Strains and failure characteristics of pyroclastic materials in deep excavation

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ABSTRACT: In this paper the results of the investigations (substantially on site) for the determination of the pyroclastic characteristics deformability soils by the Centocelle Park station open pit (Line C of the subway in Rome) are presented. To allow a correct estimation of the mechanical characteristics of the different formations, two type of investigations has been performed, both traditional (pressumeter tests, SPT, direct shear tests), that seismic (RE.MI.). Describing shortly the most representative aspects of the costituve model law and mechanical behavior of the soil at small strain, usually reached in the open pit excavation, the results of the performed tests and the criteria of choice of the deformability parameters of project are illustrated. Finally from monitoring data obtained during the excavation of the station, similes and differences on strain and bending moment on the diaphragm wall, compared to prevision obtained by finite element models are shown.

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 one 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 E_0). The numerous researches made on the behavior of soils at small strains [Rampello S. e Silvestri F., 1993; Stokoe *et al.*, 1995], have shown G_0 modulus, can have the meaning of "state parameter ", like the initial voids index e_0, the earth pressure coefficient at rest K_0 or the effective vertical lithostatic stress σ'_{v0} [Ghionna *et al.*, 2006]. From all above the necessity to define with enough approximation the course of the shear modulus G_0 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:

$$G_0=\rho\cdot V_s^2 \qquad (1)$$

in which ρ 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 inchoerent 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 o less altered called "Terrosi Tuffs" (TT). The geotecnical profile of the station is represented in Figure 1.

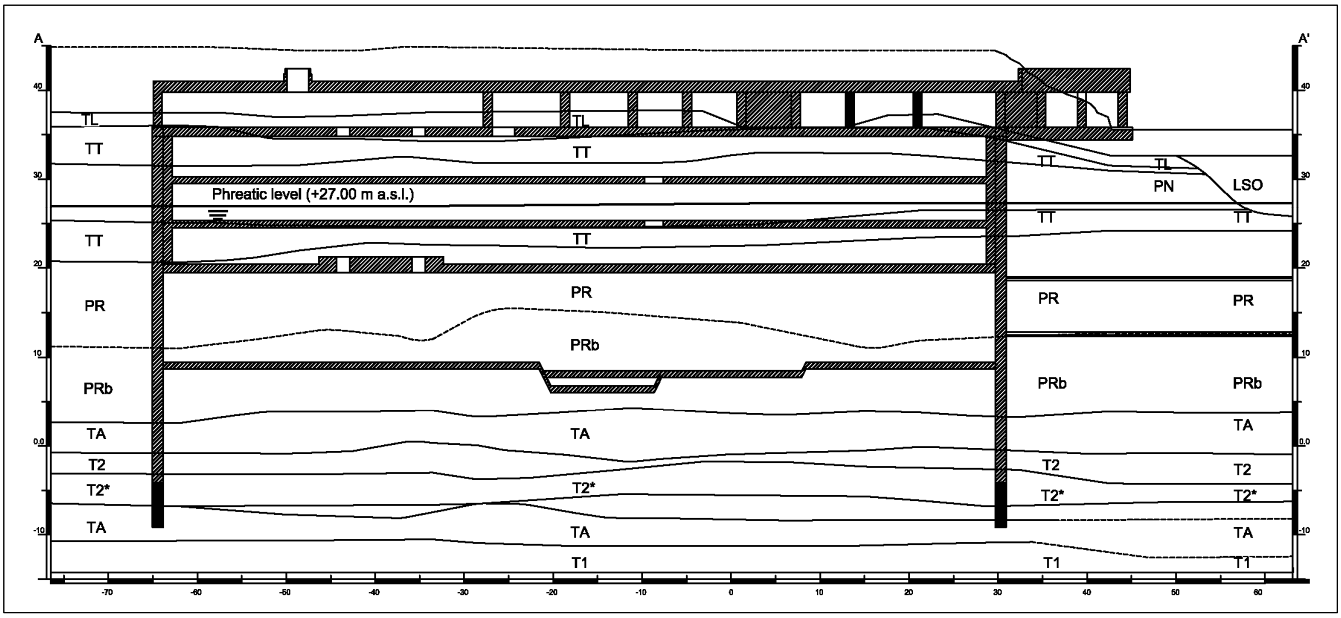

Figure 1: geological profile of Centocelle Park station

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 pressumeter 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 pressumeter 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 caracterictics 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_{ti}) have been deduced through stress-strain curves, getting the results in the Table 2.

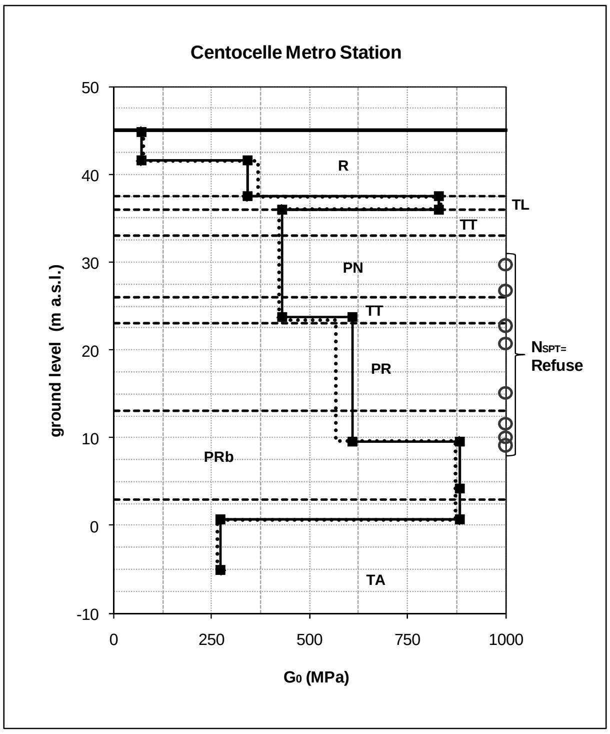

Figure 2: G_0 modulus profile from RE.MI. tests

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 Vs, that caracterize 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 i 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.

layer	ρ (Mg/m^3)	V_S (m/s)	G_0 (MPa)	E_0 (MPa)	c' (kPa)	φ' (°)
R (backfill)	1,7	330	185	407	10	25
TL	1,75	690	832	1830	25	35
TT	1,7	500	425	935	25	34
PN	1,75	500	438	963	5	35
TT	1,7	500	425	935	25	34
PR	1,85	560	580	1276	5	35
PRb	1,9	680	879	1933	300	35
TA	1,75	391	268	589	30	35
T2	1,75	-	-	-	40	35
*T2**	2	-	-	-	300	35

Table 1:geotechnical parametres of pyroclastic soil of Centocelle Park station

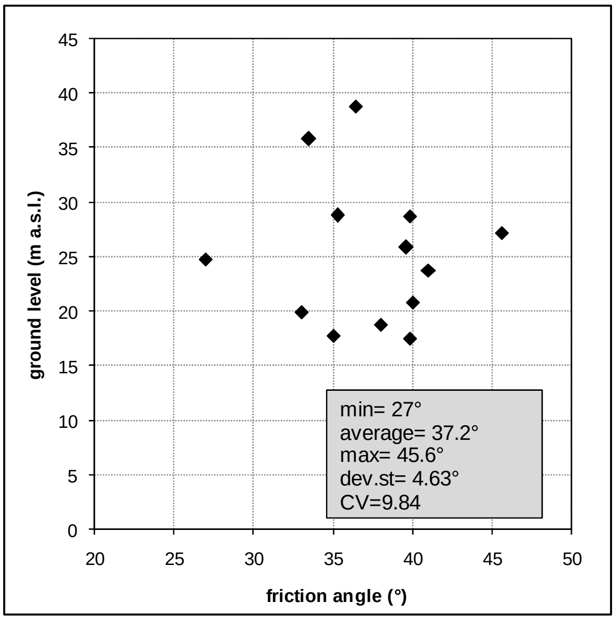

Figure 3: friction angle versus depth (m a.s.l.) for pozzolana layers (PN-PR)

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).

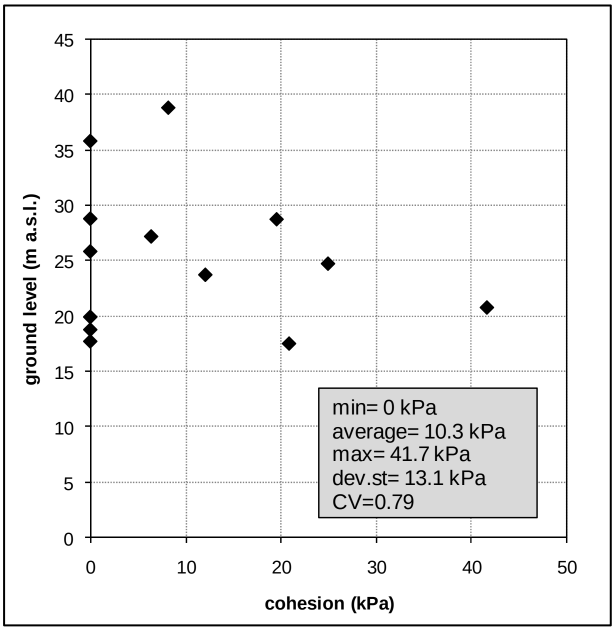

Figure 4: cohesion versus ground depth (m a.s.l.) for pozzolana layers (PN-PR)

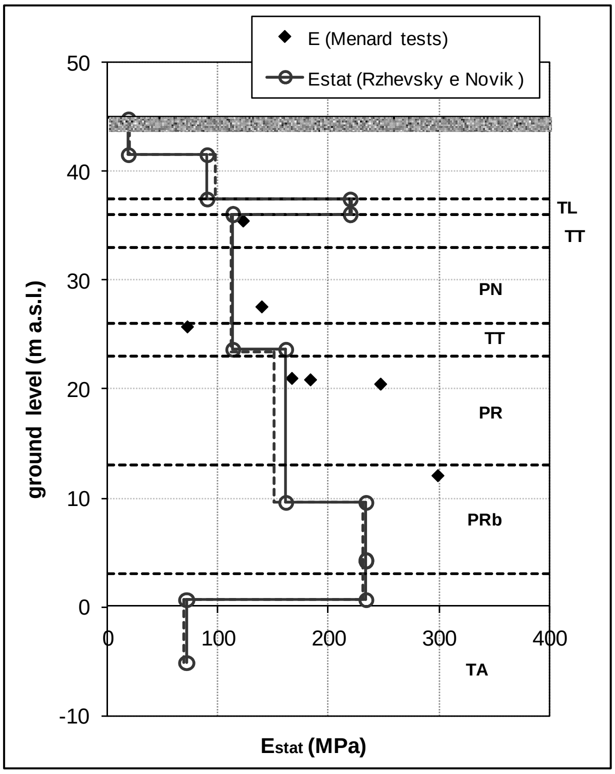

Figure 5: comparison between "static" modulus determined from RE.MI. (with Rzhevsky and Novik relationship) and pressumeter tests

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 funtion to allow the execution of a waterproof layer constituted from cement injection to isolate hydraulically the station.


Figure 6: specimen of cemented pozzolana (PRb)

parameter	average	min	max	st. dev.	CV
σ_f (MPa)	18,50	4,50	33,71	6,75	2,74
$E_{t,50\%}$ (MPa)	12044	4360	25660	4769	2,53
$E_{s,50\%}$ (MPa)	11570	5390	18960	3350	3,45
E_{ti} (MPa)	8197	2500	16667	3886	2,11
G_0 (MPa)	3726	1136	7576	1766	2,11

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

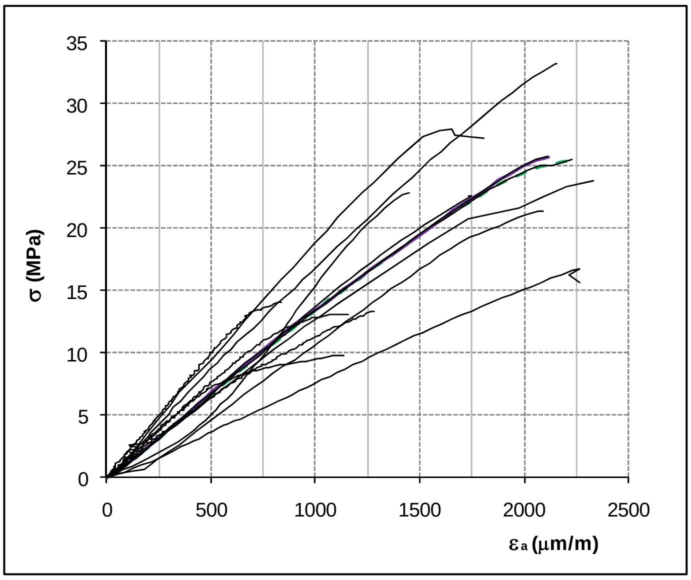

Figure 7: some stress-strain curves tired from uniaxial compressive tests on bonded pozzolanas (PRb) layer

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 concrete inner wall resist hydrostatic pressure and the remaining earth pressure.

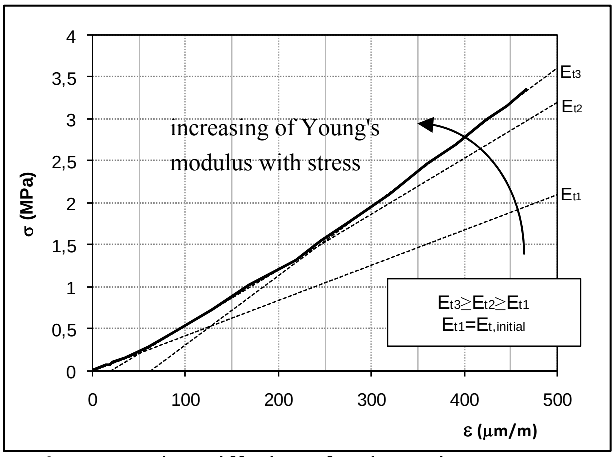

Figure 8: progressive stiffening of PRb specimens

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 m (diaphragm-wall length);
- s= 1.20 m (diaphragm-wall thickness);
- s= 1.00 m (slab thickness (top down phases));
- l= 38.70 m (station width).
- z_e= 27.10 m (maximum excavation depth);
- EJ= 4855918 kN/m^2/m (diaphragm-wall stiffness).

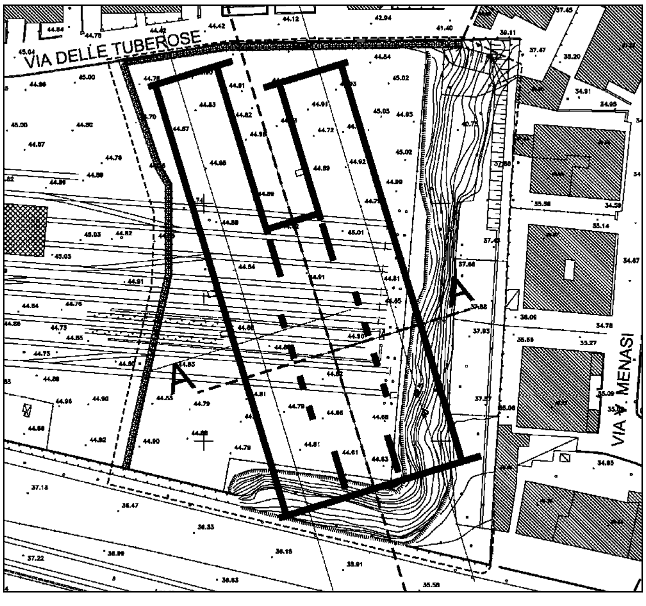

Figure 9: planimetric view of station (in evidence the calculation section analyzed)

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:

layer	γ (kN/m^3)	z_{sup} (m a.s.l.)	z_{inf} (m a.s.l.)	c' (kPa)	φ' (°)	G_0 (MPa)
R (backfill)	17	44,00	36,00	10	25	185
TT	17	36,00	33,00	25	34	425
PN	17,5	33,00	26,00	5	35	438
TT	17	26,00	23,00	25	34	425
PR	18,5	23,00	13,00	5	35	580
PRb	19	13,00	3,00	300	35	879
TA	17,5	3,00	0,00	30	35	268
T2	17,5	0,00	-4,15	40	35	879

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.

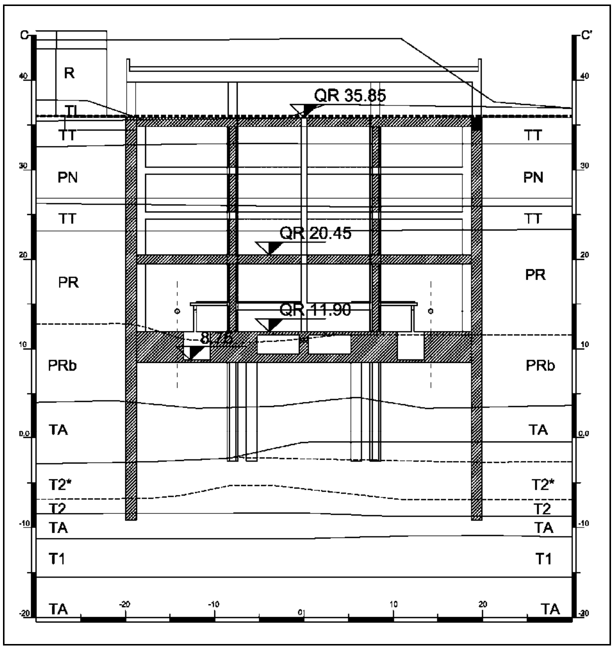

Figure 10: geotechnical cross section of Centocelle Park station

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:

$$E' = E^{ref}\left(\frac{c' \cdot \cot\varphi' + \sigma'_3}{c' \cdot \cot\varphi' + p_{ref}}\right)^m \tag{2}$$

in wich p_{ref}= 100 kPa is a reference pressure (atmospheric value), E^{ref} is the Young's modulus for $\sigma'_3 = p_{ref}$, σ'_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 reloadind phases, the model provides an elastic behavior with a reference modulus linked to the first unloading modulus by the relationship below:

$$E_{ur} = E_{ur}^{ref}\left(\frac{c' \cdot \cot\varphi' + \sigma'_3}{c' \cdot \cot\varphi' + p_{ref}}\right)^m \tag{3}$$

in which E_{ur}^{ref}= 1÷3 E^{ref}_{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^{ref}_o e $\gamma_{0.7}$. The first parameter is representative of soil stiffness for reduced levels of deformation ($\varepsilon < 10^{-6}$), while the second is the shear strain for wich the secant shear modulus is riduced to 70% of the value of G_o. The $G_o$and $\gamma_{0.7}$ parameters essentially depend from stress state and from the relative density (void index) of soil. The dependence of G_o by the stress state is represented by the law:

$$G_0 = G_{0ref}\left[\frac{(c'\cos\varphi' - \sigma'_{min}\sin\varphi')}{(c'\cos\varphi' + p_{ref}\sin\varphi')}\right]^m \tag{4}$$

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:

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

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}.

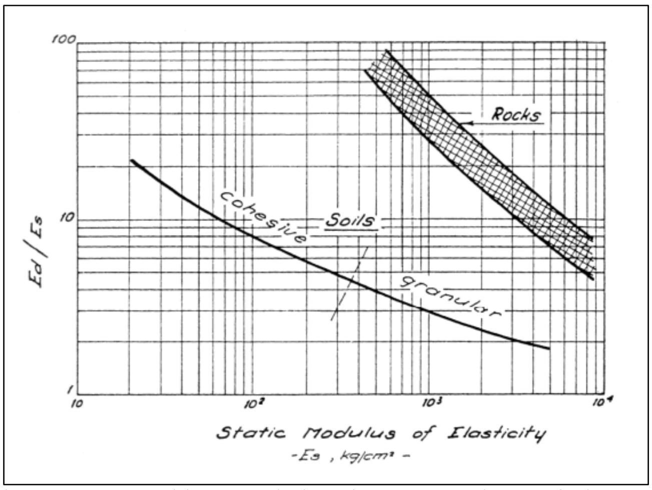

Figure 11: graphic correlation between values of dynamic modulus at small strain (E_0= E_d) and the static unloading-reloading modulus ($E_{u,rref}$~E_s) [Alpan 1970]

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.

3.5 *Analysis results and compariosons 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.

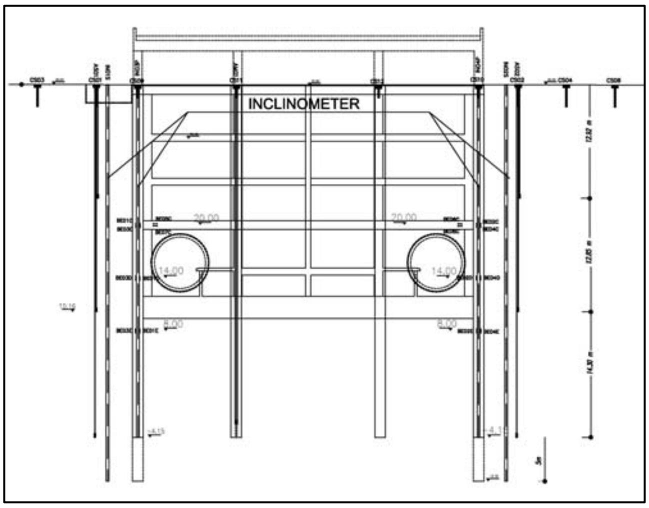

Figure 12: monitoring cross section

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 τ/τ_{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.

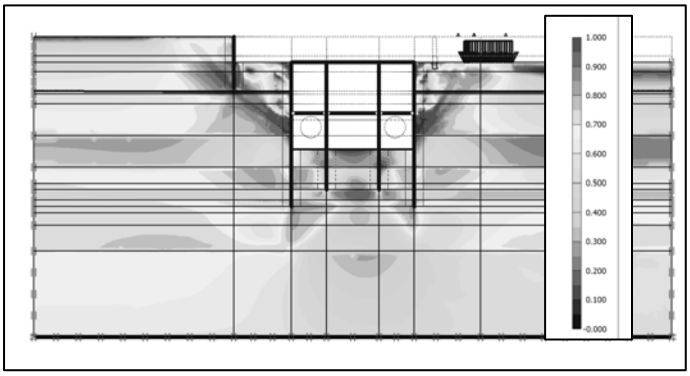

Figure 13: contours of mobilized strength: design coehsion value of PN/PR layers equal to 5 kPa

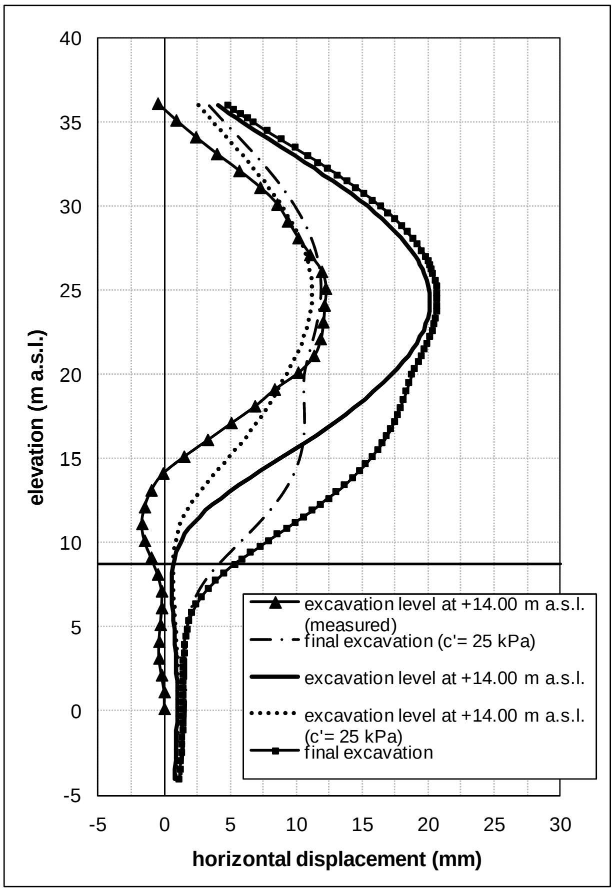

Figure 14: comparison between lateral diaphragm-wall displacements: measured values and FEM results

5 REFERENCES

Alpan I., 1970. The geotechnical properties of soils. *Earth-Science Reviews* 6:5-49.

Atkinson J.H., and Sallfors G. (1991). Experimental determination of soil properties. *In Proc. 10th ECSMFE, Florence, Vol.3, 915-956, 1991.*

Benz (2006). Small-Strain Stiffness of Soils and its Numerical Consequences. *PhD Thesis Stuttgart University*.

Callisto L., Rampello S., Calabresi G., 2007. Scelta delle modalità costruttive e previsione del comportamento di uno scavo profondo a pianta circolare. *XXIII Convegno Nazionale di Geotecnica:* 169-176. Padova-Abano Terme 16-18 Maggio 2007

Ghionna, V.N., Mortara, G., Porcino D., 2006. Caratterizzazione e modellazione dei terreni per la previsione del comportamento delle opere geotecniche. *XXIII Convegno Nazionale di Geotecnica:* 11-44. Padova-Abano Terme 16-18 Maggio 2007.

Rampello S., Silvestri F., 1993. The stress-strain behaviour of natural and reconstituted samples of two overconsolidated clays. *Proc. 1st Int. Symp. on Geotechnical Engineering of Hard Soils - Soft Rocks: 769-778*. Athens 1993, Balkema, Rotterdam, 1.

Rzhevsky V., Novik G., 1971. The Physics of Rocks. Moscow, MIR Publishers.

Schanz T., Vermeer P.A., Bonnier P.G. (1999). The Hardenimg soil model: Formulation and verification. *Beyond 2000 in Computational Geotechnics - 10 Years of PLAXIS.* Balkema, Rotterdam, ISBN 90 5809 040 X.

Stokoe K.H. II, Hwang S.K., Lee J.H.K. Andrus R.D., (1995). Effects of various parameters on the stiffness and damping of soils at small to medium strains. *Proc. Advances in the Art of Testing Soils under cyclic loading conditions, ASCE, Convention*, Detroit.