Geotechnical Engineering for the Preservation of Monuments and Historic Sites III

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Geotechnical Engineering for the Preservation of Monuments and Historic Sites III – Lancellotta, Viggiani, Flora, de Silva & Mele (Eds)

Tunnelling effects on the Basilica di Massenzio: Computed and observed displacement fields

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ABSTRACT: The contract T3 of the new Line C of the Rome underground is 2.8 km long and includes two stations and two ventilation shafts. It runs under the archaeological artefacts of the historical centre of Rome which is a UNESCO World Heritage site, an area with high density of monuments and historical buildings. Metro C, in cooperation with an interdisciplinary Scientific Technical Committee, has carried out several detailed studies to analyse the potential interactions of the monuments/historical buildings with the new line C, in order to identify the most appropriate and effective mitigation techniques to be adopted to prevent any damage. This paper describes the procedures that have been followed to safeguard the monuments/historical building interacting with the line during its construction, from the approach adopted to study the line-monument interaction at different levels of complexity, to the comparison between the computed and the observed displacement fields for the case of the *Basilica di Massenzio*.

1 INTRODUCTION

Line C is the new line of the Rome underground that includes 30 stations. Once completed, it will cross the city from the North-West to South-East, for a total length of 25.6 km, almost doubling the extent of the existing underground network. The new line C is then an infrastructure of outstanding importance for the public transport system of Rome, due to its high population. It is also the first fully automated underground line in the city.

As General Contractor, Metro C manages the construction of the Line C in its implementation and operational phases including the design, the archaeological surveys, tunnelling, stations constructions and trains manufacturing, till the start-up. The activities started in 2006 with the archaeological surveys and the design stages. At present 22 stations and 18 km of the line are in operation.

During the design stage, Metro C set up a multidisciplinary Scientific Technical Committee (*STC*) with the assignment of implementing all the necessary procedures to safeguard the historical monuments of Roman age potentially interacting with the new Line C. A methodological approach was adopted to perform the line-monument interaction studies, following procedures at two increasing levels of complexity: *green-field* (*Level 1*) analyses were first carried out ignoring the stiffness and weight of the existing monuments for a simplified evaluation of the potential damage induced by tunnelling; Finite Element (FE) interaction (*Level 2*) analyses were then carried out in both two- and three-dimensional conditions, accounting for the stiffness and weight of the monuments (Burghignoli et al. 2013).

This approach allowed the potential interaction of each monument with the line to be evaluated, this permitting to design an appropriate monitoring system to detect the displacement field induced in the monuments by the line construction. This ensured a reliable control of the construction processes providing the information needed to support the decision-making process. Based on the results of the analyses performed at the design stage, appropriate geotechnical and structural protection measures were adopted for the monuments when necessary (e.g.: Rampello et al. 2019; Masini & Rampello 2021).

The aforementioned approach was implemented for the first time in Italy by Metro C involving a deep interaction between geotechnical and structural engineering. This resulted in achieving the objective of completing contract T3 of the new Line C of Rome underground without inducing any significant effect on the monuments adjacent to the line, of outstanding archaeological and historical value. Moreover, monitoring data were seen to be in a fair agreement with the design predictions. In this paper a description of the geological and geotechnical context of the T3 Contract of the Line C of the Rome underground is first given together with a description of the activities carried out by the *STC*, and the comparison between the computed and observed displacement fields is then shown referring to the *Basilica di Massenzio*.

2 THE CONTRACT T3

The contract T3, currently under completion, runs under the historical centre of Rome starting from *San Giovanni* station, for a length of about 3 km: it includes the *Amba Aradam/Ipponio* and the *Fori Imperiali* stations and two ventilation shafts, shaft 3.2 and shaft 3.3 (Masini et al. 2021) (Figure 1). The twin running tunnels were excavated using two EPB-TBMs (Earth Pressure Balance - Tunnelling Boring Machines) at depths in the range of 20 to 50 m. The tunnels have outer and inner diameters of 6.7 m and 5.8 m, respectively, with the lining made by 0.3 m-thick precast concrete segmental elements. The face support pressure applied by the TBMs to balance the total horizontal stress acted by the soil at the tunnels depth was in the range of 200 to 480 kPa depending on the excavation depth, while the tail void injection pressure was about 50 kPa higher, to ensure the gap between the excavated soil and the lining to be filled.

Figure 1. Aerial view of Contract T3.

Several campaigns of in situ and laboratory geotechnical investigations were carried out in the design stages, dating back from to 1995 till the last one, carried out in 2010–2011. In situ tests involved dynamic and static penetration tests, the latter also instrumented with pore water transducers, cross-hole, dilatometer and Lefranc tests. About 344 undisturbed samples were retrieved to evaluate the strength e stiffness parameters of the soils, also performing triaxial tests instrumented with local axial strain transducers and bender elements, as well as resonant column tests, to define the small strain stiffness of the soils. This led to an accurate definition of both the soil stratigraphy and the mechanical parameters of the soils interacting with the line.

The geological profile of Contract T3 is shown in Figure 2: Line C runs from *San Giovanni* station towards *Venezia* station at an elevation of about 9.5 m a.s.l. (depth $z \cong 27$ m). The elevation of the tunnels gradually reduces to about 5 m a.s.l. near *Piazza Celimontana* ($z \cong 52$ m), and then increases again to about 0.25 m ($z \cong 27$ m) at *Fori Imperiali* station; from this station the line deepens again towards *Venezia* station reaching an elevation of −10 m a.s.l. ($z \cong 37$ m). From *San Giovanni* to *Amba-Aradam* stations the tunnels runs mainly into the fine grained Pleistocene and Holocene deposits, consisting of sandy silt and clayey silt (LSO, Ar-St); after a passage through the Pleistocene sandy gravel (SG), the tunnels enter the base stiff and overconsolidated Pliocene clay (Apl) to emerge again near the *Coliseum* into the overlying coarse grained soils, consisting of Pleistocene sandy gravel (SG) and volcanic medium to fine silty sand (Tb). After the *Basilica di Massenzio* the tunnels deepen being excavated in the Pliocene base clay (Apl) before reaching the *Venezia* station. Here there in an abrupt change in the geological environment, as the tunnels enter the Tiber valley with its Holocene alluvial fine grained soil (Ag). The Made Ground (MG) overlying the Pleistocene and Holocene deposits is 7 to 11 m thick with local maximum thickness of 18 m. It consists of coarse grained materials containing the remnants of the ancient city, of immense archaeological value.

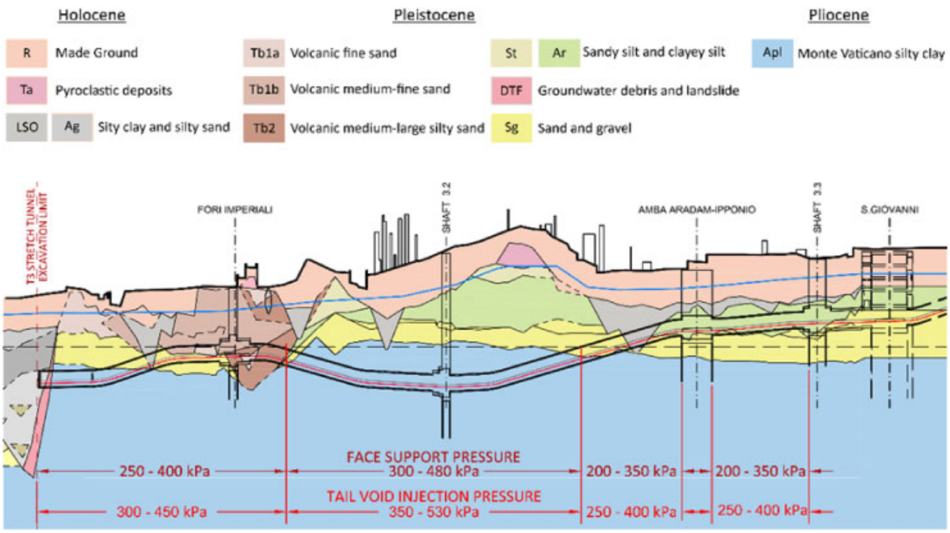

Figure 2. Geological profile along contract T3.

The pore water pressure regime in the central stretch is about hydrostatic, with the water table located at depths $z = 10$–15 m and local small downwards gradients.

3 THE SAFEGUARD OF THE MONUMENTS

The historical centre of Rome is a UNESCO World Heritage site so that Metro C set up a multidisciplinary Scientific Technical Committee (*STC*) to safeguard all the monuments or historical buildings interacting with the line. The *STC* had to ensure high quality procedures for evaluating the effects induced by tunnelling and deep open excavations on these monuments and providing design guidelines.

Prior to any analysis, preventive studies were conducted on the monuments adjacent to line C to identify materials and geometry of the foundations, the mechanical properties of the materials, the construction technologies and phases, the structural layout and the existing crack patterns.

Evaluation of tunnelling-induced settlements in *green-field* conditions was then carried out using well known empirical relationships in order to exclude from the interaction studies the monuments located outside the ground settlement trough. In these analyses, named *Level 1* analyses, the settlements were computed at the foundation level ignoring the presence of the monument.

In the empirical relationships, the settlement trough is described by a Gaussian curve in a section transverse to the tunnel axis, and by a cumulative probability function in the longitudinal direction. For sake of space, a brief reference is made in the following to the transverse section only.

The transverse surface settlement trough was evaluated through Eq. (1) (O'Reilly & New 1982; Peck 1969):

$$w(x) = \frac{V_L D^2}{4i}\sqrt{\frac{\pi}{2}}\exp\left(-\frac{x^2}{2i^2}\right) \tag{1}$$

where $w(x)$ is the settlement at a distance x from the tunnel axis, $D = 6.7$ m is the tunnel diameter, V_L is the volume loss, prescribed to be not higher than 0.5%, and i is the distance of the point of inflection of the settlement trough from the tunnel axis.

To evaluate i at depth z the expression of Moh *et al.*, (1996) was used:

$$i(z) = K \cdot z_0 \left(\frac{z_0 - z}{z_0}\right)^m \tag{2}$$

where z_0 and z are the tunnel axis and the foundation depths, respectively. For Contract T3 a width parameter $K = 0.4$–0.5 was assumed at ground surface, and an exponent $m = 0.6$.

Following O'Reilly and New (1982), the horizontal displacements were computed assuming the ground displacement vectors to be directed towards the tunnel axis.

In evaluating the settlement trough, account was taken for the variations in plan and depth of the tunnels and the presence of the twin tunnels was accounted for by superposition of each tunnel effect.

Once the subsidence profile was computed, the maximum free field deflection ratio in sagging and hogging (Δ_s/L_s, Δ_h/L_h) underneath the foundation were evaluated together with the horizontal strain ε_h: these quantities were used to evaluate the potential damage induced by tunnelling through the interaction diagrams proposed by Burland and Wroth (1974) and Burland (1995), that relate the deflection ratio and the horizontal tensile strain to six damage categories. To account for the historical value of the monuments, lower threshold tensile strains defining the damage categories, than the ones proposed by Boscardin and Cording (1989), were adopted in the study.

When damage evaluation exceeded the category 0, associated to negligible effects, *Level 2* soil-structure interaction analyses were carried out in two stages.

Specifically, the interaction between the tunnels and the monuments was studied through 2D or 3D Finite Element (FE) analyses that addressed soil-structure interaction by adopting a simplified description of the mechanical behaviour of the monuments, often assumed as linear elastic. The displacement field computed by the geotechnical interaction analyses, accounting this time for the stiffness and weight of the monument, was then applied to a detailed structural model of the monument thus computing the state of stress and strain induced by tunnelling-induced ground movements. The level of damage was finally re-evaluated using both the Burland and Wroth (1974) procedure or considering the field of maximum tensile strains computed by the structural analyses. Depending on the computed results, either the damage was deemed acceptable, or protective intervention were suggested by the *STC*.

The *Basilica di Massenzio* is referred to in the following to illustrate the adopted approach. Both *Level 1* and *Level 2* analyses were performed for this monument and safeguarding interventions were designed to prevent any damage eventually induced by tunnelling.

4 THE BASILICA DI MASSENZIO

The *Basilica di Massenzio* is located at the end of the Contract T3, close to the *Fori Imperiali* station. Its construction began on the northern side of the forum under emperor Massenzio in 308, and was completed in 312 by Costantino I.

In its original configuration, the building consisted of a central nave (about 80 m long and 25 m wide), covered by three vaults (35 m high) on four large piers, and ending in an apse at the western end, and of two flanking aisles spanned by three semi-circular barrel vaults perpendicular to the nave. Excluding the apse, the building occupied a rectangular area of about 80×60 m^2 (Figure 3a). The perimeter walls of the Basilica, as well as the internal baffles, consist of two facings of clay bricks (*opus testaceum*) and a core of Roman conglomerate of lime and pozzolana (*opus caementicium*) including aggregates of different materials. The vaulted structures as well as the foundations are made in *opus caementicium*.

In the fourth and fifth centuries the Basilica underwent several modifications, including the creation of the apse on its North-Western side, and the construction of a retaining wall to support the Velia Hill. In the sixth century, the Basilica had been already abandoned.

The south and central sections were probably destroyed by the earthquakes of 847 and 1349 (Figure 3b). What is left of the monument consists of the three large barrel vaults forming the aisle parallel to *via dei Fori Imperiali* (Figure 3c-d): each vault spans 20 m and is supported by massive walls. The two side vaults are closed by a thinner wall facing *via dei Fori Imperiali*, with two levels of three arched windows, whereas the central vault terminates with the apse constructed by Costantino.

The first excavations to restore the Basilica to its original level began in the nineteenth century and in 1932 the excavation works were carried out to remove the Velia Hill and make room for the new *via dell'Impero*. Exposed by these works, the structure revealed the presence of an extensive pattern of cracks and significant damage in the vaults and the apse. In the 1960s, Musumeci reconstructed the destroyed dome of the apse in reinforced concrete (Figure 3d).

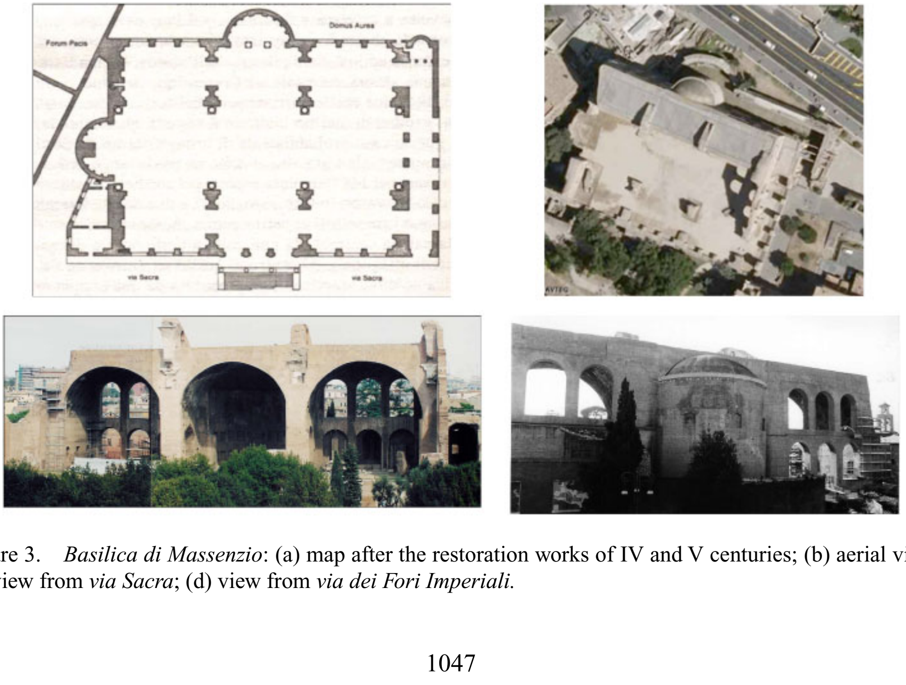

Figure 3. *Basilica di Massenzio*: (a) map after the restoration works of IV and V centuries; (b) aerial view; (c) view from *via Sacra*; (d) view from *via dei Fori Imperiali.*

4.1 *Geotechnical analyses*

Figure 4a (after CISTEC, 2001) shows a scheme of the foundations of the Basilica with the depth of the bearing walls increasing moving towards *Foro di Cesare*, from $z = 4$ m (23 m a.s.l.) for the *Pilone Colosseo* to $z = 14$ m (13 m a.s.l.) for the *Pilone Carinae*, following the original ground level of the Velia hill before the construction of the Basilica.

For the final design stage, referring to the definitive route of the Line C, *Level 2* interaction analyses were only carried out due to the complexity of the structural layout and the outstanding value of the monument. In the 3D FE analyses, the real geometry of the problem was simplified as shown in Figures 4b-c, assuming a fictitious axis of symmetry between the two tunnels, excavated at an axis-to-axis spacing of 14 m, at a depth of 25.5 m. The model assumption that the two tunnels are excavated at the same time was deemed reasonable in that on the other side of via *dei Fori Imperiali*, opposite to the Basilica, there is a high hill (the *Belvedere Cederna*) and the second tunnel is far away from the Basilica, hardly affecting its behaviour. In the model, the tunnel is assumed to be parallel to the ancient wall retaining the Velia hill, with an axis-to-Basilica distance equal to the shortest one (16 m), that occurs next to the *Pilone Colosseo*.

The numerical model was 212 m long, 91 m wide and a 37 m deep, with its base located at the top of the layer of the stiff clay (Apl); vertical boundaries of the FE mesh were restrained horizontally, for out-of-plane displacements, while the nodes of the bottom boundary were restrained both horizontally and vertically. In the direction of tunnel axis the size of the element is constant and equal to 2 m, that corresponds to the excavation step adopted in the numerical analyses. The mesh included about 72000, 10-node, tetrahedral elements with a second-order interpolation of displacements and a linear interpolation of strains.

In the simplified structural scheme adopted in the geotechnical analyses, only the embedded portions of the Basilica and of the retaining wall were modelled, using equivalent unit weights for the different portions of the Basilica in order to reproduce the vertical loads applied by the bearing walls to the foundation soils.

The mechanical behaviour of the foundation soils was described by an elastic–plastic, rate-independent constitutive model with isotropic hardening and Mohr-Coulomb failure criterion, the *Hardening Soil* model implemented in the Plaxis 3D suite (Brinkgreve et al. 2013), using the strength and stiffness parameters obtained from the available *in situ* and laboratory tests (Soccodato et al. 2013). The massive bearing piers and walls were instead modelled with an isotropic linear elastic-perfectly plastic model, with Mohr-Coulomb failure criterion, specifying a limiting tensile stress ($\sigma_t = 0.5$ MPa) (Soccodato et al. 2013).

The analyses were carried out assuming drained conditions due to the medium-high permeability of the layers of silty sand (St-Ar) and sandy gravel (SG): pore water pressure is hydrostatic with the water table located at $+10$ m, a.s.l..

The 3D FE analyses were performed as follows: (i) geostatic equilibrium at $+27$ m a.s.l.; (ii) activation of the elements modelling the Basilica and the wall; (iii) excavation at the elevation of *via dei Fori Imperiali* ($+23$ m a.s.l.); (iv) simulation of tunnels excavation. This last stage basically consists in (i) first applying a given semi-elliptical vertical displacement profile at the upper half of the tunnel boundary, immediately at the back of the shield tail, and (ii) then releasing this forced displacement profile when the permanent lining is erected (Rampello et al. 2012). The maximum displacement that had to be applied at the tunnel crown to attain the maximum volume loss ($V_L = 0.5\%$) prescribed in design was found by trial and error.

The *green-field* settlements computed by the FE analyses at section *GF1* ($+23$ m a.s.l.) for $V_L = 0.5\%$ were in a good agreement with those provided by the empirical relationships using a width parameter $K = 0.45$ at ground surface, providing a settlement $w \cong 2$ mm at the abscissa corresponding to the edge of *Pilone Colosseo*, the closest to the tunnel.

Figure 4c shows the tunnel face advancements for which the results of the 3D FE analyses were extracted, each corresponding to the position of the bearing piers of the Basilica. In Figure 4d the settlement profiles computed at the foundation level of each pier are compared with the green field evaluation. Maximum settlements of 4.8 mm, 3.5 mm and 2 mm are computed for the *Pilone*

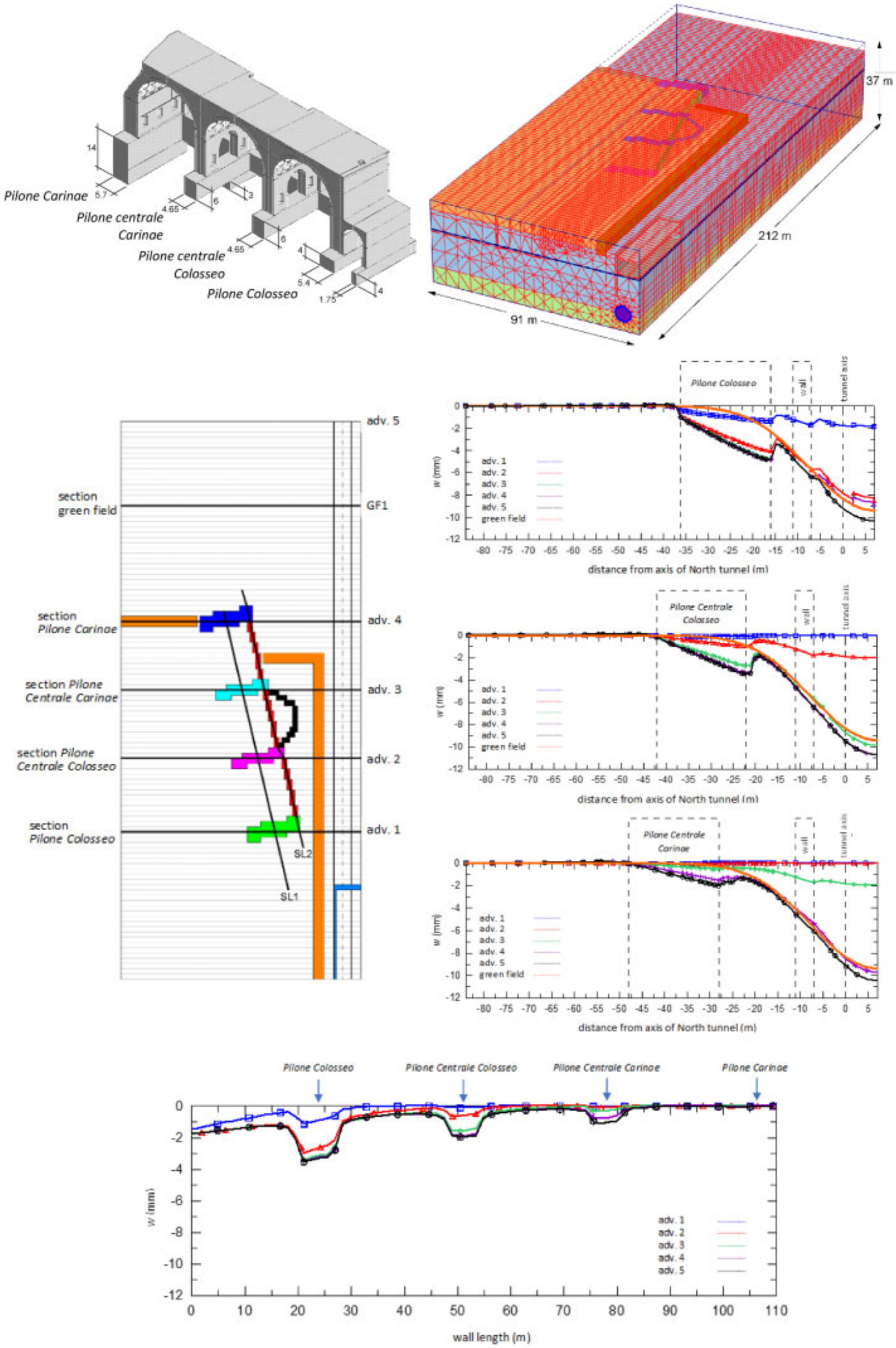

Figure 4. *Basilica di Massenzio*: (a) foundations geometry; (b) 3D numerical model; (c) plan view of the model with tunnel advancements; (d) settlement profiles with tunnel advancement for the bearing piers; (e) settlement profiles with tunnel advancement along *SL2* section.

Colosseo, *Pilone Centrale Colosseo* and *Pilone Centrale Carinae*, while nearly zero settlements, not shown in the figure for sake of space, are computed for *Pilone Carinae*, characterised by the deepest foundation. Along section *SL2*, along the wall facing *via dei Fori Imperiali*, a maximum differential settlement of about 4 mm was evaluated (Figure 4e).

It is seen that, when simulating the *Basilica di Massenzio*, the interaction analyses provide substantially larger settlements (Figure 4d), this contradicting the common belief that *green-field* analyses are more conservative than the interaction analyses. The reason for this unusual behaviour, unique among the studies carried out for Contracts T2 and T3 of Line C, can be attributed to the structural features of the Basilica. This monument is indeed characterised by an extremely small value of the ratio of the area of the bearing structural members to the total covered area: about 12%, compared to 23% of the Pantheon and 26% of S. Peter's Basilica; this is likely to be responsible for its high structural vulnerability and the many collapses experienced in the past.

Because of the significant weight of the structure, the foundation soil experienced high deviatoric stresses undergoing a reduction of the tangent stiffness. Reduction in the shear modulus is actually reproduced by the constitutive model adopted in the numerical analyses, which accounts for the dependency of soil stiffness on strain level, so that even the relatively small changes of the stress state induced at the foundation level by tunnelling produce appreciable settlements due to the reduced tangent stiffness.

Although the Burland and Wroth (1974) interaction diagrams provided negligible (class 0) damage to the Basilica, a *Level 2* structural analysis was also carried out, due to the unvaluable value of the monument, applying the displacement field computed by the geotechnical analyses to a detailed structural model of the Basilica, whose structural members were assimilated either to linear elastic or to nonlinear materials.

4.2 *Structural analyses*

In the 3D FE numerical model of the Basilica nine materials were distinguished on the basis of the in situ investigation carried out by Metro C, consisting in both mechanical tests on samples retrieved from the masonry and non-destructive tests (single and double flat jack, sonic or ultrasonic measurements). In the linear-elastic model of the Basilica, frictional interfaces were introduced at the contact between the vaults and the bearing walls, as well as at the key of the barrel vaults, in order to reproduce the most important existing cracks. In the nonlinear model (model *concrete* in ADINA) values of tensile strength in the range $\sigma_t = 0.2$–2.0 MPa were defined for the Basilica materials and the crack opening was attained in the analyses for tensile stresses higher than σ_t. The computed field of crack opening resulted consistent with the one observed, though providing slightly lower width of the cracks.

In both models, an elastic layer of foundation soil, 1 m thick, was introduced at the base of the structural model to better reproduce the stress state induced by the Basilica self-weight, as evaluated by the aforementioned tests. The elastic modulus of this layer was calibrated to reproduce the best agreement between the computed and the measured stress state.

Both the linear elastic and the nonlinear models provided very similar results so that only those computed by the second one are discussed in the following.

Figure 5a–b shows the contours of the maximum tensile strain computed in the Basilica after tunnels excavation: the most critical conditions occur for the apse, at the junction with the recent reinforced concrete dome. It is worth noting that the state of strain in the structure is mainly due to its self-weight, being only slightly modified by the tunnelling-induced displacements applied at the base of the model. In fact, the maximum tensile strain in the structure, that is equal to about $3 \cdot 10^{-4}$ before tunnels excavation, is hardly affected by tunnelling. According to the limiting tensile strain assumed in this study, the Basilica remains in a state of negligible damage after tunnelling (class 0: $\varepsilon_{lim} < 4 \cdot 10^{-4}$).

However, the historical studies and the observation of the current condition of the monument, supported by the results of the numerical analyses, suggested the opportunity of setting up some structural safeguarding interventions, consisting of tie rods, to reduce the thrusts of the large barrel

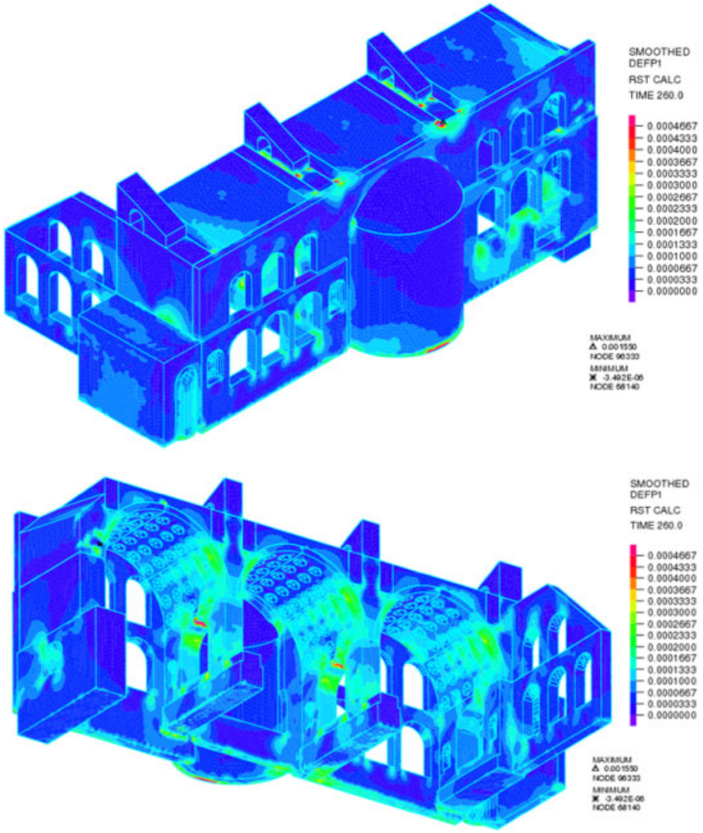

Figure 5. Contours of maximum tensile strains computed by the nonlinear model: (a) top view from *via dei Fori Imperiali*; (b) bottom view from *via Sacra*.

vaults. Also, the interaction analyses suggested the opportunity of adopting localised structural safeguarding interventions in the wall with the three-light windows facing *via dei Fori Imperiali* and at the connection between the apse and the Basilica.

4.3 *Structural interventions*

Before tunnelling Metro C implemented temporary and definitive safeguarding interventions to reduce the thrusts of the barrel vaults under the permanent loads, the cracks opening in the bearing walls and around the three-light windows, and to link the apse to the wall of the Basilica facing *via dei Fori Imperiali*.

The main definitive structural interventions consist of reinforcements with a couple of steel wire ropes of diameter $\phi = 28$ mm installed around the apse at two different elevations, 45 m and 39 m a.s.l., and preloaded to 117 kN and 235 kN, respectively (Figure 6a).

Longitudinal and transversal chains were also installed at 39 m and 26 m a.s.l.. The longitudinal chains consist of three couples of steel bars of diameter $\phi = 40$ mm installed in the perimeter walls of the Basilica and connected to the intermediate transversal walls through steel plates, 30 mm thick (Figure 6b). The transversal and longitudinal lower chains have been pre-loaded to 100 kN, while the longitudinal and transversal upper chains have been pre-loaded to 530 kN and 400 kN, respectively. This preloading was necessary to recover part of the deformation of the structure due to the distancing of the walls of the end vaults, exceeding the minimum value necessary to absorb the vault thrusts, equal to 250 kN.

On both sides of all bars to be pre-loaded, a total of 52 load cells were installed to control the forces transmitted to the bearing walls of the Basilica during the stringing activities and tunnels excavation. The stringing activities were divided into two stages, applying 50% of the design loading for each of them.

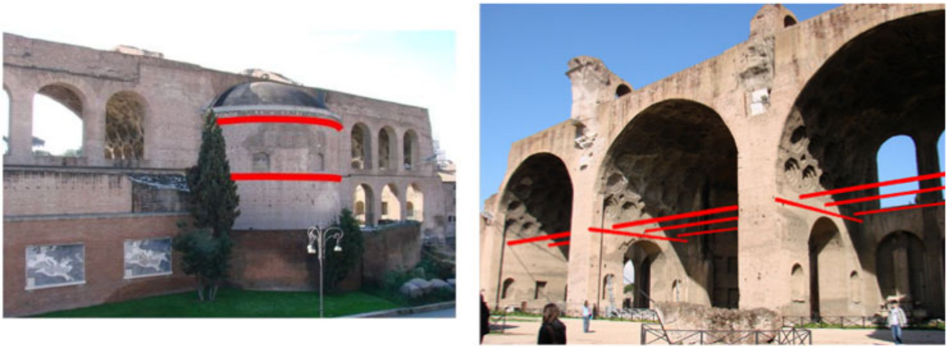

Figure 6. Structural interventions on the Basilica.

At the end of the first loading stage the monitored loads were checked to be consistent with the design assumptions, since thermal variations could induce changes as high as about 30% of the desired preloading. Each preloading stage was also divided in sub-steps in order to apply the loading as homogeneously as possible (Figure 7).

It is worth noting that the transversal section of the vault increases in thickness from about 2 m on the side facing *via dei Fori Imperiali* to about 3 m moving towards *via Sacra*. The preloaded chains reduce the tensile stresses by about 35%, these latter tending to zero where the section attains its maximum thickness. The presence of the chains also improves the behaviour of the structure as a whole thanks to the higher degree of connection between the barrel vaults and the transverse walls, connected to each other at the head.

Finally, steel temporary supports were also installed to mitigate the effects induced by the vibrations produced by the excavation activities. These consist of tube-joint structures and tower buttresses. Specifically, the support of the North-West corner of the Basilica was realised using a steel tube-joint structure, while the support of the apse was obtained via a mixed system consisting in buttresses made of steel tube-joint structures and multiprop towers. Local protections of the large openings in the bearing wall facing *via dei Fori Imperiali*, about 6 m wide and 9 m high, consist instead in metal and wooden structures (Figure 8).

4.4 *Computed and observed displacement fields*

Figure 9 shows the monitoring sections (MOR01-MOR06) set up in the area of the *Basilica di Massenzio*. These were instrumented with settlement markers installed at ground surface, inclinometer and Trivec casings, the first providing horizontal displacements only, while the second measuring the three orthogonal components Δx, Δy and Δz of the displacement vectors along the vertical measuring line, with a depth spacing of 0.5 m. In the following, reference is made to ground settlements measured by precision levelling only. The displacement markers incorporate sockets into which a removable survey plug can be screwed with good positional repeatability for manual surveying. Precise levelling was performed using a digital level which can detect the height of the plane of collimation on a suitable bar-coded staff to a resolution of 0.01 mm. Monitoring of the settlement of the bearing walls of the Basilica during tunnels excavation was performed via precision levelling on displacement markers installed at about 0.5 m height from ground surface.

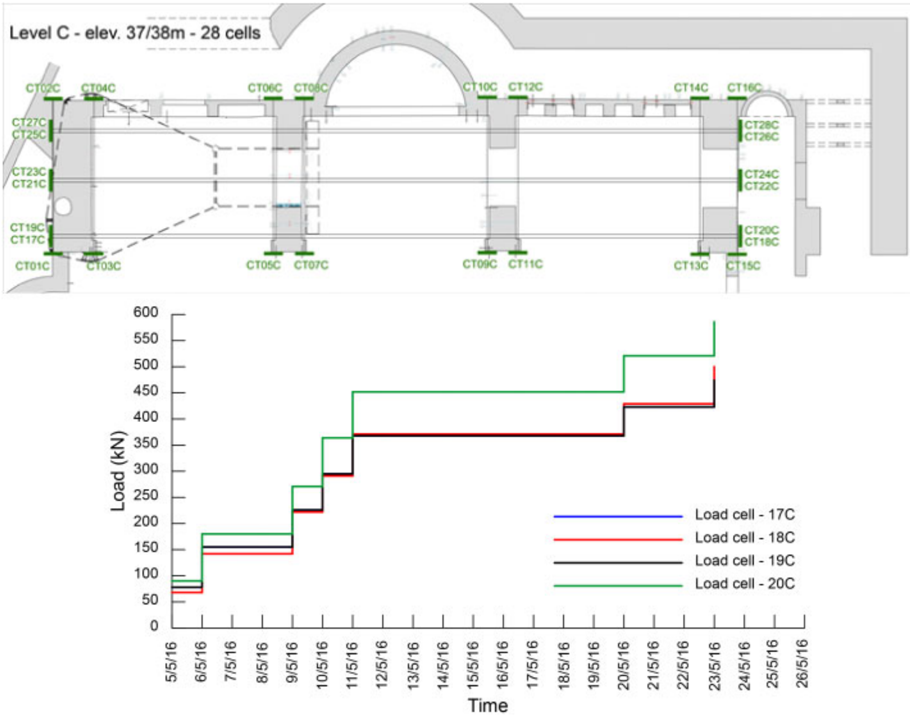

Figure 7. Plan view of the installed load cells and loading steps in the upper and inner chain (level C).

Figure 8. Temporary structural interventions installed on the Basilica.

The ground settlements measured in sections MOR01-MOR05, when the tunnel face was at a sufficient distance from the sections to assume plane strain conditions, were very small, never exceeding 3 mm. For all the sections, it was not possible to fit the observed settlement profiles with a Gaussian curve and the volume loss calculated integrating the measurements was never higher than about 0.1–0.15%. Figure 10 shows the settlements measured at section MOR04 after the excavation of both the tunnels, together with the settlement trough computed for values of volume loss $V_L = 0.15\%$ and $V_L = 0.5\%$, where the latter is the maximum threshold value allowed by design prescriptions.

The settlements of *Pilone Colosseo* and *Pilone centrale Colosseo* (alignments 1 and 2 in Figure 11a) are plotted versus time in Figure 11 b–c: they are in the range of ±2 mm, appearing hardly

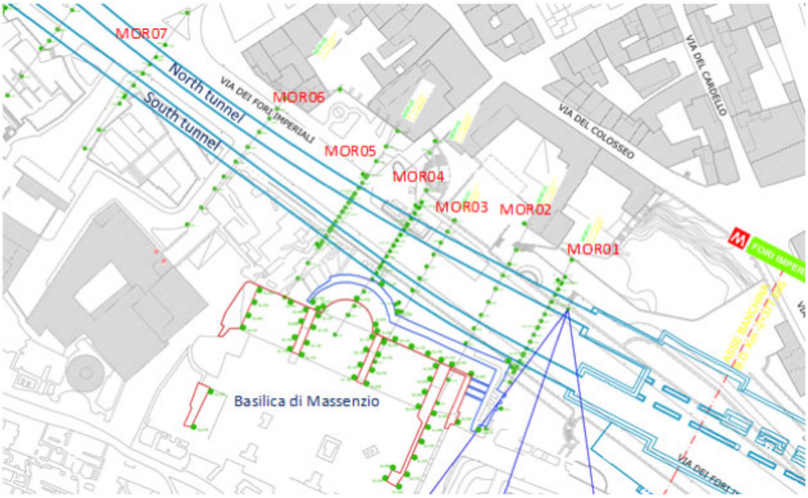

Figure 9. Plan view of the instrumented sections.

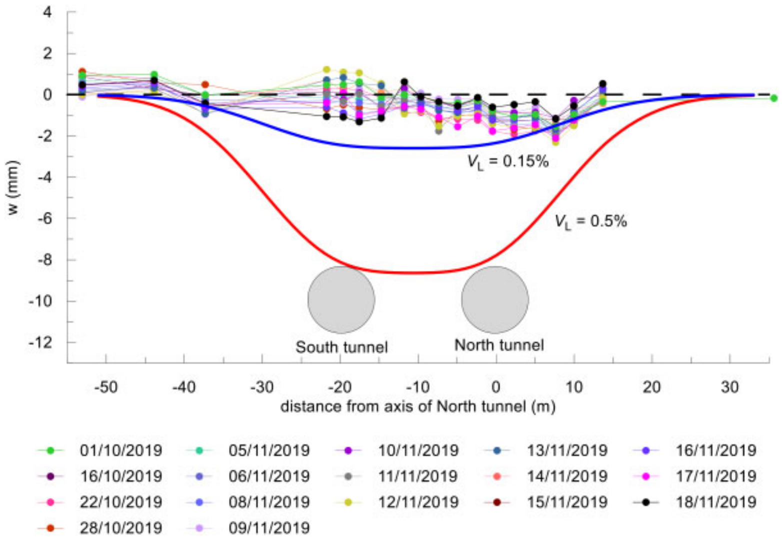

Figure 10. Measured settlements at section MOR04.

affected by tunneling, though the change from heave to subsidence is seen to occur when the face of the south tunnel, the closest to the Basilica, arrives at the alignment locations (red line in Figure 11b, c).

Figures 11 d–e show the settlement profiles measured for the two bearing walls mentioned above, together with the upper bound settlements computed assuming the threshold volume loss $V_L = 0.5\%$: isochrones of settlement do not shows any appreciable deflection confirming the substantially nihil effects of tunnels excavation on the Basilica.

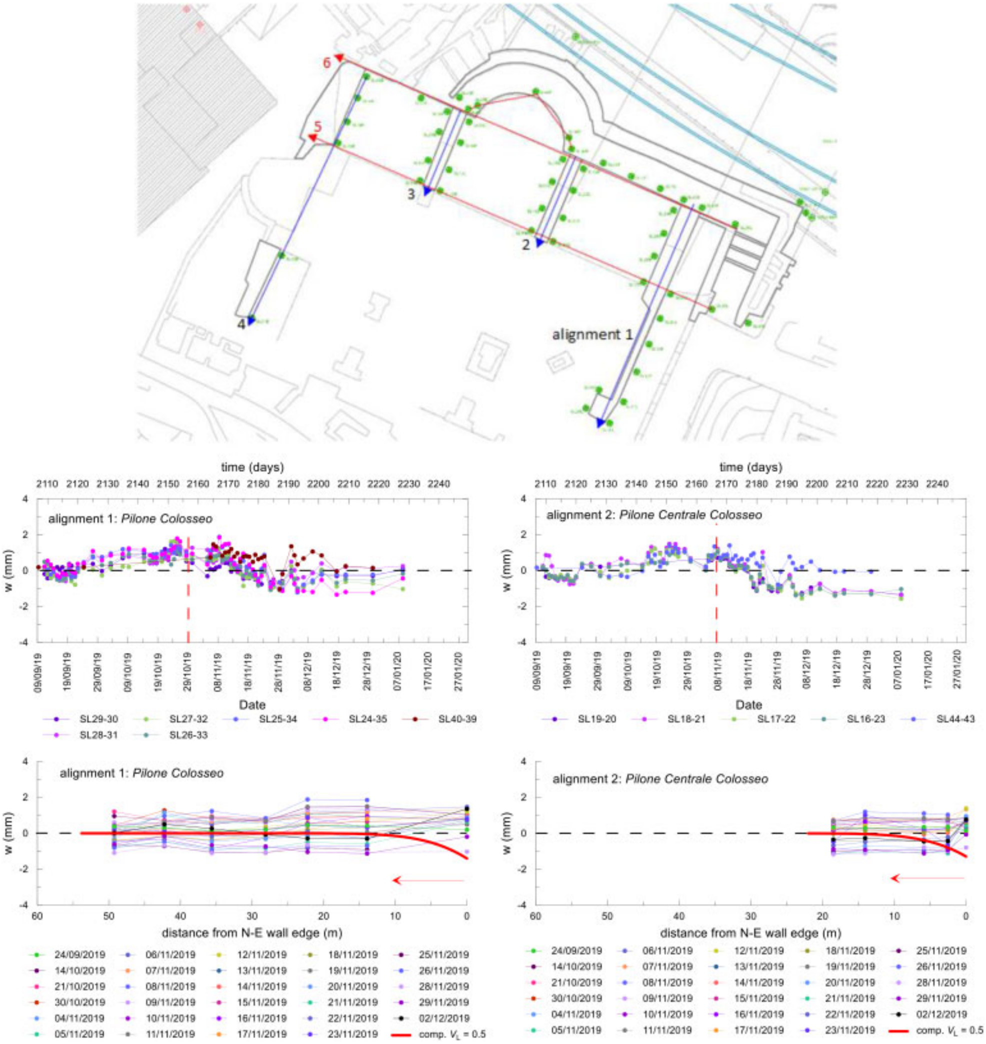

Figure 11. Monitored time histories and isochrones of monitored settlements.

5 CONCLUSIONS

The *Basilica di Massenzio* is an interesting example to illustrate the approach that was adopted in the line-monument interaction studies performed for the new line C of Rome underground. Unlike other monuments of Contract T3, the Basilica is a very heavy structure transmitting significant contact stresses to the foundation soil through its bearing walls. This resulted in an unexpected behaviour in that the settlements computed by the numerical interaction analyses were larger than the ones computed by the empirical relationships, contradicting the common belief that *green-field* analyses are more conservative than soil-structure interaction analysis.

Different positions of the tunnels layout were considered in the design of Line C to arrive at a solution that would minimize tunnelling effects on the Basilica, these concerning the depth of the tunnel, as well as their distance from the Basilica. For the solution finally adopted in the construction of Line C, both the *green-field* and the FE interaction analyses confirmed very negligible tunnelling-induced effects, so that no protective intervention was strictly necessary. Nevertheless, the high vulnerability of the monument suggested the opportunity of implementing some safeguarding interventions to reduce or contain the openings of pre-existing cracks induced by the Basilica self-weight, thus gaining some increase in the safety of the structure.

Tunneling was performed well within the design prescriptions, but it should be mentioned that a loss of control of the face support pressure of the South tunnel resulted in a high volume loss $V_L = 0.8\%$ at section MOR7, a few hundred meters after the Basilica, this suggesting that caution is never too much when a monument of inestimable historical value is involved.

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