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CPTu-based approaches for cyclic liquefaction assessment of alluvial soil profiles

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Article

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Abstract

Over the years, methods to assess cyclic liquefaction potential based on piezocone penetration tests (CPTu) have been developed. This paper presents a comparative study between three CPTu-based methodologies, mainly in terms of the normalization procedures of overburden stresses, equivalent clean sand resistance, and magnitude scaling factor (*MSF*). Four CPTu profiles from a pilot site in southwest Portugal are thoroughly analysed with different methods, in terms of factor of safety against liquefaction, the Liquefaction Potential Index (*LPI*), and the Liquefaction Severity Number (*LSN*). The site presents very heterogeneous soil profiles, composed of alluvial deposits. Due to the presence of significant sand-silt–clay interbedded layers, the influence of transition zones and the use of different soil behaviour type index (I_c) cut-off values were also considered. From these analyses, a set of recommendations is presented for CPTu-based liquefaction assessment. Based on the extensive database of CPTu results in the pilot site area, a new classification relating *LPI* and *LSN* is proposed to assess liquefaction severity and damage.

1. Introduction

Over the last years, soil liquefaction has been one of the major topics discussed and studied in geotechnical earthquake engineering, since earthquake-induced liquefaction has caused significant damage in buildings and infrastructures (Cubrinovski et al., 2011; Aydan et al., 2012). The susceptibility of soils to liquefaction depends mainly on two aspects: the soil resistance to cyclic loading and the design seismic action (SA). While earthquakes are usually sudden and unexpected, the assessment of the seismic actions can be made using reference values, namely those provided in codes and standards, or by site-specific ground response analyses. However, the soil resistance to cyclic loading can be determined from laboratory or field-testing. Laboratory tests involve either the collection of high-quality samples, which requires expensive and very difficult procedures, or the preparation of reconstituted specimens, which may be less representative of the natural soil conditions. Therefore, the use of field tests is a simpler and more economical procedure.

The piezocone penetration test (CPTu) is a widely used field test, as it provides an almost nearly continuous soil profile information, based on the soil resistance and the pore pressure developed during penetration, and is more reliable and repeatable than the SPT (Robertson, 2012). Over the years, methods to evaluate liquefaction susceptibility based on different in situ tests have been developed (Robertson & Wride, 1998; Robertson, 2009; Moss et al., 2006; Idriss & Boulanger, 2008; Boulanger & Idriss, 2014). However, there is not much consensus concerning the best criteria for evaluating liquefaction resistance based on CPT results. According to each methodology, the correlations and normalization factors are obtained differently, since the expressions were derived from different earthquake databases that have been updated over the years.

Within the framework of the European H2020 LIQUEFACT project, an extensive database of CPTu was collected and complemented, to assess the earthquake-induced risk of soil liquefaction at the Lisbon region in Portugal. This work focuses on the detailed analysis of four soil profiles, in terms of factor of safety against liquefaction (FS_{liq}), Liquefaction Potential Index (*LPI*), and Liquefaction Severity Number (*LSN*), using Robertson (2009), Moss et al. (2006), and Boulanger & Idriss (2014) methodologies. These methods are compared and contrasted, highlighting the main differences in the normalization procedures, namely in terms of computation of overburden stresses, equivalent clean sand resistance, and magnitude scaling factor (MSF). Moreover, the impact of considering the transition layers correction and the influence of the soil behaviour type index (I_c) cut off value on LPI and LSN are discussed. In the end, a correlation between LPI and LSN is proposed and verified for 37 CPTu tests performed in the testing site area.

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2. Review of the methods for liquefaction assessment

2.1. Simplified stress-based approach

Seed & Idriss (1971) developed a simplified procedure to estimate the potential for cyclic liquefaction due to earthquake loading, introducing the factor of safety against the triggering of liquefaction (FS_{liq}). This factor of safety represents the ratio between the capacity of a soil to resist liquefaction (cyclic resistance ratio, *CRR*) and a measurement of the earthquake loading induced in the soil (cyclic stress ratio, *CSR*) (Equation 1). If the *CSR* is greater than the *CRR* (i.e. $FS_{liq} < 1$), cyclic liquefaction will likely occur.

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

To estimate *CSR*, a site-specific ground response analysis should be carried out. However, Seed & Idriss (1971) proposed a simplified method based on the peak ground acceleration (a_{max}), expressed in Equation 2, where g is the acceleration of gravity, σ_{vo} and σ'_{vo} are the initial total and effective vertical stresses, respectively, and r_d is a shear stress reduction coefficient. The r_d coefficient provides an approximated correction for flexibility of the soil profile as it is a function of the non-rigid response of the soil deposit. *MSF* and K_σ are adjustment factors to account for the earthquake magnitude and the overburden stress, respectively, and are discussed in section 2.2.

$$CSR = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \frac{1}{MSF}\frac{1}{K_\sigma} \tag{2}$$

The *CRR* is a normalized value for an earthquake moment magnitude of 7.5 and effective vertical stress of 1 atm and can be obtained from correlations with field test results, namely the CPTu test. The *CRR* curve defines the boundary between liquefiable and non-liquefiable soils in the chart of *CSR versus* normalised cone resistance for clean sands.

The first "step" in the liquefaction assessment procedure based on CPT is to obtain the soil behaviour type index for each soil layer (Robertson, 1990). An iterative process relates the cone resistance (q_c), sleeve friction (f_s), and vertical stress (σ'_{vo}) normalization. The normalized cone resistance (Q_{tn}) and normalized friction ratio (F_r) are calculated using Equations 3 and 4 respectively, where σ'_{vo} is the initial effective vertical stress and σ_{vo} is the initial total vertical stress.

$$Q_{tn} = \left(\frac{q_c - \sigma_{v0}}{p_a}\right) \times \left(\frac{p_a}{\sigma'_{v0}}\right)^n \tag{3}$$

$$F_r = \left(\frac{f_s}{q_c - \sigma_{v0}}\right) \times 100\ \% \tag{4}$$

Over the years, the stress exponent (n) has been discussed and the most recent proposals (Robertson, 2009; Robertson, 2016), based on the critical-state soil mechanics framework, suggested that n varies with both I_c and effective overburden stress using Equation 5. The soil behaviour type index, which represents the normalised soil behaviour type (SBTn) zones in the $Q_{tn}-F_r$ chart, is defined by Equation 6 (Robertson & Wride, 1998).

$$n = 0.381 \times I_c + 0.05 \times \left(\frac{\sigma'_{v0}}{p_a}\right) - 0.15 \tag{5}$$

$$I_c = \left[\left(3.47 - \log Q_{tn}\right)^2 + \left(\log F_r + 1.22\right)^2\right]^{0.5} \tag{6}$$

According to the SBTn plot, $I_c < 1.31$ corresponds to gravel; $1.31 \le I_c < 2.05$ is for sand; $2.05 \le I_c < 2.60$ corresponds to silty sand to sandy silt; $2.60 \le I_c < 2.95$ is for silty clay to clayey silt and $I_c \ge 2.95$ refers to clay.

As mentioned above, the *CRR* can be obtained using correlations with CPTu results. Youd et al. (2001) reported the recommendations from the NCEER/NSF workshops in 1996 and 1998 for liquefaction assessment based on CPT measurements. Since then, many researchers have provided improvements and alternatives considering more complete liquefaction case history databases and different assumptions. The present work explores three methodologies proposed by Robertson (2009), Moss et al. (2006), and Boulanger & Idriss (2014). To simplify the representation and discussion from hereon, the analysed methods are abbreviated to R2009, MEA2006, and B&I2014, respectively. The expressions used in each method are different as the quantity and quality of case histories has increased with recent earthquake events. The reinterpretation of the new and existing data allows for the evolution and update of these methodologies, from which new approaches to assess liquefaction have been devised.

The proposal from Robertson (2009) is an update from Robertson & Wride (1998), which is similar to the recommendations from Youd et al. (2001). The CRR-$Q_{tn,cs}$ curve was based on the proposal from Robertson & Campanella (1985), which in turn was derived from the CRR-SPT relationship from Seed et al. (1985), by applying the SPT blow count and equivalent CPT tip resistance relationships. Later, CPT data from liquefaction and no liquefaction case histories validated the curve. Robertson & Wride (1998) suggested a relationship between the normalized cone resistance for clean sands with the cyclic resistance ratio for an earthquake with 7.5 moment magnitude, depending on the resistance value, later updated by Robertson (2009). Equation 7 presents the expressions, based on the value of $Q_{tn,cs}$.

$$\begin{aligned} &\text{if } 50 \le Q_{tn,cs} \le 160 \quad CRR_{7.5} = 93\left[\frac{Q_{tn,cs}}{1000}\right]^3 + 0.08 \\ &\text{if } Q_{tn,cs} < 50 \quad CRR_{7.5} = 0.833\left[\frac{Q_{tn,cs}}{1000}\right]^3 + 0.05 \end{aligned} \tag{7}$$

The method from Moss et al. (2006) is strongly based on Cetin (2000) and Cetin et al. (2004). This method was developed directly from measured CPT data and included about 200 liquefaction and no liquefaction case histories. The proposed relationship is probabilistic, however, they suggested the consideration of P_L = 15% for deterministic purposes and comparison with other methods, where P_L is the probability of liquefaction occurrence. Moss et al. (2006) presented a correlation that employs a larger database of high-quality field case histories, using a Bayesian framework to account for all the uncertainties associated with seismic demand and liquefaction resistance. Equation 8 presents the expression used to calculate CRR, where $q_{c,1}$ is the normalized tip resistance (in MPa), R_f is the friction ratio (f_s/q_c, in percent), c is a normalization exponent, M_w is the moment magnitude, σ'_v is the effective vertical stress, and $\Phi^{-1}(P_L)$ is the inverse cumulative normal distribution function.

$$CRR = \exp\left\{ \frac{\begin{bmatrix} q_{c,1}{}^{1.045} + q_{c,1}\left(0.110R_f\right) + \left(0.001R_f\right) + c\left(1 + 0.850R_f\right) \\ -0.848\ln M_w - 0.002\ln\sigma'_v - 20.923 + 1.632\Phi^{-1}\left(P_L\right) \end{bmatrix}}{7.177} \right\} \tag{8}$$

On the other hand, Boulanger & Idriss (2014) re-evaluated liquefaction triggering procedures and presented an update for the Idriss & Boulanger (2008) method, including data from recent earthquakes. The case history database was updated and the CRR curve changed slightly. This method, like MEA2006, was also derived directly from CPT case histories. The proposed correlation between $CRR_{7.5}$ and the normalized cone resistance for equivalent clean sand, q_{c1Ncs}, is presented in Equation 9.

$$CRR_{7.5} = \exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.8\right) \tag{9}$$

A common feature of the three methods is the use of an equivalent clean sand cone resistance to determine *CRR*. However, the normalizations are based on different assumptions, as presented in Table 1.

Robertson (2009) and Moss et al. (2006) methods infer the effect of fines (fines content and plasticity) from the CPT tip and sleeve measurements, as well as from the soil behaviour type index. On the other hand, Boulanger & Idriss (2014) method developed the fines content (FC) adjustment with information from case history databases, with measurements of FC from soil samples.

Table 1 summarises and evidences the differences between the methods, namely in terms of the calculation of equivalent clean sand resistance. Robertson & Wride (1998) suggested a method to calculate the apparent fines content directly from CPT results, as I_c increases with increasing apparent fines content and soil plasticity. However, since the CPT penetration resistance is also influenced by other grain characteristics, such as mineralogy, plasticity, sensitivity, and stress history, they proposed the use of a correction factor, K_c, based on the I_c.

On the other hand, MEA2006 uses an additive factor as equivalent clean sand adjustment, despite computing *CRR* from q_{c1N} and not from the equivalent clean sand resistance. The CPT normalization for overburden stress is based on cavity expansion models, in conjunction with field and laboratory tests and corresponds to a function of cone tip resistance and friction ratio (Moss et al., 2006). Note that the normalization for overburden stress is performed similarly on R2009 and B&I2014, only changing the formulas of the stress exponents.

Boulanger & Idriss (2014) proposed an equivalent clean sand adjustment, empirically derived from liquefaction case history data, which was guided by the trends in q_c/N_{60} ratios versus FC (Idriss & Boulanger 2008). The proposal involves an iterative process with q_{c1Ncs} and the value of fines content, to take into account the increase of cyclic resistance with fines content.

2.2. Adjustment factors

As the basis of the simplified stress-based formulation from Seed & Idriss (1971) was intended for a reference earthquake magnitude of 7.5 (corresponding to an equivalent number of cycles of 15) and an effective stress of 1 atm, adjustment factors were proposed to account for sites with different conditions. These adjustment factors include the magnitude scaling factor, *MSF*, that reflects the duration of shaking and the associated number of loading cycles, and the overburden correction factor, K_σ, to account for the effect of the effective vertical stress, as well as the shear stress reduction coefficient, r_d. These normalization parameters are calculated differently, according to the method used, as presented in Table 2.

Robertson (2009) adjustment factors are based on the first considerations of Seed & Idriss (1971), when they proposed the simplified procedure. Following the proposal by Robertson & Wride (1998), the *MSF* is only dependent on the earthquake magnitude, and r_d is only influenced by depth. However, Robertson (2009) introduced an update regarding K_σ. The influence of the overburden stress is reflected in the general regression, in the form of the stress exponent n. As for MEA2006, this method derived the adjustment factors directly from the liquefaction case history database and included the magnitude in the regression to compute CRR (as shown in Table 1). Moss et al. (2006) reassessed the r_d expression from Cetin et al. (2004) using the ground response, being more representative of the induced cyclic shear stress. However, the majority of case history databases

Table 1. Calculation parameters for equivalent clean sand resistance according to the method.

	Robertson (2009)	Moss et al. (2006)	Boulanger & Idriss (2014)
Equivalent clean sand resistance	$Q_{tn,cs} = K_c Q_{tn}$	$q_{c,1,mod} = q_{c,1} + \Delta q_c$	$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$
Factor to account for behaviour/fines content	$K_c = 1.0 \; if \;\; I_c \le 1.64$ $K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \;\; if \;\; I_c > 1.64$	$\Delta q_c = \left(0.38 R_f - 0.19\right) \ln CSR + \left(1.46 R_f - 0.73\right)$	$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) \exp\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right)$
Normalized cone resistance for overburden stress	$Q_{tn} = \left(\frac{q_t - \sigma_{vo}}{p_a}\right) \times \left(\frac{p_a}{\sigma'_{vo}}\right)^n$	$q_{c,1} = C_q q_c$ $C_q = \left(\frac{p_a}{\sigma'_v}\right)^c \le 1.7$	$q_{c1N} = C_N \frac{q_t}{p_a}$ $C_N = \left(\frac{p_a}{\sigma'_{vo}}\right)^m \le 1.7$
Stress exponent	$n = 0.381 \times I_c + 0.05 \times \left(\frac{\sigma'_{vo}}{p_a}\right) - 0.15$	$c = f_1 \left(\frac{R_f}{f_3}\right)^{f_2}$ $f_1 = 0.78 q_c^{-0.33}$ $f_2 = -\left(-0.32 q_c^{-0.35} + 0.49\right)$ $f_3 = \text{abs}\left[\log(10 + q_c)\right]^{1.21}$	$m = 1.338 - 0.249 \left(q_{c1Ncs}\right)^{0.264}$
Apparent fines content	$FC(\%) = 0 \; if \; I_c < 1.26$ $FC(\%) = 1.75 I_c^{3.25} - 3.7 \;\; if \, 1.26 \le I_c \le 3.5$ $FC(\%) = 100 \;\; if \;\; I_c > 3.5$	-	$FC(\%) = 80\left(I_c + C_{FC}\right) - 137$ C_{FC} considered 0

Table 2. Adjustment factors for each method.

	Robertson (2009)	Moss et al. (2006)	Boulanger & Idriss (2014)
Stress coefficient	$r_d = 1.0 - 0.00765 z$ $for \; z < 9.15 m$ $r_d = 1.174 - 0.0267 z$ $for \, 9.15 m < z < 23 m$	$r_d = \dfrac{\left[1 + \dfrac{-9.147 - 4.173 \cdot a_{max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.089 \cdot (-d \cdot 3.28 - 7.760 \cdot a_{max} + 78.576)}}\right]}{\left[1 + \dfrac{-9.147 - 4.173 \cdot a_{max} + 0.652 \cdot M_w}{10.567 + 0.089 \cdot e^{0.089 \cdot (-7.760 \cdot a_{max} + 78.576)}}\right]}$	$r_d = \exp\left[\alpha(z) + \beta(z).M\right]$ $\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$ $\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$
MSF	$MSF = \frac{10^{2.24}}{M_w^{2.56}}$	$DWF_M = 17.84 \Delta M_w^{-1.43}$	$MSF = 1 + \left(MSF_{max} - 1\right)\left(8.64 \exp\left(\frac{-M}{4}\right) - 1.325\right)$
K_σ	1	1	$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{p_a}\right) \le 1.1$ $C_\sigma = \frac{1}{37.3 - 8.27 (q_{c1Ncs})^{0.263}} \le 0.3$

were limited to earthquake magnitudes between 6.9 and 7.6 (computing *MSF* close to 1) and effective stresses around 50 to 120 kPa. Outside these intervals, care should be taken, as the regressions may not be appropriate. On the other hand, Boulanger & Idriss (2014) considered the equivalent clean sand cone resistance in the calculations of *MSF* and K_σ and added the moment magnitude to the calculation of r_d, along with depth, making the parameters more dependent on soil type. B&I2014 applied a cap for small values of magnitude ($M_w < 5.25$). Moreover, the *MSF* is dependent on a soil type parameter, being related to the equivalent clean sand resistance.

In sum, each method presents its specific considerations, and should be applied consistently with its adjustment factors. In this work, the analyses were made by critically comparing the results without any direct or weighted averaging (such as in logic tree approaches), as the combination of methods can led to unrealistic solutions.

2.3. Liquefaction Severity Indices

The factor of safety provides information about whether liquefaction is likely to occur or not, but it does not give indications about the severity of the manifestation or its cumulative effect along the soil profile. Therefore, liquefaction severity indices were developed to study the damage potential and severity of surface manifestations of liquefaction. These qualitative methods have the advantage of providing a quantitative classification of the overall liquefaction response of the entire soil profile. However, by being computed as the sum of the behaviour of each data point individually, instead of the different macro layers, the values of these indices may be, in some cases, inaccurate or misleading, especially since these do not account for cross-interactions between different layers during the development of liquefaction and post-liquefaction, as discussed by Cubrinovski et al. (2019).

One of these frameworks is the liquefaction potential index (*LPI*), proposed by Iwasaki et al. (1978), which translates the liquefaction potential damage. The *LPI* is mainly dependent on the factor of safety and is defined as:

$$LPI = \int_{0}^{20\,m} F_1 w(z)\,dz \tag{10}$$

where $F_1 = 1 - FS_{liq}$ for $FS_{liq} \leq 1.0$ and $F_1 = 0$ for $FS_{liq} > 1$ and $w(z) = 10 - 0.5z$ for $0 \leq z \leq 20$ m and $w(z) = 0$ for $z > 20$ m, where FS_{liq} is the factor of safety and z is the depth above ground surface in meters. Iwasaki et al. (1978) defined the liquefaction severity as minor for $0 < LPI \leq 5$, moderate for $5 < LPI \leq 15$ and major for $LPI > 15$. Other authors suggested slightly different intervals (Toprak & Holzer, 2003; Lee et al., 2003; Sonmez, 2003).

To indicate the liquefaction-related vulnerability of residential dwellings, a parameter was developed by Tonkin & Taylor (Tonkin & Taylor, 2013; GeoLogismiki, 2017), named Liquefaction Severity Number (*LSN*). Equation 11 defines this parameter, which considers the volumetric densification strain within soil layers (ε_v) proposed by Zhang et al. (2002) and a power law for a depth weighting factor ($1/z$). The calculation of ε_v is a function of FS_{liq} and relative density, and describes the expected post-liquefaction volumetric deformations.

$$LSN = 1000 \int_{0}^{10\,m} \frac{\varepsilon_v}{z}\,dz \tag{11}$$

Based on this parameter, the liquefaction severity was defined as little to no expression for $LSN < 10$, minor for $10 < LSN < 20$, moderate for $20 < LSN < 30$, moderate to severe for $30 < LSN < 40$, major for $40 < LSN < 50$, and severe damage for $LSN > 50$. van Ballegooy et al. (2012) did not define a specific depth, while other researchers adopted *LSN* for the first 20 m of depth (Giannakogiorgos et al., 2015; Maurer et al., 2015). For clarity, a comparative analysis is presented in this work, emphasising the influence of considering the first 10 m (LSN_{10}) or 20 m (LSN_{20}) of the soil profile.

3. Experimental program

The experimental program was developed as part of the activities of European project LIQUEFACT for the microzonation for earthquake-induced risk of soil liquefaction at the Lisbon area in Portugal (Viana da Fonseca et al., 2019a). Several CPTu tests were performed in the municipalities of Benavente and Vila Franca de Xira, in Lisbon, Portugal, from which a selection is analysed and discussed in detail in this work. Historical records show that this zone is prone to liquefaction, as observed during historical earthquakes in the area (Jorge & Vieira, 1997). The geological, geomorphological, and seismic characteristics of the site emphasise this susceptibility due to the presence of recent alluvial sand deposits in a high seismicity zone (Ferreira et al., 2020). The pilot site is located in the Lower Tagus Valley and is composed of fluvial and marine sediments, from the Pliocene to Holocene, and presents stratification irregularities, with lenticular or bevelled layers, due to the sedimentation processes. Viana da Fonseca et al. (2019b) described the area in detail. In this work, four CPTu were analysed in detail, namely SI1, SI7, NB1, and NB2, and another 33 CPTu were used to verify the proposed *LPI*−*LSN* correlation, which locations are presented in Figure 1.

The CPTu were performed according to the procedures prescribed in the European standard (ISO, 2012), with recording measurements of cone tip resistance (q_c), sleeve friction (f_s), and pore pressure (u) every 1 cm depth, providing an almost continuous profile of the soils.

The four selected sites are constituted by alluvial sand deposits with clay-silt-sand interlayers, as is observed in Figure 2. SI1 presents many clay-sand interlayers. However, two sandy layers are identified at around 2 m to 3 m and from 5 m to 7 m, this last layer interbedded with two clay layers. SI7 is very heterogeneous with no clear sand layer, but many interlayers between 6 m to 14 m. NB1 presents a distinct sand layer at around 4 m to 7 m. In NB2, between 5 m and 13 m, the layers are mostly constituted of sand with some small-interbedded clays. These different profiles will help define the influence of the different layers in the liquefaction assessment of soil profiles.

4. Results

4.1. Factor of safety against liquefaction

As described above, each CPTu profile was analysed using the three methods. The seismic considerations followed

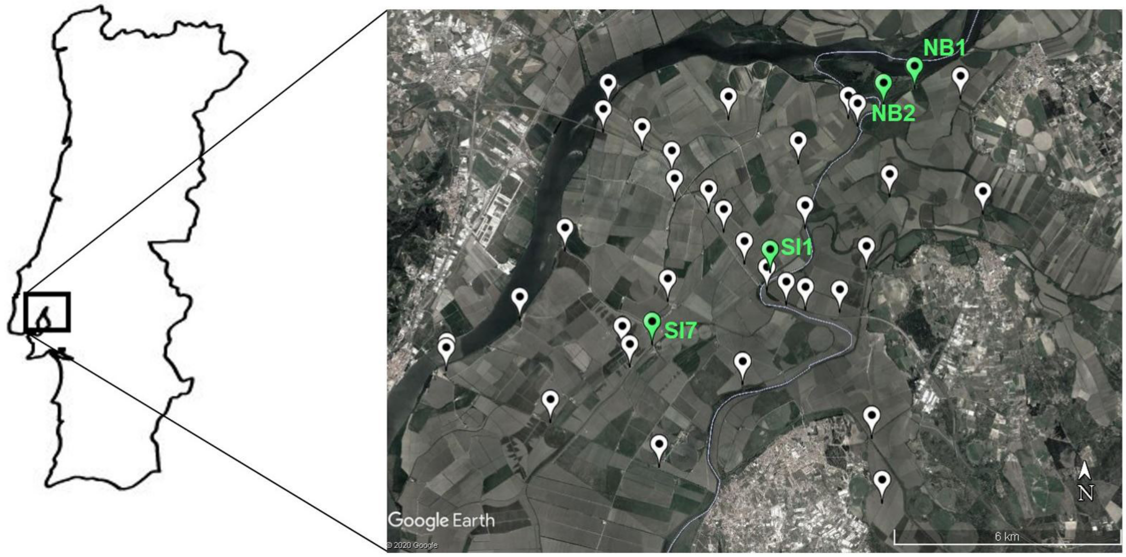

Figure 1. Location of the CPTu testing sites in the LIQUEFACT pilot site area.

the procedures included in the European Standard Eurocode 8 and the National Annex of Portugal (BSI, 2004, 2010). The seismic action was calculated for a return period of 475 years and ground type D, as the deposits were considered loose-to-medium cohesionless soil (Ferreira et al., 2020). The peak ground acceleration, a_{max}, considered at ground surface, was defined as 0.20g and 0.31g, for type 1 and 2 of seismic action (SA) respectively. As for the magnitude moment, EN 1998-5 (BSI, 2004) defines 7.5 and 5.2 for SA types 1 and 2, respectively, for the municipalities of Vila Franca de Xira and Benavente.

Figure 2 presents the profiles of cone resistance (q_c) and pore pressure (u), soil behaviour type index (I_c), and the factor of safety against liquefaction (FS_{liq}) for SA type 1, calculated according to Equation 1 for the three methods. Detailed SA type 2 results are available elsewhere (Ramos, 2021). Only FS_{liq} for layers with $I_c < 2.60$ are represented, as $I_c = 2.60$ corresponds to FC of approximately 35% in the Robertson (2009) method, and was defined in this work as the limit value for the occurrence of liquefaction. For SI1, a photograph of one of the SPT samples collected at around 4 m depth is also presented, where the interlayers are evident, with a clear distinction between thin layers of sand and clay.

SA type 1 is characterized by a lower a_{max} and higher M_w. In this case, the magnitude corresponds to the reference value of 7.5, so *MSF* is 1 and does not affect the results. In the more interlayered profiles, SI1 and SI7, the R2009 and MEA2006 methods compute more conservative results, showing lower FS_{liq} values. However, in the more homogeneous layers (for example between 4 m and 7 m in NB1 or between 5 m to 14 m in NB2), the B&I2014 is more conservative, generally resulting in lower values of FS_{liq}. Despite these differences, it is perceptible that all methods identify the same critical layers, being B&I2014 the most conservative in general, as it delivers lower values of FS_{liq}. These results also show that CPTu profile influences the prediction of liquefaction by the different methods, which is why consistency is important. The differences among the three methods are not easily distinguishable in the form of Figure 2. Therefore, in the following section, the analyses will be based only on *LPI* and *LSN*, which indirectly show the differences in FS_{liq}.

4.2. LPI and LSN

As mentioned above, the factor of safety against liquefaction is insufficient to provide indications about the severity of the manifestation or the cumulative effect throughout the soil profile. To better understand the influence of the different methods and assess the damages induced in the soil in case of liquefaction, the liquefaction potential index, *LPI* (Figure 3), and the liquefaction severity number, *LSN* (Figure 4), were analysed for the four profiles using the three methods.

For the *LPI*, the SA type plays an important role, as evidenced in the comparison of Figure 3a and 3b. The values from R2009 decrease significantly for SA type 2 (higher a_{max} and lower M_w) while B&I2014 values increase. The values of MEA2006 are nearly unaffected by SA type, despite a minor decrease for type 2. As expected, the soil profile highly influences the results. All soil profiles exhibit a high to very high risk of liquefaction, except for NB1 and NB2 for R2009 method with seismic action type 2. Once again, R2009 demonstrates higher dependency with *MSF*, since for the lower value of M_w ($M_w = 5.2$), the computed *MSF* value is very high, thus strongly decreasing *CSR* and delivering higher FS_{liq} values, hence lowering *LPI*.

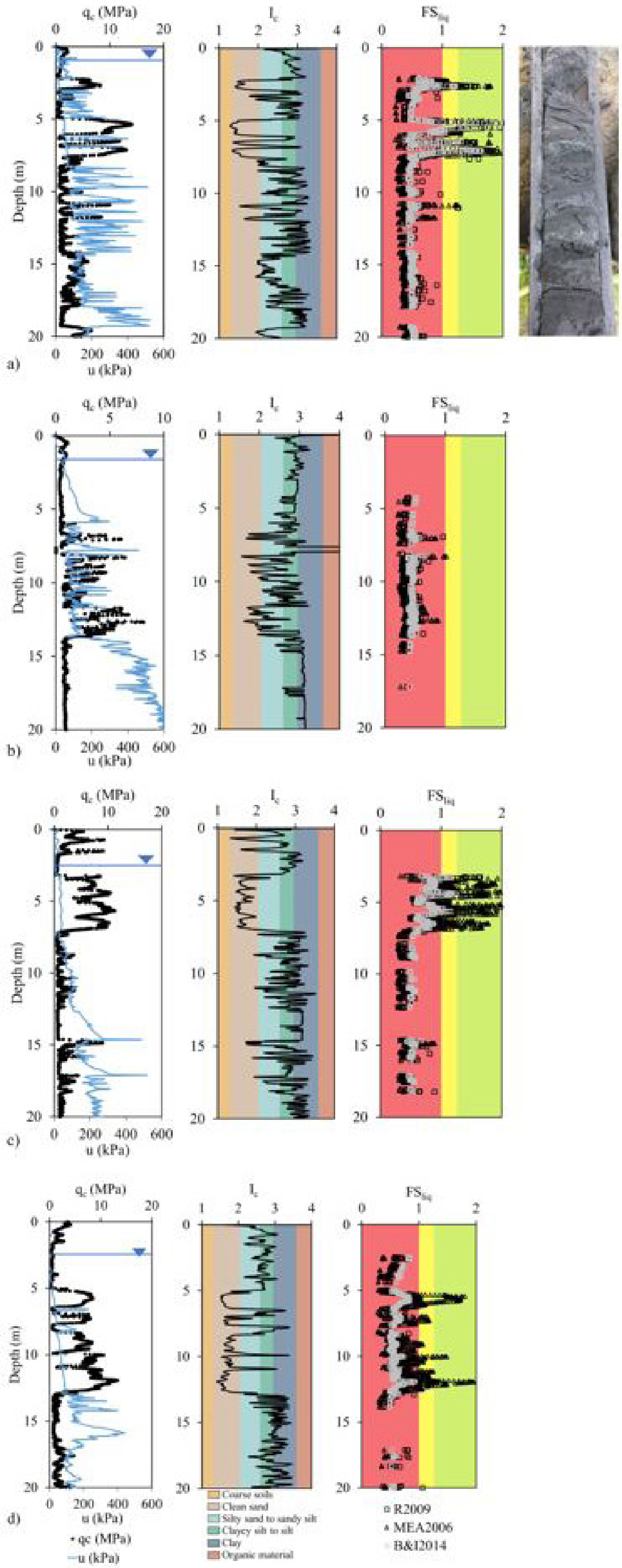

Figure 2. CPTu results in the experimental sites: (a) SI1 (including a photograph of the SPT sample collected at 4 m); (b) SI7 (c) NB1; (d) NB2.

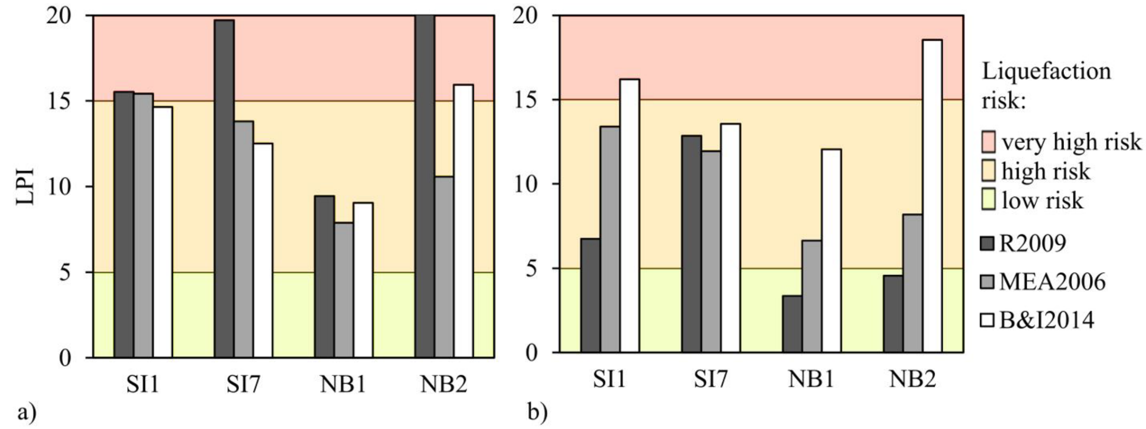

Figure 3. LPI results: (a) SA type 1, (b) SA type 2.

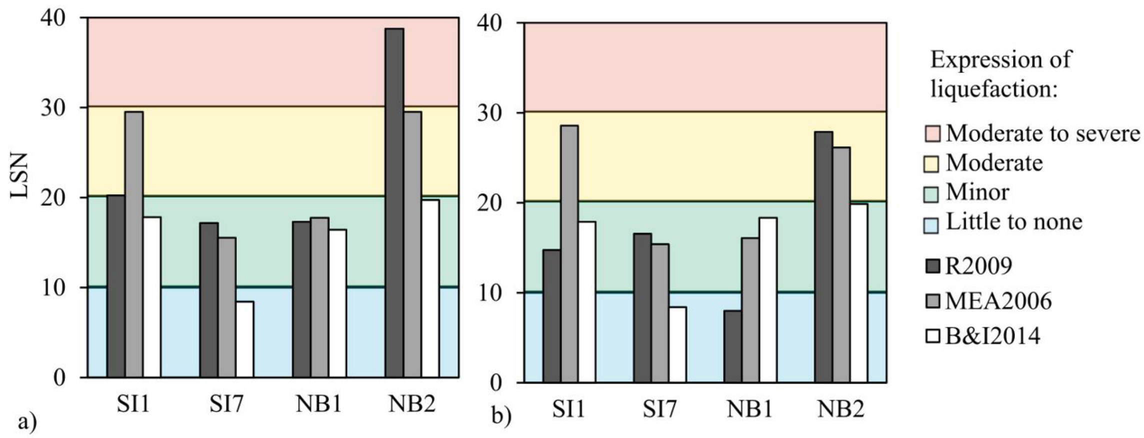

Figure 4. LSN_{10} results: (a) SA type 1, (b) SA type 2.

As for *LSN*, the differences are not so significant. Once more, the selection of the method influences the trends and the differences between the two SA types. R2009 method is the most affected by seismic action parameters, as *MSF* strongly varies according to the moment magnitude, producing higher values of FS_{liq} for SA type 2. In turn, MEA2006 and B&I2014 are less influenced by the SA type, since ε_v calculation depends on FS_{liq}. In effect, Zhang et al. (2002) calculation of ε_v considers a minimum FS_{liq} of 0.5, and since most FS_{liq} values for MEA2006 and B&I2014 are close to or lower than 0.5, the changes due to seismic action type are not visible. This also justifies the nearly identical results for SI7, even from R2009 method. The more interlayered profiles, SI1 and SI7, reveal minor to moderate expression of liquefaction. NB1 presents a minor expression of liquefaction, while NB2 is the most critical profile, presenting moderate to severe liquefaction expression. This is a consequence of the type of soils above 10 m, composed mainly of sands and silty sands with low FS_{liq}. Besides, it is interesting to note that the Zhang et al. (2002) method was proposed based on the $Q_{tn,cs}$ definition of Robertson & Wride (1998). However, in current practice, it can be assumed that the effect of the new definitions of the normalized cone tip resistance (according to Boulanger & Idriss, 2014) is expected to be negligible. Therefore, the use of Zhang et al. (2002) method with the safety factors computed according to B&I2014 is considered viable and reliable (as currently available in the CLiq software). Note that *LPI* is calculated for the layers where FS_{liq} is lower than 1.0, up to a depth of 20 m, while *LSN* is calculated for the first 10 m depth and with FS_{liq} lower than 2.0. For this reason, the two indices are not directly comparable. Nevertheless, it can be concluded that the tested area has a high to very high risk of liquefaction and minor to moderate expression of liquefaction damage. In an attempt to relate the results of *LPI* and *LSN*, *LSN* was calculated considering the first 20 m of depth (LSN_{20}). Figure 5 presents these results for SA types 1 and 2, showing that, as expected, the values are higher than those in Figure 4.

4.3. Consideration of other factors

As stated by Robertson & Wride (1998), the cone resistance is influenced by the soils ahead and behind the cone

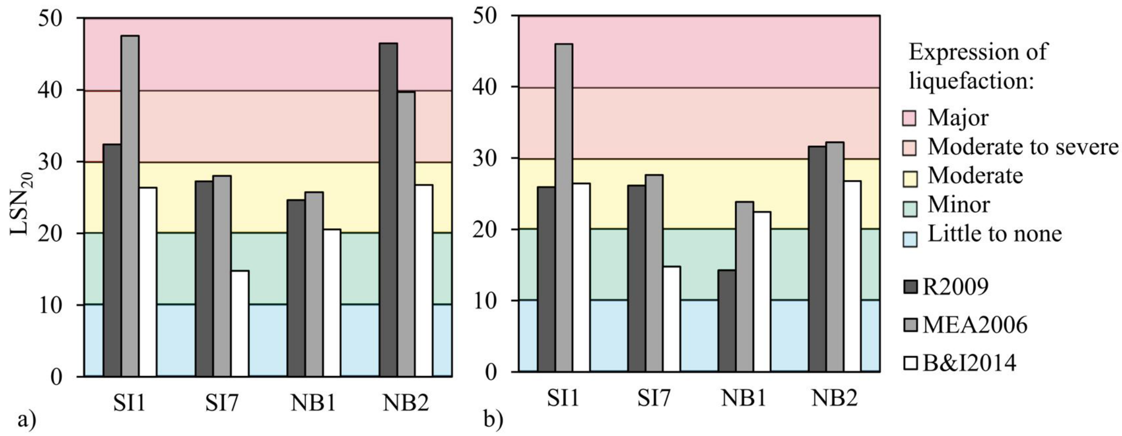

Figure 5. LSN_{20} results considering the first 20 m: (a) SA type 1, (b) SA type 2.

tip. Near the interface of two distinct soil layers, the changes in CPTu measurements are difficult to interpret and may be misleading. When the cone is moving from one soil type to another, especially if there is a significant difference in soil stiffness or strength (e.g. soft clay to sand), the CPT data within the transition zone is usually excessively conservative. This is particularly relevant when dealing with thinly interbedded soil. The analysed soil profiles are very heterogeneous, with sand-clay interlayers that affect the sensitivity of the CPTu measurements (see Figure 2). Considering the existence of such interlaying, a complementary analysis was performed excluding the transitional layers from the calculations, as a means to highlight and detect the differences between the consideration (or not) of those layers in the liquefaction assessment frameworks.

This procedure is already implemented in Cliq®, the software used to perform the CPTu calculations (version v.2.2.0.37, GeoLogismiki, 2017). The range of I_c where the transitional layers can be found was set to $1.80 < I_c < 3.00$ as these were the values considered to include silts and sandy silts. The transitional points are found when the I_c changes rapidly, defined as a rate of $\Delta I_c = 0.01$, where ΔI_c is the I_c change in a given thickness (Yi, 2018). The analysis presented below refers only to seismic action type 1, as the comparison between the two seismic actions was identical to the previous discussion.

Figure 6a presents the *LPI* and LSN_{20} values obtained with the correction of transitional layers, overlapping the results considering all layers. The values considering the correction of transitional layers are significantly lower, and consequently, the liquefaction hazard is lower than when considering all layers. It can be concluded that the initial analysis might be very conservative and the elimination of transitional layers increases the convergence of results of *LPI* and LSN_{20}. For the *LPI*, R2009 and B&I2014 are more conservative than MEA2006 and all methods are highly dependent on the soil type profile. For the LSN_{20}, the transition layer correction affects especially SI1 and SI7, which was expected, since these are the most interlayered profiles.

The $I_c = 2.60$ is normally considered as the cut-off between liquefiable and non-liquefiable soils (Robertson, 2009), corresponding to 35% of FC. However, R2009 and B&I2014 present very distinct relationships between FC and I_c. For $I_c = 2.6$, FC is around 70% in B&I2014 approach. These differences highlight the importance of the sensitivity study, using different I_c cut-off values, for the B&I2014 method, presented in Figure 6b. The values selected were $I_c = 2.80$ (a higher value to account for fine-grained soils with potentially low plasticity), $I_c = 2.60$ (the value suggested by R2009 and used throughout this work), $I_c = 2.15$ (I_c for FC = 35% in B&I2014) and $I_c = 2.35$ (an intermediate value suggested by Ferreira et al. 2020). The values of *LPI* and LSN_{20} decrease when considering lower I_c cut-off values, which is understandable as lower I_c values correspond to the consideration of fewer layers. The differences are considerable as the soil profiles are composed of many sandy silt and silty sand layers. It is important to note that I_c cut-off of 2.80 is only reasonable if the fines fraction has very low plasticity.

The selection of I_c cut-off value is a pertinent issue, since it expresses the typical behaviour of each soil layer, encompassing a variety of grain characteristics, not only the fines content. The choice of the I_c cut-off is, therefore, very conditioning, as it influences the layers considered in the calculations. Previous research (Boulanger & Idriss, 2014) stated that there is a lot of scattering when dealing with relationships between FC and I_c, likely due to the different plasticity of the fines. A soil with the same fines content can present low or high plasticity, which influences the soil to behave more like a sand (low plasticity) or a clay (high plasticity). This issue was also addressed by Facciorusso et al. (2019), when comparing the LPI obtained with various CPTu-based methods and considering different I_c cut-off values. They concluded that, if intermediate soils are considered, an increase of the cut-off from 2.6 to 2.7 can determine a significant increase in LPI. Therefore, if large differences in

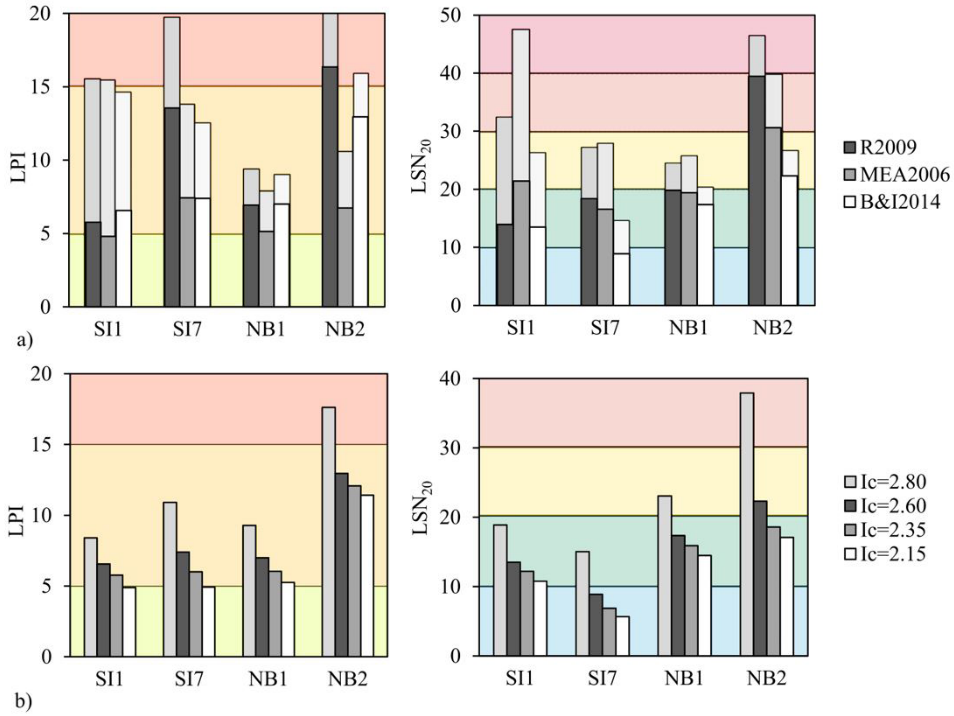

Figure 6. Impact of the consideration of different factors on the liquefaction hazard: (a) transitional layers, (b) I_c cut-off values.

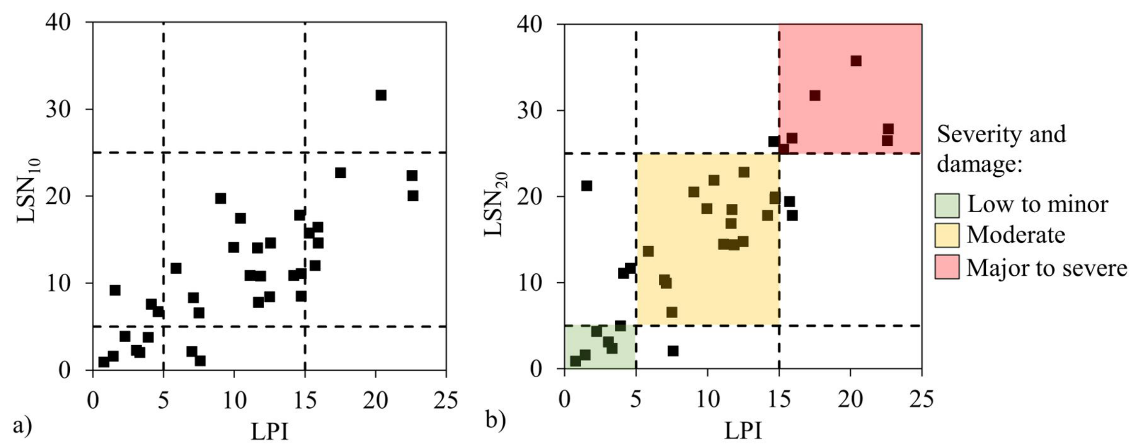

Figure 7. Severity and damage assessment using LPI and LSN results: (a) LSN_{10}, (b) LSN_{20}.

LPI are detected when adopting different I_c cut-off values, direct measurement of FC and plasticity index should be collected and integrated in the analyses. Estimates of the plasticity index (*PI*) from CPTu results are particularly difficult, due to the lack of substantial comparative results in the existing databases. Therefore, a sensitivity analysis should be performed to assess the effect of this parameter in the global liquefaction assessment.

5. Analysis and discussion

The comparison between *LPI* and *LSN* is not straightforward, as the original formulations state that *LPI* is calculated for the first 20 m and *LSN* for the first 10 m of the soil profile. However, to allow for a more direct comparison between the two indices, the *LSN* was also calculated for the first 20 m, and designated as LSN_{20}. This approach led to a convergence of the qualitative results of *LPI* and LSN_{20}, as the same critical layers were considered. Figure 7 presents the relationships between *LPI* and *LSN*, calculated for the first 10 m and 20 m, for the 4 analysed profiles together with other 33 CPTu tests, of the same pilot site (see Figure 1). The results were obtained using B&I2014 method for SA type 1. As expected, the consideration of LSN_{20} reduces the dispersion of the data points, thus increasing the compatibility of the expected liquefaction severity and damage. As a result, a classification based severity and damage assessment using both values of *LPI* and LSN_{20} is proposed in Figure 7b.

A critical analysis is recommended, as some points appear outside the selected limits but close to the boundaries, revealing the expected damage. The proposed severity and damage boundaries are: low to minor (*LPI* < 5 and LSN_{20} < 5), moderate (5 < *LPI* < 15 and 5 < LSN_{20} < 25) and major to severe (*LPI* > 15 and LSN_{20} > 25). It is important to state that this is a conceptual approach based on the current case study profiles, using *LPI* and *LSN* from CPTu results, not considering observed liquefaction damages. Some other works have suggested an *LPI-LSN* classification chart based on observed liquefaction manifestations after the Emilia-Romagna 2012 earthquake and the 2010–2011 Canterbury earthquake sequence (Giannakogiorgos et al., 2015; Papathanassiou et al., 2015). However, in this case, no data were available for that analysis.

6. Conclusions

A set of four CPTu tests was selected, from an extensive database of field tests performed at a pilot site in Vila Franca de Xira and Benavente, in Portugal, and thoroughly analysed to study the differences between various liquefaction assessment approaches, namely those proposed by Robertson (2009), Moss et al. (2006), and Boulanger & Idriss (2014). From the results, the following conclusions were drawn:

- The importance of consistency when using a CPTu-based liquefaction assessment method has been highlighted. Therefore, the implementation of averaging or logic tree approaches (Lai et al., 2020) is not recommended as it contradicts the coherence of the analyses discussed above;
- The authors propose the use of Boulanger & Idriss (2014) method, also recommended by Cubrinovski (2016). This method considers the effect of fines content, by directly introducing the fines content values in the calculations, which facilitates the use of laboratory grading data. Furthermore, the methodology was also developed for standard penetration tests (SPT) allowing to contrast and compare results from both tests. Besides, the B&I2014 is the most recent method, and is based on the largest dataset of liquefaction cases, including the 2010-2011 Canterbury earthquake sequence in New Zealand and the 2011 Tohoku earthquake in Japan, which were not available in the past. This methodology was also adopted for the analysis of an extensive database, developed under the LIQUEFACT project (Ferreira et al., 2020; Lai et al., 2020);
- The elimination of transitional layers, and the consideration of different I_c cut-off values when dealing with interbedded profiles is recommended. The I_c value expresses the typical behaviour of each soil layer, encompassing a variety of grain characteristics, not only the fines content. The plasticity of the fines is often responsible for the scatter when dealing with relationships between FC and I_c. As estimates of *PI* from CPTu results are particularly difficult, the sensitivity analysis considering different values of I_c cut-off allows for a more accurate global liquefaction assessment;
- A new classification chart relating *LPI* and LSN_{20} values was proposed to assess liquefaction severity and damage, based on an extensive database of CPTu results in the pilot site area;
- For larger projects, the CPTu results are fundamental to assess soil stratigraphy, resistance, and susceptibility to liquefaction, providing crucial information for additional field-testing and high-quality sampling.

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Declaration of interest

The authors declare there are no conflicting interests.

Authors’ contributions

Catarina Ramos: conceptualization, data curation, formal analysis, funding acquisition, investigation, methodology, visualization, writing – original draft. António Viana da Fonseca: conceptualization, funding acquisition, methodology, project administration, supervision, validation, writing – review & editing. Cristiana Ferreira: conceptualization, investigation, methodology, visualization, validation, writing – review & editing.

List of symbols

ε_v	volumetric densification strain
σ_{vo}	initial vertical total stress
σ'_{vo}	initial vertical effective stress
σ'_v	vertical effective stress
$\Phi^{-1}(P_L)$	inverse cumulative normal distribution function

ΔI_c I_c change in a given thickness
a_{max} peak ground acceleration
B_q pore pressure parameter ratio
C_{FC} fines content fitting parameter
CPTu piezocone penetration test
C_q, C_N overburden correction factors
CRR Cyclic resistance ratio
CSR cyclic stress ratio
FC fines content
FS_{liq} Factor of safety against liquefaction
F_r normalized friction ratio
f_s sleeve friction stress
g acceleration of gravity
I_c soil behaviour type index
K_c soil type correction factor
K_σ overburden stress adjustment factor
LPI Liquefaction Potential Index
LSN Liquefaction Severity Number
LSN_{20} Liquefaction Severity Number calculated for the first 20 m above ground surface
MSF Magnitude Scaling Factor
M_w moment magnitude
n, c, m stress exponents
p_a atmospheric pressure
PI plasticity index
P_L probability of liquefaction
q_c cone resistance
q_{c1Ncs} normalized cone resistance for clean sand (B&I2014 method)
Q_{tn} normalized cone resistance (R2009 method)
$Q_{tn,cs}$ normalized cone resistance for clean sand(R2009 method)
r_d shear stress reduction coefficient
R_f friction ratio
SA seismic action
SBTn normalized soil behaviour type
SPT Standard Penetration test
u pore pressure
z depth

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