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# Melbourne Metro Tunnel portal temporary earth retention works - design approach and measured performance

## Travaux temporaires de rétention de terre du portail du Melbourne Metro Tunnel - approche de la conception et des résultats mesurés e

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ABSTRACT: At the Melbourne Metro Tunnel western portal site, the temporary earth retention works near the existing South Kensington station posed several geotechnical design challenges. The geotechnical temporary works design team required working closely with the services asset owners, Metro Trains Melbourne and the Rail Infrastructures Alliance construction team to deliver safe and economical designs and construction methodologies. The mandated performance criteria for maintaining the adjacent railway tracks fully operational during a majority of the construction period, the geotechnical challenges due to the presence of soft silty clay soils, and the time and space constraints meant a non-conventional approach was necessary for the effective design of temporary earth retention systems. The paper presents the details of the temporary retention system design of a rail embankment overlying soft soils at the western portal site. It discusses the design approach adopted, that considered the past research results for the characterisation of the local soft clay soil unit to support the limited number of geotechnical investigation results available for the site, and the use of numerical analysis methods for design assessment. The results of a rigorous monitoring regime adopted for the performance evaluation of the earth retention system and the safe operation of railway tracks are also presented and discussed. The monitoring results compared reasonably well with predicted values during the design.

RÉSUMÉ: Sur le site du portail ouest du Melbourne Metro Tunnel, les travaux temporaires de rétention de terre près de la station existante de South Kensington ont posé plusieurs défis de conception géotechnique. L'équipe de conception des travaux temporaires géotechniques a dû travailler en étroite collaboration avec les propriétaires des actifs des services, Metro Trains Melbourne et l'équipe travaux de Rail Infrastructure Alliance afin de concevoir des méthodes de dimensionnement et de construction sûres et économiques. Les critères de performance imposés pour maintenir les voies ferrées adjacentes pleinement opérationnelles pendant une grande partie de la période de construction, les défis géotechniques dus à la présence de sols argileux limoneux mous, ainsi que les contraintes de temps et d'espace associées au travail dans un corridor ferroviaire ont fait qu'une approche non conventionnelle était nécessaire pour la conception efficace des systèmes de rétention de terre temporaires. L'article présente les détails de la conception du système de rétention temporaire d'un remblai ferroviaire recouvrant des sols mous sur le site du portail ouest. Il examine l'approche de conception adoptée, qui a pris en compte les résultats les recherches passées pour la caractérisation de la formation locale qu'est ce sol argileux mou afin de renforcer le nombre limité de résultats d'études géotechniques disponibles pour le site, et l'utilisation de méthodes d'analyse numérique pour l'évaluation de la conception. Les résultats d'un système de surveillance rigoureux adopté pour l'évaluation des performances du système de rétention des terres et la garantie d'exploitation des voies ferrées sont également présentés et discutés. Les résultats de la surveillance ont été raisonnablement satisfaisants comparés aux valeurs prévues lors de la conception.

KEYWORDS: Temporary earth retention works; railway; sheet pile; soft soils; numerical analysis method.

## 1 INTRODUCTION

This paper introduces the design approach adopted for the earth retention system built as a part of the temporary works required at the western portal site of the Melbourne Metro Tunnel near South Kensington station in Kensington, Victoria. A decline structure comprising concrete slabs founded on driven precast concrete piles was proposed to support the new Metro Tunnel tracks connecting to the existing Sunbury lines. The existing lines run immediately south of the proposed new decline structure on an existing rail embankment constructed from uncontrolled fill materials, inferred to be placed circa 1850s (inferred from HCV 1999) over a Quaternary aged soft silty clay unit, locally known as Coode Island Silt (CIS). The CIS layer thickness at the site is about 20 m, getting thicker towards the Metro tunnels.

Due to the proximity of the existing railway tracks, temporary earth retention systems with retained wall heights of up to 3.2 m were required to facilitate embankment cuts for the construction of the decline structure while keeping the existing lines operational throughout a majority of the period of the works. It was also required to control the displacements due to the temporary construction works for a relocated Overhead Line Electrification (OHLE) gantry and a relocated signal mast supported by a post-and-panel retaining wall also located at close proximities to the temporary retaining wall alignment. Careful considerations to minimise the effects of construction works to the existing underground services as well as fulfilling the requirements of the service asset owners were also necessary. This required close teamwork between the designers, contractors (RIA) and the asset owners (Metro Trains Melbourne, MTM) to agree on the mandated design performance requirements, and also the monitoring regimes to be put in place for maintaining the rail operation on the adjacent track for a majority of the construction period.

Figure 1 presents an indicative plan view of the extent of the proposed temporary works for the construction of the decline structure at the Metro Tunnel western portal. The proposed

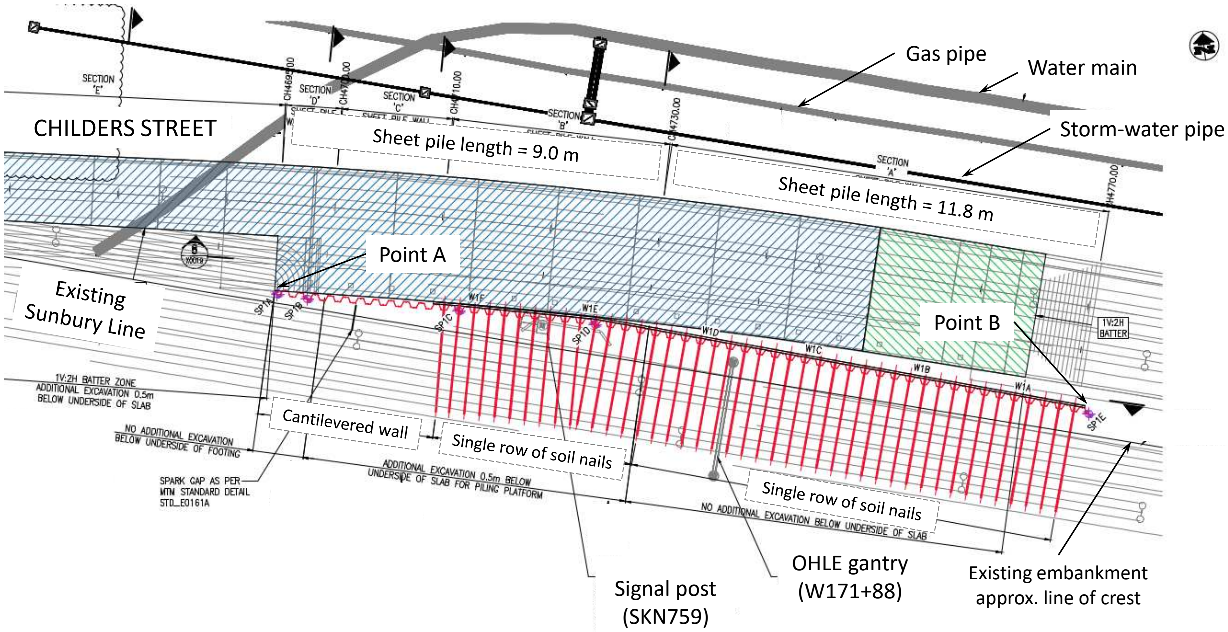


Figure 1. Plan view showing locations of the proposed temporary retaining walls.

temporary works retaining walls extended between CH4695 (Point A) and CH4763 (Point B), with higher chainages towards the eastern side of the work site extent as the decline structure approached the Metro tunnels.

## 2 PROPOSED TEMPORARY RETAINING WALLS

The existing embankment surface level elevations varied between RL5.2 m and RL5.7 m within the extent of the proposed temporary retaining walls. The northern batter of the existing embankment consisted of upper and lower batters at about 1V:2H with a service access track located in the middle of the slope, which resulted in an overall slope of about 1V:3H.

The proposed temporary retaining walls comprised unsupported cantilevered and soil nail supported sheet pile walls for maximum retaining wall heights of 2.5 m and 3.2 m from the excavation base level, respectively. The distance from the wall to the centreline of the nearest existing track varied from about 2.9 m (Point A) to about 5 m (Point B).

The proposed sheet pile wall consisted of AZ36-700N steel sheet pile sections with maximum lengths of 11.8 m and 9.0 m available for use at that time. A single row of soil nails seated on a waler beam was proposed to support the longer sheet pile sections for wall portions where the retaining wall height was greater than 2.5 m. The soil nails comprised 25 mm diameter steel bars, 10 m long installed at an inclination angle of 10 degrees to the horizontal and grouted into 150 mm diameter drilled holes. A berm (with a nominal crest width of less than 0.7 m ignored for assessment) was formed at the base of the excavation buttressing the soil nail supported wall portions.

It was understood that an underground 450 mm diameter gas pipe buried at depths of 1.5 m to 2.5 m below road level was located along Childers Street at a closest offset distance of 9 m from the northern edge of transition slab at CH4763 (Point B).

The construction sequence mainly involved an extended piling platform constructed over the existing ground for the sheet pile installation using a sheet pile piling rig, followed by the excavation of embankment fill to the base of excavation for the construction of a working platform for a driven pile piling rig. Assessments were undertaken to assess the impacts of the extended embankment access track and the excavation of the embankment fill forming the retaining wall on the underground gas pipe which showed that the impacts were negligible.

Six typical wall sections were considered for the design assessment and estimation of the wall and the surrounding ground displacements. In addition to consideration of the retained height, identification of the wall sections for design required considerations to the approximate embankment fill over the CIS layer and approximate offset distance of the nearest track centreline.

## 3 SUBSURFACE PROFILE AND DESIGN PARAMETERS

The proposed temporary works area is on the eastern side of Kensington Road. Based on the project experience involving a soil nail wall design in the embankment fill in the western side of Kensington Road, as a part of the Maribyrnong Viaduct in Regional Rail Link Project, as well as with reference to the information that the heritage-listed rail bridge over Maribyrnong River was built during 1858-59 (HCV 1999), it was understood that that the rail embankments on either side of Kensington Road were a part of the original historic embankment fill in the area. It was observed in the boreholes that embankment fill overlays the CIS layer within the temporary works site extent.

### 3.1 *Embankment Fill*

Based on the geotechnical borehole logs located in the rail embankment access track, the fill materials appeared to be highly variable in composition with medium dense to dense sandy or clayey gravel observed in the upper 1.5 m depth which is underlain by predominantly gravelly/silty clay with some clayey sand layers and traces of brick fragments in places. The existing embankment fill is considered as an uncontrolled fill by current engineering standards.

The SPT results for the embankment fill material, presented in Figure 2, indicate that the gravelly/silty clay fill is predominantly stiff to very stiff (above RL+1 m) becoming firm below RL+1 m towards the interface between the fill and CIS.

The interface between the fill and the underlying CIS is found to range between RL0 m and about RL0.5 m. For design assessment, the fill-CIS interface and the groundwater levels were taken to be at RL+0.5 m.

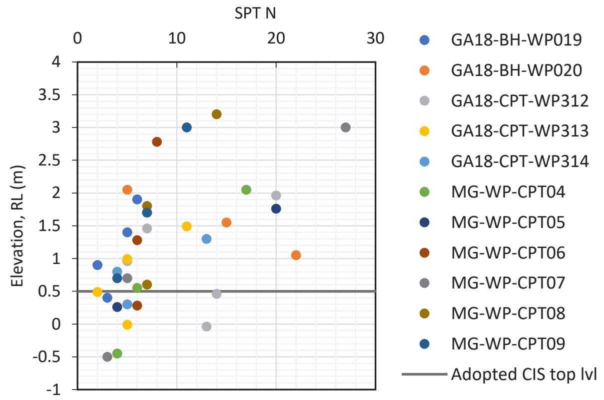


Figure 2. Measured SPT N values in rail embankment fill.

An effective cohesion of 2 kPa and an effective friction angle of 30 degrees were adopted as design parameters for the embankment fill, which were based on the back analysis of the stability of the existing embankment batters achieving a minimum FoS of 1.5. An undrained shear strength of 50 kPa and secant modulus, $E_{50}$ of 15 MPa and unload/reload modulus $E_{ur}$ of 45 MPa were adopted for design. The embankment fill was modelled using a Hardening Soil model for numerical analysis.

### 3.2 *Coode Island Silt*

The Coode Island Silt is a pale grey to grey, medium to high plasticity silty clay. As we understand the embankment was constructed over several decades, the CIS under the footprint of the embankment was inferred to have been consolidated under the embankment weight. Based on the results of the CPTs, the CIS was assessed to be of firm consistency with a gradual increase in strength to a depth of about 25 m below test surface elevation.

Four CPT locations located on the rail embankment access track were referred to for the inference of the geotechnical strength parameters. The CPTs were carried out in the CIS below the existing embankment fill.

An overlay of the variation of undrained shear strength, $s_u$, interpreted from the CPT results with depth is presented in Figure 3. The undrained shear strength profile using the cone tip resistances has been calculated adopting a cone correlation factor $N_k$ of 15, found to give reasonable estimates for CIS (Ervin 1992, Srithar 2010 and King et al. 2016).

The cone resistance indicated that the undrained shear strength of the CIS is about 30 kPa and linearly increasing at 3.5 kPa per metre with depth (orange line in Figure 3). The variation of empirical $s_u$ with depth for a given OCR of 2 has been presented, which is commonly adopted for design, e.g. based on Ervin (1992), is also shown in the figure for reference (blue dashed line). It is observed that the empirical $s_u$ profile is close to the lower bound of the CPT interpreted $s_u$ profile. The design approach considered both undrained and drained conditions. The adopted design parameters are presented as the orange and green lines in the figure. The green line represents the maximum mobilised shear strength for an effective cohesion of 2 kPa and an effective friction angle of 25 degrees.

One-dimensional consolidation tests were undertaken on CIS samples from within and adjacent to the work extent area. The consolidation test results indicated the compression index ($C_c$) ranges between 0.769 and 0.231 in the applied vertical stress range of between 100 kPa and 200 kPa. In general, the samples at shallower depths and outside the rail embankment footprint indicated higher $C_c$ values. A recompression index ratio $C_r/C_c$ of 0.08 were adopted for design assessments.

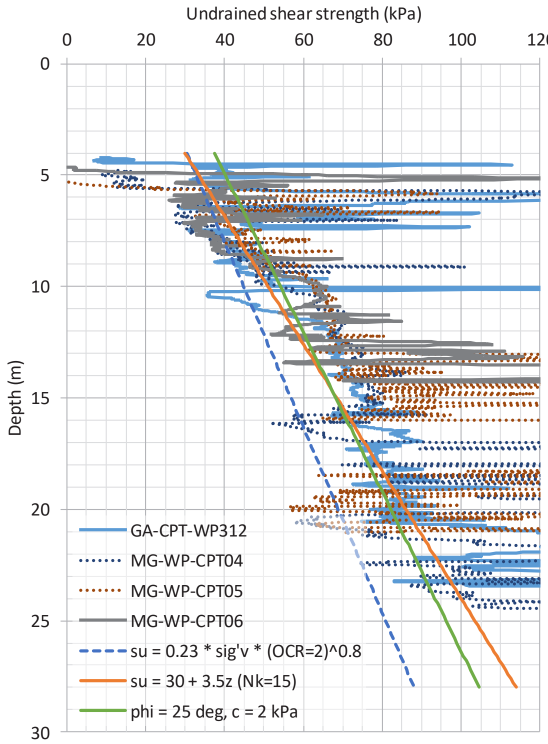


Figure 3. Overlay of CPT correlated undrained shear strength profiles (dotted lines) and profiles adopted for design (solid lines).

Out of four consolidation tests carried out, only one corresponding to borehole location was within the footprint of the existing embankment. However, the test result was considered likely to be representative of the soil behaviour which had been consolidated under the existing embankment. The design stiffness parameters adopted for the CIS were $E_{50}^{ref}$ of 2 MPa and $E_{ur}^{ref}$ of 19 MPa, which correspond to $C_c$ of 0.3 and $C_r$ of 0.024 at a reference stress of 100 kPa.

An "at rest" earth pressure coefficient of 1.0 for the CIS unit, modelled using a Hardening Soil model, was considered for the generation of initial stress conditions prior to staging of the rail embankment fill construction, during numerical assessments.

Based on the results presented in King et al. (2016), the design permeability values of $10^{-9}$ m/s in the vertical direction and $10^{-8}$ m/s in the horizontal direction for the CIS layer, with sample void ratios between 1.2 and 1.5, were considered on a cautiously conservative side for the retaining wall assessment.

### 3.3 *Soil-grout bond strength in embankment fill*

The soil nail supported temporary sheet pile wall design considered that the soil nails are installed within the rail embankment fill material. The results of soil nail pull-out tests carried out on test nails in accordance with VicRoads Specification Section 683 (VicRoads 2018) at three nominated test locations (minimum required), indicated that a nominal allowable grout-soil bond strength of 30 kPa and an ultimate of 50 kPa could be achieved for a 150 mm diameter air-flush drilled hole in the existing embankment fill material. These bond strengths were adopted for design.

## 4 ASSESSMENT METHODOLOGY

The geotechnical assessment of the retaining wall undertaken comprise an ultimate limit state (ULS) assessment of the overall stability of the structure in terms of rotation and global failure and a serviceability limit state (SLS) assessment of the wall displacements and the associated ground movements at the adjacent track level. Numerical analyses using PLAXIS 2D (version 2018) and PLAXIS 3D (version 2018.01) geotechnical finite element programs were carried out for the wall stability and displacement assessments. The assessments of localised deeper excavations, such as those required for the drainage pipe installation etc., were carried out using three-dimensional numerical analyses.

Step-by-step construction phases were considered in the numerical analyses for the ground and wall displacement evaluations, which included sheet pile rig working platform extension over the existing embankment, excavation of the rail embankment to soil nail support levels, and subsequent excavation to the underside of the working platform required for the pile driving works after installation of soil nail supports where necessary.

## 4.1 *Design Criteria*

### 4.1.1 *ULS assessment*

A minimum factor of safety (FoS) was calculated using the phi-c strength reduction method in PLAXIS. It should be noted that this method reduces the strength of the soil in both the active and passive sides of a retaining wall. This procedure yields conservative results compared to the conventional retaining wall design approaches, in which only the passive-side strength is reduced. A minimum FoS of 1.3 based on the phi-c strength reduction method in PLAXIS was thus adopted against global instability. The calculated FoS against global instability was also cross-checked using a beam and soil spring analytical program, WALLAP.

### 4.1.2 *SLS assessment*

The adjacent Sunbury railway tracks were required to be in operational during most of the construction period. Allowable track movements for a design track speed of 130 km/h was up to 20 mm that would not require any mandatory corrective actions for safe line operations as per the MTM document L2-TRK-PRO-054 (Track Procedure – Track Geometry Maintenance Tolerances). For the wall assessment, these allowable values were considered to be additional displacements, from the existing track conditions, resulting due to the excavation works on the passive side of the temporary sheet pile wall.

An observational design approach was adopted (also refer Table 2) and monitoring of the retaining wall performance by monitoring the wall and railway track movements was carried out for the duration of the construction as well as throughout the design life (effectively six months) of the wall.

The actual allowable track speed of the section within the temporary works area was understood to be 65 km/h, for which the displacement tolerances were higher than those adopted for design assessment. But following discussions, RIA was in favour of undertaking an observational approach in monitoring the wall displacement to validate the predicted wall displacements based on the allowable track movements for the 130 km/h train speed. A separate track and asset monitoring program in line with the MTM Permit to Disturb Track (PTDT) was also specified and undertaken by RIA in parallel with the wall monitoring.

## 4.2 *Design Loads*

The challenges posed by the proximity of the temporary structures to the tracks and other railway assets required careful consideration of the design surcharge loads for two-dimensional assessments.

### 4.2.1 *Rail traffic load*

The existing Sunbury line is classified as a 245LA rail loading. There is no clear guidance on the adoption of the three-dimensional rail traffic loads prescribed in AS 5100.2:2017 for a plane strain analysis, such as the one carried out for the current temporary works design, and some engineering judgement is required.

We had carried out a three-dimensional numerical assessment of the rail traffic loads to estimate an average bearing pressure generated beneath the rail sleepers, for other rail projects within Victoria, by incorporating into the model the axle and load layouts set out in AS 5100.2:2017 as well as the rails, sleepers, ballast. The results indicated that the axle loads are distributed over a considerable section of the track length. It could be observed that whilst there was a trend of concentrated loads near the edge of the sleepers, the average stress over the footprint of a sleeper width of 2.6 m along the extent of the axle layout could be reasonably approximated to a uniform surcharge load. Based on the above and considering the extent of each four-axle group is 4.5 m and the spacing between each four-axle group can be 12 m or more as well as the rail loads being transient/short-term in nature, it was considered reasonable to adopt a uniform surcharge load of 40 kPa for a 245LA train for use in a two-dimensional plane strain analysis. When comparing the results from the three-dimensional model above with a two-dimensional model applying a uniform 40 kPa surcharge train loading at the underside level of rail sleepers, the calculated displacements and stresses were found to be higher for the case of the two-dimensional model. The latter was adopted for the design.

In addition, a critical train impact loading of 1500 kN applied horizontally over a length of 2 m by 0.5 m normal to the wall (i.e. perpendicular to the sheet pile wall alignment) for geotechnical assessment of wall stability to meet an overall wall stability with a factor of safety of greater than unity.

### 4.2.2 *Equivalent infinite strip load*

For a two-dimensional assessment of a temporary wall section which is subjected to loads due to the presence of a rail asset (such as a signal post or an OHLE gantry), a point load or a line load of limited extent was converted to an equivalent infinite line load acting on the back of a wall based on the method presented in the guidance document CIRIA C760 (2017).

# 5 ASSESSMENT RESULTS

The SLS conditions were found to be the governing design conditions and the displacements were calculated based on numerical assessments. Maximum structural actions based on the numerical assessments were provided to structural engineers for checking of sheet pile structural capacities.

## 5.1 *Calculated Displacements*

Maximum track displacement of 20 mm was targeted in the design for both wall types. The wall displacement was found to be governed by, not only the retained wall height and the offset distances between the adjacent tracks or railway assets and the temporary wall, but also the expected embankment fill layer thickness between the base of excavation level and the underlying CIS layer.

The maximum retained height at the sheet pile wall portions supported by soil nails and buttressed by berms was about 3.2 m, with a maximum of up to 3.7 m locally which included allowances for up to 0.8 m over-excavations such as to facilitate installations of drainage pipes below the underside level of the decline structure. The calculated maximum horizontal displacement at the wall top level for the soil nail supported and with berm wall portion was about 25 mm. The maximum retained height at the cantilevered sheet pile wall portions was 2.5 m including allowances for up to 0.5 m over-excavation below the underside of the decline structure. The calculated maximum horizontal displacement at the cantilevered wall top level was about 20 mm.

The calculated displacements at the locally deeper excavation sections or where railway assets were at proximity to the temporary wall were generally less than or equal to the displacements calculated for the wall portions with maximum retained heights.

In addition, the settlement along the track length was expected to be relatively uniform and the calculated differential settlement was expected to be less than 5 mm at the underside of the rail sleepers of the Sunbury line tracks. Separate assessments for creep settlements were not carried out given the short-term (up to six months) nature of the proposed construction period till the piling platform and RIA undertook re-grading (tamping) on the adjacent rail tracks periodically as a part of the PTDT. It is noted that the creep rate of the CIS in the area could be expected to be in the range of 5 mm to 10 mm per year. The creep settlement was expected to result in a relatively uniform settlement and not expected to cause onerous differential settlement in the track transverse direction.

## 6 MONITORING REGIME AND RESULTS

The accuracy of the calculated displacements in a typical geotechnical analysis is highly dependent on the variability of the materials encountered. Therefore, RIA decided to adopt an observational design approach given that the rail embankment fill material was highly variable in composition, and the strength and stiffness of the underlying CIS would be highly sensitive to the changes in the in situ stress regimes. The following sub-sections discusses the monitoring regime adopted as a part of the observational design approach.

### 6.1 *Monitoring Regime*

Survey monitoring points (prisms) were set-up along the top and mid-point of the wall at 5 m longitudinal intervals along the wall (Figure 4). Prisms were also installed at the lower section of the wall near the base of excavation for the cantilevered wall.

Figure 4. Temporary sheet pile wall portion supported by soil nails and buttressed with berm at the piling platform level. Prisms installed at wall top and mid-height for displacement monitoring.

Monitoring of the wall displacements were undertaken in conjunction with track monitoring (conducted separately by RIA) immediately after the excavation reached the first row of nail level with the subsequent prisms to be installed at the middle and base of the exposed wall height. These monitoring points were used to monitor displacements of the walls and the tracks to compare the measured values with the predicted values, forming a part of the monitoring program which included a Trigger Action Response Plan (TARP) and associated alerts and actions. An example of TARP is presented in Table 2.

Table 2. Recommended actions corresponding to measured wall-top horizontal displacements

| *Trigger levels* | *Measured displacement* | *Recommended actions* |
|---|---|---|
| **0** (Review and continue) | Less than 80% calculated displacement | Review wall displacement data at excavation to wall mid-height level. |
| **1** (Alarm level) | 80% calculated displacement | Increase frequency of monitoring and notify temporary works designer, RIA and MTM. Compare track movement data |
| **2** (Work suspension level) | 100% calculated displacement | Compare track movement data and if trend of movement is increasing then excavation ceases immediately and backfill against the retaining wall or install additional/ second row of nails. . Notify temporary works designer and MTM. |

The survey monitoring for the wall was carried out daily during construction and then reduced to weekly monitoring for another three months, and monthly thereafter. A baseline reading for the wall displacements was established immediately after the survey prisms were installed on the wall-top, prior to excavation. Survey points in the railway embankment beyond the temporary wall extents were also carried out to assess the effects of background creep settlement of the CIS layer.

A daily construction activity record was documented by RIA as a part of the track monitoring program and submitted to the temporary works designer to facilitate the review of wall monitoring data.

### 6.2 *Monitoring Results*

Figure 5 shows that a uniform lateral displacement at the top of wall (both cantilevered and soil nail supported walls) of up to about 25 mm had occurred when the excavation reached the base level (underside) of piling platform at end of May 2019. The wall movement progressively increased to 30 mm over the next two months while the piling works and construction of the approach slab was carried out. The wall continued to creep and stabilised at 35 mm (maximum) for the next several months.

The lateral wall displacements of cantilevered and soil nail supported walls are shown separately in Figure 6. The figures also show the wall displacement at the middle and base of the

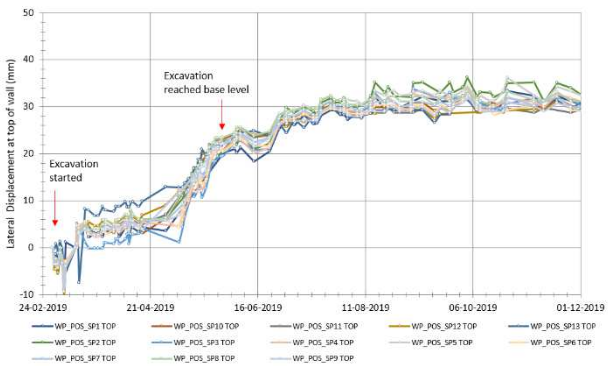


Figure 5. Measured horizontal displacements at top of wall.

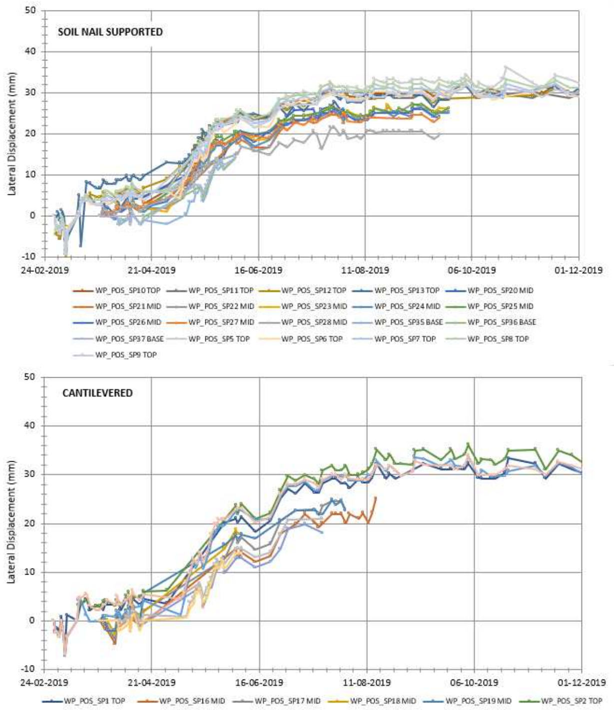


Figure 6. Wall displacement of cantilevered and soil nail supported walls with survey data at middle and base prisms.

wall. The middle and base of the wall have also displaced up to 20 mm and 25 mm for cantilevered and soil nail supported wall, respectively.

Track monitoring data obtained by RIA indicated that track settlements of 10 mm to 20 mm were recorded up to the end of August, before re-grading of tracks was undertaken by filling and tamping of ballast. Similar to the lateral wall displacement, a progressive creep settlement of up to 5 mm was observed from the time when excavation reached the maximum depth for a period of two months. Based on the recorded track settlement, we can assume that the ground settlement can be up to 75% of the wall lateral displacement.

In line with the predetermined TARP and the associated alerts applicable, daily monitoring data were provided by the RIA team to the temporary works design team for immediate review. Even though the measured lateral wall displacement was 10 mm to 15 mm higher than the predicted values, the measured track settlement was within the predicted value of 20 mm during construction. Therefore, the displacements were considered to be satisfactory from maintenance perspective set by MTM. It is noted that an option of installation of additional row of soil nails was available as a contingency measure, if any additional displacement were assessed to be unacceptable.

## 7 CONCLUSIONS

In geotechnical problems, the actual displacements can exceed those assessed during the design, which can be due to a number of reasons, and mainly due to variations in the ground conditions. Regular monitoring of displacements and critical review of the displacements observed, their impact and associated risks would provide further confidence in the design and to proceed with the work.

In the case history presented in this paper, although the measured displacements were higher than those predicted during the design, with the observational approach adopted, a satisfactory outcome has been achieved with the construction works completed as planned. Timely review and reporting for the monitoring data played a pivotal role in the success of the observational approach adopted. The wall displacement and track monitoring data were able to be provided from RIA to temporary design team for immediate review to assess the performance of the wall.

The sheet pile wall has achieved its primary objective of ensuring the wall deflections and track movements were maintained to an acceptable level during the most critical construction stage where the temporary excavation was at the deepest level.

## 8 ACKNOWLEDGEMENTS


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