Cite this manuscript as follows:
O'Kelly, B.C., and Zhang, L., 2013. Consolidated-drained triaxial compression testing of peat. ASTM Geotechnical Testing Journal, Vol. 36, No. 3. doi:10.1520/GTJ20120053.

Consolidated-drained triaxial compression testing of peat

Brendan C. O'Kelly and Lin Zhang

Brendan C. O'Kelly BE, MEngSc, PhD, FTCD, CEng, CEnv, MICE
Associate Professor,
Department of Civil, Structural and Environmental Engineering,
Museum Building, Trinity College Dublin, Dublin 2, Ireland.
e-mail: bokelly@tcd.ie; Tel. +353 1896 2387; Fax. +353 1677 3072

Lin Zhang BEng
Postgraduate student,
Department of Civil, Structural and Environmental Engineering,
Trinity College Dublin, Dublin 2, Ireland.

Corresponding author: Brendan O'Kelly

Number of Tables: 2
Number of Figures: 9

Submitted for possible publication in *ASTM Geotechnical Testing Journal*
First submission: 16th April 2012
Resubmitted: 21st December 2012
Accepted in full by ASTM Geotechnical Testing Journal on 19th February 2013

Abstract: Recent peat soil problems, including failures of dykes, foundations and slopes in peat deposits, have focused greater attention on understanding the mechanical behavior of peat. Stability calculations routinely involve effective stress analysis, with pertinent strength and stiffness parameters often determined from standard triaxial testing, without special consideration given to internal tensile reinforcement provided by the fiber content and also the high compressibility of the peat material. This paper investigates consolidated-drained triaxial compression testing applied to peat soils. Significant differences in mini-structure and fiber content between test-specimens of undisturbed, reconstituted and blended peat materials were found not to cause significant differences in shear resistance under drained triaxial compression, with mobilized shear resistance increasing approximately linearly with increasing axial strain. Hence it was concluded that c' and ϕ' deduced from drained triaxial

compression testing of peat are unlikely to be intrinsic material properties, and rather are largely a function of strain level, with higher values of ϕ' deduced for higher strain levels. End of primary consolidation should be deduced from pore-water pressure measurements rather than volume change response; although the repeatability of the triaxial consolidation tests was generally found to be poor on account of the natural variability of peat and small size of the test specimens.

Keywords: Consolidation; Constitutive relations; Organic soils; Laboratory tests; Mechanical properties; Poisson's ratio; Shear strength; Strength and testing of materials

1. Introduction

In recent years, failures of dykes (van Baars, 2005, McInerney et al. 2006), foundations and slopes (Long and Jennings, 2006) in peat deposits have focused greater attention on understanding the mechanical properties of peats (organic content > 75%). Excessive settlements that necessitate increased amounts of maintenance are not uncommon for peat deposits acting as pavement subgrades or foundations for railroad embankments (Hendry et al., 2012). Calculations routinely involve effective stress analysis, with the general consensus being that the principles of effective stress and routinely-used soil mechanics strength models (e.g. Mohr–Coulomb) developed for mineral soils also correlate with mechanical behavior in peat to a sufficiently high degree (Zhang and O'Kelly, 2013). Furthermore, the exclusively frictional nature of peat strength has now been well established (Adams, 1965, Mesri and Ajlouni, 2007), including the interpretation of undrained shear behavior based on the effective stress principle (Yamaguchi et al., 1985).

However peat soils have significant micro-structural and fabric differences compared with most inorganic soils (Zwanenberg, 2005, O'Kelly, 2005b, 2006, Hendry et al., 2012). Laboratory testing methods used to determine the strength properties of peats are generally the same as used for mineral soils, without special consideration being given to the fiber content, high compressibility or relatively high permeability and gas content of some fibrous peats (Farrell, 2012). Triaxial testing is one of the routinely used methods in practice since its repeatability is generally good. The triaxial apparatus providing close control of the specimen stress and boundary conditions, and it is also possible to measure pore-water pressure and other parameters. However difficulties arise in applying standard triaxial testing to peat soils.

It has been well documented that triaxial compression of fibrous peats produces very high effective angle of shearing resistance (ϕ') values in the range of 40° to 60° (Landva and La Rochelle, 1983, Farrell and Hebib, 1998, Long, 2005, Mesri and Ajlouni, 2007), as compared to less than 35° generally measured for soft clay and silt compositions. Normal compression of peat in the vertical direction mobilizes internal tensile reinforcement provided by the presence of peat fibers (Hendry et al., 2012), with the fibrous organic matter predominantly horizontally orientated *in-situ* (Yamaguchi et al. 1985). Interpretation of actual failure is also difficult on account of the large strains involved and excessive specimen deformation/volume change. For example, the effects of specimen end-restraint and high compressibility of the peat may result in excessive necking/uneven deformation of the cylindrical test-specimen during triaxial consolidation. For highly fibrous peats, the fiber effect will be quite dominant, with shear resistance continuing to develop at high strains, to the extent that failure as defined by peak deviatoric stress may not be achieved in triaxial compression (Farrell, 2012).

Another important effect due to the presence of fibers in peat is the development under consolidated-undrained (CU) triaxial compression of high excess pore-water pressures at failure, which can approximately equal the magnitude of the confining pressure (Yamaguchi et al., 1985, Farrell and Hebib, 1998, Boulanger et al., 1998). Induced pore-water pressures increase very rapidly under compression, and for axial strain (ε_a) beyond 5% to 10% in fibrous peat, approximately equal the applied cell pressure on account of the low Poisson's ratio of the fiber content (Farrell, 2012). Marachi et al. (1983) reported that for specimens of 'almost pure fibrous vegetation', the induced pore-water pressure can equal the confining pressure while the mobilized deviatoric stress is still increasing, with the maximum strength mobilized for ε_a >> 15–20%. Cola and Cortellazzo (2005) demonstarted that as the applied lateral effective stress approaches zero, the membrane could easily expand and the pore water move toward the lateral boundaries of the test-specimen. These responses make any interpretation of ϕ' from CU triaxial compression testing of fibrous peat difficult.

An extensive review of the literature by the authors produced relatively few studies reporting consolidated-drained (CD) triaxial compression testing of peat; namely Adams (1961), Hollingshead and Raymond (1972), Holubec and Langston (1972), Tsushima et al. (1977), Marachi et al. (1983) and Farrell and Hebib (1998). Drained triaxial compression tests are seldom performed due to gross changes in specimen shape and dimensions occurring during the course of the test (Edil and Wang, 2000) and also presumably because the compression stage necessitates significantly slower rates of axial strain ($\dot{\varepsilon}_a$) in order to allow direct measurement of the effective-stress shear response. Adams (1961) reported ϕ' = 51° mobilized at ε_a = 50% for undisturbed woody fine-fibrous peat specimens that were sheared very slowly over a period of three months under a relatively high mean effective-confining pressure (σ_3') of 138 kPa. However, data from shorter duration tests can be interpreted to give similarly high ϕ' values. For example, Hollingshead and Raymond (1972) reported ϕ' = 34° at ε_a = 24% for undisturbed fine-fibrous to amorphous peat under σ_3' = 1.8–8.5 kPa. Marachi et al. (1983) reported ϕ' = 37° and 44° at ε_a = 5% and 10%, respectively, for peat specimens described as pure fibrous vegetation. Marachi et al. (1983) also reported that fibrous peat in CD triaxial compression behaves very much like cohesionless mineral soil.

The aim of this paper is to investigate the CD triaxial compression test-method applied to peat soils through a coordinated program of testing on undisturbed, reconstituted and blended peats. In particular, the following are examined: (i) affects of peat fibers, mini-structure/fabric and rate of shear strain on measured values of effective angle of shearing resistance , drained Young's modulus and Poisson's ratio; (ii) overall repeatability of the test method. The present study presupposes that the effective-stress strength parameters (c', ϕ') are appropriate for peat and that if these parameters can be obtained from CD triaxial compression tests.

2. Test material

The test material in the present study was a pseudo-fibrous peat obtained from Clara bog (County Offaly, Ireland): a raised bog that originated from an early Holocene lake about 11,500 years ago and which subsequently in-filled, forming a fen about 8,000 years ago (Crushell et al., 2008). Saturated peat blocks were obtained from a waterlogged, recently-cut vertical face-bank from a depth of 2.5 m below the ground surface. The undisturbed blocks were excavated using a flat shovel and trimming saw. The blocks were sealed immediately after sampling using successive layers of plastic film and kept in sampling boxes during transportation and subsequent storage in the laboratory. The *in-situ* peat was heterogeneous, although close examination indicated a general cross-anisotropic fabric. The fossilized laminates consisted mainly of *Sphagnum*, but also some *Sedge*, and were interspersed with plant and *Calluna* (shrub) remnants, along with a small portion of woody fibers provided by shrub rootlets. Based on the slight amount of decay of the plant structures (H_4), the average water content (B_3), the low content of coarse fibers (R_1) and a low amount of wood remnants (W_1), the peat was classified as SCN–H_4 B_3 F_3(S) R_1(N) W_1(N) according to the modified von Post peat classification system (Landva and Pheeney, 1980).

3. Experimental Program

3.1 Material treatment and index properties

The sampled peat was treated in different ways to allow consideration of mini-structural and fiber effects. Material for reconstituted specimens was prepared by crumbling the peat, which required the addition of some bog water for thorough remolding. Water from its natural source was used since some engineering properties (e.g. value of liquid limit (Hanrahan et al. 1967)) are sensitive to the chemistry of the water.

Blended peat material was prepared from the remolded material using an electric handheld blender, having removing large fibers beforehand using tweezers. A gelatinous paste was obtained for testing by gently pressing and rubbing this blended material to pass the 425-μm sieve. The purpose of this preparation method was to remove the coarse fibrous fraction, with ~36% of the remolded material (wet mass basis) removed in obtaining the refined material. Pichan and O'Kelly (2012, 2013) reported that although the rate of degradation of solids is extremely slow under waterlogged conditions *in-situ*, the rate may increase significantly should environmental factors become more favorable. Hence the peat test-materials were preserved in hermetically sealed bins in order to maintain saturation and prevent the development of conditions favorable for aerobic decomposition from occurring over the course of the experimental program. Figure 1 shows that the fibers, with characteristic open cellular structure visible, remained largely intact for the remolded material, compared with the short serrated fibers and cellular-spongy matrix of the relatively homogeneous, blended peat material.

Figure 1. Scanning electron micrographs of peat materials.
(a) Remolded.
(b) Blended.

Selected properties of the peat materials are presented in Table 1. The water content (*w*) was determined by oven-drying representative specimens at 105°C over a period of 48 h, which provides accurate values of water content for peat materials (O'Kelly, 2004). The liquid limit (*LL*) and plastic limit (*PL*) values were determined using the 80g–30^o fall-cone *LL* apparatus and the Casagrande thread-rolling method (BSI, 1990*a*). The material for the *LL* test was prepared by first removing coarse fibers and wood fragments using tweezers (as allowed by the British Standard) and then mixing the remaining material using a broad-bladed knife to produce a homogeneous paste. However this material still included some elements that were greater than 425-mm in size. Hence the significantly higher *LL* value measured for the remolded material using the strength-based fall-cone method presumably reflects the effect of these coarser elements, which were absent from the blended peat material. The undisturbed and remolded materials showed little evidence of plasticity and the *PL* could not be determined on account of their fibrous nature (i.e., uniform soil threads could not be rolled out to 3.0/3.2 mm in diameter).

Table 1. Properties of peat materials.

The specific gravity of solids was determined using the small picnometer method and the loss on ignition was determined by igniting dry powdered material in a muffle furnace at 440^{o}C (BSI, 1990*a*, *b*). The undisturbed peat had an initial void ratio of 8.4 and an apparent pre-consolidation pressure of ~15 kPa; the latter determined from an oedometer void ratio against logarithm of effective stress plot using the Casagrande (1936) curve-fitting technique.

Constituent fibrous material was separated by washing representative peat specimens on the 150-μm sieve (as specified by ASTM (2008)), and also on the 63-μm sieve in order to assess the effect of the blending action, with the fiber content (*FC*) determined by expressing the oven-dried mass of the retained material as a percentage of the specimen dry mass at 105°C. According to ASTM (2007), the undisturbed *Sphagnum* peat is classified as Hemic (33% < *FC* < 67%). *Sphagnum* comprises a distinctive cellular-spongy fraction of mainly leaves and a fibrous fraction of leaf stalks, stems and rootlets. Water content tests performed after manually separating the fibrous and cellular-spongy fractions of a representative specimen of remolded peat indicated that the water content of these fraction was approximately 940% and 1130%, respectively, compared with the material's bulk water content of 1026%. This broadly agrees with Landva and Pheeney (1980) who reported that the water content of *Sphagnum* leaves ranged between 900% and 1100%, compared with an average value of 670% for stems. Blended peat had a high value of water content since this material contained a greater cellular-spongy fraction.

3.2 Specimen preparation

Sets of test specimens, 38-mm in diameter by 76-mm long, were prepared for triaxial testing. Undisturbed specimens were carved from the intact block sample using a soil lathe and sharp blade. Sets of reconstituted and blended peat specimens were prepared from peat cakes that had been consolidated one-dimensionally under the same stress regime.

Peat material prepared at a void ratio of ~15.3 was placed inside a 152-mm diameter consolidometer cell (O'Kelly, 2009) in successive ~20-mm deep layers to form a specimen, initially ~180-mm in height. This height was such that 76-mm long triaxial specimens could be carved vertically from the consolidated cake. Each of the sub-layers was individually tamped lightly and then vacuumed after fitting the lid of a vacuum desiccator above the consolidometer cell in order to remove air voids during the specimen preparation. Cakes of reconstituted and blended peat were formed by 1D consolidation, with two-way drainage to atmosphere under applied vertical stresses (σ_v) of 6, 12 and 24 kPa, which were maintained for periods of 15, 16 and 43 days, respectively. The duration of the load stage under σ_v = 24 kPa was such that the settlement response was well into the secondary compression stage, with ε_a values of 24% and 46% achieved for reconstituted and blended peats, respectively, by the end of the consolidometer tests.

3.3 Triaxial compression testing

Isotropic consolidated-drained (CID) triaxial compression testing was performed on the peat materials using a triaxial setup that included two pressure-volume controllers (GDS Instruments Ltd.), which provided automatic control of the applied cell- and back-pressures to an accuracy of 1 kPa, along with specimen volume-change measurement to an accuracy of 0.03 ml. Triaxial compression of the consolidated test-specimens was performed using a Tritech 50-kN load frame. The axial deformation of the test-specimen was measured using a linear displacement transducer (sensitivity 4.97x10^{-3} mm/mV), with the mobilized deviatoric stress measured using a submersible 3-kN load cell (sensitivity 2.4 mV/V) located inside the Perspex pressure cell. Some quick undrained (QU) triaxial compression tests were also performed under a cell pressure of 45 kPa and with $\dot{\varepsilon}_a$ = 120%/h.

The usefulness of the CID triaxial test-method as applied to peat was investigated by performing a number of tests under identical conditions. Sets of undisturbed, reconstituted and blended peat specimens were tested in order to investigate affects of mini-structure and fiber content. The specific test conditions involved consolidating the peat specimens under the same effective confining pressure (σ_3') of 30 kPa, against an applied back-pressure in excess of 200 kPa, followed by triaxial compression at a sufficiently slow rate for the drained condition to prevail. Hence all of the triaxial specimens were tested in a normally-consolidated state. The σ_3' value of 30 kPa was adopted considering the accuracy of pressure control for the triaxial setup, although it is accepted that *in-situ* effective stresses in peat deposits are generally considerably lower than this value. The additional specimen consolidation that occurred due to the small increase in effective stress between the end of 1-D compression under σ_v = 24 kPa in the consolidometer and the final desired stress of σ_3' = 30 kPa in the triaxial apparatus was designed to produce a relatively small dimensional change;i.e. test-specimens would remain almost right cylinders. The appropriate strain rate in triaxial compression was determined by applying standard curve-fitting techniques to the measured volumetric strain–elapsed time response (BSI, 1990*c*).

During the consolidation stage, the specimens' excess pore-water pressure (u_e) response was periodically measured by temporarily closing the specimen drainage lines for a period of about 30 min (O'Kelly, 2005a), after which time the u_e readings were found to have started to equilibrate. Filter-paper side drains were not fitted around the triaxial specimens since relatively large corrections would then have been required to the measured deviatoric stress for these soft test-materials. Instead, all of the specimens were consolidated under two-way vertical drainage conditions, against the applied back pressure. All of these triaxial tests, as well as the different stages of these tests, were of the same duration for consistency. Since the specimens had been designed to be physically identical and with the same stress history, the specimen set for a given material type would be expected to exhibit the same mechanical response, which includes consolidation–time and stress–strain–time behavior. The affect of the strain rate was also investigated by performing some CID triaxial compression tests at significantly faster $\dot{\varepsilon}_a$, which would not have allowed the same degree of dissipation of the excess pore-water pressures to occur.

4. Experimental Results and Analyses

4.1 Mini-structure and fabric

The undisturbed specimens exhibited significant fabric anisotropy, with the fibrous organic matter highly oriented in the horizontal direction, as has been reported by many researchers including Yamaguchi et al. (1985) and O'Kelly (2005b, 2006). This inherent 'mat' fabric has been attributed to the nature of fiber deposition and, following deposition, the large vertical strains experienced during 1D consolidation *in-situ* (Landva and Pheeney, 1980).

The reconstituted peat specimens exhibited a transition from isotropic to cross-anisotropic structure. This presumably arose on account of the large ε_a of 24% that had occurred in compressing the peat mass in the consolidometer cell to form the peat cake from which the triaxial specimens were subsequently prepared. Hendry et al. (2012) reported a similar development of cross-anisotropic behavior in remolded peat due to the re-alignment of peat fibers, perpendicular to the specimen axis, with increasing vertical strain. This induced anisotropic fabric for fibrous peat has been reported to remain intact after isotropic consolidation (Yamaguchi et al., 1985).

The blended peat showed a general isotropic fabric and, furthermore, the slurry material had a gel-like nature.

4.2 Physical properties of triaxial specimens

The values of water content and void ratio for the different peat specimens tested in triaxial compression are summarized in Table 2. As expected, the most variability occurred for the undisturbed peat specimens on account of the natural heterogeneity arising from plant growth patterns, with large variations expected to occur even over very small distances (Hobbs, 1986) since plants of different character live in communities, with decomposition not occurring uniformly throughout the peat mass (Hobbs, 1986). Some small shrub (*Calluna*) remnants that provided preferential flow channels were observed during preparation of the undisturbed specimens, particularly for specimen U1, which may explain its higher initial value of void ratio (Table 2).

Reconstituted and blended peat specimens (*FC* = 63.5% and 16.7% respectively) were prepared from cakes that had been consolidated from similar values of initial water content and under the same 1D loading regime. Higher values of water content and void ratio reported for the test-specimens of reconstituted peat can be attributed to fiber effects and the fact that water content, and hence void ratio, generally increase with increasing fiber content (Edil and Wang, 2000). Minor variations in measured physical properties of the test-specimens of blended peat arose from sample preparation, with some random occluded gas bubbles, originally present in the slurry, still evident as voids on dissecting the consolidated peat cake. By contrast, gas bubbles could escape more readily via intact fibers during compression of the reconstituted peat in the consolidometer cell. The saturation stage of the CID triaxial compression tests ensured full saturation of all test-specimen, confirmed by measured pore-pressure coefficient *B* values in excess of 0.98 for all of the tests.

Table 2. Selected properties of triaxial specimens.

4.3 Triaxial consolidation

The measured volumetric strain (ε_{v}) response during the triaxial consolidation stage is shown against square-root of elapsed time and logarithm of elapsed time in Figs. 2 and 3.

Figure 2. Volumetric strain against square-root of elapsed time during triaxial consolidation.
(a) Undisturbed and blended.
(b) Reconstituted.

Figure 3. Volumetric strain against logarithm of elapsed time during triaxial consolidation.
(a) Undisturbed and blended.
(b) Reconstituted.

Figure 4 shows the average degree of consolidation (*U*, Eq. 1) achieved during the triaxial consolidation stage.

$$U = \frac{\sigma_3' - u_e}{\sigma_3'} \tag{1}$$

where σ_3' is the effective confining pressure (i.e., 30 kPa in this study) and u_e is the measured excess pore-water pressure response.

Figure 4. Average degree of consolidation achieved during triaxial consolidation.

Figure 4 indicates that primary consolidation occurred relatively quickly, with between approximately 300 and 500 min generally required for substantive dissipation (U > 0.85–0.90) of excess pore-water pressures to occur. End of Primary (EOP) for the purposes of determining the rate of axial strain applied during the compression stage was assessed from curve-fitting analyses of the ε_{v} against elapsed time responses in Figs. 2 and 3. The strain rate generally adopted in this study was based on the time period t_{100} required in order to achieve EOP for the test-material that consolidated slowest (i.e. blended peat). These t_{100}

values were factored by 8.5 (specified for two-way specimen drainage condition by BSI (1990*c*)) in determining the strain rate of typically 0.085%/h required for substantive dissipation of the excess pore-water pressures to occur at shear failure. For the purpose of calculation, shear failure was assumed to occur at ε_a = 20%, which is often adopted as a limiting-strain condition in testing peats.

4.4 Triaxial compression

The test-specimen responses in terms of deviatoric stress q, principal stress ratio σ_1'/σ_3' and ε_v are plotted against ε_a for the three peat materials in Figs. 5–7. Note that the CID data presented in Fig. 5(a, c) are for $\dot{\varepsilon}_a$ = 0.054–0.089%/h, whereas the data in Fig. 5(b) are for $\dot{\varepsilon}_a$ = 0.077–0.833%/h. Some diurnal cycling in the shear resistance data was caused by minor changes in ambient laboratory temperature over the course of these longer-duration compression tests. The CID test-specimens contracted approximately uniformly in the radial direction during the triaxial consolidation stage and underwent approximately 1D compression (see later) during the triaxial compression stage. Hence membrane corrections were only applied to the measured deviatoric stress data for QU triaxial specimens which, by contrast, deformed by undergoing general bulging under triaxial compression. Note that the ε_v and ε_a data presented in Figs. 5–7 were computed based on the calculated specimen volume and height corresponding to the start of the triaxial compression stage (BSI, 1990*c*). The volumetric strain response was calculated from the measured volume of pore water that drained from the test-specimen during the triaxial compression stage.

The values of q and σ_1'/σ_3' mobilized for $\dot{\varepsilon}_a$ = 0.054–0.417 were found to increase approximately linearly with increasing ε_a, without reaching a peak value, even for CID tests continued to beyond 30% ε_a (Figs. 5b and 6). Hence none of the CID specimens could be taken to failure as defined by the Mohr-Coulomb criterion and, therefore, most of these tests were terminated on reaching 20% ε_a. This eventuality was not unexpected for undisturbed and reconstituted peat since it is generally not possible to bring test-specimens of these materials to failure in drained triaxial compression on account of the continual compression of the fibers and hence ongoing volumetric strain (Farrell, 2012), even for as much as 50% ε_a (McGeever, 1987). However the two CID tests continued to beyond 30% ε_a (specimens R1b and R1c, Figs. 5b and 6) suggest that the $q-\varepsilon_a$ response may deviate from linearity, with some evidence of a slight concaved-upward trend developing for ε_a > ~20%. The value of σ_1'/σ_3' increased approximately linearly from unity to ~3.7 over the ε_a range of 0% to 30%, under σ_3' = 30 kPa. This contrasts with the typical range of σ_1'/σ_3' = 10–100 developed by fibrous peats in undrained triaxial compression for which the effective lateral pressure often approaches zero with ε_a > 5–10%.

Figure 5. Deviatoric stress against strain mobilized in drained triaxial compression.
(a) Undisturbed.
(b) Reconstituted.
(c) Blended.

Figure 6. Effective stress ratio against axial strain for reconstituted specimens.

Figure 7. Volumetric strain against axial strain during triaxial compression.
(a) Undisturbed and blended.
(b) Reconstituted.

4.5 Drained Young's modulus and Poisson's ratio

The drained modulus of elasticity for the vertical direction (E'_{v}) was determined from the gradient of the $q-\varepsilon_a$ plots presented in Fig. 5, with E'_{v} = 110 to 160 kN/m^2 under σ'_3 = 30 kPa. Figure 8 shows the radial strain (ε_{r}) and drained vertical Poisson's ratio (ν') responses, which were determined from the gradient (ε_{A}) of the $\varepsilon_{\mathrm{v}}-\varepsilon_a$ plots presented in Fig. 7: where ε_{A} is the change in cross-sectional area of the test-specimen relative to its cross-sectional area determined at the start of the triaxial compression stage. These deduced values were consistent with values of ε_{r} determined from the final specimen dimensions recorded after dismantling the triaxial cell. Note that a value of ε_{A} of unity indicates 1D compression.

The CID test-specimens of undisturbed and reconstituted peat generally responded by bulging only very slightly, essentially undergoing 1D compression, with deduced mean ν' values of 0.02–0.03 and 0.04–0.05, respectively, for ε_a = 0–20% and $\dot{\varepsilon}_a$ = 0.077–0.417. As expected, significant increases in strain rate produced $q-\varepsilon_a$ responses more characteristic of undrained triaxial compression. For example, specimen R1d ($\dot{\varepsilon}_a$ = 0.833%/h; i.e. ε_a = 20% after 24 h period) showed signs of mobilizing peak values of q and σ'_1/σ'_3 at ε_a ~15% (Fig. 5b), with deduced ν' = 0.18–0.21 for ε_a = 0–15% (Fig. 8b). However there was no physical evidence of a shear plane having developed for any of the CID test-specimens, even for ε_a > 30%. Furthermore the uniformity of the diametrical dimension over the specimen height indicated that effects of specimen end-restraint due to platen friction were not significant.

The blended peat specimens deformed by general bulging, but again without a shear plane developing, although significantly higher mean values of ν' = 0.13–0.16 deduced for ε_a = 0–20% and $\dot{\varepsilon}_a$ = 0.054–0.084 occurred (Fig. 8b). The strain response of the three blended peat specimens (B1–B3, Fig. 7a) were almost identical, whereas test-specimens of reconstituted and in particular undisturbed peat experienced slightly different responses on account of the natural heterogeneity of these materials.

In contrast, QU triaxial compression at $\dot{\varepsilon}_a$ = 120%/h generally produced a shear failure. The undisturbed specimen QU–U mobilized a peak q value at ε_a = 16% (Fig. 5a), rupturing along a distinct shear plane inclined at ~42$^{\mathrm{o}}$ to the horizontal. The blended peat specimen QU–B (Fig. 5c) responded differently, deforming significantly by lateral bulging, with a shear resistance value close to q peak maintained for ε_a of between 10% and 40%, with this test terminated on reaching 40% ε_a. However examination of this test-specimen after dismantling the triaxial cell indicated evidence of a shear plane having developed at 52–55^{o} to the horizontal.

Fig. 8. Specimen response in drained triaxial compression.
(a) Radial strain.
(b) Poisson's ratio.

On completion of the triaxial tests, some test-specimens were sectioned longitudinally but no apparent changes in structure due to shearing were evident. Other test-specimens were sectioned and portioned for water content testing in order to determine the distribution of water content throughout the specimen. All specimens were oven dried at 105^{o}C over a period of 48-h for final water content determinations. Measured values of water content were generally uniform throughout the test-specimen, both radially and vertically, which was expected since the CID test-specimens had been practically fully consolidated by the end of the 3-day triaxial consolidation period (Fig. 4).

Fiber effects were also evident in the oven-drying responses of the different peats. Undisturbed specimens tended to part along predominantly-horizontal bedding planes. Reconstituted specimens remained completely intact. In contrast, blended peat specimens contracted in volume by about one-third, with some necking developing near the specimen ends, before disintegrating into irregular/angular fragments on nearing the fully dry state, despite having significantly lower final water contents compared with the undisturbed and reconstituted triaxial peat specimens (Table 2).

5. Discussion

5.1 Consolidation

In general, the volumetric strain response during triaxial consolidation (Figs. 2 and 3) was strongly related to the structure of the peat materials. Undisturbed specimens consolidated significantly quicker than reconstituted specimens, presumably on account of the mini-structure present in the former. Reconstituted specimens consolidated quicker than blended peat specimens, presumably on account of the significantly greater size and number of preferential flow channels provided by the largely intact fibers present in the former and also the greater cellular-spongy fraction of the latter. However the difference in volumetric strain response measured for the set of test-specimens of a given material type was unexpectedly large (e.g. up to ~7.5% strain for reconstituted peat, Fig. 3b), given that the test-specimens had been prepared in an identical manner and with the same stress history, and were subsequently tested under identical conditions in the triaxial apparatus. This indicates the natural variability of peat, but also suggests that the repeatability of triaxial consolidation tests on 38-mm diameter pseudo-fibrous and amorphous peat specimens may be generally poor on account of the small size of the test specimen.

5.2. Drained compression

5.2.1 Structure and fiber effects

The conventional consensus is that, due to fiber effects, undisturbed and reconstituted peat test-specimens should mobilize higher shear resistance compared with blended peat in the triaxial mode of shear. Peat fibers affect the geotechnical behavior by providing internal lateral resistance to shear deformation. Since an increase in pore pressure reduce this resistance, loading under drained conditions provides better stability through higher lateral resistance (Landva and La Rochelle, 1983). In the present study, mobilized values of q and σ_1'/σ_3' increased approximately linearly with increasing ε_a for all three peat materials (Figs. 5 and 6), with the test-specimens showing no signs of approaching failure, even for ε_a > 30%. This eventuality was not unexpected for the undisturbed and reconstituted peat specimens (with *FC* = 63.5%) since it is generally not possible to bring a fibrous peat specimen to failure in drained triaxial compression on account of the continual compression of the fibers (Farrell, 2012). These experimental findings are also consistent with previous research that reported the qualitative behavior of undisturbed and remolded peat is similar (Hanrahan and Walsh, 1965, Hanrahan et al., 1967) and, furthermore, pre-consolidation and anisotropic consolidation have little effect on the values of the strength parameters (Adams, 1965).

In contrast, Long (2005) reported that for peats having low *FC*, fiber reinforcement effects will be insignificant and a shear failure may be expected to occur. However this was not the case for the blended peat, whose response was somewhat unexpected, with similar $q-\varepsilon_a$ responses measured for all three peat materials in drained triaxial compression (Fig. 5), given that structural and fiber affects would have been significantly less for blended peat. Constituent fibers in the blended peat had been serrated by blending as part of the material preparation, with all of this material passing the 425-μm sieve, and a measured *FC* (based on the percentage dry mass retained on 150 μm sieve) of only 16.7%. Furthermore, undisturbed specimen U1 and reconstituted peat specimen R2 did not appear to receive any significant strength enhancement from the *Calluna* remnants that were particularly noticeable in these specimens. This would suggest that mini-structure and fiber content do not have significant effects on the resistance mobilized under drained triaxial compression of 38 mm diameter specimens. Similar findings were reported by Edil and Wang (2000) who reported that fibrous and amorphous peats show no perceptible differences in values of effective friction angle. Furthermore, the relatively large scatter in initial values of water content and void ratio measured between test-specimens of a given peat material (Table 2) did not appear to cause similar scatter in the $q-\varepsilon_a$ responses. These findings agree closely with those of Holubec and Langston (1972) from CD triaxial testing of fibrous peat and also with Adams (1965) and Hollingshead and Raymond (1972) from CU triaxial testing on Canadian muskegs.

5.2.2 Effective angle of shearing resistance

Values of q and σ_1'/σ_3' mobilized in drained triaxial compression generally increased approximately linearly with increasing ε_a (Figs. 5 and 6), without the development of a shear plane. Hence the value of ϕ' deduced from a Mohr Circle of Stress analysis is strain-level dependent (Fig. 9). The value of c' could not be determined from the available experimental data since all of the CID tests in the present study had been performed for the same σ'_3 value of 30 kPa. Values of c' = 0 have been measured in drained triaxial compression testing of fibrous peat by Tsushima et al. (1977), Marachi et al. (1983) and Farrell and Hebib (1998). Hence a value of c' of zero was assumed in the calculations for ϕ' for the three peat test-materials in the present study.

Taking ε_a = 20% as an arbitrary failure condition (often applied in such circumstances in practice) produced ϕ' values of 30.2±1.5°, 28.9±1.1° and 30.3±1.0° for undisturbed, reconstituted and blended peats respectively. These ϕ' values, deduced for *w* = 378–556% (Table 2), are in good agreement with ϕ' = 34° reported for 50-mm diameter, undisturbed fine-fibrous to amorphous peat specimens (*w* = 558% initially) tested in drained triaxial compression under much lower σ'_3 = 1.8–8.5 kPa (Hollingshead and Raymond, 1972). Like the present study, these tests had been arbitrarily terminated at ε_a = 24% since, up to this strain level, the measured $q-\varepsilon_a$ responses were practically linear (E'_v = 50–200 kN/m^2), with none of the test-specimens mobilizing peak q values.

Higher ϕ' values would be deduced from the experimental data for higher strain levels, and *vice versa*. For example, extrapolation of the experimental $\phi'-\varepsilon_a$ relationships for the different peat materials in the present study (see Fig. 9) indicated a range of ϕ' = 48–52$^\mathrm{o}$ for ε_a = 50%. This range is consistent with ϕ' = 51° reported by Adams (1961) for undisturbed fine-fibrous peat (*w* = 375–430%) at ε_a = 50% in drained triaxial compression of 48.3-mm diameter test-specimens sheared over a three-month period ($\dot{\varepsilon}_a$ = ~0.0023%/h), but under much higher σ'_3 = 138 kPa.

Hence these experimental observations would suggest that for pseudo-fibrous and amorphous peats, the value of ϕ' deduced from drained triaxial compression tests is unlikely to be an intrinsic property of the material, but rather is approximately proportional to of strain level, provided that $\dot{\varepsilon}_a$ is sufficiently slow for substantial dissipation of excess pore-water pressures to occur. Moreover, given that the value of c' is established by projecting the Mohr-Coulomb envelope back to intersect the shear stress axis, c' is also unlikely to be an intrinsic material property since it is intimately dependent on the interpretation of ϕ'.

Figure 9. Values of ϕ' deduced from drained triaxial compression of peat. Note: Hollingshead and Raymond (1972) and Marcchi et al. (1983) values were determined from reported $q-\varepsilon_a$ plots.

The engineering behavior of peat also depends on its morphology (Farrell, 2012), which is often not adequately described in literature. Values of ϕ' = 37$^\mathrm{o}$ and 44$^\mathrm{o}$ at ε_a = 5% and 10%, respectively, reported by Marachi et al. (1983) for ‘almost pure fibrous vegetation’ from the San Jaoquin Delta, California, were determined from drained triaxial compression of 76-mm diameter test-specimens. These test-specimens also had significantly lower initial water contents compared with the other peats considered in Fig. 9, which were either pseudo-fibrous or amorphous. Some research has suggested that different failure envelopes occur for the same peat material but prepared at significantly different initial water contents, with ϕ' increasing with decreasing water content (Sodha, 1966). Hence the significantly higher ϕ' mobilized at comparatively small ε_a reported by Marachi et al. (1983) may be due, at least in part, to differences in morphology and initial water content of the test-specimens, although further research is necessary in this regard.

5.3 Strain rate effects

The affect of $\dot{\varepsilon}_a$ in drained triaxial compression can be assessed by considering the responses of the six reconstituted peat specimens (Figs. 5–8). Apart from specimen R1d ($\dot{\varepsilon}_a$ = 0.833%/h), the other five specimens were found to mobilize similar values of q and σ_1'/σ_3', which increased approximately linearly with increasing ε_a, even though they had been compressed at significantly different rates ($\dot{\varepsilon}_a$ = 0.077–0.417%/h). As noted earlier, specimen R1d showed signs of mobilizing peak values of q and σ_1'/σ_3' at ε_a ~15%, somewhat characteristic of CU triaxial compression behavior of fine-fibrous to amorphous peat (Hollingshead and Raymond, 1972), and also bulged considerably (ν' = 0.18–0.21) compared with the other five reconstituted specimens, all of which essentially underwent 1D compression. Calculations based on the time periods of typically between 200 and 500 min required to achieve substantive dissipation of excess pore-water pressures (U > 0.85–0.90, Fig. 4) during triaxial consolidation suggested that $\dot{\varepsilon}_a$ = 0.28–0.47%/h would have been adequate for the triaxial compression stage. These observations indicate that for the peat materials tested in the present study, the drained triaxial compression stage could have been performed at an $\dot{\varepsilon}_a$ about ten time quicker than that deduced from standard curve-fitting analysis of the experimental ε_{v}–elapsed time data to BSI (1990*c*), without significant adverse effects on the accuracy of deduced strength and stiffness properties. Such curve-fitting techniques were developed for mineral soils, but secondary compression in peat is substantial, often exceeding the primary consolidation component. Hence in the case of peats, the authors concur with Edil and den Haan (1994) in that the value of $\dot{\varepsilon}_a$ should be based on EOP estimations deduced from measurements of pore-water pressure rather than the measured volume change response.

5.4 Drained Poisson's ratio and fiber effects

The slightly lower mean ν' range of 0.02–0.03 deduced over ε_a = 0–20% for drained triaxial compression of the undisturbed pseudo-fibrous peat material presumably reflects the greater affect of its predominantly cross-anisotropic 'mat' fabric compared with the transitional isotropic to cross-anisotropic structure of the reconstituted peat (ν' = 0.04–0.05). Furthermore, the significantly higher ν' range of 0.13–0.16 deduced for drained blended peat presumably arises from its general isotropic structure and considerably lower fiber content. The experimental ν' ranges determined for undisturbed and reconstituted pseudo-fibrous peat in the present study are considerably lower than reported ν' values for fibrous peat of

0.10 (Zwanenberg, 2005), 0.11 (Rowe et al., 1984) and 0.15 (Rowe and Mylleville, 1996, Tan, 2008), although these values were determined from undrained triaxial compression tests. This can be explained by considering the lateral resistance (internal tensile force) induced by the fibers, which is a function of the friction developed between the fibers (or between fibers and the cellular-spongy matrix) and also of the tensile strength of the fibers themselves (Landva and La Rochelle, 1983). Another possible explanation for such low ν' values could be the effects of specimen end-restraint due to platen friction and/or scale effects, considering the relative size of the peat fibers constituting the 38-mm diameter test-specimens. However drained specimens under triaxial compression at a sufficiently slow rate for substantial dissipation of excess pore-water pressures to occur essentially underwent 1D compression in all three peat test-materials suggesting that end friction was not a significant factor in the present study.

For the fully-drained condition, full friction is mobilized between the fibers and also with the matrix material under the effective normal stress, σ'_{n}. However for undrained conditions, excess pore-water pressures develop rapidly and may approximately equal the applied cell pressure for ε_a > 5–10% (Yamaguchi et al., 1985, Farrell and Hebib, 1998, Boulanger et al., 1998). Hence, although the deviatoric stress still continues to increase (Marachi et al., 1983), the fiber reinforcement diminishes under the reducing σ'_{n}, with the peak q value mobilized for $\varepsilon_a \approx$ 15 to 25%. Under these conditions, the friction between fibers and with the matrix material is only partly mobilized or not mobilized at all, depending on the magnitude of the excess pore-water pressures. Similar behavior would occur for drained triaxial compression testing performed at too high a strain rate. For example, significant lateral bulging occurred for specimen R1d (ν' = 0.18–0.21) under $\dot{\varepsilon}_a$ = 0.833%/h, which was shown earlier to be too rapid for excess pore-water pressures to dissipate sufficiently. Hollingshead and Raymond (1972) reported ν' decreasing from 0.36 to 0.24 with increasing σ'_3 from 1.8 to 8.5 kPa in CD triaxial compression testing of undisturbed fine-fibrous to amorphous peat. Hence the findings of the present study and of Hollingshead and Raymond (1972) would suggest that the measured ν' for fibrous peat is dependent on the level of effective stress, reducing significantly with increasing effective stress.

Given all of the above observations, there are major difficulties with CD triaxial compression testing of peat and the authors concur with Landva et al. (1986), in that standard drained triaxial compression testing of peat is not particularly useful for determining its effective-stress strength properties. Finally, the properties of the peats soils presented above are based on laboratory investigations. Their applicability directly in the field is not clearly understood and is beyond the scope of this paper.

Conclusions

The principal conclusions from the program of standard consolidated-drained triaxial compression testing performed on saturated pseudo-fibrous *Sphagnum* peat can be summarized as follows:

During triaxial consolidation, the ε_{v} response was strongly related to mini-structure and fiber content. Undisturbed peat, with its intact and predominantly cross-anisotropic ‘mat’ fabric, generally consolidated significantly quicker than reconstituted peat, which had a transitional isotropic to cross-anisotropic structure. Reconstituted peat consolidated quicker than the more homogeneous blended peat material, with the latter predominantly comprised of cellular-spongy material. Differences in ε_{v} responses for samples of a given peat material, having identical stress history and tested under the same conditions, were significant. This occurred due to the natural variability of peat, which arose even under the controlled conditions of material preparation, but also indicates the generally poor repeatability of triaxial consolidation tests on peat specimens of small size. It is preferable that the rate of axial strain in triaxial compression tests be based on ‘end of primary’ consolidation estimates deduced from the measured pore-water pressure response, rather than from curve-fitting analysis of the volume change response which is generally too conservative.

Standard drained triaxial compression testing of peat is not particularly useful for determining its effective-stress strength properties. It was not possible to bring any of the peat test-specimens to failure under drained compression, with mobilized values of q and σ_1'/σ_3' increasing approximately linearly with increasing ε_a, even for ε_a > 30%. The drained modulus of elasticity ranged from 110 to 160 kN/m^2 for σ_3' = 30 kPa. Furthermore, significant differences in mini-structure and fiber content did not produce significant differences in shear resistance under drained triaxial compression. Hence c' and ϕ' deduced from drained triaxial compression are unlikely to be intrinsic material properties, but rather are largely a function of strain level, with higher ϕ' values deduced for higher strain levels.

However fiber effects were significant in terms of limiting lateral expansion of the test-specimens under drained triaxial compression. The undisturbed and reconstituted peats (*FC* = 63.5%) responded by essentially undergoing 1D compression, with mean ν' ranges of 0.02–0.03 and 0.04–0.05 respectively. Blended peat (*FC* = 16.7%) deformed by general bulging, again without a shear plane developing, with mean ν' of 0.13–0.16. Significant lateral bulging of fibrous peat also occurs for test-specimens compressed at too high a strain rate for sufficient dissipation of the excess pore-water pressures to occur. Furthermore the value of ν' measured for fibrous peat was dependent on the level of effective stress, with the above ranges deduced for σ_3' = 30 kPa.

Acknowledgments

The authors would like to acknowledge the following contributions by Trinity College Dublin (TCD) personnel to the work reported herein: Martin Carney and Eoin Dunne in assisting with the geotechnical laboratory testing; Patrick Veale with the peat sampling and Derek Simpson in obtaining the SEM images. This paper was written and submitted by the first author for peer-review during a period of sabbatical leave from TCD. The second author gratefully acknowledges a Postgraduate Research Award from TCD.

References

Adams, J.I. 1961. Laboratory compression test on peat. Proceedings of the 7th Muskeg Research Conference, Ottawa, Ont. ACSSM Technical Memorandum 71, pp. 36–54.

Adams, J.I. 1965. The engineering behavior of a Canadian muskeg. Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. 1, pp. 3–7.

ASTM 2007. Standard Classification of Peat Samples by Laboratory Testing, ASTM D4427–07. ASTM International, West Conshohocken, PA, United States.

ASTM 2008. Standard Test Method for Laboratory Determination of the Fiber Content of Peat Samples by Dry Mass, ASTM D1997–91. ASTM International, West Conshohocken, PA, United States.

Boulanger, R.W., Arulnathan, R., Harder, L.F. Jr., Torres, R.A., and Driller, M.W. 1998. Dynamic properties of Sherman Island peat. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 124(1): 12–20.

BSI 1990*a*. Methods of Test for Soils for Civil Engineering Purposes (Classification Tests), BS1377: Part 2. British Standards Institution, Milton Keynes.

BSI 1990*b*. Methods of Test for Soils for Civil Engineering Purposes (Chemical and electro-chemical tests), BS1377: Part 3. British Standards Institution, Milton Keynes.

BSI 1990*c*. Methods of Test for Soils for Civil Engineering Purposes (Shear strength tests (effective stress)), BS1377: Part 8. British Standards Institution, Milton Keynes.

Casagrande, A. 1936. The determination of the preconsolidation load and its practical significance. Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, p. 60.

Cola, S., and Cortellazzo, G. 2005. The shear strength behaviour of two peaty soils. Geotechnical and Geological Engineering, 23: 679–695.

Crushell, P.H., Connolly, A., Schouten, M.G.C., and Mitchell, F.J.G. 2008. The changing landscape of Clara Bog: the history of an Irish raised bog. Irish Geography, 41: 89–111.

Edil, T.B., and den Haan, E.J. 1994. Settlement of peat and organic soils, ASCE Geotechnical Special Publication, 40(2): 1543–72.

Edil, T.B., and Wang, X. 2000. Shear Strength and Ko of Peats and Organic Soils. In: Geotechnics of High Water Content Materials, ASTM STP 1374, T.B. Edil and P.J. Fox, Eds., American Society for Testing and Materials, West Conshohocken, PA, United States.

Farrell, E.R. 2012. Organics/peat soils. In: ICE Manual of Geotechnical Engineering (Burland J, Chapman T, Skinner H and Brown M (eds)). ICE Publishing, London. Vol. 1, pp. 463–479.

Farrell, E.R., and Hebib, S. 1998. The determination of geotechnical parameters of organic soils. Proceedings of the International Symposium on Problematic Soils (IS-TOHOKU 98), Sendai, Japan. Vol. 1, pp. 33–36.

Hanrahan, E.T. and Walsh, J.A. 1965. Investigation of the behavior of peat under varying conditions of stress and strain. Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. 1, pp226–230.

Hanrahan, E.T., Dunne, J.M., and Sodha, V.G. 1967. Shear strength of peat. Proceedings Geotechnical Conference, Oslo, Vol. 1, pp. 193–198.

Hendry, M.T., Sharma, J.S., Martin, C.D., and Barbour, S.L. 2012. Effect of fiber content and structure on anisotropic elastic stiffness and shear strength of peat. Canadian Geotechnical Journal, 49: 403–415.

Hobbs, N.B. 1986. Mire morphology and the properties and behaviour of some British and foreign peats. Quarterly Journal of Engineering Geology and Hydrogeology, 19: 7–80.

Hollingshead, G.W., and Raymond, G.P. 1972. Field loading tests on Muskeg, Canadian Geotechnical Journal, 9(3): 278–289.

Holubec, I., and Langston, E., 1972. Analysis and performance of a dike on fibrous peat. Proceedings of the ASCE Conference on Performance of Earth and Earth-Supported Structures, Purdue University, Lafayette, Ind., pp. 415–434.

Landva, A.O., and Pheeney, P.E. 1980. Peat fabric and structure. Canadian Geotechnical Journal, 17: 416–435.

Landva, A.O., and La Rochelle, P. 1983. Compressibility and shear characteristics of Radforth peats. In: Testing of Peats and Organic Soils. ASTM Special Technical Publication 820. American Society for Testing and Materials, West Conshohocken, PA. pp. 157–191.

Landva, A.O., Pheeney, P.E., La Rochelle, P., and Briaud, J.L. 1986. Structures on peatland—geotechnical investigations. Proceedings of the Conference on Advances in Peatlands Engineering, Ottawa. pp. 31–52.

Long, M. 2005. Review of peat strength, peat characterization and constitutive modelling of peat with reference to landslides. Studia Geotechnica et Mechanica, 27(3–4): 67–90.

Long, M., and Jennings, P., 2006. Analysis of the peat slide at Pollatomish, County Mayo, Ireland. Landslides, 3: 51–61.

Marachi, N.D., Dayton, D.J., and Dare, C.T. 1983. Geotechnical properties of peat in San Joaquin Delta. In: Testing of Peats and Organic Soils. ASTM Special Technical Publication 820. American Society for Testing and Materials, West Conshohocken, PA, pp. 207–217.

McGeever, J. 1987. The Strength Parameters of an Organic Silt, MSc Thesis, University of Dublin, Trinity College.

McInerney G.P., O'Kelly B.C., and Johnston P.M. 2006. Geotechnical aspects of peat dams on bog land. Proceedings of the 5th International Congress on Environmental Geotechnics, Cardiff, 26th–30th June 2006. Vol. 2, pp. 934–941. doi.org/10.1680/ieg.34723

Mesri, G., and Ajlouni, M.A. 2007. Engineering properties of fibrous peat. ASCE Journal of Geotechnical and Geoenvironmental Engineering, 133(7): 850–866.

O'Kelly, B.C. 2004. Accurate determination of moisture content of organic soils using the oven drying method. Drying Technology, 22(7): 1767–1776. DOI: 10.1081/DRT-200025642

O'Kelly, B.C. 2005a. Consolidation properties of a dewatered municipal sewage sludge. Canadian Geotechnical Journal, 42(5): 1350–1358. DOI: 10.1139/t05-054

O'Kelly, B.C. 2005b. Compressibility of some peats and organic soils. Proceedings International Conference on Problematic Soils, Cyprus, 25th–27th May 2005. Vol. 3, pp. 1193–1202.

O’Kelly, B.C. 2006. Compression and consolidation anisotropy of some soft soils. Geotechnical and Geological Engineering, 24(6): 1715–1728. DOI: 10.1007/s10706-005-5760-0

O'Kelly, B.C. 2009. Development of a large consolidometer apparatus for testing peat and other highly organic soils. SUO – Mires and Peat, 60(1–2): 23–36. http://www.suoseura.fi/suo/pdf/Suo60_Okelly.pdf

Pichan, S.P., and O’Kelly, B.C. 2012. Effect of decomposition on the compressibility of fibrous peat. Proceedings of ASCE GeoCongress 2012: State of the Art and Practice in Geotechnical Engineering, 25th–29th March, Oakland, California, USA. GSP 225 (Eds. R.D. Hryciw, A. Athanasopoulos-Zekkos and N. Yesiller), GSP 225, pp. 4329–4338. DOI: 10.1061/9780784412121.445

Pichan, S.P., and O’ Kelly, B.C. 2013. Stimulated decomposition in peat for engineering applications, Proceedings of the Institution of Civil Engineers, Ground Improvement, 166. DOI: 10.1680/grim.12.00003

Rowe, R.K., MacLean, M.D., and Soderman, K.L. 1984. Analysis of a geotextile reinforced embankment constructed on peat. Canadian Geotechnical Journal, 21(3): 563–576.

Rowe, R.K., and Mylleville, B.L.J. 1996. A geogrid reinforced embankment on peat over organic silt: a case history. Canadian Geotechnical Journal, 33(1): 106–122.

Sodha, V.G. 1966. Some aspects of the behavior of peat in shear. MEngSc thesis, University College Dublin.

Tan, Y. 2008. Finite element analysis of highway construction in peat bog. Canadian Geotechnical Journal, 45(2): 147–160.

Tsushima, M., Miyakawa, I., and Iwasaki, T. 1977. Some investigations on shear strength of organic soil. Tsuchi-to-Kiso, Journal of Soil Mechanics and Foundation Engineering, 235, 13–18 (*in Japanese*).

van Baars, S. 2005. The horizontal failure mechanism of the Wilnis peat dyke. Géotechnique, 55(4): 319–323.

Yamaguchi, H., Ohira, Y., Kogure, K., and Mori, S. 1985. Undrained shear characteristics of normally consolidated peat under triaxial compression and extension conditions. Japanese Society of Soil Mechanics and Foundation Engineering, 25(3): 1–18.

Zhang, L. and O'Kelly, B.C. 2013. The principle of effective stress and triaxial compression testing of peat, Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, 166. DOI: 10.1680/geng.12.00038

Zwanenberg, C. 2005. The influence of anisotropy on the consolidation behavior of peat. Ph.D. thesis, TU Delft, Delft, the Netherlands.

List of TWO Tables:

List of NINE Figures:

Table 1. Properties of peat materials.

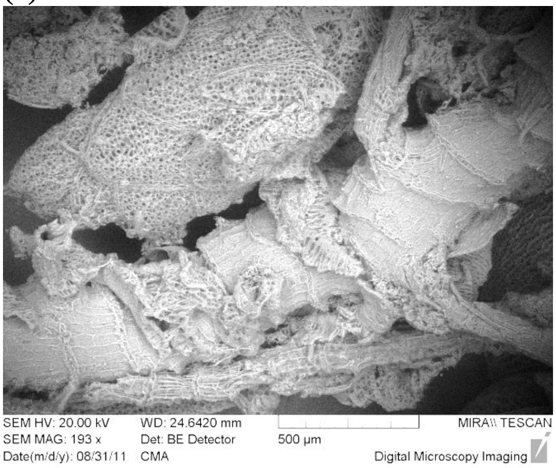
Peat type	Undisturbed (U) and remolded (R)	Blended
Water content (%)	590 (U), 1026 (R)	1065
Liquid limit (%)	1135 (R)	757
Plastic limit (%)	Non-plastic	446
Plasticity index	—	311
Specific gravity of solids	1.42	1.42
Loss in dry mass on ignition (%)	98.6	98.5
Fiber content (% retained on 63-μm sieve)	74.2	27.1
Fiber content (% retained on 150-μm sieve)	63.5	16.7
pH	3.8	3.7

Table 2. Selected properties of triaxial specimens.

Material Type	Undisturbed				Reconstituted						Blended			
Specimen ID	QU–U	U1	U2	U3	R1a	R1b	R1c	R1d	R2	R3	QU–B	B1	B2	B3
Initial water content (%)	626	623	560	584	757	799	781	746	755	767	495	523	530	502
Initial void ratio	8.9	8.8	8.0	8.3	10.7	11.4	11.1	10.6	10.7	10.9	7.0	7.4	7.5	7.1
Void ratio at end of consolidation stage	–	7.8	7.7	7.9	9.5	9.7	9.2	9.2	9.2	9.6	–	6.4	6.2	6.7
Water content at end of shearing stage (%)	626	471	463	455	543	450	403	515	553	556	495	378	386	393
Void ratio at end of shearing stage	8.9	6.7	6.6	6.5	7.7	6.4	5.7	7.3	7.9	7.9	7.0	5.4	5.5	5.6
End of primary, t_{100} (min)	–	169	64	676	841	1156	484	576	225	841	–	2401	784	2116
Theoretical strain rate (%/h)	–	0.835	2.20	0.209	0.168	0.122	0.292	0.245	0.627	0.168	–	0.059	0.180	0.067
Actual strain rate applied (%/h)	120	0.087	0.087	0.089	0.088	0.208	0.417	0.833	0.077	0.084	120	0.054	0.084	0.079

Figure 1. Scanning electron micrographs of peat materials.

(a) Remolded.



(b) Blended.

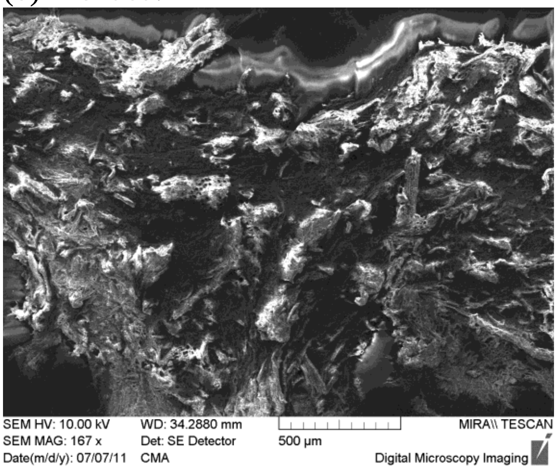
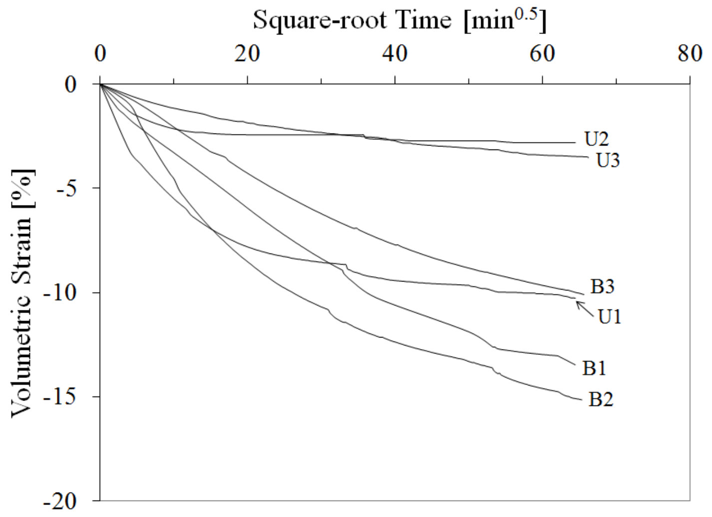


Figure 2. Volumetric strain against square-root of elapsed time during triaxial consolidation.

(a) Undisturbed and blended.



(b) Reconstituted.

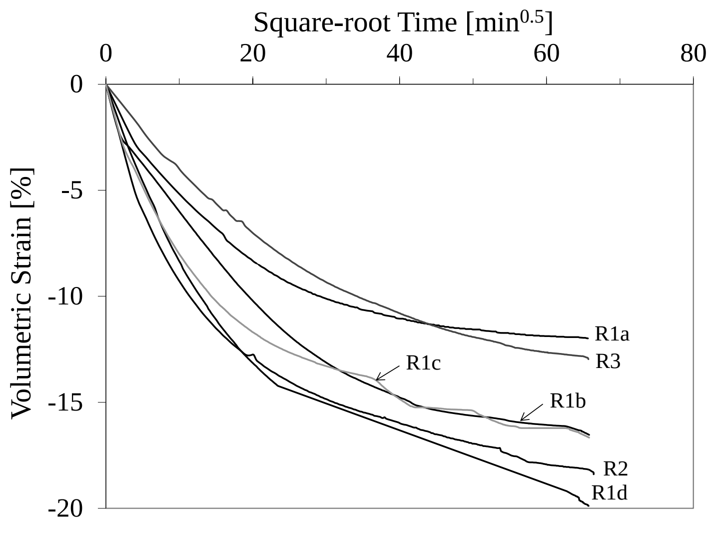
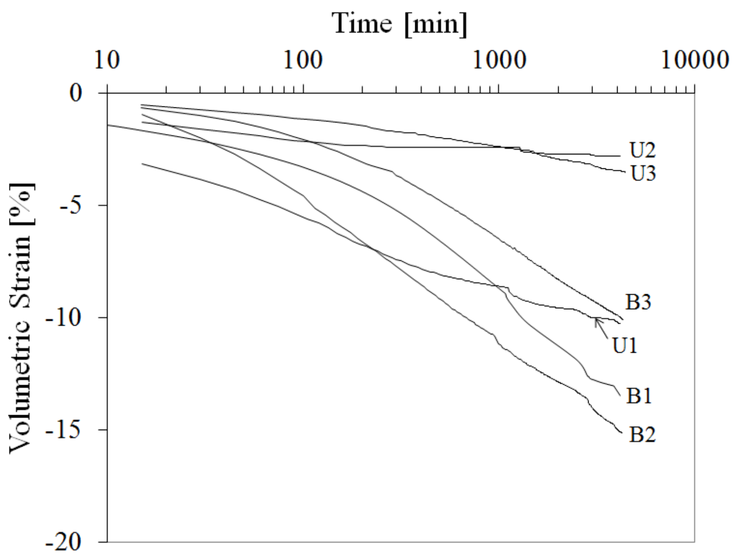


Figure 3. Volumetric strain against logarithm of elapsed time during triaxial consolidation.

(a) Undisturbed and blended.



(b) Reconstituted.

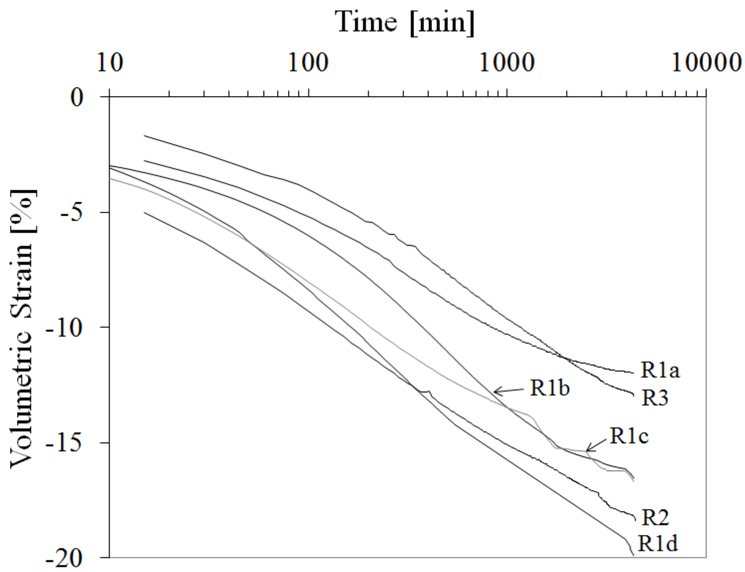


Figure 4. Average degree of consolidation achieved during triaxial consolidation.

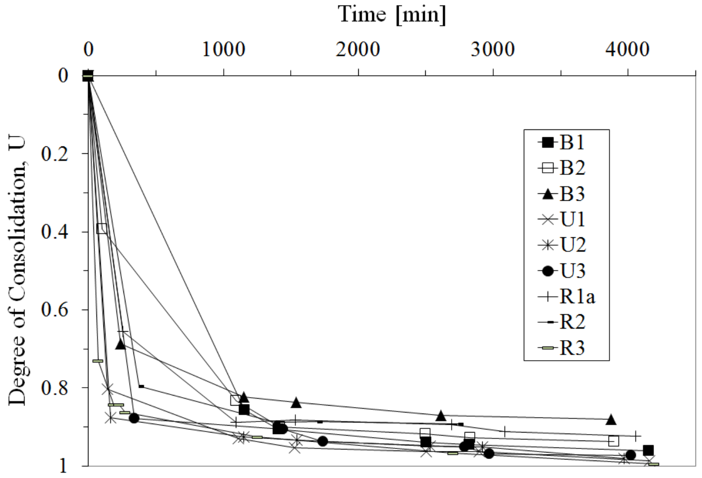
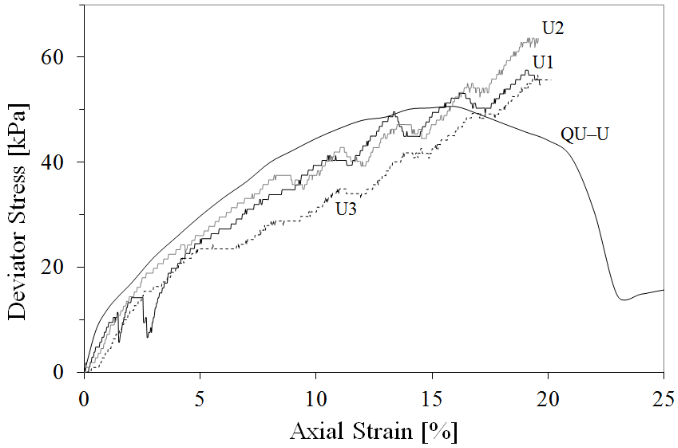
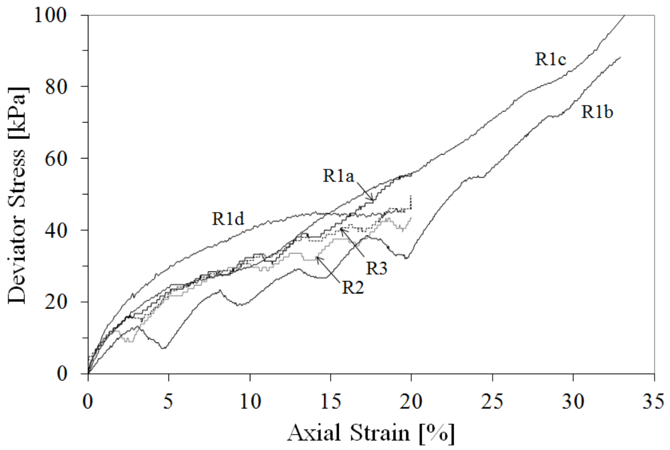


Figure 5. Deviatoric stress against strain mobilized in drained triaxial compression.
(a) Undisturbed.



(b) Reconstituted.



(b) Blended.

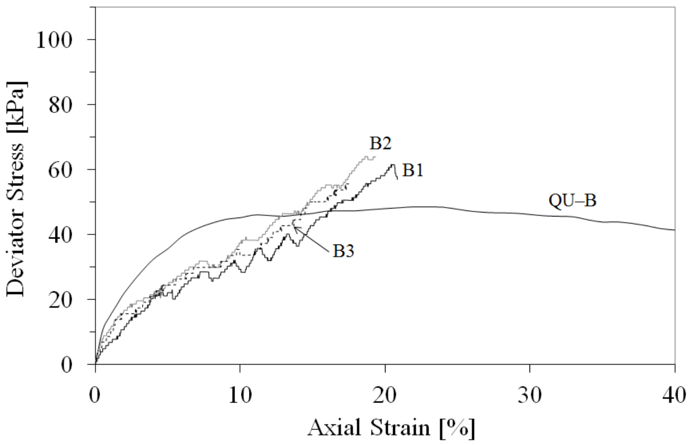


Figure 6. Effective stress ratio against axial strain for reconstituted specimens.

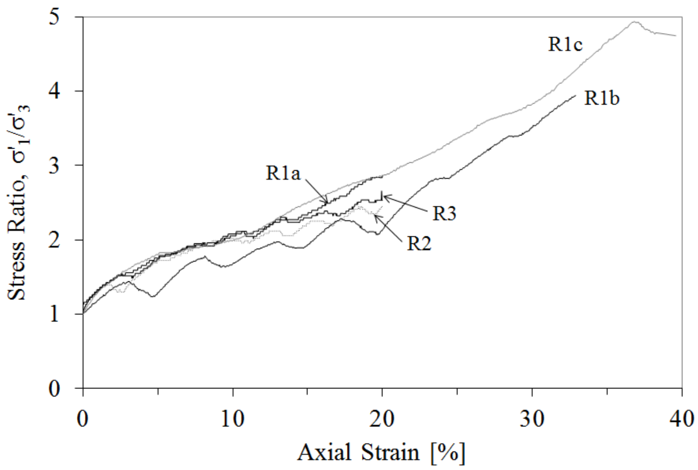
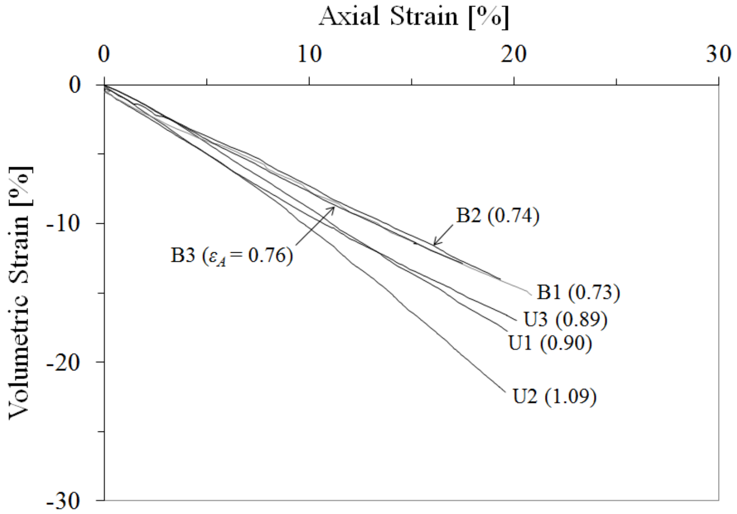


Figure 7. Volumetric strain against axial strain during triaxial compression.

(a) Undisturbed and blended.



(b) Reconstituted.

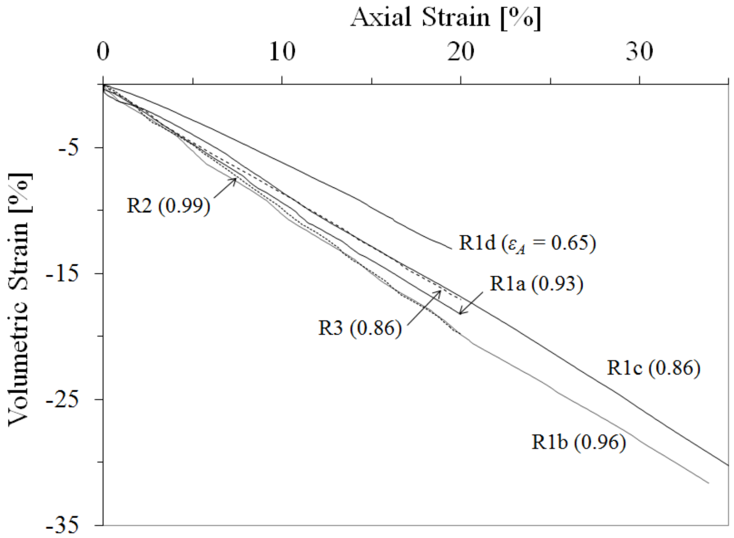
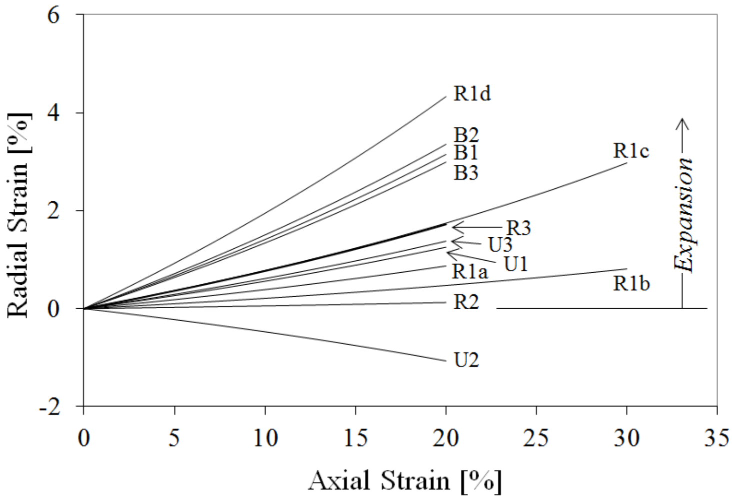


Figure 8. Specimen response in drained compression.

(a) Radial strain.



(b) Poisson's ratio.

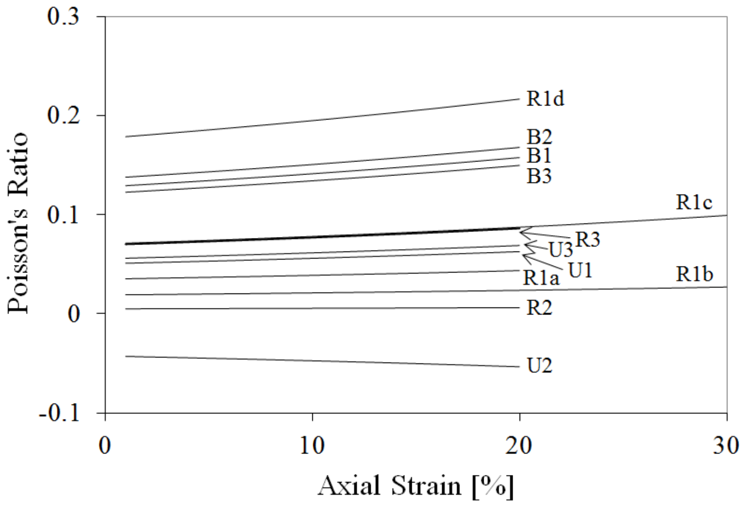


Figure 9. Values of ϕ' deduced from drained triaxial compression of peat. Note: Hollingshead and Raymond (1972) and Marcchi et al. (1983) values were determined from reported $q-\varepsilon_a$ plots.