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Numerical Analysis of Tunnelling Effects on Masonry Buildings: the Influence of Tunnel Location on Damage Assessment

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Abstract The architectural heritage is subjected to various risk factors like the lack of maintenance, the material decay and the external solicitations. Nowadays, due to the ever-increasing demand for urban space, a relevant cause of structural damage that the historical buildings experience is the ground settlement due to excavation works. In the city of Amsterdam, for example, the construction of the new North-South metro line will involve an area characterized by the presence of many ancient masonry buildings. A fundamental phase of the design of this kind of projects is the assessment of the risk of subsidence which can affect the existing structures. The actual method to perform this assessment provides for a preliminary screening of the buildings located in the area surrounding the excavation, in order to evaluate which structures are at risk of settlement induced damage. It is based on the simplification of the building as a linear elastic beam and the assumption of the absence of interaction between the soil and the structure. An improved classification system should take into account the main parameters which influence the structural response, like the nonlinear behaviour of the building and the role played by the foundation in the soil-structure interaction. In this paper, the effect on the damage mechanism of the excavation advance and the location of the tunnel with respect to the building is evaluated. Numerical analyses are performed in order to understand the effect of different settlement profiles of the ground. A coupled model of the structure and the soil is evaluated, taking into account a damage model for the masonry building and the nonlinear behaviour of the soil-structure interaction. This paper demonstrates the importance of 3D modelling; neglecting the tunnel advance can lead to an underestimation of the damage.

Keywords: Damage assessment, masonry building, tunnelling

Introduction

Nowadays the tunnelling activity in urban areas is a growing phenomenon which involves a significant risk of damage of existing structures, especially historical masonry buildings. The assessment of this risk is an essential stage of the design process and it requires the understanding of a complex three-dimensional situation.

In practice, a simple 2D approach is adopted in practice to perform a preliminary assessment of the structural damage due to the tunnelling induced settlements (Burland and Wroth 1974). However, this analytical method cannot take into account the mutual influence of the building and the ground deformation, or the three-dimensional impact of the tunnelling advance. Therefore, numerical analysis has started to be considered a suitable tool for the damage assessment of buildings subject to tunnelling. 3D finite element models have been developed, including non-linear behaviour of the materials and realistic simulation of construction stages (Augarde 1997, Franzius 2004). In all these works, the simulation of adjacent masonry buildings has been simplified to linear elastic or no-tension structures, and the effect of foundation stiffness has been neglected.

In this paper, a 3D finite element analysis of a tunnel excavation in soft soil is presented; a two unit masonry house is included, modelled using a smeared cracking constitutive law, in order to analyze the stress and stiffness redistribution due to the settlement induced damage. Shallow strip foundations are represented by a bedding interface. The advance of the tunnelling process in time is simulated by subsequent stages of soil excavation and lining installation. Damage due to the tunnel advance is

analyzed and compared with a simplified analysis where the tunnel advance is ignored and the tunnel volume losses are applied simultaneously over the full length of tunnel.

Physical model

In this study the situation illustrated in Fig.1 is considered. A tunnel with a diameter of 10 m is excavated in soft soil under a masonry building on shallow strip foundations. The structure consists of two units separated by an internal wall; the external facades contain doors and windows. Two locations of the building have been studied: directly above the centreline of the tunnel and with an eccentricity of 27 meters between the tunnel and the building centrelines.

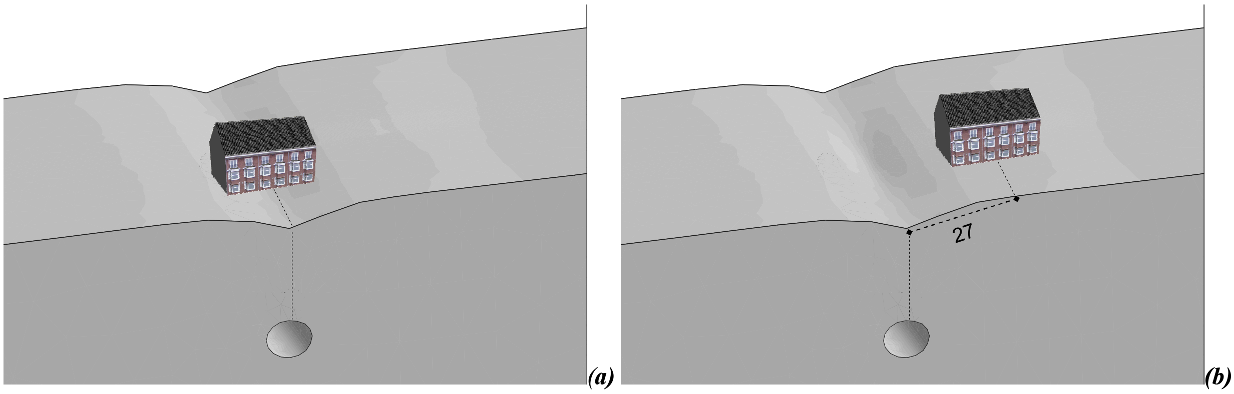


Figure 1: Geometry of the problem: building in sagging (a) and hogging zone (b)

Numerical model

Soil The soil block was modelled by bricks elements (Fig. 2a) with linear elastic behaviour and Young's modulus E linearly increasing with the depth z:

$$E = E_0 + mz \tag{1}$$

where E_0 is the Young's modulus at the surface and m is a constant gradient. The soil properties are listed in Table 5.

Building The external facades and the internal walls of the house were modelled by shell elements (Fig. 2b). The roof and the floor diaphragms were not represented because their stiffness is negligible with respect to the global stiffness of the building. The building is subject to dead and live loads. A rotating crack model is adopted for the masonry. The material is considered elastic in compression and having a linear softening behaviour in tension. The parameters for the model are reported in Table 5.

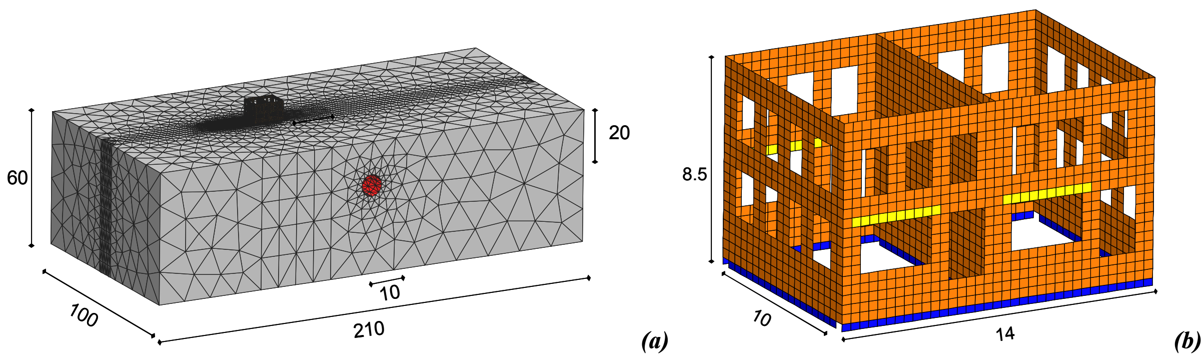


Figure 2: Meshes: global model (a) and building (b)(All the dimensions in meters)

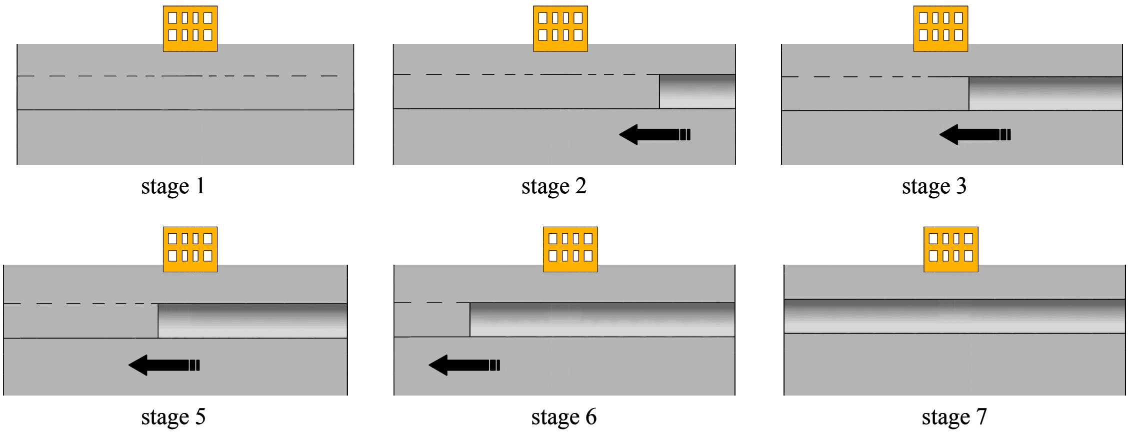


Figure 3. Staged analysis

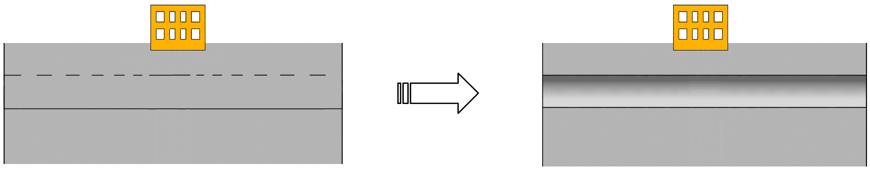


Figure 4: Non-staged analysis

Interface To simulate the transmission of vertical and horizontal deformations from the ground to the structure through the shallow strip foundations, interface elements are adopted, with a non-linear relation between the normal and shear stresses and the relative displacements across the interface. A no tension criterion in the normal direction and a Coulomb friction criterion in the tangential direction are assumed. Parameters are listed in Table 5.

Tunneling The tunnelling process was simulated by a sequence of discrete steps (Fig. 3). First, the initial state of the model without the tunnel was evaluated. In the next stages, successive tunnel ring elements were removed and a corresponding part of the lining was activated. At each step, the stresses calculated from the previous stage were applied, and the equilibrium state of the new model was obtained through an incremental adaptation to the actual loading. The concrete lining was modeled by curved shell elements with linear elastic behavior.

Table 5: Parameters for the numerical model

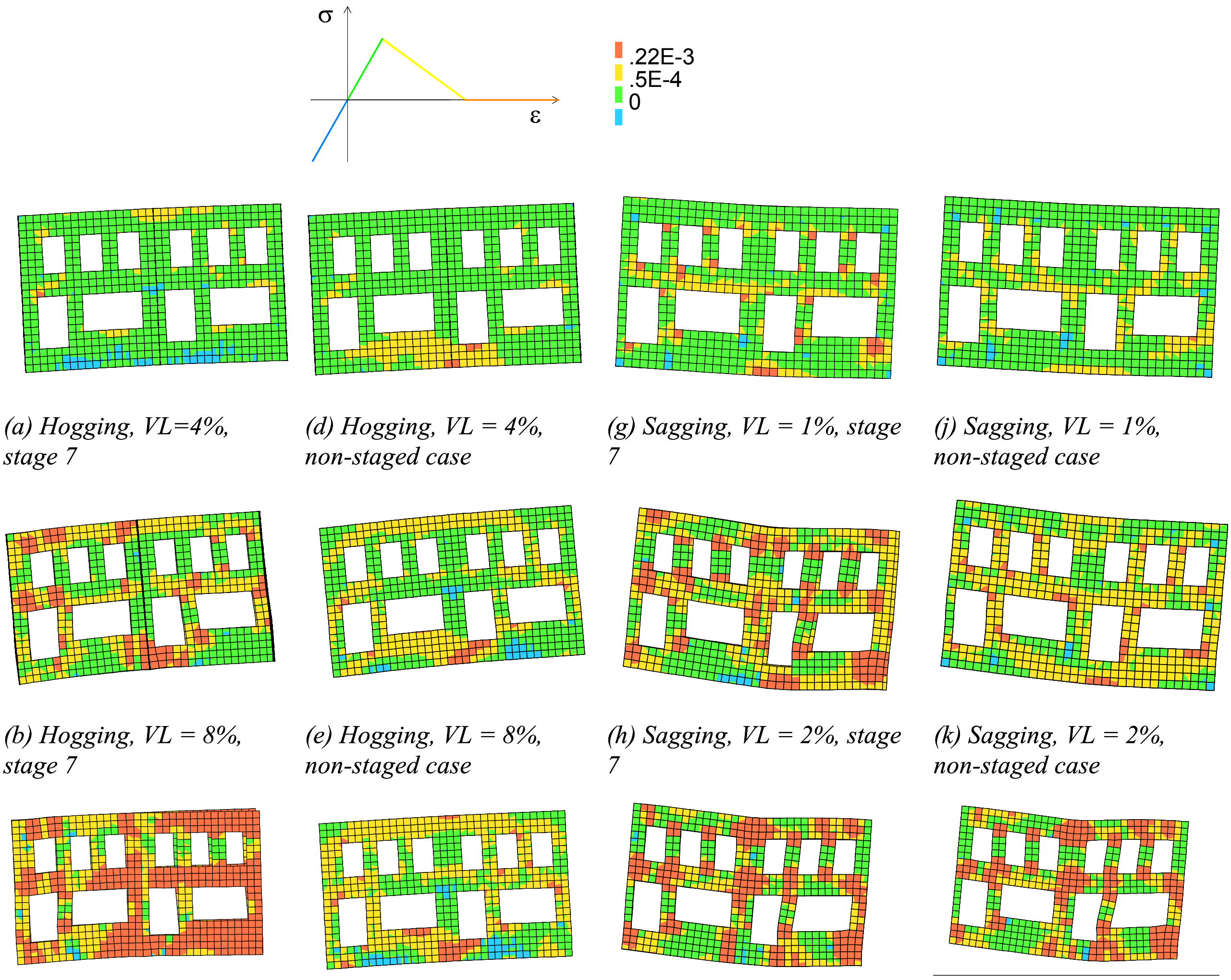
Material			*Soil*	*Masonry*	*Interface*
Young's modulus	E_0	[N/m^2]	$5\cdot10^{+7}$	$6\cdot10^{+9}$	-
Poisson's ratio	ν	[-]	0.3	0.2	-
Gradient	m	[N/m^3]	$1\cdot10^{+7}$	-	-
Density	ρ	[Kg/m^3]	$2\cdot10^{+3}$	$2.4\cdot10^{+3}$	-
Tensile strength	f_t	[N/mm^2]	-	$3\cdot10^{+5}$	0
Fracture energy	G_f	[N/mm]	-	$5\cdot10^{+1}$	-
Normal stiffness	k_n	[N/mm^3]	-	-	$2\cdot10^{+8}$
Tangent stiffness	k_s	[N/mm^3]	-	-	$1\cdot10^{+8}$
Cohesion	c	[N/mm^2]	-	-	0
Friction angle	φ	[°]	-	-	30
Dilatancy angle	ψ	[°]	-	-	0

Results

The influence on the structural response of different factors, like the building location, the soil-structure interaction and the volume loss occurring during and after the tunnelling process are evaluated in terms of global damage, cracks pattern and failure mechanism.

The principal strain contour plots and the magnified deformed shapes shown in Fig. 6 visualize the different cracked areas and highlight the different failure mechanisms.

The graphs in Fig. 8 give a representation of the total amount of relevant cracks in terms of the lengthening of the façade, as shown in Fig. 7 for typical failure mechanisms in case of sagging and hogging. In order to evaluate the importance of using a 3D analysis to assess the damage, two different models are compared: the solid line represents the results of the analysis performed including the tunnelling progress in time (Fig. 3), while the dotted line is the output of the model in which the tunnel advance is ignored and the tunnel volume losses are applied simultaneously over the full length of tunnel (Fig. 4).



(a) Hogging, VL=4%, stage 7

(d) Hogging, VL = 4%, non-staged case

(g) Sagging, VL = 1%, stage 7

(j) Sagging, VL = 1%, non-staged case

(b) Hogging, VL = 8%, stage 7

(e) Hogging, VL = 8%, non-staged case

(h) Sagging, VL = 2%, stage 7

(k) Sagging, VL = 2%, non-staged case

(c) Hogging, VL = 12%, stage 7

(f) Hogging, VL = 12%, non-staged case

(i) Sagging, VL = 4%, stage 7

(l) Sagging, VL = 4%, non-staged case

Figure 6: Maximum principal strain plots for different building location and volume loss values: comparison between staged and non-staged analysis

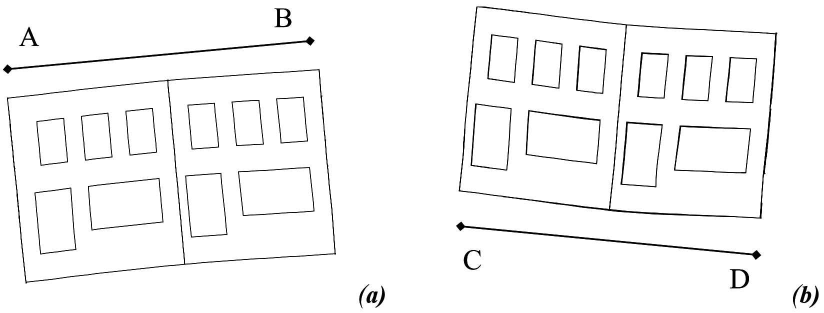
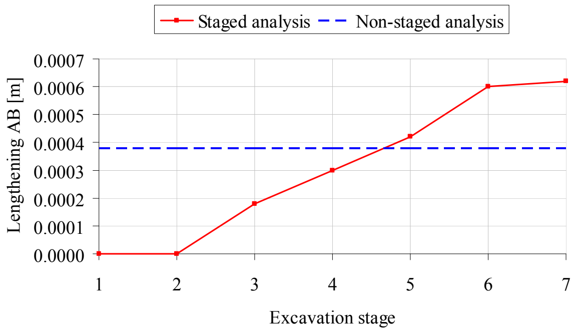
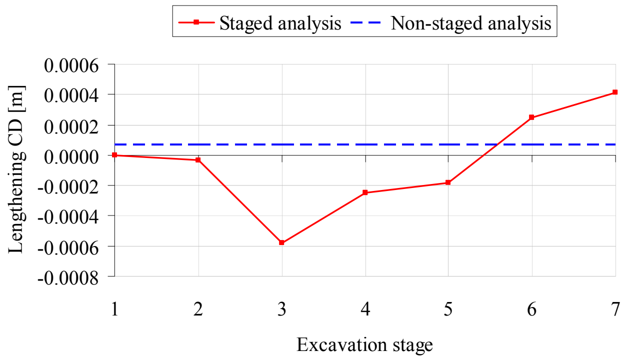


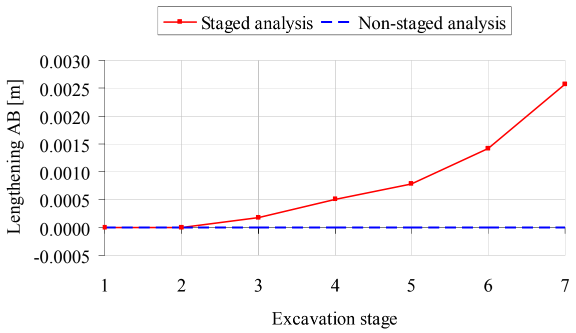
Figure 7: Distances used to represent the global damage in case of sagging (a) and hogging (b)



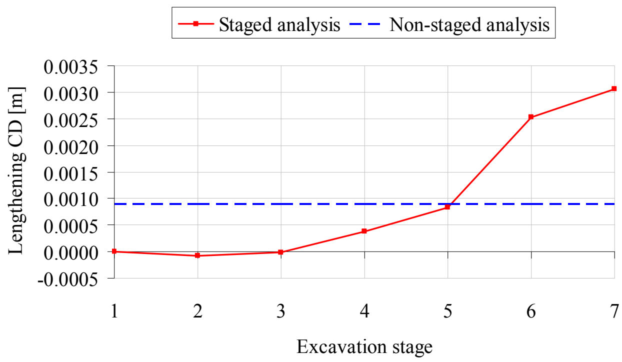
(a) Hogging, volume loss = 4%



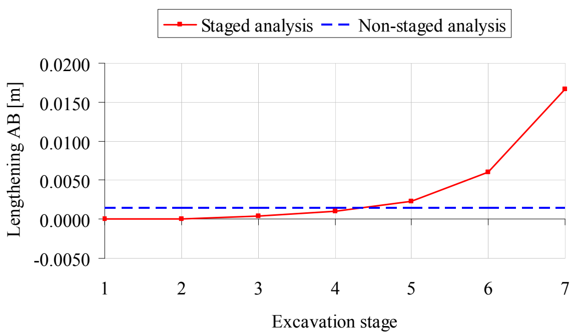
(d) Sagging, volume loss = 1%



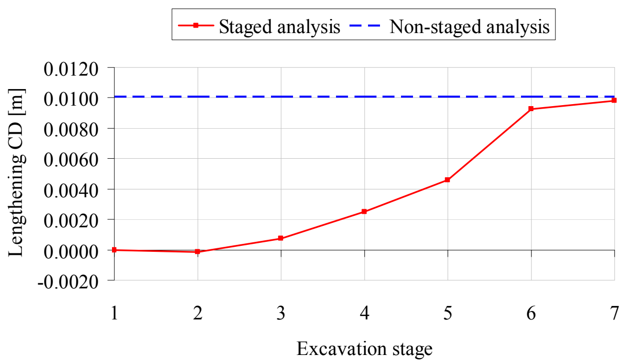
(b) Hogging, volume loss = 8%



(e) Sagging, volume loss = 2%



(c) Hogging, volume loss = 12%



(f) Sagging, volume loss = 4%

Figure 8: Potential damage for different building location and volume loss values: comparison between volume loss increasing and constant in the longitudinal direction

The graphs in Fig. 8a-c show a general increasing of the distance AB (Fig. 7c), as a consequence of the tunnelling-induced volume loss, which corresponds to the failure mechanism illustrated in Fig. 7a. Differences can be observed between the damage resulting from the staged analysis and the non-staged ones. Due to the progression of the tunnel, the transversal façade and the interfaces experience a certain amount of inelastic deformation, which is not completely recovered after the tunnel passage. Therefore, the final crack pattern for the structure in the staged analysis (solid line) is more pronounced than the one obtained neglecting the tunnel advance. In reality, this effect would be amplified in case of structures previously damaged or with foundations in poor conditions, like in the case of historic buildings.

For high values of volume loss, this difference in terms of crack width can lead to a different classification of the building in the traditional damage assessment method.

The simplifying assumption of neglecting the time variable influences also the failure mechanism. In fact, if the same volume loss is applied at the same time, then the horizontal deformation of the ground is less gradually redistributed to the structure through the interface, and a certain level of tensile strain concentrates in the lower side of the façade (Fig. 6d-f).

In the sagging zone, the settlement profile causes a different failure mechanism (Fig. 7b), with cracks arising in the lower part of the façade and propagating to the top (Fig. 6g-i). The role of the soil-structure interaction in the transmission of the deformations to the building is illustrated in Fig. 8d-f. For low values of volume loss, the horizontal restrain represented by the friction reduces significantly the tension due to the vertical settlement, causing an initial slight compression of the façade bottom (Fig. 8d). This effect tends to vanish when the vertical settlement becomes dominant (Fig. 8e-f).

For some values of volume loss, also in the sagging zone the nonlinear deformation accumulated during the tunnelling advance results in a crack pattern which is more severe than the one resulting from the non-staged analysis (Fig. 8d-e, Fig. 6g,j,h,k).

Conclusions

In this paper a 3D finite element analysis of a tunnel excavation in soft soil under a masonry building is presented. The importance of the 3D modelling in the damage assessment is demonstrated by the analysis of the results in terms of crack pattern and failure mechanism.

Neglecting the 3D tunnelling advance and building geometry leads to an underestimation of the damage; the value of this underestimation depends from different factors, like the expected volume loss and the position of the tunnel with respect to the building, which governs the type of settlement trough.

Non-linear modelling of the soil-structure interface is also a relevant issue, because the shear behaviour of the soil-foundation system affects the failure mechanism in a way which is more or less conservative, depending from the considered settlement profile.

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