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Vulnerability assessment of historical masonry buildings to excavation-induced settlements: *palazzo assicurazioni generali*

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Abstract. Historical masonry structures represent a conspicuous part of the European architectural heritage. However, these often are more vulnerable to damage risk than other constructions due to their peculiar mechanical properties. This justifies the lively interest in the development of efficient and reliable approaches for their structural capacity assessment. This work presents the vulnerability assessment of the historical masonry building, "Palazzo Assicurazioni Generali", for the effects induced by the underlying tunnel excavation required for line C of the Rome underground. The structure is a 12-floor building, built in 1902-1906 in Piazza Venezia, one of the most prestigious areas of the city. A detailed macromechanical three-dimensional numerical model of the entire structure is developed and two kinds of static analyses are conducted, assuming linear elastic and nonlinear constitutive response for masonry.

Introduction

Masonry structures compose most of the architectural heritage of many European countries. However, due to the poor tensile strength, these usually exhibit high vulnerability against natural loadings but also anthropic phenomena, such as excavation-induced ground settlement.

The assessment of structural response to differential settlements has always been a felt concern, especially in metropolitan areas. Indeed, several works have been dedicated to the estimation of damage in masonry buildings due to settlement effects. Meaningful examples and extensive reviews are reported in [1,2]. Most adopted techniques usually rely either on simplified approaches, where elastic strains in the material are compared with limit values, e.g. [3,4], or on more complex methods, based on nonlinear Finite Element (FE) analyses [5-7]. For the latter, soil-structure interaction can be neglected [8], for simpler and more conservative analyses, or included [9], for more accurate solutions.

This work is part of the studies conducted to estimate the effects induced on historical buildings by the tunnelling operations required for the construction of the new Line C of the Rome underground in Italy. These involve the realization of two tunnels that connect the station located near the Colosseum to that to be built under Venice Plaza (*Piazza Venezia*). In particular, this work presents the vulnerability assessment of the Assicurazioni Generali building (*Palazzo Assicurazioni Generali*), an important historical masonry building located in Venice Plaza, on the northeast side of the underground station.

A three-dimensional (3D) numerical model of the building is defined and structural analyses are carried out to estimate the damage distribution in the masonry due to the settlement-induced deformations. These are conducted by modeling the structure as decoupled from the soil. However, assumed vertical displacement profiles of the foundation, corresponding to the settlements, are defined according to two different approaches, namely level 1, based on the assumption that interaction between soil and building is negligible (free field), and level 2, based on numerical simulations capable of accounting for this interaction.

Two types of structural analyses are performed. In the first, the response of the building is assumed as linear elastic. Hence, the level of strains induced by the application of the subsidence fields is determined and used to evaluate the potential crack patterns occurring in the masonry. In the second, a smeared crack non-linear constitutive model (SCM) [10,11] is used to simulate the material damage and, thus, the evolution of cracks produced by the differential settlements.

Structure geometry

The Assicurazioni Generali building is a 12-floor masonry structure, occupying a large area of almost 3500 m^2 in plan. It was built in 1902-1906 by the Architect G. Sacconi, who decided to use high-quality solid brick masonry for almost the entire construction.

In the main portion of the building, the structure consists of 9 floors of different heights, 2 of which are underground, for a total height of about 32 m from the ground level (Figure 1: left). The building has a pseudo-quadrilateral layout, with one of the vertices beveled. The main façade overlooks Venice Plaza. On the southern corner of the building, a square tower rises for almost 17 m, reaching a height of approximately 48 m.

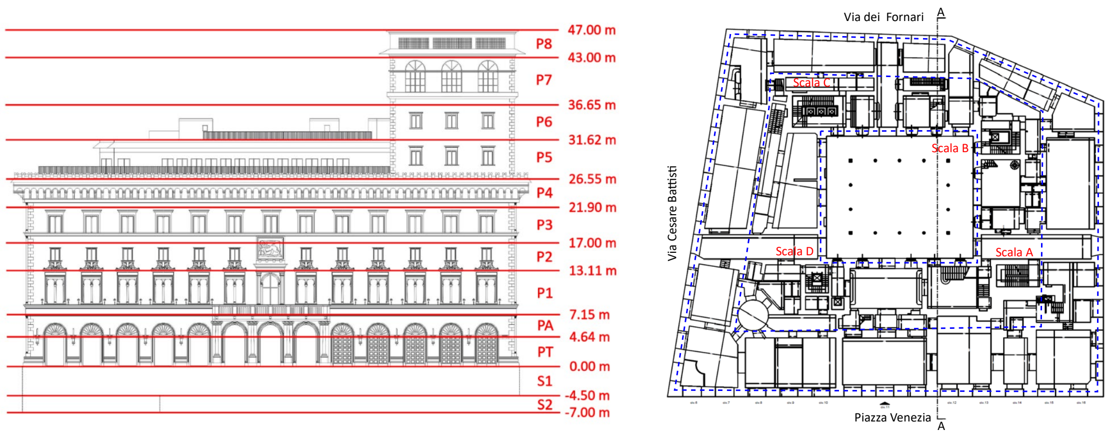

Figure 1: Façade on Piazza Venezia with indication of the building floors (left) and ground floor plan with indication of the main walls as blue dotted lines (right)

The structural plan is organized according to a "ring" system (Figure 1: right), consisting of the perimetral façades and central spine walls, connected by transverse walls and horizontal slabs. All the main walls have a large thickness, reaching at some points the value of 2.5 m. The horizontal slabs and the 4 main stairways, located at the internal courtyard vertices, have various natures, ranging from barrel and cross vaults to wood, steel or RC elements.

On the ground floor, the center of the building hosts a cloister and a travertine colonnade made with 18 cross vaults. These stand above a vast basement, covered by a reinforced concrete (RC) vault that is supported by 14 RC pillars.

The foundation is an RC slab supported by wooden piles that go beyond the first layer (about 11 m) of fill material soil and reach the underlying layer of compact silty clay (about 43.5 m).

Excavation-Induced Settlements

The evaluation of the settlement profile is carried out by the geotechnical engineering research group led by Prof. S. Rampello at the Sapienza University of Rome. The excavation operations are grouped into two main construction phases:

- phase 1: mechanized excavation of the two tunnels and the station shaft;
- phase 2: traditional enlargement necessary for the quay tunnels and the rail junction.

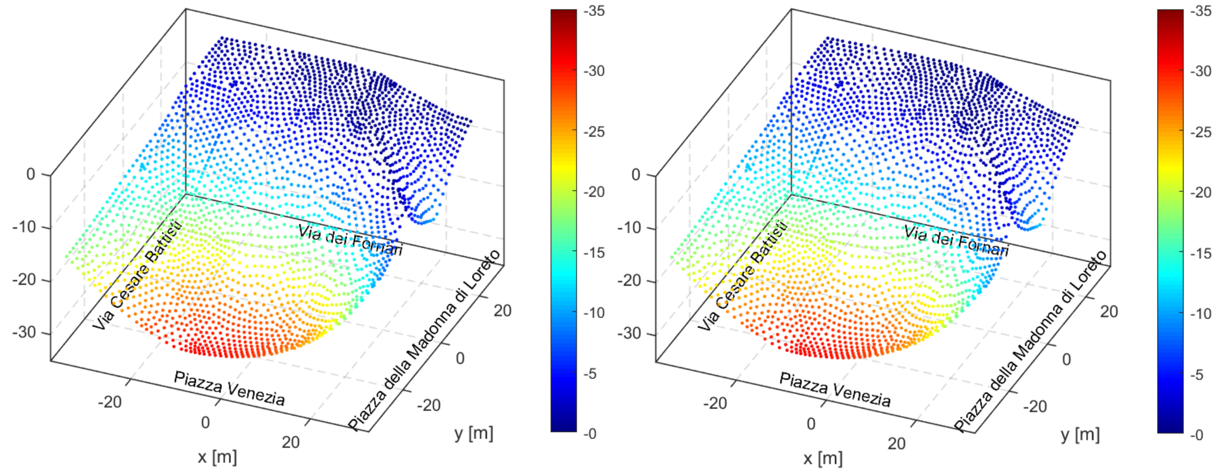

Figure 2: Settlement profile for phases 1 (left) and 2 (right), obtained from level 2 analyses (the color bar and the vertical axis indicate the value of the settlements in mm)

Two distinct procedures are followed, referred to as level 1 and level 2 analyses. Level 1 analyses are based on a semi-empirical approach that neglects the interaction between the soil and the building (free field). By contrast, level 2 analyses use FE numerical simulations where the ground is modeled as a continuous medium with non-linear behavior and the ground-building interaction is included.

For example, Figure 2 shows the settlement profile at the foundation intrados obtained from the level 2 analyses, for phases 1 (left) and 2 (right). The maximum settlement value is almost equal to 31 mm. These mainly develop during phase 1 in the central area of the façade overlooking Venice Plaza and at the corner between this façade and that overlooking Piazza della Madonna di Loreto. Very small increments (less than 2 mm) are observed during phase 2.

Finite Element numerical model

To carry out the structural analysis of the building, a 3D FE model is created (Figure 3) by using the software MIDAS FEA NX [12].

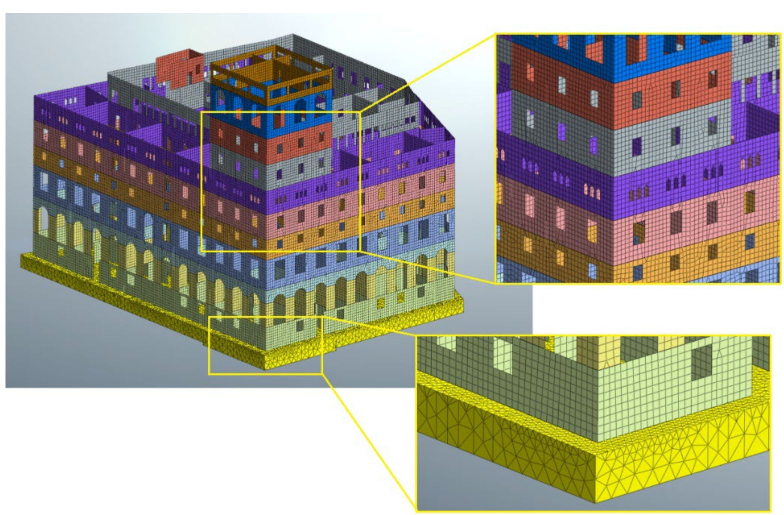

Figure 3: FE model of the building defined in MIDAS FEA NX

Walls are modeled with 3- and 4-node shell elements based on the Mindlin-Reissner plate theory and a mixed formulation (*Assumed Strain Elements* with in-compatible modes [12]) that avoids shear locking issues. The foundation is modeled through tetrahedral 4-node solid FEs, based on a classical displacement formulation. Columns, pillars and beams are modeled through 2-node frame elements, based on a force formulation. Given the uncertainty about the actual mechanical characteristics of the horizontal slabs, these and the stairs are included in the model only as loads and by applying equivalent rigid diaphragm constraints at the node of the FE mesh.

Damage distribution assessment through linear elastic analyses

In the linear elastic analyses, the structure behavior is studied by imposing at the foundation intrados the settlement vertical displacements. Hence, stress and strain distribution attained in the structural elements are analyzed. To quantitatively define the vulnerability of the structure against the settlements, the results are interpreted based on the criterion proposed by Burland et al. [3], in the version developed by Boscardin & Cording [4]. This evaluates the possible damage to the masonry from the principal tensile strains. Given the historical and architectural interest of the building, a more conservative definition of the damage levels than that provided by Boscardin & Cording is assumed as indicated in Table 1.

Table 1: Adopted damage level classification

Damage level	Severity of the damage	Tensile strain	
		min	max
0	Negligible	0.0	4.0 10^{-4}
1	Very low	4.0 10^{-4}	6.0 10^{-4}
2	Low	6.0 10^{-4}	1.0 10^{-3}
3	Medium	1.0 10^{-3}	2.0 10^{-3}
4	Severe	2.0 10^{-3}	∞

Figure 4 shows, for example, the deformed shape (left) and the distribution of the principal tensile strains (right) obtained by imposing the vertical displacements computed for phase 2 through the level 2 analyses. These represent the worst condition for potential cracks in the masonry, although no significant difference is observed with respect to phase 1. Indeed, during phase 1, significant tensile strains arise in the main façades on Venice and Madonna di Loreto Plazas. These reach values up to 3.5×10^{-4} on the upper floors, corresponding to damage level 0, and 6.0×10^{-4} in very limited areas of the lower floors, corresponding to damage level 1.

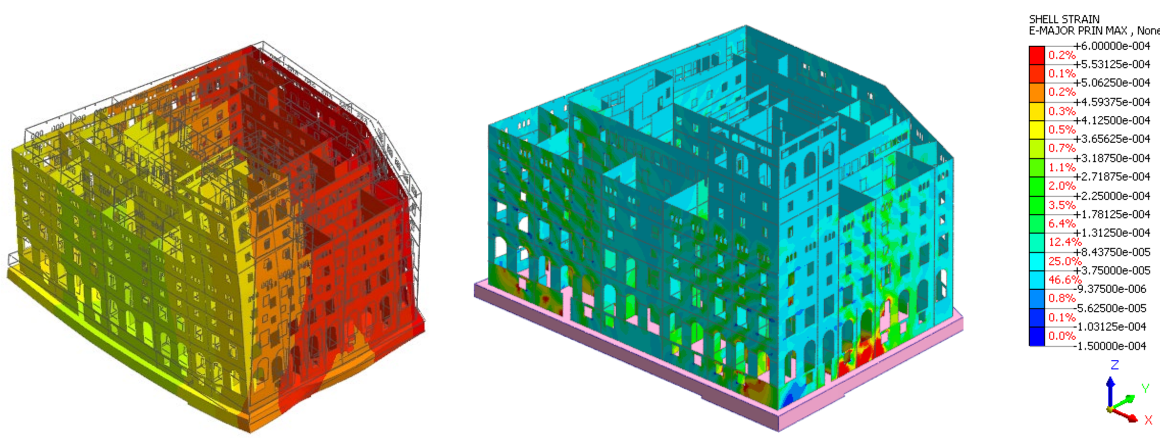

Figure 4: Deformed shape (left) and maximum tensile principal strains (right) obtained for phase 2 settlements (level 2 analyses)

Nonlinear analyses

To obtain a more accurate assessment of the damage patterns potentially occurring in the masonry, nonlinear static analyses are conducted. To limit the computational cost, a reduced model of the building is employed. This includes only the two façades on Venice and Madonna di Loreto Plazas mainly affected by the settlement effects and portions of the transverse walls, as reported in Figure 5. Moreover, the foundation is not explicitly included in the model and the settlement displacements are directly imposed at the base of the walls.

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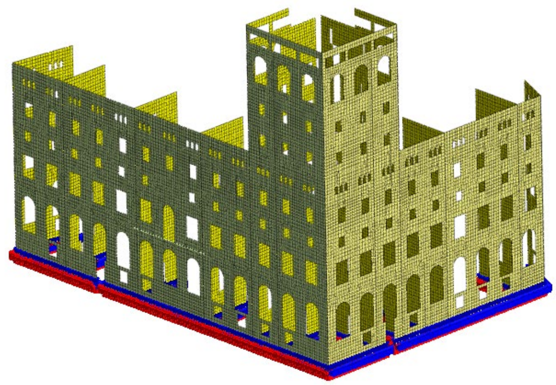

Figure 5: Reduced model of the structure adopted for the nonlinear analyses

To verify the accuracy of the reduced model, preliminary linear elastic analyses under the vertical settlements are conducted and the results are checked with those of the full model.

In the nonlinear model, the SCM formulation [10,11] is assumed for the masonry and the following mechanical parameters are adopted: Young's modulus E = 1714 MPa; Poisson ratio υ = 0.15; compressive strength f_c = -2.7 MPa and fracture energy density $g_c = G_c/h$ = 0.864 MPa; tensile strength $f_t = |f_c|/10 \simeq 0.30$ MPa and fracture energy density $g_f = G_f/h$ = 0.0035 MPa; crack band width h = 500 mm; and shear factor β = 0.01 [5].

Loads are applied to the structure in 3 successive phases, i.e. gravity loads are applied in a single step, assuming that these instantly act at the initial time of the analysis (phase 0); then the settlement vertical displacements are imposed, according to the two phases 1 and 2, by carrying out an incremental step-by-step analysis. Level 2 analyses values are considered.

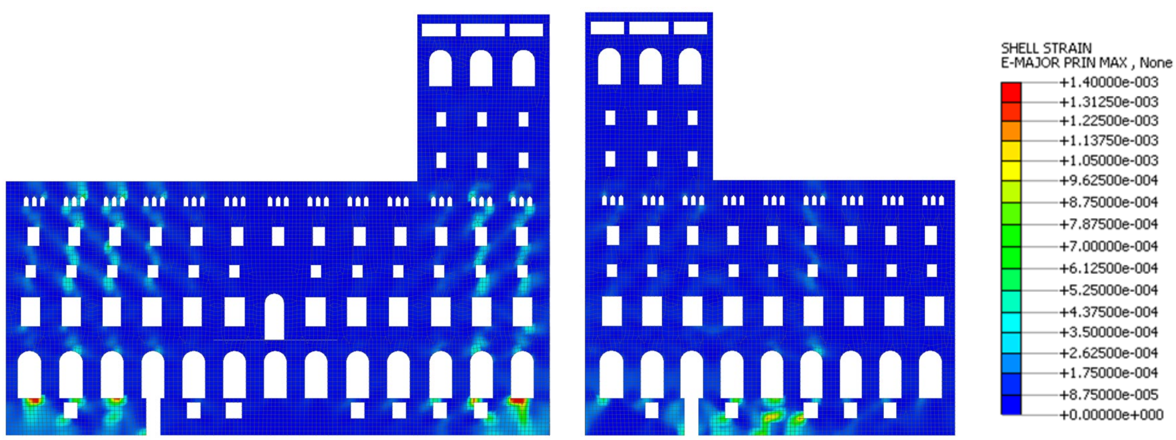

Figure 6: Maximum tensile strains in the Venice Plaza (left) and Madonna di Loreto Plaza (right) façades obtained at the final step of phase 2

Tensile strain distributions for the last step of the analysis are reported in Figure 6. As expected, for gravity loads (phase 0), strains in the structure remain in the elastic limit, although the masts of the ground floor reach values close to this limit. During phase 1, the façade on Venice Plaza shows a significant increase of tensile strains in correspondence with its two vertical edges. Indeed, this mainly bends in its plane (see also Figure 4, left) with maximum vertical displacements in the middle part and minimum displacements at the edges (clamped-clamped-beam-like deformation). By contrast, the façade on Madonna di Loreto Plaza undergoes in-plane bending with maximum displacements near the tower and minimum displacements at the opposite edge (cantilever-beam-like deformation); the maximum tensile strains are observed in the central part of the lower floors. During phase 2, there are no significant increases in the deformations, with maximum tensile strain equal to 1.4×10^{-3}, corresponding to a crack width of about 0.60 mm. These results confirm those obtained in the linear elastic analyses, showing a maximum damage level equal to 1 (very low).

Summary

This work presented the vulnerability assessment of the Assicurazioni Generali masonry building for the settlements induced by the tunneling operation required for the new Line C of the Rome underground. Linear elastic and nonlinear static analyses are conducted by employing a detailed macromechanical model. These showed that the damage risk against the excavation-induced settlement is negligible (damage level 0) in almost the entire structure and reaches very low values (damage level 1) in limited zones of the main façades on Venice and Madonna di Loreto Plazas. Hence, the tunnelling operations do not generate significant stress-strain states in the structures that require specific interventions or attention.

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