The use of CPT to evaluate the properties of a compacted embankment

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ABSTRACT: SPT is the most usual type of soil investigation in Brazil for onshore foundation design. The piezocone test is routinely used for the evaluation of geotechnical parameters for embankment design on soft clays. However, this test is not yet a regular practice for onshore foundation design. The present paper shows CPT tests for the design of shallow foundations on a compacted granular embankment, built on top of a sandy soil. Soil compaction quality and assessment of the corresponding parameters for the foundation design were evaluated. Very high cone resistance and sleeve friction values were obtained, indicating a relative density of 100%, even at shallow depths. The very high values obtained were attributed to high horizontal stresses developed during compaction.

Keywords: relative density, sandy soil, CPT test

1. Introduction

Cone penetrometer test (or CPT) arrived in Brazil in the mid-1950s, brought by the Belgian Franki Piles company to be used in pile design [1], because there was no experience in Brazil on the design of piles based on SPT, the most common in situ test in Brazil. The first equipment is illustrated in Fig. 1.

After some years, and the establishment of a database correlating SPT with CPT, the CPT was progressively abandoned for pile design.

It was through the research work in the universities that the CPT returned to practice and a new market opened for private companies, but no longer as CPT, but now as piezocone (CPTU) tests, which included pore-pressure measurement, in addition to cone resistance and sleeve friction. However, this use was not intended for pile design, but primarily for the design of embankments on soft soils [2].

In the last five years, cone (and piezocone) testing have also been used for foundation projects and have been recommended by Brazilian designers and consultants who properly know their capabilities. In this research, CPT tests were associated with SPT tests for the design of shallow foundations in sand.

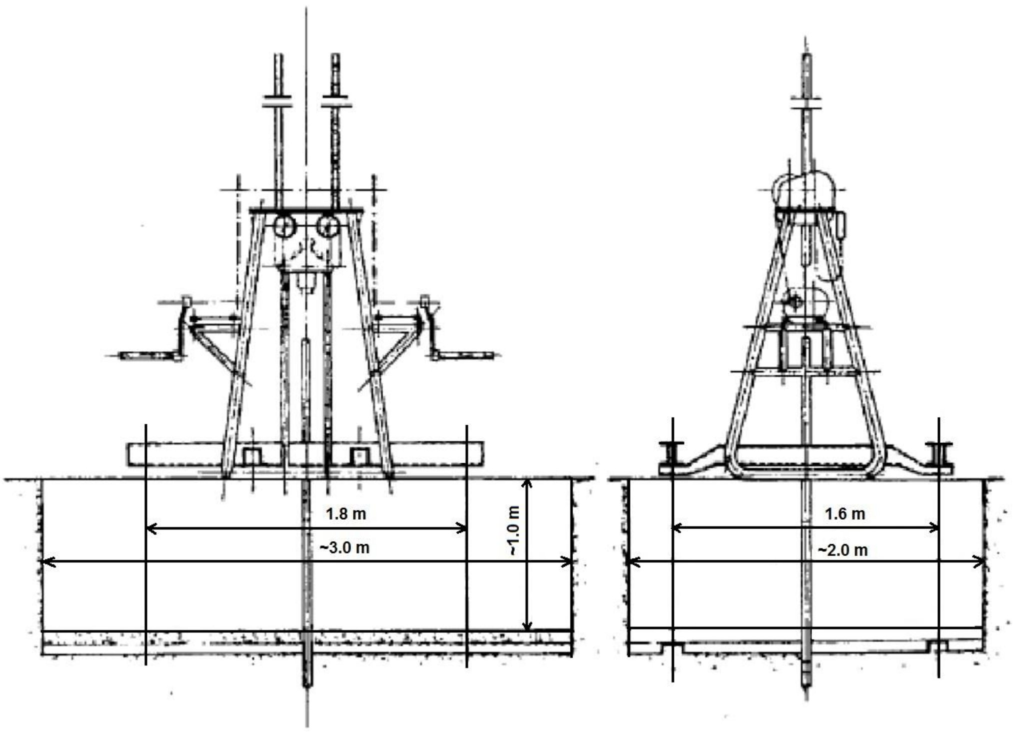

Figure 1. First CPT rig used in Brazil (Velloso, 1959).

2. The embankment and the tests performed

2.1. The embankment

The embankment was performed in a religious temple, part of Brazil's LDS Temple Complex, to be built on an area located in Barra da Tijuca, west part of the city of Rio de Janeiro. A first campaign consisting of 26 SPT tests was carried out by Soloteste Engenharia Ltd. in March 2013. The soil profile is composed only of sandy materials, relatively homogeneous in the horizontal direction. A typical test result is shown in Fig. 2.

Shallow foundations on a compacted embankment, about 3 m thick, directly over the natural material were designed. After the completion of the compacted embankment, CPT tests were specified by the designer and performed in 2017. Figure 3 illustrates the aspect of the embankment after completion.

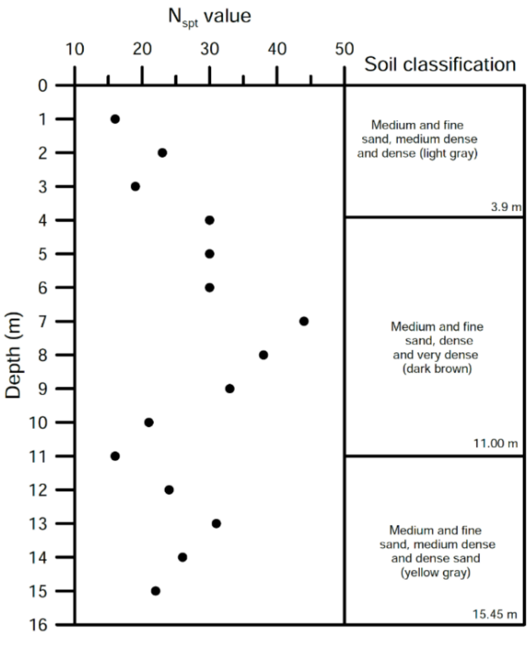

Figure 2. Typical SPT test before embankment construction.

Figure 3. Embankment after completion.

2.2. The tests performed

The following procedure was adopted for conducting the tests: i) for each test, the anchors were installed, which in all cases consisted of two sets of 3 m smooth rods and 3 m helical rods with 0.1 m in diameter. Thus, the anchor was always positioned on natural soil, since the embankment thickness was somewhat constant, around 3 m; ii) the penetration was performed at the standard rate of 20 mm/s, being interrupted every meter to add a new rod to the rod stem.

The maximum depth specified by the designer was 6 m. However, almost all the tests reached smaller depths, due to the capacity of the penetrometer rig (170 kN), rod buckling or anchor failure. The nine CPT tests are listed in table 1.

It is noteworthy to make some considerations regarding the operational issue of the CPT test in situations similar to the presented case. As mentioned, there were three limiting factors that should be analysed. The first is the capacity of the penetrometer rig, 170 kN (17 tf). The largest load component to achieve this value is the accumulated lateral friction along the rods, even in the present case of a relatively small rod length. In fact, when considering the measured values (presented below) of cone resistance q_c around 40 MPa, a tip load of 40 kN (4 tf) is obtained. Now assuming an average lateral frictional resistance of 200 kPa and 3 m rod length, a frictional load value of almost 190 kN (19 tf) is achieved, greater than the capacity of the rig. It should be noted at this point that even truck-mounted equipment has hydraulic systems limited to about 200 kN (20 tf), so it would also have difficulties in the present case. Regarding the reaction, a truck is more convenient than a rig, because the weight of a truck can be larger than the capacity of the rig. The failure of the anchor in the case of a small rig is relatively common in the authors´ experience, and it is difficult to predict the corresponding capacity. The last of the limiting element is the rod, which showed a buckling tendency even in a test without pre-hole (Fig. 4), which represents an important limitation of the test since the rod used is the same used in truck-mounted equipment. In other words, CPT tests to be performed in soils with characteristics similar to those of the material analysed here, require special operational procedures, such as the use of rods with devices that allow water jets to be used above the test region during penetration as well as removal, and the use of special high strength steel.

Figure 4. Beginning of the CPT-06 rod assembly buckling, even without pre-hole.

Table 1. Tests performed

Designation	Date	Maximum depth (m)	Reason to stop the test
CPT-01	22/08/2017	3.15	Capacity of the penetrometer rig
CPT-02	22/08/2017	1.82	Anchors failure
CPT-03	23/08/2017	2.91	Capacity of the penetrometer rig
CPT-04	22/08/2017	2.11	Capacity of the penetrometer rig
CPT-05	22/08/2017	1.75	Anchors failure
CPT-06	23/08/2017	2.07	Rods buckling
CPT-07	23/08/2017	5.80	Maximum lenght specified in contract
CPT-08	23/08/2017	3.38	Capacity of the penetrometer rig
CPT 4-	23/08/2017	3.27	Rods buckling

2.3. CPT results and discussion

Figure 5 shows the values of cone resistance (q_c) and sleeve friction (f_s) as well as the inclination versus depth of all tests performed. It may be noted that the inclination of the cone was not significant, and in fact, the interruption of the test cannot be related to a high inclination of the rods.

Good repeatability of the tests was observed, indicating that the embankment was properly executed. Only one test was not limited to the compacted embankment, and from this test a clear transition to the natural material can be observed. The measured values show that the embankment material is denser than the original soil, as it has a higher cone resistance value at a lower depth (or stress state).

Analysing the tests results, the profile shows three trends, which were considered as layers: i) a trend of linearly increasing values with depth (both q_c and f_s), up to approximately 1.75m; ii) from 1.75 m to 3.25 m, a trend of constant values, still in the embankment; c) from 3.25 m to 5.80 m, a decreasing resistance trend, already in the natural soil. The average values of the measured quantities applied to these layers were considered in the following analysis (table 2). The normalized friction ratio F_r and normalized cone resistance Q_{tn} were also included in the table.

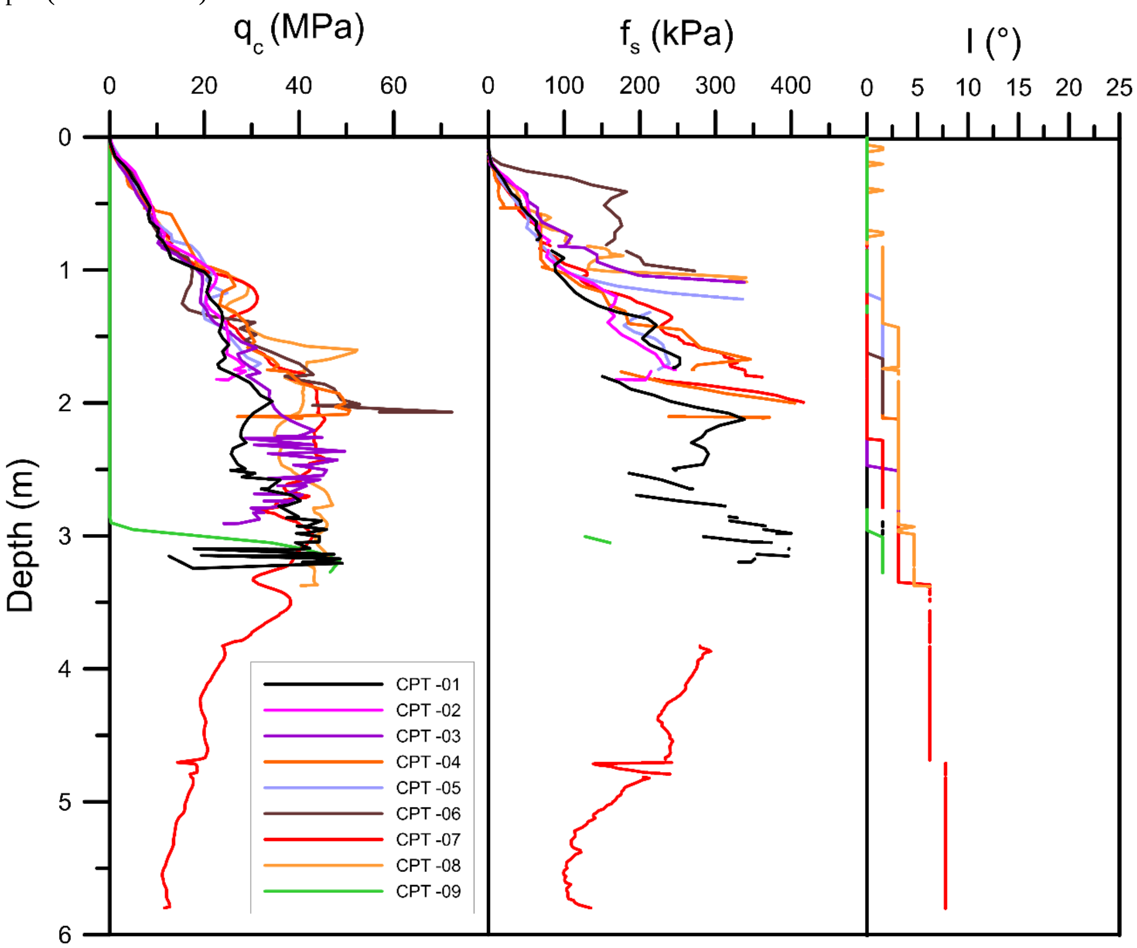

Figure 5. Results of CPT tests performed.

It is interesting at this point to verify the soil behavior type, using Robertson's proposal [3] to classify soil behavior as drained or undrained, compressive or dilating, as shown in Fig. 6.

The obtained values provide an indication that Robertson's proposal [3] was able to properly predict the behavior of the compacted soil. As for the drained behavior, there is no doubt, as the soil is unsaturated, and the dilative behaviour is explained by compaction, generating high horizontal stresses.

Table 2. Approximated average values in each layer

Layer	Average depth (m)	q_c (kPa)	f_s (kPa)	F_r (%)	Q_{tn}
1	0.88	16125	150	0.93	237
2	2.50	32250	300	0.93	393
3	4.52	21125	225	1.07	226

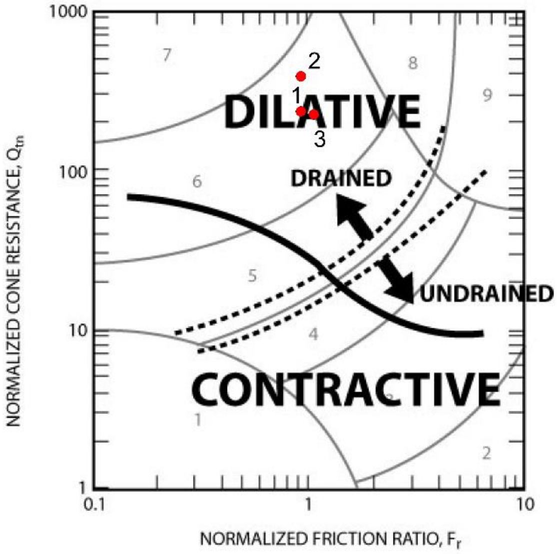

Figure 6. Approximate limits of dilated-compressive and drained-undrained behavior (modified from Robertson 2012).

Regarding geotechnical parameters, for cohesionless soils, the cone resistance can be used to predict relative density, D_r, and friction angle, ϕ'. To estimate D_r Jamiolkowsky et al. [4] suggested the equation (1) below.

$$D_r = -98 + 66(\,log_{10}\frac{q_c}{\sigma'_v{}^{0,5}}) \quad (1)$$

where q_c and σ'_{vo} are expressed in tf/m².

According to those authors, equation (1) provides an estimation of relative density with an error of +/- 20% for normally consolidated deposits (intrinsic uncertainty range of the method). For pre-consolidated deposits, the value of σ'_{vo} should be replaced by the value of σ'_{ho}, which requires some level of geotechnical judgment. In the research herein the value of σ'_{vo} was used, due to the difficulty in estimating a proper value of σ'_{ho}. Thus, the estimation of D_r is expected to be conservative.

The evaluation of the friction angle from D_r can be obtained by different approaches, for example, De Mello [5], equation (2),

$$(1{,}49 - D_r)\tan\phi' = 0{,}712 \qquad (2)$$

and Bolton [6], equation (3)

$$\phi' = 33 + 3\,(D_r(10 - ln\,p') - 1) \qquad (3)$$

where p' is the average stress in kN/m² and D_r in decimals. Similarly to the previous case the average stress was adopted as being the value of σ'_{vo}. The obtained values are presented in table 3.

Table 3. Geotechnical average parameters

Layer	D_r (%) Jamiolkowsky et al. (1985)	ϕ' (°) De Mello (1971)	ϕ' (°) Bolton (1986)
1	112	63	56
2	114	64	52
3	92	51	45

As expected, extremely high values for D_r and ϕ' were obtained for the methods employed in the case of the embankment, which were attributed to the high horizontal stresses developed during compaction.

It is also interesting to compare the measured values with cone test results from calibration chamber tests, and Ticino sand [7,8], normally consolidated, was employed for this comparison (Fig. 7). The values obtained in the embankment, in the tests reported in this research, are much above all values obtained in Ticino sand, which is explained for the high horizontal stresses from the compaction. The natural soil is at the upper limit, but still within the range measured in Ticino sand tests.

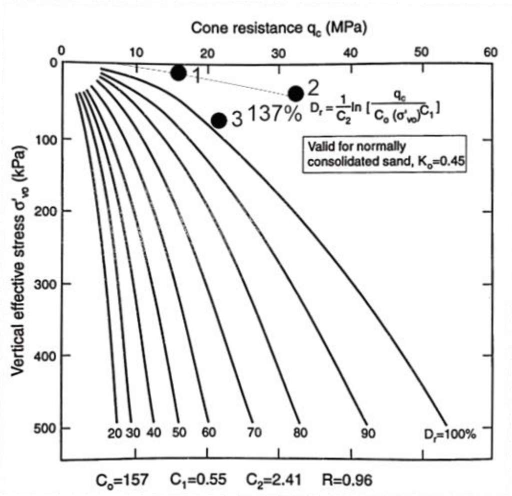

Figure 7. Relationship between q_c, σ'_{vo} and D_r for normally consolidated Ticino sand (modified from Baldi et al. 1986 by Lunne et al. 1997), with data obtained in the present tests campaign.

3. Conclusions

The results of cone penetration tests performed in a compacted sandy embankment overlying an also sandy natural soil were presented and analysed. The performed tests were not able to reach a significant penetration depth, i.e., were limited to the compacted embankment, except in only one case where the cone reached the natural soil. The difficulties to perform CPT tests in similar conditions to the present case were discussed. Extremely high values of penetration resistance (q_c, f_s) were obtained, associated with the relative density of at least 100%, even in short depths (maximum of 3 m), in other words, low vertical stress. The values obtained were attributed to high horizontal stresses developed during the compaction, which is also reflected in soil behavior type [3], drained-dilated.

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