REVIEW OF PEAT STRENGTH, PEAT CHARACTERISATION AND CONSTITUTIVE MODELLING OF PEAT WITH REFERENCE TO LANDSLIDES

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1. 0 INTRODUCTION

The correct estimation of the shear strength of the peat subsoil is one of the major aspects of assessing risk of landslip in blanket peat areas. Due to the fibrous structure of the peat, its behaviour during shear is somewhat different from that of mineral soils (e.g. soft clays) and it has been found difficult to obtain reliable values of its shear strength. To estimate the shear strength of the peat, the geotechnical engineer is faced with making use of a several testing techniques, both in the laboratory and in situ, which were actually developed for mineral soils. Whether these techniques are applicable to peat is not clear.

The objectives of this note are to review:

1. available methods of characterising peat,
2. previous research that has been carried out on estimating the shear strength of peat in Ireland and elsewhere using both laboratory and in situ testing,
3. efforts that have been made to model peat using advanced numerical modelling.

Following the review some recommendations will be made as to which test type is most applicable to peat soils and recommendations will be made for further work.

2.0 CHARACTERISING PEAT

2.1 Manual classification systems

The best known classification system for peats is that of von Post (von Post and Granlund, 1926). It is based on categorisation of botanical composition, degree of humification, water content, content of fine and coarse fibres and content of woody remnants. It was originally devised to aid the development of an inventory of peat resources in southern Sweden. According to the von Post scale peat is classified as being between H1 (completely unhumified fibrous peat) and H10 (completely amorphous non fibrous peat). Hobbs (1986) extended the system with categories for organic content, tensile strength, odour, plasticity and acidity. A recent Dutch proposal (Delft Geotechnics, 1994) for a new classification extends von Post with only organic content, but is more detailed in naming the various botanic and mineral constituents.

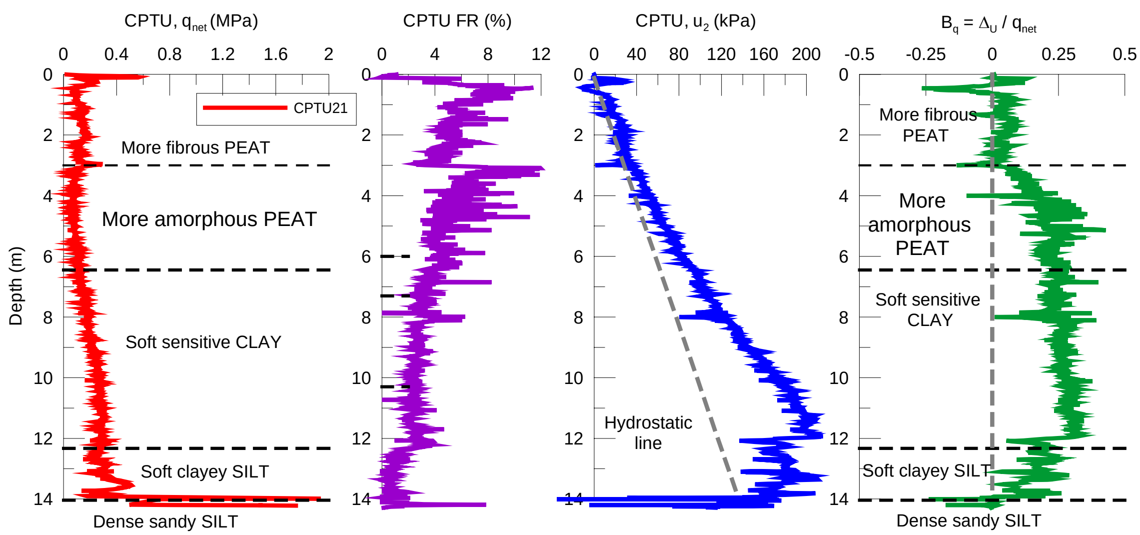
The Radforth system of classification was developed for highly organic muskegs in Canada. Landva and Rochelle (1983) speak of a Radforth peat when the mineral content is very low.

Mangan (1994) presented the French system of classification for organic soils. The 10 degrees of humification of von Post are reduced to 3 classes for fibrous, semi-fibrous and amorphous peats. It is interesting to note the similarity of this system to a modern Swedish system (Larsson, 1990). Also in North American literature, 3 classes of degree of composition are sometimes used: fibric, hemic and sapric states (den Haan et al, 1995).

Ideally of course a detailed characterisation of peat, such as that of von Post, is very useful. However much experience is needed for an accurate consistent classification. For the purposes of normal engineering works, the use of a relatively simple three point system may be sufficient.

2.2 Peat characterisation by CPTU

Lunne et al. (1997) provide a useful review of the use of the CPTU in peat and organic soils. They summarise case histories of work in peat form Holland, Germany and Canada. In conclusion they suggest that peat is characterised by a high friction ratio (R_f) greater than perhaps 5% and that negative pore pressures can be developed in fibrous zones.



Bundoran / Ballyshannon Bypass CPTU21 / T-Bars 22 & 23

Figure 1. Use of CPTU as a profiling tool

An example from some recent work in Ireland during the ground investigation for the Bundoran / Ballyshannon bypass in County Donegal is shown on Figure 1. This site is underlain by approximately 6.5 m of peat (raised bog) over soft sensitive clay. As suggested by 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 net cone resistance and the pore pressure being either hydrostatic or slightly negative. In contrast positive excess pore pressures are generated in the more amorphous lower peat. It can also be seen that the pore pressure parameter, B_q is particularly useful in delineating the two separate peat zones. In the fibrous zone B_q is close to zero, whereas in the more amorphous zone, B_q is about 0.25.

It would seem there is much promise in the use of the CPTU as a profiling tool in peat soils. However further work is necessary to relate measured (or derived) CPTU parameters to actual properties of the peat.

3.0 PREVIOUS RESEARCH INTO THE SHEAR STRENGTH OF IRISH PEAT

The assessment of shear strength of Irish peat started by Prof. E. T. Hanrahan at University College Dublin (UCD) as early as 1948, and mostly concerned the problems of road construction in raised bog areas. Landva and Rochelle (1983), in a review of the subject, conclude that this UCD work was the first reported research on the shear strength of peat. Hanrahan (1952, 1954a and 1954b) concluded that the structure of remoulded peat was unlikely to be representative of undisturbed peat and the shear strength of peat was of a cohesive nature ($\phi' = 0$) only. Subsequently Hanrahan and Walsh (1965) and Hanrahan et al. (1967) report on a comprehensive triaxial shear test program, involving 130 tests, on remoulded (macerated) peat samples and it is interesting to note that the conclusions of this paper are quite the opposite of those in the previous publications, in that it was concluded the qualitative behaviour of peat in its remoulded and undisturbed states is similar and that the strength of peat was frictional (c' = 5.5 kPa to 6.1 kPa and ϕ' = 36.6° to 43.5°).

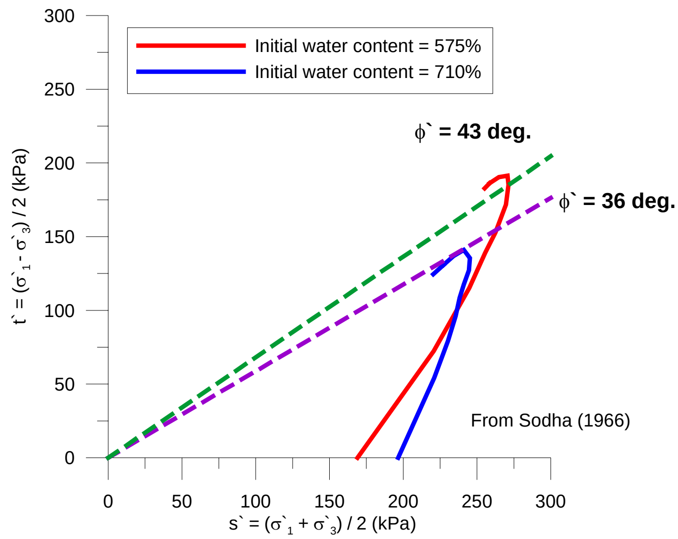


Figure 2. Behaviour of macerated peat of different water contents (Sodha, 1966)

An example from this work is shown on Figure 2 (Sodha, 1966). It can be seen that the behaviour of the material under shear in isotropically consolidated triaxial tests is not unlike that of a lightly overconsolidated mineral soil. However, unlike mineral soils, there is a different failure envelopes corresponding to different initial water contents, with ϕ' increasing with decreasing water content.

Pigott et al. (1992) made use of in situ vane test results when designing remedial works following a large failure of a canal embankment constructed of and founded on peat, see Figure 3. A back analysis of the failure suggested the mobilized shear strength along the failure plane was of the order of 5.2 kPa, whereas average vane shear strength was between 5 kPa and 20 kPa. Hanrahan (1994) acknowledges the limitations of the vane test and reports that the test tends to overestimate the shear strength of peat. However he suggested that it remains a useful simple rapid method of evaluating features such as variability with depth and hard and soft layers.

<u>Figure 3</u>. Slide on Grand Canal embankment (Pigott et al. 1992)

Mc Geever (1987), working at Trinity College Dublin (TCD), studied the difference in effective stress parameters determined from different tests and for different organic contents. The parameters determined in undrained triaxial tests with pore pressure measurements and in drained tests (for relatively low organic contents) were considerably higher in compression than extension and also considerably higher than those determined from the shear box test. Mc Geever concluded that that there was significant strength anisotropy behaviour and that this possibly accounted for the different results. It was not possible to determine the effective stress parameters from drained tests on peats because of the continuing increase in deviator stress and the continuing volumetric compression even at 50% strain.

O'Neill (1992) carried out shear box tests on artificially prepared samples of silt with varying organic contents as described by Mc Geever above. These results showed that the effective angle of shearing resistance of the silt is higher with organics than without. However the results are inconclusive in regard to a general trend.

Farrell & Hebib (1998) also working at (TCD) reported results of a comprehensive laboratory investigation into the shear strength of an Irish peat recovered from Raheenmore bog. The peat was about 98% organic and had a moisture content ranging between 1200% and 1400%. The peat was found to have undrained shear strength of about 5 kPa. The main findings of this study can be summarised as follows:

(1) The apparent effective angle of shearing resistance, as measured in undrained triaxial compression tests, was about $\phi' = 55°$,
(2) Failure as defined by peak deviator stress was not reached in drained triaxial compression tests and
(3) an effective angle of shearing resistance $\phi' = 38°$ was measured in both the direct shear box and the ring shear test whereas the direct simple shear (DSS) yielded lower value of $\phi' = 31°$.

The concurrence of the results of the ring shear and direct shear box test would suggest that ϕ' measured is representative of the matrix whereas ϕ' measured in triaxial compression is more representative of the matrix and the reinforcing effect of the fibres.

Further testing was carried out by Hebib (2001) on undisturbed peat samples recovered from Ballydermot bog. The peat was between 94% to 98% organic and had a moisture content varying between 750% to 950%. The peat samples were tested both in ring shear and in triaxial compression. An angle of shearing resistance of $\phi' = 21°$ was derived from the ring shear. Similar behaviour to Raheenmore peat was observed under undrained triaxial conditions, an angle of shearing resistance of 68° was measured. The samples tested in drained triaxial compression did not reach failure and the angle of shearing resistance was derived at an arbitrarily axial strain of 20%.

Hebib (2001) concluded that one geotechnical peculiarity of the shear strength of peat is the effect of fibres. As shown by the previous authors quoted above, the shear strength parameters of peat vary according to the type of test used. The triaxial test tends to yield higher angles of shearing resistance than the direct simple shear and this is believed to be due to the reinforcing effect of fibres. For highly fibrous peats the effect of fibres will be quite dominant, to the extent that failure may not be reached in triaxial compression. For peats with low content of fibres, the effect of fibre reinforcement will be insignificant and a shear failure may be expected to occur. In the case of the direct simple shear, direction of shearing is assumed to be parallel to the orientation of fibres therefore shear strength parameters derived from this test are likely to represent those of the peat matrix.

Farrell et al. (1998) have carried out some direct simple shear (DSS) on peat samples from the Netherlands. The peat tested was sedentary fibrous, reed-sedge and typical soils properties were, a water content of between 400% to 900% with an average of about 600% and an average loss in ignition of about 75%. The angle of shearing resistance measured from the DSS was about 34° as compared to 48° from the undrained triaxial compression.

4. 0 RESEARCH WORK IN OTHER COUNTRIES ON THE LABORATORY SHEAR STRENGTH OF PEAT

4.1 Early work

More or less in parallel with the work at UCD reported above, Adams (1961 and 1965) carried out a series of drained and undrained triaxial tests on normally and overconsolidated undisturbed peat samples of relatively low moisture content (200% –

600%). He concluded that that the shear strength of peat was frictional and measured a friction angle of 48°. He also measured $K_0 = 0.18$ and found that preconsolidation and anisotropic consolidation had little effect on the strength parameters of peat in triaxial compression.

Gautschi (1965) carried out triaxial tests on peats of different degrees of humification. Hollingshead and Raymond (1972) presented results of consolidated undrained and drained triaxial tests. Undrained behaviour was erratic and no results are given. Drained tests were terminated at 24% vertical strain without a peak strength having been reached and this point corresponded to $c' = 4$ kPa and $\phi' = 34°$.

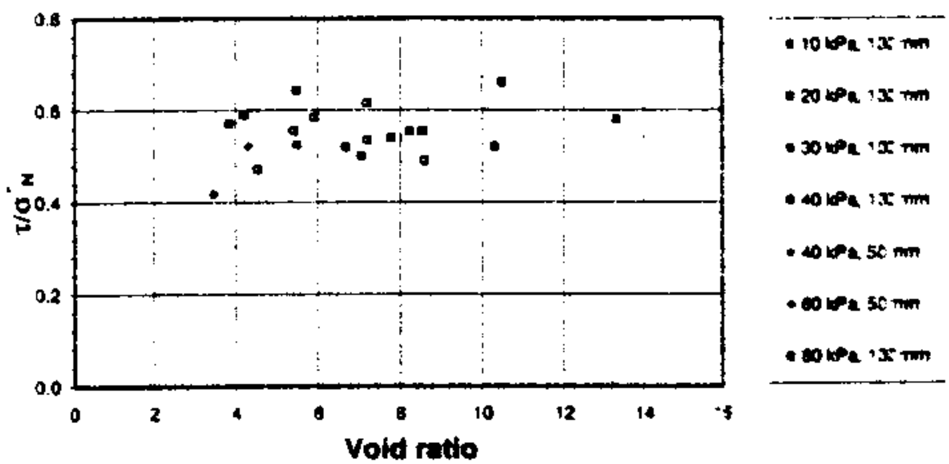
Landva and Rochelle (1983) suggested that the most reliable laboratory test for determining the effective strength parameters of peat is the ring shear test. In this test the large strains involved mean the effect of the fibres are eliminated. For peat with a moisture content of about 1200%, these authors suggested the strength parameters were typically $c' = 4$ kPa and $\phi' = 30°$.

4.2 More recent work – effective stress parameters

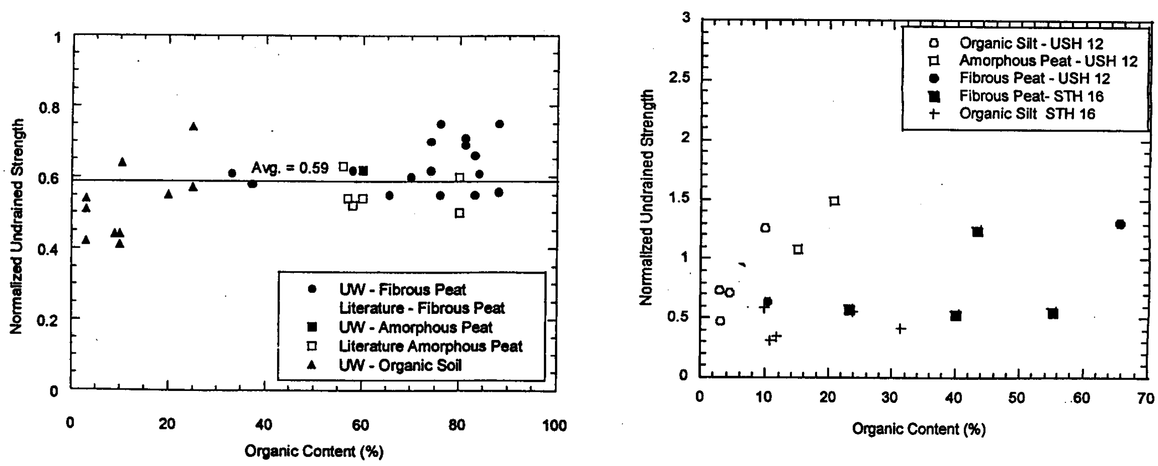
Recent work has also confirmed that effective friction angles in peat, measured in triaxial tests are very high. For example Den Haan et al. (1995) report ϕ' values of between 32° and 58°, with ϕ' increasing with decreasing density. Similar values are reported by Coutinho and Lacerda (1989) for Brazilian peat, Tsushima and Mitachi (1998) for Japanese peat and Rahadian et al. (2001) for peat in Kalimantan, Indonesia.

4.3 Undrained strength parameters

Peat is also found to have higher undrained strength ratios (s_u/σ'_{v0}) than are normally found for inorganic clays. For example, Carlsten (2000) reports values of between 0.4 and 0.55 for direct shear tests on Swedish peats, with s_u/σ'_{v0} increasing with increasing void ratio, see Figure 4.



<u>Figure 4.</u> Relationship between s_u/σ'_{v0} and void ratio for Swedish peats tested in the direct shear test (Carlsten, 2000)

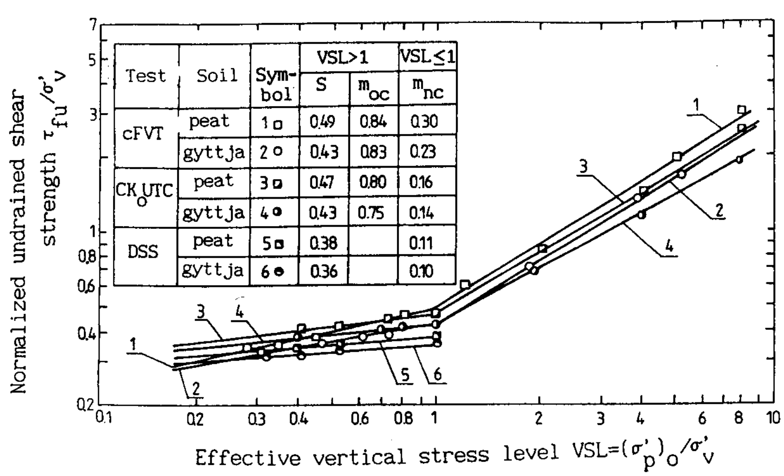


<u>Figure 5.</u> Relationship between s_u/σ'_{v0} and organic content for US peats from (a) CIU triaxial tests and (b) field vane tests (Edil, 2001)

Edil (2001) reviewed work on US peats and suggested that the range for peat is between 0.4 and 0.8 for CIU triaxial tests, see Figure 5a, with the ratio increasing with increasing organic content. For field vane testing however, there is much more scatter in the data and s_u/σ'_{v0} lies in the range 0.3 and 1.5.

Landva and Rochelle (1983) reported similar high vane strengths compared to laboratory ring shear tests (1.23 versus 0.5 to 0.7). Hanzawa et al. (1994) reported a s_u/σ'_{v0} value of 0.45 for a peat from Niigata Prefecture in Japan, based on direct shear tests and said this was lower than typical for Hokkaido peat based on field vane testing.

Work in Poland by Lechowicz (1994) and Przystański (1994) showed that the usual linear relationship for s_u/p'_c (p'_c = preconsolidation pressure), which applies to normally consolidated clays, does not work for peats. These authors suggests that a bilinear relationship applies, when the data are plotted on a double log scale, see Figure 6.



<u>Figure 6.</u> Normalised undrained shear strength (Lechowicz, 1994)

4.4 Summary of laboratory work

In brief a summary of the findings of laboratory testing of peat can be summarised as follows:

- ϕ' (triaxial) > ϕ' (DS) > ϕ' (DSS) > ϕ' (ring shear).
- This pattern is partly expected but there are clearly problems with the triaxial test values (too high?) and the DSS test (values too low?).
- s_u/σ'_{v0} values also higher than for mineral soils.
- s_u/σ'_{v0} relationship non linear for peat.
- s_u/σ'_{v0} lower and less scattered for laboratory tests when compared to field vane testing.
- Effect of fibres (anisotropy) very significant.

4.5 Anisotropy

In the early work at UCD, Hanrahan and his co-workers decided to omit the effect of the fibres from their tests by artificially macerating the peat. They were then able to how that the macerated peat behaved in a similar manner to mineral soils, see Figure 2. Furthermore they found that the results of vane tests were very similar to laboratory tests when the fibre effect was eliminated, see Figure 7a.

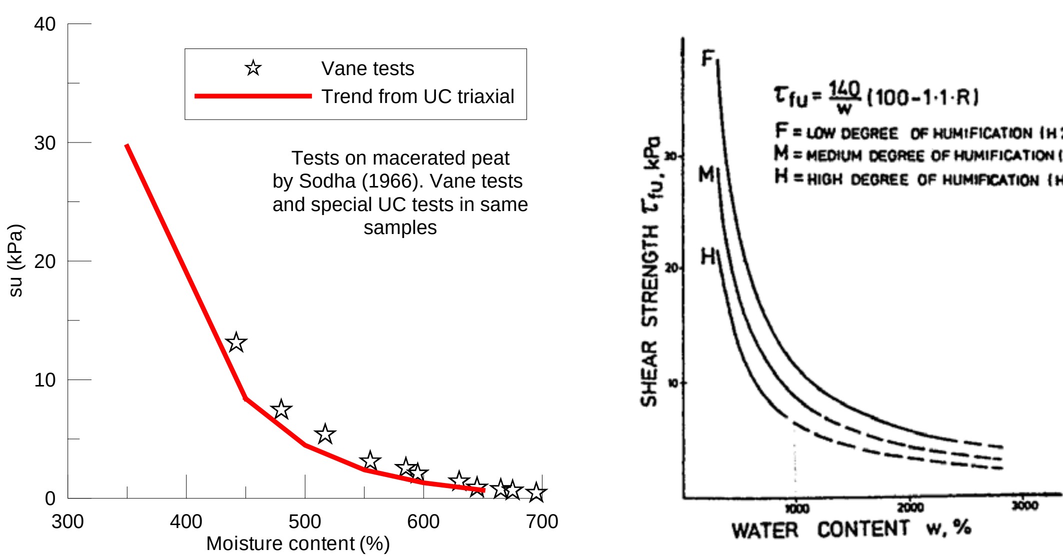


Figure 7. Vane tests and UC triaxial tests on macerated peat of different water contents (Sodha, 1966) and empirical relationship for peat strength (Carlsten, 2000)

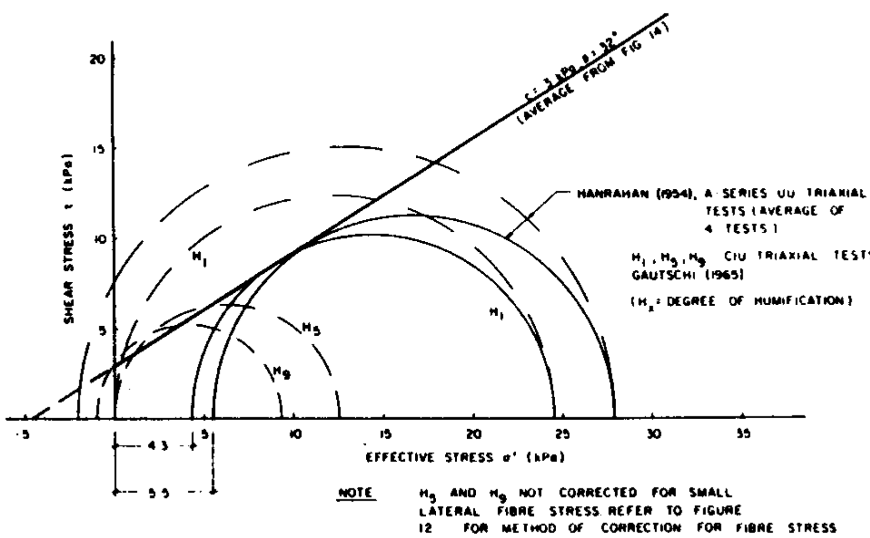
The relationship found by Sodha is very similar, in pattern, to that suggested by Carlsten (2000). The latter was based on vane testing, see Figure 7b. Carlsten (2000) found that the relationship originally published by Amaryan et al. (1973) and Helenlund (1967), i.e.

$$s_u = \frac{140}{w}(100 - 1.1R),$$

where: w = water content,

works well for Swedish peats. For a particular water content the s_u value suggested by Sodha (1966) is not surprising lower than that given by the above relationship.

In fibrous peat high ϕ' values are due to the reinforcing effect of the predominantly horizontally orientated fibres. This effect is not mobilised in the simple shear mode of deformation, such as occurs in ring shear tests. From large ring shear tests on natural fibrous peat, Landva and Rochelle (1983) found values of approximately c' = 3 kPa and ϕ' = 32°. They combined this envelope with earlier triaxial test results reported in the literature, (by Hanrahan 1954 and Gautschi 1965), where horizontal effective stress reached zero at failure, see Figure 8. Keeping σ'_1 constant they increased σ'_3 to bring the stress state on the envelope. The increase in σ'_3 is then the apparent increase in lateral resistance due to the effect of the fibres.



<u>Figure 8</u>. Triaxial tests on peat reinterpreted to determine the effect of fibres (Landva and Rochelle 1983)

The effect of the fibres can then be expressed as σ' tan α and as can bee seen on Figure 9, this effect is consistent being highest for H1 peat and more or less zero for H9 peat. There would seem to be considerable promise in using a relationship such as that illustrated on Figure 9. However further data is required, on peat of varying degree of humification, to validate the theory.

Yamaguchi et al. (1985) demonstrate the effects of organic content and anisotropy of peat by triaxial compression and extension tests on vertically and horizontally orientated samples. Thy showed that the resulting strength and pore pressure generated were closely related to the orientation of the fibres and the organic content. Termaat and Topolnicki (1994) report on biaxial plane strain tests performed on natural and artificial peats.

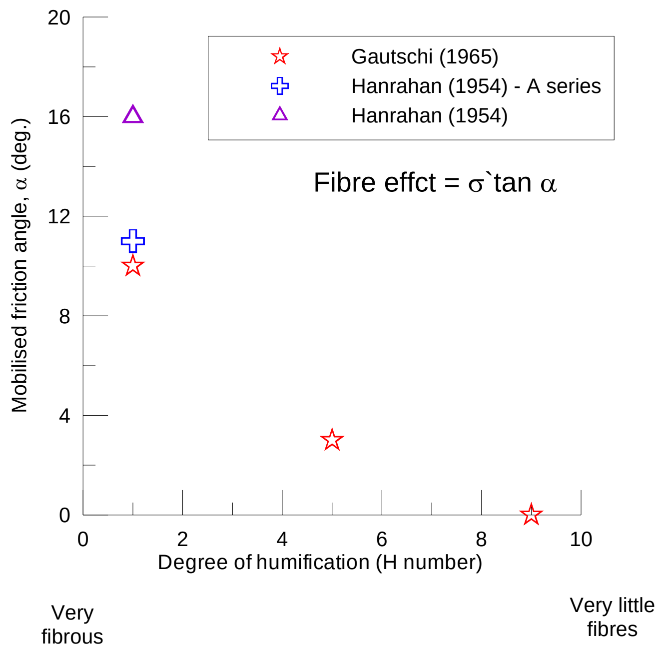


Figure 9. Consistency of effect of fibres

4.6 Problems with the triaxial test

As has been discussed above there are some difficulties with the use of the triaxial test in the laboratory testing of peat. However it is the author's opinion that the test should not be dismissed as in this test stresses can be carefully controlled and it is possible to easily measure pore pressure and other parameters. In the triaxial test the strength of the material both in compression and extension can be obtained. Extension strength may be of particular importance in analysing landslides in peat. Some of the problems with the triaxial test, which could be overcome, are as follows:

1. End platen roughness – eliminate by using special smooth end platens / silicon membrane inserts etc.
2. Membrane stiffness – can be eliminated by accurate correction.
3. Consolidation stresses too high. This results because the actual mean effective stress in a peat mass in situ is very low, perhaps of the order of 5 kPa. Even the most accurate pressure controlling device is only able to resolve to about ± 2 kPa. A solution to this problem may be to use a differential pressure controller to ensure that the differential pressure between the cell and back pressure controlling devices is constant.

4.7 Problems with the DSS test

From the above discussion it is clear in that direct simple shear tests (DSS) in peat give lower values of shear strength than triaxial tests. However as the mode of failure in a DSS test may be applicable to that in a landslide it is worth considering this further.

Wroth (1987) stated that "conventional interpretation of ϕ^1 from DSS is incorrect and leads to an underestimate". Similarly Airey and Wood (1987) and Farrell et al. (1998) found that the conventional approach to interpreting a DSS test, i.e.

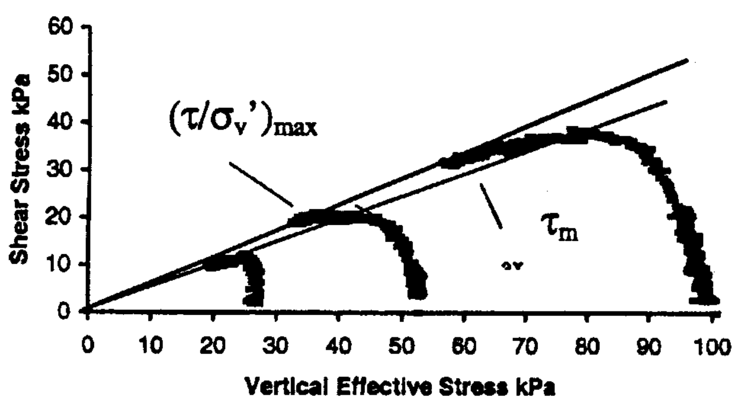
$\phi^1 = \text{Tan}^{-1}\ \tau_h / \sigma^1_v$ (τ_h = max. shear stress, σ^1_v = effective vertical stress)

often leads to an underestimate.

Jardine & Hight (1987) suggested that at large displacements in DSS tests the conventional approach often lead to reasonable estimates of ϕ^1. However theoretical work by Potts et al. (1987), shows that when volume is constant, the expected result for true simple shear is

$\phi^1 = \text{Sin}^{-1}\ \tau_h / \sigma^1_v$.

Another issue with the DSS tests is whether to take the strength at the maximum shear stress (τ_{max}) or the maximum τ_h / σ^1_v ratio. These points are illustrated by examining some typical DSS tests on peat by Farrell et al. (1998), as shown on Figure 10. It can be seen that for both of these issues, conventional interpretation will lead to an underestimate.



<u>Figure 10.</u> DSS tests on Dutch peat (Farrell et al., 1998)

There is no doubt that the stress regime in the DSS test is complicated and thus means that a simple interpretation of the results, such as illustrated above, needs to be treated with caution. In particular experience has shown that the effective stress strength parameters obtained form a DSS test can be erroneous.

5.0 IN SITU TESTING OF PEAT

5.1 Field vane tests

There are no special in situ techniques available for testing peat soils. Therefore standard techniques for inorganic soils have had to be used and adopted for peats. Perhaps most use has been made of the field vane test.

The problems with using the vane test in peat were recognised at an early stage. For example Quinn (1967) stated that the "test was open to criticism as the failure mechanism is one of tearing rather then shearing". Helenlund (1967) concluded that the "test is not reliable in fibrous peat". In a comprehensive review of the practice Landva (1980) 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 11. It seems unlikely then that the test can be truly considered "undrained".

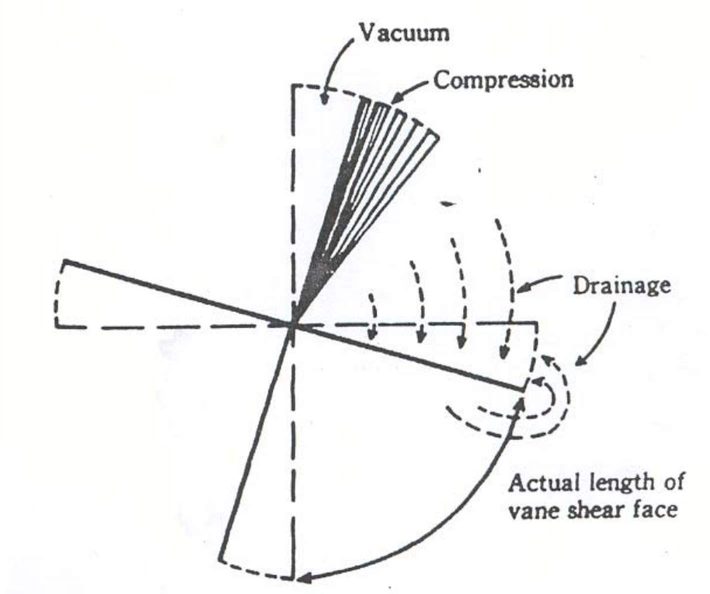
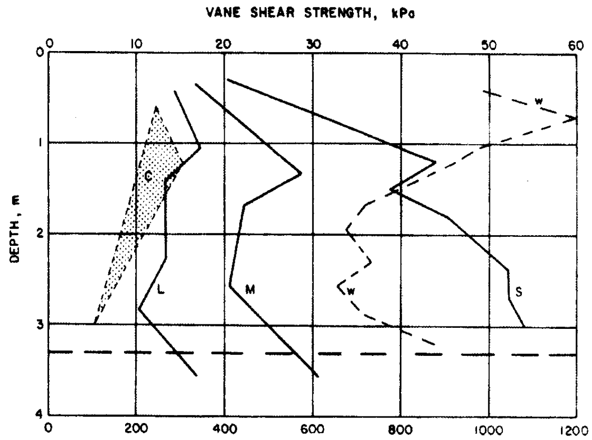


Figure 11. Interaction of vane with peat during test (Noto, 1991)

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 use of the conventional relationship between applied torque (T) and the vane dimensions (L and D) to obtain s_u is questionable, i.e. for L/D = 2:

$$s_u = \frac{0.86T}{\pi D^3}$$

Unlike mineral soils, vane strength (s_{uv}) in peat has been found to decrease with increasing diameter, possibly due to the effect of the fibres, as can be seen for example on Figure 12.



<u>Figure 12</u>. Dependency of su on vane size (Landva, 1980)

Because of these considerations Landva (1980, 1986) concluded that the vane shear test is "of little engineering value in fibrous material" and is also not suitable for organic soils.

However the review of 13 embankments designed using the vane shear test on peat including fibrous peat indicated that in 11 of the cases, embankments were constructed successfully (Muskeg Engineering Handbook, 1969). In one case the peat was purposefully loaded to failure. This shows the vane shear test has some merit and indeed it has been used widely throughout the world.

In engineering practice reduction factors have been introduced to produce safe designs (similar to the factor necessary to explain the Grand Canal failure in Ireland, Hanrahan 1994). The undrained shear strength adopted for design (s_u) is taken as:

$$s_u = \mu s_{uv}$$

These reduction factors have been developed in response to local experience and practice, as has been demonstrated previously above (see for example Figure 5). For example, the Swedish Geotechnical Institute developed the following reduction factor (Larsson et al. 1984):

$$\mu = \left(\frac{0.43}{w_L}\right)^{0.45}$$

where: w_L = liquid limit.

Golebiewska (1983) proposed μ = 0.5 to 0.55 for peat. Landva and Rochelle (1983) provide vane and ring shear data where the ring shear value is 42% to 57% of s_{uv}. Hanzawa et al. (1994) report that in Japan the mobilised shear strength in a peat deposit that failed under and embankment load was calculated to be 50% of s_{uv} and that the laboratory direct shear strength was 67% of s_{uv}.

Significant work on this topic has been carried out in Poland. For example 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) recommends that the Swedish correction factors be used in practice in Poland.

Mangan (1994) suggests that, as the mode of deformation of peat is often characterised by punching failure, corrections should be applied to vane strength with caution.

In conclusion it would seem that although in fibrous material the vane test is scientifically meaningless it can be used with caution once local experience and correlations exist. For example Hanrahan (1994) "acknowledges the limitations of the vane" but feels that it "remains a useful simple method to assess variability with depth, hard and soft layers". If the field vane is to be used in practice, it should be as large as possible. The Muskeg Engineering Handbook (1969) recommends a vane of diameter 100 mm for use in peats. Noto (1991) reports that a 55 mm vane has become standard for Hokkaido peat in Japan mostly because of ease of handling. Noto (1991) also shows a decrease in s_{uv} with increasing vane rotation and recommends a standard rate of 1 deg. / sec. should be adopted. Edil (2001) notes that in Sweden and Poland a time to failure of 3 minutes is adopted as standard.

5.2 Cone penetration tests (CPTU)

Use of the CPTU in peat characterisation has been discussed in Section 5.2 above. Application of the CPTU to characterise shear strength of soft peat soils is not as straightforward as its use in inorganic clays. Typical cone resistances often vary between 0.1 MPa and 0.5 MPa and therefore extra sensitive cones may be necessary. Landva (1986) used a large 300 mm cone and observed that negative pore pressures are induced and large vertical compression and expulsion of water takes place during CPTU penetration in fibrous peat. He also argues that the observed failure mode in laboratory model CPTU tests do not resemble the real mode of deformation under structures and therefore concludes that the CPT is of little use in determining the engineering properties of peat soils. Landva (1986) suggests that the CPTU may yield strength values similar to the remoulded value and may be of some use in modelling progressive failure in embankments.

There may be some benefits in using cones larger than the normal 10 cm^2 version. For example, for Hokkaido peat in Japan a WP-20 type cone (apex angle 30°, base area 20 cm^2) is adopted. Fugro Ltd. operate a large 33 cm^2 cone.

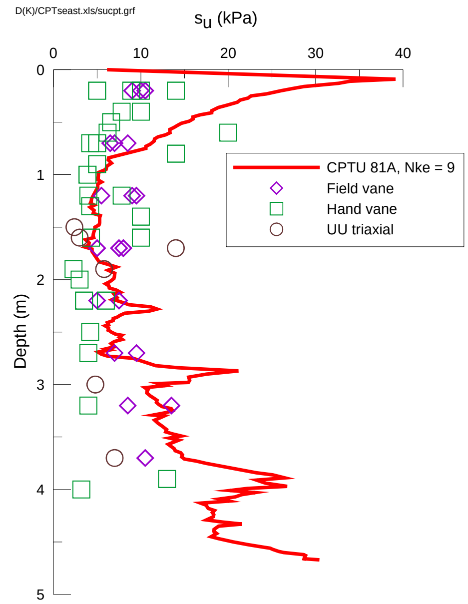


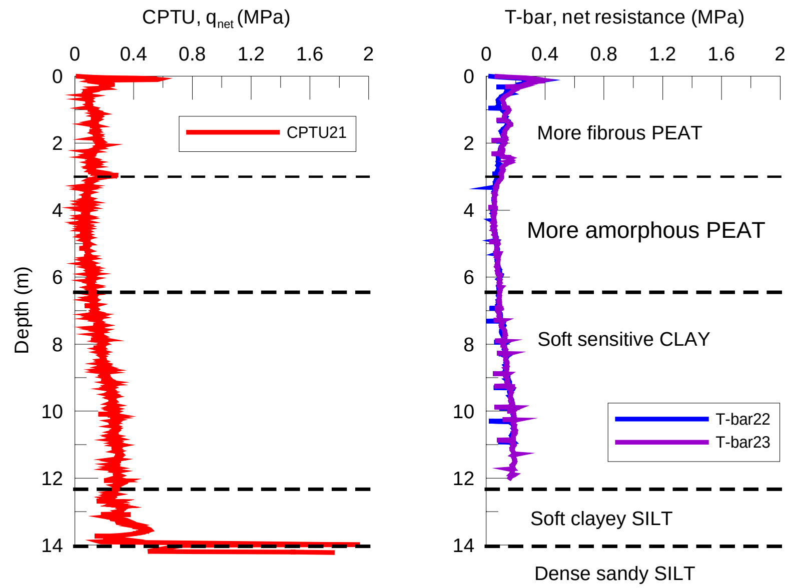
Figure 13. **Use of CPTU to obtain s_u – blanket bog site western Ireland**

Nevertheless the CPT has been used routinely in design of organic soils. For example in Ireland various publications by researchers at University College Galway (UCG) (e.g. Rodgers 1992) have made use of this approach and Rodgers (1992) states that there was good correlation between q_t – u and vane shear strength for various (including organic) soils. Similarly research at TCD (e.g. Faulkner 1992) found that N_{ke} ($s_u = q_t – u / N_{ke}$) was approximately equal to 3 for organic soils at Cavan. An example from a blanket bog site in western Ireland is shown on Figure 13. It can be seen that if $N_{ke} = 9$, then s_u as derived from the CPTU compares well with that obtained from field vane and hand vane tests and laboratory UU triaxial tests.

5.3 Other in situ testing techniques

Due to very small stresses involved and the difficulties in correcting CPTU data for out of balance pore pressure effects, recent research in several countries has advocated the use of the T-bar penetrometer. In this test the cone end is removed and is replaced by a T-bar, typically 40 mm in diameter and 250 mm long (i.e. area = 100 cm^2, 10 times at of a conventional cone), see Figure 14.

Figure 14. **UCD T-bar (Long and Gudjonsson, 2004)**



<u>Figure 15</u>. T-bar test results Bundoran / Ballyshannon bypass, Co. Donegal, Ireland

An example of some data from a typical T-bar test is shown on Figure 15. These data are from the same site as the CPTU data on Figure 1. An important finding from these tests is the repeatability of the results. The T-bar resistance confirms the findings of the CPTU in that the stronger upper more fibrous layers overly a weaker amorphous zone. More work is required to correlate T-bar resistance with field vane and laboratory shear strength.

Edil (2001) reports that there are a few examples of the application of pressuremeter and dilatometer tests in peat soils but there are no available guidelines in the interpretation of such tests. Rahardjo et al. (2004) describe 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.

Landva (1986) suggests that plate loading tests is not applicable for determining strength and deformation characteristics of peat.

Screw plate load tests were performed by Schwab (1976) and found to be useful in determining shear strength when a bearing capacity factor of 9 was applied.

Kramer et al. (1990) evaluate 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.

Some researchers have also used the back-analysis of trial pit failure to estimate s_u. However significant assumptions on the geometry of the slip surface, drainage conditions etc. are require for this technique.

6.0 NUMERICAL MODELLING OF PEAT

Hanrahan (1994) suggested that in Ireland numerical models based on "routine methods of effective stress rarely used". He recommends that for normal engineering projects undrained analyses should be carried out and s_u obtained from vanes. Termaat (1994) suggests that normal Mohr - Coulomb straight line (ϕ^1, c^1) is wrong theoretically (more than likely curved) but adequate for routine work.

Several researchers have attempted to model the behaviour of peat using "normal" soil constitutive models, mostly the conventional Mohr – Coulomb model. For example Brinkgreve et al. (1994) obtained reasonable results with the commercial finite element package PLAXIS for embankments over peat using either the Mohr-Coulomb or Modified Cam clay constitutive models. However careful choice of input parameters was required.

The biaxial tests by Termaat and Topolnicki (1994) were performed to provide a basis for validation of constitutive models including anisotropy. Topolnicki and Niemunis (1994) successfully applied an elasto-plastic model developed for clays to these tests. It takes induced anisotropy into account by allowing rotation of the yield surface.

Sellmeijer (1994) devised an analytical anisotropic model which combines an elasto-plastic matrix with orientated fibres. A disadvantage of the model is that it lacks volumetric hardening which is strong in peats.

Molenkamp (1994) constructed constitutive equations for composite material. Matrix and fibres are both given elasto-visco-plastic properties. One lesson form his work was that top and bottom platens of triaxial specimens should be given freedom to translate relative to one another.

Fox and Edil (1994) describe an application of discrete element modelling to 1D compression of fibrous peat. The method can easily be extended to 2D. Single fibres are modelled as trusses, the bars of which experience both elastic and creep deformations.
Anisotropy of peats as a subject of research is clearly very much in its infancy. Den Haan et al. (1995) suggest that effort must be directed at experimental element testing, constitutive modelling, implementation in finite element codes and validation of laboratory model tests and field prototype tests.

7. BEHAVIOUR OF PEAT IN LANDSLIDES

Slides in peat areas are relatively rare events world-wide. For example Hungr and Evans (1985) describe a slide in Canada, which they suggest is the only documented slide in a peat area in Canada. These authors also report on a slide in the Falkland Islands (Barkly 1887), some slides in Western Scotland (Bowes 1960) and suggest that slides are relatively well known in the Pennines of England (Crisp et al 1964). However as stated by Hungr and Evans (1985) these events are particularly common in Ireland, with the earliest documented peat slide having occurred in 1697.

It is outside the scope of this report to summarise or describe all of the peat slides that have occurred in Ireland. Feehan and O'Donovan (1996) provide an excellent overview of the subject. These authors state that landslides in peat areas can be sub-divided into the following types:

- Bog bursts: where excess hydrostatic pressure in the basal peat causes a "blow out" and subsequent slide.
- Bog flows: where the liquid basal peat escapes from beneath the less humified peat.
- Bog slides: where the less humified upper peat layers slides overt the base peat layer
- Peat slides: where the slide occurs in the underlying mineral soil and the failed mass breaks up into relatively intact blocks or rafts.

It has been reported in several situations that the main slide was preceded by a number of smaller slides, which gradually destabilised the peat mass (retrogressive failure). Following failure the peat is found to have very low remoulded shear strength and run out distances can be very significant, see Figures 16 & 17.

<u>Figure 16.</u> Peat slide in Slieve Bloom Mountains, Co. Laois, 1988. Note run out distance and scars of old slides.

<u>Figure 17</u>. Peat slide, Pollatomish, Co. Mayo, Oct. 2003. Note long run-out distance.

It can be seen therefore that these slides resemble slides in sensitive materials, such as quick clay. Therefore the mode of shearing that may be important may relate to ring shear tests or direct simple shear box tests (DSS).

8.0 SUMMARY AND CONCLUSIONS

Peat is characterised by:

1. High triaxial friction angles $\phi' = 40°$ to $60°$. This is because of the effect of horizontally orientated fibres and possibly due to end platen friction.
2. Somewhat lower values are measured in ring shear and direct simple shear and these tests may be more applicable to the case of landslips in peat.
3. In situ undrained shear strength values are often low but normalized values s_u/σ'_{v0} are often high (0.5 to 0.7).
4. In situ vane tests in peat are difficult to interpret and may yield too high values. Local reduction factors have been developed.
5. As the observed failure mode in CPTU tests do not resemble the real mode of deformation under structures the CPT may also be of limited use in determining the engineering properties of peat soils. However local correlations again appear to be of some use. The T-bar test is also promising.
6. If vane or CPT tests are to be used then the dimensions of the device should be as large as possible.
7. Screw plate loading tests may be of some merit in peat soils.
8. Some attempts have been made to model numerically peat and in particular peat fabric anisotropy. However this work is in its infancy and much additional effort is required.

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