The Undrained Strength of Soft Clays Determined from Unconventional and Conventional Tests

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Abstract. The laboratory fall cone test, considered an unconventional test, was performed to estimate the undrained shear strength of undisturbed samples of Brazilian coastal soft clays with different plasticity index values. The undrained shear strength determined by laboratory fall cone test was compared with the strength determined by conventional field and laboratory tests commonly used to estimate this parameter in cohesive soils: piezocone test, field vane test, unconfined compression test, unconsolidated undrained triaxial compression test and laboratory vane test. The fall cone test undrained shear strength results presented good agreement with the laboratory vane test strength results and reasonable agreement with unconfined compression test strength results. The strength results obtained by laboratory tests were compared with the continuous strength profile estimated from the piezocone test calibrated using the field vane test, and presented good agreement with fall cone test and laboratory vane test strength results. The normalised undrained shear strength was compared with some empirical correlations reported in the literature based on plasticity index, being verified some behaviour similarity.

Keywords: fall cone test, soft clays, undrained shear strength.

1. Introduction

The properties of the soil are crucial to perform a geotechnical engineering design. Estimating geotechnical parameters is complex because of the difficulty in obtaining reliable experimental data and because of the natural variability of the subsoil. In soft cohesive soils, the determination of these parameters is considered to be even more complex, as it is necessary to understand not only the soils strength properties but also its deformability properties and hydraulic conductivity. For short-term stability analyses in these soils, the undrained shear strength S_u is the most important design parameter (Shogaki, 2006).

Many factors affect the shear strength of clays, such as the types of minerals, humidity, stress history, draining during shear, load rate and soil structure, and it is not justifiable to attempt to attribute a unique shear-strength value to any given clay (Sridharan *et al.*, 1971). Moreover, according to Lunne *et al.* (1997b), there is no unique value for S_u *in situ*; this value depends on the mode of rupture, the anisotropy of the soil, the deformation rate and the stress history.

The standard tests to determine the shear strength of soils are typically classified as either laboratory or field tests. Field tests generally supply measurements of the soil strength that can be acquired more rapidly and in greater quantity than the measurements afforded by laboratory tests. However, they provide less precise measurements and, in some cases, are based on empirical correlations (Alshibli *et al.*, 2011).

The conventional tests to determine S_u in the laboratory are unconfined compression test (UCT), unconsolidated undrained triaxial compression test (UUT) and laboratory vane test (LVT), and *in situ* are piezocone test (CPTU), field vane test (FVT) and pressuremeter test (Kempfert & Gebreselassie, 2010). S_u depends on the testing method, among other factors, thus to understand the relations between the strengths determined by each test and the reliability of these determinations is important when S_u is a relevant parameter (Watabe & Tsuchida, 2001).

The fall cone test (FCT), considered unconventional test in many countries, was developed between 1914 and 1922 by the Geotechnical Commission of the Swedish State Railways and, compared with other test methods, it is considered to be a very simple method, which has led to its extensive use in Scandinavia (Hansbo, 1957). Although it was originally developed to estimate the strength of remoulded cohesive soils, it became widely used as a standard method of determining the liquid limit of clays (Koumoto & Houlsby, 2001), having already been included in the British, Swedish, Canadian and Japanese standards (Claveveau-Mallet *et al.*, 2012; Feng, 2000; Tanaka *et al.*, 2012).

The present study shows the result of five conventional and commonly applied tests for the S_u determination - CPTU, FVT, UCT, UUT and LVT - and compares these results with those of the fall cone test (FCT), also known as the Swedish cone test.

The strength results obtained by laboratory tests were compared with the strength profile obtained from CPTU

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Submitted on September 11, 2016; Final Acceptance on November 24, 2017; Discussion open until April 30, 2018.
DOI: 10.28927/SR.403291

with the cone factors (N_{kt} and $N_{\Delta u}$) calibrated using the FVT, considered a referential test to obtain reliable values of S_u (Schnaid & Odebrecht, 2012). The CPTU was adopted because it supplies a continuous profile of S_u. Also it has a strong theoretical foundation and several well-known and comprehensive publications are available concerning its interpretation (Robertson, 2009).

This study also compares the normalised undrained shear strength results with some empirical correlations reported in the literature based on the plasticity index (I_p).

2. Materials and Methods

2.1. Soil

The investigated site is located in the city of Vila Velha, Espirito Santo State, in the coastal region of Brazil, near to Rio de Janeiro, composed of recent fluvial, fluvial-marine and fluvial-lacustrine sediments. The soft clay deposits in Brazil found all along the coastline were originated in the Quaternary period. The local subsoil was formed by cycles of erosion and sedimentation which occurred during periods of regression and transgression of sea level, between the Pleistocene, 123000 years ago, and the Holocene, 5100 years before present (Suguio, 2010).

The investigated deposit is formed of a thick layer of soft clay, situated in an area near to a highway construction site, whose subsoil underwent rupture during the embankment operations. Standard penetration tests (SPT) and piezocone test (CPTU) performed locally indicate that the site (Fig. 1) is composed of a subsurface layer of a very soft organic clay, with water level 0.50 m below the surface, over a layer of very soft marine clay with thickness of 15.0 m, followed by a layer of sand. Fig. 1 also presents the clay layer SPT blow count (N_{value}) of zero values, low values of q_t and f_s, obtained from CPTU, and water content (w_n) values above the liquid limits (w_L) determined by characterization tests in SPT samples.

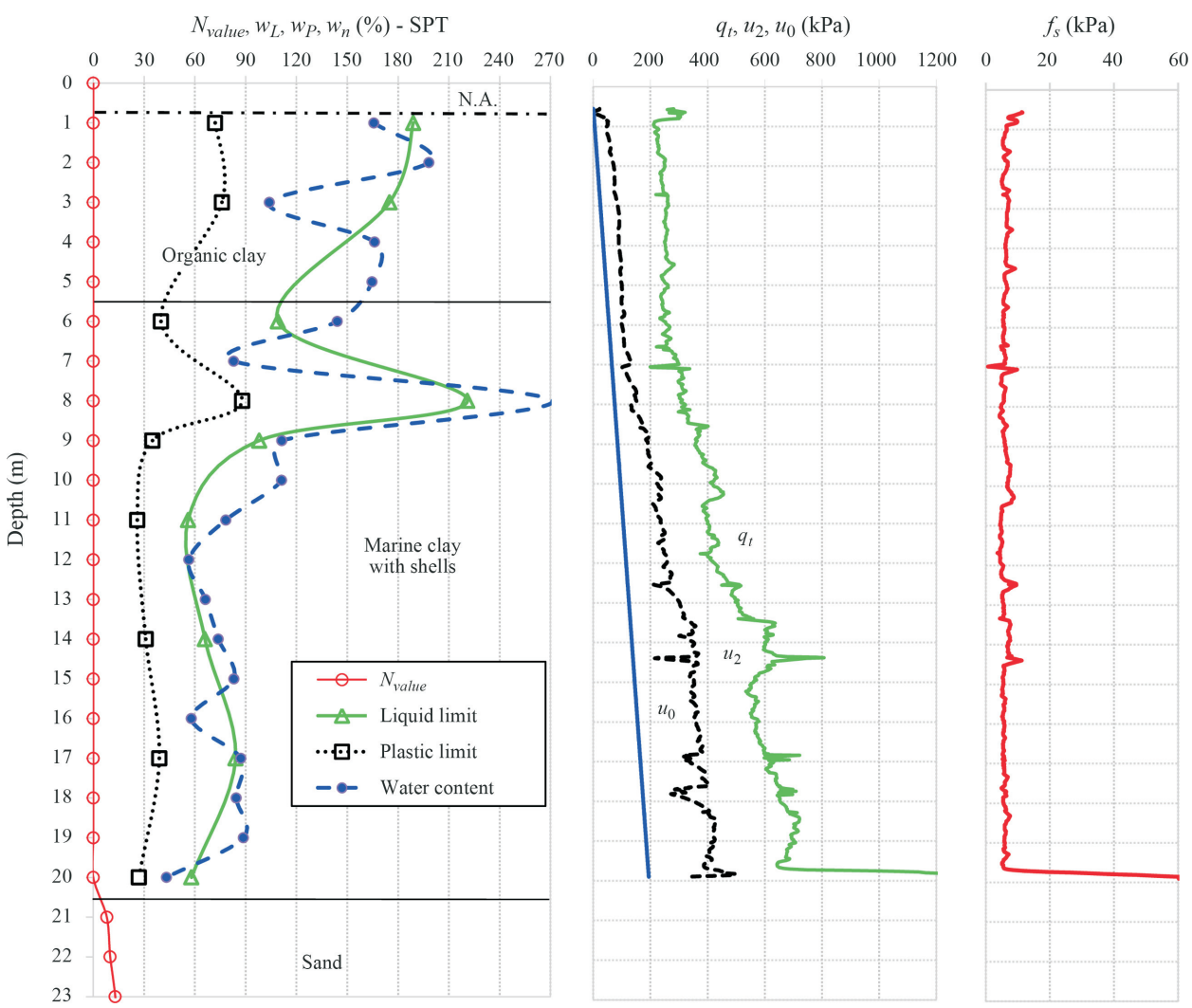

Figure 1 - Typical geotechnical profile.

2.2. Testing program

Piezocone test (CPTU) and field vane tests (FVT) were performed near to the Standard Penetration Test location whose results are indicated in Fig. 1. CPTU was performed between the depths of 0.50 and 20.0 m, with three dissipation tests being performed at depths of 6, 7 and 12 m. At depths between 7.0 and 12.0 were performed the field vane tests (FVT) and also collected six undisturbed samples. The sampling procedures, packaging and transport of the undisturbed samples followed the requirements of the Brazilian standard ABNT (1997).

The laboratory testing program comprised 24 fall cone tests (FCT), 12 vane tests (LVT), 6 unconfined compression tests (UCT) and 6 unconsolidated undrained triaxial tests (UUT).

The undisturbed sampling tubes were segmented as illustrated in Fig. 2, allowing the FCT tests to be performed on the faces of all segments. The FCT and LVT were performed with the soil sample kept in the segmented sampling tube. Subsequently the sample was extracted for moulding the specimens to UUT, UCT and oedometer (OCT) tests.

2.3. Fall cone test (FCT)

The test consists of dropping a standard cone onto the soil under its own weight and after 5 seconds measuring the penetration depth of the cone into the soil. From the penetration depth, the undrained shear strength in both undisturbed (S_u) and remoulded (S_{ur}) conditions can be estimated by the following equation:

$$S_{u(FCT)} = K\frac{W}{d^2} \quad (1)$$

where W is the mass of the cone in grams, d is the penetration depth of the cone in the soil in units of mm, and K is an empirical constant that depends on the cone tip angle (β) and on the cone roughness (ξ).

Hansbo (1957) estimated the value of K by comparing the FCT results with FVT and LVT, with K equal to 1.0 and 0.30 for cone angles of 30° and 60°, respectively, that are used in the Canadian standard CAN (2006) to estimate S_u and S_{ur} (Claveveau-Mallet *et al.*, 2012). Wood (1985 *apud* 1990) found K mean values of 0.85 and 0.29 for cones angles of 30° and 60°, respectively, by comparing results between FCT and LVT. The European standard ISO (2004) indicates K values of 0.80 to 1.0 for cone angle of 30° and 0.27 for 60°.

Houlsby (1982) has presented a theoretical analysis of the cone test for strengths in the same range as those that have already been determined empirically. This analysis reinforces the use of empirical correlations and the relevance of certain variables in the determination of the constant K, such as the cone tip angle and its roughness.

Koumoto & Houlsby (2001) have analysed the cone penetration mechanism into the soil, introducing the concept of dynamic strength for static results. They compared their theoretical K values with those obtained experimentally by other authors, concluding that there was good agreement in the results obtained for a cone with an angle of 60°, whereas for a 30° cone, the theoretical values were slightly higher than those obtained experimentally.

The fall cone tests were performed on the faces of the soil sample kept in the segmented sampling tube following the recommendations of the European standard ISO (2004). The cone has a weight (W) of 80 g, a cone tip angle of 30° and a mean roughness of 0.4 μm. Five measurements of the depth (d) were performed on each face of the segmented sample indicated in Fig. 2, keeping at least 25 mm distance between each point and from the edge of the sampler. Measurements higher than 10% of the mean value were ex-

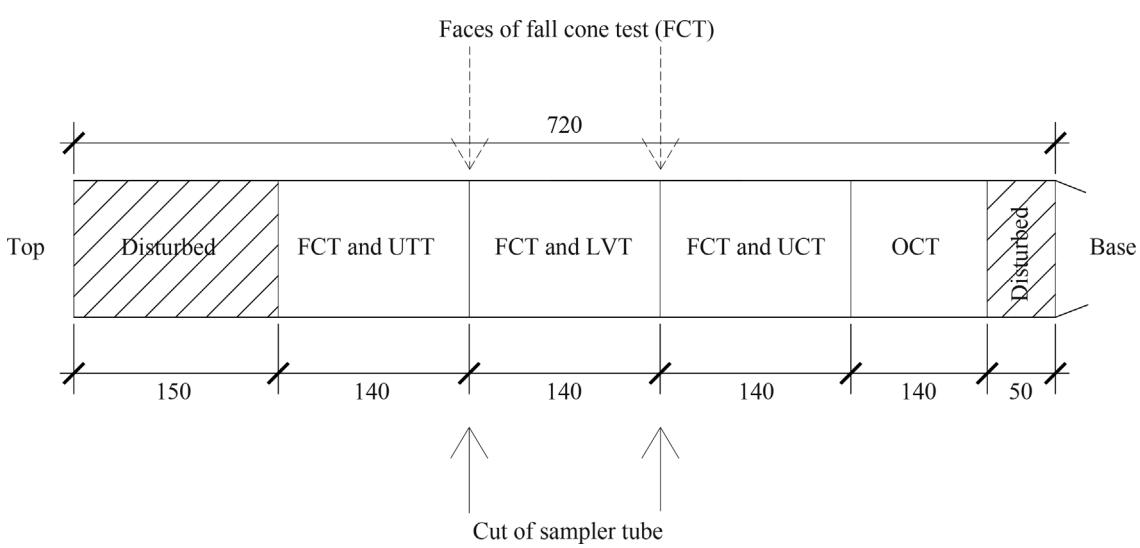

Figure 2 - Undisturbed sampling tubes segmentation for laboratory tests. Values in mm.

cluded from the estimate of S_u and *a K* value 0.80 was adopted, as recommended by the standard ISO (2004).

2.4. Conventional laboratory tests

2.4.1. Laboratory vane test (LVT)

The procedure to perform the LVT followed the recommendations of the American standard ASTM (2010), including those concerning the calibration of the springs. The vane has a height of 25.4 mm and a diameter of 12.7 mm, corresponding to the 2:1 ratio that is recommended to reduce the effects of the anisotropy on the shear strength. The vane was inserted into the soil sample kept in the segmented sampling tube, with a depth equal to twice its height, for measuring undisturbed strength (S_u). Two tests were performed for each segmented samples indicated in Fig. 2 in opposite faces. The remoulded conditions were created after the peak strength was reached. So the vane was manually rotated by ten complete turns, and the test was then repeated. The S_u and S_{ur} values were estimated based on the following equation, for the height of the vane being twice the diameter:

$$S_{u(LVT)} = 0.86 \frac{T}{\pi D^3} \tag{2}$$

where *T* is the maximum torque applied by the spring and *D* is the diameter of the vane in consistent units with strength. The relationship between vane torque *T* and spring deflection measurement in the test was established through the calibration procedure.

2.4.2. Unconfined compression test (UCT)

The UCT was performed following the recommendations of the American standard ASTM (2006). Specimens were moulded for each segmented sampling tube indicated in Fig. 2, except for sampling from 7.0 m depth that was highly fissured and was discarded. They were prepared with a constant height to diameter ratio of 2 and unconfined compression tests with controlled strain were performed. The $S_{u(UCT)}$ value was calculated as half of the unconfined compression strength (q_u).

2.4.3. Unconsolidated Undrained Triaxial Compression Test (UUT)

The UUT was performed in accordance with the recommendations of the American standard ASTM (2003). Specimens were moulded for each segmented sampling tube indicated in Fig. 2 with a constant height to diameter ratio of 2 and wrapped in a membrane. The specimen was inserted into a triaxial cell for the application of confining pressure followed by the application of an axial load. The $S_{u(UUT)}$ value was calculated as half of the deviator stress (σ_d), calculated without correction for membrane effects.

2.5. Conventional in situ tests

2.5.1. Field vane test (FVT)

The field vane tests were performed in accordance with the Brazilian standard ABNT (1989) using a steel vane retracted in the protective shoe for advancement without pre-drilling and the instrument is equipped with slip coupling. The vane prescribed by the Brazilian standard has a diameter of 65 mm, a height of 130 mm, and a vane thickness of 2 mm. The vane retracted in the protective shoe was inserted into the soil and once the desired depth was reached, it was pushed into the soil 0.50 m from the lower part of the protective shoe. Immediately was applied torque at a speed of 6 ± 0.6°/min and the torque curve *vs.* the applied rotation was recorded to determinate S_u. The remoulded conditions were created by rotating the vane rapidly through ten revolutions and the test repeated to determine S_{ur}. The $S_{u(FVT)}$ and $S_{ur(FVT)}$ values were estimated using Eq. 2, where *T* is the maximum value of torque corrected for rod friction measured by slip coupling.

2.5.2. Piezocone test (CPTU)

The cone test with porewater pressure measurements was performed following the recommendations of the American standard ASTM (2012). The penetrometer has a cross section area of 10 cm2 and the filter element located at the base (measurement of u_2). The penetration was performed at a constant speed of 20 ± 5 mm/s, taking automatic measurements of the following parameters: cone resistance (q_c), friction sleeve resistance (f_s) and porewater pressure (u_2). The corrected cone total resistance (q_t) was calculated using the following equation:

$$q_t = q_c + u_2(1 - a_n) \tag{3}$$

where a_n is the ratio between the areas obtained through calibration, which, in this case, was equal to 0.75.

A large number of studies concerning the interpretation of the CPTU to obtain the undrained strength of clays can be found in the literature, representing two different interpretation approaches: one based on theoretical solutions and another based on empirical correlations, generally preferred as reported by Lunne *et al.* (1997b). The empirical approaches estimate S_u by three empirical cone factors, N_{kt}, $N_{\Delta u}$ and N_{ke}, generally used in combination with FVT data being given by the following equations (Danziger & Schnaid, 2000):

$$S_{u(CPTU)} = \frac{(q_t - \sigma_{vo})}{N_{kt(FVT)}} \tag{4}$$

$$S_{u(CPTU)} = \frac{(u_2 - u_0)}{N_{\Delta u(FVT)}} \tag{5}$$

$$S_{u(CPTU)} = \frac{(q_t - u_2)}{N_{ke(FVT)}} \tag{6}$$

In geotechnical engineering practice in Brazil, Eq. 4 is more used (Danziger & Schnaid, 2000; Almeida & Marques, 2014; Coutinho & Schnaid, 2010). In very soft clays, the Eq. 5 has more accuracy in u_2 and u_0 measurements than q_t (Robertson & Cabal, 2015).

3. Results and Discussion

3.1. Field test results

The undisturbed and remoulded strengths obtained through the field vane test (FVT) are presented in Table 1. According to Skempton & Northey (1952) classification, the soil deposit can be considered sensitive.

The N_{kt} and $N_{\Delta u}$ values obtained by Eqs. 4 and 5 and calibrated using the FVT are shown in Fig. 3(a). Typically N_{kt} varies from 10 to 20 (Lunne *et al.*, 1997b; Robertson, 2009). For Brazilian soft clays, Coutinho & Schnaid (2010) reported N_{kt} values between 9 and 18, Schnaid & Odebrecht (2012) between 10 and 20 for normally consolidated or slightly overconsolidated clays and Baroni (2016) between 6 and 18 for soft clays of Rio de Janeiro.

Table 1 - Vane test results.

Depth (m)	$S_{u(FVT)}$ (kPa)	$S_{ur(FVT)}$ (kPa)	$S_t = S_u/S_{ur}$
7.0	2.95	1.72	1.72
8.0	6.60	1.40	4.71
9.0	11.32	1.97	5.75
10.0	14.08	3.02	4.66
11.0	12.34	1.72	7.17
12.0	11.02	1.72	6.41

Although $N_{kt(FVT)}$ values of the studied deposit vary between 17 and 37, values at depths of 7 and 8 m do not have good agreement with the range reported in the literature, so N_{kt} equal to 20 was adopted as representative of the deposit, being slightly higher than Brazilian reported clays. Almeida *et al.* (2010) compared N_{kt} values from several regions of the Brazilian coast and considered the dispersion of the values to be significant, indicating the large variabil-

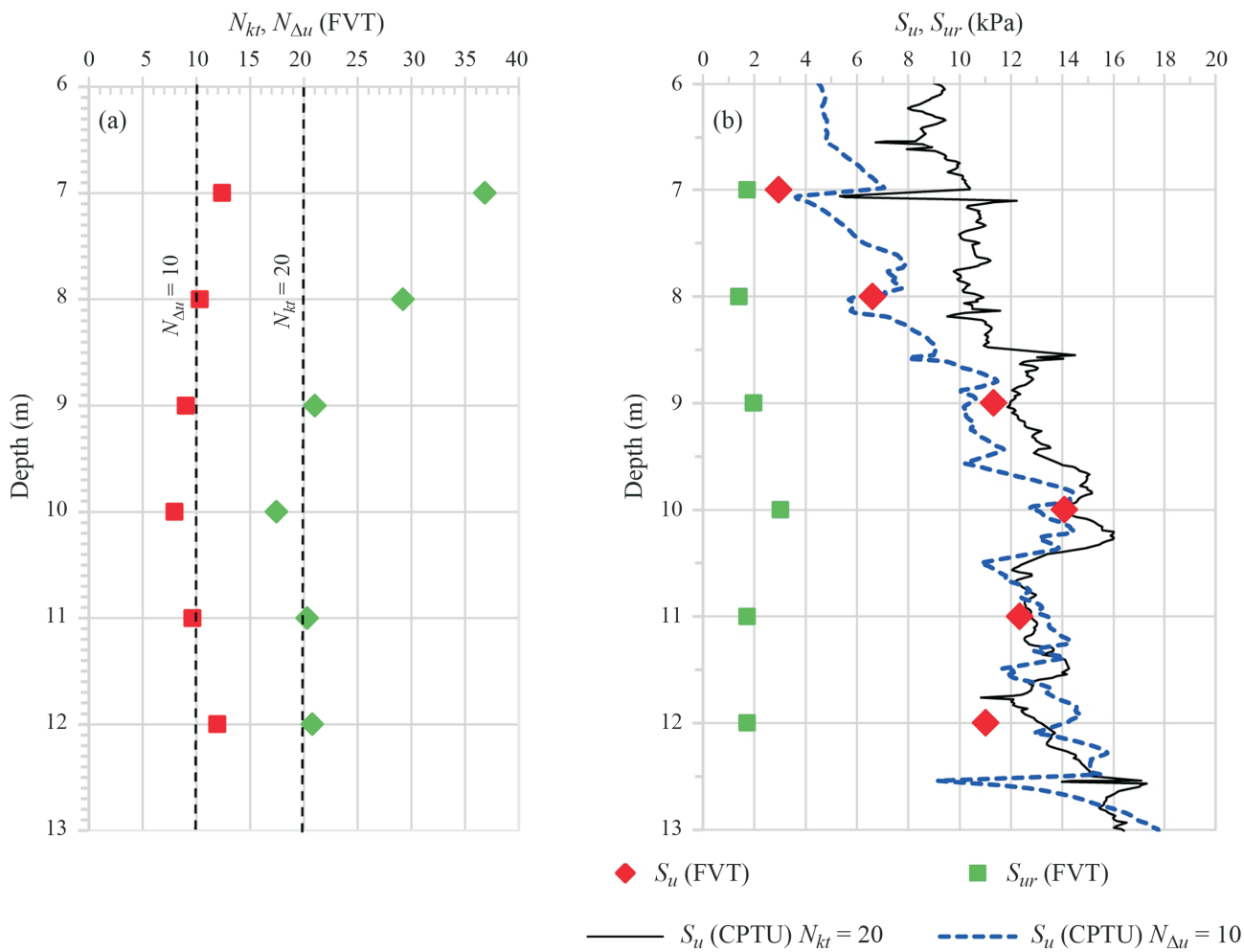

Figure 3 - (a) N_{kt} and $N_{\Delta u}$ values with depth, (b) Strength estimates by FVT and CPTU.

ity of the Brazilian coast soils and the importance of estimating the N_{kt} value for each deposit.

Roberson & Cabal (2015) reported $N_{\Delta u}$ values between 4 and 10. For Brazilian soft clays, Coutinho & Schnaid (2010) reported $N_{\Delta u}$ values between 7 and 9.5 and Coutinho & Bello (2014) between 7.5 and 11 for Recife soft clays. $N_{\Delta u(FVT)}$ equal to 10 was adopted as representative of the deposit, being similar to Brazilian reported clays.

Figure 3(b) shows the undrained shear strength values estimated by *in situ* tests. The S_u estimate from CPTU used the adopted cone factors N_{kt} and $N_{\Delta u}$, respectively, equal to 20 and 10. From 9.0 m depth there was a good agreement between the $S_{u(CPTU)}$ estimates by the two cone factors and between $S_{u(CPTU)}$ and $S_{u(FVT)}$ estimates.

3.2. Laboratory test results

Table 2 shows the quality classification of undisturbed samples based on the ratio between variation in the void ratio (Δe) and initial void ratio (e_o), proposed by Lunne *et al.* (1997a) modified by Coutinho (2007) for Brazilian clays. It is observed that samples numbers 2, 5 and 6 presented poor quality and numbers 3 and 4 presented good to excellent quality.

Overall, the results of tests performed on low-quality samples tend to underestimate S_u. Tanaka (1994, 2008) has observed for LVT tests performed on poor samples that quality of the sample has little influence on the results of S_u, but for UCT tests S_u was underestimated.

A summary of the soil properties obtained from undisturbed samples is presented in Table 3. The natural water content values are closer to the liquid limit and the samples can be subdivided into three groups depending on the I_P value: (1°) I_P greater than 60% and less than 100%, samples 1 and 2; (2°) I_P greater than 100%, samples 3 and 4; and (3°) I_P less than 50%, samples 5 and 6.

Table 2 - Quality classification of undisturbed samples by Coutinho (2007).

Sample number	$\Delta e/e_o$	Classification
2	0.10	Poor
3	0.07	Good
4	0.04	Excellent
5	0.10	Poor
6	0.18	Very poor

The X-ray diffraction measurements indicated that kaolinite and muscovite are the predominant clay minerals, being also detected the presence of quartz, illite and montmorillonite.

3.2.1 Strength results and comparison

Figure 4 shows the relationship between undrained shear strengths estimated from FCT and from conventional laboratories tests: LVT, UCT and UUT. For FCT and LVT it can be concluded that there was good agreement between the results, with a tendency for the $S_{u(LVT)}$ values to be slightly lower than those of the $S_{u(FCT)}$, as shown by regression lines ($R^2 = 0.84$). The same behaviour has also been observed by Rajasekaran & Narasimha Rao (2004) on marine clays treated with lime. Those authors concluded that the FCT test is a good alternative for estimating the undrained strength of clays.

The $S_{u(LVT)}/S_{u(FCT)}$ ratio had a mean of 0.92 with standard deviation of 0.17 and variation coefficient of 1.3%.

For FCT and UCT there was reasonable agreement between the S_u results, with a tendency for the $S_{u(UCT)}$ values to be higher than the $S_{u(FCT)}$, as shown by regression lines ($R^2 = 0.62$). The $S_{u(UCT)}/S_{u(FCT)}$ ratio had a mean of 1.14 with standard deviation of 0.34 and variation coefficient of 30%.

Table 3 - Properties of the soil studied.

Sample number	1	2	3	4	5	6
Depth (m)	7.0	8.0	9.0	10.0	11.0	12.0
Specific gravity of soils particles - G_s (kN/m^3)	27.3	27.5	27.6	26.9	27.0	27.2
Bulk unit weight - γ_{nat} (kN/m^3)	-	14.8	13.6	13.1	15.0	15.2
Clay fraction ≤ 0.002 mm (%)	40	46	52	49	38	36
Silt > 0.002-0.063 mm (%)	36	30	23	28	16	27
Sand > 0.063-2.0 mm (%)	24	24	25	23	46	37
Natural water content - w_n (%)	112	86	139	162	82	76
Liquid limit - w_L (%)	121	95	143	167	77	71
Plastic limit - w_P (%)	34	31	41	45	29	26
Plasticity index - I_P (%)	87	63	102	123	48	44
Void ratio - e_o	-	2.42	3.71	4.25	2.15	2.01
Over consolidation ratio - OCR (by OCT)	-	1.02	1.23	1.23	1.06	-

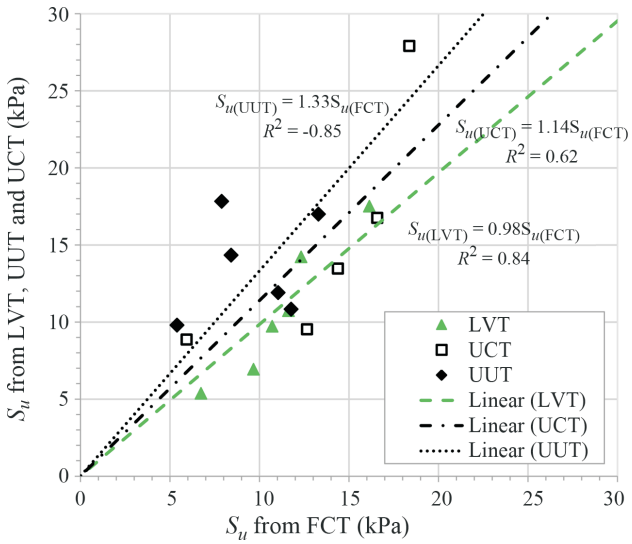

Figure 4 - Correlation between S_u values from FCT and conventional laboratory tests (LVT, UCU and UUT).

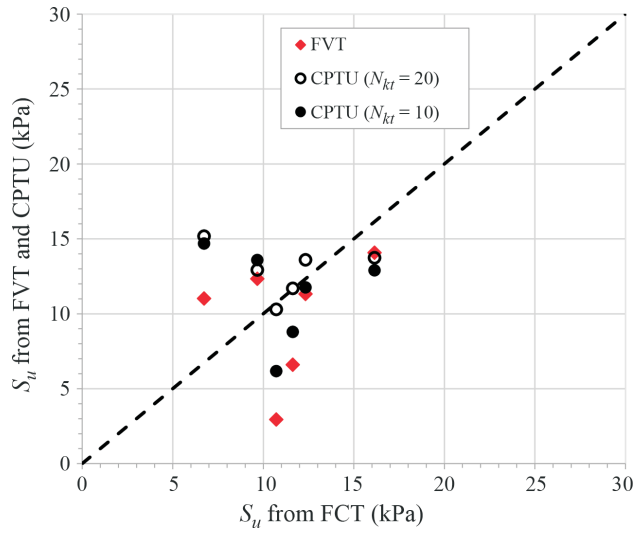

Figure 5 - Correlation between S_u values from FCT and conventional *in situ* tests (FVT and CPTU).

Tanaka *et al.* (2012) have compared S_u data estimated by UCT and FCT from four sites that have been extensively investigated in Japan (Atsuma, Takuhofu, Y-Ariake, & H-Osaka). These sites exhibit different characteristics but similar undrained shear strengths, varying between 20 and 80 kPa. In this study, the author recognised a tendency for the $S_{u(UCT)}$ values to be lower than the $S_{u(FCT)}$, except for the Y-Ariake site. It was observed that the differences could not be attributed to the quality of the samples, consistent with the study of Horng *et al.* (2011), which concluded that the effects of disturbances in the samples are similar for UCT and FCT.

Unexpectedly, the UUT results did not demonstrate good agreement with the FCT, as shown by regression lines (R^2 = -0.85). And the $S_{u(UUT)}/S_{u(FCT)}$ ratio had a mean of 1.51 with standard deviation of 0.51 and a variation coefficient of 34%.

Figure 5 shows the correlation between undrained shear strengths estimated from FCT and conventional *in situ* tests: FVT and CPTU. For these correlations there was observed larger discrepancy between S_u results and it was not possible to establish an adequate linear regression.

The $S_{u(FVT)}/S_{u(FCT)}$ ratio had a mean of 0.92 with standard deviation of 0.49 and variation coefficient of 53%. Despite the larger discrepancy, it was observed a tendency for the $S_{u(FCT)}$ values to be higher than the $S_{u(FVT)}$ values, similarly to Tanaka *et al.* (2012) results.

The $S_{u(CPTU\text{-}Nkt)}/S_{u(FCT)}$ ratio had a mean of 1.25 with standard deviation of 0.52 and variation coefficient of 41%. The $S_{u(CPTU\text{-}N\Delta u)}/S_{u(FCT)}$ ratio had a mean of 1.11 with standard deviation of 0.59 and variation coefficient of 53%.

It is difficult to judge whether variation in soil properties is caused by human factors or by the natural variability in the properties (Tanaka, 2008). The discrepancy observed in Figs. 4 and 5 can be considered to be predominantly attributable to the disturbance of the samples, as their quality was generally classified as poor. However, others important factors were observed in the samples and should be considered, such as the large vertical variability, indicated by liquid limit variations, the presence of shells (Fig. 6a), concretionary materials (Fig. 6b) and thin layers of sand and mica (Fig. 6c). These factors may have influenced the

Figure 6 - Samples variability: (a) shells, (b) concretionary material and (c) layers of fine sand and mica.

laboratory test results, particularly the FCT and LVT, and are also an indicative of horizontal variability.

Figure 7 may help to understand the larger discrepancy between S_u results presented in Fig. 5. It is observed that S_u estimated by CPTU ranged between 3.6 kPa to 17.6 kPa with considerable variations at certain depths, such as between 7.0 and 7.2 m. For correlations presented in Fig. 7, the mean value of $S_{u(CPTU)}$ in 1.0 m range was considered, thus it was not possible to verify by this analysis if there was good agreement between the FCT and CPTU results.

Despite the heterogeneity of the deposit and discarding the very discrepant values of $S_{u(UCT)}$ and $S_{u(UUT)}$, it can be visually observed in Fig. 7 that between the depths of 7.0 and 11.0 m there is a good agreement between the laboratory S_u results and $S_{u(CPTU)}$ estimated with N_{kt} cone factor. Between depths of 11.0 and 13.0 m the $S_{u(FCT)}$ and $S_{u(LVT)}$ do not present good agreement with $S_{u(CPTU)}$. These samples presented the lowest plasticity index (mean of 46) and the highest percentage of sand (mean of 42%). Maybe a drained behaviour can explain this greater variation among the S_u results. Larsson *et al.* (1987) observed that $S_{u(FCT)}$ values measured in specimens from greater depths than 10 and 15 m are often too low and the same behaviour occurs in clays of low plasticity and high sensitivity, which also can explain the lower $S_{u(FCT)}$ results between depths of 11.0 and 13.0 m.

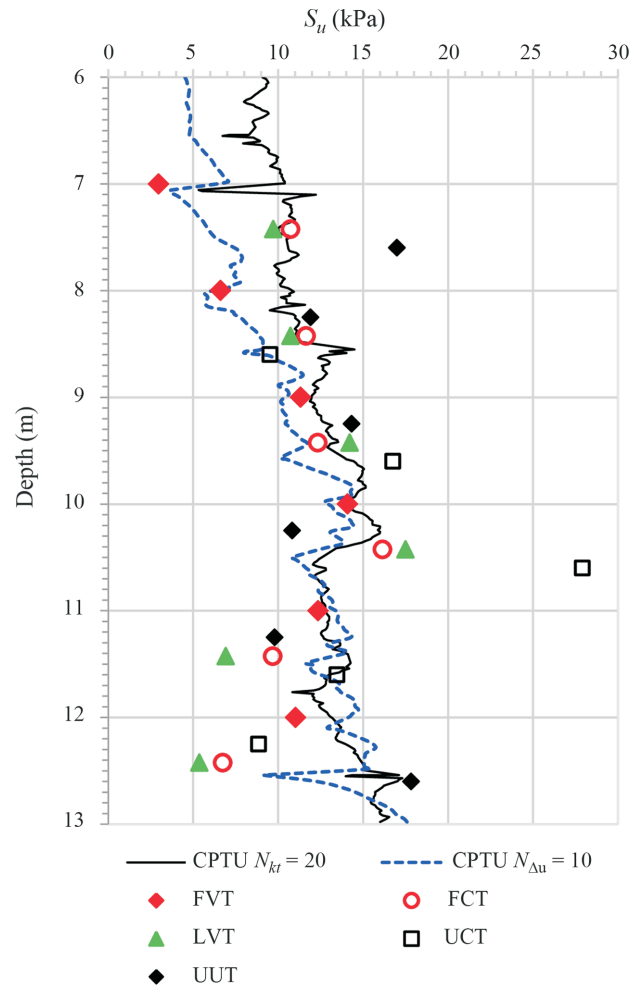

Figure 7 - Undrained shear strength estimated by conventional and unconventional tests.

3.2.2. Empirical correlations

Attempts to develop simple methods for estimating the undrained shear strength of soils based on physical indices, such as correlations based on plasticity index, have been conducted since the beginning of Soil Mechanics (Kempfert & Gebreselassie, 2010). However, several of the most well-known empirical correlations were established using data from soils obtained in countries of northern Europe and America, where the sediments were strongly influenced by the glaciers of the ice age period (Tanaka, 2000; Tanaka *et al.*, 2001).

Although the correlations developed in a given geological context are not universally applicable and should be used with caution, as well as be calibrated locally (Larsson & Ahnberg, 2005; Leroueil *et al.*, 2001), empirical correlations between the undrained shear strength (S_u) and the plasticity index (I_P) can be used to support and complement strength determinations (Larsson *et al.*, 1987). Some of these correlations are presented in Table 4.

Table 4 - Empirical correlations between normalised S_u and I_P (adapted from Kempfert & Gebreselassie, 2010).

Equation	Reference	Applicability
$S_u/\sigma'_{vo} = 0.0037I_P + 0.11$	Skempton (1957)	NC soils, $I_P > 10\%$
$S_u/\sigma'_P = 0.0024I_P + 0.2$	Leroueil *et al.* (1983)	Clays from eastern Canada, $I_P < 60\%$
$S_u/\sigma'_P = 0.003I_P + 0.14$	Lambe & Whitman (1969)	All clays
$S_u/\sigma'_P = 0.45(I_P/100)^{1/2}$	Bjerrum & Simons (1960)	NC clays
$S_u/\sigma'_P = 0.22$	Mesri (1975)	Soft clays
$S_u/\sigma'_P = 0.0043I_P + 0.129$	Wroth & Houlsby 1985	NC clays

σ'_{vo}: initial effective vertical stress; σ'_p: preconsolidation stress.

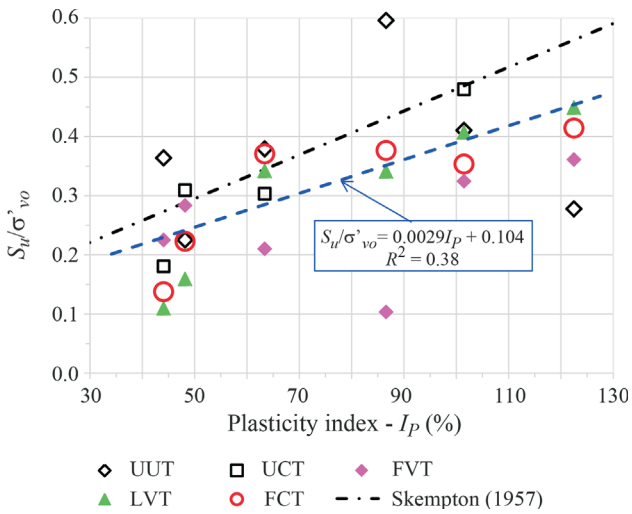

Figure 8 - Relationship between normalised undrained shear strength with effective vertical stress (S_u/σ'_{vo}) and plasticity index (I_P).

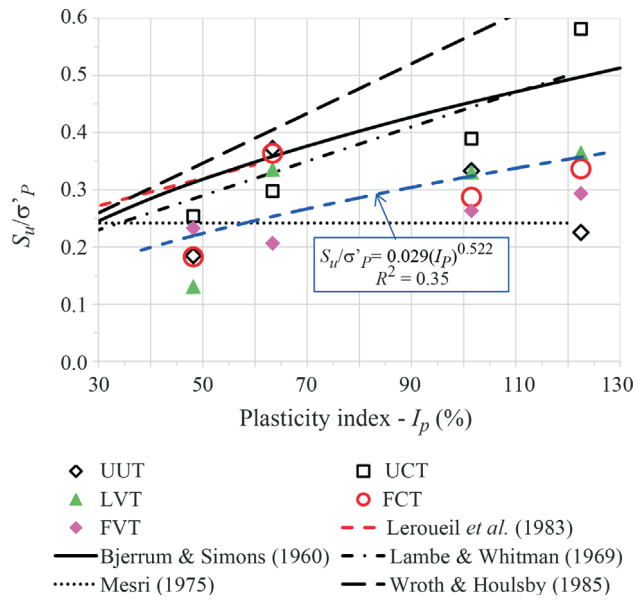

Figure 9 - Relationship between normalised undrained shear strength with preconsolidation stress (S_u/σ'_P) and plasticity index (I_P).

Many of these correlations indicate a tendency for S_u/σ'_P or S_u/σ'_{vo} increase with increasing of I_P. Figures 8 and 9 illustrates the relation between S_u/σ'_{vo} and S_u/σ'_P and I_P using the *in situ* test data (FVT) and laboratory test data (FCT, LVT, UUT, UCT) obtained in this study and the empirical correlations presented in Table 4.

Despite the large dispersion, it can be observed in Fig. 8 that exists a tendency for S_u/σ'_{vo} increase with increasing of I_P, as shown by linear correlation ($R^2 = 0.38$), similar to the Skempton linear correlation and being more accentuated for I_P lower than 70%. Baroni (2016) reported for Rio de Janeiro soft clays that there is not a tendency for S_u/σ'_{vo} increase with I_P.

Figure 9 was elaborated with S_u/σ'_P results of samples number 2 to 5 and presents the same behaviour of Fig. 8, a tendency for S_u/σ'_P increase with increasing of I_P and being more accentuated for I_P lower than 70%. The regression line presented the same behaviour of Bjerrum & Simons potential correlation. Similarly, Futai *et al.* (2008) have observed for clay deposits in Rio de Janeiro that the S_u/σ'_P ratio demonstrates a tendency to increase with increasing of I_P, similar behaviour of the Canada clays.

As reported by Tanaka (1994), the S_u/σ'_P ratios determined for various Japanese marine clays ranged between 0.25 and 0.35 and did not exhibit any significant relationship to I_P, being S_u values estimated by FVT and I_P values ranging between 20% and 150%. Chung *et al.* (2007) had also concluded for a specific Japanese marine clay deposit that the S_u/σ'_P ratio did not depend on I_P.

4. Conclusions

In the present study, the undrained shear strength (S_u) results from laboratory fall cone test (FCT) were compared with the S_u results from conventional field and laboratory tests commonly used in geotechnical engineering to estimate this parameter in cohesive soils: CPTU, FVT, UCT, UUT and LVT. The normalised undrained shear strength estimates were compared with some empirical correlations based on plasticity index. The following conclusions result from this study:

- The S_u values determined by FCT presented good agreement with the S_u determined by LVT, with $S_{u(LVT)}/S_{u(FCT)} = 0.98$ obtained by linear regression (coefficient of determination $R^2 = 0.84$).
- For FCT and UCT there was reasonable agreement between the S_u results, with a tendency for the $S_{u(UCT)}$ values to be higher than the $S_{u(FCT)}$. The $S_{u(UCT)}/S_{u(FCT)} = 1.14$ was obtained by linear regression (coefficient of determination $R^2 = 0.62$).
- FCT and UUT did not demonstrate good agreement between the S_u results, with variation coefficient of 34% for the $S_{u(UUT)}/S_{u(FCT)}$ ratio.
- The S_u values determined by FCT did not present good agreement with the S_u determined by FVT, with variation coefficient of 53% for the $S_{u(FVT)}/S_{u(FCT)}$ ratio.
- Despite the considerable variations between $S_{u(CPTU)}$ values estimated with N_{kt} cone factor for certain depth ranges, there was a good agreement with $S_{u(FCT)}$ and $S_{u(LVT)}$ for depths until 11.0 m.
- The difference between S_u values determined through laboratory and *in situ* tests can be assigned to others important factors like the large vertical variability indicated by liquid limit variations and the presence of shells, concretionary materials and thin layers of sand and mica

that may have influenced the laboratory test results, particularly the FCT and LVT.

- The normalized undrained shear strength data, S_u/σ'_p and S_u/σ'_{vo}, determined using the various test methods presented a tendency to increase with increasing of I_P, similar to some empirical correlations reported in the literature.

As a final contribution of this study, considering the simplicity and flexibility of the fall cone test (FCT) application and the possibility to collect a greater number of data, it would be appropriate to use this method to support and complement other strength determinations.

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