DYNAMIC CLAY PROPERTIES BY IN SITU AND LABORATORY TESTS FOR AN INDUSTRIAL BUILDING IN CATANIA (ITALY)

Francesco CASTELLI¹, Valentina LENTINI¹, Michele MAUGERI³

ABSTRACT

The paper deals with the dynamic geotechnical characterization of a clay soil for the construction and successively for the analysis of the settlements interesting a reinforced concrete building in the industrial area of Catania (Sicily, Italy). In this paper a review of the principal results of the in situ and laboratory investigations is described. Before of the construction of the building, the site was well investigated to determine the geological profile and the geotechnical properties of the soil foundation. The following in situ geotechnical tests were carried out: boreholes, SPT, CPT, PLT and dynamic in situ tests. Among them Down-Hole (DH), Cross-Hole (CH), SASW and Seismic Dilatometer Marchetti Tests (SDMT) have been carried out with the aim to evaluate the shear waves velocity profile ($V_s$). Laboratory tests were carried out on undisturbed samples, such as: Oedometer test, Direct Shear Test, Triaxial Test, Resonant Column and Torsional Shear Test. Static and dynamic parameters obtained by in situ and laboratory tests were reported and analyzed. The experimental results provide the parameters to evaluate the soil behavior and the geotechnical model for the soil settlements evaluation.

Keywords: dynamic properties, shear waves velocity, resonant column, shear modulus, damping modulus.

INTRODUCTION

The geotechnical earthquake engineering problems requires the evaluation of the dynamic soil properties. The mechanical properties associated with dynamic loading are shear wave velocity ($V_s$), shear modulus ($G$), damping ratio ($D$), and Poisson’s ratio ($\nu$). To determine soil dynamic properties, the current state of practice involves: estimating or measuring shear waves velocity $V_s$ in the field using geophysical methods and estimating or measuring the variation in laboratory of shear modulus $G$ and damping ratio $D$ as a function of shear strain $\gamma$.

The geotechnical characterization of Catania area by in situ and laboratory tests has a great importance because the east coast area of Sicily is considered as one of the zones of Italy with greater high seismic risk, basing on the past and current seismic history and on the typology of civil buildings and industrial activities (Cavallaro et al., 2008). The knowledge of soil dynamic properties gives the possibility to preview the soil behavior during the seismic events. A number of field and laboratory techniques have been developed over the last 35 years to measure the dynamic properties of soils.

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To the aim of soil dynamic characterization, in situ tests are available such as Down-Hole, Cross-Hole, SASW and Seismic Dilatometer Marchetti Tests and in laboratory tests such as Resonant Column and Torsional Shear Tests.

The ability of laboratory tests to provide accurate measurements of dynamic soil properties is affected by several factors, such as sample disturbance, specimen size, equipment compliance, loading conditions, and reproduction of actual field conditions such as stress, chemical, thermal, and structural conditions. The impact of many of these factors, particularly on nonlinear dynamic soil properties, remains to be quantified. Thus there is a significant need to perform nonlinear dynamic soil property measurements in situ and to compare the field values to those measured in the laboratory. In situ dynamic property measurements would eliminate many of the problems associated with laboratory testing and would allow the accuracy of laboratory methods to be evaluated (Kurtulus and Stokoe, 2008).

SOIL DYNAMIC PROPERTIES BY IN SITU AND LABORATORY TESTS

Shear modulus
The small strain ($\gamma \leq 0.001\%$) shear modulus $G_o$ can be determined from most seismic geophysical methods or tests, such as SDMT and Down Hole (DH) tests, and the shear waves velocity ($V_s$) can be used to compute the modulus $G_o$ using the following expression:

$$G_o = \rho \cdot V_s^2$$  \hspace{1cm} (1)

where $\rho$ is the mass density of the soil. The measured shear wave velocity is generally considered the most reliable means to obtain the $G_o$ for a soil deposit. The equivalent shear modulus ($G_{eq}$) can be determined in the laboratory by means of a Resonant Column test (RCT) performed on Shelby tube specimens. 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).

Variation of shear modulus and damping ratio with strain level
The Resonant Column test for determining modulus and damping characteristics of soil is based on the theory of waves propagation. Either compression waves or shear waves can be propagated through the soil specimen from which either Young’s modulus or shear modulus can be determined. In a resonant column apparatus the excitation frequency is adjusted until the specimen experience resonance. The modulus is computed from the resonant frequency and the geometric properties of the specimen and driving apparatus. A measure of the damping ratio can be obtained by either of two methods, amplitude decay or magnification factor.

The experimental results of specimens obtained by RCT 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:

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

in which:

$G(\gamma) =$ strain dependent shear modulus;
$\gamma =$ shear strain;
$\alpha, \beta =$ soil constants.
The expression (2) allows the complete shear modulus degradation to be considered with strain level (Maugeri, 1995). The values of $\alpha = 7.15$ and $\beta = 1.223$ were obtained for the Catania clay (Sicily, Italy) by Carrubba and Maugeri (1988). As suggested by Yokota et al. (1981), the inverse variation of damping ratio with respect to the normalised shear modulus has an exponential form (Maugeri, 1995):

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

(3)

in which:
- $D(\gamma)$ = strain dependent damping ratio;
- $\gamma$ = shear strain;
- $\eta, \lambda$ = soil constants.

The values of $\eta = 28.12$ and $\lambda = 2.50$ were obtained for the Catania clay (Sicily, Italy) by Carrubba and Maugeri (1988). The equation (2) assume maximum value $D_{\text{max}} = 28.12\%$ for $G(\gamma)/G_o = 0$ and minimum value $D_{\text{min}} = 2.30\%$ for $G(\gamma)/G_o = 1$. Therefore, eq. (2) can be rewritten in the following normalised form:

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

(4)

These parameters were obtained from the damping values assessed by means of the steady-state method.

**Evaluation of shear modulus $G_o$ from empirical correlations**

It was also attempted to evaluate the small strain shear modulus $G_o$ by means of the following empirical correlations based on standard (CPT) or dynamic penetration tests (SPT) and Seismic Dilatometer Marchetti Test (SDMT) results or laboratory results available in literature.

a) *Hryciw (1990)*

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

(5)

where: $G_o, \sigma'$, and $p_a$ are expressed in the same unit; $p_a = 1$ bar is a reference pressure; $\gamma_0$ 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)*

$$G_o = \frac{406 \cdot q_c^{0.696}}{e^{1.13}}$$

(6)

where: $G_o$ and $q_c$ are both expressed in [kPa] and $e$ is the void ratio. Equation (6) is applicable to clay deposits only;

c) *Jamiolkowski et al. (1995)*

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

(7)
where: $\sigma'_{m} = \left(\sigma_{v} + 2 \cdot \sigma'_{h}\right)/3$; $p_a = 1$ bar is a reference pressure; $G_o$, $\sigma'_{m}$ and $p_a$ are expressed in the same unit.

The values for parameters which appear in equation (7) 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.

**Evaluation of shear modulus $G_o$ from SPT tests**

The values of $G_o$ can be evaluated by means of the following empirical correlations based on standard penetration test (SPT) results.

a) *Ohta and Goto (1978)*

$$V_s = 54.33 \cdot \left(N_{SPT}\right)^{0.173} \cdot \alpha \cdot \beta \left(\frac{z}{0.503}\right)^{0.193}$$

where: $V_s$ = shear wave velocity (m/s), $N_{SPT}$ = number of blows from SPT, $z$ = depth (m), $\alpha$ = age factor (Holocene = 1.000, Pleistocene = 1.303), $\beta$ = geological factor (clays = 1.000, sands = 1.086).

b) *Yoshida and Motonori (1988)*

$$V_s = \beta \cdot \left(N_{SPT}\right)^{0.25} \cdot \sigma'_{vo}^{0.14}$$

where: $V_s$ = shear wave velocity (m/s), $N_{SPT}$ = number of blows from SPT, $\sigma'_{vo}$ = vertical pressure, $\beta$ = geological factor (any soil = 55, fine sand = 49).

c) *Imai and Yoshimura (1970)*

$$V_s = 76 \cdot \left(N_{SPT}\right)^{0.33}$$

where: $V_s$ = shear wave velocity (m/s), $N_{SPT}$ = number of blows from SPT.

d) *Ohba and Toriumi (1970)*

$$V_s = 8 \cdot \left(N_{SPT}\right)^{0.31}$$

where: $V_s$ = shear wave velocity (m/s), $N_{SPT}$ = number of blows from SPT.

**CASE STUDY**

On September 2000 in the industrial area of Catania (Sicily, Italy) started the construction of a reinforced concrete building (*Figure 1*) on normally consolidated clayey deposits. *Figure 2a* and *Figure 2b* show a plan view of the industrial building (M6) and the site respectively.

These soft soil deposits have a low bearing capacity and exhibit large settlements when subjected to loading. It is therefore inevitable to treat soft soil deposits prior to construction activities in order to prevent differential settlements and subsequently potential damages to structures.
To improve the compressibility of the soil foundation, the preloading technique was applied by means of the construction of an instrumented circular test embankment with a diameter of 65 m and 2.50 m high. Thirty-three vertical prefabricated drains were disposed beneath the embankment. The construction of the embankment started on September 5, 2000 and it was realized in different layers. Due to the embankment weight, consolidation started and considerable settlements took place. Several types of geotechnical instrumentation were installed before the construction of the embankment to monitor its performance. Vertical drains and monitoring instruments (assettimeters, shallow and deep piezometers, inclinometers) were used to monitor the soil foundation beneath the embankment.

**PREVIOUS STUDIES**

**Site investigation**
To evaluate the geological profile and the geotechnical properties of the soil, the site was well investigated by means of in-situ tests such as boreholes, standard and cone penetration tests, seismic tests, as well as laboratory tests (*Table 1*). Number 8 boreholes, performed to a maximum depth of 100 m, were located in the investigated area with dimensions of 212400 m², as reported in *Figure 3*. Inside the boreholes S3-S8 many standard penetration tests SPT (at different depth) and down-hole seismic tests (DH) have been performed (see *Figure 3*). Also cone penetration tests CPTU (no.22 tests) and plate
loading tests PLT (no.15 tests) have been performed. Finally, spectral analysis of surface wave (SASW) with different distance of the source and Seismic Dilatometer Marchetti Tests (SDMT) have been performed.

Oedometer, triaxial (UU and CID) and direct shear tests were performed. A comparison between laboratory and in situ tests results have been performed. For a comprehensive behavior of soil foundation an embankment with a diameter of 65 meters and 2.50 meters high have been built. The embankment have been monitored for the measurements of soil settlements and for the evaluation of time required to end the consolidation settlements.

### Table 1. Geotechnical soil properties by previous geotechnical investigation

<table>
<thead>
<tr>
<th>Tests</th>
<th>Depth (m)</th>
<th>(\gamma_{sat}) (kN/m³)</th>
<th>(w_{n}) (%)</th>
<th>(G_s) (kN/m³)</th>
<th>(e_o)</th>
<th>n</th>
<th>(S_r) (%)</th>
<th>(w_l) (%)</th>
<th>(w_p) (%)</th>
<th>(I_c)</th>
<th>(c_u) (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S7 R1</td>
<td>2.45-2.65</td>
<td>19.9</td>
<td>24.17</td>
<td>16.0</td>
<td>26.6</td>
<td>0.66</td>
<td>0.40</td>
<td>97</td>
<td>43</td>
<td>23</td>
<td>0.94</td>
</tr>
<tr>
<td>S7 I1</td>
<td>4.50-5.00</td>
<td>19.5</td>
<td>28.54</td>
<td>15.2</td>
<td>26.7</td>
<td>0.76</td>
<td>0.43</td>
<td>100</td>
<td>45</td>
<td>29</td>
<td>1.02</td>
</tr>
<tr>
<td>S7 R2</td>
<td>8.10-8.30</td>
<td>19.0</td>
<td>32.21</td>
<td>14.4</td>
<td>26.7</td>
<td>0.86</td>
<td>0.46</td>
<td>100</td>
<td>51</td>
<td>33</td>
<td>1.04</td>
</tr>
<tr>
<td>S7 I2</td>
<td>10.50-11.00</td>
<td>18.1</td>
<td>36.39</td>
<td>13.3</td>
<td>26.5</td>
<td>1.00</td>
<td>0.50</td>
<td>97</td>
<td>54</td>
<td>36</td>
<td>0.97</td>
</tr>
<tr>
<td>S7 I3</td>
<td>15.00-15.50</td>
<td>17.5</td>
<td>43.91</td>
<td>12.1</td>
<td>26.6</td>
<td>1.19</td>
<td>0.54</td>
<td>98</td>
<td>60</td>
<td>41</td>
<td>0.84</td>
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<tr>
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<td>19.30-19.50</td>
<td>17.5</td>
<td>44.96</td>
<td>12.1</td>
<td>26.7</td>
<td>1.21</td>
<td>0.55</td>
<td>100</td>
<td>65</td>
<td>38</td>
<td>0.74</td>
</tr>
<tr>
<td>S7 I4</td>
<td>20.00-20.50</td>
<td>17.2</td>
<td>48.25</td>
<td>11.6</td>
<td>26.5</td>
<td>1.28</td>
<td>0.56</td>
<td>100</td>
<td>73</td>
<td>30</td>
<td>0.57</td>
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<tr>
<td>S7 I5</td>
<td>25.00-25.50</td>
<td>17.1</td>
<td>51.13</td>
<td>11.3</td>
<td>26.7</td>
<td>1.37</td>
<td>0.58</td>
<td>100</td>
<td>77</td>
<td>25</td>
<td>0.49</td>
</tr>
<tr>
<td>S7 I6</td>
<td>28.00-28.50</td>
<td>17.3</td>
<td>47.19</td>
<td>11.8</td>
<td>26.6</td>
<td>1.26</td>
<td>0.56</td>
<td>100</td>
<td>70</td>
<td>41</td>
<td>0.79</td>
</tr>
<tr>
<td>S7 I7</td>
<td>37.00-37.50</td>
<td>17.3</td>
<td>48.27</td>
<td>11.7</td>
<td>26.7</td>
<td>1.28</td>
<td>0.56</td>
<td>100</td>
<td>65</td>
<td>48</td>
<td>0.98</td>
</tr>
<tr>
<td>S7 R4</td>
<td>38.50-38.70</td>
<td>18.9</td>
<td>32.35</td>
<td>14.3</td>
<td>26.7</td>
<td>0.87</td>
<td>0.56</td>
<td>100</td>
<td>61</td>
<td>36</td>
<td>1.14</td>
</tr>
</tbody>
</table>

### Soil properties

From soil profiles obtained by boreholes, the M6 soil foundation consists of fine alluvial deposits. Undisturbed samples were retrieved by means of Osterberg (1973) piston sampler and an 86 mm Shelby tube sampler. At the M6 industrial building site, the clay fraction (CF) is predominantly in the range of 2–54%. This percentage decreases to 0–2% at the depth of 95 m where a sand fraction of 4–9% is observed. The gravel fraction is always zero. The silt fraction is in the range of about 50–100%. The values of the natural moisture content \(w_{n}\) range from between 22 and 56%. Characteristic values for the Atterberg limits are: \(w_l = 54 - 84\%\) and \(w_p = 27 - 46\%\), with a plasticity index of \(PI = 22 - 41\%\).

The good degree of homogeneity of the deposit is confirmed by comparing the cone penetration resistance \(q_c\) from mechanical cone penetration tests (CPT) performed at different locations over the investigated area (Figure 3). 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 2.

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

The information obtained from laboratory and in situ tests is summarized in Figure 5. 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 \( \sigma'_p \) are obtained in the laboratory because of sample disturbance.

**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).

The SDMT, performed up to a depth of 42 meters, 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 coaxial cable with an oscilloscope (Martin and Mayne, 1997; 1998). The measured arrival times at successive depths provide the \( V_s \) profiles for horizontally polarized vertically propagating shear waves. The small strain shear modulus \( G_o \) is determined by the theory of elasticity by the well known equation (1).

A summary of SDMT parameters are shown in Figure 4. The \( K_d \) horizontal stress index 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. A first glance at the \( K_d \) profile is helpful to “understand” the deposit nature. Figure 4 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. 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 4. Summary of SDMTs in STM M6 area (Cavallaro et al., 2008).

Figure 5. $G_o$ from laboratory and in situ tests (Cavallaro et al., 2008).

Table 2. Soil compressibility by oedometer tests

<table>
<thead>
<tr>
<th>$z$ (m)</th>
<th>$k_v$ (m/s)</th>
<th>$c_v$ (m$^2$/s)</th>
<th>$E_{ed}$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>$3.37 \times 10^{-9}$</td>
<td>$4.86 \times 10^{-7}$</td>
<td>1413</td>
</tr>
<tr>
<td>8.3</td>
<td>$4.05 \times 10^{-10}$</td>
<td>$7.49 \times 10^{-8}$</td>
<td>1815.8</td>
</tr>
<tr>
<td>15.3</td>
<td>$1.02 \times 10^{-10}$</td>
<td>$2.43 \times 10^{-8}$</td>
<td>2347.8</td>
</tr>
<tr>
<td>25</td>
<td>$3.14 \times 10^{-11}$</td>
<td>$9.87 \times 10^{-9}$</td>
<td>3085</td>
</tr>
<tr>
<td>32</td>
<td>$1.70 \times 10^{-11}$</td>
<td>$6.27 \times 10^{-9}$</td>
<td>3617</td>
</tr>
<tr>
<td>37</td>
<td>$1.18 \times 10^{-11}$</td>
<td>$4.80 \times 10^{-9}$</td>
<td>3997</td>
</tr>
</tbody>
</table>
Geotechnical additional investigation carried out

The additional geotechnical investigation program is related to the evaluation of heterogeneity of soil profile along the longitudinal and transversal directional of the same building, the evaluation of in situ hydraulic soil properties and the evaluation of soil strength and deformability properties. The situ and laboratory tests were performed and the results obtained are summarized in this section.

As site investigation concerns, the following investigations were made: no.3 boreholes, no.3 Down-Hole (DH) seismic tests, no.4 cone penetration tests with pore pressure measurement (CPTU). As concerns laboratory test the following tests have been made on no.16 undisturbed soil samples for the evaluation of: no.15 tests for water content, no.15 tests for unit weight, no.15 for particle size distribution, no.15 for Atterberg limits, no.15 for the particle specific weight, no.5 Undrained Triaxial Tests (UU), no.2 for isotropic consolidation test (CID), no.3 oedometer tests, no.5 resonant column tests (RC).

Soil profile and in situ tests

To determine the geological profile and the geotechnical characteristics of the soil, in-situ tests such as boreholes, cone penetration tests and down-hole seismic tests will be carried out. As far as concerns the boreholes, Figures 16-18 show the soil profiles versus depth for the no.3 boreholes (S1, S2, S3). The profile shows that the underground soil is constituted mainly by clayey-silt and silty-clay up to a depth of 35-40 meters from ground surface. The water table, determined by piezometers, is located at around 1.5 meters below the ground surface.

Among the in-situ tests the down-hole seismic tests DH (3 tests) have been performed inside the no.3 boreholes S1, S2, S3, the cone penetration dissipation tests CPTU (4 tests) have been performed up to a depth of 60 meters, with the pore pressure measurement for the dissipation tests. In particular, starting from the ground level, the following layer have been found:

- **Borehole S1**
  A superficial thin layer from ground level to 3 m of sandy backfill, a thick layer (to 3÷10 m depth) of clayey silt and finally a layer (to 10÷40 m depth) of soft to medium silty clay with interlayers of thin lenses of silty sand (to 30÷40 m depth). The water table is located at around 1.20 and 7.70 meters below the ground surface.

- **Borehole S2**
  A superficial thin layer from ground level to 3 m of sandy backfill and a thick layer (to 7.5 ÷ 40 m depth) of soft to medium clay, normally consolidated. Thin lenses of silty sand and clayey silt have been encountered between 3 and 7.5 m of depth. The water table is located at around 1.30 and 24 meters below the ground surface.

- **Borehole S3**
  A superficial thin layer from ground level to 3 m of sandy backfill, thick layer (to 3÷37 m depth) of clayey silt with frequent interlayers of peat and thin lenses of silty sand. Finally a layer of soft to medium clay, normally consolidated from 37÷60 m depth.

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

Soil properties

The index properties and the mechanical characteristics of the soil have been evaluated from laboratory tests carried on undisturbed soil samples, with the aim to compare the values of the geotechnical parameters determined by laboratory tests with those derived from in situ tests. Due to the seismicity and to the geotechnical properties of the area, the soil deformability have been investigated both in static conditions by oedometer tests and in dynamic conditions by resonant column tests.
The index properties and the mechanical characteristics of the soil foundation derived from the laboratory tests are shown, as function of depth, in Table 3. A good correspondence between the values of the geotechnical parameters derived from the laboratory tests and those derived by the results of the in situ tests was observed.

The index tests classified the soil as clayey-silt and silty-clay with the following average parameters: liquidity limit $w_l$ varies from 40 up to 70%, plasticity limit $w_p$ is about 35%, consistence index $I_c$ varies from 0.4 up to 1.0. The values of the natural moisture content $w_n$ prevalently range between 30 and 50% as depth increasing, while the soil unit weight is equal around to 17 kN/m$^3$.

Table 3. Geotechnical soil properties by laboratory tests

<table>
<thead>
<tr>
<th>Tests</th>
<th>Depth (m)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$G_s$ (kN/m$^3$)</th>
<th>$w_r$ (%)</th>
<th>$w_l$ (%)</th>
<th>$w_p$ (%)</th>
<th>$I_p$</th>
<th>$I_L$</th>
<th>$I_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 C1</td>
<td>5.50 - 6.00</td>
<td>17.93</td>
<td>2.65</td>
<td>31.87</td>
<td>34.37</td>
<td>21.66</td>
<td>12.71</td>
<td>0.80</td>
<td>0.20</td>
</tr>
<tr>
<td>S1 C2</td>
<td>11.10 - 11.60</td>
<td>17.51</td>
<td>2.60</td>
<td>48.65</td>
<td>59.60</td>
<td>36.29</td>
<td>23.32</td>
<td>0.53</td>
<td>0.47</td>
</tr>
<tr>
<td>S1 C3</td>
<td>18.00 - 18.50</td>
<td>16.46</td>
<td>2.61</td>
<td>58.50</td>
<td>72.00</td>
<td>39.87</td>
<td>32.16</td>
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<td>18.01</td>
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</table>

Due to the peculiarity of the geotechnical problem, the vertical consolidation was studied by the oedometer tests and the characteristic values, reported in Table 4, were determined. In the interval of interest the oedometer modulus $E_{ed}$ is ranging between about 3800 up to 6400 kPa as depth increasing, the consolidation coefficient $C_v$ is ranging between $1.3 \times 10^{-6}$ and $2.0 \times 10^{-7}$ m$^2$/sec. These values are in good agreement with the values evaluated with dissipation tests.

Dynamic Geotechnical characterization

For the dynamic characterization of the soil foundation, resonant column tests have been performed. Resonant column tests have been carried out with a fixed-free equipment with an electric motor provided with 4 magnets and 8 windings having equal maximum ability of 1.1 Nm, connected directly on the head of test. The experimental apparatus is constituted by a signal generator connected to a power amplifier, the sensors for the measurement and an the system for the acquisition of experimental data.

The column resonant tests have been carried out in drained conditions on soil samples of 50 mm diameter and 100 mm height consolidated to the geostatic vertical pressure. The resonance condition has been obtained like the maximum value of the acceleration of a frequencies ranging. The resonant column tests results are provided in terms of strength deformation initial modulus $(G_o)$ and the dumping. Also, the decreasing of shear modulus and the increasing of damping with shear strength, have been plotted as $G$-$log \gamma$ and $D$-$log \gamma$ curves (Figures 6 and 7).
The initial shear modulus $G_0$ evaluated by resonant column test and that evaluated through the theory of elasticity by Vs Down Hole measurements are in good agreement.

<table>
<thead>
<tr>
<th>Tests</th>
<th>Depth (m)</th>
<th>$C_v$ (m²/sec)</th>
<th>$E_{ad}$ (kPa)</th>
</tr>
</thead>
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<td>6317.64</td>
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<td>9.99E-07</td>
<td>3854.05</td>
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</tbody>
</table>

Figure 6. Shear modulus $G_o$ and damping $D$ versus shear strength $\gamma$ for soil sample S2 C1.

Figure 7. Shear modulus $G_o$ and damping $D$ versus shear strength $\gamma$ for soil sample S2 C2.

**CONCLUSIONS**

A review of the results of the geotechnical investigations carried out for the construction of an industrial building (M6) in the area of Catania (Sicily, Italy) is described. The geotechnical characterization of the Catania area by in situ and laboratory tests has a great importance because the east coast of Sicily is considered as one of the zones of Italy with greater high seismic risk, basing on the past and current
seismic history and on the typology of civil buildings and industrial activities. The knowledge of soil dynamic properties gives the possibility to preview the soil behavior during the seismic events. With this aim static and dynamic parameters obtained by in situ and laboratory tests were analyzed and compared.

Figure 8. Shear waves velocity profile ($V_s$) obtained from Down Hole (a) and SCPTU (b) tests.

REFERENCES


