Analysis of settlements caused by tunneling with an earth-pressure balance machine and correlation with excavating parameters

E. Vanoudheusden*, G. Petit* & J. Robert
Arcadis ESG, Paris and Toulouse (*), France

F. Emeriault & R. Kastner
URGC Géotechnique, INSA, Lyon, France

J.Y. de Lamballerie
Maîtrise d'œuvre du Métro de l'Agglomération Toulousaine, Toulouse, France

B. Reynaud
Société du Métro de l'Agglomération Toulousaine (SMAT), Toulouse, France

ABSTRACT: This paper presents horizontal and vertical soil movements recorded by a set of monitoring devices during the excavation of a 4.7 km long tunnel by an Earth-Pressure Balance machine in the Toulouse's (France) underground. The devices included three inclinometers, five multi-points extensometer boreholes, high precision levelling and 5 pairs of vibrating wire strain gauges installed in concrete lining segments. Excavation rate, confining pressure, grouting pressure and volume of grout injected are the tunneling parameters considered in the analysis.

1 INTRODUCTION

The increasing urban transport needs lead to build underground structures such as tunnels and stations in a more and more urbanized environment. Particular attention should be paid to the range of induced soil deformations in depth and at ground level and potential effects on surrounding structures (Peck, 1969, Boscardin & Cording, 1989, Burland, 1995). At the design level, assumptions are required to characterize the behavior of the soil and nearby structures but also for the modeling of the different excavation phases (excavation, installation of the concrete lining, grouting of the annular void, ...). Therefore contractors usually install at key locations complete sets of measuring devices to evaluate the movements during and after the works and improve the design model.

The research project METROTOUL has been initiated to collect and analyze the results of the different monitoring devices installed during the excavation of the 12.6 km long subway line B of Toulouse – France (Emeriault et al. 2005, Vanoudheusden et al. 2005).

Contract 2 (4.7 km long) has been excavated by a 7.8 meter in diameter Herrenknecht earth-pressure balance machine. One monitoring section (see Figure 1) has been installed 2.9 km after the start of the tunnel boring machine (TBM). It includes inclinometers, multi-points borehole extensometers and precise levelling. Strains are also measured in the tunnel concrete lining segments. This article presents the results of this instrumentation and their analysis taking into account the main excavation parameters recorded by the TBM works.

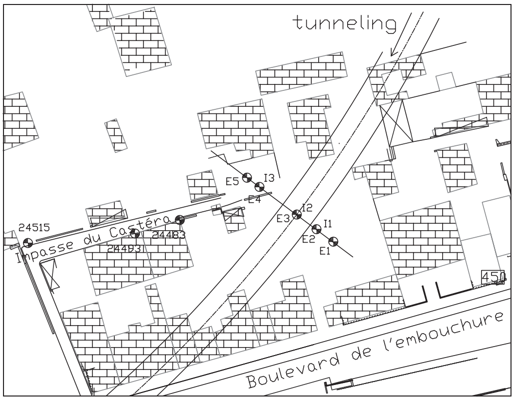

Figure 1. Location of the monitoring section.

2 GEOLOGICAL CONDITIONS

The tunnel runs through the Toulouse molasses (hard sandy clay with pockets and lenses of very dense sand). Geotechnical investigations have shown that in these formations, K_0 is greater than 1 and geotechnical characteristics are homogeneous ($\gamma = 22\,kN/m^3$, $S_u = 300\,kPa$, $c' = 30\,kPa$, $\phi = 32°$). In the vicinity of the monitoring section, the water table is found 2.8 m below ground level and the overburden thickness is approximately 12.7 m (Figure 2).

3 INSTRUMENTATION

The monitoring section is described in Figure 2. A complete set of data has been obtained regularly during the TBM passage. The monitoring devices included three inclinometric boreholes, one at the axis of the tunnel (I2), and the others 2.3 and 3.8 meters away from the tunnel sidewalls (I1 and I3). Vertical movements were recorded with five multi-points extensometer boreholes with automatic data acquisition (denoted E1 to E5). High precision levelling of borehole extensometer heads was regularly performed (after each excavation phase during the installation of the precast concrete tunnel lining elements). Besides, strains in the tunnel concrete lining segments were measured with a pair of vibrating wire strain gauges in each of the 5 segments. For surface instrumentations, reference is taken when the TBM is 30 m ahead of the monitoring section.

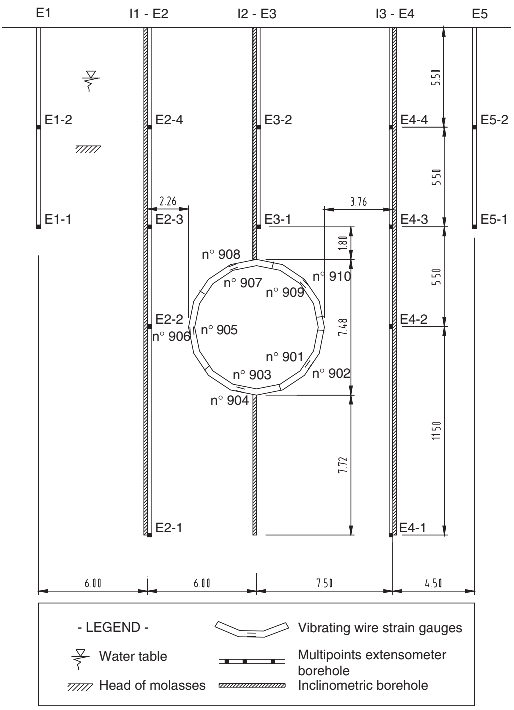

Figure 2. Monitoring section.

4 OBSERVED SOIL MOVEMENTS

4.1 *Ground surface vertical movements*

The vertical displacements of the ground surface have been measured by precise levelling of the 5 extensometer borehole heads. In the [−8 m; +12 m] range, one topographic survey is done for each installed tunnel ring (1.4 m long).

Figure 3 and Figure 4 show vertical displacements of the extensometer heads versus distance to the monitoring section. Even though amplitudes of movements are small, one can observe that, until the +4.2 m measure, extensometer heads E2, E3 and E4 remain still and that E1 and E5 (extreme extensometer) settle a little. Afterwards, heave is observed with a maximum

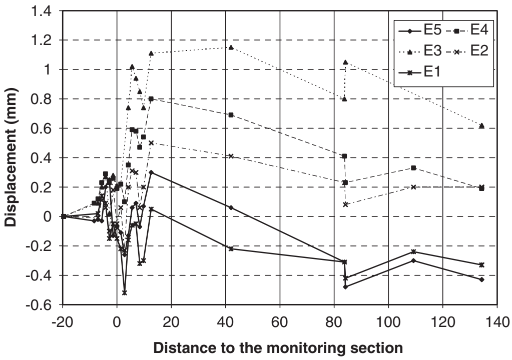

Figure 3. Vertical displacement of the extensometer heads vs. distance to the monitoring section – positive = heave – negative = settlement.

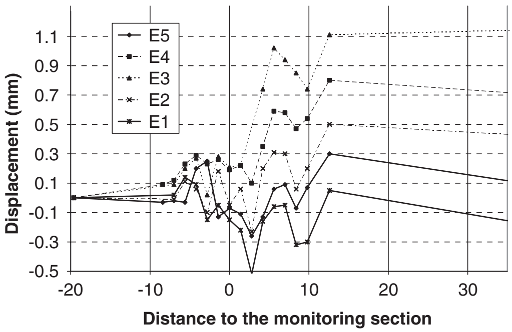

Figure 4. Vertical displacement of the extensometer heads vs. distance to the section in the [−20 m; +35 m] range.

value of 1.1 mm directly above the tunnel axis for the +12.6 m measure. After approximately 40 m, the different extensometer heads settle (see Figure 5). The final maximum settlement of 0.43 mm is reached 20 days after the passage of the TBM and is not observed directly above the tunnel axis but with a 12 m offset.

4.2 *Horizontal displacements*

The horizontal movements have been measured in three inclinometric boreholes located in the tunnel axis (I2) and on both sides of the tunnel (I1 and I3). Figure 6 shows the longitudinal horizontal movements (parallel to the tunnel axis), and Figure 7 the transversal horizontal movements (perpendicular to the tunnel axis). Due to the installation process, the directions of the inclinometer casings do not exactly correspond to the tunnel longitudinal and transversal directions. The results presented in Figures 6 and 7 account for the twisting of the casing measured with a compass-probe.

Before the cutting wheel reaches the monitoring section, the longitudinal displacements of the central inclinometric borehole I2 show that the soil in front of the TBM moves forward (see I2 Figure 6). This movement is linear on the whole length of the inclinometric borehole. The same scenario is observed for I1 with a smaller range of displacements. On the contrary, the −2 m measure of I3 shows a displacement towards the TBM excavation chamber, the maximum displacement being observed in the central part of the tunnel. After the passage of the TBM, I1 shows the formation of a bulb in the lower part of the tunnel lining, and on the contrary, in I3 the soil seems to be pushed forward when the TBM moves forward.

The transversal displacements appear 5 m after the cutting wheel has reached the monitoring section and correspond to a convergence towards the tunnel. The maximum displacement (5.3 mm) is obtained in the

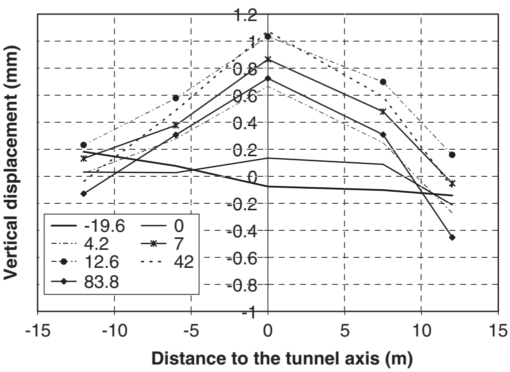

Figure 5. “Settlement” trough for different distances to the monitoring section (positive = heave – negative = settlement).

lower part of the tunnel. One can observed there is almost no horizontal displacement at ground level for I3 while the head of I1 moves of 1.7 mm towards the tunnel (value that is within the range of precision of measurements for a 27 m long inclinometer).

The simultaneous convergent horizontal displacements and ground surface heave can be explained by the high value of K_0 (greater than 1). The dissymmetry of I1 and I3 displacement profiles could be partly explained by the in-plane curvature of the tunnel lining in the vicinity of the monitoring section and/or by a local non uniform geology (local lenses of sand).

4.3 *Vertical displacements*

The vertical displacements at different locations within the soil are measured by five multi-points borehole extensometers (see Figure 2):

- two lateral 2-points extensometers at a distance equal to 2 diameters from the tunnel axis (E1 and E5),
- two 4-points extensometers at a small distance away from the tunnel sidewalls (E2 and E4)
- a central 2-points extensometer (E3).

The differential vertical displacements between the anchor and the extensometer head are automatically recorded every 5 minutes (positive values are

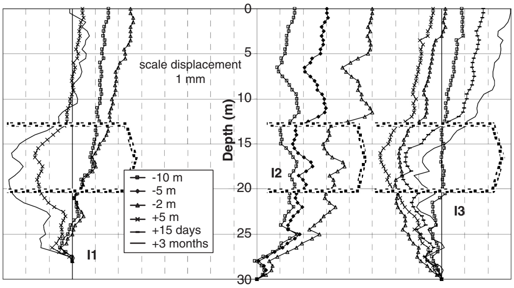

Figure 6. Longitudinal horizontal movements of the three inclinometric boreholes I1, I2 and I3.

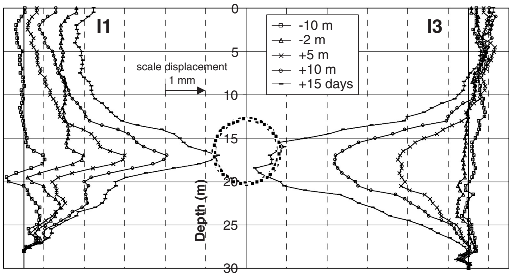

Figure 7. Transversal horizontal movements of I1 and I3.

obtained for a shortening of the distance between the extensometer head and the anchor).

A preliminary analysis of the results for extensometer E2 is first performed. The deepest anchor E2-1 is located one diameter below the tunnel and should therefore not be affected by the tunnelling. If this assumption is correct, the relative displacement measured between E2-1 and the extensometer head should be the exact opposite of the total vertical displacement measured by precision levelling of the extensometer head. Figure 8 shows that, except the two measures noted by arrows (and discarded because they correspond to measurement errors), the assumption is verified considering the intrinsic precision of levelling (approximately 0.2 mm).

As a result, the total vertical displacements of the three other anchors (E2-2 to E2-4) can be calculated as the difference between the differential displacements of the anchor and of E2-1 (Figure 9).

Before the cutting wheel reaches the monitoring section, no significant movement of the three anchors is observed. 4.5 m after the monitoring section, E2-2, E2-3 and E2-4 recorded a sudden heave. The maximum value of 0.3 mm is obtained when the cutting wheel is 5.8 m after the section which corresponds to

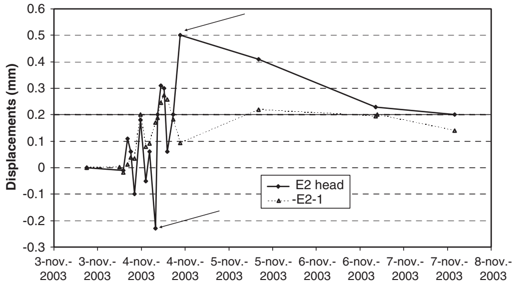

Figure 8. Precision levelling of extensometer E2 head and negative differential displacements of E2-1.

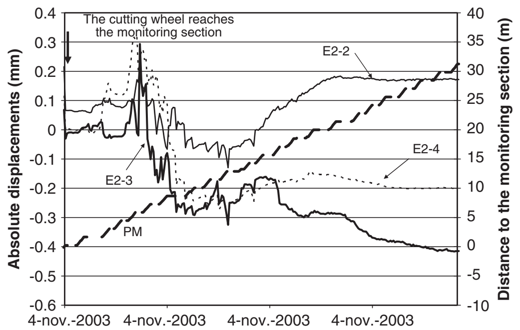

Figure 9. Total vertical displacements of the anchors E2-2, E2-3 and E2-4 and distance of the cutting wheel to the monitoring section vs. time.

the moment the end of the shield reaches the monitoring section. Then the 3 anchors settle until +9 m (with a maximum amplitude of 0.6 mm for E2-3) corresponding to the end of the tail and the beginning of grouting. This movement can be explained by the reduction of the TBM diameter (0.12 m on the total length of the shield). Settlements are stopped by the effect of grouting pressure. After 12 m, the 3 anchors start to heave, especially E2-2 (total heave of 0.3 mm). This is probably related to the buoyancy effect (the tunnel is on the whole lighter than the excavated soil). Starting at a distance ranging from 16 to 21 m depending on the anchor, settlement resume: if E2-2 is almost stable, E2-3 settles by approximately 0.26 mm and E2-4 around 0.05 mm. The consolidation of the grout can be responsible for this trend.

The final observed displacements are as follows: the extensometer head heave is 0.2 mm, the anchor E2-4 (5.5 m below ground level) settles of 0.2 mm while E2-3 settles the more (0.4 mm), the final vertical displacement of E2-2 is positive (0.17 mm) and E2-1 remains undisturbed by the tunnelling. The final vertical displacement measured for E2-2 should be compared with the horizontal convergence observed in inclinometer I1 at the same depth: 4.5 mm. The difference between soil movements in vertical and horizontal directions can be partly explained by a K_0 value greater than 1 (resulting from geotechnical investigations).

Figure 10 presents differential displacement of the soil above the tunnel axis (extensometer E3). The 2 anchors have similar movements:

– no significant differential displacements are observed during the TBM approach and while it passes the monitoring section
– 4 m after the section a sudden shortening of the distance between the anchor and the extensometer head appears indicating that the anchor moves upward with respect to the head.

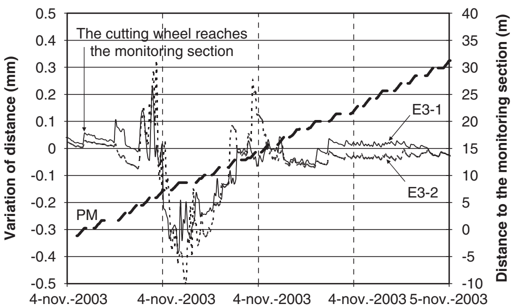

Figure 10. Differential vertical displacements of the 2 anchors of extensometer E3 vs. time.

– 7 m after when the cutting wheel reaches the monitoring section the distance increases.
– finally, the two anchors come back to their initial position. This distance decrease starts when the cutting wheel is 9.8 m after the section.

The total displacements of the surface and of the anchors E3-1 and E3-2 show a global heave of the soil above the tunnel axis. They are strongly related to the passage of the TBM in the [+4 m; +14 m] range.

The differential displacement recorded by extensometer E1, E3 and E4 are too small to be presented and analyzed (less than 0.2 mm). One can note the dissymmetry of vertical displacements between the extensometer E2 and E4; this dissymmetry has already been mentioned for the horizontal displacement profiles (see paragraph 4.2).

4.4 *Strains in the tunnel lining*

The variations of strains in the tunnel lining are measured by 5 pairs of vibrating wire strain gauges installed in the lining segments at the intrados and extrados (see Figure 2). Automatic data acquisition starts as soon as the ring is installed; one measurement is taken every 10 min.

Reference for the strains is taken before the lining segments are transported in the tunnel entrance shaft and thus before any load is applied. Negative μ-strains correspond to compression and positive values to extension.

The average strain for each pair of gauges is presented in Figure 11. First, an increase of μ-strain is recorded after the segment installation and during about 24 hours; this initial increase is affected by several factors that can not be clearly identified or assessed like temperatures of concrete or of the strain gauges for example. Then a slow decrease of μ-strain is observed with a stabilization after two months, corresponding to a compression of the lining segments. The induced increments of compressive stress can be assessed from the μ-strains: 2.75 MPa for the tunnel invert and 4.75 MPa for the tunnel crown. They can be related to an increase of the pressure applied by the soil and grout to the tunnel lining ranging from 247 kPa to 428 kPa. These stresses are close to the values of the vertical total stress σ_{v0} at the crown and invert of the tunnel (266 and 429 kPa).

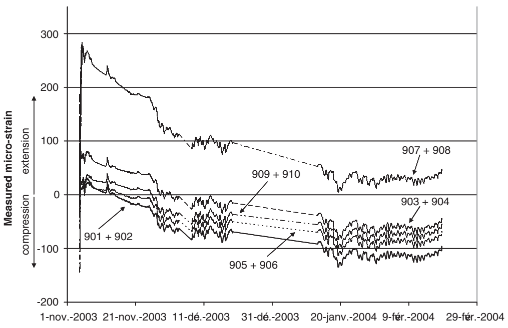

Figure 11. Strain gauges measurements vs. time.

5 TBM PARAMETERS

The tunneling parameters of interest for the analysis are the following:

– excavation rate
– pressure at the front P_{front}
– grouting pressure P_{grout}
– volume of grout injected per ring V_{grout}

5.1 *Excavation rate*

In the [−20 m; +35 m] range, the excavation rate increases slowly from 40 mm/minutes to 65 mm/minutes (see Figure 12). The excavation and installation of one ring of the tunnel lining require around 1 hour (see also Figure 9).

5.2 *Pressure at the front*

In the [−20 m; +40 m] range, the average pressure at the front is constant. The average recorded pressure is close to $0.6\sigma_{v0}$ (where σ_{v0} is the total vertical stress at the tunnel crown). Even though this pressure at the front seems rather small, in the particular geological context of Toulouse molasses (overconsolidated clayey soils), the stability of the front is not endangered. The high excavation rate is sufficient to ensure a satisfying level of stability.

5.3 *Grouting pressure*

The grout injection is performed by four pipes located in the upper half of the tunnel lining at a distance of

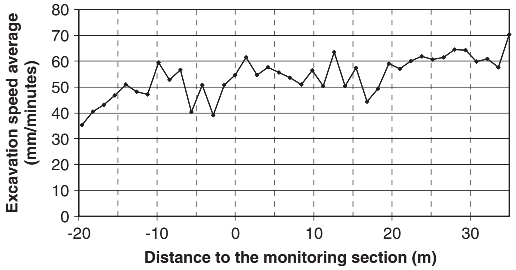

Figure 12. Excavation rate vs. distance to the monitoring section – average by ring.

8.5 m behind the cutting wheel. Therefore, the heave movements recorded by the borehole extensometers between 7 and 14 m after the cutting wheel passed the monitoring section may be due to these injections: it is clearly the case of the two anchors of E3 which were subjected to a sudden heave for 9.8 m and of E2-2 which is the only anchor to record a movement at 12 m.

The average grouting pressure is determined for each pipe and for each installed tunnel ring (1.4 m long). Figure 14 shows the grouting pressure divided by the total vertical stress at the tunnel crown versus the distance to the section.

Figure 14 shows that the average normalized grouting pressure P_{grout}/σ_{v0} (where σ_{v0} is the total vertical stress at the tunnel crown) varies between 1.1 and 1.5. It can be noted that the grouting pressure used before the TBM reaches the section is higher than 1.3; this may be the origin of the heave recorded when the cutting wheel is 7 m after the section: the injection is performed 1.5 m forward and the surrounding soil moves largely.

In the range [0 m; 10 m], the average grouting pressure decreases and then increases again. In any case,

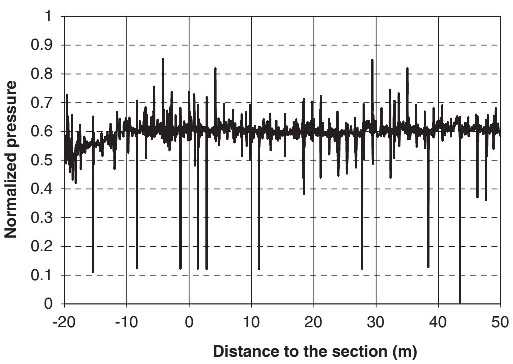

Figure 13. Normalized confining pressure P_{front}/σ_{v0} vs. distance to the monitoring section.

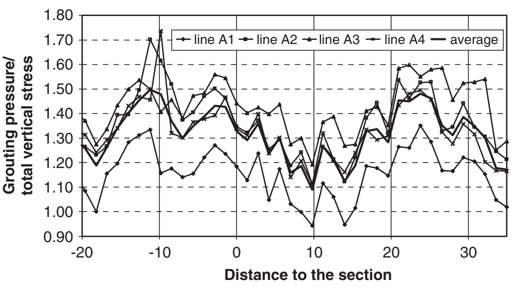

Figure 14. Normalized grouting pressure P_{grout}/σ_{v0} vs. distance to the monitoring section.

the grouting pressure is higher than the vertical stress, which may explain the global heave of the soil.

5.4 *Volume of grout injected*

It appears that the volume of grout injected per installed tunnel ring remains almost constant in the [−20 m; +30 m] range and is equal to the theoretical volume, except during injection of one ring located 11.2 m after the section (which proves not to have any consequence on the observed movements).

6 CONCLUSION

The horizontal and vertical movements induced by an Earth-Pressure Balanced machine during tunnelling on the Toulouse subway line B are presented. Besides, some tunnelling parameters are analysed to give an insight on the possible explanations of the recorded ground movements.

It appears that, due to the particular geological conditions of Toulouse molasses, the vertical movements are much smaller than the horizontal ones. Moreover, although horizontal movements appear as soon as the cutting wheel reaches the monitoring section, vertical displacements are particularly influenced by the grout injections, responsible for a global heave of the soil. Strain measurements in the tunnel lining show that progressive compression stresses appears in the lining segments.

Now it may be useful to proceed with the analysis of other monitoring sections installed on the 4 different contracts of the Toulouse's subway line B using TBMs. Since the geology, stratigraphy and tunnel cover are almost identical, the differences in soil reaction can be explained by different tunnelling parameters and different tunnelling techniques (Earth- Pressure Balanced Shield, Slurry Shield and Compressed- Air).

AKNOWLEDGEMENTS

The research project METROTOUL would not have been possible without the kind permission and constant support of the Société du Métro de l' Agglomération Toulousaine (SMAT). The authors would also like to thanks for the financial support the French Ministry of Research and the Réseau Génie Civil & Urbain (RGC&U). They finally would like to acknowledge the help of the different contractors in collecting data.

REFERENCES

Boscardin M.D. & Cording E.J. 1989. Building response to excavation induced settlement. Journal of Geotechnical Engng., ASCE, Vol. 115, No 1, pp. 1–21.

Burland J.B. 1995. Assessment of risk of damage to buildings due to tunnelling and excavation. Proc. 1st Int. Conf. Earthquake Geot. Eng., IS-Tokyo '95.

Emeriault F., Bonnet-Eymard T., Kastner R., Vanoudheusden E., Petit G., Robert J., de Lamballerie J.-Y. & Reynaud B. 2005. Ground movements induced by earth-pressure balanced, slurry shield and compressed-air tunneling techniques on the Toulouse subway line B. To be published in the Proceedings of ITA-AITES 2005, 7–12 May 2005, Istanbul, Turkey.

Peck R.B. 1969. Deep excavation and tunneling in soft ground. Proc. Of the 7th ICSMFE, State of the Art Volume, Mexico City, pp. 225–290.

Vanoudheusden E., Petit G., Robert J., Emeriault F., Kastner R., de Lamballerie J.-Y. & Reynaud B. 2005. Impact on the environment of a shallow gallery excavated in Toulouse's molasses by a conventionnal method. To be published in the Proceedings of ITA-AITES 2005, 7–12 May 2005, Istanbul, Turkey.