Expected Ground Motion at the Historical Site of Poggio Picenze, Central Italy, with reference to current Italian Building Code

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ABSTRACT

The amplification of the ground motion at the surface is greatly influenced by the geotechnical characteristics of the soil formations below the ground surface. Traditionally, analyses of the ground response are deterministic, which means no consideration of the aleatory nature of geotechnical parameters of soil layers like density, shear wave velocity, *etc*. A fully stochastic procedure for estimating the site amplification of ground motion allows taking into account the record-to-record variability in an input ground motion and the uncertainty in dynamic soil properties and in the definition of the soil model. In particular, their effect on response spectra at the ground surface can be evaluated. With this procedure, it is pretended to reduce the aleatory variability into the soil model.

In this work, the soil profile below the San Felice Martire church, at Poggio Picenze (L'Aquila area, Abruzzo, Central Italy), has been studied basically on field geologic observations and drilling and geophysical tests retrieved from previous investigation campaigns. The dynamic soil properties were obtained by literature and by the test results. Amplification effects at the site under investigation have been estimated using fully 1D stochastic site response analyses and

for the object motion 7 real records compatible to the Italian code-based spectrum referred to 475-year return period. The Italian building code defines the reference seismic action in terms of elastic acceleration response spectra derived from the results of a probabilistic seismic hazard study.

The results in terms of accelerograms and acceleration response spectrum (with the associated dispersion) indicated a clear amplification of the input motion at the basement of San Felice Martire church due to the lithostratigraphic characteristics of the soil deposits at the site under investigation.

Keywords: Ground response, 1D stochastic analysis, site amplification, seismic input definition, Poggio Picenze, Italian building code

1 INTRODUCTION

The 6.3 Mw L'Aquila earthquake (April 6, 2009, Italy, Figure 1) caused considerable damage to structures over an area of approximately 600 km^2, including the urban centre of L'Aquila and several villages of the middle Aterno valley. The earthquake occurred on normal faulting affecting the main axis of the Appenine mountain belt that runs from the Gulf of Taranto in the south to the southern edge of the Po Basin in the northen Italy. This faulting is related to the collision of the Eurasian and African plates and the opening of the Tyrrhenian Basin to the west (Global risk Miyamoto 2009). The number of fatalities was 308, more than 1500 people were injured and more than 65000 were evacuated. This event was registered by 57 record stations that belong to the Italian Government. The mainshock caused heavy damages in the centre of L'Aquila, where intensity value was reported varying between VIII and IX. Damages were even more significant in some villages located in the middle Aterno valley, where intensities as high as IX-X were experienced in Castelnuovo and Onna. In total, 14 municipalities experienced a MCS intensity between VIII and IX, whereas those characterized by MCS intensity I≥VII were altogether 45 (Galli and Camassi 2009).

The work presented here focuses on the assessment of possible amplification effects, through a 1D stochastic analysis, of the ground motion due to the specific lithostratigraphic conditions at the site under investigation in Poggio Picenze, a small village located 17 km far away from L'Aquila and with more or less 1000 inhabitants.

Why a stochastic analysis?

The analyses of the seismic vulnerability of structures can be done following non-linear dynamic methods. In this case the input motions (accelerograms) should take into account the amplification due to the transition of the shear waves from the bedrock (assumed as an infinite elastic half-space) to the ground surface. Assimaki et al. (2003) explains that the site response analysis is strongly influenced by the uncertainty associated to the definition of soil properties and model parameters. Besides, Boore (2004) explains that a given site may be affected by a variety of earthquakes each with different characteristics in the terms of frequency content, duration, correlation between phases and components. Rota et al. (2011) state that quantifying the uncertainty associated to the model is not trivial, since this is affected by a combination of epistemic (lack of knowledge) and aleatory (related to the intrinsically stochastic nature of model parameters) uncertainty. Since a deterministic or even a parametric analysis cannot capture the uncertainty in dynamic soil properties and in the definition of the soil model parameters, a stochastic analysis should be performed. The stochastic methodology adopted here allows to systematically evaluate the sensitivity of the surface ground motion to the aleatory uncertainty associated to geotechnical parameters and to reference object motion. According to Lai *et al.* (2009), Rota *et al.* (2012) and Bozzoni *et al.* (2013), the procedure to carry out stochastic ground response analysis can be subdivided into the following steps:

(a) *Definition of the seismic input*, where a set of 7 real, spectrum- and seismo- compatible records are downloaded from the SEISM-HOME web portal, (SElection of Input Strong-Motion for HOmogeneous MEsozones, Rota et al. 2012). These accelerograms, recorded at outcropping rock sites with flat topographic surface, are spectrum-compatible with the acceleration response spectrum prescribed by the Italian building code at the site under investigation and also satisfy the requirement of "seismic-compatibility" which means that they are consistent with the regional seismotectonic and seismogenic setting of the area, where the site under investigation is located.

(b) *Geological, geotechnical and lithostratigraphic characterization*, where the 1D soil profile is defined taking into account geotechnical parameters: soil thickness layer, unit weight γ of the soil layers, Vs profile and the damping and shear modulus degradation curves.

(c) *Statistical characterization*, where the probabilistic distribution for the geotechnical parameters are defined.

(d) *Stochastic modelling*, where the 1D analyses of the soil profile is computed. It is recommendable not to use less than 1000 numerical simulations to stabilize the results.

(e) *Selection of spectrum compatible output records*, where a set of accelerograms compatible with the computed response spectrum are selected for further dynamic analyses.

2 DEFINITION OF THE REFERENCE SEISMIC HAZARD

The reference seismic hazard at the site (that is referred to outcropping rock and ground levelled topographic conditions) has been defined within a probabilistic framework. In Italy, the probabilistic seismic hazard is assessed and continuously updated by the National Institute of Geophysics and Volcanology (http://esse1.mi.ingv.it/). The results of this study were adopted by the current Italian building code NTC (2008) to prescribe the design seismic action.

The reference seismic action has been represented in this work by uniform hazard acceleration spectra and natural acceleration time histories for 475-years return period.

A set of 7 accelerograms recorded from real earthquakes have been used (Figure 2). The set was downloaded from the SEISM-HOME portal. It allows downloading suites of 7 real spectrum-and seismo- compatible accelerograms for any location of the Italian territory for outcropping rock conditions. The size of the suite of accelerograms has been established equal to 7, in agreement with the Italian Building Code (NTC 2008), which specify that the number of records in a set should be equal or larger than 7 if the user wishes to use the average results of the analyses instead of the most unfavourable ones. According to Rota et al. (2012), SEISM-HOME archive does not require inputting any information related to the regional seismogenic characteristics at a specific site, since these pieces of information have already been considered during the process of record selection. When downloading the set of real records, SEISM-HOME also returns a table of *metadata* containing an identification code, the values of the scaling factors and the main seismological characteristics of the selected records (Table 1). An important aspect of the portal is that the values of the scaling factors adopted for the selected records are kept reasonably close to unity.

It has been *a priori* decided to use recorded accelerograms since, according to the current literature and Eurocode (EC8 2005) recommendations, they are preferred with respect to artificial records, especially for applications in geotechnical earthquake engineering and, in particular, for seismic site response analyses. Furthermore, the use of real time-series as input to dynamic analyses of structures should be preferred, as they are realistic in terms of frequency content, duration, number of cycles, correlation among vertical and horizontal components and energy content in relation to seismogenic parameters.

For the stochastic analyses the probability density function (pdf) for the selection of the accelerograms was uniform, so all of them have the same probability of being selected than the others during the stochastic analyses. Figure 3 shows the comparison between the elastic acceleration response spectrum (ARS) computed by the set of natural accelerograms and the ARS given by the Italian code.

The response of soil deposits and soil structures during earthquakes is largely dependent on the frequency at which they are acted upon by the dynamic loads. The frequency content of each of the 7 records was computed in order to understand how the amplitudes are distributed among the interesting frequencies and they were obtained by transforming the ground motion from time domain to a frequency domain through a Fourier transform.

3 SEISMIC GEOTECHNICAL SITE CHARACTERIZATION

Information regarding the geological, geotechnical and geophysical characteristics of the soil deposit of Poggio Picenze town has been gathered from detail field investigations compared to literature and previous works. As follows, the relative information and the steps to define the soil profile at the site of San Felice church are described.

3.1 Geological framework

The Poggio Picenze village, within the Aterno valley in the NE part of the Apennines chain, is located in a very complex geological sector placed over Quaternary continental deposits (*i.e.* colluvial deposits, alluvial fan deposits, fluvial and lacustrine deposits). Geological setting and Pliocene-Quaternary continental deposits setting have been studied since the beginning of last century, and recently after the 2009 earthquake, focusing on tectonics, stratigraphy, geomorphology, particularly concerning relict landforms (Bosi and Bertini 1970; Bertini and Bosi 1993; Bagnaia *et al.* 1992; Miccadei *et al.* 1999, 2004; Giaccio *et al.* 2012).

In a large scale, the Poggio Picenze area is located at the boundary of the Paganica-Fossa basin, a morphological depression of extensional tectonic origin, filled with an up to 200 m thick with

Quaternary alluvial and lacustrine succession (Working Group MS-AQ 2010, Boncio et al. 2011). Such tectonic basin is part of the wider L'Aquila basin, composed of a complex arrangement of several, laterally connected, fault-bounded sedimentary basins, such as the upper valley of Aterno River basin, the Scoppito Basin, the western L'Aquila Basin, the Conca Paganica-Fossa and the middle Aterno Valley (Figure 4), all of Late Pliocene-Quaternary age (Bosi and Bertini 1970; Bertini and Bosi, 1993; Bosi *et al*. 2003; Cavinato *et al*. 2010; Galli *et al*. 2010). Such intermountain basins formed after the NE-SW oriented, regional extension that affected the central Apennines since Late Pliocene.

Geological units (bedrock and Quaternary continental deposits) outcropping in the Poggio Picenze area, investigated by means of 1:5000 field geological mapping, are pertaining to different sedimentary environments (from marine to continental) which controlled the features of rocks and deposits (Figure 5a). The mapped units are as follows from top to bottom:

<u>Quaternary continental deposits</u>

1. Backfill deposits (heterogeneous deposits, thickness ≤5 m), *Holocene*
2. Recent colluvial deposits (very loose gravel, sand and silt levels, with silt-clay matrix, thickness ≤2-4 m), *Holocene*
3. Ancient colluvial deposits (heterogeneous gravel, sand and silt levels, with silt-clay matrix, thickness ≤6 m), *Holocene*
4. Alluvial fan deposits, *Middle Pleistocene*
   4a. In the upper part, gravels and slightly cemented conglomerates, with thickness ~40 m.
   4b. In the lower part, conglomerates with thickness ~50 m.
5. Fluvial and lacustrine deposits, *Lower-Middle Pleistocene*
   5a. In the upper part, stratified, stiff-to-cemented white calcareous silt, with levels of sands, gravels and breccias.
   5b. In the lower part, gray clayey silts; total thickness ~50 m, of which ~20 m of gray clayey silts and ~30 m of white calcareous siltstones.
6. Slope deposits (Calcareous breccia, inferred thickness ~5-8 m), *Lower Pleistocene*

Bedrock (Meso-Cenozoic marine succession)

7. Limestones (well stratified; minimum thickness 50 m, could be several hundreds of metres), *Miocene*

The geological map of Poggio Picenze in Figure 5a reveals three principal deposits outcropping in the area surrounding the interested place, which from top to bottom layers (from the youngest to the oldest) are: colluvial deposits (light blue), alluvial fan deposits (green) and fluvial and lacustrine deposits (yellow); backfill deposits outcrop along the main roads and bedrock outcrops only in the NW side of the area. Buried lithological units are outlined by drillings and geophysical investigation (gray clayey silts of the fluvial and lacustrine deposits; Working Group MS-AQ, 2010), or inferred from the surrounding areas (slope deposits and bedrock limestones (Miccadei 2010).

Figure 5b shows a stratigraphic and lithologic scheme of the Poggio Picenze area, outlining outcropping or estimated thickness and stratigraphic relationship among all the geological-litological units present in the area defined by detail geological and geomorphological field mapping (Miccadei 2010). Features of the main lithological units are shown as well. The studied zone is over a small conglomerate level related to alluvial fan deposits (see red square in Figure 5). The thickness of the conglomerate layer is variable but can be estimated as 4-5 m at the interested zone as shown in Figure 6.

The geological and geometrical relationship among the geological units are summarized in two perpendicular cross sections intersecting at the interested site (Figure 7) realized by means of field geological data, drillings and geophysical tests (see next section for details). The general setting show slightly SW dipping strata. In the upper part a small conglomerate layer is present (4 in Figure 5) and overlie a ~50 m thick silt and siltstone layer (5 in Figure 5), the upper part of which (5a in Figure 5) shows a lateral change (heterotopic, zigzag line in Figure 7) from gravels and breccias with levels of sands and silts (in the NE side of the area) to stiff-to-cemented white calcareous silt (in the SW side). At the interested zone, the 5a unit is made up of white calcareous silt, with thin levels of sands, gravels and breccias. In the lower part the gray silts and slope deposits thickness and the bedrock depth are derived from Working Group MS-AQ (2010) and Lanzo *et al.* (2011).

3.2 Geotechnical information and geophysical field investigation

Drillings and geophysical tests retrieved from previous investigation campaigns have been collected in order to investigate the soil profile in the studied zone. The tests are located at the base of the conglomerate zone, so they give just information about the first 30 m of the fluvial and lacustrine deposits (see 5a layer in Figure 5b). Any tests reached the depth of the bedrock. These lacking information was indirectly overcome by detail field geological mapping of the surrounding areas and the correlation with existing subsurface data in the geological cross sections (Figure 7).

The Working Group MS-AQ (2010) has carried out an exhaustive geology investigation at Poggio Picenze zone as part as the classification of Macroarea 4 (Poggio Picenze, Barisciano and S. Pio delle Camere). After the L'Aquila earthquake (April 6, 2009), they gathered information and performed drilling tests (*i.e.*, Standard Penetration Test, SPT, and Down-Hole test, DH). Besides, some ambient vibration measures, three Electrical Resistivity Tomography (ERT) surveys and Multi-Channel Analysis of Surface Waves (MASW) tests were performed to complement the study in all the Macroarea. Since any of the drilling tests was greater than 50 m depth, the position of the bedrock was basically obtained by the ERT and ambient vibration tests. Besides, four undisturbed samples from Poggio Picenze were obtained from the SPT boreholes and analyzed to understand the nonlinear behaviour of the material, specially to obtain the degradation curves for shear modulus and damping, this work was performed by the Federico II University (Naples) and Roma La Sapienza (Appendix 4.13, Working Group MS-AQ 2010).

Based on the soil tests and the Vs measures from the DH tests the physic and mechanical soil properties for using on the numerical analysis are summarized (1D and 2D ground response, from appendix 4.6, Working Group MS-AQ 2010). Especially for the place close to the church, a principal soil layer consisted on white calcareous silts with clay and gravel/sand levels (5 in Figure 5) was specified with a Vs at the surface of Vs 300 m/s and increasing according to the depth up to reach 600 m/s. The general Vs profile at Poggio Picenze for the silt layer was done based on four DH tests (DH-09, DH-10, DH-11 and DH-12), and on the laboratory tests performed at Federico II University (Naples). The Vs of the transition zone was specified as 800 m/s and the Vs on the bedrock as 1250 m/s.

According to the geologic sections, four general layers are identified down the church: conglomerate (4b in Figure 5), white calcareous silts (5 in Figure 5), transition layer (slope deposits, 6 Figure 5) and bedrock (7 in Figure 5). The Working Group MS-AQ (2010) performed eleven 1D analyses in the area and one of them was closed to the church location. In this case, it was assumed to have horizontal layers and a 2D analysis was considered useless (Lanzo *et al.* 2011).

In order to better represent the soil profile of the interested place, additional tests to the ones reported by the DPC (2012) were gathered at Poggio Picenze. Here four seismic refraction tests are included (R1, R2, R3 and R4). From all the tests, just two seismic refraction (R3 and R4), two MASW (MASW1 and MASW2) and one down-hole (S10, DH-09) tests could be considered close to the church (Figure 8).

The shear wave velocity profile of each of the tests is showed in Figure 9. Note that the places where the tests were performed are not in the same altitude. In all cases, the general trend of the Vs is the increment of its value according to the depth. Besides, the Working Group MS-AQ (2010) elaborated an equation to compute the Vs of the white calcareous siltst layer at Poggio Picenze according to the depth. Just for the Vs of the DH test a reduced of Vs from 467 m/s to

413 m/s has been observed (data interpreted by CERFIS 2010). The church was built over a 4 m conglomerate layer, which is not represented by any reported test but is showed in the geological map (Figure 5).

The MASW1 reported a shear velocity of 1491 m/s after the 30 m depth. This strong variation in Vs means a local variation of the soil layer. The presence of a local formation of conglomerate and gravel can be found at that zone (see 5a in Figure 5 and Figure 7). However, the Working Group MS-AQ (2010) did not take into account the Vs of 1491 m/s to draw sections of the geological map for Poggio Picenze, and they reported the depth of the bedrock (around 50 m) according to ambient vibration measures. Since the church is close to the place of the MASW test, the necessity of more local tests is addressed to better describe the geometry of the soil layers down the church and to evaluate the presence of local conglomerate and gravel inclusions.

A 3D preliminary model of the distribution of shear wave velocities was built with the information in Figure 9. Here 5 velocities range were created: Vs < 150 m/s, 320 < Vs < 350 m/s, 413< Vs < 484 m/s, Vs≈ 600 m/s and Vs > 1400 m/s to better see the Vs profile in the zone. The tests, which are inside a 200 m radius from the Church, are showed in Figure 10. The horizontal distance from MASW1 to S10 (DH-09) is 103 m, from MASW1 to R3i is 39 m, and from R3i to S10 is 100 m. The church dimensions are roughly estimated as 22 m x 40 m.

The numerical 3D model represented in Figure 11 suggests layers roughly parallel to the surface as in the Sections of Figure 7. In this model the Vs greater than 600 m/s was not possible to be interpolated because not all the tests reported layers with Vs greater than 600 m/s. It is important to mention that the Church is placed over a 4 m conglomerate layer, which was manually added in Figure 11 according to the geologic map (Figure 5). The amplification of the seismic response due to a possible topographic effect seems not to be important due to the geometrical configuration of the place where the church was built. The previous model validates the assumption to have horizontal sub-layers within the white calcareous silts layer and validates the assumption of a 1D ground response analysis, as it was specified by the Working Group MS-AQ (2010). The most detail information of the Vs profile below the altitude of 749 amsl is given by DH-09 test, which reached a depth of 30 m.

3.3 Definition of the soil profile

According to the geological cross section, the stratigraphic and lithological scheme of the area (Figure 5 and Figure 7), and the numerical 3D representation of the soil layers, the lithologies present in the possible soil profile of the interested zone is as follows:

Layer I: This layer is composed of conglomerate. The thickness is 4 m. The Vs= 400 m/s and the specific weight were taken from the Table 4.1 of the Working Group MS-AQ (2010).

Layer II: The thickness was taken as 17 m and basically composed of sandy silts. The Vs was computed considering the velocities reported by DH-09 from 1 m to 17 m depth. CERFIS (2010) evaluates the Vs considering even the first layer of 149 m/s, but in this work it was not taking into account.

Layer III: The thickness is 4 m and the layer is basically composed of clayey silt (consolidated). The Vs was computed considering the velocities reported by DH-09 from 17 m to 21 m depth similar to CERFIS.

Layer IV: The thickness of this silt layer is 6 m. The Vs was defined as 413 m/s by CERFIS (2010).

Layer V: The thickness was taken as 23 m and basically composed of gray clayey silts. The Vs was taken as 600 m/s considering the data reported by the MASW1 test after the 25 m depth and the value of Vs suggested by the authors.

Layer VI: It is defined by slope deposits, which is a transition between the upper layers and the bedrock. The Vs= 800 m/s and the specific weigh were taken from Table 4.1 of Working Group MS-AQ (2010).

Layer VII: Bedrock. The Vs reported by the Working Group was 1250 m/s and it was based on down-hole measures at Aterno valley (Di Capua *et al.* 2009).

The detail stratigraphy of the 30 m thick white calcareous silts (5 of Figure 5) at the site is described by the majority of the previous tests represented in Figure 9. To describe the distribution of Vs into this layer, the results of the SPT (S10) and the Down-Hole (DH-09) tests were analyzed with the interpretation given by CERFIS (2010), see Table 2. The hole made by the SPT tests was used for the set up of the DH test.

It is clearly seen that there is a first layer with Vs around 150 m/s which contains sandy silt and rest of vegetables. However, at the zone of the church this sub-layer was replaced by a 4 m of conglomerate, where the foundation of the church was built. Then, more layers can be identified looking at the Vs variation. CERFIS (2010) has divided the log profile into 4 soil layers, while the STP test indicates 6 layers within the 30 m depth, both without consideration of the conglomerate part of the Church due to the location of the tests and without identification of the depth of the bedrock due to the test limitations.

4 STATISTICAL CHARACTERIZATION OF THE SOIL PROFILE

In order to make stochastic analyses of the 1D response of a media, it is necessary to provide statistical information (i.e. probability density functions, pdf) of some parameters as thickness (h), shear wave velocity (Vs), specific weight (γ), *etc*., to define a wide range of variability in the media. Since the gathered data did not directly give this information, so it was necessary to evaluate some available data and to compute the standard deviation values (σ) taking into consideration some assumptions.

CONGEO (2009) has been computed the Vs profile from the MAWS1 test. Here just this test seems to be useful to indirectly evaluate standard deviations. Figure 12 shows the range of variation of the Vs and the layer thickness (green zone). Knowing that the probabilistic density function, especially the normal distribution, can contains three standard deviations at both sides of the mean value, it is possible to evaluate an approximately value of the standard deviation by analyzing the green zone in Figure 12. For example, to compute the standard deviation of the Vs of the second layer, the range of Vs goes from 245 m/s to 450 m/s, so the σ is computed as (450-245)/6. The same procedure is made to compute the σ of the layer thickness. Table 3 and Table 4 summarize the calculations made to compute the σ and to see its tendency. Once the σ is computed, then the coefficient of variation (CoV) can be expressed as the rate of standard deviation σ and the mean value.

The MASW1 profile was useful to understand the distribution of the Vs profile at the zone of interest. Besides, it is seen that the CoV for each layer is around 10%. In the next section this value was increased to 15% to be applied to the final numerical soil profile. According to Lai *et al.* (2009), the CoV for the specific weight of soils can be taken as 5%.

4.1 Soil modelling for using within stochastic analyses

A summary of the proposed geometry and material properties for the soil profile is specified in Table 5. In the stochastic method the thickness, the Vs and the specific weight of each layer is associated to a Gaussian distribution characterized by a mean value and by the standard deviation (σ). Here random values up to three standard deviations at each side of the mean value were generated to construct the belt Gauss. The CoV for the thickness and for the Vs were obtained from an analysis of the MASW1 results, where in general a CoV for the thickness and the Vs of 10% was computed, here the CoV was increased to 15% in all layers.

The degradation curves for shear module and damping were obtained from Table 4.1 of Working Group MS-AQ (2010) where some soil tests were performed for Poggio Picenze. The material MAT1 is specified for conglomerate and identified as **cglp**. The material MAT2 is composed of white silt and clay with sand and gravel lens and identified as **L**. The material MAT3 is a transition between silt and bedrock, it contains breccias and conglomerates and identified as **bb**. The material MAT4 is given for bedrock with characteristics obtained from the literature. All the degradation curves are shown in Figure 13.

5 1D SITE RESPONSE ANALYSIS

5.1 Deterministic 1D site response analysis

The 1D analyses of the ground response can be used just if the soil deposit is not prone to topographic amplification effects and constituted by plane parallel layers. The seismic wave is vertically transmitted and the non linear cyclic behaviour of each soil layer is represented by an equivalent linear model (Kanai 1951, Roesset and Whitman 1969, Tsai and Housner 1970). Figure 14 shows a simplified scheme of the 1D analysis. In general, the input motion at the bedrock -that is in time domain- is represented as a Fourier Series using Fourier transform in the frequency domain. Each term in the Fourier series is subsequently multiplied by the Transfer Function, which is an intrinsic soil function. The output motion is then expressed in the time domain using the inverse Fourier transform. The so-called Transfer Function, which is valid for linear behaviour of soils, takes into account the nonlinearity of the soil layer by using equivalent values of shear and damping from degradation curves through an iterative process (Idriss and Sun 1992). Large amplification of the signal occurs mostly in areas where layers of low seismic velocity overlie material with high seismic velocity (*i.e.*, impedance contrast).

To perform the 1D analysis, a group of 7 deterministic analyses has been carried out with EERA (Bardet 2000) in order to verify the stochastic results. These input accelerograms are the ones located at the outcropping rock (soil type A). The outcropping records are the results of the incident and reflected waves. However, and according with EPRI (1988), at least 75% of the power in a free surface motion is due to vertically propagated shear waves and the remaining energy may be due to scattered waves or P-waves. By deconvolution (which is automatically performed in SHAKE91), these accelerograms were computed at the bedrock and then by convolution a new set of accelerograms were obtained at the ground surface. In the deconvolution process it is accepted that all motion results from vertically propagating shear waves, but the energy beyond 15Hz should be filtered out prior to deconvolution within the equivalent linear model (Kramer 1996). The properties for the soil column were the mean values specified in Table 5, where the fundamental frequency of the soil column was 2.1 Hz.

The elastic ARS of the ground response are shown in Figure 15 with PGA going from 0.37g to 0.52g. Here a clear amplification was observed in terms of acceleration. The variability of the amplification demonstrates that its value depends on each earthquake record (frequency content). The maximum amplification ratio was around 2 Hz, which is close to the fundamental period of the soil column. The amplification is the ratio between the acceleration record at the ground surface and at the bedrock at a given frequencies.

5.2 Stochastic 1D site response analysis

For the stochastic analysis each of the soil properties specified in Table 5 were assumed to follow a Normal distribution (*i.e.*, Gauss). In Matlab (http://www.mathworks.com it), and following a Monte Carlo simulation, a total 1000 soil profiles were generated for the interested zone (Figure 16) with the Latin Hypercube sampling technique. The minimum and maximum value of Vs, thickness and specific weight were in the interval of ± three standard deviations. Then, an input file (*.txt) was automatically written to be used with Shake91 (software developed by Schnabel *et al.* 1972), where each soil layer is completely defined by its thickness, shear wave velocity, specific weight, and degradation curve for shear modulus and damping. The input accelerograms for each of the 1000 analyses was randomly selected from the set of natural accelerograms following a uniform distribution.

The procedure for each ground response analysis was equal to the one done in the deterministic analysis: the acceleration at the bedrock was computed by deconvolution and finally the ground response was obtained by convolution of the seismic record from the bedrock to the ground surface. All this procedure was performed with Shake91, software that computes the response in a horizontally layered soil-rock system subjected to transient and vertical travelling shear waves.

Shake91 assumes that the cyclic soil behaviour can be simulated using an equivalent linear model (Kramer 1996). Cyclic repetition of the motion is implied in the Fourier transform and quiet zone of 0 s are necessary to avoid interferences between cycles; which means no interference between successive trains of accelerograms. For this reason, the Shake91 source has been slightly modified to consider accelerograms with more than 4192 data points, as it is specified in Shake91 manual: "*For those rare occasions when MA= 8192 is needed, the size of the COMMON block and the length of the variable MAMAX in the MAIN Module should be changed to 51220 and 8192, respectively*." MA is the number of values for use in Fourier Transform, which should be a power of 2, and should be greater than the number of acceleration values to be read for input motion.

The elastic spectral accelerations at the ground surface are shown in Figure 17a. The minimum PGA is 0.28g and the maximum PGA is 0.72g. It can be seen the greater variability of acceleration is between 0.1 s to 1 s of period. Figure 17b shows the comparison between the mean ARS (PGA= 0.42g) with ±1 standard deviation and the elastic ARS specified by the Italian building code NTC (2008) for different soil types (the Vs_{30} was around 380 m/s). Similar to Figure 15b, a clear difference between the 1D ground response analysis and the ARS given by the code has been seen, especially for values between 0 s and 1 s. From 0 to 0.6 s, the ARS given by code is close to the Mean -1 standard deviation, which shows in evidence the non conservative proposal values of the Italian code for Poggio Picenze.

The mean of the maximum accelerations at the ground surface and at the bedrock are 0.46g and 0.20g, respectively (Figure 18). Here it can be seen that the distribution of maximum values follow a lognormal and normal distribution. According to the Italian code a PGA of 0.26g should be used for soil type A. So, an amplification factor of 0.46/0.26= 1.8 could be proposed for the study place at Poggio Picenze.

To see how each of the 7 input records affects the ground response at the surface, 7 more 1D stochastic analysis were performed using each individual record. A total of 7x1000 runs were done without varying the soil profiles defined before. Figure 19 shows the variability of the PGA at the ground surface due to the influence of each record, the bold continues line represents the distribution using all the records. By comparing the results it can be seen how for some cases is evident how much the distribution of each record varies with respect to the total distribution (see input 2 and input 4). This is related the frequency content of the record and the fundamental frequency of the soil column. Here is demonstrated the importance for using stochastic analysis since the ground response is greatly influence by the input record

6 SELECTION OF THE BEST SUITE OF 7 RECORDS AT THE GROUND SURFACE

The mean spectra computed at the free surface are the final results of the stochastic analysis at the site of San Felice Church (Poggio Picenze). However, for structural dynamic analysis, the acceleration, velocity or displacement time history may be required. Therefore, the computed spectra are not sufficient.

A possible procedure for the definition of these time histories, is to utilize the acceleration time histories computed at the free surface (1000) as a data base from which a set of 7 records is selected by imposing the compatibility with the mean spectra obtained from the stochastic site response analysis. The criterion used to select the input records is the same implemented in the code ASCONA (Corigliano *et al.*, 2012). This algorithm determines the standard deviation between spectral ordinates of the mean spectrum and the generated spectrum in a fixed range of periods, basically from 0.15 s to 2 s. Besides, the maximum negative deviation between the mean spectrum and the computed spectrum in the fixed range of periods was verified not to be greater than 10%, as it is specified by the Italian code NTC 2008.

Figure 20 shows the 7 acceleration records expected at the ground surface considering the soil profile reported before. Note that the limit of the Time axis is different in some of the figures to clearly see the acceleration record.

7 CONCLUSIONS

In this work the 1D ground response analysis at the site of San Felice Church (Poggio Picenze) has been evaluated through a stochastic analysis. First, an exhaustive investigation regarding the geology of the study area was performed, based on field mapping compared to information from preliminary tests (drillings, geophysical investigations). This allowed to integrate and correlate existing subsurface investigations and to outline a thin conglomerate and breccias level at top of the soil profile and gravel and conglomerate levels interlayered in the white calcareous silts, which could be verified by additional tests. Knowing the variability of the soil profile and its properties, and even the influence of the input motion, a stochastic analysis was suggested. The assumption to have horizontal layers at the site was corroborated by a 3D model of the Vs developed with the gathered test results.

The reference seismic hazard is defined using the Italian building code (NTC, 2008) prescriptions through a 475-year return period elastic acceleration response spectrum for soil category A, and a corresponding set of 7 real, spectrum-compatible, accelerograms. If the methodology is to be applied to a site outside the Italian territory, a site-specific PSHA should be performed for the definition of a uniform hazard response spectrum that will be subsequently used for the selection of the natural records.

The stochastic procedure allowed analyzing 1000 simulations, which means 1000 soil profiles with a random uniform selection of input motions. The probability density function (pdf) for the thickness, Vs and specific weight of each soil layer was assumed as normal distribution; while, the pdf for the input motions was uniform. This last means that every time an analysis was done, an acceleration record was chosen from the set where every record had the same probability of being selected. Then, each simulation was analyzed using the program Shake91; which assumes that the cyclic soil behaviour can be simulated using an equivalent linear model. At the end a set of 1000 acceleration records was computed at the free surface.

The results indicated a clear amplification of the PGA from a soil type A to the ground surface of the San Felice church, from 0.26g to 0.46g. To see the influence of the input record on the 1D analysis, other stochastic analyses were done considering just each record for each set of 1000 simulation. The results show that the soil response highly depends on the frequency content of the record and on the frequency of the soil layer.

Finally, a set of 7 acceleration records computed at the free surface and compatible with the mean computed ARS were selected. The stochastic analysis took into account the inherent variability and uncertainty in the soil profile and on the seismic demand, so the results expressed here can be used to assess the dynamic response of the San Felice church.

8 ACKNOWLEDGMENT

This work has been possible under the financial support of the Coimbra Group Universities under the Scholarship Programme for Young Professors and Researchers from Latin American Universities 2012. Prof. Miccadei acknowledges the University "G. D'Annunzio" of Chieti Pescara for the Faculty Funds.

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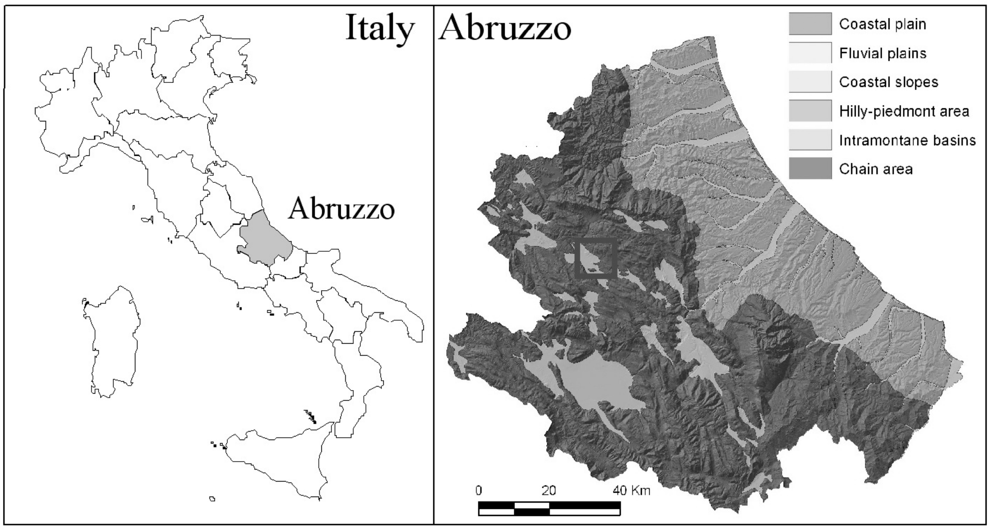
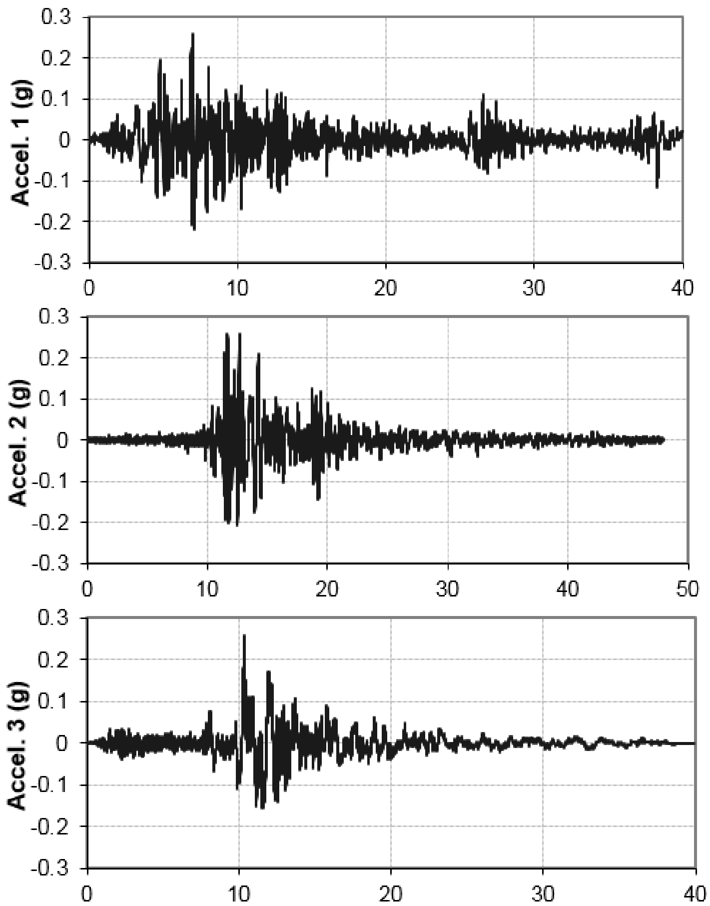


Figure 1.- Location map of the study area (red box) in the chain area of the Abruzzo region.



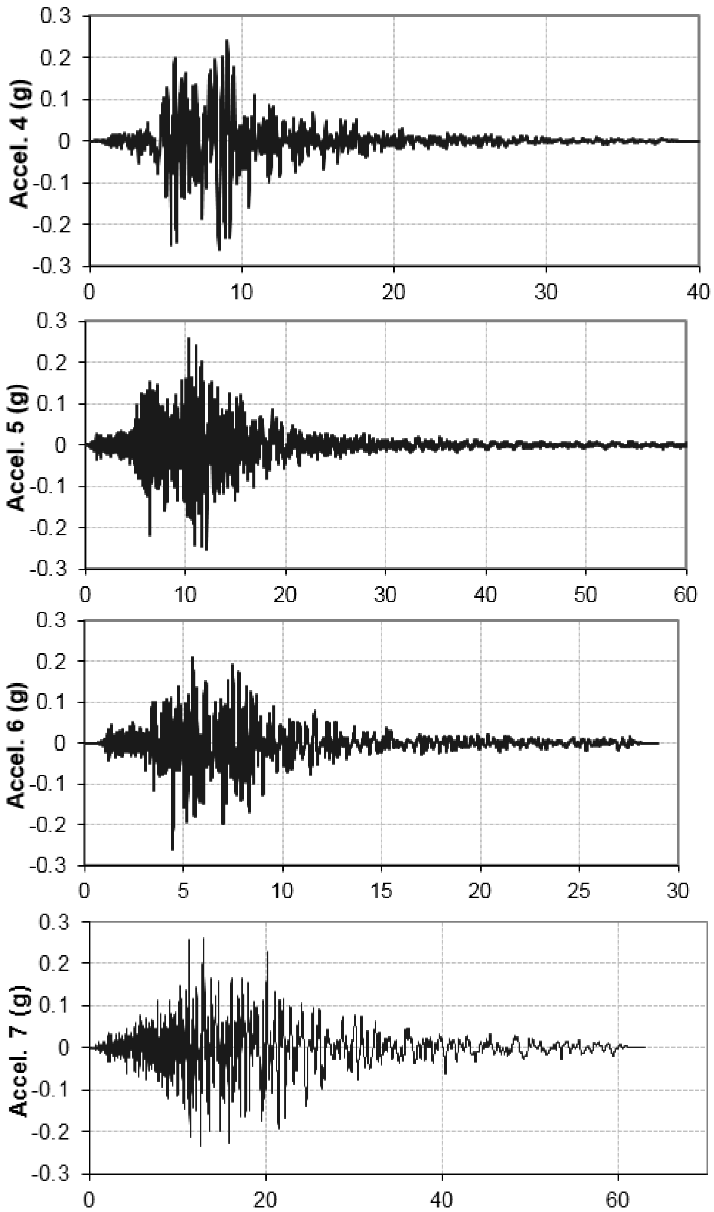
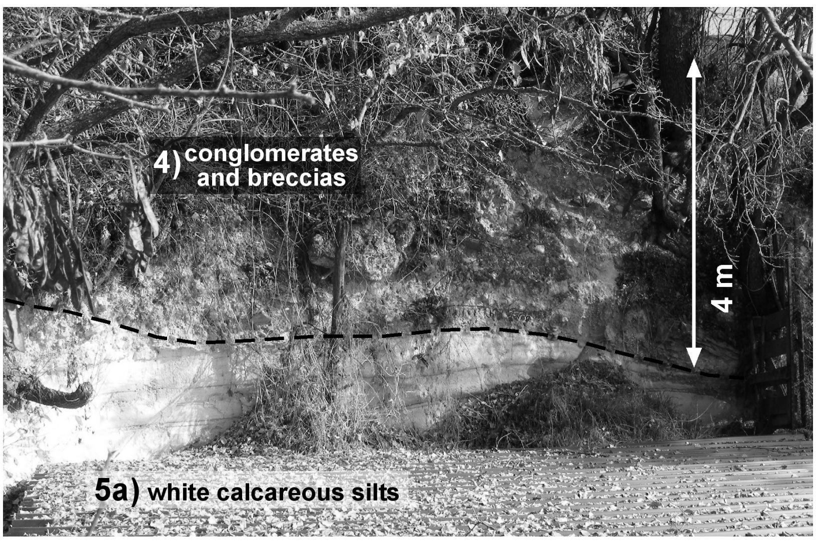


Figure 2.- Horizontal component of the selected accelerograms obtained from SEISM-HOME for Poggio Picenze (475-year return period, http://www.eucentre.it/seismhome.html).



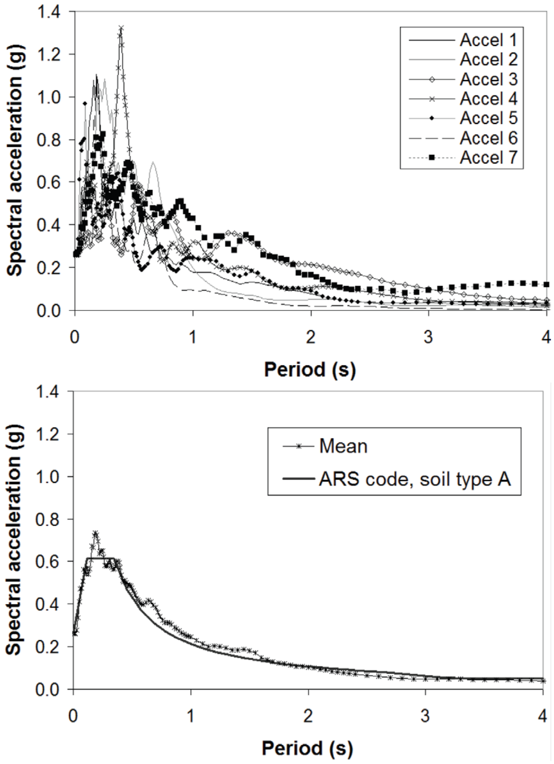


Figure 3.- (a) Elastic ARS of the set of 7 natural accelerograms for the 475 year return period imposed for outcropping rock, and (b) the comparison of the ARS given by the Italian code for soil type A and the mean ARS of the input records (damping ratio 5%).

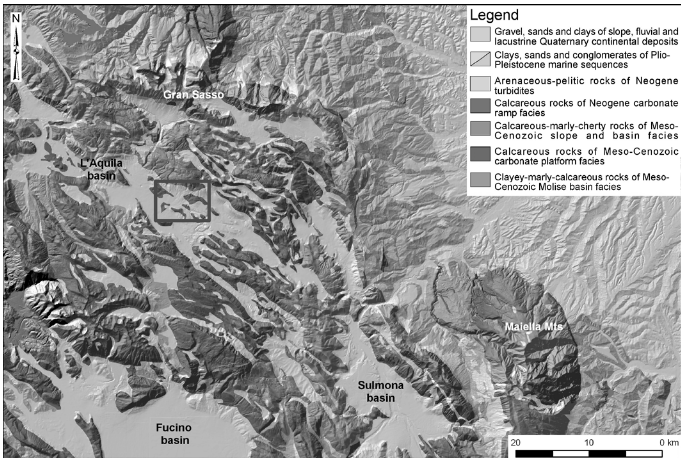


Figure 4.- Simplified geological scheme of the Abruzzo Apennines.

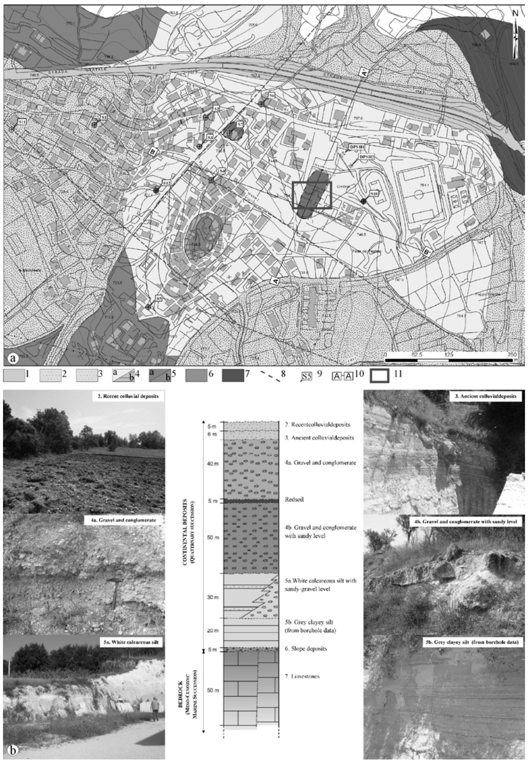


Figure 5.- a) Geological map of Poggio licenze area. Quaternary continental deposits: 1) backfill deposits; 2) recent colluvial deposits; 3) ancient colluvial deposits; 4) alluvial fan deposits (a: upper part, b: lower part); 5) fluvial and lacustrine deposits (a: upper part, b: lower part); 6) slope deposits. Bedrock (Meso-Cenozoic marine succession): 7) limestones. Symbols: 8) inferred fault; 9) geotechnical and geophysical field investigations; 10) geological profiles; 11) interested site. b) Stratigrafic and lithologic scheme of the Poggio Picenze area (Miccadei 2010).

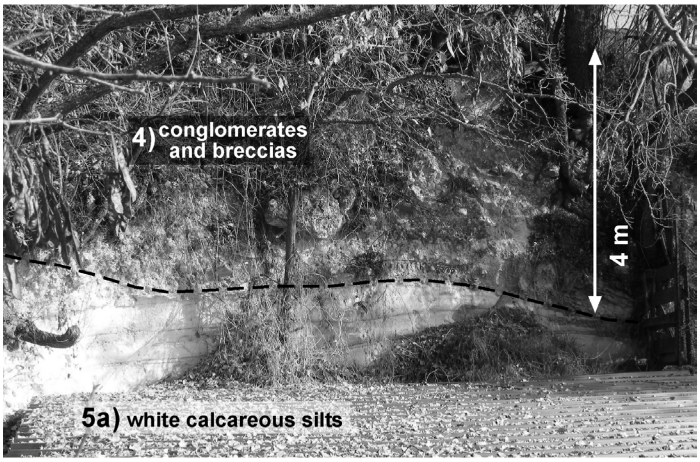


Figure 6.- Limit between white calcareous silts of lacustrine deposits and overlying conglomerates and breccias of alluvial fan deposits close to the interested site (numbers refers to Figure 5).

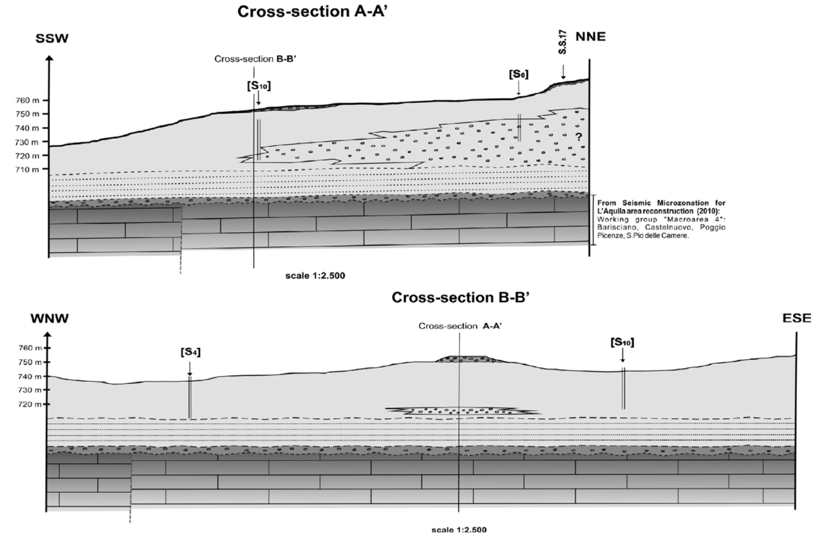


Figure 7.- Cross sections of the geologic map at Poggio Picenze (see Figure 5 for legend and location).

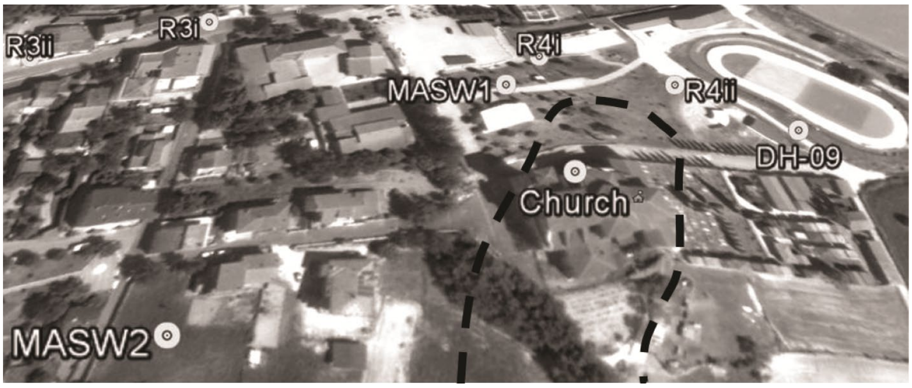
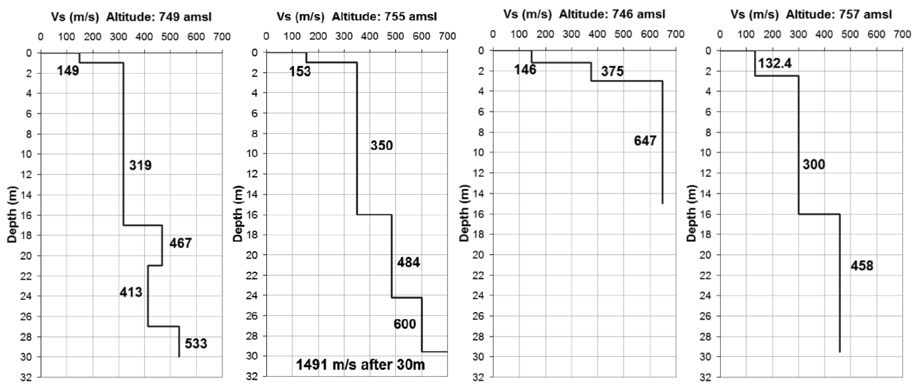


Figure 8.- Identification of additional tests performed close to the church zone. The dash line encloses the conglomerate upper layer (see Figure 5 for reference).



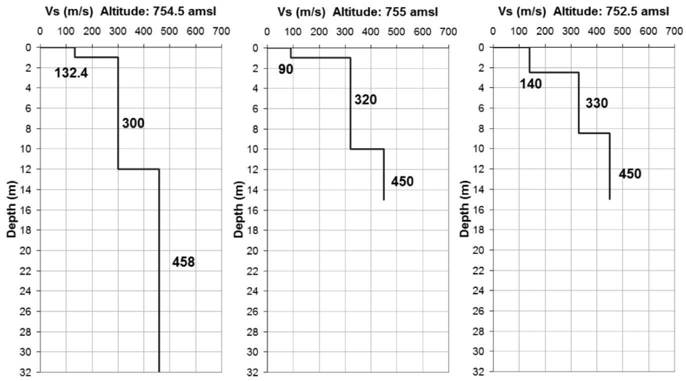


Figure 9.- Vs profiles of the tests close to the Church: a) DH-09 (CERFIS 2010), b) MASW1, c) MASW2, d) and e) R3, f) and g) R4. A Vs= 1491 m/s is specified for MASW1 after the 30m. Note that the surface of each test is not at the same altitude.

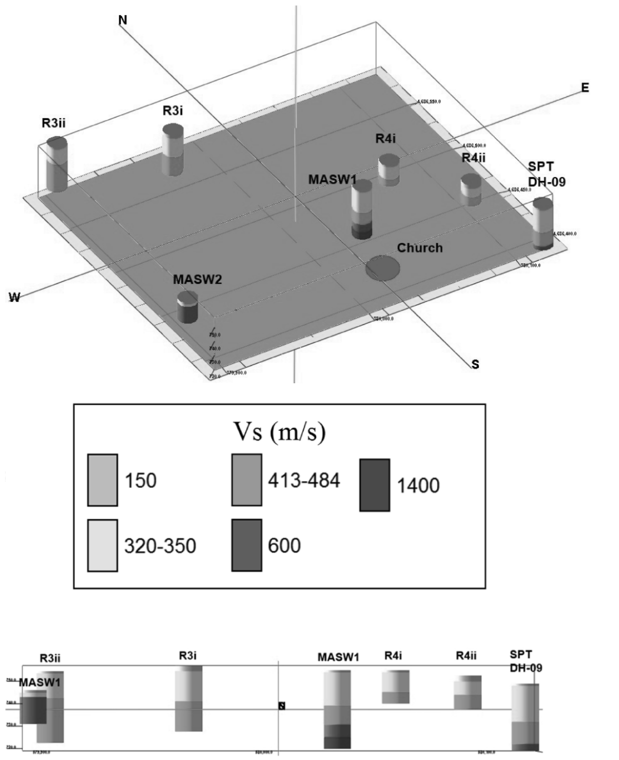
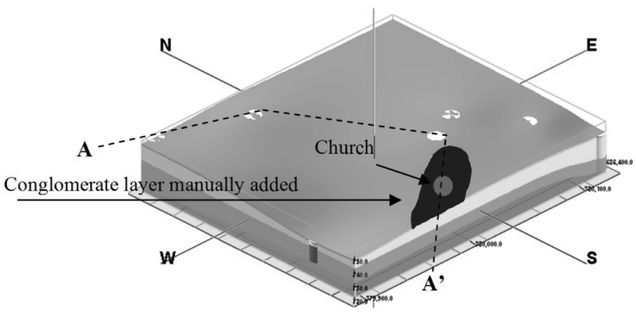


Figure 10.- Location of the tests and its Vs profile; a) Vs profile of each of the tests closet o the church; b) South view of (a).



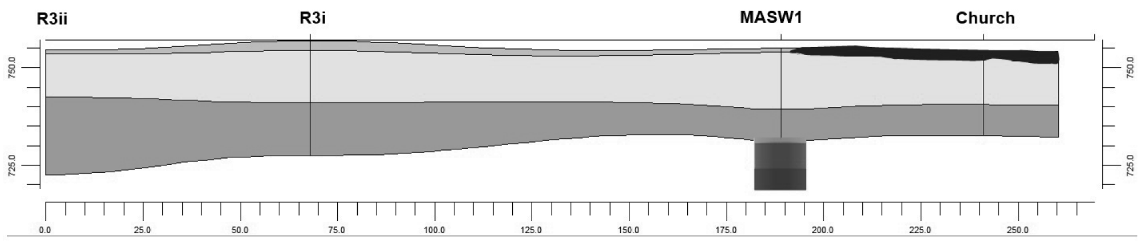


Figure 11.- a) 3D numerical model of the best representation of the Vs profiles according to the selected tests. The depth of the bedrock was not reached by any test. Here a conglomerate zone of 4 m at the top has been manually added according to Sections 3 and 8 in Figure 9; b) Cross section A-A' through R3ii – R3i – MASW2 – Church.

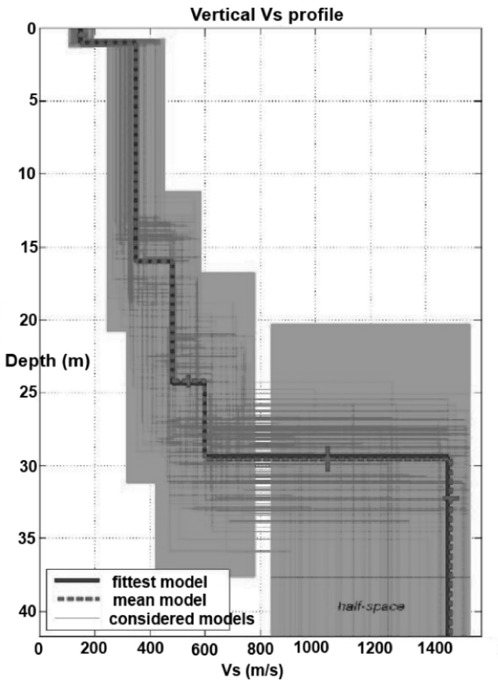


Figure 12.- Vs profile according to the MASW test (CONGEO 2009).

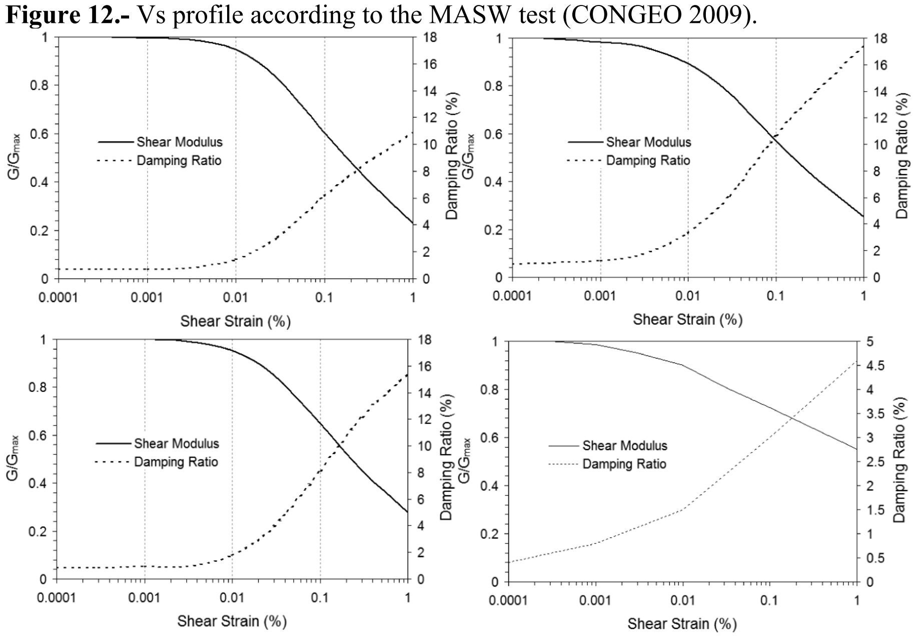


Figure 13.- Shear module and damping degradation curve for the soil layers: a) Conglomerate (cglp); b) White silt and clay with sandy-gravel lens; c) Transition between silt & bedrock; d) Bedrock.

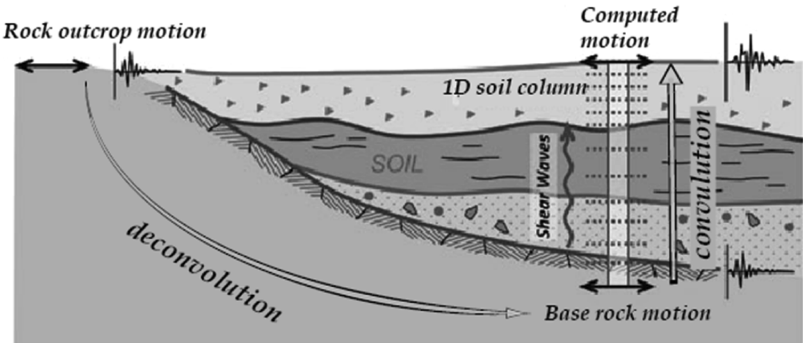


Figure 14.- Scheme of the 1D analysis of the ground response (modified from Nikolaou and Go, 2009).

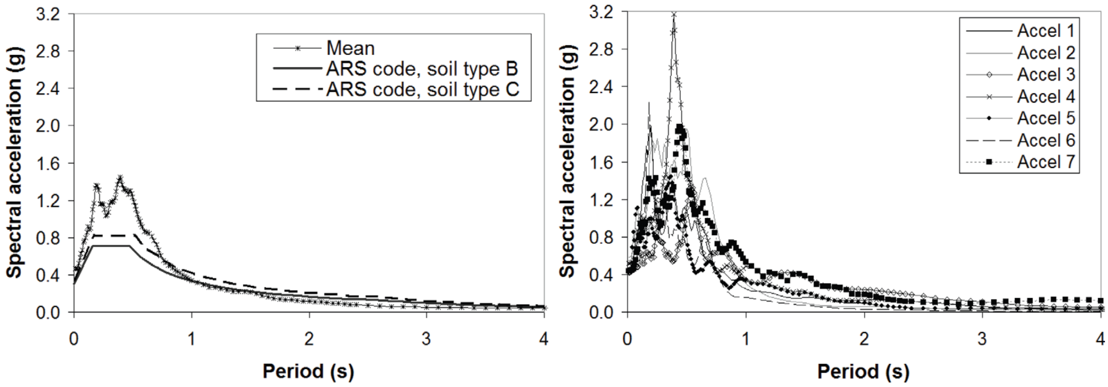


Figure 15.- a) Elastic spectral acceleration at the ground surface for the 475 year return period accelerograms, and b) the comparison of the ARS given by the Italian code for soil type B and C and the mean ARS of the ground response (damping ratio 5%), the PGA of the mean curve is 0.47g.

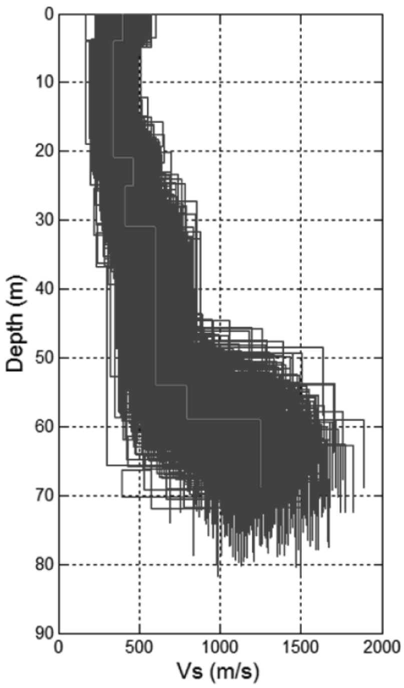


Figure 16.- Variation of thickness and Vs along the soil profile for the 1000 simulations (blue colour) and the mean value (red colour).

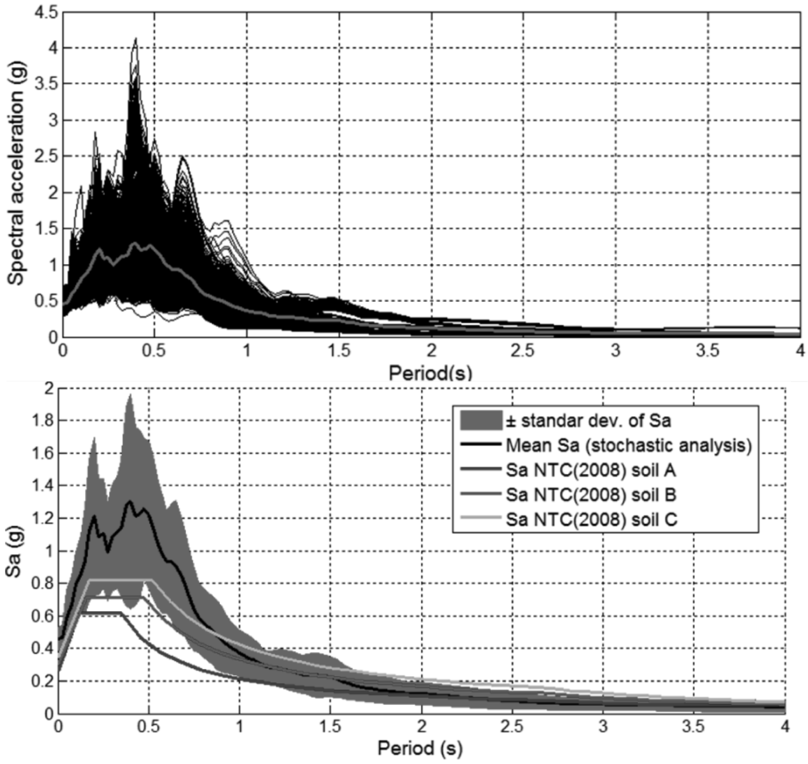


Figure 17.- Elastic ARS at the ground surface: a) 1000 simulations; b) comparison of the elastic ARS given by the Italian code for soil type B and the mean ARS of the computed records (damping ratio 5%) with the ±1 standard deviation.

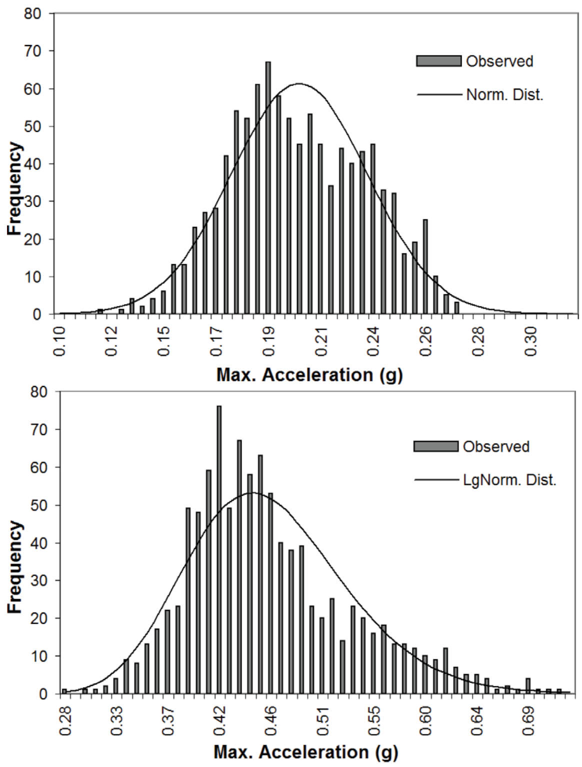
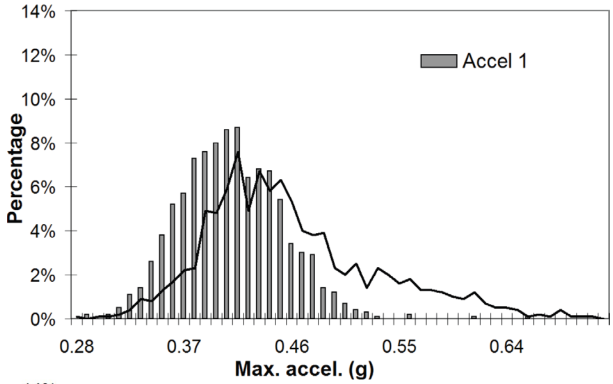
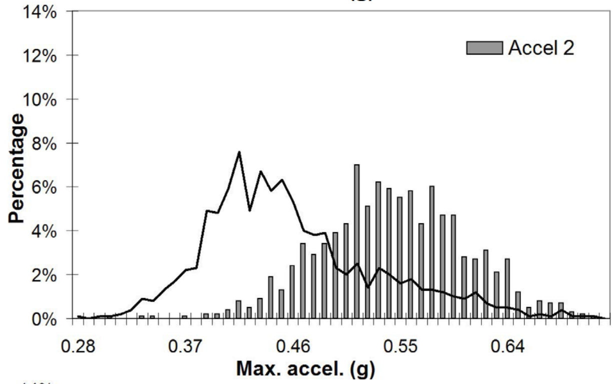
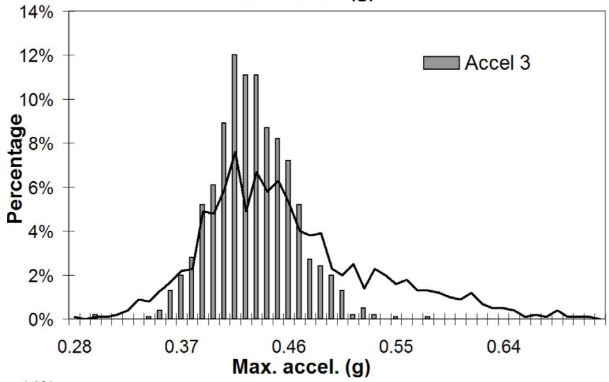
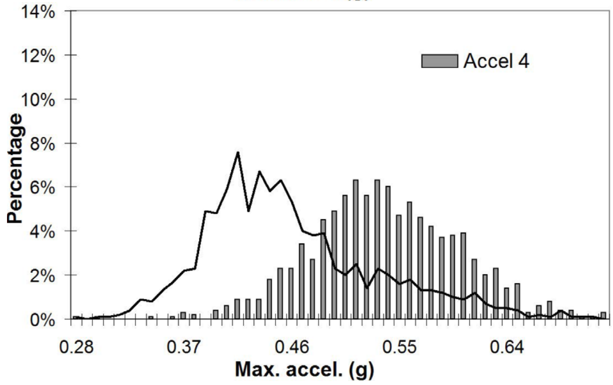


Figure 18.- Distribution of maximum accelerations at the (a) bedrock and at the (b) ground surface considering the 1000 simulations. The mean PGA at bedrock is 0.20g and at the surface is 0.46g with a CoV of 14% and 16%, respectively.


Accel 1
Percentage
0%
2%
4%
6%
8%
10%
12%
14%
0.28
0.37
0.46
0.55
0.64
Max. accel. (g)


Accel 2
Percentage
0%
2%
4%
6%
8%
10%
12%
14%
0.28
0.37
0.46
0.55
0.64
Max. accel. (g)


Accel 3
Percentage
0%
2%
4%
6%
8%
10%
12%
14%
0.28
0.37
0.46
0.55
0.64
Max. accel. (g)


Accel 4
Percentage
0%
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0.28
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0.64
Max. accel. (g)

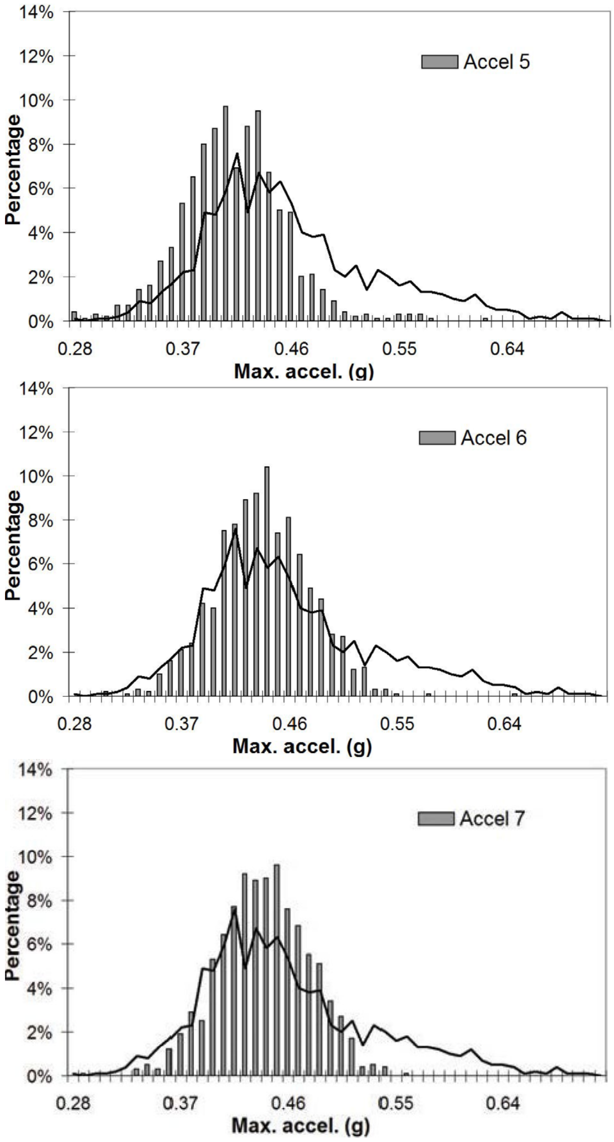
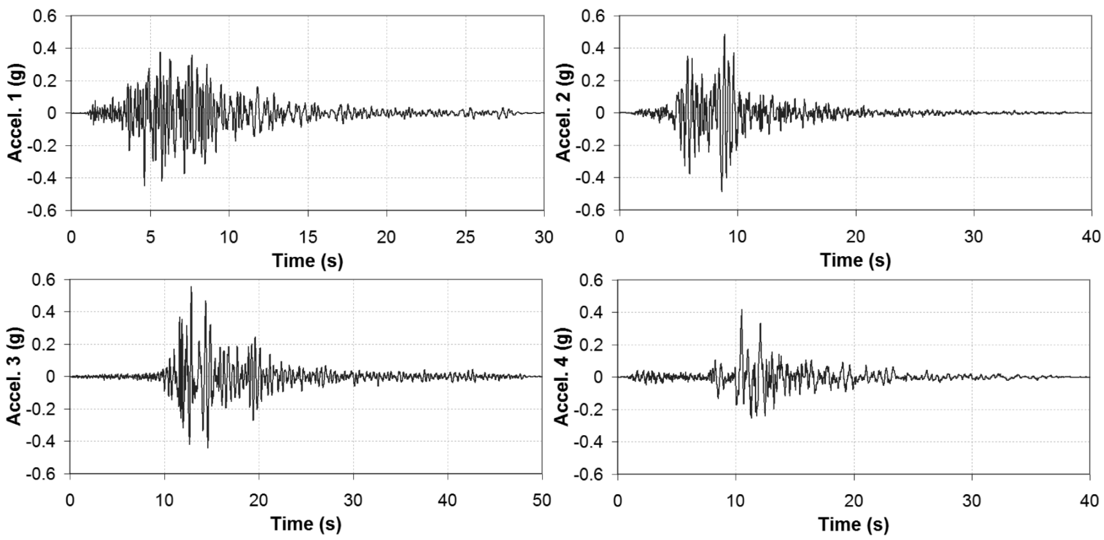


Figure 19.- Comparison of the distribution of PGA at the ground surface considering all the records selected randomly (continues line) and the distribution of PGA considering a single seismic input (histogram).



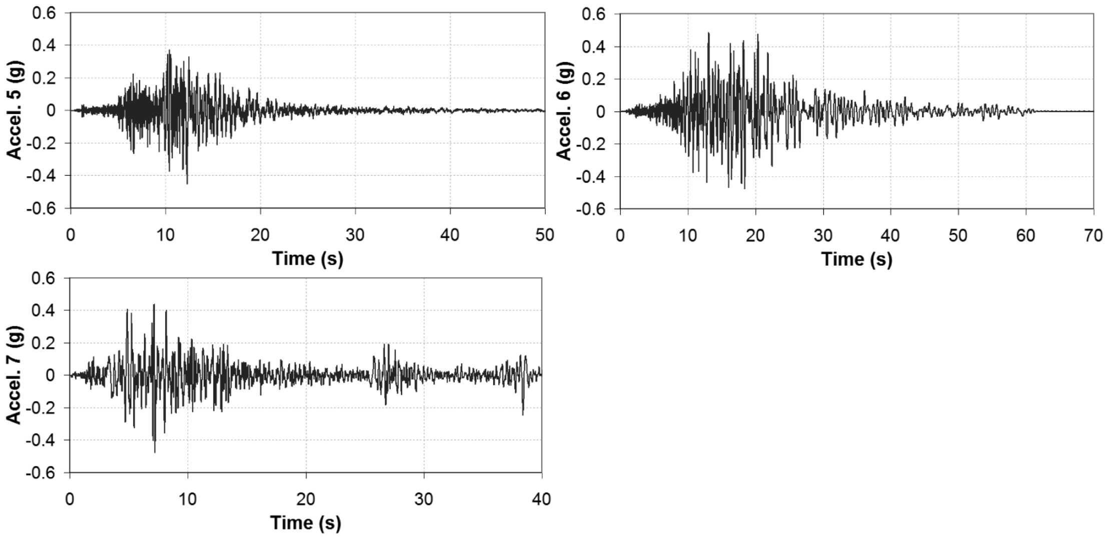


Figure 20.- Set of 7 natural records compatible with the 475-year return period.

Table 1.- Data information about the set of real records obtained from SEISM-HOME.

Return period	Group number	ID_NTC	Longitude	Latitude
475 years	1	26530	13.5580	42.3350
Magnitude (Mw)	**Epic. Dist. (km)**	**Total S.F.**	**Source file name**	
6.87	11.00	0.78	ESD 000182xa.cor	
6.68	65.00	1.16	ESD 000200xa.cor	
6.93	94.31	2.83	NGA 0797y.txt	
6.69	38.07	1.73	NGA 1091x.txt	
6.60	36.18	2.07	KNET1 SAG0010503201053.NS	
6.00	33.00	1.99	ITACA 19780415_233347ITDPC_NAS__WEC.DAT	
6.30	50.42	0.43	ITACA 20090406_013239ITDPC_SBC__WEC.DAT	

Table 2.- Comparison of the results of some tests (placed at the base of the conglomerate layer).

<table>
<tr><th>Depth (m)</th><th>DH-09
Vs (m/s)</th><th>CERFIS
Vs (m/s)</th><th>CERFIS
Soil type</th><th>SPT
Soil type</th></tr>
<tr><td>0.00 - 1.00</td><td>149</td><td rowspan="12">319</td><td rowspan="12">Sandy silt</td><td rowspan="2">Sandy silt with vegetable</td></tr>
<tr><td>1.00 - 1.40</td><td rowspan="2">285</td></tr>
<tr><td>1.40 - 3.00</td><td rowspan="2">White & gray silt</td></tr>
<tr><td>3.00 - 4.00</td><td rowspan="2">321</td></tr>
<tr><td>4.00 - 5.00</td><td rowspan="9">Consolidated clayey silt</td></tr>
<tr><td>5.00 - 7.00</td><td>333</td></tr>
<tr><td>7.00 - 9.00</td><td>379</td></tr>
<tr><td>9.00 - 11.00</td><td>325</td></tr>
<tr><td>11.00 - 13.00</td><td>357</td></tr>
<tr><td>13.00 - 15.00</td><td>329</td></tr>
<tr><td>15.00 - 16.00</td><td rowspan="2">395</td></tr>
<tr><td>16.00 - 17.00</td></tr>
<tr><td>17.00 - 17.50</td><td rowspan="2">494</td><td rowspan="4">467</td><td rowspan="4">Clayey silt</td></tr>
<tr><td>17.50 - 19.00</td><td rowspan="2">Consolidated clay (gray colour)</td></tr>
<tr><td>19.00 - 20.30</td><td rowspan="2">441</td></tr>
<tr><td>20.30 - 21.00</td><td rowspan="4">White silt with calcareous fragments</td></tr>
<tr><td>21.00 23.00</td><td>398</td><td rowspan="6">413</td><td rowspan="6">Silt</td></tr>
<tr><td>23.00 - 23.50</td><td rowspan="4">398</td></tr>
<tr><td>23.50 - 24.00</td></tr>
<tr><td>24.00 - 24.20</td><td rowspan="6">Limestone fragments and calcarenitic</td></tr>
<tr><td>24.20 - 25.00</td></tr>
<tr><td>25.00 - 27.00</td><td>442</td></tr>
<tr><td>27.00 - 29.00</td><td>568</td><td rowspan="3">533</td><td rowspan="3">Calcarenitic</td></tr>
<tr><td>29.00 - 29.60</td><td rowspan="2">498</td></tr>
<tr><td>29.60 - 30.00</td></tr>
</table>

Table 3.- Evaluation of the σ for the Vs based on the MASW-1.

	Vs min (m/s)	Vs mean (m/s)	Vs max (m/s)	σ (m/s)	CoV (%)
Layer $_{MASW-1}$ = 1	105	150	196	15.17	10.11%
Layer $_{MASW-1}$ = 2	245	348	450	34.17	9.82%
Layer $_{MASW-1}$ = 3	314	447	580	44.33	9.92%
Layer $_{MASW-1}$ = 4	419	598	776	59.50	9.95%
Layer $_{MASW-1}$ = 5	-	1491	-		

Table 4.- Evaluation of the σ for the layer thickness based on the MASW-1.

	Thickness min (m)	Thickness mean (m)	Thickness max (m)	σ (m)	CoV (%)
Layer $_{MASW-1}$ = 1	0.7	1.05	1.4	0.12	11.11%
Layer $_{MASW-1}$ = 2	10.35	14.88	19.41	1.51	10.15%
Layer $_{MASW-1}$ = 3	5.55	7.96	10.36	0.80	10.08%
Layer $_{MASW-1}$ = 4	3.52	4.97	6.42	0.48	9.73%
Layer $_{MASW-1}$ = 5	-		-		

Table 5.- Proposal soil profile for the study zone and its mean values.

Layer	Type	Depth (m)	Thickness (m)	CoV (%)	Vs (m/s)	CoV (%)	γ (kN/m^3)	CoV (%)	Mat
I	Conglomerate	4	4	15	400	15	20	5	Mat1
II	Sandy silt	21	17	15	341	15	19.8	5	Mat2
III	Clayey silt	25	4	15	467	15	20.3	5	Mat2
IV	Silt	31	6	15	413	15	20.3	5	Mat2
V	Gray clayey silts and calcarenitic	54	23	15	600	15	20.8	5	Mat2
VI	Transition	59	5	15	800	15	21.0		Mat3
VII	Bedrock	After 59 m	-	-	1250	15	22.0	5	Mat4