EPB TUNNELLING WITH SHALLOW COVER UNDER THE HISTORICAL LAVOV MOST (LIONS BRIDGE) IN SOFIA

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EPB tunnelling, shallow cover, risk analysis, mitigation measures.

INTRODUCTION

To respond to the quick growth of population and the traffic improvement in the city of Sofia, the second metro line is under construction (Figure 1). The line 2 running from North to South is 9.4km-long and consists of 11 stations as follows: the section Obelya – Nadezhda, (3km and 4 stations) at-grade in the North-West suburb of the city; the second section Nadezhda - Cherni Vrah (6.4 km and 7 stations) completely underground beneath the city centre.

The Bulgarian authorities has contracted the consortium SIM composed by SYSTRA and the Bulgarian partners INFRAPROJECT and METROCONSULT for the works supervisions of Lots 1 and 2 of the second section. The paper refers in particular to Lot 1, which starts at the North-West of the city and ends in the city centre just before station MC-9 and involves the construction of 3.8 km of bored tunnel and 3 stations (MS-5, MS-6 and MS-7). The tunnel is excavated with a 9.43m-diameter EPB TBM in soft soils. The internal diameter of the tunnel is 8.43m and the lining is 0.32m thick. The design-and-build contract for Lot 1 has been awarded to DOGUS, and TUNNELCONSULT is the contractor's designer.

Several hard points have been encountered along the tunnel alignment such as tunnelling with shallow cover beneath a railway tracks close to the TBM launching shaft, and the intersection with the Metro Line 1. However, the most sensitive tunnelling section has been the excavation under the Lavov Most (Lions Bridge), an historical masonry bridge built at the end of the 19th century over the Vladaiska River.

In this section, the average overburden is 3.5m and the vertical tunnel profile is very close to the bridge's foundations. The complexity of the excavation under the bridge is increased by the short distance (i.e., about 18m) from the TBM launching at the station MC-7.

The paper describes the approach followed to prepare and carry out the excavation of this sensitive section.

Figure 1 : Overview on the city of Sofia and alignment of the 2nd metro line in green.

BRIDGE DESCRIPTION

The structure of the bridge consists of granite blocks forming two arches of 11.70m span each. The bridge is about 22.5m long and 20.5m wide. The foundations are formed by wooden piles caissons 3.5m-long and 2.8m-deep, filled with large stone blocks. The blocks beneath the Northern and Southern abutments are 1.5m long while the foundation under the central pier is 2m long. The average distance between the tunnel lining crown and the foundation is 0.90m, while the wooden piles are only at 0.2m distance from the lining (Figure 2).

The horizontal alignment of the tunnel is almost parallel to the longitudinal axis of the bridge, the inclination between the tunnel and bridge horizontal axes being of about 12°.A preliminary visual inspection of the bridge has highlighted the presence of small cracks that could be associated to movements suffered by the bridge in the past. The existing cracks should not be reactivated by tunnelling-induced effects and hence mitigation measures should be designed to protect the bridge.

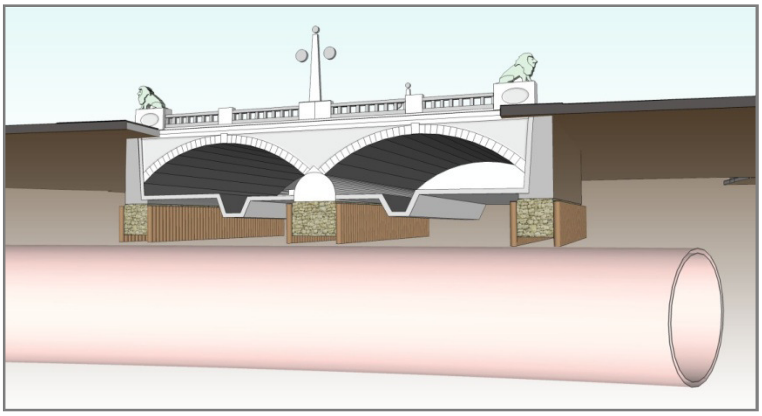

Figure 2 :3D render of the Lions Bridge.

GEOLOGICAL AND GEOTECHNICAL CONTEXT

The initial and the complementary geotechnical investigations at the Lions Bridge location allowed a detailed characterisation of the ground conditions. The geotechnical stratigraphy beneath the structure from the surface downwards can be summarized as follows:

- from ground level (537.4m a.s.l.) to elevation 532.3m: backfill consisting of gravel with sand filler, loam and silty clays (Unit 1 according to the geotechnical profile).
- from 532.3m to 528.3 m a.s.l.: Quaternary deposits (Unit 3 in the geotechnical profile) consisting of fine to medium gravel, cobbles and clayey-sand filler. The lower boundary of Unit 3 is located 4.5m below the river bed. The permeability is of the order of $8x10^{-5}$m/s
- from 528.3m: Pliocene deposits (Unit 7 in the geotechnical profile) consist of silty clays of high plasticity and high water content. The permeability ranges between $2x10^{-6}$ m/s and $1x10^{-9}$ m/s.

The Lions' bridge foundations are located in Unit 3, while beneath the bridge the tunnel is excavated mainly through Unit 7 with the very top of the tunnel crown in Unit 3, which constitutes the 3.5m of tunnel overburden. The water table is located at about 531.5m a.s.l.

During the geotechnical investigation campaign, three 30m-deep boreholes have been drilled, one close to each abutment of the Lions Bridge and the third in the river bed. In order to better investigate the bridge foundations and confirm the initial assumptions, three additional inclined boreholes have been realized. Boreholes BH1 (7.5m depth) and BH2 (15m depth) have been done in correspondence of the southern foundation and borehole BH3 (15m depth) was performed beneath the northern abutment. The additional investigations carried out in the bridge area have confirmed the foundation type and depth, and have shown that foundations are structurally independent from the bridge superstructure.

PREDICTION OF TUNNELLING-INDUCED SETTLEMENT

Numerical analyses were performed by the Designer in order to study the settlement distribution induced by tunnelling taking into consideration the option "doing nothing" (Model A) and the option of treating the Quaternary deposits of Unit 3 for the full height of the tunnel overburden, i.e., 3.5m (Model B). The 3D finite element models were performed with the software PLAXIS 3D.

The numerical models (Figure 3) were conceived to simulate the excavation of the first 40 rings after TBM launching at the station MC-7. The model simulates the real advancing of the tunnel excavation, consisting of 1.5m-long strokes, and applying the real EPB working pressure at the excavation face (is 0.5 bars at the tunnel crown and a gradient of +0.14bar/m toward the tunnel invert). The conical shape of the TBM shield and the presence of the lining installed at the rear of the shield are considered, together with the injection of the tail void. Due to the limits of the models, the tunnel alignment has been modelled parallel to the bridge axis with a distance between the two axes corresponding to the distance between the bridge and the lining at the Northern abutment (most critical condition).

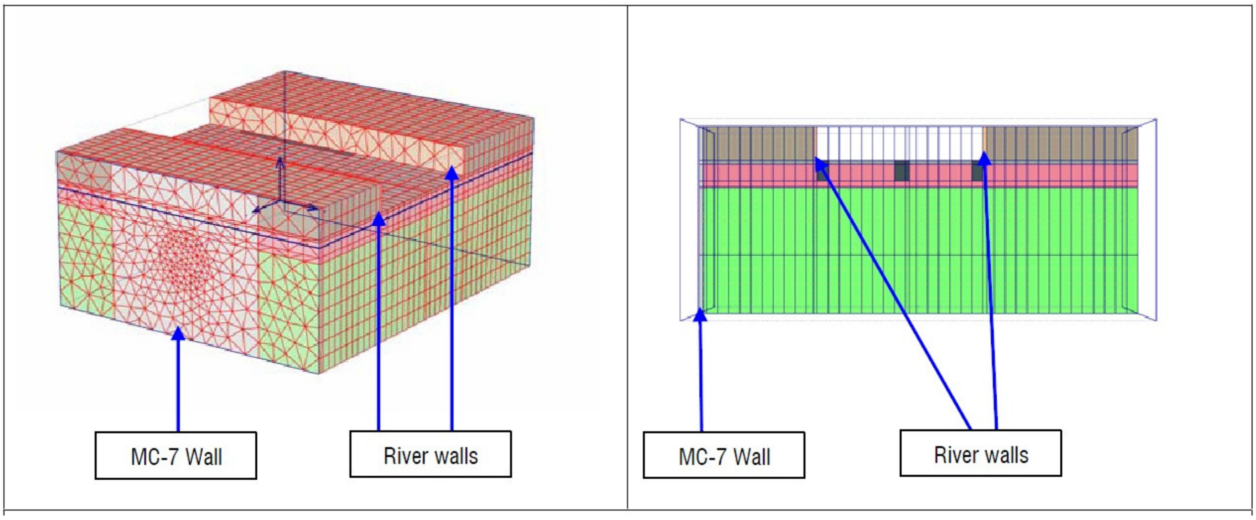

Figure 3: 3D model used for the finite element analyses.

The hardening soil model was used as constitutive law of the soil. The characteristics of the soil treatment in Unit 3 were imposed in terms of increased cohesion (i.e., from zero to 300kPa) and increased deformation modulus (i.e., from 26MPa to 300MPa).

The results of Model A showed a maximum settlement of about 8mm and an angular distortion of about 1/1000. Furthermore, the computed volume loss of 0.23% was in the expected range for an EPB excavation under controlled conditions, and confirmed the suitability of the proposed confinement pressures. However, this ideal situation could be easily perturbed considering the low tunnel cover, the soil conditions characterised by granular soil at the TBM crown, and the precision in maintaining the designed face pressure during the excavation with an EPB machine (±0.2 bar). A sudden and localised pressure loss would have been inacceptable for the project, as also shown by further numerical analyses.

Model B showed that the risk of a sudden loss pressure could be mitigated in a reasonable way by treating the gravel of Unit 3 beneath the bridge foundations (settlement limited to 3-4 mm and angular distortion to 1/3500. The soil treatment has been then designed to achieve the required characteristics of the treated soil.

RISK ANALYSIS AND MITIGATION MEASURES

The technical solution proposed by the Contractor for the TBM launching and the TBM drive under the Lion's Bridge (Figure 4) was optimized through a process of risk analysis in which all the involved parties took part (Figure 5).

The ground treatment consisted of a preliminary consolidation of the bridge foundations through cement injections through inclined sealed-in, 5m-long sleeve pipes. The pipes were left in-place till the TBM passage, in order to eventually execute additional injections in case of excessive movement induced by the tunnel excavation.

Unit 3 was then treated by permeation grouting, using a low injection pressure (between 1 and 2 bars) in order to avoid "claquage". Considering the grain size distribution of the soil to be treated, cement with high Blaine number (about 9000 cm^2/g) was recommended for the effectiveness of the treatment. Two 5m-deep cut-off walls were created upstream and downstream the bridge in order to form a limited area in which confining the grouting. The injection mesh consisted of 4m-long injection holes spaced at 1m and disposed in staggered configuration. The cement injections were realized with the method "bottom to up", without manchette tubes. An extension of 580m^2 was treated using about 283tons of grout.

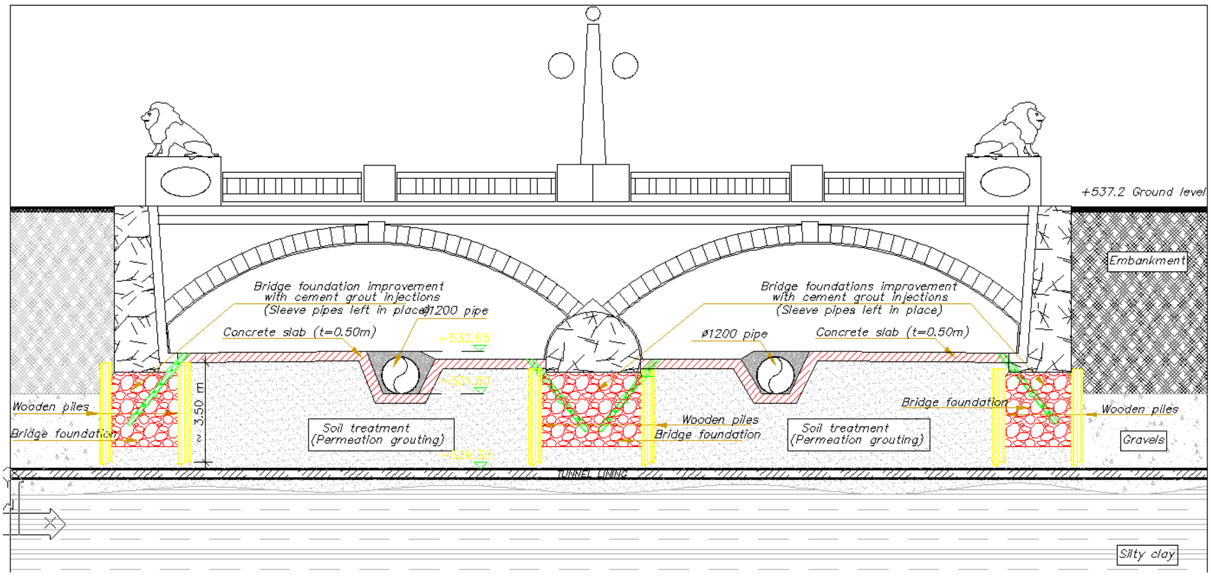

Figure 4 : Ground treatment scheme of the Lions Bridge foundations.

The following additional mitigation measures were also implemented:

- interruption of traffic over the bridge during the TBM drive in this section;
- divert the water into pipes;
- installation of a temporary a scaffolding system beneath each bridge arch in order to support the bridge deck in case of excessive settlements;
- casting of a 0.5m-thick concrete slab reinforced with wire mesh on the river bed previous to the ground improvement; this element had the double role of offering a plane working platform for the ground injections and of minimizing the possible horizontal bridge movements during the tunnel excavation;
- drilling/injection rigs ready on site;
- implementation of an accurate monitoring system associate with the interpretation of the TBM operation key-parameter during the TBM drive.

Figure 5 shows the risk register prepared for analysing the major identified risks related to tunnelling under the bridge, to assess the initial risk, to identify the required mitigation measures and to list the contingency measures to control the residual risk. The Risk assessment was extended to the break-out section in station MC-7, due to its proximity to the bridge. The lesson learned during the TBM break-out in the previous stations allowed improving the TBM launching at Station MC7 and to well define the steering procedures for tunnelling under the bridge.

Previous to tunnelling beneath the bridge, slug tests both outside and inside the improved ground area were performed to check the effectiveness of the soil treatment, showing a residual permeability of the treated gravel of $7.7x10^{-7}$ m/s.

AREA	HAZARD	CAUSES	CONSEQUENCES	INITIAL RISK	MITIGATION MEASURES	RESIDUAL RISK	CONTINGENCY MEASURES
STATION 7 BREAK OUT AREA	Water leakage at the tunnel-eye joint	- Absence of reservation for the sealing joint at the tunnel eye in the raft - Bias between tunnel axis and tympanum at the TBM start-up	- Stoppage of the TBM - Excessive settlement due to water/soil inflow - Cracks potentially appearing on the bridge piers close to the station	H	- Complete full-round reservation for sealing joint at the tunnel eye -Soil treatment at launching area	L	
	Lower then prescribed confinement pressure in the TBM working chamber	- Loss of pressure through the tunnel eye joint if not full-round. - Defect in the installation of the joint - TBM operator error	- Stoppage of the TBM - Excessive settlement - Cracks potentially appearing on the bridge piers close to the station	H	- Make a complete full-round joint - Substitute the usual break-out scheme with a more reliable system. - Review of the break-out procedure and of TBM's operator instructions before the start-up	L	- Further review of TBM procedures and operator instructions during construction based on monitoring results. - Secondary injections of the tail void through the installed segmental lining.
	Ground loss in the break out area	- Over excavation - Insufficient face pressure	- Stoppage of TBM, - Excessive settlement due to lack of face stability	H	- Continuous control of extracted weight and face pressures -Additional soil treatment of the upper gravel unit at break-out area	VL	Maintain an active drilling rig on site for interventions from the surface in case of anomalies
CROSSING the BRIDGE ABOUTMENT	Wooden piles in the TBM excavation chamber	- Defect in the soil investigation - Lack of information	- Stoppage of the TBM for removing the wooden pile under compressed air - Possible instability of the soil at the tunnel face (air loss); - Sudden increment of measures pressure due to soil collapsing into the front chamber - Volume loss and settlement under the bridge foundation	H	- Additional investigations and extraction of a wooden pile to check its length - Soil treatment to stabilise the soil around the bridge foundations - Installation of a supporting steel frame under the bridge to protect the structure. - Procedure for hyperbaric interventions in the TBM chamber	L	- Maintain an active drilling rig an injection equipment on site to be able to do interventions from the surface in case of anomalies - Fill the masonry arches with self placing concrete in between two bulkheads.
CROSSING THE RIVER	Loss of pressure with foam leakage to surface	- Face pressure above the designed value, heave and soil cracks - Sleeve pipes left open and in contact with the tunnel crown - Defect of the soil treatment or of the concrete slab	- Stoppage of TBM - Excessive settlement at river level potentially leading to damages on the bridge	H	-Concrete slab - Confine the grouting area when treating the gravels. - Fill in the injectionholes. - Monitoring system checking continuously the settlement/heave and strictly interpreted with TBM data	L	- Maintain an active drilling rig an injection equipment on site to be able to do interventions from the surface in case of anomalies .
	Differential settlement of Lions Bridge	- Defect of the soil treatment beneath the foundations or the bridge arches. - Face Pressure different than the designed value - Over-excavation or instabilities due to wooden piles pulled into the TBM chamber.	Cracks on the bridge	H	- Monitoring design + thresholds definition - Real-time Monitoring - Reinjectable upper level of TAMs under the foundations - Continuous and systematic control of excavated quantities and face pressure. - Installion of a supporting steel frame under the bridge to protect the structure.	L	-Reinjection of TAMs beneath the bridge piers)
	Possible sticky behaviour of the clay	- Presence of plastic clay (layer 7)	- Slow TBM advancing - Interventions in the chamber - Potentially increases of settlements at the surface due to slow advance	M	- Injection of polymers or water in the excavation chamber to condition porperly the excavated material - Control the trend of the TBM torque and of the total thrust	VL	- Review the use of additives - Wash the cutterhead (with high pressure)

Figure 5: Simplified risk register developed for the TBM drive beneath the bridge

BACK-ANALYSIS OF PREVIOUS TBM PERFORMANCES

A very important factual element for implementing an optimized set of mitigation measures has been the level of confidence achieved with EPB tunnelling along the previously excavated section.

An accurate back-analysis of the TBM parameters and the monitoring results for the tunnel stretch from station MC6 to MC7 (pk 5+507 – 5+990) was performed by the Supervision Team. In this 480m-long stretch, the tunnel was mainly excavated through granular soils.

In particular, the tunnel face support pressure, the extracted weight at each ring and the longitudinal grouting volume with the associated pressures recorded during EPB tunnelling were back-analysed and compared with the reference values and the relevant operational range (see Figure 6 and Figure 7). Then, the correlation between the excavation parameters and the induced surface settlements expressed in terms of "volume loss" has been considered in order to evaluate the response of the ground towards the TBM excavation procedure.

In the back-analysed stretch, a surface benchmark was installed every 10m, in correspondence with the tunnel axis, and was monitored daily. Furthermore, n.14 transverse monitoring sections were equipped with 6 levelling points at the distance of 5, 10 and 14m from the tunnel axis. The area from 50m ahead the tunnel face to 50m behind it was considered the active zone in terms of TBM influence and it was monitored at a higher frequency. The transverse settlement profiles were hence recorded and plotted.

Through a parametric analysis, the theoretical Gaussian curves describing the tunnelling-induced settlement trough were best-fitted to the measured settlement and the dimensionless parameter *k* of the well-known Gaussian formulation was back-calculated in order to match the measured settlement profile. A satisfying fitting was obtained for $k = 0.33$. This result is in agreement with the values of 0.2-0.3 suggested in the technical literature for granular soils. Finally, the volume loss has been calculated along the analysed stretch and the results have been plotted and compared with the above mentioned EPB parameters.

The controlled steering of the TBM allowed limiting the maximum settlement to 16mm (average settlement < 7mm) and the maximum volume loss to about 0.3% (average back-calculated values ranging between 0.1 and 0.2%). The maximum values were recorded in the learning curve section.

The back-analysis emphasized the following aspects:

- keeping the extracted weight and the confinement pressure within the predefined operational ranges was possible with a relative high level of confidence;
- settlement and volume loss were strictly related to face pressure and the injection pressure in the tail void, and could be easily controlled by acting on these two key-parameters;
- anomalies in the key parameters could be easily identified through a systematic follow-up of the TBM performances, could be interpreted and correlated to the induced effects at the surface, and corrective actions could be implemented through the existing TBM tunnelling procedures.

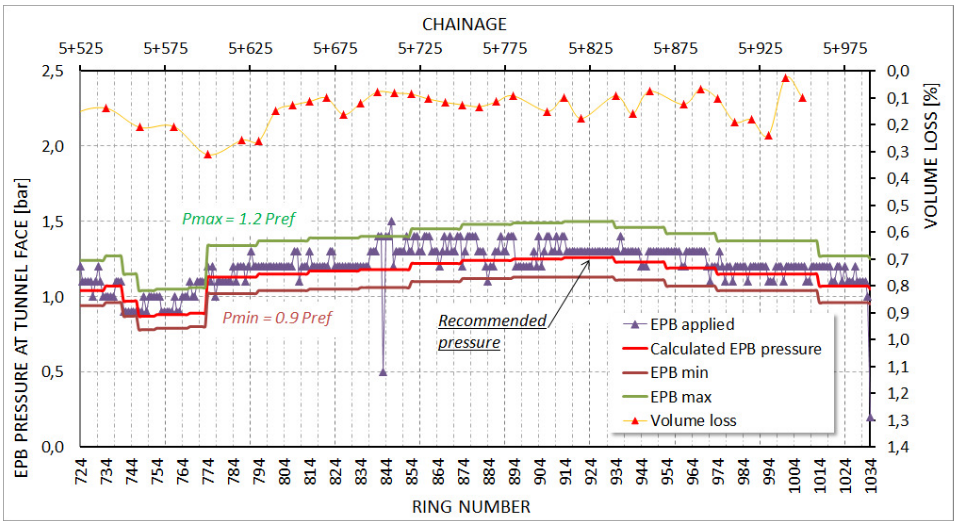

Figure 6: EPB confinement pressures measured at the sensor located 0.7m below the tunnel crown, and back-calculated volume loss in the stretch MC6-MC7

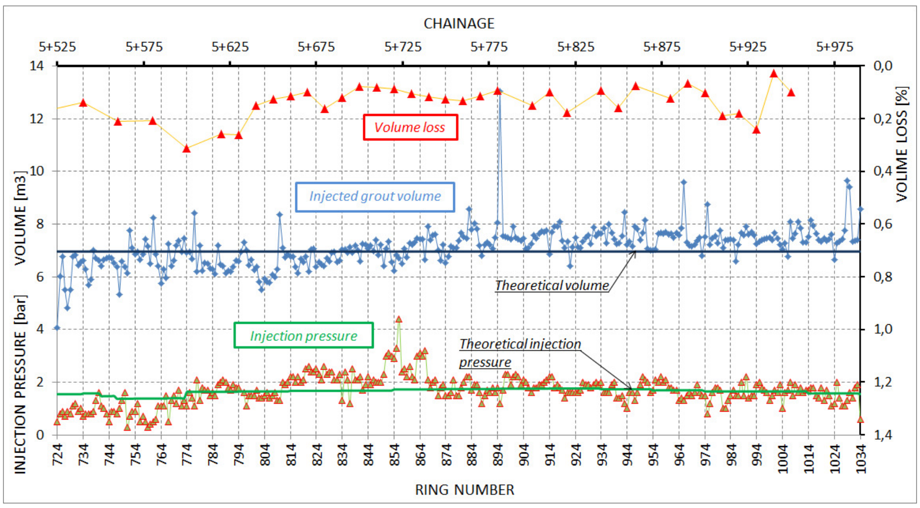

Figure 7: Grouting parameters vs. volume loss in the stretch MC6-MC7

As an example, during the excavation of ring 843 a sudden pressure loss to 0.5bar was detected. This was explained with the occurrence of highly cohesive and plastic silty-clay lenses at the tunnel excavation face, forming plugs in the excavation chamber and sort of blocks obstructing the base of the plenum of the screw conveyor and choking the muck circulation. In parallel an increase of rotation speed and torque were observed. Due to the temporary loss of pressure, the gap around the shield converged more than usual and higher grouting pressures had to be applied in the subsequent rings. The presence of silty-clay lenses in the area reduced the effect of settlement at the surface.

The grouting volumes and pressures were slightly lower than the theoretical during the learning curve. Then, grouting volumes remained almost constant, and generally 8-10% higher than the theoretical annular gap volume. An anomalous grouting volume of 12m^3 was recorded when assembling ring 895. However, this value was measured at the end of one day of TBM stoppage during which the injection system was cleaned. The sensors had probably recorded the water flow passing through the tubes and the volume was not representative of grouting (also the grouting pressures remained close to the design values).

TUNNELING BENEATH THE BRIDGE - RESULTS

A monitoring plan was defined to measure the movement induced at the surface, and at the bridge deck and foundations during both the soil treatment operations and during the TBM excavation.

Levelling points were installed at the surface in correspondence with the projection of the tunnel axis, at a spacing of 10m, and n.3 transversal monitoring sections consisting of 5 benchmarks each were installed at the ground level close to the abutments and the central pier. In addition, 6 levelling points were installed on the bridge foundation. The measures were taken by two theodolites positioned in the river bed and a levelling station at the surface level, all being positioned outside the TBM influence area.

The measured total settlement due to TBM tunnelling show a maximum value of 7mm recorded on the left side of the Northern bridge abutment, and recorded when the TBM got just outside the treated soil block for the break-out from Station MC-7. The average settlement measured on the street level along all the treated area and at the bridge foundations ranged between 1 and 4mm (Figure 8).

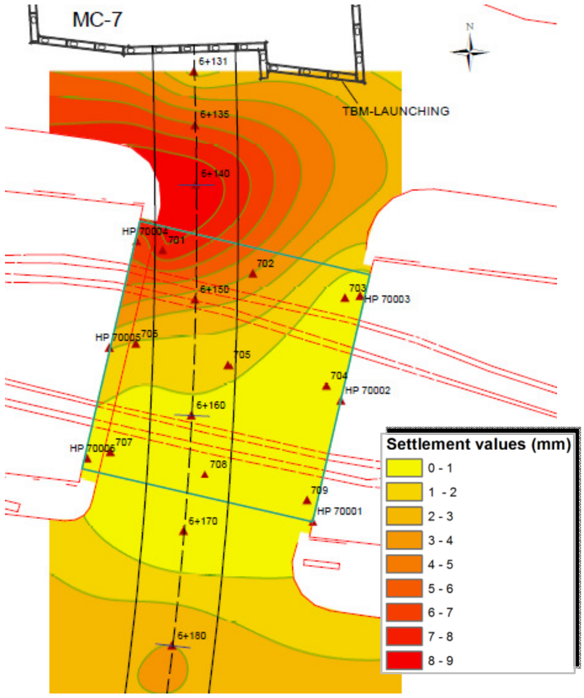

Figure 8 : Tunnelling-induced settlement from Station MC-7 to the complete crossing of the bridge

During the TBM launching the confinement pressures and injection parameters were slightly lower than recommended, leading to the mentioned maximum settlement. The TBM procedure was rapidly adjusted, especially in terms of face pressure, in order to minimise volume loss and settlement during the bridge crossing. This was also possible thanks to the strength of the treated soil above the tunnel crown, which prevented heave. The injection parameters of the tail void were generally in line with the prescriptions, but with injection volumes 15% lower than the theoretical. Secondary injections were executed directly through the segments to treat any possible residual weakness. The maximum back-calculated volume loss is 0.18% (launching section). Then the volume loss has been progressively better controlled achieving negligible values.

After a visual inspection of the bridge done by the Engineer the support frame was finally and the dismantled since any functional and structural damage occurred.

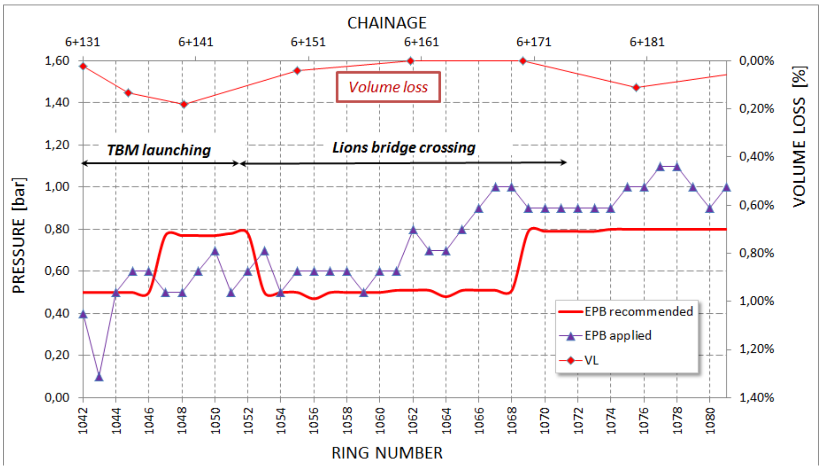

Figure 9 : Face pressures at the crown vs. back-calculated volume loss when tunnelling under the bridge.

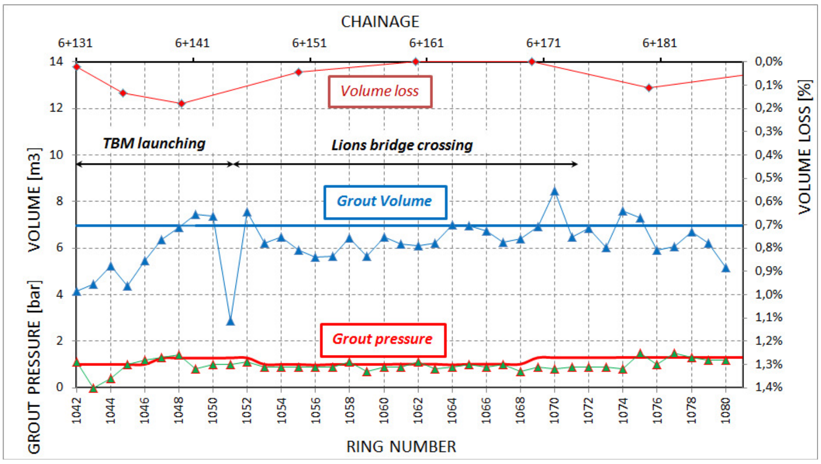

Figure 10: Tail void injection volume and pressures vs. volume loss when tunnelling under the bridge.

CONCLUSIONS

The paper presents the approach followed for tunnelling with a 9.43m diameter EPB TBM under the Lions Bridge, in Sophia, in difficult soil conditions and with a reduced overburden. Numerical models were run to justify and quantify the need of soil treatment at the tunnel crown, in Unit 3 (gravels) and a risk management approach has been used by the involved parties to identify the risks and propose adequate mitigation measures and contingency measures. Massive soil treatment and/or bridge underpinning were avoided thanks to confidence in the TBM performance and procedures, gained in the previously excavated stretches. The detailed follow-up and back-analysis of the TBM and monitoring data in the previous stretches allowed adapting the TBM launching procedure and to set up the EPB parameters for successfully tunnelling under the Lions bridge.

ACKNOWLEDGMENTS

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