1 INTRODUCTION

In this paper the results of the geognostic investigation made for Centocelle Park station design (subway line C in Rome), the determination of pyroclastic soils deformability parameters interested by the excavation are shown. The investigations on site, made by seismic tests (RE.MI. Refraction Microtremor), have allowed an estimation of the elastic modulus in the field of small deformations (from 0.01% to 0.1%) that usually reached in the open pit station (in this case approximatively 28 meters). A comparison between the results obtained from traditional investigations and those obtained from seismic investigation (pressumeter tests, uniaxial compression test only for lithoid soils). The analysis of the behavior in exercise of the deep excavations is subordinate to the knowledge of elastic modulus at small deformations (G0 or E0). The numerous researches made on the behavior of soils at small strains [Rampello S. e Silvestri F., 1993; Stokoe et al., 1995], have shown G0 modulus, can have the meaning of "state parameter ", like the initial voids index e0, the earth pressure coefficient at rest K0 or the effective vertical lithostatic stress $\sigma'_{v0}$ [Ghionna et al., 2006]. From all above the necessity to define with enough approximation the course of the shear modulus G0 with depth for all layers interested from excavation. Its nature of "fundamental" parameter is better defined by relationship between itself and speed propagation of shear waves come from "elasticity" theory:

$$G_0 = \rho \cdot V_s^2$$  \hspace{1cm} (1)

in which $\rho$ is the soil density. The equation (1) allows to pass from data obtained by RE.MI., that is propagation speed of shear waves, to the stiffness parameter of reference.

From these considerations and from the results of seismic investigation, a numeric simulation of the excavation has been done to verify the stress-strain behavior of soil-structure group and to compare to that one obtained by measurement done during excavation phases.

2 PYROCLASTIC MATERIALS INTERESTED BY THE EXCAVATION OF CENTOCELLE PARK STATION

2.1 Geological profile

From a geologic point of view, the area interested by Centocelle Park station building is covered by alluvial bakfill under which there are the products of Colli Albani volcanic apparatus and “Sabatino” volcano that are much diffuse in the Est and South-Est part of Rome.
Soil succession includes the typical Roman pozzolanas, that are, for a great part, in benches of material with a weakly bonding degree, so much to be classified as incoherent, but that in some places have also facies with a great bonding degree (with a soft rock behavior). Finally, there are successions constituted by very frequently interbedding of incoherent materials and others materials with various bonding degree. The whole thickness of Volcanites is about a few hundreds of meters and is represented by:

- Superior Complex of Colli Albani volcanic products, separated substantially from altered and remoulded pyroclastics materials and composed by Villa Senni Tuff (called “pozzolanaslle” or superior pozzolanas); by Lionato Tuff (“Tufo Lionato”, (TL)); by Black Pozzolanas (PN) and Red Pozzolanas (PR).

- Inferior Tuffs Complex composed by a succession of lithoids layers (T1-T2) and incoherent layers, sometimes partially pedogenized (TA). From Tufo Lionato and Black Pozzolanas and from last ones and Red Pozzolanas and, more in general, at the top of Inferior Tuffs, there are layers of pyroclastics materials more or less altered called “Terrosi Tuffs” (TT). The geotechnical profile of the station is represented in Figure 1.

![Figure 1: geological profile of Centocelle Park station](image)

2.2 Strain characteristics of pyroclastic soil

The RE.Mi. tests performed have furnished profiles of speed wave shear and therefore of \( G_0 \) modulus of the type suitable in Figure 2. Such profile underlines in way enough detailed the stiffness variations of the different soil layers.

On the bottom of backfill layer (R) is visible en increase of \( V_S \), due to the presence of Lionato Tuff (TL), the behavior of that will not be analyzed, considered the very small thickness and the absence of continuity along the the station. The values of speed \( V_S \), \( G_0 \) and \( E_0 \) modulus at the small strain they are presented in Table 1, together to the design geotechnical parameters.

The most important soil layer for the design of retaining structures, it is that of "Pozzolanas". The Black Pozzolanas (PN) are constituted from incoherent to pseudo-coherent pyroclastic material with fine grain size and ash – cinders matrix of blackish-grey color, sometimes of purplish color, can have a weakly bonding degree.

The Red Pozzolanas (PR), under the PN layer, are characterized instead by a purplish red color, and much coarse matrix.

The good geotechnical characteristics of such layer have been confirmed by the numerous SPT tests gone to refusal. The direct shear tests furnished friction angles above the 35° and very dispersed cohesions around the average value of 10.3 kPa (Figures 3 and 4). The average value of \( G_0 \) modulus is equal to 440 MPa for PN layer and to 580 MPa for PR. The pressurometer modulus are results equal to \( E_m = 88 \) MPa (PN) and to 70 MPa (PR).

With the purpose to effect a comparison between the elastic modulus tired from pressurometer tests and those "static" tired from the RE.Mi. tests, has been used the correlation proposed by Rzhevsky and Novik [1971]: \( E_0 = 8.3 \cdot E_{static} + 0.97 \) in which \( E_0 \) represents the elastic modulus at small strain obtained by the RE.Mi. tests.
The comparison between modulus obtained from pressumeter tests is shown in Figure 5, in which it can be noticed that in the layer of the pozzolanas and altered tuffs the values of the “static” modulus $E$ obtained from Re.Mi. tests is inferior that this one obtained from pressumeter tests for almost the whole interval of depth analyzed.

In the area of the station, during investigation, a facies very cemented has been met (indicated as PRb layer) that makes to the material assume the characteristics of a lithoid tuff (Figure 6). The Re.Mi. profile confirms an increase of the rigidity between +10.00 and +3.00 m a.s.l. up to values around 880 MPa, comparable with those one of Lionato Tuff and of the Inferiors Tuffs not altered (T1-T2). The high soil bonding degree in the basal portion of the bed of pozzolanas (PRb) has been confirmed by numerous uniaxial compression tests in which it’s was possible to obtain variable resistances between 4.5 and 33.7 MPa with an average value of 18.6 MPa. From tests given with measurement of deformations, an average tangent Young’s modulus $E_{t,50}$ of 12 GPa has been obtained. Moreover, to be able to perform a comparison between modulus values $G_0$ obtained by geophysical tests on site, initial tangent modulus ($E_{0i}$) have been deduced through stress-strain curves, getting the results in the Table 2.

![Figure 2: $G_0$ modulus profile from RE.MI. tests](image)

Regarding to stress-strain response, it is highlight that the behavior observed for small deformations shows in the almost totality performed tests a progressive increase of the tangent elastic modulus (Figure 7). This behavior is probably due to the progressive closing of present macropores and partly due to the unreliability of measure in the initial part of stress-strain curve. However it’s opportune to observe that this type of answer about stress-strain was manifested regardless of way of measurement of deformations (strain gauges or displacements transducers). The elastic modulus values obtained by laboratory tests obviously result more elevated than those obtained through the measure of $V_s$, that characterize soil at the scale of deposit: detected differences on the average value of shear modulus allow to esteem a reduction in the passage from the rock material to the mass in order to 75%. This stiffness reduction is probably overestimated because unreliability of Re.Mi. tests for depth up to the 40 m, and it doesn't result justified, given the relative structural homogeneity of deposit on studying.

![Table 1: geotechnical parameters of pyroclastic soil of Centocelle Park station](image)

<table>
<thead>
<tr>
<th>layer</th>
<th>$\rho$ (Mg/m$^3$)</th>
<th>$V_s$ (m/s)</th>
<th>$G_0$ (MPa)</th>
<th>$E_0$ (MPa)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PRb</td>
<td>1.75</td>
<td>391</td>
<td>268</td>
<td>569</td>
<td>30</td>
<td>35</td>
</tr>
<tr>
<td>PRb</td>
<td>1.75</td>
<td>391</td>
<td>268</td>
<td>569</td>
<td>30</td>
<td>35</td>
</tr>
<tr>
<td>PRb</td>
<td>1.75</td>
<td>391</td>
<td>268</td>
<td>569</td>
<td>30</td>
<td>35</td>
</tr>
<tr>
<td>PRb</td>
<td>1.75</td>
<td>391</td>
<td>268</td>
<td>569</td>
<td>30</td>
<td>35</td>
</tr>
</tbody>
</table>

![Figure 3: friction angle versus depth (m a.s.l.) for pozzolana layers (PN-PR)](image)

3 PREDICTION OF BEHAVIOR OF EXCAVATION AND COMPARISON WITH MONITORING MEASURES

3.1 Description of the station

The station, is constituted by a deep structure excavated between diaphragm wall of 1.20 m thickness to rectangular plant with geometry narrowings at one side. At this time (oct. 2010), the excavation level
has not yet reached the maximum. The actual level is at +14.00 meters a.s.l., inside the PRb layer. To realize the station diaphragm walls has been necessary to perform a preliminary excavation of maximum height equal to 8.50 m, sustained by a cantilevered retaining wall (piles of diameter equal to 1.20 m, distance 1.40 m).

Figure 4: cohesion versus ground depth (m a.s.l.) for pozzolana layers (PN-PR)

The project foresees a maximum excavation of around 28.00 m with conventional top-down method, sustained from diaphragm wall (length equal to 45.00 m) realized with hydrofraise, without steel reinforcement for the last 5 m: this part of has just the function to allow the execution of a waterproof layer constituted from cement injection to isolate hydraulically the station.

Figure 6: specimen of cemented pozzolana (PRb)

Table 2: summary of results of uniaxial compression tests on PRb specimens

<table>
<thead>
<tr>
<th>parameter</th>
<th>average</th>
<th>min</th>
<th>max</th>
<th>st. dev.</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_f$ (MPa)</td>
<td>18.50</td>
<td>4.50</td>
<td>33.71</td>
<td>6.75</td>
<td>2.74</td>
</tr>
<tr>
<td>$E_{150%}$ (MPa)</td>
<td>12044</td>
<td>4360</td>
<td>25660</td>
<td>4769</td>
<td>2.53</td>
</tr>
<tr>
<td>$E_{150%}$ (MPa)</td>
<td>11570</td>
<td>5390</td>
<td>18960</td>
<td>3350</td>
<td>3.45</td>
</tr>
<tr>
<td>$E_s$ (MPa)</td>
<td>8197</td>
<td>2500</td>
<td>16667</td>
<td>3886</td>
<td>2.11</td>
</tr>
<tr>
<td>$G_0$ (MPa)</td>
<td>3726</td>
<td>1136</td>
<td>7576</td>
<td>1766</td>
<td>2.11</td>
</tr>
</tbody>
</table>

For top-down phases only two slab will be realized (top slab and slab at +20.05 m a.s.l. level) up to the attainment of the maximum excavation level (+8.70 m a.s.l.), while the completion of the structure will happen with cast concrete slab (two mezzanine slabs and mat foundation slab), together to that of the con-

Figure 7: some stress-strain curves tired from uniaxial compressive tests on bonded pozzolanas (PRb) layer
crete inner wall resist hydrostatic pressure and the remaining earth pressure.

![Graph: Progressive stiffening of PRb specimens](image)

Figure 8: progressive stiffening of PRb specimens

It is put in evidence the fact that it was possible to reach the maximum excavation depth with just two slabs thanks to the favorable effect on earth pressure due to presence of PRb layer, characterized by a pseudo-lithoid behavior.

The principal geometric characteristics of the structures of the station are following summarized:

- \( L = 45 \, \text{m} \) (diaphragm-wall length);
- \( s = 1.20 \, \text{m} \) (diaphragm-wall thickness);
- \( s = 1.00 \, \text{m} \) (slab thickness (top down phases));
- \( l = 38.70 \, \text{m} \) (station width);
- \( z_c = 27.10 \, \text{m} \) (maximum excavation depth);
- \( E_J = 4855918 \, \text{kN/m}^2/\text{m} \) (diaphragm-wall stiffness).

![Graph: Young's modulus with stress](image)

3.2 Structural and geotechnical model

Following geotechnical and structural hypotheses are described. The succession of soil layer has been deduced by the design geotechnical profile. From the ground level of 36.00 m a.s.l., the interested layers by excavation are the followings:

<table>
<thead>
<tr>
<th>Layer</th>
<th>( y ) (kN/m²)</th>
<th>( z_{up} ) (m a.s.l.)</th>
<th>( z_{down} ) (m a.s.l.)</th>
<th>( c' ) (kPa)</th>
<th>( \phi' ) (°)</th>
<th>( G_0 ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R (backfill)</td>
<td>17</td>
<td>44.00</td>
<td>36.00</td>
<td>10</td>
<td>25</td>
<td>185</td>
</tr>
<tr>
<td>PN</td>
<td>17.5</td>
<td>33.00</td>
<td>26.00</td>
<td>5</td>
<td>35</td>
<td>438</td>
</tr>
<tr>
<td>TT</td>
<td>17</td>
<td>23.00</td>
<td>20.00</td>
<td>5</td>
<td>35</td>
<td>425</td>
</tr>
<tr>
<td>PR</td>
<td>18.5</td>
<td>23.00</td>
<td>13.00</td>
<td>5</td>
<td>35</td>
<td>580</td>
</tr>
<tr>
<td>PRb</td>
<td>19</td>
<td>13.00</td>
<td>9.00</td>
<td>500</td>
<td>35</td>
<td>879</td>
</tr>
<tr>
<td>TA</td>
<td>17.5</td>
<td>3.00</td>
<td>0.00</td>
<td>50</td>
<td>35</td>
<td>268</td>
</tr>
<tr>
<td>T2</td>
<td>17.5</td>
<td>0.00</td>
<td>-4.15</td>
<td>40</td>
<td>35</td>
<td>879</td>
</tr>
</tbody>
</table>

Table 3: geotechnical parameters of Centocelle Park station layers

The phreatic level has been considered at 27.00 m a.s.l.. The analysis has been executed in drained condition.

![Geotechnical cross section of Centocelle Park station](image)

Figure 10: geotechnical cross section of Centocelle Park station

The analysis for prediction of structures behavior has been carried on through the finite elements code PLAXIS 9.0 version.

To the edges of the model the followings boundary condition has been imposed:
- superior surface: free;
- lateral surfaces: null horizontal displacement;
- inferior surface: null horizontal and vertical displacement (hinges).

3.3 FEM analysis constitutive law

The soil behavior has been represented through an elastic-plastic constitutive law with hardening. In particular the Hardening Soil Model with Small
Strain Stiffness (HSSmall) [Benz, 2006] has been used. This model represents an evolution of Hardening Soil Model (HS) [Schanz et al. 1999]. The HS model contemplate an hyperbolic stress-strain relationship.

The behavior in the elastic field is defined by the elastic modulus of Young $E'$, that depends on the effective stress state through relationship:

$$E' = E'_{ref} \left( \frac{c' \cdot \cot \phi' + \sigma'_3}{c' \cdot \cot \phi' + \rho_{ref}} \right)^m$$  \hspace{1cm} (2)

in which $\rho_{ref}$= 100 kPa is a reference pressure (atmospheric value), $E'_{ref}$ is the Young’s modulus for $\sigma'_3 = \rho_{ref}$, $\sigma'_3$ is the principal effective minimum stress and $m$ is an adimensional coefficient, varying between 0.2 and 1 according to the grain size of the material. The hardening parameter is a function of plastic shear strains through the parameter $E_{50}$ and of plastic volumetric strain through the parameter $E_{oed}$. $E_{50}$ e $E_{oed}$ vary with stress state through relations formally similar to equation (2). For unloading and reloading phases, the model provides an elastic behavior with a reference modulus linked to the first unloading modulus by the relationship below:

$$E_{ur} = E_{ur}^{ref} \left( \frac{c' \cdot \cot \phi' + \sigma'_3}{c' \cdot \cot \phi' + \rho_{ref}} \right)^m$$  \hspace{1cm} (3)

in which $E_{ur}^{ref} = 1 + 3 E'_{50}$.

The range of deformations for which a soil has actually an elastic response, with an almost total recovery of applied deformation, is very limited. This aspect of soil behavior is highlighted by the characteristic “S” shape of decreasing curve of shear modulus at small strain $G_0$ versus shear strain [Atkinson & Sallfors 1991].

The HSSmall model can take into account to real stiffness of soil on small deformations and to its dependence on level of deformations achieved by introducing two additional parameters: $G_{50}^{ref}$ e $\gamma_{0.7}$. The first parameter is representative of soil stiffness for reduced levels of deformation ($\epsilon < 10^{-6}$), while the second is the shear strain for which the secant shear modulus is reduced to 70% of the value of $G_0$. $G_0$ and $\gamma_{0.7}$ parameters essentially depend from stress state and from the relative density (void index) of soil. The dependence of $G_0$ by the stress state is represented by the law:

$$G_0 = G_{0ref} \left[ \frac{(c' \cdot \cos \phi' - \sigma'_{min} \sin \phi')}{(c' \cdot \cos \phi' + \rho_{ref} \sin \phi')} \right]^m$$  \hspace{1cm} (4)

similar to that one used for $E_{50}$ and $E_{ur}$. The deformation $\gamma_{0.7}$ is assumed independent on the average of the stress state and equal to:

$$\gamma_{0.7} = \frac{1}{9(G_{o,ref})} \left[ \left( \frac{2c' (1 + \cos(2 \phi')) - \sigma'_1 (1 + K_o) \sin(2 \phi')} \right) \right]$$  \hspace{1cm} (5)

At the same way, operative values of modulus during loading and unloading ($E_{ur}^{ref}$), defined before, can be deduced by calculating at first elastic modulus value for small deformation, in according to the well-known relationship of the elasticity theory and by dividing the obtained value for a $k$ coefficient derived from the graphic correlation represented in figure 11 [Alpan, 1970].

During simulation of the excavation of the Centocelle Park station, $G_0$ modulus has been deduced by interpretation of Re.Mi tests, while $E_{50}^{ref}$ has been deduced by dividing for the value 3 the $E_{ur}^{ref}$ modulus obtained using Alpan's correlation. Alternatively it was possible to use the procedure suggested by Callisto et al. (2007), that involves comparison between elastic modulus values at small deformation obtained by geophysical tests (for example Cross-Hole), and that one obtained by simulation of triaxial tests to determinate the reductive factor to pass from $E_0$ to $E_{50}^{ref}$.

![Figure 11: graphic correlation between values of dynamic modulus at small strain ($E_{50}^{ref}$) and the static unloading-reloading modulus ($E_{ur}^{ref}$) $E_{0}$ [Alpan 1970]](image)

3.4 Monitoring instrumentation

To compare the results of the analyses displacement measures has been used, obtained by using inclinometer into the diaphragm wall or positioned on the back of the same. In the figure 12 below the considered section of monitoring is shown.

![Figure 12: Monitoring instrumentation](image)

3.5 Analysis results and comparisions with monitoring measures

The results of geotechnical analysis are presented below as:

1. horizontal displacement of diaphragm wall;
2. bending moment on diaphragm wall.
All parameters above are referred to wall that is adjacent to the sheet piles to support the initial excavation.

Figure 12: monitoring cross section

The finite element model shows displacements in general higher than those measured. In the figure 13, the contours of mobilized shear strength, defined as the ratio $\tau / \tau_{\text{max}}$ between the maximum shear stress acting at one point and the corresponding available strength, indicate that areas in which the ratio is around equal to 1.0 (soil near to failure conditions) are concentrated between the slab at +20.05 m a.s.l. level and mat foundation slab: at this depth the red pozzolanas are presents. For this reason, to find the best set of parameters to approximate calculated displacements to measured displacements, has been considered more appropriate to not increase values of $G_0$ modulus for soils (PN e PR).

The horizontal displacement are represented in figure 14 for excavation phases analyzed. The curves are referred to middle excavation phase (excavation at +14.00 m a.s.l.) and to final excavation phase (+8.70 m a.s.l.). In this figure, a good agreement with measured displacements has been obtained by assuming for pozzolanas (PN e PR) a cohesion value equal to 25 kPa.

This value has been chosen by observing laboratory results of tests made on pozzolanas of others station of the line C. The back-analysis executed, leads to believe that cohesion of pozzolanas (PN e PR) that are in the area of the station, has been underestimated, probably because the disturbance on samples during taking and testing. The bending moments of envelope vary from 1941 kN*m/m (earth side) e 1654 kN*m/m (excavation side). In the middle phase when excavation is on +14.00 m a.s.l., moments are equal to 1929 kN*m/m (earth side) and to 1600 kN*m/m (excavation side).

4 CONCLUSIONS

From comparison between maximum bending moment values and that at middle phase of excavation, it’s possible to expect very modest increases for the last excavation phase. The displacements of calculation obtained by assuming the cohesion of PN-PR layers equal to 25 kPa, are in good accord to that measured. This result shows the real resistance of pozzolanic superior layers. It’s proves once again that the negative effects due to a difficult sampling, leads to a wrong evaluation of the bonding degree of this material.
REFERENCES


