Optimisation of temporary support design for the Northern portal cut & cover tunnel

J. Zeerak[1], Dr. M. Wei[2], J Roper[3], B Clarke[4]

[1]Associate Principal-Geotechnics, EIC Activities Pty Ltd, 567 Collins Street, Melbourne, Australia; email: Jawad.Zeerak@eicactiv.com
[2]Technical Principal-Geotechnics, EIC Activities Pty Ltd, 567 Collins Street, Melbourne, Australia; email: Michael.Wei@eicactiv.com
[3]Principal-Structures, EIC Activities Pty Ltd, 567 Collins Street, Melbourne, Australia; email: Jeff.Roper@eicactiv.com
[4]Engineering Manager, John Holland Group, 180 Flinders Street, Melbourne, Australia; email: Ben.Clarke@jhg.com.au

ABSTRACT

The West Gate Tunnel Project is a city-shaping project that will deliver a vital alternative to the West Gate Bridge, provide quicker and safer journeys, and remove thousands of trucks off residential streets. Delivery of the WGTP project is currently underway by a joint venture of CPB Contractors and John Holland Group. Northern Portal cut and cover tunnel is one of the major structures on the project which requires excavations to a depth of 22 m to allow the launch of the twin 15.6m diameter Tunnel Boring Machines (TBM). Design of the retention system comprised 900 – 1500 mm diameter secant pile wall supported with multiple levels of temporary steel struts. A detailed Soil-Structure Interaction (SSI) analysis together with a review of proposed construction methodology indicated that a two-level propping arrangement as opposed to propping at three levels, which is what would normally be expected for a structure of this scale, would be adequate. The opportunity to remove one level of the proposed steel struts with the potential for a significant reduction in materials and time was considered critical to the completion of this critical path structure. Removal of one level of props would result in a reduction of steel tonnage in excess of 1,000 tons, in addition to improving constructability, productivity and safety. This paper discusses technical aspects of the analysis which enabled the development of the above optimised solution. In addition, the results of the instrumentation and monitoring and performance of the constructed portal structure will be discussed.

Keywords: WGTP, Cut & Cover Tunnel, Deep Excavation, Secant Pile Wall, Soil Structure Interaction, FEM, Value Engineering

1 INTRODUCTION

The West Gate Tunnel Project (WGTP) is a city-shaping project that will deliver a vital alternative to the existing West Gate Bridge, provide quicker and safer journeys, and remove thousands of trucks off residential streets. In addition to the construction of twin large diameter (15.6 m) tunnels the project will deliver:

- Widening of the West Gate Freeway from 8 to 12 lanes
- Multiple crossings across the Maribyrnong River, connected to an elevated viaduct along Footscray Rd, and
- Multiple bridges, entry and exit ramps across the eastern and western zones of the project.

The project is currently in the delivery phase in Melbourne by a construction Joint Venture of CPB Contractors and John Holland Group.

2 NORTHERN PORTAL CUT & COVER TUNNEL

The Northern Portal (NP) cut and cover tunnel at WGTP is one of the major packages of the works on the project. The portal is used in the temporary condition to facilitate the launch of twin TBM machines for the bored tunnels and forms the final cut & cover tunnel (tunnel portal) after completion of the TBM launch and construction of the permanent structural lining. The portal structure is over 330m in length and up to 22.2 m in depth at the interface of the cut & cover tunnel and the TBM tunnel. Temporary support for the northern portal excavation comprised the following retention systems:

- Secant pile walls supported with heavy steel strutting & waling
- Anchor supported sheet pile walls
- Base slab
- Tension piles

Propped secant pile wall was adopted for the deeper parts of the cut & cover tunnel at the interface with the TBM tunnels, transitioning to the anchored sheet pile wall as the excavation depth reduces towards the open trough structure. Multiple design types were incorporated in the design to suit excavation depths and ground conditions along with the portal structure. The portal structure was designed as a tanked structure (undrained) for the full length of the portal to minimise disturbance to the regional groundwater regime. Figure 1 presents a general layout of the northern portal.

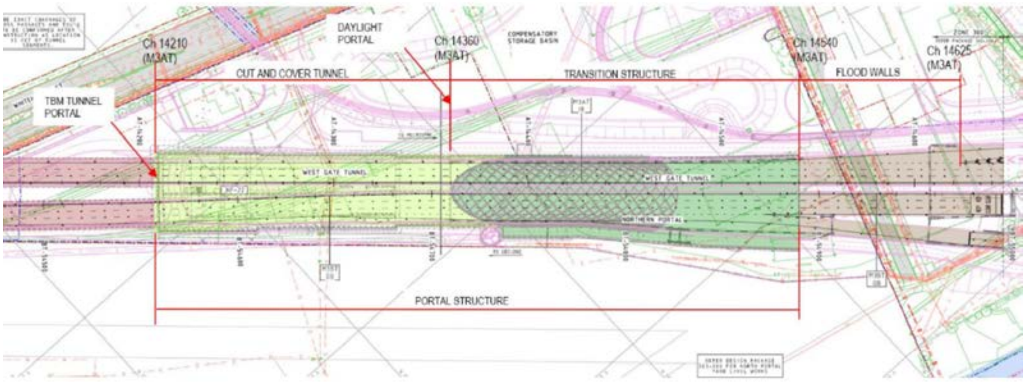

Figure 1. Northern portal site layout

It is worth noting that the northern portal cut & cover tunnel is a major structure comprising propped secant pile wall in the temporary and a structural lining wall constructed on the inside in the long term. The two structure types combined forms the cut & cover tunnel with a significant interaction between the two structure types. Discussions in this paper are limited to the analysis conducted for the structure in the temporary condition only before construction of the permanent structural lining. It is further noted that the discussion presented in this paper are based on the analysis, which are limited in nature conducted as part of a value engineering and optimisation exercise, detailed design of the structure were undertaken by others.

2.1 Geological Conditions & Geotechnical Parameters

Figure 2 presents surface geology of the general project area highlighted on an extract from the Geological Survey of Victoria's Melbourne Mapsheet.

The geological conditions at the Northern Portal consists of an upper unit comprising man-made Fill overlying a thin veneer of the soft and compressible Coode Island Silt (light green in Figure 2, not shown in Figure 3). These thinner units are underlain by a more substantial layer of Quaternary alluvial outwash, and deeper alluvial infill to a paleochannel crossing the northern end of the Northern Portal (green/dark green in Figure 3). Variable thicknesses of the Brighton group unit (yellow in Figure 3) are encountered in parts of the northern portal. The older volcanics rock (pink in Figure. 3) comprises the bedrock in the area of the portal where most of the piling and excavation works within the deeper ends of the northern portal occur.

Variable degrees of weathering of the older volcanic units were encountered during piling which comprised residual soils to extremely and highly weathered rock. Tertiary aged Werribee formation sediments underly the Older Volcanics rock. A typical geological section at the northern portal is shown in Figure 3. Groundwater levels vary along with the portal structure and ranges typically between RL0 mAHD to RL-4 mAHD. The lower levels were associated with the long-term depressurisation effects of local drains e.g. North Yarra Sewer Main. The portal structure was designed as a tanked structure with the retention structure comprising predominantly a secant pile wall which is considered watertight. It is worth noting that a row of recharge wells was proposed along the perimeter of the excavation to minimise the impact of works on the local groundwater regime. Given the secant wall system, any groundwater inflow was expected to be through the foundation rock which was expected to be minimal.

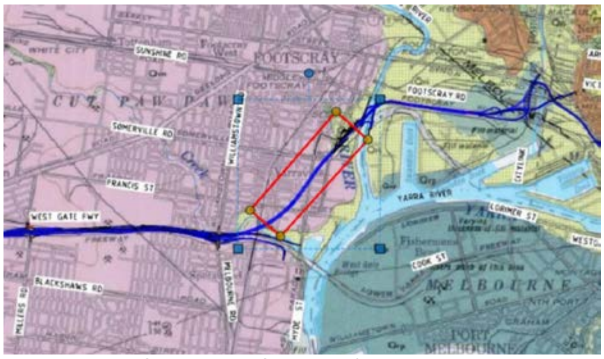

Figure 2. Geological Survey of Victoria, Melbourne Mapsheet, (1:63,360)

Geotechnical parameters adopted in the finite element analysis were derived from site investigation data, laboratory testing, published literature and past experience. Parameters for the soil units were generally derived from laboratory testing, typical correlations, and past experience. While rock mass parameters for use in the design of excavation support were assessed using the generalised Hoek-Brown strength criterion using the Roclab (RocData) software by Rocscience. Adopted parameters are summarised in Table 1.

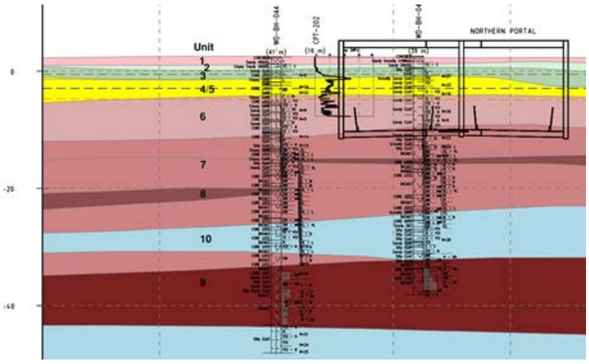

Figure 3. Northern Portal Geological Profile

In the analysis, both drained and undrained parameters were utilised relevant to each stage of the construction. In general, short term or undrained parameters were used in the short term i.e. during top down excavation to the final excavation level and casting of the base slab after which soil parameters were changed to drained parameters relevant to long term conditions.

The above covers two extremes of soil behaviour.

Table 1: Adopted Geotechnical Parameters

Unit	Unit	r [kN/m^3]	Su [kPa]	c' [kPa]	Phi' [°]	E' [MPa]	ν'
1	Fill – General	17	20	2	26	12	0.3
2	Alluvium Outwash	18	-	1	33	25/20/75	0.2
3	Paleochannel Alluvium	18	120	10	28	25/20/75	0.2
4	Brighton Group-Clays	19	150	10	30	40	0.35
5	Brighton Group-Sands	20	-	1	34	60	0.3
6	Older Volcanics-Residual Soil	20	150	15	28	60	0.2
7	Extremely weathered older volcanics	21	250	30	28	200	0.3
8	Highly weathered Older Volcanics	21	-	50	40	800	0.3
9	Slightly weathered Older Volcanics	28	-	400	55	6000	0.2
10	Werribee Formation	20	200	20	28	50/40/150	0.2

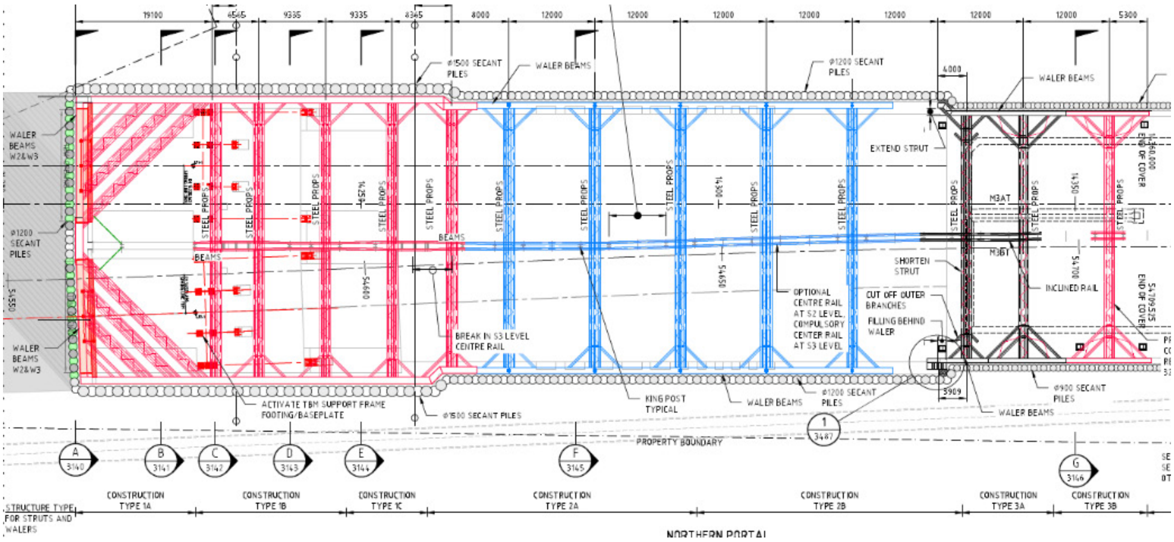

Figure 4. Northern Portal strutting layout

In reality, the soil behaves in a lot more complex manner than that in the modelling. For the purposes of covering critical cases in terms of forces on the retention structure and support elements, various sensitivity checks were carried out with changing soil properties from undrained to drained at various constructing stages. The impact of the above sensitivity checks on the overall design was found to be negligible in this case.

Excavation works are essentially 'unloading' problems. This behaviour is best captured using the constitutive soil model Hardening Soil (HS) model in a finite element analysis e.g. PLAXIS. The HS model is an advanced soil model that is able to generate a more realistic soil response in terms of non-linearity, stress dependency and inelasticity of soils. The model requires three stiffness parameters (E50/Eoed/Eur) as shown in the E' column in Table 1. In the analysis, in general an unload/reload modulus (Eur) of 3 x Secant modulus (E50) was adopted based on literature and common practice. Although experience and some cases studies in literature indicate that this ratio can be significantly higher than 3 in some cases. This may also explain the better than predicted deflection results for the structure which will be discussed in later sections.

3 NORTHERN PORTAL RETENTION DESIGN

The adopted retention system for the majority of the cut & cover structure comprised 900 mm to 1500 mm secant piles propped with multiple layers of steel struts. A secant pile wall option was required to provide a watertight structure as required by the project technical specifications. The larger diameters 1200 mm and 1500mm diameter piles were adopted for the deeper parts of the excavation in a hard-hard pile configuration, while the smaller 900 mm diameter piles were adopted for the shallower excavation depths with a hard-soft arrangement. It is noted that a hard-hard pile arrangement was incorporated in the design for constructability reasons i.e. to reduce reinforcement congestion and ease of installation.

The proposed design of the propping for the secant pile wall comprised 3 levels of steel strutting connected to a waler system against the secant pile walls. The largest steel struts comprised double 1200WB steel sections connected to a double 1200WB walers system attached to piles. Steel struts were supported in the middle using a row of king posts supported on bored piles socketed to rock. The initial strutting layout at S2 level is shown in Figure 4 with a typical section shown in Figure 5.

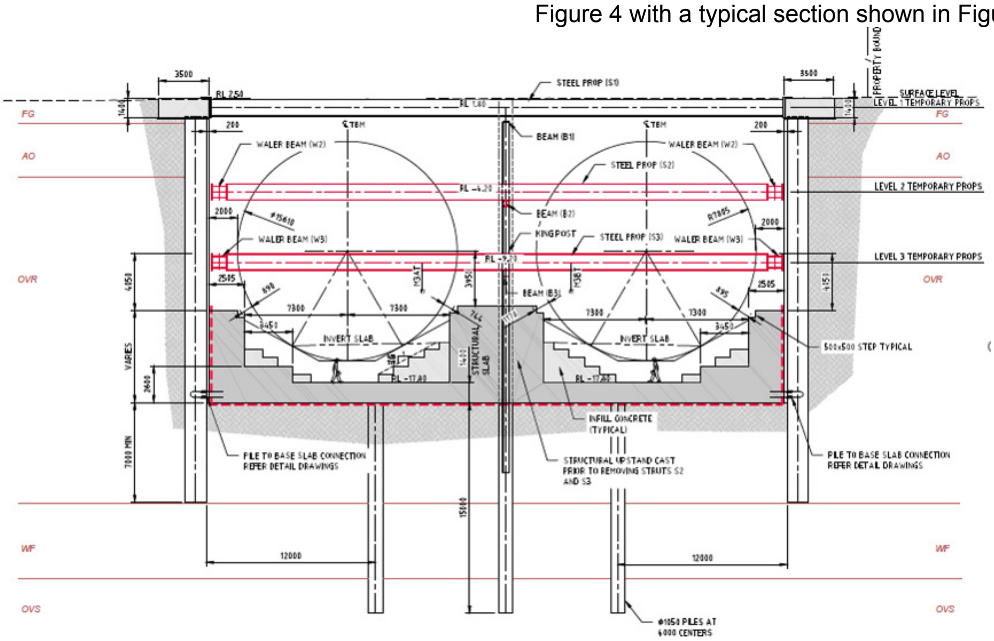

Figure 5. Typical retention design – Northern Portal

The assessment criteria relevant to the design of retention system in the temporary condition included:

- Stability during excavation
- Wall deflection and settlement behind the wall
- Crack widths
- Strength checks

Stability, deflection, and settlement checks were assessed as part of the soil-structure interaction (SSI) analysis discussed in this paper, while strength and crack widths along with other criterion relevant to structural design were assessed under several load combinations as part of a detailed structural analysis, which are outside scope of this paper.

4 SOIL STRUCTURE INTERACTION

As noted earlier, in the temporary condition the portal structure was intended to facilitate the TBM launch for the twin bored tunnels. In other words, the portal structure was on the critical path for the project. Given the required excavation depths, length, size and type of structure, there was a considerable scope of piling and strutting required. Any improvement in the construction program (and cost savings) was seen critical to the success and timely completion of the works. The above warranted a detailed review of the structure in particular the temporary support system. The review targeted geotechnical parameters, improvements in the finite element analysis (FEA) modelling approach, construction sequencing as well as piling methodology some of which is described in brief here.

The portal structure comprised a secant pile (1200 diameter for sections discussed in this paper) wall retention system in the temporary to retain the soil and water before a permanent lining structure was constructed on the inside. The temporary retention piles and final structural lining was constructed over several stages. During the top-down construction, steel props (typically 2x1000WB) were used to support the retention piles. As the final structural lining was constructed progressively in a bottom-up approach, permanent supports comprising a base slab, road deck and roof slab replaced the temporary props. Given the relatively complex nature of works and associated construction stag, involving installation and removal of multiple supports, a detailed Soil-Structure Interaction (SS) model was developed using the finite element software program PLAXIS 2D. A SSI analysis such as one described above, is able to capture interaction between structural elements (retention piles), ground and water loads particularly as the loads changes during the staged construction and locked in stress are generated at various stages. In addition to above, there is also a significant interaction between the retention piles and the permanent structural lining as the permanent lining is progressively installed and temporary supports removed. PLAXS analysis showed that loads on the retention piles (and supports) were greatest in the short term before installation of the permanent supports. This was expected given the limited support points in the temporary condition in contrast to the final configuration when the structure is propped at multiple levels with road deck, roof slab and base slab. Once the structural lining and permanent supports were installed, there is a load redistribution and load sharing occurring between the retention piles and the lining structure. Given the focus of the current paper is on the temporary supports system until excavation of the final excavation level and installation of base slab and upstand walls as shown in Figure 7, discussion on the interaction of permanent and temporary structure will be limited to the above.

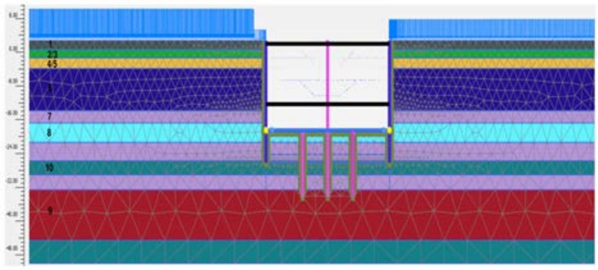

Figure 6. Typical PLAXIS 2D Model

In the PLAXIS model, all construction stages were simulated based on the proposed sequence until completion of the excavation and installation of the base slab and removal of the S3 prop. The above stages can generally be summarised as below:

- Installation of retention piles
- Progressive excavation and installation of temporary steel props
- Construction of base slab and casting of upstand walls (partially completed permanent wall)

Figure 7. View of northern portal temporary strutting towards TBM headwall

The level for S1 prop was generally fixed at the caping beam level for the portal structure as they are most effective in controlling pile head deflections and from a constructability point of view. The aim of the above exercise was to eliminate S2 prop from a design where feasible. Therefore, the configuration for the S3 (lowest level prop) was of significant interest in order to optimise the propping design. An optimum arrangement of S3 prop was considered to be one which would allow removal of S2, while limiting wall deflections and providing adequate support to the retention piles in what would be considered a significant unpropped height between the S1 and S3 (with S2 removed).

To achieve the above goal, various iterations were run in the PLAXIS analysis to test and assess the most optimum configuration for S3 prop. From the iterations of the PLAXIS analysis, it became clear that a lowered S3 prop would generally be a lot more effective in providing lateral support to the retention piles. However, the extent on how much the S3 props could be lowered was constrained by the working space required for the base slab and construction of the partial upstand walls. Therefore, the prop was lowered as far as feasible for practical purposes.

It is also noted that small section of the permanent structural lining was constructed in early stages after casting of the base slab which was called 'Upstand Walls'. The analysis also showed that it was critical that the partial upstand walls were constructed prior to removal of the S3 prop. These upstand walls would prop the retention piles at a higher level and therefore reduce additional forces generated in the piles. Analysis indicated that this would result in an increase of forces within the base slab and upstand walls slightly, however, the benefits would outweigh the small increase in forces. The above were relatively simple, yet critical modifications required to enable a two-level strutting system for spans of over 15 m between the two props which were considered unprecedented in author's experience.

In addition to the above, various refinements and improvements were also made in the finite element modelling resulting in improvements of the overall analysis outcome. In the recent versions of PLAXIS 2D, the software allows the input of flexural stiffness as 'elastoplastic' moment-curvature diagram instead of the more 'traditional' way of defining flexural stiffness of the plate elements as a linear elastic material. With the improved moment-curvature (M-K) input feature, it was found that forces generated within the retention piles were more optimised and considerably lower than those which would have been obtained using the traditional 'EI' values reduced by a factor to account for short- and long-term cracking of the concrete. The combined effects of the above led to the possibility of the S2 (middle props) being removed from the design leaving two levels S1 and S3. Although there were differences in the pile forces at individual stages, PLAXIS analysis indicated that retention pile structural actions envelopes generally remained similar between a two-strut configuration and a three-strut system. Proposed prop sizes also remained adequate for the slightly increased loads with the S2 props removed. Figure 8 shows a view of the northern portal temporary works completed.

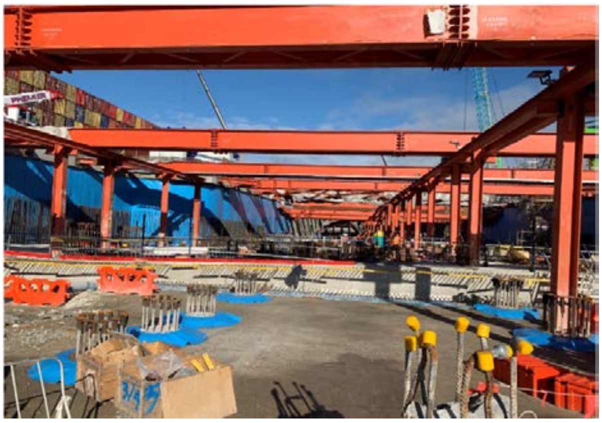

Figure 8. View of northern portal temporary strutting

5 INSTRUMENTATION AND MONITORING RESULTS

Given the size and scale of the portal structure, a detailed and comprehensive Instrumentation and Monitoring regime was recommended as part of the design. The following instruments were installed on piles and struts along the portal:

- In Place Inclinometers at regular intervals
- Reflectorless prims at multiple levels on the piles
- Settlement markers behind the excavation at regular intervals
- Groundwater monitoring wells; and
- Strain gauges on selected steel props to monitor forces on the steel struts.

The I&M requirement was a critical part of the design allowing the wall and strutting system to be monitored during progressive excavation and installation of strutting system. This was in particular critical where the S2 level props were omitted. The recommendation was that retention piles and S1 level props would be monitored continuously, and wall deflections and prop forces compared against design predictions.

The monitoring system would allow for early intervention should the monitoring results indicate a need based on the wall performance during construction. Accordingly, wall deflections were monitored continuously during bulk excavations and installation of the steel props until base slab excavation. Based on the PLAXIS analysis, a maximum wall deflection of 55mm was expected in the final stage, upon removal of the S3 prop after the base slab had been constructed. Inclinometer readings obtained during construction indicated maximum wall deflections of 20mm which was well below the design predicted value and set trigger levels. Typical inclinometer profiles showing horizontal wall deflections are shown in Figure 9.

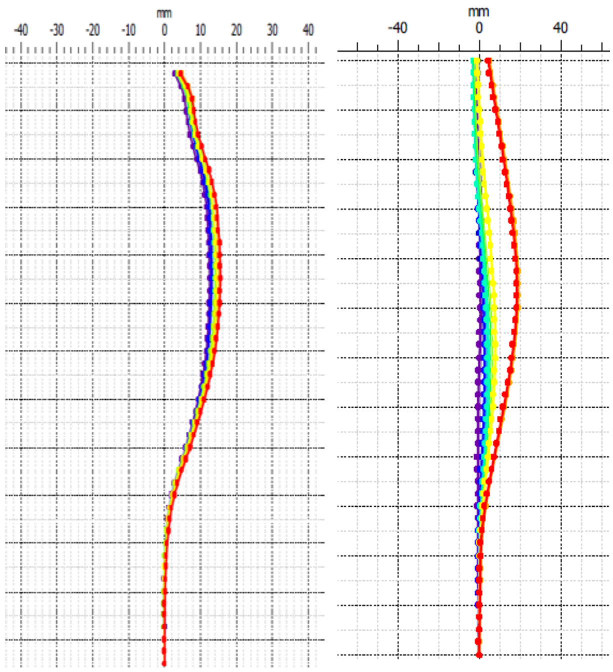

Figure 9. Typical inclinometer profile (horizontal wall movements)

Similarly strain gauges installed on the steel struts were monitored during construction to obtain prop forces and comparison against design predictions. Monitoring results indicated strut forces within the range predicted in the PLAXIS analysis. There were variations in the peak forces as shown in Figure 10. These fluctuations in the forces were attributed to the changes in the atmosphere temperature levels causing expansion and contraction of the steel struts. Progressive and real-time monitoring results obtained at each stage of the construction provided confidence in the suitability and adequacy of the design.

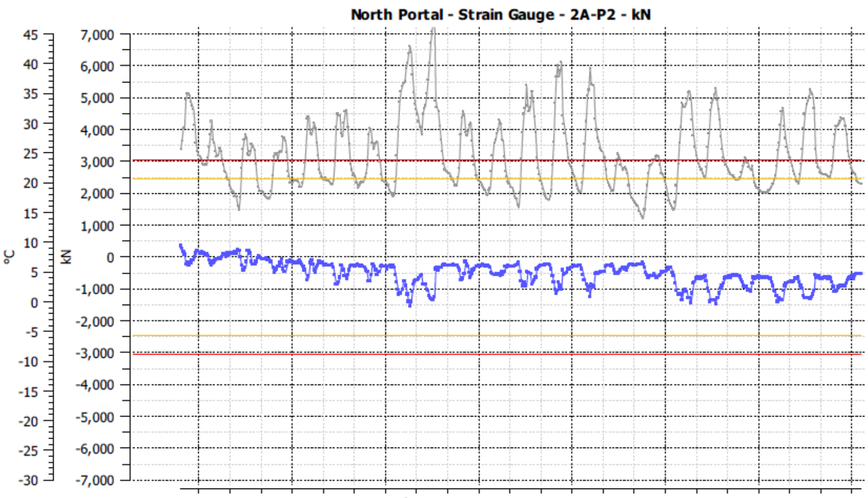

Figure 10. Strain gauge data

Observations made from the monitoring results were that significantly lower wall deflections were recorded compared to those predicted in the design and calculated in PLAXIS analysis. This is in part attributed to some level of prudent conservatism in the design as would expected for the scale and size of the excavation described in this paper. However, other observations and lessons learnt relating to better-than-expected performance of the excavation support system, particularly the retention piles could be attributed to factors stated below:

- Stiffness parameters for various rock units were assessed using the Roclab software using lower bound and upper bound test results. In the PLAXIS analysis, typically lower bound parameters were used given the scale and size of excavation, associated risks and to cover inherent variability and uncertainty associated with ground conditions. It appears, that encountered ground conditions have responded considerably better than those assumed in the analysis which has resulted in better performance of the retention system. Greater levels of confidence can be reached with additional targeted geotechnical investigations and laboratory testing to optimise parameters further during design stages.

- As noted earlier, an unload/reload (Eur) modulus of 3xE50 was adopted in the design. It is also possible that in this Eur/E50 ratio is higher than 3, potentially greater than 5. Pressuremeter testing can be undertaken to obtain more accurate estimates of the Eur/E50 ratio.

- There is a propping effect at the two corners where the portal structure meets the TBM headwall. Based on experience from other similar structures, the corner effects can be significant due to soil arching in the corners, but also the propping effect of the end walls (TBM headwall in this case) to the side walls. Given the limitations associated with a 2D plane strain analysis, this was not captured in the analysis undertaken for this structure. However, these beneficial effects can be captured well using a 3D type analysis.

6 CONCLUSION

As part of a value engineering exercise, a detailed soil structure interaction (SSI) and review of construction methodology indicated possibility of removal of one level of props for a major portal structure on the WGTP. The SSI was completed using the finite element software package PLAXIS 2D. Results of the FE analysis indicated that a two-level strutting arrangement opposed to propping at three levels, would provide adequate support to the 22 m deep excavation at the portal structure. Level of the lowest strut (along with other improvements discussed in this paper) was found to be critical in optimising the temporary support requirements for the portal structure. A thorough and continuous instrumentation and monitoring regime prescribed as part of the design, enabled a close monitoring of the performance of the structure and provided confidence during construction with the suitability and adequacy of the proposed design. Results of the ongoing monitoring indicated a generally better performance of the retention design than expected. This may be attributed to the 3D nature of the structure towards the deeper ends of the portal which was not captured in the 2D analysis and encountering better ground conditions in particular better rock stiffness than those assumed in the design. This underlines the significance of adequate site investigation and testing to enable detailed assessment and adoption of refined geotechnical parameters. The adoption of an optimising strutting design at two levels as opposed to three levels which would normally be seen reasonable for excavation depths such as those required at the northern portal, provided significant cost savings to the project in addition to an efficient, smoother and safer construction program.

7 ACKNOWLEDGEMENTS

As noted in the introduction, the discussions presented in this paper are based on limited analysis conducted as part of a value engineering and optmisation exercise. Detailed design of the structures are by others. Authors wish to acknowledge inputs and support provided by the WGTP tunnel zone personnel during design and construction phases of the northern portal structure. without whose support this paper would not have been possible. Finally, authors wish to thank Dr. Jeff Hsi of EIC Activities for a peer review of this paper.

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