WINTEN PROPERTY GROUP I FRASERS PROPERTY AUSTRALIA

396 LANE COVE ROAD, MACQUARIE PARK.

PRELIMINARY GEOTECHNICAL AND STRUCTURAL IMPACT ASSESSMENT ON ECRL INFRASTRUCTURE

FEBRUARY 2018

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396 Lane Cove Road, Macquarie Park.
Preliminary Geotechnical and Structural Impact Assessment on ECRL Infrastructure

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EXECUTIVE SUMMARY

This report presents an initial assessment of the potential impact associated with construction of a proposed commercial development at 396 Lane Cove Road, Macquarie Park on the Epping to Chatswood Rail Link (ECRL) Infrastructure.

Winten Property Group/ Frasers Property Australia obtained concept approval (MP 09_0209) for this proposed development in 2012, and currently applying this development application to modify this approval under Section 75W of the EP&A Act.

The proposed development western boundary is sited approximately 10m from the Sydney Trains rail corridor and within 15m of the Epping to Chatswood Rail Link (ECRL) Macquarie Park Station. With respect to SEPP (Infrastructure 2007) Clause 88, the proposed development falls within the horizontal distance of 25m from Sydney Trains corridor hence triggering the need to provide an impact assessment as outlined in the TIDC Guidelines for Assessing Impact of Proposed Developments on the Underground Infrastructure of the Epping to Chatswood Rail Line, 2006.

The guideline requires an impact assessment study be undertaken by the developer and to be submitted with the Project Plan application. Winten Property Group/ Frasers Property Australia has appointed WSP Australia Pty Limited (WSP) to provide a preliminary impact assessment to be included in the Concept Plan submission.

This report is limited to highlighting the possible geotechnical and structural impacts to the existing ECRL infrastructure. The proposal has been designed to minimise impact by adopting basement works with maximum clearances as possible and limiting the depth of the excavation. Additionally, the development does not encroach into the Sydney Trains stratum boundary. The developer and associated consultants have initiated a consultative meeting with Sydney Trains and have incorporated advice received on the concept plan design. The project plan submission stage will include a detailed geotechnical and a structural assessment report.

The results of the impact assessment are summarised in Table ES.1 below

Table ES.1 Calculated horizontal and vertical displacement for affected ECRL infrastructure

SECTION	CALCULATED VERTICAL / HORIZONTAL DISPLACEMENT (mm)		
	MACQUARIE PARK STATION CAVERN	MACQUARIE PARK STATION CONCOURSE CAVERN	MACQUARIE PARK STATION WEST ENTRY SHAFT
Station cavern and concourse cavern (Section 1)	2 / 1	2 / 1	N/A
Station cavern and west entry shaft (Section 2)	1 / 2	N/A	3 / 10

Where the horizontal movement is calculated to be 10mm at the west entry shaft wall, ground anchors may be required for the adjacent proposed site retention system to reduce the impact on the ECRL infrastructure.

Based on the preliminary review of the station drawings and results from the preliminary numerical modelling, it is anticipated that the construction of the proposed commercial buildings are unlikely to have an adverse impact on the existing ECRL infrastructure.

1 INTRODUCTION

1.1 BACKGROUND

This report presents an initial assessment of the geotechnical and structural impacts of a proposed multi-storey development at 396 Lane Cove Road, Macquarie Park on the adjacent ECRL Macquarie Park Station and running tunnels.

The study was commissioned by Winten Property Group/ Frasers Property Australia and Enstruct Group Pty Ltd has also been commissioned as the Structural Engineers.

The development is adjacent to the Epping to Chatswood Line (ECRL) along the north-western site boundary on Waterloo Road. Concept drawings of the development indicate up to 10m excavation (to RL 50.8m) is anticipated along the north-western property boundary for Building A. Concept drawings also indicate that up to 12m excavation (to RL 48.0m) is anticipated along the north-western property boundary for Building B and Building C.

The proposed development is approximately between 12.5m from the underground concourse cavern and 12.5m from the West Entry access shaft. Drawing PS105852-GEO-DRG-001 shows the proposed development footprint and basement, available geotechnical investigation locations and ECRL underground boundary plan.

The primary purpose of this report is to:

- outline the proposed development boundary and spatial distance from ECRL infrastructure
- examine potential impacts of the multistorey development on the existing ECRL infrastructure
- provide initial confidence to Sydney Trains that the development impacts are acceptable to obtain Sydney Trains' non-objection for the concept submission.

1.2 PROPOSED DEVELOPMENT

The proposed development consists of four (4) multi storey buildings with 3 to 4 levels of basement excavation. The tallest building (Building A) sited adjacent to the Macquarie Park Station consists of 17 storeys with 3 levels of basement and is situated in close proximity to the ECRL infrastructure.

In November 2017, WSP has been provided with a concept design for the proposed multi-storey development on the adjacent ECRL Macquarie Park Station and tunnel infrastructure. WSP has been engaged to prepare an initial assessment of the geotechnical and structural impacts of the deep excavation of the development on the ECRL infrastructure. Details of the proposed development (plan, elevation and cross sections) are included in Appendix A.

1.3 ECRL MACQUARIE PARK STATION

Macquarie Park station and the running twin tunnels intersect a sequence of Ashfield Shale, Mittagong Formation and upper Hawkesbury Sandstone. The internal diameter of the tunnel is 7.2m and these tunnels are separated by a minimum of 5.5m. The tunnel excavation was completed in 2004 and the ECRL commenced operation in 2009.

The Macquarie Park station comprises a 14m high main platform cavern, approximately 30m deep, 200m long, and 20m wide at the arched crown springline. The concourse is 70m long, 16m wide, 7.5m high arched cavern connected to the platform cavern by a 12m wide access passageway, and to street level by twin 39m long, 13m wide and 36m deep access shafts (Rozek, 2004).

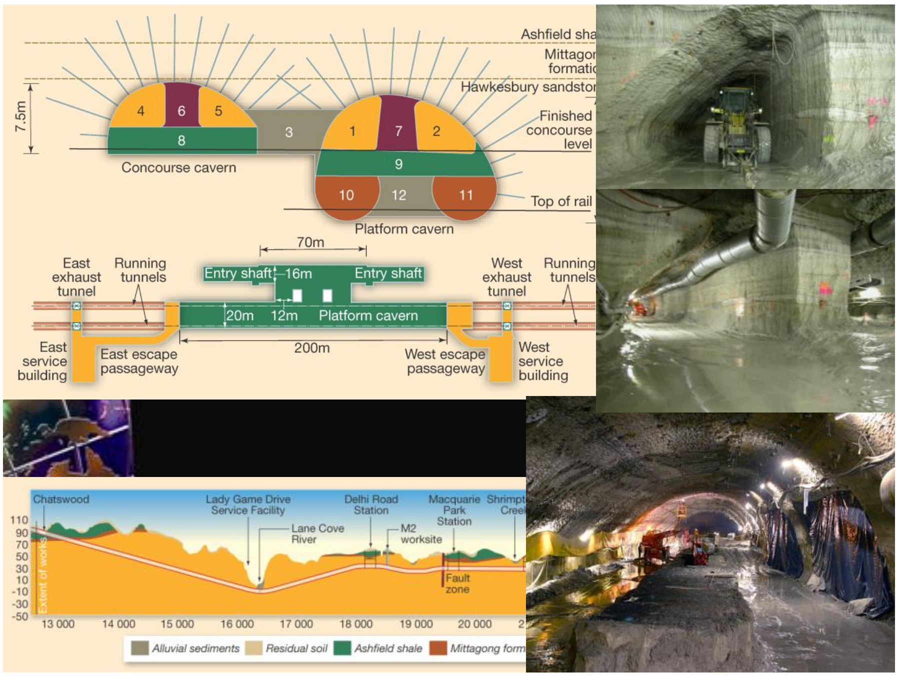


Figure 1.1 ECRL Macquarie Park Station outline details (Tunnels & Tunnelling Feb 2004)

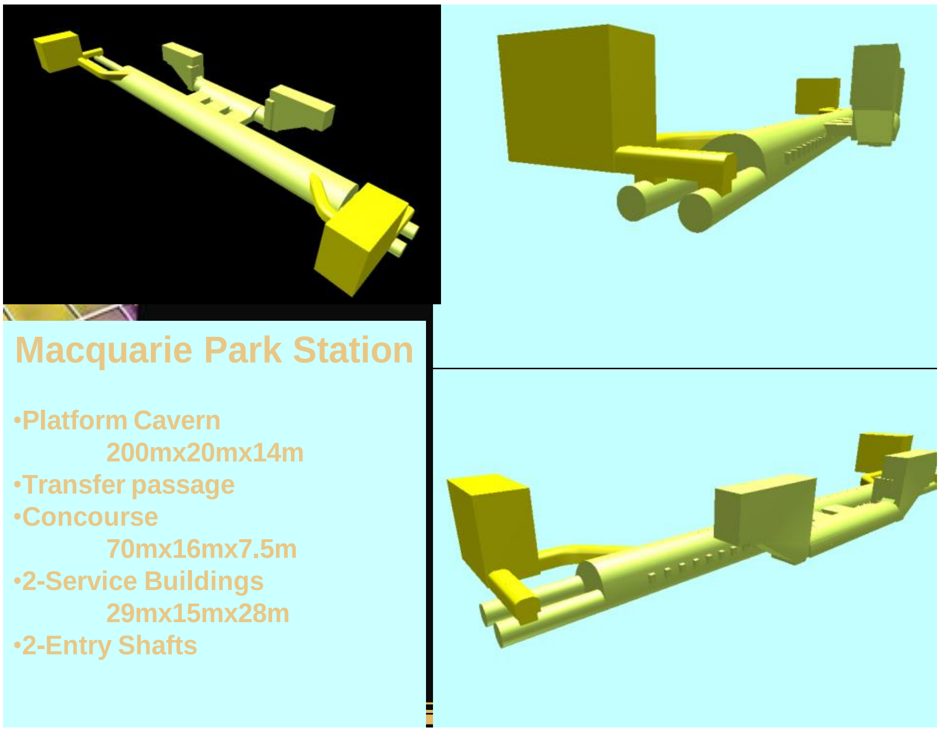


Figure 1.2 ECRL Macquarie Park Station outline details in 3D

The running tunnel permanent lining comprises circular-formed un-reinforced concrete, minimum 200mm thick, with a design compressive strength of 32MPa at 28 days. Temporary support consists of rockbolts and shotcrete lining. The tunnel invert consists of a level horizontal slab constructed 3m below the tunnel's centre.

1.4 AVAILABLE INFORMATION

The following information has been made available for consideration in our assessment:

1. Bates Smart. Architectural basement plan (ref no. PA02[REV A] 003 Basement Level 003 Plan), February 2018.
2. Bates Smart. Architectural elevation drawings (ref no. A08.01 Section AA), 23 November 2017.
3. Bates Smart. Architectural elevation drawings (ref no. A08.02 Section BB), 23 November 2017.
4. Enstruct Group Pty Ltd. Column loads for building A, B and C (ref no. 5493-171129), 29 November 2017.
5. Parsons Brinckerhoff (PB, now WSP). Geotechnical desktop study report - 396 Lane Cove Road, Macquarie Park (ref no. 2108235A/lij/0640), September 2010.
6. Parsons Brinckerhoff (PB, now WSP). Preliminary geotechnical and structural assessment on ECRL infrastructure - 396 Lane Cove Road Macquarie Park (ref no. 2108235A-PR_0637), September 2010.
7. Thiess Hochtief Joint Venture (THJV) & PB/GHD Design Joint Venture - Parramatta Rail Link (PRL). As built drawing: Macquarie Park Station - Site Layout Plan (ref no. PRL-CSD-160503 Rev 4), 4 August 2005.
8. Thiess Hochtief Joint Venture (THJV) & PB/GHD Design Joint Venture - Parramatta Rail Link (PRL). As built drawing: Macquarie Park Station - Typical Cavern and Concourse Cross Passage Permanent Rock Reinforcement (ref no. PRL- CSD-161555- Rev 6), 11 December 2007.
9. Thiess Hochtief Joint Venture (THJV) & PB/GHD Design Joint Venture - Parramatta Rail Link (PRL). As built drawing: Macquarie Park Station - Typical Cavern and Concourse Cross Passage Permanent Rock Reinforcement (ref no. PRL- CSD-161556- Rev 6), 4 August 2005.
10. Thiess Hochtief Joint Venture (THJV) & PB/GHD Design Joint Venture - Parramatta Rail Link (PRL). As built drawing: Macquarie Park Station - Cross sections DD and EE (ref no. PRL- CSD-164403- Rev 4), 4 August 2005.
11. TIDC ECRL Underground Infrastructure Protection Guidelines (Ref 20007300/PO-4532-Rev 3), 16 May 2008.

2 SYDNEY TRAINS CONSULTATION

2.1 GENERAL

Transport Infrastructure Development Corporation (TIDC) has transferred ownership of ECRL to Sydney Trains on commencement of ECRL operations hence all development assessments adjacent to ECRL are to be reviewed by Sydney Trains. Prior to commencement of this impact assessment, the developer and consultants attended a consultative meeting with Sydney Trains asset management engineers on the 28th of July 2010. In this meeting the concept plan was discussed in detail. Sydney Trains confirmed that they have not further developed specific guidelines for the ECRL project. At the time of this report no new specific guidelines has been provided by Sydney Trains and WSP has referred to the previously issued guidelines by TIDC.

TIDC ECRL Underground Infrastructure Protection Guidelines (TIDC, 2008) require that the potential impact of a proposed building development in the vicinity of the ECRL underground infrastructure be assessed in terms of potential effects on stress distribution, deformation and groundwater movement.

2.2 STATION SUPPORT ZONES

The support influence zone for the station is defined in the TIDC guidelines as 8m vertically or horizontally from the finished internal surface of the station (Figure 2.1).

2.3 STATION INFLUENCE ZONES

The TIDC guidelines define the influence zone for the station as 15m vertically above the crown of the cavern. The influence zone is bounded by a line tangent to the outside of the lining extending outwards at a slope of 2 vertical: 1 horizontal (Figure 2.1)

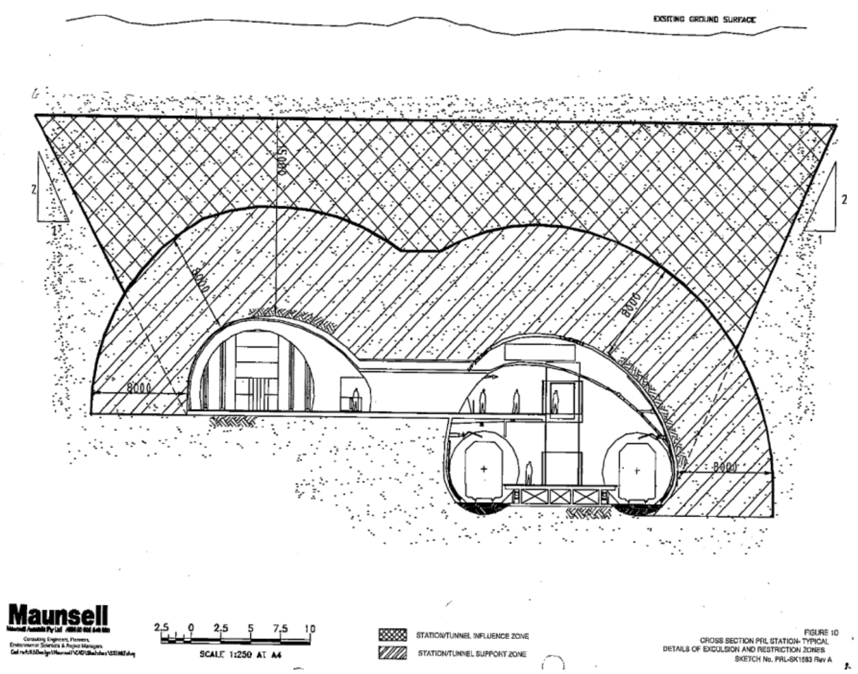


Figure 2.1 ECRL station protection and support zones

2.4 INFORMATION FOR SYDNEY TRAINS TO REVIEW

The TIDC guidelines require the following information (TIDC, 2008) be provided for any proposed development or associated foundation element that falls within the tunnel influence zone:

- assessment carried out by a suitably experienced geotechnical engineer on the predicted impact of the proposed development on the ECRL underground structure
- setting out details of the foundations for the proposed development including survey reference marks co-ordinated in MGA coordinates and all founding levels in AHD
- all dimensions of the foundation elements including anchor fixed and free lengths
- schedule of loads for all foundation elements including anchor lock off loads and design loads
- details of any foundation testing proposed at the site
- construction staging and foundation installation sequence, including any dewatering works.

Issues associated with assessment of interaction between the proposed development and the existing ECRL infrastructure include:

- stress redistribution effects around the proposed deep basement and impacts on the existing underground station and western entry shaft structure
- ground movement effects
- foundation load stresses on existing ECRL infrastructure
- loads from temporary/or ground anchors and other support (if required)
- construction impacts including vibration and staging
- variation in the groundwater regime.

3 GEOTECHNICAL MODEL DEVELOPMENT

3.1 GEOTECHNICAL DESKTOP STUDY

Geotechnical model for this assessment has been developed based on available geotechnical information around the proposed development. Summary of geotechnical desktop study for this development is presented in the geotechnical desktop study report 2108235A/LIJ/0640, dated September 2010.

3.2 GENERAL SITE SUBSURFACE CONDITIONS

The basement excavation for the proposed building development intersects, in descending order, fill materials, residual clays, (highly weathered shale) stiff to hard consistency, Ashfield Shale of varying strength from very low to medium strength with increasing depth, Mittagong Formation of medium to high strength, grading into Hawkesbury Sandstone.

The adjacent Macquarie Park platform and concourse caverns are located in Hawkesbury Sandstone. The concourse cavern has a roof elevation of approximately RL 41.5 m AHD and invert at RL 34 m AHD while the platform cavern roof is at approximately RL 40 m AHD with invert at RL 28 m AHD. The overlying Mittagong Formation occurs approximately 2 to 3 metres above the roof of the platform cavern and concourse.

Based on reports produced for the Macquarie Park Station as part of the ECRL design, the following geological profile was identified at the approximate centre of the station:

- residual soil from approximately RL 58 m AHD to RL 54 m AHD
- Ashfield Shale from approximately RL 54 m AHD to RL 46 m AHD
- interbedded/interlaminated siltstone and sandstone (Mittagong Formation) from approximately RL 46 m AHD to RL 42 m AHD
- Hawkesbury Sandstone from below RL 42 m AHD.

It should be noted that the elevation of these geotechnical units varies across the length of the station caverns as the topography and underlying weathering profile is inclined gently upwards from the southeast to northwest. There is also a gentle dip of the Ashfield Shale and Mittagong Formation units to the north.

Bedding in the Mittagong Formation and the Hawkesbury Sandstone is sub-horizontal, with bedding planes typically spaced at 100-300mm throughout the sandstone units within the Mittagong Formation, ranging to 1.0m or greater, in the siltstone units and in the Hawkesbury Sandstone. Hawkesbury Sandstone is also characterised by cross-beds dipping typically between 15 to 30 degrees, generally towards the north-east.

The station area is also characterized by two orthogonal east-west and north-south striking joint sets. Another geological feature present in the vicinity of the proposed development site is the North Ryde Fault Zone which is located approximately 150m away from the eastern end of the proposed development.

The inferred geological cross-section along northwest to southwest directions which shows the anticipated subsurface condition within the proposed development and proposed development are presented in drawings PS105852-GEO-DRG-010.

3.3 HYDROGEOLOGY

Ground water monitoring which occurred prior to the construction of ECRL tunnels and associated station indicated that groundwater table occurred at or just below approximately RL 49m AHD, within the base of weathered Ashfield Shale. Locally perched groundwater may also occur within the residual clay unit.

Drawdown of water table due to the construction of the station caverns would have occurred and would extend beyond the boundary of the proposed development site. It is noted that the station was designed as a drained station which normally result in the permanent lowering of the water table.

4 NUMERICAL MODELLING

4.1 ASSUMPTIONS

The numerical modelling was developed based on the following assumptions:

- Two sections were considered for modelling including:
  - Section 1: station and concourse caverns modelled adjacent to proposed Building A basement excavation.
  - Section 2: station cavern and west entry shaft modelled adjacent to proposed Building A basement excavation.
- The two sections modelled are in close proximity to one another and will consist of the same geotechnical model.
- Rockbolt support system for station & concourse cavern were incorporated.
- Basement excavation simulated over 4 stages at 1.5m – 2.5m depths depending on layer thickness of geological unit.
- Groundwater was not modelled as the Macquarie Park Station infrastructure was designed to be drained and construction would normally result in the permanent lowering of the water table well below the excavation of the development.
- An assumed 10kPa pedestrian traffic surcharge is applied to the ground surface between the proposed development and Lane Cove Road.
- An assumed 20kPa vehicle traffic surcharge is applied to the ground surface above the ECRL caverns within the Roads and Maritime Services' Lane Cove Road corridor.
- Footing loads of 200kPa was applied for the existing building foundation (located within the site boundary to be demolished for the construction of the proposed development).
- Adjacent building loadings are not considered in the modelling.
- The finished floor level in the lowest basement level of the proposed Building A development (3 basement levels) is RL 50.80m AHD.
- The finished floor level in the lowest basement level of the proposed Building B and C development (4 basement levels) is RL 48.0m AHD, however construction impacts from these buildings have not been assessed as the deepest basement lies outside the protection zone established for the ECRL tunnel infrastructure.
- The proposed footing locations and dimensions for proposed Building A have been approximated from drawings provided and have been modelled indicatively only.
- The proposed footing loadings for proposed Building A have been provided for one row of footings at the western excavation face only.
- Column A loading was applied to all footings except where Column B was specified.
- The shoring option for the Macquarie Park Station West Entry shaft was assumed to be comprised of 600mm dia. soldier piles on a 1.1m c/c spacing founded within 2m of competent rock (Ashfield Shale Class I/II or better).
- The shoring design for the proposed development was not provided and it is assumed to be consistent with that applied to the existing Macquarie Park Station West Entry shaft (i.e. 600mm dia. soldier piles on a 1.5m c/c spacing founded within 2m of competent rock).
- Tieback anchors were not modelled due to possible encroachment of the ECRL infrastructure protection zone.
- The ECRL infrastructure excavation walls within Sandstone Class II or better were assumed to be left unsupported.

— Joint/defect networks were modelled to represent known defects in the geology as encountered during construction of the ECRL infrastructure, previously documented by Parsons Brinckerhoff within a geotechnical report for this proposed development site (396 lane Cove Road Macquarie Park, Geotechnical desktop study report, reference: 2108235A/LIJ/0640, dated 14th September 2010).

— Utilities were assumed to be within 2m below ground surface underneath the rail corridor and road corridor and have not been considered in this assessment.

4.2 ADOPTED GEOTECHNICAL GROUND MODEL

The geotechnical ground model adopted for the numerical modelling will be consistent across both cross sections as the sections are in close proximity. Based on the Geotechnical Desktop Study Report (PB, Ref: 2108235A/lij/0640, 2010) produced for proposed site, the following profile was identified in close proximity to the Macquarie Park Station west entry shaft (at the approximate location of the cross sections used for the modelling) and adopted for the ground model:

— Residual soil (RS) from approximately RL 59.0m AHD to 57.8m AHD.

— Ashfield Shale Class V (AS_c) from approximately RL57.8 m AHD to RL 55.8m AHD.

— Ashfield Shale Class III/IV (AS_b) from approximately RL55.8m AHD to RL 51.6m AHD.

— Ashfield Shale Class I/II (AS_a) from approximately RL51.6m AHD to RL 47.5m AHD.

— Interbedded/interlaminated Siltstone and Sandstone (Mittagong Formation) Class I/II (MF_a) from approximately RL47.5m AHD to RL 42.0m AHD.

— Hawkesbury Sandstone from below RL 42m AHD.

Figure 4.1 and Figure 4.2 below contain a graphical representation of the geological model adopted for the Phase2 numerical analysis.

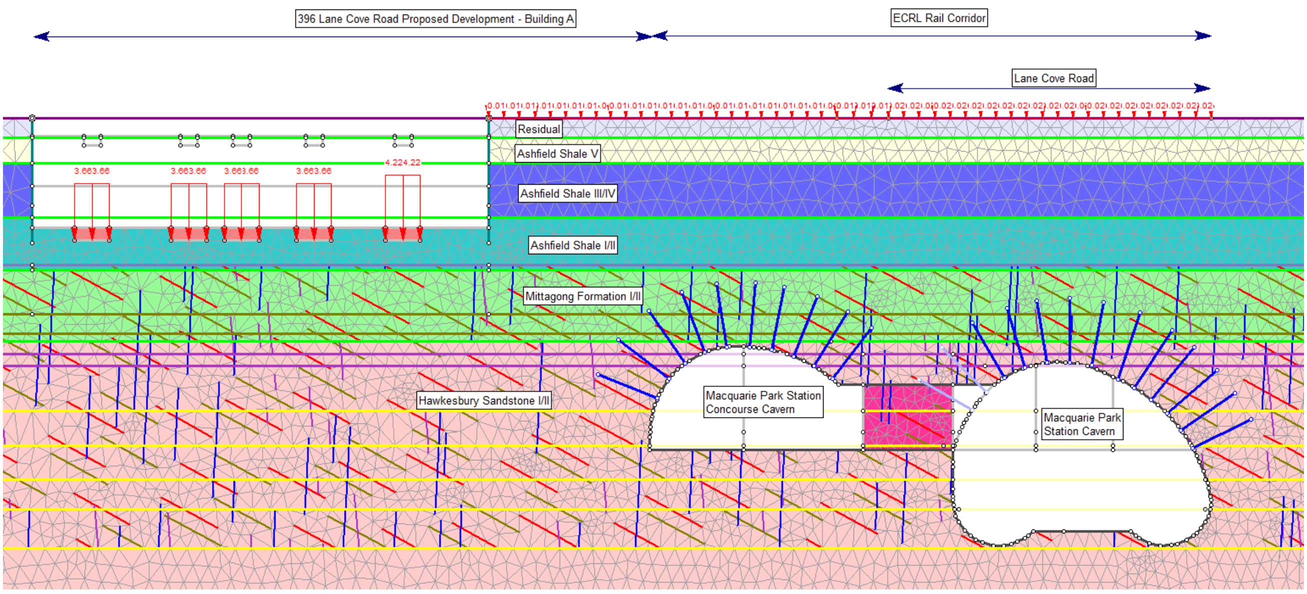


Figure 4.1 Geotechnical model for Phase2 analysis (Section 1)

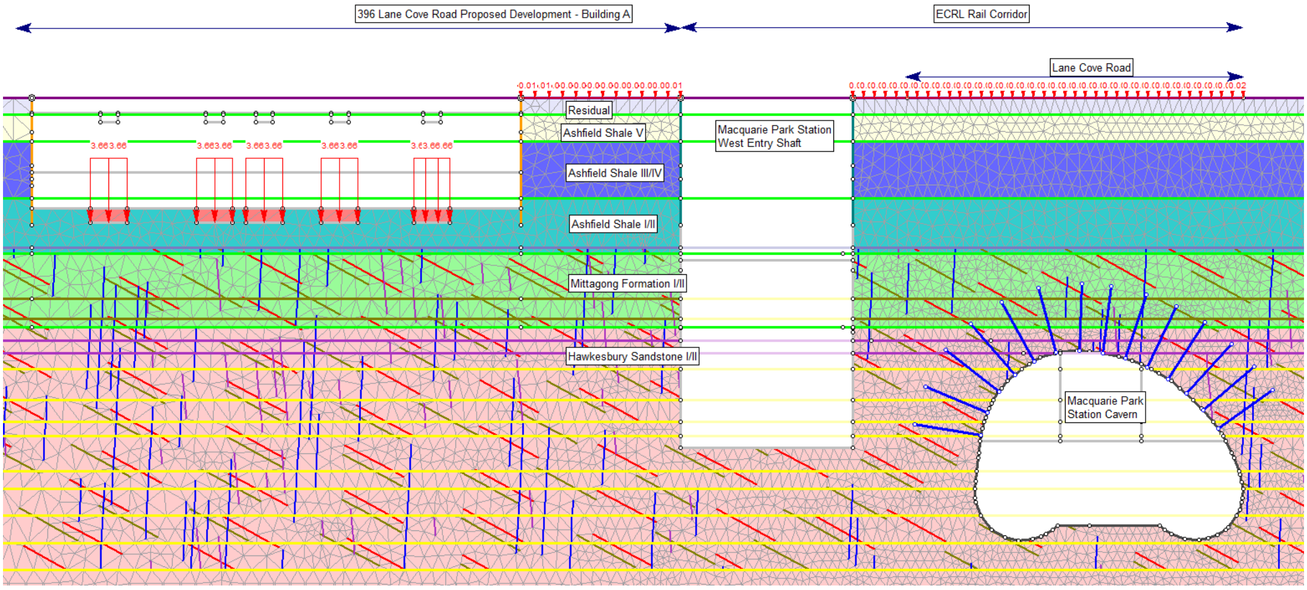


Figure 4.2 Geotechnical model for Phase^2 analysis (Section 2)

4.3 GEOTECHNICAL DESIGN PARAMETERS

Table 4.2 and Table 4.2 below presents the geotechnical parameters adopted for this impact assessment. These parameters are developed based on the industry well accepted Sydney's Rock Classification System (Bertuzzi & Pells 2002, Bertuzzi 2014) and based on a published paper titled "Back-analysis of Monitoring Results at Macquarie Park Station, Epping to Chatswood Rail Line" (K. Chan and P. Stone, June 2006).

Table 4.1 Geotechnical design parameters adopted for Phase^2 numerical analysis

MATERIAL TYPE	**UNIT WEIGHT (kN/m^3)**	**UCS (MPa)**	**MODULUS (MPa)**	**POISSON'S RATIO (ν)**	**Q RANGE**	**GSI**
Residual	20	0	40	0.30	N/A	N/A
Ashfield Shale V (AS_c)	22	1	200	0.25	0.005-0.3	20
Ashfield Shale III/IV (AS_b)	22	7	2000	0.20	0.01-20	30-45
Ashfield Shale I/II (AS_a)	22	10	3000	0.20	1.5-100	55-65
Mittagong Formation I/II (MF_a)	22	10	3000	0.20	1.5-100	55-65
Sandstone I/II (HAW_a)	23	35	8000	0.2	22.5-150	65-75

Table 4.2 Failure criteria parameters adopted for Phase[2] numerical analysis

MATERIAL TYPE	MOHR-COULOMB CRITERION		
	TENSILE STRENGTH (MPa)	FRICTION ANGLE (DEG)	COHESION (MPa)
Residual	0	26	0.005
Ashfield Shale V (AS_c)	0.05	28	0.01
Ashfield Shale III/IV (AS_b)	0.1	30	0.05
Ashfield Shale I/II (AS_a)	0.6	32	0.3
Mittagong Formation I/II (MF_a)	0.6	32	0.3
Sandstone I/II (HAW_a)	1.0	35	0.5

4.4 FIELD STRESS

The field stresses have significant impact on both deep excavation conditions and induced ground movements in the immediate area of the excavation works. High in-situ lateral stresses can be “locked in” within the bedrock stratum.

Several papers have been published giving detailed analyses of the natural stress field in the Triassic rocks of the Sydney Basin. Papers by Enever (1999) and McQueen (2004), presented recommended models to describe the stress field for tunnelling and excavation projects in the Sydney Basin:

- Enever (1999) suggests for stress magnitudes for rocks down to 200m depth:
  - Vertical Stress σ_V = Overburden pressure.
  - Major Horizontal Stress $\sigma_H = 2.5\sigma_V$, orientation 28° magnetic (40 ° true, NNE).
  - Minor Horizontal Stress $\sigma_h = 1.5\sigma_V$, orientation 118 ° magnetic (130 ° true, ESE).
- McQueen (2004) indicates the following “steeped” model of upper limit field stresses:
  - Down to 20m depth below the ground surface:
    - Major Horizontal Stress $\sigma_H = 2.5\text{MPa} + \sigma_V$ down to 200m depth.
    - Minor Horizontal Stress $\sigma_h = 2.0\text{MPa} + \sigma_V$ down to 200m depth.
  - And then down to 200m depth below the ground surface:
    - Major Horizontal Stress $\sigma_H = 6.5\text{MPa} + \sigma_V$.
    - Minor Horizontal Stress $\sigma_h = 4.5\text{MPa} + \sigma_V$.
- Pells (2002) proposed that, away from topographic effects, the stress field may be approximated by the equations:
  - Vertical Stress σ_V = Overburden pressure.
  - Major Horizontal Stress $\sigma_H = \sigma_{NS} = 1.5\text{MPa} + 1.2$ to $2.0\ \sigma_V$.
  - Minor Horizontal Stress $\sigma_h = \sigma_{WE} = 0.5$ to $0.7\ \sigma_V$.

The following models were used during design of the Epping to Chatswood Rail Link (ECRL) for all stations except Delhi Road Station:

$$\sigma_{H\ (NS)} = 1.0\text{MPa} + 4.5\ \sigma_V$$

$$\sigma_H / \sigma_{h(WE)} = 1.5$$

For the subject site at 396 Lane Cove Road, Macquarie Park, it is considered that the field stress regime adopted for ECRL station design is an appropriate basis for development of upper and lower bound modelling stresses.

The following upper and lower bound ground stresses were applied in the numerical analysis:

Upper bound: $\sigma_{H(NS)} = 1.0\text{MPa} + 4.5\ \sigma_V$ and $\sigma_H / \sigma_{h(WE)} = 1.5$

Lower bound: $\sigma_V = \sigma_H = \sigma_h = 1.0$

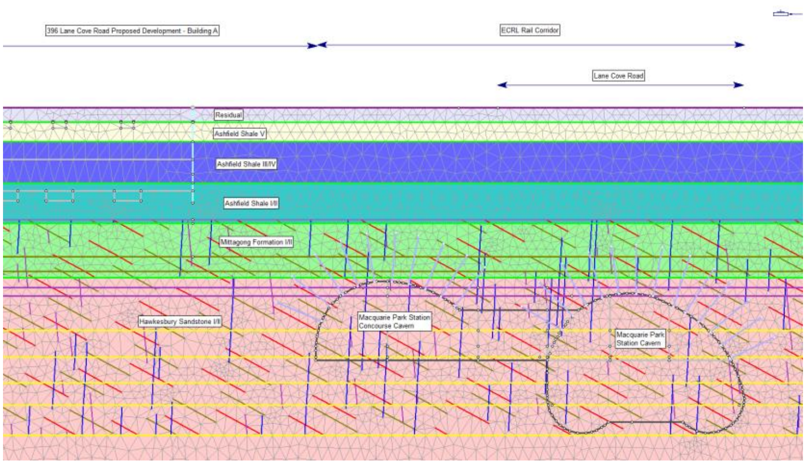
The upper bound stresses are applied to fresh, good quality sandstones and shales (Class I and II). In poorer quality rock masses, ko = 1 is applied, i.e. $\sigma_V = \sigma_H = \sigma_h = 1.0$.

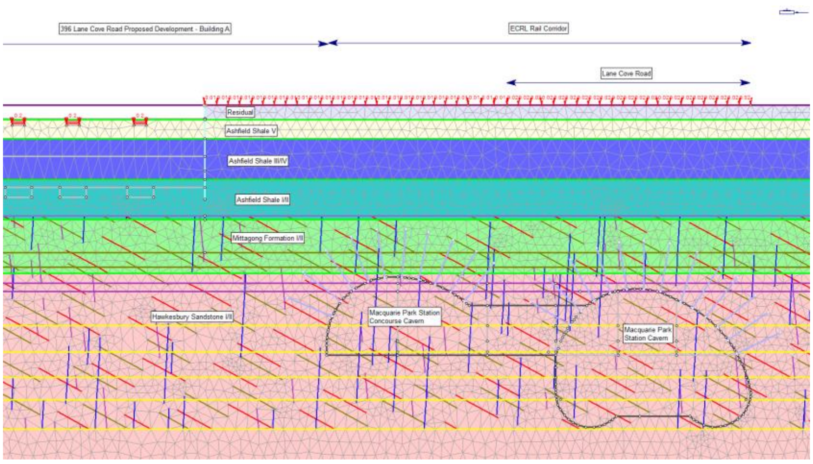
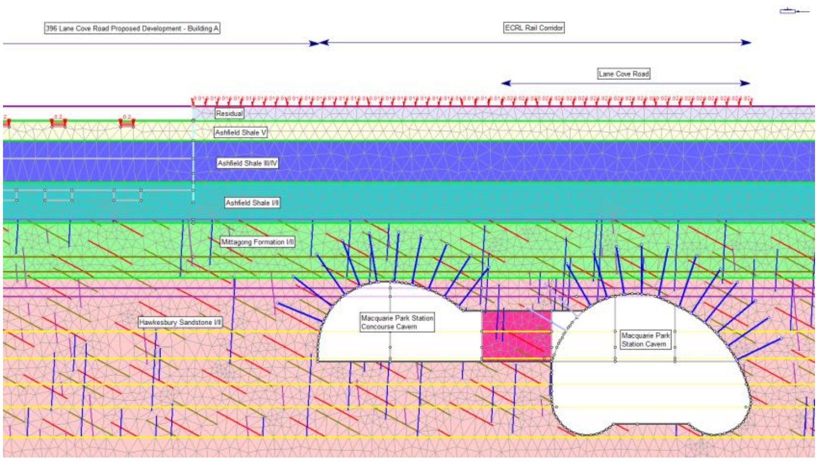
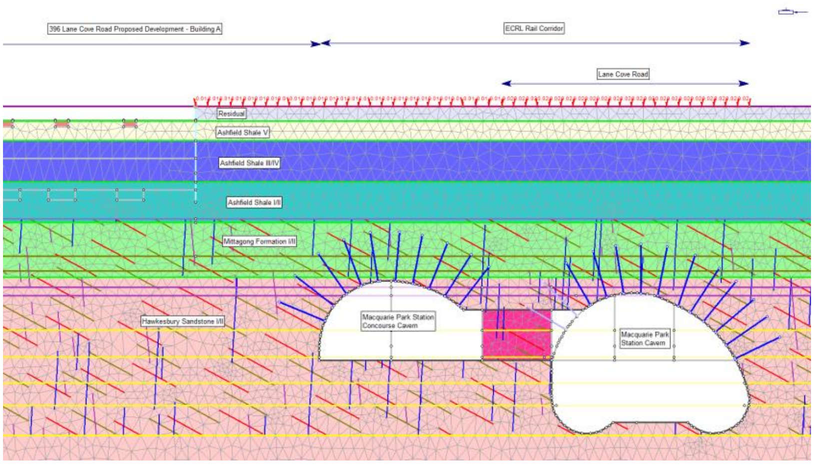
4.5 EXCAVATION AND CONSTRUCTION SEQUENCES

Basement excavation of the 396 Lane Cove Road site will be modelled to minimise the change in integrity of surrounding rock. This may include localised saw cutting of the final excavated surface to minimise development of rock mass fracturing.

Table 4.3 and Table 4.4 present the construction sequences adopted in the numerical modelling for the respective cross sections.

Table 4.3 Construction sequences adopted in the Phase[2] numerical modelling (Section 1)

STAGE NO	DESCRIPTION	ILLUSTRATIONS
1	Initialise the in-situ field stresses.	

STAGE NO	**DESCRIPTION**	**ILLUSTRATIONS**
2	Existing basement and foundation loads (assumed to be prior to ECRL Macquarie Park Station construction)	
3	Macquarie Park Station - concourse cavern and station cavern construction including installation of tunnel support	
4	Demolition of existing basement and removal of footing loads	

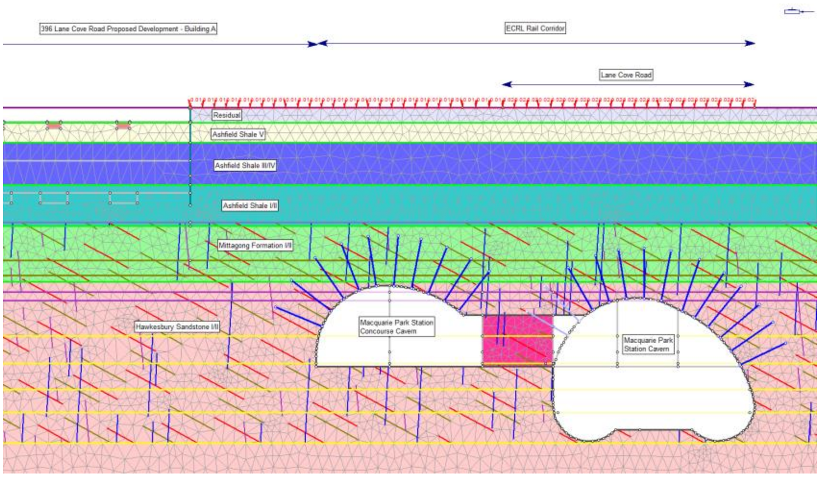
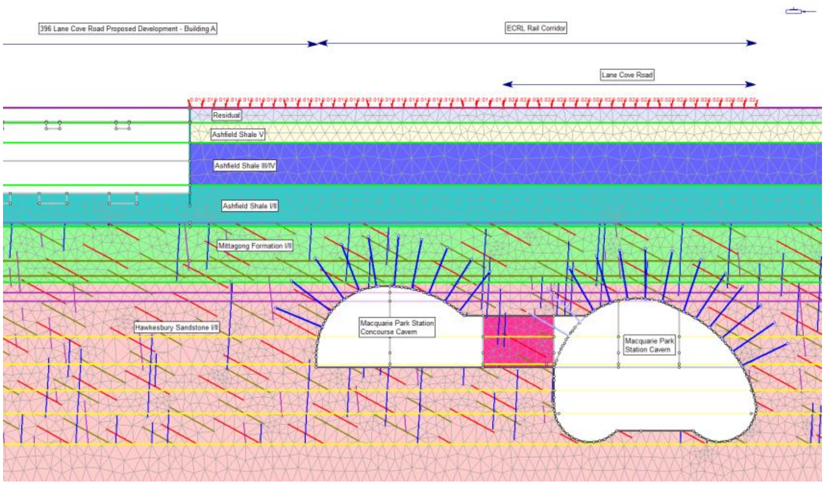
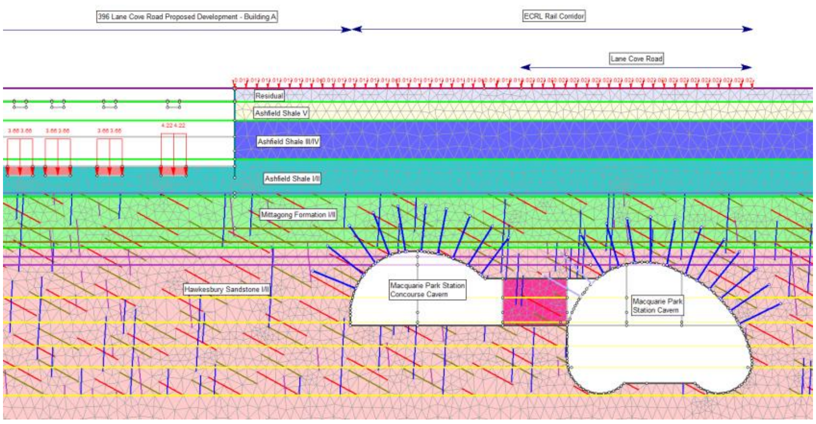
<table>
<tr><th>STAGE NO</th><th>DESCRIPTION</th><th>ILLUSTRATIONS</th></tr>
<tr><td>5</td><td>Install piled walls for proposed development basement to RL 49.6 m AHD (2.0m embedment into $AS_{a)}$</td><td>
</td></tr>
<tr><td>6</td><td>Excavate to the proposed final basement level (50.8 m RL).</td><td>
</td></tr>
<tr><td>7</td><td>Install pad footings and foundation loading</td><td>
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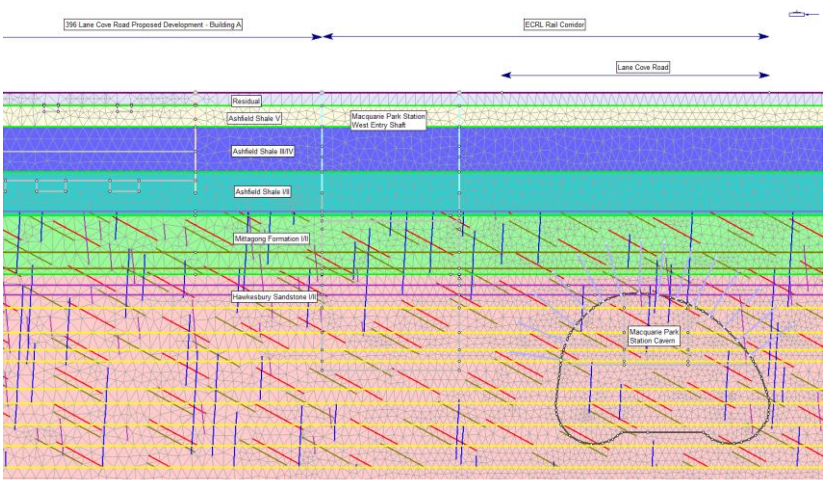
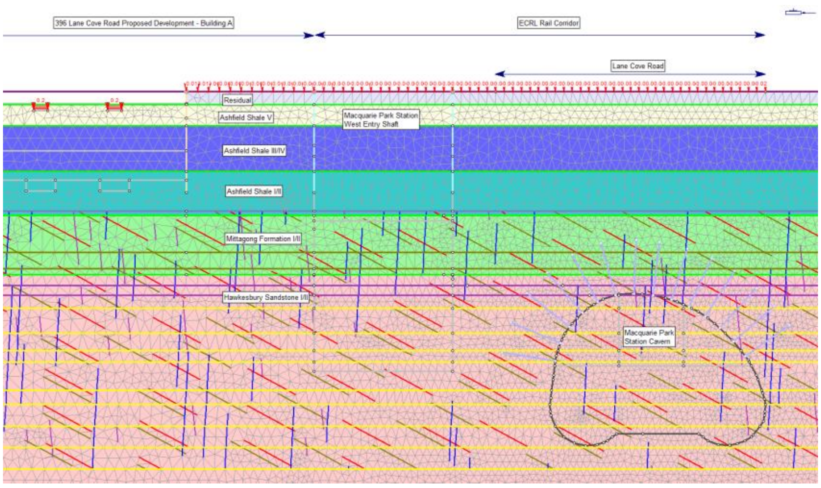
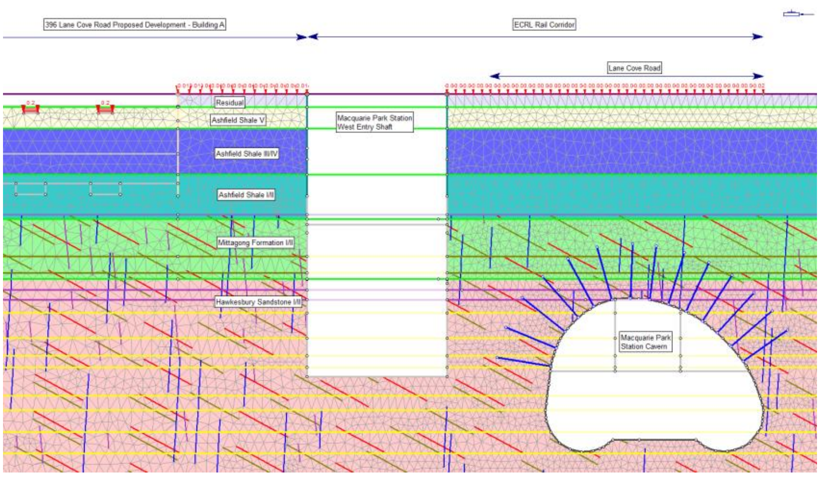
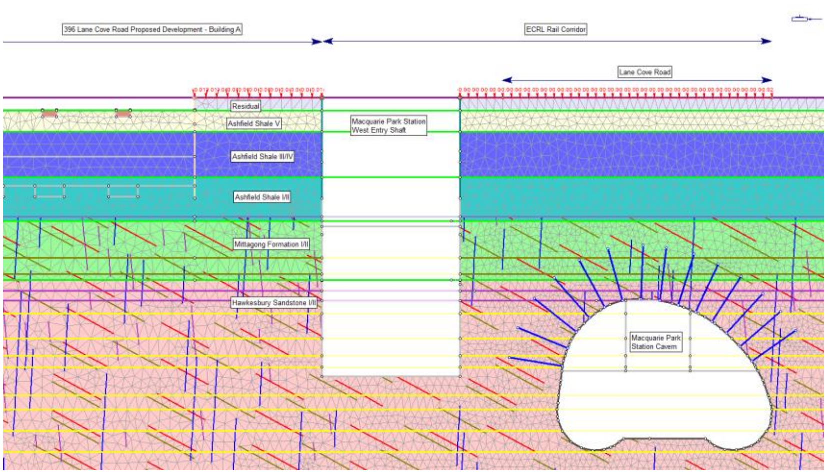
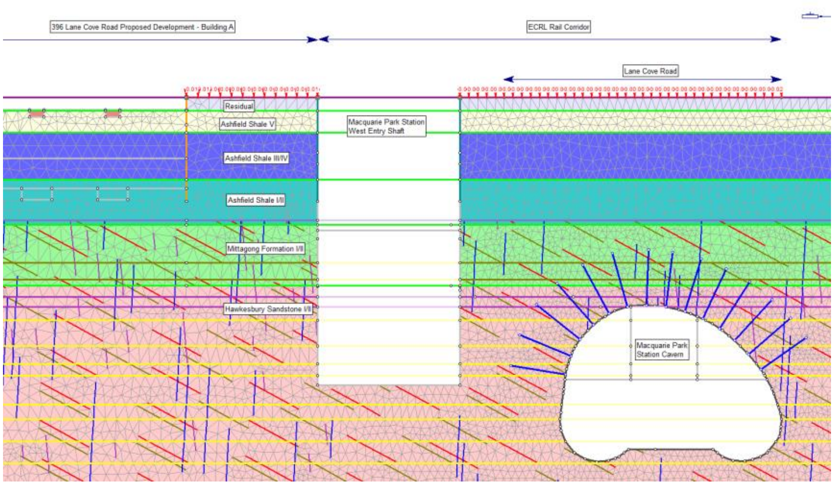
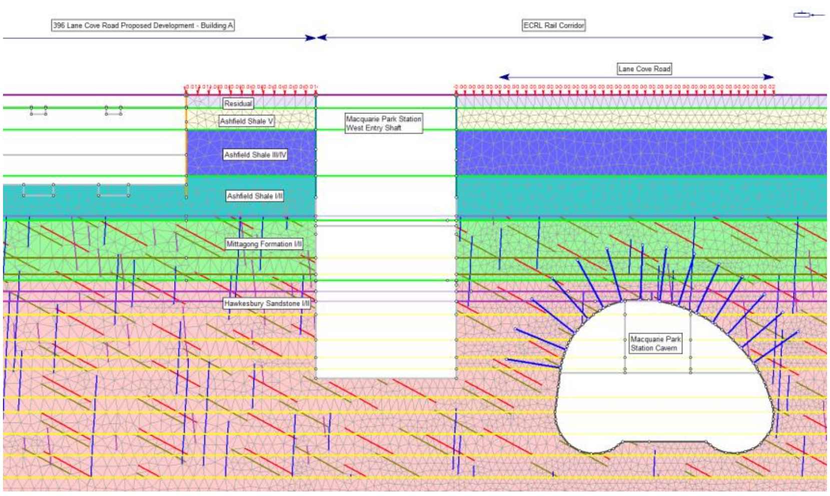
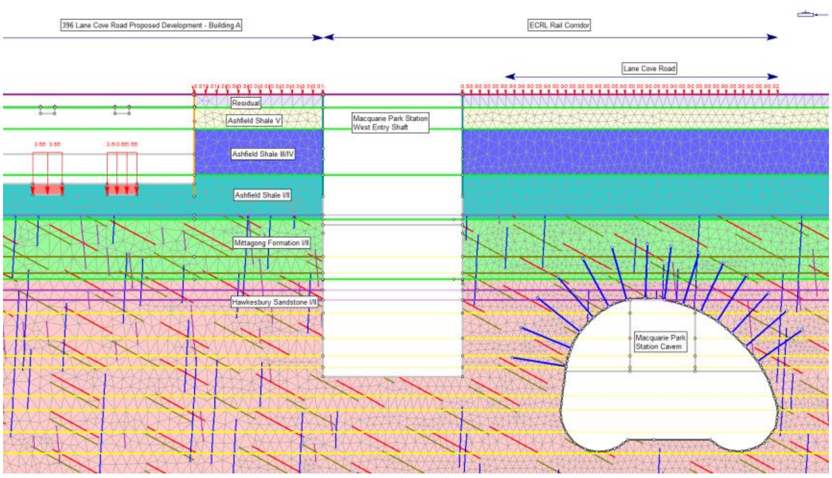
Table 4.4 Construction sequences adopted in the Phase[2] numerical modelling (Section 2)

STAGE NO	DESCRIPTION	ILLUSTRATIONS
1	Initialise the in-situ field stresses.	
2	Existing basement and foundation loads (assumed to be prior to ECRL Macquarie Park Station construction)	
3	Macquarie Park Station - concourse cavern and station cavern construction including installation of tunnel support	

<table>
<tr><th>STAGE NO</th><th>DESCRIPTION</th><th>ILLUSTRATIONS</th></tr>
<tr><td>4</td><td>Demolition of existing basement and removal of footing loads</td><td>
</td></tr>
<tr><td>5</td><td>Install piled walls for proposed development basement to RL 49.6 m AHD (2.0m embedment into AS$_{a)}$</td><td>
</td></tr>
<tr><td>6</td><td>Excavate to the proposed final basement level (50.8 m RL).</td><td>
</td></tr>
</table>

STAGE NO	DESCRIPTION	ILLUSTRATIONS
7	Install pad footings and foundation loading	

4.6 FOUNDATION AND SITE RETENTION

Design drawings for the existing west entry station shaft indicate a site retention system comprising of 600mm diameter soldier piles on a 1.1m spacing (c/c). Design drawings for the ECRL Macquarie Park Station indicated that the soldier piles will have a minimum embedment depth of 0.5m into Ashfield Shale Class II or better. The modelling adopted an embedment depth of 2.0m into Ashfield Shale Class I/II. Shotcrete application in between the soldier piles was not considered in the analysis.

The site retention system assumed for the modelling of the proposed development comprised of 600mm diameter soldier piles on a 1.5m spacing (c/c). It was assumed that the soldier piles will have a minimum embedment depth of 2.0m into Ashfield Shale Class II or better to be consistent with that adopted for the existing west entry shaft support. Shotcrete application in between the soldier piles was not considered in the analysis.

Anchors were not considered in the modelling for the site retention of the proposed development due to the proximity to the ECRL infrastructure and encroachment into the protection zones.

The foundation design for the proposed development building structures was not provided, however through consultation with Enstruct Group (the client's Structural Engineer) on the 15th of December 2017, it was advised that that foundation would likely comprise of pad footings founded within Ashfield Shale Class II or better. Given the column loading provided, Enstruct Group suggested that the appropriate pad foundation size for use in the modelling should be at least 2.7 x 2.7 x 1.0m (length x width x depth). The foundation loads provided and adopted in the modelling are:

- Column A = 26,700 kN (unfactored working load), equivalent to 3.66 MPa for suggested pad footing dimensions.
- Column B = 30,800 kN (unfactored working load), equivalent to 4.22 MPa for suggested pad footing dimensions.

The equivalent uniformly distributed load was applied across the footing base. It was assumed that Column A loads were applied at all footing locations except where specified as Column B as shown in the Figure 4.3 below.

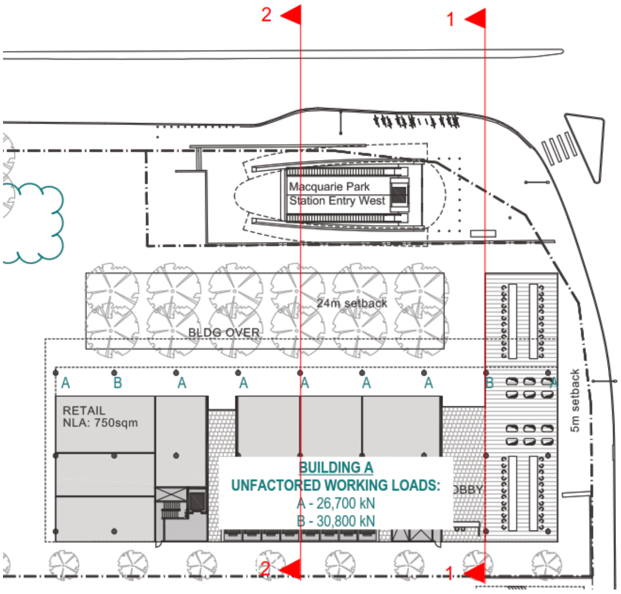


Figure 4.3 Footing locations and loads for numerical modelling Section 1 and Section 2

5 RESULTS AND DISCUSSION

5.1 GROUND MOVEMENT AND STRESS REDISTRIBUTION

The development's basement excavation will alter the in-situ stress regime (causing stress relief or concentration) in surrounding rock strata thereby causing displacements within the rock mass which will generally be concentrated along geological structures.

One option for site retention system will comprise soldier piles founded in competent rock with concrete/shotcrete infill panels, supported by temporary anchors and followed by strut bracing (if required). Excavation within the competent rock may involve temporary rock anchors/bolts followed by strut bracing from permanent works.

This method limits ground movement at the edges of the excavation in the upper weathered materials. Movement of ground deformation will be restricted to ensure no damage occurs to adjacent buildings and public infrastructure.

Figure 5.1 (Hewitt, Burkitt & Baskaran, 2005) indicates the predicted maximum horizontal movement at the top of the wall. Case history data indicates that for a 15m deep excavation, the deflection range is between 15-25mm for an anchored system.

Based on published analysis of stress relief behaviour in vertical rock cuts (Glastonbury & Fell 2002 Ref 2), the horizontal stress-relief displacement induced by the basement excavation is expected to be less than 5mm at 15m from the basement excavation.

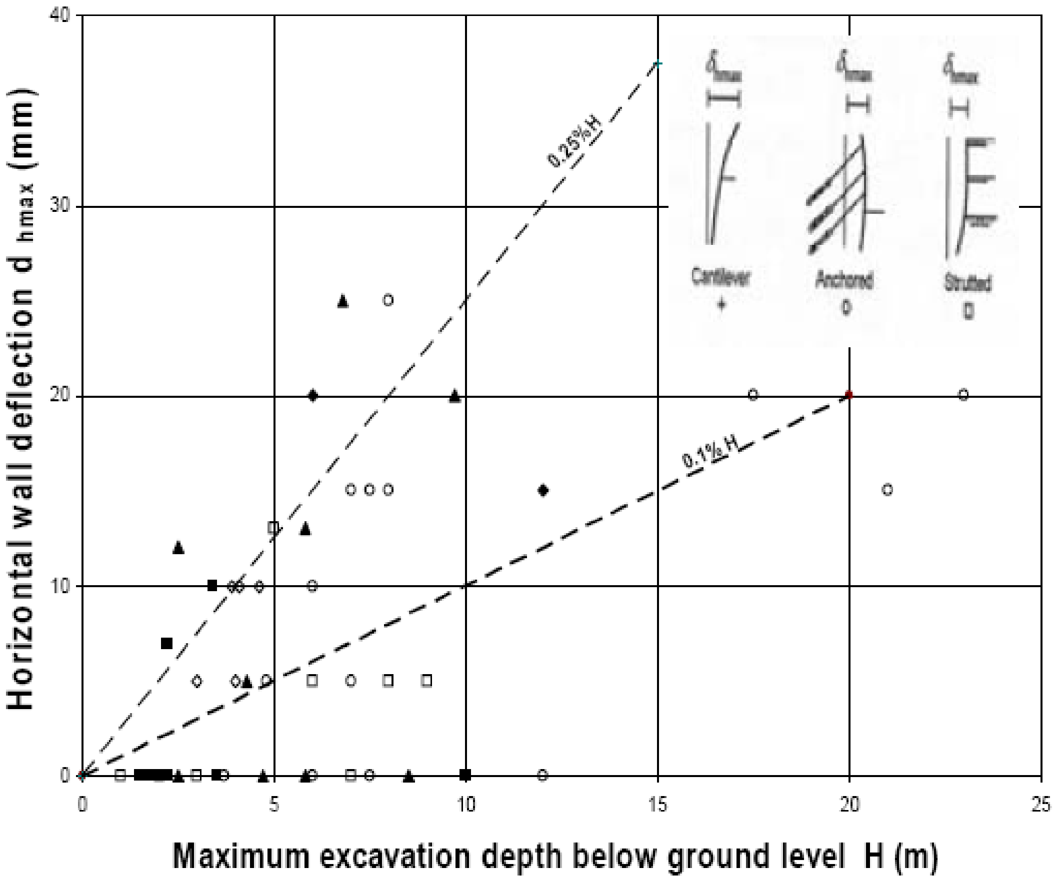


Figure 5.1 Wall deflection with respect to excavation depth

This magnitude of displacement is not expected to have an adverse effect on the rock mass or defect shear strengths in the area surrounding the existing station cavern, entry shafts and running tunnels. The horizontal stress redistribution should be studied in detail as the locked in horizontal stresses are greater than the vertical stress and the due to the near horizontal bedding planes, the magnitude of movements could cause distress of the infrastructure support systems.

A summary of results from the two dimensional Phase^2 numerical analysis of Section 1, addressing the impact to the Macquarie Park Station cavern and concourse cavern is presented in Table 5.1 and Table 5.2.

The summary of results from the two dimensional Phase^2 numerical analysis of Section 2, addressing the impact to the Macquarie Park Station cavern and west entry shaft is presented in Table 5.3 and Table 5.4

The numerical analysis outputs are presented in Appendix C.

5.1.1 STATION CAVERN AND CONCOURSE CAVERN (SECTION 1)

5.1.1.1 GROUND MOVEMENT

The calculated maximum vertical and horizontal deformations at the concourse cavern after proposed basement excavation (including application of foundation loads) are approximately 2mm and 1mm respectively. The calculated maximum vertical and horizontal deformations for the concourse cavern rock bolt support are approximately 1mm and 1mm respectively. The differential movement is calculated to be less than 1mm for the concourse cavern structure and rock bolt support.

The calculated maximum vertical and horizontal deformations at the station main cavern after proposed basement excavation (including application of foundation loads) are approximately 1mm and 2mm respectively. The calculated maximum vertical and horizontal deformations for the station cavern rock bolt supports are approximately 1mm and 1mm respectively. The differential movement is calculated to be less than 1mm for the station cavern structure and rock bolt support.

The calculated maximum vertical and horizontal deformations at the surface level adjacent to the completed basement excavation are approximately 3mm and 3mm respectively. The predicted maximum vertical and horizontal deformations at the finished floor level of the completed basement excavation are approximately 2mm and 1mm respectively. The differential movement is calculated to be less than 1mm for the soldier pile wall retention system.

Table 5.1 Summary of calculated vertical and horizontal deformation (in mm) for Section 1

DESCRIPTION	VERTICAL DISPLACEMENT	HORIZONTAL DISPLACEMENT
Station concourse cavern (at crown)	2	1
Station concourse cavern (within rock bolt support)	1	1
Station main cavern (at crown)	2	1
Station main cavern (within rock bolt support)	1	1
Ground surface	3	3
Bottom of excavation (Basement FFL)	2	1

5.1.1.2 STRESS DISTRIBUTION

The maximum stresses calculated at the concourse cavern face prior to and after proposed basement excavation including application of foundation loads are approximately 6.0 MPa and 6.75 MPa respectively. The maximum stresses calculated at the rock bolt support for the concourse cavern are approximately 3.75 MPa and 3.75 MPa respectively. No change in stress distribution was found at the rock bolt support for the concourse cavern.

The maximum stresses calculated at the station cavern face prior to and after proposed basement excavation (including application of foundation loads) are approximately 11.25 MPa and 11.25 MPa respectively. The maximum stresses calculated at the rock bolt support for the station cavern are approximately 3.0 MPa and 3.0 MPa respectively. No change in stress distribution was found at the rock bolt support for the station cavern.

The maximum stress at the surface level of the basement excavation face adjacent to the ECRL infrastructure is calculated to be approximately 0 MPa. The maximum stress at the finished floor level of the basement excavation face adjacent to the ECRL infrastructure after proposed construction is complete is calculated to be approximately 1.5 MPa.

Table 5.2 Summary of stress distribution results for Section 1

DESCRIPTION	MAJOR STRESS DISTRIBUTION PRIOR TO BASEMENT EXCAVATION (MPa)	MAJOR STRESS DISTRIBUTION AFTER BASEMENT EXCAVATION (MPa)
Station concourse cavern (at crown)	6	6.75
Station concourse cavern (within rock bolt support)	3.75	3.75
Station main cavern (at crown)	11.25	11.25
Station main cavern (within rock bolt support)	3	3
Ground surface	0	0
Bottom of excavation (Basement FFL)	2.25	1.5

5.1.2 STATION CAVERN AND WEST ENTRY SHAFT (SECTION 2)

The calculated maximum vertical and horizontal deformations at the station west entry shaft after proposed basement excavation (including application of foundation loads) are approximately 3mm and 10mm respectively.

The calculated maximum vertical and horizontal deformations at the station main cavern after proposed basement excavation (including application of foundation loads) are approximately 1mm and 2mm respectively. The calculated maximum vertical and horizontal deformations for the station cavern rock bolt supports are approximately 0mm and 2mm respectively. The differential movement is calculated to be less than 1 mm for the station cavern structure and rock bolt support.

The calculated maximum vertical and horizontal deformations at the surface level adjacent to the completed basement excavation are approximately 5mm and 38mm respectively. The predicted maximum vertical and horizontal deformations at the finished floor level of the completed basement excavation are approximately 5mm and 38mm respectively. The differential movement is calculated to be less than 1mm for the soldier pile wall retention system.

The 38mm horizontal displacement is localised to the face of the excavation of the proposed development.

Table 5.3 Summary of calculated vertical and horizontal deformation (in mm) for Section 2

DESCRIPTION	VERTICAL DISPLACEMENT	HORIZONTAL DISPLACEMENT
Station west entry shaft (at sidewall)	3	10
Station main cavern (at crown)	1	2
Station main cavern (within rock bolts support)	0	2
Ground surface	5	38

DESCRIPTION	VERTICAL DISPLACEMENT	HORIZONTAL DISPLACEMENT
Bottom of excavation (Basement FFL)	5	38

5.1.2.1 STRESS DISTRIBUTION

The maximum stresses calculated at the Station west entry shaft prior to and after proposed basement excavation (including application of foundation loads) are approximately 3.10 MPa and 3.85 MPa respectively.

The maximum stresses calculated at the station cavern face prior to and after proposed basement excavation (including application of foundation loads) are approximately 7.60 MPa and 7.60 MPa respectively. The maximum stresses calculated at the rock bolt support for the station cavern are approximately 1.60 MPa and 1.60 MPa respectively. No change in stress distribution was calculated at the excavation or the rock bolt support for the station cavern.

The maximum stress at the surface level of the basement excavation face adjacent to the ECRL infrastructure is calculated to be approximately 0.10 MPa. The maximum stress at the finished floor level of the basement excavation face adjacent to the ECRL infrastructure after proposed construction is complete is calculated to be approximately 0.85 MPa.

Table 5.4 Summary of calculated stress distribution results for Section 2

DESCRIPTION	MAJOR STRESS DISTRIBUTION PRIOR TO BASEMENT EXCAVATION (MPA)	MAJOR STRESS DISTRIBUTION AFTER BASEMENT EXCAVATION (MPA)
Station west entry shaft (at sidewall)	3.10	3.85
Station main cavern (at crown)	7.60	7.60
Station main cavern (within rock bolts support)	1.60	1.60
Ground surface	0.10	0.10
Bottom of excavation (Basement FFL)	0.85	0.10

5.2 FOUNDATION AND SITE RETENTION

Pad foundations will likely be utilised for the proposed building foundation, however detailed arrangements of the foundation have not been provided. Thus, the preliminary structure design for the shoring system and foundation was reviewed and a simplified version adopted in the analysis.

Through consultation with Enstruct Group (the client's Structural Engineer) on the 15th of December 2017, the adopted foundation design for Building A of the proposed development consists of pad footings founded within Ashfield Shale Class II or better. The adopted pad footing size was relative to the capacity to absorb the column loading provided in the preliminary structural drawings.

The Building A basement wall and the foundations lie outside the First Reserve protection zone of the ECRL structures. It is noted that the part of the proposed pad foundations for Building A is founded within the Second Reserve influence zone, hence load from the building will be transferred into the zone.

Based on the separation distance of the basement and ECRL infrastructure, it is anticipated that the horizontal load transfer to the proposed tunnels will be low. This will be further investigated in the design stage.

Temporary anchors may be utilised for support works during the excavation and these anchors will be de-stressed on completion. No anchors are to encroach the protection zone. Detailed assessments will be undertaken during the design stage to study the impact of transient temporary anchors loads within the influence zones.

Furthermore, RailCorp Engineering Specification SPC207 Track Monitoring Requirements for Undertrack Excavation, states in Section 8.3.5 that the operation (excavation of the basement) may only be continued if the track movement (within the station cavern) due to boring (or in this case the excavation of the basement and installation of foundation and site retention) is negligible (i.e. $\leq$ 4mm). The maximum differential vertical and horizontal settlements calculated within the station cavern below the rail track are less than 2mm, thus within the acceptable limits.

5.3 CONSTRUCTION IMPACTS

The staging of the basement excavation will be addressed during design stage to minimise ground movements. An excavation sequence plan will be developed using numerical modelling to predict ground movements of each stage.

Basement excavation methods are expected to be limited by local restrictions on ground vibration and noise. It is expected that impact hammering or saw-cutting will likely be employed. These methods are not expected to adversely impact ground conditions in the rock.

Expected peak ground acceleration impacts will be addressed during the designs stage.

5.4 CHANGES TO GROUNDWATER REGIME

Local groundwater levels prior to the construction of Macquarie Park Station indicate that the groundwater level could be approximately 11m deep based on ground water monitoring information gathered from adjacent sites. It is likely that the groundwater has been lowered by the construction of the drained station caverns.

6 CONCLUSIONS AND RECOMENDATIONS

The footprint of the proposed development does not encroach within the protection zone of the Macquarie Park Station cavern, concourse cavern or west entry shaft as defined by the TIDC guideline. The minimum horizontal separation between the proposed basement excavation and the ECRL infrastructure is approximately 11.5m, which is outside the protection zone of the ECRL infrastructure. While there is minor encroachment on the influence zone (Second Reserve), it considered that this encroachment would not likely to have adverse impact the integrity of the ECRL structural support systems.

The basement excavation work will result in stress relief which can result in unacceptable movement along the horizontal bedding and subsequently undermine the structural integrity of the ECRL tunnel cavern support systems. A preliminary analysis of this movement based on known geological information and proposed development foundation configuration indicate that these movement will be limited to a value of less than 3mm within the vicinity of the station caverns and less than 10mm within the vicinity of the station west entry shaft.

RailCorp Engineering Specification SPC207 allows for a maximum of 4mm track movement. Considering the maximum calculated vertical or horizontal deformation within the station cavern is less than 3mm, the calculated deformation below the rail track will be within the acceptable limits. Based on the information from ECRL investigations and as summarised in Section 3.2, it is known that bedding planes / joints are present in the cavern areas and associated tunnels, which may have an adverse effect on the ECRL infrastructure not captured in the modelling.

The modelling calculates that the soldier pile wall on the excavated wall of the proposed development opposite the west entry shaft will deform laterally up to 38mm. At a horizontal distance of 1.5m from the soldier pile wall, the 38mm calculated deformation is reduced to 10mm. The high lateral deformation appears to be localised at the excavation within the site boundary and is unlikely to adversely affect the west entry shaft support system.

Considering the site retention system adopted in the model is only a preliminary option, further analysis in the detailed design is required to reduce the lateral deformation on the development's excavation face and further reduce the impact on the adjacent ECRL infrastructure. This may involve the installation and stressing of temporary ground anchors during excavation, increasing the pile diameter of the soldier piles or decreasing the spacing between piles. Any temporary ground anchor will need to be destressed following completion of basement works. The encroachment and proposed structural arrangements within these areas need to satisfy loading requirements and should be reviewed in more detail.

The site retention system and building foundation adopted in the modelling is subject to change. Detailed analysis is required to be carried out and a staged excavation system will need to be defined during the design stage to accurately predict the movements based on more detailed site retention and foundation information as it comes available. To limit the horizontal and vertical impacts on the existing ECRL infrastructure, a suitably rigid retention system should be designed to constrain the movement of low strength Ashfield Shale layers (Class IV or worse) and residual soil/fill in the upper part of the excavation. This requires the provision of detailed foundation plans, those of which are currently not available.

Although the magnitude of movement of the caverns and the support systems is considered to be low and acceptable, any movement of the ECRL infrastructure due to construction activities from the development and due to adverse geological faults will need to be monitored to confirm the acceptance of this limit. A detailed monitoring plan is required during the design stage to address the required monitoring programme of the works. The primary objective of the monitoring is to confirm the predictive results of the numerical modelling. Additionally, it will provide a response action plan if the set triggers are breached.

7 LIMITATIONS

This report has been prepared on behalf of Winten Property Group and Frasers Property to address specific project requirements. By necessity, this report has been limited to a conceptual assessment of the geotechnical and structural constraints associated with development in close proximity to the ECRL infrastructure. Detailed assessment of these issues would be required at the design development stage.

The proposed ECRL infrastructure details have been sourced from WSP in-house records. At the time of writing this report no further information was provided by Sydney Trains, hence the details are limited in its accuracy and will be subject to further review during the design stage.

This report provides an assessment in relation to potential impacts of the proposed 396 Lane Cove Road development on the existing railway corridor and ECRL infrastructure. It is provided as a basis for obtaining a non-objection to the proposed development from Sydney Trains to address planning approval requirements. The following limitations apply to this initial ground movement and impact assessment:

- Analysis of the ground movement and stress changes has been undertaken using proven empirical techniques which are typically used for impact assessment, and generally provide reasonable correlation between calculated and actual effects. This approach is primarily based on experience and takes into account rock structure interaction between the existing structure and ground adjacent to the proposed redevelopment for the assumed construction sequence.
- The calculated displacement represents an estimate of the ground movement which has been benchmarked against actual performance from other projects in the area, however it does not necessarily reflect the effect of variations in geological conditions or the way the ground will behave and deform around an excavation. Therefore, a comprehensive program of instrumentation and monitoring is required to confirm the actual ground and building response which can be compared against predictions and used to verify the accuracy of the analyses.
- The reported results of the geotechnical analysis by finite element methods implicitly involves numerical approximations. Consequently, while results have been reported to nearest mm, it is unlikely that their accuracy is to this order.
- The Phase^2 finite element analysis presented in this report assumes two-dimensional conditions, therefore the applied loadings are more conservative than actual loading environments. Consequently, the calculated deformation and stress concentrations are likely to be conservative.
- Discrepancies between numerical calculations and measured data could occur due to stress relief effects caused by earlier ECRL construction, the existing building basement within the site boundary and other basement excavations in the surrounding area. Significant variation in subsurface conditions from those anticipated should be reported to WSP for assessment.

While the preparation of appropriate design calculations, drawings and work procedures is a necessary precursor to deep excavation, such preparation is only part of the process and will not control all the risks associated with excavation. Other factors can lead to excessive ground movements. These factors would include:

- poor decision making by the construction staff
- priority given to cost and programme control over ground control
- poor anchor installation practice
- unexpected ground behaviour.

The risk of an incident due to such factors can be reduced and/or the consequences controlled by employing sufficient qualified and experienced staff, allowing sufficient time in the construction programme for preparing and reviewing the design calculations and drawings, and developing suitable risk mitigation measures, contingency plans (including for dealing with mechanical breakdowns) and work procedures before construction commences. Most importantly, management of the excavation and site retention by experienced staff and geotechnical professionals is essential to manage the risk associated with deep excavation.

8 REFERENCES

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APPENDIX A

PROPOSED DEVELOPMENT PLANS

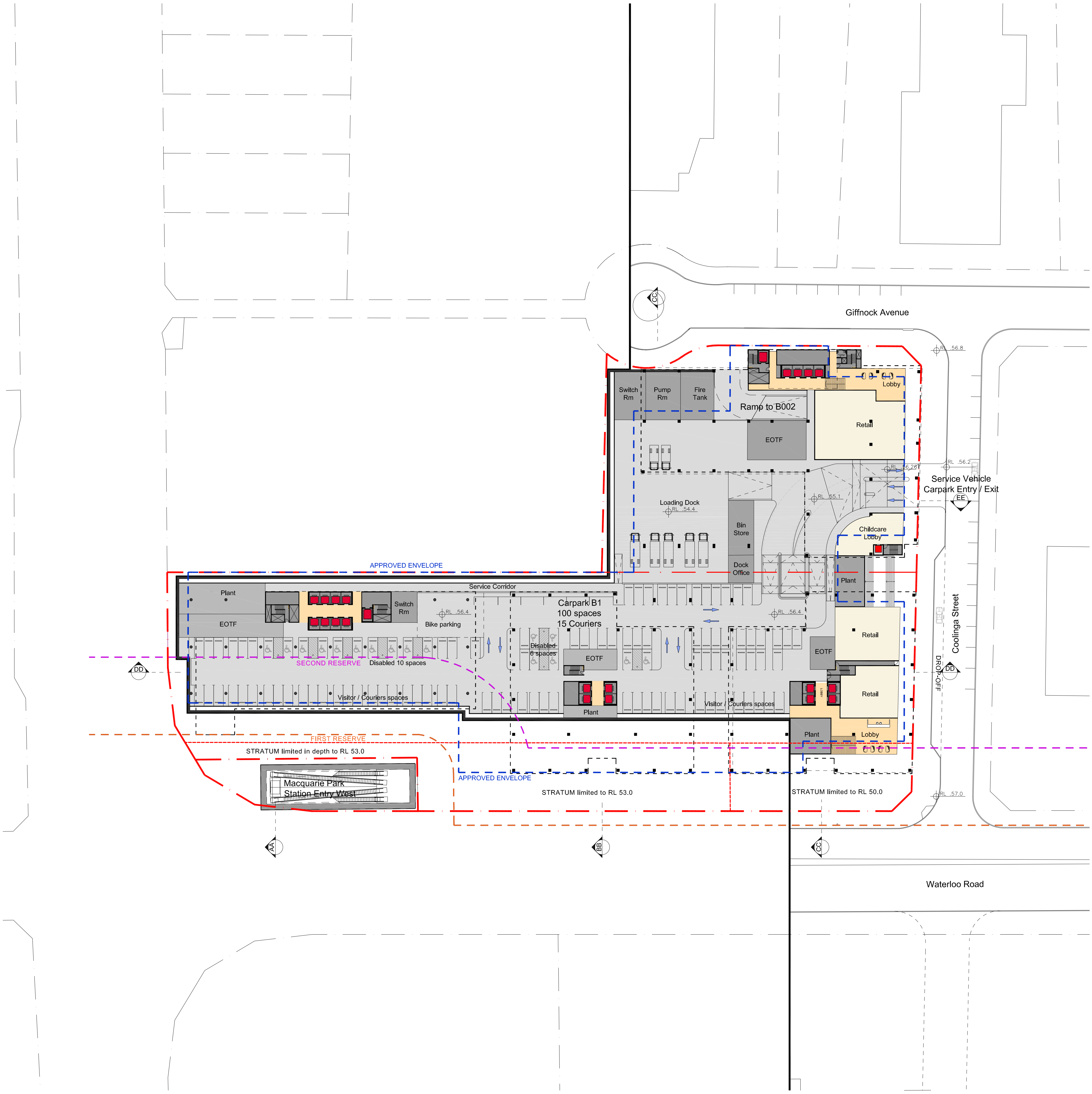
BATESSMART

Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_001 Basement Level 001 Plan

Scale 1:1000

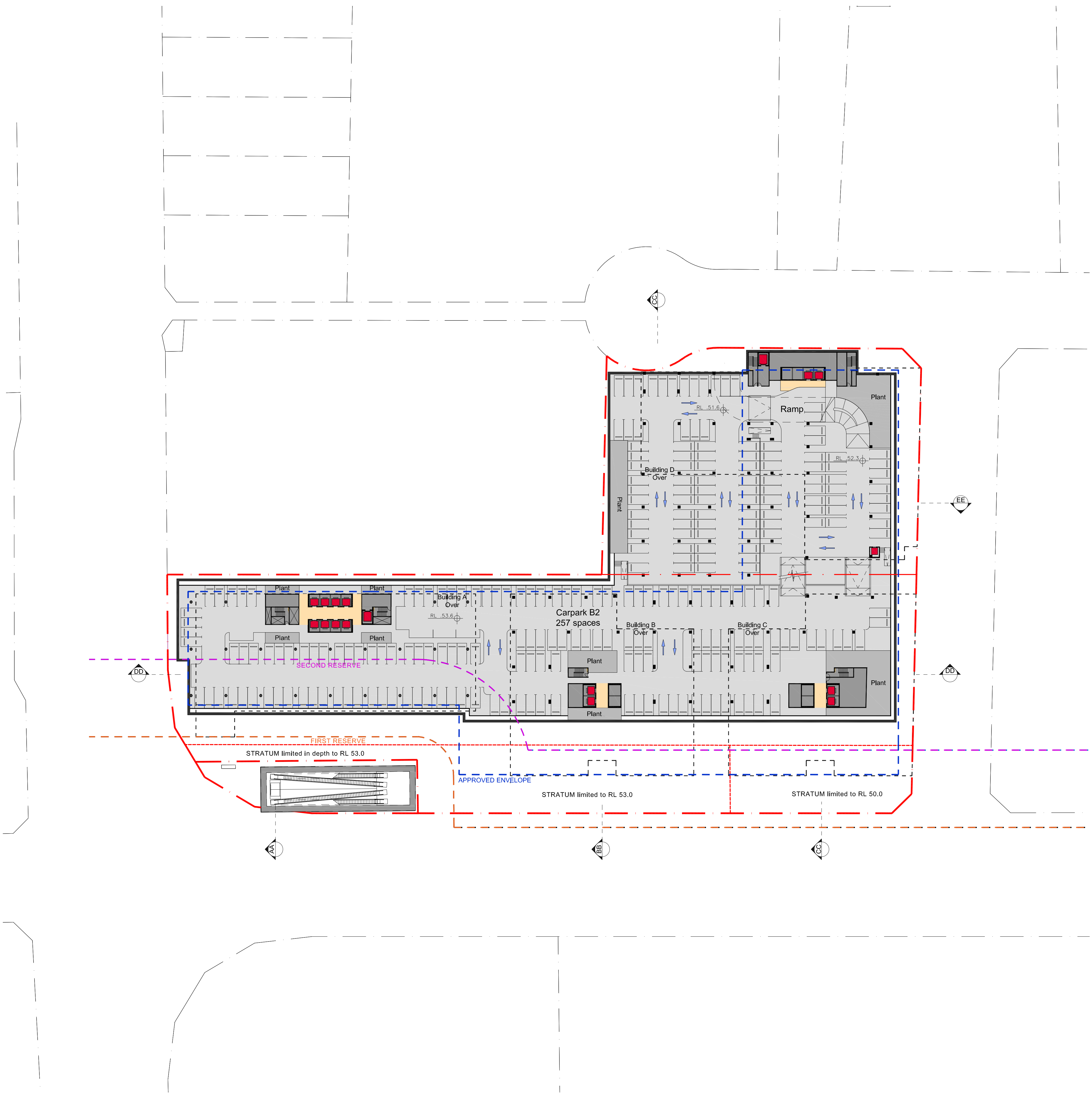


Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_002 Basement Level 002 Plan

Scale 1:1000



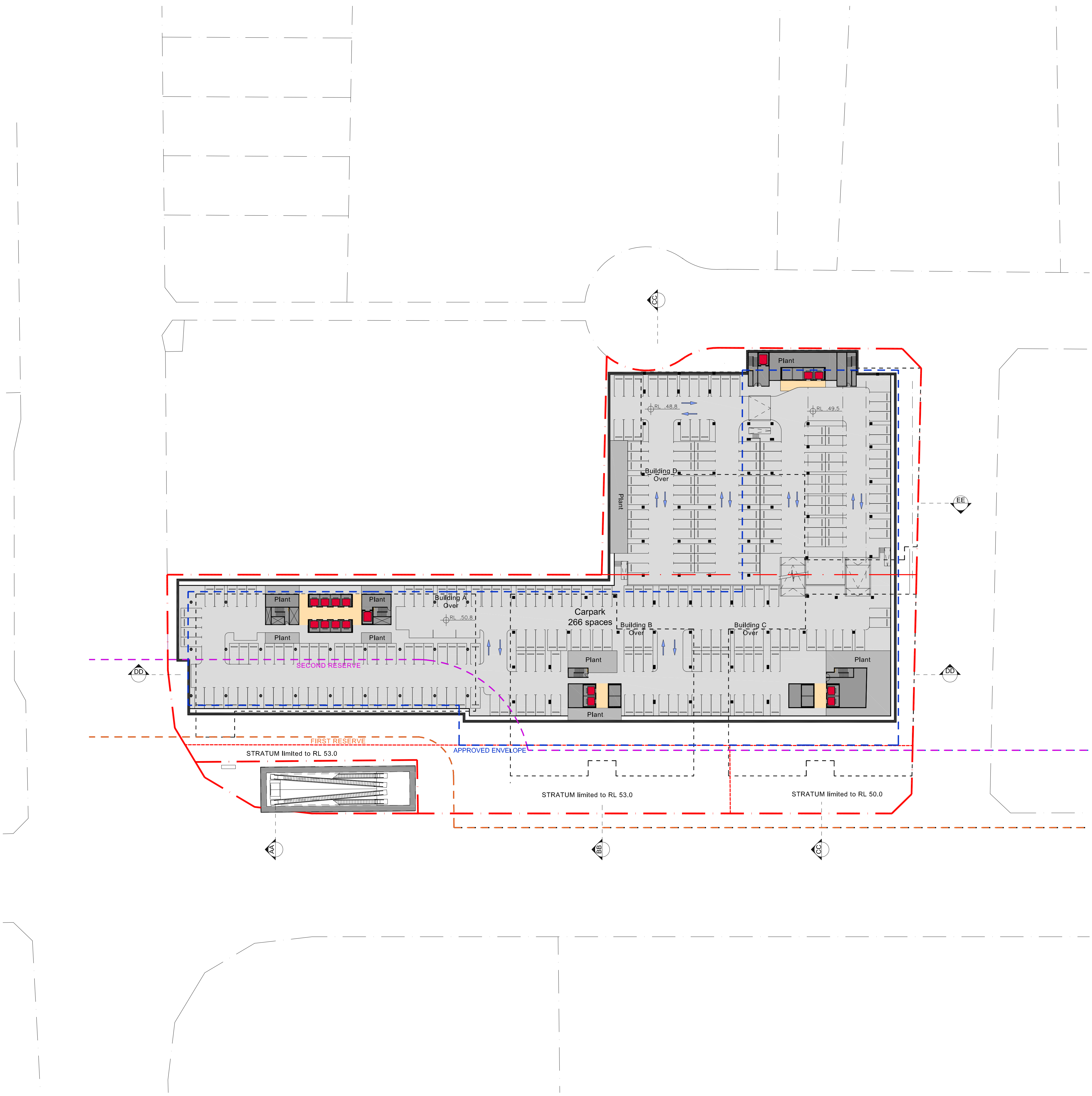
BATESSMART

Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_003 Basement Level 003 Plan

Scale 1:1000



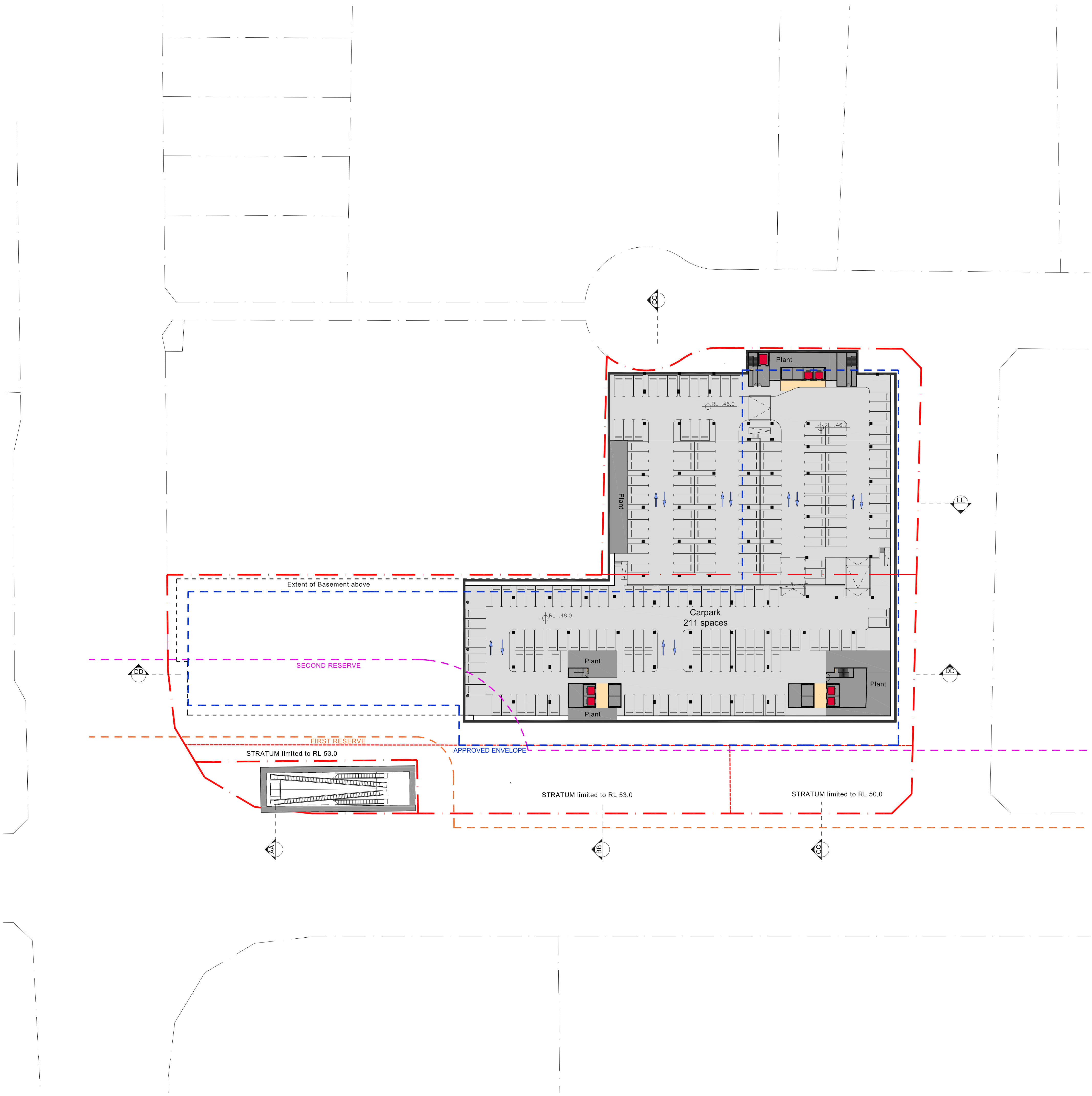
BATESSMART

Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_004 Basement Level 004 Plan

Scale 1:1000



BATESSMART

Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

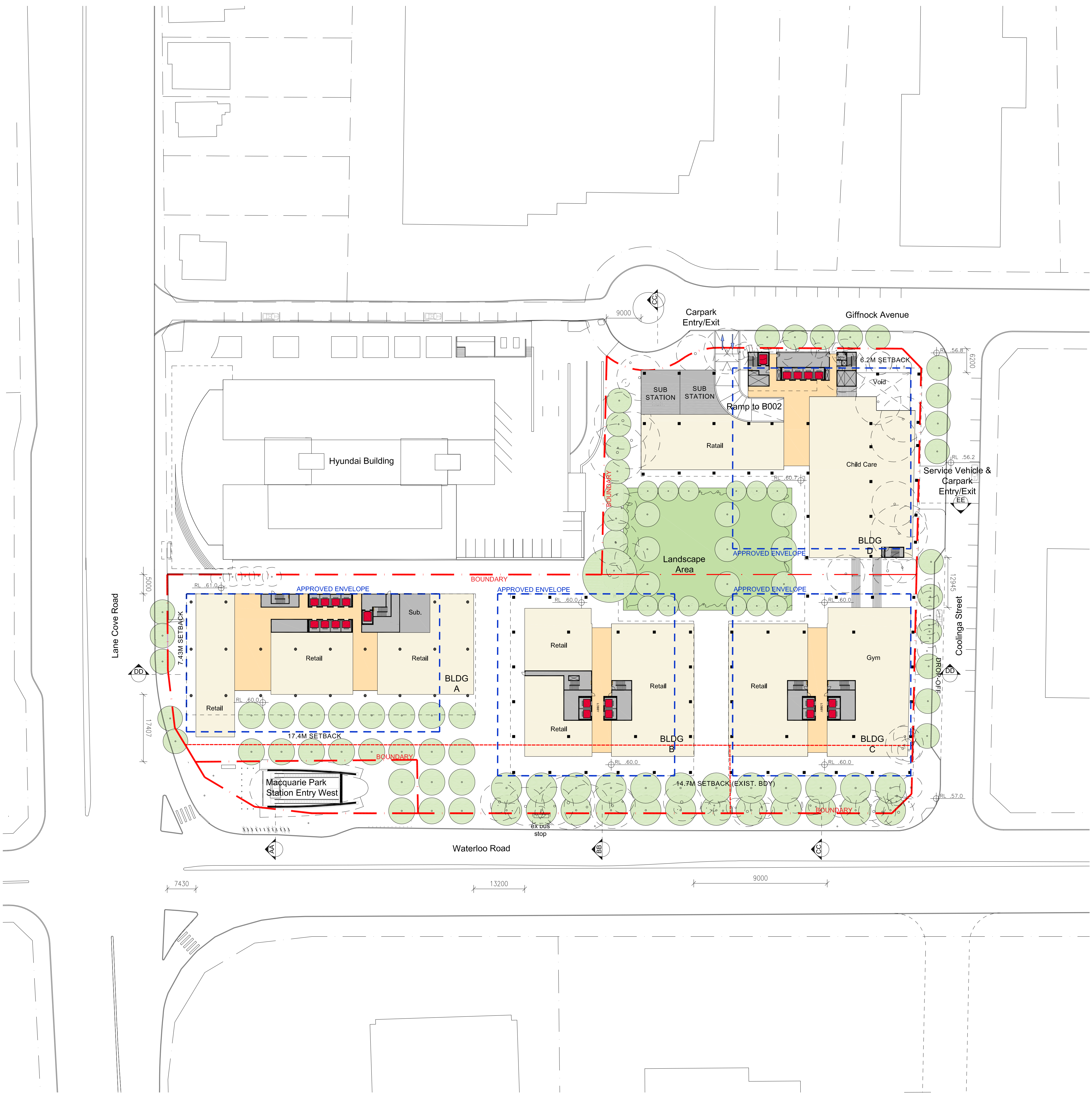
PA02[REV A]_00 Ground Level Plan

Scale 1:1000

Legend

Trees to be removed

New Trees

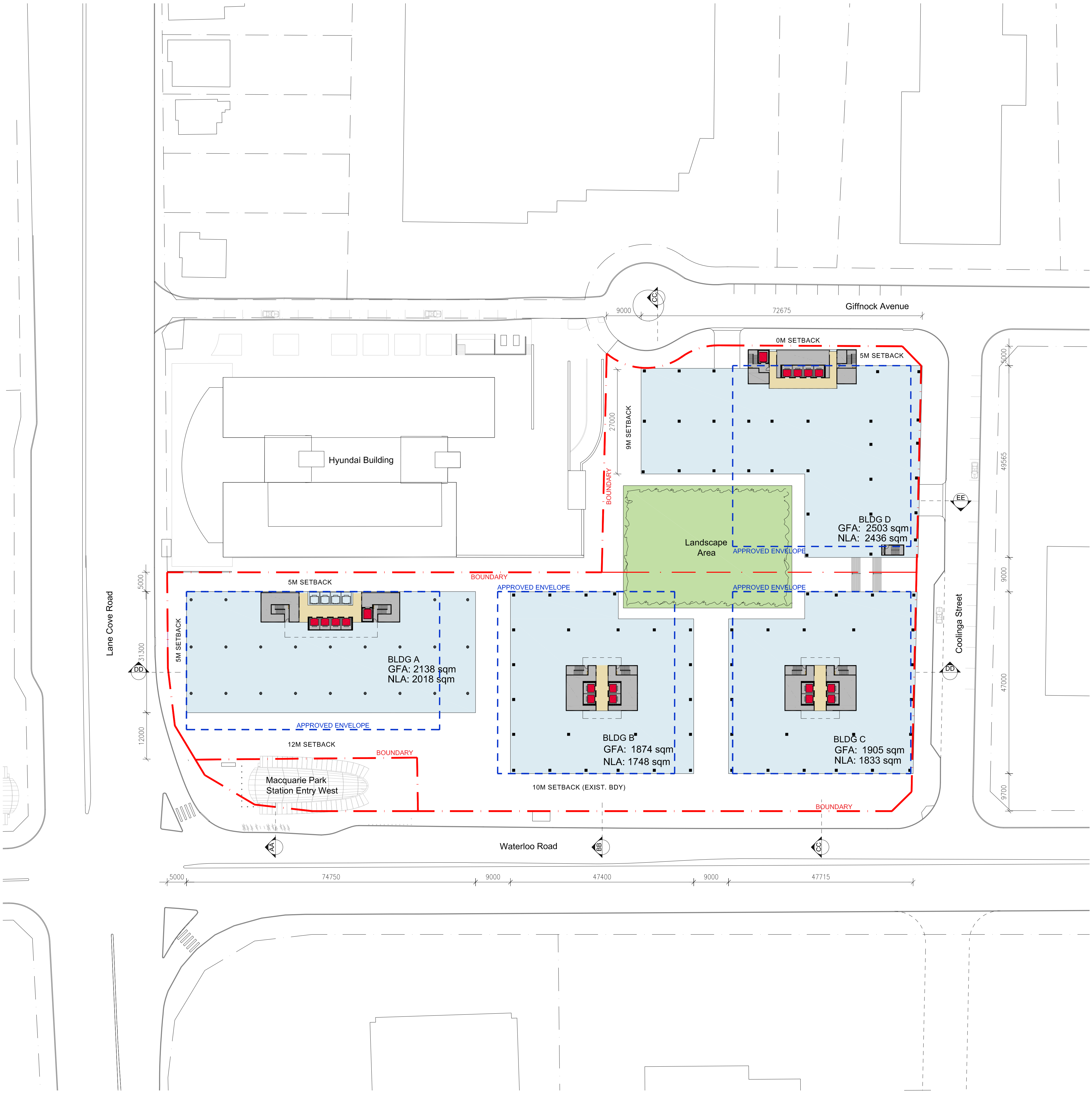


Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_01 Typical Level Plan

Scale 1:1000



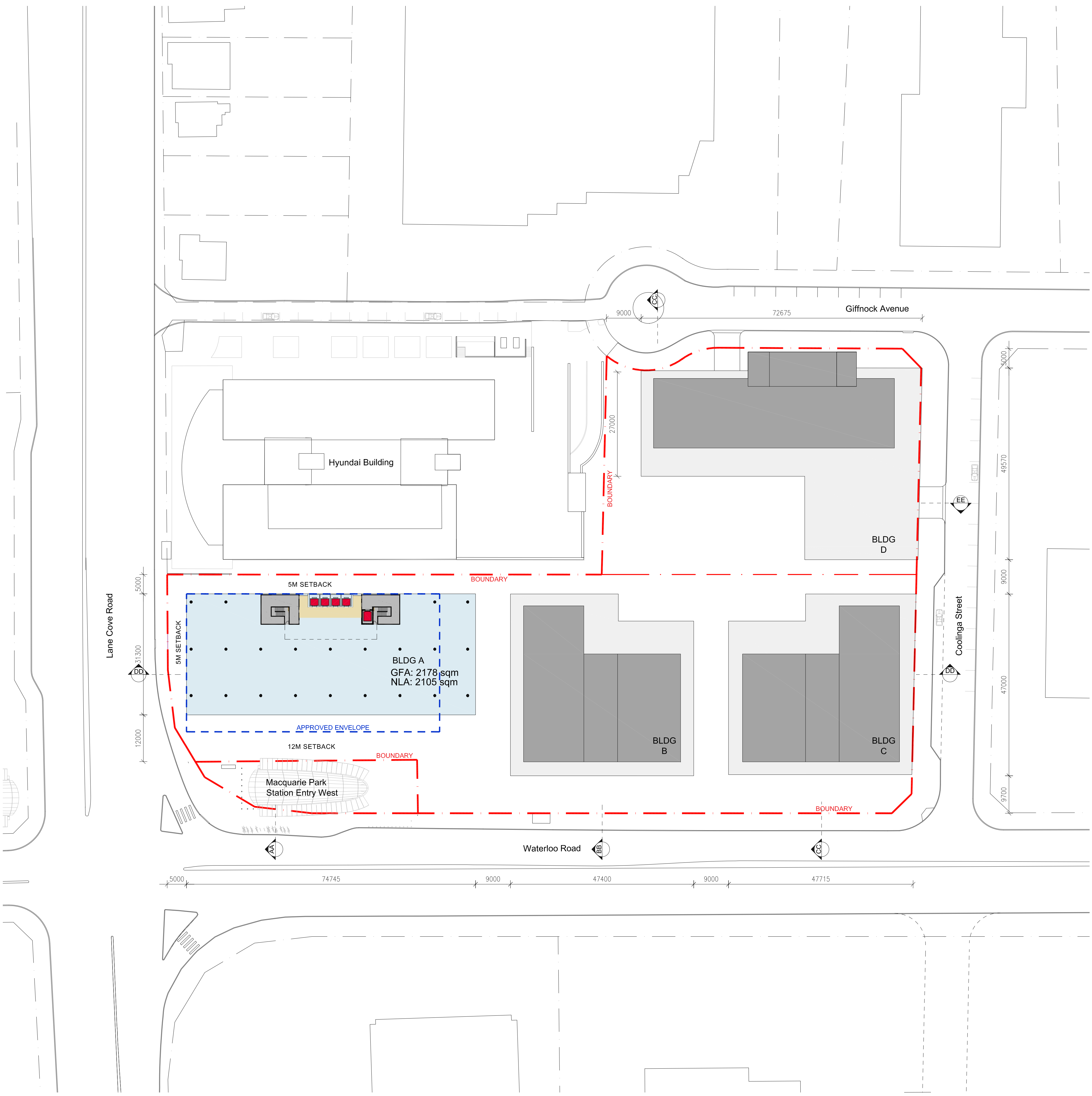
BATESSMART

Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_10 Upper Level Plan

Scale 1:1000



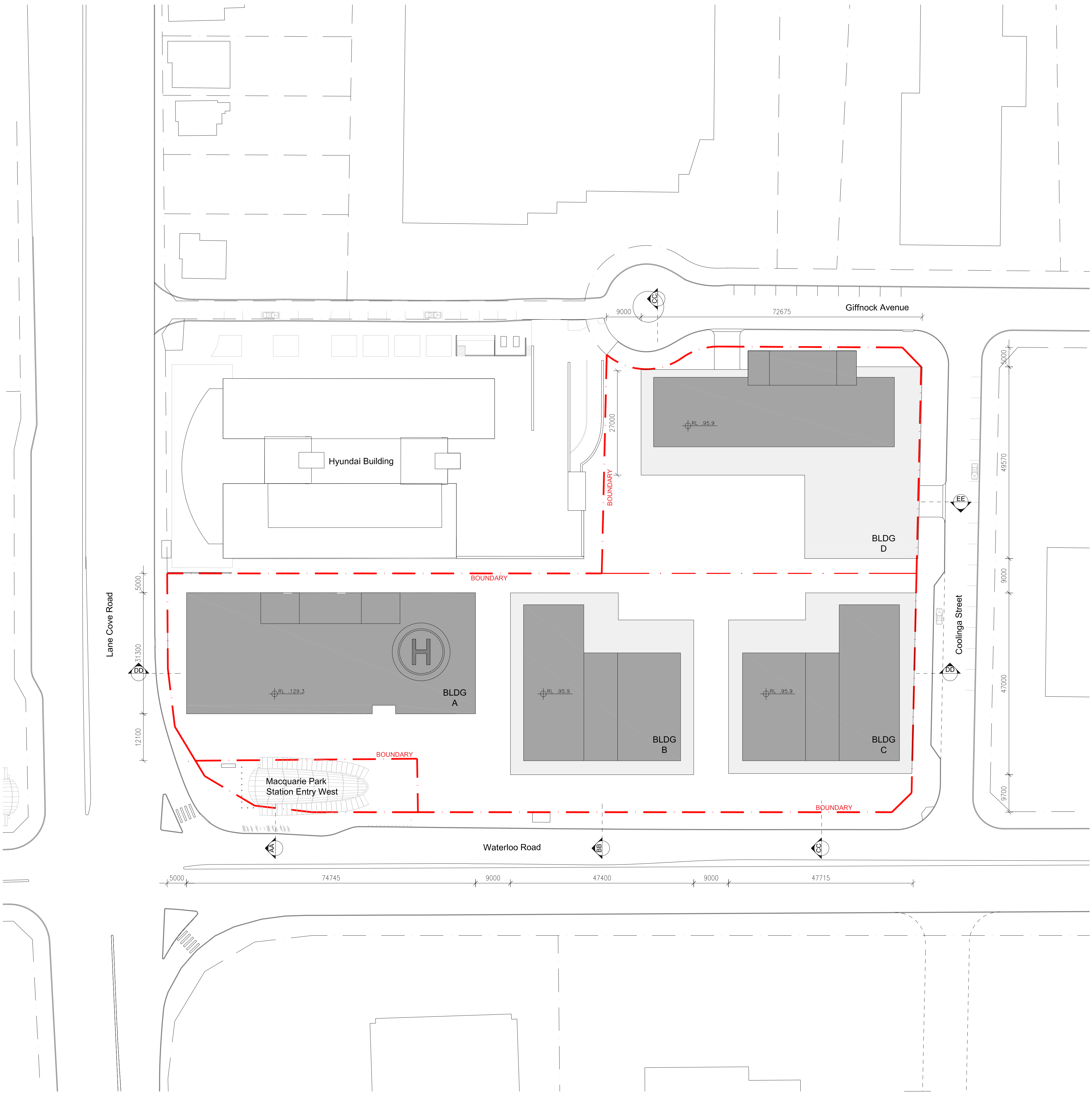
BATESSMART™

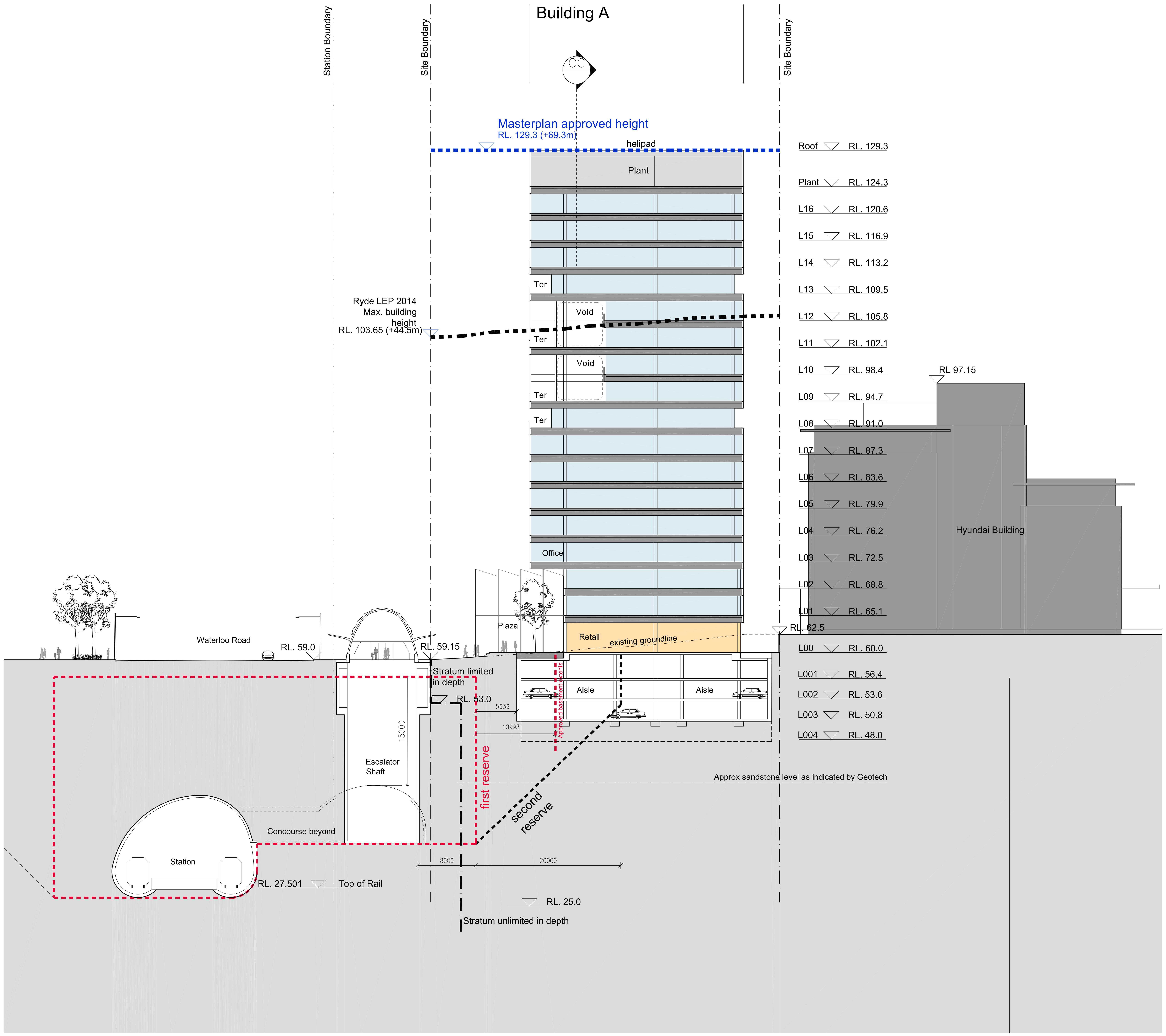
Macquarie Park Commerce Centre
Macquarie Park Station
Waterloo Road

Architectural Design Statement_Section 75W
S10758 February 2018

PA02[REV A]_20 Roof Plan

Scale 1:1000





Revision	Date	Description	Initial	Checked
B	23/11/17	For Information	AT	BD
A	19/10/17	For Information	AT	BD

Client

WINTEN PROPERTY GROUP

FRASERS PROPERTY

Macquarie Park

Section AA

Scale
1:300@A1 / 1:600@A3

Drawn
AT

Checked
BD

Project No.
S10758

Status
SK

Plot Date
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... 7\A08.01.dwg

Plot File

Drawing No.
A08.01

[Revision]
A

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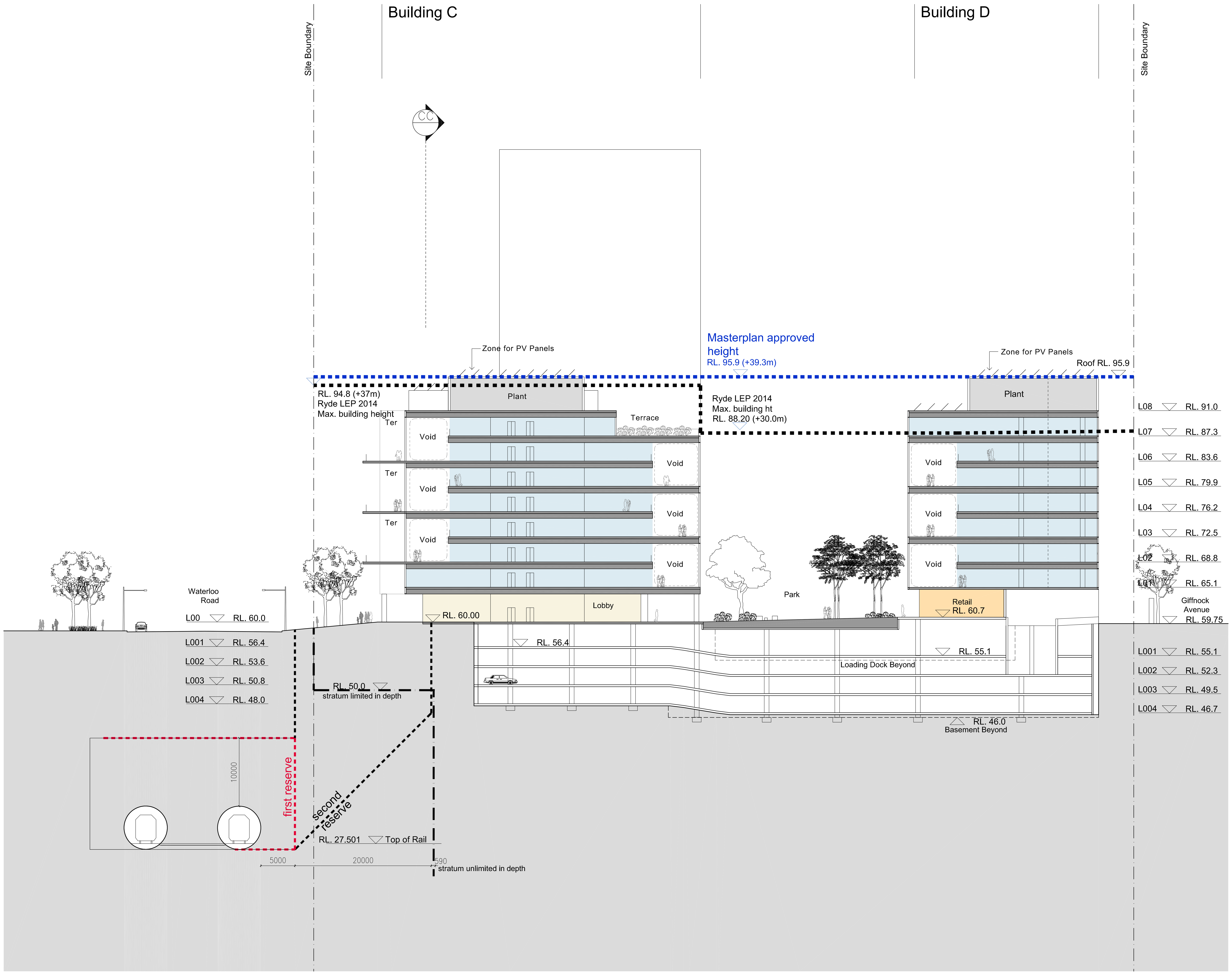
Check all dimensions and site conditions prior to commencement of any work, the purchase or ordering of any materials, fittings, plant, services or equipment and the preparation of shop drawings and/or the fabrication of any components.

All drawings to be read in conjunction with all architectural documents and all other consultants documents.

Do not scale drawings - refer to figured dimensions only. Any discrepancies shall immediately be referred to the architect for clarification.

All drawings may not be reproduced or distributed without prior permission from the architect.



Revision	Date	Description	Initial	Checked
B	23/11/17	For Information	AT	BD
A	19/10/17	For Information	AT	BD

Client

WINTEN PROPERTY GROUP

FRASERS PROPERTY

Macquarie Park

Section BB

Scale
1:300@A1 / 1:600@A3

Drawn
AT

Checked
BD

Project No.
S10758

Status
SK

Plot Date
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Plot File

Drawing No.
A08.02

[Revision]
A

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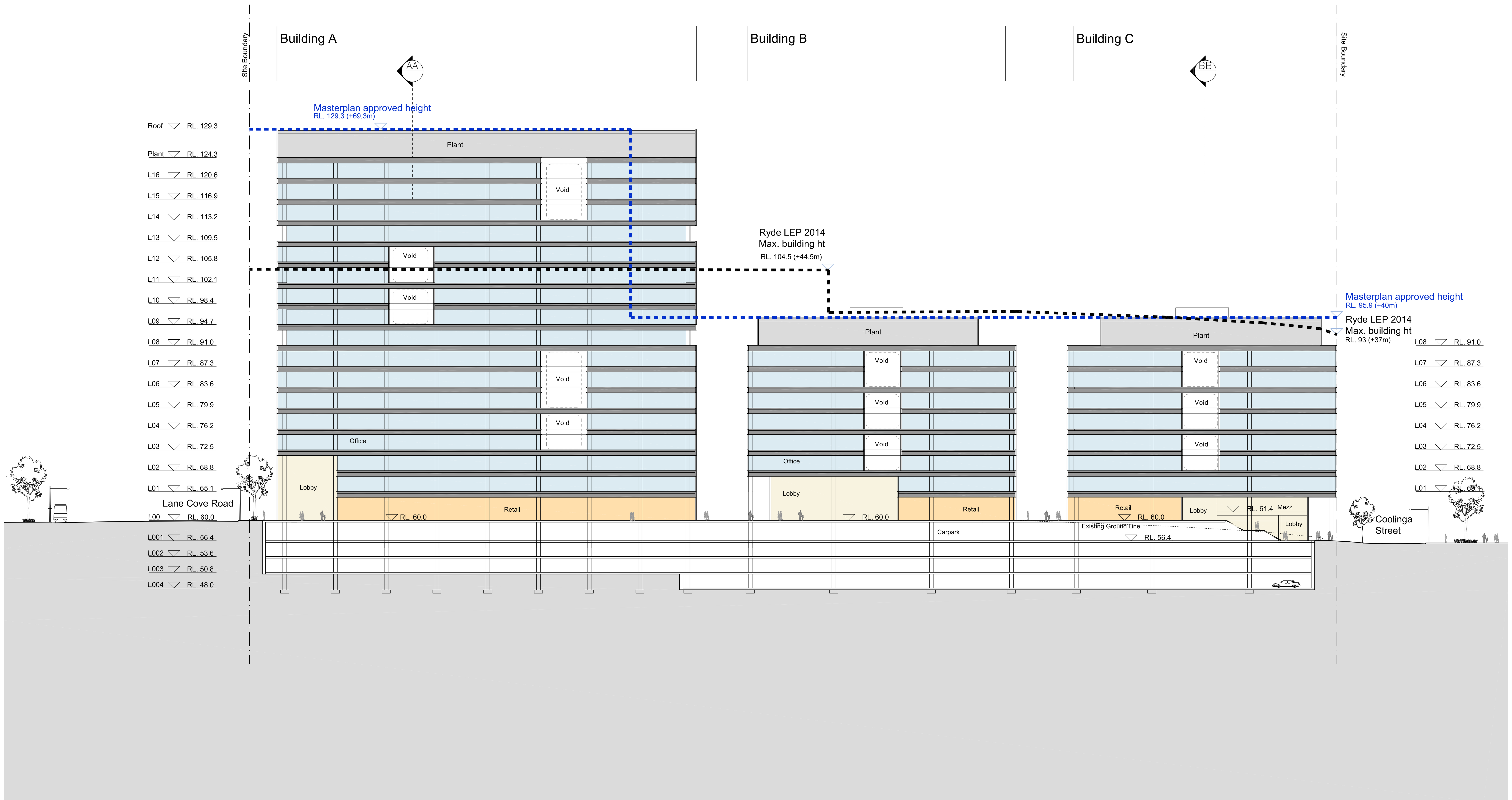

B	23/11/17	For Information	AT	BD
A	19/10/17	For Information	AT	BD
Revision	Date	Description	Initial	Checked

Client

WINTEN PROPERTY GROUP

FRASERS PROPERTY

Macquarie Park

Section CC

Scale
1:400@A1 / 1:800@A3

Drawn
AT

Checked
BD

Project No.
S10758

Status
SK

Plot Date
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... 7\A08.03.dwg

Plot File

Drawing No.
A08.03

[Revision]
A

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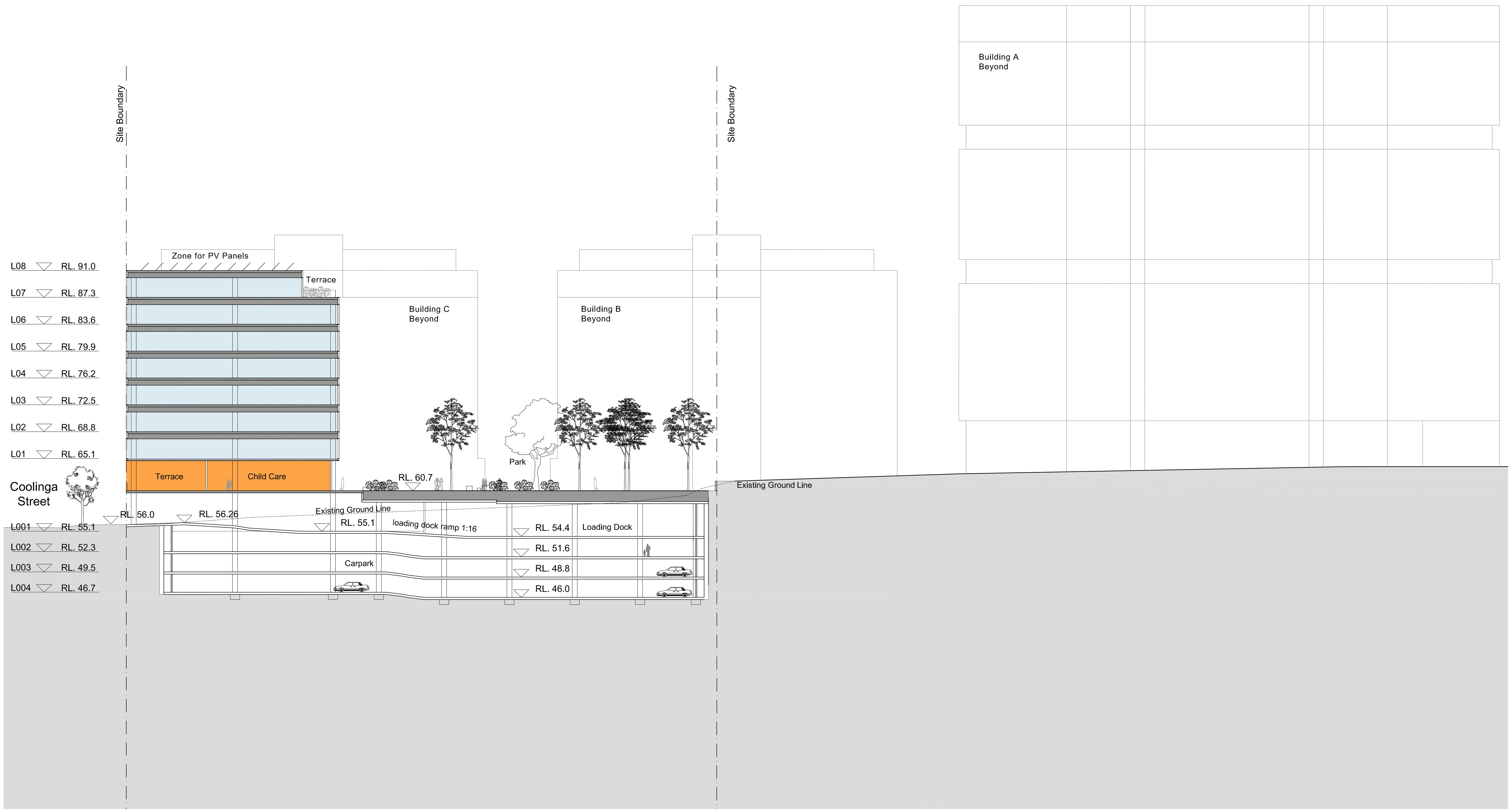

Revision	Date	Description	Initial	Checked
B	23/11/17	For Information	AT	BD
A	13/10/17	For Information	AT	BD

Client

WINTEN PROPERTY GROUP

FRASERS PROPERTY

Macquarie Park

Section DD

Scale
1:300@A1 / 1:600@A3

Drawn
AT

Checked
BD

Project No.
S10758

Status
SK

Plot Date
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Plot File

Drawing No.
A08.04

[Revision]
A

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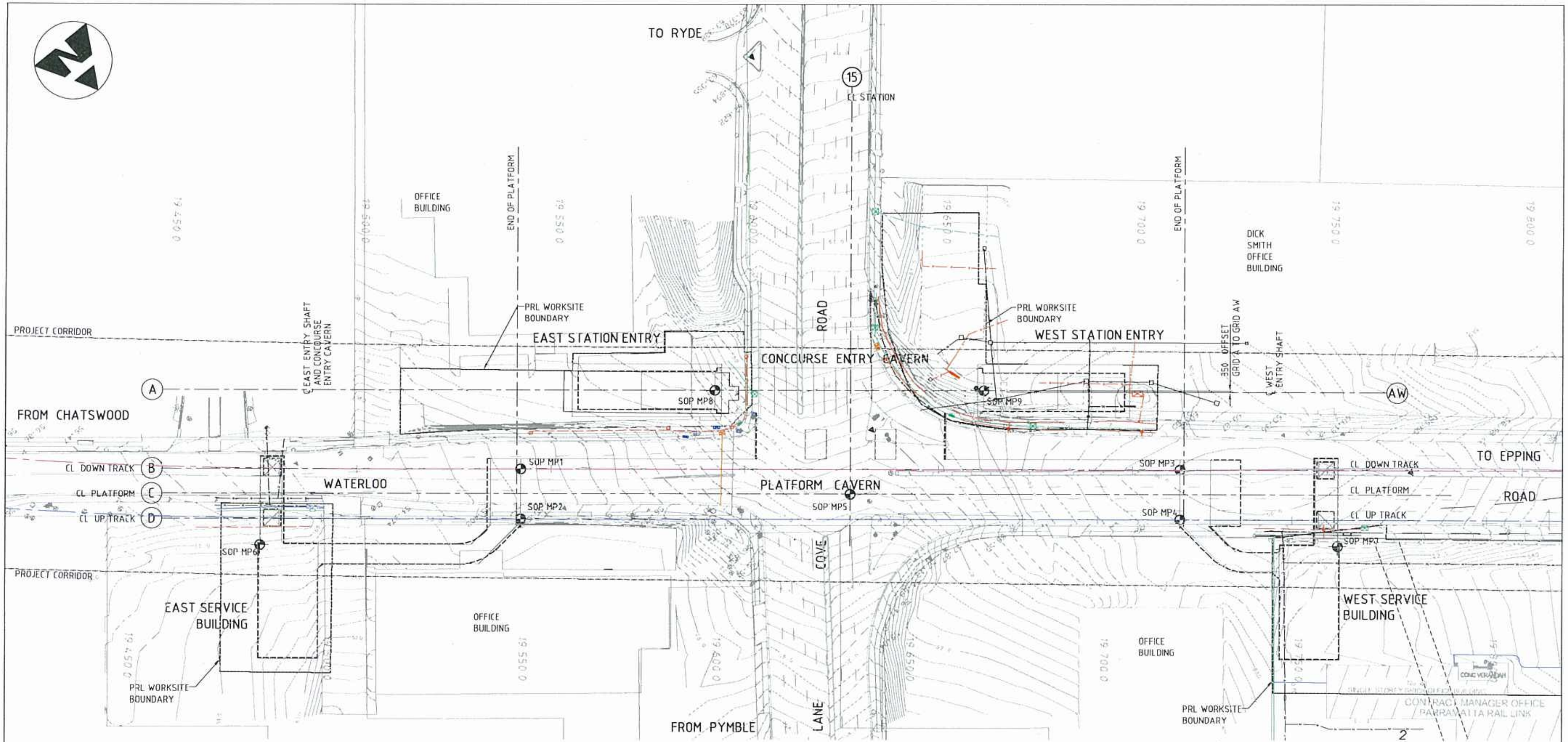
APPENDIX B

EPPING TO CHATSWOOD RAIL LINK – AS BUILT DRAWINGS



MACQUARIE PARK STATION

NOTE

1 FOR LOCATION OF SERVICES REFER DRG No PRL-CSD164804

ALL COORDINATES TO MGA

AS BUILT

A1 1:500
A3 1:1000

REV	BY	DATE	DESCRIPTION	APPD
4	GS	040805	AS BUILT	GE
3	SPG	080404	WEST ENTRY SHAFT MOVED, GRID ADDED	DW
2	EMJ	161003	NOTE ADDED	DW
1	EMJ	180603	ISSUED FOR CONSTRUCTION	DW

AB

GHD
PARSONS BRINCKERHOFF

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	PREPARED	CHECKED	DESIGN CONSULTANT APPROVED
DESIGN	JMcN		D WUNDER
DRAWN	MPR	GCS	DATE 180603
CAD FILE			C12A

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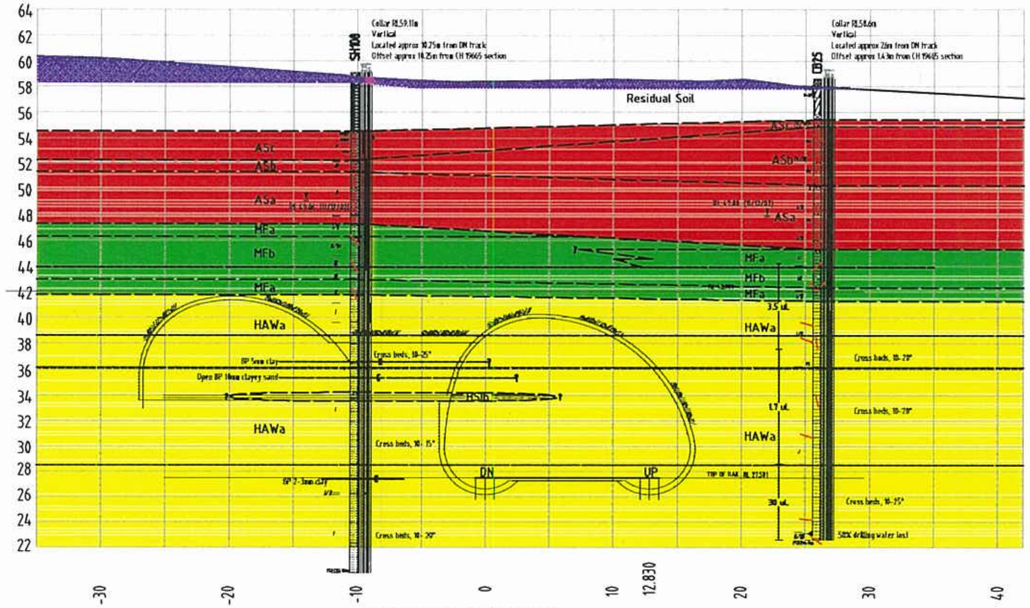
THIESS HOCHTIEF

Parramatta Rail Link
A NSW GOVERNMENT INITIATIVE

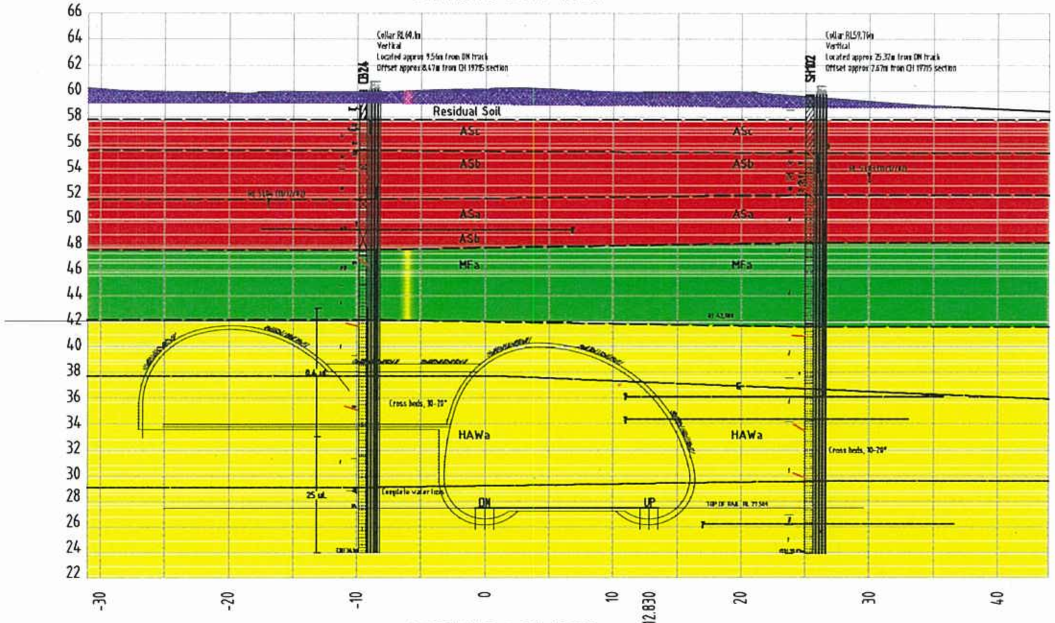
PARRAMATTA RAIL LINK

DRAWING TITLE
MACQUARIE PARK STATION
SITE LAYOUT
PLAN

SCALE	DRG NO.	REV
A1 1:500	PRL-CSD160503	4



SECTION D-D CH: 19665



SECTION E-E CH: 19715

NOTES

1. REFER TO DRAWING PRL-CSD104401 FOR SYMBOL NOTES AND EXPLANATIONS.
2. FOR SECTION LOCATIONS REFER TO DRAWING PRL-CSD164400.
3. THESE SECTIONS ARE INTERPRETIVE REPRESENTATIONS AND HAVE BEEN USED AS A BASIS FOR GEOTECHNICAL MODELLING IN CONJUNCTION WITH A FACILITY OVERLAY. PROMINANT STRUCTURAL UNITS AND FEATURES ARE SHOWN CONCEPTUALLY. FOR FURTHER DETAILS REFER TO INDIVIDUAL BOREHOLE LOGS AND GEOTECHNICAL REPORTS ON THIS FACILITY.

COLOUR KEY

- FILL
- ASHFIELD SHALE
- MITTAGONG FORMATION
- HAWKESBURY SANDSTONE

(1:200 for A1 Size Plot)

AS BUILT

REV	BY	DATE	DESCRIPTION	APPD
4	GS	04.08.05	AS BUILT	GE
3	DMS	21/06/05	AMENDED AS CLOUDED	DW
2	DMS	28.04.05	AMENDED AS CLOUDED	DW
1	JH	14/07/04	APPROVED FOR CONSTRUCTION	DW

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	PREPARED	CHECKED	DESIGN CONSULTANT APPROVED
DESIGN	DW	GPK	D.WUNDER
DRAWN	JH	GPK	DATE 14/07/04
CAD FILE			C12 A

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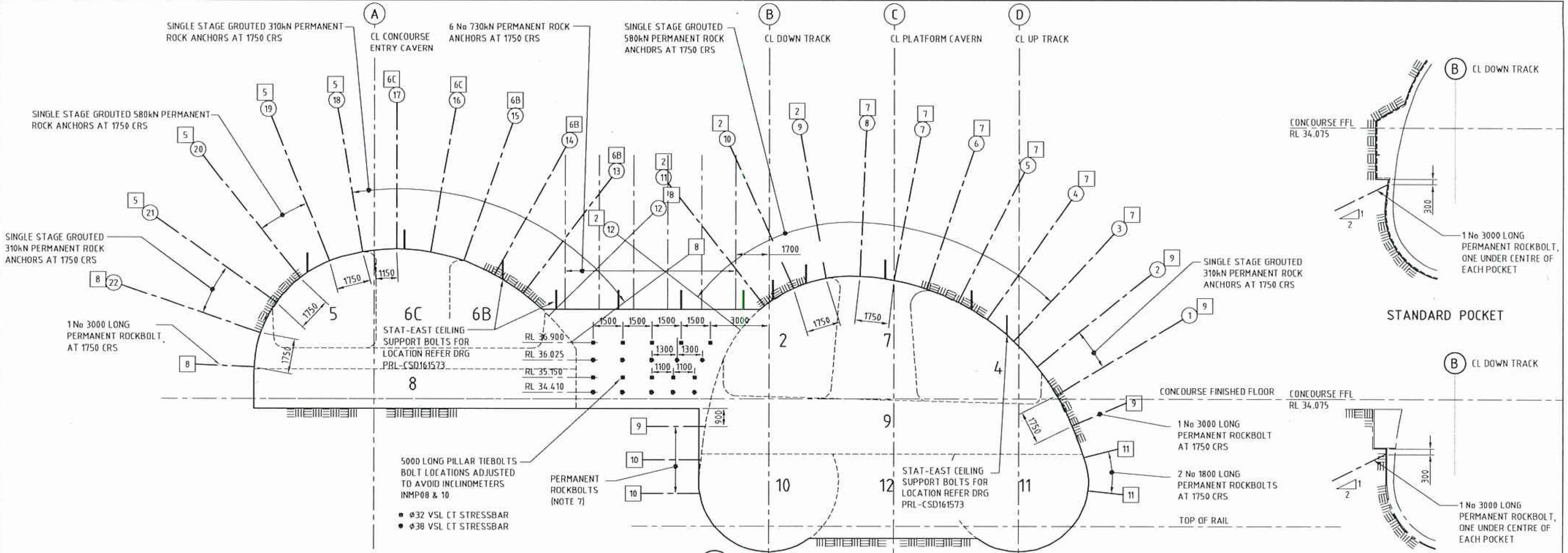
PARRAMATTA RAIL LINK

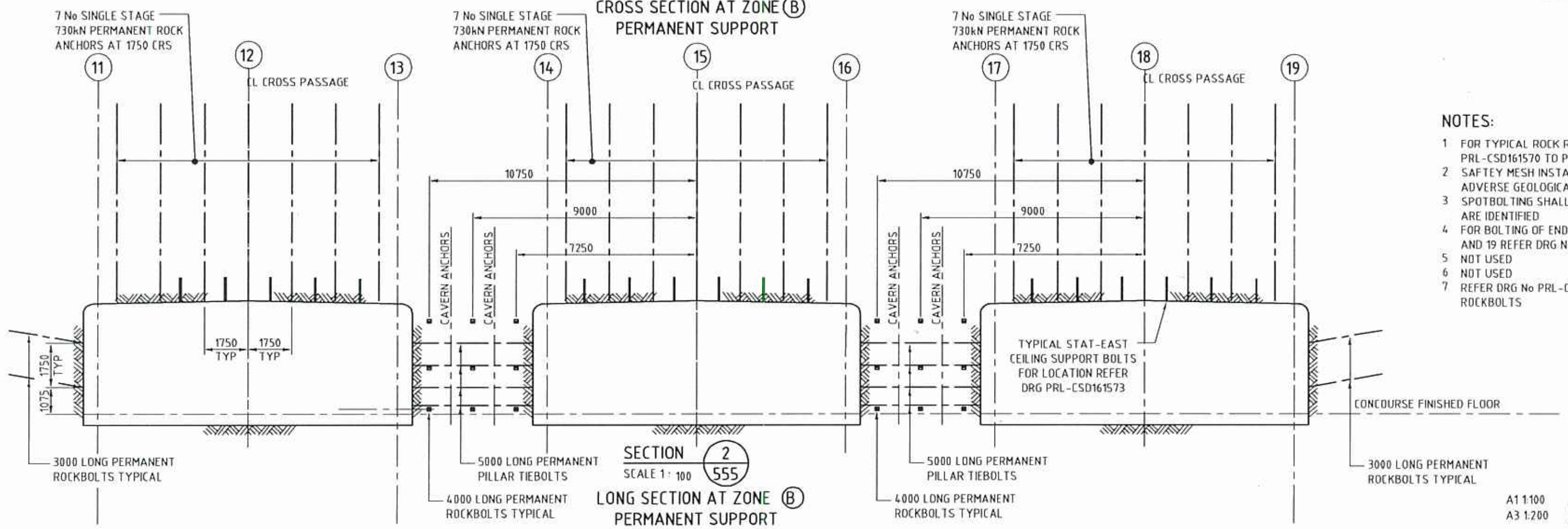
DRAWING TITLE
MACQUARIE PARK STATION
CROSS SECTIONS D-D AND E-E

SCALE	DWG NO.	REV
A1 1:200	PRL-CSD164403	4



SECTION 1A/555
SCALE 1 : 100

CROSS SECTION AT ZONE B
PERMANENT SUPPORT



SECTION 2/555
SCALE 1 : 100

LONG SECTION AT ZONE B
PERMANENT SUPPORT

NOTES:

1. FOR TYPICAL ROCK REINFORCEMENT DETAILS REFER TO DRG No PRL-CSD161570 TO PRL-CSD161572
2. SAFTEY MESH INSTALLED WITH TEMPORARY ROCKBOLTS WHERE ADVERSE GEOLOGICAL CONDITIONS ARE ENCOUNTERED
3. SPOTBOLTING SHALL BE CARRIED OUT WHERE ROCK WEDGES ARE IDENTIFIED
4. FOR BOLTING OF END HEADWALLS ADJACENT TO GRIDLINES 11 AND 19 REFER DRG No PRL-CSD161567
5. NOT USED
6. NOT USED
7. REFER DRG No PRL-CSD161574 FOR DISPOSITION OF PERMANENT ROCKBOLTS

AS BUILT

A1 1:100
A3 1:200
0 2 4 6m

REV	BY	DATE	DESCRIPTION	APPD
6	GS	040805	AS BUILT	CE
5	EMJ	071103	MINOR AMENDMENTS, PERMANENT ROCKBOLTS REVISED (RFI 000365)	DW
4	GCS	270803	STAT-EAST BOLTING REVISED	DW
3	SPG	280703	STAT-EAST & BOLTING REVISIONS	DW
2	PRW	270603	PILLAR DIMS. CLARIFIED	DW
1	EMJ	180603	ISSUED FOR CONSTRUCTION	DW

AB

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DESIGN	TS	JM/N	D WUNDER
DRAWN	LKO	GCS	DATE 180603
CAD FILE			

C12 A

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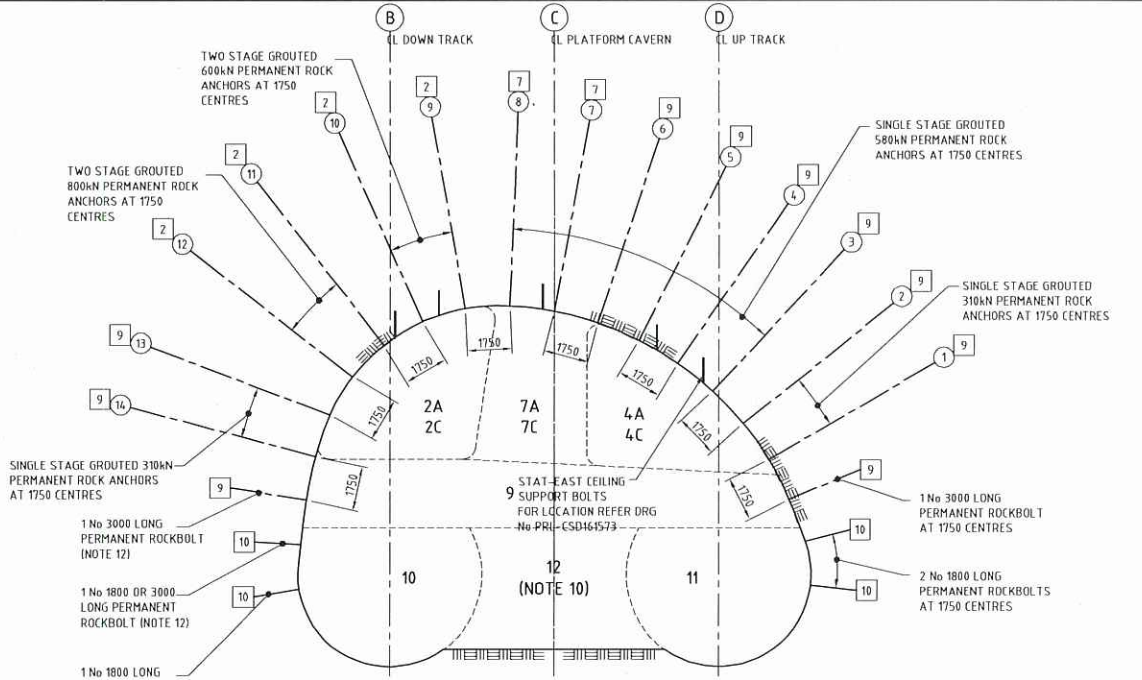
Parramatta Rail Link
A NSW GOVERNMENT INITIATIVE

PARRAMATTA RAIL LINK
DRAWING TITLE
MACQUARIE PARK STATION
TYPICAL CAVERN AND CONCOURSE CROSS PASSAGE
PERMANENT ROCK REINFORCEMENT

SCALE A1 1:100 | DRG No PRL-CSD161556 | REV 6

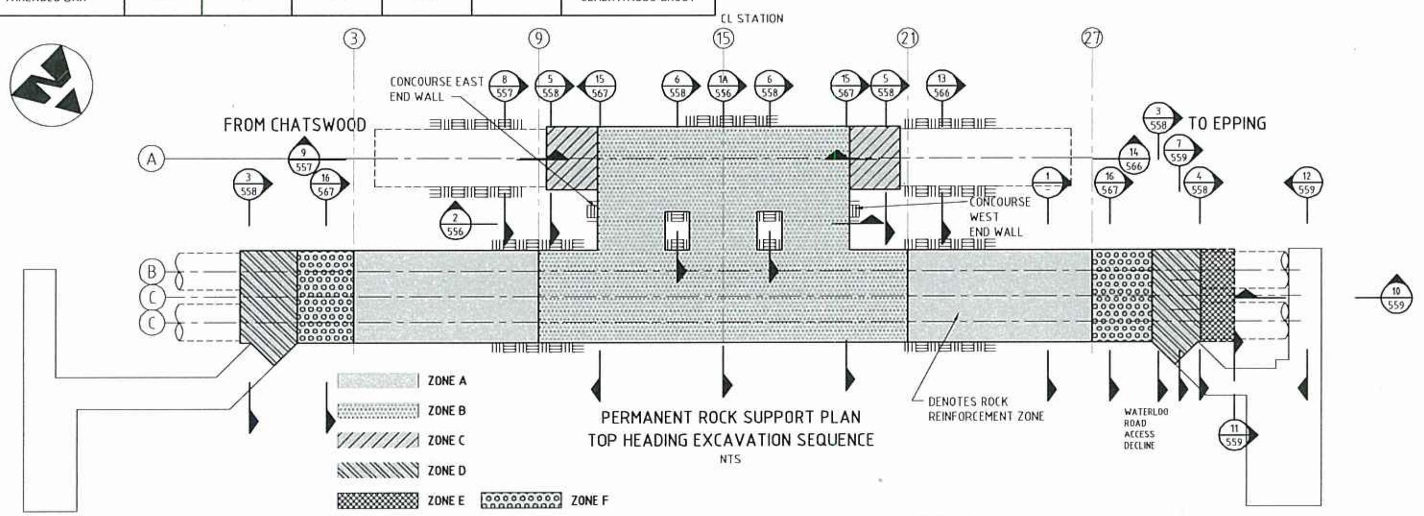
ROCK SUPPORT SCHEDULE

ROCK REINFORCEMENT	TENDON/BAR DESCRIPTION	EMBEDDED LENGTH	TENDON/BAR DIAMETER	HOLE DIAMETER	MIN ULTIMATE TENSILE CAPACITY (kN)	INITIAL PRE-TENSION LOAD (kN)	GROUTING REQUIREMENTS
SINGLE STAGE GROUTED ANCHOR	SHEATHED BLACK STEEL MULTIPLE STRAND TENDON WITH POLYPROPYLENE SLEEVE GREASED TO FREE LENGTH	7000	TENDON		310	100	MECHANICAL ANCHOR CEMENT GROUT INJECTION TO EMBEDDED LENGTH
SINGLE STAGE GROUTED ANCHOR	SHEATHED BLACK STEEL MULTIPLE STRAND TENDON WITH POLYPROPYLENE SLEEVE GREASED TO FREE LENGTH	7000	TENDON		580	175	MECHANICAL ANCHOR CEMENT GROUT INJECTION TO EMBEDDED LENGTH
SINGLE STAGE GROUTED ANCHOR	SHEATHED BLACK STEEL MULTIPLE STRAND TENDON WITH POLYPROPYLENE SLEEVE GREASED TO FREE LENGTH	7500	TENDON		730	220	MECHANICAL ANCHOR CEMENT GROUT INJECTION TO EMBEDDED LENGTH
TWO STAGE GROUTED ANCHOR	GREASE FILLED SHEATH TO FREE LENGTH BLACK STEEL MULTIPLE STRAND TENDON	7500	TENDON	TO ACCOMMODATE 40 ROCK SHEAR	600	175	MECHANICAL ANCHOR CEMENT GROUT INJECTION TO ANCHORAGE LENGTH, FREE LENGTH GROUTED LATER
TWO STAGE GROUTED ANCHOR	GREASE FILLED SHEATH TO FREE LENGTH BLACK STEEL MULTIPLE STRAND TENDON	8000	TENDON	TO ACCOMMODATE 40 ROCK SHEAR	800	240	MECHANICAL ANCHOR CEMENT GROUT INJECTION TO ANCHORAGE LENGTH, FREE LENGTH GROUTED LATER
PERMANENT ROCKBOLT	SHEATHED THREADBAR	VARIES	24	45	300	60	RESIN END-ANCHORAGE /MECHANICAL ANCHOR CEMENT GROUT INJECTION TO EMBEDDED LENGTH
TEMPORARY ROCKBOLT	SINGLE BLACK DEFORMED BAR	2700/3000	24	28	310	60	TWO SPEED RESIN ENCAPSULATION
TEMPORARY SPILING BAR (DOWEL)	SINGLE BLACK DEFORMED BAR	2400	24	36	310	-	FULL COLUMN CEMENTITIOUS GROUT INJECTION
PILLAR TIEBOLTS	EPOXY COATED STRESSBAR	5000	32	75	870	250	FULL COLUMN CEMENTITIOUS GROUT INJECTION
STAT-EAST CEILING SUPPORT	STAINLESS STEEL THREADED BAR	1000	20	28	100	-	FULL COLUMN CEMENTITIOUS GROUT
OHW HANGER	STAINLESS STEEL THREADED BAR	1000	20	28	100	-	FULL COLUMN CEMENTITIOUS GROUT



SECTION 1 / -
SCALE 1 : 100

CROSS SECTION AT ZONE (A)
PERMANENT SUPPORT



PERMANENT ROCK SUPPORT PLAN
TOP HEADING EXCAVATION SEQUENCE
NTS

NOTES
1 ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED
NOT USED
2 SPOTBOLTING SHALL BE CARRIED OUT WHERE POTENTIALLY
3 UNSTABLE ROCK WEDGES ARE IDENTIFIED
FOR TYPICAL CAVERN ROCK REINFORCEMENT DETAILS REFER
4 DRG No PRL-CSD161570
ROCKBOLT/ANCHOR DIMENSIONS AND CAPACITIES AS SHOWN ARE
5 MINIMUM
NOT USED
6 SAFETY MESH INSTALLED WITH TEMPORARY ROCKBOLTS WHERE
7 ADVERSE GEOLOGICAL CONDITIONS ARE ENCOUNTERED
NOT USED
8 FOR EACH TRANSFER PASSAGE PILLAR REFER DRG No
9 PRL-CSD161558
SUPPORT OF CENTRE PILLAR, 12, REFER RUNNING TUNNEL
10 DRAWINGS DESIGN PACKAGE C11A
ALL BOLTING ARRAYS AT 1750 CENTRES LONGITUDINALLY, UNO
11 REFER DRG No PRL-CSD161574 FOR SPACING AND DISPOSITION OF
12 PERMANENT ROCKBOLTS IN THIS ROW

AS BUILT

A1 1:100 0 2 4 6m
A3 1:200 2 1

REV	BY	DATE	DESCRIPTION	APPD
6	LMT	11.12.07	AS BUILT	GE
5	AB	160605	NOTE NOT USED	DW
4	CZ	071103	MINOR AMENDMENTS, PERMANENT ROCKBOLTS REVISED (RFI 040365)	DW
3	GCS	270803	STAT-EAST & OHW REVISIONS	DW
2	SPG	280703	STAT-EAST & MINOR BOLTING REVISIONS	DW
1	EMJ	180603	ISSUED FOR CONSTRUCTION	DW

AB

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	PREPARED	CHECKED	DESIGN CONSULTANT APPROVED
DESIGN	TS	JMcN	D WUNDER *
DRAWN	EMJ	GCS	DATE 180603

CAD FILE * ORIGINAL SIGNED COPY ON PROJECT FILE

C12 A

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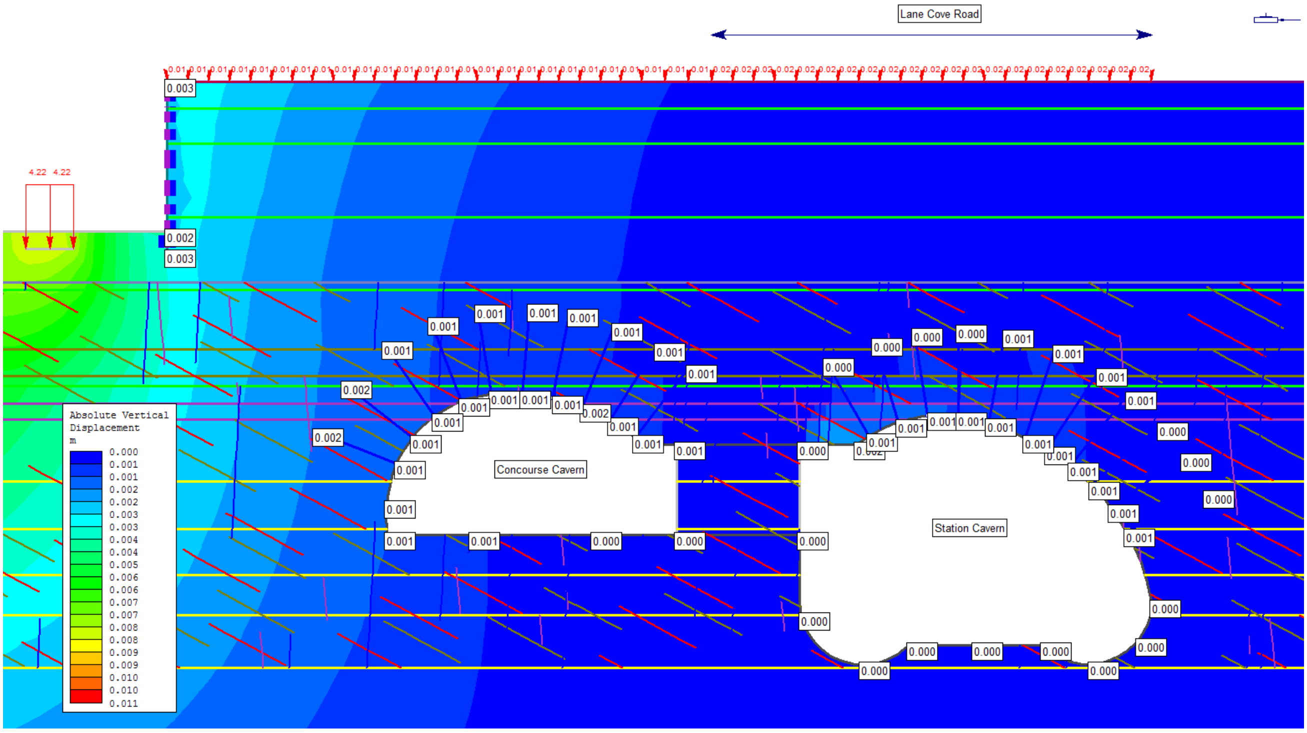
PARRAMATTA RAIL LINK
DRAWING TITLE
MACQUARIE PARK STATION
TYPICAL CAVERN AND CONCOURSE
PERMANENT ROCK REINFORCEMENT

SCALE A1 1:100 | DRG No PRL-CSD161555 | REV 6

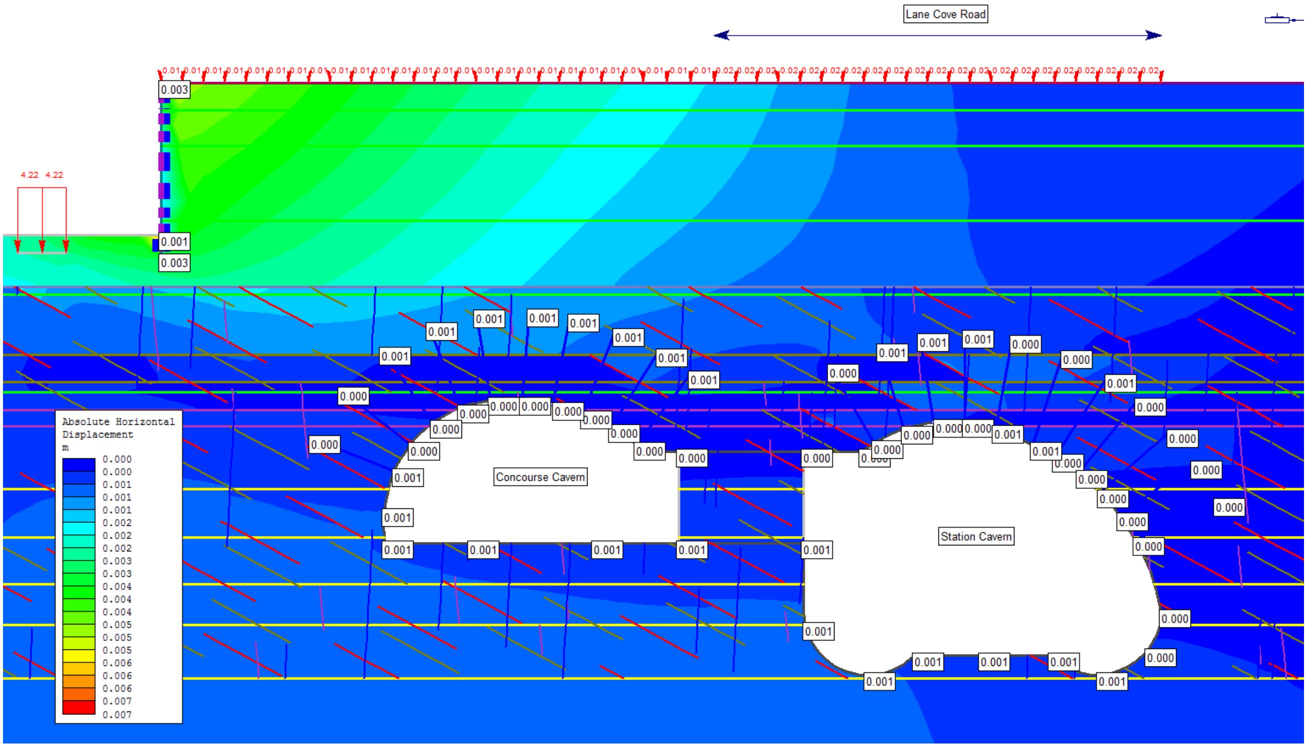
APPENDIX C

NUMERICAL MODELLING OUTPUTS

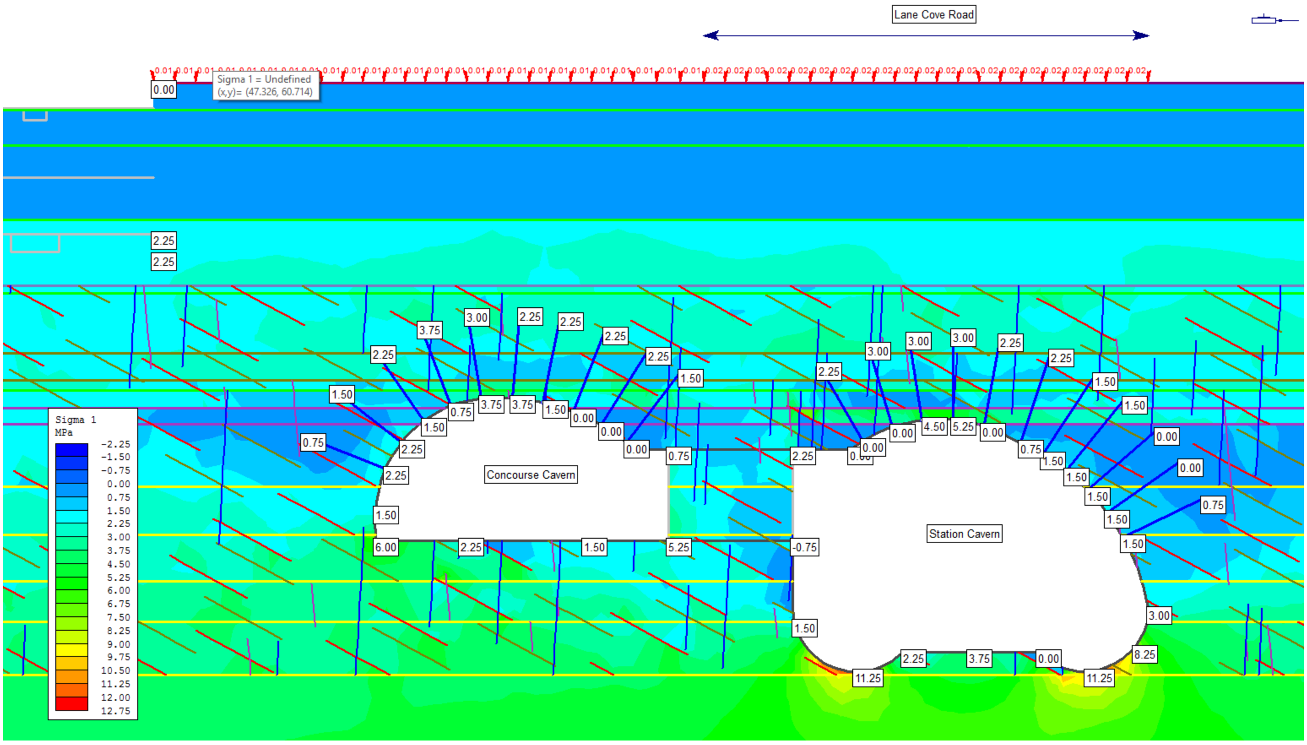
C1 STATION CAVERN AND CONCOURSE CAVERN (SECTION 1)



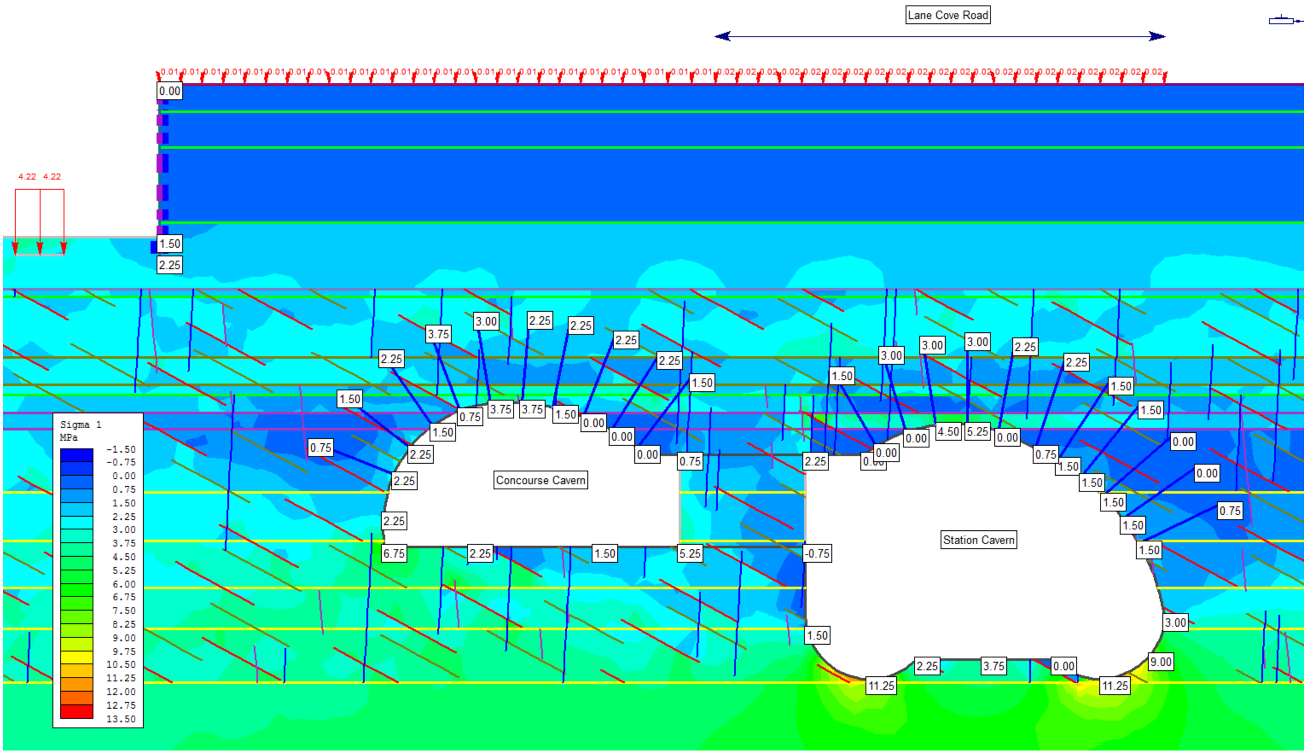
Absolute vertical displacement after excavation completion (Section 1)



Absolute horizontal displacement after excavation completion (Section 1)

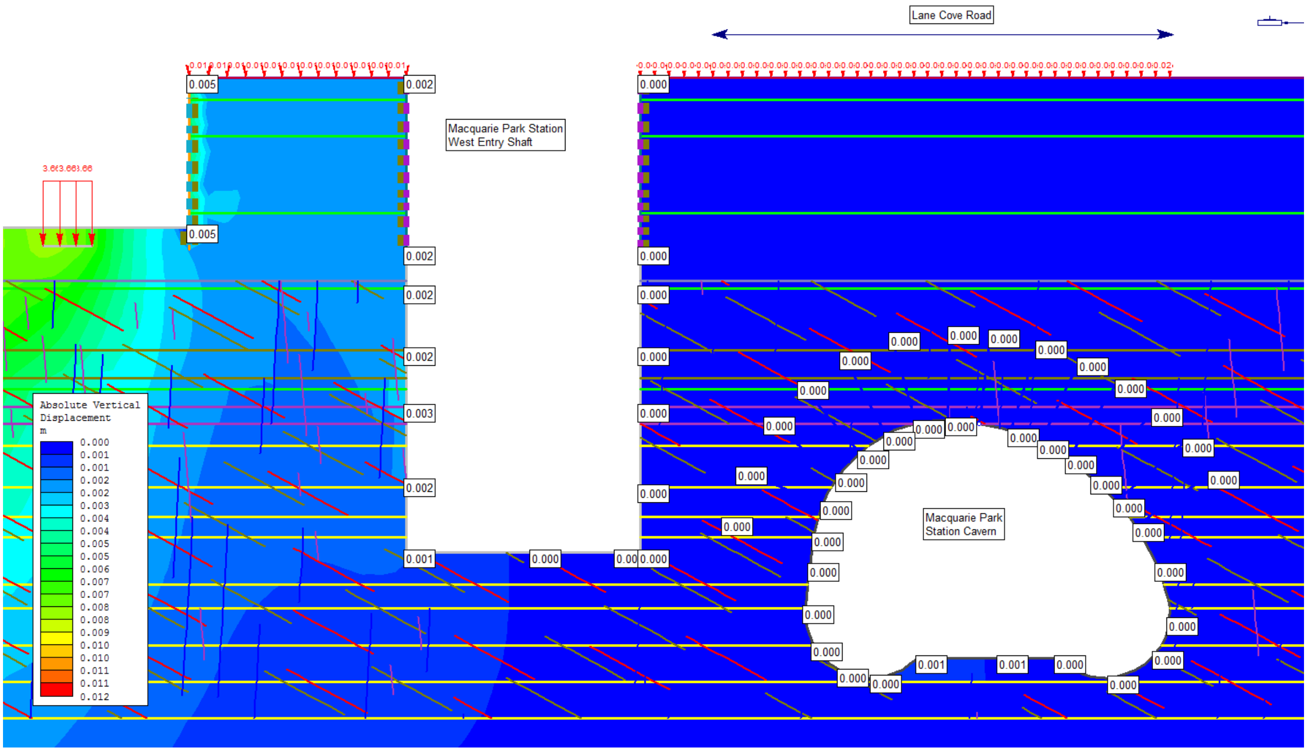


σ₁ contours – prior to development excavation (Section 1)

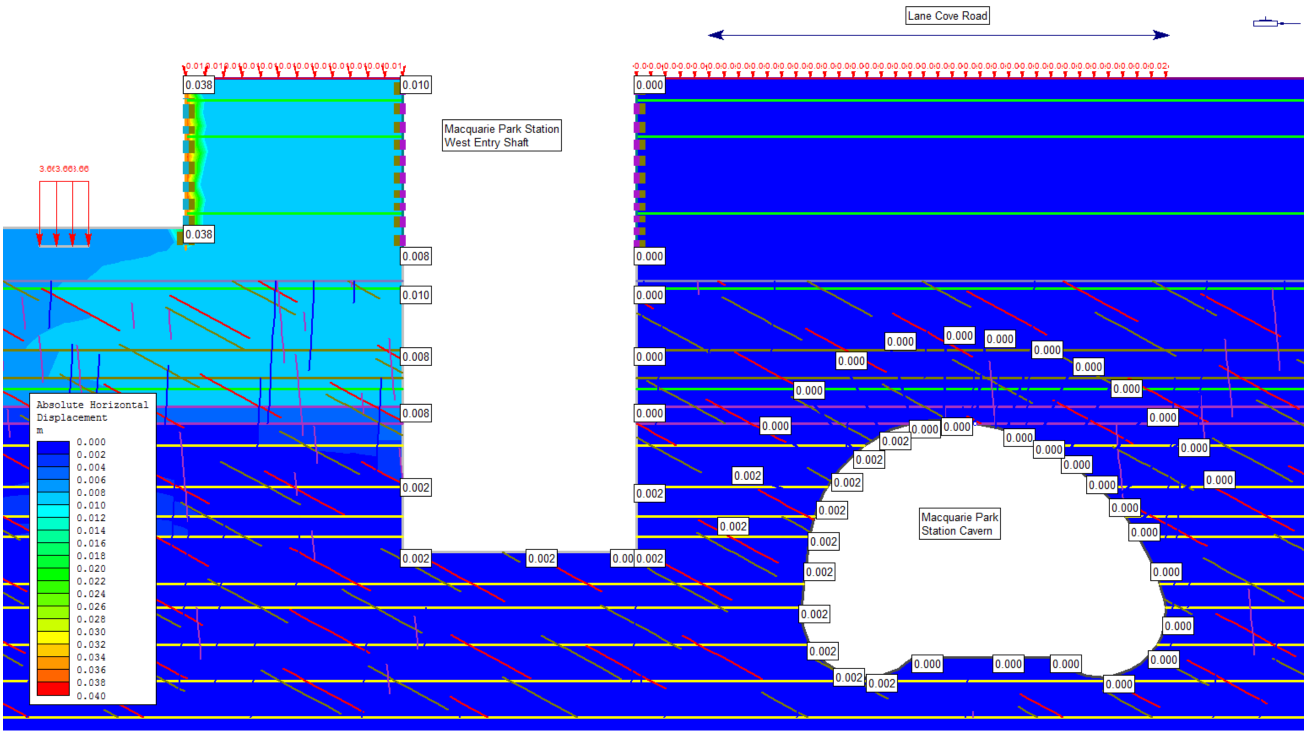


σ₁ contours – after development excavation completion (Section 1)

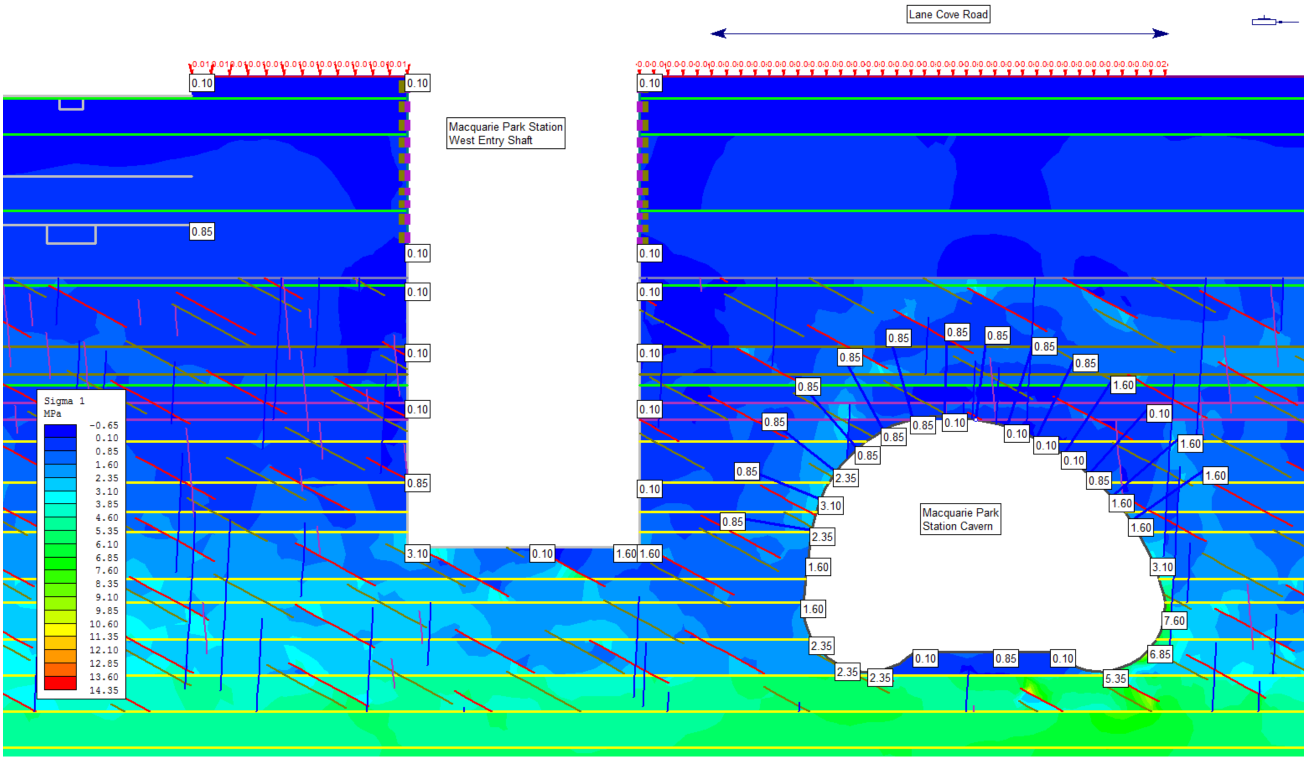
C2 STATION CAVERN AND WEST ENTRY SHAFT (SECTION 2)



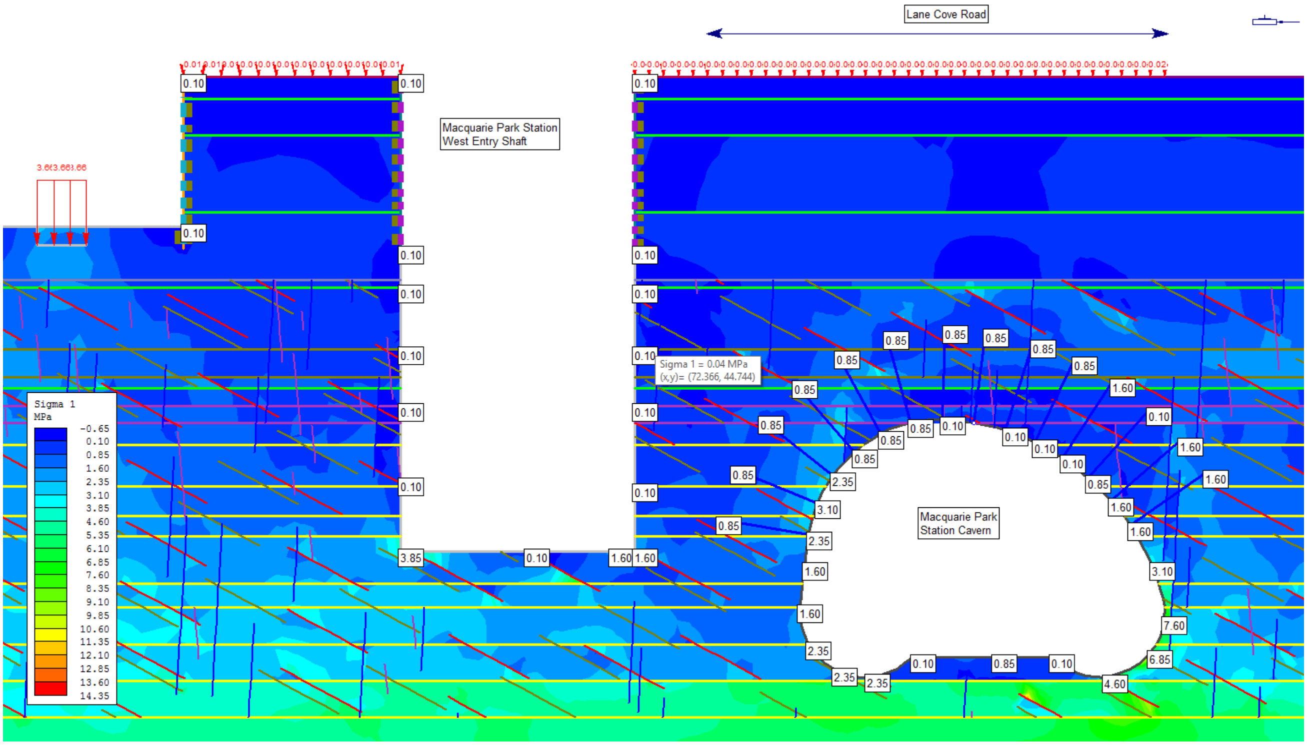
Absolute vertical displacement after excavation completion (Section 2)



Absolute horizontal displacement after excavation completion (Section 2)



σ_1 contours – prior to development excavation (Section 2)



σ_1 contours – after development excavation completion (Section 2)