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ESTIMATED VERSUS MEASURED V_S PROFILES AND V_{S30} AT A PILOT SITE IN THE LOWER TAGUS VALLEY, PORTUGAL

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ABSTRACT

A pilot site in the Lower Tagus Valley near Lisbon, in Portugal has recently been set-up for liquefaction susceptibility analysis and microzonation, within the scope of the European H2020 LIQUEFACT research project. A comprehensive geological, geophysical and geotechnical site investigation campaign has been carefully carried out, including SPT, CPTu, SDMT and geophysical methods, as well as high-quality soil sampling for laboratory characterization. The geophysical tests comprised seismic refraction, SASW and borehole seismic tests (SDMT), among others. For the definition of the V_S profiles and respective V_{S30} values, two approaches were followed: 1) predictions from geotechnical tests; 2) direct V_S measurements. Considering the detailed geotechnical field data obtained in the SPT, CPT and DMT tests, V_S profiles and V_{S30} have been estimated, based on different proposals available in the literature. The direct measurements of V_S extracted from SDMT, as well as from seismic refraction and SASW geophysical tests, were subsequently integrated for comparison with the estimated values. This paper will focus on the comparative analysis of the estimated *versus* measured results, discussing on the reliability and adequacy of V_S estimates based on the existing proposals, considering also the type of test (SPT, CPT or DMT) used in the predictions.

Keywords: Shear wave velocity; V_{S30}; Geotechnical characterization; Liquefaction

1. INTRODUCTION

The characterization of the shear wave velocity and hence of the small-strain stiffness of soils and rocks is fundamental in geotechnical and earthquake engineering design, namely for site response analysis, site classification and soil-structure interaction. The determination of shear wave velocities (V_S) can be done directly through borehole geophysical tests, such as cross-hole (CH) and down-hole (DH) tests, seismic penetration tests, namely the seismic cone penetrometer (SCPT) or the seismic dilatometer (SDMT) or superficial geophysics, such as seismic refraction (SR) and spectral analysis of surface waves (SASW), among others. In the absence of direct measurements of V_S, it is possible to estimate V_S in the field from a range of proposals, mainly based on SPT, CPT and DMT test results. Despite the distinct strain level associated with V_S and penetration tests, typically at large strains for SPT and CPT and at medium strains for DMT, both measurements are primarily dependent on void ratio, stress state and stress history, and therefore this common association can be applied to establish correlations between them (Mayne and Rix 1993; Wair et al. 2012).

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1.1 Estimate of V_S profiles from in situ penetration tests

For the estimate of V_S from penetration field results, Wair et al. (2012) combined, reviewed and discussed the numerous available proposals in the literature, according to soil type and penetration test method. For SPT data, the authors suggested the use of the proposals summarized in Table 1 for different Quaternary soils. The correlation proposed for all soils was derived from Ohta & Goto (1978). For clays and silts, the geologic age (Holocene, Pleistocene and Quaternary) and type of deposition were considered, by introducing age scaling factors (ASF) and focusing on the proposals of Ohta and Goto (1978) and Lee (1992). For sands, the representative equation was developed also taking into account the geologic age, by approximating Ohta and Goto (1978) and Piratheepan (2002) equations.

Table 1: Proposals for SPT–stress–V_S correlation for different soil types (Wair et al., 2012)

Soil type	**V_S estimates for Quaternary soils (m/s)**	**Age Scaling Factors**	
		Holocene	**Pleistocene**
All soils	$30\ N_{60}^{0.215}\ \sigma'_v{}^{0.275}$	0.87	1.13
Clays and silts	$26\ N_{60}^{0.17}\ \sigma'_v{}^{0.32}$	0.88	1.12
Sands	$30\ N_{60}^{0.23}\ \sigma'_v{}^{0.23}$	0.90	1.17

For the estimate of V_S from CPT test results, Wair et al (2012) analyzed a wide range of available proposals, according to soil type and geologic age. From their analysis, these authors suggested the calculation of V_S from the average of the values obtained using Mayne (2006), Andrus et al. (2007) and Robertson (2009) equations for all soils, as presented in Table 2. These correlations make use of different parameters measured in the CPT test, therefore taking into account not only soil strength, but also soil type in the form of the soil behavior type index, I_C. Wood et al. (2017) also considered these proposals as most representative of the CPT-V_S correlations based on global datasets. During this work, it was found that the proposal of Hegazy & Mayne (1995) was more appropriate than Mayne (2006) for very low values of f_s.

Table 2: Proposals for CPT–V_S correlation for all soils (Wair et al., 2012)

Proposal	**Geologic age**	**V_S estimates for all soils (m/s)**
Hegazy & Mayne (1995)	Quaternary	$10.1\ \log\ (q_c)\text{-}11.4)^{1.67}\ (100\ f_s/q_c)^{0.3}$
Mayne (2006)	Quaternary	$118.8\ \log(f_s)+18.5$
Andrus et al. (2007)	Holocene	$2.27\ q_t^{0.412}\ I_C^{0.989}\ z^{0.033}$
Robertson (2009)	Quaternary	$[10^{(0.55Ic+1.68)}(q_t\text{-}\sigma'_{v0})/p_a]^{0.5}$

As shown in Table 2, the correlation equations were generally developed for Holocene or Quaternary soils, which may slightly underestimate V_S for Pleistocene soils. However, Robertson (2009) found these to be generally valid for all Quaternary soils.

For the estimate of V_S from DMT results, Marchetti et al. (2008) constructed a diagram (Figure 1) and interpolated a correlation (Table 3), based on the seismic dilatometer results obtained at thirty-four different research sites, in a variety of soil types. Therefore the experimental diagram presented in Figure 1 and the equations shown in Table 3 can provide estimates of the small strain shear modulus G_0 (hence V_S) from the DMT parameters I_D (material index), K_D (horizontal stress index) and M_{DMT} (constrained modulus) calculated with the usual DMT interpretation formulae (Marchetti et al. 2001). As it clearly appears from Figure 1 and Table 3, the ratio G_0 / M_{DMT} varies in a wide range ($\approx$ 0.5 to 20 for all soils) and it is strongly dependent on (at least) both soil type and stress history, by means of I_D and K_D respectively. In this respect, Amoroso (2014) presented a comparison between CPT and DMT-predicted and measured V_S profiles at six research sites, showing that DMT-V_S predictions are more consistent than CPT predictions. This conclusion was justified by the availability on DMT of K_D, which is noticeably reactive to stress history, prestraining/aging and structure (Marchetti et al. 2001), while scarcely felt by q_c from CPT. In addition, the DMT-V_S predictions are univocal while the CPT-V_S predictions are subjected to the additional uncertainties related for example to geologic age and effective stress state.

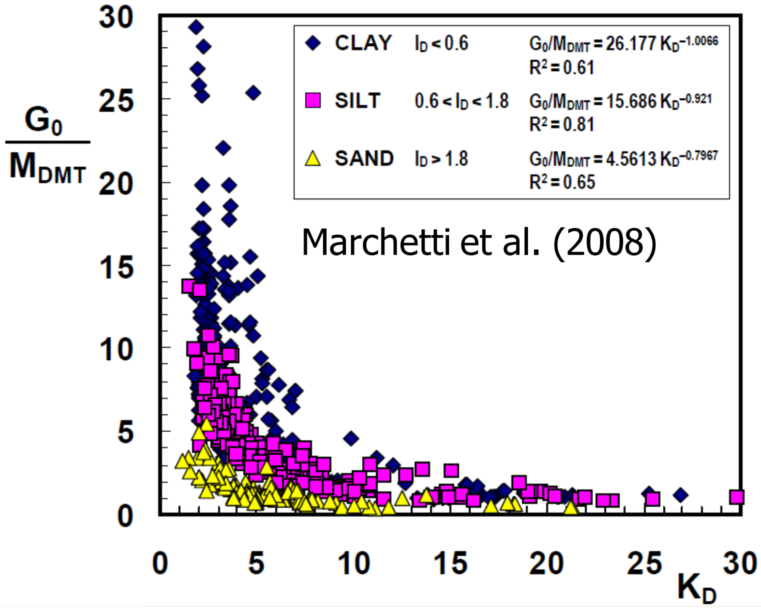

Figure 1. Ratio G_0 / M_{DMT} vs. K_D for various soil types (Marchetti et al., 2008)

Table 3: Proposals for DMT–V_S correlation for all soils (Marchetti et al. 2008)

Proposal	Material Index	Interpolated correlation
	$I_D < 0.6$	$G_0/M_{DMT} = 26.177 \cdot K_D^{-1.0066}$
Marchetti et al. (2008)	$0.6 < I_D < 1.8$	$G_0/M_{DMT} = 15.686 \cdot K_D^{-0.921}$
	$I_D > 1.8$	$G_0/M_{DMT} = 4.5613 \cdot K_D^{-0.7967}$

1.2 Definition of V_{S30}

In order to classify a soil profile for characterizing its effects on site response and ground motion, several authors have suggested the use of V_{S30}, the time-averaged shear wave velocity at the first 30 meters. V_{S30} can be computed from the recorded shear wave velocities down to such depth, by considering the harmonic average of all values, distinct from the arithmetic average of V_S. As shown in Equation 1, V_{S30} is calculated dividing the depth of 30 m by the sum of the travel times for shear waves travelling through each soil layer (Wair et al., 2012).

$$V_{S30} = \frac{30}{\sum \frac{d}{V_S}} \tag{1}$$

According to Eurocode 8, there are five typical ground types (A, B, C, D, E) and 2 special ground types (S_1, S_2) that may be used to account for the influence of local ground conditions on the seismic action, based on V_{S30} values. Alternatively, if the value of V_{S30} is not available, N_{SPT} data or the undrained shear strength C_u can be used. Figure 2 presents the description of each ground type and its defining parameters.

Also based on V_{S30}, the Caltrans Seismic Design Criteria classifies sites into six categories (Soil

Profile Types A through F), as presented in Figure 3. The Caltrans site classes are different from Eurocode 8, but consistent with those used by other American codes and standards, including the National Earthquake Hazard Reduction Program (BSSC, 2003) and American Society of Civil Engineers (ASCE 2006, 2010).

Ground type and description	V_{S30} **(m/s)**	N_{SPT}	C_u **(kPa)**
A: Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	-	-
B: Deposits of very dense sand, gravel or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C: Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180 – 360	15 – 50	70 - 250
D: Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E: Soil profile consisting of a surface alluvium layer with V_S of type C or D with thickness of about 5 m and 20 m, underlain by stiffer material with V_S > 800 m/s.	-	-	-
$\mathbf{S_1}$: Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100	-	10 - 20
$\mathbf{S_2}$: Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1	-	-	-

Figure 2. Classification of soil profile types according to Caltrans/NEHRP (Caltrans, 2006)

Site class	Soil profile type	V_{S30} (m/s)	N_{SPT}	C_u (kPa)
A	Hard rock	>1500		
B	Rock	760 a 1500	-	-
C	Very dense soil and soft rock	360 a 760	> 50	> 100
D	Stiff soil	180 a 360	15 – 50	50 - 100
E	Soft soil [1]	<180	< 15	< 50
F	Soils requiring site-specific evaluation [2]	---		

[1]Site Class E includes any profile with more than 3 m of soft clay, defined as soil with PI > 20, w > 40% and C_u < 25 kPa;
[2]Site Class F includes: (1) Soils vulnerable to failure or collapse under seismic loading (liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils). (2) Peat and/or highly organic clay layers more than 3 m thick. (3) Very high plasticity clay (PI > 75) layers more than 8 m thick. (4) Soft to medium clay layers more than 36 m thick.

Figure 3. Classification of soil profile types according to Eurocode 8 (CEN, 2010)

If the shear wave velocities (either measured or predicted from other test data) do not extend to the predefined depth of 30 m, extrapolation may be relevant for a more accurate estimate of V_{S30}. Boore (2004) carried out statistical analyses of borehole data extended at least to 30 m and proposed an extrapolation method for V_{S30}. This method considers the shear wave velocities measured to a terminal depth *d* and computes the time-average V_{Sd}, analogous to V_{S30}, which is then correlated with V_{S30} by means of Equation 2:

$$\log V_{S30} = a + b \cdot \log V_{Sd} \tag{2}$$

where *a* and *b* are regression coefficients, provided in Boore (2004) and Wair et al (2012) for depths ranging from 10 to 29 m. Naturally, correlation becomes stronger as the last depth of measurement approaches 30 m. Extrapolating shallow velocity data to calculate V_{S30} may be appropriate for most sites, where relatively uniform soil conditions are expected. However, this method could lead to errors, for sites with a clear velocity contrast within the first 30 meters.

2. PILOT SITE CHARACTERIZATION

2.1 Seismological and geological setting

The seismicity of Portugal is heterogeneous and distinct seismic behavior is considered for different regions, as prescribed in the Portuguese National Annex of Eurocode 8 (EC8-NA) (CEN, 2010). Seismicity increases in intensity from North to South and it is concentrated in the South and the Atlantic margins. Based on the available records, most earthquake epicenters are located around the cities of Lisbon and Évora, in the Lower Tagus River Valley (LTV) region, and along the Algarve coast, due to the proximity to the boundary between the Eurasian and African plates (Ferrão et al. 2016). The greater Lisbon area is considered to have higher seismic risk, since it is affected by the occurrence of not only large magnitude (M_w>8) offshore earthquakes, but also moderate (M_w~6) onshore earthquakes (Vilanova and Fonseca, 2007). The Lower Tagus River Valley (LTV), located in the densely populated and developed region of Lisbon, has been disturbed by severe earthquakes causing serious damage and many casualties (Cabral et al., 2011). This seismicity comprises distant events, as the 1755 earthquake, generated in the Eurasia-Nubia plate boundary zone, and M=6–7 local intraplate earthquakes, as in 1344, 1531, and 1909. Geologically, this area comprises up to about 2000 m of Tertiary (Paleocene to Pliocene) sediments, Pleistocene fluvial terraces, and a thick (up to 70 m) Upper Pleistocene to Holocene alluvial cover (Cabral et al., 2011). The specific seismological, geological, geomorphological and hydrological setting in this area combines all the requirements for earthquake-induced liquefaction triggering (Viana da Fonseca et al., 2018). A liquefaction potential zonation map of Continental Portugal, presented by Jorge (1993) and further developed by Jorge and Vieira (1997), also evidences the high liquefaction potential in this region.

Based on these findings and after the collection of pre-existing geotechnical information in the metropolitan region of Lisbon, a pilot site on liquefiable soils was set up in Lezíria Grande de Vila Franca de Xira, near Benavente, within the scope of the European H2020 LIQUEFACT research project (LIQUEFACT, 2017, Saldanha, 2017, Viana da Fonseca et al., 2018).

2.2 Geological, geotechnical and geophysical site characterization

The area of interest for the pilot site on liquefiable soils includes an important recently built infrastructure, the A10 highway bridge. The number and location of the site investigation points are identified in
Figure , which were selected considering the geological interpretation of the site, coupled with the information provided by the liquefaction zonation map by Jorge (1993). In this campaign, a large amount of geotechnical investigations was considered and performed, including boreholes, standard penetration tests, cone penetration tests, seismic dilatometer and geophysical tests. In addition, a seismic microzonation is being carried out, for which a series of noise measurements along the A10 Bridge and two other alignments have also been performed, to collect additional information on the existing impedance contrast and to calibrate the results of other noise measurements in the Tagus valley.

A summary of the geological, geotechnical and geophysical site characterisation is provided in Table 4. Description of these test procedures and details of the obtained results can be found in Saldanha (2017) and Viana da Fonseca et al. (2017, 2018).

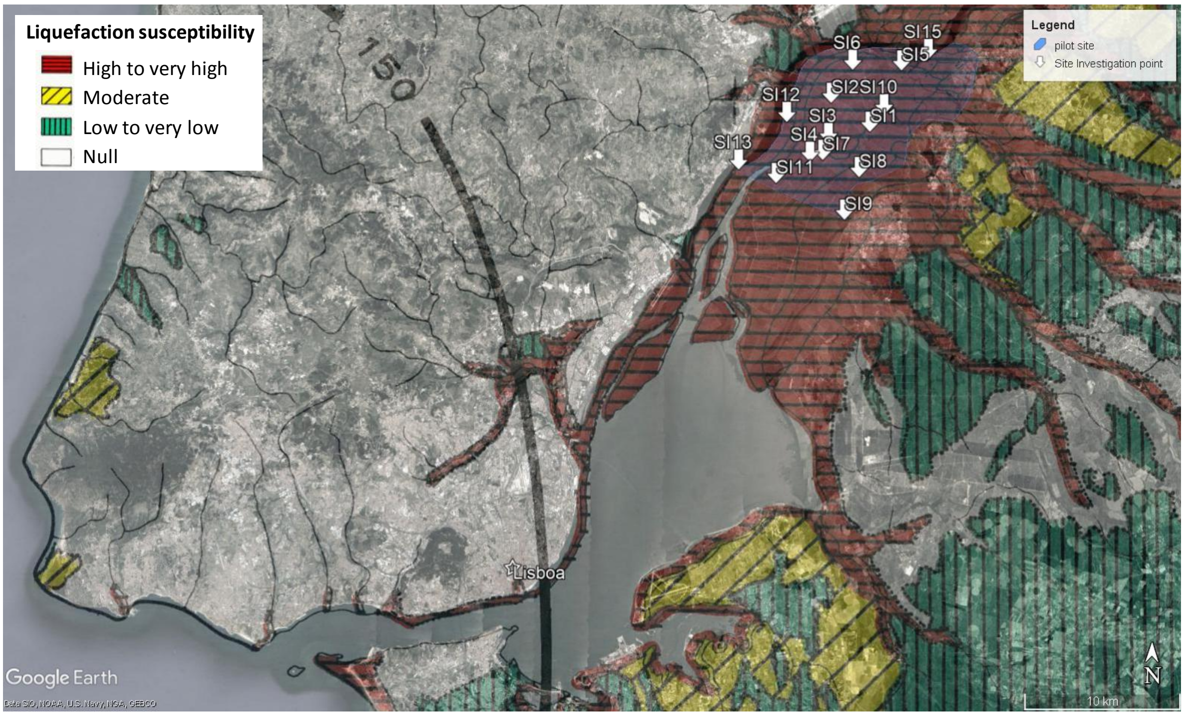

Figure 4. Location of the pilot site, superimposed on the liquefaction zonation map (from Jorge, 1993)

Table 4: Geotechnical and geophysical site characterisation performed at the pilot site

Type and number of tests		Site investigation points
Geotechnical	2 SPT	SI1; SI7
	8 CPTu	SI1, SI2, SI3, SI4, SI5, SI6, SI7, SI10
	3 SDMT	SI7, SI8, SI9
Geophysical	1 SASW	SI5
	8 Seismic Refraction (SR)	SI1, SI5, SI6, SI7, SI9, SI11, SI12 e SI13

The analysis of the SPT test results was implemented on a spreadsheet, based on Idriss & Boulanger (2010) and Boulanger & Idriss (2014). The interpretation of CPT results was made, based on the proposals of Robertson (2009) and Boulanger & Idriss (2014). For this task, a specific software package, CLiq® version v.2.0.6.92, was used, which was specifically developed for CPT liquefaction potential analysis (GeoLogismiki, 2017).

For the purpose of this paper, the analysis of the in situ characterisation results will focus on the comparison of seismic wave velocity measurements from geophysical tests and its predictions from geotechnical tests, namely SPT, CPTu and DMT.

3. ESTIMATED VERSUS MEASURED V_S PROFILES AND V_{S30}

3.1 Comparison of V_S profiles at different testing locations

The results of the geotechnical tests in this site investigation campaign are available elsewhere (Viana da Fonseca et al. 2017, 2018). Based on the V_S prediction proposals previously presented, the estimated and measured V_S profiles have been produced for each site investigation point, which are illustrated in Figures 5 to 7. In the two locations where SPT tests were conducted, namely SI1 and SI7, simplified soil profiles have been produced, as illustrated in

Figure .

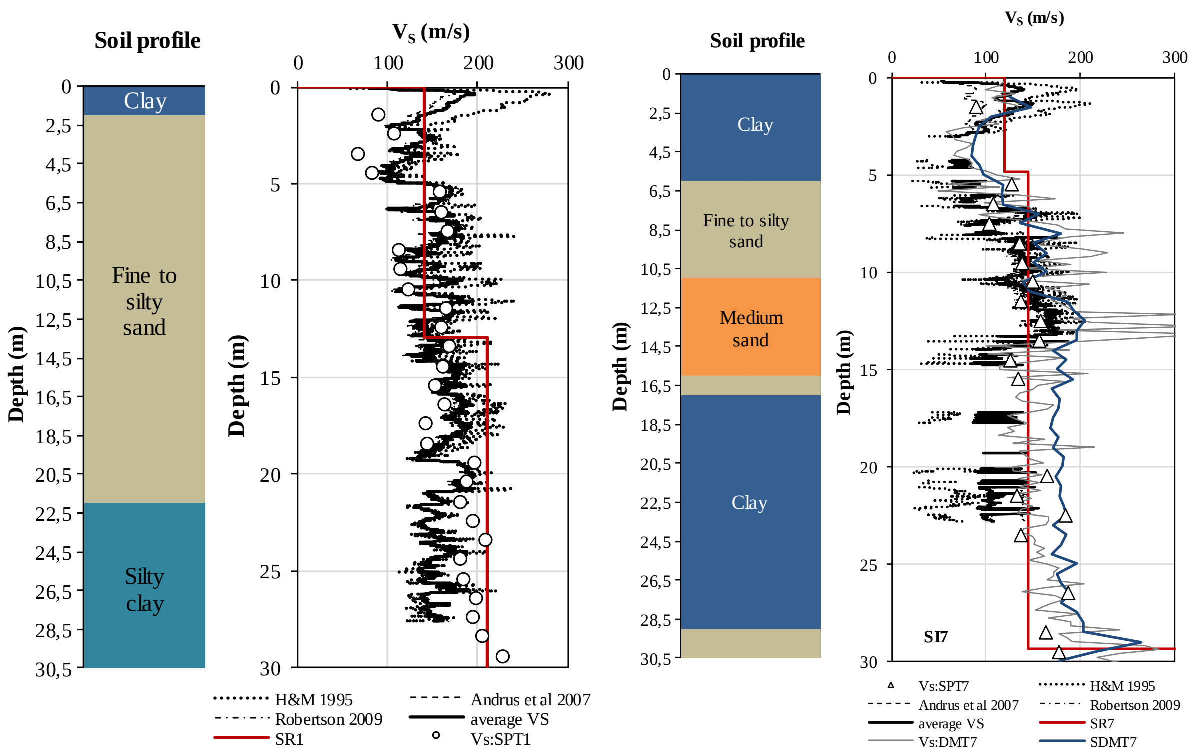

Figure 5. Estimated and measured V_S at SI1 (via SPT, CPTu, SR) and SI7 (via SPT, CPT, SR, SDMT)

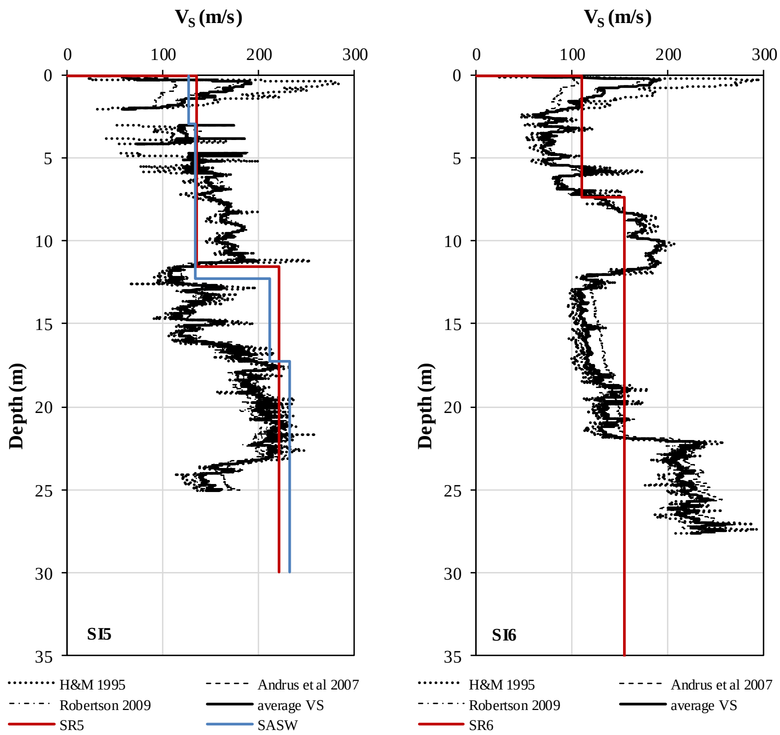

Figure 6. Estimated and measured V_S at SI5 (via CPT, SR, SASW) and SI6 (via CPT, SR)

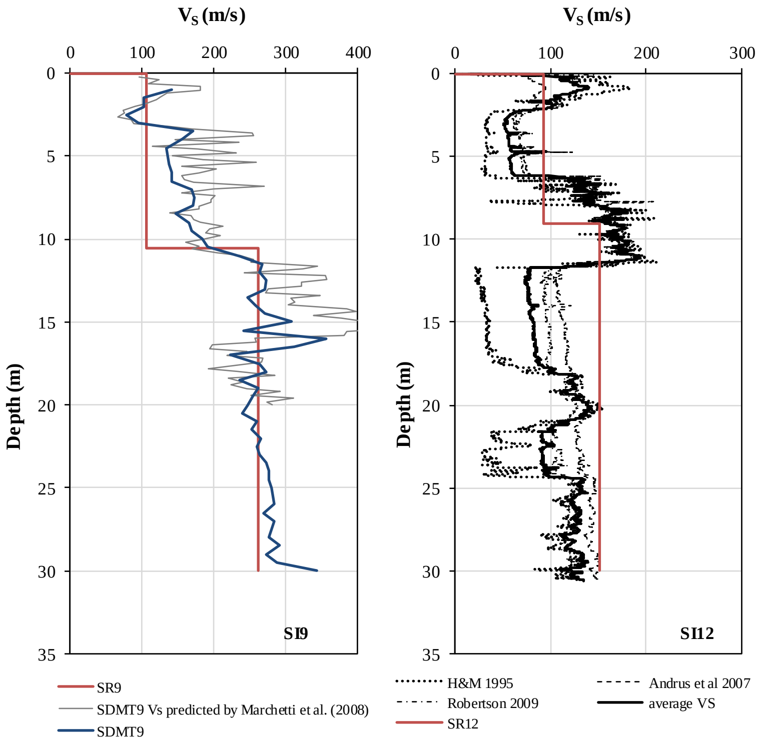

Figure 7. Estimated and measured Vs at SI9 (via DMT, SDMT, SR) and at SI12 (via CPT, SR)

From these figures, the comparison between V_S predictions and measurements generally shows a good agreement. In some cases, namely in SI9, the overlap of the profiles is substantial. Despite the clear variability and heterogeneity in depth of the V_S profiles, the values of shear wave velocity are generally below 300 m/s, except for SI9. As expected, the use of surficial wave methods produce simplified V_S profiles, which are in good agreement with the other V_S profiles only when there is an increase in stiffness with depth, as for example in SI9. On the contrary, when the stiffness decreases with depth or exhibits an erratic profile, for example in SI5, SI6 and SI7, the seismic refraction or SASW results are indicative of an average value of V_S. It can be concluded that surface geophysical methods provide good indications regarding the variation of stiffness in depth; however, complementary information, even if obtained through correlations with other in situ testing methods, may be of great use to validate and confirm those results.

In terms of the V_S predictions based on SPT, obtained in SI1 and SI7, the results are very similar in depth to those of other methods. In particular, the similarity between the V_S-SPT and V_S-SDMT profiles in SI7 is evident. For the case of the different available correlations for CPT predictions of V_S, a good agreement was found. Robertson (2009) proposal provided very similar results to the average of the three proposals; hence, it is considered the most appropriate for these soils. As for the V_S profiles estimated from DMT and measured in SDMT obtained in SI7 and SI9, an excellent agreement was observed, which confirms the good consistency and reliability of Marchetti et al. (2008) proposal for the prediction of V_S.

3.2 Comparison of V_{S30}

Following the methodology for determination of V_{S30} previously presented, the V_{S30} values have been computed from the estimated and measured V_S profiles in each site investigation point. Since the geotechnical and geophysical tests reached different depths, in order to measure V_{S30} the last-depth parameters have been extended to 30 m depth, when the test depth was smaller than 30 m. For the purpose of comparison, these values have been compiled in
Table and in
Figure .

Table 5: V_{S30} from estimated (via SPT, CPT and DMT) and measured (via SDMT, RS and SASW) V_S profiles

	Estimated values			**Measured values**		
Testing location	**SPT**	**CPTu[#]**	**DMT**	**SDMT**	**SR**	**SASW**
SI1	145.08	149.41			172.71	
SI2		125.63				
SI3		124.74				
SI4		134.40				
SI5		155.05			177.76	175.23
SI6		118.34			140.20	
SI7	136.34	120.17	138.40	154.54	142.05	
SI8			165.16	168.59		
SI9			212.26	201.43	172.79	
SI10		145.13				
SI11					122.30	
SI12		97.90			127.14	
SI13					128.54	

[#] extrapolated for 30 m, for the cases of maximum depth below 30m according to Boore (2004)

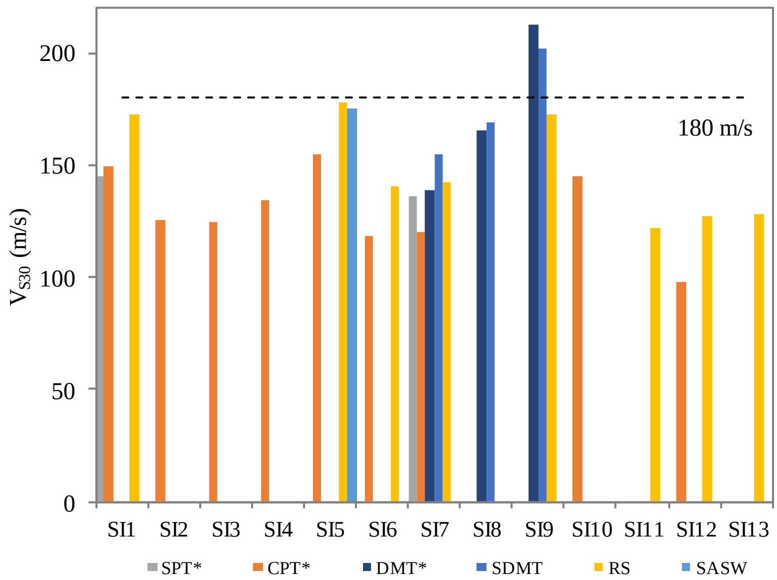

Figure 8. Comparison of estimated (*) and measured V_{S30} values at the site investigation points

From these results, it can be concluded that the soils in all site investigation points were characterized as soft soil (V_{S30} <180 m/s), except for SI9, where V_{S30} is slightly above that threshold from one measured V_S profile. Despite some variability, all the results from the various tests in all testing locations belong to the same classification class. This classification as soft soil corresponds to Ground type D, according to Eurocode 8, and to soil type E, according to the Caltrans Seismic Design Criteria (2004). In SI9, the average V_{S30} corresponds to 187 m/s, that is, in the borderline between soft and dense soil, from which the classification as soft soil is considered acceptable and conservative.

In terms of the estimated versus measured results, at SI7, it was possible to obtain four different but similar V_{S30} values, with a mean value of 138.28 m/s and a dispersion of less than 13%. In general, V_{S30} values obtained from V_S predictions correspond to the lower bound of the results, from which it can be argued that these predictions yield conservative values of shear wave velocity.

On a side note, the classification of all testing points of this pilot site as soft soils confirms the appropriate choice of this zone as a pilot site for assessment of liquefaction susceptibility.

4. CONCLUSIONS

This paper addresses the comparison between estimated and measured shear wave velocities in a pilot site on liquefiable soils in the Lower Tagus Valley near Lisbon, in Portugal. An extensive geophysical and geotechnical site investigation including SPT, CPTu, SDMT and geophysical methods, provided a good database of results for such comparisons to be made. Shear wave velocity profiles were produced from predictions from SPT, CPT and DMT tests and direct V_S measurements from seismic refraction, SASW and SDMT.

In general, the agreement was considerable and, in some cases, coincident. The correlations from SPT and CPT were found to be reliable and consistent, while yielding in general lower values than those directly measured. The correlation based on DMT results provided very similar V_S profiles to those of the corresponding SDMT, thus evidencing the quality and reliability of this prediction proposal. In addition, the estimates of V_{S30} based on those profiles were also comparable and consistent, suggesting that the use of adequate correlations for V_S predictions from in situ penetration tests can be effectively made.

5. ACKNOWLEDGEMENTS

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