

# Discussion: Measuring the plastic limit of fine soils: an experimental study

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## Contribution by S. K. Haigh and P. J. Vardanega

### 1. Introduction

Sivakumar *et al.* (2015) present an excellent experimental study that represents a major advance on efforts to get  $PL_{100}$  (Stone and Phan, 1995), termed the plastic strength limit by Haigh *et al.* (2013), into mainstream use. Given the current efforts underway to revise European soil testing standards (e.g. Orr, 2015), the paper by Sivakumar *et al.* is a welcome addition to the literature. Many authors have attempted to develop methods of determining the plastic limit using the cone penetrometer (e.g. Harison, 1988), as the thread rolling test is regarded by some as crude and inaccurate (e.g. Sherwood, 1970; Whyte, 1982). Sivakumar *et al.* offer a mechanical alternative to the original test described over a century ago by Atterberg (1911a, 1911b).

### 2. Strength variation in the plastic range

Sivakumar *et al.* rightly report that their plastic limit values are actually  $PL_{100}$  values as they assumed that the 'plastic limit' occurs at an undrained strength of 170 kPa (100 times the 1.7 kPa liquid limit proposed by Wroth and Wood (1978)). Haigh *et al.* (2013, 2014) presented work showing that the plastic limit as described by Atterberg (1911a, 1911b) does not correlate to a fixed strength as it is not a strength test, so these two different 'plastic limits' may not coincide.

Recently, analogous work by Vardanega and Haigh (2014) utilised a large database of standard fall-cone test results (80 g–30°) to reveal (among other things) that if the double logarithmic formulation utilising a logarithmic liquidity index  $I_{LN}$  (Koumoto and Houlsby, 2001) (Equation 4) is assumed, the implied ratio of undrained strengths between liquid and plastic limits is found – on average – to be around 83.5 (Equation 5). Similar work investigating the variation of

undrained strength with liquidity index has been reported by O'Kelly (2013).

$$4. \quad I_{LN} = \frac{\ln(w/PL)}{\ln(LL/PL)}$$

$$c_u = (c_L)83.5^{(1-I_{LN})}$$

5. where

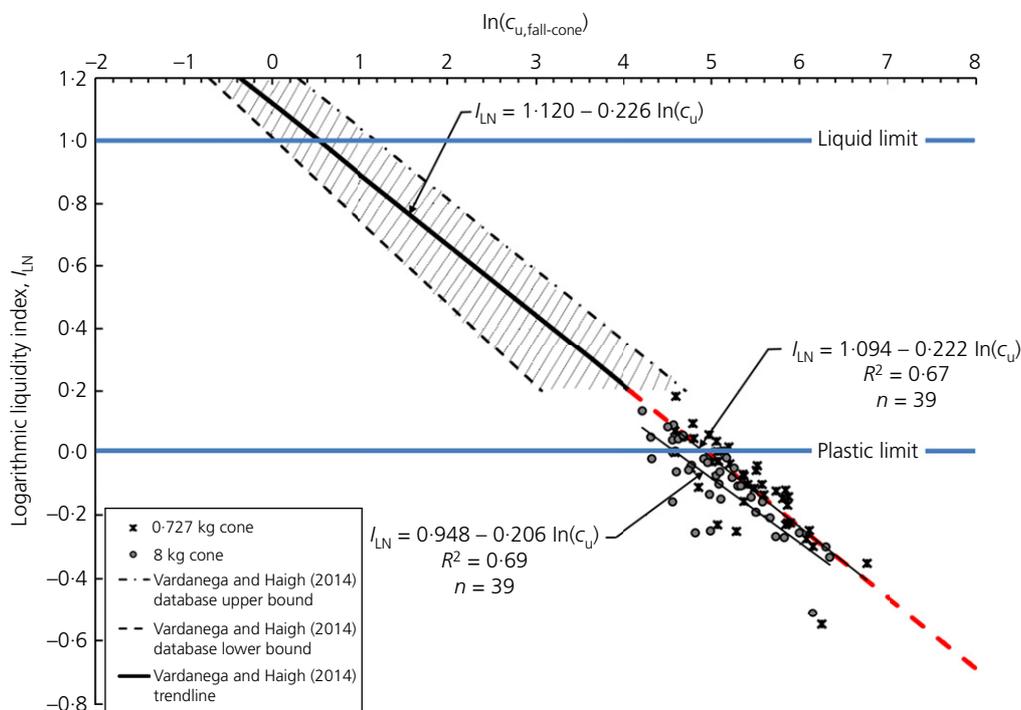
$$c_L = 1.7 \text{ kPa}$$

$$0.2 < I_L < 1.1$$

The data from Figure 7 of the paper under discussion were digitised by the contributors and transformed onto a  $c_u$ – $I_{LN}$  plot (Figure 12), the bounds of the data from Vardanega and Haigh (2014) are also shown in Figure 12. In order to convert the data presented of cone penetration  $d$  into undrained strength  $c_u$ , three steps are required. Firstly, a cone factor  $K$  needs to be assumed, as defined by Hansbo (1957) (Equation 6), linking the force applied to the cone  $F$  to its penetration  $x$

$$6. \quad F = \frac{x^2 c_u}{K}$$

Secondly, the validity of this plasticity solution for materials that are potentially brittle needs to be assumed. Thirdly, dynamic analysis of the fall-cone test should be carried out as tests in which a cone penetrates a soil surface with significant velocity are not self-similar to those in which the cone is stationary with its tip level with the soil surface at the beginning of the test.



**Figure 12.** Dataset from the paper under discussion (Sivakumar *et al.*, 2015) plotted on  $I_{LN}$ - $\ln(c_u)$  axes; bounds and mean trendline from Vardanega and Haigh (2014) are also shown

Dynamic analysis of the fall-cone test implies equating the potential energy lost by a fall-cone dropped from a height  $h$  above the soil surface with the work done by the force  $F$  in bringing it back to rest at a penetration  $d$ . This results in

$$7. \quad mg(h+d) = \int_0^d \frac{x^2 c_u dx}{K} = \frac{1}{3} \frac{d^3 c_u}{K}$$

Hence

$$8. \quad c_u = \frac{3mgK(h+d)}{d^3}$$

Equation 8 can be used to interpret the data in Figure 7 as presenting data of the variation of strength with the liquidity index or logarithmic liquidity index (as given by Equation 5). These data are shown in Figure 12 together with the bounds of the database presented in Vardanega and Haigh (2014) for soils at higher liquidity index (80 g-30° fall-cone data) (note that few data were sourced for inclusion in the database at  $I_L < 0.2$ ). It can be seen that use of the logarithmic liquidity index formulation shows the data to continue on the same linear trend, both above and below the plastic limit. For any given set of fall-cone data, the choice of cone factor can shift all the data

points horizontally by the same distance on Figure 12. However, the slope of the regression to the data would not change. The high degree of similarity between the slopes of the data from the three different cones (80 g, 8 kg and 0.727 kg) suggests that the analysis is valid if an appropriate cone factor is chosen and that Equation 5 is a good predictor of undrained shear strength across a wide range of water contents.

### 3. Sample preparation

Sivakumar *et al.* (2015) state that sample preparation was done in accordance with the procedures outlined by Sivakumar *et al.* (2009), utilising oven drying during the process. However, Mesri and Peck (2011: pp. 89-90), in their discussion of the paper by Sivakumar *et al.* (2009), cast doubt on the use of oven drying for sample preparation. Does the sample preparation process cause the material to enter at least a partially brittle (or cracked) state? The sample was also compacted (in the mould) and hence was also in an unknown saturation state. As well as water content, clay strength is also a function of overconsolidation ratio, saturation and current effective stress (e.g. Mayne *et al.*, 2009), so if this procedure is adopted for soil testing in practice, two questions may be posed.

- How can sample preparation be kept consistent enough to ensure that the test always gives the same answer for the same material in different laboratories?

- Does the strength measured during the test have a practical meaning in terms of the strength variation of clays close to the plastic limit, or does the variety of states at which compacted soils can exist at these low water contents make this only valid within the context of this test procedure?

#### 4. Difficulties with standardisation

While Sivakumar *et al.* (2015) present an excellent experimental study, the method proposed does not result in a completely automated mechanical method for determination of the plastic strength limit ( $PL_{100}$ ), as clearly sample preparation needs to be kept consistent between laboratories. While the strength at the liquid limit is a remoulded strength, with drier samples close to the plastic limit we have near-zero total stresses but an unknown amount of suction, so an unknown effective stress state and hence potentially variable strength. Could the authors comment on the difficulty (or lack thereof) of this drive towards standardisation?

#### Authors' reply

We thank the contributors for their thorough assessment of our article. There are essentially two aspects that need clarification.

#### 1. Repeatability of producing test specimens for routine applications

The water content at the plastic limit is about 10% (in terms of the percentage of difference rather than the actual water content) more than the optimum water content (OWC) for compaction (Sridharan and Nagaraj, 2005). If the compaction energy is high, then the difference will be even greater. For example, the OWC of kaolin is about 28% based on light compaction and about 26% based on heavy compaction, whereas its plastic limit is about 31%. For fine soils, the degree of saturation of compacted soil is about 85–90% (depending on the energy level) and, therefore, if compacted at the plastic limit, it could be close to 95% or more. For practical purposes, the fall-cone test specimens can be assumed to be saturated: hence the density of the compacted soil is practically independent of the compaction effort. This was also examined as part of the study under discussion, where test specimens of kaolin were prepared over a range of water contents close to the plastic limit. It was found that the bulk density remained generally the same for a given water content and soil, regardless of the level of compaction applied using the procedure described in the paper.

#### 2. Ratio between undrained shear strengths at plastic limit and liquid limit

This aspect was discussed in the last part of the article under discussion. In this, the authors stated the following (Sivakumar *et al.*, 2015: p. 62).

An equally valid and complementary explanation is obtained by considering the dynamic  $c_{ud(LL)}$  value of 2.66 kPa (Koumoto and

Houlsby, 2001) mobilised at  $h=20$  mm for the British Standard 80 g–30° fall-cone LL apparatus (BSI, 1990). On this basis, the 8 kg–0 mm cone set-up would mobilise a dynamic  $c_{ud(PL)}$  value of 266 kPa. Hence the 8 kg cone mass must be reduced by a factor of approximately 0.87 (i.e. = 230/266 and corresponding strength ratio  $c_{u(PL)}/c_{u(LL)}$  of 87) to 6.948 kg in order to mobilise an equivalent  $c_{u(PL)}$  value of 170 kPa in triaxial compression at  $\dot{\gamma} = 79\%h\dots$

However, by considering strain rate effects in the predicted fall-cone strengths, we later postulated that the strength ratio for the quasi-static condition appeared to be approximately 87. Some limited evidence has been presented to support this claim, although further investigation is required to draw any firm conclusion.

#### REFERENCES

- Atterberg A (1911a) Lerornas förhållande 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 der Bodenkunde* **1**: 4–37 (in German).
- BSI (1990) *BS 1377-2*: Methods of test for soils for civil engineering purposes – part 2: classification tests. BSI, London, UK.
- Haigh SK, Vardanega PJ and Bolton MD (2013) The plastic limit of clays. *Géotechnique* **63(6)**: 435–440, <http://dx.doi.org/10.1680/geot.11.P123>.
- Haigh SK, Vardanega PJ, Bolton MD and Barnes GE (2014) Discussion: The plastic limit of clays. *Géotechnique* **64(7)**: 584–586, <http://dx.doi.org/10.1680/geot.13.D.06>.
- Hansbo S (1957) A new approach to the determination of the shear strength of clay by the fall cone test. *Swedish Geotechnical Institute Proceedings* **14**: 5–47.
- Harison JA (1988) Using the BS cone penetrometer for the determination of plastic limit of soils. *Géotechnique* **38(3)**: 433–438, <http://dx.doi.org/10.1680/geot.1988.38.3.433>.
- Koumoto T and Houlsby GT (2001) Theory and practice of the fall cone test. *Géotechnique* **51(8)**: 701–712, <http://dx.doi.org/10.1680/geot.2001.51.8.701>.
- Mayne PW, Coop MR, Springman S, Huang AB and Zornberg J (2009). State-of-the-art paper (SOA-1): Geomaterial behavior and testing. *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, ICSMGE, Alexandria, Egypt*. Millpress/IOS Press, Rotterdam, the Netherlands, vol. 4, pp. 2777–2872.
- Mesri G and Peck RB (2011) Discussion: A new method of measuring plastic limit of fine materials. *Géotechnique* **61(1)**: 88–92, <http://dx.doi.org/10.1680/geot.2011.61.1.88>.
- O'Kelly BC (2013) Atterberg limits and remoulded shear strength-water content relationships. *ASTM Geotechnical Testing Journal* **36(6)**: 939–947, <http://dx.doi.org/10.1520/GTJ20130012>.

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- Orr T (2015) Editorial. *Proceedings of the Institution of Civil Engineers – Geotechnical Engineering* **168(1)**: 1–2, <http://dx.doi.org/10.1680/geng.2015.168.1.1>.
- 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.
- 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, <http://dx.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, <http://dx.doi.org/10.1680/geng.14.00004>.
- Sridharan A and Nagaraj HB (2005) Plastic limit and compaction characteristics of fine soils. *Proceedings of the Institution of Civil Engineers – Ground Improvement* **9(1)**: 17–22, <http://dx.doi.org/10.1680/grim.2005.9.1.17>.
- Stone KJL and Phan KD (1995) Cone penetration tests near the plastic limit. *Géotechnique* **45(1)**: 155–158, <http://dx.doi.org/10.1680/geot.1995.45.1.155>.
- Vardanega PJ and Haigh SK (2014) The undrained strength-liquidity index relationship. *Canadian Geotechnical Journal* **51(9)**: 1073–1086, <http://dx.doi.org/10.1139/cgj-2013-0169>.
- Whyte IL (1982) Soil plasticity and strength: a new approach using extrusion. *Ground Engineering* **15(1)**: 16–24.
- 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, <http://dx.doi.org/10.1139/t78-014>.