San Pasquale Station of Line 6 in Napoli

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***Abstract* – The paper reports some geotechnical aspects of the design and construction of the San Pasquale Station, intermediate along the stretch of Line 6, which walks across the coastline named Riviera di Chiaia. The station required an excavation deeper 27 m, almost entirely located in pyroclastic sand below the groundwater table. The main shaft is 85.5 m long and 24.1 m large, containing the whole length of the pedestrian platform while a single large section tunnel, built before the excavation of the station shaft, accommodates the two operating rail tracks. Monitoring data will be presented and discussed. Settlements and horizontal displacements represent certainly very significant outcomes among the observed data. Their variations along with time and main construction steps are presented in the paper. The monitoring data have also been submitted to a process of careful interpretation based on the use of numerical analyses to better understand the interaction of deep excavation in a crowded urban area. The FEM code Plaxis has been adopted for such a purpose. Advanced constitutive soil models are available in the software library; however the best compromise between available models and the concrete possibility of properly calibrating their parameters on the basis of the site and laboratory geotechnical investigations was made.**

I. INTRODUCTION

In 1997 the Municipality of Napoli approved a new City Transportation Plan, that has led to a significant pressure for the construction of new underground train lines, stations and car parks.

Metropolitana di Napoli, or Napoli Under-ground, is the metro system serving the city, including at present six underground rapid transit railway lines, a commuter rail network and four funicular lines, with planned upgrading and expansion works underway. Among the six already operating lines some are experiencing a substantial development with new stretches under construction.

One out of these is the Line 6. When completed, according to the current design state, Line 6 will connect the Western borough of Bagnoli to the city centre at Municipio station, with a total length of 8 km and 12 stations.

In the following, attention will be focused on the design and the construction problems of San Pasquale station, which will be operating in the next months.

II. SAN PASQUALE STATION

The Station is inserted near the sea in a crowded area which is one of the most notorious and touristic district of the city.

The main body of San Pasquale station has a rectangular shape in plan of 85.50 m × 24.10 m and the maximum excavation depth is approximately 27 m (26 m underground water table) (Figure 2). The long side of the station is parallel to the longitudinal tunnel axis and the closest buildings are all located on the Northern side keeping approximately a unique alignment which is again parallel to the long side of the station. On the Southern side the station is bounded by the sea with the interposition of the public garden. The main and rather large shaft contains the passenger platforms and eliminates the necessity of excavating platform tunnels underground. On both long side are located the two entrance shafts deep about 10 m.

The excavation is supported from T-section diaphragm walls made by reinforced concrete (Figure 1) and built using huge hydromill equipped by a 90° rotating head. Each panel of the diaphragm walls was built by intersecting two separate excavations with a rectangular shape in plan. The depth of the panels is about 50 m which allows a substantial embedment in the Neapolitan Yellow Tuff formation (NYT).

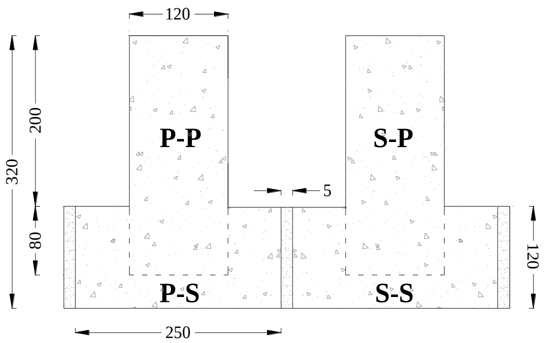

Fig. 1. T section diaphragm walls.

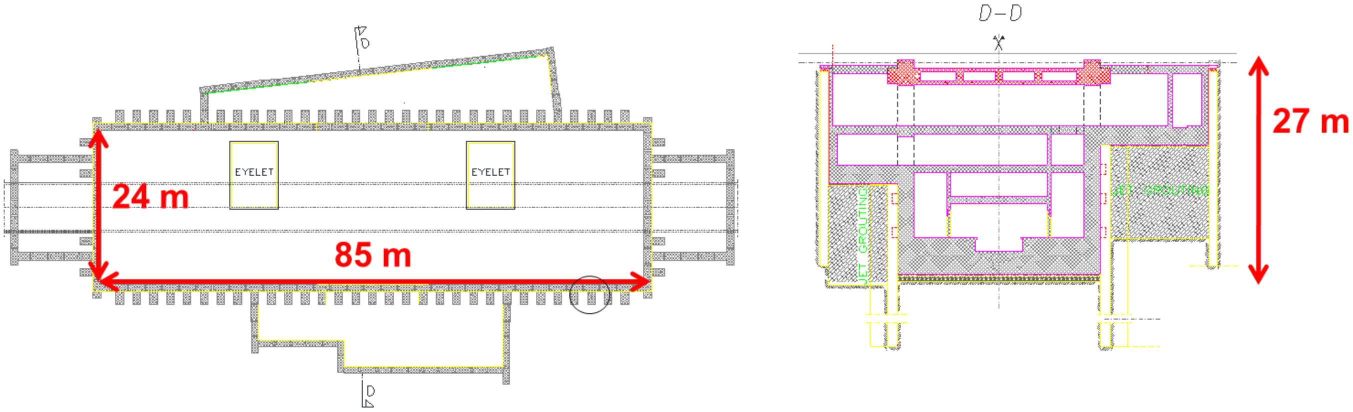

Fig. 2. Plan view and cross section of the San Pasquale Station.

The closest historical and valuable buildings involves the primary need to preserve the structures. In order to keep the settlements within tolerable limits the top-down construction method was adopted. The excavation was executed in dry conditions using pumping dewatering wells. Firstly the diaphragms have been executed, leaving soft eyes with fiberglass reinforcement bars to be drilled by TBM. The passage of the TBM was the second main step before the excavation of the station. The tunnel within the station area was filled with soil and the lining was demolished when was reached by the excavation.

At the design stage geotechnical investigations were carried out in the area occupied by the station which is approximately 2000 m^2. The site is relatively flat with the ground level located between +2 and +2.30 m a.s.l.. The groundwater table lays at +1.30 m a.s.l..

Both in situ and laboratory geotechnical tests were executed. As usual boreholes with continuous coring, drilling hole and Standard Penetration Tests were initially carried out. Further site investigations consisting in CPTs, Seismic Dilatometers test and Cross Hole tests were subsequently carried out. The plan view of the site investigations is reported in Figure 3. Figure 4 shows the geological soil profile corresponding to the transverse cross-section located in the middle of the station. The deposit is characterized by subhorizontal volcanic layers underling the yellow tuff formation. On the top marine sand and hand made soil were found to a depth of 17 m from the ground surface (layer A), followed by a layer of pyroclastic products consisting of silty sands, or ashes and pumices from the depth of 17 m down to a depth of 41 m (layer B). Going deeper a thin layer of altered yellow tuff (layer C) separates the layer B from the formation of the tuff (layer D) (Autuori et al. 2013). Due to the irregular soil profile of the tuff formation, the elevation profile of the tuff ruff was obtained from the borehole information and it is reported in Figure 5. The understanding of the correct elevation position was very helpful in the comphrension of the inclinometer profiles.

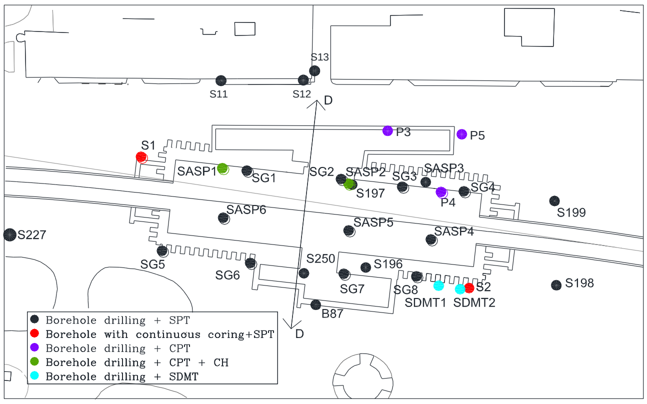

Fig. 3. Plan view of site investigations.

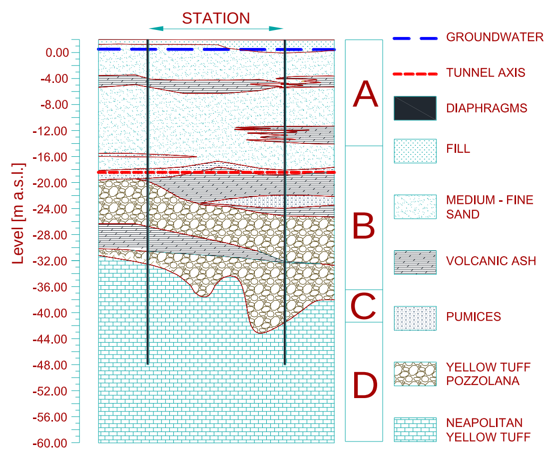

Fig. 4. Stratigraphy of San Pasquale site.

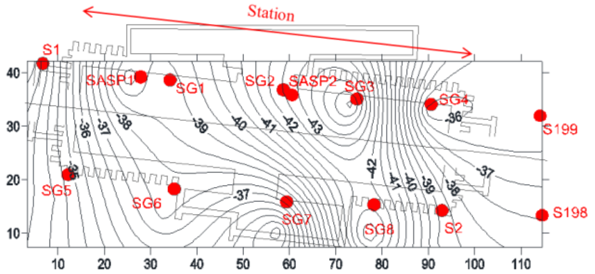

Fig. 5. Elevation profile of San Pasquale site.

In Figure 6, the values of the cone resistance q_c of CPT are plotted versus depth. The trend of the results is typical of cohesionless soils, including the relatively high scatter (from 10 MPa to 20 MPa with some peaks of 40 MPa).
In Figure 7 the shear wave velocity profile is reported, obtained by both SDMT and CH. The agreement is quite satisfactory, even though the values from SDMT are on average slightly larger than those obtained by the CH. This could be related to the different local effect of the two in situ tests: in fact, with SDMT, the soil is locally displaced (and compressed) by the dilatometer, while in the CH, the soil expands after drilling the holes. The main differences between the two tests are concentrated at a depth of about 10 and 25 m, where it seems that the dilatometer has detected two very dense or slightly cemented layers that the cross-hole has not intercepted. The shear wave velocity increases in the sand from 150 m/s near the ground surface to 500 m/s at a depth of 20 m; in the pyroclastic soils below the sand, it keeps constant with depth at around 350 m/s and increases at the bottom of the deposit to reach almost 1,000 m/s at the top of the underlying tuff. The in situ tests well confirm the adopted simplified stratigraphy.

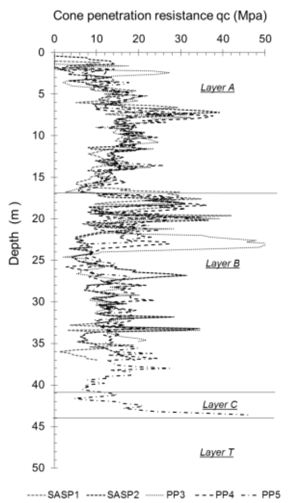

Fig. 6. Cone penetration resistance q_c obtained from CPTs.

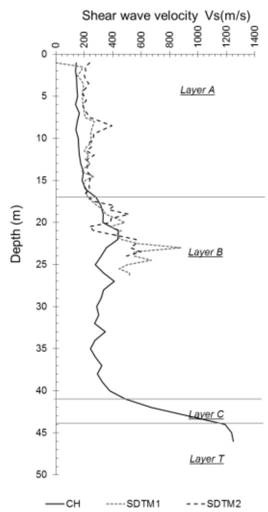

Fig. 7. Shear wave velocity V_s from CH and SDMTs.

The values of the earth pressure coefficient at rest k_0 was obtained processing the results of both CPTs and SDMTs (Figure 8). Figure 9 shows the trend of shear modulus obtained by processing the results of the CH.

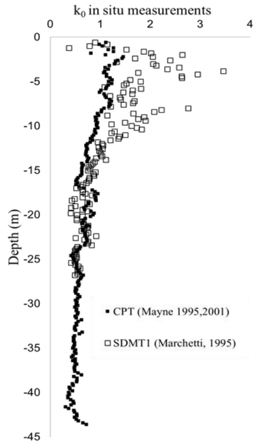

Fig. 8. Earth pressure coefficient at rest k_0 profile obtained from SDMTs and CPTs .

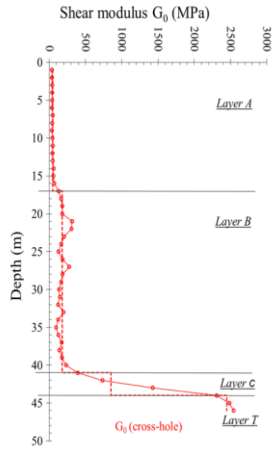

Fig. 9. Shear modulus profile from CH.

III. MONITORING

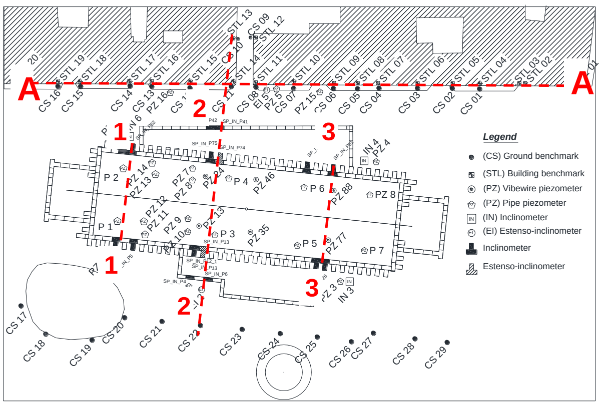

Fig. 10. Plan view of monitoring system and cross sections.

As already mentioned in the introduction, San Pasquale station is very close to some historical and valuable buildings belonging to the downtown. It is excavated in granular soils sitting above a rather homogenous and not largely altered tuff layer with a rather superficial groundwater table governed by the nearby sea level.
It is clear that the problem is a very complex one with the settlement induced by subsidence increasing the already critical settlement induced by the deformations of the diaphragm walls.

The monitoring had a very important role during the construction of the station. Figure 10 reports the plan view of the adopted monitoring system.
The total time needed to reach the designed depth of the excavation and built the underground structures was 5 years and 10 months.Analyzing the monitoring data 11 main steps of construction were individuated, which were further more simplified into 3 phases (Table 1):

- phase I: archeological excavation and the dewatering test were executed. The excavation proceeded slowly due to the possibility to find archeological remains;
- phase II: designed depth was reached. The excavation continued with a full power using dewatering wells;
- phase III: pumping wells were closed. The internal structural box station was completely waterproof and groundwater level grew back to the hydrostatic conditions.

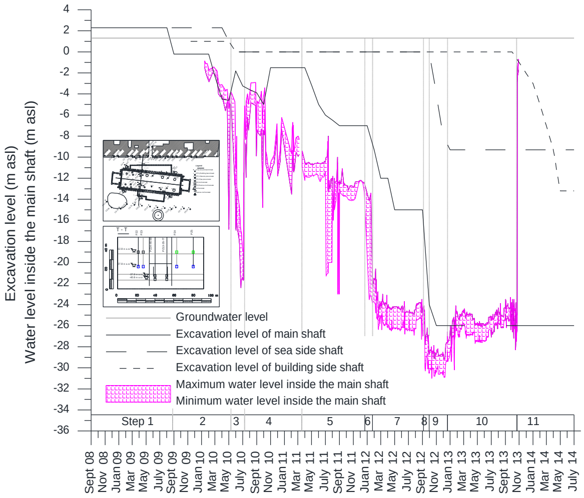

Fig. 11. Excavation depth and groundwater level vs time.

Table 1. Construction sequence of San Pasquale station.

PHASE	STEP	PERIOD: DATES		CONTRUCTION ACTIVITIES	
			Depth of excavation (m asl)	*Dewatering (m asl)*	*other*
	1	15/9/08			Diaphragm walls
I	**2**	27/8/09	Archeological excavation	-4	
	3	10/5/10		Dewatering test (-22)	
	4	11/7/10	Main excavation -7	-10	TBM and top slab
	5	23/3/11	Main excavation -9	-12	
	6	24/12/11	Main excavation -12	-25	
II	**7**	21/1/12	Main excavation -15	-25	
	8	12/9/12	Main excavation -25	-29	
	9	29/9/12	Sea side excavation -9	-29	
	10	27/12/12	Main excavation -27	-32	Bottom slab
III	**11**	23/10/13 24/07/14	Building side excavation -13	End of pumping 23/10/2013	Bottom slab

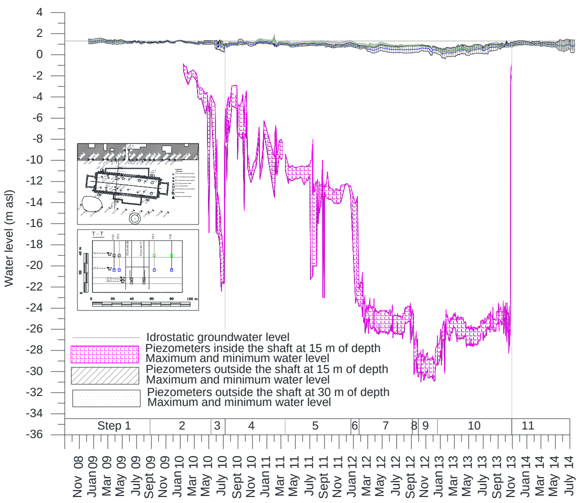

Fig. 12. Groundwater level inside and outside the excavation vs time.

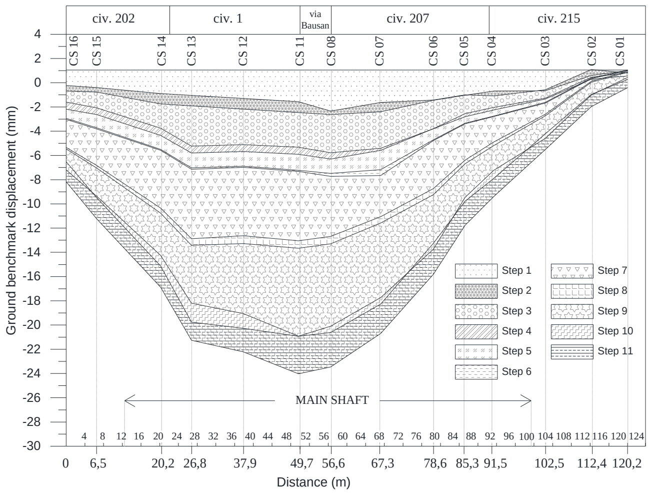

Fig. 13. Ground benchmarker measurements of the section A-A.

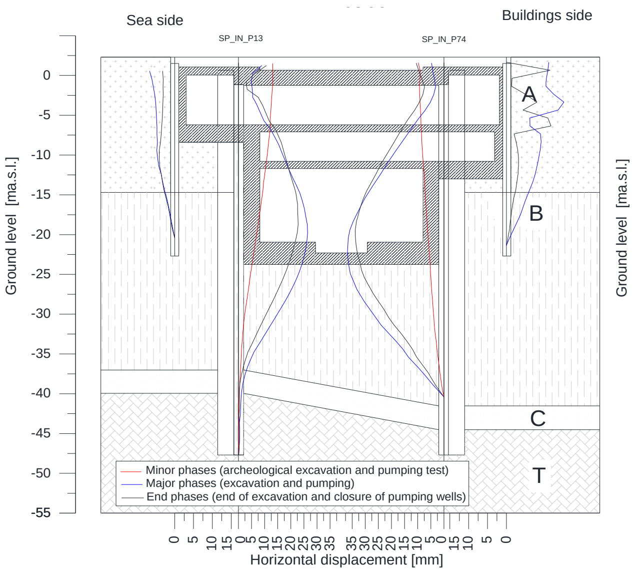

Fig. 14. Inclinometers profile of cross section 2-2.

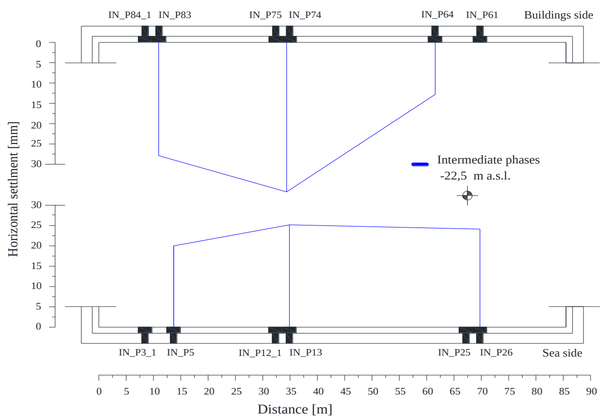

Fig. 15. Horizontal cross section of the station at a level of -22.5 m asl with horizontal displacement of the inclinometers at the end of phase II.

The trend along the time of the excavation depth and the groundwater level is reported in Figure 11. During the phase II the ground water level is on average 5 m below the maximum excavation depth in order to keep dry conditions inside the shaft. The influence of the pumping inside the main shaft in terms of groundwater level outside the excavation is reported in Figures 12. It is possible to observe that the dewatering activity influences little the groundwater level outside the shat: while the lowering inside is at a maximum 32 m, outside the groundwater level goes down at a maximum of a 1 m depth.

Figure 13 shows the ground subsidence for each step of the excavation along the buildings alignment (section A-A of Figure 10). Data show that the maximum settlement is 24 mm in correspondence of the middle of the shaft when the designed excavation depth was reached. Figure 13 shows the horizontal displacement measured from the inclinometers inside the panels located in the opposite position at the cross section 2-2 of Figure 10. During the phase I the panels exhibits a shelf behavior (red color). After the construction of the top slab, the shape profile totally changed with the maximum displacement measured at the maximum excavation depth equal to 36 mm (blue color). Figure 16 represents the horizontal displacement of the opposite panels in correspondence of the horizontal cross section at the maximum excavation depth at the phase II. Both figures 13 and 15 highlight the tridimensional behavior of the station. At the end of phase III, after the closure of the pumping wells, groundwater level grew back to the hydrostatic conditions (Figure 11 and 12), as a consequences a little amount of displacement (both vertical and horizontal) gave back (Figure 13 and black color in Figure 14).

IV. NUMERICAL ANALYSIS

The transversal cross section 2-2, reported in Figure 10, was chosen to back analyze the monitored behavior. A 2D element finite model was modelled. Figure 16 shows the geometry model adopted for the 2D parametric analyses. The calculation domain is 250m x 90m in order to minimize the edge effects. A plain strain model was assumed in the calculation with a 15 node triangular mesh. The number of the mesh elements is 3277 and the average element size is 2.62m. Soil properties and boundary condition were assigned, and also the measured depth of the tuff formation was modelled.

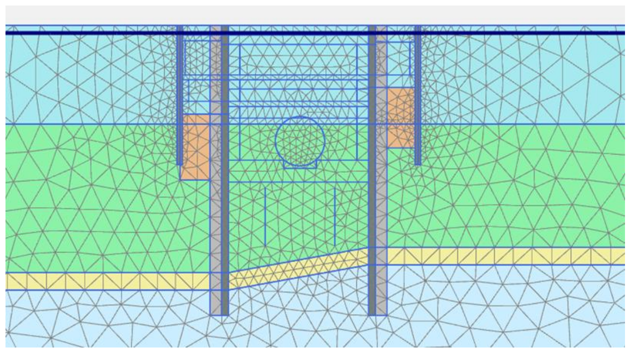

Fig. 16. Geometry model and calculation mesh.

The simplified stratigraphic sequence consists of 4 layers: the layers labelled with capital letters A, B, C and D correspond to those already identified in the simplified stratigraphy of Figure 4. As regards to the mechanical behavior of the soils, the upper layers of the loose soils were modelled with the hardening soil constitutive model and the stiffness parameters were estimated on the base of the CPT shown in Figure 6. In particular the secant stiffness was assumed to be 3 times the value of the resistance to the tip penetrometer q_c mediated within the layer. The unloading/reloading stiffness E_{ur} was assumed twice the modulus E_{50}, that is equal to 6 times the average value of the penetrometer resistance q_c. For the underlying layers were used the simpler model linear elastic perfectly plastic of Mohr -Coulomb. The calculation consists of 24 phases in order to reproduce as realistically as possible what happened during the construction. Several analyses were carried out which take into account separately the effects due to the initial stress state of the soil and the presence of buildings with different calculation mode. In this paper they describe only the results obtained from the most complex analysis which simultaneously models the measured stress state of soil and the building load referring at the end of phase II when the monitored displacements were maximum.

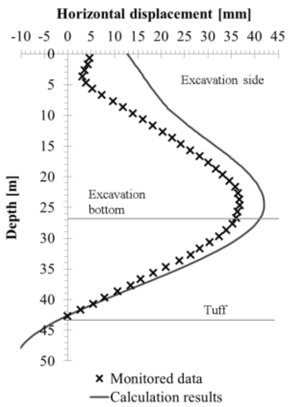

Fig. 17. Comparison between measured and calculated horizontal displacements at the P74 inclinometer on the Northern side.

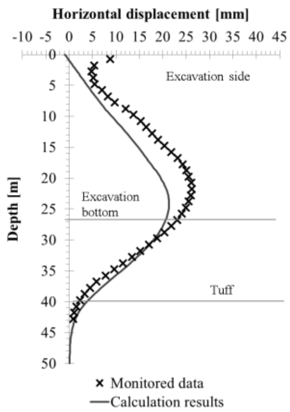

Fig. 18. Comparison between measured and calculated horizontal displacements at the P13 inclinometer on the Southern side.

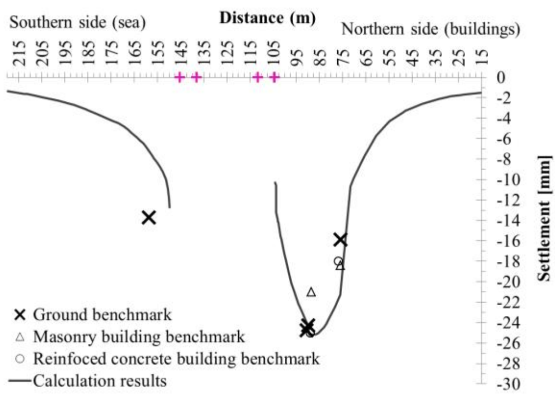

Fig. 19. Comparison between measured and calculated ground settlements.

The Figures 17 and 18 display the horizontal displacements measured and calculated respectively in panels P74 (building side) and P13 (sea side) both supported of the main shaft. The comparison of the results with the monitoring data confirms that the deformation behavior of the diaphragm walls is greatly influenced by the depth of the roof of the bedrock, which acts as a joint to the base. Figure 19 shows the calculation results of the numerical analysis in terms of ground settlement compared with some monitored points. In general the numerical simulation presented here, although able to identify the overall behavior of the excavation, are in good agreement with both horizontal and vertical displacement values monitored.

V. CONCLUSION

Huge excavations were executed and are going on in the city of Napoli to build a new line of the underground network. Some of these excavations present among the other geotechnical difficulties in area which are closely surrounded by important buildings. The numerical analysis of the excavation of the San Pasquale Station reproduces satisfactorily the behavior of the excavation process for all construction steps. The calculation phases were chosen in order to accurately repeat the construction sequence. The bedrock position was modelled and the calibration of the constitutive model parameters were mainly based on the data from the in situ investigations (CPT, CH, SDMT).

The initial stress state of soil and the presence of buildings were taken into account; these aspects are usually neglected in the numerical analyses.

The calculation provides the results in terms of subsidence of the ground surface and horizontal movements of the diaphragm walls slightly different than those measured.

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