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On the effects of Line 6 tunnel excavation in Naples

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ABSTRACT: The ever increasing demand for new transport routes led to the need of improving the current Naples underground system. It is actually composed by 7 lines connecting most of the suburban areas of the Neapolitan environment to the city center. Line 6 is one of the under construction lines of the system. Although the digging operations are still in progress, the line is already operating among Mostra and Mergellina Stations. Nowadays a 3 km tunnel, from Mergellina to Municipio Station is missing. This paper aims to discuss the tunnelling induced effects of the upcoming part of the line. A number of Finite Element Analysis has been performed with Plaxis 3D Tunnel, as to predict the tunnelling induced displacements field and the role played by both soil and machine parameters on it. The monitoring data will then be compared with the numerical analysis results in order to verify and validate them.

1 INTRODUCTION

The control of ground movements induced by tunnel excavation in urban areas is a key issue of concern for designers and many techniques are used in practice to minimize the effect of the excavation on existing buildings. This is particularly important when historical sites are affected by underground works and the limits on allowable movements are very onerous. Therefore the analysis of induced tunnelling displacements on ground surface and at depth is required.

Empirical methods have been developed on the basis of experimental data as to define the displacements field induced by tunnelling in greenfield hypothesis. However, numerical analyses can be used when more complex situations have to be analyzed.

Several Finite Element analyses have been performed, by Plaxis 3D Tunnel, in reference to the Line 6 tunnel in Naples in order to understand the soil behaviour and to back-analyze the observed induced displacements.

On the basis of collected monitoring data concerning the ground and the buildings displacements, as the machine applied pressures (front and grout pressure), the real tunnel excavation process has numerically been simulated by means of 13 construction stages. Moreover, parametric studies on soils parameters and machine pressures have been performed in order to observe their influence on induced displacements field, in terms of volume loss and maximum settlement at the ground surface.

1.1 *The Line 6*

Line 6 is a 6 km long line linking the Northern suburb of Fuorigrotta (Mostra Station) to the historic city center (Municpio Station), passing through one of the most urbanized areas of the city (Riviera di Chiaia). Nowadays the initial part of the line, between Mostra and Mergellina Stations, is already working, whereas 3 km of tunnel between Mergellina Station and Municipio Station have to be excavated as to finalize the line (Fig. 1).

The stratigraphic sequence featuring the area where the tunnel has to be excavated is made by an upper loose soils layer resting on a Yellow Neapolitan Tuff bedrock. The tunnel passes mainly through the loose soils layer, within 11 m up to 20 m depth and it is always located above the groundwater level (3 m a.m.s.l.).

An 8.15 m diameter Earth Pressure Balance (EPB) machine has been chosen to excavate the tunnel, while the final lining is made by (1.50 × 1.70 × 0.30) m segments made of RcK 45 MPa concrete.

Several investigation campaigns performed all over the years led to define the main geotechnical parameters for the soils affected by the tunnel excavation, as resumed in Table 1.

To supervise and control the tunnelling induced displacements, several monitoring instruments have been installed on the ground surface transversely to the tunnel track, and on preexisting buildings ‘facades.

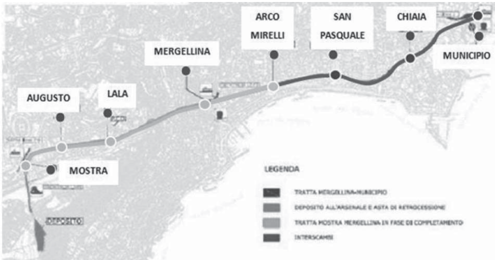

Figure 1. Line 6 plan.

Table 1. Geotechnical parameters.

	γ	c	φ	ψ	E
Soil layers	[kN/m³]	[kPa]	[°]	[°]	[MPa]
Loose soils	16	0	35	0	50
Tuff	14	500	27	0	1000

2 LINE 6: MONITORING DATA

2.1 *Tunnel excavation*

Up to August the 31st, 423 m of tunnel have been excavated and 250 lining segments have been installed within Mergellina Station and the under construction Arco Mirelli Station. Because of technological problems, the tunnel production has been interrupted for several days between May and June (Fig. 2). The best tunnel production has been reached in May, when almost 150 m of tunnel have been excavated and 75 lining segments have been installed.

Thanks to a number of sensors on the EPB machine, measurements of front pressure (FP), grouting pressure (GP) and torque moments (TM), applied for advancing with the tunnel excavation, have been collected. Figure 3 shows their trend versus the tunnel advance production. The front pressure is always linearly increasing with the tunnel advance, whereas both the grouting pressure and the torque moments have a non uniform trend. For the grouting pressure peak values have been recorded between June the 18th and July the 28th when electrical problems with the cutterhead and water inflow from the front have been encountered. In the same interval an instantaneous 50% reduction of the torque moments has been measured.

2.2 *Ground and buildings measured displacements*

Up to August the 31st, 45 ground landmarks sections have been installed along Via Piedigrotta, transversely to the tunnel track.

Looking at the maximum measured displacements, 3 zones can be identified: zone A (w_{max} = 14 mm), zone B (w_{max} = 8 mm) and zone C (w_{max} = 5 mm), as shown in Figure 4.

Zone A mainly includes landmarks sections measuring the largest displacements, zone B features the middle part of the monitored stretch and zone C shows the smaller measured displacements.

Measured displacements, versus front-section distance, have been plotted in Figure 5, whereas Figure 6 depicts such displacements at the tunnel front and at the passage of tail of shield. According to indications from Craig & Muir Wood (1978), for unsupported tunnel front in sandy soils, an average of 20% of the maximum displacements has been measured at the tunnel front and of 80% at passage of tail of shield.

Regarding the collected buildings monitoring data, up to August the 31st, 29 buildings have been monitored by means of several landmarks installed on their facades. The maximum measured settlement is 8 mm for building n°23 along Riviera di Chiaia (Fig. 7), while the average displacement is 5 mm.

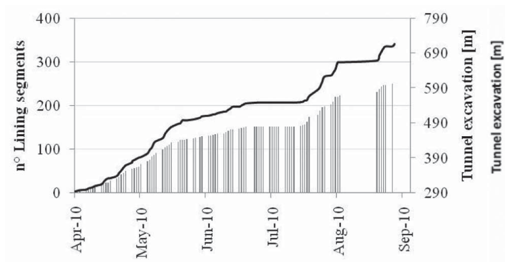

Figure 2. Tunnel advancing up to August the 31st.

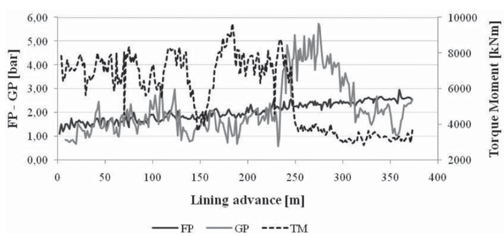

Figure 3. EPB measured pressures and torque moment.

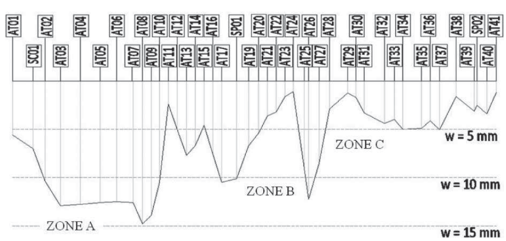

Figure 4. Landmarks on the ground surface: Maximum measured displacements in longitudinal profile.

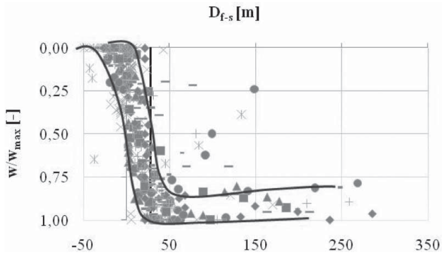

Figure 5. Maximum displacements in longitudinal direction.

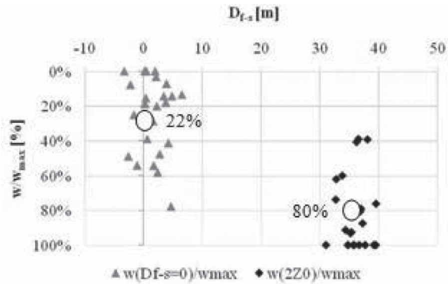

Figure 6. Maximum surface settlements at the tunnel front and at the passage of tail of shield.

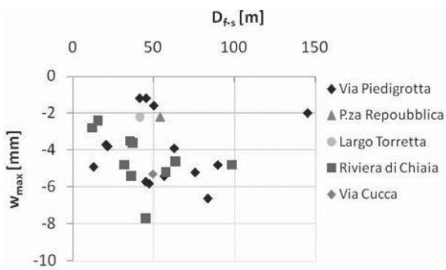

Figure 7. Maximum buildings settlements.

3 NUMERICAL ANALYSES

3.1 *The input model*

Several three-dimensional analyses have been performed by means of Plaxis 3D Tunnel, as to analyze the tunnelling induced soil behaviour.

Parametric studies have been carried on in order to observe the input soil parameters and machine pressures on the effects observed on the ground surface, hence to back-analyze the monitored effects induced by the Line 6 tunnel excavation. The monitoring data have been updated to August the 31st, and then the Mergellina-Arco Mirelli stretch has been back-analyzed.

The plane section shown in Figure 8, has been chosen along Via Piedigrotta as to perform the FE analyses. The tunnel mainly crosses the loose soil layer resting on the Tuff bedrock, 20 m deep, and is located 11 m below the groundwater table. The greenfield conditions have been considered for the analyses, then a symmetric section has been used, as to reduce the computation time.

To prevent the border effects influence on the excavation process, a (5D × 5D) size section has been defined. The number of planes in z-direction has been fixed as to reproduce at best the monitored excavation process day by day; in such a way a 125 m length model in z-direction has been constructed. The resulted 3D model is then featured by 44268 quadratic 15-node wedge elements and 122688 nodes.

Since the tunnel excavation affects at the most the upper layer made by sandy soils, "*drained*" analyses have been performed.

To describe soil and structures (EPB shield and concrete final lining) behavior, the Hardening Soil Model has been used for the former and the Linear Elastic Model for the latter; Table 2 resumes the used soils properties.

As the tunnel excavation implies recasting in soil properties, the model of soil-tunnel interaction has been required. Hence, soils materials with reduced strength parameters have been defined by means of the interface parameter R_{interf} set lower than 1.

3.2 *Parametric analyses on soil parameters*

In order to analyze the soils parameters influence on the surface effects induced by tunnelling, in terms

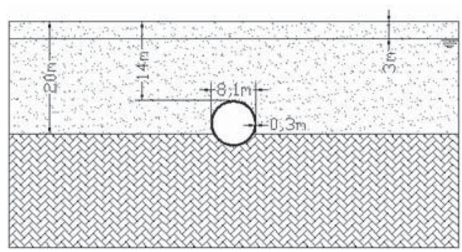

Figure 8. Plane section for 3D FE analyses.

Table 2. Geotechnical parameters of soils layers.

Soil layers		Loose soil	Tuff
γ_d	[kN/m³]	13	14
γ_{sat}	[kN/m³]	18	19
k	[m/sec]	$1e^{-4}$	$1e^{-4}$
ν	[–]	0.3	0.3
c	[kPa]	0	500
φ	[°]	35	27
ψ	[°]	0	0
E_{ed}	[kPa]	$6.73e^4$	$1.35e^6$
E_{ur}	[kPa]	$2.02e^5$	$4.04e^6$

of V_L and w_{max}, a number of FE analyses have been performed by varying soil strength and stiffness parameters within reliable ranges of values. As the deep Tuff layer did not reveal any influence on the surface displacements, a simplified homogeneous model, made only by loose soils, has been used. The soil parameters resumed in Table 2 have been used for the reference analysis, whereas Table 3 describes their ranges of variations. For all the above discussed analysis, the machine front and grout pressures have been kept constant (FP = 160 kPa, GP = 200 kPa).

Despite volume loss controlled analyses are possible with the used code, not any contraction has been imposed in this case, in order to observe the volume loss at the tunnel depth ($V_{L,d}$) changes, according to different soil parameters.

Results from the performed analyses showed a clear influence of the soil stiffness on $V_{L,d}$: the larger is the Young's modulus, the smaller is $V_{L,d}$, passing from 0.20% for $E = 1/3*E_{ref}$ to 0.02% for $E = 3*E_{ref}$. Regarding the surface effects in terms of volume loss (V_L), maximum settlement (w_{max}) and subsidence width (k), a great influence of the stiffness moduli on V_L and w_{max} (Figs. 9 and 10) has been noticed, whereas the stiffness moduli ratio $P = E_{ur}/E$ affects

Table 3. Ranges of variations for soil parameters.

γ	φ	ψ	E	$P = E_{ur}/E$
(±20%)	(–40÷0)%	(0÷10)°	$(1/3÷3)E_{ref}$	(3÷12)

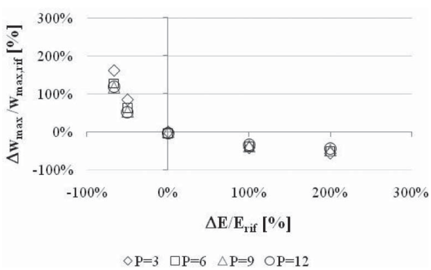

Figure 9. Effect of stiffness modulus variations on the maximum settlement on the ground surface.

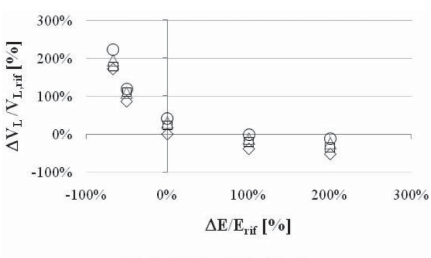

Figure 10. Effect of stiffness modulus variations on the volume loss on the ground surface.

the subsidence width by leading to its extension up to 50% when it increases from 3 up to 12 (Fig. 11).

Clear relationships can be as well detected between γ and the effects on the ground surface. For three reference planes, at the tunnel front (Plane O), 2D (Plane U) and 3D (Plane AE) forward, an almost linear relation between γ, V_L and w_{max} has been detected (Figs. 12 and 13). Reductions, rather than increments, of γ up to 20%, leads to much more pronounced variations of k (–80%) (Fig. 13).

At the same time, not any particular relationship has been noticed between the friction angle (φ) and the dilatancy angle (ψ) with the volume loss and the maximum settlement on the ground surface. Nevertheless, a linear increase of the subsidence width up to 25% has been noticed for reduction of φ up to 40%. Conversely, not any effects on the

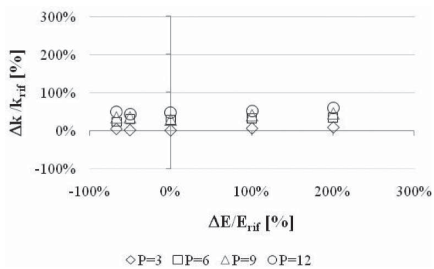

Figure 11. Effect of stiffness modulus variations on the subsidence width.

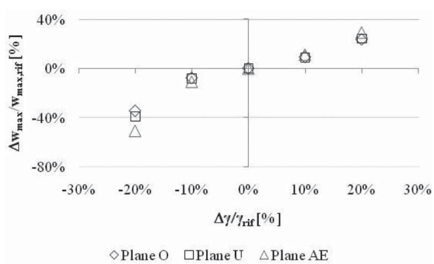

Figure 12. Effect of γ variations on the subsidence width. On the maximum settlement on the ground surface.

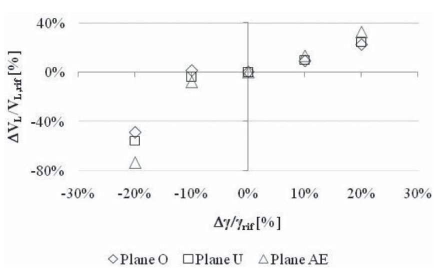

Figure 13. Effect of γ variations on the volume loss on the ground surface.

subsidence width has been detected for ψ increments less than 15%, from which an instantaneous enlargement of the surface subsidence basin, up to 20%, has been observed.

3.3 *Parametric analyses on EPB pressures*

When using and EPB machine in tunnelling, the effects on the ground surface are strictly dependant by the applied front and grout pressure.

Several FE analyses have been performed in order to identify any possible link between the applied pressures and the induced effects on the ground surface (V_L and w_{max}). Since such pressures concern also the Tuff layer, the two layered geometric shown in Figure 9 has been used.

The constitutive law describing the soil behaviour are the same as previously described, whereas volume loss controlled analyses have been performed, by imposing a 0.1% contraction at the tunnel depth.

Both front and grouting pressure have been varied within 100 kPa and 500 kPa, and 48 analyses have been carried on as to investigate their effects on the ground surface. Results have been plotted referring to a "reference" analysis where both front and grout pressures have been set equal to 100 kPa.

Looking at the front pressure (Figs. 16 and 17), beneficial effects in reducing the surface displacements (up to 30%) have been noticed, for pressures within a range of values preventing soil plastification. Indeed, results showed:

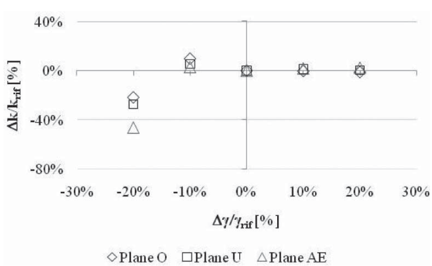

Figure 14. Effect of γ variations on the subsidence width.

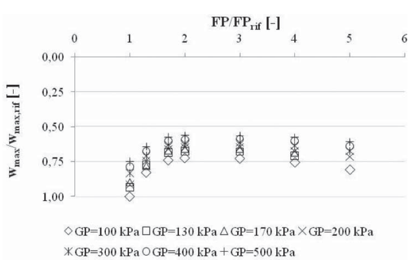

Figure 15. Front and grouting pressure effects on surface maximum settlements.

- face plastification, caused by large horizontal displacements toward the excavated tunnel, generate the lowest front pressure. This leads to the development of an active failure mechanism and of large surface settlements. Moreover, for front pressure within 100 kPa and 200 kPa, the larger is the applied pressure, the smaller are the V_L and w_{max}.
- soil plastification at the tunnel contour for front pressure exceeding 200 kPa. New increment in w_{max} and V_L developed together with horizontal displacements ahead the tunnel front, leading to a passive failure mechanism.

Concerning the grout pressure effects o the ground displacements, Figure 17 and Figure 18 show a linear reduction of both w_{max} and V_L for increasing grout pressure.

By comparing the front and the grout pressure influence, Figure 19 and Figure 20 show the

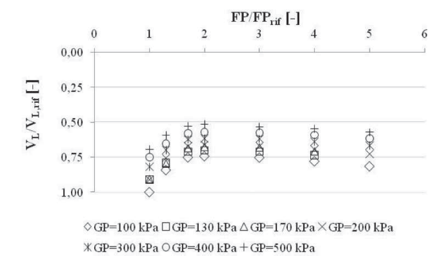

Figure 16. Front and grouting pressure effects on surface volume loss.

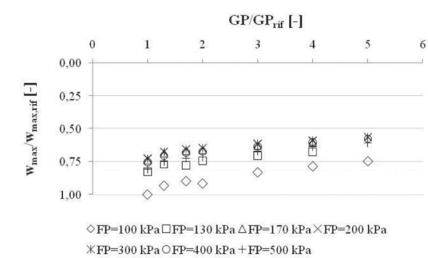

Figure 17. Front and grouting pressure effects on surface maximum settlements in dimensionless graph.

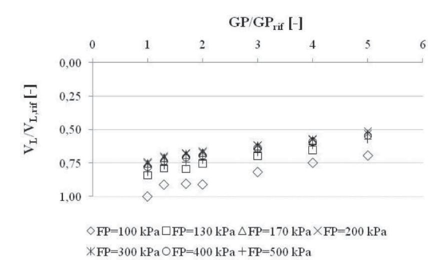

Figure 18. Front and grouting pressure effects on surface volume loss in dimensionless graph.

initial predominant effect of the front pressure up to 200 kPa (corresponding to 100% increment of the reference pressure).

Looking at the grout over the front pressure ratio effects on surface displacements, Figure 21 and Figure 22 show a sort of linear relationship in a range of values defined by the minimum and maximum front pressure values.

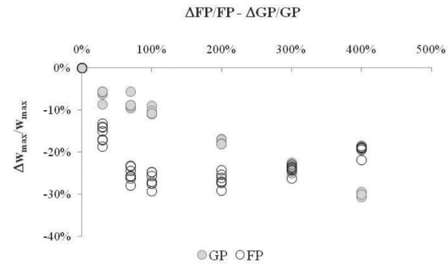

Figure 19. Comparison between the grout pressure (GP) and front pressure (FP) effects on maximum surface settlement.

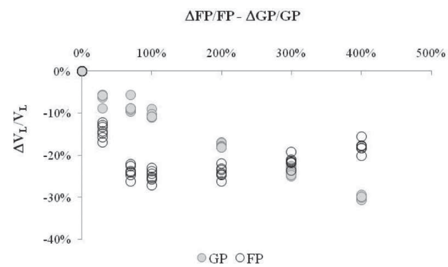

Figure 20. Comparison between the grout pressure (GP) and front pressure (FP) effects on surface volume loss.

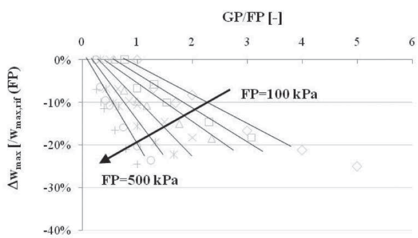

Figure 21. Influence of the grouting than the front pressure ratio on surface maximum settlement.

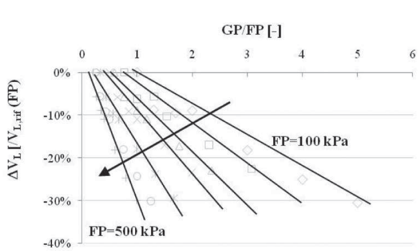

Figure 22. Influence of the grouting than the front pressure ratio on surface volume loss.

4 LINE 6: BACK-ANALYSIS OF MONITORED EFFECTS

In order to perform the Line 6 tunnelling back-analysis, the monitored construction process has been simulated and the volume loss or contraction method has been used. 13 calculation phases have been defined as to reproduce about 100 m of excavated tunnel. In each phase the same procedure to simulate the tunnel construction process has been repeated by:

- deactivating the soil clusters inside the tunnel lining.
- activating the shell elements representing the EPB shield behind the tunnel face and draining the water within it.
- applying the concrete properties to the clusters representing the installed final lining.
- applying the face and the grout pressure to prevent the tunnel collapse, and the forces the hydraulic jack driving the EPB machine, exert on the already installed lining.
- modelling the soil-tunnel interaction by assigning modified properties, with reduced strength parameters (by means of $R_{interf} < 1$), to the slices where between the tunnel front and the last lining segment.
- defining a tunnel contraction to simulate volume loss.

To reduce the influence of boundary conditions on the excavation process, 25 m of tunnel have been excavated at the start of the model and the induced tunnelling displacements have been set to zero. Both the front and grout pressure linearly increase with depth; 14 kPa and 20 kPa gradients (depending on the bentonite specific weight and on the mixture for the mortar injections) have then been used for the front and the grout pressure. On the contrary, as the hydraulic jacks' pressure on the final lining, related to the applied front pressure, is not depth dependent, a constant value has been defined.

In order to define the input values for the EPB pressures to apply in the analyses, on the basis of monitored EPB performances, the average value of measured front and grout pressures for each landmarks zone (A, B and C), has been estimated as resumed in Table 4.

On the contrary, no monitoring data about the volume loss, neither at the ground surface nor at the tunnel depth, is available. A 0.1% volume loss has then been considered as an initial reliable value

Table 4. Input EPB pressures in the FE analyses.

	Zone A	Zone B	Zone C
FE analyses	kPa	kPa	kPa
Front Pressure	160	190	240
Grout Pressure	150	150	370
Jacks Pressure	878	999	1202

Table 5. Input $V_{L,d}$ at the tunnel depth and resulting V_L at the ground surface.

FE analyses	Zone A	Zone B	Zone C
$V_{L,d}$ (Input)	0.25%	0.15%	0.10%
V_L (Output)	0.52%	0.34%	0.21%

to impose at the tunnel depth. Nevertheless, since the effects to be reproduced are significantly different from each other, on the basis of assumptions on soils behaviour and parameters, on geometric model and on applied pressures, different values for the volume loss at the tunnel depth have been needed. Table 5 resumes the required volume loss at the tunnel depth, and the consequent V_L at the ground surface, resulting always twice the imposed ones.

The analyses accuracy has been confirmed comparing the analyses results to the measured displacements both in transversal (Figs. 23, 24, 25) and longitudinal direction (Fig. 26).

5 CONCLUSION

Tunnelling in urban areas is of increasing importance over the past few decades. The prediction and the analyses of the induced effects, by means of analytical, numerical or experimental methods, play an even more important role, in order to prevent damage to the overlying buildings and services. Literature data reveal that the role of the excavation technique is far from negligible and a significant reduction in the induced effects due to the excavation techniques improvement over the years has been revealed.

Numerical analyses, by means of FE codes, represent a useful tool to account for the excavation method in the design process, since they enable the whole excavation process. In this context, the objective of this thesis has been to assess the influence of soil parameters and of technological factors on tunnelling induced displacements, thus several FE analyses, by means of Plaxis 3D Tunnel have been performed. Results have been used to back-analyze at the best the monitored surface effects induced by the Line 6 tunnel excavation.

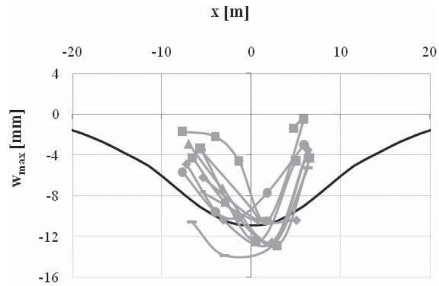

Figure 23. Transversal surface subsidence obtained by the back-analyses for Zone A.

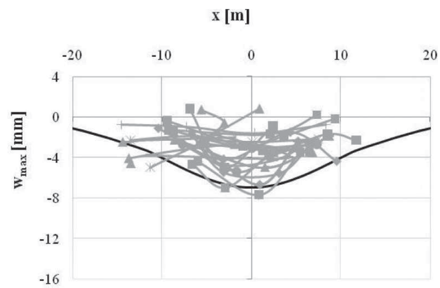

Figure 24. Transversal surface subsidence obtained by the back-analyses for Zone B.

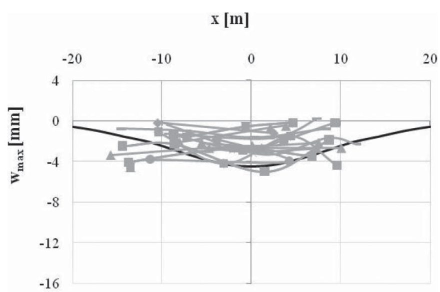

Figure 25. Transversal surface subsidence obtained by the back-analyses for Zone C.

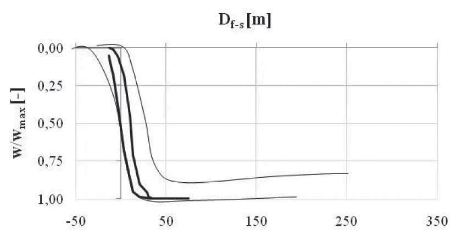

Figure 26. Longitudinal displacements obtained by the back-analyses compared to the range defined by collected monitoring data.

Looking at the soil parameters influence, main results of the parametric studies can be summarized in the following:

- the soil stiffness mainly affects V_L and w_{max} at the ground surface and influences the subsidence width (k) in a lesser degree: increasing the soil stiffness the subsidence becomes wider
- the soil stiffness moduli ratio does not significantly affects V_L and w_{max} but the subsidence width: the larger is such ratio, the larger is k
- linear relationship has been detected between γ, V_L and $w_{max;}$ however reduction of γ yields a shrinkage of the subsidence basin
- no clear relationships have been detected between φ and ψ with the surface displacements; conversely, reductions in φ lead to a sort of linear increase in k, whereas the dilatancy effect is evident only above a certain value when the subsidence width suddenly increases.

Regarding the EPB pressures effects, the main results can be resumed as follows:

- clear relationships between the applied pressures and the effects on the ground surface can be defined
- the positive effect of the front pressure on the induced displacements is evident up to a certain value, related to the front equilibrium, after which further increases leads to plasticization around the tunnel lining implying later growing in settlements and volume loss
- the face pressure governs the tunnel front behaviour leading to active or passive failure mechanism, thus strongly affecting the displacements at the ground surface
- the face pressure influence on both the volume loss and the maximum settlement at the ground surface, is much more pronounced than the grouting pressure, up to a certain value, when the observed behaviour changes and the grouting pressure becomes more influent in reducing the tunnelling induced effects
- looking at the pressures ratio, linear relationships with both the volume loss and settlement variations have been detected within a range defined by the minimum and maximum front pressure used.

As general conclusion, it seems reasonable to state that the design of tunnel cannot neglect (as usually done) technological factors, which have a strong influence on the tunnelling induced settlement.

In other words, good soil modelling coupled with advances numerical analyses, could give wrong results if technological aspects (grouting and front pressures) are not well considered at design stage and not well implemented at construction stage. A good design is based on a "right" balance of all above.

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