The application of shear wave velocity to soft ground site investigation in Ireland

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ABSTRACT: Shear wave velocity (V_s) may be determined from a number of seismic geophysical techniques. One of these techniques the Multichannel Analysis of Surface Waves (MASW) method has been successfully applied by the authors over the last number of years to site investigation in Ireland. This paper will focus on the applications of shear wave velocity, measured using the MASW method, to soft ground site investigation. Applications discussed here include the determination of subsurface stiffness profiles at sites in Athlone and Portumna, sample disturbance assessment at a site in Ballinasloe, monitoring ground improvement works at a site in Belfast and finally it will be shown that 2D profiles of V_s may be generated to determine the lateral variation of subsurface materials.

1 INTRODUCTION

Geophysical techniques, and in particular seismic methods, have received considerable attention in civil engineering over recent years, their role steadily increasing to the point where they play an important part in site and material characterisation. This popularity likely arises from recent advances in both computational power and the geophysical techniques themselves. Furthermore, many geophysical methods are non-invasive which make them well suited and cost effective in profiling spatially and temporally.

From a geotechnical engineering perspective the most popular geophysical techniques are seismic methods, presumably because they may directly measure a mechanical property, soil or rock stiffness. Seismic testing consists of monitoring seismic waves that occur either naturally or artificially from a source inside or on the surface of the earth. Seismic waves that are most frequently used are body (compression (P) and shear (S) waves) and one type of surface wave, the Raleigh wave. This paper will focus on the applications of one seismic parameter, the shear wave velocity (V_s), to soft ground site investigation in Ireland.

Several techniques are commonly used to measure V_s in both the field or in the laboratory. Intrusive field methods include cross-hole, down-hole and seismic cone methods. In these surveys seismic sources and receivers are located either between boreholes or between the surface and a point in a borehole or cone. Non intrusive field methods used to determine V_s include seismic reflection and refraction and surface wave surveys.

Laboratory methods used to compute V_s include the resonant column method and the bender element method where a shear wave is transmitted using a piezoceramic element from the top of the soil specimen and recorded with another piezoceramic element at the bottom. In this paper the Multichannel Analysis of Surface Waves (MASW) method was used for each of the sites discussed. This technique has been successfully applied by the authors to a range of materials and ground conditions over the last number of years (Donohue *et al*. 2003, Donohue *et al*. 2004, Long and Donohue 2007, Donohue and Long 2007).

2 SURFACE WAVE GEOPHYSICS

In geotechnical engineering the most widely used surface waves are Raleigh waves which travel along the earth-air interface with a retrograde elliptical particle motion. Surface waves are very easy to detect as approximately two thirds of the total energy from a vertical point source on the surface propagates in the form of Raleigh waves (Miller and Pursey, 1955).

A Raleigh wave that propagates along the surface of a uniform, homogeneous elastic half space will travel at a velocity that is independent of its wavelength (or frequency). If however the media in question is non-uniform, the propagation velocity of a Raleigh wave is dependent on the wavelength (or frequency) of that wave by:

$$\lambda = \frac{V_r}{f} \qquad (1)$$

where λ is the wavelength and f is the frequency of the Raleigh wave in question.

Raleigh waves with short wavelengths (or high frequencies) will be influenced by material closer to the surface than Raleigh waves with longer wavelengths (or low frequencies), which reflect properties of deeper material. This is illustrated in Figure 1. This dependence of velocity on frequency is called dispersion.

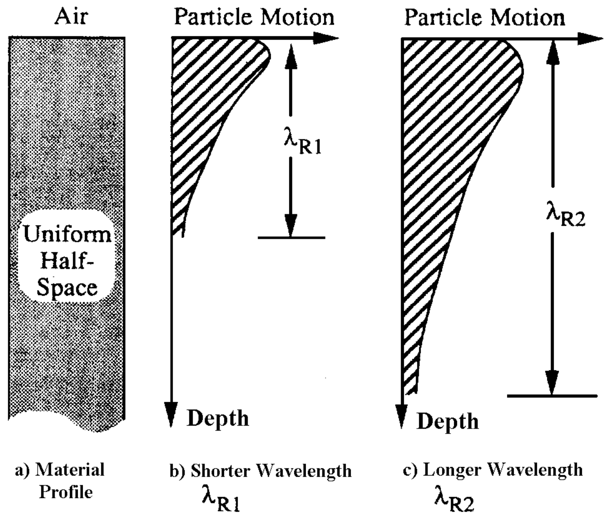

Figure 1. Approx. distribution of vertical particle motion with depth for two Raleigh Waves with different wavelengths (Stokoe et al., 1994)

Therefore by generating a wide range of frequencies, surface wave surveys use dispersion to produce velocity and frequency (or wavelength) correlations called dispersion curves.

After production of a dispersion curve the next step involves the inversion of the measured dispersion curve to produce a shear wave velocity – depth profile. Kansas Geological Survey produced the software *Surfseis*, which performs the inversion procedure using a least-squares technique developed by Xia *et al.* (1999). Surfseis is evaluated in detail using discrete particle models by Donohue and Long (2007).

A summary of the basic procedure that should be followed when utilising Raleigh waves is given below:

(i) Generate vertical ground motions using either an impulsive or a continuous wave source.

(ii) Measure these ground motions using low frequency geophones, which are arranged along a straight line directed toward the source.

(iii) Record the ground motions using either a conventional seismograph, oscilloscope or spectrum analyser.

(iv) Produce a dispersion curve from a spectral analysis of the data showing the variation of Raleigh wave velocity with frequency (or wavelength).

(v) Inversion of the dispersion curve (iv) to produce a subsurface profile of the variation of shear wave velocity with depth.

2.1 *Surface wave techniques*

The surface wave technique was probably first introduced into the field of civil engineering in the 1950's by Jones (1958). Jones used a mechanical vibrator to produce Raleigh waves, and was able to construct a dispersion curve. Abiss (1981) and Tokimatsu *et al.* (1991) further developed the continuous vibration technique using improved apparatus.

Significant advances have been made since this method was first introduced. In the early 1980's the Spectral Analysis of Surface Waves (SASW) was developed by Heisey *et al.* (1982) and by Nazarian and Stokoe (1984). Around the same time the Continuous Surface Wave (CSW) method (Mathews *et al.*, 1996) was also developed. Both the SASW and CSW techniques were developed from the initial continuous vibration technique. An increase in available computational power was the main reason for the development of these new techniques. The most significant difference between the two methods is that the SASW method uses an impulsive source (e.g. a hammer) to induce Raleigh waves in the shallow subsurface whereas the CSW method uses a vibrational source.

The Multichannel Analysis of Surface Waves (MASW) technique was introduced in the late 1990's by the Kansas Geological Survey, (Park *et al.*, 1999). The MASW method exploits proven multichannel recording and processing techniques that are similar to those used in conventional seismic reflection surveys. The most significant difference between the SASW and the MASW techniques, involves the use of multiple receivers with the MASW method (usually 12 to 60) compared to the SASW technique, which is based on a two geophone approach. The MASW approach only requires one shot gather, and when used in conjunction with the software, *Surfseis*, also maximises the signal to noise ratio and is therefore generally seen as an advancement on the SASW approach.

The MASW method was used for recording and processing of surface wave data for each of the sites discussed in this paper.

3 APPLICATION 1: MEASUREMENT OF SOIL STIFFNESS

The measurement of the small strain shear modulus, G_{max} of a soil is important for a range of geotechnical design applications. This usually involves strains of 10^{-3}% and less (Figure 2). Ground strains associated with most soil-structure interaction problems are generally accepted to be less than 0.1% and hence small strain stiffness values are required to make reasonable predictions of deformation (Jardine *et al.* 1986).

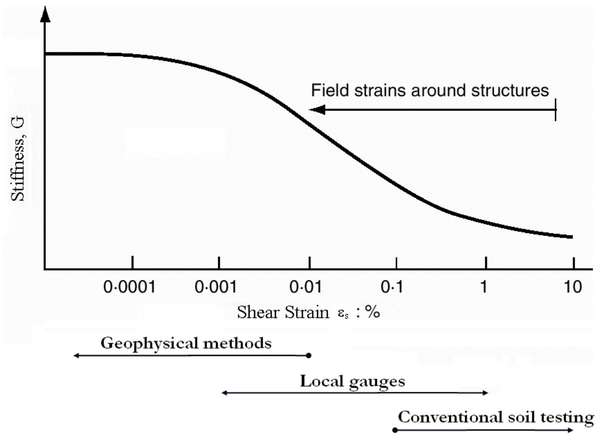

Figure 2. Range of strain for different testing techniques

According to elastic theory G_{max} may be calculated from the shear wave velocity using the following equation:

$$G_{\max} = \rho . V_s^{\,2} \qquad (2)$$

where G_{max} = shear modulus (Pa), V_s = shear wave velocity (m/s) and ρ = density (kg/m^3).

Young's modulus (E) is related to shear modulus by:

$$G = \frac{E}{2(1+\upsilon)} \qquad (3)$$

where υ is Poisson's ratio, and which may actually by measured if the P wave velocity (V_p) has also been measured during testing.

It has been shown by Stokoe *et al*. (2004) that stiffness-strain and stress-strain curves for a range of materials may contain considerable error if small strain stiffness values have not been incorporated. A significant overestimation of deformation may result, which could substantially increase the cost of a project.

From the authors experience a material may be considered soft if its shear wave velocity is less than approx. 170m/s, which corresponds to a G_{max} of less than approx. 50MPa.

3.1 *Stiffness of 2 soft Irish soils*

A number of soft soil sites have been examined by the authors using V_s over recent years. Two well characterised sites involved in this study are discussed here. They are located at Athlone and Portumna towards the centre of Ireland. Full details of the Athlone site are given by Long and O'Riordan (2001). Conaty (2002) describes the sites at Portumna. The deeper soft clay soils at these sites are glacial lake deposits, which were laid down in a large pro-glacial lake, which was centred on the middle of Ireland, during the retreat of the glaciers at the end of the last ice age some 10,000 to 20,000 years B.P.

As the climate became warmer and vegetation growth was supported on the lake-bed, the depositional environment changed and the upper soils have increasing organic content. At both sites the ground surface is underlain by two thin organic layers, calcareous marl and peat.

3.1.1 Athlone

At Athlone, two distinct strata were formed, as can be seen on Figure 3a. The lower soils are very soft brown horizontally laminated (varved) clays and silts with clearly visible partings typically 1 mm to 2 mm thick. These deposits are referred to as the brown laminated clay. Though there is some scatter in the data, there is no apparent trend in the parameters with depth. The brown laminated clay has a clay content of about 35% and has an average plasticity index (I_p) of about 18%.

As the climate became warmer, the depositional environment changed and the upper soils show only some signs of varving and have an increasing organic content. The material deposited under these conditions is homogenous grey organic clay and silt with a clay content and average I_p are of the order of and 25% and 40% respectively. These latter are higher than for the brown clay as a result of the increased organic content.

Above these layers of soft clay thin organic layers of peat and calcareous marl material were observed. As shown in Figure 3a the moisture content values are very high for these materials being consistently over 200%, with corresponding low bulk density values of the order of 1.2 Mg/m^3 and 1.4 Mg/m^3.

Piezocone (CPTU) q_{net} values are very low for all layers. They are slightly higher, however, in the peat and marl possibly due to the effects of fibrous inclusions. Values increase from about 0.15 MPa to 0.35 MPa in the grey organic clay, then fall back to about 0.2 MPa in the brown laminated clay followed by a gradual increase with depth, particularly below 10.5 m to about 0.6 MPa.

3.1.2 Portumna

As can be seen from Figure 3b, the Portumna clays are relatively uniform. In these deeper clay layers moisture content falls from about 50% in the upper clay layer to 40% in the lower layer. The corresponding bulk density values are 1.7 Mg/m^3 and 1.85 Mg/m^3. Additional data shows that the clay content is about 40% and I_p is 22%.

As at Athlone, the peat layer has a very variable natural moisture content, which ranges between 45% and 180% and a corresponding bulk density of less than 1.2 Mg/m^3. Equally the marl material has very high moisture content and a relatively low bulk density of about 1.25 Mg/m^3.

High CPTU q_{net} values were recorded in the peat, due probably to the effects of fibres. In the marl and in the layers below, values show almost no increase with depth, except perhaps in the upper clay layer and remain constant at about 0.25 MPa.

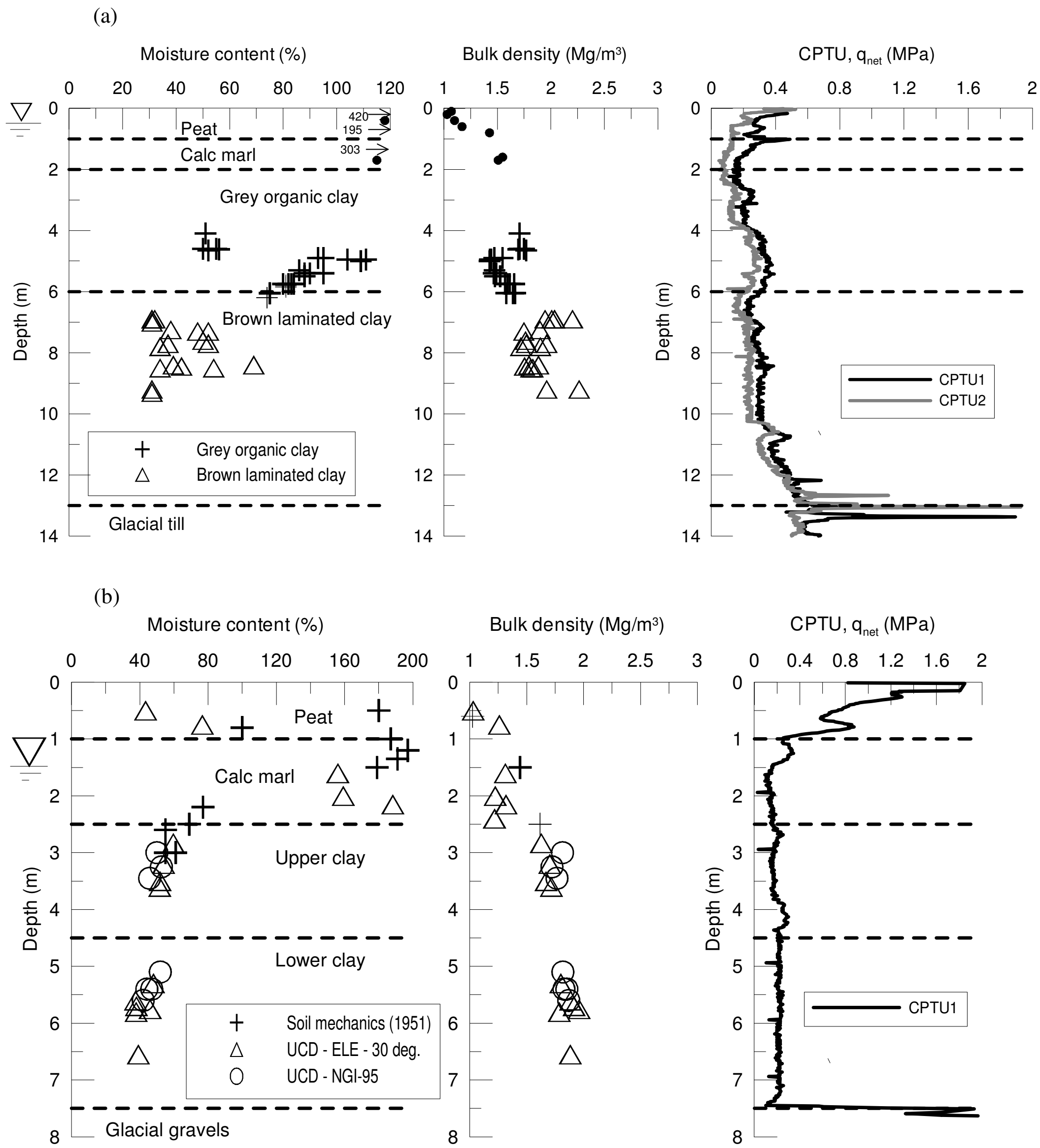

Figure 3. Basic soil parameters for (a) Athlone and (b) Portumna sites.

3.2 V_s *results*

A number of MASW tests were performed at each site and the results are compared to results from CPTU tests in terms of the cone tip resistance q_c. An empirical relation proposed by Mayne and Rix, (1993) was used to estimate G_{max} from the q_c data:

$$G_{max} = 99.5(p_a)^{0.305} * (q_c)^{0.695} / (e_0)^{1.13} \qquad (4)$$

where q_c = the measured cone tip resistance (kPa) p_a = atmospheric pressure, e_0 = in situ void ratio.

3.2.1 Athlone V_s

Three separate MASW survey lines were performed for the Athlone site to test the repeatability of the survey. The MASW lines were all parallel and located at two metre intervals.

The depth of penetration of the MASW method for each of the profiles was 8.75 m, which was adequate for the site. The limitation to this depth resulted from a lack of signal coherence at very low frequencies. G_{max} values computed for the MASW survey at Athlone are presented in Fig. 4a along with the empirically derived profiles from the CPTU tests.

There is good agreement between the three MASW profiles for this site. There is a slight increase in the variation of the three profiles with depth. The difference in the top few metres is negligible and the maximum difference at 8.75 m depth is 2.4 MPa. There is also variation in G_{max} estimated from the two CPT cone tip resistance (q_c) profiles, indicative of ground variability.

The MASW results give similar and very low G_{max} values for the peat and marl (less than 1.5MPa), with the boundary between the marl and grey organic clay being clearly defined.

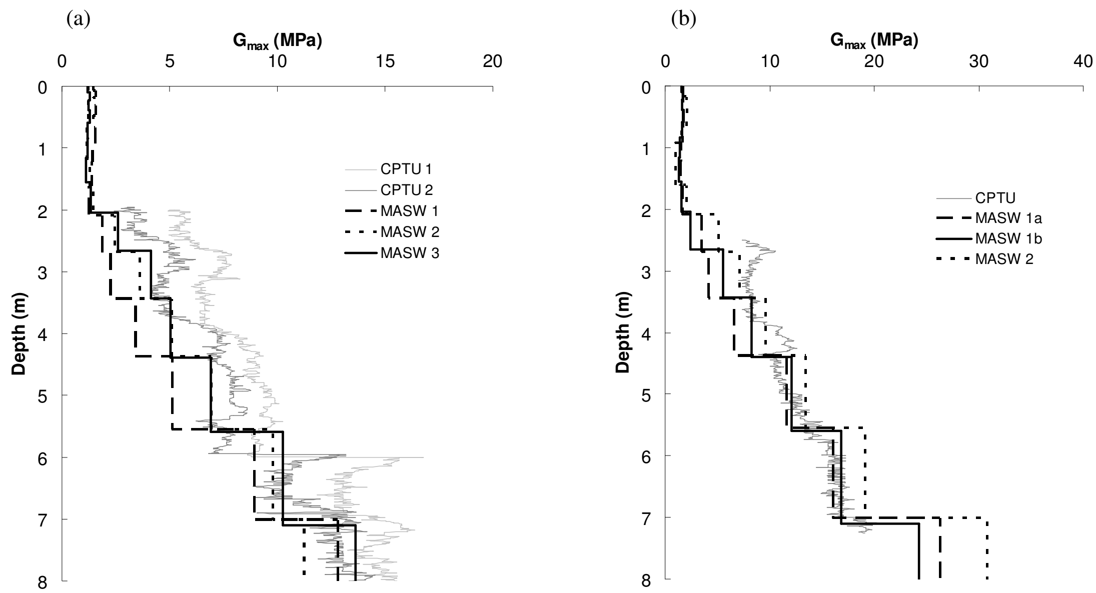

Figure 4. G_{max} measured using the MASW technique at (a) Athlone and (b) Portumna compared with corresponding CPT profiles

It is also possible to define a boundary between the grey organic clay and the lower brown laminated clay.

CPTU data for the peat and calcareous marl has been ignored as the Mayne and Rix (1993) approach was never intended to be used for such materials. All the G_{max} values are very low for these very soft clays. While the first CPTU profile (CPTU1) gives a consistently higher result than the MASW profiles the second, CPTU2, gives a very similar result. In general the CPTU estimated profiles give higher G_{max} than the MASW results. Typically the CPTU approach gives values 30% higher than for the MASW survey for the grey organic clay and 20% higher for the brown laminated clay. However the values involved are so small that these differences are considered negligible.

3.2.2 Portumna V_s

Two separate MASW survey lines were performed for the Portumna site to again test the repeatability of the survey. The MASW lines were parallel and located two metres apart and in the same location as the CPTU work and boreholes which yielded data described in Figure 4b. G_{max} values computed for the MASW surveys at Portumna are presented in Figure 4b along with the corresponding CPT profile. Two profiles for the same set-up were acquired by switching the position of the source to the opposite side (labelled MASW 1a and MASW 1b in Fig. 4b) again to test the repeatability.

The depth of penetration of the MASW method for the survey lines, MASW 1a and MASW 1b was 8.75 m and for the MASW 2 profile the maximum depth was 10 m which were more than adequate for this site.

There is very good agreement between the three MASW profiles for this site. As in the Athlone site there is a slight increase in the variation of the three profiles with depth. Also as before the MASW survey clearly delineates the interface between the marl and the underlying clays. G_{max} calculated from the CPTU cone tip resistance (q_c) shows excellent agreement with the MASW profiles. A slight difference between the profiles occurs at the top of the clay layer at a depth of 2.5 m to 3.5 m where the CPTU derived G_{max} is slightly higher than any of the corresponding MASW profiles.

3.3 *Comparison with laboratory test data*

A comparison between laboratory CAUC (anisotropically consolidated undrained compression) triaxial test data and MASW survey output is shown for Athlone brown laminated clay and Portumna clay on Figure 5a and 5b respectively. For Athlone the tests were carried out on high quality Sherbrooke block samples and for Portumna samples obtained using the NGI 95 mm diameter piston sampler was used. Strain resolution for Athlone is generally better as the axial displacement was measured using specimen mounted local gauges (Hall effect transducers).

It can be seen that the MASW G_{max} values relate well with the laboratory tests for these two lower plasticity clays.

4 APPLICATION 2: SAMPLE QUALITY ASSESSMENT

The engineering characterisation of soils for design and construction depends on acquiring samples from the ground with minimal disturbance. In the case of soft clays, evaluation of sample quality is considered essential if design parameters derived from laboratory tests are to be deemed reliable.

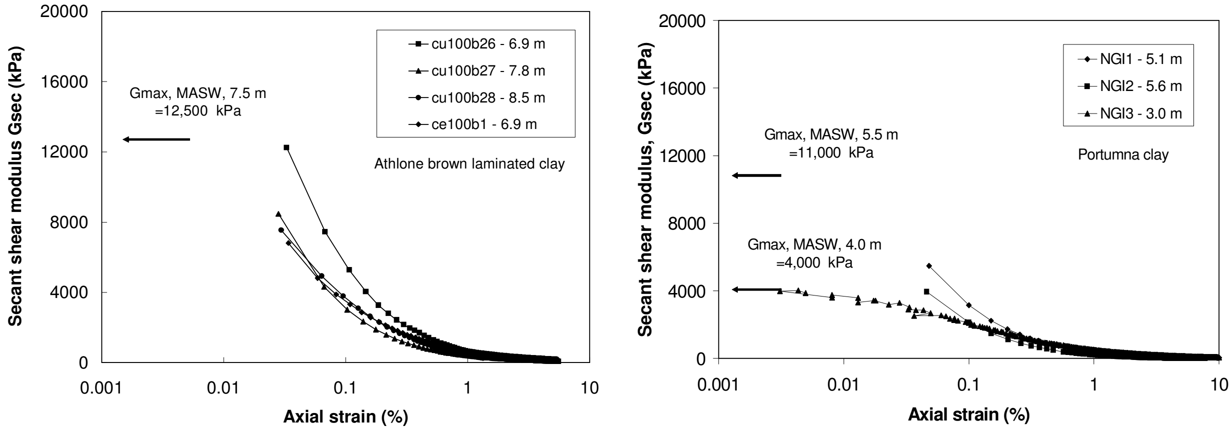

Figure 5. Comparison between laboratory test data and MASW output for (a) Athlone brown laminated clay and (b) Portumna clay.

Currently available evaluation techniques that are considered reliable, such as measurement of volumetric strain (Kleven *et al*., 1986) and the normalised change in void ratio, $\Delta e/e_0$ (Lunne *et al*., 1997), require reconsolidation back to in-situ stresses before measurement. These are therefore quite inefficient as this process may require a number of days testing. This is a particular problem for offshore sampling, where rapid assessment of sample quality may result in major economic saving.

A number of studies in recent years (Shiwakoti *et al*. (2001), Tan *et al*. (2002) and Porcino & Ghionna (2004)) have observed that laboratory determined shear wave velocities (V_s) and resulting small strain shear moduli (Equation 2) are generally lower than the in-situ equivalent and have attributed this difference to sampling disturbance. Each of these studies involved reconsolidation of laboratory specimens back to in-situ stresses before measurement of V_s, using bender elements (Dyvik and Madshus 1985), and comparison with the in-situ V_s. For a quick assessment of sample quality, Hight and Leroueil (2003) and Nash (2003) suggest using a portable bender element kit to acquire a measure of unconfined V_s immediately after removal from the subsurface while simultaneously using a portable suction probe to measure soil suction (u_r), which enables differences in stress state to be taken into account. This process only takes a matter of minutes to complete.

In the study described here (from Donohue and Long 2006) measurement of in-situ shear wave velocity was made using the MASW technique described above and measurement on recovered samples was made using the bender element technique. A number of samples of varying quality were acquired from a site near Ballinasloe and these were initially tested using the conventional disturbance measures mentioned above.

4.1 *Ballinasloe, Ireland*

The Ballinasloe test site is located approximately 170km west of Dublin and about 70 km east of Galway in the midlands of Ireland. The site is currently undergoing infrastructure development and will eventually be part of a new dual carriage motorway that will link the cities of Dublin and Galway.

The soft soil under investigation is very similar to that encountered at Athlone and Portumna, being post glacial lacustrine clay, deposited in a proglacial lake. The deposit comprises seasonally deposited layers of silt and clay (varved clay). These laminations of clay and silt are relatively thin and in general less than 1 mm to 2 mm. This deposit may actually be subdivided into two separate layers of soft clay based mainly on their CAUC failure envelopes and also on their colour which passes from grey to brown between these two layers. As the engineering properties of these materials are almost identical (Figure 6), these separate layers will be considered as a single geological unit in this paper. Above this material a layer of peat was again observed and underlying it dense sandy gravel was encountered.

For this work samples were obtained using the 100 mm diameter, 1 m long ELE fixed piston sampler, which is the most common “high quality” sampler used in the UK and Ireland. Two types of sample were obtained using this sampler. In the first the conventional technique of sampling from the bottom of a shell and auger (open percussive) borehole was used. In the second, the Scandinavian displacement approach was adopted, wherein the sampler (with the piston in front of the sampling tube) was pushed down to the desired sampling depth without preboring. In both cases the sampler cutting edge was sharpened from the normal 30º to 5º. Parallel samples were taken at four depth levels. As shown in Figure 6, there is little difference between the basic parameters of the conventional and full displacement techniques.

In addition one sample was obtained using the open drive U4 sampler. It was necessary to screw two tubes together to achieve full recovery. Despite several attempts it proved possible to recover only one sample.

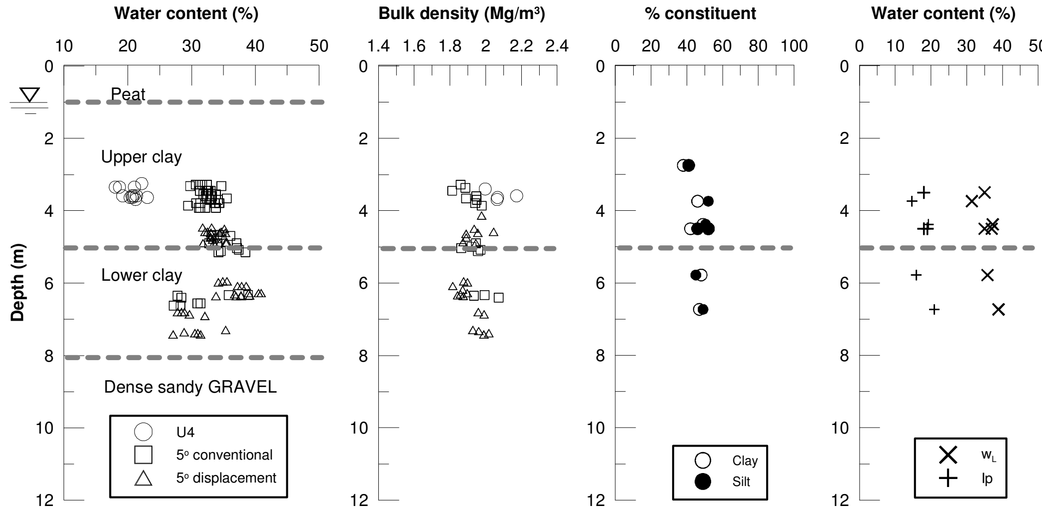

Figure 6. Basic site information for Ballinasloe

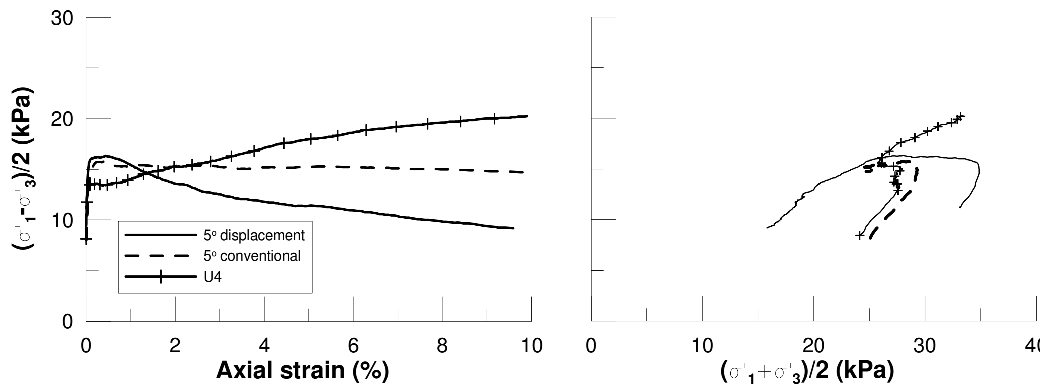

Figure 7. CAUC stress-strain and stress path plots for Ballinasloe

Depth (m)	Sample method	w (%)	ρ (Mg/m^3)	σ'$_{v0}$ (kPa)	σ'$_{ho}$ (kPa)	ε_{vol} (%)	Δe/e$_0$	Sample Quality *	s_u (kPa)	ε_f (%)
4.7	5° displacement	33	1.92	44	22	3.2	0.068	2	16.5	0.4
5.9	5° displacement	37	1.87	53	27	3.9	0.079	3	18.2	0.2
6.8	5° displacement	29	1.95	61	31	2.8	0.063	2	22.3	0.1
3.5	5° conventional	31	1.87	34	17	3.4	0.072	3	16.0	0.5
4.9	5° conventional	34	1.96	45	23	6.4	0.139	3	16.5	0.2
6.5	5° conventional	35	1.95	59	30	6.2	0.132	3	21.0	0.2
3.4	U4	18	2.01	32	16	4.4	0.118	3	20.4	>10

Table 1. Summary of CAUC tests, w = moisture content, ρ = bulk density, σ'$_{v0}$ = vertical effective stress, σ'$_{ho}$ = horizontal effective stress. * Sample quality from Lunne *et al*. (1997) where 1 = Very good to excellent, 2 = Good to fair, 3 = Poor, 4 = Very Poor

Interestingly the moisture content and bulk density of the material taken from the double U4 samples are quite different from those of the piston tubes indicating that the material has been densified by drainage of excess pore pressure from the silt lenses during sampler driving.

A similar effect was described by Long (2006) for samples of varved clay from Athlone.

4.2 *Conventional sampling disturbance measures*

A summary of the most important parameters obtained from a number of anisotropically consolidated undrained (CAUC) triaxial tests is given in Table 1. The table includes some basic classification data and measurements from the consolidation and shearing phases. The consolidation stage parameters include the volumetric stain (ε_{vol} – Kleven *et al.*, 1986) and the normalised change in void ratio ($\Delta e/e_0$ – Lunne *et al.*, 1997). The shearing stage parameters included in the table are the undrained shear strength (s_u) and the axial strain at failure (ε_f). According to the consolidation and shearing stage parameters, the 5^o modified piston tube using the displacement technique results in considerably higher quality samples of Ballinasloe clay.

CAUC stress-strain and stress path plots are shown in Figure 7 for samples between 3 and 5m depth. It is interesting to observe very different behaviour for each of the different samplers. The most startling result from the stress paths is that the U4 specimen shows dilative behaviour in comparison to contraction exhibited by the piston tubes. This is consistent with the observed densification as discussed above.

From the stress-strain plots it would seem that the full displacement samples are superior to the conventional piston samples. The peak is clearly defined, it occurs at a lower strain and there is greater strain softening post peak. The stress path for the conventional piston sample shows some dilative behaviour when it reaches the failure line whereas the displacement sample exhibits continuous contractive behaviour.

4.3 *V_s Results*

Unconfined shear wave velocities (V_{s0}) and corresponding in-situ V_s, resulting from the MASW (Donohue, 2005) method are presented in Figure 8. Remoulded shear wave velocities were obtained from sample trimmings at their in-situ density and natural water content. The measurement of unconfined V_s was performed on samples before CAUC testing thereby enabling a direct comparison between measured velocities and conventional sampling disturbance parameters, such as $\Delta e/e_0$.

Shear wave velocities measured on samples of Ballinasloe clay are significantly lower than the in-situ equivalent (Figure 8). The 5^o modified piston tube using the displacement technique displays consistently higher velocities (and therefore stiffness) than either the 5^o conventional or U4 samples. The shear wave velocity of the U4 sample is extremely low and is very close to the remoulded V_s, indicating very poor quality, consistent with the CAUC data discussed above.

A tentative criterion was proposed by Donohue and Long (2006) for the quantification of sample disturbance which combined shear wave velocities and soil suctions measured on samples from three soft clay sites (Ballinasloe and Bogganfin in Ireland, Onsøy in Norway). Having observed its relationship with a other standard sample quality measures, Donohue and Long (2006) proposed the normalised parameter L_{cx} to evaluate disturbance, where:

$$L_{vs} = \frac{V_s insitu - V_{s0}}{V_s insitu - V_s remoulded} \qquad (4)$$

The use of remoulded shear wave velocities takes into account the lowest possible V_s, where the sample is completely disturbed. An L_{vs} of zero would be considered completely undisturbed, as V_{s0} would therefore be equal to the in-situ V_s.

The criterion proposed by Donohue and Long (2006) involves plotting L_{vs} against the proposed normalised soil suction parameter L_u. Having observed the relationship between these two parameters individually and $\Delta e/e_0$ for the three sites listed, they divided the criterion into three classes; very good to excellent, good to fair and poor. For a sample to be considered very good to excellent L_{vs} must be < 0.65 and L_u must be < 0.4. A perfectly undisturbed sample would exhibit zero L_{vs} and L_u. The L_{vs} vs. L_u criterion is shown in Figure 9 for the three sites listed above. As shown the Sherbrooke block samples from Onsøy produce by far the highest quality sample. From an Irish perspective it is interesting to observe that the 5^o displacement piston samples were generally found to be of "good to fair" quality and were definitely of superior quality to samples from the other techniques tested here.

5 APPLICATION 3: GROUND IMPROVEMENT

In many types of ground improvement there may be some uncertainty regarding the effectiveness of the ground improvement method. Shear wave velocities obtained from MASW testing are ideally suited for monitoring ground improvement as they provide a rapid and direct measure of improvement during the treatment process. The MASW technique has also been found to be produce high quality data, even on sites with ongoing construction noise. This is a major advantage over other seismic techniques where first arrivals may be impossible to detect.

An MASW survey can be performed on site prior to ground improvement works, which would provide a set of baseline stiffness profiles. Further stiffness profiles can be acquired rapidly both during and after the improvement process and compared to the baseline profiles. Improvement may therefore be quantified in terms of shear stiffness. The effective depth of the improvement can also be determined.

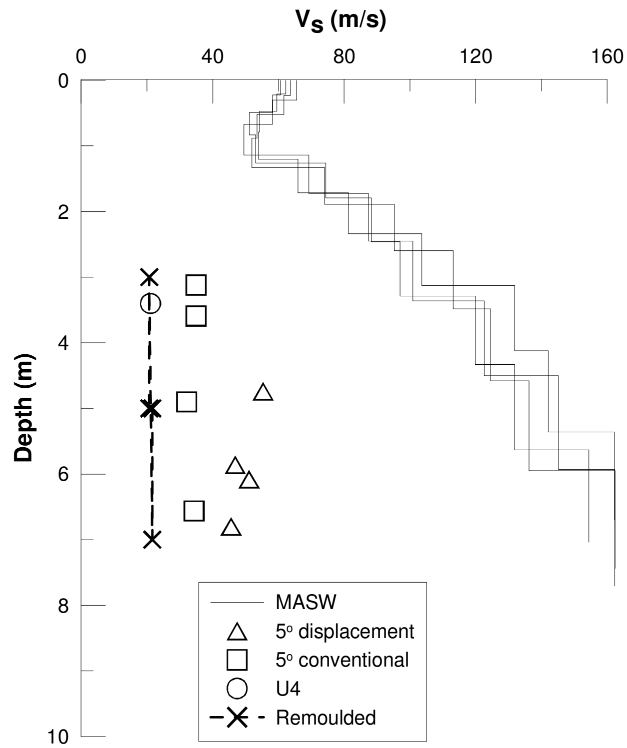

Figure 8. Unconfined V_s measurements made on samples compared with in-situ V_s for Ballinasloe

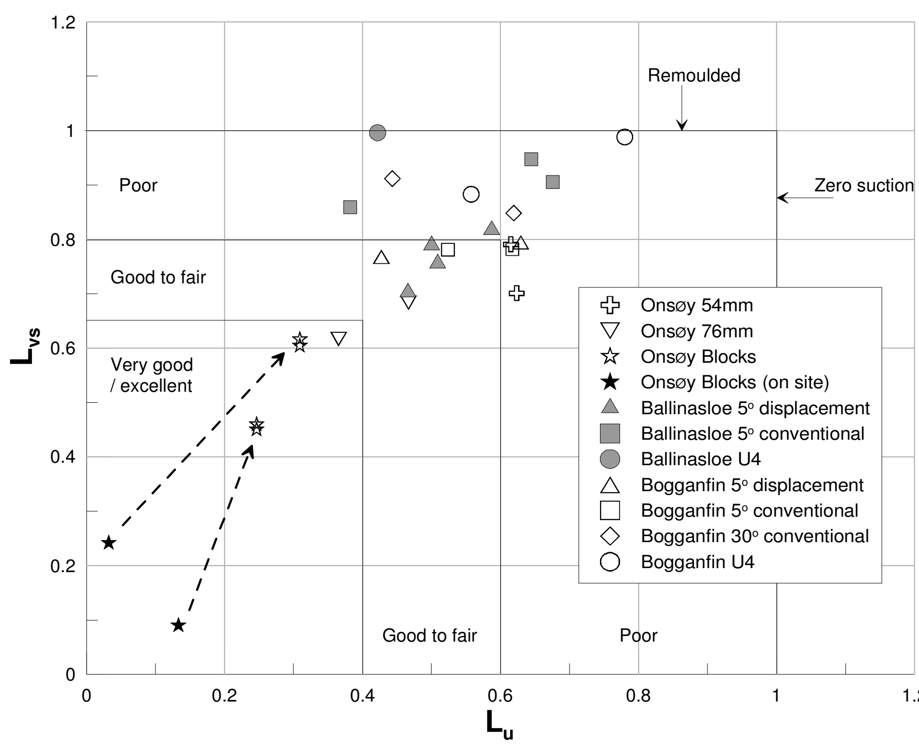

Figure 9. Variation of the normalised shear wave velocity parameters, L_{vs}, with the normalised soil suction parameter, L_u, showing estimated sample qualities.

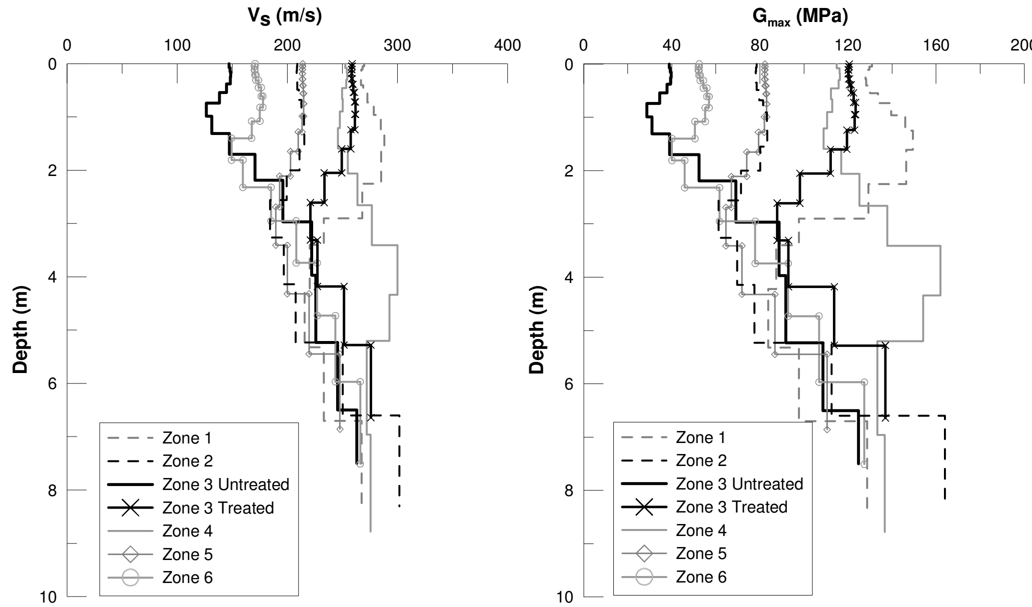

Figure 10. (a) Shear wave velocity and (b) corresponding small strain shear modulus from Jamaica St. ground improvement site

5.1 *Jamaica Street site, Belfast*

The Jamaica St. site housing development (approx. 1.25Ha), located in the Oldpark area of north Belfast was used for initially testing the effectiveness of V_s measurements during ground improvement.

The site was underlain by approx. 1.2m of made ground underneath which soft to firm sandy gravely clay/glacial till was encountered. The glacial till became stiff and very stiff with depth.

Ground improvement on site was performed by CON-FORM. They excavated in areas subject to loading from house construction down to firm bearing capacity, which was measured by means of hand-held shear vanes. The materials were then re-engineered by the addition of lime or cement to achieve bearing in backfilled material under the housing platforms. This material was also dynamically rolled to induce maximum settlement and consolidation in the backfilled material.

5.2 *V_s results*

The site was subdivided into a number of zones and MASW profiles of shear wave velocity were performed at six of these zones. Unfortunately only Zone 3 was able to be tested prior to treatment, so the results of all six zones will be compared to this baseline V_s profile. Shear wave velocity and corresponding shear modulus profiles are shown in Figures 10a and 10b respectively. As shown a significant increase in V_s and G_{max} was observed at Zones 1, 3 and 4. A significant increase was also observed at Zones 2 and 5 (although not as large as at Zones 1, 3 and 4) and only a small increase in stiffness was observed at Zone 6 (relative to the untreated Zone 3). This variation is likely due to the different zones being at different stages of curing, with Zone 6, in particular having been freshly treated.

The effective depth of improvement may also be observed for each zone, below which each of the profiles converge back to the original untreated profile (Zone 3 untreated).

6 APPLICATION 4: STRATIGRAPHIC MAPPING

Shear wave velocity may also be used to identify subsurface geological boundaries, both vertically and horizontally, if there is a difference in stiffness between adjacent geological units (e.g. between soil and rock). Both MASW and seismic refraction can be used for this, which in combination with another geophysical method, resistivity, can build up a detailed image of the shallow (<40m) subsurface.

The full set of shear wave velocity data for the Ballinasloe test site (discussed above in Section 4) is shown below in Figure 11. All profiles were performed between the boreholes used for acquiring soil samples detailed above, which enabled identification of the geological layers shown in Figure 11. As shown the MASW method detects each of the three layers identified by the subsequently drilled boreholes. It identifies the very low velocity peat layer to a depth of approximately 1.3m, and shows V_s increasing for the soft clay from 66m/s at 1.2m depth, to 162m/s at a depth of 7.7m. When low frequency geophones (4.5Hz) were used the deeper dense sandy gravel layer was detected which is evidenced by a significant jump in V_s at a depth of 7.7m depth.

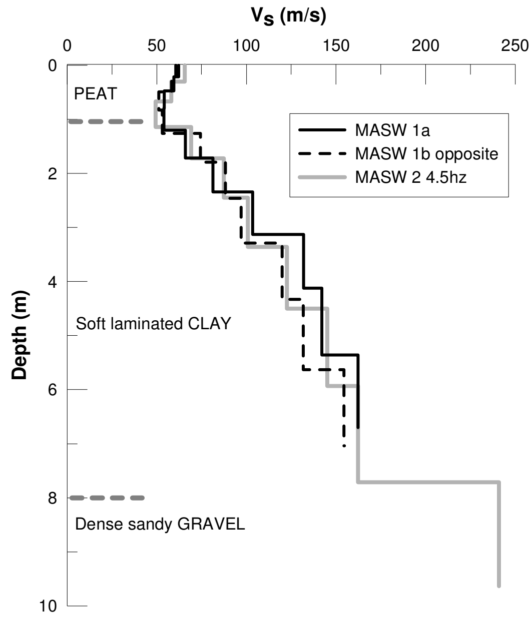

Figure 11. Using in-situ shear wave velocity profiles from Ballinasloe to identify layer boundaries

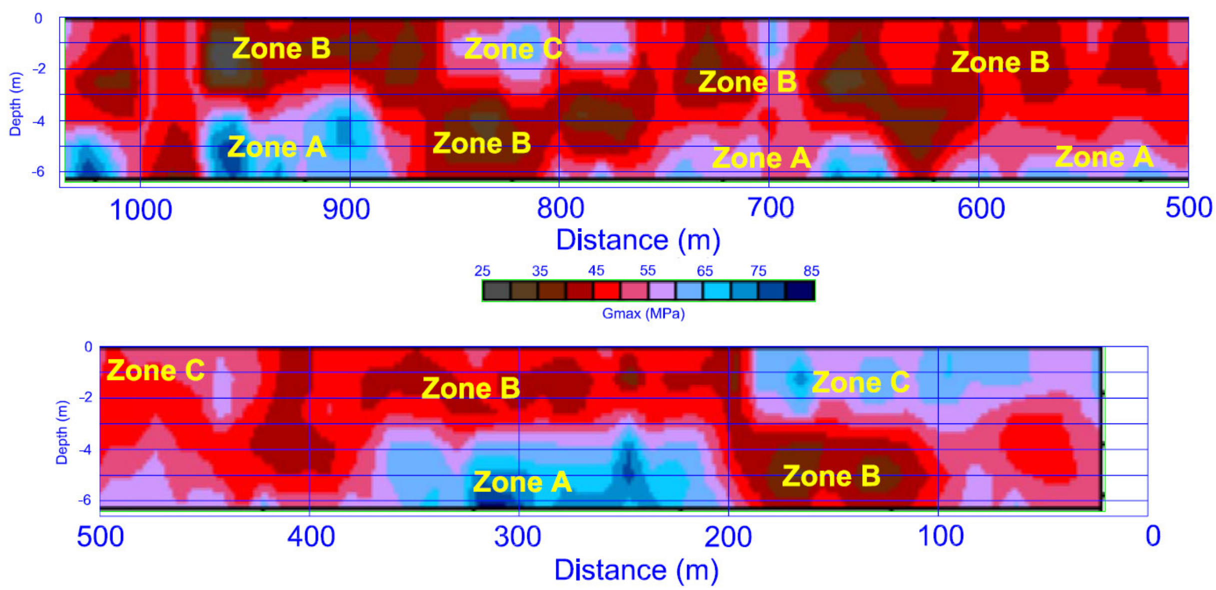

Figure 12. 2D stiffness map from an embankment along the River Dee in Chester

Shear wave velocity may also be horizontally mapped leading to a high resolution 2 dimensional stiffness profile, which may be useful for mapping geological boundaries or for detecting localised changes in stiffness. This essentially involves combining a number of closely spaced individual 1D profiles.

The example shown above is a 2D MASW profile from an embankment along the River Dee in Chester. The embankment in this area has a history of instability. To identify areas of relative low stiffness within the embankment (approx. 6m high) an MASW survey was employed. The survey was just over 1Km in length and 1D profiles were acquired every 6m.

As shown in Figure 12 a number of localised areas of relative low stiffness were identified (labelled Zone B), although overall the small strain stiffness of the embankment was quite low.

7 SUMMARY AND CONCLUSIONS

A number of geophysical techniques have received considerable attention in civil engineering over recent years, mainly due to advances in geophysical techniques and an increase in available computational power but also because they may be non-invasive, cost effective and a large area may be covered relatively quickly. The applications of shear wave velocity measurements were discussed in this paper in the context of soft ground site investigation in Ireland. The recently developed Multichannel Analysis of Surface Waves (MASW) method was used at each of the sites under investigation and was shown to produce high resolution and very repeatable profiles of V_s. The following is a summary of the applications discussed above:

1) Shear wave velocity profiles were obtained in the field using the Multi Channel Analysis of Surface Waves (MASW) method at two soft ground sites in the Irish Midlands to determine the small strain shear modulus, G_{max} and to compare the MASW derived stiffness profiles with corresponding CPT derived profiles. At both sites the MASW produced profiles compared very well with values derived empirically from CPTU results.

2) Shear wave velocity was also shown to be a very useful parameter for the rapid estimation of sample disturbance. Samples of varying quality were tested using conventional assessment techniques performed in conjunction with shear wave velocity measurements at a soft clay site in Ballinasloe.

3) The effectiveness of ground treatment may be monitored using V_s measurements which provide a rapid and direct measure of improvement during the treatment process. Significant improvement was observed at some parts of the test site in Belfast. It was interesting to observe only a small increase in stiffness on a freshly treated part of the site.

4) Shear wave velocity may also be used to identify subsurface geological boundaries, and high resolution stiffness profiles may be produced for a site relatively quickly.

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