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EPB tunnelling in deltaic deposits: Observations of ground movements

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ABSTRACT: The Line 9 of Barcelona Metro is currently under construction. The southern part of the line connects the city to the airport and is completely excavated in soft deltaic deposits of Holocene age. In this part, tunnelling is performed by two EPB machines of 9.4 m diameter. The paper first describes the specific geology of the area. Information derived from the excavation of this part of Line 9 is presented and reviewed. In particular, the following issues are addressed: history of volume loss distribution along the line, magnitude and shape of subsidence troughs, influence of pressures used during tunnelling on observed movements, and contributing causes to observed instances of larger ground movements. The paper may constitute a useful case record for other EPB tunnelling projects in soft soil.

1 INTRODUCTION

A new Metro line, named Line 9, is currently under construction in the Barcelona metropolitan area (Di Mariano et al., 2007; Gens et al., 2006). It has a total length of 47.8 km, out of which nearly 44 km correspond to tunnelling. The project includes the construction of 52 stations. For civil engineering planning purposes, the project was divided into 4 major parts (Fig. 1): Part 1, the southern part, connects Barcelona city to its airport; Part 2, the south-eastern part, links to the Logistic Activities Zone of the Port of Barcelona; Part 3, the central part, runs through the high part of the city and, finally, Part 4, the northern part, connects Barcelona to the densely populated cities of Badalona and Santa Coloma de Gramanet (Ormazabal et al., 2008). The southern part of the line, i.e. Part 1, is completely excavated in soft deltaic deposits of Holocene age, which constitute, at the larger scale, a rather homogeneous geological profile. In this area, tunnelling is performed by means of two 9.4 m diameter EPB machines excavating in opposite directions from Mas Blau Station (Fig. 1).

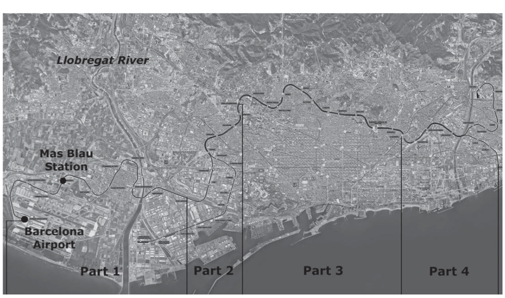

Figure 1. Plan view of the full route of the Line 9.

The present paper specifically refers to the excavation of the Line 9 between Mas Blau Station and Barcelona airport (Fig. 1). This 4.33 km-long section of the Line crosses an urban area close to some sensitive structures. Ground movements have therefore been a special concern during tunnel design and construction. An extensive instrumentation system has been deployed to control the magnitude and distribution of ground movements and also displacements of the structures during and after the excavation of the line. The article first summarily describes the special geology of the area and its implications for tunnelling. Afterwards, the information derived from this part of the excavation is presented and reviewed. In particular, the following issues are addressed: history of volume loss distribution along the line, magnitude and shape of subsidence troughs, and influence of pressures used during tunnelling on observed movements. Attention is also paid to contributing causes to observed instances of larger ground movements.

2 LOCATION AND GROUND CONDITIONS

The part of the Line 9 (L9) route described in this paper was constructed under L9 Contract T1D that starts at Mas Blau Station and ends at Terminal entre Pistes Station, inside Barcelona airport (Fig. 1). Tunnelling works started in April 2008 and lasted approximately one year. Figure 2 shows a plan view of this part of the line. It also indicates the location of the three instrumented reference sections present, the four stations of this route

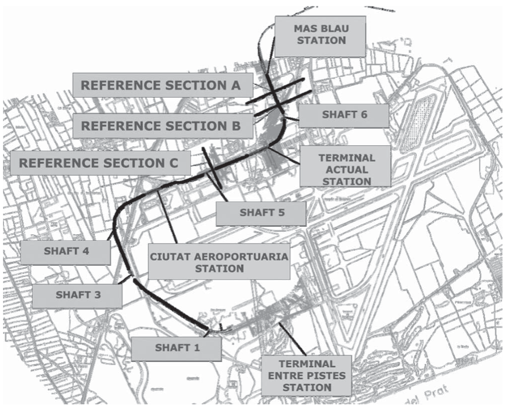

Figure 2. Plan view of the L9 contract T1D route.

and, finally, the five intermediate ventilation and emergency shafts. Of the four stations only one, the Terminal Actual Station, has been fully constructed underground and lacks a structure above ground. It will provide access to the old Barcelona Airport Terminal.

The longitudinal section shown in Figure 3 summarizes the geology along the tunnel route of Contract T1D. The top layer comprises made ground (R) of varying thickness overlying a thin stratum of brown fine silts (Ql1). Below it, gray fine sands, with some gravel inclusions (Ql2) are encountered which are in turn underlain by a gray layer (Ql3) of mixed composition: silty clays with some interlayered sands, sandy silts, clays and silts. The Ql3 layer is the main soft deposit and it reaches depths of approximately 50 m below ground level. It overlies the base gravels (QL4) where a confined aquifer is located. All of these materials are of Quaternary age and layers QL2 to QL4 belong to the deltaic deposits of the Llobregat River (Fig. 1). The water table is nearly horizontal and is located 0–1 m above sea level.

Except in the proximity of the Stations, the ground cover of the tunnel is about 16 m, with a cover/diameter (C/D) ratio of approximately 1.7. Around the Stations this ratio decreases reaching a minimum value of 1.0 in the area of Terminal Actual Station.

3 TUNNEL EXCAVATION

Tunnelling of Contract T1D was performed using a Herrenknecht Earth-Pressure-Balance (EPB) boring machine. The running tunnel was lined with 8.43 m Inner Diameter (ID) pre-cast segmental concrete rings, the segment length along the tunnel being 1.5 m. Six segments plus one key formed a full ring of 0.32 m thickness.

The advantage of EPB tunnelling is the possibility of providing substantial support to the excavated face at all times, thus allowing good ground movements control (Mair, 2008). During the excavation of the L9 Contract T1D, bentonite was systematically injected in the over-excavated annulus around the shield and the tail void was always grouted simultaneously to the EPB shield advance, in order to improve ground movement control (Wongsaroj et al., 2006).

Three different pressures were applied: the face pressure (Pf), a bentonite pressure (Pb) and the grouting pressure (Pg) (Di Mariano et al., 2009; Gens et al. 2009). Pf represents the support pressure to the excavation face provided partly by the thrust from the cutter head and partly by the chamber pressure (Mair, 2008); pressure Pb is the pressure of the bentonite injection around the shield, in the annular void between the shield and the ground due not only to the over-excavation of the cutter head but also to the conicity of the shield; pressure Pg corresponds to the pressure of the grouting injection that fills the gap between the extrados of the lining and the excavated ground (tail void). Typical face pressure values were in the range 2.0–3.0 bars, while shield pressures varied approximately from 1.3 to 2.0 bars and grouting pressures from 2.0 to 3.5 bars.

In order to achieve optimum performance with EPB machines, the excavated soil must form a suitable plastic mass of soft consistency and low friction that can readily be extruded from the head chamber through the screw conveyor (Peña Duarte, 2007). Natural ground does not usually have ideal properties when excavated, so modifications of its properties through soil conditioning may be necessary. The most common conditioners used in the tunnelling process are foams, polymers and clays (Peña Duarte, 2007). Foams were the only spoil conditioning agent used inside the chamber pressure in Contract T1D. Actually, when the excavated section was entirely in the Ql3 layer and its composition was mostly clayey, no spoil conditioning agent was needed.

After the first 250 m of the tunnel drive, access into the chamber pressure, by means of compressed air, was considered necessary to check the state of the cutting tools. No maintenance works were required. Afterwards, all the interventions at the cutter head were done at atmospheric pressure in the ventilation and emergency shafts and also in the Terminal Actual Station. In general, the wear and damage of the cutting tools was very limited in this section of the Line.

The advance of the EPB face with time is shown in Figure 4, together with the advancing tunnelling rates. The overall average advance rate, along the route of Contract T1D, was about 28.5 m/day,

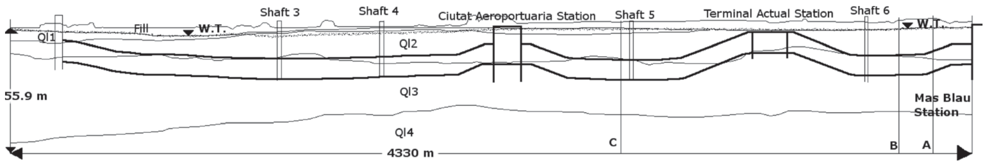

Figure 3. Sketch geological profile along the tunnel route of Contract T1D. Sections A, B and C are the instrumented reference sections at chainage 4150 m, 4000 m and 2750 m, respectively.

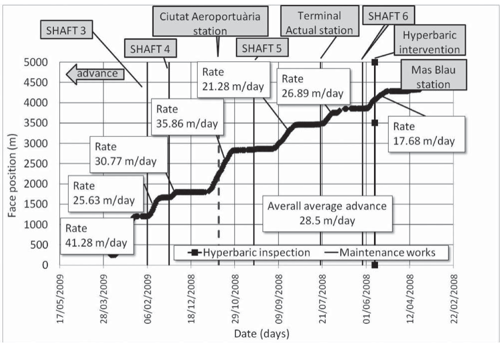

Figure 4. EPB face position vs. time. The Figure also indicates the advancing tunnelling rates.

i.e. 19 rings per day (stoppage periods are not included in these values) (Fig. 4). The maximum recorded daily advance was 90 m, or 60 rings. In Figure 4, the continuous vertical black lines indicate the positions of the shafts and the station where maintenance works were performed. The discontinuous line, which corresponds to Ciutat Aeroportuaria Station, indicates that the latter was constructed after the passage of the EPB machine and could not be used for maintenance. Finally, the vertical line, with solid black points, designates the location of the boring machine during the hyperbaric inspection.

4 GROUND MOVEMENT OBSERVATIONS

4.1 *Volume loss*

An extensive monitoring program was commissioned prior to the start of the excavation. Instrumentation was installed to measure not only the movements of nearby buildings, but also surface and subsurface movements as well as pore water pressures. Surface movements were monitored through levelling points and automatic sensors installed on the surface along longitudinal and transverse sections of the tunnel. Three instrumented reference sections were also installed to keep track of the deep ground movements at sections in greenfield situations with no buildings located nearby. Figure 5 shows, as an example, the instrumented section at chainage 4 + 000 m. The following instrumentation is deployed: seven levelling points along the transverse section of the tunnel, three extensometers (left, right and centre), two inclinometers (left and right), four open standpipe piezometers and three vibrating wire piezometers.

Two transverse settlement profiles, measured after the passage of the EPB beyond the zone of influence, are shown in Figure 6. Both of them refer to the instrumented section at chainage 4 + 000 m. The open points and the discontinuous line indicate the surface ground movement while the solid points and the continuous line represent the settlement trough at a depth of 9.5 m. Both profiles can be reasonably approximated in the form of a Gaussian distribution (Mair et al., 1993). In that case, the volume of the settlement trough per unit length, V_s is:

$$V_s = S_{\max} i \sqrt{2\pi} \tag{1}$$

where, S_{max} represents the ground vertical movement at the tunnel centre line and i the distance of the point of inflection of the curve from the tunnel centre line. The settlements caused by tunnelling are usually characterized by the volume or ground loss, V_l, which is the volume V_s expressed as a percentage of the notional excavated volume of the tunnel (Burland, 2001):

$$V_l = \frac{V_s}{\pi (D/2)^2} \tag{2}$$

Where D is the tunnel diameter. For practical purposes, the trough-width parameter at surface level (i) can be estimated with the following expression (O'Reilly & New, 1982):

$$i = K \cdot H_0 \tag{3}$$

where the parameter K depends on soil type and H_0 is the depth of the tunnel axis. For subsurface

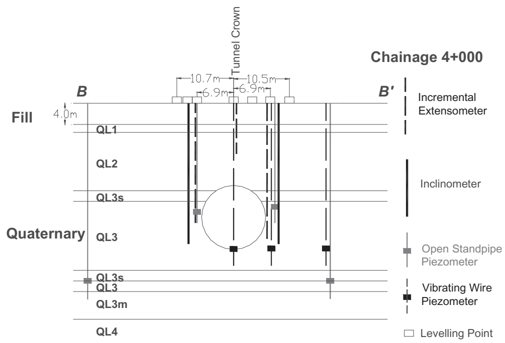

Figure 5. Instrumented reference section at chainage 4 + 000 m.

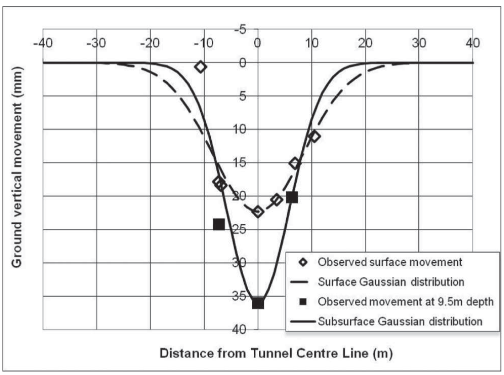

Figure 6. Surface and subsurface transverse settlement profiles in the instrumented reference section at chainage 4 + 000 m.

Table 1. K parameter and total volume loss at the three instrumented reference sections.

Reference section	Depth of tunnel axis	Volume loss	Surface settlement trough		Settlement trough at 9.5 m depth	
Chainage (m)	H_0 (m)	V_l (%)	K (–)	i (m)	K_{sub}* (–)	i_{sub}* (m)
4 + 150	18.7	0.22	0.51	9.5	0.59	5.4
4 + 000	20.2	0.68	0.42	8.5	0.55	5.9
2 + 750	21.4	0.50	0.52	11.1	0.70	8.3

*The subscript "sub" stands for subsurface.

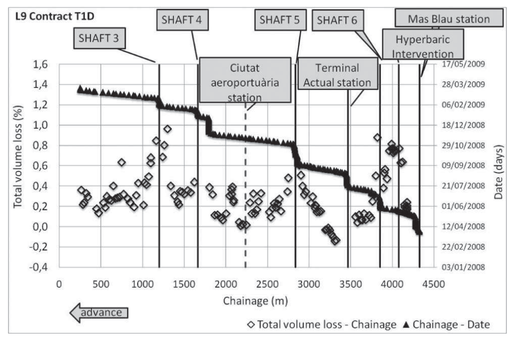

Figure 7. Total volume loss along the route of contract T1D.

settlement profiles the trough-width parameter $i_{sub}(z)$ can be expressed as (Mair et al., 1993):

$$i_{sub}(z) = K_{sub}(z) \cdot (H_0 - z) \tag{4}$$

z being the depth at which the distribution of settlements is considered and $K_{sub}(z)$ the subsurface parameter K at a depth z.

For the surface settlement trough in Figure 6, equation (2) gives a volume loss of 0.68% and a trough-width parameter (i) of 8.5 m resulting in a K parameter equal to 0.42. At a depth of 9.5 m, the subsurface parameters i_{sub} and K_{sub} equal to 5.9 m and 0.55 respectively. Table 1 summarizes the values of K, and i, relative to both surface and subsurface Gaussian distribution curves at the three instrumented reference sections. The subsurface curves are always calculated at a depth of 9.5 m. In common with field measurements in other cases, the parameter K appears to increase with depth, giving proportionally wider settlement profiles closer to the tunnel (Mair et al., 1993).

In addition to the three instrumented reference sections, several additional transverse sections were installed on which surface settlements were measured (Nyren et al., 2001). Field observations indicated that the parameter K assumes an average value of approximately 0.50 at ground level, on nearly the entire route of Contract T1D. This value was hence used to evaluate volume losses along the excavation using only the settlement above the axis of the tunnel. Figure 7 shows the computed volume loss along the route, after the passage of the EPB beyond the zone of influence. The majority of values lie between 0.0 and 0.6 per cent, a rather low range of values given the soft nature of the excavated ground. Higher volume losses were observed in two areas: the first one close to chainage 3 + 851 m where shaft 6 is located and the second one in the area of chainage 1 + 201 m, corresponding to shaft 3. Usually, the shafts along the route were used to carry out scheduled maintenance works to the cutter head under atmospheric conditions. The process of the boring machine entering into a shaft or exiting from it caused sometimes difficulties for maintaining the required face pressure. That is why volume losses in the proximity of the shafts were usually higher than in the rest of the L9 route. Moreover, whenever the EPB machine stopped, as in the case of the hyperbaric inspection, the restart of the excavation works generally represented a critical phase with higher ground losses.

Negative values of volume losses in Figure 7 refer to negative ground vertical movements, i.e. ground heave. Such negative values were registered in about 140 m of tunnelling in the gray fine sands of the Ql2 layer, just after the exit of Terminal Actual Station. Although the heave values are quite small, they probably indicate an excessively high face pressure.

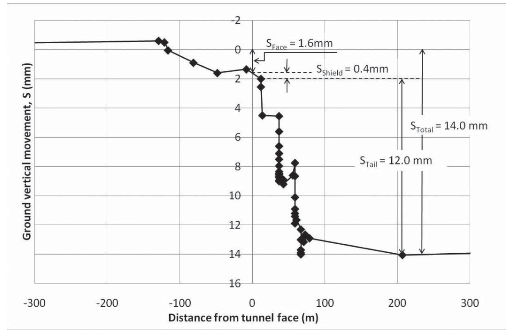

Figure 9. Ground vertical movement vs. distance from tunnel face at chainage 2+900 m.

4.2 *Components of ground movements associated with EPB tunnelling*

An example of the ground movements commonly associated with EPB tunnelling is shown in Figure 8 (Cording, 1991; Mair and Taylor, 1997; Dimmock, 2003 and Wongsaroj et al., 2006). Three components of settlements are indicated: the ground vertical movement measured when the face of the EPB machine is at the monitoring section (S_{Face}), the settlement due to the passage of the shield (S_{Shield}), and the settlement caused by both the closure of the tail void behind the shield and the deflection of the lining (S_{Tail}). Finally, the total settlement (S_{Total}) is defined as the sum of the three previous components. Delayed settlements caused by creep and/or consolidation are not explicitly considered in this paper. When measured, they were generally much smaller than the excavation-induced settlements.

As an example, Figure 9 shows the variation of the ground vertical movement with distance from the tunnel face, at a measuring point at chainage 2 + 900 m. The movement directly above the tunnel face, S_{Face}, is here less than 2 mm. In this case, S_{Face} represents approximately 10% of the total settlement. Generally, in the route of Contract T1D, the proportion of S_{Face} with respect to S_{Total} is quite low and often below 10%. Indeed, in some cases the arrival of the EPB face causes a small heave. At the same point (Fig. 9), the vertical movement due to the passage of the shield, S_{Shield}, is only 0.4 mm. The proportion of S_{Shield} with respect to S_{Total} varies greatly along the route, depending on several factors such as the excavated materials, the efficiency of the bentonite injection and the EPB rate of advance. In any case, the largest proportion of settlement by far corresponds to the closure of the tail void and the deflection of the lining (S_{Tail}). This applies to the entire route of Contact T1D and it is in fact very characteristic of EPB tunnelling.

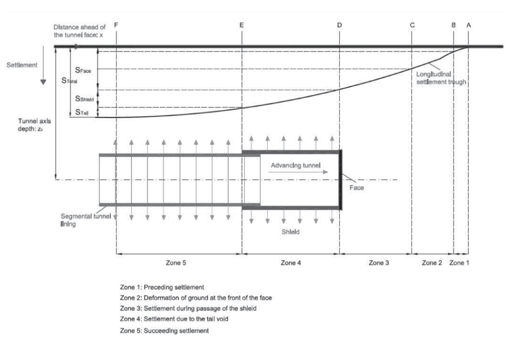

Figure 8. Components of ground settlements due to EPB tunnelling (Wongsaroj et al., 2006).

5 INFLUENCE OF EPB OPERATING PRESSURES ON VOLUME LOSS

During the EPB advance in the Contract T1D route, face, shield and grouting pressures were measured at the injection points every ten seconds. A large amount of field data is therefore available to examine the potential influence of such pressures on volume loss. A drawback, however, lies in the fact that the range of variation of the pressures employed is limited, thus reducing the scope for examining the effect of large pressure variations. Three components of volume loss have been defined (Wongsaroj et al., 2006): the volume loss measured as the tunnel face reaches the monitoring section (V_{LF}), the volume loss measured when the end of the shield passes the monitoring section (approximately 10 m after the passage of the EPB face) (V_{LS}) and the tail volume loss when the EPB has advanced enough, beyond the monitoring section, and the surface settlement has stabilised (V_{LT}). All three components defined are incremental. Accordingly, three pressure ratios have been introduced: the Face Pressure Ratio (FPR), the Bentonite Pressure Ratio (BPR) and the Grouting Pressure Ratio (GPR). The three ratios represent the face pressure, the bentonite pressure and the grouting pressure, respectively, normalised with respect to the overburden pressure at tunnel axis.

Figure 10 shows the relationship between the face volume loss, V_{LF}, and the face pressure ratio. It can be observed that the face volume loss is quite small in all cases and even negative at times. It can be concluded that, in this case, when face pressures are in the range 48–78% of the overburden pressure, the face volume loss is maintained well inside the ±0.2% range.

The volume loss V_{LS} measured when the end of the shield passes the monitoring section versus the bentonite pressure ratio is shown in Figure 11. Again, the volume loss is often very low and close to zero. The highest value of shield volume loss observed along the route of Contract T1D is only a little over 0.2%. Bentonite pressures in the range 33–74% of the overburden pressure are capable to keep the shield volume loss below 0.3% and generally much lower.

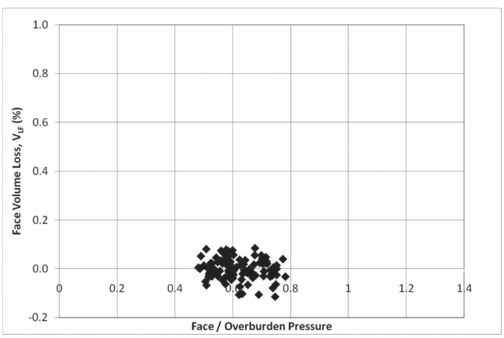

Figure 10. Face volume loss vs. face pressure ratio.

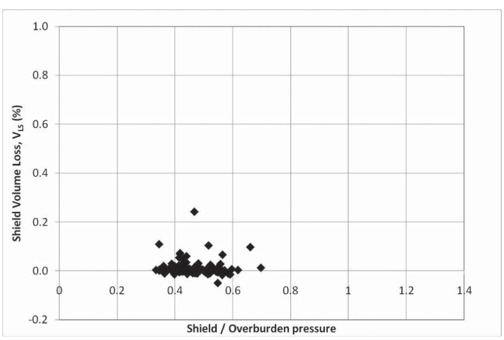

Figure 11. Shield volume loss vs. bentonite pressure ratio.

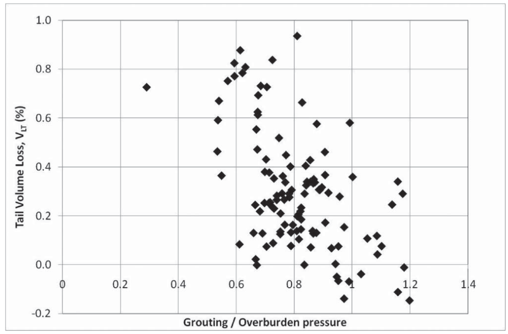

Figure 12. Tail volume loss vs. grouting pressure ratio.

The effect of the normalized grouting pressure on the tail volume loss is shown in Figure 12. Despite the large scatter in the data, there appears to be a rough tendency of the tail volume loss to decrease as the grouting pressure ratio increases. Identifying the causes of such scatter is not easy, as a number of additional factors (e.g. advanced rate, lining deflection, creep deformations) may contribute to it. As mentioned above, the volume loss due to the closure of the tail void represents the greatest percentage of the total volume loss, so the grouting injection has always been considered a very important operation. In the case described here, a full closure of the tail gap with no grouting applied would result in a volume loss of approximately 7%.

In any case, the good control of the three EPB operating pressures has resulted in quite low overall volume losses taking into consideration the soft characteristics of the ground.

6 SUMMARY AND CONCLUSIONS

The paper describes the excavation of a 4.33 km-long section of the Metro Line 9 between Mas Blau Station and Barcelona airport. The excavation has been performed in soft deltaic deposits with a number of sensitive structures close to the alignment.

The overall average advance rate of the EPB machine along the route was 28.5 m/day Volume losses lie generally in the range 0.0–0.6%, even though higher values have been observed at times. Most of the larger volume losses are associated with the entrance to or exit from shafts used for maintenance purposes. The parameter K that defines the width of the surface settlement trough has generally a value of about 0.5. Data from deep instrumentation confirms that K appears to increase with depth.

The face settlements and volume loss are always quite small and, sometimes, even negative indicating heave ahead of the EPB machine. The shield volume loss is more variable but also normally small. Most of the volume loss is associated with the closure of the gap after the passage of the shield. In spite of a large scatter of the data, there appears to be a correlation between the grouting pressures used and the tail volume loss.

In any case, the volume loss observed in the majority of cases is quite limited and the range of pressures adopted has been successful in reducing settlements to quite acceptable values in spite of the poor geotechnical characteristics of the excavated ground.

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