The Increasing Role of Seismic Measurements in Geotechnical Engineering

The 2008 Spencer J. Buchanan Lecture
By Professor Kenneth Stokoe

Application of Dynamic Methods to the Design and Installation of Driven Piles

The 2007 Karl Terzaghi Lecture
By Dr. George Goble

Friday, November 14, 2008
College Station Hilton
College Station, Texas, USA
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Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star.

He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.

Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England. They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in
Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy’s Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.
The Spencer J. Buchanan ’26 Chair in Civil Engineering

The College of Engineering and the Department of Civil Engineering gratefully recognize the generosity of the following individuals, corporations, foundations, and organizations for their part in helping to establish the Spencer J. Buchanan ’26 Professorship in Civil Engineering. Created in 1992 to honor a world leader in soil mechanics and foundation engineering, as well as a distinguished Texas A&M University professor, the Buchanan Professorship supports a wide range of enriched educational activities in civil and geotechnical engineering. In 2002, this professorship became the Spencer J. Buchanan ’26 Chair in Civil Engineering.

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The text of the lectures and a videotape of the presentations are available by contacting:

Dr. Jean-Louis Briaud  
Spencer J. Buchanan ’26 Chair Professor  
Zachry Department of Civil Engineering  
Texas A&M University  
College Station, TX 77843-3136, USA  
Tel: 979-845-3795  
Fax: 979-845-6554  
e-mail: Briaud@tamu.edu  
http://ceprofs.tamu.edu/briaud/buchanan.htm
AGENDA

The Sixteenth Spencer J. Buchanan Lecture
Friday November 14, 2008
College Station Hilton

2:00 p.m. Welcome by Jean-Louis Briaud
2:05 p.m. Introduction by David Rosowsky
2:10 p.m. Introduction of George Goble by Zenon Medina-Cetina
2:15 p.m. “Applications of Dynamic Methods to the Design and Installation of Driven Piles”
The 2007 Terzaghi Lecture by George Goble
3:15 p.m. Discussion
3:25 p.m. Introduction of Kenneth Stokoe by Jean-Louis Briaud
3:30 p.m. “The Increasing Roles of Seismic Measurements in Geotechnical Engineering”
The 2008 Buchanan Lecture by Kenneth Stokoe
4:30 p.m. Discussion
4:40 p.m. Closure with Philip Buchanan
5:00 p.m. Photos followed by a reception at the home of Jean-Louis and Janet Briaud.
Dr. Stokoe has been working in the areas of in situ seismic measurements, laboratory measurements of dynamic material properties, and dynamic soil-structure interaction for the past 35 years. He was instrumental in developing the crosshole seismic method for in-situ shear wave velocity measurement to the method that has been adopted as the standard by the American Society for Testing and Materials (ASTM D4428M) and the method that is used by geotechnical engineering firms worldwide. He has also developed a combined torsional shear/resonant column system which is now used by many universities and private firms in the U.S., Europe and Asia to evaluate dynamic material properties.

In the last 30 years, Dr. Stokoe has conducted major research efforts in the areas of: 1. nondestructive testing (NDT) of pavements, runways and geotechnical systems, and 2. laboratory evaluation of soil and rock stiffnesses under cyclic and dynamic loading conditions. He and his colleagues have developed the Spectral-Analysis-of-Surface-Waves (SASW) method for testing geotechnical and pavement systems and structural components. Dr. Stokoe has conducted major studies using the SASW method to evaluate earth dams for the U.S. Bureau of Reclamation, the Icelandic government and Delft Geotechnical Laboratories and to evaluate debris slides for the U.S. Geological Survey and the Italian government. He has also conducted comparison studies of the downhole seismic method with the SASW method for the U.S. Geological Survey and has performed extensive SASW investigations at the proposed high-level nuclear storage site at Yucca Mountain for the U.S Department of Energy.

Dr. Stokoe has participated in numerous demonstration projects involving SASW testing of airport runways and taxiways over the past 20 years. Testing has been performed at McDill,
Homestead, and Tyndell Air Force Bases. Testing was also performed at the Cannon International Airport in Reno, Nevada and the Sonoma County Airport in Santa Rosa, California. Over the past 10 years, Dr. Stokoe has applied the SASW method at John F. Kennedy Airport to evaluate changes beneath runways and taxiways due to micro-tunneling activities.

As part of his activities in NDT of pavements, Dr. Stokoe and his colleagues have been actively involved in developing dynamic linear and nonlinear analyses for modeling and understanding falling weight deflectometer (FWD) measurements. He and Dr. Roesset have presented and published papers at the Transportation Research Board Meeting in this area. They have been successful in extracting the static pavement response from the dynamic FWD test as well as calculating the depth to bedrock from free vibrations of the pavement system in 120- to 180-ms long records.

Over the past 10 years, Dr. Stokoe has been involved in the development of the Rolling Dynamic Deflectometer (RDD) with funding from Texas DOT and federal agencies. The purpose of this device is to perform continuous profiles of pavement stiffness “on-the-fly.” This deflectometer operates at speeds around 2 km/hr (about 1.2 mph) and presently represents a “one-of-a-kind” piece of equipment. Most recently, the RDD has been used to profile runways at the Dallas-Fort Worth (DFW) Airport, the Portland Airport, the Seattle-Tacoma International Airport, the Atlanta International Airport and various highway pavement sections in Texas and Pennsylvania.

The laboratory studies with which Dr. Stokoe has been involved can be divided into two groups. The first group has dealt with the use of resonant column/torsional shear (RCTS) equipment to evaluate the nonlinear shear modulus and material damping of soils. Most recently, Dr. Stokoe has completed three major laboratory studies in which nonlinear dynamic soil properties were being evaluated for: 1. the Savannah River site for the Westinghouse Corporation, 2. the Electric Power Research Institute, and 3. the ROSRINE (Resolution of Site Response Issues in the 1994 Northridge Earthquake) project. These studies have been directed towards evaluating the response of soil sites during earthquake shaking. These results were combined with other tests to develop a new nonlinear soil model (including frequency-dependent material damping). Presently, dynamic rock properties are being evaluated with the RCTS equipment for the proposed Yucca Mountain High-Level Nuclear Waste site.
The second group of laboratory studies has dealt with axially loading specimens, either with transient pulses or continuous cycling for measurement of Young's resilient, constrained and shear moduli. This work has been conducted with funding from the Air Force Office of Scientific Research, the United States National Science Foundation, the California DOT, and the Texas DOT. Much of the recent work has been performed in conjunction with resilient modulus testing of subgrade soils. Further, Dr. Stokoe has developed synthetic specimens which the SHRP project used in the evaluation of resilient modulus equipment.

Dr. Stokoe is also actively involved in research dealing with ground motions associated with blasting, and exploration activities as well as construction and railroad operations. His research activities also involve integrity investigations of deep foundations and structural components by wave propagation methods. He has adapted the SASW method for testing concrete structural elements to evaluate damage and has been awarded Patent No. SN 071 462, 404 for In Situ Testing with Surface Seismic Waves of Materials Having Properties that Change with Time.

Finally, Dr. Stokoe is the PI (along with co-PIs Prof. Rathje and Prof. Wilson) on a 4-year, $3 million NSF grant in the NEES (Network for Earthquake Engineering Simulation) program. This project will have a substantial impact on the geotechnical earthquake engineering community in the United States. The project involves the development of large-scale mobile field equipment for dynamic loading of geotechnical and structural systems. The equipment, which became operational in October 2004, presents field capabilities that never before existed and will be used in research, with the oversight of the University of Texas team, by universities and governmental researchers around the U.S. The equipment will be operated by the UT team over the next 10 years, with a yearly operating budget in excess of $700K from NSF.
THE INCREASING ROLE OF SEISMIC MEASUREMENTS IN GEOTECHNICAL ENGINEERING

The Sixteenth Buchanan Lecture

Presented by Professor Kenneth H. Stokoe, II
Jennie C. and Milton T. Graves Chair
The University of Texas at Austin
OVERVIEW

The geotechnical engineer has always been faced with the problem of characterizing near-surface materials. The near-surface region is often within 10 to 100 m of the ground surface. Traditionally, field exploration programs have involved boring, sampling, and penetration testing. In the 1960s, in situ geophysical measurements began to be employed in geotechnical engineering. This work primarily involved seismic (stress wave) measurements which were adapted from exploration geophysics. Seismic measurements were used to characterize geotechnical sites (e.g. layering, top of bedrock, depth to water table) and geotechnical materials (e.g. stiffnesses in shear and compression). The real demand for seismic measurements grew out of the need to evaluate the dynamic properties of near-surface soils, specifically the shear-wave velocity, \( V_s \). Shear-wave velocity is a key parameter in soil dynamics and geotechnical earthquake engineering. Today, however, in situ seismic measurements are used in many more applications as discussed in this lecture and in the articles included in the booklet.

ORGANIZATION

Four articles are included in this booklet which cover material in the Sixteenth Buchanan Lecture. (In addition, the 2007 Terzaghi Lecture by Professor George Goble, also part of today’s program, deals with stress waves in piles.) The first article, entitled “Some Contributions of In Situ Geophysical Measurements to Solving Geotechnical Engineering Problems” by Stokoe, Joh and Woods (2004), presents background material on field seismic testing. Case histories and applications are also presented that involve geosystems loaded statically as well as dynamically. In addition, the uses of in situ \( V_s \) measurements in evaluating sample disturbance and in predicting nonlinear soil behavior in the field are discussed. It is important to note that seismic measurements are the only mechanical-property measurements that are routinely performed today in the laboratory and in the field. As such, these measurements create a critical link in translating laboratory test results to field deformational behavior as discussed in the article.

The second article, entitled “Field Seismic Testing in Geotechnical Earthquake Engineering” by Stokoe (2007), deals with seismic methods that are used to evaluate compression-wave velocity (\( V_p \)) and shear-wave velocity (\( V_s \)) profiles of all types of geologic materials. These profiles are used to represent the stiffnesses of the geologic materials in the small-strain range. Many seismic methods are available for shallow investigations as discussed in the paper. Shallow investigations are defined as profiling to depths less than 75 m. Developments are discussed that
are occurring to profile to intermediate (75 to 225 m) and deep (greater than 225 m) depths. The seismic methods used for deeper profiling are the downhole, suspension logging and surface wave methods. Examples of deeper profiling are presented. In addition, field seismic methods are discussed that are being developed to perform parametric studies in situ. Example parametric studies are presented that show in situ measurements of the effects of: (1) stress state on $V_s$ and $V_p$, (2) nonlinear straining on shear modulus, and (3) cyclic loading leading to liquefaction.

The third article, entitled “Use of Intermediate to Large Vibrators as Surface Wave Sources to Evaluate $V_s$ Profiles for Earthquake Studies” by Stokoe et al. (2006), presents information on three new sources for dynamically loading field sites and/or structures at these sites. This equipment is mobile and has been used around the United States on more than 15 projects over the past six years. The equipment is operated for shared use by the NSF-NEES equipment site at the University of Texas (nees@UTexas) with funding from the National Science Foundation. Examples using this equipment as active seismic sources are presented. Other examples using this equipment to load soils into the nonlinear range (even loading to liquefaction) are presented in the second article. Also, several of the faculty at Texas A&M University (Professors G. Biscontin, L. Barroso and J. Roesset) are using this equipment in the development of new methods to evaluate nonlinear soil behavior under dynamically loaded prototype footings.

The fourth (and last) article by Lin, Stokoe and Rosenblad (2008) presents a case history dealing with $V_s$ profiling over a large area, much of the Imperial Valley of California, and the uniformity that might be expected if the materials are from the same parent material. The variability is studied by investigating the coefficient of variation, COV. As shown in the article, COVs in the range 0.10 to 0.15 are representative of a large natural area with relatively uniform material. As $V_s$ profiles of other materials from adjacent mountain and valley-edge areas are added to the database, COVs increase and are in the range of 0.20 to 0.25. If stiffer rock over similar depth ranges were encountered, other studies show that COV values in the range of 0.30 to 0.35 would likely result. A comparison between $V_s$ profiles determined by two seismic methods is also presented. The two methods are the surface-wave (SASW) and downhole methods. A close comparison in median $V_s$ profiles is shown when an identical site-and-depth comparison is performed, called a “red-apples-to-red-apples” comparison in the article.
Some Contributions of In Situ Geophysical Measurements to Solving Geotechnical Engineering Problems

Kenneth H. Stokoe, II
*University of Texas, Austin, Texas, U.S.A.*

Sung-Ho Joh
*Chung-Ang University, Seoul, South Korea*

Richard D. Woods
*University of Michigan, Ann Arbor, Michigan, U.S.A.*

Keywords: geophysics, geotechnics, seismic testing, in situ tests, body waves, surface waves, case histories

ABSTRACT: This paper focuses on one in situ geophysical method, seismic measurements. Seismic (stress wave) measurements have been used for more than 50 years in geotechnical engineering, primarily in the areas of soil dynamics and geotechnical earthquake engineering. In the past 30 years, their role has steadily increased to the point where they also play an important part in characterizing sites, materials and processes for non-dynamic problems. Case histories and applications are presented to highlight some examples.

1 INTRODUCTION

The geotechnical engineer has always been faced with the problem of characterizing near-surface materials. The near-surface region is often within 10 to 100 m of the ground surface. Traditionally, field exploration programs have involved boring, sampling, and penetration testing. In the 1960s, in situ geophysical measurements began to be employed in geotechnical engineering. This work primarily involved seismic (stress wave) measurements which were adapted from exploration geophysics. Seismic measurements were used to characterize geotechnical sites (e.g. layering, top of bedrock, depth to water table) and geotechnical materials (e.g. stiffnesses in shear and compression). The real demand for seismic measurements grew out of the need to evaluate the dynamic properties of near-surface soils, specifically the shear-wave velocity, $V_s$. $V_s$ is a key parameter in soil dynamics and geotechnical earthquake engineering. Today, however, in situ seismic measurements are used in many more applications as discussed herein.

The discipline of geophysics and in situ geophysical measurements encompass much more than seismic methods. Other geophysical methods include electrical, magnetic, electromagnetic, ground-penetrating radar, and gravity. All of these methods offer the geotechnical engineer new and improved in situ techniques to characterize sites, materials and processes. These opportunities arise from the strong theoretical bases upon which geophysical methods are founded, the complementary physical principles that support various field tests, and the ability to perform the same basic measurement in the laboratory and in the field. Furthermore, many geophysical methods are noninvasive which make them well suited and cost effective in profiling spatially and temporally.

The geotechnical engineering profession has not adopted other geophysical methods as rapidly as seismic methods. One reason is that seismic methods directly measure a mechanical property, the initial slope of the stress-strain relationship, which is used in the solution of many geotechnical engineering problems. However, new demands in geoenvironmental, geotechnical, and military applications are constantly increasing the need for improved and higher-resolution site characterization methods. Geophysical methods have an important role to fill in these areas. Technical papers in the Proceedings of the First International Conference on Site Characterization (Robertson and Mayne, 1998) emphasize this point.

1.1 Organization

The information and examples presented in this paper focus on the use of in situ seismic measurements. However, this material demonstrates the relevance of geophysical methods in geotechnics. A brief background on stress waves is presented in Section 2 to facilitate subsequent discussions. An overview is presented in Section 3 of four noninvasive surface-wave methods. The reason is that the use of this generalized method is rapidly increasing in geotechnical engineering. Case histories and applications are presented in Section 4 that involve geosystems loaded statically as well as dynamically. The use of in situ $V_s$ measurements in evaluating sample disturbance and in predicting nonlinear soil response is discussed in Section 5 followed by conclusions in Section 6.
2 BACKGROUND ON STRESS WAVES AND TRADITIONAL SEISMIC METHODS

2.1 Types of Stress Waves

Traditionally, in situ seismic testing has been conducted by initiating a mechanical disturbance at some point in the earth and monitoring the resulting motions (stress waves) at other points in the earth. The modes of propagation most often used are body waves, compression and shear waves, and one type of surface wave, the Rayleigh wave. These waves, in terms of their far-field particle motions, are illustrated in Figure 1. Compression waves (P waves) have particle motion parallel to the direction of wave propagation, while shear waves (S waves) have particle motion perpendicular to the direction of wave propagation. Rayleigh waves (R waves) exist because of the exposed ground surface. R waves have particle motions that are a combination of vertical (shear) and horizontal (compression) motions. Near the surface of a uniform material, R waves create particle motion that follows a retrograde elliptical pattern as illustrated in Figure 1c. The decay with depth of the vertical and horizontal components of R-wave particle motion is illustrated in Figure 2. The depth axis is normalized by the Rayleigh wavelength, \( \lambda_R \). It is interesting to see in Figure 2 that the horizontal component changes sign at a normalized depth around 0.15. The meaning of this change in sign is that R-wave particle motion changes from a retrograde ellipse to a prograde ellipse in a uniform half-space.

2.2 Stress Wave Velocities

Stress waves are non-dispersive in a uniform elastic medium. The term non-dispersive indicates that the propagation velocity is independent of frequency. These waves are also considered non-dispersive in low-loss homogeneous soil and rock at small strains and low frequencies. However, stratigraphy and other forms of heterogeneity cause frequency-dependent velocity. (This dependency is the fundamental premise on which surface-wave testing is based as noted in Section 3.) The far-field velocities of stress waves depend on the stiffness and mass density of the material as,

\[
P\text{-wave velocity:} \quad V_p = \sqrt{\frac{M}{\rho}} = \sqrt{\frac{B + \frac{4}{3}G}{\rho}} = \sqrt{\frac{E}{\rho \left(1 + \nu\right)\left(1 - 2\nu\right)}} \quad (1)
\]

\[
S\text{-wave velocity:} \quad V_s = \sqrt{\frac{G}{\rho}} \quad (2)
\]

where \( \rho \) is the mass density and \( M, B, G \) and \( E \) are the constrained, bulk, shear, and Young’s moduli, respectively, and \( \nu \) is Poisson’s ratio. For a homogeneous, isotropic material, compression and shear
wave velocities are related through Poisson’s ratio, \( \nu \), as,

\[
V_p = V_s \sqrt{\frac{1 - \nu}{0.5 - \nu}}
\] (3)

The far-field velocity of the Rayleigh wave, \( V_R \), is related to the velocities of P and S waves as (Achenbach, 1975),

\[
2 - \left( \frac{V_R}{V_s} \right)^2 - 4 \left[ 1 - \left( \frac{V_R}{V_P} \right)^2 \right] \left[ 1 - \left( \frac{V_R}{V_S} \right)^2 \right] = 0
\] (4)

A good approximation for \( V_R \) in terms of \( V_s \) and Poisson's ratio is (modified from Achenbach, 1975),

\[
V_R \approx \frac{0.874 + 1.117 \nu}{1 + \nu} V_s
\] (5)

These equations permit computing the relative values of \( V_P \), \( V_s \) and \( V_R \) as a function of Poisson’s ratio, as shown in Figure 3. At \( \nu = 0 \), \( V_P = \sqrt{2} V_s \) and \( V_R = 0.874 V_s \). At \( \nu = 0.5 \) (which theoretically represents an incompressible material; hence, an infinitely stiff material), \( V_P = \infty \) so that \( V_P/V_s = \infty \). At \( \nu = 0.5 \), \( V_R = 0.955 V_s \). The ratios of body wave velocities (\( V_P / V_s \)) typically determined with small-strain seismic tests on unsaturated soil and rock are around \( \sim 1.5 \) to \( 2.0 \), which corresponds to Poisson's ratio \( \sim 0.10 \) to \( 0.33 \); therefore, the small-strain Poisson's ratio is relatively low.

It is important to note that the S-wave velocity is the same in an infinite medium as in a rod (torsional motion). However, the longitudinal P-wave velocity is different, being \( V_P = \sqrt{(M/\rho)} \) in an infinite medium and \( V_L = \sqrt{(E/\rho)} \) in a rod. The “L” denotes a longitudinal wave. The relationship between \( V_L \) and \( E \) is for tests in which wavelengths are much greater than the radius of the rod. For shorter wavelengths, \( V_L \) decreases as frequency increases. Also, wave velocity, \( V \), wavelength, \( \lambda \), and frequency of excitation, \( f \), are related for any type of stress wave as,

\[
V = f \lambda
\] (6)

It is also worth mentioning that the terms “elastic” and “small strain” are often used to describe stress waves and associated propagation velocities and moduli when dealing with in situ seismic measurements. These terms are used because transient mechanical disturbances created in situ during testing generate stress waves in geotechnical materials that have maximum strain amplitudes less than 0.0001%. As a result, the stress waves exhibit propagation behavior that is independent of strain amplitude and possess only a minor amount of energy dissipation due to material damping.

### 2.3 Wave Velocities and Degree of Saturation

The shear wave velocity is related to the shear stiffness of the soil skeleton. In clean coarse sands, where capillary effects are negligible, the effective stress controls the shear stiffness, and the effect of saturation on shear wave velocity is only related to changes in mass density \( \rho \), through \( V_s = (G/\rho) \). The relevance of capillary forces at interparticle contacts on shear stiffness increases with fines content. And, the lower the degree of saturation, the higher \( G \) and \( V_s \) become (Cho and Santamarina, 2001).

On the other hand, P-wave velocity is controlled by the constrained modulus, \( M = B + 4G/3 \). Therefore, the fluid and the granular skeleton contribute to \( V_P \). For degrees of saturation, \( S_r \), less than about 99 percent, P-wave velocity is controlled by the stiffness of the soil skeleton in constrained compression in the same fashion as shear waves; that is, the main influence of water on \( V_P \) over this range in \( S_r \) comes from unsaturated conditions which impact the soil skeleton stiffness. However, if the degree of saturation equals 100 percent, the constrained modulus of this two-phase medium is dominated by the relative incompressibility of the water in comparison to the soil skeleton. The resulting value of \( V_P \) varies with the void ratio or porosity, \( n \), the bulk stiffness of the material that makes the grains, \( B_g \), and the bulk stiffness of the fluid, \( B_f \). The bulk stiffness of the fluid phase is very sensitive to the presence of air. Therefore, when the degree of saturation \( S_r \) is about 99.5 to 100 percent, the value of \( V_P \) is very sensitive to \( S_r \). Figure 4 shows the typical influence of degree of saturation on \( V_P \) over this very small change in degree of saturation (the shear wave velocity remains unaffected by such a small change in saturation). For completeness, it is also noted that the impact of \( S_r \) on \( V_s \) and \( V_P \) of rock is very small (only a few percent change) for \( S_r \) going from zero to 100%.
Compression Wave Velocity, \( V_p \), m/s

| Degree of Saturation, \( S_r \), Percent |
|-----------------|---|---|---|---|---|
|                 | 300 | 600 | 900 | 1200 | 1500 |

Range from Void Ratio Changes

Figure 4. Typical variation in compression wave velocity with degree of saturation changing from 99.4 to 100 % for sand (after Allen et al., 1980)

2.4 Traditional Field Methods

Field testing methods can be classified as active or passive. Active-type methods are generally employed in geotechnical engineering. In this case, a wave is radiated into the medium from a source that is energized as part of the test. Passive-type methods are used less frequently. However, a passive system can be selected when background noise can be used as the excitation source. Field testing methods can also be classified as nonintrusive if all instrumentation is placed on the ground surface, or intrusive when boreholes or penetrometers are used. The most common stress-wave based methods in field use today are briefly reviewed below.

2.4.1 Nonintrusive, active methods

Nonintrusive methods have many advantages including: (1) elimination of the time and cost of drilling, (2) avoidance of potential environmental consequences of drilling, and (3) effective coverage of large areas. These methods include surface refraction, surface reflection, and surface waves as illustrated in Figure 5.

**Surface Reflection Method** - The surface reflection method is one of the oldest and most common seismic methods. This method is well documented in numerous textbooks in geophysics (e.g., Dobrin and Savit, 1988, and Burger, 1992). The main principle of the seismic reflection method is illustrated in Figure 5a which shows one arrangement of the source and receivers. Both the source and receivers are placed on the ground surface. Typically, compression wave measurements are performed using either mechanical sources that are vertically oriented or explosive sources. Waves reflected from interfaces at depth are monitored with vertically-sensitive geophones. The main purpose of testing is typically to identify and approximately locate key interfaces at depth.

**Surface Refraction Method** - The surface refraction method is an established geophysical method for nonintrusively identifying sediment stiffnesses and layer interfaces at depth. The method is based on the ability to detect the arrival of wave energy that is critically refracted from a higher velocity layer which underlies lower velocity sediment. Seismic signals are generated with an active source, and wave arrivals are detected on the surface with an array of receivers as shown in Figure 5b. As with the surface reflection method, compression wave measurements are typically performed using vertical mechanical sources or explosives. The arrivals of refracted waves on the ground surface are monitored with vertically-sensitive geophones.

Figure 5. Generalized field testing
Surface-Wave Method – The surface-wave method can involve Rayleigh and Love waves, and testing has been conducted on land and offshore (Stokoe et al., 1994, Stoll et al., 1994, Tokimatsu, 1995, and Luke and Stokoe, 1998). Most testing in geotechnical engineering involves R waves. Several variations of the generalized method are currently being used or are under development as discussed in Section 3. The most common approach used on land is called the spectral-analysis-of-surface-waves (SASW) method. This test method involves actively exciting Rayleigh wave energy at one point and measuring the resulting vertical surface motions at various distances (receiver points) away from the source as illustrated in Figure 5c. Measurements are performed at multiple source-receiver spacings along a linear array. The generalized method and variations under development have tremendous potential in geotechnical engineering and are therefore discussed in more detail in Section 3.

2.4.2 Intrusive, active methods
Intrusive active methods have been widely used in geotechnical engineering, particularly in soil dynamics and geotechnical earthquake engineering beginning in the 1960s. The crosshole and downhole methods were initially developed/adapted for use followed by development of the seismic cone penetrometer (SCPT) and suspension logger. Each method is briefly discussed below.

Crosshole Method – Shear and compression wave velocities are determined from time-of-travel measurements between a source and one or more receivers in the crosshole method. Testing is generally conducted by placing the active source and receivers at the same depth in adjacent boreholes, as illustrated in Figure 6a. The times of travel from the source to the receivers, called direct travel times, and the times of travel between receivers, called interval travel times, are measured. Vertically oriented impacts with mechanical sources are usually applied to the borehole wall using a wedged source. Vertically oriented receivers are used to monitor horizontally propagating shear waves with vertical particle motion; hence SV waves. Radially oriented receivers are used to monitor horizontally propagating P waves. Compression and shear wave velocities are determined by dividing the borehole spacings at the testing depth by the respective travel times. The test is repeated at multiple depths to compile a complete profile of shear and compression wave velocities versus depth.

There are several strengths associated with crosshole testing. First, the source and receivers are placed closed to the material/target to be evaluated, thus enhancing resolution. Second, measurements can also be gathered along multiple inclined ray paths which can be processed together to render a tomographic image of the cross section (Menke, 1989, and Santamarina and Fratta, 1998). Third, P, SV and SH waves can be generated and measured. (SH waves are shear waves with particle motions in the horizontal direction.) The main disadvantage in crosshole testing is the time and cost associated with drilling boreholes; however, ongoing developments in penetrometer-deployed sources combined with effectively deployed receivers promise efficient crosshole implementations under the appropriate soil conditions (e.g., Fernandez, 2000).

Downhole Method – In the downhole method, the times for compression and shear waves to travel between a source on the surface and points within the soil mass are measured. Wave velocities are then calculated from the corresponding travel times after travel distances have been determined. Travel distances are typically based on assuming straight ray paths between the source and receivers, although the analysis may sometimes account for refracted travel paths. Figure 6b shows a conventional setup which requires the drilling of only one borehole. One of the main advantages of the downhole method in comparison to the crosshole method is the need for only one borehole, so the cost is less. However, the disadvantage is that wave energy has to travel increasingly larger distances as the depth of testing increases. In the writers’ experience, the optimum testing depths range from about 10 m to 50 m unless specialized personnel are involved. This depth is, of course, dependent on the energy developed by the source (various high-energy, mechanical sources have been constructed, e.g., Liu et al., 1988).

Seismic Cone Penetrometer – The cone penetrometer (CPT) is a well established tool for characterizing soil properties by measuring tip and side resistances on a probe pushed into the soil (Lunne et al., 1997). The SCPT test is a modification of the cone penetrometer test that allows measurement of shear wave velocities in a downhole testing arrangement (Campanella et al., 1986). Seismic energy is generated at the surface near the insertion point of the cone. Usually a horizontal impact on an embedded anvil is used to generate the SH waves. Travel times of the shear wave energy, either direct or interval, are measured at one or more locations above the cone tip as shown in Figure 6c. After testing at one depth, the cone is penetrated further into the soil, and the test is repeated. One of the important benefits of this method is that the seismic data can be combined with the cone resistance values to build a clearer picture of both soil type, strength, stiffness, and layering. This is an excellent example of using multiple techniques to investigate sites.

Suspension Logger – Logging tools can be lowered into a borehole to determine material properties with stress waves, electromagnetic waves, gamma radiation, and other physical principles. The main limitations in borehole logging are the effect of
casing and drilling fluids on the measured response and

Source 3 – D Receivers

Direct P and S Waves

3-D Receivers

a. Crosshole test

b. Downhole test

c. Seismic cone penetrometer test

d. Suspension logging test

Figure 6: Field arrangements used to perform intrusive seismic tests (from Stokoe and Santamarina, 2000)

the depth scanned by the technique relative to the zone affected by drilling the borehole. One of the more recent advances in borehole shear wave methods is the suspension logger (Kitsunezaki, 1980, Toksoz and Cheng, 1991, and Nigbor and Inai, 1994). This test is performed in a single, mud-filled borehole. The device is lowered on a wire line into the borehole, and seismic energy is generated and received by a receiver array in the borehole as shown in Figure 6d. The shear and compression wave velocities of the surrounding material are determined from the arrival times of these waves following standard travel-time procedures. One of the advantages of this method is that the wire-line nature of the test allows for measurements at significant depths (hundreds of meters). Two drawbacks of the method are that it generally can not be performed in a steel or thick plastic casing if soft soils are to be tested and it does not work well within about 7 m of the ground surface.

2.5 Additional Information

Most of the information presented above was extracted from the article by Stokoe and Santamarina, 2000. The information is briefly presented to facilitate the following discussion. However, much more information is available in the article and in the literature because of the strong theoretical bases upon which seismic and other geophysical measurements are founded. Textbooks such as Richart et al. (1970), Aki and Richards (1980), Ward (1990), Sharma (1997), and Santamarina et al. (2001) are excellent references. Manuals such as ASCE Press (1998) and NRC (2000) are also good references. Many important topics such as material damping, geotechnical spreading, near-field effects, mode conversions, effects of stress state on wave velocities, inherent and stress-induced anisotropies could not be covered
3 OVERVIEW OF EVOLVING SURFACE-WAVE METHODS

There is one field seismic method that is under active development and deployment today. That method is the surface-wave method. The great interest in this method arises, in large part, from the noninvasive nature of the method combined with the capabilities of imaging softer layers beneath stiffer materials and testing large areas rapidly and cost effectively. One test configuration is described in Section 2.4.1 and illustrated in Figure 5c. The method, in terms of a generalized method, can be divided into two basic parts: (1) monitoring Rayleigh-wave propagation along the ground surface (“field testing”), and (2) empirical or numerical modeling of the field measurements to yield the subsurface V_s profile. The objective of field testing is to determine the phase-velocity dispersion curve for the test site. This objective is discussed in detail below as are the different approaches to meeting this objective. Modeling of the field measurements, either empirical or numerical, can vary significantly from one analysis procedure to another. The strengths and limitations of the modeling procedures are discussed in Section 3.2. However, since the forward modeling theory of surface-wave propagation was introduced by Thomson (1950) and Haskell (1953), empirical procedures should no longer be used to analyze the field measurements.

3.1 Surface-Wave Techniques: Determination of Phase-Velocity Dispersion Curves and Associated Modeling Approaches

The steady-state, Rayleigh-wave method (Richart et al., 1970) is one of the initial surface-wave methods that was used in geotechnical engineering, which directly measure the wavelengths of Rayleigh waves for the determination of phase velocities (velocities associated with wavelengths or frequencies). The two-station method (Landisman et al., 1968 and Sato, 1971) is another surface-wave method based on the inter-station phase difference. Sato used a transfer function to determine phase velocities of surface waves for a range of frequencies, and Landisman et al. used an inter-station cross-correlogram to eliminate the adverse effects of low-energy noises. In the 1980s, the University of Texas at Austin (Heisey, et al., 1982, Nazarian and Stokoe, 1984, and Stokoe and Nazarian, 1985) established the SASW method to determine phase-velocities of Rayleigh waves. The SASW method was an innovative method to make faster and more efficient measurements than any previous methods. In addition, the dynamic stiffness matrix method (Kausel and Roësset, 1981) became the theoretical basis for modeling the phase-velocity dispersion curves to yield the V_s profile of the site. Advances continue to occur in this aspect of the test (Roësset et al., 1991, Gucunski and Woods, 1991, Al-Hunaidi, 1994, Al-Hunaidi and Rainer 1995, and Joh, 1996).

Recently many other methods have been developed for determination of phase-velocity dispersion curves in the generalized surface-wave method. The four most widely used methods are the spectral-analysis-of-surface-waves (SASW) method, the frequency-wave number (f-k) spectrum method, the multi-channel analysis of surface wave (MASW) method, and the continuous surface wave (CSW) method. Currently, the SASW method is used around the world including Asia and Europe, the MASW method is mostly used in the America and some Asian countries, and the CSW method is actively used in the United Kingdom, Australia and some Asian countries. In Table 1, general features of each method are compared. Advantages and disadvantages of each method are summarized in Table 2. In the following sections, the fundamental principles of these methods are described and some important issues are discussed.

3.1.1 SASW method

In the SASW method, the dispersive characteristics of Rayleigh waves propagating through a layered material are measured and then used to evaluate the S-wave profile of the material (Stokoe et al., 1994). SASW measurements involve generating waves at one point on the ground surface and recording them as they pass by two or more locations, as illustrated in Figure 7a. All measurement points are arranged along a single radial path from the source. Measurements are performed with several (typically six or more) sets of source-receiver spacings. In each set, the distance between the source and first receiver is kept equal to the distance between receivers. The phase shift versus frequency relationship is measured for surface waves propagating between the receivers for each receiver spacing. A typical phase plot is shown in Figure 7b. From each phase plot, the phase velocity of the surface wave is calculated at each frequency knowing the frequency, phase angle and distance between the receivers. The result is a plot of phase velocity versus frequency for a given receiver spacing, called an individual dispersion curve (Figure 7c). This procedure is repeated for all source-receiver spacings used at the site and typically involves significant overlapping in the dispersion data between adjacent receiver sets. The individual dispersion curves from all receiver spacings are combined into a single composite dispersion curve called the experimental or “field” dispersion curve (Figure 8). Once the composite dispersion curve is generated for the site, an iterative forward modeling procedure or an inversion analysis algorithm is used...
to determine a shear-wave velocity profile by matching the field dispersion curve with the theoretically-determined dispersion curve (Figure 8).

<table>
<thead>
<tr>
<th>Surface-Wave Method</th>
<th>Key Features</th>
</tr>
</thead>
</table>
| SASW method         | - phase velocities from phase differences  
                       - two to four receivers typically used  
                       - superposed-mode phase velocity (apparent phase velocity)  
                       - global property over receiver-spread area  
                       - shear-wave velocity profile from the apparent phase velocities  
                       - comprehensive forward modeling or inversion analysis  
                       - impulsive source, swept-sine source, or random vibration source |
| f-k spectrum method | - phase velocities from frequency-wave number spectrum  
                       - multiple receivers (e.g. 128, 256, etc. receivers)  
                       - fundamental and higher-mode phase velocities  
                       - global property over receiver-spread area  
                       - shear-wave velocity profile from fundamental and higher modes  
                       - impulsive source |
| MASW method         | - limited number of receivers (usually 24 receivers)  
                       - fundamental and higher-mode phase velocities  
                       - walk-away measurement  
                       - same measurement configuration as common-midpoint reflection survey  
                       - global property over receiver-spread area  
                       - shear-wave velocity profile from the fundamental mode  
                       - impulsive source or swept-sine source |
| CSW method          | - phase velocity from the average phase-angle slope over receiver-spread area  
                       - four to six receivers used  
                       - superposed-mode phase velocity (apparent velocity)  
                       - global property over receiver-spread area  
                       - shear-wave velocity profile from the apparent velocities  
                       - steady-state harmonic source |

Table 2. Advantages and disadvantages of four, widely used surface-wave methods

<table>
<thead>
<tr>
<th>Surface-Wave Method</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
| SASW method         | - good sampling of shallow material  
                       - more sensitive measurements for layer stiffness contrast, using apparent velocity inversion analysis | - multiple measurements using different source-receiver configurations are required  
                       - expertise required for phase unwrapping and forward modeling |
| f-k method          | - dispersion curves separated for fundamental and higher modes  
                       - body-wave effect extracted  
                       - dispersion curve global to the receiver-spread area | - aliasing problem in wave number domain  
                       - inaccurate mode separation in case of poor resolution in f-k spectrum  
                       - large number of traces required for good resolution in wave-number domain  
                       - limitation due to topographic constraint and instrumentation capability  
                       - long measurement time |
| MASW method         | - mode separation of surface waves | - aliasing problem in wave-number domain  
                       - use of the fundamental mode only in inversion analysis |
| CSW method          | - the effects of local anomalies minimized with the use of average phase-angle slope  
                       - no expertise required to calculate phase velocity  
                       - reliable measurements with controlled source | - dedicated inversion analysis required but not used  
                       - near-field effects included  
                       - exploration depth limited  
                       - frequency-content of vibrator is limited |

The SASW method is a simple technique that is easily implemented in terms of field testing. The requirement of several measurements using different source-receiver configurations is time and labor intensive. However, the multiple source-receiver configurations employ multiple sources which are selected appropriately for the measured wavelength range at each source-receiver configuration. Therefore, time- and labor-intensive measurements preferably lead to high-quality results.

The SASW method measures apparent phase velocities, which correspond to the superposed mode of higher-mode surface waves and body waves. Determination of apparent phase velocities incorporates phase unwrapping. In a complicated multi-layered system, phase unwrapping may be non-systematic and sometimes requires expertise, which is cumbersome to inexperienced personnel. However, the non-systematic nature of phase unwrapping can be improved by a signal processing technique such as the
impulse-response filtration technique (Joh et al., 1997) and Gabor spectrum.

Figure 7. Spectral-analysis-of-surface-waves (SASW) method: Calculation of phase velocities
Importantly, the SASW method uses the apparent phase velocity dispersion curve along with source and receiver locations in the forward modeling or inversion analysis. The dynamic stiffness matrix method (Kausel and Roësset, 1981), which is the forward modeling algorithm used in the matching or inversion process, can simulate the apparent phase velocity specific to the source-receiver configuration. The inversion analysis based on apparent phase velocities and the dynamic stiffness matrix method are key features of the SASW method, which improves the reliability and accuracy of the shear-wave velocity profile.

### 3.1.2 Frequency-wave number (f-k) spectrum method

The frequency-wave number (f-k) method is another method which has been widely used in the geophysical area and recently adopted for geotechnical engineering applications. In the f-k method, the propagating surface waves are measured at a significant number of locations in a line with the source. The measurement of propagating surface waves at many sequential locations in a line can reveal the wavelengths of the surface waves, which are basically the reciprocals of the wave numbers. Along with the frequency information obtained from the time-domain waveform, the wave number information is used to determine phase velocities.

To describe the f-k spectrum method, a total of 256 synthetic seismograms were generated using the dynamic stiffness matrix method. The layered system shown in Figure 9a was used as the model profile. The stacked traces in the time-space domain are shown in Figure 9b. These results can be transformed to the frequency-wave number domain by means of a 2-D FFT or slant-stack analysis (McMechan and Yedlin, 1981). Figure 9c displays the frequency-wave number (f-k) contour plot transformed from the time-space domain data in Figure 9b. The fundamental and higher modes of the surface-wave propagation are identified by the ridge analysis of the frequency-wave number contour plot. In the frequency-wave number contour plot, the modes of the surface-wave propagation refer to different wave numbers for a given frequency, and correspond to the loci identified by the ridge analysis. Figure 9d is the phase-velocity dispersion curve determined from the modes identified in Figure 9c. This approach to determine phase velocities from the frequency-wave number spectrum is called the f-k spectrum analysis (Gabriels et al., 1987).

The f-k spectrum method is superior to any other method in characterizing the fundamental and higher modes from the measured surface wave. However, the required use of numerous receivers is the main disadvantage both for repetitiveness problems and for required testing time. Also, the data acquisition required for a large number of traces may be expensive, and the topographic constraints may limit the reliability of the measurements.

### 3.1.3 Multi-channel analysis of surface waves (MASW) method

In the MASW method (Park et al., 1999, and Miller et al., 1999), a large array of time traces is measured using a swept-sine vibratory source or an impulsive hammer, using the walk-away method (Figure 10). The basic field configuration and acquisition procedure for the MASW measurements is generally the same as the one used in conventional common midpoint (CMP) body-wave reflection surveys. In the MASW method, the dispersion curve can be determined in two approaches: the swept-frequency record approach and the frequency-wave number spectrum approach. In the swept-frequency record approach shown in Figure 11a, the linear slope of each component of a swept-frequency record is determined and used to calculate the phase velocity. The frequency-wave number spectrum shown in Figure 11b is almost the same approach as the frequency-wave number spectrum method described in the Section 3.2. In the frequency-wave number spectrum method, the ridge of the frequency-wave number contour plot is identified and used to determine the phase velocity from the relationship among frequency, wave number and phase velocity. On the other hand in the MASW method, the phase velocity-frequency contour plot is first determined from the frequency-wave number contour plot and then the ridge of the phase velocity-frequency contour plot is identified for the calculation of the phase velocity corresponding to a frequency.

The MASW method uses only the fundamental mode for the inversion analysis. For the site with a normally dispersive dispersion curve, in which phase velocities increase with increasing wavelength, the fundamental mode alone may be enough to resolve the layer stiffness reliably. However, for a typical geotechnical site with a more complex stiffness profile, where the measured dispersion curve may be inversely dispersive or heavily fluctuating with a up-and-down pattern, the inversion analysis using the fundamental-mode only can not work well (Tokimatsu et al., 1992). To make the MASW method a reliable exploration method, it is crucial to incorporate higher modes as well as the fundamental mode in the inversion analysis. Recently, an effort to use higher modes in the inversion analysis was made (Kansas Geological Survey, 2003).
Figure 9. Numerical simulation illustrating the frequency-wave number (f-k) spectrum method: (a) layered geotechnical site, (b) synthetic seisograms, (c) f-k contour plots, and (d) phase-velocity dispersion curve.
3.1.4 Continuous surface wave (CSW) method

The continuous surface-wave (CSW) method is a geophysical exploration technique to evaluate the subsurface stiffness structure using a vibrator and more than four receivers, as depicted in Figure 12. Since the CSW method was initiated by British researchers (Matthews et al., 1996, and Menzies and Matthews, 1996), it has been used in Europe, Australia and some Asian countries. Unlike other surface-wave methods, the CSW testing only uses a vibrator to generate surface waves. The application of the CSW method is limited to shallow stiffness profiling like compaction-quality control, because the vibrator source does not generate enough energy for sampling deep material.

The CSW testing shown in Figure 12 uses four geophones to measure the particle-velocity history of the ground for sinusoidal vibration induced by the vibrator. The geophones are placed in a linear array with an equal spacing. Sometimes five or six geophones are used to improve the accuracy of the measurement. The time history of particle velocity that is measured at each geophone is transformed into the frequency domain by Fourier transformation. And the phase angle is determined for an excitation frequency at each geophone. Then, the phase angles are plotted against the location of the geophone, as shown in the lower portion of Figure 12. If the soil is homogeneous, the phase angle should have the tendency to linearly increase with the distance from the source. In some cases, the phase angle goes over 180 degree or below -180 degree because the phase angle is wrapped to fall between -180 degree and 180 degree due to the nature of Fourier transformation. Usually the phase wrapping can be easily identified in the plot of the source-to-receiver distance versus the phase angle, which has more than two parallel lines. In the case with wrapped phase angles, the phase unwrapping operation can be applied to recover the original phase angles.

After the phase velocities are determined for all the excitation frequencies, the shear-wave velocity profile can be determined from an empirical relationship or an inversion analysis like the one for the SASW method. Presently an empirical analysis is used. One advantage of the CSW method is to use an average phase-angle slope. The average of the phase-angle slope eliminates the local anomalies which may mislead the evaluation of the global S-wave velocity profile. The other advantage of using the average phase-angle slope is that expertise is not needed in determining the phase velocities, which enables the automation of the phase-velocity calculation. The controlled source like a electro-mechanical vibrator allows reliable measurements only in the frequency range compliant to the vibrator specification, and measurements of frequencies out of the vibrator specification lose reliability. This indicates that very shallow and very deep materials can not be sampled, which turns out to be a disadvantage of the CSW method. Also, the measurement time is usually long compared with other surface-wave methods. Finally, the CSW method needs to be more refined in that an inversion analysis specific to this method needs to be developed for reliable use in the future.
(a) Analysis procedure: Swept-frequency approach (Park, Miller and Xia, 1999)

(b) Analysis procedure: Frequency-wave number approach

Figure 11  Multichannel analysis of surface waves (MASW) method: Analysis Procedure (Kansas Geological Survey, 2003)

Figure 12 Continuous surface wave (CSW) method
3.2 Theoretical Aspects Associated with the Surface-Wave Methods

Most surface-wave methods as applied today are sophisticated in measurement and analysis, and therefore give rise to important issues in terms of theoretical background aspects. In this section, two major issues related to the surface-wave methods are discussed. These issues are the forward modeling procedure and the inversion analysis; that is, how to theoretically calculate phase velocities for a given layered system and how to evaluate a shear-wave velocity profile from a measured dispersion curve. These issues are important topics for better and more reliable profiling of subsurface stiffness.

3.2.1 Higher-mode velocities and apparent velocity

Fundamental and higher modes in surface-wave propagation are the distinctive features in a multi-layered system. When surface waves propagate through a multi-layered system, the stiffness of each layer affects the propagation of the surface waves (Gucunski and Woods, 1992, and Al-Hunaidi and Rainer, 1995). The different stiffnesses in the layers may confine the stress waves in some layers or cause multiple refractions and reflections, leading to different ray paths, which result in different propagation velocities even for the same frequency. Both the transfer matrix method (Thomson, 1950, and Haskell, 1953) and the dynamic stiffness matrix method (Kausel and Roësset, 1981) can determine fundamental- and higher-mode velocities. These modes correspond to plane waves in 2-D space, and can be calculated from the eigenvector analysis of the transfer matrix or the dynamic stiffness matrix.

The superposed mode in surface-wave propagation is also an important feature, because this mode is actually generated during testing. The superposed mode corresponds to the 3-D solution of propagating surface waves in a cylindrical pattern, not like the planar pattern of a 2-D wave. This propagation is often observed when the source is close to the receivers and the wavefront still has a significant cylindrical pattern. The superposed mode does not fall into specific normal modes, but is somewhere between the normal modes. The superposed mode is often called an apparent velocity or an effective velocity. Calculated or measured apparent phase velocities are dependent on the actual locations of the source and receivers.

In surface-wave measurements, there are two different approaches in terms of using surface-wave modes. The SASW and CSW methods use the superposed mode, because the source is close to the receivers and mode separation of the measured surface waves is not practical. The f-k spectrum and MASW methods use the fundamental and higher modes, because the source is far enough from the receivers and the modes are well separated. Specifically, the f-k spectrum and MASW methods use a large array of receivers which helps the separation of surface-wave modes.

Figure 13 shows the differences among the normal-mode solution, 2-D solution and 3-D solution of propagating surface waves. The 2-D solution is different from the 3-D solution in that the 2-D velocity is a superposed-mode velocity of plane Rayleigh-wave modes without body-wave interference (Roësset et al., 1991). The dynamic stiffness matrix method was used to calculate theoretical phase velocities for the layered systems in Figure 13. Case 1 is a soil system with increasing stiffness with depth, and Case 2 is a soil system with a soft layer trapped between a harder surface layer and a half-space. Phase velocities were calculated for: (1) different modes of plane Rayleigh waves, (2) the 2-D solution of a plane Rayleigh wave, and (3) the 3-D solution, which is an apparent dispersion curve for the cylindrical Rayleigh wave. The contribution of different modes to the simulated dispersion curve is presented in Figure 13b. In the case of the soil system with increasing stiffness with depth, the apparent dispersion curve essentially coincides with the fundamental mode of the Rayleigh wave over the complete frequency (hence wavelength) range. However, in Case 2, the 2-D and 3-D solutions in the higher-frequency region (smaller-wavelength region) are not just from one mode but a superposition of several modes. Also, it is important to realize that: (1) the 2-D solution resides between the modes of a plane Rayleigh wave, and (2) the 3-D solution may become lower than the fundamental mode at low frequencies. This phenomenon is probably due to the multiple reflections and refractions of body waves. The comparison of the normal-mode solution, 2-D solution and 3-D solution in Figure 13b implies that the 3-D solution may be the closest to the actual measurements contaminated with body waves and higher-mode Rayleigh waves.

3.2.2 Inversion analysis to evaluate a shear-wave velocity profile

In the surface-wave methods, two different categories of inversion analysis are available, dependent on the type of experimental dispersion curve. The first one is to use the normal-mode solution. The f-k spectrum method and the MASW method belong to this category. Most of the available inversion techniques are based on this approach (Hossain and Drnevich, 1989, Addo and Robertson, 1992, Yuan and Nazarian, 1993, and Xia, et al., 1999). In this category, the most crucial step is to well separate fundamental and higher modes. Specially in the high-frequency range, the phase velocities for
normal modes are very close to each other. Therefore, if the field measurement configuration is not good enough to differentiate modes, the inversion analysis may end up with misleading results. Figure 9c is a good example of difficulty in resolving lower modes in the high-frequency region. In the high-frequency region, the evaluated mode is not necessarily the fundamental mode, but is one of the higher modes. In this case, it is almost impossible to identify which higher-mode the measured mode belongs to. Therefore, the inversion analysis using the normal-mode solution needs to focus on only the low-frequency region to avoid the problem in miscounting the normal-mode number.

In the second category of inversion analysis, the apparent phase-velocity dispersion curve is (or should be) used. The SASW and CSW methods belong to this category (Gucunski and Woods, 1991, Rix and Leipski, 1991, Tokimatsu et al, 1992, Joh, 1996, and Ganji et al, 1998). In this case, it is very important to calculate the apparent theoretical phase velocity. The apparent theoretical phase velocity should be calculated using the exact locations of source and receivers, which can have a significant influence on the resulting phase-velocity dispersion curve. Several sets of the experimental dispersion curves from different sets of receiver combinations should be included to evaluate the layer stiffness contrast more reliably.

In inverting the dispersion curves determined in the SASW or CSW method, it is more beneficial to incorporate information on the source and receiver locations rather than to neglect them by assuming the measured phase velocities are far-field velocities. Figure 14 compares the resulting shear-wave velocities of two approaches: (1) the global inversion analysis, and (2) the array inversion analysis (Joh, 1996).

In the global inversion analysis, information on the source and receiver locations is ignored, and it is assumed that the receivers are located in the far field. In this inversion analysis, the theoretical phase velocities are calculated for receivers deployed at virtual locations of $2 \lambda$ and $4 \lambda$ ( $\lambda$ is wavelength for a specific frequency) and optimized to match the general trend of the dispersion curve. On the other hand, the array inversion analysis uses the phase velocities specific to the source and receiver locations, and finds the optimum shear-wave velocity profile to match all the individual experimental dispersion curves with theoretical dispersion curves corresponding to each source-receiver configuration. As shown in Figure 14b, the array inversion analysis made a fit between five experimental dispersion curves with the corresponding theoretical dispersion curves, while the global inversion analysis made a fit to follow
Figure 14 Comparison of global and array inversion analyses (Joh, 1996)

the general trend of the experimental dispersion curve. The resulting shear-wave velocities also show the superiority of the array inversion analysis. The array inversion analysis was able to produce the shear-wave velocity profile almost the same as the exact model assumed to generate the synthetic dispersion curves. However, in some cases, environmental noise and undesirable effects due to lateral geologic variability may intervene into real measurements so that this approach may not work perfectly and needs to be applied with care.

4 CASE HISTORIES AND APPLICATIONS

The purpose of this section is to present some case histories and applications that demonstrate the importance of in situ geophysical methods to the solution of geotechnical engineering problems. The examples focus on the use of in situ seismic measurements, but demonstrate the relevance of geophysical measurements in geotechnical engineering. The examples include problems that involve geosystems loaded statically as well as dynamically and loaded in the linear (small strain) and nonlinear ranges.
4.1 Soil Modulus for Settlement Analysis and Soil Structure Interaction

Settlement predictions/calculations based on the ultimate strength of soils have been made for a century or more. However, beginning in the late 1960s in the construction of nuclear power plants and other very large and heavy structures, it was evident that geotechnical engineers had to find better ways of analyzing the deformation behavior of soils including analyses of settlement and soil structure interaction. However, the geotechnical community has made very slow progress in pursuing a rational approach to these analyses. The following four case histories describe some attempts to use elastic modulus derived from seismic wave velocity measurements to estimate the settlement of foundations on sands, gravels, heavily overconsolidated clays, and soft rock.

4.1.1 Case history 1- settlement analysis of large and heavy structures

In an early recognition of the prevailing irrational approach to settlement prediction based on ultimate strength, William Swiger of Stone and Webster in 1974 (Swiger, 1974) suggested an improvement to the geotechnical practice of settlement prediction using elastic moduli derived from seismic waves. His description of the problem and potential solution is presented here as the basis for a modest advancement over the past quarter century in use of elastic modulus for settlement prediction.

In a rational approach to settlement (deformation) prediction, stress and strain should be related through modulus. The complication for soils is that soil is a nonlinear material, starting from very low strain levels, so application of any approach using modulus has to recognize and accommodate the nonlinear behavior. Swiger determined, based on calculations for five power plant structures, that the average strain causing settlement under large structures was on the order of $10^{-3}$ throughout a depth about equal to the minimum dimension of the loaded area. He then outlined an approach for determining a soil modulus at an appropriate strain level.

Swiger pointed out correctly that methods of modulus determination requiring sampling of soils and testing specimens in the laboratory suffered from a major inescapable drawback, sample disturbance. (The subject of sample disturbance is presented in more detail in Section 5.0). He also noted that techniques exist by which modulus can be measured in situ and without disturbance, namely seismic wave velocity measurements. Crosshole and downhole seismic tests were well established by the mid-1970s and Swiger used them to determine small-strain ($10^{-6}$) soil moduli at multiple depths in the ground.

Swiger acknowledged the benefits of using shear wave velocity ($V_s$) over compression wave velocity ($V_p$) because $V_s$ is unaffected by the water table and calculation of shear modulus from shear wave velocity requires only an estimate or measurement of soil density. Shear modulus is calculated using Equation 2. Furthermore, for isotropic materials, shear modulus can be converted to other moduli including Young’s modulus or constrained modulus using Equation 1. The simplified equation relating shear and Young’s moduli is,

$$E = 2G(1+\nu)$$

Equation 7

Swiger further pointed out that shear modulus measured at low strain can be adjusted to larger strains through relationships provided by Hardin and Drnevich (1972a and 1972b) or as shown by Seed (1969) in Figure 15. Based on the first cycle of a large load test at the Brookhaven National Laboratories and on modulus derived from crosshole shear wave velocity at the site, he found reasonable agreement in back calculated Young’s modulus and $E$ determined from seismic wave velocity at a strain level of $4 \times 10^{-5}$ as shown in Table 3 for two values of $\nu$. At the time, Swiger decided to use a range in $\nu$ but noted that $\nu = 0.3$ seemed more reasonable. Today, we realize that $\nu$ in the range of 0.15 to 0.35 is appropriate for the soil skeleton.

Table 3 Moduli calculated from load test and crosshole shear wave velocity (from Swiger, 1974)

<table>
<thead>
<tr>
<th>Poisson’s Ratio</th>
<th>$\nu = 0.3$</th>
<th>$\nu = 0.45$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Modulus (E)</td>
<td>$3.7 \times 10^6$ psf</td>
<td>$4.2 \times 10^6$ psf</td>
</tr>
<tr>
<td>Load Test Modulus (E)</td>
<td>$3.9 \times 10^6$ psf</td>
<td>$3.4 \times 10^6$ psf</td>
</tr>
</tbody>
</table>

Figure 15. Modulus-strain relations used by Swiger, 1974 (from Seed, 1969)
At the site of the turbine room of the Shippingport nuclear power station, a site underlain by about 60 ft (18m) of medium dense to dense sand and gravel, strain-adjusted modulus determined from seismic wave velocity was compared to modulus computed from observed settlement by Swiger as shown in Figure 16. These moduli show very good agreement.

Since the time of Swiger’s paper, another important seismic method has become available permitting modulus profiles to be determined without the boreholes or any ground disturbance, namely surface-wave testing as discussed in Section 3. With this nondestructive and nonintrusive method, elastic moduli profiles for homogeneous and layered soil sites can be readily obtained for the purpose of settlement analysis and soil structure interaction. Surface-wave testing mitigates one of the high-cost elements of crosshole and downhole seismic testing, namely boreholes.

### 4.1.2 Case history 2 – settlement analysis of a power plant

Konstantinidis et al. (1986) reported a case study in which moduli developed for prediction of settlement of a power plant were based on the approach suggested by Swiger, 1974. The site consisted of both sand and clay layers of about equal thicknesses to a depth of 200 ft (61 m). Seismic wave velocities measured at the site are presented in Figure 17. Other methods of estimating moduli for settlement prediction considered by Konstantinidis et al. (1986) included the CPT, pressuremeter testing (PMT) and laboratory tests including consolidation and triaxial compression. Short-term settlement measurements (initial elastic settlement) showed about 1 inch (0.68 to 1.19 inch) [17 to 30 mm] of settlement for Unit 1 of a two-unit plant. Unit 2 did not have the same sequence of settlement measurements so it could not be compared.

Moduli from all methods used except moduli from seismic crosshole tests overestimated the measured settlement by a factor of at least two. Using the moduli from seismic crosshole tests, the estimated settlements were within +/- 15% of the measured settlements.

It is noteworthy that the laboratory tests, although performed with special refinements designed to eliminating sample disturbance and conducted on carefully sampled specimens, consistently produced unrealistically low estimates of soil stiffness. It was postulated that the highly overconsolidated soils at this site were more susceptible to disturbance during sampling than “average” soils. The overconsolidated state of this site, compared to a soft soil site, may have also added to the applicability of seismically determined moduli in this case.

The authors conclude that field-determined moduli produce better estimates of soil compressibility than laboratory tests, and that the modified version of Swiger’s suggested method based on seismic wave velocity measurements produced the most comprehensive and realistic assessment of settlement.

### 4.1.3 Case history 3 – settlement of a water tank

John Burland, in his Bjerrum Lecture, focused on the need for small-strain soil properties for many geotechnical problems, including soil structure interaction (Burland, 1989). He emphasized the nonlinear behavior of soils and promoted the use of strain-appropriate strength or stiffness for analysis. Since soil strains in most geotechnical problems fall in the range of 0.1% or smaller, Burland urged that geotechnical engineers recognize the importance of strain-appropriate soil properties. The development of laboratory techniques capable of precise measurement of small strains convinced him.
International Conference on Site Characterization (ISC-2), Porto, Portugal, September 19-22, 2004

that the gap between dynamic and static measurements of soil stiffness was being closed. Previously, dynamic measurements of soil stiffness gave values so much higher than static measurements that many engineers discounted the dynamic measurements. However, Burland cited cases where accurately determined static small-strain values of stiffness were compatible with seismically measured stiffness, giving greater credibility to dynamically measured values. As an example, he presented a case where Young’s modulus deduced from a static water tank loading test on Mundford Chalk produced stiffness very nearly the same as those determined from a seismic refraction survey. Figure 18 shows Young’s modulus versus depth determined by three methods: 0.86-m diameter plate loading tests at seven depths, finite-element back calculation, and seismic refraction.

Burland concluded that these results along with others open up the way for a whole new area of study linking dynamic and static deformation properties of soils. He speculated that studies of this kind would lead to wider application of geophysical measurements for determining in situ stiffness properties of geotechnical materials. The writers agree wholeheartedly.

4.1.4 Case history 4 – settlement analysis of footings

In 1994, the Geotechnical Division of ASCE held a settlement prediction symposium in conjunction with an ASCE Specialty Conference at Texas A&M University (Briaud and Gibbens, 1994). Full-scale footings of five sizes / configurations were tested to failure with detailed measurements of load-settlement. Thirty-one predictors were bold enough to make first class predictions of settlement based on detailed characterization of the site. Three of the 31 predictors used seismically determined moduli from crosshole tests to determine modulus for settlement prediction.

The predictors used 22 different settlement prediction methods based on soil properties determined by five field tests and two laboratory tests. Several measures of accuracy of prediction were compared with predictions including settlement at several stages of loading and the factor of safety at ultimate load. Table 4 is a compilation of factors of safety for the five footings computed from predictions by the 31 predictors. Those who used seismically determined moduli were numbers 22, 23 and 28. Two of those (numbers 22 and 28) were consistently better than the mean of all predictors. The other predictor, number 23, was better than the mean for the three larger footings. Although this prediction event did not specifically showcase settlement predictions based on seismically determined moduli, it provides further evidence that this method of prediction has potential for future application.

Table 4. Factors of Safety \( F = \frac{Q_l}{Q_d} \) (measured design load / predicted design load) [*ultimate with FS = 3] (from Briaud and Gibbens, 1994)

<table>
<thead>
<tr>
<th>No.</th>
<th>Authors</th>
<th>( Q_l / Q_d ) 1m</th>
<th>( Q_l / Q_d ) 1.5m</th>
<th>( Q_l / Q_d ) 2.5m</th>
<th>( Q_l / Q_d ) 3.0m</th>
<th>( Q_l / Q_d ) 3.5m</th>
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<tr>
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<td>Sidiquee</td>
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<td>6.47</td>
<td>4.03</td>
<td>3.70</td>
<td>4.26</td>
</tr>
</tbody>
</table>

Mean: 6.64 4.29 4.72 4.99 4.48
Standard Deviation: 5.23 5.90 4.12 4.83 4.13
Measured Value: 5 5 5 5

Figure 18. Young’s modulus determined from three methods for settlement analysis of a water trunk on Mundford Chalk (from Burland, 1989)
4.1.5 Summary

A few successful demonstrations of the use of seismically determined soil modulus for settlement predictions have been presented. Other case histories can be found in conferences dealing with the pre-failure deformation characteristics of geomaterials such as Shibuya et al. (1994), Jardine et al. (1998), Jamilolkowski et al. (2001), and Di Benedetto et al. (2003). The potential for use of this more rational approach to determining soil stiffness has been confirmed, but not yet widely adopted. In some cases, engineers report that obtaining the seismic wave velocities is too expensive (Konstantinidis et al, 1986), but with the development of surface-wave methods, the cost of boreholes has been eliminated. The writers hope that elimination of this cost impediment will allow broader application of seismically determined moduli for geotechnical engineering purposes.

4.2 Crosshole Seismic Velocity for Grouting Control

When soil improvement in the form of grouting is selected in geotechnical applications, some means of confirming the expected improvement is necessary. Seismic wave velocity can be used to quantify soil improvement by measuring wave velocities before and after grouting. Following are three case histories describing the successful use of seismic wave velocity to confirm the extent of ground improvement by grouting.

4.2.1 Case history 1 - forge foundations

Seismic shear wave velocities determined from crosshole tests were used to confirm the degree and extent of soil improvement from combined compaction and chemical grouting, (Woods and Partos, 1981). New forging machines were to be installed in a forge shop located on deep beach deposits near the Atlantic coast. Figure 19 shows blow count versus depth for two locations at this site, B1 and B3. The blow count ranged from 2 to about 20 in the upper 5 meters. The new forge machines were considerably larger than the old forges and vibrations within the plant from the new, larger installations were of concern as were differential settlements of the forges. Based on preliminary calculations, it was clear that the loose sand deposits were susceptible to shake-down settlement if large-amplitude vibrations occurred. There was also a potential for transmission of large-amplitude vibrations through the plant, so soil improvement in the form of grouting was selected to mitigate these concerns. Both compaction and chemical grouting techniques were chosen to be performed in series, with compaction grouting being performed as a first stage and then chemical grouting in a second stage if sufficient stiffness had not been achieved in the first stage.

The plan view of the site, Figure 20, shows the footprint of four new foundation blocks and locations for compaction grouting, chemical grouting and boreholes for crosshole tests. Figure 21 shows the shear wave velocity versus depth profiles determined from crosshole seismic tests after each of the two stages of grouting. Compaction grouting achieved the minimum shear wave velocity at this location, but chemical grouting was performed as an added factor of safety. The crosshole tests confirmed a significant increase in shear wave velocity leading to successful operation of forging machines at this site. No excessive settlements were observed and vibration levels throughout the plant were not noticeable.
4.2.2 Case history 2 - subway construction
One route of the subway in Pittsburgh, Pennsylvania was constructed under a part of Sixth Avenue, a narrow street with old, heavy masonry structures on both sides (ENR, 1982). The invert of the subway was well below the lower elevation of the spread footings supporting the heavy buildings. The foundation material consisted of coarse sand, gravel and cobbles. Construction of the subway called for excavation in a cut-and-cover process, but stability of the adjacent building foundations was in question. Chemical grouting was chosen to improve (stabilize) the soil, and crosshole seismic tests were used to confirm achievement of sufficient improvement over the un-grouted condition.

Crosshole equipment was fabricated that made use of the grout pipes for placement of the source and receivers. Shear wave velocities were used to characterize the soil before and after chemical grouting. A target shear wave velocity was determined in the laboratory using resonant column tests, and a test section of crosshole tests was performed at the site to confirm expectations from the laboratory study. Before-grouting shear wave velocities ranged from about 500 ft/sec to 1000 ft/sec (150 m/s to 305 m/s) and after-grouting velocities ranged from about 1400 ft/sec to 3000 ft/sec (425 m/s to 915 m/s). The criteria for satisfactory soil improvement by grouting was either: (1) a doubling of the shear modulus (1.41 times increase in shear wave velocity) over the before-grouting condition, or (2) a minimum of 1400 ft/sec (425 m/s). The Sixth-Avenue section of the subway was successfully completed without disturbance of the adjacent buildings.

4.2.3 Case history 3 - old bridge support
New design loads on a railway line in Italy required structural and ground improvements along the entire line and particularly on two XIX century masonry arch bridges (Volante et al., 2004). A need for careful ground movement control during upgrading of the old bridges led to design of a multistage-multiport, low-pressure grouting technique. Careful grout control was exercised during the injection process, but in the long run it was important to determine the strength and deformation parameters of the newly grouted soil. Figure 22 shows P-wave and S-wave velocities before and after grouting at one of the bridge sites. In this case, the modulus of the ground under the piers was increased by a factor of about 2.5.

4.2.4 Summary
The three case histories presented here clearly show that seismic wave velocities can be used to advantage in confirming quality and extent of grouting operations. While the examples cited all used the crosshole seismic method, applications of surface wave methods may provide economies where there is sufficient lateral extent to apply them. For a broad-area dynamic compaction project, the writers have also successfully used the SASW method to confirm ground improvement by showing increased shear wave velocities. Stokoe and Santamarina, 2000 have also shown evaluation of blast densification by the SASW method.

4.3 Underground Cavity Detection
Many engineering situations require the determination of the existence or absence of underground obstacles, solid or void, as well as their locations and depths. Probing for these obstacles with penetrometers is a time consuming and expensive
process. Several currently available geophysical techniques have been proposed and used to identify underground anomalies. The following example and two case histories describe the use of some of these techniques.

4.3.1 Example 1 – use of GPR, SASW and crosshole testing for cavity detection

Three geophysical methods were studied for the detection of buried cavities (Al-Shayea, 1994, and Al-Shayea et al., 1994). A soil bin, 7 m in diameter and 2 m in depth, was used in this study. The soil bin is shown in Figures 23 and 24. A three-cell cavity was buried in the bin at a depth to center of the cavity of 614 mm. SASW data were collected along five lines identified by the source and receiver symbols and the skew lines marked a-c on Figure 23. The general SASW test arrangement is shown in Figure 24. Ground-penetration radar (GPR) data were also collected along the grid lines identified on Figure 23. Crosshole shear-wave tests were performed across both long and short axes of the buried cavity and in the free-field. GPR gave the most obvious identification of the cavity as indicated on Figure 25. In this figure, it is very clear where the electromagnetic wave field produced by GPR was distorted, (Figure 25b) compared with the wave field of the free-field (Figure 25a). Approximate depth and size of the cavity were calculated from Figure 25b knowing the frequency of the GPR source and the dielectric constant of the sand.

SASW tests were performed directly over the centerline of the cavity with various states of filled and empty cells. (The cells were filled using the same sand as in the bin.) Differences in cavity filling showed substantially different dispersion curves, Figure 26. A smoothed free-field dispersion curve is shown as a solid line while the empty and partially empty void dispersion curves are shown with other symbols. For this discussion, the symbols representing cavity conditions need not be identified because the key result is that the existence of a void and the size of the void both influenced the shape of the dispersion curve.
Other SASW tests were performed directly over and on lines skewed to the centerline of the cavity. These varying lines also showed substantially different dispersion curves.

Results of SV-wave seismic crosshole tests performed on lines S-R1 (free-field) and S-R2 (over cavity) in Figure 23 are presented in Figure 27. Here it can be seen that the shear wave profile for the free-field direction is quite consistent for three conditions of the void, all cells empty, 1 cell filled with sand, and all cells filled with sand. The cases of all cells filled did not exactly match the free-field condition because the cell filling process could not duplicate the free field density of the sand. In the case of line S-R2, the average shear wave velocity in the depth region of the cavity was clearly reduced by the existence of the cavity. Had the crosshole boreholes been closer together, shear wave velocity differences would have been more dramatic at depths representative of the cavity.

All three of the geophysical techniques used in this study have characteristics that allow identification of cavities or anomalies in the underground, but each have their limitations. To mention just the most salient drawbacks, GPR suffers from ability to penetrate clay, SASW is limited by the need to run multiple lines of data, and crosshole boreholes need to strategically located to straddle the cavity. Some of these limitations may be minimized through future study while others are inherent to the basic principles of the geophysical methods. Some of these drawbacks simply point to the necessity of performing suites of complementary geophysical tests for cavity detection.

It is worth noting that some of the drawbacks of the SASW method that were cited above for cavity detection may be mitigated by using recently developed wavelet theory (Shokouhi and Gucunski, 2003, and Gucunksi and Shokuohi, 2004). Continuous wavelet transforms (CWT) are a new class of transformations that can produce a time-frequency map of the ground surface from which indications of near-surface cavities can be derived. The data collection required for CWT can be achieved simultaneously with SASW data collection.

4.3.2 Case history 1- mine collapse under a highway

High resolution SH-wave reflection tests were performed along the right-of-way of Interstate Highway I-70 in southeastern Ohio, (Guy et al., 2003). A portion of the east bound lane of I-70 at this location collapsed into old underground coal-mine workings. A plan view of the east bound lanes of I-70, the location of the collapsed highway, and a projection of the old underground mine workings are shown in Figure 28. High-resolution SH-reflection surveys were performed along two lines straddling the east bound lanes of I-70, lines GUE-I70-1 and EBPassYY. The interface between soil and rock was clear for most of the lengths of both of these survey lines, but in the interval between stations 48320 and 48360 on line EBPassYY the SH-wave stacking velocity plot showed a discontinuous segment of the soil/rock interface in the region indicted by the angled bars in Figure 29.
and on the geologic cross section shown in Figure 30. The continuous soil/rock interface as interpreted is indicated on Figure 29 by the dash-dot line across the plot. Soil borings and rock coring were performed at six locations selected by interpreting the soil/rock interface reflector in Figure 29. These voids indicate stopping from the former coal mine upwards, but that stopping had not progressed to the pavement level.

This seismic method penetrated relatively deep, about 20 meters in this case. The results of the tests show a potential for future subsidence and sinkhole development along this highway. Based on this kind of information, remedial efforts could be applied to stop progression of the stopping or stabilize the ground above the current voids.

Because high resolution SH reflection utilizes steady-state ground excitation, wavelengths can be controlled for good wavelength/cavity size ratios and may permit better cavity size and depth determination than other seismic wave based cavity detection methods.

4.3.3 Case history 2 – other geophysical testing at the mine collapse under highway I-70

All of the geophysical techniques cited in Section 4.3.1 (except CWT) were applied by Hiltunen et al., (2004) at the mine-collapse site on I-70 described above. The results confirmed the draw backs cited previously. However, the work by Hiltunen et al. confirmed that quality geophysical measurements could be made in close proximity to an active interstate highway with heavy truck traffic. Both GPR and SASW testing at this site could not probe deep enough to explore the soil/rock interface, GPR because of clay in the soil and SASW because of the lack of a sufficiently energetic excitation source. The crosshole test did not happen to encounter a void or loose soil. A major conclusion from these tests was that no single technique could unambiguously detect voids or other anomalies throughout a wide range of depths.
4.3.4 Summary
While geophysical methods, including several seismic wave propagation methods, have considerable potential for locating and sizing buried objects, no single method is appropriate for all sites. In the past, engineers and geophysicists have often chosen one method in an attempt to locate cavities or buried objects and have been disappointed with the results. Indications from the cases cited herein are that, while any one method may provide some parts of the identification puzzle, a suite of tests can provide more confidence in finding cavities or buried objects.

4.4 Tunnel Investigation
Sometimes, $V_s$ measurements are used to profile constructed systems and their geotechnical foundation materials to assist in forensic studies. A forensic study of a concrete-lined tunnel in rock is described below (Stokoe and Santamarina, 2000). A generalized cross section of the tunnel is shown in Figure 31a. The tunnel is approximately 3 m in diameter, with a concrete liner that has a nominal thickness of 30 cm.

An extensive investigation was conducted in which SASW testing was performed at more than 100 locations along the longitudinal axis of the tunnel. SASW testing was performed with handheld hammers as sources and accelerometers as receivers. The accelerometers were held magnetically to metal disks attached to the liner. This general configuration is shown in Figure 31b. Testing was conducted to profile along two planes into the liner-rock system. One profile was along the springline, and the other profile was near the crown as illustrated in Figure 31b.

The SASW testing program was designed to investigate the following: 1. thickness and quality of the concrete liner in the springline and crown areas, 2. thickness and quality of any grout in the area of the crown, 3. identification of any voids in the crown area, and 4. stiffness and variability of the rock behind the liner. (Grouting in the crown area was done some time after construction of the liner.) The program successfully answered these questions. Examples showing how some of the questions were answered follow.

An interpreted $V_s$ profile at one springline location is shown in Figure 32a. The profile shows a high-quality concrete liner ($V_s > 2500$ m/s) that is about 35 cm thick. At this location, the liner is in direct contact with the rock, and the rock is stiffer (and presumably stronger) than the concrete.

Results from one crown location are shown in Figure 32b. In this case, the liner is thicker than 40 cm, and there is grout between the liner and the rock. Based on the $V_s$ values, both the concrete and grout are high quality. The concrete-grout-rock interfaces have intimate contact; hence, no voids. Also, the rock is less stiff than the concrete at this location.

Clearly, SASW testing was successfully and cost-effectively applied in the tunnel investigation. The writers have had other successful projects in many other underground applications (for instance, Madianos et al., 1990, Olson et al., 1993, and Luke et al., 1998).

4.5 Offshore Shear-Wave Velocity Profiling Using Seismic Interface Waves
As offshore construction moves into deeper water (depths greater than 1.6 km), traditional drill-and-sample geotechnical site investigations become expensive and less reliable. The expense of drilling in deep water often dictates the extraction and testing of only a few samples. Furthermore, the quality of these samples can be severely compromised when extracted through great water depths. Other geotechnical site investigation methods, such as the seismic cone penetrometer (e.g. Robertson et al., 1986), are effective on land and in shallow water, but become more difficult and costly to apply.
in the deep-water environment. One seismic method that has potential for deep-water seafloor investigation is the surface-wave method for $V_s$ measurements of the sediment. Shear-wave velocity is used because it is essentially unaffected by the presence of water (and air if the sediment is unsaturated) and because, in a saturated soil, $V_s$ is far better correlated with shear strength than $V_p$ wave, which has been used to predict shear strength in previous offshore investigations (Blake and Gilbert, 1997).

The SASW method is being applied to the offshore environment to determine shear-wave velocity profiles of the seafloor. In this application, a soil-water interface wave, called a Sholte wave (Wright et al., 1994), is measured. The measured Sholte-wave dispersion curve is used to determine a shear-wave velocity profile. Figure 33 shows the results from SASW testing performed by ConeTec, Inc. off the coast of Vancouver, B.C. (Rosenblad and Stokoe, 2001). The soils in this region were composed of loose silty-sands and sands that were unsaturated (gaseous). Testing was conducted in shallow water depths, ranging from 12.2 to 76.2 m, to demonstrate the potential of the method. Personnel from ConeTec also conducted SCPT measurements at the site. The SASW results agree reasonably well with the SCPT values at depths between 5.5 and 10 m as seen in Figure 33. The differences that are observed are likely due to the localized versus global nature of the SCPT and SASW tests, respectively. Based on the writers’ experience on land, development of a lower-frequency source and longer arrays would allow SASW profiling to significantly greater depths at this site, certainly to depths on the order of 30 to 50 m. However, the results do demonstrate the feasibility of the surface-wave method offshore. Stoll et al. (1994), Luke and Stokoe (1998), Rosenblad (2000), and Rosenblad et al. (2003) are among others who have also shown this surface-wave application.

### 4.6 Geotechnical Earthquake Engineering

The importance of the shear stiffness of geotechnical materials in calculating their response during dynamic loading initially stimulated the development of in situ seismic methods tailored to measure $V_s$. This development began in earnest in the 1960s with modifications/refinements to the crosshole and downhole methods. It is continuing today with improvements to surface-wave methods. The following two case histories and one application describe some recent work in geotechnical earthquake engineering.
4.6.1 Case history 1 - profiling hard-to-sample alluvium

Yucca Mountain, Nevada, was approved as the site for development of the geologic repository for high-level radioactive waste and spent nuclear fuel in the United States. The U.S. Department of Energy has been conducting studies to characterize the site and assess its future performance as a geologic repository. As part of these studies, a comprehensive program of in situ seismic investigations was performed at the proposed site of the Waste Handling Building (WHB). The purpose of these investigations was to characterize the velocity structure of the subsurface for seismic design of the WHB facilities.

In situ seismic velocity measurements were performed by three different methods at this site. The seismic methods include two borehole methods, downhole and suspension logging, and one surface-wave method, spectral-analysis-of-surface waves (SASW). The borehole surveys were conducted in 16 cased boreholes to a maximum depth of 198 m. SASW surveys were performed at 34 locations around much of the proposed area which was about 300 m by 450 m in plan dimensions. The SASW surveys were aimed at evaluating the top 50 m of the site and investigating lateral variability. The SASW surveys provided greater spatial coverage of the site while the borehole surveys added critical deeper information.

Stokoe et al., 2003 presented a comparison of the \( V_s \) profiles determined by the three seismic methods in the material where the most overlap in measurements existed. This material is a hard-to-sample Quaternary alluvium/colluvium (Qal) which ranges from a poorly graded gravel (GP) to a silty gravel (GW). The alluvium contains varying amounts of sand, cobbles and boulders, and it varies in thickness, depth, and amount of cementation over the WHB site. The alluvium was measured in 15 of the 16 boreholes. The results form the most comprehensive set of \( V_s \) measurements, in terms of multiple seismic methods in a localized area and in one material type, that has ever been compared. In addition, this comparison represents a “blind comparison” overseen by URS personnel, in that each measurement team was not aware of the other’s results until after they were all submitted to URS. Therefore, not only did the seismic tests at the WHB site provide the information needed for the seismic design of the facility, but they also provided an interesting comparison of \( V_s \) profiles measured by different methods as described below.

The comparison of the \( V_s \) profiles is presented as an average profile for each method. The average profiles were determined by first dividing the 1.5- to 30-m depth range into four intervals, with the smallest near the surface where the \( V_s \) gradient was the greatest and largest at depth where the gradient was the smallest. This division allowed the overall characteristic \( V_s \) profile of the alluvium to be preserved. The four depth intervals were: 1.5 to 4.6 m (layer no. 1), 4.6 to 9.2 m (layer no. 2), 9.2 to 18.3 m (layer no. 3), and 18.3 to 30.5 m (layer no. 4). An example of the results from this procedure at one borehole is presented in Figure 34. Figure 34a shows the profiles determined from downhole, suspension logging and SASW measurements in and near borehole RF-19. Figure 34b shows the averaged \( V_s \) profiles.

Comparison of the average \( V_s \) profiles is presented in Figure 35. Also shown in the figure is the variability in the \( V_s \) values measured with each method, expressed by ± one standard deviation.
The data are presented numerically in Table 5, including the coefficient of variation (COV = \( \sigma / \text{Avg } V_s \)). The comparison is shown in Figure 35 and demonstrates the strength and robustness of S-wave velocity measurements today. First, identical trends of increasing \( V_s \) with depth were measured with all three methods. Second, differences in average \( V_s \) values in each layer are small. The two largest differences are 16% and 12% and are found between the downhole and SASW measurements in layers no. 1 and no. 2, respectively. These layers are the shallowest layers (less than 9.2 m deep) and are the ones which should be expected to show the largest differences due to: (1) the SASW method having the most resolution of these methods near the surface, (2) the averaging effect of placing straight-line segments through the measured travel times in downhole data reduction, and (3) wave refraction and lateral variability in the Qal affecting each method differently. The standard deviations determined from the measurements and the COV values support points (1) and (2) above. The COV values are about 0.21 for SASW measurements in layers no. 1 and no. 2 and about 0.11 for the downhole measurements in the same layers. It should be noted that the variability shown by \( \pm \sigma \) includes both measurement uncertainty and variability in material properties.

4.6.2 Case history 2 - deep \( V_s \) profiling

Another part of the seismic investigations at the Yucca Mountain site discussed above involved deep \( V_s \) profiling along the top of Yucca Mountain (Stokoe et al., 2004). Deep profiling is defined as evaluating the shear-wave velocity structure to depths of about 200 m. This work involved the SASW method and required the use of a Vibroseis (see Figure 36) to generate the low-frequency (hence long wavelength) waves necessary to profile to 200 m. Yucca Mountain consists of stacked layers of tuffs with \( V_s \) generally above 900 m/s. Therefore, the lowest excitation frequency was in the range of 3 Hz and the farthest measurement point from the source was around 500 m.

SASW measurements were performed at 22 array sites along the top of Yucca Mountain. These sites were spread over a distance of about 5 km. The SASW surveys were aimed at evaluating: (1) the top 150 to 200 m of the mountain, (2) an apparent \( V_s \) gradient in the near surface (within about 5 to 15 m), and (3) any lateral variability over the 5-km distance. The mean \( V_s \) profile that was determined from the 22 profiles is presented in Figure 37 along with the 16th and 84th percentile \( V_s \) values. The coefficient of variation (COV) about the mean profile was calculated by assuming the \( V_s \) values follow a lognormal distribution.

Table 5 Numerical Analyses of Average \( V_s \) Profiles (from Stokoe et al., 2003)

<table>
<thead>
<tr>
<th>Depth Interval (m)</th>
<th>Downhole Surveys</th>
<th>SASW Surveys</th>
<th>Suspension Logging Surveys</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. of Meas.</td>
<td>Avg. ( V_s ) (m/s)</td>
<td>St. Dev., ( \sigma ) (m/s)</td>
</tr>
<tr>
<td>1.5–4.6</td>
<td>10</td>
<td>432</td>
<td>53</td>
</tr>
<tr>
<td>4.6–9.2</td>
<td>12</td>
<td>526</td>
<td>56</td>
</tr>
<tr>
<td>9.2–18.3</td>
<td>13</td>
<td>662</td>
<td>59</td>
</tr>
<tr>
<td>18.3–30.5</td>
<td>8</td>
<td>732</td>
<td>43</td>
</tr>
</tbody>
</table>

1 Suspension logging was not successfully performed at shallow depths due to high attenuation, backscattering and tube-wave interference.
2 Insufficient data to perform meaningful calculations.
3 SASW surveys were performed at one less borehole than the downhole surveys due to surface obstructions.
Several interesting trends are evident in Figure 37. First, it is observed that the near-surface $V_s$ gradient is quite abrupt. The values of $V_s$ change from approximately 300 m/s in the top meter to over 800 m/s at a depth of only 5 m. Below 5 m, the mean $V_s$ value increases gradually from about 900 to 1000 m/s at a depth of 150 m. A gradient near the surface is expected due to the effects of weathering on the near-surface rock. The abruptness of the gradient, which has important implications in terms of the ground motion hazard, can not be predicted without field measurements of this kind. Another important result shown in Figure 37 is the nearly constant value of the mean $V_s$ below the 5-m-thick, near-surface zone. The measurements show a mean value of approximately 1000 m/s in the depth range of 10 to 150 m.

4.6.3 Application 1 - liquefaction resistance
Evaluation of the liquefaction resistance of soils can be a critical factor in many geotechnical engineering investigations. Such an evaluation is typically performed with field test such as the standard
One of the strengths of seismic measurements, as well as other geophysical measurements, is that the same basic measurement can be performed in the field and in the laboratory. Field measurements of $V_s$ are performed in the small-strain or elastic range as discussed earlier. Therefore, laboratory measurements of $V_s$ also have to be evaluated in this small-strain range if they are to be compared directly with the field values. This comparison presently forms the way sample disturbance is evaluated in geotechnical earthquake engineering when dealing with nonlinear deformational characteristics. Field and laboratory values of $V_s$ at small strains are used to adjust the nonlinear response of soil measured in the laboratory to field conditions. The nonlinear response is typically shown in terms of the nonlinear variation in shear modulus with shearing strain ($G – \log \gamma$). This comparison, the adjustment procedure, and the impact on the $G – \log \gamma$ and stress-strain ($\tau – \gamma$) curves are discussed below.

5.1 Comparison of Small-Strain Field and Laboratory Values of $V_s$

Invariably, when field and laboratory values of $V_s$ are compared, values of $V_{s,\text{lab}}$ range from slightly less to considerably less than the in situ values, $V_{s,\text{field}}$ (Anderson and Woods, 1975, Long, 1980, Yasuda and Yamagushi, 1985, Yokoa and Konno, 1985, and Chiara, 2001). A just-completed project dealing with the resolution of site response issues in the 1994 Northridge, CA earthquake, called the ROSRINE project, involved numerous field and laboratory investigations. Sixty-three intact samples were recovered and tested in the laboratory at the University of Texas using combined resonant column and torsional shear equipment (Darendeli, 2001, and Choi, 2003). Additionally, in situ seismic measurements were performed during the field investigation phase, mainly by GeoVision Geophysical Services, Corona, CA using a suspension logger. Therefore, the ROSRINE project afforded an excellent opportunity to investigate further the relationship between field and laboratory values of $V_s$.

An example field $V_s$ profile measured in this study is presented in Figure 39. At this site, called La Cienega, in situ seismic tests (shallow crosshole testing and deep suspension logging) were performed. A depth of nearly 300 m was logged. Intact samples were recovered from depths ranging from 4 to about 240 m. The laboratory values of $V_s$, shown by the solid circular symbols, are plotted at the corresponding sample depths. There is considerable variability in the field $V_s$ profile. The
“average” field values associated with the laboratory values are shown by the short vertical lines through the field \( V_s \) profile in the vicinity of the sample depth.

A summary of all field-lab \( V_s \) comparisons from the ROSRINE project is presented in Figure 40. A total of 63 samples were tested in the laboratory. There is a clear trend in the data, with the velocity ratio \( \frac{V_{s,\text{lab}}}{V_{s,\text{field}}} \) decreasing as the in situ value of \( V_s \) increases. (There was essentially no correlation with sample depth.) In general terms, the velocity ratio is around one at \( V_s = 160 \) m/s. However, at \( V_s \approx 725 \) m/s, the velocity ratio is about 0.6, which means that the small-strain shear modulus from laboratory testing is on the order of 1/3 of the value in the field. This comparison strongly supports the need to perform field seismic tests, certainly in studies dealing with siting and retrofitting of important facilities.

### 5.2 Estimated Field \( G - \log \gamma \) Curves from Field and Laboratory Measurements

Once the \( V_s \) profile has been determined at important or high-risk sites, the next step in the geotechnical earthquake engineering investigation is determination of the nonlinear characteristics of the soil. This step typically involves cyclic and/or dynamic laboratory testing of intact specimens. In terms of nonlinear shear modulus, these results are presented in the form of the variation in normalized modulus, \( \frac{G}{G_{\text{max,lab}}} \), with shearing strain amplitude, \( \gamma \), or simply \( G - \log \gamma \). Typical examples of \( G - \log \gamma \) curves from the ROSRINE project for soft, moderately stiff and very stiff soils are shown in Figures 41a, 41b, and 41c, respectively. The laboratory \( G - \log \gamma \) curves are shown by the solid lines in the figures.

With the in situ \( V_s \) value and the laboratory \( G - \log \gamma \) curve, the final step is to estimate the field \( G - \log \gamma \) curve. This step is accomplished by scaling the laboratory \( G - \log \gamma \) curve using \( G_{\text{max}} \) determined from the field seismic tests as,

\[
G_{\gamma,\text{field}} = \left( \frac{G_{\gamma,\text{lab}}}{G_{\text{max,lab}}} \right) G_{\text{max,field}}
\]

where,

- \( G_{\gamma,\text{field}} \) = in situ shear modulus at a shearing strain of \( \gamma \),
- \( G_{\gamma,\text{lab}} \) = shear modulus determined in the laboratory with an intact specimen at a shearing strain of \( \gamma \),
- \( G_{\text{max,lab}} \) = small-strain shear modulus determined in the laboratory, and
- \( G_{\text{max,field}} \) = in situ shear modulus measured by seismic testing.

It is assumed, of course, that evaluation of the \( G - \log \gamma \) curve in the laboratory was performed at a confinement state, excitation frequency, number of loading cycles, drainage condition, etc. that represent the field conditions. Also, \( G_{\text{max,field}} \) was calculated from \( V_{s,\text{field}} \) using Equation 2.

The estimated field \( G - \log \gamma \) curves are shown by the dashed lines in Figures 41a, 41b, and 41c for the soft, medium stiff and very stiff soil examples taken from the ROSRINE project. Clearly, adjustment of the laboratory curve is critical to correctly predicting the earthquake ground motions at the site.

### 5.3 Laboratory and Field Stress-Strain (\( \tau - \gamma \)) Curves

The laboratory shear stress-shear strain (\( \tau - \gamma \)) curve can be calculated from the laboratory \( G - \log \gamma \) curve as,

\[
\tau = G \ast \gamma
\]
Figure 41. Measured laboratory $G \log \gamma$ curves, estimated field $G \log \gamma$ curves using $V_{s,\text{field}}$, and possible range in field $G \log \gamma$ curves using range in Figure 40 but no measurement of $V_{s,\text{field}}$. Examples for soils from the ROSRINE project with a range in shear stiffness

a. $V_{s,\text{in-situ}} = 180$ m/s (soft soil) and $V_{s,\text{lab}}/V_{s,\text{field}} = 0.92$

b. $V_{s,\text{in-situ}} = 360$ m/s (moderately stiff soil) and $V_{s,\text{lab}}/V_{s,\text{field}} = 0.81$

c. $V_{s,\text{in-situ}} = 540$ m/s (very stiff soil) and $V_{s,\text{lab}}/V_{s,\text{field}} = 0.60$

Figure 42. Calculated laboratory $\tau - \gamma$ curves, estimated field $\tau - \gamma$ curves using $V_{s,\text{field}}$, and possible range in field $\tau - \gamma$ curves using range in Figure 40 but no measurement of $V_{s,\text{field}}$. Examples for soils from the ROSRINE project with a range in shear stiffness
with companion sets of $G - \gamma$ values taken from the dynamic laboratory curve. The laboratory $\tau - \gamma$ curves that were derived from the laboratory $G - \log \gamma$ curves for the soft, medium stiff and very stiff soils in Figures 41a, 41b and 41c are shown by the solid lines in Figures 42a, 42b and 42c, respectively. The estimated field $\tau - \gamma$ curves for these soils were determined following the procedure expressed by Equation 9, except that the estimated field $G - \log \gamma$ curves were used. The estimated field curves are shown by the dashed lines in Figure 42.

This example is presented to show the importance of $V_s$, field in predicting the field $\tau - \gamma$ curves which are used for deformational analyses like the ones presented in Section 4.1. In such deformational analyses, shear strains rarely exceed 1% which is the reason why the $\tau - \gamma$ curves are shown with a scale of 0 to 1%. Unfortunately, many geotechnical engineers are not aware of this adjustment procedure for sample disturbance or the importance of $V_s$.

5.4 What If No In Situ $V_s$ Values Are Measured?

At times, the owner or client may elect to test only intact samples and not perform in situ $V_s$ measurements. This decision may be based on cutting cost, incomplete understanding of the importance of $V_s$, field or other reasons. In any case, the field $G - \log \gamma$ and $\tau - \gamma$ curves can not be estimated using the adjustment procedure discussed above. Therefore, a wide range in the estimated nonlinear field curve will result. Figure 40 can be used in reverse to find the range in the expected field curves if one only had laboratory $G - \log \gamma$ curves. These ranges are shown by the shaded zones in Figure 41 for the three different soil stiffnesses. The ranges are quite large, exceeding factors of two and three for the moderately stiff and very stiff soils, respectively. The same relative comparison is shown by the shaded zones in Figure 42 for the ranges in expected $\tau - \gamma$ curves.

5.5 What If No Laboratory $G - \log \gamma$ Curves Are Measured?

Obviously, the other situation that the geotechnical engineer might face is having only the in situ $V_s$ measurements. In the writers’ opinion, this situation may not be as troublesome as the case above with no $V_s$, field values. Hopefully, the engineer has a boring log and the soil types identified. In this case, empirical soil models can be substituted for the laboratory $G - \log \gamma$ curves (assuming no unusual or difficult soils). The empirical models should include variables such as soil type, confinement state, some measure of uncertainty, etc. such as the model by Darendeli (2001). However, this approach would only be used if no laboratory $G - \log \gamma$ curves were measured and will result in wide ranges for the estimated field $G - \log \gamma$ and $\tau - \gamma$ curves.

6 CONCLUSIONS

Geophysical methods have an important and ever-increasing role to play in the solution of geotechnical engineering problems. Seismic methods have been embraced by geotechnical engineers over the past 50 years. They have been heavily used in the solution of soil dynamics and geotechnical engineering problems, especially in the evaluation of small-strain shear and compression stiffnesses. Today, $V_s$, measurements in the field and laboratory form a critical link in evaluating sample disturbance and in predicting nonlinear $G - \log \gamma$ and $\tau - \gamma$ curves.

The adoption of geophysical methods in the solution of non-dynamic problems has occurred more slowly in geotechnics, excluding geoenvironmental and military applications. Seismic testing is still the most widely used method, particularly for evaluating site characteristics (layering, top of bedrock, voids, etc.) and monitoring processes (grouting, damaged or changed zones from construction activities, etc.). The use of strain-adjusted moduli in settlement and other deformational analyses offers a rational approach to the solution of many of these problems. This approach will continue to grow.

The seismic method that continues to evolve in geotechnical engineering is the surface-wave method. The nonintrusive nature of the method makes its application very cost effective, and its usefulness will continue to increase. It would be very beneficial to this method, as well as other geophysical methods, to incorporate increased automation. Adoption by the profession would also benefit from increased coverage of geophysical methods in the civil engineering curriculum. Finally, the engineer also needs to consider that the robustness of the solution is significantly enhanced in many applications by the use of a suite of geophysical measurements.

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FIELD SEISMIC TESTING IN GEOTECHNICAL EARTHQUAKE ENGINEERING

Kenneth H. STOKOE, II

ABSTRACT

Field seismic testing is an active and growing area in geotechnical earthquake engineering. The primary purpose of the field tests is to develop compression-wave velocity ($V_p$) and shear-wave velocity ($V_s$) profiles. These profiles are used to represent the stiffnesses of the geologic materials in the small-strain range. Many seismic methods are available for shallow investigations as discussed in the paper. Shallow investigations are defined as profiling to depths less than 75 m. Developments are occurring in profiling to intermediate (75 to 225 m) and deep (greater than 225 m) depths. The seismic methods used for deeper profiling are the downhole, suspension logging and surface wave methods. Examples of deeper profiling are presented. In addition, field seismic methods are being developed to perform parametric studies in situ. Examples are presented that show in-situ measurements of the effects of: (1) stress state on $V_s$ and $V_p$, (2) nonlinear straining on shear modulus, and (3) cyclic loading leading to liquefaction.

Keywords: deep profiling, field testing, in-situ parametric studies, seismic measurements

INTRODUCTION

The starting point when evaluating the response to earthquake shaking of critical facilities founded on or embedded in the earth is small-strain stiffness profiles, expressed by the variation of compression-wave velocity ($V_p$) and shear-wave velocity ($V_s$) with depth (Kramer, 1996). Profiles of $V_p$ and $V_s$ are measured in the field using seismic methods. The seismic method or combination of seismic methods employed in field investigations depends upon: (1) the geologic profile, (2) the maximum profiling depth, (3) the size of the investigation area, and (4) the critical level assigned to the facilities or structure. In many instances, the field investigation is limited to the top 30 m at one to three locations. The purpose of the investigation is to evaluate the average $V_s$ over the top 30 m ($V_{s,30}$) for use in a code-based design. Both intrusive and nonintrusive seismic methods are used in these shallow investigations and only one field method is employed. Intrusive methods used in such investigations are the crosshole, downhole, seismic cone penetrometer (SCPT), seismic flat-plate dilatometer, and suspension logger (P-S logger). The nonintrusive methods are surface-wave methods and are based on measuring Rayleigh-type waves. The spectral-analysis-of-surface-waves (SASW) and multi-channel-analysis-of-surface-waves (MASW) methods are two active-source methods employed in shallow investigations.

The state of practice in shallow seismic investigations is good and improving. As noted above, a number of field seismic methods are readily available for use. Improvements in analysis methods, instrumentation and automation associated with the methods are occurring. The number of knowledgeable engineering practitioners is also increasing, and areas of application are growing. Surface-wave testing is the most rapidly growing area, due in large part to the efficient and cost-effectiveness of nonintrusive testing. Developments are also occurring in two other aspects of field

1 Jennie C. and Milton T. Graves Chair in Engineering, Department of Civil, Architectural and Environmental Engineering, The University of Texas at Austin, Email: k.stokoe@mail.utexas.edu
seismic testing. The first is profiling to deeper depths in all types of geologic settings. The second is performing parametric studies in situ. The effects of parameters such as stress state, strain amplitude, and cyclic loading leading to liquefaction are being evaluated in situ. Developments in both aspects are briefly discussed below and are covered in more detail in the presentation.

DEEPER SEISMIC PROFILING

In the past decade, considerably deeper investigations have been required at critical sites in the United States, with profiling depths in the range of 125 to 450 m. Intermediate and deep wave-velocity profiles have been measured with two or three seismic methods at several of these sites. Intermediate depth profiles are defined as having maximum depths in the range of 75 to 225 m and deep profiles have depths exceeding 225 m. (Shallow profiles are defined as profiles less than 75 m deep.) The seismic methods used in deeper profiling have been downhole testing, surface-wave based tests and P-S suspension logging as discussed below.

Intermediate and deep profiles have been measured at several locations including: (1) Yucca Mountain, Nevada, (2) the northern Mississippi embayment, (3) the Salt Lake Valley, Utah, and (4) the Hanford Site near Richland, Washington. The spectral-analysis-of-surface-waves (SASW) method was used at each location, and measurements were performed over lateral extents ranging from 10 to 200 km. The large vibrator used as the SASW source is called “Liquidator”. Liquidator is shown in Figure 1a. It is a one-of-a-kind, low-frequency vibrator that is specially designed to give high-force output (a peak force of about 90 kN) in the low-frequency range (about 1 to 4 Hz). Liquidator is part of a shared-use equipment site that is operated by the University of Texas at Austin (nees@utexas) and funded by the U.S. National Science Foundation (NSF) as part of the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES). Statistical analysis of nine, intermediate-depth Vs profiles of the alluvium at the Hanford Site is presented in Figure 2. The median Vs profile and the associated coefficient of variation (COV) are shown in Figures 2a and 2b, respectively. These results compare well with measurements from independent downhole tests, with the average median value of Vs from each test method within 4% over the same test depths. However, due to various constraints, downhole testing was only performed to an average depth of about 70 m.

Deep Vp and Vs profiles have also been measured by the downhole and P-S suspension logging methods at the Hanford Site. In this case, testing was performed in three boreholes spaced around the footprint of a set of buildings. Testing was conducted to a maximum depth of about 440 m. In the downhole test, generating measurable compression (P) and shear (S) waves at depths of 300 to 400 m was challenging due to the alternating layers of soil interbeds and basalt. Therefore, the triaxial vibrator called, “T-Rex” was used as the seismic source. (Measurements with the P-S suspension logger were easier to perform at the deeper depths since the wireline tool containing the source and receivers is lower to the test depths.) T-Rex is another large vibrator operated by nees@utexas. T-Rex is capable of generating large dynamic forces in any of three directions (X, Y, or Z directions). The peak forces are about 270 kN in the vertical (Z) direction and about 135 kN in either horizontal direction. The vibration direction can be changed at the touch of a button. These capabilities, combined with the ability to prescribe the source signal, made T-Rex an excellent source for deep downhole profiling. The use of 20- to 50-Hz, fixed-sine input signals with 4 to 10 cycles worked well. Example travel-time records are presented in Figure 3. Sets of P- and S-wave records at depths of 137 and 290 m are shown in Figures 3a and 3b, respectively. Relative times of the direct P or S wave on each waveform are shown in the figure. These relative points were tracked throughout the complete depth range from which Vp and Vs profiles were determined.
Figure 1. Large mobile vibrators used as controllable, high-energy seismic sources: (a) the low-frequency vibrator called Liquidator and (b) the tri-axial vibrator called T-Rex

Figure 2. Statistical analysis of intermediate-depth $V_s$ profiles in an alluvial deposit from SASW testing: (a) median and ± one standard deviation, (b) coefficient of variation and (c) number of profiles (from Lin, 2007)

Figure 3. Example compression (P) and shear (S) wave records using T-Rex as the downhole seismic source: (a) receiver at an intermediate depth of 137 m and (b) a deep depth of 290 m (from Li, 2007)
IN-SITU PARAMETRIC STUDIES

Field seismic methods are also being developed to permit in-situ evaluation of various parameters that affect the dynamic response of soil during earthquakes. These parameters include: (1) state of stress, (2) strain amplitude in the nonlinear range, and (3) dynamic loading at strains creating pore pressure generation leading to liquefaction. The methods involve applying static and dynamic loads near the surface of the soil deposit and measuring the response of the soil mass beneath or around the loaded area using embedded instrumentation. In all cases, T-Rex (see Figure 1b) is an excellent source in applying both static and dynamic loads.

Two field approaches that are used to measure nonlinear shear moduli are shown in Figure 4. The first approach (Figure 4a) utilizes a surface footing that is dynamically loaded horizontally. This approach is also used to evaluate the effect on $V_s$ and $V_p$ of in situ changes in stress state as discussed below. The second approach (Figure 4b) utilizes a drilled shaft that is dynamically loaded vertically. The third parametric study, involving controlled loading at strain levels that create pore pressures, is simply an adaptation of the surface-footing arrangement in Figure 4a. In this case, the embedded instruments are placed at larger depths below the surface, with the locations being within the upper portion of the potentially liquefiable soil layer. Brief examples of each parametric study are presented below.

Example 1. Log $V_s$ – Log $\sigma_v$ Relationship

This example, shown in Figures 5a, presents one set of measurements in which the log $V_s – \log \sigma_o$ relationship was determined (Stokoe et al., 2005). A 1.2-m diameter concrete footing was used as the loading platen. The soil beneath the footing was a poorly graded sand (SP) that was lightly cemented, overconsolidated, and unsaturated. Staged loading was performed with increasing static vertical loads. At each static vertical load, small-strain, horizontal dynamic loads were generated which permitted vertically propagating and horizontally polarized shear waves ($S_{vh}$) to be measured. The stress state in the soil is expressed by the vertical and horizontal total stresses at each measurement depth, $\sigma_v$ and $\sigma_h$, respectively, as discussed by Stokoe and Santamarina (2000). Clearly, the variation in $V_s$ with stress state was measured. The log $V_s – \log \sigma_v \cdot \sigma_h$ relationship reveals several important points that are discussed in the presentation (and in Stokoe et al., 2005 and 2006).
Example 2. Nonlinear Shear Modulus Measurements Using a Surface Footing

The second example is presented in Figure 5b. In this example, a 0.9-m (3-ft) diameter surface footing was loaded with a constant vertical force. At this point, the footing was staged loaded with increasingly larger horizontal dynamic loads. Shear waves with increasing strains amplitude were measured from which the $G - \log \gamma$ and $G/G_{\text{max}} - \log \gamma$ relationships were evaluated. The soil was a poorly graded sand and silty sand (SP-SM) that was lightly cemented. Results from an intermediate dynamic loading stage are shown in Figure 5b. Measurements of $G$ in the linear range (hence, $G_{\text{max}}$) and in the nonlinear range were clearly conducted. The maximum shearing strain was only 0.02% since this was an intermediate stage. The field $G/G_{\text{max}} - \log \gamma$ relationship shows that the value of the elastic threshold strain ($\gamma_t$) in the field is around 0.002% at the imposed stress level. Comparison of the field relationship with laboratory results using an intact specimen is also shown in Figure 5b.

![Diagram](image)

Figure 5. In-situ evaluation of two parameters that affect the dynamic response of soil: (a) the variation of $V_{S\text{vh}}$ with increasing stress (from Stokoe et al., 2005) and (b) comparison of the $G/G_{\text{max}} - \log \gamma$ relationships measured in the field and laboratory (from Park, 2007)

Example 3. In-Situ Dynamic Liquefaction Test

An in-situ dynamic liquefaction test is under development (Rathje et al., 2005 and Cox, 2006). It is designed to measure pore water pressure generation under dynamic loading at field sites. T-Rex is used to provide the dynamic loading. The generalized test configuration is illustrated in Figure 6a. Shear waves dynamically load the test area. The level of shaking is controlled by specifying the number of cycles and their amplitudes. The shear waves induce cyclic shear strains which generate excess pore water pressure in the test area. One example of testing at the Wildlife Site in Imperial Valley, CA is shown in Figure 6b. This work is discussed in detail by Cox (2006).

![Diagram](image)

Figure 6. In-situ evaluation of liquefaction resistance: (a) test area that is stage loaded and (b) in-situ pore pressure generation curves obtained from one test (from Cox, 2006)
CONCLUSIONS

Field seismic testing to determine $V_s$ and $V_p$ profiles at depths less than 75 m is widely done in geotechnical earthquake engineering. This profiling is defined as shallow profiling herein, and many field methods are available. Deeper profiling is less often performed but is required in the earthquake design of some critical facilities. The downhole, suspension logging and surface-wave methods are applicable for profiling at the intermediate (75 to 225 m) and deep (greater than 225 m) depths associated with deeper profiling. In-situ testing to study parameters that are important in geotechnical earthquake engineering is also progressing. Parameters such as stress state, strain amplitude and liquefaction potential are being studied in situ with seismic tests.

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REFERENCES

USE OF INTERMEDIATE TO LARGE VIBRATORS AS SURFACE WAVE SOURCES TO EVALUATE $V_s$ PROFILES FOR EARTHQUAKE STUDIES

Kenneth H. Stokoe, II, University of Texas, Austin, TX  
Brady R. Cox, University of Texas, Austin, TX  
Yin-Cheng Lin, University of Texas, Austin, TX,  
Min Jae Jung, University of Texas, Austin, TX  
Farn-Yuh Meng, University of Texas, Austin, TX  
James A. Bay, Utah State University, Logan, UT  
Brent Rosenblad, University of Missouri, Columbia, MO  
Ivan Wong, URS Corporation, Oakland, CA

Abstract

The starting point when evaluating the response of soil and rock sites to earthquake shaking is normally a small-strain stiffness profile, typically expressed as a shear wave velocity profile. In situ shear wave velocity ($V_s$) measurements can be performed using a number of seismic methods, with noninvasive surface wave methods becoming widely used because of the cost and time savings associated with this type of testing. Both active and passive sources are presently used for $V_s$ profiling with surface waves. The writers have experience using active sources. In the early 1990’s, traditional vibroseises were used as surface wave sources. In the past five years, three new sources that have been developed for this purpose. The new sources are an intermediate-size vibrator called “Thumper” and two large vibrators called “Liquidator” and “T-Rex”. These three mobile vibrators are operated by the NSF-NEES equipment site at The University of Texas at Austin (nees@utexas). Experience and examples using a traditional vibroseis, Thumper, T-Rex and Liquidator are presented. Thumper is an excellent “urban-area” vibrator and surface wave profiling to 40+ m is routinely done. Liquidator is a unique vibrator which can operate at frequencies of 1 Hz and below. An example using Liquidator at the Yucca Mountain Site, where surface wave profiling to 400+ m has been accomplished, is discussed. T-Rex is a tri-axial vibrator that was developed for many tasks, including surface wave generation. The performance of T-Rex and a conventional vibroseis as surface wave sources are compared using studies performed in Imperial Valley, CA.

Introduction

Shear wave velocity ($V_s$) profiles of geotechnical sites are required input to the evaluation for earthquake loading of critical facilities founded on or embedded in the earth. The $V_s$ profiles are measured in the field using seismic methods. The depths of investigation are generally in the 30 to 100 m range. Field seismic measurements are performed in the small-strain or linear range where $V_s$ is independent of strain amplitude. Any nonlinearities in the geotechnical materials created during earthquake loading are evaluated in the laboratory using samples recovered from the site or are estimated from empirical relationships. No matter what approach is used to predict the nonlinear behavior, the field $V_s$ profile is required to link linear and nonlinear responses of the soil or rock.

Traditionally, invasive seismic testing involving the downhole or crosshole method has been used to measure $V_s$ profiles in earthquake studies. This approach has been employed since the 1960s. In the 1990s, noninvasive seismic testing with Rayleigh-type surface waves began to be employed for this purpose (Stokoe and Santamarina, 2000). Surface wave testing is more rapid and cost effective than
traditional invasive testing and therefore its use is steadily increasing. A key aspect in surface wave testing is measurement of surface waves over a wide range in frequencies. A wide frequency range is required so that a wide range in wavelengths is measured. As discussed below, a wide range in wavelengths translates to measurements over a significant depth range. Therefore, if $V_s$ profiling to depths of 30 to 100+ m is the goal, low frequencies in the range of 1 to 4 Hz are often required. For surface wave measurements with active sources (vertical vibrators that are used in the field), it can be difficult to generate these low frequencies with enough energy to travel the required distances along the ground surface for proper measurements to be performed. Therefore, a critical component of the field testing technique is the source and its ability to generate sufficient energy in the low-frequency range appropriate to the maximum depth of the investigation. Of course, it is also necessary to have the appropriate sensors for the frequencies being recorded and that these sensors be calibrated to account for their phase versus frequency characteristics.

As noted above, one critical component in surface wave testing using active sources is the capability of the source at low frequencies. The writers have used a variety of traditional and recently developed vibroseises as intermediate to large, low-frequency sources. These sources and their performance are discussed herein. The sources have been employed with one general methodology in surface wave testing, the spectral-analysis-of-surface-wave (SASW) method. Therefore, a brief overview of the SASW method is first presented to show the general approach and the need for low-frequency surface waves in deep profiling. Three new mobile vibrators operated for shared-use by the University of Texas are then discussed. The performances of the new vibrators are compared with each other and with a traditional vibroseis. Finally, examples of profiling with the vibrators are presented. In general, a traditional vibroseis can only be operated at frequencies of 3.5 to 4 Hz and above (to about 100 Hz). As such, profiling depths are limited to around 45 m at soft soil sites. On the other hand, the new, large vibrator specifically designed for low frequencies (named Liquidator) has been used to generate frequencies in the range of 1 to 2 Hz, which permits profiling to depths around 450 m at rock sites.

Overview of Surface Wave Testing Using the SASW Method

Surface waves propagating through a layered material are dispersive. Dispersion occurs because different wavelengths (hence different frequencies) sample different depths within the layered material. A key component in surface wave testing is the frequency range over which accurate measurements can be made. For earthquake studies, researchers would like to be able to measure the propagation of surface waves having frequencies that are as low as possible, thereby allowing one to profile as deep as possible. Both active and passive sources are presently used for deep profiling using surface wave methods. Passive-source techniques and results may be found in the literature (Tokimatsu, 1995; Louie, J.N., 2001; Kudo et al., 2002). One active-source method used presently is the spectral-analysis-of-surface-waves (SASW) method. The SASW method is briefly discussed below simply to show where the active source fits in these types of measurements and because this is the experience base of the writers. The sources presented herein can just as easily be used in other surface wave methods such as: (1) the multi-channel analysis of surface waves (MASW) method (Park et al., 1999; Miller et al., 1999), (2) the frequency-wave number ($f$-$k$) spectrum method (McMechan and Yeldin, 1981; Gabriels et al., 1987), and (3) the continuous surface wave (CSW) method (Matthews et al., 1996; Menzies and Matthews, 1996).

The SASW method is a stress-wave based method to nondestructively and nonintrusively determine the $V_s$ profile of a material (Stokoe et al., 1994). The method utilizes the dispersive nature of Rayleigh-type surface waves propagating through a layered material. In this context, dispersion arises when surface wave velocity varies with wavelength or frequency. Dispersion in surface wave velocity
occurs from changing stiffness properties of the soil and rock layers with depth. This phenomenon is illustrated in Figure 1 for a multi-layered solid. A high-frequency surface wave, which propagates with a short wavelength, only stresses material near the exposed surface and thus only samples the properties of the shallow, near-surface material (Figure 1b). A lower-frequency surface wave, which has a longer wavelength, stresses material to a greater depth and thus samples the properties of the shallower and deeper materials (Figure 1c). Spectral analysis is used to calculate the surface wave phase velocity at different frequencies (or wavelengths) to determine the experimental (“field”) dispersion curve for the site (Figure 1d). An analytical procedure is then used to generate a theoretical dispersion curve for a one-dimensional layered system of varying layer stiffnesses and thicknesses (Joh, 1996). The theoretical modeling procedure accounts for the contributions from all body wave and surface wave modes. An iterative procedure is employed to determine the one-dimensional $V_s$ profile that generates a theoretical dispersion curve which best matches the field dispersion curve. This best-match profile is presented as the $V_s$ profile of the site.

The SASW field procedure involves actively generating surface waves at one point on the exposed material surface and measuring the motions perpendicular to the surface created by the passage of the surface waves at two or more locations. All measurement points are arranged on the exposed surface along a single radial path from the source. The distance between the source and the first receiver is typically kept equal to the distance between receivers. Data are collected at shorter wavelengths (sampling material near the surface) by using small receiver spacings and a source capable of generating high frequencies. Longer wavelength data (sampling deeper material) are collected by using successively larger receiver spacings and correspondingly larger sources which generate lower and lower frequencies. Measurements are performed with several (typically seven or more) sets of source-receiver spacings, and the totality of seven or more sets of source-receiver spacings is called an SASW array.

**Figure 1:** Illustration of surface waves with different wavelengths sampling different materials in a layered system, which results in dispersion in Rayleigh-wave velocity (from Stokoe et al., 2004a).
NEES@UTexas Mobile Vibrators

The U.S. National Science Foundation (NSF) has developed the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES). As part of the NEES program, a network of fifteen advanced testing facilities, called equipment sites, have been developed. These equipment sites, which are distributed across the United States, are for shared use by researchers and practitioners. The site specializing in seismic testing with mobile, field equipment is located at the University of Texas at Austin, and is referred to as nees@UTexas. The goal of nees@UTexas was to develop three mobile vibrators with diverse force and frequency capabilities to dynamically load geotechnical and structural systems in the field. The nees@UTexas vibrators are discussed below.

As discussed in Stokoe et al., 2004b, the three mobile vibrators of nees@UTexas are called: (1) “T-Rex”, (2) “Liquidator”, and (3) “Thumper”. Each mobile vibrator was designed and built by Industrial Vehicles International, Inc. (IVI), Tulsa, Oklahoma. T-Rex was introduced by IVI in 1999 and is capable of generating large dynamic forces in any of three directions (X, Y, or Z directions) at the touch of a button. A photograph of T-Rex is shown in Figure 2a. The shaking system is placed on an off-road vehicle so that it can be operated in difficult geologic environments. Some important characteristics of T-Rex are: buggy-mounted off-road vibrator; total weight of 29,030 kg; three vibration orientations (vertical, horizontal in-line, and horizontal cross-line); and push-button transformation of shaking orientation. Additional characteristics of T-Rex are given in Table 1. These characteristics make T-Rex an excellent vibrational source for subsurface seismic exploration and earthquake motion simulation. The theoretical performance of T-Rex (the actual force output of the vibrator is site dependent) in both the vertical and horizontal modes is shown in Figure 3a. As shown in the figure, the force output in the vertical mode is about 267 kN and decreases with frequency below 12 Hz. In the horizontal mode, the maximum force output is about 133 kN, one-half of the maximum force output in the vertical mode. This force output does not begin to decrease with frequency until about 5 Hz. Several modifications to T-Rex have been made as part of the NEES project to improve its performance and capabilities for earthquake studies. The two most important modifications that have been made are: (1) to the electronic controller so that external drive functions can be used to drive the vibrator with sinusoidal, random or earthquake motions, and (2) to the static hold-down system of the vibrator so that variable vertical stresses can be applied to the ground surface during staged testing.

Liquidator is the other large mobile vibrator. Liquidator is designed to be a lower frequency vibrator than T-Rex and is a one-of-a-kind vibrator. A photograph of Liquidator is shown in Figure 2b. As seen in the photograph, Liquidator has the same off-road buggy design as T-Rex, but the shaking system is specially designed to give a higher force output in the low-frequency range, defined herein as 0.5 to 4.0 Hz. Some important characteristics of Liquidator are: buggy-mounted off-road vibrator; total weight of 27,200 kg; two vibration orientations (shop transformable in a day): vertical or horizontal transverse; 6100 kg vibrational mass; and peak-to-peak mass movement of 40 cm. Additional characteristics of Liquidator are given in Table 1. These characteristics make Liquidator an excellent low-frequency vibrational source for deep surface wave testing and earthquake motion simulation. The large peak-to-peak movement of the mass is required to create high force levels at low frequencies and demands a one-of-a-kind isolation system. The theoretical performance of Liquidator (actual force output is site dependent) in both the vertical and horizontal modes is shown in Figure 3b. As shown in the figure, the force output in both modes is about 89 kN and decreases with frequency below 1.3 Hz. Because the force from Liquidator does not start to fall off until 1.3 Hz, it can supply significantly larger forces than T-Rex in the frequency range of 0.5 to 4 Hz.

Thumper is designed to be a moderate-to high-frequency vibrator used in seismic reflection and surface wave projects. A photograph of Thumper is shown in Figure 2c. As can be seen in the photograph, Thumper is housed on a much smaller vehicle, which aids in its transportation to and from
a. High-force, three-axis vibrator called T-Rex

b. Low-frequency, two-axis vibrator called Liquidator

c. High-frequency, three-axis vibrator called Thumper

Figure 2: Photographs of the three nees@UTexas mobile vibrators (from Stokoe et al., 2004a).
Table 1: Characteristics of the three nees@UTexas mobile vibrators (from Stokoe et al., 2004a)

<table>
<thead>
<tr>
<th>Vibrator</th>
<th>T-Rex</th>
<th>Liquidator</th>
<th>Thumper</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vehicle Type</td>
<td>Buggy-mounted vibrator, articulated body</td>
<td>Buggy-mounted vibrator, articulated body</td>
<td>Built on Ford F650 Truck</td>
</tr>
<tr>
<td>Driving Speed</td>
<td>Hydraulic drive system (&lt;15 mph)</td>
<td>Hydraulic drive system (&lt;15 mph)</td>
<td>Highway Speeds</td>
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<tr>
<td>Total Weight</td>
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<td>29,030 kg (64,000 lb)</td>
<td>9980 kg (22,600 lb)</td>
</tr>
<tr>
<td>Length</td>
<td>9.8 m (32 ft)</td>
<td>9.8 m (32 ft)</td>
<td>7.1 m (23 ft)</td>
</tr>
<tr>
<td>Width</td>
<td>2.4 m (8 ft)</td>
<td>2.4 m (8 ft)</td>
<td>2.4 m (8 ft)</td>
</tr>
<tr>
<td>Height</td>
<td>3.2 m (10.5 ft)</td>
<td>3.2 m (10.5 ft)</td>
<td>2.4 m (8 ft)</td>
</tr>
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<td>207 bar (3,000 psi)</td>
<td>207 bar (3,000 psi)</td>
<td>476 bar (4000 psi)</td>
</tr>
<tr>
<td>Vibrator Pump Flow</td>
<td>757 l/m (200 gpm)</td>
<td>530 l/m (140 gpm)</td>
<td>151 l/m (40 gpm)</td>
</tr>
<tr>
<td>Vibration Orientations</td>
<td>(1) Vertical, (2) Horizontal in-line, and (3) Horizontal cross-line</td>
<td>(1) Vertical, and (2) Horizontal cross-line</td>
<td>(1) Vertical, (2) Horizontal in-line, and (3) Horizontal cross-line</td>
</tr>
<tr>
<td>Shaking Orientation Transformation</td>
<td>Push-button transformation of shaking orientation</td>
<td>Shop transformable in one day</td>
<td>Field transformable in about hour hours</td>
</tr>
<tr>
<td>Maximum Output Force: (1) Vertical, and (2) Shear</td>
<td>(1) 267 kN (60,000 lb) (2) 134 kN (30,000 lb)</td>
<td>(1) 89 kN (20,000 lb) (2) 89 kN (20,000 lb)</td>
<td>(1) 26.7 kN (6000 lb) (2) 26.7 kN (6000 lb)</td>
</tr>
<tr>
<td>Base Plate Area</td>
<td>4.11 m² (44.2 ft²)</td>
<td>4.34 m² (46.7 ft²)</td>
<td>0.698 m² (7.50 ft²)</td>
</tr>
<tr>
<td>Moving Mass: (1) Vertical, and (2) Shear</td>
<td>(1) 3,670 kg (8,100 lb) (2) 2,200 kg (4,850 lb)</td>
<td>(1) 13,475 lb (6,110 kg) (2) 13,475 lb (6,110 kg)</td>
<td>(1) 311 lb (140 kg) (2) 311 lb (140 kg)</td>
</tr>
<tr>
<td>Stroke (Peak to Peak): (1) Vertical, and (2) Shear</td>
<td>(1) 8.9 cm (3.5 in.) (2) 17.8 cm (7.0 in.)</td>
<td>(1) 40.6 cm (16.0 in.) (2) 40.6 cm (16.0 in.)</td>
<td>(1) 7.6 cm (3.0 in.) (2) 7.6 cm (3.0 in.)</td>
</tr>
<tr>
<td>Hydraulic Oil</td>
<td>Vegetable-based hydraulic oil</td>
<td>Vegetable-based hydraulic oil</td>
<td>Vegetable-based hydraulic oil</td>
</tr>
<tr>
<td>Special Features</td>
<td>(1) Cone pushing capacity (2) Hydraulic pressure take-off (3) Variable vertical hold-down force (4) Must be transported by tractor-trailer rig</td>
<td>(1) Optimized for low freq. (down to 0.5 Hz) (2) Cone pushing capacity (3) Hydraulic pressure take-off (4) Must be transported by tractor-trailer rig</td>
<td>(1) Built for high-frequency output (above 200 Hz) (2) Built for use in urban environments (3) Can be driven on highways</td>
</tr>
</tbody>
</table>
Figure 3: Theoretical force outputs of the three nees@UTexas mobile vibrators (from Stokoe et al., 2004a).
sites and also allows it to be used in urban environments. Some important characteristics of Thumper are: mounted on a Ford F650 truck; total weight of about 9,900 kg; and, three vibration orientations (field transformable in a few hours): vertical or horizontal transverse. These characteristics make Thumper an excellent vibrational source for shallow (depths less than 100 m) seismic reflection profiling and surface wave testing. The theoretical performance of Thumper (actual force output is site dependent) is shown in Figure 3c. As shown in the figure, the maximum force output is about 27 kN over the frequency range of 17 to 225 Hz. The force output decreases outside of this frequency band. The relatively low-force output (27 kN) makes Thumper an excellent vibrator for testing in urban environments where disturbance or possibly damage to existing above-ground and below-ground facilities might occur.

T-Rex and Liquidator must be transported to and from field sites on a tractor-trailer rig. However, it is important to note that the combined weight of one of the large vibrators and the tractor-trailer rig is between 45,360 and 47,627 kg, thus necessitating the use of overweight permits to transport them on highways. Additionally, two special features have been added to T-Rex and Liquidator to increase their usefulness. The first feature is a cone or sensor pushing capability that has been added on the back bumper. Pushing (or pulling) is done with a hydraulic cylinder controlled by a variable flow value. The second special feature is a hydraulic take-off so that either large vibrator can be used to power other hydraulic equipment in the field.

Comparison of Force Levels Generated by Different Vibrators

One of the key aspects in using active sources is their performance at low frequencies (below about 4 Hz). It is therefore interesting to compare the theoretical peak force levels of the three nees@UTexas vibrators and a traditional vibroseis. This comparison is shown in Figure 4 and is presented numerically at selected frequencies in Table 2. The frequency range in Figure 4 is extended to 20 Hz so that the frequency at which each vibrator reaches the theoretical maximum force level can be seen. Also, the traditional vibroseis has been assumed to have the following characteristics: (1) hydraulic system pressure of 207 bar (3,000 psi), (2) vibrator pump flow of 568 l/m (150 gpm), (3) peak stroke of 4.45 cm (1.75 in.), (4) reaction mass equal to 1,588 kg (3,500 lb), and (5) theoretical peak force of 15,717 kg (34,650 lb).

As noted above, Liquidator was designed to generate significant force levels at frequencies in the range of 0.5 to 4 Hz. The theoretical force output of Liquidator in this frequency range in comparison with other vibrators is shown in Figure 4. For example, at 1.3 Hz and below, the output of Liquidator is more than seven times greater than that of T-Rex and approximately 17 times greater than that of a traditional vibroseis. The high output of Liquidator at low frequencies enables surface wave profiling with this source to substantially greater depths than with other vibrators. The outputs of T-Rex and a traditional vibroseis surpass that of Liquidator at frequencies greater than approximately 4 Hz and 7 Hz, respectively. It should be noted that it is the writers’ experience that any large vibroseis works quite well for surface wave testing using frequencies greater than 7 Hz.

The output of Thumper is substantially lower than all other vibrators as shown in Figure 4. However, Thumper has the advantages of ease of use in urban environments and no need for an additional, large transporting vehicle. Thumper has also been proven to be adequate and quick in obtaining $V_{s30}$ over a wide range in soil/rock profiles. Several field case histories are given below to illustrate the effectiveness of using active sources, such as Thumper, Liquidator, T-Rex and traditional vibroseises.
Figure 4: Comparison of the theoretical peak force levels in vertical shaking of four different intermediate to large vibrators in the frequency range of 0.2 to 20 Hz.

**Field Case Histories**

*V<sub>s</sub> Profiling at Strong Motion Stations in the Puget Sound Area, WA, with Thumper*

The 28 February 2001 M 6.8 Nisqually earthquake was recorded by more than 70 strong motion sites within the Pacific Northwest Seismic Network (PNSN) and the National Strong Motion Network (NSMN). To evaluate the shallow site response effects on the recorded Nisqually ground motions, it was desired to develop shear wave velocity (V<sub>s</sub>) profiles at 30 of these stations. The near-surface geology of the Puget Sound region is dominated by a complex interbedded and discontinuous sequence of glacial and nonglacial deposits. Most of the surveyed strong motion sites are underlain by glacial till with the remaining sites on Holocene alluvium, glacial recessional and advance outwash deposits, or manmade fill/modified land. Ground motions in the 2001 earthquake recorded at the 30 sites, as characterized by peak horizontal ground acceleration (pga), ranged from 0.04 to 0.31 g. In general, ground motions were found to be highest at sites underlain by lower velocity materials (e.g., alluvium, outwash) relative to stiffer sites such as the glacial till (Wong et al., 2004).

The writers successfully used Thumper as an active source to generate V<sub>s</sub> profiles to depths of 30 to 100 m at 30 PNSN and NSMN sites around Puget Sound using the SASW method. Most of the sites were located in urban environments, thus necessitating testing on sidewalks, roads, parking lots and alleyways. The maneuverability of Thumper made it possible to test in these urban areas and enabled rapid movement between test locations. Typically, at least three sites were tested each day, depending on the distance between sites. A picture of Thumper being demonstrated for children at an elementary school near Seattle is shown in Figure 5 (an additional rewarding activity with this equipment). Typical dispersion curves and V<sub>s</sub> profiles from two separate test locations are shown in Figures 6 and 7. Figure 6a shows the experimental and theoretical dispersion curves for the Maple Valley Site. It can be seen that wavelengths of 150 m were measured, allowing determination of the V<sub>s</sub> profile for the site to a...
Table 2: Force output ratio between the four, different intermediate-to-large vibrators at low frequencies.

<table>
<thead>
<tr>
<th>Y</th>
<th>X</th>
<th>T-Rex</th>
<th>Liquidator</th>
<th>Traditional Vibroseis</th>
<th>Thumper</th>
</tr>
</thead>
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<tr>
<td>T-Rex</td>
<td>1*</td>
<td>7.43</td>
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</tr>
<tr>
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For Frequency = 2.0 Hz

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<th>Liquidator</th>
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<th>Thumper</th>
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<tr>
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For Frequency = 3.0 Hz

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<th>Liquidator</th>
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<th>Thumper</th>
</tr>
</thead>
<tbody>
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<td>0.03</td>
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<tr>
<td>Liquidator</td>
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<td>0.28</td>
<td>0.02</td>
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<tr>
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<td>0.08</td>
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</tr>
<tr>
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<td>47</td>
<td>13</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

* Ratio = X/Y

depth of approximately 75 m (Figure 6b). Figure 7a shows the experimental and theoretical dispersion curves for the SEATAC Airport Site. In this case, a maximum wavelength ($\lambda_{\text{max}}$) of almost 200 m was measured, allowing determination of the $V_s$ profile for the site to a depth of approximately 95 m (Figure 7b). It should be noted that the $V_s$ profile for a site is typically developed only to a depth equal to the maximum measured wavelength divided by two ($\lambda_{\text{max}}/2$) because of the lack of layer resolution at depths greater than this depth.

$V_s$ Profiling at Yucca Mountain, NV, with Liquidator

Yucca Mountain, Nevada, was approved as the site for development of the geologic repository for high-level radioactive waste and spent nuclear fuel in the United States. The U.S. Department of Energy has been conducting studies to characterize the site and assess its future performance as a
geologic repository. As part of these studies, a program of deep seismic profiling was conducted around Yucca Mountain to evaluate the $V_s$ structure of the repository block and the adjacent areas where support facilities may be located. The resulting $V_s$ data were used as input into the development of ground motions for the preclosure seismic design of the repository and for postclosure performance assessment. The noninvasive SASW method was employed in the deep profiling. Deep profiling is defined in this work as profiling to depths of 300 to 450 m.

Yucca Mountain is located in southern Nevada, within the Great Basin which is part of the Basin and Range structural/physiographic province. The area consists of stacked layers of tuffs. The tuffs are approximately 7.5 to 15 million years old, and were formed by eruptions of volcanic ash from the north. Most of the rocks are welded and nonwelded ash flow tuffs (BSC, 2003). As the ash settled, it was subjected to various degrees of compaction and fusion, depending on the temperature and pressure. When the temperature was high enough, ash was compressed and fused to produce a welded tuff, a hard, dense, brick-like rock with very little open pore space in the rock matrix. Nonwelded tuffs occur between the layers of welded tuff. These tuffs are compacted and consolidated at lower temperatures, are less dense and brittle, and have a higher porosity.

The writers successfully used Liquidator as an active source to generate $V_s$ profiles to depths of 300 to 450 m at Yucca Mountain. Typically, it was only possible to test one site per day due to the time required to deploy receivers over great distances in rough terrain and the extended duration of the vibrator sweeps at low frequencies. A picture of Liquidator at Yucca Mountain is shown in Figure 8. Typical dispersion curves and $V_s$ profiles from two separate test locations are shown in Figures 9 and 10. Figure 9a shows the experimental and theoretical dispersion curves for a site at the top of Yucca Mountain. It can be seen that wavelengths up to 900 m were measured, allowing determination of the $V_s$ profile for the site to a depth of approximately 450 m (Figure 9b). Figure 10a shows the experimental and theoretical dispersion curves for a site near the possible location of the North Portal Facility. In this case, wavelengths slightly less than 900 m were measured, allowing determination of the $V_s$ profile for the site to a depth of approximately 430 m (Figure 10b).
**Figure 6:** Acquired and developed SASW measurements at the Maple Valley site, WA, using Thumper as the surface wave source.

(a) Comparison of the experimental and theoretical dispersion curves.

(b) Shear wave velocity profile determined from forward modeling.

**Figure 7:** Acquired and developed SASW measurements at the SEATAC airport site, WA, using Thumper as the surface wave source.

(a) Comparison of the experimental and theoretical dispersion curves.

(b) Shear wave velocity profile determined from forward modeling.
Figure 8: Liquidator in operation on top of Yucca Mountain, NV.

(a) Comparison of the experimental and theoretical dispersion curves.

(b) Shear wave velocity profile determined from forward modeling.

Figure 9: Acquired and developed SASW measurements at the top of Yucca Mountain, NV, using Liquidator as the surface wave source.
Figure 10: Acquired and developed SASW measurements at the North Portal Facility at Yucca Mountain, NV, using Liquidator as the surface wave source.

Comparison of $V_s$ Profiling in Imperial Valley, CA, with a Traditional Vibroseis and T-Rex

Profiling with a Traditional Vibroseis

The writers have been involved in several projects in which $V_s$ profiling was performed in Imperial Valley, CA. The earliest project involved a number of the strong motion recording stations that exist in Imperial Valley. Many of these stations have been placed by the U.S. Geological Survey (USGS), beginning in the early 1970s. Various experimental programs have been undertaken to accumulate geotechnical and seismic data in the valley area for use in developing ground response models for strong earthquake shaking and empirical correlations (Porcella, 1984). As part of this continuing effort, the Pacific Earthquake Engineering Research (PEER) center supported SASW profiling at more than 30 sites around Imperial Valley. This work was conducted in the late 1990s using a traditional vibroseis as the source.

Imperial Valley is broad deep trough that is “filled with Cenozoic deposits consisting of deltaic sediments of the nearby Colorado River and fan and alluvial materials transported from the surrounding mountain ranges” (Porcella, 1984). In the top 30 to 60 m, these materials are quite soft as determined by the $V_s$ profiles, with shear wave velocities often ranging between 150 and 300 m/s. Unfortunately, the softness of the materials makes deep profiling (depths of 100 m or more) with a traditional vibroseis impossible. The reason is that a traditional vibroseis has a lower-bound operating-frequency on the order of 3.5 to 4 Hz. As such, the longest wavelength that can be generated is on the order of 75 to 80 m, thus generally resulting in a $V_s$ profile down to a maximum depth of about 40 m.

A traditional vibroseis operating in Imperial Valley is shown in Figure 11. One set of field measurements using this machine is presented in Figure 12a. As seen can be seen, the longest
wavelength that could be generated was approximately 70 m. The resulting $V_s$ profile is shown in Figure 12b and extends to a depth of about 35 m. At this particular site, USGS personnel had performed downhole seismic tests more than a decade earlier. Good agreement was found between $V_s$ profiles determined by the two test methods as shown in Figure 12b.

**Profiling with T-Rex**

In the summer of 2005, T-Rex was used as a dynamic source in an in-situ liquefaction study that was performed in Imperial Valley. The study was supported by the NEHRP program of USGS and was conducted at the Wildlife Liquefaction Array (WLA) site that is operated by UC Santa Barbara under the NSF-NEES program [http://nees.ucsb.edu/facilities/wla/](http://nees.ucsb.edu/facilities/wla/). As part of this study, SASW testing was performed at the WLA using T-Rex as the surface wave source. The resulting dispersion curve and $V_s$ profile are shown in Figures 13a and 13b, respectively. T-Rex was able to generate useable surface wave energy in the range of 1.5 to 2 Hz. In the soft soils of Imperial Valley, this frequency range allowed measurements of a surface wave with a maximum wavelength of approximately 200 m. As a result, a $V_s$ profile to a depth of nearly 100 m was determined. At this site, independent $V_s$ measurements were performed with a suspension logger several years earlier. Good agreement was found between the $V_s$ profiles determined by both test methods as shown in Figure 13b.

As noted above, a traditional vibroseis can not work at the low frequencies required to profile to depths of about 100 m and more in soft soils. This result is shown by comparing the dispersion curves in Figures 12a and 13a for a traditional vibroseis and T-Rex, respectively. Both sources were used in the soft soils of Imperial Valley. The maximum profiling depth of about 35 m for a traditional vibroseis resulted from a low-end operating frequency of about 4 Hz. The maximum profiling depth of about 100 m with T-Rex resulted from a low-end operating frequency of about 1.5 Hz. Further, if Liquidator were used at these sites and if a low-end operating frequency of 0.75 Hz were generated, then the maximum profiling depth would be estimated as 200 m (at least with the present way of recording and processing the surface wave data).
(a) Comparison of the experimental and theoretical dispersion curves.

**Figure 12:** Acquired and developed SASW measurements at the Holtville Post Office site, CA, using traditional vibroseis as the surface wave source.

(b) Shear wave velocity profile determined from forward modeling.

(a) Comparison of the experimental and theoretical dispersion curves.

**Figure 13:** Acquired and developed SASW measurements at the Wildlife Liquefaction Array (WLA) site, CA, using T-Rex as the surface wave source.
Conclusions

Surface wave testing is a noninvasive seismic testing approach that is used to evaluate \( V_s \) profiles in the field. This type of seismic testing is being used in more and more applications. In the geotechnical earthquake engineering area, surface wave testing with active sources is used to evaluate moderate-to-deep \( V_s \) profiles. In this application, the source is critical to accomplishing the profiling depth, with frequencies in the range of 1 to 3 Hz needed to profile to depths of 100+ m at soft soil sites and 400+ m at rock sites. Shear wave velocity profiles evaluated with three new vibrators, Thumper, T-Rex and Liquidator, and a traditional vibroseis are illustrated. Profiling to depths of 30 to 100 m was accomplished with Thumper in urban settings, while profiling with Liquidator at rock sites in the Nevada desert was accomplished to depths of 300 to 450 m. Profiling with a traditional vibroseis at soft soil sites could only be done to a depth of about 35 m because of its lowest operating frequency is about 4 Hz. However, in the same soft soil setting, \( V_s \) profiling to a depth of about 100 m was accomplished with T-Rex because its lowest operating frequency is about 1.5 Hz.

References

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Variability in $V_S$ Profiles and Consistency between Seismic Profiling Methods: A Case Study in Imperial Valley, California

Y.-C. Lin & K.H. Stokoe, II  
*University of Texas, Austin, Texas, USA*

B.L. Rosenblad  
*University of Missouri, Columbia, Missouri, USA*

ABSTRACT: The variability in $V_S$ profiles at large construction sites and locations of critical facilities is often important. The Spectral-Analysis-of-Surface-Wave (SASW) method is a robust way of determining $V_S$ profiles of the subsurface. The profiling depth can be in excess of 400 m. In this paper, the SASW method was used to determine in-situ $V_S$ profiles at 31 sites in Imperial Valley, California. Variability in $V_S$ profiles over this area was studied by investigating the coefficient of variation (COV). The $V_S$ profiles were studied to determine ranges in COV values that represent uniform and non-uniform sites and the boundary between these sites. In addition, the $V_S$ profiles determined with the downhole and P-S suspension logging methods performed at common sites are compared with the SASW measurements based on the median $V_S$ profiles to investigate the consistency between the seismic profiling methods.

1 INTRODUCTION

1.1 Background of test area

Imperial Valley, California is a region about 100 km by 80 km in southeastern California between the Salton Sea and the border of Mexico (Figures 1 and 2). Since 1872, several big earthquakes have occurred in this region, with most of them having a magnitude over 6. These earthquakes resulted in severe damage to structures and induced liquefaction in the area. To avoid serious property damage and loss of life in the future, several studies were conducted in this region based on the $V_S$ profiles obtained from different seismic profiling techniques.

1.2 Information of studied SASW $V_S$ profiles

In this paper, 31 SASW $V_S$ profiles obtained in Imperial Valley, CA were studied. These SASW measurements were performed by the personnel from the University of Texas at Austin in 2002 and 2005. 30 out of the 31 profiles were sponsored by the Pacific Earthquake Engineering Research Center (PEER) Lifelines Program. The goal of this project was to apply the SASW method to profile the subsurface of selected strong-motion recording (SMR) sites in Imperial Valley, CA and use the top 30 m of the $V_S$ profiles ($V_{S30}$) obtained from the SASW tests to classify these sites based on the International Building Code provisions. Only one among the 31 SASW profile was sponsored by the Earthquake Engineering Simulation (NEES) project. Maps of the approximate locations of test sites are shown in Figures 1 and 2.

The list of the 31 SASW test sites and other relative information are shown in Table 1. As seen, only four sites are classified as class “E” and the remaining 27 sites are all “D” sites. In addition, there are 23 downhole and three suspension logging $V_S$ pro-
files available in the area where the 31 SASW tests were performed. The 23 downhole profiles were mainly carried out by USGS (Porcella 1984) and the three suspension P-S logging tests were performed by GEOVsion and Kjima.

![SMR Sites Where SASW Tests Were Conducted by the University of Texas at Austin](image)

Figure 2. Names of 28 SASW test sites in Imperial Valley, CA.

Table 1. Information on the SASW test sites in and around Imperial Valley, CA.

<table>
<thead>
<tr>
<th>No.</th>
<th>Site Name</th>
<th>Profile Depth (ft)</th>
<th>V_s (fps)</th>
<th>Site Class</th>
<th>Other Tests</th>
<th>Downhole</th>
<th>P-S Log</th>
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<td>1144</td>
<td>D</td>
<td>2</td>
<td></td>
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<tr>
<td>25</td>
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<td>242</td>
<td>1064</td>
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<tr>
<td>26</td>
<td>Salton Sea State Park</td>
<td>186</td>
<td>870</td>
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<tr>
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<td>Salton Sea Wild Life Refuge</td>
<td>102</td>
<td>627</td>
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<tr>
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<td>152</td>
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<td>262</td>
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<td>D*</td>
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<td>562</td>
<td>E</td>
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</table>

*Based on extrapolated data

2 REVIEW OF THE SASW METHOD

The majority of the V_s profiling in this study was performed using the SASW method. Therefore, this method is briefly described below.

2.1 Theoretical background and test procedures

The SASW method is a nondestructive and noninvasive method that uses surface waves to determine the V_s profile of a material (Stokoe et al. 1994). The method utilizes the dispersive nature of Rayleigh-type surface waves propagating through a layered material. Dispersion in surface wave velocity results from changing stiffness properties of the soil/rock layers with depth. This phenomenon is illustrated in Figure 3 with a multi-layered system. A shorter-wavelength (high-frequency) surface wave only propagates through material which is near the exposed surface and thus only samples the properties of the shallow, near-surface material (Figure 3b). A longer-wavelength (lower-frequency) surface wave propagates through material to a greater depth and thus samples the properties of both of the shallower and deeper materials (Figure 3c). Spectral analysis is used to calculate the surface wave phase velocity at different frequencies (or wavelengths) to determine the field/experimental dispersion curve for the site (Figure 3d).

Once the field dispersion curve has been determined, an analytical modeling procedure (Joh 1996) is used to generate theoretical dispersion curves for one-dimensional layered systems of varying layer stiffnesses and thicknesses. The theoretical modeling procedure takes the contributions from all body wave and surface wave modes into account. The analytical procedure is a forward-modeling iterative procedure that is employed to determine the one-dimensional Vs profile that generates a theoretical dispersion curve which best matches the field dispersion curve. This best-match profile represents the V_s profile of the site.

![Figure 3. Illustration of surface waves with different wavelengths sampling different materials in a layered system that results in dispersion in Rayleigh-wave velocity (Stokoe et al., 2004).](image)
The keys of the SASW method are to generate, measure and analyze surface waves at the site of interest. First, active sources, such as hammers and vertical shakers, are employed to generate surface waves at various points on the exposed material surface. Second, vertical transducers (accelerometers or geophones) are used in pairs to measure the motions perpendicular to the surface created by the passage of the surface waves. All measurement points are arranged on the exposed surface along a single radial path from the source. The distance between the source and the first receiver is typically kept equal to the distance between receivers in each pair. With small receiver spacings and a source capable of generating high frequencies, the data of shorter wavelengths, which sample material near the surface, are collected. With longer receiver spacings and a source capable of generating low frequencies, the data of longer wavelengths, which sample deeper material, are collected. Typically, seven or more measurements with different source-receiver spacings are performed at a site, and the combination of these different source-receiver spacings is called an SASW array. After testing is completed in the field, a post-field analysis is performed to reduce the data collected in the field. The analysis software is called WinSASW which was developed by Joh (1996). This program performs an iterative, forward modeling procedure to determine the VS profiles of the site.

3 STATISTICAL ANALYSIS OF VS PROFILES

The statistical study of the VS profiles measured in and around Imperial Valley, CA is based on measurements performed with three seismic methods: SASW, downhole and P-S suspension logging. The numbers of VS profiles with each method are: 31 SASW, 23 downhole and three P-S logger. The statistical data were determined for each profiling method in terms of: (1) the median, 16th and 84th percentile profiles of VS, (2) the coefficient of variation (COV) which is equal to one standard deviation divided by the mean of the VS profiles at each depth, and (3) the number of profiles (N). An example set of data for the SASW method is shown in Figure 4. The data were analyzed assuming a log-normal distribution and the profiles were divided into 0.3-m increments. The statistics were calculated only when the number of profiles was equal to or greater than three (N ≥ 3).

Analyses of these data are divided into two parts. The first part deals with variability in the VS profiles which is expressed by the COV values. In this part, sufficient data were available only in the SASW and downhole profiles. In the second part which deals with consistency between the seismic profiling methods, the median VS profiles were compared. All three seismic methods were compared, although only three P-S logging profiles were available.

4 VARIABILITY IN VS PROFILES

4.1 Variability based on profiling the complete area using the SASW method

All 31 SASW VS profiles in and around Imperial Valley are included in this analysis. This area is shown in Figure 1 and is the largest area. The statistics of the 31 VS profiles are presented in Figure 4. As seen, the COV values are largest in the top 5 m, with values between about 0.3 to 0.4. Below 5 m, the COV ranges between about 0.2 to 0.3, with an average of 0.25.

4.2 Variability after excluding the SASW profiles from the western mountain area

Among the 31 SASW VS profiles, most sites are classified as “D” sites, with the VS,30 in the lower range of the D sites as shown in Table 1. There are, however, three sites that classify as D sites but have VS,30 in the higher range. These sites are Parachute Test Site, Superstition Mountain Base and Salton City and they are all in the mountain area west of Imperial Valley. Inclusion of the VS profiles from these sites results in a higher median profile and higher COVs in the statistical analysis of the 31 sites. To investigate how COV values change if only more similar sites are included, statistical analysis of only the 28 sites not in the mountain area was performed. These results are shown in Figure 5. In this case, the profiling area decreased only slightly. As expected, the median VS profiles and COV values in Figure 5 are smaller than in Figure 4. The only exception is the COV within about 1 m of the surface which has increased slightly. This comparison shows that it is feasible to use the COV value as an index to estimate the uniformity of the soil/rock formations in an area of interest. The COV values at
most depths below 6 m are less than 0.20, with an average COV value of about 0.17. As long as there is no significant jump in $V_S$ at some depths in the profiles, the COVs are larger near the surface where moisture changes and different materials (sands versus clays) increase the variability.

Figure 5. Statistical analysis of 28 SASW $V_S$ profiles in and around Imperial Valley (3 sites in the western mountain area were excluded).

4.3 Variability based on SASW profiling only in the central valley area

If the sites in the transition zone around the valley edge are excluded from the 28 sites used to generate Figure 5, lower COV values should be expected because the Cenozoic deposits (deltaic sediments and alluvial materials) (Porcella 1984) in the central valley area is quite uniformly soft. Based on Figures 1 and 2, and the $V_{S,30}$ information in Table 1, there are five possible sites in the transition zone around the valley edge. These sites are Sites El Centro Array #1, El Centro Array #13, Bombay Beach, Bond’s Corner and Salton Sea State Park. Statistical analysis of the remaining 23 sites in the central valley area is presented in Figure 6. As expected, the COV profile in Figure 6 has smaller values than Figures 4 and 5. Below 1.5 m, the average COV value of these 23 $V_S$ profiles is about 0.14. As expected, this average COV is the lowest of the three study areas and is presented here as representative of a relatively large area (50 km by 30 km) which is relatively soft and uniform.

Figure 6. Statistical analysis of 23 SASW $V_S$ profiles in Imperial Valley (8 sites in the western mountain area and valley edge were excluded).

4.4 Studies based on $V_S$ profiling in the central valley area with the downhole method

As seen in Table 1, 19 downhole $V_S$ profiles are available in the central valley area. To avoid placing too much weight on the same site, at sites with two downhole $V_S$ profiles, the profiles were averaged before the statistical analysis was performed. The COV values in Figure 7 are close to 0.1 at most depths. Between about 3 m and 25 m, the COV values in the SASW and downhole profiles are very similar (Figures 6 and 7). Within 3 m of the surface, the median $V_S$ profile is lower and COV values are higher in the SASW measurements compared with the downhole measurements due to the higher resolution in the SASW method at shallow depths (Lin 2007). Below 25 m, the COV values are higher in the SASW measurements, possibly due to fewer measurements in this depth range and the decreasing resolution with depth in the SASW method.

Figure 7. Statistical analysis of 19 downhole $V_S$ profiles in Imperial Valley (4 profiles in mountain area and valley edge were excluded).

5 COMPARISONS BETWEEN DIFFERENT SEISMIC METHODS

5.1 Criteria for the comparison

When comparisons are made between $V_S$ profiles determined by different seismic methods, the conclusions(s) can be strongly influenced by the criteria used to make the comparisons. This point is illustrated below by investigating the consistency between the $V_S$ profiles determined by the SASW, downhole and P-S suspension logging methods. Three different criteria were used to compare the
median $V_S$ profiles determined by the seismic methods to evaluate which criterion is most objective.

Three criteria are utilized to perform these comparisons (Lin 2007). First, all available $V_S$ profiles obtained in the same region from the different methods are compared. This comparison is called a general comparison or an “apples-to-oranges” comparison. Second, comparisons are made with profiles from the common sites where different seismic methods were used. This comparison is called a common site comparison or a “red-apples-to-green-apples” comparison. The last comparison is called an identical-site-and-depth comparison (or a “red-apples-to-red-apples” comparison) because only the $V_S$ profiles obtained by the different methods from the common sites and common depths are investigated.

One point that must be mentioned here is that the $V_S$ profiles obtained from the SASW method provide more global information than the downhole and suspension logging methods. In contrast, the downhole measurements are rather localized and the suspension logging measurements are very localized. Therefore, there should be some difference between the measurements from the SASW, downhole and suspension logging methods. However, the difference should be small if median $V_S$ profiles are compared and the “red-apples-to-red-apples” criterion is used and if the three methods have similar precision in their measurements.

5.2 General comparisons (“apples-to-oranges” comparisons)

Thirty-one SASW, 23 downhole and three suspension logging $V_S$ profiles from Imperial Valley were used in these comparisons. The median $V_S$ profiles from the three different methods are compared in Figure 8. As seen, the SASW $V_S$ profile is consistently above the downhole and suspension logging $V_S$ profiles. However, based on Figures 1 and 2 and Table 1, there are more SASW $V_S$ profiles obtained in the mountain area and around the valley edge than downhole and suspension logging profiles. Due to the higher $V_S$ values in the mountain area and around the valley edge, the median SASW $V_S$ profile is higher than the other two profiles. These differences can not be used to investigate differences in the seismic methods because they are biased by not including the same materials in each data set.

On average, the differences (value of the median $V_S$ of one method minus the median SASW $V_S$ divided by median SASW $V_S$) of the median $V_S$ profiles between the SASW and downhole and between the SASW and suspension logging are about -12% and -18%, respectively.

To perform more objective comparisons, two different comparisons are conducted to study the consistency between the three seismic profiling methods as discussed below.

5.3 Common-site comparisons (“red-apples-to-green-apples” comparisons)

It is more objective to compare the median $V_S$ profiles obtained by different seismic methods from the same locations. Therefore, the “red-apples-to-green-apples” comparisons are performed to study the consistency or inconsistency between the different profiling methods.

First, a comparison between the SASW and downhole profiles from 21 common sites was performed. The results are presented in Figure 9. In comparison to Figure 8, the difference between the SASW and downhole median $V_S$ profiles in Figure 9 is much smaller. The average difference of the median $V_S$ profiles is about -3.3%.

![Figure 8](image-url). Comparison of the median $V_S$ profiles of 31 SASW, 23 downhole and three suspension tests performed in and around Imperial Valley, CA.

![Figure 9](image-url). Comparison of the median SASW and downhole $V_S$ profiles from 21 common sites in and around Imperial Valley, CA.
Second, another comparison is performed between the SASW and suspension logging measurements at three common sites. As seen in Figure 10, the consistency of the median $V_S$ profiles of these two measurements is much better than the comparison in Figure 8. On average, the difference between the two median $V_S$ profiles is around 6.8%.

As to the comparison between the SASW and suspension logging median $V_S$ profiles, the result is shown in Figure 10 over the depth range of 6 to 20 m. As noted earlier, the average difference is about 6.8%.

5.4 Identical-site-and-depth comparisons (“red-apples-to-red-apples” comparisons)

To perform the most objective comparisons between different seismic methods, only profiles from common sites and common depths at these sites should be compared. This approach means that the profiles are from the same depth ranges which have both SASW and downhole or SASW and suspension logging measurements.

First, the same SASW and downhole profiles from 21 common sites in the previous section are included. However, only the profiles of SASW and downhole measurements at their common depths at each common site are compared. These results are shown in Figure 11. The median $V_S$ profiles from the SASW and downhole measurement are very consistent. Their difference is smaller than in the previous two comparisons. On average, the difference between the two median profiles is -2.9%. However, on closer inspection, it is seen that the downhole profile exhibits higher median $V_S$ values in the top 3 m which is likely due to the lack of resolution of the downhole method in this depth range (Lin 2007). Below about 25 m, the SASW profile exhibits slightly higher values (about -8.6% over the depth range of 25 to 40 m), likely due to averaging over larger volumes combined with a decrease in resolution with depth inherent in the present SASW forward-modeling procedure.

6 CONCLUSIONS

6.1 Coefficient of variation

Based on the study in Section 3, it is feasible to use COV of the SASW $V_S$ profiles to evaluate the uniformity of soil deposits/rock formations. In addition, the study of the SASW $V_S$ profiles obtained from Imperial Valley shows that COV value of SASW measurements between 0.15 and 0.20 is a good index to estimate the uniformity (or non-uniformity) of the geotechnical structure of an area of interest.

6.2 Comparability/reliability of the SASW method

Though, there may be too few suspension logging $V_S$ profiles (only three) to draw an objective conclusion. However, according to the studies performed in Section 4, it shows the SASW method is comparable to the downhole and suspension logging methods, as long as correct comparison criteria are adopted. In addition, it shows the most objective comparison is the “red-apples-to-red-apples” comparison based on the comparison between the SASW and downhole median $V_S$ profiles.

7 REFERENCES


George Goble received a B.S. degree in Civil Engineering at the University of Idaho in 1951 and an M.S. and Ph.D. in Civil Engineering specializing in structures with a minor in geotechnical engineering from the University of Washington, Seattle. He studied at the Technical University of Stuttgart (Germany) as a Fulbright Student.

After two years in the U.S. Air Force, he worked as a Structural Inspector for the Oregon DOT and as a Structural Designer for Marshall Barr and Associates of Seattle, Washington. In 1961, he joined the faculty of Case Institute of Technology, (now Case Western Reserve University), Cleveland, Ohio. During the next 15 years he taught structures and mechanics courses there and was active in research on the dynamics of pile driving, structural optimization, bridge testing, and experimental structural behavior. He was Chairman of the Civil Engineering Department, 1975-77. He then joined the faculty of the University of Colorado, Boulder as Department Chairman of Civil and Environmental Engineering. He retired from the University of Colorado in 1992.

He has published about 125 papers in the areas of structural optimization, structural laboratory testing, dynamics and field testing of pile driving, field testing of bridges, determination of soil properties from dynamic measurements, geotechnical centrifuge testing, and safety evaluation of foundations and structures using probability analysis.

In 1972, he founded Pile Dynamics, Inc., the manufacturer of the Pile Driving Analyzer (PDA) and other measurement equipment for the pile driving industry. The PDA is now used in
about 45 countries. He was also the founder of Goble and Associates, Inc., Consulting Engineers, now GRL Engineers, specializing in dynamic monitoring of pile driving and design of pile driving hammers and equipment. Goble and Associates developed the WEAP program for the wave equation analysis of pile driving in 1976. In 2000, after his withdrawal from Pile Dynamics and GRL, he founded George G. Goble Consulting Engineer, LLC to serve as a vehicle for his specialized consulting interests, primarily in the deep foundations area. In 1989, he founded Bridge Diagnostics, Inc., a firm specializing in field testing and evaluation of bridges.

He received the ASCE Collingwood Prize in 1965, the fifth Award of the Lincoln Arc Welding Foundation Professional Structural Design Competition of 1966, the ASCE Martin Kapp Award of 1988 and the Deep Foundations Institute Distinguished Service Award of 1995. In 2004, Geotechnical Publication No.125, “Current Practices and Future Trends in Deep Foundations” was published in his honor by ASCE. In 2007 he was invited to present the GeoInstitute Terzaghi Lecture.

Goble is a licensed Structural Engineer in Washington and a licensed Civil Engineer in Ohio and Wyoming.
DRIVEN PILE INSTALLATION AND CAPACITY DETERMINATION

George Goble

Abstract

The purpose of this paper is to review the state-of-the-art of the engineering of driven pile installation and dynamic pile capacity determination. The development of these capabilities over the past half century will also be discussed. Two developments supported the changes that occurred – the appearance of the electronic digital computer and high speed electronic devices. It became possible to use the computer to realistically simulate the pile driving process. With the new electronic capabilities, measurements during pile driving impact became routine. Pile driving hammers also used measurements to control their operations but the primary basis for their development was the creativity of hammer manufacturers.

The primary goal of this paper is to give the designer and pile installer a view of current capabilities so that practice can be improved.

I. Introduction

Pile driving, as a foundations solution, is an ancient process. The remains of timber piles that were driven centuries ago have been found, usually imbedded in lake bottoms where the timber decays very slowly. Piles found in Lake Lucerne in Switzerland are estimated to be about 4000 years old. Probably they supported dwellings and provided the inhabitants protection from their enemies. Timber piles have been used continuously to the modern age. Other pile materials only appeared in about the last 150 years.

Until about 1950 pile driving was treated as a black art. Only an “old foreman” really understood the process. By 1950, this began to change. Gradually, analytical methods that modeled the driving process were developed. This effort continues to the present day. During this time improvements in pile driving equipment and in methods for making measurements during driving also progressed.

In this paper, developments in pile installation will be discussed by reviewing 1) status of the technology at the beginning of the 20th century, 2) one dimensional wave mechanics for understanding pile driving, 3) experimental developments, 4) analytical methods and 5) modern developments in pile driving hammers. Methods for determining axial pile capacity using information obtained from subsurface investigations (static analyses) will not be discussed here. The primary use of static analyses is to determine pile lengths for bidding. With the exception of the static load test, in almost all cases pile axial capacity is determined by dynamic methods and driving blow count. Quality control of axial
capacity usually uses a dynamic method, blow count. Dynamics will be emphasized in this paper.

II. Pile Driving in 1900

In 1900, analytical methods for the determination of axial capacity were limited to pile driving formulas or static load tests. At that time, the dynamic formulas were all based on conservation of energy. As shown in Figure 1 and Equation 1, the work done by the penetration of the pile into the soil is equated to the potential energy at the top of the stroke,

\[ R_u s = Wh \]  (1)

where \( R_u \) is the force acting on the pile to resist penetration (ultimate capacity), \( s \) is the permanent set induced by the hammer blow, \( W \) is the weight of the ram and \( h \) is the ram stroke. In the case of the drop hammer, the stroke can be observed during driving. For mechanical hammers, the rated energy usually replaces \( Wh \), but sometimes measured energy is used.

If the terms are rearranged, the units for \( W \) are kips, \( h \) is feet and \( s \), inches, a factor of safety of six is applied and a modification \( c \) is added to \( s \), the Engineering News formula is obtained.

\[ R_u = \frac{2E'}{(c+s)} \]  (2)

where \( E' \) is the energy of the hammer. The symbol \( E' \) is used for energy instead of the standard \( E \) because \( E \) is used here for modulus of elasticity. A large number of formulas were developed from the middle of the nineteenth century until the recent past. Corrections were made attempting to deal with the assumptions: (1) the soil resistance force is constant, (2) the resistance force consists of only static resistance, (3) the hammer efficiency is known and does not vary, (4) the pile is elastic and the pile length is not a factor. There are probably several other factors that are not listed here. Hundreds of dynamic formulas have been proposed but none are very reliable. However, to put formulas in the proper perspective, they are much more reliable than static analyses because they are used to determine a required blow count while the static analysis implies that all piles will be driven to a specified depth.
In 1900, most piles were driven by winch operated drop hammers. However, the mechanical hammer had been developed by the Vulcan Iron Works and its use was expanding. These hammers were usually steam driven as were the cranes that handled them. A photograph of a typical system is shown in Figure 2. These hammers were usually single acting.

III. One Dimensional Wave Mechanics

III.A. The Wave Equation and its Solution

The one dimensional wave equation has been with us for a long time. It was derived and solved in the 18th century (Timoshenko, 1934). An understanding of its possibilities was further developed in the 19th century, but it could not be applied to pile driving because the necessary fast measurement capabilities did not exist.

The one dimensional wave equation can be derived by modeling a segment of a slender rod during a wave passage including the deformation of the rod. If the equation of motion is applied to this model a second order, partial differential equation is obtained.

\[
\frac{\partial^2 u}{\partial t^2} = c^2 \frac{\partial^2 u}{\partial x^2}
\]  

(3)

where \( u \) is the longitudinal displacement of a point on the rod, \( c \) is the velocity of wave propagation, \( t \) is time and \( x \) is the position of the point on the rod. A solution to the equation is

\[
u(t,x) = f(x + ct) + f_1(x - ct)
\]  

(4)

It can be shown that \( f_1(x - ct) \) is a wave traveling in the positive direction along the rod while \( f(x + ct) \) is a wave traveling in the negative direction where \( c \) is the velocity of wave propagation.

III.B. Velocity of Wave Propagation

From the wave equation and its solution the velocity of wave propagation is

\[
c = \sqrt{E/\rho}
\]  

(5)
where $E$ is the modulus of elasticity of the pile material and $\rho$ is its mass density. So, wave speed is a material property. For steel it is 16,800 feet per second while concrete can have a wave speed ranging from about 11,000 to 14,000 feet per second. Timber has similar values. The concrete wave speed varies because both the modulus and mass density vary. The timber wave speed will depend on the species and the moisture content.

III.C. Wave Mechanics Concepts for Application to Pile Driving

Pile behavior observed during pile driving can be explained based on the wave equation solution. Illogically, today it is often the practice to define hammer performance for the purpose of hammer selection using rated hammer energy. But, for example, two hammers may have the same rated energy, but one has a short stroke and heavy ram and the other a long stroke and a light ram. These two hammers will behave differently – a difference that can be understood based on wave mechanics. Engineers and contractors that work in this area will benefit by gaining an elementary understanding of applied wave mechanics.

III.C.1. Force-Velocity Proportionality

The wave equation gives us relationships between force and particle motion. The relations between force, stress and strain and between displacement, velocity and acceleration are well known. The wave equation gives the relationship between these two sets of variables. For application to pile driving problems the relationship between force (or stress or strain) and velocity are the most useful and they are

$$F = \frac{EA}{c}v \quad (6)$$

$$\sigma = \frac{E}{c}v \quad (7)$$

$$\varepsilon = \frac{v}{c} \quad (8)$$

where $F$ is the force in the wave, $A$ is the pile cross sectional area, $\sigma$ is the stress, $\varepsilon$ is the strain and $v$ is the particle velocity in the wave. $EA/c$ is commonly known as the impedance and $E/c$ will be referred to here as the stress impedance. $E/c$ is usually more useful that $EA/c$ because it is independent of the pile area and so provides more generality.

If measurements are available they are usually shown graphically with $F$ and $(EA/c)v$ as functions of time. Since they are equal (Equation 6) they can be shown with the same units. The same exists with $\sigma$ and $(E/c)v$. This gives velocity in the same units as force or stress.

III.C.2. Fundamentals of Wave Propagation

From the wave equation solution the basic rules for wave propagation in a uniform cross section rod can be summarized using the usual sign convention: compression force positive, down velocity positive. In the text that follows the right will be referred to as down.
II.C.2.a. Simple Wave Propagation

Imagine that a force (and therefore a particle velocity) is applied instantaneously at the end of the rod so that a wave is induced. For simplicity, the instantaneously applied force is assumed to induce a vertical front on the wave. It propagates unchanged along the rod. Since the force and velocity are proportional if the velocity is multiplied by $EA/c$ it will have the same magnitude as the force; they will plot on the same line. This wave is illustrated in Figure 3d.

III.C.2.b. Wave Reflections from the Pile End

A compression wave will reflect upward as a tension wave (negative) from a free end and the velocity will reflect up the pile as a down velocity (positive). The sum of the downward traveling compression wave and the reflected tension wave will produce zero force at the end of the pile during the reflection. This satisfies the free end boundary condition. If the downward traveling compression wave ends the net result will be the upward traveling tension wave. The downward velocity will reflect upward as a downward (positive) velocity. Therefore, the velocity doubles at the end of the rod when it is reflected.

A compression wave will reflect up the pile as a compression wave (positive) from a fixed end and the velocity will reflect as an upward (negative) velocity. Thus, the force will double at the rod end when it is reflected and the velocity will be zero.

III.C.3 Action of a Resistance Force on a Downward Traveling Compression Wave

It is useful to be able to determine the location and magnitude of a resistance force that acts upward at some point along the pile against a downward traveling wave. The process is illustrated in Figure 3. An upward acting force is shown in Figure 3a. It induces an upward acting compression wave and a downward acting tension wave as shown in Figure 3b. Equilibrium requires that the two compression forces have same magnitude equal to one half the applied resistance, $R$, with the upward traveling wave in compression and the downward one in tension.

The particle velocity induced by $R$ is shown in Figure 3c. Since the applied displacement induced by $R$ is upward, the motion is upward. Compatibility requires that the particle velocity be upward. Proportionality between force and velocity requires that the downward traveling velocity be one half the applied value. As mentioned above an induced downward traveling wave is shown in Figure 3d.

Now imagine that $R$ is induced by the downward traveling wave (such as can happen during pile driving). Then $R$ will act to reduce the downward traveling wave below the point of application of $R$ and increase it above as shown in Figure 3e. When the velocity induced by $R$ is applied to the downward traveling velocity it reduces the velocity above and below the point of application of $R$. 
Now we see that the difference between the upward traveling force and velocity wave is equal to the magnitude of the applied resistance $R$.

All of the wave representations $a$, $b$, $c$, $d$ and $e$ show the wave as a function of position. Figure 3f shows the wave propagating as a function of time. The reflected force $R$ arrives at the top of the pile and the magnitude of the force can be measured. Also, the time of arrival gives the location of the resistance force.

An example of a force and velocity measurement is shown in Figure 4. The $2L/c$ time is illustrated and it shows the time from the force and velocity peak to the peak of the reflection from the pile toe. The magnitude of the sum of the
resistance force above one point is shown as indicated by A. It gives the magnitude of the total resistance from the pile top down to the point illustrated. This resistance consists of the sum of a static force component and a dynamic component.

III.C.4. Pile - Ram Weight Ratio

When an idealized rigid ram impacts a uniform cross section pile as shown in Figure 5 the pile top force-time response can be calculated. The model assumes that at the instant of impact all the particles at the pile end take on the velocity of the ram. This velocity is proportional to the stress (Equation 7) that propagates through the pile. Of course, the force induced at the pile top acts to reduce the velocity of the ram and so as the ram velocity decreases the top force decreases producing a decay that is exponential.

The equation of motion is applied to a freebody of the ram producing a first order differential equation that has a solution shown in Figure 5, where $\sigma_i$ is the stress at impact, $M_H$ is the mass of the ram, $M_P$ is the mass of the pile and $L$ is the pile length. So, $L/c$ is the time required for the stress wave to travel the length of the pile and $\alpha$ is the number of times the wave has traveled the length of the pile. The time $2L/c$ is a basic parameter that is used and often referred to in applied pile driving applications.

The solution of the differential equation is given in Figure 5. If $\alpha L/c$ is substituted for time and some arithmetic manipulations are performed a relationship for the stress becomes a function of the pile - ram weight ratio. The
normalized stress is plotted for the first $2L/c$ time in Figure 5. The stress declines exponentially as a function of $\alpha$. As the pile-ram weight ratio increases the stress decays more rapidly as a function of the $M_p/M_h$ ratios. This ratio is a useful parameter for understanding the ram weight implications in hammer selection.

If the stress record is increased in time beyond $2L/c$ the reflection will have arrived at the top from the toe. The top force can be extended beyond $2L/c$ by including the effect of the reflection. For examples of the extended records see Timoshenko, (1934).

Of course, in an actual case the stress does not increase instantaneously at impact. The measurements shown in Figure 4 illustrate an example of the rate of increase of the force that actually occurs at impact.

### III.C.5. Energy Calculation

Because there has been a strong interest in the energy transmitted to the pile (and to an SPT rod) it is useful the show the means of determining the transmitted energy. Figure 6 shows a force acting at the pile top (it could be at any point along the pile where measurements are available). The Force as a function of the displacement, $\delta$, is shown in the graph. An increment of energy, $\Delta E'$, passes through the measurement point when an increment of displacement occurs due to the action of the force. So the increment of energy is

$$\Delta E' = F(\delta) \Delta \delta$$

where $F$ is the force and $\Delta \delta$ an increment of displacement. The displacement increment is
\[ \Delta \delta = v \Delta t \]  \hspace{1cm} (10)

where \( t \) is time. Then, if \( F \) is given as a function of time and substituting for \( \Delta \delta \) into Equation 9, in the limit

\[ E'(t) = \int F(t) v(t) \, dt \]  \hspace{1cm} (11)

The problems associated with the application of Equation (11) will be discussed further in Section IV.C.3.b

IV. Experimental Developments

IV.A. Force Measurements

As indicated in Section III.A, the tools for measurement of very fast phenomena such as occurs in pile driving did not exist until the twentieth century. But methods and devices for deformation measurement were available early in the twentieth century. However, these devices could not respond nearly fast enough to be useful in measuring the shock phenomena characteristic of wave propagation.

Electric resistance strain gages were first conceived by Lord Kelvin in 1856 (Dally and Reilly, 1978) when he observed that the electrical resistance of copper and iron wire changed when they were deformed and that the change in resistance could be measured with a Wheatstone Bridge. This method was used to some degree but its usefulness was limited by the difficulty of obtaining an accurate calibration.

In the mid-1930’s, Ruge and Simmons (Simmons, 1940) independently developed the means of making a resistance strain gage that could be calibrated and easily attached. They mounted a carefully measured length of wire on a paper backing. If the length of the wire was known and the resistance change measured for a unit deformation, the gage could be calibrated. The paper backed gage could be mounted easily in the laboratory. The resistance change was measured with a Wheatstone Bridge circuit and the strain determined (Dally and Reilly, 1978).

It is interesting to note that even though Simmonds obtained a patent it probably could have been challenged by Ruge so they apparently joined together in commercializing it. The product was known as the SR-4 gage and it was widely used. It is remarkable that these two men, working independently, were able to reach an agreement to work together to commercialize the gage. Many lawyers were probably disappointed.

Finally, the etched foil strain gage was invented by Sanders and Roe in 1952 (Dally and Reilly, 1978). This gage improved the accuracy of a strain gage and simplified its manufacture. The response of the resistance strain gage is related to the length of the gage and it is much greater than required to measure the pile
hammer impact event. It satisfied the requirements for making measurements of the force in a wave that is induced by pile driving hammer impact.

IV.B. Devices for Motion Measurements

It was shown in Section III.C. that motion measurement, and particularly particle velocity, is necessary for evaluating wave propagation during pile driving. Until recently, a direct measurement of either velocity or displacement was not possible because to make such a measurement a fixed reference frame is necessary. There is sufficient ground motion in the vicinity of pile driving to make it impossible to use the ground near the pile to provide a fixed reference frame for supporting the instrument. In addition, it would be necessary to continually reset the device as the pile penetrated. However, recently non-contact displacement measurement devices based on the use of laser technology have become available. The laser follows the motion of a point at a distance. It has been used in making measurements during Statnamic testing. The laser measurement device does have to be reset during pile penetration. These devices will be discussed further in Section V.B.3.

Acceleration can be measured to obtain velocity since the reference is the acceleration of gravity. The acceleration measurement can be integrated to obtain velocity. The primary problem with acceleration measurement is that a very large acceleration immediately followed by much smaller values must be measured accurately. An accelerometer that can survive and accurately measure the large acceleration will be less sensitive for small accelerations than is desirable.

Two basic devices are in use for measuring acceleration – piezoelectric and strain measurement sensors. In both cases, the acceleration force is measured to obtain acceleration. In the case of the piezoelectric accelerometer, a mass is mounted on a quartz crystal (more recently the crystal has been replaced by manufactured materials). When the mass is accelerated it applies a force to the crystal inducing a potential. The potential is amplified in both charge and power. Piezoelectric accelerometers have the advantage of a high natural frequency for use in high frequency applications.

Another accelerometer type uses a strain measurement to obtain the acceleration. For example, a strain gaged cantilever beam can sense the acceleration force by measuring the bending strain. The basic difficulty with these devices is that they have a low natural frequency. To deal with this problem, the unit can be incased in a fluid to damp the high frequency component of the motion and avoid resonance in the accelerometer. The strain gage accelerometer will continue to measure a constant acceleration or a slowly changing one while the piezoelectric device will gradually lose the charge and return to zero. For that reason the strain gage accelerometers are known as D.C. devices and this characteristic can be very desirable in some applications.

Recently, extremely small accelerometers have been developed for use in automobile crash testing. These are D.C. devices that sense the acceleration with a measurement of deformation of an extremely small membrane.
IV.C. Measurement Applications

IV.C.1. British Building Research Institute

The first strain measurement on a driven pile was performed at the British Building Research Institute in 1938. They built a transducer that attached directly to a concrete pile and measured strain. The measurements appear to be correct and reasonable. It seems that no attempt was made to make use of the measurements.

IV.C.2. Michigan Project

In 1961, a major field testing project was undertaken by the Michigan Highway Department and directed by Professor House of the University of Michigan (Michigan State Highway Commission, 1965). The purposes of the project were very broad to include the study of a variety issues including static performance, pile type selection and driving problems. However, the primary motivation for the program was to evaluate the performance of diesel pile driving hammers and to establish a means of determining an energy rating. Of course, this required the development of the capability for making difficult field measurements. It was a very ambitious project to complete in a short time. New measurement systems require time for testing and modifying the equipment, particularly electronic measurement systems. In spite of the short time, they succeeded in producing a working measurement system and collecting a lot of field measurements.

Eighty-eight piles were driven at three locations around the state. Twenty-five were dynamically tested and nineteen were load tested statically. The dynamic testing consisted of force and acceleration measurements at the pile top during driving. The force measurements were made with a large force transducer that was instrumented with resistance strain gages and statically calibrated.

Most interesting was the use of an accelerometer attached to the force transducer. This was the first time that acceleration measurements were made during pile driving. The purpose of those measurements was to determine particle velocity. The accelerometer had a maximum range of 500 g, possibly too small to measure acceleration peaks on steel piles, now known to be as large as 1000g or more. This may not have been critical because the high acceleration peaks were very short. The data was recorded on an oscillograph that ran at speeds up to 80 inches per second, sufficient to provide a satisfactory record.

Figure 7 shows a schematic of the Michigan test system together with the later Case system. An example of the Michigan measurements is shown in Figure 8. The peak force measured at impact was about 1000 kips, much greater than the yield strength of the steel so there was an error of some sort. The difficulty with the measurements was not understood for several years until Goble and Rausche (1972) provided the explanation.
There was a large mass (adaptor) under the force transducer. It generated an inertia force during the large downward acceleration at impact and that force was measured by the force transducer. That explains the very large force peak at the beginning of the record shown in Figure 8a and also the large variability throughout the force record. The measured force could be adjusted by using
acceleration measurements applied to the adaptor. When the measured force was modified a much smaller peak force at impact resulted. The adjusted force is quite variable due to the variable acceleration but it was smoothed and that produced a force record similar those measured on the Case Project. It can be concluded that the measurements would have been good if the adaptor had not been present.

In spite of the problems with the measurements, the energy values obtained from the measurements should be correct. This project made major steps forward in pile experimental work in spite of the measurement problems.

**IV.C.3. Case Project**

In 1964, a research project was initiated at Case Institute of Technology supported by the Ohio DOT and the Federal Highway Administration. The project had the goal of developing a method for predicting pile capacity from dynamic measurements. The basic concept for capacity measurement was based on prior research at Case (Eiber, 1958). That research will be discussed in a later section. The Ohio DOT project continued to study several problems of driven pile installation and capacity determination until 1977.

**IV.C.3.a. Force Measurement Sensors**

At the beginning of the Case project, steel pipe piles, the only pile type tested at that stage of the project, were instrumented with foil strain gages applied directly to the pipe near the top. Two pairs of two element gages were attached on opposite sides of the pile and each pair connected in a Wheatstone Bridge. The pair provided redundancy for the measurements and this measurement system gave excellent results. However, for field applications and particularly in winter weather the gage application was very difficult, time consuming and unrealistic for routine measurements. In the case of concrete piles, the direct use of resistance gages would not have been satisfactory because of the non-uniform characteristic of concrete and the possibility of concrete cracking.

In 1966, a pile top transducer was developed for use on the pipe piles. All of the pipe piles used by the Ohio DOT were 12-3/4 inches in diameter with a variety of wall thicknesses. The transducer consisted of a 12-3/4 inch pipe that was strain gaged on the inside and calibrated in a materials testing machine. The accelerometers were also attached on the inside of the transducer. A removable fitting that fit inside the pipe pile was bolted
on the bottom end. A heavy pipe that fit in the helmet was bolted to the bottom of the transducer. One of these transducers is shown in Figure 9. Because it weighed over one hundred pounds it was difficult to use when air travel was required. However, it was quite satisfactory for testing in Ohio. While this type of transducer is no longer used in dynamic pile testing it is probably still the most efficient for testing a large number of pipe piles at a particular site.

An effort was made over an extended period of time to develop a strain transducer that could be easily attached near the pile top. It was finally discovered that the device had to be quite flexible to avoid the attachment slipping during the passing of a stress wave. A large series of devices were developed over a period of time. They are shown in Figure 10. The development of the transducers proceeded from right to left. The final device was very effective and it is still in use for dynamic pile testing and also in other applications.

IV.C.3.b. Acceleration Measurement

Acceleration measurements proved to be much more difficult than strain measurements. Piezoelectric accelerometers were selected because of their high frequency response. Their primary disadvantage was the difficulty in amplifying and recording the signal using the long lead wires that were required in the pile driving application. The weak signal required both charge and power amplifiers and noise problems were always present. Piezoelectric Accelerometers that contained the necessary amplifiers internally became available and they solved most of the acceleration measurement problems. Of course, there was the fundamental problem with the need to measure the large impact acceleration and then the small acceleration after impact.

But, an occasional problem appeared with the accelerometer measurement. The accelerometer would simply fail to operate. It was particularly frequent during measurements made on the Standard Penetration Test (SPT) for the purpose of determining transferred energy. When Schmertmann began studies on the efficiency of energy input to the Standard Penetration Test he was unable to
obtain satisfactory accelerometer measurements. He took advantage of the force-velocity proportionality to calculate energy. Using this approach the calculation of transferred energy is made using Equation (11),

$$ E'(t) = \int F(t) v(t) \, dt $$

(11)

The force-velocity proportionality is

$$ F = \frac{E A}{c} (v) $$

(6)

So, substituting in (11) for \( v \)

$$ E'(t) = \int F^2(t) \frac{c}{EA} $$

(12)

For this approach to give correct values of energy the signal must not be more than \( 2L/c \) long. If that condition is not satisfied the integration will include some input force still coming in when the toe reflection arrives back at the top. To avoid this problem the SPT rod must be quite long. This work of Schmertmann was valuable because it caused geotechnical engineers to become concerned about SPT energy standardization (Pallacios, 1977).

Hauge (1977) presented an explanation of the problem. Strain measurements were made on SPT rods in the laboratory and the problem was found to be the generation of very high frequency signals, in the range of the resonance frequency of the accelerometer during the steel-to-steel impact in SPT testing. It was determined by making force measurements using a very fast oscilloscope. A record of one of the force signals measured in the laboratory during such an event is shown in Fig. 11. Frequencies in the range 40 to 80 kHz were recorded but they were not reproducible. These frequencies, observed in the strain signals, were near the resonance frequency of the accelerometer and this overloaded the amplifier. This recording was made in 1980 at a time when the capability for recording fast events could only be made with a high frequency analog oscilloscope.

This problem was only infrequently observed in tests on pile driving because the driving system usually had a cushion and that avoided steel-to-steel impact that generated the high frequencies. Because of this problem the pile measurement equipment had not been used up to that time for measurement of SPT transmitted energy.
When D.C., beam-type, fluid-damped accelerometers became available the problem was solved. Later damped piezoelectric accelerometers that gave satisfactory measurements also became available. Today this problem does not occur.

IV.C.3.c. Recording Equipment

At the time the Case Project began, the only realistic recording equipment was an oscillograph. Analog tape recorders were available but they all used one half inch tape and they were not realistically portable. In addition, the cost of the tape was prohibitive. The best available oscillograph, the same as was used in the Michigan project, had a maximum speed of 80 inches per second. For typical pile records this speed was adequate. With care it was possible to record a dozen hammer blows on the standard 100 foot roll of paper. The data was manually digitized for analysis.

V. Applied Analytical Developments

V.A. Wave Equation Analysis

The original developer of a discrete method of analyzing the pile driving process chose to call it “Wave Equation” analysis. This tool has become standard for predicting pile driving characteristics for a variety of piles, hammers and soils. The name is somewhat unfortunate in that it is related to a discrete method of analysis and it should not be confused with the differential equation that defines wave propagation.

V.A.1. Smith Wave Equation

One of the most remarkable Civil Engineering achievements of the 20th century was the development of the “Wave Equation.” This work was done from original concept to application by E. A. L. Smith, Chief Engineer of the Raymond Company. After developing the basic concept he took it to the International Business Machines Company (IBM), then a new company, to program it for computer analysis (Smith, 1951, 1957 and 1960).

Smith’s model of the pile driving system is shown in Fig. 12. The hammer, driving system and pile are modeled as a discrete, one dimensional series of masses, springs and dashpots. The soil resistance is represented as an elastic-plastic spring with a linear damper as illustrated. The static soil resistance is

\[ R_s = \left( \frac{R_u}{q} \right) u_i \quad \text{if } u_i \leq q \]  

(13)

for the linear elastic portion and

\[ R_s = R_u \quad \text{if } u_i = q \]  

(14)

for the plastic portion.

\[ R_s \] is the
static resistance (a function of $u$), $R_u$ is the ultimate static resistance, $u_i$ is the displacement and $q$ is the quake, the displacement where the resistance becomes plastic.

Three methods have been used to define the dynamic resistance. The procedure known as the Smith Method and like the Smith static resistance it has two definitions of damping

$$R_D = j_0 R_s v \quad \text{if} \quad u_i \leq q \quad (15)$$

and

$$R_D = j_0 R_u v \quad \text{if} \quad u_i > q \quad (16)$$
where $j_s$ is the Smith damping constant. This definition of the resistances is known as Smith resistances. The second method was developed and used during the Case research project. The dynamic resistance was related to the pile impedance as

$$R_D = j_c (EA/c) v$$

(17)

where $j_c$ is known as Case damping. A third approach designated as resistance damping is a modification of Smith damping to use only the ultimate static capacity instead including the loading portion of the force displacement relation.

$$R_D = j_R R_u$$

(18)

where $J_R$ is the damping constant. Smith and Resistance damping have the unusual characteristic that $j_s$ and $j_R$ have the units sec/ft and it seems intuitively that they should be dimensionless. The Case constant is dimensionless. On the other hand, it is difficult to justify that damping resistance should be related to the pile impedance.

To perform the wave equation analysis the necessary input data is selected by an examination of the soil investigation and from knowledge of the hammer, driving system and pile. The ram is dropped and the forces propagate along the series of masses and springs. The pile forces, soil resistances and displacements are calculated.

Smith also produced an approach to use the results of the analysis. The analysis is repeated for successive increases in capacity and the blow count is calculated for each selected capacity. This produces a curve known as a bearing graph such as is shown in Fig. 13. For a given blow count the associated capacity is given. Likewise, for a given capacity the blow count is given. This development also represented a creative accomplishment. Maximum driving stresses, force and velocity versus time records and stroke for diesel hammers is also available.

His modeling of the problem as a discrete system may have been the first application of this concept that we find so common today. Its development must have been early 1940’s. The computer program that he prepared may have been the first civilian engineering application of a digital computer (Schiffman, 1998).

V.A.2. Texas Transportation Institute Wave Equation Program (TTI)

The Smith program was proprietary to the Raymond Company. In about 1962, the Texas Highway Department was having problems with breaking concrete piles. They contracted with the Texas Transportation Institute of Texas A&M University to develop a wave equation program following the Smith approach. This program became known as the TTI program (Samson et al 1963). It went into practice and was widely used by the offshore oil industry beginning in about 1964.

It also
gained some usage in land practice but there the application developed more slowly. It has been surprising that the Wave Equation was so slow in coming into practice. The TTI program was easy to use and it represented the pile driving process well, but as late as 1980, implementation of wave equation analysis was far from complete. Today, not all pile foundation designers do drivability analyses routinely.

V.A.3. Wave Equation Analysis of Piles (WEAP)

In 1974, work was undertaken by Goble and Associates on a program funded by the Federal Highway Administration and the New York DOT that was intended to develop a wave equation program that included a model for the diesel hammer
(Goble and Rausche, 1976). In the case of a diesel hammer, the stroke of the ram varies with the driving resistance and probably several other factors such as axial pile flexibility and soil dynamic response. The primary development was to treat the ram motion in a more realistic fashion. Diesel hammer operation is described in Section VI.

This computer program, Wave Equation Analysis of Piles (WEAP) was supported by FHWA for several years and then it became a GRL proprietary program for the purpose of continued support. It is the primary wave equation analysis program in use today.

V.A.4. The Fischer Method

Professor Hans Fischer proposed a method for analyzing one dimensional wave propagation in 1954 (Fischer, 1954, Fischer, 1959). Subsequently, the entire work was reviewed at the Second International Conference on the Application of Stress-Wave Theory on Piles (Fischer, 1984).

The Fischer Method is based on the fact that a stress wave travels unchanged in a uniform cross section pile at a wave speed defined by the pile material properties. At a change in area of the pile cross section, wave propagation forces arriving at the section will be transmitted and reflected. If the pile is represented by a series of discrete elements, they can each be treated as a uniform cross section element. If resistance forces or cross section changes are present at the element boundaries the wave propagation can be represented as a progression of constant forces traveling through the elements. The resistance forces are applied at the element boundaries. Transmitted and reflected forces are determined at each boundary at the appropriate time intervals. This process is repeated, in time, for each element boundary until the full wave propagation analysis has been determined for all of the element boundaries.

One problem with the Fisher Method is modeling of the hammer and driving systems in the wave equation analysis. As yet this problem has not been solved.

V.B. Capacity Determination from Measurements

V.B.1. Initial Development

Nara and Eiber (Eiber, 1958) were the first to attempt to determine pile capacity by dynamic measurements and analysis. They measured accelerations near the top of a rod that was driven in the laboratory into a container filled with sand. They modeled the system by treating the ram and the pile as a single element and then applying Newton's Second Law to the model.

\[ R_u(t) = ma(t) + c'v(t) \]  (19)

where \( m \) is the mass of the ram and the rod together, \( c' \) is a constant applied to the velocity to determine the dynamic resistance and \( R_u \) is the soil resistance. The value of \( R_u \) at the time that the velocity is zero is
\[ R_u = ma(t_{v=0}) \]  

and \( R_u \) is the predicted pile capacity. The assumptions made in this approach are that the dynamic resistance is zero when the velocity is zero, the rod is rigid and the ram and the rod move together. In the limited number of tests performed the results were promising as shown in Figure 14.

V.B.2 Rigid Body Case Method

In 1964, a project was started at Case Institute of Technology (now part of Case Western Reserve University) funded by the Ohio DOT and the Federal Highway Administration. This project continued studies of pile problems until 1977.

Dynamic tests were made during re-strike on piles that had been statically load tested. Force and acceleration was measured at the pile top. As in the case of the Nara and Eiber work the pile was assumed to be rigid and the capacity was determined at the time when the pile top velocity was zero. The model shown in Figure 15 was used. Since the pile top force was measured it was not necessary to include the mass of the hammer and driving system in the model. This was necessary because there was significant relative motion between the ram and the pile top due to the hammer cushion.
The general relationship, given in Equation 19 is similar to that used by Nara and Eiber with the force measurement included. The capacity is determined by

\[ R_u(t) = F(t) + c'v(t) + ma(t) \] (21)

Again, if the expression is evaluated at the time when the velocity is zero the capacity, \( R_u \), is

\[ R_u(t_{v=0}) = F(t_{v=0}) + ma(t_{v=0}) \] (22)

Data was collected on a large number of piles that had been statically load tested. Ohio was an ideal location to undertake the work because a large number of piles were statically load tested every year.

At that time the Ohio pile installation practice was divided into two parts. In northern Ohio, piles were usually driven to rock while in Southern Ohio most piles were either friction piles or had a resistance that was divided between the shaft and the toe. In Northern Ohio, H-piles were used and a standard blow count was specified – usually 10 blows per inch. In Southern Ohio the resistance was divided between the shaft and the toe and the driving criterion was established by static load test. The Ohio DOT specified 12-3/4 inch closed end pipe piles that were filled with concrete after driving and testing.

In an unusual practice, the DOT did not specify the wall thickness of the pipe, but only the required capacity. The contractor was required to select the pile wall thickness and prove the driving criterion by static load test to three times the design load. A contractor that was able to drive very thin-walled piles could save on the project cost. Therefore, a large number of static load tests were performed each year.

The research project was organized so that, when a static test failed on a DOT Project a dynamic re-strike test was performed and dynamic measurements were made by the Case Project. The load test was carried to as much as three times the design load. If it did not carry twice the design load the pile was driven further and the load test repeated. So the piles tested had a capacity between twice and three times the design load. (It is of interest to note that the contractor was not paid for the re-strike). When the static test failed the research project was informed by the DOT and the project contacted the contractor and asked him to perform the re-strike test when the load test frame was removed. In more than 70 cases, only one contractor refused to perform the test free.

V.B.3 STATNAMIC

The STATNAMIC concept was developed and patented by Patrick Bermingham (Bermingham and Janes, 1989). The concept uses a large mass on top of the
pile supported by an explosive system between the pile top and the mass. Actually, the “explosive” burns relatively slowly in order to generate a force acting on the pile that is long enough to avoid problems with elastic wave propagation. The mass is accelerated upward and, of course an equal force acts downward and the downward acting force loads the pile. A sketch of the system is shown in Figure 16. An example of a force versus time record is shown in Figure 17.

Pile capacity is determined using the rigid body Case Method. Stokes et al (2008) have collected STATNAMIC data where static load tests were performed and have developed associated correction factors based on soil characteristics.

![Schematic of the STATNAMIC System](image)

Figure 16

V.B.4. Elastic Case Method

Rausche developed a method for predicting pile capacity using an assumption of elastic pile behavior (Rausche, 1970 and Rausche et al, 1985). He showed that

\[ R_{\text{TL}} = \frac{1}{2} \left[ F(t_1) + F(t_2) \right] + \frac{1}{2} \left[ v(t_1) - v(t_2) \right] \frac{EA}{c} \]  

(23)

where \( R_{\text{TL}} \) is the total resistance to penetration of a pile during driving, \( F \) and \( v \) are the force and velocity measured at the gage location. The time of the peak force and velocity at impact is designated \( t_1 \). The time \( 2L/c \) after impact is \( t_2 \), the time of the arrival at the pile top of the initial impact reflected from the toe.
[\nu(t_1) EA/c + F(t_1) - R_{TL} ] \quad (24)

However, $R_{TL}$ includes both static and dynamic resistance. Then Rausche assumed that the dynamic resistance is concentrated at the pile toe and that the toe velocity in force units can be approximated by

The dynamic part of the driving resistance is obtained by assuming it is linearly related to the toe velocity. It is multiplied by a damping constant, $j$. Then if the dynamic resistance is subtracted from the total resistance the static resistance is

$$R_u = R_{TL} - j [\nu(t_1) EA/c + F(t_1) - R_{TL}] \quad (25)$$

where $j$ is the user input damping constant. The damping constant was assumed to be dependent on soil type. Experience has shown that the relation between soil properties and the damping constant is rather poor. However, if the damping constant is calibrated by a static load test or by a signal matching analysis the Case Method can be quite usable. The derivation of the Case Method is presented in Rausche et al, (1985).

V.B.4. SIGNAL MATCHING ANALYSIS OF MEASUREMENTS

The concept of signal matching analysis is based on the fact that in one dimensional wave propagation when there are no resistance forces acting along the pile shaft and at the toe the force and particle velocity must bear a known relationship to each other. Likewise, when resistance forces are active the
relationship between force and velocity is affected in a known way as shown in Figure 3. If the force at the pile top is known as a function of time, and the resistance forces are known, the particle velocity can be calculated as a function of time. The same is true in determining the force from a known velocity. However, the resistances cannot be determined directly. Given a known force and velocity as a function of time the resistances can be determined in iterative manner using an interactive procedure.

The force consists of a static and a dynamic component but only the static resistance is of interest in determining the pile capacity. Rausche (Rausche, 1970) solved this problem and produced a capability for performing the computation iteratively using a computer program known as Case Pile Wave Analysis Program (CAPWAP). This analysis is now in routine use.

As originally developed by Rausche (1970, Rausche et al, 1972) the pile model was the same as the Smith lumped-mass-spring model. The soil was also represented using the Smith model. More recently the Fischer Method of analysis has been used to produce a more realistic analysis and a better quality match. An example of a CAPWAP analysis is shown in Figures 20 - 23.

![Image of force and velocity measurements]

**Dynamic Measurements**

Figure 20

Figure 20 shows the measurements. The dashed line is the measured velocity and the solid line is the force. Figure 21 shows the match between the measured and calculated force. The dashed line is the calculated force. The match between the two is very close. Figure 22 is the simulated load test curve showing both the top and bottom force-displacement curve. The shaft resistance distribution is given in Figure 23.
CAPWAP Match of Force and Velocity
Figure 21

“Static” Load Test
Figure 22
VI. PILE DRIVING HAMMERS

VI. A. Air/Steam Hammers

The hammers used in 1900 were limited to winch operated drop hammers and steam operated mechanical single and double acting hammers. At that time almost all of the mechanical hammers were manufactured by the Vulcan Company. The dominant hammer type was still probably the drop hammer. The air/steam hammers were operated exclusively by steam when they first appeared but with time as larger air compressors became available, compressed air became more common than steam. Today, all of these hammers are operated on compressed air in land work. In the offshore application, some steam hammers are in use but today the offshore work is usually done with hydraulic hammers. During this time there was little change in air/steam hammer design.

The typical single acting air/steam hammer has a stroke of three feet or sometimes a bit more. A few larger stroke hammers have been built. For example the Vulcan 6300 has a six foot stroke with a ram weight of 300 kips.

There are very few double acting hammers in use today except where the piles are being driven to depth. For example, in the soft clays of Southern Louisiana double acting air hammers are commonly used because their fast operation is more efficient than single acting hammers and the piles are driven to a required penetration. Capacity is then proven, after a wait time, by static load test or a re-strike dynamic test.
Double acting air hammers are difficult to inspect to assure capacity when piles are driven to a blow count. They have been found to be unreliable because the impact velocity and speed of operation does not change substantially when the operation is unsatisfactory.

Single acting air hammers run at about 60 blows per minute while double acting hammers will run at 120 to 140 blows per minute. The few air hammers with larger strokes run slower that 60 blows per minute.

VI.B. Diesel Hammers

The diesel hammer was the next major development in pile driving hammers. It was invented in Germany in the 1920’s but it developed quite slowly. When it first appeared in the United States in the 1950’s it was viewed with considerable suspicion. The variable stroke made it difficult to believe the rated energy for use in a dynamic formula, the most common means of determining pile capacity at that time. The diesel hammer performance was the major reasons for the Michigan tests.

The most common diesel hammer is open ended – that is, the top of the cylinder is open allowing the ram top to go above the top of the cylinder. A drawing showing the major parts of the diesel hammer is shown in Fig. 21. It operates as a two cycle engine with an Otto Cycle. The name diesel has been attached to it because it burns diesel fuel. Some diesel hammers do use an approximate diesel cycle by injecting the fuel in an atomized state causing it to begin burning immediately. However, that has the disadvantage of slowing the velocity of the ram prior to impact more than liquid injection. With liquid injection combustion takes place after impact. If combustion occurs prior to impact the ram velocity is reduced and driving is less effective. This is called pre-ignition and it is undesirable. Pre-ignition can be caused by the use of a fuel with a low ignition temperature or possibly when the hammer becomes very hot in extended operation in hot weather.

The operation can be described briefly with reference to Figure 22. Beginning on the down stroke (Figure 22a) starting with the ram at the top of its stroke it falls free until it passes the ports (Figure 22b). At that time, the air in the chamber is compressed and a gradually increasing force is applied to the pile top. The ram continues its down-stroke in compressing the air in the chamber. When the ram impacts the anvil (impact block) it atomizes the fuel and it burns (Figure 22c). The ram bottom and the top of the impact block are manufactured so that with the ram in the full down position there is still a small chamber that contains the compressed gas. A diesel hammer will typically have a compression ratio of about 13 to 1. With impact, the pile deflects downward due to the impact force. If the driving is hard the pile will have a large rebound and this rebound will provide an initial upward velocity. The gas pressure will also act to increase the velocity. In easy driving there may be little or no rebound and then the cylinder pressure must raise the ram. When the ram passes the port the gas pressure exhausts and the ram continues up pulling in fresh air and scavenging the cylinder Figure 22d. The cycle then repeats.
The stroke will depend on the pile rebound together with the combustion pressure. In very easy driving the stroke may not be adequate to provide sufficient scavenging for the hammer to run. The open end Diesel will run at about 40 blows per minute.

Another diesel hammer type is the closed end hammer. This machine has an operation that is similar to the open end hammer except the top of the cylinder is closed. On the up stroke air is compressed in the upper cylinder, slowing the ram down on the upstroke and then accelerating it downward. Some time after passing the ports the fuel is injected in an atomized form and it begins to burn. After impact the cycle repeats. The closed end diesel hammer has the advantage of running at a higher speed, about 80 blows per minute and it will run in very easy driving while the open end hammer may not. The principal disadvantage is that it has a lower impact velocity and therefore it will not drive as hard. The closed end diesel is now used less frequently.

Figure 23 shows a very large hammer, the APE 180 diesel hammer with a ram weight of about 40 kips.
Diesel Hammer Operation
Figure 22

APE D180 Diesel Hammer
(Courtesy of American Piledriving Equipment)
Figure 23
VI.C. Hydraulic Hammers

Probably the first hydraulic hammer was built by the Raymond Company in about 1970. That hammer is shown in Figure 24. It was a double acting hammer that had been constructed by modifying an existing double acting air hammer. The hammer did not succeed in achieving broad usage, perhaps because the Raymond Company failed not long after it appeared. Measurements made by the author in 1972 showed a rather poor performance. However, that was probably due the timing of the downward acting pressure.

Today hydraulic hammers have become very popular and there are several manufacturers. Most of them are single acting.

In the single acting hammer the hydraulic fluid lifts the ram and then, on the down stroke, the pumped hydraulic fluid is stored in an accumulator for use on the next up stroke. One of the major advantages of the hydraulic hammer is the ability to change the stroke continuously, even during driving. Of course, if the stroke becomes shorter, the hammer operates faster. Because of the continuously variable stroke it is necessary to measure the hammer performance during driving and this is now a common feature of installation specifications in the United States. That is usually accomplished by measuring the impact velocity, calculating the kinetic energy and displaying it.

References


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Spencer J. Buchanan '26 Chair in Civil Engineering

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