Site effects and soil liquefaction of the sandy soil in Catania harbour

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Abstract

The harbour of the city of Catania, located on the eastern zone of Sicily, is an area exposed to high seismic hazard. The city of Catania in the South-Eastern Sicily has been affected by several destructive earthquakes of magnitude about 7.0+ in past times; repetition of events with similar characteristics would generate the additional risk of a damaging tsunami, as well as liquefaction phenomena around the coast. In situ investigations of sandy harbour soil were carried out in order to determine the soil profile and the geotechnical characteristics for the site under consideration, with special attention to the variation of shear modulus and damping with depth. Seismic Dilatometer Marchetti Tests (SDMT) have been carried out, with the aim to evaluate the soil profile of shear wave velocity (V_S). Moreover the following investigations in the laboratory were carried out on undisturbed samples: resonant column tests; direct shear tests; triaxial tests. The available data obtained from the SDMT results enabled the evaluation of the shear modulus profile. In addition, using some synthetic seismograms of historical scenario earthquakes at the bedrock, the ground response analysis at the surface, in terms of time history and response spectra, has been performed by the 1-D non-linear code EERA. The results of the site response analysis will be also used for the evaluation of liquefaction hazard of the investigated area.

1 Introduction

The coastal plain of the city of Catania, which is recognized as a typical Mediterranean city at high seismic risk, was investigated by Seismic Dilatometer Marchetti Tests (SDMT). Seismic liquefaction phenomena were reported by

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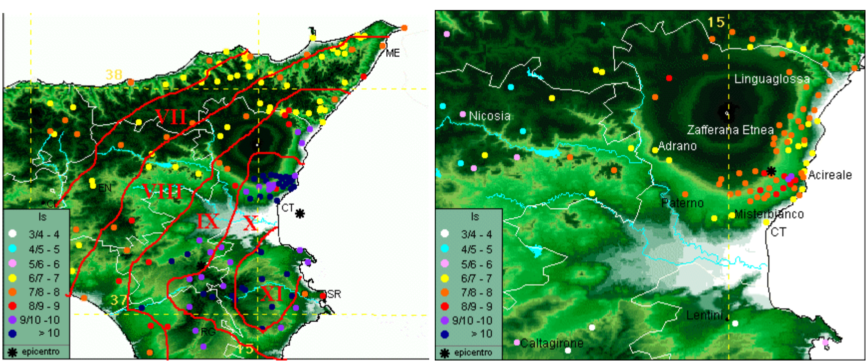
historical sources following the 1693 (M_s = 7.0-7.3, I_o = X-XI MCS) and 1818 (M_s = 6.2, I_o = IX MCS) Sicilian strong earthquakes (fig. 1) [1–3]. The most significant liquefaction features seem to have occurred in the Catania area, situated in the meisoseismal region of both events. These effects are significant for the implications on hazard assessment mainly for the harbour of the city, where most facilities are located. The Val di Noto earthquake of January, 11 1693 is the best remembered by Sicilians. The shock of January 11 which propagated from the epicentre (situated at sea but not far from the coast) measured XI on MCS [5, 6] (see fig. 1). The life toll was enormous: estimates of victims vary from 11,000 (more probable) to 20,000 from a total of about 23,000–27,000 inhabitants. In contrast, the Etna earthquake that took place on February 20, 1818 was one of the feeblest ever to have occurred but its effects were noticed over a vast area. This event as a whole shows that the quake reached the peak of IX on MCS [7] (see fig. 1). The isoseismal map explains that the earthquake was felt almost in every part of Sicily from Siracusa to Noto and Palermo.

In order to study the dynamic characteristics of soils in the Catania harbour area, laboratory and in situ investigations have been carried out to obtain soil profiles with special attention being paid to the variation of the shear modulus (G) and damping ratio (D) with depth. SDMTs have been also carried out in the zone of the harbour area, with the aim of an accurate geotechnical characterisation, including the evaluation of the soil shear wave velocity (V_S) profile, as well as the profile of the horizontal stress index K_D. Moreover the following investigations in the laboratory were carried out on undisturbed samples: resonant column tests; direct shear tests; tri-axial tests.



(a) (b)

Figure 1: Isoseismal maps with shocked localities. (a) Earthquake of January 11, 1693; (b) Earthquake of February 20, 1818; after [4], modified.

2 Geotechnical characterisation by SDMTs

To evaluate the geotechnical characteristics of the soil, the following in situ and laboratory tests were performed in the Catania harbour area: 5 SDMTs; 3 direct shear tests; 3 tri-axial CD tests; 6 resonant column tests (RCT). The investigation programme was performed in the zone of "*Acquicella Porto*" in the Catania harbour. The 5 SDMTs (SDMT1-5) have an effective depth of 30.50 m, 32.00 m, 31.00 m, 30.00 m and 32.00 m. Fig. 1 shows the location of the SDMTs in the Catania harbour. The SDMT [8–10] 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 by a co-axial cable with an oscilloscope [11, 12]. The measured arrival times at successive depths provide pseudo interval V_S profiles for horizontally polarized vertically propagating shear waves. The small strain shear modulus G_0 is determined by the theory of elasticity by the well-known relationships: $G_0 = \rho V_S^2$ where ρ is the mass density. SDMT obtained parameters are: I_d, a material index giving information on soil type (sand, silt, clay), fig. 3a; M, the vertical drained constrained modulus, fig. 3b; Phi, the angle of shear resistance, fig. 4a; K_D, fig. 4b; V_S, fig. 5a; G_0, fig. 5b.

The profile of K_D is similar in shape to the profile of the overconsolidation ratio (OCR). $K_D = 2$ indicates that, in clays, OCR = 1, $K_D > 2$ indicates overconsolidation. A first glance at the K_D profile is helpful to "understand" the deposit. The "Acquicella" site along the southern coast line of Catania is characterized by fine sands with thin limestones.

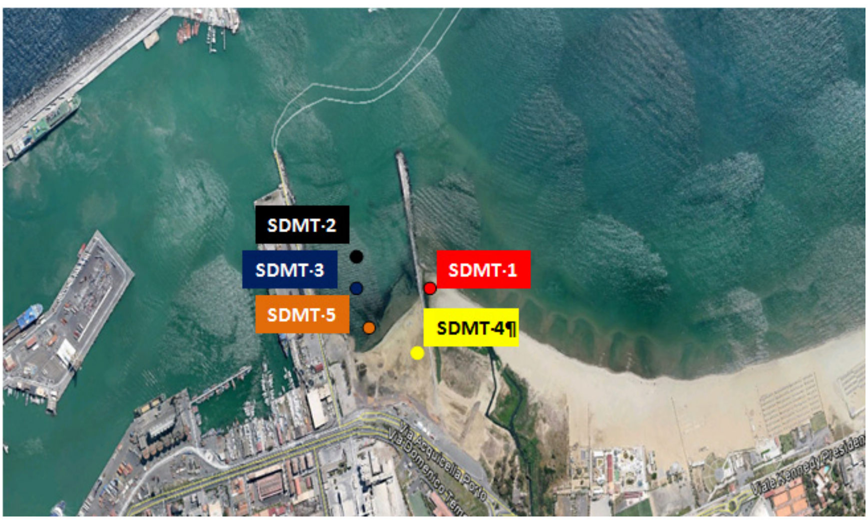
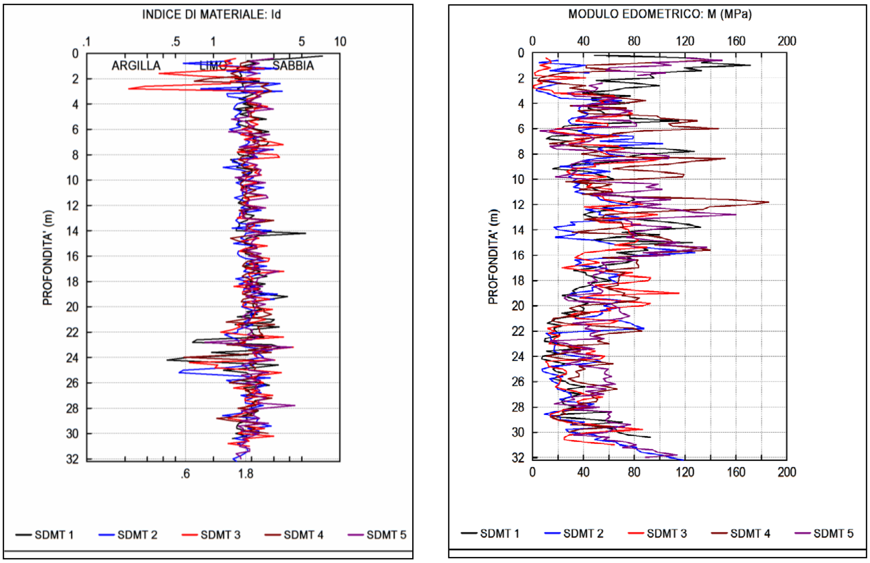
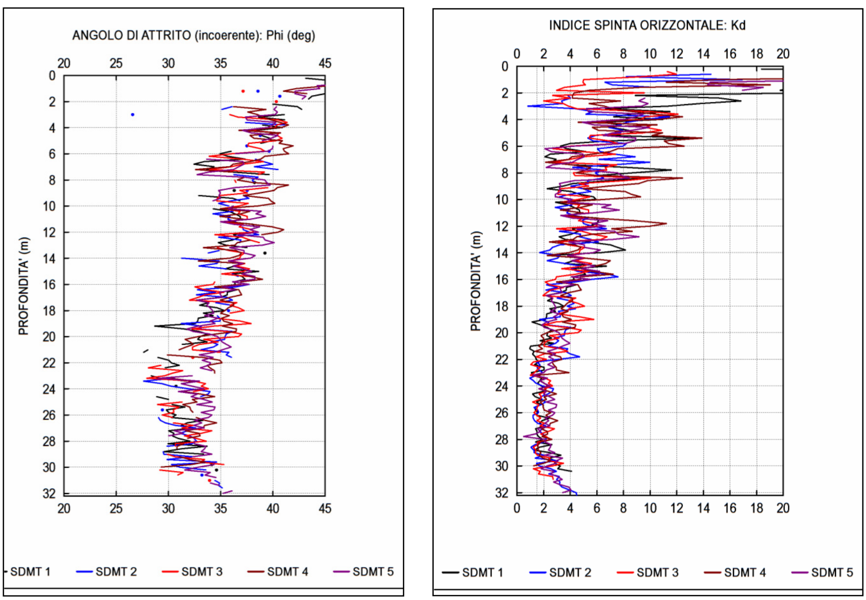


Figure 2: Location of the 5 SDMTs in the *"Acquicella Porto"* Catania harbour.



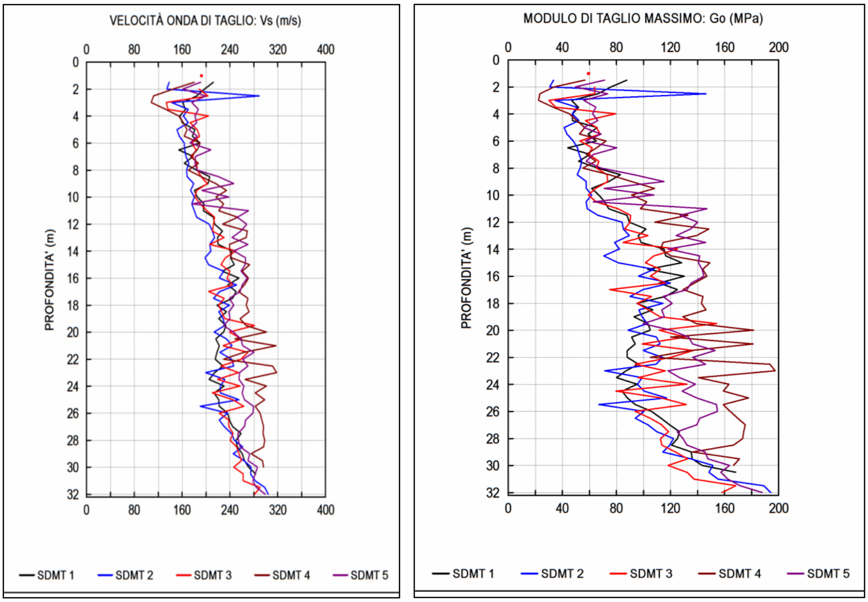
(a) (b)

Figure 3: (a) Id: material index; (b) M: vertical drained constrained modulus.



(a) (b)

Figure 4: (a) Phi: angle of shear resistance; (b) K_D: horizontal stress index.



(a) (b)

Figure 5: (a) V_S shear wave velocity; (b) G_0: small strain shear modulus.

Fig. 6 shows as an example the results of the direct shear tests performed on the sample retrieved at the depth of 8.50 m from borehole SDMT3. Other two tests have been performed on samples retrieved from borehole SDMT1 at the depths of 13.20 and 39.00 m. Results of the laboratory tests (direct shear tests; tri-axial tests) performed on samples show that the soils characterised are cohesionless, with values of the angle of shear resistance of about 37°–38°.

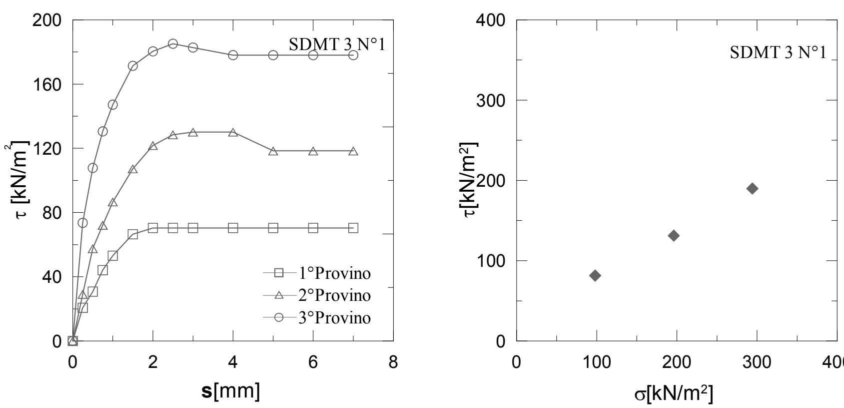
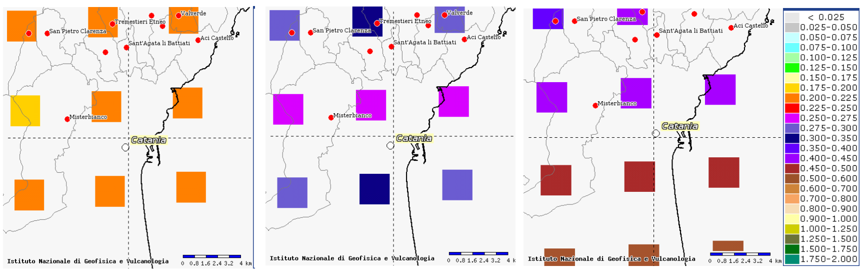


Figure 6: Results of the direct shear test performed on the sample retrieved at the depth of 8.50 m from borehole SDMT3.

3 Seismic acceleration for the evaluation of site effects

Site response analysis, performed by the EERA code [13], was carried out for all the normalized shear modulus and damping ratio reported. 1-D columns have a height of 30–32 m and are excited at the base by accelerograms obtained from the synthetic seismograms of 1693, with a PGA of 0.225g (fig. 7a) corresponding to a return period of 475 years in the current Italian regulatory text “seismic hazard and seismic classification criteria for the national territory” obtained through a probabilistic approach in the interactive seismic hazard maps [14, 15]. Further analyses have been performed using scaled seismograms, to the maximum PGA of 0.275g (corresponding to the return period of 975 years, fig. 7b) and to the maximum PGA of 0.400g (corresponding to the return period of 2475 years, fig. 7c).



(a) (b) (c)

Figure 7: Interactive seismic hazard map of the city of Catania. (a) 10% probability of exceedance in 50 years (return period of 475 years); (b) 5% probability of exceedance in 50 years (return period of 975 years); (c) 2% probability of exceedance in 50 years (return period of 2475 years).

Using these time histories, response spectra concerning the investigated site have been deduced. The soil response at the surface was modeled using the computer code EERA (Equivalent–linear Earthquake site Response Analyses of Layered Soil Deposits) for the calculation of the amplitude ratios and spectral acceleration. Fig. 8 shows the results in terms of maximum accelerations with depth for SDMTs No. 1–5, for the 475, 975 and 2475 return periods. Results of the site response analysis show high values of soil amplification factors especially for the 475 and for the 975 return periods of the scenario earthquake. Probably this fact is due to a nonlinear behaviour of soil that often occurs [16], especially in the presence of the strong accelerations of the 975 and 2475 earthquake scenarios. High values of soil amplification factors often occur in the city of Catania due to the characteristics of soils, both stratigraphic and topographic [17–20]. The results of the site response analyses have then been used for the evaluation of liquefaction hazard of the investigated area, in terms of the maximum acceleration of the scenario earthquake chosen in the analyses.

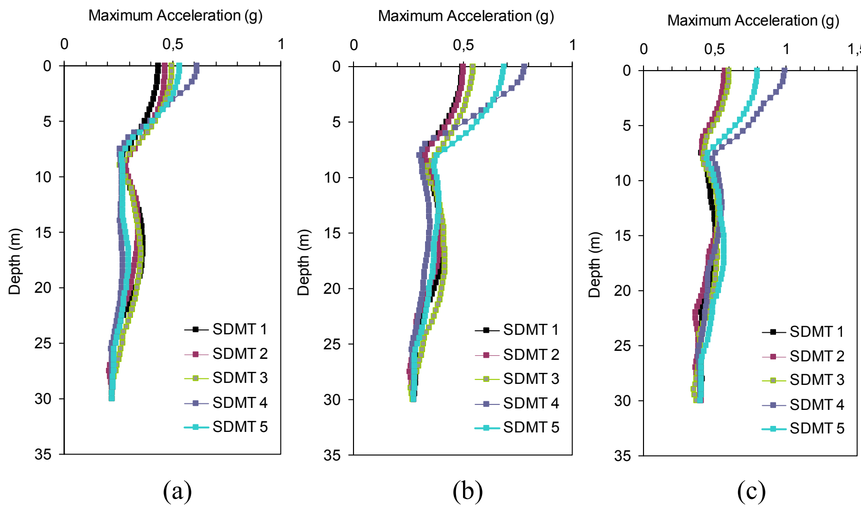


Figure 8: Maximum accelerations with depth for SDMTs No. 1–5 profiles. (a) 475 years earthquake scenario return period; (b) 975 years earthquake scenario return period; (c) 2475 years earthquake scenario return period.

4 SDMT-based procedure for evaluating soil liquefaction

Seismic liquefaction phenomena were reported by historical sources following the 1693 and 1818 earthquakes. The most significant liquefaction features seem to have occurred in the Catania area, situated in the meisoseismal region of both events. Extensive liquefaction effects occurred in the Catania area following the January 11, 1693 mainshock. Probably due to the severity of the earthquake (M_s = 7.0-7.3, I_o = X-XI MCS), contemporary sources tended essentially to describe the catastrophic consequences of damage suffered by the towns, providing only generic information on seismogeological effects among which the liquefaction-induced features. Often during strong earthquakes, effects of liquefaction phenomena are visible also far from the epicentral area [21]. Previous studies performed in the industrial area of the city of Catania revealed a high liquefaction hazard during a possible repetition of the scenario earthquakes [22, 23].

4.1 Cyclic shear stress ratios induced by earthquake ground motions

The susceptibility of a site to seismic-induced liquefaction may be assessed by comparing the cyclic soil resistance to the cyclic shear stresses due to the ground motion. The latter is of course a function of the design earthquake parameters, while the former depends on the soil shear strength and can be computed using results from in situ tests.

The traditional procedure, introduced by Cavallaro *et al.* [20], has been applied for evaluating the liquefaction resistance of *"Acquicella Porto"* harbour sandy soils. This method requires the calculation of the cyclic stress ratio (CSR), and cyclic resistance ratio (CRR). If CSR is greater than CRR, liquefaction can occur. The cyclic stress ratio CSR is calculated from the following equation [24]:

$$\mathrm{CSR} = \tau_{av}/\sigma'_{vo} = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d/\mathrm{MSF} \qquad (1)$$

where τ_{av} is the average cyclic shear stress, a_{max} the peak horizontal acceleration at the ground surface generated by the earthquake, g the gravitational acceleration, σ_{vo} and σ'_{vo} the total and effective overburden stress, respectively, r_d a stress reduction coefficient depending on depth and MSF a magnitude scaling factor. Seed and Idriss [24] introduced the stress reduction coefficient r_d as a parameter describing the ratio of cyclic stresses for a flexible soil column to the cyclic stresses for a rigid soil column. As regards the peak horizontal acceleration, the value of 0.45g has been chosen, It is the value of the acceleration with the 5% probability of exceedance in 50 years (return period of 975 years), amplified with an amplification factor of 1.80 given by the seismic response analysis. The magnitude scaling factor, MSF, has been used to adjust the induced CSR during earthquake of magnitude M (M =7.3 of the 1693 scenario earthquake) to an equivalent CSR for an earthquake magnitude, M = 7½. Fig. 9 shows typical CSR profiles obtained from boreholes SDMT1-3.

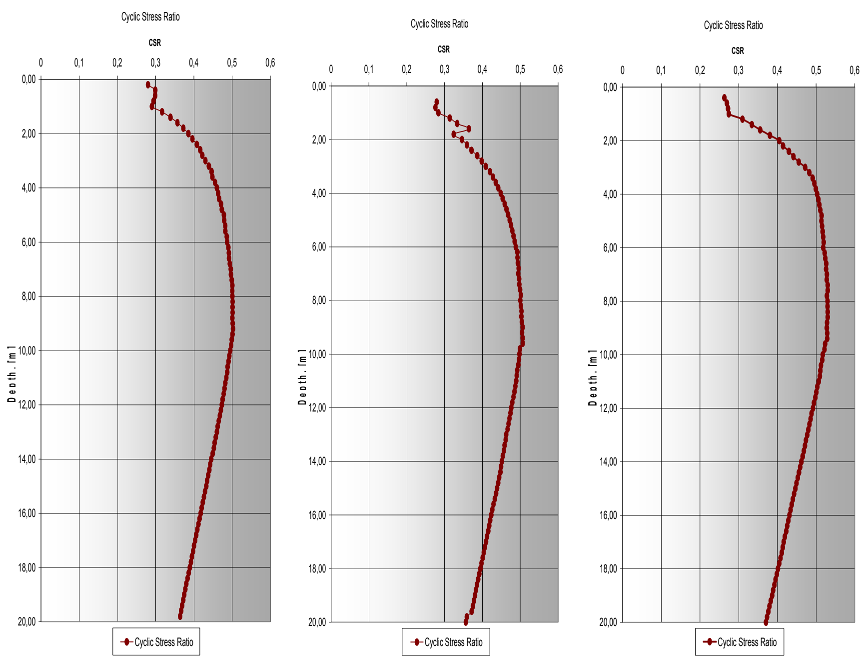


Figure 9: CSR profiles obtained from equation (1) for boreholes SDMT1-3.

4.2 Evaluation of CRR from the DMT horizontal stress index K_D

Marchetti [25] and later studies suggested that the horizontal stress index K_D from DMT ($K_D = (p_o - u_o)/\sigma'_{vo}$) is a suitable parameter to evaluate the liquefaction resistance of sands. Previous CRR-K_D curves were formulated by Marchetti [25]. The following CRR-K_D curves have been used in the present study, approximated by the equations:

$$\mathrm{CRR} = 0.0107K_D^3 - 0.0741K_D^2 + 0.2169K_D - 0.1306 \tag{2}$$

$$\mathrm{CRR} = 0.0242\mathrm{e}^{(0.6534K_D)} \tag{3}$$

$$\mathrm{CRR} = 0.0084\,K_D^{2.7032} \tag{4}$$

Eqn (2) has been developed by Monaco *et al.* [26]; eqns (3) and (4) have been developed by Grasso and Maugeri [23]. Fig. 10 shows CRR- K_D trends for SDMT1-2.

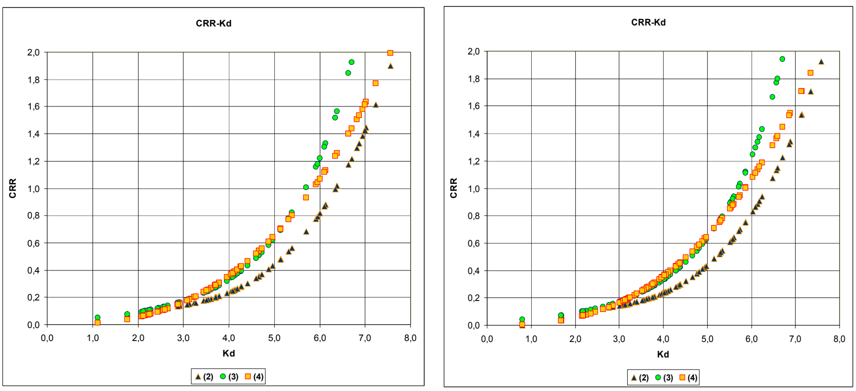


Figure 10: CRR- K_D trends obtained used K_D values from SDMT1-2.

4.3 Evaluation of CRR from shear wave velocity V_S measured by SDMT

The use of the shear wave velocity, V_S, as an index of liquefaction resistance has been illustrated by several authors [27, 28]. The V_S based procedure for evaluating CRR has advanced significantly in recent years. The correlations between V_S and CRR used in the present study are given by Andrus and Stokoe [27] and Andrus *et al.* [28]:

$$\mathrm{CRR} = a\left(\frac{V_{S1}}{100}\right)^2 + b\left(\frac{1}{V_{S1}^* - V_{S1}} - \frac{1}{V_{S1}^*}\right) \tag{5}$$

$$\mathrm{CRR} = \left[0.022\left(\frac{K_{a1}V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{S1}^* - (K_{a1}V_{S1})} - \frac{1}{V_{S1}^*}\right)\right]K_{a2} \tag{6}$$

where: V^*_{S1} is the limiting upper value of V_{S1} for liquefaction occurrence; $V_{S1} = V_S(p_a/\sigma'_{vo})^{0.25}$ is the corrected shear wave velocity for overburden-stress; a and b in eqn (5) are curve fitting parameters, while K_{a1} and K_{a2} are aging factors equal to 1.0 for uncemented soils of Holocene age. The correlations given by eqns (2), (3), (4), (5) and (6) have been then used for the evaluation of liquefaction potential index P_L [29], using the K_D and V_S values measured by SDMT instead. Figs. 11 and 12 show P_L values obtained, respectively, from CRR-K_D and CRR-V_S correlations for SDMT1-2.

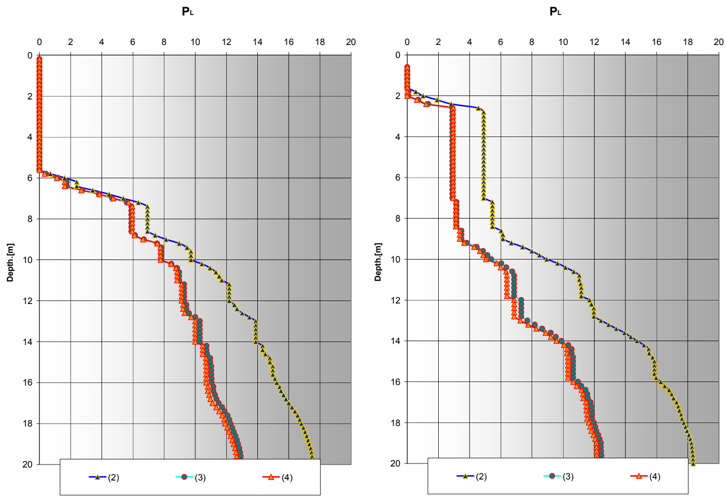


Figure 11: Liquefaction potential index P_L obtained from CRR- K_D correlations.

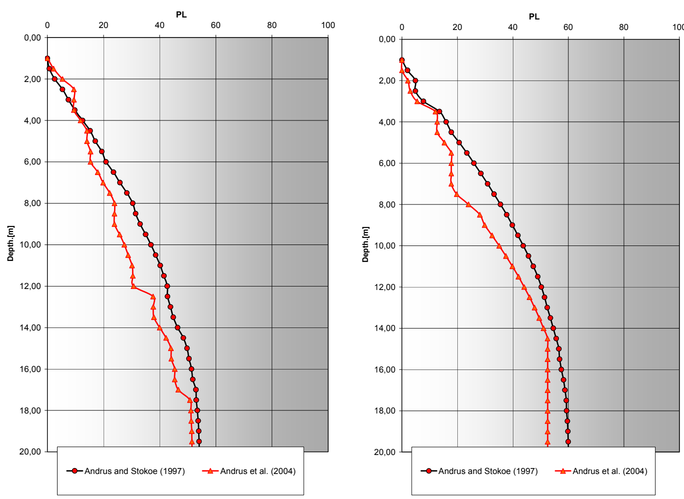


Figure 12: Liquefaction potential index P_L obtained from CRR- V_S correlations.

However the CRR-V_S correlations are not reliable when V_S exceeds the value of 225 m/s. In addition, the V_S measurements are made at small strains, whereas pore-pressure build up and liquefaction are medium- to high-strain phenomena. Thus, it could be preferable to evaluate liquefaction by K_D measurements, which is related to medium-high strains [30–32].

5 1-D Local site response analyses

Synthetic seismograms have been drawn for the sites along a set of receivers placed at different depths, starting from the surface up to almost 30 m, both for the 1693 scenario earthquake [33–37] and for the 1818 scenario earthquake. Using 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 a 1-D non-linear code in correspondence of five SDMTs. Local site response analyses have been brought for the "la Plaja" beach by a 1-D linear equivalent computer code EERA. Using this time history, response spectra concerning the investigated site have been deduced. Fig. 13 shows the results in terms of time history of maximum acceleration at the surface for SDMTs No. 1–5, for the 475, 975 and 2475 return periods.

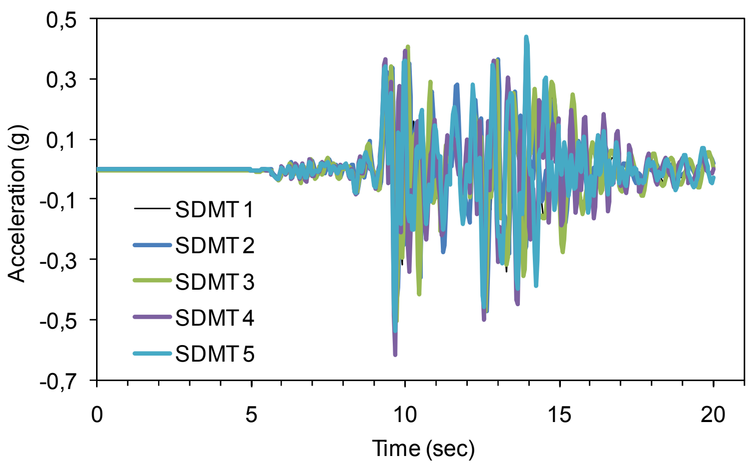


Figure 13: Maximum accelerations at the surface for SDMTs No. 1–5 profiles (475 years earthquake scenario return period).

Results of the site response analysis show high values of soil amplification factors especially for the 475 and for the 975 return periods of the scenario earthquake. Probably this fact is due to nonlinear soil behaviour, especially in the presence of the strong accelerations of the 975 and 2475 earthquake scenarios.

Fig. 14 shows the results in terms of response spectra for SDMTs 1–5 profiles, for the 475 years earthquake scenario return period.

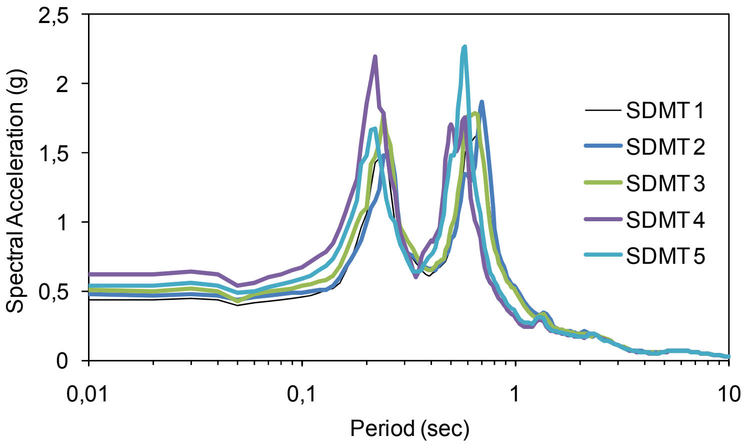


Figure 14: Response spectra for SDMTs 1–5 profiles (475 years earthquake scenario return period).

6 Conclusions

In this paper some information concerning the geotechnical characterisation by SDMT tests for soil liquefaction evaluation of the *"Acquicella Porto"* zone in the Catania harbour (Italy) have been presented. Local site response analyses have also been carried out for the "Acquicella Porto" area by a 1-D linear equivalent computer code EERA for the evaluation of the amplification factors of the maximum acceleration [38–46]. The results of the site response analysis have also been used for the evaluation of liquefaction hazard of the investigated area. Results of the site response analysis show high values of soil amplification factors especially for the 475 and for the 975 return periods of the scenario earthquake, higher than those obtained by the current Italian regulatory text. CRR-K_D correlations have been used for the evaluation of liquefaction potential index, P_L. The results obtained from SDMT1 show that the liquefaction potential index P_L is below 5 (low risk) up to a depth of about 7 m; while the results obtained from SDMT2 show low risk up to a depth of 10 m [47]. By the way, it is unlikely to have liquefaction at a depth greater than 7–10 m.

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