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Displacements and stresses induced by a tunnel excavation: Case of Bois de Peu (France)

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ABSTRACT: The tunnel of Bois de Peu is part of a project of the south-eastern Besançon (France) by-pass. An exploration gallery permitted to highlight the presence of eighteen geological units and to distinguish four sorts of materials: limestone, marls, clays and interbeddings of marls and limestone. Despite of the important number of laboratory and in situ tests carried out, many uncertainties remained on the mechanical parameters value and on the position of the different geological units. So, it was decided to apply the interactive design method during the construction to adapt the excavation and support to the actual conditions found. In the framework of this method, an important monitoring program was foreseen. This article shows the behavior observed in the different kind of soils. In the clayey zone where the support is the most complex, strains in the ground and in steel ribs are monitored. Finally, the available measurements permit to better understand the behavior of soil-structure interaction problems.

1 INTRODUCTION

The digging of a tunnel induces a modification of the initial stress field in the ground which creates an unbalance state. This unbalance results in movements of soil like convergence of the cavity, pre-convergence ahead of the face, extrusion of the face and settlements.

During the excavation an arching effect is created. If it is not sufficient to stabilize the cavity, lining supports are set up in order to limit soil movements and thus, to avoid the failure of the structure.

The lining support can include shotcrete, ribs and/or radial bolting. In certain cases, when the ground has low mechanical characteristics, additional support like ground reinforcement or ground improvement at some stage of the excavation is necessary. Several reinforcement systems have been developed. Pelizza & Peila (1999) presented the different methods of soil and rock improvement used to permit safe tunneling in difficult geological conditions. Lunardi (2000) divided the support methods into three groups: pre-confinement, confinement and pre-support. Each group exerts a different kind of effect on the cavity.

When the traditional method of digging is used in difficult geological conditions, the main problem is the control of movements. Without support or adapted treatment, the ground tends to sink into the opening (tunnel face failure, tunnel face extrusion): it is the phenomenon of decompression. In order to reduce this phenomenon, an action of pre-confinement may be

required. A pre-confinement action is defined as any active action that increases the formation of an arch effect in the ground ahead of the tunnel face. The pre-confinement can be achieved by reinforcement or protective intervention ahead the tunnel face. The umbrella arch method and the face bolting are included in protective interventions or pre-support methods.

The support and forepoling introduce many three dimensional soil-structure interactions. Consequently, it is difficult to understand these phenomena analytically. Moreover, although the umbrella arch method is widely used, there are no simple approximations to simulate this method in numerical analyses. The design of an umbrella arch is still based today on empirical considerations or on simplified schemes (Oreste & Peila 1997). Several numerical studies carried out in 2D and in 3D, were focused on the manner of taking into account umbrella arch (Tan & Ranjith 2003, Bae et al. 2005). In the same way, several numerical modeling were carried out in 2D and in 3D on the manner of taking into account the face bolting (Yoo 2002, Dias 1999).

For some large geotechnical engineering projects, a monitoring program is generally defined in project phase to record the soil movements which really occur during construction and to evaluate the performance of the construction design. In most of the cases, the recorded data are just used to control the construction process. But, these data can be also used to update predictions by using inverse analysis processes on

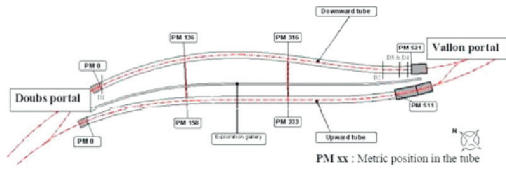


Figure 1. A plan view of the Bois de Peu tunnel project.

these measurements in order to decide of a possible adaptation of the construction process in the case where unsafe values would be predicted. This later practice is a part of the observational method (Peck 1969, Powderham & Nicholson 1996). The AFTES guidelines (2005) related to the monitoring methods of underground works present the major methods and give advices on the measurements frequency.

In this article, monitoring results obtained in the different geological units found during the excavation of the downward tube of the Bois de Peu tunnel are presented. First, this communication introduces the tunnel project. Then, in a second part, the monitoring results are presented. They permit to better understand the soil and the umbrella arch behavior set up in the clayey zone.

2 PRESENTATION OF THE TUNNEL

2.1 General presentation

The tunnel of Bois de Peu (cf. Fig. 1) is part of the project of the south-eastern Besançon (France) by-pass entitled “La Voie des Mercureaux”. This project includes several engineering structures: two tunnels and one bridge, and several retaining walls. The tunnel is composed of two tubes of 520 m length. The cover height varies between 8 m and 140 m. The excavation, achieved in September 2006, was carried out full face by drill and blast for the major part of the tunnel.

2.2 Geology and geotechnics

An exploration gallery was dug in 1995 in order to assess the mechanical properties of the ground. It has a width of 3 m and a height of 3.5 m. Various laboratory and in situ tests were carried out. The results lead to conclude that the tunnel is situated in a disturbed area. Eighteen geological units are identified. A geological cross section is showed in Figure 2. Among these eighteen units, four sorts of materials can be distinguished: clays, marls, limestone and interbeddings of marls and limestone. Geotechnical properties of these materials defined at the end of site investigations are summarized in Table 1. Two types of characteristics were defined for the marl (probable and

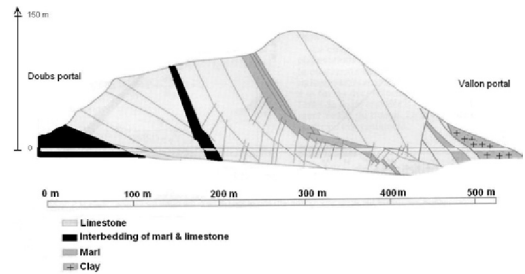


Figure 2. The geological cross section.

Table 1. Geotechnical properties of material.

Material	γ kN/m ³	E MPa	C	ϕ MPa
Limestone	26.3	3290–10000	—	—
Marl	24.8	1600	0.7	40
(probable)				
Marl	24.0	750	0.21	36
(exceptional)				
Clay	23	80	0.25–0.4	13–17

exceptional) because the in situ and laboratory tests leaded to variable values. The Poisson ratio, the dilatancy angle and the earth pressure ratio are the same for the materials and are respectively equal to 0.3, 0° and 0.7.

The exact position of the different units is difficult to know before the digging. Finally, following the observation made in this exploration gallery, many uncertainties are remaining. So, it was decided to apply the observational method during the digging in order to adapt the lining support to the real ground conditions. In this framework, an important number of experimental measurements were foreseen and four sorts of support were defined in project phase. During construction, the choice of the adapted support depended on the monitoring results and on the quality of the soils assessed by geological surveys. In this context, two other supports were realized during the excavation. The digging step for each support was defined variable.

3 MONITORING RESULTS

Four monitored sections are presented: one in interbeddings of marl and limestone (entitled D1), one in marls (D2) and two in clay (D3 & D4). Each of them are not circular. For two of them, only convergence and leveling measurements are available. For the others, more specific measurements were carried out such as strains measurements ahead the face by extrusometers.

Table 2. Main characteristics of each studied section.

Section	PM m	Material	H m	D _{face} m
D1	9.75	marl/limestone	22	3.1
D2	457	marl	40	13
D3	510.3	clay	15	1.3
D4	493.8	clay	22	3.84

Each section is presented and analyzed. Table 2 reports the localization in the tube (PM, referred to Fig. 1), the type of material, the overburden from the tunnel axis (H) and the distance from the face at the origin of the convergence and leveling measurements (D_{face}) for each studied section. D1, D3 and D4 are situated near one of the two tunnel portals (see Fig. 1, low overburden). For the sections D1, the wall support is composed of shotcrete and steel ribs set up every 0.75 to 1.75 m. For D2, it is made up by shotcrete and radial bolts. The digging step varies between 3.5 and 4.5 m. For D3 and D4, the support is more complex. It includes a wall support and an arch invert by shotcrete and steel ribs, set up at the tunnel advance every 1.5 m. And a forepoling by umbrella arch and face bolting are realized every 9 m in the general case. In D3, the excavation was made in partial face otherwise it was made in full face in D4.

3.1 Convergence and leveling

Tunnel wall convergences between reference points are realized by optical sights. Five optical reflector targets are installed in the monitored sections where the excavation is made in full face (at the crown, at 45° and at the spring line). When the excavation is realized in partial face, the five targets set up after the excavation of the half higher section are completed by two other targets installed after the excavation of the lower part at the side walls. Leveling measurements are less accurate than convergences (± 5 mm against ± 1 mm) because of the use of several reference stations. But, for all studied sections in the downward tube, leveling curves are exploitable.

The maximal convergence and leveling values are obtained in the clayey zone. In the half higher section D3, leveling measurements reached 25 mm for targets 1 and 2 (Fig. 3) while convergences remain lower than 7 mm.

The maximal convergence value is recorded for wire 4. Although the section is closed by a temporary buton, vertical movements are important. As targets 1, 2 and 3 present higher displacements, the deformation of the section is dissymmetric. The side wall located near the other tube presents larger vertical movements.

After the excavation of the lower part, convergences measurements are higher and reach 12 mm for wire 4

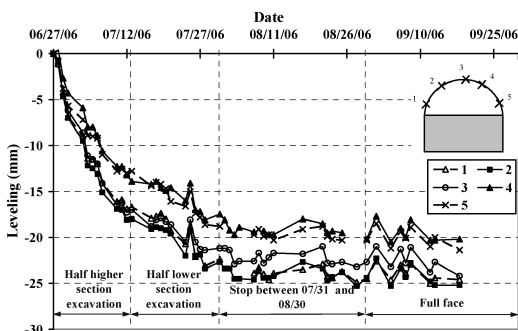


Figure 3. Leveling measurements in the half higher section D3.

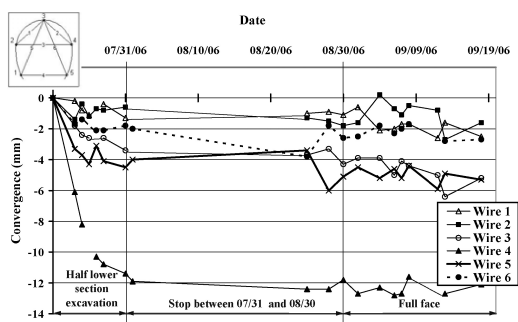


Figure 4. Convergences measurements in the half lower section D3.

(Fig. 4). For the other wires, convergences remain lower than 7 mm. The leveling measurements are not so high than in the half higher section. They remain lower than 6 mm. All targets show similar displacements so the deformation of the section is symmetric after the excavation of the lower part. Measurements in section D3 seem to be stabilized after the return towards an excavation in full face.

In section D4, 25 mm of convergence is registered for wire 4 (Fig. 5) and the speed of convergence is high despite of the distance from the face at the origin which is important (4 m). Vertical movements are important but remain lower than those registered during the digging of the half higher section D3. They reach 14 mm. The section is closed by an arch invert set up at the tunnel advance every 3 m. So, the section is not immediately closed. This can explain the leveling values. Measurements are not stabilized at the end of the excavation.

Displacements recorded in the clayey zone in the downward tube are more important than those measured in the other tube (Eclaircy-Caudron et al. 2007). Consequently, the clayey zone seems to be of better quality in the downward tube. This conclusion is also

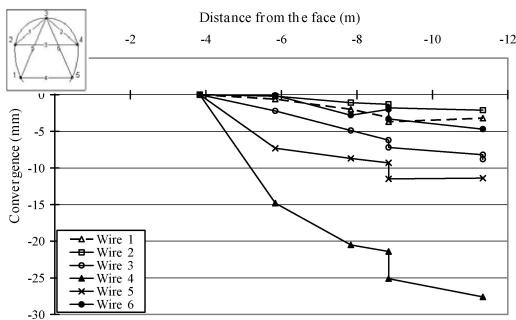


Figure 5. Convergences measurements in section D4.

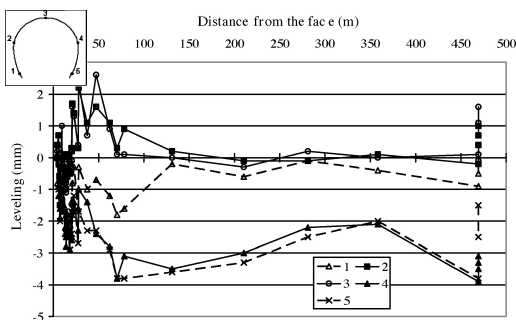


Figure 6. Leveling measurements in section D1.

verified by the strains measurements performed by extensometers ahead the face.

The section D1 is situated in interbeddings of marls and limestone. The convergence measurements show an important dissymmetry confirmed by the leveling measurements (Fig. 6). In fact, vertical displacements are more important for targets 4 and 5 and reach 4 mm. Maximal convergences values are obtained for wires 3 and 4 and reach 8 mm.

This section presents lower measured values than sections situated in a similar soil in the upper tube (Eclaircy-Caudron et al. 2007). This can be explained by the dip of the interbeddings which is more favorable in the downward tube (Fig. 7). In fact, the face survey shows almost horizontal interbeddings of marls and limestone. Moreover, the face survey presents less fractures and faults in this section than in similar sections in the other tube. So, the soil seems to be of better quality in the downward tube. Measurements are stabilized at a distance from the face equal to 60 m so 9R against 150 m in the other tube.

In section D2, located in marls, the maximal convergence value is obtained for wire 4 as in the section D1 and in marls in the upward tube. It reaches 12 mm before the end of the excavation from the Doubs portal at 03/17/06. Leveling measurements show a dissymmetric deformation of the section as in the upward

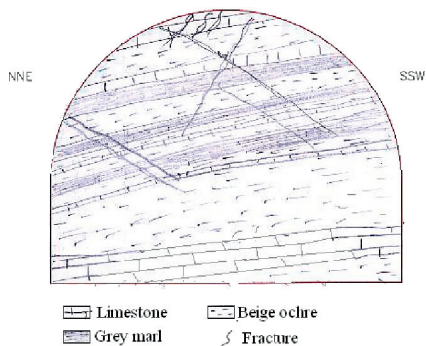


Figure 7. Face survey in section D1.

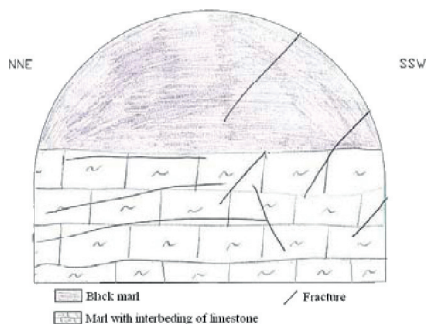


Figure 8. Face survey in section D2.

tube. Maximal vertical movements are recorded for targets 3 and 2 and reach 15 mm before the end of the excavation from the Doubs portal and almost 20 mm after. The side wall situated on the side of the other tube presents more displacements than the other while in section D1 the opposite is noticed. This dissymmetry can be explained by the presence of the exploration gallery and not by the geology which is perfectly symmetric (Fig. 8). Convergences measurements are stabilized before the end of the excavation from the Doubs portal (at 25 m). After the excavation from the Doubs portal and before the beginning of the excavation from the Vallon portal, the measured convergences increase. These displacements increments can be due to the construction phases (drilling of the first face bolts and of the first umbrella arch...). After the beginning of the excavation from the Vallon portal, the convergence measurements stop increasing. So, the digging from the other portal does not influence the convergences measurements of this section while in the other tube the opposite was noticed in this kind of soil. Concerning the leveling, no conclusion can be drawn because after the beginning of the excavation from the Vallon portal, these measurements present some unexplained variations.

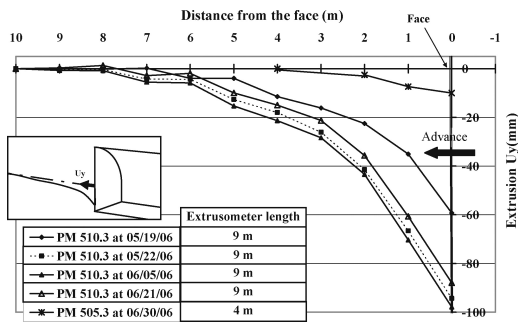


Figure 9. Extrusion evolution versus the distance from the face at different face positions (extrusometer 1).

3.2 Radial displacements in marl at PM 325

These measurements are realized by three borehole extensometers installed radially from the tunnel wall (at the crown and at 45°). Each extensometer has a length of 12 m and included 7 measurements points at 0, 1, 2, 4, 6, 8 and 10 m from the tunnel wall. These measurements can be used to assess the extent of the zone of influence around a tunnel. Movements are measured automatically with an accuracy of 0.02 m. The face was at a distance of 4 m at the origin of the measurements. The point located at 10 m is assumed to be outside of the influence zone. So, the displacement of each point is computed by considering that this point is fixed. The computed displacements can be compared to the convergences measurements realized in marls. Displacements are very low compared to convergences and remain lower than 2 mm. The distance from the face being equal to 4 m at the origin of displacements measurements, low displacements can be explained by the fact that a part of them was lost.

3.3 Displacements measured ahead the face in the clayey zone

Two extrusometers of 20 meters length were set up at PM 521 and 501. The feature consists to measure relative displacements between two successive points spaced by one meter. It is destroyed at the tunnel advance. If the anchor point can be considered outside the zone influenced by the excavation then absolute displacement of each point can be computed. Generally, the zone of influence extends until one radius ahead the face. So, to consider that the last point is fixed, the extrusometer has to be longer than one radius. The measurements obtained for each extrusometer are presented.

3.3.1 Extrusometer 1 (PM 521)

Figures 9 and 10 present the extrusion evolution versus the distance to the face and the PM.

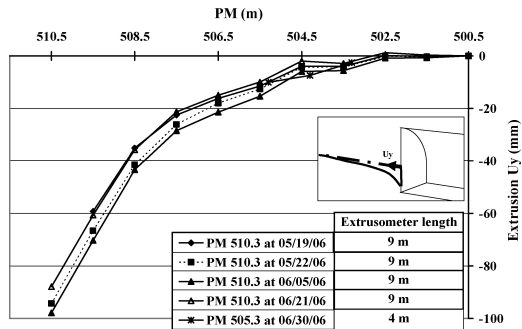


Figure 10. Extrusion evolution versus the PM at different face positions (extrusometer 1).

The length of the extrusometer is indicated in each case. Four measurements were performed when the face was stopped at PM 510 after collapses which took place in May 2006. After the resumption of the excavation in partial face only one measurement was realized and it is not exploitable due to the short length of the extrusometer. The maximal value of extrusion reaches 10 cm (so a strain U/R equal to 1.5% with $R = 6.8$ m). This value is four times greater than in the other tube. 80% of this extrusion is obtained in the first 4 m so at a distance lower than one radius as in the other tube. Due to a lack of measurements, it is not possible to determine the radius of influence of the face. Measurements should be realized at each digging phases.

3.3.2 Extrusometer 2 (PM 501)

Figure 11 shows the most important extrusion measurements performed with the second extrusometer. Fourteen measurements were recorded. One was made during the excavation of the half higher section. Four were performed between the end of the digging of the half higher section and the beginning of the excavation of the lower part. Four were realized after the beginning of the lower part excavation and three were made after the complete excavation of the part in partial face. Finally, only two measurements were realized after the return towards an excavation in full face. The curves evolution versus the distance from the face is different. This can be explained by a different behavior of soil or of the bolts. The maximal extrusion reaches 4 cm (so $U/R = 0.7\%$). This value is similar to the one obtained in the other tube.

3.4 Strains in the steel rib at PM 493

These measurements are realized by 14 vibrating wires extensometers installed on steel ribs as shown in Figure 12. They permit to obtain the strain of the steel rib. Movements are measured with an accuracy of $1 \mu\text{m/m}$. Temperatures are also monitored. The face

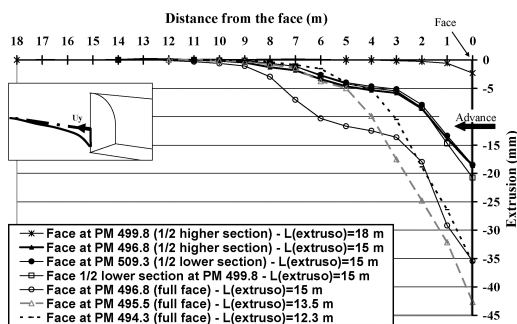


Figure 11. Extrusion evolution versus the distance from the face at different face positions (extrusometer 2).

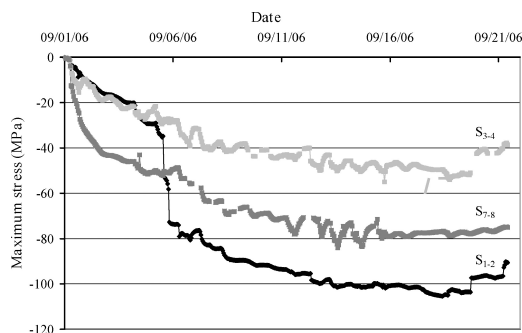


Figure 13. Evolution of the stresses in the steel rib at PM 493.

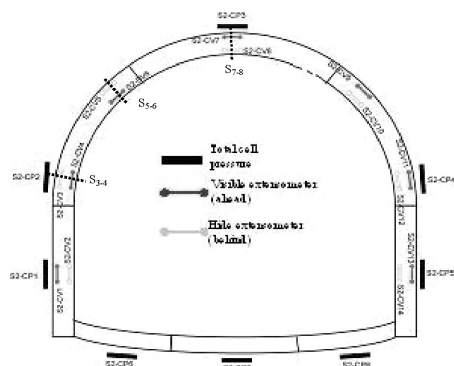


Figure 12. Location of the instrumentation at PM 495.

was at a distance of 1 m at the origin of the measurements. Maximal values are recorded in extensometers S2-CV1 and S2-CV2 and reach $400 \mu\text{m}/\text{m}$. So, the deformation of the rib is dissymmetric. However, the geology is the same in both sides. From the strains measured by a pair of extensometers it is possible to compute the maximum stress induced in the steel rib. For example, the stress S_{1-2} is determined from the measurements of S2-CV1 and S2-CV2. These stresses are important and the maximal value is obtained in extensometers 1 and 2 and reaches 100 MPa. However, this value remained lower than the limit stress (180 MPa). The computed stresses are shown in Figure 13. The same behavior than in the other tube is observed. Stresses induced in the steel rib permit also to compute the normal force and the bending moment. It appears that the steel rib works essentially in compression as in the upward tube. As regard to the measurements, structural elements are less loaded in the downward tube than in the other tube but displacements are higher.

4 CONCLUSIONS

This paper presents the most representative measurements which occurred during the excavation of the downward tube of the Bois de Peu tunnel in Besançon (France). The measurements permit to better understand the behavior of the complex support set up in the clayey zone and to adapt the support and excavation to the found conditions during the construction. The convergences measurements vary between 6 and 25 mm and the leveling between 4 and 25 mm following the soil. It appeared that structural elements are less loaded in the clayey zone of the downward tube than in the upward tube but displacements are higher. In fact, vibrating wires extensometers set up on the steel rib permit to conclude that this structural element is loaded at 55% of its admissible stress against 70% in the upward tube where the clayey zone seems to be of better quality.

In the upward tube, higher displacements are observed in the geological unit including interbeddings of marl and limestone where a heterogeneous geology is observed with many faults. In the downward tube, this unit is less fractured and displacements are reduced. The displacements monitored by extensometers show that the perturbed zone ahead the face extends to one diameter. And, 80% of the extrusion at the face is obtained at a distance lower than one radius. Comparisons between predictions and measurements permitted to evaluate the actual mechanical parameters of the soils found by the tunnel. Comparisons in the clayey zone showed that this layer had similar properties than those defined at the end of site investigations.

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