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Ground stiffness measurement by the continuous surface wave test

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The continuous surface wave (CSW) test is a seismic technique for determining ground stiffness by measuring the velocity of Rayleigh wave propagation along the ground surface. A sinusoidal force is generated by a shaker placed on the ground surface and the response is detected by an array of geophones also at the surface. Measurements are made for a range of shaker frequencies thereby allowing a profile of stiffness with depth to be established. The continuous surface wave test is performed relatively quickly and is less expensive than conventional stiffness measurement techniques; in addition it is non-intrusive and non-destructive thus making it attractive for civil engineering applications. This paper describes the continuous surface wave test, the execution of the test, analysis of the data as well as interpretation of the results. Calibration results as well as typical results from full scale field tests are presented.

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INTRODUCTION

Seismic tests are used to measure the velocity at which stress waves propagate through the ground. They have been used for many years by seismologists and geophysicists for applications such as characterisation of the earth's interior as well as mineral exploration. Shallow seismic techniques such as reflection and refraction methods have been used in civil engineering applications since the 1930s to obtain the geological stratigraphy of the subsurface. Recent developments in seismic techniques, however, have focussed on obtaining engineering design parameters as opposed to characterisation of the subsurface stratigraphy (Clayton *et al* 1995). The continuous surface wave test is an example of such a 'new generation' seismic test.

Seismic energy applied at the ground surface generates four seismic wave types; two surface waves and two body waves. Compression waves (also known as primary waves) are body waves with the particle motion in the same direction as the wave propagation. They are the fastest of all the seismic waves and are transmitted by both the soil skeleton and the pore fluid. For a soft saturated soil the first arriving compression wave will be propagated through the water at a velocity of approximately 1 500 m/s. The compression wave propagated by the soil skeleton will only arrive some time later and its arrival is often obscured by the first arriving wave transmitted by the pore fluid.

Shear waves (or secondary waves) are the second fastest seismic wave type and are body waves which induce particle motion perpendicular to the direction of wave propagation. Shear waves are only transmitted by the soil skeleton and not by the pore fluid. The velocity of shear waves is therefore independent of the pore fluid conditions making them particularly useful for geotechnical investigations. It may be shown from elasticity theory that the shear wave velocity (V_s) of a material is related to its bulk density (ρ) and shear stiffness (G_o) as follows:

$$G_o = \rho V_s^2 \tag{1}$$

Love waves (Love 1927) are surface waves with particle motion parallel to the ground surface and perpendicular to the direction of wave propagation. Only a small proportion of the induced energy is transmitted as Love waves and for this reason until recently these waves have not been widely used in shallow seismic investigations (Guzina & Madyarov 2005). In contrast, two thirds of the energy imparted at the ground surface produces the second type of surface wave known as Rayleigh waves (Rayleigh 1885), with the remaining one third producing all of the other three wave types. Compared with the other wave types Rayleigh waves attenuate slowly at a rate of $1/\sqrt{r}$ where r is the distance from the source. These factors make Rayleigh waves attractive for seismic surveys as high-quality Rayleigh wave signals are relatively easily obtained. A soil particle at the surface propagating a Rayleigh wave will undergo both vertical and horizontal motion following a retrograde elliptical path in a vertical plane parallel to the direction of propagation. Rayleigh waves travel slower than shear waves with the ratio of Rayleigh wave velocity (V_r) and shear wave velocity (V_s) a function of Poisson's ratio (ν):

$$\frac{V_r}{V_s} \cong \frac{0{,}874 + 1{,}117\nu}{1+\nu} \tag{2}$$

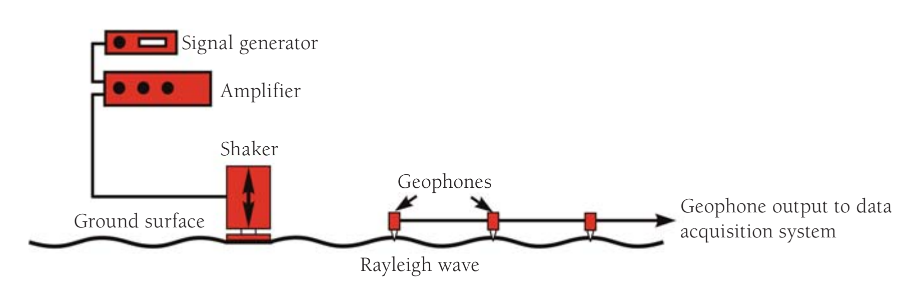

Figure 1 Layout of the continuous surface wave system

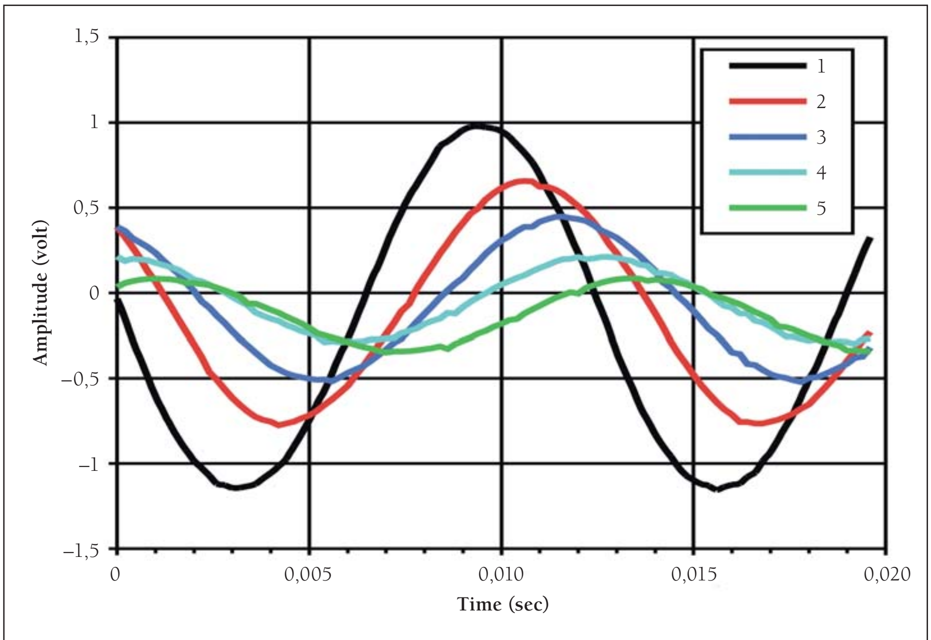

Figure 2 Output response of the five geophones

The continuous surface wave test focus on the generation, detection and interpretation of Rayleigh waves. The Rayleigh wave velocity may be used in conjunction with Equations 1 and 2 to determine the shear stiffness of the material and the continuous surface wave test is therefore a test which produces potentially valuable soil mass parameters for design engineers.

DESCRIPTION OF THE CONTINUOUS SURFACE WAVE TEST

The continuous surface wave test uses a shaker to generate the seismic energy at the ground surface by applying a vertical sinusoidal force of known frequency. High frequencies produce short Rayleigh waves which penetrate only to shallow depth whereas low frequencies produce long wavelengths which penetrate to greater depths. Sweeping through a range of frequencies allows a Rayleigh wave velocity profile to be established. The Rayleigh wave propagation is detected by an array of geophones placed at the surface in a line radiating away from the shaker. Uni-axial geophones are generally used with the measurement axis of the geophone vertical and therefore only the vertical component of the ground motion is measured. Equally spaced geophones at a fixed distance apart are commonly used but Tokimatsu *et al* (1991) suggested that the geophones spacing should be varied according to the wavelengths of the surface waves. The response of the geophones may be used to determine both the wavelength and the velocity of the Rayleigh wave at any particular frequency.

The continuous surface wave test is part of a family of surface wave tests which includes spectral analysis of surface waves (SASW), multi-channel analysis of surface waves (MASW) and the frequency-wave number (f-k) spectrum method (Stokoe *et al* 2004). The SASW, MASW and f-k spectrum methods typically use an impact source to generate the seismic energy. Typical impact sources are hammer blows to the ground surface. The continuous surface wave test requires a variable frequency shaker to generate the seismic energy. However the main advantage of the continuous surface wave test is that the operator has full control over the frequencies being produced and may therefore target the depth of interest by selecting the required frequency range. In contrast, the spectrum of frequencies generated by an impact source depends on the type of source used and the operator has limited control over the frequencies generated.

Diverse types of continuous surface wave systems exist but all have a shaker of which the frequency can be controlled and a detection system to measure the ground surface response. The following section describes the continuous surface wave system developed by the author.

System components

Two shakers of different operating frequency and input energy were developed as seismic sources to allow a range of depths to be covered. The high frequency shaker for shallow depths consists of a 14 kg electromagnetic actuator commonly used in industry for vibration testing. The signal to drive the shaker is provided by an electronic signal generator amplified by a linear power amplifier. The actuator was designed to operate over a very large frequency range between 5 Hz and 9 kHz but when used as a continuous surface wave source for geotechnical applications it is typically operated in the frequency range 15 Hz to 200 Hz. For road pavement applications frequencies as high as 10 kHz may be required to detect the stiffness of the uppermost layers (Svensson 2001). At frequencies below about 15 Hz the signal quality from the electromagnetic shaker deteriorates due to the limited energy of the shaker. The maximum depth of measurement depends on the stiffness of the ground but for the high frequency shaker the typical maximum depth is approximately 5 m to 8 m.

For deeper measurements a low frequency shaker is used comprising of a counterbalanced eccentric weight shaker driven by a 1,5 kW three-phase electric motor with angular velocity control. The total mass of the two eccentric weights is 5,24 kg with an eccentricity of 56 mm. Due to safety considerations the moving parts are contained within a metal strongbox supported by a system of helix springs. The shaker has a weight of 250 kg and is high-tuned as it has a resonant frequency of 6 Hz and is operated in the frequency range 7 Hz to 22 Hz. Measurements have been made to depths of 20 m with the low-frequency shaker.

The ground response detection system consists of an array of five geophones each with a resonant frequency of 4,5 Hz. The output from the geophones is passed through a pre-amplifier before being logged on a notebook computer using a 12 bit data acquisition card. Figure 1 shows a typical layout of the continuous surface wave system.

Data analysis

Figure 2 shows the typical output response of the five geophones in the time domain with the shaker vibrating at a frequency of 80 Hz. It may be seen that the output of each geophone corresponds well to a sine wave which is the vertical component of the ground surface motion. A surface wave response which closely conforms to coherent sine waves demonstrates the correct behaviour of the system. It also allows the opera-

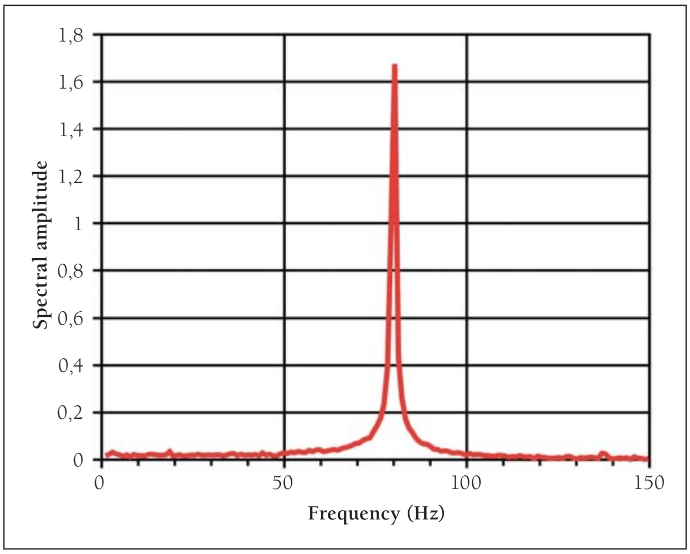

Figure 3 Geophone frequency spectrum

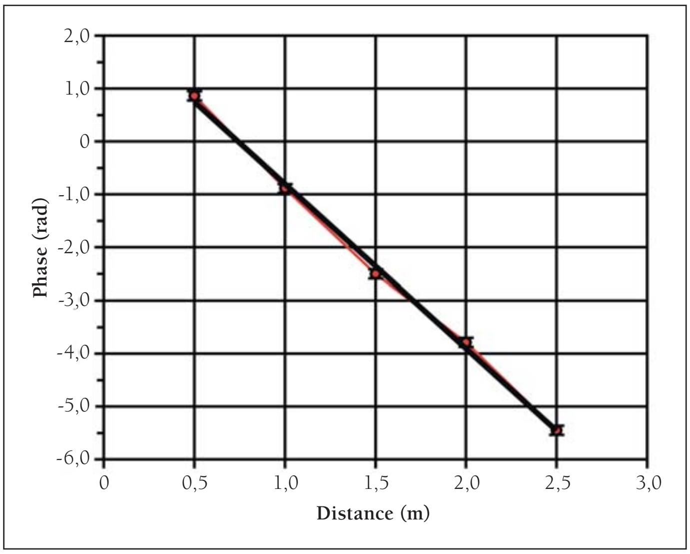

Figure 4 Geophone phase angles

tor to verify the expected operation of the system in the field. Poor quality data will result in a non-sinusoidal response which will be easily observed by the operator. Such field analysis avoids the problem inherent to some seismic systems where data analysis is conducted in the office and any shortcomings only become apparent after the field testing has been completed. In addition, Stokoe *et al* (2004) pointed out that interpretation of continuous surface wave test results are less subjective that SASW, MASW and the f-k spectrum method due to the single frequency sinusoidal response of the continuous surface wave test.

Figure 2 also shows the time offsets between the geophone outputs indicating the time required for the Rayleigh wave to travel from one geophone to the next. The attenuation of the wave amplitude may be observed as the decreasing maximum output from geophone 1, closest to the source, and geophone 5 furthest from the source.

Processing of the data is aimed at determining the wave length and velocity of the Rayleigh wave for each vibration frequency. This is accomplished by calculating the phase difference between geophones for the continuous wave generated by the shaker. First the dominant frequency at each geophone is determined by calculating the Fourier transform of the dataset by means of the fast Fourier transform (FFT) algorithm. Each frequency may be represented by a vector **z** consisting of a real (Re(**z**)) and an imaginary (Im(**z**)) component. The magnitude of the vector (|**z**|) represents the spectral amplitude at the particular frequency. A typical geophone frequency spectrum is shown in Figure 3 and the result shows that a clear peak exists at the dominant frequency of 80Hz again confirming the sinusoidal nature of the vertical ground motion at the geophone. The phase angle at the dominant frequency may also be determined from the FFT from the real and imaginary components of the phase vector as:

$$\phi = \tan^{-1}\left(\frac{\mathrm{Im}(\mathbf{z})}{\mathrm{Re}(\mathbf{z})}\right) \tag{3}$$

The wavelength of the Rayleigh wave (λ) may be determined as:

$$\lambda = \frac{d}{\left(n + \frac{\Delta\phi}{2\pi}\right)} \tag{4}$$

Where:

$\Delta\phi$ is the phase change from one geophone to the next

d is the distance between the two geophones

n is an integer which depends on the number of wavelengths between the geophones

As a number of geophones are utilised to record the ground surface response, a least squares fit may be used to determine the phase change with distance along the array of geophones. Some systems use only two geophones to detect the ground response. However using five stations to observe the ground response significantly improves the accuracy of the system as a larger number of data points allow the wave velocity to be determined with a higher level of confidence. In addition multiple stations allow judgement on whether the phase velocity is constant for the material under investigation. If the wave velocity is not constant it may indicate interference from other sources, wave reflections from nearby boundaries or other influences such as near field effects. Clearly the multiple station method which allows the variability of the wave velocity to be observed significantly increases the robustness of the test method.

Figure 4 shows the output of the five geophones with spacing of 0,5 m. The figure shows that the phase for each of the five geophones and the least squares fit conforms closely to that of a straight line indicating near constant velocity of the Rayleigh wave between the geophones.

The Rayleigh wave velocity is calculated as the product of the frequency (f) and the wavelength (λ):

$$V_r = f\lambda \tag{5}$$

Equations 4 and 5 allow the dispersion curve of the soil profile to be determined. The dispersion curve is the relationship between wave velocity and wavelength and is the fundamental seismic response 'signature' of the profile. It may be used for a number of applications such as vibration analysis of dynamically loaded foundations, earthquake response analysis of the ground, as well as to determine the stiffness profile of the ground. Figure 5 shows a typical dispersion curve.

At a depth of about half to one third of the wavelength both the vertical and horizontal components of the Rayleigh wave amplitude reaches a maximum and diminishes below this depth (Richart *et al* 1970). For this reason the simplifying assumption is often made that the effective depth of penetration of a Rayleigh wave is between half to one third of the wavelength (Gazetas 1982; Butcher & Powell 1996).

To determine the stiffness profile from the dispersion curve the bulk density (equation 1) and the Poisson's ratio (equation 2) of the material is required. Often these values are known, but if they are not known typical values may be assumed without greatly affecting the stiffness profile. For instance from equation 2 it may be seen that the ratio of Rayleigh wave velocity and shear wave velocity (V_r/V_s) is relatively insensitive to Poisson's ratio. For the typical range of Poisson's ratio for soils of between 0,2 and 0,5 the ratio (V_r/V_s) ranges between 0,91 and 0,95. In addition, equation 1 shows that the small strain shear stiffness is related to the bulk density (ρ) and the square of the shear wave velocity (V_s). As shear wave velocities are typically a few hundred meters per second, clearly the small strain shear stiffness is

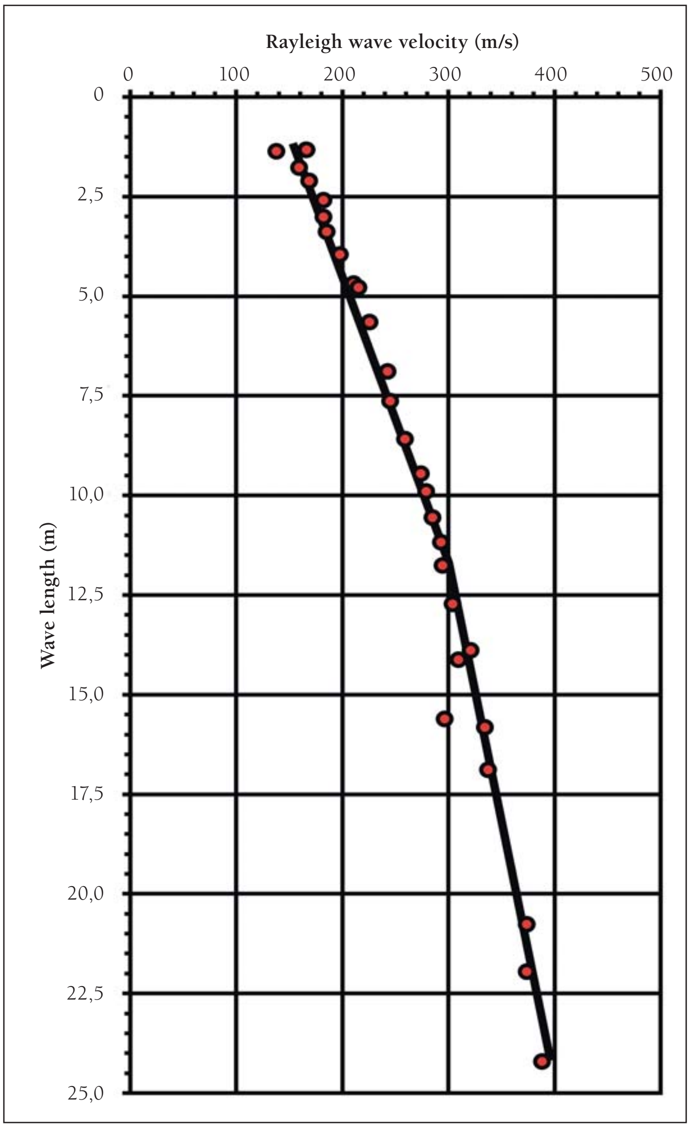

Figure 5 Dispersion curve

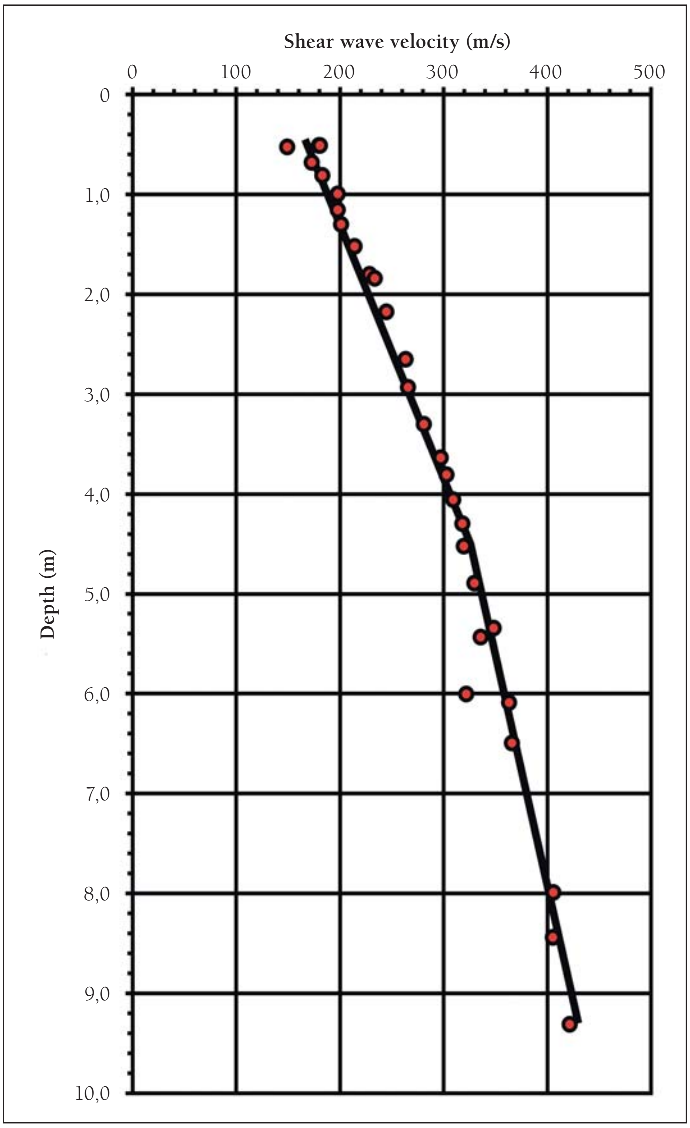

Figure 6 Shear wave velocity profile

much more sensitive to the shear wave velocity than the bulk stiffness.

Figure 5 showed a typical dispersion curve of a profile as the relationship between the Rayleigh wave velocity and the wavelength. Figure 6 shows the shear wave velocity profile obtained after inversion of the dispersion curve assuming a ratio of wavelength to depth of 2,6 and a Poisson ratio of 0,26. The small strain shear stiffness profile (G_0) of the material is shown in figure 7 using equation 1 and assuming a bulk density of 2 000 kg/m^3.

DISCUSSION

The technique of assuming the effective depth of penetration to be a fraction of the wavelength only allows an average stiffness to be determined for the material to a particular depth. More advanced analysis uses inversion techniques which matches the measured dispersion curve with an assumed layer model (Thompson 1950; Haskell 1953). However, mathematically this nonlinear inverse problem is ill-posed and therefore more than one solution may exist. For this reason knowledge of the layer thickness may be required to find a global solution to the problem. Various algorithms have been developed to search for the solution to this multi-dimensional problem. Some algorithms assume that only fundamental mode response of the soil occurs and this is an acceptable assumption for a layered profile where the stiffness of the layers increases with depth. This condition often occurs for natural sediments or soil profiles produced by in situ weathering. However, important exceptions to this type of profile include pavements where the stiffness of the pavement layers generally reduces with depth. Other examples include pedocretes such as ferruginous or calcareous material where the upper layers may be more stiff than the material below. For such profiles higher order modes may be generated (Tokimatsu *et al* 1992; Ryden *et al* 2004) and requires suitable algorithms which incorporate higher mode response (Lai & Rix 1999). The choice of inversion technique depends on the characteristics of the ground profile as well as the resource available for conducting the analysis.

Careful interpretation of the stiffness profile is required for engineering design applications. Soils typically exhibit highly nonlinear stress strain response at intermediate and large strain levels. Figure 8 shows the stiffness of three geomaterials with widely differing stiffnesses including soft clay, stiff clay and weak rock measured in a triaxial apparatus with local strain instrumentation (Heymann 1998). All three materials exhibit constant stiffness up to a strain level of approximately 0,002 %. The continuous surface wave test generally induces strain levels smaller than 0,002 % and the strain conditions are therefore in the linear-elastic range. For this reason the subscript 0 is used for the shear stiffness (G_0) in equation 1 to emphasize the fact that the continuous surface wave test measures the shear stiffness at very small strains. For dynamically loaded foundations this is the stiffness appropriate for design but it is not directly applicable for design of structures which induce larger strains such as static foundations, retaining structures, tunnels etc. To measure the stiffness of soils at intermediate and large strain levels other field or laboratory tests are required. Examples of such field tests include the plate load and pressure meter test. In the laboratory the triaxial test with the use of local strain instru-

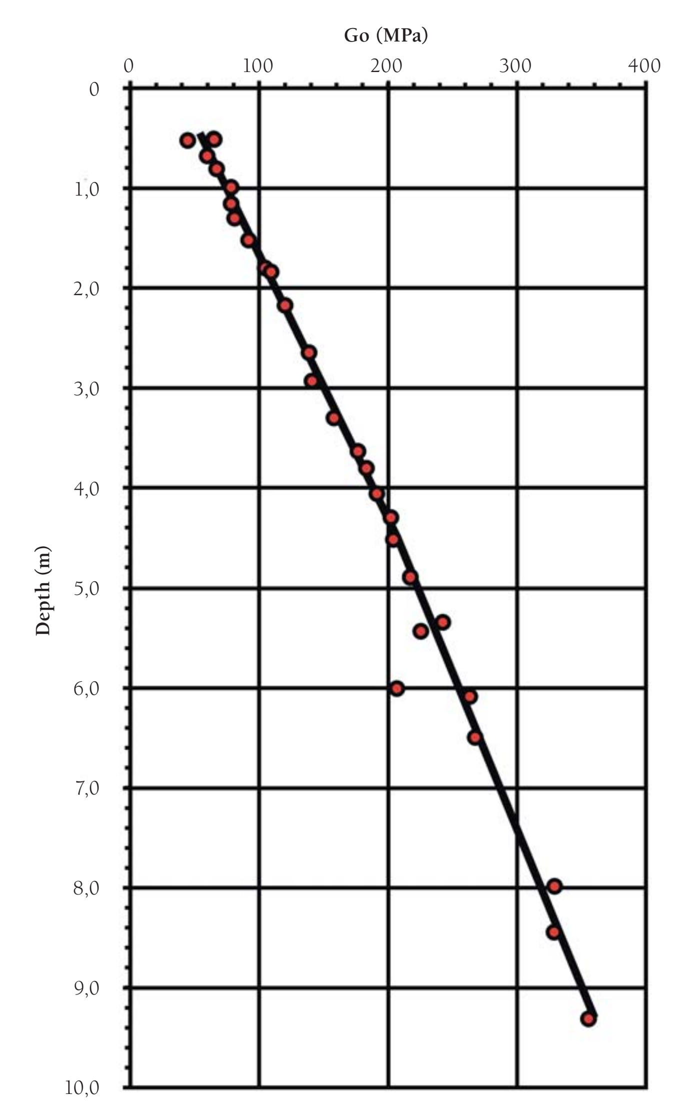

Figure 7 Stiffness profile

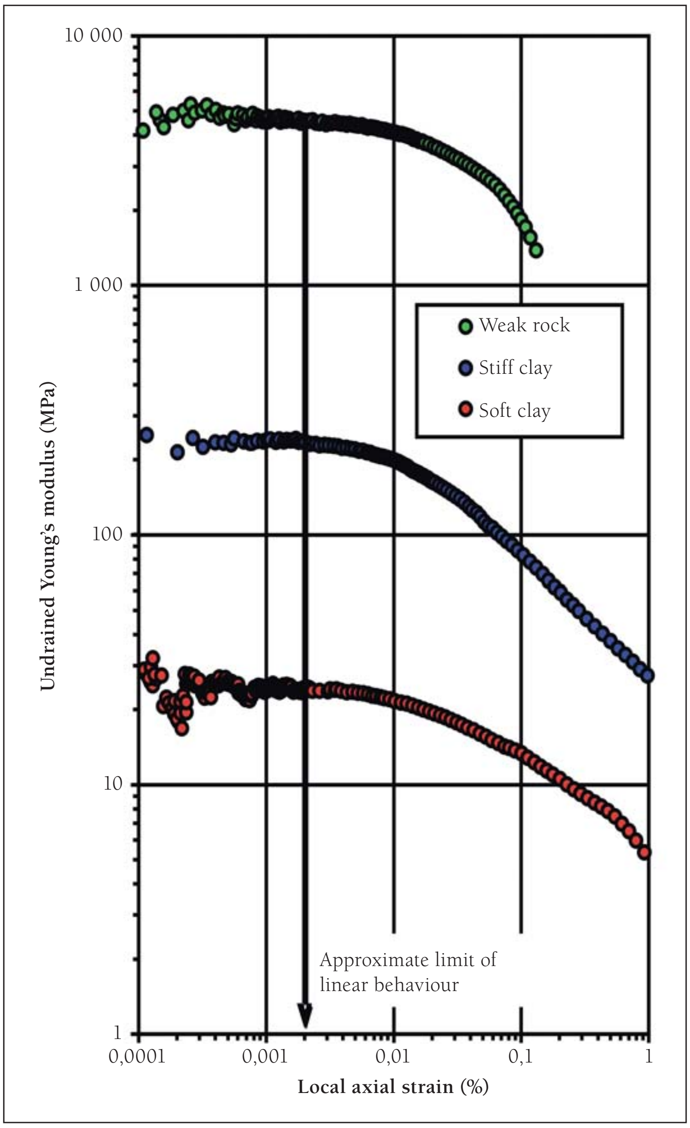

Figure 8 Geomaterial stiffness (Heymann 1998)

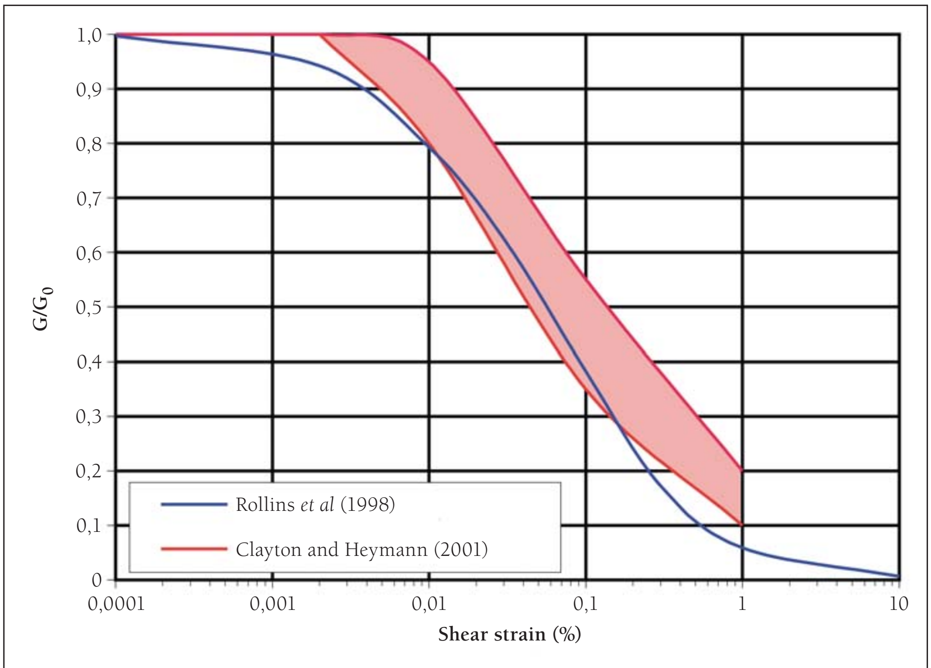

Figure 9 Softening functions for soils

mentation (Heymann 2000) may be used for this purpose.

As the strain level of soils increase the stiffness reduces but the rate at which the stiffness reduces is relatively independent of soil type (Clayton & Heymann 2001; Shibuya *et al* 2004). Vinale *et al* (2001) also showed the stiffness reduction rate of compacted soils to be independent of the compaction moisture content. Therefore, if the small strain stiffness of a soil is known, the stiffness at higher strain levels may be estimated from softening functions found in the literature.

Clayton and Heymann (2001) found the following stiffness values at intermediate strain levels from triaxial tests on undisturbed geomaterials which included soft clay, stiff clay and weak rock respectively:

$$0{,}80 \leq \frac{G_{0,01\%}}{G_0} \leq 0{,}95 \qquad (6a)$$

and

$$0{,}35 \leq \frac{G_{0,1\%}}{G_0} \leq 0{,}55 \qquad (6b)$$

and

$$0{,}10 \leq \frac{G_{1\%}}{G_0} \leq 0{,}20 \qquad (6c)$$

where the subscripts indicate the strain level and G_o is the shear stiffness at very small strains.

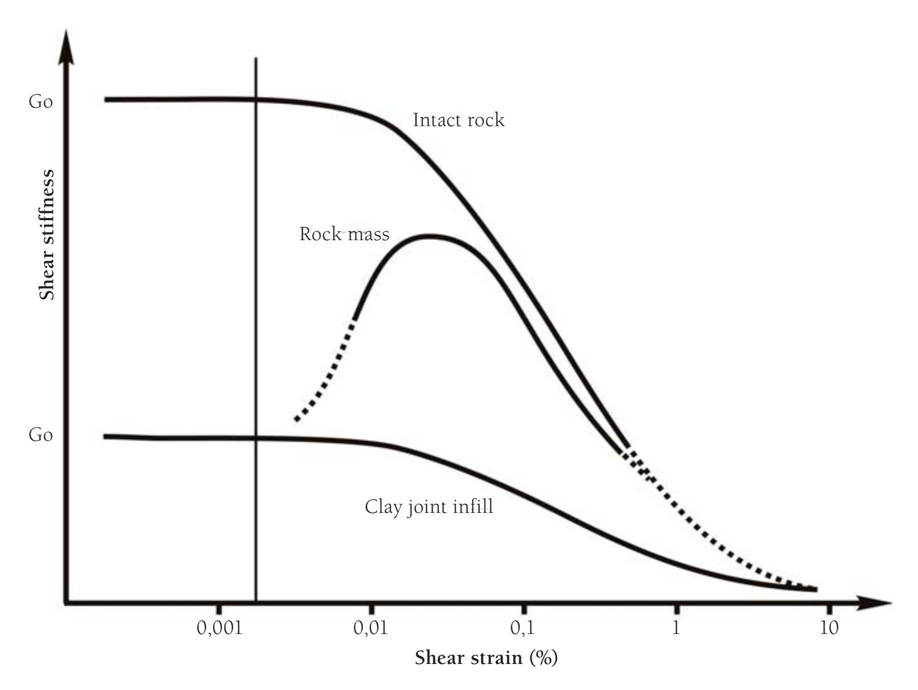

Figure 10 Idealised stiffness behaviour of jointed rock

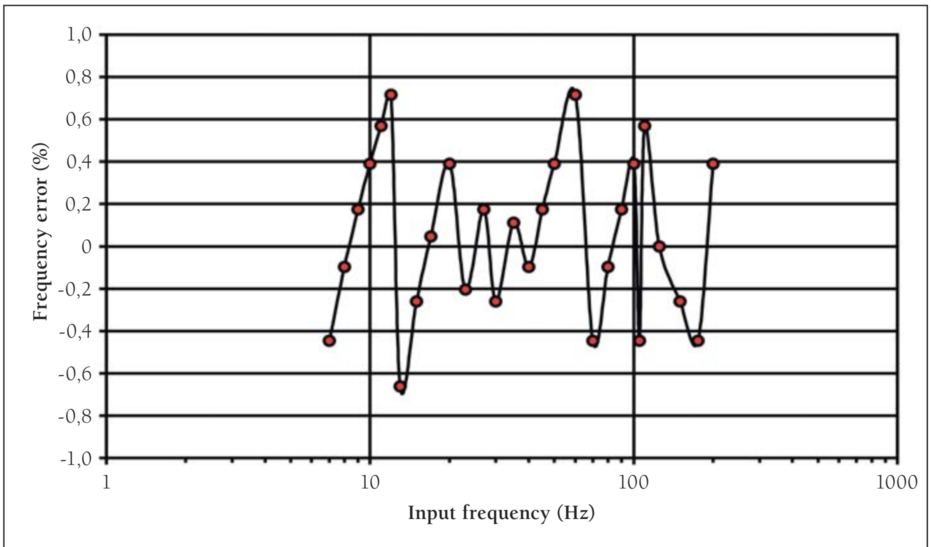

Figure 11 Frequency error of continuous surface wave system

Rollins *et al* (1998) observed the stress-strain response of sands and gravels and suggested the following softening function:

$$\frac{G}{G_o} = \frac{1}{\left[1+16\gamma\left(1+10^{-20\gamma}\right)\right]} \qquad (7)$$

Where:
γ is the shear strain

The softening functions from equations 6 and 7 are shown in figure 9.

Massarsch (2004) investigated the behaviour of sands and clays in the resonant column apparatus and concluded that the stiffness degradation depends inter alia on the plasticity index of the material. He found that the stiffness degradation with strain occurs more rapidly for soils with lower plasticity and suggested a relationship similar to equation 7, but included the plasticity index as a variable.

Caution has to be used when extrapolating continuous surface wave stiffness values to intermediate strain levels for jointed rock masses. At small strain levels the stiffness of the rock mass is dominated by the joint stiffness whilst at intermediate and large strain levels the stiffness of the rock mass is dominated by the stiffness of the intact material (material between the joints).

Clayton *et al* (1994) investigated a range for rock masses of the same origin with similar fracture spacing but different intact stiffnesses. They found that the range of mass stiffnesses, derived from continuous surface wave testing, did not vary significantly even though the intact stiffness of the rock varied by a factor of five. In addition they found the mass stiffness to be consistently lower than the intact stiffness. This indicates that the mass stiffness at very small strains is dominated by the joint stiffness and not the intact stiffness.

The author postulates an idealisation of the stiffness behaviour of a jointed rock mass with stiff intact material and soft clay as joint infill, as shown in figure 10. At very small strain levels the rock mass stiffness will be similar to that of the clay infill and at large strain levels the rock mass stiffness corresponds to that of the intact material. The intact material is an upper bound and the joint material a lower bound over the entire strain range of the rock mass. Figure 10 attempts to describe the stiffness of rock mass simplistically and ignores such effects as the wave length to block size ratio (Fratta & Santamarina 2002), loading direction (Matthews 1993), infill thickness, etc. However, figure 10 clearly illustrates the difficulty faced by the design engineer having to characterise the stiffness behaviour of a rock mass over the full strain range from very small strains to failure.

CONCLUSIONS

The continuous surface wave test measures the velocity at which Rayleigh waves are propagated along the surface of the ground. The depth to which the waves penetrate depends on the wavelength of the wave and by investigating the velocity at a range of wavelengths the dispersion curve of the profile may be determined. The dispersion curve may be used for various analyses including earthquake response analysis and predicting the behaviour of dynamic loaded foundations. The test is a non-destructive and non-intrusive test that allows ground stiffness profiles to be determined relatively quickly. In the author's experience the typical production rate is three to four profiles per day.

Some care is required to use the stiffness determined from the continuous surface wave test for design. For soils the stiffness at intermediate and large strain levels will always be less than the stiffness as measured by the continuous surface wave method and a 'softening function' such as shown in equations 6 and 7 will be required to estimate the appropriate stiffness for design. For jointed rock masses the stiffness as measured by the continuous surface wave test will be directly applicable for design under small strain conditions such as vibration analysis. However, at intermediate strain levels the design stiffness may be either greater or lesser than that measured by the continuous surface wave method depending on the relative stiffness of the joints and the intact material.

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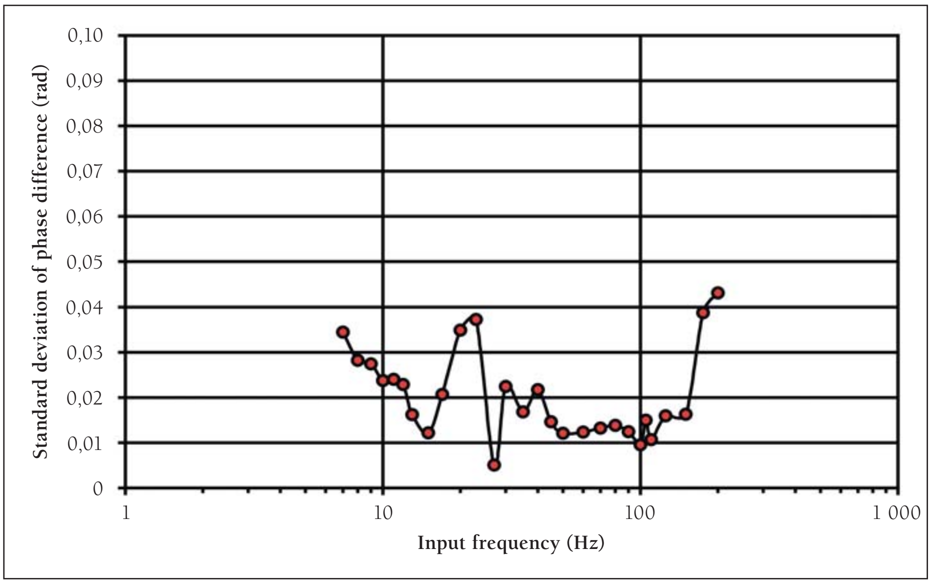

Figure 12 Phase difference between geophones during calibration

APPENDIX – SYSTEM CALIBRATION

The continuous surface wave system consist of various components including the measurement components (geophones), electronic components (amplifiers), logging components (data acquisition card) as well as the analysis component (FFT algorithm). Each one of these components may be calibrated individually; however, such calibrations will not necessarily demonstrate the accurate operation of the system as a whole. From equations 4 and 5 it may be seen that apart from the geophone spacing the system output required to determine the Rayleigh wave velocity consists only of the phase difference between the geophones and the dominant frequency detected by each geophone. The system as a whole can therefore be calibrated by evaluating the measurement accuracy of these two parameters.

The system as a whole was calibrated by attaching the five geophones to an electromagnetic shaker powered by a calibrated reference source with an accuracy of ±0,002 %. As the geophones were subjected to the same vibration, the response of the geophones should be identical. This implies that the phase difference between the geophones should be zero and the dominant frequency of all the geophones should be equal to that of the reference source.

Figures 11 and 12 show the calibration results. Figure 11 shows that for the frequency range used in the field of between 7 Hz and 200 Hz, the maximum error between the input frequency and the dominant frequency produced by the system was less than ±0,75 %. In addition the maximum standard deviation of the phase angles between the five geophones was 0,043 radians (2,5 degrees), as shown in figure 12. The error bars shown in figure 4 shows the 90 % confidence limits of the phase angle for each geophone. It indicates that the uncertainty of the measurements is small relative to the phase angle change between the geophones. These results confirm that the overall accuracy of the continuous surface wave system is good when viewed in the context of uncertainties inherent to the test such as the heterogeneity and anisotropy of the soil.

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