Near-surface site characterisation by ground stiffness profiling using surface wave geophysics.

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1. Geophysics in civil engineering

Geophysics is probably best known for its applications in the petroleum industry for use in oil prospecting. In civil engineering, however, traditionally geophysics has been used to target and map subsurface features (e.g. ground radar). But now civil engineers and engineering geologists who specialise in ground engineering (called geotechnical engineers) are doing something new - measuring ground parameters. The science of ground engineering is called “rock mechanics” when applied to rocks and called “soil mechanics” when applied to soils and soft rocks. In the relatively young science of soil mechanics, the use of geophysics is not new. The soil mechanics pioneers like Terzaghi (1943) and Hvorslev (1949) were both interested in using geophysics in the study of machine foundations subject to vibration in the 1940s. During the 1950s, Jones (1958) at the Transport Research Laboratory in the UK used surface wave geophysics to assess materials under road pavements but his work was hampered by the unsuitable technology of the time that involved manually using an oscilloscope.

With the advent of spectral analysis and portable computers that enable the application of fast Fourier transforms online, the appeal of surface wave geophysics has grown with the technology. At the same time, the emphasis in geotechnical design has swung away from stability and strength measurement towards ground deformations and stiffness measurement. This is because in many urban environments sites are very congested and construction of tunnels, retaining walls and deep excavations cause ground movements that have an adverse effect on nearby structures. With numerical modelling able to predict ground movements qualitatively, the assessment of stiffness has become the key element to enable quantitative predictions.

There has been a move towards the use of field techniques for the measurement of stiffness because laboratory methods are subject to sampling disturbance and to unrepresentative sampling. Many field tests, however, are unsuitable for measuring soil parameters in the (vertical) direction of foundation load application. Plate loading tests simulating the size and loading of real foundations are very expensive and are rarely carried out except to verify less expensive methods. The pressure-meter, which is inserted into a borehole and expands a shell, gives horizontal ground parameters only. Penetration testing (pushing a pointed probe into the ground and measuring the force penetration relationship e.g. CPT – Cone Penetration Test) can only be interpreted by empiricism.

Seismic methods overcome these problems by measuring a ground property, maximum stiffness, as distinct from a ground parameter (like strength) that depends on the method of measurement. Research is now focussed on two main areas. How do we factor maximum stiffness down to that at operational strain levels? How do we convert (the process is called “inversion”) the shear wave velocity-wave length relationship (this is the output of spectral analysis of surface waves and is called the “dispersion curve”) into a useful shear modulus-depth profile?

2. Geotechnical problems that the surface wave method solves

2.1 Overview

The surface wave method is non-invasive and tests a large zone of ground and so the method avoids the problems of borehole based methods i.e. drilling and sampling disturbance and unrepresentative sampling and hence testing of samples. It is particularly suited to soils containing stones and rock debris like glacial tills, residual soils, boulder clay, and to fractured and jointed rock, where penetration testing like CPT (Cone Penetration Test) and boreholes cannot be used. The method uses very small strains that are now known to be near the operational stiffnesses operating around real civil engineering structures like foundations, retaining walls and tunnels.
2.2 Stiffness in geotechnical engineering

2.2.1 The variation of soil stiffness with strain level

It is well known that conventional approaches to settlement prediction greatly over-estimate ground movement. Typically, the methods use Young’s modulus, $E$, measured in compression in the triaxial test. Jardine et al. (1978) measured small strains ($\approx 0.01\%$ to $0.1\%$) in the triaxial test using local strain transducers inside the triaxial cell. They showed that small strain stiffness is very much greater than large strain ($= 1\%$) values derived from conventional measurements. These measurements are made outside the cell and include end cap bedding errors.

Finite element back-analysis from observed movements around real civil engineering structures give smaller strains (typically $< 0.1\%$) with stiffnesses greater than measured by these new triaxial tests. Dynamic tests using the resonant column apparatus apply very small strains (typically $< 0.001\%$) and give stiffnesses greater than the back-analysed values. Clearly, soil stiffness is dependent on strain level.

The typical variation of shear or bulk stiffness with strain for most soils is given in Fig.1. It is believed that most soils behave elastically at very small strains (i.e. $< 0.001\%$) giving rise to a constant stiffness. The strain induced by the propagation of seismic waves is within this range and hence provides a measure of the upper bound for stiffness ($E_0$ or $G_0$ i.e. the maximum modulus value that occurs at strains very near zero). It is also now generally accepted that ground strains associated with most soil-structure interaction problems are less than $0.1\%$ and hence small strain stiffness values are required to make reasonable predictions of deformation (Jardine et al., 1986).

Fig.2 Typical shear modulus-depth profiles measured using the CSW, seismic cone, cross-hole and triaxial methods (Matthews et al., 1993).

The upper bound stiffness is clearly a fundamental parameter in defining the curve of Fig.1 and hence in situ seismic measurements of stiffness will become even more important in the future. At the present time these measurements may be used in conjunction with laboratory measurements for soil and in situ loading test measurements in rock which provide a lower bound for stiffness. As shown in Fig.2, it can be seen that local strain measurement in the triaxial test provides a lower bound to stiffness measurement while surface wave geophysics data define an upper bound.

2.2.2 Advantages of geophysical methods for stiffness measurement

Like all methods of parameter determination, field geophysical techniques have both advantages and disadvantages. For most engineers, the primary difficulty has been a belief that geophysics measures dynamic stiffness at very small strain levels, and that this stiffness is very different from that required for geotechnical design. This is now known not to be the case. Matthews (1993) suggests that for fractured Chalk the ratio between stiffnesses predicted using geophysics and those from large-diameter plate tests is close to unity and that geophysics provides the best way to determine stiffness.
For clays it has also been found that the maximum stiffness in highest quality laboratory specimens is close to that determined from field geophysics (Clayton & Heymann, in press). Table 1 shows the stiffness degradation with strain level for soft clay (Bothkennar Clay), stiff fissured clay (London Clay) and intact weak rock (Chalk) determined in the triaxial apparatus using local strain instrumentation described by Heymann et al. (1997). It is clear from Table 1 that the stiffness at operational strain levels, $E_{op}$, is between 40% and 80% of the maximum stiffness, $E_0$. Given the rates of stiffness degradation discussed above, it would seem entirely reasonable to estimate operational stiffness, $E_{op}$, from field geophysical results in combination with some (perhaps conservative) reduction factor to take account of the expected strain level around the proposed construction. Referring to Table 1, notional values for factoring $E_0$ could be as follows (Matthews et al., 2000):

$$E_{op} = 0.50E_0$$ for soft clays, and

$$E_{op} = 0.85E_0$$ for stiff clays and weak rocks.

<table>
<thead>
<tr>
<th>Material</th>
<th>$E_{0.01}/E_0$</th>
<th>$E_{0.1}/E_0$</th>
<th>$E_{1.0}/E_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact Chalk</td>
<td>0.87-0.93</td>
<td>0.42</td>
<td>failed</td>
</tr>
<tr>
<td>London Clay</td>
<td>0.83-0.97</td>
<td>0.35-0.58</td>
<td>0.11-0.20</td>
</tr>
<tr>
<td>Bothkennar Clay</td>
<td>0.75-0.81</td>
<td>0.36-0.55</td>
<td>0.11-0.21</td>
</tr>
</tbody>
</table>

Table 1. Examples of stiffness degradation for a soft clay (Bothkennar Clay), a stiff clay (London Clay) and a weak rock (Intact Chalk), after Heymann, 1998.

Values of stiffness estimated in this way can be expected to be far superior to many techniques used routinely today. For example, oedometer testing (a one-dimensional compression test), external strain triaxial testing, and penetration testing, where poor performance in predicting stiffness has been known for decades (e.g. Burland & Hancock, 1977; Clayton et al., 1988; Izumi et al., 1997). As an example, Fig. 3 shows not only stiffnesses back-analysed from ground movements around a number of major structures in the London area, but also the very much lower values from routine oedometer and triaxial tests conducted on samples from the Grand Buildings site. It can be seen that these stiffnesses are about an order of magnitude too low. In contrast, the cross-hole stiffnesses for the London Clay sites are close to those derived from back analysis (Matthews et al., 2000).

Geophysics has other advantages. For example, although it may be necessary to install cased holes for sources and receivers, the vast bulk of the ground tested remains at its in-situ stress and saturation level, and undisturbed. In contrast, laboratory tests require that specimens be taken. Sample disturbance involves not only mechanical disturbance to the soil structure, but also stress relief. In addition, field geophysical results are representative of a large volume of ground, so that layering and fracturing are taken into account. The Continuous Surface Wave method and Spectral Analysis of Surface Waves, however, are non-invasive. Because boreholes are not required such tests are very fast, and the results can be obtained relatively cheaply, and avoid significant contact with contaminated land (Matthews, et al., 2000).

To sum up, the method measures the (very small strain) maximum value of shear modulus of the ground i.e. a property that can be used to characterise the site. This property may be distinguished from a parameter that depends on the method of measurement (e.g. undrained soil strength).

Fig.3 Comparison of stiffness profiles from laboratory tests, back analysis and seismic cross-hole surveys at various sites on London Clay (Matthews et al., 2000).
3. Principles of the surface wave method

3.1 Elastic waves: definitions and terminology

The nature of elastic waves and the rules they observe (e.g. dispersive nature) make them particularly suited to subsurface stiffness profiling. Referring to Fig.4, seismic waves propagated in elastic solids may be classified into the following two types:

- **Body waves.** These can be Primary or P waves. This is a longitudinal wave in which the directions of motion of the particles are in the direction of propagation. This motion is irrotational and the wave is one of dilation propagated with speed \( V_p \). Compressional and dilatational waves are called in seismology the primary or P (or “push”) waves. Body waves can also be Secondary or shear or S waves. This is a transverse wave in which the direction of motion of the particles is perpendicular to the direction of propagation. The motion is rotational and propagated with speed \( V_s \). Since \( V_p > V_s \) the first waves to arrive from any disturbance will be P waves. In seismology they are known as Secondary or shear or S waves. A fluid such as pore water does not transmit shear waves. Accordingly, in soil, shear waves are conducted through the soil skeleton only.

- **Surface waves.** In a uniform, infinite medium only P and S waves appear. If the medium is bounded or non-uniform (as surface soils are) other simple types of wave appear. The most important are the surface waves that are propagated near the surface of a solid. Surface waves have depths of penetration depending on their wavelength. In non-uniform media, surface waves travel at a velocity dependent on their frequency. In seismology this phenomenon is described as “dispersive”. The analogy is with optics. The ground separates the radiation according to wavelength like a prism disperses or separates white light according to wavelength. Surface waves can be Rayleigh waves or Love waves. Rayleigh waves are waves beneath a free surface in the plane \( z = 0 \) whose amplitude diminishes exponentially in the \( z \)-direction being propagated along the \( x \)-axis. They are waves in which the particles of the medium move in vertical planes. The particle motion describes vertical ellipses in which the vertical axes are about 1.5 times the horizontal. At the highest points of the ellipses the particle motion is opposite to the direction of wave advance. If a solid is stratified so that the regions are of different materials, a type of Rayleigh wave appears. These waves are known as “Love” waves and are horizontally polarised.

\[
V_s = \sqrt{G/p} \quad (1)
\]

Fig.4 Elastic waves in ground.

3.2 Seismic wave relationships with elastic properties

Seismic methods utilise the propagation of elastic waves through the ground. The waves propagate at velocities that are a function of the density and elastic properties of the ground. The Rayleigh wave is a surface wave that travels parallel to the ground surface at a depth of approximately one wavelength. In an isotropic elastic medium the velocity of a shear wave, \( V_s \), is:

\[
V_s = \sqrt{G/p} \quad (1)
\]
where $G$ is the shear modulus and $\rho$ the density. According to the theory of elasticity, Young's modulus $E$ is related to $G$ by:

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

where $\nu$ is the Poisson's ratio. Thus $G$ can be obtained from measurements of $V_s$ alone.

Surface waves may also be used to determine shear stiffness in soils and rocks. 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. These waves travel at speeds governed by the stiffness-depth profile of the near surface material. 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:

$$V_r = C V_s$$  \hspace{1cm} (3)$$

where $C$ is a function of Poisson's ratio, $\nu$. 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 therefore probably less than 10%.

3.3 Seismic methods for stiffness measurement

The seismic methods employed to determine stiffness-depth profiles are divided into subsurface and surface methods as shown schematically in Fig.5. Most of the subsurface methods require one or more boreholes that need to be cased with special plastic casing that adds to the cost of the survey.

These methods are most useful where the depth of investigation is greater than 15m and involve the measurement of the transit times of seismic waves over known distances. The seismic cone has the advantage of providing both stiffness and strength-related data and does not require a borehole since the cone is pushed into the ground. The depth of penetration is limited, however, by the strength of the ground and any obstructions such as boulders, claystones or rock layers.

A simple and cost-effective surface method makes use of surface waves. 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.

To sum up, surface wave methods are attractive both for speed and to save the cost of drilling boreholes.

3.4 The surface wave method

Surface waves may be used to determine shear stiffness in soils and rocks. These 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 (eg Hertwig, 1931; Jones, 1958; Heukolum and Foster, 1962; Abbiss, 1981).

Two distinct surface-wave methods are available.

- **Spectral Analysis of Surface Waves** (SASW). This method makes use of a hammer as an energy source. The field technique is described by Ballard and McLean (1975), Nazarian and Stokoe (1984), Addo and Robertson (1992), and by Matthews et al (1996).

- **Continuous Surface Wave** (CSW). This method makes use of a vibrator as an energy source. The field technique is described by Ballard and McLean (1975), Abbiss (1981), Tokimatsu et al. (1991), and by Matthews et al. (1996).

The SASW method relies on the frequency spectrum of the energy source used. In most cases a range of hammers of different mass are
employed to achieve the necessary frequency range (3-200Hz). However, it is inevitable that certain frequencies will be missing from the spectra of these sources. This can result in gaps in the stiffness profile data. This serious disadvantage may be overcome by replacing the hammers with a frequency-controlled vibrator. This is the basis of the CSW method.

Fig.6 Schematic diagram of the GDS Continuous Surface Wave System

The basic procedure for the CSW method is shown in Fig.6 and Fig.10. The vibrator generates a predominantly vertical ground motion at a constant frequency. This ground motion is recorded using low natural frequency (2Hz) geophones placed in line with the vibrator. The analogue signal from each of these geophones is digitised and recorded with respect to time (time-domain). These data are then subjected to a Fourier transform in order to determine the phase of the signal received by each geophone with respect to the vibrator. The gradient of the phase-distance relationship permits the wavelength of the Rayleigh wave to be determined from:

$$ \lambda = \frac{2\pi}{\Delta \phi / \Delta d} \quad (4) $$

where $\lambda$ is the wavelength of the Rayleigh wave and $\Delta \phi / \Delta d$ is the gradient of the plot of phase against distance. The phase velocity of the Rayleigh wave $V_r$ of frequency $f$ is given by the relationship:

$$ V_r = f \lambda \quad (5) $$

The method is summarised in Fig.7 and Fig.10.

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.

Fig.7 Definition figure for calculating phase velocity.

The process of converting a field dispersion curve to a Rayleigh wave velocity-depth relationship is known as inversion. There are three principal inversion methods:

- The factored wavelength method.
- Finite element approaches.
- Linear models

The factored wavelength method is the simplest, but least exact, of the methods. It is of practical value because it offers a relatively quick way of processing data on-site and so enables preliminary assessment. If using either of the other techniques, then the factored wavelength method can provide a useful initial estimate of the velocity-depth profile to input to the other algorithms. In the factored wavelength method the representative depth is taken to be a fraction of the wavelength, i.e. $\lambda/\Delta z$ is assumed to be a constant. A ratio of 2 is commonly, but arbitrarily, used (Jones, 1958; Ballard & McLean, 1975; Abbs, 1981). Gazetas (1982) recommended that $D = \lambda/4$ is used at sites where the stiffness increases significantly with depth, and that $D = \lambda/2$ is suitable at more homogeneous sites. Gazetas also suggested that taking $D = \lambda/3$ is a reasonable compromise. The profiles given in Fig.14-18 have used the factored wavelength method of $D = \lambda/3$. A physical understanding of why this approach could give sensible results in some cases can be appreciated by considering Fig.8. By approximating the Rayleigh wave amplitude distribution to a triangle over an effective depth of one wavelength, it can be seen that the centre of gravity of the motion is at a depth of one third the wavelength.
Fig. 8 The amplitude of surface wave displacements as a function of distance to the surface (after Richart, Hall and Woods, 1970).

Using the finite element method a synthetic dispersion curve is generated and the stiffness distribution is progressively adjusted until the synthetic dispersion curve matches the curve obtained in the field (Clayton et al., 1995). The ground is divided into layers on constant stiffness. For a simple sub-surface geometry a two-dimensional, axially symmetric, idealisation of surface wave tests can be made. The equations of motion are integrated with respect to time to model the ground motion at the actual geophone locations used in the field. These data are used to determine the synthetic dispersion curve.

Linear models have been proposed by Nazarian and Stokoe (1984) (the so-called “Haskell-Thompson” method) and by Lai and Rix (1998).

4. Use of the surface wave method in practice

4.1 Introduction to the GDS Surface Wave Systems

Two surface wave systems are available that share several common elements like field computer, geophones, geophone take-out cable, etc. The SASW (Spectral Analysis of Surface Waves) method makes use of a hammer as an energy source (see Fig. 9). The CSW (Continuous Surface Wave) method makes use of a vibrator as an energy source (see Fig. 10). Both techniques are described in detail by Matthews et al., (1996).

Fig. 9 Schematic diagram showing the steps followed in the determination of a dispersion curve using the SASW (hammer-based) method (after Matthews et al., 1996).

As indicated in Fig. 9, the SASW system comprises a hammer as the seismic source, medium frequency (6Hz) geophones and a field computer.

As shown in Fig. 11, the CSW system comprises a frequency-controlled vibrator, low frequency (2Hz) geophones and a field computer. The frequency of the vibrator and frequency of the geophone data acquisition are controlled by the PC. Once the vibrator and geophones have been positioned and the details of the location and layout entered into the computer, the vibrator is taken from 5 to 100Hz in 5Hz increments and the resulting stiffness profile is displayed. Based on this preliminary survey, the user can then select a number of frequency ranges and assign suitable frequency increments in order to provide a more comprehensive stiffness profile. The minimum frequency increment is 0.1Hz. This is required in order to ensure continuity of the stiffness profile when operating at frequencies less than 10Hz. The
raw data are stored on disk for further analysis in the office. The stiffness profile derived from the factored wavelength inversion method described above is displayed on the computer screen and is continuously updated during the test. In general a typical test in which over 50 stiffness measurements are taken at different depths will take less than 2 hours.

4.2 Applications of surface wave methods

The applications of the surface-wave method include:

- Profiling in terms of shear stiffness (Gordon, et al., 1995)
- Prediction of ground deformation (Matthews, 1993)
- Assessment of lateral variations in stiffness
- Quality control for ground improvement (Cuellar et al., 1995), rail track ballast and sub-grade and road pavement sub-grade (Jones, 1962)
- Rock mass assessment (Matthews, 1993)
- Determination of landfill thickness (Butcher & Tam, 1994)
- Assessment of liquefaction potential (Stokoe & Nazarian, 1985)
- Assessment of shear damping ratio (Lai and Rix, 1998)

The most common applications at the present time are profiling in terms of shear stiffness and the prediction of ground deformations. For example, Fig.2 shows results from an investigation carried out to provide stiffness parameters for the structural design of cut and cover tunnels and retaining walls associated with a major junction improvement scheme. Stiffness data were also required to assess the effects of construction on nearby buildings. The site is underlain by over 30m of London Clay. It will be seen from Fig.2 that data obtained from the seismic cone penetration test (SCPT), cross-hole and continuous surface-wave (CSW) surveys show good agreement and form an upper bound for the stiffness measurements. The lower bound for stiffness measurements was provided by the laboratory triaxial tests with local strain measurement. In general the shear modulus at very small strain (e from the seismic tests) is about two times that measured at 0.01% strain in the triaxial tests. This indicates that the drop in stiffness from very small strain levels to the lower bound of field operational strain levels is perhaps not as severe as indicated by the idealised model of stiffness-strain behaviour shown in Fig.1.

Using large scale loading tests and observations of full-scale foundations, it has been shown that the load-settlement behaviour of chalk is more or less linear elastic up to the yield stress which varies between 200kPa and 400kPa (Ward et al., 1968, Burland & Lord, 1970, Matthews, 1993). It seems reasonable, therefore, to use stiffness-depth profiles determined using seismic methods directly in predictions of ground deformation such as foundation settlements. Matthews (1993) carried out 9 large diameter (1.8m) plate loading tests on weathered chalcos with similar discontinuity patterns but different intact stiffnesses. In each case the stiffness-depth profile beneath the plate locations was determined using surface-wave geophysics, the standard penetration test and visual assessment based on Ward et al. (1968). Fig.12 shows a comparison between the observed and predicted settlement for these nine tests. Clearly, the Standard Penetration Test (SPT) either grossly over-predicts or under-predicts the settlement, whereas predictions based on geophysics yields reasonable agreement.

Fig.10 Schematic diagram showing the steps followed in the determination of a dispersion curve using the CSW (vibrator-based) method (after Matthews et al., 1996).
5. Case studies of site characterisation by the GDS Continuous Surface Wave system

- The profiles in Fig.13 are part of a set of 10 produced by one operative in one day. The site is a trial road in the U.K. Measurements were made directly on the sub-grade (see also Fig. 11). The dip of the interface between the overlying stiffer layer and the underlying softer layer is clearly seen. When the road is constructed, tests will be repeated from the road surface.

- The profile in Fig.14 is for a site in the U.K. The transition from fill to underlying dense chalk is not clearly seen because of the scale of stiffnesses involved. Chalk is a jointed weak rock and the stiffness values measured are for the chalk mass i.e. including the effect of the jointing.

- The profile in Fig.15 is for a demonstration site in northern France. The profile was produced under computer control as usual and then the test was run again and the results superimposed. All the points from the second run plotted directly on the points from the first run and were indistinguishable from them. This demonstrates good repeatability of the system and its reliability in producing raw data.

- The profile in Fig.16 is for a coastal site in southern England. The upper desiccated layer is clearly seen with stiffness decreasing down to the water table and increasing with depth thereafter.

- The depth profiles in Fig.17 compare CPT cone resistance and CSW shear modulus. Looking at the shapes, the inherent averaging of the CSW method is apparent.

- The depth profiles in Fig.18 compare CPT cone resistance and CSW shear modulus to demonstrate ground improvement following treatment by vibro-stone columns. The CSW plot clearly shows an approximate doubling of modulus for the test station located midway between the stone columns for before and after installation. The important point here is that the demonstration of improvement by CSW is quantitative and provides a real engineering parameter, shear modulus. The improvement shown by CPT, however, is qualitative only and requires empirical interpretation to relate cone resistance to an engineering parameter.
Fig. 13 CSW stiffness-depth profiles along the sub-grade of a trial road showing the dip of a softer underlying soil.

Fig. 14 CSW stiffness-depth profile for a site in southern England.

Fig. 15 CSW stiffness-depth profile for a site in northern France.

Fig. 16 CSW stiffness-depth profile for a coastal site in southern England.
Fig. 17 Comparison of CSW and CPT profiles at a coastal alluvial site in southern England showing a soft alluvial layer at shallow depth between stiffer fill and medium dense gravel bed.

Fig. 18 Comparison of “before and after” CSW stiffness-depth profiles and CPT cone resistance-depth profiles for loose sandy fill between vibro-stone columns for a site in the U.K.

6. Summary of main points

- The SASW/CSW method is non-invasive and tests a large zone of soil and so the method avoids the problems of borehole based methods i.e. sampling disturbance and unrepresentative sampling/testing.

- The method is particularly suited to soils containing stones and rock debris like glacial tills, residual soils, boulder clay, and to fractured and jointed rock, where penetration testing and boreholes cannot be used.

- The method is suited to near surface applications only. Using present day signal conditioning and software, stiffness profiles in soft ground can successfully be obtained up to 10m. In hard soils/soft rocks, profiles to a depth of 50m can be obtained.

- The method uses very small strains that are now known to be close to the operational stiffnesses near real civil engineering structures like foundations, retaining walls and tunnels. An approximate relationship between maximum stiffness ($E_0$) measured by the method and the lower operational stiffness ($E_{op}$) around real structures (see Fig.1) is about

$$E_{op} = 0.50E_0$$ for soft clays, and

$$E_{op} = 0.85E_0$$ for stiff clays and weak rocks.

- The method provides an upper bound to operational stiffness while small strain triaxial is a lower bound. Most engineers are comfortable with the idea that their operational values lie between known upper and lower bounds.

- The stiffness profiles are logical (i.e. when an engineer looks at them they make sense) and correlate well qualitatively with ground-truth crosschecks such as CPT profiles and borehole logs.

- The results are remarkably repeatable (e.g. Fig.15) and so indicate that the system and its interaction with the ground is consistent and dependable as a source of raw data.

- The results are online and rapidly obtained using the factored wavelength method of dispersion curve inversion i.e. $D = \lambda/3$ (e.g. Fig.13).

- Correlation with “ground truth” like CPT is (subjectively) fairly good (e.g. Fig.17)

- The system is ideal for before and after measurements of ground improvement (e.g. stone columns case study of Fig.18) and for
monitoring changes in site characteristics e.g. between the wet and dry seasons.

• Because the method is an inherently averaging technique, minor variations such as thin seams are overlooked, but the mass properties of the ground are well represented and usually in foundation engineering it is this average value which matters.

• The drawback of the method is the uncertainty with which modulus measurements are assigned to depth, particularly in layered ground. Finite element methods and linear modelling do address this problem and will shortly enable online interpretation to be incorporated. For the present, given that the SASW/CSW methods are probably part of an overall site investigation scheme, “ground truth” data like borehole logs and CPT can be used to verify or adjust depth based on the simple factored wavelength relationship of \( D = \lambda/3 \).

• SASW is a low cost (about the cost of a small automobile) system that is highly effective at relatively shallow depth. The GDS field computer and geophone take-out cable are common to both the SASW and CSW systems. There is a logical upgrade path, therefore, from the impact seismic source system, SASW, to the considerable advantages of the harmonic seismic source system, CSW. These advantages include greater depth of stiffness profile and the ability to instruct the computer to map features (e.g. interface between layers) in greater detail.

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Dr Bruce Menzies, PhD, DSc, CEng

Bruce Menzies graduated in 1961 from the University of New Zealand. Following his Masters degree in soil dynamics he worked for a firm of civil engineering consultants before moving to England. He then spent periods lecturing at Kingston University and University College London where he completed his PhD in soil mechanics in 1970.

As Lecturer in Soil Mechanics at the University of Surrey his research in the early 1970s concentrated on applying the then emergent microcomputer to the control of stress paths in the triaxial test. Other research activities included numerical analyses of progressive failure of footings and of the performance of buried pipelines adjacent to trench excavations. He also carried out field and analytical investigations of the shear vane with particular reference to the design of footings and embankments.

He was also active in consulting and was an expert witness in several court cases relating to foundation failures. He has invented and patented a number of testing instruments. He is an author of over forty publications including technical books and papers. In recognition of his published work the University of Auckland awarded him the degree of Doctor of Science in 1990.

In 1979 he founded GDS Instruments Ltd. Widely known as simply “GDS”, the company are world leaders in computer controlled testing systems for geotechnical engineers and geologists. He left the University of Surrey in 1982 to concentrate full time on developing the company. He invented the microprocessor controlled pressure regulator that is the basic element in most GDS lab systems. He devised the computer controlled triaxial testing system based on the stress path cell. He gives many invited seminars at conferences and symposia. He is now Chairman of GDS and has returned part time to the University of Surrey.