This paper reviews the percussion-cup liquid limit, thread-rolling plastic limit (PL) and various fall-cone and other approaches employed for consistency limit determinations on fine-grained soil, highlighting their use and misuse for soil classification purposes and in existing correlations. As the PL does not correspond to a unique value of remoulded undrained shear strength, there is no scientific reason why PL measurements obtained using the thread-rolling and shear-strength-based fall-cone or extrusion methods should coincide. Various correlations are established relating liquid limit values deduced using the percussion-cup and fall-cone approaches. The significance of differences in the strain-rate dependency on the mobilised fall-cone shear strength is reviewed. The paper concludes with recommendations on the standardisation of international codes and the wider use of the fall-cone approach for soft to medium-stiff clays in establishing the strength variability with changing water content and further index parameters.

**KEYWORDS:** clays; laboratory equipment; laboratory tests; soil classification; silts; shear strength

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**INTRODUCTION**

The liquid limit (LL) and plastic limit (PL) tests are among the most commonly specified tests in the geotechnical engineering industry and originate from the original research of Atterberg (1911a, 1911b), which was subsequently standardised for use in civil engineering applications by Terzaghi (1926a, 1926b) and Casagrande (1932, 1958), and adopted for the classification of fine-grained soils. These Atterberg limits have been used for numerous purposes, including the estimation of shear strength, deformation and critical-state soil mechanics parameter values (e.g. Skempton, 1944, 1954, 1957; Karlsson & Viberg, 1967; Stroud, 1974; Wroth & Wood, 1978; Wroth, 1979; Carrier & Beckman, 1984; Larsson et al., 1987; Nakase et al., 1988; Wood, 1990; Tripathy & Mishra, 2011; Sorensen & Okkels, 2013; Farias & Llano-Serna, 2016).

The coincidence of Atterberg limit values obtained using different testing methods has been a subject of considerable discussion. This paper begins by defining the various consistency limit parameters, their measurement methods and associated problems. The significance of differences in operator performance and judgement in PL determinations from the rolling out of soil threads is assessed in terms of some established correlations with the consistency limits. Alternative methods for PL determination are reviewed, including various fall-cone approaches, but because these are shear strength-based they do not measure the onset of brittleness and hence cannot measure the true PL. The significance of plausible differences in the strain-rate dependency on the mobilised fall-cone shear strength for different test soils is demonstrated. Various correlations are established relating LL values deduced using the main measurement techniques and standards, such that discrepancies between the different LL measures can be taken into account when these are substantial. The paper concludes with recommendations on the standardisation of international codes and the wider use of the fall-cone approach as appropriate for soft to medium-stiff clays in establishing the variability of shear strength with changing water content and further index parameters.

**Consistency limits**

Figure 1 shows schematically the relative locations of various index parameters positioned on the scale of water content, with their indicative remoulded undrained shear strength ranges presented in Fig. 2. A logarithmic scale is used in Fig. 2 for undrained shear strength, as the correlation between the increase in undrained shear strength with reducing water content for a given soil can be derived from a semi-logarithmic plot or, alternatively, from a bi-logarithmic plot (after Kodikara et al., 1986, 2006). Each of these parameters is defined and their relative merit discussed in the following sections.

**Liquid limit**

Notionally the LL of a soil is the water content at which it transitions from liquid to plastic behaviour. As the soil never has zero shear strength, the LL is determined as the water content associated with an arbitrarily chosen (low) shear strength on a continuum of ever-weakening behaviour with increasing water content. The LL value is strongly dependent on the soil grading, composition and mineralogical properties, particularly those of the clay fraction, and also the quantity of interlayer water in the case of expanding clay minerals (e.g. Wood, 1990; Dolinar & Trauner, 2004; Trauner et al., 2005).

As the LL is only precisely defined by the test used to measure it, rather than representing some sudden change in behaviour, the value obtained for the LL is dependent on...
hardness test
soil
variability exists even within each of these categories.

distinguished as soft- and hard-base devices, considerable
countries (Haigh, 2016). While such devices are often
approach of mandating the range of devices in use in their
standardised, each country appears to have taken the
penetration depth (\(d\)), relying on the work of Hansbo (1957), who related
the technique used to measure it. This is problematic owing
to the lack of worldwide standardisation of LL techniques
and equipment. Two techniques, the Casagrande percussion-
cup and fall-cone (cone penetrometer) methods have been
adopted as the standard measurement approaches, with the
former favoured in the USA (AASHTO, 2010; ASTM, 2010)
and the latter adopted as the preferred approach in the UK
(BSI, 1990) and by Eurocode 7 (BSI, 2007).

Within each of these two methods further variation exists.
Casagrande (1958) bemoaned the lack of standardisation
in percussion-cup device bases in use at that time, two decades
after the test was introduced, saying 'Unfortunately, no effort
was made to specify the [base] hardness by a standard
hardness test' (Casagrande, 1958: p. 85). When the test was
standardised, each country appears to have taken the
approach of mandating the range of devices in use in their
country at that time, leading to a wide variety of base
hardness and resilience values being specified for the
percussion-cup device, with no standardisation between
countries (Haigh, 2016). While such devices are often
distinguished as soft- and hard-base devices, considerable
variability exists even within each of these categories.

The fall-cone test is essentially an assessment of soil shear
strength, relying on the work of Hansbo (1957), who related
the penetration depth (\(d\)) of a fall-cone of weight \(W\) to the
soil's undrained shear strength by way of

\[
\text{\(s_{\text{dFC}} = \frac{KW}{d^2}\)}
\]

where \(K\) is the fall-cone factor.

The effect of cone angle on the \(K\) factor from equation (1)
(and by definition the computed undrained shear strength)
has been studied by various researchers (e.g. Houlbsy, 1982;<br>

The fall-cone LL test suffers from less variability in
equipment and execution than the Casagrande cup test,
with most localities utilizing a standard 30°–80 g cone
penetrating 20 mm at LL (i.e. LL\(_{\text{FC}}\)), this corresponding to
an undrained shear strength of approximately 1·7 kPa (cf.<br>
Wroth & Wood, 1978). Other cone angles and masses have
been used, such as the 'Swedish cone' (i.e. 60°–80 g cone
penetrating 10 mm at LL\(_{\text{FC}}\) (e.g. Karlsson, 1961)), which was also advocated by Koumoto & Houslsby (2001).

'Non-standard' cones have been reported; for example, a
30°–148 g cone was used in the study of Sivapullaiah &<br>Sridharan (1985). As with the Casagrande cup apparatus, the
variations in the fall-cone LL approaches specified in
different codes (involving cones of different masses and
apex angles, with the index property value usually deduced
for different cone penetration depths) means that the
undrained shear strength assumed for the fall-cone LL
condition varies somewhat between different codes (cf.<br>Budhu, 1985; Leroueil & Le Bihan, 1996; Koumoto &<br>Houslsby, 2001).

**Plastic limit**

The PL of a soil is the water content at which it transitions
from ductile to brittle behaviour. Unlike the LL, this is a
sudden definite change in behaviour that could, in theory, be
measured with a variety of tests, each of which would be
expected to give essentially the same result. The international
standard method for PL determination involves manually
rolling out a thread of soil on a glass plate until it crumbles at
a specified diameter (BSI, 1990; ASTM, 2010), possibly
being caused by air entry or cavitation within the soil thread
(Haigh et al., 2013). It has been shown that the thread
 diameter requirement for the crumbling condition – specified
as about 3·0 mm (BS 1377-2 (BSI, 1990)) or 3·2 mm (ASTM
D4318-10e1 (ASTM, 2010)) – is not critical, with no
statistically significant trend of varying water content with the
soil thread diameter at the crumbling condition (2–6 mm
range investigated) reported for a variety of mineral (Prakash<br>et al., 2009; Haigh et al., 2013, 2014) and organic (O'Kelly,<br>2015) soils.

**REPEATABILITY OF THE THREAD-ROLLING TEST**

It has been argued that the PL values deduced by the
thread-rolling method are overly dependent on operator
performance and judgement (e.g. Sherwood, 1970;<br>Sherwood & Ryley, 1970; Whyte, 1982; Belviso et al., 1985;<br>Sivakumar et al., 2009). To investigate this point, reported
PLs determined independently by four laboratories for 11
inorganic fine-grained soils of intermediate to very high
plasticity (see Table 1) were considered. The maximum
difference in the measured PLs for a given soil type was
8%, although Sherwood (1970) reported that the variation
for engineering practice can be up to 12%. Using the data in<br>Table 1, the significance of the maximum variation in the
present study for four established and widely used corre-
lations that make use of plasticity index \(I_p\) or \(I_{Lc}\)

\[(a) \text{ In situ undrained shear strength } (s_{\text{dFC,in situ}}) \text{ as a function of } I_p \text{ for normally consolidated soil given by} \]
USE OF FALL CONES TO DETERMINE ATTERBERG LIMITS: A REVIEW

Table 1. Liquid limits and PLs of soils obtained through different laboratories operating in Northern Ireland to BS EN 1377 (BSI, 1990)

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>LL_{FC}: %</th>
<th>Thread rolling PL: %</th>
<th>Average PL: %</th>
<th>Maximum difference: %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GSI</td>
<td>CPD</td>
<td>WF</td>
<td>QUB</td>
</tr>
<tr>
<td>Sleech</td>
<td>50</td>
<td>25</td>
<td>25</td>
<td>24</td>
</tr>
<tr>
<td>Belfast Clay</td>
<td>55</td>
<td>24</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>Oxford Clay</td>
<td>55</td>
<td>24</td>
<td>22</td>
<td>23</td>
</tr>
<tr>
<td>Canadian Clay</td>
<td>73</td>
<td>27</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Glacial till</td>
<td>36</td>
<td>17</td>
<td>17</td>
<td>16</td>
</tr>
<tr>
<td>Tennessee</td>
<td>72</td>
<td>28</td>
<td>33</td>
<td>35</td>
</tr>
<tr>
<td>Amphill</td>
<td>77</td>
<td>31</td>
<td>32</td>
<td>33</td>
</tr>
<tr>
<td>Donegal Clay</td>
<td>43</td>
<td>21</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>London Clay</td>
<td>71</td>
<td>28</td>
<td>27</td>
<td>30</td>
</tr>
<tr>
<td>Enniskillen</td>
<td>36</td>
<td>18</td>
<td>19</td>
<td>17</td>
</tr>
<tr>
<td>Kaolin</td>
<td>70</td>
<td>33</td>
<td>36</td>
<td>37</td>
</tr>
</tbody>
</table>

GSI, Glover Site Investigation Ltd; CPD, Central Precambient Division, NI; WF, Whiteford Geoservices; QUB, Queen’s University Belfast (table adapted from Sivakumar et al., 2015).

equation (2) (e.g. Skempton (1954, 1957), which was later validated by an extended database in Wood (1990) – albeit with more scatter being shown than originally present in the work of Skempton).

\[
\frac{s_{u_{\text{min}}}}{\sigma_0} = 0.11 + 0.37I_P
\]

where \(\sigma_0^*\) is the in situ vertical effective stress.

(b) Effective angle of shearing resistance as a function of logarithm \(I_P\) for normally consolidated reconstituted and undisturbed clays (equation (3), reported in Sørensen & Okkels (2013), based on a database of previously published data)

\[
\phi_u = 43 - 10\log_{10}(I_P) \quad (R^2 = 0.41, n = 233)
\]

(c) The empirical factor \((\alpha_{UV})\) used to obtain the overconsolidation ratio (OCR) from normalised field vane shear strength \((s_{u_{\text{adV}}}/\sigma_0^*)\) data presented in Mayne & Mitchell (1988)

\[
\alpha_{UV} = \frac{OCR}{s_{u_{\text{adV}}}/\sigma_0^*} = 2/(I_P)^{-0.48} \quad (n = 263)
\]

(d) Remoulded undrained shear strength as a function of liquidity index (equation (5), after Wroth & Wood (1978)).

\[
s_u (\text{kPa}) = 170 \exp(-4.6I_L)
\]

Based on the data in Table 1; for equations (2)-(4) which make use of \(I_P\) the percentage variation in \(s_{u_{\text{min}}}/\sigma_0^*\) from its mean value would range between 2.2% and 10.7% considering all 11 soils, with respective values of 0.33% and 1.72% for \(\phi_u\) and 1.1% and 5.2% for \(\alpha_{UV}\). In all cases considered, the minimum and maximum variations from the mean occurred for the Donegal Clay and kaolin material, respectively, with these examples demonstrating that depending on the correlation and soil type considered, the potential variation can be significant (e.g. in the case of the \(s_{u_{\text{min}}}/\sigma_0^*\) value for the kaolin), but in many correlations may not be. However, other correlations that make use of liquidity index (and activity) to evaluate other soil characteristics are likely to be influenced to a more significant degree. For instance, differentiating equation (5) gives

\[
\delta s_u/s_u = -4.6\delta I_L
\]

so that an error of, for instance, 0.1 in \(I_L\) would give rise to an error of 46% in the estimate of \(s_u\) (Wroth & Wood, 1978).

ALTERNATIVE APPROACHES FOR PL DETERMINATION

Mechanical thread rolling

Attempts to improve on the standard PL test include the thread-rolling methods proposed by Gay & Kaiser (1973) and Bobrowski & Griekspoor (1992), a mechanically adapted version of the Bobrowski and Griekspoor’s device (Temyingyong et al., 2002), and Barnes (2009, 2013a, 2013b). The Barnes’ apparatus can measure indicative stress and toughness values for the soil thread during the rolling out procedure, with control of the strain rate, but the added complexity introduced into the test generally does not substantially alter the results obtained for PL. Apart from the Bobrowski & Griekspoor (1992) approach (a thread-rolling device that comprises two flat plates covered with paper), which was subsequently adopted as a PL rolling device in ASTM (2010) and AASHTO (2000), none of the other proposed rolling methods have, to date, been adopted more widely. Further, the PLs obtained using the Bobrowski and Griekspoor device have been shown to generally underestimate the standard (thread-rolling) PLs (Bobrowski & Griekspoor, 1992; Rashid et al., 2008; Ishaque et al., 2010), most likely because the paper tends to lead to inhomogeneity of the soil thread, the outside becoming drier than its core, during the rolling out procedure.

Strength-based approaches

Many researchers have attempted to devise various strength-based approaches to the measurement of PL. These are, in general, based on the assumption of a 100-fold gain in strength between the LL and PL, as proposed by Wroth & Wood (1978). As evident from Fig. 3, the strength gain factor \((R_{SW})\) for the traditionally defined plastic range is generally significantly different (and more often than not substantially less) than the assumed 100-fold increase. Prakash (2005) and Nagaraj et al. (2012) cautioned against assigning a fixed strength value at PL. As explained in Haigh et al. (2013), the assumption of a 100-fold factor increase derives from the following passage in Schofield & Wroth (1968), who were examining the data of Skempton & Northey (1952) (shown in Fig. 4).

The experimental results with four different clays give similar variation of strength with liquidity index... From these data it appears that the liquid limit and plastic limit do correspond approximately to fixed strengths which are in the proposed ratio of 1:100 (Schofield & Wroth, 1968: p. 155).
Houston & Mitchell (1969) also recognised that variability of strength at PL was present (their bounds are shown also in Fig. 4). However (as reviewed in the papers by O’Kelly (2013) and Vardanega & Haigh (2014b)), the data of Skempton & Northey (1952) show variations in the $R_{MW}$ value, which ranged between 70 and 160 for the four soils considered. Whyte (1982) suggested $R_{MW}/C_2^{70}$. Vardanega & Haigh (2014b) demonstrated using a database of 101 soils that the ratio of computed undrained strengths from PL to LL was on average closer to 34·3 (when fall-cone undrained strength, $s_{uFC}$, was fitted to $I_L$) and 83·5 (when $s_{uFC}$ was fitted to logarithmic liquidity index). Simply based on analysis of historical data, as the ratio of strengths at the PL and LL varies substantially between soils, these strength-based approaches can only coincidentally give correct PL values, actually measuring what might be termed the plastic strength limit ($PL_{100}$); that is the water content corresponding to $s_{uFC} = 100 \times s_{uFC}(LL)$.

Fall cone (Wood & Wroth, 1978; Belviso et al., 1985; Wasti, 1987; Harison, 1988; Feng, 2000, 2001, 2004; Koumoto & Houlsby, 2001; Sharma & Bora, 2003; Lee & Freeman, 2009; Shimobe, 2010; Sivakumar et al., 2015), steady monotonic penetration (Stone & Phan, 1995; Stone & Kyambadde, 2007), fast-static loading (Sivakumar et al., 2009) and extrusion (Timár, 1974; Whyte, 1982; Medhat & Whyte, 1986; Kayabali & Tufenkci, 2010a, 2010b; Kayabali, 2011a, 2011b, 2012; Kayabali et al., 2016) approaches for PL determination have all been suggested as alternatives to the standard thread-rolling approach. As mechanical tests, these strength-based approaches are seen by some researchers as means of achieving higher degrees of repeatability and reproducibility of results, although, to date, most fall-cone research has been conducted on well-behaved clay-rich soils that lie above the A-line on the standard plasticity chart. Although these strength-based tests do not measure the onset of brittleness and hence cannot measure the true PL, they may in many cases be measuring a more useful parameter. If what is wanted is an indication of the variability of undrained strength with changing water content, a strength test seems much more appropriate than a test of the onset of brittleness.

Other proposed approaches

Some researchers have attempted to devise relationships between the PL and other soil parameter measurements, including suction data (Uppal, 1966; McBride, 1989; McBride & Bober, 1989), effective stresses from consolidation tests (Youssef et al., 1965; Nuyens & Kockaerts, 1967; McBride & Bober, 1989; McBride & Baumgartner, 1992) and soil moisture tension (Livneh et al., 1970; Gadallah, 1973). However, since there is no unique value of suction,
effective stress or undrained shear strength at the PL for all soils, this invalidates these techniques for PL determinations.

As the PL occurs at the onset of brittleness, methods of measurement based on the onset of cracking should in theory have a better chance of giving similar results. Attempts to do this include the cube method (Abdun-Nur, 1960), indentation test (de Oliveira Modesto & Bernardin, 2008) and thread bending test (Moreno-Maro & Alonso-Azcárate, 2015); the latter is based on the measurement of bending deformations. For the indentation test proposed by de Oliveira Modesto & Bernardin (2008), the force applied to a 30° cone was slowly and steadily increased in order to indent the soil test specimen, which was considered to be in a plastic state if the perforation mark printed on it presented no cracks. In other words, the deformation response indicates whether the soil is in a brittle (crack formation) or plastic state, rather than the magnitude of the applied force or indentation hardness. This approach can be contrasted with cone penetrometer methods in which a specified indentation depth for a particular load (i.e. the soil strength) is taken as the measurement of the plastic strength limit (e.g. Stone & Phan, 1995). Andrade et al. (2011) present a review of some other approaches for the determination of soil plasticity, such as the ‘Pfeifer hornet’, ‘capillary rheometer’ and ‘torque rheometer’ methods.

Other factors influencing deduced Atterberg limit values

Other factors, including the soil fraction tested, sample preparation technique adopted (i.e. testing of fine-grained soil in its natural condition or of the homogeneous soil paste produced using wet (preferred) or dry sample preparation techniques), and the chemistry and pH of any water added to the soil sample in preparing the soil paste for testing (Jang & Santamarina, 2016), can also influence the deduced values of LL and PL. For instance, the LL and PL values measured for peats and other highly organic soils are invariably strongly dependent on these factors (Hanrahan et al., 1967; Hobbs, 1986; Yang & Dykes, 2006; Asadi et al., 2011; O’Kelly, 2015). In the case of fibrous peat material, preloading (which allows the organic solids some stress history because of their compressible nature) produces lower LL values (O’Kelly, 2015). Greater mechanical breakdown of the peat solids during sample preparation produces lower LL, PL and IP values, especially for less humified material (O’Kelly, 2015), such that the measured plastic ranges are arbitrary and unlikely to sensibly correlate with mechanical behaviour (Hobbs, 1986; O’Kelly & Zhang, 2013; O’Kelly, 2015). Further, the pH of water affects the cation exchange capacity of fine-grained soil, such that even usage of distilled water in changing the consistency of the soil material for laboratory testing can lead to different LL than what might happen for the field material (Torrance & Pirnat, 1984). Sridharan (1991, 2014) calls into question the use of Hansbo’s equation (1) for non-ductile materials. For PL < PL, the strain-rate dependence and deformation mode of the soil test specimen will be significantly different for water contents between the PL and the PL (i.e. brittle state), as compared with w > PL, which brings into question the validity of any data extrapolation techniques for the scenario described.

An alternative and prudent approach, therefore, is to employ a lower RMW value (≈ 100) in defining the water content corresponding to the chosen fall-cone upper strength value (i.e. giving PL > PL). For instance, Kounomo & Housby (2006) (p. 708) suggested that, if the definition of the new input value at, say, a strength that is only a factor of 10 higher than that at the liquid limit could be useful in the context of determining the plastic limit from cone testing (i.e. RMW = 10 mobilising an sFC of 17 kPa), although this would result in a narrow strength range of 17–17 kPa in considering correlations between water content and sFC values. By adopting a higher strength gain factor (RMW > 10), the likelihood of the test soil occurring in a brittle state for water contents about the associated upper sFC Value will progressively increase (refer to Fig. 3). In other words, these tend to be conflicting requirements – on the one hand seeking to encompass a wide enough range of undrained strengths, but also requiring that the soil test is in a plastic state for water contents about the lower sFC Value. On the basis of the ratios of strengths at the PLs and LLs reported in Haigh et al. (2013), the water content corresponding to 25 times the strength mobilised at LL (defined as PL-25, i.e. sFC = 42.5 kPa) would approximate the lowest expected strength value at the PL for inorganic soils and also allow a good prediction to be made of the strength variation between LL and PL. For the standard 30°–80 g fall cone, the proposed PL-25 corresponds to a 4 mm penetration depth.

STRAIN-RATE EFFECTS

For the fall-cone test, the strain rate changes continuously as the cone accelerates under gravity from a stationary position, penetrating the test specimen and then decelerates before coming to rest, with the strain rate also dependent on the cone characteristics. For instance, typical mean strain rate (y) values of 1.0 × 10−1%/h (0.89 × 10−1–1.15 × 10−1%/h for d = 15–25 mm) and 2.5 × 10−1%/h (1.94 × 10−1–3.37 × 10−1%/h for d = 15–25 mm) were reported for the 30°–80 g and 60°– 60 g cones, respectively (Kounomo & Housby, 2001). For fall cones incorporating a falling distance before the cone tip contacts the surface of the test specimen (e.g. Sivakumar et al., 2015), the strain rates would be greater.

**PL100: a new parameter for soil mechanics practice?**

Having recognised the important distinction between the true PL and that measured by strength-based tests, the ‘PL’ determined by the fall-cone approach has been referred to as the ‘plastic strength limit’ (Haigh et al., 2013) PL100 (Harison, 1988; Stone & Phan, 1995; Stone & Kyambadde, 2007; Kyambadde & Stone, 2012; Haigh et al., 2013; O’Kelly, 2013; Kyambadde et al., 2014; Sivakumar et al., 2015, 2016), with the subscript 100 indicating that the defined strength is 100 times the strength mobilised for the fall-cone LL (sFC). (LL). This assumes that cones having identical apex angle and surface roughness values are used in identifying both LL and PL, and furthermore, that the strain-rate dependency of the soil remains the same (as considered in the next section).

Vardanega & Haigh (2014b) demonstrated from analysis of a large database of British standard (30°–80 g) fall-cone test results that, for any given soil, acceptable linear correlations could be drawn between both the logarithm of undrained strength and liquidity index and the logarithm of undrained strength and the logarithmic liquidity index. While the ratios of strengths at the PLs and LLs varied between soils, defining any two (or more) points on these linear relationships would give good predictions of undrained strengths at intermediate water contents (O’Kelly, 2013, 2016b). The measurement of PL100 together with the LL would achieve this. However, more often than not, one would end up testing soils in their brittle state (i.e. w < PL) for water contents around PL100. This has implications for the preparation of the test specimens for fall-cone testing near the PL100 (Wood & Wroth, 1978; Whyte, 1982; Wasti & Bezirici, 1986; Harison, 1988; Stone & Phan, 1995; Feng, 2000), in that for many cases sample preparation is difficult and some test specimens are likely not to be saturated, and calls into question the use of Hansbo’s equation (1) for non-ductile materials. For PL < PL, the strain-rate dependence and deformation mode of the soil test specimen will be significantly different for water contents between the PL and the PL (i.e. brittle state), as compared with w > PL, which brings into question the validity of any data extrapolation techniques for the scenario described.
The undrained strength of soil increases by approximately 10% per tenfold increase in strain rate (Ladd & Foft, 1974; Kulhawy & Mayne, 1990; Koumoto & Houlsby, 2001) (i.e. \( \mu = 0 - 1 \), where \( \mu \) is the rate dependence parameter). It is, however, not uncommon for the rate of strength increase to range between 5 and 15% (Ladd & Foft, 1974), with values of up to 30% measured for soils with high organic content (O’Kelly, 2014, 2016b). Hence, for soil having a greater rate dependence of strength, the average undrained strength value mobilised over the course of the cone penetration into the soil test specimen would be lower than that deduced from analysis of the fall-cone data using equation (1), and vice versa. In other words, the value of the cone factor depends on the strain rate (strain rate dependence) as well as the cone characteristics.

To demonstrate the effect of plausible differences in strain-rate dependence on the mobilised fall-cone strength for different mineral soils, it can be deduced from equation (7) and Fig. 5 that, compared with the commonly assumed \( \mu \) value of 0.10, the \( K \) value for the same smooth 30° cone could potentially vary by –16.9% (\( \mu = 0.15 \)) to +25.4% (\( \mu = 0.05 \)). In other words, putting aside uncertainty regarding the cone roughness (adhesion factor value), the static strength mobilised for the 30° fall cone can vary by up to ±20.3% from the value computed using equation (1), depending on the soil's level of strain-rate dependence in the probable range of \( \mu = 0.05 - 0.15 \).

\[
K = \frac{3\zeta}{\pi N_{ch} \tan^2(\beta/2)}
\]  

(7)

where \( \beta \) is the cone apex angle, \( N_{ch} \) is a dimensionless bearing-capacity factor that takes into account the surface heave of the soil test specimen resulting from the cone's penetration and \( \zeta \) is the ratio of the 'static' \( s_{uFC} \) to fall-cone dynamic \( s_{dFC} \) undrained strength values.

For the 30°–80 g fall-cone test (BS EN 1377-2 (BSI, 1990)) and assuming a semi-rough cone (i.e. with adhesion factor (\( \alpha \)) value of 0.5 = \( N_{ch} = 7.952 \), after Hazell (2008)), this ±20.3% variation would imply an \( s_{uFC} \) range of 1.6–2.4 kPa for the \( LL_{FC} \) condition, as defined by \( d = 20 \) mm. Note, using \( K \) values of 0.80 and 0.27 for the 30° (80 g) and 60° (60 g) cones, respectively, Farrell et al. (1997) computed \( s_{uFC(L)} \) of 157 and 1.59 kPa, respectively, consistent with the lower end of the identified \( LL_{FC} \) undrained strength range. Assuming the \( \mu \) value of a given test soil remains unchanged with reducing water content and providing the test soil remains in a plastic condition, on this basis, the \( s_{uFC} \) value mobilised for a heavier 30°–8 kg fall cone at \( d = 20 \) mm (i.e. at \( PL_{100} \) could range between 160 and 240 kPa. Note that, with \( R_{SW} = 100 \) and \( s_{uFC(L)} = 1.7 \) kPa, the \( s_{uFC} \) value of 170 kPa is near the lower end of the identified \( PL_{100} \) strength range.

Therefore, it has generally been taken that the \( LL_{FC} \) corresponds to a fixed undrained strength value; for example, from theory, \( s_{uFC} \approx 2.66 \) kPa for the 30°–80 g fall cone at \( LL_{FC} \) after Koumoto & Houlsby (2001), although this undrained strength value seems rather high, with the Casagrande LL value normally taken, on average, as 1.7 kPa (Wroth & Wood, 1978). However, the above example demonstrates that, even for a given fall-cone set-up, the \( s_{uFC(L)} \) value mobilised for different soils can vary relatively significantly and will also vary between set-ups having different cone characteristics and penetration depths used in defining the \( LL_{FC} \).

For pile design, studies of glacial soils, submarine soil investigations for offshore structures, and so on, the engineer is interested in the remoulded undrained strength, but as demonstrated earlier, the soil’s level of strain-rate dependence in the plausible range of \( \mu = 0.05 - 0.15 \) can have a significant influence on the mobilised \( s_{uFC} \) value. From this point of view, displacement-controlled cone devices (e.g. the soil mini-penetrometer for quasi-static undrained strength determinations described by Stone & Kyambadde (2007)) offer a more reliable approach in determining undrained strength and \( PL_{100} \) values because adjustments for strain-rate effects are not necessary.

GEOTECHNICAL CORRELATIONS

It has been demonstrated that the precise LL and PL values obtained for any given soil depend substantially on the techniques used to measure them. The values of LL and PL obtained are used both in order to classify soil and to determine other soil parameter values through correlation. It is the outcome of these processes that is more important to design practice than the precise values of LL and PL obtained.

The standard plasticity chart (ASTM, 2011; BSI, 2015) was developed from that proposed by Casagrande (1947) based on LL and PL values deduced using the ASTM standard percussion-cup and thread-rolling methods. Hence, from a purist’s viewpoint, only the Casagrande LL (\( LL_{cup} \)) (but not \( LL_{FC} \) (Prakash & Sridharan, 2006; Prakash et al., 2009) and thread-rolling PL values should be used for soil classification purposes using the standard plasticity chart or in the multitude of correlations with directly useful soil (design) parameters built up over the decades using \( LL_{cup} \) and standard PL data. As in many countries the \( LL_{cup} \) is no longer routinely measured, it is useful to investigate the correlation between \( LL_{FC} \) and \( LL_{cup} \) values such that account can be taken of discrepancies between the different LL measures when these are substantial.

COMPARISON OF THE FALL-CONE LL AND CASAGRANDE LL

Liquid limits obtained using the Casagrande cup and fall-cone apparatus share a similar approach, despite the differences in measurement techniques. The Casagrande cup (Haigh, 2012) and the fall cone (Koumoto & Houlsby, 2001) measure the undrained shear strength of the soil specimens and this is associated with LL. The Casagrande cup device imposes shock loading to the soil test specimen as the cup repeatedly impacts against the apparatus base, initiating a slope failure to close the standardised groove pre-cut into the test specimen. This scenario has been shown to measure a specific strength (i.e. undrained strength divided by soil density) value at \( LL_{cup} \) of approximately 1 m²/s² (Haigh, 2012). The \( LL_{FC} \), on the other hand, corresponds to a fixed
<table>
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<th>Percussion cup used</th>
<th>Notes</th>
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<tr>
<td>Sherwood &amp; Ryley (1970)</td>
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</tr>
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</table>

**Correlation between LLcup and LLFC**

Many studies have reported on the relationship between LLcup and LLFC (e.g., Karlsson, 1961; Skopek & Ter-Stepanian, 1975; Garneau & LeBihan, 1977; Karlsson, 1977; Littleton & Farmilo, 1977; Moon & White, 1985; Queiroz de Carvalho, 1986; Wasti & Bizerci, 1986; Wasti, 1987; Christaras, 1991; Koester, 1992; Leroueil & Le Bihan, 1996; Farrel et al., 1997; Mohajerani, 1999; Feng, 2001; Prakash & Sritharan, 2006; Deka et al., 2009; Clause-Mallet et al., 2012), with the divergence of these measurements well noted for $w > \sim 120\%$ (Skopek & Ter-Stepanian, 1975; Wasti, 1987; Mohajerani, 1999; O’Kelly, 2013).

For soil having a low LL (<50% (Budhu, 1985); <60% (Prakash & Sritharan, 2006), the LLcup deduced for the hard base cup and the LLFC deduced for the 30°–80 g fall cone produce broadly comparable results (Wasti & Bizerci, 1986), since this fall-cone set-up was benchmarked to produce essentially the same results as the Casagrande cup device. For the low- to medium-LL soils commonly used in engineering works, LLcup is generally slightly lower than LLFC as demonstrated by Belviso et al. (1985), Wasti & Bizerci (1986) and Di Matteo (2012), to name a few. For instance, Di Matteo (2012) reported that, for fluvial-lacustrine soils from central Italy, LLFC was about 2.2–2.8 points higher than LLcup. Hence, with PL obtained from thread rolling, a general small increase in $I_P$ occurs for low- to medium-LL soil when LLFC is used in the calculation. Although this small change in the measured LL value with a change in method does not represent a fundamental change in material behaviour, in some instances it is sufficient to change the classification of a soil from suitable to unsuitable (or vice versa) owing to precise thresholds of allowable LL and (or) $I_P$ values. For instance, Di Matteo et al. (2016) reported specific problems that arose when LLFC was adopted over LLcup in preliminary liquefaction risk assessments of the suitability of soil deposits for two earthworks projects in Italy. It was found that, for 18% of the soil samples investigated, the classification position according to the standard plasticity chart changed, moving them toward groups with poorer geotechnical qualities, resulting in contradictory and wrong classification compared with that deduced for LLcup.

Inconsistencies may also arise for fall-cone LL testing of fine-grained soils having high silt and (or) sand contents, which plot below the A-line on the standard plasticity chart, and also for high- and very high-plasticity soils (Prakash & Sritharan, 2006; Poulsen et al., 2012). These inconsistencies should be taken into account when changing the standard method of testing, with classification boundaries being moved to respect the inherent relationship between the LL values obtained using the two different approaches.

**Correlating Fall-Cone LL with Casagrande LL**

In order to achieve the desired corrections to soil classification procedures, correlations are required between reference undrained strength value, independent of soil density. This difference accounts for the systematic bias between these two approaches, with higher LL values being obtained for the Casagrande cup device compared to the fall cone for high-LL materials. A semi-logarithmic relationship of decreasing undrained shear strength for the LLcup with increasing values of LL was identified by Youssef et al. (1965). Haigh (2012) demonstrated that using an appropriate correction for soil density gave good agreement between LLcup and LLFC results, without the necessity of invoking different strength regimes for high- and low-$I_P$ soils, as has been suggested by Sridharan et al. (1999) and Sridharan & Prakash (2000).

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**Correlating Fall-Cone LL with Casagrande LL**

In order to achieve the desired corrections to soil classification procedures, correlations are required between...
results obtained from the two approaches for LL determinations. In this section, using a large database (see Table 2) assembled from the literature, correlations are established relating LL_{FC} with LL_{cup} for different standards. For each dataset considered, LL_{cup} results determined for the British and ASTM standards' soft- and hard-base percussion cups, respectively, were reported along with the corresponding British standard (BS) LL_{FC} test results. The available data allowed separate regression analyses considering: (a) LL_{FC} plotted against BS ‘soft-base’ cup (LL_{BScup}) (Figs 6 and 7); (b) LL_{FC} plotted against ASTM ‘hard-base’ cup (LL_{ASTMcup}) (Figs 8 and 9). The following regression curves were obtained from Figs 6–9:

\[
\begin{align*}
\text{LL}_{\text{FC}} &= 1.86 \times \text{LL}_{\text{BScup}}^{0.84} \quad (R^2 = 0.98, n = 216) \\
\text{full range of LL} \\
\text{LL}_{\text{FC}} &= 1.62 \times \text{LL}_{\text{BScup}}^{0.88} \quad (R^2 = 0.96, n = 199) \\
\text{for LL}_{\text{BScup}} < 120\% \\
\text{LL}_{\text{FC}} &= 1.90 \times \text{LL}_{\text{ASTMcup}}^{0.85} \quad (R^2 = 0.97, n = 199) \\
\text{full range of LL} \\
\text{LL}_{\text{FC}} &= 1.45 \times \text{LL}_{\text{ASTMcup}}^{0.92} \quad (R^2 = 0.97, n = 188) \\
\text{for LL}_{\text{ASTMcup}} < 120\% \\
\end{align*}
\]

Equations (8)–(11) are shown plotted in Fig. 10. Compared to the hard Micarta base of the ASTM cup device, the softer rubber base of the BS cup device consistently gives higher LL values because more energy is absorbed by it during the repeated impacts of the cup holding the soil test specimen (Norman, 1958; Whyte, 1982; Sridharan & Prakash, 2000; Haigh, 2016). For this reason, Haigh (2016) cautioned against direct comparisons of LL_{cup} results from the soft- and hard-base Casagrande cup approaches owing to differences in base hardness.

Consistent with the findings of Belviso et al. (1985), Wasti & Bezirci (1986), Prakash & Sridharan (2006) and Di Matteo (2012), from equations (8)–(11), the BS LL_{FC} is slightly greater than LL_{cup} (i.e. LL_{BScup} and/or LL_{ASTMcup}) for low- and intermediate-LL soils. For soils having higher LL, strong divergence between LL_{cup} and LL_{FC} is evident for the combination of BS LL_{FC} with both LL_{BScup} and LL_{ASTMcup}, as evident in Figs 6, 8 and 10 (supporting the findings of Skopek & Ter-Stepanian, 1975; Wasti, 1987; Mohajerani, 1999).

**RECOMMENDATIONS FOR THE FUTURE**

**Methods for measuring LL**

Despite the long history of the Casagrande cup apparatus and the enormous amount of data derived from it used in correlations, the lack of consistency between different cup apparatus (even when nominally they correspond to the same standard) makes it non-ideal for such a widely used test. Even if the will were present to do so, the complexity of ensuring that base hardness was standard between devices at manufacture and remained so through their working life would be great with such a wide variety of devices in current usage. A standardised fall-cone device is a more appropriate means for measuring LL in such a way as to get the same result, independent of where and when the test is undertaken.

An internationally standardised fall-cone LL set-up should specify the cone mass, apex angle, surface roughness and penetration depth at the LL. Although the 60° cone is less sensitive to variations in cone roughness (Koumoto & Houlsby, 2001) and, as a result, can arguably produce greater repeatability between geotechnical laboratories, the 30° cone is in much wider use and from this consideration would be the more obvious choice for international standardisation. However, an internationally standardised fall-cone LL set-up will not overcome variations in mobilised s_{a,F(LLL)} arising from differences in the strain-rate dependency of undrained strength between different soils.

**Proposed method for measuring PL_{25} and PL_{100}**

At present, no substantially better method of measuring the onset of brittleness has been developed than Atterberg’s thread-rolling method. If a standard fall-cone set-up is to be used for the LL test, however, it would be of value to consistently report a further parameter, termed the PL_{25} that is, the water content corresponding to 25 × s_{a,F(LLL)} at which the undrained strength is approximately 42.5 kPa. As
explained earlier, this strength value approximates the lowest expected strength value at the PL for inorganic soils, ensuring that the associated fall-cone testing is performed on the test soil in its plastic state, while covering a wide enough undrained strength range for correlation with water content. For the standard 30°–80 g fall cone, the proposed PL25 corresponds to a 4 mm penetration depth. Note, the strengths corresponding to the LLFC and PL25 are termed the fall-cone lower strength parameter and the water content corresponding to the fall-cone upper strength parameter (suFC(PL25)), respectively. This approach would allow better correlations to be achieved between strength and a new fall-cone consistency index (I_{FC}; equation (12)) for soft to medium-stiff clays than can be achieved with a conventional liquidity index based on the onset of brittleness at I_L = 0.

\[
I_{FC} = \frac{\log LL_{FC} - \log w}{\log LL_{FC} - \log PL_{25}} \quad (12)
\]

with \( I_{FC} \) being defined in logarithmic form since the bi-logarithmic undrained strength–water content correlation provides a regression coefficient value closer to unity compared with the semi-logarithmic form when considering a wide water content (plastic range) for a given soil.

In the proposed framework, the fall-cone undrained strength (\( su_{FC} \)) value corresponding to any water content value within the plastic range (\( LL_{FC} < w < PL \)) can then be approximated as

\[
\log su_{FC} \approx I_{FC} \log (su_{FC}(PL_{25})/su_{FC}(LL)) + \log su_{FC}(LL)
\]

\[= I_{FC} \log(25) + \log su_{FC}(LL)\]

(13)

which simplifies to the following equation (i.e. assuming \( su_{FC}(LL) = 1.7 \) kPa for \( I_{FC} = 0 \))

\[
\log su_{FC} \approx 1.4I_{FC} + 0.23
\]

(14)
Equation (14) gives an $s_{uFC}$ value of 42.5 kPa for $I_{FC} = 1$ (i.e. at PL25), with the approximation sign in this equation reflecting probable differences in the mobilised $s_{uFC}$ value on account of the different rate dependence of different soils. In a similar way, these equations can be used to estimate the $s_{uFC}$ values corresponding to PL100 (i.e. $I_{FC} = \log 100/\log 25 = 1.43$) and more generally PL$x$, including the corresponding water content values. Further, if the standard PL has also been measured using the thread-rolling method, the corresponding $s_{uFC}$ value and hence $R_{MW}$ value can be estimated using the same approach.

Consistency of reporting using appropriate terminology

Liquid limit and PL values are often reported in the literature without reference made to the methods and (or) standards used for their determination, which introduces additional uncertainty in using these data correctly for soil classification purposes or in correlations. Hence, it is important that appropriate terminology, including references to the test methodologies employed in deducing these index values, are reported (e.g. the fall-cone LL test performed to the British standard gives the British standard LLFC value (BS 1377-2 (BSI, 1990)), both for the test results and when reporting allowable ranges in design codes of, for instance, LL or in correlations with other soil parameters.

SUMMARY

The variation of techniques and equipment used to measure LL can result in significant variations in the measured values for a given soil. The fall-cone LL device is a more appropriate methodology, with the 30°–80 g fall cone recommended as the international standard. As demonstrated in the paper, the mobilised liquid-limit undrained strength will still vary slightly between different soils, depending on their strain-rate dependence of strength.

Although Atterberg’s thread-rolling method may appear unscientific, it is currently the most appropriate technique to use if the water content for the brittle–ductile state transition

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**Fig. 9.** British standard fall-cone LL plotted against ASTM Casagrande cup LL (data of LL < 120%)

**Fig. 10.** Comparison of fitting equations

Equation (14) gives an $s_{uFC}$ value of 42.5 kPa for $I_{FC} = 1$ (i.e. at PL25), with the approximation sign in this equation reflecting probable differences in the mobilised $s_{uFC}$ value on account of the different rate dependence of different soils. In a similar way, these equations can be used to estimate the $s_{uFC}$ values corresponding to PL100 (i.e. $I_{FC} = \log 100/\log 25 = 1.43$) and more generally PL$x$, including the corresponding water content values. Further, if the standard PL has also been measured using the thread-rolling method, the corresponding $s_{uFC}$ value and hence $R_{MW}$ value can be estimated using the same approach.
is required. The strength-based approach employed with the fall-cone methods cannot be used to determine Atterberg’s PL. Further, since the strength gain over the plastic range is, on average, significantly less than 100, the PL-100 water content is significantly less than Atterberg’s PL water content; that is, the soil would be tested while in a brittle state for water contents near the PL-100.

To overcome difficulties (e.g. the need for significant extrapolation on cone penetration depth against water content plots and significantly different strain-rate dependence expected for the brittle and plastic soil), the authors recommend PL-25 (to replace PL-100) as defining the fall-cone dence expected for the brittle and plastic soil), extrapolation on cone penetration depth against water content. On average, significantly less than 100, the PL100 water content cannot be used to determine Atterberg’s liquid limit. The strength-based approach employed with the LLASTMcup Casagrande liquid limit derived from ASTM liquidity index.

To overcome difficulties (e.g. the need for significant extrapolation on cone penetration depth against water content plots and significantly different strain-rate dependence expected for the brittle and plastic soil), the authors recommend PL-25 (to replace PL-100) as defining the fall-cone upper strength parameter, which can be readily determined along with the LLFC Parameter value using the standard 30°–80 g fall cone. From these two measurements, a methodology has been presented for the determination of the undrained shear strength corresponding to any water content within the plastic range, allowing substantially better strength predictions than existing correlations based on liquidity index.

NOTATION

- \( d \): cone penetration depth
- \( I_{FC} \): fall-cone consistency index
- \( I_L \): liquidity index
- \( I_P \): plasticity index
- \( K \): cone factor
- \( \text{LLASTMcup} \): Casagrande liquid limit derived from ASTM ‘hard-base’ cup
- \( \text{LLBScup} \): Casagrande liquid limit derived from BS ‘soft-base’ cup
- \( \text{LLcup} \): Casagrande liquid limit
- \( \text{LLFC} \): fall-cone liquid limit
- \( N_{10} \): dimensionless bearing capacity factor
- \( n \): number of data points used to generate a regression
- \( PL_n \): water content corresponding to \( y \) times \( s_{uFC(LL)} \)
- \( PL_{25} \): water content corresponding to fall-cone upper strength parameter
- \( PL_{100} \): water content corresponding to \( s_{uFC} = 100 \times s_{uFC(LL)} \)
- \( R_{MW} \): strength gain factor
- \( R' \): coefficient of determination
- \( s_b \): saturated remoulded undrained shear strength
- \( s_{lim} \): in situ undrained shear strength
- \( s_{uFC} \): fall-cone shear strength
- \( s_{uFC(LL)} \): fall-cone shear strength at LL (i.e. fall-cone lower strength parameter)
- \( s_{uFC(LL)_{fcp}} \): fall-cone upper strength parameter (i.e. \( 25 \times s_{uFC(LL)} \))
- \( s_{uFV}/s_0 \): normalised field vane strength
- \( s_{uF} \): dynamic undrained strength mobilised in fall-cone test
- \( s_{uLL} \): undrained shear strength at LL
- \( W \): weight of fall cone
- \( w \): water content
- \( a \): cone adhesion factor
- \( q_{ov} \): ratio of overconsolidation ratio to normalised field vane strength
- \( \beta \): cone apex angle
- \( \gamma \): strain rate
- \( \zeta \): ratio of \( s_{uFC} \) to \( s_u \)
- \( \mu \): rate-dependence parameter
- \( s_{uFF} \): in situ vertical effective stress
- \( \phi_{tec} \): effective angle of shearing resistance of normally consolidated material

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AASHTO (American Association of State Highway and Transportation Officials) (2000). T90-00: Determining the plastic limit and plasticity index of soils. Washington, DC, USA: AASHTO.

Corrigendum

O’Kelly, B. C., Vardanega, P. J. & Haigh, S. K. (2018). Use of fall cones to determine Atterberg limits: a review. Géotechnique 68, No. 10, 843–856, https://doi.org/10.1680/jgeot.17.R.039. On page 844, in the section headed ‘Repeatability of the thread-rolling test’, the unit of plasticity index was not definitively stated; for completeness, it should have read ‘… plasticity index (Ip, %)…’.

Consequently, the final term in equation (2) is incorrect; the corrected equation is as follows.

\[
\frac{s_{u(insitu)}}{\sigma_v^0} = 0.11 + 0.0037(I_p)
\]  

Furthermore, in the text immediately below equation (5) on page 845, the values given for the percentage variation in \( s_{u(insitu)}/\sigma_v^0 \) were incorrect; the text should read ‘… the percentage variation in \( s_{u(insitu)}/\sigma_v^0 \) from its mean value would range between 1·0% and 6·0%…’.

The authors apologise to the readers of Géotechnique for these mistakes in the original paper.