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Observed and computed performance of mechanised tunnelling in the historical centre of Rome[☆]

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ABSTRACT

The project of the Line C of the Rome underground involved the use of two earth-pressure balance (EPB) tunnelling boring machines (TBMs) to excavate the twin tunnels in the city centre, having the priority of minimising potentially detrimental interferences with existing monuments and historical buildings of invaluable historical value. This gave rise to a massive monitoring program to evaluate the effects of the mechanised tunnelling on the nearby monuments, providing the opportunity for field investigation of ground response.

This paper presents and discusses the monitoring results collected from a greenfield test site located close to the Basilica of San Giovanni and instrumented with extensive surface and subsurface instrumentation. Volume losses observed at the test site were low, being about 0.3 % and 0.45 % for the first and second tunnel drives, respectively, the higher volume loss resulting from plastic strains induced by the excavation of the first tunnel. This also induced non-symmetrical settlement troughs and cumulative subsurface horizontal displacements directed towards the second tunnel.

The paper also presents a 3D numerical analysis to describe the ground response to tunnels excavation, simulating the progressive advance of their faces. The analysis results were found to satisfactorily reproduce the observed surface and subsurface ground movements induced by tunnelling. Comparing the observed and computed displacement fields helped to identify some uncertainties in the field measurements, demonstrating how numerical analyses can be used to predict the outcome of complex geotechnical problems, provided a suitable constitutive model is carefully calibrated using laboratory and site investigations.

1. Introduction

Line C of the Rome underground is the third line of the city: once fully operative, it will ease the connection with the South-eastern suburbs, thus reducing the traffic congestion in the city centre. The central part of the route (contract T3) represented an impressive engineering challenge, in that it has been excavated in the historical city centre, interfering with many existing monuments of Roman Age, of invaluable archaeological and historical value. To minimise the ground movement during construction, the twin-bore tunnels were excavated using two earth pressure balance machines (EPBMs) from the shield insertion pit, close to San Giovanni's square, to the future Venezia station.

Much has been learnt about the complex interactions between the ground and the EPBMs, mainly thanks to outstanding case histories.

Most of them refer to tunnelling in London Clay: worth mentioning are those reported by Attewell and Farmer (1974), Barratt and Tyler (1976), New and Bowers (1994), Dean and Bassett (1995), Macklin and Field (1998), Nyren (1998), Van der Berg et al. (2003), Clayton et al. (2006), Standing and Selemetas (2013), Wan et al. (2017a,b). However, there is still a limited number of comprehensive case histories also covering the subsurface ground response to EPBM tunnelling.

Despite the remarkable research and recent improvements in EPBM excavation techniques, tunnelling still causes the surrounding soil to move. Therefore, the accurate prediction of ground settlements remains one of the most challenging aspects of geotechnical engineering. In this context, field monitoring plays a fundamental role, in that it provides a deeper understanding of the observed behaviour and constitutes an invaluable benchmark to calibrate both numerical and empirical

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approaches (Avgerinos et al., 2018).

In the light of the above, the construction of the Line C of the Rome underground promoted a massive monitoring program to evaluate the effects of the construction of the line on the historical heritage and eventually suggested the adoption of advanced mitigation techniques for any potential detrimental effect.

This paper presents the results of the greenfield field monitoring carried out during the construction of the twin-bore tunnels of Line C, at an instrumented research site. Comprehensive field monitoring was undertaken to provide data on the surface and subsurface ground movements induced by mechanised tunnelling and to assess the ability of manageable numerical procedures to reproduce the observed behaviour. The research site also included an embedded barrier to test its effectiveness at mitigating displacements induced by tunnelling. The results of this test are presented in companion papers (e.g. Masini and Rampello, 2021).

Previous studies have made use of this case history, focusing primarily on specific aspects of the ground response or the effectiveness of the embedded barrier. These studies have mostly referenced the condition of fully-excavated tunnels. Fantera et al. (2016) used the site geometry and soil parameters, together with the semi-empirical surface settlement trough computed for a given volume loss, to calibrate 2D and 3D numerical models with planar geometry, and referencing a single fully-excavated tunnel. They investigated the effect of the pile-wall barrier. Similarly, Masini and Rampello (2021) and Masini et al. (2021b) developed 3D numerical models to investigate the effect of the pile-wall barrier, calibrated against the measured greenfield settlement trough in this case. In these models, reference was made to fully-excavated tunnels. Losacco et al. (2019) and Franza et al. (2021) compared, respectively, advanced FE analyses and an analytical solution with preliminary monitoring data from the site, without considering the displacement field during TBM advancement and assuming a straight tunnel alignment. Losacco and Viggiani (2019) presented a rigorous class A numerical prediction of the site response prior to construction. They used an advanced computational approach to simulate tunnel advancement coupled with a hypoplastic constitutive model to describe the mechanical soil behaviour.

In contrast to previous studies, this paper provides a description of the displacement field caused by excavating both tunnels in greenfield conditions. This includes vertical and horizontal displacements at the ground surface and at depth as the TBMs advance past the monitoring section. These observations are compared with the results of a 3D FE model that reproduces the progressive advancement of both tunnel faces along their actual, curved alignment despite adopting a simplified tunnel excavation procedure.

Short-term ground movements induced by mechanically bored tunnels result from the interaction of mechanisms that develop at the cutterhead of the TBM, along the tapering shield, and around the lining. Five primary components that contribute to ground loss induced by an EPBM have been identified (Wan et al., 2017b): soil movement in front of the TBM; over-cutting by the cutterhead; shield body tapering; tail void closure; and lining deformation.

Several advanced numerical approaches have been developed to reproduce these mechanisms, accounting for the main physical processes occurring around the TBM shield. These include the application of face support pressure, the interaction between the shield and the surrounding soil, lining installation, and backfill grouting of the tail void (e.g.: Jenck and Dias, 2004; Kasper and Meschke, 2004; Meschke et al., 2011; Lambreght et al., 2012; Chakeri et al., 2013; Dias and Kastner, 2013; Do et al., 2013; Founta et al., 2013; Comodromos et al., 2014; Kavvadas et al., 2017; Ochmański et al., 2018; Losacco and Viggiani, 2019). A detailed review of these studies can be found in Losacco and Viggiani (2019).

However, modelling the complex interactions between these mechanisms necessitates the adoption of simplifications, particularly when dealing with complex geometries, extensive numerical models, and the

evaluation of ground movements during tunnel excavation.

The aim of this paper is to evaluate the ability of the simplified numerical procedure proposed by Broere and Brinkgreve (2002) to reproduce the ground response to tunnelling as observed at the green-field test site. This is achieved by calibrating the excavation parameters against the surface volume loss measured during the excavation of the first tunnel.

Ultimately, when comparing the results of the numerical analyses with the observed behaviour, uncertainties in some of the field monitoring data were identified.

2. Instrumented site

2.1. The metro C project in the city centre

The new Line C of the Rome underground will cross the city from Northwest to Southeast, including 29 stations for a total length of about 26 km. Currently, 24 stations and 22 km of the line are in operation. The last stations to open were *Porta Metronia* and *Fori Imperiali*, on 16 December 2025. *San Giovanni* station provides a connection with Line A, while *Fori Imperiali* station provides a connection with Line B of the Rome underground.

Contract T3 runs between San Giovanni and Venezia stations and consists of about 4 km of twin running tunnels, two stations, and two shafts. Construction of contract T3 has been more problematic than the eastern ones (contracts T4 to T7) as the running tunnels and the stations were to be built in the historical centre of the city, with significant problems resulting from the buried archaeological remains and from the need to minimise the effects of the construction activities on the existing historical and monumental heritage. Invaluable historical monuments of Roman Age, from I to V centuries, are mainly encountered along the route of contract T3: the Aurelian walls at *Porta Asinaria* and *Porta Metronia*, the church of *Santo Stefano Rotondo*, the *Acquedotto Celimontano*, the *Anfiteatro Flavio (Coliseum)*, the *Basilica di Massenzio*, the *Colonnacce* and the *Foro di Cesare*. For this reason, a multidisciplinary Steering Technical Committee (STC) was set up by the general contractor, Metro C SCpA, with the assignment of implementing all the necessary procedures to safeguard the historical monuments of Roman age potentially interacting with the new Line C. A methodological approach was adopted by the STC to perform the line-monument interaction studies, following procedures at two increasing levels of complexity: *greenfield (Level 1)* analyses were first carried out ignoring the stiffness and weight of the existing monuments for a simplified evaluation of the potential damage induced by tunnelling and, when damage evaluation exceeded the category 0 defined by Burland and Wroth (1974), soil-structure interaction analyses were carried performing FE (*Level 2*) analyses under both two- and three-dimensional conditions, accounting for the stiffness and weight of the monuments (Rampello et al., 2012; Burghignoli et al., 2013).

This approach enabled the potential interaction between each monument and the line to be evaluated. This allowed an appropriate monitoring system to be designed to detect the displacement field induced in the monuments by the construction of the line. Reliable control of the construction processes was thus ensured, providing the necessary information to support the decision-making process.

In this context, a fully instrumented greenfield control section was installed between San Giovanni and Amba Aradam/Ipponio stations to monitor the performance of the two EPB-TBMs. This section was chosen because its ground conditions are representative of those encountered throughout Contract T3.

The two tunnels were excavated from the Multifunctional Shaft 3.3 (Masini et al., 2021a; Masini et al., 2020) towards the *Amba Aradam/Ipponio* station; the southbound tunnel was excavated about 18 days after the completion of the northbound tunnel. Average driving speeds were about 42 m and 53 m per week for the first and the second tunnel, respectively. The tunnel axes run approximately from 13 m to 18 m

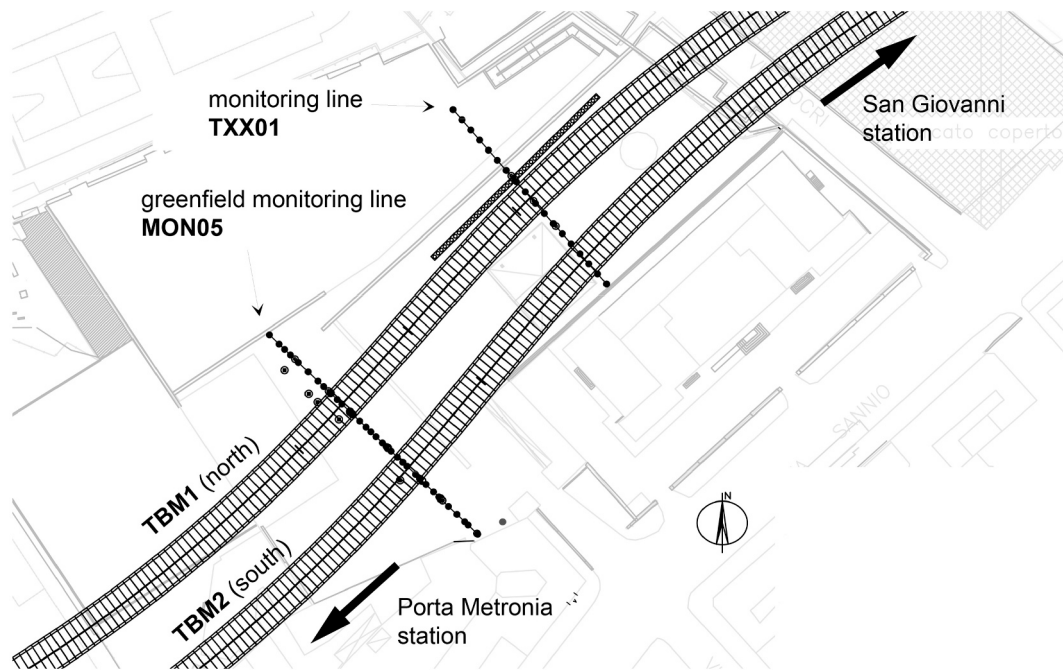


Fig. 1. Plan view of the experimental site.

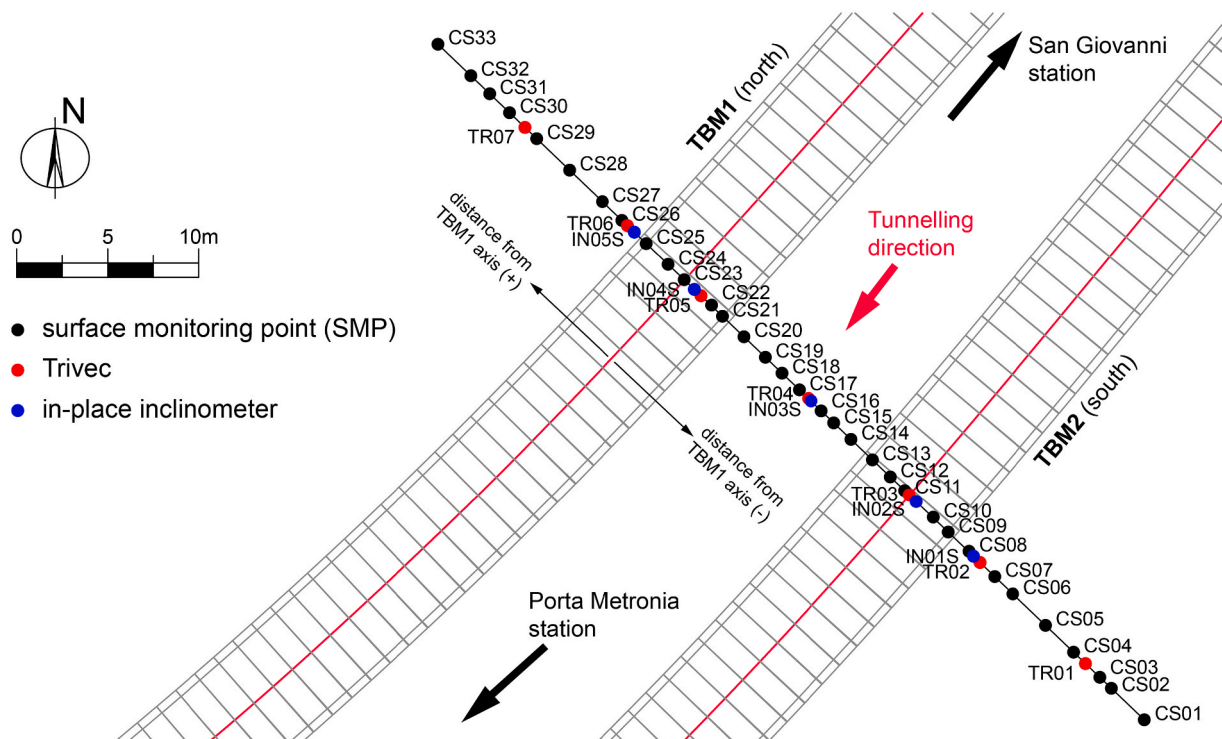


Fig. 2. Plan view of the instrumentation layout installed at the green field monitoring alignment (MON05).

apart, at a depth $z_0 = 26$ m below the ground level.

The two EPB-TBMs that excavated the Metro C tunnels have a maximum excavation diameter at the cutterhead of 6.71 m and a tapered shield length of 11.8 m. The diameter of the shield reduces from 6.69 m behind the rotating head to 6.67 m at the tail, resulting in an annular gap increasing from 10 mm to 20 mm towards the shield tail. Conventional bolted precast concrete segmental lining rings were erected within the shield body; they have a nominal length of 1.4 m and an inner and outer diameter of 6.1 m and 6.4 m, respectively (i.e., ring

thickness of 0.3 m). The resulting tail void gap of 155 mm between the tunnel lining extrados and the excavated ground was filled using a two-part grout injection system. A comprehensive system of instrumentation allowed various operational variables to be monitored continuously and the TBM to be controlled effectively.

2.2. Field instrumentation

Fig. 1 shows a plan view of the test site and the instrumentation

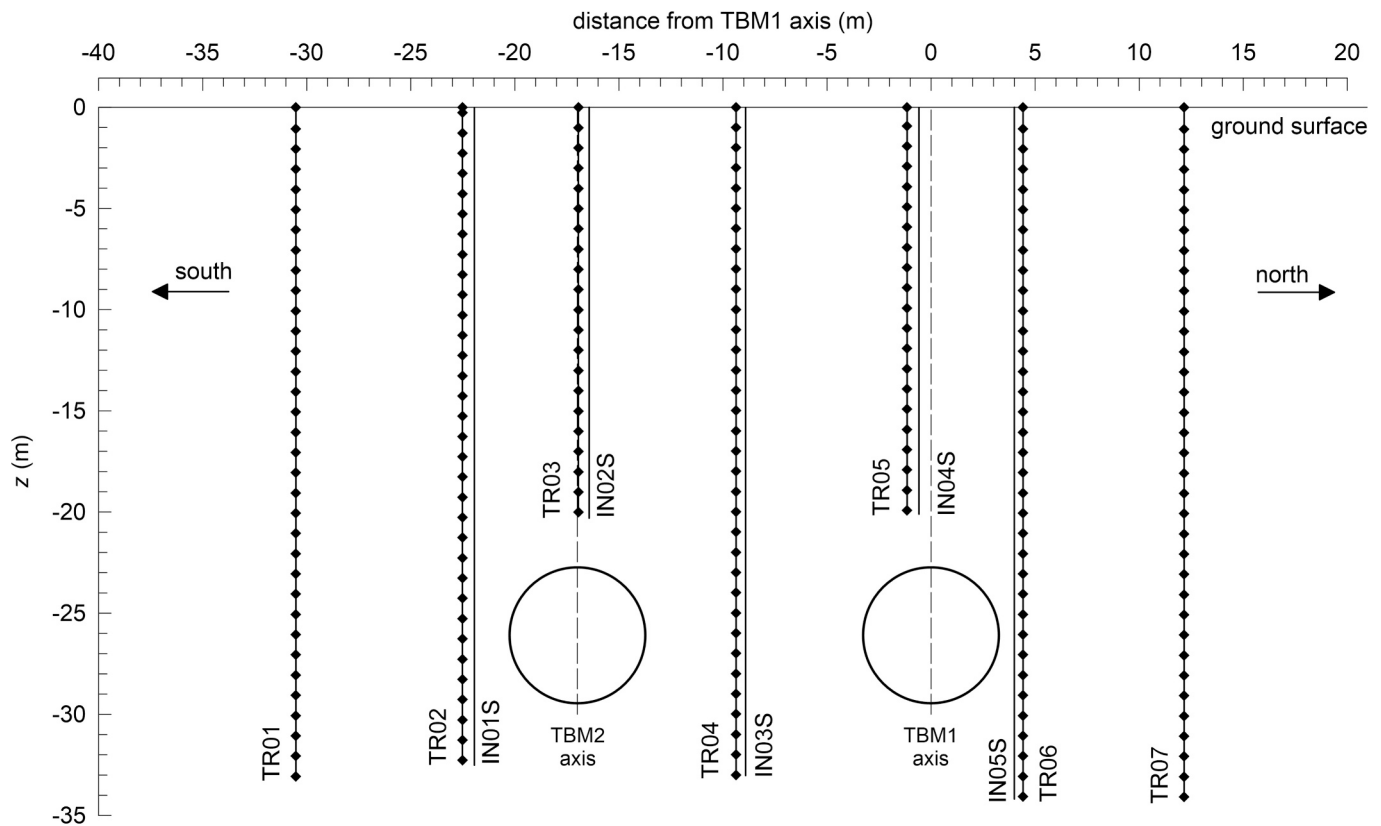


Fig. 3. Cross section of the green field monitoring alignment (MON05).

layout which was conceived to monitor the surface and subsurface ground response to tunnelling. The instruments were installed along the MON05 and TXX1 arrays, which are perpendicular to the tunnel axes: the MON05 array is representative of greenfield conditions encountered along contract T3, while the TXX1 array was located perpendicular to the centreline of a line of bored piles that was pre-installed to evaluate the effectiveness of an embedded barrier in reducing the ground movements induced by tunnelling.

This paper focuses on the ground response induced by tunnel excavation in greenfield conditions, therefore referring to the monitoring data collected from section MON05 only, while the displacement field monitored at array TXX1 is discussed and compared with the displacements computed by FE numerical analyses in [Masini and Rampello \(2021\)](#).

[Fig. 2](#) and [Fig. 3](#) show a plan view and a cross section of the green-field array MON05, with the position and the nomenclature of the installed instruments. Twelve boreholes were equipped with instruments for subsurface monitoring. Five of these were inclinometer casings and seven were Trivec casings ([Köppel et al., 1983](#)). The Trivecs allowed the three orthogonal components Δx , Δy and Δz of the displacement vectors to be measured along the vertical alignment, at a depth spacing of 0.5 m. Trivec and inclinometer boreholes were paired to compare the horizontal displacements obtained by the two instruments. Five Trivec and three inclinometer casings extended about one diameter below the tunnel invert and were embedded at a depth of 33 to 34 m below the ground level; a Trivec casing and an inclinometer casing were installed at about the centreline of each tunnel, with the toe of the casings located at a depth of 20 m, that is about 3 m above the crown of the tunnels. The top of each vertical monitoring alignment was equipped with an optical prism screwed into a socket installed at ground surface to relate the measured subsurface settlement profiles to the absolute surface settlements.

Ground surface settlements were measured by precise levelling,

using 33 surface monitoring points (SMPs) installed along the array. The displacement markers incorporate sockets into which a removable survey plug can be screwed with good positional repeatability for manual surveying. Specifically, precise levelling is performed using a digital level which can detect the height of the plane of collimation on a suitable bar-coded staff to a resolution of 0.01 mm.

The reliability of the measurements obtained from the inclinometers was found to be limited. The inclinometer casings were installed approximately one and a half years prior the start of excavation and were initially monitored using a mobile biaxial inclinometer probe, which generally yields reliable readings at 1 m intervals. However, once the excavation of the first tunnel had begun and the cutterhead was about 64 m from section MON05, fixed-in-place probes were installed within the same casings. These probes had a coarser measurement interval of 2 m and automatic data acquisition. This change in instrumentation, which was carried out when TBM1 was approximately 60 m from the MON05 station, is believed to have negatively impacted the reliability of the subsequent readings for two main reasons. Firstly, an insufficiently time interval was available to establish a reliable zero reading for the fixed probes. Unlike the other instruments, the zero reading for these probes was not established over a period of about 90 days. Secondly, the fixed transducer readings may already have been partially affected by the ongoing excavation at the time of installation. The unreliability of the inclinometer data was assessed based on the occurrence of rapid sign reversals (+/-) occurring in the horizontal displacement profiles with depth. These reversals were not observed in the corresponding Trivec measurements or in the FE results and could not be given a physical interpretation. There was also an absence of a reliable topographic reference for horizontal displacement at the top of the casing. Consequently, this discussion will focus solely on the data obtained from the surface monitoring points (SMPs) and the Trivec alignments (TR).

Due to the absence of reliable data on horizontal displacements at ground level, the relative horizontal displacements measured by the

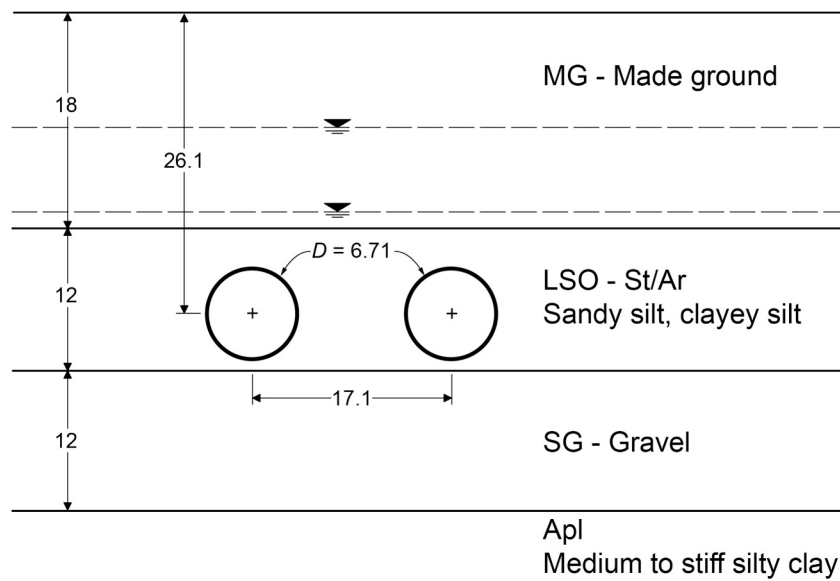


Fig. 4. Soil profile and tunnels layout.

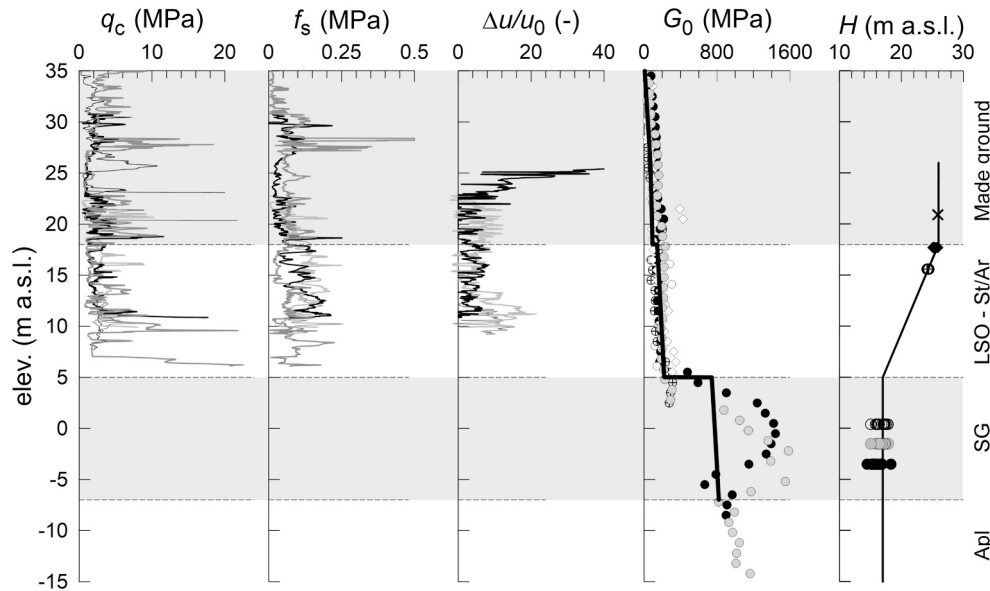


Fig. 5. Tip resistance, sleeve friction, excess pore pressure ratio, small-strain shear stiffness and pore water regime at the experimental site.

Trivec instruments were integrated under two hypotheses. Bottom-up integration was adopted for the deepest casings (TR01-02-04-06-07), assuming zero displacement at the deepest point of these casings, which are embedded in the stiff gravelly layer. For the shorter casings positioned above the tunnel axes (TR03-05), the displacement at the top of the casing was assumed to be zero, based on the symmetry of the settlement trough when excavating the tunnel beneath the casing, and the large distance from the opposite tunnel. The relative displacements measured by these casings were therefore integrated from the top of the instrument.

2.3. Soil conditions

Soil conditions at the test site are typical of those encountered in the historical centre of Rome (Rampello et al., 2012; Burghignoli et al., 2013), the soil profile being based on the site investigations carried out during several campaigns, from 1995 to 2011. Fig. 4 shows a geotechnical section of the site. The first layer encountered from the nearly

horizontal ground surface (+35 m a.s.l.) is 18 m thick and consists mainly of coarse-grained sand and gravel in a pyroclastic matrix. This layer is known as heterogeneous made ground (MG). Beneath this are recent alluvial deposits from the Tiber River, dating from the Pleistocene era, which extend to a depth of $z = 42$ m (−7 m a.s.l.). The alluvium consists of fine-grained soils up to a depth of $z = 30$ m (+5 m a.s.l.), which vary in grading but have similar physical and mechanical properties. Organic clayey silts (LSO) are mostly found at the top of sandy and clayey silts (St/Ar), with an overall thickness of around 12 m. This layer sits on top of a 12-m-thick layer of sand and gravel (SG), which extends to a depth of $z = 42$ m (−7 m a.s.l.). Beneath the sandy gravel (SG), is a thick layer of stiff, overconsolidated silty clay, known as the blue Vatican clay, which is of Pliocene age. Both tunnels run entirely within the sandy and clayey silts (LSO-St/Ar layer). The mechanical properties of these units are described in the following section.

Fig. 5 shows the profiles of tip resistance q_c , sleeve friction f_s and the excess pore pressure ratio $\Delta u/u_0$, as measured by CPTU tests. Also shown are the small-strain shear modulus G_0 , obtained from Cross Hole tests,

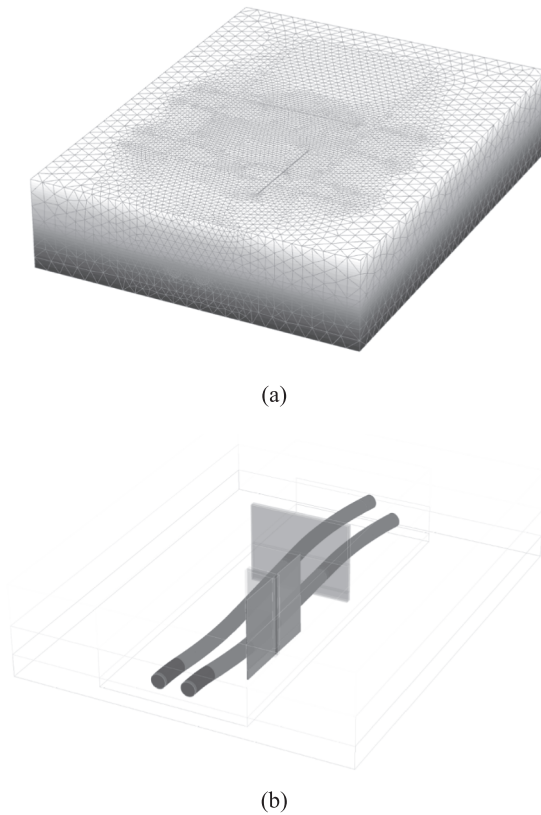


Fig. 6. 3D numerical model: FE mesh (a) and tunnels layout (b).

and the hydraulic head, evaluated from pore water pressure measurements. The average values of tip resistance and sleeve friction remain relatively constant in the coarse-grained made ground and in the layer of sandy and clayey silt (LSO-St/Ar), where the tunnels were excavated, with values of $q_c = 2.0\text{--}4.0$ MPa and values of $f_s = 0.05\text{--}0.15$ MPa. The small-strain shear modulus G_0 increases with depth, ranging from around 75 MPa at the ground surface to 300 MPa at the bottom of the sandy and clayey silt layer, with a sharp increase in the underlying layers. G_0 values of about 1300 MPa and 1000 MPa were measured in the gravel and stiff clay, respectively, while the CPT tip exhibited refusal at the top of the gravelly layer. As with the q_c and f_s profiles, there is no significant change in the excess pore water pressure ratio, $\Delta u/u_0$, in the made ground and in the sandy and clayey silt which is attributed to the composite nature of the LSO-St/Ar layer. The CPTU test results, in terms of q_c , f_s , and $\Delta u/u_0$ profiles, therefore seem to support the assumption of a drained response of the LSO-St/Ar layer to tunnel excavation.

The pore water pressure regime is characterised by downward seepage in the silty soils from the made ground, where a constant hydraulic head $H = 26$ m a.s.l. was measured, to the deep layer of sandy gravel, at a constant head $H = 17$ m a.s.l.. This condition is typical of that encountered along line C of the Rome underground and is caused by pumping from the deep, permeable layer of sandy gravel for anthropic purposes, which lowers the hydraulic head in this layer significantly. The hydraulic head in the made ground is primarily sustained by the Tiber River, with a minor contribution from meteoric infiltration. This results in the observed constant-head, downward seepage regime through the clayey silt.

3. Numerical model

3.1. FE discretisation and constitutive models for soils

A 3D FE back-analysis of the displacement field induced by the

excavation of the twin tunnels was performed with Plaxis 3D (Brinkgreve et al., 2013), using the technique proposed by Broere and Brinkgreve (2002), to simulate the three-dimensional process of mechanised tunnel excavation.

Fig. 6 shows the FE model used for the calculations. It represents a soil volume measuring 150 m in width, 42 m in height, and 190 m in length. These dimensions were chosen to minimise the impact of boundary conditions on the computed results. The soil profile reproduces the soil layers described above (see Fig. 4). The bottom boundary of the mesh is located at the bottom of the sandy gravel (SG), 42 m below the ground surface. The free-field monitoring section is located approximately halfway along the tunnels' path, with a total length of 160 m. The mesh consists of 535,453 nodes and 37,036 10-node tetrahedral elements with second-order displacement interpolation. This density was chosen following a preliminary set of analyses to strike a balance between solution accuracy and reasonable calculation times.

The nodes at the bottom of the mesh are fixed in both the vertical and horizontal directions, while the displacement of the nodes on the side faces of the model is constrained in the normal direction. At the start of the analysis, a constant hydraulic head of 26 m a.s.l. was set at the point where the made ground meets the sandy and clayey silt, and a hydraulic head of 17 m a.s.l. was set at the point where these meet the sandy gravel, in order to replicate the observed downward stationary seepage through the LSO-St/Ar layer.

In the numerical model, the actual path of both tunnels is reproduced, with the tunnel axes located at a depth of $z_0 = 26.1$ m, and the horizontal distance between the axes equal to 17.1 m at the free field monitoring section MON05, as in the reference case study.

Analyses were performed in terms of effective stresses, assuming drained conditions for the soils due to their high ($k = 7 \cdot 10^{-5}$ m/s) and medium ($k = 0.25\text{--}1 \cdot 10^{-5}$ m/s) permeability, respectively, in the coarse-grained layers of made ground (MG) and sandy gravel (SG), and in the fine-grained clayey silt.

Given the relatively fast rate of TBM advance (about 11 m/d) and the medium permeability of the sandy and clayey silt in which the tunnels were excavated, it is acknowledged that these soils may have experienced partial drainage in the close proximity to the tunnels during excavation. This is consistent with the modest excess pore water pressure recorded by the vibrating wire piezometer cells which were installed 1.30–1.90 m from the tunnel lining (not shown for the sake of brevity). These cells measured between 10 and 30 % of the initial in situ pore water pressure during tunnel passage, and dissipated in less than 4 days. Therefore, drained conditions were assumed in the FE analyses, in line with the relatively high permeability of the clayey silt and the rapid dissipation of excess pore water pressures observed in the field.

According to Wongsaroj et al. (2013) and Mair (2008), an impervious lining was assumed in the analyses, as the dimensionless soil-lining permeability ratio (RP) was found to be less than 0.1. This value was computed using an average lining permeability $k_L = 1.5 \cdot 10^{-11}$ m/s (Rampello et al., 2012), a clayey silt layer thickness $C = 4.74$ m, a soil permeability $k_S = 0.25 \cdot 10^{-5}$ m/s and a total lining thickness (grout and concrete lining) $t_L = 0.45$ m, giving a relative permeability $RP = (k_L \cdot C)/(k_S \cdot t_L) = 6.3 \cdot 10^{-5}$. This assumption is supported by piezometric measurements taken close to the tunnels, whose final readings returned to the initial in-situ values, confirming that the passage of the tunnels did not permanently alter the pore water pressure regime.

The mechanical behaviour of the made ground (MG) and sandy gravel (SG) layers, which were mainly characterised via in situ tests, was described using the *Hardening Soil* (HS) model (Schanz et al., 1999). In contrast, the behaviour of the clayey silt (LSO-St/Ar), for which accurate laboratory test results were available (Rampello et al., 2019), was described using the *Hardening Soil* with *small-strain stiffness* (HSsmall) model. This is a combination of the HS model proposed by Schanz et al. (1999) and the elastic small-strain model developed by Benz et al.

Table 1
Strength, stiffness, and initial state parameters of soils.

soil layer	$\gamma(\text{kN/m}^3)$	ν' (–)	$c'(\text{kPa})$	$\phi'(^{\circ})$	OCR (–)	K_0 (–)	G_0^{ref} (MPa)	$\gamma_{0.7}(\%)$	$E_{\text{ur}}^{\text{ref}}$ (MPa)	m (–)	$E_{50}^{\text{ref}}/E_{50}^{\text{ref}}$ (–)	$E_{50}^{\text{ref}}/E_{\text{ocd}}^{\text{ref}}$ (–)
MG	17	0.2	5	34	3.5	0.441	–	–	240	1	10	1
LSO – St/Ar	19.5	0.2	28	27	1.3	0.62	125	0.035	150	0.8	18	1.5
SG	20	0.2	0.1	40	7.5	0.36	–	–	900	0.4	10	1

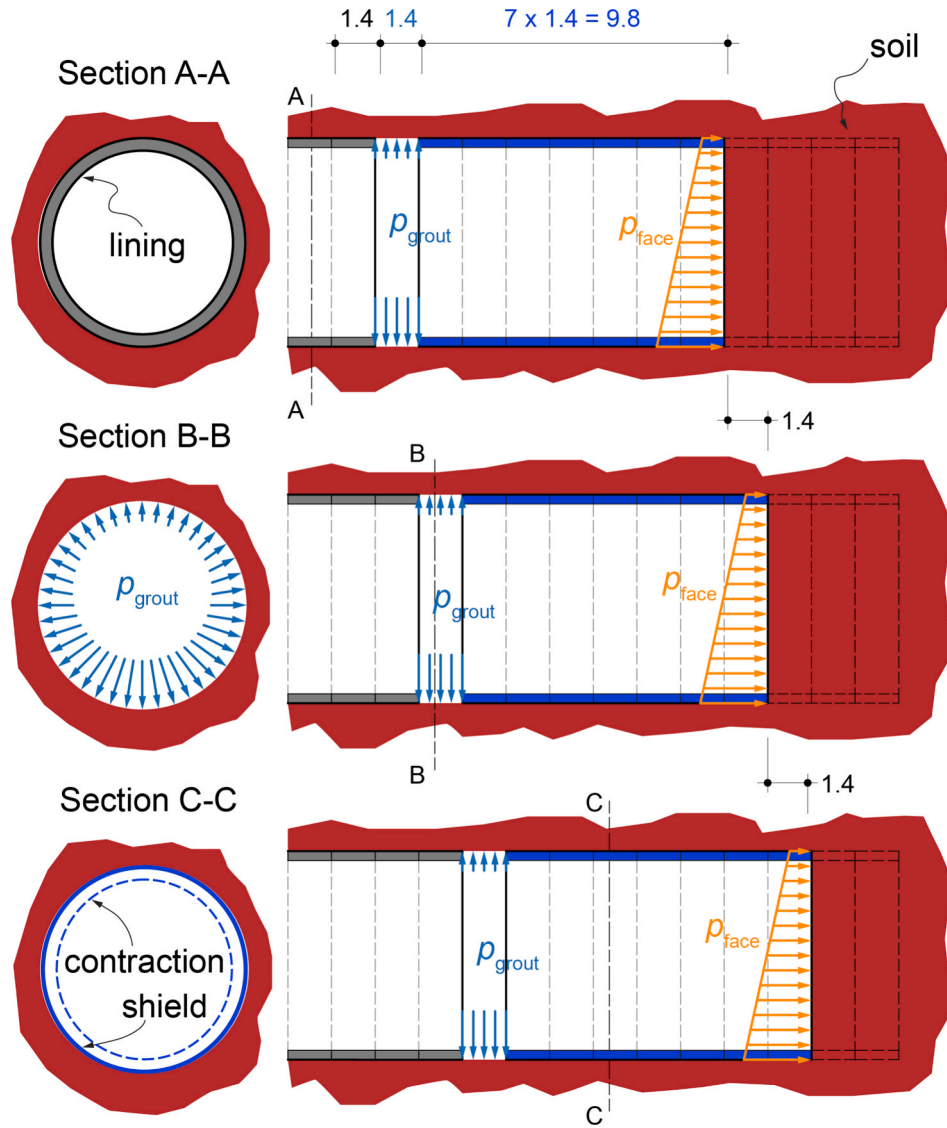


Fig. 7. Stepwise procedure adopted to simulate tunnel excavation (dimensions in metres).

(2009). The *HSsmall* model shares the same features as the *HS* model and is also capable of accounting for the high shear modulus at small strains G_0 and the decay of the secant shear stiffness ratio G_s/G_0 with shear strain γ .

Both models account for non-linear stress–strain behaviour under monotonic loading, with an initial modulus G_0 (*HSsmall*) or $E_{\text{ur}}^{\text{ref}}$ (*HS*), as well as the early development of irreversible strains. During unloading, they exhibit either non-linear (*HSsmall*) or linear (*HS*) elastic behaviour, thus describing a marked change in stiffness. Table 1 lists the values of the constitutive parameters adopted in the calculations.

It is important to note that the FE analyses presented in this paper were performed using the same set of input model parameters which were calibrated using the results of a detailed geotechnical

characterisation of the site, and were successfully used to simulate the observed behaviour of other excavation works carried out close to the test site. These works included the excavations of the *Amba Aradam* and *San Giovanni* stations, the ventilation shaft 3.2 and the launch pit 3.3 (Masini et al., 2021a), as well as the simulation of the ground response in the presence of an embedded pre-installed barrier (Masini and Rampello 2021; Rampello and Masini 2022).

The variation of the small-strain shear stiffness with depth was obtained by calibrating the parameters m and G_0^{ref} (for the *HSsmall* model) and m and $E_{\text{ur}}^{\text{ref}}$ (for the *HS* model) against the small-strain shear modulus (G_0) profile evaluated by cross-hole tests, assuming $\nu' = 0.2$. Fig. 5 shows the observed profile of G_0 alongside that assumed in the numerical analyses (continuous line).

Table 2

Model parameters assumed for the TBM advancement simulation.

Tunnel	Grout pressure (kPa)	vertical gradient(kPa/m)	face pressure (kPa)	vertical gradient(kPa/m)
northbound	350	20	320	15
southbound	360	20	330	15

The shear strain $\gamma_{0.7}$ and the unloading–reloading modulus E_{ur}^{ref} were calibrated to best approximate the curves describing the decay of the secant shear modulus G_s and the increase of the damping ratio ξ with the shear strain γ , as observed in resonant column tests.

The parameters E'_{50} and E'_{oed} were evaluated by fitting the stress–strain curves obtained from standard and non-standard triaxial tests, which were carried out using local axial-strain transducers on undisturbed samples of the clayey silt (LSO-St/Ar layer). Details of the laboratory investigations and the calibration of the constitutive models can be found in [Rampello et al. \(2019\)](#).

After initialisation of the effective stress state and simulation of the observed downward seepage, simulation of the northbound tunnel (TBM1) excavation was performed first, followed by simulation of the southbound tunnel (TBM2) excavation, as described in the following.

3.2. 3D modelling of tunnel excavation

[Fig. 7](#) shows the simplified procedure adopted for simulating tunnelling excavation in 3D FE analyses ([Broere and Brinkgreve, 2002](#)). The EPBM tunnel excavation process is divided into construction stages. The same steps are repeated at each stage to simulate support pressure at the tunnel face, the tapering shape of the TBM shield, soil excavation, installation of the tunnel lining and grouting of the gap between the soil and the recently installed lining. At any given stage, the tunnel cavity is lined either by the shield or the permanent lining. The only exceptions are the tunnel face, where support pressure is applied, and the intermediate zone between the shield and the permanent lining where the tail grout injection is applied, which is 1.4 m long. The shield extends 11.2 m (equal to 8 tunnel rings) and has been modelled using 0.03 m-thick shell elements. It has a diameter $D = 6.4$ m, a Young's modulus $E = 210$ GPa and a Poisson's ratio $\nu = 0.15$. Overcutting at the rotating head and tapering of the TBM body are modelled by applying a global contraction to the shell elements of the shield, expressed as a percentage of the cross-sectional area of the TBM shield located directly behind the cutterhead ($D = 6.4$ m). This contraction increases linearly from 0 % at the head to 0.55 % at the tail. This value was chosen to match the measured volume loss at the ground surface after the northbound TBM had passed, and it was also used to simulate the excavation of the southbound tunnel. It should be noted that the global contraction does not generally result in a uniform radial contraction of the shield; rather, the actual deformed shape of the shield cross-section is governed by the stiffness of the surrounding soil and the soil–shell interaction.

The weight of the shell elements was taken as $\gamma = 90$ kN/m³, which included the weight of the enclosed machinery and muck in the excavation chamber. The final concrete lining, installed one ring away from the shield in the analysis, was modelled as a 0.3 m-thick continuous ring with an inner diameter of 6.4 m using solid elements with a unit weight $\gamma = 25$ kN/m³, a Young's modulus $E = 30$ GPa and a Poisson's ratio $\nu = 0.2$. Purely frictional interface elements were used at the soil–shield and soil–lining contacts to reduce the angle of shearing strength of the adjacent soil, ϕ' , by a coefficient $R_{int} = 0.7$. The face support pressure and the pressurised backfill injection were modelled as distributed loads, increasing linearly with depth. The former was applied to the soil at the face of the excavation, while the latter was applied to the intermediate zone between the shield and the permanent lining. [Table 2](#) shows the values used for both tunnels, calculated as an average of those measured at the test site by the TBM contractor.

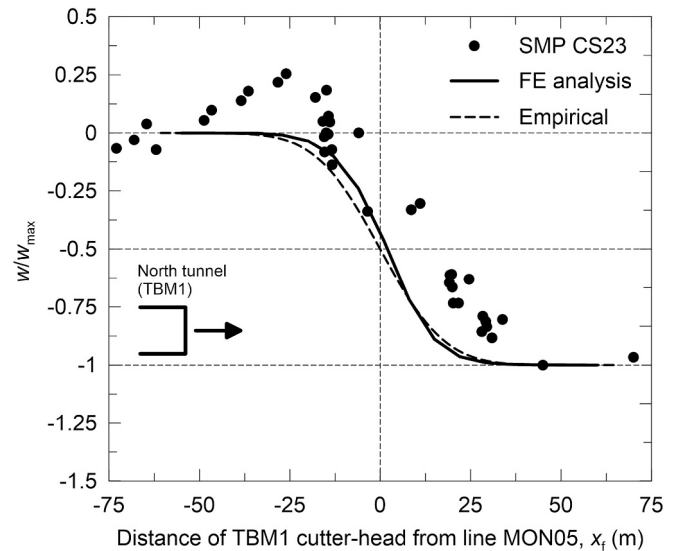


Fig. 8. Longitudinal profiles of surface settlements measured above tunnel axis (SMP CS23) during the excavation of northbound tunnel TBM1 (positive values indicate heave).

Tunnelling was modelled in discrete steps by removing one tunnel ring-length (1.4 m) of soil and advancing the support pressure, shield, and lining by an equal length, from one side of the mesh to the other, covering a total path of 160 m.

Therefore, global contraction is the only parameter governing the excavation procedure that was calibrated against monitoring data. Specifically, it was calibrated against the surface volume loss measured after the first tunnel was excavated. All other input parameters, including the soil constitutive parameters, were derived independently from laboratory and in situ testing, and were not modified. The same calibrated contraction was then used, without further adjustment, to simulate the excavation of the second tunnel and to calculate the sub-surface ground displacements caused by both tunnels. As the numerical analyses were performed after both tunnels had been excavated, the outcomes cannot be considered class A predictions in the sense of [Lambe \(1973\)](#), where the outcome of the event is unknown at the time the prediction is made. Nevertheless, no parameters were recalibrated during the simulation of southbound tunnel excavation or subsurface response. Therefore, the corresponding comparisons provide a genuine assessment of the predictive capability of the adopted procedure.

3.3. Observed and computed ground response

In the following, vertical displacements w are taken as positive upward (heave) and negative downward (settlement). Horizontal displacements u are taken as positive when directed towards the North, consistent with the positive direction of the x -axis shown in [Fig. 3](#).

3.4. Northbound tunnel excavation (TBM1)

3.4.1. Surface settlements

The data from the surface monitoring points (SMPs) discussed in this section show the incremental changes from the 'zero reading' which was taken at the end of 2017, three months before excavation began in March 2018. [Fig. 8](#) shows the normalised vertical displacement, w/w_{max} against the distance of the cutterhead (x_f) from section MON05 during the passage of the TBM. The maximum surface settlement, w_{max} was measured by SMP CS23, which is located above the tunnel centreline (see [Fig. 2](#)). Also plotted in the figure, in dotted form, is the cumulative probability function adopted in semi-empirical methods. This was computed assuming a value of the width parameter $K = 0.398$ ([Masini](#)

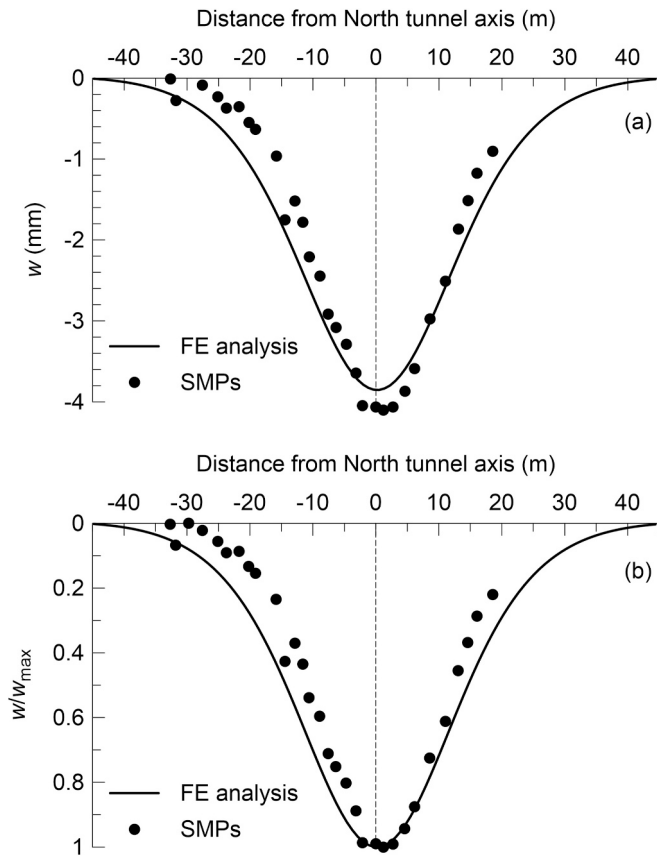


Fig. 9. Transversal surface settlement troughs induced by northbound tunnel excavation: absolute (a) and normalised (b) troughs.

and Rampello 2021). The results of the FE simulation are plotted using a solid line in the figure. Surface displacements began to occur when the cutterhead was -48 m away from the monitoring section ($x_f = -1.84 \cdot z_0$). Due to the supporting pressure applied at the excavation face (300–350 kPa), upward movements were first measured as the cutterhead approached the greenfield monitoring section. The maximum heave, equal to about 1.1 mm (25 % of w_{max}), occurred when TBM1 was -26 m away from section MON05. Downward displacements then developed quickly. Once the cutterhead had passed beneath the monitoring section, the measured settlement was -1.4 mm (about 34 % of $w_{max} = -4.1$ mm). Displacements ceased to increase once TBM1 was 45 m beyond the MON05 station. Despite the scatter in the data, the observed behaviour suggests that passing TBM1 beneath the MON05 alignment resulted in 13 % – 40 % of the maximum settlement. Although FE analysis failed to reproduce the heave as the TBM approached the monitoring section, the computed profiles are in a fair agreement with the upper-bound settlement measured at SMP CS23. This discrepancy suggests that, although the face support pressure applied in the analysis is equal to the average value measured on site, it may not fully capture the transient, construction-related fluctuations in face pressure that are likely to be responsible for the observed heave ahead of the cutterhead. While the model does not reproduce the displacement path ahead of the tunnel face, it satisfactorily reproduces the final settlement trough once the TBM has passed. This is consistent with the aim of the present analysis, in which face pressures are set directly equal to the measured values and the single free parameter governing the excavation procedure, the global shell contraction, is calibrated to match the measured surface volume loss.

Fig. 9 shows the absolute and normalised transverse settlement troughs, as measured and computed at the ground surface after TBM1 had fully passed the MON05 monitoring section. The numerical analysis

Table 3

Parameters of the interpolating Gaussian curve at the ground surface along the MON05 station after the first tunnel excavation.

Data	trough width factor i_y (m)	trough width parameter K (–)	volume loss V_L (%)	maximum settlement w_{max} (mm)
monitoring data	10.4	0.40	0.30	–4.04
FE values	10.9	0.42	0.31	–3.85

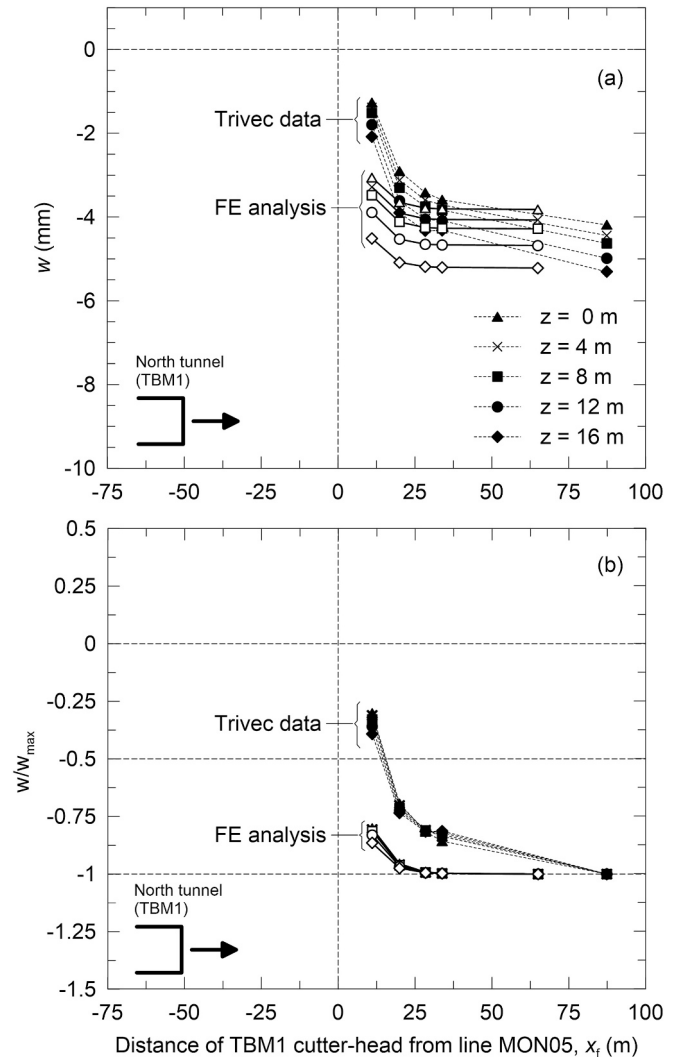


Fig. 10. Longitudinal profiles of subsurface settlements measured above tunnel axis (Trivec TR05) plotted against cutterhead distance during northbound tunnel excavation (TBM1): absolute (a) and normalised (b) profiles (positive values indicate heave).

provides a slightly wider profile at a distance greater than about 15 m from the TBM1 axis.

The observed and computed surface settlement troughs can be compared in terms of trough width factors (i_y) and volume losses (V_L). These parameters were derived by plotting the logarithm of normalised displacements ($\ln(w/w_{max})$) against the square of the normalised transverse offset distances ($(y/z_0)^2$), as described by Wan et al. (2017a). If the settlement profile follows a Gaussian distribution, the data will plot on a straight line passing through the origin. Additionally, interpolating the two halves of the trough separately allows the symmetry of the profile to

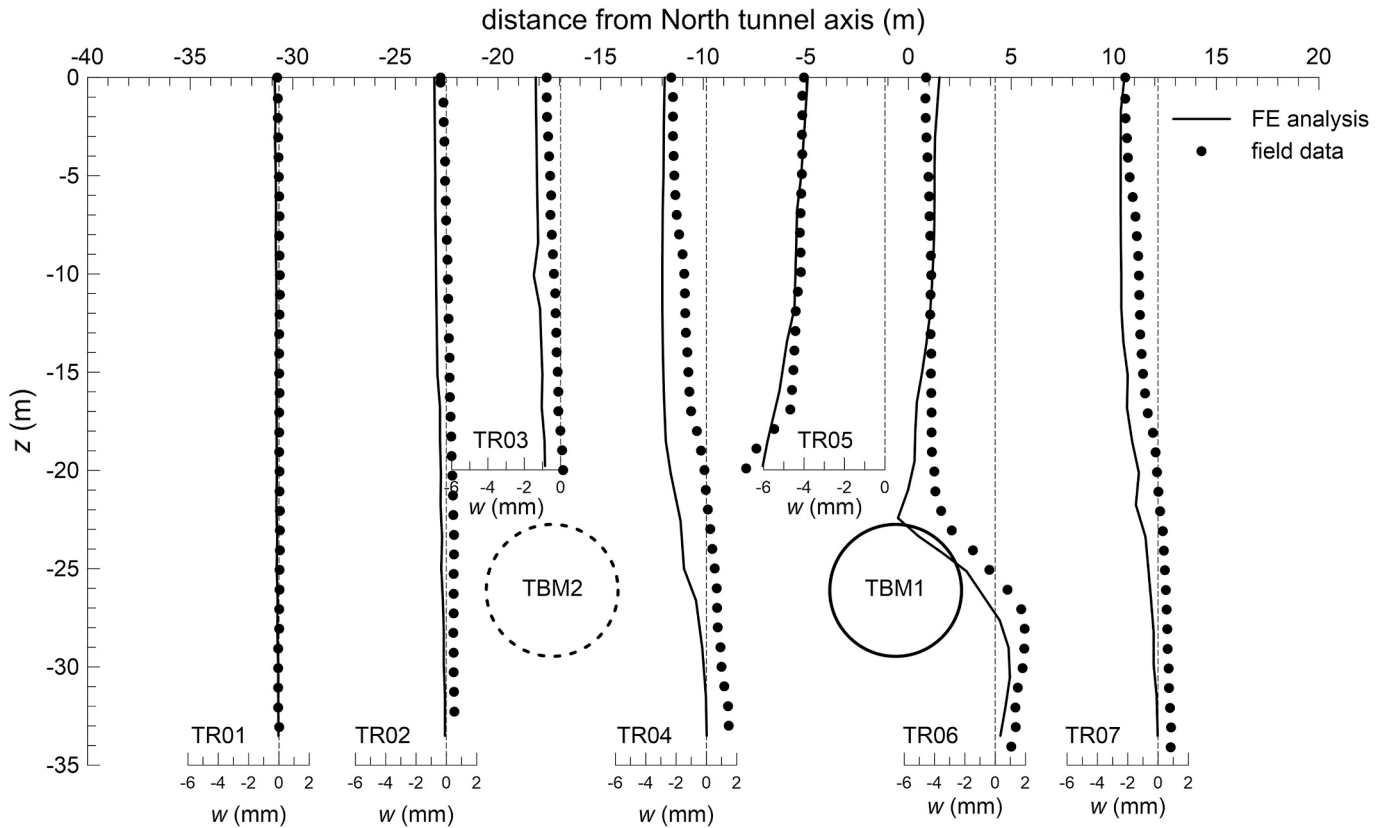


Fig. 11. Vertical displacements provided by Trivec casings at the end of northbound tunnel excavation (positive values indicate heave).

be assessed. For the MON05 section, the measured and computed data can be suitably interpolated by a single Gaussian curve, as would be expected for the ground's response to tunnelling in greenfield conditions. The parameters of the Gaussian curve interpolating the monitoring data and the FE settlement trough are compared in Table 3. The surface trough width parameter, $K = i_y / z_0$, is defined as the ratio of the distance between the tunnel centreline and the point of inflection, i_y , to the depth of the tunnel axis, z_0 . The computed and the observed troughs have nearly the same volume loss, $V_L \cong 0.3\%$, and trough width parameter, $K \cong 0.41$. The slightly smaller computed maximum settlement, $w_{\max} = -3.85$ mm, compared to the observed value of -4.04 mm, reflects the somewhat wider computed profiles, $i_y = 10.9$ m, compared to the observed value (10.4 m), with a difference of less than 5 % between the computed and the measured values.

3.4.2. Subsurface ground displacements

The subsurface ground response was investigated using data collected from the seven Trivec casings. Unlike the SMPs, the first measurement was taken 10 m after the TBM had passed section MON05. Therefore, the incremental displacements were referred to the top of each casing, with the initial displacement assumed to be equal to the measurement obtained from the nearest SMP. Fig. 10 shows the development of subsurface vertical displacements measured at Trivec TR05, which was installed above the northbound tunnel axis (see Fig. 3), as TBM1 passed beneath the MON05 alignment. Fig. 10a plots the absolute values measured at five depths ($z = 0, 4, 8, 12$ and 16 m). Fig. 10b shows the corresponding normalised values.

The maximum measured settlement was -5.2 mm at $z = 16$ m, which is 6.75 m above the tunnel crown. This occurred once TBM1 had passed 87 m beyond the instrumentation alignment. Significant ground settlements developed for $11 \text{ m} < x_f < 20 \text{ m}$. The first available reading (at $x_f = 11$ m) showed $w/w_{\max}$ values of -0.3 to -0.4 , confirming that less than 50 % of the maximum settlement occurs directly beneath the

cutterhead during closed-face shield tunnelling. For $20 \text{ m} < x_f < 33 \text{ m}$, the rate of settlement decreased; this appeared to be slightly more pronounced at greater depths, likely due to the soil being supported directly by the lining and tail-skin grouting. The remaining 12 % of the settlement (about -1 mm at $z = 16$ m) occurred between $30 \text{ m} < x_f < 80 \text{ m}$; however, there were insufficient measurements to identify a clear trend.

Fig. 10 also shows the results of the FE simulations. The numerical model predicts greater settlements as x_f approaches the monitoring section. The computed normalised settlement $w/w_{\max}$ is equal to $-0.80 \div -0.85$ for $x_f = 11$ m, reaching full development for $x_f > 20$ m. This discrepancy can be attributed to the heave that developed immediately in front of the cutting head (see Fig. 8), which the numerical model was unable to reproduce. Nevertheless, the FE simulation satisfactorily reproduces the final settlement, with the difference between the measured and computed subsurface settlements being less than 2 % at all depths for $x_f = 16$ m and around 10 % at ground level.

Fig. 11 shows the vertical subsurface displacement profiles obtained from the Trivec measurements once the TBM1 had completely passed the MON05 alignment. Negative transverse distances relate to the southern half of the settlement trough, which is situated away from the Aurelian Walls. Casing TR05, installed 1.15 m from the tunnel axis, exhibited the maximum subsurface settlement at the lowest measuring point, 2.75 m above the TBM crown ($w = -6.8$ mm at $z = 20$ m). For the other Trivec casings, maximum settlement was measured at the ground surface. Due to tunnelling-induced stress relief, the ground at TR06 (4.2 m from the tunnel axis) heaved by up to 1.9 mm at $z = 28$ m (1.45 m above the TBM invert level, $z = 29.45$ m). Beyond a transverse offset distance of 22.5 m (TR02), which is about 3.4 times the tunnel diameter, the measured displacements were barely detectable. This suggests that TR02 and TR01 were outside the zone of influence of TBM1.

Similar trends were also evident in the numerical simulations shown in Fig. 11. These are in good agreement with the measured surface

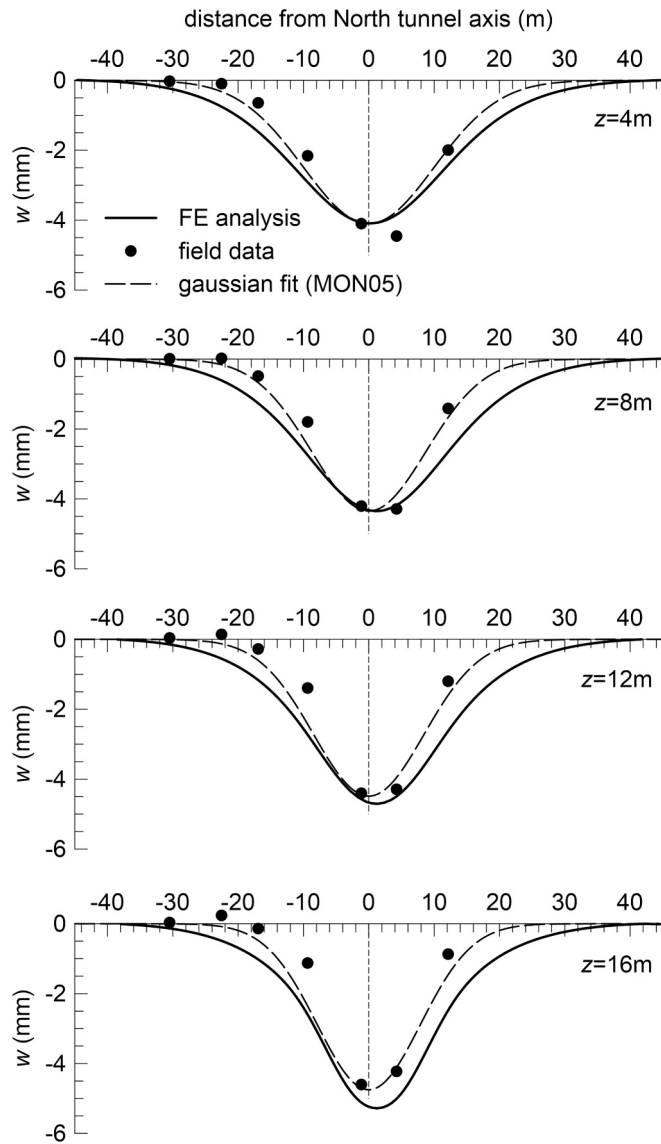


Fig. 12. Transversal subsurface settlement troughs induced by northbound tunnel excavation at different depths below MON05 alignment.

settlements and the extension of the influence zone. At the TR05 and TR06 locations ($y = -1.15$ m and $y = 4.22$ m, respectively), the FE model satisfactorily reproduced the observed profiles for $z < 20$ m. However, the numerical analysis yielded larger settlements and consequently smaller heaves closer to the tunnel.

Fig. 12 shows the transverse subsurface settlement troughs obtained from the Trivec casings installed along the MON05 alignment at various depths, at the end of northbound tunnel construction. For comparison, the figure also shows the settlement troughs computed by numerical analysis at the same depths.

As expected, the observed profiles are narrower and deeper at greater depths. The maximum measured settlement is -4.1 mm at $z = 4$ m, 1.15 m away from the tunnel axis (Trivec TR05), increasing to -4.6 mm at $z = 16$ m. The numerical simulations reasonably reproduce the maximum observed subsurface settlement, which is approximately equal to the measured value at the shallowest depth, differing by $+15\%$ ($w_{\max} = -5.28$ mm) at $z = 16$ m. However, they overpredict the width of the settlement troughs.

As with the surface settlements, Gaussian distribution curves were fitted to the subsurface settlement measurements, and the results are shown in Fig. 12 with a dashed line. The values of the subsurface

Table 4

Characteristics of the subsurface vertical displacement transverse trough along the MON05 stion after the first tunnel excavation.

depth (m)	trough width factor i_y (m)		trough width parameter K (—)		volume loss V_L (%)	
	mon. data	FE val.	mon. data	FE val.	mon. data	FE val.
4	10.0	12.3	0.45	0.56	0.29	0.35
8	8.8	11.8	0.48	0.65	0.27	0.36
12	8.5	11.2	0.60	0.79	0.27	0.36
16	7.9	10.5	0.78	1.04	0.27	0.37

transverse trough characteristics (width i_y , width parameter $K = i_y/(z_0 - z)$, and volume loss V_L) are listed in Table 4. According to the greenfield conditions, both the computed and the measured troughs were assumed to be symmetric about the TBM1 centreline. As the transverse troughs narrowed with depth, the trough width (i_y) decreased, while the subsurface trough width parameter (K) increased. Specifically, the values of K obtained from the monitoring data and the FE analysis were averaged using the power function proposed by Moh et al. (1996). The values of the power parameter $m = 0.4$ and the trough width parameter $K = 0.45$ at $z = 0$ were used (see Fig. 20). The FE analysis predicts larger values for the transverse trough characteristics listed in Table 4, ranging from 20 % to about 35 %. The volume loss (V_L) changes slightly with depth: measured values are in the range 0.27–0.29 %, whereas computed values are in the range 0.35–0.37 %.

Fig. 13 shows the profiles of the subsurface horizontal displacement measured from the Trivec casings after the TBM1 had fully passed through; the reference points and integration procedure adopted for these measurements are described in the “Field instrumentation” section. The integration hypotheses adopted are confirmed by the results of the FE analyses, which are plotted in Fig. 13. The computed profiles demonstrate that the toe of the deep instruments experienced negligible movement, and that displacements along the TBM1 axis are minimal.

Horizontal displacements are generally less than 2 mm. Considerable deformation is only observed near TBM1 (Trivec TR06), where displacement towards the tunnel amounts to approximately 1 mm. While the numerical analyses showed slightly larger displacements close to the ground surface than the measurements (Trivec TR02-03-04-07), good agreement was obtained for the two alignments near (TR06) and above (TR05) the TBM1.

3.5. Southbound tunnel construction (TBM2)

3.5.1. Surface settlements

Excavation of the southbound tunnel from the launch shaft began 18 days after the northbound tunnel was completed. Fig. 14 shows the normalised surface incremental settlements ($\Delta w/\Delta w_{\max}$) measured at the MON05 stion while TBM2 approached and passed through the monitoring array. These data refer to SMP CS12, which was installed above the centreline of TBM2. The maximum incremental settlement $\Delta w_{\max}$ was approximately -5.6 mm. Also plotted in the figure, in dotted form, is the cumulative probability function employed in semi-empirical methods. This was computed using the same value of $K = 0.398$ as for the passage of TBM1. The results of the FE simulation are plotted as a solid line. Despite the significant scatter in the data, surface settlements are evident over a range of cutterhead distances of -25 m $< x_f < 35$ m.

The monitoring data show that, when the TBM2 cutterhead is beneath the MON05 array ($x_f = 0$), ground settlement is around 20 % of the maximum incremental value, against the 40 % predicted by the FE analysis. However, due to the observed scatter in the data, no definitive conclusions can be drawn, even though these values represent the upper and lower bounds of the displacements observed in practice for closed and open shields, respectively.

Fig. 15 shows the absolute (Fig. 15a) and normalised (Fig. 15b)

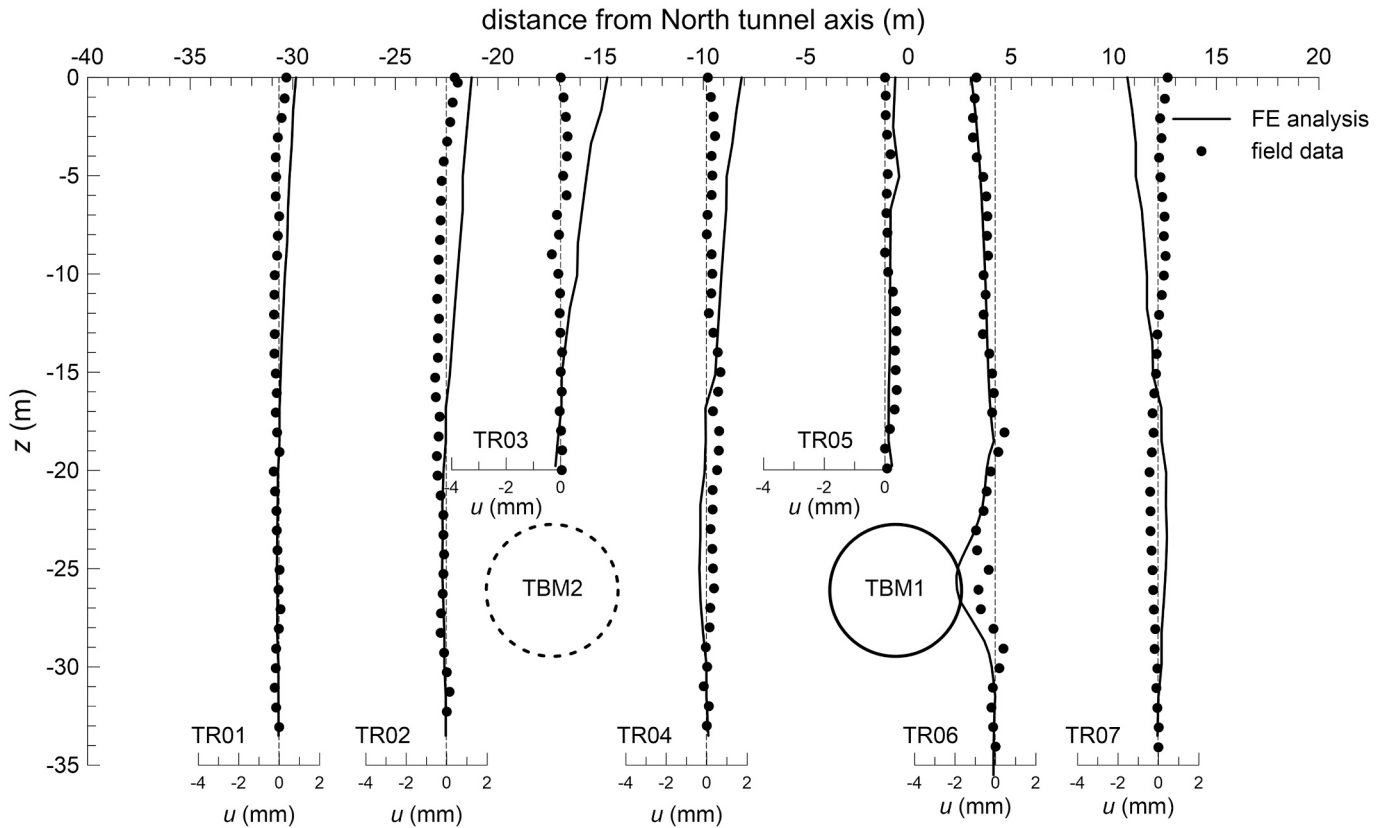


Fig. 13. Horizontal displacements provided by Trivec casings at the end of northbound tunnel excavation (positive u is consistent with the positive direction of the x -axis).

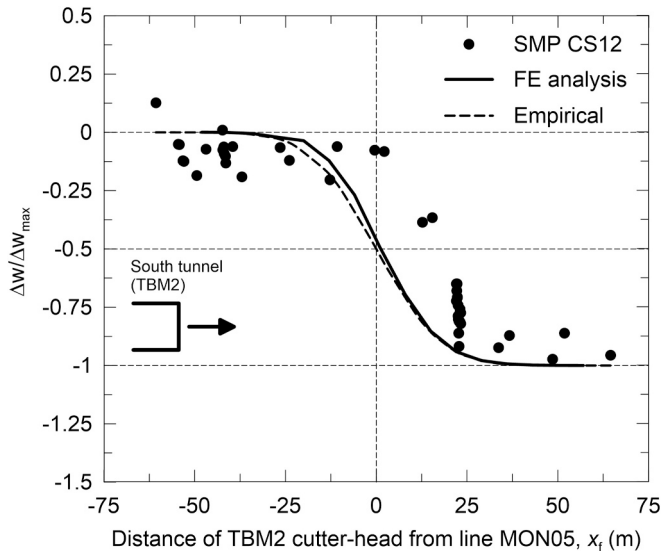


Fig. 14. Longitudinal profiles of measured and computed incremental settlements at the ground surface above the tunnel axis (SMP CS12), during the excavation of the second tunnel (TBM2, positive values indicate heave).

transverse settlement troughs that were observed and computed at the ground surface after the completion of both tunnels. For comparison purposes, the data from Fig. 9 are also plotted in the same figure. It is worth noting that the FE analysis was calibrated to reproduce the volume loss measured at the ground surface after the northbound tunnel boring machine (TBM1) had passed through, with no further adjustment made to match the response to the southbound tunnel. In light of this,

there is a very good agreement between the observed and computed behaviour in terms of both the maximum surface settlement and the shape of the settlement trough. The maximum settlement ($w_{\max} = -8.25$ mm) occurs closer to the TBM1 axis, at a distance of 7.37 m. As the profiles were asymmetric, it was necessary to fit different Gaussian curves to each side of the troughs. The parameters of the Gaussian curve that interpolates the monitoring data and the FE settlement trough are compared in Table 5. Notably, the characteristics of the observed and computed trough are nearly coincident. In both cases, the northern-half trough is wider than the southern-half trough due to the soil's non-linear and irreversible response to the passage of TBM1. The corresponding trough width parameters are $K = 0.61$ and $K = 0.56$ ($K = 0.60$ and $K = 0.57$ for the computed trough), respectively. The estimated volume losses for the measured trough were $V_L = 0.43$ % for the southern half and $V_L = 0.46$ % for the northern half. The same values were obtained for the computed trough. Overall, this corresponds to a volume loss of 0.45 %, which is greater than that calculated for the passage of TBM1 ($V_L = 0.30$ %). As the operating parameters of the two TBMs were similar, this confirms that the higher volume loss induced by TBM2 is due to the ground being softened by the excavation of the first tunnel.

3.5.2. Subsurface ground displacements

Fig. 16a shows the development of the subsurface incremental vertical displacements recorded at the Trivec TR03, which is installed above the southbound tunnel axis (see Fig. 3), as TBM2 passed beneath the MON05 alignment. Fig. 16b shows the same data plotted as a percentage of the maximum incremental displacement ($\Delta w / \Delta w_{\max}$). The response to the construction of the southbound tunnel was similar to that observed for the northbound tunnel (see Fig. 10). The maximum incremental settlement, $\Delta w_{\max} = -5.1$ mm, was measured at $z = 16$ m when TBM2 was about 23 m beyond the greenfield section. Furthermore, subsurface ground settlements developed after the cutterhead

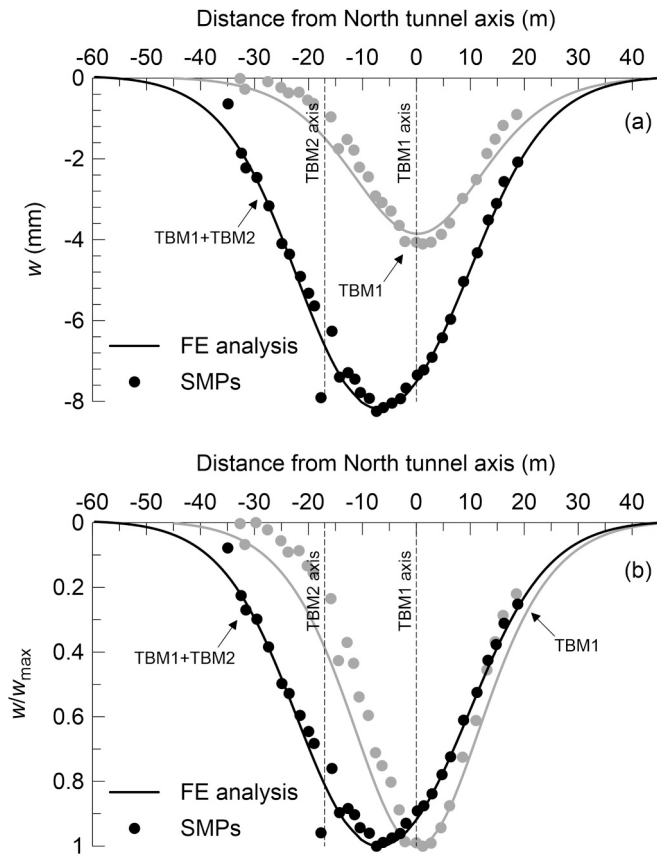


Fig. 15. Transversal surface settlement troughs induced by southbound tunnel excavation: absolute (a) and normalised (b) troughs.

passed the instrumented array, for $0 \text{ m} < x_f < 23 \text{ m}$.

As TBM2 approached the MON05 alignment, settlements at lower depths were greater than those observed closer to the TBM crown. This behaviour was reversed once the TBM cutterhead passed beneath the MON05 stion ($x_f > 0 \text{ m}$), with the measuring points closer to the tunnel crown experiencing greater settlements than the shallower ones. Similar behaviour was also observed during the excavation of the Crossrail tunnels in London, as reported by Wan et al. (2017b). Unlike the surface displacements measured by the SMP CS12, the Trivec TR03 recorded a small upward subsurface ground displacement of up to 0.4 mm when the cutterhead was at about -11 m from the MON05 section. This may be attributed to the pressure applied at the excavation face. TBM2 was stopped for about 10 days, by which time the cutterhead was 22 m beyond the greenfield section and 82 % of the maximum incremental settlement had already occurred (Fig. 16b). During this time, a small upward displacement of less than 0.5 mm was first observed, due to the ground being pushed up by the tail-skin grouting. Then, as the grout pressure decreased, the tail void began to close, resulting in an additional settlement of -1.2 mm . This settlement was only partially recovered (by about 0.3 mm) when the tail-skin grouting was restored. Therefore, the standby period resulted in a further 17 % increase in

settlement (Fig. 16b). For $x_f > 25 \text{ m}$, the increase in settlement was lower than 0.2 mm.

The FE simulation shows a fair agreement with the measured settlement profile at ground surface, with a final difference of about 0.5 mm (+10 %). However, displacements occur earlier in the FE model than was observed. This may be due to the TBM face support pressure, which was found to induce positive vertical subsurface displacements immediately in front of the tunnel face. The maximum heave measured at a depth $z = 16 \text{ m}$ was approximately 0.5 mm when the tunnel face was -10 m away from MON05 alignment. Consequently, the numerical model overestimates the final settlements, with the difference between the computed and observed settlements increasing with depth, to reach 1.78 mm (30 %) at $z = 16 \text{ m}$.

Fig. 17 shows the vertical displacement profiles measured by the Trivecs after TBM2 had fully passed. Unlike after the passage of TBM1 through the MON05 array (see Fig. 11), none of the Trivecs can be considered outside the zone of influence of TBM2 in this case. This is due to the nonlinear and irreversible soil response to the passage of the first TBM. In fact, the effects of excavating the southbound tunnel extend further from the TBM2 axis than those induced by the passage of TBM1 (see Fig. 11). Notably, both the TR07 and TR01 casings are both positioned outside the tunnel outline at a similar distance from both tunnels. The Trivec TR07 casing is installed 12 m apart from the TBM1 axis and 29 m from the TBM2 axis. Trivec TR01 is installed 13.5 m from the TBM2 axis and 30.5 m from the TBM1 axis. TBM1's passage induced a settlement of -2.17 mm at the top of the closest Trivec (TR07), whereas the value measured at the top of TR01 (30.5 m away from TBM1) was only -0.1 mm (Fig. 11). In contrast, the passage of TBM2 induced an increase in settlement of 2.35 mm at the top of the closest Trivec TR01, resulting in a final value of -2.45 mm . It also resulted in an increase in settlement of 1.67 mm at the top of the farthest Trivec TR07, which is 29 m away from TBM2, resulting in a final settlement of -3.84 mm . In other words, the effect of excavating the first tunnel fades after a distance equal to approximately 4.5 times its diameter, while the passage of the second TBM still produces significant effects at the same distance.

At the lowest measuring point of Trivec TR03, which is installed 2.65 m above the crown of TBM2 ($z = 20 \text{ m}$) the maximum settlement was $w = -7.6 \text{ mm}$. No heave was detected below the tunnel spring line by Trivec TR04, which is located between the two tunnels. Similarly, no heave was detected by Trivec TR02, which is installed about 4.5 m away from the axis of the southbound tunnel. However, a maximum heave of 1.6 mm was observed at approximately the level of the invert by Trivec TR06, which is about 21 m away from the southbound tunnel.

The results of the numerical analysis show satisfactory overall agreement with the field measurements. However, the profile of the vertical displacements computed between the two tunnels, at the location of Trivec TR04, exhibits a nearly constant settlement for $z < 19 \text{ m}$ which is different from what was observed.

Fig. 18 shows the transverse subsurface settlement troughs obtained from the Trivec casings installed along the MON05 alignment at various depths, at the end of the construction of the southbound tunnel. The settlement troughs computed by the numerical analysis at the same depths are also plotted in the figure for comparison. As with the ground surface data, asymmetry of the settlement troughs was observed after construction of both tunnels, with larger settlements developing towards

Table 5

Parameters of the interpolating Gaussian curve at the ground surface along the MON05 stion after the second tunnel excavation.

data	trough width factor $i_y(\text{m})$		trough width parameter K (-)		volume loss $V_L(\%)$		maximum settlement $w_{\max}$ (mm)
	South	North	South	North	South	North	Total
mon. data	14.5	15.8	0.56	0.61	0.43	0.46	0.45
FE values	14.8	15.7	0.57	0.60	0.43	0.46	0.45

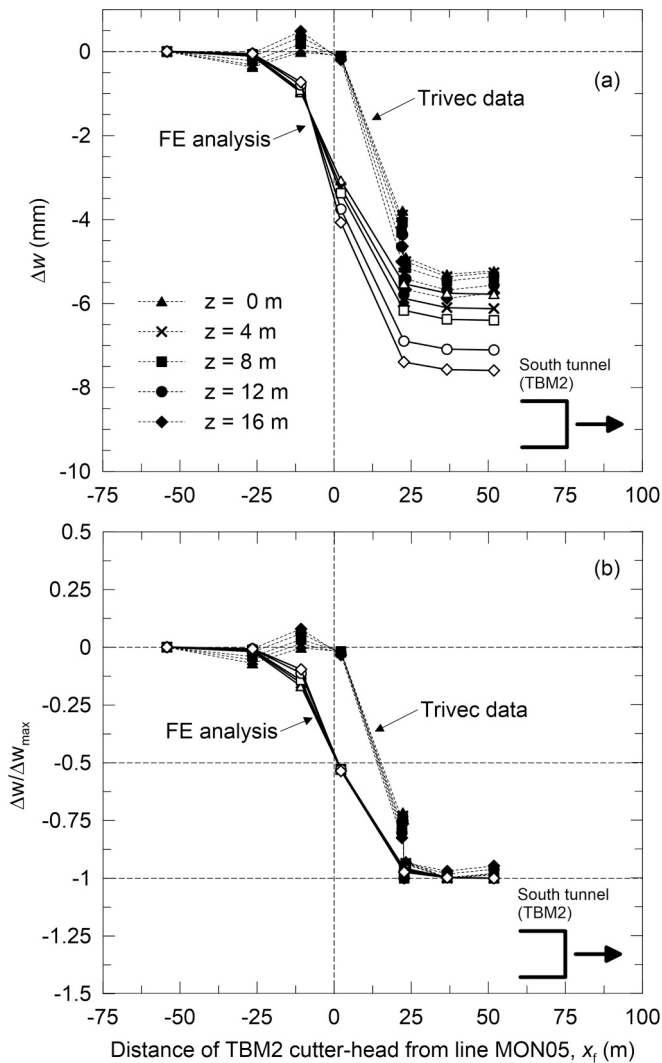


Fig. 16. Absolute (a) and normalised (b) longitudinal profiles of measured and computed subsurface incremental settlements above the tunnel axis (Trivec TR03) during the excavation of the second tunnel (TBM2, positive values indicate heave).

the TBM2 side. At the shallowest depth ($z = 4$ m, Fig. 18a), the maximum settlement ($w = -7.7$ mm) was measured by the Trivec TR04, between the TBM1 and TBM2 centrelines. Conversely, data taken from the same casing at deeper measuring points showed smaller displacements than those of the two adjacent instrumented casings, which are located above the tunnels. This behaviour differs from that observed during the excavation of twin tunnels with diameters, spacing and depths similar to those of Rome's Line C (e.g.: Wan et al., 2017b), where the maximum displacement usually occurs between the tunnels. FE analyses support this assertion, showing that the maximum settlement always occurs between the tunnels. Specifically, at shallower depths, the maximum computed settlement ($w = -8.5$ mm) occurs closer to the TBM1 axis (6.92 m). However, as the depth increases, the maximum settlement moves closer to the TBM2 axis: at $z = 16$ m, the maximum computed settlement ($w = -9.4$ mm) occurred at a distance of 7.24 m from the TBM2 axis and 9.76 m from the TBM1 axis.

FE simulations suggest that errors may have affected the readings from the deepest Trivec TR04 measuring points and that larger displacements may have occurred between the tunnels. However, this comparison constitutes an interpretive back-analysis rather than an independent validation of the model. The FE results were only used to constrain the Gaussian fitting of the deep TR04 data and not as

supporting evidence for the model itself.

As with the ground surface data, two distinct Gaussian fits were applied to the two sides of the observed and computed settlement trough shown in Fig. 18. Table 6 lists the parameters of the interpolating Gaussian curve for the southern half, while those obtained for the northern half are reported in Table 7. Due to potential errors affecting the Trivec TR04 dataset, the Gaussian fitting of the monitoring data for depths $z > 4$ m was performed by constraining the maximum settlement to the value obtained from the FE analysis at the corresponding depth. The best-fitting curves are shown in Fig. 18 with a dashed line.

For the southern half of the transverse settlement troughs (Table 6), the trough width decreases with depth, indicating progressive narrowing, whereas the subsurface width parameter K increases, consistently with the trend observed during the passage of TBM1. The volume loss (V_L) changes only slightly with depth. A similar trend is evident in the parameters derived from the numerical results for the northern half of the troughs (Table 7). However, due to the prior passage of the first TBM, the northern troughs are on average 19 % wider than those in the southern half, resulting in an approximately 20 % increase in the estimated volume loss. In contrast, regression analyses based on the monitoring data do not reveal a consistent trend when compared either with surface observations or with typical results reported in the literature for twin-tunnel excavations. Specifically, for $z > 4$ m, the fitted Gaussian curves for the northern half exhibit smaller width and lower volume loss than the corresponding curves for the southern half. This discrepancy suggests that the lack of a reliable estimate of the maximum settlement may have adversely affected the accuracy of the fitted parameters for the deeper transverse settlement troughs derived from the monitoring data. Conversely, a consistent trend was observed at the shallowest depth ($z = 4$ m).

Regarding the long-term settlement response, it is worth noting that the last available SMP readings, taken 105 days after both TBMs had fully passed through section MON05, show a modest additional settlement of around 8 % of the value measured once the TBMs had fully passed, with no discernible trend of increase over time. This additional settlement includes contributions from grout hardening and the progressive loading of the concrete lining. Therefore, it can be assumed that drained conditions have indeed been achieved consistently with excess pore water pressure measurements.

Fig. 19 shows the subsurface horizontal displacement profiles measured from the Trivec casings after the TBM2 had fully passed through the MON05 alignment (black dots in Fig. 19). The monitoring data suggest that the passage of TBM2 did not produce a significant change in horizontal displacements compared to the passage of TBM1 (Fig. 13). However, this unexpected behaviour may be due to measurement errors. In fact, comparing the TR06 and TR07 measurements in Fig. 13 and Fig. 19 suggests an anomaly in the data acquisition process, as it is unlikely that excavating the second tunnel would result in a small reduction in the horizontal displacements induced by TBM1 close to northbound tunnel. Additionally, the final horizontal displacements provided by TR02 appear to be significantly lower than those measured by TR06 after the passage of TBM1, which is in a symmetrical position with respect to TR02. The FE model predicts much larger horizontal displacements after the passage of TBM2 (thick black line in Fig. 19), exhibiting a response pattern that is consistent with that observed and computed for TBM1 (thick grey line in Fig. 19). The magnitude and direction of subsurface horizontal displacements are largely influenced by the displacement field induced by the passage of TBM1. Along Trivecs TR02 and TR04, which are the closest to the southbound tunnel, the computed horizontal displacements are directed towards TBM2, with a maximum value of 2.1 mm at the tunnel spring-line. At shallower depths, TBM2's passage results in increased horizontal displacement along all Trivec alignments that had already moved towards TBM2 after first tunnel's excavation (TR01-02-06-07). At TR04, positioned between the tunnels, the horizontal displacement direction reverses due to second tunnel's excavation. Overall, the results of the FE analyses are

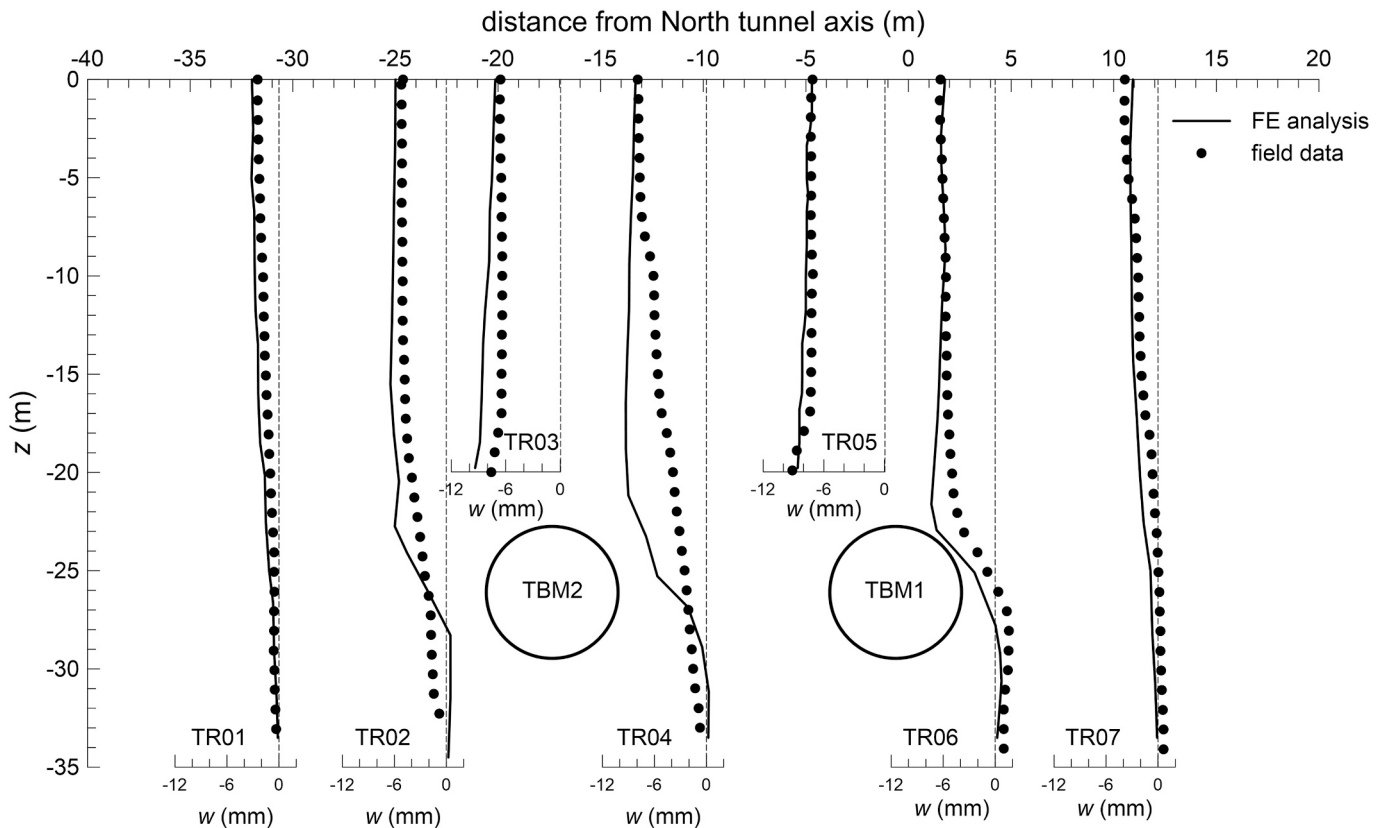


Fig. 17. Vertical displacements provided by Trivec casings at the end of southbound tunnel excavation (positive w indicates heave).

consistent with the observations and computations related to vertical displacements (Fig. 16) and confirm a larger influence zone on the side where the first tunnel was excavated.

These observations emphasise the importance of reliable horizontal displacement measurements at ground level for interpreting subsurface monitoring data. They also highlight how numerical simulations can be used to identify local deviations and potential errors in the data acquisition process.

Finally, Fig. 20 shows the width parameters of the greenfield subsurface settlement transverse trough, as determined at the Rome Metro C test site following the excavation and simulation of the first tunnel (North). These are compared with data from other case studies of tunnelling in fine-grained soils, as well as centrifuge test data (Mair et al., 1993). The K parameter values obtained for the Rome test site are consistent with other field data. The relationship proposed by Mair et al. (1993) appears to overestimate the width parameter at depths smaller than $0.6z_0$, whereas better agreement is obtained using the equation proposed by Moh et al. (1996). The K values computed from the FE analysis are reasonably consistent with the field data, although they are closer to the upper limit of the K value range.

4. Conclusions

Monitoring data on the surface and subsurface responses to the twin-bore tunnels of Line C of the Rome underground, collected at an instrumented greenfield test site, were analysed and discussed using a 3D Finite Element (FE) model. The analyses simulated the actual layout of the two tunnels and used the same set of constitutive model parameters that had previously been shown to successfully simulate the observed behaviour of other excavation works in the same area. The parameters were calibrated using the results of a detailed geotechnical characterisation of the site. The volume loss caused at the ground surface by the passage of the first TBM was used to calibrate the global

contraction, which is the only free parameter governing the excavation procedure adopted to simulate the excavation of both tunnels. Consequently, the computed subsurface ground displacements and the ground's response to the excavation of the second tunnel could be obtained without further calibration. Comparing these with the corresponding field measurements provides an assessment of the predictive capability of the numerical procedure adopted.

The main results of this study can be summarised as follows:

- The monitoring data showed that the volume losses induced by excavating the northbound and southbound tunnels were limited to 0.3 % and 0.45 % respectively. This demonstrates the efficiency of the excavation techniques, the adequacy of the face pressure control, and the reliability of the grouting operations adopted in minimising surface settlements. The increase in volume loss observed during the excavation of the second tunnel reflects irreversible, nonlinear behaviour of the soil which underwent plastic deformations during the excavation of the first tunnel. This also led to: (i) asymmetry of the settlement troughs after excavation of both tunnels, the northern half being larger than the southern half by approximately 19 %, (ii) a greater volume of soil affected by the passage of TBM2 than by TBM1, and (iii) an inward horizontal displacement field surrounding both tunnels. These findings are consistent with observations from other major urban tunnelling projects.
- The three-dimensional FE analyses, based on the *HSsmall* soil model, satisfactorily reproduced the main features of the observed ground response. The numerical results captured the magnitude and the shape of the surface and subsurface displacement profiles, thus validating the adopted constitutive parameters that had been independently calibrated from laboratory and in-situ tests. Although the lack of a reliable estimate of the maximum deep settlement has limited a meaningful comparison between measured and predicted displacement profiles after the passage of both TBMs, the good

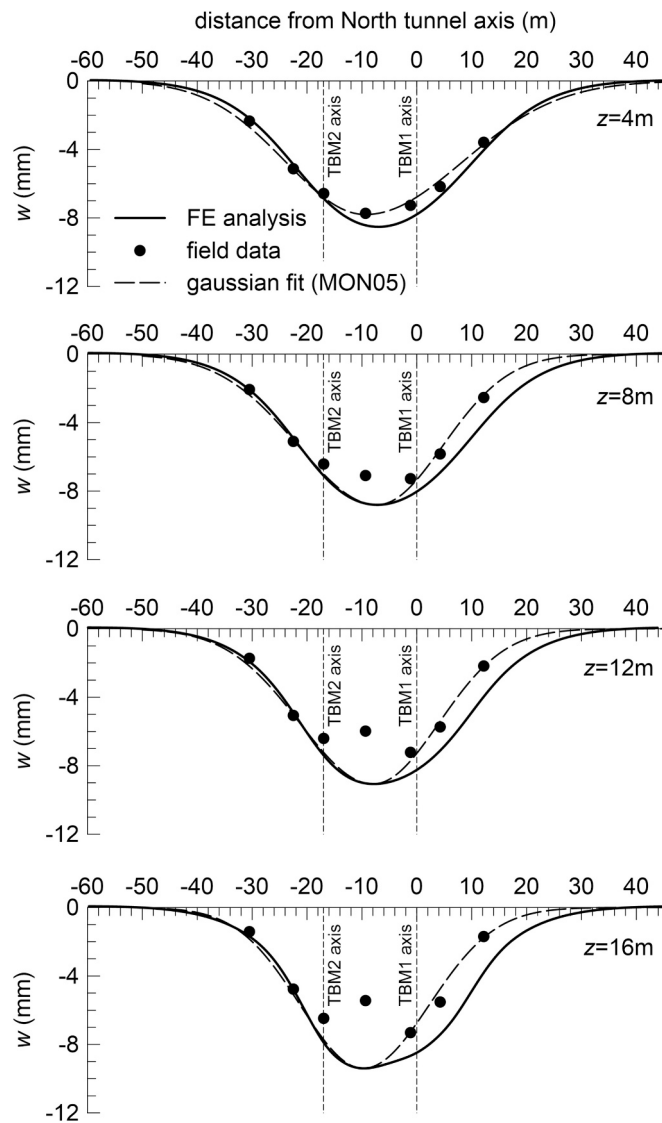


Fig. 18. Transversal subsurface settlement troughs induced by southbound tunnel excavation at different depths below MON05 alignment.

Table 6

Characteristics of the subsurface vertical displacement transverse trough along the MON05 station after the second tunnel excavation. Southern half.

depth (m)	trough width factor i_y (m)		trough width parameter K (—)		volume loss V_L (%)	
	mon. data	FE val.	mon. data	FE val.	mon. data	FE val.
4	13.6	14.4	0.62	0.65	0.38	0.44
8	14.4	13.8	0.80	0.76	0.45	0.44
12	13.4	12.8	0.95	0.91	0.43	0.42
16	11.5	11.4	1.14	1.13	0.38	0.38

NOTE: For $z > 4$ m, the maximum settlement was constrained to the FE value due to potential errors in the Trivec TR04 dataset.

agreement observed at ground surface confirms the effectiveness of the adopted procedure (Broere and Brinkgreve, 2002) in reproducing the progressive TBM advancement and the associated stress release mechanisms, including shield tapering, face pressure variation, and tail void grouting.

Table 7

Characteristics of the subsurface vertical displacement transverse trough along the MON05 station after the second tunnel excavation. Northern half.

depth (m)	trough width factor i_y (m)		trough width parameter K (—)		volume loss V_L (%)	
	mon. data	FE val.	mon. data	FE val.	mon. data	FE val.
4	17.7	15.5	0.80	0.70	0.49	0.47
8	12.0	15.2	0.66	0.84	0.38	0.48
12	12.0	15.4	0.85	1.09	0.39	0.50
16	12.0	15.6	1.19	1.55	0.40	0.55

NOTE: For $z > 4$ m, the maximum settlement was constrained to the FE value due to potential errors in the Trivec TR04 dataset.

- At the same time, comparing the computed and measured displacement fields has been instrumental in identifying potential sources of uncertainty in the monitoring data, particularly with regard to horizontal displacement profiles and local measurement inconsistencies. This emphasises the complementary nature of high-quality field monitoring and advanced numerical modelling: the former provides an invaluable benchmark for validation, while the latter allow for a more in-depth analysis of ground–structure interaction mechanisms that cannot be observed directly.
- Subsurface transverse settlement half-troughs were found to be essentially Gaussian in shape. Volume losses of approximately 0.3 % were observed at different levels above the tunnel crown after the passage of TBM1. However, the estimation of volume loss for TBM2 is subject to greater uncertainty, likely due to potential errors in measuring the maximum deep settlement. The derived values range from 0.40 % to 0.49 %, whereas the FE analyses provide slightly higher values (0.47 %–0.55 %). The width parameter K of the greenfield subsurface transverse settlement trough at the Metro C test site, derived after the first (North) tunnel was excavated, falls within the range of values reported in the literature for tunnelling in fine-grained soils, including centrifuge tests. The relationship proposed by Moh et al. (1996) closely agrees with the observations when using $K = 0.45$ and $m = 0.4$. The FE results are consistent with the field data, although they are generally positioned towards the upper bound of the reported range.
- From a broader perspective, the results of this research contribute to the refinement of predictive tools for similar projects. Integrating detailed site characterisation, continuous field monitoring, and calibrated numerical analyses has proven to be an effective methodology for both the design and control phases of mechanised tunnelling operations.
- Overall, the study confirms that EPBM tunnelling can be safely performed in complex and sensitive urban contexts when the geotechnical site conditions are accurately defined and the construction process is adequately controlled. Furthermore, the ability of the 3D numerical model to replicate the observed performance highlights its potential as a predictive tool for evaluating future tunnelling projects, optimising design parameters and supporting decision-making processes that aim to mitigate the impact of underground construction on existing structures and monuments.

CRedit authorship contribution statement

L. Masini: Writing – original draft, Methodology, Investigation, Conceptualization. **F. D’Annunzio:** Methodology, Investigation, Conceptualization. **S. Rampello:** Writing – review & editing, Supervision, Project administration, Methodology, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial

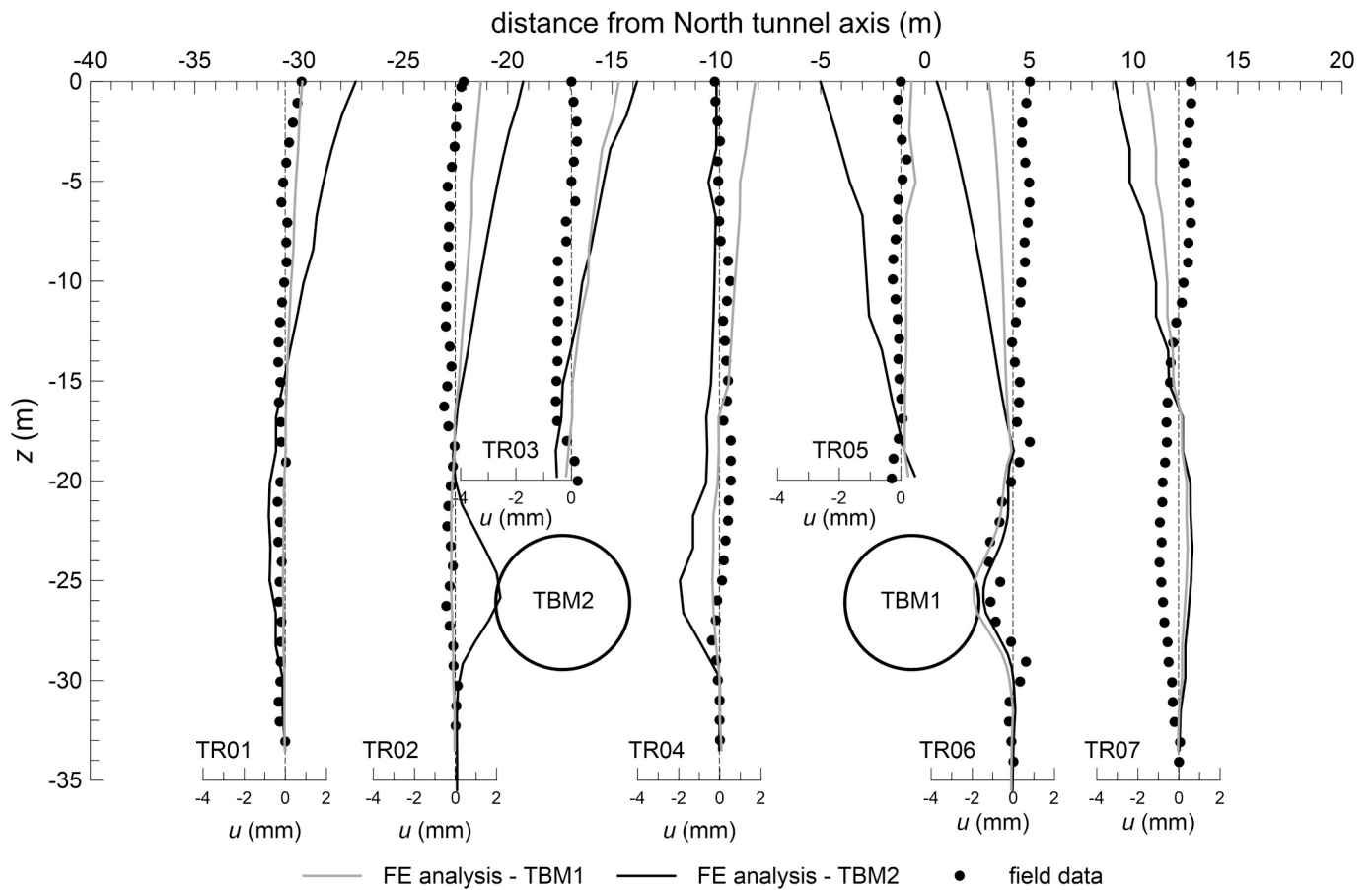


Fig. 19. Horizontal displacements provided by Trivec casings at the end of southbound tunnel excavation (positive u is consistent with the positive direction of the x -axis).

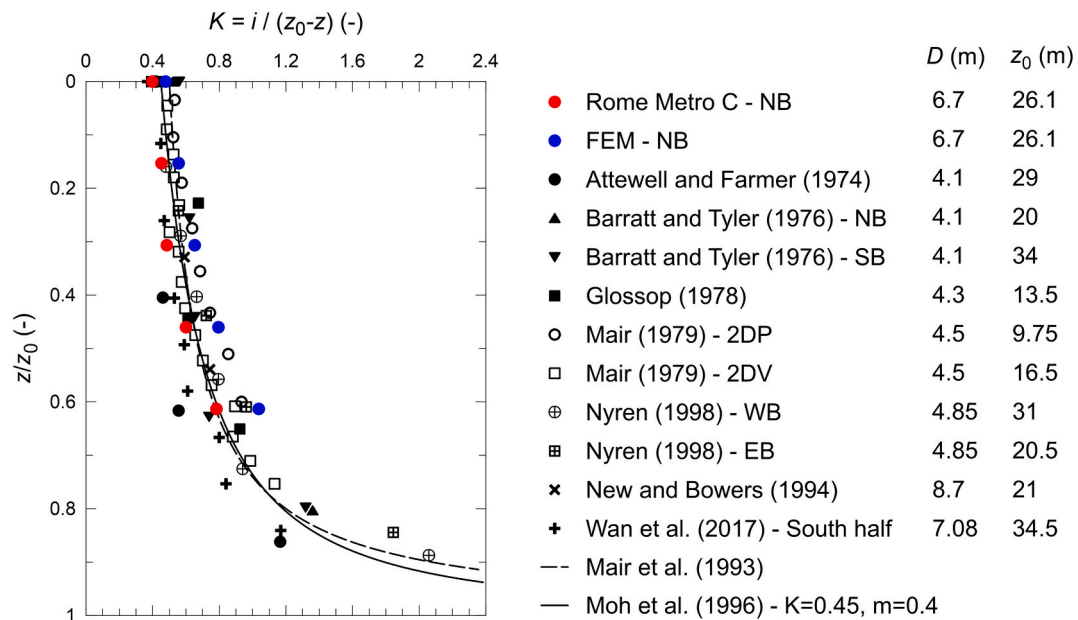


Fig. 20. Transverse trough width parameter K for the greenfield subsurface settlement trough, computed in fine-grained soils, plotted against the non-dimensional depth z/z_0 (D = tunnel diameter; z_0 = depth of tunnel axis; NB = northbound; SB = southbound; EB = eastbound; WB = westbound; 2DP = centrifuge model test 2DP; 2DV = centrifuge model test 2DV).“.

interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

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