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ScienceDirect
Procedia Engineering 158 (2016) 176 – 181
Procedia Engineering
www.elsevier.com/locate/procedia

VI ITALIAN CONFERENCE OF RESEARCHERS IN GEOTECHNICAL ENGINEERING – Geotechnical Engineering in Multidisciplinary Research: from Microscale to Regional Scale, CNRIG2016

Modelling of reinforced concrete framed structures interacting with a shallow tunnel

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Abstract

This paper deals with the numerical analysis of tunnelling-induced settlements on concrete framed structures founded on strip footings. In particular, a single frame with a variable number of storeys is taken into account in the present study, with an orientation perpendicular to the tunnel axis and a null eccentricity. The Finite Element simulations, carried out in three-dimensions with the code Abaqus, highlight the influence of the building stiffness and weight on the displacement field and provide an insight into the loading transfer mechanism occurring in the structural elements during tunnel excavation.

Peer-review under the responsibility of the organizing and scientific committees of CNRIG2016

Keywords: Tunnelling; soil-structure interaction; framed structures; numerical modelling

1. Introduction

The construction of tunnels in urban areas inevitably entails the interaction with existing structures. Current design approaches for the evaluation of tunnelling induced damage on buildings are based on semi-empirical evaluations of the deflection ratios and horizontal tensile strains at foundation level, assuming that the structure will conform to the greenfield displacements. If the stiffness of the structure is deemed significant, coupled numerical

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Peer-review under the responsibility of the organizing and scientific committees of CNRIG2016
doi:10.1016/j.proeng.2016.08.425

analyses should be performed including a model of the building. The latter can be simulated using either an equivalent solid [e.g. 1,2], for which appropriate equivalence criteria have to be defined, or a detailed structural model [e.g. 3,4,5].

This paper focuses on the soil-structure interaction due to mechanised tunnel excavation, with special reference to reinforced concrete framed buildings. The study is aimed at identifying those cases in which the building significantly alters the settlement trough induced by the excavation as compared to greenfield one. In such cases the relative contribution of the foundations and of the structural members to the global stiffness of the building is assessed, seeking for the optimal level of simplification of the structural model.

The research was carried out by performing three-dimensional parametric finite element analyses of the problem at hand, adopting the geotechnical conditions and the tunnel characteristics of the Milan metro-line 5 [5].

2. Description of the problem

The subsoil of Milan mainly consists of sand and gravel with variable hydraulic conditions. The tunnel has a circular section, with diameter D = 6.7 m and axis depth z = 25 m from the ground surface (Fig. 1). The average volume loss observed at the ground surface was 0.5%.

The investigated structure is a single reinforced concrete frame founded on a strip footing. The frame is L=20 m long, with five 4 m long bays; all the beams and columns have the same square cross section, with 0.4 m long side. The strip footing is 21.2 m long, 0.7 m high and 1.2 m wide. Frames with 5, 10 and 15 storeys, each having a height of 3.2 m, were simulated, with zero eccentricity with respect to the tunnel.

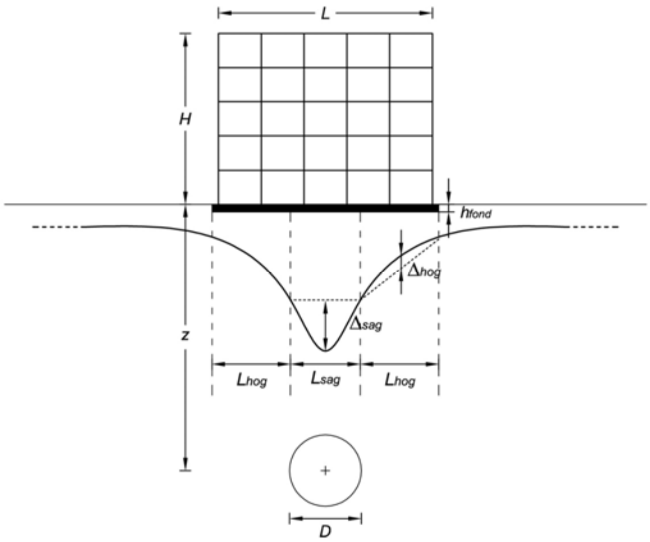

Fig. 1. Problem layout.

Table 1. Input parameters for soil and structure.

SOIL		STRUCTURE	
γ (kN/m^3)	20	γ (kN/m^3)	25
E' (MPa)	80-300 varying with depth	E (MPa)	30000
ν'	0.2	ν	0.2
c' (kPa)	2		
φ' (°)	33		
ψ (°)	11		

3. Description of the numerical model

The numerical model was set up using the commercial Finite Element (FE) code Abaqus (version 6.14). Only one quarter of the problem was analysed, taking advantage of the vertical planes of symmetry passing through the tunnel axis and the longitudinal axis of the frame, as all the investigated frames were symmetric and centred with respect to the tunnel and the latter was excavated in a single calculation step (Fig. 2). The soil domain extends 35 m and 20 m in the horizontal directions x and y, respectively, and 30 m along the vertical direction z. The limited extension of the model in this latter direction was purposely introduced to obtain a realistic subsidence trough at the ground surface, as discussed below. Boundary conditions consisted of constrained horizontal displacements perpendicular to the lateral faces and null displacement components at the base of the model.

The soil was modelled as an isotropic linear elastic, perfectly plastic material, with a Mohr-Coulomb strength criterion having the parameters listed in Table 1. The variation of the Young's modulus with depth, aimed at reproducing the stiffness profile described in [5], assuming an average reference deviatoric strain of 0.1%, was obtained by splitting the soil domain into six layers of increasing thickness with depth. The position of the water table was fixed at 15 m depth. For the frame and the foundation elements an isotropic linear elastic law was considered, with parameters summarised in Table 1.

Floors were not modelled directly. In fact, their influence was considered negligible in terms of stiffness, assuming that the direction of the joists is perpendicular to the frame. However, their weight was taken into account by fictitiously increasing the unit weight of the beams. Assuming typical composite reinforced concrete and masonry floors on both sides of the frame, with a hypothetical 4 m spacing between adjacent frames, a unit weight γ_{beams}=149.1 kN/m^3 was obtained for the beams.

20-node quadratic brick elements with reduced integration were selected for the soil and the foundation, while 3-node quadratic beam elements were adopted for the beams and the columns of the frame. In order to transmit the rotations of the foundation to the columns, 8-node shell elements, with zero self-weight and negligible stiffness, were attached to the extrados of the foundation, sharing common nodes with the foundation and the column elements. The FE mesh employed for the analysis is shown in Figure 2a, for the sample case of a 5 storeys frame. Distinct meshes were used for the foundation beam and for the soil, without shared nodes (Fig. 2b). The contact between the two bodies was simulated by enforcing a no-penetration/sliding-friction contact interaction, with coefficient of friction equal to tan (2/3 ϕ), thus using contact mechanics laws rather than interface elements.

The analyses comprised the following stages: 1) gravity activation in the soil domain; 2) deactivation of soil elements within the foundation volume and simultaneous activation of foundation elements and contact interaction; 3) activation of the frame elements; 4) deactivation of soil elements inside the tunnel volume and application of incremental displacements at the nodes of the its boundary. These displacements were set to obtain a homothetic contraction of the tunnel cross-section about its centre, in order to get a realistic greenfield subsidence profile at the ground surface. Their magnitude was calibrated to obtain a target volume loss of 0.5 %. The settlement profile was found to fairly match a Gaussian curve with trough width parameter K=0.45. All steps simulating the tunnel excavation were performed under drained conditions, given the high permeability of the soil.

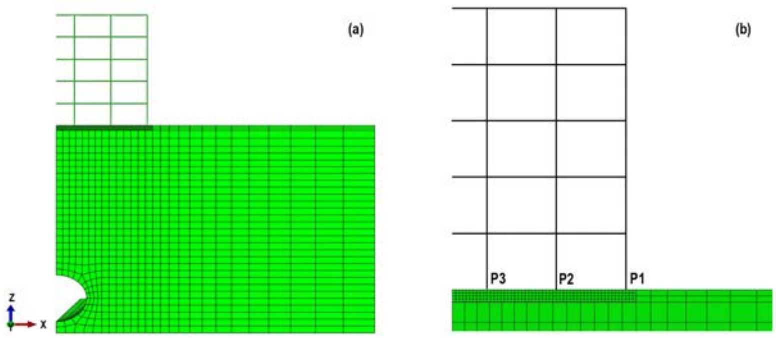

Fig. 2. (a) Finite element discretisation; (b) detail of the mesh.

4. Numerical results and discussion

The distribution of vertical soil displacements obtained at the end of the analyses at the foundation depth is shown in Figure 3a. The profiles calculated for frames with different number of storeys (i.e. 5, 10 and 15) are compared to that obtained in greenfield conditions. The presence of the structure induces smaller differential settlements beneath the foundation as the number of storeys increases. Additional analyses, in which the frame is not explicitly simulated and only its weight is accounted for by applying concentrated loads at the extrados of the foundation, clearly indicate that the stiffness contribution of the latter is predominant, especially for lower buildings. The vertical settlements of the foundation perfectly match those of the soil, suggesting that no gap has formed, contrarily to the experimental results reported by [6] for stiffer structures and larger volume losses.

It is interesting to note that the presence of the structure induces a hogging zone in the outer part of the frame, despite the fact that, in greenfield conditions, the soil deforms entirely in sagging in the same area. This phenomenon is related to the peculiar load distribution occurring in this specific kind of structures. In fact, as shown in Figure 3b for the 10-storey case, if the weight of the frame is disregarded or if it is accounted for by applying a distributed load at the extrados of the foundation, without simulating the entire frame explicitly, the hogging mode of deformation is not visible and the vertical settlement profile is very similar to that obtained in greenfield conditions.

An explanation of this load distribution mechanism can be deduced by inspecting Figure 4, which shows the forces at the base of columns P1, P2 and P3 (Fig. 2) as monitored during the analyses with the whole frame. The forces N are normalised with respect to the corresponding values N_{calc} evaluated through a preliminary hand calculation, considering the weight acting on the tributary area of each column. The figure points out that, at the gravity activation stage, the external column P1 is subjected to a larger force than that calculated analytically, while the opposite occurs to the internal columns P2 and P3, the difference being larger for the taller buildings. During the simulation of the tunnel excavation, the building stiffness is responsible of a load redistribution mechanism that produces an increase in the force at the base of column P1 and a corresponding decrease at the base of column P3, the value of N remaining constant for the intermediate column P2. These force variations are larger for the lower building.

The horizontal displacements and strains calculated at foundation depth for the soil and for the frame at the end of the analysis are reported in Figures 5a and b. Also in this case, an increase in the structural stiffness is associated to a decrease in the ground movements beneath the building with respect to those obtained in greenfield conditions, irrespective of whether the model includes the whole frame or its simplified version (foundation only and concentrated loads). However, there is a substantial difference between soil and foundation displacements, the latter being significantly smaller (Fig. 5a). This observation indicates that the strength at the soil-foundation interface has been reached and sliding occurred. The corresponding horizontal strains (Fig. 5b) are thus negligible, a particularly beneficial effect with respect to the damage induced in the building.

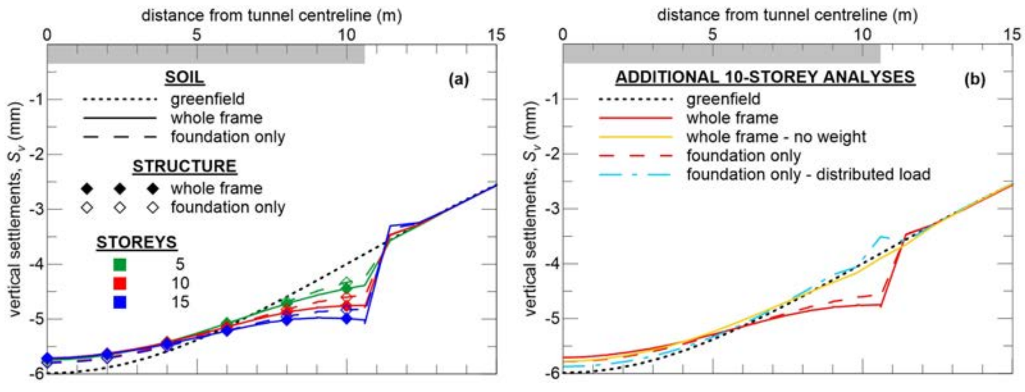

Fig. 3. (a) Vertical displacements at the foundation level for different structural configurations; (b) vertical displacements at the foundation level for different schematisation of the 10-storey structure.

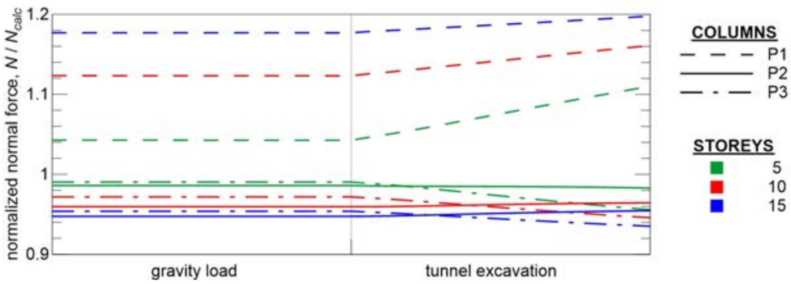

Fig. 4. Normalised normal force at the base of the three columns P1, P2 and P3 for different structural configurations (analyses with the whole frame).

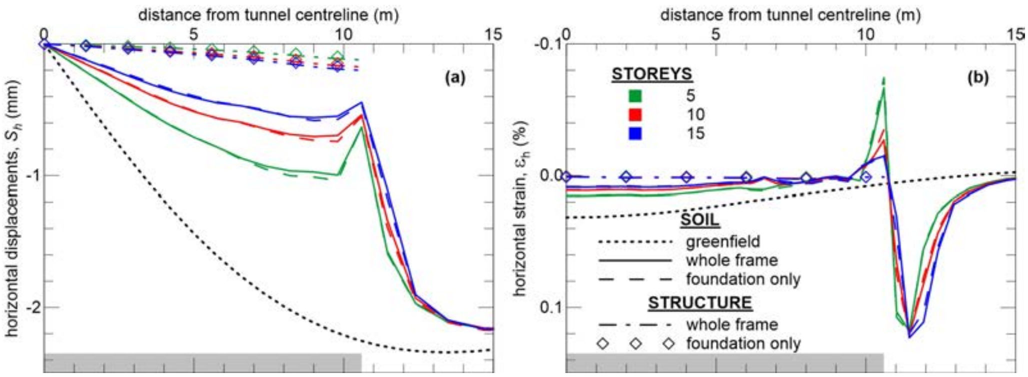

Fig. 5. (a) Horizontal displacements at the foundation level for different structural configurations; (b) horizontal strains at the foundation level for different structural configurations (positive values indicate shortening).

In addition to the horizontal strain, the damage in the building is also associated to the deflection ratio *DR*. Figure 6 shows the deflection ratios calculated in the sagging ($DR_{sag} = \Delta_{sag} / L_{sag}$) and hogging ($DR_{hog} = \Delta_{hog} / L_{hog}$) zones for the different analyses (see Fig. 1 for the meaning of the symbols).

The deflection ratio in the sagging zone DR_{sag} (Fig. 6a) is found to decrease with the number of storeys, lower values being associated to the analyses with the whole frame in comparison to those with the sole foundation. The opposite is true for the deflection ratio in the hogging zone DR_{hog} (Fig. 6b), for which the increase in the number of storeys produces an intensification of the structural deflection, in contrast with previously published results computed disregarding the influence of the building weight [e.g. 2,7] or taking into account equivalent beams with distributed loads [e.g. 8]. As previously discussed, this behaviour is related to the effect of the building weight, and in particular to its load distribution. DR_{hog} values are much lower than DR_{sag} for the 5-storey building, while they become comparable for the 15-storey case.

A convenient manner to summarise the previous results was proposed in [1] in terms of modifications factors. These factors indicate the extent to which the structure modifies the greenfield predictions of the relevant damage parameter. The modification factors for the deflection ratio in the sagging zone and for the horizontal strain are respectively defined as follow:

$$M_{DR_{sag}} = \frac{DR_{sag}}{\left(DR_{sag}\right)_{greenfield}} \quad ; \quad M_{\varepsilon_{hc}} = \frac{\varepsilon_{hc}}{\left(\varepsilon_{hc}\right)_{greenfield}} \qquad (1,2)$$

In Equation (1) the denominator represents the deflection ratio in the sagging zone calculated for the portion of the greenfield settlement trough which lies beneath the foundation. The corresponding quantity could not be computed for the hogging zone because the greenfield profile is characterised by sagging deformation only, as already pointed out. Figure 7a shows that the modification factor for the sagging zone decreases with the number of storeys, according to previously published results [e.g. 2,7,8].

In Equation (2) ε_{hc} indicates the maximum horizontal compressive strain (no tensile horizontal strains were obtained beneath the foundation, see Fig. 5b). This parameter (Fig. 7b) decreases with the number of storeys if referred to the soil, while it shows a modest increase, although with significantly lower values, if the foundation is considered. This result, appropriate for the evaluation of the potential building damage, is in contrast with the typical behaviour observed in the case of masonry structures, as described in [e.g. 2].

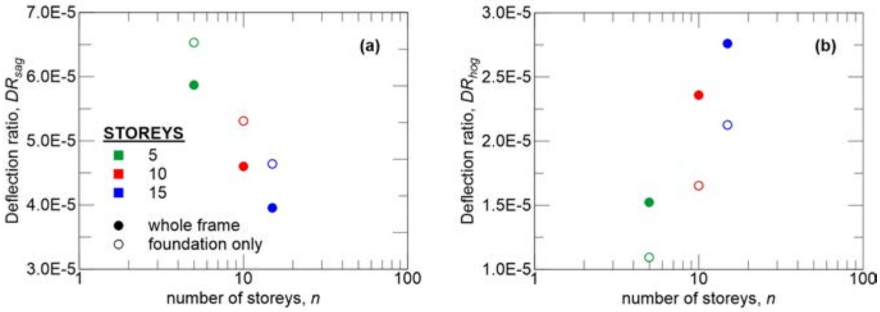

Fig. 6. (a) Deflection ratio in the sagging zone; (b) deflection ratio in the hogging zone.

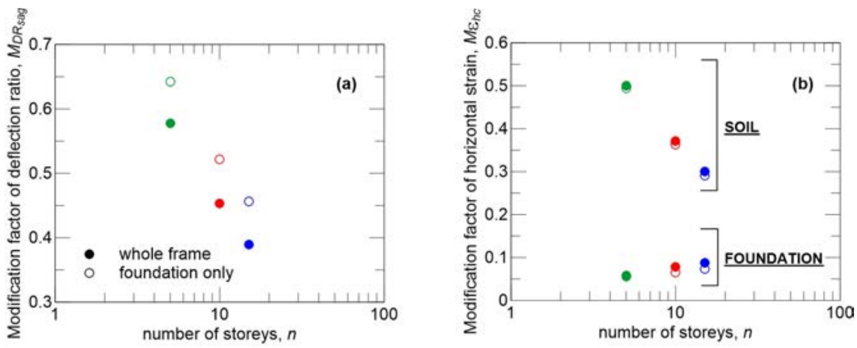

Fig. 7. (a) Modification factors of deflection ratio; (b) modification factors of horizontal strain.

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