GEOTECHNICAL CHALLENGES OF A DEEP BASEMENT EXCAVATION: A CASE STUDY

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ABSTRACT

A 59 storey tower underlain by a 30m deep basement is currently under construction within the expanding central business district of Parramatta, NSW. The tower is surrounded by and abuts two multi-storey buildings with basement levels, a heritage listed church, a 100 year old water main, and is set back approximately 50m from the Parramatta River. A comprehensive site investigation program was carried out to characterise the complex geotechnical conditions of the site, including deep cored boreholes, insitu permeability testing and borehole imaging. Due to the challenging ground conditions encountered, proximity of the neighbouring structures and shallow groundwater, the shoring system consists of secant pile walls terminated above bulk excavation level in competent bedrock, below which a grout curtain extends approximately 50m below surface level to limit seepage inflows. An Instrumentation and Monitoring (I&M) program was implemented to assess actual basement wall movements against those predicted from the numerical analyses.

1 INTRODUCTION

Construction of a 198m high, 59 storey tower overlying nine basement levels is currently underway in Parramatta, NSW. Figure 1 presents a rendered image of the proposed tower. The site is roughly rectangular in plan area being approximately 48m long by 30m wide. The almost 100 year old 'St Andrew's Church' abuts the western site boundary. The original church hall and a five storey office building were located within the site. Due to heritage constraints, the front facade of the church hall needed to be retained as part of the development.

The tower is supported by a 900mm thick hydrostatic slab across Basement Level 9 (B9) with internal thickenings at the column locations, as well as eight 1.5m diameter (tower) piles integrated into the western basement wall. The deeper B10 lift core is supported by an approximately 2m thick hydrostatic slab. Over 200 micro-piles (200mm and 225mm diameter) and 50 vertical rock anchors installed below and tied into the B9 and B10 slabs resist hydrostatic uplift pressures.

The retention system consists of anchored and internally propped secant pile shoring walls that terminate within competent bedrock approximately 15m below ground surface level. Below the toes of the secant pile walls, the competent bedrock was cut vertically. To limit seepage inflows, a grout curtain extends below the toe level of the secant pile walls to a maximum depth of approximately 50m below ground surface; that is, approximately 20m below a B9 bulk excavation level (BEL) of RL-19.8m.

A comprehensive site investigation program was undertaken to assess the subsurface characteristics across the site. From the results of the investigations and cognisant of the many design and construction challenges, detailed numerical analyses were undertaken to model the ground-structure interactions of the shoring system, tower piles and hydrostatic slab, as well as a large diameter cast iron cement lined (CICL) water pipe. Geotechnical and structural instrumentation has been installed and is being monitored to confirm the analysis predictions.

This paper summarises the geotechnical challenges faced, the engineering solutions devised by the authors, and how they were incorporated into the project. The authors note this paper was originally published in the proceedings of the 14th Australia and New Zealand Conference on Geomechanics in Cairns, 2023. However, significant updates to the original paper (Egan, Jackaman and Parsa-Pajouh, 2023) have been made in this current version, including a more comprehensive discussion of the numerical analysis results, and back-analysis based on actual monitoring data.

2 GEOLOGY

The site is located approximately 20km west of the Sydney CBD and lies within relatively flat alluvial topography associated with the Parramatta River. The geology within and immediately surrounding the site comprises Quaternary alluvial soils, overlying Triassic-aged Hawkesbury Sandstone (i.e. at least 200 million years old). There is limited published information of the structural geology in the vicinity of the site, however various geological features (i.e. faults) are not uncommon in this part of western Sydney.

Figure 1: Rendered image of the proposed 59 storey tower (image courtesy of Woods Bagot)

3 GEOTECHNICAL INVESTIGATIONS

Due to site access constraints, the fieldwork for the investigations was carried out in a staged manner both pre and post demolition, as detailed below.

Stage 1 (Pre-Demolition): The preliminary investigation comprised the drilling of two cored boreholes to about 40m depth, and a 13m deep borehole down to the bedrock surface. Regular Standard Penetration Tests (SPT) were completed throughout the soil profile in the boreholes. A total of six, five-stage packer tests were completed in the two cored boreholes. Standpipe piezometers were installed in all three boreholes. Pump-out tests were completed in each standpipe piezometer. Data loggers monitored the groundwater recharge rate during borehole infiltration testing as well as the steady state groundwater levels over a five month period.

Stage 2 (Post-Demolition): The detailed investigation comprised the drilling of nine boreholes to depths ranging between about 13m and 60m. The six deeper cored boreholes were drilled to depths between about 40m and 60m, primarily to assess the levels at which seepage cut-off could be achieved, and to provide advice on the design of deep tensile elements required to resist hydrostatic uplift pressures acting on the lowest basement floor slabs. Regular SPT's were carried out throughout the soil profile in the boreholes. A total of fourteen, five-stage packer tests were carried out in four of the six cored boreholes. Three additional standpipe piezometers were installed into the three shallower boreholes for long-term groundwater monitoring. Four test pits were also excavated to confirm the footing details and foundations conditions below the abutting eastern wall of St Andrews Church.

At the time of demolition on the subject site, the neighbouring property to the north-east was being developed and their bulk excavation had just been completed. The neighbouring basement excavation was about 25m deep, set back at least 1.1m from the common boundary, and had been designed as a drained structure. Permission was granted by the developer of the neighbouring property to inspect the basement walls closest to the subject site. The shoring system within the neighbouring basement comprised anchored and internally propped secant pile walls that terminated approximately 14m below ground surface level and which were underlain by vertically cut sandstone rock faces. From observations within the neighbouring basement excavation, an approximately 2m thick, sub-horizontal shear zone (upper shear zone) was evident at approximately 18m depth. This shear zone was discontinuous in lateral extent and it was difficult to ascertain whether it was dipping into the subject site. As such, optical and acoustic televiewer imaging was carried out during the detailed investigation. The imaging was completed in two cored boreholes located in close proximity to the common boundary with the neighbouring property to assess the orientation of the upper shear zone.

Stage 3 (Detailed Design): A supplementary investigation was carried out and comprised the drilling of four additional cored boreholes to about 40m depth. The focus of the supplementary investigation was to provide updated and more specific advice on the design, construction and inspection of the heavily loaded tower piles located along the western shoring wall.

Laboratory soil aggression and point load strength index testing were carried out for each stage of investigation. Unconfined compressive strength tests were also attempted, but the results were considered to be unreliable based on a poor correlation with the abundance of point load tests completed, and the results were never reported.

4 GROUND MODEL

From the results of the geotechnical and hydrogeological investigations, the subsurface profile comprised fill overlying variable and interlayered alluvial soils to about 13m depth, then sandstone bedrock which contained localised horizontal shear zones and laminite beds. The alluvial soil profile consisted of silty/sandy clays and sands with various proportions of silt and clay fines. The upper sandstone profile was weathered but of limited thickness. The underlying sandstone was typically fresh and of medium, high and in some instances very high strength. The fresh laminite bed was also of medium to very high strength and was encountered below approximately 25m to 27m depth, and was about 4m thick. The sandstone and laminite bedrock were typically Class II/I (Pells et al., 1998) below 15m to 21m depth, with occasional laterally discontinuous bands of Class III to Class V rock between 1.4m and 4.2m thick.

In the north-eastern corner of the site, two of the cored boreholes encountered an upper shear zone (as discussed in Section 3) between approximately 17.5m and 19.5m depth. Within these boreholes, the upper shear zone manifested as fractured bedrock bound by two distinct sheared/crushed seams or no core zones. The shear seams encountered in one of the boreholes is shown in Figure 2.

The borehole imagery further confirmed the presence of the upper shear zone and indicated apertures within individual defects typically up to 8.5mm wide, however, aperture widths of between approximately 35mm and 100mm were also recorded; possibly widened due to drill flush water effects. These defects were mostly dipping down to the west and south-west; that is, into the proposed basement excavation. The fracturing/shearing observed in the cored boreholes was less extensive than anticipated following reinspection of the shear zone from within the neighbouring basement. In addition, the rock cores at similar depths within the remaining boreholes did not indicate any such features and it was inferred that the upper shear zone was laterally discontinuous to the south and west. A second, deeper shear zone, comprising closely spaced defects and no core zones, was also encountered in several other boreholes between about 38m and 46m depth.

Figure 2: Sandstone bedrock containing two distinct shear seams

A total of twenty, five-stage packer tests were completed as part of the preliminary and detailed investigations. The packer test results indicated that the insitu permeability of the sandstone and laminite bedrock typically ranged between about 0.01 to 9.0 Lugeons (uL) above 37.5m depth, but with a single result of 48uL. However, between 37.5m and 50m depth, the results generally indicated higher values between about 18uL and 133uL. Prior to dewatering commencing within the adjacent neighbouring basement, groundwater levels on the subject site fluctuated between approximately 5m and 6m depth, with a slight hydraulic gradient down to the north toward the Parramatta River. A summary of the encountered ground conditions is presented in Table 1.

Table 1: Simplified summary of the geotechnical model

Unit	Description	Top and Bottom of Unit (RL mAHD) [Unit Thickness (m)]
1. Fill	Variable silty/sandy clays and sandy soils generally assessed to be poorly compacted	9.6 to 6.3 [0.4 to 3.0]
2. Alluvial Soils	Alluvial clays of soft to hard strength and alluvial sands of very loose to dense relative density	9.1 to -3.5 [9.2 to 12.2]
3. Sandstone (See Note 1)	Highly to moderately weathered rock of very low or low strength (Class V and IV)	-2.3 to -11.6 [0.7 to 8.3]
4. Sandstone	Slightly weathered to fresh rock of medium, high or very high strength (Class III & II/I)	-3.9 to -17.6 [5.7 to 13.7]
5. Laminite	Fresh rock of medium, high or very high strength (Class II/I)	-15.9 to -21.6 [2.1 to 4.1]
6. Sandstone (See Note 2)	Fresh rock of medium, high or very high strength (Class II/I)	-17.3 to -50.8 [10.1 to 30.6]

[1] Unit 3 was overlain in places by a thin layer of extremely weathered sandstone which has been excluded from this summary.

[2] Unit 6 contained bands of Class III and IV rock up to 4.2m thick

5 GEOTECHNICAL STRATEGY

5.1 PRIMARY ISSUES

The following primary geotechnical and structural issues were identified for the proposed deep basement excavation:

- Transmission of ground borne vibrations from construction related activities causing settlement of the heritage listed St Andrews Church, which is supported on high level footings, partly founded in very loose and loose alluvial sands;
- Support of the church hall brick facade located inside the basement footprint at its southern end;
- Retention of the soil and weathered bedrock profiles, and limiting seepage inflows during construction and in the long-term;
- Deflection of the 100 year old, 900mm CICL water main, which was set back approximately 7.3m from the southern site boundary;
- Interaction of the proposed basement levels with the neighbouring basement structures, requiring a combination of temporary internal props and rock anchors to provide lateral support to the shoring system; and
- Stability of a 1.1m to 2.3m wide plinth of soil and rock located between the neighbouring 25m deep basement excavation and the northern side of the proposed 30m deep basement excavation.

The methodologies that were adopted to overcome the various design and construction issues are discussed in detail below.

5.2 ST ANDREW’S CHURCH

St Andrew’s Church is located immediately behind the western basement wall. In order to reduce the risk of potential vibration induced settlement, jet grouting was carried out along the eastern side of the church (including short return distances along the northern and southern sides) in advance of the basement excavation. The process of jet grouting required a cementitious grout to be pumped into the underlying soils at high pressure, creating a soil-cement mix of greater strength than the insitu material. The grout was delivered via a small diameter nozzle. The drill rods were slowly extracted during grouting in an attempt to create closely spaced soil-cement columns of at least 1m diameter.

The high-pressure grout mix was injected into the soil profile to a target depth of approximately 9m, which correlated with the base of the very loose and loose alluvial sands below the church. To reduce the risk of damage, instrumentation was set up along the church facade to detect vertical movement of the walls during jet grouting. From the monitoring records, grouting pressures and in turn the soil-cement column diameters were reduced during the works in order to limit upward jacking forces along the base of the strip footings supporting the Church.

5.3 CHURCH HALL

The single storey church hall with an approximate 284m^2 plan area occupied the south-western portion of the basement footprint. Prior to the commencement of piling, the majority of the brick hall was demolished except for the southern facade, which had to be integrated into the ground and first floors of the proposed tower.

Following installation of the shoring piles and perimeter capping beams, the footings of the brick facade were strengthened, needled, and incorporated into a post-tensioned (PT) concrete ground floor slab. The PT slab was supported on the capping beams at the southern end of the site, and internally on six steel ‘H’ beam plunge columns (587mm diameter) socketed between 3.8m and 6.7m below BEL. The plunge columns were laterally braced off the basement shoring walls, underlying sandstone bedrock cut faces, and adjacent plunge columns as the excavation deepened. Figure 3 shows the PT slab and plunge columns supporting the hall facade at the southern end of the site.

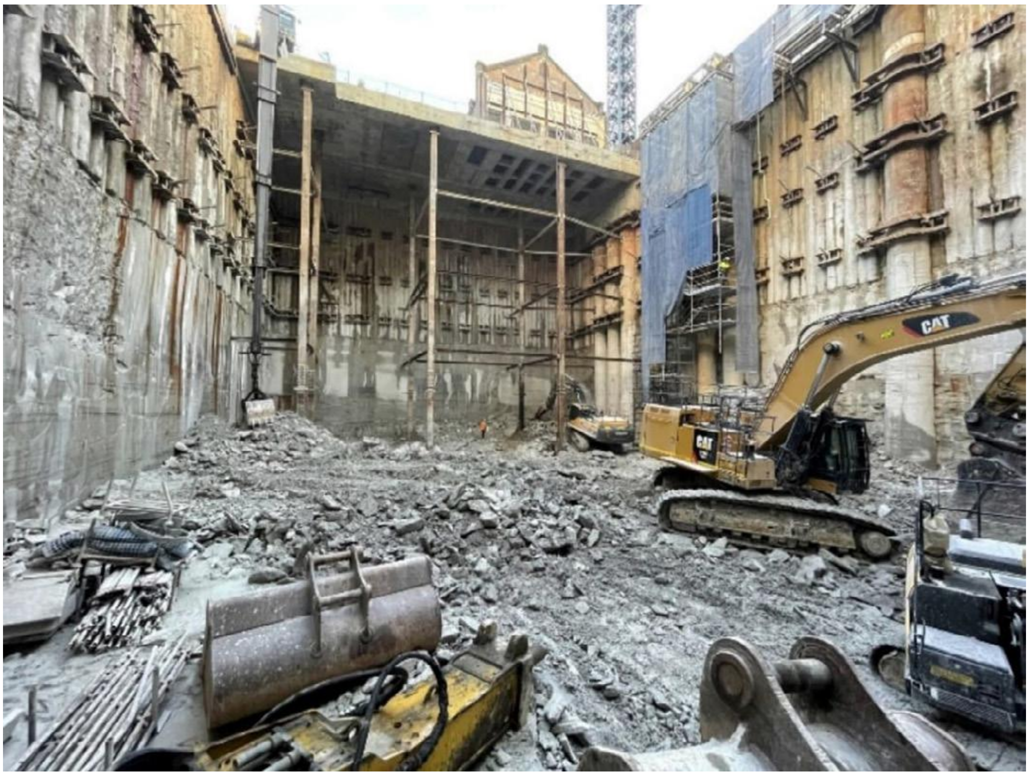

Figure 3: PT slab and plunge columns supporting the hall facade, looking south

5.4 RETENTION SYSTEM

Based on the results of the geotechnical investigations, including the long-term groundwater level monitoring and packer test results, a secant pile shoring wall was nominated to support the soil and upper weathered bedrock profiles. Diaphragm walls were also considered however, the limited site area and spatial constraints around the retained church hall facade rendered this option more costly than secant pile walls.

The 630mm diameter secant piles were installed by Bachy Soletanche Australia (BSA) using bored piling techniques with segmental casing. The secant piles were generally installed to approximately 15m depth below ground surface. In the vicinity of the rock plinth between the two deep basements, the hard piles were extended to 19.5m depth in order to engage and support the upper shear zone.

The secant pile walls were partly supported by three rows of temporary rock anchors (3 to 7 strands) between approximately 10m and 24m long, with maximum bond lengths into medium (or greater) strength bedrock of about 7m. Near the neighbouring basements at the northern end of the site, the anchors were complemented by internal props (up to four rows). The secant pile walls, temporary rock anchors and corner props are shown in Figure 4.

Below the toe level of the secant pile walls (i.e. below RL-5.5m), the sandstone and laminite bedrock was cut vertically. Regular inspections were completed during excavation to assess rock face stability and to design stabilisation measures, where necessary.

Six inclinometers were installed to progressively monitor the lateral deflections of the shoring walls and underlying vertically cut rock faces during bulk excavation. Load-cells were also utilised to measure the horizontal forces within three of the corner props in the north-eastern corner of the site.

5.5 GROUT CURTAIN

In order to control groundwater inflows, bedrock curtain grouting was carried out prior to the installation of the secant pile walls. From the results of the seepage analysis presented in Section 6.1 below, the grout curtain was installed to roughly 50m depth, with a 1.5m overlap between the top of the grout curtain and toe of the secant pile walls.

The adopted methodology comprised the drilling of ‘primary’ grout holes at approximately 6m centres, followed by intermediate ‘secondary’ grout holes. The primary and secondary holes (spaced 3m apart) were grouted until a satisfactory grout take was recorded and the closure criteria (less than 2uL) in the secondary grout holes was achieved. Inflatable rubber packers and digital flow meters/pressure gauges were utilised to assist the rock grouting process. Where closure of the open joints, seams etc. was not achieved, tertiary holes were drilled, grouted and tested. At several locations along the basement perimeter, quads, quaternary and quinary holes were required in order to achieve closure.

Within the north-eastern quadrant of the site, curtain grouting under pressure was abandoned due to connectivity of several persistent rock defects and leakage into the neighbouring 25m deep excavation. In one instance, grouting at 40m depth was leaking into the neighbouring basement at about 18m depth.

In total, approximately 340kL of grout was injected under pressure into the bedrock. For the remainder of the grout curtain adjacent to the neighbouring 25m deep basement, a method specification was designed including grout holes at 0.75m centres that were filled with grout under gravity only, and then progressively topped up, as required.

5.6 ROCK PLINTH

The 25m deep neighbouring basement to the north-east of the subject site is set back a minimum 1.1m, and extends at an oblique angle from the common boundary. The neighbouring retention system comprised anchored and internally propped secant pile walls underlain by vertically cut rock faces, similar to the subject site. In the long-term and adjacent to the subject site, the neighbouring secant pile walls are supported vertically and laterally by the four uppermost slabs, including ground floor, B1, B2 and B3. Concrete walls constructed directly in front of the rock faces and secant pile walls formed the permanent basement structure. Drainage provisions were reportedly installed behind the concrete walls.

As a result of the basement excavation on the subject site, a localised plinth of soil and rock would remain between the two basements. The primary geotechnical issue associated with a slender rock plinth was the possible reorientation and redistribution of the anticipated high horizontal stress field around the adjacent basements. It was assessed that a concentration of the horizontal insitu stresses could potentially cause the plinth of rock to crush and/or collapse during excavation within the subject site. Due to the presence of the unfavourably dipping upper shear zone above both BEL’s, the potential collapse was expected to occur into the proposed excavation, however, there was also the potential that the plinth could at least partially collapse onto the back of the concrete wall within the neighbouring basement. Several options to mitigate the risk were devised and the option accepted by the neighbouring developer, as the plinth was located entirely within their property, was the provision of a narrow excavated slot between the two basement excavations to essentially remove the narrowest section of the plinth. It was expected that this slot would allow controlled stress relief within the bedrock profile during excavation of the proposed basement. The secant piles along the northern basement wall on the subject site were then returned in a northerly direction, on either side of the 3.2m wide slot, towards the neighbouring basement secant pile wall.

The faces of the return walls were progressively internally propped during excavation. Below the toes of the 19.5m deep hard piles, as discussed above, an approximately 1m wide slot in the bedrock was excavated down to the neighbouring BEL. The upper portion of the slot was excavated to the back of the neighbouring secant pile wall. The lower portion of the excavated slot exposed the toe of the neighbouring secant pile wall and back of the concrete wall. The excavated slot is shown in Figure 4 below.

Inclinometer monitoring of lateral movements during basement excavation confirmed that convergence into the excavated slot had occurred. For long-term restraint, the excavated slot was incrementally mass filled with concrete up to the ground surface, removing the internal braces as backfilling progressed. Drainage cells were also installed between the concrete backfill and neighbouring basement wall to continue to facilitate groundwater flow.

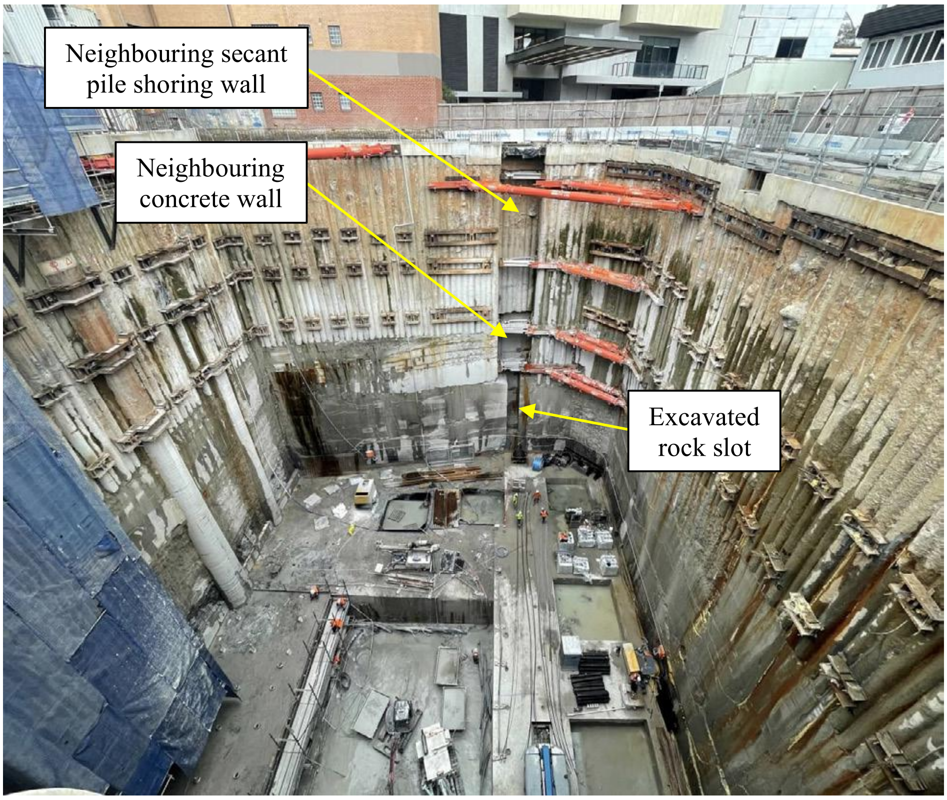

Figure 4: Rock slot extending toward the neighbouring basement walls, looking north

5.7 CICL WATER MAIN

An approximately 100 year old, 900mm diameter CICL water main was present a short distance behind the southern basement wall. The lateral deflection of the southern basement wall and of the water main itself were predicted by 2D & 3D finite element modelling. The deflections, joint pull-out, joint rotation, and tensile strain of the CICL water main were estimated from the results of our initial 2D analysis and using empirical relationships provided by various authors. The results were compared to the criteria outlined in Bracegirdle et al. (1986) as during the time of our analysis (i.e. nearly five years ago), the asset owner had no set criteria for assessment of a 900mm diameter pipe.

Due to the concerns raised by the asset owner in relation to the estimated high joint pull-out for the heritage listed pipe, as well as the anticipated delays with further analysis and obtaining approval to begin excavation, the design team (and the asset owner) ultimately decided to replace the section of pipe immediately adjacent to the proposed excavation, but with flexible connections with the retained portion of the original pipe. For this option, a more robust geotechnical assessment comprising 3D numerical modelling to predict the maximum displacement of the existing and new sections of pipe was carried out and the results provided to Warren Smith Consulting Engineers (WSce), who completed their own structural analysis of the water main. Due to the predicted lateral movements of the retention system from the 3D analysis, the new joint connections were approximately 15m beyond the eastern and western basement walls.

6 NUMERICAL ANALYSIS

Comprehensive 2D & 3D numerical analyses were carried out by the authors using PLAXIS (a software for finite element geotechnical analysis) for the following purposes:

- Predicting the potential seepage volumes into the proposed excavation during construction and in the long-term;
- Assessing the impact of the proposed deep excavation on the 900mm CICL water main;
- Predicting the shoring system behaviour and ground movements due to the proposed deep excavation; and
- Estimating the settlement, earth pressures and subgrade reaction modulus across the B9 hydrostatic slab and deeper B10 lift core slab.

The analysis of the tower piles and plunge columns was carried out by BSA on a D&C basis and is not included in this paper.

6.1 SEEPAGE ANALYSIS

The seepage analysis was carried out by developing a hydrogeological model based on the investigation results, specifications of the secant pile walls, details of the perimeter grout curtain extending below the toes of the secant pile walls, and the geometry of the basements both within and adjacent to the subject site. The permeability values adopted for the subsurface units based on the results of the borehole pump-out tests carried out in the soils, packer test results in the bedrock, well established correlations and experience with similar soils/rock, are presented in Table 2.

Table 2. Adopted permeability values for the various subsurface units

Geological Unit[1]	Permeability (m/sec)
1. Fill	Not Required
2. Alluvial Soils	2.0×10^{-4}
3. Sandstone	7.4×10^{-7}
4. Sandstone	6.0×10^{-7}
5. Laminite	6.0×10^{-7}
6a. Sandstone	4.0×10^{-7}
6b. Sandstone	5.4×10^{-6}
6c. Sandstone	2.7×10^{-9}

[1] Unit 6 sandstone was sub-layered due to the variability in the permeability test results

Based on the groundwater monitoring results within the subject site and available monitoring records within the neighbouring property to the north-east, the original groundwater level within the soil profile contained a slight hydraulic gradient down to the north from approximately RL4.2m at the southern end of the subject site, to RL2.9m at the northern end of the neighbouring property, adjacent to the Parramatta River. The water level of the adjoining non-tidal portion of the Parramatta River was taken to be approximately RL1.1m.

As groundwater levels within the subject site were partially being drawn down due to ongoing basement dewatering within the neighbouring property, the additional extracted water volumes from the long-term pumping were considered. The 3D seepage model was calibrated based on the reported daily groundwater pump-out rate in the neighbouring basement excavation of approximately 174m^3/day (i.e. equivalent to 2L/sec). The perimeter secant piles and grout curtain were then incorporated into the 3D seepage model. For the purpose of developing the model, an equivalent permeability of 0.02 uL for an assumed minimum 1m wide grout curtain, and an equivalent permeability of 0.002 uL for the secant pile walls, were adopted.

With the perimeter grout curtain cut-off at RL-41.0m, the 3D seepage analysis indicated a groundwater inflow into the basement excavation of about 5.7ML/year (i.e. 15.6m^3/day). Sensitivity analyses were completed to assess the impact of varying the toe level and equivalent permeability of the grout curtain. Based on these analyses, the inflow rates were very sensitive to the grout curtain toe level, mainly due to the more permeable sandstone layer (Unit 6b sandstone) being typically encountered between about RL-35m and RL-39m. From the sensitivity analyses, if the grout curtain was terminated above RL-35m, the rate of groundwater inflow into the excavation would be significantly higher at approximately 138m^3/day.

The actual seepage inflows which occurred between completion of bulk excavation and construction of the B9 hydrostatic slab were considerably less than predicted due to the effectiveness of the grout curtain and were readily controlled using submersible pumps.

6.2 CICL WATER MAIN ANALYSIS

The initial 2D analysis consisted of modelling the proposed southern basement shoring wall and two CICL water mains of 900mm and 150mm diameter. The pipes were set back approximately 7.3m and 12.6m from the southern site boundary, respectively with the pipe crowns at depths of approximately 2.5m (900mm diameter) and 1.0m (150mm diameter) below the road surface.

At the time of carrying out our initial analysis, no criteria existed for assessing the impact of construction related activities on 900mm diameter CICL pipes. Nevertheless, based on the results of the analysis, the expected lateral movements of the

900mm and 150mm diameter pipes, mid-length along the southern side of the proposed basement excavation, were in the order of approximately 32mm and 29mm, respectively.

As the analysed section was located mid-length along the southern side of the proposed basement excavation, and the analysis was carried out using PLAXIS 2D, the lateral movements were expected to be maximum, and possibly conservative values. Towards the eastern and western extremities of the basement excavation, the lateral movements were expected to be negligible as the southern secant pile basement wall would be buttressed by the eastern and western basement walls. Across the 30m width of the excavation, the predicted maximum curvature of both CICL pipes was in the order of 2.6mm/m. Figure 5 shows a heat map of the predicted lateral deflections of the shoring wall and pipes. A surcharge load of 20kPa along the road surface and adjacent footpath above the pipes was adopted in the model.

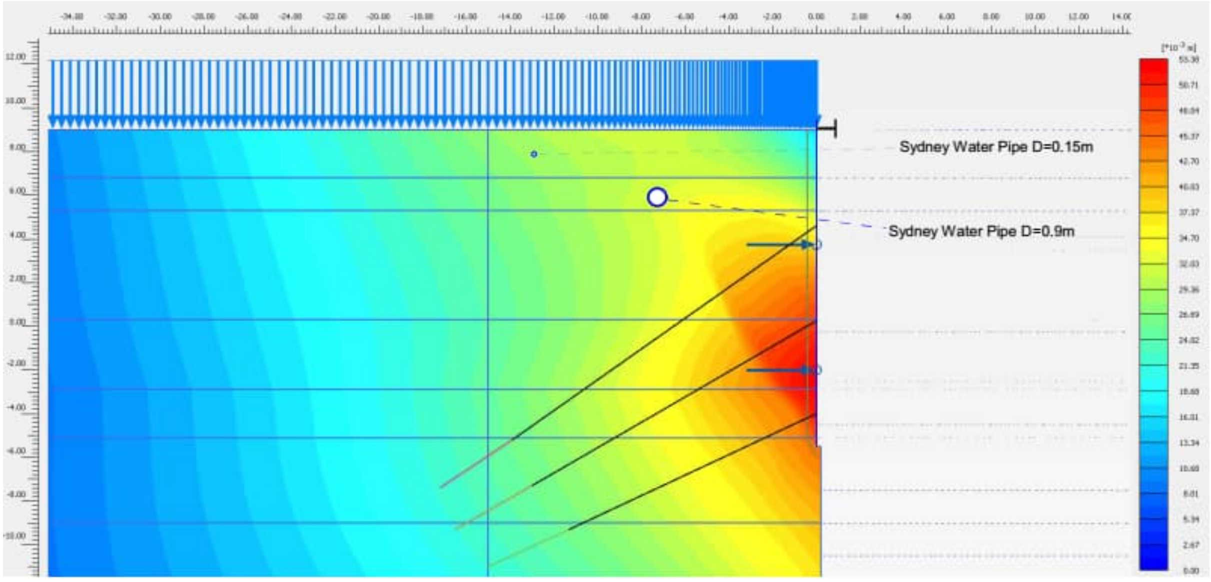

Figure 5: Heatmap of PLAXIS 2D results showing the lateral deflections of the anchored secant pile shoring wall

Following submission of our initial 2D analysis, further more rigorous assessments using the methodologies outlined in Bracegirdle et al (1996), Vorster et al (2005) and Vorster (2009) were carried out for the 900mm diameter CICL pipe only. Using these methods, the calculated empirical criteria (Smax/i), joint rotation (θ) and pipe strain (ε) were estimated as outlined in Table 3 below. The estimated lateral displacement of the pipe obtained from the 2D analysis was taken as the joint pull out (R).

Table 3. Empirical results for the 900mm diameter CICL water main

Factor	Allowable Values as Outlined in Bracegirdle et al. (1996)	Calculated Values
Empirical Criteria, $S_{max/i}$	0.012	0.0064
Joint Rotation, θ	1.5° (assuming lead-yard joints)	0.3°
Joint Pull Out, R	15mm (assuming lead-yard joints)	32mm
Cast Iron Pipe Tensile Strain, ε	>2000 με	570 με

The calculated values provided in Table 3 were below the permissible values outlined in Bracegirdle et al. (1996), with the exception of joint pull out. It is worth noting that Bracegirdle et al. (1996) stated *"again conservatively, it may be assumed that the maximum potential pull-out is equal to the maximum predicated horizontal ground displacement due to tunnelling for both longitudinal and transverse directions",* but no further discussion was given in the paper. As the lateral movement of the CICL pipe was expected to behave in a 'bell shaped' manner (i.e. Gaussian curve), the joint pull out was postulated to be less than predicted.

In addition to the numerical and empirical analyses outlined above, a monitoring plan and contingency action plan during basement excavation were prepared to support the Building Over or Adjacent to Our Stormwater Assets (BOA) submission to the asset owner. However, due their concerns, specifically that the "*level of deflection has a high potential to cause failure*", the BOA application for the proposed development was rejected. In response, the design team (and asset owner) cooperatively decided to replace the section of pipe adjacent to the proposed excavation.

A more robust 3D numerical model was developed to further assess the impact of the proposed development on the existing and replaced section of the 900mm diameter CICL pipe. The 3D model included detailed construction staging, the shoring system components including walers, rock anchors, corner props and the PT slab, as well as joints in the CICL pipe spaced at 3 feet centres. A snapshot of the 3D numerical model is shown in Figure 6.

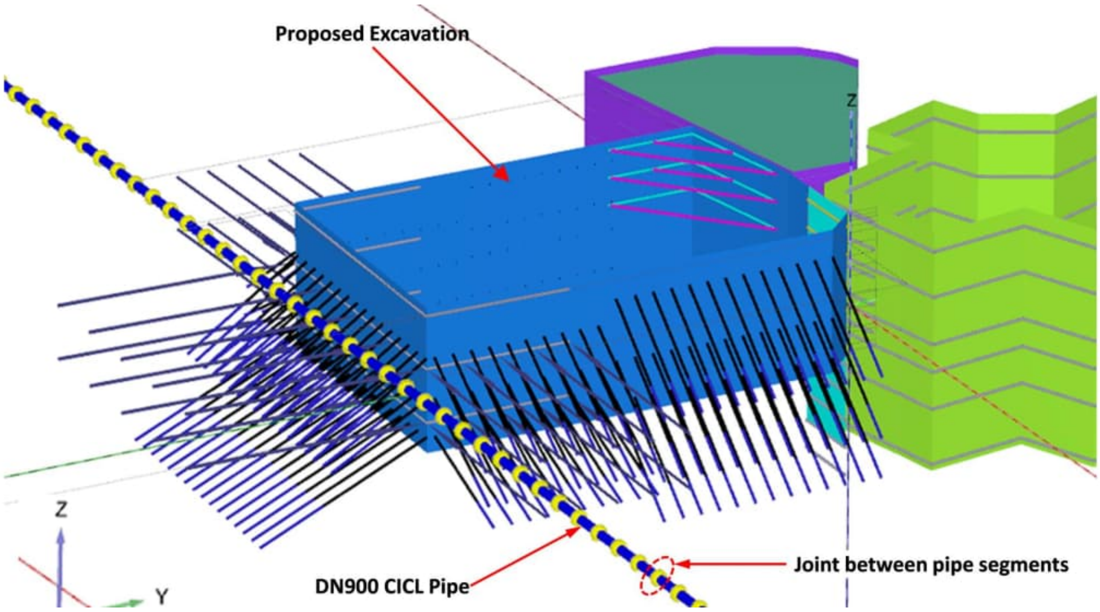

Figure 6: A snapshot of the 3D numerical model for the CICL water main analysis

The soil and rock parameters adopted in the numerical analysis were based on the results of the geotechnical investigations, past experience with similar materials and published literature (including Pells et al., 1998), and are presented in Table 4. A drained Mohr-Coulomb model was used to simulate the behaviour of the subsurface profile. Where soil/poor quality bedrock was in contact with the structural elements, a reduction factor (R_{inter}) of 0.67 was adopted. This was applied to the soil strength parameters to model the reduction in shear strength between the soils/poor quality bedrock and structural elements.

Table 4: Adopted soil and bedrock parameters in the numerical analysis

Parameter	Material [1]						
	1	2	3	4	5	6	7
γ (kN/m^3)	17	18	19	21	24	23	24
c' (kPa)	0	5	0	60	300	250	500
φ (°)	26	25	33	32	50	40	50
E (MPa)	5	10	20	80	900	2000	2000
v	0.35	0.3	0.3	0.3	0.25	0.15	0.2

[1] 1: Fill, 2: Alluvial Clay of Stiff to Very Stiff Strength, 3: Alluvial Sand of Medium Dense to Dense Relative Density, 4: Class V Sandstone, 5: Class III Sandstone, 6: Class II/I Laminite & 7: Class II/I Sandstone

The secant pile walls were modelled using plate elements representing only the hard (reinforced) piles, with an equivalent diameter and bending stiffness. This approach captures the primary structural role of the reinforced piles, which resist lateral earth pressures and control out-of-plane bending behaviour. In reality, in-plane bending transfer between adjacent secant piles is limited due to the presence of soft piles and the nature of loading conditions. To reflect this, no structural connection was introduced between adjacent plates, thereby avoiding artificial in-plane stiffening of the wall. Although inter-pile connections could be modelled using Custom Connection elements in PLAXIS 3D, this was not implemented due to the added complexity and computational cost. The selected modelling approach provided a practical balance between computational efficiency and structural realism under the governing loading conditions.

The initial stress field for the soil profile was modelled by the adopted K_o values relating horizontal and vertical stress. For the soil profile, K_o was calculated on the basis of the relationship $K_o = 1 - \sin Ø$. The insitu stresses within the sandstone and laminite bedrock were simulated based on the well-recognised relationships proposed by Bertuzzi (2014).

The numerical model was run through a number of stages in an attempt to simulate the construction sequences, such as pre-development in order to generate the insitu stresses, construction of the retention system, application of ground surcharges and excavation to BEL. An initial 'greenfield' analysis was completed without the presence of the existing water main to conservatively assess the lateral displacements of the pipe assuming it moved entirely with the surrounding soils. A second 'ground-structure' model was developed and included the existing pipe adjacent to, and extending well beyond the subject site, to estimate the movement of the pipe for both rigid (i.e. continuous pipe) and free (i.e. semi-rigid) pipe joints (initially spaced at 3 feet centres).

The results of the geotechnical analysis indicated a total maximum displacement of the existing pipe due to the basement excavation was around 20mm, regardless of whether the pipe was or was not included in the model, and provided the basis for confirming the exact location where the water main would be replaced, which was approximately 15m beyond the eastern and western basement shoring walls. The results of the geotechnical analysis were then used by WSce to carry out their own structural assessment of the retained and replaced sections of pipe.

A first pass structural assessment used the predicted ground movements from the geotechnical 'greenfield' analysis to calculate the worst-case joint pull-out, joint rotation and strains in the existing pipe (i.e. assuming the pipe follows the ground movement) using the methods outlined in Bracegirdle et al. (1996). The interim criteria provided by the asset owner at the time of our analysis (i.e. 2.5mm for joint pull-out, 0.05° for joint rotation, and 40 $\mu\varepsilon$ for tensile strain) was slightly exceeded but the impact on the retained sections of pipe was considered negligible. Similar methods were adopted for a second round of structural assessment using the predicted ground movements from the geotechnical 'ground-structure' analysis and were all well below the interim criteria outlined above.

Finally, the geotechnical 'ground-structure' analysis model was slightly modified and the estimated ground movements within a 'bounding box' approximately 4m x 4m in size around the CICL pipe was provided to WSce to assist with their design of the new pipe using Strand7 (a software for finite element structural analysis). The new section of pipe is Mild Steel Cement Lined (MSCL), of 900mm diameter and contains rubber ring joints (6m spacings) to facilitate future ground movements below/around the pipe.

6.3 RETENTION SYSTEM ANALYSIS

The 3D numerical model was developed further by including the narrow excavated slot between the two adjacent deep basements and updated shoring details to support the adjacent soil and weathered bedrock profiles. The assessment methodology, geotechnical parameters and excavation stages adopted for the analysis of the retention system were broadly similar to those outlined above for the CICL water main.

For comparative purposes, the maximum predicted lateral displacements of the shoring walls and underlying bedrock faces were reported at six inclinometer locations. After reaching BEL, the predicted lateral displacements into the proposed excavation at the inclinometer locations exceeded the maximum lateral predictions. It was postulated that the overpredictions of lateral movements were associated with the following:

- **Surcharge Loads** – Surcharges were applied immediately behind the western, eastern and southern shoring walls to simulate the additional loads from St Andrews Church and along the road/footpath surfaces. These surcharge loads were probably at the upper end of what may be expected for the church/roadways and would almost certainly increase the predicted shoring wall lateral displacements.
- **Elastic Modulus (Concrete)** – To account for long-term creep effects of the shoring system, a roughly 70% reduction of the modulus of elasticity of concrete was adopted based on advice provided by the structural engineer. It is likely that these long-term effects had not yet occurred and the concrete within the shoring system remained relatively stiff. A reduced concrete stiffness would increase the predicted shoring wall lateral displacements.
- **Elastic Modulus (Soil and Rock)** – The adopted modulus of elasticity for the soil and bedrock units were based on the results of the previous geotechnical investigations at the site, published literature and experience with similar materials. It is possible that the adopted geotechnical parameters were slightly conservative, particularly for the soil units. The underlying Class II/I sandstone and laminate bedrock exposed within the sides and base of the excavation may also have been closer to Class I rock parameters in some areas, i.e. greater stiffness. The slightly conservative soil and bedrock parameters would result in an increase and overestimation of the predicted shoring wall lateral displacements.

- **Groundwater** – The basement excavation was designed as a hybrid tanked system with secant pile shoring walls supporting the soil and weathered bedrock profiles to approximately 15m depth, below which was a grout curtain installed to approximately 50m depth, including a 1.5m overlap between the top of the grout curtain and toe of the secant pile walls. This system was very effective in reducing groundwater seepage inflows based on the low closure criteria achieved around much of the basement perimeter and only minor seepage inflows noted during basement excavation.

  During excavation, groundwater levels within the standpipe piezometers installed around the site were relatively stable within the soil profile. However, the groundwater level in one other standpipe piezometer sealed within the bedrock had been lowered by approximately 5m, which is surprising given the limited groundwater pumping required to maintain a dry basement. The lower groundwater level within the bedrock was attributed to the drainage occurring within the neighbouring basement to the north-east. It is possible that due to the lateral expanse and effectiveness of the grout curtain, the pore water pressures immediately behind the cut rock faces were reduced.

- **In-situ Stresses** – The in-situ stresses within the Class IV and better quality bedrock were estimated using the relationships proposed by Bertuzzi (2014). Recognising that these ‘locked-in’ stresses cannot be accurately simulated using the K_0 procedure or gravity loading alone, the following two-step approach was adopted in PLAXIS 3D to generate these stresses:

  - Step 1: Initial horizontal stresses proportional to vertical stress were generated using the K_0 procedure; and
  - Step 2: A subsequent plastic analysis stage was carried out by assigning a specified horizontal volumetric strain to the bedrock layers. This induced additional horizontal stresses, which were superimposed on the existing in-situ horizontal stresses calculated in Step 1.

  Based on the proximity of the nearby Parramatta River and neighbouring deep basement, as well as the presence of the upper shear zone and localised faults on the northern and southern elevations, it is possible that some of these ‘locked in’ stresses had already been released, both in geological time (i.e. Parramatta River) and in the past few years (i.e. due to the neighbouring development).

  A summary of the topographical effects on insitu stresses is presented in McQueen (2004). The author states that “*features such as valleys, coastal cliffs and paleochannels (bedrock channels infilled by recent alluvial or marine deposits) can realign the stress field and concentrate or reduce its magnitude*”. The main geotechnical structures identified at the site were predominantly along the northern elevation (i.e. discontinuous sub-horizontal shear zone) and southern elevation (i.e. faulting), with the Parramatta River located approximately 50m to the north-west. Figure 7 and Figure 8 below show these geotechnical features. The identified geotechnical structures are likely related to variations in stress magnitude and possibly the formation of the Parramatta River. Lower in-situ stresses would result in an overestimation of the predicted shoring wall lateral displacements.

Figure 7: Upper shear zone exposed along the northern elevation

Figure 8: Localised fault within the laminite bed along the southern elevation

The geotechnical parameters and structural inputs adopted in the numerical model resulted in predicted maximum lateral displacements well above actual displacements measured during and following basement excavation. Back-analysis of the lateral displacements of the shoring wall and underlying bedrock was carried out considering the above issues.

Case 1 of the back-analysis considered no surcharge loads acting on the back of the secant pile shoring walls, a short term concrete modulus (instead of a long-term modulus) in the secant pile walls, and reduced insitu stresses (i.e. $\sigma_H = 2$ instead of $\sigma_H = 2.5\text{MPa} + 2\sigma_v$).

Case 2 of the back-analysis was similar to Case 1, but the groundwater level was lowered to below BEL to eliminate the groundwater pressure behind the shoring walls and excavation faces. This second case is obviously an extreme case and does not represent actual site conditions, but it is interesting to note the impacts of the reduced porewater pressure on the predicted lateral displacements. Reference should be made to Figures 9 and 10 below.

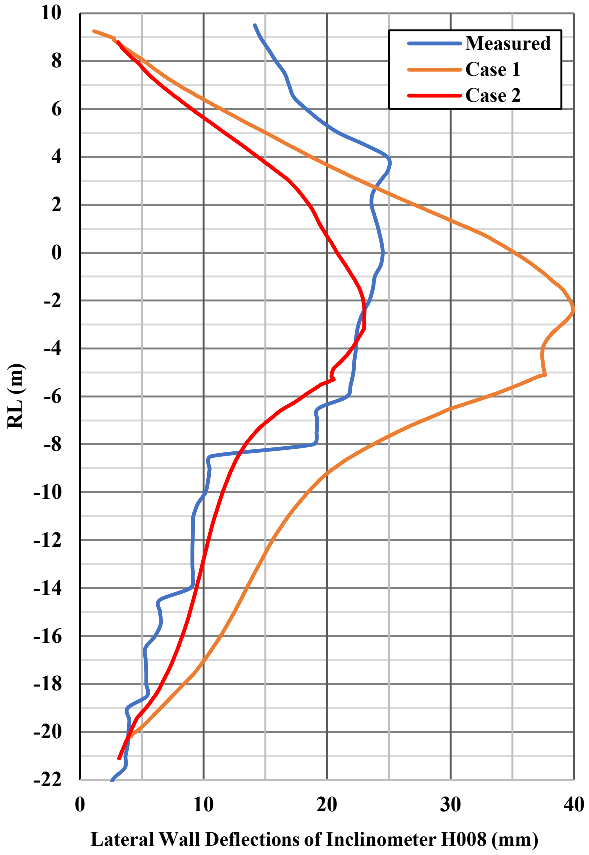

Figure 9: Measured and predicted a-axis lateral displacements

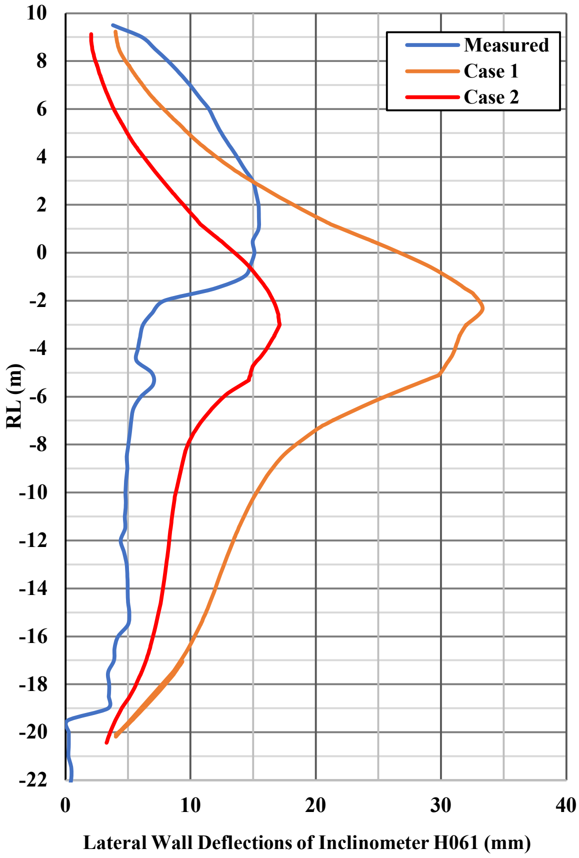

Figure 10: Measured and predicted a-axis lateral displacements

The above figures indicate that with reduced surcharge loads, insitu stresses and porewater pressures, and an increase in concrete stiffness, a better comparison with the actual lateral displacements can be made. Additional iterations of the back-analysis assessing variations in groundwater levels/pore pressures and in-situ stresses would potentially increase the accuracy of the model.

6.4 B9 HYDROSTATIC AND B10 LIFT CORE SLAB ANALYSIS

The developed 3D numerical model was also utilised to predict the settlements, earth pressures and subgrade reaction modulus values across the B9 and B10 slabs due to the application of building loads and anchor forces. The analysis was carried out by including the eight tower piles along the western shoring wall elevation and the more than 50 permanent and vertically drilled rock anchors installed below the B10 slab. The analysis did not consider the micro-piles installed below the B9 slab. The compressive building loads on the B9 columns and B10 lift core ranged between approximately 1.2MN and 113MN. The results of the analysis indicated maximum settlements below the B9 and B10 slabs of approximately 14mm, as shown in Figure 11 below.

The rock anchors (24 to 37 strands) installed to resist uplift forces on the B10 lift core ranged between 25m and 34m long and had been designed to resist jacking forces up to nearly 770 tonnes. As a result of the jacking forces on the B10 slab, additional 3D settlement analyses were undertaken and indicated maximum settlements up to approximately 5mm; a portion of this is in addition to the estimated settlements under the compressive building loads.

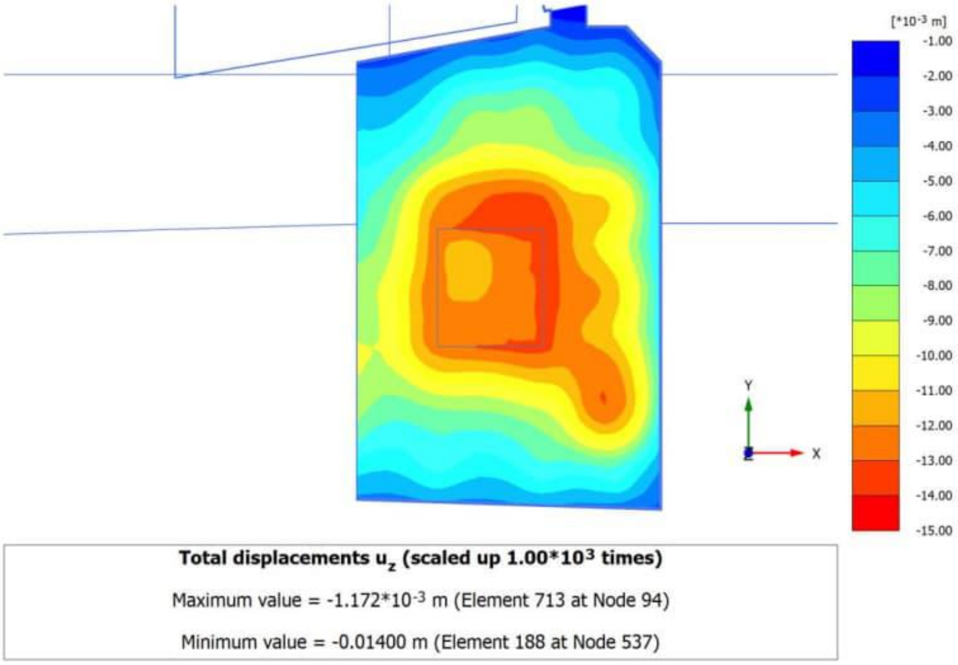

Figure 11: Heatmap of vertical settlement on the B9 & B10 hydrostatic slabs for a load combination of 1G + 1Q

The design head of water dictated the design of the micro-piles and vertical rock anchors to restrain the B9 & B10 hydrostatic slabs against uplift pressures. The design head of water was RL9.2m, which corresponds to a 1 in 100 year flood event with an additional 500mm freeboard. This level was similar to the original ground surface level across the site.

Prior to the commencement of the micro-pile and rock anchor design (completed by others), and in an effort to reduce the design head of water which was considered overly conservative, a detailed rainfall analysis of daily and antecedent rainfall (2 day to 90 day) events was prepared from available rainfall records for the period between 1 January 1832 and 28 March 2022. The rainfall data was obtained from the Bureau of Meteorology's (BOM) rainfall records for their nearby monitoring stations. A Gumbel probability plot showing the predicted return period for daily and antecedent rainfall events in Parramatta was prepared and is shown in Figure 12 below.

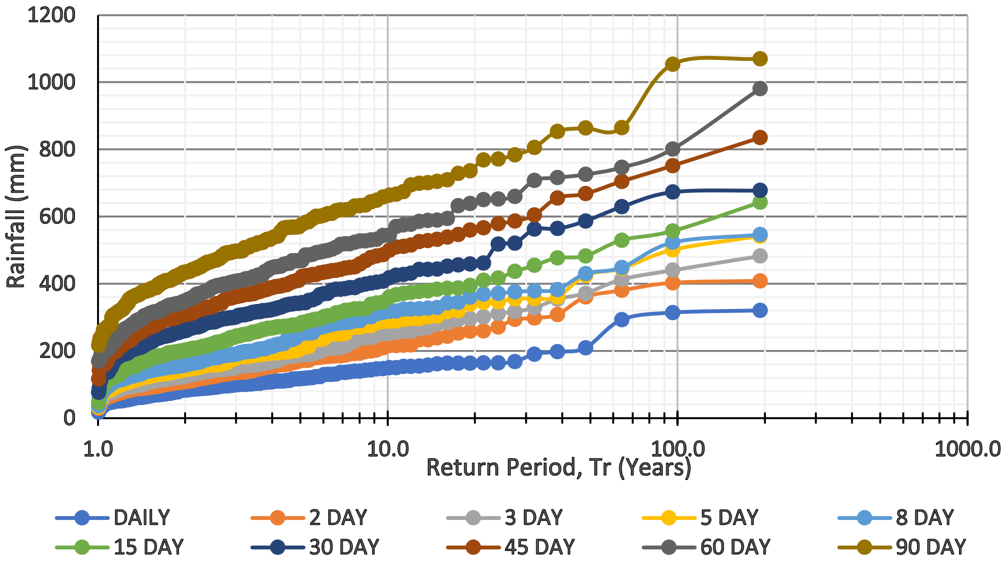

Figure 12: Probability plot of actual rainfall and antecedent rainfall between 1 January 1832 and 28 March 2022

Table 5 presents the long-term groundwater level data obtained from the three standpipe piezometers installed around the perimeter of the site against the return periods of various rainfall events.

Table 5: Historical daily and antecedent rainfall events during the groundwater level monitoring period

Date	Rainfall (mm)	Rolling Total No. of Days	Return Period (Years)	Highest Groundwater Level Recorded During Rainfall Event (RL mAHD)		
				JK107	JK108	JK109
10/2/2020	158.0	1	13.7	5.45	4.07	3.07
10/2/2020	260.0	2	21.3	5.45	4.07	3.07
10/2/2020	311.0	3	24.0	5.45	4.07	3.07
10/2/2020	360.6	5	38.4	5.45	4.07	3.07
14/2/2020	382.6	8	38.4	5.45	4.08	3.09(2)
17/2/2020	416.6	15	24.0	5.45	4.08	3.09(2)
9/3/2022	529.6	15	64.0	5.18	3.42	3.42
14/2/2020	462.0	30	21.3	5.45	4.08	3.09(2)
19/3/2022	586.8	30	48.0	5.74(1)	3.42	3.56
17/3/2020	515.4	45	12.0	5.45	4.08	3.09(2)
28/3/2022	668.2	45	48.0	5.99(1)	3.42	3.56
16/3/2020	570.0	60	10.7	5.45	4.08	3.09(2)
28/3/2022	716.2	60	38.4	5.99(1)	3.42	3.56
28/3/2022	864.6	90	64.0	5.99(1)	3.94	3.66

[1] The elevated groundwater levels recorded in JK107 between 14 March 2022 and 8 April 2022 were considered to be at least partially influenced by a leaking sewer pipe located adjacent to the monitoring well, as well as from ponded water above the protective cast iron Gatic cover.

[2] No groundwater levels were recorded between 13 February 2020 and 23 November 2021 due to a damaged water level data logger.

From Table 5, the highest recorded groundwater level at the site was RL5.99m (i.e. JK107). This level was recorded during a 1 in 38.4 year, 1 in 48 year & 1 in 64 year rainfall event for 45, 60 & 90 day rolling totals, respectively. As outlined above, the highest recorded groundwater level in JK107 was considered to be partially elevated due to a leaking sewer pipe and ponded water, which may have infiltrated into the standpipe piezometer. Notwithstanding this, for the historical daily rainfall events and recorded groundwater levels presented in Table 5, it was postulated that a 1 in 100 year rainfall event at the subject site could temporarily raise groundwater levels up to a maximum of approximately RL7.0m.

Cognisant that rainfall and flood events are different, there was no evidence based on the 3 years of groundwater level monitoring to suggest that groundwater levels (i.e. hydrostatic pressures) could rise above RL7.0m. It was further expected that groundwater levels across the site would be lower than any flood levels due to the relatively short duration of the flood events. Although there was significant evidence to suggest groundwater levels would not rise above RL7m, the design head of water for a 1 in 100 year flood event plus a 0.5m freeboard remained at RL9.2m, and the design of the micro-piles and vertical rock anchors to restrain the B9 & B10 hydrostatic slabs was carried out on this basis.

7 INSPECTION AND MONITORING PROGRAM

7.1 GEOTECHNICAL INSPECTIONS

Geotechnical inspections were completed during bulk excavation to: 1) identify potentially unstable wedges or blocks of rock that could detach from the vertically cut faces, 2) confirm the tower piles and internal slab thickenings were founded within sandstone bedrock suitable for the design ultimate bearing pressures, and 3) confirm the jet and rock grouting works were carried out in accordance with the design methodology.

During these inspections, the upper shear zone observed within the neighbouring basement excavation and confirmed during the detailed geotechnical investigation was encountered in the cut faces at the northern end of the basement excavation. The fractured, gouged and extremely weathered material within the shear zone was expected to soften and potentially spall/topple from the cut faces if left unsupported, both in the short and long-term. As a result, the upper shear zone was stabilised with reinforced shotcrete and rock bolts. Other potentially unstable blocks or wedges of rock that had been isolated by inclined joints and/or sub-horizontal partings, seams etc. were identified and removed from the cut faces, as necessary.

7.2 INSTRUMENTATION AND MONITORING

Regular inclinometer monitoring was carried out during and following the completion of bulk excavation, as outlined above. Load cells were also placed in three separate corner props (Prop No. S3 [Row 1], S7 [Row 2] and S9 [Row 3]) in the north-eastern corner of the site. The design working load of the props was 1215kN (S3), 1065kN (S7) and 2260kN (S9), respectively. Following their installation and during bulk excavation over the preceding months, the loads cells indicated that the horizontal forces within the corner props were 75% (S3), 28% (S7) and 38% (S9) of their design working loads, possibly a result of the issues outlined in Section 6.3 above.

7.3 GEOTECHNICAL TESTING

Due to the heavily loaded tower columns and B10 lift core, six additional cored boreholes were completed from the B9 and B10 excavation levels in order to further assess the suitability of the foundation material (i.e. proof coring). Based on the results of the additional investigation, the foundation material below the B9 columns and B10 lift core was confirmed to comprise Class II/I sandstone, which was considered suitable for the design ultimate bearing pressure of 100MPa. A senior geotechnical engineer further inspected the footings prior to pouring concrete to confirm their bases had been adequately cleaned of all loose and water softened material.

8 CONCLUSIONS

This case study outlines many of the design and construction challenges that the project had encountered and were overcome, as well as touching on the numerical analysis of various structural elements and the inspection and monitoring program.

Numerical back analysis of the shoring system has been carried out due to the variability in the predicted and measured lateral displacements in order to better assess the ground-structure interactions. The overestimation of the predicted lateral displacements of the shoring walls and horizontal forces in the corner props is not unexpected due to the high surcharge loads applied behind the back of the shoring walls, the use of a long-term concrete modulus, a stiffer rock mass than initially assessed, and the possibility that some of the assumed ‘locked in’ insitu stresses had already been released, both in geological time (i.e. Parramatta River and through the shear zones) and in the past few years (i.e. due to the adjacent basement excavation).

9 ACKNOWLEDGEMENTS

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CRediT authorship contribution statement

Michael Egan: Formal analysis, Writing – original draft. **Andrew Jackaman:** Writing – review and editing. **Ali Parsa-Pajouh:** Writing – review and editing.

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