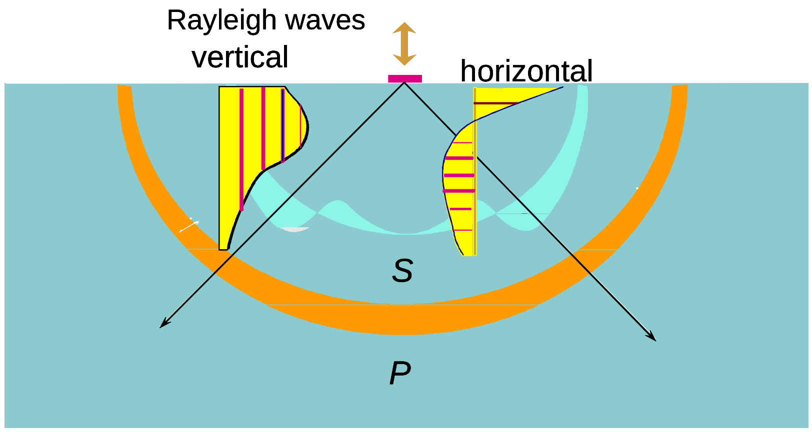

Wave propagation and microstrain behaviour of soils

Research Report

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2007

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SCOPE

This research report aims at a review of wave propagation in soils, measurements methods, wave velocities and resulting shear modulus. It covers a brief theoretical introduction, followed by a review of intrusive and non-intrusive methods for wave velocity measurements. Among those, the seismic piezocone is widely used and the author's work is discussed in more detail.

Finally this report addresses soil stiffness non-linearity as a function of strain amplitude and shows results of a broad research programme carried out by the author on saprolitic soils in Hong Kong and Rio de Janeiro.

An appendix presents the detailed procedure for seismic piezocone tests.

THE MEASUREMENT OF STIFFNESS USING SEISMIC METHODS

Seismic methods utilise propagation of elastic waves through the ground. There are two categories of seismic waves: body waves, comprising compression *(P)* and shear (S) waves, and surface waves, which include Rayleigh (*R*) waves. The modes of propagation of these wave types are well known and are described in most texts on seismic methods (*e.g.* Telford et al 1990). The waves propagate at velocities which are a function of the density and elastic properties of the ground.

Figure 2 presents an experiment to observe the phenomenon of wave propagation through an accelerometer firmly installed into the ground. The accelerometer is hooked to an oscilloscope. Some metres away from the accelerometer one hits the ground with a heavy hammer impacting on steel plate. The impact generates body waves and displacements *u* and *w* in the directions *x* and *z* respectively. The oscilloscope (Figure 2) shows the resulting signals with three peaks corresponding to compression, shear and surface waves, which propagates at different velocities.

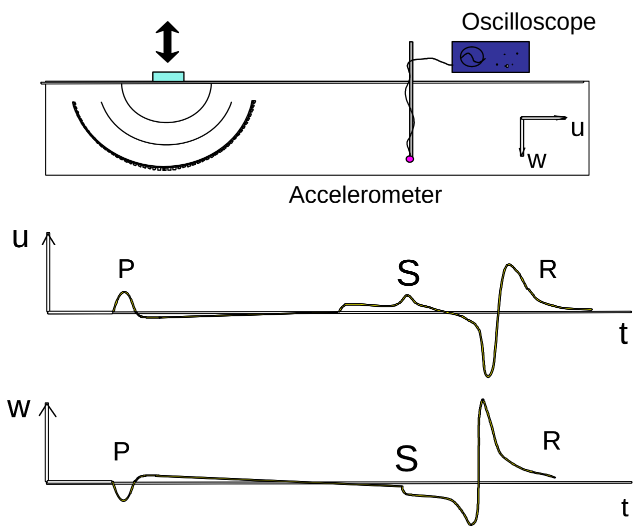


Figure 1 Measurement of *ground wave propagation*

Table 1 Range of velocities values of S and P waves

Material	V_p	V_s
	m/s	m/s
Clay	1500	150
Sand	480	250
Gravel	750	180

Table 1 shows the range of wave propagation velocities for *P* and *S* waves for soils with different stiffness. The *P* or compression waves propagate at much higher velocities, followed by *S* and *R* waves. Figure 2 presents their shapes. Rayleigh waves travel close to the surface at velocities 90% of the *S* wave. This relationship will be addresses later in this report.

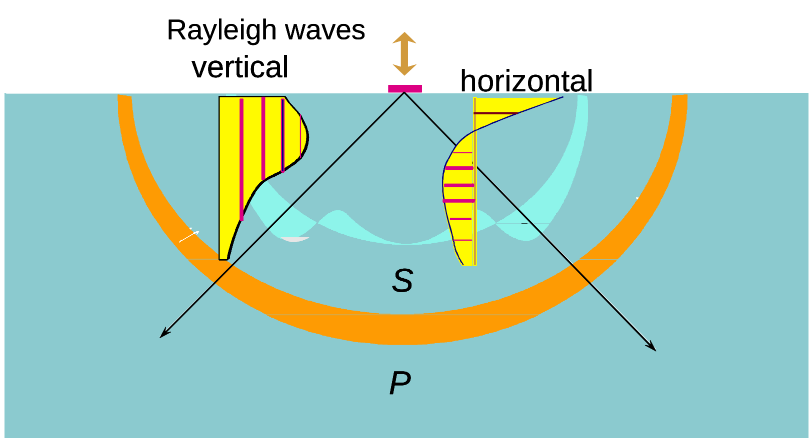


Figure 2 Wave propagation in the ground

In an isotropic elastic medium, the velocity of a compression wave, V_p, is given by:

$$V_p = \left[\left(\frac{K + 4/3G}{\rho}\right)\right]^{1/2}$$

and the velocity of a shear wave, V_s, is:

$$V_s = \sqrt{G / \rho}$$

where *K* is the bulk modulus, *G* the shear modulus and ρ the density. According to the theory of elasticity, the Young's modulus, *E,* is related to *G* and *K,* thus:

$$K = \frac{E}{3(1-2\nu)}$$

and

$$G = \frac{E}{2(1+\nu)}$$

where ν is Poisson's ratio. Therefore, *G* can be obtained from measurements of V_s alone, but V_s and V_p are needed to determine *E, K* and ν.

It should be pointed out that in saturated uncemented soils the propagation of *P*-waves will represent a short term undrained loading. In such a case, the compressibility of the pore water will tend to dominate the compressibility behaviour of the soil. The result is that the measured *P*-wave velocity is likely to be close to that of water (*i.e.,* 1500m/s) since most of the energy will travel through the pore water and will not reflect the true undrained stiffness E_u of the soil.

As the degree of cementation increases, the rigidity of the mineral skeleton increases such that first arrival *P*-waves become more representative of the material. In saturated rock the elastic modulus *E* measured from the *P*-wave velocity will be representative of the stiffness of the mineral skeleton. Thus, for stiffness measurements in soils, only shear-wave velocities should be used, since these are not affected by the compressibility of the pore fluid.

RAYLEIGH WAVES

Surface waves may also be used to determine shear stiffness in soils and rocks.

Examples of surface waves are Rayleigh, Love, and Stoneley waves, but only the first two types are commonly used in site-characterisation measurements. Rayleigh waves are vertically polarised shear waves. Love waves are horizontally polarized. If a small-amplitude body wave propagates in an elastic, isotropic, and homogeneous medium, its phase velocity depends solely on the elastic properties of the medium and is unaffected by the presence of boundaries.

Approximately two thirds of the energy from an impact source propagates away in the form of surface waves of the type first described by Rayleigh in 1885.

Exploration geophysicists have traditionally regarded Rayleigh waves, or 'ground roll', as a nuisance. However, Rayleigh waves travel at speeds governed by the stiffness-depth profile of the near-surface material. Geotechnical engineers have long recognised that Rayleigh waves offer a useful non-invasive method of investigating the ground in situ (*e.g*. Hertwig 1931; Jones 1958; Heukolom & Foster 1962; Abbiss 1981). It can be shown from the theory of elasticity that the relationship between the characteristic velocity of shear waves V_s and Rayleigh waves V_r in an elastic medium is given by:

$$V_r = CV_s$$

The constant *C* is dependent on Poisson's ratio and may be found from the expression:

$$C^6 - 8C^4 + 8\left(3 - \frac{1-2\nu}{1-\nu}\right)C^2 - 16\left(1 - \frac{1-2\nu}{2(1-\nu)}\right) = 0$$

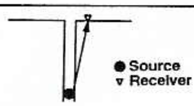
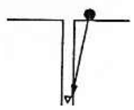
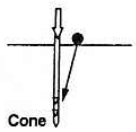
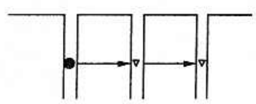
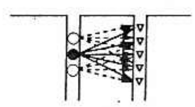
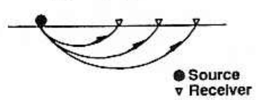
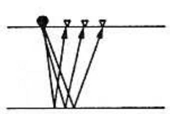
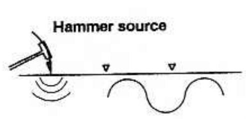
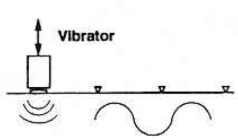
The range of *C* is from 0.911 to 0.955 for the range of Poisson's ratio associated with most soils and rocks if anisotropy is ignored. The maximum error in *G* arising from an erroneous value of *C* is less than 10%.

MEASUREMENT OF WAVE VELOCITY PROFILES

Geotechnical engineers generally require stiffness measurements to be made at different depths in order to determine a stiffness-depth profile. Using direct methods of investigation, this can be achieved by taking samples at different depths and subjecting them to appropriate laboratory tests or by conducting in situ loading tests (*e.g.* pressuremeter or plate loading tests). Stiffness is frequently determined indirectly using the Standard Penetration Test (SPT). This appears to be attractive, since the SPT is carried out routinely as part of most site investigations. However, stiffness parameters are determined using empirical relationships (e.g. Wakeling 1970; Kee & Clapham 1971; Stroud 1988, Lee, 1992)) many of which have very limited accuracy. Field seismic techniques allow stiffness to be determined on representative volumes of ground, and at the in situ stress state, and for this reason may provide valuable data which are unaffected by borehole sampling disturbance, or by penetration effects. In many cases the cost per stiffness measurement may be less using geophysical methods than for the direct methods outlined above.

The seismic methods employed to determine stiffness-depth profiles are described in Table 2 and can be classified as intrusive and nonintrusive methods.

Table 2 Seismic methods used for *determination* of *stiffness-depth profiles (Mathews et al, 1997)*

Method	Diagram	Advantages	Disadvantages
Up-hole		Only single borehole required. Tests can be carried out in all soil and rock types. Average velocity is measured in layered materials.	Need to install plastic casing to provide stable borehole.
Down-hole		Only single borehole required. Tests can be carried out in all soil and rock types. Average velocity is measured in layered materials. Higher energy sources (eg explosives) can be used without damaging the borehole.	Need to install plastic casing to provide stable borehole.
Seismic cone		No borehole required; probe is pushed into the ground. Provides other geotechnical parameters in addition to stiffness Average velocity is measured in layered materials.	Penetration limited by strength of ground. Not suitable for rock.
Cross-hole		Can detect low velocity (ie low stiffness) layers, provided they are thick compared to borehole spacing. Tests can be carried out in all soil and rock types.	Quality of data diminishes at shallow depths. Maximum velocity is emphasized in thinly layered soils due to head waves.
Cross-hole tomography		Gives two-dimensional distribution of stiffness. Tests can be carried out in all soil and rock types.	Expensive Artefacts make interpretation difficult. Specialist processing facilities required.
Refraction		No borehole required.	Cannot detect low velocity (low stiffness) layers below higher velocity layers. Cannot detect thin layers. Problems with interpretation of continuous velocity increase with depth. Cannot use in situations of continuous velocity decrease with depth, although such cases are rare.
Reflection		No borehole required.	Expensive; high-resolution seismic reflection is required for engineering surveys. Method only effective in layered ground.
Spectral Analysis of Surface Waves (SASW) method		No borehole required. Field method is quick and relatively simple.	No selective control over the frequencies generated; therefore measurements are limited to those frequencies which can be generated in the medium by a given impulsive seismic source. It may be necessary to use a number of different impulsive energy sources.
Continuous surface-wave (CSW) method		No borehole required. Selective frequency control of vibratory seismic source. Field method is relatively quick and simple. Preliminary stiffness-depth profile may be viewed on site.	Depth of investigation is currently limited to about 10m unless large lorry mounted vibrators are employed.

Intrusive methods

Intrusive methods require physical penetration of the ground or borehole drilling to determine compression-wave (V_p) and shear-wave (V_s) velocities directly. Tests included in this category are as follows: *uphole* (Meisner 1965), *downhole* (Schwarz and Musser 1972), *crosshole* (Stokoe and Woods 1972), *bottomhole* (Stokoe et al. 1978), *inhole* (Ogura 1979), and *seismic cone penetration* (Campanella and Robertson 1984).

Downhole

In the downhole method, a source located at the at the ground level emits seismic waves that are received at various depths along the borehole. The depth and travel-time are measured for various positions of the source to determine near-vertical velocity profiles. The downhole and uphole methods are similar except that the positions of source and receiver are switched.

This method is suitable for hard residual soils and rocks where drilling is needed. The downhole method requires just one borehole.

Cross-hole

The crosshole test requires a minimum of two boreholes for a source and one or more receivers. At a depth where velocity measurements are required, the source and receivers are aligned in their respective boreholes at the same elevation. Travel time and the distance between receiver boreholes are used to determine the appropriate body-wave velocity. Wherever feasible, the cross-hole test is probably the preferred choice among the intrusive methods, as it is considered the most accurate (Larsson and Mulabdic 1991). Unfortunately, the crosshole test is limited by high costs.

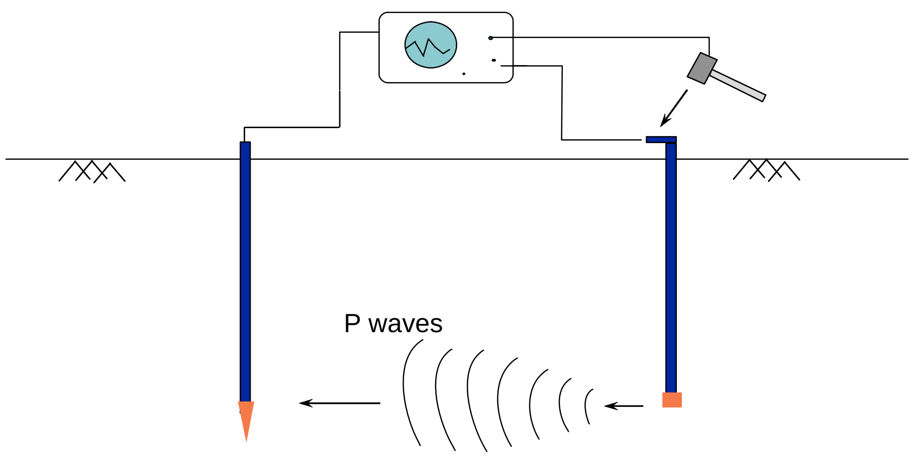


Figure 3 Cross-hole tests

CPTS Seismic piezocone

In the seismic cone penetration test (CPTS), another variation of the downhole test, a receiver is incorporated in a conventional electric cone that is pushed into the ground. At desired depths, usually at every metre, a surface source emits shear waves that are picked up by the receiver in the cone. Velocity computation is the same as in the downhole test. The method is restricted to relatively soft soils, and existing results have compared favourably with those obtained from the crosshole method (Gillespie 1990).

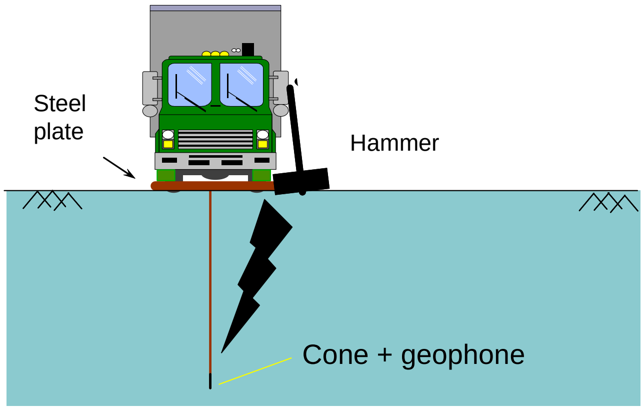


Figure 4 CPTS Seismic piezocone test arrangement

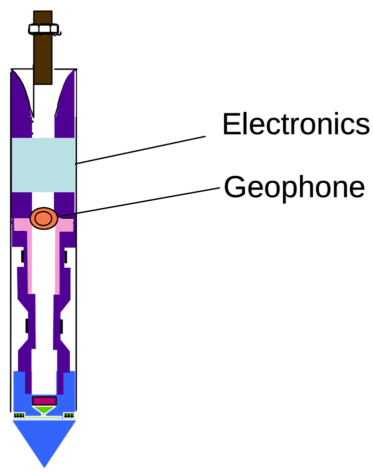


Figure 5 Seismic piezocone

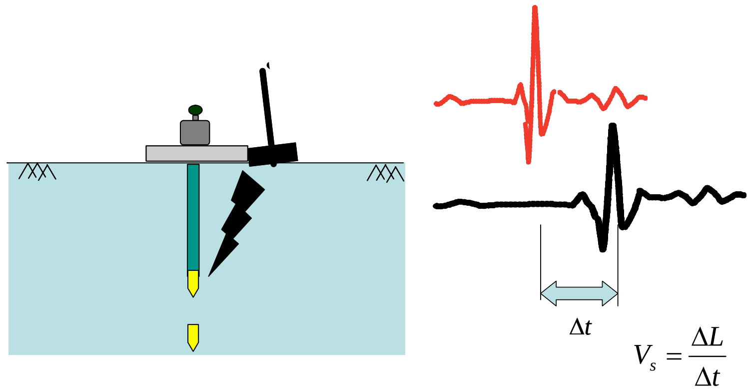


Figure 6 Shear wave velocity calculation

Nonintrusive methods

In this section, nonintrusive methods based on surface-wave dispersion are reviewed. Such methods include steady-state vibration and the spectral analysis of surface waves and involve the determination of layering and body-wave velocity profiles from measured in situ dispersion curves.

Nonintrusive methods are used to determine velocity profiles from the ground surface. These include surface reflection (Borm 1977), surface refraction (Redpath 1973), steady-state vibration (Jones 1953; Ballard 1964), and the spectral analysis of surface waves (Nazarian and Stokoe 1984). Surface reflection and refraction are generally more suited to geophysical exploration work and do not generally provide accurate profiles of body-wave velocities for geotechnical engineering purposes. To a varying extent, the last two nonintrusive methods use surface wave dispersion to indirectly determine compression- and shear-wave velocities.

Surface-wave methods exploit the dispersive nature of Rayleigh waves: the speed of propagation of a Rayleigh wave travelling at the surface of inhomogeneous ground depends on its wavelength (or frequency) as well as the material properties of the ground Measurements of phase velocity of Rayleigh waves of different frequencies (or wavelengths) can be used to determine a velocity - depth profile. Two distinct surface-wave methods are available:

- *Spectral analysis of surface waves* (SASW) method. This method makes use of a hammer as an energy source. The field technique is described by Ballard & McLean (1975), Nazarian & Stokoe (1984) and Addo & Robertson (1992).
- *Continuous surface-wave* (CSW) method. This method makes use of a vibrator as an energy source. The field technique is described by Ballard & McLean (1975), Abbiss (1981), Tokimatsu *et al.* (1991) and Matthews (1993).

The velocities determined over a range of frequencies will form a characteristic dispersion curve for the ground under investigation. This can be inverted using a variety of different methods to give a velocity-depth profile from which the stiffness-depth profile can be determined.

Steady-state vibration test

The steady-state vibration method was probably first used for civil-engineering purposes by Jones (1953). The procedure currently involves the generation of a sinusoidal surface wave of a known frequency, the wavelength of which is determined by progressively varying the relative positions of two seismic receivers until the signals detected by them are in phase. In the original procedure, a single receiver was progressively moved away from a vertically vibrating source, and distances to successive in-phase positions were measured and averaged. Based on the findings of Miller and Pursey (1955), approximately 67% of the energy of a vertically vibrating source on the surface of a half-space propagates as Rayleigh waves. For an excitation of known frequency (*f*) and measured average wavelength (*L*), the phase velocity (*c*) was computed from the relation:

$$c = fL$$

The procedure was repeated for different frequencies to obtain a relationship between phase velocity and wavelength, which yields an in situ Rayleigh-wave dispersion curve. The interpretation or inversion scheme in the steady-state method is based on two approximate relationships. The first is based on the theoretical observation that, within the normal range of Poisson's ratio ($0 < \nu < 0.5$), the average ratio of Rayleigh-wave to body shear-wave velocities (kR) is about 0.91. Shear-wave velocities are therefore obtained by dividing Rayleigh-wave velocities by 0.91, i.e.,

$$V_s = \frac{V_r}{0.91}$$

The second relationship is based on the observation (e.g., Das 1983) that the displacement components of a Rayleigh wave are only significant within a depth of about one wavelength from the surface. The depth to the centred of this zone of significant displacement is thus a fraction of the wavelength. This observation gives some theoretical support to the assumption that a Rayleigh wave of wavelength *L* propagates at an average depth, z, given by

$$z = k_z L$$

where k_z is a numeric constant. Different values of k_z have been used, e.g., 0.33 (Heisey 1981), 0.5 (Heukelom and Foster 1962; Abbiss 1983), and 1.0 (Ballard 1964).

The major advantage of this technique is its analytical simplicity because the shear-wave velocity profile is obtained directly from the field dispersion curve. It also has low initial cost and requires neither time measurement nor signal analysis. The field procedure is, however, time-consuming and impractical for investigating relatively deep-seated deposits with monochromatic signals, since receiver spacing become inconveniently large. The weakness of the method lies in its neglect of the dispersive characteristics of surface waves in the inversion process, a limitation that the SASW approach has overcome.

Introduction

In early 80's Campanella and co-workers (Campanella et al, 1986, 1987, 1989, 1994, Campanella and Stewart, 1992) introduced seismic piezocone tests (CPTUS or CPTS). The main output of this test is the shear wave velocity and the small strain shear modulus G_{max}. CPTS is relatively simple to be carried out in the field and at a marginal additional effort relatively to the piezocone test. The results lead to a direct measure of soil modulus, not a correlation as in many other tests, at a small fraction of the costs of other in situ or laboratory test that give equivalent soil parameters.

At early stages of CPTS test development, the analysis was carried out employing very simple graphical techniques. This was the case of the early stages of this research programme for the tests conducted at a research site in Rio de Janeiro soft clay. Although simple techniques lead to satisfactory results with clear and noiseless signals, the simple method is difficult to apply in a noisy environment.

Therefore, efforts were made to enhance the analysis procedure. This paper describes an automated method of analysis and the results obtained at a few sites in Brazil.

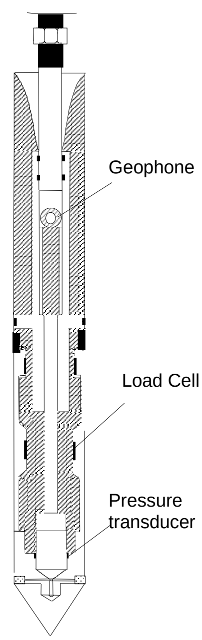


Figure 7 Seismic piezocone

Test method

Seismic piezocone tests are aimed at giving the shear modulus at small (micro) strains. The equipment used (Figure 7) consists of a standard piezocone, 10 cm^2 of cross-sectional area and 60^o of apex angle, in which a small device at the top is capable of detecting the arrival of shear waves. This device can consist of an accelerometer or a geophone (Campanella and Stewart 1992). This paper, however, concentrates on the use of the latter device. The cone includes an on board electronic module capable of amplifying and conditioning signals generated by the transducers. A PC notebook data acquisition system operates in penetration mode and seismic mode. In the first case it measures piezocone standard data and in the seismic mode it receives, digitises and stores the geophone signals at 12 bit resolution and at a highly conservative rate, typically 5 kSamples/s.

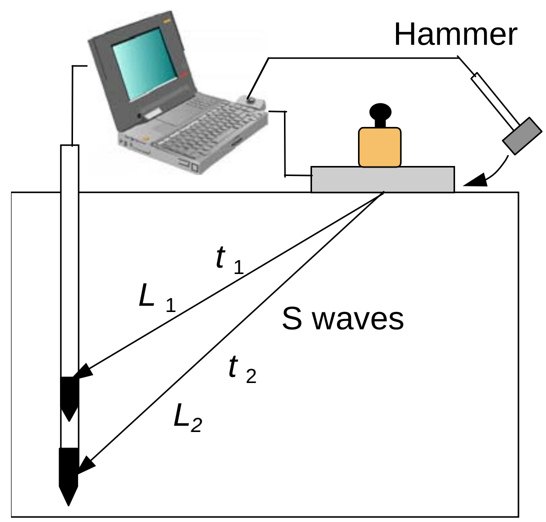


Figure 8 Seismic test

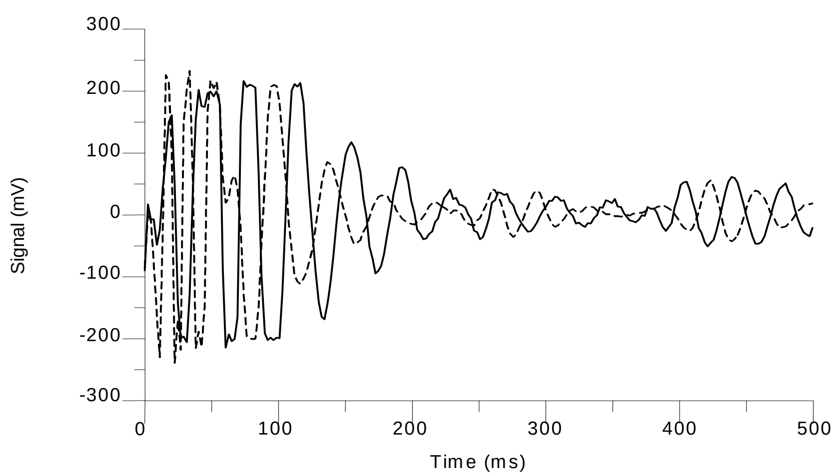


Figure 9 First series of CPTUS data

The test is carried out as shown in Figure 8. Shear or *S* waves are generated at the soil surface by striking a steel plate with a hammer. The steel plate is securely fixed on the soil surface by a heavy weight, which can be the wheel of a vehicle or the site investigation lorry. The hammer is electric connected through a wire to a triggering circuit in the data acquisition system.

The seismic test is normally carried out during cone penetration pauses to add new rod lengths. As the cone is at depth z_i, the strike plate is hit by the hammer and the triggering circuit starts the data acquisition. A signal versus time is obtained at each test depth z_i and corresponds to the arrival time t_i of the shear wave. The shear wave velocity V_s between depths z_i and z_{i+1} is given by:

$$V_s = \frac{z_{i+1} - z_i}{t_{i+1} - t_i} = \frac{\Delta z}{\Delta t}$$

The shear modulus G_{max} is then computed by:

$$G_{max} = \frac{\gamma}{g} \cdot V_s^2$$

where γ is the soil unit weight g is the acceleration of gravity.

Small strain G_{max} values can be corrected to the macro-strain domain, which corresponds to most geotechnical engineering applications (*e.g.,* Ortigao et al, 1997), by the use of a single laboratory test or theoretical degradation curves.

Tests at Rio de Janeiro soft clay test site

The initial stage of testing was aimed at to set up the equipment and to obtain the first results in a soft Rio de Janeiro clay, close to Sarapui River, which properties have been studied since the late seventies (Sayão, 1980, Ortigão et al, 1983, Ortigão, 1995). The details of the equipment and how it was set up were described in detail by Francisco (1996).

Typical signals acquired by striking the plate with a hammer are shown in Figure 8. The time gap between these signals was manually obtained by plotting the signals and graphically determining the time gap between them. This process, however, is very tedious and sometimes can be in error due to noise. The final results of this test series are presented in Figure 10, showing considerable scatter and no information for error checking.

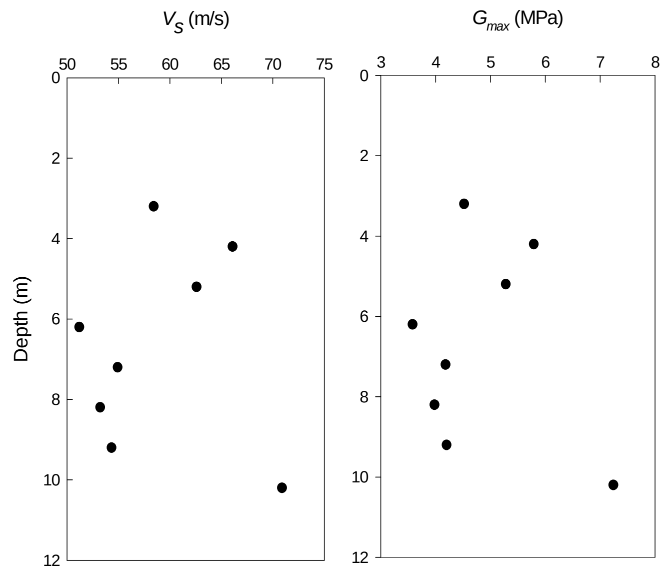


Figure 10 Results of CPTUS tests in Rio de Janeiro soft clay (Sarapui test site)

Second phase of the research programme

The second phase of the research programme was carried out to:

- Improve the quality of the data acquisition system
- Checking the repeatability of signals
- Automate the analysis.

A series of CPTUS boreholes were carried out at a soft clay site near the northern City of Sao Luiz, Maranhão. Typical interpreted piezocone profile is presented in Figure 11. The plots present the tip resistance q_t, the friction ratio R_f and the porepressure u and the interpreted stratigraphy. It consists of a 3 m thick layer of sand fill followed by soft clay.

Seismic tests were carried out at regular intervals in the soft clay. At regular depth intervals of one meter the plate was hit five times by the hammer and the signals were recorded. The analyses consisted in the application of signal analysis techniques and the results are discussed in the following text.

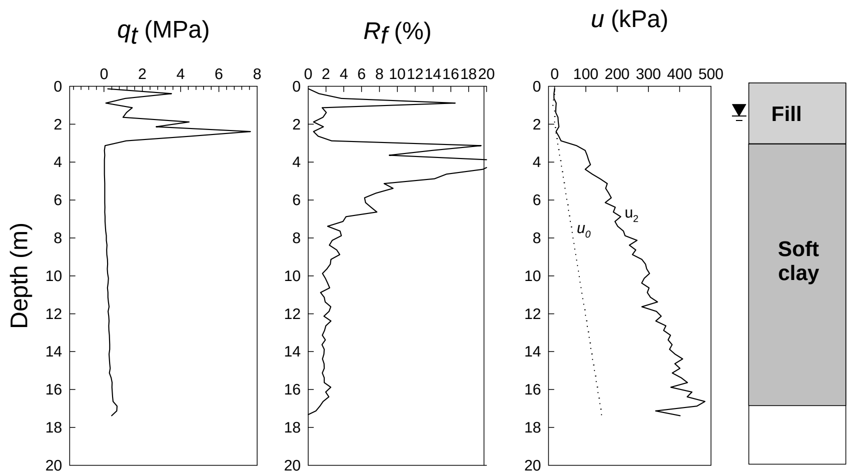


Figure 11 CPTU interpreted profile, São Luiz, Maranhão

Visual inspection

The first stage in signal processing is a check on signal repeatability. This is carried out by visual analysis of all signals at a time in the time and frequency domain. The frequency domain is obtained through Fast Fourier Transform (FFT) analysis. The computer program used by the authors displays both graphs on the PC screen (Figure 12). The user can decide to keep all signals at a certain depth, or to delete an undesired signal.

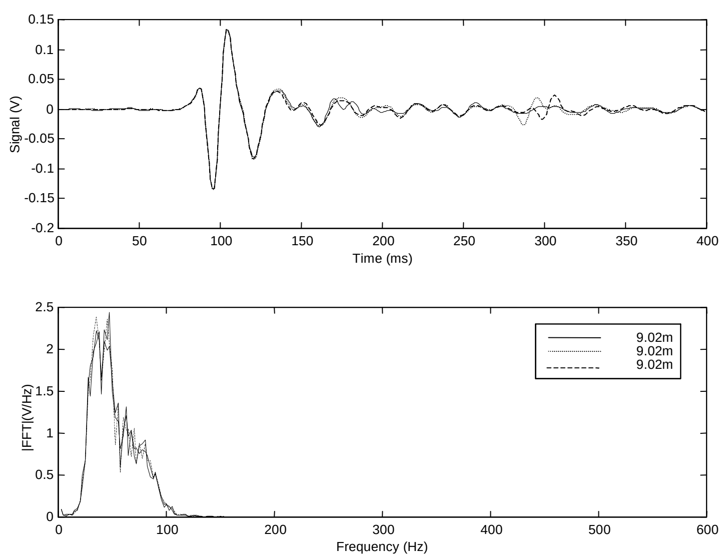


Figure 12 Repeatability of signals in time and frequency domain

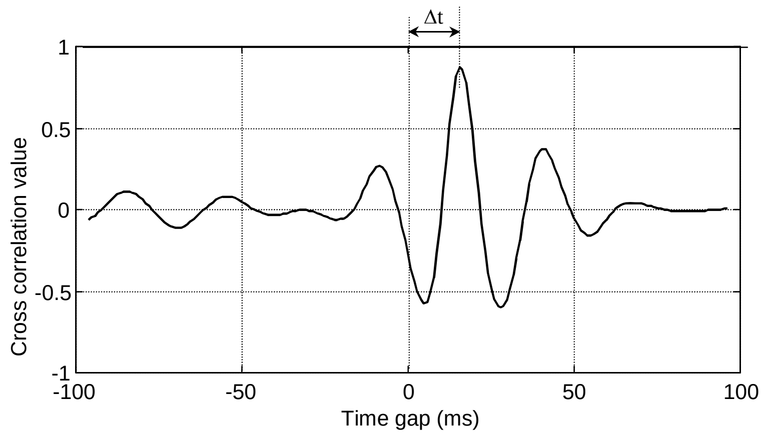


Figure 13 Typical cross correlation function of two seismic signals

Analysis

Velocity calculations requires the determination of the time gap Δt between two signals at different depths

If the signals are well behaved, graphical analysis and determination Δt is easy. However this is seldom the case. All sorts of unavoidable ambient noise interfere and even the most sophisticated filter technique is not able to make this process easy.

Therefore, a considerable effort was made to automate the process of signal comparison by using a mathematical procedure called *cross-correlation*. It has been used by (Campanella and Stewart, 1994). The cross-correlation ($R_{jk}(\tau)$) of two signals is given by:

$$R_{jk}(\tau) = \int f_j(t) f_k(t+\tau) dt$$

where $f_j(t)$ is the signal corresponding to the first signal (j^{th} signal) and $f_k(t+\tau)$ is the second (k^{th} signal) which has been shifted in time by an amount of τ milliseconds.

Signal noise presents an important property: its mean tends to present a nil value, implying that the integration of a signal over the range of time will tend to give a nil result. Cross-correlation functions get their robustness from this property. Equation (3) can be optimised by using its analogue form in the frequency domain. This is accomplished by applying the FFT to the signals before obtaining the cross-correlation:

$$R_{jk} = IFFT\left\{FFT\{f_j\} \cdot FFT^*\{f_k\}\right\}$$

IFFT refers to the inverse transformation, *i.e.*, from frequency to time domain and the asterisk refers to the complex conjugate needed in this equation.

Given two signals $f_j(t)$ and $f_k(t)$, the cross-correlation function $R_{jk}(\tau)$ will show a maximum exactly at the true time gap $\tau = \Delta t$ between these signals (Figure 7).

The analysis of a borehole with *N* signals $f_j(t),\ j=1\ldots N$, including several signals at the same depth, will result in N^2 cross-correlation functions $R_{jk}(\tau)$, where $j=1\ldots N$, $k=1\ldots N$. The resulting N^2 time gaps are, then, arranged in a time gap matrix $\mathbf{[T]}_{NxN}$.

Detailed procedure

Figure 14 presents the detailed procedure for signal processing built-in Spas.

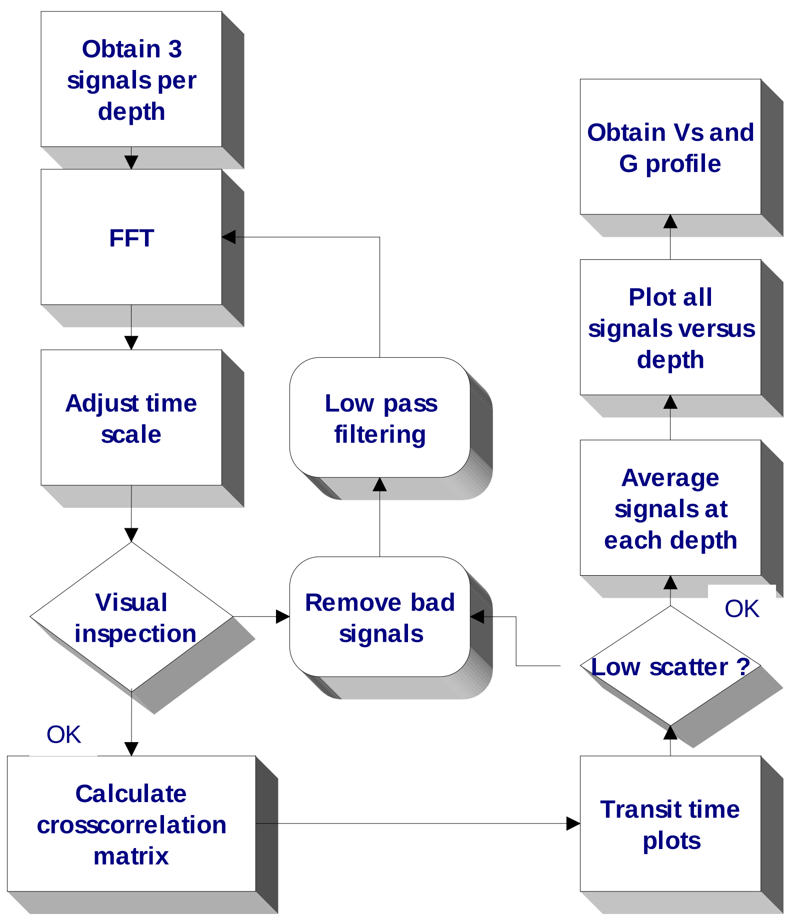


Figure 14 Flow chart for analysing seismic signals

Results

The final result of the cross-correlation analysis is shown in Figure 15. Shear wave velocities and shear modulus are presented. The shaded area corresponds to +/- the standard deviation around the mean value, and since it is a quite narrow area, it is an indication of the quality of the results. As part of the output of this research a dedicated PC program called *Spas* (Seismic Processing and Analysis Software) was developed.

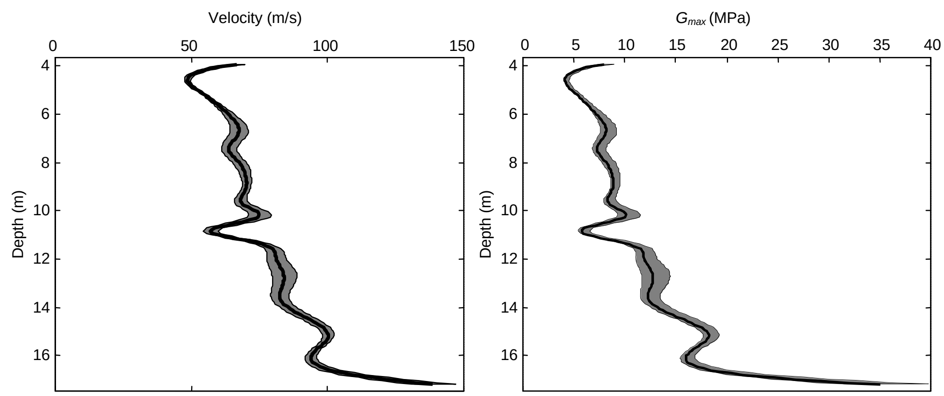


Figure 15 Results from the cross-correlation analysis

NON-LINEAR BEHAVIOUR OF SOILS

Soils behave extremely non-linear, as can be understood from Figure 16, which presents a typical relationship between shear stress and strain from a laboratory test such as the simple shear. The G_{max} is the shear modulus corresponding to the beginning of the stress-strain curve. It can only be measured at very small strains, which prevail, for instance in all seismic tests.

Once the simple shear test progresses, the *G* value presents degradation (Figure 17) and its curve can be divided into micro, medium and macrodeformation ranges. The microdeformation range takes place for most soils at very small strain levels, which are less than 10^{-3} (%) or 10^{-5}. Soils behave elastically only within this range.

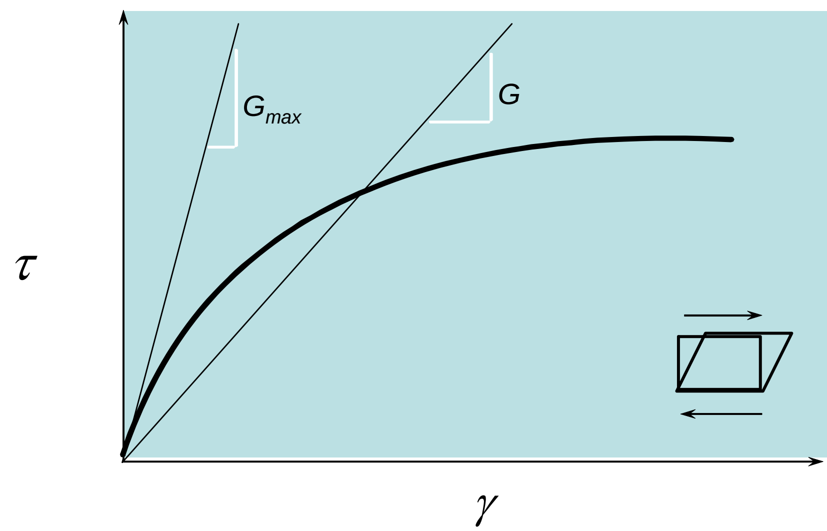


Figure 16 Non-linearuty in the stress-strain curve

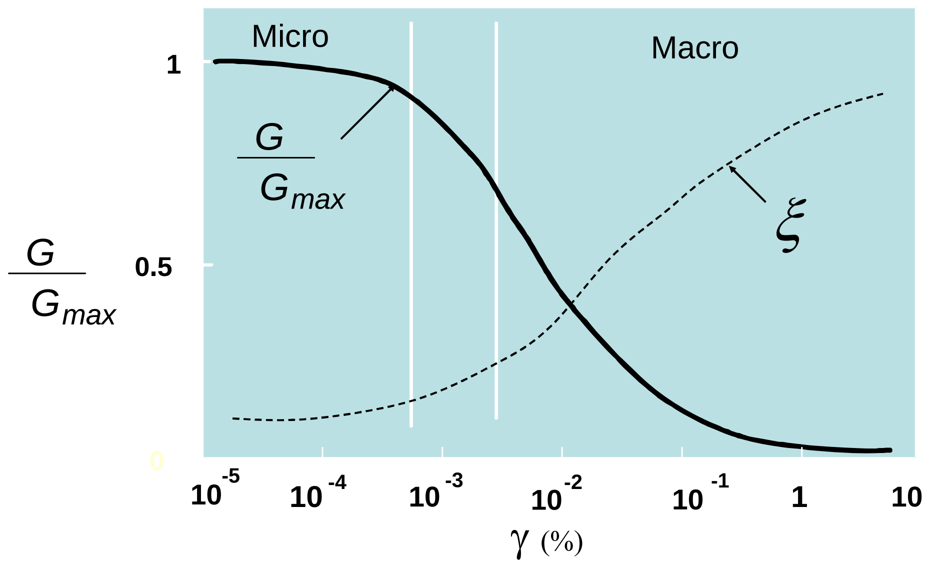


Figure 17 Shear modulus and damping versus strain amplitude (Vucetic, 1994)

Strong non-linearity dominates soil behaviour at the medium and macrodeformation ranges, as given in Figure 17. The damping parameter, which is a measure of energy dissipation, presents an inverse behaviour: it is smaller at small strains and become larger as strains increase.

A joint research programme was carried out in Brazil and Hong Kong to investigate the behaviour of saprolitic soils at various strain levels. The results were published by Ortigao et al (1997) and Schnaid et al (2000) and it is summarised here.

The scope of the work was to compare test results at the same test site at Hong Kong Bay. Figure 18 presents a summary shear modulus versus depth from all tests, including laboratory triaxial, selfboring pressuremeter (SBPM) and dilatometer (DMT) tests. The wide scatter of these results is due to the fact that they do not refer to one single strain amplitude.

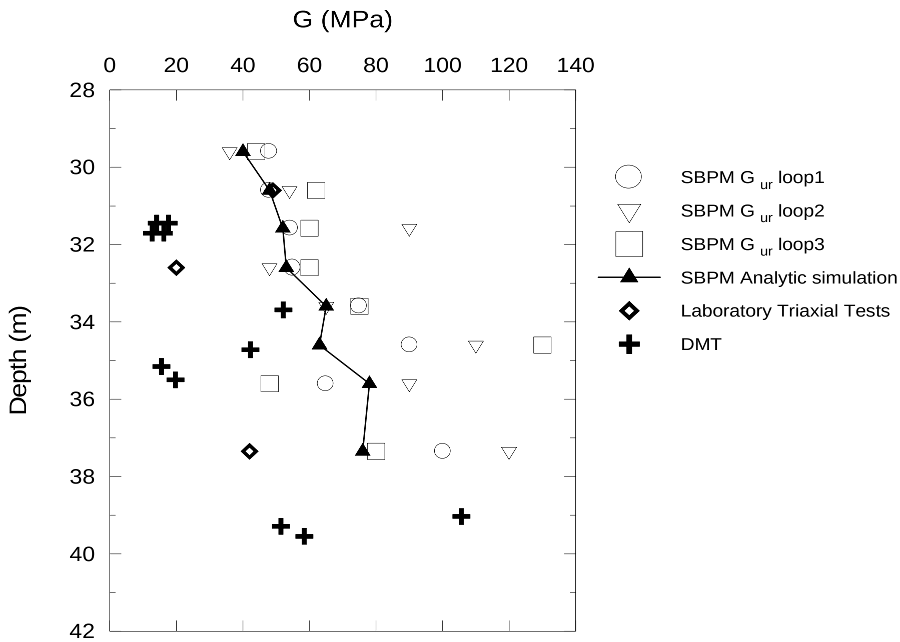


Figure 18 Shear modulus from in situ and laboratory tests

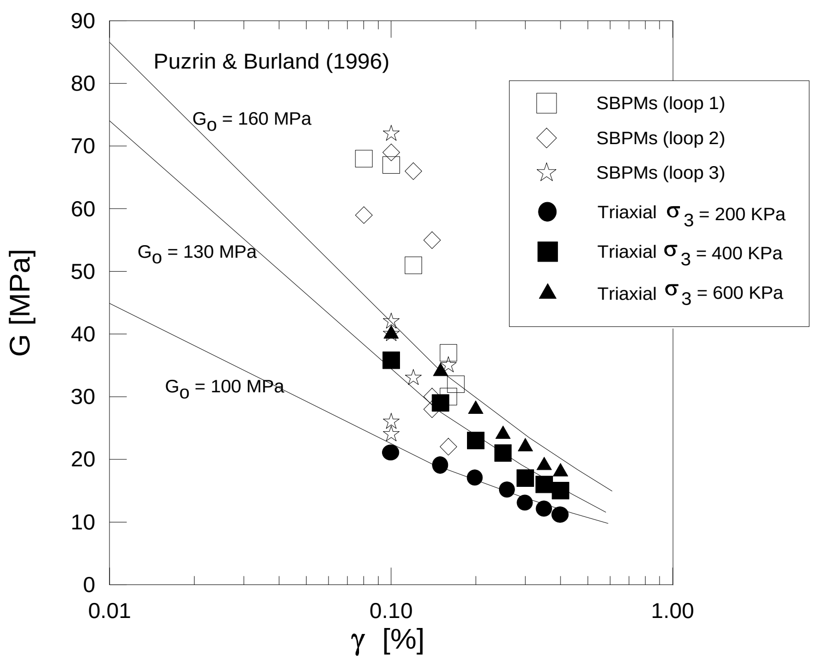


Figure 19 Shear modulus versus strain, Hong Kong Bay saprolites

These data were, then, processed and re-plotted against strain amplitude, as given in Figure 19.

The trend of reducing *G* with increasing γ is clear, but significant scatter is observed. An attempt is made to reproduce the pattern of reduction on shear stiffness with shear strain amplitude. Among the several methods that can be used to match experimental data the logarithmic formulation proposed by Puzrin & Burland (1996) was selected:

Typical G_{max} values were assumed to be within the range of 100 MPa to 160 MPa. This formulation fits the pattern given by the triaxial data SBPM moduli are scattered and produce an upper bound to existing laboratory data.

CONCLUSIONS

Field seismic methods are no longer restricted to locating and profiling sub-surface features. These methods may now be used to provide valuable engineering parameters which enable ground movements associated with structures such as foundations, retaining walls and tunnels to be predicted.

Developments in laboratory instrumentation for the measurement of strain together with back-analysed stiffness based on measurements of ground deformation have demonstrated that most soils exhibit a strongly non-linear stress-strain behaviour with very high stiffness at small strains. This has resulted in a growing interest in the measurement of stiffness at very small strain levels in soils using seismic methods.

Stiffness measured using field seismic methods overcome the problems of representative sampling, sample disturbance and insertion effects in both soil and rock and provide an upper bound for both Young's modulus, E, and shear modulus, G. Although E_{max} and G_{max} for soil cannot be used directly in design calculations they provide a valuable benchmark for the assessment of stiffness parameters determined by other methods. In rock, stiffness parameters determined from seismic velocity measurements may be used directly in predictions of ground deformation. This is particularly import-ant since these materials are often fractured rendering laboratory tests and many in situ tests unsuitable due to their inability to sample a representative volume of ground.

Of all the seismic methods used for determining stiffness-depth profiles the seismic cone and the surface-wave methods show the greatest potential. neither method requires a borehole. The seismic cone permits some measure of strength as well as stiffness. The surface-wave method is quick and easy to perform.

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APPENDIX: SEISMIC PIEZOCONE TEST STANDARDS AND PROCEDURES

Introduction

This documents describes equipment and procedures for carrying out site investigation through CPTU and seismic CPT. It also discusses de advantages of seismic CPT in relation to cross-hole tests.

Background

The seismic piezocone is one of most adequate tools for site characterisation, detailed profiling and assessment of soil properties and behaviour. It is used worldwide and also in Brazil in most major projects and should also be used in this project.

The purpose of this in situ investigation, to be complemented by other borings and laboratory tests is:

1. Detailed profiling and stratigraphy of foundation soils;
2. Site characterisation and assessment of soil behaviour;
3. Assessment of bearing capacity, strength and compressibility of soil layers
4. Assessment of consolidation and permeability of soil layers by means of dissipation tests;
5. Determination of shear wave velocity profile and in a few boreholes;
6. Determination of shear modulus profile in a few boreholes

CPT standard

Piezocone tests will be carried out according to

- ASTM D5778 (2000) Standard method for performing electronic friction cone and piezocone penetration testing of soils
- ISSMFE Report TC 16 - International reference test procedures for cone penetration tests
- ABNT NBR 12069 - Solo - Ensaio de penetração de cone in situ (CPT);

The electronic seismic piezocone

The electronic seismic piezocone (Figure 20) conforms with ASTM D5778 standard and has the following characteristics:

- 100 MPa tip capacity;

- Friction measurements at the friction sleeve
- Porepressures measurement at the shoulder (u_2 position),
- Contains a geophone on the top of the instrument for shear wave velocity measurements

The automatic data acquisition system is shown in Figure 21.

Figure 20 Electronic cone just before insertion

Figure 21 Data acquisition

CPT equipment

The CPT truck (Figure 22) is mounted on a Mercedes Benz 1313 chassis and has a hydraulic rig for CPT testing with a hydraulic downward thrust of 200 kN capacity. The hydraulic rig was made in Holland by Goudsche Machinenfabriek in the 80's.

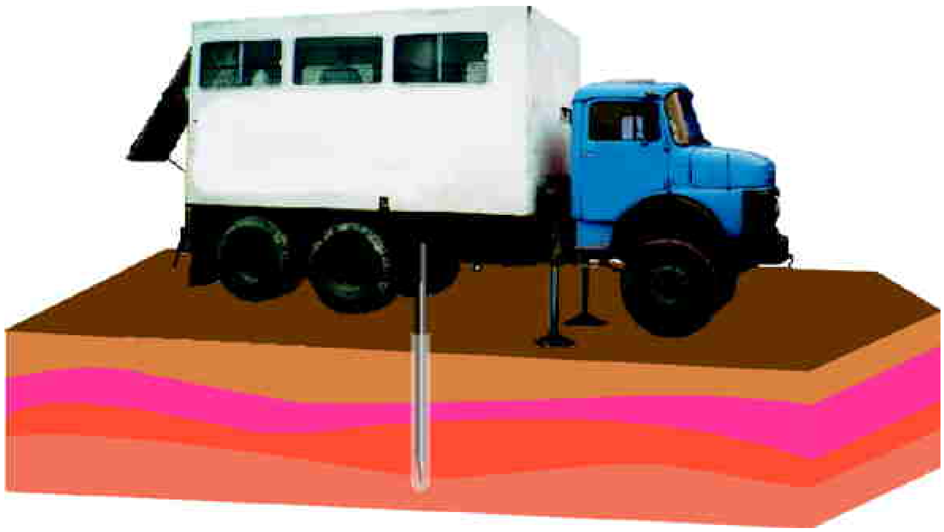

Figure 22 CPT truck

CPT Borehole log

Figure 23 presents a sample of a CPTU borehole log, which conforms with ASTM D3441. This standard only requires the following plots against depth::

- q_t is the tip resistance, corrected for the effect of the area ratio on the cone tip. The cone has an area ratio of 0.80 (Robertson and Campanella, 1989)
- R_f is the friction ratio $R_f = q_t / f_s$
- *u plot* presents the measured u_2 porepressure and a linear plot of the hydrostatic porepressure u_0

In addition, our company provides:

- $\Delta u / q_t$ ratio, which is a porepressure parameter
- Interpreted stratigraphy according to Robertson and Campanella (1989)

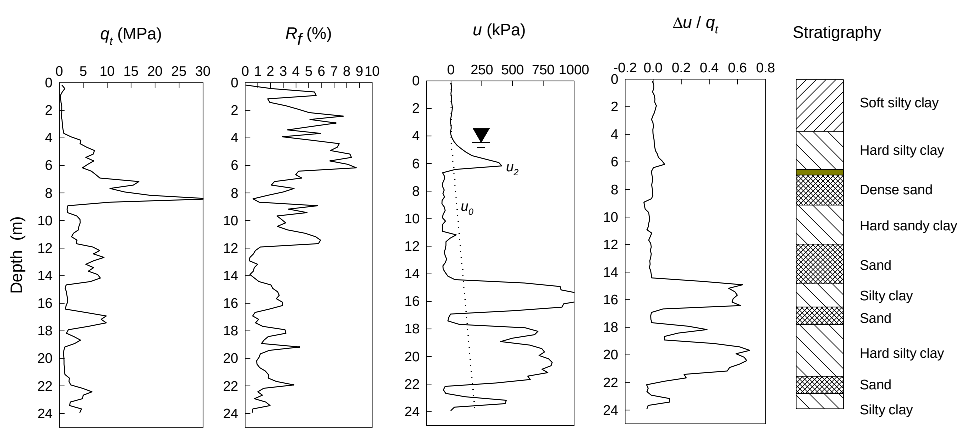

Figure 23 CPTU borehole log, with interpreted stratigraphy

Dissipation tests

Dissipation tests shall be carried out in the soft clay layer. The duration of the dissipation tests considered in our proposal is 30 minutes. Should the client require additional time, this could be arranged at an additional charge.

When penetration stops the system enters automatically in dissipation mode in which porepressures (u) are observed with elapsed time (t). The porepressures are plotted with $\sqrt{t}$, as in Figure 24.

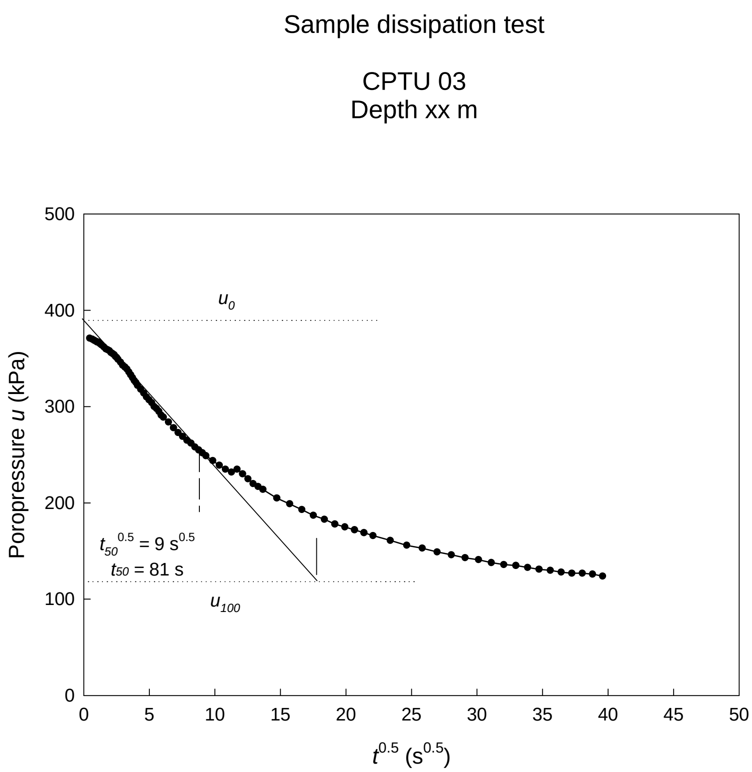

Figure 24 Sample dissipation test plot of porepressure against square root of time in seconds

The graphical technique suggested by Robertson and Campanella (1989), yields a value for t_{50}, which corresponds to the time for 50% consolidation.

The value of the coefficient of consolidation in the radial or horizontal direction c_h was then calculated by Houlsby and Teh's (1988) theory using the following equation:

$$c_h = \frac{Tr^2 I_r^{0.5}}{t_{50}}$$

where:

T = time factor given by Houlsby and Teh's (1988) theory;

r = piezocone radius;

I_r = stiffness index, equal to G divided by the undrained strength of clay (c_u). This index is taken as 100.

SEISMIC CPT

Seismic piezocone tests are carried out to measure shear wave velocity. We analyse the results through our own software *Spas* (Seismic processing analysis software) which employs state-of-the-art digital signal analysis techniques. Seismic piezocone presents advantages in relation to cross-hole tests and avoids avoiding problems related to loose contact between the geophone and the soil.

It is important to emphasise that shear modulus obtained through seismic testing is *not empirical* , but obtained through a *rigorous equation and a rigorous method*. Therefore, site-specific measurements of shear modulus are obtained. This is particularly important in tropical environments, in which empirical relationships like those obtained by Robertson and Campanella (1989) may not be applicable. Indeed residual, lateritic and other tropical soils do present weak particle bonds in the microstrain domain which can be assessed by seismic tests. On the other hand, our company has considerable experience with local soils to analyse small strain shear modulus and to apply degradation corrections to obtain large strain modulus.

Background

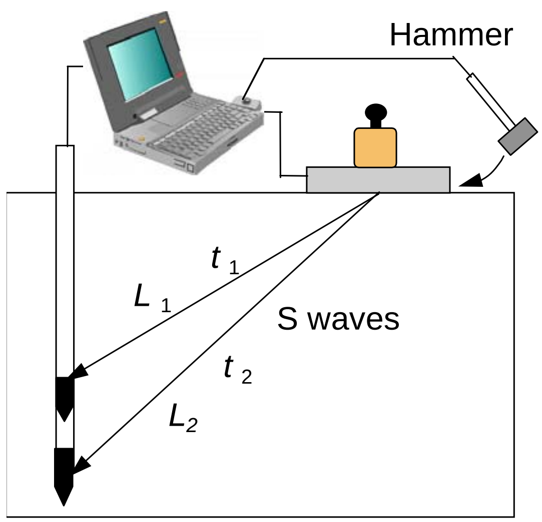

Figure 25 seismic tests

Seismic piezocone tests give shear modulus at small (micro) strains. The equipment (Figure 25) consists of a standard electric cone in which a small electronic device mounted at the top of the cone, which is capable of detecting the arrival of shear waves. This device can consist of an accelerometer or a geophone.

The test is carried out as shown in Figure 25 and Figure 26. Shear or S waves are generated at the soil surface by striking a steel plate with a hammer. The steel plate should be securely fixed on the soil surface by a weight or other method. The hammer is electrically connected to a triggering circuit in the data acquisition system.

Figure 26 Details of hammer and strike plate for shear wave velocity measurements (with a CPT truck)

The seismic test is normally carried out during cone penetration pauses to add new rod lengths. As the cone is at depth z_i, the strike plate is hit by the hammer and the triggering circuit starts the data acquisition. A signal versus time is obtained at each test depth z_i and corresponds to the arrival time t_i of the shear wave. The shear wave velocity V_s between depths z_i and z_{i+1} is given by

$$V_s = \frac{z_{i+1} - z_i}{t_{i+1} - t_i} = \frac{\Delta z}{\Delta t} \qquad (1)$$

The shear modulus G_{max} is then computed by:

$$G_{max} = \frac{\gamma}{g} \cdot V_s^2 \qquad (2)$$

where γ is the soil unit weight g is the acceleration of gravity.

Small strain G_{max} values can be corrected to the macro-strain domain, which corresponds to most geotechnical engineering applications by the use of a single laboratory test or theoretical degradation curves.

Visual inspection

The first stage in signal processing is a check on signal repeatability. This is carried out by visual inspection of all signals at a time in the time and frequency domain. *Spas* enables the user to select a depth (on the left side of Figure 27. The upper plot of the same figure displays the signal in time domain. *Spas* automatically generates a signal plot in frequency domain through a Fast Fourier Transform (FFT) transformation. The user can decide whether to keep or to delete one or more signals.

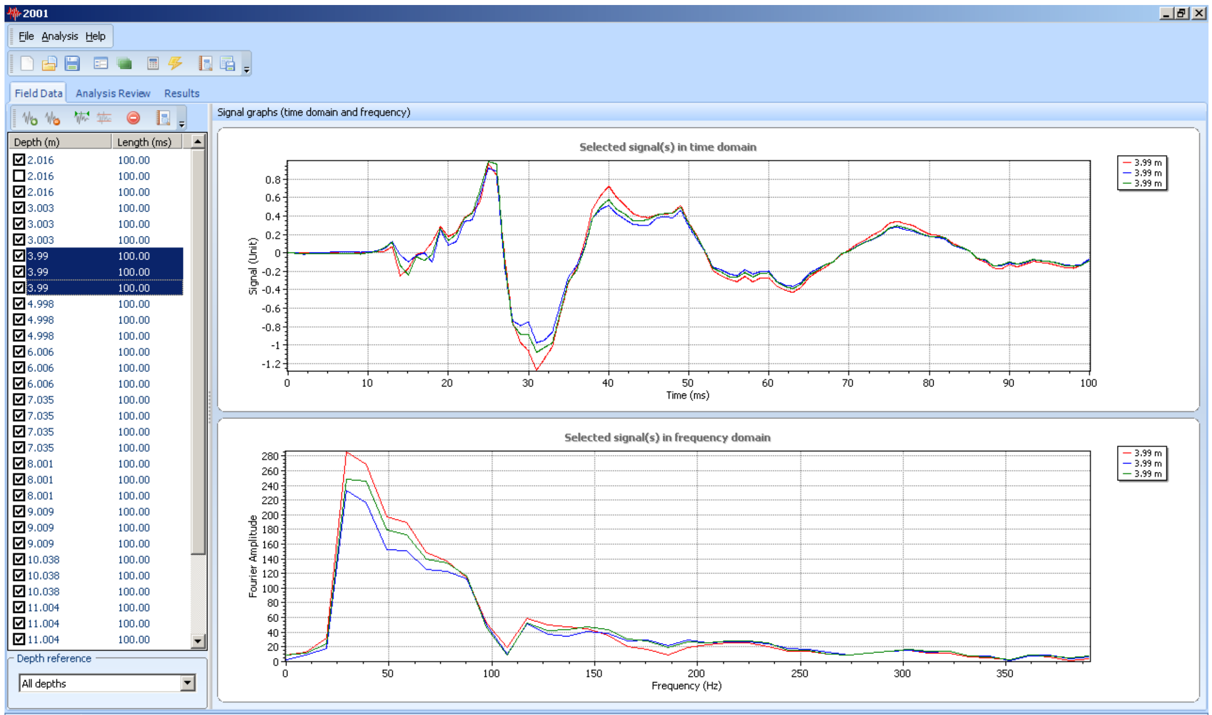

Figure 27 Visual inspection of three signals at the same depth

Transit time inspection

Figure 28 shows a plot of transit time taken by the shear wave from a selected reference, which can be any test depth. This plot enables checking the quality of cross-correlation of every single signal against all others. Any large difference can be detected and any signal can be deleted.

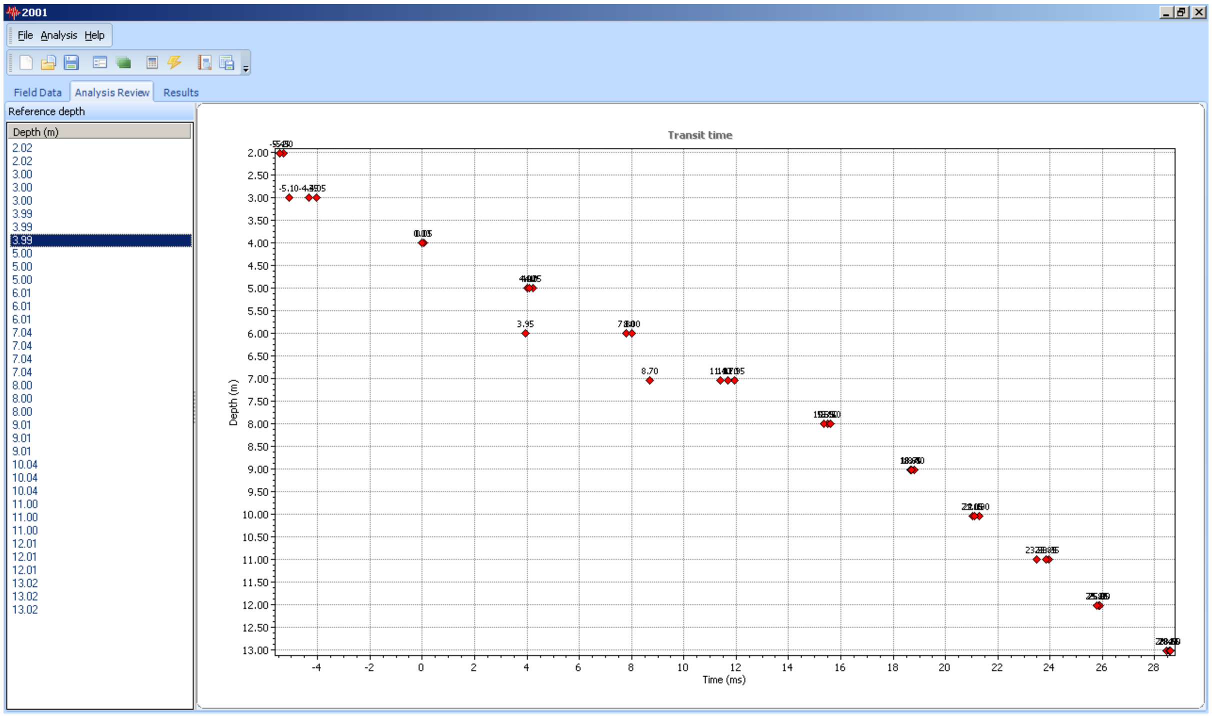

Figure 28 Transit time plot

Automatic analysis

Once all individual signal analysis has taken place and the user has deleted spurious results, Spas obtains the average signal from each depth, calculates cross-correlation and obtains the shear wave velocity profile. Figure 29 shows a typical result from these analyses.

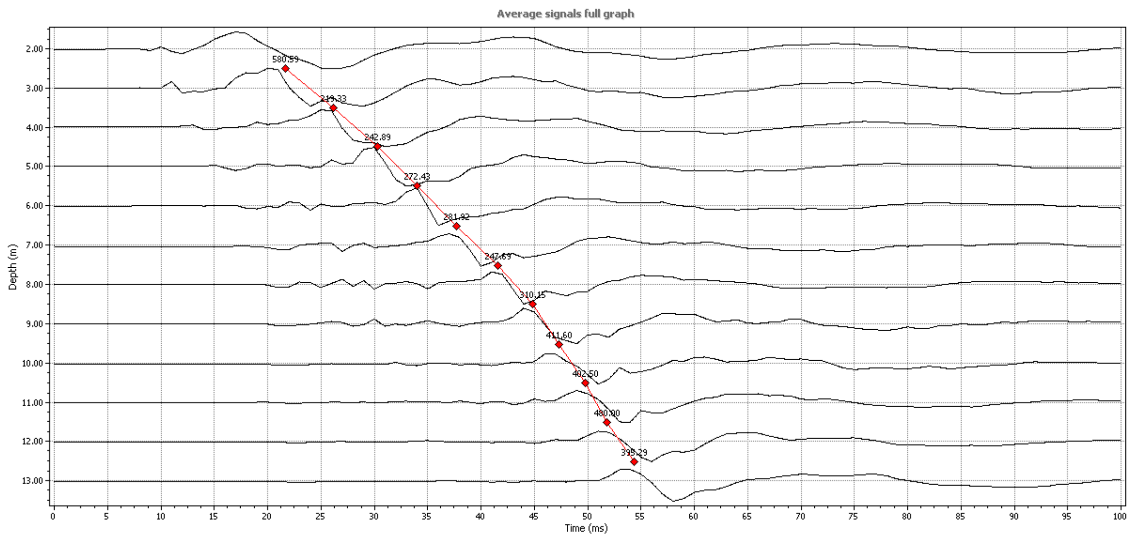

Figure 29 Plots of average signals at each depth and shear wave velocity profile

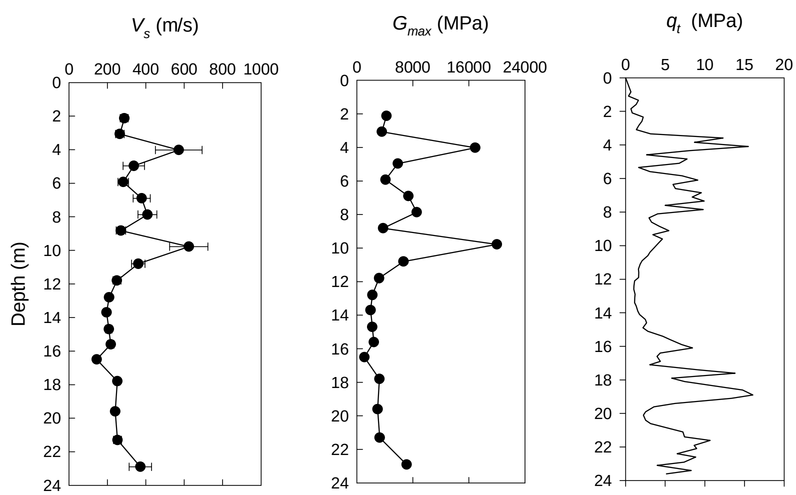

Figure 30 Sample seismic piezocone test results showing the shear wave velocity, the shear modulus and a tip resistance plot against depth

ADVANTAGES OF SEISMIC CPT IN RELATION TO CROSSHOLE TESTS

Seismic CPT present the following advantages in relation to cross-hole tests:

1. The piezocone tip is firmly inserted into the soil and is not allowed to vibrate, leading to much better dynamic signals;

2. CPT testing gives a complete package of information on stratigraphy, strength and compressibility of soils, while the cross-hole yields seismic modulus only;

3. The cost of preparing the borehole for cross-hole tests, including drilling and cementing the PVC casing is much higher than seismic piezocone.

4. Seismic CPT tests are much faster than the cross-hole.

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