GEOPHYSICAL CHARACTERIZATION OF SITES

Volume Prepared by ISSMFE
Technical Committee # 10

for
XIII ICSMFE, 1994, New Delhi, India

Edited by
Richard D. Woods
(Chairman, TC # 10)

OXFORD & IBH PUBLISHING CO. PVT. LTD.
New Delhi Bombay Calcutta

THE SEISMIC PIEZOCONE: A PRACTICAL SITE INVESTIGATION TOOL

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SYNOPSIS: In-situ soil testing procedures with the downhole seismic piezocone have been under continuous development at the University of British Columbia since 1980. Currently, this rapid and cost effective tool can be used to measure the average dynamic properties of shear wave velocity and damping with depth along with the interpretation of detailed stratigraphy and geotechnical parameters. The entire testing procedure can take less than two hours for a 20 m deep sounding including pore water pressure dissipations at selected depths.

The purpose of this paper is to briefly introduce the evaluation of cone penetration technology and then to present, in a simple and practical way, the latest seismic piezocone procedures for testing and signal processing to determine low strain dynamic properties of soil such as shear wave velocity measurements and, a much recent development, soil damping. Typical results will be presented and discussed from several different sites around the world including some where results from the seismic piezocone are compared to traditional, more expensive and involved, crosshole seismic test results.

INTRODUCTION

The cone penetration test has become increasingly more popular as an in-situ test for site investigation and geotechnical design. As a logging tool this technique is unequaled with respect to the delineation of stratigraphy and the continuous rapid measurement of parameters like tip resistance, sleeve friction and pore pressure.

Recent publications have provided vast amounts of information about cone penetration methods and their interpretation (e.g. ASCE In-Situ 86, 1986, ASCE, 1991 and ISOPT 1, March 1988). In addition, much experience has been developed at the University of British Columbia (UBC) over the past 15 years in research, development, interpretation and application of cone testing from many service to industry projects as well as thesis research. A summary of this experience at UBC has recently been updated by Campanella and Robertson (1992).

Starting in the mid-1980's, equipping standard electric cones with seismic pick-ups has greatly increased the value of these tools for site investigation. Small strain wave velocities, and more recently soil damping ratio, can now be determined in an accurate, rapid and highly repeatable fashion. A further attraction of the seismic cone technology is the much lower cost involved than standard geophysical seismic methods.

CONE PENETRATION TEST (CPT)

Probing with rods through weak soils to locate a firmer stratum has been practised since about 1917. It was in the Netherlands in about 1934 that the CPT was introduced in a form recognizable today. The method has been referred to as the Static Penetration Test, Quasi-static Penetration Test, Dutch Sounding test and Dutch Deep Sounding Test. The first electronic cone was introduced in 1948 and later vastly improved in 1971 (de Ruiter, 1971), and used strain gauged load cells to measure the tip load and the frictional sleeve load.

In the cone penetration test (CPT) a 60° apex and typically 35.7 mm diameter (10 sq. cm. area) cone on the end of a series of rods of the same or lesser diameter as the cone is pushed into the ground at a constant rate (2 cm/sec) and continuous or intermittent measurements are made of the resistance to penetration of the cone. Measurements are also made of the resistance to penetration of a surface sleeve, which is typically 150 cm^2 surface area by 35.7 mm O.D. located behind the tip.

In soft soils, cone penetration to depths in excess of 100 metres (330 feet) may be achieved provided verticality is maintained. Gravel layers and boulders, heavily cemented zones and dense sand layers can restrict the penetration severely and deflect and damage cones and rods, especially if overlying soils are very soft and allow rod buckling.

PIEZOCONE TEST (CPTU)

One of the most significant developments in CPT technology was the addition of pore pressure measurements (CPTU). The addition of pore pressure measurements has added a new dimension to the interpretation of geotechnical parameters, particularly in loose or soft, saturated deposits. Campanella and Robertson (1988) outline the CPTU's advantages, limitations, general state of development, testing and interpretation procedures, and experiences and usage around the world.

Briefly, the main advantages of the CPTU over conventional CPT are:

- ability to distinguish between drained, partially drained and undrained penetration;
- ability to correct measured cone data to account for unbalanced waterforces due to unequal end areas in cone design;
- ability to evaluate hydrological flow and consolidation characteristics;
- ability to assess equilibrium groundwater conditions;
- ability to assess stress history;
- improved soil profiling and identification; and
- improved evaluation of geotechnical parameters.

The primary purpose of the CPT and CPTU is stratigraphic logging and screening evaluation of geotechnical parameters. Other in-situ test methods or sampling and laboratory testing can be better suited for use in critical areas that have been defined by screening with the CPT or CPTU. The CPT or CPTU should be used to determine the locations and elevations at which other in-situ tests and/or sampling should be carried out.

SEISMIC PIEZOCONE TEST (SCPTU)

The addition of a seismic pick-up in a standard piezocone was first reported by Campanella and Robertson, 1984. The idea for this research was the outcome of a joint research project between Fugro Inc., Long Beach, California, and UBC, Vancouver in 1980. Fugro had developed the first working prototype. The addition of a seismic pick-up in the cone allows one to carry out a downhole seismic technique with a surface shear source during a pause in cone penetration. This simple procedure which will be described in this paper allows the direct determination of average shear wave velocity, V_s, over depth increments which are usually chosen as 0.5 m or 1 m. Using elastic theory one relates the maximum shear modulus, G_{max}, shear velocity, V_s, and total mass density, ρ, as

$$G_{\max} = \rho V_s^2 \tag{1}$$

Virtually all soils show nonlinear stress strain behaviour even at very low strains. Therefore, a knowledge of the stress strain behaviour is necessary in connection with many engineering geotechnical problems. The typical nonlinear stress strain relationship of soil can be represented by a hyperbolic curve with a knowledge of the maximum shear modulus and the shear strength. It is important to realize that the shear modulus decreases with increasing shear strain. The shear modulus is almost constant at shear strains less than 10^{-4} and is generally referred to as the dynamic shear modulus, G_{max}. The shear modulus is a fundamental soil property which relates shear deformation to shear loading. As the shear strength can be determined from the cone bearing, a stress-strain curve can be developed.

In general, the intensity of a seismic signal in soil decreases as distance from seismic source increases due to wave attenuation. This attenuation is due to geometric spreading and to energy dissipation within the soil mass caused by material damping. Attenuation is usually modelled as viscous damping, where the simulated damping force is assumed to be proportional to the velocity of the soil element. The resulting constant of proportionality is termed the coefficient of viscous damping. The damping ratio, D_s, is defined as the ratio of the coefficient of viscous damping to a critical value of the coefficient where the motion is attenuated within one wave cycle. The subscript, s, is used to indicate that the parameter is used for characterizing the behavior of the shear waves. The damping ratio is an important soil property which is required for dynamic analyses when simulating the unload-reload behavior of the soil subjected to transient loading conditions.

A schematic diagram of a typical seismic piezocone (SCPTU) is shown in Fig. 1. Independent strain gauged load cells measure bearing and friction sleeve forces and a pressure transducer measures the pore water pressure during penetration and dissipation. A slope sensor is used to indicate verticality and a temperature sensor to indicate temperature of load cells for thermal shift corrections.

The seismic receiver currently within the piezocone units at UBC consists of a piezo-resistive sensor, damped at 70 percent of critical damping (IC Sensors: 3021-002-N accelerometer), which has an essentially flat amplitude response from 0 Hz to about 300 Hz. Mechanical swing hammers are used at UBC as the source of seismic waves, although any appropriate and consistent source will work. However, in order to compare the intensity of signals at various depths in a soil profile, a source capable of generating repeatable signals must be used. The repeatability for the current work was ensured by selecting a single hammer weight and height of fall. Even with a

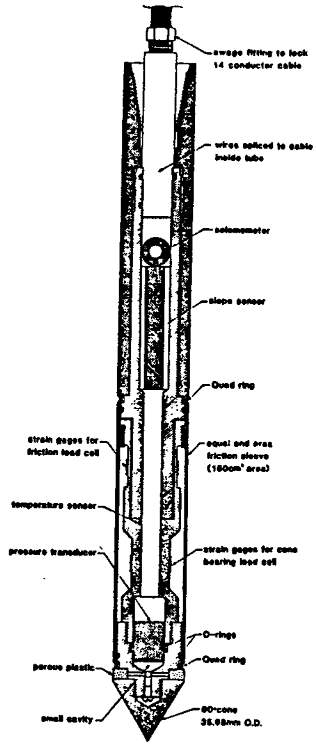


Fig. 1. Typical Seismic Piezocone (SCPTU)

consistent energy input, particularly in the case of soft soil sites, signals contaminated by vibrations from traffic or like sources are occasionally received. In order to eliminate the unwanted noise and maintain a quality signal, blows are repeated at each depth. After removing the spurious data, the rest of the seismic signals obtained from the repetition of blows at a given depth are averaged to increase the signal to noise ratio. For most of the current work carried out at UBC, it is typically convenient and deemed sufficient to record four signals at each depth.

While UBC's cone is primarily for research and development, Hogentogler and Co. manufacture a very similar SCPTU cone which also meets all the current ASTM specifications and measures bearing, friction, pore pressure (on face or just behind tip), and verticality with options which include seismometer and temperature. With either system, measurements can be taken either every 2.5 cm or 5 cm of penetration and all parameters are digitized at the surface with results printed and displayed live in the field. Both the UBC and Hogentogler systems also allows transferring data files to any form of personal computer for detailed analyses and the production of high quality plots in the field or the office.

STRATIGRAPHIC LOGGING AND GEOTECHNICAL PARAMETERS

Stratigraphic logging with the piezocone is one of its primary uses in site investigation work. Current practice using tip, friction sleeve and pore pressure information allow for very comprehensive logging with layer disremability in the order of a few centimeters. Stress normalization (e.g. Robertson, 1990, Jefferies and Davies, 1991) of measurements prior to logging and parameter evaluation is becoming more accepted and may become viewed as state-of-practice within the next few years for deep soil sites.

In addition to soil stratigraphy and type, it is also possible to use CPTU data to determine depth to groundwater table, equilibrium pore pressures, define layers of drained and undrained penetration and estimate the following geotechnical parameters for

- Drained Penetration (e.g. sands)
  - relative density, D_R
  - friction angle, ϕ
  - deformation moduli like M, E and G_{max}
- Undrained Penetration (e.g. clays)
  - undrained shear strength, s_u
  - sensitivity, S_t
  - stress history, OCR
  - deformation moduli like M, E_u, G_{max}
- Dissipation Pore Pressures
  - coefficient of consolidation, c_h
  - coefficient of permeability, k_h
- Other Correlations
  - liquefaction susceptibility
  - equivalent SPT N value.

Details concerning the procedures to estimate geotechnical parameters have been summarized by Campanella and Robertson (1992).

It is important to emphasize at this point that all the estimates of geotechnical parameters listed above are obtained through correlations with laboratory test results, large chamber test results, and other field in-situ tests. Soil mechanics theories are used to guide us in a qualitative way to identify the significant parameters or variables which might affect those correlations. Those variables include depth, pore pressure, stress history, lateral stress, compressibility, layer boundaries, partial drainage, compressibility and sensitivity among others.

SEISMIC TESTING FOR SHEAR WAVE VELOCITY

The typical arrangement utilized for conducting the SCPTU is shown in Figure 2. As noted above, the modified cone penetrometer with seismic data recording capability is also capable of providing the static soil properties and stratigraphic information. In contrast, conventional geophysical methods do not provide stratigraphic or *ground proofing* information.

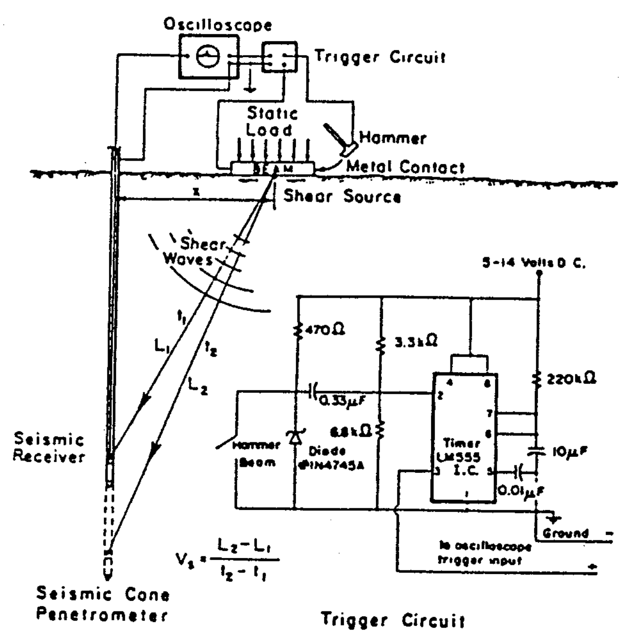


Fig. 2. A Typical Arrangement for Conducting SCPTU (after Campanella and Stewart, 1992)

For routine shear wave velocity measurements, detailed signal processing is not usually required and the velocities can be easily determined from arrival times in most cases. However, for damping measurements, it is necessary to evaluate the quality and nature of the signal in advance of the calculation. Such a signal processing also leads to an understanding of the properties of the measured signals. The signals obtained from the SCPTU can be systematically analyzed in five steps as follows:

- review the average of four hits at each depth obtained by repetitions of hammer blows;
- removal of noise on sites where signal contamination can be a problem. A low pass filter with flat response up to about 100 Hz has been found to be sufficient in most cases;
- select the main shear wave pulse from the signal;
- window the signal so that the windowed waveform contains all the characteristics of only the shear wave (a tacit assumption); and
- review Fourier transforms for each layer identified.

As shown schematically in Figure 2, the shear wave velocity, V_S, can be calculated by dividing the difference in travel distance between two depths by the time difference between the two recorded signals. The time difference can be found manually by picking the arrival time of the main shear wave pulse or by the *cross-over* technique (Campanella and Stewart, 1992). Alternatively, the time lag can be taken as the time shift of the maximum cross-correlation of the signals. The time can also be calculated as a function of frequency, f, using the phase of the cross spectrum of the signals. Dividing this time into the difference in travel distance gives the velocity as a function of frequency, $V_S(f)$. The windowed main shear wave pulse will be used as the input signal for the procedure outlined below.

1. The upper (shallow) and the lower (deeper) data sets are transformed to the frequency domain using a fast Fourier transform (FFT) algorithm in the same manner as Step 5 under Signal Processing.

2. The complex conjugate of the Fourier transform of the upper data set is then multiplied by the Fourier transform of the lower data set to give the cross spectrum of the signal.

3. The phase (in any angular units: degree or radian) of the cross spectrum can now be evaluated within a range of frequencies of practical importance for geotechnical materials (say, 0 to 250 Hz).

4. For each frequency, f, the time interval can be calculated using the following expression:

$$t(f) = \frac{phase\ in\ degree}{360^\circ\ x\ f} = \frac{phase\ in\ radians}{2\ \pi \times f} \qquad (2)$$

where

t(f) = time lag at frequency, f, corresponding to the calculated phase.

5. The shear wave velocity at a certain frequency, $V_s(f)$, is evaluated by dividing the difference in travel, distance of seismic signals, d_s, by the time lag, t(f). The frequency range over which the shear wave velocity remains relatively constant provides the frequency range for which the signals may be considered to be similar, and this velocity can be considered to be appropriate for the depth range between the upper and the lower signals.

Recent experience by a local Vancouver firm (ConeTec Investigations Ltd.) who use a Hogentogler Seismic Piezocone have demonstrated that high signal resolution is much less important than very high accuracy timing and trigger delay. Using a low cost 8 bit A/D with programmable gain amplifier they have been able to obtain very accurate and repeatable shear wave travel times to depths in excess of 50 m (165 ft).

COMPUTATION OF DAMPING BY THE SPECTRAL RATIO SLOPE (SRS) METHOD

Once the data has been processed, and phase velocities computed, the procedure for determining the small strain damping ratio is relatively straightforward. The basis of the methodology is the *S*pectral *R*atio *S*lope (SRS) which has been previously developed and used by others (Redpath et al., 1986; and Meissner and Theilen, 1986) for interpreting seismic waves. A full setp by step procedure for the SPTU is given in Campanella et al. (1994).

The damping ratio can be evaluated using the following expressions:

$$k = \frac{\partial^2\left(-\ln\frac{A_R}{A_0}\right)}{\partial f\ \partial d} \qquad (3)$$

and

$$D_s = \frac{k\,V_s}{2\pi} \qquad (4)$$

where

f	=	frequency, Hz,
d	=	depth, m
$\partial/(\partial d)$	≡	slope with respect to depth,
$\partial/(\partial f)$	≡	slope with respect to frequency,
A_0, A_R	=	amplitudes of the Fourier transforms of a reference signal and the signal at a depth where the damping ratio is to be evaluated,
V_s	=	the shear wave velocity of the layer,
A_R/A_0	=	Spectral Ratio,
D_s	=	the small strain damping ratio of the layer (decimal).

The parameter, k, in Equation (3) has a unit of (time/length). The derivation of these equations can be found in Stewart and Campanella (1993a). The essential feature of the double differentiation (first with respect to frequency and then with respect to depth) in Equation (3), is that the radiation damping component is eliminated from the expression. Thus, it is not necessary to assume or attempt to evaluate the amount of radiation damping at each depth. However, a variation of this approach can be used to study the distribution of radiation damping with depth in different soils. The procedure for implementing these expressions to evaluate the small strain damping ratio is given below.

1. Select a reference signal. This may be most conveniently defined as the signal at the shallowest depth within the layer of interest which is not affected by reflection and refraction from the surface.

2. Select the *best* frequency range for computation by inspecting the Fourier transforms of all the signals to be analyzed; the range of frequency for which all transforms exhibit a relatively similar response can be selected. Other methods such as the variation of phase velocities with frequency can also assist in the selection. Alternatively, 3 through 5 described below may be repeated with different frequency ranges with a goal to obtain the best overall statistics for the entire depth. The chosen frequency range is kept constant with depth for a particular soil layer.

3. At each successive depth, the Fourier transform of the windowed and filtered signal is divided by the Fourier transform of the windowed and filtered reference signal to evaluate the spectral ratio, A_R/A_0. The negative of the natural logarithm of this ratio is then calculated and plotted against frequency. The slope of this relationship, approximated as a straight line, is now evaluated within the frequency range selected in 2. The computation of $-\ln(A_R/A_0)$ is repeated for the same frequency range for all of the seismic signals from the soil layer of interest. Different frequency ranges can be used for different soil layers, which may be necessary where large differences in stiffness between adjacent soil layers exist (e.g. peat underlain by dense sand).

4. The slopes of the spectral ratios with respect to frequency (which are taken to approximate the first partial derivative of the spectral ratio with respect to frequency) are then plotted against depth. The slope of this plot can then be evaluated and is taken as an approximation of the second derivative of $-\ln(A_R/A_0)$, k (Equation 3).

5. The damping ratio for the layer can now be evaluated using Equation (3) from the knowledge of the phase velocity of shear waves evaluated earlier. As will be shown for a case history example, it is often necessary to divide the site to two or more layers due to stiffness variations as noted in 3.

CASE EXAMPLES OF SHEAR WAVE EVALUATION

McDonald's Farm Site, Vancouver, Canada

The data shown in the first example was obtained at the UBC research site at the Vancouver International Airport called McDonald's Farm. The cone penetration profile for this site, shown here in Figure 3, clearly indicates the upper sand layer to be relatively free of silt and the clayey silt layer to be free of sand interbedding. The water table is about 1 m below ground surface and the data in the soil profile column is the average of lab test data.

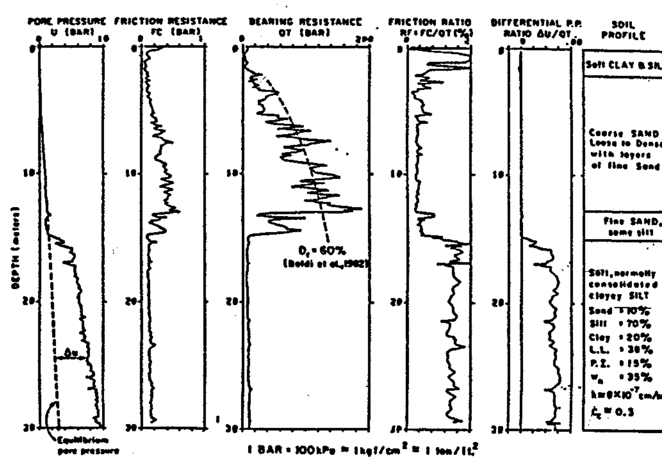


Fig. 3. Soil Profile for Research Site at McDonald's Farm Site, Vancouver

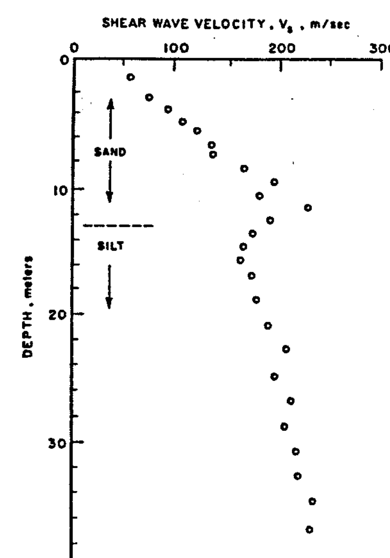


Fig. 4. Calculated Shear Wave Velocity Profile from Seismic CPT at McDonald's Farm Site, Vancouver

The shear wave velocities calculated from the difference of arrival times are shown in Figure 4. Note that the results in Figure 4 indicate that the interval shear wave velocity, and therefore maximum shear modulus, increases with depth.

Annacis Island, Vancouver

Extensive geotechnical investigations were carried out at the site of the new Annacis Bridge project near Vancouver. The area around the north main pier of the cable stayed bridge consists of Fraser River sands to a depth of about 40 m. The water table fluctuates with river level but is nominally about 4 meters below ground level.

A summary of the interval shear wave velocities and the cone bearing from the seismic CPT is shown on Figure 5. The CPT seismic downhole profile was carried out approximately 5 meters (16.4 ft) from a three hole array used for a conventional crosshole seismic survey, which was carried out by others for the B.C. Ministry of Transportation and Highways. The crosshole data was obtained at depth intervals of 2.5 meter and is also shown on Figure 5. The CPT downhole data was consistently slightly higher more than the crosshole data but generally the two sets of data compare within 20 percent. The seismic data generally follows the trend indicated by the cone bearing profile with little in the way of dramatic velocity changes.

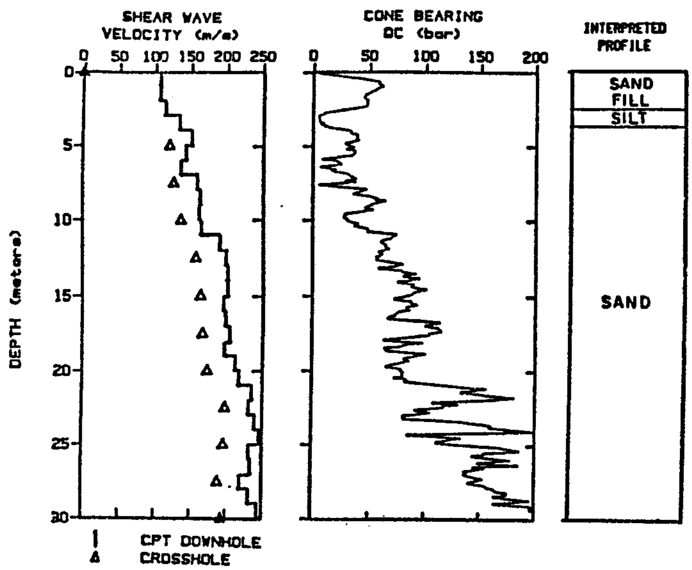


Fig. 5. Comparison of Seismic CPT Downhole and Crosshole Data at Annacis Site, Vancouver

Imperial Valley, California, U.S.A.

In the spring of 1984, seismic CPT tests were performed at several sites in the Imperial Valley, California with the cooperation of the U.S. Geological Survey and Purdue University. These sites were subjected to significant earthquakes in 1979 and 1981. Figure 6 presents the SCPT data from the Wildlife site. Full details of the site are given by Bennett et al (1984). The Wildlife site is located next to the Alamo River and exhibited extensive liquefaction during the 1981 earthquake. Also included in Figure 6 are the shear wave velocities determined by crosshole tests (Nazarian and Stokoe, 1984). The two seismic profiles compare very favourably with velocities from the two independent methods generally within about 20 percent. It is worth noting how the shear wave velocities from the SCPT respond to variations in the soil profile.

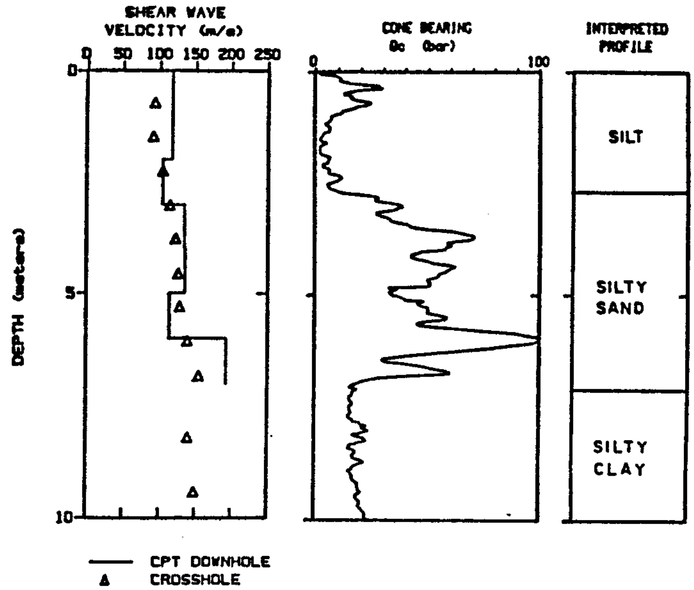


Fig. 6. Comparison of Seismic CPT Downhole and Crosshole Data at Wildlife Site, Imperial Valley

Holmen and Museumsparken Sites, Drammen, Norway

In the fall of 1984 seismic CPT tests were performed at several sites in Norway by Dr. Don Gillespie (former UBC Ph.D. student) with the cooperation of the Norwegian Geotechnical Institute (NGI) (Eidsmoen et al, 1984). These sites are well documented with extensive field and laboratory data.

The Holmen site consists of loose, medium to coarse sand to a depth of 25 m. Figure 7 shows the seismic CPT data compared to the adjacent crosshole data. On average the two seismic shear-wave velocity profiles are almost identical at this site showing little, if any, discrepancy between the CPT downhole and conventional crosshole. The seismic CPT data responds well to variations in the soil profile observed from the cone bearing.

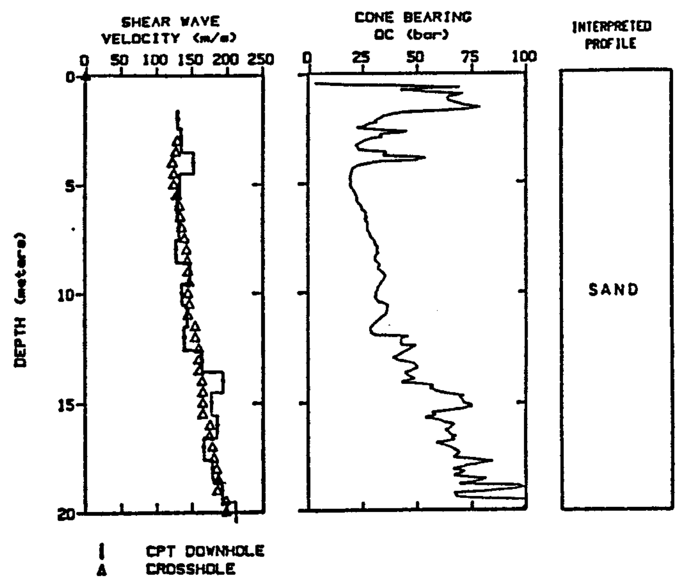


Fig. 7. Comparison of Seismic CPT Downhole and Crosshole Data at Holmen Site, Norway

The Museumsparken site consists of the well documented Drammen clay to a depth of 15 m. Figure 8 shows the seismic CPT data compared to adjacent crosshole data. Again the two seismic profiles compare very favourably.

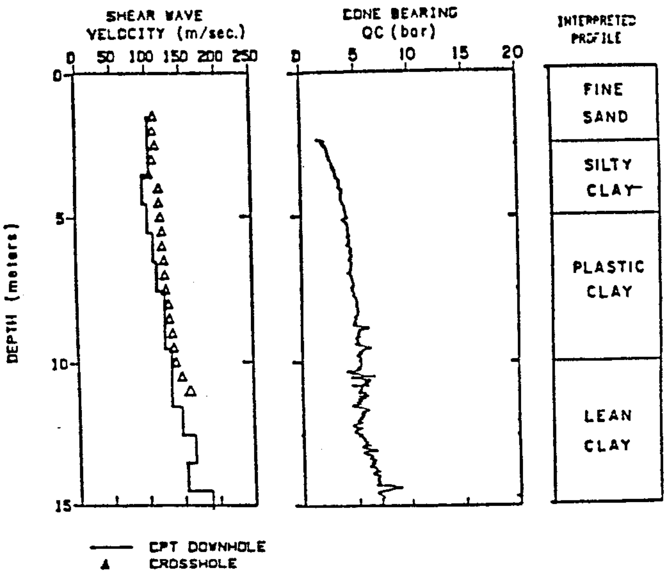
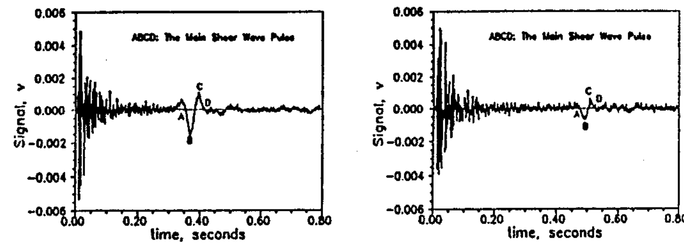
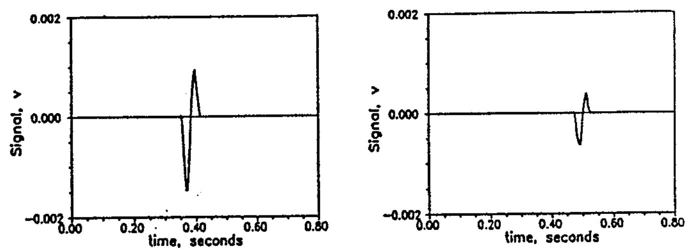


Fig. 8. Comparison of Seismic CPT Downhole and Crosshole Data at Museumsparken Site, Drammen, Norway



(a) at 9.9 m depth (b) at 14.4 m depth

Fig. 9. Averaged Time History of Seismic Signals at Two Depths



(a) at 9.9 m depth (b) at 14.4 m depth

Fig. 10. Clean Shear Wave Pulse for the Data Used for Constructing Figure 9

CASE EXAMPLE OF SOIL DAMPING EVALUATION

Kensington Site, Burnaby

The procedure briefly outlined for computing soil damping was used to compute the low strain damping ratio for a site consisting primarily of organic soils at a location in Burnaby, British Columbia. The site at which the SCPTU investigation was carried out lies at the intersection of the Trans Canada Highway and Kensington Avenue. The site can be broadly characterized by two layers to roughly 18 m depth. Beneath a surficial fill lies an organic-rich material. This organic layer is predominantly peat to a depth of about 5 m, becoming an organic-rich silt to silty clay as the mineral content of the layer increases. The organic materials show a fibrous to amorphous trend with depth. The organic-rich layer is underlain by a layer of soft sensitive clay to silty clay which is extremely uniform. The average undrained shear strength, for the top layer of organic soil is lower than 20 kPa while that for the bottom clay layer is lower than 15 kPa.

The method for evaluation of the low strain damping ratio described earlier has been automated with a digital signal processing program. These automated procedures were then utilized to evaluate the shear wave characteristics and small strain damping ratio for the soils between depths 2.5 m and 15.5 m. Figure 9a and Figure 9b illustrate two seismic signals from the data set. The windowed signals were then subjected to a low pass filter using a filter characterized by a very flat response between 0 Hz and 90 Hz and a steep decay in the region between 90 and 110 Hz (Figure 10a and Figure 10b).

A frequency range between 5 and 35 Hz was used for the computation of shear wave velocities. The choice of this range was made after examining the apparent relationship between the shear wave velocity, V_s, and frequency, f. The results of the phase velocity computation for the main shear wave pulse is shown in Figure 11. The univariate statistics shown pertain to the variation of the phase velocity against frequency within the selected frequency range.

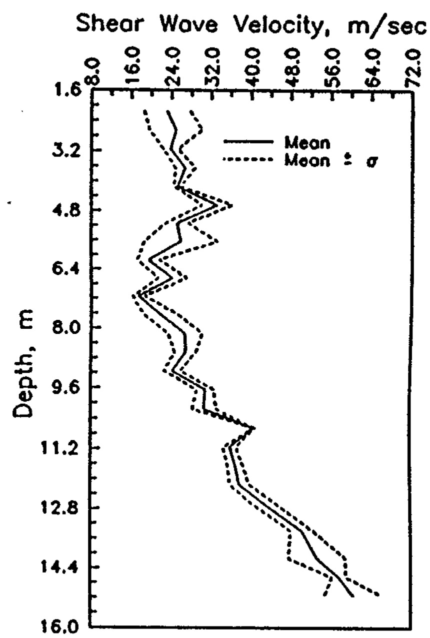


Fig. 11. Shear Wave Velocities at the Kensington Site

For the computation of the small strain damping ratio for the upper organic and the lower clay layer, the seismic signals at 1.9 m and 6.4 m depth were chosen as reference. As noted earlier, the soil profile of the site consists of two main soil type layers; between depths 1.9 m and 6.4 m, and between 10.0 m and of 15.0 m. A transition layer between 6.4m and 10.0 m depths is also apparent. Therefore, it was decided to fit the $-\ln(A_R/A_0)$ versus depth data from three soil units using three linear relationships. The slopes with respect to depth for Equation (3) are shown in Figure 12.

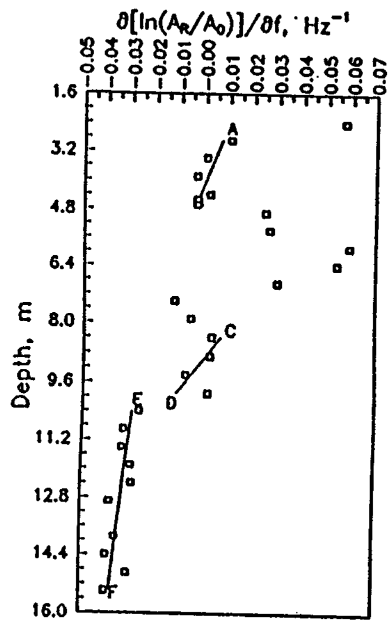


Fig. 12 $\partial[\ln(A_R/R_0)]/\partial f$ Plotted Against Depth for Kensington Site

The shear wave velocity and the appropriate value of k of the layer were then substituted into Equation (4) to calculate the small strain damping ratio. Using average values of shear wave velocity of 25 m/sec and 46 m/s for data from depths above and below 10 m, the small strain damping ratios for the three layers are:

- 2.8 m to 4.4 m = 2%
- 8.4 m to 9.9 m = 5%
- 10 m to 16 m = 1%.

The computed small strain damping ratios are well within the range of reported values for cohesive soils (Stewart and Campanella, 1993b).

SUMMARY

The piezocone is an excellent site characterization tool that is particularly superior at identifying stratigraphic detail. Interpretation procedures have progressed to the point where an extensive range of geotechnical parameters can be estimated for soil layers penetrated by the piezocone and design procedures can be applied to many foundation engineering problems. Interpretation procedures are largely empirical and local correlations are recommended for accuracy and confidence. The interactive computer program CPTINT has been developed to aid with the interpretation of piezocone data.

With the seismic piezocone downhole, seismic shear wave velocity measurements can be made during brief pauses in the cone penetration while pore pressure dissipation data is recorded. The shear wave velocity data can be used to provide a reliable determination of shear wave velocity and the maximum dynamic shear modulus. In addition, soil damping can be estimated by systematic signal processing. Accurate depth determination is assured with modern cone testing and seismometer orientation is easily maintained throughout the sounding. Hole verticality is monitored throughout the sounding with a small slope sensor installed in the cone. The combination of the seismic downhole method and piezocone logging provide an extremely rapid, reliable and economic means of determining stratigraphy, strength, consolidation, modulus and damping information in one sounding.

Comparison of the seismic CPT downhole shear wave velocity measurements with those obtained by conventional crosshole techniques show excellent agreement. The seismic CPT is, however, considerably less expensive and a more rapid procedure than the crosshole technique as well as providing comprehensive stratigraphic and geotechnical information.

ACKNOWLEDGEMENTS

Numerous graduate students (especially Don Gillespie) and the technical staff (especially Scott Jackson) of the Civil Engineering Department, University of British Columbia contributed significantly to this document. The assistance of the Natural Sciences and Engineering Research Council Canada; the British Columbia Ministry of Highways and Transportation; Jaap de Ruiter, Fugro, Holland; T. Lunne and T. Eidsmoen of the Norwegian Geotechnical Institute; L. Youd of the U.S. Geological Survey; and J.L. Charmeau, Purdue University, is also appreciated.

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Due to length restrictions, a full annotated reference list is not presented herein. However, the authors would be pleased to provide elaboration on the items in this paper or within the reference list upon written request.

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