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Assessment of the effects induced by mechanised tunnelling on masonry buildings in the urban area of Rome

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ABSTRACT: The growing population and increasing demand for sustainable mobility necessitate the transition of road and rail traffic to underground infrastructure. In urban environments, such projects are often constructed at shallow depths, this emphasising the need to evaluate interactions with existing structures. This issue is particularly critical in Rome, due to its dense concentration of monuments, historical buildings, and churches, of exceptional historical and artistic significance. It is therefore apparent that advanced numerical analyses of the excavation-structure interaction are essential to properly model the problem. However, such analyses are not always applicable to all architectural pre-existences within the displacement field generated by tunnel excavation, making it necessary to resort to empirical methods calibrated on observations in green-field conditions. In this context, over the past three decades, and particularly in London, UK, several studies have examined the modification of the displacement and deformation field caused by the presence of the structure. Different authors, through extensive parametric analyses where the building stiffness, weight, and location, were varied, introduced design charts to improve the deformation predictions. In this paper, the results from these previous studies are compared with those from an extensive parametric study performed with 2D-equivalent Finite Element (FE) analyses, which have been conducted referring to the stretch line T2 of the new Line C of Rome underground. The FE models incorporate geometries derived from detailed analyses of historical Roman buildings, highlight the applicability of existing methods and their potential adjustments for Rome's unique architectural heritage.

KEYWORDS: Mechanised Tunnel, Masonry Building, Architectural Heritage, Rome, Finite Element Analysis.

1 INTRODUCTION

In urban areas, assessing the effects that tunnel excavation may have on existing buildings and infrastructure has become a crucial stage of the design process. Estimating the induced deformations and the resulting risk of structural or functional damage requires particular attention, especially in a city such as Rome, which is densely built and characterised by invaluable historical assets.

Traditional methods, developed primarily for green-field conditions, do not consider the inherent characteristics of buildings, such as their stiffness and self-weight. Damage assessment is often carried out applying the green-field displacement field directly to the foundation (Peck, 1969) and, assuming the structure to be flexible, two parameters are derived, namely the deflection ratio, $DR = \Delta/L$, and the horizontal strain, ε_h. These parameters allow the damage category to be determined using damage charts (Boscardin e and Cording, 1989; Burland, 1995) (Figure 1)

This simplified procedure, named hereafter level 1 analysis (*e.g.* Burghignoli, 2011; Rampello *et al.*, 2012), is considered sufficient if the resulting damage falls within category zero (very slight damage). Otherwise, a more accurate analysis is carried out (level 2), which involves interaction analyses typically performed using numerical methods, explicitly modelling both the tunnel and the building within the same numerical domain.

Over the last three decades, a method based on the concept of relative stiffness between the building and the soil has been developed to overcome the above-mentioned assumptions on which the level 1 analysis is based, while still maintaining the ease of a simplified approach. This method, first introduced by Potts and Addenbrooke (1997) and further developed by Franzius *et al.* (2006) and Mair (2013), is based on an extensive FE and centrifuge dataset, considering a wide range of configurations, with the aim of providing a more realistic damage estimate, closer to that obtained through full numerical analyses, which is summarised in the following section.

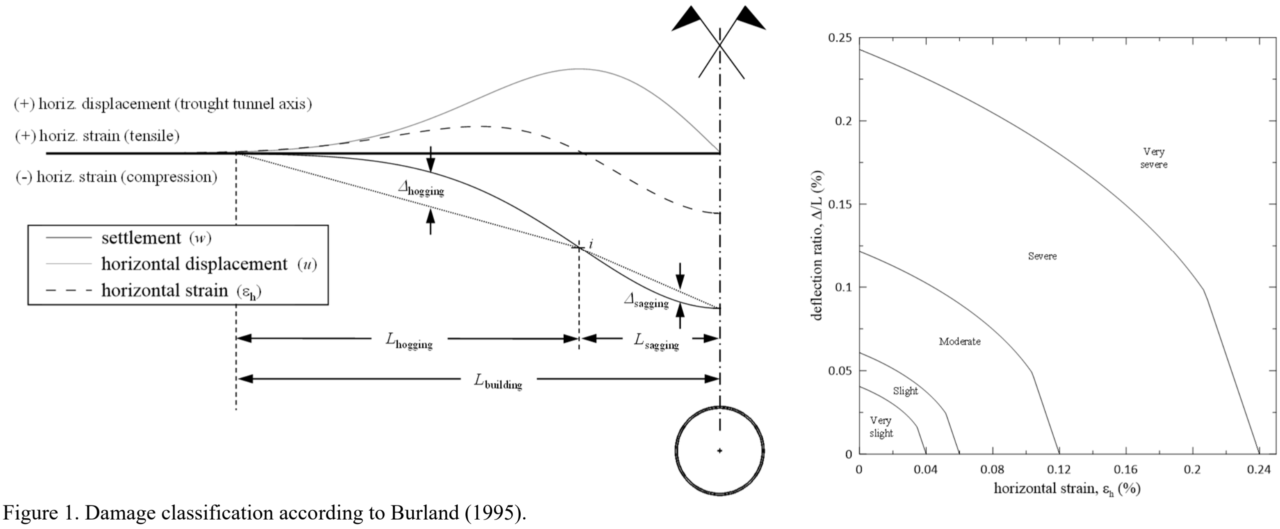

Figure 1. Damage classification according to Burland (1995).

2 MODIFICATION FACTOR

The term modification factors, M^{DR}, refers to a parameter that describes how the tunnel excavation-induced displacement and strain fields vary compared to green-field conditions, when considering the presence of a structure at the ground surface:

$$M^{DR} = \frac{DR^{\text{str}}}{DR^{\text{gf}}} \tag{1}$$

where DR^{gf} and DR^{str} are the deflection ratios computed in green-field conditions and in the presence of the building, respectively. Previous studies, based on several numerical and centrifuge parametric analyses, correlated these modification factors to the relative soil-structure stiffness, proposing various formulations. In the present study, the following definition of the dimensionless relative bending stiffness has been adopted (Mair, 2013):

$$\rho_{sag} = \frac{EI}{E_s L_{\text{sag}}^3} ; \rho_{\text{hog}} = \frac{EI}{E_s L_{\text{hog}}^3} \tag{2}$$

where EI is the bending stiffness of the building, where E represents the Young's modulus and I the relevant moment of inertia, while E_s is the representative Young's modulus of the soil column above the tunnel, which can be obtained from *in-situ* test, and L is the length either in sagging or hogging deformation mode. The design chart proposed by Mair is shown in Figure 2.

3 FE PARAMETRIC STUDY

3.1 *Reference layout*

Following the aforementioned approach, a parametric study was conducted by performing 3D FE analyses, where the layout shown in Figure 3 was reproduced. The study was carried out by fixing several parameters, such as the tunnel diameter D = 6.7 m, consistent with the design of Line C, and the tunnel depth z_0 = 25.75 m. The volume loss, V_L, defined as the ratio between the volume of the subsidence and the nominal excavation area, was assumed equal to 1.0%, as a conservative assumption. Two different cover-to-diameter ratios, C/D, were investigated, equal to 2.45 and 2.90, which were obtained by varying the foundation depth of the building, z_{pp} = 6 m and 3 m, respectively (note: $z_0 = z_{pp} + C + D/2$). In addition, four different eccentricity ratios (e/B) were considered: 0, 0.2, 0.4, and 0.6. Finally, the effect of the building's rotation with respect to the tunnel axis was studied by considering angles α = 0, 45° and 60°. In the following, the effect of the eccentricity ratio is only discussed for the sake of brevity, by assuming the extreme cases e/B = 0 and 0.6, while keeping the remaining parameters as constant (C/D = 2.45, α = 0).

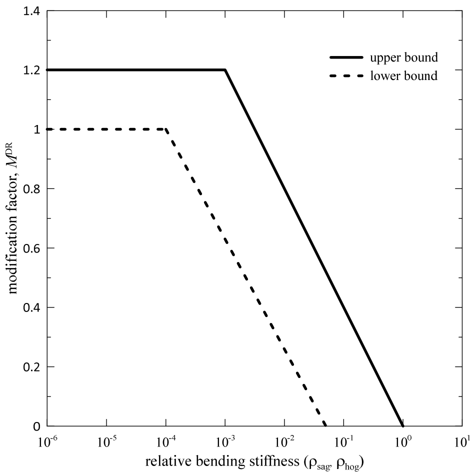

Figure 2. Design chart to compute the modification factor (Mair, 2013).

3.2 *Representative building in Corso Vittorio Emanuele II*

The geometry of the building used in the numerical analyses was defined based on a preliminary study about the historical buildings located in Corso Vittorio Emanuele II, beneath which the construction of Line C is planned. The analysis of the historical buildings made it possible to determine the above-ground height of the structure, H, the aspect ratio of the façade, H/B, and the opening percentage, O, the latter defined as the ratio between the total area of openings and the overall façade area, affecting the stiffness of the building significantly (Melis and Rodriguez Ortiz, 2001).

The processing of the collected data, presented in histogram form in Figure 4, allowed for the identification of the following average quantities representing the building: H = 21.3 m, H/B = 0.64, and O = 16.3%. A similar analysis was carried out about the plan size of the buildings, which enabled for the definition of a typical internal space.

The parameters of the façade used in the FE analysis are listed in Table 1.

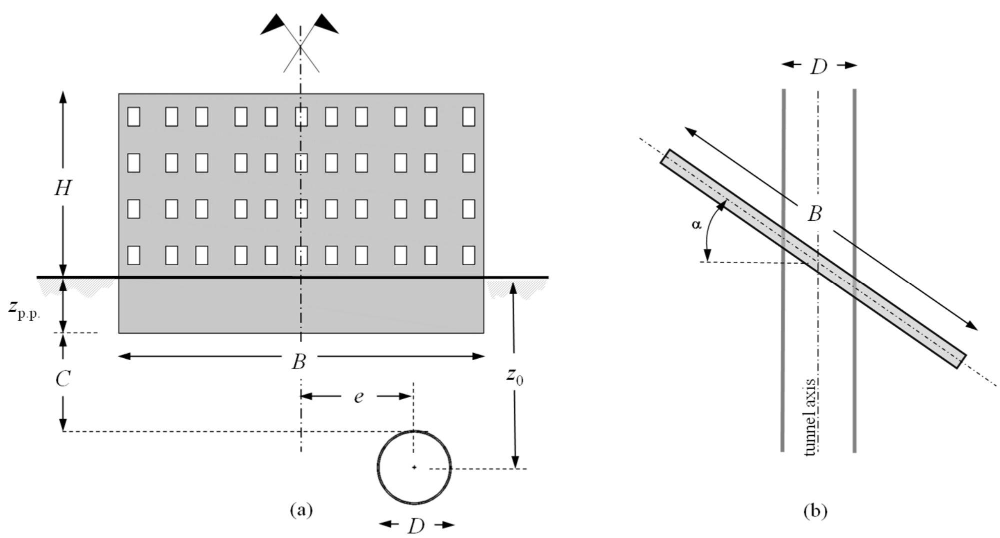

Figure 3. Reference layout adopted in the parametric analysis: (a) front and (b) top view.

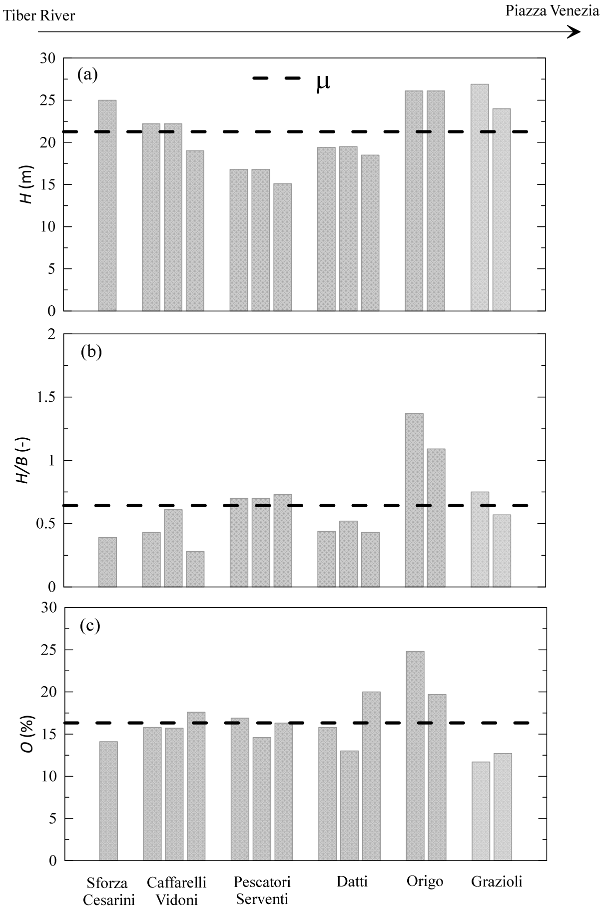

Figure 4. Distribution of geometric parameters: (a) building height above ground level; (b) aspect ratio; (c) opening percentage.

Due to their proximity to the average values determined previously, these parameters make the façade adopted in the numerical analyses representative of the typical historical building in the study area.

Table 1. Parameters adopted in the analyses for the building façade.

H	*H/B*	*O*
(m)	(-)	(%)
20	0.50	17.9

3.3 *Soil mechanical properties*

The soil profile assumed in the analyses resembles the typical geotechnical conditions of Piazza Venezia, which is located in the historic centre of Rome. The soil deposit therefore consists of a shallow 9-m-thick layer of anthropogenic fill (*R*), underlain by a 46-m-thick deposit of silty clays and clayey silts (*Ag*), whose mechanical properties are listed in Table 2.

The groundwater table is located at the interface between the two layers, and the pore water pressure regime is hydrostatic.

Table 2. Soil mechanical properties.

layer	γ	c'	φ'	$k_{0,nc}$	*OCR*
	(kN/m^3)	(kPa)	(°)	(-)	(-)
R	18	15.0	29.0	0.515	-
Ag	18	15.4	26.2	0.558	1.2

* γ = unit weight; c' and φ' = effective cohesion and angle of shearing resistance; $k_{0,nc}$ = earth pressure coefficient at rest; *OCR* = overconsolidation ratio.

3.4 *Constitutive modelling*

The constitutive model adopted for the anthropogenic fill (*R*) is the *Hardening Soil Model* (*HS*) (Schanz *et al*., 1999), developed within the framework of isotropic hardening plasticity, while its advanced formulation, the *Hardening Soil with Small-Strain Stiffness* (*HS-small*) (Benz, 2006), was used to simulate the mechanical behaviour of the *Ag* layer. The constitutive model parameters adopted for layer *R* are: E_{ur}^{ref} = 262.80 MPa, ν'_{ur} = 0.2, m = 0.643, and E_{50}^{ref} = E_{oed}^{ref} = 26.28 MPa, while the following were adopted for the layer *Ag*, namely G_0^{ref} = 61.88 MPa, m = 0.655, $\gamma_{0.7} = 4.8\cdot10^{-4}$, E_{ur}^{ref} = 59.41 MPa, ν'_{ur} = 0.2, E_{50}^{ref} = 9.37 MPa, and E_{oed}^{ref} = 7.96 MPa. These parameters result from the calibration performed on cross-hole tests, resonant column tests, and triaxial Tx-CIU tests carried out over the area of Piazza Venezia.

For the masonry, two different constitutive models were considered in the study, such as a linear elastic, homogeneous and isotropic model, as well as an elasto-plastic model based on the Hoek-Brown failure criterion (Hoek *et al*., 2002) (Table 3). For both models, a Young's modulus E = 1500 MPa and a Poisson's ratio ν = 0.2 were adopted. The unit weight γ was set to 23.1 kN/m^3 for the above-ground masonry, while γ = 18.5 kN/m^3 was assumed for the foundation.

Table 3. Hoek-Brown model parameters adopted for the numerical modelling of the masonry.

m_i	*GSI*	σ_{ci}	m_b	s_b	a	σ_t	σ_c
(.)	(-)	(MPa)	(-)	(-)	(-)	(kPa)	(kPa)
13.7	52	32	2.47	0.0048	0.5	62.1	2217

*m_i = material constant for intact rock; *GSI* = Geological Strength Index; σ_{ci} = uni-axial compressive strength of the intact rock; m_b, s_b, a = rock mass material constants; σ_t = tensile strength; σ_c = uni-axial compressive strength.

3.5 *FE modelling*

The parametric analysis was carried out using the FE software Plaxis 3D (*v22.01*) (Bentley, 2022). The 2D-equivalent model, shown in Figure 5, permitted to impose the plane-stress condition (σ_y = 0) to the building as well as the plane-strain constraint (ε_y = 0) to the soil deposit.

The soil layers, the building and the tunnel were all modelled using 10-node tetrahedral finite elements with 4 Gauss integration points, adopting second order shape functions for displacements and first order interpolation for strain, for a total amount of 29392 finite elements and 49938 nodes. The base of the model is fully constrained, whereas the nodes along the lateral boundaries are constrained only in the horizontal direction. Interface elements at the soil-foundation contact were not used, which implies a perfect bonding condition. The tunnel excavation was simulated by imposing elliptically shaped displacements along the upper semi-circumference of the tunnel. This approach proved to be the most effective in reproducing the subsidence profile predicted by the well-known analytical Gaussian distributions under green-field conditions.

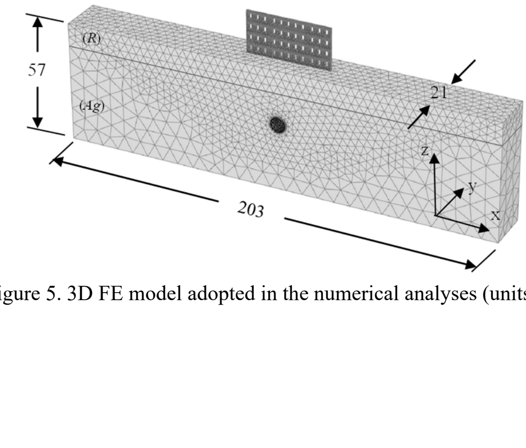

Figure 5. 3D FE model adopted in the numerical analyses (units: m).

4 APPLICATION OF THE DESIGN CHART BY MAIR (2013) TO THE LINE C OF ROME UNDERGROUND

To evaluate the effectiveness of the design chart proposed by Mair (2013) to the city centre of Rome, the results of the parametric analysis were plotted into the chart previously shown in Figure 2, after determining the relative stiffness (ρ).

In this study, the moment of inertia was calculated based on the masonry section with a thickness d = 1 m. The neutral axis position varied with the deformation mode: in sagging it was assumed at mid-height of the wall, considering both the elevation and the foundation ($I_{sag} = [H + z_{pp}]^3/12$); in hogging, it was located at the foundation base level ($I_{hog} = [H + z_{pp}]^3/3$). Structural flexural stiffness, EI, was reduced by applying a coefficient $\alpha = 0.6$, corresponding to an opening percentage of 17.9 % (see Tab. 1; Melis and Ortiz, 2001). Here the soil Young's modulus was estimated as a weighted average of the Young's moduli of the soil layers above the tunnel, obtained from the interpretation of the cross-hole test results carried out over the area of Piazza Venezia. Since the FE model returned an average shear strain $\gamma = 0.1 \cdot 10^{-3}$ and to $0.9 \cdot 10^{-3}$ for the R and Ag layers, respectively, the following decay of the Young's moduli were computed: 0.70 and 0.73. These values fall well within the range of values reported in the literature for tunnel excavation (Atkinson *et al*., 1991), which hence may be taken as a reference for preliminary calculations. Figure 6 presents the results obtained from both the linear elastic and elasto-plastic modelling of the masonry. The two sets of points are overlapping, which indicates that, under the assumed volume loss V_L, the influence of the constitutive soil model on the displacements at the foundation level is negligible, since the soil-induced deformations are not sufficient to cause a reduction in the masonry stiffness. It is also worth noting that the six data points obtained from the FE analyses fall within or close the upper-bound envelope by Mair (2013), ranging between $M^{DR} = 0.2$ and 0.7, which demonstrates the key role played by the building stiffness in reducing the deflection ratio DR. This suggests that the chart may be considered useful for preliminary assessments in the city of Rome.

5 CONCLUDING REMARKS AND FURTHER DEVELOPMENTS

This study aimed to extend the analysis of modification factors previously proposed by Mair (2013) for London, UK, to the urban context of Rome, Italy, where construction of the new Metro Line C is currently underway. The adopted approach is based on the relative stiffness between the building and the soil. When applied correctly, this method offers the twofold advantage of being a simplified procedure, which does not require time-consuming computation tools, while still providing a structural damage assessment comparable to that obtained through advanced FE analysis.

Through 2D-equivalent model parametric FE analyses, varying the eccentricity ratio *e/B*, the influence of the structure on the ground displacement and strain field was determined. At the foundation level, the presence of the structure produced a lower deflection compared to the green-field solution. The study confirmed the correlation between the structural influence on the ground displacement field induced by tunnel excavation and the relative bending stiffness. The numerical results fell within or close the upper bound envelope proposed by Mair (2013). Further research is needed to assess the influence of the structure on the displacement and strain fields more accurately. 3D FE analyses may extend this study, aiming to assess when a full structural modelling of the building become necessary.

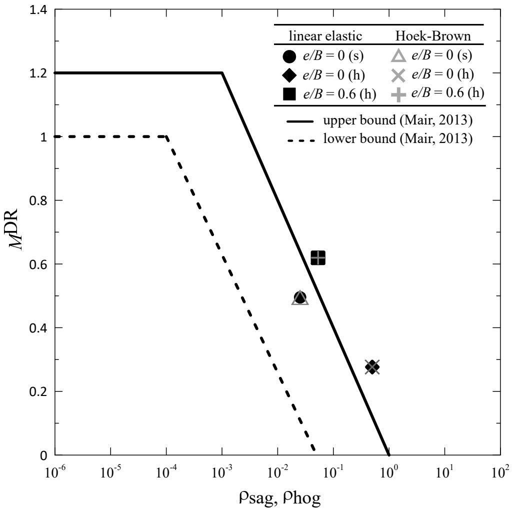

Figure 6. Result of the FE parametric analyses.

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