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BEHAVIOR PREDICTION AND MONITORING OF A DEEP EXCAVATION IN THE HISTORIC CENTER OF BRESCIA

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ABSTRACT

The construction of a new underground car park in the historic center of Brescia (Northern Italy) required the execution of a 130m long, 23m wide and 15-20m deep excavation which was supported by concrete multi-anchored diaphragm walls. The site is located between two facing ancient walls (15-16th century) that support 3-4 storey buildings in precarious conditions. The soil stratigraphy comprises a superficial layer of made ground over a clayey to sandy soil deposit supporting a perched water table. A comprehensive monitoring system was set up before construction that included inclinometers, precise leveling and automatic structural monitoring by means of a high-precision total station. A number of finite element numerical analyses were conducted using different constitutive laws for the soil to evaluate the behavior of the retaining structure and safety of adjacent buildings. The simple linear elastic-perfectly plastic constitutive law predicted unrealistic soil behavior and unreliable effects on adjacent structures. The result of numerical analyses performed with soil models that include isotropic hardening with stress and strain stiffness dependency compared well with measurements.

INTRODUCTION

The excavation for the construction of the new underground car park in Piazzale Arnaldo in the city of Brescia (Northern Italy) was completed between January and March 2012. Having a 130 m length, 25 m width and 15-20 m total depth, the new car park represents one of the biggest underground structures within the limits of the historic center of the city (Fig. 1).

The construction site is surrounded by a number of buildings, particularly on the East and West sides, and the boundary conditions are highly affected by the presence of a large section of the defense walls of the city, built between the 15th and 16th century, under the domination of the Republic of Venice (Fig. 2). In particular, the construction site is located between two facing ancient walls made of stone blocks and masonry. During the 18-19th century a group of 3-4 storey buildings was constructed along the East and South side of the site (Fig. 3), together with a municipality building that was part of the structures dedicated to receive and control people and goods entering the city (the "Porta Venezia" area, named after the direction of the main road departing from the city). The Rigagnoli-Pusterla buildings are approximately 13 m high (3 levels), 6-8m wide and 55 m long, the structure is made from masonry and stone blocks in elevation, wooden floors and roof and no basement.

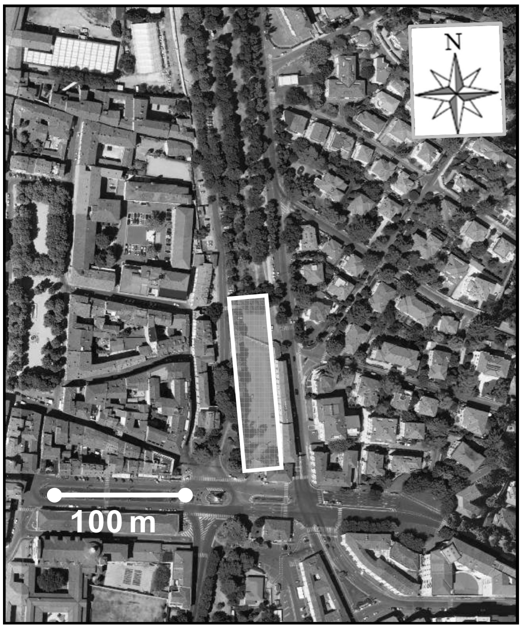

Fig. 1. Site location in the center of Brescia, Italy (present time).

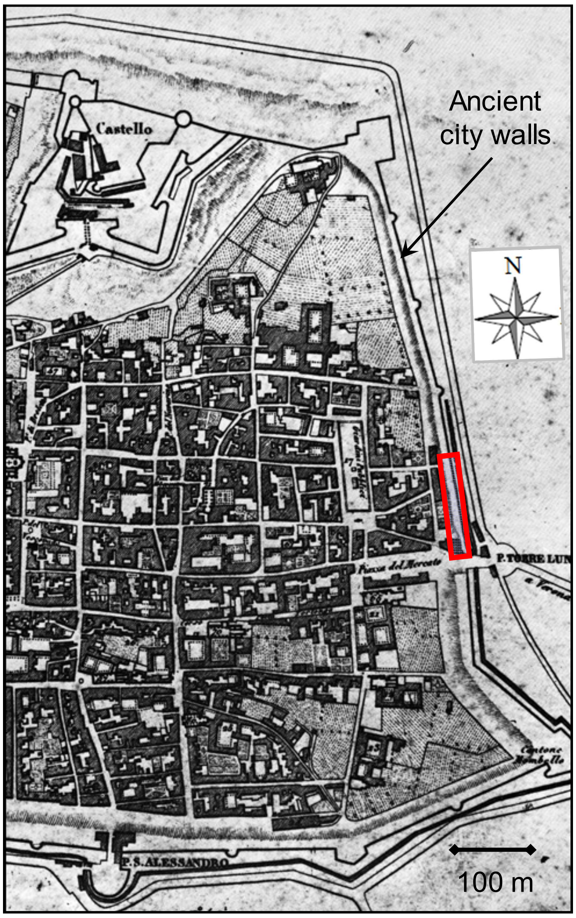

Fig.2. Historic map (1826, excerpt) of the city of Brescia and site location along the city ancient walls.

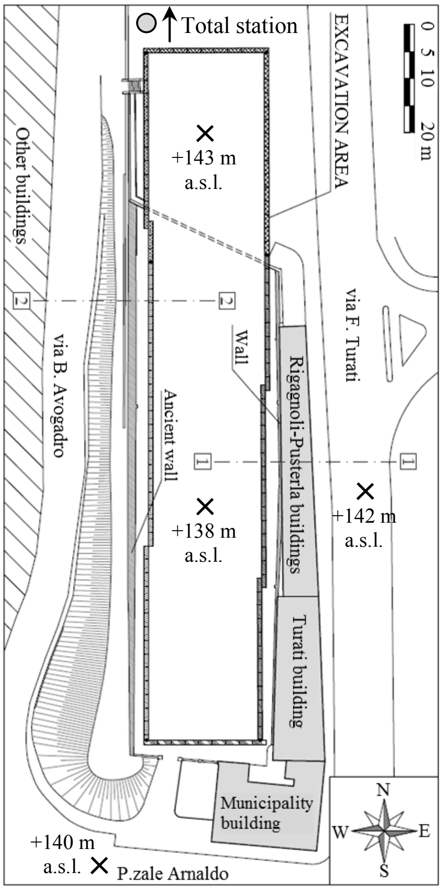

Fig. 3. Plan view of the construction site with nearby buildings.

The Turati building is wider and higher (4 levels, 9-11 m wide) and was partially reconstructed with reinforced concrete after World War II. The Municipality building is a 2 level masonry structure with a large basement, wooden floors and roof, and a portico. The Rigagnoli-Pusterla and a section of the Turati buildings were partially founded on top of the East ancient wall (Fig. 4). At the time of the car park excavation these buildings served as private dwellings and commercial activities but were mostly kept in precarious conditions by the private owners, as confirmed by an official inspection conducted by the car park owner (Brescia Infrastrutture S.r.l., Brescia, Italy), the contractor (Pac S.p.A., Brescia) and the designers. Also, the inspection showed that the Rigagnoli-Pusterla buildings had a very simple structure with two parallel main walls (thickness 0.35-0.60 m, variable with elevation), and wooden floor beams as the only connection between the two façades (with the exception of two stairwells); the wooden floor beams were embedded and/or simply supported by the masonry walls with recesses 0.07-0.10 m deep. The observed damage included cracks in the masonry main walls and partitions, and deterioration of the wooden floor beams.

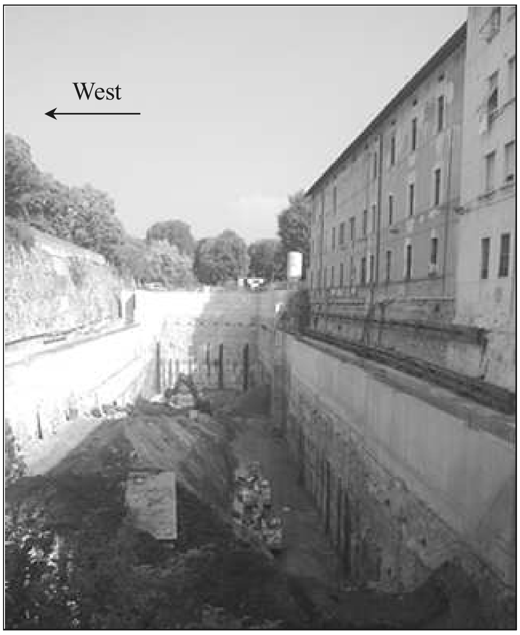

Fig. 4. Excavation in progress, summer-autumn 2011.

Due to the proximity to the Colle Cidneo hill and Monte Maddalena mountain (belonging to the Brescia and Garda Pre-Alps system), the subsoil conditions at the construction site are different from the usual stratigraphy beneath the city of Brescia (Sanzeni et al., 2010) and the soil stratigraphy comprises a superficial layer of made ground over a clayey to sandy soil deposit supporting a local perched water table.
A comprehensive monitoring system was set up before construction that included inclinometers in the East and West diaphragm wall panels, precise leveling, automatic structural monitoring by means of a high-precision total station together with manual and automatic monitoring of existing building cracks. The real-time output of the monitoring system was interpreted in the light of excavation procedures and progress.
To evaluate the behavior of the retaining structure, the safety of adjacent buildings and the effect of the excavation on other nearby structures, a number of two-dimensions finite element numerical analyses were conducted with the commercial program Plaxis 2D (version 2011; Plaxis 2D, 2010, Delft, The Netherlands), using different constitutive laws for the soil. The results of the analyses are presented and compared with some results of the monitoring system.

SOIL CONDITIONS AND EXCAVATION PROCEDURES

The excavation was conducted in a heterogeneous soil deposit that comprised a layer of made ground and clayey to sandy soils with gravel. Soil characterization was performed in 2006 and 2009, mainly before the author's involvement, and consisted of a large number of borings (30) with execution of Standard Penetration Tests, geophysical down-hole tests and measurements of water table. Some horizontal and sub-horizontal borings were executed to define thickness and compositions of the ancient walls and nature of retained soils (below the existing buildings), and exploration trenches were carried out to investigate wall foundations and underground services.
Figure 5 shows a schematic soil profile obtained by the interpretation of borings in the center area of the excavation (near section 1-1 in Fig. 3), and Fig. 6 shows typical SPT results. From ground level (elevation 138 m a.s.l.), a 3 m thick layer of loose sandy made ground (N_{SPT} < 10) is encountered; this layer was removed for environmental reasons before excavation. Below the made ground layer the borings indicated the presence of a heterogeneous deposit of clayey to sandy soils with gravel (N_{SPT} = 10-40). The clayey fraction is more significant between 6.5 and 18 m below g.l. and gradually decreases below elevation 120 m a.s.l. (no laboratory classification tests were executed during the geotechnical investigation campaigns). A number of open tube piezometers were installed to locate and characterize the water table. The 2006 campaign piezometers were merely open wells and gave unreliable water table location, whereas the 2009 piezometers consisted of standpipes located at different depths in the above soil layers. The latest measurements indicated the presence of the city main water table at an elevation of approximately 119-120 m a.s.l. and also the

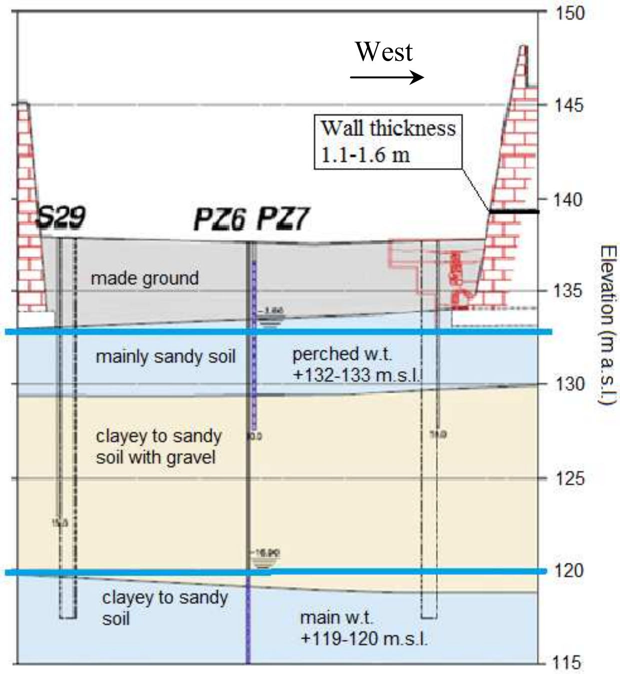

Fig. 5. Simplified soil stratigraphy and measured water tables.

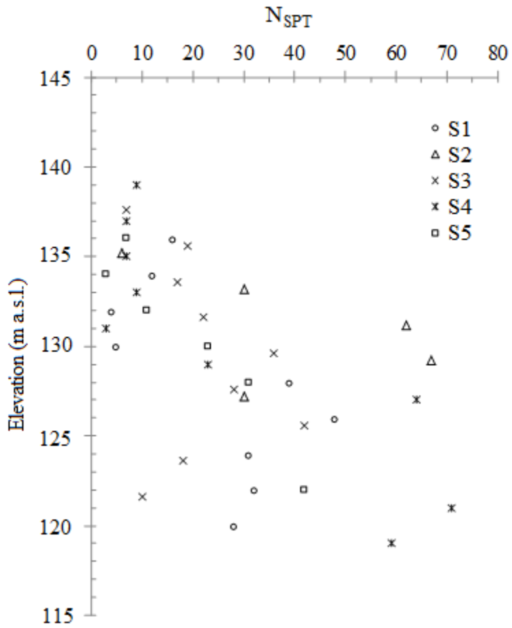

Fig. 6. Typical SPT results at the excavation site.

existence of a perched water table at an elevation 132-133 m a.s.l., confined by the clayey sandy deposit.
The 2009 sub-horizontal borings showed that the existing ancient walls had a heterogeneous composition (stone blocks and masonry, no reinforcement), 1.1-1.6 m variable thickness,

and predominantly granular medium density retained soil. The excavation was supported by concrete diaphragm walls with 4-5 levels of tiebacks (depending on the depth of excavation) with applied pre-loads of 400-500 kN each. Diaphragm wall panels were 20-26 m long and 0.8 m thick, the embedment depth was 9.5 m (elevation 114 m a.s.l.). The ground level was generally at an elevation of 138 m a.s.l. in the main area ("fossa") and at an elevation 142 m a.s.l. in the North section of the construction site (as the nearby road via Turati). The base of the excavation is 123.5 m a.s.l. The foundations of the ancient walls (East and West side) were found at an elevation 133-134 m a.s.l., characterized by the presence of a number of toe spurs that needed to be cut and demolished before the execution of the diaphragm walls (Fig. 7). The excavation was executed in stages progressing from the top to the base (installing subsequent levels of tiebacks), and moving from North to South of the construction site to allow soil transportation and discharge.
The precarious conditions of the buildings on the edge of the excavation (partially founded on the East ancient wall), the composition and thickness of the wall, and the need to demolish part of the wall foundations before the excavation of the diaphragm wall, required the design and construction of a reinforcement system to guarantee the safety of dwellings and commercial activities. The system was installed before the excavation of the diaphragm walls and is described in Fig. 7. It comprised the execution of three levels of temporary tiebacks with a temporary load distribution metal frame (partially visible in Fig. 4), and the installation of a number of micro-piles at the toe of the wall (25 m long, inter-axis distance 1.25 m) together with a connection concrete beam. The intervention was completed with the installation of a system of internal bracing metal cables at each floor level of the most precarious sections of the buildings to connect the masonry main walls (the East and West façades) and stiffen the whole structure.

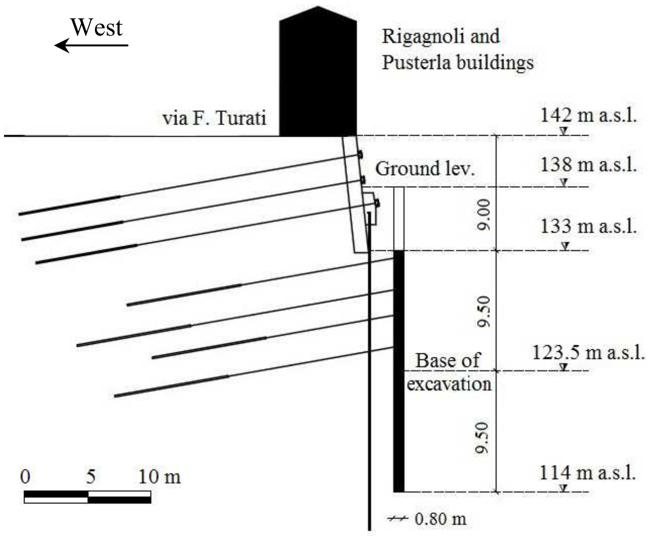

Fig. 7. Cross section of the retaining structure and reinforcement of the existing ancient wall (East side of the construction site).

NUMERICAL ANALYSES

To evaluate the behavior of the retaining structure, the safety of adjacent buildings and the effect of the excavation on other nearby structures, a number of two-dimension finite element numerical analyses were conducted with the commercial code Plaxis 2D (version 2011, Plaxis bv, Delft, The Netherlands), using different constitutive laws for the soil.
The excavation was simulated with a plain-strain, 15-node triangular element model with variable mesh fineness. The model is 80 m wide and 40 m deep and the soil is modeled in two geotechnical units A and B (Unit A: mainly granular; Unit B: mainly cohesive; Fig. 8 represents the East side of the excavation). Due to the heterogeneity of the soil deposits, the behavior of both units was assumed to be drained and therefore no consolidation analysis was performed. The water tables located at 119-120 and 132-133 m a.s.l. were included in the analysis and therefore a seepage calculation was conducted before the excavation stages. The presence of the buildings was conservatively taken into account by applying two lines of 130 kPa loads at the foundation level (142 m a.s.l.), therefore neglecting the stiffness of the structures in elevation. The behavior of the existing wall was assumed to be linear elastic as well as the plate, anchor and grid elements that were adopted to model the diaphragm wall, the tiebacks and the micro-piles.

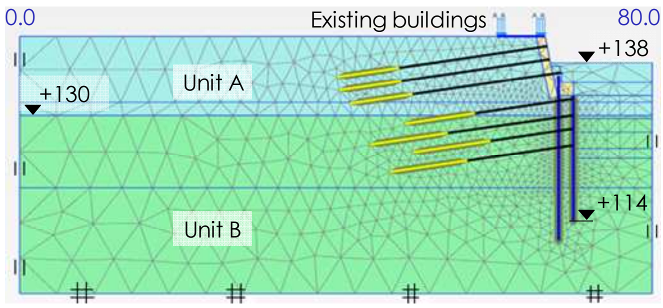

Fig. 8. Model geometry and mesh for numerical finite element analyses.

<u>Constitutive laws and soil parameters</u>

The mechanical behavior of the excavated soil was described using three different constitutive laws available in the code library: 1) simple linear elastic-perfectly plastic with Mohr-Coulomb failure criterion (named "M-C" model); 2) elastic-plastic rate independent model with isotropic hardening and stress-dependent stiffness (named "H-S" model); 3) hardening soil model with small strains stiffness and non-linear dependency on strain amplitude ("H-S small").

The shear resistance parameters were determined based on the authors' experience with local soil deposits and with empirical correlations with results of SPT tests (such as Shioi and Fukuni, 1982; Yoshida et al., 1988). In the Hardening soil constitutive model (H-S) the soil stiffness was estimated from experimental data obtained from interpretation of geophysical down-hole tests performed near the construction site, as described by Rampello and Callisto (2003). In the Hardening soil model with small strains, the small-strain stiffness was estimated from experimental data obtained from geophysical down-hole tests, and the decay of stiffness was described with the relation by Hardin and Drnevich (1972), later modified according to Santos and Correira (2001):

$$\frac{G_s}{G_0} = \frac{1}{1 + 0.385 \cdot \left| \frac{\gamma}{\gamma_{0.7}} \right|} \quad (1)$$

where G_s is the secant shear modulus, G_0 is the initial (very small-strain shear modulus, γ is the current shear strain, and $\gamma_{0.7}$ is the shear strain level at which the secant modulus G_s is reduced to approximately 72% (Plaxis materials model manual, 2011):

$$\gamma_{0.7} \approx \frac{1}{9G_0}\left[2c'\left(1+\cos\left(2\phi'\right)\right) - \sigma'_v\left(1+K_0\right) sen\left(2\phi'\right)\right] \quad (2)$$

where K_0 is earth pressure coefficient at rest, calculated assuming normally consolidated soils.
Table 1 reports the principal geotechnical properties and parameters.

Table 1. Principal geotechnical properties and parameters of the soil units

Unit	γ/γ_{sat} (kN/m^3)	ϕ' (°)	c' (kPa)	E_0 (MPa)	E_{vc} (MPa)	E_{ur}/E_{vc} (-)	$\gamma_{0.7}$ (%)
A	18/20	34	1	95	20	a=3-7	0.33
B	19/21	30	10	280	40	b=7-10	0.04

Poisson's ratio ν = 0.2

Result of numerical analyses

Figure 9 shows the comparison between computed horizontal displacements of the East diaphragm wall, obtained by numerical analyses performed with the three soil models considered for the simulation, and it refers to the final stage of excavation (base of excavation at an elevation of 123.5 m a.s.l.). The chart reports several curves that represent the output of the numerical analyses carried out using the "M-C" model and two pairs of simulations with the "H-S" and "H-S small" models with variable ratios of unloading-reloading and virgin compression Young's moduli E_{ur}/E_{vc} (*a* and *b* in the legend of Fig. 9) and friction reduction coefficient *R* of interface elements between the diaphragm wall and the retained soil. The maximum predicted value of horizontal displacement δx varied in the range of 34-45 mm and appeared to be most significantly affected by the value of the ratio E_{ur}/E_{vc}, whereas the effect of a variation of coefficient R between 0.6 and 0.7 (a reasonable value of wall-soil friction reduction) appeared to be negligible.
The H-S and H-S small models predicted very similar behavior with decreasing horizontal displacement with depth: average displacement at the base of the excavation was 22-26 mm and approximately 2-4 mm at the bottom (114 m a.s.l.). The M-C model response appeared significantly different from the H-S models, with large displacements over the entire diaphragm embedment length (δx = 34 mm at the base of excavation, 22 mm at the bottom). Also, the deformed diaphragm wall obtained with the M-C model simulation presented an unrealistic shape between 0 and -4 m that is not compatible with the stiffness of the 0.8 m concrete beam element. The reasons for this response are uncertain and the result may be explained with a concentration of high shear stresses and possible numerical problems in the region of the model around the toe of the ancient wall on the edge of the excavation. Figure 10 illustrates a detail of the stage construction analysis, performed with the M-C model, with distribution of plastic points in the soil retained by the wall, under the building load, and in the volume of soil below and to the right of the wall base.

Figures 11 and 12 and Table 2 describe the result of numerical analyses in terms of vertical ground movements. Figure 11 reports model vertical displacements of the East side of the excavation, obtained from a simulation performed with the H-S model (final stage of excavation); three measure points are labeled with letters A, B and C and are used in Table 2 to indicate predicted vertical movements of the East and West façade of the Rigagnoli-Pusterla buildings (section 1-1 in Fig. 3), and of the base of the excavation. Figure 11 shows that the ancient wall and the buildings above may experience significant vertical settlements of 30 to 40 mm. The settlement distribution at ground level is affected by the geometry of the soil profile, by the presence of the ancient wall and the applied reinforcement system (tiebacks and micro-piles). Table 2 reports values of vertical movement δy (negative downward), obtained by numerical analyses performed with the three previously described soil constitutive laws, and it refers to the final stage of excavation (base of excavation at an elevation of 123.5 m a.s.l., letters A, B and C as in Fig. 11). The H-S and H-S soil models predicted similar building settlements (measure points A and B: δy-*A* = 23-24mm; δy-*B* = 36-37mm) and uplift of the base of excavation uplift (δy-*C* = 24-25mm). The M-C model response appeared significantly different from the H-S models, with larger building settlements (30-35 mm) and much (up to five times) higher uplift at the base of the excavation. Figure 12 illustrates the comparison between ground settlement profiles near the East side of the construction site (section 1-1 in Fig. 3), obtained at the end of the excavation. As for Fig. 9, the chart reports several curves that represent the output of the numerical analyses performed using the "M-C" model and two pairs of simulations with the "H-S" and "H-S small" models with variable moduli ratios E_{ur}/E_{vc} (*a* and *b* in the legend of Fig. 12), and friction reduction coefficient *R*. Simulations conducted with H-S soil models indicated that the maximum settlement is concentrated below and around the building on the edge of the excavation (average values reported in Table 2), with a distribution that appeared to produce a rotation of the buildings towards East (i.e. away from the excavation).

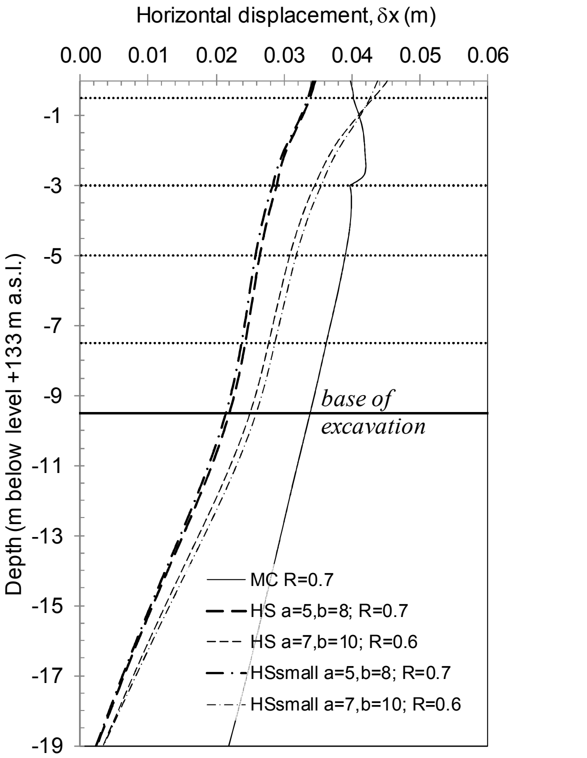

Fig. 9. Diaphragm wall horizontal displacements at the end of the excavation (East side of the construction site), computed with numerical analyses performed with different soil models.

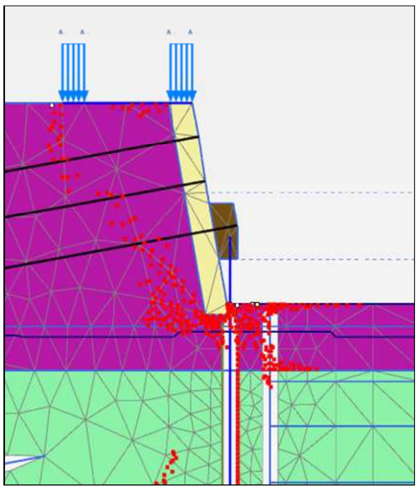

Fig. 10. Detail of a numerical analysis with plastic yielding points within the soil continuum.

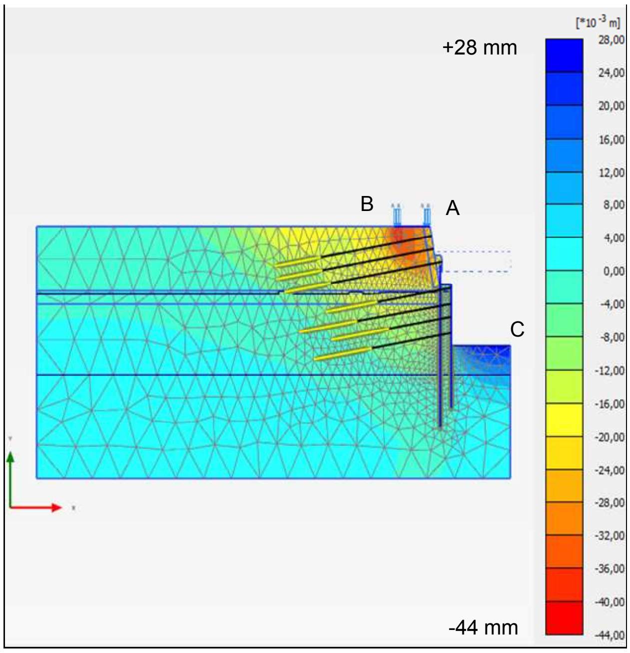

Fig. 11. Model distribution of vertical displacements at the end of the excavation (East side of the construction site), computed with numerical analyses performed with H-S model.

Table 2. Computed vertical ground movements (average values in mm), end of excavation, East side of the site

	A	B	C
M-C	-35	-30	+130
H-S	-23	-37	+24
H-S small strains	-24	-36	+25

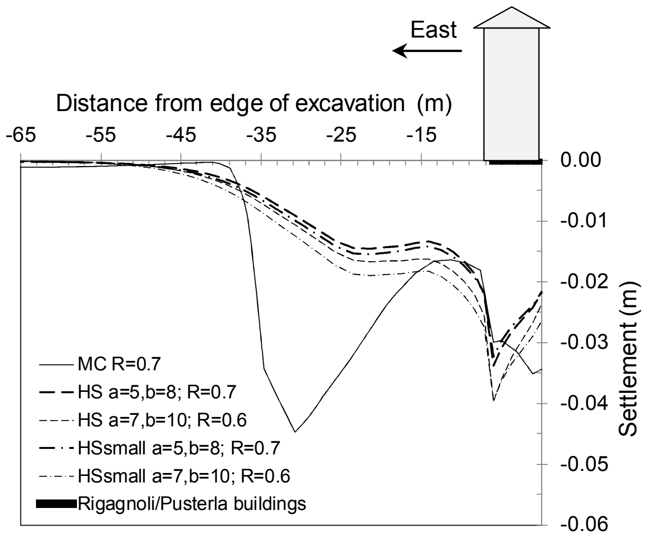

Fig. 12. Vertical displacement distribution at ground level of the East side of the site (end of excavation), computed with numerical analyses performed with different soil models.

The vertical settlement then decreases to values below 1 mm at a distance of 35-40 m from the edge of the excavation. The M-C model response appeared significantly different from the H-S models, producing comparable average building settlement but inducing a rotation in the opposite direction (towards the excavation); at greater distance from the edge of the excavation the soil profile obtained from the analysis conducted with the linear elastic-perfectly plastic model appears to be significantly and un realistically affected by the installation and pull of the tiebacks.

MONITORING SYSTEM AND MEASUREMENTS

A comprehensive monitoring system was set up before construction to control the behavior of the retaining structure and the effect of the excavation on the surrounding buildings.
Special consideration was given by the owner Brescia Infrastrutture Srl (former Brescia Mobilità SpA) and the contractor Pac SpA to a system that was capable of informing in real time on the structural stability of the buildings constructed on the old stone wall on the East edge of the excavation. An automatic high-precision total station was therefore permanently installed near the North-West corner of the construction site (Fig. 3). The apparatus was capable of measuring vertical and horizontal movements of tens of target points regularly distributed on the North and West façades of the Pusterla-Rigagnoli and Turati buildings (and later on the North façade of the Municipality building on the South side of the construction site), as showed in Fig. 13. The façades of the buildings on the East wall were also instrumented with automatic extensometers installed on existing cracks. Automatic measurements were published in real time on a web page available to people involved in the construction.
The system was completed with two load cells installed on wall reinforcement tieback jacks, repeated manual precise leveling around the buildings (to confirm measurement of vertical movements of the West and North façades and to provide measurements of vertical movement of the East and South façades), and with repeated manual measurements of amplitude of internal building cracks (existing and new cracks). A number of vibrometers and electrolevel beam sensors were installed on top of the East wall, at the base of the Pusterla-Rigagnoli buildings, to detect excessive vibrations and differential settlements produced during the demolition of the existing wall spurs and, and during the construction of the diaphragm walls and installation of tiebacks.

Regarding the behavior of the diaphragm walls, the monitoring system included two inclinometers installed in the East and West wall panels (namely "INC PAR 23" and "INC PAR 56", Fig. 14) and two load cells. The inclinometer tubes were installed together with the steel reinforcement before concreting and unfortunately could not be made longer than the diaphragm panels in order to allow measurement of relative wall-soil movements below the diaphragm wall.

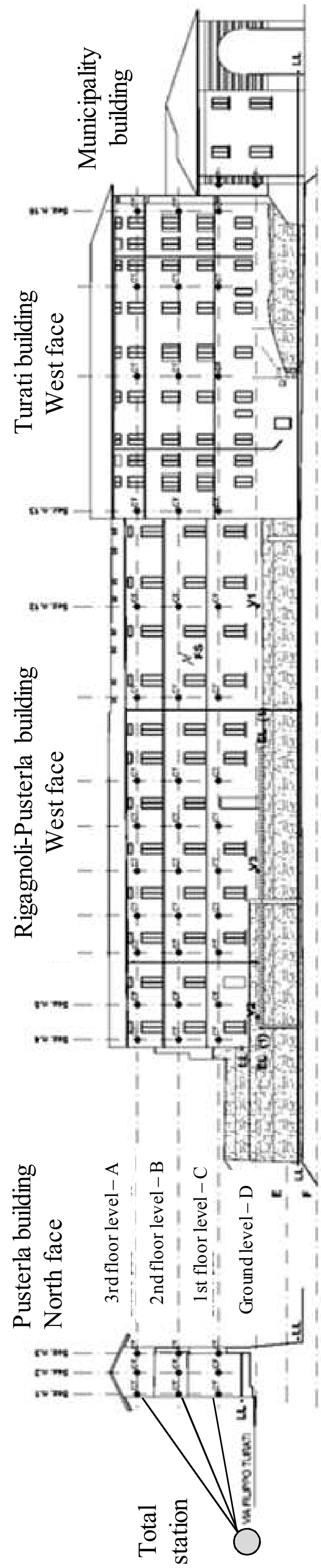

Fig. 13. Total station and target points on buildings near the excavation.

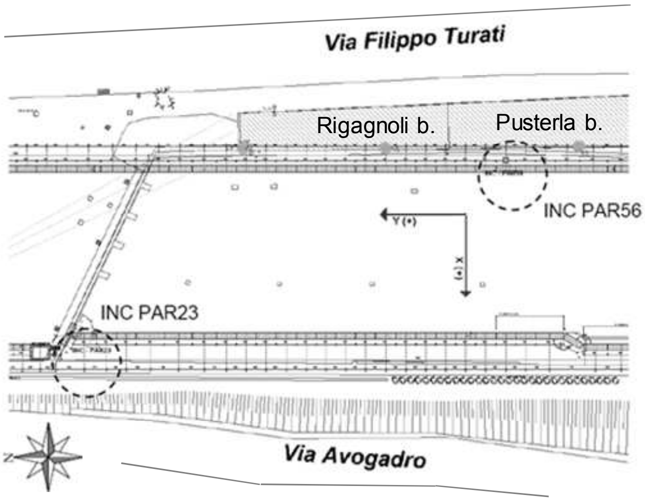

Fig. 14. Location of inclinometers in the East and West diaphragm walls.

Selected measurements and comparison with FE numerical analyses

Figure 15 shows the comparison between computed and measured horizontal displacements of the East diaphragm wall (data interpreted from inclinometer "INC PAR 56" in Fig. 14). Because of the similarity between results of analyses carried out with "H-S" and "H-S small" models, the computed displacement curves are derived from a pair of analyses performed with the H-S model with variable ratios E_{ur}/E_{vc} (*a* and *b* in the legend of Fig. 15); the comparison is presented for the intermediate and final excavation stages and it demonstrates that the numerical model was capable of describing the behavior of the retaining structure well. At the intermediate excavation stage, the recorded maximum horizontal displacement is within the range of computed displacement and the structure response is generally well reproduced by the model: the horizontal displacement gradually reduces to zero towards the bottom of the wall. At the end of the excavation the maximum recorded displacement (top section of the wall) is still well reproduced by the numerical model, but the shape of the deformed wall obtained by the FE analyses is different from the measurement obtained by the inclinometer. The difference between computed and measured wall response may be related to a conservative estimate of the stiffness of the deepest soil layers. Also, the model predicted some horizontal displacement at the bottom of the diaphragm wall that the inclinometer was not capable to capture because of the limitation on the embedment length (inclinometer tube length equal to diaphragm wall).

The limited delay in the response of the retaining structure to the progress of the excavation was generally registered which appeared to confirm the drained behavior of the soil involved in the construction.

Figure 16 illustrates the comparison between simulated settlement profiles and result of interpretation of settlement data (obtained from precise leveling performed around the building and vertical movements recorded by the total station), for section 1-1 across the Rigagnoli-Pusterla buildings (Fig. 3) at the end of the excavation. As for Fig. 15, the chart presents settlement curves obtained from simulations performed by using the H-S soil model with variable ratios E_{ur}/E_{vc} (a and b in Fig. 16 legend) and it demonstrates that the numerical analyses were capable of describing the settlement induced by the excavation in the buildings.

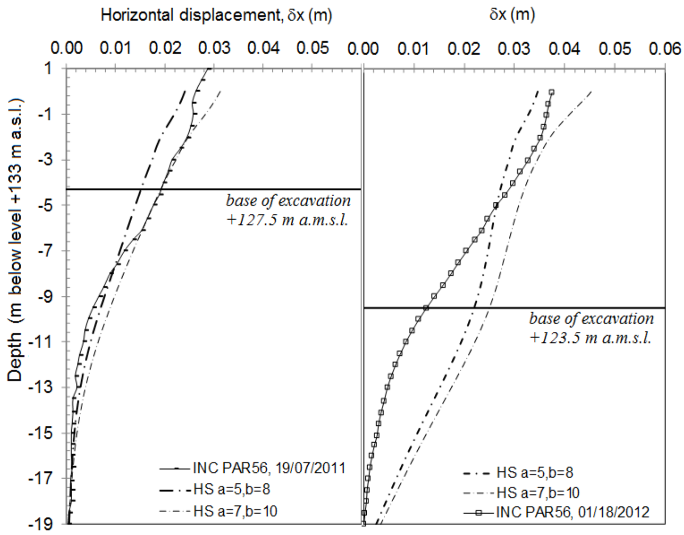

Fig. 15. Comparison between measured and computed horizontal displacements of the East diaphragm wall during and at the end of the excavation.

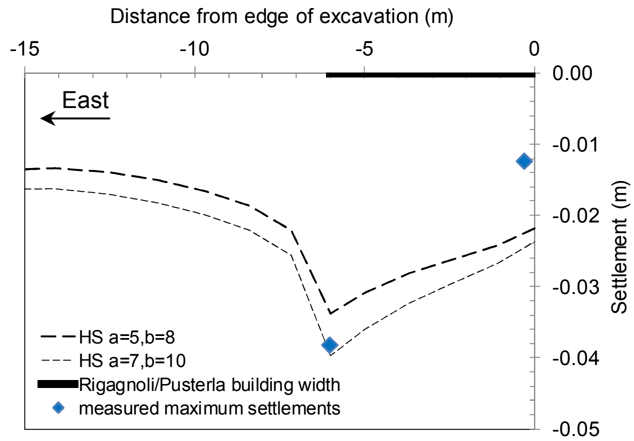

Fig. 16. Comparison between measured (interpretation of total station and geometric leveling measurements) and settlements of the Pusterla/Rigagnoli buildings.

The Rigagnoli-Pusterla buildings appeared to experience a field of settlements that produced a rigid rotation of the edifice towards East, away from the excavation. The East façade recorded settlement was in agreement with the computed vertical movement, whereas the West façade settlement was lower than the computed value (13 mm measured versus 23 mm calculated). The difference between measured and calculated settlements of the West façade may depend upon the modeling of the complex geometry near the edge of the excavation (with an unavoidable degree of uncertainty), the presence and geometrical characteristics of the existing stone wall and interaction with the reinforcement system. Figure 17 shows a typical result of interpretation of the readings obtained from the high precision total station in terms of horizontal displacement of the West façade of the buildings on the East side of the construction site.

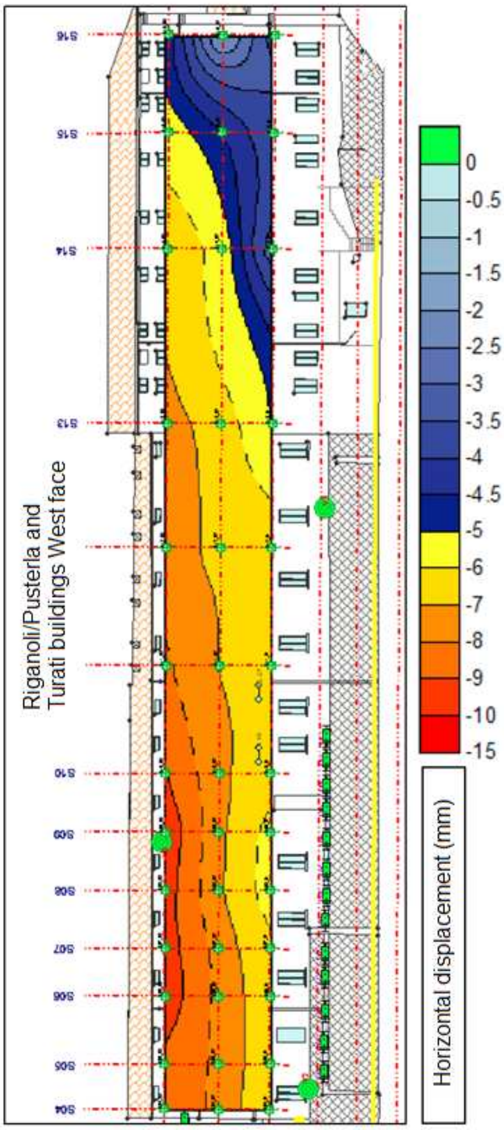

Fig. 17. Measured horizontal displacements of the West face of the Rigagnoli/Pusterla and Turati buildings (interpretation of total station data, negative values indicate a movement toward East, away from the excavation).

The experimental data confirm the rotation of the Rigagnoli-Pusterla structures away from the excavation with a maximum horizontal displacement of 14-15mm at the top and approximately 5 mm at ground level of the West façade; the Turati building had approximately 5 mm of horizontal displacement and generally appeared to be less affected by the excavation due to its wider proportions and structural organization and also because of it being only partially founded on the existing wall.

CONCLUSIONS

The construction of the Piazzale Arnaldo underground car park represented a challenging project for the city of Brescia, requiring the execution of a 130 m long and a 15-20 m deep excavation in a developed urban area in the historic center of the city. The construction site was located between two facing stone and masonry walls (part of the city's ancient defense system), supporting the foundations of some 3-4 storey residential and commercial buildings in precarious equilibrium conditions.

The excavation was conducted in a heterogeneous soil deposit that comprised a layer of made ground and clayey to sandy soils with gravel. The 2009 geotechnical investigation campaign provided extensive information on the soil stratigraphy and heterogeneity, the location and characteristics of perched and phreatic water tables, thickness and compositions of the ancient walls and nature of retained soils.

The excavation of the car park was supported by concrete diaphragm walls with 4-5 levels of tiebacks and was anticipated by the installation of a reinforcement system of the existing East wall that included three levels of temporary tiebacks and a row of micro-piles.

The underground car park project comprised the design of a monitoring system that was extended to include the installation of a permanent automatic high-precision total station, vibrometers, extensometers, electrolevel beam sensors, together with inclinometers in the diaphragm panels and load cells.

To evaluate the behavior of the retaining structure and the effect of the excavation on the nearby buildings, a number of 2D finite element analyses were performed with a commercial program using different constitutive models for the soil. The simulations performed with a linear elastic-perfectly plastic model predicted unrealistic soil response that produced excessive soil uplift at the base of the excavation and unreliable ground movements induced near the edge of the excavation. The result of numerical analyses performed with constitutive laws that include isotropic hardening and dependence of soil stiffness on stress and strain amplitude compared well with measurements obtained by the monitoring system.

Due to the complex ground geometry near the East edge of the construction site and to the effect of the reinforcement system installed before the excavation, part of the nearby buildings experienced a field of settlements that produced an almost rigid rotation in the opposite direction of the excavation. This response could be simulated by FE numerical analyses performed with advanced soil models.

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