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Site Response Analysis in the STM-M6 Industrial Area of the City of Catania (Italy)

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ABSTRACT

The paper presents the case history of the geotechnical characterization of a seismic site for the re-use of an industrial building for producing solar panels in the industrial area of Catania (Sicily, Italy). To determine the geological profile and the geotechnical characteristics of the soil, the site was well investigated by means of in situ and laboratory tests. The following in situ geotechnical tests were carried out: Borings, SPT, CPT, PLT and dynamic in situ tests. Among them Down-Hole (D-H), Cross-Hole (C-H), SASW and recently Seismic Dilatometer Marchetti Tests (SDMT) have been carried out, with the aim to evaluate the soil profile of shear waves velocity (V_s). Moreover the following laboratory tests were carried out on undisturbed samples retrieved with a 86 mm diameter Shelby sampler: Oedometer tests, Direct shear tests, Resonant Column and Torsional shear tests. Static and dynamic parameters obtained by in situ and laboratory tests were reported and analyzed. Moreover the Sicilian earthquake of December 13, 1990 (M_L= 5.4) heavy damaged the site, also due to soil amplification. Using the recordings of this earthquake, to evaluate the input motion at the conventional bedrock, the ground response analysis has been obtained by the 1-D non-linear code EERA at the industrial building site. In particular the study has regarded the evaluation of site effects in terms of acceleration time history at the surface, soil amplification factors, as well as in terms of time history and response spectra.

INTRODUCTION

The ability of laboratory tests to provide accurate measurements of dynamic soil properties is affected by several factors, such as sample disturbance, specimen size, equipment compliance, loading conditions, and reproduction of actual field conditions such as stress, chemical, thermal, and structural conditions.
The impact of many of these factors, particularly on nonlinear dynamic soil properties, remains to be quantified. Thus there is a significant need to perform dynamic soil property measurements by in situ tests and to compare the field values to those measured in the laboratory. In situ dynamic property measurements would eliminate many of the problems associated with laboratory testing and would allow the accuracy of laboratory methods to be evaluated (Kurtulus and Stokoe, 2008).
The geotechnical earthquake engineering problems requires the evaluation of the dynamic soil properties. The mechanical properties associated with dynamic loading are shear wave velocity (V_s), shear modulus (G), damping ratio (D), and Poisson's ratio (ν). To determine soil dynamic properties, the current state of practice involves: estimating or measuring shear waves velocity V_S in the field, using geophysical methods and estimating or measuring the variation in laboratory of shear modulus G and damping ratio D as a function of shear strain γ.
The geotechnical characterization of Catania area by in situ and laboratory tests has a great importance because the east coast area of Sicily is considered as one of the zones of Italy with greater high seismic risk, basing on the past and current seismic history and on the typology of civil buildings and industrial activities (Cavallaro et al., 2008; Castelli et al., 2012). The knowledge of soil dynamic properties gives the possibility to preview the soil behavior during the seismic events.
On September 2000 in the industrial area of Catania (Sicily, Italy) started the construction of a reinforced concrete building (Figure 1), founded on normally consolidated clayey deposits.
These soft soil deposits have a low bearing capacity and exhibit large settlements when subjected to loading. It is therefore inevitable to treat soft soil deposits prior to construction activities, in order to prevent differential settlements and subsequently potential damages to structures.
To evaluate the compressibility of the soil foundation, a preloading was applied by means of the construction of an

instrumented circular test embankment with a diameter of 65 m and 2.50 m high.

Thirty-three vertical prefabricated drains were disposed beneath the embankment. The construction of the embankment started on September 5, 2000 and it was realized in different layers. Several types of geotechnical instrumentation were installed before the construction of the embankment to monitor its performance.

Vertical drains and monitoring instruments (shallow and deep piezometers, inclinometers) were used to monitor the soil foundation beneath the embankment. The evaluation of the consolidation settlements below the embankment were evaluated by Castelli et al. (2008).

The industrial building was re-used in the 2012 for producing solar panels. Because of that, a new site investigation by in situ and laboratory tests has been performed. The results obtained are reported in the next paragraph.

Fig. 1. Industrial building M6.

GEOTECHNICAL SOIL CHARACTERIZATION

To determine the geological profile and the geotechnical characteristics of the soil, in-situ tests such as boreholes, cone penetration tests and down-hole seismic tests have been carried out. The obtained profile shows that the underground soil is constituted mainly by clayey-silt and silty-clay up to a depth of 35-40 meters from ground surface. The water table, determined by piezometers, is located at around 1.5 meters below the ground surface.

Among the in-situ tests the down-hole seismic tests D-H have been performed up to a depth of 36 meters, the cone penetration dissipation tests CPTU (4 tests) have been performed up to a depth of 60 meters, with the pore pressure measurement for the dissipation tests. In particular, starting from the ground level, the following layer have been found:

A superficial thin layer from ground level to 6 m of clayey sandy silt, a thick layer (to 6 - 12 m depth) of alternance of silty clay and clayey silt, a layer (to 12 - 23 m depth) of dark gray silty clay, a layer (to 23 - 27 m depth) of brown silty fine sand, a layer (to 27 – 28.5 m depth) of clayey silt and sity clay, a layer (to 28.5 – 30.5 m depth) of sandy clayey silt, a layer (to 30.5 – 32.5 m depth) of sandy gravel, a layer (to 3.25 - 34 m depth) of brown-gray silty sand, a layer (to 34 – 35.5 m depth) of fine gravel, a layer (to 35.5 - 40 m depth) of alternance of blue gray clay and brown sandy silt, a layer (to 40 - 45 m depth) of blue gray silty clay.

From the soil profile can be highlight the soil layer have a same nature in all the boreholes but the thickness of each layer can be significantly different from one boreholes to another.

The index properties and the mechanical characteristics of the soil have been evaluated from laboratory tests carried on undisturbed soil samples, with the aim to compare the values of the geotechnical parameters determined by laboratory tests with those derived from in situ tests.

Due to the seismicity and to the geotechnical properties of the area, the soil deformability have been investigated both in static conditions by oedometer tests and in dynamic conditions by resonant column tests.

The index properties and the mechanical characteristics of the soil foundation derived from the laboratory tests are shown, as function of depth, in Figure 2. The index tests classified the soil as clayey-silt and silty-clay with the following average parameters: liquidity limit w_L varies from 43 up 84 %, plasticity limit w_P is about 25 - 46 %, consistence index IC varies from 0.42 up 1.90. The values of the natural moisture content w_n prevalently range between 32 and 72 % as depth increasing, while the soil unit weight is equal around to 17 kN/m^3.

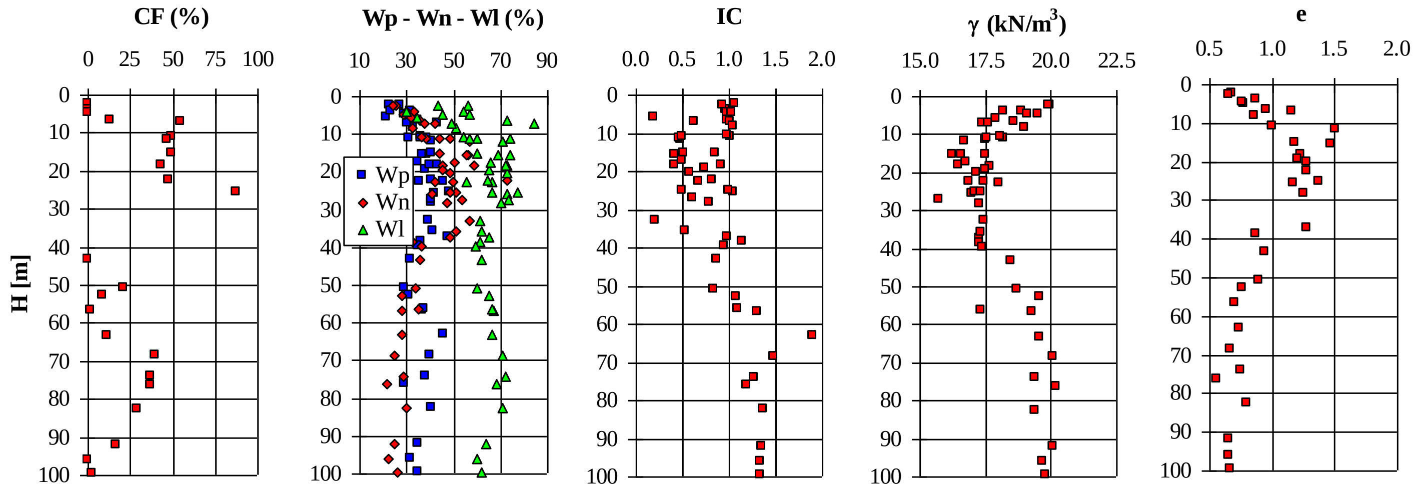


Fig. 2. Typical range of index properties of the clayey soils at the site of Industrial area of Catania STM-M6 (Italy).

Figure 3 shows the plasticity properties of numerous soil samples taken in the clayey-silt and silty-clay formation in the industrial area of Catania. The soil deposits can be classified as MH-OH inorganic silts and organic clays of low plasticity.

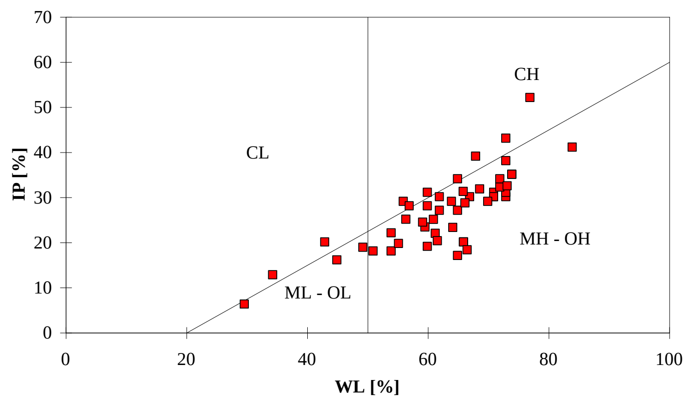


Fig. 3. Plasticity chart.

The variability of the grain size distribution is somehow confirmed by the material index, I_D, evaluated by flat dilatometer tests (SDMT) carried out in the same formation.

At the M6 industrial building site, the clay fraction (CF) is predominantly in the range of 2 - 54 %. This percentage decreases to 0 - 2 % at the depth of 95 m where a sand fraction of 4 - 9 % is observed. The gravel fraction is always zero. The silt fraction is in the range of about 50 - 100 %.

The good degree of homogeneity of the deposit is confirmed by comparing the cone penetration resistance q_c from mechanical cone penetration tests (CPT) performed at different locations over the investigated area. The variation of q_c with depth clearly shows the very poor mechanical characteristics of soil. Typical values of q_c are in the range of 0.01 to 0.49 MPa. The soil deposits can be classified as inorganic silt of high compressibility and organic clay.

The preconsolidation pressure σ'_p and the overconsolidation ratio OCR = σ'_p/σ'_{vo} were evaluated from the 24 hours compression curves of 5 incremental loading (IL) oedometer tests. Moreover, a SDMT was used to assess OCR and the coefficient of earth pressure at rest K_o following the procedure suggested by Marchetti (1980).

The OCR values obtained from SDMT range from 1 to 10 (K_o = 0.5 to 1) with an average value equal to 1.2 up to about 10 for the 40 m deep sounding. The OCR values inferred from oedometer tests are lower than those obtained from in situ tests.

One possible explanation of these differences could be that lower values of the preconsolidation pressure σ'_p are obtained in the laboratory because of sample disturbance (Cavallaro et al, 2008).

Due to the peculiarity of the geotechnical problem, the vertical consolidation was studied by the oedometer tests and the characteristic values, reported in Table 1, were determined. In the interval of interest the oedometer modulus E_{ed} is ranging between about 1413 up to 6317 kPa as depth increasing, the consolidation coefficient C_v is ranging between $1.28 \cdot 10^{-6}$ and $9.87 \cdot 10^{-9}$ m²/sec. These values are in good agreement with the values evaluated with dissipation tests.

Table 1. Soil compressibility properties by laboratory tests.

Tests	Depth [m]	C_v [m²/s]	E_{ed} [kPa]
S7R1	2.45 – 2.65	$4.86 \cdot 10^{-7}$	1413.00
S1C1	5.50 – 6.00	$8.52 \cdot 10^{-7}$	6317.64
S7R2	8.10 – 8.30	$7.49 \cdot 10^{-8}$	1815.80
S2C2	10.90 – 11.40	$1.28 \cdot 10^{-6}$	5921.52
S7I3	15.00 – 15.50	$2.43 \cdot 10^{-8}$	2347.80
S3C2	15.30 – 15.80	$9.99 \cdot 10^{-7}$	3854.05
S7I5	25.00 – 25.50	$9.87 \cdot 10^{-9}$	3085.00
S7I6	28.00 – 28.50	$6.27 \cdot 10^{-9}$	3617.00
S7I7	37.00 – 37.50	$4.80 \cdot 10^{-9}$	3997.00

SHEAR MODULUS

The small strain ($\gamma \leq 0.001$ %) shear modulus, G_o, was determined from SDMT and a Down Hole (DH) test. The equivalent shear modulus (G_{eq}) was determined in the laboratory by means of a Resonant Column test (RCT) performed on Shelby tube specimens by means of a Resonant Column. Moreover it was attempted to assess G_o by means of empirical correlations, based either on penetration test results or on laboratory test results (Jamiolkowski et al. 1995).

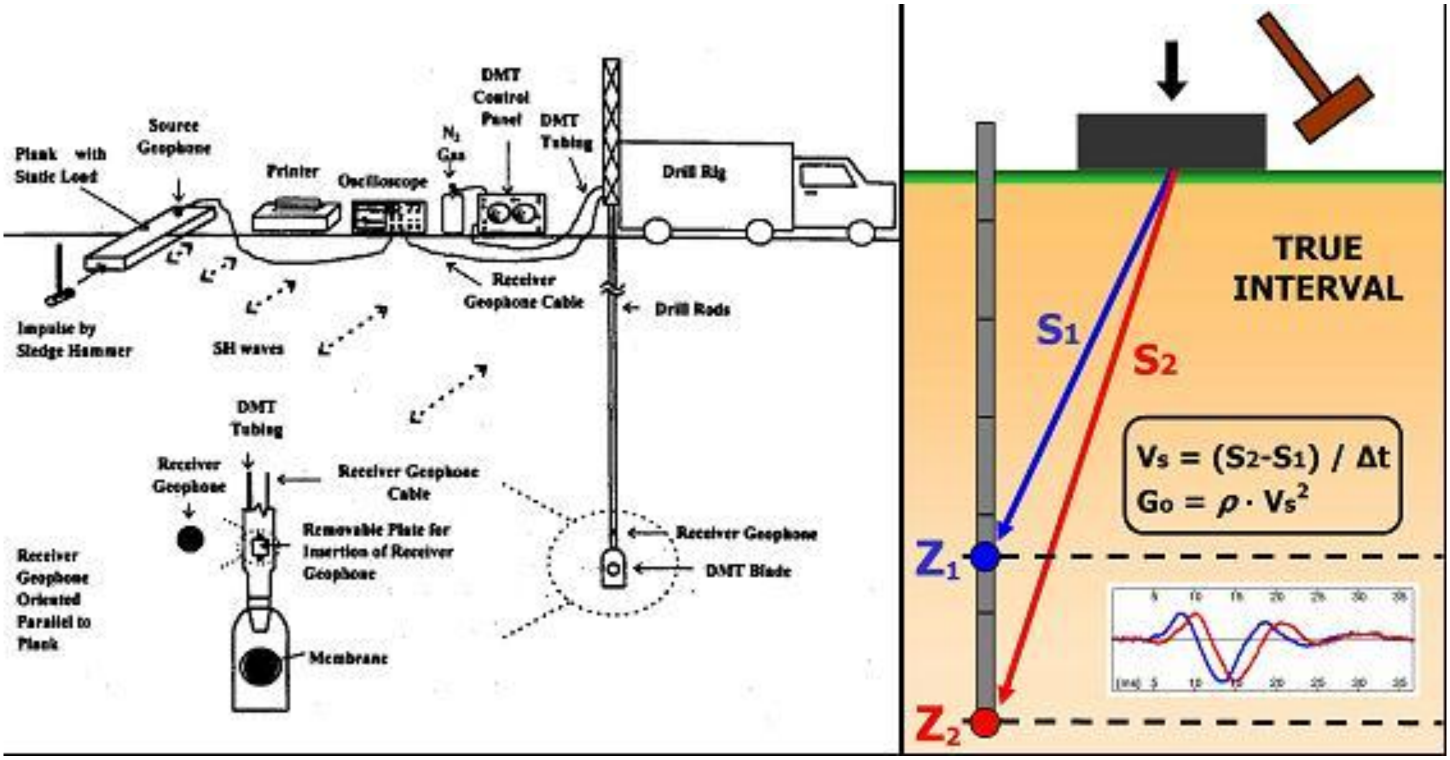


Fig. 4. The Seismic Dilatometer Marchetti Test (SDMT), resulting from the combination of the DMT blade with a modulus measuring the shear wave velocity V_S.

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 (Marchetti, 2008).
Source waves are generated by striking a horizontal plank at the surface that is oriented parallel to the axis of a geophone connects by a co-axial cable with an oscilloscope (Martin & Mayne, 1997, 1998). The measured arrival times at successive depths provide pseudo interval V_s profiles for horizontally polarized vertically propagating shear waves (Figure 4).
The small strain shear modulus G_o is determined by the theory of elasticity by the well known relationships:

$$G_o = \rho V_s^2 \qquad (1)$$

where: ρ = mass density.

A summary of SDMT parameters are shown in Figure 5 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; 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, KD > 2 indicates overconsolidation. A first glance at the K_d profile is helpful to "understand" the deposit;
- V_s: Shear Waves Velocity.

The Poisson ratio variation with depth, obtained from a Down Hole (D-H) test oscillates around 0.49.
Figure 6 shows the values of G_o obtained in situ from a D-H test and SDMT and those measured in the laboratory from RCT performed on undisturbed solid cylindrical specimens which were isotropically reconsolidated to the best estimate of the in situ mean effective stress. The G_o values are plotted in Figure 6 against depth (Carrubba and Maugeri 1988). In the case of laboratory tests, the G_o values are determined at shear strain levels of less than 0.001 %.
Quite a good agreement exists between the laboratory and in situ test results. On average the ratio of G_o (Lab) to G_o (Field) by SDMT and DH was equal to about 0.90 at the depth of 29.5 m.

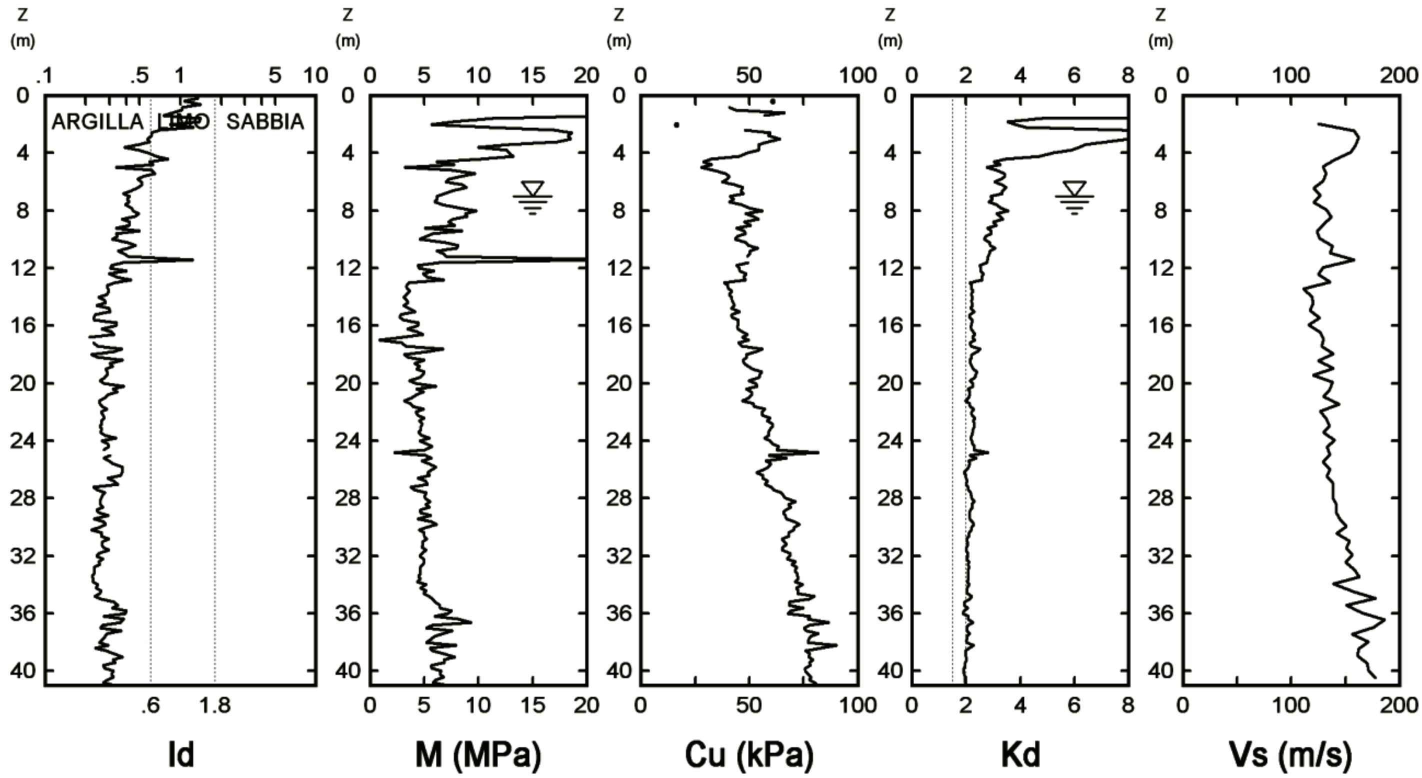


Fig. 5. Summary of SDMTs in STM M6 area.

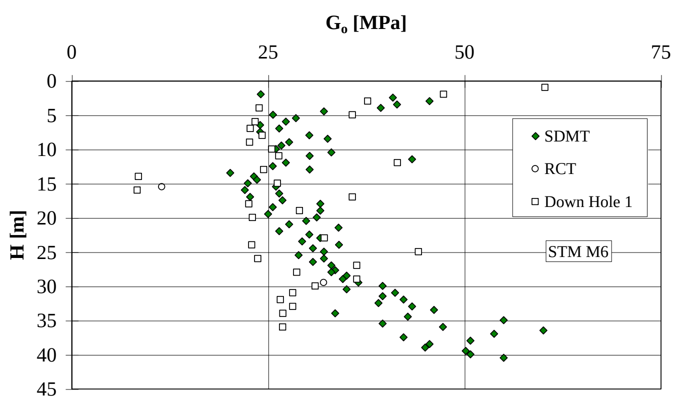


Fig. 6. G_o from laboratory and in situ tests.

In the superficial strata G_o by SDMT assumed the value of 45 MPa. In the medium Holocene strata G_o values are between 20 and 35 MPa. In the lower Holocene soil G_o increases with depth to 55 MPa.

The experimental results of specimens obtained by RCT were used to determine the empirical parameters of the equation proposed by Yokota et al. (1981) (Figure 7) to describe the shear modulus decay with shear strain level:

$$\frac{G(\gamma)}{G_o} = \frac{1}{1+\alpha\gamma(\%)^{\beta}} \tag{2}$$

in which:
$G(\gamma)$ = strain dependent shear modulus;
γ = shear strain;
α, β = soil constants.

The expression (2) allows the complete shear modulus degradation to be considered with strain level (Maugeri 1995). The values of α = 7.15 and β = 1.223 were obtained for STM M6 clay by Carrubba and Maugeri (1988).

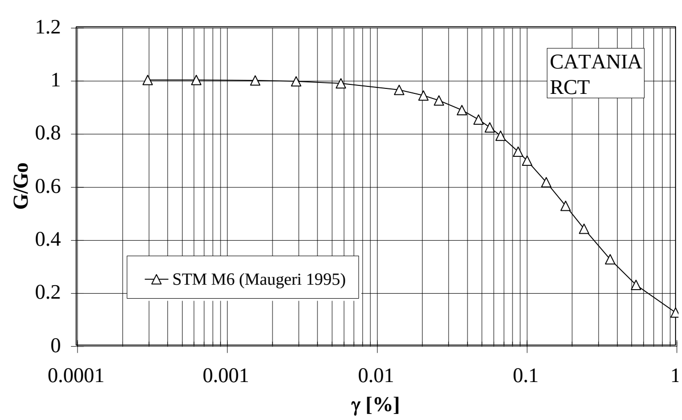


Fig. 7. G/G_o-γ curves from RCT tests.

As suggested by Yokota et al. (1981), the inverse variation of damping ratio with respect to the normalized shear modulus has an exponential form as that reported in Figure 8 for the central area of Catania (Maugeri 1995):

$$D(\gamma)(\%) = \eta \cdot \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \qquad (3)$$

in which:
$D(\gamma)$ = strain dependent damping ratio;
γ = shear strain;
η, λ = soil constants.
The values of η = 28.12 and λ = 2.50 were obtained for STM M6 clay by Carrubba and Maugeri (1988).
The equation (3) assume maximum value D_{max} = 28.12 % for $G(\gamma)/G_o$ = 0 and minimum value D_{min} = 2.30 % for $G(\gamma)/G_o$ = 1.
Therefore, eq. (3) can be re-written in the following normalised form:

$$\frac{D(\gamma)}{D(\gamma)_{max}} = \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \qquad (4)$$

These parameters were obtained from the damping values assessed by means of the steady-state method.
These equation were used to evaluate the empirical parameters for different test sites for the municipal area of Catania. The values are reported in Table 2 (Cavallaro et al. 2012).

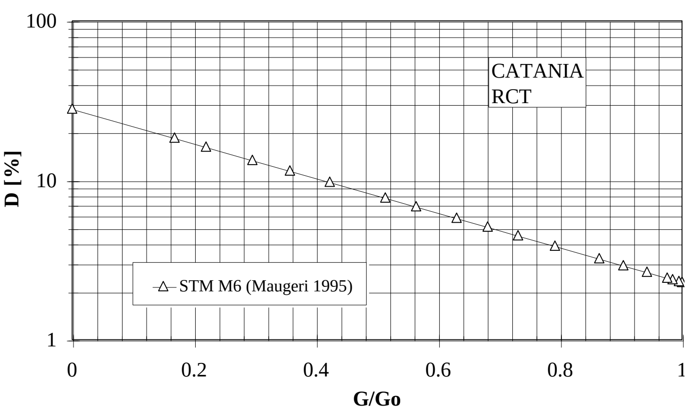


Fig. 8. D-G/G_o curves from RCT tests.

Table 2. Soil constants for the municipal area of Catania.

Site	α	β	η	λ
1. Piana di Catania (STM – M5) 12. Piana di Catania (STM – M6)	7.15	1.223	19.87	2.16
2. ENEL bo x	12.8	0.67	12.14	1.583
3. Plaja beach	9	0.815	80	4
13. San Giuseppe la Rena				
4. Tavoliere	-	-	-	-
5. Via Stellata 11. Villa Comunale	11	1.119	31	1.921
6. Piazza Palestro	6.9	1	23	2.21
7. San Nicola alla Rena Church 9. Via Monterosso	7.5	0.897	90	4.5
8. Via Dottor Consoli	16	1.2	33	2.4
10. Monte Po	-	-	-	-

EVALUATION OF V_S FROM EMPIRICAL CORRELATIONS

It was also attempted to evaluate the small strain shear modulus G_o, and than the share waves velocity V_s by elasticity theory, by means of the following empirical correlations based on cone penetration tests (CPT) and Seismic Dilatometer Test (SDMT) results or laboratory results available in literature.

a) Hryciw (1990):

$$G_o = \frac{530}{(\sigma'_v/p_a)^{0.25}} \frac{\gamma_D/\gamma_w - 1}{2.7 - \gamma_D/\gamma_w} K_o^{0.25} \cdot (\sigma'_v \cdot p_a)^{0.5} \qquad (4)$$

where: G_o, σ'_v and p_a are expressed in the same unit; p_a = 1 bar is a reference pressure; γ_D and K_o are respectively the unit weight and the coefficient of earth pressure at rest, as inferred from SDMT results according to Marchetti (1980);

b) Mayne and Rix (1993):

$$G_o = \frac{406 \cdot q_c^{0.696}}{e^{1.13}} \qquad (5)$$

where: G_o and q_c are both expressed in [kPa] and e is the void ratio. Eq. (5) is applicable to clay deposits only;

c) Jamiolkowski et. al. (1995):

$$G_o = \frac{600 \cdot \sigma'^{0.5}_m p_a^{0.5}}{e^{1.3}} \qquad (6)$$

where: $\sigma'_m = (\sigma'_v + 2 \cdot \sigma'_h)/3$; p_a = 1 bar is a reference pressure; G_o, σ'_m and p_a are expressed in the same unit. The values for parameters which appear in equation (6) are equal to the average values that result from laboratory tests performed on quaternary Italian clays and reconstituted sands. A similar equation was proposed by Shibuya and Tanaka (1996) for Holocene clay deposits.
Equation (6) incorporates a term which expresses the void ratio; the coefficient of earth pressure at rest only appear in equation (4). However only equation (3) tries to obtain all the

input data from the SDMT results.

The V_S values obtained by elasticity theory from G_o are plotted against depth in Figure 9. The method by Jamiolkowski et al. (1995) was applied considering a given profile of void ratio. The coefficient of earth pressure at rest was inferred from SDMT.

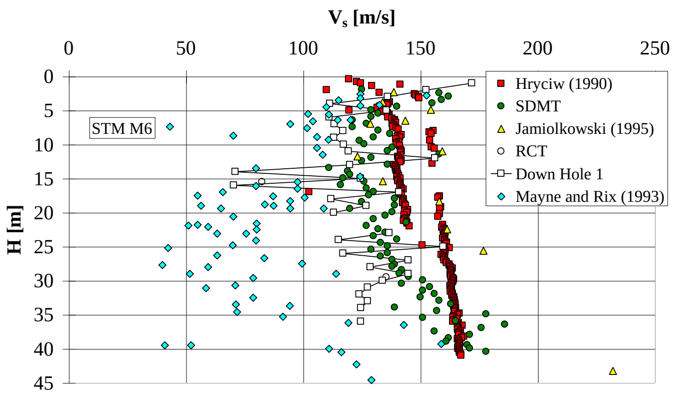


Fig. 9. V_S from laboratory and in situ tests.

1-D LOCAL SITE RESPONSE ANALYSES

The Industrial Area of the city of Catania has been damaged by the Sicilian Earthquake of December 13, 1990 (M_L= 5.4). For the ground response analysis, the Sortino recording on rock (a_{max}= 0.1g) of the Sicilian earthquake of the December 13, 1990, was used. However, the scenario earthquake of the city of Catania is represented by a M = 7.0-7.3 earthquake. According to the microzonation of the scenario earthquake by deterministic approach, an acceleration of 0.30g at the conventional bedrock will be expected (Grasso et al., 2005; Grasso and Maugeri, 2009). By the way, the Italian Regulation NTC 2008, suggest to use as input at the bedrock an acceleration of 0.225g based on the probabilistic approach, with a probability of exceedance less than 10% in 50 years.

Because the industrial building is located in the Catania plain area, the site response has been obtained by a 1-D non-linear code in correspondence of a borehole with D-H test and in correspondence of a SDMT test. The soil response at the surface was 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. The code implements a one-dimensional simplified, hysteretic model for the non-linear soil response. The Down Hole Test (D-H) was performed up to a depth of 40 m. The Seismic Dilatometer Marchetti Test (SDMT) was performed up to a depth of 41 meters (Figure 5). The results show a very detailed and stable shear waves profile. The borehole profile 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. The two 1-D soil columns above the conventional bedrock have a height of 40 m and are excited at the base by accelerograms obtained from the Sortino recording (a_{max}= 0.1g) of the Sicilian earthquake of the December 13, 1990 (Figure 10).

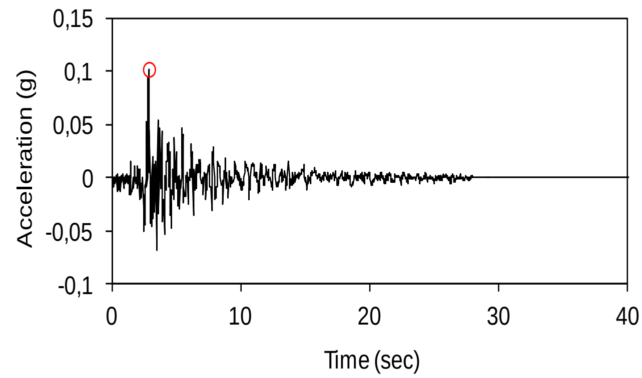
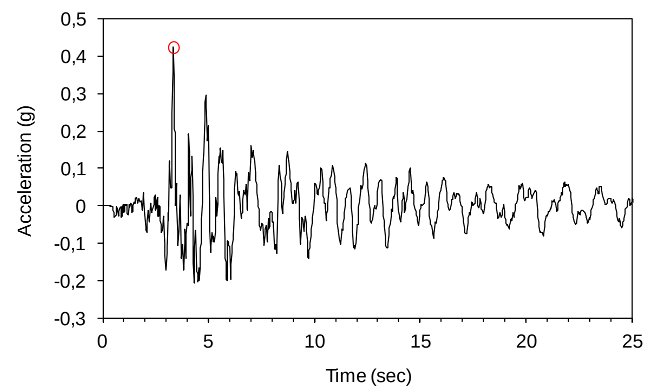
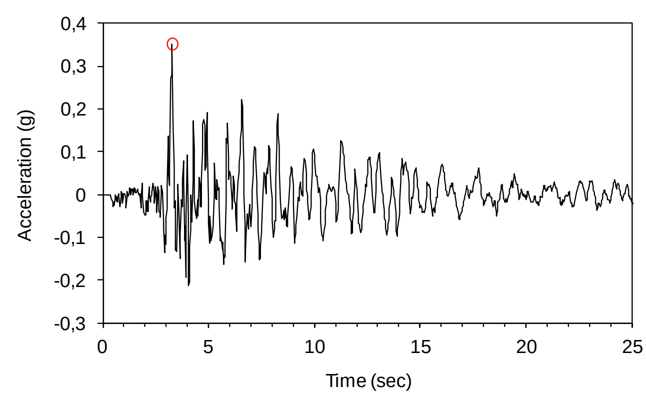


Figure 10. The Sortino E-W component (amax= 0.1g) of the Sicilian earthquake of the December 13, 1990.

The Sortino time history has been scaled to a maximum acceleration of 0.3g and of 0.225g, to simulate the input accelerograms respectively for the deterministic and probabilistic approach. Figure 11 shows the acceleration time history at the surface, for the deterministic approach (Figure 11a) and for the probabilistic approach (Figure 11b).



(a)



(b)

Figure 11. Soil response analysis: a) deterministic approach (a_{max} = 0.42g); b) probabilistic approach (a_{max} = 0.35g).

In Figure 11 is reported the soil response for the SDMT Vs profiles. The soil response by the D-H Vs profiles give maximum acceleration of 0.35g for the deterministic approach ad of 0.31g for the probabilistic approach.
Figure 12 shows the results in terms of maximum accelerations with depth for the deterministic approach; Figure 12 shows that the peak acceleration is higher using the Vs profile obtained from the SDMT than that obtained using the Vs profile obtained from the D-H. Results of the site response analysis show also higher values of soil amplification factors obtained by probabilistic approach, than those obtained by the deterministic approach; because in the latter case the input acceleration is bigger and so the non-linearity soil behavior produces a decreasing of soil amplification.

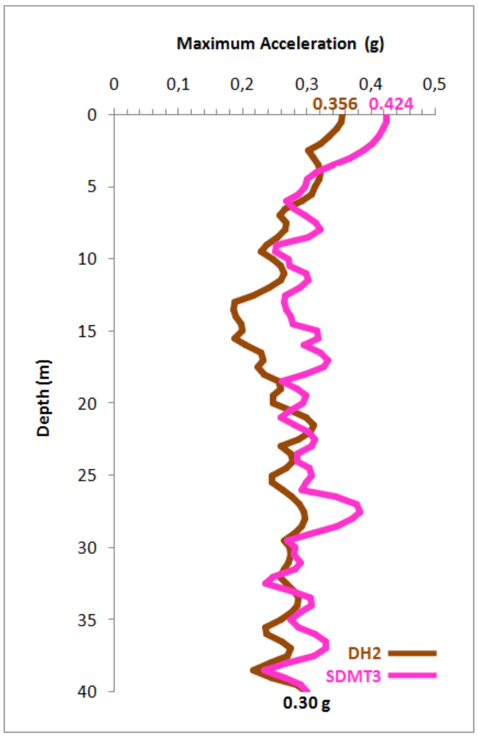


Fig. 12. Maximum accelerations with depth obtained from the response analyses for D-H and SDMT Vs profiles, using the Sortino input accelerogram scaled to 0.3g (deterministic approach).

Figure 13 shows the results in terms of Amplitude Ratio, Fourier amplitude and response spectrum for the deterministic approach, using the SDMT soil profile. The maximum values of the amplification ratio, Fourier amplitude are obtained for the frequency 0.55 and 1.4 Hz. The maximum values of spectral acceleration and relative velocity are obtained for about the period 0.7s and 1.8s; the maximum spectral relative displacement is obtained for a period of about 1.8 s.
Figure 14 shows the similar results for the deterministic approach using the D-H soil profile. The maximum values of the amplification ratio, Fourier amplitude are obtained for the frequency 0.55 and 2 Hz.

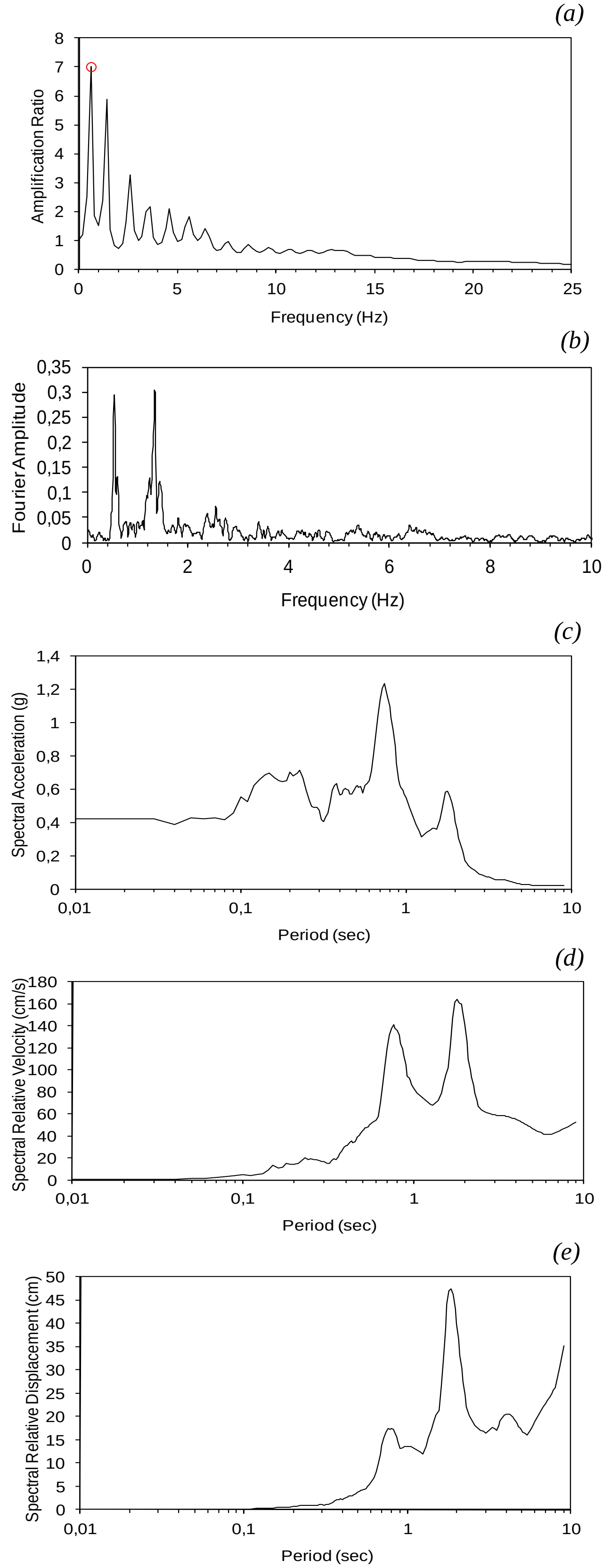


Figure 13. Site Response analysis results for the deterministic approach using the SDMT soil profile. a) Amplification Ratio; b) Fourier amplitude; c) Spectral Acceleration; d) spectral velocity; e) spectral relative displacement.

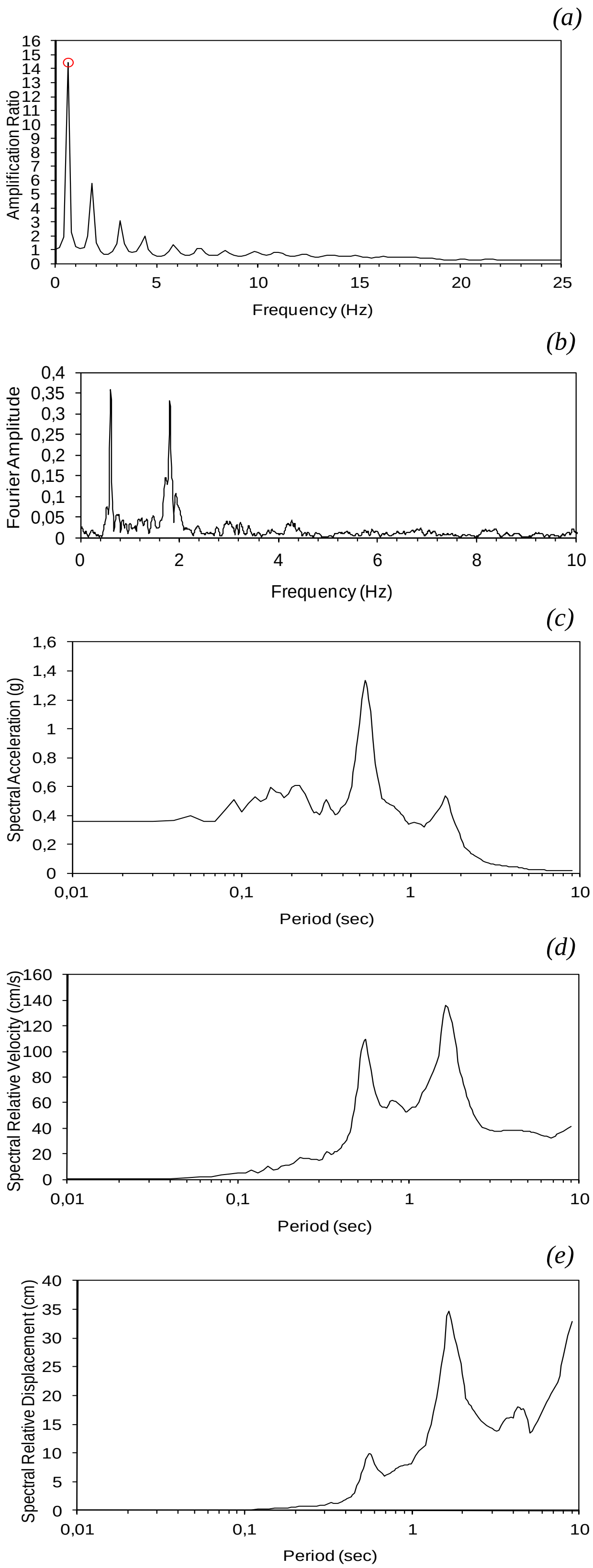


Figure 14. Site Response analysis results for the deterministic approach using the D-H soil profile. a) Amplification Ratio; b) Fourier amplitude; c) Spectral Acceleration; d) spectral velocity; e) spectral relative displacement.

The maximum values of spectral acceleration and relative velocity are obtained for about the period 0.5s and 1.8s; the maximum spectral relative displacement is obtained for a period of about 1.8 s.

CONCLUSIONS

An industrial building is located in an area prone to high seismic hazard. Recently the building was re-used in the 2012 for producing solar panels and a detailed site investigation was carried out by means of in situ dynamic D-H and SDMT and by laboratory tests RCT for the evaluation of soil non linearity. Based on the Vs soil profile evaluated by in situ tests and Gγ and Dγ curves evaluated by laboratory tests, a site response analysis has been performed, by 1-D seismic code because the industrial building is located in a plain area. As input motion was used the Sicilian 1990 earthquake recorded at Sortino station, scaled up to the deterministic values of the maximum acceleration 0.3g and the probabilistic values of the acceleration 0.225g given by the Italian Regulation with the probability of exceedance less than 10% in 50 years. The maximum values of acceleration at the surface was of 0.42g by the SDMT Vs profile, higher than that of 0.35g obtained by the D-H profile. Soil amplification is higher for the SDMT Vs profile than that for D-H profile. Moreover, soil amplification value is higher by the probabilistic approach, rather than obtained by the deterministic approach; because in the latter case the input acceleration is bigger and so the non-linearity soil behavior produces a decreasing of soil amplification. The predominant periods are about 0.5s and 1.9s.

REFERENCES

Bardet, J. P., Ichii, K., Lin, C. H. [2000]. “EERA: a computer program for equivalent-linear earthquake site response analyses of layered soil deposits”, user manual, University of Southern California, 2000, 40 pp.

Carrubba, P. and Maugeri, M. [1988]. “Determinazione delle Proprietà Dinamiche di un’Argilla Mediante Prove di Colonna Risonante”. *Rivista Italiana di Geotecnica*, No. 2, pp. 101 - 113.

Castelli F., Lentini V., Maugeri M. [2008]. “Experimental and numerical analysis of the behavior of an embankment stabilized with vertical drains". Sixth International Conference On Case Histories In Geotechnical Engineering, Arlington, VA, August 4-9, 2008. (paper 2.37). Copyright © 2008 Proceedings: Missouri University of Science and Technology, 7 p.

Castelli F., Lentini V. and Maugeri M. [2012]. “Dynamic Clay Properties by in Situ and Laboratory Tests for an Industrial Building in Catania City (Italy)”. *Proceedings of the 2nd International Conference on Performance-Based Design in Earthquake Geotechnical Engineering,* Taormina, 28 - 30 May 2012, paper n°. 02_15, pp. 341 - 353.

Cavallaro, A., Ferraro A., Grasso, S., Maugeri M. [2012]. “Topographic effects on the Monte Po Hill in Catania (Italy)”. *Soil Dynamics and Earthquake Engineering*, 43 (2012), pp. 97–113.

Cavallaro, A., Grasso, S., Lentini V. and Maugeri M. [2008]. “Geotechnical Characterization of a Soft Clay Soil Subjected to a Preloading Embankment”. *Proceedings of 6th International Conference on Case Histories in Geotechnical Engineering*, Arlington, VA, August 11 - 16, 2008.

Cavallaro, A., Capilleri, P., Grasso S., and Maugeri, M. [2012]. “Dynamic Soil Characterization by Different Test Sites for Microzonation of the City of Catania”. *Proceedings of the 2nd International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, Taormina, 28 - 30 May 2012, paper n°. 02_16, pp. 354 – 368.

Grasso S., Laurenzano G., Maugeri M. And Priolo E. [2005]: “Seismic response in Catania by different methodologies”. Chapter IV in: Seismic Prevention of Damage: A Case Study in a Mediterranean City. M. Maugeri Editor. WIT Press, Southampton (UK).

Grasso S., Maugeri M. [2009]. The Seismic Microzonation of the City of Catania (Italy) for the Maximum Expected Scenario Earthquake of January 11, 1693. Soil Dynamics and Earthquake Engineering. ISSN: 0267-7261. Vol. 29 No. 6 (2009): 953-962.

Hryciw, R.D. [1990]. “Small Strain Shear Modulus of Soil by Dilatometer”. *JGED, ASCE*, Vo. 116, N°. 11: 1700-1715.

Jamiolkowski, M., Lo Presti, D.C.F. and Pallara, O. [1995]. “Role of In-Situ Testing in Geotechnical Earthquake Engineering”. *Proceedings of 3rd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamic*, State of the Art 7, St. Louis, Missouri, April 2-7, 1995, vol. II, pp. 1523 - 1546.

Kurtulus, A. and Stokoe, K. H. [2008]. “In Situ Measurement of Nonlinear Shear Modulus of Silty Soil”. *Journal Geotechnical Geoenvironmental Engineering* ASCE Vol. 134, No. 10, pp. 1531 - 1541.

Marchetti, S. [1980]. “In situ tests by flat dilatometer”. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 106, No. GT3, pp. 299 - 321.

Marchetti, S., Monaco, P., Totani, G., Marchetti, D. [2008]. “In Situ Tests by Seismic Dilatometer (SDMT)”. In: “*From Research to Practice in Geotechnical Engineering*", ASCE Geotech. Spec. Publ. No. 180 Honouring John H. Schmertmann, 292 - 311.

Martin, G.K. and Mayne, P.W. [1997]. “Seismic Flat Dilatometers Tests in Connecticut Valley Vaeved Clay”. *ASTM Geotechnical Testing Jornal*, 20 (3), pp. 357 - 361.

Martin, G.K. and Mayne, P.W. [1998]. “Seismic Flat Dilatometers Tests in Piedmont Residual Soils”. *Geotecnical Site Characterization*, Vol. 2, Balkema, Rotterdam, pp. 837 - 843.

Maugeri, M. [1995]. “Discussions and Replies Session IX”. *Proceedings of International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, 2 – 7 April 1995, pp. 1323 - 1327.

Mayne, P.W. and Rix, G.J. [1993]. “G_{max} -q_c Relationships for Clays”. *Geotechnical Testing Journal,* Vol. 16, N°. 1: 54 - 60.

Yokota, K., Imai, T. and Konno, M. [1981]. “Dynamic Deformation Characteristics of Soils Determined by Laboratory Tests”. *OYO Tec. Rep.* 3, pp. 13 - 37.