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"DYNAMIC CHARACTERISATION OF A TEST SITE IN MESSINA (ITALY)"

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ABSTRACT

A dynamic characterisation for the test site of Regional Civil Defence Department in the city of Messina, Italy, has been carried out with the aim of forecasting the distribution of seismic geotechnical hazard within the city. These results can be used by the Civil Defence Department and by other services for disaster planning management. On December 28, 1908 at 5:20 a.m. local time, a devastating earthquake occurred along the Strait of Messina between eastern tip of Sicily and the western tip of Calabria in the south of Italy. The Messina Earthquake, also known as the Messina-Reggio Calabria Earthquake, caused severe ground shaking throughout the regions and triggered also a local tsunami. The dataset clearly points out the vulnerability of the physical environment to the occurrence of 1908 Messina like-earthquakes (and associated tsunamis). The identification and characterization of the most vulnerable sites nowadays exposed to the occurrence of earthquake environmental effects is essential in the seismic risk assessment of the Messina area. Within this aim field and laboratory experimental data allowed the definition of a geotechnical model of the subsoil in the city of Messina. For the site characterisation of soils 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, also with the aim to evaluate the soil profile of shear wave velocity (Vs). Undisturbed samples were retrieved from boreholes for static and dynamic characterisation through laboratory tests.

1. INTRODUCTION

In order to study the dynamic characteristics of soils in a test site in the city of Messina (Italy), laboratory and in situ investigations have been carried out to obtain soil profiles with special attention to the variation of the shear modulus G and damping ratio D with depth. Seismic dilatometer tests (SDMT) have been also carried out in the zone of the "Civil Defence Department" (DPC), with the aim of an accurate geotechnical characterisation [Castelli and Lentini, 2013; Castelli et al., 2016e; 2017], including the evaluation of the shear wave velocity Vs profile, as well as the profile of the horizontal stress index KD. Moreover, the following investigations in the laboratory were carried out on undisturbed samples: direct shear tests, triaxial tests, cyclic loading torsional shear tests (CLTST) and resonant column tests (RCT). This paper tries to summarize the geotechnical information in a comprehensive way in order to provide a case record of site characterization for the attenuation of seismic risk and for the evaluation of site response analysis [Castelli et al., 2008a, 2008b, 2016d, 2016f; Cavallaro, 2006b, 2008a, 2008b] within structural improvement of buildings. Similar geotechnical studies were also successful performed for significant other historical test sites [Castelli et al., 2016a; Cavallaro et al., 1999b, 2003, 2004a, 2004b, 2013a].

2. GEOLOGY AND SEISMICITY OF THE AREA

Due to the residual subduction process (from the Messina Strait to the Lamezia plain), superficial crustal structures are subjected to a regime of detachment stress due to the slow trench retreat, to the deformation of the surface portion of the slab and to the concurrent low-coupling regime between the Ionian lithosphere and the Tyrrhenian coast lithosphere. This scheme of detachment stress is at the origin of violent earthquakes in normal fault mechanism of the Arco Calabro-Peloritano [Pappalardo et al., 2016], especially with the earthquakes of February 5 and 7, 1783 (Gioia Tauro plain and Mesima Valley) and with the December 28, 1908 earthquake (Strait of Messina) [Pino et al., this issue].

On the southern edge, in the North-eastern part of the Sicily-Messina Strait, there is a transition between the residual subduction process just described (located in the Southern part of the Calabria Region) and the collisional Europe-Africa process established in the western part of the North-Eastern Sicily (Figure 1).

The Strait of Messina lies between the Calabrian region of southern Italy and Sicily, forming a part of the

Calabrian Arc, where the African Plate is thrust beneath Calabria, Sicily, and the Tyrrhenian Sea. The

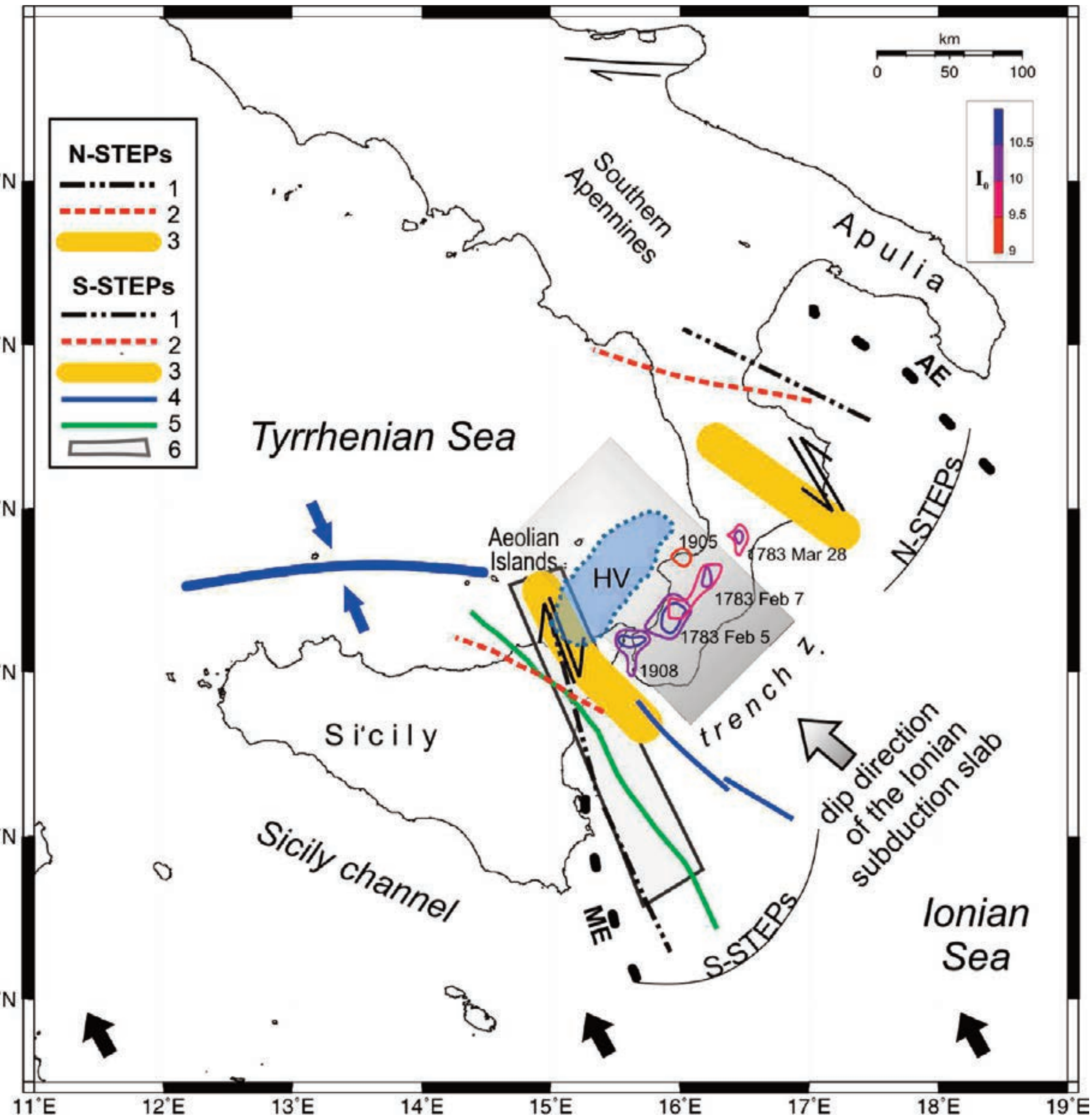

FIGURE 1. Structural sketch of Calabrian Arc Region with localization of historical earthquakes and of STEP fault systems, after Pino et al. this issue.

Strait is bounded on either side by a series of normal faults which approximately follow the north-south and the northeast-southwest trending Sicilian and Calabrian coastlines. Normal faulting is the dominant type of faulting in the region around the Strait, with many of the large historical earthquakes associated with a normal faulting mechanism.

The December 28, 1908, Southern Calabria - Messina earthquake (Intensity MCS XI, Mw 7.24) was the strongest seismic event of the 20th century in Italy with the most ruinous in terms of casualties (at least 80,000). Its epicenter was located at sea in the southern part of the Messina Strait. Few minutes after the earthquake both sides of the strait were inundated by several tsunami waves, worsening the disastrous effects of the earthquake. The damages were particularly catastrophic along the Calabrian coast, between south of Reggio Calabria and south-west of Scilla, and along the eastern coast of Sicily from its easternmost tip to south of Messina. Messina and Reggio Calabria were almost totally destroyed.

The region around the Strait of Messina has experienced some of Italy's most destructive earthquakes. The January 9 and 11, 1693 earthquakes were centered in southeastern Sicily, devastating Catania, Noto, Ragusa, Siracusa, and other towns and resulting in at least 60,000 fatalities. The February 5–March 28, 1783 earthquake sequence in Calabria, with up to 50,000 casualties, was an event that caused severe damage to both Messina and Reggio Calabria. These earthquakes, measuring between M5.7 and M7.0, have been blamed for the poor performance of many older buildings in the worst-affected northern part of Messina during the 1908 earthquake due to inadequate and hasty maintenance (RMS, 2008).

The building of the DRPC in Messina is located in a downtown intensely urbanized area flat weakly inclined eastward (Ionian Sea). This workforce reflects the geological conditions: in fact, it falls in a wide coastal plain characterized by alluvial deposits of Holocene age (Figure 2) with average width around 700 meters and long coastline in direction NNE-SSW [Pino et al., this issue].

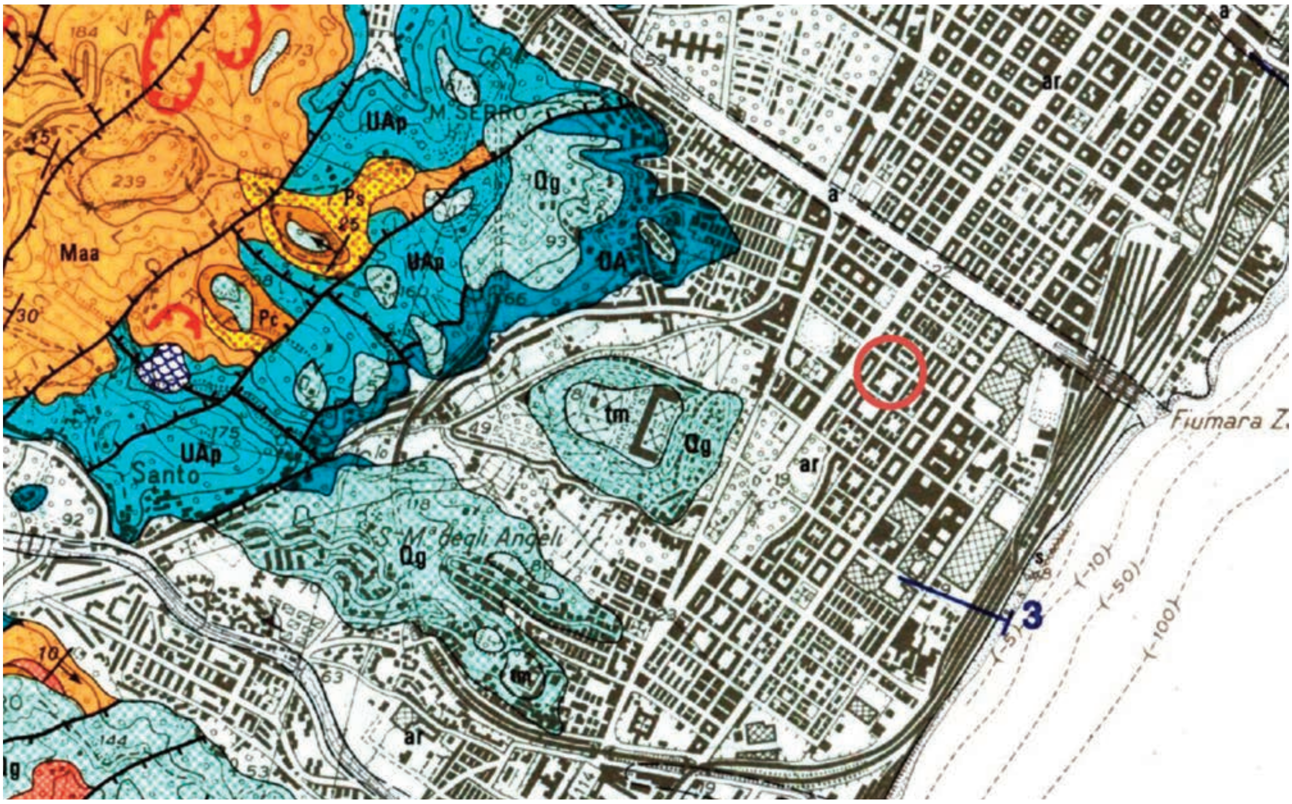

FIGURE 2. Geological sketch of the city of Messina (scale 1:25.000), after Gargano [1994].

Such alluvial formation constitutes the soil at the site of the building. The results of boreholes, driven to the depths of 60 meters (S1 and S2) and 80 meters (S3) and illustrated in Figure 3, show up to a depth of approximately 37 meters the presence of composite weathered soils formed by sandy-silty gravel fractions from prevailing metamorphic quazoso-biotitic, interspaced at around 24 meters depth from one horizon of blackish-brown silt of maximum thickness of 6 meters (Figure 4).

FIGURE 3. Layout of the test site with boreholes location in the "Civil Defence Department" (DPC) area.

Layering is horizontal, marked by grain size and color of various types. Within the alluvial formation was found the presence of a water table at a depth of about 22 meters.

3. SITE CHARACTERIZATION PROGRAMME AND BASIC GEOTECHNICAL SOIL PROPERTIES

The static and dynamic study of soils in the city of Messina was performed within one investigation test site: the "Civil Defence Department" (DPC) in the eastern area of the city. The investigation study reached a maximum depth of 80 m. Laboratory tests have been performed on undisturbed samples retrieved by means of a 101 mm tube sampler [Cavallaro et al., 2007].

To evaluate the geotechnical characteristics, the following in situ and laboratory tests were performed in the soil located in the DPC area of Messina: n. 3 Boreholes (Figure 3), n. 3 Standard Penetration test (SPT), n. 1 Down-Hole tests (DHT), n. 1 Cross-Hole tests (CHT), n. 2 Seismic Dilatometer Marchetti Tests (SDMT); n. 6 Direct Shear Tests (DST), n. 2 Consolidated Undrained Triaxial Tests (CUTXT), n. 4 Cyclic Loading Torsional Shear Tests (CLTST), n. 4 Resonant Column Test (RCT).

The "Civil Defence Department" (DPC) area mainly consists:

S1: a paving slab of basalt with concrete base (to 0.00 - 0.40 m depth), a layer of backfill (to 0.40 - 1.80 m depth), a layer of brown to green - grey sandy silty sand and gravels with included centimeter limestone with different rounding and densification degree (to 1.80 - 11.70 m depth), a layer of black - brown slightly clayey sandy silt (to 11.70 - 12.80 m depth), a layer of brown to grey slightly sand and gravels with included centimeter limestone with different rounding and densification degree (to 12.80 - 17.80 m depth), a layer of black - brown slightly clayey sandy silt (to 17.80 - 20.50 m depth), a layer of brown to green - slightly silty sand and gravels with included centimeter lime-

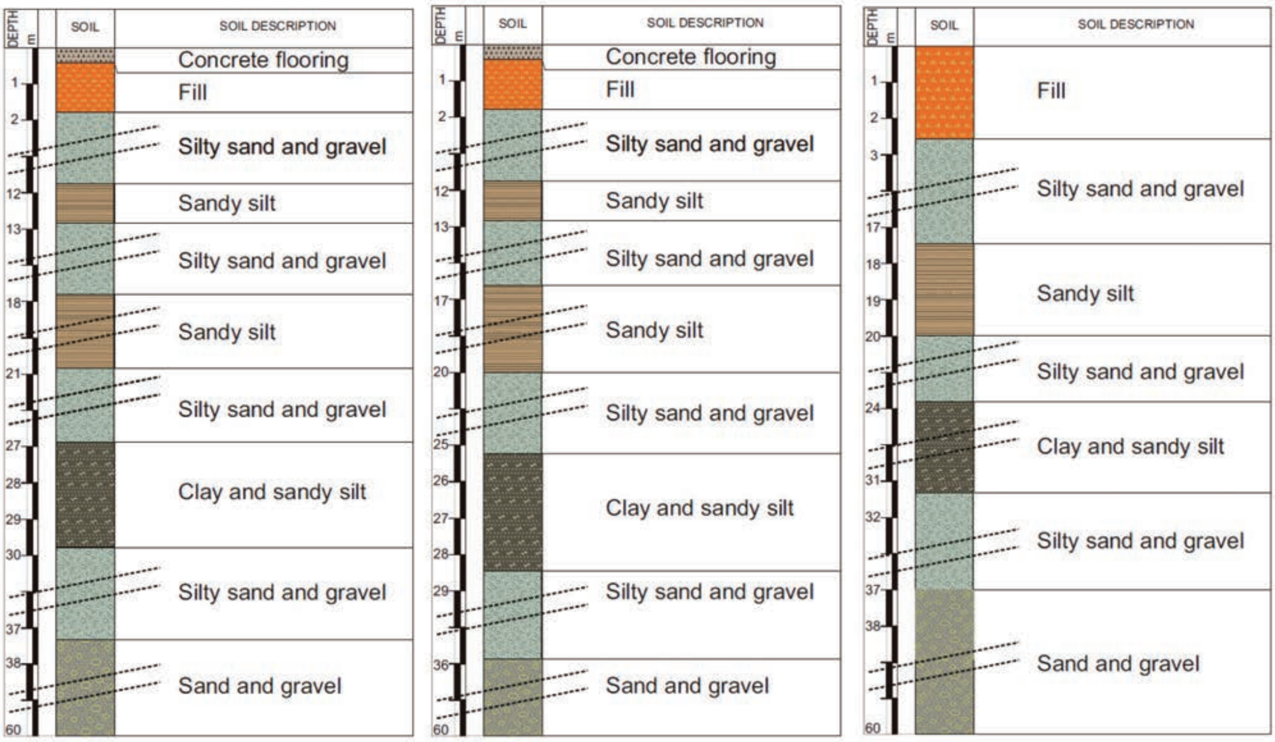

FIGURE 4. Soil profiles of boreholes S1, S2, S3, on DPC.

stone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 20.50 - 22.50 m depth), a layer of grey ash fine sand (to 22.50 - 23.20 m depth), a layer of brown to green - grey slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 23.20 - 26.90 m depth), a layer of black - brown slightly sandy clay and silt (to 26.90 - 29.80 m depth), a layer of brown to green - grey slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 29.80 - 37.30 m depth), a layer of green - grey sand and gravels; sometimes decimeter levels of rounded gravels (to 37.30 - 60.00 m depth).

S2: a paving slab of basalt with concrete base (to 0.00 - 0.40 m depth), a layer of backfill (to 0.40 - 1.80 m depth), a layer of brown to green - grey sandy silty sand and gravels with included centimeter limestone with different rounding and densification degree (to 1.80 - 11.70 m depth), a layer of black - brown slightly clayey sandy silt (to 11.70 - 12.80 m depth), a layer of brown to grey slightly sand and gravels with included centimeter limestone with different rounding and densification degree (to 12.80 - 17.30 m depth), a layer of black - brown slightly clayey sandy silt (to 17.30 - 20.50 m depth), a layer of brown to green - slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 20.50 - 22.00 m depth), a layer of grey ash fine sand (to 22.00 - 22.50 m depth), a layer of brown to green - grey slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 22.50 - 25.20 m depth), a layer of black - brown slightly sandy clay and silt (to 25.20 - 26.60 m depth), a layer of brown to green - grey slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 26.60 - 35.90 m depth), a layer of green - grey sand and gravels; sometimes decimeter levels of rounded gravels (to 35.90 - 60.00 m depth).

S3: a layer of backfill (to 0.00 - 2.70 m depth), a layer of brown to green - grey sandy silty sand and gravels with included centimeter limestone with different rounding and densification degree (to 2.70 - 17.40 m depth), a layer of black - brown slightly clayey sandy silt (to 17.40 - 20.00 m depth), a layer of brown to grey slightly sand and gravels with included centimeter limestone with

Site	H [m]	γ kN/m^3	$W_{n\,(\%)}$	G_s	e	n	S_r (%)	DST				CUT_XT			
								c' (kPa)	φ' (°)	c_r (kPa)	φ_r (°)	c_u (kPa)	φ_u (°)	c' (kPa)	φ' (°)
S1C1	7.00 - 7.40	18.34	11.93	2.79	0.67	0.40	49.95	12	34	-	-	-	-		-
S2C1	3.00 - 3.50	18.63	18.90	2.75	0.72	0.42	72.03	15	32	-	-	-	-		-
S2C2	6.00 - 6.40	19.81	13.19	2.74	0.54	0.35	67.15	30	41	-	-	-	-		-
S3C1	5.50 - 6.00	19.12	15.08	2.79	0.65	0.39	64.87	16	23	-	-	96	17	85	22
S3C2	14.60 - 15.00	19.91	212.51	2.74	0.52	0.34	65.57	9	33	-	-	-	-	-	-
S3C3	21.00 - 21.40	16.18	33.61	-	-	-	-	-	-	-	-	-	-		-
S3C4	27.00 - 27.50	17.95	39.78	2.76	1.11	0.53	99.11	15	21	0	11	82	14	38	20

TABLE 1. Mechanical characteristics for DPC of Messina area.Where: c' = Cohesion and φ' = Angle of Shear Resistance; Direct Shear Test (DST), CU Triaxial Tests (CUT_XT).

different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 20.00 - 20.70 m depth), a layer of grey ash fine sand (to 20.70 - 21.80 m depth), a layer of brown to green - grey slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 21.80 - 23.80 m depth), a layer of brown to green - slightly sandy clay and silt (to 23.80 - 31.30 m depth),), a layer of brown to green - grey slightly silty sand and gravels with included centimeter limestone with different rounding and densification degree; sometimes centimeter levels of grey ash fine sand are present (to 31.30 - 37.10 m depth), a layer of green - grey sand and gravels; sometimes decimeter levels of rounded gravels (to 37.10 - 60.00 m depth).

Using information made available from in situ boreholes, soil profiles of the Messina test site could be designed. On the basis of statistical analysis of boreholes results it is possible to define four soil classes, namely: basaltic rock, gravels, sand, silt and clay (Figure 4).

Based on the laboratory tests typical range of physical characteristics, index properties and strength parameters of the deposits mainly encountered in these areas are reported in Table 1.

On the basis of SDMT the DPC deposits mainly consist of slightly over-consolidated slightly clayey sandy silt and sand and gravels soil. A good agreement between the material index I_d with the soil profile description of the borings is shown in the following Figure 12 based on the values of the parameters proposed by Marchetti [1980]. In the same Figure 12, the proposed K_d parameter for the evaluation of OCR should apply only to the deep clay layer, because in sandy soils the correlation between K_d and OCR is site-dependent and requires further verifications [Schmertmann, 1983; Choo et al., 2015].

The value of the natural moisture content w_n prevalently ranges from between 12 - 40 %. Characteristic values for G_s (specific gravity) ranged between 2.74 and 2.79. Figure 5 shows index properties of DPC area. The water level is of 22.00 m from the ground surface. Concerning the strength parameters of the deposit mainly encountered in this area (Table 1), c' from DST ranged between 9 kPa and 30 kPa while φ' from DST ranged between 21° and 41°.

4. SHEAR MODULUS BY IN SITU TESTS

Seismic tests are conventionally classified into borehole and surface methods. These methods enable one to determine the velocity of body waves [compressional (P) and/or shear (S)] and surface waves [Rayleigh] respectively which induce very small strain levels into soil, i.e. $\varepsilon_{ij} < 0.001$ % [Woods, 1978]. It is possible, on the basis of

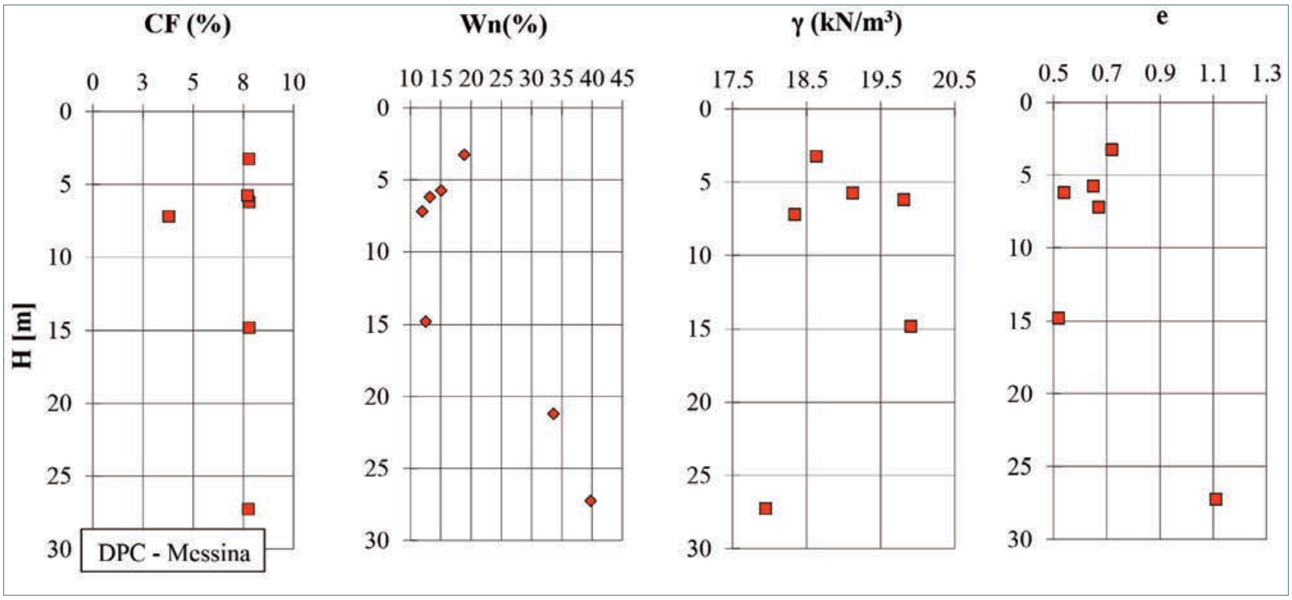

FIGURE 5. Index properties of DPC area.

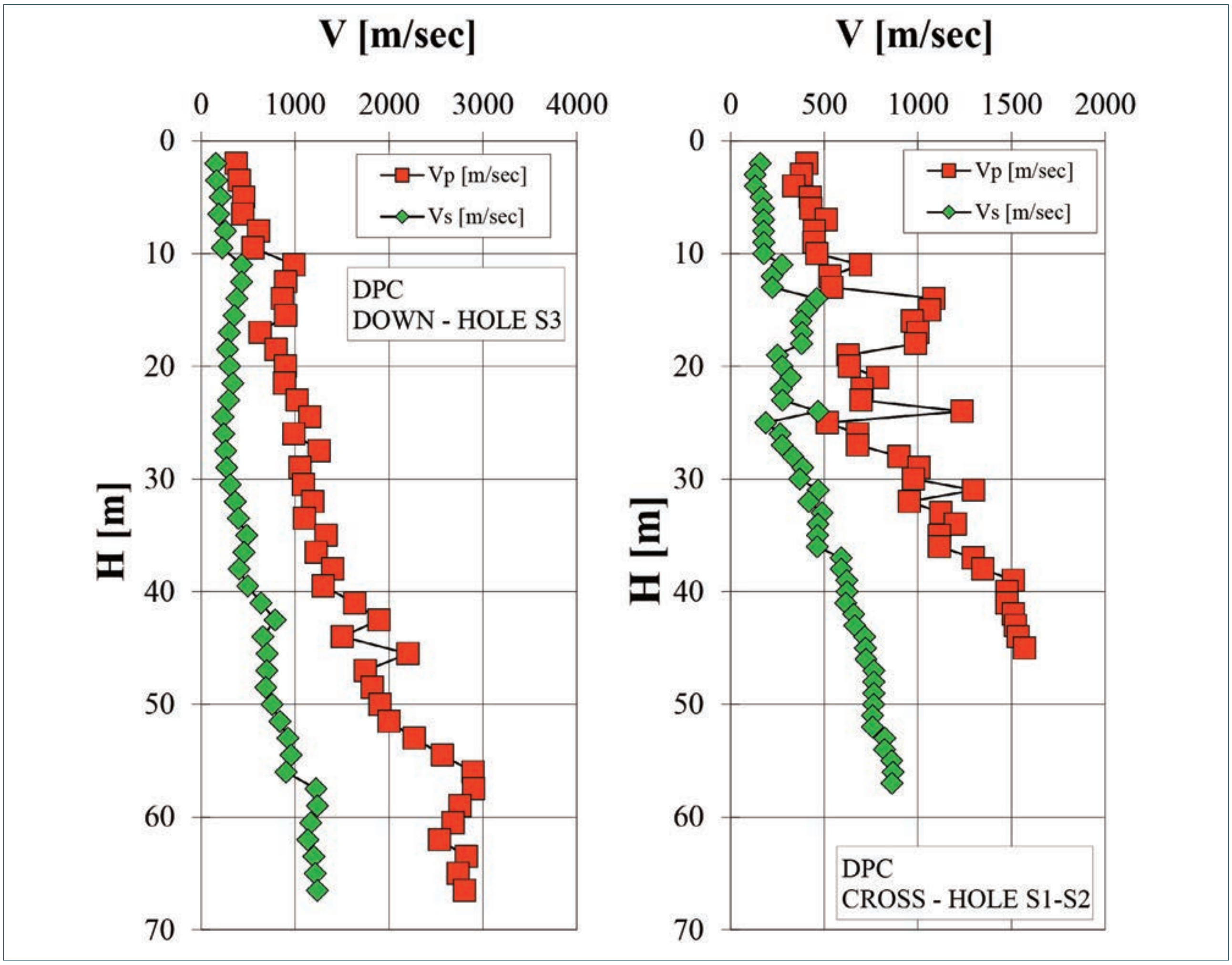

FIGURE 6. V_s and V_p from Down Hole and Cross Hole tests in DPC area.

the measured wave velocities, to obtain the small strain deformation characteristics according to the well known relationships of the linear elasticity theory.

Among the various borehole seismic methods, the

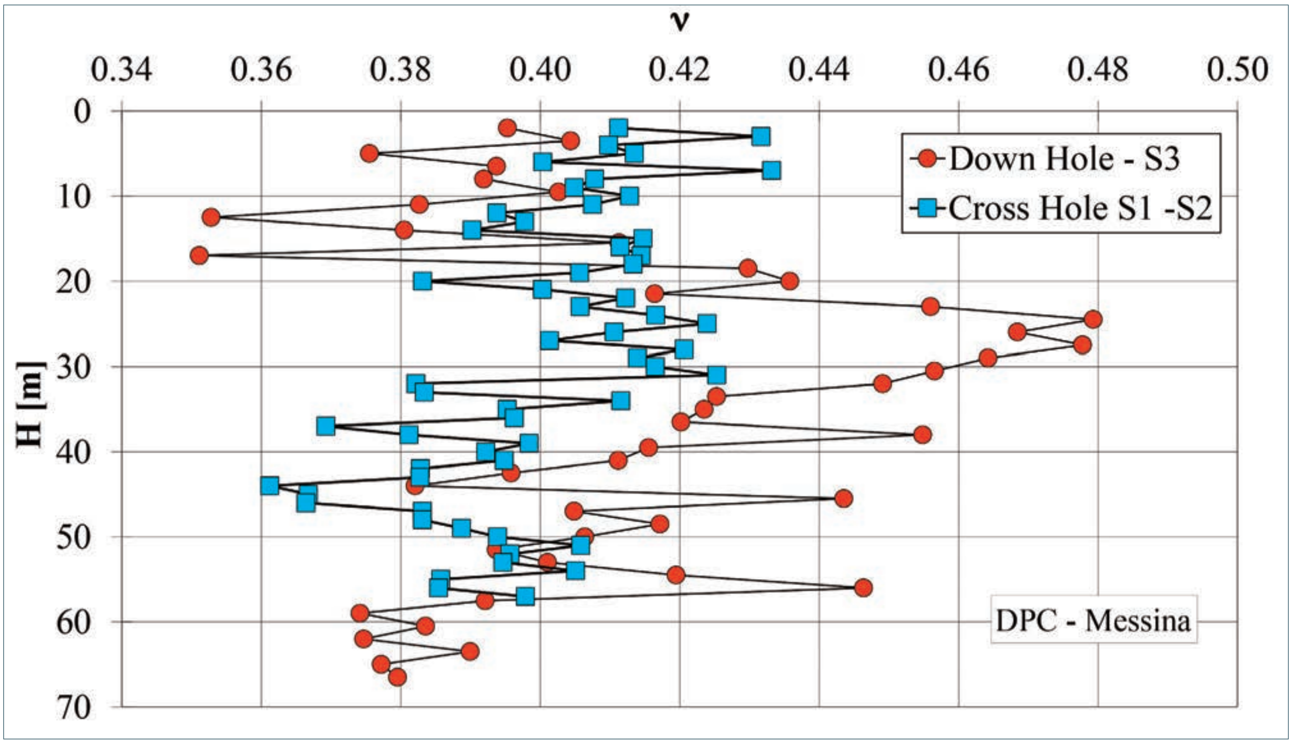

FIGURE 7. Poisson ratio from Down Hole and Cross Hole tests.

well known Down Hole (DH) and Cross Hole (CH) tests were performed in this study. Current practice and recent innovations of borehole methods for seismic exploration are covered by many comprehensive works [Auld, 1977; Stokoe and Hoar, 1978; Woods, 1978; Woods and Stokoe, 1985; Woods, 1991 and 1994; Jamiolkowski et al., 1995].

In Figure 6, the shear and compression wave velocities are reported against depth. In Figure 7 the dynamic Poisson ratio variation with depth, obtained from Down Hole (DH) and Cross Hole (CH) tests, is plotted to show site characteristics. It is shown that the values oscillate around 0.35 - 0.48.

In Figure 8 the coefficient of earth pressure at rest Ko variation with depth, obtained from Down Hole (DH) and Cross Hole (CH) tests, is plotted to show site characteristics. It is shown that after the top of about 6 m, the values oscillates around 0.55 - 0.92.

A comparison between G_o values obtained from in situ tests performed on the area under consideration is shown in Figure 9. The Down Hole and Cross Hole tests performed in DPC area show G_o values increasing with depth. Very high values of G_o are obtained for depths greater than 60 m. According to these data, it is possible to assume G_o values oscillate around 47 - 1500 MPa for depths smaller than 60 m.

The seismic dilatometer (SDMT) is the combination of the standard DMT equipment with a seismic module for measuring the shear wave velocity Vs. Initially conceived for research, the SDMT is gradually entering into use in current site investigation practice. The SDMT [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 intermediate level of strains in natural soil deposits [Castelli and Maugeri, 2008; Castelli and Lentini, 2016; Castelli et al., 2016b; Cavallaro 1999c; Cavallaro et al., 2012, 2015]. This apparatus was also used in offshore condition by Cavallaro et al. [2013b, 2013c].

The test is conceptually similar to the seismic cone (SCPT). First introduced by Hepton [1988], the SDMT was subsequently improved at Georgia Tech, Atlanta, USA [Martin and Mayne, 1997, 1998; Mayne et al., 1999].

The seismic modulus is a cylindrical instrumented tube, located above the DMT blade [Marchetti, 1980], housing two receivers at a distance of 0.50 m (see Figure 10). 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).

Moreover the pair of seismograms recorded by the two receivers at a given test depth corresponds to the same hammer blow and not to different blows in sequence, which are not automatically identical. The adoption of the “true interval” configuration considerably enhances the repeatability in the Vs measurement (observed repeatability $V_s \approx 1$ - 2 %). V_s is obtained as the ratio between

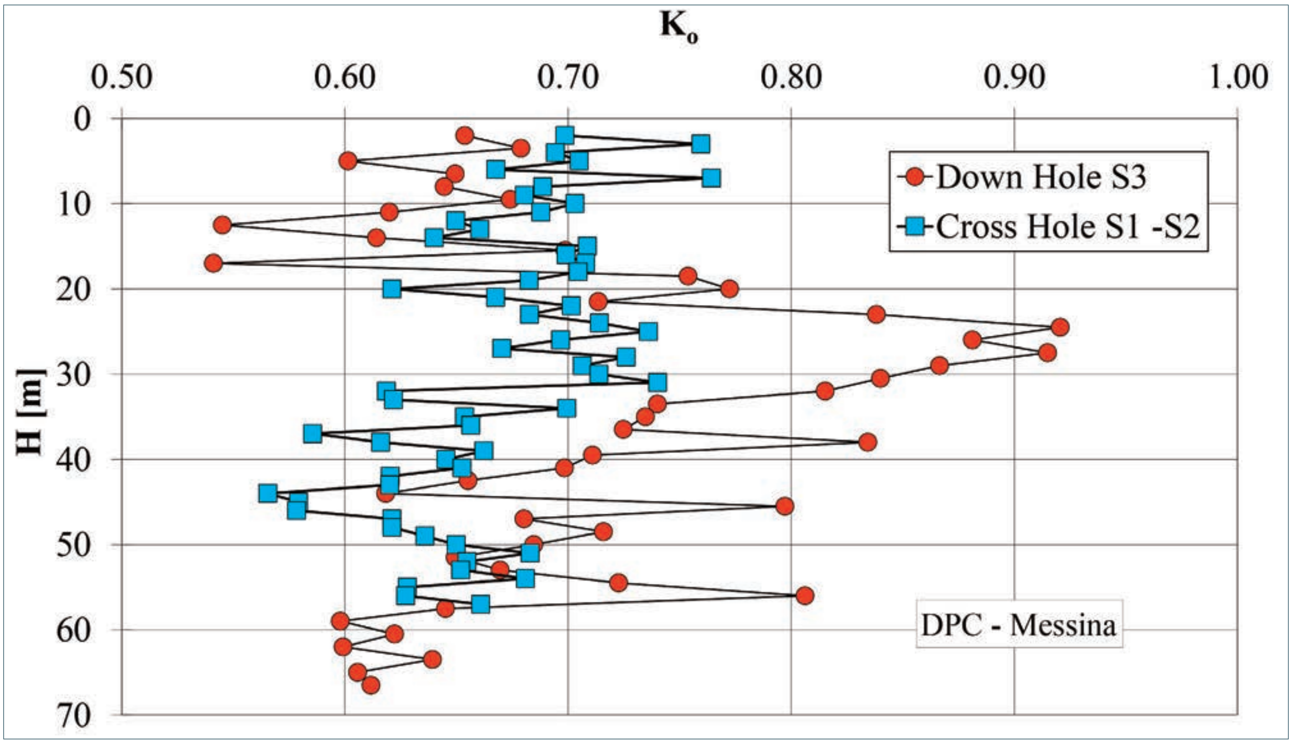

FIGURE 8. The coefficient of earth pressure at rest K_o from Down Hole and Cross Hole tests.

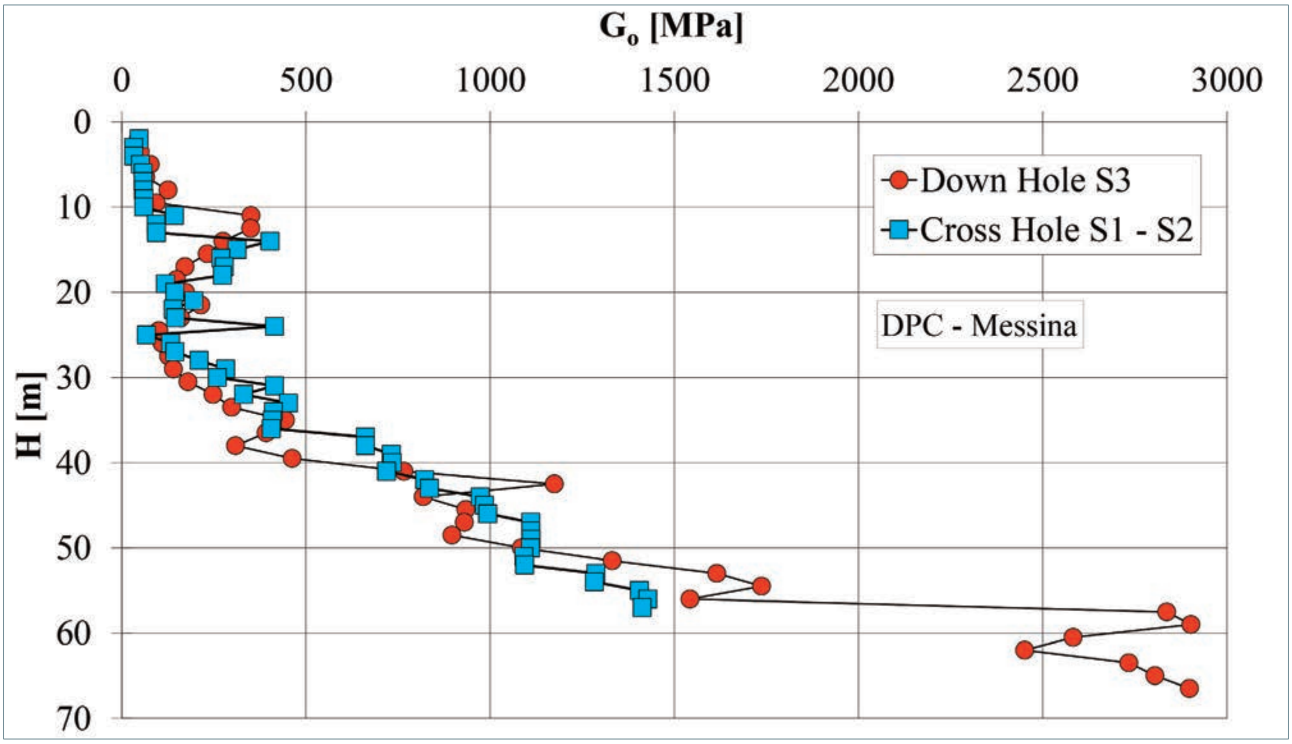

FIGURE 9. G_o from Down Hole and Cross Hole tests.

the difference in distance between the source and the two receivers (S2 - S1) and the delay of the arrival of the impulse from the first to the second receiver (Δt). Vs measurements are obtained every 0.5 m of depth.

The shear wave source at the surface is a pendulum hammer (≈ 10 kg) which hits horizontally a steel rectangular base pressed vertically against the soil (by the weight of the truck) and oriented with its long axis parallel to the axis of the receivers, so that they can offer the highest sensitivity to the generated shear wave.

In Figure 10 it is shown the SDMT scheme for the measure of V_s while Figure 11 shows an example of seismograms obtained by SDMT at various test depths at the site of DPC area (it is a good practice to plot side by-side the seismograms as recorded and re-phased according to the calculated delay). The determination of the delay from SDMT seismograms, normally carried out using the cross-correlation algorithm, is generally well conditioned, being based on the two seismograms - in particular the initial wavelets - rather than being based on the first arrival time or specific marker points in the seismogram (Figure 11).

V_s may be converted into the initial shear modulus Go by the theory of elasticity by the well-known relationships:

$$G_0 = \rho V_s^2 \tag{1}$$

where: ρ = mass density.

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., 2006a; Castelli et al. 2016c].

A summary of SDMT parameters are shown in Figure 12 where:

- I_d: Material Index; gives information on soil type (sand, silt, clay); clay for $I_d > 0.6$, silt for $1.2 \geq I_d > 0.6$, sandy silt for $1.8 \geq I_d > 1.2$ and sand for $I_d > 1.8$;
- M: Vertical Drained Constrained Modulus;
- Cu: Undrained Shear Strength;
- φ': Angle of Shear Resistance;
- K_d: Horizontal Stress Index; the profile of Kd is similar in shape to the profile of the overconsolidation ratio OCR. Kd = 2 indicates in clays OCR = 1, $K_d > 2$ indicates overconsolidation.

A first glance at the K_d profile is helpful to "understand" the deposit;

- Vs: Shear Waves Velocity (Figure 13).

Moreover SDMT allows the evaluation of the dependence of the soil stiffness decay curve (G/G_{max}) on the

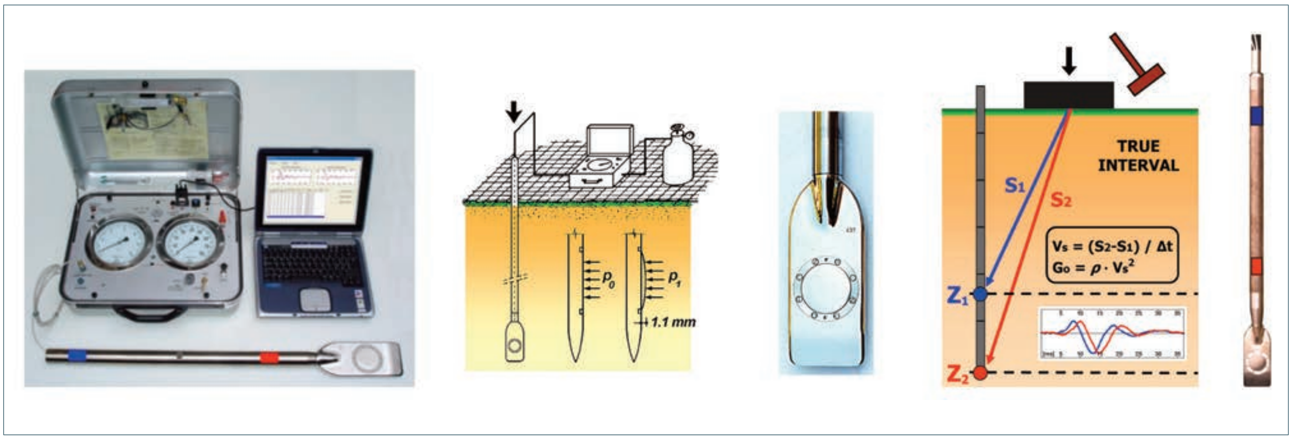

FIGURE 10. Seismic dilatometer equipment (a). Schematic layout of the flat dilatometer test (b) and of the seismic dilatometer test (c).

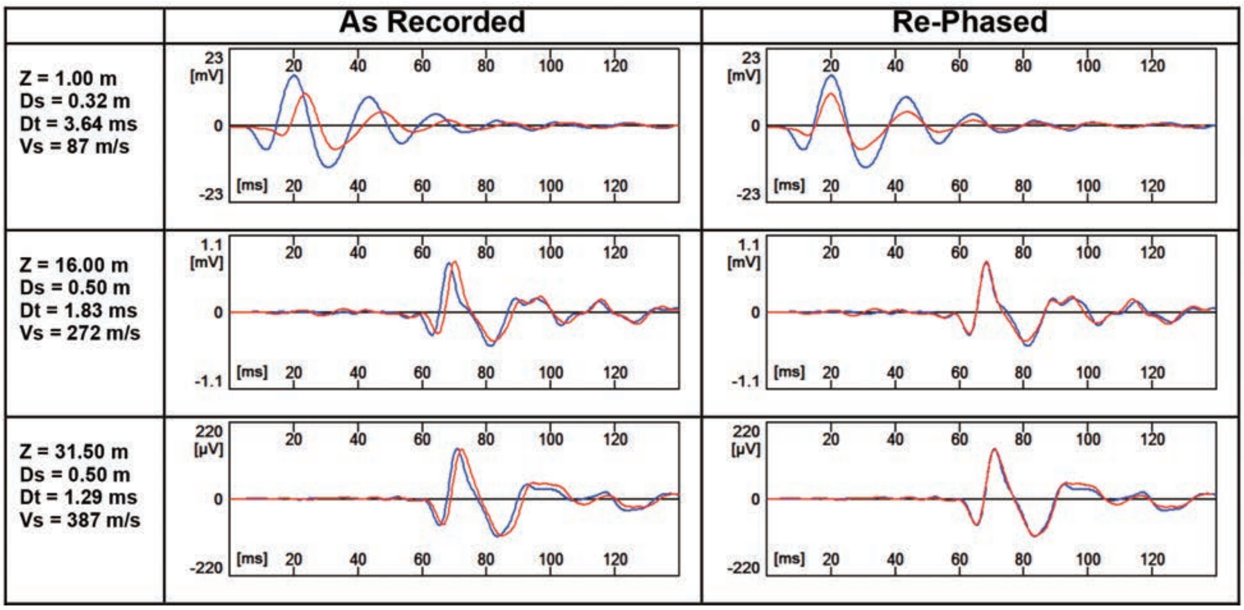

FIGURE 11. Example of seismograms obtained by SDMT at the site of DPC.

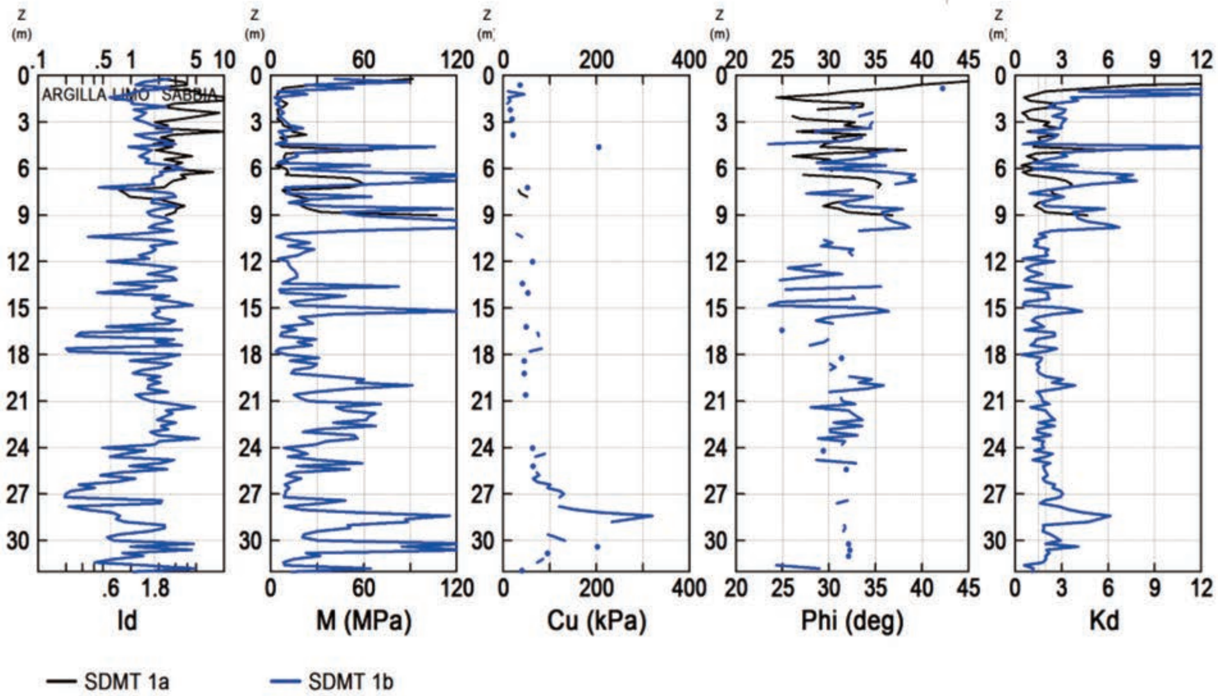

FIGURE 12. Results of the SDMTs in terms of geotechnical parameters for DPC area.

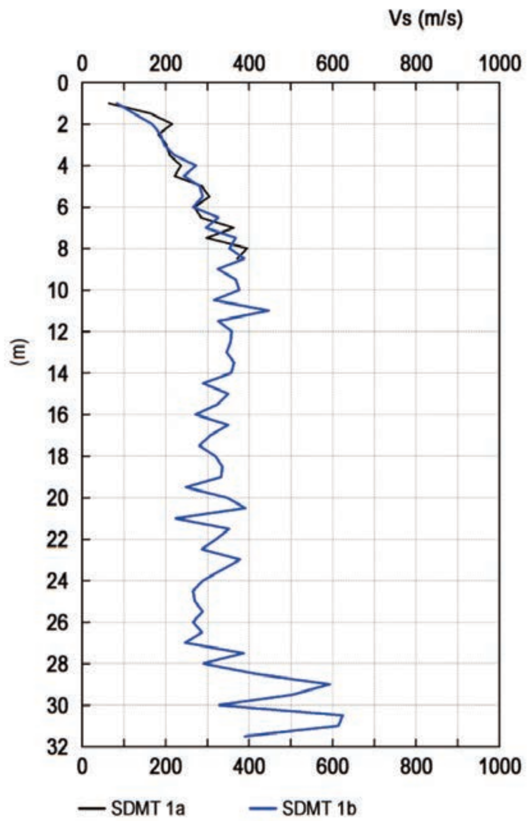

FIGURE 13. Shear wave velocity profiles obtained from DPC area.

shear strain level [Amoroso et al., 2014]. Following the approach suggested by Amoroso et al., values of the working strain shear modulus G_{DMT} can be derived by the values of the constrained modulus M reported in Figure 12 provided by the DMT tests. The intersection of G_{DMT}/G_0 horizontal ordinate line with the stiffness decay curve reported in Figure 18 provides a shear strain value referred as γ_{DMT} which can be compared with the results given by laboratory tests.

It was also possible to evaluate the small strain shear modulus Go in the DPC Messina test site by means of the following empirical correlations available in literature based on laboratory test results, Standard Penetration Tests (SPT) or seismic Marchetti dilatometer tests results.

a) Jamiolkowski et al. [1995]

$$G_o = \frac{600 \cdot \sigma_m^{'0.5} p_a^{0.5}}{e^{1.3}} \tag{2}$$

where: $\sigma'_m = (\sigma'_v + 2\ \sigma'_h)/3$ effective medium stress with σ'v = effective vertical normal stress and σ'_h = effective horizontal normal stress; p_a = 1 bar is a reference pressure; e = void ratio index; G_0, σ'm and pa are expressed in the same unit.

The values for parameters, which appear in Equation (2) are equal to the average values 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.

b) Ohta and Goto [1978]

$$V_s = 69 \cdot N_{60}^{0.17} \cdot Z^{0.2} \cdot F_A \cdot F_G \tag{3}$$

where: V_s = shear wave velocity (m/s), N_{60} = number of blow/feet from SPT with an Energy Ratio of 60%, Z = depth (m), F_G = geological factor (clays=1.000, sands=1.086), F_A = age factor (Holocene = 1.000, Pleistocene = 1.303).

c) Hryciw [1990]

$$G_o = \frac{530}{\left(\sigma_v^{'} / p_a\right)^{0.25}} \frac{\gamma_D / \gamma_w - 1}{2.7 - \gamma_D / \gamma_w} K_o^{0.25} \cdot \left(\sigma_v^{'} \cdot p_a\right)^{0.5} \tag{4}$$

where: G_o, σ'_v and p_a are expressed in the same unit; pa = 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].

Figure 14 shows the values of G_o obtained in situ from a SDMT and those obtained by means of the empirical correlations.

On the whole, with the aim of identifying the reliability of empirical correlations proposed with direct measurements through Down-Hole and SDMT, Equation (2) seems to provide the most accurate trend of G_o with depth, as shown in Figure 14. A good agreement exists between empirical correlations and SDMT. However, the method by Hryciw [1990] was not capable of detecting the SDMT results for sandy soil as shown in Figure 13.

Figure 13 shows also the value of G_o measured in the laboratory from RCT performed on undisturbed solid cylindrical specimens. In the case of laboratory tests, the G_o values are determined at shear strain levels of less than 0.001 %. At the depth of about 30 m high values of G_o are registered by SDMT probably due to the presence of a layer of green - grey sand and gravels. High values of G_o were obtained for levels higher than 40 m depth.

5. SHEAR MODULUS AND DAMPING RATIO BY LABORATORY TESTS

Shear modulus G and damping ratio D of Catania tests sites deposits were obtained in the laboratory from Resonant Column Tests (RCT) and Cyclic Loading Torsional Shear Tests (CLTST) performed by means of a Resonant Column/Cyclic Loading Torsional Shear Apparatus [Cascante et al., 1998]. This apparatus is in use at the Geotechnical and Soil Dynamics Laboratory of the Faculty of Engineering and Architecture, University of Enna "Kore".

The equivalent shear modulus G_{eq} is the unload-reload shear modulus that is evaluated from RCT in function of velocity V_s and density ρ of the sample, while G_o is the maximum value or also "plateau" value as observed in the G-log(γ) plot; G is the secant modulus [Castelli et al., 2016c; Cavallaro, 1999a; 2016; Lo Presti et al., 1999a;

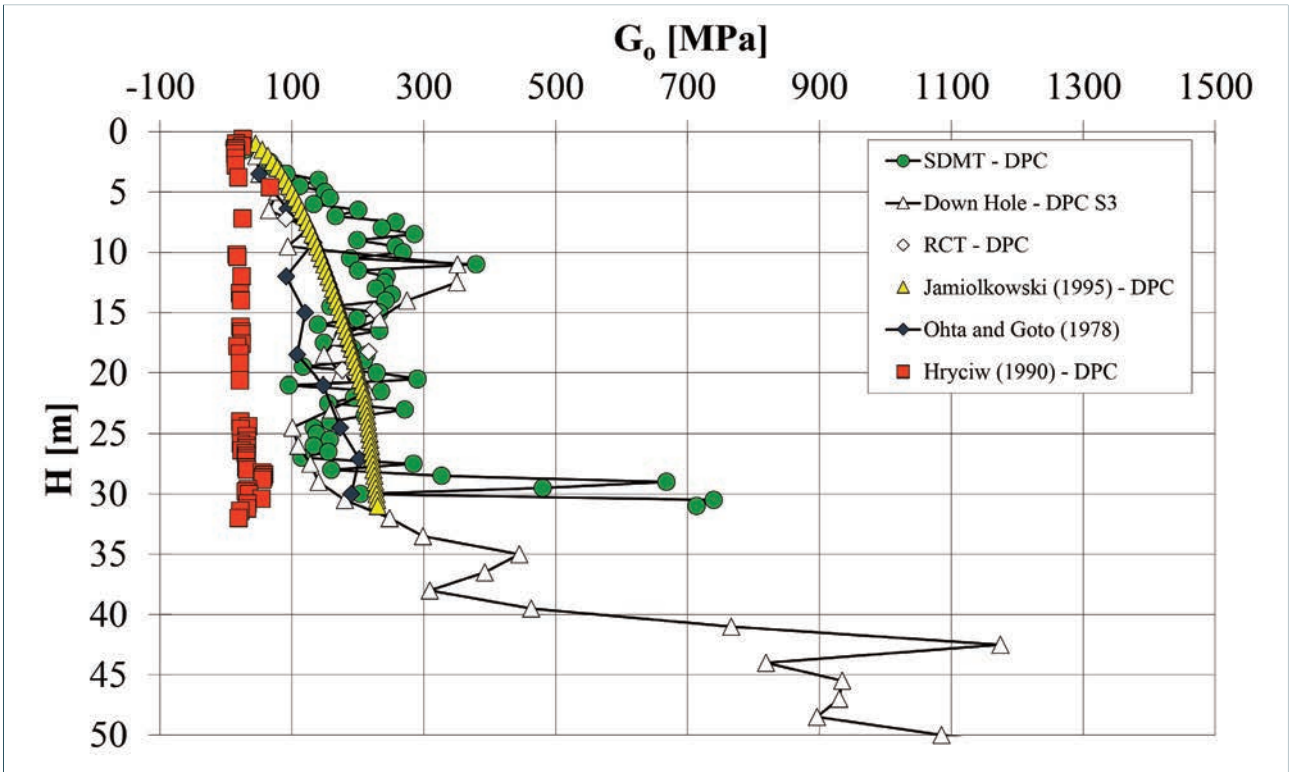

FIGURE 14. G_o from SDMT and by empirical correlations.

Borehole	H (m)	σ'_{vc} (kPa)	e	CLTST RCT	G_o (1) (MPa)	G_o (2) (MPa)	G_o (3) (MPA)	γ_{max}(%)
S2C1	3.25	70	0.72	U	-	50	50	0.483
S3C1	5.75	110	0.65	U	-	64	94	0.618
S1C1	7.20	100	0.67	U	91	-	351	0.210
S3C2	14.80	300	0.52	U	225	-	160	0.212
S2C3	18.25	450	0.52	U	216	164	128	0.133
S1C2	19.75	380	0.67	U	176	140	180	0.197
S3C4	27.25	450	1.11	U	-	127	766	0.171

TABLE 2. Test condition for DPC area specimens. where: U = Undrained. G_o (1) from RCT, G_o (2) from CLTST after 24 hrs, G_o (3) from Down-Hole.

Maugeri and Cavallaro, 1999].

Generally G is constant until a certain strain limit is exceeded. This limit is called elastic threshold shear strain (γ_t^e) and it is believed that soils behave elastically at strains smaller than (γ_t^e). The elastic stiffness at $\gamma < \gamma_t^e$ is thus the already defined G_o [Capilleri et al., 2014; Lo Presti et al., 1999b].

For CLTSTs the damping ratio (D) was calculated as the ratio between the area enclosed by the unloading-reloading loop and represents the total energy loss during the cycle and W is the elastic stored energy. For RCTs the damping ratio was determined using the steady-state method during the resonance condition of the sample.

The laboratory test conditions and the obtained small strain shear modulus G_o are listed in Table 2. The undisturbed specimens were isotropically reconsolidated to the best estimate of the in situ mean effective stress. The same specimen was first subject to RCT, then to CLTST after a rest period of 24 hrs with opened drainage. CLTST were performed under stress control condition by applying a torque, with triangular time history, at a frequency of 0.1 Hz. The size of solid cylindrical specimens are Radius = 25 mm and Height = 100 mm.

The G_o values [G_o (1) and G_o (2)], reported in Table 2, indicate an influence of strain rate even at very small strain where the soil behaviour is supposed to be elastic.

In order to appreciate the rate effect on G_o, it is worthwhile to remember that the equivalent shear strain rate experienced by the specimens during RCT can be three orders of magnitude greater than those adopted during CLTST. The effects of the rate and loading conditions become less relevant with an increase of the shear strain level, as can be seen in Figure 16 where the G-γ curves obtained from CLTST and RCT are compared.

Values of shear modulus G [MPa] and damping ratio D [%] versus γ [%] from RCT and CLTST tests are reported in Figure 15 and 16. The shear modulus obtained during CLTST shows the effect of the soil degradation because of RCT. Finally higher values of the initial shear modulus [G_o (3)] have been obtained from Down - Hole tests.

In general higher values of D are obtained by RCT than CLTST. The damping ratio values obtained from RCT by steady-state method increase with the strain level, while higher values of D have been obtained from strain level higher than 0.02 %. It is possible to observe that the damping ratio from RCT, at very small strains, is equal to about 0.013 % probably for the presence of sandy soil. Greater values of D are obtained from RCT for strain level higher than 0.02 %.

The experimental results 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 (Figure 18):

$$\frac{G(\gamma)}{G_0} = \frac{1}{1+\alpha\gamma\,(\%)^{\beta}} \tag{5}$$

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

The expression (5) allows the complete shear modulus degradation to be considered with strain level. The values of α = 20 and β = 0.87 were obtained for DPC area.

As suggested by Yokota et al. [1981], the inverse variation of damping ratio with respect to the normalised shear modulus has an exponential form as that reported in Figure 19 for the DPC area:

$$D(\gamma)(\%) = \eta \cdot \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_0}\right] \tag{6}$$

in which: D(γ) = strain dependent damping ratio; γ = shear strain; η, γ = soil constants.

The values of η = 19 and γ = 2.3 were obtained for DPC area.

The Equation (6) gives the maximum value D_{max} = 19 % for $G(\gamma)/G_o$ = 0 and the minimum value D_{min} = 1.90 % for $G(\gamma)/G_o$ = 1.

Therefore, Equation (6) can be re-written in the fol-

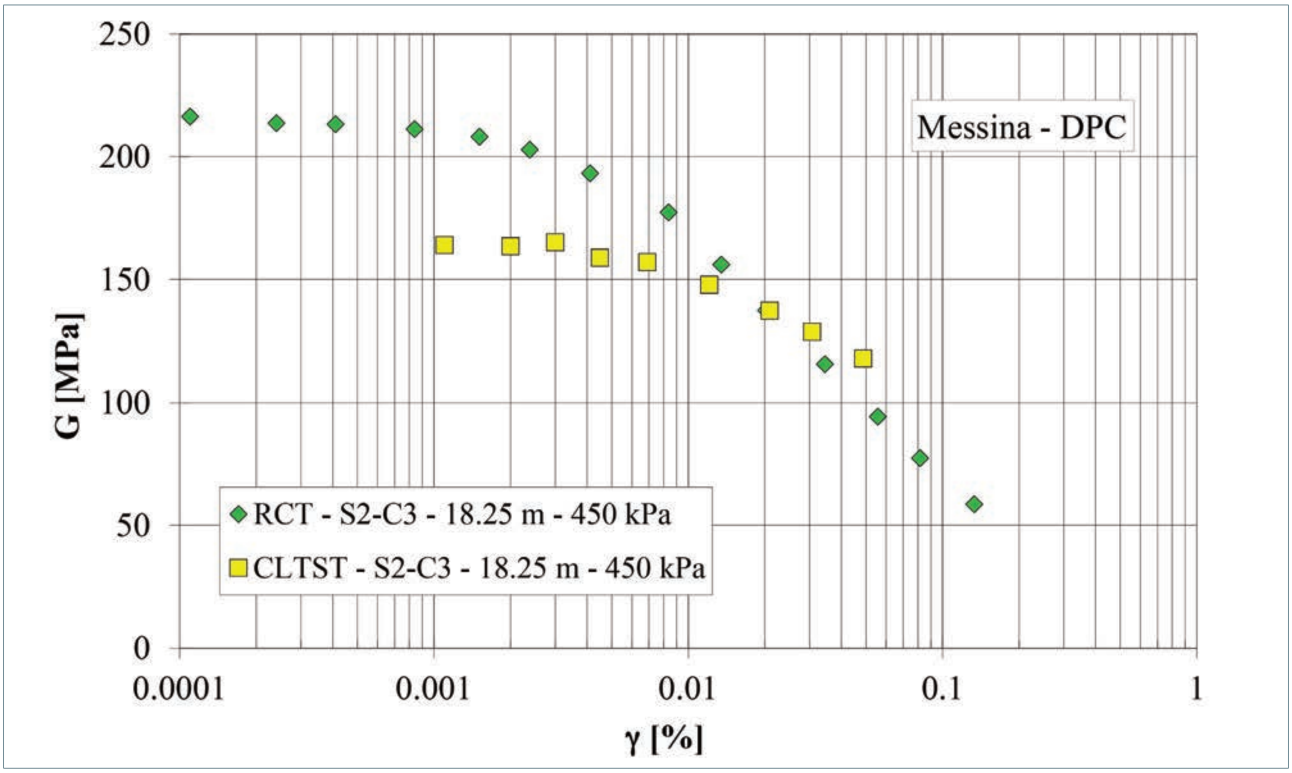

FIGURE 15. G vs γ curves from RCT and CLTST tests.

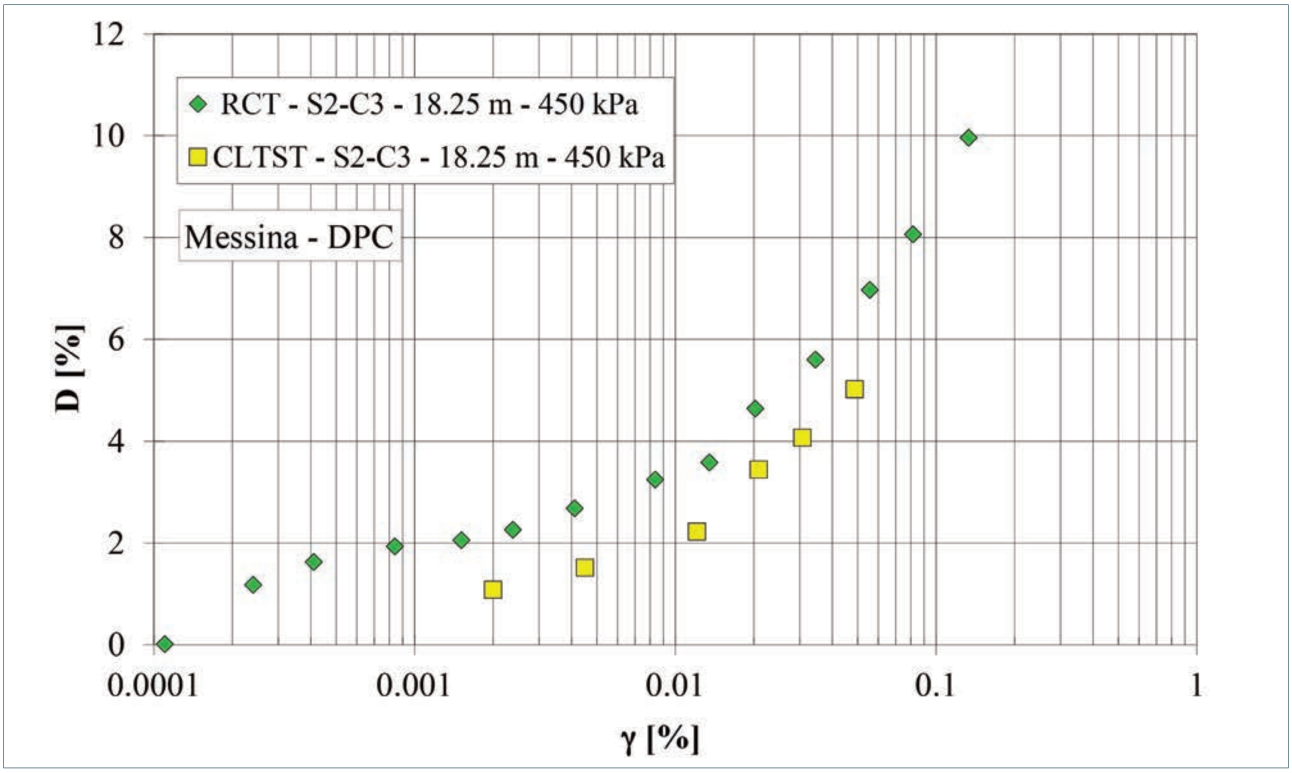

FIGURE 16. D vs γ curves from RCT and CLTST tests.

lowing normalised form:

$$\frac{D(\gamma)}{D(\gamma)_{max}} = \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_0}\right] \quad (7)$$

6. CONCLUSIONS

This study benefited of a great availability of borehole data, geophysical surveys and laboratory tests carried out from various campaigns of geological surveys that have

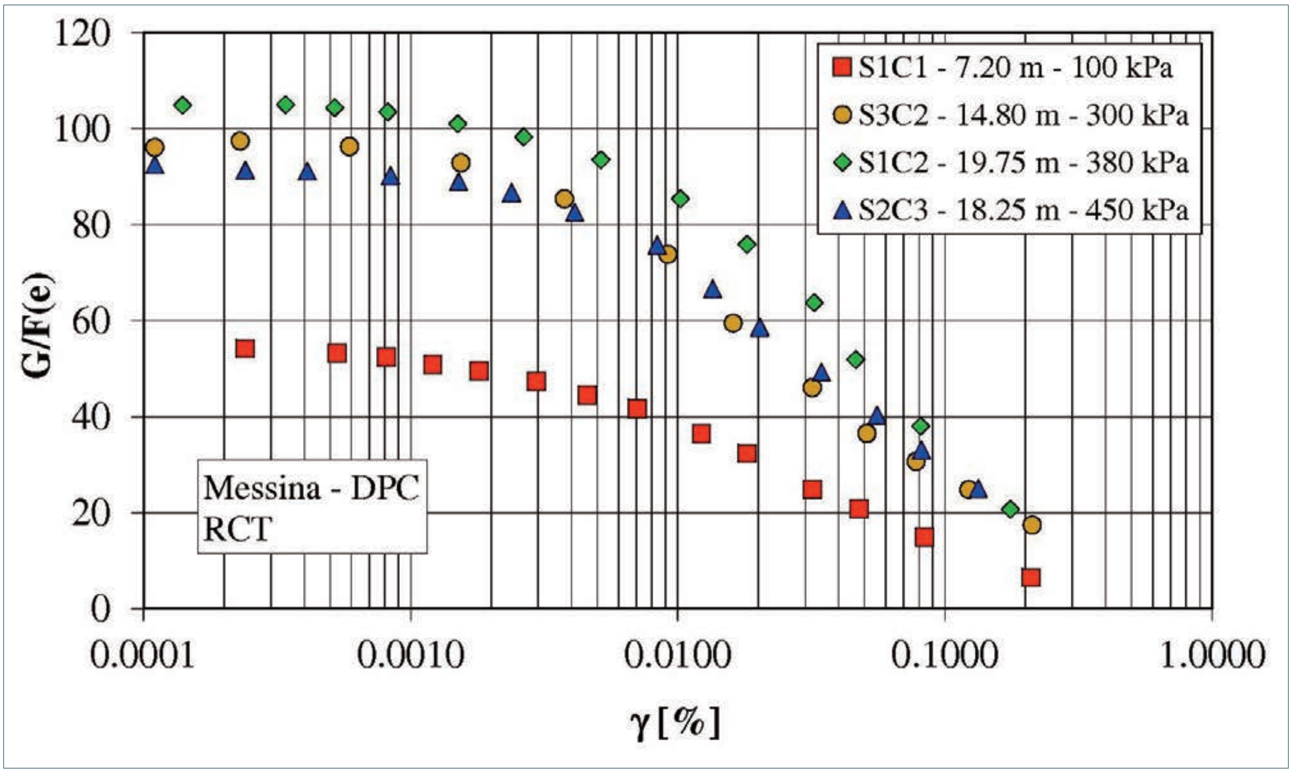

FIGURE 17. G/F(e) - γ curves from RCT tests.

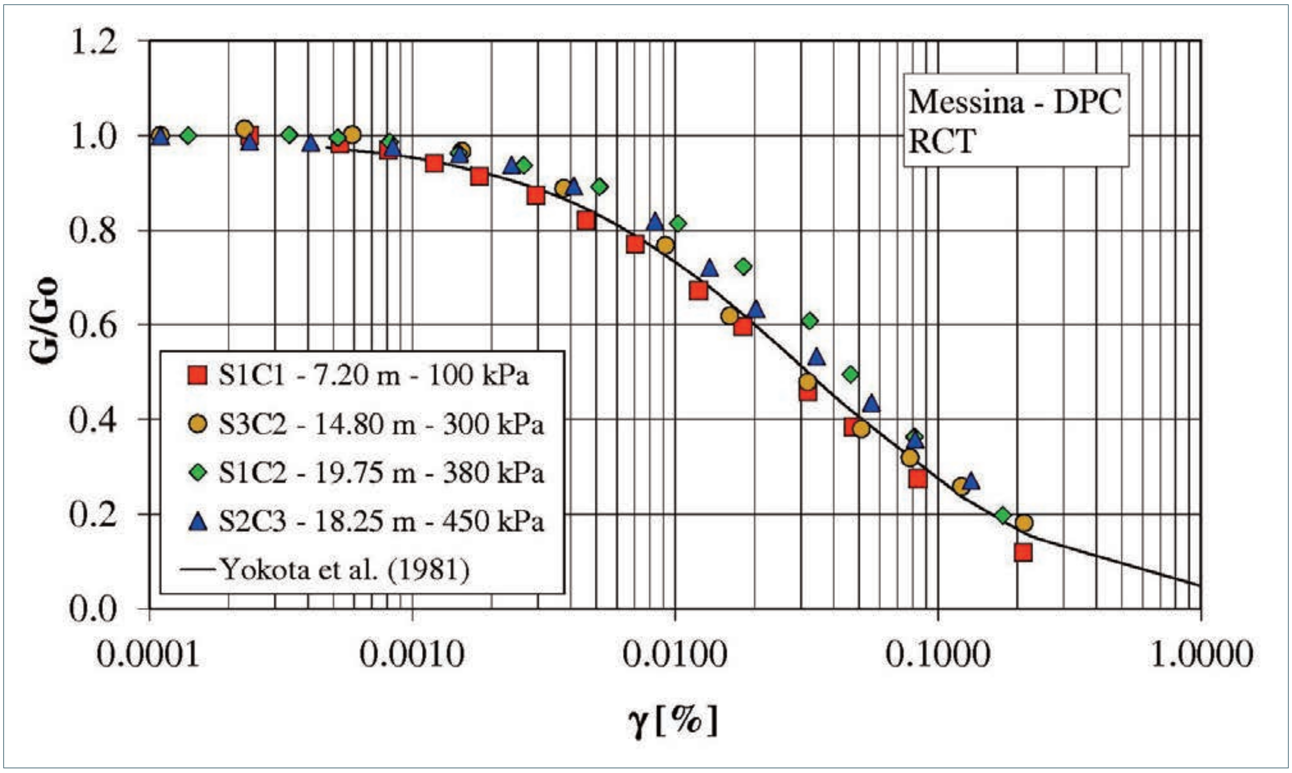

FIGURE 18. G/Go- γ curves from RCT tests.

interested the area of one test site in the city of Messina. A detailed site characterization of the "Civil Defence Department" (DPC) area is presented for the seismic response analysis evaluation. The available data enabled to define for this site the small strain shear modulus G_0 profile variation with depth. On the basis of the obtained results it is possible to draw the following conclusions: I) a degradation phenomenon occurs during CLTST because of the previous RCT on the same specimen; II) differences between initial G values from RCT

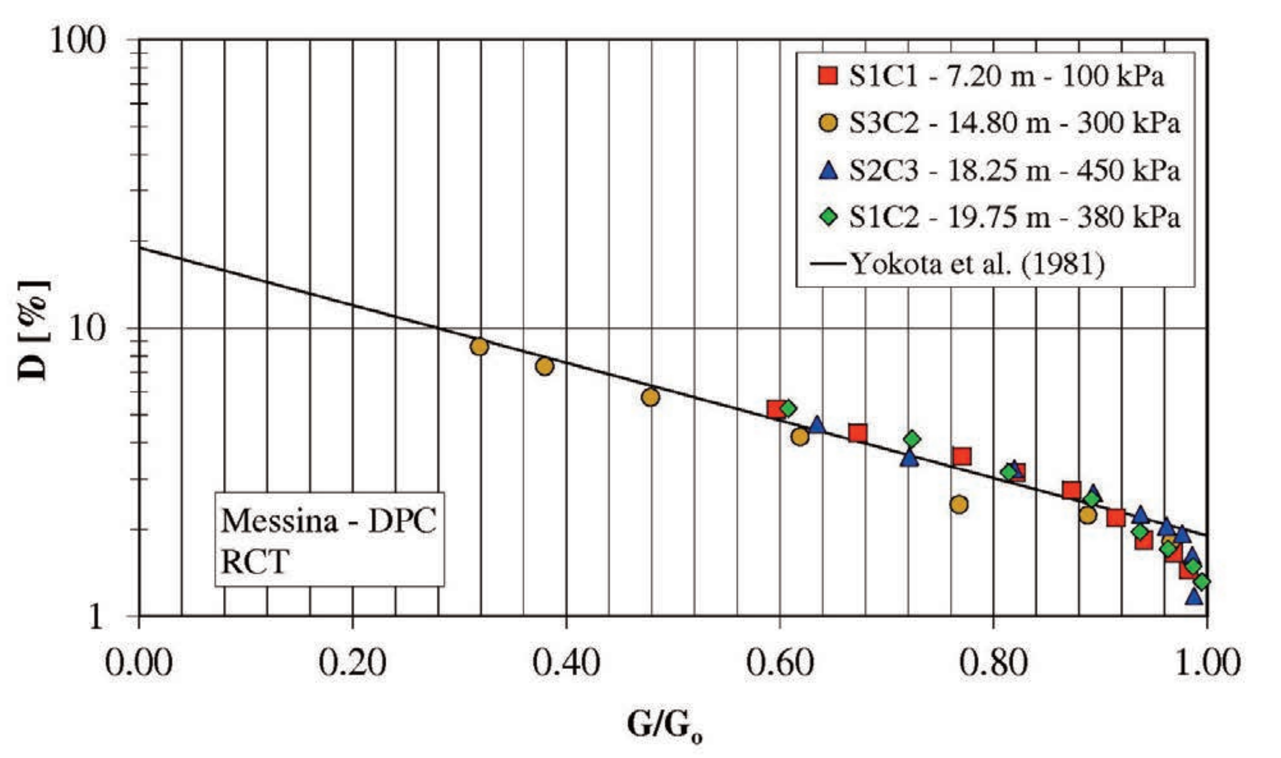

FIGURE 19. D-G/G_0 curves from RCT tests.

and CLTST results are probably due also to degradation phenomena; III) damping ratio values obtained by RCT are greater than those obtained by CLTST; IV) higher values of G_0 were obtained by Down-Hole tests; V) a good agreement of G_0 values by SDMT, Down Hole and empirical correlations was obtained.

On the basis of in situ test results, it is possible to stress that the small strain shear modulus measured in the laboratory by RCT is in a good agreement with that obtained in situ by means of SDMT and DH tests.

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