Sustainable Underground Infrastructure Design Using Innovative Artificial Intelligence (AI) Back Analysis

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ABSTRACT

Singapore's first integrated transport corridor, the North South Corridor, is designed to alleviate traffic, enhance connectivity, and foster a cleaner, greener land transport system, thereby improving the quality of life for future generations. A characteristic approach to the geological modelling was adopted in the design of an 123m section of the project as presented in a previous paper (Thi et al, 2023). Given the novel design approach, a potential mitigation for unanticipated ground movement in the form of a contingency strut was considered. Excavation of this section has been partially completed. The resultant ground movements recorded through the instrumentation and monitoring systems have validated the approach up to the current excavated roof slab level.

Numerous studies have been conducted on the performance of Earth Retaining Stabilising Structures (ERSS) in Singapore through back analysis. A digital automated AI back analysis tool was used to calibrate non-linear soil stiffness parameters to the recorded site movements from the instrumentation and monitoring systems in this paper. The calibrated parameters allowed a forward prediction of the anticipated movements to the completion of the site work, which indicates that a contingency strut is unlikely to be necessary.

The characteristic approach undertaken has provided programme and cost savings in the design, while also achieving carbon savings quantified using a digital carbon calculator, which aligns with Singapore's sustainability goals. The paper underscores the importance of instrumentation and monitoring, and a useful checklist is included with examples to provide context should the performance of an ERSS system not match expectations. This is aimed at young engineers in particular as they learn about the implementation, and interpretation, of instrumentation and monitoring.

Keywords: Characteristic Approach, Sustainability, Instrumentation Monitoring and Interpretation, Varying Soil Stiffness, Verification AI Back Analysis, Forward Prediction

1. INTRODUCTION

The North South Corridor (NSC) is 21.5km and Singapore's first integrated transport corridor.

Figure 1 North South Corridor Positive Outcomes (Diagram produced from Land Transport Authority North South Corridor redefining journeys video)

Contract N107 pertains to the design and construction of the NSC tunnel section from Toa Payoh Rise to Marymount Lane. The scope of the contract comprises the creation of 1.37km of dual-cell vehicular tunnel structures, a ramp structure measuring 431m in length, at-grade road, and a proposed facility building at Marymount.

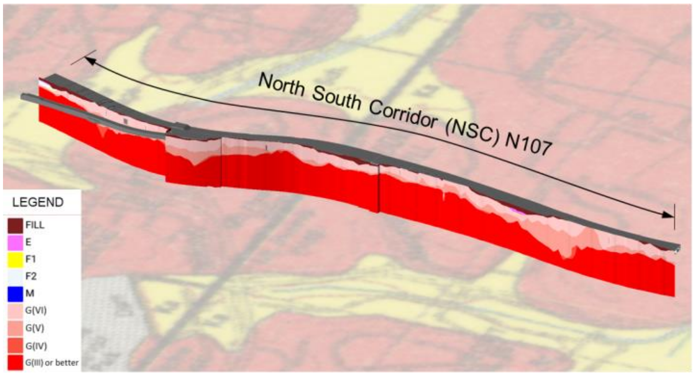


Figure 2 3D Geological Model along North South Corridor N107

The Ground

Three main strata of soil and rock define the proposed alignment of the NSC tunnel. The Fill material forms the top layer. The middle layer is made up of residual G(VI) and fully weathered G(V) soils from the Bukit Timah Granite Formation. The bottom layer comprises granitic rock, also a part of the Bukit Timah Granite Formation. The formation level of the NSC tunnel lies within the G(VI) or G(V) layers, exhibiting an SPT-N value between 20 and 40. The top of the rockhead vary from 15m to 40m below the formation level (Geotechnical Interpretative Base GIBR, 2017).

The design strategies and methods relating to the characteristic approach were first presented in a paper Thi et al (2023). A summarised table of this work can be found in Appendix B. This paper is a continuation of the previous work, focusing on the verification of the approach through careful back analysis of the instrumentation and monitoring data using automated back analysis with non-linear soil modelling.

The final design adopted a combined solution, lowering the final reinstatement level (thus reducing backfill) and using the characteristic profile approach to optimize both the foundation and ERSS (Refer to Figure 3).

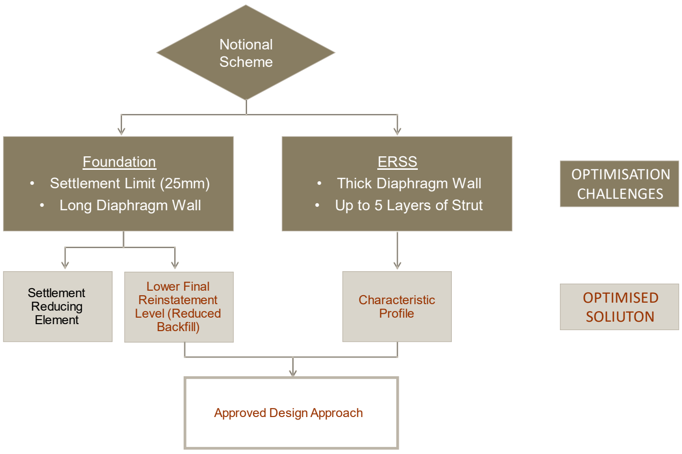


Figure 3 Design approach for the design section

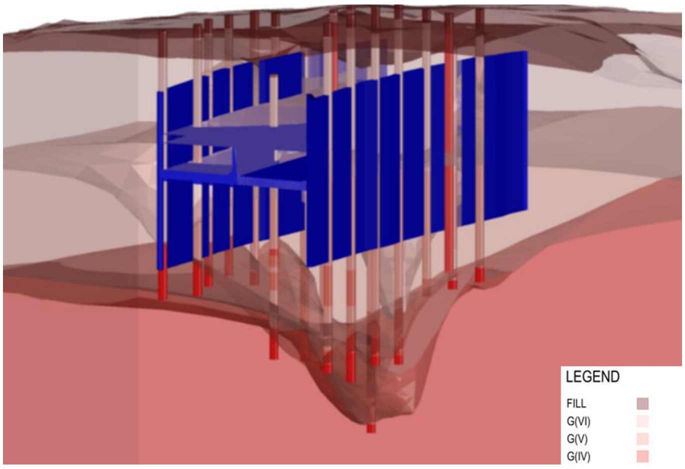


Figure 4 3D Geological model of the design cross-section

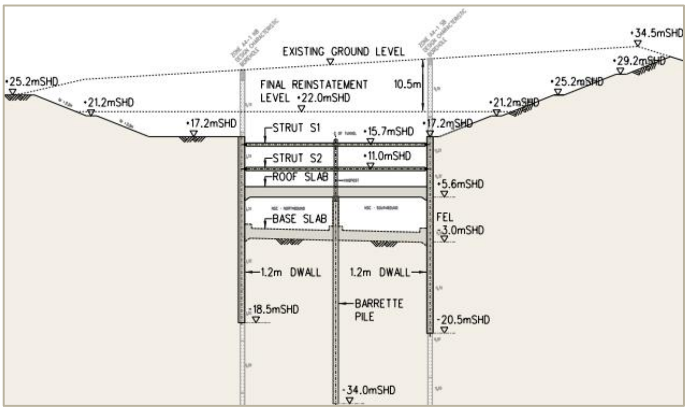


Figure 5 Cross-section for the design section

2. PROJECT CONTEXT

This project is believed to be the first of its kind in Singapore to implement and publish a paper on the characteristic profile approach, approved by the Building and Construction Authority in 2021, and now currently under construction adopting a top-down construction method. The excavation has progressed to the roof slab 11.6m depth, showing movements within anticipated design bounds (Refer to Figure 7). The next phase involves excavation to the final base slab level and offers a chance to review the design for safety and efficiency through the back-analysis, and review whether a contingency measure may be required, such as an additional strut.

This paper focuses on the verification and back-analysis using the automated digital AI tool DAARWIN and Carbon Moata for automation of the carbon savings' calculation.

The paper also focuses on the importance of reliable instrumentation data and shares best practices for its planning. The paper aims to provide guidance on key review steps when actual instrumentation readings deviate from their predictions.

3. SUSTAINABLITY – VALUE ENGINEERING

The scheme's optimisation, using the characteristic profile, is driven by value engineering considerations such as programme efficiency and cost savings. Specifically, the design scheme eliminated the need for the additional layers of strutting and a 1.5m thick diaphragm wall. This has enhanced the design's sustainability by reducing its carbon footprint, a result of evaluating the carbon savings generated. Figure 6 provides an illustration of the various sustainability-related policies, the social and sustainable outcomes targeted by the North-South Corridor demonstrating how the entire cycle of engineering tasks is integrated to these objectives. It is worth noting that the notional scheme was developed in 2017, before Singapore's commitment to the Green Plan in 2021 which has enhanced the country's existing commitment to sustainable infrastructure. Indeed, in alignment with the UN Sustainable Development Goals (SDGs), Singapore has made significant strides through the Green Plan, aiming for net-zero emissions by 2050. These principles have been applied to the design characteristic profile, a robust design innovation that reduces the thickness and length of the ERSS which substantially lowered the carbon emissions (634,319kgCO_2eq) associated with the optimised design (a 20% decrease in embodied carbon).

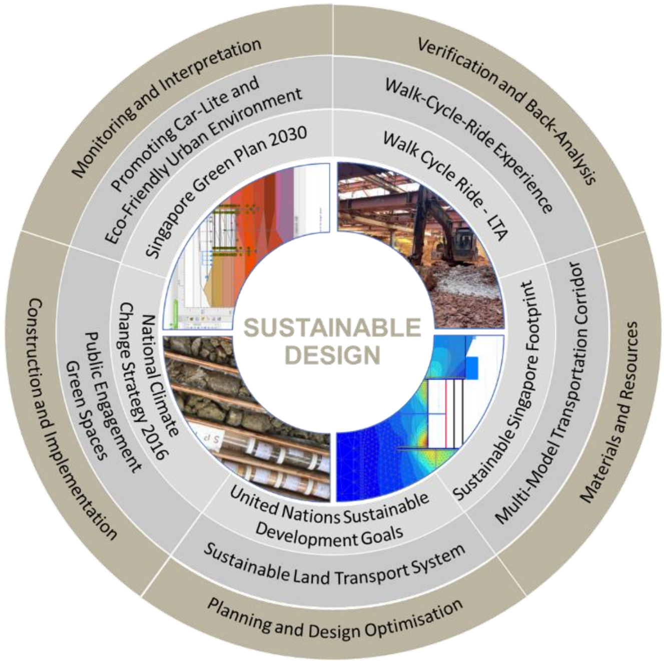


Figure 6 Sustainability in geotechnical engineering framework for this project (Diagram produced from SDGs, LTA North South Corridor Redefining Journeys video, Singapore Green Plan 2030)

Carbon emissions can be quantified through various tools, in this paper an in-house tool was used, the Moata Carbon Portal (MCP). This intuitive digital solution calculates infrastructure's carbon emissions. Its primary goal is to democratize decarbonization, empowering all infrastructure project professionals to confidently reduce carbon in their projects, regardless of their prior experience with carbon.

The design's use of the MCP aligns with SDG Goal 9, which promotes the development of resilient infrastructure, encourages inclusive and sustainable industrialization, and fosters innovation.

4. ERSS PERFORMANCE

Three excavation phases, (Excavation to Strut S1, Strut S2 and Roof Slab, as shown in Figure 8) have been completed to a depth of 11.6m. The next construction phase on site is the excavation to the final excavation level with total depth of 20.2m. Based upon the maximum inclinometer readings, the diaphragm wall deflections are approximately 33% and 90% of the predicted values at the left and right wall, respectively. Figure 7 compares the measured wall deflection for each excavation stage against the design prediction. The current excavation data provides an opportunity to reassess the design prior to the final excavation, to assess the likelihood for the need of a contingency measure such as an additional strut. This is assessed as part of the back analysis verification in the next section of this paper.

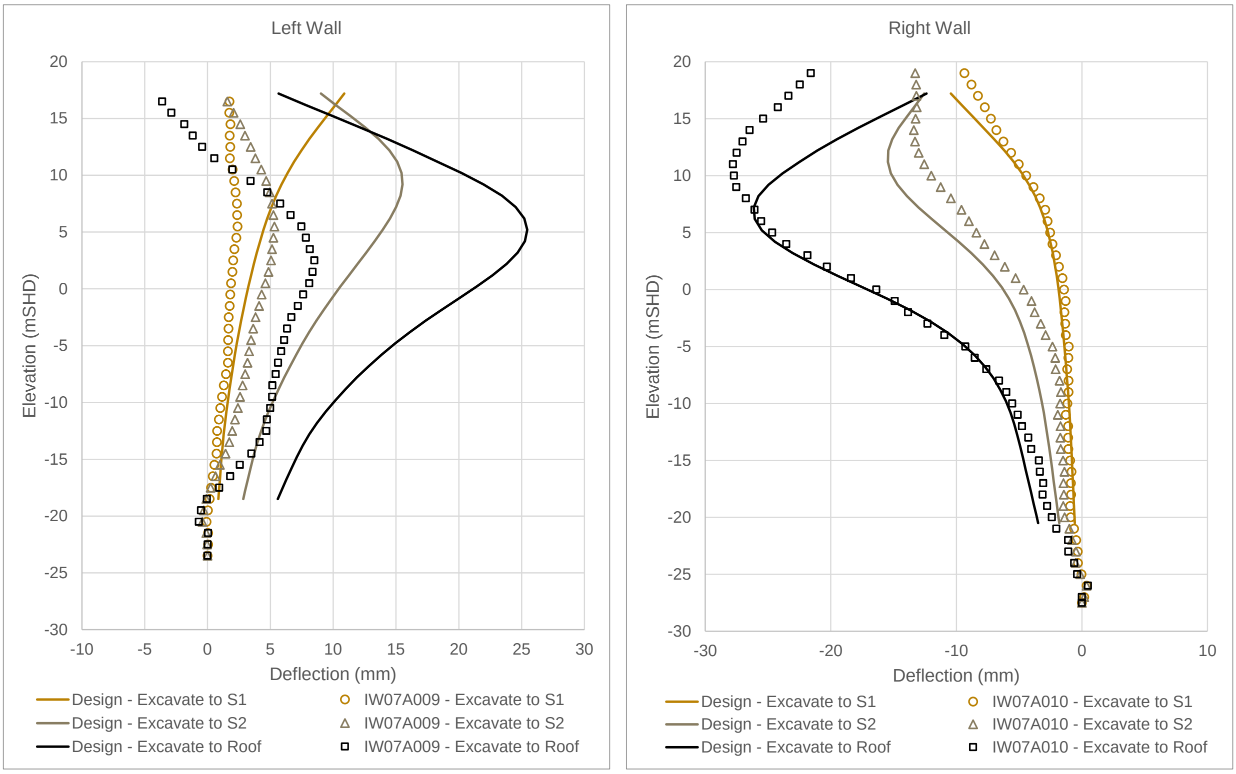


Figure 7 Wall deflection comparison of predicted (design) and actual (IW07A010) movement

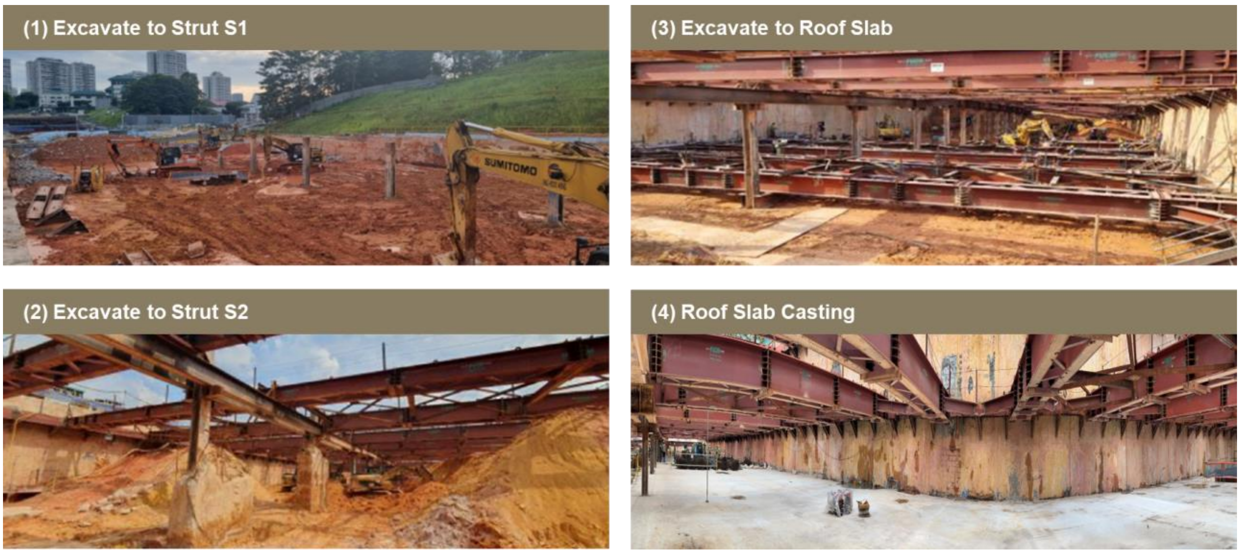


Figure 8 Site photos of the various excavation and roof slab casting stage

5. IMPORTANCE OF INSTRUMENTATION AND MONITORING

The success of back-analysis is fundamentally dependent on the data derived from instrumentation and monitoring. Given the inherent variability of the ground and the limitations of ground investigation procedures in identifying all significant properties and conditions of natural materials at every site position, designers are required to make assumptions that do not fully represent the actual conditions. Instrumentation and monitoring, along with data analysis, ensure project safety as shown in Figure 9. They verify that construction aligns with design specifications, provide early failure warnings, and enable preemptive remedial measures. This approach prioritizes safety and cost-effectiveness in execution.

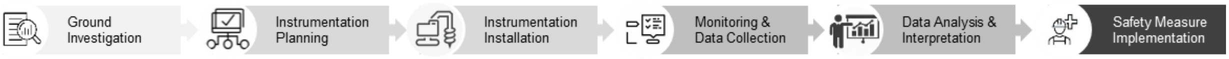


Figure 9 Full cycle of site investigation, design (instrument planning), construction, monitoring for the safety of the work

6. BACK-ANALYSIS VERIFICATION WITH DAARWIN – an automated AI digital tool

The performance of the characteristic profile design is assessed using an automated AI digital tool, DAARWIN. Figure 10 provides a summary of DAARWIN's functionality and purpose; its ability to efficiently run iterations of finite element models (Plaxis) varying multiple parameters automatically, using a genetic algorithm to minimize runs, to match the measured performance.

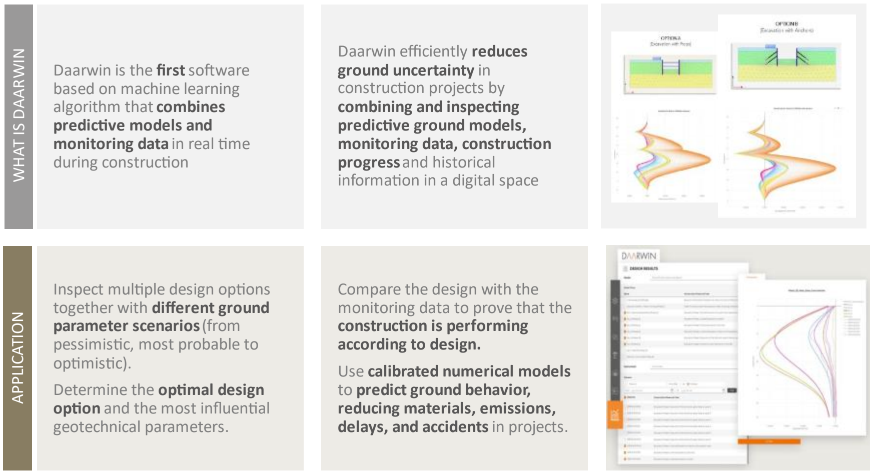


Figure 10 Summary of the AI digital tool DAARWIN

The flow chart in Figure 11 provides a summary of the methodology for the AI back analysis in this paper.

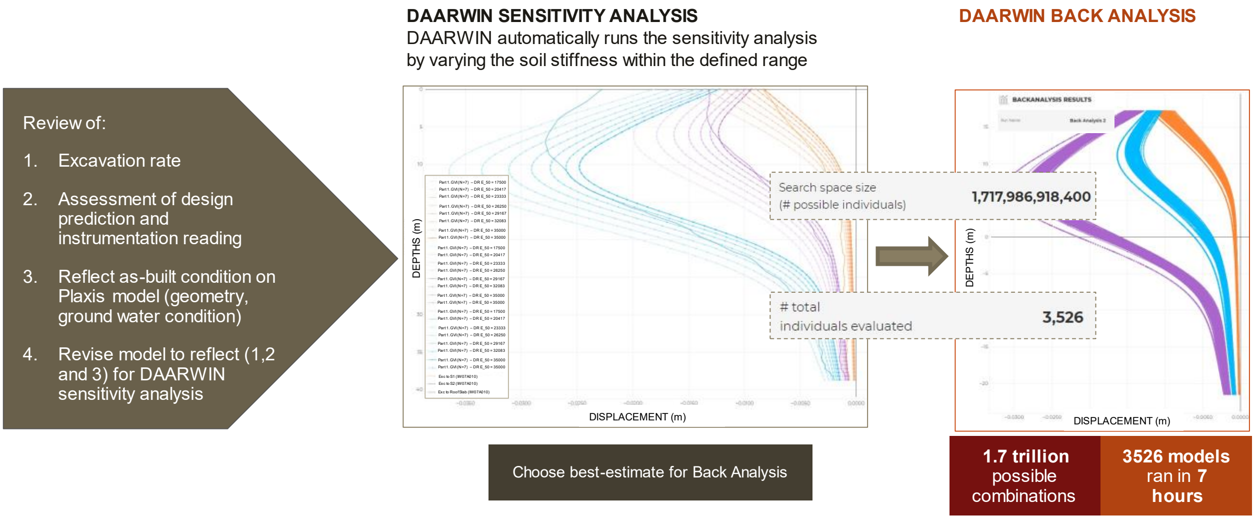


Figure 11 Summary of back analysis methodology

6.1. EXCAVATION RATE

The construction timing to the casting of roof slab is given in Table 1. Alongside the excavation rate as defined in Equation 1 as detailed in Tay et al (2021). It is noted that the DAARWIN back analysis tool has the capability to analyse both permeability and time dependency. However, considering the typical field permeability of Bukit Timah Granite (1E-7m/s) the ER/k values would indicate that the material is fully drained as shown in Figure 12. As such, the back analysis will focus solely on conditions that are fully drained, excluding those that are fully undrained or partially drained.

Equation 1 Excavation rate

$$Excavation\ Rate = \frac{Excavation\ Depth}{Excavation\ Time}$$

Table 1 Excavation rate

	Description	Excavation/ Slab Casting Days	Excavation Depth (m)	Excavation Rate (m/s)	ER/k
1	Excavate to Strut S1	43	2.5	6.73E-7	6.73
2	Install Strut S1	36	-	-	
3	Excavate to Strut S2	67	4.7	8.12E-7	8.12
4	Install Strut S2	44	-	-	
5	Excavate to Roof Slab	110	5	5.26E-7	5.26
6	Cast Roof Slab	47	-	-	

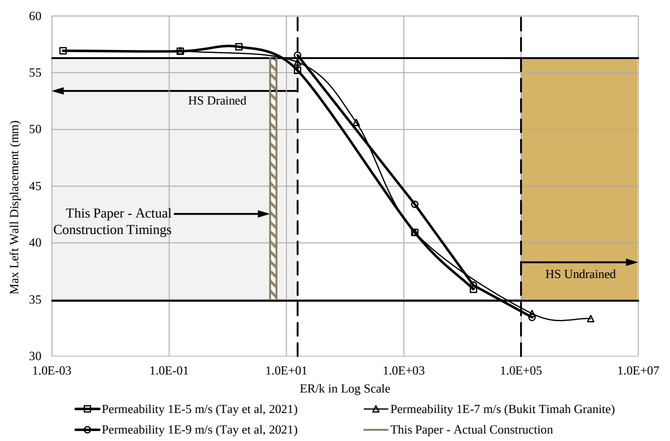


Figure 12 Wall Displacement for Soil Permeability with Actual Construction Timings – Figure Reproduced after Tay et al (2021)

6.2. BACK-ANALYSIS

The following were reviewed as part of the back analysis,

- As Built Geometry
- Characteristic Profile
- Groundwater
- Soil Stiffness

Each are discussed in further detail below.

A. As Built Geometry

The Plaxis model was revised to match the as-built geometry at the location of the inclinometers. The diaphragm wall, barrette pile, and rock-head level were revised to match the as-built information.

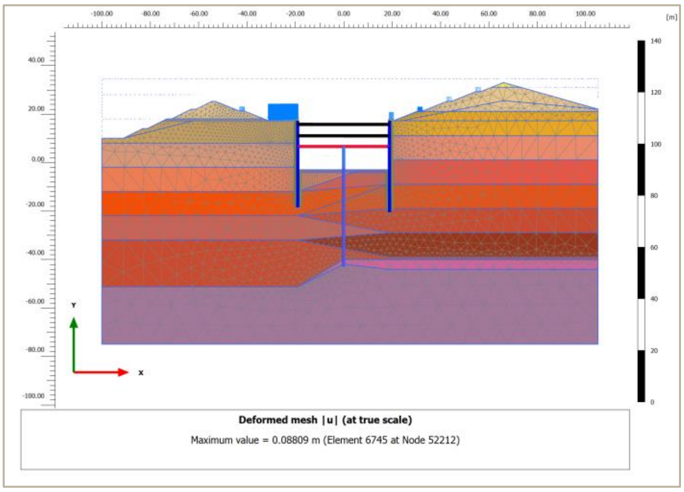


Figure 13 Plaxis output for approved design

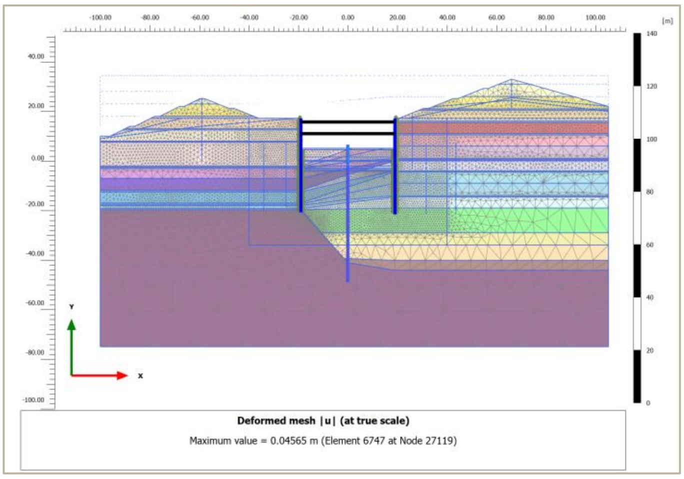


Figure 14 Plaxis output for back analysis

B. Characteristic Profile

As discussed in the previous paper, a characteristic profile was implemented with an interval of 10m for design, whilst the back analysis adopts a 5m interval (Figure 15 and 16).

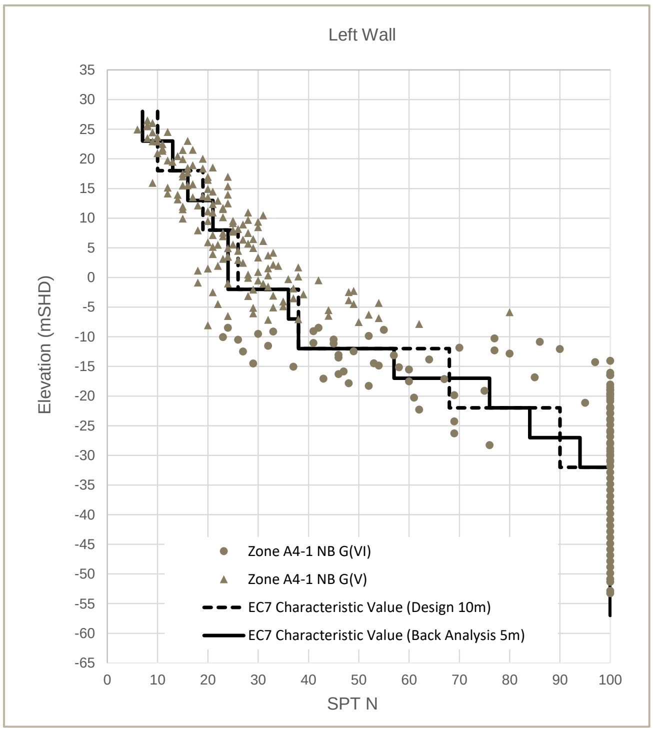


Figure 15 Left wall SPT-N scatter plots with characteristic profile for design (10m interval) and back analysis (5m interval)

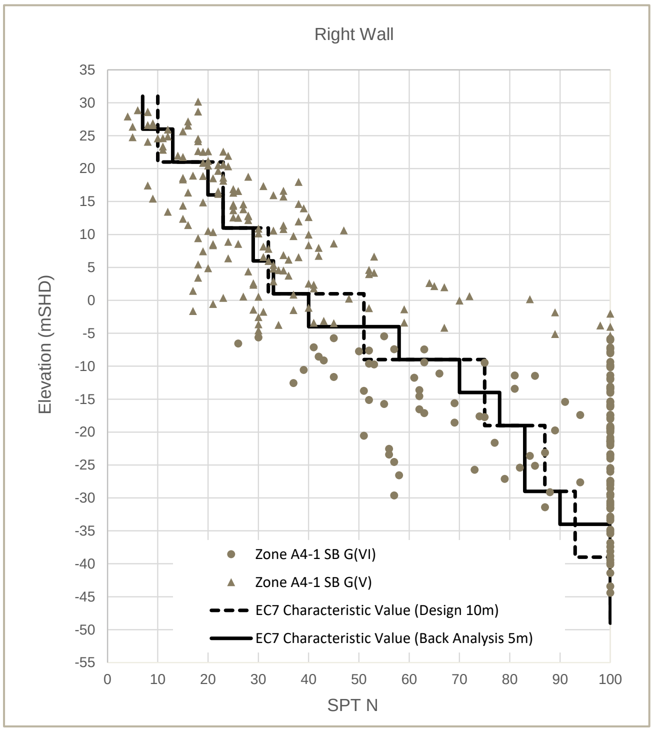


Figure 16 Right wall SPT-N scatter plots with characteristic profile for design (10m interval) and back analysis (5m interval)

C. Groundwater

Due to the variation in ground levels across the site, the measured pore water pressures from the groundwater monitoring support the flow from the higher ground on the right side to lower ground on the left. This was modelled as recharge at the left and right boundary.

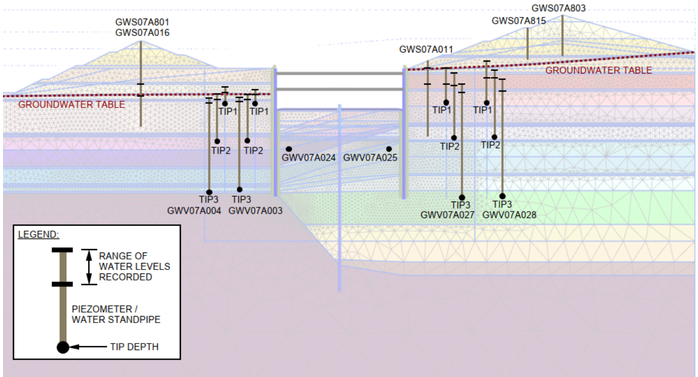


Figure 17 Groundwater condition for back-analysis

Pore water pressures, measured by the piezometer and water standpipe, correspond with daily rainfall (Figure 18). It appears that decreases in water pressure may not be solely attributed to excavation activities but could also be influenced by dry weather conditions. Significant rainfall causes a noticeable rise in pressure, which quickly recovers, indicating a drained soil response. This supports the assumption in Section 6.1 where, the soil behavior can be characterised as exhibiting fully drained behaviour.

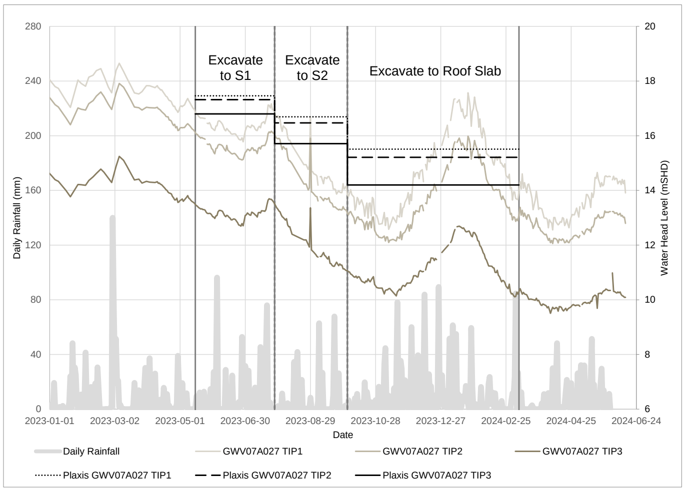


Figure 18 Comparison of piezometer reading, back-analysis water head level and daily rainfall

D. Soil Stiffness

The original design and analysis were conducted for the robust structural design of the ERSS and associated structural members (struts, walers, slabs etc.). The stiffness parameters used were based upon the Mohr Coulomb model in line with the Ground Interpretation Baseline Report (GIBR), alongside ULS groundwater levels.

The observed wall movements as shown in Figure 7 were less than those of the moderately conservative SLS design. As has been shown by numerous authors (Tay et al, 2021, Teo and Wong, 2012) the Hardening Soil is a soil model capable of capturing the non-linearity of soil and has been used in Singapore for both design and back-analysis.

A summary of Hardening Soil parameters is given in Table 2 from previous studies. These were the starting point for the back-analysis study.

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

The parameters given by Tay et al (2021), Wu (2024), and Wong (2020) did not provide a good-fit showing softer response in comparison to the actual movement as shown in Figure 19 below.

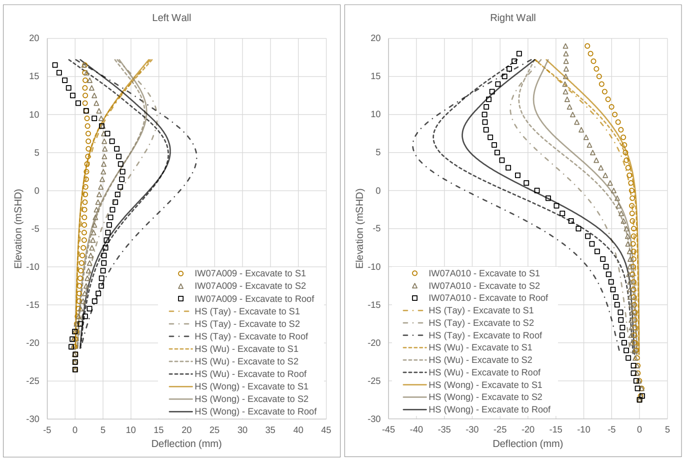


Figure 19 Comparison of predicted wall deflection using Tay et al (2021), Wu (2024), Wong (2020) and actual movement

Various sensitivity analyses were undertaken in DAARWIN prior to undertaking a full automated back analysis in DAARWIN, to obtain an optimized set of parameters for all of the soil layers. Based upon these sensitivity studies the results of the best estimate wall-deflection parameters are shown in Figure 20 below. Though the prediction is relatively good for the right wall, the fit is much poorer for the left wall.

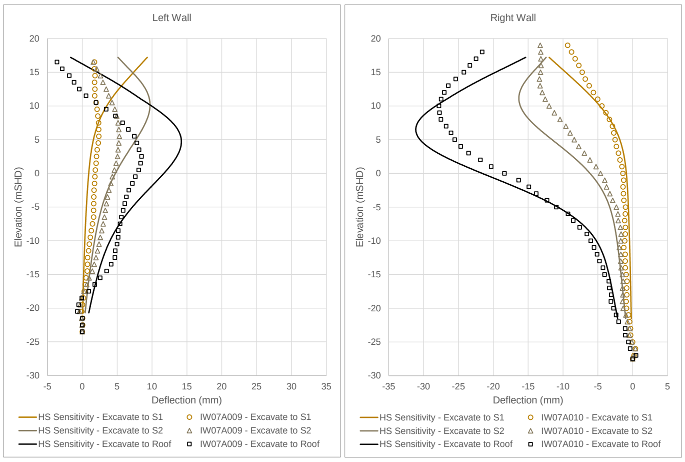


Figure 20 Wall deflection comparison of actual vs sensitivity analysis (best fit by varying soil stiffness on DAARWIN)

A full back analysis was undertaken using the previous best estimate as a baseline. The parameter $E_{ur}ref$ was varied in the analysis for each layer independently. The parameter m was fixed to 0.8 in line with previous studies and $E_{50}ref$ and $E_{oed}ref$ taken as 1/3 $E_{ur}ref$.

The $E_{ur}ref$ values in Table 2 were compared with the back analysed $E_{ur}ref$ from this project for each soil layer. It can be observed that the correlations of the SPTN and the $E_{ur}ref$ on both sides of the wall are relatively uniform. The back analysis showed that the maximum stiffness increases linearly below SPTN20, and above SPTN20 it is capped at 250MPa as shown in Figure 21. This implies that an advantage of the Hardening Soil (HS) model is its ability to represent the soil layer with SPTN ≥ 20 as a single layer, thereby eliminating the need for multiple layers to account for varying stiffness levels, as required by the Mohr Coulomb model.

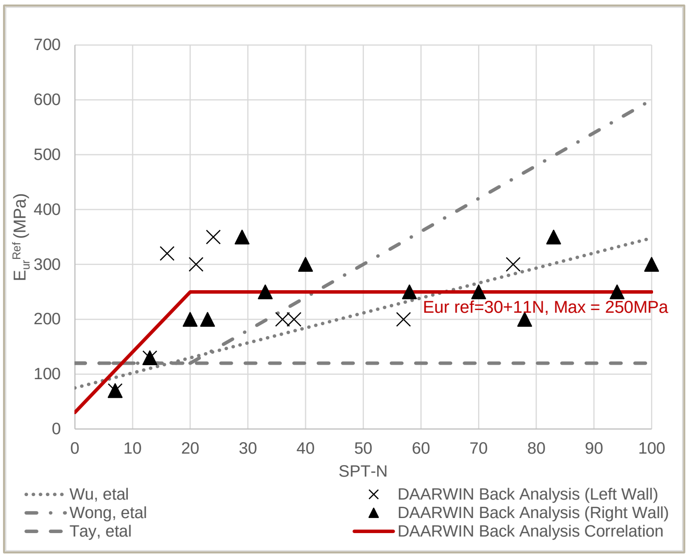


Figure 21 E_{ur}ref plot against SPT-N comparison

The comparison of the optimised stiffness from DAARWIN Back Analysis and the actual movement is compared in Figure 22 showing a good fit on both walls.

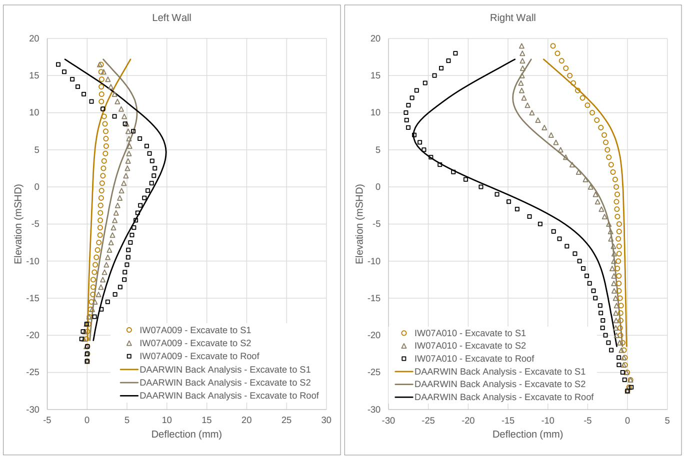


Figure 22 Wall deflection comparison of back analysis (optimised stiffness from DAARWIN back analysis) vs prediction

For Hardening Soil (HS) model, it is observed that the initial stiffness is much higher due to its strain dependency, with no strain at the start of the analysis. Between 0mSHD and -15mSHD at the Excavation to FEL stage, the HS stiffness is closer to the Mohr-Coulomb value for the right wall, a similar behavior is observed with the wall deflection. The left wall's HS soil stiffness is much higher compared to the Mohr Coulomb model which helps align the wall deflection with the actual movement monitored on-site.

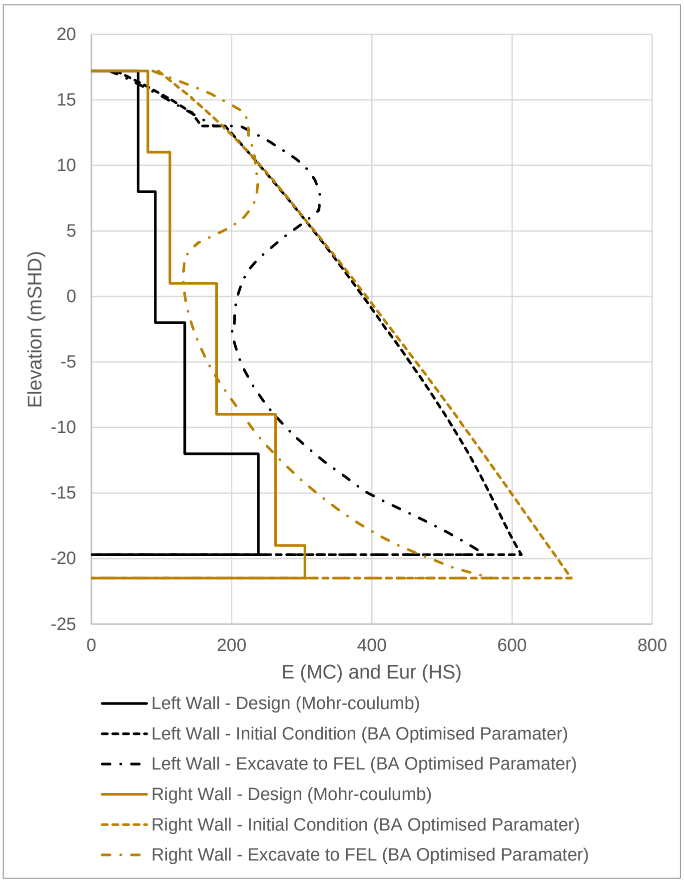


Figure 23 Mohr Coulomb and Hardening Soil Model stiffness comparison

6.3 FORWARD PREDICTION

With the parameters matched by the back-analysis, a forward prediction of excavation to the final level is reviewed with the optimised parameter (Correlation between best fit stiffness from the DAARWIN back analysis and SPTN value in Figure 21). Figure 23 shows that the predicted movement to the final excavation level is within the actual design movement.

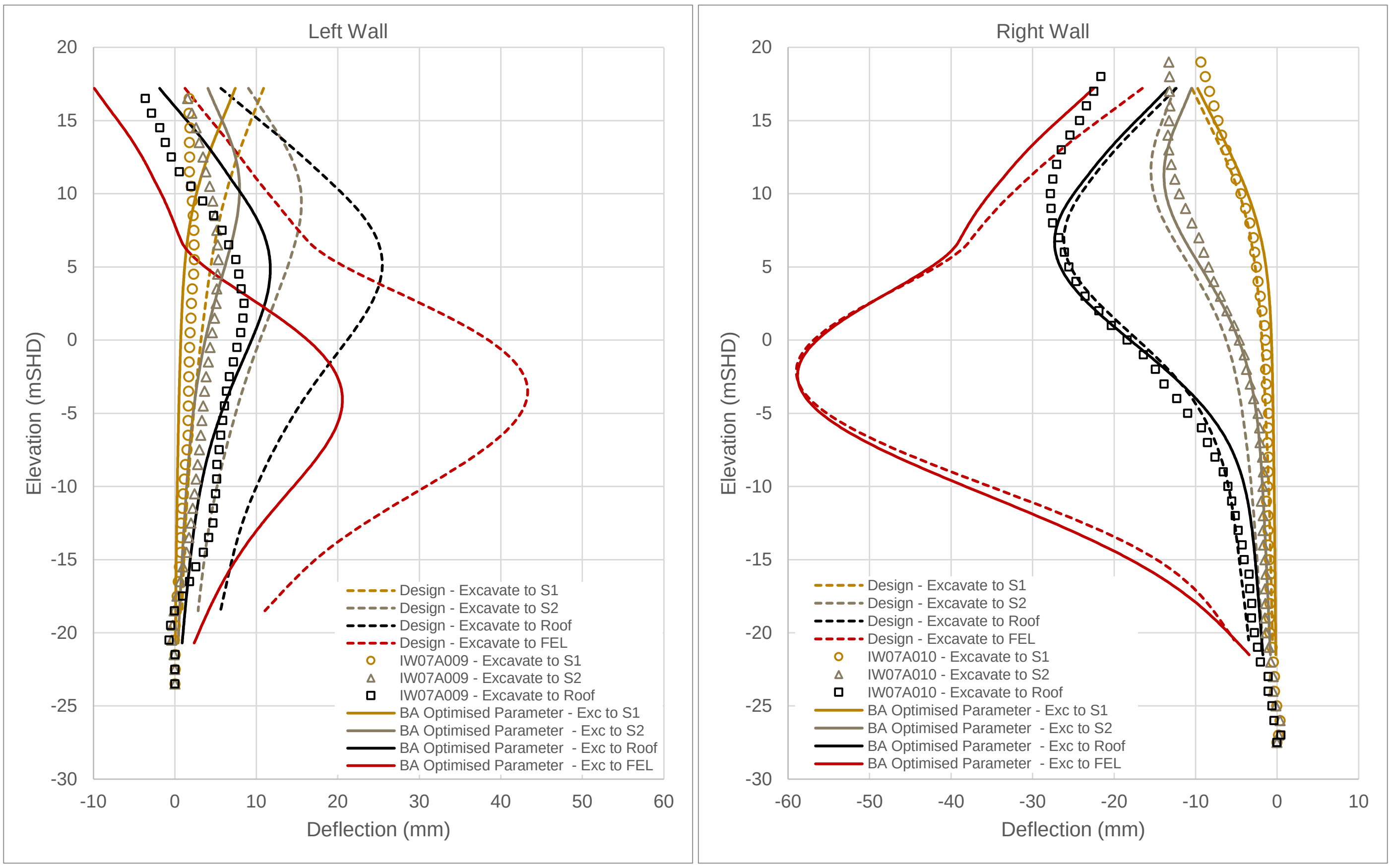


Figure 24 Wall deflection comparison of back analysis correlation vs actual movement

Calculations for Design Approach 1, Combination 1, and Combination 2 were conducted to align with the code requirements as per the Ultimate Limit State (ULS) design. These calculations are shown to be within the envelopes of the original ULS design. The envelope of the diaphragm wall bending moment for the final excavation level is compared in Figure 25. Given that the forward prediction of wall deflection and bending moment comparison indicates that the prediction is within the design, it is unlikely that any additional contingency strut will be needed.

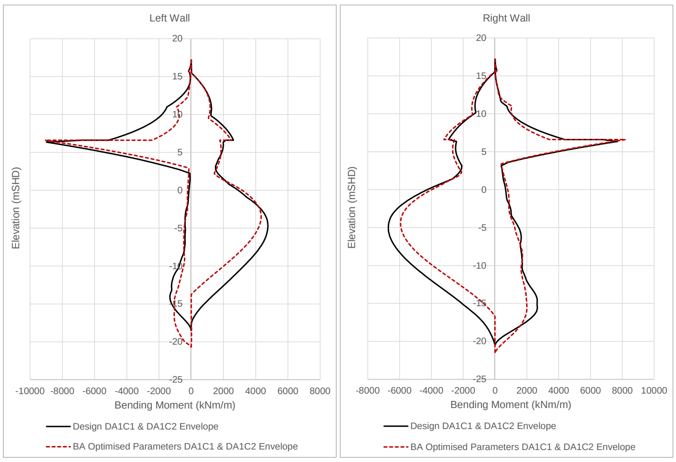


Figure 25 Bending moment comparison of back analysis correlation predicted at FEL and design envelopes

Overall, the DAARWIN back analysis optimised parameters provided good match to the data compared to the design model, but it is important to note that achieving a ‘perfect’ match is highly unlikely due to numerous variables such as actual ground layer, soil permeabilities, strut preloads etc.

Where clear variation exists between the predicted and measured values, it is important to review both the design and the instrumentation as described in the following section.

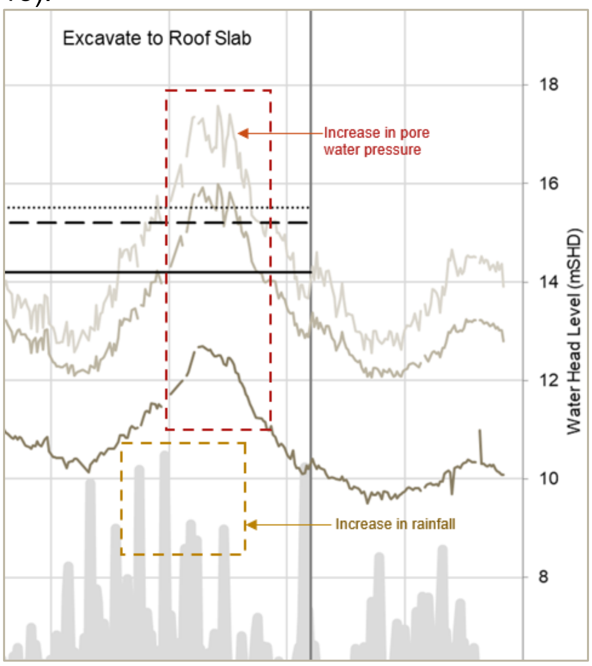
7. INSTRUMENTATION INTERPRETATION

This paper recognises that the interpretation of instrumentation is not merely a function of textbook knowledge or a sound understanding of the fundamentals. Rather, it’s closely tied to both design expertise and practical on-site experiences. As such, this paper presents a checklist as a best practice guide on what to inspect if the actual instrument does not reflect the predicted movement. The checklist can be found in Appendix C.

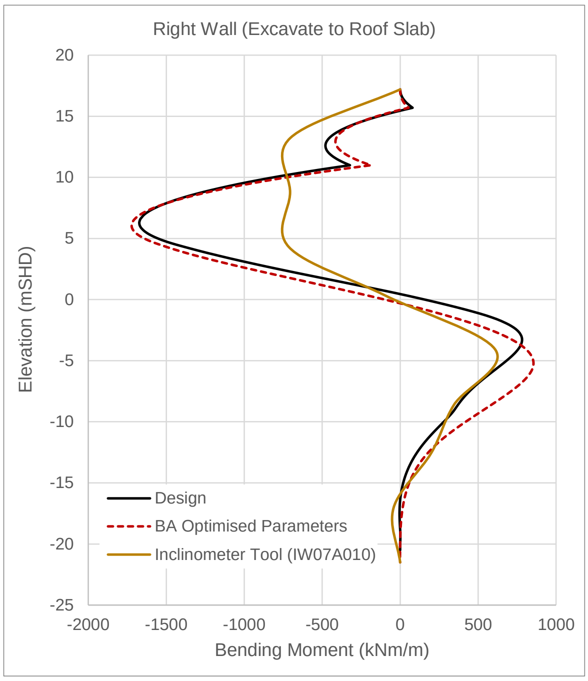
7.1. APPLICATION OF CHECKLIST

As can be inferred from Figure 7, the top 10m of the south wall moved beyond the predicted range at excavation to roof slab, though it was observed that it did not exceed the maximum review level at this point. Despite this, the movement was still examined as part of due diligence to ensure the safety of the current work stage. The following aspects were reviewed:

a. There were no other works other than excavation to roof slab at the area, and the review level monitored aligns with the prediction.
b. There are no spikes observed on the readings, and in-soil inclinometer behind the inclinometer indicated similar trend.
c. An increase in piezometer trend outside the excavation was observed, and this is due to the heavy rainfall recorded (Refer to Figure 18).



d. A rise in pore water pressure could potentially result in an increase in lateral pressure exerted on the wall.
e. There are no significant movements observed from the in-soil inclinometer, ground settlement markers and extensometer, indicating that there are no immediate concerns on the slope movement.
f. The strain-gauge and load cell readings were within the review level.
g. The pre-load of the strutting achieved the design requirement.
h. In terms of design capacity, there are no concerns as the curvature at the maximum movement location is smaller than the predicted. It was further noted that, the monitored reading is 75% of the design moment at this stage.
i. An in-house curve-fitting automated inclinometer tool was utilised with a polynomial fit to convert wall deflection into bending moment to verify the design's bending moment was within design limits at this stage. The comparison below indicates that the observed bending moment obtained from the curve fitting method aligns reasonably well with the back analysis. As such, a quick check of the moment from such inclinometer tool demonstrated a more effective method to assess the wall response rather than solely relying on limits based on wall movement.



It was concluded that there were no immediate concerns, and as such excavation to the next level commenced.

7.2. GROUNDWATER MONITORING INSTALLATION

It's evident that aligning with the groundwater profile is also crucial during back analysis, which highlights the significance of the location where the tips of the ground water monitoring instrumentation are installed. The best practice installation tip for piezometer and water standpipe can be found in Appendix D.

8. CONCLUSION

Frequently, a conventional design tends to result in a more conservative approach, which is not optimal from an industry or sustainability perspective. This paper has shown that the back analysis with the AI tool, DAARWIN, effectively shortens the back-analysis and progressive soil-stiffness modification cycles and has verified the characteristic approach design to date. The forward prediction suggests that the wall is operating within its design resistance, making the need for contingency struts highly improbable. Moreover, DAARWIN as a tool can be used with an observational method approach with progressive modifications to justify omission of planned struts, rather than only confirming that a contingency addition of a strut was not required as in this case. This shows that, with the availability of comprehensive instrumentation and monitoring, engineers, whenever feasible, should explore alternatives and advocate for a more sustainable and robust design.

The parameters outlined in this paper could be applied to design and back analysis tasks for similar ground conditions. While the checklist was originally aimed to help young engineers in understanding the implementation and interpretation of instrumentation and monitoring, it is hoped that it can also serve as a useful reference for anyone dealing with unexpected instrument behavior.

9. LESSONS LEARNT

a. **Infiltration** – From the back analysis, it was observed that pore water pressure significantly influences the movement of the wall. In terms of stability, infiltration can lead to decrease in effective stress and therefore shear strength. In addition, an increase in the pore water pressure differential across the excavation exerts additional out of balance lateral force on the retaining wall system.

The integration of infiltration measures, such as the Capillary Barrier System (CBS), sub-soil drains, and tarpaulin sheets, addresses many of the effects of infiltration as a result of climate change. These measures are in line with the 2023 BCA Framework for Risk-Based Slope Designs (BCA Annex A, 2023)

Figure 26 Geocell arrangement (Satyanaga et al, 2019)

b. **Ground-water modelling** – The initial analysis phase is often modelled as phreatic, which is then transitioned to a steady state seepage during the excavation stages when hydraulic cut off walls i.e., retaining walls are already in the ground. However, when these two approaches are compared, significant changes in lateral effective stress are observed. These changes in pore water pressure result in changes in load on the retaining walls which may not be correctly captured. To ensure this is correctly captured as well as the lateral wall movements, water pressures must be set as steady state seepage in a prior stage, before introducing any structural elements which influence the pore water pressure profile.

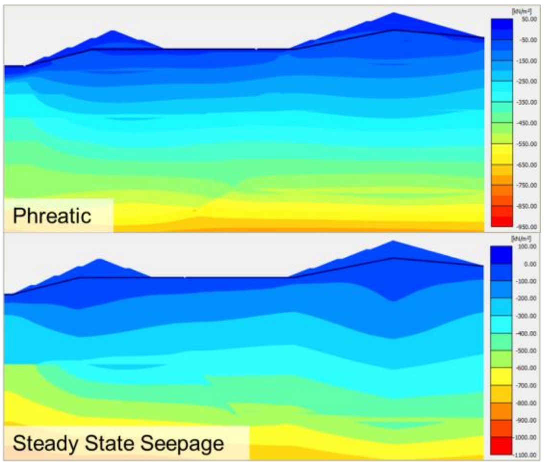


Figure 27 Comparison of Lateral Effective Stress with different ground-water modelling

c. **Wall flexural stiffness (EI)** – For a reinforced concrete section, the value of EI changes over time, with creep and relaxation causing ~50% reduction from the short-term uncracked value over the long term. It is often considered appropriate to adopt 0.7EI during the construction (CIRIA 760, 2017). The back-analysis demonstrates a good fit (Refer to Figure 24), suggesting that the use of 0.7EI for the diaphragm wall in the current design and back analysis was appropriate.

d. **Active communication** – Instrumentation interpretation extends beyond textbook knowledge, linking closely between design skill and field experience. Therefore, maintaining pro- active communication with the various on-site stakeholders (Qualified Person Supervision, Instrumentation Contractor etc.) to understand the site progress and condition is essential.

Figure 28 The author with Qualified Person Supervision (GEO), Resident Engineers, Instrumentation Contractor on site-walk

10. REFERENCES

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Appendix A – Terminology

Back Analysis Best fit stiffness from DAARWIN Back Analysis

BA Optimised Parameter Correlation between best fit stiffness from the DAARWIN Back Analysis and SPT N value.

Design Scheme that was approved and adopted for the design and construction.

HS Analysis with Hardening Soil model for the residual soil.

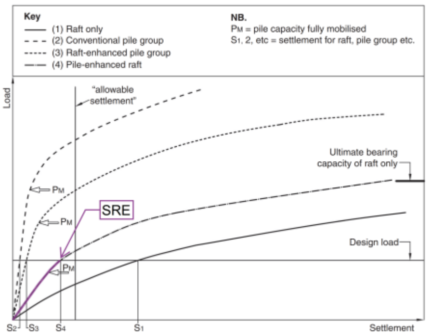
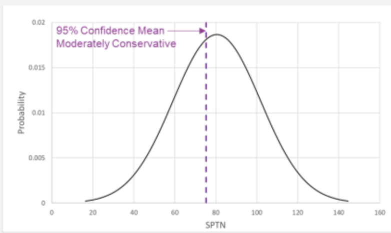
HS Sensitivity Best estimate or fit by varying soil stiffness on DAARWIN Sensitivity Analysis.

Line Colour Differentiates excavation stages.

Line Type Differentiates authors or analysis.

Markers (○, △, □) In-wall inclinometer readings.

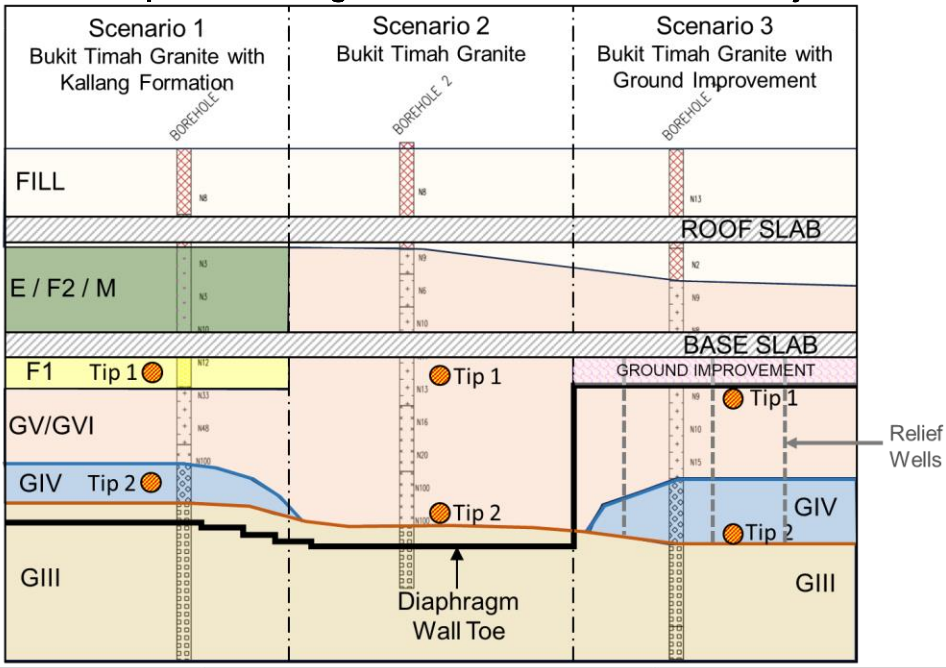
Appendix B – Summary of Design Approach (Thi et al, 2023)

Scheme	Summary of Approach	Challenge / Optimisation
Notional Scheme	Adopting the single worst borehole for design  • 1.2m thick diaphragm wall + 5 layers of strut • 1.5m thick diaphragm wall + 2 layers of strut	In an attempt to optimise the thickness, length of the diaphragm wall and number of struts from the notional scheme, with 1.2m thick diaphragm wall and 2 layers of strut, the following challenges were observed. <u>Foundation</u> 1. Should the toe of the diaphragm wall be reduced without being embedded into rock owing to the deep weathering profile (inability to find rock), and the condition is further aggravated by the thick backfill, it would result in the allowable long-term settlement of 25mm being exceeded. 2. Long diaphragm walls and barrette piles are required to act as deep foundation elements. 3. A deep weathering profile leads to a deep rockhead, which results in long diaphragm walls. <u>Earth Retaining Stabilizing System (ERSS)</u> A thicker diaphragm wall and more layers of strut are required due to poor soil conditions, a conservative assumption based on the single worst borehole.
Option 1 Scheme **Settlement Reducing Element (SRE)**	Pile enhanced raft foundation where it aims to reduce the length of piles (D-walls and barrettes) by increasing the proportion of load resisted by soil beneath the raft while satisfying serviceability limit state of the tunnel box.  	• 1.2m thick diaphragm wall • 2 layers of strut • Reduced backfill thickness (25m to 14.5m) • Reduction in diaphragm wall and barrette pile length (DFE) <u>Note:</u> As illustrated in the table, the Settlement Reducing Element (SRE) method was initially considered as a potential solution for foundation optimisation. While the SRE was still under development, a rigorous review of the reinstated longitudinal soil profile was requested. During the development of the design, consultations were held with the relevant authority regarding the reinstatement level. The outcome was a waiver that led to a lowering of the final reinstatement level and reducing the backfill thickness. This reduction effectively mirrored the anticipated outcome of the SRE approach and thus was not pursued further.
Option 2 Scheme **Characteristic Profile**	The selection of the interpreted geological profile has a direct impact on the design, as the strength and stiffness of G(VI) and G(V) soils, given in the Ground Interpretation Baseline Report (GIBR), are dependent on the SPT-N values. Therefore, a single-worst borehole approach may result in a less sustainable design.  For the approved design, a statistical method is used to select the characteristic profile, which involves a cautious estimate of the mean value with a 95% confidence level. This means that a 'moderately conservative' scenario is chosen for the design. This method aligns with the LTA Civil Design Criteria, which stipulates that designs should not be based on individual logs. Instead, they should be derived from the geotechnical model, taking into account variability and uncertainty. (Reference: E/GD/09/106/A2/16.5.3).	• 1.2m thick diaphragm wall • 2 layers of strut • Reduced backfill thickness (25m to 14.5m) • Reduction in diaphragm wall and barrette pile length (DFE)

Appendix C – Instrument Checklist

General Items to be Checked:	Y/N	Actions
1. What are the current site activities?		
2. Assess the overall data, any sudden spikes, inconsistent data?	Y	Request for check-sum check Request for readings verification
3. Any possibility of site disturbance?		
4. Does the current site activity agree with the construction sequence?		
5. Does the stage prediction review level match the current site activity?		
6. Are there any site distress reported by Qualified Person Supervision team?		
7. Review all instruments within the vicinity, do we observe a same trend?		
Inclinometer Breach Check:	Y/N	
1. Assess for any external factors, were there heavy rainfall?	Y	Check against groundwater monitoring for any fluctuations? For example, on slopes, Water standpipe: Any water level rise observed? Piezometer: Any rise in water pressure observed?
2. Assess any in-soil inclinometers behind the ERSS or prisms installed at the top of the retaining wall, if available. Are they consistent with the in-wall movements?		
3. Is there any structure nearby?	Y	Assess the building and ground settlement markers, are there any settlement observed?
4. Is there any slope nearby?	Y	Assess if slope is moving by checking any in-soil inclinometer, ground settlement markers and extensometer.
5. Assess strutting instrumentation (load cell and/or strain gauge), if applicable. Is this within the prediction, or is a sudden spike observed?		
6. Assess preload record if specified preload is achieved on site		
7. Assess bending moment capacity with a curve fitting tool – care should be taken around struts and slabs where instantaneous changes in the differential of curvature (shear) occur		
Piezometer/Water Standpipe Breach Check:		
1. Is it due to seasonal fluctuation?	Y	Assess any movement on ERSS? Review inclinometer and in-soil readings.
2. Piezometer outside excavation – Review if base reading captures the seasonal fluctuations, Is the draw-down expected at the stage of excavation?		
3. Assess the building and ground settlement markers, are there any settlement observed?	Y	Assess if recharge wells should be activated, if not already.
4. Piezometer inside excavation – are there any flow from the relief well? (If applicable), is drawdown observed from piezometer outside excavation? Is there any flow observed inside the excavation? Was there proper cut-off? Reviewed grouting records?	Y	If relief well has no flow, check functionality of relief wells? Any choke?
5. Does the Piezometer/water Standpipe show similar trend?		
Ground Settlement Markers		
1. Any consolidation observed? Is there soft ground i.e., clay present? If yes, is the ground settlement steadily over a period?	Y	Is there any building? Check differential settlement from building settlement markers. Is there any slope? Check for slope movement. Plot ground settlement marker against reading against piezometer or water standpipe reading

Appendix D – Best Practice Tip for Installing Piezometers Inside for This Project



Tip 1- 1m below F1 Tip 2- GIV/GIII Interface Tip 1 – While excavating, it is important to check the pore pressure in F1 is not going to cause base heave and blow out Tip 2 – G(IV) is highly fractured rock and has a higher permeability. Therefore, Tip 2 is to check on the effectiveness of cut-off measures (D-wall or fracture grouting) and also for basal heave and blow out deriving from this deeper layer.	Tip 1 – 1m below the base slab Tip 2 – soil rock interface Tip 1 – Any potential flow within the G(VI)/G(V) layer Tip 2 – A thin layer of very pervious gravelly sand layer is often present above the bedrock, and this is where the flow tend to occur	Tip 1 – D-wall toe Tip 2 – GIV/GIII interface or toe of relief well Tip 1 and 2 – To ensure pressure relief wells are functional Wall is short due to presence of existing live tunnels. As such, the ERSS system relies on a Ground Improvement (GI) slab for toe stability. Pressure relief wells are introduced to prevent any pore water pressure build up below the GI slab

For Scenario 1 and 2, these piezometers will determine if any contingency measures, i.e., more grouting or pressure relief wells are required

These are scenarios for piezometers inside excavation. These instruments are crucial as they monitor for any potential abrupt failure of the excavation and adjacent infrastructure (live tunnels etc.)