DEEP EXCAVATIONS IN SOFT CLAY ON BRISBANE AIRPORT LINK PROJECT

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ABSTRACT

This paper presents a case history of a 22 m deep excavation in deep soft clay on the $5.6 billion Brisbane Airport Link Project, one of the biggest infrastructure projects in Australia. A 25m deep cut and cover tunnel was constructed using a bottom up sequence to connect twin driven TBM tunnels at Toombul. Diaphragm walls with two layers of slabs and two layers of horizontal steel struts were adopted to support the deep excavation and to control the ground movements and ground water drawdown in order to ensure a safe excavation and to protect the adjacent existing residential buildings, utilities and North Coast Railway. Two-dimensional (2-D) finite element analyses were conducted during detailed design to predict the retaining wall displacement, ground settlement behind the wall and pore water pressure variation during construction and in the long term. Inclinometers, settlement markers and piezometers were installed to monitor the behavior of the diaphragm walls and the retained ground, pore water pressures during construction. The monitoring data is compared with the design predictions and comments offered on the success of the deep excavation and accuracy of predictions.

Keywords: deep excavation, cut and cover tunnel, soft clay, 2-D FE analysis, diaphragm walls, inclinometer, piezometers

1 INTRODUCTION

A 22-25 m deep cut and cover tunnel was constructed in Toombul as part of the Brisbane Airport Link project, one of the largest infrastructure projects in Australia to date. This cut and cover tunnel connects the mainline East-West bored tunnels with Sandgate Road and East-West Arterial Road. Construction of this section commenced in mid of 2009 and was completed in March 2010. Zhang and Zhu (2011) discussed the corner effects of the portal wall for the Tunnel Boring Machine (TBM) launching shaft of the bored tunnels at the same site.

Figure 1 shows the plan view of this cut and cover tunnel and Figure 2 shows the cross section of the tunnels at Chainage 54595 that this paper will discuss about. The cut-and-cover tunnels are approximately 100 m long, 40 m wide and 22–25 m deep. This paper presents finite element (FE) analyses and monitoring results on the cut and cover tunnel which is denoted as CC410.

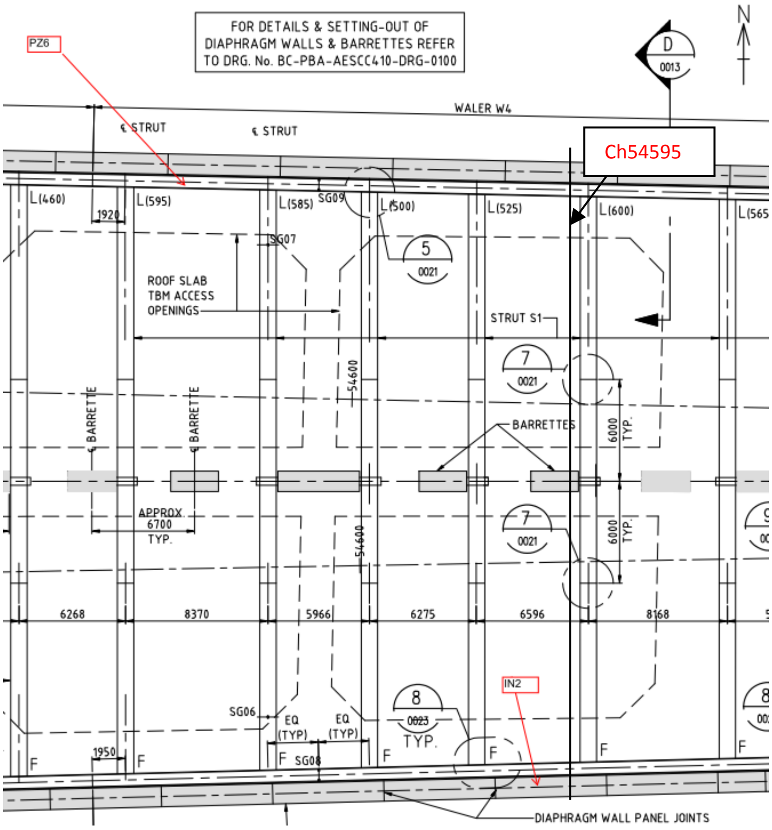

Figure 1: Cut and cover tunnel at CC410 - Plan view

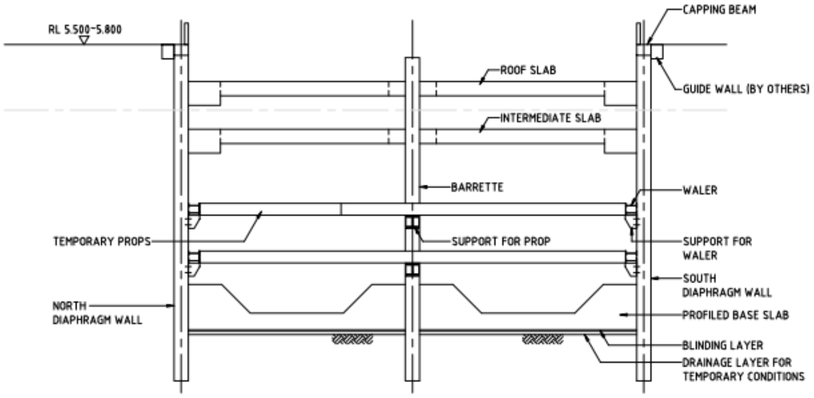

Figure 2: Cut and cover tunnel at CC410 – cross section (Ch54595)

2 GROUND CONDITIONS AND SITE CONSTRAINTS

The existing ground level at the cut and cover tunnel is at approximately RL4.0 Australian Height Datum (AHD); the groundwater level is at RL2.5 AHD. A temporary platform was built on top of the existing ground level to RL5.6 for the construction of diaphragm walls.

As shown in Figure 3, the subsurface conditions consist of three units: topsoil and fill, Quaternary alluvium and Tingalpa Formation. The material type and thickness of the three units are:

1. Unit 1: RL5.6 m to RL5.0 m, medium-dense sandy gravel;
2. Unit 2: RL5.0 m to RL4.0 m, desiccated firm clay;

   RL4.0 m to RL-7.5 m, very soft to soft clay and loose clayey sand;

   RL-7.5 m to RL-9.5 m, medium dense gravelly sand and clayey gravel;

RL-9.5 m to RL-16.5 m stiff to hard silty clay;

3. Unit 3: RL-16.5 m to RL-19.5 m, extremely low strength (ELS) siltstone; below RL-19.5 m, low to medium strength (MS) siltstone.

The compressible clays had cone resistance (qc) of 0.2–0.8 MPa, moisture content of 60–80% and standard penetration test (SPT) N value of 0–4.

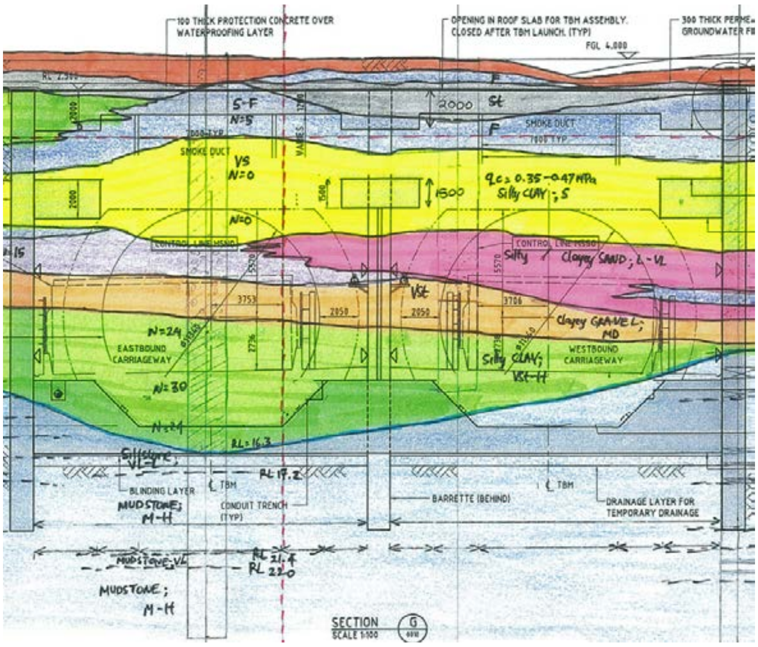

Figure 3: interpreted ground profile at the Ch54595

3 RETAINING SYSTEM AND CONSTRUCTION SEQUENCE

A robust retaining system for this section was selected at the start of the design to: a) limit ground movements to protect the adjacent buildings, structures and services; b) minimise groundwater ingress through the retaining wall into the tunnel; c) serve as the TBM launching box and facilitate subsequent TBM tunnel construction. The retaining system comprised two diaphragm walls (26 m deep, 1.2 m thick) and four-layer horizontal supports of which the two upper layers are in a form of concrete slabs forming the permanent smoke duct in the complete tunnel and two lower layers of temporary cross-lot steel struts. The permanent roof slab is 1.2m thick and the intermediate slab is 1.5 m thick with large openings during construction for TBM and tunnel lining segments access. The two slabs were supported by barrettes in between of the two walls. The two-layer temporary struts consisted of 965 mm outer diameter steel circular hollow sections with 20 mm wall thickness.

A top-down sequence was adopted to limit the excavation induced ground movements. The construction sequence was as follows:

1. Backfill to RL5.6 m;
2. Install two diaphragm walls (Dwall) and barrette and apply surcharge behind two Dwalls
3. Excavate to RL0.5 m
4. Cast 1.2 m roof slab with opening at RL1.6 m
5. Excavate to RL-4.0 m
6. Install 1.5 m thick intermediate slab with opening at RL-3.2 m
7. Excavate to RL-7.5 m
8. Install temporary strut layer 1 at RL-5.7 m
9. Excavate to RL-11.1
10. Install temporary strut layer 2 at RL-9.3 m
11. Excavate to RL-16.7 m
12. Construct base slab
13. Remove temporary struts layer 2 and layer 1
14. Infill roof and intermediate slabs openings and change all materials to drained for long term condition

4 INSTRUMENTATION AND MONITORING

A comprehensive instrumentation monitoring (I&M) scheme was implemented through design and construction stage in order:

- To monitor the performance of the retaining system,
- To verify design prediction, and
- To control deep excavation safety

The instruments included in-wall inclinometers (IN), strain gauges (SG) on struts, piezometers, settlement markers in the ground and prisms on the walls. Refer to Figure 1 for details of these instruments. The results of the instruments around the analysed section, particularly the inclinometers and piezometers will be presented and discussed in later sections of this paper.

5 FINITE ELEMENT ANALYSIS AND DESIGN PREDICTIONS

Due to the significance of the project and risks related to deep excavations in urban areas, a comprehensive finite element (FE) analysis was adopted to predict the potential wall deflection, ground water variation and ground movements. A 2-D plane-strain FE analyses using Plaxis v8.6 was performed to model soil–structure interaction and assess the wall deflection and forces, ground settlement behind the wall, strut load and pore pressure variation during construction. The details of the FE analyses are discussed in the following sections.

5.1 ENGINEERING PROPERTIES

Mohr-Coulomb Model was used to model all the soils and rocks. For cohesive soils and extremely low to very low strength (ELS) rocks, two set of parameters were assumed: undrained total stress parameters were for the short term and drained effective stress parameters for the long term. For cohesionless soils, drained effective stress parameters were adopted in both long-term and short-term conditions. For medium strength rocks (MS), drained effective stress parameters were used for both short term and long term conditions. Table 1 presents the adopted design parameters.

For structural elements, the diaphragm wall, the roof and intermediate slabs were modelled as linear-elastic plate element. The struts were modelled as linear-elastic node-to-node anchor element.

Table 1: Engineering properties adopted in design

Soil/rock	Unit weight (kN/m^3)	Undrained modulus of elasticity Eu (MPa)	Undrained shear strength Cu (kPa)	Drained modulus of elasticity E'(MPa)	Drained Poisson ratio (ν')	Drained Cohesion (c') (kN/m^2)	Drained Friction angle (ϕ') (degrees)
1. Fill (gravel)	20	-	-	15	0.2	0.3	32
2. Firm clay	18	12	30	10	0.3	1	26
3. Very soft to soft clay	16	4.5	15	3.75	0.3	1	22
4. Stiff clay	20	30	65	25	0.3	1	28
5. Dense sand	20	-	-	50	0.3	1	35
6. ELS siltstone	22	75	250	60	0.3	5	27
7. MS siltstone	22	-	-	500	0.2	50	38

Notes: Undrained Poisson ratio is 0.495; undrained friction angle is 0.

Pore water pressures were considered reaching steady state seepage condition after each stage of excavation. They were calculated by the program assuming no groundwater drawdown on the active side and the groundwater level at the excavation level on the passive side. This resulted in a higher than hydrostatic pressure on the passive side and lower than hydrostatic pressure on the active side.

5.2 PREDICTIONS FROM FE ANALYSIS

Figure 4 shows the predicted wall deflection profiles. Figure 5(b) shows the predicted pore water pressures at the wall toe level. In comparison, monitored wall deflection (at IN2) and pore pressures (at PZ6) are also presented in Figure 4 and 5 respectively. PZ6 was installed at RL-19 m on the passive side of the retaining wall. The wall toe is at RL-20.2 m.

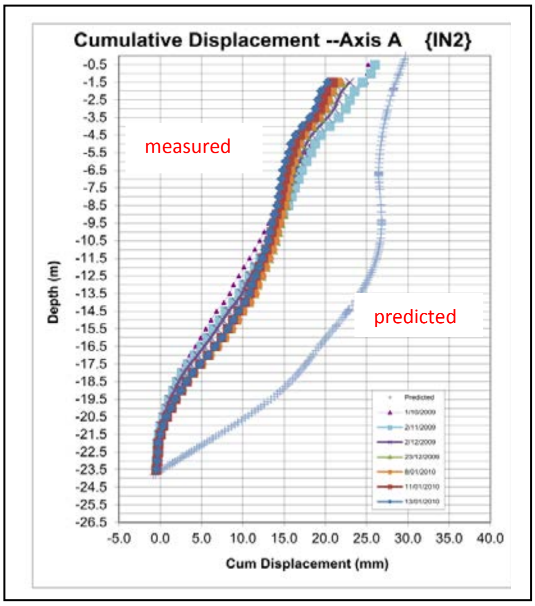

Figure 4: Predicted wall deflection after removal of 2 layers of struts

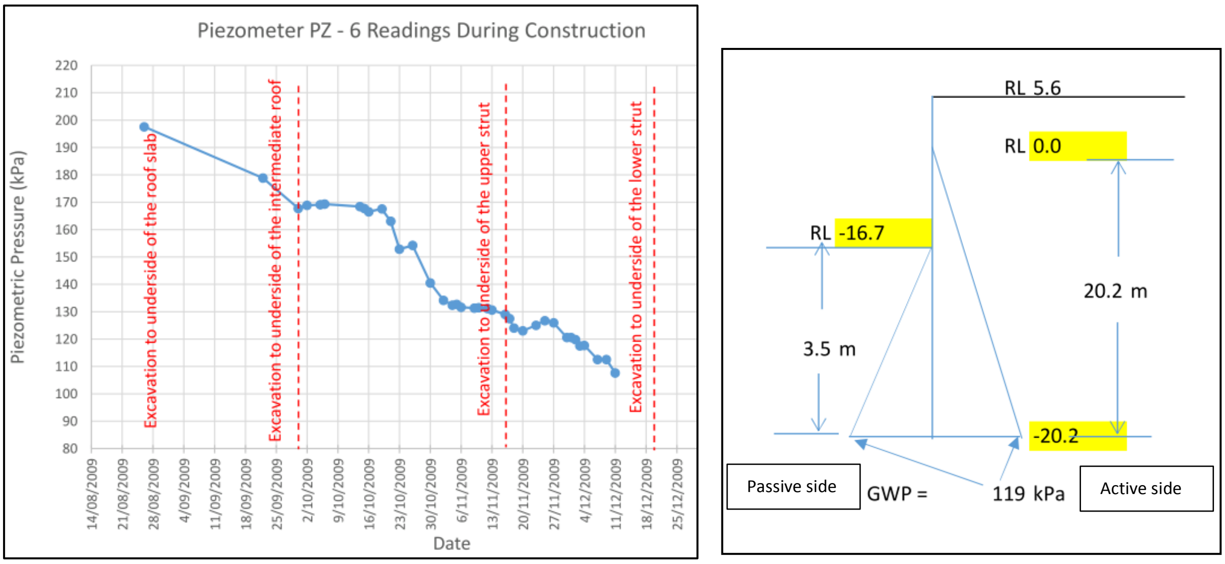

Figure 5: (a) Measured pore pressures at PZ6 (b) Design pore pressures at both sides of the wall

6 DISCUSSIONS

The following observations can be made from Figures 4 and 5.

- Deflections: the predicted wall lateral displacement is generally greater than the measured from inclinometer IN2 with a maximum over prediction of 10mm. This may be explained by the fact that the adopted constant ground moduli as part of the Mohr-Coulomb Model did not account for small strain effects which resulted in the input stiffness smaller than the actual and conservative estimations for ground movements. The prediction can be improved by using advanced soil model such as Hardening Soil Model in Plaxis that can automatically calculate higher modulus for unloading and reloading stress conditions, which is the dominant phenomenon in deep excavations problems.
- Pore water pressures: the measured pore pressure at PZ6 reduced from hydrostatic pressure of 200kPa before excavation to 107kPa when the excavation under strut layer 2 was in progress. The measurement stopped due to possible access issues or damage after the last reading of 107kPa. The design pore pressure at RL-19 (PZ6 level) is approximately 110 kPa (refer Figure 5(b)) which is close to the measured 107kPa, indicating the simplified method of evaluating water pressure around the retaining wall toe in a wide excavation as suggested in CIRIA C580 (2003) is appropriate in this design.

7 CONCLUSIONS AND RECOMMENDATIONS

A case history of a deep excavation for the cut and cover tunnels of Brisbane Airport Link project was presented. Monitored wall deflection is less than the predicted using finite element analysis, which indicates that the original design is conservative. A higher stiffness of soils as a result of small strain effects should be considered in further design. A reasonably good agreement was made between the measured and design pore pressure in front of the wall toe indicating that the simplified method of evaluating water pressure suggested in CIRIA C580 is adequate for a wide excavation where the steady state seepage condition is met.

8 ACKNOWLEDGEMENTS

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