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Toulouse Metro (France): First feedback on deep excavations in molassic soils for the new line C

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ABSTRACT: A third metro line is currently under construction in the Toulouse metropolitan area (Line C). The previous, shallower lines had already been the subject of feedback, which is briefly recalled. The authors present the current Toulouse project, particularly the southeastern end of the new Line C, along with a reminder of the geotechnical and hydrogeological context. Special attention is given to the geotechnical characteristics of the Toulouse molasse and the support wall design procedures adopted from the design phase. Two recently excavated stations are highlighted: Ormeau and Montaudran Gare. The support wall calculations and the estimation of displacements behind the diaphragm wall are compared with monitoring results, including topographic, inclinometric, and strain gauge data at certain supports. Feedback is also provided on specific construction methods, such as the implementation of pre-installed barrettes, the use of thermal insulation on steel struts to limit thermal expansion effects, and the construction of a draining raft. The important role of instrumentation during the construction and monitoring phases of the work is thus highlighted.

KEYWORDS: Diaphragm wall, Toulouse Molasse, Instrumentation.

1 INTRODUCTION

Tisséo Ingénierie has undertaken the construction of a third metro line in the Toulouse metropolitan area, creating 21 new stations to cover 27 km. The engineering firms Systra and Arcadis are part of the project management consortium and were specifically responsible for the civil engineering design and construction supervision.

The Bouygues TP / Soletanche Bachy consortium of contractors is currently working on one of the line's sections, namely Lot No. 4, located in the southeast of Toulouse, extending from the Montaudran area to the Saint Sauveur Shaft near the Garonne River, crossing the Jolimont Hill. The works include the construction of a 520-meter cut-and-cover tunnel, four stations, three ancillary structures, and 4.2 km of tunnel.

This paper recalls the geological and geotechnical context of the project, the design assumptions adopted for the diaphragm wall retaining structures and will then focus on the construction of the first two completed excavations, the monitoring implemented, and initial feedback.

2 PROJECT PRESENTATION

2.1 *Geological and geotechnical context*

The project is located within molassic terrains from the Tertiary period (Oligo-Miocene). The Toulouse molasse consists of fine fluvial deposits of silts, fine sands, and clays. Some layers of coarser sand are also observed. The section under study is specifically situated beneath the molassic hill of Jolimont, along the edge of which slope formations may be encountered. At the southern end of the section, Montaudran Gare station is located at the Montaudran plain, where sandy deposits or anthropogenic formations conceal the molassic substratum over a depth of 1 to 3 meters.

Table 1 gives geotechnical parameters used in the design. These parameters and other assumptions are based on a reconnaissance campaign dedicated to the project, as well as feedback from previous work (Houhou, 2010; Givet, 2019; Daktera, 2020). The soil resistant parameters are given in short term (CT) and medium term (MT) load duration and are used for passive and active pressures until the raft is constructed. Long term (LT) values are used during the final construction phases and in service conditions. The pressuremeter values (Em, pl*, α) used for anthropogenic formations (AN) and weathered molasse are respectively (5 MPa, 0.5 MPa, 1/2) and (50 MPa, 3 MPa, 1/2). For intact molasse, a limit pressure of 6 MPa is used with α = 2/3. The Menard modulus is a function of depth z, based on all tests conducted along the project alignment, and is given by the formula:

$$Em = 60 + \frac{(6 * z)}{(1 + 0.025 * z)} \qquad (1)$$

The at-rest earth pressure coefficient is 0.5 in AN and varies with depth in the molasse: it is 1.5 for the first 20 meters, 1.3 between 20 and 30 meters, and then decreases by 0.1 per additional 10 meters until a value of 1.0.

Table 1. Geotechnical parameters used in the design

Geological formation		γ	φ	c' (kPa)	c_u (kPa)
Anthropic soils AN	CT/LT	20	25	0	-
Altered finer/coarser Molasse Mag	CT	21,5	20	90	-
	LT	21,5	32	15	-
Altered finer Molasse Ma	CT	21,5	-	-	150
	MT	21,5	30	45	-
	LT	21,5	30	30	-
Coarser Molasse Mg	CT/LT	21	35	0	-
Finer Molasse Mf	CT	21,5	-	-	250/ 300
	MT	21,5	30	70	
	LT	21,5	30	45	

Two aquifers are encountered in the project: the molassic aquifer, which is low-yield and connected to the Garonne alluvial aquifer, and a surface aquifer on the Jolimont hill, mainly fed by meteoric water. When these two aquifers are not in equilibrium, the water pressure profiles shown in Figure 1 are used, based on all the interstitial pressure measurements obtained.

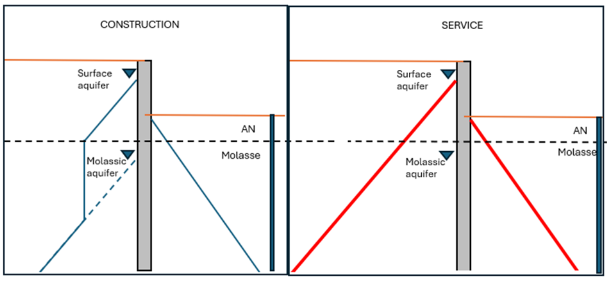


Figure 1. Hydraulic pressure profil used in the design

2.2 *Assumptions for diaphragm walls dimensioning*

The retaining wall calculations were carried out using subgrade reaction models (MISS-K) and the geotechnical characteristics previously presented. The subgrade reaction coefficient is determined using P. Schmitt's formula (Schmitt, 1998).

The design of the retaining structure adopts a safety factor of 1.5 for the passive earth pressure during construction and 1.9 for the service phase. Safety is calculated using the MISS-B procedure (Utter 2017; CNJOG 2018), in which the passive earth pressure coefficient is divided by the required safety factor. This method accounts for load redistribution within the wall, particularly in the final strut level (last excavation phase) or in the lower-level slab (service phase). The MISS-A procedure, which computes the ratio of limit passive earth pressure to mobilized passive forces and compares with the required safety factor, is also applied. The minimum safety factor associated with this procedure and used in the design is 1.25.

During the execution studies, the influence of the position of sandy pockets at the excavation bottom was studied. A layer with a thickness equal to the thickest one observed in boreholes is systematically assumed at the bottom excavation level, unless a nearby borehole clearly indicates otherwise. This conservative approach is due to the erratic nature of these lenses and the sensitivity of the results to their presence or absence.

2.3 *Montaudran Gare and Ormeau stations*

Montaudran Gare station is a rectangular box measuring 20 m by 51 m, excavated to a depth of 23 m, and located near a railway line. This structure (Figure 2), positioned at the northern end of the cut-and-cover tunnel, serves during construction as the launch shaft for the tunnel boring machine (TBM). Due to planning constraints, the excavation was carried out in open air with the installation of three levels of metal struts during the descent, prior to constructing the raft and removing the third level of struts to allow TBM assembly.

Ormeau station is a box structure with a highly constrained footprint, excavated to a depth of 26 m (Figure 3). Its general rectangular shape (29 m by 52 m) includes re-entrant corners at all four corners. The main box is bordered by three shallower structures used for station access or ventilation. The construction sequence includes the installation of two descending slab levels (DC and SS1), followed by reaching the excavation bottom with the temporary installation of two levels of metal struts. The raft and SS2 slab are constructed after the TBM has passed through, allowing the removal of the temporary struts. For this structure, the span of the slabs and the fact that two of them are constructed during the excavation required the implementation of pre-installed barrettes at the locations of the final columns.

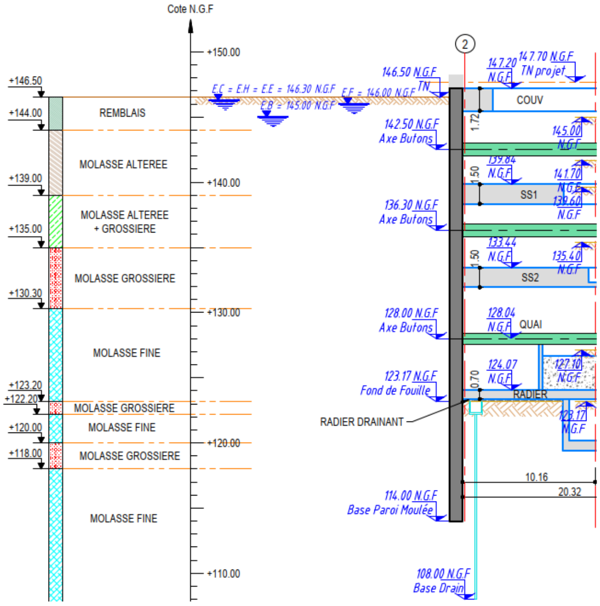


Figure 2. Montaudran Gare Station – typical cross section view.

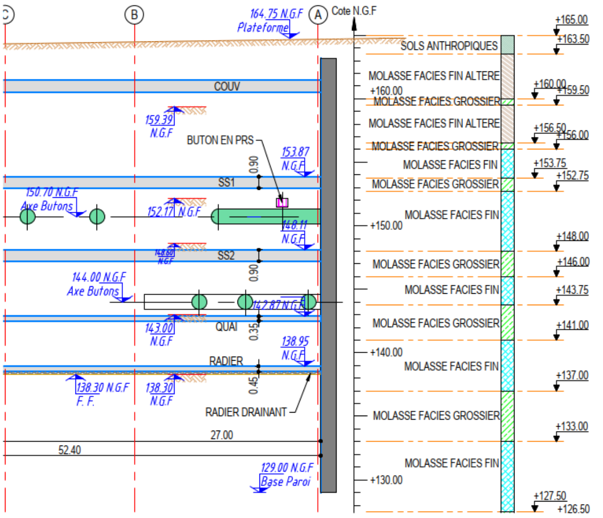


Figure 3. Ormeau Station - typical cross section view.

3 MONITORING DURING EXCAVATION

A monitoring program for the excavation phases was included in the tender, in order to assess the impact of the works on nearby structures. Inclinometers installed in the diaphragm walls allow for tracking their displacements (Schmitt, 2009), and automatic strain gauges welded onto the struts provide data to estimate the forces within these structural elements. Simultaneous temperature measurements help eliminate the influence of thermal expansion, allowing for a more accurate analysis of support reactions (Daktera, 2020).

3.1 *Montaudran Gare Station*

At Montaudran Gare Station, the relative proximity (about 20 m) to railway tracks required the implementation of a denser monitoring system. Figure 4 shows the maximum displacements recorded by each inclinometer: the values are close to those predicted by the calculations for the TBM operation phase. In Figure 5, an example monitoring report of the opposing walls and their temporary struts revealed the following:

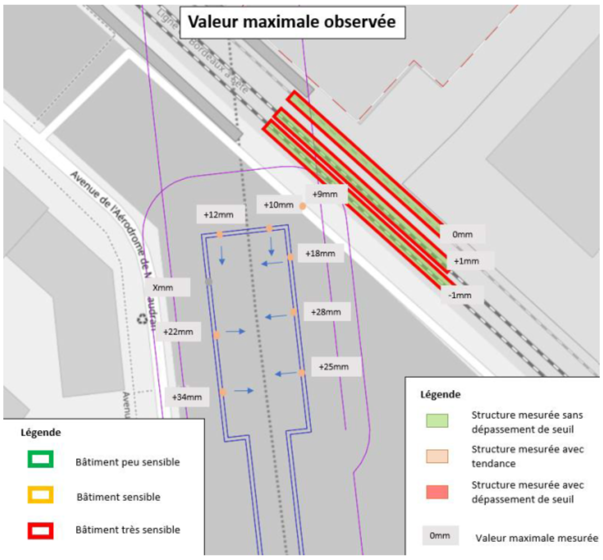


Figure 4. Montaudran Gare Station – Monitoring of Diaphragm wall movements and track settlement

- The deformation shape is similar to the calculated one;
- Forces in struts B102 and B103 exceed the calculated values and evolve during excavations;
- The force in the last strut level is slightly below the calculated values.

A critical analysis of the strain gauge data was conducted, including an exercise to isolate the effect of temperature (Daktera 2020), and a comparison between forces derived from the gauges and those inferred from the relative displacement of opposing walls. The gauge measurements were deemed credible.

This increase in strut forces was not observed throughout the entire station, making it difficult to draw definitive conclusions about the origin of these discrepancies. This led the consortium to conduct some back-analysis and compare different modeling approaches: the MISS-EF (finite element with Plaxis2D, HSM behavior law) and MISS-K (PARIS, an internal software developed by Soletanche-Bachy) calculations showed some differences in maximum support reactions (R) as follows:

- Strut level 1: $R_{measured} > R_{max}$ (MISS-EF) $> R_{max}$ (Miss-K)
- Strut level 2: $R_{measured} > R_{max}$ (Miss-K) $> R_{max}$ (MISS-EF)
- Strut level 3: R_{max} (Miss-K) $> R_{measured} > R_{max}$ (MISS-EF)

The comparison of these calculation approaches has been widely discussed (Nejjar, 2019; Daktera, 2020; Nejjar & Bergère, 2022; Schmitt, 2023). In the case of Montaudran Gare Station, both approaches yield similar total earth pressure (Figure 6), but with different distributions. The flow modeled in the finite element analysis explains part of the discrepancy. Neither model fully captured the forces measured by the strain gauges on the struts: the magnitude of the measured support reactions remains a subject for further discussion.

The struts were calculated using forces derived from a subgrade reaction model (PARIS) and a very conservative assumption regarding articulation at the ends. Given the stringent displacement criteria, the first two support struts were oversized to ensure greater stiffness. However, the doubling of forces in some struts in bed 1 led the consortium to install a thermal insulation system to mitigate the effects of thermal expansion (§ 4.2).

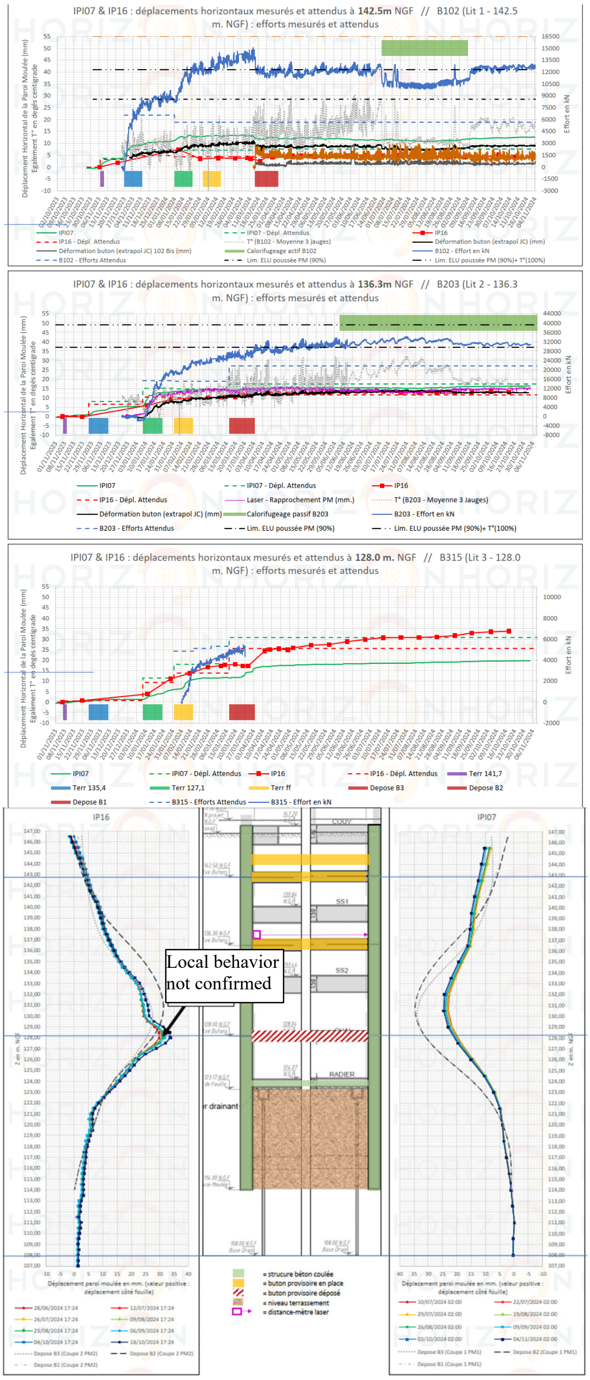
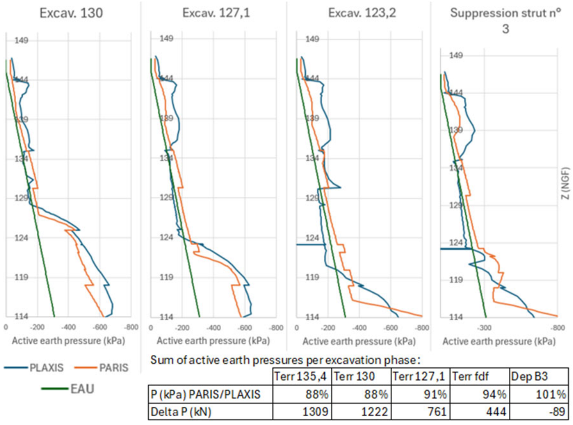


Figure 5. Montaudran Gare Station – Ex. of instrumentation report.



Sum of active earth pressures per excavation phase:

	Terr 135,4	Terr 130	Terr 127,1	Terr fdf	Dep B3
P (kPa) PARIS/PLAXIS	88%	88%	91%	94%	101%
Delta P (kN)	1309	1222	761	444	-89

Figure 6. Earth pressure calculation -Comparison MISS-K vs MISS-EF

3.2 *Ormeau Station*

Unlike Montaudran Gare station, the monitoring of Ormeau station did not reveal any excessive forces in the struts. The measured displacements and forces remained below the estimated values, and no back-analysis was required during the works. Figure 7 shows the displacements recorded by all inclinometers (ranging from 1 to 3 cm). Figure 8 illustrates the state of the retaining structure upon reaching the excavation bottom. Figure 9 and Figure 10 are taken from wall monitoring reports and allow comparison between measured values and estimated ones (dashed lines) along a cross-section at the longer span of the structure.

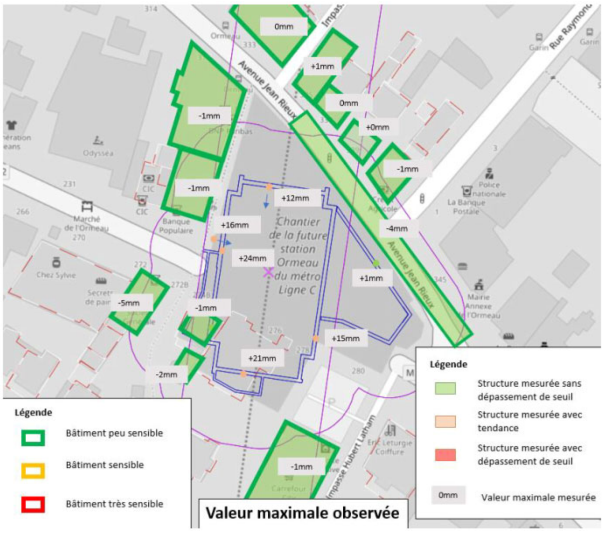


Figure 7. Ormeau Station – Monitoring of Diaphragm wall movements and settlement of neighbouring structures.

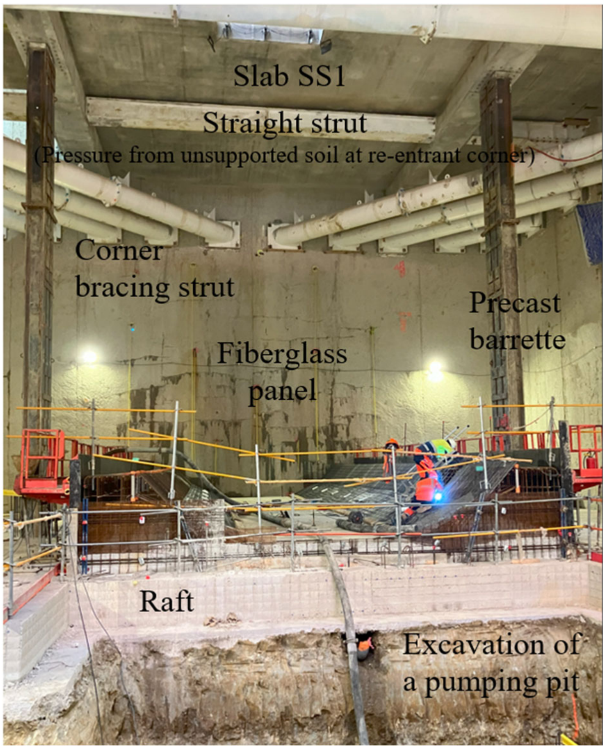


Figure 8. Ormeau Station – Photography of excavation bottom

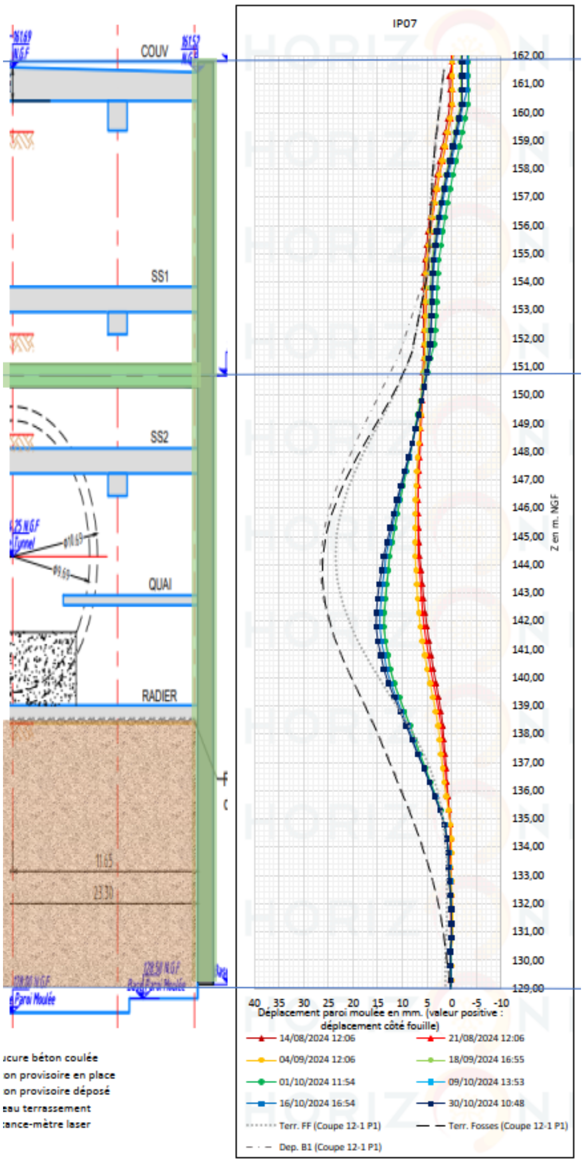


Figure 9. Ormeau Station – inclinometric report

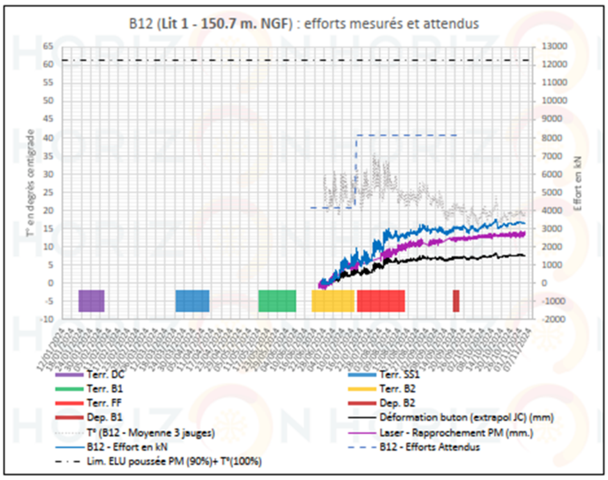


Figure 10. Ormeau Station – strain gauge report on strut B12

4 FEEDBACK ON CONSTRUCTIVE MEASURES

4.1 *Pré-installed columns (barrette)*

Two stations in the section include pre-installed columns, created by inserting a steel structure into a foundation barrette, with concreting stopped at the excavation bottom level. The consortium chose to position these temporary pre-installed elements directly at the location of the definitive columns. This concrete columns are then built around the steel profile (Figure 13), which is neglected during the service phase. As a result, the static schemes of the slabs are preserved during the construction of the final columns. The key challenge in these works lies in ensuring the verticality of the temporary steel pre-installed column. To achieve this, in addition to real-time monitoring of drilling deviations using inclinometer-equipped excavators, an alignment system is placed on the guide walls during barrette installation to adjust the position of the steel profile (Figure 11).

From an execution study perspective, the critical points are the buckling of the system and the dowel systems that transfer forces from the slab to the profile, and below the excavation bottom, from the steel profile to the foundation concrete. For buckling, a second order analysis of the system is used to compute a critical force and perform checks according to Eurocode 3. Since the steel profiles are separated to allow reinforcement bars to pass through at slab connections, buckling is more critical in their lower inertia direction. Reinforcement plates are regularly installed to link the profiles together. Within the foundation, the dowels are combined with barrette reinforcement to transmit the force within the foundation concrete (Figure 12).

Figure 11. Column head wedging system combined with topographical measurements

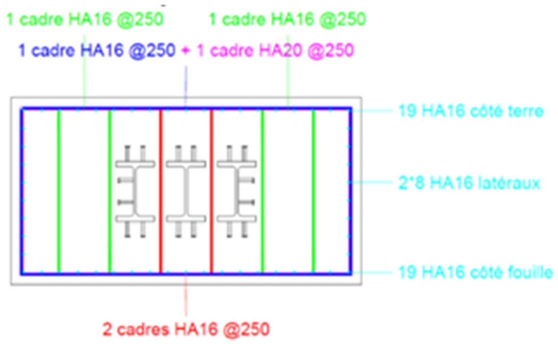


Figure 12. Plan view of the structure and foundation reinforcement

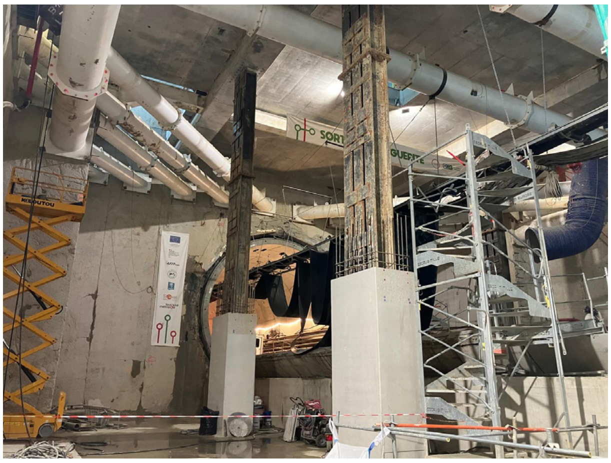


Figure 13. Ormeau Station – Photography of pré-installed columns (with the final column partially completed)

4.2 *Installation of metal strut insulation*

Excavation at the Montaudran Gare station began in late summer / early autumn, when temperatures were relatively mild during the installation and passive loading of the struts. Given the forces recorded during the initial excavation phases (3.1), and in order to secure the struts for the next two years (duration of tunnel operation), especially with the approach of the Toulouse summer, the consortium decided to install additional pre-stressed struts and a thermal insulation system, a solution previously documented (Nejjar, 2022) (Figure 14).

Two thermal systems were implemented: an active system with cold water circulation for the most critical struts, and a passive system preventing solar radiation on the other tubes. The effect of installing and activating these systems can be seen in Figure 5, where a drop in internal force in strut B102 is observed immediately after the start of cold-water circulation around the strut. The internal force was reduced by 15%, effectively eliminating the contribution of temperature during summer. It was also noted that once the high temperatures subsided, the insulation system was deactivated and the force returned to levels observed prior to its installation. For strut B203, which had only passive insulation (a reflective coating), it was not possible to eliminate the temperature effect, but only to smooth out daily variations.

Figure 14. Montaudran Gare Station – Additional pré-stressed struts and insulation system

4.3 *Draining raft*

Given the low permeability of the molassic substratum (on the order of 5×10^{-8} m/s), the project management team opted for the construction of a draining raft, consisting of a drainage mattress combined with a network of agricultural drains and permanent pumping. This system, installed beneath the excavation bottom, is connected to about ten vertical drains extending below the base of the diaphragm walls, in order to dissipate pore water pressures beneath the excavation.

The consortium proposed replacing the 60 cm thick drainage mattress with a draining mat (Somtube® –Figure 15) connected to a network of drainage trenches equipped with 100 mm agricultural drains (Figure 16). This adaptation, like the one used on the previous metro line, allowed for a 40 cm reduction in excavation depth.

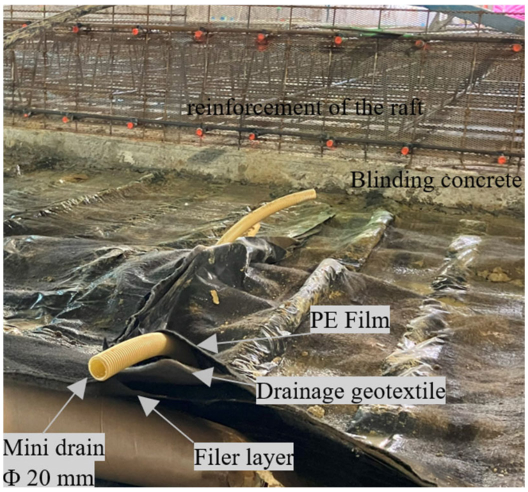


Figure 15. Filtration/drainage/protection geocomposite

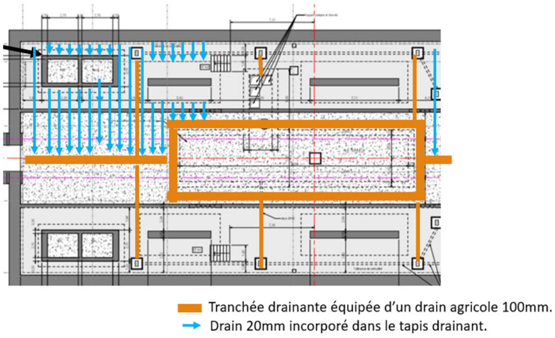


Figure 16. Schematic diagram of the drainage network

5 CONCLUSIONS

The Toulouse metropolitan area had already undergone major construction works for two previous metro lines. These projects shared significant similarities with the current one, although the excavation depths were shallower.

This experience highlights several key aspects to the successful execution of such a project: First, it should be noted that past experience—well-documented and analyzed—played an important role in the design of the current project.

Second, monitoring is essential for the proper supervision of retaining structures and provides valuable support for adapting construction methods. For retaining structures some useful tools are inclinometers, topographic monitoring, strain gauges on struts and linked to temperature tracking, as well as selective piezometers inside or outside the excavation.

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