Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-240

Effective Implementation of the Observational Method for the HS2 Old Oak Common Station Box

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ABSTRACT: When completed, High Speed Two (HS2) will be the UK's second high-speed train line and will provide important extra capacity on Britain's rail network. Old Oak Common, located in the western suburbs of London, will be the only intermediate station on the line, providing interchange to the Elizabeth Line, Great Western Main Line and Heathrow Express. The station box incorporates provision for track crossover and as a consequence is longer than a standard station box at 840m long and is up to 23m deep. The box was formed by diaphragm wall construction with a top-down construction sequence. At the eastern end of the station, one level of temporary props was included in the original construction sequence to accommodate the larger than normal structural span required to accommodate the openings for the tunnel boring machines. This paper describes the successful collaboration between the designers of the station box (WSP), the main contractors responsible for delivery of the project (BBVS), the sub-contractor responsible for the excavation and concrete packages (Expanded) and the overall client (HS2) in the implementation of the Observational Method to remove the single level of temporary props. The Observational Method was implemented in accordance with the principles of *ipso tempore* (Type B according to CIRIA C760, Gaba et al. 2017). The paper describes the changes from the "characteristic" design implemented in the reference design and the collaborative changes made by all parties for the "best estimate" design that demonstrated the temporary props could be eliminated. The paper discusses the derivation of the trigger limits, the instrumentation used to assess the retaining wall behaviour and the back analysis of the parameters that provide an important fully calibrated case history for deep retained excavations in London Clay.

KEYWORDS: Observational Method.

1 INTRODUCTION

The Observational Method (OM) in the context of geotechnical engineering was firstly introduced by Terzaghi and Peck (1967) as a 'best way out' process to deal with unforeseen conditions.

Since then, the OM has evolved as a design approach in which the performance of the structures is assessed in a range of possible conditions allowing for predetermined modifications and/or mitigation measures to be implemented during construction depending on the actual behaviour of the structures.

The OM is a design method recognised in EN 1997 (EC7). The implementation of the OM in the UK typically follows the frameworks presented in CIRIA 185 (Nicholson *et al.* 1999) and CIRIA C760 (Gaba *et al.* 2017).

This paper presents an overview of the successful implementation of a late proposal by the Contractor to eliminate a level of temporary propping originally designed to support a deep retained cut excavation. This is another example of the use of the OM in the context of large civil engineering projects in the UK (Chen et al. 2015).

2 DESCRIPTION OF THE OLD OAK COMMON SCHEME AND STATION BOX

The HS2 Old Oak Common (OOC) Station (Figure 1) is the London interchange station of the UK's second high-speed railway.

The station box is an 840m long reinforced concrete retained-cut structure that incorporates tunnel portals at each end connecting with the twin-bore Euston tunnels to the east and the Northolt tunnels to the west. The width of the station box varies between approximately 48m at the ends and 66m at the central part, with a narrower 16m width 'trouser leg' at the northeastern corner. The depth of the station box varies between 15m and 23m at the eastern extent to accommodate the launching of the tunnel boring machines for the Euston tunnels.

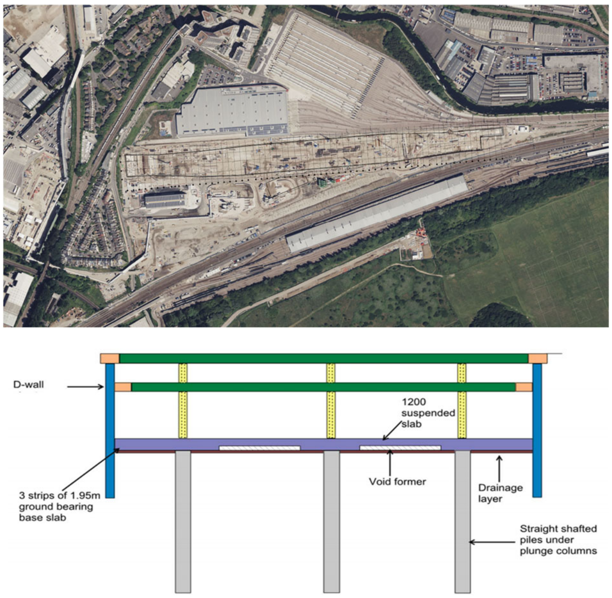

Figure 1. Drone view of the HS2 OOC Station site and indicative cross-section

The retained-cut is formed of diaphragm walls, typically 1.0m thick, laterally supported by the base slab, intermediate level props and walling beams and the ground floor deck/concourse slab structure constructed in a top-down fashion. At the tunnel portals and where the span between the intermediate level and base slab increases, 1.2m thick diaphragm walls were adopted.

Plunge columns supported on straight shafted piles were required for the top-down construction. Following construction of the base slab, the foundation acts as a combined slab/pile system.

An aerial photograph of the eastern end of the box is presented in Figure 5.

3 DESCRIPTION OF CONTRACTUAL RELATIONSHIPS

Working in partnership with HS2 Ltd, the Balfour Beatty, VINCI, SYSTRA Joint Venture (BBVS), supported in design by WSP, is responsible for the final design, construction and commissioning of the station.

Pile and diaphragm wall construction works were subcontracted to BBVS/Bachy Soletanche JV (SB3).

Construction works for the station box were subcontracted to Expanded Structures, a division of Laing O'Rourke (Expanded). Monitoring works associated with the scheme were undertaken by Sixense Ltd.

4 GROUND AND GROUNDWATER CONDITIONS

Prior to the start of the detailed design of the station box, a detailed ground investigation was undertaken at the site. The interpreted geological design profile is summarised in Table 1, together with a brief description of each stratum.

Table 1. Interpreted geological design profile.

Stratum	Elevation (Thickness)	Reference Soil Description
Made Ground	+26.50mOD (1.00m)	Generally consistent across the site, comprising black sandy gravel with a variable clay content. The gravel size fragments consisted of ash, clinker, brick, railway ballast, concrete, slab, pottery and glass.
Weathered London Clay	+25.50mOD (5.00m)	Firm to stiff, thinly laminated clay occasionally with grey gleying, sand partings and selenite crystals.
London Clay	+20.50mOD (65.00m)	Stiff, becoming very stiff, greenish grey, micaceous fissured clay, with bands of claystone.
Lambeth Group	-44.50mOD (20.00m)	The general sequence of the Lambeth Group beds encountered comprised the Upper Shelly Beds, Upper Mottled Beds, Lower Mottled Beds and Upnor Formation.
Thanet Sands	-64.50mOD (3.00m)	Homogeneous silty, fine to medium sand. The unweathered formation is grey to brownish grey, and at the surface it weathers to a pale yellowish grey.
Chalk	-67.50mOD	For design purposes, the top surface of the Chalk was assumed as a rigid boundary.

The substructure and foundations of the station box remained embedded within the London Clay formation. A comprehensive regime of in-situ and laboratory tests was scheduled as part of the site-specific ground investigation works targeting the strength, in-situ stress state, permeability and stiffness of the London Clay formation (Figure 2).

The 'characteristic' geotechnical parameters for the London Clay formation adopted in the geotechnical design of the station box substructure are summarised in Table 2. Parameters for the non-linear stiffness-strain dependent model HSS (Schanz et al. 1999) were derived from the available site-specific ground investigation targeting the measurement of soil stiffness at different strain ranges.

Long term monitoring of piezometers identified a hydrostatic ground water profile from the top of the London Clay.

5 ORIGINAL DESIGN AND ASSUMPTIONS

The geotechnical design philosophy for the diaphragm walls was based on Design Approach 1 (DA-1) of EN 1997-1 (EC7), as prescribed in the UK National Annex of EC7, and complemented by the good practice guidance provided in CIRIA C760, Gaba *et al* 2017.

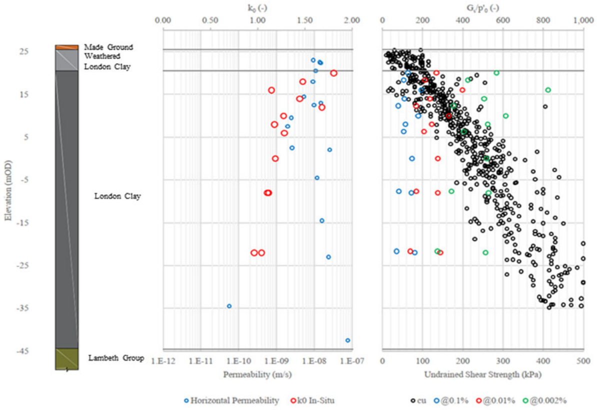

Figure 2. London Clay formation properties

Table 2. Geotechnical 'characteristic' parameters for London Clay formation

Parameter	Symbol	Value	Unit
Unit weight	γ	20	kN/m^3
Angle of shear friction	ϕ'	25	°
Cohesion	c'	5	kPa
Undrained shear strength	c_u	30 at +25.50mOD 214.5 at +5.00mOD	kPa
Young's modulus	E	Non-linear	MN/m^2
Poisson's ratio	ν	0.2	-

Two-dimensional and three-dimensional Finite Element (FE) analyses were carried out using Plaxis 2D and Plaxis 3D to assess the performance of the diaphragm walls. To reflect uncertainty in construction periods at the design stage, a period of 12 months was allowed for to reflect advice received at that stage.

The results from these analyses were used to calibrate limit equilibrium models (Wallap) that were then used for the design to speed up the wall analysis calculations and the implementation of changes in geometry and/or construction sequence. A mixed earth approach was adopted in the limit equilibrium analyses with drained conditions on the active side of the wall and undrained conditions on the passive side (see Chapter 5.6.3 in CIRIA C760, Gaba *et al* 2017).

Structurally, the diaphragm walls were designed to satisfy the ultimate limit state and the project requirement of limiting crack widths to 0.2mm at the serviceability limit state. It was found that satisfying crack width criteria generally governed the structural design.

As part of the design development, a temporary propping system was introduced at the deeper, eastern end of the station box, to control wall forces and deflections where the span between the intermediate and base slabs increases for the construction of the towards Euston tunnels.

6 EXPANDED PROPOSALS AND REVISED TIMESCALES

Following their appointment as the contractor for the works, Expanded considered the possibility of implementing the OM to eliminate the need for the temporary propping system at the eastern end of the station box. This reappraisal was prompted by Expanded's programme for the works in this area, which indicated that the excavation from the underside of the intermediate level down to the formation level, and the construction of the base slab, was scheduled to take less than three months. This represented a significant reduction in the duration of the works compared to the assumptions made in the geotechnical design of the diaphragm walls.

At the time this proposal was presented by Expanded, SB3 had completed the installation of the diaphragm walls around the station box.

7 FEASIBILITY ASSESSMENT

A comprehensive design review to assess the feasibility of the proposal presented by Expanded was undertaken by WSP as designers of the wall.

The wall analysis models from the original design were progressively revisited and updated by implementing the changes illustrated in Figure 3.

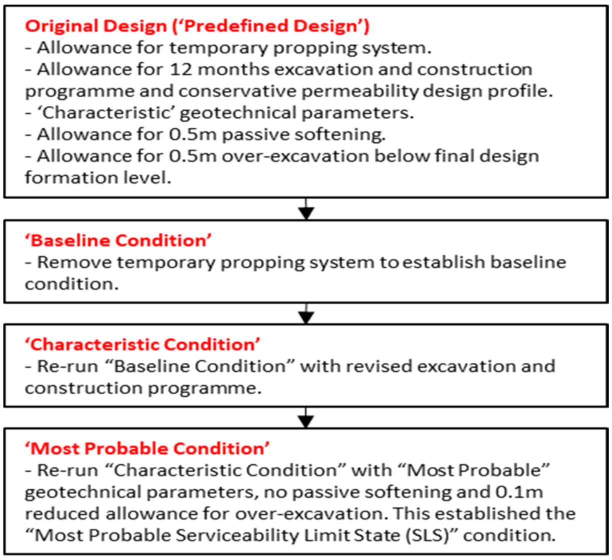

Figure 3. Flowchart illustrating the approach in WSP's feasibility assessment

This approach identified the range of acceptable behaviours for the diaphragm wall and derived trigger levels for the implementation of the OM.

For the revised construction programme provided by Expanded, the output of the coupled two-dimensional FE analysis carried out using Plaxis indicated the wall forces reduced by circa 20 to 25%, primarily due to the shorter construction period and reduction in the degree of consolidation behind the diaphragm walls. These results were used to recalibrate the Wallap models by reducing the depth of drained conditions on the active side of the wall.

Sensitivity checks were undertaken as part of this process to assess the impact of the permeability/drainage assumptions, together with a revised assessment of soil parameters to establish "Most Probable" strength and stiffness design values.

The design review concluded that there was an acceptable probability that the actual behaviour of the diaphragm walls without the temporary propping system would be within the acceptable limits imposed by the as built reinforcement cages, and that compliance with the crack width criteria was maintained.

8 OBSERVATIONAL METHOD IMPLEMENTATION

The OM was implemented after the commencement of the construction works and therefore the *ipso tempore* (Type B according to CIRIA C760, Gaba *et al* 2017) principles were broadly adopted to devise the overall strategy presented in Figure 4.

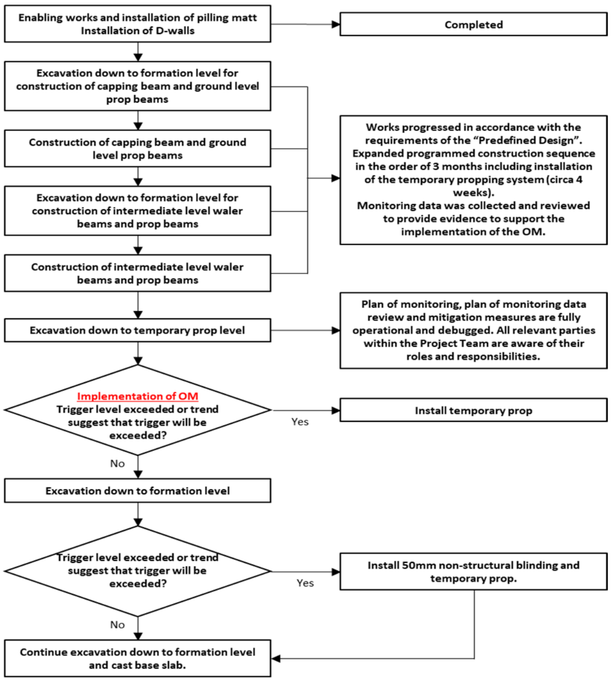

Figure 4. Flowchart illustrating the overall strategy for the implementation of the OM

The temporary propping system was designed and procured by Expanded for installation as required in the "Predefined Design". If the evidence from the initial construction stages indicated that the behaviour of the diaphragm walls would remain within the "Most Probable Condition", the temporary props would not be installed but would remain as a contingency measure throughout the subsequent construction stages. Otherwise, the temporary props would be installed as required in the "Predefined Design".

9 PLAN OF MONITORING

The primary means of excavation control was the deflection and curvature of the diaphragm walls, supplemented by complementary measures such as prop loads and settlement at the back of the wall.

The primary monitoring system comprised eight Shape Accel Array in-place inclinometers (SAA) with 500mm gauge lengths supplemented by 3D targets at the head of the reservation tubes (capping beam monitoring). One of the eight inclinometers installed was extended below the toe of the diaphragm wall to give information on global movement as well as the deflected shape.

10 TRIGGER LEVELS

The philosophy of the trigger levels was based on using the wall deflections to ensure that the performance of the diaphragm walls remained within acceptable limits:

- The green condition was set for wall deflections lower than 'Most Probable' Serviceability Limit State (SLS) prediction.
- The amber condition was set for where wall deflections had exceeded the 'Most Probable' SLS prediction, but the back analysed wall forces were lower than the SLS structural capacity of the wall.
- The red condition was when both the 'Most Probable' SLS wall deflection had been exceeded, and the back analysed SLS capacity of the wall was projected to be exceeded. In this condition, the temporary prop would be installed.

11 ASSOCIATED SITE WORKS AND MONITORING

The excavation and construction works progressed in two main areas (to the east and to the west of gridline 59.5) illustrated in Figure 5 that reflected the strategy for the construction of the base slab. A summary of the key construction dates is provided in Table 3. The excavation works were generally undertaken progressing in an east-to-west direction.

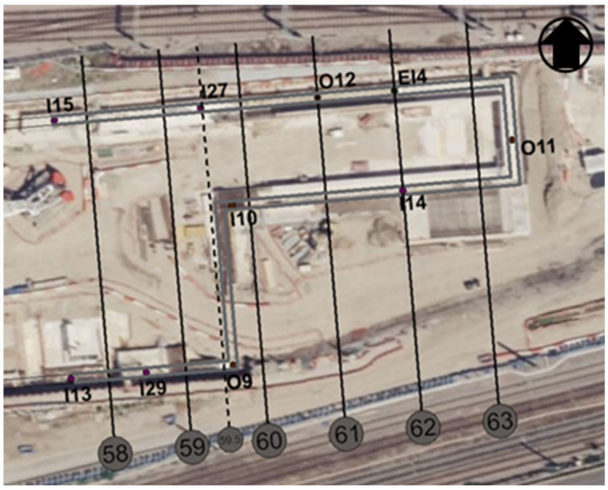

Figure 5. Progress of the excavation and construction works

Table 3. Summary of key construction dates

Stage	East of GL 59.5	West of GL 59.5
Excavation ground level prop	None	None
Excavation intermediate level prop	20/08/2022 to 15/09/2022	23/09/2022 to 18/10/2022
Excavation temporary level prop	13/02/2023 to 07/03/2023	27/03/2023 to 27/04/2023
Excavation formation level	20/03/2023 to 20/04/2023	27/04/2023 to 05/07/2023
Construction base slab	14/07/2023 to 21/07/2023	17/07/2023 to 23/08/2023

Excavation from ground level to intermediate and temporary slab levels proceeded in accordance with the anticipated programme and a collaborative decision was taken by the Project Team to progress the excavation to formation level without installing the temporary props.

Due to issues associated with site logistics, actual construction timeframes exceeded those advised with typical time periods increasing from the proposed 79 days to typically 150 days. Hence, there was a relatively large unsupported height of diaphragm wall, although the contractor left a limited thickness of soil in-situ to negate the impacts of weather on the soil at formation level.

Most of SAA in-place inclinometers recorded trends of increasing movement immediately after the baseline period. The rates of movement were different from instrument to instrument. In some of the instruments, the rates of movement become relatively stable 6 to 8 weeks after the baseline, while other instruments recorded ongoing trends of movement for larger periods of time. Upon review and discussion inside the Project Team, a collaborative decision was taken that such trends of movement were not indicative of the actual behaviour of the diaphragm walls and the monitoring data from the SAA instruments was offset to zero at different points of time to reflect this.

A series of trends of movement towards the excavation and towards the soil mass were also recorded by most of the SAA in-place inclinometers throughout the works. These trends correlated with the overall trends in temperature recorded during the same period (trends of decreasing temperature were associated with movements towards the excavation while trends of increasing temperature were associated with movements towards the soil mass).

A number of SAA in-place inclinometers recorded sudden changes in the magnitude of movement between consecutive readings. Given the sudden nature and the magnitude of such changes, they were not interpreted to be related with genuine ground movement induced by site works and were generally corrected by applying an offset to the monitoring results in the reading after the sudden step change by the magnitude of the step change.

The monitoring data from the 3D targets on the capping beam was used to fix the top of the SAA in-place inclinometer deflection profile and allow the measurement of the amount of inwards movement of the toe of the diaphragm walls. The fixed point was usually taken as the head of the inclinometers once the ground floor slabs and props have been cast. Several issues with the maintenance of lines of sight throughout the works were identified, which required continuous liaison between Sixense and Expanded to ensure the continuity of the monitoring data acquisition.

To illustrate the general principles adopted and challenges faced, a more detailed consideration of the behaviour of SAA's EI4 and I27 is described in the following section.

11.1 *SAA in-place inclinometer EI4*

SAA EI4 extended 50m bgl (approximately 18m below the toe level of the diaphragm wall) and was located in gridline 62 (northern side). The instrument measured the actual behaviour of the diaphragm walls to the east of gridline 59.5.

The available monitoring data (Figure 6) shows a clear response of the diaphragm walls to the progress of the excavation to intermediate prop level and subsequently to temporary prop level.

The instrument responded to the excavation down to formation level. The monitoring data show some reversal of movement (movement towards the soil mass) around the time the base slab pours were completed in the vicinity of the SAA instrument, followed by trends of increasing movement before stabilising. Wall deflections remained below the amber trigger level throughout.

11.2 *SAA in-place inclinometer I27*

SAA I27 extended to the toe level of the diaphragm wall and was located between gridlines 59 and 60 (northern side). The instrument measured the actual behaviour of the diaphragm walls in the transition between the two main construction areas.

Due to the manner in which the excavation was undertaken, the available monitoring data (Figure 7) shows that the diaphragm wall experienced movements from areas to the east and to the west of gridline 59.5 in separate phases. This evidence was corroborated by the available monitoring data from the other SAA in-place inclinometers to the west of gridline 59.5.

The extension of the period between starting the excavation below the intermediate slab and casting the base slab resulted in wall deflections exceeding the 'Most Probable' SLS prediction and hence the requirement to back analyse wall forces.

Figure 7 shows ongoing movement of the London Clay of circa 0.5 to 1.0mm per month at formation level during the stagnant period. This rate of consolidation movement is similar to that noted by the authors for another retaining wall project in London Clay where a similar unsupported height of wall was present, and the base slab was not cast during Covid 19 related lockdown.

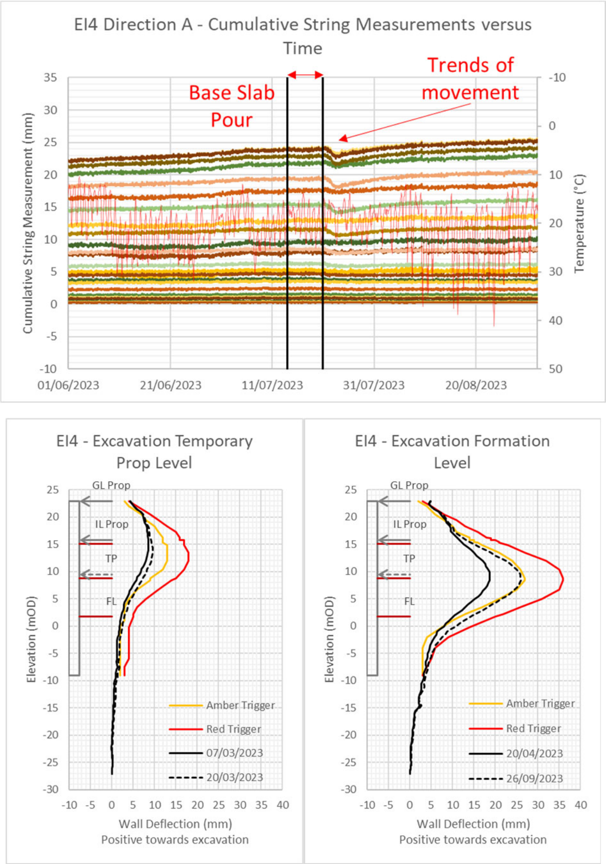

Figure 6. Monitoring data from SAA in-place inclinometer EI4

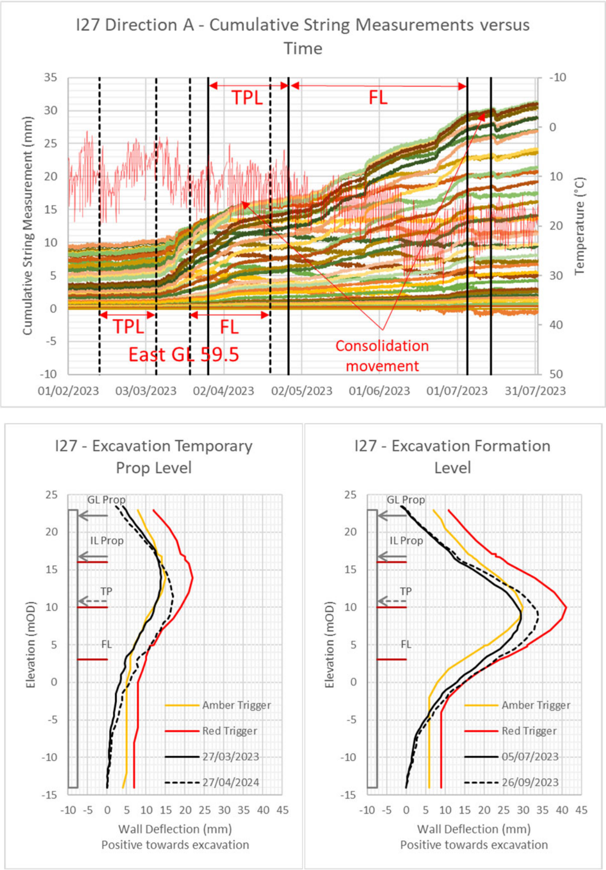

Figure 7. Monitoring data from SAA in-place inclinometer I27

12 BACK ANALYSIS METHODOLOGY AND FINDINGS

Where wall deflections exceeded the 'Most Probable' SLS prediction, wall forces were back analysed using the methodologies described in Ooi et al. (2003). This condition was only encountered in SAA in-place inclinometer I27.

After completion of the project, a back analysis of the I27 was undertaken using the geotechnical finite element code Gofer and the back-analysis tool Gofer META developed by Arup. Gofer META uses a polynomial surrogate model to find the set of geotechnical parameters that give the statistical best fit to the inclinometer data. The calibrated Gofer model was used to calculate the bending moments in the wall for comparison with the other methodologies proposed by Ooi et al. (2003).

The various methodologies provided values that were broadly consistent and were enveloped to compare to the cage capacities (Figure 8). The bending moment profile from the Gofer back analysis correlates well with the other methods but has the advantage of providing a smooth continuous profile.

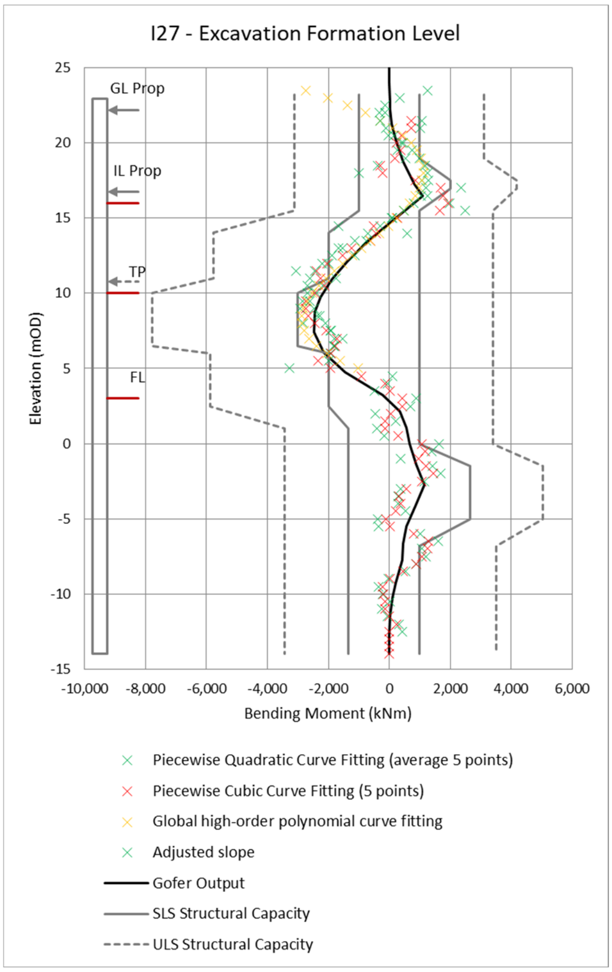

Figure 8. Back analysed wall force I27

Data regression to estimate future trends and establish point of no return for the installation of the temporary props were also undertaken on a regular basis, based on the rates of movement identified from the available monitoring data.

For sections of the wall around I27, wall deflections and forces crept towards the red trigger. The decision was made that the benefits of expediting the casting of the base slab to prop the wall outweighed the additional time required to install the temporary props. This proved the correct decision as the wall forces remained within the as designed envelopes.

13 OTHER OBSERVATIONS

Other observations from the monitoring around the station box (Figure 9) included the monitoring of ground displacements in areas around the box. The data indicated that the zone of influence extended to about twice the excavation depth.

For ground movements outside of the excavation, where large movements were recorded during wall installation, correspondingly smaller movements were recorded during the excavation phase – similar findings were reported by Fernie *et al*. (2001) during the Harrods project and by Hardy *et al* (2021) at the Exhibition Road Quarter at the V&A Museum.

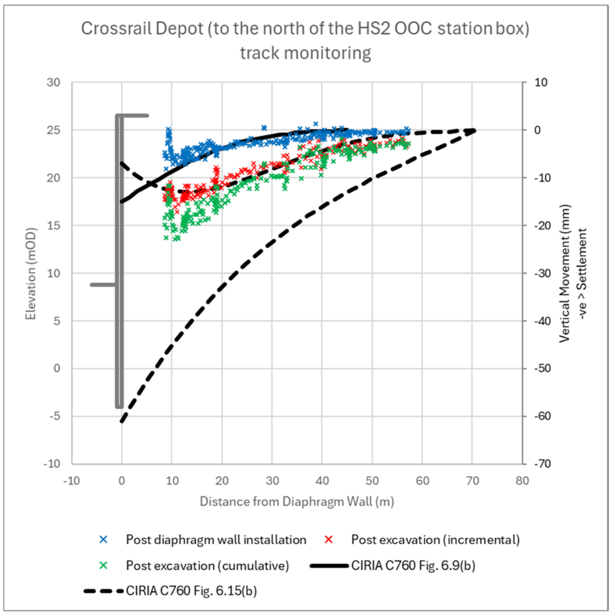

Figure 9. Monitoring results around the station box

14 CONCLUSIONS

The observational method was successfully implemented at the eastern end of the HS2 OOC station box through the successful removal of the temporary prop.

The assumption of undrained conditions in the design of retained cut excavations in London Clay has proved over the years to provide a reasonably economical and safe design. The implementation of the OM in OOC highlights the importance of construction periods, excavation geometry and permeability when assessing the behaviour of excavation induced effects on embedded retaining walls.

For large and deep retained cut excavations, such as the HS2 station box in OOC, the assumption of undrained conditions without further consideration regarding the behaviour of the embedded retaining walls for different drainage conditions within the anticipated construction periods may however underpredict wall forces and deflections. For such situations, a detailed review of the anticipated construction periods and of the soil permeability should be undertaken to inform the design with adequate provision for the anticipated range of possible drainage conditions. Sensitivity analysis should be carried out of the effects of variability in assumed construction periods and soil permeability, and as well of the robustness of the design to accommodate unforeseen circumstances.

The initially advised construction period of 12 months at the design stage was arguably too long but allowed implementation of the OM. The final outturn construction period of 150 days was longer than the planned 79 days at implementation stage and therefore required additional detailed assessment of wall forces. These timescales proved to be important factors in the performance of the wall, measured deflections and the need or otherwise for temporary props, which was managed by the OM.

The finite element analysis proved reasonably accurate in its assessment of wall deflection – subject to the incorporation of the appropriate timescales. Enveloped wall forces were calculated by back analysis of the various methodologies described and remained within the permissible envelope.

15 ACKNOWLEDGEMENTS

The authors would like to thank Dan Williams (BBVS), Peter Hewit (Expanded) and Eduard Casas (Sixense) for their contributions during the implementation of the OM. The authors would also like to thank HS2 for the permission to publish the data presented in this paper.

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