Earth Pressure Balance (EPB) TBM excavation: the case of Serravalle tunnel (Terzo Valico dei Giovi - HS/HC Milan-Genoa line)

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ABSTRACT: Earth Pressure Balance (EPB) TBMs are able to deal with the main issues in urban tunnelling that include surface settlement control, maintaining of the existing structures requirements and stability of the excavation face. Tunnels of several kilometres can also include quite different conditions along the tunnel alignment in terms of rock / soil characteristics, groundwater and overburden. In this regard, a proper design of the EPB TBM is a key point in order to avoid unexpected issues during the excavation phase. The paper presents the case of the Serravalle Tunnel, in the Piedmont region (Italy), which is one of the tunnel of the Terzo Valico dei Giovi project, on the High Speed / High Capacity (HS / HC) Milan – Genoa line. The Serravalle tunnel consists in two parallel tubes with a length of approximately 7 km each, of which 6.4 km excavated by 9.73 m diameter EPB TBMs. In particular, the main technical aspects faced from the design stage until the execution phase are presented in the paper, with particular regard to the surface settlements prediction and control during the underpass of a residential and commercial populated area.

KEYWORDS: Earth Pressure Balance (EPB) TBM excavation, Urban Tunnelling, Surface Settlements Prediction and Control, HS/HC Milan-Genoa line, Terzo Valico dei Giovi

1. INTRODUCTION

Demand of long tunnels typically involves dealing with different contexts and conditions during the excavation, including variable ground properties and conditions, groundwater and different levels of overburden. Tunnelling in urban areas also includes the presence of existing structures on the surface, requiring the control of the surface settlements. In these contexts, EPB-TBMs excavation is commonly use, since it is usually able to face with the above-mentioned conditions while ensuring a high level of safety for labour and equipment and reducing the completion time compared to conventional tunnelling.

This paper presents the case of the already completed 7 km Serravalle Tunnel, which is one of the main tunnels included in the Terzo Valico dei Giovi project, on the High Speed / High Capacity (HS / HC) Milan – Genoa line. The study described the main technical aspects of the EPB-TBM excavation addressed at the design stage and during the execution phase in order to deal with the different conditions and issues along the tunnel alignment, with a special focus on the surface settlements prediction and control during the underpass of an urban and highly populated area.

2. TERZO VALICO DEI GIOVI PROJECT

The Terzo Valico dei Giovi is a stretch of the High Speed/High Capacity (HS/HC) railway line Genoa-Milan currently under construction. The project is part of one of the corridors of the Trans-European Transport Network called the Rhine-Alpine Corridor which connects the North Sea ports of Rotterdam and Antwerp to the Mediterranean basin in Genoa (Figure 1).

The line runs along the Genoa-Milan route reaching Tortona, and proceeds along the Genova-Alessandria-Turin route up to Novi Ligure, crossing the provinces of Genoa and Alexandria. The new line will be connected to the South at Voltri and Bivio Fegino through interconnections with the railway facilities at Genoa hub and with dock basins of Voltri and Porto Storico, while connection to the North will be ensured by the existing railway lines Genoa-Torino and Tortona-Piacenza-Milan. The total length of the Terzo Valico dei Giovi will be approximately 53 km, including more than 36 km of tunnels running through the Apennine mountains between Piedmont and Liguria with an overburden up to approximately 600 m (Figure 2). The overall scope of underground works of the line, including dual tube single-track running tunnels, adits and interconnection tunnels, exceeds 90 km, of which more than 30 km excavated by mechanised method (Lunardi et al, 2019).

Figure 1 Rhine-Alpine Corridor

The Terzo Valico dei Giovi underground works includes two main tunnels: the 27 km Valico Tunnel and the 7 km Serravalle Tunnel. The Valico tunnel is currently under excavation by conventional method from the southern entrances and by mechanised method from the northern entrances. The Serravalle Tunnel was almost entirely realized by mechanised method; the excavation of the second tube was completed in April 2021.

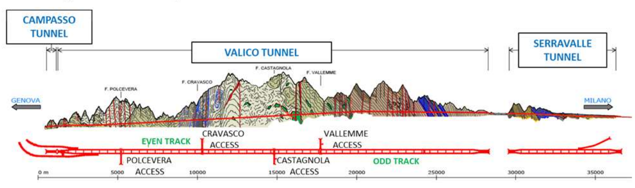

Figure 2 Geological longitudinal section of the tunnelled route section

3. SERRAVALLE TUNNEL

3.1. General information

The Serravalle Tunnel consists in two tubes with a final length of approximatively 7 km, of which 6.4 km was excavated by two 9.73 m diameter EPB-TBMs (one TBM for each tube).

The tunnel includes a first part, from the northern portal (according to the direction of excavation), which across a lowland context where the overburden is less than 20 m; this part includes an urbanized area with a commercial area of the municipality of Serravalle, in a geological context characterized by the presence of a paleo-riverbed and a tectonized area. The second part of the tunnel is in a context of transition between relief and plain, with the overburden increase up to a maximum of 130 m before decreasing with the approach of southern portal.

3.2. Geological context

The tunnel crosses five main sedimentary formations (Figure 3). From the northern portal the tunnel initially across the formation of Lugagnano clays (aL), consisting in clayey silt with sandy and arenitic intercalations. Then, the tunnel enters in the formation of Cassano Spinola (cC) which includes polygenic conglomerates in a sandy matrix with coarse-grained, calcareous and arenaceous, heterogeneously cemented clasts intercalated with the conglomerate strata are silty-sandy, poorly cemented layers with thicknesses on the order of centimetres. Halfway through the tunnel, there is the formation of Gypsiferous-Sulphuriferous (gS) consisting of prevalent pelites, siltstones and fine sandstones with chaotic intercalations of selenitic gypsum blocks, layers of gypsiferous sandstones and blocks of limestones with vacuoles formed by the dissolution of gypsum crystals (Pagani & Cassani, 2016). After that, the tunnel across the formation of Sant'Agata Fossili marls (mA1 – mA2) consisting in calcareous marl interbedded with finely laminated diatomite. The final part of the tunnel involves the formation of Serravalle sandstones (aS) with an alternation of medium coarse sandstone and silty marls.

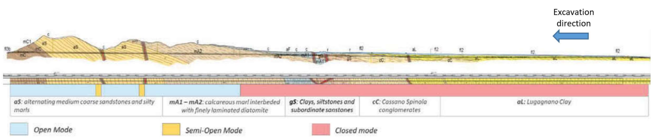

Figure 3 Geological longitudinal section of the Serravalle Tunnel and TBM working modes selected

3.3. Geotechnical parameters

The geotechnical parameters of the geological formations encountered are summarized in Table 1.

Table 1 Geotechnical parameters. γ is the unit weight, k is the ground permeability coefficient, σc is the uniaxial compression strength of the intact rock, σt is the tensile strength of the intact rock, m_i is the Hoek & Brown parameter, GSI is the Geological Strength Index, E_{RM} is the rock mass

Formation	γ [kN/m³]	k [m/s]	σc [MPa]	σt [MPa]	m_i [-]	GSI [-]	E_{RM} [MPa]
aS (outside of the fault)	21÷22	10^{-7}÷ 10^{-9}	5÷10	0.5÷1	10÷ 13	40 ÷60	1200÷ 2800
aS (contact with mA2)	20÷21	10^{-5}÷ 10^{-6}	2÷5	0.5÷1	10÷ 13	40 ÷60	700÷ 1200
aS (fault)	21	10^{-6}÷ 10^{-8}	5÷10	0.5÷1	10÷ 13	30 ÷40	650÷ 1000
mA (outside of the fault)	20	10^{-7}÷ 10^{-10}	2÷5	0.5÷1	5÷9	40 ÷45	700÷ 1200
mA (fault)	19÷20	10^{-7}÷ 10^{-10}	2÷5	0.5÷1	5÷9	30 ÷40	450÷ 650
Formation	**γ [kN/m³]**	**k [m/s]**	**φ [°]**	**c' [kPa]**	-	-	**E_{RM} [MPa]**
gS	19÷20	10^{-7}÷ 10^{-10}	25 ÷35	40 ÷45	-	-	150 ÷350
cC (up to H=30m)	20÷21	10^{-6}÷ 10^{-10}	25 ÷30	10 ÷30	-	-	100 ÷200
Formation	**γ [kN/m³]**	**k [m/s]**	**φ [°]**	**c' [kPa]**	**Cu [kPa]**	**OCR [-]**	**E_{RM} [MPa]**
aL (up to H=20m)	19.5	10^{-8}÷ 10^{-10}	21 ÷27	10 ÷30	100 ÷500	3÷ 2	20 ÷50
aL (over H=20m)	20.5	10^{-8}÷ 10^{-10}	27 ÷32	30 ÷60	500	1.5÷ 1	50 ÷60

3.4. EPB-TBM specifications

The technical characteristics of the EPB-TBMs employed for the excavation of the Serravalle Tunnel are summarized in Table 2. Then tunnel lining with an outer diameter of 9.4 m and 0.4 m of thickness consists in 6 segments plus 1 key segments (Figure 4).

Table 2 Main technical characteristics of the EPB-TBMs

Parameter	Unit	Value
Nominal excavation diameter	[m]	9.73
Cutterhead power	[kW]	3850
Nominal torque	[kNm]	17988
Maximum breakout torque	[kNm]	24103
Cutterhead rotational speed	[rpm]	0 - 3.2
Maximum thrust force	[kN]	128692
Bulkhead pressure (max. design value)	[bar]	5
Length of screw conveyor	[m]	18.5
Rotational speed of screw conveyor	[rpm]	0 - 24.4
Number of disc cutters (including no. 10 gauge cutter)	[-]	16 double 22 single
Diameter of disc cutters	[inches]	17
Shield length	[m]	10.2
Segmental lining outer diameter	[m]	9.4
Segmental lining inner diameter	[m]	8.6
Segmental lining length	[m]	1.8
Segmental lining thickness	[m]	0.4
Back-up length	[m]	90 m

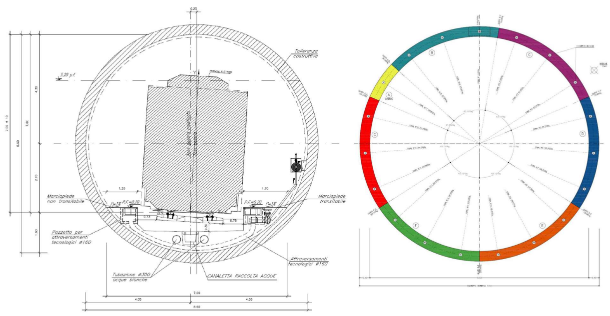

Figure 4 Scheme of tunnel section and segmental lining elements

4. MAIN TECHNICAL ASPECTS FACED AT THE DESIGN STAGE

As noted before, the Serravalle Tunnel across two main sectors in terms of geological and geomorphological contexts, i.e. the lowland sector and the hilly sector, with the presence of varying conditions along the tunnel alignment, involving several issues to face during the design stage.

The main issues in the lowland sector include:

- Stability problems with low overburden and/or low ground properties;
- Underpass of water surfaces and a paleo-riverbed area;
- Underpass of residential and commercial buildings with overburden less than 20 m;
- Underpass of area with overburden not sufficient (< 5 m) to allow a tunnel boring advance.

The hilly sector involves the presence of:

- Potential high groundwater level;
- Mixed face;

- Underpass of landslide site;
- Abrasive ground conditions.

Tectonized areas are present in both sectors. The water inflows during the excavation phase (transitory conditions) and after the excavation (stationary conditions) need to be taken into account. Clogging problems are also possible during the machine excavation.

In order to face with the above-mentioned heterogeneous conditions and issues the EPB-TBMs employed were carefully selected, able to work in:

- closed mode with face support pressure in unstable/not self-supporting ground and/or when groundwater pressure is to be balanced, controlling surface settlements;
- open mode (without face pressure) in stable grounds without needing to balance groundwater pressure.

These machines are quite flexible, allowing the adaptation the working modes on the basis of different conditions encountered with relatively short conversion times. The working modes selected along the tunnel alignment are shown in Figure 3.

The closed mode was chosen for the excavation of the entire first sector in lowland area, in order to ensure the tunnel stability, minimize the surface settlement in urbanized area and control the hydrogeological equilibrium of the context crossed. For this purpose, the face support pressure was designed for balancing the horizontal ground pressure and the potential groundwater pressure, resulting in values of maximum face support pressure selected is typically within 3.5 bar on the crown.

The open mode was selected for driving in the second sector (hilly context) where the stability analysis show face conditions from stable to stable in short term, according to ADECO-RS approach (Lunardi, 2008). However, the semi-open mode was conservatively recommended in the tectonized areas of this sector, as an additional guarantee to control the tunnel stability and manage the potential water inflows. This "transition" working mode involves maintaining a half-filled excavation chamber with conditioned muck and is indicated in presence of self-supporting stable ground with moderate water income, giving pressure only by compressed air to stop water ingress (Bilgin et al, 2014).

With regard to the two areas in the lowland sector with inadequate overburden (arch effect does not occur), a specific solution was designed, consisting in the creation of "artificial overburden" by means of a "prosthesis" of improved ground created over the crowns of tunnels before excavation (Figure 5). This solution allows to avoid the construction of artificial tunnels that involves deep incisions into the slopes to be crossed with consequent problems of safety and environmental impact.

The TBM cutterhead specifications were selected in order to deal with every type of condition expected, including exceptional scenario. The thrust system requirements were defined according to Maidl et al (1995); the maximum value required takes into account the worst possible scenario of the machine restart after a downtime, i.e. application of the maximum pressure at the face and on the shield with high ground-shield friction and high level of force provided to the cutting tools.

The front of the TBM cutterhead was equipped to allow the use of different cutting tools layout, varying the number of disc cutter and/or scraper tools employed on the basis of the ground boreability encountered. Boreability and abrasivity conditions were detailed analysed at the design stage by means of a series of laboratory tests carried out for each geological formation crossed, providing specific indexes such as DRI, CLI and CAI (e.g. Dahl et al, 2012). The analysis showed an increase of the ground abrasivity in the formations of the hilly sector. Clogging potential risk was evaluated according to Thewes & Burger (2004), resulting in a medium-high clogging risk expected in the mA, gS and aL formations. Clogging and abrasivity issues are addressed through a proper muck conditioning together with the adaptation of the machine operating level. Local decreases of ground boreability are overcome by the above-mentioned cutterhead layout arrangements.

TBM was designed to be equipped for performing probe drilling (with or without core recovery) in order to investigate the ground and water conditions ahead of the tunnel face. The drilling rig can be located in the area at the end of the tail shield and the drilling rods can pass through dedicated ports within the shield. In case of exceptional scenario, the same passages on the shield could be also used to carry out interventions of ground properties and/or permeability improvement.

Depending on ground hydraulic behaviour, specific drainage interventions were designed in order to maintain a long-term hydraulic load conditions consistent with the design capacity of the tunnel lining.

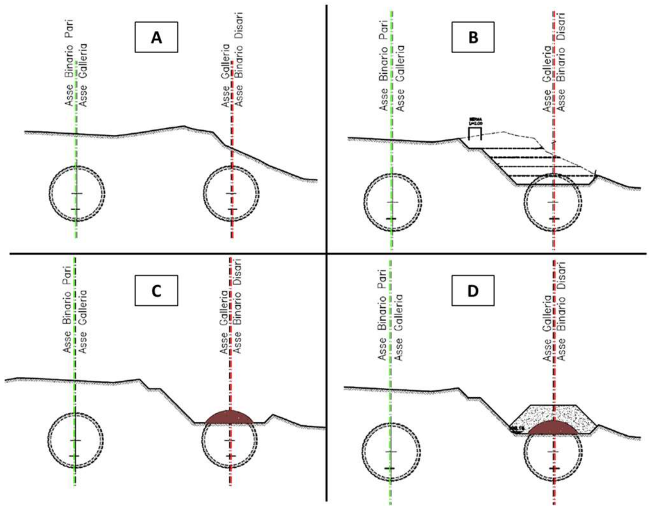

Figure 5 Phases of ground prosthesis creation. A: initial stage; B: removing upper ground layers; C: filling over the crown area with compacted ground; D: second filling with improved ground

5. UNDERPASS OF SERRAVALLE RESIDENTIAL AND COMMERCIAL AREA

5.1. General information

For about 1.2 km the tunnel underpasses the Serravalle residential and commercial area (Figure 6), crossing the gS and cC geolgolical formations, with a surface layer that generally consists of fluvial deposits (fl2). The overburden are between 18 and 25 m. The area is highly populated, including several residential and commercial buildings, streets and parking spaces.

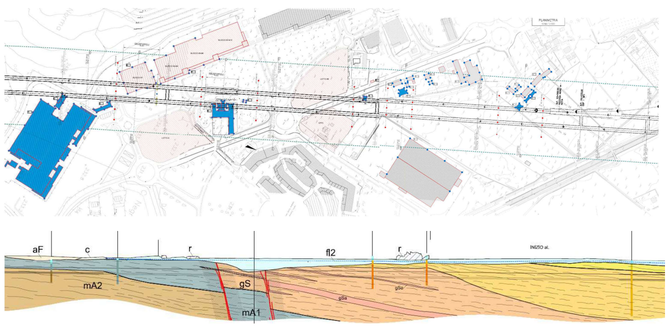

Figure 6 Plan and geological profile of the Serravalle residential and commercial area

5.2. Design stage

At the design stage a detailed analysis for predicting surface settlements and assessing the risk of building damage was performed. The procedure adopted consisted in 3 main phases of analysis.

5.2.1. Phase 1

This phase consisted in the assessment of the overall ground movements induced by the TBM excavation in "green field" conditions according to the most commonly used empirical formulations (e.g. Peck, 1969; Attwell & Woodman 1982, O'Reilly e

New 1982, Rankin, 1988). The analysis were performed for three levels of volume loss (0.5%, 1% and 2%), i.e. volume of the ground loss after excavation, and different ranges of overburden and K parameters.

5.2.2. Phase 2

This phase has included the evaluation of the potential buildings category of damage (C.D.) induced by the excavation of both tubes according to two main classification methods: Rankin (1988) and Boscardin & Cordin (1989) (e.g. Mair et al, 1996).

For each building located in the area (within 75 m of the tubes) the ground movements were assessed for three levels of volume loss (0.5%, 1% and 2%) according to the phase 1, fixing overburden and K parameter (depending on the ground formation). On this basis, maximum tensile strain (ε_{max}) induced in the building was calculated according to Mair et al (1996) allowing the assessment of the expected category of damage based on Boscardin & Cordin (1989) classification method (modified from Burland et al, 1977), which involves six different damage categories from 0 (negligible) to 5 (very severe). According to Burland et al. (1977), each building was idealised as a beam with span L and height H (depending on actual building geometry) deforming under a central point load. Maximum vertical settlement (S_{max}) and angular distortion (β_{max}) were also assessed in order to predict the potential category of damage according to the classification proposed by Rankin (1988), which includes four different categories depending on the degree of severity from 1 (negligible) to 4 (high).

The results of the building potential damage category assessment is summarised in Figure 7 and Figure 8. The analysis related to the Serravalle residential and commercial area included a total of twenty-two buildings identified by ID numbers between 25 to 46. As noted by the authors of the classification methods employed in the analysis (e.g. Boscardin & Cordin, 1989; Mair et al, 1996), the approach used is usually very conservative, since the inherent stiffness of the building tends to reduce both the deflection ratio and the horizontal strains. Therefore, the buildings category of damage estimated in this study should be understood as the maximum potential degree of damage expected for a certain value of volume loss.

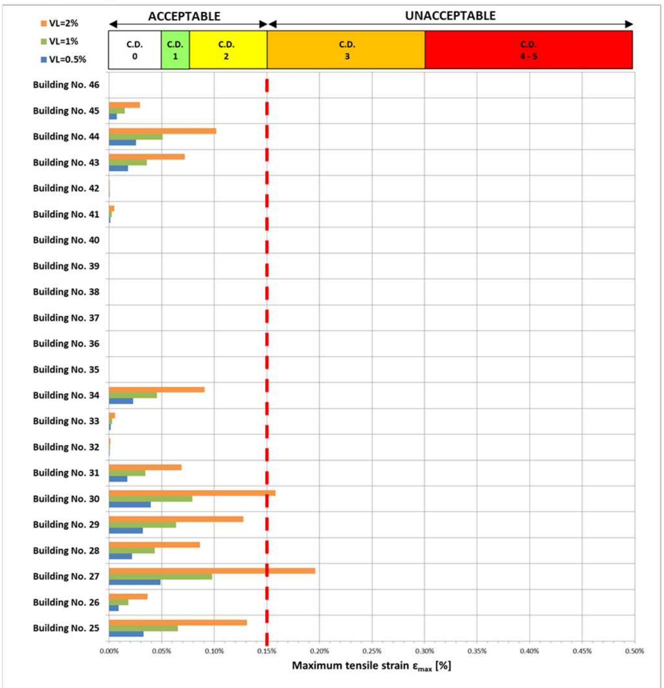

Figure 7 Assessment of the potential building category of damage according to Boscardin & Cordin (1989) classification method (depending on the maximum tensile strain). VL is the volume loss; C.D. is the category of damage

Figure 8 Assessment of the potential building category of damage according to Rankin (1988) classification method, depending on the maximum vertical settlement (above) and angular distortion (below). VL is the volume loss; C.D. is the category of damage

5.2.3. Phase 3

The final phase consisted in the selection of the acceptable category of damage (C.D.) for each building and the definition of the design requirements and mitigation measures. The acceptable degree of damage was defined depending on construction type, designated use and current conditions of each building. On this basis, a C.D. 2 was considered acceptable for all the buildings affected by the excavation.

As shown in Figure 7 and Figure 8, according to the analysis performed all the buildings result in an acceptable category of damage providing a volume loss within 1%, whereas an acceptable category of damage is not guaranteed for 6 buildings in case of a volume of loss of 2%. In this case, the exceeding of an acceptable damage is basically caused by the values of the maximum vertical settlement estimated for the buildings located in correspondence with the tunnel axis. On the other hand, acceptable conditions are normally ensured (except for 3 buildings) in terms of building tensile strain and angular distortion even with a volume loss of 2%.

The acceptable condition considered as design requirement in this study has conservatively involved a category of damage within 2 for both classification methods employed. On this basis, a closed excavation mode with a rigid control of the ground deformations avoiding machine stoppages in correspondence with more susceptible buildings (i.e. No. 25-27-29-30-31-43) was required during the underpass of the Serravalle residential and commercial area in order to limit the volume loss within 1% (threshold design value).

An intensive monitoring plan was designed in order to control the effect of the tunnel excavation before, during and after the passage of the machine. Specific threshold limits were defined for the parameters monitored in order to perform an efficient control of the tunnel excavation and to properly evaluate the possible countermeasures in case of threshold exceeding. Two main threshold limits were defined, called the attention threshold and the alarm threshold. The attention threshold was selected as a proportion (e.g. 60% or 70%) of the design threshold value of the parameters monitored; the exceeding of this limit involves the increase of the frequency of the measurements allowing a close monitoring of the progress of the parameters investigated. The alarm threshold was fixed close to the threshold design value (e.g. 90%); overcoming this limit involves a specific evaluation of the unexpected phenomenon occurred, with Work Management involvement, and assessment of proper countermeasures in order to restore acceptable conditions. With regard to the parameters involved in the control of the buildings potential damage, the following threshold limits were selected.

5.3. Execution phase

Before the passage of the TBM the extensive monitoring system required at the designed stage was arranged on the surface in the Serravalle residential and commercial area, including the use of monitoring instrumentation to measure the ground settlement on the surface and the building movements. The knowledge of the ground settlements with particular regard to the areas before the building underpass was employed to know in advance the level of the volume loss induced by the excavation. The measure of the building displacement during and after the TBM passage was used to verify the building category of damage.

Figure 9 and Figure 10 show an overall picture of the levels of volume loss and buildings category of damage registered after the excavation on the basis of the results of the monitoring system employed. The acceptable conditions required at the design stage were ensured without the use of additional mitigation measures. The area showed ground movements expected for a volume loss within 1%. All the buildings are included in the C.D. 1, except four buildings (i.e. No. 27, 28, 29, 31) that fall within C.D. 2, which is included in the design requirements. It should be noted that such buildings fall in the C.D. 2 only for the maximum vertical settlement (S_{max}) registered, whereas they shows values of angular distortion (β_{max}) and tensile strain (ε_{max}) within the upper limit of C.D. 1. This is in accordance with the background provided by the past projects (e.g. Boscardin & Cordin, 1989; Mair et al, 1996), proving as the approach employed at the design stage is conservative.

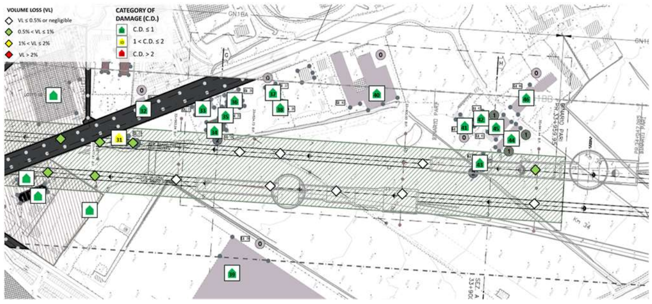

Figure 9 Plan of the first part of the Serravalle residential and commercial area with levels of volume loss and buildings category of damage achieved after the excavation

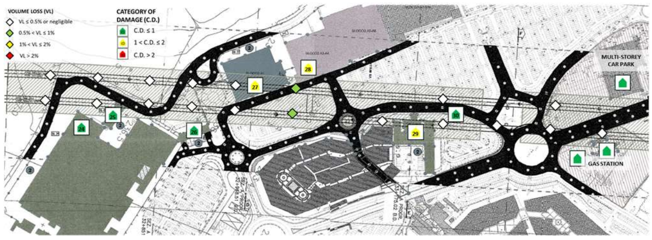

Figure 10 Plan of the second part of the Serravalle residential and commercial area with levels of volume loss and buildings category of damage achieved after the excavation

5.3.1. Underpass of the building No. 29

One of the most sensitive element of the TBM excavation under the Serravalle residential and commercial area is the building No. 29. Figure 11 shows some details about this building, which consists in three floors above grounds and one underground floor. In terms of supporting structures, the building can be divided in three main units, including framed units with steel structure (A) or reinforced concrete (B) and a mixed structure unit (C) made of masonry and reinforced concrete. The geological section includes three different geological formations over the tunnel crown, with gS at the tunnel level, cC in the middle and fluvial deposits on the surface. The overburden is around 19 m. The building was equipped with a monitoring system that includes topographic target, velocimeter and biaxial tiltmeter; the surrounding area was also equipped with topographic target on the surface.

Figure 11 Plan, geological section and pictures of building No.29

The evolution of the settlements during the advance of the TBM is reported in Figure 12. Maximum final settlements registered on the buildings after the passage of the TBM are between 8 and 13 mm, expected on the surface for a volume loss between 0.3% and 0.5%. The surface settlements registered in the surrounding area are attributable to a volume loss within 0.6% (i.e. attention threshold).

On the basis of the result provided by the monitoring system, after the excavation the building No. 29 falls in C.D. 2 according to Rankin (1988) classification method due to maximum vertical settlements (S_{max}) exceeding 1 cm, even if they are well within the attention threshold value required. On the other hand, the values of angular distortion (β_{max}) registered are well within the upper limit of C.D. 1 (β<1/500) and the attention threshold value required at the design stage (β<1/285).

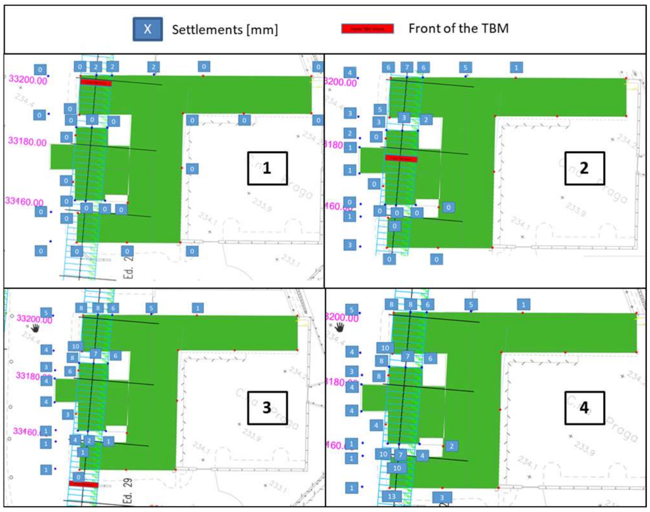

Figure 12 Settlements developments at four different instants during the progress of the excavation under the building No. 29. Instant 1: start of the underpass; Instants 2: front of the TBM in the middle of the building underpass; Instant 3: front of the TBM just outside the building underpass; Instant 4: TBM far from the building underpass

6. CONCLUSION

The paper has presented the case of the twin tubes 7 km Serravalle Tunnel, excavated by two 9.73 diameter EPB-TBMs in a context that includes variable ground characteristics and overburden, groundwater and the presence of susceptible existing structures on the surface to be preserved.

The main issues faced at the design stage and the mitigation measures provided were described, including machine requirements, selection of the excavation mode along the tunnel alignment and definition of the face support pressure.

A special focus was on the underpass of a residential and commercial area in a lowland sector. The approaches employed for the prediction of the surface settlements and expected building category of damage were discussed. The design requirements and measures in order to maintain the acceptable conditions required for the buildings preservation were explained. The on-site response observed in the area during the excavation phase was finally presented, proving as the measures and requirements provided at the design stage has allowed to respect the acceptable conditions required for the existing structures on the surface.

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