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P. Hewitt
Technical Executive, Geotechnics, WSP Australia Pty Ltd, Sydney NSW 2000

M. Kitson
Principal Tunnel and Geotechnical Engineer, WSP Australia Pty Ltd, Sydney NSW 2000

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P. Hewitt[1] & M. Kitson[2]

[1]*Technical Executive, Geotechnics*
WSP Australia Pty Ltd, Sydney NSW 2000

[2]*Principal Tunnel and Geotechnical Engineer*
WSP Australia Pty Ltd, Sydney NSW 2000

ABSTRACT: A deep basement was recently constructed in Hawkesbury Sandstone for a property development on Sydney's North Shore, in Australia, that is located adjacent to critical transport infrastructure. With a depth of over 43m, the excavation is among the deepest basement excavations in Australia. This paper presents the role of Finite Element Method (FEM) as a key design tool in the Observational Method (OM), to make accurate predictions and back-analysis, to overcome geotechnical design challenges, improve construction outcomes, and allow important decisions that may lead to cost savings through an innovative design and construction approach. Applications of smart technologies were use in the instrumentation and monitoring that provided accurate data that was incorporated into the finite element back-analysis.

When unusual displacement was observed adjacent to the transport corridor due to structural uncertainties and construction activities, the OM and verification process provided a flexible framework within which to reassess ongoing movements and effects on adjacent infrastructure to ensure construction could proceed safely. The case history demonstrates the benefit of adopting the OM for excavations to achieve savings in time and cost, and to react to unexpected movements during construction.

1 INTRODUCTION

Ground movements during deep basement excavations may potentially seriously impact adjacent infrastructure and utilities. We need to consider the displacements that deep excavations induce to assess the impact on those assets and the necessary mitigation measures. This paper describes the excavation for a property development on Sydney's North Shore. The project involved excavating over 43 m deep, and is among the deepest building basement excavations in the world. The excavation took place adjacent to critical road and rail transport infrastructure. The influence of excavation on existing transport infrastructure was an important design consideration. An added complexity was the potential impact that building excavation and imposed building loads could have on nearby utilities and underground structures.

2 PROJECT BACKGROUND

The project development (known as the "Eighty Eight") is currently under construction at 88 Christie St, St Leonards, NSW. **Figure** 1 shows the proposed development and an aerial perspective of the development.

The new development includes two residential towers (maximum 47 storeys high) and a commercial tower (15 storeys) over a large retail precinct with 10 levels of below-ground basement up to 43 m deep. The structure for the building is founded on pad footings. The 8,000 m2 basement excavation extends to the Sydney Trains boundary along the west of Lithgow Street with Civic Plaza on the ground level, and is located adjacent to major rail and road infrastructure as shown in **Figure** 3.

2.1 *ADJACENT INFRASTRUCTURE*

The site boundary is surrounded by sensitive buried utilities, road and rail infrastructure, and nearby buildings for which ground movement was a key consideration. An initial scheme of the development was about 14 m from the earlier-planned CBD Rail Link. The earlier-planned rail transport project comprised twin tunnels connecting south of the city in the Redfern/Airport region to the north at St Leonards beneath the western edge of site and track quadruplication that added two more tracks, one on each side of the existing lines and within the existing protection corridor, as shown in the schematic diagram in **Figure** 2.

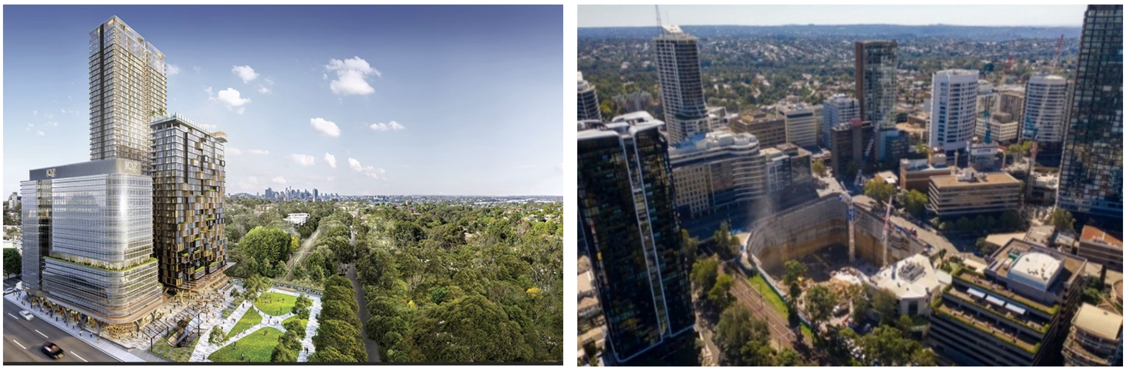

Figure 1: Aerial view perspective of the excavation

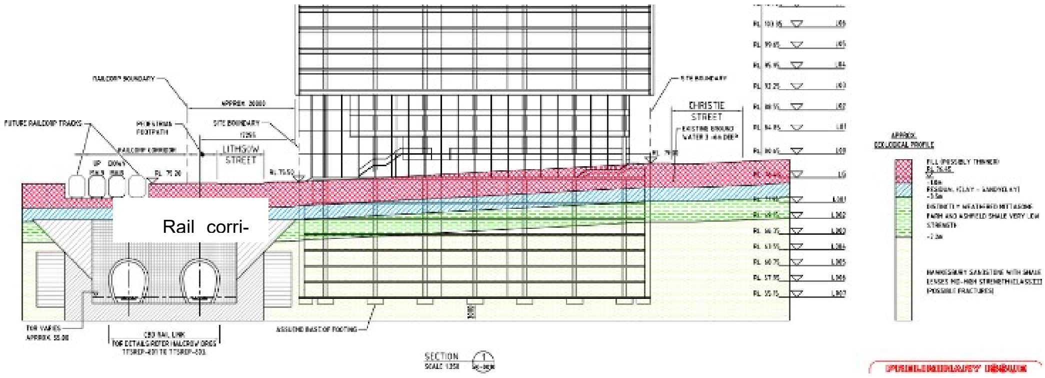

Figure 2: Schematic diagram with initial development scheme and earlier-planned CBD rail link

The earlier-planned CBD rail tunnel project was abandoned and a different route selected for CBD rail link which is now the Sydney Metro City & Southwest project (under construction).

Figure 3: Location plan showing adjacent transport infrastructure and aerial view of excavation progress

3 EXCAVATION DESIGN

3.1 *GEOLOGY*

The site geology comprises the upper sedimentary formations of the Sydney Basin stratigraphic sequence, which consists of sub-horizontal beds of Triassic-aged rock comprising (youngest to oldest) Mittagong Formation and Hawkesbury Sandstone.

3.2 *SUBSURFACE CONDITIONS*

We assessed ground conditions from geotechnical information, including 52 borehole locations, from investigations carried out by WSP and others within the proposed development and surrounding areas. Several rock strength index tests (unconfined compressive strength, UCS and point load) on recovered rock core samples were tested. In addition to the geotechnical borehole investigation, we used downhole geophysical surveys/borehole imaging to learn details of orientation, spacing, aperture, and infill characteristics of various rock mass defects including joints and bedding partings.

The development site is underlain by uncontrolled fill and residual soil to maximum 1.0m deep, followed by Mittagong Formation and Hawkesbury sandstone of varying weathering and strength. The Mittagong Formation is characterised by interlaminated sandstone and siltstone comprising fine-to-medium grained, light grey sandstone with dark grey siltstone bands that are (generally) extremely weathered and extremely low strength. The Hawkesbury Sandstone is characterised as medium-to-coarse grained, grey, with cross bedding and medium-to-high strength. The subsurface conditions encountered are summarised in **Table 1**.

The rock classification adopted the Sydney classification system (Pells et al, 2019). The Sydney classification system, which was developed for foundations, is based on UCS of saturated substance (i.e. intact sandstone or shale), defect spacing and percentage of seams within a defined vertical interval of the near-horizontal bedded rock. Both strength testing and borehole imaging identified a weaker shale/ laminite band at about 40m depth close to final excavation depth. The summary of rock strength data is shown in **Figure 4**.

Table 1: Subsurface condition

Material/origin	Material description	Thickness (m)	Top of the unit, RL (m AHD)
Fill and residual soil	Clay and Sand	1.5 - 2.0	80.5 - 75.0
Mittagong Formation	Class IV Sandstone and Shale	4.5 - 7.5	78.5 - 73.0
Hawkesbury Sandstone	Class IV Sandstone	2	74.0 - 68.5
	Class II Sandstone	3.5	72.0 - 66.5
	Class I Sandstone	19.6 - 24.0	68.5 - 63.0
	Class II Interbedded Sand and Siltstone	1.5	44.5 - 42.8
	Class I Sandstone	Not penetrated	43.0 - 41.3

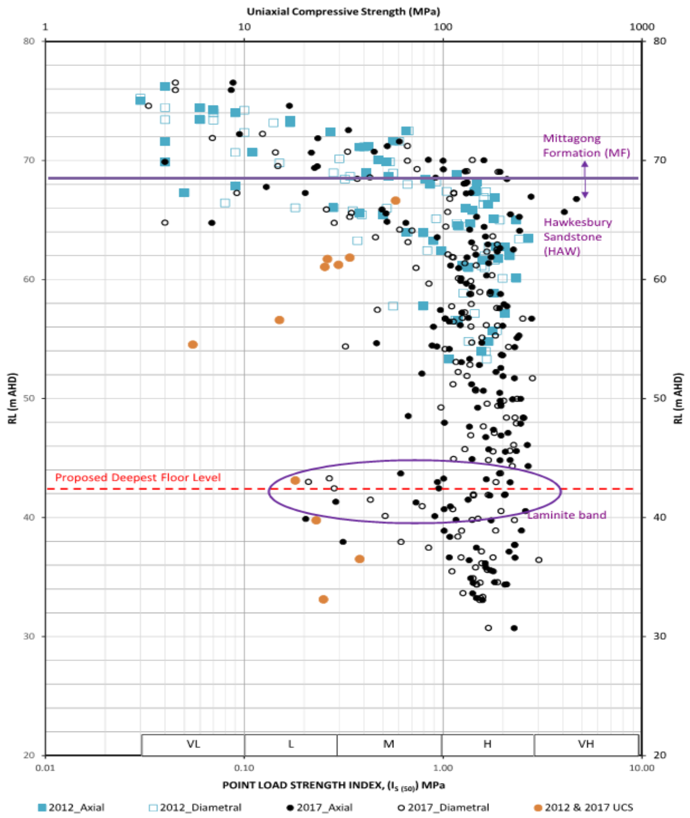

Figure 4: Rock strength data (WSP, 2018)

3.3 *EXCAVATION RESTRICTIONS*

Restrictions by the adjacent asset owners, TfNSW (ASA, 2015) and RMS (RMS, 2012) for the development included:

- Anchor systems cannot be used in the rail easement.
- Construction and operation of external developments shall not affect the stability and integrity of railway infrastructure through loading from the development and ground deformation.
- Maximum displacement of 30mm on the Pacific Highway.
- Monitoring of ground movements due to bulk excavation and monitoring of and track structures required.

The selection of retention systems for the excavation to satisfy these criteria and avoid both temporary and permanent anchors within the rail corridor and road along the project boundaries had the major influence on basement geometry and the choice of retention systems.

3.4 *EXCAVATION SUPPORT*

Most of the basement excavation face retains a sequence of weathered shale and sandstone. The temporary shoring system supporting the ten levels of basement along the Sydney Trains boundary (**Figure 5**) consisted of:

- A contiguous concrete pile wall in combination with an anchored shotcrete wall located within the middle of the site.
- Ground anchors within a 20m wide square buttress of rock on the south corner that was not to be excavated.
- Ground anchors for 20m length in the north corner adjacent to the Pacific Highway railway bridge. The north corner was not fully excavated which allowed angled ground anchors to be installed within the remaining triangle of rock.

Figure 5: Excavation support along Sydney Trains (west) boundary

The middle section of wall along the railway side was about 40m along, that could not be anchored. We used two piled walls to avoid installing long anchors under the railway line, where the upper soldier piled wall was a permanent wall installed on the railway boundary, comprising 750mm diameter piles spaced at 2.5m founded in min. Class II/III Sandstone up to 4.5m height. The lower contiguous piled wall was temporary, comprising 600mm diameter piles spaced at 1.5m also founded in min. Class II/III Sandstone anchored with short rock dowels 4.5m long in every pile. This lower wall provided the passive toe support to the upper wall.

We adopted the following proposed excavation sequence along the Sydney Trains boundary:

- Install upper permanent piled wall on railway boundary to 4.5m depth.
- Install lower temporary piled wall 3m from Sydney Trains boundary.
- Excavation to commence. Install rock dowels in every pile and then shotcrete within piled wall.
- Geotechnical engineer/ geologist to inspect exposed rock face every 2.0m intervals of excavation depth and advise requirements for localised rock bolting and shotcrete.
- Excavate to a maximum of 500mm below the next level of rock dowels and install, then install strip drains and shotcrete. Repeat for each level of anchors until reaching high strength rock.
- Continue excavation in this manner until basement excavation level.
- Construct basement structure including all pad footings, column, walls and slabs back up to the loading dock slab LB01.
- Once loading dock slab has reached full strength continue construction.

The shoring system to the three remaining sides of the excavation comprised shotcrete concrete walls spanning to 600mm diameter soldier piles. The piles are spaced 2.5m apart with ground anchors providing temporary support. The ground control solution with a pattern of rock bolts and ground anchors was selected to support the excavation below 10m to 12m below the surface level.

4 IMPACT ASSESSMENT

The impact assessment involved geotechnical analysis that considered the effects of the proposed development including basement excavation and building loads, and the effects of key values and the correction of the natural stress field based on rock mass quality.

The numerical assessment to assess the rock mass responses, and installed support subject to the design variables. Initially, a numerical two-dimensional finite element model using 2D finite element numerical modelling program Phase 2 for the Development Application (DA) submission that provided initial estimates of the potential impacts on the existing infrastructure.

The assumptions and limitations of a two-dimensional model were too restrictive for the model to provide detailed estimates of ground movements and their impacts on the existing infrastructure. The numerical assessment using 2D and 3D finite element numerical modelling programs for the detailed design and to support ground movement estimates that consider the proposed excavation's 3D geometry and provide more realistic estimates of the impact of construction on the existing infrastructure. These numerical analyses included continuum (using RS2 and FLAC 3D) and discontinuum analyses (using RS2).

4.1 *DESIGN PARAMETERS*

The geotechnical design parameters adopted for this impact assessment were selected based on the Sydney classification system, results of the geotechnical investigation, the intact parameters, the estimate of GSI for each rock mass class, published data on sandstone and shale strength and modulus, and the excavation depth of proposed development.

The adopted geotechnical design parameters are summarised in **Table** 2. To address sensitivity, we reduced the values of cohesion and tensile strength for Sandstone IV and Mittagong Formation to less than 50% of design values (see values in brackets).

Table 2: Adopted design parameters

Material type	Unit weight (kN/m³)	UCS (MPa)	Mass modulus (MPa)	GSI	Mohr-coulomb criterion		
					Tensile Strength (kPa)	Friction Angle (deg)	Cohesion (kPa)
Sandstone I	24	30	3000	75	300	55	1000
Sandstone II	24	25	2000	65	100	50	500
Sandstone IV	24	10	500 (200)	45	25 (0)	45 (35)	250 (30)
Shale II	24	15	1000	50	60	40	250

4.2 *IN-SITU STRESS*

The field in-situ stresses have a significant impact on both deep excavation conditions and induced ground movements in the immediate area of the excavation works, due to high in-situ lateral stresses, which can be 'locked in' within the bedrock stratum.

We incorporated adjacent deep excavations, including "The Forum" building, located north of the development (Figure 10) within the modelling as part of the impact assessment to provide a holistic approach to the major and minor stress distribution within the subsurface geological units adjacent to the excavation.

We adopted the following in-situ stress relationship based on WSP's work on the reference design for the nearby Sydney Metro City & Southwest project:

- Upper bound:
  $\sigma_{H(NS)} = 1.0\text{MPa} + 3.5\sigma_v$; $\sigma_{H(NS)} / \sigma_{h(WE)} = 1.5$ (1)
- Lower bound:
  $\sigma_v = \sigma_H = \sigma_h = 1.0$ (2)
  (stress field assumed to be lithostatic)

We used the upper-bound stresses applied to fresh, good quality sandstone and shale (Class I and II). In poorer quality rock masses, the horizontal stresses are expected to be less, and the lower bound stresses were applied. The minor horizontal stress was applied perpendicular to the excavation's eastern and western walls; the major horizontal stress is applied perpendicular to the excavation's northern and southern walls.

4.3 *PREDICTED GROUND PERFORMANCE*

The proposed retention system wall design addressed the following displacement mechanisms which have been observed to cause ground surface deformation adjacent to the excavation and could affect the railway.

- Lateral earth (soil) pressure acting on the shoring system causing it to deflect.
- Relaxation of the rock mass resulting from a reduction in lateral stress (stress relief).
- Anchor hole drilling and installation.

The proposed shoring system with soldier piles and anchors was designed to control the ground surface deformation due to lateral soil pressure in the upper parts of the proposed excavation (mechanism 1). We chose the layout and stiffness of the shoring system to minimise the ground movements and the impact on the railway tracks, and rail overbridge.

The relaxation of the rock mass due to stress relief (mechanism 2) from the deeper parts of the basement excavation will happen irrespectively of the shoring system type. The numerical assessment was calibrated against monitoring results from various deep excavations around Sydney, including monitoring results of the Embassy Residences (see **Figure 3**) basement excavation located on the western side of the rail corridor. Moving the boundary of the proposed development footprint and installing anchors just outside of the rail corridor will not significantly decrease ground movements of the rail corridor caused by rock stress relief, which are a result of an adjacent deep excavation.

Figure 6shows the east-west cross sections of the development with the predicted ground movements from 2D numerical assessment due to the proposed excavation. As shown in **Figure 6**, the basement excavation below LB01 at RL69.5m AHD was extended to 3.1m inside the boundary to temporarily support the cantilever pile wall along the western boundary. This revised basement plan was adopted to address Sydney Trains' preference, such that no (temporary or permanent) anchors are constructed inside the rail property. The wall configuration was based on a similar cantilever post-tensioned pile wall which was successfully adopted for the Gore Hill Freeway widening at Artarmon at a location where project boundary constraints dictated that no ground anchors could be installed (Hewitt et al, 2008).

The adopted loadings for the 2D numerical assessment are more conservative than actual loading environments and the realistic loading reduces the impact of 3D effects. The predicted deformation from 3D numerical assessment indicates that the predicted deformations and stress concentrations obtained from the 2D assessment are likely to be conservative and reduces the displacement by up to 40% due to the 3D "buttressing effects". **Figure 7** shows the site's north east quadrant with the predicted 3D ground movements within the rail corridor, Pacific Highway and rail overbridge.

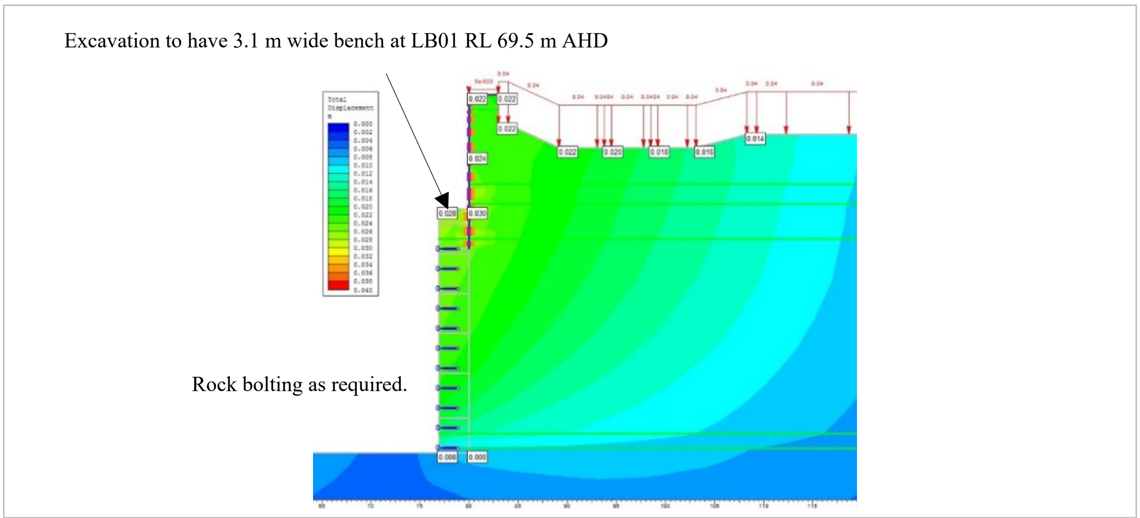

Figure 6: Predicted total ground movements of east-west section from 2D assessment

4.3.1 *Deformation within rail corridor*

The maximum predicted total vertical and horizontal deformations below the existing railway tracks after excavation completion are approximately 2mm and 6mm respectively (**Figure 6**). The maximum differential vertical and horizontal settlements below the existing rail track within the rail corridor due to the excavation were calculated to be less than 1 mm and 2mm. Trigger levels for “Line Alarm Level 1” were 10mm for 60km/h track and 14mm for 40km/h track. Lateral movement affects the line value, with the line value determined by three track locations over 8m.

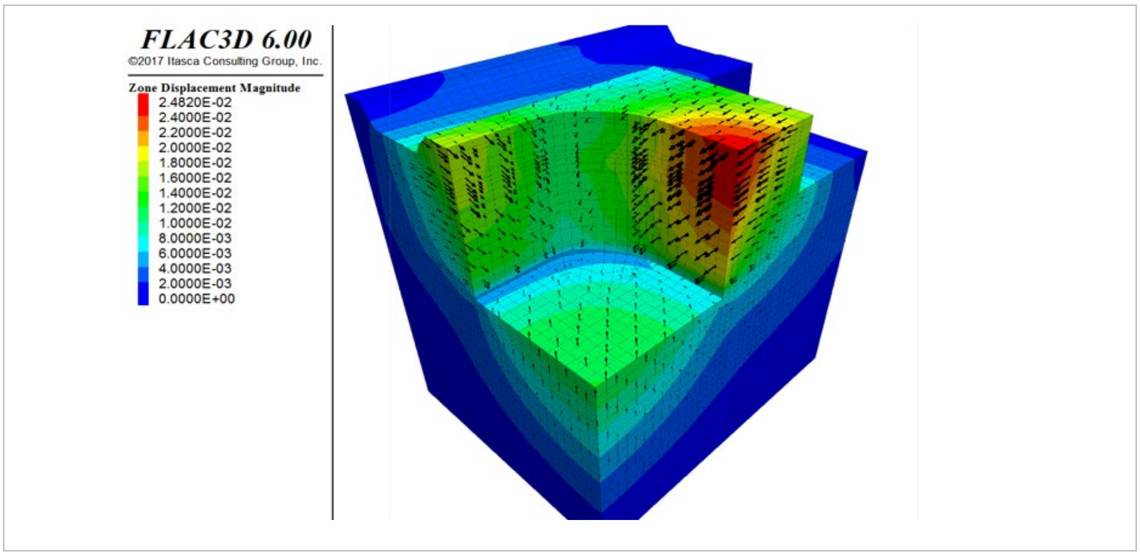

Figure 7: Predicted ground movements from 3D assessment

The estimated ground performance from the numerical assessment, a database of movements for walls using published case history data, and monitoring data on of other nearby projects (Hewitt et al, 2008) indicates that lateral wall movements are generally in the 0.5mm to 2mm range per metre depth of excavation in rock. **Figure 8** indicates the field performance of the Embassy development adjacent to the “Eighty Eight” development and the typical rates of movement observed in northern Sydney, in similar ground conditions. The lateral wall movement at the adjacent Embassy development was approximately 0.5mm or less per metre depth of excavation in rock. Trigger levels addressing total serviceability deflection (lateral displacement) of the wall in any one direction were 30mm adjacent to Pacific Highway.

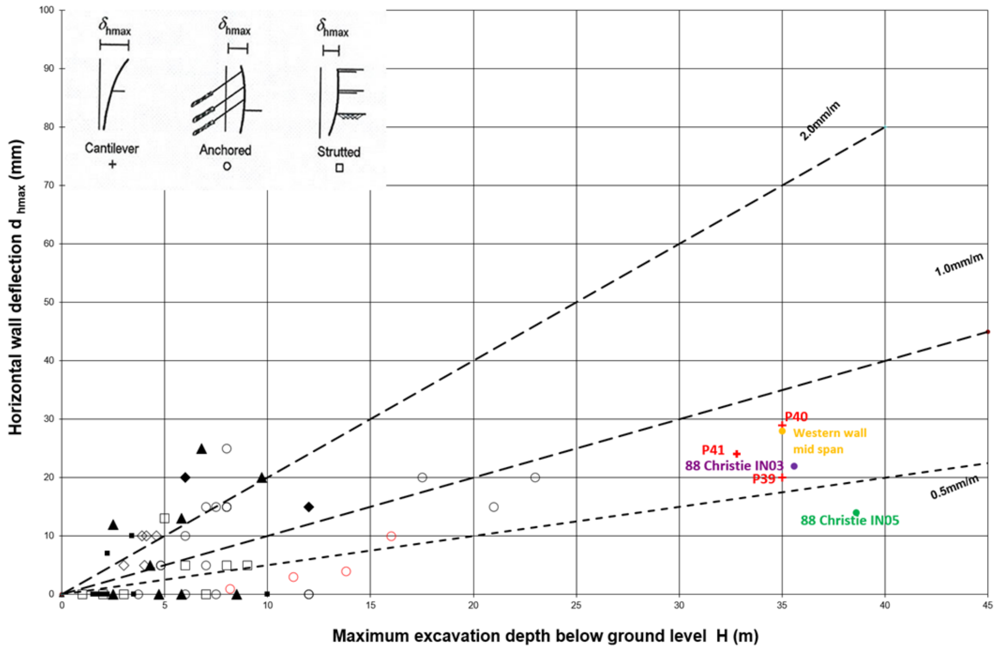

Figure 8: Field performance with different retention systems

5 CONSTRUCTION PERFORMANCE

We adopted the observational method with some contingency measures to prevent a SLS or ULS from occurring as described in CIRIA C760 (Gaba et al, 2017), including the following:

- The installation of a temporary high stiffness anchor at a high level early in the excavation sequence to control ground movements due to wall deflection, with waler and allowance for additional pre-stressing.
- Excavation start in the south-eastern corner of the site to obtain evaluation of actual wall performance, recalibration of ground and analytical models and identification of recalibrated parameters.
- Along northern boundary, excavation to proceed in 6m to 9m wide vertical panels for horizontal distance of 6m to south of Pacific Highway retaining wall to provide "berm" effect.
- Allowance for additional anchoring/cable bolts along potential sub-horizontal shear/ laminite planes identified from borehole imaging and instrumentation installation, ideally prior exposure/ displacement (adopt endoscope methods if necessary).
- Limiting temporary excavation depths along northern boundary if SLS trigger level approached.
- Trigger limits were identified at key construction stages to enable appropriate and timely decisions and interventions to made by the project team in relation to how the site retention scheme is performing and how movements are developing compared to the recalibrated and SLS characteristic predictions.

5.1 *BASEMENT EXCAVATION*

The bulk excavation progress, rock condition and shoring adopted along the south and northern (Pacific Highway) boundaries are shown in **Figure 9**, as at July 2020.

Figure 9: Bulk excavation works - view to south and north

5.1.1 *West (railway) wall*

The investigation indicated there could be weaker shale/laminite bands at about 40m depth. (see **Figure 4**). We therefore planned the installation of a temporary high stiffness anchor to control ground movements due to wall deflection, depending on the observed displacement. During the bulk excavation, the automated inclinometer measured more pronounced horizontal sliding movement on these two shale bands (**Figure 10**). The sliding movement on the horizontal shale bands was caused by the release of in-situ stress within the sandstone that allowed the block of sandstone above the shale band to move more than had been predicted. We used finite element analysis using RS3 by Rocscience. Initially we used a slice model to calibrate the model to match the movement in the inclinometer (**Figure 10** to **Figure 12**). The shale band was modelled as a ubiquitous joint model in conjunction with the Mohr-Coulomb parameters. The dip direction was set at 10 degrees into the excavation and the friction angle was reduced to 24 degrees to calibrate with the inclinometer results. Once the model was calibrated we assessed various option to reduce the movement and provide additional support to the wall, so that movements would not impact the railway.

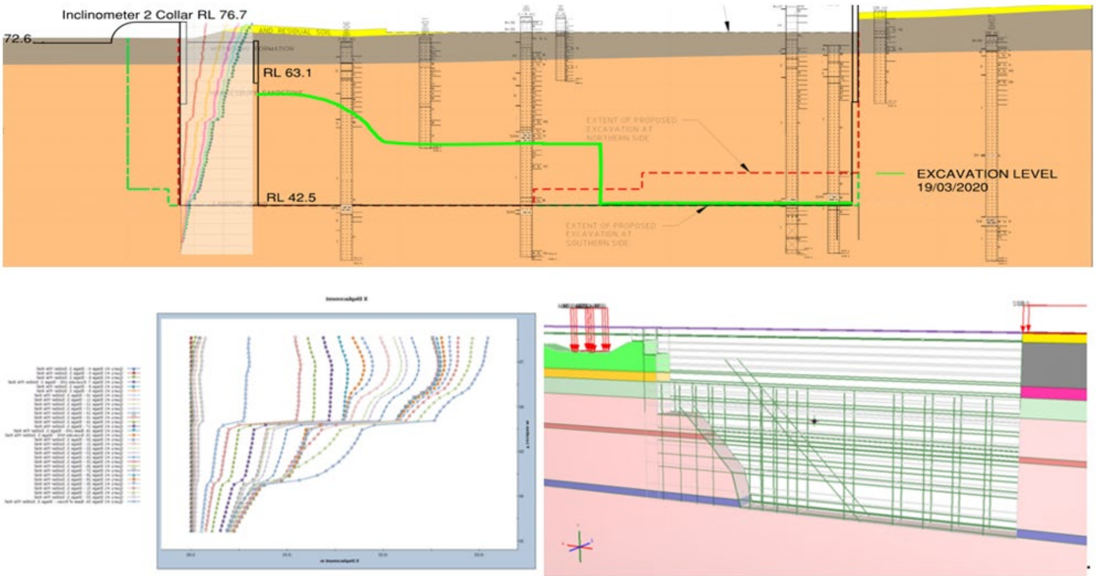

Figure 10: Bulk excavation progress from west to east and RS3 slice model including shale bands calibrated with inclinometer measurements – 14 November 2019 to 19 March 2020

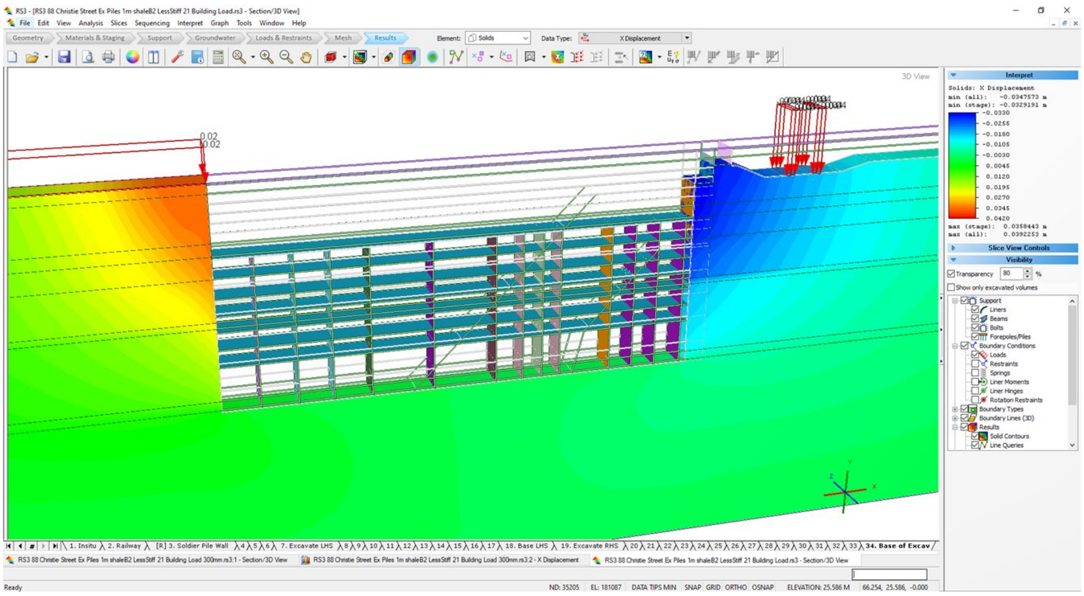

Figure 11: RS3 slice model with shale bands including building floor slabs and walls and showing predicted movement of the railway

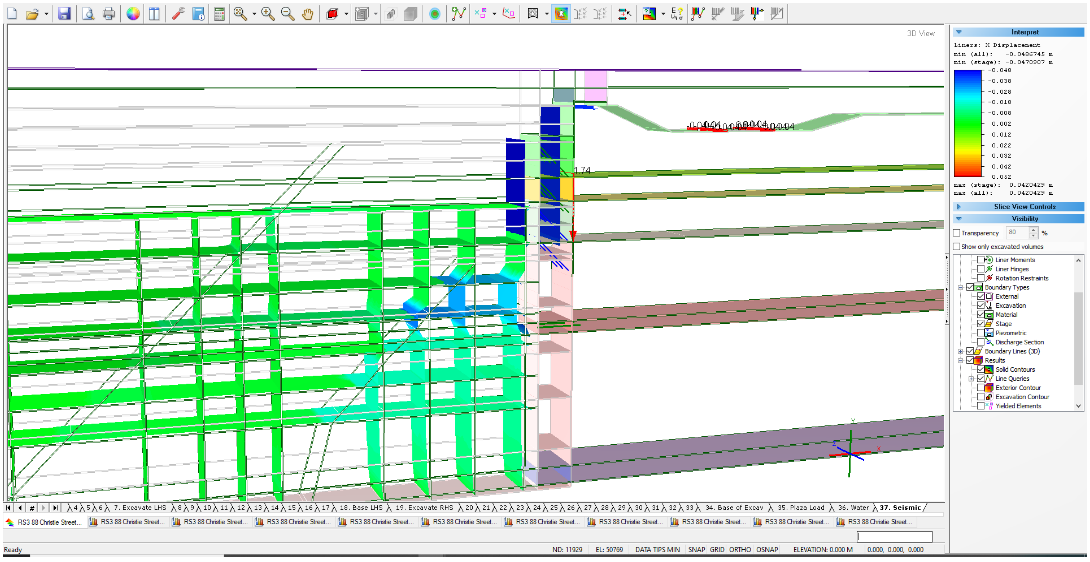

Figure 12: RS3 slice model with shale bands calibrated showing effect on movement of building floor slabs and walls

We used a 3D model to assess the bulk excavation staging and it was decided to leave a rock buttress against the wall to be excavated last, with corner propping to be installed against the southern rock buttress (**Figure 13** to **Figure 15**). To reduce the potential for movement on the railway we modified the anchor length of the southern 20m wide rock buttress anchors to have an adequate bond length below the shale band, and installed hydraulic corner propping with 2.5MN force applied to the wall (**Figure 5**). Additional anchors were installed on the opposite corner to counteract this force.

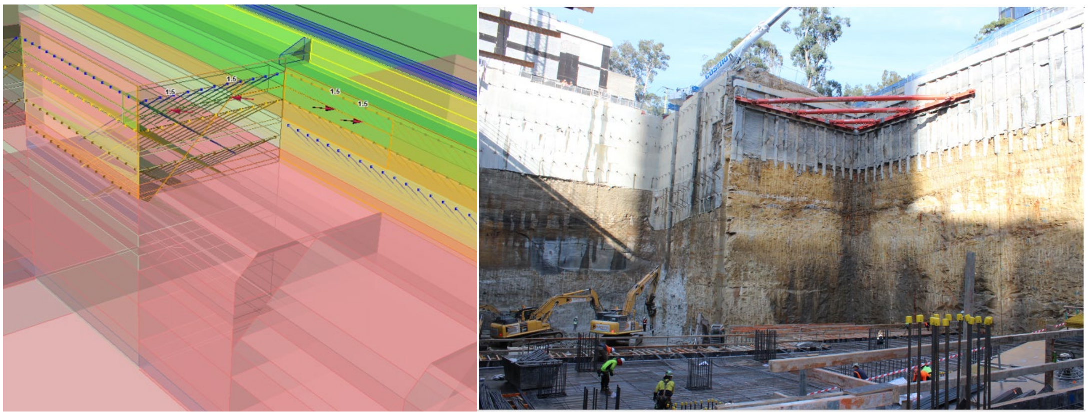

Figure 13: RS3 full excavation model with pile and anchor support and additional corner propping support

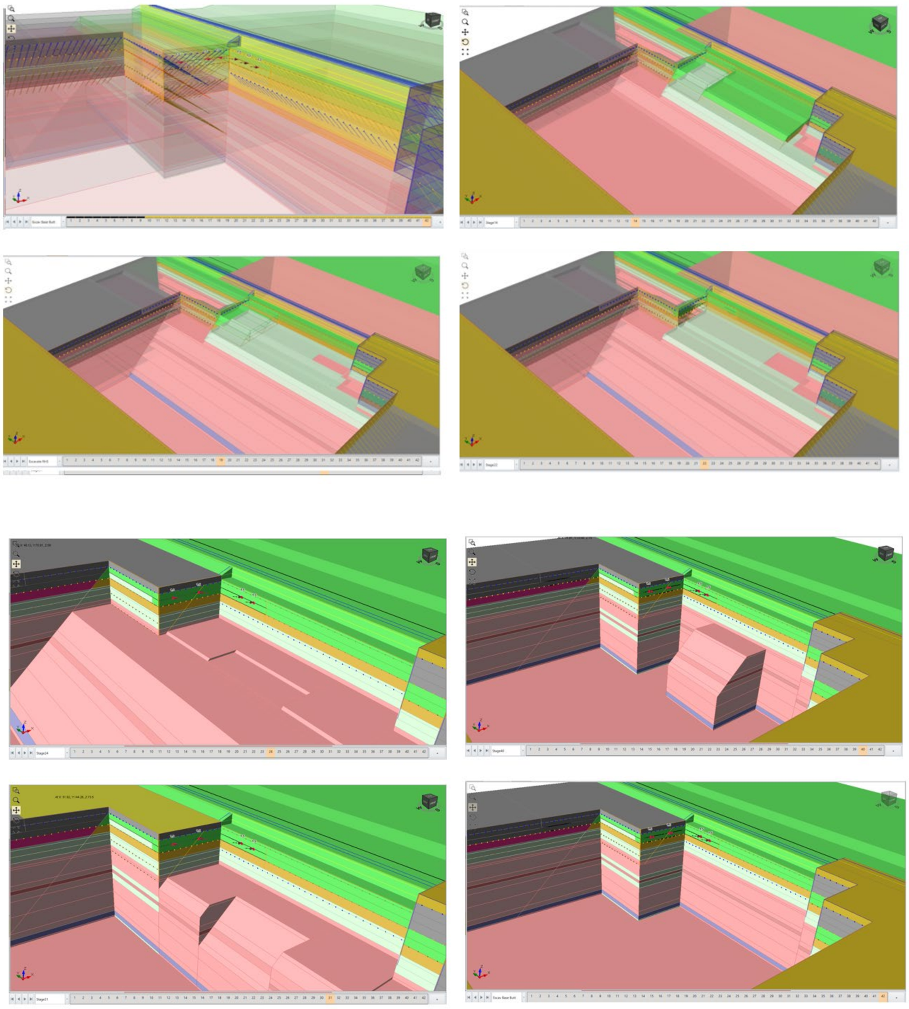

Figure 14: RS3 full staged excavation model leaving a rock buttress below the railway wall to be excavated last.

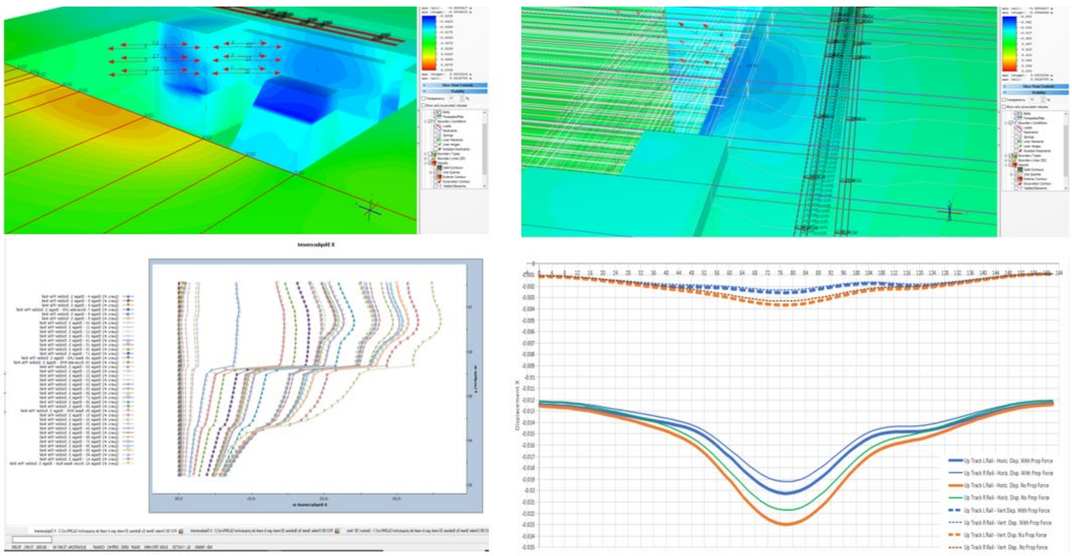

Figure 15: RS3 full staged excavation model predicted effects on the railway line leaving a rock buttress below the railway wall to be excavated last and introduction of corner propping and additional anchoring.

5.2 *INSTRUMENTATION AND MONITORING*

A project team objective was to streamline data collection to maximise system and project integration, and shorten the review and decision-making process to improve construction safety. As a general trend, advances in construction monitoring are moving away from physical measurements at a limited number of points, towards widely distributed, wirelessly connected sensor networks, and to digital scanning techniques. This data allowed the excavation contractor to optimise construction processes and increase project safety performance. Monitoring for enhanced safety was also critical due to the project scale and proximity to existing infrastructure (e.g., rail bridge, underground utilities and road pavement).

Methods included surveys of deflections and rotation of the walls, laser wall scanning (**Figure 16**), ground settlement/heave and rail track. The analyses helped set trigger values based on the 'traffic light' principle to ensure we could anticipate and control excessive ground movements.

As part of controlling the excavation process, instrumentation and monitoring points were adopted as part of the excavation protection strategy as shown in **Figure 17**. The frequency of instrumentation monitoring was based on the excavation pace and was conducted in conjunction with regular visual observation. **Table 3** shows the instrumentation schedule. Walls were monitored to check the actual design performance during construction and to provide data for reviewing design and performance, and for risk management. Monitoring satisfied the designer that the geotechnical models employed in the design were representative, that predictions of the ground and rock support behaviour were accurate, and verified compliance with the design requirements. The maximum measured horizontal wall movement was 28mm at the mid-point of the west (railway) wall, which addressed restrictions by adjacent asset owners and demonstrated excellent agreement with design predictions.

Automated Remote Monitoring and Precise manual surveying was undertaken of the railway track and associated infrastructure on the North Shore Line as shown in **Figure 18**. The monitoring network extends from approximately 8.20km to 8.31km. Monitored assets include the Up Track North Shore Line, Pacific Highway bridge and piers, OHW structures and upside crib wall. The Track Geometry was monitored in accordance with ASA Standard ESC 210 Track Geometry and Stability. A Track Certifier was engaged by JQZ in the early stages of the bulk excavation to inspect the track as a baseline, and then subsequently inspected the track during various stages of the bulk excavation.

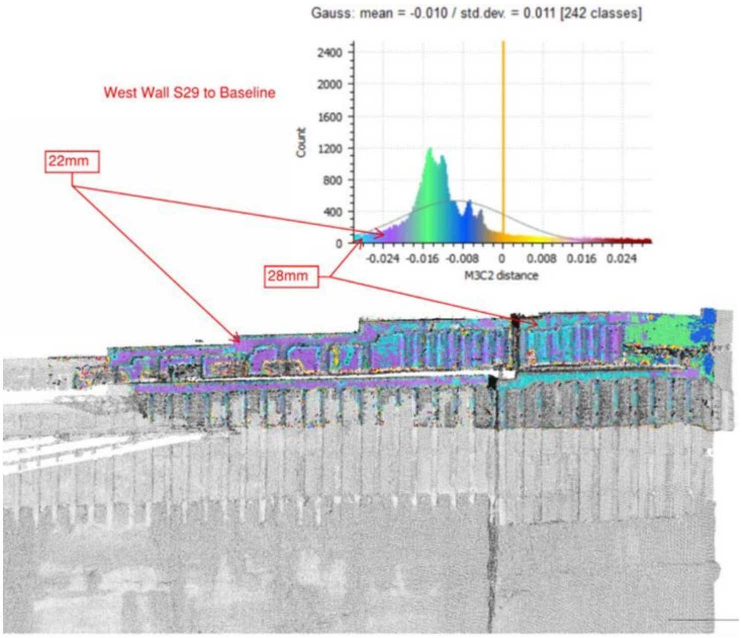

Figure 16: Laser wall scanning on west wall indicated mean displacement of 10mm and maximum 28mm

Table 3: Instrumentation schedule

Instrument	Number	Remarks
Inclinometers	5	3 manual, 2 real-time, remote GeoFlex type. See Figure 15
Displacement Points	112	Around site perimeter and buildings
Bridge displacement	3	Tiltmeters
Laser scan		Individual point accuracy of 2mm on site perimeter
Track monitoring	30	Automated remote monitoring and precise manual monitoring of rail corridor to west of site
Vibration Monitoring	5	Site perimeter
Crack gauge	~30	Visual check

Figure 17: Instrumentation plan and measured displacement

Figure 18: Automated Remote Monitoring provided Real Time Continuous Track Monitoring

6 SUMMARY AND CONCLUSIONS

The construction of the project "Eighty Eight" required excavation to over 43 m below ground level and adjacent to existing railway and highway assets. The project demonstrates how appropriate numerical analysis can be a valuable method in assessment of the influence between underground infrastructure and high-rise building foundations/deep basement excavations. Design, excavation and construction of the site retention system incorporated several critical issues, including addressing stringent settlement and angular distortion criteria, construction safety, constructability, and the constraints of defined road and rail reserves. The design process was successful and effective in addressing the concerns of all parties involved in the project.

Limited construction data available to-date indicates the displacement has been significantly reduced to achieve the performance criteria and validate the design implemented for the project. Monitoring data to date indicates the pre-construction geotechnical models and design parameters were appropriate, and that an observation-based approach allows selection of adequate retention support design to manage the risks associated with elevated stress conditions.

- Finite difference analyses were carried out to assess the behaviour of the rock, site retention system, and adjacent infrastructure.
- The accuracy of this type of interaction assessment was significantly influenced by ground model parameters and rock mass properties. This emphasises the value of detailed ground investigations prior to modelling.
- The laser is a state-of-the-art development for monitoring wall movement. It offered precision and broad area coverage of wall movements through rain, dust and smoke (which occurred due to Sydney bushfires in January 2020).
- The real-time display of the movement of basement walls allowed continuous management of deformation during excavation, with the remote monitoring inclinometer system providing early detection of horizontal sliding along shale bands.
- The real-time, automated remote monitoring and precise manual surveying of the railway track geometry and engaging a Track Certifier early in the bulk excavation works assisted in addressing impacts on track geometry.
- The maximum measured horizontal wall movement was 28mm at the mid-point of the west (railway) wall.

7 ACKNOWLEDGEMENTS

The work reported in this paper was carried out during studies by WSP (formerly Parsons Brinckerhoff) from 2010 to 2020, for various schemes. ABC Consultants Pty Ltd were structural engineers for the project. Moit and Sons (NSW) Pty Ltd carried out the main excavation. JQZ, which owns the site, generously allowed us to publish this work. The authors gratefully acknowledge the contributions of fellow staff.

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