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Predictions of settlement in peat soils

Abstract: Laboratory predictions of the compression behaviour of peat are compared with field data through use of a database of laboratory tests from 14 sites and information from full scale field loading at 5 sites. Data presented confirms the complexity of the deposits. Nonetheless from the point of view of normal engineering works, calculations based on laboratory test data are likely to give reasonable predictions of the magnitude of immediate and primary compression.Standard (20 mm thickness) samples may give misleading data on time for primary consolidation.Thicker samples, e.g. 50 mm, should be used. Sampling by conventional samplers, as used for mineral soils, can cause densification of the peat resulting in non-conservative design parameters. It was found that the data presented follow the C_α/C_c law of compressibility. There is also is some evidence to suggestthat the H^2 scaling law may be applicable. Good correlations were found between vertical yield stress (p_{vy}') and compression index (C_c) and index parameters such as water content (w_i) and void ratio (e_0). Conventional staged construction with surcharge loading may be successfully applied to peat soils as long as adequate drainage exists to permit consolidation over reasonable time intervals.

Keywords: Peat; Compressibility; Soil compression; Soil consolidation; Permeability;Foundation settlement; Laboratory testing; Field measurements

Introduction

The high compressibility of peat soils and their potential for long term creep have posed difficulties for engineers in many places around the world for hundreds of years. Examples of cases where excessive settlement of peat layers has affected the serviceability of structures include those published by Lewis (1956), Lea and Brawner (1963),Hanrahan (1964) and Edil and Simon-Gilles (1986) for roads in the United Kingdom, Canada, Ireland and the United States respectively. However, despite many years of experience in observing such settlements, there remains much uncertainty as to how these settlements should be predicted and especially how the peat parameters necessary for the calculations are obtained. These topics will be dealt with in this paper by briefly summarising relevant research so as to identify the most important research questions and subsequently addressing the issues with reference to a database of laboratory tests on peat and information from full scale field loading.

Literature review

General stress / strain behaviour of peat

Frequently engineers use the classical one dimensional compression method(Terzaghi, 1923) to model the stress / strain behaviour of peat. This method assumes that forprimary compression the relationship between void ratio (e) and the log of vertical effective stress (log σ_v') is linear and that the behaviour can be characterised by the compression index (C_c), and the swelling index (C_s). For creep a linear relationship is assumed between the change in void ratio (Δe) and log time during a loading stage and the slope of this curve is taken to be the creep coefficient (C_α). The shortcomings of this method,for both peat and mineral soils,relate mainly to the assumed linearity of the compression / swelling index, the fact that void ratio of a soil cannot decrease indefinitely under load and the separation of the primary consolidation and creep processes, see for example Bjerrum (1967).

Use of the term “preconsolidation stress” is considered inappropriate for peat as the process of material formation was not by normal sedimentation. Here the term “yield stress” (p_{vy}')is used to nominally divide the behaviour into that which approximately elastic and plastic (Chandler et al., 2004; Vaughan et al., 1988). The reasons why peat shows a yieldstress are complex. Lefebvre et al. (1984) attribute snow loading, drainage, water table fluctuations and the creep characteristics as the cause of this.In addition Hobbs(1986) suggests that thestructure of the composing plants and the decaying process which takesplace in the acrotelm (upper layers of peat) also contributes to this “critical pressure”.

For peat the classical log σ_v' versus e plot is often curved and it is difficult to estimatep$_{vy}$'. Therefore, in Scandinavia for example, use is made of the alternative procedure (Janbu, 1963)which attempts to relate the stiffness of the soil in primary consolidation to stress using the tangent modulus, M.

$$(M = \Delta\sigma'/\Delta\varepsilon) \tag{1}$$

where $\Delta\varepsilon$ = change in strain. Creep is assumed to occur contemporaneously with the primary compression and is described using the creep resistance (r_s).Examples of the application of these concepts for Norwegian peat are given by Janbu (1970). Carlsten (1988) and Sas et al. (2011) used this approach to analyse oedometer test result for Swedish and Polish peat respectively. Lefebvre et al. (1984) used the technique to compare predicted and measured settlements beneath highway embankments in Canada.

Several advanced models and calculation methods for determining 1D compression in peat soils are also available to practicing engineers. A number of these use the “isotache” principal, for examplethe “abc” method (Den Haan, 1996) which was based on the original work of Bjerrum (1967). A number of authors have reported successful use of these techniquesin peat. For example O’Loughlin (2007)concluded that the “abc” method adequately encapsulated both the total settlement and the field time – settlement response. O’Loughlin (2001) concluded that the isotache principle appeared to be valid for peat except for loading less than the yield stress.

Nevertheless, for many small to medium scale projects or for preliminary settlement estimations many engineers continue to use the classical techniques of Terzaghi (1923)or Janbu (1963)and these methods will be the focus of this paper.

Influence of sample thickness

Due to the high compressibility of the material many researchers have recognised that the use of standard 20 mm thick samples, as used for mineral soils, may not be appropriate for peat. A compromise needs to be made between having a sample which is as thick as possible but is also practical to handle and does not unduly extend the test duration. In Sweden (Carlsten, 1988) and Norway (Janbu, 1970) standard sample thickness for 1D compression tests on peat is 45 mm and 50 mm respectively. In Canada 38 mm high specimens are used for compressible peat (Lefebvre et al., 1984).

The effect of sample thickness on the laboratory test results is of significant importance as findings fromstandard thin laboratory specimens are extrapolated to thick conditions in the field.Based on the original work of Hanrahan (1954), various researchers have attempted to relate the time to primary consolidation of peat in the field, t_f, to that in a sample, t_s, using the following relationship:

$$\frac{t_f}{t_s} = \left(\frac{H_f}{H_s}\right)^i \tag{2}$$

where: H_f and H_s are the field and sample thicknesses respectively

Hanrahan (1964), Barden (1969) and Berry and Poskitt (1972) referred to the "H^2 scaling law", i.e. they found that the exponent i = 2. More recently Ajlouni,(2000)has supported the applicability of the H^2 scaling rule. Other researchers have suggested other values for i, for example Samson and La Rochelle (1972) found it to be in the range 1.6 to 2, Lefebvre et al. (1984) suggested values of 1.1 to 1.5 and Hobbs (1986) suggested that the average i value was about 1.5. The differences are considered to be due to natural material variability, influence of creep in incrementally loaded (IL) tests and the load increment ratio.

There has also been significant debate in the literature with respect to the uniqueness of the time corresponding to the end of primary compression (EOP). Some researchers consider that EOP is independent of sample thickness.A summary of some of the discussionscan be found in Mesri (2003) and Leroueil (2006).However, for clay soils many engineers now accept that EOP is not a unique parameter and depends on various factors such as applied stress and sample / layer thickness(Degago et al., 2011).

Sampling disturbance effects

Previous research on the influence of sample disturbance on peat (organic content > 80%) is very limited. Early studies by Hanrahan (1954)on undisturbed and remoulded samples of peat clearly showed the presence of structure, thus indicating that peat could possibly be susceptible to sample disturbance by destructuration. Helenelund et al. (1972) investigated the influence of sample disturbance on the strength and compressibility properties obtained from fibrous, slightly decomposed sphagnum peat samples taken with different types of thin walled tubes with various cutting edges and also block samples. Oedometer tests on intact specimens obtained from tube and block sampling and a remoulded specimen showed that the peat examined has the potential for destructuration and that the tube sampling densified the soil resulting in a stiffer response than the intact block sample. The differences identified between conventional tube sampling and block samples were found to be largely overcome by using serrated cutting teeth with a small degree of cyclic rotation during penetration.

Hobbs (1986) suggested that piston samples of peat will have higher p_{vy}' and lower C_c than block samples. He recommended that peat compression in piston samples should always be measured. An axial strain of 10% will increase p_{vy}' by about 60%, with effects being greatest for peat with a lower water content.

Testing rate effects

It is well known that strain rates used in laboratory testing influence the test results. For example Hight and Leroueil (2003) illustrate that, as the entire limit state surface of clays is strain rate dependant, then as a result the yield stress, the entire 1D compression curve and the undrained shear strength must be strain – rate dependant.Various authors, for example Leroueil et al. (1985), have shown that yield stress is particularly strain rate dependant and relationships betweenp$_{vy}$' and strain rate have been developed for clay soils. There is only limited similar work reported for peat. Ajlouni (2000)found no effect on the results for James Bay peat but found a slight increase in p$_{vy}$' with strain rate for Middleton peat.

For peat soils, it is not clear whether the differences in the loadingrate in the laboratory and the field influence settlement predictions based on laboratory measurements.

Aims of paper

This paper seeks to give guidance to engineers making practical predictions of one dimensional compression of structures founded on peat and specifically onspecifying laboratory tests and choosing soil input parameters for their calculations. If the classical Terzaghi(1923) or Janbu(1963) approaches are adopted several practical questions need to be addressed as follows:

- How does the laboratory based prediction compare to field measurements?
- How well do the compression parameters relate to the basic properties? Oedometer testing is relatively expensive and time consuming and correlations between compression parameters and simple index properties may provide a good understanding of the likely behaviour of the peat, without extensive compression tests.
- What is the effect of the thickness of the oedometer sample on the test results?

- How important is the quality of the sample and what are the effects of sample disturbance?
- Is the rate of loading important?

In order to address these points a database of laboratory test results from ten Irish sites, two Dutch and two Norwegian peats has been assembled. In situ settlement records from full-scale loading of the peat at five sites is also presented and compared to the expectations from the laboratory data.Two of these sites are also included in the laboratory test database.

Study sites

Details of the sites where laboratory test data has been considered are summarised in Table 1.Five of the sites are western Irish upland blanket bogs and the remainder are raised bogs. Both the blanket and raised bogs initially formed in water filled depressions on low lying ground. In both cases the water bodies would have gradually become overgrown with fen vegetation. Later and with the onset of a wetter climate, plants such as sphagnum mosses, which depend on rainfall for nutrient supply tend to dominate(An Foras Taluntais, 1980). Peat at the three sites located near Trondheim in Norway was formed in a similar manner to the Irish raised bogs. In the case of the Dutch sites a similar mode of deposition also prevailed. However the depositional environment in this case was of fluvial type and hence it is common to find clay, clay overlying peat and woody peat(Berendsen, 2005).

The peat at the raised bog sites is older than that at the blanket bogs (formation began approximately 10000 and 4000 years before present respectively) and hence the peat is generally thicker (up to 6 m) at these sites than at the blanket bogs where it is generally about 3 m or less in thickness.

According to (BSI, 2006)the peat can be described as “fibrous” as it has recognisable plant remains and these retain some strength. Generally it is almost totally organic and very soft.

The exception is perhaps the Dutch Bodegraven site where the material has some mineral soil content.

Details of the 5 sites where full scale loading trials were carried out are given on Table 2. Two of the full scale loading trials were done largely for research purposes;at Longfordpass, Ireland(Sheedy and Plant, 1968) and Heimdalsmyra, Norway (Hove (1972). The remaining three full scale in situ loadings were infrastructural projects in Ireland. Two were for highway schemes at Athloneand Knock. The final project at Carrick on Shannon also involved filling low lying ground for an office development car park.

Methods used

Laboratory compression testing

Three different test types were used. Most tests were conventional 24 hour incrementally loaded (IL) oedometer tests toBS1377 (1990). The ratio of the applied load to the previous load was usually 2.0 two but never was less than 1.0. Occasionally the loading period was longer than 24 hours. No correction was applied to the measurements, as this would give a misleading picture of the compression curve, because the stress state for later steps would then be for a denser state than the curve suggests. These longer increments had the advantage that the creep parameter could be determined more accurately.

In order to investigate sample thickness effects comparative testing was undertaken using the Janbu törvodometerand standard oedometers at four of the sites. The törvodometer, see Janbu (1970)and Figure 3a, was designed to test standard Norwegian 54 mm diameter samples without trimming and has a sample height of 50 mm, compared to 19 mm or 20 mm for standard oedometers.

Lastly continuous rate of strain (CRS) tests were also carried out in order to investigate strain - rate effects. These tests were undertaken on 20 mm high samples at the Norwegian

University for Science and Technology (NTNU) and on 50 mm high samples at University College Dublin (UCD) using the procedures outlined by Sandbækken et al. (1986).

Index testing

The degree of decomposition of the peat was semi-quantitatively assessed using the von Post and Granlund (1926) method. Peat is classified on a scale of H1 to H10 in increasing degrees of decomposition. Organic content was obtained by determination of the loss on ignition at 440°C(Arman, 1971).

Laboratory testresults

Typical index test results

The data in Table 1, encompasses a wide variety of peat soils with initial water content (w_i) and bulk density (ρ) ranging from 290% to 1720% and 0.93 Mg/m^3 to 1.12 Mg/m^3 respectively and degree of humification (H) after von Post and Granlund (1926) in the range 3 to 8 on the von Post scale.

The results for the raised bog at Grigg Road, N2 site are shown in Figure 1 and are typical for Irish peat (Hanrahan, 1954). At this site the peat thickness is about 6.5 m.Above the water table $\rho_i$values are relatively high and w_i is correspondingly low. Below the water table, in the fully saturated peat,w_i ranges between 700% and 1000% and shows a gradual reduction with depth. Towards the base of the bog the w_i values fall to about 400%. Bulk density is relatively constant at about 1 Mg/m^3 but does increase slightly at the base of the sequence.

The loss on ignition values are between 95% and 100%, indicating that the peat is almost entirely organicwhich is typical of Irish peat. The degree of decomposition(von Post and Granlund, 1926)varies between 3 and 9 and has no clear relationship with the other index parameters.

General stress – strain - time behaviour

Although the standard 24 hour loading interval was generally used it should be recognised that the resulting stress – strain curve may include both primary compression and creep components. Traditionally only the log stress - strain(log σ_v' v ε) curve is used to analyse a test result.According to Janbu (1963), (1991), (1998) the full test data, i.e. σ_v' v ε (log scales not used), σ_v' v constrained modulus (M), σ_v' v coefficient of consolidation (c_v) and σ_v' v creep coefficient (C_{sec}) should be studied in order to fully understand the test result. For clays the tangent modulus, M and coefficient of consolidation, $c_{v,}$initially show high values, which reduce to a minimum and subsequently increase approximately linearly with stress. Resistance to creep (here the coefficient $C_{sec} = \Delta\varepsilon/\Delta\log$ time is used) is high pre yield and decreases significantly as the yield stress is passed. According to the Janbu approach the point where the M, c_v and C_{sec} values reach a minimum corresponds to the yield stress where the initial structure of the material has been completely broken down.

An example for Charlestown peat is shown in Figures 2a to 2d. The peat behaviour is very similar to that of clay as described above. Of the 61 tests reported here, 49 of them show behaviour similar to that of a classical Janbu(1963) "clay".It is also important to note the curved nature of the log σ_v' v ε plot. This has important implications for the selection of C_c as will be discussed later.

For the tests shown in Figure 2, p_{vy}' is estimated to be 25 kPa and 35 kPa for the Casagrande and Janbu techniques respectively. Lunne et al. (2008) showed that for tests on 22 high quality Sherbrooke block samples of marine clay the p_{vy}' from the Janbu approach was on average 7% (or about 8kPa) higher than that obtained from the classical Casagrande (1936) technique.

Effect of sample thickness

As noted by Ajlouni (2000), the end of primary consolidation will occur very rapidly for stress increments less than or near the yield stress and can be significant for larger stresses.

Therefore it is necessary to consider these two conditions separately. A comparison of standard (20 mm thick) and 50 mm thick (törvodometer)test results for West Mayo peat at a stress of about 20 kPa, i.e. around the yield stress, and 80 kPa, i.e. well beyond the yield stress, are shown in Figure 3b and 3c respectively. Yield stress was determined using the Janbu approach. For the 20 kPa stress the thicker samples show much less strain and the time to reach the end of primary consolidation (EOP) is much larger than for the thin samples. However, at 80 kPa stress, i.e. well beyond the yield stress, the results for the two sets of samples are more or less the same. This finding is the same as that of other researchers for clay soils as outlined above in the literature review.This in turn supports the idea that the isotache concept may be appropriate to capture the in situ time settlement behaviour of peat.

Sample disturbance effects

At the Grigg Rd., N2 site samples of peat were obtained using a conventional 101.4 mm diameter piston sampler and a specially fabricated 200 mm diameter piston sampler.Values of water content and bulk densitymeasuredon the specimens obtained using each of the samplers are shown on Figure 1. The water content of the peat obtained using the smaller diameter sampler is lower than that obtained using the larger sampler, suggesting that peat underwent drainage (or to a more significant degree than for the larger diameter sampling) during the sampling process. Similarly the bulk density values are on average higher for the smaller diameter sampler.

Boylan (2008)also examined the influence of sampling disturbance on the engineering properties of peat soils using a detailed case history for the Vinkeveen site in the Netherlands. He compared results from three sampler types namely the Dutch hollow auger sampler (which is a conventional auger with a hollow inner tube containing a freely rotating sample tube), the standard UK 101 mm piston sampler (with 1.7 mm thin walled tubes and 30° cutting edge angle) and the high quality Sherbrooke block sampler (Lefebvre and Poulin, 1979). Water content values are shown on Figure 4a. For the piston and hollow auger samples, pparticularlybetween 4m and 5.25m, there is a trend ofincreasing water content with depth. This trend is replicated for piston samples at otherdepths and to a lesser degree for the Sherbrooke sample at 2.2m. This would appear toindicate that there has been a migration of water towards the bottom of the samplesduring the period between sampling and testing. Aside from this, the water content forthe piston sample between 4.35m and 5.25m is on average 200% lower than theSherbrooke sample at the same depth. The corresponding hollow auger sample water content is onaverage 60% lower than that of the Sherbrooke block sample at this depth. Between 2m and2.3m, water content for the piston sample is on average 190% lower than that of theSherbrooke block sample of that depth.

The range of bulk densities (Figure 4b) is broadly similar for all samplers exceptfor a few low values (< 1 Mg/m^3) mainly from piston samples.

Yield stress values (Figure 4c) for the UK piston and hollow auger sampler are significantly higher than for the Sherbrooke samples. Constrained modulus at in situ stress (M_0 at σ_{v0}') (Figure 4d) are highest for the UK piston samples and lowest for the block samples (i.e. piston samples are stiffest). Similarly compression index values (C_c), shown on Figure 4e, areon average highest for the Sherbrook samples (i.e. most compressible). Boylan (2008) concluded that poorer quality sampling (such as that by standard piton tubes)can lead to densification and reduction in the water content. This can lead to the tests on the poorer specimens overestimating the yield stress and both the pre and post yield stiffness from oedometer tests. Thus unlike clay soils the influence on engineering design parameters is not conservative.

Rate effects

In order to assess the effects of the rate at which the oedometer tests are carried out, constant rate of strain (CRS) tests were carried out at four rates between 1.5% / hour and 10% / houron the peat from 4 of the sites (see Table 1). It is acknowledged that creep may play a role in the test results, particularly for the slower tests. Typical results, from the West Mayo site, are shown in Figure 5 in terms of log σ_v' v ε and σ_v' v M. The curved nature of the log σ_v' v ε plot is again noted.Examination of Figure 5 and the data summarised on Table 1 suggests there is only marginal difference between the test results. The resulting parameters such as M and $C_c$are not influenced by the test rate. As has been found for clay soils there does appear to be a relationship between p_{vy}' and test rate. For example for the West Mayo peat (Figure 5a), p_{vy}'increases from 12 kPa at a low rate of strain to about 15 kPa at higher rates. Similarly for the Charlestown and Grigg Rd., N2 sites an increase in p_{vy}' with increasing strain rate can be observed.For the Crockagarron site there is no difference between the test results. Once again the results support the use of the isotache concept for peat.

Summary

Stress – strain curves for the peat suggest they are similar to those assumed in the classical Terzaghi (1923) or Janbu (1963) models despite the material being composed of organic rather than mineral matter and having much higher water content and lower density than the traditional clay or sand soils. Samples should be as thick as possible as, similar to the findings for clay soils, thin samples can give misleading data on time for primary consolidation (EOP). Similar to the findings of Ajlouni (2000) the speed at which the test is carried out seems only to have a modest effect onp_{vy}' and has no significant effect on other parameters such as M and C_c.It is interesting to note the recommendation of Ajlouni (2000) who considered that the end of primary consolidation of peat takes place so rapidly that the extra effort required in setting up CRS tests specimens (for example application of back pressure and more sophisticated control system) cannot be justified in comparison to performing standard IL tests.

Engineering properties of peat

Constrained modulus at in situ stress (M_0)

The stiffness of peat in one dimensional compression for loading less than the yield stress can be assessed by considering the constrained modulus M_0 (=$\Delta\sigma_v'/\Delta\varepsilon$) at σ_{v0}' (in situ vertical effective stress). These valuesare plotted against water content in Figure 6. The values are generally very low and there appears to be a weak trend of decreasing M_0 with increasing w_i. It should be noted that any trends emerging here should be treated with caution as M_0 will also be influenced by stress history.

Yieldstress (p_{vy}')

Values of p_{vy}' obtained from both the Janbu(1963) and Casagrande(1936) techniques are plotted against water content in Figures7a and 7b respectively. There is a clear relationship between increasingp_{vy}' and decreasing water content as the material becomes denser and stiffer. Similar to the results shown for the Vinkeveen site (Figure 4c), p_{vy}' values for the block samples are in general less than those from the tube samples. The Janbu values are on average about 10 kPalarger than the Casagrande ones, similar to the findings for clay soils reported above.

Kogure and Ohira (1977) first suggested that p_{vy}' of peat was closely linked to initial void ratio e_0. Ajlouni (2000) re-examined their original relationship and suggested p_{vy}'(kPa) = 150/e_0. This relationship fits very well with the data under study here (see Figure 7c).

Compression index (C_c)

As can be seen from the data presented in Figures 2 and 5, the value of the compression index (C_c) needs to be chosen with caution as the e (or ε) versus log σ_v' plot is non linear. Here C_c has been chosen for stresses just greater than the yield stress ($\approx p_{vy}'$+50kPa) as this is often the range of loading of most interest to practicing engineers. Values of C_c are plotted against initial water content in Figure 8a. As would be expected there is very good agreement between C_c and w_i. The data, particularly that for the block samples, agrees well with the linear relationship $C_c = w_i/100$ proposed by Mesri and Ajlouni (2007). On average the values for the tube samples are a little lower than $w_i/100$ and follow a trend of $C_c = w_i/125$ which is the same as the values suggested by Hobbs (1986) for fen peats.

Design engineers often make use of the compression coefficient ($C_c/1+e_0$) in and these data are also presented in Figure 8b. Although there is more scatter than for C_c, there remains a reasonably strong linear relationship with w_i. The trend in the data falls below the (extrapolated) relationship suggested by Lambe and Whitman (1979) but agrees reasonably well (at least up to $w_i \approx 1000\%$) with the relationship suggested for Danish soils by Andersen (2012). Here the influence of sample disturbance is not so clear due to the normalisation by e_0.

Swelling index (C_s)

Unfortunately, only 6 of the tests studied included an unloading stage,which allowed the determination of swelling index C_s. The average C_s/C_c value was about 0.08 which is a little less than the range 0.1 to 0.3 suggested by Mesri and Ajlouni(2007).

Creep coefficient (C_{sec})

Design engineers also often use the creep coefficient C_{sec} ($=\Delta\varepsilon/\Delta\log t$) for the purpose of predicting secondary compression. It is accepted that the determination of C_{sec} from oedometer tests is prone to error as the resulting value will be influenced by stress, the load increment ratio and the time for loading, the accuracy and resolution of the measuring equipment as well as other factors. In addition for peat it is sometimes difficult to interpret the settlement – log time curves using standard methods. Therefore, in this study only the most reliable data have been chosen. Similar to C_c, the values here were chosen for the load increment beyond p_{vy}'(i.e. $\approx p_{vy}'+50$ kPa) and the resultingvalues of C_{sec} are plotted against initial water contentin Figure 9a.

There is a clear linear relationship of increasing C_{sec} with increasing w_i. This is in contrast to the finding of Hobbs (1986) who suggested C_{sec} was largely independent of water content, especially when $w_i$exceeds about 300%. Carlsten (2000) reports that C_{sec} values for Swedish peatsgenerally vary between 0.01 and 0.045 depending on w_i. The average is about 0.025. These values are very similar to those reported here.

Creep coefficient (C_α)

Many researchers and engineers prefer to use C_α (=$\Delta e/\Delta \log t$) rather than C_{sec} and have made use of the C_α/C_c law of compressibility, first introduced by Mesri and Godlewski(1977). This proposesthat C_α/C_c is in the range 0.01 to 0.07 for all geotechnical materials. For peat Mesri and Ajlouni (2007)suggest C_α/C_c= 0.06 ±0.01. Mesri et al. (1994) and others have pointed out that C_c and C_α need to be chosen consistently and to that end the values just beyond p_{vy}' (i.e. at ≈ p_{vy}'+50 kPa) have been selected here. O'Loughlin (2001) for example found average $C_\alpha/C_c$to be 0.042 and 0.056 respectively for Clara and Ballydermot peat in Ireland but in this case the C_α/C_cratio was taken as the average over the entire loading range.

The data plotted in Figure 9b suggest $C_\alpha/C_c$is independent of water content and confirm the C_α/C_c law of compressibility. The overall average C_α/C_c of 0.072 is in general agreement albeit perhaps slightly higher than the range suggested by Mesri and Ajlouni (2007).

Coefficient of consolidation (c_v)

Values of the coefficient of consolidation, c_{v0}, at in situ stress (i.e. around the yield stress) and taken past the yield stress(≈ p_{vy}'+50 kPa)are considered separately and are shown in Figures 10a and 10b respectively. Following the advice of Gruen and Lovell (1983), who suggested the Casagrande (1936) "log time" method is not suitable for fibrous peat, the Taylor(1948) "root time" construction has been used to determine c_v. The decrease in c_v value between the two sets of data is striking with the values from past the yield stress being approximately one order of magnitude less than c_{v0}. This finding is consistent with other researchers, for example Carlsten (2000), O'Loughlin (2007) and Hobbs (1986), who demonstrated the significant reduction in peat permeability with increasing stress. All the peat samples tested in this study showed a continuous decrease in c_v with increasing stress (for example see Figure 2c). This behaviour is in contrast to that of structured clays. which show a minimum c_v value around p_{vy}', and suggest that,as proposed by Ajlouni (2000), compression of peat involves a gradual process without abrupt changes in structure.

Given the scatter in the data there is no clear pattern of decreasing c_v with increasing water content. However, there is a clear difference in the results from the törvodometer and the CRS tests with 50 mm high samples compared to the tests on conventional 20 mm high samples.

Ajlouni (2000) summarises c_{v0} values obtained from the literature for 9 peats. The values measured range between 3 m^2/yr and 64 m^2/yr. Ajlouni's(2000)own data for Middleton and James Bay peat gives c_{v0} in the range 20 m^2/yr to 300 m^2/yr.The values for the larger samples in the present study, i.e. 30 m^2/yr to130 m^2/yr, are consistent with these data from the literature and confirm the findings from the literature review that c_{v0} values obtained from thin samples needs to be treated with caution.

Summary

Similar to the findings from the stress – strain curves the engineering properties of peat follow similar correlations to index properties as those for mineral soils. In particular good correlations exist between M_0, p_{vy}' and C_c with water content. Again similar to mineral soils the data for peat follows the C_α/C_c law of compressibility. The coefficient of consolidation shows a much more marked reduction with increasing stress than that shown by mineral soils and is also a function of the sample thickness used.Sample disturbance effects seem to increase p_{vy}' and reduce M_0 and C_c (material appears to be stiffer due to densification).

Full scale field loading

Details of the five sites where full scale field loading data is available is summarised on Table 2.

Longfordpass

The Longfordpass site is located adjacent to the N6, Dublin – Cork road, about 6.5 km south of Urlingford, Co. Tipperary, Ireland. The purpose of the trial was to assess whether small bore slotted vertical drains would be effective in draining the peat following loading. As part of the study a control section without vertical drains was also installed and monitored. Data for this pilot section is presented here.

The siteis underlain by 6.8 m of very soft fibrous peat over very stiff silty clay. Groundwater is close to ground level in winter and atabout 0.25 m below ground level in summer. Average natural water content and vane shear strength (using a 150 mm x 75 mm hand vane) values were about 1250% and 8.7 kPa respectively (Table 2).

Loading comprised 2 stages over an area of 13.7 m x 13.7 m (Figure 11). An initial load of about 6 kPa (300 mm gravel layer) was maintained for 215 days before application of an additional 225 mm of gravel (4.5 kPa). Monitoring continued for 550 days. Water content and vane shear strength values were measured throughout the monitoring period. The final measured values showed very little change in water content (though natural material variability dominates the data) and an approximate 30% increase in vane shear strength.

Little settlement occurred during the first load increment. Data in Figure 7 suggests that the p_{vy}' values for the site is about 8 kPa. The first load increment is therefore less than the yield stress. The back calculated M_0 value is about 280 kPa which is consistent with the data presented in Figure 6.

For the second load increment, Taylor's (1948) construction suggests the end of primary compression occurred at about day 260. This then suggests the peat has a c_v value of about 300 m^2/yr, which is much higher than would be inferred from the data in Figure 10a. The rapid rate of pore water dissipation is consistent with the applied stress being less than the yield stress. The subsequent creep settlements would imply the material has a C_{sec} of about 0.04. Note that this value is not directly comparable to the data presented in Figure 9a as it relates to a stress nearer to yield stress.

If the in situ performance of the Longfordpass trial had been forecasted using the laboratory test data, the magnitude of settlement would have been predicted reasonably accurately. However the time for primary consolidation would have been significantly overestimated.

Heimdalsmyra

Heimdalsmyra is located about 10 km south of the city of Trondheim in Norway. Full scale loading trials were undertaken to assess the feasibility of constructing noise protection bunds of locally excavated peat and the results are reported by.

Index data for the two trial sections at Heimdalsmyra are summarised on Table 2. The site is underlain by on average about 2 m of peatover soft marine clay.According to BSI (2006) the peat can described as “fibrous” as it has visible plant remains which possess some structure. It has average natural water content and density of about 1000% and1.0 Mg/m^{3}respectively. Generally the peat is “very soft” with vane shear strength of 18 kPa. There is a clear softer zone, with strength of about 12 kPa, between about 1.0 m to 1.5 m.

A number of loading trials were undertaken and two typical examples (Figure 12), for the 2.5 m high noise protection bund at Station 5 (≈ 25 kPa load) and the 0.5 m gravel platform at Station 3 (≈ 10 kPa load). Inspection of the settlement versus time plots and the pore water pressure data from Station 3 confirm that the consolidation was rapid and that primary compression was completed about 2 days after initial loading. It would seem that applied stresses do not exceed the yield stress (especially for Station 3) and hence the rapid rate of consolidation.

Laboratory tests for the nearby Dragvoll site (see Table 1) suggested that the time for primary consolidation varied between about 7 minutes for the 20 mm thick samples to 1.5 minutes for the 50 mm high törvodometer specimens. These data suggest the exponent i in Equation 2 is about 1.3 for the 20 mm specimen and 1.6 for the thicker specimen, these values being consistent with those discussed in the literature review.

For Station 3, where loading is at a stressgreater than yield stress, the backanalysed M_0 is approximately 115 kPa, which is very similar to that suggested by Figure 7. For Station 5, where the loading is likely to be greater than the yield stress, a $C_c/1+e_0$ value of 0.36 can be backcalculated for the measured primary compression of 490 mm. This value is on the lower bound of the lab data presented in Figure 8b.

Backanalysed C_{sec} values from the two sets of field data are about 0.04 for the Station 3 gravel platform and 0.085 for the Station 5 2.5 m noise bund. It is likely that the 10 kPa loading imposed by the gravel platform is still less than the yield stress and hence the lower rate of creep. Overall these C_{sec} values would seem to be larger than the trend suggested by the data in Figure 9a.

Athlone Bypass

The Athlone Bypass was the first major road constructed over soft soils in Ireland and as a result considerable effort was made in detailed monitoring of the foundations (Long and O'Riordan (2001). Extensive monitoring of the compression of a 1.2 m peat layer beneath the embankment fill was undertaken and some of these data are presented here. Work on this site demonstrated the successful use of vertical drains and surcharging to accelerate drainage and consolidation and to attempt to reduce creep settlements.

Some basic properties of the peat at the Athlone bypass site are summarised onTable 2. It can be seen that the peat has somewhat lower average water content (about 360%) and higher average bulk density (1.05 Mg/m^3) than for many of the other sites presented here.

Figure 13 shows peat compression data for Athlone Profile E where the embankments were highest and monitoring data is available for approximately 3.5 years. Peat compression was determined by subtracting data from a magnet extensometer plate located at the base of the stratum from total settlement measured by a gauge rod at the base of the overlying fill. Fill height reached about 9.2 m and total compression of the 1.2 m peat layer was approximately310 mm (strain = 26%). A piezometer located at the centre of the peat stratum shows a clear response to water levels in the adjacent River Shannon (about 35 m away) but during the construction also records an approximate 1.5 m increase in pore water pressure despite a very closely spaced network of vertical drains at 0.9 m centres.

Given the staged nature of the construction, and the vertical drains, it is difficult to back calculate c_v values from the data. However it is possible to approximate $C_c/1+e_0$ (using the total recorded compression) and the value obtained of 0.3is consistent with the lab data presented above in Figure 8b.

Knock Bypass

The construction of the bypass road around the town of Knock in Western Ireland involved three embankments on peat. The three embankments cover a length of approximately 30 m (Fill 1), 140 m (Fill 5) and 220 m (Fill 7) and were the order of 3 m high. Work at this site also confirmed the successful use of surcharging in peat soils.

Ground conditions and index properties for the three fills are summarised on Table 2. Peat thickness is typically 2 m for Fills 1 and 7 and on average 3 m for Fill 5. The material shows significant variability with average water content of about 600%.

Fill heights, surcharge thicknesses and settlement measurements for the gauges which showed most settlement for each fill (typically those located at the centre of the fill) are shown in Figure 14. Although no pore pressure data was available it was judged that the primary settlement was completed for the data shown and the surcharge loading was subsequently removed. Maximum measured settlements varied between 50 cm (Fill 1) and 250 cm for Fill 5. The settlement at Fill 5 was unexpectedly large. As a result a borehole was drilled through the fill near the location of the settlement gauge and confirmed that the peat was about 4.5 m thick at this location.

The actual measured settlements are compared to those that would have been predicted using the parameters measured from lab oedometer tests on Figure 14. It can be seen that for Fills 1 and 7 the predicted value of about 880 mm overestimates the actual measured values of 500 mm and 780 mm respectively whereas for Fill 5 the predicted value of 2200 mm was less than the measured 2455 mm. From a practical engineering point of view these predictions can be considered to be reasonable.

Individual load increments were not held for sufficient time to allow backanalysis of c_v or C_{sec}. However the measurements can be used to back-calculate $C_c/1+e_0$ (assuming the peat stress exceeds the yield stress) and this gives values of 0.21, 0.42 and 0.33 for Fills 1, 5 and 7 respectively which seem consistent with the data presented in Figure 8b.

Carrick on Shannon

The Carrick on Shannon site is underlain by 0.4 m to 1.8 m of peat over about 2 m of soft silty clay over stiff boulder clay. The peat was fresh and fibrous and had an average water content of 470% and bulk density of 1 Mg/m^3 (Table 2).

Monitoring data is shown in Figure 15. Maximum fill thickness was 2.5 m. Unlike the Knock and Athlone projects, vertical drains were not installed. The specification for the final surface gradients of the car park was very tight and the addition of fill was carried out using staged construction with surcharge loading. Peat compression, measured by magnet extensometers, depended on original thickness and varied between 60 mm and 270 mm.Data from piezometers located in the peat and underlying silty clay showed a reasonably rapid response to loading but no subsequent dissipation of the excess pore pressure during the monitoring period. This observation is consistent with the rapid reduction in permeability of peat on loading past the yield stress.

It can be seen from the monitoring data that primary compression had not been completed by the end of the monitoring period. Maximum settlements measured were of the order of 250 mm compared to predicted settlements (from the oedometer test data) of about 570 mm. Due to the long time period that would be required for settlement to be completed, it was decided to remove the soft material by bulk excavation and to replace it with imported granular fill.

Discussion on full scale loading trials

A summary of all the measured strain in the peat strata for all of the full scale loading cases is shown in Figure 16. Some observations are as follows:

- The Knock data show rapid development of strain because of the presence of vertical drains.
- The Heimdalsmyra and Longfordpass cases have relatively rapid rate of consolidation as the applied stresses were likely below the yield stress.For Heimdalsmyra the exponent i(Equation 2), which relates field and laboratory peat thickness,was of the order of 1.5.Although there was an extensive network of vertical drains at Athlone the rate of consolidation was low, probably because the applied stress was well in excess of the yield stress.
- The Carrick on Shannon piezometer and settlement data shows lower rate of consolidation due to the combined effects of no vertical drains and a load which exceeded the yield stress.

The parameters M_0, $C_c/1+e_0$ and C_{sec}obtained by backanalysisof the case history data are shown in Figure 17a to 17c respectively. The in situ M_0 data are plotted against average water content for the same location and compared to the trend from the lab data presented above. It is important to emphasise that M_0 is likely to be influenced by stress history as well as water content. Nonetheless the available in situ data fits reasonably well with the limits obtained from the lab data.

Reliable $C_c/1+e_0$ data can be obtained from five of the full scale loading cases. It can be seen that the full scale loading cases fall within but close to the lower bound lab data boundary.

Sufficient monitoring data, at a stress greater than p_{vy}', is only available for one of the cases for the purposes of reliably backanalysing C_{sec}. The single data point lies well above the trend from the lab tests, suggesting in situ creep may be much more significant than predicted by lab tests. C_{sec} is available for a further two case histories corresponding to loading less than the yield stress. For these two cases the backanalysed value is close to the laboratory data corresponding to stresses greater than the yield stress. The finding that laboratory tests can underestimate field creep settlements is consistent with other published data, for exampleLea and Brawner(1963), Samson and Rochelle(1972) and Lefebvre et al. (1984).

Conclusions

For the laboratory and field 1D compression data presented the following conclusions can be made.

Variability of peat

The data presented in this paper confirms previous knowledge that peat is a highly complex, inhomogeneous and variable material. Its properties can vary widely from site to site and from location to location within a particular site despite visually seeming to be identical. Any engineering analysis will need to rationalise and simplify the ground properties and as a result will be prone to uncertainty. Modelling of peat is therefore also very difficult. These complexities support the use of a relatively simple calculation model as discussed in this paper.

Laboratory based predictions and field measurements

- From the point of view of design of regular engineering works, settlement predictions based on laboratory test data are likely to give reasonable predictions of the magnitude of immediate and primary compression but may overestimate the coefficient of consolidation and underestimate the creep rate.
- It is essential to understand the stress history (i.e. the yield stress) and the relationship between the any additional load and the previous stress history of peat deposits.
- The limited available data suggest that the H^2 scaling law, which relates laboratory and field peat thickness may be applicable to the peat under study.

Correlations between compression and simple index parameters

- The form of correlations developed between compression and simple index properties for mineral soils also apply to peat.
- Particularly good correlations exist between p_{vy}' and e_0 and between C_c and w_i.
- Somewhat weaker correlations exist between M_0 and $C_c/1+e_0$ and w_i.
- The data presented here follows the C_α/C_c law of compressibility. On average C_α/C_c = 0.072 and is possibly slightly higher than suggested by others.

Effect of sample thickness

- Laboratory sample thickness is very important. Standard 20 mm samples can give misleading data on time for primary consolidation and may underestimate c_v. Thicker samples of about 50 mm should be used.

Sample quality

- The limited data available suggest sampling disturbance will increase p_{vy}' and reduce M_0 and C_c. This will result in an underestimation of field compression. This finding is consistent with increased densification in standard pushed in tube samplers. Block samples are recommended. However, following on from the work of Helenelund et al.

(1972),if this is not possible sampling tubes should have serrated edges and be penetrated into the peat by combined twisting and pushing.

Rate of test

- An increase in testing rate in CRS tests will result in a modest increase on the resulting p_{vy}'. Other parameters such as M and C_c seem largely unaffected by test rate. As field loading is generally at a much lower rate than applicable in the laboratory, oedometer test derived p_{vy}' values need to be carefully assessed.

Lessons for construction on peat

- There is no clear difference in the behaviour of Irish, Dutch and Norwegian raised bog peats. Perhaps this is not surprising given the similar aspect, latitude and climate of the three locations. Loading of the peat leads to large compression with significant creep at all locations.
- The field data presented shows conventional staged construction with surcharge loading can be successfully applied to peat soils. However, for loading past the yield stress it would seem that the rapid reduction in peat permeability means that some form of vertical drainage is necessary to permit sufficiently rapid consolidation to render this form of ground treatment efficient for practical use.

Recommendations for future work

- Data presented in this paper supports the application of the isotache principle to peat. This warrants further study. Nonetheless the complexity and variability of the material would need to be taken into account in any such work.
- The lack of homogeneity of the material also supports a statistical analysis of peat test data. Probabilistic methods of analyses may be superior to deterministic ones and this deserves further work.

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Tables

Table 1

Database of 1D oedometer tests on peat at UCD (see separate document)

Table 2: Summary of full scale field loading sites

Site	Location	Depth range (m)	Material properties	Comment
Longfordpass	52.6902°N 7.6598°W	0 – 6.8: fibrous peat 6.8 – depth: very stiff silty clay	w_i≈ 1250 % $s_{u\text{-vane}}$ ≈ 8.7 kPa	Water table at 0.25 m.
Heimdalsmyra	63.3552°N 10.3729°E	0 – 2: fibrous peat 2 - depth: soft	w_i≈ 1000 % ρ_i≈ 1.0 Mg/m^3 $s_{u\text{-vane}}$ ≈ 18 kPa H = 2 - 3 Station 3 and 7 – 8 Station 5	Water table at 0.25 m. $s_{u\text{-vane}}$ ≈ 12 kPa 1.0 – 1.5 m

		marine clay		
Athlone Bypass	53.4302°N 7.9587°W	0 – 1.2: fibrous peat 1.2 – depth: soft clay	w_i≈ 360 % ρ_i≈ 1.05 Mg/m^3	Water table at 0.15 m.
Knock Bypass	53.7977°N 8.9321°W	0 – 2 (Fills 1 and 7) and 0 – 4.5 (Fill 5): fibrous peat 2/4.5 – depth: glacial deposits	w_i≈ 600 % ρ_i≈ 0.95 Mg/m^3	Peat very variable. Water table at 0.3 m.
Carrick-on-Shannon	53.9454°N 8.072°W	0 – 0.4 to 1.5: fibrous peat 0.4/1.5 – depth: soft clay	w_i≈ 470 % ρ_i≈ 1.0 Mg/m^3	Water table at 0.3 m.

Summary of Figures

Fig. No.	**Title**	**File Ref. (All unless stated DELL/Papers/QJEGH/1DComp Peat/)**
1	Index test results N2 Grigg Rd. N2 site(a) initial water content, (b) bulk density, (c) organic content and (d) von Post H.	DELL/Labtests/N2/Basics-GriggRd.grf
2	Törvoedometer results – Charlestown (a) Log σ_v' v ε, (b) σ_v' v M, (c) σ_v' v c_v and (d) σ_v' v C_{sec}	DELL/Labtests/Charlestown/torvos.grf
3	Effect of sample thickness on test results (a) Janbu's törvodometers, (b) West Mayo peat load about 20 kPa and (c) West Mayo peat load about 80 kPa	DELL/Photos/Work/NTNULab/torvos.jpg DELL/Labtests/Corrib/MLvstorvologtime.grf

		And MLvstorvologtime2.grf
4	1D compression parameters for Vinkeveen peat (a) water content, (b) bulk density, (c) yield stress, (d) constrained modulus at in situ stress and (e) compression index	DELL/LabTests/A2Netherlands/1DcompparametersVinkeveen
5	Constant rate of strain (CRS) tests West Mayo site (a) Log σ_v' v ε, (b) σ_v' v M	DELL/Labtests/Corrib.CRSoeds.grf
6	Constrained modulus M_0, i.e. M at σ_{v0}'	WatercontentM0.grf
7	Yield stress from the (a) Casagrande and (b) Janbu techniques and (c) Yield stress (Casagrande) versus initial void ratio	WatercontentVoid ratio Preconsolstress.grf
8	(a) Compression index C_c, and (b) compression coefficient $C_c/(1+e_0)$	WatercontentCcandm.grf
9	Creep coefficients (a) C_{sec} and (b) C_α/C_c	WatercontentCsec.grf
10	Coefficient of consolidation (a) at σ_{v0}' and (b) past p_{vy}'	Watercontentcv.grf
11	Longfordpass – applied load and settlement monitoring data	LongfordPass.grf
12	Heimdalsmyra – excess pore pressure and settlement monitoring data	Heimdalsmyra.grf
13	Athlone Bypass – fill thickness, pore water pressure and peat compression monitoring data	AthloneSettlement.grf
14	Knock Bypass – fill thickness and settlement data	LabTests/Knock/Settlement/ KnockSettlement.grf
15	Carrick on Shannon – fill thickness pore water pressure and peat compression data	CarrickSettlement.grf
16	Strain in peat – all sites	PeatStrainfromCaseHistories.grf
17	Parameters backanalysed from case histories (a) M_0, (b) $C_c/1+e_0$ and (c) C_{sec}	Ccover.grf

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Table 1: Database of 1D oedometer tests on peat at UCD

Site	Location	Sample				Index			*Often assumed=1.5					Preconsolidation stress p_c`				Compressibility						Time related (in nc range just beyond p_c`)				
		Oedometer Test	CRS speed %/hour	Sample Depth (m)	Sample type (P = piston sampler)	σ_{v0}` (kPa)	w_i (%)	ρ_i (Mg/m^3)	G_s*	e_0	von Post H	LOI (%)	ε_{v0} %	Casagrande kPa	OCR	Janbu kPa	OCR	M_0 i.e at σ_{v0}` MPa	$C_c/1+e_0$	Cc	Cs	Cc/Cs	m	c_{v0} m2/yr	c_v* m2/yr	C_{sec}	c_α*	cα/Cc
Irish																												
Carrick on Shannon (MBNA)	53.9454°N	CoSoed1		2.5	101 mm P	39	486.5	0.972	1.5	8.05			17.00	40	1.03	45	1.15	0.25	0.35	3.17	0.29	0.092	4.5	25	3			
	8.072°W	CoSoed2		2.6	101 mm P	39	449.5	1.03	1.5	7.00			11.00	55	1.41	70	1.79	0.42	0.35	2.80	0.21	0.075	4.5	65	1			
Charlestown	53.9532°N	ILOed2		0.57	101 mm P	5	1026	1.002	1.5	15.86	4.0	98.3	1.60	15	3.00	20	4.00	0.3	0.68	11.46				42	3	0.053	0.893	0.078
	8.793°W	Torvo1		0.32	101 mm P	5	899.9	1.046	1.5	13.34	4.0	98.3	1.20	25	5.00	35	7.00	0.42	0.60	8.60				30	2	0.04	0.574	0.067
		Torvo2		0.24	101 mm P	5	763.8	1.09	1.5	10.89	4.0	98.3	1.10	25	5.00	35	7.00	0.47	0.65	7.73				45	6	0.044	0.523	0.068
		CRS1 - 5%/hour	5	0.42	101 mm P	5	1081.7	0.972	1.5	17.24	4.0	98.3	4.20	15	3.00			0.1	0.55	10.03			6.5	2.5	1			
		CRS2 - 25%/hour	25	0.44	101 mm P	5	944.6	1.066	1.5	13.70	4.0	98.3	2.40	20	4.00			0.1	0.55	8.08			4.5	7.5	2.5			
Tuam	53.5155°N	ML2 (1day)		3.53	101 mm P with serrated edge	10	882.7	1.053	1.5	13.00	5.5	97.4	6.3	15	1.50	30	3.00	0.15	0.42	5.88			3	2	3	0.024	0.336	0.057
	8.8552°W	ML3 (3days)		3.48	101 mm P with serrated edge	10	840.2	1.006	1.5	13.02	5.5	97.4	9.4	15	1.50	30	3.00	0.1	0.45	6.31			2.5	7	3	0.022	0.308	0.049
		ML4 (45 mins)		3.43	101 mm P with serrated edge	10	783	1.014	1.5	12.06	5.5	97.4	7.2	15	1.50	30	3.00	0.1	0.47	6.14			2.5	20	1			
West Mayo	54.2°N	MSL3		0.97	SGI sampler	9.5	1076.3	1.002	1.5	16.61	4.5	97.9	17.4	10	1.05			0.06	0.4	7.04			4.5	3	0.5			
	9.7°W	MSL4		0.97	SGI sampler	9.5	1104.8	0.997	1.5	17.13	4.5	97.9	16.3	10	1.05			0.05	0.55	9.97			3	5	0.5			
		Tovo1		1.29	SGI sampler	9.5	996.6	1.043	1.5	14.77	4.5	97.9	10.7	10	1.05	20	2.11	0.1	0.42	6.62			4	24	3			
		Tovo2		1.29	SGI sampler	9.5	927.2	1.019	1.5	14.12	4.5	97.9	6.6	10	1.05	20	2.11	0.1	0.45	6.80			4	135	5			
		Tovo3		1.29	SGI sampler	9.5	1099.1	1.055	1.5	16.05	4.5	97.9	6.67	10	1.05	20	2.11	0.12	0.55	9.38			3	55	5			
		Tovo4		1.29	SGI sampler	9.5	1200.6	1.041	1.5	17.74	4.5	97.9	10.5	10	1.05	20	2.11	0.06	0.6	11.24			3	40	1			
		CRS1 - 1.5%/hour	1.5	2.63	SGI sampler	10	900	0.997	1.5	14.05	7.5	97.0	8.7	15	1.50			0.25	0.4	6.02			6	2	1.5			
		CRS3 - 2.25%/hour	2.25	1.43	SGI sampler	10	1157	0.996	1.5	17.93	4.5	97.9	8.8	12	1.20			0.2	0.5	9.47			6.5	3	1.5			
		CRS5 - 1%/hour	1	1.78	SGI sampler	10	961	1.053	1.5	14.11	4.5	97.9	8.4	12	1.20			0.3	0.5	7.56			6	2.5	1.5			
		CRS6 - 50%/hour	50	1.66	SGI sampler	10	1153.2	1.037	1.5	17.13	6.0	97.0	4.5	15	1.50			0.1	0.6	10.88			4	3.5	1.7			
		CRS7 - 5%/hour	5	1.66	SGI sampler	10	1113	1.065	1.5	16.08	6.0	97.0	10.7	13	1.30			0.15	0.55	9.40			6	3	1.5			
		CRS8 - 3%/hour	3	1.49	SGI sampler	10	1231	1.035	1.5	18.29	4.5	97.9	5	15	1.50			0.2	0.6	11.57			6.5	2.5	0.7			
		CRS9 - 10%/hour	10	1.09	SGI sampler	10	1149	1.04	1.5	17.01	4.5	97.9	8	15	1.50			0.1	0.45	8.11			6	3	2.5			
Derrybrien	53.0673°N 8.629°W	Lough_CRS001	4	0.7	Hand pushed 100 mm P	5	1290	1.049	1.2	14.90	4.5	95.0	2.8	7.5	1.50	8	1.60	0.15	0.48	7.63			6	120	5			
South West Donegal	54.6926°N	IL1		1.5	Hand pushed 100 mm P tube	8	596.3	1.037	1.5	9.07	4.0	94.4	5	15	1.88	22	2.75	0.15	0.42	4.23	0.42	0.099	4.5	15	1.8	0.029	0.292	0.069
	8.5136°W	IL2		1.5	Hand pushed 100 mm P tube	8	571.9	1.034	1.5	8.75	4.0	94.4	3.5	15	1.88	22	2.75	0.21	0.43	4.19	0.42	0.100	4.5	22	5.7	0.029	0.283	0.067
Glinsk	54.3154°N	ML1		1.6	U150	11.6	320.7	1.092	1.5	4.78	7.0	96.9	4.8	30	2.59	30	2.59	0.25	0.32	1.85			8	20	3	0.022	0.127	0.069
	9.6219°W	ML2		1.6	U150	11.6	372.7	1.095	1.5	5.48	7.0	96.9	4	20	1.72	30	2.59	0.3	0.35	2.27			6	9	1	0.03	0.194	0.086
Knock	53.7977°N	BH53 at 0.5 m		0.5	U100	5	670	0.96	1.5	11.03			11.4					0.08	0.4	4.81			3.5	10	0.3			
	8.9231°W	BH71 at 1.0 m		1	U100	10	616	0.98	1.5	9.96			8.4	10	1.00	40	4.00	0.12	0.45	4.93			4	12	0.5	0.038	0.416	0.084
		BH11 at 1.5m		1.5	U100	10	547	0.94	1.5	9.32			10	10	1.00	40	4.00	0.1	0.4	4.13			4.5	2	0.15	0.029	0.299	0.073
Grigg Rd., N2	54.1399°N	CRS1 - 1.5%/hr	1.5	6.2	200 mm dia. P tube	10	648	0.935	1.5	11.00			3					0.25	0.45	5.40			5.5	15	8			
	6.7793°W	CRS2 - 1.5%/hr	1.5	6.2	200 mm dia. P tube	10	656	0.923	1.5	11.29			1.1	15	1.50				0.45	5.53			6	7	5			
		CRS3 - 10%	10	6.2	200 mm dia. P tube	10	674	0.996	1.5	10.66			2.5	25	2.50			0.2	0.49	5.71			4	14	8			
		CRS4 - 5%/hour	5	6.2	200 mm dia. P tube	10	611	1.068	1.5	8.99			1.8	25	2.50				0.55	5.49			4	10	6			
Set1		100 mm spec 200 mm sample - MSL3		6	200 mm dia. P tube	10	739	0.972	1.5	11.95			15	10	1.00	10	1.00	0.12	0.36	4.66	0.20	0.043	5.5	80	4	0.03	0.388	0.083
		100 mm spec 200 mm sample - T1		1.75	200 mm dia. P tube	10	710	1.01	1.5	11.03	5.0	97.0	14	10	1.00	10	1.00	0.07	0.36	4.33			5.5	1.1	0.5			
		100 mm spec 200 mm sample - T2		1.75	200 mm dia. P tube	10	788	1.08	1.5	11.33	5.0	97.0	10	10	1.00	10	1.00	0.1	0.36	4.44			5.5	1.3	0.9			
Set2		100 mm spec 100 mm sample - MSL1		6	101 mm P	10	583	1.069	1.5	8.58			20					0.12										
		100 mm spec 100 mm sample - MSL2		6	101 mm P	10	594	1.082	1.5	8.62			20	13	1.30	26	2.60	0.08	0.4	3.85	0.19	0.050	4	0.2	0.2	0.035	0.337	0.088
Set3		76 mm spec 100 mm sample - S1		2.75	101 mm P	10	742	1.07	1.5	10.80	6.0	97.0	15	10	1.00			0.06	0.37	4.37			5.5	0.75	0.4			
		76 mm spec 100 mm sample - S2		2.75	101 mm P	10	752	0.95	1.5	12.45	6.0	97.0	14	10	1.00			0.12	0.37	4.98			5.5	0.75	0.6			
Crockagarron	54.5401°N	CRS1 - 1.5% / hr.	1.5	0.85	Hand cut block	5	1387	1.102	1.5	19.24	6.0		9	8	1.60				0.7	14.17			7	5	0.7			
	6.9989°W	CRS2 - 1.5% / hr.	1.5	1.24	Hand cut block	5	1721	1.051	1.5	24.99	6.0		2.5	7	1.40				0.7	18.19			8	15	1			
		CRS3 - 4.5% / hr.	4.5	0.85	Hand cut block	5	1537	1.138	1.5	20.58	6.0		15	6	1.20				0.6	12.95			8	15	1			
		CRS4 - 3.0% / hr.	3	1.24	Hand cut block	5	1596	1.08	1.5	22.56	6.0		15	6	1.20				0.55	12.96			8	20	2			
		Torvo 1		1.24	Hand cut block	5	1606	0.938	1.5	26.28	6.0		11.9	5	1.00	10	2.00	0.04	0.6	16.37			4	15	5			
Dutch																												
Bodegraven N11 site	52.0671°N	N11_SB2_3_CRS1	1.5	1.28	Sherbrook	7	311	1.159	1.965856	5.97	5.5	41.0	1.14	48.0	6.86		0.00	0.6	0.45	3.14			5	40	13			
	4.7476°E	N11_SB2_3_CRS2	1.5	1.27	samples only	7	295	1.179	1.965856	5.59	5.5	41.0	1.63	36.0	5.14	65.0	9.29	0.5	0.4	2.63			5.5	35	10			
		N11_SB2_9_CRS2	1.5	4.28	used	9	293	1.086	1.992659	6.21	6.5	39.0	2.47	24.0	2.67	35.0	3.89	0.35	0.48	3.46			5.5	50	2.5			
Vinkeveen A2 site	52.2396°N	A2_SB2_2_CRS1	4	2.3	Sherbrook	5.5	733	1.061	1.6	11.56	6.5	70.5	2.06	12	2.18	20	3.64	0.45	0.48	6.03			5.5	50	6			
	4.9755°E	A2_SB1_10_CRS1	4	4.78	samples only	6.5	835	1.024	1.5	12.70	5.5	85.0	1.7	18	2.77	25	3.85	0.55	0.50	6.85			5	75	10			
		A2_SB1_10_CRS2	4	4.78	used	6.5	820	1.037	1.5	12.31	5.5	85.0	3.1	10.5	1.62	30	4.62	0.24	0.45	5.99			5.5	40	6			
Norsk																												
Dragvoll	63.4057°N	UCD ML-1		2.63	54 mm composite	7	758	1.044	1.5	11.33	3.0		4.4	8	1.14	10	1.43	0.1	0.52	6.41			4	10	3			
	10.4693°E	UCD ML-2		2.68	54 mm composite	7	722	1.05	1.5	10.74	3.0		4.55	8	1.14	11	1.57	0.1	0.52	6.11			4	12	5			
		Torvo1		4.75	54 mm composite	10	740	1.05	1.5	11.00	3.0		4.1	20	2.00	30	3.00	0.25	0.55	6.60			3.5	255	145			
		Torvo2		4.25	54 mm composite	10	760	1.04	1.5	11.40	3.0		3.8	17	1.70	20	2.00	0.21	0.5	6.20			2.5	245	165			
Steinan	63.3999°N	1 og 5		2.5	54 mm composite		1100											0.2					6.5					
	10.4526°E	2 og 4		2.2	54 mm composite		1025											0.1					5					

Footnote: σ_{v0}' = estimated in situ vertical effective stress, w_i, ρ_i, e_0 = initial water content, density and void ratio, G_s = specific gravity, H = degree of humification, LOI = at 440°C, p_c' = preconsolidation stress, M_0 = constrained modulus at σ_{v0}', C_c, C_s = compression and swelling indices, m= modulus number, c_v = the coefficient of consolidation at σ_{v0}' and just beyond p_c' , C_{sec} and C_α = creep coefficients with respect to change in strain and change in void ratio

	w_i	ρ_i	von Post H
Min	293	0.923	3
Max	1721	1.179	7.5

Figs for Long and Boylan on: Predictions of settlement in peat soils

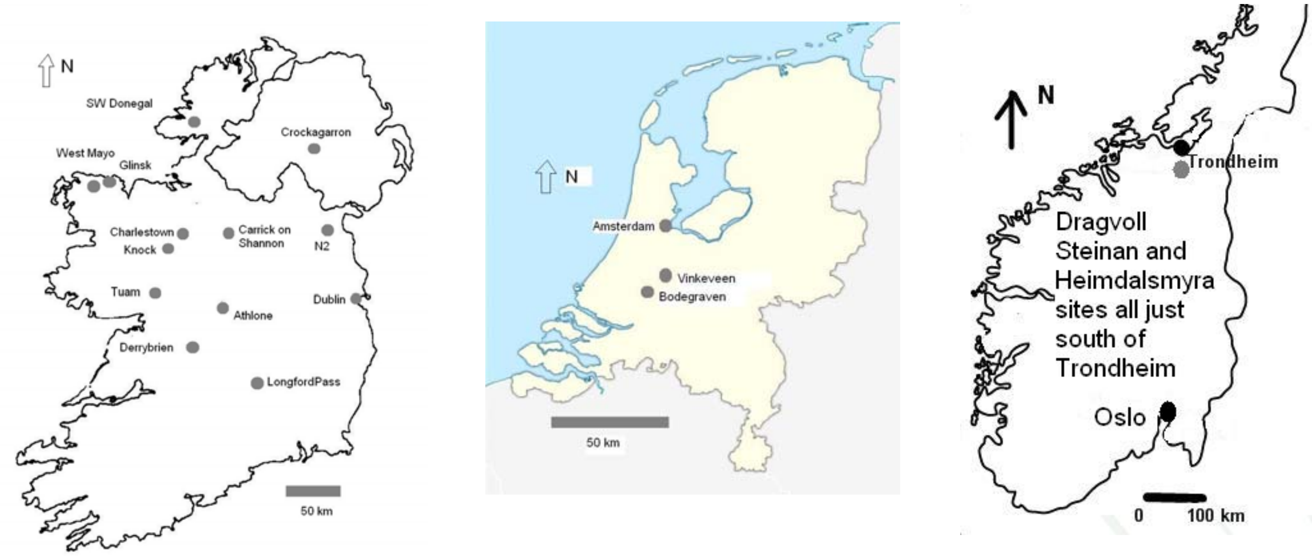

Fig. 1. Site locations in (a) Ireland, (b) the Netherlands and (c) Norway

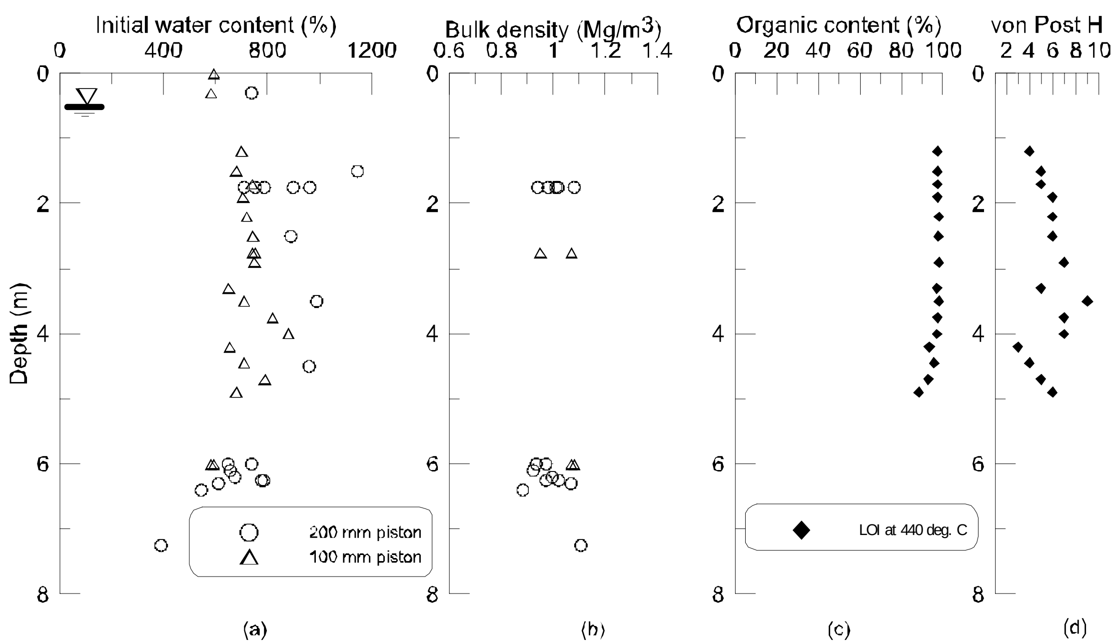

Fig. 2. Index properties – Grigg Rd., N2 site **(a)** initial water content, **(b)** bulk density, **(c)** organic content and **(d)** von Post H. Note LOI = loss on ignition.

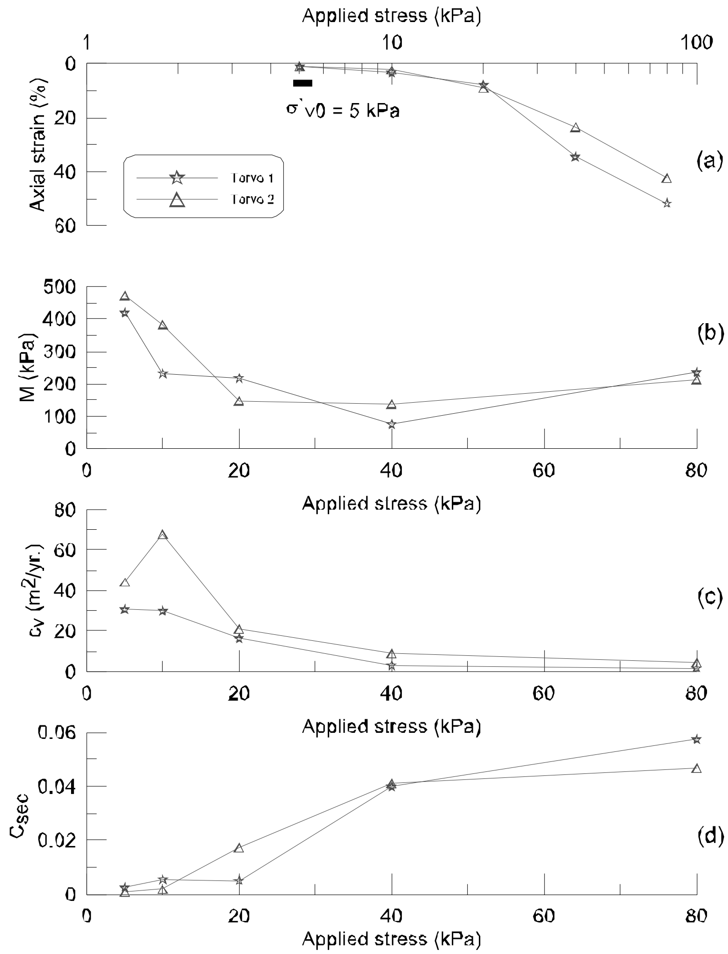

Fig. 3. Torvødometer results – Charlestown **(a)** Log σ_v' v ε, **(b)** σ_v' v M, **(c)** σ_v' v c_v and **(d)** σ_v' v C_{sec} (Torvo = Torvødometer test)

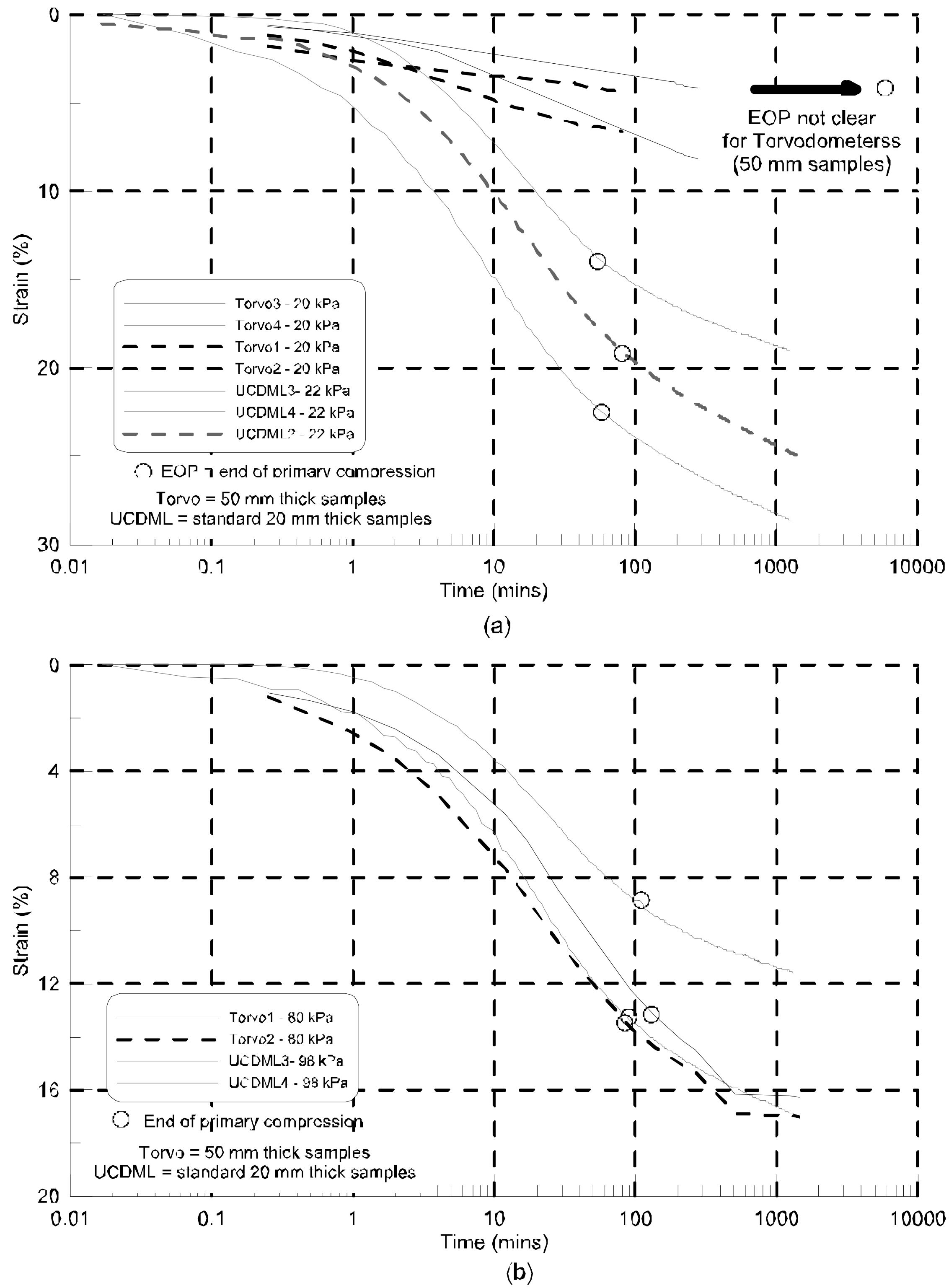

Fig. 4. Effect of sample thickness on test results **(a)** West Mayo peat load about 20 kPa and **(b)** West Mayo peat load about 80 kPa

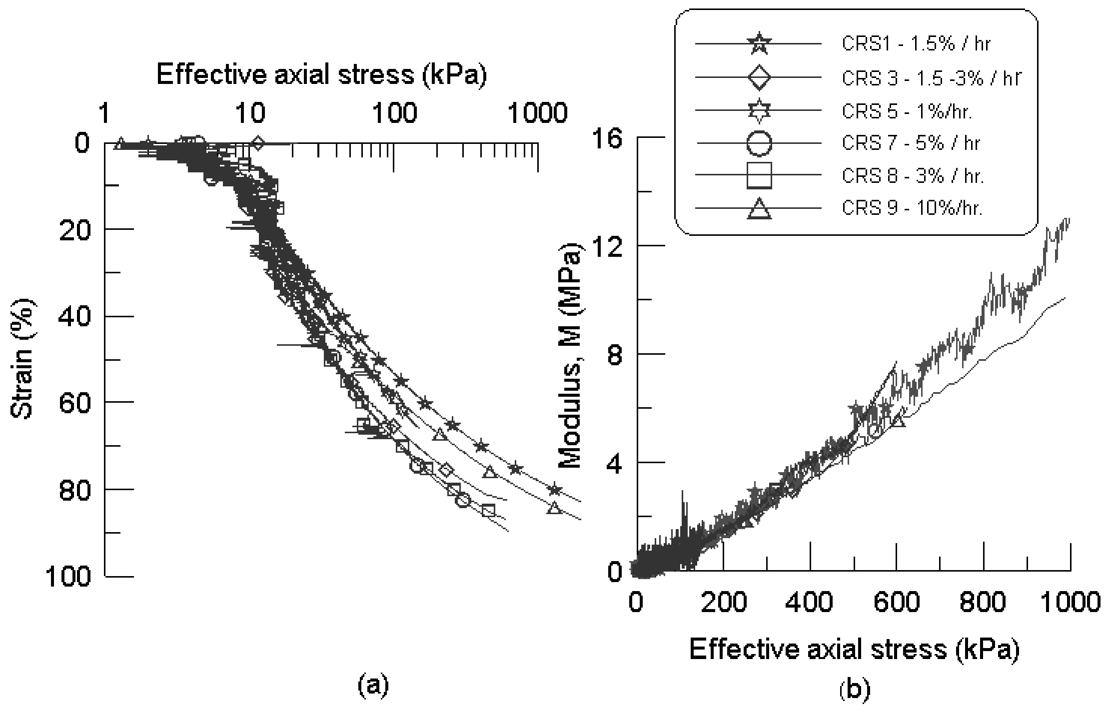

Fig. 5. Constant rate of strain (CRS) tests West Mayo site **(a)** Log σ_v' v ε, **(b)** σ_v' v M

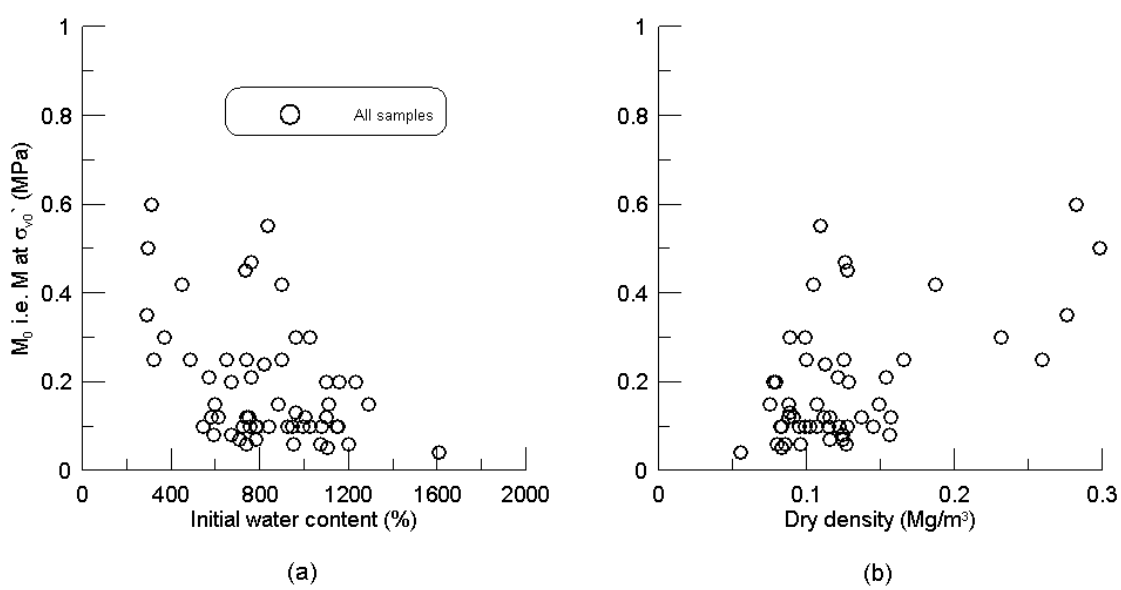

Fig. 6. Constrained modulus M_0, i.e. M at σ_{v0}' versus (a) water content and (b) dry density

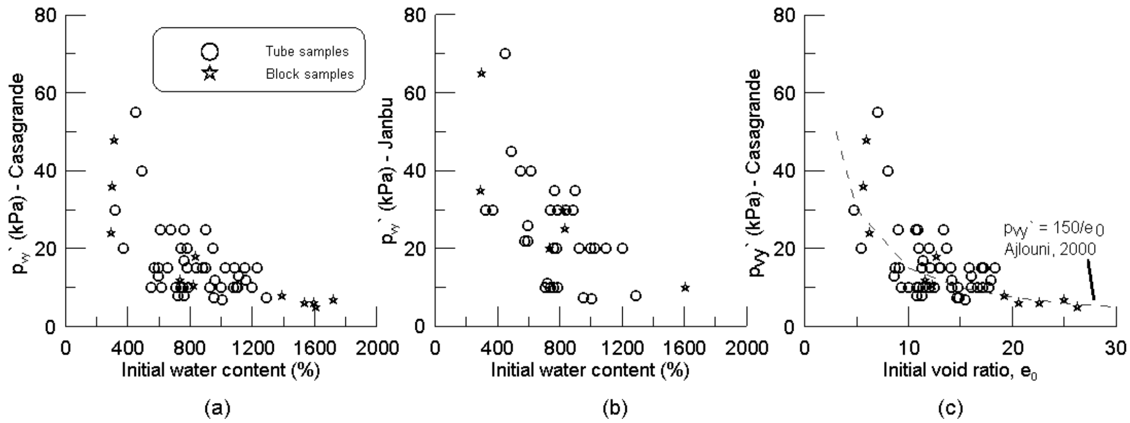

Fig. 7. Yield stress from the **(a)** Casagrande and **(b)** Janbu techniques and **(c)** yield stress (Casagrande) versus initial void ratio

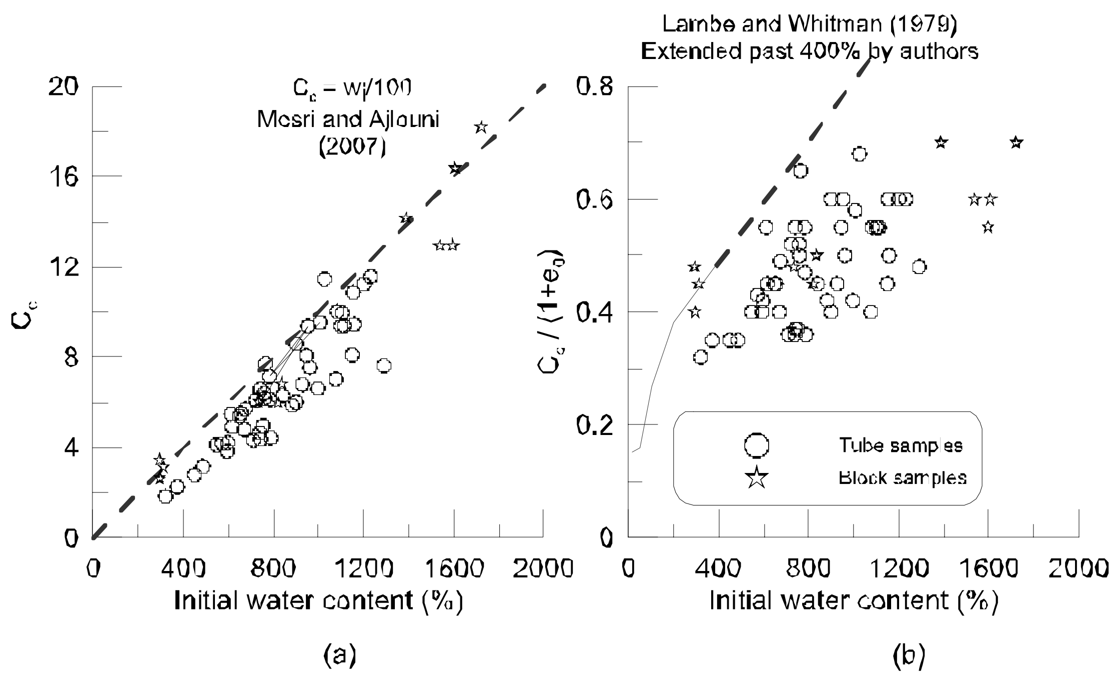

Fig. 8. **(a)** Compression index C_c and **(b)** compression coefficient $C_c/(1+e_0)$ (all relate to stress ≈ $p_{vy}' + 50$ kPa)

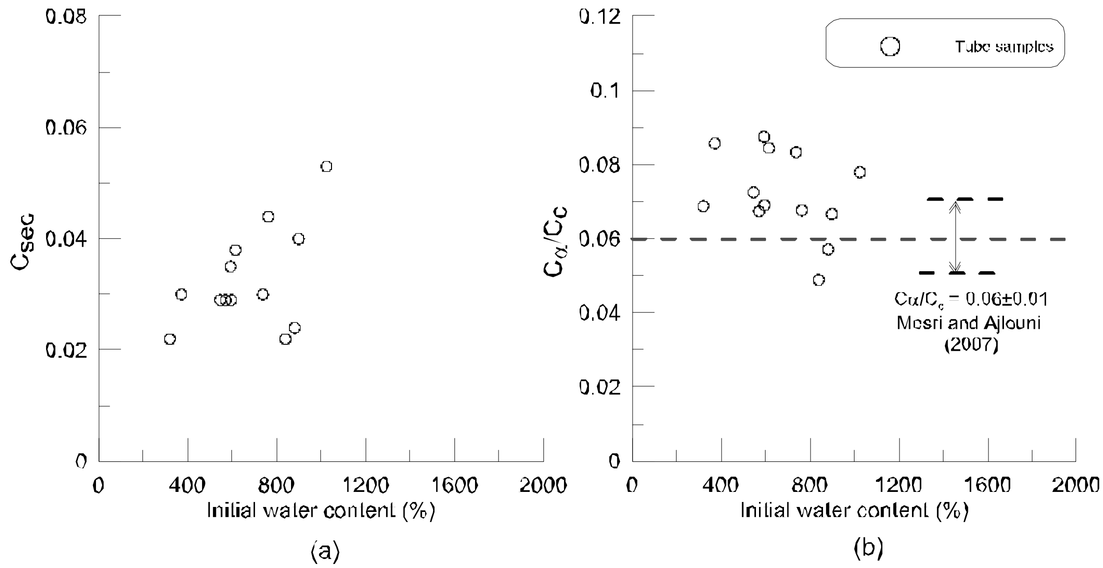

Fig. 9. Creep coefficients **(a)** C_{sec} and **(b)** C_{α}/C_c (both relate to stress ≈ p_{vy}' + 50 kPa)

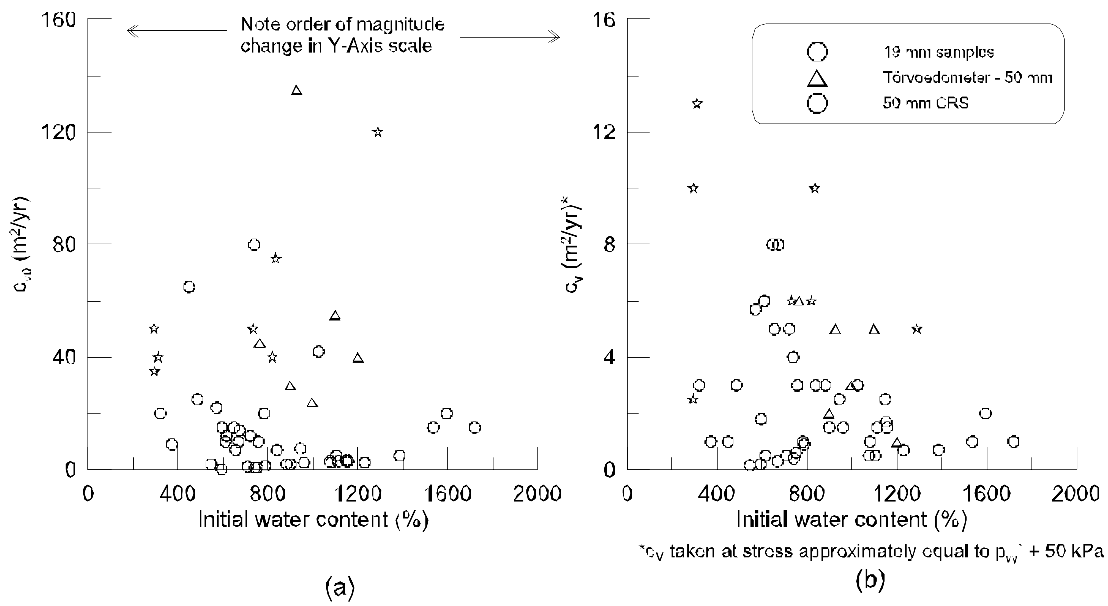

Fig 10. Coefficient of consolidation **(a)** at σ_{v0}' and **(b)** stress ≈ p_{vy}' + 50 kPa

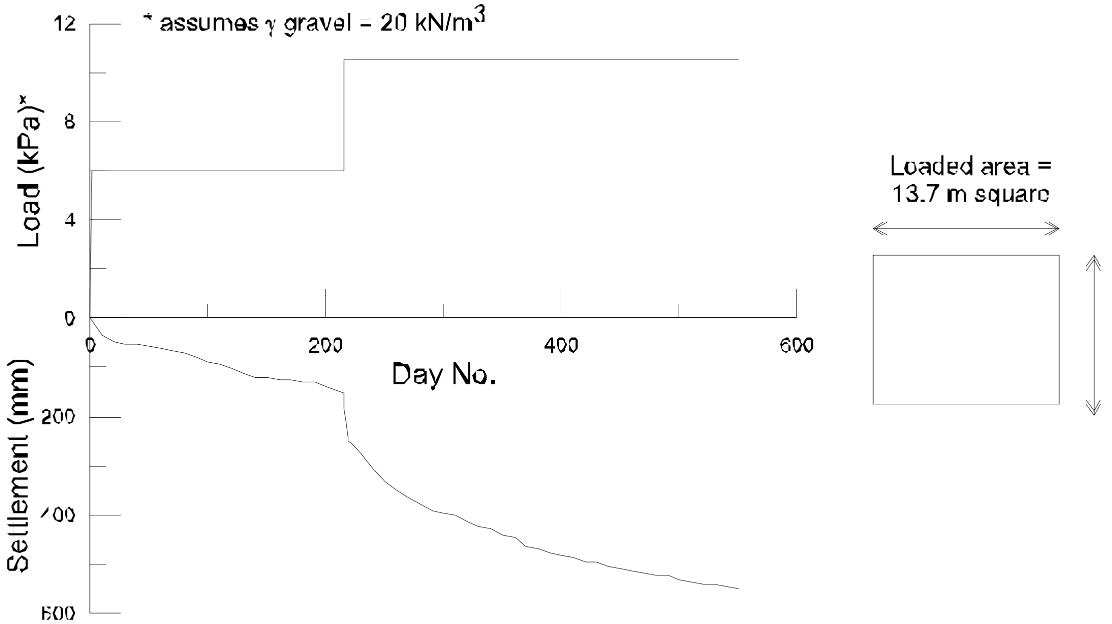

Fig 11. Longfordpass – Applied load and settlement monitoring results (Sheedy and Plant, 1968)

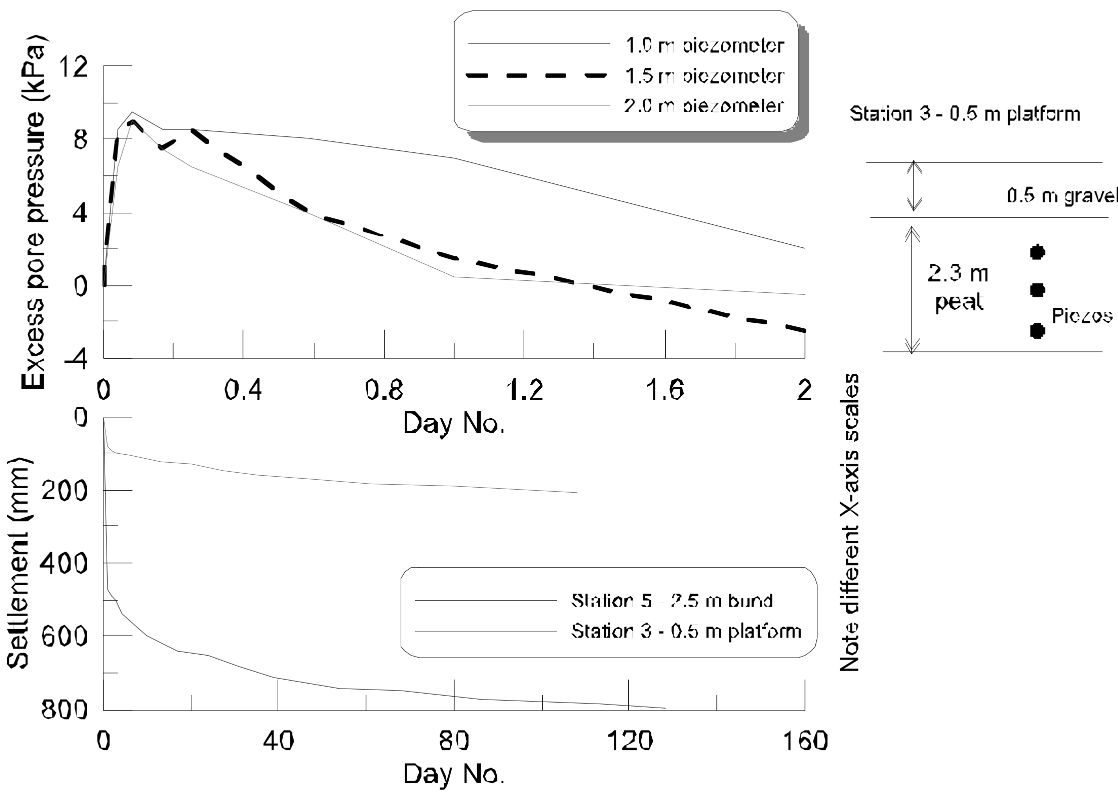

Fig 12. Heimdalsmyra – excess porewater pressure and settlement monitoring results (from Hove, 1972)

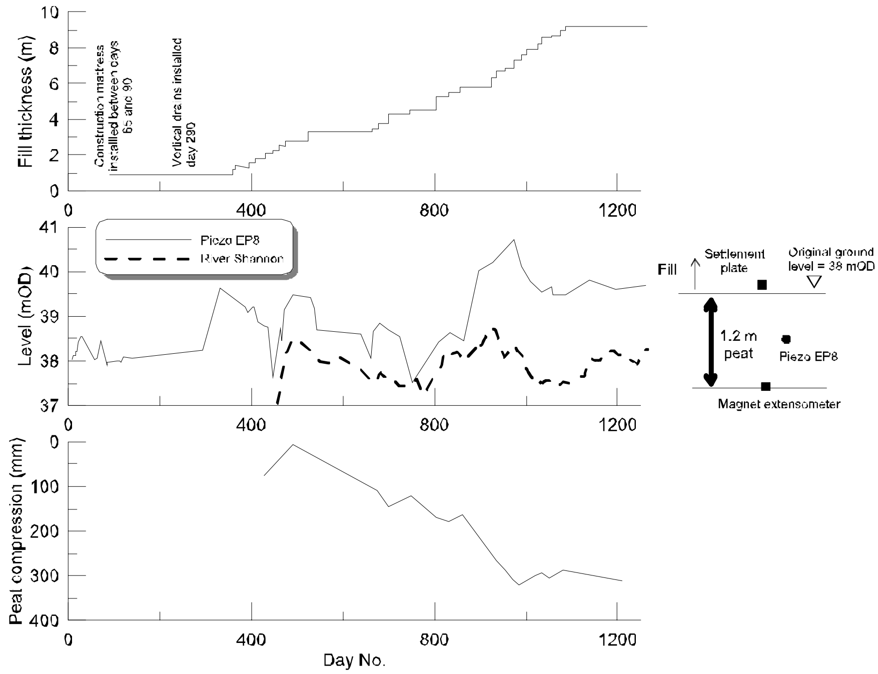

Fig 13. Athlone Bypass – fill thickness, porewater pressure and peat compression monitoring data

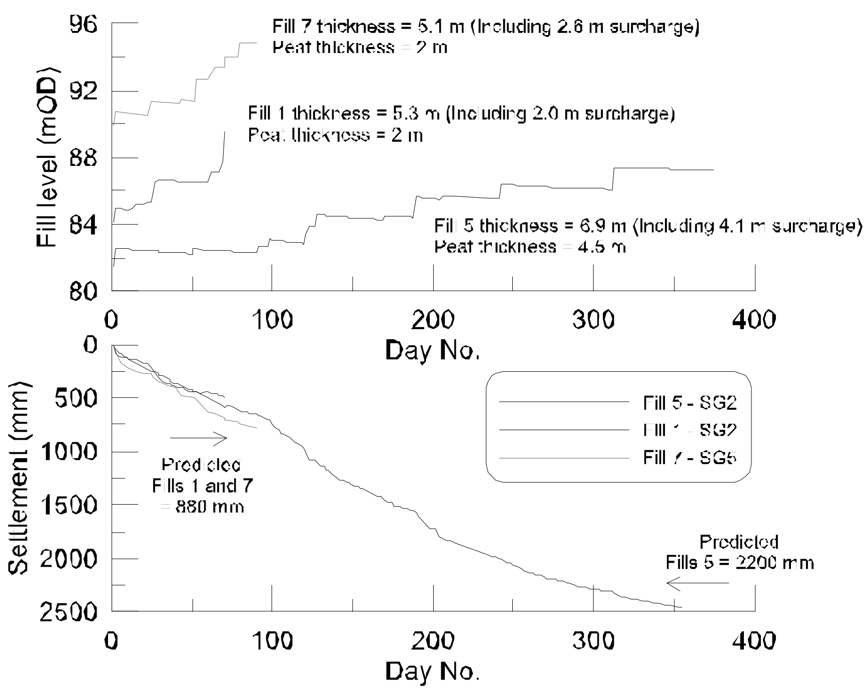

Fig 14. Knock Bypass – fill thickness and settlement data

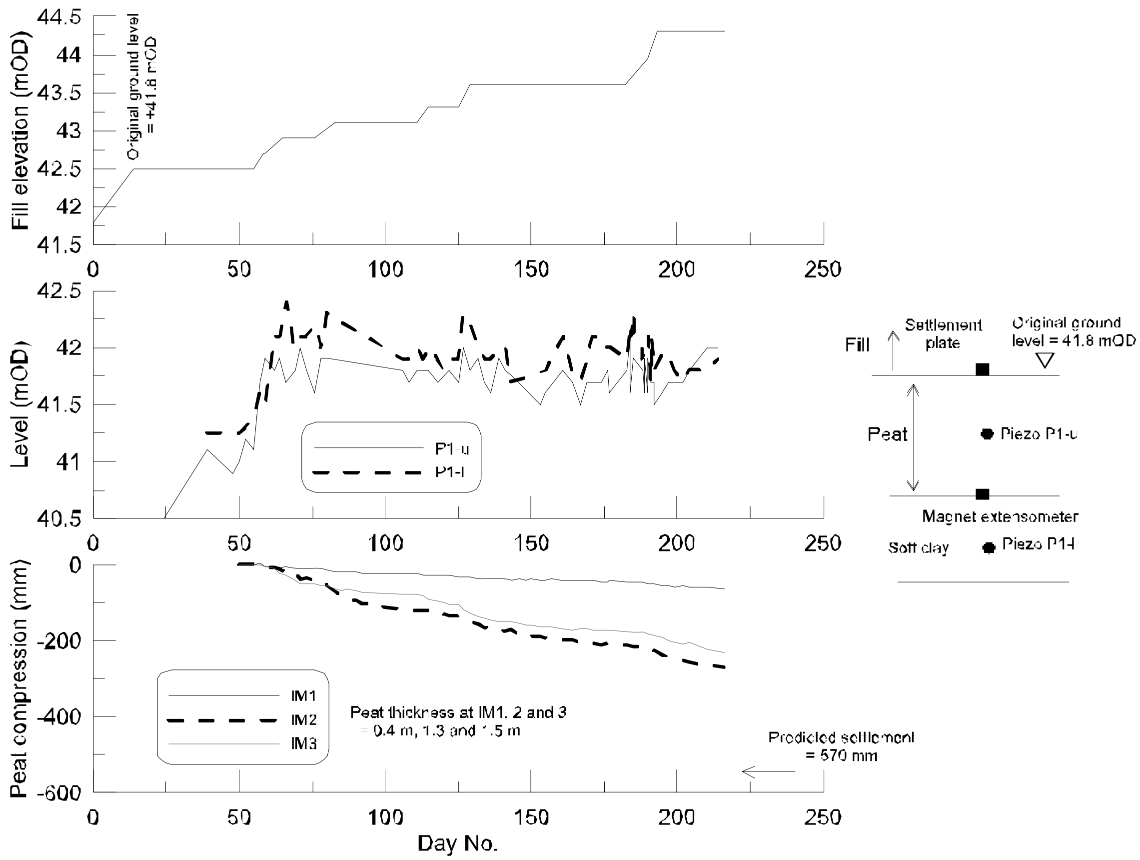

Fig 15. Carrick-on-Shannon fill thickness, porewater pressures and peat compression data

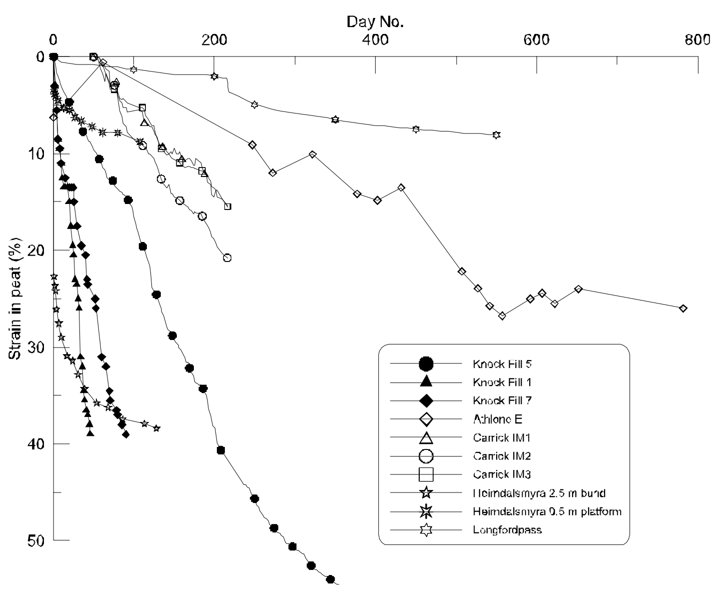

Fig 16. Strain in peat – all sites

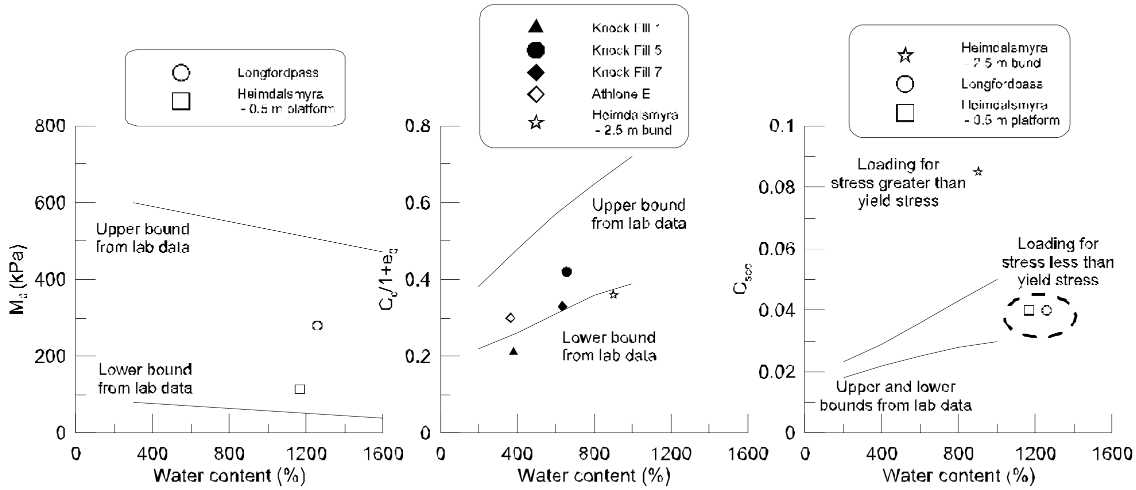

Fig 17. Parameters backanalysed from case histories**(a)** M_0, **(b)** $C_c/1+e0$ and **(c)** C_{sec}

oOo