Site Response Analysis and Liquefaction Hazard Evaluation in the Catania Harbour (Italy)

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ABSTRACT: In this paper some information concerning the geotechnical characterisation of the *"Acquicella Porto"* zone in the Catania harbour have been presented. The harbour of the city of Catania, located on the eastern zone of Sicily, is an area subjected to high seismic hazard. 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 for 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. 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 1-D linear equivalent codes. The results of the site response analysis have been also used for the evaluation of liquefaction hazard of the investigated area. This paper presents a K_D-CRR corrected relation for sandy soils potential liquefaction evaluation under cyclic loading, on the basis of previous tentative K_D-CRR curves proposed.

1 INTRODUCTION

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. This paper tries to summarise this information in a comprehensive way in order to provide a representative model of ground condition of an important zone in the city of Catania (Bonaccorso et al., 2005), for realistic seismic scenarios response analysis. The city of Catania, located on the eastern part of Sicily, is one of the most seismically active areas of Italy ((Grasso et al., 2004; Grasso and Maugeri 2005, 2009a, 2009b; 2012). The earthquake of January, 11, 1693 is considered one of the biggest earthquakes which occurred in Italy. This earthquake, with an intensity of XI degree of MCS scale in many centres, struck a vast territory of south-eastern Sicily and caused the partial, and in many cases total, destruction of 57 cities and 60000 casualties (Abate et al., 2009; Cavallaro et al., 2003; Massimino and Maugeri, 2003; Di Prisco et al., 2006; Maugeri et al., 2012).

2 GEOTECHNICAL CHARACTERISATION BY SDMT TESTS

To evaluate the geotechnical characteristics of the soil, the following in situ and laboratory tests were performed in the Catania harbour area:

N°. 5 Seismic Dilatometer Tests (SDMT);
N°. 3 Direct Shear Tests;
N°. 3 Triaxial CD Tests;
N°. 6 Resonant Column Tests (RCT);

The investigation programme was performed in the zone of *"Acquicella Porto"* in the Catania harbour. The 5 Seismic Dilatometer Tests (SDMT1-5) have an effective depth of 30.50 m, 32.00 m, 31.00 m, 30.00 m, 32.00 m. Fig. 1 shows the location of the SDMTs in the Catania harbour. The SDMT (Marchetti, 1980; Marchetti et al., 2008;

Monaco et al., 2009) 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.

Fig. 1. Location of the 5 SDMTs in the Catania harbour.

Soil stiffness, at small strains, is a relevant parameter in solving boundary value problems such as: seismic response of soil deposits to earthquakes; dynamic interaction between soils and foundations; design of special foundations for which the serviceability limit allows only very small displacements. 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 coaxial cable with an oscilloscope (Martin and Mayne, 1997; 1998).

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 = mass density.

SDMT obtained parameters by the equipment shown in Fig. 2 at the site are: I_d: Material Index; gives information on soil type (sand, silt, clay); M: Vertical Drained Constrained Modulus; Phi: Angle of Shear Resistance, Fig. 3; K_D: Horizontal Stress Index, Fig. 4 (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.

Fig. 2. SDMT equipment at the *"Acquicella Porto"* site.

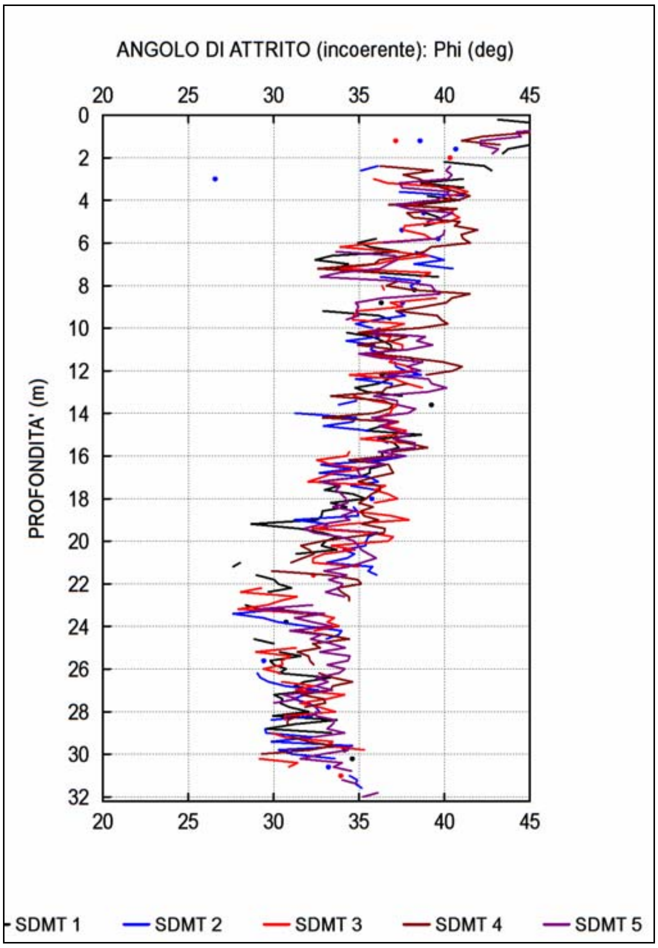

Fig. 3. Phi: Angle of Shear Resistance of the 5 SDMTs in the Catania "Acquicella" harbour.

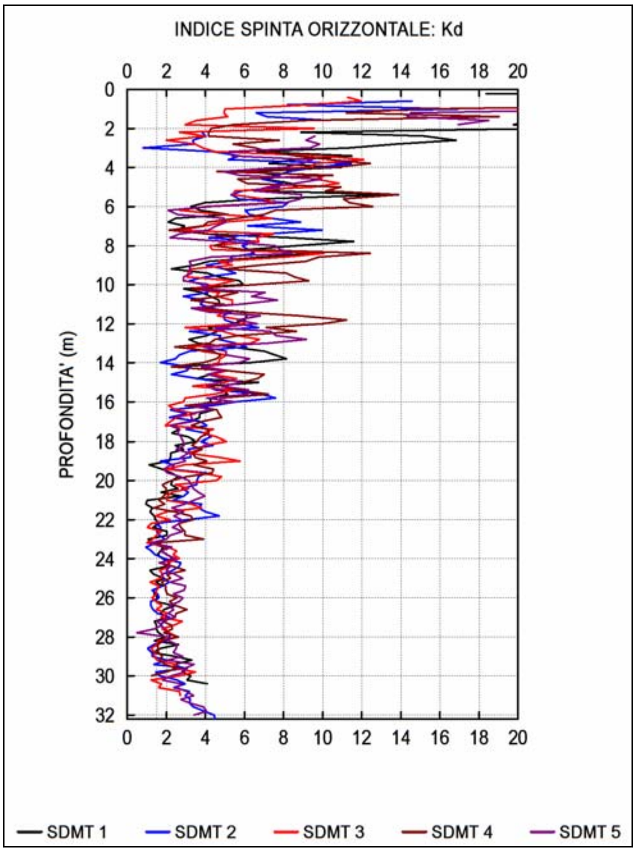

Fig. 4. K_D: Horizontal Stress Index of the 5 SDMTs in the Catania "Acquicella" harbour.

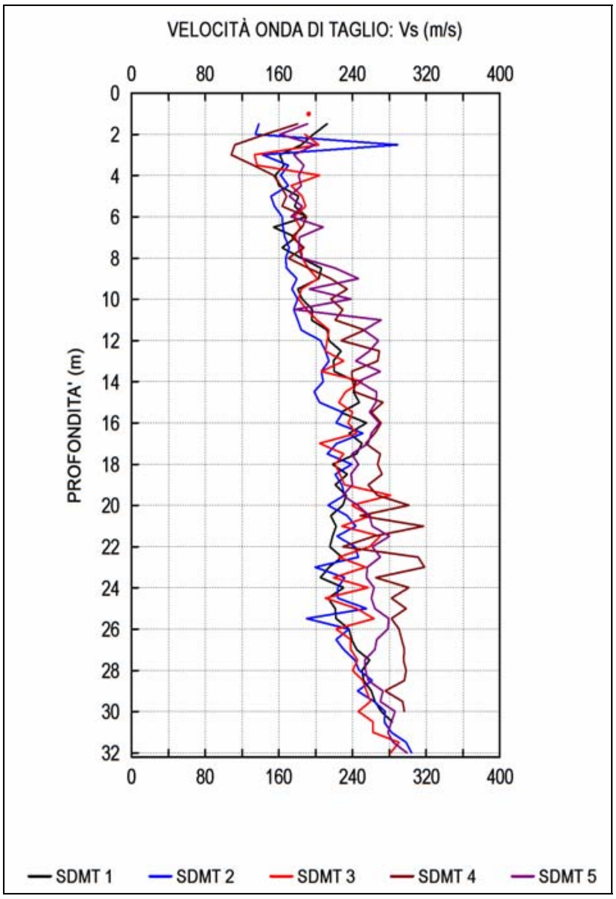

Fig. 5. Vs: Shear Wave Velocity of the 5 SDMTs in the Catania "Acquicella" harbour.

A first glance at the K_D profile is helpful to "understand" the deposit); V_s: Shear Waves Velocity, Fig. 5; $G_0 = \rho V_s^2$ Small Strain Shear Modulus. The *"Acquicella"* site along the southern coast line of Catania is characterized by fine sands with thin limestones.

3 DYNAMIC BEHAVIOUR OF SOIL DURING LABORATORY TESTS

The experimental results of specimens (Fig. 6a) from uncohesive soil, such as *"Plaja beach"* site, were used to determine the empirical parameters of the equation proposed by Yokota et al. (1981) to describe the shear modulus decay with shear strain level:

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

The values of $\alpha = 9$ and $\beta = 0.815$ were obtained for uncohesive soil.

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 reported in Fig. 6b for uncohesive soil:

$$D(\gamma)(\%) = \eta \cdot \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \tag{2}$$

in which: $D(\gamma)$ = strain dependent damping ratio; γ = shear strain; η, λ = soil constants.
The values of $\eta = 80$ and $\lambda = 4$ were obtained for the "Plaja beach" area.

4 SITE RESPONSE ANALYSIS

Local site response analyses have been brought for the *"la Plaja"* beach by 1-D linear equivalent computer codes. The Seismic Dilatometer Marchetti Tests (SDMTs) were performed up to a maximum depth of 32 meters (Figg. 3-5).

The results show a very detailed and stable shear waves profile. The S-wave propagation obtained by SDMT occur on a 1-D column having shear behaviour.

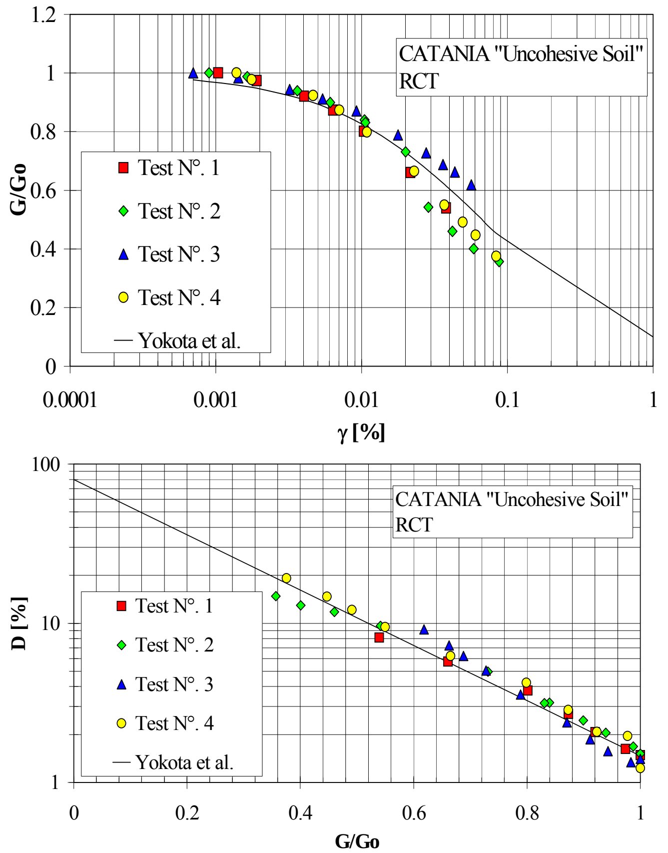

Fig. 6. a) G/G_o- γ curves from RCT for "Plaja beach" site; b) D-G/G_o curves from RCT for "Plaja beach" site.

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. The five 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 corresponding to a return period of 475 years in the current Italian seismic code "seismic hazard and seismic classification criteria for the national territory" obtained by a probabilistic approach in the interactive seismic hazard maps. Further analyses have been performed using scaled seismograms, to the maximum PGA of 0.275g (corresponding to the return period of 975 years) and to the maximum PGA of 0.400g (corresponding to the return period of 2475 years). The analysis provides the time-history response in terms of displacements, velocity and acceleration at the surface. 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 non linear behaviour of soil that often occur (Maugeri et al., 2011a), especially in 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 (Cavallaro et al., 2008a; 2012a). Figs. 7-9 show the results respectively in terms of maximum accelerations with depth, in terms of time history of maximum acceleration at the surface and in terms of response spectra for SDMTs No. 1-5, for the 475 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 (Maugeri et al., 2011b, Monaco et al., 2011b, Cavallaro et al., 2008b). Probably this fact is due to a non linear behaviour of soil, especially in presence of the strong accelerations of the 975 and 2475 earthquake scenarios. Table 1 reports stratigraphic soil amplification factors Ss obtained through 1-D analysis. The results of the site response analyses have been also 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.

Table 1. Stratigraphic soil amplification factors Ss obtained through 1-D code.

SDMT	Ss (475)	Ss (975)	Ss (2475)
1	1.97	1.80	1.45
2	2.12	1.82	1.42
3	2.27	1.98	1.50
4	2.79	2.84	2.48
5	2.42	2.50	2.00

5 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 seismo-geological effects among which the liquefaction-induced features.

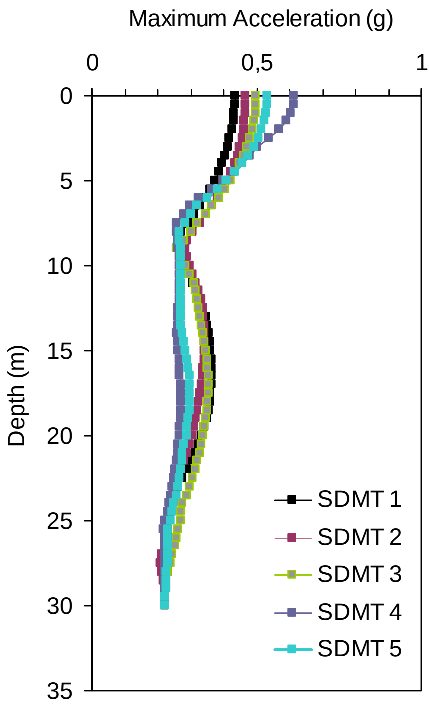

Fig. 7. Maximum accelerations with depth for SDMTs No. 1-5 (475 years earthquake scenario return period).

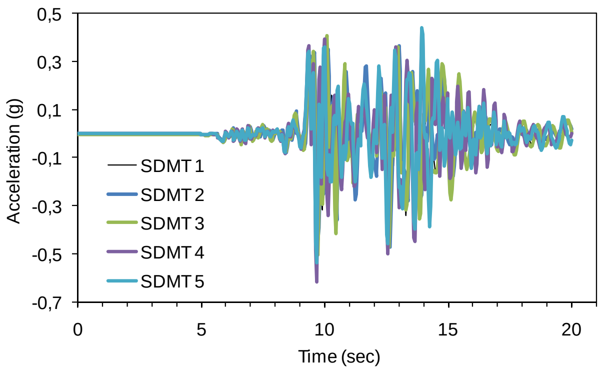

Fig. 8. Maximum accelerations at the surface for SDMTs No. 1-5 (475 years earthquake scenario return period).

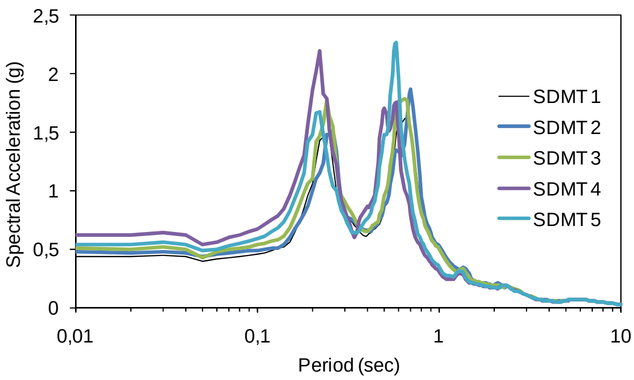

Fig. 9. Response spectra i.e. for SDMTs 1-5 (475 years earthquake scenario return period).

Often during strong earthquakes, effects of liquefaction phenomena are visible also far from the epicentral area (Monaco et al., 2011a). 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 (Grasso et al., 2006; Grasso and Maugeri, 2008).

The susceptibility of a site to seismic-induced liquefaction may be assessed 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 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 by the following equation:

$$CSR = \tau_{av} / \sigma'_{vo} = 0.65\,(a_{max} / g)\,(\sigma_{vo} / \sigma'_{vo})\,r_d / MSF \qquad (3)$$

where τ_{av} = average cyclic shear stress, a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake, g = acceleration of gravity, σ_{vo} and σ'_{vo} = total and effective overburden stresses, r_d = stress reduction coefficient depending on depth and MSF is magnitude scaling factor.

As regards the peak horizontal acceleration, the value of 0.45 g 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 magnitude M (M=7.3 of the 1693 scenario earthquake) to an equivalent CSR for an earthquake magnitude, M = 7½.

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

$$CRR = 0.0107\,K_D^3 - 0.0741\,K_D^2 + 0.2169\,K_D - 0.1306 \qquad (4)$$

$$CRR = 0.0242\,e^{(0.6534K_D)} \qquad (5)$$

$$CRR = 0.0084\,K_D^{2.7032} \qquad (6)$$

Equation (4) has been developed by Monaco et al. (2005); equations (5) and (6) have been developed by Grasso and Maugeri (2008). Fig. 10 shows CRR- K_D trends i.e. for SDMT1.

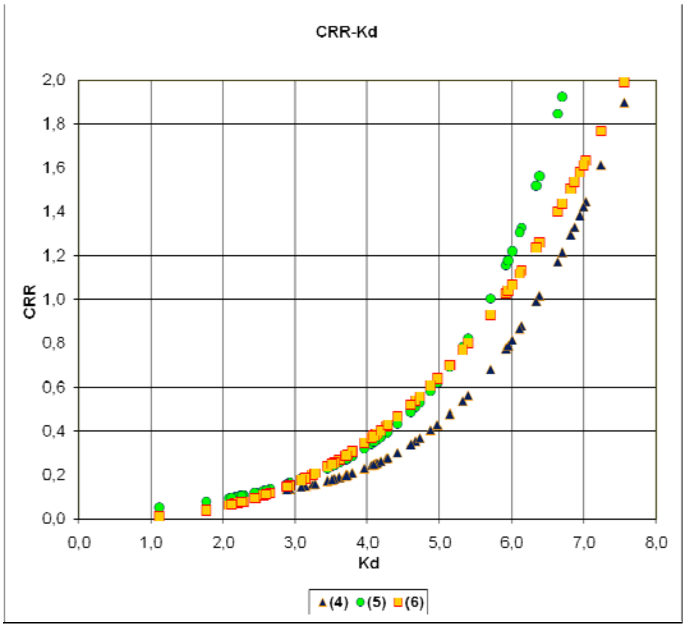

Fig. 10. CRR- K_D trends obtained used K_D values from SDMT1.

The use of the shear wave velocity, V_S, as an index of liquefaction resistance has been illustrated by several authors (Andrus and Stokoe, 2000; Andrus et al. (2004). 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 & Stokoe:

$$CRR = a\left(\frac{V_{S1}}{100}\right)^2 + b\left(\frac{1}{(V^*_{S1} - V_{S1})} - \frac{1}{V^*_{S1}}\right) \quad (7)$$

$$CRR=\left[0.022\left(\frac{K_{a1}V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{V^*_{S1} - (K_{a1}V_{S1})} - \frac{1}{V^*_{S1}}\right)\right]K_{a2} \quad (8)$$

where: V^*_{s1} = limiting upper value of V_{s1} for liquefaction occurrence; $V_{S1} = V_S\,(p_a/\sigma'_{vo})^{0.25}$ is corrected shear wave velocity for overburden-stress; a and b of equation (5) are curve fitting parameters, while K_{a1} and K_{a2} are aging factors = 1.0 for uncemented soils of Holocene age. The correlations given by equations (4), (5), (6), (7) and (8) have been then used for the evaluation of liquefaction potential index, P_L, (Iwasaki et al.. 1978), using the K_D and Vs values measured by SDMT instead. Figg. 11-12 show P_L values obtained respectively from CRR-K_D and CRR-Vs correlations i.e. for SDMT1-2.

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.

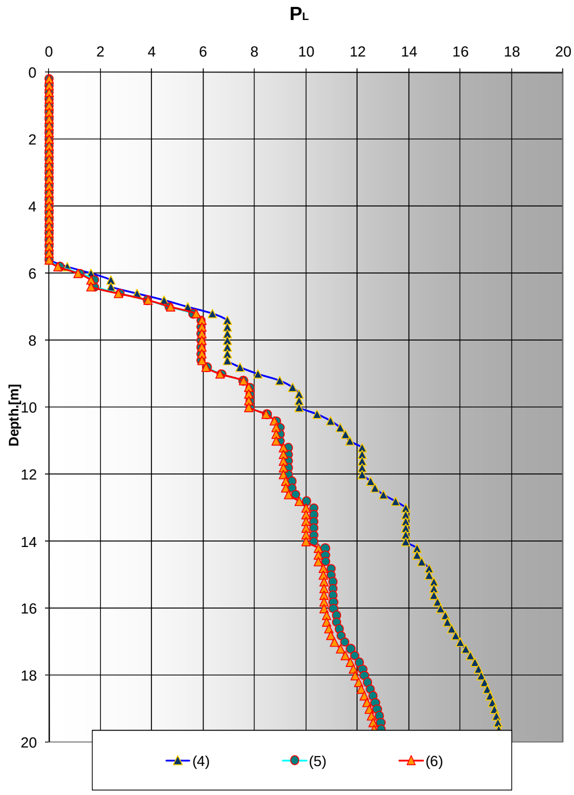

Fig. 11. Liquefaction potential index P_L obtained from CRR- K_D correlations.

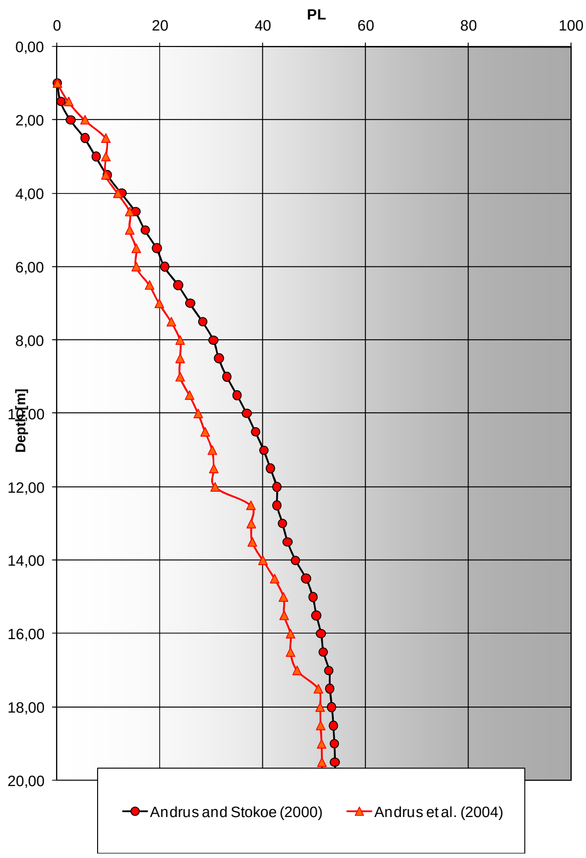

Fig. 12. Liquefaction potential index P_L obtained from CRR- Vs correlations.

Thus, it could be preferable to evaluate liquefaction by K_D measurements which is related to medium-high strains (Cavallaro et al., 2012b; 2012c).

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. Available data enabled one to define the small strain for uncohesive soil and empirical equations to describe the G and D variation with strain level. In addition, vertical drained constrained modulus, angle of shear resistance, horizontal stress index K_D and shear wave velocity V_s profiles have been evaluated by SDMT tests. Local site response analyses have been brought for the "Acquicella Porto" area by 1-D linear equivalent computer codes for the evaluation of the amplification factors of the maximum acceleration. 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 Seismic Code. CRR-K_D correlations obtained for the *"Acquicella Porto"* zone in the Catania harbour have been used for the evaluation of liquefaction potential index, P_L. The results obtained by the SDMT1 show that the Liquefaction Potential Index P_L is below 5 (low risk) up to a depth of about 7 meters; while the results obtained by SDMT2 show low risk up to a depth of 10 m. By the way it is unlikely to have liquefaction at a depth greater than 7-10 m.

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