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Performance of a shield driven sewer tunnel in the Val-de-Marne, France

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ABSTRACT: Several tunnels have been constructed in the Val-de-Marne, over the past ten years, as part of a major sewer network development, in the eastern suburb of Paris. Most of these tunnels are around 3 meter wide ; they were completed in soft soils, at shallow depth and in urban areas, using the pressurized shield technology. One of these tunnels, termed the VL4 project, was more carefully instrumented, as part of a research project on the numerical analysis of shield driven tunnels. This tunnel, 3.35 m in diameter, was bored in alluvial deposits, using a slurry shield, and lined with precast concrete segments. Instrumentation of the VL4 project included bore-hole extensometers and inclinometers installed in the ground, as well as strain gauge extensometers, set in the precast concrete liner segments. This paper summarizes the results obtained from this instrumentation program.

1 INTRODUCTION

In 1987, the Val-de-Marne Département, undertook a major sewage improvement project, including the construction of several kilometers of large size sewers and of a water treatment plant at Valenton, in the eastern suburb of Paris ; the project, termed "Seine Propre", was aimed at improving the water quality of the river Seine, upstream of Paris. These sewers were mainly constructed at shallow depth, in soft soils and in an urban environment, using tunnel boring machines ; the average size of the tunnels was in the order of 3 m.

Because of the environmental constraints associated with these projects, several tunnel sections were instrumented, to monitor the ground movements induced by the construction process. In particular, the VL4 project, which included the completion of a 2 kilometer long, 8 m deep tunnel, was carefully instrumented at one section, as part of a research project on the numerical analysis of shield driven tunnels. The tunnel is located next to the river Seine, between the cities of Vitry-sur-Seine and Créteil, in the Val-de-Marne Département.

For practical purposes, tunneling sections were termed with reference to their metric point (PM), with respect to the starting station. The instrumented section was installed at PM 1200, where the tunnel was entirely bored in a uniform layer of alluvial deposits. The present paper summarizes the main results obtained from this instrumentation program. More detail on the instrumentation devices and recorded data can be found in Atahan (1995).

2 INSTRUMENTED SECTION

The instrumented section at PM 1200 is shown in Figure 1 ; the soil profile encountered at this location includes : a man made fill, between 0 and 2.5 m ; a layer of recent alluvial deposits, consisting of 2.5 m of silt and 1 m of silty sands, between 2.5 and 6 m ; a layer of older alluvial deposits, consisting of sands and gravels, between 6 and 10 m. The substratum, made of Saint-Ouen marly limestone, is found at a depth of 10 m. The water table is 4.75 m deep.

Typical soil properties for the alluvial deposits are displayed in Table 1. The recent formations consist of silts and silty sands of low density (the undrained shear strength of the silts was estimated to 60 kPa) ; both layers include cohesive materials of low plasticity. The older alluvial deposits are very permeable (with permeability coefficients in the order of 3.5×10^{-3} m/s) and significantly stronger than the more recent formations (pressuremeter tests showed limit pressures ranging between 1.5 and 2.5 MPa in the older deposits and of 0.4 and 0.5 MPa in the silt and silty sand layers). The substratum is relatively impervious.

The tunnel is located at a depth to center-line of 7.75 m, and lies completely within the very permeable older alluvial layer at PM 1200. It was excavated with a 5.60 m long, 3.35 m diameter

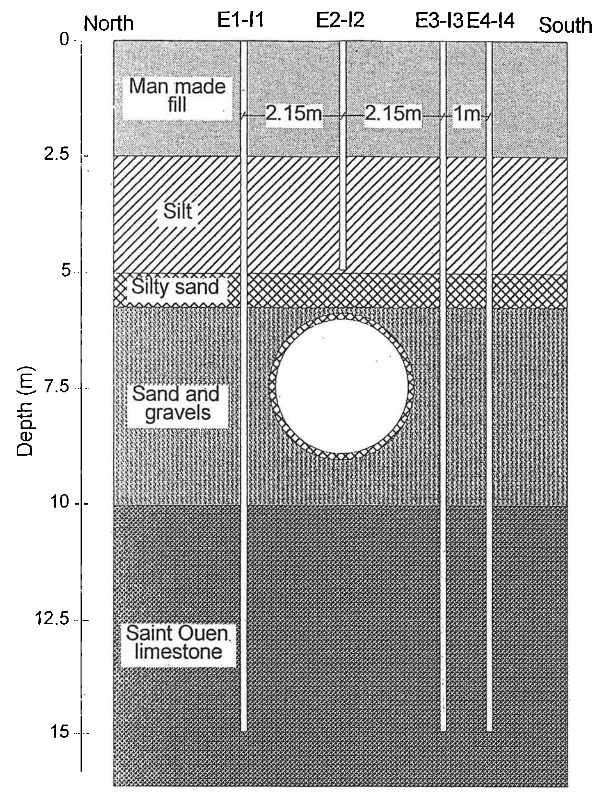

Figure 1. Instrumented Section at PM 1200

slurry shield, manufactured by Fives-Cail Babcock, and lined with a ring of 15 cm thick precast concrete segments. An additional liner, made of 20 cm thick cast-in-place concrete was to be installed after completion of the excavation works. Slurry pressures used at PM 1200 were in the order of 65 kPa.

Table 1. Soil Properties For the Alluvial Deposits

Soil Layer	γ (kN/m^3)	w_L (%)	I_P (%)	w (%)	k (m/s)	S_u (kPa)
silty sands	18	27	10	22		
silt	20	46	20	40		60
older deposits	18				$3.5\ 10^{-3}$	

The PM 1200 section was instrumented with four lines of bore-hole extensometers and inclinometers, installed from the ground surface, along the center-plane (extensometer E2 and inclinometer I2) and on each side of the tunnel (extensometers E1, E3, E4 and inclinometers I1, I3, I4). Because of the small size of the tunnel, and subsequent expected influential area, the side apparatus was installed relatively close to the tunnel, with lines E1, I1, E3 and I3 being as close as 50 cm to the springlines of the tunnel (Figure 1).

Surface settlements in the instrumented area were recorded through a network of 35 markers (Figure 2), set into the ground along five cross-sections, at PM 1200, as well as 5 and 10 m ahead of and behind the instrumented section. Two additional instrumented lines (extensometer E5 and inclinometer I5) were installed along the tunnel center-plane at PM 1205, i.e. 5 m father than PM 1200 (Figure 2). Both E5 and I5 were set at a depth of 15 m, to gain insight into the ground motion induced ahead of the tunnel face by the slurry shield.

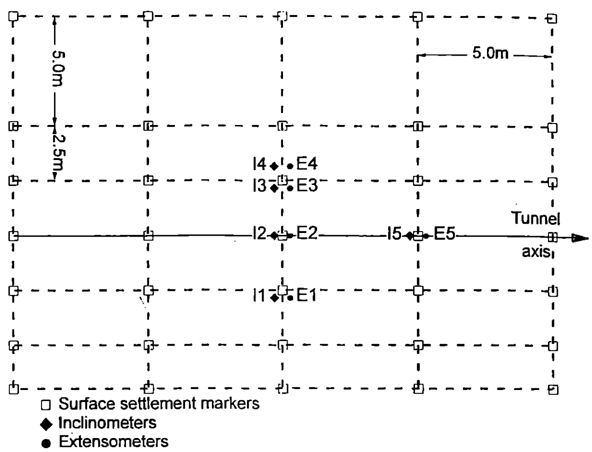

Figure 2. Surface Settlement Markers

The liner ring installed at PM 1200 was also instrumented, by means of strain gauge extensometers, cast into the concrete segments, to monitor the loads carried by the initial liner during and after construction. Figure 3 shows the location of the measurement points. Each ring consisted of four main segments (A and B, on the southern side ; C and D, on the northern side) and one key segment. The southern side of the ring was more carefully

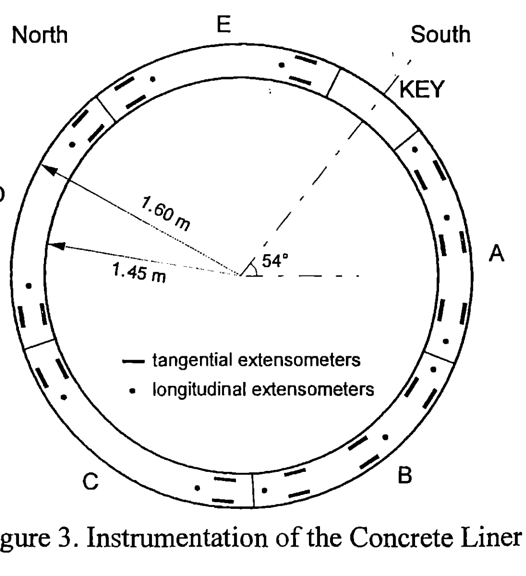

Figure 3. Instrumentation of the Concrete Liner

instrumented, with both segments A and B being equipped with three sets of strain gauge extensometers, whereas only two sets of extensometers were installed on the north side segments (C and D). Each measurement point included three strain gauge extensometers : two tangential extensometers, set along the inner and outer surfaces of the liner to measure hoop stresses, and one longitudinal extensometer, set along the tunnel axis in the center part of the concrete section to measure axial stresses in the liner.

3 TUNNELING INDUCED GROUND MOTION

Ground movements started to be recorded as the tunnel front approached PM 1191, i.e. 9 m before reaching the instrumented section at PM 1200. The monitoring devices were hand operated. As the machine passed through the instrumented area, construction was halted after each drive to allow for the readings to be made on the settlement markers, as well as extensometer and inclinometer lines. The whole monitoring operation lasted about 40 hours. Additional readings were taken a few days later, as the tunnel front reached PM 1250.

Some measurement difficulties were encountered during the monitoring period, which resulted in inaccurate surface settlement data in the vicinity of PM 1200 ; in addition, inclinometer readings could not be recovered for three drives (PM 1208, PM 1209 and PM 1212), because of a power loss in the measuring apparatus, and no measurement could be made below 7 m on the two extensometers (E1 and E3) installed nearby the tunnel center-plane, after the tunnel front had passed PM 1200, probably as a result of excessive displacements induced at this depth by the machine.

Because of these monitoring difficulties, only part of the settlement readings could be analyzed. Figure 4 shows the surface settlements recorded at PM 1200 for 5 locations of the tunnel face : PM 1206, PM 1207, PM 1209, PM 1212 and PM 1216. Settlement values are relatively low, with a maximum of 5 mm at the tunnel center-line. The shape of the surface settlement troughs observed across the tunnel section, resembles that of a reversed error function curve, as described by Peck (1969), for most of the data. For two sections, however (PM 1206 and PM 1207), a much flatter settlement curve was obtained. It is to be noticed that these unusual settlement profiles were only observed in the vicinity of the shield tail, which very likely reflected the tendency of the machine to restrict the vertical ground movements above the tunnel crown, as it passed underneath the instrumented section.

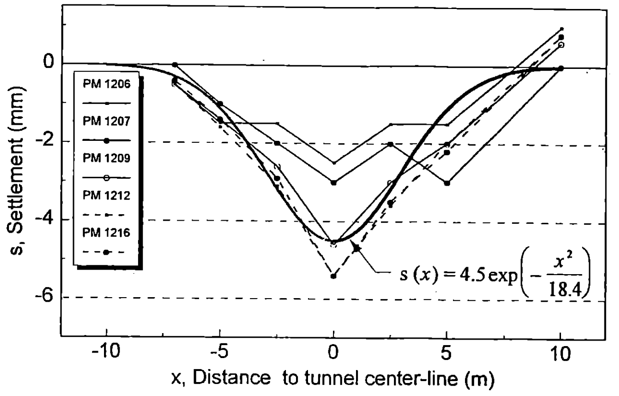

Figure 4. Surface Settlements Recorded at PM 1200

The analysis of extensometer readings could not yield any absolute value of the vertical displacements induced into the ground by tunneling, as a result of the above mentioned inaccuracies in surface settlement measurements. Some insight into the ground deformations at depth could, however, be obtained through these records. Figure 5 shows the deformation profiles obtained on E2 for 6 locations of the tunnel front : PM 1198, PM 1200, PM 1201, PM 1205, PM 1206, and PM 1209. It is apparent that very little deformation occurred ahead of the tunnel face (PM 1196), whereas some compression (up to 0.6 mm displacement between two subsequent 1 m spaced reference rings) took place in the ground above the tunnel crown, as the shield reached the instrumented section (PM 1200 and PM 1201) : this compression could be related to some ground heave induced by slurry pressures applied at the tunnel

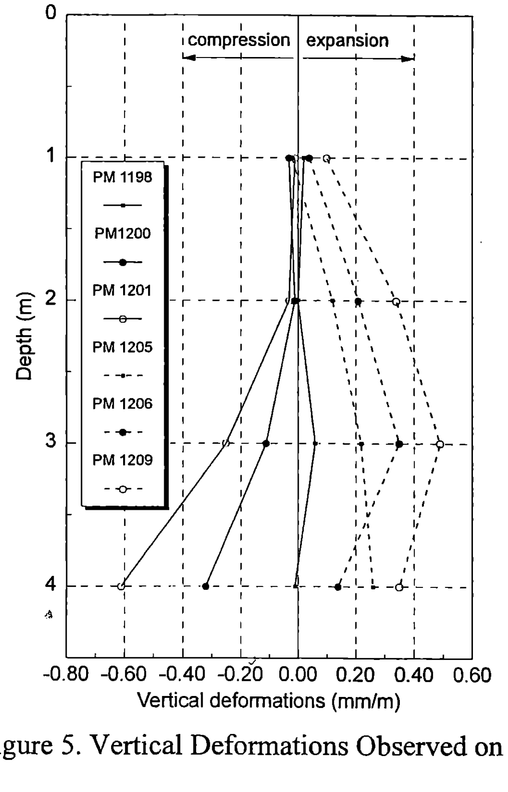

Figure 5. Vertical Deformations Observed on E2

face, and is consistent with the flatter settlement curves observed above the shield at the ground surface (Figure 4). As the tail of the shield moved away from the instrumented section (tunnel front at PM 1206 and PM 1209), the deformation process tended to be reversed, with some expansion occurring in the ground between 1 and 4 m depth. Ground expansion seemed to be restricted between 3 and 4 m depth after PM 1206, probably because of grouting operations into the tail void. Records at PM 1250 showed a reduction in ground expansion with time.

The influence of face pressures on the deformations observed over the tunnel crown is emphasized in Figure 6, where slurry pressures at the tunnel face are plotted together with the cumulated vertical deformations monitored within the ground located between the soil surface and tunnel crown as the shield approached the E2 (Figure 6a) and the E5 (Figure 6b) lines. Working face pressures in this area were set equal to 60 kPa ; the higher pressure values displayed on both Figures 6a and 6b were monitored during the driving stages. Figure 6a shows that ground expansion ahead of the tunnel front remained small at E2, where face pressures were kept above 60 kPa ; instead, some compression occurred when the shield reached PM 1200. At E5, on the other hand, ground compression was recorded 3 m ahead of the tunnel front, where a local increase in face pressure was experienced (Figure 6b), whereas some expansion occurred at PM 1203, after the face pressure had temporarily dropped to 50 kPa.

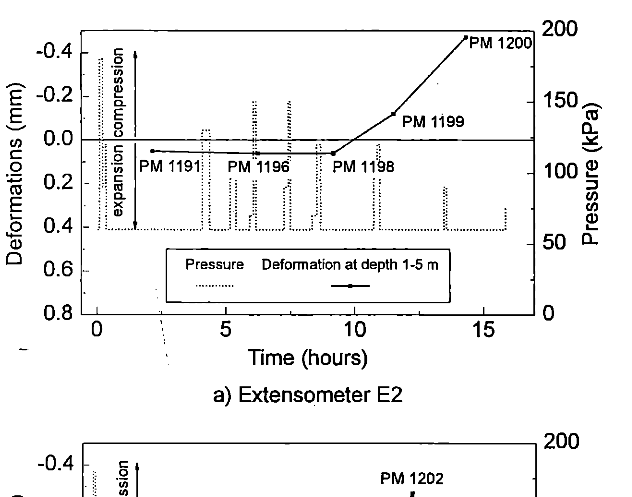

Figure 6. Interrelation Between Face Pressures and Vertical Ground Deformations at E2 and E5

Figure 7 shows the horizontal displacements measured on inclinometers I3 and I4. Outward displacements in the order of 1 mm were recorded on I4 at tunnel depth, as the shield reached PM 1200 probably because of the slurry pressure applied at the front (Figure 7b). This lateral displacement was followed with some convergence towards the tunnel, after the tail of the shield had passed the instrumented section (tunnel face at PM 1207). Typical ground movements observed on I3 are displayed in figure 7a ; records on I1 showed similar trends with the displacements being less significant that on I3. Figure 7a shows that a slight convergence was observed on I3 ahead of the tunnel face ; as the shield reached PM 1200, the ground tended to converge towards the tunnel center-plane between 1 and 6 m depth, and to move away from the tunnel between 6 and 11 m depth, with a peak in lateral displacements at a depth of 10 m. This trend was emphasized as the shield proceeded, with displacements in the order of 3 mm above tunnel level and 5 mm at 10 m depth. Because of the peak in horizontal displacement observed at 10 m depth, these data were difficult to analyze. In particular the lateral displacements induced by the shield at tunnel level were lower than those recorded at I4, even though this latter line was set 1 m farther from the

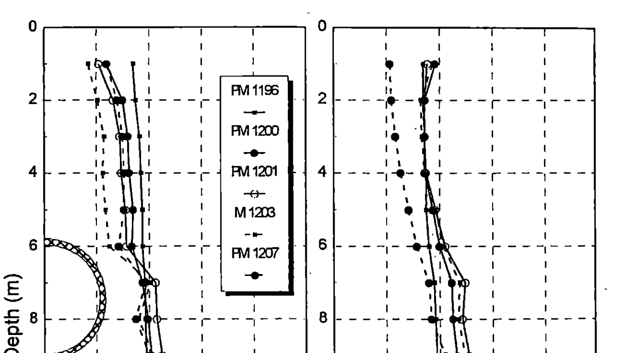

Figure 7. Horizontal Displacements Observed at I3 and I4

tunnel center-plane than I3. Part of this phenomenon could be due to the inclinometer response to the applied slurry pressure at the face, since the data actually reflected pipe deflections at I3 and were related, but not necessarily identical to ground movements at this location. Observed deflections were probably emphasized because this inclinometer was set very close to the tunnel springline. The same trend was observed along both the longitudinal and transverse directions for I1 and I3. It should also be noticed that the larger deformations took place at the interface between the pervious sand and gravel layer and the stiffer impervious limestone formation.

4 LOADS TRANSFERRED TO THE LINER

The loads carried by the tunnel liner were derived from the strain measurements performed in each of the four instrumented precast concrete segments. The instrumented ring reached PM 1200, as the shield face was located at PM 1204, and readings were taken for each drive thereafter until the tunnel front reached PM 1209 ; additional measurements were made for the following locations of the front : PM 1212, PM 1230, PM 1250, PM 1306 and PM 1250. These data were corrected to account for temperature changes.

The evolution of average liner forces with time is plotted in Figure 8. Three measurements were performed when the shield face was at PM 1204 : the data in Figure 8 show an increase in liner forces between the first and third readings, before the liner ring was actually exposed to ground loads; these forces could result from the ring's weight, as well as the action of the adjacent ring. The liner forces increased steadily with time until the tunnel front reached PM 1230, and a sharp reduction in ring forces was observed. Because some compressed air had to be used inside the tunnel, as the shield

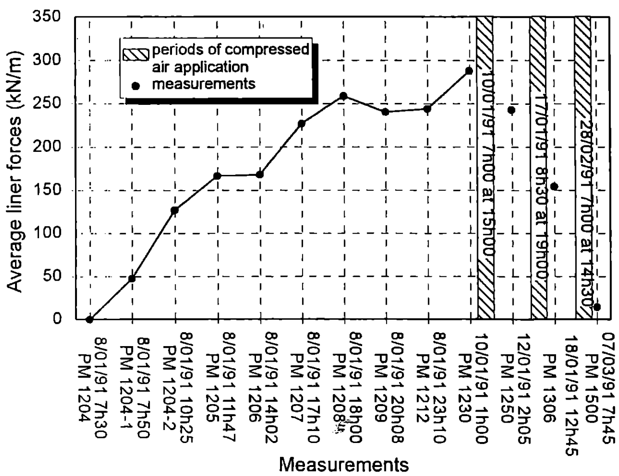

Figure 8. Average Liner Forces versus Time

proceeded between PM 1230 and PM 1500, it could be inferred that these force reductions were related to the applied air pressures. The periods when compressed air was used inside the tunnel are indicated in Figure 8 : it is apparent that a strong reduction in liner forces was observed each time compressed air was applied. Liner forces remained unchanged after removal of compressed air. It should be noticed that the maximum value of liner force (270 kN/m), obtained at PM 1230, is close to the hoop force derived from the full application of earth loads to the liner. No reduction in liner forces apparently occurred as a result of stress redistribution ahead of the tunnel face, which could result in part from the grout pressures used during the tail void filling operations ; the application of compressed air after PM 1230 seemed to move part of the load back to the surrounding ground. These latter results should however be taken with care, because of the potential for temperature and moisture effects on the observed liner response.

Figure 9 shows the distribution of liner moments obtained for four positions of the tunnel front : PM 1209, PM 1212, PM 1230 and PM 1500. It is apparent from these data that almost no moment was carried by the liner until PM 1212 was reached, i.e. 16 hours after the installation of the liner segments at PM 1200, and 12 hours after exposure of the instrumented ring to earth loads. Liner moments tended to increase with time as the shield progressed between PM 1212 and PM 1230, and to be stabilized after this latter stage, i.e. two days after installation of the instrumented ring.

The stabilized distributions of liner moments are non-symmetrical, with the moment on the northern side of the liner being close to zero, whereas a more classical distribution was obtained on the southern side. Maximum moments were found slightly above the southern springline of the

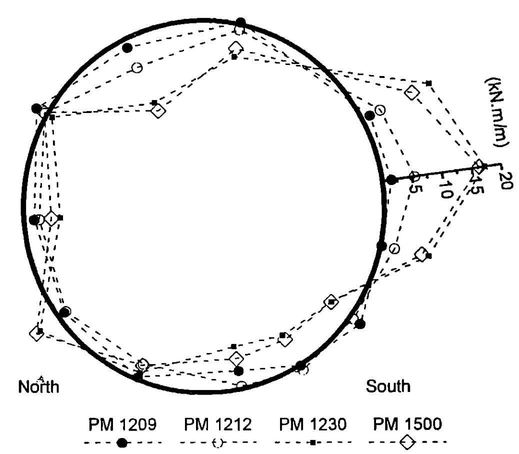

Figure 9. Distributions of Liner Moments

liner, with values in the order of 17 kN.m/m. Unlike what was observed for liner forces, these moments did not vanish when compressed air was applied inside the tunnel. It is commonly argued that joints between concrete segments of a singular liner ring could act as hinges, thus reducing moments induced by earth loads to the structure : this theory could explain why almost no moment was evidenced on the northern half of the liner ; however, the moment values observed on the southern half of the liner support the idea that hinge formation did not occur completely within the liner ring.

5 CONCLUSIONS

The instrumentation program set at PM 1200 of the VL4 project allowed analysis of the soil-structure interaction related to the construction of a moderately large sewer tunnel in soft soil using the slurry shield technology. The analysis of the recorded data was made difficult by the relatively low level of observed deformations in the ground and liner, as well as experimental problems encountered during the instrumentation program. For these reasons the conclusions driven from this study should be taken with care. The following points can, however, be highlighted :

(1) the slurry shield used to excavate the VL4 sewer tunnel was successful at controlling ground displacements in the highly permeable water-bearing deposits found at PM 1200 ;

(2) based on observed ground deformations at PM 1200 and 1205, the minimal pressure to prevent ground losses to occur at the face could be estimated to be 5 to 15 kPa higher than the natural ground water pressure at invert level ;

(3) the slurry pressures applied at the face resulted in some ground compression above and on the side of the excavated tunnel ; this apparently resulted in a relatively flat settlement curve at the ground surface, as the shield passed underneath the instrumented section, before a more classical reversed error function profile could be achieved ;

(4) the close vicinity between the tunnel springlines and the inclinometers installed to monitor lateral ground movements probably affected the recorded horizontal displacements ; in particular, a peak in lateral displacements was observed underneath the tunnel invert, which might have resulted in part to slurry flows around the tunnel ;

(5) liner forces derived from strain measurements performed in the precast concrete liner reached the maximum value due to full earth pressure, regardless of the potential for stress redistribution ahead of the tunnel face ; the measured forces were, however, strongly reduced during the subsequent construction stages ; it is to be noticed that this phenomenon was observed after compressed air had been used inside the tunnel ; the data could also have been affected by temperature and moisture changes within the concrete liner segments ;

(6) non-symmetrical moment distributions were observed around the liner ring, with almost no moment being recorded on the northern half of the liner, whereas moments in the order of 17 kN.m/m were obtained on the southern half ; this supports the idea that hinge formation between adjacent segments of the liner ring only partly occurred on the instrumented section.

ACKNOWLEDGMENTS

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