Clay Soil Characterization by the New Seismic Dilatometer Marchetti Test (SDMT)

Cavallaro A.
CNR – IBAM, Catania, Italy

Grasso S. & Maugeri M.
Department of Civil and Environmental Engineering, University of Catania, Italy

Keywords: small shear modulus, in situ tests, non linearity.

ABSTRACT: This paper describes and compares the results of in situ laboratory investigations performed on Catania soil that were carried out to determine the variation of shear modulus with depth and strain level by Seismic Dilatometer Marchetti Test (SDMT), Down-Hole (DH) Test and Resonant Column Tests (RCT). Some considerations on shear modulus degradation evaluation by SDMT are proposed. The available data also enabled one to compare the shear modulus profile obtained by empirical correlations based on CPT or laboratory results with Down Hole Test and Seismic Dilatometer Marchetti Test.

1 INTRODUCTION

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.

However, it was been pointed out by many researchers that the strain level which often occurs in geotechnical problems is quite small even under the static loading condition and the case of conventional foundations (Jardine et al. 1986, Battaglio and Jamiolkowski 1987, Burland 1989, Berardi and Lancellotta 1991, Maugeri et al. 1998).

On the other hand, the hypotheses of homogeneity, elasticity and isotropy are unrealistic for soils. In reality soil behaviour is non linear (non linear elasticity or plasticity) and anisotropic. In particular, some researchers (Hardin 1978, Jardine et al. 1984, 1986) have postulated that an elastic or apparently elastic soil response occurs only at small strains (i. e. less than 0.001 %).

In this paper the seismic flat dilatometer test (SDMT) was used to provide shear wave velocity (V_s) measurements to supplement conventional inflation readings (p_o and p_1).

Soil stratigraphy and soil parameters are evaluated from the pressure readings while the small strain stiffness (G_o) is obtained from in situ V_s profiles.

A comprehensive in situ and laboratory investigation has been carried out to study the STM M6 test site in the city of Catania.

The results obtained by SDMT were compared with those evaluated by in situ and laboratory tests during the seismic microzonation study performed in the city of Catania.

2 INVESTIGATION PROGRAM AND BASIC SOIL PROPERTIES

The investigated STM M6 area, located in the South zone of the city, has plane dimensions of 212400 mq and a maximum depth of 100 m. The area pertaining to the investigation program and the locations of the boreholes and field tests are shown in Figure 1.

The STM M6 site consists of fine alluvial deposits. Undisturbed samples were retrieved by means of Osterberg (1973) piston sampler and an 86 mm Shelby tube sampler.

In the Catania STM M6 area, 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 values of the natural moisture content, w_n, range from between 22 and 56 %.

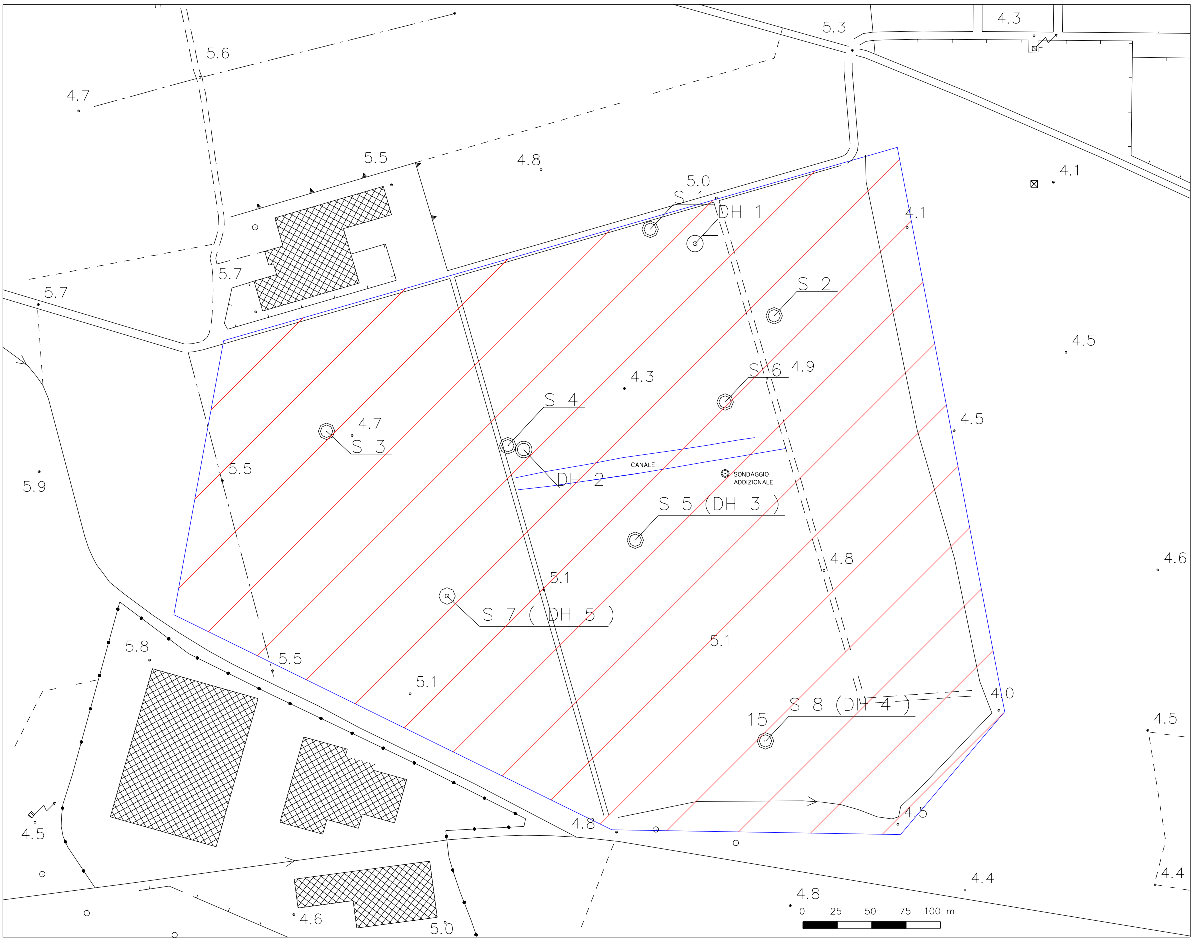

Figure 1. Layout of investigation area with locations of the boreholes and of field tests.

Characteristic values for the Atterberg limits are: w_L = 54 - 84 % and w_p = 27 - 46 %, with a plasticity index of PI = 22 - 41 %.

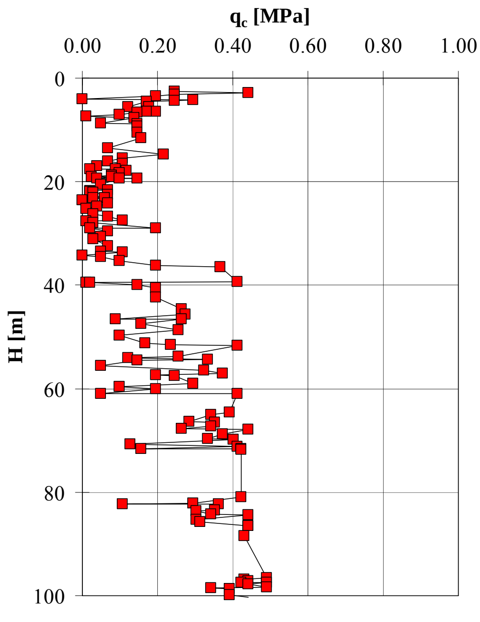

Figure 2. Static cone penetration test results.

The good degree of homogeneity of the deposit is confirmed by comparing the penetration resistance q_c from mechanical cone penetration tests (CPT) performed at different locations over the investigated area (Figure 2). 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.

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

Table 1. Mechanical characteristics for Catania STM M6 area.

Site	γ [kN/m^3]	e	c_u [kPa]	c' [kPa]	ϕ' [°]
STM M6	16.6-20.2	0.56-1.51	28.75-203.61	2.41-21.7	16-18

where: c_u (Undrained shear strength), c' (Cohesion) and ϕ' (Angle of shear resistance) were calculated from and C-U and C-D Triaxial Tests.

BACK

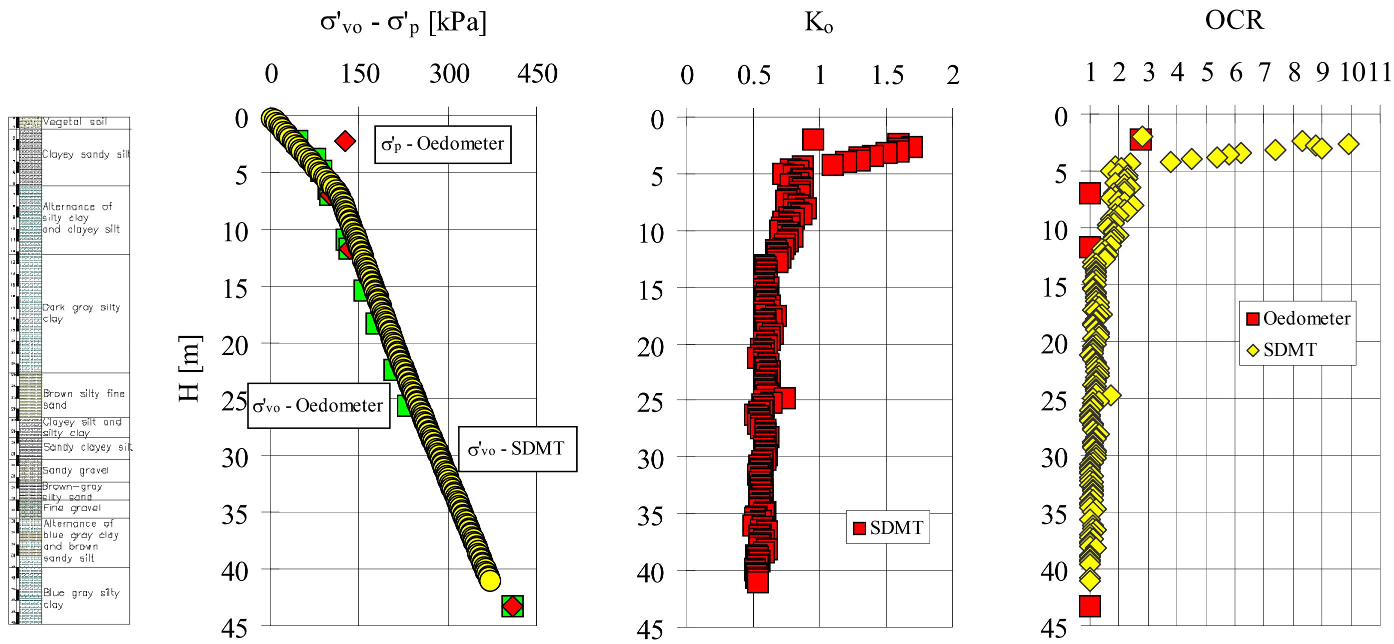

Figure 3. Stress history from in situ and laboratory tests.

The preconsolidation pressure σ'_p and the overconsolidation ratio OCR = σ'_p/σ'_{vo} were evaluated from the 24^h 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 information obtained from laboratory and in situ tests is summarized in Figure 3.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.

3 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).

3.1 *Small strain shear modulus G_o: in situ vs. laboratory measurements*

The SDMT provides a simple means for determining the initial elastic stiffness at very small strains and in situ shear strength parameters at high strains in natural soil deposits.

Source waves are generated by striking a horizontal plank at the surface that is oriented parallel to the axis of a geophone 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).

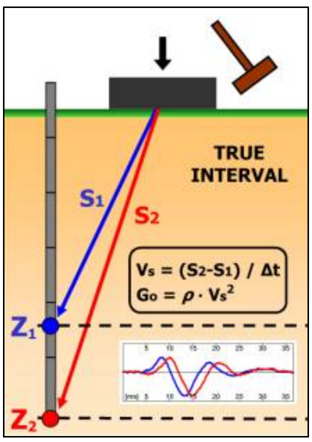

Figure 4. SDMT scheme for the measure of V_s.

BACK

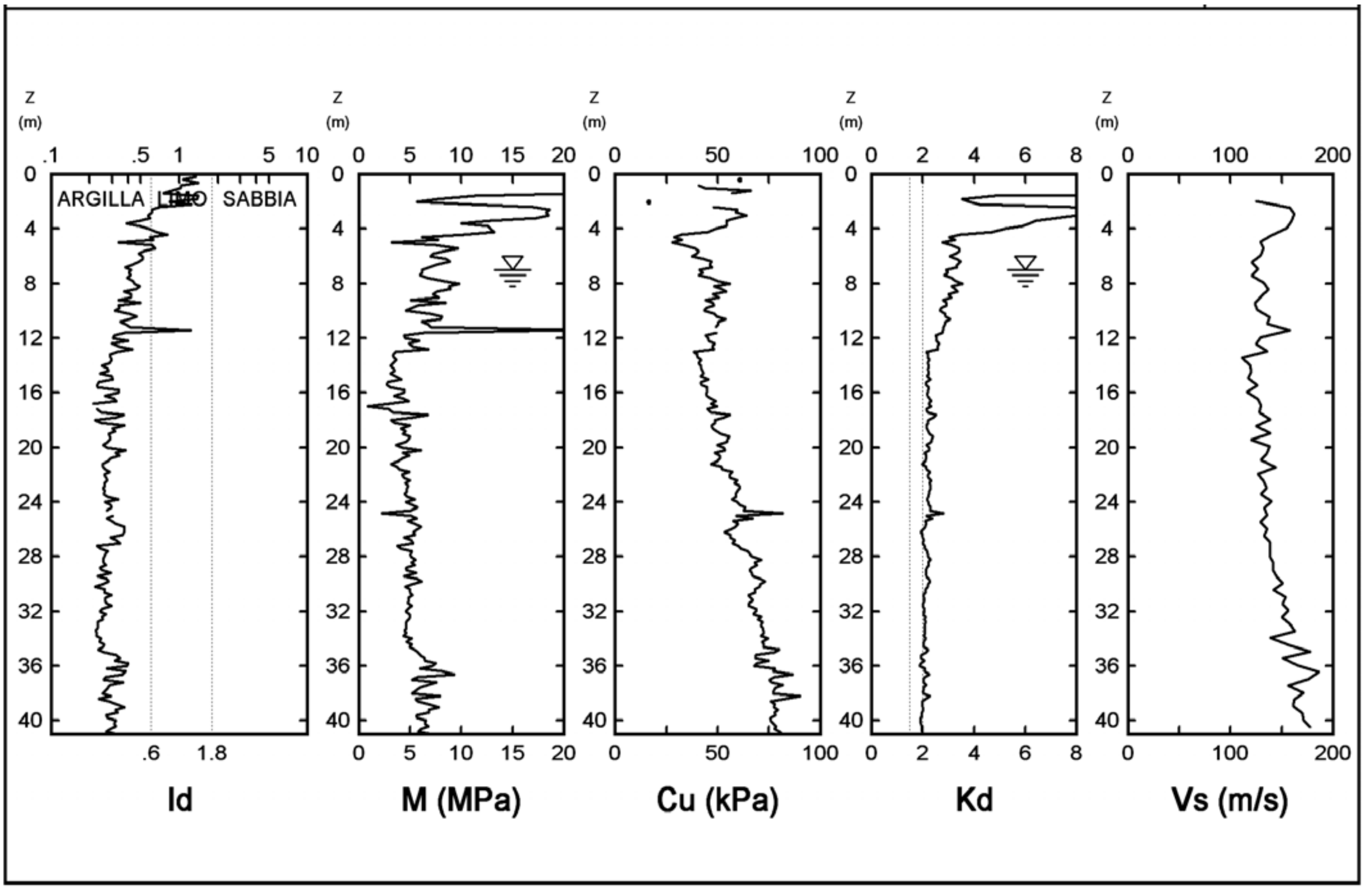

Figure 5. Summary of SDMTs in Catania STM M6 area.

The small strain shear modulus G_o is determined by the theory of elasticity by the well known relationships:

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

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.

Figure 6 shows the values of G_o obtained in situ from a DH 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 & 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.

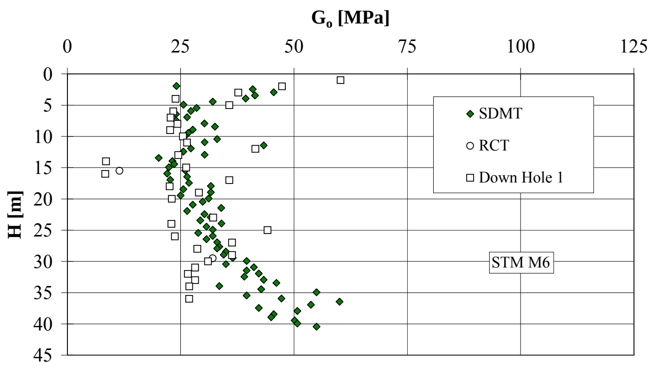

Figure 6. G_o from laboratory and in situ tests.

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.

3.2 *Shear modulus degradation from SDMT*

G is the unload-reload shear modulus evaluated from RCT, while G_o is the maximum value or also "plateau" value as observed in the G-log(γ) plot. 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. At strains greater than γ_t^e some plastic deformation occurs and the stress-strain relationship becomes non-linear. When a certain limit strain is exceeded, degradation phenomena are observed. This limit strain is called volumetric threshold shear strain (γ_t^v) and is rate dependent. For shear at a strain rate of about 0.4%/min γ_t^v ranges between 0.05 and 0.1 % and increases for increasing strain rates (Lo Presti 1989, Vucetic 1994).

A key feature distinguishing SDMT from other seismic tests is that in adition to G_o, a "working strain" shear modulus, G_{ws} is determined. The availability of two datapoints (G_o and G_{ws}) may help in selecting the G-γ decay curve, important in soil dynamics.

G_{ws} can be evaluated by the following equation based on M_{DMT} values:

$$G_{ws} = \frac{(1-2\cdot\nu)}{2\cdot(1-\nu)}\cdot M_{DMT} \qquad (2)$$

where ν (Figure 7) is the Poisson ratio, obtained from Down Hole (DH) test.

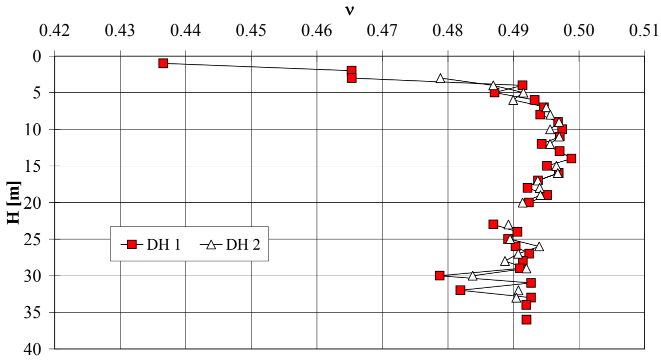

Figure 7. Poisson ratio from Down Hole (DH) test.

As regard the evaluation of "working strain" γ_{ws}, we must distinguish the settlements predicted during the analysis of case histories (γ = 0.05 to 0.1 %) and the real strain investigated by SDMT to measure the dilatometer modulus E_D.

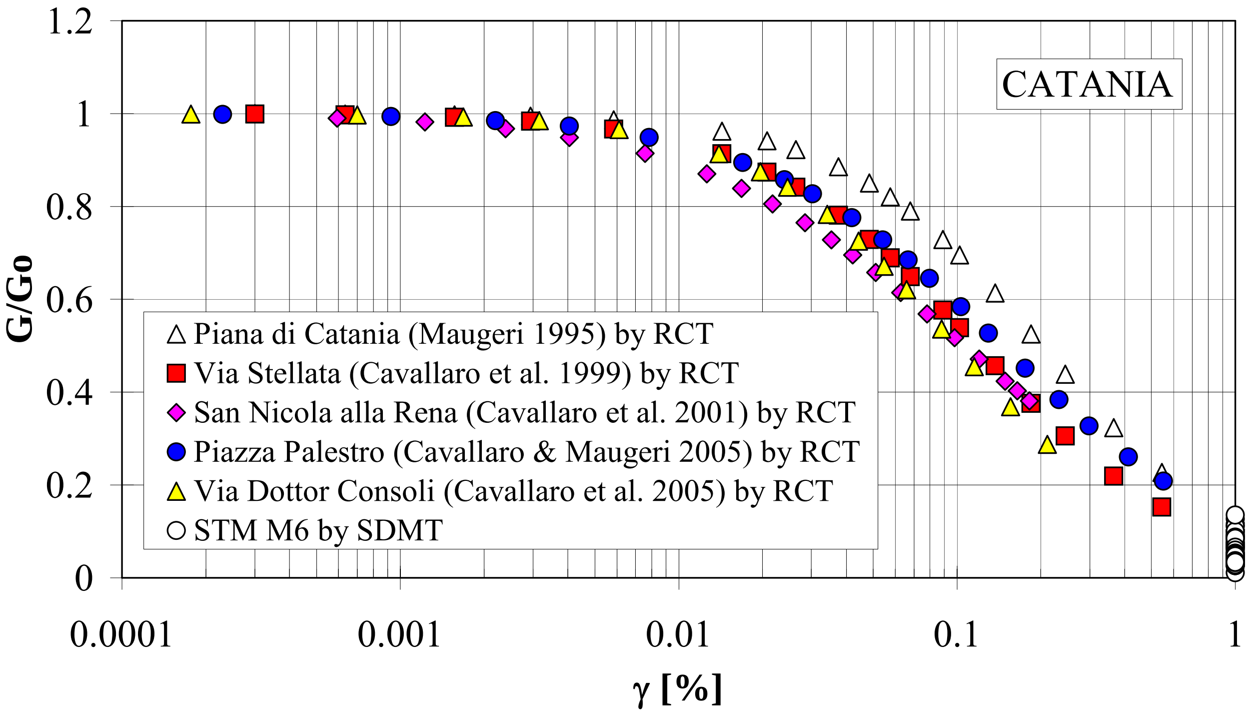

Figure 8. G/G_o vs shear strain for Catania area.

BACK

In the vicinity of the probe, the flat dilatometer blade is expected to produce shear similar to the cylindrical probes of the piezocone and smaller than the push-in pressuremeter (Lacasse & Lunne, 1988). Tentatively reported in Figure 8 is the comparison between RCT for different Catania site and SDMT results at large strain for STM M6 area.

3.3 *Evaluation of* G_o *from empirical correlations*

It was also attempted to evaluate the small strain shear modulus by means of the following empirical correlations based on penetration tests results or laboratory results available in literature.

a) Hryciw (1990):

$$G_o = \frac{530}{(\sigma'_v/p_a)^{0.25}} \frac{\gamma_D/\gamma_w - 1}{2.7 - \gamma_D/\gamma_w} K_o^{0.25} \cdot (\sigma'_v \cdot p_a)^{0.5} \quad (3)$$

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):

$$G_o = \frac{406 \cdot q_c^{0.696}}{e^{1.13}} \quad (4)$$

where: G_o and q_c are both expressed in [kPa] and e is the void ratio. Eq. (4) is applicable to clay deposits only;

c) Jamiolkowski et. al. (1995):

$$G_o = \frac{600 \cdot \sigma_m'^{0.5} p_a^{0.5}}{e^{1.3}} \quad (5)$$

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 (5) 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 (5) incorporates a term which expresses the void ratio; the coefficient of earth pressure at rest only appear in equation (3). However only equation (3) tries to obtain all the input data from the SDMT results.

The G_o values obtained with the methods above indicated 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.

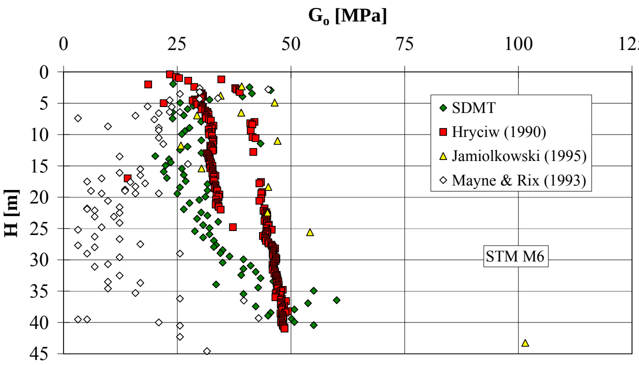

Figure 9. G_o from different empirical correlations.

All the considered methods show very different G_o values of the Holocene soil. On the whole, equation (3) and (5) seems to provide the most accurate trend of G_o with depth, as can be seen in Figure 9. It is worthwhile to point out that equation (5) overestimated G_o for depths greater than 25 m.

4 CONCLUSIONS

A site characterization for seismic response analysis has been presented in this paper. On the basis of the data shown it is possible to draw the following conclusions:

- SDMT were performed up to a depth of 42 meters. The results show a very detailed and stable shear wave profile. The shear wave profiles obtained by SDMT compare well with laboratory tests;
- the small strain shear modulus measured in the laboratory is on average 0.90 of that measured in situ by means of SDMT and DH tests;
- empirical correlations between the small strain shear modulus and penetration test results were used to infer G_o from CPT and SDMT. The values of G_o were compared to those measured with SDMT and DH tests. This comparison indicates that some agreement exists between empirical correlations and SDMT and DH test;

BACK

- moreover SDMT measurements are much more stable and repeatable than DH test, so the SDMT is a powerful investigation tool.

- SDMT, because of three independent measurements of p_o, p_1 and V_s, gives shear modulus at small strain and large strain for detecting soil non linearity.

ACKNOWLEDGMENTS

The authors wish to thank the geotechnical engineer Alessio Carbonaro for his contribution to the work.

REFERENCES

Battaglio, M. & Jamiolkowski, M. 1987. Analisi delle Deformazioni. *XII CGT*, Politecnico di Torino.

Berardi, R. & Lancellotta, R. 1991. Stiffness of Granular Soils from Field Performance. *Geotechnique* Vol. 41, N°. 1: 149-157.

Burland, J.B. 1989. Small is Beatiful - The stiffness of Soil at Small Strains. *Proceedings of the 9th Laurits Bjerrum Memorial Lecture, Canadian Geotechnical Journal*, Vol. 26, N°. 4: 499-516.

Carrubba, P. & Maugeri, M. 1988. Determinazione delle Proprietà Dinamiche di un'Argilla Mediante Prove di Colonna Risonante. *Rivista Italiana di Geotecnica*, N°. 2, Aprile-Giugno1988: 101-113.

Cavallaro, A., Maugeri, M., Lo Presti, D.C.F. & Pallara O. 1999. Characterising Shear Modulus and Damping from in Situ and Laboratory Tests for the Seismic Area of Catania. *Proceedings of the 2nd International Symposium on Pre-failure Deformation Characteristics of Geomaterials*, Torino, 28 - 30 September 1999: 51-58.

Cavallaro, A., Grasso, S. & Maugeri, M. 2001. A Dynamic Geotechnical Characterization of Soil at Saint Nicolò alla Rena Church Damaged by the South Eastern Sicily Earthquake of 13 December 1990. *Proceeding of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, Satellite Conference "Lessons Learned from Recent Strong Earthquakes"*, Istanbul, 25 August 2001: 243-248.

Cavallaro, A. & Maugeri, M. 2005. Non Linear Behaviour of Sandy Soil for the City of Catania. *Seismic Prevention of Damage: A Case Study in a Mediterranean City*, Wit Press Publishers, Editor: Maugeri M.: 115-132.

Cavallaro, A., Grasso, S. & Maugeri, M. 2005. Site Characterisation and Site Response for a Cohesive Soil in the City of Catania. *Proceedings of the Satellite Conference on Recent Developments in Earthquake Geotechnical Engineering*, Osaka, 10 September 2005: 167-174.

Hardin, B.O. 1978. The Nature of Stress-Strain Behaviour of Soils. *Earthquake Engineering and Soil Dynamics*, Vol. 1, Pasadena, CA, ASCE, New York: 3-90.

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. & 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: 1523-1546.

Jardine, R.J., Symes M.J. & Burland J.B. 1984. The Measurement of Soil Stiffness in the Triaxial Apparatus. *Geotechnique*, Vol. 34, N°. 3 : 323-340.

Jardine, R.J., Potts, D.M., Fourie, A. & Burland, J.B. 1986. Studies of the Influence of Non-Linear Stress-Strain Characteristics in Soil-Structure Interaction. *Geotechnique*, Vol. 36, N°.3 : 377-396.

Lacasse S. & Lunne T. 1988. Calibration of Dilatometer Correlations. *Proceedings of 1st International Symposium on Penetration Testing*, ISOPT-1, Orlando: 539-548.

Lo Presti, D.C.F. 1989. Proprietà Dinamiche dei Terreni. *XIV C.G.T.* Torino.

Marchetti, S. 1980. In Situ Tests by Flat Dilatometer. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 106, N°. GT3, March, 1980: 299-321.

Martin, G.K. & Mayne, P.W. 1997. Seismic Flat Dilatometers Tests in Connecticut Valley Vaeved Clay. *ASTM Geotechnical Testing Jornal*, 20 (3): 357-361.

Martin, G.K. & Mayne, P.W. 1998. Seismic Flat Dilatometers Tests in Piedmont Residual Soils. *Geotecnical Site Characterization*, Vol. 2, Balkema, Rotterdam: 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: 1323-1327.

Maugeri, M., Castelli, F., Massimino, M.R. & Verona, G. 1998. Observed and Computed Settlements of Two Shallow Foundations on Sand. *Journal of the Geotechnical and Geonvironmental Engineering*, ASCE, Vol. 124, N°. 7, July, 1998: 595-605.

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

Osterberg J.O. 1973. An Improved Hydraulic Piston Sampler. *Proceedings of 8th ICSMFE,* Moscow. Vol 1.2.

BACK

Shibuya, S. & Tanaka, H. 1996. Estimate of Elastic Shear Modulus in Holocene Soil Deposits. *Soils and Foundations*, Vol. 36, N°. 4: 45-55.

Vucetic M. 1994. Cyclic threshold shear strains in soils. *Journal of Geotechnical Engineering, ASCE*, Vol. 120, N°. 12: 2208-2228.

BACK