EVALUATION OF SEISMIC SITE EFFECTS BY MEANS OF 1D, 2D AND 3D FINITE ELEMENT ANALYSES. A CASE STUDY

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ABSTRACT

Predicting the seismic site effects in complex real case histories is a non-trivial scientific and technical task. In fact, it requires the overall geometry of the problem (i.e. surface and underground) to be properly described and combined to adequate soil models, accounting for the peculiar aspects of the dynamic soil behaviour. These latter should include the dependency of the initial stiffness on the stress state and its non-linear evolution with strain amplitude, together with the corresponding one of the damping ratio. A further ingredient to be considered when approaching this class of problems is that of selecting a robust and accurate numerical scheme, thus a suitable numerical code, to solve it.

All the above issues are addressed in this paper, which describes the different numerical approaches that were adopted to analyse the seismic site-effects expected at a real site. This latter is characterized by a soft soil valley and rock outcrop hills of relatively complex geometry. It was analysed by performing time domain 1D and 2D Finite Element (FE) analyses through two different codes: QUAKE/W, based on the equivalent-linear approach, and PLAXIS 3D, adopting the Hardening Soil model with small strain stiffness available in the material model library. 3D simulations were also carried out by means of the code PLAXIS 3D. A real accelerogram was selected as the outcrop motion, characterized by a duration of 22 seconds and a frequency content ranging from 0 to 10 Hz.

In order to validate the two FE codes, the results of 1D analyses relative to specific soil columns extracted by the site were preliminary compared to those obtained by the code EERA, based on the equivalent-linear approach in the frequency domain. Analogous results were obtained, indicating a low impact of the constitutive assumptions in the investigated case study, provided non-linearity was similarly accounted for in the different adopted approaches.

Next, the influence of the geometrical schematization was analysed, revealing that the seismic site response at the same location can strongly depend on the employed dimensional approach (i.e. 1 to 3D).

In general, assuming the outcrop signal as the reference motion, greater amplifications were observed at the centre of the valley and, with a minor extent, at the upper sides of the hills: those trends were significantly influenced by the multidimensional schematization (2 and 3D), while they were less evident or barely detectable in the 1D approach.

Keywords: seismic site response; multi-dimensional numerical simulation; site effects; Finite Element method; non-linear analysis

1. INTRODUCTION

The seismic response of a real site depends on the local stratigraphy, the geometry of ground surface and the dynamic behaviour of soil deposit. The so called local effects, related to the site conditions, may lead to different patterns of instability process and of damages on structures and infrastructures.

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due to the same seismic event (e.g. Earthquake Spectra 1988, Bulletin of the Seismological Society of America 1991).

Different geometrical schematizations can be adopted to study the local seismic response, namely:
• one-dimensional (1D): this scheme is appropriate for deposits having horizontal ground surface and soil layers (homogeneous or vertically layered deposit);
• two-dimensional (2D): this approach should be adopted if a vertically and laterally heterogeneous deposit is investigated (i.e. irregular ground surface and/or soil layers);
• three-dimensional (3D): necessary to describe the local effects for a vertically, laterally and transversally heterogeneous deposit.

The 1D geometrical schematization can handle the so called stratigraphic effects, which causes modifications of ground motion in terms of peak ground acceleration and frequency content (Kramer 1996).

2D approaches can be employed to investigate more complex geometrical conditions, as those represented by an irregular contact between the bedrock and a homogeneous soil deposit, as pointed out by previous studies using analytical solutions (Aki et al. 1970, Bard and Bouchon 1980a, 1980b, Geli et al. 1988) or numerical models (Bouckovalas and Papadimitriou 2005, Gatmiri and Arson 2008, Rizzitano et al. 2014).

The presence of a soft soil valley and/or of a hill, possibly associated to the focalization of seismic waves at the center of the valley’s ground surface and at the crest respectively, should lead to the adoption of 2D or 3D numerical schematizations.

Nowadays, the increasing computational power allows the numerical simulation of the seismic propagation processes to be carried out by rather large 3D numerical models (Smerzini et al. 2011, Sahar and Narayan 2015, Amorosi et al. 2016, Hartzell et al. 2016). Related to this, it is worth remarking that whatever is the adopted approach, a numerical procedure should always be validated against alternative solutions prior to tackle such complex geomechanical applications (Maufroy et al. 2016, Régnier et al. 2016).

Following the above statement, in this paper two different finite element (FE) codes, QUAKE/W (Krahn 1994) and PLAXIS 3D (PLAXIS 2016) are used to perform seismic ground response analyses under 1D and 2D conditions. In addition, more complex 3D analyses are also carried out by means of the PLAXIS code. All the analyses are aimed at predicting the seismic response at a specific location, the Bovino village in the northern part of the Apulia region (Italy), which is characterized by a rather complex geological setting and topographic surface.

2. GEOLOGICAL AND GEOTECHNICAL MODEL

The geological model was based on the information derived from the following investigations: 12 continuous coring boreholes, 10 to 40 m deep, 8 shear wave velocity profiles, determined by means of down-hole prospections, and 8 measurements of horizontal to vertical spectral ratio. Moreover, 32 undisturbed soil samples were collected during the borehole drilling.

The geological setting adopted here is that proposed by Petti (2010) and Cotecchia et al. (2016); the same Authors also pointed out the presence of landslide phenomena in the Bovino urban area, which will not be dealt with in the following for the sake of brevity.

The area under investigation is characterized by four geological units briefly described in the following:
• Bovino Synthem (BOV) that is a Pliocene sedimentary succession formed by silty clays with sands, lying on grain-supported conglomerates and sandstones;
• Faeto Flysch Formation (FAE), which can subdivided into an upper marly-calcareous member (FAE1) and a lower clayey-marly one (FAE2). FAE1 consists in calcarenites and calcareous marls interbedded with a few clayey levels, while FAE2 is formed by high plasticity, locally sheared, clays interbedding few and thin calcareous strata;
• Unit W1 representing the effects of either shallow landsliding or weathering in the FAE2 member, or remoulding due to tectonic movements;
• Unit V1 that is the result of further fluvial remoulding of FAE2 clays.

The geotechnical model was derived from the geological one by comparing boreholes and $V_S$ logs to
highlight strata characterized by the same mechanical behaviour (Falcone 2017). It results in three geotechnical units for each of which a constant value of the shear wave velocity with depth was assumed, as summarized in Table 1.

The map of the outcropping geotechnical units is sketched in Figure 1a), while 1b) shows three geotechnical sections. It can be observed that the maximum depth of Unit 1 is about 30 m, decreasing from section G1 to G3, and the average Unit 2 thickness is 10 m.

Table 1. Characteristics of the geotechnical units.

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>Geotechnical Unit</th>
<th>$\gamma_{sat}$ (kN/m$^3$)</th>
<th>$V_S$ (m/s)</th>
<th>$\nu$</th>
<th>PI (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1, W1$^1$</td>
<td>1</td>
<td>18</td>
<td>200</td>
<td>0.25</td>
<td>22</td>
</tr>
<tr>
<td>W1$^2$</td>
<td>2</td>
<td>18</td>
<td>800</td>
<td>0.25</td>
<td>22</td>
</tr>
<tr>
<td>FAE, BOV</td>
<td>3</td>
<td>18</td>
<td>1200</td>
<td>0.25</td>
<td>/</td>
</tr>
</tbody>
</table>

$^1$ inside the main landslide area  
$^2$ outside the landslide area

Figure 1. a) Map of the outcropping geotechnical units in Bovino urban area and b) geotechnical sections

Due to the lack of specific laboratory tests, $G(\gamma)/G_0$ and $D(\gamma)$ curves for the geotechnical Units 1 and 2 were based on literature data, selected as a function of the plasticity index PI = 22% (see Table 1). In particular, the analytical expression formulated by Darendeli (2001) was calibrated against the one proposed by Vucetic and Dobry (1991) for soils having PI = 15% and PI = 30% (Figure 2).

The geotechnical Unit 3, considered as the elastic seismic bedrock, was assumed to behave linearly. A 3D model was defined based on the geological and geotechnical settings discussed above, appropriately simplified to be implemented in the adopted numerical FE codes.

3. REFERENCE MOTION

The Calabria earthquake, occurred on March the 11th 1978 and characterized by a moment magnitude of Mw = 5.2, was selected as the reference outcrop motion, after a preliminary seismic hazard analysis (Falcone 2017; http://esse1-gis.mi.ingv.it/; Grupp di Lavoro, 2004). The signal corresponds to the waveform 169, earthquake 80, recorded at the station ST45 of the ESD on a site class A (http://www.isesd.cv.ic.ac.uk/; Ambraseys et al. 2000, 2004). In particular, its y component, scaled up to 0.22g and low filtered assuming $f_{max} = 10$ Hz, was considered in the numerical analyses.

The accelerogram and the Fourier spectrum of the scaled and filtered seismic event are shown in Figure 3.
4. NUMERICAL MODEL

The Bovino urban area was covered with 22 sections, according to the grid shown in Figure 4. The local reference system for the examined sections, together with their 3D surface patterns, are reported in Figure 5. In particular, sections 1-7, aligned along the x-axis, develop along the SW-NE directions, while sections 8-22, oriented as the y axis, follow a NW-SE direction. The first set of sections covers a length of 1400 m, while the second group has a variable extension, up to 1000 m, consistently with the variable dimension of the village.

1D vertical columns, fifty meters apart from each other, were also selected along different positions of the sections (see the black dots in Figure 5).

The thickness of the geotechnical Unit 1 is represented in Figure 6. It ranges from 2 to 30 m, this latter reached for x between 200 and 600 m along section 6. Sections 11, 15, 3 and 7 bound the valley area.

The numerical simulations were carried out in the time-domain by the FE codes QUAKE/W and PLAXIS adopting 1D and 2D geometrical schematizations. Additional 3D analyses were also performed using the code PLAXIS.

For the sections aligned along x-axis the boundary conditions employed in the two numerical codes are summarized in Table 2. For those along the y-axis, similar boundary conditions were adopted.
Figure 5. Considered sections and related reference system: a) plane view, b) 3D view

Figure 6. Thickness of geotechnical unit 1 in the 3D numerical model

Figure 7. Shear stiffness ratio $G(\gamma)/G_0$ and damping ratio $D(\gamma)$ curves predicted by the adopted constitutive models in QUAKE/W and PLAXIS

The quantity $\tau_{\text{bedrock}}(t)$ used in the QUAKE/W analyses is defined as following:

$$\tau_{\text{bedrock}}(t) = 0.5 \cdot V_{S,\text{bedrock}} \cdot \rho_{\text{bedrock}} \cdot v_{\text{outcrop}}(t)$$

where $V_{S,\text{bedrock}}$ and $\rho_{\text{bedrock}}$ are the shear wave propagation velocity and the mass density of the bedrock, respectively, and $v_{\text{outcrop}}$ is the time history of the velocity recorded at the outcrop. The outcrop signal is that presented in § 3.

The maximum finite element size, $h_{\text{max}}$, was determined according to the following relationship (Bathe 1996):

$$h_{\text{max}} = \frac{V_s}{8 - 10 \cdot f_{\text{max}}}$$

where $V_s$ is the shear wave propagation velocity of the soil and $f_{\text{max}}$ is the maximum frequency of the outcrop signal.

The time step of each numerical analysis is assumed as equal to the sample time of the reference motion (Bathe 1996).

An equivalent linear elastic approach is available in the QUAKE/W software. The decay curves of the shear stiffness and damping ratio adopted for the geotechnical Units 1 and 2 are shown in Figure 7.

The PLAXIS simulations were performed using a non-linear constitutive model named Hardening Soil Model with small strain stiffness (HSsmall in the following), available in the material library of the code (Benz 2006, Benz et al. 2009, di Lernia 2014, Amorosi et al. 2014, 2016, PLAXIS 2016). Table 3 summarizes the parameters selected for the geotechnical Units 1 and 2, while the corresponding
stiffness and damping ratio curves are reported in Figure 7. Rayleigh damping was also implemented to provide a small amount of damping ($D_0 = 1.05\%$) at the very small strain level, as suggested in Régnier et al. (2016) and Falcone (2017).

To compare the numerical results obtained by the two FE codes and the related soil constitutive models, the strength parameters of the HSsmall model were fixed at very high values, such that plasticity would not be activated during the analyses. Similarly, the initial dimension of the shear hardening yielding surface was artificially enlarged, by simulating loading and unloading phases, to obtain a purely non-linear para-elastic response in the calculations (di Lernia 2014).

Table 2. Boundary conditions for numerical model.

<table>
<thead>
<tr>
<th>Vertical boundary</th>
<th>Horizontal base</th>
<th>Ground surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>QUAKE/W</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$u_x$ = free</td>
<td>$\tau_x = 2\cdot\tau_{bedrock}(t)$</td>
<td>$u_x = u_z = $ free</td>
</tr>
<tr>
<td>$u_z = 0$</td>
<td>$\tau_z = 0$</td>
<td></td>
</tr>
<tr>
<td>PLAXIS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$u_x$ = free</td>
<td>$a_x = 0.5\cdot a_{outcrop}(t)$</td>
<td>$u_x = u_y = u_z = $ free</td>
</tr>
<tr>
<td>$u_y = u_z = 0$</td>
<td>$a_y = a_z = 0$</td>
<td></td>
</tr>
<tr>
<td>free-field</td>
<td>compliant base</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. Material parameters of the HSsmall model for the geotechnical Units 1 and 2.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Name</th>
<th>Unit 1</th>
<th>Unit 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure parameters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$c$ (kPa)</td>
<td>Effective cohesion</td>
<td>1E35</td>
<td>1E35</td>
</tr>
<tr>
<td>$\varphi$ (°)</td>
<td>Effective friction angle</td>
<td>89</td>
<td>89</td>
</tr>
<tr>
<td>$\psi$ (°)</td>
<td>Dilatancy angle</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Stiffness parameters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$m$ (-)</td>
<td>Power for the stress-level dependency of stiffness</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$E_{50,ref}$ (MPa)</td>
<td>Reference secant stiffness in standard drained triaxial test</td>
<td>21.8</td>
<td>349.5</td>
</tr>
<tr>
<td>$E_{oed,ref}$ (MPa)</td>
<td>Reference tangent stiffness for primary oedometer loading test</td>
<td>21.8</td>
<td>349.5</td>
</tr>
<tr>
<td>$E_{ur,ref}$ (MPa)</td>
<td>Reference unloading/reloading stiffness at engineering strains</td>
<td>65.5</td>
<td>1048.5</td>
</tr>
<tr>
<td>$\nu_{ur}$ (-)</td>
<td>Poisson’s ratio for unloading/reloading</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>$G_{0,ref}$ (MPa)</td>
<td>Reference shear modulus at very small strains</td>
<td>73.4</td>
<td>1174.3</td>
</tr>
<tr>
<td>$\gamma_{0.7}$ (-)</td>
<td>Shear strain at which $G_s = 0.722G_{0,ref}$</td>
<td>0.025</td>
<td>0.025</td>
</tr>
<tr>
<td>Other parameters</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$p_{ref}$ (kPa)</td>
<td>Reference stress for stiffness</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$R_i$ (-)</td>
<td>Failure ratio</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>$\sigma_t$ (kPa)</td>
<td>Tensile strength</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$\epsilon_{increment}$ (kPa/m)</td>
<td>Increment of cohesion with depth</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
5. RESULTS

In the following the results are first discussed comparing the outcomes of the two different codes and then focusing on the adopted geometrical schematization, considering the 1D, 2D and 3D numerical solutions. For the sake of brevity, in the following reference is only made to sections 4 and 6, which are considered as representative of the case under study.

A synthetic representation of the numerical results obtained by the 2D QUAKE/W analysis for section 4 is presented in Figure 8 in terms of acceleration time histories as predicted at the ground surface. The reference motion is also included in the upper part of the graph. The position of each calculated signal along the geotechnical section can easily be detected by this latter as plotted on the left side on the figure. The black line indicates the topographic surface, while the grey and black dashed lines represent the interfaces between the geotechnical units.

Figure 8 clearly shows that the 2D simulation catches both the amplification due to the presence of the hill (between 1000 < x < 1200 m) and that in the valley area (between 500 < x < 700 m).

For the same section, a comparison in terms of pseudo-velocity response spectra is provided in Figures 9 and 10 for the 1D and 2D schemes respectively. The data refer to point (600, 700), in the middle of the valley at the intersection of sections 14 and 4 (see Figure 5), and point (1300, 700), near the top of the hill identified by sections 21 and 4 (see Figure 5). The 1D seismic response results (Figure 9) are also compared to those obtained by the code EERA, which adopts the equivalent-linear approach in the frequency domain (Bardet et al. 2000). Both Figures 9 and 10 highlight the very good agreement among the results obtained by the different codes. In fact, Figure 9 demonstrates the reliability of the FE approach in comparison to the well-established frequency domain solution provided by the EERA analysis, while Figure 10 illustrates the comparable results obtained under plane strain conditions by the two different FE codes.

The central valley, characterized by the presence of a soft material on top of a stiffer one, leads to larger values of spectral velocity as compared to those detected at the outcropping hill, both in the 1D and 2D geometrical schematization.

Figure 8. Accelerograms predicted at the ground surface of section 4 by means of 2D QUAKE/W calculation

Figure 9. Comparison between EERA and 1D FE results in terms of pseudo-velocity response spectra for a) point (600, 700) and b) point (1300, 700)
The evaluation of the site effects computed by the different numerical approaches was also carried out in terms of different amplification factors, as proposed in several studies (Gruppo di Lavoro MS 2008, Santucci de Magistris et al. 2014). In particular, the adopted amplification factors, referring to the ground surface point \((x, y, z)\), are:

\[
F_{H_{01-05}}(x, y, z) = \frac{\int_{0.1}^{0.5} PSV_i(x, y, z) dT}{\int_{0.1}^{0.5} PSV_o dT} 
\]

(3)

\[
F_{H_{05-1}}(x, y, z) = \frac{\int_{0.5}^{1} PSV_i(x, y, z) dT}{\int_{0.5}^{1} PSV_o dT} 
\]

(4)

\[
F_{PGA}(x, y, z) = \frac{PGA_i(x, y, z)}{PGA_o} 
\]

(5)

where PGA is the peak ground acceleration, PSV is the pseudo-spectral velocity (5% structural damping) and \(T\) is the period. The subscripts \(i\) and \(o\) refer to the outcrop signal and to the output one, determined at the ground surface by means of numerical analyses. The subscripts 01-05 and 05-1 indicate the period intervals considered in the integral quantities. In particular, this selection corresponds to the range of periods containing the dominant period of vibration of buildings 6 or more floors tall, respectively.

Figure 11 shows the results obtained by the 2D QUAKE/W and PLAXIS analyses for section 4 in terms of the amplification factors. As already discussed before, the results obtained by the two FE codes are in very good agreement. The presence of the valley amplifies of three times the reference signal in the range of period \(0.1 < T < 0.5\) s (Figure 11a) and of about twice in the range \(0.5 < T < 1\) s (Figure 11b). Also the PGA is significantly increased at the centre of the valley, while elsewhere stays between 1 and 1.4 times the reference one. Values of amplification factors slightly greater than 1 are attained at the crest of the outcropping hill, while at the toe of it a slight de-amplification of the reference motion occurs.

Finally, Figures 12 and 13 summarize the outcome of 1D, 2D and 3D PLAXIS analyses for sections 4 and 6 respectively. The acceleration time histories determined at the ground surface by 2D analyses refer to points minimum five and maximum fifty meters apart.

Section 4 (Figure 12) is characterised by similar amplification factors irrespective of the geometrical schematization adopted in the numerical simulation, thus indicating a prevailing stratigraphic influence on the computed amplification factors.

On the contrary, for section 6, characterized by the deepest and largest valley condition (Figure 13),
the 3D seismic amplification in the valley area is about twice than that predicted by 1D and 2D schemes in the period range $0.5 < T < 1$ s (Figure 13b).

More in general, it is possible to observe that the profiles of the three amplification factors computed for the shallow and narrow shape of the valley along section 4 are characterized by a single peak value, while those obtained for the deep and large basin of the valley along section 6 display two different peaks.

Figure 11. Comparison between 2D profiles of amplification factors obtained by means of PLAXIS and QUAKE/W analyses for section 4: a) FH\textsubscript{01-05}, b) FH\textsubscript{05-1} and c) F\textsubscript{PGA}

Figure 12. Comparison between 1D, 2D and 3D amplification factors obtained by PLAXIS analyses for section 4: a) FH\textsubscript{01-05}, b) FH\textsubscript{05-1} and c) F\textsubscript{PGA}
6. CONCLUSIONS

The seismic response of the Bovino urban area, a village located in Southern Italy, was evaluated by means of different numerical approaches.

Two different FE codes were adopted: QUAKE/W, based on the equivalent-linear approach, and PLAXIS 3D, which employs the Hardening Soil model with small strain stiffness available in the material model library.

The simulation of site response was carried out with reference to 1D, 2D and 3D schemes, the latter being only performed by the PLAXIS software. A real accelerogram, recorded at the outcrop, was selected as the reference signal.

The satisfactory agreement between the results obtained by the 1D QUAKE/W and PLAXIS analyses with those of the more traditional EERA ones, conducted in the frequency domain, proved the reliability of the adopted numerical approaches to simulate the seismic response of the selected real site.

The predicted signals at the ground surface result amplified in the valley and hill areas and slightly demagnified at the toes of the hill.

In general, local effects, and especially the valley one, are the results of both stratigraphic and geometric features, related to the profiles of the soil layers below the topographic surface, and as such can only be detected by 2D and 3D schematizations. The amplification pattern found for the shallower and narrower portion of the valley (e.g. along section 4) seems to be only affected by the local stratigraphic conditions, since the 1D, 2D and 3D simulations provided very similar results. On the contrary, where the valley is deeper and larger, the 3D approach predicted larger values of the amplification factors than the other geometrical schemes.

In conclusion, thanks to nowadays computing capacity, multi-dimensional numerical analyses allow to understand the seismic response of an area characterised by complex geology and irregular ground surface, as the one studied here, for which in situ measurements of strong seismic events are barely available.
7. ACKNOWLEDGMENTS

Financial support provided by Autorità di Bacino della Puglia (ADBP), Eng. Isabella Trulli of ADBP who strongly encouraged Seismic Microzonation of Apulia region and Eng. Micha van der Sloot of PLAXIS support are gratefully acknowledged.

Authors acknowledge Dott. Francesca Santaloia of National Council of research (CNR-IRPI), Prof. Federica Cotecchia of Technical University of Bari (POLIBA) and Eng. Rossella Petti (POLIBA) for their helpful support and wide personal communication in order to understand and define the complex geological setting of Bovino urban area.

Finally, Eng. Francesco Mazzone is kindly acknowledged for his intense support in defining the Bovino topography and creating the CAD model.

8. REFERENCES


