Comparative assessment of soil behavior by *in situ* and laboratory tests

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ABSTRACT: Soil characterization can be carried out by means of *in situ* and/or laboratory testing. Both methodologies are complementary, but sometimes present differences between their results. This paper aims to compare the soil behavior estimated by cone penetration tests (CPTu) with triaxial test results performed on remolded specimens, namely in terms of contractive or dilative behavior.

For this purpose, a soil classified as silty sand was tested in a triaxial apparatus under consolidated drained and consolidated undrained conditions. The tested samples were reconstituted using the unit weight obtained from CPTu correlations. Such values were used to define the initial void ratio of the samples, i.e, the state parameter of the soil on the field. However, when comparing the CPTu interpretation against laboratory tests results, the soil behavior of both methodologies differs according to initial void ratio depending on the correlation used for estimating the *in situ* state-condition. Since the estimation of the *in situ* unit weight has a very important influence on the interpreted behavior, it should be evaluated carefully during a field campaign by retrieving undisturbed samples which is difficult in sandy deposits.

Keywords: CPTu, unit weight, void ratio, dilative soil, contractive soil

1. Introduction

The actual methodologies for the interpretation of CPTu tests are based on empirical correlations, which cover a large range of soils. Among different approaches, the one proposed by Robertson [1–3] has become very popular. This methodology normalizes the basic values obtained from CPTu, cone tip resistance (q_t) and sleeve friction (f_s), in terms of *in situ* vertical effective stress (σ'_{vo}). In addition, using this data it is possible to compute the Soil Behavior Type Index (I_c) [4].

The results obtained from *in situ* tests provide a very good approach to estimate the soil behavior. However, to obtain a complete soil characterization, the *in situ* values should be confirmed through laboratory tests. For this purpose, usually, the experimental plan in the laboratory includes physical and mechanical tests, namely triaxial tests. Undisturbed samples are not always available, especially in sandy deposits where undisturbed sampling is more challenging [5]. In those cases, triaxial tests are performed on reconstituted specimens prepared to the estimated *in situ* conditions in terms of void ratio, water content and consolidation stresses. For this purpose, the estimation of the soil unit weight at depth is needed and without undisturbed samples, this is often performed relying on CPTu correlations. In some cases, where the geological conditions are similar to the ones used to develop those correlations, the estimation is very good. However, they cannot be generally applied for all conditions and it is not straight forward to know in advance if a certain correlation is adequate or not.

This paper discusses the comparison of the different behavior (contractive or dilative) obtained from CPTu interpretation against triaxial tests results. The studied soil was collected at 4 m depth in an experimental site located in Vila do Conde (Portugal).

2. Test Site and CPTu interpretation

The experimental site of this study is in Vila do Conde, at the north of Portugal, where the new shipyards of Vila do Conde were installed. This area is very close to the mouth of Ave River in the Atlantic Ocean. Because of the position, this area has a strong tidal influence. The tides in this region of the coast are wide frequently achieving 4 m of water height between the low and the high tide [6–8]. Figure 1 displays the location of the experimental site and the three tests points.

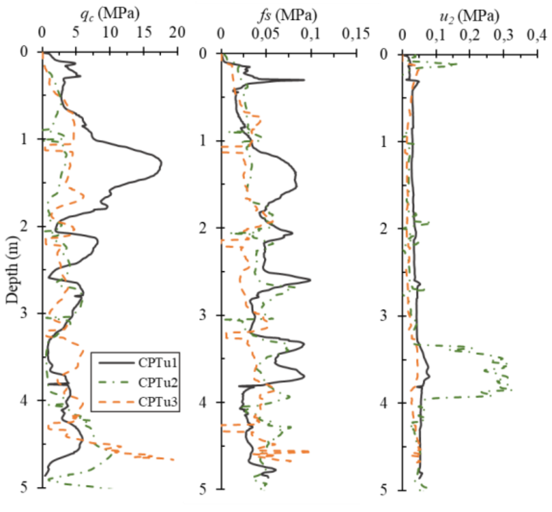
Figure 1. Location of the experimental site (adapted from Google Earth).

The site presents coastal or estuarine situation, subject to sedimentation predominantly due to the sea agitation, as well as the effect of tidal currents and fluvial inflows.

For this reason, it is not expected that cementation could be significantly developed. The activity of shipbuilding and repair is relevant in Vila do Conde [9]. In the last decade of century XX, in this site, a new modern shipyard was installed, and the old services and surroundings of the shipyard were abandoned. In 2003, it was decided to re-qualify the area, installing a shipbuilding museum, using new facilities in the old yard area and the old customs building [10].

Light Dynamic Probing (DPL) and Cone Penetration tests (CPT) tests up to 5.6 m depth were performed in three different points. The DPL were performed as a preliminary inspection to identify the depth with higher strength at which the CPT cone could be damaged [8].

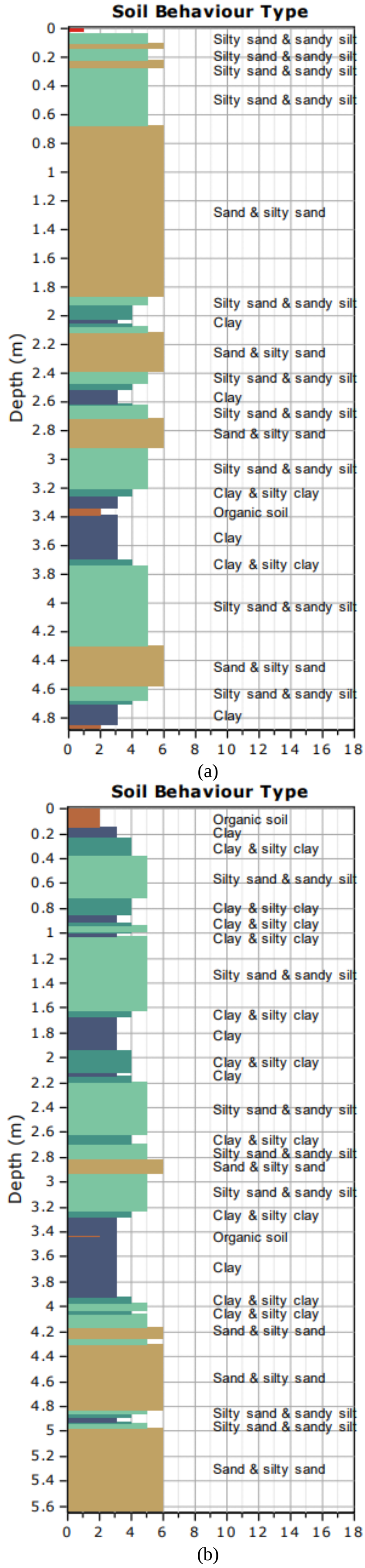
The CPTu were performed on shore in an area, that was dry (and accessible) on the low tide, and consequently the water table depth was not constant throughout the tests. After removing the cone penetrometer device, the water table depth was measured in the hole left by the cone ($CPTu_1$ = 0.5, $CPTu_2$ = 0.0m and $CPTu_3$ = 0.2m). This measurement was used for the CPTu tests interpretation. Figure 2 presents the basic results of the three CPTu.



(a) (b) (c)

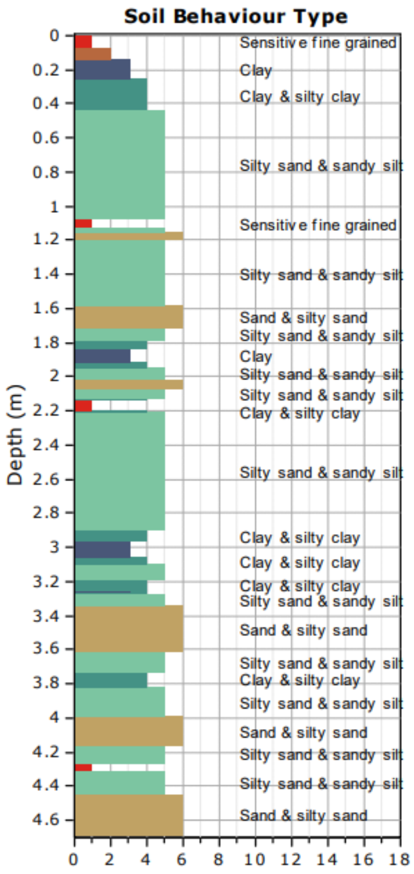
Figure 2. Results of the CPTu: a) cone resistance; b) sleeve friction; c) pore-water pressure.

By analyzing and interpreting CPTu data in CPeT-IT® [11] the soil profiles based on the classification of normalized soil behavior type (SBTn) proposed by Robertson [1–3] were obtained. Figure 3 presents the SBTn results. From SBTn profiles, it was identified layers of silty soils with the presence of sandy interlayers from the surface until 5.60 m depth. Disturbed samples were collected using a simple auger, and such soils were studied in the laboratory [6].



(a)

(b)



(c)

Figure 3. Soil profiles based on SBTn: (a) CPTu1; (b) CPTu2; (c) CPTu3.

On the other hand, CPTu data was used to estimate the soil unit weight (γ) by applying two different correlation models. Eq. (1) and Eq. (2) present the correlations used in this study, which correspond to the proposal of Robertson and Cabal [12] and Mayne *et al.*,[13], respectively. Robertson and Cabal [12] made a summary of their research on about 20 different types of soils among loose sand, silty sand and sensitive clays. Mayne *et al.*,[13] correlation was developed from 14 offshore deposits (Gulf of Mexico (3 sites), East Indian Ocean and West Australia, for example) that were subjected to comprehensive *in situ* testing, sample logging and laboratory testing. According to the author that methodology can be used in a variety of soil types, including clays, silts, sands, and mixed soils of varying consistencies.

$$\frac{\gamma}{\gamma_w} = 0.27 \cdot \log\left(Rf\right) + 0.36 \cdot \log\left(\frac{q_t}{Pa}\right) + 1.236 \tag{1}$$

where:
γ_w is the water unit weight,
Rf is the friction ratio $Rf = f_s/q_c$,
q_t is the corrected cone resistance $q_t = q_c + (1 - \mathrm{a}) \cdot u_0$ (for this specific cone a = 0.58),
p_a is the atmospheric pressure.

$$\gamma = 11.46 + 0.33 \cdot \log\left(z\right) + 3.10 \cdot \log\left(f_s\right) + 0.70 \cdot \log\left(q_t\right) \tag{2}$$

where:
z is the depth,
f_s is the cone sleeve friction,
q_t is the corrected cone resistance $q_t = q_c + (1 - \mathrm{a}) \cdot u_0$.

3. Laboratory tests

A sample from 4 m depth, collected in the second point (CPTu2), was used for the experimental plan in the laboratory. Table 1 summarizes the most relevant physical parameters of the studied soil. Figure 1 presents grain size distribution analysis, indicating that this soil is mainly composed by sand and a minor portion of silt (13%).

Table 1. Geotechnical properties of the soil

Plastic Limit	NP
Liquid Limit	NP
D_{50}	0.35 mm
Specific gravity	2.66
Fines fraction	13%
Uniformity Coefficient	16.83
Curvature Coefficient	7.41
Maximum void ratio	0.83
Minimum void ratio	0.44

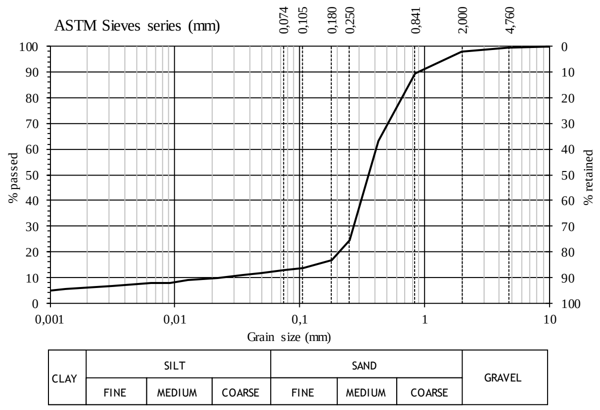


Figure 4. Grain size distribution of the soil taken from point 2 using a simple auger at 4 m depth.

On the other hand, two series of triaxial tests were conducted to assess the soil behavior in the laboratory. In the first series of triaxial tests, specimens were prepared for the state-condition obtained using the γ correlation proposed by Robertson and Cabal [12], while the second series were prepared using the γ correlation proposed by Mayne *et al.*,[13]. The first gave average values 18.9 kN/m^3 while the second an average value of 20.3 kN/m^3. From Eq. (3) the *in situ* void ratio (*e*) was calculated based on the unit weight (γ) values leading to 0.85 and 0.94 and 0.61 respectively.

$$e = \frac{G_s \cdot \gamma_w - \gamma}{\gamma - \gamma_w} \tag{3}$$

where:
e is the void ratio,
G_s is the specific gravity (Table 1),
γ is the soil unit weight,
γ_w is the water unit weight (assumed 9.81 kN/m^3).

The specimens were prepared using the moist tamping method [14], mixing the soil with 5% of water content.

This mixture was poured in six layers into a metallic mold with 70 mm diameter and 140 mm height [14]. Once in the triaxial cell, specimens were percolated with Carbon-Dioxide, CO_2, and deaired water to facilitate air removal. This procedure allowed obtaining saturated specimens with B-Skempton parameters close to 1 (B-value > 0.98). The shearing phase was conducted with controlled displacement, using a rate of 0.01 and 0.05 mm/min for drained and undrained, respectively.

The first four specimens, corresponding to the estimation of Robertson and Cabal [12] were prepared very loose. These samples were consolidated under different isotropic effective consolidation pressures (10, 35 and 50 kPa). Two additional tests in denser specimens were performed and the specimens were therefore prepared and consolidated to 35 and 150 kPa. Table 2 summarizes the triaxial test program indicating the specimens void ratios during the tests. It should be noted that the e_{max} gave a value of 0.79, while specimen 1 was prepared with 0.82. This slightly higher value of the void ratio is due to the suction induced by the moist tamping procedure.

Table 1. Summary of the specimens with their void ratios during the different test stages

Test name	Draining conditions	Effective isotropic consolidation Stress (kPa)	Final void ratios	
			Cons.	Shear
1 - Robertson and Cabal unit weight	Undrained	35	0.82	0.82
2 - Robertson and Cabal unit weight	Drained	10	0.78	0.75
3 - Robertson and Cabal unit weight	Drained	35	0.75	0.70
4 - Robertson and Cabal unit weight	Drained	50	0.76	0.72
5 – Mayne et al., unit weight	Drained	35	0.66	0.71
6 – Mayne et al., unit weight	Undrained	150	0.66	0.60

4. Results and Discussion

First, the state parameter was evaluated from the interpretation of CPTu results. According to Been and Jefferies [15] the state parameter (Ψ) is the best way to evaluate the behavior of sands. The state parameter combines effects such as stress level and density, while the relative density (D_r) just informs about the soil density. The state parameter obtained by CPTu data was show in Figure 5. The gray square in Figure 5A and 5B evidences the depth at which the soil was collected to prepare the samples for the tests. Figure 5A and 5B show the state parameter chart for the case where all the layers have a unit weight 18.08 kN/m^3 obtained by Robertson and Cabal [12] (CASE A), and for the case where all the layers have a unit weight 20.3 kN/m^3 obtained by Mayne *et al.*,[13] (CASE B). The differences between both approaches are minor, and all layers was are on the dilatant side.

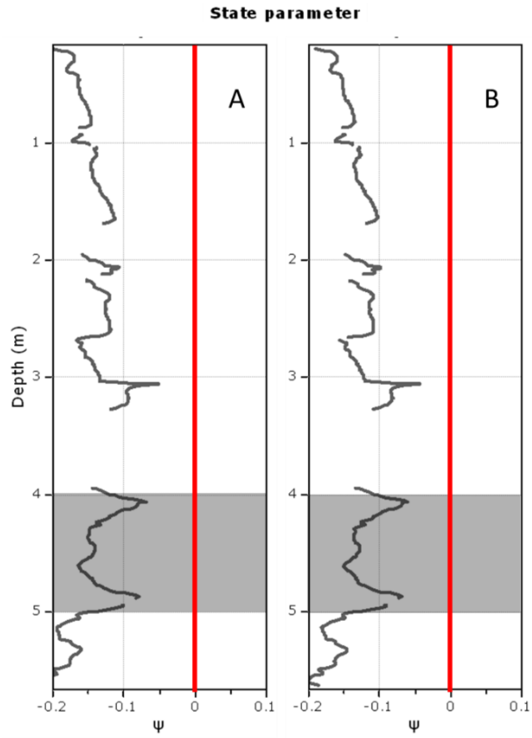


Figure 5. State parameter chart for CPTu$_2$, where A, all the layers with unit weight 18.08 kN/m^3 obtained by Robertson and Cabal [12] and B all the layers with unit weight 20.3 kN/m^3 obtained by Mayne *et al.*,[13].

The initial version of the chart proposed by Robertson [1] indicated 9 zones based on the physical characteristics of soils from sensitive fine grained soft clays to silts, sands, and very stiff fine grained soils providing good agreement between the unified classification system ASTM [16] and CPT based soil behavior type (SBTn). The new updated version, Figure 6, provides areas where soils will be likely dilative or contractive through the introduction of a boundary (marked $CD = 70$) represented by Eq. (4). When $CD > 70$, the soils are likely dilative at large strains.

$$CD = 70 = (Q_{tn} - 11)(1 + 0.06F_r)^{17} \qquad (4)$$

where,

Q_{tn} is the normalized cone resistance
F_R is the normalized friction

According to Figure 5 all layers in point 2 have a negative state parameter which would indicate a dilative behavior. This is clearly seen in Figure 6 since all points plot over the CD>70 line, meaning that a dilatant behavior is expected.

The soil behavior type index (I_c) proposed by Robertson [1] was also updated to I_B defined in Eq. (5) which has a more hyperbolic shape,

$$I_B = 100(Q_{tn} + 10)/(Q_{tn}F_r + 70) \qquad (5)$$

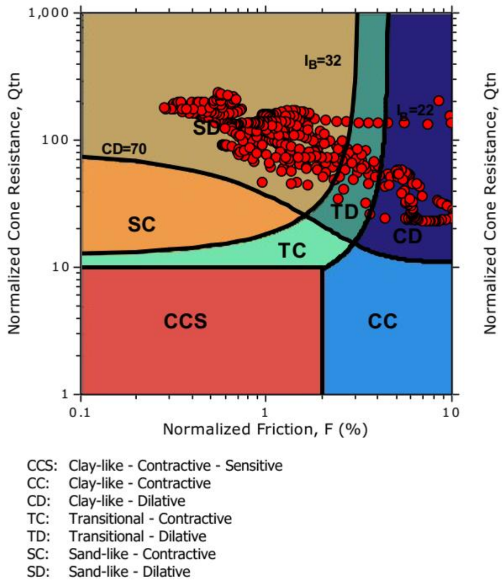


Figure 6. Modified Soil Behavior Type normalized.

The boundary represented by I_B = 32 represents the lower boundary for most sandlike soils corresponding to the areas identified as SD and SC in Figure 6. The boundary I_B = 22 represents the upper boundary for most claylike soils corresponding to the areas identified as CD and CC in Figure 6. The region represented by 22< I_B < 32, Figure 6, is defined as "transitional soil" to represent soils that can have a behavior somewhere between that of either sandlike or claylike ideal soil. For these soils, an expected partial drained behavior may be expected during a CPT test, and so these soils are also called as intermediate soils.

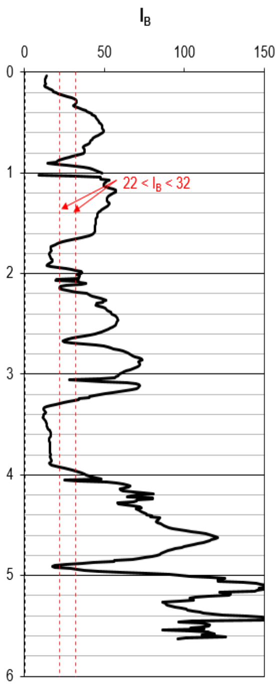


Figure 7. Estimated parameters from the CPTu$_2$ test performed at new soil behavior type index (IB) from Robertson [3].

In the chart proposed by Schneider *et al.*, [17], relating the change in pore pressure (u2) normalized by the effective stress and the Q_{tn}, it is possible to understand the soil is falling in the Sensitive clays area. Figure 8 shows that this is not the case for this soil and therefore this is not the reason for the dilative behaviour of the soil. This reinforces the idea that Mayne et al. correlation is more adapted to this soil than the Robertson and Cabal [12].

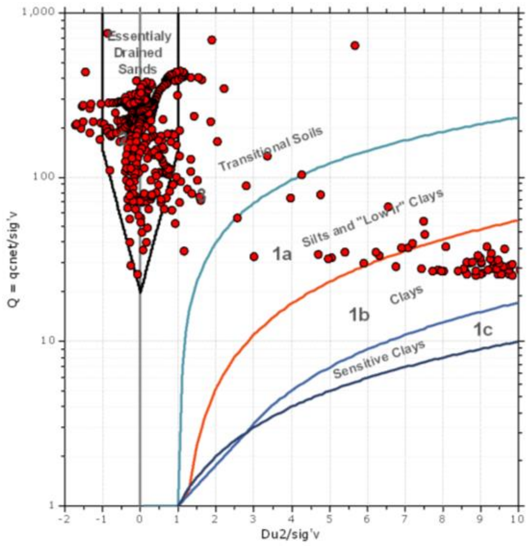


Figure 8. Chart proposed by Schneider *et al.*, [17], relating the change in pore pressure (u2) normalized by the effective stress and the Q_{tn}

The interpretation of the triaxial tests data, Figure 9, showed different soil behaviors. The 1, 2, 3 and 4 specimens prepared with the void ratio obtained from the Robertson and Cabal [12] correlation had a contractive behavior, which is different from what could be expected from the *in situ* results. The others two samples, 5 and 6, prepared with the correlation from Mayne *et al.*,[13] show dilatant behavior.

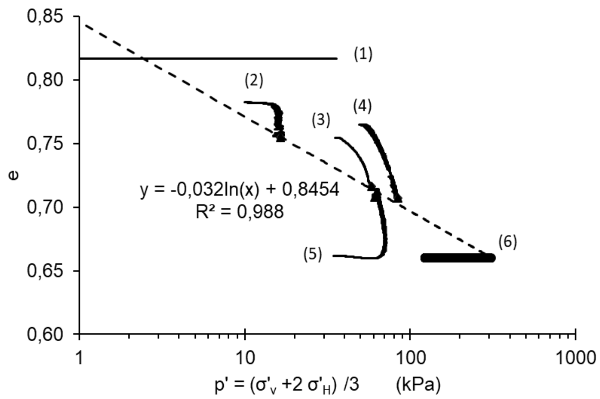


Figure 9. Critical state envelope: mean effective stress (p') versus void ratio (e).

In the graph of Figure 9 it is clear that the soil behaves differently if the void ratio was calculated using the unit weight obtained by expressions 1 or 2. It is important to note that the difference in behavior between the CPTu and triaxial tests was found when the unified approach from Robertson [1–3] was applied. This problem raises the question if, for this kind of soil, the state parameter is reliable. For this reason, the analysis of the relative density was performed to understand if this parameter is more sensitive than the state parameter for the behavior of this soil. According to Cubrinovski and Ishihara [18] and Ishihara *et al.,* [19] the relative density (D_r), is a key parameter to express the looseness or the denseness of cohesionless soils relative to their maximum and minimum states of compactiveness, being D_r defined by Eq (6).

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{6}$$

where:
e is the void ratio
e_{max} is the maximum void ratio
e_{min} is the minimum void ratio

The e_{max} and e_{min} used in Eq. (6) were obtained using the precepts contained in Japanese Standards and Explanations of Laboratory Tests of Geomaterials (JGS) [20]. For e_{max} only the JGS method is necessary but for the e_{min} calculation it is necessary to adjust the e_{min} values according to the fines content using Eq. (8) [14].

$$\frac{e^*_{min}}{e_{min}} = -0.012Fc + 1.06 \tag{8}$$

where,
e^*_{min} is the adjusted void ratio
e_{min} is the minimum void ratio obtained by (JGS) [20]
Fc is the fine content defined as 13% (see Table 1)

In the unified approach for Robertson [1–3] the relative density is calculated by the Normalized CPT Soil Behavior Type (SBTn) using the Q_{tn} parameter as shown in Eq. (7)

$$D_r = 100\sqrt{\frac{Q_{tn}}{k_{DR}}} \tag{7}$$

where:
Q_{tn} is the normalized cone resistance
k_{Dr} is the relative density constant

Actually, the conventional method to calculate D_r (reported in Ishihara *et al.,* [19]) is applied to estimate the post-liquefaction settlements but the simplicity of the procedures contained in (JGS) [20] make it favorable to this kind of assessment. For the soil present in this study, the calculated D_r is present in Table 3.

Table 3. Summary of the specimens with their D_r(%)

Test name	D_r(%)
1 - Robertson and Cabal unit weight	1.76
2 - Robertson and Cabal unit weight	12.0
3 - Robertson and Cabal unit weight	19.7
4 - Robertson and Cabal unit weight	17.1
5 – Mayne et al., unit weight	42.7
6 – Mayne et al., unit weight	42.7

On the unified approach from Robertson [1-3], the calculation of D_r with Eq (7), gave a minimum value of 36% for the area involved in this study, corresponding the depth between 4 and 5 m on point 2 (gray square in Figure 5). This minimum value was obtained with Q_{tn} being calculated either using the unit weight of Robertson and Cabal [12] or the unit weight of Mayne *et al.,*[13]. This means that values between 12 and 19 % are not very reasonable.

Been and Jefferies [15] points out that the problem with the relative density approach is that is does not considers the influence of stress on soil behavior. However, the same authors in Been *et al.,* [21] explains that in a silty sand soil, the same of the present work, is not incorrect the use of the relative density to access the soil behavior, not only to the laboratory tests but also in in situ tests like CPTu. This is not far from the methodology followed in the actual interpretation of the CPTu tests where the unified approach uses the same parameter to calculate both D_r and the state parameter Ψ - the normalized cone resistance (Q_{tn}) as show in Eq. (9).

$$\psi = 0.56 - 0.33\log(Q_{tn,cs}) \tag{9}$$

where,

ψ is the state parameter
$Q_{tn,cs}$ is the normalized cone resistance

Another point that may influence the soil behavior is the soil formation, and this is not expressed in the unit weight correlations. In works of Young [22], Vaughan *et al.,* [23], Sandeep and Senetakis [24], Sridharan [25], Quigley [26] and Churchman *et al.,* [27] is evident that soil formation may be the one important factor governing soil behavior, whether it will dilate or contract.

5. Conclusions

In this work, a very different behavior (dilatant or contractive) was observed in conventional triaxial compression tests when the void ratio changed from 0.82 to 0.60 in the initial void ratio. Without having undisturbed samples, the *in situ* unit weight was calculated by two different empirical correlations, which gave small differences, yet enough to lead to distinct behavior. To understand which correlation was providing the most reliable unit weight, the laboratory results were compared to the CPTu data interpreted by the unified approach proposed by Robertson [1-3]. The CPTu data indicated always a dilatant behavior, independently of the correlation used for the unit weight. This indicates that the correlation proposed by Mayne et al. [13] is probably the most adequate.

In addition, a comparison was made between the state parameter and the relative density to understand the differences in the behavior showed in the interpretation of *in situ* (CPTu) tests and laboratory tests (Triaxial). Again the relative density calculated by the unified approach seems to agree well with the unit weight proposed by Mayne et al. [13]. This is also in agreement with the state parameter obtained by the unified approach and with the laboratory tests prepared with a void ratio corresponding to Mayne et al. [13] expression. It should be noted that both the state parameter and the relative density obtained by the unified approach are both dependent of a single parameter – the normalized cone resistance (Q_{tn}).

Moreover, it should be noted that the disturbed sampling performed in this work with a simple auger, could have led to the collection of a mixture of soils from different depths. In this sense, it is possible that the fines content of the soil *in situ* is higher than one of the collected samples. However, it is not expected a significant difference that could completely change the results presented herein as the CPTU results plot always in the dilatant side irrespectively of being sandy or clayey soils. A better sampling procedure [5] to collect undisturbed samples is therefore needed to confirm or not these results. It is though recognized that good quality undisturbed samples in sandy soils is not very frequent as it involves advanced and expensive technologies. So, in these cases the site investigation relies mostly on in situ tests.

Unfortunately, there was no availability to carry out (as it is a preliminary study) SCPTu or SDMT tests. Still, it is expected to be perform soon, in order to evaluate eventual cementation by means of the G_0/q_t parameter. In the case of marine sediments from a jetty/dyke in an area with reliable wave breakers such as these ones, cementation should not develop significantly, although an eventual specific fabric may exist.

The unit weight has a very important influence in field interpretation by means of CPTu or laboratory tests and therefore an accurate measure of this parameter is essential. However, this is often not possible in sandy deposits where sampling is very challenging [5].

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