Preserving Historical Structures in Urban Tunneling: Diaphragm Walls Embedded in Bedrock

F. Capata[1], M. Iacorossi[2], F. De Santis[3]

[1]Engineer, SGS Studio Geotecnico Strutturale srl, Rome, Italy, email: f.capata@studiosgsroma.com

[2]Engineer, SGS Studio Geotecnico Strutturale srl, Rome, Italy, email: m.iacorossi@studiosgsroma.com

[2]Architect, SGS Studio Geotecnico Strutturale srl, Rome, Italy, email: f.desantis@studiosgsroma.com

ABSTRACT

The construction of the West Link tunnel in Göteborg, particularly in the Kvarnberget section, presented significant geotechnical challenges due to the presence of a thick layer of soft clay and a moraine layer with large boulders. To ensure stability and protect nearby historical buildings, a top-down excavation method was employed, with reinforced concrete diaphragm walls (D-walls) embedded into the bedrock. Given the variable depth and steep inclination of the rock surface, the diaphragm walls were anchored using small-diameter steel piles (D=120mm) and jet grouting injections to enhance load transfer and minimize differential settlements.

During construction, difficulties arose in achieving the intended connection between the diaphragm walls and the bedrock due to the heterogeneity of the moraine and high-pressure groundwater conditions. An initial attempt at jet grouting failed, necessitating adjustments in injection parameters and pre-cutting techniques. Subsequent testing verified the improved effectiveness of this approach. Structural analyses carried out confirmed the adequacy of the modified design in maintaining stability.

Monitoring of adjacent historical buildings, such as Nordstaden 13.12 and 13.7, indicated that vertical and horizontal displacements remained within acceptable limits, with no significant structural damage attributable to the excavation. The observed settlement trends align with historical patterns, suggesting that the mitigation measures effectively preserved the structural integrity of these buildings. This study highlights the success of combining deep foundation techniques with continuous monitoring in protecting historical structures during underground construction.

Keywords: diaphragm walls; top-down construction; jet grouting; steel micro-piles; moraine; historical buildings; monitoring; settlements.

INTRODUCTION

The West Link (Västlänken) is a major underground railway infrastructure project in Göteborg, Sweden, with extensive works in dense and historically sensitive urban areas. In the Kvarnberget lot, a segment of approximately 190 m of cut-and-cover tunnel was designed and constructed with excavation depths on the order of 15–18 m. The site is particularly challenging due to a deep bedrock surface (approximately 30–40 m below ground level in places), overlain by glacial deposits and thick soft clay. These ground conditions require excavation support systems with high stiffness to limit ground movements, as well as robust measures to protect nearby buildings, including historical structures.

A top-down construction approach was selected to minimize deformations, combining early slab installation with diaphragm walls (D-walls) serving as both temporary retaining structures and permanent tunnel box walls/foundations. A key design requirement was the reliable load transfer and restraint at the base of the D-walls despite the presence of heterogeneous moraine and an irregular bedrock interface.

This paper presents the design intent, field challenges, and the final adopted toe stabilization solution—steel small-diameter piles and jet grouting—implemented to connect D-walls to bedrock and reduce differential settlements. The paper also links the adopted solution to the observed response of adjacent historical buildings, with emphasis on Nordstaden 13.12, where cracks were notified during construction, and on the adjacent building 13.7, given the role of long-term differential settlements at their interface.

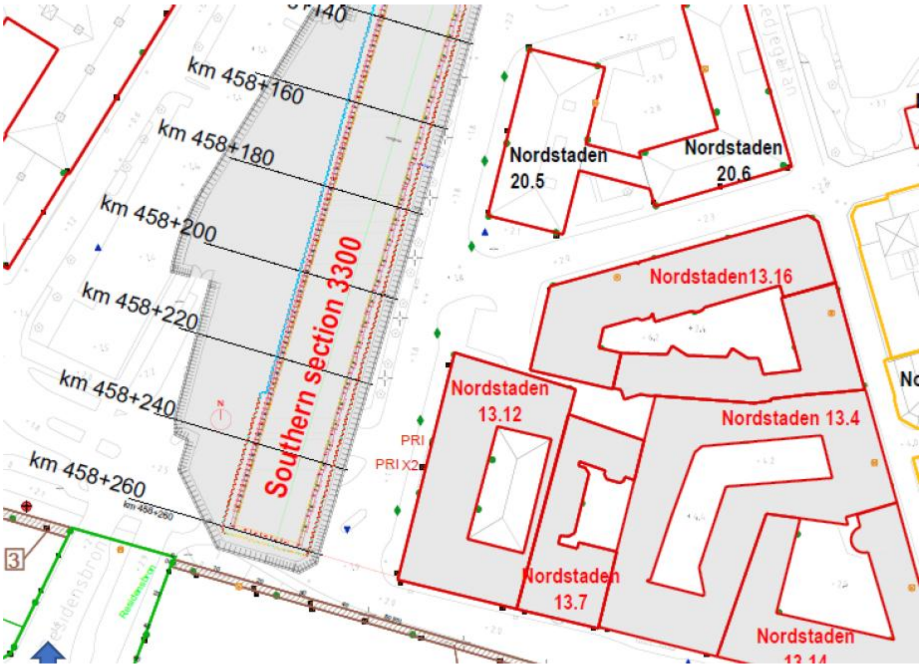

Figure 1. Project location and plan view of Kvarnberget Southern section (tunnel alignment and nearby buildings).

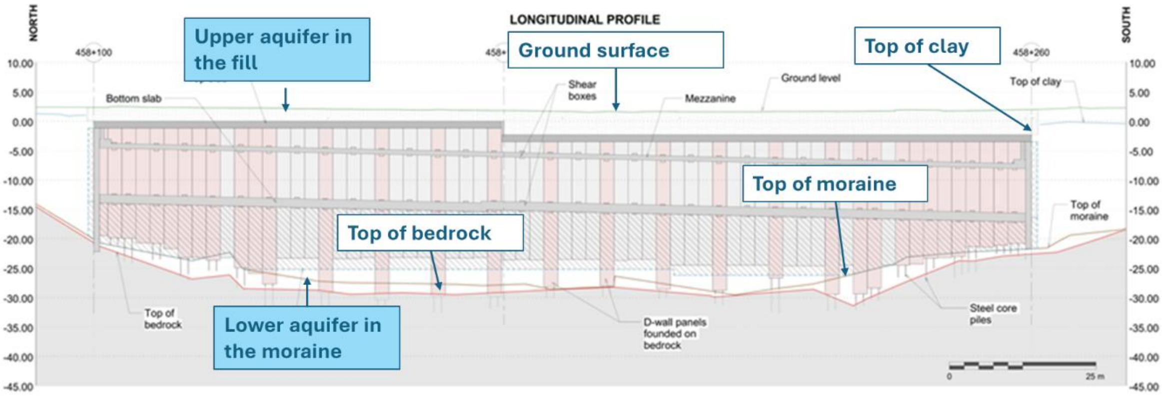

Figure 2. Longitudinal geological profile with excavation level, D-walls, moraine and bedrock variability.

GEOLOGICAL AND GEOTECHNICAL SETTING

The stratigraphy in the Kvarnberget area consists, from top to bottom, of:

A thick sequence of soft clay deposits, with typical thickness in the order of 20–28 m, characterized by high compressibility and long-term consolidation settlements.
An underlying moraine layer with variable thickness, generally between 1 m and 4 m, composed of sands and gravels with frequent large boulders, resulting in highly heterogeneous mechanical and hydraulic properties.
The bedrock surface, encountered at highly variable depths, typically between 25 m and 35 m below ground level, with local steep inclinations ranging approximately from 15° to 45°.

Groundwater conditions represent a major complicating factor. The moraine layer hosts a pressurized aquifer with piezometric heads locally reaching up to about 30 m above excavation level, these conditions strongly affect both excavation stability and groutability during toe treatment works.

The combination of thick soft clay, boulder-rich moraine, an irregular and inclined bedrock surface, and high groundwater pressure creates particularly unfavourable conditions for achieving a reliable and continuous contact between diaphragm wall toes and bedrock, while the tolerance for induced ground movements is extremely limited due to the proximity of sensitive historical buildings.

DESIGN CONCEPT AND CONSTRUCTION METHOD

Top-down cut and cover solution

To limit deformation and protect the surrounding area, the tunnel box was constructed using a top-down method. Diaphragm walls were installed first, followed by excavation stages coordinated with the construction of slabs from top to bottom. This method provides early lateral restraint and reduces wall deflections compared to bottom-up approaches, particularly beneficial in soft ground and in the presence of strict deformation criteria near existing buildings.

Diaphragms walls

The D-walls were designed to serve dual functions:

- Excavation support during staged excavation,
- Permanent structural components of the tunnel box, contributing to long-term stability and serviceability.

Panels are typically in the order of 1.0 m thickness with widths around 2.4 m (panel geometry), with lengths varying depending on bedrock depth and design needs. Selected “long” panels were intended to reach bedrock to provide global stability and to control settlements.

Toe stabilization with steel piles and jet grouting

Given the inclination of the bedrock surface and the potential for toe sliding and non-uniform support, a combined stabilization solution was adopted to ensure an effective connection between the diaphragm wall toe and the underlying rock. Small-diameter steel piles (D = 120 mm), socketed into bedrock, were installed to provide a direct mechanical anchorage of the wall. In addition, jet grouting was executed beneath the wall toe to improve contact continuity, enhance vertical load transfer and reduce the risk of differential settlements along the wall alignment. The combined system was conceived to ensure adequate stiffness and reliability of the toe restraint under the expected geological and hydraulic conditions.

SITE CONSTRUCTION AND CHALLENGES

During field execution, starting in early 2021, the moraine layer proved substantially more resistant and heterogeneous than anticipated in the design phase and included frequent large boulders, in several cases exceeding 0.5 m in diameter. In multiple locations the excavation tools were unable to reliably penetrate the moraine layer down to the bedrock surface. As a consequence, the realized distance between the diaphragm wall toe and the bedrock increased significantly with respect to the design assumptions, from the expected value of less than 1 m to local values exceeding approximately 4 m, as schematically illustrated in Figure 3.

This geometrical deviation had important structural and execution implications. The steel piles intended to connect the diaphragm wall to the bedrock became longer than originally foreseen, increasing both their structural demand and their sensitivity to alignment accuracy and rock socket quality. At the same time, the initial design assumption that jet grouting would be required only to fill a limited toe gap was no longer valid, particularly under the prevailing pressurized groundwater conditions in the moraine layer.

The increased toe-to-rock gap also implied that a larger portion of the vertical load and bending effects had to be transferred through the combined pile–grout system rather than through direct wall–rock contact. This condition required additional verification of the structural capacity of the steel piles and of the global stiffness of the toe restraint system.

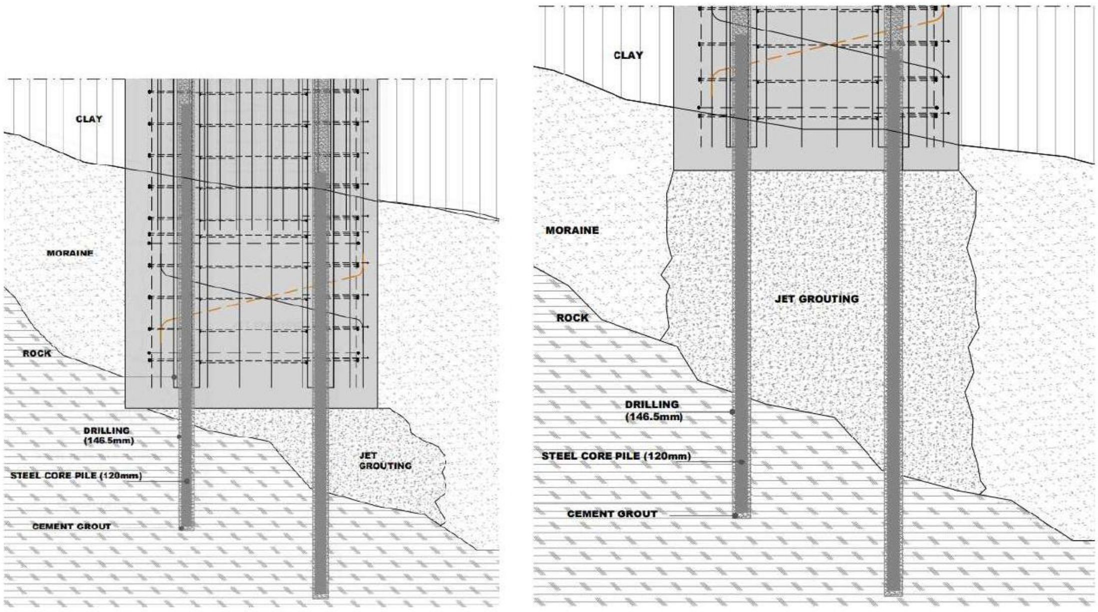

Figure 3. Comparison between design assumption (left) and as-built toe condition (right) showing increased wall toe–bedrock gap.

Figure 4. Example of large boulders extracted from the moraine layer during diaphragm wall excavation.

MODIFIED TOE TREATMENT AND QUALITY CONTROL

The initial jet grouting attempts beneath the diaphragm wall toe did not provide the intended continuity and material quality. Verification coring showed that the treated material was often discontinuous or poorly cemented, mainly due to the combined effects of high groundwater pressure, unfavourable soil fabric and non-optimized execution parameters.

A revised execution procedure was therefore adopted, including a preliminary pre-cutting phase performed at reduced pressures to condition the ground and decrease local permeability, followed by optimized injection parameters tailored to the heterogeneous moraine. Quality control was carried out through systematic control drilling and core recovery beneath selected panels.

Representative cores extracted from the treated zone, shown in Figure 5a, confirmed the continuity of the cemented mass and the effective incorporation of coarse moraine fragments within the grout matrix. Acoustic probes installed at radial distances between approximately 0.6 m and 1.0 m from the nominal column axis were used to estimate the resulting column diameter and verify the lateral extension of the treated volume. Unconfined compressive strength tests performed on selected specimens, as illustrated in Figure 5b, yielded strength values consistent with the project requirements for toe improvement and load transfer.

Based on these results, the revised execution parameters were adopted for the production works beneath the diaphragm wall panels, ensuring continuous treated zones and improved contact conditions at the wall–rock interface.

a)  b)

Figure 5. (a) Core samples from control drilling showing continuity of jet-grouted material; (b) specimen prepared for unconfined compressive strength testing.

STRUCTURAL VERIFICATIONS

Additional structural analyses were carried out to verify the performance of the steel piles and of the combined toe restraint system under the revised geometrical conditions observed in the field. Finite element models were developed to represent the interaction between the diaphragm wall, the steel piles and the bedrock, accounting for the variable embedment depth and inclination of the rock surface.

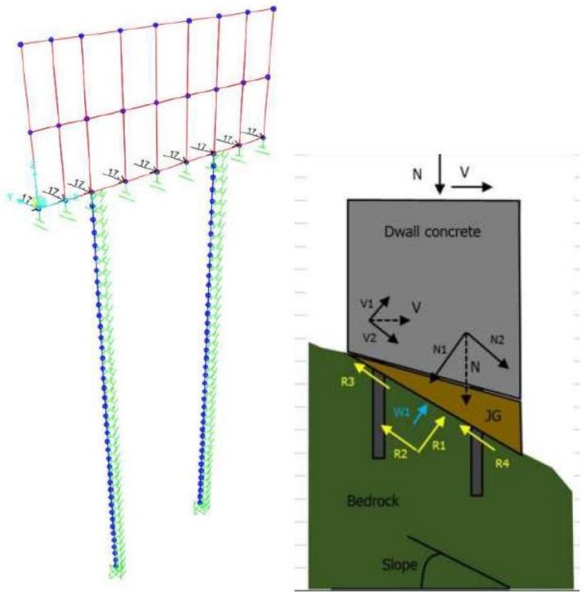

Figure 6. Structural model of diaphragm wall toe with steel piles and load transfer mechanism at the wall–pile–rock interface.

Several geometrical configurations were analysed, considering different values of the toe-to-rock gap and different rock inclinations, in order to evaluate the sensitivity of internal forces and displacements to the observed deviations from the design geometry. Particular attention was paid to the axial force and bending moments developing in the steel piles, as well as to the stress transfer mechanisms at the pile–rock and pile–wall interfaces.

The analyses confirmed that, provided a minimum rock socket length was achieved, the steel piles were able to safely transfer the vertical loads and restrain potential toe sliding with adequate safety margins. The presence of the jet-grouted zone contributed to increasing the stiffness of the toe support and to distributing the loads more uniformly among adjacent panels, thereby reducing differential movements along the diaphragm wall alignment.

MONITORING OF DIAPHRAGM WALL BEHAVIOUR AND ADJACENT BUILDINGS

A comprehensive monitoring system was implemented in order to control both the behaviour of the diaphragm walls and the response of the adjacent historical buildings during the excavation and construction phases. The monitoring program included automatic total station measurements on wall reference points and building façades, levelling pins for vertical displacement control and systematic visual inspections.

The monitoring focused on the historical buildings Nordstaden 13.12 and Nordstaden 13.7, part of the Nordstaden 13 complex located on reclaimed land in the former harbour area of Göteborg. Building 13.12 was originally constructed in approximately 1850 and later extended in 1926 (Figure 7); the original masonry structure is founded on a traditional timber mattress foundation ("rustbädd"), consisting of horizontal timber layers resting on soft ground, while the later extension is supported by a concrete and steel structure founded on end-bearing piles. Building 13.7 (Figure 8) was constructed in approximately 1882 and is founded on timber friction piles driven in soft cohesive deposits, typical of foundation systems adopted in harbour areas during the nineteenth century. The two buildings are directly adjacent and share a common interface wall, making them particularly sensitive to differential settlements and relative movements.

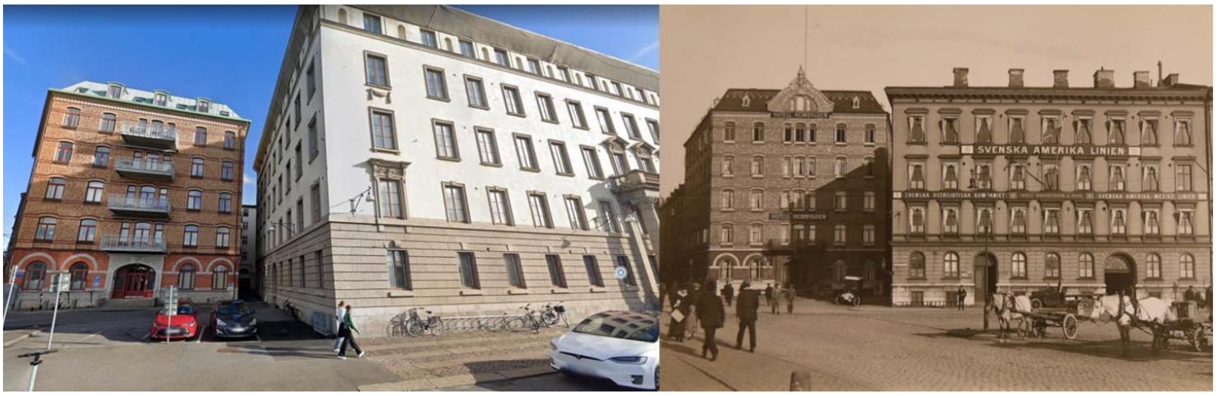

Figure 7. Building 13.12: comparison between recent status (left) and historical status at 1920 (right).

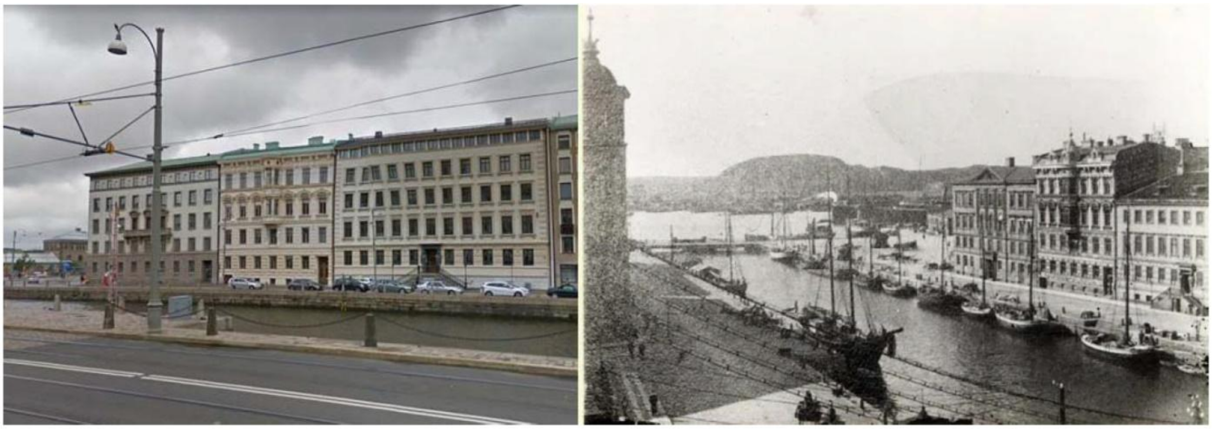

Figure 8. Building 13.12 and 13.7: comparison between recent status (left) and historical status at 1920 (right).

Project threshold values for vertical displacement, horizontal displacement and tilting were defined in the design phase and continuously checked during construction. For Building Nordstaden 13.12, the adopted criteria prescribe maximum additional vertical displacements of ±5 mm (in addition to ongoing natural settlements), maximum horizontal displacements of ±20 mm and a maximum tilting of 0.1%. For Building Nordstaden 13.7, the corresponding limits are ±20 mm for vertical displacements, ±20 mm for horizontal displacements and 0.2% for tilting.

Historical levelling data indicate that both buildings experienced significant long-term settlements since their construction, exceeding approximately 30 mm for Building 13.12 and 55 mm for Building 13.7, with a progressive reduction of settlement rate in recent years. During the construction phases involving diaphragm wall installation, Deep Soil Mixing and early excavation works, additional vertical displacements recorded at Building 13.12 remained generally within ±2 mm, while horizontal movements were typically limited to a few millimetres and always below 10 mm, as shown in Figure 9 and Figure 10.

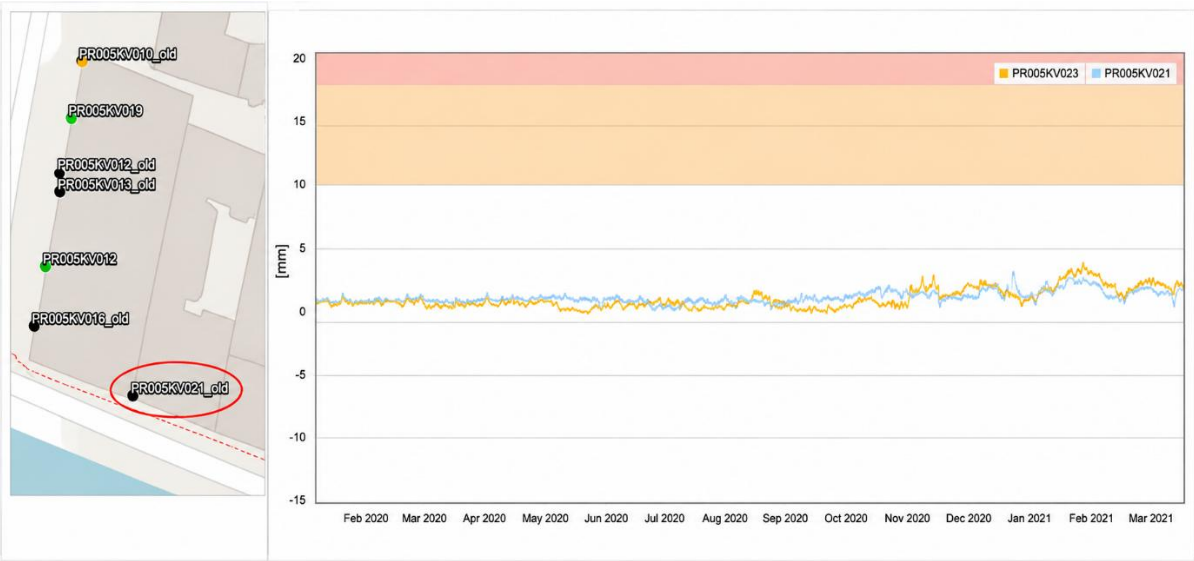

Figure 9. Horizontal displacement of Building 13.12 versus construction progress.

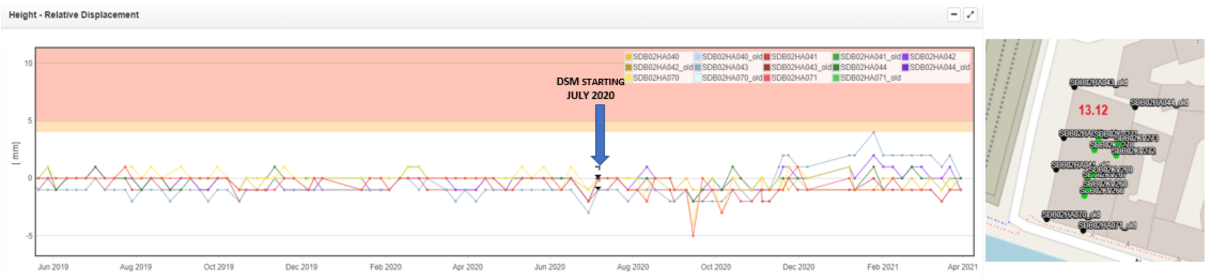

Figure 10. Vertical displacement of Building 13.12 versus construction progress.

Similar trends were observed for Building 13.7, with vertical displacements within ±5 mm and horizontal movements below 10 mm, as illustrated in Figure 11 and Figure 12.

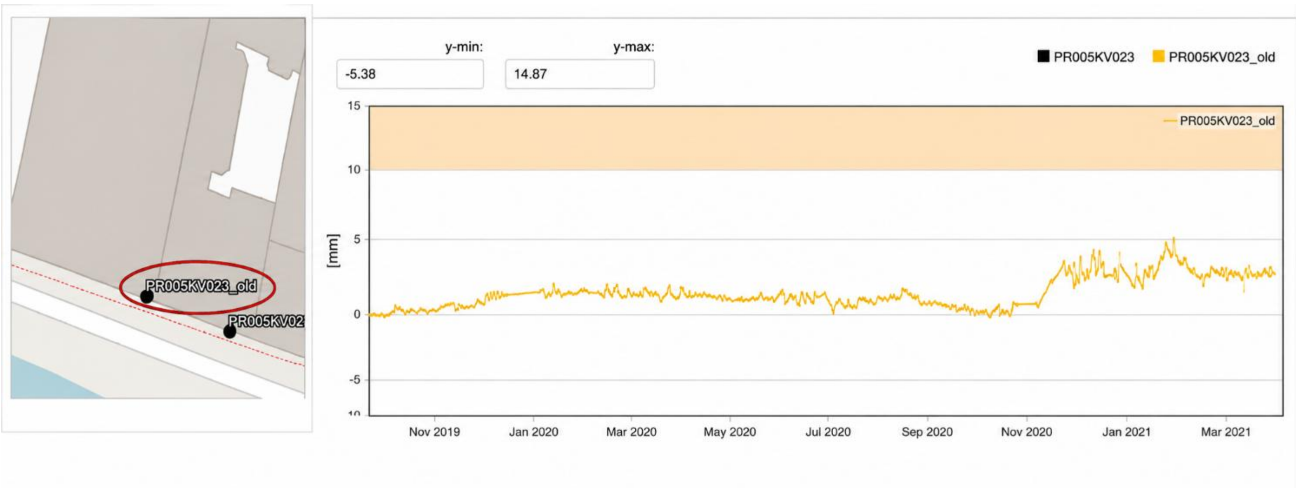

Figure 11. Horizontal displacement of Building 13.7 versus construction progress.

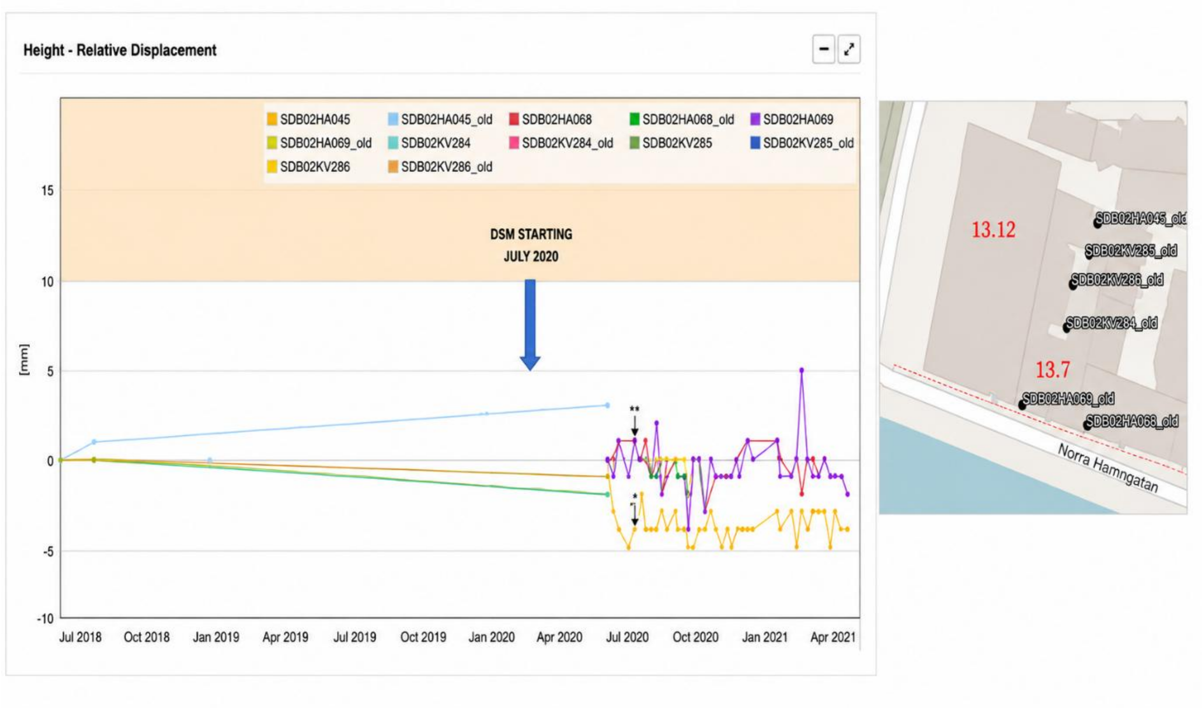

Figure 12. Vertical displacement of Building 13.7 versus construction progress.

All recorded displacements remained well below the project threshold limits throughout the monitored construction phases. These results indicate that the adopted diaphragm wall system and toe stabilization measures provided sufficient stiffness to effectively control excavation-induced deformations and to preserve the structural integrity of the adjacent historical buildings.

IMPACT ON ADJACENT BUILDINGS

Cracks notified inside Building 13.12 during construction were mainly located along the interface with the adjacent Building 13.7, suggesting a potential interaction mechanism between the two structures. Review of historical documentation and inspection records indicates that similar crack patterns and façade distortions were already present prior to the start of the West Link works, pointing to a pre-existing deformation process.

Analysis of historical settlement trends shows that the two adjacent buildings have experienced markedly different long-term settlement behaviour. Building 13.7 exhibits a significantly higher cumulative settlement than Building 13.12, likely related to differences in foundation typology and structural history. This differential behaviour may generate restraint effects along the common interface, leading to stress concentration and cracking even in the absence of external perturbations.
The construction-phase monitoring data show that vertical and horizontal displacements in the vicinity of the crack locations remained limited to a few millimetres and did not display any systematic increase associated with excavation stages. In particular, no acceleration of movements was detected during diaphragm wall installation, Deep Soil Mixing or early excavation phases. Visual inspections carried out before and after the notified event confirmed the absence of significant crack propagation or widening.

These elements support the interpretation that the observed damage is predominantly associated with long-term differential settlements between the adjacent buildings rather than with excavation-induced effects.

CONCLUSIONS

Diaphragm walls were successfully employed as both excavation support and permanent foundation elements in the Kvarnberget section of the West Link project. The presence of heterogeneous moraine with large boulders and an irregular, inclined bedrock surface prevented uniform toe founding and generated local toe-to-rock gaps exceeding 4 m, requiring the adoption of a combined stabilization strategy. The use of small-diameter steel piles socketed into bedrock, together with jet grouting beneath the wall toe, provided a reliable load transfer mechanism and ensured adequate restraint against sliding and differential support conditions. Additional structural verifications confirmed the adequacy of the steel piles and of the combined toe restraint system under the revised geometrical conditions observed in the field. Monitoring data indicate that excavation-induced displacements of the adjacent historical buildings remained limited to a few millimetres and well within the prescribed project thresholds. The observed cracks in Building 13.12 and 13.7 are interpreted as primarily related to long-term differential settlements between adjacent structures rather than to construction-induced effects. Overall, the adopted solution proved effective in limiting deformations and preserving the structural integrity of sensitive historical buildings during deep excavations in complex glacial deposits.