Appendix G

Engineering Services Report

ANNANDALE CHRISTIAN COLLEGE

(PROPOSED NEW ADMINISTRATION BUILDING AND EXTENSION)

ENGINEERING SERVICES REPORT

ANNANDALE CHRISTIAN COLLEGE

LANGTREE CONSULTING

Project No.: 1194_2

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TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

Langtree Consulting has been engaged by RPS Consulting on behalf of Annandale Christian College to undertake this Engineering Services Report (ESR). This report has been prepared in support of a Development Application for Annandale Christian College situated at 104-156 Yolanda Drive, Annandale. The application pertains to land identified as Lot 2 on RP850718 and Lot 3 on SP256046, as well as the recently acquired property at 9 Jonquil Crescent, Annandale described as Lot 6 on RP747166.

The proposed development application will seek to increase enrolment and staffing numbers, a demolition and reconstruction of H Block (Heart & Hub) as well as extension of the existing car park along the eastern boundary of the site.

The engineering services report addresses the following with regards to the proposed development:

- **Water servicing:** Assessment and design of potable water supply infrastructure to meet applicable authority requirements.
- **Wastewater servicing:** Assessment and design of wastewater infrastructure to ensure adequate servicing capacity and compliance with authority standards.
- **Stormwater management:** Assessment and design of the stormwater network and drainage system including overland flow paths, and identification of a lawful point of discharge.
- **Stormwater quality**: MUSIC modelling of treatment train and compliance reporting.
- **Flooding:** Hydraulic and hydrological modelling, flood impact assessment and design mitigation.

2.0 SITE

The proposed development is located approximately 7km from the Townsville CBD at:

- 104-156 Yolanda Drive, Annandale (Lot 2 on RP850718 and Lot 3 on SP256046); and
- 9 Jonquil Crescent, Annandale (Lot 6 on RP747166).

Lot 2 on RP850718, Lot 3 on SP256046 and Lot 6 on RP747166 is hereon in referred to as the subject site.

The subject site is 6.215ha in area and is bounded by Yolanda Drive to the south, Jonquil Crescent to the north, Cypress Drive to the northwest and neighbouring residential land on all other sides. Refer to **Figure 1** in red for site locality.

Figure 1. *Site Locality (Source: Queensland Globe)*

3.0 PROPOSED DEVELOPMENT

The proposed development includes the demolition H Block (OSHC and classrooms) and is to be replaced with a new two storey administration building (also referred to as Hub and Heart Building). In addition, a car park extension located on the eastern boundary of the subject site is proposed. The recently acquired land located at 9 Jonquil Crescent (Lot 6 on RP747166) is approved to be utilised as office space and will be utilised as a temporary administration office during the demolition of H Block and construction of the proposed Hub & Heart Building. Refer to **Figures 2** and **3** for the proposed new administration building locality plan, and **Figure 4** for the proposed car parking extension. The proposed development plans are attached to **Appendix A.**

Figure 2. *Existing and Proposed Development Locality (Source: Burling Brown)*

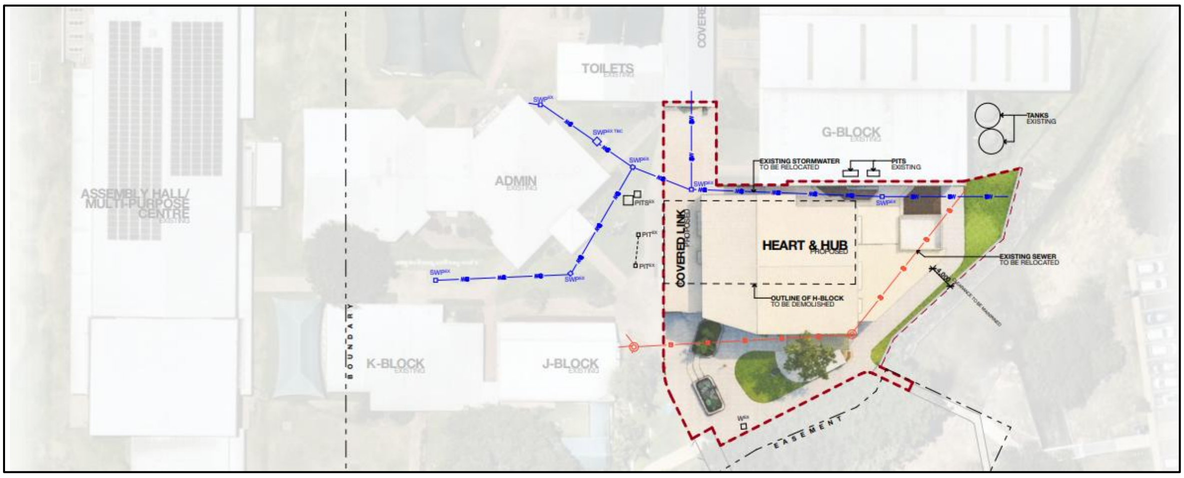

Figure 3. *Proposed Development Layout Plan (Source: Burling Brown)*

Figure 4. Proposed car parking extension

4.0 WATER SERVICING

A water supply and sewerage capacity assessment was undertaken by DPM Water for the proposed Annandale Christian College development to assess the ability of the existing infrastructure to service the projected student and staff population in accordance with Townsville City Council (TCC) Planning Scheme and Cairns, Townsville and Mackay (CTM) Code standards.

The full technical report dated 7 April 2026 (Revision B) is provided in **Appendix B.**

The key assumptions and findings of the report are as follows:

<u>Trunk infrastructure assumptions</u>

- The site is serviced from the **Douglas No. 1A/B reservoirs** via the established Annandale trunk and reticulation water network.
- Existing trunk mains (DN375, DN300 and DN225 AC) and local reticulation mains, including **DN100 AC mains along Jonquil Crescent**, are retained with no upgrades required.
- Water network modelling was undertaken using Council's **WaterGEMS** model.

<u>Demand</u>

- Design per **CTM Code**:
  - Average Day: 600 L/EP/day
  - Peak Hour: 0.033 L/s/EP (peaking factor 2.56)
- A fire flow requirement of **15 L/s** was applied for all stages, consistent with "Park Residential" zoning.
- The projected equivalent population for water servicing is approximately **188 EP**, inclusive of students, staff, and the additional building at **9 Jonquil Crescent**.

Network performance

- Standard requirements:
  - Peak Hour Pressure ≥ **220 kPa**
  - Fire Flow Pressure ≥ **120 kPa**
  - Headloss Gradient ≤ **0.005 m/m**
- **Peak hour pressures** remain well above the minimum required **220 kPa**, with typical pressures of approximately:
  - **518 kPa** during peak school demand (midday)
  - **456 kPa** during peak residential demand (7 pm)
- With the inclusion of a **15 L/s fire flow**, network pressures reduce to approximately **404 kPa**, exceeding the minimum **120 kPa** requirement.
- **Pipe velocities** remain below the CTM Code limit of **2.5 m/s**.
- Short sections of DN100 mains experience higher theoretical headloss gradients during peak school demand; however, these occur over short durations, with acceptable velocities and pressures maintained at all times.

Conclusion

- The existing trunk and reticulation water network has **sufficient capacity** to service the projected student and staff population at Annandale Christian College. All peak hour and fire flow performance criteria are met, and no water infrastructure upgrades are required to support the development under Council standards.

5.0 WASTEWATER SERVICING

5.1 EXTERNAL WASTEWATER SERVICING

A sewerage network capacity assessment has also been undertaken by DPM Water for the proposed Annandale Christian College development to assess the ability of the existing gravity sewer infrastructure to service the projected student and staff population.

The full SewerGEMS modelling report is provided in the relevant **Appendix B.**

The key assumptions and findings of the report are as follows:

<u>Existing sewer infrastructure</u>

- The site is serviced by an existing **reticulated gravity sewer system** discharging to **Pump Station PS S4A**.
- The main school site drains via **DN150 gravity sewers** from MH 3/S4A7A and MH 7/S4A.
- The building at **9 Jonquil Crescent** is serviced independently via a **DN150 gravity sewer** from MH 1/S4A7B1.
- Downstream infrastructure includes **DN300 and DN375 trunk gravity sewers** progressing to PS S4A and the wider catchment.

<u>Demand</u>

- Equivalent population (EP) generation was calculated in accordance with the **TCC Planning Scheme**.
- The projected sewer equivalent population is approximately **291 EP**, inclusive of students, staff, and the additional building at 9 Jonquil Crescent.
- Sewer modelling was undertaken at **5 × Average Dry Weather Flow (5 × ADWF)**, consistent with CTM Code requirements.

<u>Network performance</u>

- Standard requirements:
    - Maximum allowable sewer flow: 75% full (CTM Code).
    - All assessed sewer infrastructure operates within allowable capacity limits.
- The SewerGEMS capacity assessment confirms:
- DN150 gravity sewer servicing the main school site flows to approximately **57% full** at 5 × ADWF.
- DN150 gravity sewer servicing 9 Jonquil Crescent flows to approximately **48% full**.
- Downstream DN300 trunk sewer flows up to **71% full**.
- DN375 sewer immediately upstream of PS S4A flows up to **49% full**.

Conclusion

- The existing gravity sewer system and associated downstream infrastructure, including PS S4A, have **sufficient capacity** to accommodate the projected student and staff population at Annandale Christian College. All sewer mains operate below the CTM Code maximum capacity thresholds at 5 × ADWF, and **no sewer infrastructure upgrades are required** to service the development.

5.2 INTERNAL WASTEWATER SERVICING

As part of the proposed reconstruction of the H Block (Hub and Heart) building, a realignment of the existing internal sewer network is anticipated to avoid building over or in close proximity to sewer infrastructure. While the internal sewer network is a privately owned and maintained asset, the realignment will be undertaken to achieve appropriate building clearances. The proposed realignment will also improve long-term serviceability of the internal sewer network. The proposed realignment is shown in **Figure 5**.

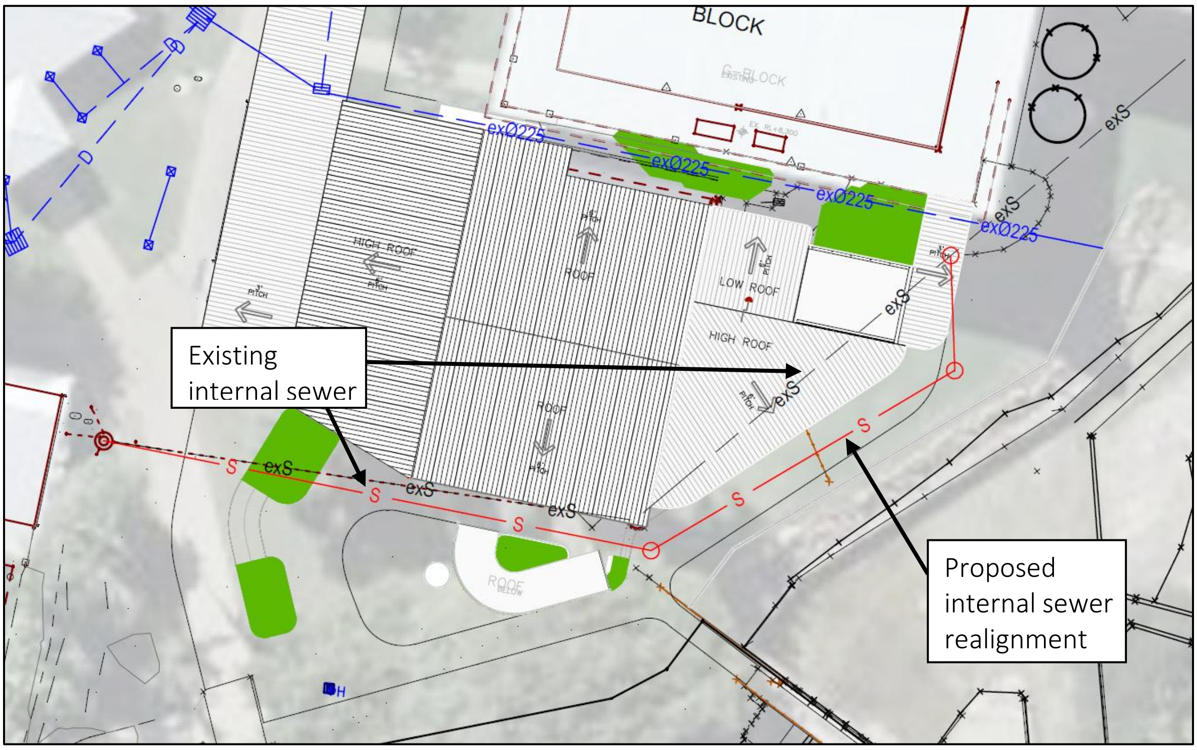

Figure 5. *Proposed internal sewer realignment*

6.0 TRAFFIC NETWORK

A Traffic Impact Assessment (TIA) report has been prepared to assess any impacts the development may have on the existing road network and mitigation measures that may be required as part of the development. The report is available in **Appendix C** of this report.

The report has found the following with regards to the existing site and access conditions and operational performance:

<u>Existing site and access conditions</u>

- Site is accessed via **seven (7) existing driveways** as shown in **Figures 6 and 7**:
  - four (4) on Yolanda Drive; and
  - three (3) on Jonquil Crescent.
- No new accesses are proposed.
- Surrounding roads include
  - Yolanda Drive (major collector, 60 km/h);
  - Cypress Drive (residential streets, 50 km/h); and
  - Jonquil Crescent (residential street, 50km/h).
- All key intersections and accesses are **unsignalised**.
- Crash history within 150m of key intersections and accesses shows **no systemic safety issues**, with only three isolated crashes in the last 10 years.

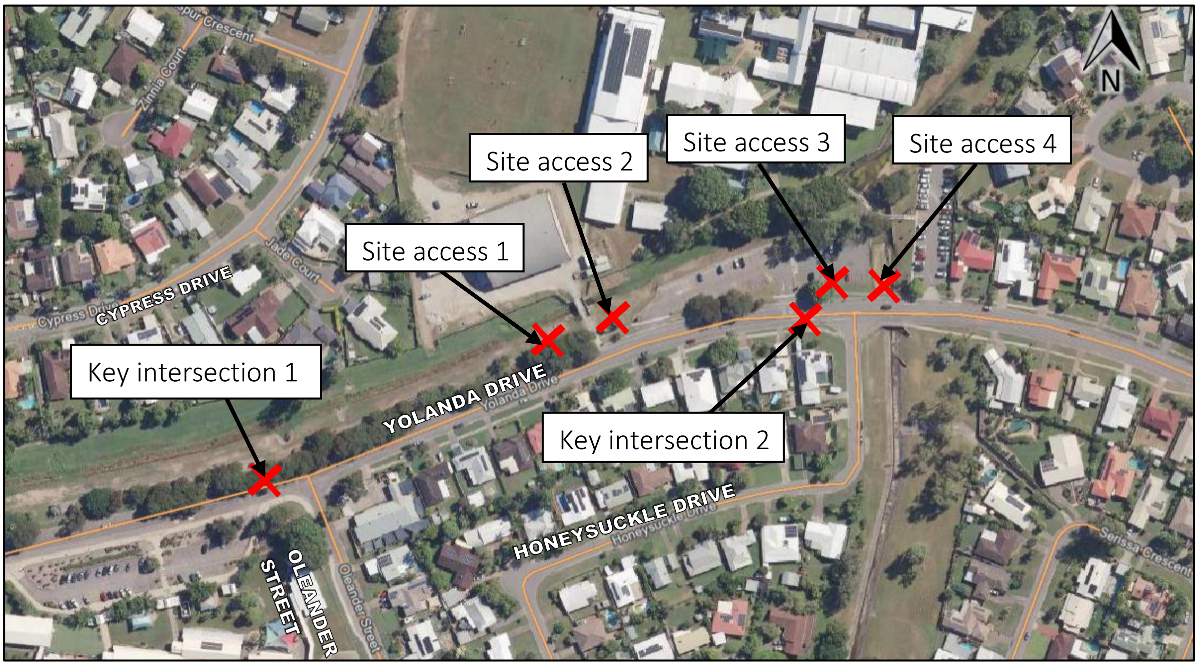

Figure 6. Key intersections and accesses on Yolanda Drive (Source: Queensland Globe)

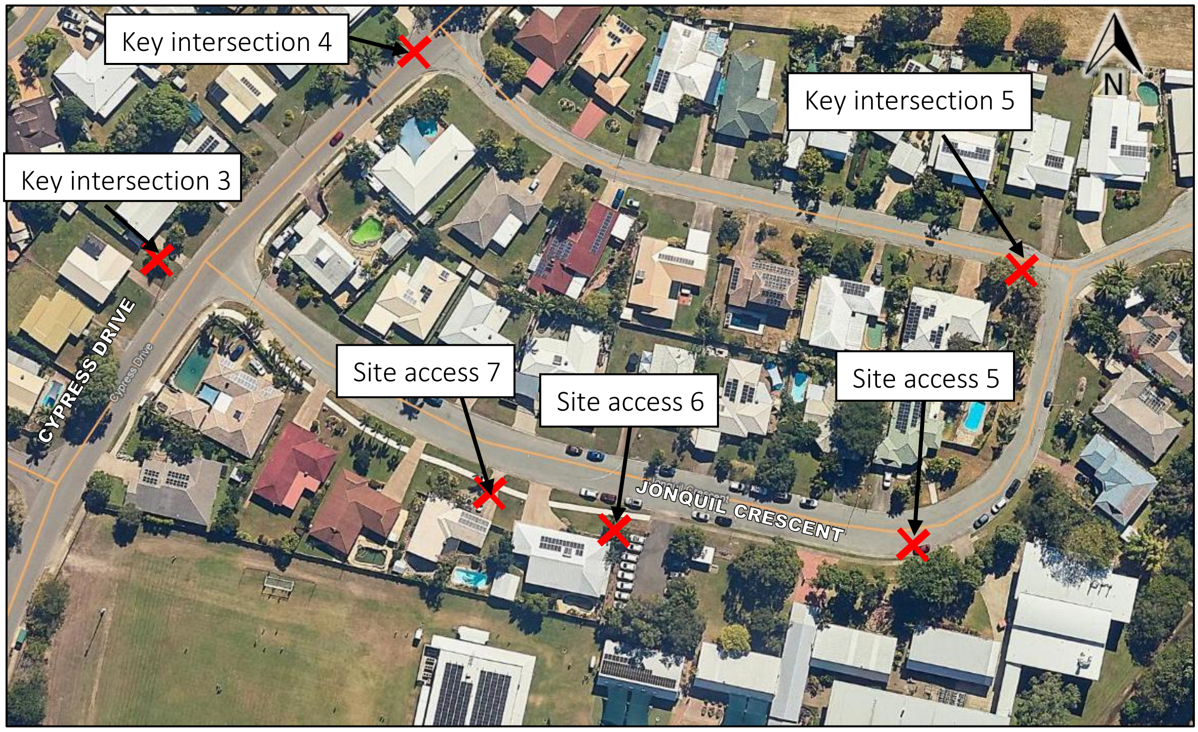

Figure 7. Key intersections and accesses on Cypress Drive and Jonquil Crescent (Source: Queensland Globe)

<u>Operational Performance</u>

- SIDRA Intersection modelling was undertaken for 2027 (base year) and 2037 (10-year horizon).
- Across all assessed intersections and site accesses, **Levels of Service remain A or B**, with Degrees of Saturation well below capacity thresholds in both AM and PM peaks.
- The development results in **minimal increases in delay and saturation**, with no significant deterioration in overall network performance.

The TIA confirms that no major road network upgrades are required. However, a number of **localised corrective measures** are recommended to ensure ongoing compliance with relevant standards, maintain sight distance, and reinforce pedestrian safety within the school environment.

Turn treatments and intersection configuration

Turn warrant assessments indicate that existing arrangements remain appropriate:

- All site accesses currently operate with **Simple Left (SL)** and **Simple Right (SR)** turn treatments under existing conditions.
- Yolanda Drive functions as a major collector road with posted speeds of 60 km/h (40 km/h during school hours). For the majority of site accesses along Yolanda Drive:
- Turn warrant assessments indicate that a **Basic Left-turn (BAL)** and **Basic Right-turn (BAR)** arrangement represents the appropriate minimum standard treatment.
- An urban BAL on a two-lane, two-way road generally only consists of no obstructions on the upstream approach to the intersection. No set dimensions are provided in Austroads, however, parking should be restricted to not limit the sight distance to or from the intersection.
- Whilst it is acknowledged that a BAR is generally the accepted minimum, no right turn treatment is proposed at the accessed due to the school environment and given the following:
    - Very low right-turn volumes associated with the site;
    - Consistently low delays and Degrees of Saturation (LOS A–B);
    - Absence of crash history indicative of a turning-movement deficiency; and
    - The school zone environment, where pedestrian safety, speed control and minimisation of crossing distances take precedence over vehicular capacity.
    - BARs would increase the number of potential conflict points and introduce geometric mania to the area and resulting in increased driver decisions;
    - Through traffic may potentially travel at a higher speed when pass and reduce yielding behaviour which reduces pedestrian safety; and
    - In accordance with Queensland Road Safety Technical User Volumes (QRSTUV): Guide to Schools, “Design should reduce crash severity and exposure before seeking to optimise traffic flow.” and “Treatments that favour capacity or speed at the expense of safety are inappropriate in school environments.”
- Given the above and the school environment, it is not considered practical or safe to provide right turn treatment at the site accesses. It is proposed that there is no change to the right turn treatments of existing site accesses.

Vehicle Sight Distance and Parking Management

To maintain minimum sight distances for safety at the access in accordance with **AS 2890.1**, the access-specific measures shown in **Figures 8** and **9** (or **Appendix C** of the TIA) are required .

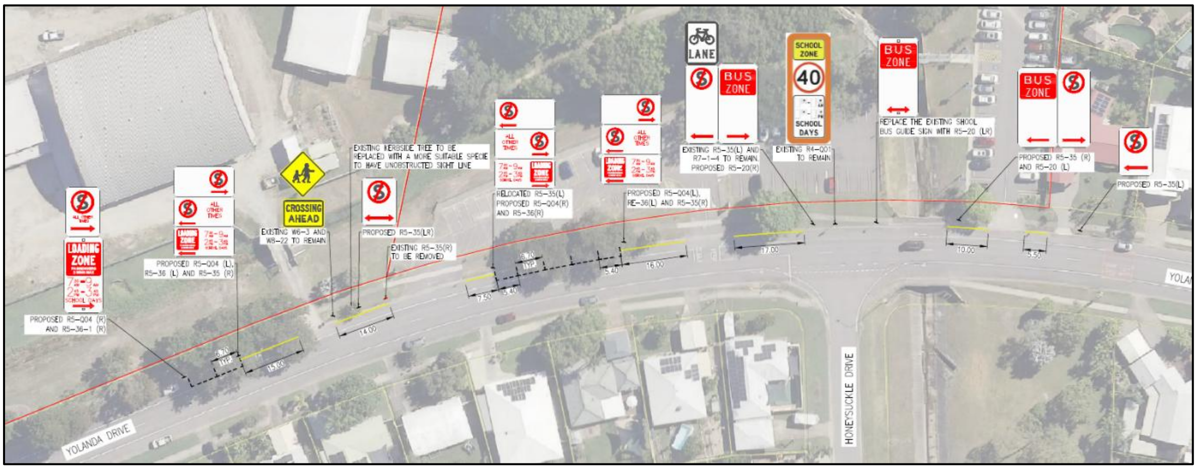

Figure 8. Proposed signage and linemarking plan – Yolanda Drive

Figure 9. Proposed signage and linemarking plan – Jonquil Crescent

Pedestrian and Cyclist Safety Enhancements

No high-risk conflict points were identified between vehicles, pedestrians, or cyclists provided certain measures are implemented. Risk assessments identified all access as overall **low risk** provided the following is implemented:

- **routine vegetation trimming** at Site access 5,
- relocation of obstructive signage at Site access 3 as necessarily identified in the report.
- Whilst not required consideration may be given to application of **pavement markings or contrasting surface treatments** across accesses which do not already have them.

Parking Provision and Internal Circulation

- The proposed development provides a total of **170 parking spaces**, exceeding the Townsville City Plan requirement of **153 spaces** for the ultimate development scenario.
- The eastern car park extension and drop-off areas are designed to comply with **AS 2890.1**, including internal circulation, manoeuvring, and turning requirements.
- Modification to the existing staff car park on Jonquil Crescent is required to meet AS2890.1 (off-street parking) and NCC requirements. Modifications include, provision of an accessible parking space, and provision of a turnaround facility in the form of an empty bay.
- It is noted that the existing blind aisle east of Site access 3 on Yolanda Drive is more than maximum number of spaces for a blind aisle before a turnaround provision is required. The facility also does not have a 1m extension at the end of the aisle, however, due to the wide aisle (12m width at eastern end) swept paths have been checked and does not pose an issue with vehicles departing the end bays and a B99 is still able to perform a 3-point turn within the aisle to depart in a forward motion.

Summary

The TIA demonstrates that the proposed development at Annandale Christian College will operate safely and efficiently within the existing road network for both the 2027 and 2037 assessment horizons. All intersections and site accesses maintain **acceptable Levels of Service (A–B)** with **Degrees of Saturation well below capacity** thresholds, and the additional traffic generated by the development does not adversely affect network performance. Subject to implementation of the recommended localised mitigation and correction measures (primarily involving parking controls, signage, linemarking and minor vegetation management) the development is considered acceptable from a traffic engineering and road safety perspective, provided the mitigation measures identified are implemented.

7.0 STORMWATER REGIME

7.1 EXISTING STORMATER REGIME

7.1.1 Topography

Site elevations and contours have been obtained from survey and ELVIS Spatial 2018 LiDAR. The general overland flow direction for the site and surrounding area is shown in **Figure 10**.

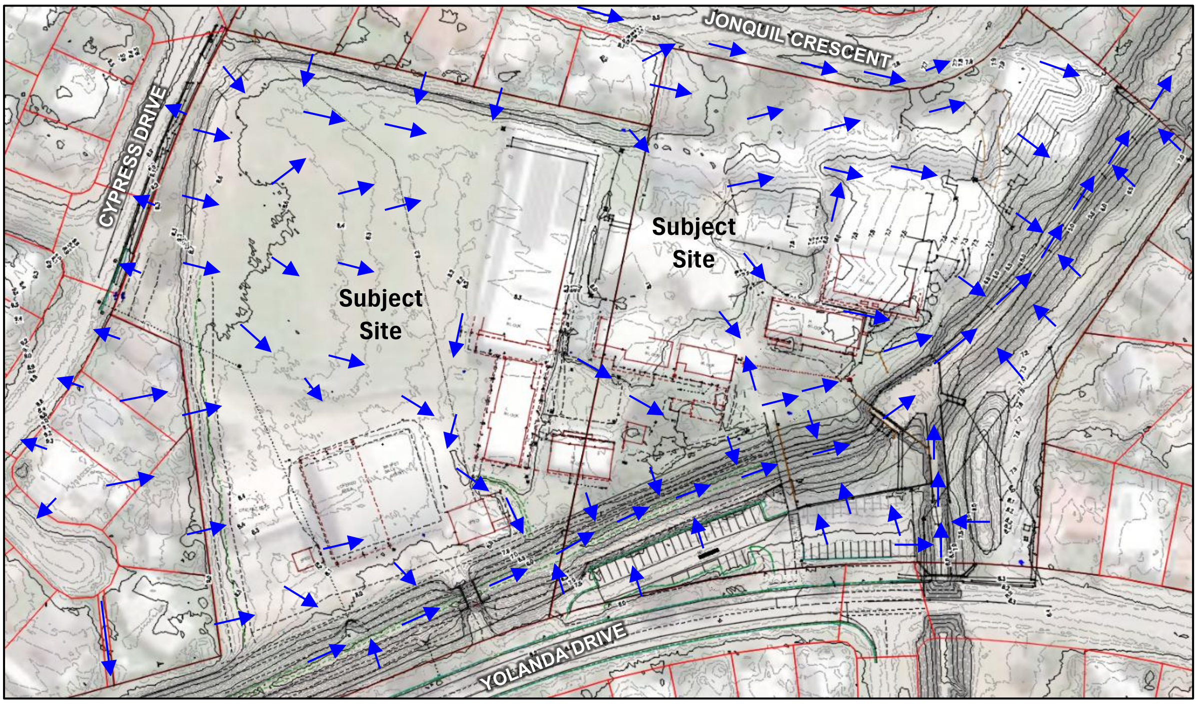

Figure 10. *Existing overland flow directions (Source: ELVIS Spatial)*

The lawful point of discharge for the subject site is the existing open drain, classified as an Order 1 watercourse (canal) under the Queensland Watercourses framework, which runs generally parallel to the southern and eastern site boundaries. Review of existing site topography indicates that post-development and existing overland stormwater flows predominantly in an easterly direction toward this drain. The open drain functions as the primary receiving drainage infrastructure for the site, conveying flows along the southern and eastern extents before discharging to the **Ross River**, which ultimately drains to the Pacific Ocean. **Figure 11** illustrates the broader stormwater drainage pathway from the subject site, through the downstream open drain network, and into the Ross River catchment.

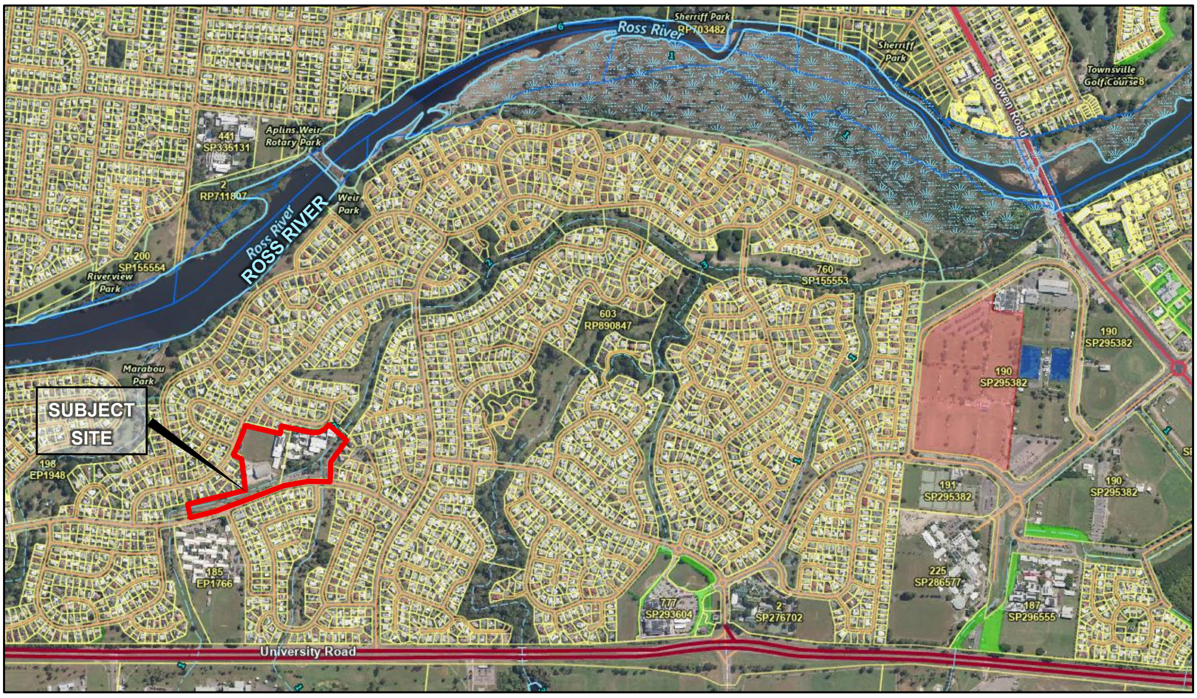

Figure 11. *Existing watercourses (Source: Queensland Globe)*

7.1.2 Flood overlay

Flood mapping from the Townsville flood information portal indicates that the subject site may be affected by flooding during a 1% AEP (ARI 1) event. Refer to **Figure 12** for flood mapping of the subject site and surrounding area.

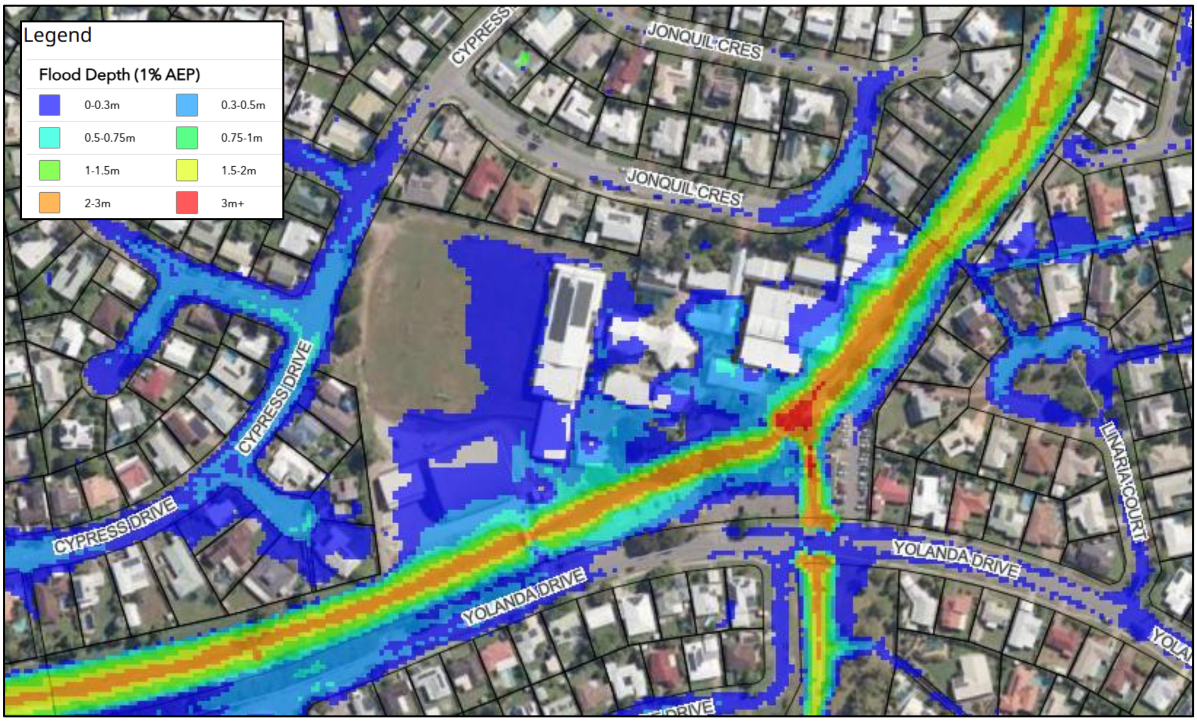

Figure 12. *Flood mapping (Source: Townsville - Flood Information Portal)*

7.2 HYDROLOGICAL ASSESSMENT

Design of the stormwater network has been analysed using the Rational Method. The guideline utilised for the calculation of the rational method is as per the Queensland Urban Drainage Manual (QUDM) 2017.

$$Q_y = (C_y \,.\, {}^tI_y \,.\, A) / 360$$

Where: Q_y = peak flow rate (m^3/s) for average recurrence interval (ARI) of ''y' years

C_y = coefficient of discharge (dimensionless) for ARI of 'y' years

A = area of catchment (Hectares)

${}^tI_y$ = average rainfall intensity (mm/h) for a design duration of 't' hours and an ARI of 'y' years

t = the nominal design storm duration as defined by the time of concentration

The recommended design ARI rainfall event for underground drainage from the Queensland Government design standards for Queensland education facilities is an **ARI 20 (5% AEP).**

The following steps have been undertaken for calculation of peak flow rates:

a) Analysis of possible flow paths based on available contours;
b) Determination of the time of concentration of each flow path;
c) Adoption of the flow path with the longest time of concentration (t_c) for design;
d) Determination of the runoff coefficients; and
e) Calculation of peak flow rate for drainage structures.

Standard inlet time to the first field inlet pit plus estimated pipe flow time has been adopted for the assessment.

7.3 CATCHEMNT AT SITE POINT OF DISCHARGE

Stormwater onsite discharges to the existing formalised drainage channel before leaving the site. The overall catchment of to the downstream site discharge point is shown in **Figure 13**. Stormwater is currently typically allowed to sheet flow to the open drainage channel.

Figure 13. *Catchment to site discharge point*

It is noted that whilst the development proposes to increase the impervious area, the change in discharge as a result of the change in impervious area is not anticipated to have any significant impacts on downstream areas. The proposed change in impervious area is 1,627m2. Impacts of the change have been assessed though rain on grid flood modelling which are described in **Section 8.0** of this report. The following pre- and post-development stormwater assessments reviews the capacity of the existing onsite stormwater structures and any mitigation that may be required.

7.4 PRE-DEVECLOPMENT CATCHMENT

The existing catchment, discharge point between H Block and G Block, and impervious areas of the site are shown in **Figure 14**. Pre-development impervious area is 60 % of the catchment area.

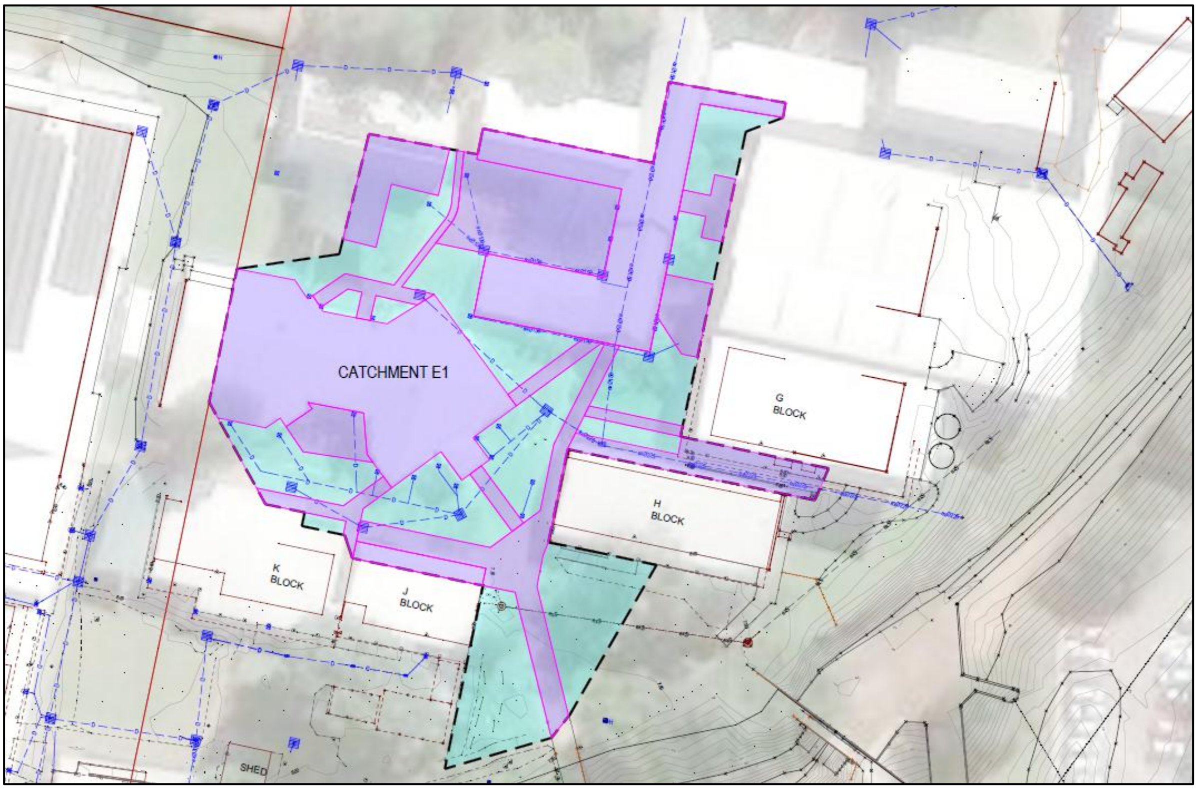

Figure 14. *Existing catchment (cyan) and impervious areas (magenta)*

7.5 PROPOSED SITE DRAINAGE REGIME

No changes are proposed to the discharge location of the existing pipe network between G Block and H Block. However, minor system augmentation is proposed to ensure the network has sufficient capacity to convey flows up to an ARI 20 (AEP 5%) design event.

The pipe infrastructure has been designed assuming an impervious catchment fraction of 80%, thereby allowing for potential future intensification of impervious area (i.e. pathways, courtyards etc.) and associated increases in runoff. This conservative design approach mitigates the need for future pipe replacement, which would likely be logistically challenging and cost prohibitive. The proposed network is shown in **Figure 15** .

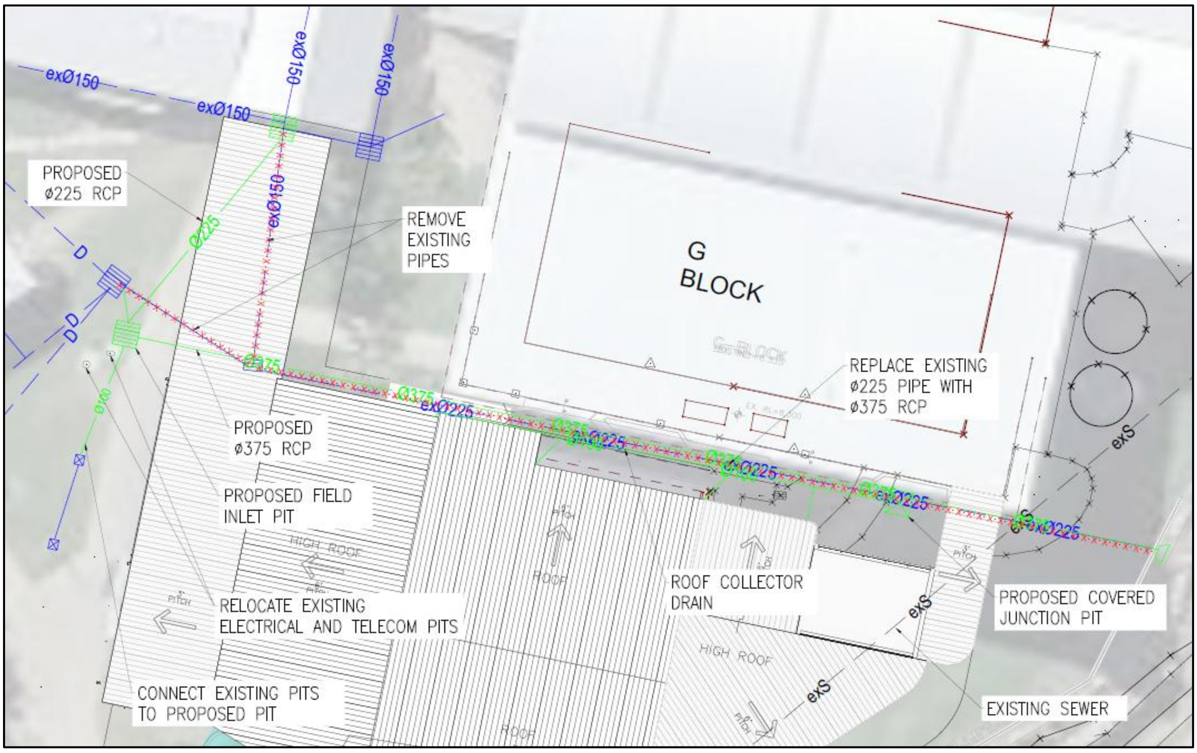

Figure 15. Proposed site drainage and culverts

7.6 POST-DEVELOPMENT CATCHEMENT

The post-development catchments are shown in **Figure 16**. Post-development the impervious area of the catchment to the outlet is 70% of the catchment area. An impervious area of 80% has been adopted for the design of the network. Catchment P2 is a sub-catchment as a result of the of the new proposed roof. Catchment P3 feeds into Catchment P2, however, has been broken out for MUSIC modelling.

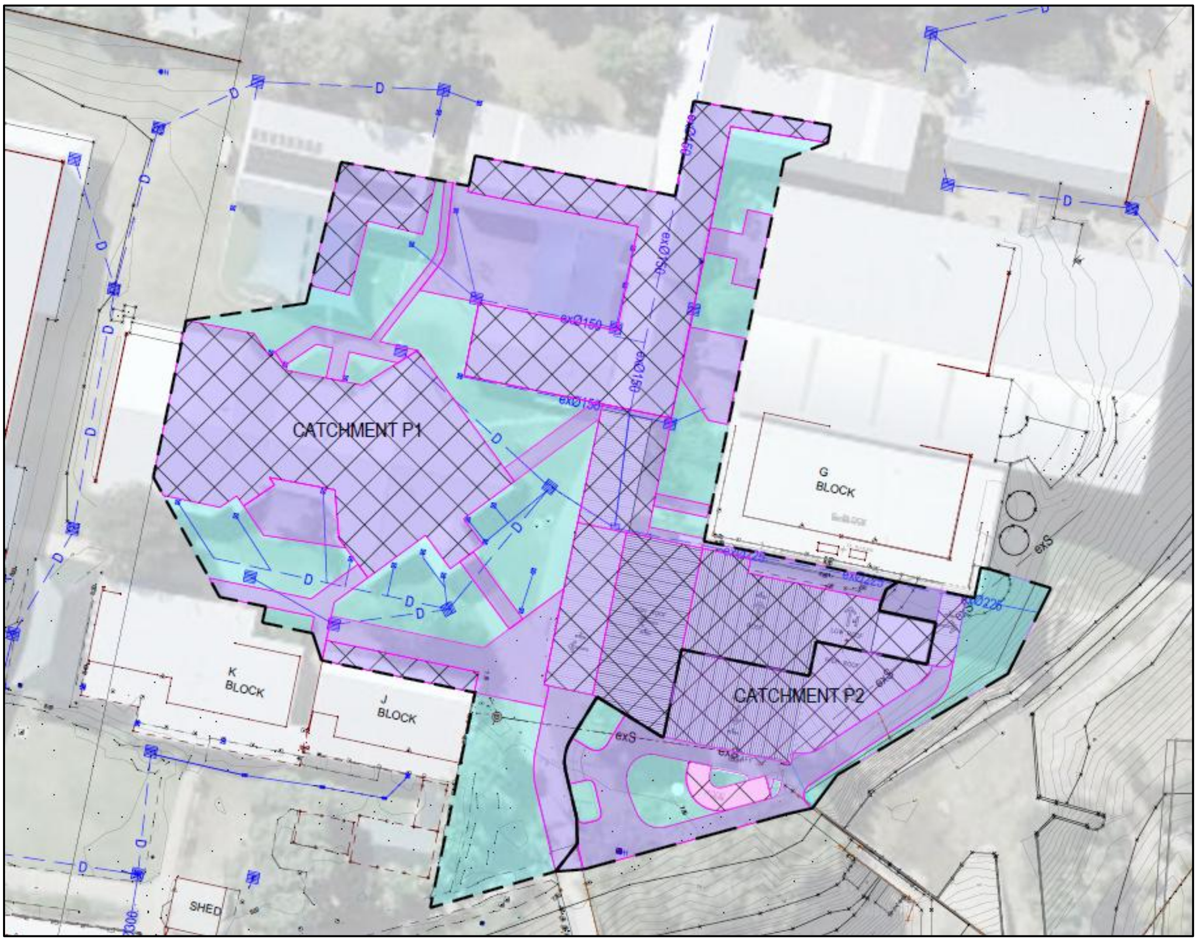

Figure 16. *Post-development H Block catchments and impervious areas (magenta)*

No stormwater structures are required with respects to the carparking extension and stormwater can be discharged via sheet flow to the existing proposed open channel drain. As stated above, the minor increase will be insignificant compared to the overall catchment and this has been verified with flood modelling. It is noted that from stormwater quality assessment a bioretention swale will be required to mitigate stormwater quality from the carparking area. Refer to **Section 9.0** of this report for details.

7.7 HYDRAULIC ASSESSMENT

Utilising the formulas in **Section 5.0** of this report the pre- and post-development catchments areas and discharges flows are summarised in **Table 1**.

Table 1. Pre vs post-development catchment area and discharge

Catchment	Pre-development				
	Area	t_c	Impervious area	Discharge	
				Storm Event	
	m²	**min**	**%**	ARI 20 (5% AEP) **m³/s**	ARI 100 (1% AEP) **m³/s**
Catchment E1	3,960	5.5	0.6	0.215	0.313
Catchment P1	4,350	5.5	0.8 Adopted (0.7 actual)	0.248	0.349
Catchment P2	882	10.6	0.8 adopted (0.7 actual)	0.041	0..057

Based on the post-development catchment the existing 225mm dia. pipe must be replaced with a 375mm dia. RCP to ensure that it can accommodate the proposed development and future development in the catchment up to 80% impervious.

As part of the stormwater quality management plan, Catchment P2 must release into a swale. The swale profile must have a minimum 0.4m wide base, a depth of 0.3m (including 0.15m freeboard) and 1 in 2 batters.

Refer to **Appendix D** for hydrological and hydraulic calculations.

8.0 FLOODING

Flood modelling of the subject site has been conducted by Venant Solutions (Ref L.M00676.FloodReport.001.00.docx) to assess the flood impact of the proposed works. Refer to **Appendix E** for full flood modelling report.

Council provided their 2021 Ross River TUFLOW hydraulic model to Venant Solutions to undertake this impact assessment. The model was refined based on site survey where available. It is noted the refinement of the model resulted in some changes in levels compared to Council's provided model including:

- Reductions in flood level in the drain parallel to Yolanda Drive. These are due to the reduced cell size improving the representation of the drain conveyance which allows more flow along the drain, particularly at some of the foot bridges across the drain.
- Downstream of the Site there are minor increases in flood level of around 13 mm in the drain. A significant reduction in flood level in the drain parallel to Yolanda Drive, with decreases of up to 900 mm.
- These reductions are mainly due to the better representation of the area under the east footbridge because of the survey resulting increased waterway area and hence flow through the bridge.
- Increases in flood level in the drain downstream of the Site are around 60 mm and are due to the increased flow capacity of the area immediately upstream as a result of the improved model.

A comparison of the existing scenario and proposed development scenario were run and compared. The change in 1% AEP flood levels is shown below in

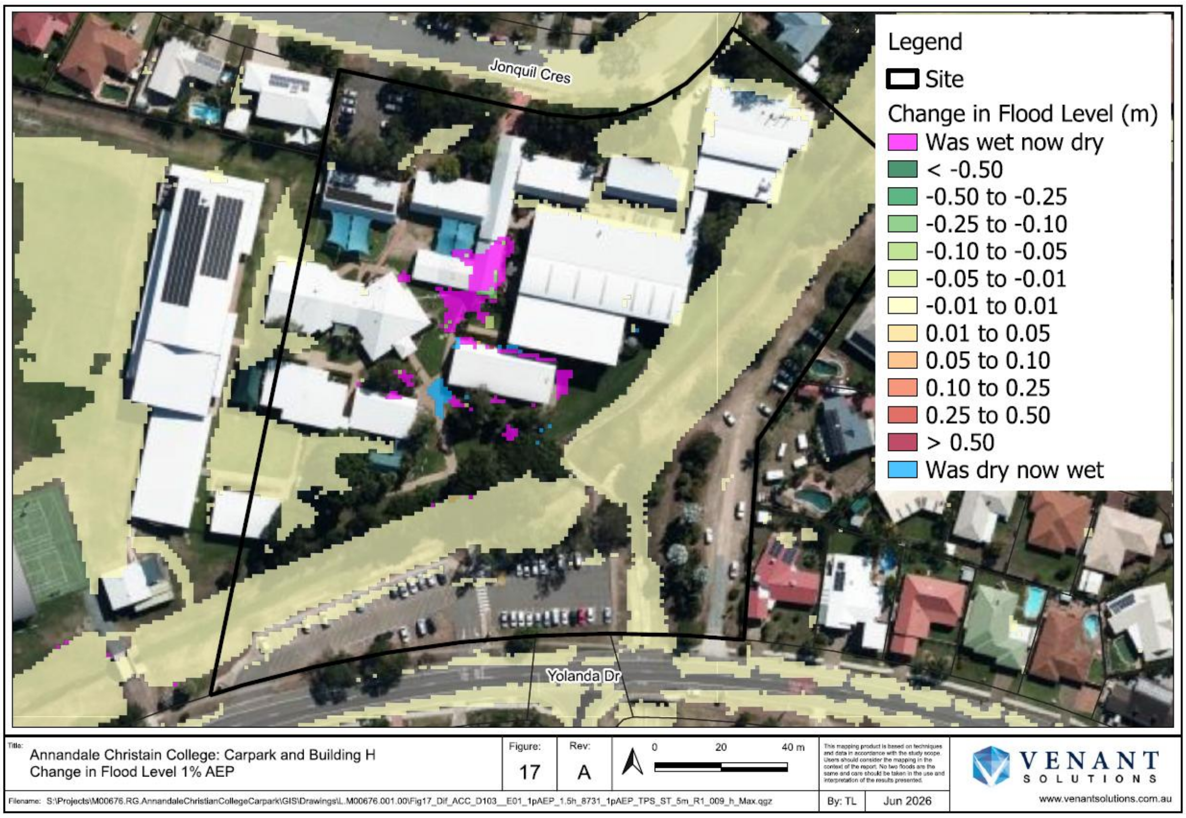

Figure 17. Subject

The carpark area is proposed on top of the bank with little change to the levels as such it will not result in significant changes in flood levels to the area if any at all. Sampling points were taken at the proposed access points to determine the 1% AEP flood levels and required minimum floor levels which are summarised **Table 2**.

Table 2. Sampling point for 1% AEP Flood Levels

Location	1% AEP Flood level (m AHD)	Required Floor Level (m AHD)	Proposed Floor Level (m AHD)
GLA1	8.02	8.32	8.335
GLA2	N/A	N/A	8.335
East Entrance	6.95	7.25	8.335
Staff Lounge	N/A	N/A	8.335
Delivery Room	N/A	N/A	8.335
Lobby	7.81	8.11	8.335

The report has found that the following with respects the development:

- Does not impede the flow of flood waters through the site or worsen flood flows external to the site;
- Ensures any changes to the depth or velocity of flood waters are contained within the site;
- Does not directly, indirectly or cumulatively worsen flood characteristics outside the development site, having regard to increased scour and erosion; and
- Provides 1% AEP flood levels so that floor levels can be designed with appropriate freeboard.

9.0 STORMWATER QUALITY ASSESSMENT

Water quality modelling has been conducted using the Model for Urban Stormwater Improvement Conceptualisation (MUSICX) software version 1.50.1.14015 to demonstrate the TCC target reduction criteria. Stormwater treatment trains were developed and modelled for the sites to determine the effectiveness of the water quality measures in achieving the relevant water quality objectives.

The proposed lot layout has been modelled to demonstrate compliance with State Planning Policy (SPP) and TCC target reduction criteria. Stormwater treatment trains were developed and modelled for the sites to determine the effectiveness of the water quality measures in achieving the relevant water quality objectives.

9.1 MODELLING ASSUMPTIONS

- The rainfall, runoff and pollutant parameters adopted within the MUSICX Model are detailed in **Tables 2, 3 and 4**.
- In accordance with Water by Design, MUSIC Modelling Guidelines (2018), Table 3.1, urban source nodes for residential has been adopted.
- Exfiltration of 0mm/hr has been adopted for all relevant nodes.

Table 3. Rainfall Parameters

Input Parameter	Data Used in Modelling
Rainfall station	32040 Townsville Aero
Time step	6 minutes
Modelling period	31/03/2000 - 31/03/2010
Rainfall runoff parameters	Residential
Pollutant export parameters	Residential

Table 4. Runoff Parameters

Input Parameter	Data Used in Modelling
Land use	Residential
Rainfall threshold (mm)	1.0
Soil storage capacity (mm)	500
Initial storage (% capacity)	10
Field capacity (mm)	200
Infiltration capacity coefficient (a)	211
Infiltration capacity coefficient (b)	5.0
Initial depth (mm)	50
Daily recharge rate (%)	28
Daily baseflow rate (%)	27
Daily deep seepage rate (%)	0

Table 5. Pollutant Export Parameters for land use

Surface Type	Flow Type	Total Suspended Solids (log mg/L)		Total Phosphorous (log mg/L)		Total Nitrogen (log mg/L)	
		Mean	Std Dev.	Mean	Std Dev.	Mean	Std Dev.
Residential Roof	Baseflow parameters	N/A	N/A	N/A	N/A	N/A	N/A
	Stormwater parameters	1.30	0.39	*0.89	0.31	0.26	0.23
Residential Roads	Baseflow parameters	1.00	0.34	-0.97	0.31	0.20	0.20
	Stormwater parameters	2.43	0.39	-0.30	0.31	0.26	0.23
Residential Roads	Baseflow parameters	1.00	0.34	-0.97	0.31	0.20	0.20
	Stormwater parameters	2.18	0.39	*0.47	0.31	0.26	0.23

9.2 WATER QUALITY OBJECTIVES (WQO)

It is acknowledged that Townsville City Council (TCC) set the following design objectives for stormwater quality:

- ≥ 80% reduction in total suspended solids load
- ≥ 65% reduction in total phosphorus load
- ≥ 40% reduction on total nitrogen load
- ≥ 90% reduction in gross pollutant load.

While the planning policy expresses stormwater quality outcomes in terms of percentage pollutant load reductions, these targets are generally intended to apply to new or substantially redeveloped sites where there is sufficient flexibility to integrate treatment infrastructure. In this case, the subject site is predominantly existing and almost fully developed, with limited opportunity for meaningful retrofitting of stormwater quality measures.

Accordingly, a **performance-based assessment** has been undertaken comparing **pre-development** (scenario based on site 10 years ago) **and post-development residual pollutant loads (expressed in kg/year)** rather than percentage reductions. This approach provides a more appropriate and transparent evaluation of the development's actual impact on receiving water quality. On this basis, the stormwater quality management objective for the development has been set to **no worsening of existing conditions**, ensuring that post-development pollutant loads do not exceed pre-development levels, which is considered a reasonable and pragmatic outcome given the site constraints.

9.3 MUSICX MODELLING RESULTS

9.3.1 Pre-development

An aerial of the adopted pre-development case taken 6 Jun 2016 is shown in **Figure 16**. The adopted catchments, treatment train and results are provided in **Figure 19**, **Figure 20** and **Table 6** respectively.

Figure 18. *Subject site pictured 6/06/2016 (Source: QLD Globe – Townsville_2016_10cm_SISP_Urban)*

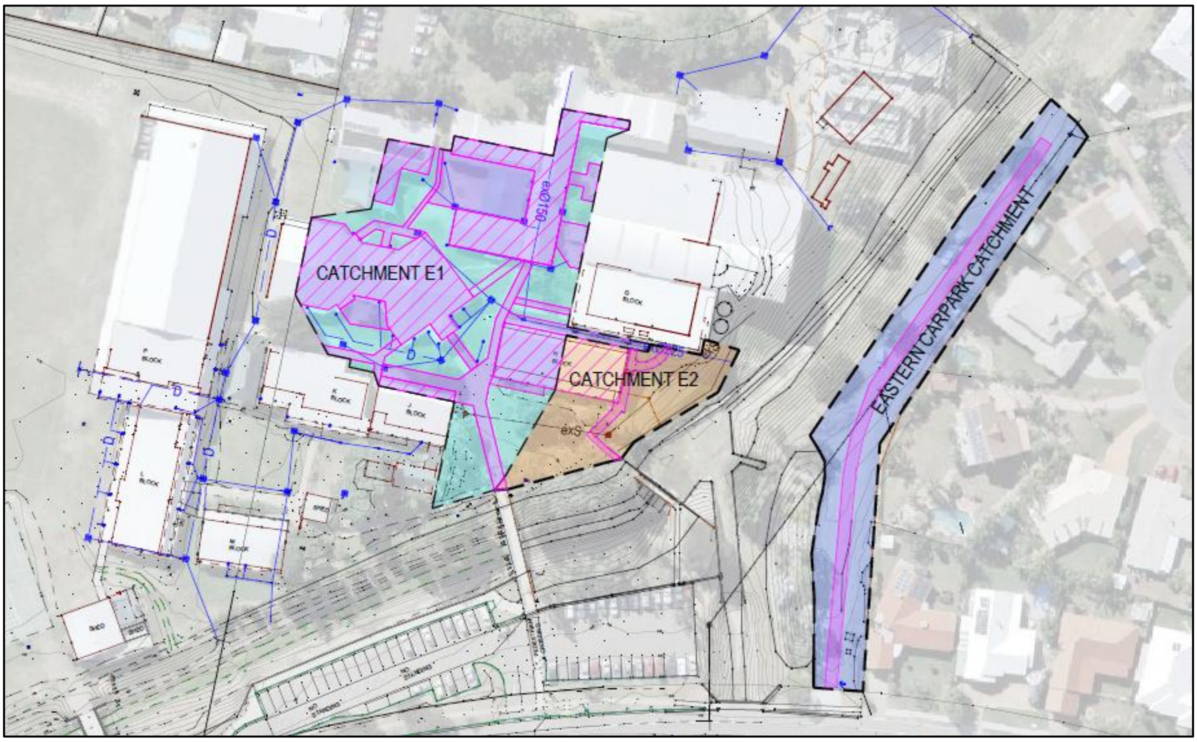

Figure 19. *Pre-development MUSIC catchments*

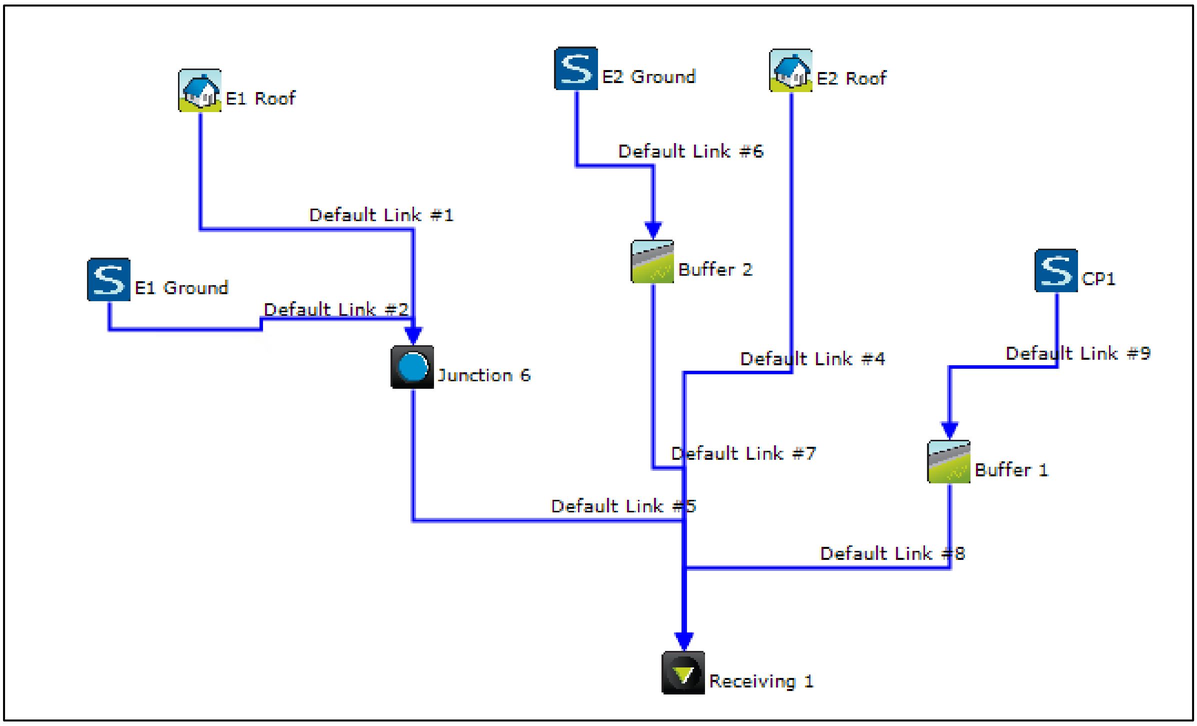

Figure 20. *Pre-development MUSIC treatment train*

Table 6. *MUSICX modelling results pre-development*

	Sources	Residual Load
Flow (ML/yr)	3.956	3.956
Total Suspended Solids (kg/yr)	545.5	374
Total Phosphorus (kg/yr)	1.235	1.026
Total Nitrogen (kg/yr)	8.125	7.611
Gross Pollutants (kg/yr)	53.71	53.71

9.3.2 Post-development

The adopted catchments, treatment train and results are provided in **Figure 19**, **Figure 20** and **Table 7** respectively.

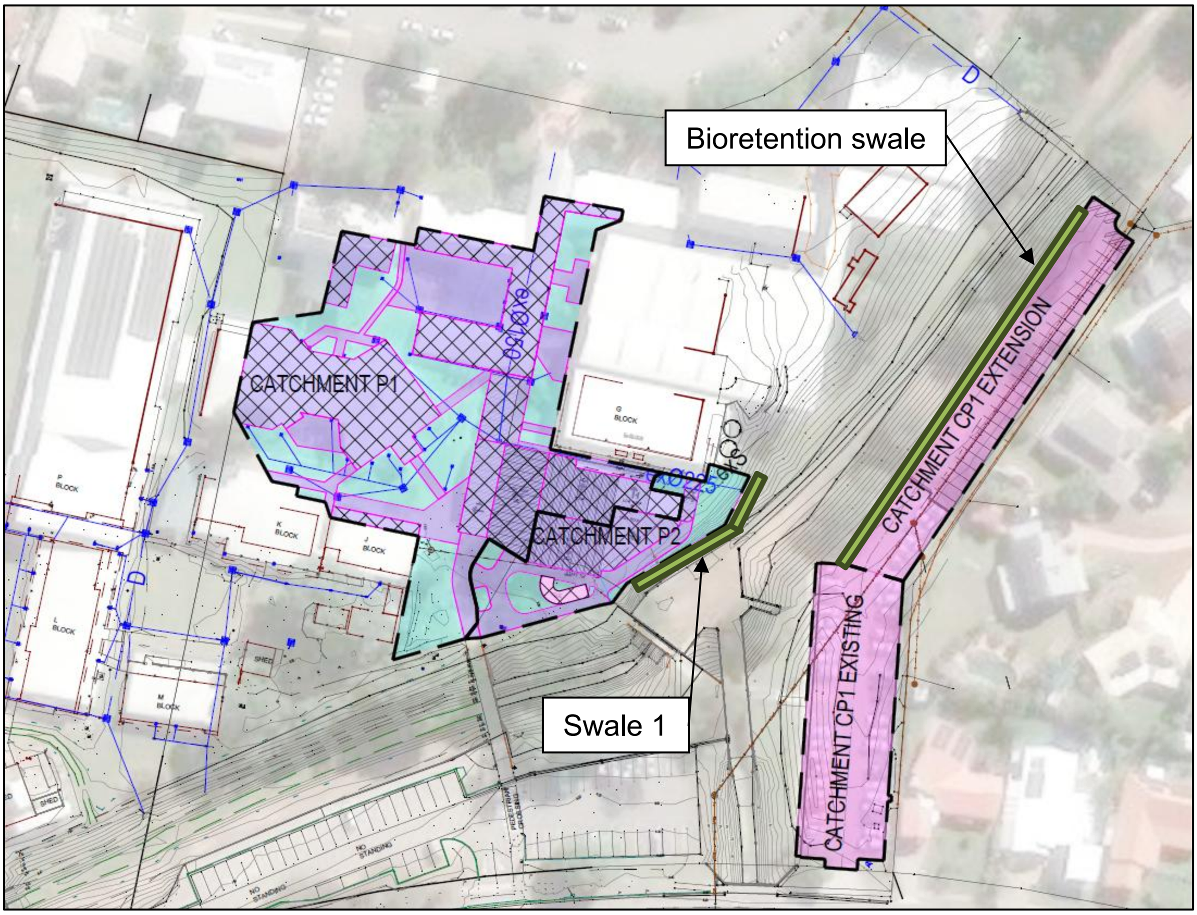

Figure 21. Post-development MUSIC catchments

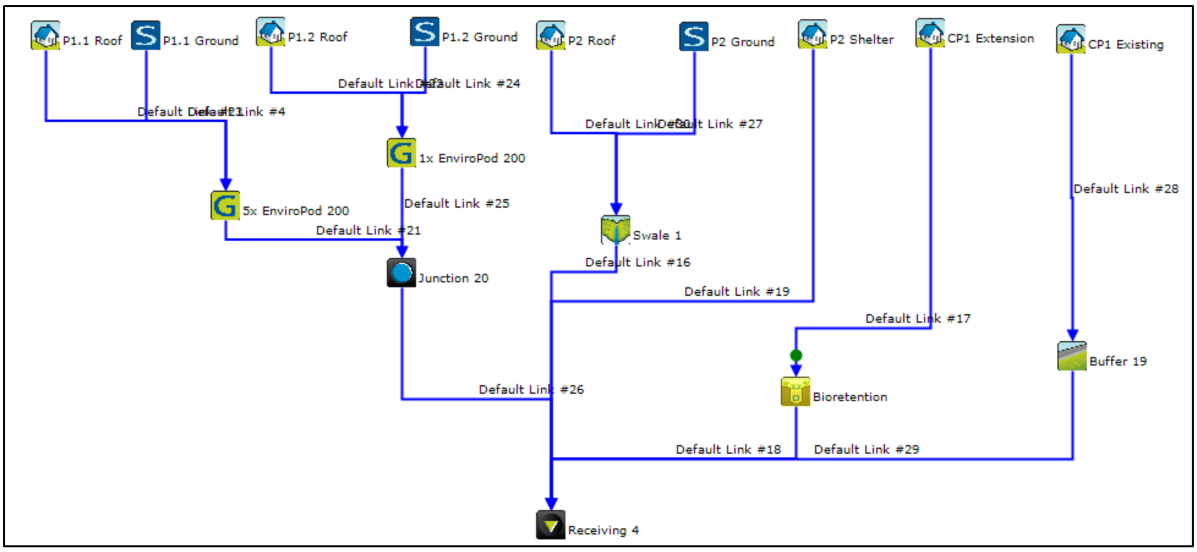

Figure 22. Post-development MUSIC treatment train

Table 7. MUSICX modelling results post-development

	Sources	Residual Load
Flow (ML/yr)	4.389	4.354
Total Suspended Solids (kg/yr)	358.6	89.12
Total Phosphorus (kg/yr)	1.052	0.6571
Total Nitrogen (kg/yr)	9.157	6.876
Gross Pollutants (kg/yr)	72.4	53.58

Based on the comparison of pre-development and post-development residual pollutant loads (kg/year) presented in **Table 6** and **Table 7**, the proposed development is not anticipated to result in a significant worsening of stormwater quality relative to existing site conditions.

While the developed scenario results in an increase in annual runoff volume, the implementation of targeted stormwater quality mitigation measures achieves net reductions in key pollutant loads when assessed on a residual load basis. Specifically, the post-development scenario achieves:

- a reduction in **Total Suspended Solids** from approximately **374kg/yr** to **89kg/yr**;
- a reduction in **Total Phosphorus** from approximately **1.03kg/yr** to **0.65kg/yr**;
- a reduction in **Total Nitrogen** from approximately **7.61kg/yr** to **6.88kg/yr**; and
- a reduction in **Gross Pollutants** from approximately **53.71kg/yr** to **53.58kg/yr**.

Assessed in terms of **absolute residual pollutant loads (kg/year)** rather than percentage reduction targets, the proposed development achieves a clear **no-worsening outcome** compared to the existing conditions.

This outcome is achieved through the implementation of the following mitigation measures:

- installation of **six (6) EnviroPod 200 units** fitted to existing stormwater pits to provide distributed pre-treatment of runoff;
- provision of a **vegetated swale along the retaining wall edge of Catchment P2** to provide flow attenuation and particulate removal prior to discharge to the downstream drainage network; and
- construction of a **95-metre-long bioretention swale** along the western boundary of the eastern carpark, providing enhanced treatment of nutrients and suspended solids. The bioretention swale has been modelled with a minimum saturation depth of 0.15m, 0.2m base width and 1 in 4 batters.

While the planning framework articulates stormwater quality objectives using percentage-based reduction targets, these targets are predicated on greenfield or redevelopment scenarios where comprehensive treatment systems can be readily integrated. In contrast, the subject site is predominantly existing and effectively fully developed (hence the need for the demolition and reconstruction of H Block), leaving little practical scope for retrofit stormwater infrastructure. As such, a performance-based residual load assessment represents evaluating stormwater quality outcomes in terms of kg/year as opposed to reduction targets has been deemed more appropriate for this site. The assessment confirms that the proposed development does not worsen stormwater quality compared to existing conditions and delivers measurable improvements in residual pollutant loads.

10.0 SUMMARY

This report provides an assessment of the existing site and proposed conditions with respects to water and wastewater servicing, traffic, stormwater (quantity and quality) and flood impacts and any mitigation measures which have been deemed required.

Key findings of the report are as follows:

Water servicing

- The assessment is based on a projected equivalent population of approximately **188 EP**, inclusive of students, staff, and the ancillary building at 9 Jonquil Crescent.
- Water demands were assessed for peak hour operating conditions and with an assumed **15 L/s fire flow**, consistent with Council servicing standards.
- Hydraulic modelling confirms that the existing trunk and reticulation water network servicing the site via Jonquil Crescent is capable of accommodating the proposed demand.
- Minimum pressure requirements are satisfied under both peak demand and fire flow conditions, and pipe velocities remain within acceptable limits.
- No upgrades to the existing Council water supply infrastructure are required to service the proposed development.

Wastewater servicing

- The assessment is based on a projected sewer equivalent population of approximately **291 EP**.
- Sewer flows were assessed at **5 times Average Dry Weather Flow (5 × ADWF)**, consistent with Council design and capacity criteria.
- Network modelling confirms that the existing DN150 reticulation sewers, downstream DN300 and DN375 trunk sewers, and Pump Station PS S4A operate within acceptable capacity limits.
- All sewer infrastructure assessed operates below the maximum allowable **75% full flow** criterion.
- The existing sewerage network has sufficient capacity to service the development, and no upgrades to Council sewer infrastructure are required.
- Minor realignment of the internal sewer network is proposed as part of the development works to avoid build over or near the internal sewer network and ensuring ease of future maintenance activities. Refer to **Figure 5** for proposed internal sewer realignment.

Traffic impact

- Across all assessed intersections and site accesses, **Levels of Service remain A or B**, with Degrees of Saturation well below capacity thresholds in both AM and PM peaks.
- The development results in **minimal increases in delay and saturation**, with no significant deterioration in overall network performance.
- No change is proposed with regards to turn warrants, however, changes to parking restrictions are proposed to ensure required minimum sight distances are available. All proposed signage and linemarking is summarised in **Appendix C** of the Traffic Impact Assessment.
- In addition to the above, it is recommended that the tree between Site access 1 and Site access 2 is replaced with a more suitable species.
- Modifications to the existing staff parking off Jonquil Crescent is required to ensure compliance with AS2890.1 and NCC. As part of the modifications an accessible parking space is required and due to the number of spaces within the blind aisle, a turnaround provision is also required.

Stormwater quantity

- It is proposed that the existing 225mm dia. pipe between H Block and G Block is replaced with a 375mm dia. RCP.
- A swale at the top of the proposed retaining wall is proposed to assist with stormwater quality. The swale must have a minimum 0.4m wide base, a depth of 0.3m (including 0.15m freeboard) and 1 in 2 batters.

Flood modelling

- The proposed development does not impede the flow of flood waters through the site or worsen flood flows external to the site.
- Any changes to the depth or velocity of flood waters are contained within the site.
- It does not directly, indirectly or cumulatively worsen flood characteristics outside the development site, having regard to increased scour and erosion.
- 1% AEP flood levels are provided within the report and **Table 2** of this report so that floor levels can be designed with appropriate freeboard.

Stormwater quality

- The stormwater quality assessment has been done based on a performance-based residual load assessment, evaluating stormwater quality outcomes in terms of **kg/year** as opposed to reduction targets due to the site being predominantly existing and effectively fully developed (hence the demolition and reconstruction of H Block) leaving little practical scope for retrofit stormwater infrastructure.

- The report has found that provided that the mitigation measures outlined in **Section 9.3.2** of the report are implemented the proposed development does not worsen stormwater quality compared to existing conditions.

APPENDIX A

SITE PLANS

APPENDIX B

WATER AND SEWER REPORT

APPENDIX C

TRAFFIC IMPACT ASSESSMENT REPORT

APPENDIX D

HYDROLOGICAL AND HYDRAULIC REPORTS

APPENDIX E

FLOOD MODELLING REPORT