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Reducing settlement caused by shield tunnelling in alluvial soils

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ABSTRACT: Shield tunneling methods have become the main methods for tunnel construction in urban areas. However, under extremely unfavorable geological conditions, adjacent structural damage may occur by face instabilities and tail void. It is, therefore, necessary to develop the technology to repress as much as possible the ground deformation caused by shield advancing. This paper describes two sites of the Lyons Metro D line extension (1993-1995). They were intensively monitored and were bored with slurry shield. Based upon numerical modeling and site measurements of the behavior phases of the ground related to the shield advancement, this paper also presents the specified requirements for an inert and very economical back-fill together with a summary of the grouting operations. They have been proved to be very successful in reducing ground settlement caused by shield tunneling in alluvial soils.

1 INTRODUCTION

Tunneling in soft ground cause ground movements, which may induce damage or failure in adjacent existing structures in urban areas. Based on the recorded results related to tunneling operation from different projects, many authors (Rowe et al. 1983, Fujita 1989, Moroto et al. 1995, Kastner 1996) have suggested that the soft ground movements are very sensitive to tunneling operations. For this type of tunnels, which is often constructed by pressurized shield, the direct prediction of the final ground movements by empirical or numerical methods have proven to be insufficient (Rowe 1983, Clough & Finno 1985, Benmebarek et al. 1998). Therefore, it is necessary to develop a technology to repress as well as possible as the ground deformation induced by shield advancing and also accurate simulation tools, which allow prediction ground movements in relation with technical choices. In order to prevent excessive settlement and obtain precise and reliable experimental data, the extension of the D line of metro of Lyon-France (1993-1995), built with slurry shield, has been the topic of intensive and specific instrumentation realized by the Geotechnical Laboratory of INSA-Lyon-France.

After a project presentation, this paper describes the tunneling method and the monitoring procedure as well as the behavior phases of ground related to shield advancement. Based on monitoring results and back analysis, we try to find out the relationship between ground response and the influencing factors due to slurry shield. We also present the specified requirements for an inert and very economical back-fill, which have been proven to be very successful in reducing ground settlement caused by shield tunneling in alluvial soils.

2 GROUND CONDITIONS

The extension of the Lyon metro line D between the station 'Gorge de Loup' and 'Gare de Vaise' (Fig. 1), is situated in a built up area. 6.27 m diameter and 900 m in length twin tubes were bored in very soft soil and included 500 m under old buildings. The excavation is carried out entirely under water table.

From the surface, the encountered ground lay consist mainly of:

1. Average thickness fill of 3 to 5 meters,
2. Silt alluvial deposits in more or less sandy or clayey thin soils. On occasions these silts have a natural water content close to liquid limit, making these layers very sensitive to disturbances,
3. Sandy-gravely alluvial deposits, few elements having a diameter of more than 500 mm that require the use of a crusher,
4. The gneiss substratum is encountered at more than 80 meters depth in Vaise station and less than 20 meters depth in Gorge de Loup.

Hydrogeological studies revealed two water-bearing aquifers:

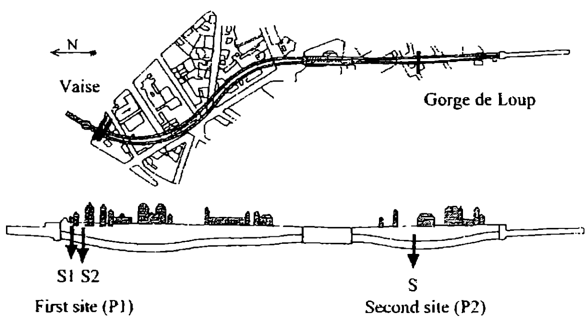

Figure 1. Plan view of construction site.

1. An upper aquifer in the silts and fill (Kh = 10^{-6} to 10^{-7} m/s)
2. A lower aquifer in the permeable alluvial deposits fed by the River Saône (Kh = 10^{-3} to 10^{-5} m/s).

3 TUNNELING METHOD

Due to the settlement risk, the technique of a slurry shield was preferred to earth pressure balance shield "EPBS". Problems may be encountered with heterogeneous ground conditions (Clough et al. 1993). This technique allows precise adjustments of slurry pressure in the chamber.

The tunnel boring machine (TBM) used in this project was built by the Herrenknecht Company and consists of shield of 6.85 m length and 42 m length train consisting of 4 trucks. The shield has a conicity of 3cm resulting from the difference between face diameter 6.27 m and tail diameter 6.24 m (Fig. 2). This conicity facilitates the controlling of shield pose and reduces soil-shield friction.

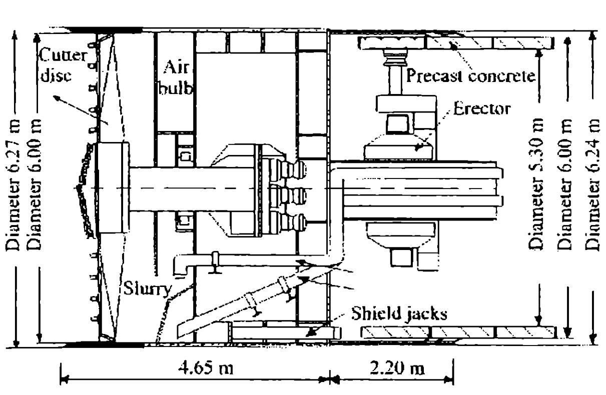

Figure 2. Schematic view of Herrenknecht shield.

The tunnel is lined with rings of 1m length installed under the protection of the tail. Rings are made up of six precast concrete segments with 6 m of external diameter and 0.35 m thickness. The annular space remaining between the external side of the tunnel lining segments and excavation side is 13.5 cm (Gap = 27 cm).

In soft ground, it is required to fill tail voids with back-fill grouting as quickly as possible for controlling ground movements, so as not to develop any tail void. Therefore a simultaneous and continuous grouting method is adopted for the back-filling with the machine advance and even during stop phases from six injection pipes distributed uniformly at the tail side of the TBM (Fig. 3). Pressure sensors are fixed at each pipe and related to a station, which allows controlling working pipes.

For the material of the back filling, an inert and very economic grout constituted of sand, filler bentonite and water is employed. This grout has a liquid behavior at prepared water content w = 15 %.

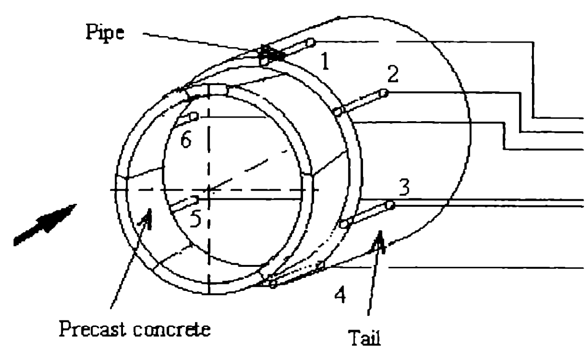

Figure 3. Injection pipes distribution.

4 EXPERIMENTAL SITES AND EQUIPMENT

In order to study with some precision the soil-TBM interaction, two sites are selected to monitor the ground response due to shield tunneling. The instrumentation is divided into two monitoring sites (Fig.1). The first site (P1) includes 2 monitoring sections (S1 and S2) that are located respectively at 30m and 65 m from the starting shaft in order to aid operation and adjustment of the TBM. The second site (P2) is located at the end of the layout, at a distance of about 800 m from the first, with the aim of recording data on an area where the working process is well controlled and the geometry is symmetric.

The vertical movements are measured by means of multipoint extensometers with 4 points of measurement, connected to an automatic data logger. The surface settlement measurements are ensured by a system of leveling vessels and complemented by more punctual topographic measurements. The horizontal movements are measured on both sides of the tunnel by means of inclinometers. The recording occurs manually via one measurement every 50 cm. It can be noted that the nearest measuring points are situated within 1 meter of the tunnel section. The pore pressures of silt layers are monitored by pore pressure cells (CPI).

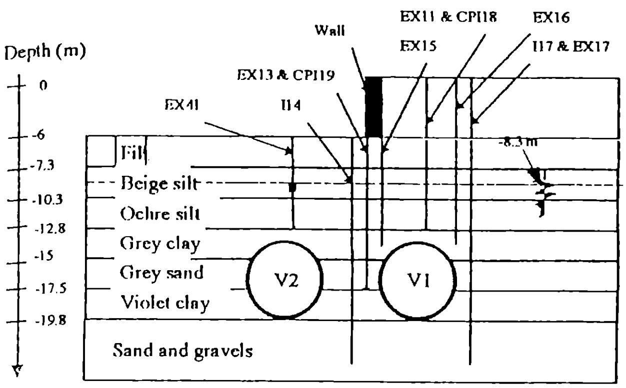

Figure 4. P1-S1 experimental section.

Figure 4 shows, for example, the ground layers encountered from the surface and the measurement system implimentated on the first section S1 of the first site (P1-S1).

5 GROUND RESPONSE DUE TO SHIELD TUNNELING

Figure 5 plots the recording settlements evolution with time at different depths of the centered extensometers EX11 (P1-S1) and EX31 (P2-S).

The horizontal movements in the direction of the axis remain small but they are more important in the perpendicular direction. Figure 6 shows, for example, the horizontal displacements recorded by inclinometers in the direction perpendicular to the tunnel axis during the shield passage at the first tube of (P1-S1).Figure 7 shows the pore pressure evolution with position of shield and time for section (P1-S1). The progression of the shield leads to increase in pore pressure, which dissipates to their original value practically between two boring cycles. This would indicate that drained conditions are suitable for simulation of the soil behavior. The trends of both vertical and horizontal displacements for all measurement sections are similar.

Ideally, as shown in figure 8, settlements of deeper points of the centered extensometers and inclinometer displacements in perpendicular direction can be divided into five featured phases:

Phase 1) The first phase (Ph1) consists of small settlements with an outward movement of the ground occurs at the axis level, which indicate a very good stabilization of the excavation front. The monitor of pore water pressure (Fig. 7) at the tunnel axis (depth = 16 m) before the cutting face goes through the observational section indicates no increase in pore pressure.

This confirms that the permeation of slurry into the soil forms the mud film then the slurry pressure acting on the cutting face is transmitted through the ground.

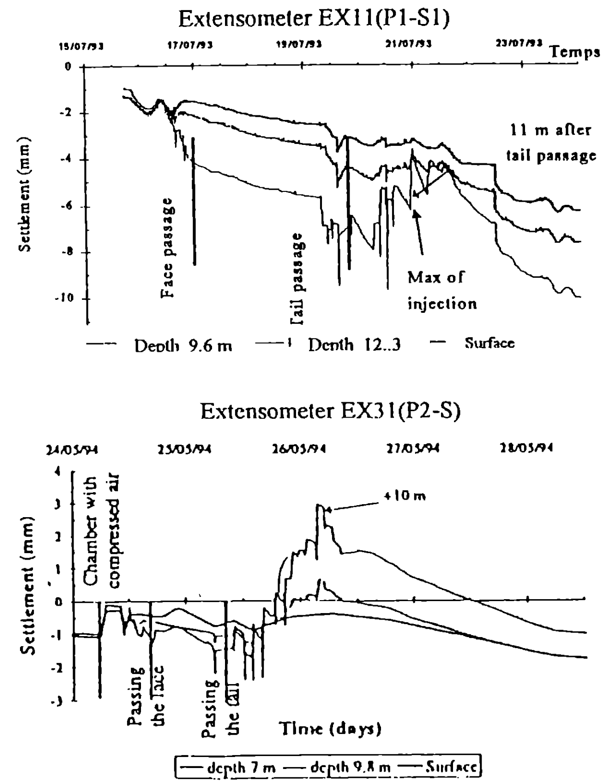

Figure 5. Evolution of vertical movements.

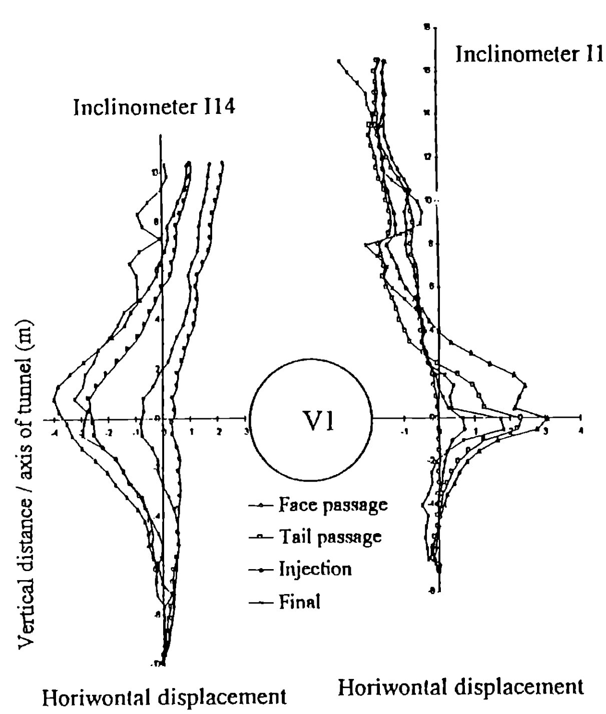

Figure 6. Inclinometric displacements (P1-S1).

Phase 2) The second phase (Ph2) regard the passage of the shield face, which induces an immediate settlement of the crown and a beginning lateral soil repulse. At this stage, the shield wall prevents all inward movements of soil (Fig. 2).

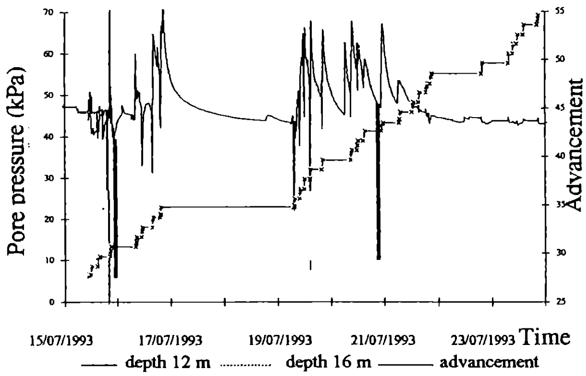

Figure 7. Pore pressure response to shield advance.

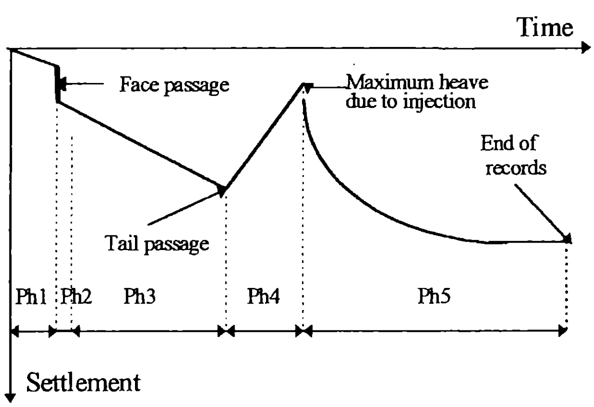

Figure 8. Idealized vertical displacement time curve.

Therefore, these occur without over cutting. The observed movements could be attributed to a reduction of the shear stress at the periphery of the excavation.

Phase 3) For the third phase (Ph3) the vertical movements increase with the progression of the shield due to a decreasing diameter of 3 cm over its length. The inclinometers show an unexpected continuity of lateral outward soil movements at the tunnel axis in spite of decreasing shield diameter.

Phase 4) For the forth phase (Ph4), at the tail exit, the effect of the injections back-filled grout is very marked by not only the lateral soil repulse but also by some settlement-heave cycles of the crown that decreased at surface. These cycles were closely related to cycles of shield advancement. It seems that the settlement-heave cycles depend of the pressure fluctuation of and the grout injected volume. All experimental measurements show an upheaval at the deepest point of about 4 mm synchronized with maximum lateral repulse. This indicates that the simultaneous injection of liquid inert grout has a good filling performance into tail void.

It should be noted that on section P2-S, the voluntary increase of the pressure and the grout volume injected has formed a grout bulb that leads to 3 mm heave at the deepest point over it's initial position and the same value for lateral repulse. Laboratory tests show that the behavior of this grout is greatly influenced by water content. A 3 % reduction of water content permits at grout to pass from a liquid to a consistent behavior. Therefore, grout strength is obtained by a few reduction of water content under ground pressure.

Phase 5) For the final phase (Ph5), the lateral movements are stabilized by a little convergence, so that the settlement takes place again in order to reach a stabilization to the tip 60 days. These movements can be linked to the phenomenon of soil creep on the one hand and to the hydraulic consolidation of the inert grout on the other. It is important to note that, after 3 months, the voluntary increase of the pressure and volume of grout injected for section P2-S has repressed the surface settlement within 2 mm and inverted surface settlement and subsurface settlements. That is deep settlements have become less than surface settlement. We can deduce that the little expansion caused by grouting with little pressure fluctuation is very efficient for repressing settlements. Final settlement values recorded at this point after excavation of the first tunnel are: 15.2 mm at EX11A (depth 12.3 m) and 1.5 mm at EX31A (depth 9.8 m).

6 BACK ANALYSIS

There have been many attempts to model tunneling operations by using empirical and numerical methods (Attewell 1977, Rowe et al. 1983, Mair et al. 1993, Bakker et al. 1996). However, the very sensibility of ground movements to the progress of tunneling operations and the input parameters are still a major problem for routine design.

The distances from centerline to the point of inflection are compared, in table 1 to value proposed by Attewell (1977). The width of trough surface settlement for section P1-S1 is well indicated in the proposed range. On the other hand for section P2-S which is not that deeper, the through surface settlement appears clearly narrow.

In the aim to take a count the tunneling operations, this paper presents numerical results of an improvement in enforced displacement concept on modeling excavation process by separating radial and tangential displacements.

Table 1. Comparison of experimental surface settlement curves to values proposed by Attewell (1977)

Section	z (m)	z/2a	i/a (measure)	i/a (Attewell)
P1-S1	16,9	2,7	2,4	1,9 - 3
P2-S	13,4	2,1	1,25	1,6 - 2,5

(z : depth to tunnel axis, a : tunnel radius)

The principle of the present method is to use a ring with diameter at first equal to the shield face diameter. This ring first represents the shield wall, then the segment lining and the grout. Interaction between soil-shield and soil-grout were modeled by interface element, applying Mohr-Coulomb Yield criterion. The different stages were modeled by activating interface element with mechanical characteristics related to the stage and decreasing or increasing progressively the diameter of the ring, with fixed bottom, adjusted at the displacement evolution of the reference point. The deepest extensometer point, placed on the axis of tunnel excavation, was chosen as a reference point for all simulations.

The computation was performed in high deformation using two-dimensional plan strain explicit finite difference code FLAC (Itasca 1995). The initial stresses were calculated by considering a geostatic stress state with a coefficient of earth-pressure at rest K_o equal to 0.5. Figure 9 shows computed displacement vectors at shield face passage. It indicates that the movements observed are mainly caused by the reduction of the shear stress along the interface between soil and wall shield, allowing irreversible soil sliding around the tunnel section.

The shield passage was simulated by reducing progressively both the diameter of the ring, and the mechanical characteristics of the interface elements attested by the very weak recorded resistance to shield advancement. At first, this phase was simulated, without adjustment to record results, by 3 cm diameter reduction, which corresponds to the theoretical gap due to shield conicity. Simulation results show the soil closing to the ring (Fig. 10) with 3.1 cm settlement at reference point. Back-analysis shows a little diameter reduction (-4 mm for section P1-S1 and -1 mm for section P2-S) gives a good simulation of observed movements. This indicates that the soil closes partially the gap under back-filling effects. The design of the shield with little conicity has thus not harmful effect. The phase of injection was simulated by increasing the diameter of the ring progressively adjusted to the displacement of the deepest point of the centered extensometer. In order to take into account the fluid character of back-fill grouting, which transmits badly the shear stress to ground, the mechanical characteristics c and φ of the interface are taken very weak.

This simulation reproduces well not only the lateral repulse of soil but also the amortization of the settlements toward the surface.

For the final phase, the ring diameter was reduced progressively and adjusted to the displacement of the deepest point of the centered extensometer. It reproduces well observed movements for all sections. Figure 11 is the computed lateral displacements evolution of inclinometers I14 and figure 12 is the computed surface settlement evolution compared to final measurements (P1-S1). However, the adjustment of volume loss of grout based on laboratory tests gives a fairly good estimation for section P2-S and an underestimate for section P1-S1. As the same grout was injected, this discrepancy is probably due to the pressure fluctuation effect of back-fill grouting which clearly marked by some falls of irreversible settlement (Fig. 6).

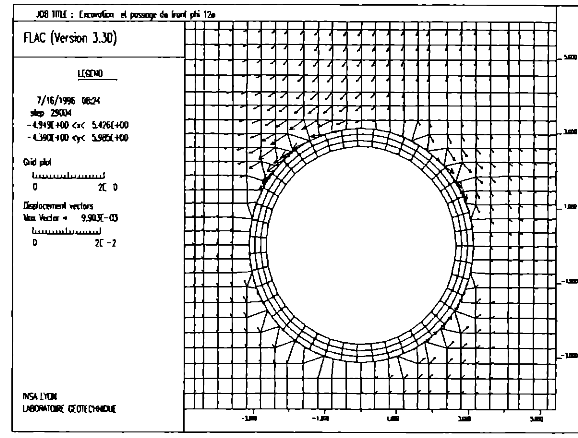

Figure 9. Computed displacement vectors at face passage.

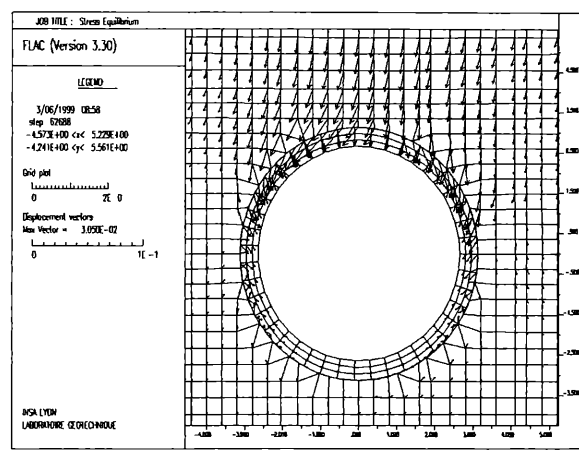

Figure 10. Computed displacement vectors at tail passage by 3 cm diameter reduction.

7 CONCLUSIONS

The following conclusion can be drawn from the analysis of the recording and the simulation results:

- The settlement in front of the shield was less than 2 mm for all sections of the present project. The efficiency of face stabilization is confirmed by the control of the slurry pressure fluctuation with a compressed air bulb in the chamber.

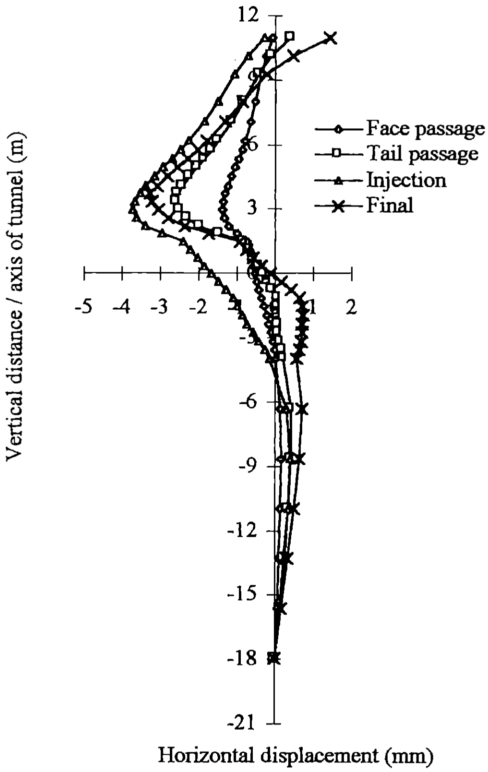

Figure 11. Computed lateral displacements evolution of inclinometers I14 (P1-S1).

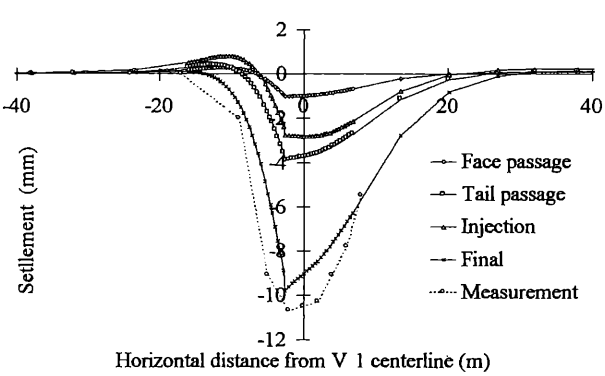

Figure 12. Computed surface settlement evolution compared to final measurements (P1-S1).

- During the shield passage, soil closes partially the gap and it is influenced by grouting back-fill. So that shield conicity has no residual effect on ground movements and it is very efficient and economic in reducing soil-shield friction.
- The fluctuation pressure reduces the efficiency of the injection grout by increasing the settlements in the consolidation stage.
- The light repulse of surrounding soil over its initial position by a grout bulb is very suitable to repress settlements in a permeable soil.

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