Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
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In situ monitoring of the Lyons Metro D line extension

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ABSTRACT : The Lyons Metro D line extension to Gare de Vaise is situated in a built up area. 6.27 m diameter twin tubes were bored with a slurry shield due to the risk of settlement of these shallow tunnels, partly situated in fine silty sands and clays. This paper presents monitoring results from two experimental tunnel sections, in order to highlight the soil response to shield tunnel operations and to obtain a reliable data base to improve and validate computer simulation tools.

1 INTRODUCTION

The development and extension of large cities creates a need for multiple shallow tunnel projects in the soft ground of built up areas. The presence of existing sensitive structures close to these tunnels, both on the surface and below ground (buildings, water and sewerage pipes...) requires that the movements induced by excavation works do not affect the stability and proper operation of these structures.

For such shallow tunnels, often built with pressurized shields, the empirical rules to evaluate settlements prove to be unsuitable (PANTET 1991). It is therefore necessary to develop and validate more accurate simulation tools to allow assessment of the amplitude and distribution of soil movements in relation to the selected technical choices.

In order to obtain precise and reliable experimental data, monitoring was carried out on two sites.

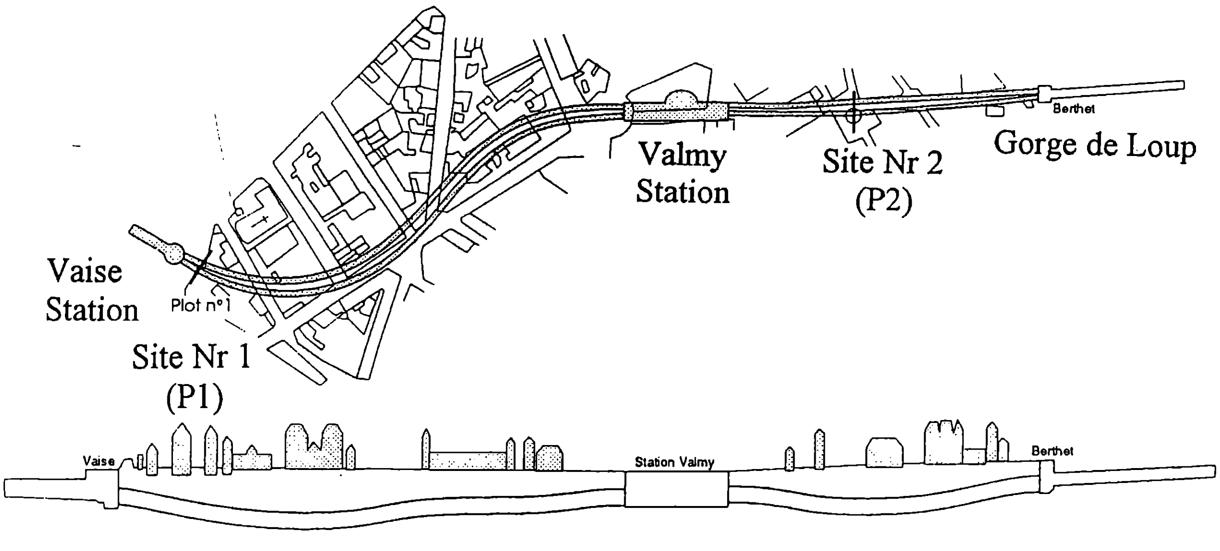

Figure 1. Tunnel alignment

After a description of project conditions, experimental equipment and soil reaction, the report concludes with analyses of the horizontal and vertical displacements, and the pore water pressures due to the progression of the tunneling works.

2 PROJECT DESCRIPTION

The tunnels are almost parallel to the River Saône, in a built up area and consist of two 900 m. long, 5.3 m internal diameter tubes passing through the 110 m long Valmy station (figure 1). The minimum cover over TBM entrances and exits is one diameter since more than half the route is under residential areas.

2.1 *Site Geology and HydroGeology*

Four major types of ground were encountered from the surface (table 1)

Table 1. Soil properties

stratum	γh KN. m^{-3}	w %	wl %	Ip %	Ic	C' kPa	φ' °	qc MPa	E MPa	Pl MPa
1 - Fill	18								4	0.5
2 - Beige Silt	16 à 21	35	58	26	0.9	0 à 30	29 à 35	<2	<4	<0.8
3 - Ochre Silt	16 à 21	27	29	8	0.25 à 0.5	0 à 30	29 à 35	0.5 à 4.5	1 à 20	0.5 à 1.4
4 - Grey Clay	21	67	78	21	0.5	30 à 40	20 à 30	1	<4	<0.8
5 - Grey sand	21	33	58	20	1.25	5	35	10	5 à 15	2
6 - Violet clay	18.5	30 à 40	47	20	0.1	15	20	3	5	0.7 à 1.4
7 - Sand and gravels	21					0	37	10 à 30	5 à 15	2

1. Fill 3 to 5 metres average thickness,
2. Silty alluvial deposits in thin, more or less sandy or clayey soils. On occasions these silts have a natural water content close to the liquid limit, making these layers very sensitive to disturbance. The mechanical characteristics shown up by pressuremeter tests, CPT and laboratory tests were poor, and indicated that there could be significant deformations.
3. Sandy-gravelly alluvial deposits, some elements having diameter of more than 50 mm with a few of over 500 mm that require the use of a crusher.
4. The gneissic substratum is encountered at depths of more than 80 metres at Vaise Station to depths of less than 20 metres at Gorge de Loup. A micro-gravimetry study enabled more precise location of the position of this substratum at Place Valmy where it is the highest.

Hydro-geological studies revealed two water-bearing aquifers :

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).

2.2 *The Slurry shield*

The contract was awarded to a consortium made up of SOGEA, Campenon-Bernard and GTM. The technique of a slurry shield was chosen in preference to « EPBS » which was likely to be more difficult to control in these soil conditions. The TBM was built by the Herrenknecht Company and consists of the following elements :

1. A 6. 27 m diameter shield fitted with a bulkhead and a compressed air bubble allowing precise adjustments to be made to the slurry pressure in the chamber. The cutting head is a six-pointed star wheel fitted with excavation tools.
2. A 42 m-long train consisting of 4 trucks with mucking pipes, segment transfer conveyor, the electro-hydraulic generator and a pilot cabin.

The lining of 6 m external diameter and 0.35 m thickness consists of 1 m long rings wich are made up of six precast concrete segments. The annular void is continuously backfilled by pumping inert grout from six injection pipes situated at the tail periphery.

Overall operation is automatically controlled and data is continuously recorded by a data logger.

3 EXPERIMENTAL SITES

In order to study precisely the soil-TBM interaction, two experimental sites have been equipped with a measurement system. These experimental sites have two main objectives :

1. To enable the operator to define the control parameters of the TBM.
2. To set up an experimental data base in order to develop and validate computing tools with the aim of modelling the soil-TBM interaction.

The first site (P1) including 2 monitoring sections (S1 et S2) is located near the starting shaft in order to aid operation and adjustment of the TBM. A second experimental site was decided upon after the Valmy station, with the aim of recording data on an area where the working process is well controlled. On site P1, the two measuring sections are situated entirely in silty layers. On site P2, the base of the section penetrates to a depth of 2 metres the sandy gravel layer. Table 2 presents the main characteristics of the three monitored sections. Notice that on section P1-S1, it was also possible to monitor the boring of the second tube.

Table 2 : Description of the experimental sections

Section	P1-S1	P1-S2	P2
Extensometer	EX11	EX21	EX31
cover over the tunnel	13,3 m	10,3 m	10,5 m
shield passing time	78h	22 h	24 h
Grouting pressures (Mpa) (upper part/ lower part)	0.3/ 0.2	0.35/ 0.25	0.25/ 0.25
Volume of grout injected in the annular void (m^3/m)	3.7	3.75	4.05

4 EXPERIMENTAL EQUIPMENT

Figure 2 shows, as an example, the measurement system implemented on section P1-S1.

This system is composed of :

1. vertical borehole extensometers with 4 points of measurement to monitor the vertical displacements.
2. Communicating vessels installed on top of extensometers in order to have the same reference level through an extensometer with a deep reference point.
3. measurement of surface settlement by surveying
4. Inclinometer casings situated on both sides of the tunnel, permitting the determination of horizontal displacements

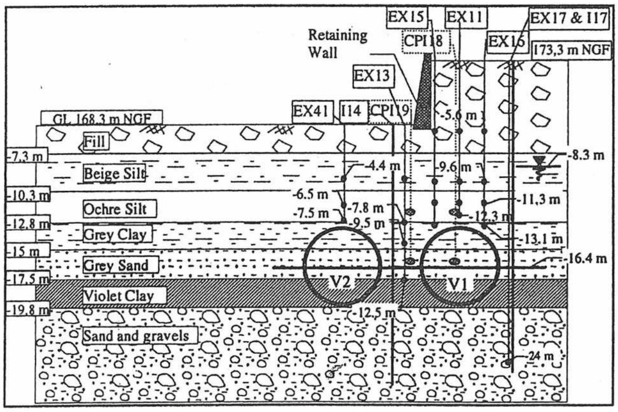

Figure 2. P1-S1 experimental section

5. Pore pressure cells to monitor the pore pressure reaction of silty layers with respect to stresses induced by the boring process.

With the exception of inclinometer measurements and surveying which require manual operation, all other measurements have been made automaticallly by means of a data logger. This data logger records information from 40 transducers at 5 second intervals during the passage of the TBM in the experimental sections. It can be noted that the nearest measuring points are situated within 1 meter of the tunnel section.

4.1 *Accuracy of measurement*

The experimental equipment was put in place 3 months prior to the start of shield tunnelling operations to assess measurement precision. The absolute settlement values are the sum of the extensometer and automatic levelling values and have a resolution in the order of 0.1 mm and an error not greater than 0.5 mm.

Calibration measurements performed before shield passage indicated a 1 mm error in horizontal displacement for a 20 m high inclinometer tube. As regards pore water pressure, the metrologic features of the sensors and the data logger yielded an error value of approximately 2 kPa.

5 EXPERIMENTAL RESULTS

The following results are typical of the soils reaction to the construction of the tunnel.

5.1 *Vertical movements on the tunnel axis.*

Figure 3 shows the development of the vertical movements with time of extensometers located on the tunnel axis, at the surface, and near the tunnel section.

Over the 3 monitored sections, 4 phases can be characterized. Table 3 summarises the surface and deep settlement values observed during these 4 phases.

Phase 1 : during the shield approach, the soil movements remain very low. The surface settlement is equal to, or greater than the deep settlements which would indicate good face stabilisation.

Phase 2 : on sections P1-S2 and P2 the shield front passage causes immediate settlements which became more important with depth. The settlements increase during the progression of the shield due to a decreasing diameter of 3 cm over it's length.

Phase 3 : After tail passage, settlement and heave peaks can be related to grouting of the annular void. The maximum heave at the deepest point depends on the value and distribution of grouting pressures around the tunnel. It is negligible on section P1-S1. On section P2, the filling of the annular void leads to à 3 mm heave at the deepest point over its initial position, without noticeable influence on immediate surface movements.

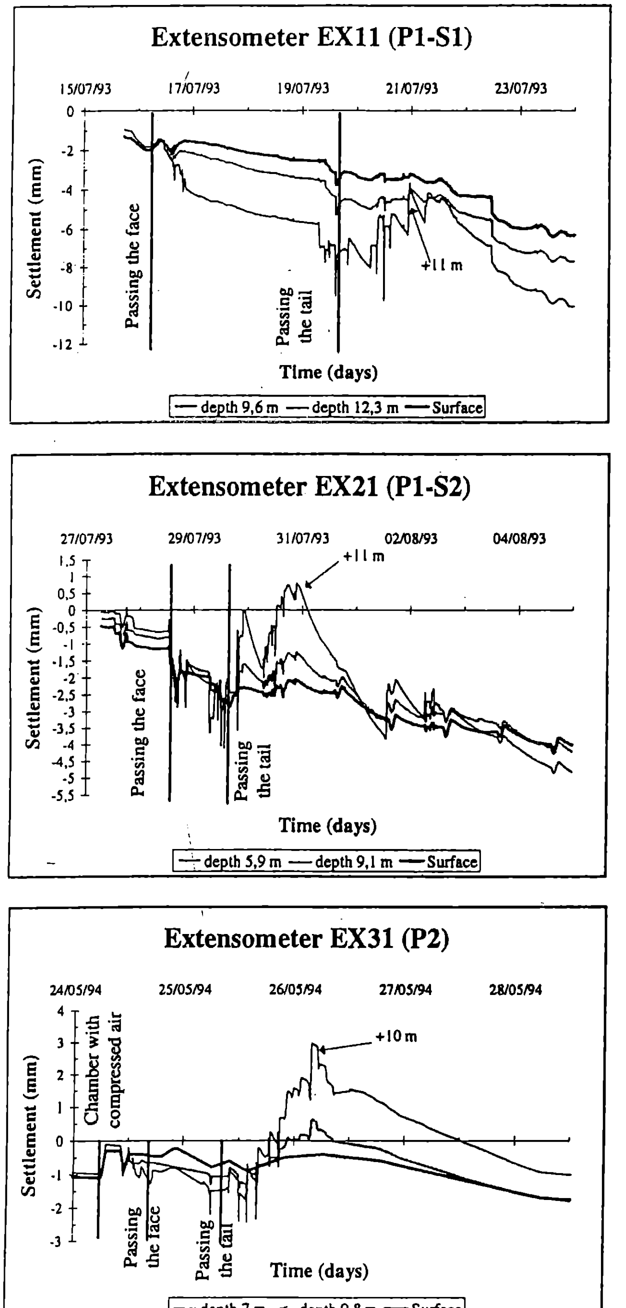

Figure 3. Evolution of vertical movements

Phase 4 : 20 metres after the shield passage, grouting effects disappear and consolidation settlement of the soil and the grouted void take place. Settlement distribution along the vertical axis of the tunnel depends on the grouting process : whereas surface settlement is lower than deep settlement on section P1-S1, the heave observed near the tunnel on section P2 leads to a final surface settlement higher than deep settlement.

Tableau 3 : surface settlement/ deep settlement (mm)

Section	P1 S1	P1 S2	P2
Phase 1	2/2	1.2/0.5	0.3/0.3
Phase 2	3.5./7.5.	3/3	0.8/1.5
Phase 3	4.5/5	2.5/-0,75	0.5/-3
Phase 4	10/15	6.5/8	2.5/2

5.2 *Surface settlement profiles*

Despite different soil-TBM interaction between the 3 experimental sections, the final settlement profiles (figure 4) are reasonably well described by the Gaussian distribution curve. Figure 5 presents the settlement profiles on section P1-S1 where the boring of the 2 tubes were monitored. By approximating these settlement profiles by Gaussian distribution curve, one can obtain the parameter « i », a characteristic of the width of the curve. The values obtained are similar for the basic profiles corresponding to each tube, and are within the limits proposed by Attewell (1977). The final profile resulting from the excavation of the 2 tubes is slightly larger than that of the individual tube, and a maximum settlement of 20 mm is satisfactory with regards to the difficult soil conditions.

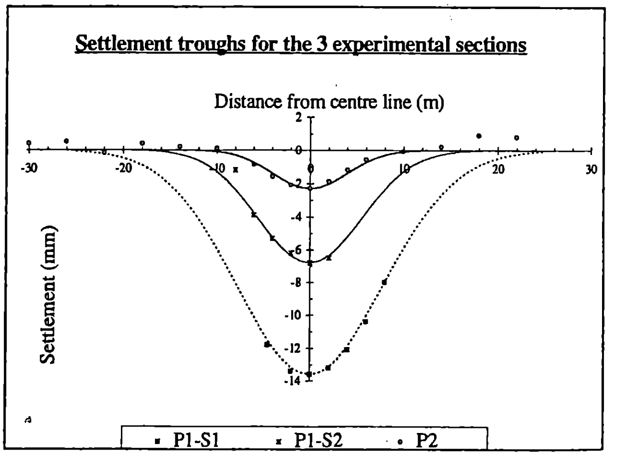

Figure 4. Surface settlement profiles

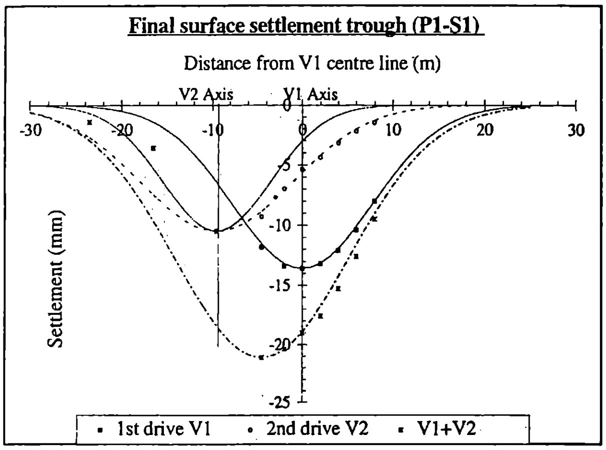

Figure 5. Surface settlement profiles (section P1-S1)

5.3 *Horizontal displacements*

The horizontal movements recorded by inclinometer measurement can be analysed within the 4 phases defined earlier. Figure 6 shows the horizontal displacements on site P1-S1, in the direction perpendicular to the tunnel axis.

Phase 1 : At the shield approach, a low magnitude inward movement of soil is to be observed.

Phase 2 : At the passage of the cutter wheel, the soil at the tunnel axis level moves outward while settlement occurs on the vertical axis. These soil movements occur as the TBM advances without over cutting. Finite difference calculations (figure 7) show these simultaneous displacements caused by the reduction of the shear stress along the interface between the soil and the shield, allowing sliding of soil around the tunnel section

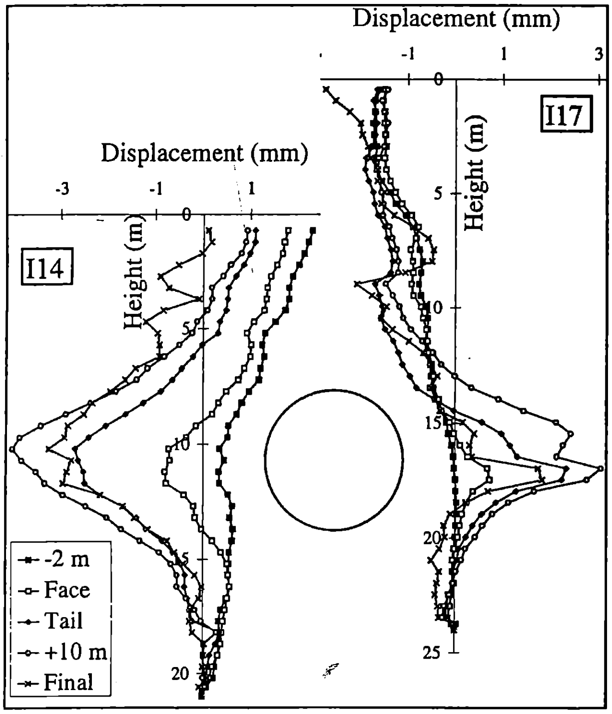

Figure 6. Section P1-S1 : horizontal displacements (1st drive V1)

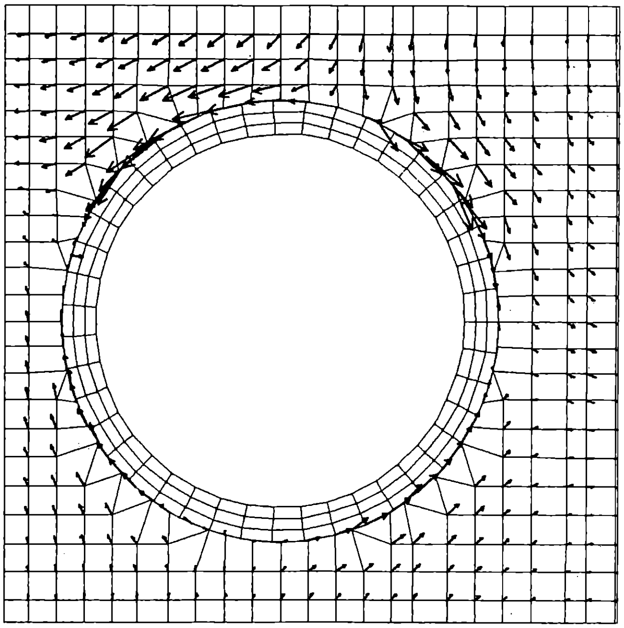

Figure 7. Calculated displacements at face passage

Phase 3 : Grouting produces a lateral outward movement larger than the vertical one which may be explained both by the difference between the grouting pressure in the lower and upper part of tunnel section and by Ko.

Phase 4 : Finally, the consolidation of the grout and the soil around the tunnel leads to a slight reduction of the outward lateral displacement. The final horizontal movements remain small compared to the vertical ones.

5.4 *Soil TBM interaction*

Figure 8 shows the soil response (vertical displacement, pore pressure) during one boring sequence in relation to the TBM advance and the grouting. It appears that grouting begins as soon as the TBM advances, but the discharge of grout seems to be not high enought to fill completely the annular void, as indicated by a fall in pore pressure and simultaneous settlement.

At the end of the boring phase, the grouting continues if the pressure target is not achieved. This fills the void and recompresses the soil around the tunnel as indicated by the parallel response in displacement and pore pressure. The quick

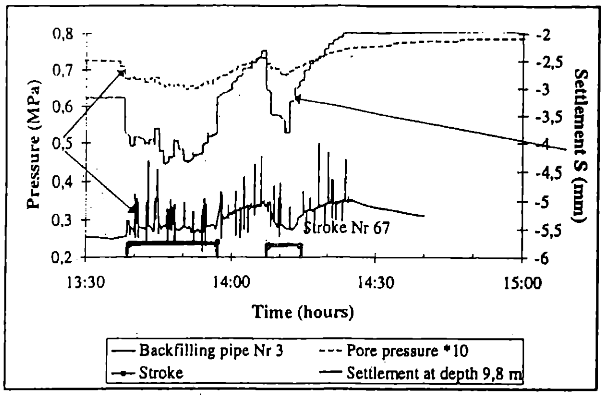

Figure 8. Soil response to shield advance and grouting

progression of such phases leads to increases in pore pressure which dissipate to their original value in under 2 days. This would indicate that drained conditions are suitable for simulation of the soil behaviour.

6 CONCLUSIONS

The measuring equipment placed on the experimental sites and on the TBM, allowed continuous monitoring with a good precision of soil response in relation to tunnelling progress. These observations, particulary in the vicinity of the tunnel section permitted clear identification of the origin of the soil movements. It appears that :

The negligible settlements in front of the shield confirmed effective face stabilization.

The shield passage induces settlements related to the reduction of shear stress at the soil-shield interface and to the decrease of the TBM diameter

The grouting system permits an effective filling of the annular void, with a local recompression of the soil. However, these injections of grout have little effect on irreversible surface settlements.

The tunnelling process chosen appears to effectively control the soil movements since the surface settlements resulting from the twin tube boring do not exceed 20 mm for the P1-S1 section within unfavorable soil conditions.

This experimental data provide a precise description of the soil mass at each phase of works allowing the validation of a modelling procedure (BERNAT & al, 1996) based on the deconfinement coefficient hypothesis (PANET 1988) This data is also the base of modelling works done by the « GEO » network of laboratories (BERNAT & al,1995) to improve the prediction of movements induced by shield tunnelling.

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ACKNOWLEDGEMENT

This study was supported by SYTRAL, and MESR (French Department of Higher Education and Research). It grouped SEMALY, INSA de LYON, ECOLE CENTRALE de LYON and CETU. We would like to acknowledge for their assistance the contractors SOGEA, CAMPENON-BERNARD SGE and GTM.