Optimisation of the Underground Station Wall Design Through Ground-Structure Interaction Analysis

Chenhui Lee[1], Thomas Miller[2] and Jeff Hsi[3]

[1]Senior Associate, SMEC, Sydney, Australia
[2]Senior Engineer, SMEC, Sydney, Australia
[3]Technical Principal – Geotechnical, EIC Activities, Sydney, Australia
Corresponding author's E-mail: jeff.hsi@eicactiv.com

Abstract

The design and construction of the underground Sydney Metro Northwest stations is delivered under two separate contracts. The temporary support (10 years) for the excavation, part of the Tunnels and Station Civils (TSC) contract, typically comprises prestressed ground anchors and soldier pile walls. The permanent support (100 years) and station structure, part of the Operations Trains and Systems (OTS) contract, comprises reinforced concrete station walls propped by station slabs constructed in front of the temporary support. Due to the limited design life of the temporary support, the ground pressure will ultimately be transferred to the permanent station structure. Therefore, the design of the station permanent walls requires modelling of the ground stress history due to the excavation, followed by modelling of the load transfer from the temporary support onto the permanent walls. Capturing this, as well as the beneficial effects of the stress relaxation due to the initial excavation, is crucial in achieving an optimised design for the permanent walls. This paper presents the integrated ground-structure interaction approach adopted for the permanent walls design at Norwest Station, which resulted in optimised design ground pressures and station wall thicknesses.

Keywords: Retaining structures, Ground-Structure Interaction, Numerical methods and back analysis

1. BACKGROUND

The $8.3 billion Sydney Metro Northwest involves the construction of a 36km rapid transit link and eight new stations. As part of this project, five new underground stations will be constructed. One of the key challenges for these underground stations was to develop a holistic approach enabling the effects of intricate ground stress relaxation history and ground-structure interaction to be considered. This is a crucial step toward optimising the station wall thicknesses. Moreover, the weight of the wall will dictate the suitability of the segmental precast concrete panel construction technique, which is desirable from the constructability standpoint and has several advantages such as faster construction cycles, better quality control etc. As such, accurate prediction of ground pressures post-degradation of the temporary support and optimising the wall thicknesses play a pivotal role in the design.

2. NORWEST STATION

Norwest station is approximately 196m long, 22m wide and has an excavation depth of approximately 23m within the Permo-Triassic aged sedimentary rock comprised of Ashfield Shale and Hawkesbury Sandstone. The station is located at the corner of Norwest Boulevard and Brookhollow Avenue (Figure 1A). The station is a fully underground station and is accessible via the concourse level through escalators and lifts. The station has been designed to allow natural light to flow through several openings to the platform located 20m underground. This requirement increases the floor to floor heights and reduces the possibilities for internal propping of the station wall. Therefore, the internal structure needs to be carefully designed to ensure sufficient horizontal restraint is provided to the permanent station box walls.

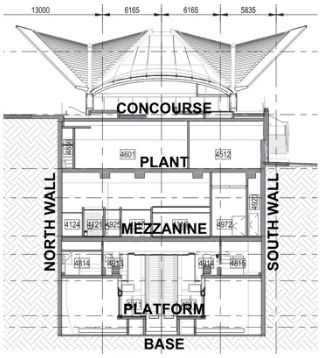

Figure 1A Figure 1B

Figure 1. Aerial photo (Figure 1A). Typical station structure layout (Figure 1B)

3. SUBSURFACE PROFILE

The subsurface profile of the Norwest Station is typically comprised of 2-3m of fill above Ashfield Shale up to 9.0m thick, that overlies Mittagong Formation up to 1.8m thick and then Hawkesbury Sandstone to the floor of excavation. At the centre of the station, there is a historical infilled gully up to 6m thick, comprised of ripped sandstone, siltstone and clay with gravel and cobbles. The rock structure of the Ashfield Shale is dominated by continuous, clay filled faults that dip sub vertically in the upper Kellyville Laminite, sub-horizontal in the Rouse Hill Siltstone and terminate in the Mittagong Formation. Over 80 discontinuities have been recorded during the excavation, some of which had unfavourable dip directions that may result in rock wedges that impose loads on the wall. The Ashfield Shale is underlain by a slightly weathered to fresh, fine to medium grained, high-strength Hawkesbury Sandstone with high locked-in tectonic stress up to 6 times the overburden stress.

4. DESIGN METHODOLOGY

When assessing the pressure acting on the station box walls, it is necessary to distinguish two types of failure mechanisms: those due to the failure of rock wedges; and those due to ground stress relaxation (Lee *et al.* 2017). The impact of earthquakes and groundwater must also be considered in the station design. The way in which these were assessed is summarised below:

1. The failure of rock wedges is associated with distinctive blocks of rock formed by the intersection of discontinuities, which may be free to fall or slide from the excavation periphery. This mechanism is examined through kinematic stability analysis.
2. The ground stress relaxation is associated with the stability and lateral pressure imposed by the retained ground mass to the adjoining retaining wall. The pressure is assessed through finite element analysis (FEA).
3. The additional load and deformation imposed on the station box walls during an earthquake due to (a) instability of rock wedges; and (b) ground shaking and the associated deformation.
4. The groundwater pressure acting on the station box walls. Due to the drained structure design requirements, the groundwater pressure acting on the wall is minimal.

The estimated ground pressure acting behind the temporary support piles was determined as the envelope of the kinematic wedge pressure and residual lateral pressure after the excavation.

4.1. Kinematic Analysis

Kinematic analysis was undertaken using the defect orientation data obtained from the geology and discontinuity mapping records. The critical defect information recorded comprises the (1) physical location; (2) type (e.g. fault, joint); (3) orientation (i.e. dip, dip direction); (4) roughness; (5) infill material and thickness; (6) shape (e.g. undulation or planar); (7) spacing; and (8) persistence, length and height. Figure 2 shows the typical subsurface profile and discontinuity records. At Norwest Station, most of the discontinuities in shale are planar shape with some undulation, typically slightly rough between discontinuities with either clean or with some iron staining or clay infill. The height of the exposed rock discontinuities recorded on the mapping sheets is typically less than 2m for the steeply dipping joints and, for the low angle joints, the height ranges from less than 1m to 15m. The underlying Hawkesbury Sandstone is typically not affected by joints/faults.

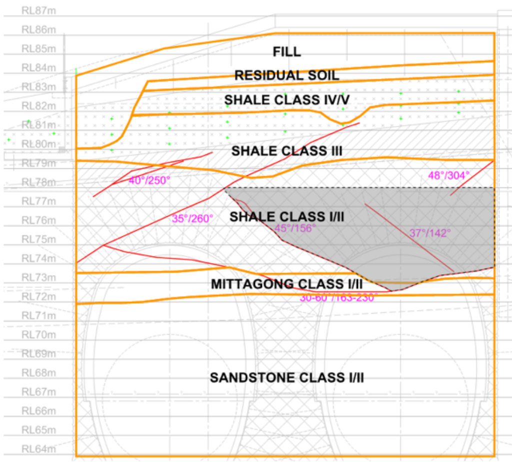

Figure 2A

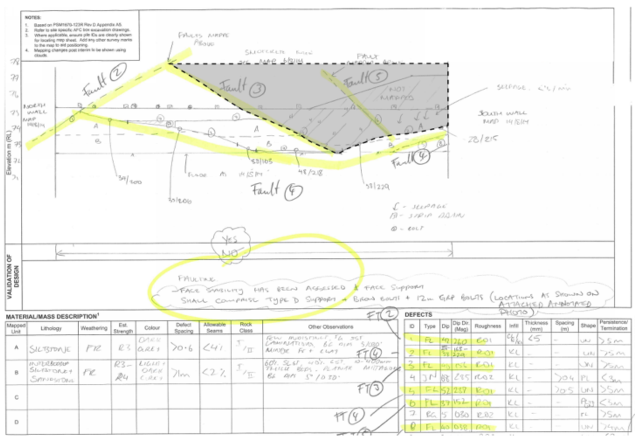

Figure 2B

Figure 2. Typical subsurface profile with rock wedge (Figure 2A) and the corresponding geology mapping sheet (Figure 2B)

Kinematic analysis using the stereographic projection software Rocscience DIPS was used to determine the likely rock failure modes (i.e. planar, wedge, toppling etc.). The risk rating associated with different modes of failure was assessed. The assessment indicated that the risk rating associated with wedge sliding is high, whereas the risk rating associated with topping failure and planar is low considering this mode of failure is limited to a small single wedge caused by the intersection of two joints, rather than a series of joints that can trigger a large-scale instability. The pressure due to toppling and planar failures was assessed to be lower than that imposed by wedge failures.

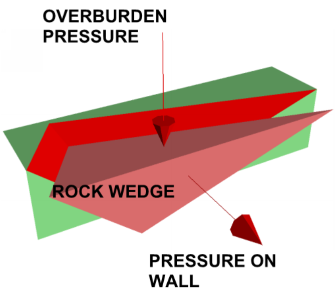

Figure 3A

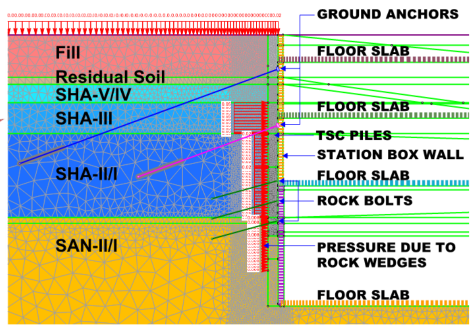

Figure 3B

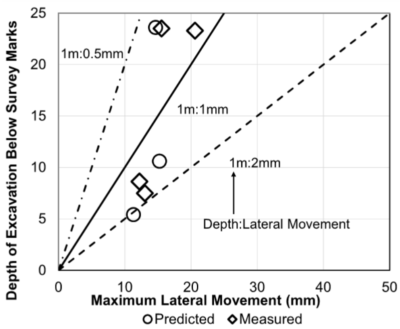

Figure 3C

Figure 3. Kinematic analysis of the rock wedge (Figure 3A). Finite element mesh (Figure 3B) and the predicted vs. measured lateral movement of the excavated face (Figure 3C)

The pressure imposed by rock wedges has been assessed using Rocscience SWEDGE as shown in Figure 3A. The orientation of the intersecting discontinuities was analysed. The discontinuity shear strength was derived based on the shear box testing and available information such as the discontinuity roughness, infill material, weathering and lithology of the rock. Scaling of the rock wedges based on the height of exposed face was undertaken to match the mapping sheet (Figure 2B). This is crucial because, for a given set of discontinuities considered, the maximum possible wedge size can be much larger than that encountered in the field. The scaling ensures the predicted pressure is relevant.

Table 1. Structural properties

Structural Type	Element Type	E' (MPa)	ν	Thickness (m)	Bolt Diameter (mm)	Bond Shear Stiffness (MN/m/m)	Bore Diameter (mm)	Spacing (m)
TSC Piles	6 Noded Triangle Element	10,000	0.3	–	–	–	900	2.5
Degraded TSC Piles		Ground properties		–	–	–	–	2.5
TSC Anchors	Tieback	200,000	–	–	34	808	100	2.5
TSC Bolts	Fully Bonded	200,000	–	–	24	–	–	1.75
Wall	Liner	32,000	0.2	0.4	–	–	–	–
Slab	Liner	32,000	0.2	0.3	–	–	–	–

4.2. Finite Element Analysis

Finite element analysis was undertaken using Rocscience RS2. The station box excavation and ground anchor/bolt installation were modelled in stages in the FEA. Because the plane strain FE model cannot explicitly consider the blocks formed by discontinuities, which may be free to fall or slide, the pressure associated with the kinematic instability needs to be explicitly considered as an applied pressure. As an individual rock wedge starts to detach from the ground, the rock pressure acting on the block would reduce to zero. Hence, the applied wedge loading was derived as the difference between the kinematic and the residual pressure. No additional pressure will be applied when the residual pressure is higher than the kinematic pressure.

Table 2. Subsurface profile and geotechnical parameters

Layer	Wall RL (Top)	γ_{sat} (kN/m^3)	c' (kPa)	φ' (°)	E' (MPa)	ν	In-situ Horizontal Stress: Perpendicular to long wall (MPa)	In-situ Horizontal Stress: Parallel to long wall (MPa)
Fill	87.5	20	10	28	10	0.3	$0.53\sigma_v$	$0.53\sigma_v$
Residual Soil	83.8	20	8	28	20	0.3	$0.7\sigma_v$	$0.7\sigma_v$
SHA-V/SHA-IV	83.2	21.5	15	29	100	0.3	$1\sigma_v$	$1\sigma_v$
SHA-III	81.6	23	100	30	300	0.3	$1\sigma_v$	$1\sigma_v$
SHA-II/SHA-I	78.9	24	350	30	1500	0.2	$1.4\sigma_v$+0.35	$2.0\sigma_v$+0.5
SAN-II/SAN-I	71.6	24	450	37	5400	0.2	$2.0\sigma_v$+2.5	$2.0\sigma_v$+2.5

SHA – Ashfield Shale, SAN – Hawkesbury Sandstone, σ_v – Overburden stress

The FE mesh with the temporary and permanent ground support and structural properties are shown in Figure 3B and Table 1. The excavation is temporarily supported by post-tensioned ground anchors and 900mm diameter piles installed at a centre-to-centre spacing of 2.5m. In the FEA, the ground anchors and rock bolts were modelled using the *Tie-back Bolt* and *Fully Bonded Bolt*. The piles were modelled as an elastic *6 Noded Triangle Element* with equivalent 'smeared' properties during excavation and reassigned to the in-situ ground properties in long-term to simulate fully-degraded piles. The

permanent station walls and floor slabs were modelled in front of the piles as a *Liner* element.
The subsurface profile and properties presented in Table 2 were developed based on the geological mapping recorded during excavation as shown in Figure 2. The locked-in stresses in the high-strength rock were first determined based on the in-situ stress measurements through over-coring and hydro-fracture methods. The rock stress perpendicular to the station wall was then derived from the major and minor principal lateral rock stresses using Mohr's circle analysis and calibrated with the measured ground movement. The equivalent Mohr-Coulomb criteria for the rock were derived from Hoek-Brown failure criteria. The initial groundwater table was assumed at RL70mAHD based on the measured groundwater level from the grout-in-place Vibrating Wire Piezometers. Groundwater monitoring shows that the water level has been drawdown to the base of excavation. This drawdown has been modelled in the FE analysis using steady-state seepage analysis.

The results show that the FEA predicted a similar magnitude of deformation compared to the field measurements, as shown in Figure 3C. Thus, the FE model was calibrated and adopted for the ground-structure interaction as described in Section 4.3.

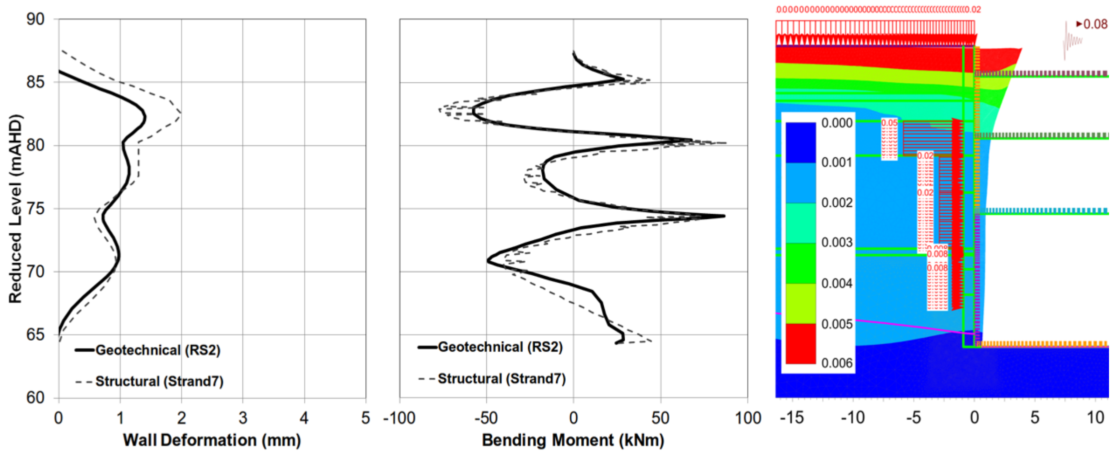

Figure 4. Calibration of wall deformation (Figure 4A) and bending moment (Figure 4B) between geotechnical and structural model. Additional seismic shear displacement (Figure 4C)

4.3. Integrated ground-structure interaction approach

An integrated ground-structure interaction approach was adopted between the geotechnical and structural analysis. The aim was to obtain agreement between the geotechnical and structural analysis, in term of the ground pressure acting on the wall, wall deflection and the wall stresses. Due to the station structure elastic, creep and shrinkage deformation, the ground pressure acting on the wall at the end of the excavation could be different to the long-term pressure port-degradation of the temporary support. Because of this, the ground pressure acting behind the temporary support piles was adopted to develop the station structures. Once the station structures were developed, the structural details were then incorporated into the geotechnical analysis and the temporary support was removed to model the load transfer onto the station structure. The ground relaxation due to long-term deformation of the station structure was considered. The refined ground pressures were adopted in the structural design and further iterations carried out until there was an agreement in the geotechnical and structural analyses, as shown in Figure 4.

Next, to analyse the additional deformation and strains caused by seismic waves on the station box walls, a pseudo-static FE approach using RS2 was adopted. The ground shearing displacement was assessed using a pseudo-static seismic coefficient for Sydney region of 0.08g in RS2. This approach accounts for the interaction between the structural supports, the surrounding ground, and the effect removal of earth mass removed during excavation, similar to that described in FHWA-NHI-09-010 (2009). The assessed additional wall deformation (Figure 4C) is considered in the structural design.

5. RESULTS AND DISCUSSION

Figure 5A shows the pressure envelope adopted for the design. The pressure is lower compared to the equivalent pressure back-calculated from the TSC ground anchors/bolts due to the additional stress relaxation that occurs post-construction. Figure 5B shows the wedge loading applied to the south wall in the FE analysis, which was determined from the difference between the kinematic and the residual pressure.

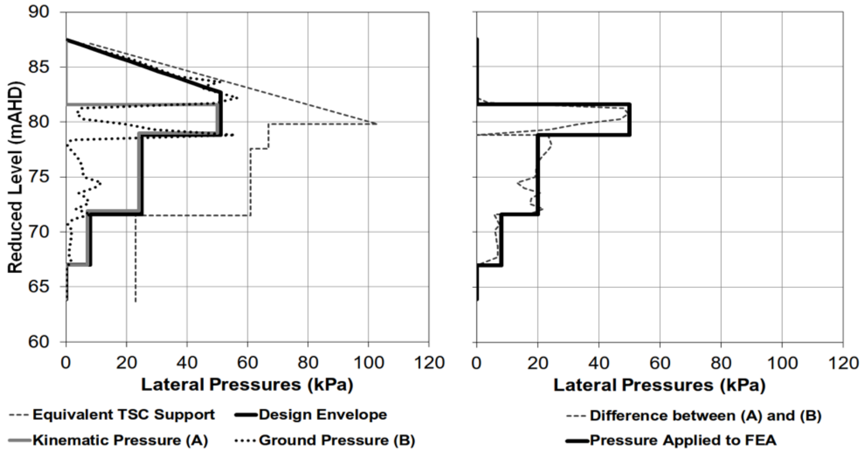

Figure 5A Figure 5B

Figure 5. Ground pressure on the station wall (Figure 5A). The kinematic load applied to the FE analysis (Figure 5B)

6. CONCLUSION

This paper demonstrates the importance of taking holistic design approach to capture the effects of intricate ground stress relaxation history and ground-structure interaction. The approach took advantage of the geology mapping records and monitoring data to fine tune the ground properties, wedge loading and tectonic locked-in stresses. The calibrated FE model was then used to predict the ground pressure for the station structures design which was optimised by considering the stress relaxation during excavation and post-construction. The final design pressure envelope after degradation of the temporary support is assessed to be lower than the equivalent load back-calculated from the ground anchors. Through an integrated ground-structure interaction approach, an agreement in the bending moment response of station wall has been achieved between the geotechnical and structural analysis and the station wall thicknesses were optimised from 600mm to 400mm.

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