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An integrated interpretation of SCPTu for liquefaction assessment in intermediate alluvial deposits in Portugal

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ABSTRACT: The state parameter approach applies the concepts of critical state soil mechanics to describe the behaviour of cohesionless soils combining density and stress level. Such parameter can also be applied to estimate liquefaction susceptibility and can be obtained from in situ and laboratory tests. This paper aims to interpret SCPTu data using integrated classification charts for assessing liquefaction susceptibility. An extensive database was obtained from site characterisation of liquefiable soils in the greater Lisbon area. The state parameter was estimated using the relationship between stiffness and strength in the soil layers with soil behaviour type index below 2.60. In order to validate the liquefaction susceptibility assessment using the state parameter, the results were categorised in terms of contractive/dilative behaviour and drainage conditions of the soils by combining stiffness and strength ratio with state parameter. Validation was based on the comparison of these results with the factor of safety against liquefaction from SCPTu data.

Keywords: state parameter, liquefaction, in situ testing, soil stiffness.

1. Introduction

The Cone Penetration Test (CPT) is widely used for in situ soil characterisation because of its continuous data measurement and repeatability. Besides, the CPT apparatus can be improved by incorporating a pressure transducer to measure pore-water pressure during the cone insertion into the ground and accelerometers or geophones to measure seismic wave velocities. This device configuration is known as the seismic piezocone (SCPTu), which has the advantage of combining independent measurements of shear strength and stiffness in soils, among others.

The interpretation of this type of tests has a strong theoretical background, which allows identifying the soil profile and mechanical properties of these materials. From such theoretical background, Robertson [1] proposed an update to the soil classification, introduced initially by Robertson [2,3], to incorporate the dilative/contractive behaviour of soils. Such approach is very attractive when assessing liquefaction. In soils susceptible to trigger such phenomena (e.g. clean sands and non-plastic silts), the dilative/contractive behaviour is controlled by the stress and density states.

The evaluation of dilative/contractive behaviour of soils uses the state parameter (ψ), which was proposed by Been and Jefferies [4]. This parameter combines both the effect of void ratio (e) with the effect of effective mean stress (p') to establish the dilative/contractive behaviour of granular soils. The state parameter is defined as the void ratio difference between the current state (e_0) of the soil and the critical state (e_{cs}) at the p' [5]. Therefore, soils with a state denser than the critical state, ψ<0, will be dilative; in turn, soils with a state looser than the critical state, ψ>0, will be contractive. Soils with contractive behaviour are more susceptible to liquefaction than soils with dilative behaviour.

This paper explores the combination of shear strength and stiffness of soils with the state parameter criteria for assessing liquefaction susceptibility in soil deposits. For this purpose, soil layers with soil behaviour type index (Ic) below 2.60 were selected. The data were obtained through SCPTu conducted in a pilot site in the greater Lisbon area for liquefaction assessment. The results address the classification of liquefiable soils in terms of contractive/dilative behaviour and drainage conditions. Furthermore, a discussion based on the comparison of the classification findings versus the factor of safety against liquefaction from SCPTu data is presented.

2. Experimental site

Within the scope of two research projects on soil liquefaction developed in the CONSTRUCT-GEO research centre of FEUP, a vast site characterisation campaign was conducted in a pilot site in the greater Lisbon area (south of Portugal). Such characterisation aimed at the definition of microzonation maps of liquefaction risk for an area of about 14683 ha, in the municipalities of Vila Franca de Xira and Benavente.

The south of Portugal is probably the zone, in this country, with greater seismic risk because of its proximity to the boundary between the African and Eurasian plates [6]. Hence, this region is affected by the occurrence of large magnitude (>8) distant earthquakes and medium magnitude (>6) near earthquakes [7]. There is historical evidence of soil liquefaction phenomena, fairly well documented after the last large earthquake that occurred in the 23rd April 1909 (Mw = 6.0 with epicentre near Benavente) in the greater Lisbon area [8]. Table 1 summarises the seismic action, recommended in the National Annex of Eurocode 8 [9], for the two municipalities.

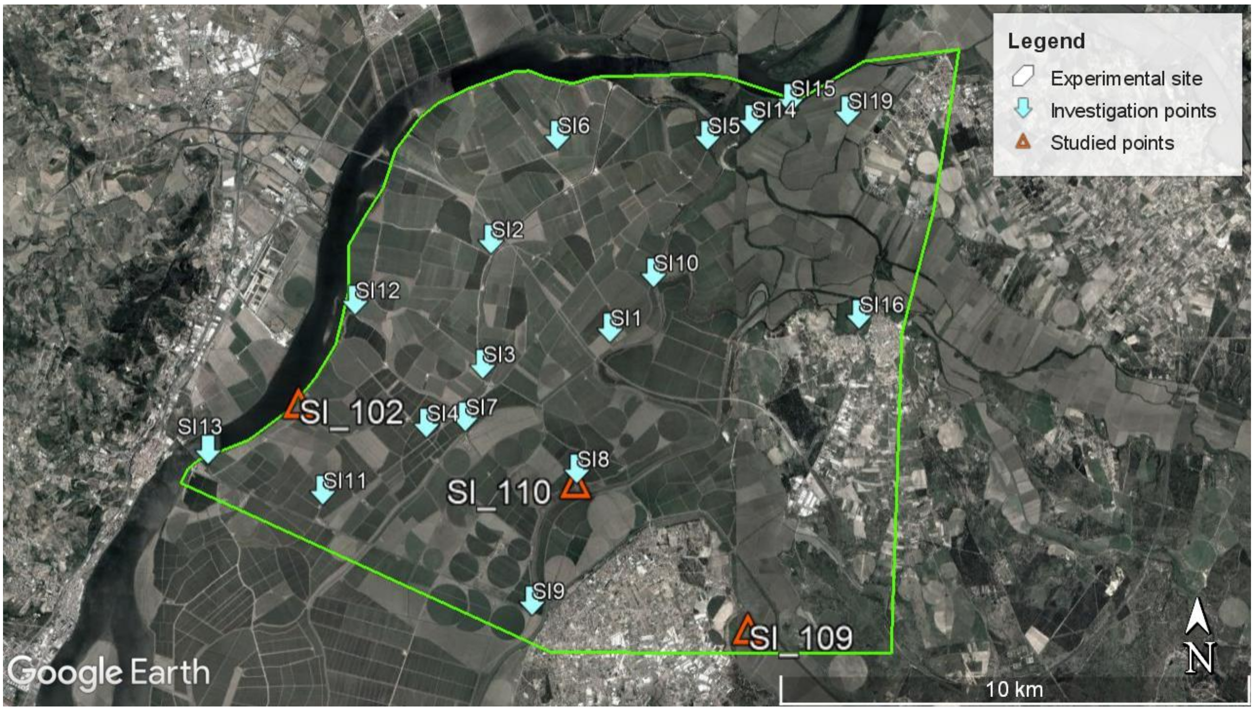

Figure 1. Map of the experimental site and location of the investigation points (after [10]).

Table 1. Seismic action for a return period of 475 years for Benavente and Vila Franca de Xira (after [9]).

Seismic action	Type 1	Type 2
Seismic zone	1.4	2.3
M_w	7.5	5.2
a_gR (m/s^2)	1	1.7
γI	1	1
ag	1.0	1.7
Ground type	D	D
S_{max}	2	2
S	2.00	1.77
a_{max} (m/s^2)	2.00	3.00
MSF	1.00	1.80

Viana da Fonseca *et al.* [10] presented the results of the geotechnical site characterisation of an experimental site in Lezíria Grande. Ferreira *et al.* [11] performed a comparative analysis of liquefaction susceptibility assessment in the greater Lisbon area. In situ tests for geotechnical and liquefaction characterisation were performed at specific locations, referenced as site investigation points (SI). Fig. 1 shows the map and the SI location within the experimental site.

For this study, SI102, SI109 and SI110 were selected. In these points, piezocone penetration tests with measurement of seismic wave velocities (SCPTu) were carried out. Shear wave velocities (Vs) were measured at each 0.50 m depth. The groundwater level was identified about 1.0 m depth in the three SI. Fig. 2 presents the SCPTu test results in terms of cone resistance (q_c), sleeve friction (f_s), pore-water pressure generated during cone penetration (u_2) and shear wave velocity (Vs).

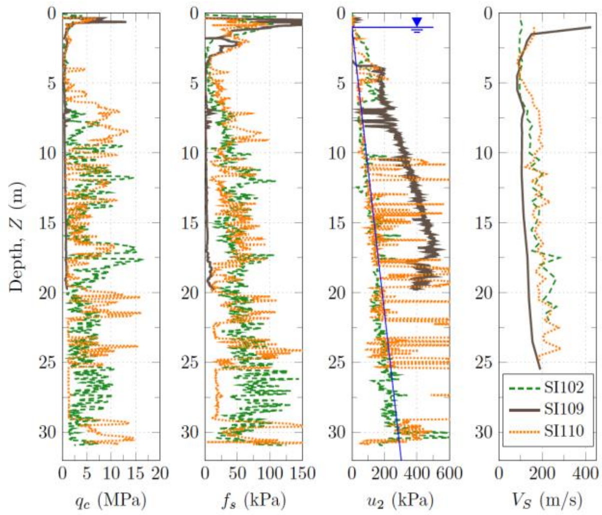

Figure 2. SCPTu results.

3. Interpretation and analysis of test results

The SCPTu data were analysed using the software Cliq® v.3.0. [12]. The analyses were carried out based on the unified approach proposed by Robertson [3]. Such approach includes the classification of soil behaviour type (SBT) zones according to the soil behaviour type index (Ic). The CPT-based SBT method suggested by Robertson [2] is based on the following normalized parameters:

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \quad (1)$$

$$Fr = \left(\frac{f_s}{q_t - \sigma'_{vo}}\right) 100\% \quad (2)$$

$$Ic = [(3.47 - \log Q_t)^2 + (\log Fr + 1.22)^2]^{0.5} \quad (3)$$

where q_t is the cone resistance corrected for pore water effects, $q_t = q_c + u_2(1 - a)$; a is the cone area ratio; σ_{vo} is the current in situ total vertical stress; σ'_{vo} is the current in situ effective vertical stress. Fig. 3 presents the interpretation of the soil profiles in terms of soil type, based on SBT.

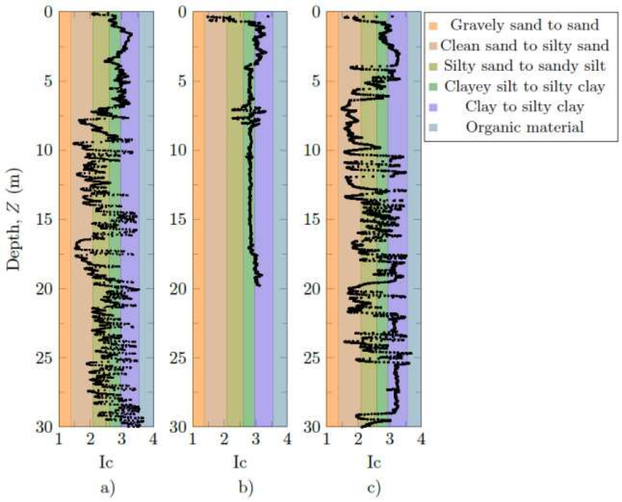

Figure 3. Soil profiles based on SBT: a) SI102; b) SI109; c) SI110.

By comparing the SCPTu results, similarities were observed between SI102 and SI110 profiles, since both profiles exhibit layers of clean sands to silty sands (Ic<2.6). On the other hand, the SI109 profile is composed by clayey silt to silty clay soils. The three profiles are clearly representative of the soils in the greater Lisbon area, that are a product of alluvial deposition from the Tagus River after a long period of successive marine transgressions from the Atlantic Ocean [13]. Due to the geological origin of the soils in this region, these soil profiles evidence interstratified layers composed of sand and clays.

The combined information from the SCPT has the potential to aid in the identification of possible microstructure and fabric in soils. The maximum shear modulus (G_0) describes small-strain stiffness of soils, which are controlled by microstructure and fabric. G_0 is related to the mass density of the soil (ρ) and shear wave velocity (Vs). Eq. 4 shows the most common procedure to analytically compute G_0 from Vs.

$$G_0 = \rho V s^2 \tag{4}$$

Robertson and Wride [14] suggested a normalised cone parameter (Q_{tn}) in terms of the atmospheric pressure (Pa) to evaluate soil liquefaction using a stress exponent variable (n) dependent of the nature of the soil, defined as follows:

$$Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{Pa}\right)\left(\frac{Pa}{\sigma'_{vo}}\right)^n \tag{5}$$

In order to assess the liquefaction phenomena in the layers of cohesionless soils, SCPTu data were filtered for the layers with Ic<2.60. At such depths, the state parameter, ψ, was estimated using Eq. (6) [15].

$$\psi = 0.56 - 0.33 \log Q_{tn} \tag{6}$$

Fig. 4 shows the ψ profiles for the depths composed by cohesionless soils with Ic<2.60. These profiles indicate that the granular soils in SI102 and SI110 have a predominantly dilative behaviour (ψ<0), while the soils in SI109 present a contractive behaviour (ψ>0) at 5.5 to 6.5 m depth.

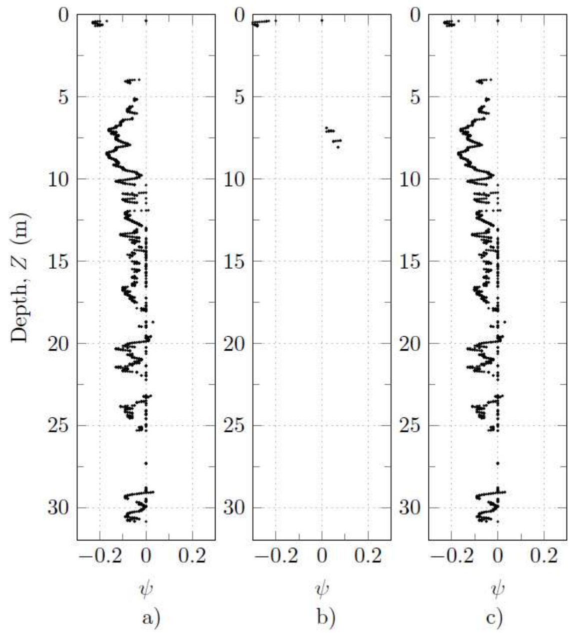

Figure 4. Soil state parameter profiles for the layers with Ic<2.60: a) SI102; b) SI109; c) SI110.

On the other hand, the values of q_t, Q_{tn}, Fr and ψ were averaged each ± 0.25 m in depth (ranges of 0.5 m), taking into account the depth of the Vs measurements for each profile. Such procedure was implemented to obtain data at the same depths for classifying the soil, in terms of contractive/dilative behaviour and drainage conditions.

Shuttle and Cunning [16] and Jefferies and Been [5] suggested that when a soil has a state parameter ψ>-0.05, strain softening and strength loss in undrained shear conditions can be expected. Therefore, Robertson [17] defined a region on the SBT chart that represents a state parameter of about ψ>-0.05 to identify the liquefaction susceptibility based on historical cases. Subsequently, Robertson [1] stated that such region is conservative and presented a chart to divide the predominantly dilative from the contractive soil behaviour.

Fig. 5 presents the classification in terms of the dilative/contractive tendency of the soils. From such Figure, it can be observed that, in general, soils with Ic<2.60 tend to plot in the dilative region of the chart. In addition, transitional soils were identified in the three profiles. Therefore, the classification results in terms of dilative/contractive behaviour (Fig. 5) indicate that the soil layers of this experimental site are composed mainly of soils with dilative behaviour, which are not susceptible to trigger soil liquefaction.

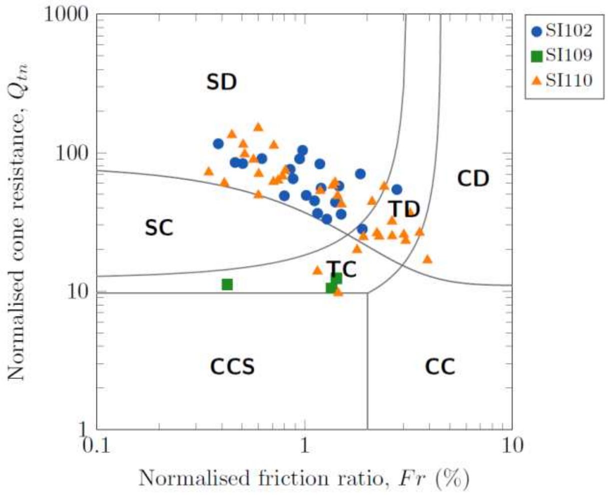

Figure 5. Soil behaviour type classification of the soils with Ic<2.60 in the greater Lisbon area.

The classification taking into account the drainage conditions used in this study was proposed by Nierwinski [18]. Such classification uses a dimensionless log-log space Q_{tn}:G_0/q_t, which divides the soil into different classes. Such classes range from 1 to 7 for soils with low plasticity (sands and sandy mixtures) and I to V for soils with high plasticity (clays and clayey mixtures). All classes provide a good signal of the strength loss index or brittleness. Table 2 presents the classes derived from the stiffness index, G_0/q_t, based on the drainage conditions.

Table 2. System for soil classification system based on drainage conditions using SCPTu test results (after [18]).

Non-plastic soils	
1	Sensitive non-plastic soils
2	Sandy silts
3	Sandy silt to silts with gravels or cemented silts
4	Sands with gravels or cemented sands
5	Silts
6	Sands to silty sands
7	Clean sands
Plastic soils	
I	Clays
II	Organic clays
III	Silty clays
IV	Clayey silts
V	Clayey sands

The chart classification is based on the boundaries defined by Schnaid *et al* [19], who compiled results of q_t and G_0 in natural sands. Such boundaries are parallel and indicate the limits for cemented and uncemented materials. Eq. 7 defines the shape for those boundaries lines.

$$\frac{G_0}{q_t} = \theta \sqrt[3]{Q_{tn}} \tag{7}$$

where θ is a parameter that ranges from 20 (line D-D), 110 (line A-A), 280 (line B-B) and 800 (line C-C). From this classification, Nierwinski [18] added vertical boundaries to define the drainage conditions of the soil, using a line series with Q_{tn}=50 and Q_{tn}=10 for drained and undrained conditions, respectively. In addition, the author recommended an intermediate empirical line, Q_{tn}=20 to separate possible mixtures with higher proportion of sand (right) from higher proportion of silty fines (left). Fig. 6 presents the classification chart in terms of drainage conditions with the results for the SCPTu tests.

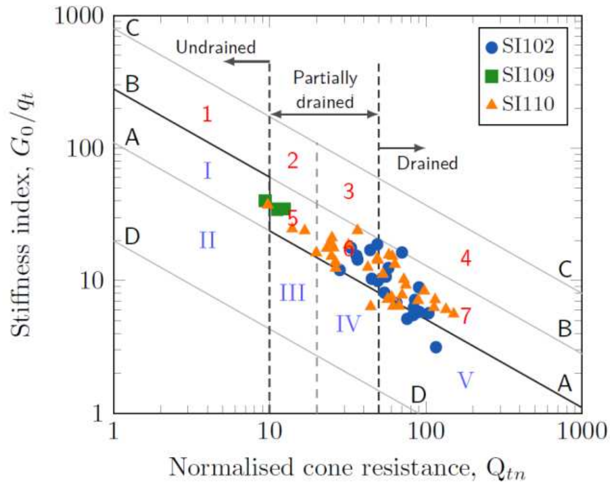

Figure 6. Soil classification in terms of drainage conditions of the soils with Ic<2.60 in the greater Lisbon area.

The studied soils were classified mainly as sands to silty sands and clean sands (zones 6 and 7), except for most of SI109 profile. Such classification indicated that the soils of SI102 and SI110 are expected to respond in drained to partially drained conditions, which are typical for soils susceptible to trigger liquefaction. The combination of stiffness and strength (G_0/q_t) allows obtaining an alternative classification of soil behaviour, because it takes advantage of two intrinsic properties of the soils. Besides, this ratio is not sensitive to changes in stress-state or relative density of granular materials [20]. Robertson [1] stated that the classification must be applied in uncemented and young soils, because the G_0/q_t ratio is sensitive to cementation and ageing. This means that it is applicable to the soils in this study, originated from the Holocene [6, 11].

To describe the soil state and its susceptibility to trigger liquefaction, Schnaid and Yu [20] proposed an empirical relationship between G_0/q_t and ψ from a database of six different sands (mostly clean) based on physical models in a large calibration chamber. Eq. 8 presents such empirical relationship.

$$\psi = \alpha \left(\frac{p'}{Pa}\right)^{\beta} + \chi \ln \frac{G_0}{q_t} \tag{8}$$

where α=0.52, β=0.07 and χ=0.18 are average coefficients obtained from calibration chamber data; p' is the mean effective stress of the soil and Pa is the atmospheric pressure. The data reported by Schnaid and Yu [20] ranged for p' stresses between 50 and 500 kPa.

The analysis of SCPTu in the studied soils presented partially drained conditions and generated pore-water pressure during cone penetration. In the light of the findings of DeJong and Randolph [21], a correction of the cone resistance in terms of drainage conditions must be applied, since the Schnaid and Yu's proposal was developed for clean sands under drained conditions. The partial drainage condition of soils with medium permeability coefficient (10^{-5} to 10^{-8} m/s) is controlled by the cone penetration rate during testing [22].

Nierwinski [18], based on the theoretical evidence of Senneset et al [23], suggested a correction for obtaining a normalisation in terms of the standard penetration rate (2 cm/s), using the ratio between the drained cone resistance (q_{tD}) and the undrained cone resistance (q_{tND}), and the pore pressure ratio ($Bq=(u_2 - u_0)/(q_t - \sigma_{vo})$). Eq. 9 describes the drainage correction for SCPTu results.

$$\frac{q_{tD}}{q_{t(20)}} = 1 + \left(\frac{q_{tD}}{q_{tND}} + 1\right) Bq^{0.5} \tag{9}$$

where q_{tD}/q_{tND} has two boundary conditions, according to the Bq value (see Eq. 10), and depends on the effective friction angle of the soil, ϕ', (see Eq. 11) [23]. For the soils of the greater Lisbon Area, the ϕ' values reported by Ramos et al. [24] were used. Fig. 7 presents the drained correction chart for different q_{tD}/q_{tD} values.

$$\frac{q_{tD}}{q_{tND}} = \begin{cases} 1 \rightarrow Bq = 0 \\ \frac{q_{tD}}{q_{tND}} \rightarrow Bq = 1 \end{cases} \tag{10}$$

$$\frac{q_{tD}}{q_{tND}} = 1.6921 exp^{0.0458\phi'} \tag{11}$$

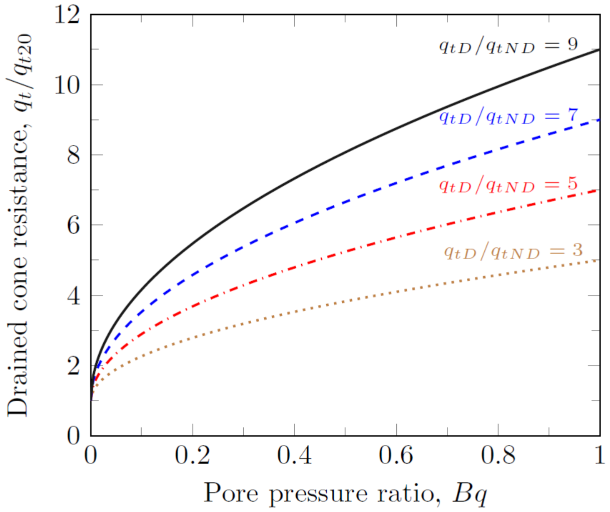

Figure 7. Drained cone resistance correction chart.

Fig. 8 presents the classification of the soils after the computation of the cone resistance corrected to full drainage conditions. The new classification shows that the soils with Ic<2.60 in the greater Lisbon Area were classified as sandy silts to silts with gravels and sands with gravels. This classification was validated by comparing the results of the sampling campaign presented by Molina-Gómez *et al* [25] in the greater Lisbon Area. In addition, results showed that such soils have a fragile failure behaviour because of the high values of G_0/q_{tD}. Such behaviour is typical of soils with high strength loss index (brittleness) and is associated with low liquefaction resistance.

Using the G_0/q_{tD} and ψ values, the Schnaid and Yu's proposal was estimated for the soils with Ic<2.60 in the greater Lisbon area. Fig. 9 shows the plot of the theoretical correlation between G_0/q_{tD} and ψ for the soils with Ic<2.60 from these three profiles.

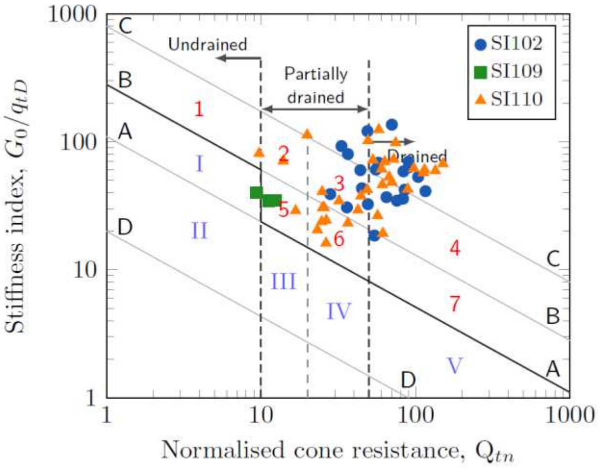

Figure 8. Soil classification in terms of drainage conditions of the soils with Ic<2.60 after drained cone resistance correction.

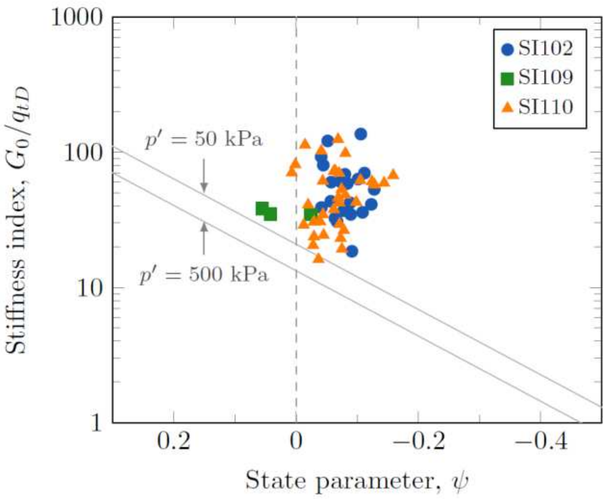

Figure 9. Results of the theoretical correlation between G_0/q_{td} and ψ.

The results confirm the tendency to dilate of the analysed soil profiles, presented in Fig. 4 and Fig. 5. However, the chart displays some dispersion of the data with respect to the narrow band of p' bounds (50 kPa to 500 kPa) obtained by Schnaid and Yu [20]. Such dispersion was observed even after applying the drained cone resistance correction. In addition, the variability of results can be attributed to the interlayering of the alluvial deposits, which is more pronounced in SI102 and SI110 profiles.

On the other hand, the classification results in terms of contractive/dilative behaviour and drainage conditions were contrasted and validated through liquefaction susceptibility profiles. This analysis was conducted for the Type 1 seismic action (see Table 1) using the simplified procedure, originally introduced by Seed & Idriss [26] and recommended in Eurocode 8 [9].

This method states that the factor of safety against liquefaction (FS_{liq}) can be computed from the ratio between the cyclic stress ratio (CSR) and the cyclic resistance ratio (CSR). Eq. 12 describes the procedure to compute CSR [26] and Eq. 13 defines the deterministic correlation to estimate CRR from CPTu data [27].

$$CSR = \frac{\tau_{av}}{\sigma'_v} = 0.65 \left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_v}{\sigma'_v}\right) r_d \tag{12}$$

$$\text{CRR}_{M=7.5,\sigma'v=1atm} = \exp\left[\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.80\right] \quad (13)$$

where τ_{av} is the average equivalent uniform cyclic shear stress caused by the earthquake and is assumed to be 0.65 of the maximum induced stress, a_{max}/g is the maximum horizontal acceleration (as a fraction of gravity) at the ground surface, σ'_v is the initial effective vertical (overburden) stress at the depth evaluated, σ_v is the total overburden stress at the same depth, and r_d is the shear stress reduction factor that accounts for the dynamic response of the soil profile. q_{c1Ncs} corresponds to the normalised equivalent clean sand cone resistance. Fig. 9 presents the results of liquefaction susceptibility profiles based on FS_{liq}.

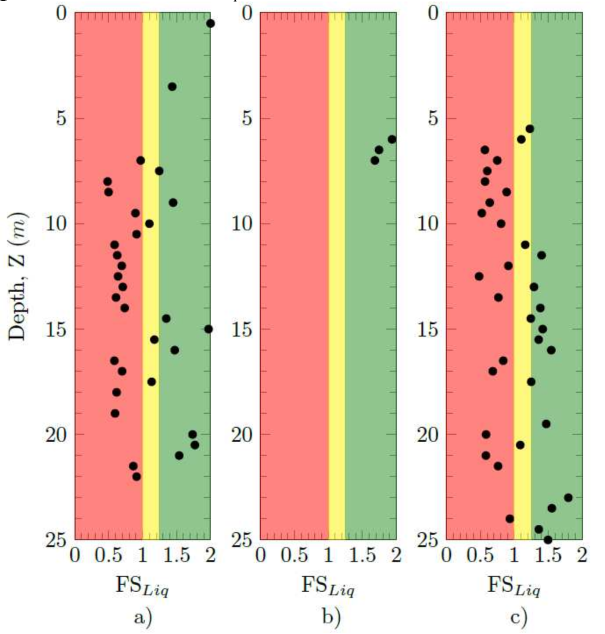

Figure 10. Liquefaction susceptibility profiles based on FS_{liq}: a) SI102; b) SI109; c) SI110.

By comparing results of soil classification in terms of drainage conditions (Fig. 8) against liquefaction susceptibility profiles (Fig. 10), it can be observed that soils in SI102 and SI110 have a drained condition and are susceptible to soil liquefaction because of the brittleness of this type of materials.

Likewise, the soils in SI109, which present an undrained behaviour, do not have susceptibility to trigger liquefaction. From this comparison, it may be concluded that both interpretations converge for the purpose of liquefaction susceptibility assessment.

4. Summary and concluding remarks

Different methodologies for the interpretation of seismic piezocone tests have been applied in this study. These methodologies provided an additional level to classify the liquefaction susceptibility of the alluvial soil deposits in an experimental site in the Lower Tagus Valley (LTV) in the greater Lisbon Area. In fact, the combination of stiffness and strength (G_0/q_t) together with the normalised cone resistance (Q_{tn}) allowed obtaining an alternative characterisation of such soils in terms of contractive/dilative behaviour and drainage conditions. For the case study of this paper, which only involved soils with Ic<2.6, the following conclusions can be drawn:

- The granular soils in the LVT, close to Lisbon, have a predominant dilative behaviour.
- The interpretations based on G_0/q_t ratio could be applied for the granular soils in this area because of its low cementation degree and due to its Holocene origin.
- The G_0/q_{td} ratio is a good complement to Q_{tn} for representing the brittleness of recent alluvial soil deposits, such as the sandy soils of this valley.
- Based on critical state soil mechanics, the chart G_0/q_t and ψ only explains the contractive/dilative behaviour of the studied soils because the results are disperse in terms of G_0/q_t in comparison to the stress range (50kPa to 500 kPa) of the original proposal.
- The soil classification in terms of drainage conditions describes well the liquefaction susceptibility of the studied soils and this statement was validated by comparing such results against the FS_{liq} profiles.

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