MODELLING AND MONITORING OF A URBAN UNDERGROUND EXCAVATION

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ABSTRACT: Expansion works are in progress at Montesanto Station (Naples-Italy) to build, what is more, a pedestrian tunnel providing an alternative exit to travellers from two lines currently in service. The high urbanization of the area where the opera is located and the need to guarantee the transportation service to travellers during the works gave birth to the plan of the combined use of 3D numerical analyses and the real time monitoring of significant parameters (displacements, strain, stresses and temperature) to confirm the set of design criteria assumed and calibrate the design parameters affecting the problem faced.

The 3D analyses simulated the step by step excavation predicting the stress-strain behaviour; hence the comparison of the analytical predictions with the corresponding values derived through the monitoring (185 points being monitored so far) allowed to calibrate the model as the excavation advanced, thus refining the analysis itself and improving the safety level.

KEYWORDS: Calibration, Modelling, Monitoring, Tunnel.

1. INTRODUCTION

The Montesanto Station of Cumana Railway Line, owned by S.E.P.S.A. (“Limited Public Service Company”), is located in the very historical hearth of Naples (Italy) and can be deemed a critical junction of the city transportation network, given the closeness both to the Montesanto Station of the cable railway line and to the Montesanto Station of Metro Line 2, a few meters far on the West side.

Since 2004 expansion works are in progress at the Station in order to build, what is more, a pedestrian tunnel providing an alternative exit to travellers from the two lines currently in service (Cumana and Circumflegrea Lines). The high urbanization of the area where the opera is located, the nature of soils crossed, and mostly the need to guarantee the transportation service to travellers during the works, gave birth to the plan of different measures to protect the existing tunnels and to soil improvements in the excavation area.

The design of the technical interventions that could minimize the effects of the excavation on the existing tunnels was carried out by means of both 2D and 3D numerical analyses. In order to control the proper execution of the ongoing works and to verify the adequacy of the design solutions adopted, an intensive monitoring activity was planned under the supervision of the

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Civil Engineering Department of the Second University of Naples (SUN): a monitoring system based on the observation method was thus contrived; starting from the data collected through the monitoring (sidewalls displacements, stress state in the liners, etc.), the numerical model selected for the design analyses was controlled or calibrated, if necessary; in the latter case, a new verification of the design solutions adopted would have been achieved and more appropriate auxiliary solutions would have been set.

![Figure 1 - Location of Montesanto Station](image)

2. DESCRIPTION OF THE OPERA & WORK CHRONOLOGY

The 40m long, 7m large pedestrian tunnel will be perpendicular to the two upper tunnels in service (see Figure 2-a); the distance between the upper rails and the design tunnel crown will be of just 3.5m (see Figure 2-b); the connection of the pedestrian tunnel with the station outside will be guaranteed by escalators, therefore a 30° sloped 8m diameter tunnel has been already constructed (see Figure 2-c), turning into a big service tunnel (11m diameter) when going down at the design tunnel depth.

The intervention area soil, upper than the water table, is rather heterogeneous: the descending tunnel housing the escalators and the big connection service tunnel lie within a region made of Neapolitan yellow tuff, whereas the tunnel to build will lie in loose volcanic soil (except for a short first portion). The excavation, carried out by employing the traditional cut and cover technique (by cutting and temporary supporting with steel arches and shotcrete, before the permanent concrete casting) will be executed after the accomplishment of different consolidation interventions: a ring of sub-horizontal metallic pipes in the crown (7° sloped, 10m long), 5.5m overlapped with those installed in the previous step; the excavation ground will be improved by glass-reinforced plastic nailing (20mm diameter, 12m long); jet-grouting sub-horizontal columns will support the pre-support bases. Moreover, consolidation interventions have been already executed to sustain the top tunnels sidewalls, by means of 10m long micro-piles; a 0.60m deep parterre on piles has been also constructed to sustain the rails.
The excavation of the descending tunnel started on Oct. 4, 2005 and was completed by Sept. 2006 (see Figure 2), whereas the big service tunnel was completed by Dec. 2006. In the early 2007 the first connection portion with Metro Line 1 was built; the concrete casting of the as built tunnels was carried out before the excavation of the pedestrian tunnel. However, many works slowdowns occurred mainly due to the complexity of the specific yard position.

Figure 2  -  Pedestrian tunnel views: a) plan; b) section 1-1; c) section 2-2

In April 2008 the construction of the pedestrian tunnel below the two railways started. According to the design, 7 advancement steps had to be performed: a first 6m advancing step, five further 4.5m steps, and a final 6m step. At each step, before cutting and placing the presupport liner, the consolidation interventions were accomplished, while the permanent reinforced concrete liner was planned to be cast in place backwards after reaching the dig bottom, in order to speed up the advancement, given the reduced room available.

For the time being the first two excavation steps (10.5m) and the consolidation activities related to the third breakthrough have been completed, whereas the third excavation step is ongoing. The activities are proving to be very complex, due to both the reduced room available (Figure 3-a) and the need to manually carry out the cutting of the sub-vertical micro-piles consolidating the ground from the sidewalls and the foundation slab of the upper tunnels, where these interfere with the excavation front (Figure 3-b).

Figure 3  -  Tunnel excavation snapshots
3. ANALYSIS

The numerical analyses were based on both two-dimensional and three-dimensional simulations, performed by means of *Plaxis* software and *FLAC3D* software, respectively; given the complexity and the significance of the project, the two-dimensional-based design assumptions were confirmed and refined with the 3D analyses; in particular, the *FLAC3D* is an explicit finite-difference program for engineering mechanics computation that can simulate the behavior of three-dimensional structures built of soil, rock or other materials that undergo plastic flow when their yield limits are reached. The full dynamic equations of motion are used, even when modelling systems are essentially static; this enables *FLAC3D* to follow physically unstable processes without numerical distress.

The model assumed in the *FLAC3D* is constituted by a $72 \times 40 \times 50 \text{m}^3$ parallelepiped, having 8 different solid groups, each with different and specific mechanical properties (see Figure 4); besides the modelling of the different types of soil and the structural elements of the *(tuff)* existing tunnels, the step by step consolidation interventions foregoing the tunnel excavation have been integrated as well. All the mentioned groups are characterized by the Mohr-Coulomb constitutive model.

Several parametric analyses were performed in order to weigh the main variables affecting the simulation; in particular, the elastic modulus of the loose volcanic stone $E_s$ was assessed; the geotechnical investigation lead to assume a value of $E_s$ ranging between 50 and 100MPa; within this range the displacement derived with the analyses varied of about 5mm, whereas for $E_s > 100\text{MPa}$ the displacement variation highly reduced.

The results that were finally accounted for corresponded to $E_s = 100\text{MPa}$, considered the most reliable value according to the experimental campaign previously performed; the displacement of the four sidewalls relating to the Circumflegrea and the Cumana Lines
tunnels were thus derived, together with the corresponding stresses; moreover, a further inspection was provided by the stress analysis of the temporary support of the design tunnel. The displacements of the most relevant points on the existing tunnels (corresponding to some of the points monitored, see next section) were derived for each of the 7 advancing steps; Table 1 reports the vertical and the horizontal components of the theoretical displacements relating to the four sidewalls of the two existing tunnels caused by the excavation occurring beneath (the values are ordered by following the excavation advancement direction).

Table 1 - Displacement of sidewalls relating to the existing tunnels

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4. MONITORING AND CONTROL PLAN

The monitoring and control plan (Figure 5) is given by two independent measurement systems, both automatic and manual. The automatic system provides the control of vibrations, movements and stresses variations induced in the liners by the excavation operations: the first is based on the use of 4 speedometers laid on structures placed both outside and inside the station; the second is based on the employment of 56 electro-levels (36 horizontal and 20 vertical) to control the sidewalls movements, placed on a 18m long line along the new tunnel axis; the third system is based on the use of load cells inserted within the existing tunnels liners and on vibrating wire gauges installed on both the tension and the compression sides of the pre-support steel arches of the pedestrian tunnel. In particular, to control the stress state of the existing tunnels liners, 4 monitoring sections are identified, two for each tunnel, each one having 5 load cells; as for the control of the stress state in the pre-support liner of the pedestrian tunnel, five sections were instrumented, each one having 6 control points, with a total of 12 gauges per section liner.

In February 2007 two cracks-gauges were installed as well, in order to monitor the opening over time of a crack occurred on Vicereale Wall, retaining the cable railway line. All sensors installed converge into automatic switchboards that record the data at time intervals scheduled depending on the excavation operations and via modem transfer them to the Administration, to Building Contractor Bureau, and to the Faculty.

The manual system is given by a network of benchmarks, controlled by means of topographic levelling: currently 40 measurement benchmarks have been installed; plus, 13 benchmarks needed to monitor the displacements of both the existing tunnels sidewalls and the structures nearby the station (a residential building and the Wall 1, see Figure 5); 34 optical surveys have been placed as well, inspected by employing a total station, in order to gather the convergence measurements in the tunnel and to monitor a supporting wall that will be directly influenced by the work in progress activities (“Paradise Stairs” supporting wall).

The monitoring plan is completed by the temperature control on site by means of thermal sensors properly placed. The total number of points under control is 185 so far (see Figure 5).
5. MEASURES

The most significant displacement measures gathered so far relate to the cable railway restraint wall and to the first sidewall of the Circumflegrea Line (that closer to the excavation front); Figure 6 reports the displacements observed along this wall from the 29th of June 2006 (zero value) to the 8th of July 2008 (last available scanning; vertical dot-dashed lines point out the most significant dates of the excavation history so far).

All the benchmarks had vertical displacements, with a maximum value of 25mm at benchmark # 6. The deformation layout clearly shows a sagging zone and a hogging zone. The maximum angular distortion recorded so far ($\beta_{\text{max}} = 7.1 \times 10^{-4}$, recorded on June 25th 2008 between benchmarks # 4 and 5) is lower than the threshold prescribed in the scientific literature to avoid the structural damage of the type of structure investigated ($8 \times 10^{-4} < \beta_{\text{adm}} < 3 \times 10^{-3}$, Day 2000).

Figure 7 depicts the displacements gauged on the 1st Circumflegrea sidewall so far, confirming that the automatic and manual control systems are in good agreement.

As for the accuracy of the tunnels displacements measured, the scientific literature does not give specific indications, despite the existence of case studies relating to tunnels interested by new forthcoming tunnel excavations. Nevertheless, it is certainly possible to set a displacement threshold related to the structural safety of the two tunnels in service. For this purpose it was necessary to know the mechanical and geometrical characteristics of the existing tunnels liners, and to assess the initial stress state. Hence, two different experimental campaigns were performed along with time (June ‘05 and February ’07), made of coring, single and double jack tests carried out on specimens taken within the intervention area soil.

The tunnels liners were found to be made of piperno masonry, red bricks masonry and variable thickness concrete strata. The single jack tests allowed measuring the service stresses, ranging between 0.35 and 2.40MPa; the double jack tests showed an ultimate stress value higher than 3.50MPa. The displacement scannings were aided by stress scannings given by load cells installed on two sections of the Circumflegrea tunnel astride the tunnel to build...
Figure 6 - Time evolution of displacements
(1) Work start; (2) Completion of descending tunnel; (3) Completion of service tunnel; (4) Pedestrian tunnel excavation start.

Figure 7 - Displacements of 1st sidewall
(the 1st in March '07, and the 2nd in June '08); the pressure variations are reported in

Figure 8, together with the temperature variations. The maximum compression value (~8bar) was detected on cell B2 during the 2nd excavation step. It can be observed that the pressure variations seem to be strongly influenced by the temperature variations.
6. COMPARISON OF THEORETICAL AND EXPERIMENTAL RESULTS

The calibration of numerical analyses carried out to predict the displacements expected after each excavation step, strictly depended on the mechanical parameters initially based on the experimental test results; afterwards, with the measures progressively gathered in terms of sidewalls displacements, existing and as built liners stresses, a further calibration of such parameters could be performed in the pipeline.

After the recent completion of the second excavation step, the comparison between the analytical and measured displacement values could be performed, as shown in Figure 9 (the zero displacement value refer to the construction of the pedestrian tunnel below the two railways).

It is clear that the expected displacements overestimate the values measured: it is believed that such disagreement might be due to three main factors:

- the uncertainty related to the assessment of the Elastic modulus, $E_s$;
- the model did not take into account the aforementioned consolidation interventions of the upper sidewalls;
- the simulation of the progressive excavation determines displacement values that do not take into account the time dependence; this implies that the displacements expected involve delayed adjustment phenomena (due to the viscosity behaviour of soil) that might occur along with time.

Hence, the set of esteemed displacements shall be deemed as an upper bound for the measured data.

Nevertheless, the trend line of the two curves confirms that the model approach fits the effective general behaviour of the complex space investigated.

The stability verification of the 3D model corresponding to the displacements derived was supported by the verification of the stress levels of the existing liners; in particular, the maximum differential vertical displacements observed between the adjacent sidewalls after the 7 advancing steps simulated was equal to 8.4mm, derived between the two sidewalls of Circumflegrea tunnel, occurring at the 2nd excavation step; the maximum compressive stress value derived on the relevant tuff liner was found to be 3.5MPa, which is lower than the ultimate experimental value. Figure 10 illustrates the stress state just discussed.
Figure 9 - Comparison between theoretical and measured vertical displacements

Figure 10 - Maximum compressive stresses on the liners at 2nd excavation step

7. CONCLUSIONS

In the present paper the main issues related to the accomplishment of underground structures in deeply urbanized and highly historical and monumental areas has been illustrated.

With reference to the new pedestrian tunnel serving the Montesanto Station, it has been shown how an integrated approach between 3D numerical analyses and real time monitoring of significant parameters as displacements, strain, stresses and temperature can confirm the set of design criteria assumed for the specific project, and in the meantime can refine their use by extending its application field through the proper calibration of the manifold parameters involved in the definition of complex works interacting with earthy materials which behavior is very unclear and thus needing deep investigations to support the design.
8. ACKNOWLEDGMENTS

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9. REFERENCES
