Geotechnical characterization of the Aterno valley for site response analyses

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Summary
The paper reports about the in-situ and laboratory tests executed in the aftermath of the April 6, 2009 L’Aquila earthquake, aimed at contributing to the definition of geotechnical models for site response analyses in the Aterno valley. The investigation started immediately after the earthquake and was entrusted by the Civil Protection Department to different public institutions and private companies; the experimental activities here reported were coordinated by the Italian Geotechnical Society.

In the paper, after briefly depicting the geological setting of the L’Aquila basin and the Aterno river valley, an overview of their subsoil conditions is given, based on the geotechnical tests executed before the 2009 earthquake. Thereafter, the specific investigations planned for the characterization of reconstruction sites and seismic microzonation are widely described. The in situ tests consisted of seismic dilatometer and multi-receiver surface wave tests; the resulting shear wave velocity profiles were compared each other, and against independent Down-Hole tests and ambient noise measurements. The laboratory investigation included cyclic/dynamic simple/torsional shear tests on undisturbed samples. The results, summarized in terms of variation of stiffness and damping with shear strain, highlighted some peculiar properties of the soils in the Aterno valley, which could be related to their index properties.

1. Geological setting of the L’Aquila basin and the Aterno river valley
The area affected by the April 6th, 2009 earthquake is located within the central section of the Apennines, the mountain chain which traverses most of the length of the Italian peninsula. The city of L’Aquila and most of the small towns damaged by the earthquake lie in a vast intra-Apennine tectonic basin, elongated in NW-SE direction, parallel to many of the active normal faults and surrounded by the high peaks of the Gran Sasso and the Velino-Sirente mountains. These chains are mostly constituted by Meso-Cenozoic carbonate rocks and occasionally by marly-arenaceous rocks.

The Aterno River is the main hydrographic element of the basin; the middle Aterno river valley is about 10 km wide and is elongated in the NW-SE direction for more than 15 km. The current geological setting of the L’Aquila basin results from a complex sequence of depositional events, due to erosion and tectonics.

The geological setting of the L’Aquila basin is illustrated in the geological map and the schematic section drawn in figure 1.

The bedrock consists of Meso-Cenozoic limestone formations, generally outcropping along the sides of the valley and on ridges located within the Aterno River basin.

The bottom of the valley was filled during the Quaternary with continental deposits of variable genesis and deposition age, resulting from lacustrine to subsequent fluvial sedimentations. The maximum thickness of the Quaternary deposits is estimated as high as 400-500 m.

The older Pleistocene lacustrine deposits, placed on the calcareous bedrock, form a complex depositional sequence of silt, sand and conglomerate
units [Bosi and Bertini, 1970; Bertini et al., 1989]. In detail, three different units may be distinguished within the lacustrine formation:
- an older unit at the bottom (placed on the bedrock) of highly variable composition, mostly gravels, sands and clays in variable combination;
- an intermediate unit, predominantly consisting of gravels and sands;
- a more recent unit at the top, mainly consisting of sands and clays, having a thickness of some tens of meters.

The recent alluvial Holocene soils, placed on the top of the Meso-Cenozoic and Pleistocene deposits, consist of sands and cobbles, while sands and silts are sometimes found. The foot of the valley flanks and the ridges located within the valley are covered by talus debris and, locally, by large debris alluvial fans. The relationship among the sedimentary styles is very complex, due to the interplay of tectonics and climate changes.

The area on the left side of the Aterno River basin (drainage from NW to SE), where the old town
center of L’Aquila is located, is characterized by the presence of vast deposits of Pleistocene heterometric breccias (associated with Quaternary paleolandslides), known as “Megabrecias” or “Brecce dell’Aquila”, overlying the lacustrine sequence placed on the bedrock. The “Megabrecias” consist of limestone boulders and alluvial clasts with a size from few centimeters to few meters, embedded in sandy-silty matrix having a highly variable degree of cementation.

2. Geotechnical properties from routine in situ and laboratory tests

The collection and analysis of existing data from previous geotechnical investigations, typically carried out by routine in situ and laboratory tests, was the primary step in planning the post-earthquake site investigation programs. Particular attention was devoted to possible sites for new temporary housing and to seismic microzonation, as well as to public buildings damaged by the earthquake.

Soon after the main shock of the seismic sequence, a large amount of data from previous investigations carried out in the area of L’Aquila was collected by the Civil Protection Department (DPC) and organized into an on-line database, which was made available to decision makers and investigators.

A preliminary selection of results of in situ and laboratory tests, collected throughout the past years by the geotechnical group of the University of L’Aquila [Bertini et al., 1992], points out that the Quaternary deposits of the L’Aquila basin are characterized by a significant heterogeneity.

Figure 2 shows the grain size distribution (Fig. 2a) and the plasticity properties (Fig. 2b) of numerous soil samples from the lacustrine formation in the upper part of the Aterno valley. The variability of the grain size distribution is confirmed by the material index, $I_d$, evaluated by flat dilatometer tests (DMT) carried out in the same formation, at two locations close to the river (Fig. 3). The layering is characterised by a cover of silty-sandy soil overlying a finer clayey deposit. In the same figure, the mechanical parameters usually evaluated by DMT test (the constrained modulus $M_c$, the undrained shear strength $c_u$ and the horizontal stress index $K_d$, related to OCR) show a gradual but irregular increase with depth. The profiles of the shear wave velocity, $V_s$, estimated from empirical correlations with DMT [Monaco et al., 2009], show that $V_s$ values seldom exceed 400 m/s, even at the largest investigated depth, that is 14 m in this case.

The Holocene alluvial deposits lying in the bottom part of the Aterno valley are also characterised by a remarkable heterogeneity. As an example, figure 4 compares the profiles of tip resistance, $q_t$, resulting from 25 cone penetration tests (CPT) carried out along the banks of the Aterno River from Onna to Fossa. Below the first 4 m depth, the profiles become very irregular; the relatively low $q_t$ values also indicate the poor geotechnical properties of the river bank deposits, which could contribute to explain the high concentration of damage caused by the earthquake in this zone, including the collapse of a R.C. bridge near Fossa [Lanzo et al., 2010].

A typical stratigraphic profile in the “Megabrecias” formation, as retrieved from a borehole drilled in the historical centre of L’Aquila (Piazza del Tea-
is reported in figure 5, together with the 

\begin{align*}
V_s \text{ profile obtained by a seismic dilatometer test (SDMT) carried out in the borehole after backfilling with sand.}
\end{align*}

\(V_s\) measurements by SDMT in backfilled boreholes are normally taken every 0.50 m of depth. The “gaps” in the \(V_s\) profile in Figure 5b, as well as in figure 6 ahead, correspond to depth intervals greater than 0.50 m in which \(V_s\) measurements were missed during testing. The shear wave velocity increases with depth from about 400 m/s up to 1600 m/s, evidencing the existence of a weathered cover in the first 10-15 m. Some boreholes executed in the centre of L’Aquila retrieved layers of fill material, having thickness of few meters; most of these man-made fills are supposed to be the disposal of rubbles of masonry buildings destroyed by past earthquakes [Monaco et al., 2012].

3. Shear wave velocity from in situ tests

The emergency scenario imposed very strict time constraints for decision on reconstruction. Few weeks after the mainshock, the Civil Protection Department identified about tentative 20 sites to locate the first temporary houses for homeless people.

These buildings (C.A.S.E. project) were conceived to be seismically isolated, so that one of the requirements to be satisfied for the definitive location was that the natural frequency of the subsoil should be larger than 0.5 Hz. This was verified by ambient noise measurements and by performing 1-D linear equivalent site response analyses (e.g., Evangelista et al., 2011). For an accurate yet quick dynamic subsoil characterization of such sites, surface wave tests were planned in all of them. Downhole (DH) and laboratory tests were carried out only in sites with apparently deep seismic bedrock. A task force of the Italian Geotechnical Society (AGI) was committed to investigate several of such sites by seismic dilatometer and surface wave tests (see Fig. 1a). The testing techniques and some representative results will be reported in the following.
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3.1. Seismic dilatometer: testing and interpretation procedures

Seismic dilatometer tests (SDMT) (e.g., Marchetti et al., 2008) are executed employing, as a wave source, a pendulum hammer (\( \approx 10 \) kg) that hits horizontally a steel rectangular plate, pressed against the soil by the weight of a truck. Two horizontal receivers, assembled with a spacing of 0.50 m along the DMT blade, can record the shear wave arrival. The \( V_S \) measurements are typically taken every 0.50 m of depth, while the mechanical DMT readings are taken every 0.20 m. The \( V_S \) value is obtained as the ratio between the difference in distance between the two receivers and in time between the seismograms at the first and the second receiver. The latter is computed through a cross-correlation algorithm and systematically checked through the visual inspection of the recorded tracks. The test procedure proves to be an effective, quick and cost-saving alternative to conventional DH tests in loose and soft soils; however, \( V_S \) profiles can be obtained also in non-penetrable soils by inserting the SDMT blade in a preliminary drilled and backfilled borehole. In this case, the other DMT parameters are not measured because they would be meaningless in the backfill soil. Details of the technique can be found in Totani et al. [2009].

As a general rule, it is advisable to measure directly \( V_S \); however, at sites where only mechanical DMTs have been executed, the small strain shear modulus \( G_0 \) (hence \( V_S \)) can be estimated with empirical correlations based on the parameters \( I_D, K_D, M_{DMT} \) [Monaco et al., 2009].

Fig. 4 – a) Location and b) profiles of the cone penetration resistance \( q_c \) from 25 CPT tests carried out along the left hand side (PS) and right hand side (PD) banks of the Aterno River from Onna to Fossa (after Monaco et al., 2012).

Fig. 4 – a) Ubicazione e b) profili della resistenza penetrometrica \( q_c \) da 25 prove CPT condotte lungo le rive destra (PD) e sinistra (PS) del fiume Aterno fra Onna e Fossa (da Monaco et al., 2012).

Fig. 5 – a) Typical stratigraphic profile of the subsoil (“Megabrecias”) in the town center of L’Aquila; b) SDMT measurements of \( V_S \) in the backfilled borehole (after Totani et al., 2009).

Fig. 5 – a) Tipico profilo stratigrafico del sottosuolo (“Megabrecia”) presente nel centro della città di L’Aquila. b) Profilo di \( V_S \) da prova SDMT (da Totani et al., 2009).
The SDMT can be also helpful to estimate the decay of the equivalent shear modulus $G$ with shear strain $\gamma$, as it will be shown at the end of the paper.

3.2. SDMT results in the Aterno valley

The tests were carried out at 5 locations (Pettino, Cese di Preturo, Sant’Antonio, Roio Piano and Pianola, see Figure 1a) in the first months following the earthquake [Università dell’Aquila, 2009].

Figure 6a shows the profiles of DMT parameters and $V_s$ for a site (Pettino) where a silty cover about 8 m thick overlies a thick gravelly layer; most of the subsoil was therefore non-penetrable and the borehole backfilling procedure was followed. As in the case shown in figure 5, the $V_s$ measured in the gravel is higher than 400 m/s, reaching values in the order of 1000 m/s for a depth greater than 15 m.

The results in figure 6b were gathered at Pianola, one of the sites of the C.A.S.E. project, where a 5 m silty cover overlies a 20 m thick layer of sandy soil and deep bedrock is expected. The measured $V_s$ values increase with depth and exceed 400 m/s at the largest investigated depths. For both sites a good agreement exists between the $V_s$ profile directly measured by SDMT with that estimated from mechanical DMT, using the correlation by Monaco et al. [2009]. For the Pianola site, the comparison among the $V_s$ profiles by SDMT and by other techniques (surface waves and Down-Hole tests) is also satisfactory, as discussed in details by Monaco et al. [2012].

Fig. 6 – SDMT results at a) Pettino and b) Pianola sites.

Fig. 6 – Risultati di prove SDMT condotte presso i siti di a) Pettino e b) Pianola.
3.3. Surface waves: testing and interpretation procedures

The Surface Wave Method (SWM) is a cost- and time-effective technique for evaluating the shear wave velocity profile. It is based on the dispersion of surface waves in vertically heterogeneous media: the velocity of each harmonic component of the surface wave depends on the properties of the medium affected by the wave propagation and its penetration depth is proportional to the wavelength. An experimental ‘dispersion curve’ (velocity versus wavelength or frequency) can be extracted from field data using several processing techniques (e.g., SOCCO and STROBBIA, 2004; FOTTI, 2005; EVANGELISTA, 2009).

The shear wave velocity profile can be inferred by solving an inverse problem, based on the minimization of the distance between the theoretical dispersion curve and the experimental one.

In the investigated sites, multi-receiver surface wave (MASW) tests were performed using both active and passive acquisition techniques. The multi-station active layout used harmonic (f>5Hz) or impulsive sources and linear receiver arrays; the passive layout (f<10Hz) used two-dimensional receiver arrays, necessary to determine the direction of the dominant wave and consequently its velocity. Dispersion curves were obtained as the amplitude peaks in the f-k (frequency-wavenumber) domain, where different curve branches identify the multi-modal response of the layered subsoil [ZWICKI, 1999].

The inversion procedure was based on the Haskell-Thomson matrix method for the solution of the forward problem [THOMSON, 1950; HASKELL, 1953]. For an elastic horizontally layered medium, the dispersion curve corresponds to the zeros in the velocity-frequency domain of the Haskell-Thomson matrix determinant, which depends on model parameters (layer thickness, shear wave velocity, density, Poisson’s ratio). In figure 7 the shape of the absolute value of the Haskell-Thomson matrix determinant for a given profile is shown.

A Monte Carlo algorithm was used to perform the inversion of the experimental data [MARSCHINI and FOTTI, 2010]. This algorithm evaluates the misfit of a set of soil profiles randomly generated between boundaries chosen by the user from the experimental dispersion curve. The misfit function is the L1 norm of the vector of the Haskell-Thomson determinant, evaluated in correspondence of the experimental dispersion curve. This misfit function allows higher modes of propagation of Rayleigh wave to be properly taken into account with low computational cost [MARSCHINI et al., 2010].

Moreover, with the adoption of a stochastic approach, it is possible to appreciate the uncertainty associated to non-uniqueness of the solution, which is typical of any method based on the solution of an inverse problem. A discussion of the consequences of such uncertainty on seismic site response can be found in FOTTI et al. [2009].

For the present study, the inversion procedure was carried out as a ‘quick and blind prediction’, i.e. with no knowledge of the layering from the boreholes which, in most cases, were afterwards executed nearby.

A first validation of the MASW shear wave velocity profiles was possible by means of the DH and SDMT tests subsequently available in some of these sites. A further validation was attempted using single station measurements of microtremors. Indeed,
the experimental horizontal to vertical spectral ratio (HVSR) of ambient noise provides a reasonable estimate of the fundamental natural frequency of the site. This can be compared to the fundamental frequency computed for the same subsoil, modeled as a visco-elastic linear layered medium with the MASW velocity profile. To compute such transfer function, the depth ($z_B$) and velocity ($V_{S,B}$) of the stiffest and deepest layer were taken, in an approximate way, as conventional bedrock parameters.

3.4. Surface Wave Tests in the Aterno Valley

The MASW tests in the Aterno valley were carried out in the six C.A.S.E. sites shown in figure 1a; geological surveys and instrumental records of aftershocks and microtremors permitted a preliminary classification of the sites with apparently shallow and deep bedrock. Therefore, for three ‘shallow bedrock’ sites (Bazzano, San Giacomo, Sant’Antonio), only active MASW tests were carried out; for the ‘deep bedrock’ sites (Roio Piano, Pianola, Il Moro), passive measurements were also used to reach higher investigation depths.

A synthesis of the experimental data collected on the six C.A.S.E. sites is shown in table I. In three cases, a nearby Down-Hole test was available and reliable; at the sites of Roio Piano and Pianola, however, the DH did not detect seismic bedrock because of the limited extension of the boreholes, while the MASW tests were able to investigate down to higher depths ($z_{max}$ in the table) and identified a seismic bedrock.

The table shows that the experimental peak frequency ($f_{HVSR}$) and the fundamental frequency computed analytically on the basis of the MASW velocity profiles ($f_{MASW}$) are in a good agreement. In the following, two representative examples (one with shallow the other with deep bedrock) will be reported; further details are given in Boiero et al. [2009] and d’Onofrio et al. [2010].

3.4.1. Sites with Deep Bedrock

At the ‘deep bedrock’ sites, investigated by the researchers of the Politecnico di Torino, the integration of active and passive data allowed dispersion curves to be retrieved over a broad frequency range; thus, the investigated depth was extended without losing resolution in shallow layers.

Specifically, in Roio Piano the subsoil is characterized by colluvial deposits and debris covers overlying sandy and silty layers; the latter lie on a stiff calcareous bedrock with variable depth [Tallini et al., 2009]. Figure 8 shows three branches of the experimental dispersion curve; two of them result from the active-source acquisitions, using a linear receiver array, and overlap to the passive data in the common frequency region (5-10 Hz). The whole experimental dispersion curve was inverted using the multi-modal Monte Carlo algorithm described above. The inversion was performed using a population of $10^6$ randomly generated profiles.

In figure 9a a set of twenty trial profiles is plotted, including the best-fitting profile (which corresponds to the darkest color). The Haskell-Thomson matrix determinant of the latter is compared to the experimental dispersion curves in figure 9b. The higher velocity branch of the experimental dispersion curve is associated to the first higher mode.
The shear wave velocity profiles, obtained considering two different source positions in the active tests (forward and reverse shots), are compared in Figure 10 with the layering and the $V_s$ profiles from the Down-Hole (DH) and the seismic dilatometer (SDMT) tests carried out nearby. It can be noted that MASW, DH and SDMT test results are in very good agreement with each other; the surface wave test allows a deeper velocity profile to be retrieved, detecting a seismic bedrock with $V_s = 800$ m/s at depth of 40-45 m.

The 1-D amplification function computed using the average $V_s$ profile resulting from the MASW test is plotted in Figure 11; the shaded area represents the frequency range with $H/V$ spectral ratio of microtremors higher than 2, as required by the SESEAME [2004] guidelines to identify a clear resonance peak [MS-AQ WORKING GROUP, 2010]. The fundamental frequency $f_{MASW}$ is about 3.1 Hz, close to the peak frequency of the experimental records, $f_{HVSR} = 2.8$ Hz.

3.4.2. Sites with shallow bedrock

The ‘shallow bedrock’ sites were investigated by a joint research team of Universities of Molise and...
Naples, operating with high-frequency active MASW acquisition setup. The source of surface waves was an electro-mechanical shaker controlled by a function generator, producing a harmonic force in the frequency range of 5-120 Hz. The receivers are 14 piezo-electric 1-D accelerometers with a wide dynamic range (0.1-300 Hz), arranged in a linear array with a total length of 29 m. EVANGELISTA [2009] reports a detailed description of the test procedures and results in the three selected locations.

At the Bazzano site, the geological survey reports a sandstone formation locally covered by alluvial deposits [CAVUTO and MOCATELLI, 2009]. The MASW test was executed at the transition between the outcropping sandstone and the alluvial cover. The experimental shear wave velocity profile identifies a stiff stratum at a depth of 8 m (Fig. 12). In this case, however, no borehole was available in the vicinity to confirm this result.

Figure 13 shows the amplification function of the subsoil model corresponding to the MASW profile. The range of frequencies with H/V spectral ratio higher than 2 recorded during aftershocks [MS-AQ WORKING GROUP, 2010] correspond exactly to that computed from the shear wave velocity profile, assuming the soil below 8 m from the ground level as a bedrock. This assumption was made considering the lack of deeper data and especially because of the large seismic impedance ratio between the alluvium and the sandstone.
that could explain the high frequency measured by H/V spectral ratio. Both experimental and numerical frequencies are about 8 Hz (Tab. I); such consistency reciprocally validates the different measurements.

4. Stiffness and damping from laboratory tests

For the C.A.S.E. project, cyclic and dynamic laboratory tests were carried out on 17 undisturbed samples retrieved in eight sites. A network of five soil dynamics laboratories was involved: ISMGEO (Bergamo), Politecnico di Torino, Universities of Florence, Naples Federico II and Rome ‘La Sapienza’.

The testing programme, summarized in table II, consisted of standard classification tests, one-dimensional compression tests, cyclic-dynamic torsional shear tests (RC–CTS) and double specimen direct simple shear tests (DSDSS). Cyclic and dynamic torsional shear tests are the most common tool employed in earthquake geotechnical engineering to detect the relationship between the shear modulus and the damping ratio and the shear strain. As for the DSDSS apparatus, two specimens of the same soil, rather than one, are sheared simultaneously in simple shear mode. Due to its specific configuration and to the large stiffness

Fig. 13 – Site response at Bazzano: comparison between the linear numerical amplification function and the frequency range defined by SESAME criteria on the experimental spectral H/V ratio.

Fig. 13 – Risposta in frequenza per il sito di Roio Piano: confronto fra la funzione di amplificazione numerica in ipotesi di comportamento del terreno visco-elastico lineare ed il campo di frequenze sperimentali definito, in accordo con i criteri SESAME, attraverso il rapporto spettrale H/V.

Tab. II – Physical properties of the tested soils.

<table>
<thead>
<tr>
<th>Sample</th>
<th>depth (m)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$e_0$ (%)</th>
<th>$w$ (%)</th>
<th>Ip (%)</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
<th>Laboratory</th>
<th>Tests</th>
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<tr>
<td>Cese S3-C1</td>
<td>4.0-4.8</td>
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<td>0.752</td>
<td>26.1</td>
<td>36.7</td>
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<td>59</td>
<td>37</td>
<td>57</td>
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<td>RCCTS</td>
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<td>26.0</td>
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<td>RCCTS</td>
</tr>
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<td>40</td>
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<td></td>
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<td>RCCTS</td>
</tr>
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<td>1.058</td>
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<td>27.1</td>
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<td>36</td>
<td></td>
<td>Naples</td>
<td>RCCTS</td>
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<td>25.1</td>
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<td>RCCTS</td>
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<td>RCCTS</td>
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<td>RCCTS</td>
</tr>
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<td>14.0</td>
<td>81</td>
<td>19</td>
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<td></td>
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<td>DSDSS</td>
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<td>32.9</td>
<td>20</td>
<td>28</td>
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<td>RCCTS</td>
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<td>Tempera S1-C2</td>
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<td>1.167</td>
<td>39.4</td>
<td>28.0</td>
<td>18</td>
<td>52</td>
<td>30</td>
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<td>19.0</td>
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<td>DSDSS</td>
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<td>0.616</td>
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<td>60</td>
<td>33</td>
<td>7</td>
<td></td>
<td></td>
<td>Rome</td>
<td>DSDSS</td>
</tr>
<tr>
<td>Camarda S1-C1</td>
<td>4.5-5.0</td>
<td>14.50</td>
<td>1.957</td>
<td>57.2</td>
<td>15.6</td>
<td>1</td>
<td>51</td>
<td>41</td>
<td>7</td>
<td>Turin</td>
<td>RCCTS</td>
</tr>
</tbody>
</table>
of the device components, all the problems associated with false deformations and system compliance were reduced to negligible levels, thus enabling the measurement of soil properties even at very small strains [DOROUDEIAN and VUCETIC 1995; LANZO et al., 2009].

In the following, a summary of the experimental data is reported and some comparisons between the non-linear geotechnical properties evaluated with different techniques are shown.

4.1 Physical properties

The values of the main index properties are summarized in table II, while the grain size distributions are shown in figure 14. Most of the samples are fine-grained soils, classifiable as silty clays to sandy silts. Those retrieved at Tempera (S1-C3), Pianola (S1-C1) and Camarda (S1-C1) are silty sands. The range of grain size distribution shows a lower percentage of clayey material with respect to the data collected by previous investigations (see again Fig. 2).

According to the particle size distribution, the tested samples can be classified into two main groups on the basis of clay fraction (CF): lower than 30% and higher than 30% (respectively grey and black curves in figure 14). The Atterberg limits measured on the same samples show a lower percentage of clayey material with respect to the data collected by previous investigations (see again Fig. 2).

4.2. Non-linear stiffness and damping

Soil non-linear behaviour was analyzed by means of fixed-free Resonant Column/Torsional Shear (RC-CTS) devices and a Double Specimen Direct Simple Shear (DSDSS) apparatus. Details on the testing techniques and the whole collection of the data are reported by MS-AQ WORKING GROUP [2010].

The specimens were consolidated, either isotropically (RC-CTS tests) or one-dimensionally (DSDSS tests) to the estimated in situ stress. At the end of the consolidation stage, the cyclic and/or dynamic tests were performed with increasing shear load levels, to investigate the behaviour of the soils for shear strains spanning between 0.0001% and 1%. As usual, the tests were interpreted in terms of linear equivalent parameters, i.e. shear modulus, \( G \), and damping ratio, \( D \).

Figure 16 shows the experimental results obtained from the resonant column tests in terms of normalized shear modulus, \( G/G_0 \) (Figs. 16a-b), and
damping ratio, $D$ (Figs. 16c-d), versus shear strain, $\gamma$. In the same figure cyclic torsional shear rather than resonant column tests are reported for Camada 1-C1 and Roio Piano S3-C4 specimens. The data are plotted with the same symbols and color code used for the grain size distribution curves and for the plasticity chart. In the same plots, the curves suggested by Darendeli [2001] for soils with plasticity indexes equal to 15%, 30% and 100% are reported for comparison. The curves proposed by Darendeli for mean effective confining pressure $\sigma'_p$ equal to 100 and 400 kPa were used for soils tested in the range 75-150 kPa and 200-500 kPa, respectively.

The comparison among the different groups of samples consistently reflected their differences in physical properties, confirming that clay fraction and plasticity index are key parameters to represent soil non-linearity. In fact, the grey curves, relevant to silty - low plasticity soils, define a range of linear behavior not exceeding a threshold strain level of 0.005%, beyond which the decay of stiffness (Figs. 16a-c) and the increase of damping (Figs. 16b-d) are quite pronounced. On the other hand, the black curves (clayey – high plasticity soils) are characterized by higher values of the linear threshold strain level of the order of 0.005%, beyond which the decay of stiffness (Figs. 16a-c) and the increase of damping (Figs. 16b-d) are quite pronounced. On the other hand, the black curves (clayey – high plasticity soils) are characterized by higher values of the linear threshold strain level of the order of 0.001%, showing a less evident reduction of stiffness (Figs. 16a-c) and lower damping values (Figs. 16b-d) in the non-linear range.

Note that the overall trends of the two groups of samples do not exactly overlap the average literature curves pertaining to low ($I_p=15\%$) and medium ($I_p=30\%$) plasticity soils for $\sigma'_p=100$ kPa (Figs. 16a-b). In particular the curves of the tested soils are shifted rightward with respect to literature ones, i.e. the soils show a more linear and less dissipative behavior. On the contrary, the curves for $\sigma'_p=400$ kPa (Figs. 16c-d) better approximate the experimental behavior of soils characterized by similar plasticity characteristics. An exception is constituted by the peculiar behaviour shown by the organic samples for which the influence of fabric cannot be captured by the ‘standard’ literature curves [Page et al., 2013]. For instance, the finest organic sample, taken at Roio Piano site in Figs. 16c-d (S3-C8, $CL=48\%$, $w_0=90\%$, $I_p=33\%$) shows a non-linear behaviour comparable to that described by the standard literature curves relevant to highly plastic soils ($I_p=100\%$), rather than those relevant to medium plasticity ($I_p=30\%$). Moreover, the large initial damping ratio exhibited by the Pagliare di Sassa S2-C1 specimen could be due to the relatively low confining stress employed in executing the test.

The non-linear behaviour of stiffness and damping obtained by DSDSS tests on four silty - low plasticity samples is summarized in figure 17. As shown for RC tests, the cyclic behavior is not satisfactorily captured by Darendeli curves at $\sigma'_p=100$ kPa being experimental data shifted downward. Right both in terms of normalized stiffness modulus and damping ratio (Figs. 17a-b). At $\sigma'_p=400$ kPa, the results of the soil sample taken Tempera site in Figs. 17(c-d) (S1-C3, $CL=32\%$, $w_0=37\%$, $I_p=19\%$) shows a cyclic behavior well reproduced by Darendeli curves relevant to highly plastic soils ($I_p=100\%$). This could be ascribed to the presence of relevant organic matter content.

The samples subjected to DSDSS tests show linear threshold strains lower than 0.005% and more gradual variations of the equivalent parameters with $\gamma$, if compared to those measured by RC tests on soils of comparable plasticity and grain size. This evidence could be attributed to the effects of loading rate and the number of cycles, being the DSDSS frequency (about 0.3 Hz) and number of cycles much lower than those relevant to RC tests.

This hypothesis is further confirmed by the plots in figure 18, which compare the stiffness and damping measured on a single specimen of the clayey – high plasticity group, during the RC test and series of CTS tests driven at variable frequency (0.5, 1, 2 Hz). The non-linear stiffness and damping curves (Fig. 18a) approximately identify the same linear strain range, but the decay of the shear modulus in the RC test is clearer than that defined by all CTS tests; this could be probably due to material degradation associated to the higher number of cycles in the RC test.

The influence of frequency on the small-strain stiffness ($G_0$) and damping ($D_0$) is demonstrated in figure 18b, where the values of $G_0$ and $D_0$ measured during CTS and RC tests are plotted against the logarithm of the frequency. The shear modulus shows a continuous 10% increase for log cycle of frequency, while the initial damping is characterized by a low rate-dependency below 1 Hz and by a sharper increment for higher frequencies. This rate-dependent
behaviour was frequently observed in literature on medium to high plasticity silts and clays (e.g., Shibuya et al., 1995; d’Onofrio et al., 1999) and should be properly taken into account for site response analyses. As a matter of fact, considering that the expected dominant frequency of the reference input signals generally falls below 10 Hz, the choice of RC test results might lead to errors in computing the amplification function at a given site.

The availability of both field and laboratory tests in the C.A.S.E. sites made also possible some calibration of empirical estimates of non-linear parameters from SDMT. A ‘working strain shear modulus’, $G_{DMT}$, was derived from the constrained modulus, $M_{DMT}$, obtained from the usual DMT interpretation. It is assumed that $M_{DMT}$ is a reasonable ‘working strain’ modulus (i.e. the modulus that, introduced into the linear elasticity formulae, provides realistic estimates of the settlement of a shallow foundation under working load). This assumption is supported by the good agreement observed in a large number of well documented comparisons between measured and DMT-predicted settlements or moduli [Marchetti et al., 2008]. As a first approximation, by referring to linear elasticity:

The ‘working strain modulus’ was evaluated with the above procedure from SDMT carried out at Ro-
Geotechnical characterization of the Aterno Valley for the site response analyses.

Table III reports the measured values of shear wave velocity (hence, the field evaluation of $G_0$) and constrained modulus at the depth of the samples tested in the laboratory. A Poisson’s ratio $v$ equal to 0.2 was used to calculate the working strain shear modulus, given by $G_{DMT} = 0.375 \cdot M_{DMT}$ as resulting from Eq. (1). The values of the ‘normalized working strain shear modulus’ $G_{DMT}/G_0$, also reported in table III, fall in the range ±0.1 to 0.4 (0.10 to 0.23 in silt and clay, 0.37 in silty sand).

In figure 19 each $G_{DMT}/G_0$ data point (grey symbols) is superimposed on the corresponding same-depth laboratory $G/G_0$ curve. The range of values of the shear strain $\gamma_{DMT}$ inferred from the “intersection” of the $G_{DMT}/G_0$ data points with the laboratory curves (rectangular areas in Fig. 19), also reported in

Fig. 16 – Normalised shear modulus (a-c) and damping ratio (b-d) versus shear strain from RC or CTS tests compared with literature curves by Darenelli [2001] for $\sigma_0 = 100$ kPa (a-b) and 400 kPa (c-d).

Fig. 16 – Modulo di taglio normalizzato (a-b) e rapporto di smorzamento (c-d) in funzione della deformazione tangenziale da prove RC o da prove CTS confrontate con le curve proposte da Darenelli [2001] per le pressioni di confinamento di 100 kPa (a-b) e 400 kPa (c-d).
This result is in agreement with the indications by Ishihara [2001], who classified SDMT test among the medium strain measurement methods, i.e. providing estimates of the soil deformation parameters at a strain level \( \gamma \) ranging between 0.01 and 1%, as well as with “typical ranges” of shear strain associated to working strain moduli \( G_{DMT} \) in various soil types detected by Amoroso et al. [2012].

In order to explore how the choice of the Poisson’s ratio affects the result, the above procedure was repeated assuming \( \nu = 0.3 \). In this case the ‘normalized working strain shear modulus’ \( G_{DMT}/G_0 \) was found in the range 0.07-0.28 and the corresponding shear strain \( \gamma_{DMT} \) was found between 0.2 and 0.7%. This result indicates that the range of \( \gamma_{DMT} \) is not significantly affected by the assumed Poisson’s ratio.

5. Conclusions

An experimental geotechnical campaign was performed, by in-situ and laboratory tests, along the Aterno valley in the aftermath of the April 6th, 2009 L’Aquila earthquake.
This investigation allowed defining subsoil models for seismic response analyses, for the location of temporary housing and for urban re-planning.

In this paper, some of the obtained data are presented and scrutinized in an attempt to give a picture of the main soil properties relevant for ground response analysis and seismic microzonation. Particular attention was devoted to collecting and interpreting in-situ and laboratory tests near the epicentral area that was in the L’Aquila basin in the northern part of the Aterno river valley. Therefore, even quite complete, the investigation campaign does not encompass all the data that would be necessary for an advanced site response analysis of the whole Aterno valley.

In the analyzed area, the heterogeneity of the fluviolacustrine soils was already well known, before the earthquake, from previous investigations data. The new results permitted to improve significantly the static and dynamic characterization of the soils in the area.
The post-earthquake investigations included measurements of the shear wave velocity $V_s$ by different in situ testing techniques, i.e. Down-Hole tests (DH), surface wave tests (MASW) and seismic dilatometer tests (SDMT). Comparisons at various test sites, illustrated in the paper, indicate that MASW, DH and SDMT results are in very good agreement, with surface wave tests often allowing defining a deeper shear wave velocity profile. In lack of boreholes, MASW tests proved to be effective to assess the frequency response of the subsoil, recorded instrumentally by HVSIR method.

Undisturbed samples from various sites were subjected to cyclic and dynamic tests carried out by a network of research laboratories in Italy. The cyclic/dynamic shear tests – Resonant Column/Torsional Shear tests and Double Sample Direct Simple Shear tests were aimed at characterizing the non-linear and dissipative behaviour (stiffness and damping) of medium- to fine grained soils. The
The normalized shear modulus $G / G_0$ and damping ratio $D$ versus shear strain $\gamma$ obtained for the tested fine-grained materials, synthetically described in the paper, consistently reflect their different grain size distribution and plasticity. Reference literature curves do not overlap the non-linear overall trend of the experimental curves, in some cases highlighting the peculiar nature of the soils of the Aterno valley. The combined use of laboratory and SDMT tests to date appears promising to calibrate simplified procedures to estimate soil non-linear behaviour.

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References


Caratterizzazione geotecnica della valle dell’Aterno per analisi di risposta sismica locale

**Sommario**

L’articolo riassume i risultati di alcune prove in sito e in laboratorio, eseguite subito dopo il terremoto dell’Aquila del 6 aprile 2009, che hanno contribuito alla definizione di modelli geotecnici per analisi di risposta sismica locale nella valle dell’Aterno. Le attività di indagine sono state affidate dal Dipartimento della Protezione Civile a varie istituzioni pubbliche e private; le attività riportate in questo lavoro sono state coordinate dall’Associazione Geotecnica Italiana.

L’articolo, dopo un breve inquadramento geologico della conca dell’Aquila e della valle del fiume Aterno, riporta una panoramica delle condizioni di sottosuolo nell’area, ricavate da prove geotecniche eseguite prima del terremoto del 2009. In seguito, si descrivono le specifiche indagini programmate per la caratterizzazione dei siti da ricostruire e per la microzonazione sismica. Le prove in sito hanno comportato indagini con il dilatometro sismico e l’analisi multistazione delle onde superficiali; i relativi profili della velocità delle onde di taglio sono stati confrontati con i dati di prova Down-Hole e di misure di microtremori. Le indagini di laboratorio includono prove cicliche/dinamiche di taglio semplice/torsionali eseguite su campioni indisturbati. I risultati, sintetizzati in termini di variabili di rigidezza e smorzamento con la deformazione a taglio, evidenziano alcune proprietà peculiari dei terreni della valle dell’Aterno, che possono essere messe in relazione con le loro proprietà indice.