Cite this article
O'Kelly BC (2024)
Theory of liquid and plastic limits for fine soils, methods of determination and outlook.
Geotechnical Research **11(1)**: 43–61,
https://doi.org/10.1680/jgere.23.00038

Research Article
Paper 2300038
Received 01/07/2023; Accepted 27/11/2023
First published online 04/12/2023

Geotechnical Research

emerald PUBLISHING ice Publishing

Theory of liquid and plastic limits for fine soils, methods of determination and outlook

Brendan C. O'Kelly PhD, FTCD
Associate Professor, Department of Civil, Structural and Environmental Engineering, Trinity College Dublin, Dublin, Ireland (Orcid:0000-0002-1343-4428) (bokelly@tcd.ie)

Albert Atterberg introduced various consistency limits (state transitions) for fine-grained soil in the 1910s. Of these, the liquid limit (LL) and plastic limit (PL) are ubiquitous in geotechnical engineering practice, including their usage for soil classification and in deducing useful geotechnical parameters through correlations. Given that it is about 110 years since they were first introduced, it seems timely to review critically the current state of play regarding various definitions and theories of these index parameters and their standardised testing methods, as described in majorly used codes worldwide. Because different codes allow different LL apparatus types and employ some dissimilar criteria (e.g. in establishing the end point for the PL test), a change in method or code may produce different consistency limits results for testing the same soil. These differences are rationalised in terms of the controlling soil and test parameters. Some potential pitfalls in consistency limits testing are highlighted. Attention then turns to strength-based approaches, mostly employing fall-cone (FC) set-ups, emphasising their unsuitability for determining Atterberg's PL. Considering the general poor reproducibility of the PL test, this paper concludes with an alternative way forward, obtaining useful FC index parameters that provide new possibilities for strength predictions and in the classification of fine-grained soils.

Keywords: characterisation techniques/codes of practice & standards/penetrometers/plasticity/soil classification/testing, apparatus & methods

Notation

d	cone penetration depth (mm)
d_L	cone penetration depth at the fall-cone liquid limit (mm)
h_d	free-fall height before the cone tip contacts the surface of the test specimen (mm)
$I_{C(FC)}$	fall-cone consistency index parameter (dimensionless)
$I_{F(FC)}$	semi-logarithmic fall-cone flow index (%)
$I_{F(PC)}$	semi-logarithmic percussion-cup flow index (%)
I_L	liquidity index (= $(w - w_P)/I_P$) (dimensionless)
I_P	plasticity index (= $w_L - w_P$) (%)
K	cone factor (contacting cone) (dimensionless)
K_d	cone factor accounting for the free-fall height h_d of the cone (dimensionless)
I_{P100}	numeric difference between $w_{L(FC)}$ and w_{P100} (%)
LL_{FC}	fall-cone liquid limit
LL_{PC}	percussion-cup liquid limit
N	number of bumps (for the percussion cup) (dimensionless)
n	number of observations or data points (dimensionless)
PL_{25}	plastic strength limit, defined for $s_u = 25 \times s_{u\text{-}L(FC)}$ (%)
PL_{100}	plastic strength limit, defined for $s_u = 100 \times s_{u\text{-}L(FC)}$ (%)
R^2	coefficient of determination (dimensionless)
S_S	specific strength (= $s_{u\text{-}L(PC)}/\rho_{sat}$) ($m^2/s^2$)
s_u	saturated remoulded undrained shear strength (kPa)
$s_{u\text{-}FC}$	saturated remoulded fall-cone-derived undrained shear strength (kPa)
$s_{u\text{-}L(FC)}$	saturated remoulded undrained shear strength mobilised at the fall-cone liquid limit (kPa)
$s_{u\text{-}L(PC)}$	saturated remoulded undrained shear strength mobilised at the percussion-cup liquid limit (kPa)
$s_{u\text{-}P}$	saturated remoulded undrained shear strength mobilised at Atterberg's plastic limit (kPa)
W	cone weight (N)
w	water content (%)
w_L	water content at the liquid limit (%)
$w_{L(FC)}$	water content at the fall-cone liquid limit (%)
$w_{L(PC)}$	water content at the percussion-cup liquid limit (%)
w_P	water content at Atterberg's plastic limit (%)
w_{P25}	water content corresponding to the PL_{25} parameter (%)
w_{P100}	water content corresponding to the PL_{100} parameter (%)
$\dot{\gamma}$	shear strain rate (%/h)
μ	strain-rate dependence of undrained shear strength (dimensionless)
ρ_{sat}	saturated bulk density (kg/m^3)
σ	standard deviation (dimensionless)

Introduction

Albert Atterberg (1911a, 1911b) first introduced the 'limits of consistency' state transitions for fine-grained soil in the 1910s. With reducing water content (w), Atterberg defined (*a*) the upper limit of viscous flow; (*b*) the liquid limit (LL) (his 'flow limit' or upper plastic limit); (*c*) the sticky (or adhesion) limit; (*d*) the plastic limit (PL) (his 'roll-out limit'); (*e*) the cohesion limit; and (*f*) the shrinkage limit (ASTM, 2017). In current geotechnical engineering usage, the term 'consistency limits' usually refers only to the LL and PL state transitions, but in some cases, it can also include the shrinkage limit (ASTM, 2017). The water contents associated with the LL and PL (i.e. w_L and w_P, respectively) define the plastic range of the investigated fine-grained soil (i.e. understood as comprising soil particles passing a 425 μm sieve size). The LL and PL have wide importance for civil/geotechnical engineering applications, including those in soil classification and for deducing the values of useful geotechnical parameters (e.g. compaction, California bearing ratio (CBR), hydraulic conductivity, swelling potential and pressure, consolidation and shear strength) through a myriad of empirical correlations built up over the decades (e.g. see O'Kelly *et al*., 2018). Atterberg's original LL flow test and manual (hand) thread-rolling PL test were subsequently standardised for use in civil engineering applications by Terzaghi (1926a, 1926b), Wintermeyer (1926) and Casagrande (1932, 1958). Given that it is 112 years since their first introduction, it seems timely to review critically the current state of play regarding the various definitions of these soil index parameters (herein focusing on the LL and PL) and their standardised testing methods, as described in several globally well-known standards for soil geotechnical tests (i.e. standards issued by the American Association of State Highway and Transportation Officials (Aashto), American Society for Testing and Materials (ASTM) and British Standards Institution (BSI) and also the European standards (EN)).

In engineering science, plasticity (or plastic behaviour) is understood as the material property that enables significant deformation to occur, without fracture, on the application of sufficiently high stresses (i.e. exceeding the yield stress of the material), along with the substantive retention of the deformed material shape on removal of those stresses. In the case of inorganic fine-grained soil, plasticity is associated with its clay fraction, with the plastic soil exhibiting a range of water contents over which it displays plastic behaviour and retaining its shape on drying (ASTM, 2006). Atterberg (1911a, 1911b) gave his definition of plasticity for fine-grained soils as their ability to be rolled out into threads, whereas non-plastic (NP) inorganic fine-grained soil, being devoid of clay mineral content, does not have any water content range over which it exhibits plasticity. The PL is used together with the LL to determine the plasticity index I_P (= $w_L - w_P$) — that is, the magnitude of the water content range over which the fine soil exhibits plastic/ductile behaviour. A fine-grained soil is NP if it has an I_P of zero (i.e. $w_P = w_L$) or if its w_P cannot be determined by the thread-rolling PL test (BSI, 1990a, 2018a). Plotting I_P against w_L in the Casagrande plasticity chart, the LL and PL are extensively employed for the classification of fine-grained soils (as reviewed in the paper by Moreno-Maroto *et al*. (2021)) — for instance, in the specification of soils for road construction and for other applications in the broader construction industry.

As elaborated below, the aims of this paper are to examine critically (*a*) the definitions of the LL and PL parameters appearing in several globally well-known standards for soil geotechnical tests; (*b*) the fundamental basis for these index parameters; and (*c*) the standardised testing methods, as well as other suitable testing approaches, employed for their determination. Some potential pitfalls in consistency limits testing are highlighted (e.g. inadequate maturing of the prepared fine-grained test material before performing the LL testing) and various issues associated with the testing of some unconventional geomaterials are described.

Two main testing approaches (i.e. percussion-cup (PC) and fall-cone (FC)) are used for LL determination, with various PC LL (i.e. LL_{PC}) apparatuses specified in the different codes having some significant differences (Haigh, 2012, 2016). Hence, in testing the same fine-grained soil, a change in LL method or code may produce different w_L values (O'Kelly *et al*., 2018). In the present paper, these differences are rationalised in terms of the controlling parameter of saturated remoulded undrained shear strength (s_u) that is mobilised at the associated w_L. Also investigated in the present paper are differences in some details prescribed for the standardised manual thread-rolling PL method between the majorly used codes, potentially the most significant of which relate to the manifestations of the specified end point and how they specifically relate to the plastic/ductile–brittle state transition. The handful of rolling device PL methods, as well as other proposed methods for which the soil deformational response is the discerning factor, are also critically reviewed. Attention then turns to the so-called strength-based approaches, mostly using various FC set-ups, and their unsuitability for determining Atterberg's PL, because they cannot discern the onset of brittleness.

Considering the general poor reproducibility of the PL test, this paper concludes with presenting an alternative way forward that involves obtaining various useful strength-based FC index parameters, providing new possibilities for s_u predictions and in the classification of fine-grained soils.

Liquid limit

ASTM D 653 (ASTM, 2006: p. 18), 'Standard terminology relating to soil, rock, and contained fluids', defines the LL as 'the water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil'. ASTM D 4318 (ASTM, 2017: p. 2) defines the LL as 'the water content, in percent, of a soil at the arbitrarily defined boundary between the semi-liquid and plastic states', while the British standard (BS) BS 1377-2 (BSI, 1990b: p. 11) describes the LL as 'the empirically established moisture content at which a soil passes from the liquid state to the plastic state'. On these bases, suitable methods for LL determination would associate it with a threshold viscous resistance or a small arbitrarily chosen value of s_u on the continuum of ever-weakening behaviour with increasing water content (O'Kelly *et al*., 2018). ASTM D 4318 (ASTM, 2017), for instance, considers the value of s_u for fine-grained soils at their (Casagrande) w_L to be ~2 kPa. As elucidated by Sharma (2012), fine-grained soils do not pass abruptly from one state to another (here referring to the LL state transition), so that drawing up of a limit in such a transition phase must, of necessity, involve an element of arbitrariness, hence the inclusion of terms such as 'arbitrary limit' and 'arbitrarily defined boundary' in the ASTM D 653 (ASTM, 2006) and ASTM D 4318 (ASTM, 2017) terminologies for LL. However, rather than the 'arbitrary' nature, the author of the present paper would suggest a more nuanced definition of LL, since, for instance, remoulded fine-grained soil at its w_L would not be expected to mobilise a superhigh s_u value of 12 kPa (O'Kelly, 2019a) but rather a much smaller s_u in the range of approximately 1–3 kPa (Federico, 1983; Haigh, 2012; O'Kelly, 2019a; Youssef *et al*., 1965).

The w_L magnitude is strongly dependent on the gradation, composition and mineralogical properties (of the clay fraction) of the fine-grained soil and also on the quantity of interlayer water in the case of expanding clay minerals, such as montmorillonite (Dolinar and Trauner, 2004; Trauner *et al*., 2005; Wood, 1990). In practice, because of the inclusion of 'arbitrary' in the terminologies reported for the LL state transition in the different codes, the w_L value is defined in terms of the experimental method/criteria prescribed by a testing standard used for obtaining it (O'Kelly *et al*., 2018). In other words, the codes do not explicitly state the precise value of s_u for obtaining w_L; rather, they define the experimental approach (method and specific set of criteria) to be followed for its determination. For instance, BS 1377-1 (BSI, 1990a), ASTM D 4318 (ASTM, 2017) and BS EN ISO 14688-2 (BSI, 2018a) define the LL as the water content at which a fine-grained soil passes from the liquid to the plastic state, as determined by standardised LL tests. In the case of the BS (i.e. BSI, 1990b), two main types of LL test were specified: the 80 g/30° FC (cone penetrometer) method, being an indentation test, and the much earlier Casagrande (PC) type of test, which imposes shock loading to the soil test specimen (O'Kelly *et al*., 2018). For both types of test, BS 1377-2 (BSI, 1990b) allowed an alternative rapid 'one-point' method – that is, the LL one-point FC method and the LL one-point PC method – which may give less accurate results. Note that the BS 1377-2 code has been superseded, with the current standardised methods of LL and PL testing presented in BS EN ISO 14688-2 (BSI, 2018b). The multipoint and one-point FC LL (i.e. LL_{FC}) methods are preferred to those employing the PC device for the reasons of greater repeatability and reproducibility (BSI, 1990b; Sherwood and Ryley, 1970). In other words, when compared with the test using the PC-type device, the FC test is easier to perform and is less dependent on the operator's skill and judgement, with the FC apparatus also being simpler to maintain in correct adjustment with the codes (Haigh, 2016). However, satisfactory results can be obtained for the PC approach provided that care is taken to ensure that the PC device is correctly maintained and that the LL_{PC} test procedure is strictly adhered to (BSI, 1990b). In the present global context, both the FC and PC approaches are generally considered equally valid means for LL determination (O'Kelly *et al*., 2022a), although in many jurisdictions, one approach is often favoured over the other. Note that in the Aashto and ASTM standards, only the PC (i.e. not the FC) test procedure is specified for LL determination.

A brief description of the standard PC type of test is presented here, as context to the discussion that follows. For this test, four or more water contents are investigated over the range N = 10–50 bumps of the standard brass cup containing the soil paste specimen, to cause closure over a specified length of a preformed groove in the specimen (ASTM, 2017; BSI, 1990b, 2018a). In preparing the fine-grained soil specimen for LL_{PC} testing, the said groove is formed by drawing a standard grooving tool through the placed soil specimen on a line joining the highest point to the lowest point on the rim of the cup, thereby dividing the specimen into two separated 'wedges' of soil. Each bump impact (blow) is produced by allowing the cup to fall through a standard 10 mm height of drop and impact on the apparatus base. Depending on the adopted code, the base material may be broadly categorised as 'soft' (e.g. BSI, 1990b) or 'hard' (e.g. ASTM, 2017)) and the difference between them may have a significant influence on the obtained w_L results (O'Kelly and Soltani, 2022b; O'Kelly *et al*., 2018). Furthermore, while they are often distinguished as being soft- or hard-base PC devices, considerable variability exists within both categories between the various codes (Haigh, 2016). Despite being otherwise identical PC devices, compared to those devices with soft-base material, their hard-base counterparts produce lower w_L results, because of the greater energy release of the falling cup impacting on the apparatus base to cause closure of the preformed groove (Haigh, 2016; Sridharan and Prakash, 2000). With the water content as ordinate on a linear scale and the number of bumps (N) as abscissa on a common logarithmic scale, the best straight line is fitted to the four or more w:N data points obtained for the tested soil. The LL_{PC} water content (i.e. $w_{L(PC)}$), expressed to the nearest integer value, corresponds to N = 25 bumps (ASTM, 2017; BSI, 1990b, 2018a). Considering the importance of the cup drop height (of 10 mm) to achieve correct measurements, frequent checking throughout the LL testing procedure is important, as is it ensuring that the values of base hardness and resilience are standard between PC devices at their manufacturing and that they remain so through their working life (Haigh, 2016; O'Kelly *et al*., 2018). The criteria of LL for the PC type of test have been analysed as an undrained slope stability problem (Haigh, 2012, 2016), with the flow movement together of the two initially separated soil wedges in the brass cup essentially a dynamic slope stability test (Wroth, 1979). Employing a particular PC type (i.e. for either soft- or hard-base material), with a defined number of bumps (i.e. N = 25) assigned to cause slope failure and closure of the preformed groove over a specified length, the investigated fine-grained soil can be deemed to mobilise an approximately fixed s_u (i.e. $s_{u\text{-}L(PC)}$) at the LL_{PC} condition. Here, however, one must also consider that the soil's saturated bulk density (ρ_{sat}) at the LL condition reduces with increasing value of w_L, such that the undrained slope analysis requires that the value of s_u mobilised at the $w_{L(PC)}$ progressively reduces with increasing value of w_L (identified experimentally by Youssef *et al*. (1965)). For instance, $s_{u\text{-}L(PC)}$ typically decreases from ~2.5 to 1.6 kPa for $w_{L(PC)}$ increasing from 20 to 70% (O'Kelly, 2019a). Taking into account the change in soil density with varying water content, it can be shown that for testing of a wide range of inorganic fine-grained soils, the value of the specific strength ($S_S = s_{u\text{-}L(PC)}/\rho_{sat}$)) is constant, being, on average, equal to 0.86 and 0.47 m^2/s^2 for standard hard- and soft-base type PCs, respectively (Haigh, 2016). Again, this outcome (i.e. in terms of S_S) is consistent with hard-base PC-type devices producing a lower w_L for a given fine-grained soil compared with that of their soft-base counterparts.

For the LL_{FC} method, a range of cone penetration depth (d) values are investigated in four or more test runs covering a range of water contents either side of this state transition. In the case of using the 80 g/30° cone (BSI, 1990b, 2018a), a range of $d \approx$ 15–25 mm is investigated, with the value of d measured after a 5 s period following the cone's release to penetrate into the soil test specimen. From the best-fitting line to the four or more w:d data points (presented on linear scales), the LL_{FC} water content (i.e. $w_{L(FC)}$) of the tested fine-grained soil is obtained for d_L = 20 mm and expressed to the nearest integer (BSI, 1990b, 2018a). Being originally developed as an undrained strength test, in its adaptation for LL determination and incorporation into the BSI codes during the 1970s, the LL_{FC} approach was calibrated to produce essentially w_L results equivalent to those obtained using the earlier LL_{PC} test of Casagrande (1932, 1948) (O'Kelly *et al*., 2018). In other words, using Equation 1 (after Hansbo, 1957) and assuming an s_u value at w_L (i.e. $s_{u\text{-}L(FC)}$) of ~1.7 kPa, the LL_{FC} condition can be defined in terms of a certain penetration depth (i.e. d_L) of a standardised cone, with specified characteristics of cone weight, apex angle and surface roughness.

1. $$s_{u\text{-FC}}\ (\text{in kPa}) = \frac{KW}{d^2} \times 10^3$$

where K is the cone factor; W is the cone weight (N); and d is the cone penetration depth (mm).

Note that the $s_{u\text{-}L(FC)}$ value of ~1.7 kPa was deduced by Wroth and Wood (1978) based on synthesis of a large database of $s_{u\text{-}L(PC)}$ data compiled for various remoulded fine-grained soils. Accordingly, from the FC theory (i.e. based on Equation 1), various LL_{FC} definitions (i.e. employing several combinations of cone weight and apex angle and for different values of d_L) could be employed provided that they would result in mobilising approximately the same value of $s_{u\text{-}L(FC)}$. For instance, BS EN ISO 17892-12 (BSI, 2018b) allows the determination of LL_{FC} using 80 g/30° and 60 g/60° cones, with associated d_L of 20 and 10 mm, respectively. Note that, referring to Equation 1, the determination of the values of $s_{u\text{-}L(FC)}$ associated with these different LL_{FC} criteria is critically dependent on the empirical cone factor (K). The value of K for a given cone set-up is usually back-calculated (i.e. calibrated) relative to direct strength measurements, typically obtained using miniature vane-shear tests or less frequently using unconsolidated-undrained (UU) triaxial compression tests (this aspect is reviewed by O'Kelly (2019a)). For instance, O'Kelly (2014) presented an example case of deducing the experimental value of K for the BS 30° cone based on the results of both strength test methods performed over a wide range of water contents for an organic clay. Essentially, the value of K depends on the cone apex angle and surface roughness (Houlsby, 1982; Koumoto and Houlsby, 2001), and it is somewhat dependent on the strain rate dependence of undrained strength (μ) for the investigated fine-grained soil (O'Kelly, 2018; O'Kelly *et al*., 2018). The μ parameter gives the proportional variation in undrained shear strength for one log-cycle change in the shear strain rate ($\dot{\gamma}$), with the value of K (and hence s_u) decreasing for increasing value of the μ parameter (O'Kelly, 2018). As elaborated later in the section headed 'Way forward?', with the assumed value of $s_{u\text{-}L(FC)} \approx$ 1.7 kPa pertaining to $\mu \approx 0.10$ and a range of $\mu = 0.10 \pm 0.05$ (Ladd and Foott, 1974) not uncommon when considering a variety of inorganic fine-grained soils, this means that the value of $s_{u\text{-}L(FC)}$ for the 80 g/30° FC set-up, with d_L = 20 mm (BSI, 1990b, 2018b), could plausibly range between 1.6 and 2.4 kPa (O'Kelly *et al*., 2018). This arises because the average shear strain rate for the 30° cone penetrating into the soil specimen of $\dot{\gamma} \approx 1.0 \times 10^6\ \%/\text{h}$ for d = 15–25 mm (Koumoto and Houlsby, 2001) is approximately four orders of magnitude faster compared with that generally employed, for instance, in performing a typical standard UU triaxial test. Another point worth mentioning is that the FC approach always produces an experimental flow curve (i.e. even for NP soil), from which the FC flow index, defined by Sridharan *et al*. (1999) as $I_{F(FC)} = \Delta w/\Delta \log_{10} d$, and other strength-based index parameters that are described later in this paper can be computed. In contrast, for Casagrande LL testing of NP soils and some soils of low plasticity (Sherwood and Ryley, 1970; Sivapullaiah and Sridharan, 1985; Sridharan *et al*., 1999), after several trials at successively higher water contents, the two initially separated soil wedges of the prepared test specimen continued to slide together on the surface of the brass cup (rather than soil flow movement occurring). The number of drops (N) required to close the groove for investigating these soils was always less than 25 (ASTM, 2017). Consequently, in the case of NP soil, the PC flow curve and hence the associated flow index $I_{F(PC)}$ (=$\Delta w/\Delta \log_{10} N$) could not be obtained.

With the LL_{PC} and LL_{FC} criteria defined on the basis of S_S (Haigh, 2012, 2016) and s_u values (O'Kelly *et al*., 2018), respectively, and considering the choices of standard FCs with different cone characteristics and of soft- and hard-base PC types available, it is inevitable that the w_L results obtained for testing the same fine-grained soil may differ (O'Kelly *et al*., 2018). This point has been demonstrated experimentally, in comparing, for instance, the results of ASTM $w_{L(PC)}$ against BS $w_{L(FC)}$ (Belviso *et al*., 1985; Dragoni *et al*., 2008; Özer, 2009; Sampson and Netterberg, 1985) and of BS LL_{PC} against BS LL_{FC} (Budhu, 1985; Dragoni *et al*., 2008; Feng, 2000; Özer, 2009; Prakash and Sridharan, 2004; Sherwood and Ryley, 1970; Sridharan *et al*., 1999) for diverse ranges of fine-grained soils investigated.

Furthermore, in investigating a wide variety of fine-grained soils, with an $s_{u\text{-}L(FC)}$ of approximately constant magnitude (of 1.7 kPa) being mobilised using a given cone weight, apex angle and d_L combination, with increasing w_L, the $s_{u\text{-}L(PC)}$-to-$s_{u\text{-}L(FC)}$ ratio decreases and, consequently, the $w_{L(PC)}$-to-$w_{L(FC)}$ ratio increases (O'Kelly, 2021). According to BS 1377-2 (BSI, 1990b), the difference between the soft-base BS $w_{L(PC)}$ and the 80 g/30° $w_{L(FC)}$ obtained for a given fine-grained soil is generally not significant for w_L < 100% and is less than the normal variation likely to be obtained using the PC device. Whereas, the hard-base ASTM $w_{L(PC)}$ and the 80 g/30° $w_{L(FC)}$ obtained for the same fine-grained soil are broadly comparable with w_L < 50% (Budhu, 1985) or <60% (Prakash and Sridharan, 2006). It is worth mentioning that the resulting changes in the measured w_L and hence I_P (= $w_L - w_P$) due to a switch in the LL method employed do not represent a fundamental alteration in the material (mechanical) behaviour of the investigated soil (O'Kelly *et al.*, 2018). Rather, these changes are simply related to distinct differences in the LL measurement approaches. For specific engineering applications, in some instances, the switch in LL method (e.g. from using PC to FC devices) may be sufficient to cause a change in the classification of fine soils from suitable to unsuitable materials (or vice versa) (Di Matteo *et al.*, 2016). This scenario is more likely to arise for those fine-gained soils plotting close to the A-line boundary (distinguishing between clay- and silt-type soils) in the plasticity chart. The switch in the LL method may also cause a shift to occur in the plasticity level class, with the various plasticity level classes defined in the codes for prescribed LL ranges (e.g. see BSI, 2018a) – that is, the switch in LL method could result in small but potentially significant changes occurring in both the ordinate and coordinate values for the soil when plotted in the plasticity chart. If deemed necessary (although generally not currently done in routine practice), numerous correlations reported in the literature could be employed to relate values of w_L deduced using the different LL measurement techniques and codes (e.g. see the review papers by O'Kelly *et al.* (2018), Shimobe and Spagnoli (2019) and O'Kelly and Soltani (2022b)). In applying these correlations, it is important to consider their LL calibration ranges and the types of (clay) mineralogy comprising the investigated soils – that is, different regression equations are obtained depending on the data set and correlation method used. For instance, from investigations of LL data for 368 different fine-grained soils reported in the literature, O'Kelly *et al.* (2018) produced power regression curves to relate the 80 g/30° BS $w_{L(FC)}$ to both the BS $w_{L(PC)}$ and ASTM $w_{L(PC)}$ results, considering separately the w_L domains of <120% (as shown in Figure 1) and up to 600%. Other researchers have opted to use linear regression in obtaining such correlations. However, with the $s_{u\text{-}L(PC)}$-to-$s_{u\text{-}L(FC)}$ ratio decreasing for increasing w_L (O'Kelly, 2021), the power regression approach is considered more appropriate when considering a wide w_L range.

Plastic limit

As a fundamental basis, Haigh *et al.* (2013) stated that the PL relates to the capillary suction at which the water phase ceases to act as a continuum, caused either by air entry or heterogeneous cavitation during the thread rolling-out procedure, leading to brittleness (see also the recent experimental work by Murray and Tarantino (2019), which supports this hypothesis). The air-entry value, the point of desaturation of soil drying from a fully saturated state, has been found to occur at a water content just above w_P (Cafaro, 2002; Haigh *et al.*, 2014; Marinho and Oliveira, 2012; Marinho and Pinto, 2000).

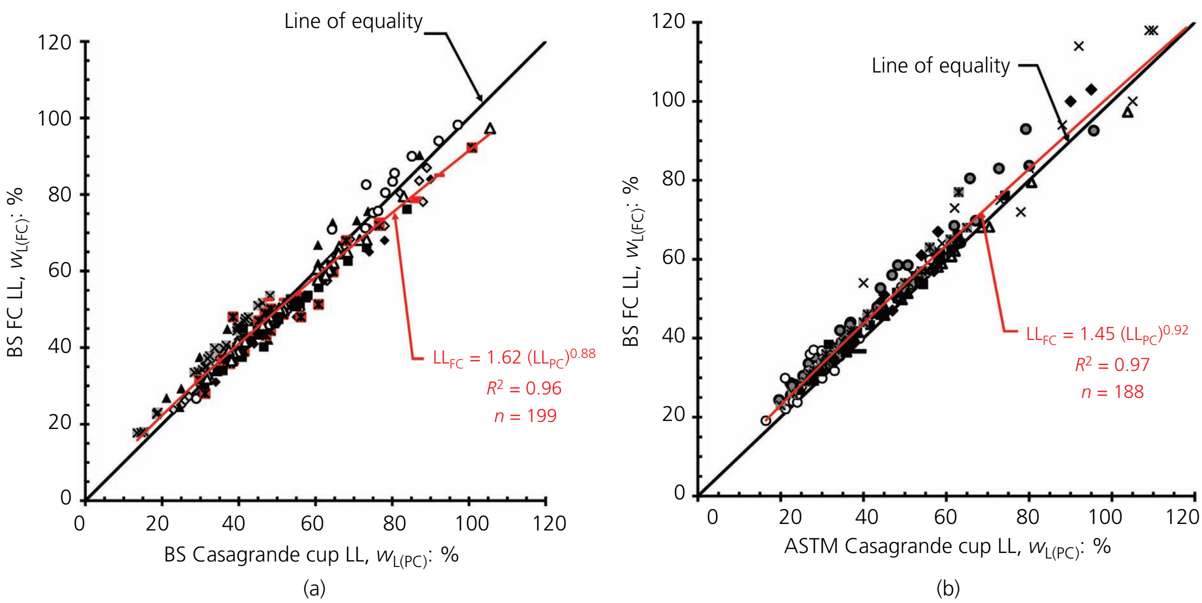


Figure 1. Correlations of FC LL with PC LL: (a) BS LL_{FC} plotted against BS LL_{PC} (BSI, 1990b); (b) BS LL_{FC} plotted against ASTM LL_{PC} (data of w_L < 120%) (adopted from O'Kelly *et al.* (2018))

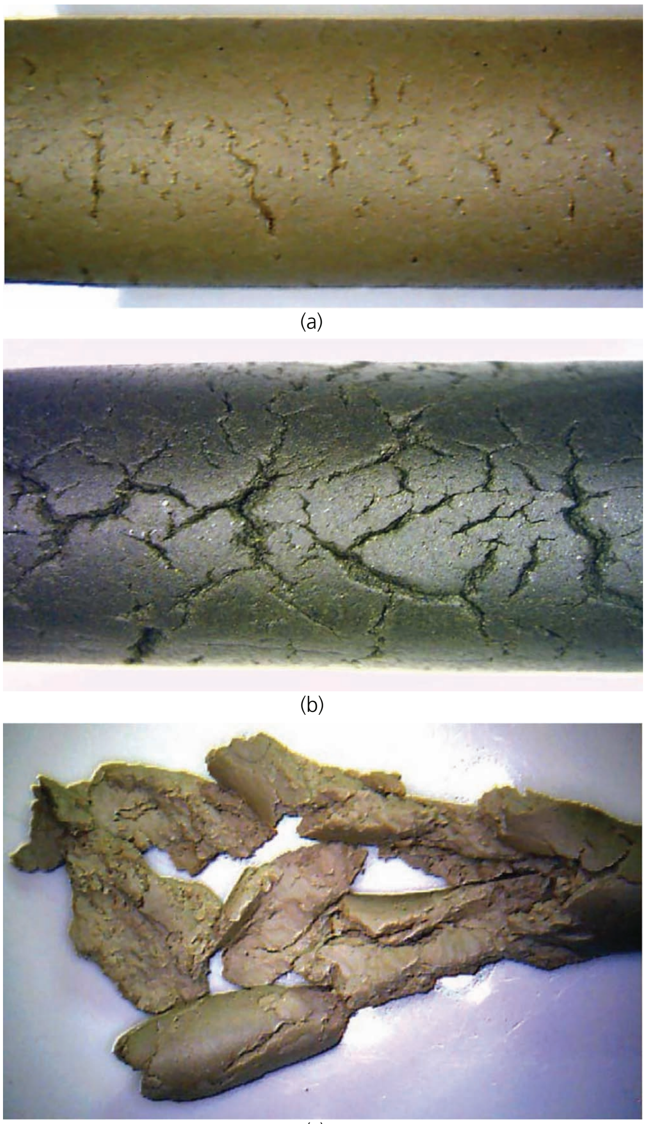
Largely following Atterberg's original procedure, the almost universally accepted method for PL determination is the 'rolling of threads' method (O'Kelly *et al*., 2018), in which the soil thread undergoes a complex process of compression, tension, torsion and bending (Barnes and O'Kelly, 2011). In relation to the majorly cited codes, BS 1377-2 (BSI, 1990b: p. 20), for instance, defines the PL state transition as 'the lowest moisture content at which the soil is plastic', as determined by the thread-rolling PL test. BS EN ISO 17892-12 (BSI, 2018b: p. 9) defines the PL as 'the water content at which a soil ceases to be plastic when dried further', as determined by the thread-rolling PL test. Aashto T90 (Aashto, 2000) defines the PL as the lowest water content determined in accordance with the thread-rolling PL test at which the soil remains plastic. Meanwhile ASTM D 4318 (ASTM, 2017: p. 2) terms the PL as 'the water content, in percent, of a soil at the boundary between the plastic and semi-solid states', with both the ASTM and Aashto standards providing the conventional manual (hand) rolling and alternative rolling-device (after Bobrowski and Griekspoor, 1992) PL test methods for its determination. Note that in §4 'Terminology' of the ASTM D 653 code (ASTM, 2006), two terms are used in describing the PL, one being the same as the above PL term reported in ASTM D 4318 (ASTM, 2017) and determined using the thread-rolling method. Whereas the second term could arguably be interpreted slightly differently, describing the PL as 'the water content corresponding to an arbitrary limit between the plastic and the semi solid states of consistency of a soil' (ASTM, 2006: p. 22) and not appearing to tie the PL state transition to the PL test. The potential ramifications of the latter/second term (interpretation) are discussed later in this section.

However, all four codes referenced in the preceding paragraph, as well as the wider soil mechanics/geotechnical engineering literatures, are consistent in that they define the PL as the lowest water content for which fine-grained soil behaves plastically, as exclusively assessed using the thread-rolling PL test. The one exception is the Chinese standards (see the test description reported in Vardanega *et al*. (2020)), which uses a 76 g/30° FC to define the 'PL' as the water content for d = 2 mm, as determined by extrapolation of the log d–log w flow curve. The implications of this deviation from the otherwise universally employed thread-rolling PL method are also discussed later in this section.

According to Haigh *et al*. (2013), the thread-rolling PL test is, in principle, actually well designed to investigate the plastic/ductile–brittle state transition, which is generally not the case for many of the various alternatives proposed to date (with the main ones being described later). For $w > w_P$, a thread of fine-grained soil will be plastic (ductile) and reduces in diameter while extruding longitudinally during the rolling-out process due to its plastic deformation response. Whereas, the soil will be brittle for $w < w_P$, which manifests in performing the PL test as crumbling of the soil thread when rolled out (BSI, 1990b). Obviously, to establish this state transition, the investigated fine-grained soil must be tested for water contents on the 'wet' side and (just) 'dry' side of w_P. With proper care exercised in performing the PL test, the continuous ductile response of the thread during its plastic elongation allows this point of brittle transition (identified as a definite change in deformational behaviour type, i.e. from plastic/ductile to brittle) to be observed (Barnes, 2009, 2013a, 2013b).

In performing the PL test, the soil thread is rolled between the fingers (from the fingertip to the second joint, of one hand) and the surface of the glass rolling plate, closely examining the thread condition to identify the onset of the brittle state – that is, the end point. According to BS 1377-2 (BSI, 1990b), enough hand (finger) pressure is applied to reduce the thread diameter from initially ~6 mm down to ~3 mm. in five to ten complete strokes (forward-and-back movements of the hand), maintaining a uniform rolling pressure. Some heavy clays require 10–15 strokes when the soil is near w_P because the soil hardens at this stage (BSI, 1990b). In other words, the amount of hand pressure required can vary greatly according to the remoulding toughness of the soil being tested, with the required pressure typically increasing with greater soil plasticity (Barnes, 2009, 2013a, 2013b; Casagrande, 1948; Moreno-Maroto and Alonso-Azcárate, 2018; O'Kelly *et al*., 2022b, 2023). Fundamentally, within the plastic range, saturated clayey materials develop suction, such that active/high-plasticity clays (characterised by the highest air-entry values) have the greater toughness. Consequently, at (just above) w_P, active clays exhibit high toughness, whereas low- to medium-plasticity soils (e.g. silty clays) display medium toughness, with silty, sandy and peaty soils presenting slight toughness (O'Kelly *et al*., 2022b, 2023). For the PL test, the kneading/rolling processes are repeated until the thread is found to shear both longitudinally and transversely (BSI, 1990b) when it has been rolled out to ~3 mm dia., with the first crumbling point taken as the w_P result. ASTM D 4318 (ASTM, 2017) and BS EN ISO 17892-12 (BSI, 2018b) give similar methodologies for performing PL testing. However, in terms of the obtained w_P results, there are potentially significant differences between the prescribed details of these codes (Barnes, 2021), including the rate and amount of rolling and the manifestations of the specified end point.

Regarding the manifestations of the specified end point in the standard PL test, the initiation of transverse cracks or crazing on the surface of the plastic thread (see Figure 2(a)) provides an indication of approaching the w_P. Importantly, the first appearance of the broken/dilated thread condition (see Figure 2(b)), as specified in BS EN ISO 17892-12 (BSI, 2018b: p. 21) (i.e. 'when the threads just begin to break apart'), should represent the PL. Barnes (2021) describes this condition as 'shear fracturing into aggregations and segments but with the thread remaining loosely intact and without separation or dispersion of the segments'. Furthermore, dilation in the thread core with shear rupture and a central opening frequently occurs, observed on the cross-section by splitting the soil thread (Barnes, 2021). According to Barnes (2021), continuing the rolling-out procedure until the thread segments disperse into individual crumbs (see Figure 2(c)), as required by the ASTM D 4318 (ASTM, 2017) and BS 1377-2 standards (BSI, 1990b), results in an exaggerated end point, causing unnecessary drying and therefore



(a)

(b)

(c)

Figure 2. Examples of thread conditions for Atterberg's PL testing: (a) initiation of transverse cracks/crazing on the thread surface provides an indication of approaching the w_P; (b) at its first appearance, the shown broken/dilated thread condition should represent the PL (BSI, 2018b); (c) dispersed crumbled thread (images reproduced from Barnes (2021))

produces an underestimate of w_P. The author of the present paper would concur with this viewpoint.

Furthermore, the examined codes for the PL test place much emphasis on rolling out of the soil threads to 3.0 or 3.2 mm dia. for the crumbling condition. It seems, however, that this requirement is not critical, with Haigh *et al*. (2013), Prakash *et al*. (2009) and Barnes (2021) reporting no statistically significant trend of varying w_P with the soil thread diameter (up to 6 mm dia. range investigated) for the crumbling condition. Of much greater importance is observing the condition of the soil thread during the rolling-out procedure to identify the first occurrence of the brittle state (Barnes, 2021).

Operators of the manual-rolling PL method require some experience before attaining a consistent rolling procedure (Barnes, 2021), with generally good repeatability of the test achieved for a single experienced operator in a single laboratory (Sherwood, 1970). However, because of factors such as the amount of finger pressure, rate of rolling and hand warmth, different operators will produce 'rolling paths' (a term coined by Barnes (2021)) that reduce the thread diameter quickly and thereby reduce the water content slowly, or vice versa, with a range of possible rolling paths in between (Barnes, 2021). This range of variation in the rolling paths for different operators, combined with the ambiguity about the end point (i.e. based on either the crumbling (ASTM, 2017; BSI, 1990b) or broken/dilated (BSI, 2018b) thread condition) and the operators' subjective judgement of same, means that the reproducibility of the standard PL test is generally poor in practice (Belviso *et al*., 1985; Sherwood, 1970; Sivakumar *et al*., 2009, 2015; Sridharan *et al*., 1999; Whyte, 1982), which reflects on I_P (= $w_L - w_P$). In addition to the above, it is also worth considering that the repeated kneading/rolling actions in performing the PL test produce incremental reductions in the soil thread's water content. Consequently, the w_P result, obtained for the first occurrence of the end point, would nearly always lie below the actual w_P (i.e. ductile–brittle state transition), thereby producing underestimated values of w_P, often seriously (Barnes, 2021). This aspect, hitherto unreported, adds to the poor repeatability of the standard PL test. To reduce its impact, close to w_P, the manual PL test should be performed with frequent short-duration kneading/rolling actions to produce smaller incremental water content reductions (Barnes, 2021). In this way, the underestimation of the actual w_P value would be reduced. Additionally, Barnes (2021) included an overemphasis on rolling to a particular thread diameter (i.e. 3 or 3.2 mm) as a further reason for the poor reproducibility of the standard PL test. Considering all of the above, it is not surprising why many researchers have sought to developed alternative testing methods to the manual PL test, as well as various indirect approaches for PL determination, which are described later in this section.

As mentioned earlier, ASTM D 653 (ASTM, 2006: p. 22) provides a second terminology used in relation to the PL – namely, as 'the water content corresponding to an arbitrary limit between the plastic and the semi solid states of consistency of a soil' – but seemingly not appearing to tie directly the determination of the PL state transition to the thread-rolling PL test. In other words, based on this terminology, it could be possibly construed that the ductile–brittle state transition may be experimentally established using appropriate methods (i.e. other than the thread-rolling PL test) that can distinguish between the plastic/ductile and brittle deformational behaviour responses of fine-grained soil with a varying water content. Hence, this ASTM D 653 terminology (ASTM, 2006), not tying the PL condition to the thread-rolling PL test, could potentially lead to some confusion regarding the PL's determination (definition) – that is, clay soil exhibits plasticity during many types of strain, with the standardised thread-rolling PL test establishing a useful threshold for a certain type of soil deformation (i.e. for the soil thread undergoing the repeated rolling paths of the PL test). Whereas, adopting different types of soil deformation (strain) testing for PL determination strain could potentially result in obtaining dissimilar values of water content for assigning to the associated ductile–brittle transition of the investigated fine-grained soil. It is for this very reason that the various international codes (apart from the Chinese standards) exclusively determine the PL condition using the thread-rolling PL test. It should also be considered that the A- and U-line boundaries of the Casagrande plasticity chart, as well as myriads of existing correlations relating w_P, I_P and/or the liquidity index (I_L) to other useful geotechnical parameters (e.g. compaction, CBR, hydraulic conductivity, swelling potential and pressure, consolidation and shear strength), have been deduced based on LL_{PC} and Atterberg's PL results.

Because of its general poor reproducibility across different operators (Belviso *et al*., 1985; Sherwood, 1970; Sivakumar *et al*., 2009, 2015; Sridharan *et al*., 1999; Whyte, 1982), various proposals have been made over the decades to improve on the thread-rolling PL test. These have largely focused on mechanising the manual-rolling procedure (e.g. Barnes, 2009, 2013a, 2013b; Bobrowski and Griekspoor, 1992; Temyingyong *et al*., 2002) while retaining the essential features of the soil deformation occurring in the manual PL test. Of these proposals, only the plate-rolling PL method (original presented by Bobrowski and Griekspoor (1992)) has been codified, being adopted in the Aashto and ASTM standards (Aashto, 2000; ASTM, 2017) but not in the BSI or EN standards. However, in practice, the manual thread-rolling PL test remains far more extensively used worldwide than the plate-rolling PL method. In using the Aashto and ASTM rolling device (see Figure 3), downward force and back-and-forth rolling motion are simultaneously applied to the soil thread through the to-and-fro horizontal movement of the top acrylic flat plate until it comes into contact with the 3.2 mm deep side-rails fastened to the acrylic bottom plate (both plates being covered with unglazed paper). However, most of the issues responsible for the poor repeatability of the manual thread-rolling PL test remain for the plate-rolling PL method. In other words, different operators would still produce a range of possible rolling paths (because of factors such as the amount of downward plate pressure and rate of rolling employed), different durations of the kneading/rolling actions would result in varied incremental water content reductions that produce different severities of the underestimation in the actual w_P value, the overemphasis on rolling to a particular thread diameter (i.e. 3.2 mm) remains, along with the subjective operator judgement of the end point.

Comprehensive statistical analysis performed on a database of 60 diverse fine-grained soils by Soltani and O'Kelly (2021) demonstrated that the standard (manual) thread-rolling and plate-rolling PL methods produce essentially similar w_P values. Furthermore, compared with the thread-rolling w_P, the likelihood of achieving consistent soil classifications in the Casagrande plasticity chart employing the plate-rolling w_P (along with the w_L) was shown to be 98%, with the identified discrepancies being cases that plot relatively close to the A-line (Soltani and O'Kelly, 2021). As such, the standard plate-rolling PL method can be used with confidence for soil classification.

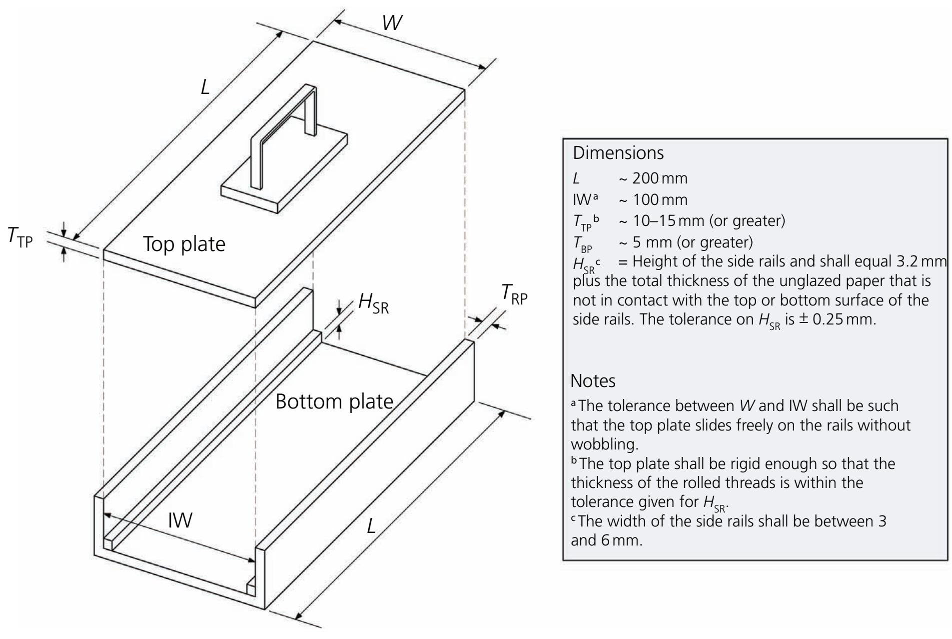


Figure 3. Schematic diagram of the ASTM/Aashto rolling device employed for PL determination (adopted from ASTM D 4318 (ASTM, 2017) and reproduced from Soltani and O'Kelly (2021))

Closely replicating Atterberg's manual-rolling method, Barnes (2009, 2013a, 2013b) developed a mechanical thread-rolling apparatus and test method that allow nominal applied stress and diametrical strain measurements for a uniform soil cylinder (thread) during the rolling-out procedure. Nominal remoulding toughness measurements are computed considering the areas beneath the obtained experimental stress–strain plots for the initially 6 mm dia. plastic soil threads prepared at different water contents. These values of remoulding toughness are plotted against water content, from which various soil toughness-related properties and the value of w_P (occurring at the sharply defined ductile–brittle transition) can be determined (Barnes, 2009, 2013a, 2013b). However, compared with the standard thread-rolling PL test, the Barnes's apparatus procedure takes longer and is more labour intensive, such that it is unlikely to replace commercially the simple, quick and cheap, but regrettably unreliable, standard thread-rolling PL test (Barnes, 2021).

Other PL testing alternatives include the thread-bending PL test of Moreno-Maroto and Alonso-Azcárate (2015), but the different type of soil thread deformation compared with that of the thread-rolling PL test may possibly result in dissimilar w_P results (e.g. bending may reveal some signs of surface distress nearer the identified PL (Barnes, 2021)). Additionally, de Oliveira Modesto and Bernardin (2008) proposed an indentation test, whereby the force applied to a 30° cone was slowly and steadily increased to indent the fine-grained soil test specimen, which was considered to be in a plastic state if the printed perforation mark presented no cracks, or in a brittle state for crack formation. With the soil deformational response as the discerning factor, this approach seemed reasonable, although the soil deformation system, being dissimilar from that of the rolling actions of the standardised PL test, could potentially produce different w_P results.

A review of other methods for soil plasticity measurement was presented in the paper by Andrade *et al*. (2011), including FC- and extrusion-type test methods, Pfefferkorn (impact deformation), capillary rheometer, torque rheometer and stress–strain curves obtained from compression tests. Additionally, suction or soil–moisture tension measurements, effective consolidation pressures from one-dimensional consolidation tests (reviewed by Barnes (2013b), O'Kelly *et al*. (2018) and O'Kelly (2021)) and, most recently, a power-based approach considering the work undertaken for extrusion of a known soil volume in a set time period (O'Kelly, 2022a) have been proposed for consistency limits determination, with differing levels of success achieved. A lot of research efforts over recent decades have centred on various strength-based FC and extrusion test approaches (here 'strength-based' meaning that certain values of s_u are effectively assigned (assumed) for identifying both the LL and PL states), and these methods are the focus of the remainder of this section. From the outset, it is important to point out that strength-based approaches are fundamentally inappropriate for Atterberg's PL determination (O'Kelly *et al*., 2018), since they cannot demonstrate the significant change in deformational behaviour, from plastic/ductile to brittle, for water contents each side of w_P, relying instead on correlations with the thread-rolling PL method to adjust the test apparatus configurations. Some researchers (e.g. Belviso *et al*. 1985; Feng, 2000, 2001, 2004; Harrison, 1988; Koumoto and Houlsby, 2001; Lee and Freeman, 2009; Sharma and Bora, 2003; Sivakumar *et al*., 2009; Wasti and Bezirci, 1986) have claimed to measure w_P using strength-based FC approaches. They assigned a specified cone penetration depth for the PL, invariably based on an assumed 100-fold increase in s_u with reducing water content over the full plastic range, as proposed by Wroth and Wood (1978). However, when using this strength-based approach, they are not actually measuring w_P but a completely different index parameter called PL_{100} (Haigh *et al*., 2013; Kyambadde and Stone, 2012; O'Kelly, 2013a; O'Kelly *et al*., 2018; Sivakumar *et al*., 2015, 2016; Stone and Kyambadde, 2007; Stone and Phan, 1995). These aspects are further elaborated in the section headed 'Critique of alternative consistency limits determination methods'. Using the same strength-based rationale, other researchers (e.g. Medhat and Whyte, 1986; Timár, 1974; Whyte, 1982) have claimed to measure w_P using soil extrusion approaches. They typically associate the 'PL' with an extrusion pressure 100 times greater than that required to cause steady extrusion of the saturated remoulded fine-grained soil at its $w_{L(FC)}$ to occur from the extrusion chamber (see the review paper on this topic by O'Kelly (2019b)). However, like the strength-based FC approaches, these extrusion approaches are essentially attempting to measure a PL_{100}-type (i.e. not Atterberg's PL) parameter. Note that from reassessing a large data set compiled for many hundreds of different fine-grained soils reported in the literature, O'Kelly (2019b) concluded that conventional analysis (i.e. considering the continuous extrusion: for example, in hot/cold extrusion of long metal billets) did not seem to provide reliable s_u estimates consistently in the case of soil extruded through a die orifice from a small cylindrical container. This suggested that for the soil extrusion apparatuses, different shearing mechanisms (zones) were at play for the test soil contained in the cylindrical container (Xu *et al*., 2023) and possibly also localised billet consolidation was occurring for the combination of slow die displacement rates employed and relatively high extrusion pressures required (O'Kelly, 2019b).

Meanwhile, other researchers have attempted to determine I_P indirectly, and hence w_P (= $w_L - I_P$), using, for instance, correlations between the I_P and the $I_{F(PC)}$ or $I_{F(FC)}$ parameters (Fall, 2000; Fang, 1960; Haigh and Vardanega, 2014; Nagaraj and Srinivasa Murthy, 1987; Soltani and O'Kelly, 2022; Spagnoli *et al*., 2019; Sridharan *et al*., 1999). In other words, using I_P:$I_{F(PC)}$ and I_P:$I_{F(PC)}$ correlations, the values of w_L, I_P and w_P could potentially be determined for fine-grained soils from only the experimental flow curve results. This aspect is further elaborated in the section headed 'Way forward?'.

Some pitfalls of consistency limits testing

This section highlights some potential pitfalls in performing consistency limits testing, including inadequate maturing (curing) of the prepared fine-grained soil materials before performing the LL tests and, for testing of some unconventional geomaterials are also described.

Before performing consistency limits tests, the remoulded soil paste (prepared at the desired water content) is stored in an airtight container and allowed to stand long enough to enable the pore water to permeate and distribute uniformly through the soil. ASTM D 4318 (ASTM, 2017), BS 1377-2 (BSI, 1990b) and BS EN ISO 14688-2 (BSI, 2018a) recommend material maturing periods of ~16 h (overnight), ~24 h and up to 24 h, respectively, before starting LL_{PC}/LL_{FC} testing. While a shorter time may be acceptable for soils of low clay content (BSI, 1990b), longer maturing periods may be needed for high-plasticity clays, such as bentonites. For instance, with $w_{L(PC)}$ increasing with maturing period, a minimum 4-day curing period was needed to reach nearly equilibrium values of $w_{L(PC)}$ measured for three bentonites investigated by Bharat *et al*. (2020). In other words, an inadequate maturing period for the LL test material may result in a (considerable) underestimation of the actual value of w_L.

Consistency limits testing is synonymous with the soil fraction passing the 425 µm sieve size, but there is a small yet increasing number of papers published in the recent literature reporting on LL and PL testing of fibre-reinforced fine-grained soils. Examples include studies of various fine-grained soils examining the effects of 2% human-hair-fibre additive, each fibre being 20–50 mm long (see the paper by Ayothiraman *et al*. (2022) and discussion of the same by O'Kelly and Soltani (2022a)), and up to 10% recycled sheep-wool additive (Omer *et al*., 2022) on the consistency limits test results. This practice is not correct, since consistency limits testing first requires the removal during the sample preparation stage of any coarse particles and fibres present (BSI, 1990b), with the testing performed on the fine soil fraction (i.e. passing the 425 µm sieve size). The literature also includes an increasing number of investigations reporting on the consistency limits testing of fine-grained soils amended with various biopolymers at low dosages (e.g. see the review paper by Sujatha and O'Kelly (2023)). Compared to the natural soil, the texture of biopolymer-treated soil is quite different. Hence, the question arises as up to what biopolymer dosage can the plasticity behaviour of the mixture be still regarded as typical of fine-grained soil (i.e. above a certain generally small dosage, the texture (deformation behaviour) of the wet mixture becomes a typical of fine-grained soil behaviour). For example, Kannan *et al*. (2023) described the effect of 0.25–1% additions of the gellable biopolymer sodium–carboxymethylcellulose (NaCMC) to low-plasticity organic silt soil. The NaCMC additive produced dramatic increases in $w_{L(PC)}$, w_P and the computed I_P, which resulted in a change in soil classification (from silt- to clay-type soil). Kannan *et al*. (2023) explained that the addition of water molecules in the NaCMC-treated soil increased its water-imbibing capacity, such that with an increased NaCMC dosage, more water was needed for the soil to lose shear strength.

Furthermore, in the case of fibrous peats, greater mechanical breakdown of the intact peat fibres in producing the <425 µm sized soil solids test material resulted in lower measured values of w_L, w_P and I_P. Consequently, their adoption for behavioural characterisation could be (grossly) misleading, since the measured w_L, w_P and I_P values did not give sufficient insight into the likely mechanical behaviours of the original fibrous peat materials in the field (O'Kelly, 2015a, 2016a, 2022b). O'Kelly (2015a, 2016a) went as far as to conclude that the consistency limits concepts should not be applied/extended to fibrous peat materials. Unlike remoulded inorganic fine-grained soils, the reality of individual (distinct) soil particles strictly does not always apply for peats that are not completely humified, with connectivity between the constituent fibres in fibrous peat materials provided by cellular connections and fibre entanglement (O'Kelly, 2015b; O'Kelly and Orr, 2014). This does not mean that peat soil deposits do not exhibit plasticity within a range of water contents; rather, they reveal the shortcomings of the standardised laboratory tests in accurately identifying their associated plastic ranges. Other anomalies arise, for instance, in regard to consistency limits testing of diatomaceous earth soils, where a large amount of water is contained in the intra-skeletal pore spaces of the frustules, and thus, it barely interacts with the soil particles (Bandini and Al Shatnawi, 2017). Consequently, the results of standard consistency limits testing performed on these soils also may not provide reliable information in respect to their likely mechanical properties (Vardanega *et al*., 2023).

Critique of alternative consistency limits determination methods

Over the decades, various assumptions ('rules of thumb') have been employed in developing correlations between the consistency limits, and with other geotechnical parameters, for use in geotechnical engineering practice. These include, for instance, correlations that can be used to obtain w_P data when reliable w_P measurements are unavailable or when difficulties are encountered (e.g. for marginally plastic soils) in executing the standard PL test. Some of these assumptions have subsequently been found baseless or unreliable, such that the continued use of correlations based on them should be discouraged. The first example examined here is the fallacy of a strength-based definition for Atterberg's PL, as originally proposed by Wroth and Wood (1978), which assumes the following:

- a 100-fold variation in s_u over the full plastic range
- $s_{u\text{-}L(FC)} = 1.7$ kPa, such that the value of s_u at w_P (i.e. $s_{u\text{-}P}$) would be assigned as 170 kPa (Wroth and Wood, 1978).

On these basis, some researchers (e.g. Belviso *et al*., 1985; Feng, 2000, 2001, 2004; Harrison, 1988; Koumoto and Houlsby, 2001; Lee and Freeman, 2009; Sharma and Bora, 2003; Sivakumar *et al*., 2009; Wasti and Bezirci, 1986) have claimed that w_P can be determined using the FC approach. Thus, from Equation 1 and based on their assumptions with the LL_{FC} established using the 80 g/30° cone for $d_L = 20$ mm (BSI, 1990b, 2018b), according to these researchers, the 'PL' could be established using the same cone set-up as the water content for $d = 2$ mm. One of a few proposed extrapolation techniques (typically of the best-fitting correlation line to the data plotted in log d–log w (see e.g. Feng, 2000, 2001)) is applied to the experimental LL_{FC} data to deduce the water content corresponding to d = 2 mm. There is some evidence to suggest that an approximately bilinear I_L (w)–log s_u relationship occurs over the brittle and plastic states, with its turning point occurring at w_P (i.e. I_L = 0), as deduced from vane-shear investigations of remoulded clay soils by Vinod *et al*. (2013). Hence, valid extrapolation of the experimental d:w data for d progressively reducing to 2 mm with decreasing water content would strictly necessitates that the soil remains in a plastic state (i.e. $w_P \leq w_{P100}$, where w_{P100} is the water content associated with PL_{100}). In other words, soil tested with water content w would occur in a brittle state for $w_{P100} < w < w_P$. Alternatively, a different cone set-up – for example, employing an 8 kg/30° contacting cone with d = 20 mm (Sivakumar *et al*., 2015; Wood, 1990) – could be employed to avoid the need for extrapolation of the experimental d:w data for PL_{100} determination. This approach would still require $w_P \leq w_{P100}$ for the investigated fine-grained soil, such that the plastic strength analysis of Equation 1 remains valid. In the section headed 'Liquid limit', it was explained that for the 80 g/30° FC set-up with d_L = 20 mm (BSI, 1990b, 2018b) and considering a range of μ = 0.10 ± 0.05 (Ladd and Foott, 1974) for different inorganic fine-grained soils, the value of $s_{u\text{-}L(FC)}$ had a plausible range of 1.6–2.4 kPa (O'Kelly *et al*., 2018). On this basis, the assumption of $s_{u\text{-}L(FC)}$ = 1.7 kPa adopted in the strength-based FC approach was strictly not correct for all fine-grained soils. Regarding the assumed 100-fold s_u variation over the full plastic range, considering their diverse natures in terms of physiochemical and mechanical behaviour/properties, from the author's perspective, it seems too much of a stretch thatxall fine-grained soils would universally have the same $s_{u\text{-}P}$-to-$s_{u\text{-}L(FC)}$ ratio value (of 100). It would seem more plausible that when considering a wide range of diverse fine-grained soils, their values of $s_{u\text{-}P}/s_{u\text{-}L(FC)}$, and also of $s_{u\text{-}P}$, would have continuous probability distributions with significant values of standard deviation (σ). After many decades of research in this area, including the development of various FC apparatuses and d:w data extrapolation techniques, this has been definitively shown to be the case – that is, generally $s_{u\text{-}P} \neq 100 \times s_{u\text{-}L(FC)}$ and $s_{u\text{-}P} \neq$ 170 kPa (Haigh *et al*., 2013; Nagaraj *et al*., 2012; O'Kelly, 2013a; Vardanega and Haigh, 2014). In other words, when considering a diverse range of fine-grained soils, the magnitudes of $s_{u\text{-}P}$ (see Figure 4) and of the $s_{u\text{-}P}/s_{u\text{-}L(FC)}$ ratio can both vary widely (Haigh *et al*., 2013; Nagaraj *et al*., 2012; O'Kelly, 2013a; Vardanega and Haigh, 2014), with $s_{u\text{-}P}$ more often than not <170 kPa (O'Kelly *et al*., 2018).

The realisation of the strength-based 'PL' definition fallacy led to the introduction of a new index parameter, denoted as PL_{100} (Haigh *et al*., 2013; Kyambadde and Stone, 2012; O'Kelly, 2013a; O'Kelly *et al*., 2018; Sivakumar *et al*., 2015, 2016; Stone and Kyambadde, 2007; Stone and Phan, 1995) and coined as the 'plastic strength limit' by Haigh *et al*. (2013). In other words, Atterberg's PL and the PL_{100} are fundamentally different parameters (see the papers by O'Kelly *et al*. (2018) and O'Kelly (2021) and the discussions by Sivakumar *et al*. (2016) and O'Kelly *et al*. (2022a)), with, for example, experimental data presented in the studies by Feng (2000) and Hrubesova *et al*. (2020) showing ±20% variation of w_{P100} from w_P. Note that for NP fine-grained soil, I_P = 0 (i.e. $w_P \geq$ the measured $w_{L(PC)}$ or $w_{L(FC)}$) or w_P cannot be determined by the PL test (BSI, 1990a, 2018a), but it can always be assigned a value of w_{P100} (i.e. $\ll w_{L(FC)}$). As described in the next section, the FC-derived $I_{F(FC)}$, w_{P100} and I_{P100} (= $w_{L(FC)} - w_{P100}$) parameters, and variants thereof, provide new possibilities for FC remoulded undrained shear strength (i.e. $s_{u\text{-}FC}$) predictions and in the classification of fine-grained soils (O'Kelly *et al*., 2018; Vardanega *et al*., 2022). However, it is worth repeating that it would not be correct to refer to (equate) PL_{100} as Atterberg's PL or to refer to (equate) I_{P100} as the I_P parameter; the juxtaposition of these standard notations would cause much confusion.

Also proposed, with varying predictive capabilities for I_P and hence w_P, are empirical correlations based on the flow index concept (e.g. Fall 2000; Soltani and O'Kelly, 2022; Spagnoli *et al*., 2019; Sridharan *et al*., 1999), which work on the premise that the $I_{F(PC)}$ and $I_{F(FC)}$ parameters provide a measure of soil plasticity (Soltani and O'Kelly, 2022). Hence, they could be used, either independently or in conjunction with other index parameters – for example, LL_{PC} and LL_{FC} (see Fall, 2000) – to provide estimates of the values of I_P and hence w_P (= $w_L - I_P$).

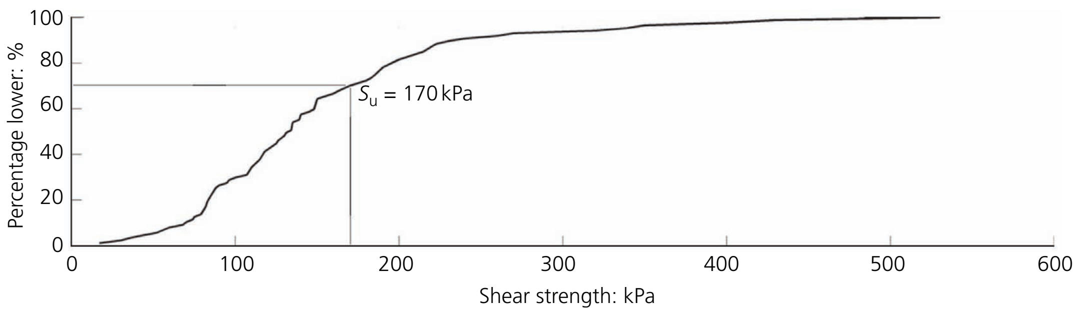


Figure 4. Cumulative distribution of shear strengths of fine-grained soils mobilised at their Atterberg's PLs (adopted from Haigh *et al*. (2013))

However, recent investigations involving the author comprising comprehensive statistical analyses performed on large and diverse data sets of I_P:$I_{F(FC)}$ test results have demonstrated that such correlations, at best, can provide only rough approximations of the actual I_P and hence w_P (O'Kelly and Soltani, 2021a; Soltani and O'Kelly, 2020, 2022; Vardanega *et al*., 2022).

Furthermore, over the decades, numerous empirical correlations have been proposed in the published literature for deducing the values of LL, PL and/or I_P (e.g. see O'Kelly and Soltani, 2021b). However, when considering large and diverse collections of fine-grained soils with widely different mineralogical and gradation properties, the LL and PL parameters generally do not correlate with one another. Moreover, for a fine-grained soil with a certain LL value, its value of I_P can range from zero (i.e. NP soil) to an approximate upper bound given by the associated U-line value in the Casagrande plasticity chart. Consequently, when considering large and diverse collections of fine-grained soils, the LL and I_P typically only weakly correlate, with the correlation largely arising from the fact that I_P is itself calculated using the LL (i.e. $I_P = w_L - w_P$) (O'Kelly and Soltani, 2021b; Soltani *et al*., 2023). Greater predictive performance may be achieved in cases where the LL, PL and/or I_P correlations are developed using data sets pertaining to specific fine-grained soil formations (i.e. pertaining to narrow ranges of soil composition and mineralogical properties). When employed for different soil types, these correlations invariable produce poor predictions (O'Kelly and Soltani, 2021b, 2023a).

Way forward?

Various issues and shortcomings regarding the conventional consistency limits and their methods of determination were highlighted above, particularly the poor reproducibility of the standard thread-rolling PL test (Belviso *et al*., 1985; Sherwood, 1970; Sivakumar *et al*., 2009, 2015; Sridharan *et al*., 1999; Whyte, 1982). Approached from the viewpoint of obtaining correlations with soil strength and stiffness, the PL_{100} and I_{P100} parameters could be seen as favourable choices, being implicitly linked to the variation of s_u with water content (Haigh *et al*., 2013; Kyambadde *et al*., 2014) – that is, for fine-grained soil existing in a plastic state (i.e. $w_P \leq w_{P100}$, such that the plastic strength analysis of Equation 1 remains valid), I_{P100} gives the range of water contents producing a 100-fold gain in s_u relative to the $s_{u\text{-}L(FC)}$ value of ~1.7 kPa. (Note that, for reasons elaborated earlier, with $w_{P100} \leq w_P$, there would be uncertainties regarding the actual strength gain for the identified I_{P100} value.) As described in the previous section, w_{P100} can be determined using an 8 kg/30° contacting cone for d = 20 mm or from extrapolation of the 80 g/30° LL_{FC} flow curve (typically obtained for d = 15–25 mm) to d = 2 mm. Both approaches have drawbacks, including the predictive capability of the extrapolation technique adopted (from the various ones proposed in the literature) and potential health and safety issues related to using the heavier 8 kg cone during routine laboratory investigations (Sivakumar *et al*., 2015). This section focuses on some recent developments in the measurement of the strength-based PL_{100} and I_{P100}, their use in obtaining s_u predictions and in the classification of fine-grained soils.

A major advancement on efforts to bring the PL_{100} parameter into mainstream use was the development of an FC set-up that incorporates a free-fall height of h_d (Sivakumar *et al*., 2015), resulting in the tip of the falling cone contacting the surface of the soil test specimen with an initial (impact) velocity. Based on an energy-conservation approach and with consideration of the higher strain rates occurring compared with those of the 8 kg/30° contacting cone, Sivakumar *et al*. (2015) settled on a modified FC set-up (Figure 5), employing a 0.727 kg/30° cone with a free-fall height of h_d = 200 mm (i.e. resulting in a 2.0 m/s velocity at the start of the cone penetration) for determining the value of w_{P100}, obtained for d = 20 mm. Of course, the w_{P100} of fine-grained soil can be determined using the modified FC test set-up employing any equivalent combination of cone weight and apex angle, h_d and d (O'Kelly, 2022c).

Additionally, employing finite-element analysis and energy-conservation approaches, Dastider *et al*. (2021) and O'Kelly (2022c) respectively showed how the associated cone factor K_d (i.e. accounting for the inclusion of a free-fall height h_d) can be calculated from the conventional cone factor K (i.e. for the set-up with the cone tip initially just contacting the surface of the test specimen) as $K_d = K(1 + h_d/d)$. Hence, FC set-ups that include a free-fall height can also be conveniently used as strength measurement devices for plastic fine-grained soils – for example, in establishing their s_u variability with changing water content and sensitivity (i.e. ratio of undisturbed to remoulded undrained strength, without any change in water content).

For a given FC apparatus set-up, the associated value of K can bexcalculated from rearranging Equation 1, inputting the assumed $s_{u\text{-}L(FC)}$ of ~1.7 kPa (Wroth and Wood, 1978) and related value of d_L. O'Kelly *et al*. (2018) and Haigh *et al*. (2021) elaborated on this approach, with various pitfalls highlighted in the discussion paper by O'Kelly and Soltani (2023b). For instance, considering the contacting cone set-up for the 80 g/30° cone and d_L = 20 mm (BSI, 1990b, 2018b), this approach gives K = 0.87, in agreement with the experimentally derived value of 0.85 (σ = 0.05) reported by Wood (1985). Whereas, for the 60 g/60° cone and d_L = 10 mm (BSI, 2018b), the value of K is deduced as 0.29, in agreement with experimentally derived K values of 0.3 (Hansbo, 1957), 0.29 (σ = 0.04) (Karlsson, 1961) and 0.29 (σ = 0.05) (Wood, 1985). Experimentally derived values of K (or K_d) are obtained by calibration (back-calculation) with direct strength measurements, typically obtained using miniature vane-shear tests or less frequently from UU triaxial compression tests (see e.g. O'Kelly, 2014, 2019a). The derived value of K (or K_d) will depend on the different shearing modes of these strength tests (O'Kelly, 2013b, 2014, 2023), the shear strain rates they employ, and, as elaborated below, also on the strain rate-dependence of undrained strength for the investigated fine-grained soils. Furthermore, the strength

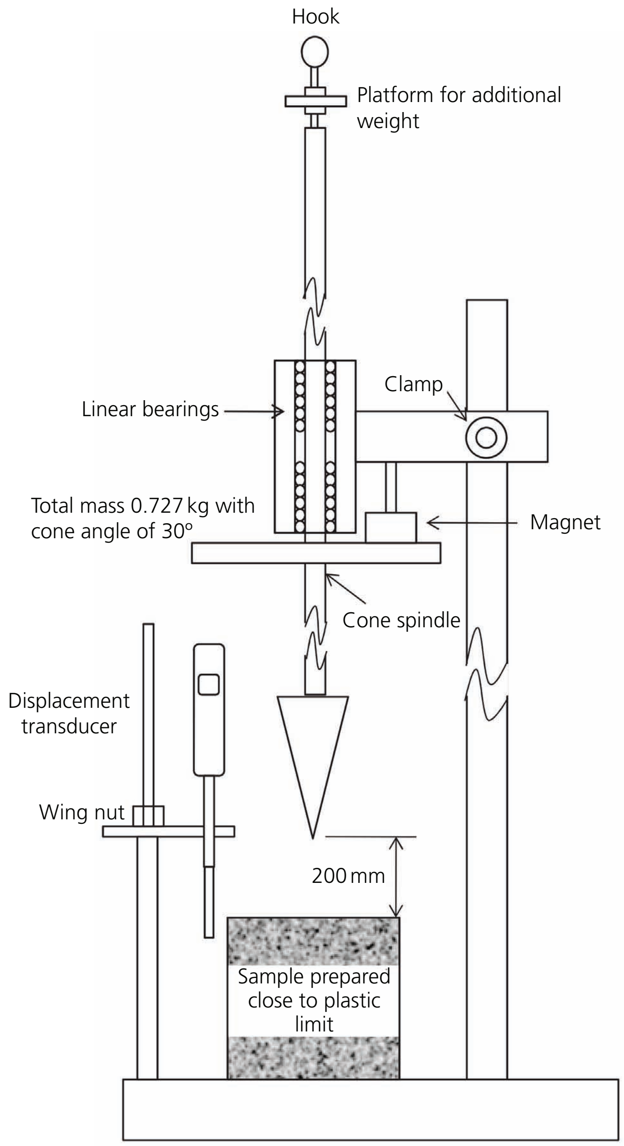


Figure 5. Sivakumar *et al.*'s modified FC test set-up, employing a 0.727 kg/30° cone and a free-fall height of h_d = 200 mm, for determination of the PL_{100} parameter, interpolated from the best straight-line fitting of four or more *w*:*d* data points for *d* = 20 mm (reproduced from Sivakumar *et al.* (2015))

apparatus employed for the calibration must have the required measurement sensitivity, and in the case of the vane apparatus, the shearing rate employed must be sufficiently fast to maintain a truly undrained specimen shearing condition (Llano-Serna and Contreras, 2020; O'Kelly, 2019a). Even with these two testing conditions achieved, O'Kelly (2018) and O'Kelly *et al.* (2018) demonstrated that when investigating different fine-grained soils, the deduced value of *K* for a specified value of $s_{u\text{-}L(FC)}$ (or, conversely, the $s_{u\text{-}L(FC)}$ associated with a certain value of *K*) could vary somewhat depending on the natural variation in experimental μ between different fine-grained soils. This essential point was also reported by Llano-Serna *et al.* (2022). In other words, many inorganic fine-grained soils have a μ value of ~0.10 (Koumoto and Houlsby, 2001; Kulhawy and Mayne, 1990; Ladd and Foott, 1974), with a range of μ = 0.10 ± 0.05 (Ladd and Foott, 1974) not uncommon, although μ values of up to 0.30 may occur for some high-organic-content soils (O'Kelly, 2014, 2016b, 2018; O'Kelly *et al.*, 2020a). Therefore, in determining the value of *K* for relating $s_{u\text{-}FC}$ to measured vane-shear or UU triaxial strength results, one needs to take into account the significantly different shear strain rates occurring in these different strength tests. For example, in the case of the contacting 80 g/30° cone, the $\dot{\gamma}$ of ~1.0×10^6 %/h for the range of *d* = 15–25 mm (Koumoto and Houlsby, 2001) is approximately four orders of magnitude faster compared with that generally employed in performing a typical standard UU triaxial test. Accordingly, with the assumed value of $s_{u\text{-}L(FC)} \approx 1.7$ kPa taken as pertaining to $\mu \approx 0.10$, for those fine-grained soils with μ = 0.10 ± 0.05, the 80 g/30° contacting cone set-up, with d_L = 20 mm (BSI, 1990b, 2018b), could plausibly mobilise an $s_{u\text{-}L(FC)}$ range of 1.6–2.4 kPa (O'Kelly *et al.*, 2018). Accordingly, for PL_{100} established at *d* = 20 mm using the 8 kg/30° contacting cone set-up, the mobilised s_u (commonly taken as ~170 kPa) could range between 160 and 240 kPa (O'Kelly *et al.*, 2018).

An important consideration regarding the PL_{100} parameter is that for $w_{P100} < w_P$ (more often than not the case when investigating many different fine soils (O'Kelly *et al.*, 2018)), the fine-grained soil being tested at water contents of about w_{P100} will exist in a brittle state. Consequently, the validity of applying extrapolation techniques (i.e. to the experimental FC flow curve results obtained from LL_{FC} testing) for obtaining w_{P100} and the plastic strength analysis of Equation 1 for $w_{P100} < w_P$ both become questionable (O'Kelly *et al.*, 2018, 2020b). To overcome this, O'Kelly *et al.* (2018) proposed using the PL_{25} parameter in place of PL_{100}, the former defining as the soil water content (i.e. w_{P25}) that mobilises an s_u value of 25 × $s_{u\text{-}L(FC)}$ (≈ 42.5 kPa), considered an expected lower bound value of $s_{u\text{-}P}$ mobilised for inorganic fine-grained soils. PL_{25} can be measured experimentally as the water content corresponding to *d* = 4 mm of the standard 80 g/30° contacting cone. Alternatively, analogous to the modified FC set-up of Sivakumar *et al.* (2015), PL_{25} can be established by employing any equivalent combination of cone weight, apex angle, h_d and *d* (for $d \gg 4$ mm), as pointed out by O'Kelly (2022c). The latter approach would have the advantage of shear testing a larger portion of the soil specimen and be potentially more accurate since, with $s_{u\text{-}FC} \propto 1/d^2$, small inaccuracies in *d* measurements have greater impact on the deduced $s_{u\text{-}FC}$ for smaller values of *d*.

For obtaining s_u predictions, taking the value of $s_{u\text{-}L(FC)}$ as 1.7 kPa, O'Kelly *et al.* (2018) proposed that the value of $s_{u\text{-}FC}$ mobilised for water content *w* could be computed as follows:

$$2.\quad s_{u\text{-}FC} = 10^{\left(1.4\, I_{C(FC)} + 0.23\right)}$$

where $I_{C(FC)}$ is the FC consistency index parameter given as follows:

$$3. \quad I_{C(FC)} = \left(\log w_{L(FC)} - \log w\right) / \left(\log w_{L(FC)} - \log w_{P25}\right)$$

Note that the $I_{C(FC)}$ parameter is defined in logarithmic form. This arises since, compared with the semi-logarithmic form, the double-logarithmic s_u–w correlation for a given fine-grained soil provides a regression coefficient value closer to unity when considering a wide water content range (O'Kelly *et al.*, 2018).

Although $I_{F(FC)}$- and $I_{F(PC)}$-based correlations should generally not be used for estimating I_P and hence w_P (= measured LL minus the flow index-deduced I_P) (O'Kelly and Soltani, 2021a; Soltani and O'Kelly, 2020, 2022; Vardanega *et al.*, 2022), it appears that they are suitable for use in routine classification of fine-grained soils (Soltani and O'Kelly, 2022; Vardanega *et al.*, 2022). In this respect, Vardanega *et al.* (2022) proposed a new plasticity chart (see Figure 6), plotting $I_{F(FC)}$ against $w_{L(FC)}$ obtained for the 80 g/30° cone with d_L = 20 mm (BSI, 1990b, 2018b).

In producing the new chart, Vardanega *et al.* (2022) made appropriate adjustments to reposition the A- and U-lines (of the Casagrande plasticity chart) to take into account (*a*) the change in the ordinate – using $I_{F(FC)}$ in place of I_P; (*b*) the fact that the abscissa of the new chart plots $w_{L(FC)}$ – rather than the $w_{L(PC)}$ data employed in developing the original (Casagrande-type) plasticity chart. The distinct advantage of the Vardanega *et al.* (2022) chart over existing plasticity charts (the major ones being reviewed in the paper by Moreno-Maroto *et al.* (2021)) is that it allows classification of fine-grained soils to be achieved without the need to perform the thread-rolling PL test, which can have poor reproducibility. In other words, using the BS 80 g/30° cone, the $w_{L(FC)}$ of the fine-grained soil is established for d_L = 20 mm from the best-fitting line to four or more data points in a w–d plot, investigating the range of $d \approx$ 15–25 mm (BSI, 1990b, 2018a). Plotting the same four or more data pairs in the w–log d space, the FC flow index is obtained from regression analysis as $I_{F(FC)} = \Delta w/\Delta \log_{10} d$. Then, using the obtained $w_{L(FC)}$ and $I_{F(FC)}$ data pair, the soil can be classified as clay- or silt-type soil, with its associated fine-grained soil plasticity level class, using the Vardanega *et al.* (2022) plasticity chart (Figure 6).

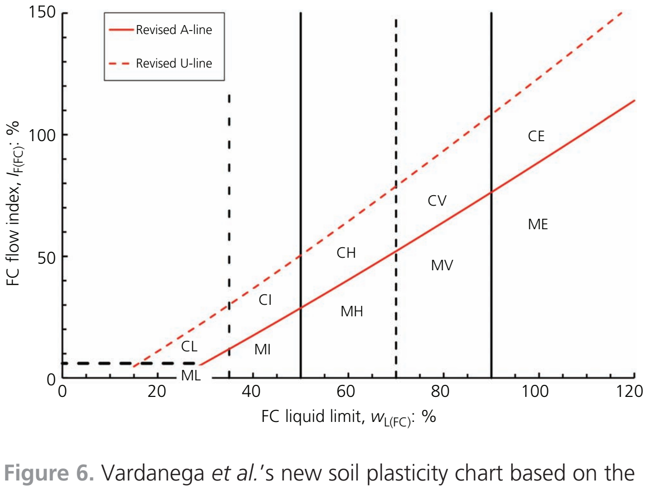


Figure 6. Vardanega *et al.*'s new soil plasticity chart based on the BS FC flow index and LL parameters (adopted from Vardanega *et al.* (2022)). Note that the soil classes (i.e. CL, CI, CH, CV, CE, ML, MI, MH, MV and ME) labelled in the chart refer to the BS soil plasticity classification obtained using the Casagrande-type plasticity chart (BSI, 1990b)

In a more recent investigation considering 125 very different fine-grained soils, Karakan (2023) showed that the Vardanega *et al.* (2022) $I_{F(FC)}$–$w_{L(FC)}$ plasticity chart produced 92% agreement with the soil classifications obtained using the Casagrande-type plasticity chart. Note that $I_{F(FC)}$- and $I_{F(PC)}$-based correlations allocate a value of I_P irrespective of the inherent plasticity characteristics of the investigated soil. In other words, applying these correlations to NP fine-grained soils would result in them being erroneously assigned as having some plasticity, such that they would incorrectly plot in both plasticity charts (Soltani and O'Kelly, 2022; Vardanega *et al.*, 2023). To overcome this limitation, before plotting in the $I_{F(FC)}$–$w_{L(FC)}$ plasticity chart, the operator would first need to confirm the soil's plasticity credentials, generally judged by touch rather than requiring a standard PL test to be performed (Vardanega *et al.*, 2023). However, if the water content indicating the transition from the plastic state to the brittle state is needed, then the standardised thread-rolling PL test must be performed.

Summary and conclusions

This paper addresses Atterberg's consistency limits of fine-grained soil using a comprehensive critical review of the literature and standardisation. The strengths, weaknesses and limitations of the various methods for determining the consistency limits, particularly the PL, were elaborated. For LL determination, in the present global context, the hard- and soft-base PC and the 80 g/30° and 60 g/60° cone FC approaches are generally considered equally valid, although one approach may be preferred over another in certain territories. Standardised LL methods define this state transition using particular testing criteria for the PC and FC methods. It can be shown that both the LL_{PC} and LL_{FC} relate to narrow s_u ranges of typically $s_{u\text{-}L(PC)}$ = 1–3 kPa and plausibly $s_{u\text{-}L(FC)} \approx$ 1.6–2.4 kPa, although the latter range is often simply reported in the literature as 1.7 kPa. For testing the same soil using particular PC and FC devices, the value of $s_{u\text{-}L(PC)}$ progressively reduces with increasing value of $w_{L(PC)}$ (typically from 2.5 to 1.6 kPa for LL_{PC} of 20 to 70%), whereas $s_{u\text{-}L(FC)}$ is dependent on the μ parameter value of the test soil, with $s_{u\text{-}L(FC)} \approx$ 1.7 kPa for an average $\mu \approx 0.10$. Before performing LL testing, the test soil must be allowed to mature adequately, which for high-plasticity clays may require (significantly) longer than the specified 16–24-h standing period stated in the codes. Inadequate maturing of the test material causes an underestimation of the actual w_L value, particularly for $w_{L(PC)}$. Considering the range of different LL testing apparatuses employed by the various codes and with the $w_{L(PC)}$-to-$w_{L(FC)}$ ratio increasing for increasing w_L, it is inevitable that w_L results obtained for carefully testing the same fine-grained soil in accordance with these codes may differ. Resulting changes in measured w_L (and hence I_P) due to a change in the LL method may be significant (e.g. in some instances being sufficient to cause a change in soil classification), particularly for higher levels of plasticity, in which case carefully chosen published correlations could be employed to relate values of w_L deduced using different measurement techniques and codes.

The PL (i.e. Atterberg's PL) is understood as the plastic/ductile–brittle state transition, and apart from the Chinese standards, it is exclusively determined in the codes using the manual 'rolling of threads' PL test and also the ASTM/Aashto device-rolling PL method. There are potentially significant disparities between some details of the PL test method among different codes, including thexmanifestations of the specified end point. At first appearance, the broken/dilated thread condition, as specified in BS EN ISO 17892-12, should be taken as representing the PL, whereas continuing rolling until the thread segments disperse into loose, separate crumbs, as required by the ASTM D 4318 and BS 1377-2 standards, causes unnecessary drying and therefore underestimates the w_P value. The PL test itself establishes a useful threshold for a certain type of soil deformation – that is, repeating rolling paths – although inevitably, different operators produce a range of possible rolling paths (e.g. either reducing the thread diameter quickly, thereby reducing the water content slowly, or vice versa). Adopting different types of soil deformation testing for PL determination (e.g. using the thread-bending PL test) could potentially result in obtaining dissimilar w_P results. Strength-based test approaches (e.g. using FC apparatus) are fundamentally unsuitable for PL determination, since they cannot demonstrate the significant change in soil deformation behaviour, from plastic/ductile to brittle, for water contents each side of w_P. Rather employing the same FC set-up for both consistency limits, they assign a value of d_L (i.e. $s_{u\text{-}L(FC)}$) and then usually assume a 100-fold s_u variation over the full plastic range, thereby defining a value of $s_{u\text{-}P}$ obtained for $d_L/10$. However, when considering a diverse range of inorganic fine-grained soils, as well as $s_{u\text{-}L(FC)}$ potentially ranging 1.6–2.4 kPa depending on the μ values of the soils in the credible range of 0.10 ± 0.05, the values of $s_{u\text{-}P}$ and hence of the $s_{u\text{-}P}$-to-$s_{u\text{-}L(FC)}$ are both found to vary widely. In other words, strength-based FC approaches cannot be used to obtain Atterberg's PL consistently and reliably.

However, FC approaches can be used to obtain various useful strength-based index parameters, providing new possibilities for $s_{u\text{-}FC}$ predictions and in the classification of fine-grained soils. Compared with the PC-type LL and thread-rolling PL tests, being easier to perform and its apparatus being easier to maintain in correct adjustment with the codes give the FC-deduced parameters (i.e. $w_{L(FC)}$, w_{P100} (PL_{100}), I_{P100} and $I_{F(FC)}$) greater repeatability and reproducibility. Valid determination of the PL_{100} parameter strictly requires that $w_P \leq w_{P100}$, such that the plastic analysis of Hansbo's FC-strength equation and/or the adopted flow curve (w:d data) extrapolation technique applied to LL_{FC} data obtained for a higher water content range remains valid. Hence, the PL_{25} parameter can be regarded as a better choice than PL_{100}. For instance, PL_{25} can be measured using the 80 g/30° contacting cone for d = 4 mm. Being strength based, PL_{100} and PL_{25} can be determined for any suitable combinations of cone weight, apex angle, free-fall height (h_d) and penetration depth (d). For instance, w_{P100} can be obtained using an 80 g/30° cone for h_d = 0 and d = 2 mm or using a 0.727 kg/30° cone for h_d = 200 mm and d = 20 mm.

Having confirmed the soil's plasticity credentials, typically judged by touch, the Vardanega *et al*. (2022) revised plasticity chart (plotting $I_{F(FC)}$ against $w_{L(FC)}$) allows classification of fine-grained soils to be achieved based solely on analysis of BS LL_{FC} testing results – that is, without the need to perform the PL test, which has high operator variability.

REFERENCES

Aashto (American Association of State Highway and Transportation Officials) (2000) Aashto T90: Standard method of test for determining the plastic limit and plasticity index of soils. Aashto, Washington, DC, USA.

Andrade FA, Al-Qureshi HA and Hotza D (2011) Measuring the plasticity of clays: a review. *Applied Clay Science* **51(1–2)**: 1–7, https://doi.org/10.1016/j.clay.2010.10.028.

ASTM (2006) D 653: Standard terminology relating to soil, rock, and contained fluids. ASTM International, West Conshohocken, PA, USA.

ASTM (2017) D 4318: Standard test methods for liquid limit, plastic limit, and plasticity index of soils. ASTM International, West Conshohocken, PA, USA.

Atterberg A (1911a) Lerornas forhållande till vatten, deras plasticitetsgränser och plasticitetsgrader. *Kungliga Lantbruksakademiens Handlingar och Tidskrift* **50(2)**: 132–158 (in Swedish).

Atterberg A (1911b) Die plastizität der tone. *Internationale Mitteilungen für Bodenkunde* **1**: 4–37 (in German).

Ayothiraman R, Sahu R and Bhuyan P (2022) Strength and deformation behavior of fine-grained soils reinforced with hair fibers and its application in pavement design. *Journal of Natural Fibers* **19(14)**: 7646–7663, https://doi.org/10.1080/15440478.2021.1954129.

Bandini P and Al Shatnawi HH (2017) Discussion of 'Fines classification based on sensitivity to pore-fluid chemistry' by Junbong Jang and J. Carlos Santamarina. *Journal of Geotechnical and Geoenvironmental Engineering* **143(7)**: 07017011, https://doi.org/10.1061/(ASCE)GT.1943-5606.0001691.

Barnes GE (2009) An apparatus for the plastic limit and workability of soils. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **162(3)**: 175–185, https://doi.org/10.1680/geng.2009.162.3.175.

Barnes GE (2013a) An apparatus for the determination of the workability and plastic limit of clays. *Applied Clay Science* **80–81**: 281–290, https://doi.org/10.1016/j.clay.2013.04.014.

Barnes GE (2013b) *The Plastic Limit and Workability of Soils*. PhD thesis, University of Manchester, Manchester, UK.

Barnes GE (2021) A review of the plastic limit test by means of rolling paths. *Geotechnical Testing Journal* **44(6)**: 1658–1677, https://doi.org/10.1520/GTJ20210059.

Barnes GE and O'Kelly BC (2011) Discussion: An apparatus for the plastic limit and workability of soils. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **164(4)**: 293–294, https://doi.org/10.1680/geng.2011.164.4.293.

Belviso R, Ciampoli S, Cotecchia V and Federico A (1985) Use of cone penetrometer to determine consistency limits. *Ground Engineering* **18(5)**: 21–22.

Bharat TV, Yadav H, Mahaur JP and Kushwaha S (2020) Effect of aging time on consistency limits of bentonites. *Geotechnical and Geological Engineering* **38(4)**: 3737–3749, https://doi.org/10.1007/s10706-020-01251-3.

Bobrowski LJ and Griekspoor DM (1992) Determination of the plastic limit of a soil by means of a rolling device. *Geotechnical Testing Journal* **15(3)**: 284–28, https://doi.org/10.1520/GTJ10025J.

BSI (1990a) BS 1377-1:1990: Methods of test for soils for civil engineering purposes. General requirements and sampling. BSI, London, UK.

BSI (1990b) BS 1377-2:1990: Methods of test for soils for civil engineering purposes. Classification tests. BSI, London, UK.

BSI (2018a) BS EN ISO 14688-2:2018: Geotechnical investigation and testing. Identification and classification of soil. Principles for a classification. BSI, London, UK.

BSI (2018b) BS EN ISO 17892-12:2018: Geotechnical investigation and testing. Laboratory testing of soil. Determination of liquid and plastic limits. BSI, London, UK.

Budhu M (1985) The effect of clay content on liquid limit from a fall cone and the British cup device. *Geotechnical Testing Journal* **8(2)**: 91–95, https://doi.org/10.1520/GTJ10515J.

Cafaro F (2002) Metastable states of silty clays during drying. *Canadian Geotechnical Journal* **39(4)**: 992–999, https://doi.org/10.1139/t02-039.

Casagrande A (1932) Research on the Atterberg limits of soils. *Public Roads* **13(8)**: 121–136.

Casagrande A (1948) Classification and identification of soils. *Transactions of the American Society of Civil Engineers* **113(1)**: 901–930.

Casagrande A (1958) Notes on the design of the liquid limit device. *Géotechnique* **8(2)**: 84–91, https://doi.org/10.1680/geot.1958.8.2.84.

Dastider AG, Chatterjee S and Basu P (2021) Advancement in estimation of undrained shear strength through fall cone tests. *Journal of Geotechnical and Geoenvironmental Engineering* **147(7)**: 04021047, https://doi.org/10.1061/(ASCE)GT.1943-5606.0002535.

de Oliveira Modesto C and Bernardin AM (2008) Determination of clay plasticity: indentation method versus Pfefferkorn method. *Applied Clay Science* **40(1–4)**: 15–19, https://doi.org/10.1016/j.clay.2007.06.007.

Di Matteo L, Dragoni W, Cencetti C, Ricco R and Fucsina A (2016) Effects of fall-cone test on classification of soils: some considerations from study of two engineering earthworks in Central Italy. *Bulletin of Engineering Geology and the Environment* **75(4)**: 1629–1637, https://doi.org/10.1007/s10064-015-0808-8.

Dolinar B and Trauner L (2004) Liquid limit and specific surface of clay particles. *Geotechnical Testing Journal* **27(6)**: 580–584, https://doi.org/10.1520/GTJ11325.

Dragoni W, Prosperini N and Vinti G (2008) Some observations on the procedures for the determination of the liquid limit: an application on Plio-Pleistocenic clayey soils from Umbria region (Italy). *Italian Journal of Engineering Geology and Environment* **2008(Special Issue 1)**: 185–198, https://doi.org/10.4408/ijege.2008-01.s-12.

Fall DA (2000) A numerical model for rapid determination of plasticity of fine-grained soils. *Ground Engineering* **33(9)**: 43–45.

Fang HY (1960) Rapid determination of liquid limit of soils by flow index method. *Highway Research Board Bulletin* **254**: 30–35.

Federico A (1983) Relationships (c_u–w) and (c_u–δ) for remolded clayey soils at high water content. *Rivista Italiana di Geotecnica* **17(1)**: 38–41.

Feng TW (2000) Fall cone penetration and water content relationship of clays. *Géotechnique* **50(2)**: 181–187, https://doi.org/10.1680/geot.2000.50.2.181.

Feng TW (2001) A linear log d–log w model for the determination of consistency limits of soils. *Canadian Geotechnical Journal* **38(6)**: 1335–1342, https://doi.org/10.1139/t01-061.

Feng TW (2004) Using a small ring and a fall-cone to determine the plastic limit. *Journal of Geotechnical and Geoenvironmental Engineering* **130(6)**: 630–635, https://doi.org/10.1061/(ASCE)1090-0241(2004)130:6(630).

Haigh SK (2012) Mechanics of the Casagrande liquid limit test. *Canadian Geotechnical Journal* **49(9)**: 1015–1023, https://doi.org/10.1139/t2012-06.

Haigh SK (2016) Consistency of the Casagrande liquid limit test. *Geotechnical Testing Journal* **39(1)**: 13–19, https://doi.org/10.1520/GTJ20150093.

Haigh SK and Vardanega PJ (2014) Fundamental basis of single-point liquid limit measurement approaches. *Applied Clay Science* **102**: 8–14, https://doi.org/10.1016/j.clay.2014.10.011.

Haigh SK, Vardanega PJ and Bolton MD (2013) The plastic limit of clays. *Géotechnique* **63(6)**: 435–440, https://doi.org/10.1680/geot.11.P.123.

Haigh SK, Vardanega PJ, Bolton MD and Barnes GE (2014) Discussion: The plastic limit of clays. *Géotechnique* **64(7)**: 584–586, https://doi.org/10.1680/geot.13.D.06.

Haigh SK, Vardanega PJ and O'Kelly BC (2021) Discussion of 'Factors influencing undrained strength of fine-grained soils at high water contents' by H.B. Nagaraj, M.V. Sravan and B. S. Deepa. *Geomechanics and Geoengineering* **16(5)**: 417–419, https://doi.org/10.1080/17486025.2019.1674453.

Hansbo S (1957) *A New Approach to the Determination of the Shear Strength of Clay by the Fall Cone Test*. lvar Hæggströms Boktryckeri AB, Stockholm, Sweden, Royal Swedish Geotechnical Institute Proceedings No. 14.

Harison JA (1988) Using the BS cone penetrometer for the determination of the plastic limits of soils. *Géotechnique* **38(3)**: 433–438, https://doi.org/10.1680/geot.1988.38.3.433.

Houlsby GT (1982) Theoretical analysis of the fall cone test. *Géotechnique* **32(2)**: 111–118, https://doi.org/10.1680/geot.1982.32.2.111.

Hrubesova E, Lunackova B and Mohyla M (2020) Mohajerani method: tool for determining the liquid limit of soils using fall cone test results with strong correlation with the Casagrande test. *Engineering Geology* **278**: article 105852, https://doi.org/10.1016/j.enggeo.2020.105852.

Kannan G, O'Kelly BC and Sujatha ER (2023) Effect of chitin, chitosan and NaCMC biopolymers on the consistency limits of organic silt. *International Journal of Environmental Science and Technology*, https://doi.org/10.1007/s13762-023-05022-4.

Karakan E (2023) Flow index-liquid limit relationship by fall-cone tests in clay–sand mixtures. *Engineering Science and Technology, an International Journal* **41**: article 101405, https://doi.org/10.1016/j.jestch.2023.101405.

Karlsson R (1961) Suggested improvements in the liquid limit test, with reference to flow properties of remoulded clays. In *Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, France*. Dunod, Paris, France, vol. 1, pp. 171–184.

Koumoto T and Houlsby GT (2001) Theory and practice of the fall cone test. *Géotechnique* **51(8)**: 701–712, https://doi.org/10.1680/geot.2001.51.8.701.

Kulhawy FH and Mayne PW (1990) *Manual on Estimating Soil Properties for Foundation Design*. Electric Power Research Institute, Palo Alto, CA, USA, Report No. EL-6800.

Kyambadde BS and Stone KJL (2012) Index and strength properties of clay–gravel mixtures. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **165(1)**: 13–21, https://doi.org/10.1680/geng.2012.165.1.13.

Kyambadde BS, Stone KJL and Barnes GE (2014) Discussion: Index and strength properties of clay–gravel mixtures. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **167(1)**: 83–86, https://doi.org/10.1680/geng.12.00116.

Ladd CC and Foott R (1974) New design procedure for stability of soft clays. *Journal of the Geotechnical Engineering Division* **100(7)**: 763–786, https://doi.org/10.1061/AJGEB6.000006.

Lee LT and Freeman RB (2009) Dual-weight fall cone method for simultaneous liquid and plastic limit determination. *Journal of Geotechnical and Geoenvironmental Engineering* **135(1)**: 158–161, https://doi.org/10.1061/(ASCE)1090-0241(2009)135:1(158).

Llano-Serna MA and Contreras LF (2020) The effect of surface roughness and shear rate during fall-cone calibration. *Géotechnique* **70(4)**: 332–342, https://doi.org/10.1680/jgeot.18.P.222.

Llano-Serna MA, Contreras LF and O'Kelly BC (2022) Discussion: The effect of surface roughness and shear rate during fall-cone calibration. *Géotechnique* **72(10)**: 935–937, https://doi.org/10.1680/jgeot.20.D.003.

Marinho FAM and Oliveira OM (2012) Unconfined shear strength of compacted unsaturated plastic soils. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **165(2)**: 97–106, https://doi.org/10.1680/geng10.00027.

Marinho FAM and Pinto CS (2000) Discussion: Use of liquid limit state to generalize water retention properties of fine-grained soils. *Géotechnique* **50(3)**: 295–296, https://doi.org/10.1680/geot.2000.50.3.295.

Medhat F and Whyte IL (1986) An appraisal of soil index tests. In *Site Investigation Practice: Assessing BS 5930* (Hawkins AB (ed.)). The Geological Society, London, UK, vol. 2, pp. 317–323.

Moreno-Maroto JM and Alonso-Azcárate J (2015) An accurate, quick and simple method to determine the plastic limit and consistency changes in all types of clay and soil: the thread-bending test. *Applied Clay Science* **114**: 497–508, https://doi.org/10.1016/j.clay.2015.06.037.

Moreno-Maroto JM and Alonso-Azcárate J (2018) What is clay? A new definition of 'clay' based on plasticity and its impact on the most widespread soil classification systems. *Applied Clay Science* **161**: 57–63, https://doi.org/10.1016/j.clay.2018.04.011.

Moreno-Maroto JM, Alonso-Azcárate J and O'Kelly BC (2021) Review and critical examination of fine-grained soil classification systems based on plasticity. *Applied Clay Science* **200**: article 105955, https://doi.org/10.1016/j.clay.2020.105955.

Murray I and Tarantino A (2019) Mechanisms of failure in saturated and unsaturated clayey geomaterials subjected to (total) tensile stress. *Géotechnique* **69(8)**: 701–712, https://doi.org/10.1680/jgeot.17.P.252.

Nagaraj TS and Srinivasa Murthy BR (1987) Consistency limits of soils – principles and potentials. In *Civil Engineering Practic,* vol. 3. *Geotechnical/Ocean Engineering* (Cheremisinoff PN, Cheremisinoff NP and Cheng SL (eds)). Technomic Publishing, Lancaster, PA, USA, pp. 23–44.

Nagaraj HB, Sridharan A and Mallikarjuna HM (2012) Re-examination of undrained strength at Atterberg limits water contents. *Geotechnical and Geological Engineering* **30(4)**: 727–736, https://doi.org/10.1007/s10706-011-9489-7.

O'Kelly BC (2013a) Atterberg limits and remolded shear strength–water content relationships. *Geotechnical Testing Journal* **36(6)**: 939–947, https://doi.org/10.1520/GTJ20130012.

O'Kelly BC (2013b) Undrained shear strength–water content relationship for sewage sludge. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **166(6)**: 576–588, http://doi.org/10.1680/geng.11.00016.

O'Kelly BC (2014) Characterisation and undrained strength of amorphous clay. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **167(3)**: 311–320, https://doi.org/10.1680/geng.11.00025.

O'Kelly BC (2015a) Atterberg limits are not appropriate for peat soils. *Geotechnical Research* **2(3)**: 123–134, https://doi.org/10.1680/jgere.15.00007.

O'Kelly BC (2015b) Effective stress strength testing of peat. *Environmental Geotechnics* **2(1)**: 33–44, https://doi.org/10.1680/envgeo.13.00112.

O'Kelly BC (2016a) Briefing: Atterberg limits and peat. *Environmental Geotechnics* **3(6)**: 359–363, https://doi.org/10.1680/envgeo.15.00003.

O'Kelly BC (2016b) Geotechnics of municipal sludges and residues for landfilling. *Geotechnical Research* **3(4)**: 148–179, https://doi.org/10.1680/jgere.16.00013.

O'Kelly BC (2018) Fall-cone strength testing of municipal sludges and residues. *Environmental Geotechnics* **5(1)**: 18–30, https://doi.org/10.1680/jenge.15.00080.

O'Kelly BC (2019a) Fallacy of wide undrained strength range at the Casagrande liquid limit. *Geotechnical Research* **6(3)**: 205–217, https://doi.org/10.1680/jgere.18.00040.

O'Kelly BC (2019b) Reappraisal of soil extrusion for geomechanical characterisation. *Geotechnical Research* **6(4)**: 265–287, https://doi.org/10.1680/jgere.19.00006.

O'Kelly BC (2021) Review of recent developments and understanding of Atterberg limits determinations. *Geotechnics* **1(1)**: 59–75, https://doi.org/10.3390/geotechnics1010004.

O'Kelly BC (2022a) Appraisal of novel power-based extrusion methodology for consistency limits determinations of fine-grained soils. In *Proceedings of the Civil Engineering Research in Ireland 2022 Conference, Dublin, Ireland* (Holmes N, de Paor C and West RP (eds)). Civil Engineering Research Association of Ireland, Dublin, Ireland, vol. 1, pp. 317–322.

O'Kelly BC (2022b) Discussion of 'Physio-chemical properties, consolidation, and stabilization of tropical peat soil using traditional soil additives – a state of the art literature review' by Afnan Ahmad, Muslich Hartadi Sutanto, Mohammed Ali Mohammed Al-Bared, Indra Sati Hamonangan Harahap, Seyed Vahid Alavi Nezhad Khalil Abad, Mudassir Ali Khan. *KSCE Journal of Civil Engineering* **26(8)**: 3455–3459, https://doi.org/10.1007/s12205-022-2313-5.

O'Kelly BC (2022c) Discussion of 'Advancement in estimation of undrained shear strength through fall cone tests' by Abhishek Ghosh Dastider, Santiram Chatterjee, and Prasenjit Basu. *Journal of Geotechnical and Geoenvironmental Engineering* **148(6)**: 07022005, https://doi.org/10.1061/(ASCE)GT.1943-5606.0002807.

O'Kelly BC (2023) Discussion of "Practical transitions among undrained shear strengths of remolded samples from pocket penetrometer tests and other laboratory tests" by Budak T.O., Gurbuz A. and Eksioglu B. [Catena 213 (2022) 106148]. *Catena* **288**: article 107129, https://doi.org/10.1016/j.catena.2023.107129.

O'Kelly BC and Orr TLL (2014) Briefing: Effective-stress strength of peat in triaxial compression. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **167(5)**: 417–420, https://doi.org/10.1680/geng.13.00143.

O'Kelly BC and Soltani A (2021a) Discussion: Determining the plasticity properties of high plastic clays: a new empirical approach [Arab J Geosci (2020) 13 (11), 394]. *Arabian Journal of Geosciences* **14(8)**: article 715, https://doi.org/10.1007/s12517-021-06757-5.

O'Kelly BC and Soltani A (2021b) Discussion: A comparative study on the application of artificial intelligence networks versus regression analysis for the prediction of clay plasticity [Arab J Geosci (2021) 14(7), 534]. *Arabian Journal of Geosciences* **14(20)**: article 2150, https://doi.org/10.1007/s12517-021-08566-2.

O'Kelly BC and Soltani A (2022a) Comments on 'Strength and deformation behavior of fine-grained soils reinforced with hair fibers and its application in pavement design' by Ayothiraman et al. [Journal of Natural Fibers, DOI: 10.1080/15440478.2021.1954129]. *Journal of Natural Fibers* **19(17)**: 15846–15850, https://doi.org/10.1080/15440478.2022.2134260.

O'Kelly BC and Soltani A (2022b) Machine learning techniques for relating liquid limit obtained by Casagrande cup and fall cone test in low-medium plasticity fine grained soils [Eng. Geol. (2021) 294, 106381]. *Engineering Geology* **306**: article 106746, https://doi.org/10.1016/j.enggeo.2022.106746.

O'Kelly BC and Soltani A (2023a) Discussion: Development of a single-point method to determine soil plastic limit using fall-cone data [Geotech Geol Eng 41:4473–4485, 2023]. *Geotechnical and Geological Engineering*, https://doi.org/10.1007/s10706-023-02679-z.

O'Kelly BC and Soltani A (2023b) Discussion of 'Factors influencing undrained strength of fine-grained soils at high water contents' [Geomechanics and Geoengineering 13(4), 276–287]. *Geomechanics and Geoengineering* **18(2)**: 170–174, https://doi.org/10.1080/17486025.2021.2015457.

O'Kelly BC, Vardanega PJ and Haigh SK (2018) Use of fall cones to determine Atterberg limits: a review. *Géotechnique* **68(10)**: 843–856, https://doi.org/10.1680/jgeot.17.r.039.

O'Kelly BC, Oettle NK and Ramos JA (2020a) Geotechnical properties of compacted biosolids for monofill design, As-Samra, Jordan. *Environmental Geotechnics* **7(6)**: 404–434, https://doi.org/10.1680/jenge.17.00107.

O'Kelly BC, Vardanega PJ, Haigh SK and Barnes GE (2020b) Discussion: Use of fall cones to determine Atterberg limits: a review. *Géotechnique* **70(7)**: 647–651, https://doi.org/10.1680/jgeot.19.D.003.

O'Kelly BC, Vardanega PJ and Haigh SK (2022a) Discussion of 'Mohajerani method: tool for determining the liquid limit of soils using fall cone test results with strong correlation with the Casagrande test' by E. Hrubesova, B. Lunackova and M. Mohyla [Engineering Geology 278(2020) 105852]. *Engineering Geology* **302**: article 106623, https://doi.org/10.1016/j.enggeo.2022.106623.

O'Kelly BC, Moreno-Maroto JM and Alonso-Azcárate J (2022b) Discussion of 'Determining soil plasticity utilizing Manafi method and apparatus' by Masoud S.G. Manafi, An Deng, Abbas Taheri, Mark B. Jaksa, and Nagaraj HB, published in Geotechnical Testing Journal 45, no. 4 (2022): 797–818. *Geotechnical Testing Journal* **45(6)**: 1144–1150, https://doi.org/10.1520/GTJ20220093.

O'Kelly BC, Alonso-Azcárate J and Moreno-Maroto JM (2023) A comprehensive review of soil remolding toughness determination and its use in the classification of fine-grained soils. *Applied Sciences* **13**: article 5711, https://doi.org/10.3390/app13095711.

Omer B, Kalpakcı V and Ali HH (2022) A research on consistency limits, compaction, and strength properties of sheep wool–fine-grained soil mixtures. *Arabian Journal of Geosciences* **15(1)**: article 77, https://doi.org/10.1007/s12517-021-09397-x.

Özer M (2009) Comparison of liquid limit values determined using the hard and soft base Casagrande apparatus and the cone penetrometer. *Bulletin of Engineering Geology and the Environment* **68(3)**: 289–296, https://doi.org/10.1007/s10064-009-0191-4.

Prakash K and Sridharan A (2004) Free swell ratio and clay mineralogy of fine-grained soils. *Geotechnical Testing Journal* **27(2)**: 220–225, https://doi.org/10.1520/gtj10860.

Prakash K and Sridharan A (2006) Critical appraisal of the cone penetration method of determining soil plasticity. *Canadian Geotechnical Journal* **43(8)**: 884–888, https://doi.org/10.1139/t06-043.

Prakash K, Sridharan A and Prasanna HS (2009) A note on the determination of plastic limit of fine-grained soils. *Geotechnical Testing Journal* **32(4)**: 372–374, https://doi.org/10.1520/GTJ101960.

Sampson LR and Netterberg F (1985) The cone penetration index: a simple new soil index test to replace the plasticity index. In *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering* (Publications Committee of the XII ICSMFE (eds)). Balkema, Rotterdam, the Netherlands, 2nd edn., pp. 1041–1048.

Sharma B (2012) Discussion of 'Re-examination of undrained strength at Atterberg limits water contents' by H.B. Nagaraj, A. Sridharan, and H.M. Mallikarjuna. *Geotechnical and Geological Engineering* **30(4)**: 1035–1036, https://doi.org/10.1007/s10706-012-9515-4.

Sharma B and Bora PK (2003) Plastic limit, liquid limit and undrained shear strength of soil – reappraisal. *Journal of Geotechnical and Geoenvironmental Engineering* **129(8)**: 774–777, https://doi.org/10.1061/(ASCE)1090-0241(2003)129:8(774).

Sherwood PT (1970) *The Reproducibility of the Results of Soil Classification and Compaction Tests*. Department of Transport, London, UK, Transport and Road Research Laboratories Report LR 339.

Sherwood PT and Ryley MD (1970) An investigation of a cone-penetrometer method for the determination of the liquid limit. *Géotechnique* **20(2)**: 203–208, https://doi.org/10.1680/geot.1970.20.2.203.

Shimobe S and Spagnoli G (2019) A global database considering Atterberg limits with the Casagrande and fall-cone tests. *Engineering Geology* **260**: article 105201, https://doi.org/10.1016/j.enggeo.2019.105201.

Sivakumar V, Glynn D, Cairns P and Black JA (2009) A new method of measuring plastic limit of fine materials. *Géotechnique* **59(10)**: 813–823, https://doi.org/10.1680/geot.2009.59.10.813.

Sivakumar V, O'Kelly BC, Henderson L, Moorhead C and Chow SH (2015) Measuring the plastic limit of fine soils: an experimental study. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **168(1)**: 53–64, https://doi.org/10.1680/geng.14.00004.

Sivakumar V, O'Kelly BC, Henderson L *et al.* (2016) Discussion: Measuring the plastic limit of fine soils: an experimental study. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **169(1)**: 83–85, https://doi.org/10.1680/jgeen.15.00068.

Sivapullaiah PV and Sridharan A (1985) Liquid limit of soil mixtures. *Geotechnical Testing Journal* **8(3)**: 111–116, https://doi.org/10.1520/gtj10521j.

Soltani A and O'Kelly BC (2020) Discussion of 'The flow index of clays and its relationship with some basic geotechnical properties' by G. Spagnoli, M. Feinendegen, L. Di Matteo, and D.A. Rubinos, published in Geotechnical Testing Journal 42, no. 6 (2019): 1685–1700. *Geotechnical Testing Journal* **44(1)**: 216–219, https://doi.org/10.1520/gtj20190423.

Soltani A and O'Kelly BC (2021) Reappraisal of the ASTM/AASHTO standard rolling device method for plastic limit determination of fine-grained soils. *Geosciences* **11(6)**: article 247, https://doi.org/10.3390/geosciences11060247.

Soltani A and O'Kelly BC (2022) Reappraisal of fall-cone flow curve for soil plasticity determinations. *Geotechnical Testing Journal* **45(1)**: 225–243, https://doi.org/10.1520/GTJ20200312.

Soltani A, Azimi M and O'Kelly BC (2023) Reappraisal of linear shrinkage test for plasticity index determination and classification of fine-grained soils. *Applied Clay Science* **238**: article 106920, https://doi.org/10.1016/j.clay.2023.106920.

Spagnoli G, Feinendegen M, Di Matteo L and Rubinos DA (2019) The flow index of clays and its relationship with some basic geotechnical properties. *Geotechnical Testing Journal* **42(6)**: 1685–1700, https://doi.org/10.1520/gtj20180110.

Sridharan A and Prakash K (2000) Percussion and cone methods of determining the liquid limit of soils: controlling mechanisms. *Geotechnical Testing Journal* **23(2)**: 236–244, https://doi.org/10.1520/GTJ11048J.

Sridharan A, Nagaraj HB and Prakash K (1999) Determination of the plasticity index from flow index. *Geotechnical Testing Journal* **22(2)**: 169–175, https://doi.org/10.1520/GTJ11276J.

Stone KJL and Kyambadde BS (2007) Determination of strength and index properties of fine-grained soils using a soil minipenetrometer. *Journal of Geotechnical and Geoenvironmental Engineering* **133(6)**: 667–673, https://doi.org/10.1061/(ASCE)1090-0241(2007)133:6(667).

Stone KJL and Phan CD (1995) Cone penetration tests near the plastic limit. *Géotechnique* **45(1)**: 155–158, https://doi.org/10.1680/geot.1995.45.1.155.

Sujatha ER and O'Kelly BC (2023) Biopolymer based soil treatment for geotechnical engineering applications. In *Handbook of Biopolymers* (Thomas S, AR A, Jose Chirayil C and Thomas B (eds)). Springer, Singapore, pp. 609–626.

Temyingyong A, Chantawarangul K and Sudasna-na-Ayudthya P (2002) Statistical analysis of influenced factors affecting the plastic limit of soils. *Agriculture and Natural Resources* **36(1)**: 98–102.

Terzaghi K (1926a) Simplified soil tests for subgrades and their physical significance. *Public Roads* **7(8)**: 153–170.

Terzaghi K (1926b) Principles of final soil classification. *Public Roads* **8(3)**: 41–53.

Timár A (1974) Testing the plastic properties of cohesive- and intermediate-type soils by extrusion. *Acta Technica Academiae Scientiarum Hungaricae* **76(3–4)**: 355–370.

Trauner L, Dolinar B and Misic M (2005) Relationship between the undrained shear strength, water content, and mineralogical properties of fine-grained soils. *International Journal of Geomechanics* **5(4)**: 350–355, https://doi.org/10.1061/(ASCE)1532-3641(2005)5:4(350).

Vardanega PJ and Haigh SK (2014) The undrained strength–liquidity index relationship. *Canadian Geotechnical Journal* **51(9)**: 1073–1086, https://doi.org/10.1139/cgj-2013-016.

Vardanega PJ, O'Kelly BC and Haigh SK (2020) Discussion of 'Reclaimed lignin-stabilized silty soil: undrained shear strength, Atterberg limits, and microstructure characteristics' by Tao Zhang, Guojun Cai, and Songyu Liu. *Journal of Materials in Civil Engineering* **32(3)**: 07020001, https://doi.org/10.1061/(ASCE)MT.1943-5533.0003064.

Vardanega PJ, Haigh SK and O'Kelly BC (2022) Use of fall-cone flow index for soil classification: a new plasticity chart. *Géotechnique* **72(7)**: 610–617, https://doi.org/10.1680/jgeot.20.p.132.

Vardanega PJ, Haigh SK, O'Kelly BC *et al.* (2023) Discussion: Use of fall-cone flow index for soil classification: a new plasticity chart. *Géotechnique* **73(7)**: 648–654, https://doi.org/10.1680/jgeot.21.00268.

Vinod P, Deepa KA and Sridharan A (2013) Remoulded shear strength at plastic and semi-solid states. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **166(4)**: 415–424, https://doi.org/10.1680/geng.11.00071.

Wasti Y and Bezirci MH (1986) Determination of the consistency limits of soils by the fall-cone test. *Canadian Geotechnical Journal* **23(2)**: 241–246, https://doi.org/10.1139/t86-033.

Whyte IL (1982) Soil plasticity and strength – a new approach using extrusion. *Ground Engineering* **15(1)**: 16–24.

Wintermeyer AM (1926) Adaption of Atterberg plasticity tests for subgrade soils. *Public Roads* **7(6)**: 119–122.

Wood DM (1985) Some fall-cone tests. *Géotechnique* **35(1)**: 64–68, https://doi.org/10.1680/geot.1985.35.1.64.

Wood DM (1990) *Soil Behaviour and Critical State Soil Mechanics*. Cambridge University Press, Cambridge, UK.

Wroth CP (1979) Correlations of some engineering properties of soils. In *Proceedings of the 2nd International Conference on the Behaviour of Off-shore Structures* (Stephens HS and Knight SM (eds)). British Hydromechanics Research Association, Fluids Engineering, Cranfield, UK, vol. 1, pp. 121–132.

Wroth CP and Wood DM (1978) The correlation of index properties with some basic engineering properties of soils. *Canadian Geotechnical Journal* **15(2)**: 137–145, https://doi.org/10.1139/t78-014.

Xu S, Lai J, O'Kelly BC and Zhao B (2023) Reverse extrusion test for fine-grained soil characterisation: internal flow pattern with ANN-enhanced particle tracking. *Proceedings of the Fourth International Symposium on Machine Learning and Big Data in Geoscience, Cork, Ireland,* extended abstract 94.

Youssef MS, El Ramli AH and El Demery M (1965) Relationships between shear strength, consolidation, liquid limit and plastic limit for remolded clays. In *Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering*. University of Toronto Press, Toronto, ON, Canada, vol. 1, pp. 126–129.

How can you contribute?

To discuss this paper, please submit up to 500 words to the editor at support@emerald.com. Your contribution will be forwarded to the author(s) for a reply and, if considered appropriate by the editorial board, it will be published as a discussion in a future issue of the journal.