




Assessing the undrained strength of very soft clays in the SPT

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Technical Note

Keywords

Site investigation
Standard Penetration Test
In-situ tests
Undrained shear strength of soft clays

Abstract

The log of a SPT in very soft clay may simply indicate a zero blow-count, or present information on the penetration – under self-weight – of the composition (sampler, rods and hammer) as recommended by some standards. The second type of information is often disregarded by design engineers due to the lack of a standard procedure for measuring these penetrations or because the test is regarded as not sensitive enough to give an indication on the undrained shear strength of soft clays. The penetration under the composition's self-weight, however, can indicate the magnitude of S_u , which, along with other more specific and sensitive tests, can help in assessing the spatial distribution of clay consistency in a large deposit. A proposed test procedure and interpretation had been given in an earlier technical note. This note presents an extended formulation and an evaluation of S_u via the SPT at a construction site in Rio de Janeiro, including comparisons with results of piezocone and vane tests. The values of S_u obtained with the SPT lie between the profiles given by vane tests, corrected by Plasticity Index, and the Critical State Theory, the latter representing a lower bound to the clay strength.

1. Introduction

The Standard Penetration Test (SPT) is part of any site investigation campaign, being the first test performed, with the aim of defining the stratigraphy. During the test, when a layer of very soft clay is encountered, the sampler does not need to be driven by hammer blows, and penetrates under self-weight. Some SPT standards prescribe the recording of the penetration of the sampler plus rods with or without the hammer (the Brazilian standard, with the hammer resting on top of the rods; ABNT, 2020). In this case, the log presents, at the depth of starting the test, the information W/L , where W indicates “self-weight” and L is the recorded penetration of the composition. Other standards just require that, if the sampler penetrates 45 cm under self-weight, the operator records $N=0$ and removes the sampler so that boring proceeds to the next test depth (usually 1.0 m ahead). Due to the lack of a standard test procedure and an interpretation guidance, or to the concept that SPT is not sensitive to provide any data for very soft clays, most design engineers disregard the information on self-weight penetration.

The undrained shear strength of clays, S_u , is usually determined by in-situ and laboratory tests (e.g., Almeida & Marques, 2013). For a number of reasons, in-situ tests are generally preferred for defining S_u , and this topic was covered in detail by Wroth (1984). Both in-situ and laboratory

tests are scheduled after an initial site investigation, based on SPTs, to define the stratigraphy of the site. Therefore, it would be interesting if an assessment of the undrained shear strength of the clays is made in this first campaign, which is the objective of this note.

In a previous note (Lopes, 1995), a standard procedure for the case of self-weight penetration and a formulation for data interpretation was put forward. The interpretation is analogous to that of a pile at failure in a static load test.

This note suggests a procedure for evaluating a lower bound value for S_u from the SPT and presents results of tests carried out at a site in Rio de Janeiro, Brazil, where a very soft clay layer was investigated by other methods (CPTu and vane). SPT tests in soft clays have also been useful for providing a water content profile, with data obtained from the clay at the sampler tip; the water content is then related to S_u by other tests, such as vane and UU triaxial tests (e.g., Almeida & Marques, 2013). An evaluation of S_u of clays is also possible via the torque applied to the sampler, in the so-called SPT-T (Décourt, 2002).

2. Proposed test interpretation

The problem of penetration, followed by stabilization, of a sampler under the static action of self-weight and hammer can be considered as being equivalent to that of the

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Submitted on November 25, 2020; Final Acceptance on June 29, 2021; Discussion open until February 28, 2022.

<https://doi.org/10.28927/SR.2021.060520>



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failure of a pile in a static load test. When the penetration of the sampler stabilizes, the configuration is that of a cylindrical element in which the weight of the sampler plus rods and hammer reaches equilibrium with the soil resistance developed along its external lateral surface and its base or tip area (Figure 1).

A very similar condition occurs when a pile develops its full bearing capacity in clay. In this condition, the ultimate bearing capacity of the pile – in a homogeneous, undisturbed clay – can be obtained with:

$$Q_{ult} = q_{ult} A + U \sum f_s \Delta l = (S_u N_c + \sigma_{vo}) A + U L S_u \quad (1a)$$

As for the sampler, if its penetration, L , is less than its length (approximately 800 mm), the bearing capacity equation above can be modified to (Figure 1a):

$$Q_{ult} = (S_u N_c + \sigma_{vo}) A + U L \eta_1 S_u \quad (1b)$$

where η_1 is a disturbance factor for the clay along the sampler shaft. After a 45 cm penetration, the sampler is filled with soil and functions as a close-ended pile.

Equation 1b can be rewritten with the ultimate load replaced by the sum of the weights of the sampler, rods and hammer:

$$W_s + W_r + W_h = (S_u N_c + \sigma_{vo}) \frac{\pi D^2}{4} + \pi D L \eta_1 S_u \quad (2)$$

If the sampler penetrates more than its length (approx. 800 mm), the clay will then act on the rods and will be strongly remolded (Figure 1b). In this case, a given length of the rod, L_r , penetrates the clay, and a slightly more complex expression is necessary (Figure 1b):

$$W_s + W_r + W_h = (S_u N_c + \sigma_{vo}) \frac{\pi D^2}{4} + \pi (D L_s \eta_1 S_u + d L_r \eta_2 S_u) \quad (3)$$

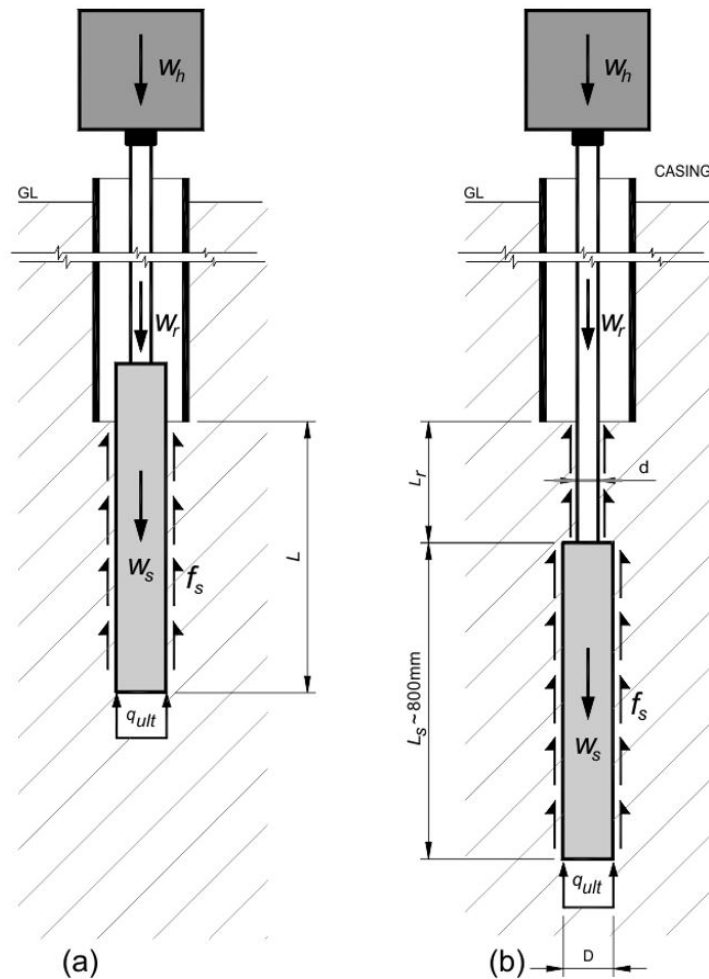


Figure 1. Test scheme, with sampler penetration (a) less than and (b) greater than the sampler length (L_s = sampler length, L_r = rod penetration).

where η_2 is a second clay disturbance factor. If the weight of the composition and its penetration are known, and the disturbance factors are estimated, the above expression leaves S_u as the sole unknown.

For tip unit resistance of both the pile and the sampler, the classical factor $N_c = 9.0$, suggested by Skempton (1951), can be used.

The disturbance factors are the ratio of the mobilized shear strength of the clay on the side of the sampler/rod to its undisturbed strength, S_u . They can be evaluated for each clay or investigation site, but some typical values can be derived from the experience with the relation between S_u and side friction f_s in CPTu. In clays of low to moderate sensitive, such as in Rio de Janeiro, where $S_t \leq 5$, the (local) sleeve friction is typically half of S_u , i.e., $f_s/S_u \sim 0.5$ (e.g., Jannuzzi et al., 2015). This would be an upper bound to the disturbance factor as the sleeve displacement in the CPTu does not cause full disturbance of the clay. For a large relative displacement, as expected for the SPT sampler in very soft clays, full disturbance is likely to occur, and it can be hypothesized that η_1 could ultimately be as low as the inverse of clay sensitivity (i.e., $\eta_1 = 1/S_t$). Considering $S_t = 5$, η_1 would be in the range of:

$$0.2 \leq \eta_1 \leq 0.5 \quad (4)$$

For η_2 , as the clay around the rod is fully remolded mainly due to the reduction in diameter, it can be assumed that η_2 is the inverse of the clay sensitivity, as described in Equation 5.

$$\eta_2 \sim \frac{1}{S_t} \quad (5)$$

3. Proposed test procedure

The following test procedure is suggested:

- At the test depth, the sampler and necessary rods are carefully lowered in the borehole. The penetration should be slow, controlled with the help of a rod holding (or “U”) key (Figure 2a). The rod length must exceed the casing by at least 1.0 m.
- The penetration under the action of the weight of the sampler and rods is recorded for a first calculation of S_u (Figure 2b) making use of Equation 2 without the hammer weight.
- The hammer is then put on the top of the rods while the rods are held with the U key (Figure 2c). The key is then loosened slowly to ensure a quasi-static penetration. In a very soft clay one (or more) rod segments can be necessary as the hammer comes close to seat on the top of the casing. The hammer can be lifted to allow the addition of a new rod segment, and is then put on the top of the rods as before.

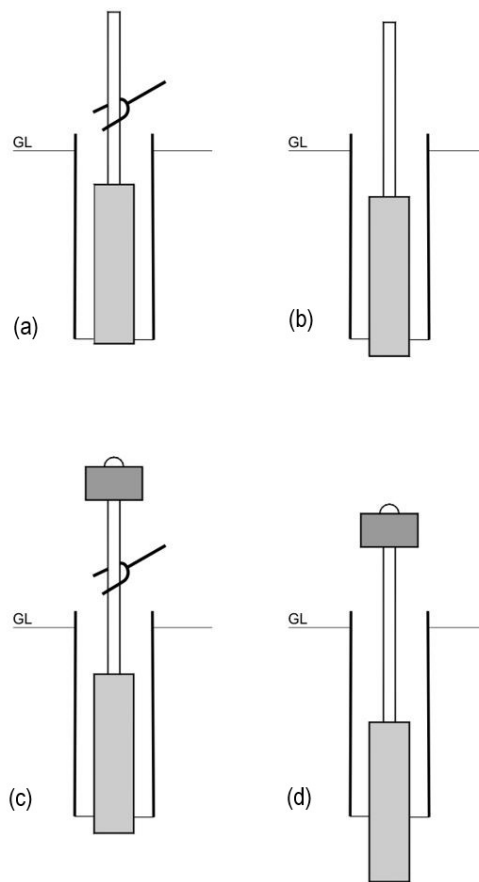


Figure 2. Suggested procedure: (a) the sampler and rods lowered in borehole, (b) the composition slowly released until penetration stops, (c) hammer rests on the top of the (held) rods and (c) the composition slowly released until penetration stops.

- The final penetration is then recorded (Figure 2d) and Equation 2 is used.

It should be noted that the proposed calculations are applicable only if the thickness of the remaining clay layer is greater than the penetration observed in the test.

On the shear strength profile, the calculated strength should be indicated at the mid-point of the penetration. As there is usually an increase in strength with depth, the calculated shear strength corresponds to the average along the penetrated length.

4. Application at a construction site

The proposed test procedure and interpretation were applied during the investigation of a construction site in Jacarepagua neighborhood, Rio de Janeiro, Brazil. The site presents a soft clay deposit, 11-12 m thick, with the water level almost at ground level. Vane tests, piezocone tests, laboratory vane and UU triaxial tests were also performed (Figure 3). A comprehensive description of the site investigation campaign can be seen in Almeida (1998) and Almeida et al.

Table 1. S-14 SPT data and interpretation.

Depth at start of test (m)	Pene-tration, L (m)	Rods		Hammer ?	Com-position weight (kN)	σ_{vo} at tip (kN/m ²)	η_l	S_u (kN/m ²)
		Length (m)	Weight (kN)					
4.9	0.3	6	0.18	No	0.28	68	0.4	5.1
4.9	2.2	6	0.18	Yes	0.93	92	0.2	8.7
7.4	1.6	8	0.24	Yes	0.99	117	0.2	9.1
9.0	1.9	12	0.36	Yes	1.11	142	0.2	10.1

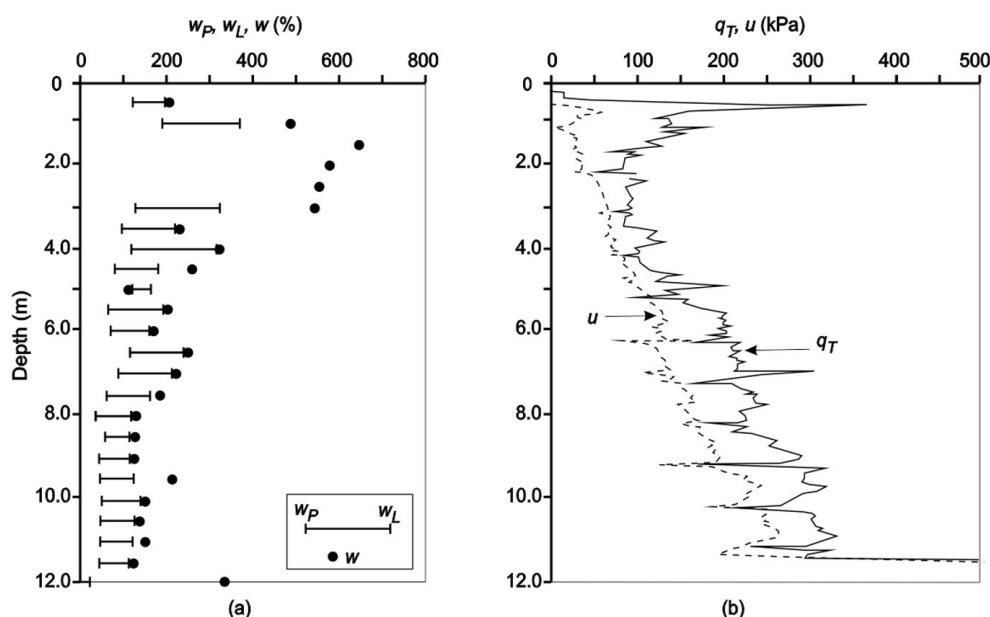


Figure 3. Profiles of (a) water content and (b) typical pore-pressure and tip resistance from CPTu (Almeida, 1998).

(2000). Further data on the clay can be seen in Baroni & Almeida (2012) and Riccio et al. (2013).

Water content and piezocone profiles presented in Figure 3 indicate that three layers can be distinguished: (i) an upper organic clay crust, 3 m thick, with a water content as high as 600%, (ii) a second clay layer, 4 m thick, with a water content typically of 200%, and (iii) a third layer, 4 m thick, with a water content typically of 100%. Laboratory tests also indicated an OCR of 2.2 near the ground surface (typical of water table fluctuations), reducing to 1.5 (typical of aging) at 4 m depth and then remaining constant.

Undrained shear strength profiles from vane tests, as measured and corrected, are shown in Figure 4. The corrected S_u profile shown in Figure 4 was obtained with Bjerrum's correction by the PI values (Bjerrum, 1973). The correction factor μ used in the present case was 0.6 as the clay's PI value is around 100%, which is consistent with the Brazilian experience (Sandroni, 1993). It can be observed that corrected S_u values varied from 5 to 15 kPa.

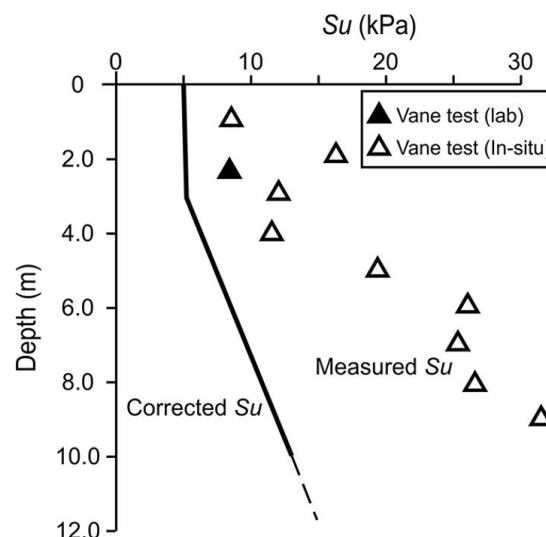


Figure 4. S_u profile as determined by vane tests.

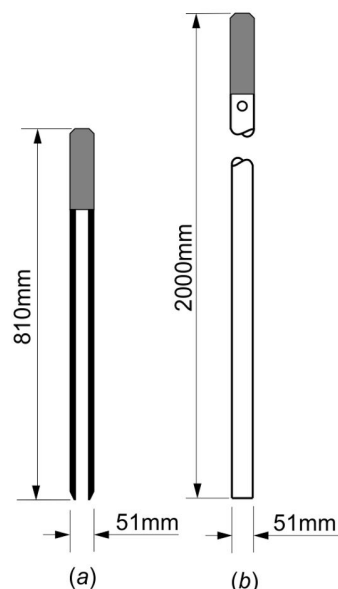


Figure 5. (a) Standard SPT sampler and (b) 2.0 m false sampler.

The proposed procedure for the SPT was applied at a few borings, using a longer, false sampler to avoid the question of a change in diameter between the sampler and the rods. The false sampler was a 2 m long aluminum tube, close-ended, with a rough external surface to ensure full adherence to the clay (Figure 5b).

Penetrations without the hammer weight did not exceed 0.3 m, but reached 2.00 m with the hammer. A typical record of the SPT field data and subsequent interpretation is presented in Table 1. In the computations shown in this table, $\eta_1 = 0.2 - 0.4$, $\eta_2 = 0.2$ and $\gamma_{sat} = 13 \text{ kN/m}^3$ (assumed to calculate σ_w). The weight of the composition considered the false sampler, with 70 N, the rods, with 30 N/m, and an anvil with 35 N.

S_u values obtained with the proposed SPT procedure and the corrected vane test profile are shown in Figure 6, with values marked at the center of the penetration length. S_u values obtained with the SPT were close to the corrected vane test profile but did not exhibit the same increase with depth. Another profile was added to this figure, based on the soil stress history (Ladd et al., 1977) and anchored on the Critical State Theory (Almeida, 1982; Wroth, 1984). S_u values were obtained with:

$$\frac{S_u}{\sigma_{vo}} = K \cdot OCR^m \quad (6)$$

where K and m for Rio de Janeiro clays have typical values of 0.3 and 0.8, respectively (Almeida, 1982).

Considering $OCR = 1.5$ and $\gamma' = 3 \text{ kN/m}^3$, for $z = 3 \text{ m}$, $S_u = 3.7 \text{ kN/m}^2$; for $z = 12 \text{ m}$, $S_u = 14.9 \text{ kN/m}^2$, as indicated by the dashed line in Figure 6.

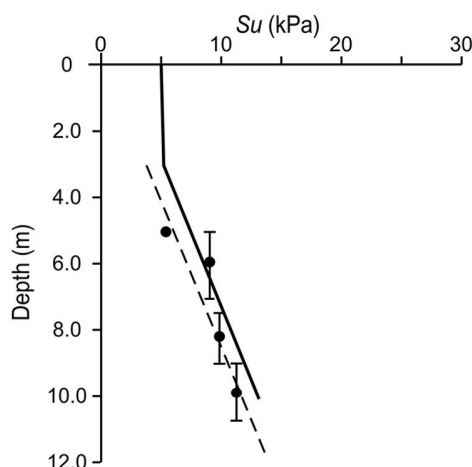


Figure 6. S_u values determined in S-14 (penetration lengths indicated) and profiles of corrected vane tests (solid line) and based on stress-history/critical-state (dashed line).

5. Concluding remarks

A procedure that can aggregate information to the SPT in very soft clay has been proposed. The proposal does not intend to replace more precise and well-established in-situ tests for these soils, such as the vane test and CPTu. It has been proposed that an information that is usually obtained without standardization – and usually disregarded – should be improved in order to be useful, combined with other tests, to present a spatial distribution of consistency in a large soft clay deposit.

The test and interpretation procedures were applied in a site investigation campaign in Rio de Janeiro, Brazil, and produced S_u values close to profiles given by corrected vane tests and the Critical State Theory, the latter representing a lower bound to the clay strength. These results are believed to encourage further investigation of the procedure, and – combined with other test results – appear to be suitable for use in preliminary stability analysis of embankments on soft clays.

Acknowledgements

The second author received a MSc scholarship from CNPq - Brazilian Research Council. This study was partly financed by CAPES - Coordenação de Aperfeiçoamento de Pessoal de Nível Superior, Brazil, under Finance Code 001.

Declaration of interest

There is no conflict of interests in the material presented.

Authors' contributions

Francisco Lopes: conceptualization, methodology, original draft preparation. Osvangivaldo Oliveira: investigation, tests.

Marcio Almeida: conceptualization, discussion of results, review and approval of the final version of the manuscript.

List of symbols

A	sampler base or tip area
D	sampler diameter (51 mm or 2")
d	rod diameter (usually 25 mm or 1")
f_s	unit shaft resistance
K	parameter in normalized undrained strength equation
L	penetration length (general)
L_s	sampler length
L_r	rod penetration length
m	power in normalized undrained strength equation
N_c	pile bearing capacity factor due to cohesion
OCR	overconsolidation ratio
PI	Plasticity Index
Q_{ult}	bearing capacity of a pile (also of the sampler)
q_{ult}	unit base or tip resistance
S_t	clay sensitivity
S_u	undrained shear strength
U	perimeter
W	weight of the composition sampler + rods + hammer
W_h	weight of hammer
W_r	weight of rods
W_s	weight of sampler
w	water content
w_L	Liquid Limit
w_p	Plasticity Limit
η	disturbance factor
μ	Bjerrum's correction factor
σ_{vo}	total vertical geostatic stress at the tip level
σ'_{vo}	effective vertical geostatic stress

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