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Monitoring a deep excavation in the historical center of Napoli: Università Station

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Abstract

The completion of the downtown stretch of Line 1 of the Napoli (Italy) underground involved the construction of many stations in the historic centre of the city, including the *Università* station. The name of the station is derived from the closeness to the historical buildings of the administrative offices of the *Università Federico II*. This station consists of two shafts excavated with the protection of reinforced concrete diaphragms, and two platform tunnels. The main and deeper shaft reaches the line tunnels excavated with TBMs in the formation of the *Neapolitan Yellow Tuff*, at a depth of more than 36 m below ground level and more than 33 m below the groundwater table; excavations of the shaft involved loose materials for a depth of 23 m, and the Neapolitan Yellow Tuff formation for the remaining. A secondary and shallower shaft was excavated to realize the mezzanine dedicated to ticket machines and checks. The two platform tunnels, each of them consisting in two separate stretches about 50 m long, were excavated from the main shaft using the Artificial Ground Freezing, AGF, technique to guarantee excavation stability and hydraulic seal. For space reason the paper presents the results of the monitoring carried out along the partial construction process with reference to the deep shafts for a total duration of about four years; the presented data consists of: measurements of pore water pressures inside and outside the excavations; measurements of displacement of retaining structures and ground-anchors forces; measurements of subsidence of surrounding buildings. The overall case history is a very complete one and valuable lessons are learnt on the effectiveness of the construction techniques further than on the design methodologies adopted. Qualitative and quantitative relationships among the various sets of measurements are also outlined, such an aspect representing a valuable feature of the presented case history.

Keywords: Deep excavation, Settlement, Horizontal displacement, Artificial Ground Freezing

1. Introduction

This paper is devoted to describing an interesting and rather complex case study with reference to the construction of a deep subway metro station in a rather crowded urban area. This station is part of the finale route of Linea 1, a closed ring connecting the North outskirts of the city of Napoli, the area of the hills, the historical centre, the administrative district, and the airport, for a total length of about 30 km and 25 stations. In the so called *Tratta Bassa*, which is the downtown stretch of the Linea 1, is included the *Università* station whose name is inspired by the closeness of the historical buildings of the *Università Federico II* which are now dedicated to the administration offices including the Rector's main office. As for most of the stations of the *Tratta Bassa* the construction of the station required deep open excavations in coarse-grained soils below the water table (up to a maximum depth of about 32 m below the ground surface), in a densely built urban environment. For space reason the paper reports only the results of the monitoring carried out for the four years taken for the construction of the deep station shafts while the interesting data on the platform tunnels excavated under the protection of AGF maybe found elsewhere. The monitoring was focussed on the horizontal displacement of the embedded diaphragm walls and on the settlement of the surrounding buildings. Both quantities were rather small and the whole operation was safely carried out to the end. On the other hand, settlements larger than expected were recorded during the platform tunnel excavation. On this aspect major details can be found elsewhere (Nicotera and Russo, 2021).

2. Construction site and construction steps

The *Università* station is around the middle section of the stretch of Linea 1 named *Tratta Bassa* (Figure 1) and is located in a rather large square (*Piazza Bovio*). The area surrounding the construction site is a relatively flat plain slightly sloping towards the sea. The site itself, a few hundreds of meters far from the seashore, is nearly flat with the ground level located between 5.40 m and 4.80 m above sea level. The groundwater table is located at 2.25 m a.s.l. very close to the ground surface. The station is built close to several buildings the minimum distance from these buildings being 2.80 m at the southern edge of the main shaft (see Figure 2). The location of the station within the Bovio square was primarily chosen so that both the platform tunnels and the inclined passageways were not built below the existing surrounding buildings; this conservative criterion also ensured that the two longest parallel sides of the main shaft, which would have undergone the greatest displacement during construction, were well away from the facing residential buildings (about 60 m)

2.1 Subsoil investigations

Plan views of the station with the site investigations devoted to pore pressure measurements (i.e. various types of piezometers) are reported in Figure 2. The depth of the *Yellow Tuff* bedrock in the site area is represented by *iso*-elevation contours referred to the average sea level. The top surface of the bedrock is rather regular at a level ranging between -15 m and -17 m a.s.l. within the limits of the station plan; however in the investigated area this surface gradually deepens in a south-southwest direction until it reaches a level ranging from -20 m a. s.l. to –22 m a.s.l. in the corners of the square. In the station area the shallow made ground layer is rather thick (i.e. from 7 m up to 8 m). Below the made ground layer about 5 m thick deposit of marine sandy silt representing redeposited pyroclastic material. The above layer overlays a 7 m thick stratum of pyroclastic sands and the above mentioned tuff bedrock.

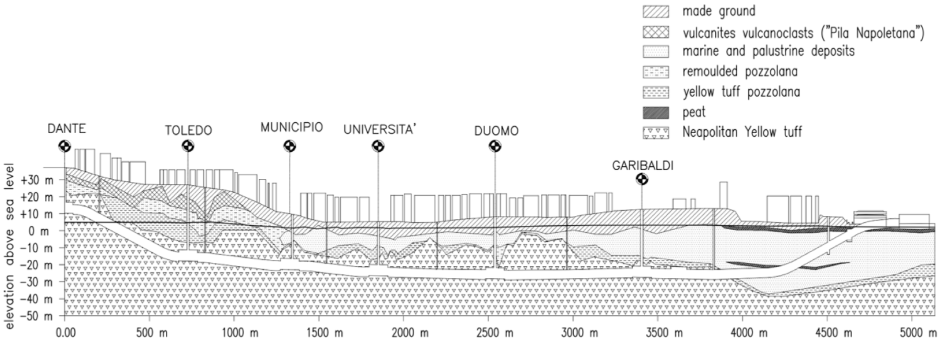

Figure 1: Naples Linea 1 – Geological profile of the *Tratta bassa* (after Maiorano et al., 2002**).**

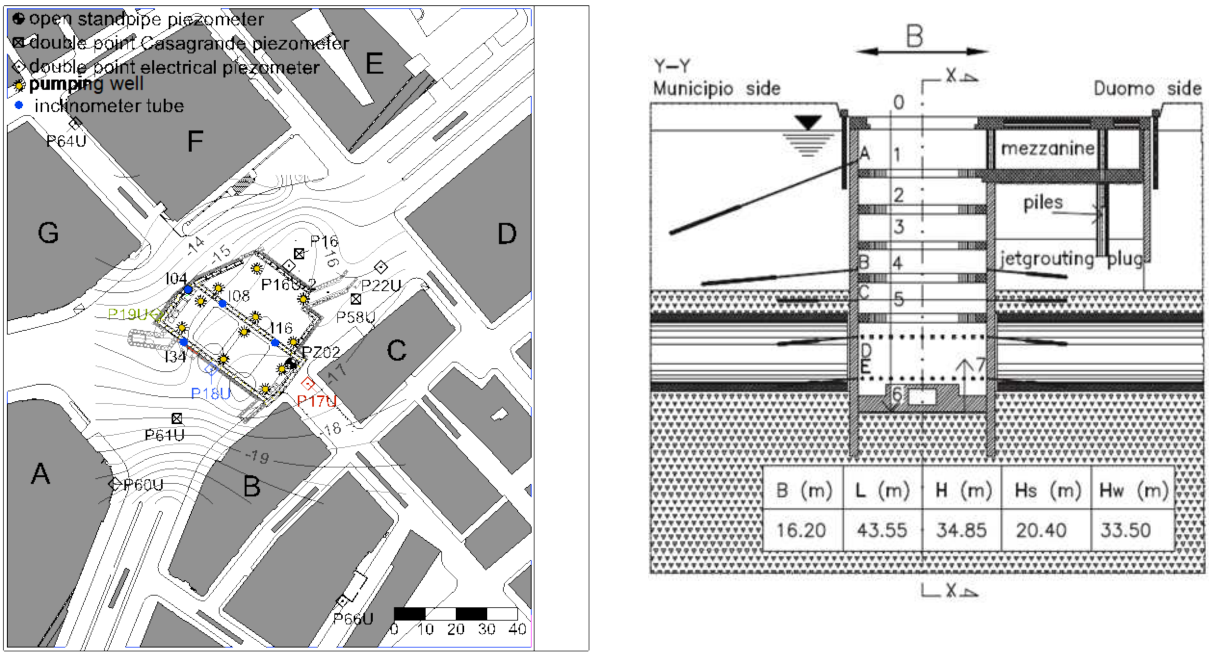

Figure 2: Station site, subsoil investigations and piezometers with level of *Yellow Tuff* top boundary (m.a.s.l.)-plan view. Section of the station shaft on the short side with the construction

2.2 Construction steps

The station is made up of a mezzanine floor (see Figure 2) that accommodates the accesses and a narrower main shaft that contains escalators and elevators for reaching the tracks level. The main shaft (see Figure 3), realized in a deep excavation (bottom level equal to - 31.25 m a.s.l.), is rectangular in plan and its inner net surface is 43.55 m × 16.20 m; the mezzanine is realized in a shallower excavation (bottom level - 4.25 m a.s.l.).

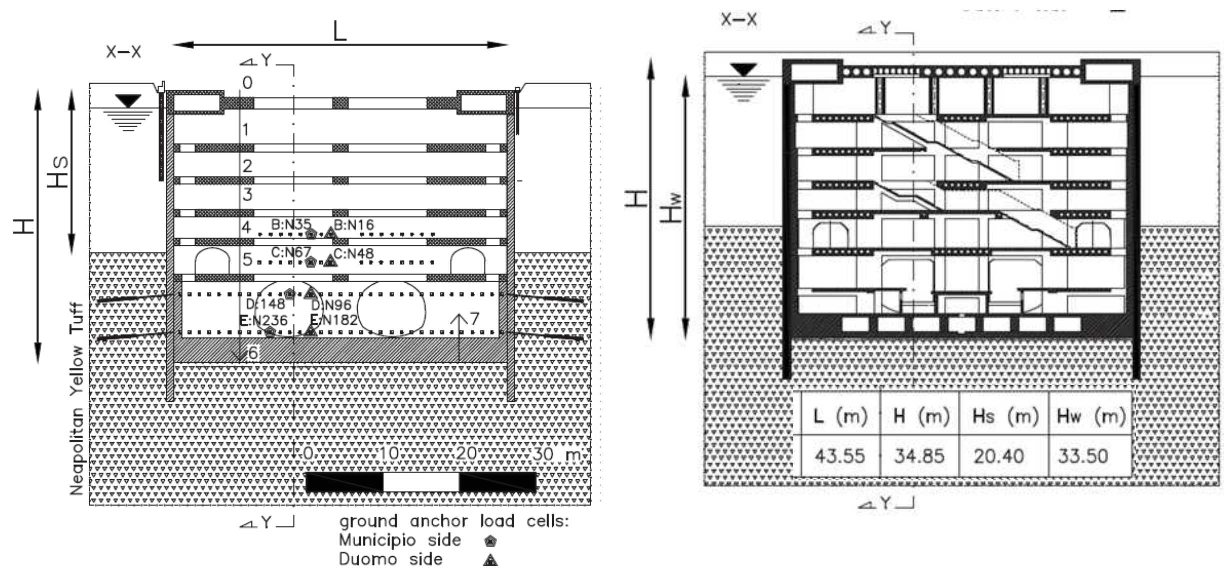

Figure 3: Sections of the station shaft: construction sequence (left) with provisional ground anchors and load cell location; final configuration (right)

The station, as described above, was built inside two shafts excavated side by side. The main shaft was excavated down to a maximum depth of 36.45 m. The upper 21.00 m were excavated through cohesionless granular deposits and the remaining 15.45 m in the *yellow tuff* soft rock formation; the hydraulic head at the bottom of the main shaft excavation was equal to about 33.50 m. The mezzanine excavation was supported by reinforced concrete diaphragm panels 2.50 m wide 0.80 m thick and 16.50 m deep, dug with a mechanical grab; the hydraulic sealing of the joints between the panels was obtained by the use of PVC stop end tubes. The main shaft excavation was supported by reinforced concrete diaphragm panels 2.80 m wide 1.00 m thick and 39.00 m deep, dug with an hydro-mill. Displacement caused by the diaphragm walls construction (i.e. excavation and casting) were not measured but results of monitoring carried out on other works in very similar subsoil conditions allow to estimate the occurrence of only a few millimetres displacements in the area immediately close to the r.c. walls (L'Amante et al. 2012; Autuori et al. 2019). The construction procedure involved partial construction of the shaft roof (identified by #0 in Figure 3); top-down construction of five r.c. slabs (id. from #1 to #5 in Figure 3); deepening of the excavation in the tuff formation down to the bottom level (id. #6 in Figure 3); construction of the bottom slab (id. #7 in Figure 3) and excavation of the platform and passageway tunnels. The design of the bracing system of the diaphragm walls was one of the most complex design issues. The bracing system was conceived to achieve two main objectives: limiting the stresses on the r.c. panels; reduce to the minimum technically possible the horizontal movements of the panels themselves in order to minimize subsidence around the excavation. The geometry of the slabs and of the roof during construction included two large openings for the vertical movements of materials; therefore, two rows (B and C in Figure 2) of prestressed anchors were realised on the side of these openings to counteract the high horizontal loads acting at least on the two deepest slabs (id. #4 and #5 in Figure 2). An additional anchors row was disposed along the longer side of the main shaft in correspondence of the shallow slab immediately below the roof (level A in Figure 2); these anchors were installed with the specific role to absorb the unbalance between the earth pressures acting on the two facing walls due to the presence of the adjacent mezzanine excavation. The contrast of the diaphragm walls in the tuff section was guaranteed by two further anchors rows (level D and E in Figure 2) realised along the entire perimeter of the shaft. A jet-grouting plug was executed below the bottom of the mezzanine; the jet-grouting columns (Ø 1200 mm) were realised upwards starting from the top surface of the yellow tuff bedrock in order to obtain a quite impervious bottom plug to minimize the water inflow towards the excavation. Later on, the r.c. roof slab was constructed at the top of the diaphragm walls in the main shaft and mezzanine; the roof slab was based both on the peripheral diaphragm walls and upon intermediate steel piers. These last piers were founded both on the main shaft long side diaphragm wall and on large diameter piles realised below the bottom level of the mezzanine excavation.

3. Monitoring plan and instrumentation layout

An intense programme of monitoring was set up at the design stage and implemented during the construction to address safety issues and to adjust design predictions in updated real time forecasting. The mitigation and prevention of excavation effects on surrounding buildings was of course the main goal. The buildings are rather tall (i.e. 6 or 7 floors) masonry brick buildings dated back to the beginning of the 20th century. The foundations are relatively shallow and consist of an enlargement of the structural section of the bearing walls.

3.1. Instrument layout

The hydraulic head variations outside the shafts were measured initially by four Casagrande twin-tube double cell piezometer (see Figure 2: P16U-1, P58U, P59U and P61U). Further measurements were carried out via both electric double cell piezometers and Casagrande piezometers (reported in Figure 2) in the period of construction of platform tunnels which is not documented in this paper. Several tens, about fifty, of optical survey points consisting of marks on the buildings around the construction area were monitored to measure the settlement. In the long-term monitoring, the overall construction process took nearly 7 year, replacement of many instruments was needed to solve both damages to the instruments and bad functioning. The marks were installed at about 3 m above the ground surface to prevent accidental hit or removal from pedestrian walking on sidewalks. Four inclinometers were installed inside the r.c. panel of the diaphragm walls (see Figure 2); the inclinometers pipes were as long as the panels they were embedded in (i.e.: 41 m for I04; 33 m for I08; 32 m for I16; 41 m for I34). The tension forces acting on eight anchors (see Figure 3) at installation and in live conditions during the excavation were measured by cylindrical vibrating wire load cells

3.2 Measurements results

An overall picture of the excavation progress and of the water table variations is provided in the diagram of Figure 4. The time axis covers the whole period of construction of the station shaft for a total duration of about four years. As a general comment on Figure 4 it is evident that at any time during the shaft construction the inner water level (empty circles) was just below the current excavation level. In the same period the outside groundwater table was only marginally affected by the pumping activities inside the station shaft, the four couples of Casagrande piezometers recording about 1 m of maximum drawdown. The progressive shaft dewatering was obtained by 11 deep pumping wells; 6 were located inside the main shaft and 5 inside the mezzanine as reported in Figure 2.

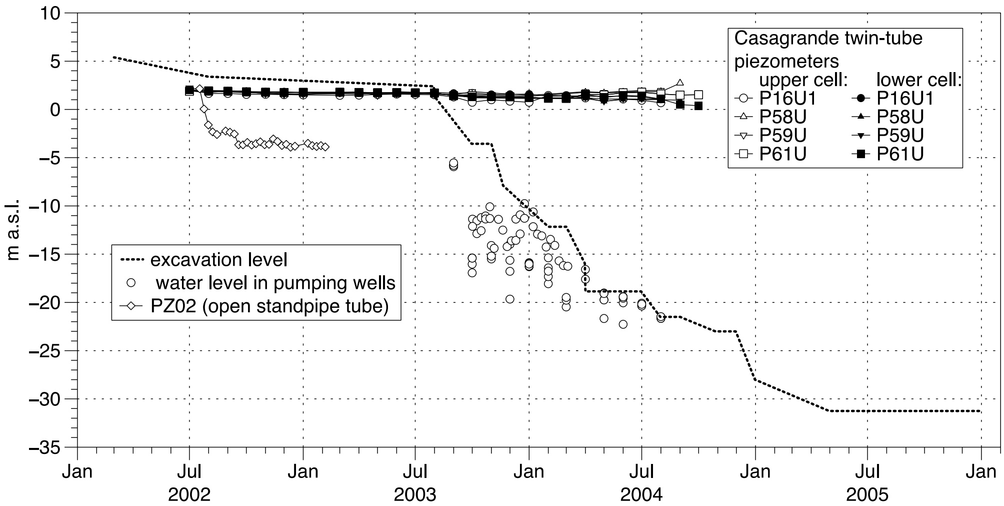

Figure 4: Excavation steps, water level in pumping wells and piezometric measurements.

Lateral movements of the diaphragm walls were monitored at all the major excavation steps. The pipes were periodically explored with manual torpedo and the readings are reported in the plots of Figure 5 for two inclinometers (I08; I16) located on the northeast long side. In Figure 6 are plotted the displacement obtained by the inclinometer I34 on the southwest long side.
It deserves mention the fact that I08 and I16 were located on the long side wall shared between the main station shaft and the mezzanine. This wall was realized with the top situated in correspondence of the elevation of the mezzanine floor extrados; for this reason, inclinometers profiles are shorter and starts from the elevation corresponding to the bottom slab of the mezzanine. Furthermore, the inclinometer I16 was damaged in the bottom part during the construction as shown by the interrupted profiles of Figure 5.

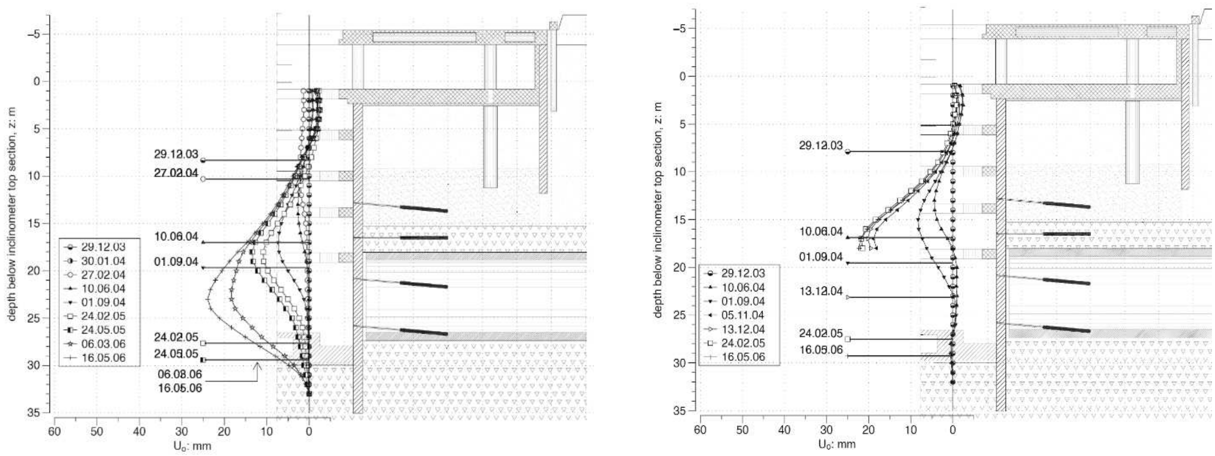

Figure 5: Horizontal displacement profiles during the main shaft construction inclinometer I08 (a) and I16 (b)

The readings of inclinometers I08 and I16 are very similar and were both affected by the unbalanced overall pressure towards the mezzanine area and the top displacements were correspondingly directed towards such area. The profile started showing bulging towards the main shaft only when the excavation proceeded down and got close to the maximum excavation depth. For the I08 and I16 the maximum horizontal movements recorded ranged between 20 mm and 25 mm and occurred at a depth below the ground surface ranging between 20 m and 25 m. Also, the inclinometer I34 was on the long side of the station shaft but opposite to the mezzanine (Md) and it was damaged during the construction process at mid depth of the shaft excavation. Initially a cantilever type movement was observed because the main station shaft moved towards the mezzanine even being the top slab a very rigid strut and the inclined top row of ground anchors working to counteract this trend. Maximum displacement at the head of I34 was however only a couple of millimetres. The measured profiles of the inclinometer I34, in Figure 6, bulged towards the inner side of the shaft at the depth of r.c. slab #3 with subsequent deepening of the excavation. The development of the horizontal displacements of the diaphragms in the zone of the inclinometers I34, was conditioned by the excavation procedure used in this area of the shaft in front of the passageway tunnels where the excavation was locally deepened of more than 5 m below the general level without the support of the r.c. slab #3 (see the red dotted line in the first diagram of Figure. 7 corresponding to the period around January 2004). Similar trends occurred in the inclinometer I04 that is not reported here for space limitations. This observation is corroborated by the diagrams reported in Figure 7 where the depth of the excavation in the main shaft is compared with the development of just the maximum horizontal displacements of the diaphragms and the tensile force in the anchorages. The maximum horizontal movement was about 10 mm for I34 whose last available readings however referred to an intermediate excavation step. al., 2012). The ground anchors were used at various levels and with different design purposes as recalled in previous sections. The range of the pretension forces extended from about 600 kN up to 1050 kN. Most of the measurements simply confirms the design value of the pretension force with a few exceptions (showing lower values). During the excavation process for many anchors the force increased with top values reaching values as high as 1500 kN (ultimate design values ranging from 2200 kN for anchors in soil layers up to 3300 kN for anchors in tuff). For the anchors on the rows D and E the load increase occurred after the completion of the excavation. In the same period horizontal boreholes for the installation of the freezing pipes were drilled.

These operations reduced locally the bending stiffness of these structural elements and caused additional displacements of the panels (see inclinometers I08 and I16) and increases in the load on the anchors.

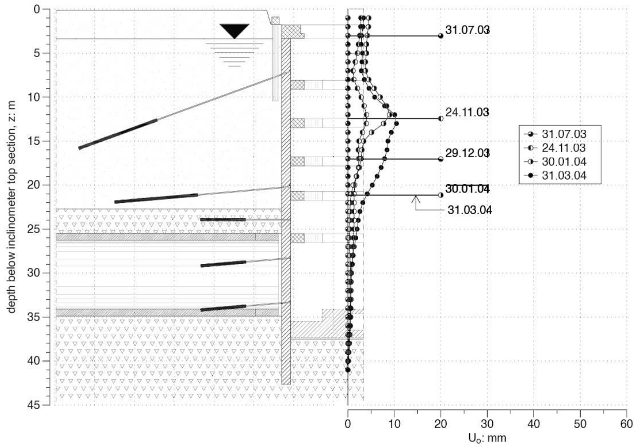

Figure 6: Horizontal displacement profiles during the main shaft construction inclinometer I34.

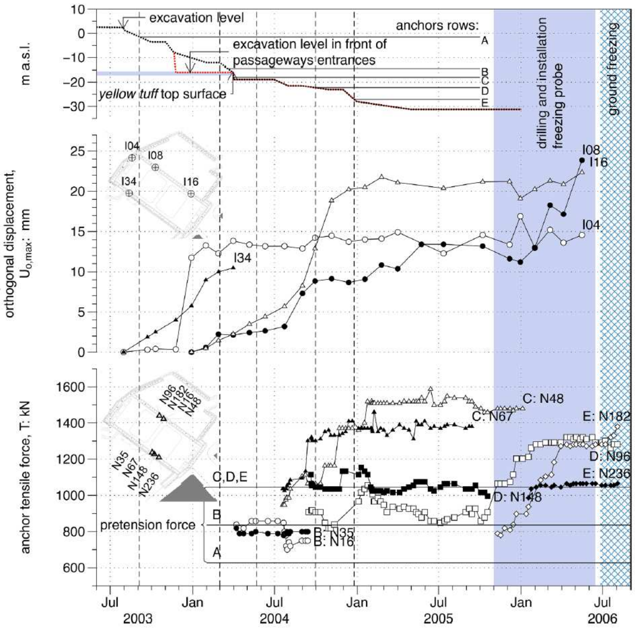

Figure 7: Panels maximum horizontal displacements and anchors tensile forces measured during main shaft construction and AGF installation

In Figure 8 a plot of the settlements measured during the station construction versus time is represented. At first glance the measured settlements are rather small, the maximum value being about 5 mm. Shaded coloured areas are used for enveloping the data measured on various buildings labelled with capital letters in the plan view of Figure 2. In the early monitoring period (January 2003 -January 2004), the recorded small movements of the buildings were in close correlation with the outdoor temperature measured at the site and reported in the upper window of the plot with a dotted line. After this initial period a clear correlation may obviously be found between the settlement trend of the buildings and the progressive deepening of the bottom of the excavation (bold line referred to the internal z axis). The settlement for the building B, i.e. the closest to main shaft, is reported in the plot with shaded cyan area and the average value at the end of the shaft construction was in the range of 2–3 mm. The building A represented by the red shaded area exhibited a slightly larger settlement in the range 4-5 mm at the end of the measurement period. These values were on the safe side and rather close to the design prediction with a maximum expected settlement less than 10 mm at the closest building. The construction operations of the station shaft were thus a great success and, as it could be expected, no damages occurred at all the surrounding structures. An important factor playing a role on the expected settlement was the groundwater table control. As confirmed by Figure 4 the groundwater table below the buildings in the area outside the station box was only marginally modified by the huge pumping activities during the excavation of the station shaft.

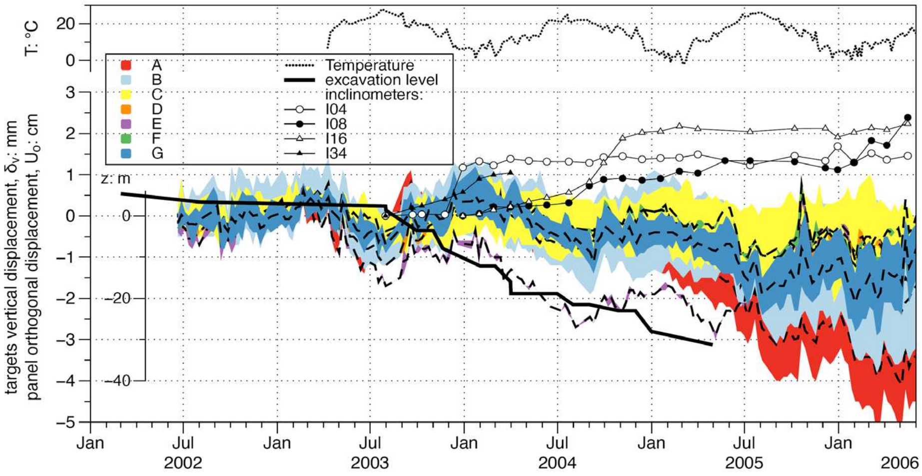

Figure 8: Settlement at all the buildings, maximum horizontal displacement of retaining diaphragms via inclinometers and progressive deepening of the excavation in the station shaft.

4. Conclusions

The data here reported have been collected along a rather long period during the construction of the station Università of Line 1 of Napoli underground network. The measurements substantially confirmed the displacement predicted at the design stage these latter being just slightly conservative as expected in complex works in densely urbanised areas with a lot risky construction stages involving groundwater management and deep excavation below and close to huge masonry built buildings.

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