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Analysis of diaphragm wall displacement caused by jet grouting

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ABSTRACT: As part of the construction of a tunnel using cut-and-cover method a jet grouting plug was installed below the maximum excavation depth between two embedded retaining walls (Diaphragm walls) for ground water control during construction and to prevent basal uplift failure of the braced excavation system.

The treated soils, consisting of clayey silts, had very low strength and were naturally in a very plastic to fluid state. The installation of jet grouting columns within these soils caused significant movements in the diaphragm walls already in place, as observed through inclinometers, even though no excavation work had been carried out.

The high-pressure injection of large volumes of grout, combined with the limited spoils evacuation observed through the annular drilling space, was identified as the cause of the lateral displacement of the diaphragm walls. Although measures were taken to improve the fluidity of the injection grout —and thus the spoils— to reduce the risk of blockage of the annular space preventing spoil evacuation, the walls continued to move. Monitoring results indicated that the jet grouting works ultimately caused horizontal displacements of the retaining walls up to 140 mm soil side (away from excavation).

These horizontal movements in turn induced large bending moments in the diaphragm walls, which had not been anticipated in the original design. Hence, the construction process had to be adapted to ensure the structural integrity of the Diaphragm walls by installing additional struts during excavation stages or reducing the depth of excavation passes.

This paper presents an analysis of the phenomena that caused these wall displacements due to horizontal jet grouting, based in particular on back-calculations performed using finite element models.

KEYWORDS: Jet grouting, diaphragm wall.

1 INTRODUCTION

In order to prevent a basal instability due to uplift in clayey silt without having to dewater the sand and pebbles present at depth, a jet grouting plug was installed below the maximum excavation's depth for the construction of a tunnel using both cut-and-cover and open-trench methods. Diaphragm wall solution was proposed to facilitate the excavation works and to serve as permanent retention system.

The jet grouting plug did not figure in the initial design and the solution, where injection at the toe of the diaphragm wall was designed.

Due to certain evolutions in the project, the jet grouting works were approved after the installation of the diaphragm wall.

2 PROJECT DESCRIPTION

2.1 *Structure description*

The construction of the tunnel involved a braced excavation in soft alluvial clayey silt supported by diaphragm wall. The excavation was 8 to 14 m deep, 9.14 m wide and about 700 m long.

The excavation was supported by a 0.8 m thick diaphragm wall.

The jet grouting plug was installed over a cumulated length of approximately 300 meters. Along the remaining section, the ground water was pumped in the sandy aquifer, to prevent an uplift failure of the clayey silt during excavation. The Sand and Pebbles layer between the diaphragm walls was pressure grouted to create a cement grout plug beforehand at the wall toe level to reduce the ground water flow, as primarily designed. This treatment proved insufficiently effective to be implemented throughout the entire tunnel while adequately reducing pumping rates.

On the design section examined in this article, the final excavation depth was around 10 m (-5.0 m NGF), and the diaphragm wall was supported at the top by a 0.8 m thick cover slab located at 1.85 meters below the natural ground level (+2.5 m NGF), constructed following the top-down method.

A level of temporary prestressed struts was installed at a depth of 6.8 meters (-2.0 m NGF) and removed after the 0.8 m thick base slab (- 4.6 m NGF) was placed.

To follow the displacements of the diaphragm walls during excavation, 1 inclinometer for every 50 m length of diaphragm walls was installed up to the base level of the steel cage, on both sides of the trench. Topographic monitoring targets were also installed at the wall crest at 5-meter intervals.

2.2 *Subsurface conditions*

A cross section showing the soil profile and some of the characteristics measured is presented in Figure 1.

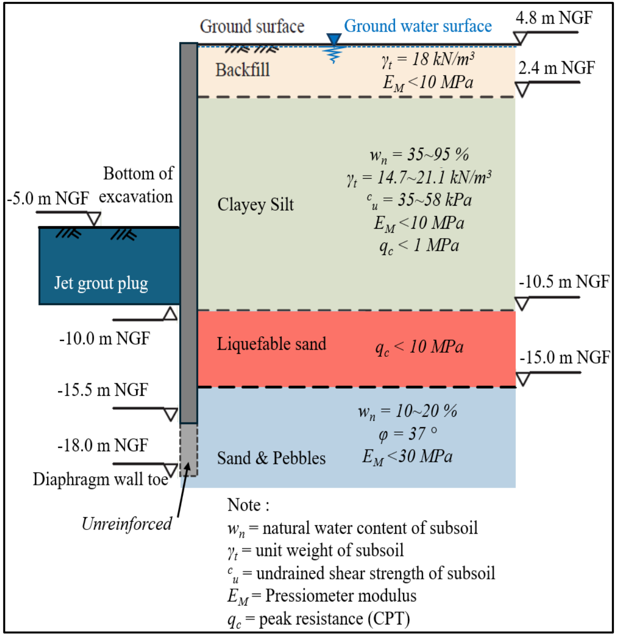

Figure 1. Geotechnical profile and soil parameters

The subsurface soils consisted of a silty sandy fill layer overlying a very soft clayey silt. Granular deposits (sand and pebbles) are present at depth up to the toe of the diaphragm wall.

The clayey silts, which made up most of the excavated soils, exhibited very low mechanical properties, with measured limit pressures inferior to 0.8 MPa and pressuremeter moduli of less than 10 MPa (up to a minimum value of 1 MPa)

Laboratory-tested samples also showed very high-water contents, in most cases significantly exceeding the Atterberg liquid limit, indicating an almost liquid behavior of the in-situ materials, as illustrated by the samples presented in Table 1.

Peat lenses with organic matter contents ranging from 8% to 29% were also locally identified in some samples in the area.

Table 1. Water contents and Atterberg limits measured on clayey silt samples of the area

Sample depth (m)	Water content w_n (%)	Plastic limit w_p (%)	Liquid limit w_l (%)
3.4	34.1	28	39
10.7	73.9	35	47
4.5	37.5	39	47
8.05	95	43	65
12.25	63	31	48

3 JET-GROUTING OPERATION

3.1 *Jet Grouting System*

Jet grouting is carried out prior to the start of excavation works to form a 5-meter-thick plug directly beneath the final excavation level over two areas of the trench: Zone 1, located west of the trench, which is about 100 m long and Zone 2, which is 200 m long on the east side of the project. These two zones are 110 m apart from each other.

The target diameter of the columns was 1.8 meters, and they were installed on a 1.0 m × 1.16 m grid to create a continuous mass of treated soil.

The double jet method was used, involving the simultaneous injection of two high-pressure fluids to create the jet grout plug.

Prior to commencement of jet grout works, a field trial was carried out to test various operation parameters and the effectiveness of the treatment on the different soil layers. It turned out that it was not possible to achieve satisfactory treatment in the sand and pebbles layers, as the presence of pebbles hindered soil erosion and grout diffusion, commonly termed as the “shadow effect”. It was therefore decided to carry out the treatment in the clayey silts to form a jet grout plug resistant to uplift pressures.

Since the treatment was carried out in impermeable soils with low mechanical properties, the jet grouting plug was not intended to prevent water ingress but rather to withstand the uplift pressures exerted beneath the clayey silt. These upward forces were counteracted through an arching mechanism within the plug and were transferred to the diaphragm wall via friction at the interface between the treated soil and the concrete structure.

Table 2. Jet-Grouting Operation Parameters.

Operation parameter	Value / Range
Borehole diameter	114 mm
Grout injection pressure	45 MPa
Grout flow rate	420 l/min
Compressed air pressure	0.8 to 1.2 MPa
Rod withdrawal rate	18 to 21 cm/min
Rod rotation rate	6 to 7 rpm
Water cement ratio	1:1
Cement content	750 kg/m^3 grout
Superplasticizer content	6 L/m^3 grout

The treatment was carried out according to the parameters presented in Table 2.

It is to be noted that the trial tests were carried out in Zone 1. After the trials, the jet grouting plug was constructed all over Zone 1. The inclinometers installed inside the concrete structure of the diaphragm wall in Zone 1 recorded limited horizontal displacements, ranging from 5 mm to 35 mm away from excavation depending on the monitored panel.

However, in Zone 2, and particularly in the cross section discussed in this article and described above, horizontal displacements of several centimeters towards the retained ground were recorded as soon as the first columns were grouted. This section is located 250 meters west of Zone 1. In both zones, the jet grout plug is installed in the same clayey silt layer.

The surprising difference in behavior between Zones 1 and 2 is likely due to the heterogeneity within the clayey silt layer, which locally exhibits a more fluid consistency and includes peat elements in some areas in Zone 2, as described in Section 2.2.

3.2 *Discharge of spoil*

During Jet Grouting works, discharge of spoil developed as observed during trial tests.

But some issues during discharge of the spoil were identified in two zones, even after using conventional admixtures that improve fluidity of spoils to facilitate its discharge.

The problem in evacuation of spoils and its impact on the deformations in diaphragm walls was first detected due to large movements measured in the neighboring inclinometers. In the absence of any other work being carried out that may cause diaphragm wall movements, it was inferred that the pressure build up caused by the blockage of spoils was the cause of the deflections seen in the diaphragm wall panels during jet grouting execution.

The construction methodology and execution of works were therefore carefully reviewed, in order to verify if the diaphragm wall displacements were caused by unsuitable construction methodology or the characteristics of the soil in place. Some tests were conducted, and changes were proposed such as, modifying admixtures (such as MasterRheobuild 100 and MasterSet R 100), enlarged diameter of casing, reduction of production rates or sequential order of columns execution.

Finally, some measures were implemented in order to reduce the problem of discharge of the spoil; pre-bored duct for evacuation with a 200 mm diameter casing and 13 m length, was modified to 250 mm casing diameter and 10.5 m length (adding an initial phase excavation of 2.5 m).

Unfortunately, the problem did not completely stop, increasing the horizontal movement in the neighboring inclinometers up to the end of the Jet Grouting Works.

3.3 *Effects of jet grouting on diaphragm walls*

The following figures show the horizontal displacements measured on two inclinometers installed in two panels of the same section, one on the north and one on the south, following the jet grouting works. The construction stage corresponding to the date of these inclinometer measures is presented in the Table 3. These inclinometers recorded the largest displacements observed in the project.

Since the inclinometer data are derived from cumulative angular measurements, the reference point at the inclinometer base is conventionally fixed at zero horizontal displacement. Therefore, it is not possible to measure any potential horizontal movements at the base of the inclinometer.

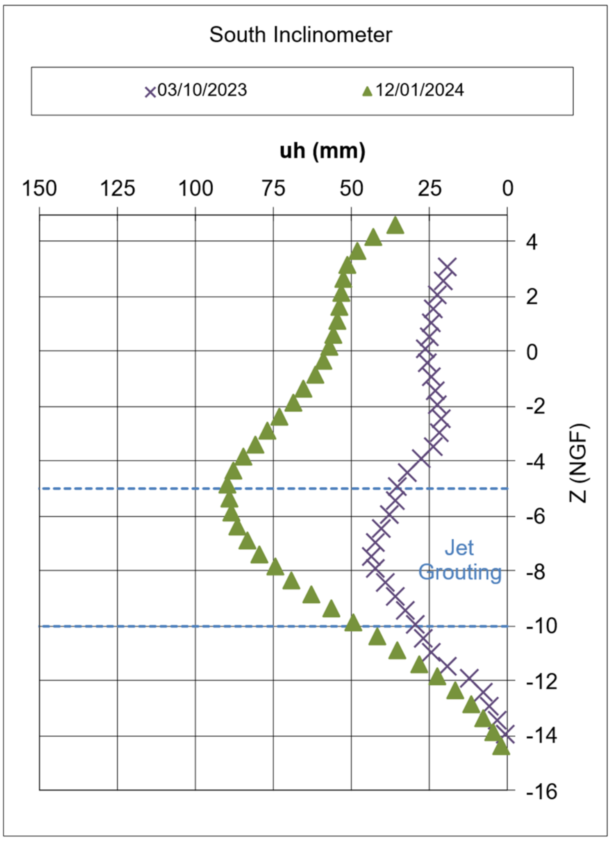

Figure 2. Horizontal movement of the south diaphragm wall following the grouting works (positive values correspond to displacements in the direction of the retained ground).

Most of the displacements occurred at the beginning of the jet grouting operations, before corrective measures for spoil discharge improvement were implemented.

The spoil discharge improvement measures did not fully rectify the problem of displacements in diaphragm walls but did reduce the maximal predicted displacement at the end of Jet Grouting works.

The two inclinometers show distinctly different horizontal deformation profiles. The north diaphragm wall exhibits a rotational movement with a maximum displacement of 140 mm at the top, whereas the south diaphragm wall shows a bulging effect near the plug, with a displacement of approximately 90 mm.

Table 3. Works stage in the studied area following date of inclinometer measure

Date	Works stage
03/10/2023	25 % of jet grout columns. No measures for spoil discharge improvement.
23/11/2023	60% of jet grout columns realized. Measures for spoil discharge improvement.
12/01/2024	100% of jet grout columns realized. Measures for spoil discharge improvement.
17/09/2024	Base slab poured, temporary struts removed

The explanation adopted for the difference in horizontal deformation profiles between the north and south walls is the presence of abandoned underground structures within the backfill on the south side. These structures acted as a support on the wall, limiting its head displacement. This condition was modeled by introducing fixed-end anchors on the wall within the southern backfill, which allowed the simulated profile to match the inclinometer readings.

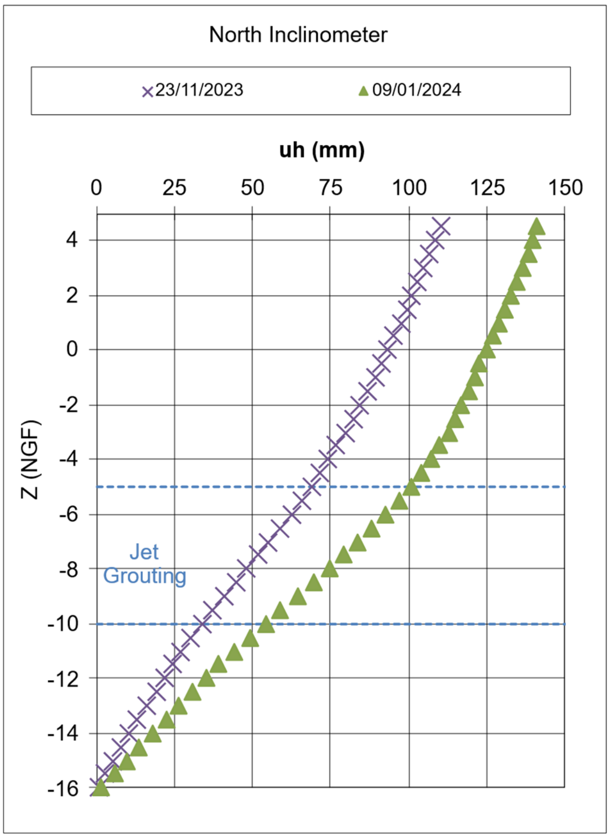

Figure 3. Horizontal movement of the north diaphragm wall following the grouting works (positive values correspond to displacements in the direction of the retained ground).

4 CONSTRUCTION SEQUENCE

Due to additional displacements during Jet Grouting works, the construction sequence of excavation already approved, was not valid anymore.

Considering the displacements of diaphragm walls during Jet Grouting operation, a new construction sequence was defined, to verify structural capacity of reinforced concrete sections of diaphragm walls during and after the excavation works.

4.1 *Initial state of diaphragm walls after Jet Grouting works*

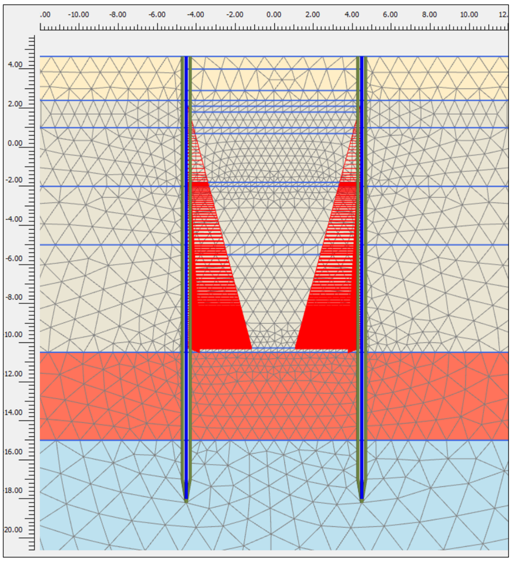

Figure 4. Additional hydrostatic pressure due to Jet Grouting

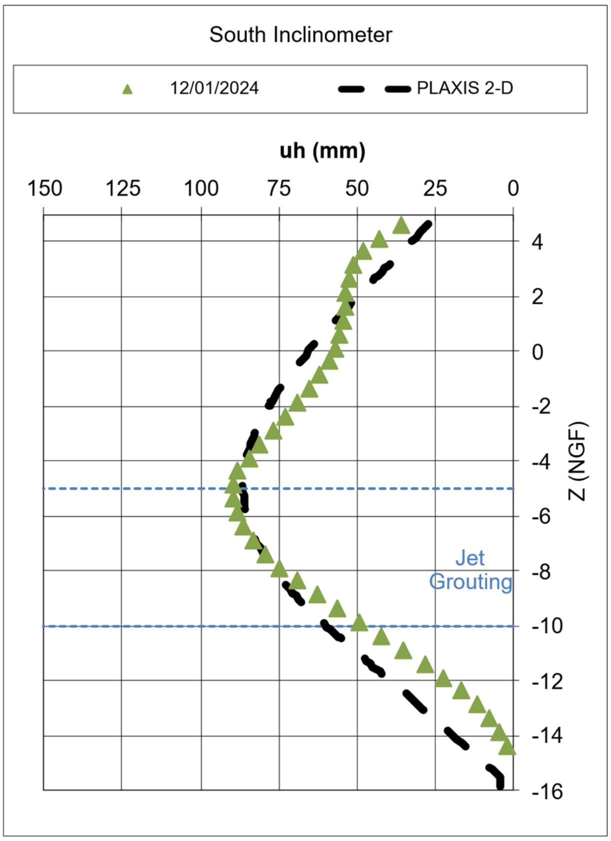

Figure 5. Initial state before excavation of the south diaphragm wall (positive values correspond to displacements in the direction of the retained ground).

Considering the liquid state of the clayey silt in the zone concerned by the problems of spoil discharge, an assumption of incremental hydrostatic pressure on the diaphragm walls caused by the Jet Grouting works has been implemented into a 2D finite element software, as shown in the Figure 4. Given the fluid-like behavior of the soil, the transmission of grout pressure is expected to follow a triangular distribution, comparable to a hydrostatic load, rather than a rectangular and uniform one.

Adding 75kPa on the North diaphragm wall and 105kPa on the South diaphragm wall, it was possible to obtain similar displacements as measured in the inclinometers, as illustrated in the charts of Figure 5 and Figure 6.

4.2 *Modification of construction sequence*

The pre-approved construction sequence was a conventional one (7 phases): excavation, provisional props close to the diaphragm wall head (+4.3 NGF), excavation, roof slab (+2.4 NGF), excavation, base slab (-5.0 NGF) and fill from roof slab up to surface (removal of provisional props).

Table 4. Excavation Construction sequence conducted

Phase	Description
1	Excavation up to +2.5 NGF
2	Prestressed struts (327kN) at +4.3 NGF
3	Excavation up to +2.1 NGF
4	Roof Slab at +2.5 NGF
5	Fill from roof slab up to surface (removing provisional propping system).
6	Excavation up to -1.8 NGF
7	Prestressed struts (700kN) at -1.3 NGF
8	Excavation up to -5.0 NGF
9	Base slab -4.6 NGF
10	Removal of Prestressed struts at -1.3 NGF

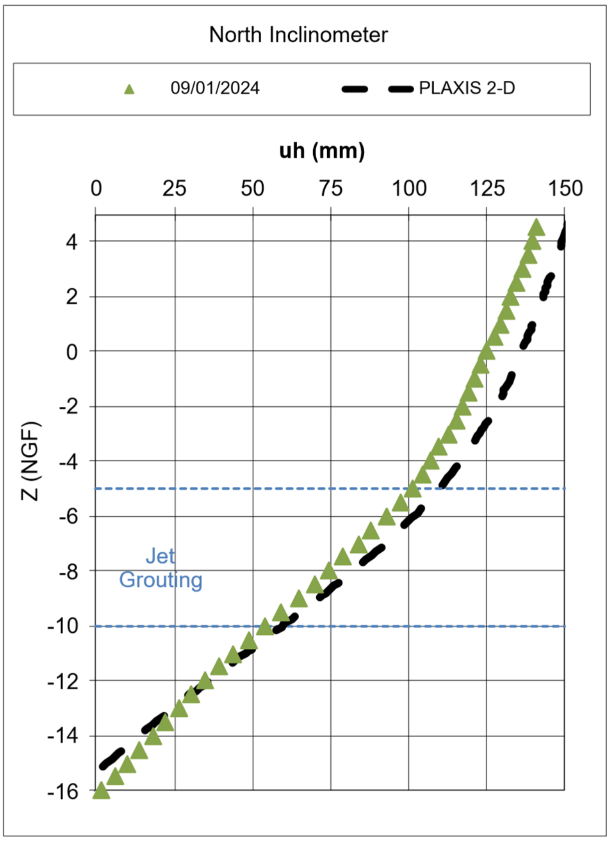

Figure 6. Initial state before excavation of the north diaphragm wall (positive values correspond to displacements in the direction of the retained ground).

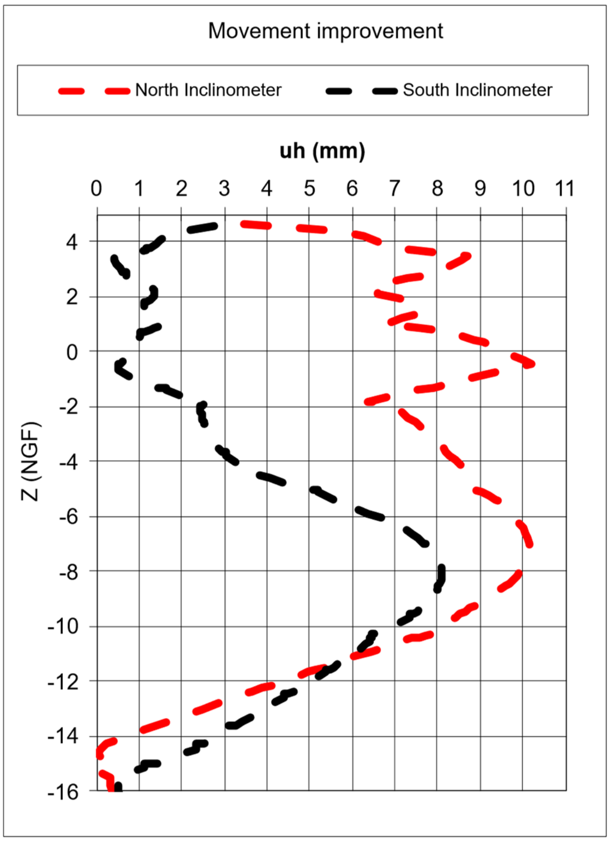

Figure 7. Differences -Absolute value- on horizontal displacement for pre-approved and conducted construction sequences.

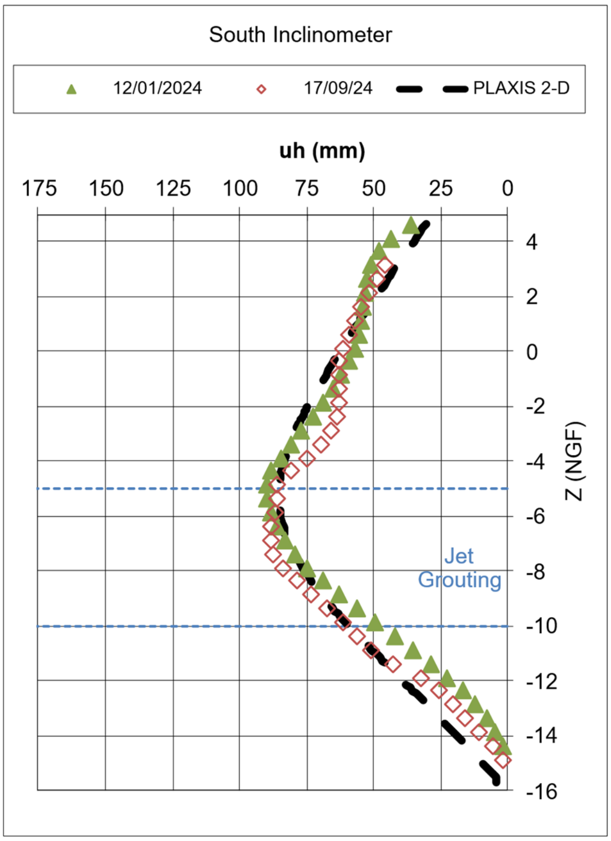

Figure 8. Final state of the south diaphragm walls at the end of excavation construction sequence.

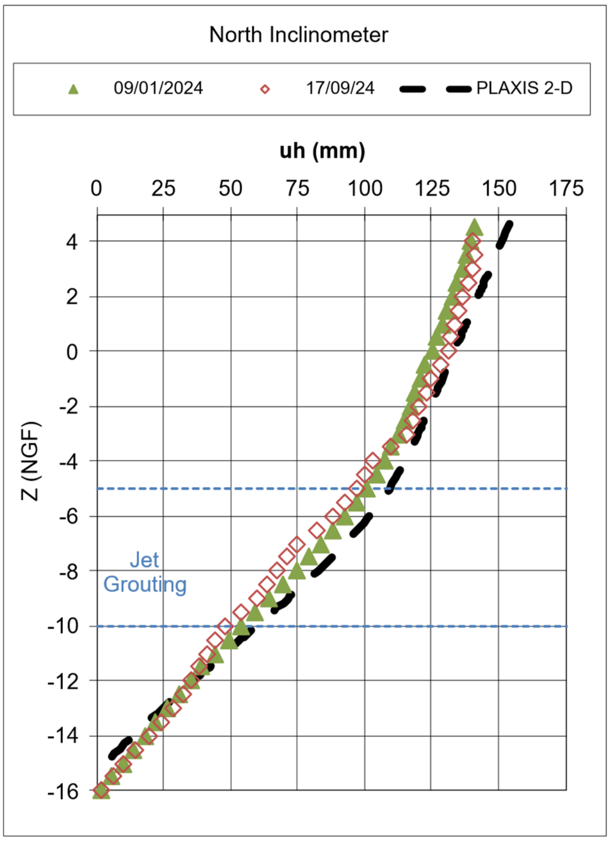

Figure 9. Final state of the north diaphragm walls at the end of excavation construction sequence.

Considering the initial state of diaphragm wall, the revised construction sequence shown in Table 4 was designed, using the soil reaction towards the diaphragm walls, and additional force from the prestressed struts, to recover the original position before the jet grouting works, minimizing the curvature and limiting the bending moment for the already deformed diaphragm wall.

The chart in Figure 7 shows the difference between the predicted horizontal deformations with the pre-approved construction sequence and that with the adapted/ modified construction sequence calculated on the 2D finite element model. The high prestress initially planned for the upper struts would have worsened the displacements triggered by the jet grouting works on the north side. Reducing this prestress and adding an additional strut level with high prestress just above the base slab successfully mitigated the wall bulging near the excavation base and the grout plug, which was indeed the area most structurally affected by the jet grouting works. The 10 mm reduction in wall bulging restored structurally acceptable stress conditions for the diaphragm wall, particularly in terms of bending moment distribution, ensuring compliance with design limit states.

5 EXCAVATION AND MONITORING

The readings of the wall inclinometers, from the end of the Jet Grouting works to the end of Phase 10, confirm the predicted effects of the construction sequence on the behavior of diaphragm walls. The monitoring results at the end of the excavation are shown in Figure 8 and Figure 9.

These results were used to evaluate long term structural behaviour of the diaphragm walls, roof slab and base slab.

6 CONCLUSIONS

The monitoring data from the project and their analysis led to the following conclusions:

1. In the zone where the clayey silts are the most sensitive — in a very plastic to liquid natural state— the diaphragm wall experienced significant horizontal displacements, ranging from 7 to 14 cm.
2. The water content of the clayey silts ranged from 80 % to 160 % of the Atterberg liquid limit. Liquid behavior of natural soils can explain the displacement measured, even with a fraction of the jet pressure at the nozzle (around 400 bars)
3. Displacement of the diaphragm wall can be reproduced roughly in a finite elements model using small values (around 100 kPa) of hydrostatic pressure to model the jet grouting works on an impermeable and liquid state soil.
4. Adaptations of the jet grouting method (reduction of jetting pressure, improvement of the fluidity of the injection grout, reduction of the height of central columns of the plug, …) could not completely prevent the pre-excavation diaphragm wall movements. A new construction sequence was defined in order to guarantee long term stability of the diaphragm wall.
5. Measurements during excavation confirm the results of the model used for soil structure interaction studies and prove the efficiency of adaptations to construction sequence undertaken.

7 REFERENCES

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