

In-Situ Testing of Peat – a Review and Update on Recent Developments

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ABSTRACT: This paper reviews the techniques used and some recent developments on in situ testing of peat for the purposes of the design and surveillance of engineering structures. Geophysical techniques, especially ground penetrating radar, are now being used extensively in peatlands. All geotechnical in-situ tests in peat can be influenced by partial drainage and therefore can give misleading results if not used carefully to well established guidelines and if not interpreted correctly. There is therefore a benefit in multi-measurement tests (e.g. CPTU and piezoball) which give additional information to help assess the drainage condition. There seems promise in the use of the pore pressure measurements for both CPTU and piezoball for the purposes of profiling peat decomposition and possibly shear strength assessment. Field vane testing will frequently give misleading results. Other standard geotechnical techniques may only be useful when used in conjunction with locally derived empirical correlations. Pore pressure measurements in peat may be influenced by the presence of gas in the deposits.

1. INTRODUCTION

Throughout the world, construction on peat soils presents engineers with many challenges arising from their high compressibility and relative low shear strength. While avoiding construction on these soils may be a favoured option, in certain conditions development on these soils cannot be avoided. For example, in Ireland and Scotland, to utilise the renewable energy resources available from wind and water, developments often take place in peatland environments in both upland and lowland settings. Catastrophic peat landslides that have occurred in these environments have increased awareness of this geohazard and the importance of properly characterising this material. In the Netherlands many dikes are constructed of and on peat / strongly organic soils and ongoing monitoring and stability assessment of these structures is of significant importance. Highway and infrastructure developments on peat in other parts of the world, for example Canada, the USA, Malaysia and Indonesia, all require knowledge of the properties of the peat.

Due to the difficulties in sampling of peat and subsequently preparing specimens for laboratory testing engineers have made significant efforts to develop in situ techniques for characterisation of peat deposits and for estimating the relevant engineering properties. This paper provides a review of this work and gives an update of some recent developments.

2. FIELD VANE TESTING

2.1 Introduction

There are no special in situ techniques available for testing peat soils. Therefore standard techniques for inorganic soils are generally used in peat. The field vane test is often used to determine the shear strength of peat (s_{u-FV}). Despite its use in peat being heavily criticised it has continued to be used throughout the world and remains perhaps most common test. Therefore a review of the issues involved and its application is appropriate.

2.2 Problems with field vane testing in peat

The problems with using the vane test in peat were recognised at an early stage in several countries. For example in Ireland, Quinn (1967) stated that the “test was open to criticism as the failure mechanism is one of tearing rather than shearing”. In Finland, Helenelund (1967) concluded that the “test is not reliable in fibrous peat”. For work in France, Mangan (1993) suggested that, as the mode of deformation of peat is often characterised by punching failure, field vane strength should be applied with caution.

Perhaps the most well known and comprehensive review of the practice is that of Landva (1980). He observed that a void was

generated behind the blade into which the compressed peat in front of the blade drained resulting in a modified peat, see Figure 1 (Noto, 1991). This partially drained / drained behaviour would lead to strength parameters that are higher than the truly undrained s_{u-FV} . Noto (1991) confirmed this by carrying out vane tests at rotation rates from 0.1°/sec to 10°/sec, which showed a trend of decreasing strength with increasing rotation rate (Figure 2). Attempts to observe the influence of strain rate by Landva (1980), were masked by the variability of the material and no consistent trends could be observed. For the range of vane sizes and rotation rates generally used in practice, and considering the typical consolidation properties of peat, it is unlikely that undrained conditions could be obtained using this test in peat.

Landva (1980) and Helenelund (1967) also reported that a cylindrical shear surface occurred at a diameter 7 mm to 10 mm outside the edge of the blade and the length of the vane shear face was shorter due to the compression / void mechanism described above. Therefore the assumed failure surface, from which s_{u-FV} is calculated, is quite different to the actual failure surface.

Additionally in fibrous peat, fibres often wrap around the vane during rotation and increase the resistance being measured. Landva (1980) concluded that the field vane test is “of little engineering value in fibrous material” and is also not suitable for organic soils.

2.3 Influence of size of vane

Unlike mineral soils, s_{u-FV} in peat has been found to decrease with increasing diameter, possibly due to the scale effect of the fibres (Landva, 1980). Figure 3 shows the results of in-situ vane tests which were carried out at the Vinkeveen research site in the Netherlands (Boylan, 2008).

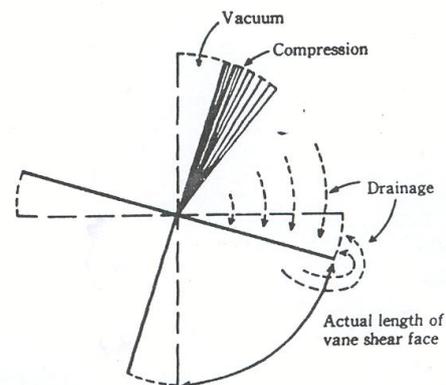


Figure 1 Interaction of vane with peat during test - illustrated in (Noto, 1991)

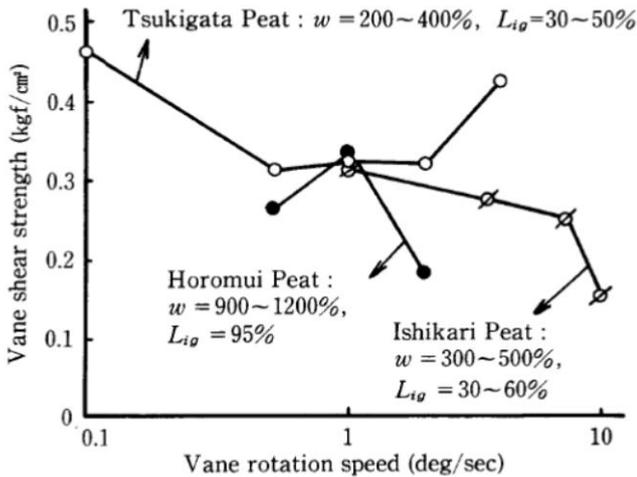


Figure 2 Influence of rotation rate on s_{u-FV} (Noto, 1991)

An electrical GEOTECH vane (www.geotech.se) was used. This apparatus is computer controlled and torque data is logged on a laptop computer. The vane head which applies the torque to the rods is mounted on a frame to ensure stability during testing. Tests were conducted both a 280 mm x 140 mm and 172 mm x 80 mm size vane.

Figure 3 shows the shear strength (s_{u-FV}) from these tests to lie between 7-15 kPa. The results of tests with the smaller size vane are more scattered and generally higher than those with the larger size vane. This is similar to the findings of Landva (1980). The results of remoulded vane tests are also indicated.

2.4 Correction factors

In engineering practice reduction factors have been introduced to modify the measured strength and provide strength parameters representative of undrained conditions and account for viscous rate effects in some cases. These reduction factors have been developed in response to local experience and conditions. The undrained shear strength adopted for design (s_u) is taken as:

$$s_u = \mu_{FV-C} s_{u-FV} \tag{1}$$

Golebiewska (1983) proposed $\mu_{FV-C} = 0.5$ to 0.55 for peat. Landva and Rochelle (1983) provided vane and ring shear data where the ring shear value is 42% to 57% of s_{u-FV} .

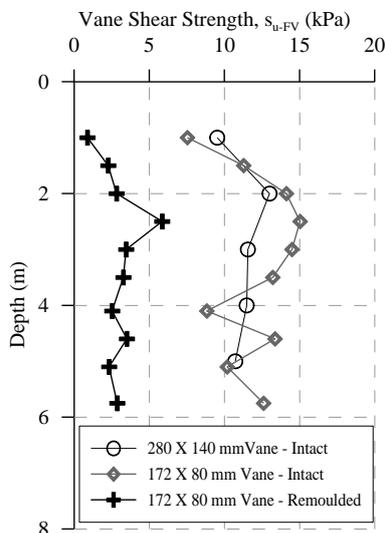


Figure 3 Vinkeveen in-situ vane tests

The Swedish Geotechnical Institute developed the following reduction factor (Larsson et al., 1984):

$$\mu_{FV-C} = \left(\frac{0.43}{w_L} \right)^{0.45} \tag{2}$$

where: w_L = liquid limit.

In Poland Sanglerat and Mlynarek (1980) and Mlynarek et al. (1983) found that the relationship between laboratory UU triaxial strength and vane strength varied between 0.26 for sedge moss peat and 0.69 for carbonate sedimentary peat. Lechowicz (1994) recommended that the Swedish correction factors be used in practice in Poland.

Hanzawa et al. (1994) reported that in Japan the mobilised shear strength in a peat deposit that failed under an embankment load was calculated to be 50% of s_{u-FV} and that the laboratory direct shear strength was 67% of s_{u-FV} .

For work in the US Edil (2001) suggested a vane correction factor, $\mu_{FV-C} = 0.4 - 0.5$. Mesri and Ajlouni (2007) suggested a correction, $\mu_{FV-C} = 0.5$ be applied to the results of vane tests in peat.

In the experience of the author's it is not possible to determine the liquid limit of peat, using either the fall cone or the Casagrande cup, due to the effects of the fibres and in Irish practice a single factor of 0.5 is often employed.

2.5 Comparison with laboratory DSS strength

A comparison is made here between the results of direct simple shear (DSS) tests and in situ vane tests in peat. Simple shear tests provide strength parameters appropriate for stability analyses of translational type failure, which peat slope failures often resemble (Long and Boylan, 2012). In addition the simple shear strength is often taken as the average strength mobilised under an embankment or large shallow foundation.

It would be also possible to compare the field vane test results with those from triaxial testing. However, as detailed by Long (2005), there are many problems associated with the triaxial test in peat, for example; controlling the very low effective stresses required for consolidation, the large volume changes which occur during consolidation, accounting for the effects of end platen roughness, membrane stiffness effects and maintaining the verticality of the specimen during testing. These factors play a role in the scatter of shear strengths reported from triaxial tests on peat.

A comparison of field vane test results and laboratory direct simple shear strength measurements at three Irish sites are shown on Figure 4. The simple shear specimens were consolidated to the best estimate of the in situ vertical effective stress (typically 5 kPa to 10 kPa). Simple shear tests are carried out as constant volume tests thereby measuring an undrained strength, that is generally lower than the partially drained / drained field vane test (Long and Boylan, 2012). The anisotropy of peat strength due to the presence of fibres, allied with the different failure mechanisms in simple shear and vane tests also plays a role in these differences.

In order to classify the material and distinguish between different types of peat, engineers often use the classifications system of Von Post and Granlund (1926). This is based on a visual inspection of the peat and a simple hand squeezing test. The peat is classified on a scale of H1 (no decomposition) to H10 (completely decomposed) as outlined on Table 1.

Table 1 Determination of decomposition Von Post and Granlund (1926)

Degree of Humification	Decomposition	Plant Structure	Material Extruded on Squeezing
H1	None	Easily identified	Clear, colourless water
H2	Insignificant	Easily identified	Yellowish water
H3	Very slight	Still identifiable	Brown, muddy water; no peat
H4	Slight	Not easily identifiable	Dark brown, muddy water; no peat
H5	Moderate	Recognisable but vague	Muddy water and some peat
H6	Moderately strong	Indistinct (more distinct after squeezing)	About one third of peat squeezed out; water dark brown
H7	strong	Faintly recognisable	About one half of peat squeezed out; any water very dark brown
H8	Very strong	Very indistinct	About two thirds of peat squeezed out; also some pasty water
H9	Nearly complete	Almost unrecognisable	Nearly all peat squeezed out as fairly uniform paste
H10	Complete	Not discernible	All peat passes between fingers; no free water visible

Data for the West Donegal site, shown on Figure 4a, is typical for Irish peat and illustrates that the water content of the peat decreases from about 1000% at 0.5 m to 600% at 2.5 m. Broadly the peat can be described as moderately decomposed throughout but can be divided into an upper less decomposed zone with von Post and Granlund (1926) H = 4 to 8 to 1.5 m and a lower more decomposed region with H = 5 to 6. Shear strength values obtained from simple shear tests (s_{u-DSS}) show a clear increase with depth from about 5 kPa near the surface to 11 kPa with depth. The average s_{u-DSS} value is about of about 8.0 kPa. As expected there is a clear tendency for an increase in s_{u-DSS} with decreasing water content. There is no clear relationship between s_{u-DSS} and von Post and Granlund H. Vane test results (s_{u-FV}) are in general higher and very scattered. There is a greater difference between the vane and simple shear test results in the deeper more decomposed zone.

Similar results can be seen for the Glinsk site in Co. Mayo where the peat is very similar to that at the West Donegal site (Figure 4b). Here the vane test results are even more scattered.

Data for the Crockagarron site in Co. Tyrone is interesting because here the peat has unusually high water content, being of the order of 1200% to 1600% (Figure 4c). Simple shear values are correspondingly lower with an average of about 4.5 kPa. Hand vane s_u values are higher than those of the larger vanes. The larger vanes, particularly the 55 mm x 110 mm Geonor H10 vane, give s_{u-FV} values slightly greater than those from the simple shear tests.

Long and Boylan (2012) compared in situ vane test and simple shear tests for 8 sites (including the three detailed above). The ratio s_{u-FV}/s_{u-DSS} versus degree of decomposition, H, for depths at which vane tests and simple shear tests exist are compared on Figure 5. The ratio of s_{u-FV}/s_{u-DSS} ranges from 1 to 5.7, with the highest ratios observed for lower values of decomposition.

This is as expected due the greater concentration of fibres at low decomposition levels. In addition, the effect of partially drained / drained conditions on the vane tests would be greater in the more compressible peat of low decomposition.

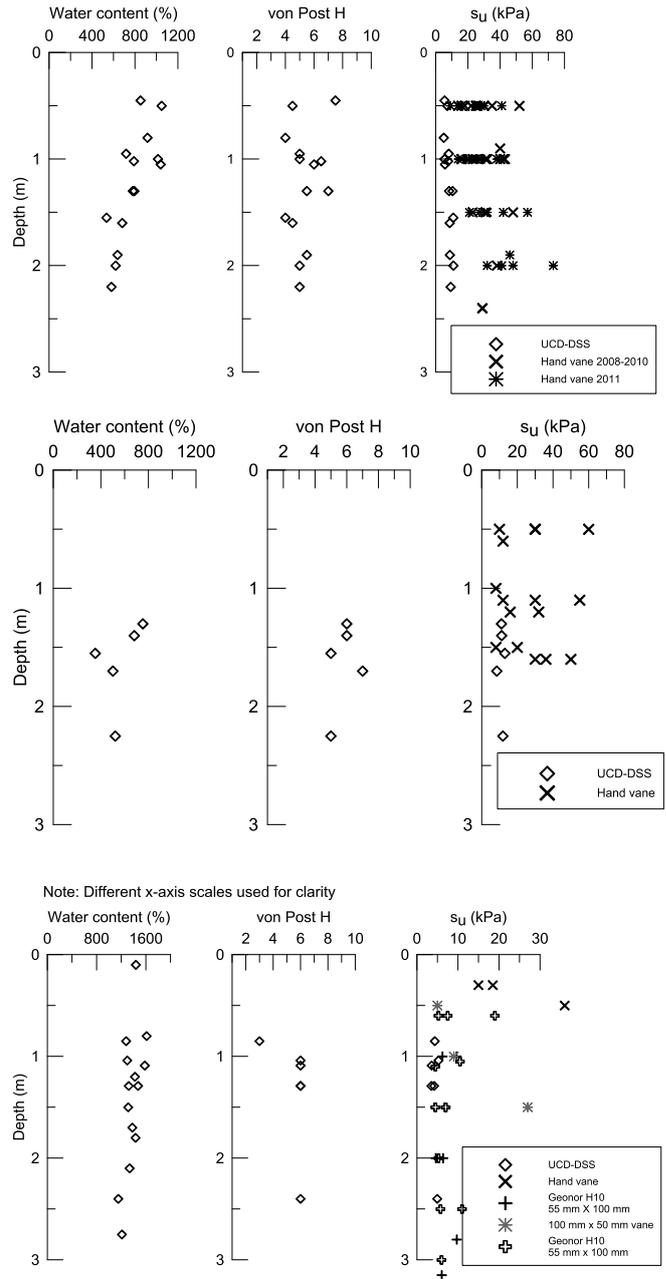


Figure 4 Comparison of s_u from field vane and laboratory DSS tests (a) West Donegal site, (b) Glinsk site, Co. Mayo and (c) Crockagarron, Co. Tyrone

The wide variation of ratios and the high values, far greater than 1.0, suggests that in-situ vane tests may grossly overestimate the shear strength of peat deposits. Considering the s_{u-FV}/s_{u-DSS} ratio of 2.0 implied by the vane correction factors discussed above approximately 70% of the values lie above this level meaning that a universal factor is insufficient for correcting vane tests in peat.

2.6 Conclusion

It is clear from the discussion above that vane tests in peat may give misleading and non-conservative results and should be treated with great caution.

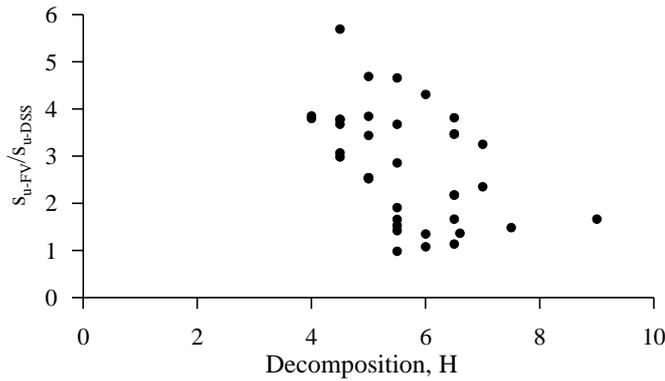


Figure 5 Ratio of in situ vane strength compared to DSS

3. CONE PENETRATION TESTING

3.1 Introduction

The cone penetration test (CPT) or the cone penetration test with pore pressure measurements (CPTU) (commonly referred to as the piezocone test) test is widely used in peat deposits due to the availability of lightweight equipment to access peat sites, the relatively low cost, speed and minimal site disturbance caused by the testing, see Figure 6.

A typical set of CPTU test results, for the Crockagarron Wind Farm site in Co. Tyrone, Northern Ireland is shown on Figure 7. It can be seen that about 4.7 m of peat overlies granular mineral soil. Corrected cone resistance (q_t) and sleeve friction (f_s) values are very low being of the order of 0.1 MPa and 5 kPa respectively. A wood fragment was encountered at about 3 m. Pore water pressures (u_2) are greater than hydrostatic in the peat and drop off suddenly on encountering the mineral soil.

It can be seen that a significant issue with CPTU tests in peat is the low tip resistances and some standard cones (10 cm² bearing area) have difficulty resolving these resistances accurately.



Figure 6 Lightweight CPT equipment used on peat in Ireland by Lankelma Ltd and In Situ Site Investigations Ltd

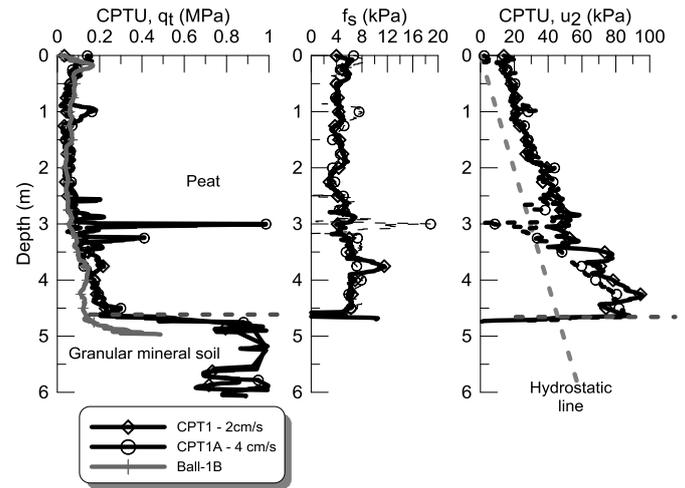


Figure 7 CPTU and ball results – Crockagarron Wind Farm site, Co. Tyrone, Northern Ireland

In addition to this the measured resistance can be very scattered due to the variable interaction with fibres.

To improve resistance measurements in these very soft deposits, Viergever (1985) performed tests in organic soil using cones with large projected areas (50 and 100 cm²) and the traditional cone (10 cm²). The results of tests using the largest cone measured resistances that were 30%–45% lower than that of the 10 cm² cone. The larger cones measured a more homogenised soil volume, possibly under conditions closer to fully undrained, than the smaller cones, resulting in a substantial reduction in the standard deviation of the penetration resistance.

Landva (1986) similarly conducted tests with a large 300 mm diameter cone while Noto (1991) used a larger cone with a 20 cm² bearing area.

3.2 Accuracy of CPTU testing in peat

Boylan and Long (2006a) and Boylan et al. (2008) explored the accuracy of CPTU testing for the characterisation of organic soils. Laboratory studies on a number of cones show that the measured parameters can be greatly influenced by differences in the temperature at which the cone is zeroed and the temperature in the ground itself, even if the cone is temperature compensated. This effect can result in significant positive or negative shifts in the measurements from this test.

An example of this effect on two tests from the Vinkeveen research site are shown on Figure 8. In the first test, the cone was stabilised and zeroed as normal in air (11°C - 12°C) before the test commenced. In the second test, the cone was equilibrated to the ground temperature (7°C - 8°C) by immersing the cone in a bucket of water from a stream adjacent to the test location. Both of these tests were carried out as standard CPTU tests at a penetration rate of 2 cm/sec. It can be clearly seen, especially in terms of q_t , that the cone which has not been equilibrated to the ground temperature has been affected by the temperature differential. In the early stage of the test (< 2.5m) q_t is extremely scattered. At greater depths q_t becomes less scattered and there is a clear disparity between the two tests of approximately 170kPa.

Sleeve resistance (f_s) measurements were also affected by the temperature differential. Similar to the q_t measurements above 2.5m f_s values are slightly more scattered for the cone which has not been equilibrated. Below 3.5m there is a clear difference of approximately 2 kPa.

Note that the negative values of q_t and f_s recorded in these tests are purely due to temperature effects. The change in temperature between the CPTU truck and the ground causes an electronic shift in the transducer readings hence recording negative values in some cases.

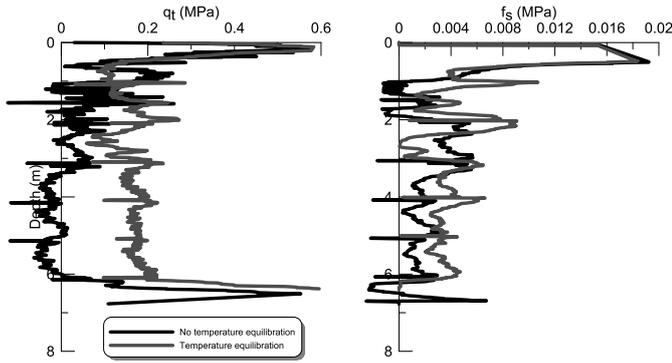


Figure 8 Temperature effects on CPTU results in peat

This problem can be overcome by equilibrating the cone to the ground temperature (ideally in a bucket of water taken from the ground itself) prior to the test commencing. It is therefore especially important in peat that CPTU testing is carried out to a recognised standard. Up to the present the best official guideline for performing CPTU was the IRTP (International Reference Test Procedure) published by the International Society for Soil Mechanics and Geotechnical Engineering in 1999 (ISSMGE, 1999). A European Standard (ENISO 22476-1, 2007) has been completed and will be officially available from late 2012 or early 2013. This document is an updated version of the IRTP, based on the same principles.

Boylan et al. (2008) also detailed the results of CPTU tests which were also carried out at the Vinkeveen site using various cones with differing accuracies. Figure 9 shows the results of all tests in terms of q_t , f_s and u_2 . Also indicated on this figure are the accuracy ranges required for an Application Class 1 test by the European Standard for piezocone testing (ENISO 22476-1, 2007). This application class is the intended accuracy class for soft to very soft soil deposits. For q_t the accuracy range corresponding to Class 1 is 35 kPa or 5% of the measured value whichever is less. For f_s and u_2 the corresponding values are 5 kPa / 10% and 10 kPa / 2% respectively.

For q_t the agreement between the various cones is generally good. However, the range of values falls outside the accuracy range for an Application Class 1 test if all the tests are considered together. Similarly, the range of f_s values fall outside the accuracy range of an Application Class 1 test if all the tests are considered together. During testing all cones recorded pore pressures close to the hydrostatic pore pressure and the range of measured values is generally within the required accuracy range.

The pore pressure (u_2) appears to be more reliable than the corrected resistance (q_t) or friction sleeve resistance (f_s). Long (2008) presented similar findings from a number of comparative cone trials in clay.

Piezocone testing of soft organic soils often involves measuring parameters which are close to the accuracy of the equipment and can be highly influenced by factors such as temperature, zero offsets, poor calibration, testing procedures etc. The European Standard for piezocone testing (ENISO 22476-1, 2007) provides comprehensive guidelines on all aspects of piezocone testing and the factors which influence its accuracy. Adoption of this standard is crucial for accurate investigation with CPTU in organic soils.

3.3 Profiling peat from CPTU tests

Lunne et al. (1997) provide a useful review of the use of the CPTU in peat and organic soils. They summarised case histories of work in peat from Holland, Germany and Canada. They suggest that peat is often characterised by a high friction ratio ($R_f = f_s/q_t$) greater than perhaps 5% and that negative pore pressures can be developed in fibrous zones.

Long (2005), Boylan and Long (2006a) and Boylan and Long (2006b) investigated the use of the pore pressure parameter (B_q) in order to characterise the degree of decomposition of the peat.

$$B_q = \text{pore pressure parameter} = \frac{\Delta u}{q_{net}} = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \quad (3)$$

where: u_0 = ambient or in situ pore water pressure and σ_{v0} = in situ total vertical stress

This was motivated by the finding, discussed above, that pore pressure (u_2) tends to be the most reliable measured CPTU parameter. Peat permeability changes as it decomposes and this should be reflected in measured B_q values.

An example of such profiling for the Crockagarron site (Figure 4c) is shown on Figure 10. The degree of decomposition, as expressed by the Von Post and Granlund (1926) H value increases with depth in a similar manner to B_q and both reach a maximum at about 2.7 m.

Although this approach shows some promise further work is needed before definite recommendations on the link between B_q and H can be made. A particular issue here is the consistency of definition of degree of decomposition as this can be a subjective and operator dependant parameter.

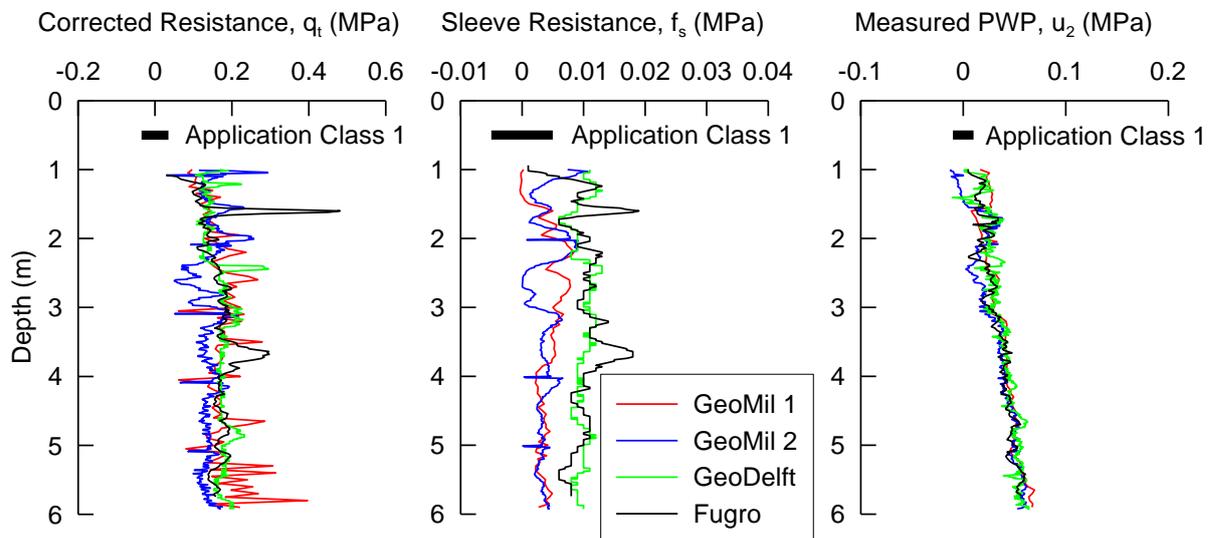


Figure 9 CPTU tests using different cones at Vinkeveen site

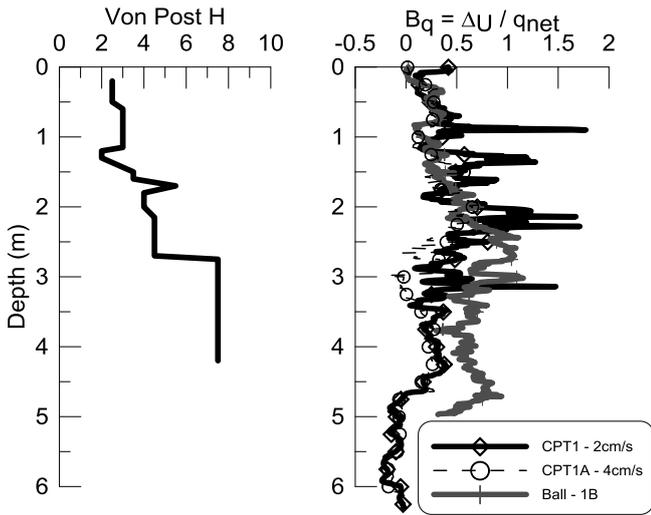


Figure 10 Peat degree of decomposition profiling using B_q for Crockagarron site

3.4 Use if standard CPTU classification charts for peat

Use of the CPTU for classifying soil has now gained world-wide acceptance. A number of well established soil classification or soil behaviour charts exist. Generally these charts use a combination of corrected cone resistance (q_t), sleeve friction (f_s) and pore water pressure (u_2) data or normalised parameters derived using these measurements.

Work by Mollé (2005) and Long (2008) suggested that the soil behaviour charts of Robertson et al. (1986) and the similar normalised chart of Robertson (1990) were perhaps the most widely used charts world-wide. They found that these charts are adequate to reasonably accurately characterise uniform soft to medium stiff clay and uniform sand sites and to a lesser degree for some intermediate soils such as silty clay or clayey silt and sandy silt. This work also highlighted the importance of reliable f_s measurements. Inaccuracies in f_s measurements can decrease the reliability of the charts.

However there seems to be difficulties with the use of the charts for characterising peat and organic clay soils. An example for the Bundoran – Ballyshannon Bypass site, Co. Donegal, Ireland is shown on Figure 11a (Long, 2005; Long and Phoon, 2004). This site is underlain by approximately 3 m of peat over calc marl (soft silt) over, soft sensitive clay. Consistent with the suggestion of Lunne et al. (1997), R_f values are high in the peat being in the range 4% to 12%. The more fibrous upper peat is clearly distinguished from the deeper more amorphous peat by the higher q_{net} and R_f values. However the underlying calc marl shows similar R_f values to the peat, albeit with higher u_2 or B_q values.

Although the Robertson et al. (1986) chart, shown on Figure 11b, accurately classifies the deeper soft silty clay it fails to separate the peat from the calc marl despite these two strata having significantly different geotechnical properties. They are classified as either “organic material” or “clay” on both charts. In addition in many circumstances fibrous peat can have high q_t values and the soil behaviour charts will then classify the material in zones 4 to 7, i.e. mixed silt and clay soil.

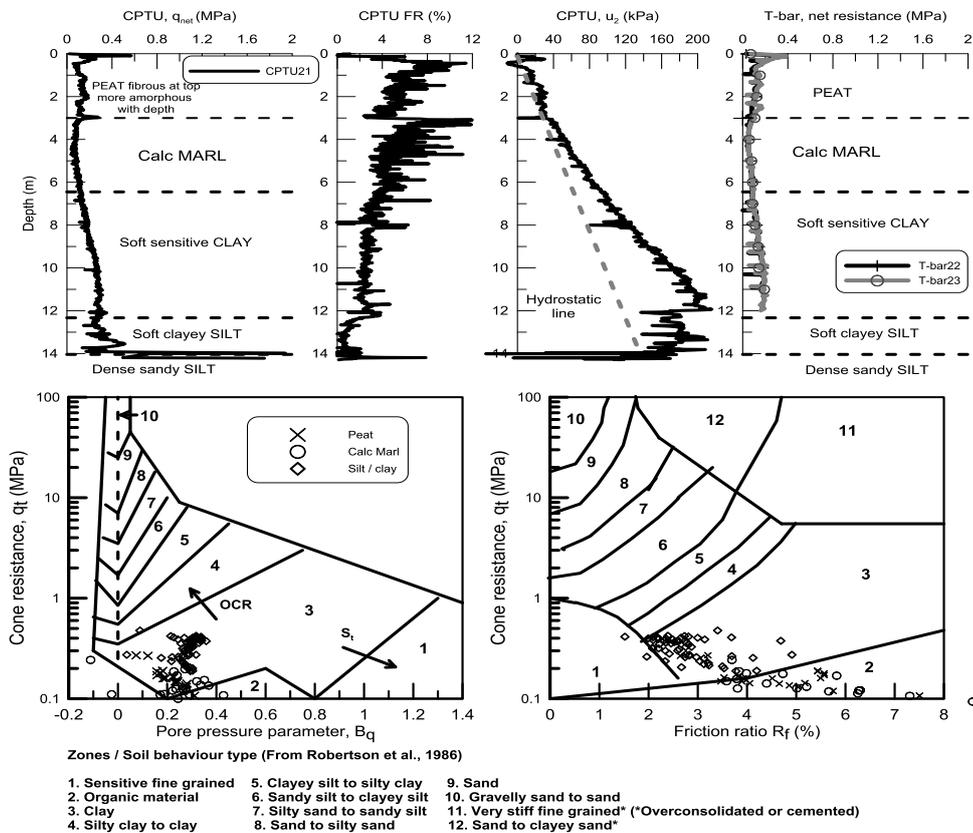


Figure 11 Bundoran – Ballyshannon Bypass (a) CPTU and T-bar test results and (b) Robertson et al. (1986) soil behaviour type chart

Similar findings were made by Long et al. (2010) for the organic soils at Crayford, east of London and Liew (2008) for a site near Kuala Lumpur. Misclassification at these sites was due to the partially drained nature of the penetration, leading to high measured resistances, the unreliability in the sleeve friction readings and the influence of the reinforcing fibres.

3.5 Strength of peat from CPT tests

Landva (1986) and Boylan (2008) carried out laboratory tests which studied the deformation around a half cone penetrated into peat behind a glass screen. These tests showed that large amounts of vertical compression were required to mobilise the strength of the peat, indicating that the peat undergoes considerable consolidation during CPT penetration. Landva (1986) concluded that the CPT is “of little use” in determining the engineering properties of peat soils.

A very significant issue is that cone penetration in peat may not be fully undrained. Pore pressure parameter (B_q) values from CPTU in Irish peat are generally less than 0.3 (Boylan and Long, 2006a; Long, 2005). This indicates that the material behaves in a partially drained to drained manner (Schnaid et al., 2004). Therefore, any correlations between q_{net} and s_u will be influenced by the level of drainage which takes place during penetration. It is also worth exploring correlations based on u_2 .

Despite this the standard approach used to determine s_u for clay soils is often applied to peat. A series of empirical bearing capacity factors N_{kt} , N_{ke} and $N_{\Delta u}$ (Lunne et al., 1997) have been used for this purpose, i.e.:

$$s_u = \frac{q_{net}}{N_{kt}} = \frac{q_t - \sigma_{v0}}{N_{kt}} \quad (4)$$

$$s_u = \frac{q_t - u_2}{N_{ke}} \quad (5)$$

$$s_u = \frac{\Delta u}{N_{\Delta u}} \quad (6)$$

There are few published studies which look at the range of N_{kt} factors for peat soils. Hanzawa et al. (1994) suggested the use of an N_{kt} of 10, based on an empirical relationship with s_u results from the direct shear test. However, the CPT penetration profiles used to develop this relationship were not corrected for out of balance pore pressure effects. Long (2005) found good agreement using an $N_{ke} = 9$, with the results of in-situ vane tests and unconsolidated undrained (UU) triaxial tests for a site in Co. Mayo.

Due to the large correction required for out of balance pore pressure effects in soft soils and peat, Den Haan and Kruse (2007) preferred to divide q_c directly by a factor to obtain s_u and suggested a value of 7.8 based on triaxial compression tests. They also emphasised the need for more empirical relations between q_c and s_u to substantiate this relationship.

Some of the author’s experience of use of these parameters will be presented in Section 4.3 together with the full flow probe data.

4. FULL FLOW PROBES

4.1 Introduction

Full flow probes including the T-bar and ball (see Figure 12) have been introduced in an attempt to overcome the problems associated with measuring resistances in very soft sediments. Very early work in the area was carried out in the late 1930’s at the Swedish Geotechnical Institute (SGI) as reported by Kallstenius (1961) who describes the SGI Iskymeter, which is not unlike the T-bar penetrometer. Flaate (1962) in a discussion of the shearing resistance of peat, also suggested the iskymeter “may be of some help”.

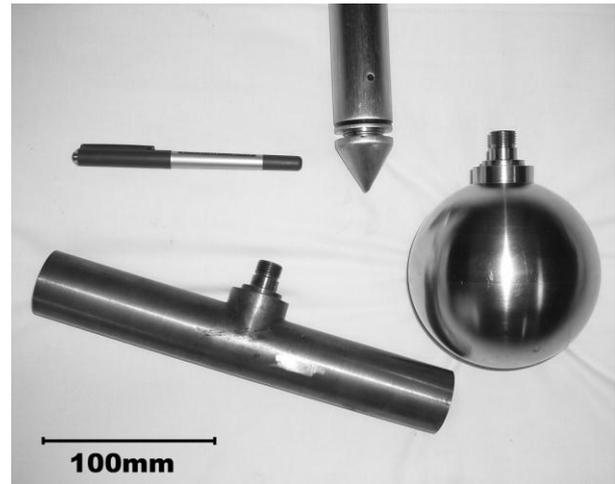


Figure 12 Full flow probes Boylan and Long (2006a)

The T-bar and ball probes are becoming increasingly popular for characterising soft sediments, particularly in offshore environments. Ball probes are often used for deep profiling as they can fit inside casing used for offshore works. T-bars are regularly used for shallower studies such as for pipelines. In peat some problems can be encountered with the T-bar due to bending effects on the T-bar load cell and some difficulties in the buckling of the driving rods when it is eccentrically loaded (Long and Gudjonsson, 2004). In these tests the cone end is removed and is replaced by either a T-bar, typically 40 mm in diameter and 250 mm long or a ball of diameter 113 mm (i.e. area = 100 cm², 10 times that of a conventional cone).

Given the known reliability of pore pressure measurements in soft clays, pore pressure sensors have been added to both the T-bar (Peuchen et al., 2005) and the ball (Kelleher and Randolph, 2005), (Peuchen et al., 2005) and (Boylan et al., 2007). The balls used were developed by Benthic Geotech, Fugro and Lankelma respectively.

Some balls have recently been introduced which permit the measurement of pore pressure at several locations, see Figure 13. The UWA piezoball, described by Boylan et al. (2011b) is 60 mm in diameter has 4 small sensors around the equator of the ball and one at its tip. The In Situ Site Investigations ball is 113 mm in diameter and allows pore pressure measurement at the ball tip, mid face and equator.

An example of some data from the In Situ Site Investigations probe at the Camster site in Scotland is shown on Figure 14. For this site it was found that u_{ball} data from the mid face position showed highest values and data from the equator was very similar to the in situ pore pressure values.

Long (2008) reviewed the use of this equipment and found that although they produced useful and promising results a significant issue is that there is no standardisation in the design of these instruments, particularly with respect to the location of the pore water pressure transducer.

4.2 Application of full flow probes to peat - general

Application of full flow probes to peat and organic soils has been discussed by Oung et al. (2004), Boylan and Long (2006a), Long et al. (Long et al., 2010) and Boylan et al. (2011a). A particular feature of the latter study is that the full flow probe results are compared to laboratory testing on high quality Sherbrooke block samples.

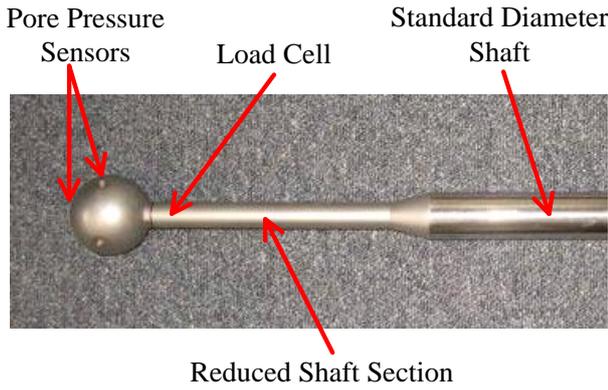


Figure 13 Piezoballs developed by (a) University of Western

The results of penetration tests using the T-bar and ball show that they overcome some of the problems of the CPTU in peat namely the scattering due to interaction with fibres. The resistance profiles are more repeatable and more uniform than those of the cone, which may be explained by the larger volume of material mobilised during penetration and reduced sensitivity to small fibres. This effect can be seen clearly in Figures 7, 11a and 14 for the Crockagarron, Bundoran – Ballyshannon and Camster sites respectively.

Resistance profiles from the T-bar and Ball are similar with those from the T-bar tending to be higher than those from the Ball. Analytical solutions suggest that Ball resistances should be higher. It is the opinion of the author's that the end effects of the T-bar and its interaction with the fibres may be the origin of this difference.

The Crockagarron and Bundoran – Ballyshannon examples above show that resistance profiles from the CPTU show a tendency to increase with depth at a rate higher than the T-bar and Ball which has been noted in other soft soils (Chung and Randolph, 2004; Long and Gudjonsson, 2004).

The pore pressure parameter (B_{Ball}) from the ball penetrometer test has been shown to be a useful parameter to differentiate the decomposition of material within peat deposits, similar to CPTU B_q described above in Section 3.3. B_{Ball} (and B_q) values show a tendency to increase with peat decomposition.

4.3 Undrained strength of peat from full flow probes

Perhaps the principal objective for engineers carrying out full flow probes in peat is to determine the undrained shear strength for the design of infrastructure located on the peat or for the assessment of slope stability in peatlands.

Similar to the CPTU, described above in Section 3.5, s_u can be determined from the piezoball measurements using empirical factors N_{ball} and N_{u-ball} as follows (Boylan et al., 2011a):

$$s_u = \frac{q_{ball}}{N_{ball}} \quad (7)$$

$$s_u = \frac{u_{ball} - u_0}{N_{u-ball}} \quad (8)$$

Given the likely influence of partial drainage on penetration resistance, care needs to be taken when interpreting penetration tests in these soils. Dimensional analysis shows that the degree of partial drainage during continuous penetration is controlled by the normalised velocity V , defined as (Finnie and Randolph, 1994):

$$V = \frac{vd}{c_v} \quad (9)$$

where: v is the penetration rate, d is the diameter of the penetrometer and c_v is the coefficient of consolidation of the soil. For penetration processes, drained conditions exist for $V < 0.01$, while for $V > 10$ undrained conditions exist (House et al., 2001). The tests conducted with the larger penetrometers will therefore have a greater normalised velocity (V) than the cone. Depending on the consolidation properties of the soil it may measure a resistance that is representative of a lower degree of partial drainage.

Therefore, the use of full flow probes appears beneficial in increasing the sensitivity of measurements and possibly reducing the effects of partial drainage on the measured resistance.

For peat sites variable rate penetration tests should be undertaken over the widest range of penetration rates available to provide insight into the drainage conditions during penetration, and possibly the penetration rate required for undrained penetration. The insight that could be gained from variable rate tests in these soils is often limited by the range of penetration rates available on most commercial penetration rigs.

This can be seen, for example, for the data from the Camster site on Figure 14b.

Available bearing capacity factors derived from the results of CPTU, piezoball tests and laboratory simple shear tests carried out at in situ vertical effective stress (Long and Boylan, 2012) are shown on Figure 15. A summary of the values is given on Table 2.

N_{kt} and N_{ball} factors show significant scatter. The scatter is greatest for the shallower tests possibly due the effects of partial drainage resulting in relatively higher resistance values. Long and Gudjonsson (2004), Boylan and Long (2006a) and Boylan et al (2011a) found that computed N_{T-bar} and N_{ball} factors for the full flow probes showed less variance than the N_{kt} factor for the CPTU. A similar finding is made here with N_{kt} showing slightly more variation than N_{ball} . N_{kt} values are generally higher than typical values published for clays (Karlsrud et al., 2005). Note that the work of Karlsrud et al. used s_u from anisotropically undrained triaxial compression tests (CAUC) as reference, whereas DSS tests are used here. For clays s_{u-DSS} is typically 0.7 to 0.85 s_{u-CAUC} .

Table 2 Summary of empirical bearing capacity factors

Item	N_{kt}	N_{ball}	N_{Au}	N_{u-ball}
No. of data points	12	25	18	13
Minimum	8.5	7.2	0.9	1.3
Maximum	44.0	37.0	8.2	7.8
Average	21.2	18.9	3.7	3.7
Standard deviation	13.0	8.8	1.9	1.9

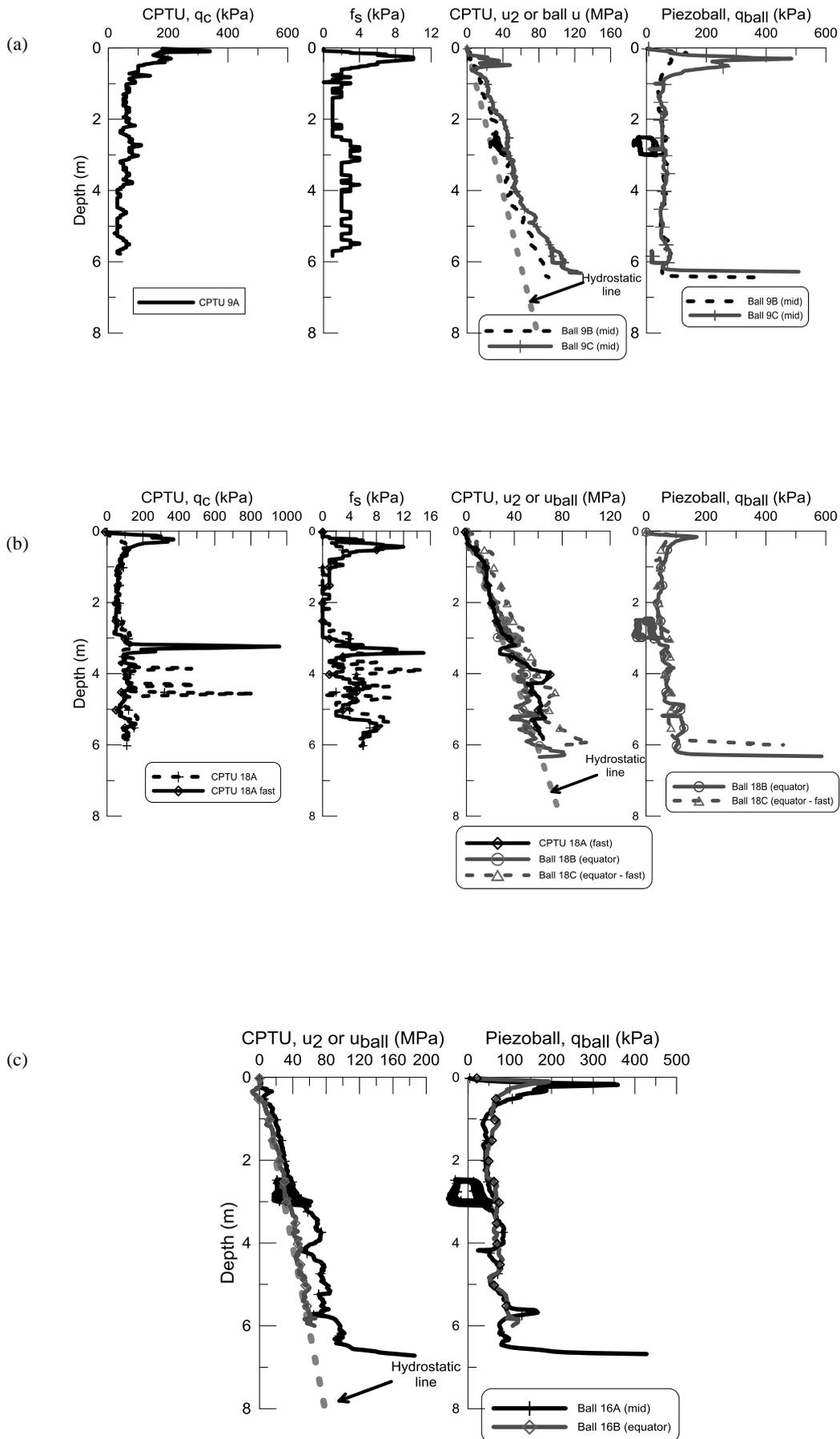


Figure 14 - CPTU and full flow probe data - Camster site, Scotland (a) Location 9 and (b) Location 18 and (c) Location 16

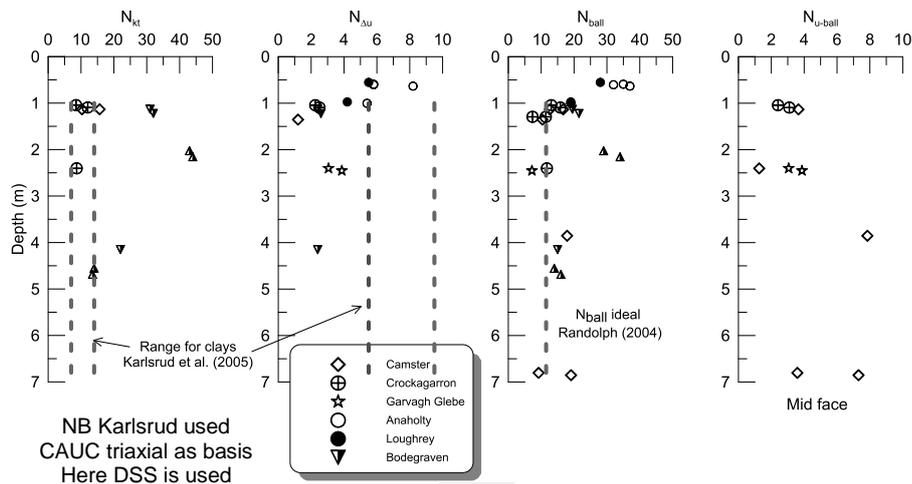


Figure 15 – CPTU and piezoball empirical bearing capacity factors as related to s_u from simple shear tests

Theoretical resistance factors have been derived for the ball based on plasticity solutions, e.g. Randolph (2004). N_{ball} was theoretically found to range between 11 and 15.3 for undrained conditions depending on the interface roughness of the penetrometer. N_{ball} values presented here are generally higher than this range due to particle drainage effects increasing the q_{ball} measurement above the value that would have occurred if undrained conditions were achieved.

N_{Au} and N_{u-ball} also show significant scatter. N_{Au} values are less than those typically applied to clay soils, again because of partial drainage effects. The range of values as well as average and standard deviation value are similar for N_{u-ball} and N_{Au} . It is clear that the effects of partial drainage on the measured values need to be carefully assessed before applying these factors.

4.4 Summary

There would seem to be good evidence to suggest that full flow penetrometers, particularly the piezoball, can be useful tools in profiling peat. The ball should be used in conjunction with traditional CPTU tools. There seems particular promise in the use of the pore pressure measurements for both the CPTU and piezoball for the purposes of profiling peat decomposition. Partial drainage effects need to be carefully assessed prior to application of bearing capacity factors to the test results so as to derive undrained shear strength. The benefit of multi-measurement tests, such as the CPTU and piezoball, is that there additional information available to help assess the drainage condition.

The ranges of bearing factors presented in this paper are examples and should not be interpreted as recommended values. Laboratory tests should be carried out on an adequate range of samples to determine a site specific bearing factor.

5. USE OF GEOPHYSICAL TECHNIQUES IN PEAT

5.1 Ground penetrating radar (GPR)

Ground penetrating radar (GPR) techniques involve the transmission and reflection measurement of electromagnetic waves. The penetration depth achievable depends on the nature of the peat (especially its electrical conductivity), the location of the water table and on the frequency of the transmitted wave.

Work at Lund University in Sweden (Ulriksen, 1979; Ulriksen, 1980; Ulriksen, 1983), (Bjelm and Ulriksen, 1980), (Bjelm, 1980) investigated such factors as the effect of frequency of the transmitted wave, the transmission velocity and the technique used for moving the antenna over the peat on the measured results. This work also showed that not only could the peat thickness be estimated accurately, some information can be obtained on the material beneath the peat.

These techniques have been also used successfully for many years in Sweden (Carlsten, 1988) and Finland (Saarenketo et al., 1992) for the determination of the thickness of both the road pavements and that of the underlying peat. Edil (2001) reported similar findings for work in the US.

To date most equipment has involved moving a single frequency transmitter over the surface of the peat. For example Trafford (2009) reported on use of a 100 and 250 MHz transmitter for the survey of a large area of peatlands in Central Ireland, either by man hauling the antenna or by use of all terrain vehicle (Figures 16a and 16b). A variety of challenging conditions can therefore be dealt with. Trafford (2009) found that the maximum depth of penetration for the 100 MHz transmitter in Irish raised bogs was typically 6 m.

Transmitters with varying input frequency have also been used. For example for the equipment shown on Figure 16c the input frequency can be altered by changing the length of the boom. In Ireland it has been found that a good compromise between depth of penetration and resolution of data can possible be found by combining results from two different frequency inputs, e.g. 80 MHz and 40 MHz.

Some output from the work at Clara raised bog in Central Ireland in shown on Figure 17. Probing (left hand side on Figure 17) revealed approximately 5.6 m of peat over silt and clay. This boundary is clearly identified in the GPR data. In addition GPR is able to resolve some internal boundaries son the peat for example that at about 2.5 m between the sphagnum and underlying fen peat. Further work in this area is well warranted.

GPR work is now usually linked to an accurate GPS system which allows spatial relocation to GPS co-ordinates as well as providing topographic information. These systems are now being used regularly in design and risk assessment for infrastructural works on peatlands. The example on Figure 18a is for a windfarm site in Co. Donegal, Ireland where the GPR trace clearly identifies the shallow peat filled valleys in between competent soils or rock. On Figure 18b the GPR and GPS data are integrated to produce a useful image of the variation in ground surface and peat bottom for a raised bog at Roosky, Co. Longford.

5.2 Other geophysical techniques

Geophysics experts will normally recommend that in any application two or more geophysical techniques should be used in parallel. Work on peat sites is no exception. For example in Ireland a combination of GPR, electrical resistivity tomography techniques (ERT) and multi channel analysis of surface waves (MASW) is often used to characterise the peat thickness and its engineering properties as well as those of the underlying mineral soils and rock.



Figure 16 (a) Man hauling 250 MHz antenna and (b) all terrain “quad” with 100 MHz antenna and (c) variable frequency antenna

6. MEASUREMENT OF IN SITU PORE WATER PRESSURE IN PEAT

Pore water pressure measurements in peat are important, for example, for monitoring dike and embankment stability (e.g. the Netherlands), for assessment of slope stability and for hydrogeological studies. A major issue with measurement of pore pressure in peat is the effect of the accumulation of gas bubbles within the measuring instrument (Baird and Gaffney, 1994; Greeuw et al., 2003; Waddington et al., 2009).

Greeuw et al. (2003), for example, reported on a number of long term measurements in the Netherlands where unexpected increases in measured pore pressure occurred. These authors give some useful recommendations for the design of piezometers for peat, for example that a larger filter area than normal is necessary.

7. OTHER IN SITU TESTING TECHNIQUES

7.1 Marchetti dilatometer (DMT . SDMT)

A dilatometer test consists of pushing a flat blade located at the end of a series of rods into the ground (Marchetti, 1980). Once at the testing depth, a circular steel membrane located on one side of the blade is expanded horizontally into the soil. The pressure is recorded at specific moments during the test (p_0 on contact and p_1 at 1.1 mm expansion). The blade is then advanced to the next test depth. Various soil parameters can then be derived empirically from these measurements together with knowledge of the in situ effective stress and pore water pressure. Recently Marchetti et al. (2008) have introduced seismic piezocone (SCPTU) technology into the DMT, by the inclusion of geophones, to form the seismic dilatometer (SDMT).

Application of the DMT in peat has been reported by Nichols et al. (1989) and (2006; Mlynarek et al., 2010). The technique was shown to be simple to use, robust and repeatable. Results were mixed but there seems to be good scope for developing local correlations. Edil (2001) also reported that there are a few examples of the application of DMT tests in peat soils but there are no available guidelines in the interpretation of such tests. A significant issue with the test is that the maximum displacement of the DMT blade is 1.1 mm and this may not be adequate in highly organic fibrous peat. Rahardjo et al. (2004) described the DDMT (dual dilatometer), which is a standard Marchetti DMT with additional thicker blade attached at the top, to produce larger strains that are postulated to improve sensitivity in soft soils. The equipment was tested in soft clays and peats at Pelintung, Sumatera. No clear improvement in interpretation was evident.

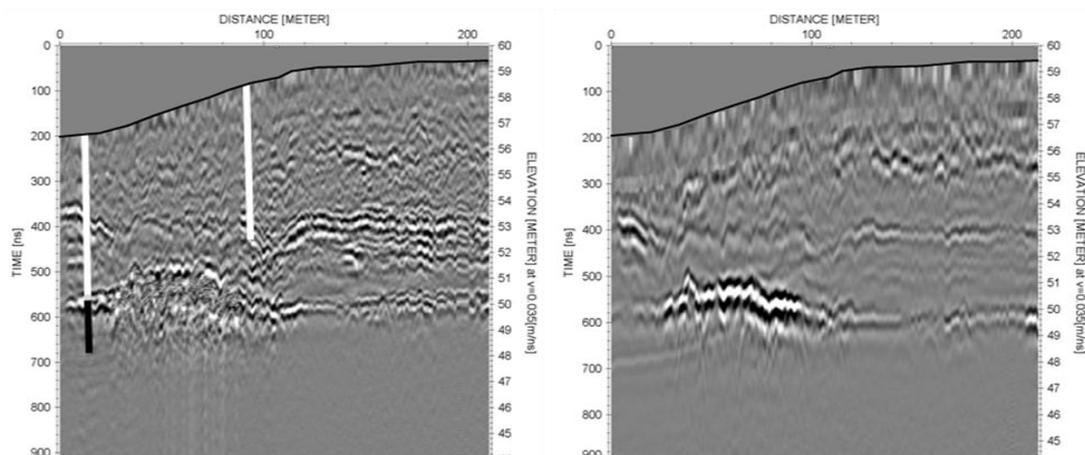


Figure 17 Output for 80 and 40 MHz transmitters at Clara bog

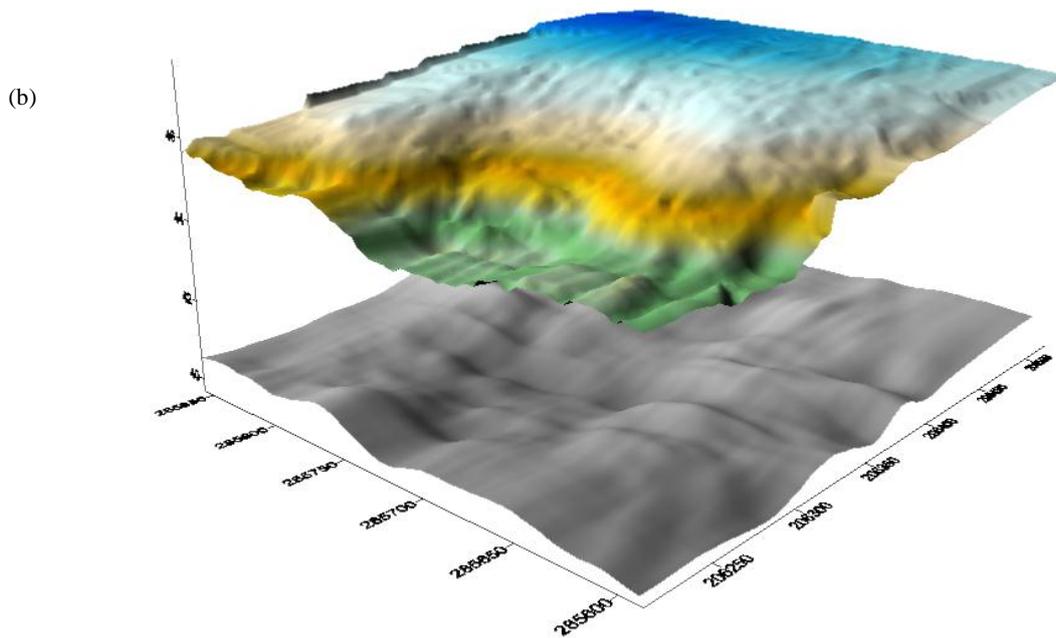
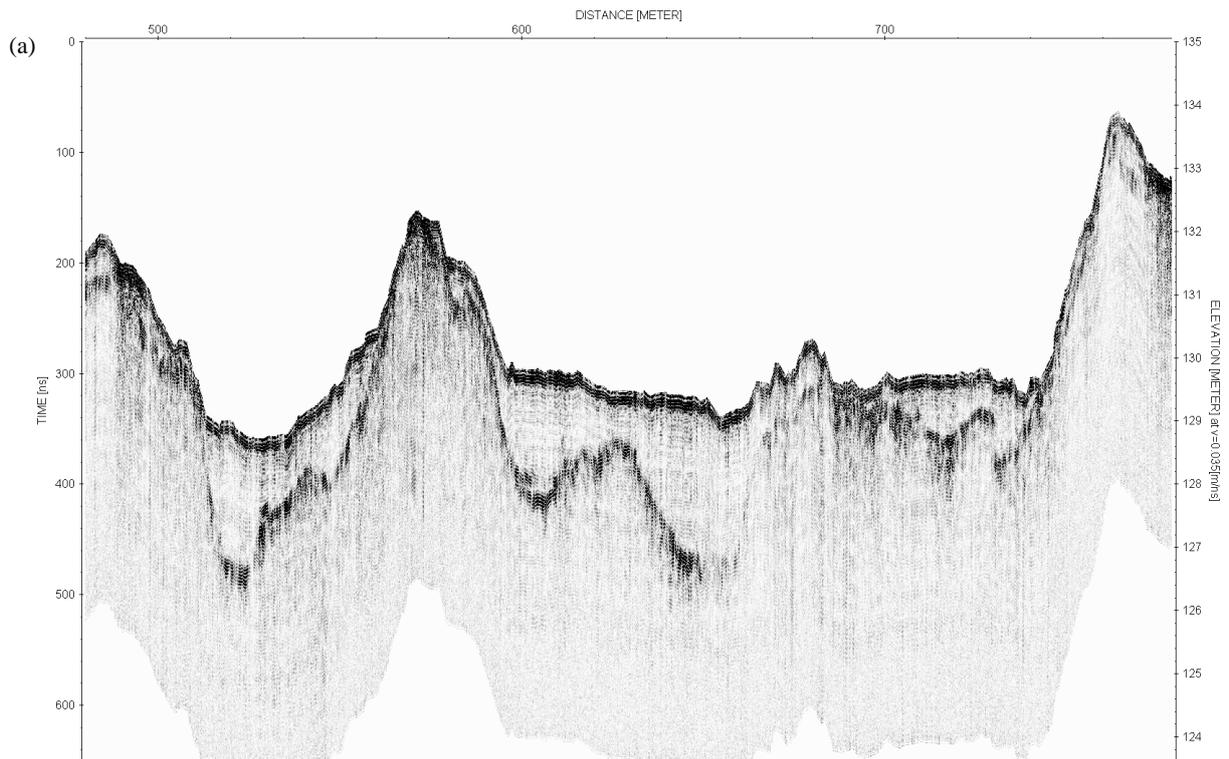


Figure 18 (a) 250 MHz GPR trace for windfarm in Co. Donegal, Ireland, (b) image of ground surface and peat bottom produced by integrating GPT and GPS data for the Roosky raised bog in Co. Longford. Note y-axis reads in mOD

7.2 Pressuremeter

There are also a few examples of the use of the pressuremeter in the literature, see for example Nichols et al. (1989). The test suffers from similar problems to the DMT, i.e. disturbance on penetration, lack of sufficient strain and no published guidelines on carrying out the tests and interpretation of the results. Landva et al. (1986) stated that the test is not recommended for work in peat.

7.3 Plate load tests

Landva (1986) and Landva (2007) gave some detailed analyses of plate load tests on peat. It was found that the results of plate load tests could not easily be applied to peat either for studies of compression or failure. This is because the test is not representative of the mode of deformation of real structures on peatland and is therefore "of little geotechnical significance". An exception was found to be the study of very concentrated loads, e.g. from vehicles.

7.4 Other in situ tests

Kramer et al. (1990) evaluated the strength of peat based on the results of full-scale lateral load tests on 8 inch diameter steel pipe piles. Undrained strength backanalysed from the trial was about twice that obtained from field vane tests and UU triaxial tests.

8. CONCLUSIONS

This paper has provided a review and an update on some recent developments on in situ testing of peat for civil engineering purposes. It was found that:

- Geophysical techniques, particularly ground penetrating radar, are very useful for profiling peat deposits rapidly and economically.
- CPTU testing can also be useful for profiling these materials. However due to the lack of homogeneity in the material and the very low measured values, the tests need to be carried out carefully to well established guidelines. The influence of partially drained penetration and the reinforcing effects of fibres need to be thoroughly assessed.
- Full flow probe testing is a useful compliment to the CPTU. In particular the u_{ball} and u_2 measurements may be very useful for both assessment of peat shear strength and assessment of the degree of decomposition of the peat. Further work is required to develop these methods and engineers should develop site specific correlations based on tests using high quality samples to determine the range bearing capacity factors to be used.

- Some standardisation of full flow probe testing (especially for pore pressure measurement) would be of great value.
- Vane testing in peat will often give misleading results and should only be used with great caution.
- Pore pressure measurements in peat can be problematic due to the presence of gas in the deposits.
- Many other standard geotechnical tests have been tried in peat but only seem useful if they are used in conjunction with locally developed empirical correlations.

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LIST OF SYMBOLS

- B_q = CPTU pore pressure parameter
 c_v = coefficient of consolidation
 d = instrument diameter
 f_s = CPTU sleeve resistance
 H = degree of decomposition
 N = bearing capacity factors
 q_t / q_{net} = CPTU corrected and net end resistance
 R_f = CPTU friction ratio
 s_u = undrained shear strength (s_{u-DSS} from direct simple shear test, s_{u-FV} from field vane, s_{u-CAUC} from triaxial compression test)
 u = pore water pressure
 v = penetration rate
 V = normalised velocity
 w_L = liquid limit
 μ_{FV-C} = field vane correction factor
 σ_v / σ'_v = total / effective vertical stress (σ'_{v0} = in situ)

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