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Ground movements induced by tunnel boring in Naples

E. Bilotta & G. Russo
University of Napoli Federico II, Naples, Italy

ABSTRACT: Major underground works have been carried out in the city of Naples (Italy) in the last two decades to enhance the public transport system and they are still in progress. Very densely urbanized areas of the city were interested, including historical and monumental buildings. Therefore, prediction and control of ground movements have been issues of great concern for designers and contractors. The paper refers about the ground movements measured during the excavation of tunnels of stretches belonging to Line 1 and Line 6 of the underground railway system. The collected data confirmed that the use of tunnel boring machines has nowadays largely reduced the level of ground loss during excavation and, consequently, the concern for possible interaction with the existing buildings.

1 INTRODUCTION

The ground movements induced by tunnel excavation are mainly generated by the larger volume of soil removed during operations compared to the volume occupied by the tunnel. This 'loss' of volume occurs at the front of the excavation ('face loss') and around the lining in radial direction ('radial loss'): both these components can be controlled during mechanised tunnelling. By using a Tunnel Boring Machine the face loss can be largely reduced by pressurising the front while the radial loss can be controlled by the volume and pressure of the backfilling grout. As a matter of fact, these parameters are also used as input value for advanced numerical predictions of TBM-induced ground movements (Koelewijn & Verruijt, 2001; Moller & Vermeer, 2008).

During the construction of new stretches of the underground in Naples, the ground movements were monitored for two main reasons: (i) controlling the construction processes and (ii) accumulating experimental data for the advancement of the knowledge on the topic. Some of the collected data are reported in this paper and discussed also in the light of the key parameters of the mechanised excavation (Bakker & Bezuijen, 2009), which were routinely controlled during the TBM driving procedures.

2 UNDERGROUND WORKS

2.1 Line 1 extension

The lower stretch of the Naples Metro Line 1 was constructed underground by boring two parallel tunnels at variable depth in various ground conditions (Fig. 1). From the intake located in a shaft at about 17 m below the ground level, the tunnels were bored in alluvial sands and silty sands, below the groundwater table (typically a few metres below the ground level). At 1 km far from the intake the two tunnels enter a soft rock formation (Yellow Tuff). The paper deals with the measurements carried out in the first part of the line where the two tunnels were excavated by two earth pressure balance shields ($D = 6.74$ m) through sandy soils and installing a precast r.c. segmental lining ($5.85$ m ID, $6.45$ m OD).

The average values of the properties of the sandy soil present in the first part of the line are summarized in Table 1.

2.2 Line 6 extension

The line 6 was partially constructed several years ago and a new long stretch is under construction. The terminal part of the existing line and the beginning of the new stretch are mainly excavated through sandy soils (Fig. 2).

The shallower layer is usually constituted by antropic fills, with thicknesses that reach up to
ten meters. The groundwater table is a few meters below surface. Both pyroclastic soils and landfills are characterized by medium-high density and high permeability. The sandy layers are limited at the bottom by the tuff formation. The average values of the main soil properties are shown in Table 2.

In the paper displacements obtained by two monitoring sections in the terminal part of the existing line will be presented and discussed. The stretch was bored by means of a hydroshield ($D = 9.25$ m) at a depth almost constant around $12$ m bgl (axis depth, $z_0$), and lined by bolted rings of precast reinforced concrete segments. In the area the groundwater table is found around $7$ meter deep, that is on average at the roof of the tunnel.

### 3 SOIL MONITORING DURING EXCAVATION

#### 3.1 Line 1 monitoring set-up

The lower stretch of line 1 approaching the Garibaldi Station from the intake shaft located about $1$ km away was partly (about $400$ m) excavated in soft ground (sand) and partly excavated in the upper part of the underlying tuff formation, under a cover of overburden layers of sand (Fig. 1). Due to the presence in the area of several infrastructures, as the highway entrance and a regional railway embankment, and several buildings, among which the city main railway station, particular care was dedicated to control the ground movements induced by the excavation. Survey of ground surface was performed by levelling optical landmarks regularly deployed along the tunnel longitudinal axes. Moreover a number of transverse sections were equipped for measuring the ground movements both at surface and in depth. A plan view of the line with the traces of the fifteen monitored sections is shown in Figure 3.

All the fifteen sections were provided with arrays of landmarks for measuring settlements at the ground level. In some of them also vertical tubes were installed with inclinometer casing and magnetic ring targets for measuring deep vertical and horizontal displacements by means of inclinometer and extensometer probes. A sketch of a typical monitoring section is shown in Figure 4: A, B, and C are the verticals along which both settlements and horizontal displacements were measured while S and N are centered on the tunnel vertical axes and for this reason only settlements were measured. In Table 3 the distance of the tubes A, B and C from the vertical axis of the tunnels are reported for each section.

#### 3.2 Line 6 monitoring set-up

The extension of Line 6 under construction is being monitored similarly to Line 1 for both surface and deep ground movements. These data are still being collected and they will not be available for an overall interpretation before the works will be completed.

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Table 1. Average values of geotechnical parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight, $\gamma_d$ [kN/m$^3$]</td>
<td>15</td>
</tr>
<tr>
<td>Saturated unit weight, $\gamma_{sat}$ [kN/m$^3$]</td>
<td>19</td>
</tr>
<tr>
<td>Permeability, $k$ [m/s]</td>
<td>$10^{-6}$–$10^{-7}$</td>
</tr>
<tr>
<td>Cohesion, $c'$ [kPa]</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle, $\phi'$</td>
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</tr>
<tr>
<td>Small strain shear modulus, $G_o$ [MPa]</td>
<td>$10 + 5z$ ($z$ = depth)</td>
</tr>
<tr>
<td>Earth pressure coefficient, $K_o$</td>
<td>0.4–0.5</td>
</tr>
</tbody>
</table>

Table 2. Average values of geotechnical parameters.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Sand</th>
<th>Pyroclastic soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight, $\gamma_d$ [kN/m$^3$]</td>
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<td>14</td>
</tr>
<tr>
<td>Saturated unit weight, $\gamma_{sat}$ [kN/m$^3$]</td>
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<td>16</td>
</tr>
<tr>
<td>Permeability, $k$ [m/s]</td>
<td>$10^{-6}$–$10^{-7}$</td>
<td>$10^{-6}$–$10^{-7}$</td>
</tr>
<tr>
<td>Cohesion, $c'$ [kPa]</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Friction angle, $\phi'$</td>
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<td>36°</td>
</tr>
<tr>
<td>Small strain shear modulus, $G_o$ [MPa]</td>
<td>$10 + 5z$ ($z$ = depth)</td>
<td>$10 + 5z$ ($z$ = depth)</td>
</tr>
<tr>
<td>Earth pressure coefficient, $K_o$</td>
<td>0.4–0.5</td>
<td>0.4–0.5</td>
</tr>
</tbody>
</table>

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Figure 2. Ground conditions of Line 6.

Figure 3. Plan view of the monitored stretch of Line 1.
Table 3. Distance of tubes from the tunnel axis.

<table>
<thead>
<tr>
<th>Section</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_s$; $b_s$ (m)</td>
<td>8.1</td>
<td>5.8</td>
<td>6.6</td>
<td>6.5</td>
<td>8.1</td>
</tr>
<tr>
<td>$b_n$; $c_n$ (m)</td>
<td>8.0</td>
<td>8.5</td>
<td>8.4</td>
<td>8.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

Two monitoring sections belonging to the terminal part of the old existing stretch of the Line 6 are reported in Figure 5.

3.3 Surface settlements

Figure 6 shows the evolution of the maximum measured vertical displacement at four of the monitored sections of Line 1 during the construction of both south and north tunnels. The north tunnel was excavated after the south tunnel and for each section a new zero reading was taken before the construction of the north tunnel started. Therefore the values of surface settlement shown for the north tunnel are to be considered as increments subsequent the completion of the south tunnel. As shown in the figure, settlement started before the arrival of the excavation front to the monitored section: around 50 m before for the north tunnel and a bit less for the south tunnel.

The settlements measured along the monitoring sections after plane strain condition were reached are shown in Figures 7 and 8. In the same figures the relevant Gaussian best fits are also shown. It is worthy reminding that the widely used Gaussian curve is represented by the following equation:

$$w = w_{\text{max}} \cdot \exp\left(-\frac{x^2}{2\ell^2}\right)$$  \quad (1)

With $w_{\text{max}}$ equal to the maximum settlement above the tunnel axis, $x$ equal to the horizontal distance from the tunnel axis, $\ell$ equal to the distance of the inflexion point from the tunnel axis. The parameter $\ell$ and the maximum settlement, $w_{\text{max}}$, are further
linked by the following equation which introduces the volume loss, $V'$:

$$V' = w_{\text{max}} \cdot 4 \sqrt{\frac{2}{\pi}} \frac{i}{D}$$

(2)

The values of the parameter $V'$ range between 0.3% and 0.5%, except for the section 7 of the north tunnel where $V'$ = 1% was back-calculated. The values of the parameter $K (= i/z_0)$ are in the range between 0.3 (section 1-south tunnel) and 0.65 (section 7-north tunnel) an average value being around 0.5. The upper bound of the range is consistent with the indication of Peck (1969) for tunnels in sand below the water table.

Figure 9 shows the longitudinal profiles of the maximum settlements, $w_{\text{max}}$, measured over a span of about 80 m when the south tunnel front was at five different locations between chainages 236 m and 378 m. In the figure are also shown the fitting curves based on the integration of the normal cumulative curve (Attewell & Woodman, 1982) and assuming: $w_{\text{max}}(\infty) = 6 \text{ mm}$ according to measurements; the inflexion distance, $i_{\text{long}}$, equal to that of the Gaussian fit of the nearest monitored transverse section, $i_{\text{trasv}}$.

Although the plane strain maximum settlement, $w_{\text{max}}(\infty)$, over the considered span is the same, the settlement at TBM face is different and ranges between 25% $w_{\text{max}}(\infty)$ and 0. According to Craig (1975) this is to be expected for shielded tunnels in sand below the water table.

Similar plots are shown in Figure 10 referring to a larger span (from chainage 99 m to 513 m). The plane strain maximum settlement varies between 6 mm and 15 mm without correlation with the front settlement.

During the excavation of the stretch of Line 6 tranverse and longitudinal profiles of settlement were measured and results similar to those for Line 1 were obtained. In this case values of the parameter $V'$ equal to 0.18% (section 1) and 0.33% (section 2) and corresponding values of $K$ equal to 0.43 and 0.27 were determined by Gaussian fit of the measured settlement in the tranverse section. The settlements measured in the longitudinal direction were best-fitted by assuming $i_{\text{long}} = 2 \cdot i_{\text{trasv}}$, the front settlement at section 2 being almost negligible.

3.4 Deep displacements

The deep settlements measured along the extensometer tubes (Fig. 4 and Table 3) are shown in the following Figures 11–13. The relative settlements measured between the magnetic targets in the tubes were referred to the top settlement of the extensometer tube, which was separately determined by optical levelling.

The vertical displacements (negative = downward) are plotted along the elevation from the tunnel axis. It can be seen that the maximum settlement measured at the ground level generally increases with the tunnel depth. Furthermore this settlement tends to remain constant with the depth, at least until the tunnel section is approximately met. Suddenly the observed settlements decrease and below the tunnel axis they are very small: in some cases the displacements at that level are directed downward and in some others upward.

The vertical displacements measured in section 3 above the crown of both south and north tunnels (S-3 and N-3 in Figure 13) are substantially the same, indicating the same performance of the TBMs at that section. This is confirmed also by the corresponding measurements at the lateral tubes (A-3 and B-3 in Fig. 11 and B-3 and C-3 in Fig. 12) and by the similar values of $V'$ and $K$ back-calculated for the two tunnels at that section.

On the other hand, the settlements measured in section 1 above the tunnel crown (S-1 and N-1 in Figure 13) are significantly different. In fact the settlement trough of the north tunnel was wider and lower settlements were measured at surface...
(Figs. 7 and 8), although the same \( V' \) was back-calculated for the two tunnels at that section.

The deep settlements above the crown of both tunnels at sections 1 and 3 have been fitted by means of several Gaussian curves. It was found that the parameter \( V' \) generally increased with depth: a volume loss at the tunnel crown almost double than that back-calculated at surface was obtained for both sections. This measurement simply confirms that tunnel excavations in sandy soil is not an undrained process (as it is in clayey soil) which should imply constant volume loss from the tunnel upwards to the ground surface. Moreover, the distance from the axis of the tunnel to the inflexion point, \( i \), decreased with depth, as shown in Figure 14.

Moh et al. (1996) proposed a formula to evaluate the trough width parameter, \( i \), at a depth of \( z \):

\[
i(z) = bD \left( \frac{z_0 - z}{D} \right)^m
\]

Where \( D \) is the tunnel diameter, \( (z_0 - z) \) the elevation from the tunnel axis depth, \( z_0 \), and \( b \) and \( m \) two parameters. Following Moh et al. (1996), the exponent \( m \) was determined from the settlement profiles above the tunnel centre shown in Figure 13 as the slope of the same profiles plotted in a bi-logarithmic plane. The coefficient \( b \) was obtained by fitting the settlement trough at surface. The results are shown in Table 4.

The corresponding profile \( i (z-z_0) \) are shown in Figure 14 as fitting curves. A further profile is shown \( (m = 0.8, b = 0.7) \) as this fitted better the set of data corresponding to the south tunnel at section 3. However, in three over four cases the Eq (3),

<table>
<thead>
<tr>
<th>m (south)</th>
<th>b (south)</th>
<th>m (north)</th>
<th>b (north)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>0.1</td>
<td>0.7</td>
<td>0.2</td>
</tr>
<tr>
<td>Section 3</td>
<td>0.4</td>
<td>1</td>
<td>0.4</td>
</tr>
</tbody>
</table>
calibrated on the deep settlements above the tunnel centre and on the surface trough, enabled an accurate prediction of the profile of $i(z)$ that was back-calculated by Gaussian fitting the deep settlements measured along different vertical lines.

The horizontal (in-plane) displacements measured in the transverse sections along the inclinometer tubes (Fig. 4 and Table 3) are shown in the following Figure 15 for the south tunnel. In most cases the horizontal components of ground movements at the tunnel sides were directed towards the tunnel (positive in the figures) and a significant decrease of their magnitude was observed with the depth. The horizontal displacements below the tunnel invert are generally very small.

The direction of the vectors of total displacement above the tunnel crown level are represented in Figure 16 as a ratio between the in-plane horizontal displacements, $u_h$, and the settlement, $w$. The ratio $u_h/w$ is plotted against the ratio $x/|z_0 - z|$, indicating at each point in the ground the inclination of a vector pointing to the tunnel centre. It is worth noting that in most cases the vectors of displacement close to surface were directed toward the tunnel centre $(u_h/w = x/|z_0 - z|)$. The plot shows also that the ratio $u_h/w$ presents only minor changes with the depth indicating that the displacement vectors keep parallel with the depth pointing at different locations below the tunnel axis.

4 DISCUSSION

The results of monitoring presented in the previous section have been interpreted in the light of the usual empirical criteria based on the Gaussian fit of surface and subsurface ground movements. The influence of the excavation technology was already evidenced by the low values of volume loss back-calculated from the experimental data, typical of a TBM excavation. For both case histories the value of $V'$ derived by surface settlement troughs were rather small, ranging in the field 0.2%–0.5%. The values of the parameter $\lambda$ were on the average higher for Line 1 compared to Line 6. The difference is expected if the different groundwater table location is considered: for Line 1 the most of the covering sandy soil is under the groundwater table while for Line 6 the most of the covering sandy soil is above the groundwater table.

However the observed pattern of movements appeared in some cases locally conditioned by parameters related to the control of the tunneling machine which are usually neglected in the design stage. Therefore, available parameters such as the applied face pressure, the volume and weight of the excavated soil, the volume and pressure of the injected grout at the shield tail, registered during the excavation, are analysed and discussed in the following.

A key parameter to control ground movements during the TBM excavation is the ratio between the volume of extracted soil and the theoretical volume of the excavation. The average value of such a ratio over the south tunnel of line 1 was roughly equal to 1 and over the north tunnel it was equal to 1.1. This would indicate an ideal performance of the TBM excavation, according to AFTES (2001), and would justify the low volume loss generally back-calculated. However, despite the low variability of such a ratio, along the tunnel a significant variability of the magnitude and of the shape of surface settlement through was observed. Such a variability cannot be explained by the typical suggestions of the so called Gaussian method which only asks for the definition of the type of subsoil and of the amount of volume loss $V'$. 
Another parameter which could in principle be important to control settlement induced by excavation is the back-filling grout volume. In Figure 17, for instance, the maximum surface settlements measured along the south tunnel of Line 1 are plotted against the volume of the grout injected at the shield tail in the corresponding tunnel section. Although the large scattering of the experimental data, it clearly appears that the settlement decreases as far as the injected grout volume increases.

In Figure 18 the same data are plotted in another and more complex form to take into account all the main factors. The dimensionless ratio between the measured and the theoretical grout volume was computed as:

\[
R_{\text{inj}} = \frac{4V_{\text{GROUP\_meas}}}{\pi (D_S^2 - D_L^2)}
\]  

Where \(V_{\text{GROUP\_meas}}\) is the measured injected grout volume per ring of lining, \(D_S\) is the diameter of the TBM shield and \(D_L\) the external diameter of the lining. Such a ratio was plotted on the left y-axis (grey line) along the corresponding chainages of the south tunnel (the fifteen monitored sections are also indicated on the chainage axis) together with the maximum settlement measured at surface

Figure 17. Influence of injected grout volume on settlement (Line 1—south tunnel).

Figure 18. Relative injected grout volume and settlement along the south tunnel of Line 1.

Figure 19. Face and grout pressures measured along the south tunnel of Line 1.

The three dashed lines represent a simplified prediction of the maximum settlement by assuming a constant value of \(K = 0.5\) (average value in the monitored section, see §3.3) and three different values of \(V' = 0.3\%\), 0.5% and 1%. Such lines show clearly that in the first stretch of the Line 1, due to the shallower location of the tunnel axis, larger settlement at the surface should occur at the same fixed volume loss \(V'\). As a matter of fact, even with some scattering, the recorded surface settlements along the whole stretch of the line fall always in the same range 3 mm–12 mm, independently from the depth of the tunnel axis. Such a finding may be explained by the higher values of the injected volume in the initial stretch of the line.

Further parameters which could help in explaining such a finding is the average earth pressure measured in the excavation chamber (at face) or at the shield tail. These values as recorded during the excavation process of the stretch are plotted in Figure 19. In the same chart two continuous lines labeled as the upper and lower allowable pressure at face are also plotted. The upper allowable pressure has been assumed as equal to the total vertical stress at the tunnel axis level, following a practical “rule of thumb” to avoid the risk of “blow out” of the ground. The lower allowable pressure has been calculated as the hydrostatic pressure at the tunnel axis incremented of the effective stress needed to balance the unstable wedge in front of the tunnel face, according to the Anagnostou and Kovári (1996) method. While the pressure at the face is almost constantly close to the lower bound, the pressure at tail is closer to the upper allowable bound in the first part of the stretch, getting closer to the lower bound in the second part of the stretch.

Again, the use of relatively higher pressure at tail in the initial part of the stretch together with the use of larger volume of grout volume seem the main factors successfully adopted to prevent larger
settlement in the initial part of the stretch were the tunnel was located at shallower depth.

The patterns of the measured deep horizontal displacement (Fig. 15) are schematically summarised by the sketch in Figure 20. It has been observed in section 3.4 that the horizontal displacement around the tunnel axis level are directed sometimes toward the tunnel (positive) and sometimes outwards (negative). In Figure 20 the horizontal displacements measured about one tunnel diameter away at the tunnel axis level are plotted versus the injected grout pressure, showing a general trend of the ground movement to be directed outwards when the back-filling grout pressure increases.

5 CONCLUSIONS

The paper reports data obtained by the observations of both surface and deep movements recorded during the excavation of new tunnels with TBM in Naples (Italy). It is important to underline that very often tunnels excavated in urban areas interact also with buried structures: in such cases the empirical methods such as the Gaussian one should be adapted in the design stage to predict both surface and deep movements. To date the experimental evidence on deep movements around tunnels is rather scanty especially when compared to the data available on surface induced settlements.

The excavations in Naples were mainly carried out in sandy soil below the groundwater table. Surface settlements were measured using precision survey. Deep ground movements were measured using down hole extensometer and inclinometer probes.

Volumes of the surface trough as low as 0.3–0.5% of the tunnel volume were back-calculated; the volume of the trough deduced by deep movements data increased with depth up to values about twice the volume observed at the surface. The distance of the inflexion point from the tunnel axis, \( i \), allowed the empirical parameter \( K \) of the Gaussian fit to be calculated: the values varied in the typical range proposed by literature suggestions, with an average value of 0.5. The distance to the inflexion point, \( i \), decreased with depth slightly less than linearly, with power law with exponents between 0.7 and 1.

Interestingly, the vectors of displacements measured along vertical at the tunnel sides pointed towards different focuses located mainly below the tunnel axis.

The paper has shown also, at least on a qualitative basis, that the movements induced by TBM around the tunnel depend on technological and constructional procedures and details, usually non completely defined at the design stage. By a careful record of many operational parameters governed by the TBM drivers it has been possible to demonstrate that the volume of the injected grout at the tail and the pressure adopted for such injections are indeed factors controlling the induced surface and deep movements.

REFERENCES


