Comparative analysis of liquefaction susceptibility assessment by CPTu and SPT tests

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ABSTRACT: The assessment of liquefaction susceptibility from field tests is conventionally based on the factor of safety (FS_{liq}) against liquefaction, relating the cyclic resistance ratio (CRR) with the cyclic stress ratio (CSR). The calculation of CSR is relatively straightforward, whereas CRR strongly depends on the in situ technique from which it is derived. Distinct approaches have been proposed based on quantitative liquefaction risk indexes, namely the Liquefaction Potential Index (LPI) and the Liquefaction Severity Number (LSN). In Portugal, a pilot site for liquefaction assessment has been set up in the Lower Tagus Valley, near Lisbon, within the European H2020 LIQUEFACT project. In this paper, the geotechnical field data from SPT and CPTu is integrated in the three approaches to liquefaction assessment. A comparative analysis of the results is presented and discussed, highlighting the differences and limitations of these in situ tests in the assessment of liquefaction susceptibility in loose granular soils.

1 BACKGROUND

Portugal’s mainland and its Atlantic coast are located on the western and southern margins of the Iberian Peninsula. The seismicity of the Portuguese territory is heterogeneous and is classified according to regions with distinct seismic behavior, as in the Portuguese National Annex of Eurocode 8 (EC8-NA) (CEN, 2010). Seismicity increases in intensity from North to South and is concentrated in the South and the Atlantic margins. According to existing records, earthquake epicentres are mostly located in the Lisbon region, in the Lower Tagus River Valley (LTV) region, and along the Algarve coast (Ferrão et al. 2016). The greater Lisbon area is probably the zone with greater seismic risk, and it is affected by the occurrence of large magnitude (>8) distant earthquakes and of medium magnitude (>6) near earthquakes (Azevedo et al. 2010). An example of a distant event is the 1755 Lisbon earthquake (M>8.5) generated in the Eurasian-Nubia plate boundary zone, and local intraplate (M≈6-7) earthquakes occurred more frequently, namely in 1344, 1531 and 1909.

Earthquake-Induced Liquefaction Disasters (EILDs) are responsible for significant additional structural damage and casualties, particularly in zones where specific geologic, geomorphological, hydrological and geotechnical characteristics indicate liquefaction potential of soils. The presence of thick profiles of recent alluvial sand deposits in a high seismicity area is a good example of the combination of the necessary liquefaction triggering conditions (LIQUEFACT, 2017). Information regarding liquefaction in Portugal has been collected and analyzed by Jorge (1993). Subsequently, Jorge & Vieira (1997) identified the locations of historical liquefaction events coupled with a reliability classification. A liquefaction potential zonation map of Continental Portugal was developed by Jorge (1993) and further discussed by Jorge & Vieira (1997). This zonation map was derived from the superposition and generalization of two basic maps: the liquefaction opportunity map and the liquefaction susceptibility map. For the greater Lisbon area, the authors produced a detailed representation, which identified high to very-high liquefaction susceptibility areas, mostly along the NE region of Lisbon, in the Lower Tagus Valley (LTV).

Based on this map, the region for the pilot site was selected and later refined from the analysis of existing geotechnical data, mainly covering the municipalities of Vila Franca de Xira, Benavente, Montijo and Barreiro.

2 APPROACHES TO THE ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

2.1 *Factor of safety*

The most common approach to the assessment of the liquefaction suscetibility is the "Simplified Procedure", originally introduced by Seed & Idriss (1971), also recommended by Eurocode 8. This procedure consists on the computation of the factor of safety to liquefaction, as the ratio between the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR), expressed in Equation 1. The CRR is a measure of the capacity of the soil at a given point in depth to resist to liquefaction and the CSR refers to the expected seismic action on the soil at a specific location.

$$FS_{liq} = \frac{CRR}{CSR} \tag{1}$$

The expression for calculating CSR, according to the proposal of Seed and Idriss (1967), is as follows:

$$CSR = \frac{\tau_{cyc}}{\sigma'_{v0}} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot r_d \tag{2}$$

where a_{max} is the peak ground acceleration at the site, g is the acceleration of gravity, σ_{v0} and σ'_{v0} are the total and effective vertical stresses at the specific depth and r_d is a shear stress reduction coefficient.

The local peak ground acceleration (PGA or a_{max}) at the site was defined based on the National Annex of Eurocode 8 (EC8-NA) (CEN, 2010), as summarised in Table 1. For a return period of 475 years and a corresponding building importance class of II, the importance factor, γ_I, for the seismic zone of Vila Franca de Xira/Benavente is equal to 1.0. The corresponding magnitudes of Seismic Type 1 and Type 2 are 7.5 and 5.2, respectively, and a_{max} are equal to 1.0 and 1.7, respectively. These reference a_{max} must then be corrected to the local ground type, typically type D, corresponding to 'deposits of loose-to-medium cohesionless soil, or of predominantly soft-to-firm cohesive soil' (CEN, 2010) using the parameter *S*.

Table 1: Calculation of a_{max} for Vila Franca de Xira and Benavente, according to EC8-NA (CEN, 2010)

Parameter	Seismic action Type 1	Seismic action Type 2
Seismic zone	'1.4'	'2.3'
M_w	7.5	5.2
a_{gR} (m/s^2)	1.0	1.7
γ_I	1	1
Soil type	D	D
S	2.00	1.77
$a_{máx}$ (m/s^2)	2.00	3.00

The parameter r_d is a stress reduction coefficient, can be computed as proposed by Liao & Whitman (1986), also recommended by Youd et al. (2001), as a function of depth. However, other authors included the earthquake magnitude in its computation, namely Idriss (1999), also suggested by Boulanger & Idriss (2014) and adopted in this work, as follows:

$$r_d = e^{[\alpha(z)+\beta(z)\cdot M]} \tag{3}$$

$$\text{with } \alpha(z) = -1.012 - 1.126\sin\left(\frac{z}{11.73} + 5.133\right) \tag{4}$$

$$\text{and } \beta(z) = 0.106 + 0.118\sin\left(\frac{z}{11.28} + 5.142\right) \tag{5}$$

where z = depth (m) and M = earthquake magnitude.

The capacity of the soil to resist liquefaction is provided by the CRR, which can be evaluated from in situ test and lab results. The standard penetration tests (SPT) and cone penetration test (CPT) are particularly convenient, given the extensive worldwide database and past experience. The most recent approaches to the assessment of liquefaction potential, following the proposals of Idriss & Boulanger (2010) and Boulanger & Idriss (2014), for SPT and CPT, respectively were adopted. These approaches consider the computation of the cyclic resistance ratio (CRR) from the normalized penetration resistance, adjusted to an equivalent clean sand (cs), as indicated below:

$$CRR_{7.5} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \tag{6}$$

$$CRR_{7.5} = \exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.8\right) \tag{7}$$

where $(N_1)_{60cs}$ is the normalized equivalent clean sand SPT penetration resistance, and q_{c1Ncs} corresponds to the normalized equivalent clean sand cone resistance. A clean sand is considered to have a fines content (FC) below 5%, as suggested by Idriss & Boulanger (2004). The introduction of the percentage of fines in these approaches reflects its importance in the liquefaction susceptibility of the soil. However, it should be noted an estimate of the FC, especially below 25%, may be inaccurate if based solely in the lithological descriptions of the SPT logs, in the absence of grain size distribution of those soils.

Details of the calculations based on SPT and CPT are provided in Boulanger & Idriss (2014). For ease of computation, all CPTu analyses were made using CLiq® software (v.2.0.6.92, GeoLogismiki, 2017).

For earthquake magnitudes other than 7.5, the cyclic stress ratio needs to be corrected by a magnitude scaling factor *MSF*. In this work, the calculation of *MSF* was made according to Idriss and Boulanger (2010) taking into account the type of soil, as follows:

$$MSF = 6.9\exp\left(-\frac{M_w}{4}\right) - 0.058 \le 1.8\text{, for sandy soils} \tag{8}$$

$$MSF = 1.12\exp\left(-\frac{M_w}{4}\right) + 0.828 \le 1.13\text{, for clayey soils} \tag{9}$$

2.2 *Liquefaction Potential Index*

An alternative approach to liquefaction assessment is based on Liquefaction Potential Index (LPI). This index, originally developed by Iwasaki et al. (1978), combines the safety factor with depth, *z*, to 20 m:

$$LPI = \int_0^{20m} F \cdot W(z) dz \tag{10}$$

where $W(z) = 10 - 0.5z$ (11)

and $F = 1 - FS_{liq}$, if $FS_{liq} \leq 1$ and $F = 0$, if $FS_{liq} > 1$ (12)

where FS_{liq} is the factor of safety previously defined in Equation 1. Based on correlations between computed LPI values and observations of liquefaction events from Japanese earthquakes, surficial liquefaction damages were classified. Table 2 shows the classification proposed by Sonmez (2003).

Table 2: Classification of liquefaction potential based on LPI (Sonmez, 2003)

LPI	Liquefaction potential
0	Very low
0 <LPI <5*	Low
5* <LPI <15**	High
15**> LPI	Very high

2.3 *Liquefaction Severity Number*

The Liquefaction Severity Number (LSN) is a quantitative indicator of the consequences of liquefaction, developed by Tonkin & Taylor (2013), and represents the expected damage effects of shallow liquefaction on direct foundations, based on post-liquefaction reconsolidation settlements and is defined as:

$$LSN = 1000 \cdot \int \frac{\varepsilon_v}{z} dz \tag{13}$$

where ε_v is the volumetric densification strain due to post-liquefaction consolidation of soil layer *i*, calculated from Zhang et al. (2002), and *z* is the depth of the soil layer in metres, below the ground surface, referring only to the top 10 m of the soil profile. Liquefaction severity can be classified in terms of expected damage, as follows:

Table 3: Liquefaction severity and damage based on LSN (Tonkin & Taylor, 2013)

LSN range	Typical performance
0 – 10	Little to no expression of liquefaction
10 – 20	Minor expression of liquefaction, some sand boils
20 – 30	Moderate expression of liquefaction, sand boils and some structural damage
30 – 40	Moderate to severe liquefaction, settlement can cause structural damage
40 – 50	Major expression of liquefaction, damage ground surface, severe total and differential settlements
>50	Severe damage, extensive evidence of liquefaction at surface, severe total and differential settlements affecting structures, damage to services

3 SELECTION AND CHARACTERISATION OF THE PILOT SITE

3.1 *Collection and analysis of existing information*

The selection of the location of the pilot site was based on the analysis of existing geological and geotechnical information in the metropolitan region of Lisbon along the Lower Tagus Valley. With the collaboration of public institutions, governmental agencies, private companies, contractors and design offices, 95 geotechnical reports were selected, summing up to more than 350 test results. The analysis of the collected reports was carried out according to the type of geotechnical data, and the classification of the liquefaction susceptibility of each soil profile was made. A minimum factor of safety of 1.00 was considered, associated with a minimum thickness of the liquefiable soil layer of 3 m. For ease of visualization and interpretation, each data point was geographically located and color-coded according to its liquefaction susceptibility, preliminarily on Google Earth®.

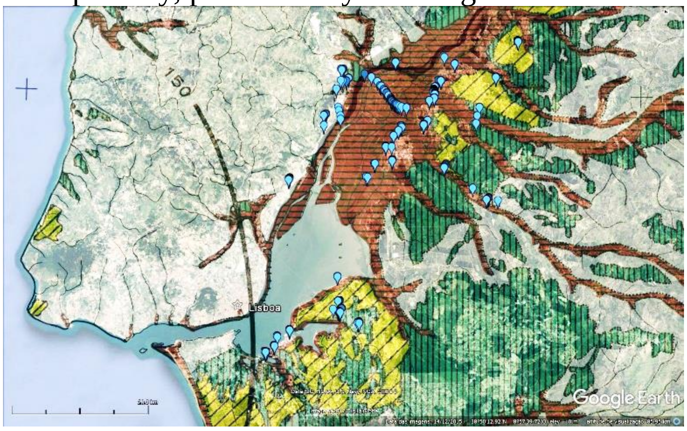

Figure 1: Location of the geotechnical reports collected in the greater Lisbon area, superimposed on the liquefaction zonation map (from Jorge, 1993). Red, yellow and green correspond to high to very high, moderate, and low liquefaction susceptibility zones, respectively; markers indicate geotechnical information.

3.2 *Location of the pilot site*

The municipality of Vila Franca de Xira is adjacent to Benavente where the 1909 earthquake occurred. Important works associated to the construction of a major highway (A10), including a 12km bridge and viaduct, provided a wealth of information from extensive geological and geotechnical site characterization. The area in the agricultural plains of the 'Lezíria Grande de Vila Franca de Xira' was found to have the ideal geological, hydrogeological and geotechnical, as well as operational conditions, for setting up a research pilot site on liquefiable soils. The area of the pilot site was divided into zones, named Site Investigation (SI) points, identified by the respective number. The geotechnical tests consists of 2 SPT at SI1 and SI7, 8 CPTu at SI1 to SI7 and SI10, as well as 3 SDMT and a wide range of geophysical tests (seismic refraction, SASW and HVSR) (not discussed in this paper). Figure 2 indicates the testing locations, including the geophysical measuring points.

Figure 2: Location of site investigation points for in situ characterisation at the pilot site

3.3 *Liquefaction susceptibility assessment at the pilot site*

For the purpose of comparison, two specific testing locations have been selected, where a greater number of tests has been performed, namely SI1 and SI7. Figure 3 illustrates the SPT results obtained in these two locations, in terms of simplified soil profiles and the normalized penetration resistance $(N_1)_{60cs}$ relevant for liquefaction studies. The soil profiles are significantly different, but it is worth noting that the values of the normalized penetration resistance do not exceed 15 blows in the first 30 m at both locations.

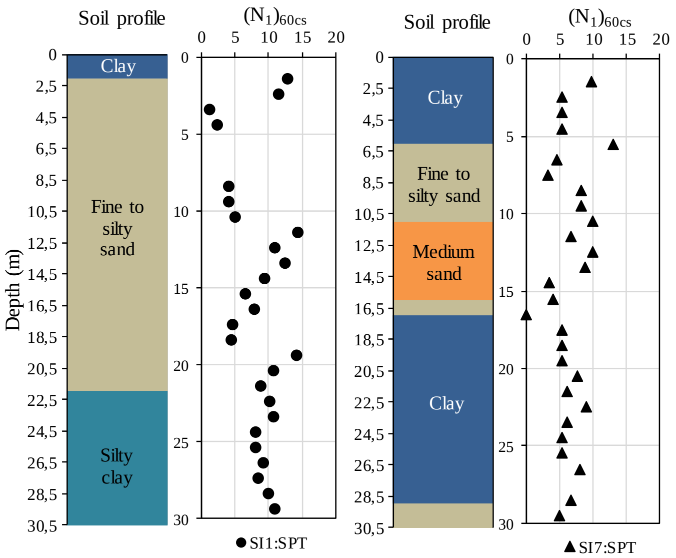

Figure 3: SPT results at the pilot site (SI1 and SI7)

Figures 4 and 5 show the CPTu tests results in terms of cone resistance, sleeve friction ratio, pore pressure and soil behavior type index (according to Robertson and Wride, 1997) for testing locations SI1 and SI7, respectively. Again, the comparison between the two soil profiles evidences considerable differences, not only in terms of the nature of the soil in depth, but also in terms of strength.

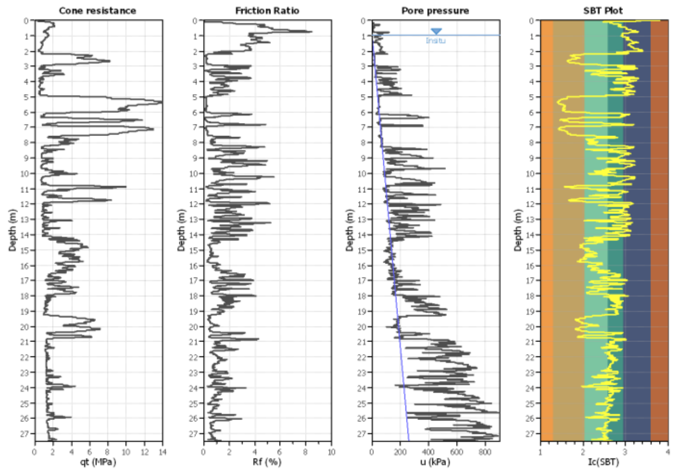

Figure 4: CPTu results: q_t; R_f; u; I_c for SI1

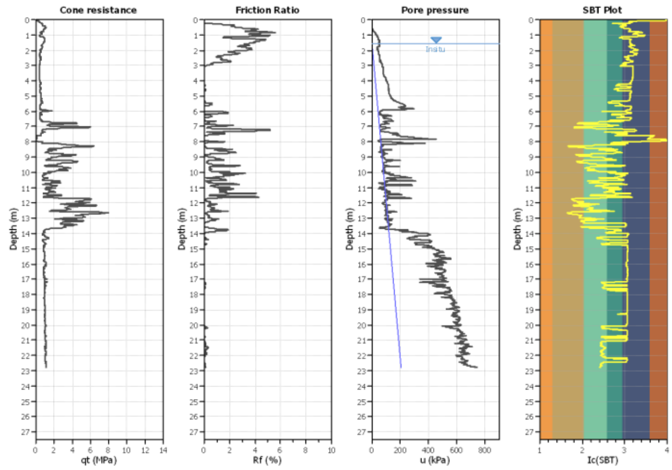

Figure 5: CPTu results: q_t; R_f; u; I_c for SI7

Comparing the soil profiles in Figure 3 with the soil behavior profiles in Figs. 4 and 5, the distinction is clear. While SPT results lead to the interpretation of homogenous soil layers, CPTu results reveals the existence of thin interbedded layers of clay/silt in the sandy deposits, which are clearly identified in the CPTu by means of the pore pressure measurements above the hydrostatic line. This fact will necessarily have an impact in the response of the soil in the context of liquefaction.

From these results, the assessment of liquefaction susceptibility was made. The factors of safety against liquefaction have been computed in depth for the two locations based on SPT and CPTu results in Figures 6 and 7. The low values of the factors of safety obtained in depth suggest the existence of thick layers of highly susceptible soils to liquefaction at both locations. The differences between the absolute values of FS_{liq} obtained from SPT and CPTu are not significant. It is however interesting to note the variability of the CPTu-based FS_{liq}, associated with the presence of interbedded layers of fine and granular soils.

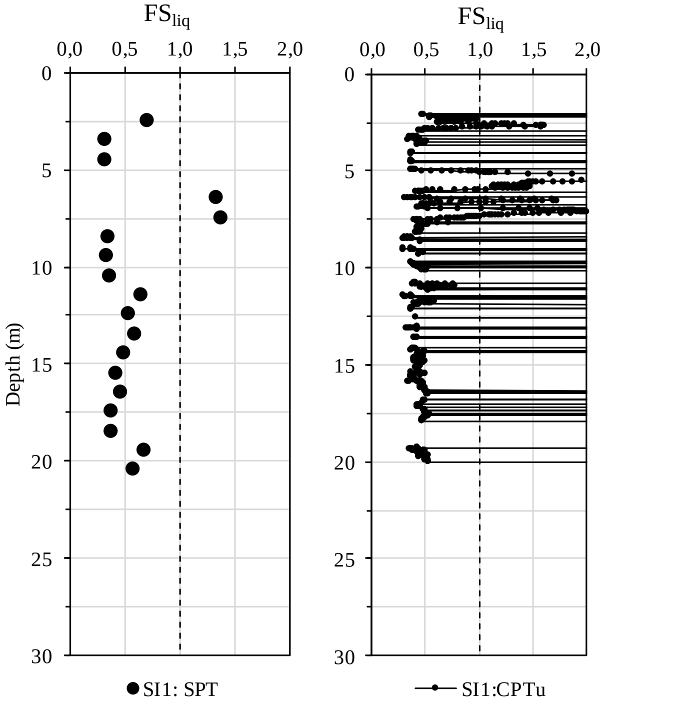

Figure 6: Comparison of SPT and CPTu assessment of liquefaction potential via FS_{liq} at SI1

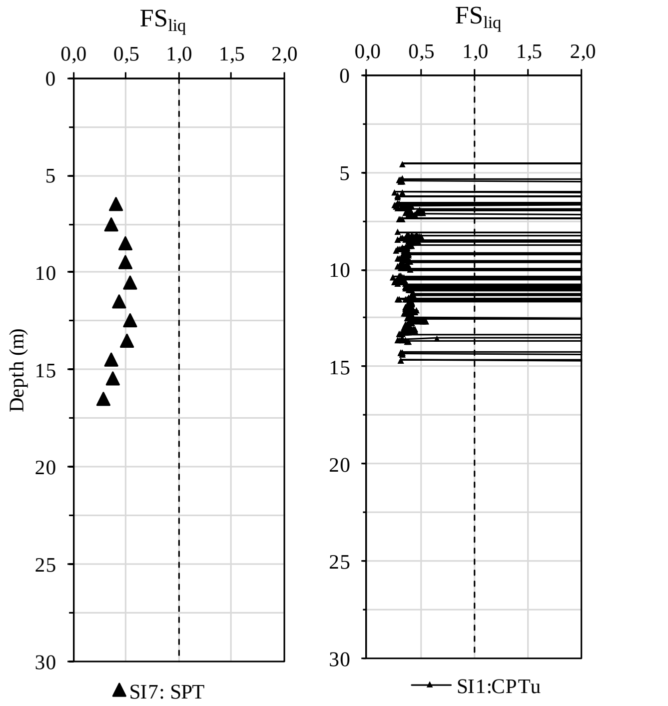

Figure 7: Comparison of SPT and CPTu assessment of liquefaction potential via FS_{liq} at SI7

The quantitative liquefaction indexes LPI and LSN have also been computed from these test results. A summary is provided in Table 4.

Table 4: LPI and LSN indexes computed from SPT and CPT results (for seismic action Type 1)

Test location	LPI		LSN	
	SPT	CPTu	SPT	CPTu
SI1	31.8	15.0	83.1	42.7
SI7	26.7	12.5	54.4	22.8

Comparing the values of LPI and LSN obtained from SPT and CPT tests, significant differences can be observed, especially at SI1. Given the presence of interbedded layers in the soil profiles, which were only identified by the CPTu test, the discrepancies in the quantitative indices are expected. In these soils, SPT-based liquefaction risk indices fail to accurately assess soil behavior with regard to liquefaction, providing considerably higher values that those estimated from CPTu results.

For assessing the variability of these indices across the pilot site, the overall results obtained in the 8 CPTu tests in terms of LPI and LSN are presented in Figures 8 and 9, respectively.

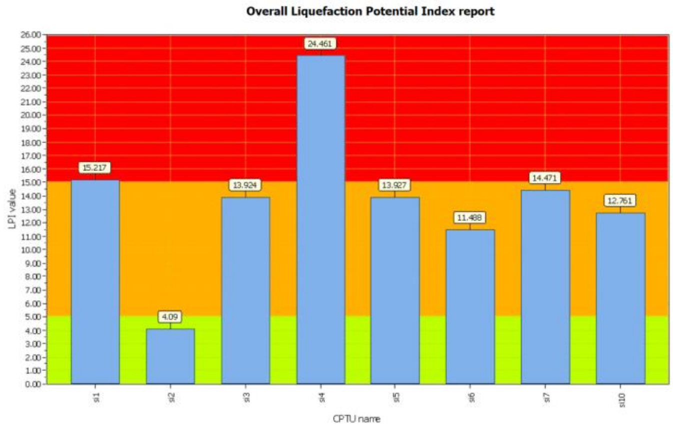

Figure 8: LPI values for CPT tests

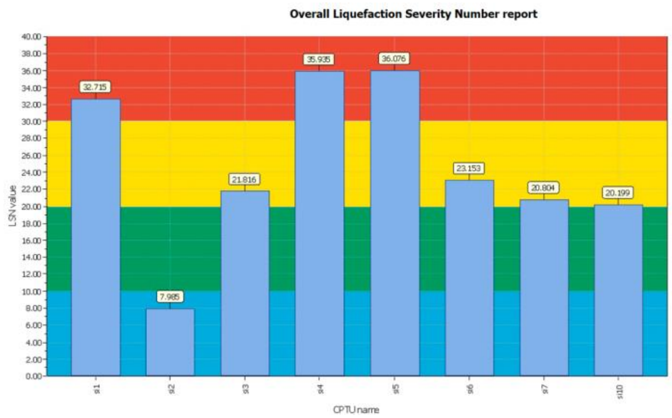

Figure 9: LSN values for CPT tests

The results exhibit considerable variation of the LPI and LSN values across the pilot site, with only one location with low liquefaction potential (SI2). The test locations with higher LPI also have high LSN, although the expected damage associated with liquefaction should not be severe in any location (LSN lower than 40).

4 CONCLUSIONS

Different methodologies for the assessment of liquefaction susceptibility by means of in situ penetration tests have been applied in a pilot site in liquefiable soils. The comparison of these methodologies provided an additional level of information and enabled to highlight the limitations of some of the approaches. In fact, the comparison between the derived values of the liquefaction risk indexes based on SPT and CPTu tests evidenced considerable discrepancies, which are a reflection of the level of detail of the characterization offered by each testing method.

For the case study of this paper, which involved profiles with interbedded layers of clay and silt in sand deposits, the results obtained using SPT data were satisfactory in terms of the factor of safety, but unrepresentative in terms of LPI and LSN. The combination of different approaches for liquefaction susceptibility assessment enabled to define and identify the most affected zones, to be subsequently applied for microzonation of the site.

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