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A study of the interaction between the new C line of Roma underground and the Aurelian Wall

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ABSTRACT: A study of the effects of the construction of the C Line of Roma Underground on the Aurelian Wall has been carried out as a research project sponsored by the municipality of Roma through STA S.p.A. This paper summarises the results of the study for a section where the two closed-shield (10 m in diameter) running tunnels pass directly beneath the Wall, with a skew of about 45°. At first, ground movements induced from construction of the tunnels, for different positions of the excavation face, were estimated in *greenfield* conditions using empirical methods. In a second stage, three dimensional finite difference analyses were carried out to simulate the interaction between the Wall and the construction of the tunnels. A distinctive feature of the research was the extended investigation aimed at an adequate definition of a model of the subsoil and of the geometry and quality of the foundations of the Wall.

1 THE C LINE OF ROMA UNDERGROUND

In October 2002 the Municipality of Roma approved the preliminary design of the third line of Roma Underground (C Line). In its central part the route crosses the historical centre of the city posing significant problems connected to the high risk of interferences with buried archaeological remnants, the geotechnical characteristics of the soil, mainly medium grained and below the water table, and the necessity of minimising the effects on the architectural and monumental heritage existing at surface.

In order to avoid direct intersections with the archaeological layer, which in some areas can reach a thickness of more than 12 m, the running tunnels were placed almost everywhere at depths between 20 and 35 m bgl; moreover, to limit subsidence, both the running tunnels and the station tunnels will be excavated using the same closed shield TBM with a diameter of about 10 m (Carriero *et al*. 2002).

The geological sequence along the route comprises a base deposit of stiff overconsolidated clay of Pliocene age (*Monte Vaticano* unit, *APl*) overlain by a layer of fluvio-palustrine gravel of Pleistocene age (*SG*) followed by either clayey silts and sandy silts (*Paleotevere* units *Ar*, *St*), or sandy and silty sinvolcanic soils (*Valle Giulia* unit, *Tb*), both of Pleistocene age; very occasionally (e.g. in the area of Colosseo) pyroclastic deposits are found (*Tl* and *Ta*). Frequent and relatively deep ancient ditches filled by alluvial silty-clayey and sandy deposits (*Alluvioni dei fossi* unit, *LSO*) cut into the Pleistocene deposits. A layer of made ground (*R*) of varying thickness including relicts of ancient structures covers everywhere the natural soil profile. The tunnels are generally contained within the Pleistocene deposits, running into the layer of made ground only between Largo Amba Aradam and S. Giovanni, where they become progressively shallower to permit exchange with the existing A Line.

Despite the measures adopted in design to minimise excavation-induced subsidence, the Municipality of Roma sponsored a research project on the topic of the interaction between construction activities and existing monuments in a limited number of sections preliminarily identified as particularly vulnerable. This paper summarises the results of the study for the section at Largo Amba Aradam, where the running tunnels pass directly beneath the Wall; construction works include also a connecting tunnel, several passageways, and a deep excavation between diaphragm walls with multiple levels of support to accommodate Amba Aradam station, as detailed in the following.

2 THE AURELIAN WALL

According to the historical sources, the first defensive wall of the city of Roma dates back to the age of the Tarquini (VI century b.C.) and was erected by

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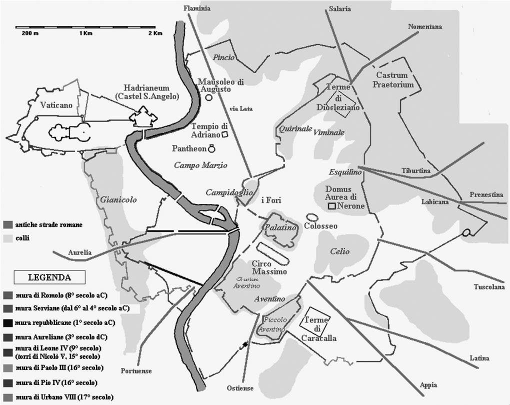

Figure 1. The Walls of Roma (VIII cent. b.C XVII cent. a.D.)

king Servio Tullio (578–535 b.C.). The structure consisted of a large ditch in the ground further protected by amassing the excavated soil on the inside of the perimeter formed by the ditch (the so-called "aggere" from the Latin "*ad gerere*" or "to push against"). The oldest true defensive wall for the city, which is commonly and improperly called Servian Wall, was in fact erected in the first half of the IV century b.C. i.e. Republican Age, and enclosed an area of about 426 ha for a total length of about 11 km (Fig. 1).

The construction of the Aurelian Wall was started in 271 a.D. by emperor Aureliano (270–275 a.D.) and proceeded rapidly under the pressure of the barbaric invasions from the north, as the expansion of the city had completely transcended the limits of the old Republican Wall; the Aurelian Wall was completed in only 6 years under Probo (276–282 a.C.).

The brick wall was about 19 km long, 6 m tall and 3.5 m thick, rested on a 3–4 m wide foundation and ended at the top with an open walkway. In 401–402 emperor Onorio doubled the height of the wall; the original open walkway was covered and provided with openings towards the city.

Over its life the Aurelian Wall has undergone numerous transformations aimed at strengthening the original structure, adapting it to changing needs or rebuilding collapsed sections, but also due to abandon and improper use, particularly in the late nineteenth and early twentieth century. As these transformations are not always documented, a significant joint effort of archaeologists and engineers was required to define reliable models of the monument and of the foundation soil.

3 LARGO AMBA ARADAM

Figure 2 is a plan of the Aurelian Wall at Largo Amba Aradam, showing the position of the running tunnels,

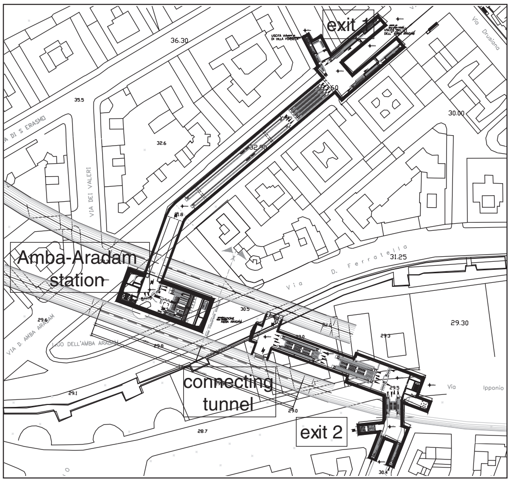

Figure 2. Plan of Largo Amba Aradam.

Figure 3. View of the Aurelian Wall at Largo Amba Aradam.

of Amba Aradam Station and of service tunnels and passageways for pedestrian access.

The axes of the running tunnels are about 21 m below the level of the ground on the outside of the Mura and intersect them at a skew of about 45°. Directly beneath the Wall a tunnel of the diameter of 8 m connects the two running tunnels at platform level. The open excavation to accommodate Amba Aradam Station is 46.4 m long and 17.7 m wide and has a maximum depth of about 34 m.

The state of preservation of the monument in this section is rather poor as the inner wall delimiting the covered passageway collapsed and only part of the internal wall is standing at present for a height of about 2 m from the level of the ground on the inside of the Wall (Fig. 3).

The subsoil of the area was explored by several geotechnical site investigations in 1995–97, 2000, 2002, and 2003, these latter mainly to define the position and consistency of the foundations of the Wall. The

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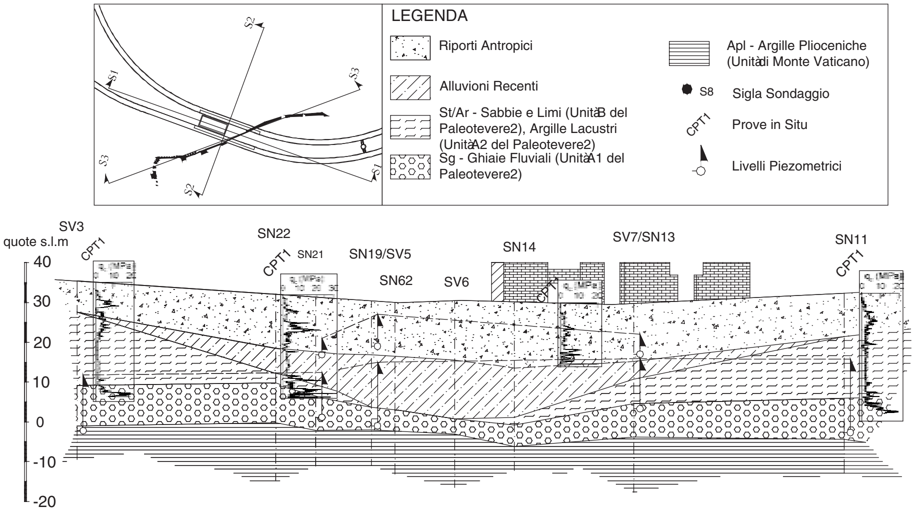

Figure 4. Geotechnical profile parallel to the tunnel axes.

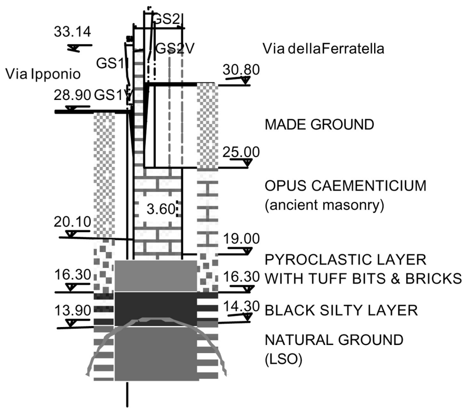

Figure 5. Schematic section through the Aurelian Wall at Largo Amba Aradam.

subsoil profile is consistent with the geological setting described above; the main local feature is a ditch running from north-east to south-west with a maximum depth of about 16 m, cutting into the Pleistocene clayey and sandy silts down to the layer of gravels, filled by alluvial silty-clayey and sandy deposits (Fig. 4).

The relative position of the foundations of the Wall and the tunnel was largely unknown before this study; the site investigation permitted to establish the significant thickness of the layer of made ground, which was found to be about 15 m.

This is of variable nature, mostly man made, generally coarse grained, and often including bricks and tuff blocks. In fact, it turned out that a significant proportion of the layer of made ground was created at the turn of last century by accumulation of material excavated to develop the area of Esquilino. The clearance between the crown of the 10 m o.d. tunnels and the base of the foundation is less than 6 m, between the crown and the pyroclastic layer with tuff bits and bricks, probably a Roman reclamation of the bottom of the ditch along which the Wall was built, only about 3 m (Fig. 5).

4 EMPIRICAL PREDICTIONS

The study of the effects of the construction of the running tunnels and of the station box on the Wall has been conducted in stages of increasing complexity and approximation following the methodological approach outlined by Burghignoli *et al.* (2005) in a companion paper to this Symposium. In the first stage of the study, the displacement fields induced by construction activities in greenfield conditions have been computed using well established empirical relationships. For the running tunnels empirical predictions were based on Gaussian settlement profiles (O'Reilly & New 1982); in this case, the input parameters are the expected value of the surfaces settlement trough, V_S, and the horizontal distance, i, of the point of inflexion of the surface settlement profile from the axis of the tunnel.

The surface volume loss was taken to be 0.5% or 1.0% of the nominal volume of the tunnel; the smaller value is the design value and the most likely given the tunnelling technique and the mechanical characteristics of the soil, the larger value was adopted as a worst scenario value. The width of the settlement profile at different depths was defined using the relative amplitude, K ($i = K(z_0 - z)$, where z_0 is the depth of the tunnel axis). This was taken to increase with depth

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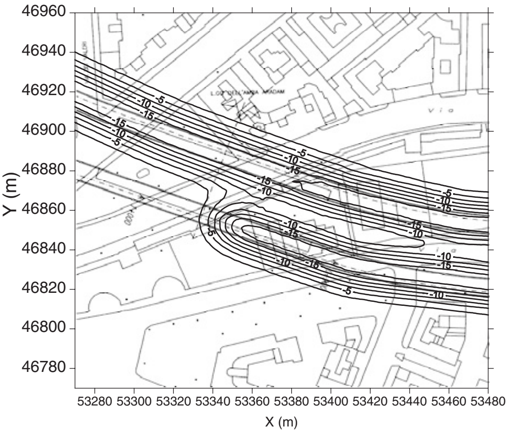

Figure 6. Surface settlement contours (mm, $V_S = 1.0\%$).

below ground level as from the available experimental evidence (Mair *et al*. 1993, Moh *et al*. 1996, Dyer *et al*. 1996) and the results of three-dimensional finite element analyses of case histories with similar geometry and foundation soils (Venturi & Viggiani 2003).

As an example, Figure 6 shows the greenfield surface settlement contours computed using the empirical relationships, in a situation where the first tunnel is completed and the face of the second tunnel has reached the axis of the Wall.

The results made it possible to recognise that, at ground surface: (i) the settlement troughs due to the running tunnels overlap as their widths are larger than the horizontal distance between the axes; (ii) the maximum settlement is about 19 mm (9 mm for $V_S = 0.5\%$) and corresponds to a situation when both tunnels are completed; (iii) the horizontal displacement parallel to the axis of the tunnel is transient as it reaches a maximum when the tunnel face is directly beneath the Wall, once the tunnel is completed it takes relatively small values of 1–2 mm (0.5–1 mm); (iv) the horizontal displacement transverse to the tunnel axis (0.5–1 mm) increases as tunnel excavation proceeds and its final value is equal to ± 5 mm (± 2.5 mm). At foundation depth, the subsidence troughs are narrower than at surface so that they overlap for a lesser extension, however the maximum settlement is larger than at surface and equal to 29 mm (14.5 mm).

5 NUMERICAL ANALYSES

Numerical finite difference analyses of the interaction between the running tunnels and the Wall were carried out with a commercial code, FLAC^{3D} (Itasca 2000). The basic mesh, which was adopted also to carry out preliminary analyses in greenfield conditions, includes only one running tunnel and is 140 m wide and 100 m long. Out of plane horizontal displacements are restricted on the four sides of the mesh and all displacements of the base are restricted.

In the model the pyroclastic layer with tuff bits and bricks has been take to belong to the Wall so that its foundation is at an elevation of 16 m asl, or only 3 m above the crown. The contacts between the layers are horizontal and the soil profile corresponds to the average soil profile in the area. The groundwater regime has been conservatively taken to be hydrostatic with a ground water level at 22 m asl, or about 7 m below ground level on the outside of the Wall.

All soils were modelled as elastic perfectly plastic with Mohr Coulomb yield criterion. Table I summarises the main physical and mechanical properties that were adopted in the analyses. These were obtained as average values of the mechanical properties of the different units from *in situ* and laboratory tests. The stiffness of the soil was taken to be about 40% of the maximum shear stiffness obtained either in laboratory resonant column tests or in cross-hole site surveys. The mechanical properties of Units *LSO* and *St/Ar* are very similar and therefore have not been distinguished in the analyses.

Table 1. Physical and mechanical properties adopted in the analyses.

	γ (kN/m^3)	E′ (MPa)	ν' (−)	c' (KPa)	φ' (°)
R	18.0	52.5	0.25	0	35
LSO St/Ar	19.5	$52.5 \div 175^1$	0.25	20	30
SG	20.0	252.0	0.20	0	40
Apl	20.5	$182 \div 273^1$	0.30	50	26
Wall	22.0	300–2000	0.15	240–800	13

[1] increasing linearly with depth.

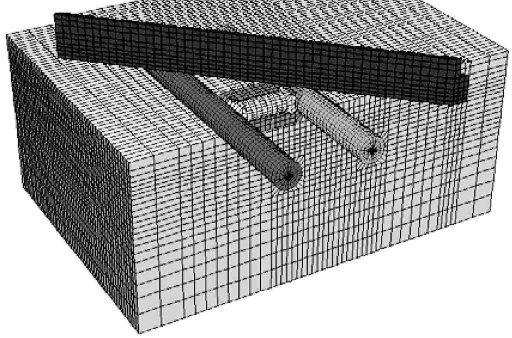

Figure 7. Three-dimensional mesh adopted for the analyses.

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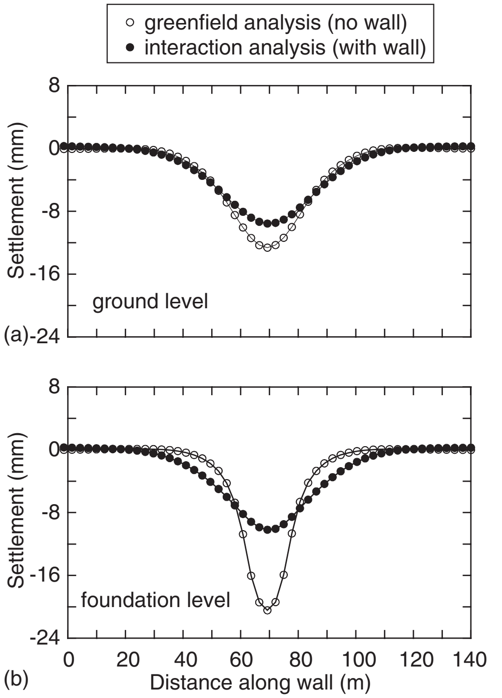

Figure 8. Vertical displacements along the wall: (a) ground level, (b) foundation level.

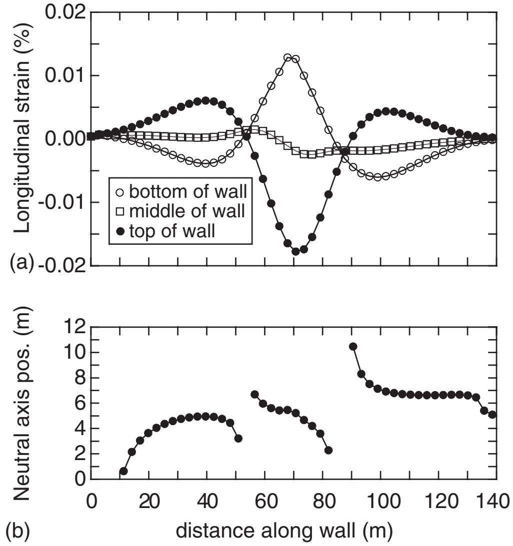

Figure 9. (a) Computed maximum tensile strain in the mid plane of the Wall and (b) neutral axis position.

The Wall was also taken to behave as an elastic perfectly plastic solid with a Mohr Coulomb strength envelope and a tension cut off. The range of values adopted for the stiffness and strength of the Wall (see Table I) was established based on available literature data on roman concrete and brickwork (Lamprecht 1986, CISTeC 2001); in fact, pressuremeter tests carried out within the body of the Wall in this section yielded values of stiffness and strength that were closer to the lower limit of the adopted range, confirming that the consistency of the structure in this section is relatively poor.

In the analyses, shield tunnelling was modelled by imposing a field of displacements to the nodes immediately behind the shield, before installing the permanent lining, so that a prescribed value of the volume loss is attained, as described in further detail by Viggiani & Soccodato (2004). Because the soils are prevalently coarse grained and relatively permeable the analyses were carried out in drained conditions by keeping the initial hydrostatic values of pore water pressures.

The complete interaction analysis models the excavation of the two tunnels in succession, ground treatment in the area of the connecting tunnel and excavation of the connecting tunnel (Fig. 8). Treated ground has been given increased stiffness (2.5 times the original value) and cohesion of 200 kPa; the friction angle of the soil is not affected by ground treatment. The excavation of the connecting tunnel was modelled by removing the corresponding soil elements all at once, after having installed a temporary support in sprayed concrete and centrings.

Figure 8 shows computed vertical displacements along the wall at ground surface and at foundation level in a situation where the first tunnel has been completed, both for greenfield conditions and with a stiff wall ($E = 2000$ kPa). At foundation level the presence of the wall reduces the maximum vertical displacement from about 20 mm to about 10 mm.

Figure 9(a) shows the computed longitudinal strain, ε_{LL}, within the mid plane of the wall, corresponding to bending of the wall, with the bottom fibre extended in its central part, directly above the tunnel crown, and compressed at the sides, and the top fibre compressed in the middle and extended at the sides. The computed position of the neutral axis, defined by $\varepsilon_{LL} = 0$, turned out to be practically constant along the wall at about mid height, see Figure 9(b).

Figure 10 shows the computed maximum tensile strain contours, and the associated levels of likely damage, in the points corresponding to the mid plane of the wall for greenfield conditions (no wall), for a stiff wall ($E = 2000$ kPa) and for a flexible wall ($E = 300$ kPa).

The contours are plotted in each case for the step of advancement of the tunnel corresponding to the maximum tensile strain at point A, directly above the tunnel crown at foundation level. From the data in Figure 10 it is evident that for a relatively stiff wall the most

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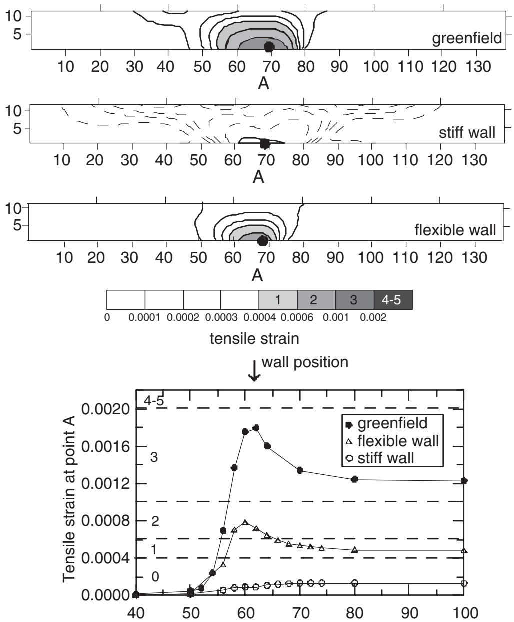

Figure 10. Contours of maximum tensile strain and corresponding levels of damage.

dangerous situation occurs when the tunnel has been completed, i.e. in plane strain conditions. On the contrary, for greenfield conditions and for a flexible wall, probably more representative of the actual behaviour of the Mura Aureliane in this section, the tensile strains are maximum in an intermediate situation, in which the shield is passing under the wall; these conditions cannot be reproduced in a plane strain analyses, no matter how sophisticated the model for the soil and/or the wall.

To explore the effects of a variation of the strength of the Mura, after completion of one tunnel, the strength parameters were progressively reduced by a factor F, such that $c_F = c/F$, $\sigma_{tF} = \sigma_t/F$ and $\tan\varphi_F = \tan\varphi/F$, whilst monitoring the response of the wall in terms of strain and stress increments.

Figure 11 shows the maximum tensile strain obtained factoring the strength parameters divided by the maximum tensile strain obtained for $F = 1$ as a function of F in the four points located directly above the tunnel crown at foundation level (A) and at the crest of the wall (B) and on either side of the tunnel at the crest of the wall (C and D). The same figures also shows the position of the elements in the mid plane of the Wall that reach plastic failure. Despite the fact that for $F = 10$ the maximum strain ratio at point B is only about 3.3, it is evident that the wall has reached plastic failure almost everywhere in its mid-plane. This may be attributed to the fact that in the problem under examination the displacements of the wall are constrained by the surrounding ground and therefore, even for a failing wall, deformations do not increase dramatically.

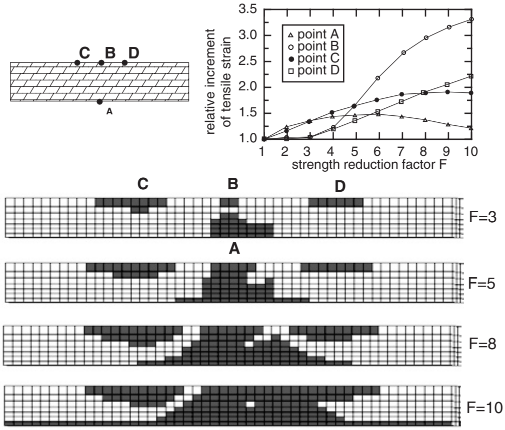

Figure 11. Effects of a strength reduction on the computed maximum tensile strains.

6 CONCLUSIONS

The ability to predict with confidence tunnelling-induced displacements of structures and the associated potential for damage is extremely important for urban tunnelling, particularly in situations where the built environment is of monumental and historical value, as in the example detailed in the paper.

The presence of structures may modify substantially greenfield displacements. Geometrical details and mechanical parameters of the materials of which the structures are composed are very important but often, particularly in the case of old structures, largely unknown. In this case the geometry of the buried part of the monument was defined through a significant joint effort of archaeologists historians and engineers. Ideally, the geometry of the foundations and of all other underground parts of the structure, and the mechanical properties of the materials should always be obtained directly through investigation programmes designed *ad hoc*. The example of Aurelian Wall well illustrates how the stiffness of the wall affected not only the magnitude of the computed displacements but also, and more importantly, the pattern of the displacement field and associated deformation.

Existing criteria for the assessment of the likely level of damage are largely based on the concept of associating a given category of damage to the attainment of a threshold value of tensile strain in the structure. However, in some cases, other indicators of the state of damage in the structure, such as yield stress

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states, ought to be considered together with the tensile strains. This is particularly important for relatively confined problems, such as that of the Aurelian Wall, where failure is reached but the surrounding ground prevents significant deformations of the wall.

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