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Antonio Ferraro
University of Catania, Italy

Salvatore Grasso
University of Catania, Italy

Michele Maugeri
University of Catania, Italy

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TOPOGRAPHIC SITE EFFECTS EVALUATION FOR THE MONTE PO HILL IN THE CITY OF CATANIA (ITALY)

Antonio Ferraro
University of Catania
Viale A. Doria, 6
95125 Catania (Italy)

Salvatore Grasso
University of Catania
Viale A. Doria, 6
95125 Catania (Italy)

Michele Maugeri
University of Catania
Viale A. Doria, 6
95125 Catania (Italy)

ABSTRACT

The Monte Po Hill is located in the North-eastern part of the city of Catania; this area is prone to high seismic risk due to the presence of several constructions, including a school, in the vicinity of a toe of a slope, characterized by precarious stability conditions. The study has regarded the evaluation of site effects in correspondence of the site, to which corresponds a different value of the Seismic Geotechnical Hazard. In the beginning of 2007 a seismic station has been also located into the school building, with the aim of recording seismic events. Seismograms obtained by the seismic station have been also used to evaluate the ground response analysis at the surface. Finally the 1-D computer code EERA was also used to model the equivalent-linear earthquake site response analyses of layered soil deposits of the hill. The detail with which the hill has been studied has allowed the construction of a detailed 2-D model of its structure. It has been explored the differences between the computed ground motion for different Vs profiles using QUAD4M and QUAKE/W 2-D codes. It has been also possible to compare the results from different 1-D models reflecting current approaches to the determination of site response.

INTRODUCTION

Monte Po hill is located in the north-eastern part of the city of Catania. In the area a school and private constructions were built in the past in the nearness 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 condition. During the first construction times, instability phenomena occurred in the hill without determining an interruption in the works. In the same time damages in some of the buildings existing near the hill occurred and failure of several earth-retaining structures were observed in the area. Successively, damages and instability phenomena were observed as a consequence of meteoric events, during and after some excavations performed near the toe and, finally, after the December 13, 1990 Sicilian earthquake.

Due to the significant seismic geotechnical hazard related to this site an intensive research activity was performed in the framework of the research project “Detailed scenarios and actions for seismic prevention of damage in the urban area of Catania” (Maugeri 2000). The aim of the present research was the study of the site response of the Monte Po hill. The detail with which the hill has been studied has allowed the construction of a detailed 2-D model of its structure. In this paper it has been explored the differences between the computed ground motion for different Vs profiles. 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 occurred in the area was considered.

For site characterization of the soil, deep site investigations have been undertaken. Borings and dynamic in situ tests have been performed. Among them Down-Hole (D-H) and Seismic Dilatometer Marchetti Tests (SDMT) have been carried out, with the aim to evaluate the soil profile of shear waves velocity (Vs). The Seismic Dilatometer Marchetti Tests were performed up to a depth of 15 meters.

Moreover, data obtained during the research programme carried out by the geological office of the Catania municipality as a consequence of the instability phenomena occurred in the area were also analyzed.

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 analyzed in the attempt of evaluating the possible occurrence of an earthquake-triggered landslide focusing the attention on the effects of the earthquake-induced permanent displacements and on the post-seismic serviceability of the structures involved in the area.

SEISMICITY OF THE AREA

The study area is located in one of the most seismically active zone 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 magnitude 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 intensity equal to X MCS and XI MCS and estimated magnitudes equal to 7.0 - 7.4. According to Azzaro et al. (Azzaro et al. 1999) the first level scenario event for the Catania area may be reasonably assumed 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 to rupture with a normal pure mechanism along the escarpment (Postpischl 1985).

In particular at the beginning of 1991 during the construction of the school, a shallow instability phenomenon occurred causing a high-risk conditions for the work-yard of the school and for the existent private constructions. As a consequence of the occurred instability, the Catania municipality carried out an intense site investigation activity and monitored of the hill with piezometers and inclinometers.

BASIC SOIL PROPERTIES

The investigated Monte Po hill area, located in the North-West zone of the city, has plane dimensions of 40,000 m2 and a maximum depth of 20 m. The area pertaining to the investigation program and the locations of the boreholes and field tests are shown in Figure 1.

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 unit 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 sub-grade.

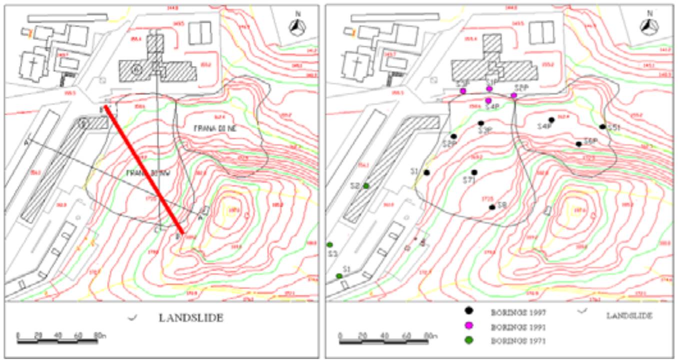


Fig. 1. Lay-out of Monte Po hill area with landslide localization and indication of the three cross sections considered, localization of borings location.

In the Catania Monte Po hill area, the clay fraction (CF) is predominantly in the range of 28 - 44 %. This percentage decreases to 17 % at the depth of 7 m where a sand fraction of 42 % is observed. The gravel fraction is always zero. The silt fraction is in the range of about 3 - 42 %.

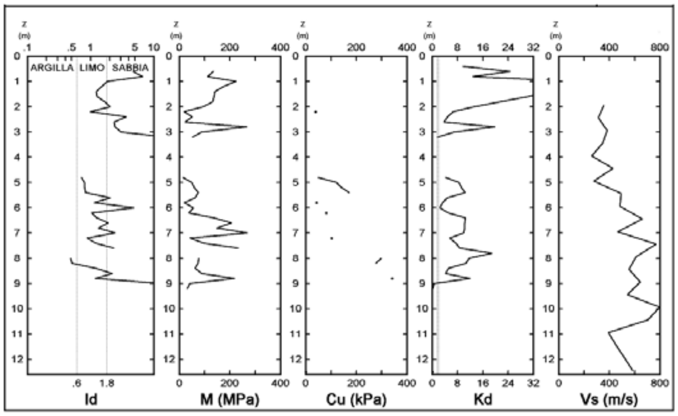
Typical range of physical characteristics, index properties and strength parameters of the deposit are reported in Table 1.

Table 1. Mechanical characteristics for Catania Monte Po hill area

Depth [m]	**0 - 5**	**5 – 10**	**10 – 15**	**15 – 20**
γ [kN/m^3]	18.9÷20.4	19.7÷20.1	20.6÷20.3	20.1÷20.7
γ_s [kN/m^3]	26.8÷26.9	20.1÷27.2	27.0÷27.0	2.71÷27.1
γ_d [kN/m^3]	15.8÷17.2	15.5÷16.2	16.3÷16.8	16.2÷17.6
γ_{sat} [kN/m^3]	20.4÷20.8	19.7÷20.2	20.2÷20.6	20.2÷21.1
w_n [%]	17.5÷ 25.6	21.5÷26.9	20.2÷23.0	17.1÷23.9
e	0.56÷0.67	0.65÷0.72	0.59÷0.66	0.62÷0.67
S_r [%]	76.3÷91.4	87.9÷99.2	91.0÷96.2	87.7÷97.1
w_L [%]	45.3÷51.2	44.3÷49.8	47.1÷54.2	37.7-51.5
w_P [%]	19.5÷27.8	18.5÷19.3	21.3÷22.6	16.5÷20.5
PI	17.5÷31.7	25.8÷30.5	25.8÷31.6	21.2÷32.1
IC	1.03÷1.20	0.69÷0.86	1.02÷1.04	0.86÷0.97
c_u [kPa]	17.4÷19.6		46.5÷63.6	14.4÷20.1
c' [kPa]	12.4÷18.6		24.0÷35.1	10.6÷12.0
φ' [°]	21.8÷22.8		21.6÷23.7	21.3÷24.7

SHEAR MODULUS

The small strain ($\gamma \leq 0.001$ %) shear modulus, G_0, was determined from SDMT and Down Hole (D-H) tests. The Seismic Dilatometer Marchetti (SDMT) is an instrument resulting from the combination of the DMT blade with a modulus measuring the shear wave velocity. The seismic modulus is an instrumented tube, located above the blade (see Figure 2), housing two receivers at a distance of 0.50 m. The test configuration "two receivers"/"true interval" avoids the problem connected with the possible inaccurate determination of the "first arrival" time sometimes met with the "pseudo interval" configuration (just one receiver). Also the pair of seismograms at the two receivers corresponds to the same blow, rather than at two successive blows - not necessarily identical. The adoption of the "true interval" configuration considerably enhances the repeatability in the Vs measurement. The SDMT provides a simple means for determining the initial elastic stiffness at very small strains and in situ shear strength parameters at high strains in natural soil deposits. Source waves are generated by striking a horizontal plank at the surface that is oriented parallel to the axis of a geophone connected to a co-axial cable with an oscilloscope (Martin and Mayne 1997), (Martin and Mayne 1998). The measured arrival times at successive depths provide pseudo interval V_s profiles for horizontally polarized vertically propagating shear waves (Figure 2). V_s may be converted into the initial shear modulus G_0. The combined knowledge of G_0 and of the one dimensional modulus M (from DMT) may be helpful in the construction of the G- γ modulus degradation curves (Cavallaro et al 2006). The V_s determinations are executed at 0.50 m depth intervals. A summary of SDMT parameters are shown in Figure 2 where: I_d: Material Index; gives information on soil type (sand, silt, clay); M: Vertical Drained Constrained Modulus; C_u: Undrained Shear Strength; K_d: Horizontal Stress Index; V_s: Shear Waves Velocity.



(a) (b) (c) (d) (e)

Fig. 2. Summary of SDMT's in Monte Po hill area. a) I_d: Material Index; b) M: Vertical Drained Constrained Modulus; c) C_u: Undrained Shear Strength; d) K_d: Horizontal Stress Index; e) V_s: Shear Waves Velocity.

The profile of K_d is similar in shape to the profile of the overconsolidation ratio OCR. $K_d = 2$ indicates in clays OCR = 1, $K_d > 2$ indicates overconsolidation. A first glance at the Kd profile is helpful to "understand" the deposit. Figure 3 shows the values of V_s obtained in situ from a D-H test and SDMT. In the superficial strata V_s by SDMT is about 350 m/s. The V_s values increased with depth. At the depths of 6 and 10 m V_s is about 700 m/s in correspondence of sandy strata. These high V_s values by SDMT show the effect of soil disturbance during the test. The V_s values, experimentally determined during D-H tests, show an important variation in the transition zone at depths of 8 and 19 m, where thin layers of sandy soil exist.

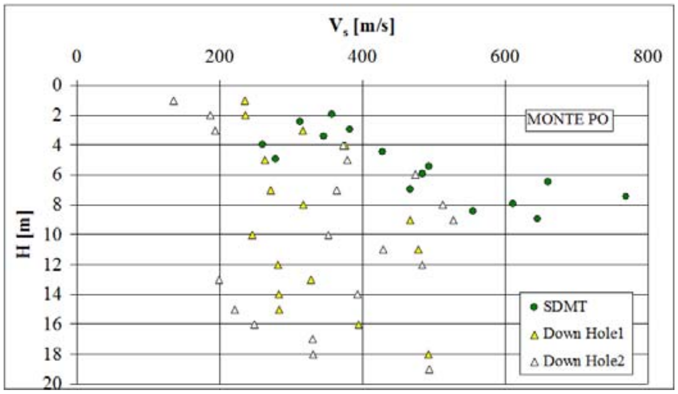


Fig. 3. V_s from different in situ tests.

1-D LOCAL SITE RESPONSE ANALYSIS USING DIFFERENT V_S PROFILES

The Monte Po hill in the city of Catania, located in the South-Eastern Sicily (Italy), has been affected by several destroying earthquakes of about magnitude 7.0+ in past times. It is so reasonable to assume in Catania a maximum expected earthquake as a repetition of the January 11, 1693 event, with intensity XI MCS and estimated magnitude M = 7.3. Synthetic seismograms have been drawn for the site long a set of receivers placed at different depths, starting from the surface up to almost 40 m. After evaluating the synthetic accelerograms at the bedrock, the ground response analysis at the surface, in terms of time history and response spectra, has been obtained by some 1-D non-linear codes. The Sicilian earthquake of December 13, 1990 brought to an end a period of seismic dormancy lasted a long time thus reaching a local magnitude ML = 5.4 with a focus depth of about 6-12 km. Even if it was internationally recognized as a "moderate" earthquake it provoked serious damages to many buildings.

By the beginning of 1991, during the building up of a public school situated at the bottom of the Monte Po hill, signs of gravitational motions were noticed, which proved to be dangerous both for the building in progress and for some council buildings some decades old. It is well-known that the slope is linked to rain phenomena of a certain intensity, which often cause a relevant rise in its piezometric surface, thus risking to jeopardize the stability of the whole slope. The heavy precipitations occurred in the days before the sliding, with the concurrence of the 1990 earthquake represented the cause of the hydrogeological breakdown of the Monte Po hill, which had lost its passive defenses that had previously and so far granted a good surface drainage; moreover, during the same period, cuttings and excavations for the marking of the external perimeter of the school were performed. The flowing and gliding landslide which came out and affected the north-western side of the hill at about 190 mt. upon sea level, shook huge masses of ground with a roto-traslational kinematic mechanism, causing in some cases a significant damage to buildings situated in the valley. Another detachment, occurred later in the north-eastern side, affected by collapses of the sub-vertical faces of the surfacing conglomerates, but that motion did not jeopardize any houses.

In the beginning of 2007 a seismic station has been also located into the school building, with the aim of recording seismic events. Seismograms obtained by the seismic station have been also used to evaluate the ground response analysis at the surface.

Local site response analyses have been brought for the Monte Po Hill by 1-D linear equivalent computer codes. The codes implement a one-dimensional simplified, hysteretic model for the non-linear soil response. The Seismic Dilatometer Marchetti Test (SDMT) was performed up to a depth of 15 meters. The results show a very detailed and stable shear waves profile. The S-wave propagation obtained by D-H and SDMT occurs on a 1-D column having shear behavior. The column is subdivided in several, horizontal, homogeneous and isotropic layers characterized by a non-linear spring stiffness G(γ), a dashpot damping D(γ) and a soil mass density ρ. Moreover, to take into account the soil non-linearity, laws of shear modulus and damping ratio against strain have been inserted in the code (Frenna and Maugeri 1995). The 1-D columns have a height of 20 m and of 40 m and are excited at the base by accelerograms obtained from the seismograms and from the recordings of local earthquakes. The analysis provides the time-history response in terms of displacements, velocity and acceleration at the surface. Using this time history, response spectra concerning the investigated site have been deduced. The soil response at the surface was also modeled using the Equivalent–linear Earthquake site Response Analyses of Layered Soil Deposits computer code EERA (Bardet et al 2000) for calculus of amplitude ratios and spectral acceleration. An evaluation on buildings most directly affected or menaced by the landslide (see Figure 1) has been carried out, in order to focus on possible existing damage. As the NE landslide section did not affect any buildings, the attention has been directed on NW landslide section, which seriously jeopardized both a public school situated to the north of the landslide (building A), and some IACP buildings to the west of it (building B).

In Figure 4 is reported cross section B-B of the Monte Po hill (red line in Figure 1), plotted in the direction of maximum gradient just next to the point where the ground overlaps a restraint wall, with localized borehole points S2 (near the school), S7 and S8 (on the hill) along which local site response has been calculated. Figure 5 shows maximum shear stresses and maximum accelerations obtained using 1-D code EERA, considering as input the Sortino recording (E-W component) of the 1990 earthquake.

Figure 6 shows maximum shear stresses and maximum accelerations obtained using 1-D code EERA, considering as input the synthetic accelerograms of the 1693 earthquake. Figure 7 shows amplitude ratios obtained using 1-D code EERA, while Figg. 8 and 9 show response spectra obtained using 1-D codes EERA and GEODIN.

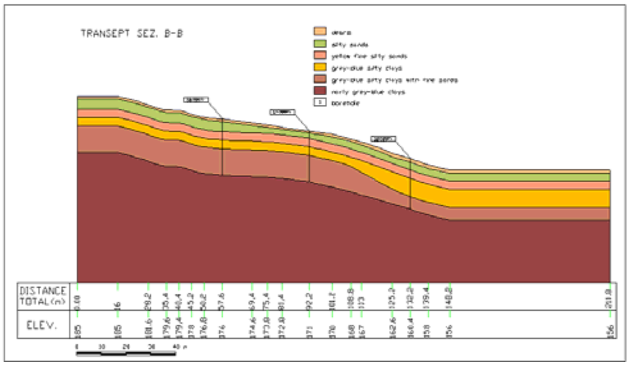


Fig. 4. Section B-B of the Monte Po hill with localization of the borehole points S2, S7, S8.

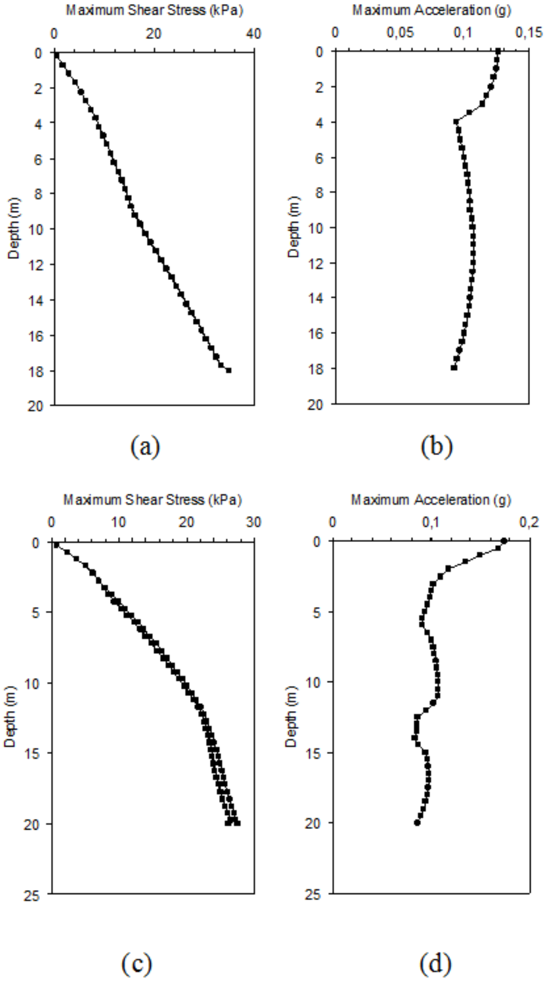


Fig. 5. Maximum shear stress (kPa): (a) S2 point; (c) S7 point. Maximum accelerations (g): (b) S2 point; (d) S7 point, considering as input the December 13, 1990 earthquake.

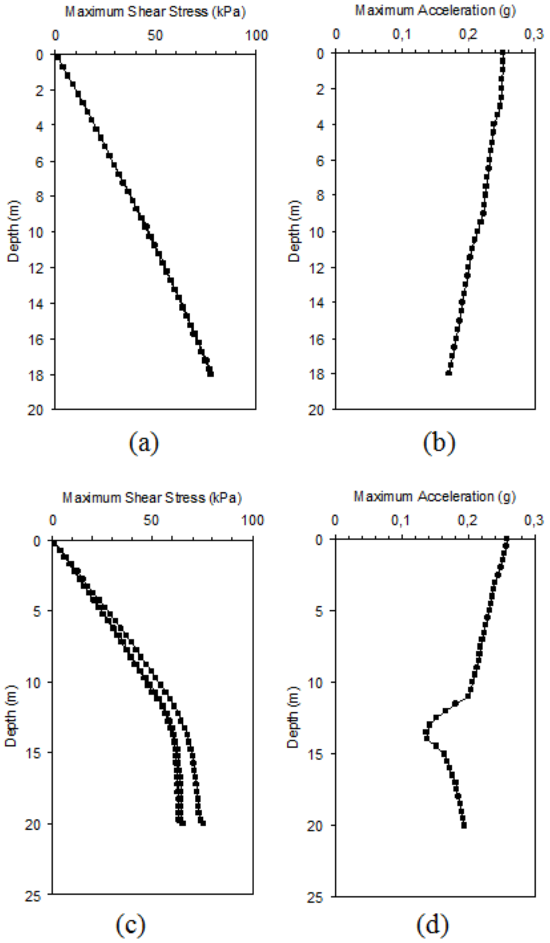


Fig. 6. Maximum shear stress (kPa): (a) S2 point; (c) S7 point. Maximum accelerations (g): (b) S2 point; (d) S7 point, considering as input the January 11, 1693 earthquake.

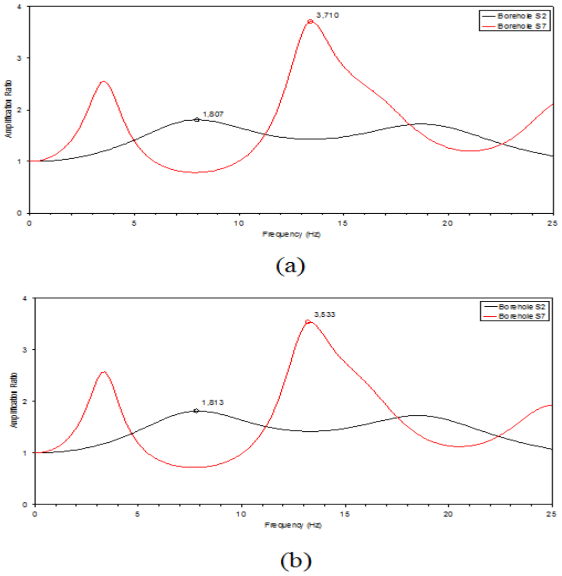


Fig. 7. Amplitude ratios: (a): 1990 earthquake input; (b): 1693 earthquake input.

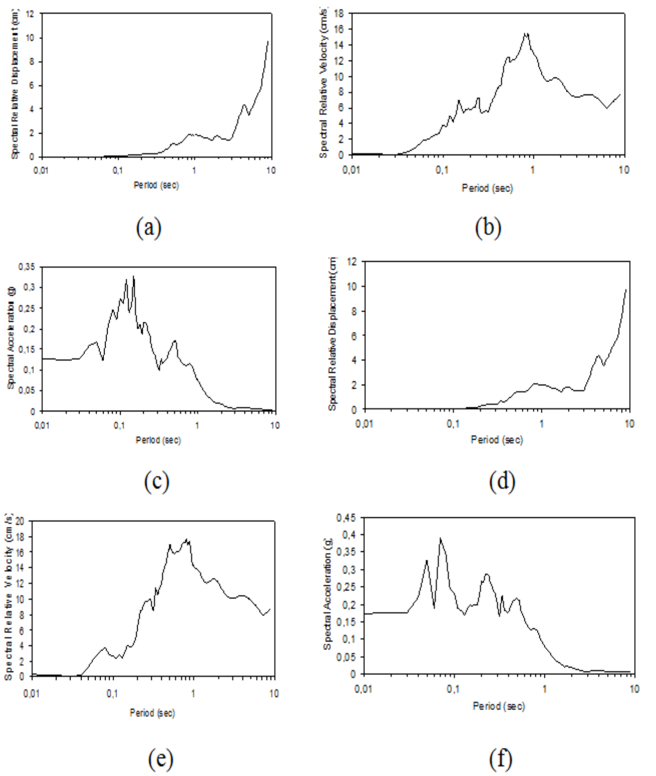


Fig. 8. Response spectra using EERA code of relative displacement, relative velocity and acceleration, considering 1990 earthquake input: (a), (b), (c) S2 point; (d), (e), (f) S7 point.

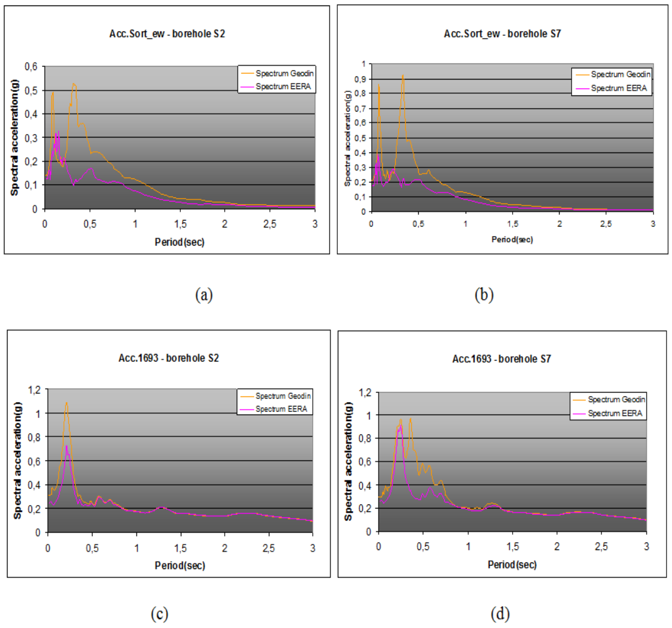


Fig. 9. Comparisons between response spectra obtained by 1-D codes EERA and GEODIN. a) 1990 earthquake input S2 point; b) 1990 earthquake input S7 point; c) 1693 earthquake input S2 point; a) 1693 earthquake input S7 point.

2-D SITE RESPONSE ANALYSES USING QUAD4M AND QUAKE/W CODES

A detailed 2-D model has been constructed and validated for the Monte Po hill. The advantage of this model is to investigate the parameters that, in addition to surface soil conditions, can be used to correctly characterize site response in a 2-D structure. Through analyses using 2-D numerical simulations for SH waves, differences between the computed ground motion along some points of the Monte Po hill structure have been evaluated. It was also possible to compare the results from different 1-D models used reflecting current approaches to the determination of site response.

Figure 10 shows the cross section B-B modeled into the 2-D FEM code QUAD4M (Hudson et al 1994), modified from QUAD4 (Idriss et al. 1973). QUAD4M is a dynamic, time-domain, equivalent linear two dimensional computer program, as a modification to QUAD4 to implement a transmitting base and an improved time stepping algorithm. Figure 10 shows also the 2m x 2m mesh (1185 finite elements) of the cross section B-B adopted for calculations and the green points along which seismic response has been monitored. Figure 11 shows the cross section B-B modeled into the the 2-D FEM code QUAKE/W (Geo-Slope International 2008) and also shows the mesh (3116 finite elements) adopted for calculations.

Figure 12 shows the results obtained using 2-D code QUAD4M in correspondence of point S2. Results compare well to EERA 1-D code. Figure 13 shows the results obtained using 2-D code QUAD4M in correspondence of point S7. Results compare well to EERA 1-D code.

Figure 14 shows the results obtained using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W, in correspondence of S2 point.

Figure 15 shows the results obtained using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W, in correspondence of S7 point.

It is possible to observe that in correspondence of point S2 (near to the slope toe) the stratigraphic amplification factor is more relevant than the topographic amplification factor. Moreover, in correspondence of point S7 (near to the slope crest) the topographic amplification factor is more relevant than the stratigraphic one.

Figure 16 shows the elastic response spectra obtained in correspondence of point S2, using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W. Figure 17 shows the elastic response spectra obtained in correspondence of point S7, using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W.

Figure 18 shows the Topographic Aggravation Factors (TAF) obtained following the approach proposed by Kallou et al.

(2001) for the 1693 and 1990 input earthquakes, using 2-D code QUAD4M. Figure 19 shows the Topographic Aggravation Factors (TAF) obtained following the approach proposed by Bouckovalas and Kouretzis (2001) for the 1693 and 1990 input earthquakes, using 2-D code QUAD4M.

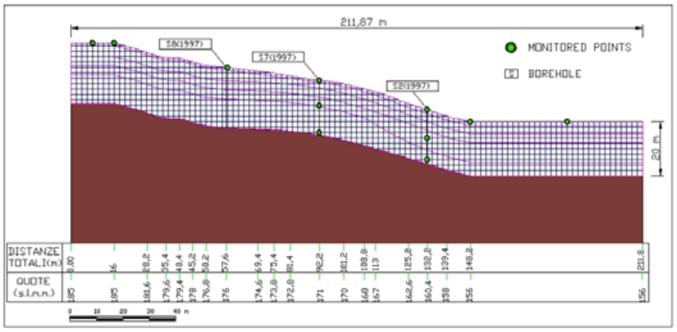


Fig. 10. Mesh (2m x 2m) of the cross section B-B modeled into QUAD4M code.

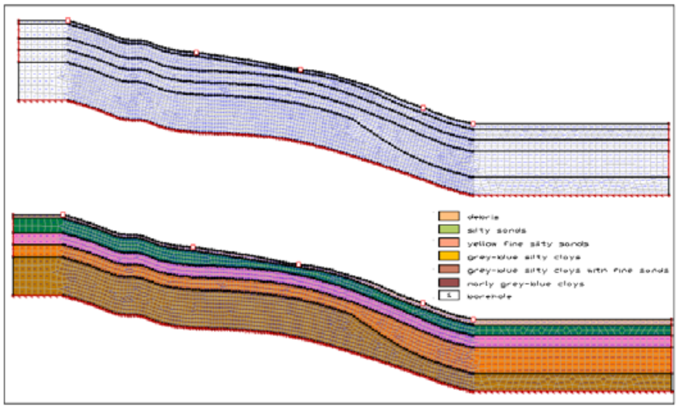
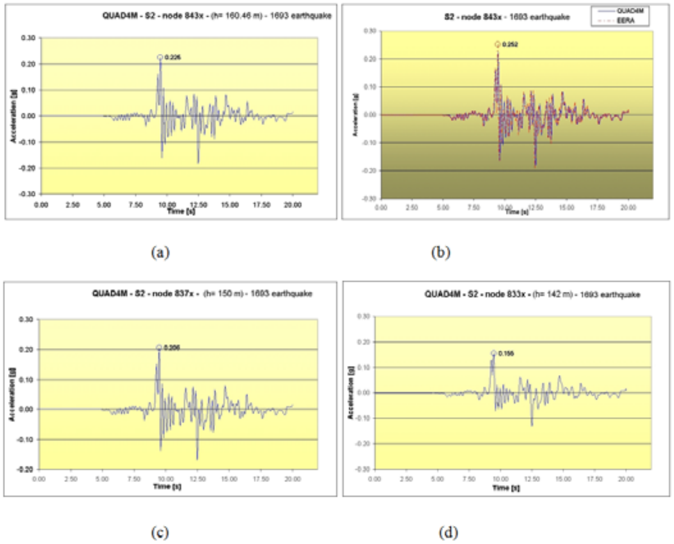


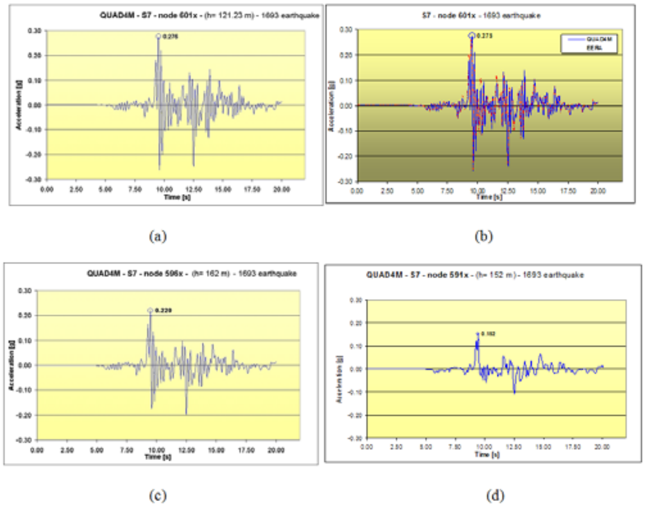
Fig. 11. Mesh (2m x 2m) of the cross section B-B modeled into QUAKE/W code.



(a) (b)

(c) (d)

Fig. 12. Seismic response analysis of the Monte Po hill. Borehole S2: (a) surface acceleration with QUAD4M; (b) comparison between QUAD4M and EERA surface accelerations; (c) acceleration at a depth of 10.5 m; (d) acceleration at a depth of 18.5 m.



(a) (b)

(c) (d)

Fig. 13. Seismic response analysis of the Monte Po hill. Borehole S7: (a) surface acceleration with QUAD4M; (b) comparison between QUAD4M and EERA surface accelerations; (c) acceleration at a depth of 9.5 m; (d) acceleration at a depth of 19.5 m.

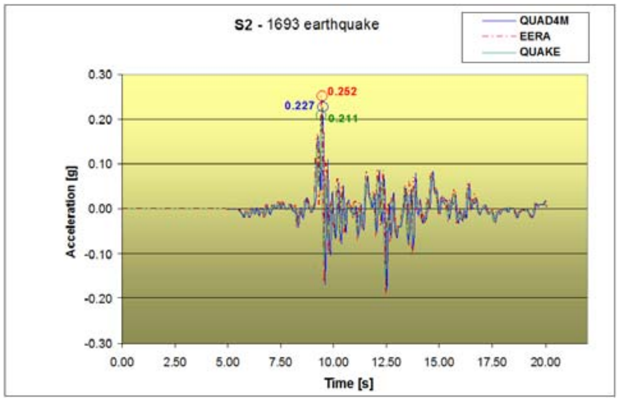


Fig. 14. Results obtained using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W, in correspondence of S2 point.

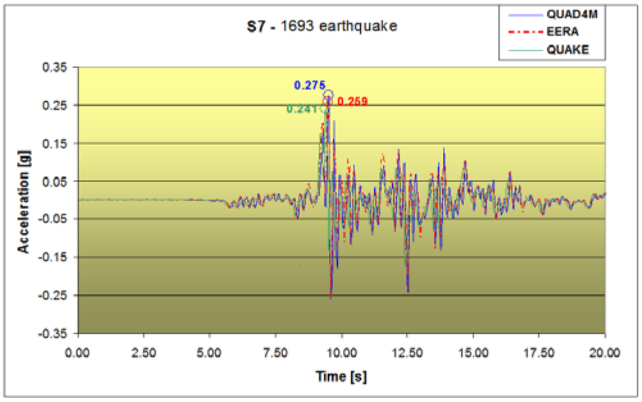


Fig. 15. Results obtained using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W, in correspondence of S7 point.

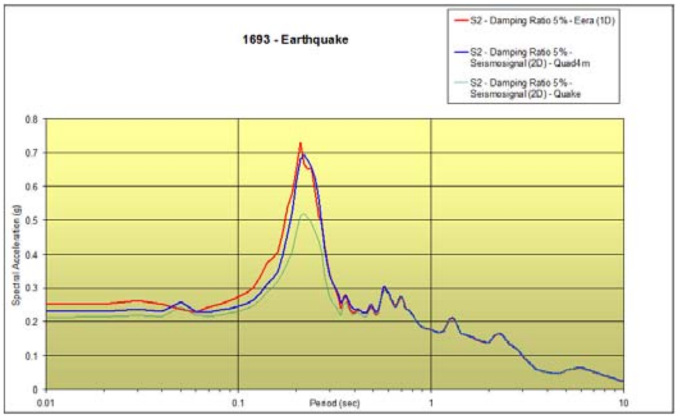


Fig. 16. Response spectra obtained using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W, in correspondence of S2 point.

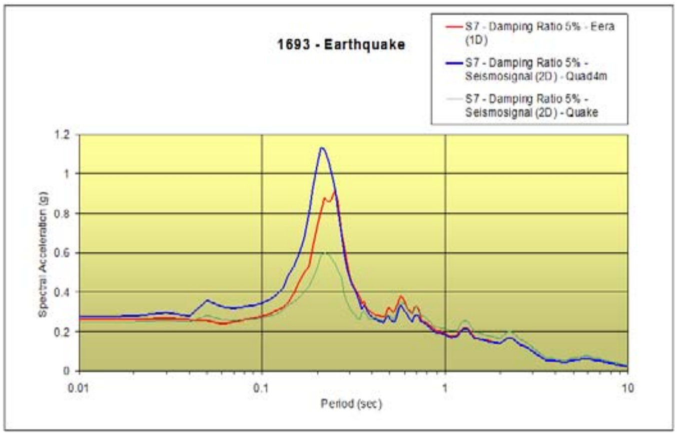


Fig. 17. Response spectra obtained using 1-D code EERA and the 2-D codes QUAD4M and QUAKE/W, in correspondence of S7 point.

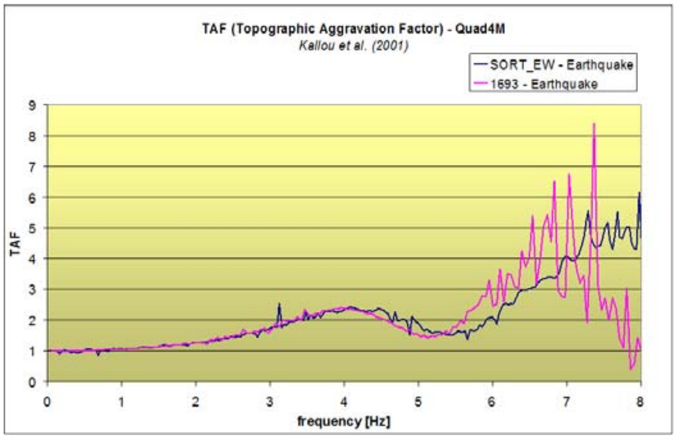


Fig. 18. Topographic Aggravation Factors (TAF) obtained following the approach proposed by Kallou et al. (2001) for the 1693 and 1990 input earthquakes, using 2-D code QUAD4M.

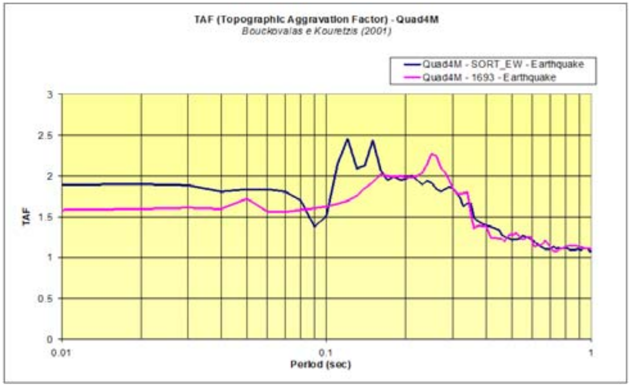


Fig. 19. Topographic Aggravation Factors (TAF) obtained following the approach proposed by Bouckovalas and Kouretzis (2001) for the 1693 and 1990 input earthquakes, using 2-D code QUAD4M.

CONCLUSIONS

It has been presented a study of the site response of the Monte Po hill, in the city of Catania. The detail with which the hill has been studied has allowed the construction of a detailed 2-D model of its structure. In the paper it has been explored the difference between the computed ground motion for different Vs profiles. It has been also possible to compare the results between 1-D model and 2-D models reflecting current approaches to the determination of site response. It is possible to observe that in correspondence of point S2 (near to the slope toe) the stratigraphic amplification factor is more relevant than the topographic amplification factor. Moreover, in correspondence of point S7 (near to the slope crest) the topographic amplification factor is more relevant than the stratigraphic one. The evaluation of site response analysis using 1-D model gives a stratigraphic amplification factor of about S_s= 1.5. The evaluation of site response analysis using 2-D models suggests a topographic amplification factor of about ST= 1.2 and an apparent amplification (1-D) + (2-D) of about A_a= 1.7.

The seismic response analysis performed with 2-D models shows how for a correct evaluation of the topographic amplification factor it is necessary to work in terms of maximum accelerations and in terms of topographic aggravation factor (TAF).

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