An Assessment on Correlation between Dynamic Cone Penetration Blow Count and Liquid Limit of NSW Clays

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ABSTRACT

The Dynamic Cone Penetration (DCP) test is one of the simple and economical geotechnical tests used in the field, and its results are commonly used for practical applications, such as checking consistency with the results from borehole loggings or roughly estimating soil bearing capacity. Even though the test is widely used in practice, particularly in Australia, its relationships with clay moisture limits, such as Liquid Limit (LL), have not been extensively studied. Hence, this paper describes the energy-based behaviour of DCP in relationship with LL via the theoretical model of pore collapse and empirical correlation. As a result, the method predicts well the plasticity behaviour of NSW clays from 8 DCP blows per 100mm. Discussion on the method, testing results and relevant limitations are also provided.

Keywords: cone resistance, driving energy, dynamic cone penetration, Liquid Limit, NSW clays.

1 INTRODUCTION

Among field soil tests, dynamic cone penetration (DCP) test is one of the oldest geotechnical methods for soil characterisation and was originally developed in Australia by Scala (1956). This in-situ test is simple with quick set-up, low cost and applicable to all kinds of soils, particularly to evaluate the consistency of in-situ soil and assist soil-logging descriptions. Furthermore, the portable DCP apparatus can characterise soil in field, particularly granular materials, such as sandy soils or road subgrades and pavement (Mohammadi et al. 2008; Lee et al. 2019). Of the correlated parameters, relative density (D_r) and California Bearing Ratio (CBR) are so far intensively investigated in relationship with DCP index (DPI) (Lin et al. 2019; MacRobert et al. 2019; Sagar 2022). The investigation is inspired from the correlation of DCP driving energy with cone tip resistance and skin friction along the rod in cohesionless soils (Escobar et al. 2013; Gholami et al. 2020). However, the analysis can be affected by reading of observer, rod tiltation after DCP extension during penetration, and energy loss. This loss can be caused by the friction between hammer and guide rod, and transferring energy through upper rod of DCP equipment (Sadrekarimi and Seyyedi 2009).

To gain the more reliable and meaningful DCP results, the energy-based assessment on soil resistance values was performed with the inclusion of load cell and accelerometer on the anvil (Byun and Lee 2013). The parameters of cone resistance were proven to be consistent with DPI values and soil properties via using instrumented DCP for shallow soils (Byun et al. 2014, Navarrete et al. 2021). The result is in good agreement with the study on DPI in lime-treated clays, which was performed by Vakili (2021), mentioning that DPI-values can also be correlated with CBR, Unconfined Compressive Strength (UCS) and subgrade reaction coefficients (K_s). The authors indicated that apart from curing effects, moisture content apparently affects DPI, along with density and pore water pressure (Ampadu et al. 2017). Using DCP test to predict the moisture change in lateritic soil subgrade was also researched by Ampadu and Fiadjoe (2015), confirming that DPI is significantly dependent on the small changes in water content of sandy clay materials. In the changing range of water content, optimal moisture content (OMC) of the soil is the most concerning parameter since DPI tends to increase significantly when moisture is larger than OMC and the soil turns to wet state. Hence, soil state obviously affects DCP behaviour in soil.

Although the soil state of clay related to Liquid Limit (LL) is one of the most important parameters for soil classification, few studies have been conducted to research correlations between DCP results and LL of clayey soils. In a similar mechanism, another common field test, named Standard Penetration Test (SPT) has been revealed with a strong relationship between SPT blows (N) and relative void ratio (Mujtaba et al. 2018). Meanwhile, initial void ratio is closely related to moisture content and Atterberg limits of intact clays (Chung et al. 2016; Pineda et al. 2016), suggesting correlations between dynamic penetration parameters with these values. Furthermore, Rahim et al. (2004) formulated the relationship between DPI and porosity, resulting in a good agreement with experiment data. Therefore, it is possible to establish a relationship between DPI and moisture content in limit values from Atterberg's methods.

In this paper, this relationship will be investigated for clayey soil at 30 sites in and around Sydney City, New South Wales, named NSW clays. Field DCP and laboratory Casagrande tests were performed for samples at shallow depths. The analysis goes from theoretical to empirical analysis to produce the predicted curve between field DCP results and laboratory-based values of LL. The discussion on the results and limitations is also conducted to evaluate the analysis method.

2 ANALYSIS METHOD

2.1 DCP analysis and initial-void-ratio prediction

According to AS 1289.6.3.2 (1997), DCP test is a simple penetrometer test in which a hammer of 9kg falls from a height of 510mm, pushing a 20-mm diameter cone with apex angle θ of 30° into layers of ground. The amount of penetration in mm for each falling or blow is called Dynamic Cone Penetration Index, denoted as DPI (mm/blow). According to Australia Standard HB 160 (2006), there is also another parameter to express DCP result, named Dynamic Cone Penetration Blow Count (*N*). This count is the blow number of hammer to push the DCP rod penetrate 100mm into the ground. From this, it can obtain:

$$\text{DPI (mm/blow)} = 100 / N \text{ (blow)} \tag{1}$$

In each blow, the energy to push the DCP cone penetrating into the ground is originated from the kinetic energy (*E*), which is produced by dropping the 9-kg hammer to the 16-mm diameter rod head from the height of 510mm. However, in DCP test, when the energy is transferred from the rod head to the cone tip, there is an amount of energy loss through the rod, which increases with an extension of the rod length (Odebrecht et al. 2005; Byun and Lee 2013). Assuming that this energy is totally turned into cone tip resistance (denoted as q_c, kPa), and rod friction in the ground is insignificant because the cone diameter is larger than the rod diameter (d=20 mm, compared with 16 mm), the energy of DCP test can yield as follows:

$$E = \text{DPI}.A.q_c / LR \tag{2}$$

Where, A (m^2): cone tip area, $A = \pi d^2/4$
LR: energy loss ratio, $LR = 0.64$ without extension rod (after Byun and Lee 2013)

Substituting Eq. (1), *A, E* and *LR*-value into Eq. (2), cone tip resistance (q_c) can be obtained:

$$q_c \text{ (kPa)} = 917N \tag{3}$$

Adopting the analysis of slip pattern under the DCP cone, conducted by Salgado et al. (1997), and assuming a slip shape of logarithmic spiral around the cone (Bolton 1979), the DCP cone resistance q_c can be computed as:

$$q_c = 2p\, e^{\pi\tan\Phi} [(1+C)^m - mC - 1] / [C^2 m(1-m)] + c.\cot\Phi(e^{\pi\tan\Phi}-1) \tag{4}$$

Where, p (kPa): the limiting cavity expansion pressure from cone tip,
c (kPa): soil cohesion, Φ (°): frictional angle,
$m = 1+1/[\tan(45+\Phi/2)]^2$
$C = e^{(\pi/2)\tan\psi}\cot(\theta/2)$, where ψ: the dilatancy angle [ψ=0 for normally consolidated clay (Vermeer and de Borst 1984)], and θ: DCP apex angle (θ = 30°).

Considering when the cone tip penetrates into the ground, it creates and widens a cavity in the cylindrical shape, the final porosity (*n*) of penetrated soil can be formulated with the limiting cavity expansion pressure (*p*). Based on the theoretical model of pore collapse, suggested by Rahim et al. (2004), *p* can be calculated as below:

$$p = c \cot\Phi\; c\, (n^{1.34\sin\Phi/(\sin\Phi-1)} - 1) \tag{5}$$

Substituting Eq. 3-5 and values, the final porosity *n* can be calculated from *N*, *Φ* and *c* as follows:

$$n = \{1 + 13.93m(m-1)\, [458.5N - c.\cot\Phi(e^{\pi\tan\Phi}-1)] / [c \cot\Phi\, e^{\pi\tan\Phi}(4.73^m - 3.73m - 1)]\}^{(\sin\Phi-1)/(1.34\sin\Phi)} \tag{6}$$

By using the model of pore collapse, it is assumed that the soil matrix is incompressible during pore collapse occurring. This means that very little or no change in material matrix volume occurs due to compression of the soil. In other words, the volume change is mostly caused by effective pore collapse during cone penetration. Therefore, with the pore collapse theory suggested by Rahim et al. (2004) and applied in DCP test, the final porosity (n) can be relative to initial porosity (n_0) as the following equation:

$$n^{1.34} = (n-n_0) / (n-1) \qquad (7)$$

From the initial porosity (n_0), the initial void ratio (e_0) can be calculated as below:

$$e_0 = (\mathrm{n}+\mathrm{n}^{1.34}-\mathrm{n}^{2.34})/(1-\mathrm{n}-\mathrm{n}^{1.34}+\mathrm{n}^{2.34}) \qquad (8)$$

Observing Eq. 6 and 8 and with assumed values of clay cohesion (c) and frictional angle (Φ), initial void ratio (e_0) of the soil can be predicted from DCP blows/100mm (N). By investigating the next possible relationship between the ratio and LL, one can obtain the bridging relationship between N and LL.

2.2 Empirical prediction of Liquid Limit (*LL*) from initial void ratio (e_0)

The relationship between void ratio and Liquid Limit is extensively investigated by intensive studies on both remoulded and intact soils. For disturbed samples, Komurlu (2020) studied the influences of soil void ratio on LL, which is determined by fall-cone and Casagrande method. Before the LL tests were performed, void ratios of specimens had been measured in volumes determined with particular specific gravities of studied soils. The experimental results show that void ratio has an important effect on LL-values determined by both fall-cone and Casagrande testing mechanisms. However, there is no unique correlation between the two parameters of various kinds of remoulded soils. Instead, the LL-e_0 relationship differs among various disturbed soils in wide ranges of LL; therefore, the good correlation should be adopted for each kind of studied soil (Komurlu 2020). This statement is also associated with the research conducted by Sridharan and Honne (2011), confirming that there is a relationship between LL and change in void ratio. Nevertheless, the correlation is poor, and it is related to compressibility of soil, particularly when the studied soils are remoulded with the low ratio of void ratio ($\Delta e/e_0$), indicating the high disturbance level of tested sample. Hence, it is in need to investigate whether there is a good correlation between LL and e_0 from in-situ samples by high-qualified sampling and with low levels of disturbance.

In this study, researching the relationship between LL and in-situ void ratio (e_0) is reasonable because DCP test is a kind of on-site experiment, and its results come from the behaviour of intact ground. The high quality of tested soil sample plays an important role to produce a good correlation of LL-e_0, on which DCP blows N can rely on predicting acceptable LL-values. From the literature review on intact samples at shallow depths, various studies of high-quality clay specimens were conducted to characterise their features in soil mechanics (e.g. stress-strain behaviour by Pineda et al. (2016) or penetration behaviour by Chung et al. (2017). In their sampling technique, advanced modified samplers were utilised, such as Osterberg-type or hydraulic fixed-piston sampler (Pineda et al. 2016) and Oil-operated fixed-piston sampler (Chung et al. 2017). From extracting relevant results of in-situ void ratio e_0 and LL, their correlation curve can be shown in Figure 1a, which results in a parabolic fitting equation with the coefficient of determination R^2=0.93, as follows:

$$LL = 6.4e_0^2 + 10.3\, e_0 + 29 \qquad (9)$$

It is noted that LL in Figure 1a is determined by fall cone method, while LL in this study is based on values from Casagrande testing mechanism. However, since the LL from fall cone tests is quite equal to those defined from Casagrande method (Christaras 1991; Grønbech et al. 2011; Spagnoli 2012), it is reasonable to consider LL-values in Eq. 9, determined by both methods. It is also notable in Figure 1a that most samples are medium to super high plasticity clay with LL-values ranging from 30 to 120%, according to Unified Soil Classification System (Grønbech et al. 2011). To cover a lower and higher range of LL-value, two boundary lines are depicted in Figure 1a, illustrating the variation of LL for possibly lower or higher plasticity of other clays.

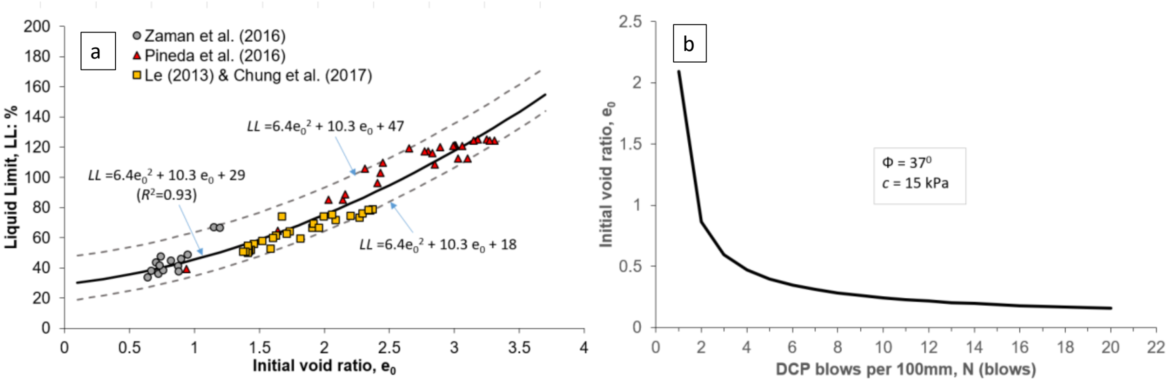

Figure 1: (a) *Empirical correlation between LL and* e_0, *(b) Predicted curve of N vs* e_0

By adopting Eq. 9 into Eq. 6 and 8, *LL* at a specific depth can be predicted from *N*-values in DCP test at that depth with assumed values of cohesion (*c*) and frictional angle (Φ) (see Figure 1b). For studied NSW clays (e.g. Ballina clay), the average frictional angles of soil at the shallow depths can vary from 32° to 41° (Pineda et al. 2016). Therefore, a representative angle of 37° can be adopted, while clay cohesion can be averaged at around 15 (kPa). With these assumed values, the input blows (*N*) from in-situ DCP tests at a specific depth can roughly output the in-situ or initial void ratio at that depth. However, this rough estimation is based on the consideration that at the tested depth, the studied soil is homogeneous without any interference of impurity (e.g. rootlets, sandstone, gravel, ironstone or shale fragments). Furthermore, the depth should be shallow (up to just below 1.0m) since the extension can significantly reduce the energy loss ratio (Odebrecht et al. 2005; Byun and Lee 2013), which underestimates the values of initial void ratio (see Eq. 6 and 8), leading to the resulted *LL* smaller than expected in Eq.9.

3 EXPERIMENTAL VALIDATION

3.1 Testing sites and program

To investigate and verify the proposed relationship between DCP blow count (N) and LL at particular depths, DCP tests were conducted at thirty (30) different sites located in and around Sydney city, NSW, Australia (Figures 2a-b). There were totally 104 samples collected from the sites to bring them to Geotesta Laboratory for testing Liquid Limit (sample numbers shown on each pinpoint in Figures 2a-b). The sample depths are tabulated in Table 1 with NSW postcodes.

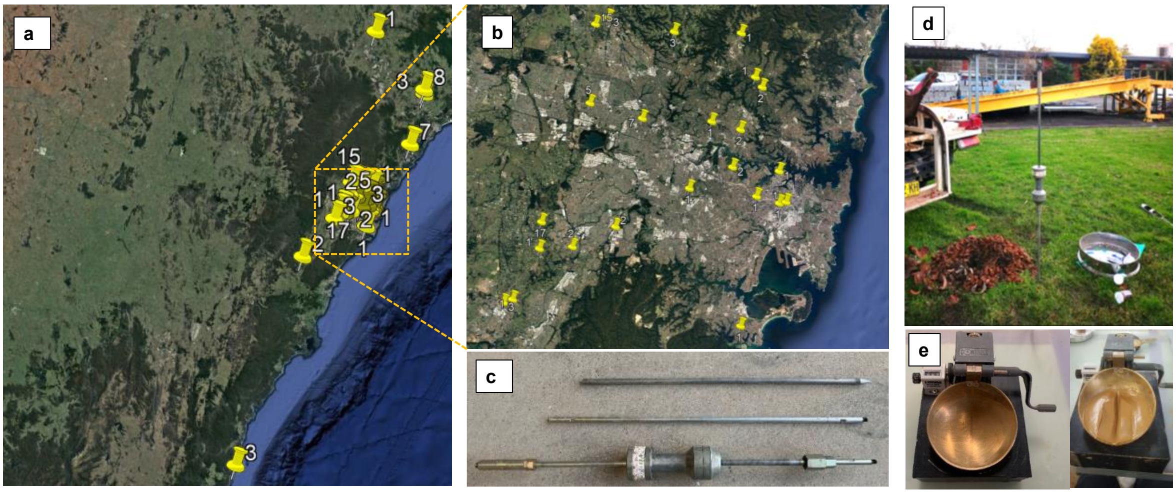

Figure 1: (a) NSW site, *(b) Sydney site, (c) DCP equipment, (d) DCP test and (e) LL test*

On the surveyed site, DCP tests were conducted following the guidance mentioned in Australian Standard AS 1289.6.3.2 (1997) for Determination of penetration resistance of a soil – 9 kg dynamic cone penetrometer test, while *LL* is determined by Casagrande method, following AS 1289.3.1.1 (2009). The photographs of equipment and tests are shown in Figures 2c-e.

3.2 Testing results and discussion

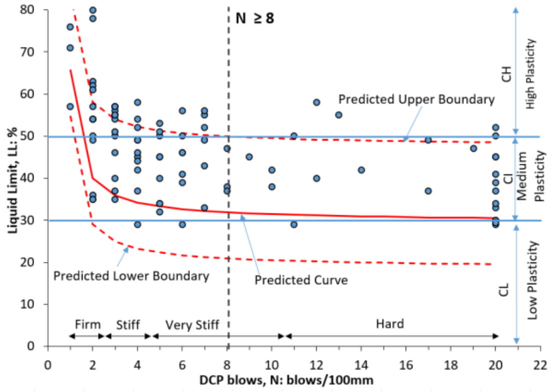

Figure 3: Predicted curve and measured data of LL vs N

Table 1: Sample depths at sites in NSW postcodes

Postcode	**2039**	**2042**	**2043**	**2072**	**2073**
Depth (m)	1.0	1.2	0.8	0.6-0.8	0.9
Postcode	**2077**	**2112**	**2114**	**2130**	**2137**
Depth (m)	1.6	0.5-1.0	0.4	0.8	0.9
Postcode	**2143**	**2145**	**2148**	**2155**	**2158**
Depth (m)	0.8	0.5-1.2	0.8-1.3	0.4-0.9	0.5-0.7
Postcode	**2170**	**2174**	**2179**	**2229**	**2259**
Depth (m)	0.7-1.5	0.8-1.0	0.4-1.1	0.7	0.6-1.2
Postcode	**2320**	**2321**	**2333**	**2546**	**2557**
Depth (m)	0.5-1.4	0.5-0.8	0.6	0.8-1.2	0.5
Postcode	**2565**	**2567**	**2575**	**2765**	**2800**
Depth (m)	0.4	1.2-1.9	1.0-1.2	0.6-1.2	0.7-0.8

As a result of this study, Figure 3 illustrates the relationship between DCP blows per 100mm (*N*) and LL of NSW clays. It can be seen from this figure, most investigated clays are medium and high plasticity clays (LL>30%), equivalent to CI and CH, respectively (AS 1726: 2017). Furthermore, the predicted curve and upper boundary well cover measured points, which have a majority of *N*-values smaller than 8 with higher LL up to about 80%. With *N* larger than 8, most samples turn to be in medium plasticity (30%<LL<50%), and the predicted upper boundary reflects this tendency correctly by intersecting the medium-high-plasticity boundary (LL=50%) right at *N* = 8 (blows/100mm). According to Australian Standard No. HB160 (2006), the *N*-value of 8 indicates the clay consistency of Very Stiff, as shown on the horizontal axis in Figure 3. When considering the change of plasticity from medium-to-high (MH) to only medium (M) degree, very stiff NSW clays can be categorised into two levels, very-stiff-MH (5 < *N* < 8) and very-stiff-M (8< *N* <10).

From the investigation on testing postcode, the very-stiff-M to hard clays (N ≥ 8), indicating medium plasticity state (CI category), distribute in the areas with NSW 2039, 2072, 2077, 2145, 2155, 2158, 2170, 2229, 2259, 2320, 2321, 2546, 2567 and 2765. These postcodes mostly belong to the northern areas of testing Sydney and NSW sites (refer to Figures 2a-b). Based on the geological map of bedrock under very-stiff/hard clay in these areas, most of the surveyed sites are underlain by Wianamatta Group (Rh) with medium to coarse-grained quartz sandstone (DMRS 1983), corresponding to the very-stiff-M clay (medium plasticity). Meanwhile, to the south, the firm to very-stiff-MH clay (medium-to-high plasticity) is related to fine-to-medium-grained lithic sandstone (Rwb and Rwm) or black-to-dark shale (Rwa). This means that the differences in the grained level of bedrock between the northern and southern studied areas are in light with the categorisation in very-stiff clay based on plasticity degree; the predicted upper curve of DCP versus LL clearly demonstrates this categorisation.

However, using the prediction method has limitations. Firstly, the method analysis assumes that no skin friction occurs during penetration since cone diameter is larger than rod diameter and the depth is limited within shallow degrees. This leads to an overestimation of frictional angle value at 37^0, which is based on Ballina clay from the study of Pineda et al. (2016). Secondly, unexpected intervention of impurity in clay, such as ironstone, shale, gravel pieces, can significantly increase N-value, which may not reflect the right category of very-stiff-M clay. Likely, DCP test with the depths deeper than 0.8-1.0m (see Table 1) can remarkably increase the DCP blows due to further energy loss from rod extension. Future research is expected to consider these limitations to reflect better the correlation between DCP blows *N* and *LL*.

4 CONCLUSIONS

An analysis method was proposed to predict the correlation between dynamic cone penetration (DCP) blows per 100mm (*N*) and Liquid Limit (*LL*). The method is based on the theoretical model of pore collapse to build the relationship of *N* versus initial void ratio (e_0) and empirical curves between e_0 and *LL*. This correlation was investigated using the DCP tests at 30 sites in NSW, Australia and Casagrande tests for samples collected from these sites. The significant conclusions can be drawn as follows:

- Predicted curve and upper boundary reflect well the *N*-*LL* correlation: high LL with N-value lower than 8, and medium LL from 30% to 50% with N-value larger than 8. Therefore, the N-value of 8 can categorise very-stiff clay into two kinds: (i) very-stiff-M (medium plasticity) and (ii) very-stiff-MH (medium-to-high plasticity), which is in light of the discrepancies in geology origins of bedrock under the layer of studied clays.

- However, the study has limitations: assumptions with overestimated frictional angle, high DCP blows by extending rods and encountering unexpected intervention of impurity in NSW clays.

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