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Statistical analysis between intensity measures at the bedrock and the seismic response of sites

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ABSTRACT

This research work focuses on the seismic response of sites considering uncertainties. There are two main objectives. The first consists on assessing the effectiveness of intensity measures to predict properties of the expected wave motion at the surface. According to recent studies, special attention has been paid to predicting velocity-related ground motion parameters, which are highly correlated to the nonlinear dynamic response of civil infrastructures. The second objective is to analyse the evolution of the dynamic properties of the soil because of seismic waves. The main database of acceleration records in Colombia has been analysed, as well as three soil profiles that correspond to real building projects designed and built in the same country. From this information, a series of site seismic response analyses have been carried out using the equivalent linear method. Then, using statistical regression techniques, the correlation between input variables (parameters of the seismic records) and output results (intensity measures at the surface, maximum soil deformations, damping and shear wave velocity variations) have been evaluated. The paper shows that a series of highly correlated variables can be used to incorporate, in a simplified manner, site effects in the analysis of seismic risk at a regional scale.

Keywords: Site effects; soil profiles; geotechnical characterization.

1. Introduction

The world is becoming increasingly urbanized. Since 2007, more than half of the global population has been living in cities (ONU, 2023), where infrastructure plays a crucial role. In many areas, seismic hazard becomes one of the most pressing challenges in the field of structural design and civil engineering. Seismic events can have devastating effects on the population, resulting in economic and human losses attributed to the partial or total collapse of civil infrastructures (Trujillo, 2020). Accordingly, it is of importance the understanding and mitigating of this risk for the sustainable development of society. In this context, risk corresponds to the potential losses that may occur to the exposed system, resulting from the convolution of hazard and vulnerability (Cardona, 2021). Understanding seismic risk extends beyond predicting the magnitude and occurrence of earthquakes; it also encompasses assessing how these actions affect civil infrastructure and, consequently, the safety of people and property.

In this context, one of the goals of earthquake-resistant design is to ensure that buildings do not exhibit appreciable damage during low or moderate-intensity earthquakes, while allowing for a certain level of damage in some elements of the structure during high-intensity events, but never the collapse (Adam, 2023). The characterization of seismic hazard is directly linked to the intensity of ground motion during an earthquake. This is usually parametrized based on intensity measures (IMs). A seismic IM quantifies the characteristics of the induced motion that are important for both describing its destructive potential and its ability to predict structural response (Chavez et al., 2012). These measures are derived from physical quantities such as acceleration, velocity, displacement, etc., of the recorded ground response over time. However, representing seismic hazard solely through IMs is an incomplete approach because subsurface characteristics also play a crucial role in the propagation of the seismic waves. Therefore, a key aspect of seismic risk study is the characterization of the dynamic response of the soil profile (Pagliaroli et al., 2014). Alongside this, there is a need to analyze modifications in ground motion caused by nonlinear effects in the soil. In summary, the soil, acting as a filter, can amplify or deamplify the energy at certain frequencies of the seismic waves (Biglari et al., 2016), leading to significant variations in response at different locations, even within the same urban area. To analyze this variability, it is common to model site seismic response using the linear equivalent method proposed by (Schnabel et al., 1972) in the SHAKE program.

This method helps consider non-linearities associated with soil stiffness loss and the consequent increase in damping due to shear deformations, allowing an efficient evaluation of how site characteristics affect structural response. Identifying patterns and relationships between soil characteristics and IMs allows for optimizing structural design based on geographical and geological location, thereby improving community resilience to seismic events.

The main objective of this article is twofold. The first consists on assessing the effectiveness of IMs to predict parameters of the expected ground motion at the surface, after propagation of seismic waves. The second objective is to analyse the evolution of the dynamic properties of the soil associated to the pass of seismic waves. To characterize seismic hazard, it has been employed a set of 160 acceleration records taken from seismic stations in Colombia, which are located in firm rock. Additionally, the analysis is expanded by considering three soil profile obtained from building projects in the Santander department, Colombia. The combination of these seismic records and precise geotechnical profiles data provides a unique opportunity to explore and quantify correlations between seismic IMs and soil parameters. The aim is to contribute to the advancement of knowledge in the field of seismic engineering and to the reduction of seismic risk, with significant implications for urban planning and safety of infrastructure that shapes our built environment.

2. Cases of study

The geotechnical campaigns performed for designing three buildings in Colombia have been used in this research. They are briefly described in the following.

2.1. Bosques del Venado

The Bosques del Venado project is located on the right side of the Bucaramanga – Cúcuta road, in the municipality of Bucaramanga, which belongs to the department of Santander, Colombia.

2.1.1. Geotechnical study

According to (NSR-10, 2010), for a building classified as a special category, the minimum depth of exploration is 30 meters beneath surface. The minimum number of explorations is 5. Furthermore, in article H.2.3.6, it is specified that "*explorations conducted at the boundary between adjacent construction units of the same project can be considered valid for both units*" and "*for projects with several similar units, the total number of explorations will be calculated from the second construction unit onwards as half (50%) of that found for the first unit.*" Accordingly, the required number of explorations is 11. Thus, in the geotechnical study, 11 standard penetration tests (SPT) were conducted, of which six reached a depth of 18 meters and the remaining a depth of 4 meters. Samples were also extracted for 15 laboratory tests to classify the soils according to SUCS. The main conclusions from the geotechnical study of the Bosques del Venado project are as follows:

- Proposed excavation depth for foundation: Variable between 10.5 and 3.5 meters.
- The water table appeared at depths between 4.20 and 5.30 meters.
- Between 9.0 meters and undetermined depths exceeding 18.0 meters, residual soils composed of silty sands, clayey sands, and sandy clays from the Bucaramanga Neis were found.
- Liquefaction during seismic events is unlikely; therefore, no settlements related to earthquakes are expected.

Additionally, two Down-Hole tests were conducted, reaching depths of 16 and 18 meters. Another test was conducted, establishing shear wave velocities up to a depth of 60 meters, using the Refraction Microtremor (ReMi) method. The conclusions from this test were as follows:

- The average shear wave velocity (Vs) up to 30 meters depth is 450.7 m/s. In the 60-meter profile, velocities increase with depth.
- According to both the classification of soil profiles in the NSR-10 and the project's characteristics, the ReMi test results lead to the conclusion that the soil profile for seismic-resistant design is C.

2.2. Casa Bosque

The Casa Bosque project is located on 41st Street, 23rd Avenue, in the Cañaveral neighbourhood of the municipality of Floridablanca, department of Santander.

2.2.1. Geotechnical study

According to the NSR-10, for a building classified as a special category and consisting of two construction units, the minimum number of explorations is 8, in compliance with the previously mentioned article H.3.2.5. Seven explorations were conducted in 2013, and data from two explorations carried out in 2004 in the study area were also available. Four explorations reached a depth of 20 meters, and the others reached depths of 6 meters. Samples were extracted for 15 laboratory tests to classify soils according to SUCS (ASTM, 1985). The main geotechnical conclusions from the study conducted are as follows:

- Proposed excavation depth for foundation: 10 meters.
- The water table appeared at depths between 4.20 and 5.30 meters.
- Liquefaction during seismic events is unlikely; therefore, no settlements related to earthquakes are expected.

Additionally, a Downhole seismic wave test was conducted (reaching depths of 18 and 16 meters) to determine Vs and compressional wave velocity (Vp) values. The conclusions from this test were as follows:

- The Vs at depths greater than 12.0 meters (projected foundation depth is 10 meters) range between 1500 m/s > Vs ≥ 760 m/s.
- It is concluded that the soil profile at the study site, according to NSR-10, is B.

2.3. Torre Mayor

This project has been built in the Sotomayor neighbourhood located in Ward 12 (Main Ward of the Llano) of the municipality of Bucaramanga, in the department of Santander.

2.3.1. Geotechnical study

In the geotechnical study, 12 Standard Penetration Tests (SPT) were carried out, with 5 explorations reaching a depth of 24 meters and the others at depths between 2 and 4 meters. Samples were extracted for 15 laboratory tests to classify soils according to SUCS. Additionally, for the seismic wave study, a Downhole test was conducted, reaching a depth of 50 meters.

Another test, establishing shear wave velocities up to a depth of 45 meters, was the ReMi test. The main conclusions of this geotechnical study of the Torre Mayor project are as follows:

- Foundation depth: 12 meters.
- The water table appears at variable depths between 5.60 and 10.43 meters.
- The studied area does not present liquefaction potential; therefore, no settlements related to earthquakes are expected.
- The thicknesses of the strata and the depth at which the rock is found were determined using information from the 312.0 m depth exploration conducted in San Pío Park by the Metropolitan Aqueduct of Bucaramanga.

3. Seismic hazard characterization

3.1. Ground motion records

A set of ground motion records selected from the Colombian Geological Survey database (SGC, 2020), compiled from 1993 to 2017, was used for the seismic hazard characterization. The selection has been made with a proximity algorithm that identifies the closest stations to a given location. In this case, the selected location is the centroid of the area formed by the three cases studied, whose geographical coordinates are 7°06'07.4" N 73°06'31.8" W, as shown in Figure 1. From this point, and considering a radius of 110 km in the proximity algorithm, the nearest recording stations were searched for. This has been done to obtain a more accurate representation of the seismic hazard of the area.

Seismic records from the aforementioned database have been acquired in soil and rock. However, for the purpose of this study, these records act at the bedrock of the soil profiles, therefore, only those acquired in rock have been used. In this way, 10 stations have been identified (see Figure 2), from which 160 records of accelerations in rock have been extracted.

In order to achieve high-intensity levels, the seismic records have been scaled based on the proposal of (Vargas-Alzate et al., 2022). It has been used as IM to scale to the average of the spectral accelerations around 1 second. This period could be roughly linked to that of 10-storey reinforced concrete buildings. In terms of scaling, the choice was made to distribute the 160 records in 10 incremental bands every 0.1 g, from 0 to 1 g.

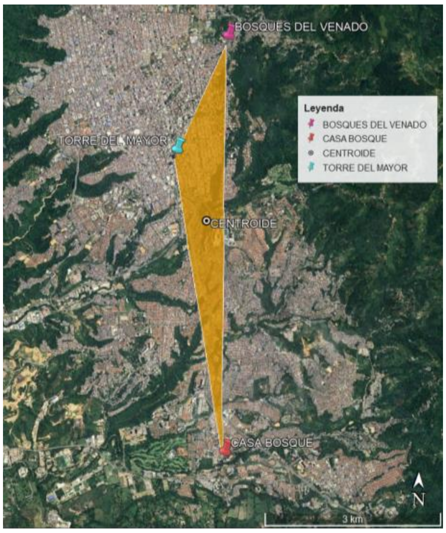

Figure 1. Location to identify the nearest seismic records.

3.2. Intensity measures

Seismic assessment and design methodologies require knowledge of how seismic actions, in terms of IMs, affect the structural response. The variables that represent this response are commonly known as engineering demand parameters, EDP, (Vargas-Alzate et al., 2022). Ideally, an IM should contain sufficient information about ground motion so that the EDP can be confidently predicted (Pejovic & Jankovic, 2015). In this investigation, since no structural analysis has been performed, the EDPs have been parameters of soil response and the resulting ground motion at the surface.

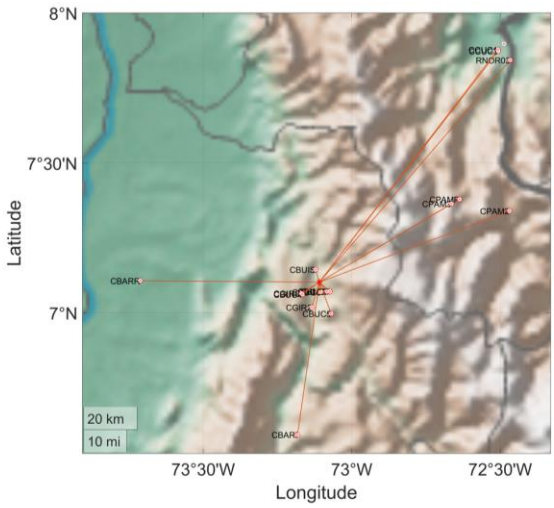

Figure 2. Seismic stations closest to the reference point

In order to characterize the intensity of the signals, 8 IMs have been calculated (see Table 1); these IMs have been used in the statistical analysis. More information on how to calculate these IMs can be found in Vargas-Alzate et al., 2022.

Table 1. Intensity measures

Intensity Measure	Variable
Spectral acceleration at T_1	$Sa(T_1)$
Spectral velocity at T_1	$Sv(T_1)$
Spectral displacement at T_1	$Sd(T_1)$
Average spectral acceleration	$AvSa$

Average spectral velocity	$AvSv$
Average spectral displacement	$AvSd$
Equivalent velocity at T_1	$VE(T_1)$
Average equivalent velocity	$AvVE$

4. Soil profiles

4.1. Bosques del Venado

According to the geotechnical information carried out to characterize the main features of the site, the geotechnical profile shown in Figure 3, from a depth of 3.5 metres (foundation depth) to 55 metres has been developed. Below this level, shear wave velocities are greater than 700 m/s according to data from the ReMi test. Therefore, it has been assumed that it appears at 55 metres. The soil stratum to be analysed is formed by sands of the Bucaramanga Neis. In other words, it is composed of sands and clays that have shear wave velocities between 370 m/s and 630 m/s.

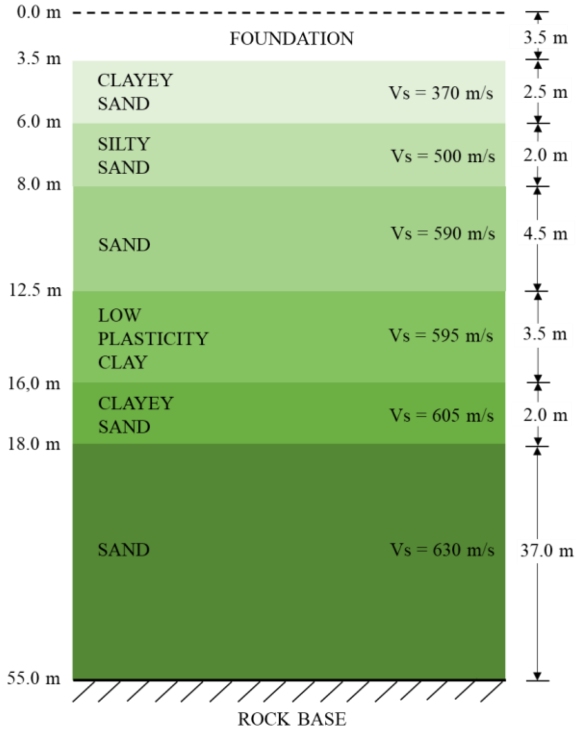

Figure 3. Soil profile of the Bosques del Venado Project

In order to consider the modification of the elastic properties of the soil profile, Table 2 shows the unit weight as well as the degradation curves assigned to each stratum according to the type of soil and depth at which it has been found.

Table 2. Bosques del Venado Project Profile Parameters

Layer	Unit weight (kN/m^3)	Degradation curves
1	18.99	EPRI (93), 0-20 ft
2	19.1	EPRI (93), 20-50 ft
3	19.1	EPRI (93), 20-50 ft
4	19.1	Vucetic & Dobry, PI = 15
5	19.1	EPRI (93), 50-120 ft
6	19.1	EPRI (93), 120-250 ft

4.2. Casa Bosque

Since the identification of firm rock is not indicated in the study, it has been assumed that it appears at 12 meters. Note that at this depth, the Vs range between 1500 m/s > Vs ≥ 760 m/s. Accordingly, the geotechnical profile shown in Figure 4, from a depth of 10 m (foundation depth) to 12 m, has been developed for this case. This profile is composed of a low plasticity clay stratum with a wave velocity of 788 m/s.

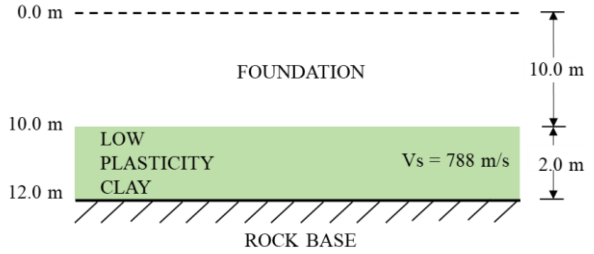

Figure 4. Soil profile of the Casa Bosque Project

Table 3 shows unit weight of the soil profile for the Casa Bosque project and the degradation curve assigned to the stratum according to the type of soil and depth at which it is located.

Table 3. Casa Bosque Project Profile Parameters

Layer	Unit weight (kN/m^3)	Degradation curves
1	18.83	Vucetic & Dobry, PI = 15

4.3. Torre mayor

According to the tests carried for the project, together with the 312.0 m deep borehole carried out in the San Pío park (located less than 1 km from the area under study) by the Acueducto Metropolitano de Bucaramanga, the geotechnical profile shown in Figure 5, from a depth of 12 m (foundation depth) to 300 m (depth at which the rock is indicated to appear) has been developed for this case. The profile is composed of strata of sands, clays and gravels with Vs varying from 400 m/s to 850m/s.

Table 4 shows the unit weight for this project. In addition, the degradation curves assigned to each stratum, according to the type of soil and depth at which it is found, are also shown.

Table 4. Torre Mayor Project Profile Parameters

Layer	Unit weight (kN/m^3)	Degradation curves
1	18.77	EPRI 1993, 20-50 ft
2	18.77	EPRI 1993, 50-120ft
3	18.8	EPRI 1993, 50-120ft
4	18.8	EPRI 1993, 50-120ft
5	18.8	EPRI 1993, 120-250ft
6	19.1	Vucetic & Dobry, PI = 15
7	19.1	EPRI 1993, 250-500ft
8	19.1	EPRI 1993, 250-500ft
9	19.1	EPRI 1993, 250-500ft
10	19.1	Vucetic & Dobry, PI = 15
11	19.1	EPRI (93), 500-1000 ft
12	19.5	Vucetic & Dobry, PI = 15
13	19.5	EPRI (93), 500-1000 ft
14	19.5	Vucetic & Dobry, PI = 15
15	19.5	EPRI (93), 500-1000 ft
16	19.5	Vucetic & Dobry, PI = 15

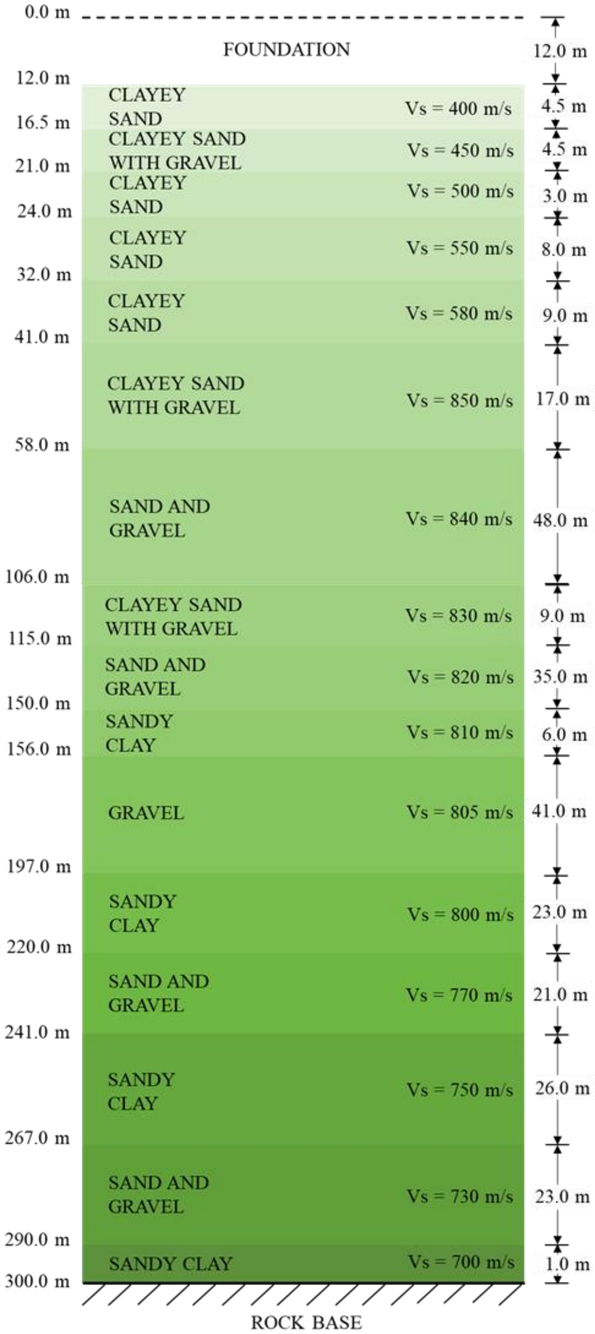

Figure 5. Soil profile of the Torre Mayor project

5. Results

By using the 160 records identified in section 3.1, and the 3 soil profiles described in the previous section, 480 equivalent linear analysis (ELA) have been performed in this research (Schnabel et al., 1972). Thus, for each soil profile, 160 surface ground motions have been obtained, for which IMs can also be calculated. In addition, since the ELA allows estimating the degradation of the elastic properties of the soil, results in this respect are also available for each profile.

5.1. Statistical analysis

Statistical analyses to measure the predictive capacity of IMs at the bedrock have been carried out. These analyses have been performed to fulfil the two main objectives of the research work.

Regarding the first objective, it has been identified the IM, calculated from the records at the bedrock level (Indicated with subscript b), most correlated with AvSv at the surface (Indicated with subscript s). AvSv has been selected since (Vargas-Alzate et al., 2022) and (Zapata-Franco et al., 2023) showed that it is the most correlated with an EDP in buildings called *maximum inter-story drift*. It should be noted that the IMs that depend on the fundamental period of the system (T_1) have been calculated for a value equal to 1 second. Figure 6, Figure 7 and Figure 8 shows the bivariate distributions, in the log-log space, between IMs and $AvSv_s$ for BV, CB and TM, respectively. Two types of regression analyses have been performed (Subscript L and NL denote linear and quadratic, respectively). It can be observed that $AvSv_b$ has the highest predictive capacity for estimating $AvSv_s$.

In the case of CM, due to the small thickness of the soil deposit (2 m) and the high shear wave velocity (788 m/s), negligible modifications of the motion have occurred due to propagation in this case, which explain the perfect correlation; basically, the same intensity measure is being computed from two virtually identical time histories. Then, the different correlation values for the other IMs represents the correlation between the different them with $AvSv_s$ directly from the input motion. Table 5 summarizes the correlation exhibited between IMs at the bedrock level and $AvSv_s$. It can be seen that AvSv at the bedrock ($AvSv_b$) is the most correlated IM to predict AvSv at the surface.

In order to verify steadfastness (See Vargas-Alzate et al. 2022), which is a statistical property related to the invariability of the efficiency when grouping results stemming from different models, it has been analyzed the bivariate distribution of the entire set of results (See Table 5). Under this grouping, a decrease in efficiency has been observed, yet, $AvSv_b$ remains as the IM with the highest predictive capacity of $AvSv_s$. It indicates that this IM exhibits steadfastness.

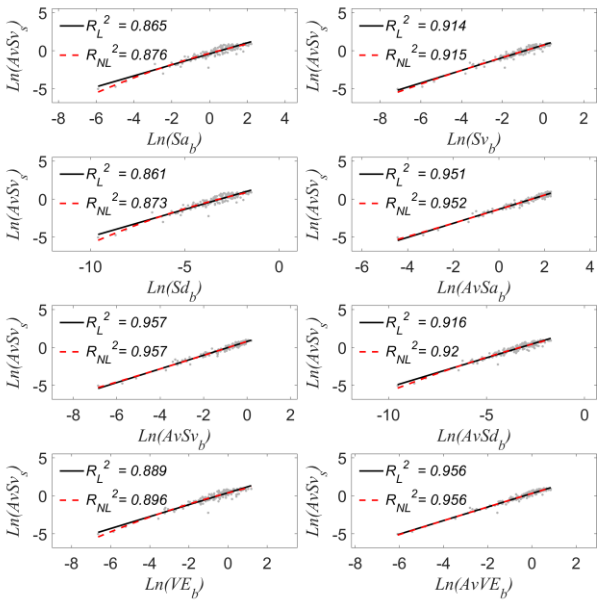

Figure 6 Bivariate distributions IM-$AvSv_s$ for BV

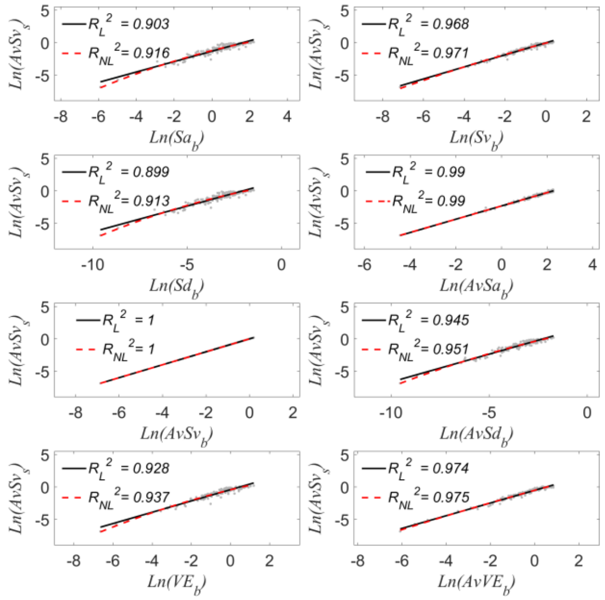

Figure 7 Bivariate distributions IM-AvSv$_s$ for CB

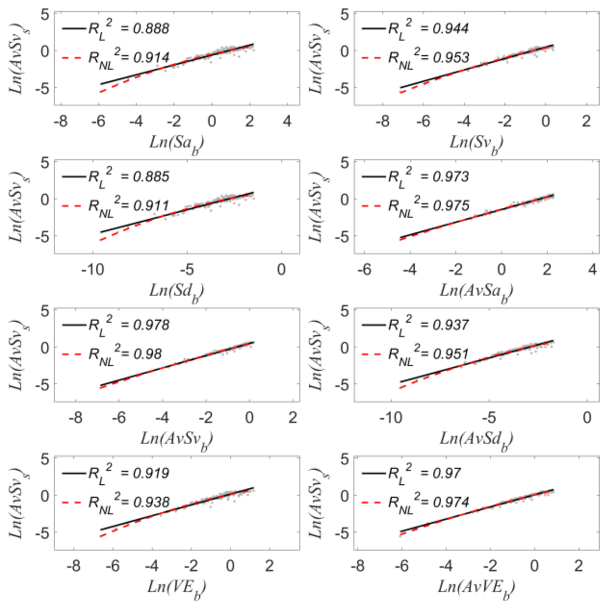

Figure 8 Bivariate distributions IM-AvSv$_s$ for TM

Table 5. Correlation coefficcient between AvSv at the surface vs IMs at the bedrock level

	BV	**CB**	**TM**	**BV+CB+TM**
$\boldsymbol{Sa}$	0.8647	0.9029	0.8884	0.7751
$\boldsymbol{Sv}$	0.9140	0.9684	0.9436	0.8248
$\boldsymbol{Sd}$	0.8615	0.8994	0.8854	0.7722
$\boldsymbol{AvSa}$	0.9512	0.9899	0.9730	0.8504
$\boldsymbol{AvSv}$	**0.9572**	**1.0000**	**0.9783**	**0.8567**
$\boldsymbol{AvSd}$	0.9165	0.9449	0.9370	0.8164
$\boldsymbol{VE}$	0.8893	0.9284	0.9193	0.7985
$\boldsymbol{AvVE}$	0.9556	0.9744	0.9702	0.8461

For the second objective, four cases of correlation have been analyzed for each soil profile. They are mainly related to the modification of the soil properties because of the seismic wave propagation:

- Correlation between the 8 IMs shown in Table 5 with the maximum shear strain reached, γ.
- Correlation between the 8 IMs shown in Table 5 with the average damping gained, $\delta\xi$. This variable has been calculated as the difference between the resulting damping after propagation and the elastic damping.
- Correlation between the 8 IMs shown in Table 5 with the average loss of shear wave velocity, δVs. This variable has been calculated as the difference between the elastic Vs and the resulting Vs after propagation.
- Correlation between the 8 IMs shown in Table 5 with the maximum shear strain reached by the shallowest layer, γ_{sh}.

It should be noted that the IMs that depend on the fundamental period have been calculated using the value of the soil profile. Histograms shown in Figure 9, Figure 10, Figure 11, Figure 12 show the statistical distributions of the soil response variables. It can be seen that, for the CB case, no significant modifications of the dynamic properties of the soil have been observed. Moreover, the shear strains reached during propagation are much lower compared to BV and TM.

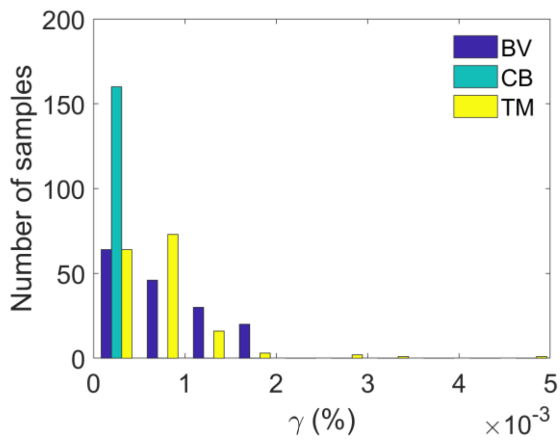

Figure 9 Histograms for the maximum shear strain

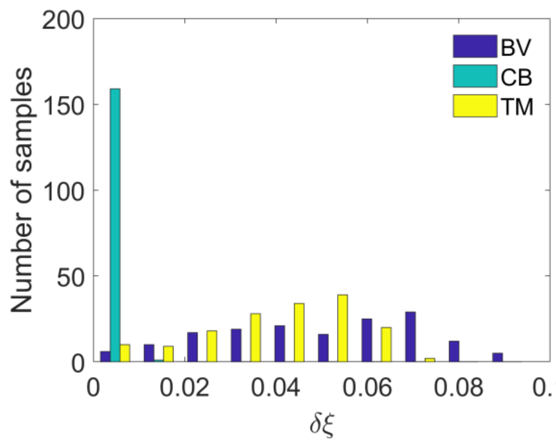

Figure 10 Histograms for the average dampoing gained

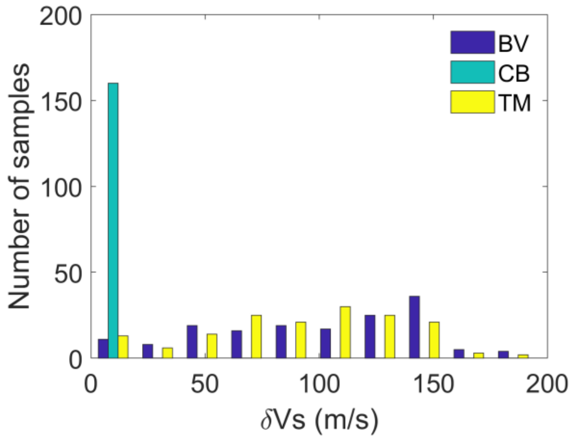

Figure 11 Histograms for the average loss of Vs

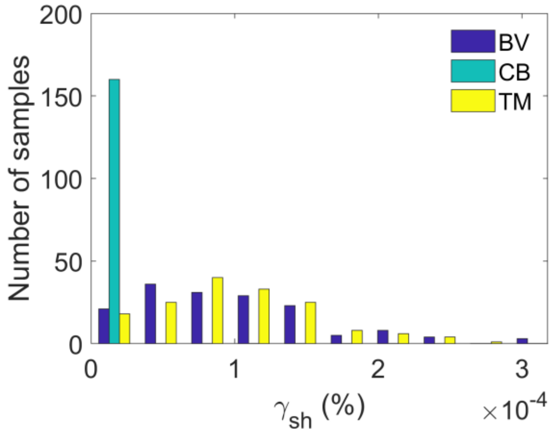

Figure 12 Histograms for the maximum shear strain at the shallowest layer

In Table 6, Table 7, Table 8 and Table 9 results have been summarized considering data from each location (BV, CB and TM) and for a combination of BV+TM. Data from CB have not been included in the combination of results since the variations in velocity and damping with this profile have been relatively insignificant.

Table 6. IMs at the bedrock level vs γ

	BV	CB	TM	BV+TM
$\boldsymbol{Sa}$	0.9124	0.0040	0.7251	0.8149
$\boldsymbol{Sv}$	0.8865	0.0477	0.7340	0.8057
$\boldsymbol{Sd}$	0.9125	0.0037	0.7249	0.8148
$\boldsymbol{AvSa}$	0.8247	**0.4552**	0.7611	0.7866
$\boldsymbol{AvSv}$	0.8958	0.2017	0.7935	0.8390
$\boldsymbol{AvSd}$	**0.9514**	0.0296	**0.8163**	**0.8793**
$\boldsymbol{VE}$	0.8838	0.0012	0.6880	0.7823
$\boldsymbol{AvVE}$	0.8979	0.0750	0.7682	0.8283

Table 7. IMs at the bedrock level vs δξ

	BV	CB	TM	BV+TM
$\boldsymbol{Sa}$	0.8262	0.0003	0.8078	0.7832
$\boldsymbol{Sv}$	0.8687	0.0339	0.8250	0.8113
$\boldsymbol{Sd}$	0.8255	0.0003	0.8075	0.7827
$\boldsymbol{AvSa}$	0.9351	**0.3361**	0.8839	0.8716
$\boldsymbol{AvSv}$	**0.9384**	0.1277	**0.9068**	**0.8842**
$\boldsymbol{AvSd}$	0.8744	0.0074	0.9053	0.8525
$\boldsymbol{VE}$	0.7799	0.0079	0.7745	0.7444
$\boldsymbol{AvVE}$	0.8769	0.0627	0.8757	0.8394

Table 8. IMs at the bedrock level vs δVs

	BV	CB	TM	BV+TM
$\boldsymbol{Sa}$	0.8144	0.0011	0.7937	0.7993
$\boldsymbol{Sv}$	0.8538	0.0383	0.8070	0.8254
$\boldsymbol{Sd}$	0.8138	0.0010	0.7934	0.7988
$\boldsymbol{AvSa}$	0.9142	**0.4347**	0.8643	0.8840
$\boldsymbol{AvSv}$	**0.9178**	0.1834	0.8895	**0.8982**
$\boldsymbol{AvSd}$	0.8558	0.0208	**0.8953**	0.8694
$\boldsymbol{VE}$	0.7661	0.0008	0.7626	0.7594
$\boldsymbol{AvVE}$	0.8571	0.0688	0.8643	0.8550

Table 9. IMs at the bedrock level vs γ_{sh}

	BV	CB	TM	BV+TM
$\boldsymbol{Sa}$	0.7040	0.0040	0.6229	0.6622
$\boldsymbol{Sv}$	0.7782	0.0477	0.7236	0.7496
$\boldsymbol{Sd}$	0.7030	0.0037	0.6215	0.6610
$\boldsymbol{AvSa}$	**0.9214**	**0.4552**	**0.8894**	**0.9034**
$\boldsymbol{AvSv}$	0.8859	0.2017	0.8283	0.8553
$\boldsymbol{AvSd}$	0.7707	0.0296	0.6664	0.7170
$\boldsymbol{VE}$	0.6647	0.0012	0.5809	0.6216
$\boldsymbol{AvVE}$	0.8067	0.0750	0.7138	0.7588

6. Conclusions

In this article, it has been sought to identify efficient relationships between parameters representing the seismic hazard at the bedrock level and the soil response because of the propagation of seismic waves. Two main objectives have been pursued. First, to identify IMs calculated at the bedrock level able to predict AvSv at the surface level. Second, identify the IMs calculated at the bedrock level to predict the effects of seismic wave propagation through the soil profiles.

Regarding the first objective, it has been concluded that the IM calculated at the bedrock level, for the three profile cases, that presents the highest correlation with $AvSv_s$ at the surface is $AvSv_b$. This good correlation is maintained when the results from the three profiles are grouped together, which is an indicator that this IM is steadfastness.

When analyzing the evolution of the dynamic properties of the soil due to the pass of seismic waves (second objective), the following conclusions can be drawn:

- For the prediction of the maximum shear strain reached, the best IM for CB is AvSa; for the BV and TM profiles it is AvSd..
- For the prediction of the average damping gained, the best measure for CB is AvSa; for TM and BV is AvSv.
- For the prediction of the average loss of Vs, the best measure for CB is AvSa; for BV is AvSv; for TM is AvSd.
- For the case of BV, CB and TM the IM with the highest correlation for the prediction of

the shear strain reached by the shallowest layer is AvSa.

- By grouping the results stemming from all the profiles, the correlation significantly reduced since the variations in velocity and damping with the CB profile have been relatively insignificant.
- When grouping only the results of the BV and TM profiles, it has been observed that the IM that best predicted the maximum shear strain reached has been AvSd; the one that best predicted the average damping gained has also been AvSv; the one that best predicted the loss of average shear wave velocity has been AvSv; the one that best predicted the maximum shear strain reached by the shallowest layer has been AvSa.

Future research should be oriented to quantify the predictive capacity of IMs at the bedrock by considering a probabilistic set of soil profiles and new seismological environments. In this way, it will be possible to develop improved relationships to quantify site effects more comprehensively.

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