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ASSESSMENT OF SEISMIC SITE RESPONSE BASED ON MICROTREMOR MEASUREMENTS

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ABSTRACT

Microtremor measurements is a cost-effective and non-invasive technique based on the ambient vibrations recordings of three components at ground surface. It is used to estimate the fundamental frequency of soils, f_0, and its amplification ratio, A_0, based on the spectral ratio between the horizontal (H) and vertical (V) components of the measurements.

In the scope of the H2020 EU funded, LIQUEFACT project, which addresses the mitigation of the risks associated with the liquefaction induced due to the seismic action, in situ geotechnical tests were performed, including microtremor measurements, in the Lisbon area in Portugal.

Each measurement had an approximate duration of 40 minutes at 26 different sites, using a SYSCOM velocity sensor (MS2003+) connected to an SYSCOM acquisition unit (MR2002), considering an acquisition frequency of 400 Hz. The H/V curves at some points exhibit clear single peaks with large amplitude, which could be associated to sharp discontinuities corresponding to a profile with a single fairly homogeneous layer with a low value of the shear wave velocity contrasting a much higher value at a certain depth ("seismic bedrock"). The studied areas are characterized by peak frequencies ranging from 0.92 to 11.01 Hz and peak amplitudes ranging from 2.58 to 4.73. The linear equivalent model was used to assess seismic site effects, using Cross-Hole data to build the soil profile, along with strain-dependent curves from resonant column and cyclic torsional tests.

The peak horizontal acceleration computed through numerical simulation was then compared with the frequency, the amplitude and the shape of HVSR curves to assess HVSR curves reliability in the prediction of seismic site effects.

Keywords: Seismic site response; Nakamura method; Fundamental frequency;

1 INTRODUCTION

Over the past decades, earthquakes caused large human and economic losses. These losses depend not only on the type of seismic action (intensity, epicentre distance, duration), but also depend of local characteristics such as type of construction and geological formations (Panzera et al. 2017). An increasing interest in the study of local ground conditions, regarding the estimation and evaluation of site effects, in order to predict and mitigate these effects.

The local seismic site response depends on several physical phenomena (reflections and wave diffractions, resonance effects, non-linear behaviour of the soil) which may imply an amplification or attenuation of the seismic waves recorded near the surface (Panzera et al. 2017). The local amplification ratio has a direct influence on the behaviour registered at the surface, and to do so, it is important to develop a detailed characterization of the soil behaviour (Poggi et al. 2017).

Nakamura method (or HVSR) has been widely used due to its low cost, non-invasive nature and simplicity. This method uses measurements of ambient vibrations at surface to estimate the fundamental frequency, f_0, and the local amplification ratio, A_0, of soil deposits (Nakamura, 2008). These parameters are obtained through the spectral ratio between the horizontal and vertical components (H/V).

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While the estimate of f_0 is relatively accurate, in most cases the value of the amplification factor, A_0, is not reliable. Another disadvantage is the inadequacy of Nakamura method to characterize site effects under complex conditions (Ghofrani and Atkinson, 2014).

In this paper the results of ambient vibration measurements obtained in the pilot site selected in Portugal for the tasks of LIQUEFACT project (www.liquefact.eu) are presented and the assessment of the adequacy of Nakamura technique for the purpose of identifying seismic response is discussed. The Linear Equivalent method is used to compare the local site effects with Nakamura method. This paper shows the first results obtained.

2 EXPERIMENTAL CAMPAIGN

The ambient vibration was recorded at Lezíria Grande area, in the municipality of Vila Franca de Xira, Portugal. This area was selected within the scope of the H2020 EU LIQUEFACT project, due to its high susceptibility to liquefaction (Saldanha et al., 2017).

2.1 The study area

The geotechnical characterization of the zone was carried out based on the several in situ tests carried out in the construction of the A10 Highway, namely SPT (S) tests, Cross-hole (CS) profiles, CPTu (Cpt) tests, of which 2 are SCPTu, whose location is presented in Figure 1.

Figure 1. Location of the several tests along A10 Highway (adapted from LIQUEFACT, 2017)

The geological map reveals the existence of large Holocene deposits of sandy sediments on the south bank of the Tagus River, covering the municipality of Vila Franca de Xira. This basin is composed of tertiary sediments that can reach a depth of 2000 m and, the deepest sediments consist of a layer with 200 to 400 m of thickness of continental sediments of the Palaeocene, on which was found a layer of continental and marine sediments of the Miocene that can reach 800 m of thickness in some places. Figure 2 shows the geological profile along A10 Highway, including several Vs profiles from CH tests (adopted from Vis et al., 2016).

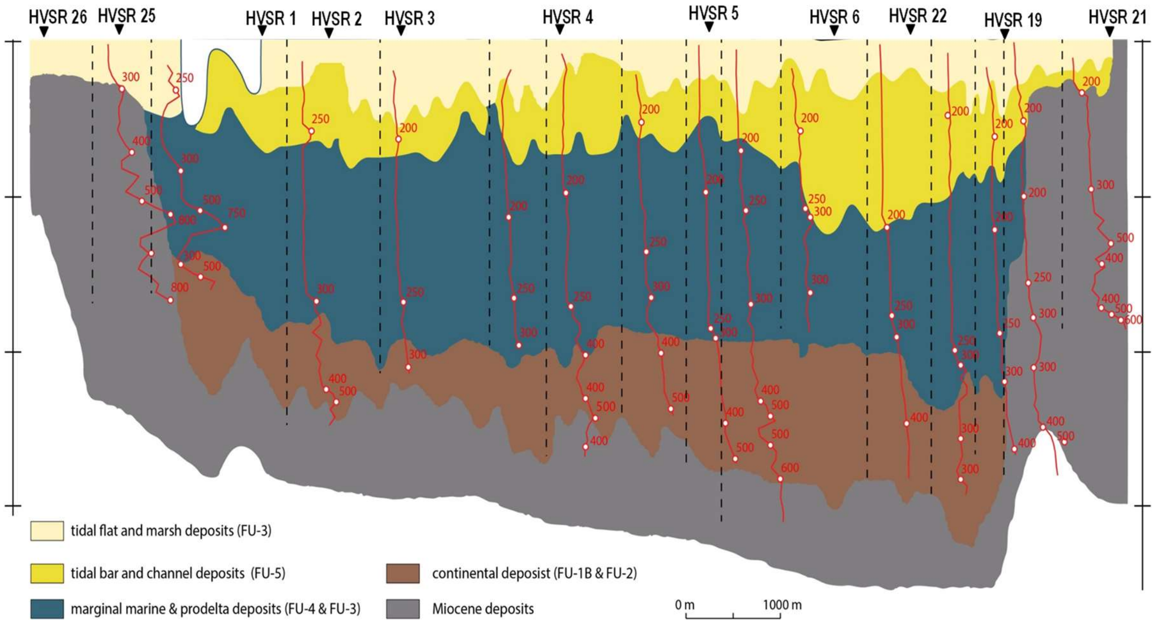

Figure 2. Geological profile of the A10 Highway (adapted from LIQUEFACT, 2017)

From Figure 2 , the A10 Highway geological profile can be defined as:

- Surface layer with a thickness between 5 to 6 meters (FU-3), formed by muddy clays, silty clay and clays, intercalated with silts and fine sand, and coarser sands at surface, with $200 \leq V_s \leq 300$ m/s;
- Sediment layer of fine, medium to coarse sands intercalated with silts and clays (FU-5), with an average thickness of 15 meters, with $150 \leq V_s \leq 250$ m/s;
- Marine fine grained sediments (FU-4 & FU-3), formed by large volumes of clays and silty clays, with a thickness between 20 and 30 meters and $200 \leq V_s \leq 250$ m/s;
- Alluvial deposits (FU-1B & FU-2) formed by coarse sand, sands and gravels, with a low percentage of fine grains, between 40 and 60 meters, with an average thickness between 12 to 15 meters, and $400 \leq V_s \leq 500$ m/s. These deposits, in the upper zone, between 30 and 40 meters, are formed by materials with a smaller particle size, silts and silty clays, intercalated with clays and fine sands, with an average thickness between 5 to 7 meters and $300 \leq V_s \leq 500$ m/s;
- Miocene deposits, lithified, but not very hardened, with a thickness of approximately 300 meters, with $600 \leq V_s \leq 800$ m/s.

Based on the Vs profiles, it was determined the shear wave velocity of the upper 30 meters, $V_{s,30}$, and the NP EN 1998-1 (2010) ground type classification scheme was applied (Table 1).

Table 1. $V_{S,30}$ and NP EN 1998-1 (2010) ground type for the A10 Highway.

Cross-hole	**S203**	**S11**	**S6**	**S208**	**S16**	**S220**	**S22**	**S25**	**S225**	**S28**	**S32**	**S232**	**S37**	**S304**	**S42**
$V_{s,30}$ (m/s)	292	167	203	167	168	181	181	160	185	172	165	164	159	167	216
Ground type	C	D	C	D	D	C	C	D	C	D	D	C	D	D	C

$V_{s,30}$ varies from 160 m/s (S25) to 292 m/s (S203). According to the classification proposed in NP EN 1998-1 (2010), the ground type is near the limit between ground Type C (180 m/s < $V_{s,30}$ < 360 m/s - Deep deposits of dense or medium-dense sand, gravel or stiff clay) or Ground Type D ($V_{s,30}$ < 180 m/s - Deposits of loose-to-medium cohesion less soil, with or without some soft cohesive layers, or of predominantly soft-to-firm cohesive soil). 7 profiles are classified as ground type C and 8 profiles as type D.

The experimental campaign comprised single station ambient vibration measurements (HVSR) at 11 sites along A10 profile (Figure 3).

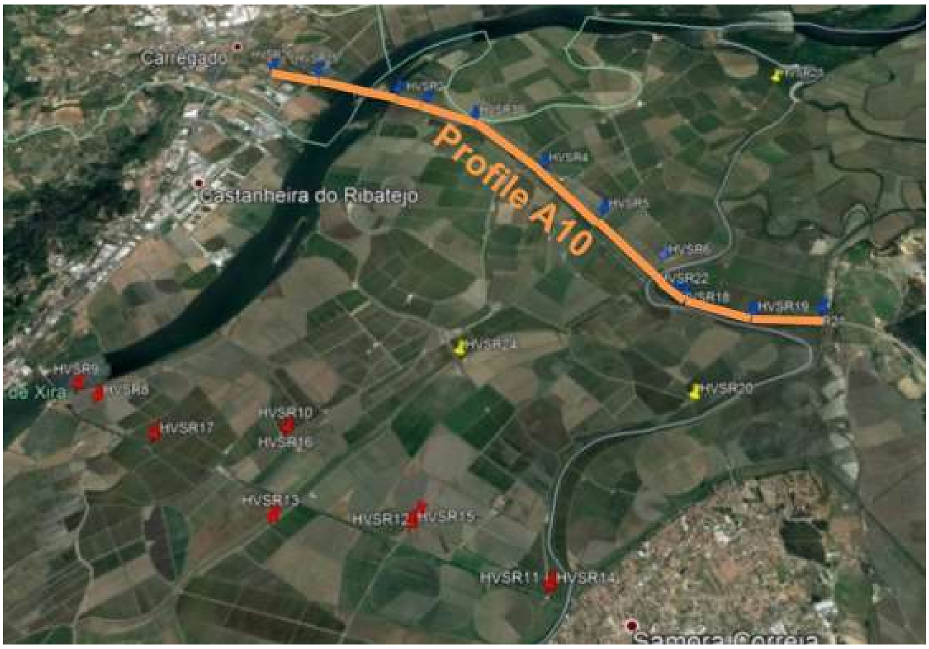

Figure 3. Location of ambient vibration measurements

The measurements were carried out using a three dimensional velocity sensor (Model MS2003+) (Figure 4a)), connected to an acquisition unit (Model MR2002) (Figure 4 b)), which was connected to a portable computer. The recordings had duration of approximately 40 minutes, with an acquisition frequency of 400 Hz.

a) 

b) 

Figure 4. a) Three-dimensional velocity sensor (Model MS2003+); b) Acquisition unit (Model MR2002)

3 HVSR RESULTS

The analysis of the ambient noise recordings was developed using Geopsy software, that is an open source software, developed within the SESAME Project (2004). In

Table 2, the values obtained for HVSR measurements in the A10 profile are presented. Figure 5 a) and Figure 5b) plot the H/V curves obtained in the borders of the A10 Highway (HVSR 19, 21, 25 and 26). Figure 5 c), d) and e) plot the H/V curves obtained in the central zone of the A10 Highway (HVSR 1 to 6 and HVSR 22).

Table 2. Obtained values from the H/V curves for the A10 profile.

Measurement	**HVSR 26**	**HVSR 25**	**HVSR 1**	**HVSR 2**	**HVSR 3**	**HVSR 4**	**HVSR 5**	**HVSR 6**	**HVSR 22**	**HVSR 19**	**HVSR 21**
f_0 (Hz)	1.98	1.14	1.14	0.90	1.61	1.18	1.14	1.18	1.22	1.06	11.01
A	4.65	3.71	3.88	4.02	2.58	3.51	3.70	4.16	4.73	4.05	3.32

Regarding the obtained values for the Amplification ratio, A_0, the obtained H/V curves show that is possible to identify a single and clear peak (HVSR 1,4,5,6 and 22). The values of A_0, range from 2.58 to 4.73. Only for measurements HVSR 2,6,19,22 and 26 $A_0 > 4$.

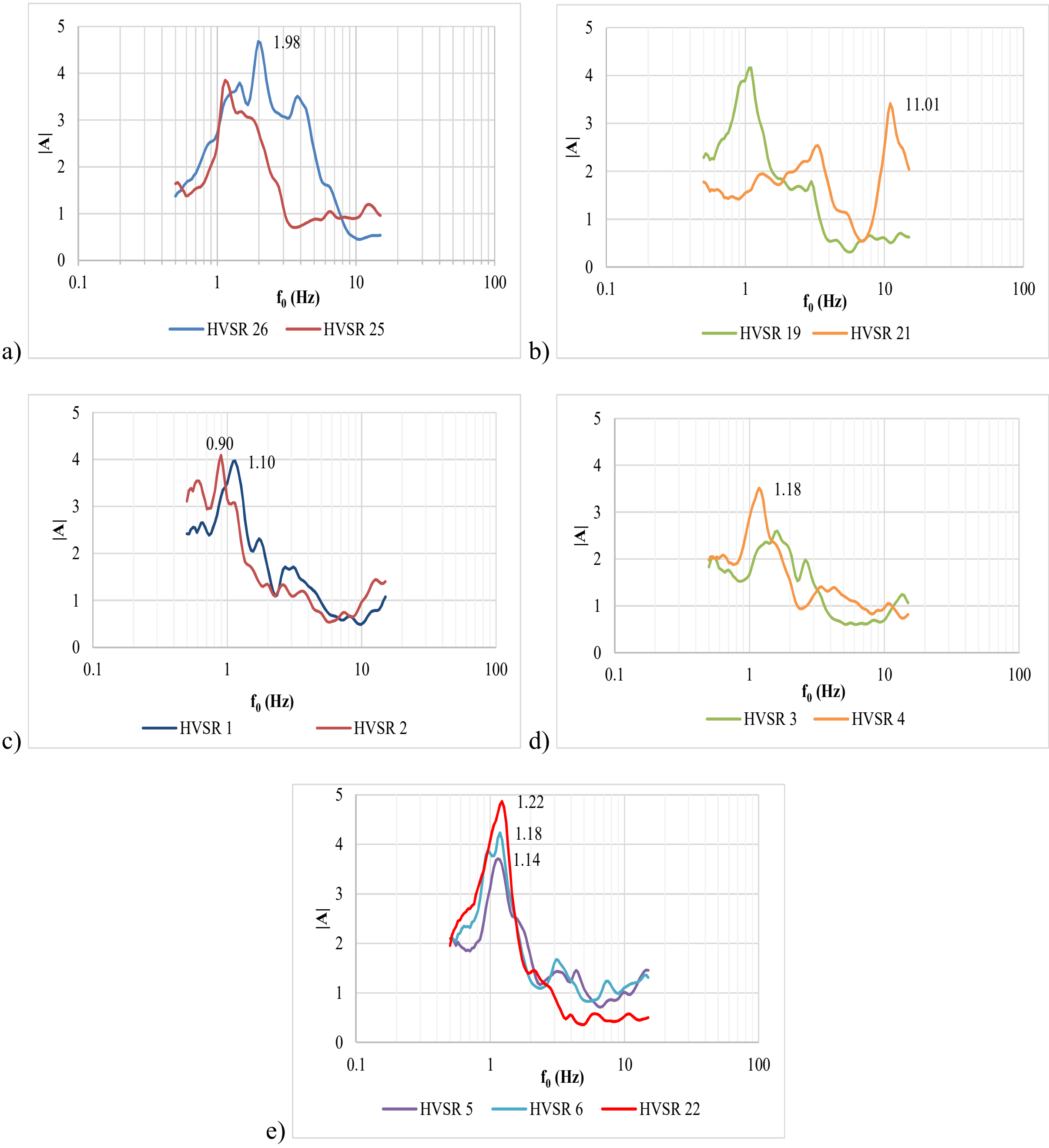

Figure 5. H/V curves for HVSR measurements in A10 Highway profile: edge of the valley
c), d) and e) - central zone

The values of the fundamental frequency, f_0, reveal variability in the stratigraphy all along the profile, existing a clear evidence that this is a river valley zone:

- Edges of the valley with amplification peaks at high frequencies, namely f_0 = 1.98 Hz (HVSR 26) and f_0 = 11.01 Hz (HVSR 21), consistent with the geological profile (Figure 2).
- Central zone of the valley with amplification peaks at low frequencies, f_0 = 0.90 Hz (HVSR 2) until f_0 = 1.22 Hz (HVSR 22);

4 ASSESSMENT OF SEISMIC SITE RESPONSE

The linear equivalent method, using EERA software (Bardet et al. 2000), was used to assess seismic site response for two situations:

- A_1 – Linear elastic analysis, to validate the numerical model against f_0 from Nakamura method;
- A_2 – Linear equivalent analysis, to evaluate the site response for a representative strong motion.

The assessment of the seismic site response was developed in three different locations of the A10 profile, namely L_1, L_2 and L_3 (Figure 6). These places were defined according to the existent geotechnical information, namely SPT and Cross-hole (CH) tests. L_1 corresponds to CH S25, while L_2 corresponds to CH 32 and L_3 corresponds to CH 232

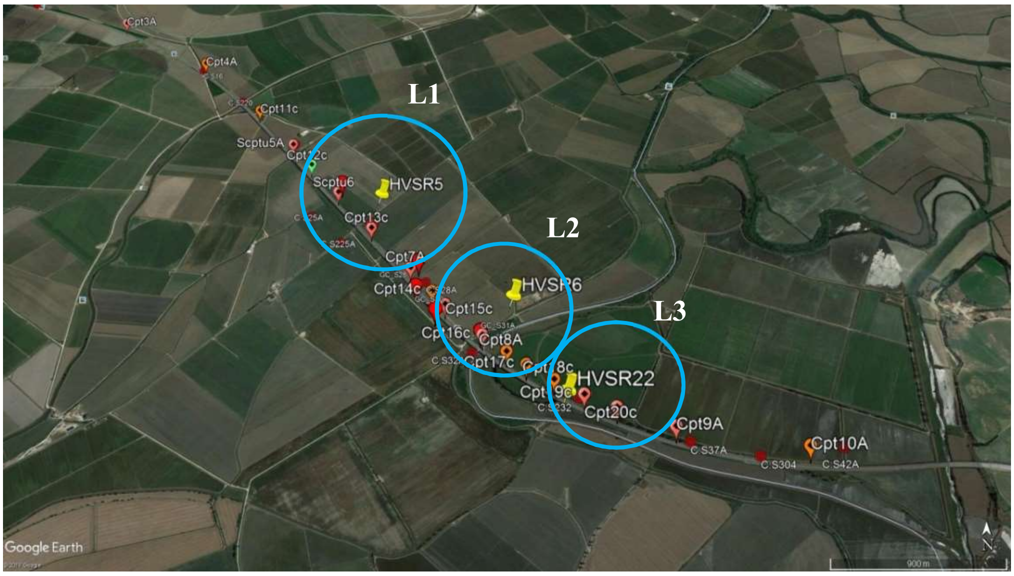

Figure 6. Sites where seismic response was evaluated

The three sites where the seismic site response is assessed are classified as ground type D ($V_{s,30}$ < 180 m/s; see Table 2).

4.1 Seismic actions

The assessment of the seismic site response was developed considering a local seismic action based on strong motion recorded during M_s = 7.60 Izmit earthquake in 1999 (Figure 7) on a very soft soil (ground type D). The record has a peak acceleration a_{max} = 2.55 m/s^2.

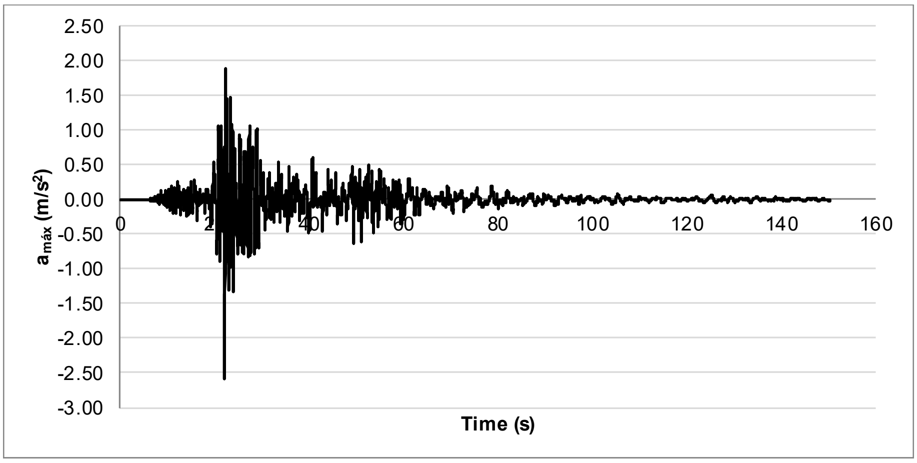

Figure 7. Strong ground motion considered for the local seismic action

4.2 Vulnerability index, Kg

Nakamura defined the vulnerability index, K_g, (equation 1) as an index to estimate the ground strain, γ (equation 2). K_g is obtained from the fundamental frequency, f_0 and the amplification factor A_0 of the microtremor measurement (Nakamura, 2012).

$$K_g = \frac{A^2{}_0}{f_0} \tag{1}$$

$$\gamma = \frac{a}{\pi \times V_b} \times K_g \tag{2}$$

Where:

- K_g – Vulnerability index;
- A_0 – Amplification ratio obtained from the H/V curves;
- f_0 – Fundamental frequency obtained from the H/V curves;
- $a_{máx}$ – Maximum acceleration;
- V_b – Shear wave velocity at the bedrock

Using equation (1), the vulnerability index, K_g, was determined for the three places, while equation (2) was used to estimate the ground strain, γ (Table 3). For a_{max}, was used the peak acceleration value obtained from the linear equivalent analysis presented in 4.4.

Table 3. Vulnerability index, K_g, and estimative of ground strain, γ

	L_1	**L_2**	**L_3**
A_0	3.70	4.16	4.73
f_0 (Hz)	1.14	1.18	1.22
Kg	12.01	14.64	18.29
$a_{máx}$ (m/s²)	3.30	2.95	4.00
V_b (m/s)	800	800	800
Kg	12.01	14.64	18.29
γ	1.57E-02	1.72E-02	2.91E-02

Table 3 shows that as the value of K_g increases proportionally with ground strain. From the analysis of the values of the estimated ground strain, γ, is possible to conclude that the three sites are vulnerable to high shear strains larger than $1x10^{-2}$.

4.3 A_1 – Linear elastic analysis

A linear elastic analysis was done to validate the numerical model against f_0 from Nakamura method. Figure 8 plots the amplification curves from the numerical model and HVSR curves. Because the difference in the value of f_0 from the linear elastic analysis and from Nakamura method is less than 10%, the numerical model is validated.

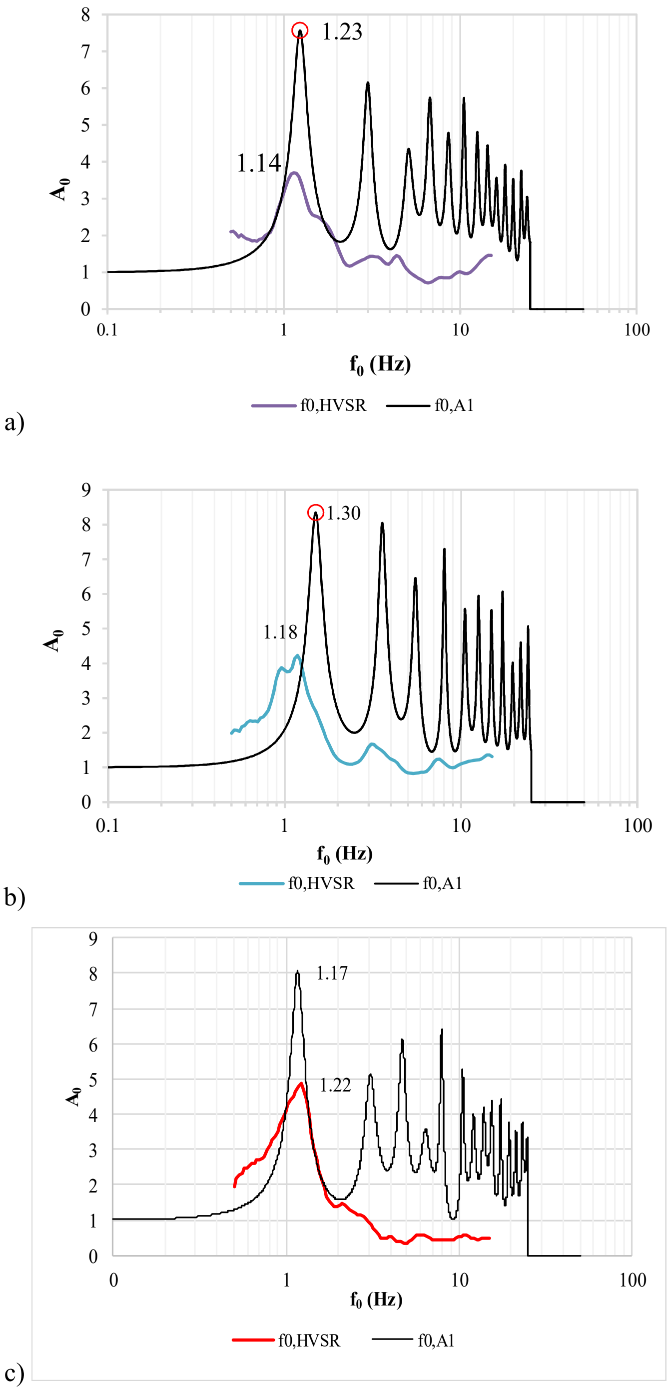

Figure 8. $f_{0,H/V}$ values obtained from the H/V curves for the three sites: a) L_1; b) L_2; c) L_3

4.4 A_2 – Linear equivalent analysis

The linear equivalent analysis was applied to evaluate the seismic site response for the three sites. The peak acceleration at surface ($a_{g,max}$), the maximum shear strain (γ_{max}), and the correspondent amplification ratio computed as the ratio between the peak acceleration at surface and outcrop (AR – see equation 3). In Table 4 are presented the obtained values for the three places, in Figure 9 is presented the variation in depth of the maximum acceleration and while in Figure 10 is presented the variation in depth of the maximum shear strain.

Table 4. Values obtained for linear equivalent analysis

Site	$a_{g,max}$ (m/s^2)	$\gamma_{máx}$	AR
L_1	1.77	0.0192	0.69
L_2	2.71	0.0197	1.06
L_3	2.72	0.0109	1.07

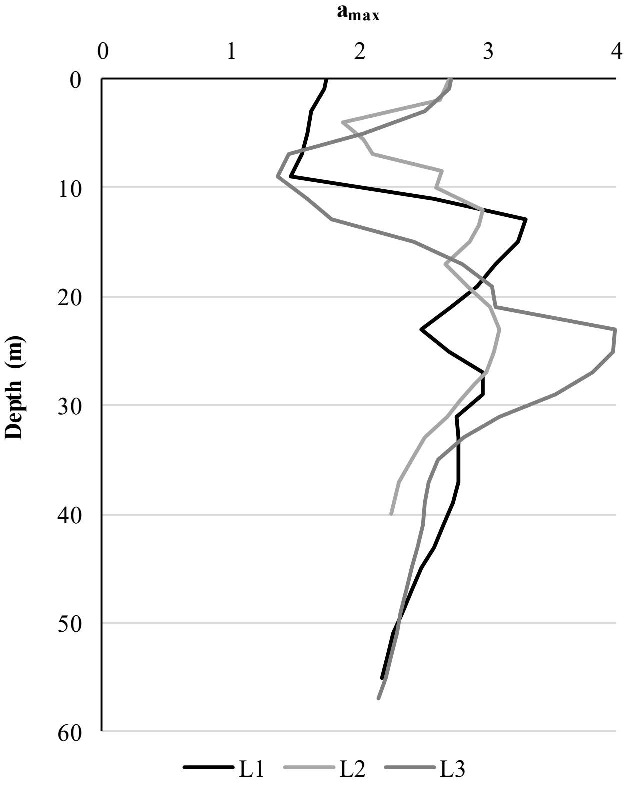

Figure 9. Variation of a_{max} for the three places

Considering the peak acceleration at surface, $a_{peak,surface}$, and the peak acceleration at the outcrop, $a_{peak,outcrop}$, the amplification ratio, AR, was determined considering equation (3) (see Table 4):

$$AR = \frac{a_{peak,surface}}{a_{peak,outcrop}} \tag{3}$$

From the values in Table 4, it can be observed that AR is close to unity for sites L_2 and L_3, while site L_1 has a lower AR value equal to about 0.7.

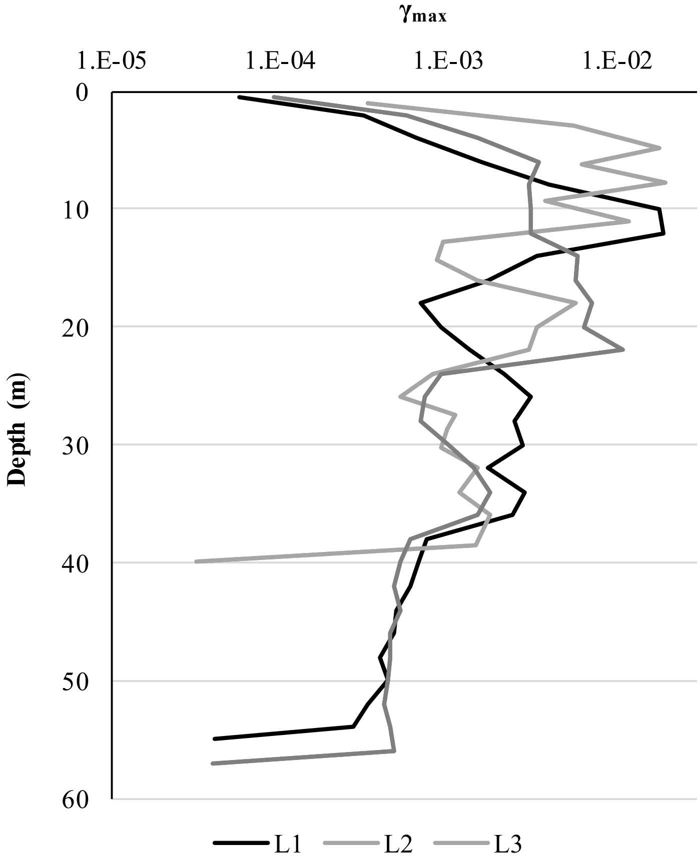

Figure 10. Variation of γ_{max} for the three places: a) L_1; b) L_2; c) L_3

For L_1, the higher values of strain are between 8 and 14 meters of depth, with $\gamma_{max,L1}$ = 1.92E-02 at 14 meters, which is a zone that is essentially composed of fine sands and silts, with low values of V_s = 130 m/s. For L_2, the higher values of strain are between 2.50 and 12.50 meters, with $\gamma_{max,L2}$= 1.97E-02 at 8 meters, which is a zone that is essentially composed of fine sands and clays, with lower values of V_s = 147 m/s. For L_3, the strain values increase in depth until the maximum value of γ_{max} = 1.09E-02 at 24 meters. Up to this depth, the soil is essentially composed by fine sands slightly muddy, with V_s = 150 m/s.

For places L_1 and L_2, the values of ground strain obtained using the K_g value, are similar, slightly smaller, than the ones obtained from the linear equivalent analysis, with a variation of 18 % for L_1 and 13 % for L_2. However, for L_3 there is a significant difference between the obtained values of ground strain, using K_g values, the ground strain is about two times the value that is obtained from the linear equivalent analysis. This difference is due to the fact that for L_3, the peak acceleration, a_{max}, is higher than the one that obtained in the other sites, and also due to the parameters f_0 and A_0.

Analysing the parameters f_0 and A_0 that influence the values of strain obtained using K_g is possible to conclude that f_0 is the parameter that has a lower influence on the obtained values, and the that the parameter A_0 is the one that has the higher influence, since higher values of A_0 translates into a higher probability of amplification of the ground motion, which will lead to higher ground strains. For instance, for L_2, if we consider the value of A_0 for L_3, A_0 = 4.73, instead of the value of A_0 = 4.12, the strain would be γ_{L2}= 2.23E-02 instead of the obtained value of γ_{L2}= 1.72E-02, which translates into a significant higher value of strain. The ground acceleration has also some influence in the obtained results.

It is also important to refer that, regarding the obtained values, the shear strain that is obtained using K_g, refers to a maximum value that is estimated at the surface, while the values that are obtained with the linear equivalent method are related to the distribution of the shear strain in depth (Figure 10), is possible to observe that the maximum values are not registered at surface.

5 CONCLUSIONS

The microtremor measurements were used to estimate the dynamic characteristics of the soil, namely the fundamental frequency, f_0, and corresponding local amplification factor, A_0, using the Nakamura method. These results were used to validate the soil profile used in the linear equivalent method.

Regarding the seismic site response, it is possible to conclude that the site response for site L_1, is different than the seismic site response for L_2 an L_3. Considering an acceleration on the bedrock of 0.26 g, in L_1 there is an attenuation of seismic action ($AR_{,L1} = 0.69$), and in L_2 and L_3 there is slight seismic amplification of the seismic action ($AR_{L2} = 1.06$ and $AR_{L3} = 1.07$).

For the estimation of the ground strain, using the vulnerability index, K_g, (Nakamura, 1996), comparing the values of ground strain, γ, using the K_g index, and the ones that are obtained with the linear equivalent analysis, for L_1 and L_2, there is a difference between 18% and 13%, however for L_3 is larger. The difference is mostly due to the fact that for L_3 the amplification ratio, A_0, is significantly higher than the values in the other places L_1 and L_2. The maximum acceleration, a_{max}, as also some influence in the obtained results, for L_3 the values are higher because in this location a_{max} is significantly higher than the acceleration registered in the other places.

This paper shows that Nakamura method and vulnerability index provided relatively good estimates of soil deposit fundamental frequency, and also can predict the maximum ground strain from strong motion. More studies will be done to confirm the trends identified in this paper.

6 ACKNOWLEDGMENTS

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