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The seismic cone test

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The seismic cone test is an in situ test used to measure body wave velocities in soils. Geotechnical parameters that may be derived from the test include void ratio, small strain stiffness and Poisson's ratio. In addition, results from the seismic cone test allows judgement to be made on the static and dynamic liquefaction potential of soils. The test makes use of a cone containing several geophones, which is hydraulically pushed into the soil under investigation. Seismic waves are generated at the surface and the time required for the waves to propagate through the soil to the cone is measured. The information is used to determine the velocities of different wave types in the material. This paper describes the seismic cone test. Field results are presented and parameters that may be derived from the results are discussed.

INTRODUCTION

Seismic tests measure the velocity at which elastic waves travel through soil. These wave velocities may be used to determine geotechnical properties of the material, such as void ratio, small strain stiffness and Poisson's ratio. Few other methods exist to determine these properties in the field and for cases where undisturbed samples cannot be taken, seismic tests may be the only method to determine these parameters. In addition, seismic wave velocities may be used to predict the behaviour of the material during static and dynamic loading.

Seismic wave velocity may be measured in the field by a variety of techniques. All have a source that generates waves and receivers to monitor the ground motion some distance away. The source and receivers may be placed either at the surface or in the ground, giving rise to different techniques. For up-hole, down-hole and cross-hole methods, the energy sources and geophones are installed in boreholes (Matthews *et al* 1997). For seismic refraction and reflection surveys, both the source and the receivers are at the ground surface (Clayton *et al* 1995).This configuration is similar to that used for the continuous surface wave (CSW) and spectral analysis of surface wave (SASW) techniques (Heymann 2001). The seismic cone test is typically used in soft ground where the energy source is at the surface and the geophones are installed inside a cone pushed into the ground. The cone is advanced in increments and the travel times of the waves between the surface and the geophones are measured at various cone depths.

A seismic cone has been built at the University of Pretoria and has been successfully implemented at a number of sites to determine the geotechnical properties of the subsurface materials. This paper describes the test equipment and test procedures used during the fieldwork and shows typical results determined from the field data.

SEISMIC WAVES

Energy imparted at the surface of a semi-infinite half space by way of an impact generates four wave types in the material. Of these wave types, two are surface waves (Rayleigh and Love waves) that are propagated along the surface of the material, and two are body waves (compression and shear waves) propagated within the semi-infinite medium. The seismic cone test measures shear and compression wave velocities only.

Compression waves

Compression waves are the fastest of the seismic waves. They are propagated as spherical fronts radiating from the seismic disturbance with the direction of particle motion being the same as that of wave propagation. Compression waves can be transmitted both by the soil skeleton and the pore water, and in water travel at a velocity of approximately 1 500 m/s (Clayton *et al* 1995). Consequently, for a saturated soil with a volumetric stiffness of the soil skeleton significantly less than that of water, the first-arriving compression waves will be propagated by the pore fluid and travel at a velocity of approximately 1 500 m/s. In unsaturated soils, however, the soil skeleton will propagate the compression waves, and the wave velocity may be faster than 1 500 m/s in a stiff medium and slower than 1500 m/s in a soft medium. This benchmark velocity allows differentiation between compression and shear waves that make up a single wave trace.

Shear waves

Shear waves also radiate from the seismic source as spherical fronts, but are transverse waves, with the direction of particle motion perpendicular to the direction of wave propagation. Particle motion may therefore have components parallel and perpendicular to the ground surface. Shear waves can only be propagated by the soil skeleton and not by the pore water on account of the negligible shear stiffness of water. As a result, shear waves are typically significantly slower than compression waves, particularly in a soft medium. In a very stiff medium, however, the shear wave velocity may be as high as 70 % of the compression wave velocity (Clayton *et*

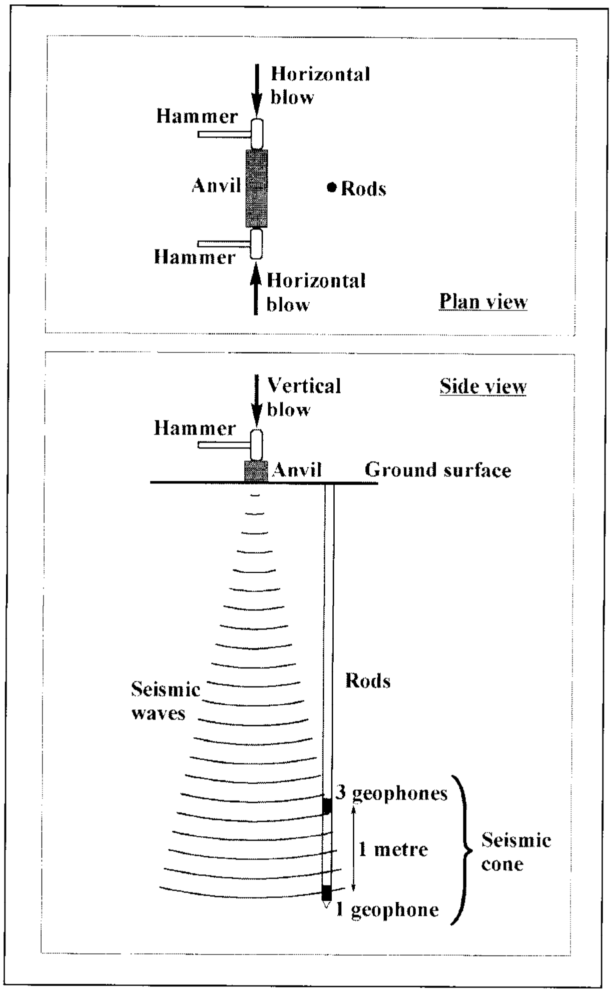

Figure 1 Layout of the seismic cone penetration tests

al 1995). In addition, shear waves have the attribute of being 'reversible'. This means that if the polarity of the seismic event is reversed, the phase angle of the shear waves will change by 180 degrees.

THE SEISMIC CONE TEST

Various sources are used to generate the seismic event at or near the ground surface. These include hammers and explosives. The source should be tailored to produce a seismic event rich in the particular wave of interest (either shear or compression waves), and it is advantageous if the polarisation of the shear wave can be reversed. This significantly increases the confidence with which a shear wave can be identified in a recorded trace that contains different wave types.

Figure 1 shows the arrangement of the seismic cone test. The seismic source consists of an anvil in the form of a wooden sleeper tied to the ground by an auger-type anchor. The sleeper is positioned as close as possible to the position of the hole, without interfering with the hydraulic probe rig. Hitting the sleeper with a heavy hammer generates the seismic energy. A seismic event rich in compression waves may be produced by hitting the sleeper vertically downwards. In contrast, seismic events dominated by shear waves may be generated by hitting the sleeper horizontally from the side, and shear waves with reverse polarity, by hitting the sleeper from the opposite side.

The 15 cm^2 seismic cone contains four geophones positioned at two levels inside the cone – three near the top of the cone and one near the bottom at a distance of one metre below the top phones. One of the top geophones is placed vertically, and two are positioned horizontally, orientated perpendicular to one another. The bottom geophone is placed horizontally and parallel to one of the top geophones. The orientation of the vertical geophone maximises its sensitivity to detect compression waves, as the direction and particle motion of compression waves are near vertical for the test arrangement shown in figure 1. The three horizontal geophones are orientated to maximise their sensitivity to shear waves for which the particle motion is parallel to the ground surface.

The outputs from all four geophones are amplified by a set of eight amplifiers with a response time of 2 μs/volt. The fast response time is required in order to minimise signal delay. Each geophone output is amplified both ten and one hundred times and the information is recorded in digital format by a data acquisition system. The multiplexed system records the seismic event at a frequency of 333 kHz.

The ten and one hundred times dual amplification of each channel allows the most suitable trace to be used for analysis. At shallow depths the seismic energy reaching the geophones may be sufficiently large to saturate the channel amplified one hundred times, leading to clipping of the signal. In such instances the data amplified ten times will be more suitable for analysis.

An external geophone is fixed to the sleeper at the ground surface to record the start time of the seismic event and initiate data logging. Once the data acquisition system is armed, data is streamed to a continuous buffer. Hitting the sleeper generates an output from the external geophone that triggers the logging event. A pre-triggering facility is used to capture data from before the seismic event up to a specified time period after the seismic event. This period should be sufficiently long to capture the entire wave trace of interest.

Prior to insertion, the seismic cone is orientated such that two horizontal geophones are positioned parallel to the direction of the wooden sleeper. This ensures maximum sensitivity of these geophones to the shear wave event generated by sideway blows to the sleeper. The third horizontal geophone is perpendicular to these two geophones, and if the cone rotates about the vertical axis during penetration, it will have an orientation sensitive to shear wave events, ensuring high output regardless of cone orientation.

The cone is pushed into the ground in increments of one metre. After penetrating a metre the probe rig motor is shut down and three tests are performed – one with a hammer blow vertically down onto the sleeper and two with hammer blows horizontally on opposite sides of the sleeper. For each test the response of the geophones is digitally recorded. Penetration of the seismic cone is continued until the required depth is reached or refusal of the cone occurs.

SEISMIC WAVE VELOCITY MEASUREMENT

The first objective of wave trace analysis is to differentiate between compression wave and shear wave first arrivals at the geophones. Compression and shear wave events may be distinguished by considering the following aspects:

- In soft, saturated soils compression waves travel faster than shear waves: the compression waves travel at approximately 1 500 m/s and shear waves typically between 100 m/s and 300 m/s.
- The polarisation of shear waves is reversible, whereas compression waves cannot be reversed.
- The amplitude of a particular wave type will depend on the dominant wave type produced by the seismic source.
- The frequency response of a compression wave is typically much higher than that of a shear wave (Clayton *et al* 1995). However, the frequency response depends not only on the wave type but also on the physical properties of the cone, in particular its inertia.

Figure 2 shows a typical shear wave trace and the reversal. For the first trace the wooden sleeper was hit from the one side. The first arrival of the shear wave event can be identified at 14 milliseconds after the time of impact. For the second trace the cone was in the same position, but the sleeper was impacted from the opposite side. Clearly the time history recorded by the geophone was similar for both events, but the polarity of the response was reversed.

Inaccuracies in determining seismic wave velocity during field measurements may arise from a number of causes (Matthews *et al* 2000). These include:

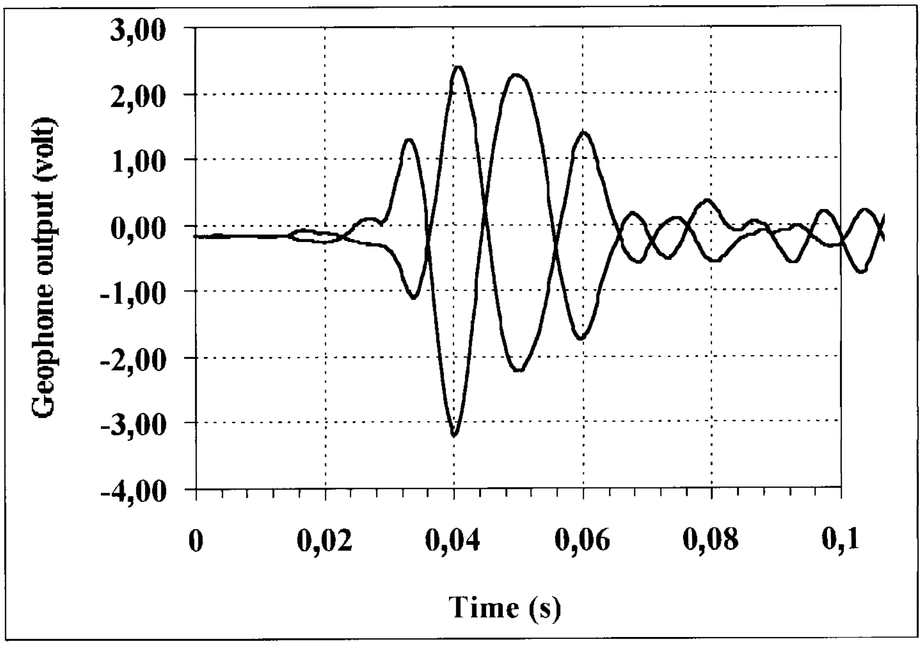

Figure 2 Reversal of a shear wave trace

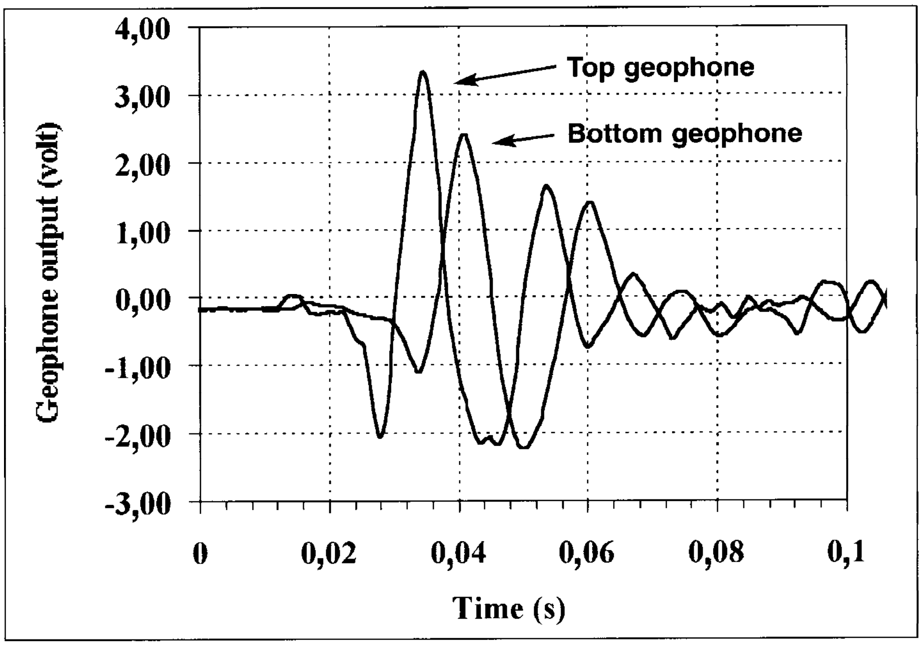

Figure 3 Shear wave arrival at the top and bottom geophones

- selecting the wrong shear or compression wave event through misinterpretation
- masking the onset of a wave event by background noise
- timing errors when triggering the logging device
- poor time resolution of the measured data
- uncertainties in the wave path length.

Selecting the wrong shear or compression wave event through misinterpretation could lead to large errors in determining the shear or compression wave velocity. Such errors are generally easy to detect. The other causes of inaccuracies will result in less significant errors in the measured wave velocities and are often more difficult to detect, but good field and interpretation techniques may readily overcome these difficulties. Some of these techniques are discussed below.

Trigger timing errors are systematic errors that occur when a disparity exists between the moment taken as zero time and the initiation of the seismic event. Systems utilising a switch mechanism to activate the logging system are particularly prone to trigger timing errors. It is often difficult to judge whether measurements contain trigger timing errors, but techniques exist by which trigger timing errors may be minimised. These include pre-triggering and dual receivers.

Pre-triggering utilises a data buffer that records data prior to the seismic event and thus allows data to be stored prior to the seismic event. The onset of the seismic event can then be identified from the captured data and no reliance is made on a switch to establish zero time. A logging system with a pre-trigger facility greatly reduces the likelihood of trigger timing errors.

A two-level seismic cone having a dual geophone configuration (see figure 1), where two geophones detect the same seismic event but are placed some distance apart, may be used to eliminate the effect of trigger timing errors. As the wave will arrive at the top geophone some time before it reaches the bottom one, the travel time between the geophones may be used in conjunction with the ray path length to determine the wave velocity. Any trigger timing error will be excluded from the measurement. Figure 3 shows the response of two horizontal geophones for the same seismic event. It may be observed that a time offset of approximately 7 milliseconds exists between the geophones, representing the travel time of the shear wave between the two geophones one metre apart.

Field results

Figure 4 shows typical shear wave velocity traces recorded for a single hole at increasing depths in a gold tailings dam. The figure shows consistency in the time history of the wave events occurring at each depth. From the data the average wave velocity of a profile may be measured, as well as the wave velocity at a particular depth in the profile. The average wave velocity is measured using the time required for the wave to travel from the source to the cone, taking the distance between the source and the geophone as the ray path length.

The wave velocity 'at depth' may also be determined by using a subtraction technique, whereby the travel time to a particular depth is subtracted from the travel time to another depth and the assumption is made that the difference in travel times represents the travel time between the two depths. However, this technique, is susceptible to trigger timing errors. To determine the wave velocity at a certain depth in the profile using a two-level dual seismic cone, the cone is advanced to the depth in question and the travel time of the wave between the top and the bottom geophones is measured for the same wave event by comparing the response of the two geophones (see figure 3). The time difference of the first arrivals may be used to determine the travel time. However, determining the exact time of each first arrival is often difficult as 'lift-off' of the trace that signifies the first arrival is often not well defined. More precise methods to determine the time offset between the two geophones include the comparison of the first reversals as well as the cross-correlation technique (Baziw 1993), which compares the time history of both complete traces. A typical profile of shear wave velocity with depth is shown in figure 5 using cross-correlation. Four tests were conducted at each depth, as recorded by the two horizontal geophones for each of the two horizontal blows. Figure 5 shows the average and range of the four tests at each depth.

A number of workers have demonstrated that the seismic cone test provides wave velocities similar to those measured by other methods such as cross-hole and surface wave methods (see for example Campanella *et al* 1986 and Eidsmoen *et al* 1985). Butcher and Powell (1996) demonstrated for anisotropic soil that, as expected, differences in wave velocities may occur between methods if waves with different direction of propagation or different direction of particle motion are employed.

ENGINEERING PARAMETERS FROM THE SEISMIC CONE TEST

Field seismic tests require no sampling of the soil and material parameters are determined at in situ conditions. These tests are therefore well suited to measure the properties of materials that are sensitive to sampling disturbance. In engineering practice, seismic wave velocities may be used both as a direct and an indirect method to obtain information of value to the engineer. These include:

- applying empirical relationships between wave velocity and an engineering parameter of interest such as void ratio
- using theoretical relationships between wave velocity and elastic parameters such as stiffness and Poisson's ratio
- measuring the dynamic properties of the soil when investigating a dynamic soil-structure interaction problem such as a foundation subjected to vibrations (Richart *et al* 1970)
- using observed field performance to judge the susceptibility of soils for dynamic liquefaction (Andrus & Stokoe 2000) or static liquefaction (Robertson *et al* 1995 and Papageorgiou *et al* 1999)

Examples of using theoretical and empirical relationships to obtain engineering parameters are discussed below.

Void ratio

Hardin and Drnevich (1972) used the resonant column to investigate the relationship between shear wave velocity and numerous soil parameters for a wide range of soils including sands, silts and clays. They found that shear wave velocity was highly dependent on the void ratio and mean effective stress of the soil but less influenced by factors such as degree of saturation, over-consolidation ratio, shear stress level, thixotropy, soil structure, as well as the grain size, shape, gradation and mineralogy. From these findings it follows that if the influence of stress can be accounted for by normalisation, then a direct relationship should exist between void ratio and shear wave velocity. Robertson and Fear (1995) used empirical data for a number of sands and suggested that the void ratio of sand may be estimated from the normalised wave velocity using the linear relationship:

$$V_{s1} = (A - Be)K_o^{0,125} \qquad (1)$$

Where:

V_{s1} = normalised shear wave velocity ($V_{s1} = V_s(P_a/\sigma'_{vo})^{0.25}$)
V_s = shear wave velocity
P_a = atmospheric pressure
σ'_{vo} = vertical effective stress
e = void ratio
K_o = coefficient of earth pressure at rest
A and B = material constants in m/s

Robertson *et al* (1995) suggested values of 381 m/s for A and 259 m/s for B to be appropriate for Ottawa sand. These values in association with equation 1 are superimposed on the measured data for a number of sands shown in figure 6. It indicates that for a K_0 of 1,0, equation 1 gives a reasonable estimation of void ratio for all the sand types using the suggested values for A and B for void ratios less than 0,9. However, it becomes unreliable for void ratios in excess of 0,9, and clearly a non-linear relationship between normalised shear wave velocity and void ratio may be more appropriate.

Hardin and Drnevich (1972) did not investigate the effect of interparticle bonding or cementation on shear wave velocity. However, Stokoe *et al* (1995) observed an increase in shear wave velocity with age. Also, current state-of-the-art methods to quantify dynamic liquefaction potential, using shear wave velocity, recognise the effect of interparticle bonding on shear wave velocity (Andrus & Stokoe 2000). Further research is required to quantify the effect of bonding on shear wave velocity, and the relationship shown in equation 1 should therefore be regarded as applicable for uncemented sands only.

Figure 7 shows the void ratio profile calculated using equation 1 for the gold tailings dam for which the wave velocities were presented in figure 5. It may be observed that the void ratio ranges between approximately 0,6 and 0,8 and that some reduction of void ratio is evident with depth. Direct comparison of field void ratio and field shear wave velocity are particularly difficult for hydraulically placed tailings dams, because it is difficult to obtain undisturbed samples from below the water table. However, one undisturbed block sample was taken from the daywall of the tailings dam. The material was

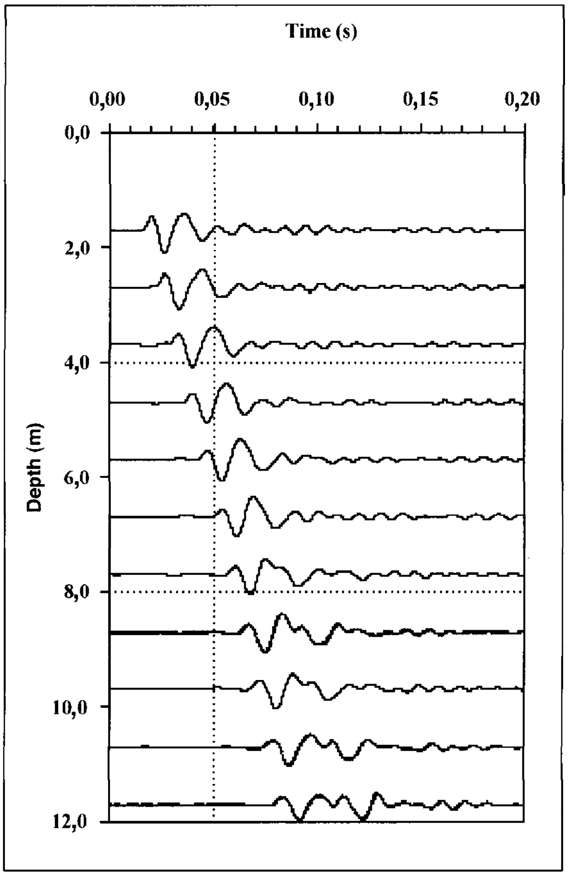

Figure 4 Typical shear wave trace profile for a gold tailings dam

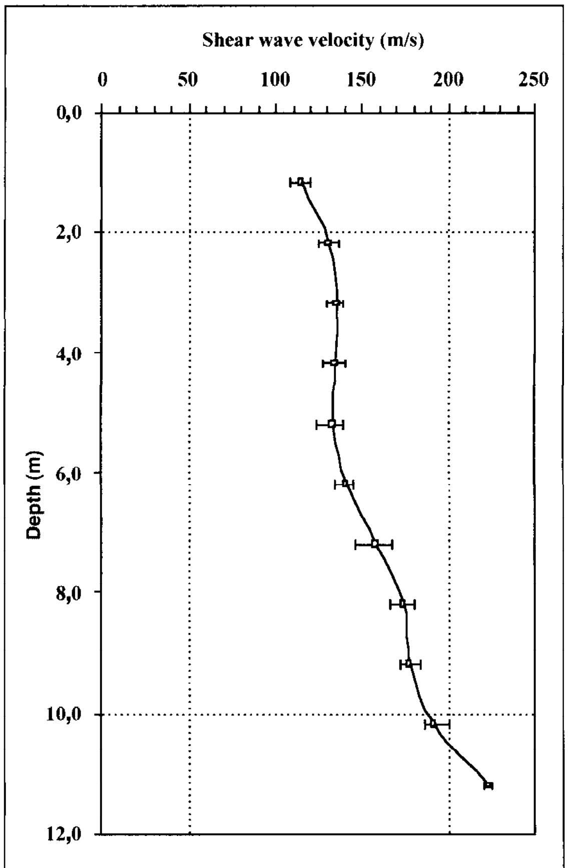

Figure 5 Typical shear wave velocity profile for a gold tailings dam

highly layered, as is typical for hydraulically placed tailings, and the void ratio of four individually cut specimens ranged between 0,65 and 0,77. This compares well with the void ratios shown in figure 7 giving confidence in using the seismic cone to determine void ratio of young uncemented geomaterials.

Soil stiffness

It can be shown from elasticity theory that the shear stiffness (G_o) of a homogeneous, isotropic linear elastic continuum may be expressed in terms of its mass density (ρ) and the velocity at which shear waves (V_s) are propagated through the medium:

$$G_o = \rho V_s^2 \qquad (2)$$

In addition, the constrained stiffness at very small strain (M_o) may be determined from the compression wave velocity (V_p) and mass density of the material (ρ) as:

$$M_o = \rho V_p^2 \qquad (3)$$

From the shear and constrained stiffness, other elastic parameters such as Young's modulus (E_o) and volumetric stiffness (K) may be determined:

$$E_o = 2G_o(1+\nu) \qquad (4)$$

and:

$$K = \frac{1}{3}M_o\left(\frac{1+\nu}{1-\nu}\right) \qquad (5)$$

Where: ν is the Poisson's ratio

It should be noted that the stiffnesses shown in equations 2 to 5 are only appropriate under small strain conditions. Such conditions often occur during dynamic loading from vibrating machines, transportation and construction equipment, wave action etc. However, for static loading conditions the stiffness depends on the strain level of the soil. Figure 8 shows a schematic representation of the stiffness degradation of a soil as it is sheared (Mair 1993). The figure show that for soil the stiffness at small strains (G_0) is higher than the stiffness required for design of statically loaded structures. Clayton and Heymann (2001) presented data to demonstrate that geomaterials with widely differing stiffnesses, ranging from soft clay to weak rock, all have a linear stress-strain response up to a strain level of approximately 0,002 %. They also showed that the rate of stiffness degradation is similar for materials with varying stiffness. The operational stiffness at 0,01 % strain typically varies from 80 % to 95 % of the maximum stiffness, and from 35 % to 55 % at 0,1 % strain. Therefore, stiffnesses determined using seismic wave velocity, when used in conjunction with the stiffness degradation characteristics, provide valuable information to the design engineer. In fact, a number of authors have demonstrated that if soil stiffness is measured accurately, the displacement of structures may be predicted with a high degree of certainty (Jardine *et al* 1991, and Mayne 2000).

Poisson's ratio

Poisson's ratio is defined as the ratio of strain perpendicular to the loading direction to strain parallel to the loading direction. When considering Poisson's ratio for a soil it is useful to distinguish between drained and undrained Poisson's ratio. If a fully saturated soil, specimen undergoes distortion without water being allowed to enter or leave the specimen, the material is said to be loaded under undrained conditions. For such conditions the volume of the specimen remains constant, and by definition the Poisson's ratio is 0,5. Whether undrained conditions will occur in practice depends on the loading rate relative to the time required for excess pore pressure to dissipate. In practice, undrained conditions often occur in clays under engineering loading, but rarely occur in sands as excess pore pressures generally dissipate quickly. Undrained conditions may, however, occur in sands during dynamic loading such as earthquake events or seismic field tests.

The small strain Poisson's ratio of soil may be determined from seismic field tests if both the shear wave and compression wave velocity are measured. Equation 6 shows the relationship between Poisson's ratio and the shear and compression wave velocities (Abbiss 1981).

$$\nu = \frac{\frac{1}{2}\left(\frac{V_p}{V_s}\right)^2 - 1}{\left(\frac{V_p}{V_s}\right)^2 - 1} \qquad (6)$$

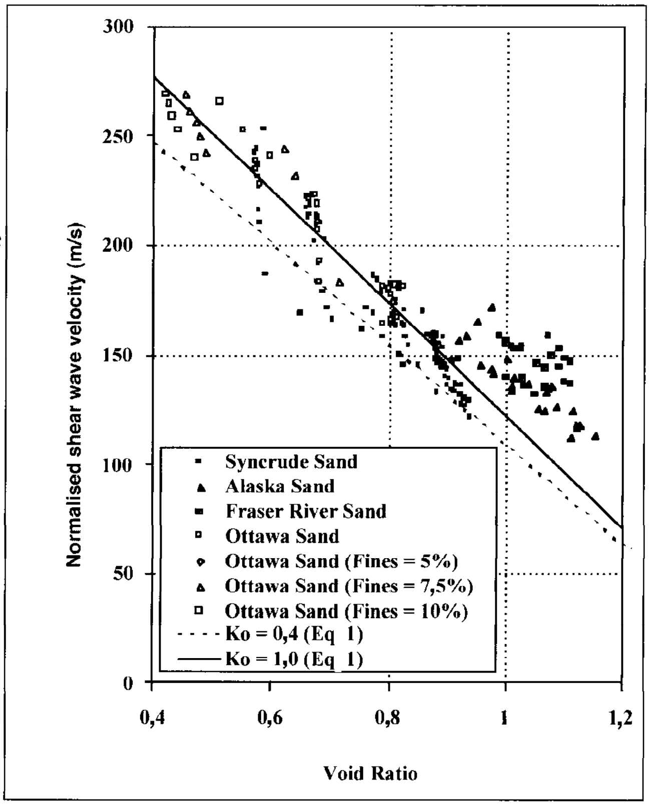

Figure 6 Relationship between normalised shear wave velocity and void ratio for a number of sands (from Robertson & Fear 1995)

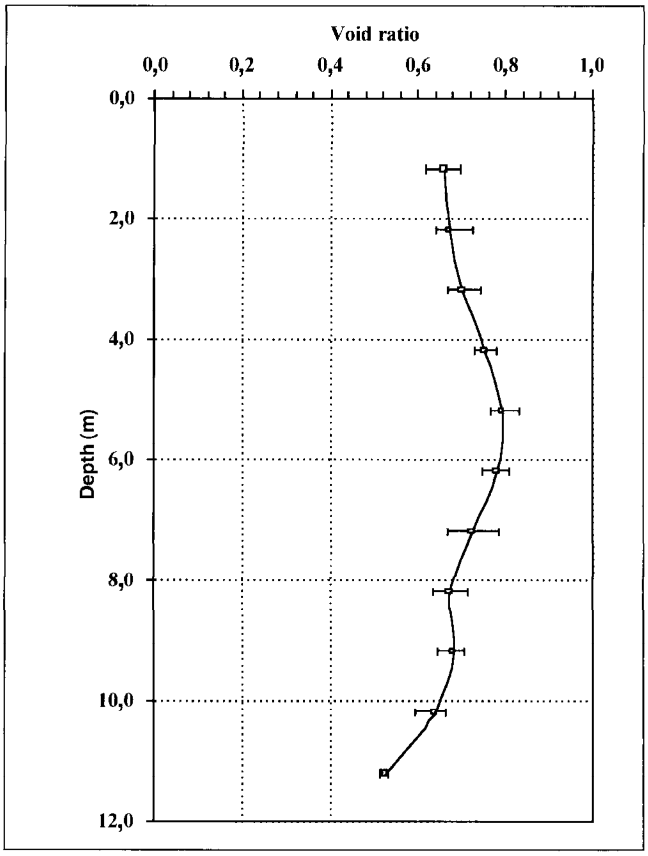

Figure 7 Typical void ratio profile for a gold tailings dam

From equation 6 it is clear that for soil subjected to undrained conditions, where the compression wave velocity is significantly in excess of the shear wave velocity, the Poisson's ratio will approach the expected value of 0,5.

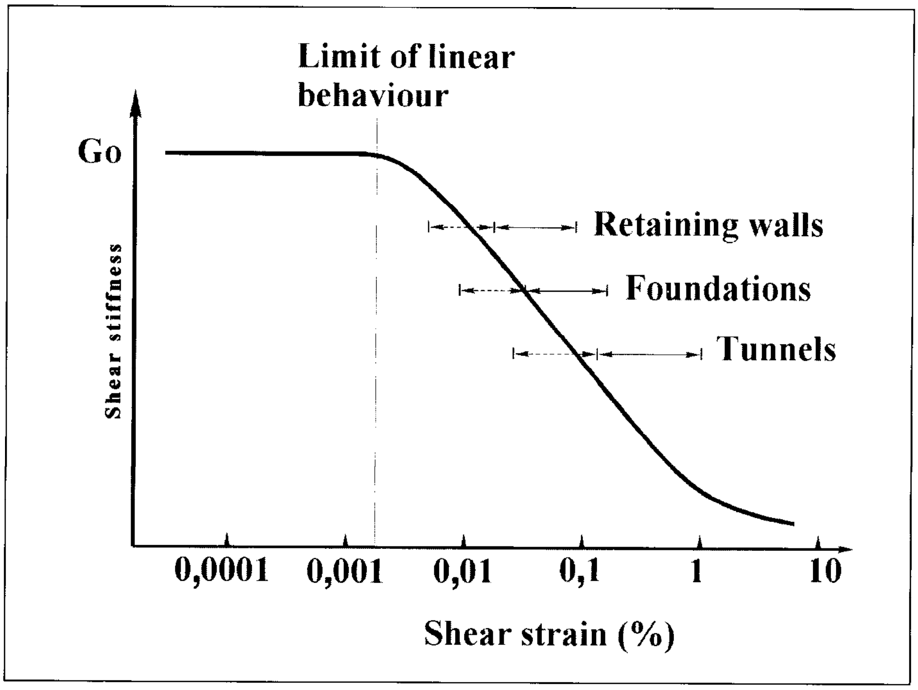

Figure 8 Shear stiffness reduction with strain (Mair 1993)

CONCLUSIONS

The seismic cone is used to measure the in situ shear and compression wave velocities in soils. These wave velocities may be used to obtain valuable information regarding the behaviour of the soil. Perhaps one of the greatest advantages of the seismic cone is that it allows measurement of soil parameters, such as void ratio on undisturbed material at the in situ stress condition. In the past, direct measurement of these parameters in materials sensitive to sampling disturbance has proved to be difficult, if not impossible. Field data was presented to show the void ratio profile for a gold tailings dam. The void ratio as determined with the seismic cone compared well with that of undisturbed samples taken from the daywall of the same dam. The ability to measure in situ void ratio of mine tailings in the field, below the water table, using the seismic cone is an exciting development, as few other techniques exist to determine the void ratio under these conditions.

Wave velocity measurement may also be used in conjunction with elasticity theory to determine small strain material stiffness and Poisson's ratio. Stiffness determined from wave velocity measurements may be used directly in cases where small strains occur, such as for foundations subjected to vibrations. At higher strain levels, if stiffness degradation is taken into account, the stiffness appropriate for engineering design may be derived. In addition, judgement on the susceptibility for static or dynamic liquefaction of soils may be made from wave velocities measured by the seismic cone.

The seismic cone is an exciting development that provides a valuable tool for practising engineers and its use may lead to improved and more cost-effective design for a wide variety of engineering structures.

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References

Abbiss, C P 1981. Shear wave measurements of the elasticity of the ground. *Géotechnique*, 31(1):91–104.

Andrus, R D & Stokoe, K H 2000. Liquefaction resistance of soils from shear wave velocity. *Journal of Geotechnical and Geoenvironmental Engineering*, 126(11):1015–1025.

Baziw, E J 1993. Digital filtering techniques for interpreting seismic cone data. *Journal of Geotechnical Engineering*, 119(6)998–1018.

Butcher, A P & Powell, J J M 1996. Practical considerations for field geophysical techniques used to assess ground stiffness. In *Advances in site investigation practice*. London: Thomas Telford, pp 701–714.

Campanella, R G, Robertson, P K & Gillespie, D 1986. A seismic cone penetrometer for offshore applications. *Proceedings* of the Oceanology International '86 international conference: Advances in Underwater Technology, Ocean Science and Offshore Engineering, Brighton, UK, 6, chapter 51.

Clayton, C R I & Heymann, G 2001. The stiffness of geomaterials at very small strains. *Géotechnique*, 51(3):245–256.

Clayton, C R I, Matthews, C *and* Simons, N E 1995. *Site investigation*. 2nd edition. Oxford: Blackwell Science.

Deresiewicz, H 1974. Bodies in contact with applications in granular media. In G Herrmann (ed), *R D Mindlin and applied mechanics*. New York: Pergamon Press, pp 105-147.

Duffy, J & Mindlin, R D 1957. Stress-strain relations and vibrations of a granular medium. *Journal of Applied Mechanics*, ASME, 24:585–593.

Eidsmoen, T, Gillespie, D, Lunne, T & Campanella, R G 1985. Tests with UBC seismic cone at three Norwegian research sites. Norwegian Geotechnical Institute, Oslo, Norway. Report 59040-1.

Hardin, B O & Drnevich, V P 1972. Shear modulus and damping in soils: design equations and curves. *Journal of the Soil Mechanics and Foundations Division*, ASCE, 98(7):667–692.

Heymann, G 2001. Seismic surface wave methods to measure soil stiffness profiles. South African Institution of Civil Engineers, Geotechnical Division: Ground Improvement Seminar. Johannesburg, October 2001.

Jardine, R J, Potts, D M, St John, H D and Hight, D W 1991. Some practical applications of a non-linear ground model. *Proceedings* of the 10th European Conference of Soil Mechanics and Foundation Engineering, Florence, vol 1:223–228.

Johnson, K L 1985. *Contact mechanics*. Cambridge: University Press.

Mair, R J 1993. Developments in geotechnical engineering research: application to tunnels and deep excavation. Unwin Memorial Lecture. *Proceedings* of the Institution of Civil Engineers, Civil Engineering, 93:27–41.

Matthews, M C, Clayton, C R I and Own, Y 2000. The use of field geophysical techniques to determine geotechnical stiffness parameters. *Proceedings* of the Institution of Civil Engineers, Geotechnical Engineering, 143:31–42.

Matthews, M C, Hope, V S & Clayton, C R I 1997. *The geotechnical value of ground stiffness determined using seismic methods. Modern geophysics in engineering geology*. Engineering Geology Special Publication No 12:113–123. London: Geological Society.

Mayne, P W 2000. Enhanced geotechnical site characterization by seismic piezocone penetration tests. Fourth International Geotechnical Conference, Cairo University, pp 95–120.

Papageorgiou, G, Fourie, A B & Blight, G E 1999. Static liquefaction of Merriespruit gold tailings. In G R Wardle *et al* (eds), Geotechnics for developing Africa. *Proceedings* of the 12th African Regional Conference of the ISSMGE. Rotterdam: Balkema.

Richart, F E Jr, Hall, J R Jr & Woods, R D 1970. *Vibration of soils and foundations*. Englewood Cliffs, NJ: Prentice-Hall.

Robertson, P K & Fear, C E 1995. Application of CPT to evaluate liquefaction potential. *CPT '95*, 3:57–79. Swedish Geotechnical Society.

Robertson, P K, Sasitharan, S, Cunning, J C & Sego, D C 1995. Shear wave velocity to evaluate in situ state of Ottawa sand. *Journal of Geotechnical Engineering*, 121(3):262–273.

Stokoe, K H, Hwang, S K, Lee, J N K & Andrus, R D 1995. Effects of various parameters on the stiffness and damping of soils at small to medium strains. *Proceedings* of the First International Conference on Pre-failure Deformation Characteristics of Geomaterials, vol 2:785–816. Rotterdam: Balkema.