Seismic site response: a comparative study considering Italian standard code

G. Cassani[1], A. Zambon[1], M. Chiarini[1] and C.L. Zenti[1]
[1]*Rocksoil S.p.A, Milan, Italy*
E-mail: carla.zenti@rocksoil.com

ABSTRACT: Italy is one of the countries in the Mediterranean with the highest seismic risk. The Italian technical standards for construction since 2008 requires the estimation of seismic site amplification. The paper attempts to give a critical overview of the field of site response analysis evaluating the influences of the main sources of uncertainties considering Equivalent Linear and Non Linear approach. Steps involved in ground response analyses to develop site-specific response spectra at a soil site are briefly summarized. Through a case study the authors define a procedure that could be carried out during the different stage of a project considering the sensitivity of different parameters and the available data.

KEYWORDS: Seismic site response, amplification, equivalent linear analysis, non linear analysis

1. INTRODUCTION

Italy is characterized by a particular geographic position, the country is located at the convergence of the African and Eurasian plates and for this reason it is one of the countries in the Mediterranean with the highest seismic risk.

After an earthquake, the observation of damages on constructions and infrastructures often highlights substantial differences in different built-up areas, even at short distance among them.

Observations from earthquakes over the past 50 years have shown that local soil conditions can significantly influence the characteristics of ground shaking during earthquakes.

The amplification or attenuation effects of the ground motion during an earthquake is the outcome of a complex combination of concurring factors. Lithostratigraphic and topographic variabilities play an important role in the local seismic response, especially for the Italian territory that is geologically young and characterized by complex and frequent vertical and lateral heterogeneities in terms of impedance (Amanti et al., 2020; Giallini et al., 2020).

The seismic amplification is usually assessed by means of different approaches in function of the level of zonation. ISSMGE (1999) introduces three grades of approach to zonation:

1) the first level of zonation (grade-1, general zonation) is the lowest-cost approach, generally based on compilation and interpretation of existing information available from historic documents, published reports, and other available databases. This is a qualitative level study, aimed at defining homogeneous areas in terms of ground shaking intensity;
2) at the second level of zonation (grade-2, detailed zonation), ISSMGE (1999) suggests to integrate already available information with geotechnical data from engineering reports, to perform field surveys, to better define the soft covers in terms of shear wave velocity and thickness of the layers. The knowlegde referred to this level of zonation should be sufficient to use the simplified approach usually consisting in abacuses that evaluate the amplification effect in function of a limited number of parameters;
3) at the third level of zonation (grade-3, rigorous zonation), ISSMGE (1999) requires site specific information and therefore additional investigations, whose results are generally used to perform specific site response analyses to quantify rigorously the amplification.

In presence of inversion of Vs profile, the Italian Building Code (since 2008) not allow to apply simplified approach based on regional rather than national amplification charts and abacuses, in favour of specific site response analyses typically performed in level 3 of the above mentioned list.

Site effects are quantified via site response analysis, which involves the propagation of earthquake motions from the base rock, through the overlying soil layers, to the ground surface. Site response analysis provides surface acceleration time series, surface acceleration response spectra, and spectral amplification factors based on the dynamic response of local soil conditions (C-C., Tsai & C-W. Chen, 2014).

The main sources of uncertainties for the definition of the seismic site response are:

- the variability of the input motion;
- the intrinsic variability of the soil properties (e.g., shear wave velocity and soil nonlinearity);
- the adopted analysis method (e.g., non-linear or equivalent-linear methods).

These uncertainties may lead to a significant overestimation or underestimation of the ground shaking (Pagliaroli et al., 2015).

Generally, one-dimensional (1D) ground response analysis is preferred to evaluate the effect of local site conditions subjected to an earthquake ground motion. For a site with complex and irregular stratigraphy and geometry, two-dimensional (2D) and three-dimensional (3D) ground response is preferred over 1D wave propagation for more realistic evaluation of ground response under seismic load.

The present work addresses the problem from a practical point of view, considering the fact that nowadays seismic site response is increasingly requested in the face of scarce data available.

The preliminary sensitivity analysis of the parameters governing the uncertainties typical of the seismic site response is the fundamental aspect for cost control and optimization of the project, in terms of both integrative investigation and civil structure design. The aim is to define a procedure that could be carried out during the different stage of a project considering the available data and the necessity of integrative survey campaign. The authors consider a general case and for this reason they consider only one-dimensional approach.

2. SITE RESPONSE ANALISYS

One of the most important controlling parameters of the seismic site response in terms of earthquake ground motions modification is the shear wave velocity (Vs) profile of the subsoil, that generally increases with depth because of geological age, cementation and overburden stress. There exist, however, geological settings where the velocity profile is characterized by inversions, when a stiffer layer (exhibiting higher Vs) overlies a softer one (with a lower Vs). When referring to the Italian territory, these conditions are widespread and require specific seismic site response analyses, both for structures and infrastructures design and for land planning.

This particular subsoil condition has an important impact on the design of many geotechnical works (e.g., pile foundation, retaining walls and others), and in particular in the design of underground infrastructures like tunnels, shaft and stations, both in static and dynamic conditions (Fabozzi and Bilotta, 2016). The soft soil layer in fact, could affect the underground structure stability by increasing the plastic zones, causing asymmetrical stress distribution and aggravating the corresponding internal forces. In general, typical inversions of the Vs profile are associated with well defined geological conditions such as alluvial fans and volcanic settings also in the form of multiple Vs inversions due to the alternating soft and lithoid layers in the subsoil.

The Italian technical standards, as mentioned above, requires the estimation of seismic site amplification in case of shear wave velocity inversion (Fabozzi et al. 2021).
In most cases, 1D site response analysis is performed to assess the effect of soil conditions on ground shaking because vertically propagating and horizontally polarized shear waves dominate the earthquake ground motion wave field. Frequency domain (FD) equivalent linear (EQL) (e.g. Schnabel et al., 1972) and time domain (TD) nonlinear (NL) analyses (e.g. Hashash and Park, 2001) are the most common approaches used to perform 1D seismic site response analysis. The dynamic responses computed via these methods can vary considerably because of the inherent differences in the numerical approaches (FD vs. TD solutions) and differences in how nonlinear soil response is modelled (EQL vs. fully NL).
TD-NL site response analysis does not only propagate input ground motion through the soil deposit in TD, but also varies soil properties with time. This approach allows more realistic modelling of NL soil response than EQL, which only approximates transient nonlinear behaviour as a strain compatible parameter. Therefore, the nonlinear method is generally assumed to provide more accurate site response results, especially for high-intensity input motions.
In the following, through the use of an example the authors, discuss the procedure adopted during the project development considering the available data.

3. THE CASE

The case represents an example of a 1D seismic site response analysis performed by the code DEEPSOIL (Hashash et al., 2012), which is capable of performing TD-NL and FD-EQL analyses

3.1 Subsoil model

In the present study, a one-dimensional subsoil model has been considered (figure 1).
Geometric characterization of the deposit:

- horizontal and indefinitely extended bedrock
- horizontally stratified deposit

Parameters required for numerical modelling:

- thickness of the layers
- velocity of the shear waves in the single identified seismic layers
- volume weight
- dynamic soil parameters

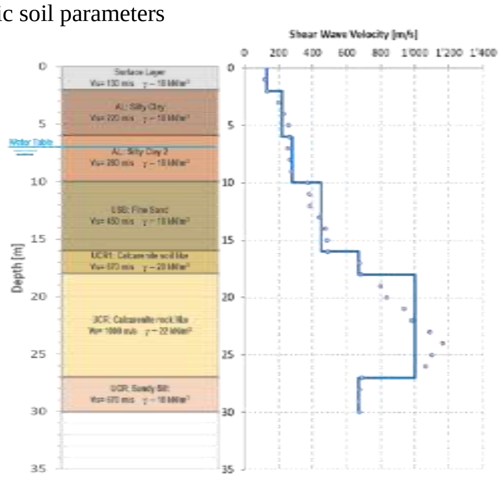


Figure 1 Graphical representation and corresponding shear-wave velocity of soil profile

3.2 Disaggregation data analysis

Seismic hazard disaggregation is commonly used as an aid in ground-motion selection for the seismic response analysis. The results of seismic hazard disaggregation can be used to assign relative weights to a given ground motion record based on its corresponding magnitude, distance and deviation from the ground motion prediction model (epsilon) in order to make probability-based seismic assessments. The reference data for Italy are available on http://esse1-gis.mi.ingv.it/ (INGV: National Institute of Geophysics and Volcanology) through *Interactive Maps of Seismic Hazard*. Figure 2 shows the reference disaggregation data obtained considering the geographical location (a town in south of Italy identified by its geographic coordinates) and the probabilistic assessment of seismic hazard (PSHA) in terms of horizontal peak ground acceleration (PGA) with 5% probability of exceedance in 50 years on hard ground.

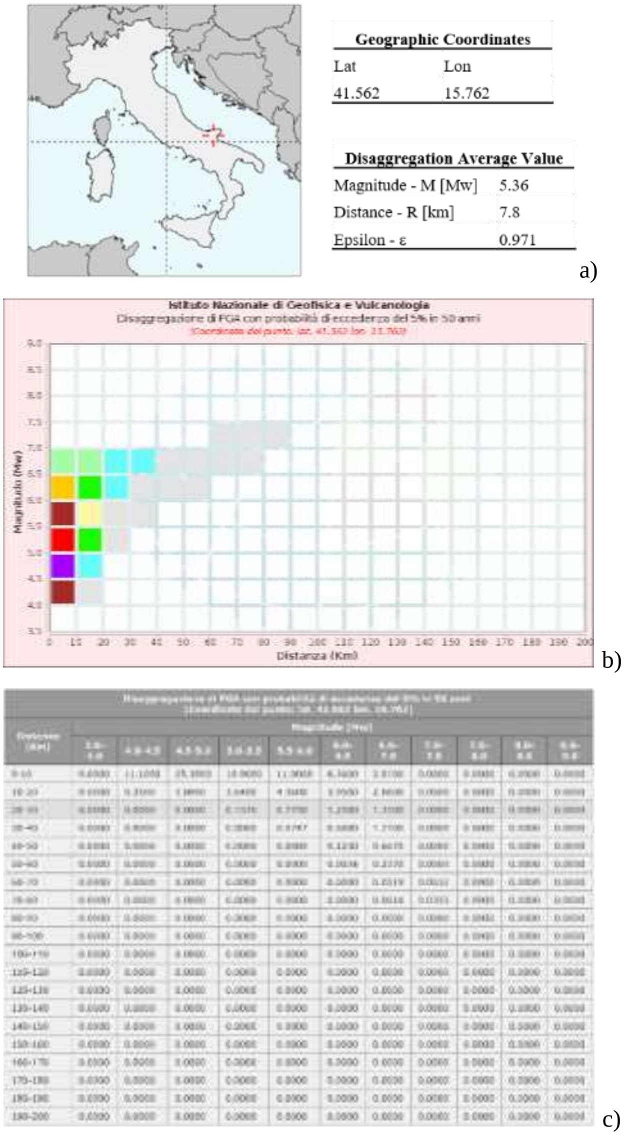
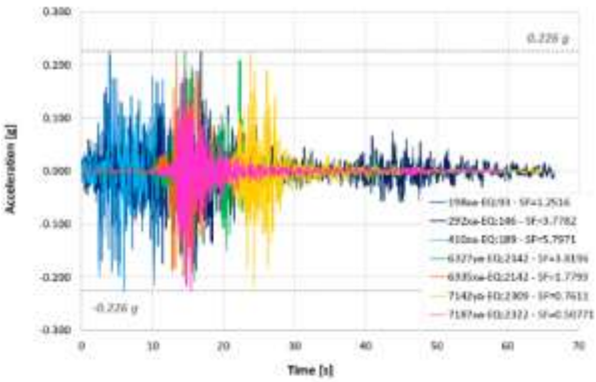


Figure 2 Disaggregation: a) data synthesis; b) graph; c) data detail (http://esse1-gis.mi.ingv.it/)

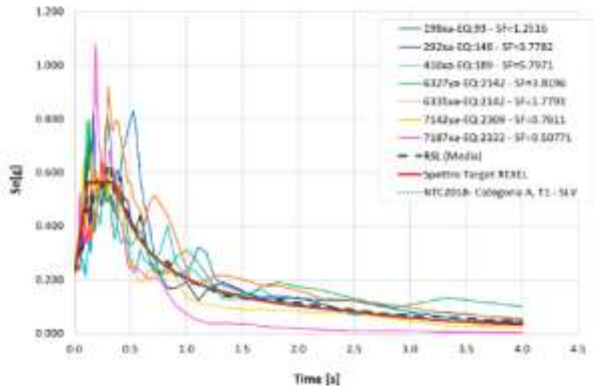
3.3 Seismic input motion

One of the key issues for the seismic site response is the selection of appropriate seismic input, which have to be representative of the seismic hazard of the local area. REXEL (Iervolino et al., 2019), freely available at the website of the Italian network of earthquake engineering university labs (http://www.reluis.it/index_eng.html), allows to search for suites of waveforms, currently from the European Strong-motion Database, compatible to a reference spectra being either user-defined or automatically generated according to Eurocode 8 (EN 1998-1, 2004) and the Italian Building Code (2008;2018).
Considering the geographical location and disaggregation data (M,R and Epsilon, see Fig.2.a), Rexel selected seven input signals and scaled up to the reference value of the peak ground acceleration, a_g, for the selected design limit state (Fig.3.a), according to the probabilistic seismic hazard approach adopted by the Italian Building code to guarantee the spectrum-compatibility of the selected signals

(Fig.3.b) (here and in the following will be considered only vertical spectrum). For the selected case the hazard curve provides a value of a_g about to 0.226 g, by assuming a 'life safety limit state' (i.e., Pr=10%) and a reference life cycle equal to 100y, which correspond to a return period of the design earthquake as high as 946y.



a)



b)

Figure 3 Rexel Output: a) selected signals; b) spectrum-compatibility of the seven selected signals

3.4 Analysis

The use of advanced soil constitutive models is appropriate when detailed information on soil behaviour is available.
However, for most applications the only information available are the modulus reduction and damping curves. Therefore, use of more simplified models - especially models that belong to the family of hyperbolic soil models - are often used (Hashash et al. 2010).
The performed analysis (FD-EQL and TD-NL) consider as soil model (Backbone Curve) the General Quadratic/ Hyperbolic Model with Masing Rules (Hashash et al., 2012).

3.4.1 Reference curve

Resonant Column Test is the laboratory test used to determine the shear elastic modulus and damping properties of soils. Unfortunately, data on this parameters are not always available. In this case the designer have to carry out a sensitivity analysis on the reference curves existing in literature. Typically, the ones related to cohesive soil vary on the base of Platicity Index (P.I.). The ones related to granular soil vary on the base of relative density.
Table 1 and Figure 4 are referred to the reference curves adopted by the authors for the present case as the results of a sensitivity analysis performed on the dynamic parameters of the soil model represented in Figure 2.

3.4.2 Software procedure

The user creates the layered domain in DEEPSOIL software and select the available reference curve (damping curve & stiffness reduction curve). Upon constructing the layered domain, GQ/H curve fitting routine calculates the shear strength corrected shear strength - shear strain curve and provides the necessary parameters. These values will be directly used in soil hysteretic material.
The user has the option of obtaining the site response results using the equivalent linear method automatically whenever nonlinear site response analysis is conducted. By this way the selection of the shear modulus and damping ratio are automatically extracted by the software from the backbone curve of the nonlinear models. The use of this option it is highly recommended.

Table 1 Reference Curve

Soil Type	Reference Curve
Surface Layer	Seed Idriss Mean
AL - Silty Clay	Vucetic&Dobry (P.I. 30)
USB - Fine Sand	Seed Idriss Lower
UCR1 - Calcarenite soil like	Seed Idriss Mean
UCR - Calcarenite rock like	Seed&Idriss Upper
UCR-LS: Sandy Silt	Vucetic&Dobry (P.I. 0)

Seed&Idriss (1970) - Lower Limit

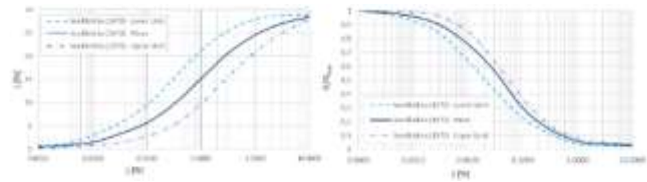
Strain [%]	G/Gmax	Damping [%]
0.0001	1	0.75
0.0003	0.98	1.1
0.001	0.93	3
0.003	0.84	5.5
0.01	0.64	9.5
0.03	0.43	15
0.1	0.23	21.2
0.3	0.12	25.4
1	0.04	28
3	0.03	28.8
10	0.025	29

Seed&Idriss (1970) - Upper Limit

Strain [%]	G/Gmax	Damping [%]
0.0001	1	0.24
0.0003	1	0.42
0.001	0.99	0.8
0.003	0.96	1.4
0.01	0.85	2.8
0.03	0.64	5.1
0.1	0.37	9.8
0.3	0.18	15.5
1	0.08	21
3	0.05	25
10	0.035	28

Seed&Idriss (1970) - Mean

Strain [%]	G/Gmax	Damping [%]
0.0001	1	0.48
0.0003	0.99	0.8
0.001	0.96	1.5
0.003	0.9	3.2
0.01	0.76	5.7
0.03	0.57	9.5
0.1	0.3	15.2
0.3	0.15	20.5
1	0.06	24.6
3	0.04	27
10	0.03	28.5



Vucetic&Dobry - IP15 (1991)

Strain [%]	G/Gmax	Damping [%]
0.0001	1	1
0.0003	0.998	1
0.001	0.995	1
0.003	0.946	2.32
0.01	0.822	4.34
0.03	0.656	7.11
0.1	0.413	11.27
0.3	0.226	15.48
1	0.086	19.58

Vucetic&Dobry - IP30 (1991)

Strain [%]	G/Gmax	Damping [%]
0.0001	1	1
0.0003	1	1
0.001	0.999	1
0.003	0.977	2
0.01	0.904	3.55
0.03	0.763	5.54
0.1	0.545	8.25
0.3	0.352	11.69
1	0.164	16.51

Vucetic&Dobry - IP0 (1991)

Strain [%]	G/Gmax	Damping [%]
0.0001	1	1
0.0003	0.998	1
0.001	0.962	1.45
0.003	0.885	2.77
0.01	0.719	5.21
0.03	0.498	9.44
0.1	0.25	14.94
0.3	0.113	19.37
1	0.02	23.25

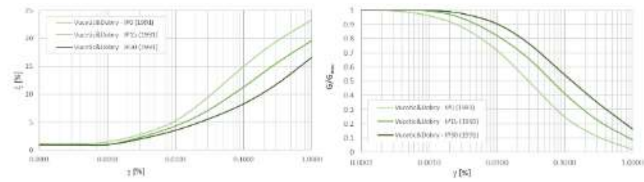


Figure 4 Reference Curve: damping curve & stiffness reduction curve

4. RESULTS

As mentioned in the introduction, the aim of the paper is to define a procedure that could be carried out during the different stage of a project considering the available data and the sensitivity analysis of the parameters governing the uncertainties of the seismic site response.

4.1 Bedrock

Downhole seismic testing is one field test that is commonly used to determine compression wave (P) and shear wave (S) velocity profiles in geotechnical earthquake engineering investigations. These profiles are required input in evaluations of the responses to earthquake shaking of geotechnical sites and structures at these sites. In some case the test does not identify the bedrock depth. This parameter has a fundamental importance because deeply influenced the analysis result. The authors considered the soil profile of Figure 2 and made two different hypotheses:

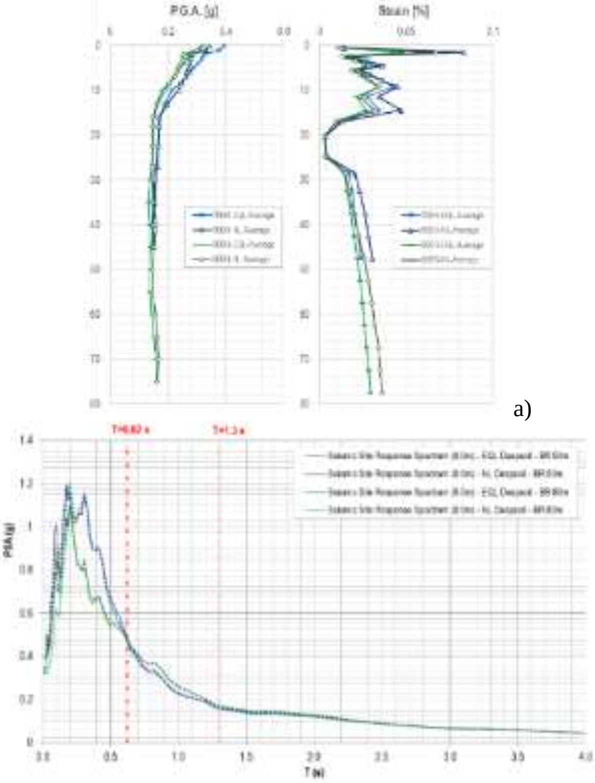
- Bedrock Depth 50m (50BR). The thickness of the soil unit UCR-LS: Sandy Silt is 2.0m + 23.0 m and it is characterized by a constant shear wave velocity with depth equal to Vs:670m/s and volume weight equal to γ = 18 kN/m^3
- Bedrock Depth 80m (80BR). The thickness of the soil unit UCR-LS: Sandy Silt is 2.0m + 53.0m and it is characterized by a constant shear wave velocity with depth equal to Vs:670m/s and volume weight equal to γ = 18 kN/m^3

EQL and NL analyses have been carried out on both hypotheses.

The seismic site response of the above defined 1D columns was evaluated, here and in the follow, in terms of modification of the outcropping rock reference signal.

4.1.1 Comments

The influence of the different hypotheses is demonstrated by the analysis results comparison of Figure 5. The bedrock depth has a high influence in terms of P.G.A. and Strain profile (Fig.5.a) and, as a consequence, it influenced also the Seismic Site Response Spectrum (Fig.5.b). In this specific case there is a difference between ELQ and NL analysis up to the first peak. The bedrock variability causes a Spectrum inversion at T=0,62 while it does generate no substantial difference after T=1,3s. Therefore, the period of the structure under design becomes a discriminating factor and in case of a civil works with different type of structures, the depth of bedrock is always subjected to an integrative investigation request.



a)

b)

Figure 5 Bedrock Influence: a) P.G.A. & Strain average profiles; b) Seismic Site Response Spectrum

4.2 Soil Profile Detail

Before going on with the discussion, a distinction must be made in terms of calculation methodology based on the specific design stage (conceptual design, preliminary design, detailed design) and related to the types of works the local seismic response analysis is addressed to. The first discriminating factor concerns the design phase: more embryonic is the project and less specific investigations (DH) are available. The second aspect to be evaluated is the significant volume investigated and the scale of the civil work.

The design practice of the large scale civil work prefers a robust geotechnical model, even if characterized by less information about the heterogeneity of the parameters in each single geotechnical unit. The characterization of the dynamic properties of the geotechnical units also provides the possibility to reconstruct a reliable velocity profile even where there is no direct evidence of Vs, and therefore to develop analysis at any point of the construction site (e.g. red profile in figure 6) and to define large scale numerical models.

On the other hand, for specific works or whose significant volume is small (i.e. viaducts foundation pile), and in particular for 1-D models, the adoption of a very detailed profile resulting from the single test allows to take into account the local heterogeneity that characterize the specific vertical and therefore the local stratigraphy.

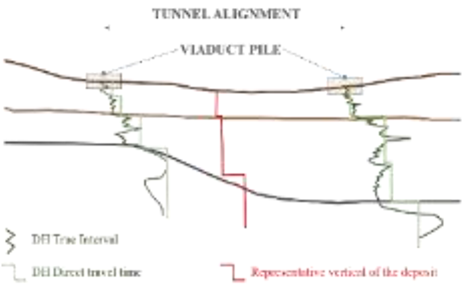


Figure 6 General scheme

Down-hole tests (DHT) represent a compromise in terms of accuracy and execution costs. The downhole method requires only one borehole, and this make it a cost-effective method. The data interpretation may be performed following several approaches. One of those determines the arrival time for the P wave or S wave directly from the record as the lapsed time between time zero (activation of the seismic source) and the arrival of the respective wave trains at each of the receiver depths (distance between receivers typically 1.0÷2.0m) assigning the velocity to the mid-point. This approach allows high spatial resolution and local estimation at different depth; however, it is affected by the determination of the instant of first-arrival at the two receivers (typically based on the assumption that raypaths between the source and receivers are straight), especially when the signal/noise ratio of the recordings is low. For this reason, it is to be avoided interpretation based on velocity obtained using just one transducer positioned at different depths (pseudo interval).

Another approach is based on direct time interpretation through the definition of the variation of first arrival with depth. The average slopes allow the estimation of the average propagation velocity for each layer. This interpretation is characterized by a lower resolution and sensitivity to small variations accompanied with a greater robustness. The hypoteses listed in the following have been defined considering the different DHT interpretation approach.

The last group of analysis consider the soil profile of Figure 7. The bedrock depth has been identified at a depth equal to 70.0m and it is characterized by a shear wave velocity equal to Vs=800m/s and a volume weight equal to 22.0kN/m^3. The soil profile has been divided in 1.0m layer up to the depth of 30.0m. The remain thickness up to the bedrock depth (40.0m) has been divided in layer characterized by a thickness of 5.0m, in in compliance with:

$$h_{max} = \frac{\lambda_{min}}{6 \div 8} = \frac{V_S}{6 \div 8\, f_{max}} \qquad (1)$$

where f_{max} is the maximum frequency of the seismic input, considering it as that frequency beyond which the frequency content of the Fuorier spectrum is negligible (for seismic input f_{max} = 20 Hz).

Two different hypotheses related to the shear wave velocity profile have been made considering the distinction mentioned at the beginning of this paragraph.

HP01. Each layer is characterized by the shear wave velocity related to the specific soil unit (Figure 1). The thickness of the soil unit UCR-LS: Sandy Silt (40.0m) is characterized by a constant shear wave velocity with depth equal to Vs:670m/s and volume weight equal to γ = 18 kN/m^3. The reference shear wave velocity profile is DH direct travel time and its progression (Figure 7).

HP02. Each layer is characterized by the shear wave velocity recorded during the specific downhole test (Figure 7). The thickness of the soil unit UCR-LS: Sandy Silt (40.0m) is characterized by a linear increasing shear wave velocity value, starting from Vs=670m/s (30.0m depth) to Vs=800m/s (70.0m depth) with depth equal to and volume weight equal to γ = 18 kN/m^3. The reference shear wave velocity profile is DH true-interval and its progression (Figure 7).

EQL and NL analyses have been carried out on both hypotheses.

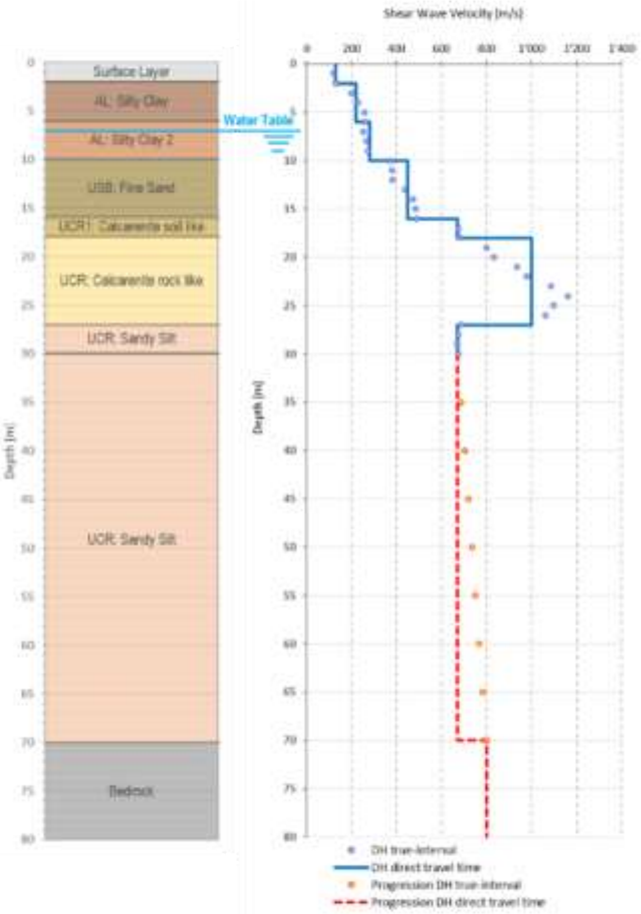


Figure 7 Soil profile detail

4.2.1 Comments

The influence of the different hypotheses is demonstrated by the spectrum comparison reported in Figure 8. It is important to underline that the scope of the comparison is not the identification of the better approach but it is to create an awareness about the way by which every single approach governs the results.

The first comment is related to analysis method. The dynamic responses computed via EQL and NL could vary considerably because of the inherent differences in the numerical approaches and differences in how nonlinear soil response is modelled. The nonlinear method is generally assumed to provide more accurate site response results, especially for high-intensity input motions. However, the NL analysis, compared to the EQL, is more affected by the completeness of the model adopted and therefore by the quality of the available information. The case study points out different results for ELQ and NL analysis especially referring to the peak periods. The EQL analysis results may leads to an over-dimensioning of the civil work structure. The approach governs the structural choices of designer and, as a consequence, the cost of the project. However, especially in the early design stages, when only few data are available and it is essential not to quantify but to identify the problem, the adoption of EQL is a convenient choice.

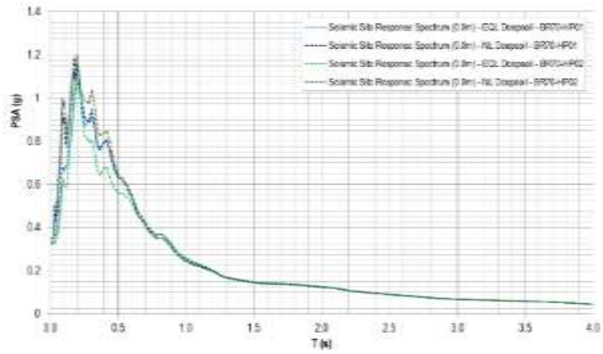


Figure 8 Soil profile detail: Seismic Site Response Spectrum related to HP01 and HP02 in case of EQL and NL analysis

The quantification of the problem, requires a detailed analysis. The evaluation of the P.G.A. & Strain profiles (Figure 9.a) lead to identify the layer characterized by the highest amplification. This information gives the possibility to the designer, if it is physically and geometrically possible, to move out from this layer the civil work, for instance, adopting deeper foundation work. The Figure 9.a underlines the maximum strain at a 2.0m depth. The analyses carried out at a 2.0m depth (figure 9.b), instead of the surface level (Figure 8), show a lower spectrum. In case of study at 2.0m depth the difference between ELQ and NL analysis are few, this is not representative of the common situation, but it is the specific condition detected in the proposed case study.

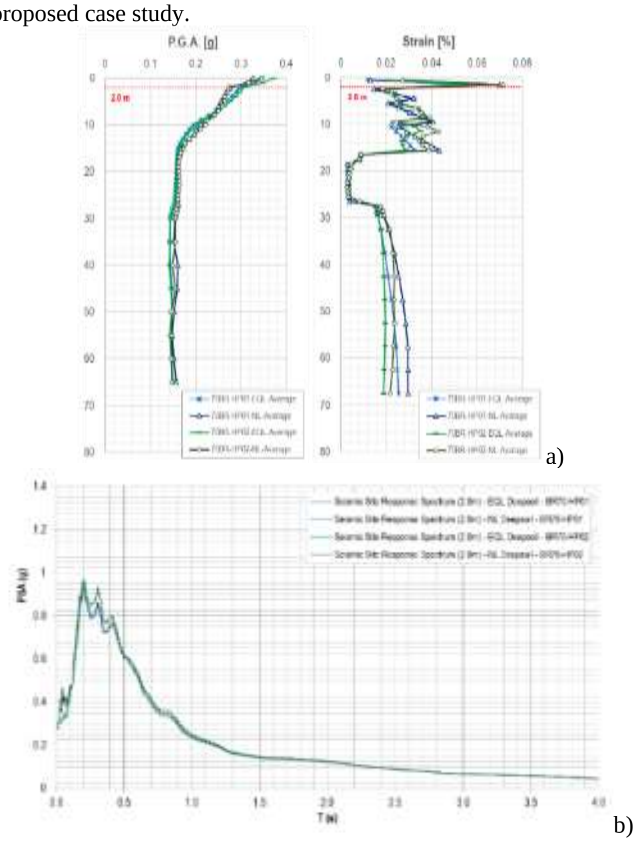


Figure 9 Soil profile detail: a) P.G.A. & Strain average profiles; b) Seismic Site Response Spectrum & Normalized Spectrum (2.0m)

Furthermore, in the case study, the adoption of a Vs profile interpreted through direct travel time method or the one obtained by true-interval methos does not cause a substantial difference. from the results obtained considering the true interval values. The difference in terms of acceleration (P.G.A. profile) decreases even more not considering the surface layer (up to 2.0m depth). The difference in terms of deformation is remarkable, between depth interval from 2.0m to 15.0m. This evidence is particularly interesting when the structure analysis (performed with the results of Seismic Site Reponse

Analysis) will be based on the strain profile (i.e design of underground infrastructures such as tunnels, shafts and stations) or using the PGA value at the foundation level.

The last comment is related to the design spectrum comparison. Figure 10 compares the ones calculated from the normalization of seismic site response spectrum and the one of Italian Building Code for a ground type B. NTC 2018 Spectrum is associated to the DH of Figure 7 in case of site test ends to 25.0m depth (without the identification of Vs inversion). This fact underlines the importance to detected shear wave velocity inversion to avoid the risk of underestimate civil works structures.

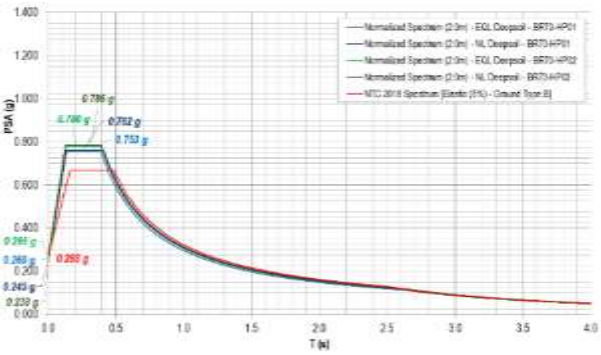


Figure 10 Soil profile detail: SSR Normalized Spectrum (2.0m) and Italian Building Code Spectrum (B Category)

5. CONCLUSION

The seismic characteristics of the Italian territory require the adoption of soil-structure interaction analysis in seismic conditions increasingly topical. Therefore, the ground response analysis is necessary to investigate the problem where is not possible adopting the simplified regulatory approach. Such analyses require a detailed geotechnical and geological model reconstruction, since the results of the analysis are very sensitive to the input data provided.

Often the designer, particularly at the early design stages, has to handle the problem on the base of few available surveys, typically not widespread in the significant volume of the civil works, and on the basis of large-scale geological model. The unknowns related to the lack of data add up to the main sources of uncertainties typical of the seismic site response definition. However, in this phase is fundamental to develop ground response analysis in order to identify the potential criticalities of the project defining which data are indispensable and quantifying the commitment for the subsequent integrative investigation campaign.

The aim of the paper is to suggest a sensitivity analysis on the main uncertainties typically to face in the daily planning even of large civil works. The influence of several parameter, such as bedrock depth, heterogeneity of the single geotechnical units and the consequent local variations of Vs, have been study considering their variation with depth. The reference curve selection has been made considering available data for each geotechnical unit and comparing that with the damping and stiffness reduction curves present in scientific literature. The sensitivity analysis performed on above mentioned parameters lead to evaluate their incidence on the results of seismic site response analysis carried out through equivalent linear and nonlinear method.

The case study describes the proposed approach. The sensitivity analysis shows the difference from the scenario defined by the simplified regulatory approach, it also highlights the data that can most influence the results of the analyses based on the type of civil work under design. In addition, the sensitivity analysis defines a methodological path for the identification of the integrative survey campaign in order to collect the design data essential.

Furthermore, given the purposes of the work, the analyses were conducted in one dimension including only the lithostratigraphic effect so that the discussed results and conclusions are valid in this condition only. In more complex morphological settings that can lead also to 2D and 3D effects, the influence of inversion of Vs profile could be considered with the morphological ones.

6. REFERENCES

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