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Geotechnical design and performance of a rock excavation under a cut and cover tunnel structure in Sydney

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ABSTRACT: Several major motorway tunnel projects have been completed in Sydney over the last ten years. These projects required complex interfaces and construction staging of main structures including cut and cover tunnel structures. This paper presents the geotechnical design aspects and performance of the top-down rock excavation under a cut and cover tunnel structure, which permanent concrete works were completed as part of an earlier project stage. The excavation works comprised excavation to a maximum depth of 12.5 m in Ashfield Shale. The instrumentation and monitoring data during and after construction have been analysed and are discussed in this paper. A comparison between observed displacements and design predictions is presented, while considering their correlation with the geotechnical design and numerical model assumptions.

1 INTRODUCTION

The project comprised a new major motorway in Sydney which included a cut and cover tunnel structure for the tunnel exit and entry ramps which was designed and constructed in two stages by two different contractors. The Stage 1 project scope included the detailed design of soldier piles, locally with anchors, and headstocks under a new bridge. As part of the Stage 1 work, the excavation progressed to top of bedrock or below the strut beam. Stage 2 project scope comprised the detailed design of the excavation beneath the cut and cover tunnel structure, including excavation rock support, as well as the construction of the motorway to the finished surface level.

This paper presents the geotechnical design of the excavation with a focus on the assessment of lateral displacements. The predicted displacement values are compared with those measured by the instrumentation installed in the excavation.

1.1 *Stage 1 Cut and cover*

The existing cut and cover tunnel structure comprised a roof structure, with super-T girders and a cast in-situ composite deck, which was supported on three rows of soldier piles. A bridge deck with road live traffic was located on top of the cut and cover tunnel structure. A photograph of the existing cut and cover tunnel structure, prior to the rock excavation works, is shown on Figure 1. The general arrangement plan and a typical cross section of the cut and cover tunnel structure are shown on Figure 2 and Figure 3, respectively.

The structure foundations comprised:

- The roof abutments, i.e. the western and eastern retaining walls, and the tunnel portal retaining walls. The soldier piles that formed the retaining walls had a diameter of 900 mm and a spacing of 2.3 m. The portion of the western retaining wall at the deepest north end of the excavation utilised ground anchors for additional lateral support.
- The central pier between the two carriageways, comprised 900 mm diameter piles with a spacing that varied between 2.7 m and 5.5 m.

Figure 1 Existing cut and cover tunnel structure (southern end, photograph taken prior to stage 2 rock excavation works)

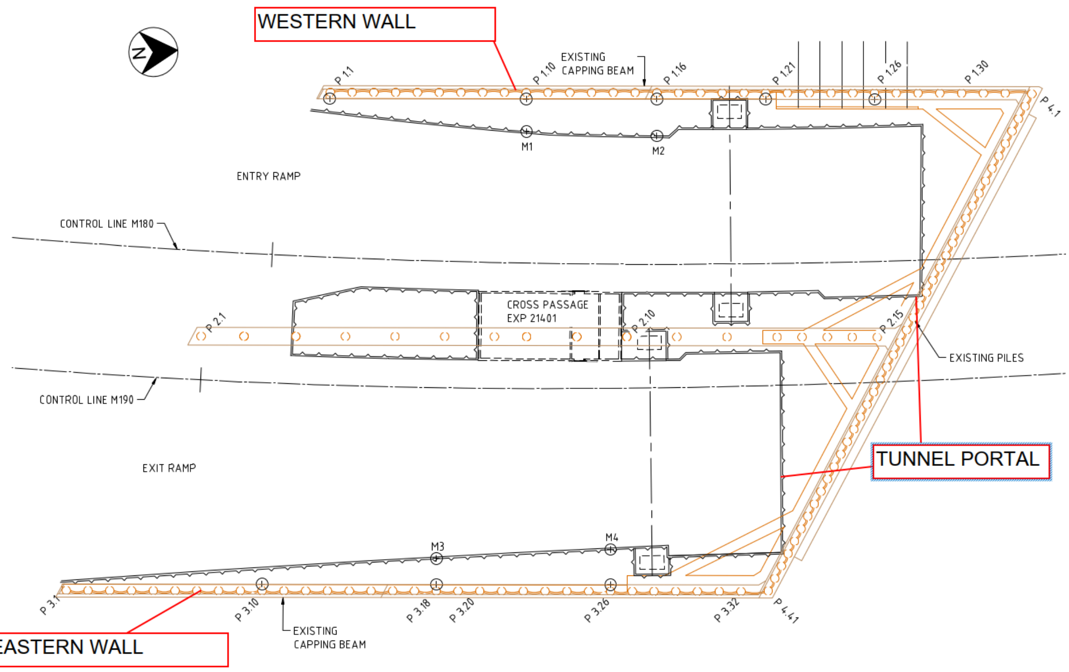

Figure 2 General arrangement plan showing location of survey points

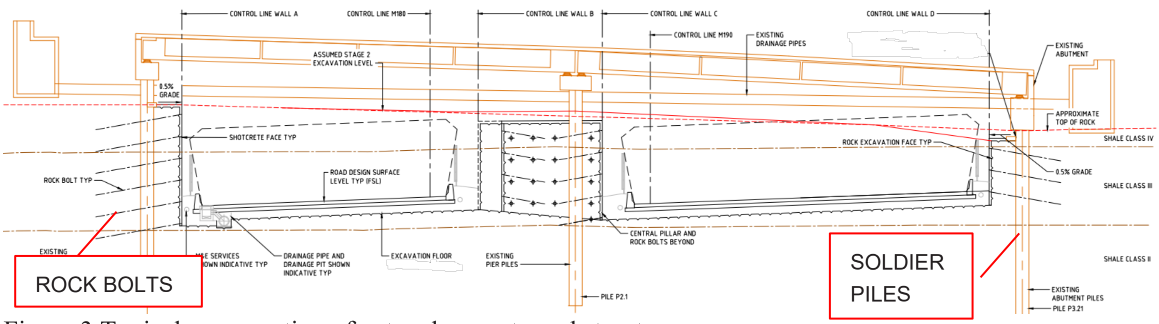

Figure 3 Typical cross section of cut and cover tunnel structure

1.2 *Stage 2 Rock excavation*

The Stage 2 rock excavation works comprised advancement of the excavation works below the top of rock to the underside of the motorway pavement, extending approximately 60 m from the southern end of the cut and cover tunnel structure to the tunnel portals. The detailed design scope comprised the geotechnical design of the rock excavation under the existing structure, and the structural assessment of the elements of the existing structure that were affected by the excavation. The adopted design solution comprised pattern rock bolting and the surface of the excavation walls covered with shotcrete, which is a typical design solution for deep excavations in Ashfield Shale. Pattern rock bolting was specified at the abutment/retaining walls and central pier to ensure stability against a wedge or planar failure case based on regional and site-specific defect data. Rock bolts of 22 mm in diameter were adopted with vertical spacing between 1.0 m and 1.5 m, and horizontal spacing between 0.6 m and 1.7 m.

2 GROUND MODEL AND GROUNDWATER CONDITIONS

The ground model was developed based on ground investigations carried out as part of Stage 1 and Stage 2 designs. The ground conditions comprised uncontrolled fill of up to 2 m thick, overlying a relatively shallow residual soil overlying Ashfield Shale bedrock. The fill and part of the residual soil/extremely weathered material had been excavated as part of the Stage 1 project. The Stage 2 excavation was carried out predominantly in Ashfield Shale. The shale was assessed as highly weathered and of very low to low strength in the upper 2 m (Class IV, Pells 2004). Below this, the weathering of the shale improved with depth to be typically moderately weathered and of medium strength (Class III or better, Pells 2004). The Ashfield Shale geotechnical design parameters are presented in Table 1.

Based on the available groundwater data, perched water pressure at the top of the residual soil was considered in the geotechnical design.

Table 1 Ashfield Shale Geotechnical design parameters for the Stage 2 rock excavation

Parameters	Class IV *	Class III *	Class II *
Unit weight, γ (kN/m3)	23	24	24
Unconfined Compressive Strength, UCS(MPa)	2	6	12
Rockmass Elastic Modulus, E (MPa)	250	500	1,200
mi (Material constant)	8	8	8
GSI (Geological Strength Index)	35	50	60
Poisson's Ratio, ν	0.3	0.25	0.2
Ultimate Bond Stress (kPa)	300	600	900

Parameters	Class IV *	Class III *	Class II *
Horizontal to vertical stress ratio	1.4	1.4	1.4

*Pells (2004)

3 GEOTECHNICAL ANALYSIS

Soil structure analyses were carried out to assess the deformations of the excavation during construction and in the long-term using finite element program Plaxis 2D. The deflections of the retaining walls at the northern end of the abutment piles and at the tunnel portal piles, where retaining walls are restrained by the capping beam and an intermediate strut, were assessed using Plaxis 3D.

Seven representative sections of the eastern and western abutment/retaining walls were analysed using Plaxis 2D. The representative sections, their excavation depth (to underside of pavement) and rock ledge width, are presented in Table 2.

A typical Plaxis 2D model is shown on Figure 4. Analysis was not carried out at the central pier where the piles were predominantly vertically loaded and the excavation did not affect their design loading conditions.

The 3D model is shown on Figure 5. The same construction sequence and assumptions were adopted in the Plaxis 2D and Plaxis 3D analysis.

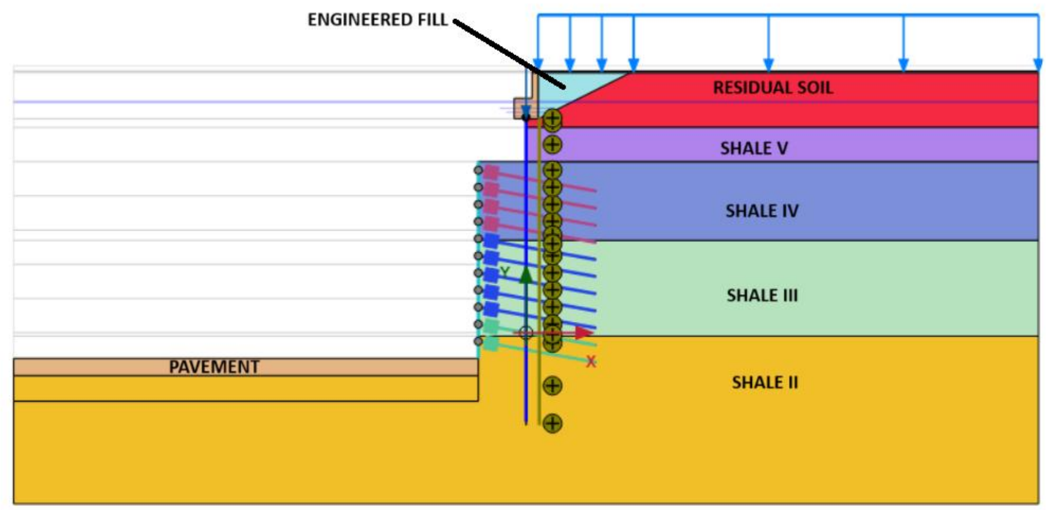

Figure 4 Plaxis 2D model

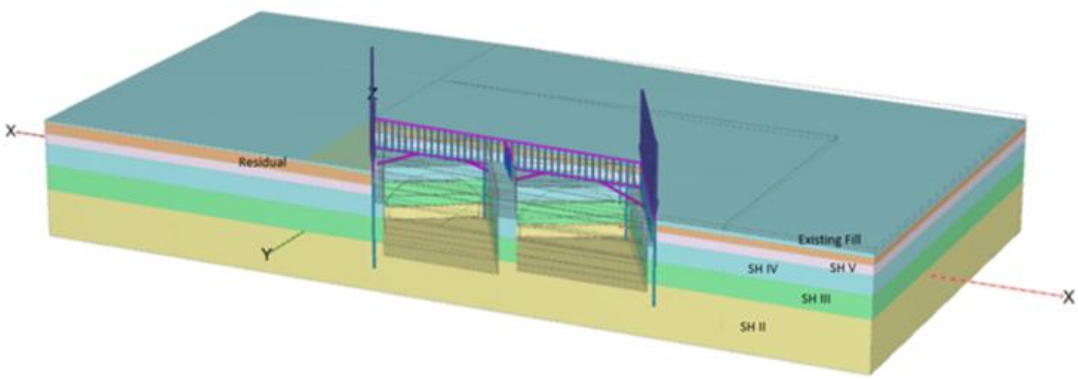

Figure 5 Plaxis 3D model

3.1 *Main inputs and assumptions*

The main inputs and assumptions of the geotechnical analysis are listed below:

- As the deck was simply supported on the abutments/ capping beam, no connection was modelled between the retaining walls and the deck.
- Axial load from the upper structure was applied on the piles based on the values reported on the Stage 1 as constructed drawings.
- Displacements were reset to zero at the end of Stage 1 works so that estimated displacements corresponded to Stage 2 works only.

For the additional Stage 2 excavation works the following construction sequence was adopted:

- Excavation to top of Class IV Shale;
- Excavation from top of Class IV Shale with 2 m deep lifts;
- Application of shotcrete and installation of rock bolts;
- Repetition of the same procedure using 2 m lifts until the final excavation level; and
- Construction of pavement.

3.2 *Predicted deflections*

The predicted pile head deflections based on the geotechnical analysis are presented in Table 2. The predicted values were below the allowable limiting value of 30 mm, taking into account also the Stage 1 deflections that were estimated to vary between 3 mm and 5 mm. The deflections due to the Stage 2 rock excavation varied between 1.1 mm and 2.4 mm per meter of rock excavation. The lower values correspond to a higher percentage of the rock excavation in better quality rock (Class III or better), typically on the eastern abutment. The maximum value of 2.4 mm/m was estimated at pile P1.2. The values of deflections per meter of rock excavation are consistent with the typical ranges reported for deep excavations in Ashfield Shale (e.g. Salcher et al. 2024, Oliveira & Chan 2016, Toh & Mostyn 2015, Walker 2004, Hewitt et al. 1999).

Table 2 Representative sections for geotechnical analysis and predicted and observed deflections

Pile number*	Stage 2 Excavation Depth (m)	Excavated material thickness (m) per rock class**	Rock Ledge Width (m)	Total retained height (m)	Predicted Pile Head Deflection due to Stage 2 excavation		Observed Pile Head Deflection due to Stage 2 excavation	
					(mm)	Ratio of deflection to excavation depth (mm/m)	(mm)	Ratio of deflection to excavation depth (mm/m)
P1.2 (West Abutment)	7.4	3.2/ 4.2/ 0.0	1.2	12.5	18	2.4	7	0.9
P1.9 (Western Abutment)	9.7	3.8/ 5.9/ 0.0	3.2	14.2	18	1.9	12	1.2
P1.16 (Western Abutment)	11.3	4.4/ 5.2/ 1.7	3.7	15.7	16	1.4	NA***	NA***

P1.21 (Western Abutment)	12.5	4.6/ 5.6/ 2.3	2.8	17.8	19	1.5	25	2.0
P3.9 (Eastern Abutment)	5.7	0.8/ 3.2/ 1.7	2.0	9.7	6	1.1	5	0.9
P3.15 (Eastern Abutment)	6.3	1.0/ 3.1/ 2.2	2.8	11.9	10	1.6	8	1.3
P3.26 (Eastern Abutment)	10.7	3.7/ 6.3/ 0.7	3.9	16.0	15	1.4	14	1.3

*Pile locations are shown on Figure 2
**Excavated material thickness (m) per rock class: Ashfield Shale Class IV/ Class III/ Class II
***Not available

4 COMPARISON OF MEASURED AND PREDICTED DEFLECTIONS

Instrumentation for the Stage 2 excavation works comprised optical prisms installed at the pile head (‘P’) and at the top of rock ledge (‘M’) at selected sections of the abutment walls. The location of survey points is shown on Figure 2.

The predicted and measured deflections are summarised in Table 2. The measured deflection response during and after excavation is shown at selected monitoring points of the western and eastern abutment on Figure 6 to Figure 8. The excavation works of the western carriageway started first, with a direction south to north, followed by the excavation of the eastern carriageway with a direction north to south. The start and end dates of the excavation for each carriageway are also shown on the figures.

The following observations are made:

- Overall, the maximum measured deflection values are in good agreement with the predicted values. The accuracy of the predicted deflections is typically between 5% and 25% and can be classified based on the prediction quality classification as ‘excellent’ to ‘fair’ (Morgenstern, 2000). The main exception is survey point location P1.2 where the measured values may be erroneous.
- At P1.21, deflections exceeded the maximum predicted value but remained within the allowable deflection limit. This is attributed to the position of the pile which was the last pile at the deepest section of the excavation that was not supported by ground anchors. Sudden transitions between anchored and unsupported areas may trigger increased movement. At P1.26, where the piles were supported with ground anchors, the measured deflection is in good agreement with the predicted value.
- At the eastern abutment, the deflections at P3.26 started increasing at the start of the excavation until approximately 20 June. After that, deflections are shown to stabilise prior to the end of the excavation works. This response is attributed to the lateral displacement at P3.26 and at the respective pile (P1.21) on the western abutment closing the abutment gap and causing a propping effect on the top of P3.26.
- Based on the measured deflections of the western abutment, deflections varied between 0.9 mm and 2.0 mm per meter of rock excavation, that is slightly less than the predicted values.

- Where there were survey points placed at the top of rock ledge, the measured deflection response was consistent with that placed on the respective pile behind the rock ledge.

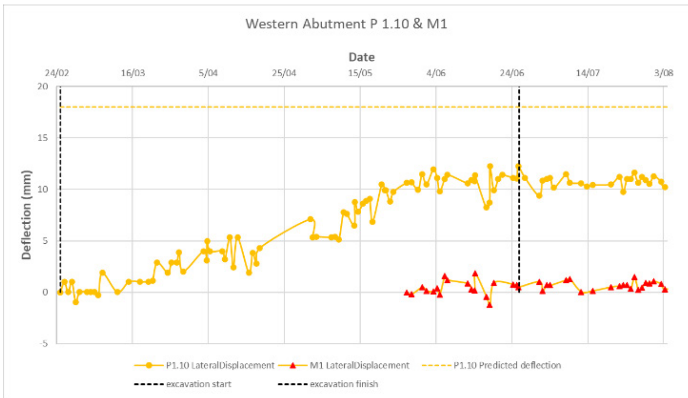

Figure 6 Western Abutment P1.10 measured and predicted lateral displacements

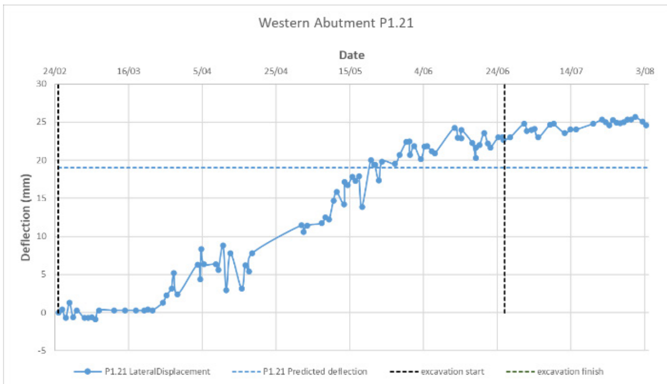

Figure 7 Western Abutment P1.21 measured and predicted lateral displacements

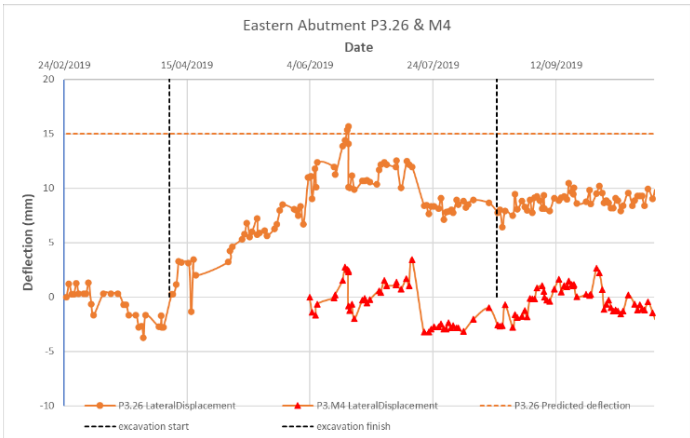

Figure 8 Eastern Abutment P3.26 measured and predicted lateral displacements

5 CONCLUSIONS

The adopted design solution for a deep excavation in Ashfield Shale beneath an existing cut and cover tunnel structure, comprising soldier piles with pattern rock bolting and shotcrete on the wall surface, performed satisfactorily. Deflections stabilised after the end of excavation and remained below the allowable limiting value.

Based on the comparison of the predicted deflection values of the excavation retaining walls with those measured by the instrumentation, the following conclusions were made:

- Deflections are shown to be a function of excavation depth and rock quality (strength and defects).
- The measured deflections varied between 0.9 mm and 2.0 mm per meter of rock excavation, and were slightly lower than the predicted values of 1.1 mm and 2.4 mm per meter. Lower values correspond to higher percentage of the rock excavation in better quality rock (Class III or better).
- Sudden transitions between anchored and unsupported areas of the excavation may have triggered increased deflection at one survey point location compared to that predicted by the 2D finite element geotechnical analysis.
- The accuracy of predictions was generally classified based on the prediction quality classification as ‘excellent’ to ‘fair’ (Morgenstern, 2000)

6 REFERENCES

Hewitt, P.B., McQueen, L.B. and Davies, P.R., 1999. Genting Centre, Sydney-Deep Excavation Adjacent to Railway Tunnels. In Australia-New Zealand Conference on Geomechanics (8th: 1999: Hobart, Tas.) 611-617. https://www.issmge.org/uploads/publications/89/88/8ANZ_075.pdf

Morgenstern, N.R., 2000. Performance in geotechnical practice. Hkie Transactions, 7(2), 2-15. https://www.tandfonline.com/doi/abs/10.1080/1023697X.2000.10667819

Pells, P.J.N., 2004. Substance and mass properties for the design of engineering, structures in the Hawkesbury sandstone, Australian Geomechanics journal, 39 https://geomechanics.org.au/admin/wp-content/uploads/2015/03/39_3_1.pdf

Toh, J. and Mostyn, G., 2015. Ground surface deformation adjacent to a deep excavation in shale. Australian Geomechanics journal, 50(2), 57-67. https://www.australiangeomechanics.org/papers/ground-surface-deformation-adjacent-to-a-deep-excavation-in-shale/

Oliveira, D.A.F. and Chan, G., 2017. Ground control for a deep basement excavation in Sydney’s GPO fault zone. In Proceedings of the 20th Australian Geomechanics Society Sydney Chapter Symposium, Sydney. https://www.australiangeomechanics.org/papers/ground-control-for-a-deep-basement-excavation-in-sydneys-gpo-fault-zone/

Walker, B.F., 2004. Stress relief on hillsides and hillside excavations. Australian Geomechanics journal,, 39(3), 59-72. https://australiangeomechanics.org/admin/wp-content/uploads/2015/03/39_3_5.pdf