Seismic safety assessment for Architectural Heritage in Italy: a procedure for local response spectra determination.

A. Cividini[a]*, F. Pergalani[b], M.A. Parisi[a], M. Compagnoni[b] and C. Tardini[a]

[a]*ABC, Politecnico di Milano, Italy*

[b]*DICA, Politecnico di Milano, Italy*

*annamaria.cividini@polimi.it

Abstract

The definition of a seismic action for the structural analysis of existing buildings generally results in the selection of a response spectrum. The Italian design code offers a basic procedure to define a reference spectrum that includes amplification effects from site conditions. However, when considering architectural heritage buildings, a more accurate determination of the local spectrum could be of interest, given the need to effectively protect these assets.

The main aspects of the procedure adopted for the evaluation of local response spectra are recalled, and results are presented for three sites having different stratigraphic conditions. Finally, the results from the commonly used, code-defined procedure and from the one based on the local geophysical investigation are compared.

Keywords: acceleration response spectrum, architectural heritage, geophysical investigation, seismic assessment procedure, amplification phenomena, structural design code

1. Introduction

A large part of the Italian territory is subject to earthquakes and classified with different levels of seismicity. The Italian structural design code (NTC 2008), mentioned as “NTC code” in the following, covers the earthquake resistant design of new buildings as well as the strengthening of existing ones.

The seismic response of existing buildings may be adequately estimated with proper modeling both of the nonlinear behavior of structural elements and foundations (e.g. Cividini and Gioda 2017) and of the local seismic motion expected for an assigned return period, RP. According to the NTC code, the seismic action is defined by the response spectrum to be adopted for the site at ground level.

In order to construct such spectrum starting from the national seismic hazard map (MPS 2004), which supplies data referred to bedrock, the NTC code proposes a conventional procedure, indicated as “standard” in the following, to account for amplification effects from site conditions, including both stratigraphy and topographic irregularities.

Soil stratigraphy is considered, by performing a classification of ground deposits from rocky to increasingly softer soils, based on the estimated shear wave velocity, similarly to other International codes like Eurocode 8 (EN 1998-1 2004). Site dependent amplification factors for defining the response spectrum are assigned depending on the ground types.

During the many earthquakes that arrived in Italy in the last decades, often in a mountainous or hilly area, topographic irregularities produced strong and highly damaging amplifications. Consequently, an additional amplification factor has been introduced to cover topographic effects; the peak ground acceleration, PGA, increases up to 40 percent as a function of the ground slope or when the building is on a ridge.

The NTC code allows as well to adopt more accurate procedures for the determination of the local spectrum encompassing geophysical investigation (e.g. Pergalani 2015; Monaco and Amoroso 2016), which may be more appropriate in many situations, like those relevant to the safety assessment of architectural heritage. Such buildings, often centuries old and erected according to traditional construction rules, may be affected by significant seismic vulnerability and may not satisfy current safety requirements. At the same time, strict conservation principles are in effect for heritage assets, limiting the extension and modality of the interventions that may be performed for improving their seismic response. An accurate definition of the seismic action to be considered when assessing seismic safety will bring to better calibrated interventions. The situations of inadequate improvement, resulting from adopting a standard NTC code spectrum while the actual local response is higher, or, vice-versa, practicing unnecessary interventions if the real local response is lower than estimated, are avoided.

This study focuses on the determination of local response spectra and on the effect of the standard procedure accuracy on the global evaluation of seismic safety, within a broader work devoted to the definition of procedures suitable for assessing the seismic safety of listed architectural heritage.

A connotative aspect of dealing with cultural heritage assets is complexity and, consequently, the need of intertwining contributions from different disciplines, with experts collaborating in a synergic modality rather than in the more common process of carrying out their specific task and passing on the results for the next step. This work aims to testify the positive effect of interdisciplinarity and peer interaction, often disregarded, in local site response characterization.

The main characteristics of the general research project and the procedure adopted for evaluating the local seismic response, LSR, are outlined. The steps of the procedure are exemplified referring to three museums hosted in historical buildings at sites that present different local conditions.

2. The context of seismic safety assessment

The need to assess the seismic safety of a large number of existing buildings is a challenge that many countries need to face. In Italy, a large fraction of the building stock is composed of masonry structures, some of which were built centuries ago according to heuristic, good practice rules. The NTC 2008 Code that has been the base for this work devotes a large section to seismic evaluation and strengthening of existing buildings, with particular focus on masonry. This code has been updated very recently (NTC2018, 2018), but modifications do not concern the parts used as reference in this work. The code aims at protecting common buildings in general. Existing buildings listed as cultural heritage, however, are unique, may host contents of particular value, and are often subjected to intense public use. Specific guidelines, indicated as "Guidelines" in the following, have been issued by the Ministry of Cultural Heritage, MiBACT, (MiBACT 2011) to supplement the general NTC code for the seismic protection of cultural heritage assets.

2.1 Guidelines for seismic safety of cultural heritage

The proposal of guidelines for dealing with different issues related to the seismic protection of architectural heritage stemmed from the need to harmonize Internationally recognized restoration principles that are intended to preserve the original integrity of heritage assets with a modern vision of structural safety that may require modifications of their original or current structural condition. The document issued by MiBACT, after explaining to the professional community the rationale and the consequences of special treatment for these structures, outlines good practice procedures aimed at satisfying both demands.

The Guidelines indicate a path for acquiring knowledge on the building and on the building site, which includes contributions from different experts, acknowledging the interdisciplinary character of the problem of heritage protection and conservation.

The path is defined by a list of steps to be taken in order to collect information on the asset and finally be in condition to formulate an educated judgement on its seismic safety level. Steps are very diverse, as they cover different facets composing a complete knowledge of the building. The main objective is to reach a satisfactory confidence level in the data for structural analysis in spite of the limitations applying to these cases.

The first step concerns precisely the site characterization and local seismic response, which is the particular focus of this work; a totally different kind of information to be acquired is related to the history of the construction (e.g. Cantini et al. 2016a). Locating its origins in a particular historic period, associated with the relevant constructional practice then in use, and following the development of structural modifications performed during the building life so far as a consequence of new social needs, new destination of use, or simply maintenance and repair, is extremely rewarding in terms of knowledge acquired on the state of the building, but at times rather difficult to collect and interpret. Subsequent steps are related to accurate geometric survey of the building and its rendering in architectural models, the survey and interpretation of possible crack patterns, the mechanical characterization of construction materials with the least invasive testing methods possible, up to the inspection and evaluation of service plants and their possible interference with the structural response (e.g. Cantini et al. 2016b). The final part of the path develops structural modeling and analysis. This is a main goal of the entire path. All the collected information contributes to develop significant geometrical and mechanical models for these complex structures. The final safety level is estimated from structural analysis. The capacity, which may be typically expressed as the structure's limit acceleration, is compared to the demand, in terms of reference acceleration from the spectrum; hence, safety indices may be derived. Yet a more thorough safety assessment is possible considering also the critical issues evidenced along the entire path that cannot be inserted in the general numerical model, either because related to non-structural aspects, or for the modeling difficulties involved. This entire picture, which stems from collaboration of different expertise, allows a comprehensive appreciation of the state of the building.

2.2 The research project

The analysis of local seismic response presented here is part of a research program of the Ministry of Cultural Heritage on the seismic safety of Italian museums. According to the project, a series of buildings were attributed to different research teams nationwide. The teams analyzed their set of buildings according to the Guidelines (MiBAC 2011). The goal was twofold, that is, to

1. acquire an in-depth knowledge of the seismic vulnerability of the buildings;
2. test and calibrate different analysis procedures indicated in the Guidelines by comparison and discussion among the different teams.

In the context of the research program, the definition of the local response spectrum through a geophysical analysis versus a standard one, as discussed in the following, becomes fundamental, as the development of this research has shown (Parisi et al. 2016).

Analogously, structural analyses were to be carried out following different methods, that is,

- considering synthetic procedures to assess the global shear capacity of the structure,
- checking the possibility of local failure of parts of the masonry body as a kinematic chain by means of limit equilibrium method in the so-called analysis of mechanisms (e.g. D'Ayala and Speranza 2003; Lagomarsino 2006), and finally
- assessing the dynamic structural characteristics by modal analysis, and
- analyzing the global response and the progression of post-elastic deformations by means of a static nonlinear "pushover" analysis, conducted according to Fajfar (2000); this choice is in line with current recommendations that recognize this analysis as the most suitable for a viable and sufficiently reliable evaluation of the ultimate seismic capacity of existing masonry buildings. Recourse to nonlinear dynamic analyses is ruled out in practice, being extremely cumbersome for these cases and affected by several uncertainties that blur the expected accuracy of the method; therefore, it was not addressed in the program.

Each of the analyses listed above is meant for a particular purpose and is associated to specific approximations (e.g. D'Ayala and Lagomarsino 2015). When a global capacity value will be used for a first assessment of strength, a relatively simple structural model may be used. Here the global shear value was estimated, according to the Guidelines, accounting for the shear-resisting areas of the main walls in two orthogonal horizontal directions; torsional effects caused by eccentricity between the centers of mass and stiffness, as well as other irregularities due to geometry or materials that may affect the response, are included by means of modification factors. This simplified approach, which does not require a detailed model of the structure, is particularly useful for comparing assets when performing a general assessment of vulnerability of the building stock in a territory. Yet, in the course of the research project, it has become clear that this first evaluation of capacity has been meaningful and very useful for a first overview and classification of the structural behaviour, to be examined in depth and confirmed by subsequent analyses.

The pushover analysis, which is basically a static nonlinear analysis for monotonically increasing horizontal loads distributed according to the most significant modal shape, allows a more refined evaluation of the global structural behavior and it is widely recognized as particularly suitable to detect the ultimate response conditions of existing structures, both in terms of strength capacity and of the associated post-elastic displacements. The structure may be modeled by finite elements or, when applicable, with a simpler equivalent frame model. The field of applicability and significance of this method, related to many building characteristics, is well known and its specification goes beyond the scope of this work. It is worth confirming that for the cases examined the method was deemed in condition to supply a significant description of the structural response.

The search of possible damage mechanisms that involve only a limited part of the structure with little interference with the rest and the corresponding evaluation of the collapse limit situation by means of a kinematic limit analysis is necessary in masonry structures. Earthquake damage observations have shown that localized damage, which nonetheless may be extremely dangerous and devastating, is extremely frequent. Determining the lateral force level, and the associated acceleration for which local collapse may be expected, indicates whether local mechanisms may develop and which are the most likely. The purpose of this analysis is to supply a basis for planning possible local strengthening interventions.

All these structural analyses require reference to the relevant response spectrum, in order to compare the resistance capacity with the demand and to assess the corresponding safety level. It is worth mentioning that in Eurocode 8, as in other codes, the reference spectrum usually corresponds to a standard return period of 475 years, and the seismic action intensity is subsequently increased in the case of building categories of particular importance or large public use. The Italian code defines initially the reference return period according to the importance and use category of the building itself and derives a response spectrum on this basis, drawing from the available seismic hazard database that contains corresponding data for the national territory for a variety of return periods. The procedure is explained in detail in the following sections. For ordinary buildings, the resulting return period is the common case of 475 years, corresponding to a probability of exceedance of 10 percent in a nominal life of 50 years according to a Poisson's model of earthquake recurrence. In the special case of museums, due to the importance of the building and the more extended public use, non-standard return periods have been initially selected in agreement with MiBACT for all the case studies included in the research program. The selection was shared by different groups in order to make comparisons possible among the cases studied, ranking the buildings by vulnerability. The return period associated to the ultimate limit state has been fixed to 712 years rather than the more common 475, considering a more extended reference life for these buildings in their quality of heritage assets; additional limit states and relevant return periods considered have been: 30 years, that is typical of the operational limit state connected to building plants, and 201 years for the damage limit state that controls maximum displacements within an elastic behavior. This value of return period is particularly demanding, as the displacement limit state is commonly checked for a much lower one and, therefore, for a milder seismic action. However, the selection was due to the need to set a high level of protection for artistic contents of the museums, some of which may be particularly sensitive to structural displacements.

2.3 The case studies

Three buildings are examined here. The three areas are located in southern Italy, two of which in Napoli (buildings indicated in the following as PB and MP) and the third one in the small town of Sala Consilina (AM building). The seismic history of the municipalities of Napoli and Sala Consilina is summarized by the diagrams in Figure 1, showing the intensity of seismic events that occurred in the last 800 years. The data in the diagrams are available in the Italian Macroseismic Database DBMI11 (Locati et al. 2011).

The building locations are shown in Figure 2, together with the map of seismic hazard, that provides a first general view of the situation.

The buildings have different origins, namely: PB is a rather common late-18th century building in tuff stone masonry (Figure 3a), located on the top of a large hill within the precincts of the Royal Manor of Capodimonte (Figure 3b), now a national museum. It is part of the subsidiary buildings of the main manor; about half of the building had been torn down during a World War II bombing and was reconstructed with a reinforced concrete frame, posing an interesting problem of compatibility of parts in the dynamic behaviour. The building denoted as MP is a 19th century villa constructed in tuff masonry as family residence by a nobleman in Napoli (Figure 4); it is located near the seashore, at the base of the hill of Vomero, that is densely inhabited. The third building, AM, is an ancient convent that was built in limestone masonry on a hillside at the end of the 16th century; after many modifications and changes of use, it was recently refurbished and retrofitted for use as a museum (Figure 5).

The main steps of a geophysical analysis used in order to define the seismic action for these three case studies are outlined in the next section. All the cases presented different stratigraphic and topographic conditions that indicated them as possible candidates for a local response analysis. A comparison between the results from the standard code procedure and from the complete geophysical analysis reveals significant differences, showing that in many situations the commonly used procedure for spectrum definition could either overestimate or, more dangerously, underestimate the local amplification effects.

3. Definition of the seismic action according to the standard procedure

According to the Eurocode 8, the earthquake motion on the surface can be basically represented by an elastic ground acceleration response spectrum; the NTC code offers two possibilities for the definition of the response spectrum.

The first, standard one, is a simplified procedure while according to the second possibility the definition of the elastic ground acceleration response spectrum is obtained by means of a local response analysis. Its characteristics main steps, and the data relevant to the investigated building sites, are reported in Section 4.

Both procedures require the definition of the seismic hazard at the rock base. The Italian seismic hazard map was considered (http://esse1.mi.ingv.it/). For each building site, the values of the reference horizontal ground acceleration (a_g) for each considered return period are indicated in Table 1. Note that the values are referred to the bedrock (i.e. subsoil A, characterized by average value of propagation velocity of shear waves V_{S30} greater than 800 m/s in the upper 30 m of the soil profile) and are obtained as weighted average of the values available at the four surrounding nodes of the reference grid (step 0.05° both in latitude and longitude) provided by the database of seismic hazard in Italy (MPS 2004).

Table 1. Values of reference acceleration at the bedrock a_g for the PB, MP and AM building sites and for the three different return periods.

return period RP (years)	30	201	712
	a_g (g)	a_g (g)	a_g (g)
PB and MP sites (Napoli)	.044	.120	.192
AM site (Sala Consilina)	.053	.148	.272

When adopting the standard procedure, the response spectrum at the bedrock (or characteristic response spectrum) is immediately available, since its significant parameters are provided in the database MPS (2004). For the return period, RP = 712 years these spectra are shown by black dots in Figure 6, where the acceleration values at period T = 0 s correspond to the data in Table 1.

Then the response spectrum at the foundation level can be obtained, accounting for amplification effects from site conditions including stratigraphy and topographic irregularities. It can be observed that apart from latitude and longitude of the building, only the average shear wave velocity is required for the evaluation of the response spectrum according to the standard procedure.

4. Analysis of local seismic response

The Local Seismic Response, LSR, analysis consists of an interdisciplinary study, aiming at assessing changes in amplitude, frequency and duration of ground shaking due to the specific litho-stratigraphic and morphological conditions that characterize the area under investigation. The objective is to evaluate the seismic motion expected for a given return period, at the ground surface or at a specific reference area (e.g. the foundation level). The evaluation requires the geological, geotechnical and seismic characterization of the site, an a prior assumption on the expected seismic motion at the bedrock for a given return period and a numerical investigation.

The analysis allows consequently to quantify the overall/global seismic amplification effects that characterize the site under investigation through a comparison between the seismic motion, in terms of acceleration versus time history and in terms of elastic acceleration response spectra for a selected value of critical damping, at the investigated area and at the bedrock.

With reference to the three sites considered here, the LSR analyses are characterized by five main steps:

1. definition of the reference events;
2. evaluation of the geological and geomorphological condition in the area;
3. acquisition of geotechnical and geophysical data;
4. derivation of the stratigraphic scheme of the soil column;
5. numerical analysis for the evaluation of the seismic amplifications.

In the following, the main aspects, the essential steps and the obtained results of the LSR approach are presented.

4.1 Reference events

In order to carry out the numerical modeling for the evaluation of the local seismic effects, it was necessary to refer to available recorded accelerograms to be used as input in the subsequent modeling. In particular, seven recorded accelerograms were selected from ITACA database (Pacor et al. 2011) and scaled, as required in the NTC code. As suggested by the same code, the choice was based on:

– type of the source, which is mainly normal fault for all the considered sites;
– maximum expected horizontal acceleration;
– results of the disaggregation analyses performed by a National Working Group in 2004 (MPS 2004) that allow to have information about the characteristics, in terms of magnitude and epicentral distance, of the event contributing most to the seismic hazard assigned to the investigated area;
– recordings on the seismic bedrock, corresponding to the type A subsoil, as defined in the NTC code as well as in Eurocode 8;
– spectrum similarity of the real recording spectrum with the characteristic response spectrum (or reference spectrum) for subsoil A available on the basis of the NTC code (see Section 3 and Figure 6).

All the selected recordings were scaled, in order to optimize the spectrum-compatibility of the target response spectrum, limiting the differences in terms of spectral ordinates within 10% in the period interval between 0.15 seconds and 2T, where T is the dominant vibration period of the analyzed system.

Table 2 presents the main characteristics for eight seismic events considered in preparing the set of seven acceleration-time series for each one of the three considered return periods and for each one of the investigated sites. In particular, name, date and time of the event, geographical coordinates of the earthquake epicenter, the depth, the moment magnitude Mw, the local magnitude ML and the tectonic characteristic are indicated in the table.

In the present case, the events and the acceleration-time recordings, having characteristics in accordance with code criteria, were directly chosen by the authors; however the software REXELite (Iervolino et al. 2009; 2011) could be used for selecting automatically in the ITACA strong motion database the records necessary for the subsequent seismological and structural analyses.

For completeness, for each considered return period some characteristics of the seven records selected for Napoli and for Sala Consilina are listed in Tables 3 and 4, respectively, and include code, name and coordinates of the recording station, its distance from the earthquake epicentre, event name and date, component of the horizontal accelerogram and its peak ground acceleration and finally the scale factor adopted in the present LSR analyses.

Note that the same recorded accelerogram with different scale factor could be included in the set relevant to each return period and that the EW and NS components of an event have been used, as independent element, to obtain the target spectrum.

For the return period of 712 years, the acceleration response spectra at the bedrock for the seven selected accelerograms are shown in Figure 6a for PB and MP sites in Napoli and in Figure 6b for the AM site in Sala Consilina. In addition, the average of the seven curves, represented by a red solid line in the figures, allows to verify the spectrum similarity with the corresponding reference spectrum (black dotted line) available from the NTC code.

Table 2. Main characteristics of the selected seismic events.

Event name	*Date*	*Time*	*Lat (°)*	*Lon (°)*	*Depth (km)*	*Mw*	*ML*	*Tectonic char.*
Irpinia-1	23/11/1980	18:34:53	40.76	15.31	15.0	6.9	6.5	Normal fault
Irpinia-2	01/12/1980	19:04:29	40.88	15.31	9.0	-	4.6	Normal fault
Pescopagano	16/01/1981	0:37:45	40.84	15.44	10.5	5.2	4.6	Normal fault
Ap. Lucano	09/09/1998	11:28:00	40.06	15.95	29.2	5.6	5.6	Normal fault
L'Aquila-1 a	07/04/2009	9:26:29	42.34	13.39	9.6	5.1	4.8	Normal fault
L'Aquila-1 b	07/04/2009	17:47:37	42.30	13.49	17.1	5.5	5.4	Normal fault
Gran Sasso	09/04/2009	0:53:00	42.49	13.35	11.0	5.4	5.1	Normal fault
Mormanno	25/10/2012	23:05:24	39.88	16.01	6.3	5.3	5.0	Normal fault

Table 3. Main characteristics of the records selected for LSR analysis at the PB and MP building sites in Napoli.

ID	*Code - Station*	*Lat (°)*	*Lon (°)*	*Epicentral distance (km)*	*Event Name, Date*	*Comp.*	*PGA (g)*	*Scale factor*
Return Period RP = 30 years								
Acc1	SLA - Sant'Angelo dei Lombardi	40.93	15.17	24.5	Pescopagano, 16-01-1981	EW	- 0.025	1.0
Acc2	SLA Sant'Angelo dei Lombardi	40.93	15.17	24.5	Pescopagano, 16-01-1981	NS	0.022	1.0
Acc3	OPB - Oppido Balzata	40.87	15.21	8.7	Irpinia-2, 01-12-1980	EW	- 0.084	1.0
Acc4	OPB - Oppido Balzata	40.87	15.21	8.7	Irpinia-2, 01-12-1980	NS	0.058	1.0
Acc5	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	EW	0.116	1.4
Acc6	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	NS	- 0.116	0.4
Acc7	AQP - L'Aquila	42.38	13.37	5.5	L'Aquila 1 a, 07-04-2009	EW	0.098	0.5
Return Period RP = 201 years								
Acc1	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	EW	- 0.083	0.8
Acc2	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	NS	- 0.096	0.8
Acc3	ALT - Auletta	40.55	15.39	23.8	Irpinia-1, 23-11-1980	EW	0.057	1.5
Acc4	ALT - Auletta	40.55	15.39	23.8	Irpinia-1, 23-11-1980	NS	0.056	2.0
Acc5	VGG - Viggianello	39.96	16.05	13.5	Ap. Lucano, 09-09-1998	EW	- 0.070	2.0
Acc6	VGG - Viggianello	39.96	16.05	13.5	Ap. Lucano, 09-09-1998	NS	0.073	1.5
Acc7	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	NS	- 0.116	2.5
Return Period RP = 712 years								
Acc1	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	EW	- 0.083	1.0
Acc2	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	NS	- 0.096	1.0
Acc3	ALT - Auletta	40.55	15.39	23.8	Irpinia-1, 23-11-1980	NS	0.056	4.0
Acc4	VGG - Viggianello	39.96	16.05	13.5	Ap. Lucano, 09-09-1998	EW	- 0.070	4.0
Acc5	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	NS	- 0.116	4.0
Acc6	AQP - L'Aquila	42.38	13.37	13.2	L'Aquila 1 b, 07-04-2009	NS	0.076	3.0
Acc7	AQP - L'Aquila	42.38	13.37	11.8	Gran Sasso, 09-04-2009	EW	- 0.071	2.0

Table 4. Main characteristics of records selected for LSR analysis at the AM building site in Sala Consilina.

ID	*Code - Station*	*Lat (°)*	*Lon (°)*	*Epicentral distance (km)*	*Event Name, Date*	*Comp.*	*PGA (g)*	*Scale factor*
Return Period RP = 30 years								
Acc1	SLA - Sant'Angelo dei Lombardi	40.93	15.17	24.5	Pescopagano, 16-01-1981	EW	- 0.025	1.0
Acc2	SLA Sant'Angelo dei Lombardi	40.93	15.17	24.5	Pescopagano, 16-01-1981	NS	0.022	1.0
Acc3	OPB - Oppido Balzata	40.87	15.21	8.7	Irpinia-2, 01-12-1980	EW	- 0.084	1.0
Acc4	OPB - Oppido Balzata	40.87	15.21	8.7	Irpinia-2, 01-12-1980	NS	0.058	1.0
Acc5	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	EW	0.116	0.8
Acc6	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	NS	- 0.116	1.0
Acc7	AQP - L'Aquila	42.38	13.37	5.5	L'Aquila 1 a, 07-04-2009	EW	0.098	1.5
Return Period RP = 201 years								
Acc1	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	EW	- 0.083	1.0
Acc2	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	NS	- 0.096	1.0
Acc3	ALT - Auletta	40.55	15.39	23.8	Irpinia-1, 23-11-1980	EW	0.057	1.5
Acc4	ALT - Auletta	40.55	15.39	23.8	Irpinia-1, 23-11-1980	NS	0.056	1.5
Acc5	VGG - Viggianello	39.96	16.05	13.5	Ap. Lucano, 09-09-1998	EW	- 0.070	3.0
Acc6	VGG - Viggianello	39.96	16.05	13.5	Ap. Lucano, 09-09-1998	NS	0.073	2.5
Acc7	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	NS	- 0.116	3.5
Return Period RP = 712 years								
Acc1	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	EW	- 0.083	3.0
Acc2	BSC - Bisaccia	41.01	15.37	28.3	Irpinia-1, 23-11-1980	NS	- 0.096	3.0
Acc3	ALT - Auletta	40.55	15.39	23.8	Irpinia-1, 23-11-1980	NS	0.056	4.0
Acc4	VGG - Viggianello	39.96	16.05	13.5	Ap. Lucano, 09-09-1998	EW	- 0.070	4.0
Acc5	0LAI - Laino	39.95	15.97	8.5	Mormanno, 25-10-2012	NS	- 0.116	4.5
Acc6	AQP - L'Aquila	42.38	13.37	13.2	L'Aquila 1 b, 07-04-2009	NS	0.076	4.0
Acc7	AQP - L'Aquila	42.38	13.37	11.8	Gran Sasso, 09-04-2009	EW	- 0.071	4.0

4.2 Geological and geomorphological structure

In the following, the main geological and geomorphological characteristics (Geological Survey of Italy 2010) are briefly summarized for the three building sites considered in this study.

4.2.1 PB building site

The building is located in Capodimonte (Napoli). The area consists of a slab of rigid tuff, covered with loose pyroclastic materials of variable thickness. The grain size of the pyroclastic materials is not homogeneous but varies from silty sand to sandy silt, with a limited gravel component consisting of pumice and rock fragments.

Below the pyroclastic deposits there are deposits of “Sintema Vesuviano-Flegreo” and in particular the “Neapolitan Yellow Tuff” (“tufo giallo napoletano”, TGN), which outcrops occasionally around the site. The TGN has two distinct components: the first one, with maximum thickness of 10 m, consists of volcanic ash and levels of lapilli, the other, having greater thickness, is formed by volcanic lithoid or loose ash.

4.2.2 MP building site

The area, located at the sea coast (Riviera di Chiaia, Napoli), is almost at sea level, where the submerged deposits consist of pyroclastic sandy gravel and sand from coarse to medium, well graded, with relevant amount of quartz and feldspar, mixed with pumice and rounded bio-clasts, without pelitic matrix. On the other hand, the pelitic matrix increases in areas protected from wave action and towards the external limit of the coastline. Below the beach deposits, the materials of the “Sintema Vesuvio-Flegreo” are found, and in particular the TGN, outcropping upstream of the site.

4.2.3 AM building site

The site is located at the eastern side of the “Vallo di Diano”, an intermountain basin extending in the NW to SE direction, generated by tectonic causes related to the presence of a regional structural fault (Line of *Vallo di Diano*).

The slope, presenting an average inclination of about 15 degrees, is characterized by a series of flat man-made terraces. The area is nowadays stable since the presence of active and potential instability or of particular erosion phenomena were not detected. The slope is characterized by a colluvial deposit, decametric in thickness, resting on the carbonate bedrock, which is more or less fractured, and it is part of the “Campano-Lucana” carbonate platform.

Note, however, that the site is located in an active deposit, between two alluvial fans. The deposit is characterized by a superficial layer, more or less thin, that consists mainly of colluvial material with relevant fine matrix, widely outcropping in the area, and resting on a Holocene colluvial layer. This layer, outcropping along the edge of the site, is in turn on a layer of Upper Pleistocene gravel, not outcropping in the area, resting on a dolomitic bedrock.

4.3 Geotechnical and geophysical data

Some already available geotechnical investigations were used to define the geotechnical characteristics of the deposits in the building area, while the geophysical characteristics of the sites were investigated in the proximity of the three buildings.

At the time of construction of the three buildings few geotechnical data, if any, were available on the foundation subsoil. Subsequently the planning of rehabilitation works (in 1958 for Capodimonte Manor, close to the PB building, in 1990 and 2010 for the MP villa and in 2003 for AM museum) required the re-analysis of the bearing capacity of the foundations and to this purpose some geotechnical investigations were carried out, consisting of borings, SPT standard penetration and DPM continuous dynamic penetrometric tests and of laboratory tests on the recovered samples to evaluate the mechanical resistance of the layers.

The series of borings were carried out with continuous core recovery for identifying the soil profile. As for the laboratory investigation, the quantitative data available, beside the soil profile, consist of the results of gran-size analyses and of some direct shear tests on the samples recovered from the upper soil layers.

In the PB and AM areas, the presence of the water table at depth of interest was not detected, while for the MP building area the water table is located at a depth of 5-6 m below the ground level.

The schematic representation of the soil layers in the three building areas and the corresponding mechanical parameters from the geotechnical investigation are not presented here for sake of brevity but contributed to the identification of the soil stratigraphy discussed in Section 4.4.

The geophysical tests, adopted according to the Italian guidelines (Associazione Societa' di Geofisica 2010) and carried out in the PB, MP and AM building sites, are as follows:

- Horizontal to Vertical Spectral Ratio, HVSR (SESAME Project, 2004; Nogoshi and Igarashi, 1970; Nakamura 1989; Konno and Ochamachi 1998; www.geopsy.org for the software);
- Electric lines, ERT (ASTM D5777:2011; Groot-Hedlin and Constable 1992; Sasaki 1992; www.geotomosoft.com for the software);
- Seismic Refraction in longitudinal waves, SR_P (ASTM D6431:2010; Achenbach 1999; Aki and Richards 1980);
- Multichannel Analysis of Surface Waves, MASW (Park et al. 1999; www.winmasw.com for the software).

The field investigation required the presence of three technicians and was completed in eight-ten hours at each site, while the interpretation of the geophysical data was more time consuming (approximately two days).

The geophysical tests are not described here being of more conventional type; only some results are presented in Figures 7, 8 and 9 and briefly commented in the following.

In the building area, the ambient vibration (or microtremors) at the ground surface were recorded for about one hour using a single observation station. In the PB and MP sites, the HVSR seismometer was placed in more than two locations to verify the geological and litho-stratigraphical variability in the areas. This preliminary investigation that can be applied even in areas of low or moderate seismicity allows to evaluate the variation of the spectral ratio between horizontal H and vertical V components of the recorded vibration data. Then the fundamental natural frequency f_0 of the soil deposit (corresponding to the peak values of the H/V ratio) is identified. The values at each measuring location and the mean values for each site are shown in Table 5.

The low variability of the natural frequencies indicates uniform litho-stratigraphy in the investigated site, while there is marked difference in the values at PB and MP sites characterized by different geological conditions, even if they are part of the same municipality.

Table 5. H/V peak frequency f_0 from HVSR tests.

point	1	2	3	4	5	6	mean value
	f_0 (Hz)	f_0 (Hz)	f_0 (Hz)	f_0 (Hz)	f_0 (Hz)	f_0 (Hz)	**f_0 (Hz)**
PB site (Napoli)	3.0	2.1	3.7	2.8	4.0	---	**3.12**
MP site (Napoli)	2.5	2.4	1.9	2.5	2.4	2.4	**2.35**
AM site (Sala Consilina)	3.5	3.8	---	---	---	---	**3.65**

The ERT electrical resistivity measurements were carried out by means of 48 electrodes located along a line, with spacing of 4 m, 1.5 m and 2 m respectively at PB, MP and AM sites. Adopting the so-called Wenner arrangement, the back analysis of the measurements led to identify the subsoil apparent resistivity in a triangular area extending on the vertical plane through the electrode line. Taking into account the number of electrodes and their spacing, the maps shown in Figures 7a, 8a, 9a extend down to a depth of about 34 m, 13 m and 17 m, that is, to a depth equal to 8.5 times the adopted electrode spacing.

Note that (a) the maximum resistivity at the AM site (Sala Consilina) is almost one order of magnitude larger than for the other two sites located in Napoli and that (b) the data indicate different water level at the three sites. In fact, the resistivity at the PB and AM sites is lower in the upper part of the section and increases with depth, while the overall resistivity of the MP deposit is lower than in the other two cases, and low resistivity is identified also in the lower part of the investigated area, indicating the presence of water in the soil deposit.

The subsurface investigation was continued using the seismic refraction method (SR_P test) and the obtained compressional wave velocity V_P maps are shown in Figures 7b, 8b and 9b. The impulse source for ground vibrations was provided by an 8 kg hammer at the MP and AM sites, while projectile (gun) source was used in the park area close to the PB building. A set of 24 geophones, equally spaced along a line, provided the electric signal to the seismograph. Five to seven shot-points were located within the geophone line, while two were set outside each end of the line. The length of each investigated area depends, obviously, on the geophone number and spacing (namely 4 m, 2 m and 3 m at the PB, MP, and AM sites) while the depth is associated to the length of the seismic line and to the distance of the external shots from the geophones.

From the maps, it can be observed that the sub-horizontal layers are characterized by a gradual increase of the velocity values with depth. Particularly at the MP site, V_P velocity values of 1200-1300 m/s are attained about 6 m below the ground level, confirming the presence of the water table detected by the ERT resistivity tests.

The field investigation was completed by the multichannel analysis of surface waves (MASW). The tests were carried out with an impulsive source as in SR_P testing. The analysis of the field data allows to derive the phase velocity versus frequency curve (or dispersion curve, shown on the left in Figures 7c, 8c and 9c). The dispersion data then are used as input data (together with the compressional wave velocity profile previously obtained by the

SR_P test). For the subsequent steps of the measurement interpretation, an initial estimate of the unknown parameters (layer thickness and shear wave V_S velocities) is progressively modified by the iterative back calculation; at the end of the process, the profile of the shear wave velocity in the soil deposit is obtained (diagram on the right hand side in Figures 7c, 8c, 9c).

Summarizing, the preliminary geological information has allowed to assume a uniform condition in the horizontal planes. This assumption is confirmed by the information obtained from geophysical tests carried out in the proximity of the three buildings.

4.4 Soil stratigraphy

On the basis of the geotechnical data and of the results from the geophysical investigations mentioned in the preceding section, it was possible to set up the reference stratigraphy for the three investigated areas.

The results are reported in Tables 6, 7 and 8 for the PB, MP and AM buildings respectively. In the tables, *s* is the thickness of the layer (or unit), γ the natural unit weight, V_P the longitudinal wave velocity, V_S the shear wave velocity, ν the Poisson ratio and G_0 is the small strain shear modulus. Note that the elastic parameters were obtained on the basis of the longitudinal and shear wave velocity values available from the field investigation. The high values of the Poisson ratio in the MP subsoil are consistent with the presence of the water table mentioned in Section 4.3.

Table 6. Reference stratigraphy at the PB building site.

Unit	Lithology of the unit	s (m)	γ (kN/m^3)	V_P (m/s)	V_S (m/s)	ν	G_0 (MPa)
U1	Loose pyroclastic deposit	3	12	300	125	0.39	19
U2	Loose pyroclastic deposit	8	13	350	200	0.26	53
U3	Loose pyroclastic deposit	4	14	450	290	0.14	120
U4	Partially cemented pyroclastic d.	14	15	600	400	0.10	245
U5	Semi-lithoid tuff	7	17	1000	670	0.10	778
U6	Lithoid tuff	---	18	1300	850	0.13	1325

Table 7. Reference stratigraphy at the MP building site.

Unit	Lithology of the unit	s (m)	γ (kN/m^3)	V_P (m/s)	V_S (m/s)	ν	G_0 (MPa)
U1	Man-made deposit	3	16	350	100	0.46	16
U2	Fine silty sand	3	17	550	200	0.42	69
U3	Saturated fine silty sand	2	18	1400	200	0.49	73
U4	Loose coarse sand	10	18	1400	240	0.48	106
U5	Moderately thickened coarse sand	6	18	1400	380	0.46	265
U6	Coarse sand	4	18	1400	500	0.43	458
U7	Lithoid tuff	---	18	1400	750	0.30	1032

Table 8. Reference stratigraphy at the AM building site.

Unit	Lithology of the unit	s (m)	γ (kN/m^3)	V_P (m/s)	V_S (m/s)	ν	G_0 (MPa)
U1	Colluvial deposit	3	18	450	240	0.30	105
U2	Debris-colluvial deposit	7	19	550	300	0.29	174
U3	Debris deposit	16	20	900	500	0.28	509
U4	Bedrock	---	24	2000	1000	0.33	2446

The average shear wave velocity V_{S30} and then the subsoil type (NTC 2008) in the three areas are as follows:

PB building site (Napoli): V_{S30} = 263 m/s, type C subsoil

MP building site (Napoli): V_{S30} = 244 m/s, type C subsoil;

AM building site (Sala Consilina): V_{S30} = 418 m/s, type B subsoil.

Finally, for the numerical investigation aimed at evaluating the motion at seismic amplification, the shear modulus G/G_0 decay curve and the variation of damping ratio D as function of the shear strain γ are necessary.

These curves are shown in Figure 10 and are obtained as follows:

– for the pyroclastic materials at the PB site and for the coarse to medium, well graded sand of the MP site reference is made to the investigation carried out in the municipality of Napoli (ATI 1996);
– for the colluvial deposits (units U1 and U2) at the AM building site, the data are available in the database of the National Seismic Service (SSN 2004);
– and, finally, for the underlying debris deposit (unit U3) the data are obtained from Rollins et al. (1998).

4.5 Evaluation of the seismic amplification

The geological, geotechnical and geophysical investigation permitted to identify for the sites a stratigraphy characterized by horizontal layers; a revised version of the widely used one dimensional (1D) code SHAKE91 (Idriss and Sun 1992). was adopted for the numerical investigation.

The numerical analyses were repeated by considering the seven different accelerograms, for each considered return period and for each site, in order to evaluate the local amplification. For the various acceleration-time histories at the ground surface, the elastic acceleration response spectra were then evaluated, considering 5% of critical damping, in accordance with NTC code procedure.

In Figure 11 the average acceleration response spectra for the three building sites and for three return periods (red lines) are compared with the corresponding ones proposed by the code procedure (black dotted lines). It is worth noting that the difference between the spectra obtained by the two approaches at all three museum sites is significant.

In addition, on the basis of the data from the performed geophysical tests, of the available geotechnical data and of the numerical investigation, it can be observed that:

– The sites hosting the PB and MP buildings are characterized by a strong motion at the bedrock and by highly non-linear material behavior of the subsoil. Increasing the return period, a decrease in the values of the amplification factor and, consequently, of the local spectra is observed.

- Some of the accelerograms having return period equal to 201 years and 712 years lead units U2, U3 and U4 (see Tables 6, 7, 8) to conditions close to the volumetric threshold estimated for these materials, with consequent possible development of non-negligible pore pressure. The deposits in the PB site are not saturated, they will not be interested by liquefaction, but it would be possible to have a densification effect.
- On the other hand, the deposits in the MP area are saturated, so it seems possible to have instability effects in the case of RP = 712 years, and consequently, the influence of the seismic amplification effects is less relevant.
- Consequently, for the MP site, it was decided to check the occurrence of liquefaction using a simplified method (Idriss and Boulanger 2004, 2008; Juang et al. 2010). The method consists in the evaluation of the liquefaction factor, that depends on the Cyclic Resistance Ratio, CRR, and on the Cyclic Stress Ratio, CSR. On the basis of the available data from the SPT and DPM tests, the layer more susceptible to liquefaction was identified and its CRR value calculated, while the CSR value has been evaluated considering the shear stress provided by the LSR numerical analyses. In particular, for the return periods of 30 and 201 years, liquefaction is not expected, while for 712 years the probability of having liquefaction is slightly greater than zero and the phenomenon could develop in the sandy layer at a depth between 6 and 9 m.
- For the AM site, the analysis of local seismic response led to detect significant amplification for low values of the period, since the subsoil is characterized by the presence of a stiff substrate covered with thin soft deposits. In these cases, the elastic response spectra obtained by the LSR analyses are higher than the spectra proposed by the NTC code.

5. Effects on seismic safety assessment

The final step in the general project was to assess the compliance of the buildings with each of the limit states. Structural analyses were carried out according to the different methods commented in Section 2.2: a global value of resistance to lateral forces, ultimate local equilibrium analyses of elements and substructures, and global push-over analyses were performed, checking results for the return periods corresponding to the different limit states and the relevant response spectra.

The specific structural results obtained from this large set of analyses go beyond the scope of this work. In order to give a general measure of the efficacy of local spectrum characterization, the ratio of maximum spectral ordinates of the locally derived spectrum versus the standard one may be considered (Figure 11). In particular, for the return period of 712 years, which refers to life safety checking, this ratio was favorable for the two cases in Napoli, with a reduction factor of 0.71 for MP and 0.68 for PB, and very demanding, with an amplification of 1.7, for AM at Sala Consilina. Incidentally, the AM building appeared in conditions to reasonably face this situation thanks to strengthening interventions performed a few years earlier that had significantly increased its original resistance.

Masonry structures are globally rather stiff and present low natural periods. For each of the buildings the fundamental period, which dominates the response, was in the maximum amplification range, where the difference between standard spectral values and those obtained by local analysis was impressive. When considering ultimate limit states where post-elastic behavior develops, the relationship is not so direct, and other factors affecting the dynamic response should be considered. Yet, the comparison remains significant for a first framing of the situation. As a result, the indication given in the Guidelines (MiBACT 2011), which stresses the importance of an accurate determination of local spectra for heritage assets, seems extremely appropriate.

6. Concluding remarks

The seismic safety analysis of three historical buildings, conducted within a research project of the Italian Cultural Heritage Ministry, has been the occasion for an interdisciplinary work involving different disciplines including geology, soil mechanics, architectural history, material and element diagnostics, structural engineering. The general project has been carried out in close collaboration and decision sharing among experts.

A fundamental step has been the definition of the seismic action, performed by means of a local seismic response analysis, this step has seen on site the collaboration of the authors: geologists, geotechnical and structural engineers. The response spectra that were obtained showed some important differences with the code derived spectra that would be commonly applied in design practice. Specifically, for one of the sites spectra with significantly higher accelerations were derived, while the other sites showed a reduction of spectral values with respect to the standard code spectrum. These results have pointed out the importance of performing an accurate local seismic analysis when dealing with cultural heritage assets, that require enhanced safety levels and for which the retrofitting operations that may be considered compatible with the status of heritage asset are very limited. An overestimation of the action would produce excessive interventions, while underestimation could bring to unexpected damage and possibly to the loss of an irrecoverable value.

Acknowledgements

The investigations summarized in this study were carried out in the framework of research projects supported by the Italian Ministry of University and Research, the Civil Protection Department (Reluis 2014-2018), the Ministry of Cultural Heritage, Project MiBACT-ARCUS “Verifica della sicurezza sismica dei Musei Statali”.

References

Achenbach JD. Wave Propagation in Elastic Solids. Amsterdam: North-Holland Publishing, 1999.

Aki K, Richards PG. Quantitative Seismology, Theory and Methods. Vol. 1-2. NewYork: WH Freeman & Co., 1980.

Associazione Società di Geofisica. Linee Guida per indagini geofisiche (in Italian), 2014.

ASTM D6431. Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation, 2010.

ASTM D5777. Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation, 2011.

ATI, Fondedile Costruzioni s.r.l., Aquater s.p.a. *I*ndagini geologiche per l’adeguamento del PRG alla legge regionale 07-01-1983 n. 9 in difesa del territorio dal rischio sismico. 1996 (in Italian).

Cantini L, Parisi MA, Tardini C, Cardani G. The analysis and the diagnostic investigation of tuff masonry structures of a historic villa in Naples. In Brick and Block Masonry – Trends, Innovations and Challenges Conference, 2016, Padova eds. Modena C, da Porto F, Valluzzi M, London: Taylor & Francis Group 2016a:1477-1483.

Cantini L, Bonavita A, Parisi MA, Tardini C. Historical analysis and diagnostic investigations in the knowledge acquisition path for architectural heritage. In Structural Analysis of Historical Constructions – anamnesis, diagnosis, therapy, controls, 2016, Leuven eds. van Balen K, Verstrynge E, London CRC Press: Taylor and Francis Group, 2016b:166-172.

Cividini A, Gioda G. Application of the Finite Element Method to the Conservation of Old Foundations. Indian Geotech J, 2017;47(2):195-207.

D’Ayala D, Speranza E. Definition of Collapse Mechanisms and Seismic Vulnerability of Historic Masonry Buildings. Earthquake Spectra, 2003;19(3):479-509.

D’Ayala D, Lagomarsino S. Performance-based assessment of cultural heritage assets: outcomes of the European FP7 PERPETUATE project, Bulletin of Earthquake Engineering, 2015;(13):5-12.

EN 1998-1. Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. CEN European Committee for Standardization, 2004.

Fajfar P. A nonlinear analysis method for performance-based seismic design. Earthquake Spectra, 2000;16(3):573–592.

Geological Survey of Italy. CARG Project – Foglio geologico 447 Napoli, 2010.

Groot-Hedlin C, Constable S. *Occam's inversion to generate smooth, two-dimensional models form magnetotelluric data.* Geophysics, 1992;(55):1613-1624.

Idriss IM, Boulanger RW. Semi-empirical procedures for evaluating liquefaction potential during earthquakes. In 11th ICSDEE & 3rd ICEGE, eds. Doolin et al. Berkeley, 2004;(1):32-56

Idriss IM, Boulanger RW. Soil liquefaction during earthquakes. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, 2008.

Idriss IM, Sun JI. User's manual for SHAKE91, A computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits. Davies: University of California, 1992.

Iervolino I, Galasso C, Cosenza E. REXEL: computer aided record selection for code-based seismic structural analysis. Bulletin of Earthquake Engineering, 2009.

Iervolino I, Galasso C, Paolucci R, Pacor F. Engineering ground motion record selection in the ITalian ACcelerometric Archive. Bulletin of Earthquake Engineering, 2011;9(6):1761-1778.

Juang CH, Ku CS, Chen CC. Simplified model for evaluating soil liquefaction potential using CPTU. In 2nd International Symposium on Cone Penetration Testing, Huntington Beach; 2010.

Konno K, Ohmachi T. Ground motion characteristics estimated from spectral ratio between horizontal and vertical components of microtremors. Bull Seism Soc Am, 1988;88(1):228-241.

Lagomarsino S. On the vulnerability assessment of monumental buildings. Bulletin of Earthquake Engineering, 2006;(4):445-463.

Locati M, Camassi R, Stucchi M eds. DBMI11, the 2011 version of the Italian Macroseismic Database. http://emidius.mi.ingv.it/DBMI11, 2011.

MiBACT, Ministry of Cultural Heritage. Guidelines for Evaluation and mitigation of seismic risk to Cultural Heritage. DPCM 26.02.2011 Linee Guida per la valutazione e riduzione del rischio sismico del patrimonio culturale con riferimento alle norme tecniche per le costruzioni, 2011 (in Italian).

Monaco P, Amoroso S. Site effects from the building scale to the seismic microzonation scale: examples from the experience of L'Aquila. Procedia Engineering, 2016;(158):517-522.

MPS, Working Group. Redazione della mappa di pericolosità sismica prevista dall'Ordinanza PCM 3274 del 20 marzo 2003. Final report for the Civil Protection Department, INGV, http://zonesismiche.mi.ingv.it/, 2004.

Nakamura Y. A method for dynamic characteristics estimation of subsurface using microtremor on the ground surface. Quarterly Report of Railway Technical Research Institute (RTRI), 1989;30(1):25-33.

NTC2008, Italian Building Code. D.M. 14.01.2008: Norme Tecniche per le Costruzioni. Roma: Ministero delle Infrastrutture (Italian Ministry of Infrastructures), 2008 (in Italian).

NTC2018, Italian Building Code. D.M. 17.01.2018: Norme Tecniche per le Costruzioni. Roma: Ministero delle Infrastrutture e dei Trasporti (Italian Ministry of Infrastructures and Transportation), 2018 (in Italian).

Nogoshi M, Igarashi T. On the propagation characteristics of microtremor. J Seism Soc Jpn, 1970, (23):264-280.

Pacor F, Paolucci R, Luzi L, Sabetta F, Spinelli A, Gorini A, Nicoletti M, Marcucci S, Filippi L, Dolce M. Overview of the Italian strong motion database ITACA 1.0. Bull Earthquake Eng, 2011;9(6):1723–1739.

Park CB, Miller RD, Xia J. *Multichannel analysis of surface waves.* Geophysics, 1999;64(3):800–808.

Parisi MA, Della Torre S, Tardini C, Cantini L. Learning from experience: the case of Villa Pignatelli and other regional sites. In Accademia Nazionale dei Lincei, "Enhancing Resilience of historic cities to earthquakes", Roma: Bardi Edizioni, 2016;(306):497-517.

Pergalani F. La microzonazione sismica; procedure ed esempi applicativi. In: Effetti sismici locali e modelli geotecnici. In Atti della Giornata di Studio, Brescia, 20.11.2014, eds. Carini A, Sanzeni A, Brescia: Edizioni Associazione Scientifica Osvaldo De Donato, 2015:3-14 (in Italian).

Rollins KM, Evans MD, Diehl NB, Daily WD. Shear modulus and damping relationships for gravels. J Geotech Geoenviron Eng, 1998;124(5):396-405.

Sasaki Y. Resolution of resistivity tomography inferred from numerical simulation. Geophysical Prospecting, 1992;(40):453-464.

SSN Servizio Sismico Nazionale. Banca dati delle curve di degrado del modulo di taglio e del rapporto di smorzamento. CD, 2004 (in Italian).

SESAME Project. Guidelines for the implementation of the H/V spectral ratio technique on ambient vibrations. Measurements, processing and interpretation, WP12, deliverable no. D23.12, 2004.

WEB sites and Software

http://esse1.mi.ingv.it/

Software WinMASW, Eliosoft 2009 Available at: http://www.winmasw.com/ (Accessed: 12 December 2009)

Software RES2DINV, Electrical Imaging Surveys 2009. Available at: http://www.geotomosoft.com/ (Accessed: 12 April 2009)

Software open source Geopsy 2009. Available at: http://www.geopsy.org/ (Accessed: 12 March 2009)

CAPTIONS to the Figures

Fig. 1. Seismic history for Napoli (a) and Sala Consilina (b).

Fig. 2. PB, MP and AM building locations on the map of seismic hazard in Italy for return period of 475 years (map available at www.protezionecivile.gov.it/jcms/en/descrizione_sismico.wp Sept.12, 2018).

Fig. 3. View of (a) the PB building and of (b) the Capodimonte area in Napoli.

Fig. 4. View of the MP building area in Napoli.

Fig. 5. View of the AM building area in Sala Consilina.

Fig. 6. PB and MP sites in Napoli (a) and AM site in Sala Consilina (b): reference spectrum at the bedrock for the return period RP of 712 years available from the NTC code standard procedure (black dotted line) and response spectra of the seven accelerograms selected for the Local Seismic Response LSR analysis (the average value is represented by a thick red solid line).

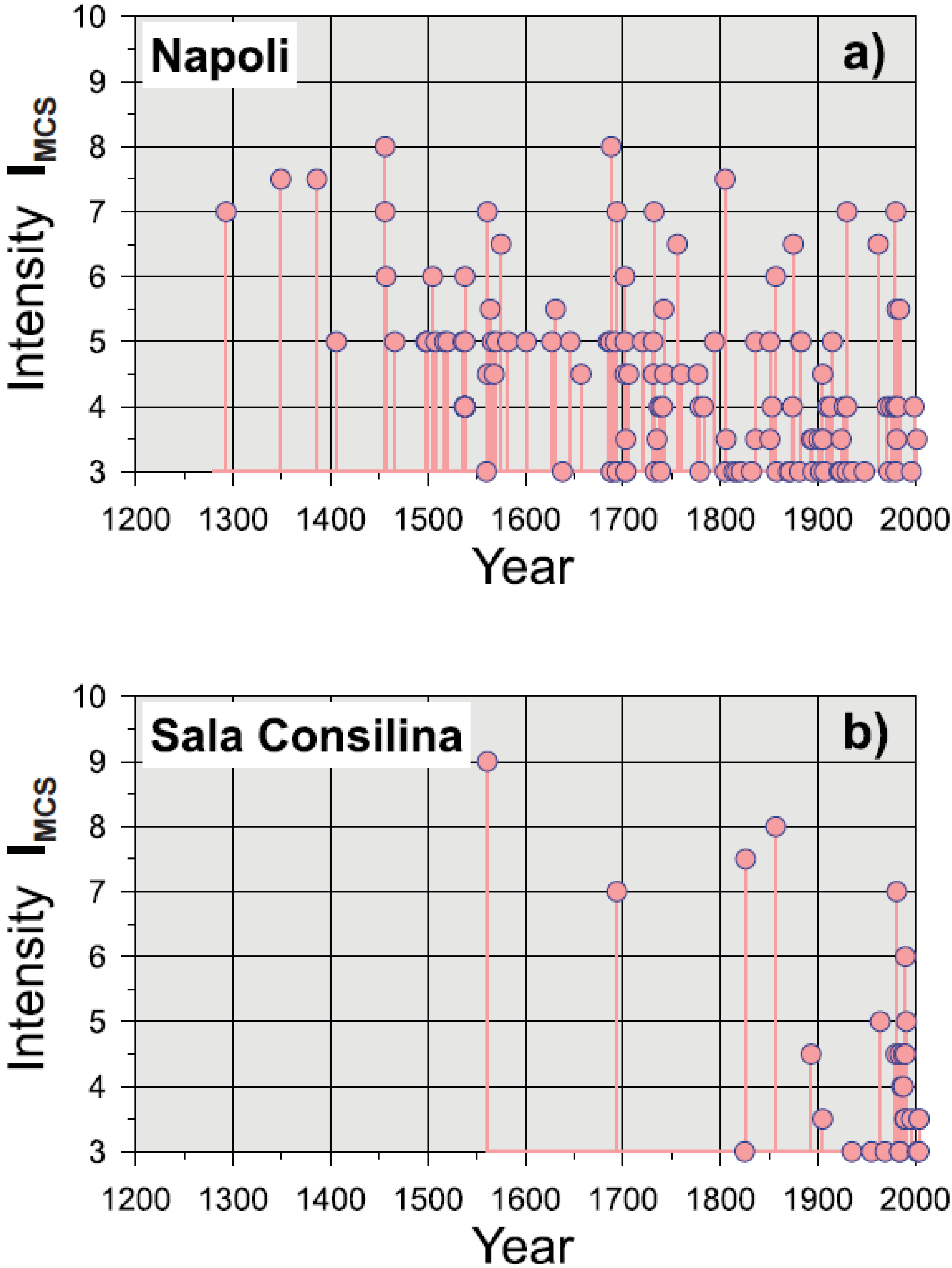
Fig. 7. Results from the geophysical investigation in the PB building area (Napoli): (a) distribution of the resistivity values from the geoelectrical survey ERT (Wenner configuration); (b) distribution of wave velocities V_P from seismic refraction SR_P; (c) MASW phase velocity versus frequency (dispersion curve) and profile of the shear wave velocity V_S with depth.

Fig. 8. Results from the geophysical investigation in the MP building area in Napoli: (a), (b), (c) as in Figure 7.

Fig. 9. Results from the geophysical investigation in the AM building area in Sala Consilina: (a), (b), (c) as in Figure 7.

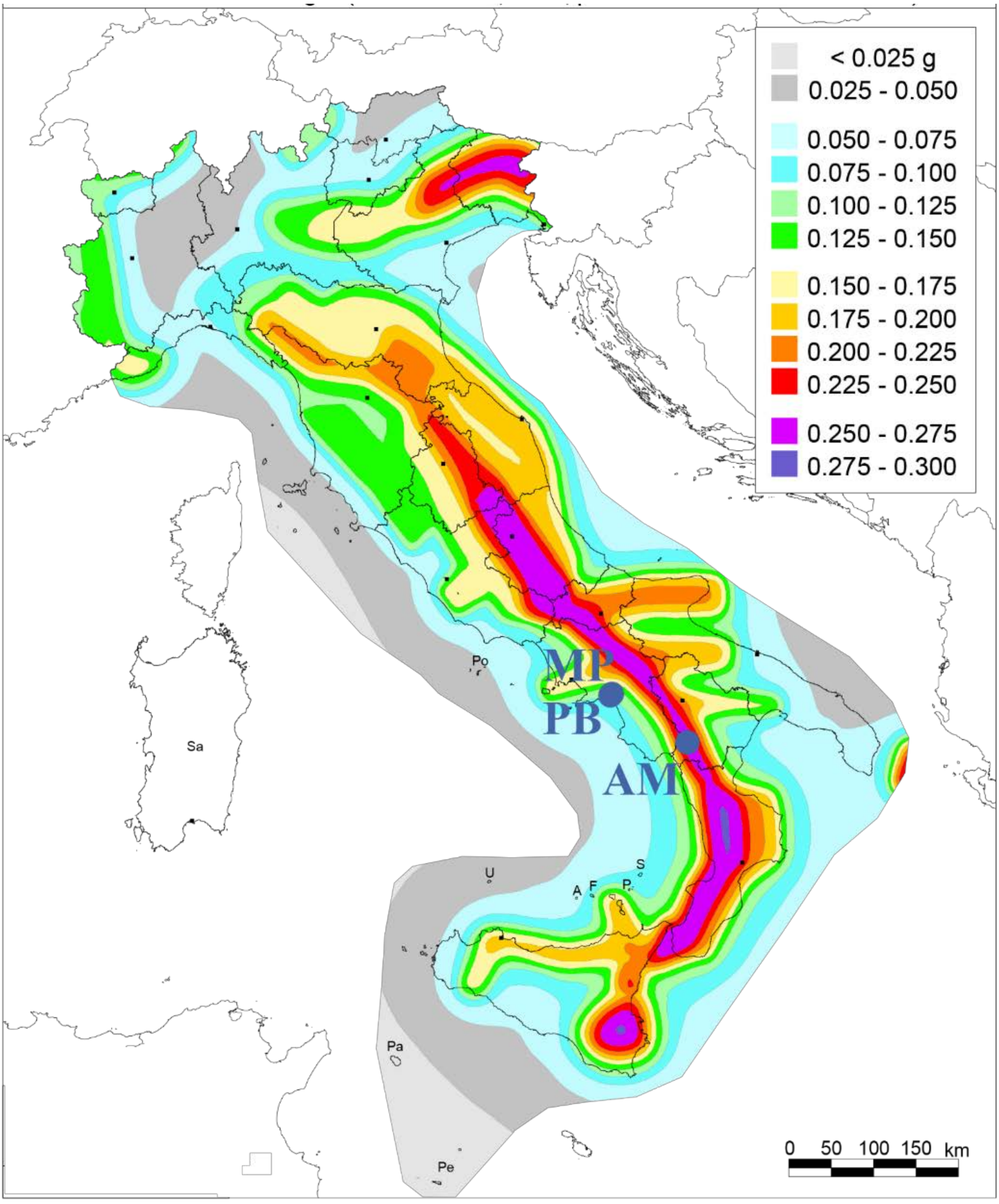
Fig. 10. Variation of (a) normalized shear modulus G/G_0 and of (b) damping ratio D versus shear strain γ adopted in the numerical analyses.

Fig. 11. Acceleration response spectra for the PB, MP and AM buildings, evaluated at the foundation level, obtained from Local Seismic Response LSR analysis (red line) and from the standard procedure (black dotted line).



Napoli
a)
Sala Consilina
b)
Intensity I_{MCS}
Year
10
9
8
7
6
5
4
3
1200
1300
1400
1500
1600
1700
1800
1900
2000

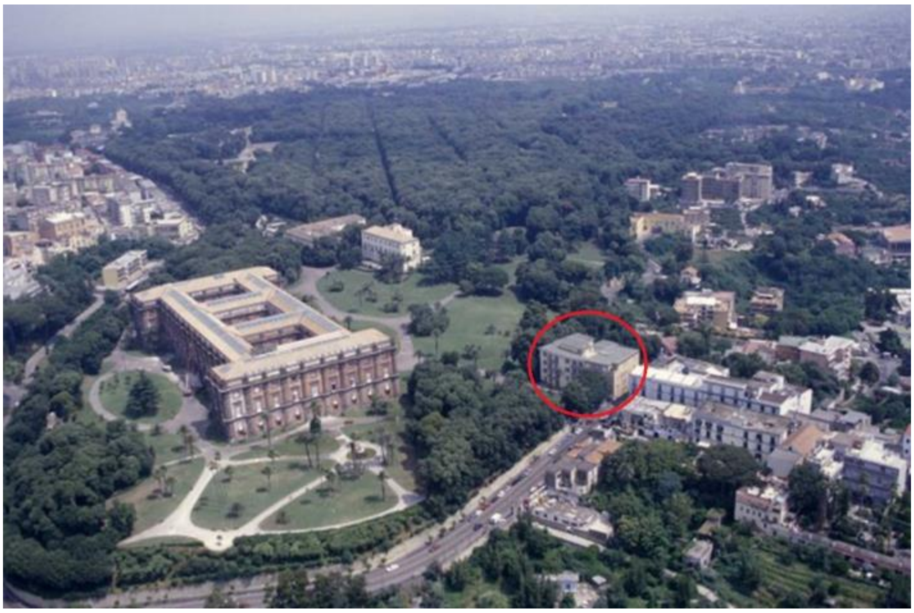
Figure 1



< 0.025 g
0.025 - 0.050
0.050 - 0.075
0.075 - 0.100
0.100 - 0.125
0.125 - 0.150
0.150 - 0.175
0.175 - 0.200
0.200 - 0.225
0.225 - 0.250
0.250 - 0.275
0.275 - 0.300
MP
PB
AM
Po
Sa
U
S
A
F
P
Pa
Pe
0 50 100 150 km

Figure 2

a)



b)

Figure 3

Figure 4

Figure 5

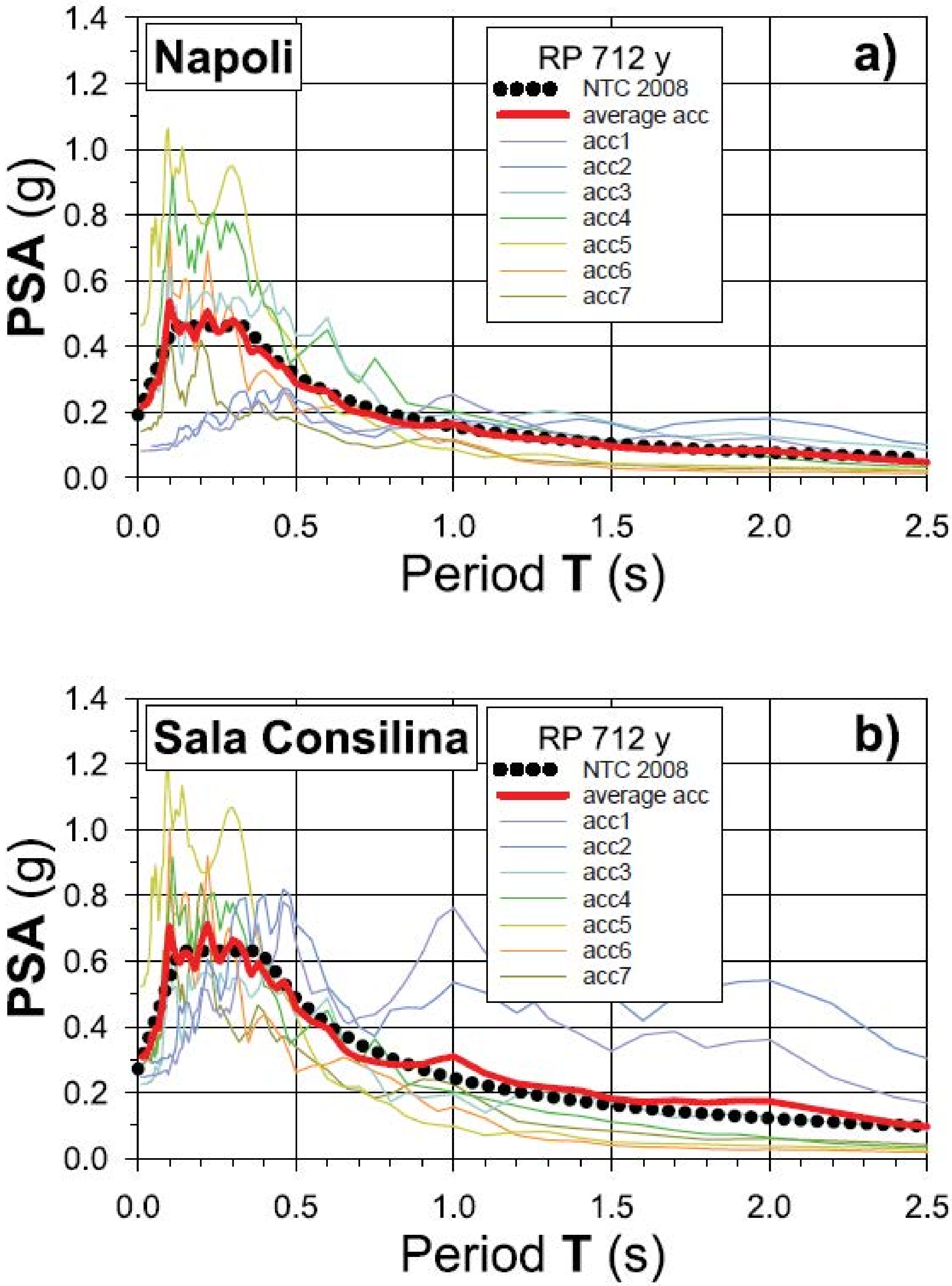
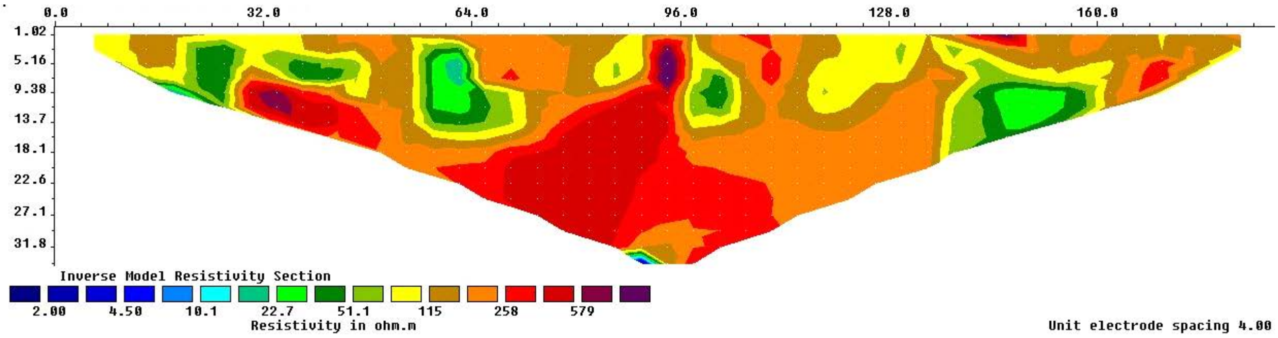
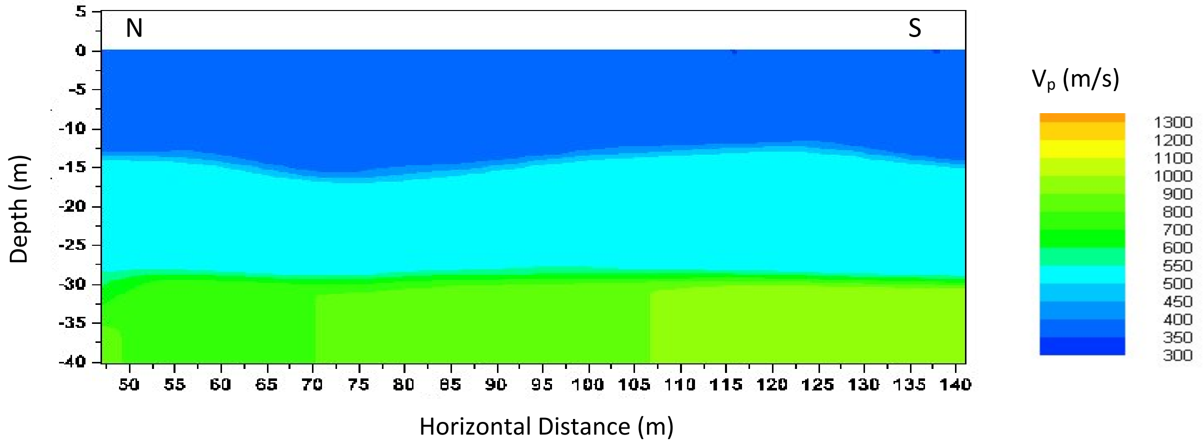

Napoli
a)
RP 712 y
NTC 2008
average acc
acc1
acc2
acc3
acc4
acc5
acc6
acc7
PSA (g)
Period T (s)
Sala Consilina
b)
RP 712 y
NTC 2008
average acc
acc1
acc2
acc3
acc4
acc5
acc6
acc7
PSA (g)
Period T (s)

Figure 6



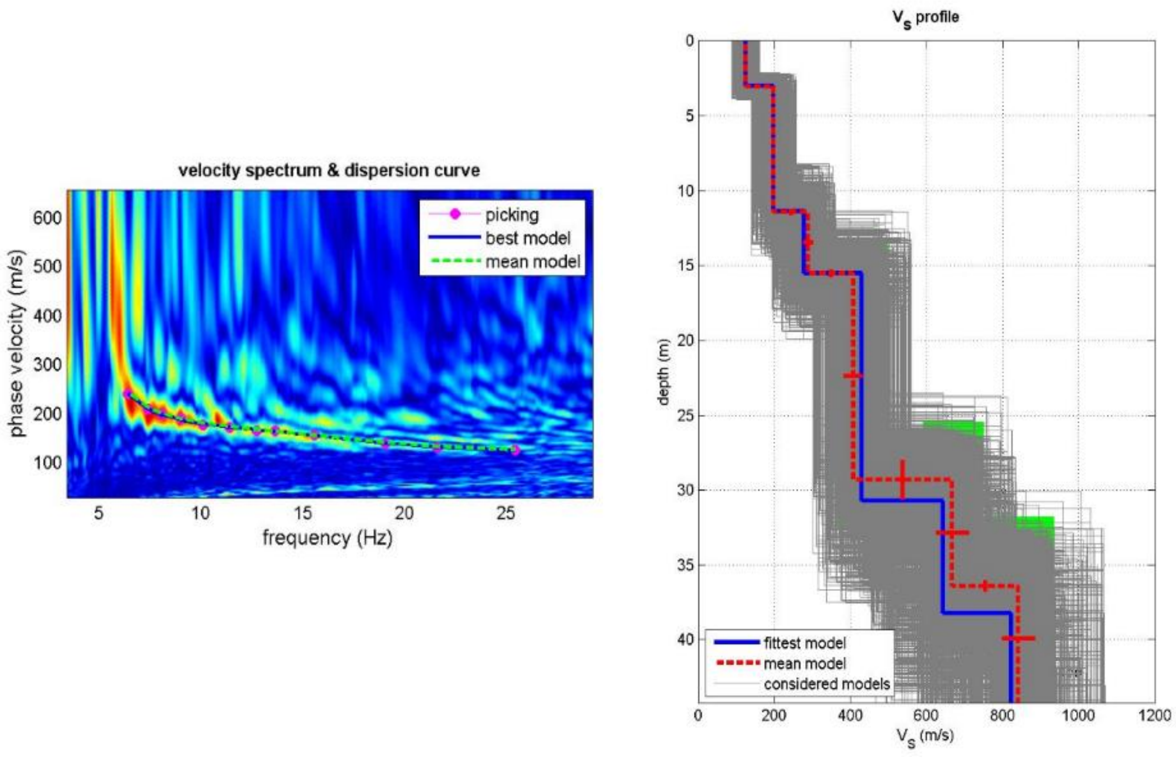
0.0
32.0
64.0
96.0
128.0
160.0
1.02
5.16
9.38
13.7
18.1
22.6
27.1
31.8
Inverse Model Resistivity Section
2.00
4.50
10.1
22.7
51.1
115
258
579
Resistivity in ohm.m
Unit electrode spacing 4.00

a)



N
S
Depth (m)
Horizontal Distance (m)
5
0
-5
-10
-15
-20
-25
-30
-35
-40
50
55
60
65
70
75
80
85
90
95
100
105
110
115
120
125
130
135
140
V_p (m/s)
1300
1200
1100
1000
900
800
700
600
550
500
450
400
350
300

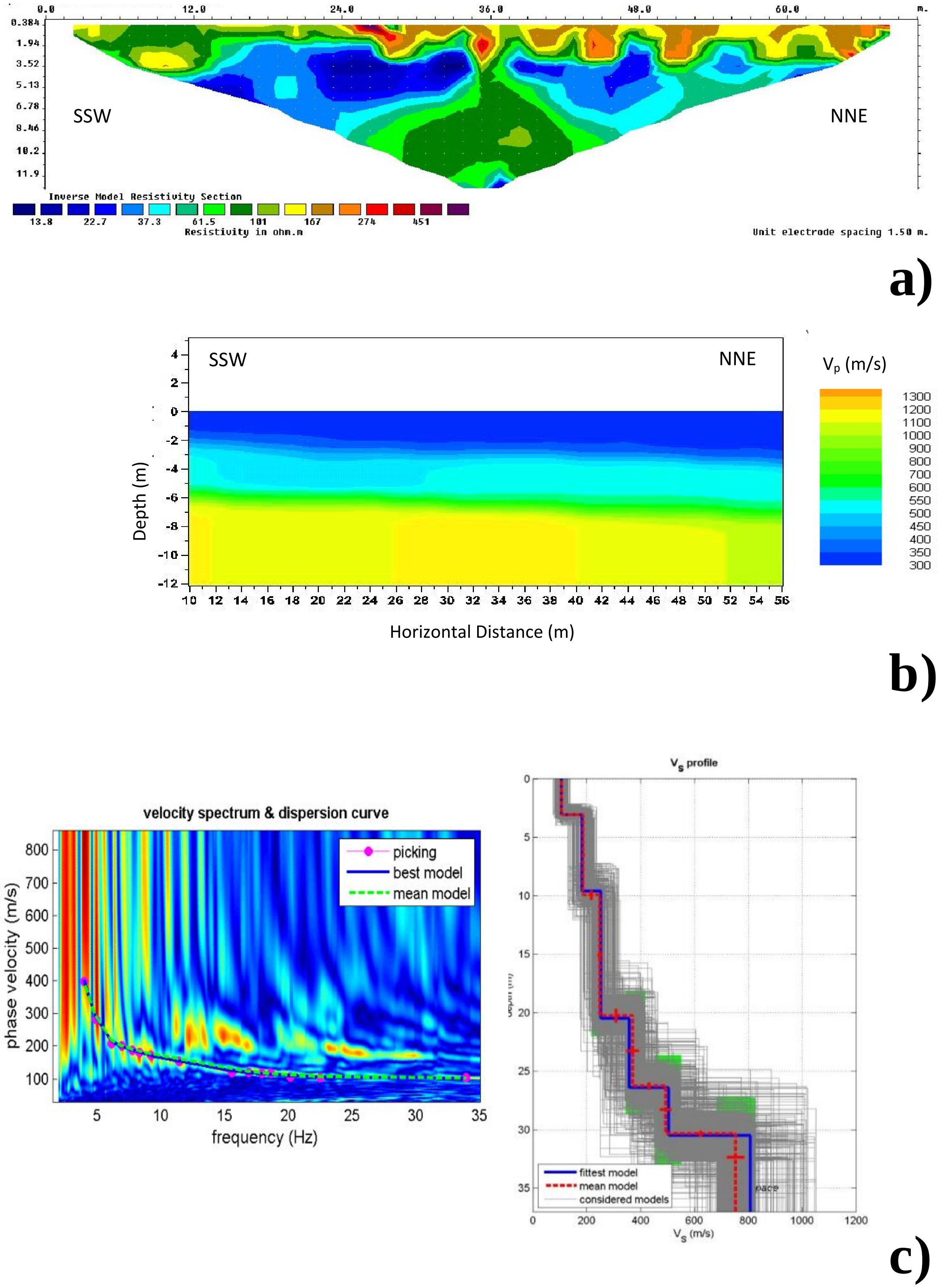
b)



velocity spectrum & dispersion curve
picking
best model
mean model
phase velocity (m/s)
frequency (Hz)
600
500
400
300
200
100
5
10
15
20
25
V_s profile
depth (m)
V_s (m/s)
fittest model
mean model
considered models
0
5
10
15
20
25
30
35
40
0
200
400
600
800
1000
1200

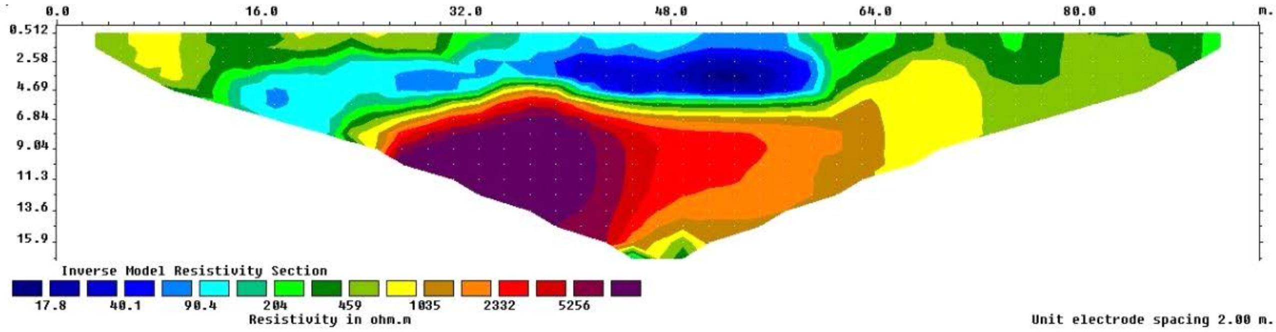
c)

Figure 7

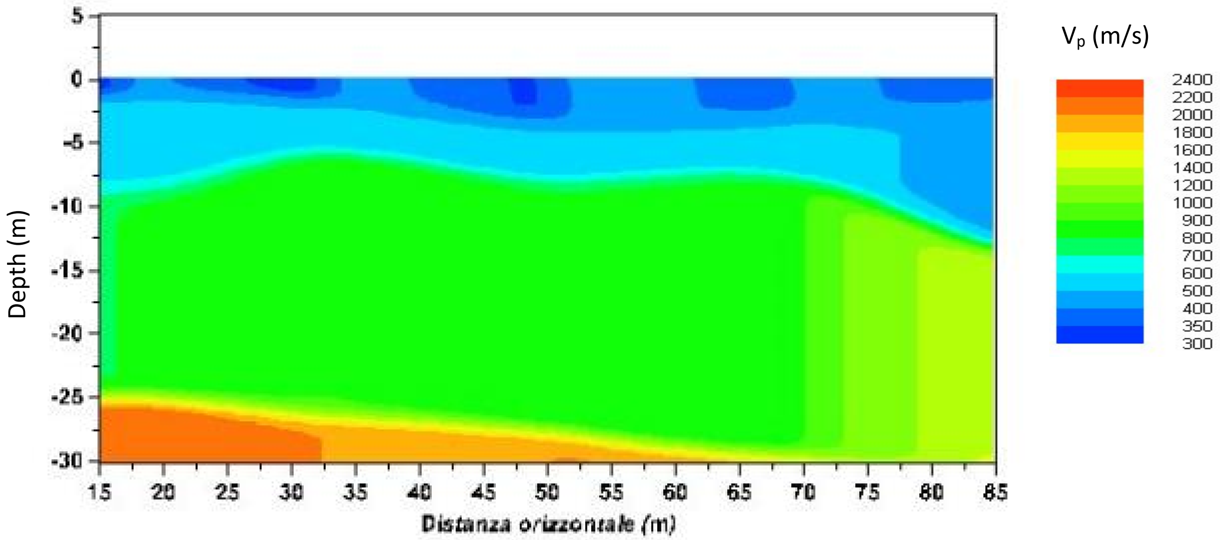


0.0 12.0 24.0 36.0 48.0 60.0 m.
0.384
1.94
3.52
5.13
6.78
8.46
10.2
11.9
SSW
NNE
Inverse Model Resistivity Section
13.8 22.7 37.3 61.5 101 167 274 451
Resistivity in ohm.m
Unit electrode spacing 1.50 m.
a)
SSW
NNE
Depth (m)
4 2 0 -2 -4 -6 -8 -10 -12
10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42 44 46 48 50 52 54 56
Horizontal Distance (m)
Vp (m/s)
1300 1200 1100 1000 900 800 700 600 550 500 450 400 350 300
b)
velocity spectrum & dispersion curve
picking
best model
mean model
phase velocity (m/s)
800 700 600 500 400 300 200 100
5 10 15 20 25 30 35
frequency (Hz)
Vs profile
0 5 10 15 20 25 30 35
fittest model
mean model
considered models
0 200 400 600 800 1000 1200
Vs (m/s)
c)

Figure 8

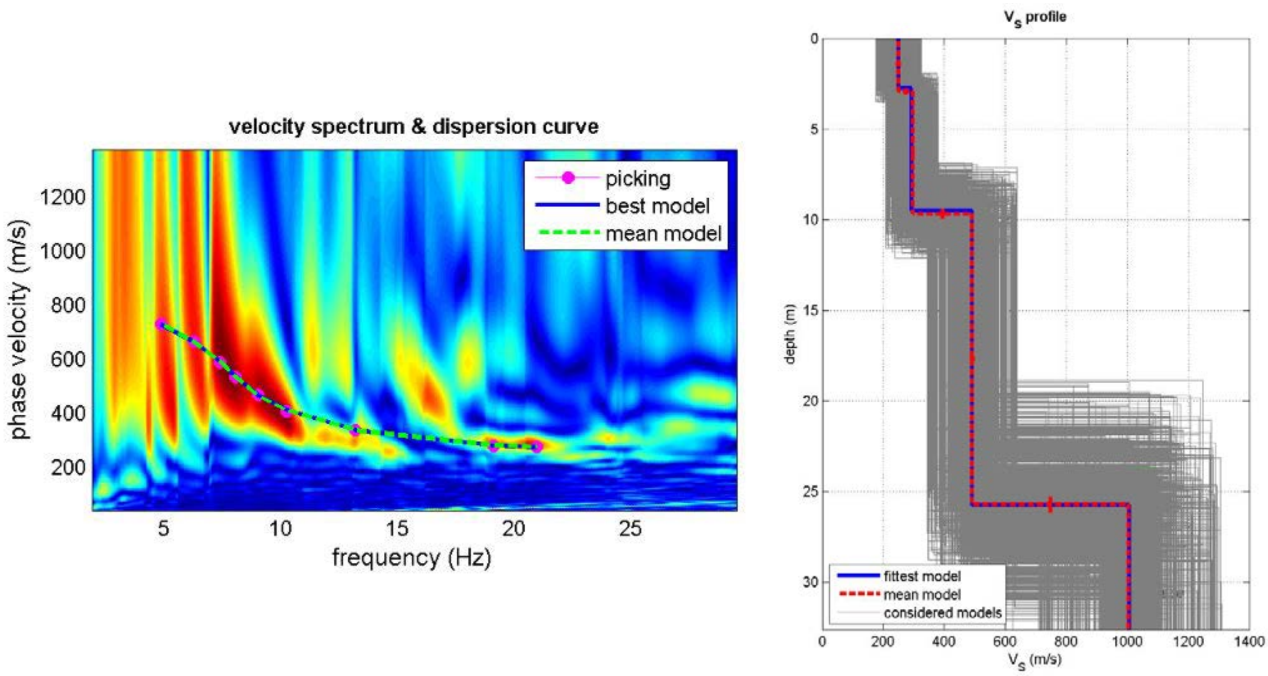


a)



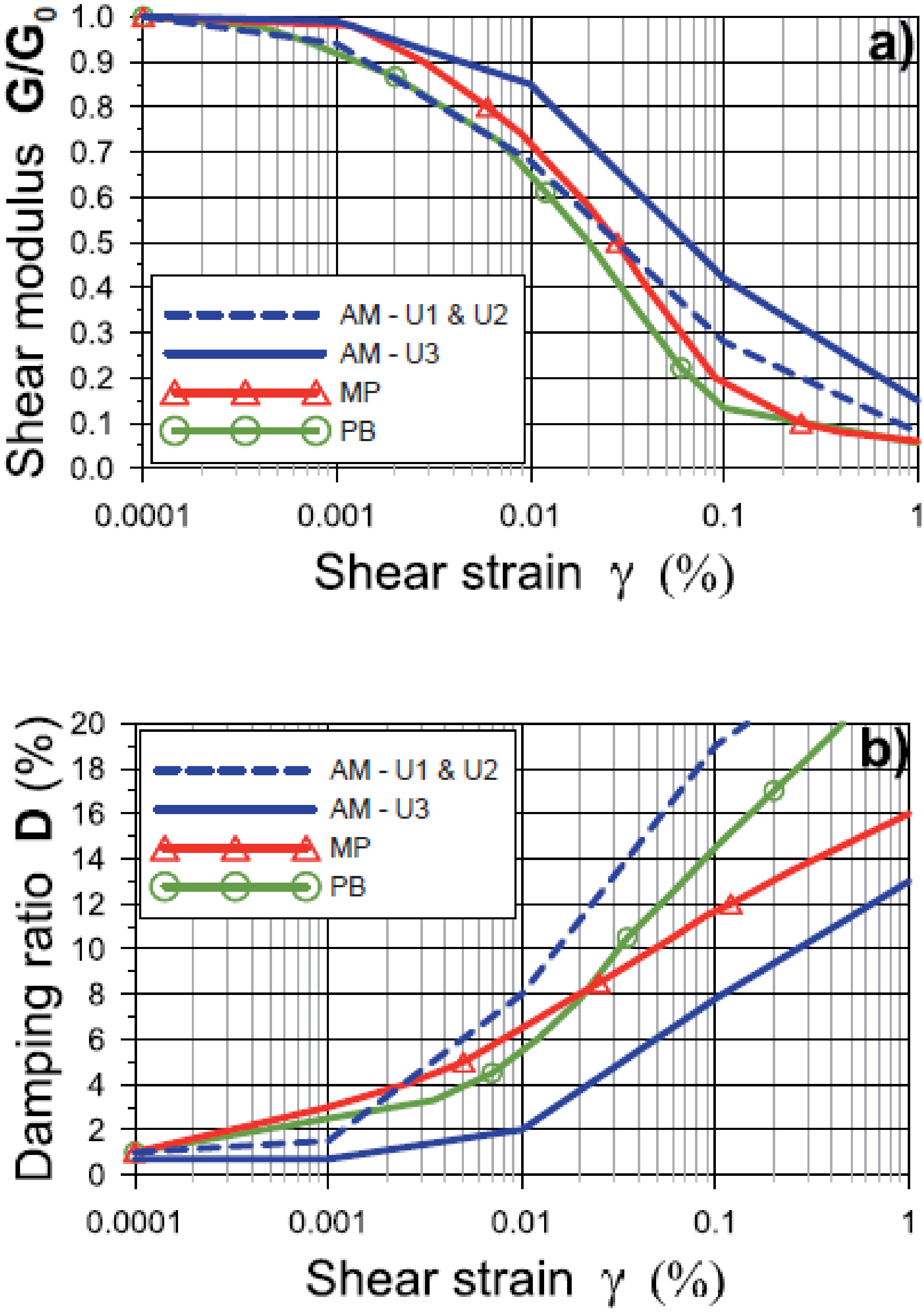
b)

Horizontal Distance (m)



c)

Figure 9



a)
b)
Shear modulus G/G0
Damping ratio D (%)
Shear strain γ (%)
AM - U1 & U2
AM - U3
MP
PB
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
20
18
16
14
12
10
8
6
4
2
0
0.0001
0.001
0.01
0.1
1

Figure 10

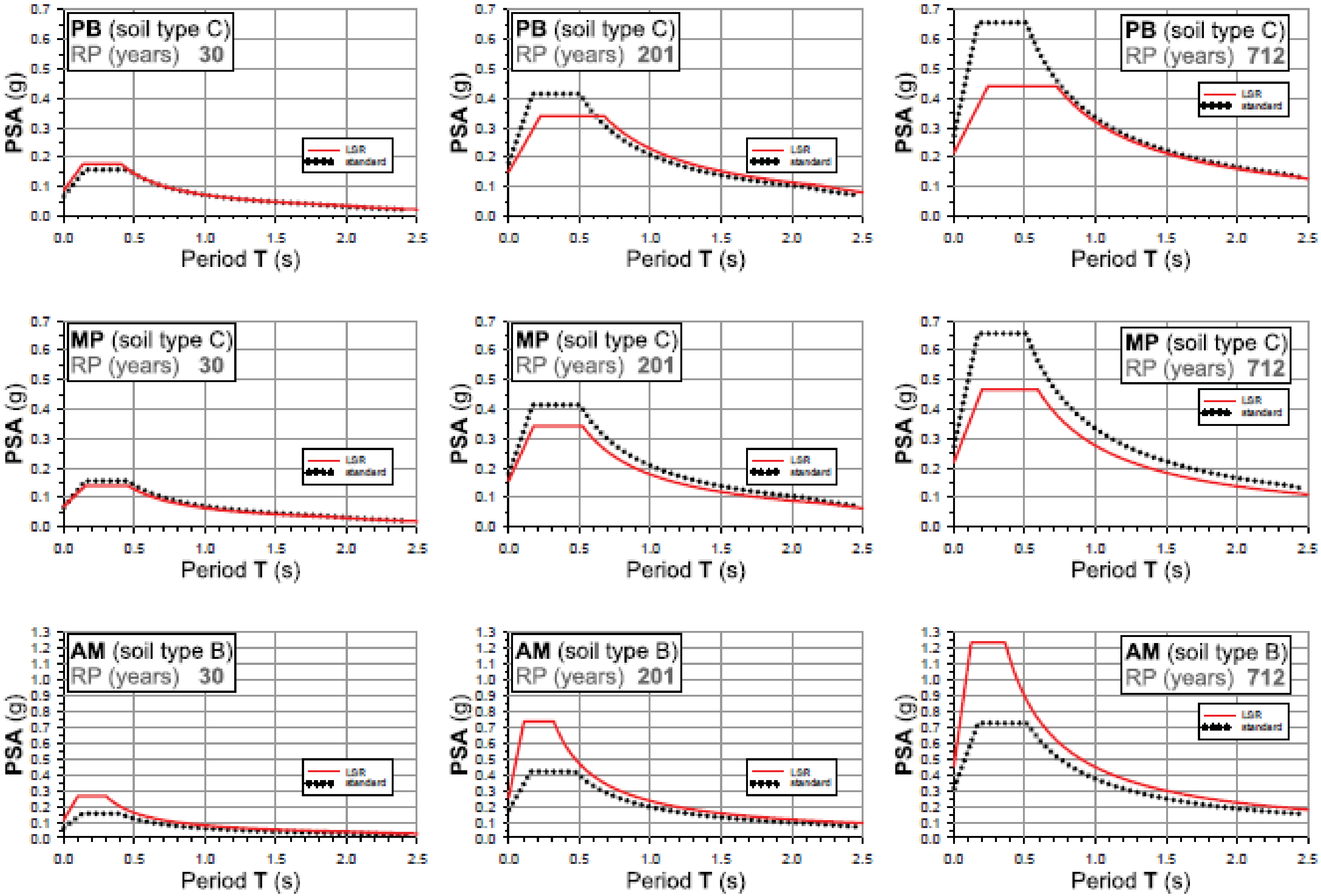

PB (soil type C)
RP (years) 30
PB (soil type C)
RP (years) 201
PB (soil type C)
RP (years) 712
MP (soil type C)
RP (years) 30
MP (soil type C)
RP (years) 201
MP (soil type C)
RP (years) 712
AM (soil type B)
RP (years) 30
AM (soil type B)
RP (years) 201
AM (soil type B)
RP (years) 712
LSR
standard
PSA (g)
Period T (s)

Figure 11