Available online at www.sciencedirect.com

ScienceDirect

Procedia Engineering 158 (2016) 254 – 259

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

A mitigation technique to reduce ground settlements induced by tunnelling using diaphragm walls

Laura Fantera[a,*], Sebastiano Rampello[a], Luca Masini[a]

[a]*University of Rome La Sapienza, Via Eudossiana 18 – 00184 Roma*

Abstract

Ground movements due to the excavation of shallow tunnels may cause damage or loss of functionality to nearby buildings: embedded diaphragm walls pre-installed between the tunnel and the building can be effective in reducing these movements thus preventing damage on existing structures. Plane strain finite element analyses are first presented in the paper in which a continuous diaphragm made by adjacent panels was modelled to evaluate the effects of different geometrical and mechanical parameters. For a limited set of these parameters, 3D FE analyses were also performed maintaining a plane strain excavation scheme to evaluate the effectiveness of an infinite diaphragm made by closely spaced piles. A 3D study is finally presented in which the horizontal diaphragm length needed to attain the same efficiency provided by an infinite diaphragm is evaluated. Completion of tunnel construction was assumed in both 2D and 3D studies.

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

Keywords: tunnelling; mitigation of ground movements; diaphragm wall; three-dimensional numerical analysis

1. Introduction

The main design restriction for tunnel excavation in urban areas is the induced ground deformation when the tunnel line passes near an existing building. Both active and passive techniques have been developed to control the movements of structures affected by tunnelling. For example, site observations and laboratory investigations have

* Corresponding author.
E-mail address: laura.fantera@uniroma1.it

Peer-review under the responsibility of the organizing and scientific committees of CNRIG2016
doi:10.1016/j.proeng.2016.08.438

shown the effectiveness of compensation grouting as an active mitigation technique [1,2]. In this paper, a numerical study was carried out to evaluate the possibility of reducing ground movements induced by tunnelling by pre-installing a passive protective barrier between the tunnel and the buildings exposed to potential damage, to limit the lateral propagation of displacements from the tunnel axis. Mitigation barriers of bored piles and micropiles have been used in Shanghai [3,4] and Barcelona [5,6] showing to be effective in reducing ground movements induced by tunnelling. The effectiveness of protective barriers has also been demonstrated by centrifuge tests [7] and numerical studies have been performed to investigate the role of geometrical and mechanical characteristics of the diaphragm [8,9,10].

In this study, parametrical analyses were performed using the FE codes Plaxis 2D and 3D to define the soil-structure interaction mechanisms that allow the reduction of ground movements induced by tunnel excavation. Plane strain numerical analyses were first carried out to evaluate the effects of the wall length and of the roughness at the soil-diaphragm interface. A three-dimensional study was then carried out to evaluate the efficiency of a diaphragm made by piles installed at different spacing, that is generally preferred to a continuous diaphragm wall made by adjacent panels due to the minor impact related to pile construction.

Finally, a 3D model was developed to evaluate the minimum horizontal length needed for a diaphragm to reach in a given section about the same efficiency provided by a diaphragm of infinite horizontal extension. Completion of tunnel construction was assumed in both the 2D and 3D analyses.

2. Model definition

2.1. Soil properties

Ground conditions assumed in the analyses are representative of those encountered in the historical centre of Rome, close to the *Aurelian Walls* at *Porta Asinaria*. The soil profile consists of a 17 m thick gravelly made ground (MG) overlying a slightly overconsolidated sandy silt layer, about 13 m thick, and a sandy gravel with a thickness of about 12 m which rests over a thick deposit of stiff Pliocene overconsolidated clay. In the analyses the conservative assumption of a diaphragm wall entirely installed in the sandy silt was made.

The mechanical behaviour of the MG was described using the Hardening Soil model (*HS*), while the behaviour of the sandy silt layer was described by the Hardening Soil model with small-strain stiffness (*HSsmall*) [11,12], both implemented in the code Plaxis. In addition to the features of the *HS* model, *HSsmall* model is able to describe the hysteretic para-elastic soil behaviour at very small strain. The strength and stiffness parameters of the soils were selected to reproduce data from cross-hole tests and from resonant column and triaxial tests carried out on undisturbed samples retrieved from the site. Figure 1a lists the values adopted in the analyses.

Measurements of pore water pressure show a downwards seepage in the silty soil, from the made ground towards the gravel, with the groundwater head decreasing from 26 to 17 m o.d..

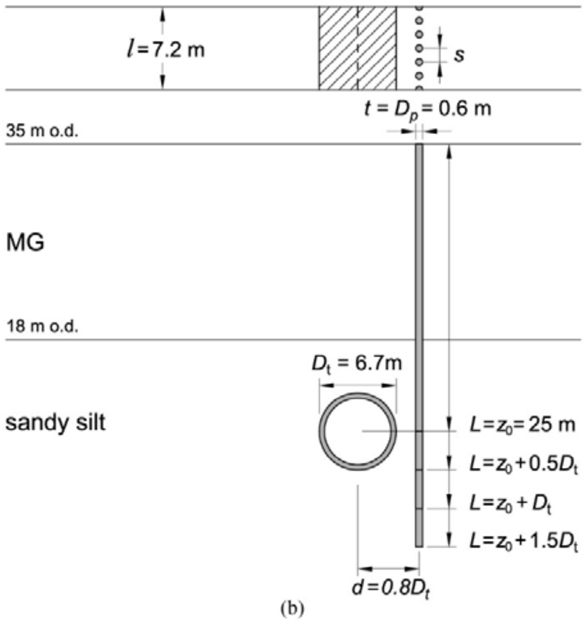
2.2. Problem definition

The model layout is schematically represented in Figure 1b: the tunnel has a diameter D_t = 6.7 m and its axis is at a depth z_0 = 25 m. The offset d of the diaphragm from the tunnel axis is equal to $0.8D_t$. Four possible wall lengths were investigated: $L = z_0$, $z_0 + 0.5D_t$, $z_0 + D_t$ and $z_0 + 1.5D_t$. The diaphragm was assumed to be continuous, made by adjacent panels 0.6 m thick, or discontinuous, made by a line of piles with a diameter D_p = 0.6 m, installed at spacing $s = D_p$, $1.5D_p$ or $2D_p$. In both the 2D and 3D analyses, the vertical boundaries of the transverse section of the models were located 12 tunnel diameters away from the tunnel axis, while the meshes extended about 4 diameters below the tunnel: vertical boundaries were restrained horizontally, while the base was fixed. A horizontal width l of 7.2 m was adopted for the 3D FE mesh, that is capable to model a line of piles with 12, 8 and 6 piles installed at spacing $s = D_p$, $1.5D_p$ and $2D_p$, respectively. Drained condition of soil was considered in all the analyses.

Plaxis interface elements were used to reproduce the soil-structure contact: they are defined by the R_{int} parameter that reduces both strength and stiffness parameters of a selected soil. The interface of the diaphragm wall was assumed either partially (R_{int} = 0.7) or fully rough (R_{int} = 1.0), while it was assumed R_{int} = 0.7 for the contact between the tunnel lining and the soil.

soil		MG	sandy silt
γ	(kN/m^3)	17.0	19.5
c'	(kPa)	5	28
φ'	(°)	34	27
OCR		3.5	1.3
k_0		1.0	0.62
constitutive model		*HS*	*HSsmall*
G_0^{ref}	(MPa)	--	125
$\gamma_{0.7}$	(%)	--	0.035
ν'		0.2	0.2
$E'_{ur}{}^{ref}$	(MPa)	240	150
$E'_{50}{}^{ref}$	(MPa)	24	8.21
$E'_{oed}{}^{ref}$	(MPa)	24	5.52
m		1.0	0.8

(a)



(b)

Fig. 1. Model definition: (a) strength and stiffness parameters of soil and initial state condition; (b) geometry of the model.

2.3. Simulation of tunnel excavation and wall construction

In all the analyses a simple two-dimensional excavation scheme was adopted to simulate tunnel construction: tunnel excavation was simulated imposing a set of vertical displacements at the upper portion of the tunnel, while leaving the invert fixed; in a second stage the structural elements that simulate tunnel lining were activated and the condition on displacement was removed.

This procedure appears more suitable than a uniform radial contraction for tunnels excavated with closed-shield tunnel boring machines [13]. The maximum displacement δ imposed at the tunnel crown was determined by trial and error to meet the requirement of a notional volume loss V_L equal to 1.0 % after completion of the tunnel, both in the presence or the absence of the diaphragm. Tunnel lining, made of precast concrete elements with a thickness of 0.3 m, was simulated using plate elements.

The diaphragm wall was modelled as wished in place and described as a solid element. All structural elements were modelled as linear elastic materials.

3. Analyses results

The effectiveness of the different schemes of embedded diaphragms in reducing settlements induced by tunnelling was evaluated by comparing the transverse settlement profiles, assuming the structure to follow the ground movements. A quantitative evaluation of the mitigation effect is provided by the integral efficiency η_{int}, defined using the ratio between the areas of the transverse settlement trough computed beyond the barrier position in the presence (w_{dw}) and in the absence (w_{ff}) of the diaphragm wall:

$$\eta_{int} = 1 - \frac{\int w_{dw}(x)dx}{\int w_{ff}(x)dx} \tag{1}$$

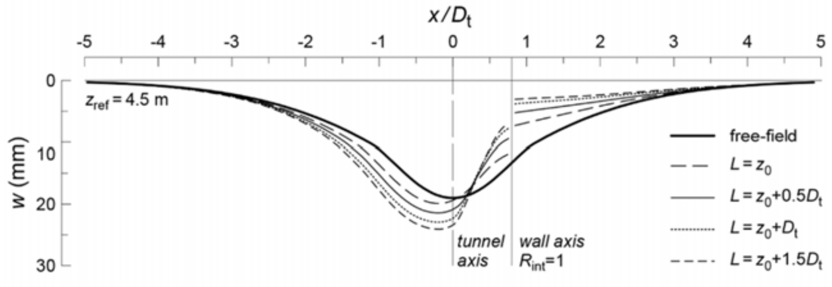


Fig. 2. Profile of vertical displacement (2D analyses).

Table 1. Integral efficiency computed for different lengths and surface roughness of the diaphragm.

	η_{int}			
	$L = z_0$	$L = z_0 + 0.5D_t$	$L = z_0 + D_t$	$L = z_0 + 1.5D_t$
$R_{int} = 1.0$	0.26	0.40	0.52	0.58
$R_{int} = 0.7$	0.36	0.46	0.53	0.57

The closer to 1 is the value of η_{int}, the greater is the settlement reduction. The following evaluations were made at a depth z_{ref} of 4.5 m from ground surface, which can be assumed to resemble the typical depth of a building basement. Figure 2 shows the profiles of vertical displacement computed from 2D analyses. The presence of a pre-installed diaphragm wall reduces both the ground settlements beyond its location and the curvature of the settlement trough; therefore, the barrier is capable to reduce the building damage that is usually related to the free-field profile of settlements induced by tunnelling. Settlements reduce with the increase of the diaphragm length, but the benefit becomes negligible for $L > z_0 + D_t$.

Table 1 shows the values of integral efficiency η_{int} as a function of the wall length and the soil-wall interface roughness. It increases with diaphragm length but with a reducing rate: values of $L > z_0 + D_t$ result in minor increments of η_{int}. Higher values of η_{int} are computed assuming a partially rough interface ($R_{int} = 0.7$), but differences are seen to reduce with wall length, becoming negligible for $L > z_0 + D_t$.

The observed trend can be attributed to the interaction between the displacement field induced by tunnelling and the diaphragm that results in the development of shear stresses along the wall side. Figure 3a shows the mobilised strength ratio τ/τ_f along the two vertical faces of the barrier: positive values indicate upwards shear stresses τ. On the tunnel-side interface soil settles relative to the diaphragm generating downwards shear stresses, while the diaphragm settles relative to the portion of soil facing to the building so that downwards shear stresses are applied by the wall to the soil. The reduction of interface roughness reduces the downdrag force transmitted by the wall to the soil with an increase in efficiency. The increase of diaphragm length offers a longer portion of wall for the development of the shear stresses making them reaching lower values; as a consequence, the mobilised shear strength reduces and the influence of wall roughness becomes less important.

The efficiency of a discontinuous diaphragm made by a line of piles was evaluated through 3D analyses in which the simple scheme of plane strain tunnel excavation was maintained. Piles installed at spacings $s = D_p$, $1.5D_p$ or $2D_p$ were modelled, firstly assuming a fully rough contact between the piles and the soil ($R_{int} = 1$). Figure 4a shows the values of η_{int} computed for a line of piles compared to those obtained for a continuous barrier. The row of piles is still able to reduce ground settlements induced by tunnelling, but the efficiency is lower than that of the continuous diaphragm and it decreases with increasing piles spacing: closely spaced piles with $s \leq 1.5D_t$ should be used to obtain a significant reduction of the ground movements.

The line of piles is less able to act as a screen for the displacement induced by tunnelling: downwards vertical displacements still develop between and just behind the piles, leading to values of efficiency lower than that provided by a continuous diaphragm. Differently from the case of a continuous diaphragm wall, Figure 3b shows that for a line of piles downdrag shear stresses develop not only on the tunnel-side interface of the wall but they are still

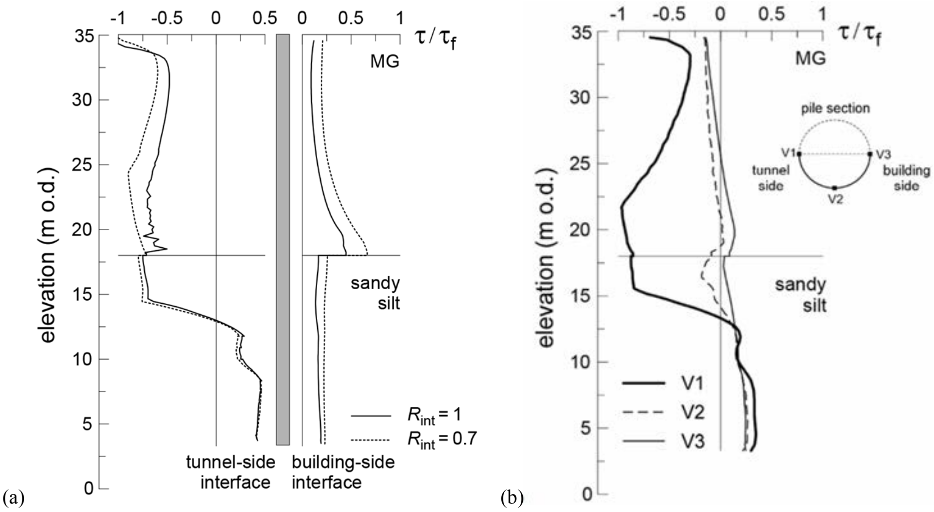
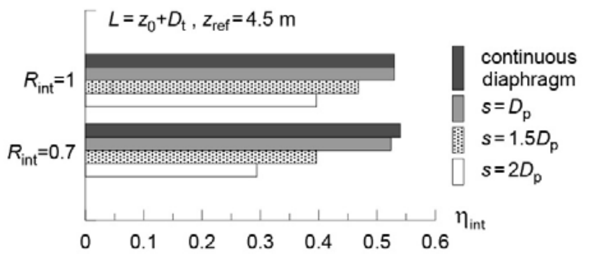


Fig. 3. Shear stresses mobilized at the diaphragm surface: (a) 2D analyses, continuous diaphragm ($L = z_0 + D_t$); (b) 3D analyses, line of piles $s = 1.5D_p$ ($L = z_0 + D_t$; $R_{int} = 1$).

	η_{int}		
diaphragm		*L*	
wall	$z_0 + 0.5D_t$	$z_0 + D_t$	$z_0 + 1.5D_t$
continuous	0.39	0.53	0.60
$s = D_p$	0.37	0.53	0.60
$s = 1.5D_p$	0.33	0.47	0.55
$s = 2D_p$	0.29	0.40	0.50

(a)



(b)

Fig. 4. 3D analyses: (a) integral efficiency computed for a continuous diaphragm and for a line of different spaced piles ($R_{int} = 1$); (b) integral efficiency computed assuming different surface roughness ($L = z_0 + D_t$).

present on the building-side interface. Piles are entirely enveloped by the subsidence field induced by tunnelling and a fully rough interface is more suitable to support the soil than a partially rough one. Figure 4b shows that for values of pile spacing $s \geq 1.5D_t$ a fully rough interface is fundamental to reach greater values of efficiency.

Finally, 3D analyses characterised by a diaphragm of finite horizontal length L_h were carried out to evaluate the minimum horizontal extension of the wall needed to reach in a given control section the same efficiency computed in plane strain condition ($L_h = \infty$). The diaphragm is supposed to be both continuous, made by adjacent panels, and discontinuous, made by a line of piles installed at spacing $s = 2D_p$. The diaphragm has a length $L = z_0 + D_t$ and an interface roughness factor $R_{int} = 1$. The analyses domain extends from the control section to the section in which free-field conditions are reached again. The efficiency of both the layouts increases with L_h but with a decreasing rate (Table 2): about 80% of η_{int} under plane strain conditions is computed for $L_h \cong 2D_t$ ($L_h = 2l$), while about 90% of the plane strain integral efficiency is evaluated for $L_h \cong 3D_t$ ($L_h = 3l$), irrespective of the diaphragm type.

Table 2. Variation of the integral efficiency ($R_{int} = 1$; $L = z_0 + D_t$) with the wall horizontal length.

	η_{int}				
diaphragm wall	$L_h = l$	$L_h = 2l$	$L_h = 3l$	$L_h = 4l$	$L_h = \infty$
continuous	0.34	0.45	0.48	0.50	0.53
$s = 2D_p$	0.21	0.32	0.36	0.37	0.40

4. Conclusions

Diaphragm walls pre-installed between the tunnel and the buildings show to be effective in mitigating the effects induced by tunnelling: the interaction between the displacement field induced by tunnel excavation and the diaphragm produces a favourable change in the shape of computed settlement troughs.

To optimise soil-structure interaction and to obtain a significant settlement reduction the diaphragm length should be extended below the tunnel invert, at a depth of about D_t from tunnel axis. The roughness at the diaphragm-soil interface plays an important role in the effectiveness of the intervention, affecting the shear stresses transmitted between the soil and the wall. It was seen that for a continuous diaphragm wall made by adjacent panels higher values of integral efficiency are computed for a smoother wall interface; conversely, for a line of piles a rough surface is needed to obtain higher values of efficiency. In fact, for a barrier made of a line of piles, the displacement field developing between the piles and the shear stress mobilised along the lateral surface of the piles mainly affects the effectiveness of this mitigation technique. Therefore, to account for the three-dimensional character of the interaction mechanism, 3D models should be used rather than 2D model, in which a piled wall can be described only by a continuous equivalent diaphragm of reduced stiffness and weight.

While analyses results suggest the convenience to adopt construction solutions capable to provide partially or fully rough interfaces depending on the type of the protective barrier, it is worth mentioning that the behaviour of a real interface is strongly dependent on the construction process that is not easy to keep under control. A safe design of a continuous diaphragm wall should then be carried out assuming fully rough interfaces, while for a line of piles partially rough interfaces should be assumed: this does not result in an excessive underestimation of wall effectiveness in that influence of interface assumptions was seen to decrease with diaphragm length.

Acknowledgements

The research work presented in this paper was partly funded by the European research project NETTUN (New Technologies for Tunnelling and Underground Works).

References

[1] R.J. Mair, D.W. Hight, Compensation grouting, World Tunnelling (1994) 361-367.
[2] L. Masini, S. Rampello, K. Soga, An approach to evaluate the efficiency of compensation grouting, Journal of Geotechnical and Geoenvironmental Engineering 140(12) (2014) 401-407.
[3] Y. Bai, Z. Yang, Z. Jiang, Key protection techniques adopted and analysis of influence on adjacent buildings due to the Bund Tunnel construction, Tunnelling and Underground Space Technology 41 (2014) 24-34.
[4] X. Chen, Y.H. Lui, W.H. Cao, Z.F. He, Protection for the former observatory during construction of the Yan An Dong Lu Tunnel, in: A. jr Negro, A.A. Ferreira (Eds.), Tunnels and Metropolises, Balkema, Rotterdam (1998) 1083-1088.
[5] A. Di Mariano, A. Gens, J.M. Gesto, H. Schwartz, Ground deformation and mitigating measures associated with the excavation of the new Metro line, in: Geotechnical Engineering in Urban Environments, Proc. 14th ECSMGE, Millpress Science Publisher, The Netherlands, Rotterdam (2007) 1901-1906.
[6] R. Katzenbach, S. Leppla, M. Vogler, M. Seip, S. Kurze, Soil-structure interaction of tunnels and superstructures during construction and service time, in: Proc. 11th Int. Conf. on Modern Building Materials, Structures and Techniques, MBMST 2013, Procedia Engineering 57 (2013) 35-44.
[7] E. Bilotta, R.N. Taylor, Centrifuge modelling of tunnelling close to diaphragm wall. Int. J. Phys. Modelling in Geotechnics 1 (2005) 25-41.
[8] E. Bilotta, Use of diaphragm walls to mitigate ground movements induced by tunnelling, Géotechnique 58(2) (2008) 143-155.
[9] E. Bilotta, G. Russo, Use of line of piles to prevent damages induced by tunnel excavation, Journal of Geotechnical and Geoenvironmental Engineering 137(3) (2011) 254-262.
[10] E. Bilotta, S.E. Stallebrass, Prediction of stresses and strains around model tunnels with adjacent embedded walls in overconsolidated clay, Computers and Geotechnics 36 (2009) 1049-1057.
[11] T. Benz, P.A. Vermeer, R. Schwab, A small-strain overlay model, Int. Journal for Numerical and Analytical Methods in Geomechanics 33 (2009) 25-44.
[12] T. Schanz, P.A. Vermeer, P.G. Bonnier, Hysteretic damping in a small-strain stiffness model, Proc. of the 10th Int. Symp. on Numerical Models in Geomechanics, NUMOG 10, London (1999) 737-742.
[13] S. Rampello, L. Callisto, G. Viggiani, F.M. Soccodato, Evaluating the effects of tunnelling on historical buildings: the example of a new subway in Rome, Geomechanics and Tunnelling 5(3) (2012) 254-262.