

Back Analysis and Observational Method for Strut Removal for a Deep Underground Excavation in Singapore using Genetic Algorithm Tools

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ABSTRACT: The observational method was used on Singapore's North South Corridor Contract N107 to assess the necessity of a contingency strut when one of the first published use of the characteristic approach in Singapore was applied (Thi et al, 2023). This paper seeks to explore the feasibility of an observational method to study whether further progressive modifications could provide a more optimized and informed future design. Through the completion of construction for this section, further back analysis was undertaken in this paper and applied to other sections in adjacent zones to evaluate the risk of construction, as well as feasibility of the observational method and omission of planned struts as well as the potential of derived parameters being used in other projects with similar ground conditions.

Careful review of the measured wall movements validated the characteristic approach up to the roof slab level. Subsequently, forward predictions to the final excavation level using the calibrated parameters through SAALG's DAARWIN software, a genetic algorithm-based automated back analysis tool was conducted, as detailed in a previous paper (Vong et al, 2025). This previous study focused on soil stiffness alone and whilst yielding a relatively good fit, the later excavation induced wall movements were less than the predictions. This paper builds on the previous study comparing the previous predictions with measured movements and further examines the impact of soil permeability, structural element stiffness, and wall movements in a top-down excavation again using SAALG's DAARWIN software. The calibrated parameters and structural element stiffness values are then applied to adjacent sections to assess whether changes to the construction sequences and strutting layouts can be optimised. A number of valuable lessons learned for instrumentation in Singapore's tropical residual soil were made, which played an important role in the back analysis and are shared in the paper.

KEYWORDS: Back Analysis, Observational Method, Construction Sustainability, Risk Management

1 INTRODUCTION

Contract N107 covers the design and construction of the North South Corridor (NSC) tunnel section between Toa Payoh Rise and Marymount Lane in Singapore. The scope includes 1.37 km of dual-cell vehicular tunnels, a 431 m ramp structure, at-grade roads, and a proposed facility building at Marymount.

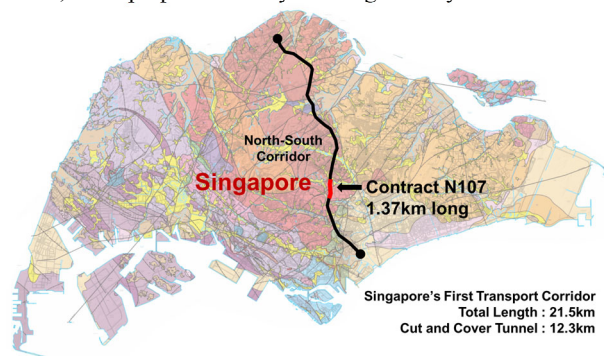


Figure 1. Map of the North South Corridor.

This paper focuses on Zone A4 and A3 of the project. Zone A4 comprises a 123m section constructed using the top-down excavation method, with an excavation depth of 20.2m. The excavation is supported by two layers of struts and 1.2m

thick diaphragm walls and is completed with roof and base slabs. Zone A3 consists of a 12m section constructed using the top-down excavation method, with a depth of 20m. It is supported by three layers of struts and 1.2m thick diaphragm walls. The geological profile comprises residual soil (GVI) and G(V) from Bukit Timah granite, with the rock head located 35 m below FEL. The NSC tunnel formation lies within the G(VI) and G(V) layers, where SPT N-values range from 20 to 40. The design strategies and methods based on the characteristic approach were first introduced in a paper by Thi et al. (2023).



Figure 2. Aerial photograph of Zone A3 and Zone A4.

Singapore has not historically adopted the ab initio (from the start) approach to the observational method. Recently an ab initio framework has been proposed by Singapore's Building and Construction Authority (BCA). However, at present this has not been commonly adopted as discussed in O'Brien et al. (2026). Instead, the observational method was applied in real time as a practical way to manage risk, enabling informed decisions, promoting more sustainable sequencing, and improving the overall programme while reducing safety risks through favourable site conditions and monitoring results. This application does not contradict BCA's regulatory framework and remains fully compliant within the context of prevailing statutory requirements. This paper builds on a previous study where the ipso tempore method was used to proactively assess the need for a contingency strut before final excavation, based upon instrumentation data available up to the excavation to the roof slab level (Vong, et al, 2025). Expanding on that approach, the current study evaluates the effectiveness of forward prediction using partially completed works and compares it with actual outcomes. The objective is to demonstrate how real-time data interpretation through the observational method and forward prediction can reduce safety risks, optimise sequencing, and enhance programme efficiency. In this case, back-analysed parameters at Zone A4 were used to justify omitting a strut layer in an adjacent section Zone A3. The findings aim to guide future projects with similar ground conditions, promoting adaptive, data-driven risk management and sustainable construction practices.

2 BACKGROUND

A previous study was undertaken by Vong et al. (2025), which undertook a back analysis of the soil stiffness using SAALG's DAARWIN software (Martinez, 2024), a genetic algorithm-based automated back analysis tool. The back analysis incorporated the review of the as-built geometry, characteristic soil profile, ground water and ground stiffness, supported by the use of instrumentation data from inclinometers, piezometers and water standpipes. The results verified that the characteristic soil approach was robust and appropriate up to the roof slab excavation level.

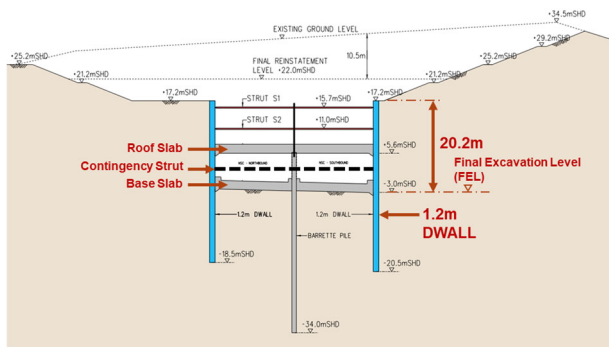


Figure 3. Zone A4 Cross Section

Using calibrated parameters from the back-analysis to roof slab level, forward predictions of wall deflections and pore water pressures were made for the final excavation stage. A contingency strut was initially considered, as the characteristic approach had not been validated in Singapore. However, after careful review of instrumentation and forward prediction, the strut was deemed unnecessary, and excavation proceeded to the final excavation level (FEL) with continued monitoring. Although the characteristic approach has not been widely applied to cut-and-cover tunnels in Singapore, its successful implementation on this project has led to its adoption in several other projects within the organization.

Although optimized parameters from the DAARWIN back-analysis provided a much better match to the data when compared with the design model, there were obvious limitations to the approach which were acknowledged. Only a best fit match to the data is feasible due to the numerous variables such as actual ground layers, soil permeability, wall stiffness etc. Additional factors such as diaphragm wall and slab stiffness, and soil strength were not studied in the previous paper, as they were beyond its scope.

3 FORWARD PREDICTION VERIFICATION

The forward prediction (FP) of wall movements to the FEL based upon the previous back analysis (BA) is compared with the on-site measured results in Figure 4 and Figure 5; alongside the prediction of the pore water pressures in Figure 6.

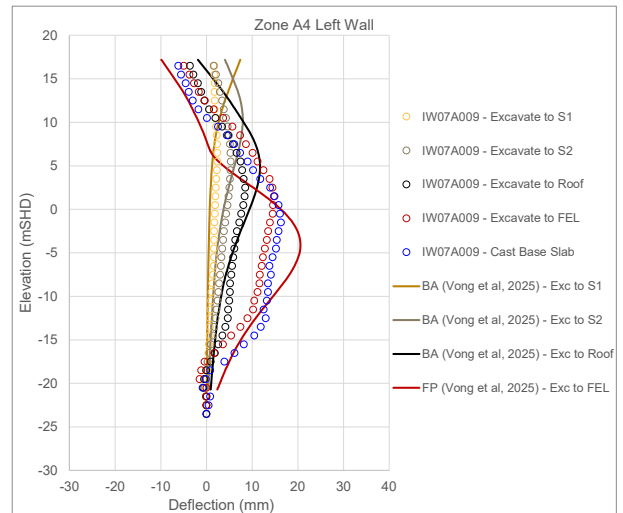


Figure 4. Left wall deflection comparison of forward prediction (Vong et al, 2025) and actual (In-wall inclinometer IW07A009).

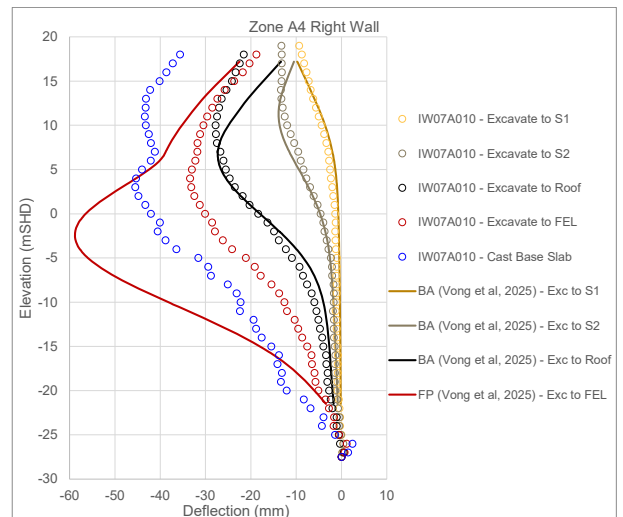


Figure 5. Right wall deflection comparison of forward prediction (Vong et al, 2025) and actual (In-wall inclinometer IW07A010).

For the right wall forward prediction, although the initial stages correlate well with the observed movements, it exhibits a noticeably softer response as it progresses to FEL. Specifically, increased wall curvature is predicted below the roof slab at +5.6m SHD, with predicted deflections approximately twice the magnitude of those recorded on site. This discrepancy suggests that the forward prediction may be underestimating the deeper ground strength and stiffness,

alongside other influencing factors, such as groundwater or structural stiffness.

It should be noted that Figure 5 shows continued wall movement following blinding of the formation, attributed to rising pore water pressure (as shown in Figure 6), resulting in increased active loading and reducing passive resistance. This highlights the importance of, continued passive water pressure monitoring after the formation is sealed (blinding and/or slab cast) to understand and monitor this behaviour.

As found in the preceding study, groundwater conditions have a key influence on the performance of the Earth Retaining System (ERSS). As such, the initial step in this back-analysis involves comparing the groundwater levels predicted in the forward analysis with the actual site monitoring data.

3.1 Groundwater

Figure 6 illustrates that the observed groundwater drawdown exceeded the levels predicted in the previous forward prediction. To address this discrepancy, an initial calibration introducing recharge boundaries on the left and right sides of the PLAXIS 2D model was attempted. However, extensive on-site drawdown was seen due the complexity of the soil permeability, topography, flow, and recharge (precipitation and groundwater), as such, this approach could not yield a good match.

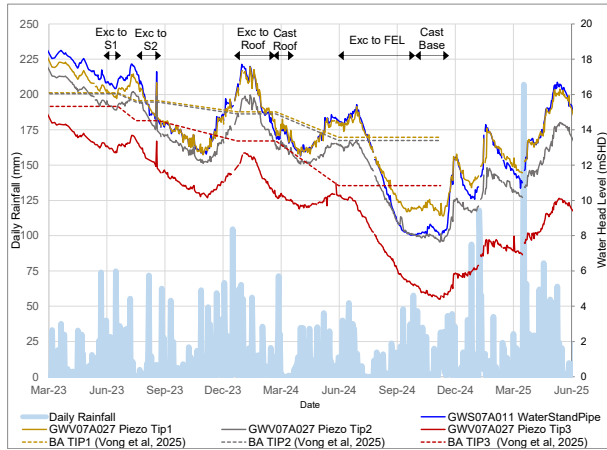


Figure 6. Comparison of piezometer reading, back-analysis water head level and daily rainfall.

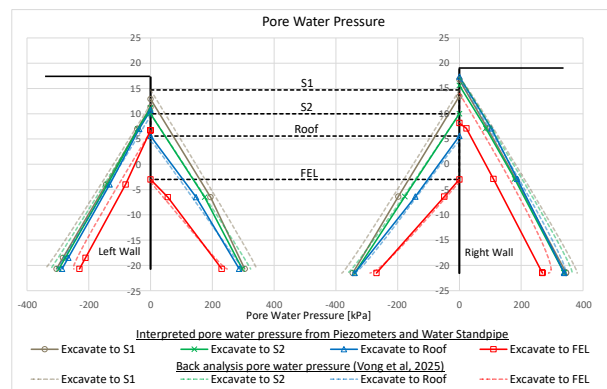


Figure 7. Comparison of interpreted piezometer readings and back-analysis water pressure.

Accordingly, pore water pressure lines (PWPL) as shown in Figure 7 were manually applied to the soil clusters to directly represent the groundwater conditions observed on site. Although the calibration allowed for a significantly improved fit for the walls as shown in Figure 8 and Figure 9, additional parameters were subsequently assessed which were not considered in the previous analysis including:

- The flexural stiffness (EI) of the diaphragm wall (lumped factor and moment-curvature).
- The characteristic concrete stiffness (E_{cm}) of both the diaphragm walls and the roof slab.
- The structural modelling approach of the slabs and diaphragm walls.
- The interrelation between strength and stiffness in the chosen soil model.

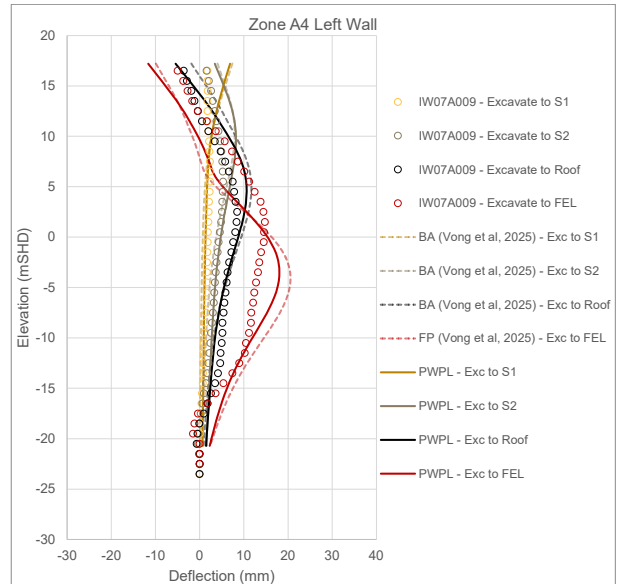


Figure 8. Comparison of predicted wall deflection with the updated pore water pressure

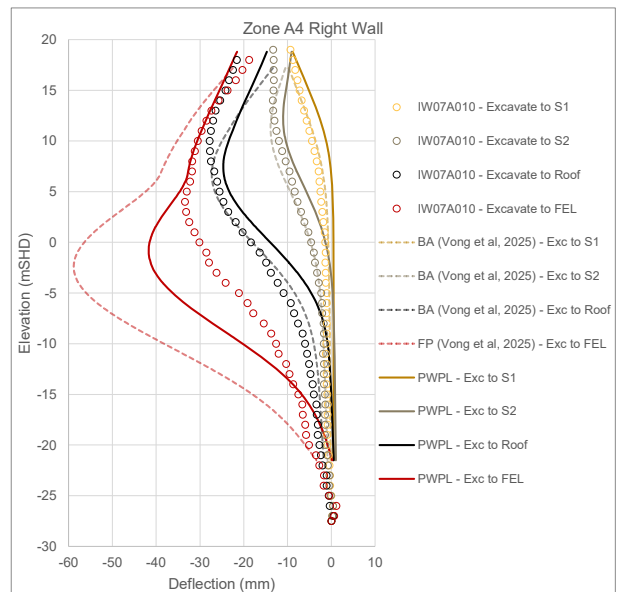


Figure 9. Comparison of predicted wall deflection with the updated pore water pressure

3.2 The flexural stiffness (EI) of the diaphragm wall

A sensitivity analysis was carried out by varying the EI of the diaphragm wall at increments of 50%, 70%, 100% and 120% of the concrete section (i.e. minimum feasible, long term, to the maximum feasible, uncracked with steel contribution). The corresponding EI values adopted in the sensitivity study are tabulated in Table 1.

Table 1. Flexural stiffness (EI) for sensitivity analysis.

Sensitivity Increments	Flexural stiffness EI (kNm ² /m)
50% EI	2400840
70% EI (Design)	3361176

It can be seen from Figure 10, that variation in the EI changes the amplitude of the deflection profile, however, does not have a significant impact on the overall shape.

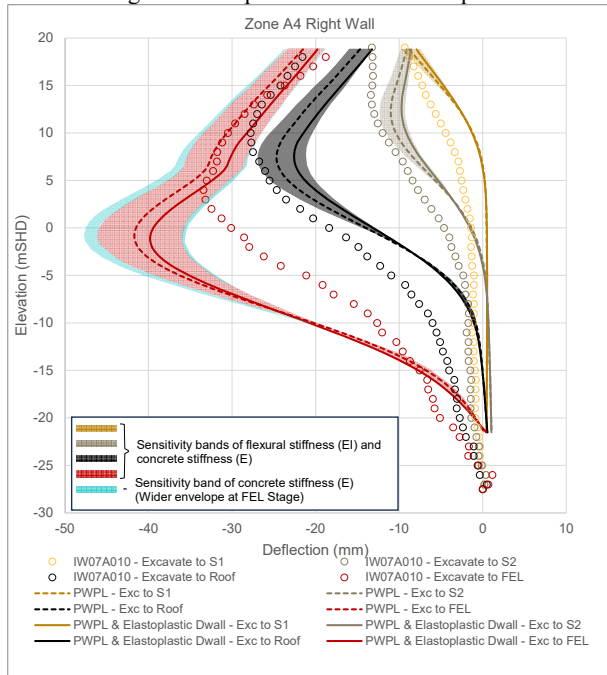


Figure 10. Wall deflections from the sensitivity analyses.

3.3 The characteristic concrete stiffness of both the diaphragm walls and the roof slab

The cube test result was reviewed as plotted in Figure 11. As observed, there is some variation in the concrete cube strength between 38 MPa to 70 MPa. The corresponding stiffness can vary up to 12%, with this. Due to sensitivity study of EI in Section 3.2 showing limited impact, greater variation of concrete stiffness of the diaphragm wall and roof slab (-50% to +20% of C32/40 E stiffness) due to the cracking behaviour were reviewed. The result suggests that concrete stiffness is not a dominant parameter in improving the fit of the diaphragm wall in this context.

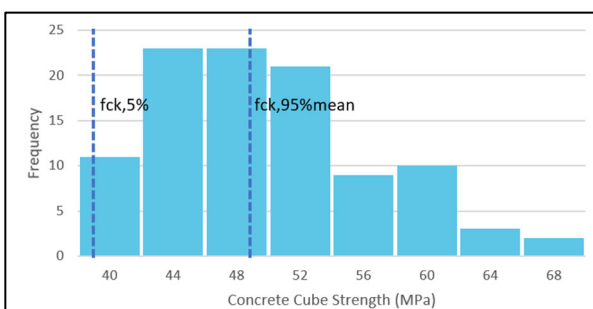


Figure 11. Test results of the concrete cube strength.

3.4 The structural modelling approach of the slabs and diaphragm walls.

The diaphragm wall and roof slab were modelled using the plate element in Plaxis 2D where the actual wall or slab thickness is not modelled volumetrically. This may not be suitable in capturing the complex geometry of the 2m thick roof slab with 0.6m thick haunch and the interaction with the diaphragm wall.

To better capture this, both the slab and D-wall were modelled as volume element with concrete-equivalent stiffness

and density, allowing for more realistic representation of the flexural stiffness and interaction at the slab-wall interface.

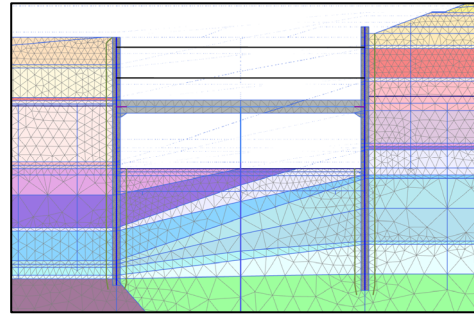


Figure 12. Idealised Plaxis 2D model with volume elements.

As shown in Figure 13, the curvature ("belly") beneath the slab is reduced, resulting in a better fit with observed behaviour. This modelling approach also yields a more realistic stress distribution and bending stiffness at the slab-D-wall interface.

Though the soil cluster approach provides a better fit as a back analysis, the design intent was for a structurally robust solution suitable for potential progressive modification (Powderham & O'Brien, 2020).

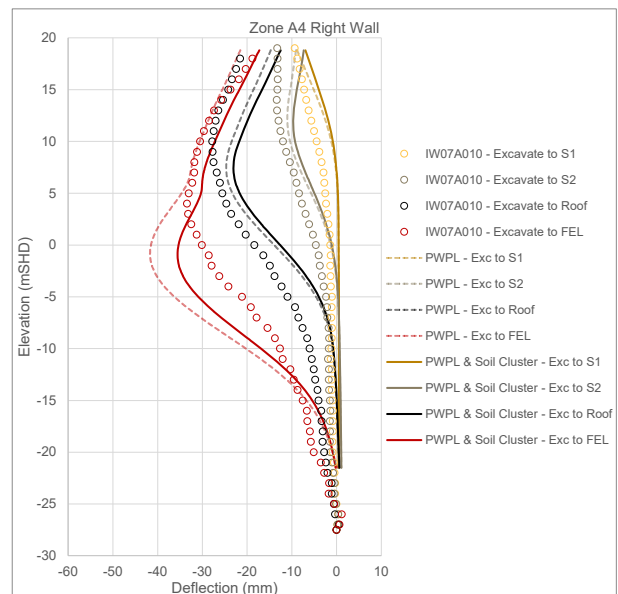


Figure 13. Wall deflections with soil clusters modelling approach.

3.5 The interrelation between strength and stiffness in the chosen soil model.

A full back analysis of the overall construction sequence was undertaken using DAARWIN to compare the derived soil strength and stiffness parameters, with the following settings as the baseline.

- Groundwater (Observed groundwater pressure)
- Diaphragm wall flexural stiffness (70% EI)
- Concrete stiffness (E_{cm} of C32/40)
- Modelling approach for the diaphragm wall and the roof slab using soil cluster

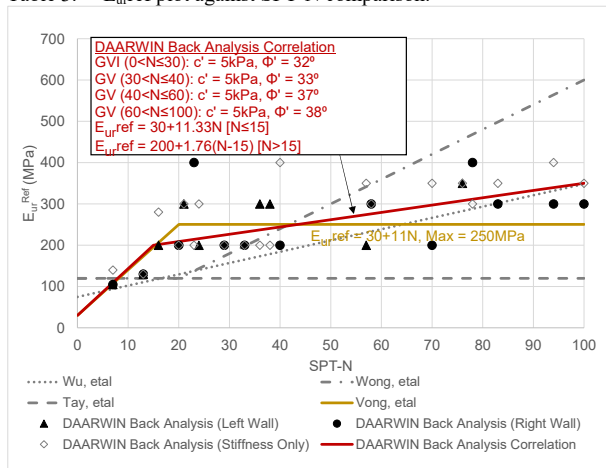
A full back analysis was undertaken using the previous best estimate as a baseline. The parameter $E_{ur,ref}$ was varied in the back analysis for each layer independently. The parameter m was fixed to 0.8 in line with previous studies and $E_{50,ref}$ and $E_{oed,ref}$ sensibly taken as $1/3 E_{ur,ref}$ as there were no settlement concerns in this case.

Table 2. Soil stiffness properties.

Bukit Timah Granite G(VI), G(V)	$E_{ur,ref}$ (MPa)	m
Tay et al (2021)	120	0.8
Wu (2024)	75+2.73N	1
Wong (2020)	120 + 6N for N>20 120 for N≤20	0.8
Vong et al (2025)	30+11N, Max 250MPa	0.8

Although the result showed a good fit, a further back analysis was carried out by including the soil strength as a variable in DAARWIN to assess if further improvement could be made.

Table 3. $E_{ur,ref}$ plot against SPT-N comparison.



The $E_{ur,ref}$ values in Table 2 were compared with the back analysed $E_{ur,ref}$ from DAARWIN for each soil layer. In the preceding study, the back analysis was undertaken up to the roof slab level, in which showed a linear increase in stiffness and capped at 250MPa with SPT-N beyond 20. The updated back analysis up to the FEL, showed non-linear behaviour where greater soil strain is observed. The $E_{ur,ref}$ continued to increase linearly with SPT-N values beyond 15.

As can be seen on Figure 14 and Figure 15, a significant improvement to the shape of the deflection profiles was obtained from the back analysis.

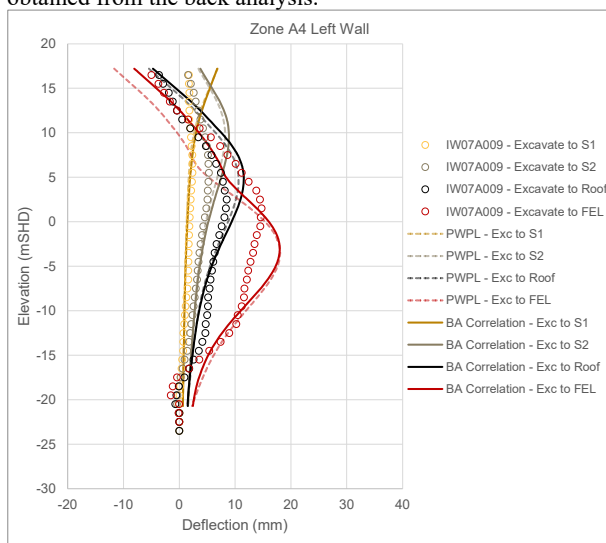


Figure 14. Zone A4 left wall deflection comparison of back-analysis correlation vs actual movement.

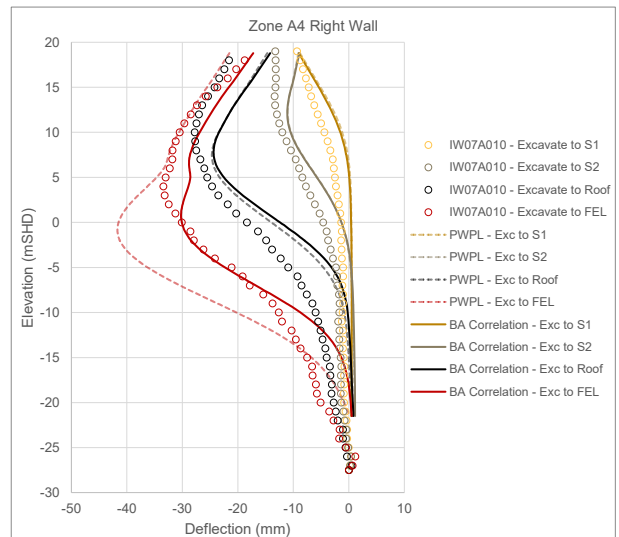


Figure 15. Zone A4 right wall deflection comparison of back-analysis correlation vs actual movement.

4 CLASS B PREDICTION OF A3 USING CLASS C OF A4 FINDINGS

Lessons learnt from the preceding study supported the approval of Zone A3, where one layer of struts was successfully omitted based on groundwater being the dominant factor. The full back analysis undertaken in this study predicted significantly smaller wall deflections, which closely matched the observed moments further validating the approach and increasing confidence in the data-driven design decisions.

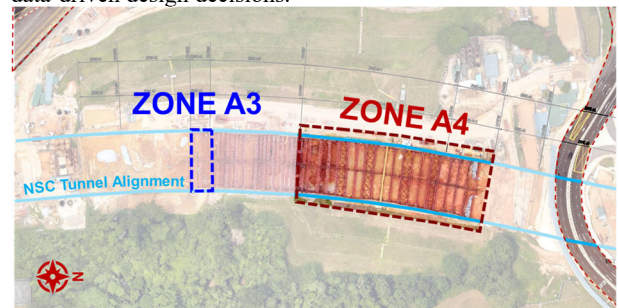


Figure 16. Aerial view of Zone A3 and Zone A4.

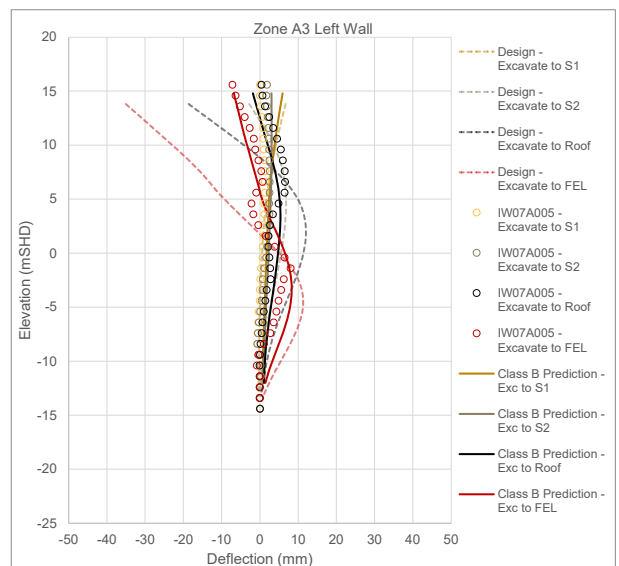


Figure 17. Wall deflection comparison of Zone A3 Class B prediction vs actual movement (In-wall inclinometer IW07A005).

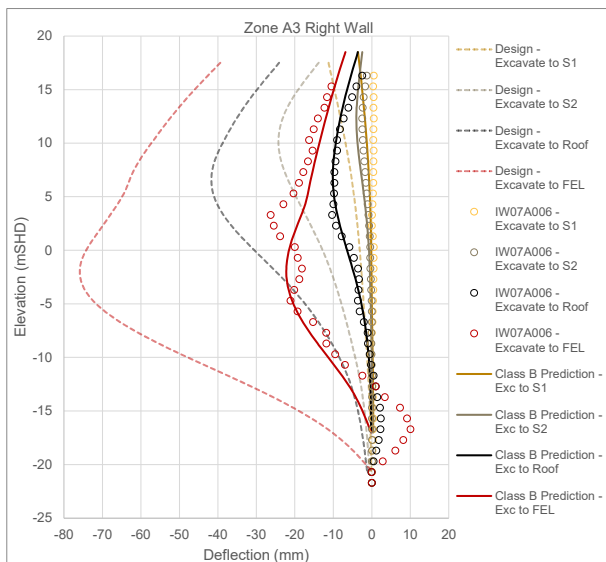


Figure 18. Wall deflection comparison of Zone A3 Class B prediction vs actual movement (In-wall inclinometer IW07A006).

As shown in Figure 17 and Figure 18, the back-analysed prediction aligns well with the actual wall movement. Notably, both the predicted and observed movements remained well within design limits.

5 LESSONS LEARNT AND POTENTIAL APPLICATIONS

Through this study, there several key observations and takeaways which should be highlighted for future design and OM applications, particularly in granitic residual soils.

1. Reliability of instrumentation data and the duration of the monitoring. In this case, the client was supportive to keep the monitoring after lean concrete casting for the roof slab. With that, we were able to assess the impact of lean concrete casting on the ERSS as described under Section 3.
2. Groundwater drawdown during excavation was the primary driver of the back analysis. This was identified in preceding paper and further validated in this study, showing a much better fit after calibrating groundwater pressure to actual site conditions.
3. The modelling of structural members as soil clusters given their thickness, significantly improved the shape of the curve, particularly at and below the slab levels.
4. The preceding back analysis, which was limited to the roof slab excavation stage, indicated a relatively softer soil response. This is likely due to the lower levels of induced strain and limited deformation within the soil, as the excavation had not yet mobilised the deeper strata. At that stage, groundwater pressure was the primary driver of ground movement.
5. As noted in Point 2, groundwater pressure is the main factor influencing ERSS performance. However, with deeper excavation to FEL, increased soil strain and strength mobilisation at lower depths enabled more accurate calibration of soil parameters, resulting in a better model fit.

6 CONCLUSION

This study reinforces the value of the observational method in deep excavation works, particularly under complex geotechnical conditions. In Zone A4, one contingency strut was safely omitted following a validated Class 3 prediction, supported by sensitivity and back-analysis using the AI tool DAARWIN. Further, Zone A3 with the lessons learnt at Zone A4, achieved a Class 2 prediction, enabling the removal of a planned strut while maintaining structural integrity, again with DAARWIN.

These design optimizations were made possible through real-time instrumentation data, demonstrating how actual site observations can inform adaptive decision-making, enhance safety, manage risks and improve construction efficiency.

While it is acknowledged that fully capturing all ground and structural behaviours within numerical models is not feasible and a perfect back-analysis fit is beyond the scope of this study the calibrated parameters and lessons learned obtained remain valuable. They provide a practical reference for future projects applying the observational method in similar ground conditions, highlighting its effectiveness in managing uncertainty and refining design through data-driven engineering.

7 REFERENCES

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