Estimated settlements during the Brescia Metrobus tunnel excavation

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ABSTRACT: The paper describes ground subsidence and effect on historic buildings induced by the earth pressure balanced (EPB) shield single tunnel construction of the first line of the Brescia Metrobus (Italy, 2005-2009). The diameter of the shield is 9.15 m, the tunnel is 5.6 km long and excavation was carried out mainly in alluvial gravelly soil deposits. Among the buildings in Brescia the Palazzo della Loggia has been the venue of the city municipality since its construction between the 15th and 16Ith century. The progressive deterioration of the building massive piers and the forthcoming tunnel construction – located 25m from the building and 20m below ground level – required the consolidation of the soil beneath the foundations by means of low-pressure grouting and assessment of building settlements induced by the excavation. A number of finite element numerical simulations were carried out on a calibration-purpose model and preliminary results were compared with measured subsidence obtained from tunnel sections previously constructed. Predicted settlements and settlement distribution at the Loggia section were found in good agreement with movements measured during construction. Among the factors affecting subsidence prediction, particular attention was given to the ground loss during tunnel excavation and the presence of loads due to the building foundations.

1 INTRODUCTION

This paper presents the study carried out to predict subsidence induced by the construction of the tunnel for the first underground metropolitan line in the city of Brescia, Italy, the "Metrobus Brescia" project.

The project comprises the construction of a light-railway line which includes surface and underground cut-and-cover trunks (3.4 km and 3.8 km respectively) and a 5.9 km single tunnel trunk, for a total extent of 13 km. The tunnel was excavated between 2005 and 2009 with a Tunnel Boring Machine equipped with a shield and earth pressure balance technology (EPB-S). The machine is suitable for most soils, can operate below water table and is capable of automatically laying the permanent tunnel lining. The shield diameter is 9.15 m and the permanent lining diameter is 8.85m, consisting of pre-cast reinforced concrete elements of 0.35 m thickness. The excavation was carried out mainly in alluvial gravelly soil deposits and the soil cover is generally in the range of 17-20 m. Face pressure was approximately equal to 130 kPa at the top and 250 kPa at base, backfill injection of the tunnel lining was 4.5 m^3/m and advance rate was 30 mm/min (average measured values).

The tunnel layout lies beneath the historic center of Brescia where a number of buildings of historical and social interest are located (Fig. 1). Among the most important structures, Palazzo della Loggia has been the venue of the city administration since its construction between the 15th and 16th centuries, during the Republic of Venice domination. Figure 2 shows the main East façade of the building, Figure 3 reports a schematic section showing the position of the tunnel and Figure 4 illustrates the tunnel layout and indicates the control system installed around the building area. The two storey building ground plan is 47 m x 30 m and about 30 m high. The structure is made of bricks and stones in elevation, covered by wooden floors and cross vaults, and the dome is shaped like a trough vault covered with lead sheets (Marini and Riva 2003, Giuriani 2007). The foundations are continuous in the West part and are isolated piers in the East part; the foundation level is approximately 5.0 m below ground surface. As documented by Giuriani (2007), the building suffers foundation problems due to partial degradation of the short wooden piles driven for soil improvement purposes in the 15th century (Fig. 2). The tunnel axis is located between 23 m and 25 m from the West side of the building and 24.5 m from ground level.

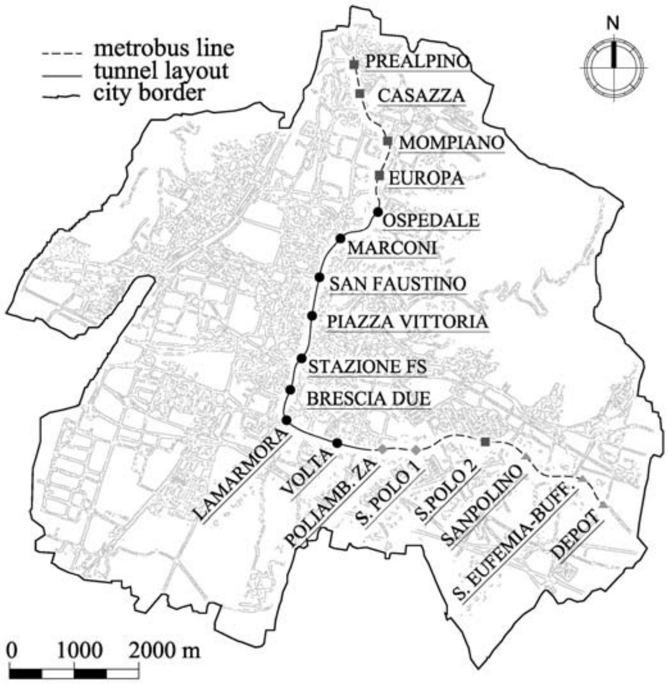

Figure 1. Layout of the Brescia Metrobus light-railway line.

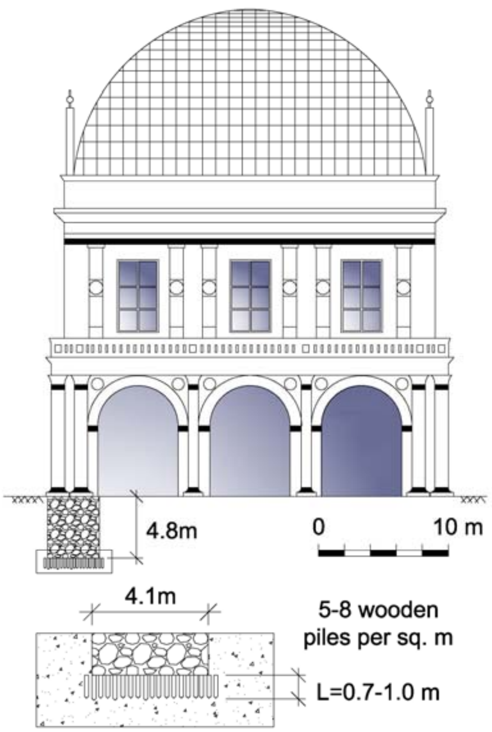

Figure 2. Palazzo della Loggia East façade.

A number of other buildings of different sizes, mainly devoted to residential purposes are located near the palace (Figs. 3-4). The progressive deterioration of the building foundations and the forthcoming tunnel construction required the improvement of soil conditions beneath the foundations and assessment of building settlements induced by the excavation; the improvement of the building foundation was accomplished only one month before the tunnel excavation, by means of low-pressure grout injection of the cavities left by the degraded wooden piles (preparatory studies are described by Giuriani 2007).

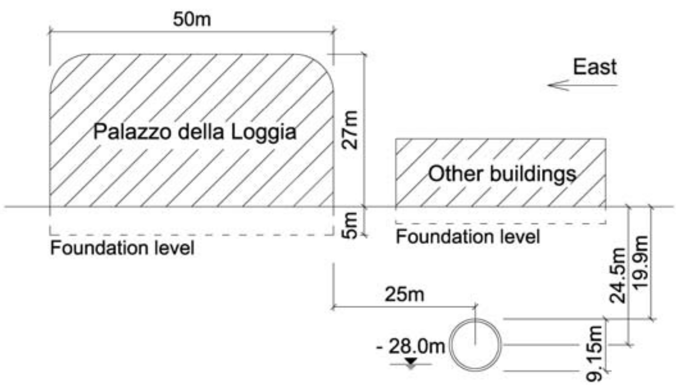

Figure 3. Schematic section at Palazzo della Loggia.

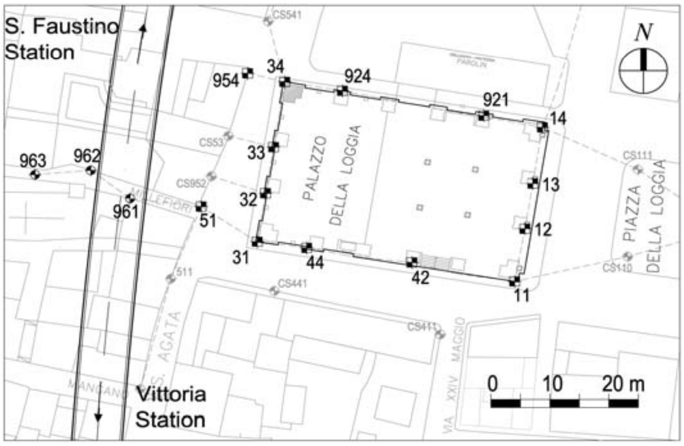

Figure 4. Plan view of Palazzo della Loggia and tunnel location; benchmarks around and connected to the building.

Before the authors' involvement the ground subsidence induced by the Metrobus tunnel was computed using the classic empirical equation proposed by Peck in 1969; a similar approach has been recently used with acceptable results to estimate subsidence induced by the excavation of Line-1 extension in Milan (Antiga and Chiorboli 2007). In this study however the effect of tunneling was analyzed using a plain-strain finite element model.

To predict soil movement and building response, a number of numerical simulations were carried out on a calibration-purpose model and preliminary results were compared with measured settlements obtained from a number of tunnel sections, previously constructed. The model was subsequently used to predict subsidence induced at Palazzo della Loggia section (Zinelli et al. 2010).

2 SOIL PROFILE AND PARAMETERS

2.1 *Site characterization*

The tunnel excavation was carried out mainly in alluvial gravelly soil deposits.

Soil characterization was performed before the authors' involvement and included a comprehensive preliminary desk study with results of pre-existing investigations, and a site investigation campaign.

The latter was carried out in 2003 and 2004 and consisted of 22 borings with execution of Standard Penetration Tests, geophysical down-hole tests, continuous dynamic penetration tests and soil sampling.

Figure 5 shows a schematic soil profile obtained by collecting results of bore-holes and SPT tests along the layout of the tunnel near Palazzo della Loggia. From the ground surface (146.3 m a.s.l.) the following layers are encountered:

– Made ground: mainly cohesionless, medium to loose soil, N_{SPT} = 15-25. The thickness of this layer varies between 1.0 m to 6.0 m along the line layout and is approximately 5.0 m at the Loggia palace section.
– Gravel: well graded cohesionless (45-60% gravel, 20-35% sand, 10-25% silt and clay), medium to dense soil, N_{SPT} = 35-60. Local sandy and clayey soil deposits as well as weak cemented volumes are encountered along the tunnel layout.
– Weathered limestone: this geological unit was encountered only at Palazzo della Loggia section at an elevation below 118-119 m a.s.l. (z = 27-28 m).

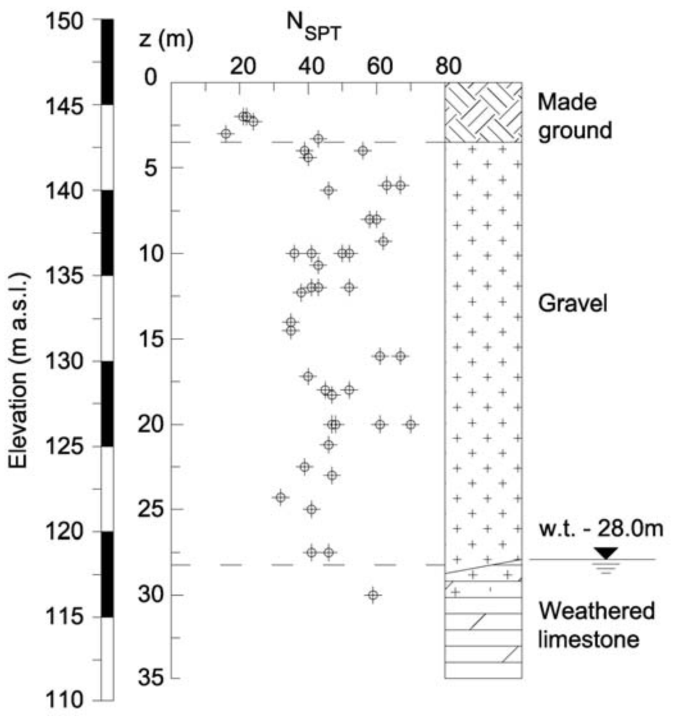

Figure 5. Soil profile at Palazzo della Loggia with SPT test results.

The water table is generally a few meters below the tunnel axis, is located at an elevation 118 m a.s.l. at the Loggia section, and rises above the tunnel axis southward of the city.

2.2 *Constitutive Model and Soil Parameters*

The subsidence induced by tunnel excavation was studied through a number of finite element analyses using the code Plaxis (version 8.6, Delft University) with a plain-strain, 15-node triangular element model. The mechanical behavior of the soil around the excavation was described using the constitutive model Hardening Soil, available in the code library (Shanz et al. 1999). This is an elastic-plastic rate independent model with isotropic hardening and stress-dependent stiffness according to a power law.

The shear resistance parameters were determined based on the authors' experience with local soil deposits and with empirical correlations with results from SPT tests (De Mello 1971, Shioi and Fukuni 1982, Yoshida et al.1988). The soil stiffness was estimated from experimental data obtained from geophysical down-hole tests performed along the tunnel axis, as described by Rampello and Callisto (2003). In the Hardening Soil (H-S) constitutive model the elastic behavior of granular soils is defined by isotropic elasticity through a stress-dependent Young's modulus:

$$E' = E'^{ref}\left(\frac{\sigma'_3}{p^{ref}}\right)^m \qquad (1)$$

where σ'_3 is the minimum principal effective stress, $p^{ref} = 100kPa$ is a reference pressure, $E'^{\,ref}$ and m are model parameters. The Young's modulus $E'^{\,ref}$ has been related to the shear modulus at small strain G_0 obtained from down-hole tests. Figure 6 shows experimental values of G_0 estimated from measurements of the shear wave velocity V_s obtained from several down-hole tests (SB 23, 7, 11, 13 and, more recently, S5). Although there is some scatter in the experimental data, it is possible to identify a unique profile for the alluvial gravel deposit.

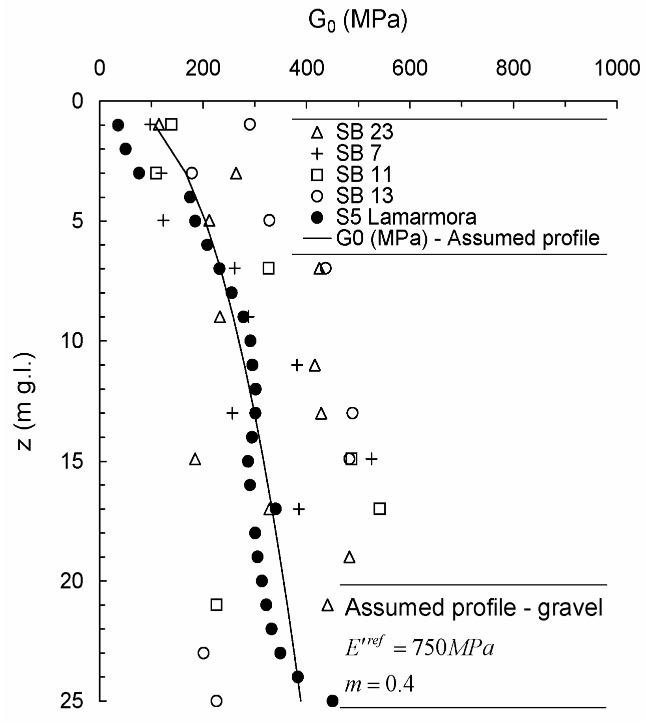

Figure 6. Small strain shear stiffness obtained from down-hole tests and assumed profile for the alluvial soil deposit.

The parameter E'^{ref}_{50}, which identifies the secant stiffness modulus at the reference confining pressure p^{ref}, was estimated assuming the Poisson's ratio $\nu = 0.20\text{-}0.25$ and introducing a ratio $E'^{ref}/ E'^{ref}_{50} = 10\text{-}12$ in relation to the expected soil shear strains.

The mechanical behavior of the superficial layer of made ground and of the altered limestone were described using a perfectly plastic model with Mohr-Coulomb failure criterion (M-C) and a liner elastic model (L-E) respectively.

Table 1 reports soil constitutive models and parameters adopted in the analyses.

Table 1. Soil constitutive models and parameters.

Soil layer	Soil model	φ'	E'^{ref}	m	E'^{ref}_{50}
		°	MPa	-	MPa
Made ground	M-C	32	-	-	25
Gravel	H-S	36	750	0.4	65
Weathered rock	L-E	-	-	-	270

3 EXCAVATION ANALYSIS

3.1 *Evaluation of Ground Loss*

Ground conditions and tunneling method combine to control the ground movements which result from subsurface excavation. To normalize the volume of lost ground with respect to tunnel size, the volume of the settlement basin at the surface, V_s, can be expressed as a percentage of the excavated tunnel volume, V_{exc}. Before the magnitude of ground movements can be predicted it is necessary to estimate the expected ground loss V_s/V_{exc}. This estimate is generally based on case history data and should include an engineering appraisal that takes into account the adopted tunneling method and site conditions (New and O'Reilly 1991). Alternatively, the movement of soil can be estimated through the determination of the so called gap parameter (Lee et al. 1992), which takes into account the ground loss as a function of soil strength and deformation behavior, physical clearance between the excavated diameter and the lining, and workmanship.

In the numerical simulations, that were performed using a plain-strain model, the effect of ground loss during excavation and construction was simulated by applying a contraction to the tunnel lining equal to the ratio V_s/V_{exc}. The numerical value of the applied contraction was estimated with measurement of surface settlements obtained from a number of sections, taken from a trunk of tunnel already constructed, where the soil profile had geotechnical features similar to those encountered nearby Palazzo della Loggia. Figure 7 shows one of the sections (named SCBF 8, located at km7+597) with indication of the soil profile, depth of the water table and tunnel location, while Figure 8 reports vertical settlements taken during tunnel excavation and construction. The most appropriate value of lining contraction was calculated integrating the settlement profiles such as the one reported in Figure 8 and dividing the obtained value by the area of the TBM face. Figure 9 shows the variation of surface vertical settlement along the longitudinal tunnel crown axis with the advance of the TBM machine: the vertical settlement is normalized by the maximum measured value and the machine advance is expressed both in meters and in shield diameters.

The reported data show that vertical displacement ahead of the TBM face amounts to 20-22% of the total and starts showing approximately at a distance equal to 2 diameters; ground loss over the shield consists of some other 25-28% of the total and finally, although backfill grouting of tunnel lining is systematically executed, the settlement registered over the permanent lining is approximately 50% of the total. At the time of the authors' involvement an estimated ground loss 0.45-0.50% represented the most likely value with frequency of occurrence in the range of 86-90% (the frequency of occurrence of ground loss higher than 0.5% was 10-14%).

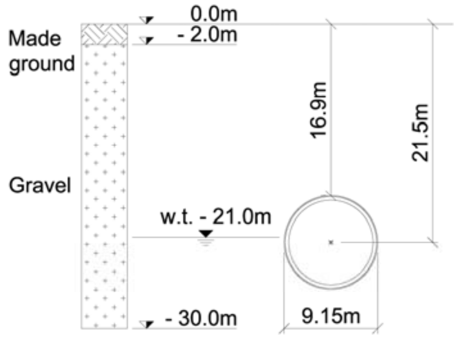

Figure 7. Calibration section SCBF 8 at km 7+597.

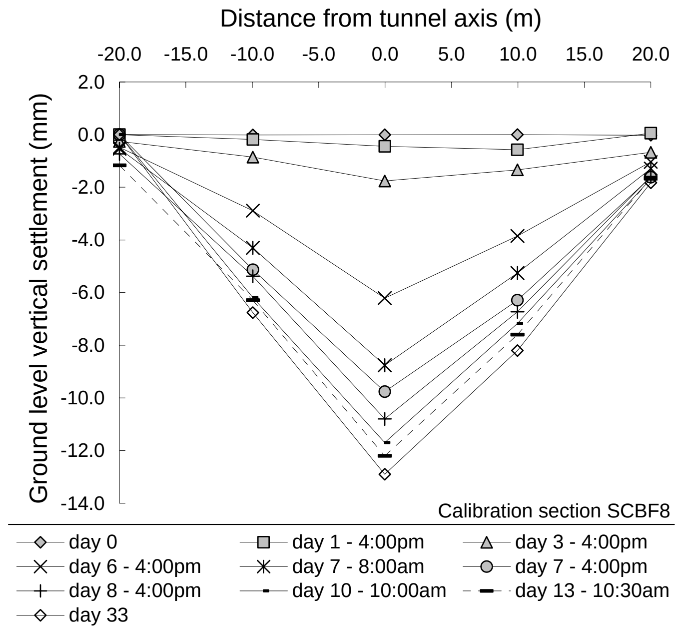

Figure 8. Ground surface settlements at section SCBF8.

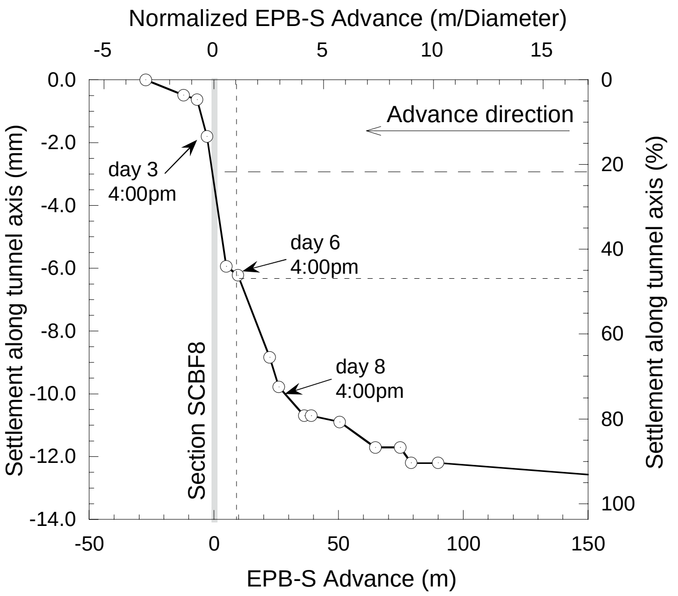

Figure 9. Ground surface settlements with TBM advance.

3.2 *Back analysis and prediction simulations*

The back analysis simulations of tunneling through the reference sections were performed with a numerical model assuming the soil profile illustrated in Figure 7 and absence of buildings at the ground surface. Since the examined sections were symmetric, only one half of the tunnel section was taken into account. The soil behavior was described with constitutive models and parameters reported in Table 1. The lining contraction was applied instantly during one calculation phase – because the permanent lining is constructed immediately after the excavation – and the effect of a higher contraction as a direct consequence of technical problems encountered during excavation was examined.

The prediction analysis at Palazzo della Loggia section (Fig. 3) considered the soil profile represented in Figure 5 and soil constitutive models and parameters reported in Table 1. The presence of the palace and other buildings were conservatively taken into account by applying distributed loads at the foundation levels (150 kPa for Palazzo della Loggia and 50 kPa for the other buildings), therefore neglecting the stiffness of the structures in elevation.

4 COMPARISON OF OBSERVED AND COMPUTED BEHAVIOR, CONCLUSIONS

In Figure 10 the vertical displacements measured at ground level 33 days after the excavation and construction of section SCBF 8 are compared with those computed in the numerical analysis and with those estimated with the empirical equation proposed by Peck in 1969 assuming parameter $K = 0.35$, in agreement with the settlement prediction in the Metrobus project. Measurements and computations show a satisfactory agreement, particularly regarding the amount of settlement above the tunnel axis (with maximum settlement not exceeding 13-14 mm). However, the amplitude of the ground surface affected by subsidence is significantly underestimated by Peck's empirical correlation, due to the numerical value assumed for parameter K, which affects the transverse distance of the point of inflection from the tunnel longitudinal axis, whereas the FEM computed settlement distribution reproduced the real subsidence well. The comparison also demonstrates that the value of applied lining contraction, estimated as the ratio between measured subsidence and the area of the TBM face, V_s/V_{exc}, is suitable for describing the effects of excavation and construction process on ground movement around the tunnel, although the plain-strain numerical model does not allow to appreciate three dimensional and time effects as observed in Figure 9.

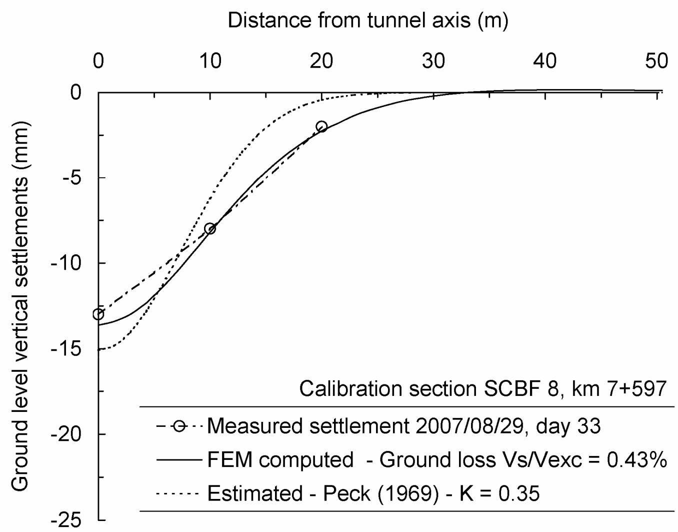

Figure 10. Comparison of observed and computed vertical settlements at section SCBF8.

To investigate the effects of ground loss a number of numerical analyses were performed applying values of lining contraction in the range between 0.3% and 1.5%, as this was intended to represent the highest possible figure in case of technical difficulties during tunneling such as a sudden stop, sudden loss of face pressure, uncontrolled machine tilting and overcutting, or interruption of backfill grouting of tunnel lining. The expected vertical settlement above the tunnel axis greatly increases with applied contraction values higher than 0.7%. Similarly, the amplitude of the subsidence basin increases as a result of the increase in volume of soil subjected to high shear strains near the tunnel lining.

Figure 11 shows the results obtained from the prediction analysis at Palazzo della Loggia in comparison with selected measurements of vertical settlements obtained from the control system in Figure 4. The computed settlement is taken from a horizontal section located at the building foundation level of the numerical model. The measured settlement is obtained from geometric leveling of benchmarks connected to the building and refers mainly to the north and south side of Palazzo della Loggia, plus some other points across the tunnel. The maximum vertical settlement, measured along the West side of the building at points 34 and 31, is less than 2 mm, and decreases to values comparable with the control system error within 10 m from the West side of the building (benchmarks 44 and 924).

The resulting angular distortion is estimated as 1/5000. The settlement prediction compares well with measurements and confirms that, due to the distance of the building from the longitudinal axis of the tunnel, the risk of damage is limited, although the group of buildings directly above the tunnel axis is subjected to more severe effects. The vertical settlements measured above the tunnel show the effect of the buildings stiffness in the sagging zone of the subsidence profile.

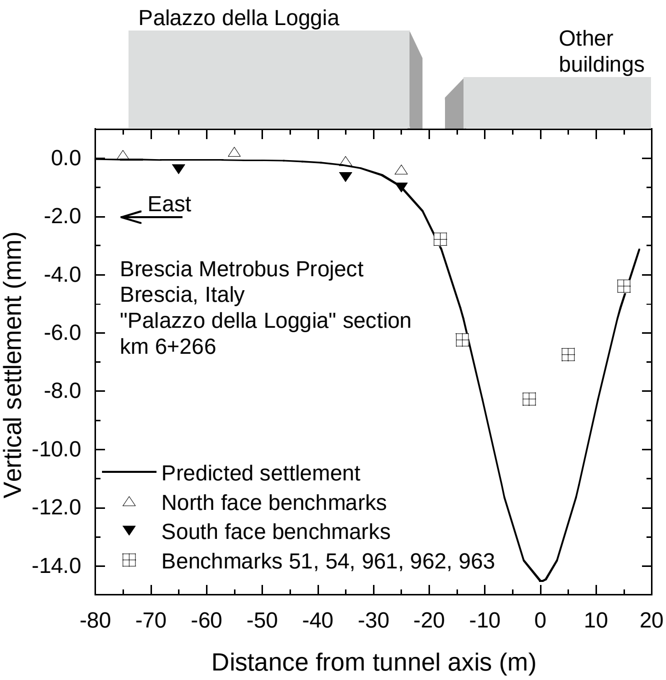

Figure 11. Comparison of observed and predicted vertical settlements at Palazzo della Loggia.

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