Chapter 10

Seismic response analysis of Monte Po hill (Catania)

G. Biondi & M. Maugeri

*Dipartimento di Ingegneria Civile ed Ambientale, University of Catania*

**Abstract**

In the paper the results of a large number of seismic response analyses of the Monte Po hill (Catania, Italy) are presented. The interest for this site is due to the presence of several constructions, including a school, in the vicinity of the toe of the slope. In the past, these structures suffered damage related to the precarious stability conditions of the slope and the consequences of some excavation performed during the construction of some buildings in the area. Since all the damaged buildings are still in use, a study of the seismic stability condition of the slope is required. To develop a reliable soil model, available *in situ* and laboratory test results were analysed. The seismic response analyses were performed using two records of the December 13, 1990 Santa Lucia Earthquake and artificial acceleration time-histories developed referring to a $M=7.0$ scenario earthquake. The results show significant differences in the hill response depending on the dynamic characteristics of the involved soils. In particular, considerable effects are related to the variation of the shear modulus and damping ratio with shear strain adopted in soil modelling. The obtained acceleration responses show the possible occurrence of permanent deformations of a potentially unstable soil mass; these phenomena can significantly affect the post-seismic serviceability conditions of some structures involved in the hill.

1 Introduction

Monte Po hill is located in the north-eastern part of the Catania district. In the area a school and private constructions were built in the past in the environs of the hill and sometimes near the toe. Then, high-risk conditions arose since the
stability of the area is poor even under static loading conditions. During the first construction period, instability phenomena occurred in the hill without causing an interruption in the works. At the same time damage to some of the buildings existing near the hill occurred and failures of several earth-retaining structures were observed in the area. Later, damage and instability phenomena were observed as a consequence of meteoric events, during and after some excavations performed near the toe and, finally, after the 13 December 1990 Santa Lucia earthquake.

Due to the significant geotechnical seismic hazard related to this site intensive research activity was performed in the framework of the research project “Detailed scenarios and actions for prevention of seismic damage in the urban area of Catania” (Maugeri [1]). In particular, Grasso et al. [2] performed a geotechnical characterization of the area and stored the obtained data in a digital database. Following on, the area of study was included in the landslides hazard zonation proposed by Grasso & Maugeri [3] for the Catania district.

The aim of the present research was the evaluation of the slope stability condition under seismic loading. To this end, a scenario earthquake for the area under study was selected and several seismic response analyses were performed.

In order to obtain a reliable model of the subsoil, data concerning the soil geotechnical properties were collected using both in situ and laboratory test results. In particular, results of the geotechnical characterization performed during the different building activities in the area was considered. Moreover, data obtained during the research programme carried out by the geological office of the Catania municipality, as a consequence of the instability phenomena which occurred in the area, were also analysed.

In this paper the procedure adopted to detect the more reliable subsoil model to be used in the local seismic response analyses is described. The results were analysed in an attempt to evaluate the possible occurrence of an earthquake-triggered landslide, focusing attention on the effects of the earthquake-induced permanent displacements and on the post-seismic serviceability of the structures involved in the area.

2 Description of the area

Monte Po hill is located in a populated area of the western part of Catania. Thus the study area is located in one of the most seismically active zones of the Mediterranean. In the last 900 years, the east cost of Sicily has been struck by various disastrous earthquakes with MKS intensity varying in the range IX-XI, and estimated magnitudes ranging from less than 5.0 to greater than 7.0. The most probable source of earthquakes in the area is the Malta Escarpment, a system of sub-vertical normal faults, NNW-SSE oriented, which runs for about 70-100 km offshore along the Ionian coast of Sicily. This structure appears to be subdivided into different segments, the northernmost ones bordering the eastern Hyblean coast and extending inland as far as the Etna volcano area.

Using the historical seismicity data available for the area it is reasonable to assume as a maximum expected earthquake the repetition of the two $M > 7$
events which hit western Sicily in the past and destroyed Catania: the 1169 and the 1693 earthquakes. These catastrophic events were characterized by intensities equal to X MCS and XI MCS and estimated magnitudes equal to 7.0-7.4. According to Azzaro et al. [4] the first level scenario event for the Catania area may be reasonably assumed as the January 11, 1693 earthquake that caused the largest seismic catastrophe in eastern Sicily. The most probable source of this event is located along the northern part of the Ibleo-Maltese fault and is commonly associated with rupture to a normally pure mechanism along the escarpment (Postpischl [5]).

Concerning local characteristics of the area, the slopes of the hill range from 0°-5° in the flatter area near the toe, to 15°-20° in the hilltop area. The topography of the area is shown in Fig. 1a). In the site, a deep layer of grey clay represents the base formation and layers of conglomerate and sand outcrop.

Near the toe of the hill a school was built in 1991 and several constructions for lodgings were built in the seventies by the Istituto Autonomo Case Popolari (IACP) of the Catania municipality. Over this time, several instability phenomena were observed in the hill, especially during the building activities.

In particular, at the beginning of 1991 during the construction of the school, a shallow instability phenomenon occurred causing high-risk conditions for the work-yard of the school and for the existing IACP lodgings. In Fig. 1b) a detail of the area of interest is shown together with the location of the school and of the IACP lodgings that suffered damage. As a consequence of this instability, the Catania municipality carried out an intense site investigation and monitoring of the hill with piezometers and inclinometers. The excavation performed near the toe of the hill during the activities in the school work-yard and the intense rainfall which occurred some days before the 1991 landslide, were identified as the main causes of the instability.

These phenomena mainly concerned the north-western part of the hill and were characterized by both rotational and translative failure mechanisms that involved the school and the IACP buildings. However, shallow instability and toppling of the outcropping conglomerates were also observed in the north-eastern part of the hill; in this side of the hill the instability did not cause significant damage due to the absence of structures and infrastructures. In Fig. 1b) the localization of the two soil masses involved in the landslide occurred in the north-western (NW) and in the north-eastern (NE) part of the hill as shown. Several months after the end of the construction activities at the school, another instability phenomenon occurred without of any other construction activities. These phenomena damaged the surface drainage system installed on the hill (Fig. 1b) and some existing earth-retaining structures that were built to stabilize the area. Moreover, significant damage was also observed in the structure of the school building and in the earth-retaining structure delimiting the school area. Concerning the IACP lodgings, less evident damage was observed due to the presence of the earth-retaining wall protecting the buildings from earth-slides.

The results of the research and of the monitoring activities performed during the whole of 1991 allowed us to characterize the instability phenomena, in particular, the phenomena consisting of shallow movements of the upper
conglomerates and sand layers on the clay base formation. For the north-western soil mass a maximum length and width equal to 180 m and 100 m respectively were estimated and a medium depth equal to 10 m were detected based on investigations. One of the causes of the instability was detected as the effect of the 13 December 1990 earthquake that shook the site about one or two months before the start of the work for construction of the school. In particular, the soil shear strength reduction due to the earthquake-induced pore pressure could be considered among the possible causes that triggered the instability.

Figure 1: Plan view of the area of study: a) topography of the area; b) detail of the hill.

Figure 2: a) Location of the boreholes (S), piezometers (P) and inclinometers (I); b) location of the down-hole (DH) and seismic refraction (TSR) tests.
Concerning the reduction of hazards in the area, the north-western unstable soil mass represents the main problem; in particular the school building located at the toe of the hill and in use at the moment represents the major concern. Since the IACP lodgings are placed in the lateral side of the unstable area and are protected by earth-retaining structures, they appear to be less exposed to potential instabilities.

3 Geotechnical characterization

Several geotechnical investigations have been performed in the past due to the intense construction activities in the area at the toe of the hill and near the hill. In Table 1, the main characteristics of the performed investigation are presented.

In the seventies, during the activities related to the construction of the IACP lodgings, three boreholes (S) and laboratory and *in situ* tests were carried out. In particular, laboratory tests were performed on three undisturbed samples, consisting of tests for soil description and classification, direct shear tests and oedometer tests. At the site, seismic refraction (TSR) and electrical survey (SE) were also carried out.

In 1990 geotechnical investigations were performed for the construction of the school building; additional investigations were carried out in 1991 after the instability phenomena occurred during and after the construction of the school.

Four boreholes (S) were performed in the area near the toe of the slope; standard penetration tests (SPT) and cone penetration tests (CPT) were also performed; finally, laboratory tests for soil description and classification, direct shear tests and oedometer tests were carried out on seven undisturbed samples.

Due to the high-risk condition of the hill the Catania municipality carried out geotechnical investigations in 1997 together with monitoring activities of the slope. In detail, eight boreholes (S) and several laboratory and *in situ* tests were performed; moreover, inclinometers and piezometers were installed both in the detected NE and NW unstable soil masses. Standard penetration tests (SPT), seismic refraction (TSR) and seismic down-hole tests (DH) were also carried out. Finally, laboratory tests for soil description and classification were performed together with direct shear tests, triaxial tests and oedometer tests on 19 undisturbed samples.

Table 1: Description of the geotechnical investigation performed in the area of study.

<table>
<thead>
<tr>
<th>Year</th>
<th>Boreholes</th>
<th>Penetration tests</th>
<th>Seismic refraction test</th>
<th>Laboratory tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971</td>
<td>3</td>
<td>-</td>
<td>8 TSR</td>
<td>Soil description and classification, Direct shear test, Oedometer tests</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>11 SE</td>
<td>Soil description and classification, Direct shear test, Oedometer tests</td>
</tr>
<tr>
<td>1990</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>Soil description and classification, Direct shear test, Oedometer tests</td>
</tr>
<tr>
<td>1991</td>
<td>4</td>
<td>5 SPT 14 CPT</td>
<td>3 SE</td>
<td>Soil description and classification, Direct shear test</td>
</tr>
<tr>
<td>1997</td>
<td>8</td>
<td>30 SPT</td>
<td>7 TSR</td>
<td>Soil description and classification, Direct shear test</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2 DH</td>
<td>Triaxial tests, Oedometer tests</td>
</tr>
</tbody>
</table>
Fig. 2 shows the location of the boreholes and of the in situ tests carried out during the investigations. In Fig. 3 the results of the down-hole tests and of the standard penetration tests performed during the 1997 investigations are shown.

Figure 3: a, b) Profile of the soil shear wave velocity obtained in the DH tests performed in the 1997 geotechnical investigation; c) profile of the shear wave velocity adopted in the seismic response analyses; d) results of SPT.

Using the results of all the investigations a scheme of the soil profile was detected for each of the boreholes. In particular, attention was focused on the boreholes of the 1991 and 1997 investigations which seem to be more accurate with respect to those performed in 1970. Figs. 4 and 5 show the profiles obtained from the investigations of 1991 and 1997 respectively.

Generally, a thin layer of altered soil can be observed in the area with thickness ranging from 0 to about 1 m. Then, four different units can be recognized: a layer of medium-stiff alluvial silt of medium plasticity with thickness ranging from 50 cm to 4 m; a layer of very sandy gravel which was detected only in some boreholes with thickness ranging from 25 cm to about 4 m; a formation of conglomerate and sand with thickness ranging from 50 cm to about 3 m; finally a layer of clay of upper plasticity range locally representing the subgrade.
Concerning the clay and silt layers, the laboratory test results obtained during the 1997 investigation represent the most accurate data. During the laboratory activities 16 of the 19 undisturbed samples were analysed obtaining the physical properties listed in Table 2. The soils are fully saturated starting from a depth of about 5 m from the ground surface coincident with the contact between sand and silt. As a result of SPT shown in Fig. 3d) the sand and gravel layers can be classified as dense or medium dense soils. Steady flow seepage parallel to the clay layer was detected and a water table level coincident with the sand-silt interface was estimated using piezometric data.

From the results of the down-hole tests described in Figs. 3a) and b) average values of shear wave velocity equal to 300 m/s and 260 m/s were evaluated for boreholes 5 and 7 respectively. Using all the collected data, geotechnical sections of the hill were developed (Fig. 6). In particular three sections, denoted as AA, BB and CC in the plan view of Fig. 2, were considered. Section AA is orthogonal to the earth-retaining wall that delimits the IACP building nearest to
The sections of the hill previously described were analysed by Grasso et al. [2] through a stress-strain analysis performed in plane strain condition, with the finite difference code FLAC 2D referring to the static conditions. For the same sections, 1-D seismic response analyses were performed using Shake91 (Idriss & Sun [6]) with reference to the different soil profiles involved in each section.

Figure 5: Geotechnical schemes of 1997 boreholes adopted for seismic response analyses.

4 Geotechnical characterization

The sections of the hill previously described were analysed by Grasso et al. [2] through a stress-strain analysis performed in plane strain condition, with the finite difference code FLAC 2D referring to the static conditions. For the same sections, 1-D seismic response analyses were performed using Shake91 (Idriss & Sun [6]) with reference to the different soil profiles involved in each section.
Since the slopes of the hill are generally lower than 20°, a 1-D approach was adopted for seismic response analyses. In this paper, attention was focused on the soil columns S3 and S8 developed during the 1997 geotechnical investigation; these columns are located along section CC passing through the school building.

Figure 6: Geotechnical sections of the hill developed using the results of the performed investigations.
For the analyses some records of the 13 December 1990 earthquake were considered; in particular the two horizontal components of the Catania and Sortino records were selected. The Catania records refer to a soil site while the Sortino accelerograms were recorded on a rock-like site. Figs. 7 and 8 show the four accelerograms considered in the analysis together and the corresponding Fourier amplitude spectra.

Concerning the Catania records, since the accelerograms were recorded on the surface of a soil deposit, for each of the considered soil columns the analysis were carried out using a signal obtained through a deconvolution process. To evaluate the response of the slope to the selected earthquake scenario, artificial records developed by Grasso et al. [7] were adopted. In the Grasso et al. (2004) analysis the 1693 earthquake was assumed as the reference event for the Catania area and the source of the event is located along the northern part of the Ibleo-Maltese system, which is simplified in a segment about 25 km long. The source considered in the analysis consists of the summation of five subevents, which reproduce approximately the rupture propagation along the fault segment and the heterogeneous distribution of the seismic moment along the fault. In the Grasso et al. [7]) analysis a bi-dimensional vertical plane containing the source and different receivers was considered and the elastic behaviour of the rock is assumed to be linear and isotropic. Using the Chebyshev spectral element...
method to solve the propagation of the seismic field through the considered structural model, seismograms were computed in seven sites of the Catania Municipality.

Figure 8: Fourier amplitude spectra of the 13 December 1990 acceleration records adopted in the seismic response analyses.

Figure 9: Seismograms computed by Grasso et al. [7] for site n.3 at a depth of 31 m using the IBM B distribution of the seismic moment.

In particular seismograms were computed up to a maximum frequency of 8 Hz for a total propagation time equal to 20 s. For each site the seismograms were computed for six receivers located at increasing depths from the free surface to
about 170 metres deep. In the present paper the seismograms computed using the
distribution of the seismic moment denoted as IBM B for the site n. 3 at a depth
of 31 m were adopted (Fig. 9). This choice is due to the nature of the prevalent
lythology involved in the site n.3 that consists of a deep layer (thickness more
than 250 m) of clay interbedded with sand lying on a base formation of pliocenic
sediments and allochonous.

The results of DH tests previously described were used to evaluate the
profile of shear wave velocity $V_s$ in the soil columns shown in Figs. 4 and 5. In
Fig. 3c the profile obtained interpolating the experimental results and used in the
seismic response analyses of soil columns S3 and S8 is shown. For soil depths
down to 20 m the $V_s$ profile used in the analyses is that obtained from the DH
test performed in the borehole S7 (1997) while data obtained from the DH test
performed in the borehole S5 (1997) were excluded, since these are relevant to
the soils of the NE landslide. For soil depths larger than 20 m the $V_s$ profile was
obtained through a relationship proposed by Gazetas [8]. The complete profile of
the shear wave velocity assumed in the analyses is shown in Fig. 3c.

To account for the variation of the shear modulus $G$ and the damping factor
$D$ with the shear strain $\gamma$ some results obtained in clay sites in the city of Catania
(Carrubba & Maugeri [9]; Cavallaro & Maugeri [10]) and the experimental
curves proposed by Vucetic & Dobry [11] were considered. Consistent with the
laboratory test results reported in Table 2, the curves proposed by Vucetic &
Dobry [11] for $I_p = 0\%$, 15% and 30% were selected. Figs. 10 and 11 show the
curves $G-\gamma$ and $D-\gamma$ adopted in the seismic response analyses.

![Figure 10: Variation of the normalized shear modulus and of the damping factor with the shear strain obtained for clay soils of the city of Catania.](image)

Analyses were carried out using the accelerograms recorded during the
Santa Lucia earthquake, combining all the $G-\gamma$ and $D-\gamma$ curves with the soil
profiles shown in Fig. 12. For the soil column S3 four layers were considered:
Layer 1 consisting of the shallow soils and sandy silts down to 2.5 m, Layer 2 consisting of sandy gravels down to 5.5 m, Layer 3, only 30 cm thick, consisting of silty sands and Layer 4 consisting of silty clay down to 20 m. For the soil column S8 only three of the four layers described above were detected: Layer 1 down to 2.5 m, Layer 3 down to 5.7 m and Layer 4 down to 20 m.

For Layers 1 and 4 the $G\gamma$ and $D\gamma$ curves due to Carrubba & Maugeri [9] and Cavallaro & Maugeri [10] and also the curves given by Vucetic & Dobry [11] corresponding to $I_p = 15\%$ and 30\% were used. For sandy gravels (Layer 2) and silty sands (Layer 3) the curves given by Vucetic & Dobry [11] corresponding to $I_p = 0$ and 15\% and to $I_p = 15$ and 30\% were adopted, respectively. For each of the columns considered in the analyses two different bedrock depths were assumed: $z_b=20$ m and $z_b=30$ m for column S3; $z_b=20$ m and $z_b=40$ m for column S8. The thickness of Layer 4 was adjusted accordingly. Then, for each column, four schemes of analysis were considered, as shown in Fig. 12.

![Figure 11: Variation of the normalized shear modulus and of the damping factor with the shear strain obtained by Vucetic & Dobry [11].](image)

5 Description of seismic response analysis results

The study consisted of 64 seismic response analyses. In the following some of the results are presented focusing on the influence of different assumptions on maximum values of soil acceleration and shear strain.

In Figs. 13 and 14 the profiles of maximum acceleration and shear strain obtained using the Sortino record are shown with reference to the four subsoil schemes and the two different bedrock locations adopted in the analysis. Maximum acceleration values at the ground surface range between 0.17 g and 0.35 g. Maximum values of 0.30-0.35 g are attained using the scheme 4 and $z_b=30$ m for column S3 and $z_b=20$ m for column S8. The scheme of subsoil significantly affects the maximum value of the shear strain. In particular,
maximum values are observed for the scheme 4 at the level of gravels and silty sands for column S3 and at the level of silty sands and silty clays for column S8. Maximum shear strains are generally smaller than 0.05%, reaching only for the case of column S3 and scheme 4 the value of 0.085%, close to the volumetric threshold (Carrubba & Maugeri [9]; Cavallaro & Maugeri [10]).

Since maximum accelerations and maximum shear strains were generally obtained in the analyses performed using the scheme 4, this scheme was used to repeat analyses for both columns and for each column for both bedrock depths, assuming the scenario artificial accelerogram as input motion.

Fig. 15 shows the profiles of maximum acceleration, shear strain and shear stress, together with the amplification functions of the ground motion with respect to the bedrock motion and the acceleration time-histories obtained at ground level. In all the considered cases the ground peak acceleration was larger than 0.4 g, attaining maximum values of 0.515 g and 0.525 g for S3 and $z_0=20$ m and for S8 $z_0=40$ m, respectively. Maximum amplification factors equal to about 2.4 were observed. Moreover, in the topmost 10 m of the soil deposit maximum acceleration is always larger than 0.20 g, with a mean value of about 0.35 g.

<table>
<thead>
<tr>
<th>a)</th>
<th>S3</th>
<th>Scheme 1</th>
<th>Scheme 2</th>
<th>Scheme 3</th>
<th>Scheme 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5 m</td>
<td>1</td>
<td>$G_{y1}, D_{y1}: I_p=15%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: I_p=15%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: CCC$ Cavallaro &amp; Maugeri (1999)</td>
<td>$G_{y1}, D_{y1}: CPC$ Carrubba &amp; Maugeri (1988)</td>
</tr>
<tr>
<td>3.0 m</td>
<td>2</td>
<td>$G_{y1}, D_{y1}: I_p=0%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: I_p=15%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: I_p=0%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: I_p=0%$ Vucetic &amp; Dobry (1991)</td>
</tr>
<tr>
<td>8.7 m</td>
<td>4</td>
<td>$G_{y1}, D_{y1}: I_p=15%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: I_p=30%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: CCC$ Cavallaro &amp; Maugeri (1999)</td>
<td>$G_{y1}, D_{y1}: CPC$ Carrubba &amp; Maugeri (1988)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>b)</th>
<th>S8</th>
<th>Scheme 1</th>
<th>Scheme 2</th>
<th>Scheme 3</th>
<th>Scheme 4</th>
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<tbody>
<tr>
<td>2.5 m</td>
<td>1</td>
<td>$G_{y1}, D_{y1}: I_p=15%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: I_p=15%$ Vucetic &amp; Dobry (1991)</td>
<td>$G_{y1}, D_{y1}: CCC$ Cavallaro &amp; Maugeri (1999)</td>
<td>$G_{y1}, D_{y1}: CPC$ Carrubba &amp; Maugeri (1988)</td>
</tr>
</tbody>
</table>

Figure 12: Geotechnical schemes of the boreholes S3 (1997) and S8 (1997) adopted for seismic response analyses.
Figure 13: Profiles of maximum acceleration and shear strain obtained for the borehole S3 (1997) using the Sortino record and two different bedrock locations: a, b) Component EW, \( z_b = 20 \) m; c, d) Component EW, \( z_b = 30 \) m; e, f) Component NS, \( z_b = 30 \) m.

Finite difference stability analyses, carried out using the code FLAC2D (Grasso et al. [2]), showed that in this shallow part of the soil deposit instability might occur; in particular a potential translational failure mechanism was detected for each of the three sections shown in Fig. 6. For the same sections limit equilibrium analyses were also performed; in particular, in order to estimate the displacement response of the hill, values of slope critical accelerations were evaluated using the pseudo-static approach and referring to the infinite slope.
scheme. For section CC, critical acceleration values ranging from 0.09 g to 0.27 g were evaluated using both peak and residual strength parameters and considering different directions of the seismic actions and depths of failure surface involved in the mechanisms. Thus, results obtained from both pseudo-static and seismic response analyses point to a condition of potential instability of the area that under seismic loading may undergo permanent deformations.

Figure 14: Profiles of maximum acceleration and shear strain obtained for the borehole S8 (1997) using the Sortino record and two different bedrock locations: a, b) Component EW, z₀=20 m; c, d) Component NS, z₀=20 m; e,f) Component NS, z₀=40m.
Figure 15: Seismic response of the boreholes S3 (1997) and S8 (1997) to the artificial seismogram: a) profiles of maximum acceleration; b) profiles of maximum shear strain; c) profiles of maximum shear stress; d) amplification function; e, f, g, h) acceleration responses at ground surface.
In all the considered cases the maximum seismic-induced shear strains were almost constant with depth with values larger than 0.05%. For column S3, at the silty sand and gravel layers maximum shear strains are significantly larger than the volumetric threshold (Cavallaro & Maugeri [12]), implying the possible occurrence of excess pore water pressures that may affect severely the slope stability in seismic and post-seismic conditions.

Comparing the ground acceleration time-histories (Figs. 15 e, f, g, h) with the input motion (Fig. 9) it can be observed that the soil deposit induces a remarkable change in the frequency content. Amplification functions show that motion is amplified in a wide range of frequencies for both the soil columns. Maximum amplification for column S3 occurs at 3-4 Hz and the soil response is mostly affected by large strains in the gravel and silty sand layers. For column 3 and $z_b=40$ m the input motion is amplified in the whole range of frequency of practical interest; larger strains are obtained when the bedrock depth is at $z_b=20$ m, and the non-linear effects produce in this case smaller amplifications and even de-amplification of the high frequency components.

6 Conclusion

In this paper, some results of an extensive study of the seismic response of the hill of Monte Po were presented. The site is characterized by a stiff clay subgrade overlain by alluvial silts, sands and gravels.

The mechanical properties of the soils were obtained through a comprehensive amount of data available for the area of study from both in situ and laboratory test results. Concerning the cyclic behaviour of the soils, data from cyclic laboratory test results performed on soils of the Catania area were considered. In particular, to account for the variation of the shear modulus and of the damping factor with the shear strain, experimental results obtained for the clay soil of the Catania plain area and for the central Catania clay were adopted. Moreover available published results concerning the variation of shear modulus and damping factor were also considered in the soil modelling.

The seismic response analyses were performed using different subsoil schemes and assuming two possible bedrock depths; moreover both real and artificial earthquake records were considered in the analyses. In particular, the Catania and Sortino records of the 13 December 1990 earthquake were adopted; the analyses were performed considering also an artificial seismogram developed in a 2-D spectral element analysis assuming the 1693 earthquake as the scenario event for the area of study.

The results of the analyses show that for part of the considered scheme the acceleration response is characterized by peak values higher than the critical acceleration of the slopes involved in the area. Then, the seismic response of the hill can be characterized by the occurrence of instabilities. Moreover, for some of the examined situations, the shear strain level obtained in the seismic response analyses attains values close to the volumetric threshold. Then, the seismic response of the soil involved in the hill may be affected by an increment of pore water pressure. Thus, since the actual stability conditions of the hill are globally
poor, seismic-induced effects, namely shear stresses and excess pore water pressures, might trigger instability phenomena and accumulation of permanent deformations. Based on the obtained acceleration response, the need of an estimation of earthquake-induced permanent displacements is highlighted. This assessment will enable evaluation of the post-seismic serviceability of the structures potentially involved in the instability phenomena.

References


