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Investigation of the Atterberg limits and undrained fall-cone shear strength variation with water content of some peat soils

P. J. Vardanega[a]*, C. L. Hickey[b,c], K. Lau[b], H. D. L. Sarzier[b], C. M. Couturier[b], G. Martin[a]

[a]*Department of Civil Engineering, University of Bristol, Bristol, United Kingdom*
[b] *Formerly, Department of Civil Engineering, University of Bristol, Bristol, United Kingdom*
[c]*Civil engineer, Arup, London, United Kingdom*

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Abstract

Road construction in peatlands is challenging. The ability to make rapid estimates of the response of construction soils derived from natural peats to changes in water content is useful for pavement and geotechnical engineers. This paper details some laboratory test results on peat soils sourced from two sites in the South-west of England. The samples were sieved and the roots and natural fibres removed prior to laboratory testing. Water contents on the natural specimens were determined. The percentage of roots in the samples was determined. The thread rolling test was used to estimate the plastic limit of the peat soil material. A series of fall cone tests were conducted at varying moisture contents to determine the liquid limit of the peat soil as well as study the variation of fall cone undrained shear strength with the liquidity index, logarithmic liquidity index and the water content ratio. Both the liquidity index and logarithmic liquidity index are able to predict the fall cone undrained strength to within ± 40% around 90% of the time. When using the water content ratio to predict the fall cone undrained shear strength an accuracy of ± 40% is achieved around 85% of the time. The study concludes that the liquidity index and logarithmic liquidity index are better predictors of fall cone undrained shear strength but the water content ratio approach may be preferred if the engineer is less confident in plastic limit determination for peat soils.

Keywords: Peats; Fall cone testing; Atterberg limits; Undrained strength; Liquidity index

1. Introduction

Road construction in peat areas is very difficult due to the high settlement potential of peat deposits [1]. According to Spedding [2] peat "…is invariably found with significant moisture content at the surface of the ground, within a depth of between 2 and 15 meters." Peat soils exhibit low values of undrained shear strength in natural conditions [3,4]. Natural peats contain very high water content, "…low strength, high compressibility and high shrinkage on drying" [1]. Various studies detailing construction challenges in such deposits have been published [5-8]. Edil [3,9,10] has reviewed construction over peat materials – focusing on the compressibility characteristics of the material. The review paper by O'Kelly and Pichan discussed the decomposition and compressibility characteristics of peat soils and concluded that "uncontrolled or unexpected decomposition in fibrous peat deposits may cause significant additional settlement of bearing strata, adversely impacting on the performance of engineering structures founded on or within such deposits" [11]. Nie et al [12] also studied the influence of organic content on decomposition and concluded that for the Chinese peats in their study organic content had a major influence on the mechanical properties. Use of peat as a lightweight fill material for construction purposes is assisted by the fact that upon drying "…an irreversible change takes place in the colloidal fractions and the peat will 'take-up' only a small fraction of the water it originally contained on being immersed again" [1]. This investigation aims (in part) to see the potential variations in undrained shear strength due to variations of water content (w). This paper aims to: (a) Compare the measured Atterberg limits and loss on ignition (LOI) for peat soils from two locations in the South-West of England to those for some other English peat soils – reported in the literature [13-14] and (b) Determine the undrained shear strength variation with increasing water content for the peat soils studied and compare the results to previously published studies [15-18]. A summary of the sample collection, preparation and testing details is given in Section 3 of this paper (for further details see the dissertations of Hickey [19], Lau [20] and Sarzier and Couturier [21]).

* Corresponding author
E-mail addresses: p.j.vardanega@bristol.ac.uk (P. J. Vardanega); Claire.Hickey@arup.com (C. L. Hickey); kaikyology@gmail.com (K. Lau); helena.sarzier@laposte.net (H. D. L. Sarzier); chloe.couturier@outlook.fr (C. M. Couturier); gary.martin@bristol.ac.uk (G. Martin)
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2. Background

2.1. Plastic Limit (w_P)

O'Kelly [22] has criticised the use of Atterberg limits for peat soils, the major reasons being the presence of fibres and the variability of the test results. Skempton and Petley [13] stated "The plastic limit cannot be readily determined, even with the highly humidified peats, and the test is undoubtedly subject to personal variations". However, the thread rolling test remains an important test and arguably the only way to determine the brittle transition point [23] and will therefore, be retained in this study. The challenges of determining the w_P are clear when the soil fibres are included in the sample. This study focuses on the soil component of the peat material, where the thread rolling test is easier to perform.

2.2. Liquid Limit (w_L)

The definition of the liquid limit as the water content of 20mm penetration of a 30 degree, 80 g cone was proposed in Sherwood and Ryley [24] and subsequently adopted into the British Standard BS1377-1975 [25]. O'Kelly et al. [26] recently published a review of the use of fall cones to determine the Atterberg limits of fine-grained soils, concluding in part, that the fall cone should be adopted universally to determine the liquid limit.

2.3. Undrained strength at the liquid limit

Based on a review of previously published data-sets, Wroth and Wood [15] adopted an undrained shear strength at w_L (c_L) of 1.7 kPa. Nagaraj et al [27] also reviewed the ranges of undrained shear strength measured at the liquid limit from different publications with different methods of c_u determination arguing that a wide range exists in the reported literature and that a unique strength should not be assigned to c_L. In discussion of [27], Haigh and Vardanega [28] argued ascribing an undrained strength at w_L is valid as the fall cone test is essentially a strength measurement and if w_L is to be assigned to a particular fall cone penetration, then an undrained strength can be associated with it. The fall cone test can be used to estimate undrained shear strength. Hansbo [29] gave the following Equation which can be used to back-analyse fall-cone tests and estimate undrained shear strength (c_u):

$$c_u = K\left(\frac{mg}{d^2}\right) \tag{1}$$

where, K = the cone factor; m = fall cone mass, g = gravitational acceleration and d = fall cone penetration. When using the British standard cone [30] which has a mass of 80 g and a cone angle of 30 degrees as well as assuming that the c_u at 20 mm penetration (liquid limit) is equal to 1.7 kPa a K value of 0.867 is calculated. A theoretical value of 1.33 for the cone factor (semi-rough condition) was given by Koumoto and Houlsby for a 30 degree cone [31]. Wood [32] explored the effect of changing cone angle on the cone factor. Brown and Huxley [33] reviewed the cone factor for the 30° cone in more detail and showed slightly lower cone factors than 0.867 for the BSI cone as they took c_L = 1.5 kPa. K=0.867 is used in this paper and therefore for the analysis presented in this work a c_L of 1.7 kPa is assumed throughout. It must be noted that the values of fall-cone undrained shear strength ($c_{u,FC}$) quoted in this work are as accurate as the assumed value of 1.7 kPa at the liquid limit [16].

2.4. Variation of c_u with changes in w

Many researchers have related changes in undrained shear strength to changes in liquidity index (I_L)[15-18,34], which is computed by Eq. (2):

$$I_L = \frac{w-w_p}{w_L-w_p} \tag{2}$$

Wroth and Wood [15] gave an equation of the form:

$$c_u = 100c_L \exp(-4.6I_L) \tag{3}$$

Eq. (3) implies a strength variation of 100 from w_L to w_P (an assumption made in Schofield and Wroth [35], based on examination of data from Skempton and Northey [36], an assumption that was used in [15] and [37]). Wood [34,38] gives a more general form of Eq. (3) which can be written as:

$$c_u = c_L {R_{MW}}^{(1-I_L)} \tag{4}$$

where, R_{MW} = the computed factor increase in c_u as the w decreases from w_L to w_P.

Koumoto and Houlsby [31] advocated the use of logarithmic liquidity index (I_{LN}) (Eq. (5)) when modelling the change of undrained shear strength with changes in water content:

$$I_{LN} = \frac{\ln(\frac{w}{w_P})}{\ln(\frac{w_L}{w_P})} \tag{5}$$

Koumoto and Houlsby [31] showed that Eq. (6) matched the data for six fine grained materials reasonably well. Eq. (6) retains the factor 100 strength increase assumed in [15] but introduces the I_{LN} concept. Eq. (6) gives as strength of 1.38 kPa at liquid limit: Koumoto and Houlsby [31] were using a 60g, 60° cone.

$$I_{LN} = 1.070 - 0.217\ln(c_u) \tag{6}$$

Vardanega and Haigh [16] compiled a large database of 101 soils (641 fall cone measurements, all taken using the BSI cone) and after performing regression analysis produced two equations of the form suggested in Wood [34], Eqs. (7) and (8). Eqs. (7) and (8) link undrained shear strength (back-analysed from the fall cone) to I_L and I_{LN} respectively:

$$c_{u,FC} = c_L 34.3^{(1-I_L)} \tag{7}$$

where, c_L = 1.7 kPa and $0.2 < I_L < 1.1$

$$c_{u,FC} = c_L 83.5^{(1-I_{LN})} \tag{8}$$

where, c_L = 1.7 kPa and $0.2 < I_{LN} < 1.1$

Kuriakose et al. [17] advocated the use of water content ratio (w/w_L) (or the void ratio (e) normalised with the void ratio at w_L (e_L)) to predict the undrained shear strength. Earlier studies have also shown the benefits of the (w/w_L; e/e_L) ratio when studying aspects of soil behaviour [39,40,41,42]. Regression relationships linking changes in c_u to changes in w/w_L has been reported in various publications [43,44,45,46]. Vardanega and Haigh [18], when discussing Kuriakose et al. [17], re-analysed the same database presented in Vardanega and Haigh [16] and reported the following regression Equation:

$$log_{10}(c_{u,FC}) = 2.662 - 2.432\left[\frac{w}{w_L}\right] \tag{9}$$

Vardanega and Haigh [18] did note that the coefficient of determination calculated for Eq. (9) was lower than that obtained when using I_L or I_{LN} as the predictor of $c_{u,FC}$ [16].

3. Material and methods

3.1. Sample Collection

The 'Exmoor peats' were sourced from the Exmoor National Park near the town of Dulverton in Squallacombe (site 1) and Blackpitts (site 2). At the Exmoor sites, the superficial grassed surface was removed and the samples were taken from a depth of approximately 100 mm and placed in plastic bags. The 'Glastonbury peats' were sourced from the RSPB Ham Wall Nature Reserve in Glastonbury. At the Glastonbury site, samples were collected from two locations about 100 m apart at a depth of approximately 200 mm.

3.2. Sample Preparation

The samples were brought back to the Geomechanics Laboratory at the University of Bristol having been sealed in plastic bags on site. Several moisture contents were measured from each bag (results summarised in Table 1). Free water was not used as part of the moisture content testing at this stage. In accordance with BS 1377 [30], a portion of the peat sample from each bag was weighed and put in a bucket with de-ionised water added which was then mixed until the roots and organic fines were separated out. The material was then washed through a 0.425 mm sieve into another bucket – material retained in the sieve was oven dried until constant mass was achieved to determine the mass (percentage) of roots (see Table 1). Material that passed through the 0.425 mm sieve was allowed to settle in the bucket until supernatant water appeared which was then poured off. The residual sludge was then transferred into trays and oven dried at approximately 50ºC with the aim of achieving a water content co-incident with a 15 mm fall cone drop (modified from BS 1377, Part 2, 1990: clause 4.2.4.8, [30]). In some cases, a hand drier was used to further dry the sample prior to initial fall cone testing. When penetrations were sought lower than 15 mm the hand drier was also used during sample preparation. After fall-cone testing water contents were determined using an oven drying temperature of about 105ºC as per BSI (1990).

3.3. Laboratory Testing

The Atterberg limits [47,48] were determined in accordance with BSI [30] using the remoulded material that had had the roots and other material removed by sieving using the 0.425 mm sieve (as described in the previous section).

3.3.1. Fall Cone Liquid Limit

The liquid limits were determined using (BSI [30]) using the standard British Cone (80 g, 30 degrees with the w_L corresponding to the water content at 20 mm penetration). Previous research by Kodikara et al. [49] and Feng [50, 51] advocated the use of power law functions to describe the variation of water content (w) with fall cone penetration (d) over the more traditional semi-logarithmic formulations [15]. For the data-set analysed in this paper, little variation in the computed values of w_L between the semi-log and power law approaches was observed. The w_L values shown in Table 1 and used in this study were determined using the regression function generated from fitting ln(w) to ln(d) (power relationship) and using this to derive the water content at 20mm penetration. In addition to fall cone penetrations taken about the liquid limit, penetrations on drier samples were taken so as to capture the undrained strength increase with the concomitant decrease in water content. Table 2 shows the full set of fall cone penetrations which range from 4.97 to 26.45 mm (n=126).

A summary of the measured fall cone test data is given in Table 2, note that some tests were conducted at penetrations less than 16 mm (generally not used in w_L determination) to estimate undrained strengths closer to the w_P. The c_u calculated using Eq. (1) range from about 1 kPa to 27.5 kPa with an average value of 3.8 kPa. While these values are derived from back-analysis of remoulded, sieved peat soil this range does compare well with that given by Moayedi and Nazir [4] in a recent review paper who suggested a range of c_u of 3 to 17 kPa for peat materials. Edil [3] stated a range of c_u values for peats in natural conditions of between 5 and 20 kPa. While the fall-cone undrained shear strength values reported in this paper are for the soil component only the range of values are comparable with those quoted in both [3,4]. Therefore, the relationships derived in this paper may prove useful for geotechnical engineers wishing to make estimates of undrained shear strength and its variation as water content changes.

3.3.2. Thread Rolling Plastic Limit

The w_P is a very useful soil mechanics test as it determines the brittle transition point of the material [23]. The plastic limit test

Table 1
Summary index test data (a) water content (w) and Atterberg limits (w_L and w_P) and (b) loss on ignition.

(a)	w (%)				w_L (%)				w_P (%)			
Site	μ	σ	COV(%)	n	μ	σ	COV(%)	n	μ	σ	COV(%)	n
Glastonbury 1	980	53.9	5.5	8	657	75	11.5	6	390	40.7	10.4	7
Glastonbury 2	762	47.5	6.2	6	607	71	11.7	7	394	27.6	7	5
Exmoor 1.1	769	-	-	1	494	-	-	1	220	15	6.9	8
Exmoor 1.2	606	-	-	1	545	-	-	1	310	31	10	7
Exmoor 2.1	720	36.2	5	2	634	-	-	1	324	26.2	8.1	8

(b)	LOI (%) un-sieved				LOI (%) sieved			
Site	μ	σ	COV(%)	n	μ	σ	COV(%)	n
Glastonbury 1	91.3	1.3	1.4	8	90.6	3.9	4.3	6
Glastonbury 2	91.5	0.1	0.2	7	90	1.1	1.2	8
Exmoor 1.1 Exmoor 1.2	83.6	1.6	1.9	2	91.2	0.05	0.1	2
Exmoor 2.1	89.7	0.7	0.8	2	94.5	0.1	0.1	2

n = number of data points; μ = mean value; σ = standard deviation; COV = coefficient of variation

Table 2
Fall cone test data.

No.	Test Series	d (mm)	w (%)	No.	Test Series	d (mm)	w (%)	No.	Test Series	d (mm)	w (%)
1	Glastonbury 1.1	16.57	626	43		18.34	755	85		15.33	521
2		19.85	658	44		19.62	773	86		16.88	538
3		17.96	640	45	Glastonbury 2.1	19.68	681	87		18.33	553
4		21.45	673	46		17.13	666	88		19.74	567
5		22.14	682	47		18	667	89	Glastonbury 2.6	6.26	422
6	Glastonbury 1.2	16.8	675	48		20.86	689	90		8.37	437
7		16	668	49		21.28	695	91		8.58	432
8		17.79	683	50	Glastonbury 2.2	16.25	598	92		7.77	436
9		19.02	685	51		16.56	595	93		9.23	447
10		20.29	695	52		17.69	606	94		9.98	448
11	Glastonbury 1.3	18.1	670	53		19.44	615	95		10.94	458
12		19.25	672	54		21.41	625	96		10.19	457
13		18.59	671	55	Glastonbury 2.3	16.09	593	97		11.33	463
14		22.77	676	56		16.54	595	98		12.2	474
15		21.2	674	57		19.05	608	99		13.63	483
16	Glastonbury 1.4	11.64	571	58		19.71	609	100		14.44	489
17		11.43	570	59		20.81	616	101		15.81	507
18		12.79	573	60	Glastonbury 2.4	10.26	481	102		16.84	518
19		13.5	581	61		10.32	482	103		19.03	545
20		14.11	584	62		10.81	509	104	Glastonbury 2.7	21.22	766
21		15.32	589	63		11.24	507	105		16.67	686
22		15.09	583	64		11.46	511	106		15.72	683
23		15.91	593	65		11.92	502	107		17.01	624
24		16.61	595	66		12.15	500	108		4.97	463
25		17.88	606	67		12.63	492	109		14.87	636
26		19.4	618	68		13.49	498	110		14.62	637
27		22.56	636	69		13.9	509	111		11.01	560
28		21.79	636	70		14.35	517	112	Exmoor 1.1	15.05	459
29	Glastonbury 1.5	8.13	475	71	Glastonbury 2.5	11.29	487	113		18.75	476
30		8.96	481	72		11.28	490	114		22	500
31		8.58	477	73		12.61	501	115		23.6	529
32		10.95	493	74		11.27	486	116	Exmoor 1.2	14.3	488
33		11.5	496	75		13.09	506	117		17.2	502
34		12.32	501	76		11.82	491	118		17.85	519
35		13.03	503	77		12.2	495	119		20.1	541
36		13.28	505	78		11.09	485	120		22	570
37		13.72	512	79		13.08	502	121		23.7	589
38		14.56	515	80		13.72	509	122	Exmoor 2.1	14.7	565
39	Glastonbury 1.6	9.5	509	81		14.08	513	123		16.75	599
40		17.46	687	82		15.03	521	124		19	609
41		16.1	637	83		15.15	524	125		22.9	667
42		11.56	586	84		14.67	517	126		26.45	712

Data summary		
n	126	126
max	26.45	773.5
μ	15.5	565.5
min	4.97	422.4
σ	4.2	82.8
COV = σ/μ (%)	27.3	14.6

is criticised as being crude as it is not a mechanised test. However, the thread-rolling test was reported to be repeatable to ± 3% by Sherwood and Ryley [24] when performing a benchmarking study across many laboratories in the United Kingdom. As previously mentioned, it is reported to be a difficult test to perform on peat soils [13,14]. However, it must be stated that a unique value of undrained shear strength should not be assigned at the w_P [23,27,52]. Table 3 shows the individual results of the thread-

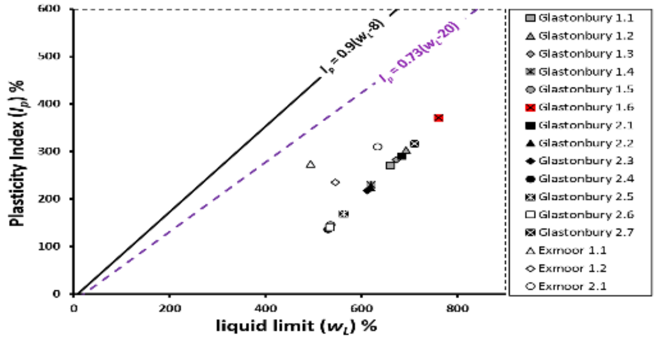

Fig. 1. Casagrande chart [53,54] showing the new data collected during this study.

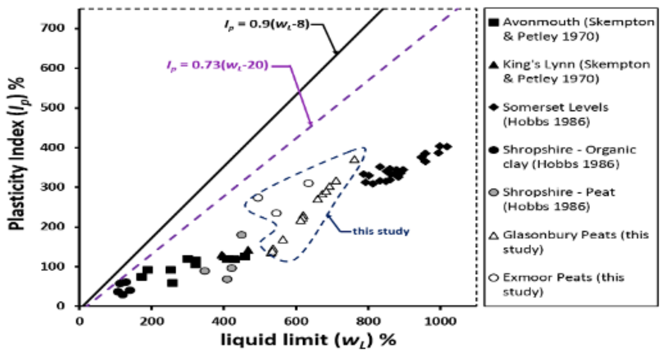

Fig. 2. Data collected in this study compared with data from some other UK peats (Avonmouth and King's Lynn data from Skempton and Petley [13]; Shropshire Data digitised from Figure 18 in Hobbs [14] where Hobbs cites Soil Mechanics Ltd 1983 as the original source of the data; Somerset Levels Data also digitised from Figure 18 in Hobbs [14] where Hobbs cites Petley 1983 (pers. comm.) as the original source of the data).

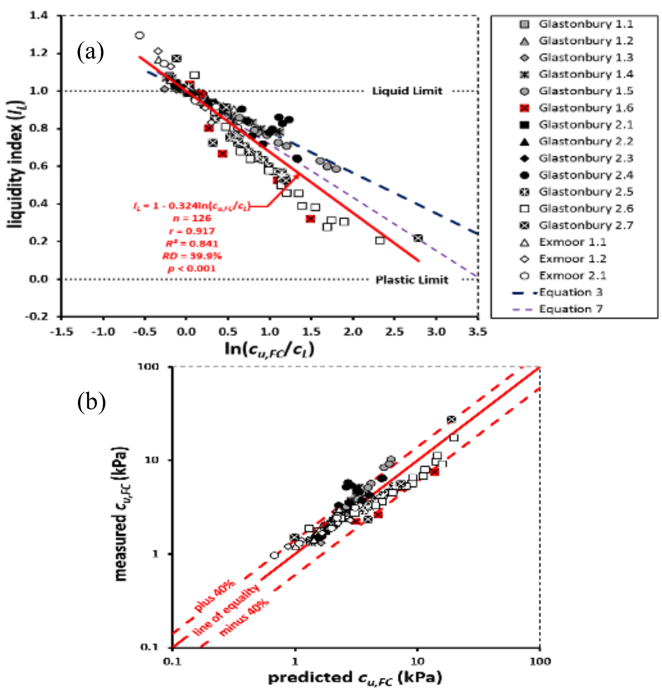

Fig. 3. (a) Plot of I_L versus natural logarithm of normalised fall cone undrained shear strength $\ln(c_{u,FC}/c_L)$ and (b) Measured $c_{u,FC}$ versus predicted $c_{u,FC}$ plot (using Eq. (11)).

Table 3
Individual thread rolling test results.

Site	w_P (%)	Site	w_P (%)
Glastonbury 1	344	Exmoor 1.1	213
	328		199
	394	Exmoor 1.2	303
	431		352
	385		348
	432		315
	413		264
Glastonbury 2	345		295
	410		297
	404	Exmoor 2.1	344
	404		352
	406		318
Exmoor 1.1	241		306
	230		355
	220		307
	224		333
	231		280
	200		

rolling testing carried out for the sites used in this study. The average values of w_P from Table 1 are used in the subsequent analyses presented in this paper.

3.4. Classification using the Casagrande Approach

Fig. 1 shows the peat soil materials in this study (n=16) plotted on the Casagrande-style chart [53]. The Equations of the A-line and U-lines are also shown (Eq. (10)) [54]. (Some recent research suggests that the relationship between the plasticity index (I_p) and w_L is non-linear, [55]). Fig. 2 shows the data shown on Fig. 1 compared with the results from other studies from peat soils from the UK.

$$I_P(\%) = 0.73(w_L - 20) \quad (10a)$$

$$I_P(\%) = 0.9(w_L - 8) \quad (10b)$$

3.5. Loss on Ignition

LOI was determined in accordance [56]. The summary results from this testing are also given in Table 1. The LOI values are above 80% and are higher than that reported by [13] for the Avonmouth (range 26.1 to 74.7%), and King's Lynn (range 74.2 to 82.0%) peat soils.

4. Results

4.1. Correlation of normalized c_u with I_L

The regression shown in Fig. 3(a) is given as Eq. (11) with the relevant statistical measures given in square brackets:

$$I_L = 1 - 0.324\ln(c_{u,FC}/c_L)$$
$$[n = 126, r = 0.917, R^2 = 0.841, RD = 39.9\%, p < 0.001] \quad (11)$$

where r = correlation coefficient; R^2 = coefficient of determination; RD = relative deviation (defined in [57]) and p = p-value.

Eq. (11) was adjusted so that the c_u at I_L = 1.0 corresponds to 1.7 kPa, the loss of R^2 when this condition is imposed is observed in the third decimal. Eq. (11) can be rearranged as Eq. (12):

$$\frac{c_{u,FC}}{c_L} \approx 21.9^{(1-I_L)} \qquad 0.2 < I_L < 1.2 \qquad (12)$$

where c_L = 1.7 kPa

The computed value of $R_{MW} \approx 22$, is lower than the value reported in Vardanega and Haigh [16] for clays and silts (cf. Eq. (7) in this paper). The R_{MW} value is dependent on the accurate determination of the w_P which is a challenge for natural peat soils but is reasonable to determine w_P for the soil component of said soils. It is recommended to use Eq. (12) in the range $0.2 < I_L < 1.2$ as this is the approximate range of data available. Additionally, the R_{MW} value in Eq. (12) is lower than the commonly used value of 100 [15,34,35]. Examination of Fig. 3(b) shows that most of the data fits within ± 40% prediction bounds (i.e. 112 out of 126 data-points or 89%) which are a similar level of accuracy to that obtained using the database of Vardanega and Haigh (±50%)[16].

4.2. Correlation of normalised c_u with I_{LN}

The regression shown in Fig. 4(a) is given as Eq. (13) with the relevant statistical measures given in square brackets:

$$I_{LN} = 1 - 0.292\ln(c_{u,FC}/c_L)$$

$$[n = 126,\ r = 0.917,\ R^2 = 0.840,\ RD = 40.0\%,\ p < 0.001] \qquad (13)$$

Eq. (13) has been adjusted so that the c_u at I_{LN} = 1.0 corresponds to 1.7 kPa, the loss of R^2 when this condition is imposed is observed in the third decimal. Eq. (13) can be expressed as Eq. (14):

$$\frac{c_{u,FC}}{c_L} \approx 30.7^{(1-I_{LN})} \qquad 0.2 < I_{LN} < 1.2 \qquad (14)$$

where c_L = 1.7 kPa

The computed value of $R_{MW} \approx 31$ is lower than the value computed using the database of Vardanega and Haigh [16] of 83.5 (cf. Eq. (8) in this paper). As with Eq. (12), it is recommended to use Eq. (14) in the range $0.2 < I_L < 1.2$ as this is the approximate range of data available. Examination of Fig. 4(b) shows that most of the data also fits within ± 40% prediction bounds (i.e. 113 out of 126 data-points or 90%) which are a similar level of accuracy to that obtained using the database in Vardanega and Haigh (±50%) [16].

4.3. Correlation of normalised c_u with w/w_L

The regression shown on Fig. 5(a) is given as Eq. (15) with the relevant statistical measures given in square brackets:

$$\frac{w}{w_L} = 1 - 0.102\ln(c_{u,FC}/c_L)$$

$$[n = 126,\ r = 0.857,\ R^2 = 0.734,\ RD = 51.6\%,\ p < 0.001] \qquad (15)$$

Eq. (15) has been adjusted so that the c_u at w/w_L = 1.0 corresponds to 1.7 kPa, the loss of R^2 when this condition is imposed is observed in the third decimal. Examination of Fig. 5(b) shows that most of the data also fits within ± 40% prediction bounds (i.e. 107 out of 126 data-points or 85%) which is a similar level of accuracy reported for the database in Vardanega and Haigh [16] (±50%), but the quality of fit is not as good as those seen in Figs. 3(b) and 4(b): note the lower R^2 value and the increased dispersion of the data [17,18]. However, Eq. (15) does not require the determination of the thread rolling plastic limit and therefore may be a more useful way to model changes with the water content of natural peats.

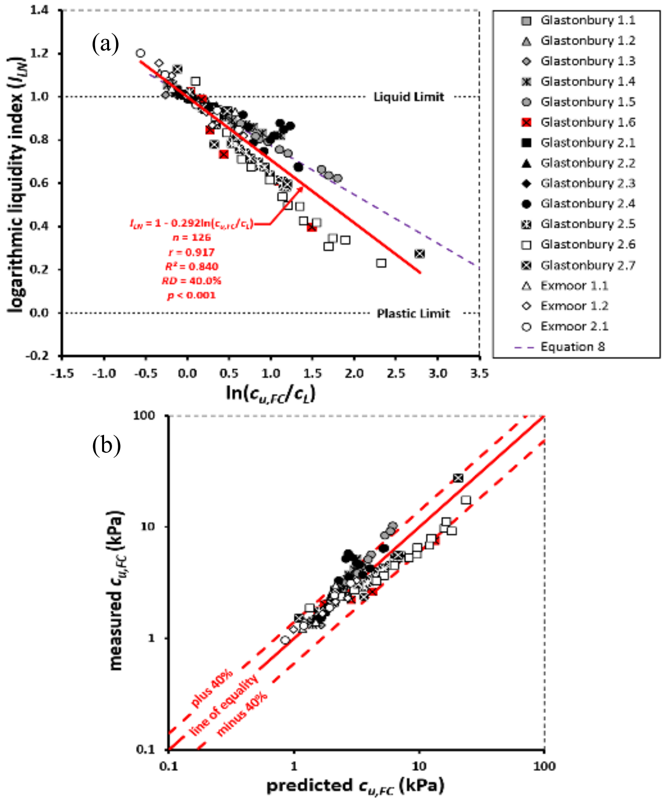

Fig. 4. (a) Plot of I_{LN} versus natural logarithm of normalised fall cone undrained shear strength $\ln(c_{u,FC}/c_L)$ and (b) Measured $c_{u,FC}$ versus predicted $c_{u,FC}$ plot (using Eq. (13)).

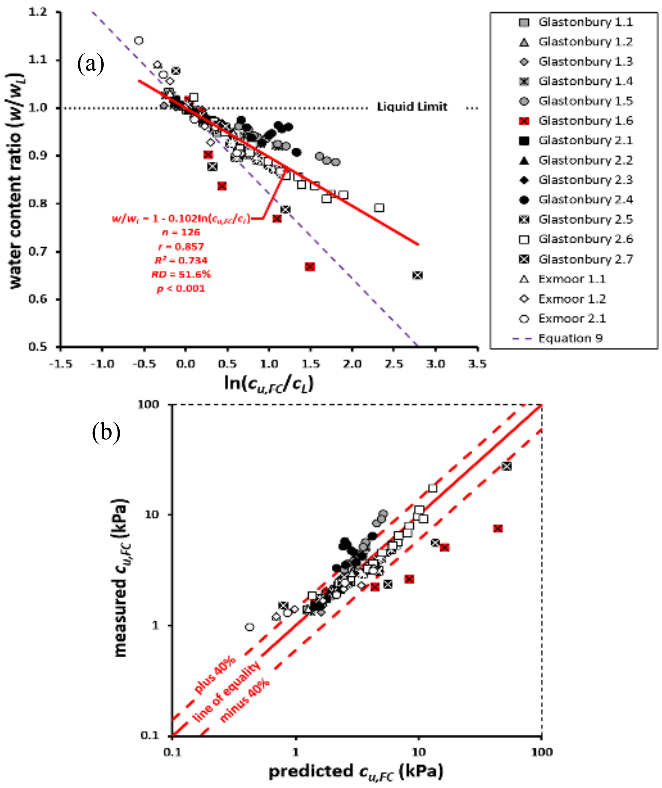

Fig. 5. (a) Plot of w/w_L versus natural logarithm of normalised fall cone undrained shear strength $\ln(c_{u,FC}/c_L)$ and (b) Measured $c_{u,FC}$ versus predicted $c_{u,FC}$ plot (using Eq. (15)).

5. Discussion and conclusions

The advantage of using the I_L (or I_{LN}) is that an R_{MW} value can be calculated. R_{MW} is a useful parameter for engineers as it allows approximation of the rate of decrease of undrained shear strength that may be expected as water content increases for peat soils. However, it does require the determination of the w_P. If the w_P cannot be determined to acceptable accuracy when characterising a peat deposit then the w/w_L is also good predictor of undrained shear strength. However, the engineer using a correlation between undrained shear strength and w/w_L must accept that the proximity to the brittle transition (which is defined by the thread rolling test not a specific strength) will not be captured [18].

This paper has presented the results of some laboratory characterisation of two English peat soils, focussing on the soil index properties and the variation of $c_{u,FC}$ with changing I_L or w/w_L. The following concluding points are made:

1. By examination of the Casagrande chart, the Atterberg limits obtained for the tested peat soils fit within the bounds suggested by previous testing on English peat soils (although the degree of scatter is high);
2. The LOI for these soils is high compared with historical data from Avonmouth and King's Lynn;
3. The R_{MW} value for the data-set presented in this paper was computed to be about 22 when the I_L is correlated with normalised $c_{u,FC}$ and 31 when the I_{LN} is correlated with normalised $c_{u,FC}$. It is reiterated that these computed values are dependent on the successful measurement of the w_P – a measurement that is difficult, even for these sieved peats. For the peat soils studied in this paper the R_{MW} values are lower than the average value computed by Vardanega and Haigh [16] using a large database of measurements of clays and silts;
4. The w/w_L was able to predict the $c_{u,FC}$ variation to ±40% 85% of the time which is not as accurate as using the I_L (or I_{LN}) which are accurate to ±40% 89% and 90% of the time respectively. However, the use of w/w_L may be a more appropriate method to capture the undrained shear strength variation of natural peat soils with changing water content.

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