

Attachment F

Stormwater Management Plan

Precinct-Wide Flood Modelling, Stormwater Design and Site Grading

FINAL

Bundaberg Health and Enterprise Precinct

Prepared for CPB / 14 January 2026

236019

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1.0 Introduction

TTW (NSW) Pty Ltd has been engaged by the HRP NBH project on behalf of Bundaberg Regional Council (BRC) to complete a hydraulic and hydrological assessment of the Bundaberg Health and Enterprise Precinct. This assessment aims to test how the proposed precinct development can reduce flood behaviour across the Precinct and meet hydraulic performance criteria, flow neutrality, flood immunity for lots, and grading constructability.

This project involves:

- Development of a precinct-scale hydraulic model,
- Modelling of existing and post-development flood scenarios,
- Concept stormwater network design and site grading across the precinct
- Detention basin sizing across the precinct to manage flows and meet planning requirements,
- Site grading design to support the stormwater and flood performance objectives and lift developable areas above the flood planning levels.

Given the risks associated with the Saltwater Creek catchment, the full range of design storm events (i.e. from 63% AEP to 1% AEP, including climate change scenarios) are to be assessed for all relevant durations to ensure a comprehensive understanding of flood and stormwater impacts under current and future conditions.

This report summarises the existing flow behaviour within the precinct and at the key assessment areas.

1.1 Precinct Description

The Bundaberg Health and Enterprise Precinct is an integrated health and medical precinct, part of the new Bundaberg Hospital project, designed to provide a hub for healthcare services, medical education, and commercial activities in the region. The precinct is expected to generate significant local employment and economic growth through the construction and operation of new hospital facilities and supporting commercial ventures.

Key aspects of the precinct are summarised below.

- New Bundaberg Hospital:
- The precinct is centered around a new, state-of-the-art Bundaberg Hospital, enhancing regional healthcare services and patient care.
- Integrated Health Services:
- It aims to bring together a range of health and medical services, potentially including medical education and training, to create a comprehensive health hub.
- Economic and Employment Growth:
- The project is a significant investment in the Wide Bay-Burnett region, with the potential to create a large number of local jobs during construction and ongoing operation.
- Supporting Commercial Development:
- The precinct is also designed to include commercial and enterprise spaces that will complement the health services, further boosting local economic activity.
- Future-Focused Vision:
- The concept focuses on creating a sustainable, integrated environment that fosters health innovation, education, and community well-being.

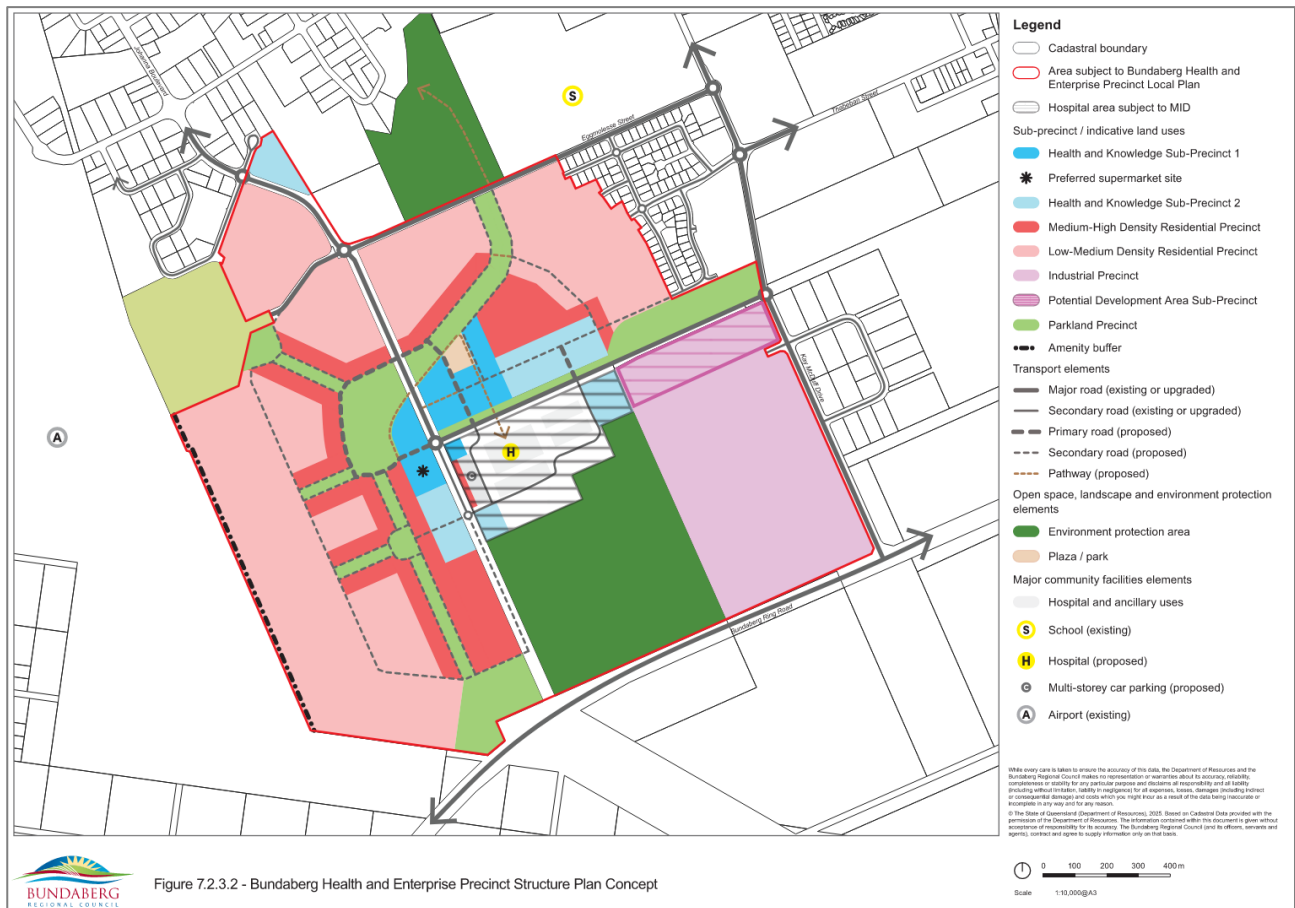


Figure 1: Preliminary Concept Precinct Plan

1.2 Site Topography

The precinct area is approximately 268 ha and has a gradual fall from west to east. The elevation at the western boundary is approximately 29.50m AHD and approximately 27.50m AHD at the eastern boundary, with an average grade of approximately 1%. The topography across the precinct is shown in Figure 2.

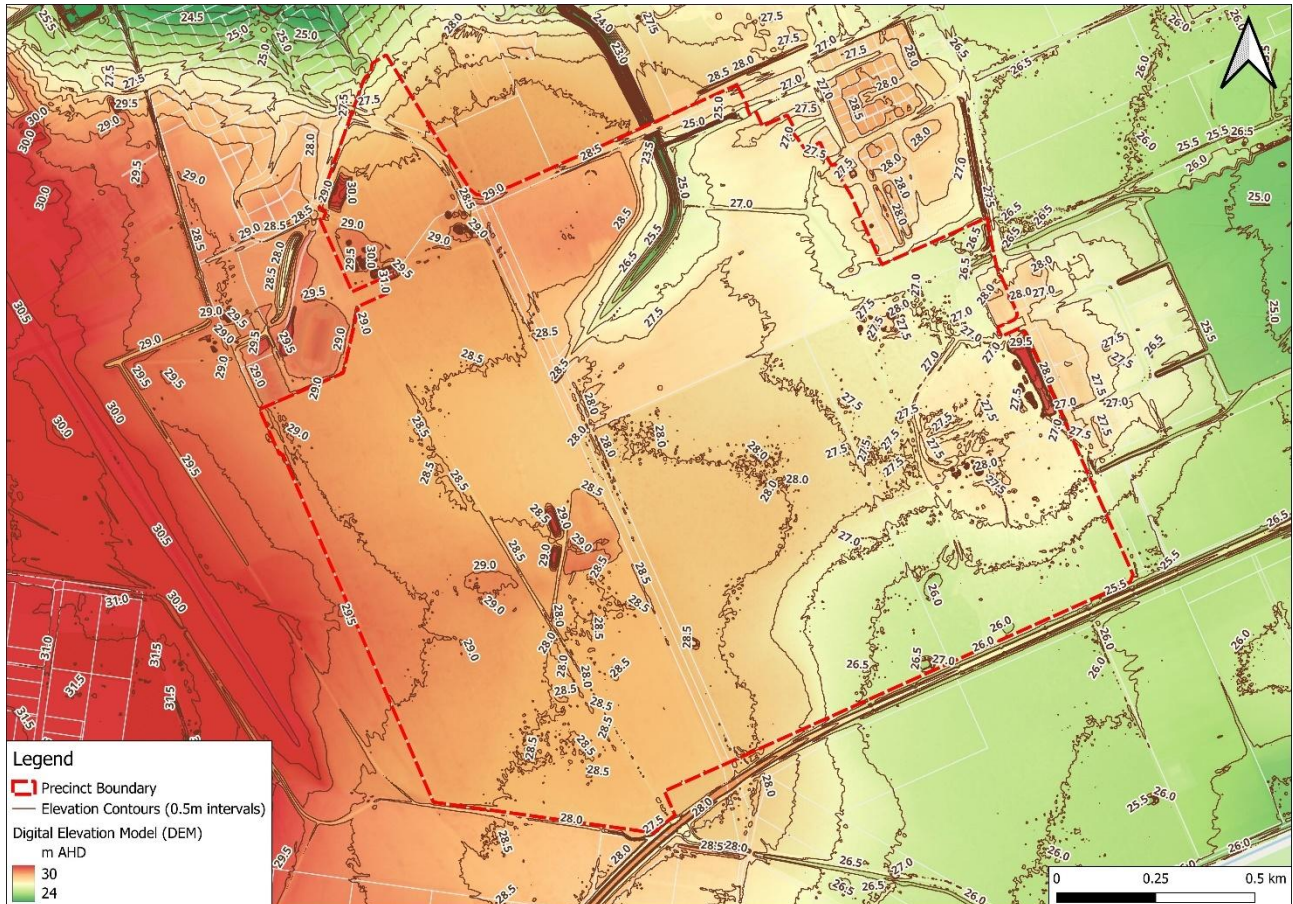


Figure 2: Topography and elevations across the precinct

1.3 Performance Criteria

TTW met with Bundaberg Regional Council on 28th July 2025 to confirm the performance criteria and objectives for the precinct development. BRC provided a detailed response on 31st July 2025. The key criteria are outlined below.

1.3.1 No Actionable Nuisance

The overall objective is to ensure there is no *actionable nuisance* as a result of the precinct development. To assess this, flood behaviour at five key locations will be assessed. This includes four discharge points around the precinct boundary:

- **Location A:** to the north, downstream of Eggmolesse Street
- **Location B:** to the east, upstream of Kay McDuff Drive
- **Location C:** to the southwest upstream of Bundaberg Ring Road.
- **Location D:** to the southeast upstream of Bundaberg Ring Road (i.e. where the hospital site discharges).

Actionable nuisance in this context means a substantial and unreasonable damage or loss due to flooding.

1.3.2 Specific Requirements for Saltwater Creek

Bundaberg Regional Council specifically requested that an additional residential area downstream of the precinct along Saltwater Creek is included in the hydraulic impact assessment: *“This section of the creek presents a high risk of above floor level flooding and is considered a ‘residential hotspot’. As such, any increase in flood impact resulting from the proposed development will be considered unacceptable.”*

- **Location E:** Between Branyan Street and Walker Street (spot Location taken at Puie Street)
- Impact assessment must demonstrate no increase in peak flow, level, velocity, and duration of above-floor inundation.

Refer to Figure 3 below for the location of the five key assessment locations.

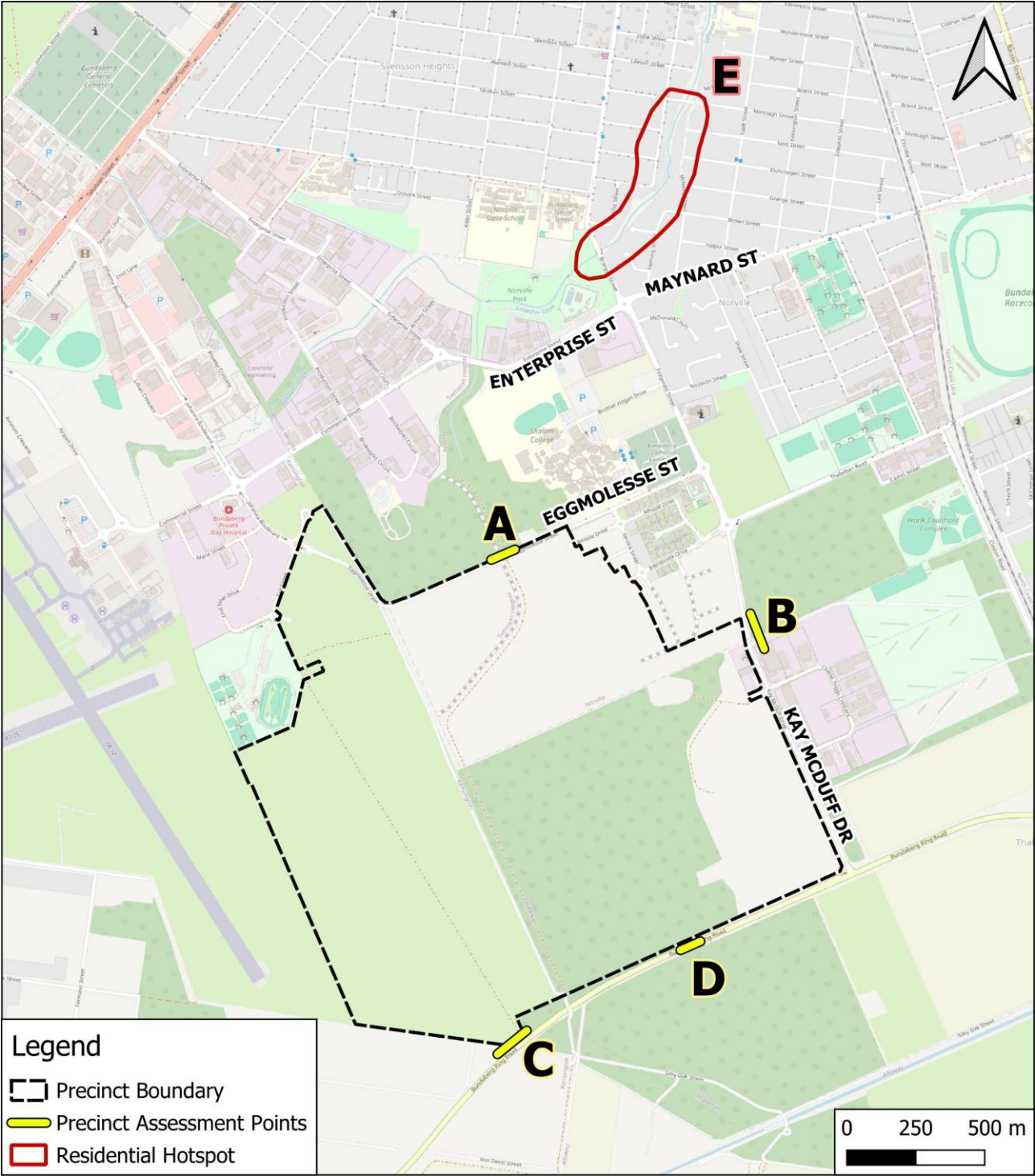


Figure 3: Key locations of assessment across the precinct

2.0 TUFLOW Existing Base-Case Flooding

2.1 Bundaberg Citywide Overland Flow Study

Council provided TTW with an updated version of the Bundaberg Citywide Overland Flow Study TUFLOW model (.tcf version 003) with guidance notes on the parameters, storm events, durations and temporal patterns to be used for the assessment. No change has been made to the parameters of Council's flood model.

A table of the storm durations assessed for each flood event is shown below in Table 1.

Table 1: Storms and events modelled

Flood Event	Storm Durations (minutes)
1% AEP +CC	30, 60, 90, 120, 270, 360
1% AEP	30, 60, 90, 120, 270, 360
2% AEP	30, 60, 90, 120, 270, 360
5% AEP	30, 60, 120, 180, 270, 360
10% AEP	30, 60, 120, 180, 270, 360
18.1% AEP	30, 90, 180, 270, 360, 540
39.4% AEP	30, 90, 180, 270, 360, 540
63.2% AEP	30, 90, 180, 270, 360, 540

2.2 Critical Duration Assessment

No flood results files were provided by Council, so the full set of events and durations were run to obtain the complete set of result. A critical duration assessment was then completed for each flood event to understand the flood behaviour across the precinct and wider catchment, and to determine the critical storm duration at the five key locations across the precinct and Saltwater Creek, as shown in Table 2. The critical duration assessment for the 1% AEP + Climate Change (+CC) is presented in Figure 4.

Table 2: Critical storm duration (mins) for each flood event at each key location

Flood Event	Location A	Location B	Location C	Location D	Location E
1% AEP +CC	270	270	270	270	360
1% AEP	270	270	270	270	360
2% AEP	270	270	270	270	360
5% AEP	270	270	360	360	360
10% AEP	180	270	180	360	180
18.1% AEP	360	360	360	360	180
39.4% AEP	180	360	360	360	180
63.2% AEP	180	360	360	360	180

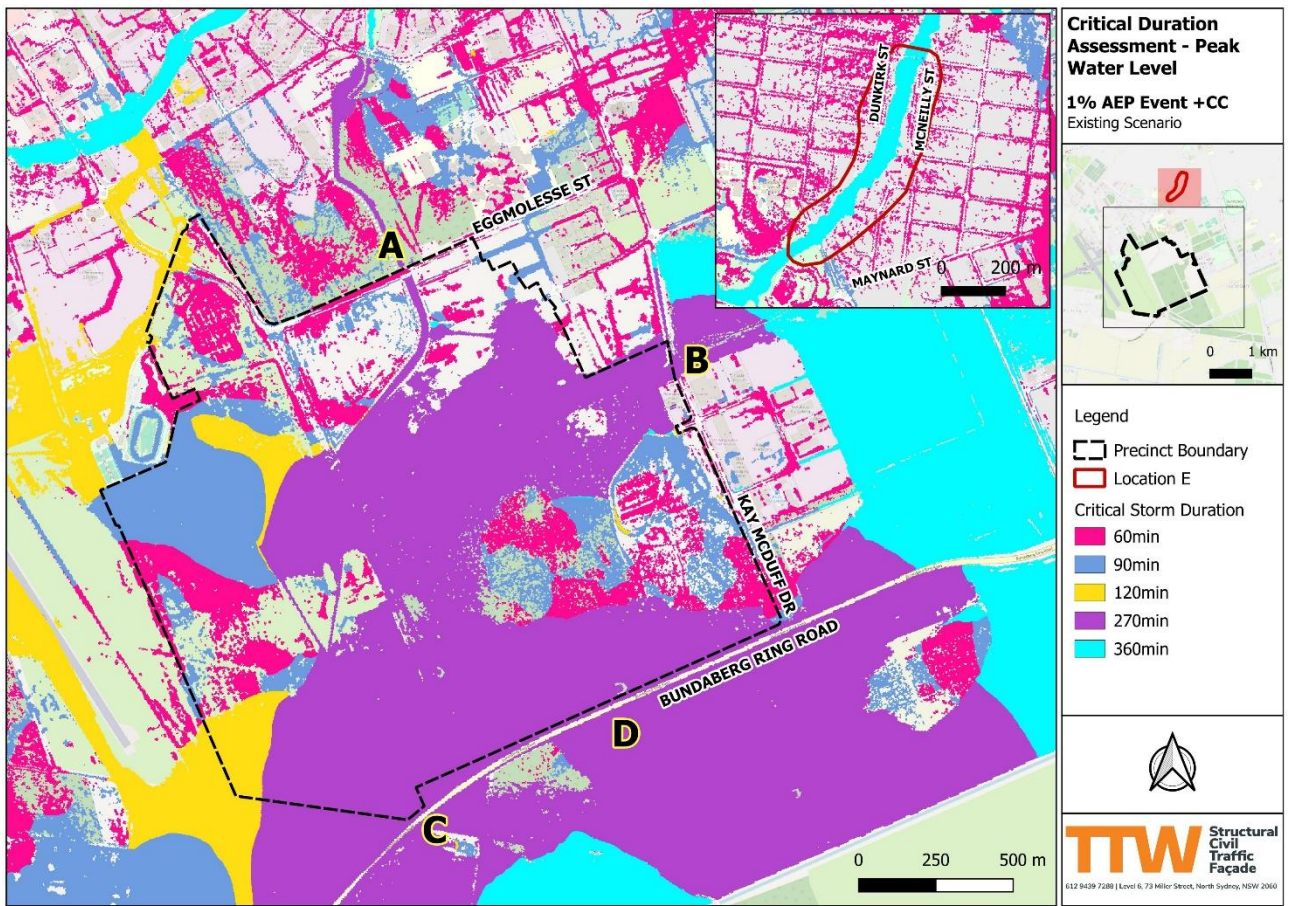


Figure 4: 1% AEP +CC critical duration assessment

2.3 Summary of Peak Flows and Levels

The design flood event is the 1% AEP +CC. 1% AEP +CC flood depths and levels, flood velocity, and flood hazard maps are shown in Figure 5, Figure 6 and Figure 7 respectively.

Flood results show that overland flow is generated from the west of the precinct with a ridge running west to east through the precinct that splits the overland flow into two separate flow paths. The southern overland flow discharges via multiple culverts under the Bundaberg Ring Road. The central overland flow path primarily discharges to the east (Point B) at Kay McDuff Drive and flows to the downstream basins at the Bundaberg Clay Target Club. This central overland flow is also partly conveyed by the Edenbrook Drain to the north (Point A) and into Saltwater Creek further downstream.

The overland flow is shallow sheet flow (typically less than 300mm) with greater depths adjacent to flow controls such as culverts or raised embankments/roads. Flow velocities are low (typically less than 0.5m/s) across the site due to the rough terrain and shallow grade topography. Flood hazard is also low across the precinct with mostly H1 and H2 hazard except for deeper ponding at culverts, channels and trapped ponding areas.

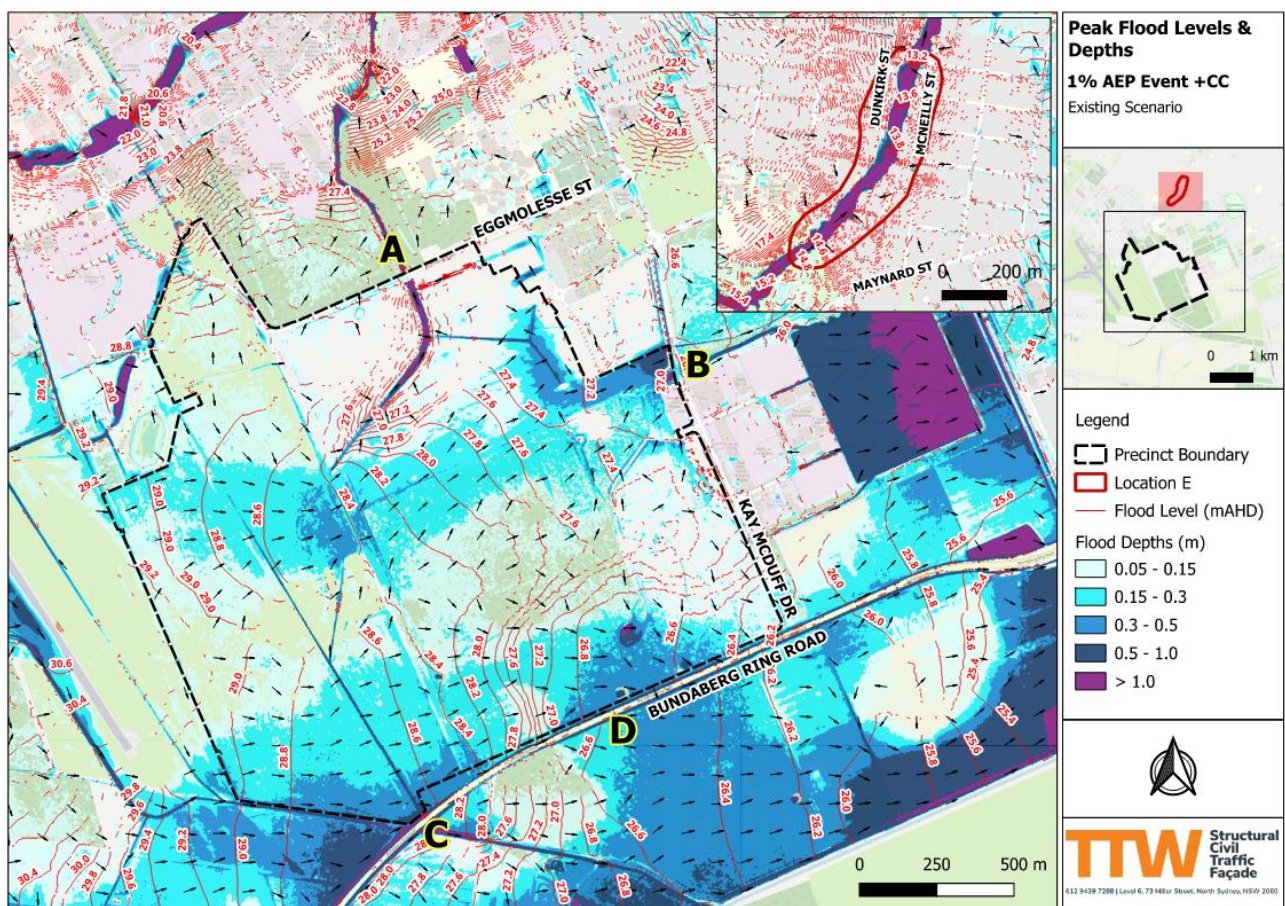


Figure 5: Existing 1% AEP +CC flood levels and depths

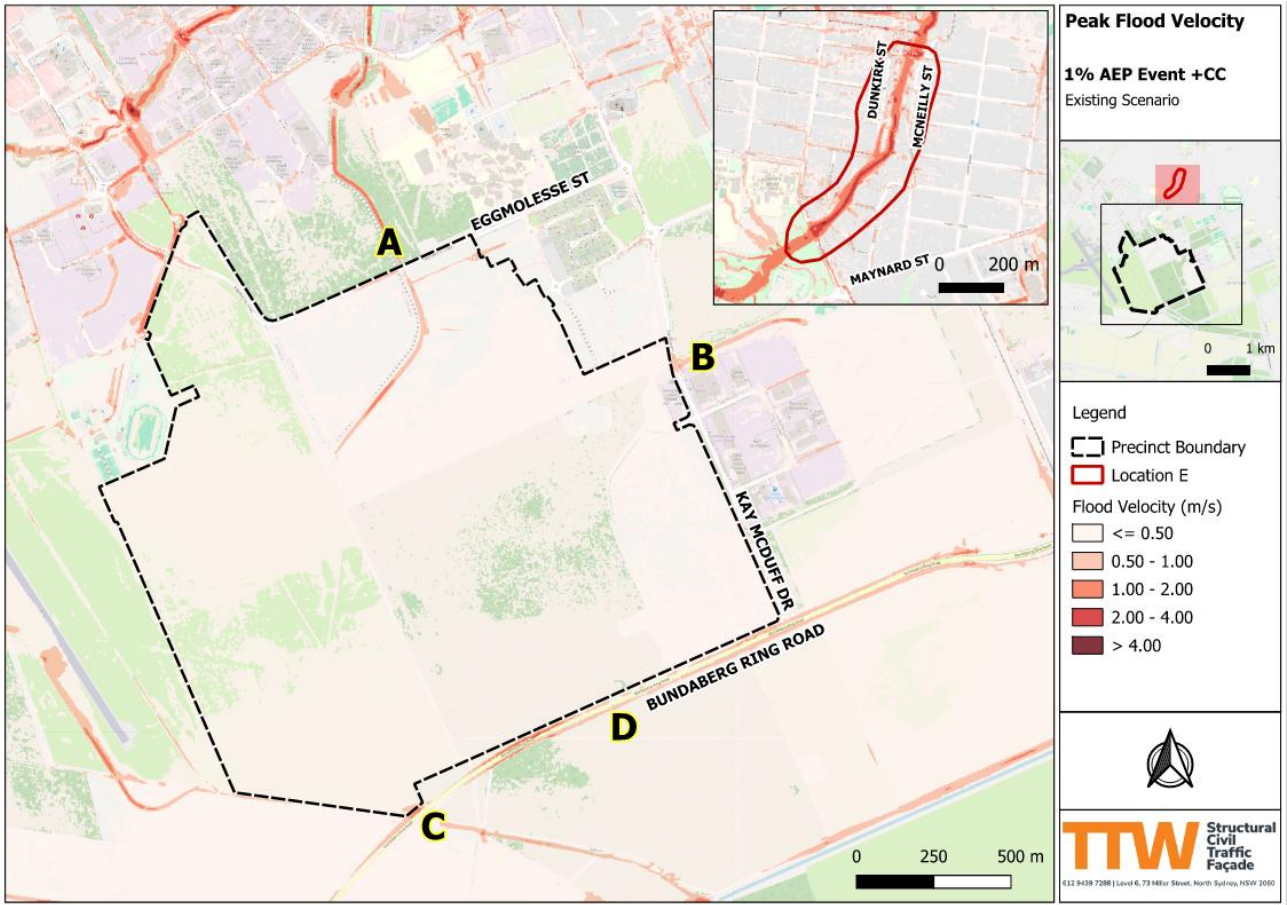


Figure 6: Existing 1% AEP +CC flood velocity

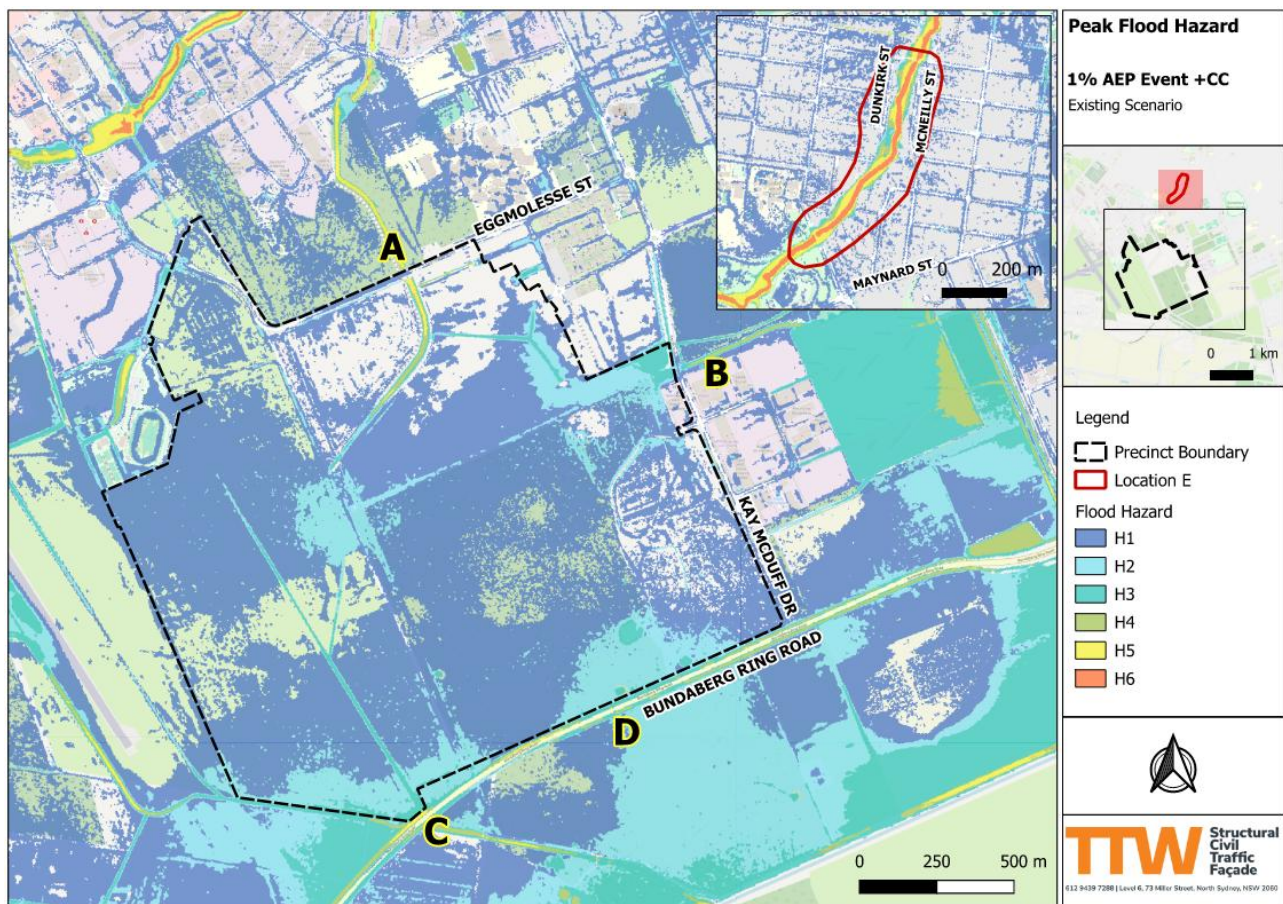


Figure 7: Existing 1% AEP +CC flood hazard

Peak flow hydrographs for the 1% AEP +CC for various durations at each of the key locations are shown below in Figure 8, Figure 9, Figure 10, Figure 11, Figure 12 and Figure 13. The critical duration in the 1% AEP +CC event is generally the 270-minute storm with the exception of the residential area adjacent to Saltwater Creek, with the 360-minute creating a slightly higher peak. However, due to the shape of the hydrograph and the change in peak flow over time, the 270-minute storm causes a longer period of peak flow compared to the 360-minute duration storm and is therefore deemed to be the critical storm duration for this event.

Peak flood flows, peak flood levels, and peak flood velocities for all storm events at the key locations are shown in Table 3, Table 4 and Table 5, respectively.

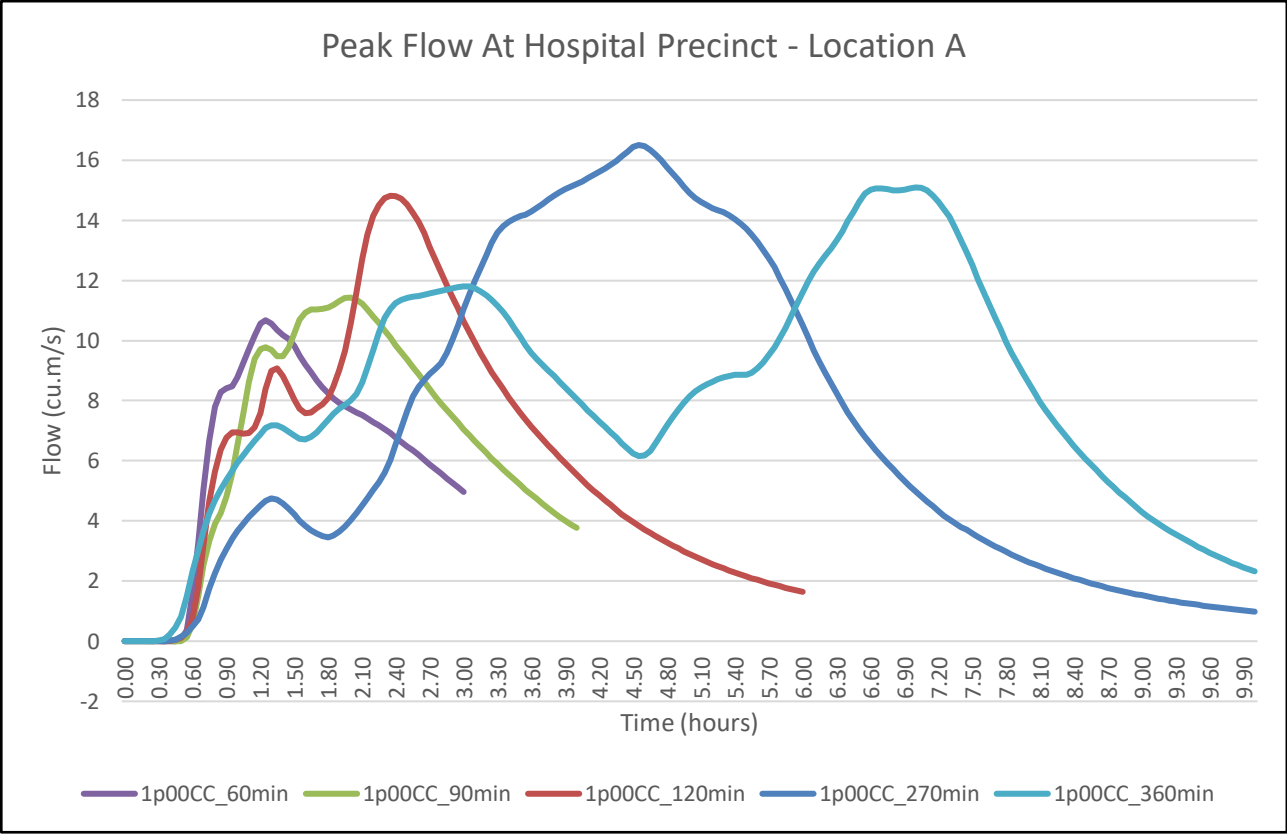


Figure 8: 1% AEP +CC flow hydrographs for different storm durations at Location A

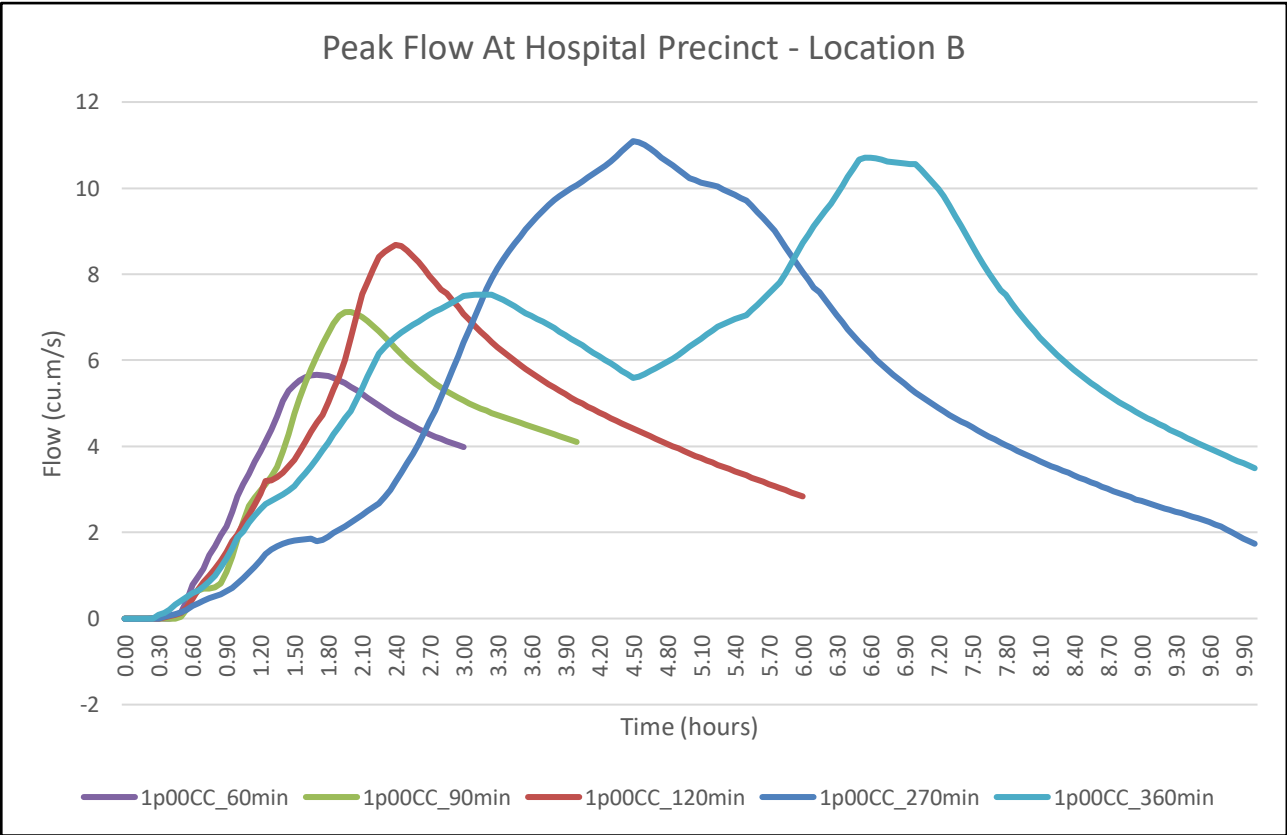


Figure 9: 1% AEP +CC flow hydrographs for different storm durations at Location B

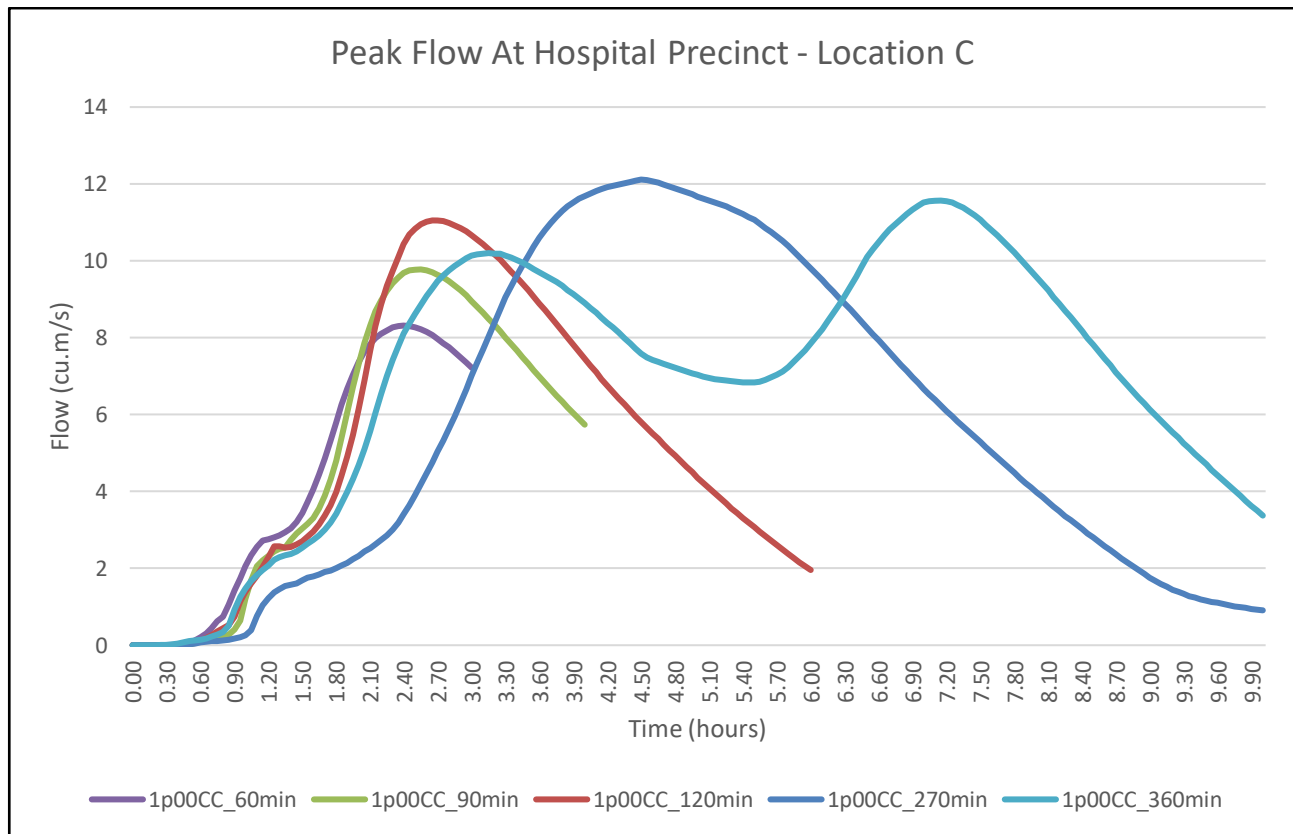


Figure 10: 1% AEP +CC flow hydrographs for different storm durations at Location C

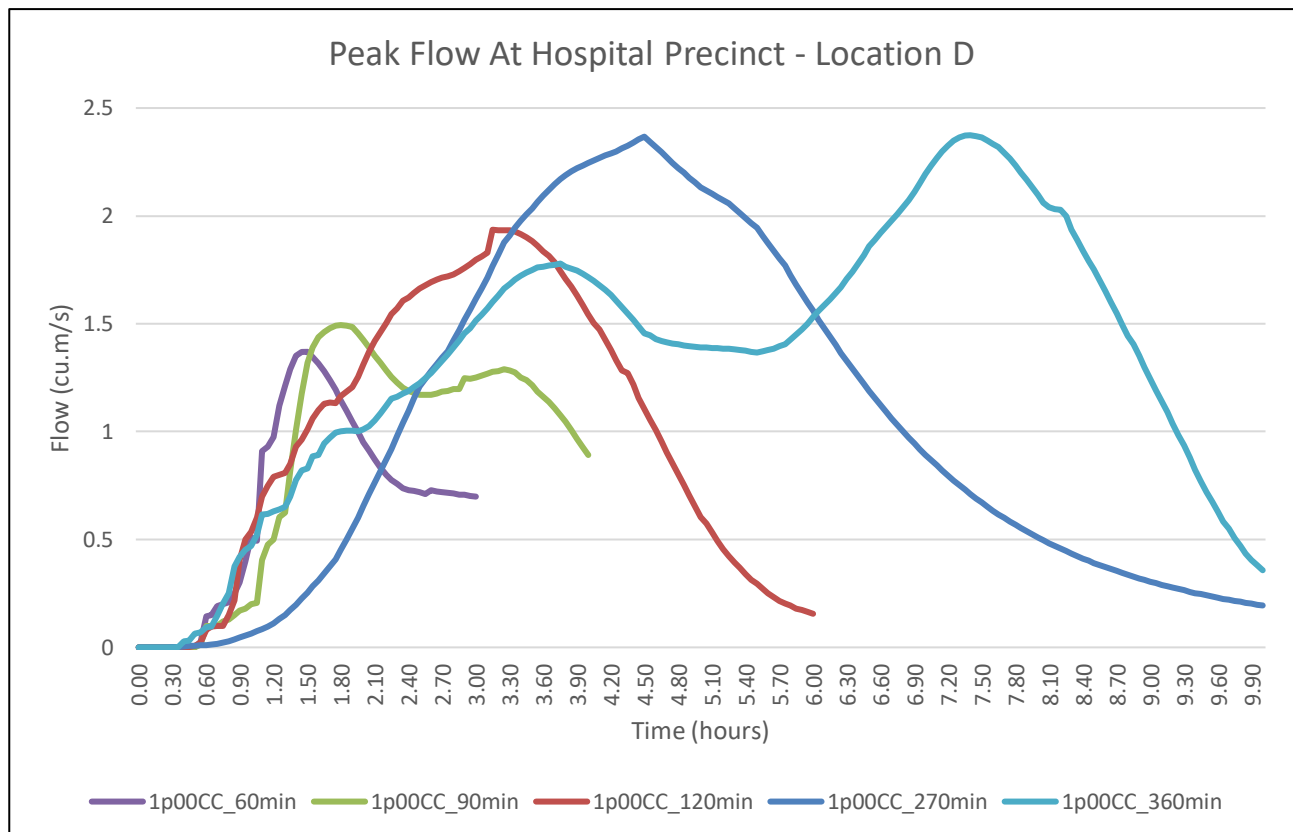


Figure 11: 1% AEP +CC flow hydrographs for different storm durations at Location D

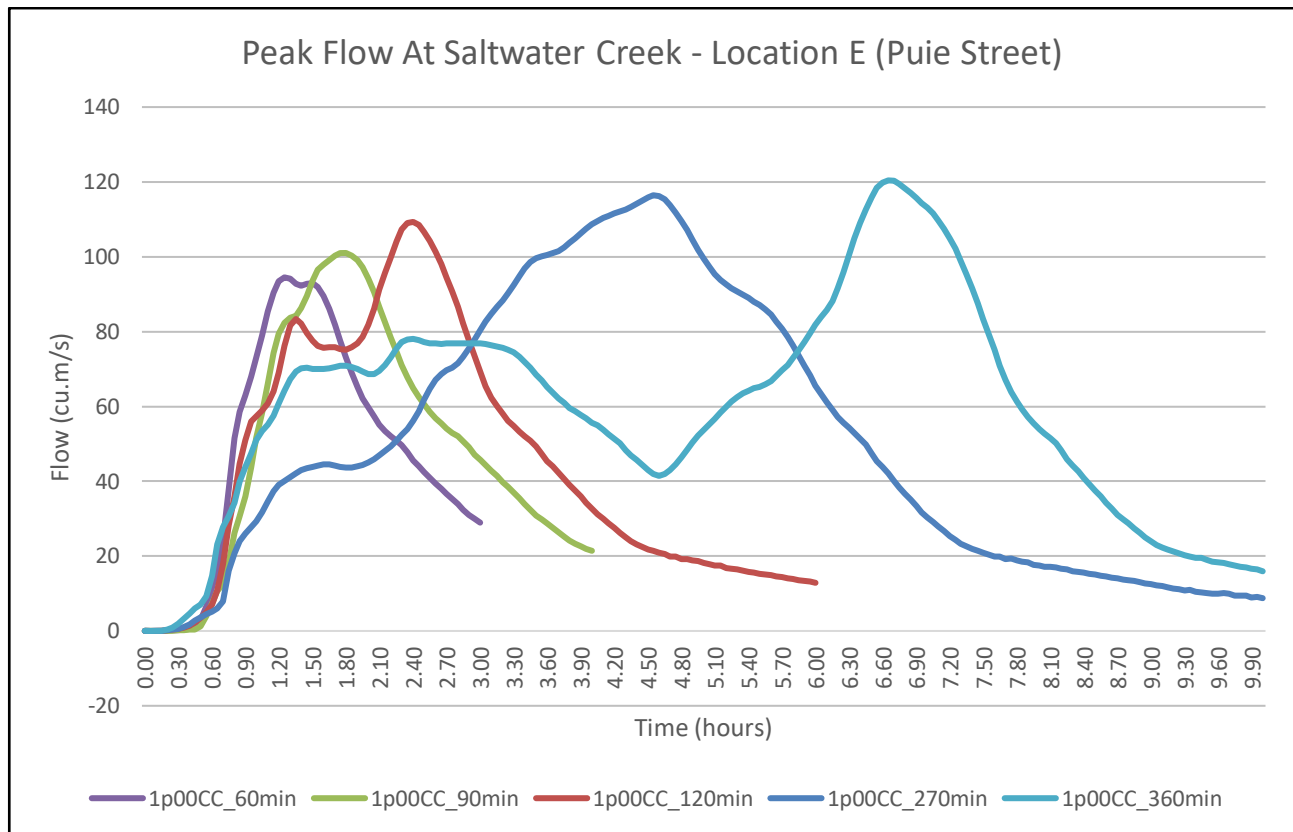


Figure 12: 1% AEP +CC flow hydrographs for different storm durations at Location E

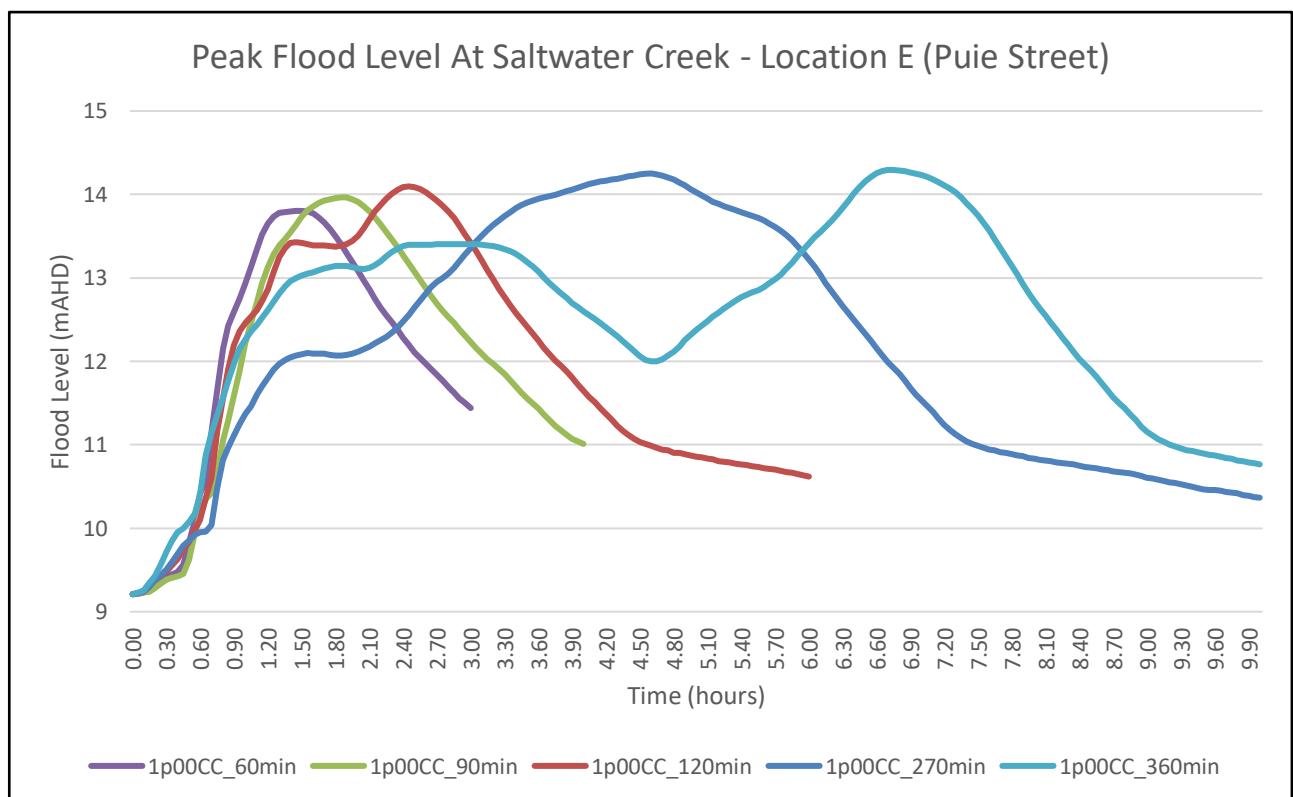


Figure 13: 1% AEP +CC flood level hydrographs for different storm durations at Location E

Table 3: Existing peak flows at key assessment locations

Peak Flows Across Precinct (cu.m/s)					
Flood Event	Location A	Location B	Location C	Location D	Location E
1% AEP +CC	16.5	11.1	12.1	2.9	120.4
1% AEP	13.5	9.2	10.9	2.3	94.1
2% AEP	11.5	7.7	9.7	1.8	76.3
5% AEP	8.6	6.2	7.2	1.1	69.4
10% AEP	7.4	5.0	6.3	1.0	63.3
18.1% AEP	6.4	4.1	5.5	0.9	59.1
39.4% AEP	4.4	2.6	3.8	0.9	46.7
63.2% AEP	2.3	1.6	2.1	0.8	22.3

Table 4: Existing peak flood levels at key assessment locations

Peak Flood Levels Across Precinct (mAHD)					
Flood Event	Location A	Location B	Location C	Location D	Location E
1% AEP +CC	25.2	26.6	28.6	26.7	14.3
1% AEP	25.1	26.5	28.6	26.6	13.9
2% AEP	25.0	26.5	28.5	26.6	13.5
5% AEP	24.8	26.4	28.4	26.5	13.1
10% AEP	24.7	26.3	28.4	26.4	13.0
18.1% AEP	24.6	26.2	28.3	26.4	12.9
39.4% AEP	24.5	26.1	28.2	26.3	12.2
63.2% AEP	24.2	26.1	28.2	26.2	11.2

Table 5: Existing peak flood velocities at key assessment locations

Peak Flood Velocities Across Precinct (m/s)					
Flood Event	Location A	Location B	Location C	Location D	Location E
1% AEP +CC	0.6	0.7	0.5	0.9	2.3
1% AEP	0.5	0.7	0.5	0.7	2.2
2% AEP	0.5	0.7	0.4	0.6	2.2
5% AEP	0.4	0.6	0.3	0.5	2.1
10% AEP	0.4	0.6	0.2	0.5	2.1
18.1% AEP	0.3	0.5	0.2	0.5	2.1
39.4% AEP	0.3	0.4	0.1	0.5	2.1
63.2% AEP	0.2	0.3	0.1	0.5	2.0

3.0 DRAINS Pre-Development Calibration

3.1 Introduction

This section outlines the hydrologic and hydraulic modelling methodology adopted to assess the stormwater performance of the proposed Precinct Plan. The modelling approach involved the use of both TUFLOW (2D hydraulic modelling) and DRAINS (1D hydrologic/hydraulic modelling) to simulate pre-development and post-development conditions across the site.

Given the complex nature of surface flows within the precinct – characterised by sheet flow, ponding, and multiple overland flow paths – TUFLOW was used to accurately represent surface topography and flow routing, while DRAINS required manual representation of these features through nodes, basins, and weirs. To ensure consistency and reliability, the DRAINS model was calibrated against TUFLOW for the pre-development scenario, confirming that the DRAINS and TUFLOW model results are comparable and sufficient for the purposes of testing the proposed Precinct Plan stormwater strategy, grading, and basin design before final confirmation in the TUFLOW modelling.

Key comparison points were established in consultation with Bundaberg Regional Council, focusing on five critical locations around and downstream of the precinct. These locations were selected to assess potential flood impacts and ensure the development does not result in actionable nuisance. For the DRAINS model, Locations A, B, and C were prioritised, with overflow constraints from Location C to D incorporated into the post-development design.

Hydrologic assumptions included catchment delineation based on outlet points, time of concentration calculations using Friend's equation, and surface roughness values derived from QUDM. Hydraulic assumptions involved the representation of ponding and overflow structures using basins and weirs, informed by TUFLOW outputs and QGIS contour data.

3.2 Methodology

3.3 Calibration Approach

Within the precinct site, multiple catchments exist with significant flooding and ponding observed. Surface water sheet flow disperses into several overland flow paths, which ultimately contribute to separate sub-catchments (refer to 1% AEP+CC flood levels and depths in Figure 5). TUFLOW, as a 2D hydraulic modelling software, utilises existing surface TINs to accurately represent surface topography and predict flow directions. In contrast, DRAINS is a 1D model that does not simulate surface topography or overland flow paths directly; these must be manually represented in the model as links and nodes. Accordingly, calibration against the TUFLOW model is required to ensure an accurate representation of pre-development conditions.

The results from the Pre-Development DRAINS model will then be used to ensure flows from the Post-Development DRAINS model does not exceed the Pre-Development flows. The use of DRAINS modelling to preliminary sizing up the detention measures required for the precinct prior to testing in TUFLOW model significantly reduces the TUFLOW modelling time required (i.e. DRAINS simulations are much faster to complete than TUFLOW simulations).

The scenarios tested are consistent with the TUFLOW model, for which Council provided TTW with an updated version of the Bundaberg Citywide Overland Flow Study TUFLOW model (.tcf version 003). The storms and events modelled are outlined in Table 1.

3.4 Modelling Assumptions

3.4.1 Catchment Delineation/Time of Concentration

Refer to Figure 14 below for the Pre-Development Catchment Plan, where catchments have been delineated based on their respective outlet points. Notably, Point C has a large upstream catchment that includes the airport and existing sites to the west of the precinct.

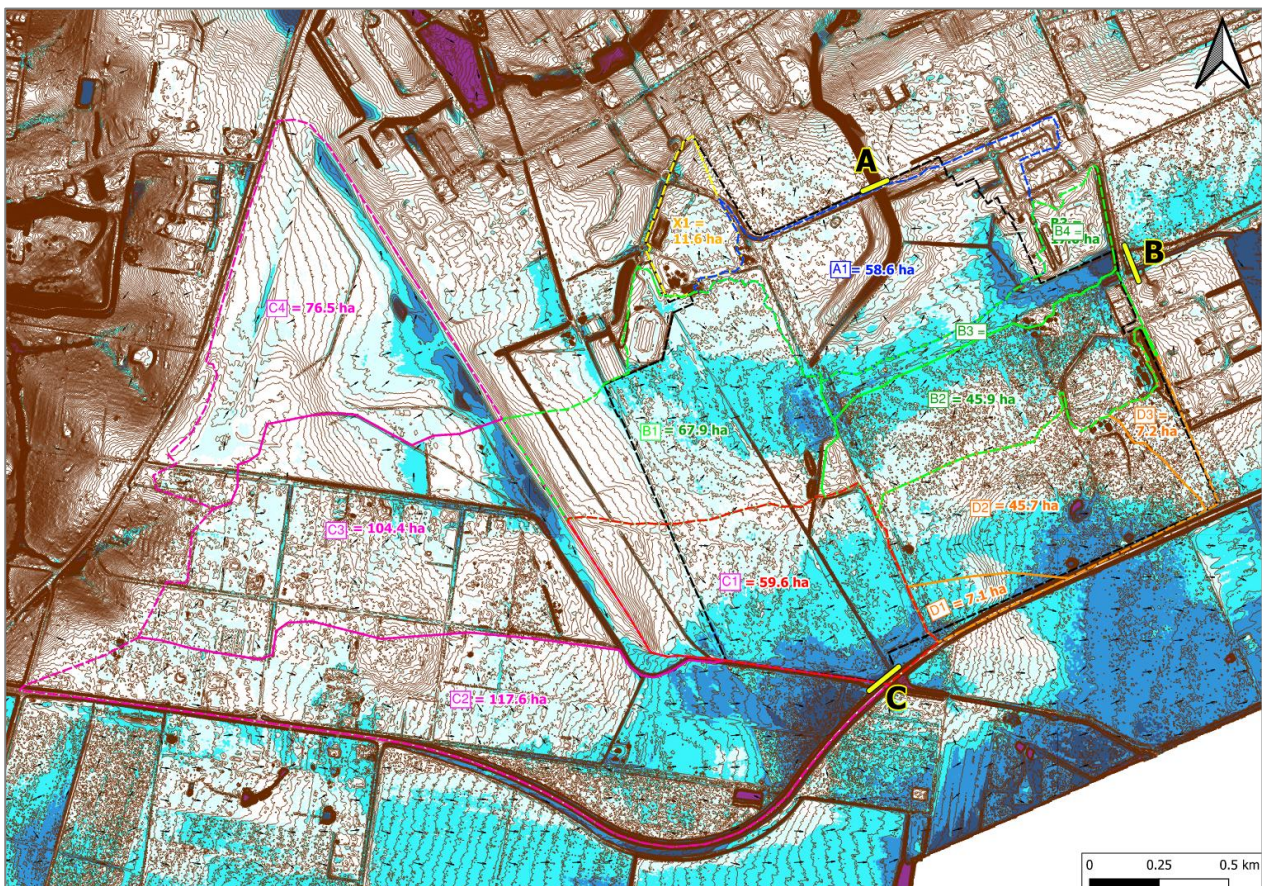


Figure 14: Pre-Development Catchment Plan

Further, please refer to Appendix B - Pre-Development Catchment Plan for more detailed catchment plan information, including basin locations and overflow paths/levels.

TUFLOW simulates sheet flow within the model; therefore, for the pre-development DRAINS model, the Time of Concentration (TOC) for each catchment was calculated using the Friend's equation to estimate overland sheet flow travel time. The Horton's surface roughness factor (n^*) was assigned a value of 0.1 for the majority of catchments, reflecting predominately short grass cover, based on Table 4.6.5 – Surface Roughness or Retardance Factors from QUDM.

Exceptions include:

- Catchments C1, where $n^* = 0.2$ for the 1% + CC, 1%, 2%, and 5% AEP storm events due to significant flooding/ponding on site during these events.
- Catchments C2 and C3 are more developed than the other catchments. Accordingly, an n^* value of 0.01 is used for impervious areas (reflecting concrete), and 0.07 for pervious areas (reflecting sparse vegetation, as most of the catchment consists of dirt/gravel roads and open spaces).
- For the 1% AEP + CC event in Catchment C2, an n^* value of 0.10 is applied to the pervious areas to reflect increased ponding and attenuation.
- Catchments B2 and B3, where $n^* = 0.2$ for all storm events, reflecting widespread ponding.
- Catchment B1, $n^* = 0.15$ for all storm events reflecting localised ponding.

3.4.2 Losses

Losses for Hydrologic Modelling is consistent with the guidance provided in **Section 5.6.4, Table 12** of the BRC Stormwater Management (see Figure 15 below).

Surface Type	Initial Loss	Continuing Loss
Pervious	25 mm	1.5 mm/h
Impervious	1 mm	0 mm/h

Figure 15: Calibrated Hydrological Model Rainfall Losses:

The rainfall losses applied in the TUFLOW model are based on surface type and typically range up to 15 mm. In contrast, the DRAINS model applies a uniform initial loss of 25 mm, consistent with Council's recommended values. This difference is considered acceptable because the two models represent hydrologic processes at different levels of detail and scale.

TUFLOW is a distributed 2D hydraulic model that explicitly represents topography, land surface roughness, and infiltration behaviour across the catchment. As a result, it tends to generate greater surface runoff and accounts for additional hydrologic losses through overland flow interaction and local storage effects within the model domain. The slightly higher initial loss adopted in the DRAINS model (an additional 10 mm) effectively compensates for these additional physical processes not directly represented in the lumped hydrologic approach.

Therefore, it is reasonable for the loss parameters to differ between the two models while still producing consistent and representative peak flow outcomes. The key consideration is that both models are calibrated to reflect expected catchment responses under pre-development conditions.

3.4.3 Roughness (Mannings Values)

The Manning's n value used for overland flow paths in DRAINS reflects the existing ground cover and surface characteristics. In TUFLOW, Manning's n values vary depending on the exact location and water depth; for this study, values representing shallow sheet flow and the majority of the catchment have been applied. Table 6 below summarises the Manning's n values used for the overland flow routes in each catchment.

Table 6: Mannings Values

Catchment	Land Use Description	Manning's Value
C1	Open (Rural)	0.09
C2	Open (Urban)	0.06
C3	Open (Urban)	0.06
C4	Open (Urban)	0.06
B1	Open (Rural)	0.09
B2	Open (Rural)	0.09
B3	Open (Rural)	0.09
B4	Open (Rural)	0.09
A1	Low Density Vegetation (Rural)	0.10

3.4.4 Climate Change Factor for 1% AEP + CC

Since DRAINS only allows a single rainfall multiplier to be applied across all storm events, the climate change (CC) factor was derived from the TUFLOW model by comparing the 1% AEP baseline and 1% AEP + CC design rainfalls. The average increase in rainfall depth was calculated to be 19.7%; therefore, a rainfall multiplier of 1.197 was applied in DRAINS for the 1% AEP + CC event.

3.4.5 Representation of Ponding and Overflows

As noted above, TUFLOW is a 2D hydraulic modelling software that utilises existing surface DEM's to accurately represent surface topography and predict flow directions. In contrast, DRAINS is a 1D model that does not directly simulate surface topography or overland flow paths; these must instead be manually represented as 1D linear paths. Accordingly, within the pre-development DRAINS model, basins were used to simulate ponding during storm events, with overflows and weirs defined based on contours extracted from the QGIS model. The Pre-Development Catchment Plan in Appendix B highlights the location of these basins and their corresponding overflow/weir structures.

For example, within the B1 catchment, TUFLOW models indicated significant ponding during flood events, with water discharging northeast through Edenbrook Drain and east through East-West Channel to Point B (Kay McDuff Drive). To represent this in DRAINS, a basin was modelled to capture the ponding water, with weirs/overflows directed to both Edenbrook Drain and the East-West Channel. Similarly, two basins were modelled at Point B, each associated with the culverts passing under Kay McDuff Drive. In this configuration, the southern basin fills first and overtops into the northern basin, which subsequently overflows towards Edenbrook Drain once flood levels reach RL 26.90.

Additionally, the Bundaberg Ring Road along the southern portion of the precinct appears to act as a bund, causing water to pond around Location C and overtop eastward along Bundaberg Ring Road. This has been represented in the pre-development DRAINS model as a basin, with weirs/overflows along the Bundaberg Ring Road swale and southeast corner of Pressler site.

3.4.6 Tailwater Conditions

At Point C, significant ponding is observed across all storm events; therefore, tailwater conditions derived from the TUFLOW pre-development model have been adopted. Refer to Table 7 below for the Tail Water Levels at each point obtained from the TUFLOW results

Table 7: Existing peak flood levels at key assessment locations

Peak Flood Levels Across Precinct (mAHD)					
Flood Event	Location A	Location B	Location C	Location D	Location E
1% AEP +CC	N/A	N/A	28.6	N/A	N/A
1% AEP	N/A	N/A	28.6	N/A	N/A
2% AEP	N/A	N/A	28.5	N/A	N/A
5% AEP	N/A	N/A	28.4	N/A	N/A
10% AEP	N/A	N/A	28.4	N/A	N/A
18.1% AEP	N/A	N/A	28.3	N/A	N/A
39.4% AEP	N/A	N/A	28.2	N/A	N/A
63.2% AEP	N/A	N/A	28.2	N/A	N/A

3.5 Results

Table 8: DRAINS - TUFLOW Peak Flow Comparison

AEP	Location	TUFLOW	DRAINS	Difference
1.00% + CC	A (Culv0211)	16.703	15.7	-6.00%
	B (HP5)	11.09	10.66	-3.88%
	C (Culv0445)	22.53	22.7	7.86%
1.00%	A (Culv0211)	13.65	13.1	-4.03%
	B (HP5)	9.21	9.21	0.00%
	C (Culv0445)	21.44	19.9	-3.45%
2.00%	A (Culv0211)	11.65	11.3	-3.00%
	B (HP5)	7.753	8.1	4.48%
	C (Culv0445)	20.22	19.2	-1.09%
5.00%	A (Culv0211)	9.61	10.2	6.14%
	B (HP5)	6.18	6.3	1.94%
	C (Culv0445)	18.08	18.5	2.32%
10.00%	A (Culv0211)	7.572	8.35	10.27%
	B (HP5)	4.995	5.041	0.92%
	C (Culv0445)	16.044	16.4	2.84%
18.10%	A (Culv0211)	6.604	6.16	-6.72%
	B (HP5)	4.077	3.9	-4.34%
	C (Culv0445)	14.807	14.9	-2.75%
39.40%	A (Culv0211)	4.61	4.22	-8.46%
	B (HP5)	2.642	2.73	3.33%
	C (Culv0445)	11.699	11.9	-10.25%
63.20%	A (Culv0211)	3.14	3.09	-1.59%
	B (HP5)	1.798	1.879	4.51%
	C (Culv0445)	7.92	8.47	-9.97%

Table 9: DRAINS - TUFLOW Differences

Location	1% AEP + CC	1% AEP	2% AEP	5% AEP	10% AEP	18.10% AEP	39.40% AEP	63.20% AEP
A (Culv0211)	-6.00%	4.03%	-3.00%	6.14%	10.27%	-6.72%	-8.46%	-1.59%
B (HP5)	-3.88%	0.00%	4.48%	1.94%	0.92%	-4.34%	3.33%	4.51%
C (Culv0445)	0.75%	7.18%	-5.04%	2.32%	2.22%	0.63%	1.72%	6.94%

Table 10: DRAINS - TUFLOW Critical Duration

AEP	Location	TUFLOW Critical Duration	DRAINS Critical Duration
1.00% + CC	A (Culv0211)	270min	120min
	B (HP5)	270min	270min
	C (Culv0445)	270min	180min
1.00%	A (Culv0211)	270min	120min
	B (HP5)	270min	270min
	C (Culv0445)	270min	180min
2.00%	A (Culv0211)	270min	120min
	B (HP5)	360min	270min
	C (Culv0445)	270min	180min
5.00%	A (Culv0211)	180min	180min
	B (HP5)	270min	360min
	C (Culv0445)	180min	180min
10.00%	A (Culv0211)	180min	180min
	B (HP5)	270min	360min
	C (Culv0445)	180min	180min
18.10%	A (Culv0211)	360min	180min
	B (HP5)	360min	360min
	C (Culv0445)	360min	270min
39.40%	A (Culv0211)	180min	120min
	B (HP5)	360min	360min
	C (Culv0445)	360min	180min
63.20%	A (Culv0211)	180min	180min
	B (HP5)	540min	360min
	C (Culv0445)	270min	180min

3.6 Summary

The peak flows between DRAINS and TUFLOW are generally within $\pm 10\%$, which is considered acceptable, except at Point A during the 10% AEP event, where differences are just over 10%. The larger flow discrepancies for this event are likely due to the natural attenuation effects within the TUFLOW model which cannot be fully represented in DRAINS. The change in flow discrepancies from higher to lower between all storm events (i.e. from +% to -%) suggested that there are likely changes in attenuation within this local catchment which cannot be properly accounted for in DRAINS due to its modelling limitations.

Also, please note that the critical durations between the DRAINS and TUFLOW models differ slightly for certain locations and storm events. This variation is primarily due to the differences in how each model routes flows through the catchment. Refer to Figure 16 below, which presents the Peak Flow Chart for Point C during the 63.20% AEP storm. The peak flows for the 180-minute and 360-minute storms are very similar, with only minor discrepancies between the longer-duration events. A similar pattern is observed in the TUFLOW model results, where the critical durations also show minor variations between storm events.

As shown in Table 9, DRAINS peak flows at Points A and C generally increase and then decrease compared with TUFLOW peak flows. These consistent trends of decreasing or increasing flows between DRAINS and TUFLOW indicate differences in how each model represents catchment response and flow routing. The decreasing flows at Points A and C suggest that DRAINS may simulate greater attenuation or slower routing for smaller storms, while TUFLOW captures additional overland flow and storage effects. Overall, these

patterns highlight model differences and the sensitivity of certain catchments to storm intensity, which should be considered when interpreting results.

Overall, the DRAINS and TUFLOW model results show good agreement, with peak flows generally within $\pm 10\%$, which is considered acceptable. Minor variations at certain locations and storm events are attributed to differences in model representation of catchment response and flow routing. These results are considered comparable and sufficient for use in testing the proposed Precinct Plan stormwater, grading, and basin design.

Refer to Appendix C for images of DRAINS model and results.

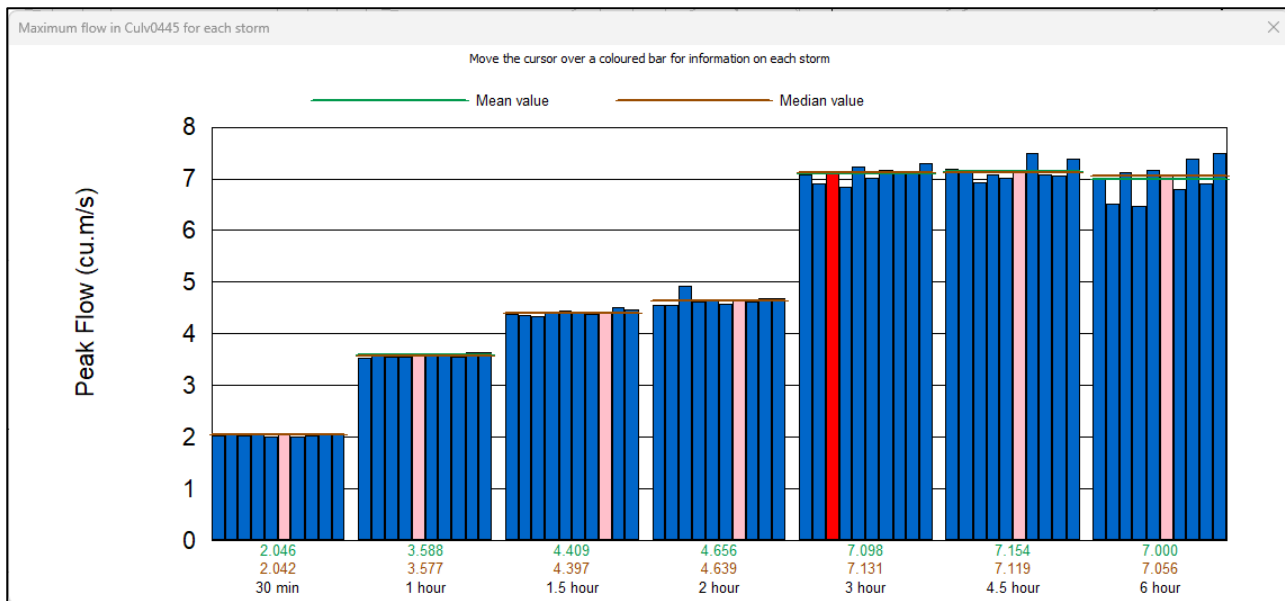


Figure 16: Peak Flow Chart:

4.0 DRAINS Post-Development Comparisons

4.1 Introduction

A precinct-scale DRAINS model has been developed to test concept stormwater and site grading designs efficiently. The modelling covered both existing and post-development flood scenarios across the full range of design storm events (63.2% to 1% AEP, including climate change) to ensure comprehensive assessment under current and future conditions.

Three key sub-catchments were assessed: Pressler North, Edenbrook, and Pressler South.

Further verification with TUFLOW will be carried out to confirm the adequacy of the detention system design using the DRAINS model, also ensuring displaced ponding does not affect upstream properties. Overall, the proposed detention and grading strategy should demonstrate that post-development flows can be effectively managed, maintaining or improving existing flood behaviour to meet BRC stormwater and flood performance requirements.

4.2 Modelling Assumptions

4.2.1 Catchment Delineation/Time of Concentration

The catchment areas for the Pressler and Edenbrook sites were delineated into sub catchments of approximately 10,000 m², each assigned to the nearest corresponding pit within the stormwater network. The delineation process ensured accurate representation of surface flow paths and contributing areas. Refer to Appendix D for the Pressler Catchment Plan, Appendix E for the Edenbrook Catchment Plan, and Appendix F for the Upstream Catchment Plan.

For urban residential, commercial, and road corridor areas, a uniform time of concentration of 5 minutes was adopted for each sub catchment. However, for the as built section within Edenbrook, the time of concentration was determined using the Friend's Equation, incorporating the longest flow path and average slope of the respective catchment, and Hortons roughness of $n^*=0.01$, reflecting predominately concrete, based on Table 4.6.5 Surface Roughness or Retardance Factors from QUDM.

For basin and park areas, vegetated swales, and upstream catchments, the time of concentration was determined using Friend's Equation, incorporating the longest flow path and average slope of the respective catchment, and Hortons roughness of $n^*=0.1$, reflecting predominately short grass cover, based on Table 4.6.5 Surface Roughness or Retardance Factors from QUDM.

4.2.2 Fraction Of Impervious

The fractions of impervious area adopted for the post-development scenario at both the Pressler and Edenbrook sites were based on Table 11 of the Bundaberg Regional Council Stormwater Management Guidelines (refer to Figure 17 below). The following impervious percentages were applied:

- Urban residential (low density): 70%
- Urban residential (high density): 90
- Commercial/industrial: 90%
- Road corridors: 90%
- Open space (including parks): 0%.

Table 11 Fraction Impervious by Development Class (Adapted from QUDM Table 4.5.1 and Informed from Analysis of Council's Fraction Impervious Database)

Development Class	Average Allotment Size	Fraction Impervious
Open Space (including Parks)	-	0%
Rural residential (4,000-10,000m ²)	4,000-10,000m ²	20%
Rural residential (2,000-4,000m ²)	2,000-4,000m ²	40%
Urban residential - Low density (>1,000m ²)	>1,000m ²	50%
Urban residential - Low density (600-1,000m ²)	600-1,000m ²	60%
Urban residential - Low density (<600m ²)	<600m ²	70%
Urban residential - Medium density	-	80%
Urban residential - High density	-	90%
Commercial / Industrial	-	90%
Central Business District	-	100%
Road Corridor	-	90%

Figure 17: Fraction of Impervious by Development Class (Table 11 BRC Stormwater Guidelines)

4.2.3 Losses

Losses for Hydrologic Modelling is consistent with the guidance provided in **Section 5.6.4, Table 12** of the BRC Stormwater Management (see Figure 15 below).

Table 12 Regional Calibrated Hydrologic Model Rainfall Losses - Bundaberg City & Coastal Urban Areas

Surface Type	Initial Loss	Continuing Loss
Pervious	25 mm	1.5 mm/h
Impervious	1 mm	0 mm/h

Figure 18: Hydrological Model Rainfall Losses

4.2.4 Climate Change Factor for 1% AEP + CC

Since DRAINS only allows a single rainfall multiplier to be applied across all storm events, the climate change (CC) factor was derived from the TUFLOW model by comparing the 1% AEP baseline and 1% AEP + CC design rainfalls. The average increase in rainfall depth was calculated to be 19.7%; therefore, a rainfall multiplier of 1.197 was applied in DRAINS for the 1% AEP + CC event.

4.2.5 Tailwater Conditions

At Point C, significant ponding is observed across all storm events; therefore, tailwater conditions derived from the TUFLOW pre-development model have been adopted. Refer to Table 7 in Section 3.4.6 for the Tail Water Levels at each point obtained from the TUFLOW results.

4.2.6 Minor Drainage System

The pipes within the DRAINS model were sized to convey the minor drainage storm, in accordance with Table 2 of the Bundaberg Regional Council Stormwater Management Guidelines (refer to Figure 19 below). The following design storm events were applied to each land use category:

- Commercial/Industrial: 10% AEP
- Urban Residential (Low Density): 18% AEP
- Urban Residential (High Density): 10% AEP
- Roads: As per the applicable development category

Development Category (QUDM)	Council Planning Scheme – Zone	Design Storm AEP
Central business and commercial	Principal centre zone, Major centre zone, district centre zone, Local centre zone, Neighborhood centre zone, Specialised centre zone	10% AEP
Industrial	Industry zone, High impact industry zone	10% AEP
Urban residential high density	High density residential zone	10% AEP
Urban residential low density	Medium density residential zone, Low density residential zone, Emerging community zone, Limited development zone, Community facilities zone	18% AEP
Rural Residential	Rural residential zone, Sport and recreation zone	39% AEP
Open space – parks, etc.	Rural zone, Open space zone, Environmental management and conservation zone	63% AEP
Roadway Criteria		Design Storm AEP
Major Road (i.e., Arterial, Sub-arterial, Trunk Collector (Suburban), Industrial Collector, Principal Rural Road)	Table Drain/Kerb & Channel	10% AEP ⁽¹⁾
	Cross Drainage (Culverts)	2% AEP ^(2,3)
All other Roads	Kerb and Channel	Use relevant Development Category above
	Cross Drainage (if Rural Culverts ⁽⁴⁾)	10% AEP ⁽³⁾

Figure 19: Design Storms for Minor Drainage System (Table 2 BRC Stormwater Management Guidelines)

4.3 Results

As discussed in Section 3.2, the three critical locations assessed in the pre-development DRAINS model were Points A, B, and C. The post-development DRAINS model focuses on Points A and C only. No modelling was undertaken for Point B, as the flows contributing to the East–West Swale (Point B) originate from Stage 4 of the Edenbrook development, where the basin has already been designed by others. In addition, the proposed commercial developments along the East–West Road corridor, including the Bundaberg Hospital, will incorporate their own on-site detention systems to limit post-development peak flows to pre-development levels.

4.3.1 Pressler North Results

Table 11: Pressler North towards Edenbrook Drain Post-Dev Comparison

Pressler North towards Edenbrook Drain

AEP	Pre	Post	Reduction	TWL	Peak Storm
1.00% + CC	6.65	5.02	24.51%	27.52	360
1.00%	5.55	3.95	28.83%	27.29	270
2.00%	4.81	3.63	24.53%	27.13	270
5.00%	4.00	2.93	26.75%	27.00	360
10.00%	3.21	1.68	47.66%	26.86	360
18.10%	2.49	1.52	38.96%	26.66	360
39.40%	1.63	0.987	39.45%	26.48	360
63.20%	1.13	0.756	33.10%	26.35	360

Table 12: Pressler North towards East West Swale Post-Dev Comparison

Pressler North towards East -West Swale

AEP	Pre	Post	Reduction
1.00% + CC	4.06	2.45	39.66%
1.00%	3.44	2.05	40.41%
2.00%	3.02	1.85	38.74%
5.00%	2.54	1.7	33.07%
10.00%	2.09	1.49	28.71%
18.10%	1.66	1.27	23.49%
39.40%	1.13	1.06	6.19%
63.20%	0.804	0.83	-3.23%

4.3.2 Pressler North Summary

Within the pre-development model for Pressler North, it was evident that the site discharged stormwater both northeast towards the Edenbrook Drain and east towards the East–West Swale. The objective of the post-development design is to maintain these same discharge locations; with majority of the catchment draining to an above-ground basin, except for the commercial area adjacent to East–West Road. The discharge from the commercial area towards the East–West Swale remains un-detained (no on-site detention), and as shown in Table 12, it remains below pre-development peak flows for all storm events except the 63.2% AEP. Consequently, during detailed design, some form of detention may be required to limit peak flows for less frequent storm events.

For the discharge towards the Edenbrook Drain, detention is proposed to reduce peak flows by more than 20% for all storm events. As shown in Table 11, the post-development peak flow rates are lower than the pre-development flows for all storm events. This is achieved using a basin of 68,800 m³ storage capacity and a 1 × 900 mm pipe to control the 63.2% AEP event flows, a 20m weir at RL 26.90 to control the 5% AEP event flows, which then conveys stormwater to another 1 × 900 mm pipe controlling the 2% AEP event flows, and a final emergency 40m weir at RL 27.45 to manage the 1% AEP plus climate change event flows. The design intent is to limit the peak flow from the Pressler site so that it does not coincide with the peak from the Edenbrook site, and to reduce the amount of cut required in the Edenbrook Drain for detention purposes. The top water level during the 1% AEP + climate change event is 27.52, as such the surrounding roads have been designed to be at least RL 27.82 high, to achieve minimum 300 mm freeboard.

4.3.3 Edenbrook Results

Table 13: Edenbrook Drain (Point A) Post-dev Comparison

Edenbrook Drain (Point A)

AEP	Pre - TufLOW	Pre - DRAINS	Post - DRAINS	Reduction	TWL
1.00% + CC	16.70	15.70	14.60	7.01%	25.96
1.00%	13.65	13.10	9.92	24.27%	25.80
2.00%	11.65	11.30	9.38	16.99%	25.69
5.00%	9.61	10.20	8.46	17.06%	25.55
10.00%	7.57	8.35	5.86	29.82%	25.45
18.10%	6.60	6.16	4.68	24.03%	25.4
39.40%	4.61	4.22	2.77	34.36%	25.07
63.20%	3.14	3.09	2.78	10.03%	24.82

Table 14: Edenbrook Drain (Point A) All Storms Post-dev Comparison

Edenbrook Drain (Point A) All Storms Comparison

AEP	Storm (mins)	Pre - DRAINS	Post - DRAINS
1% + CC	30	12.00	11.67
	60	14.80	13.35
	90	15.50	13.83
	120	15.70	14.62
	180	14.70	14.49
	270	14.70	14.59
	360	14.10	14.10
1.00%	30	10.00	9.25
	60	12.20	9.66
	90	12.70	9.81
	120	13.10	9.63
	180	12.20	9.47
	270	12.20	9.44
	360	11.40	9.92
2.00%	30	8.32	8.26
	60	10.40	9.25
	90	10.90	9.34
	120	11.30	9.27

	180	10.70	9.19
	270	10.60	8.94
	360	9.91	9.39
5.00%	30	6.09	4.84
	60	8.19	7.55
	90	8.83	7.84
	120	9.17	8.46
	180	10.20	8.24
	270	8.98	8.00
	360	8.92	8.08
10.00%	30	4.82	2.91
	60	6.63	5.28
	90	7.33	5.75
	120	7.34	5.87
	180	8.35	5.65
	270	7.52	5.81
	360	7.48	5.85
18.10%	30	3.74	2.76
	60	5.27	3.10
	90	5.89	3.66
	120	6.03	4.18
	180	6.16	4.01
	270	5.99	4.68
	360	5.19	4.11
39.40%	30	2.48	2.75
	60	3.59	2.75
	90	4.15	2.76
	120	4.22	2.77
	180	4.12	2.77
	270	4.12	2.78
	360	3.61	2.69
63.20%	30	1.42	2.74
	60	2.32	2.79
	90	2.79	2.76
	120	2.92	2.74
	180	3.09	2.76
	270	2.91	2.55
	360	2.64	2.41

4.3.4 Edenbrook Summary

The existing Edenbrook Drain is proposed to be utilised as a detention basin for the Edenbrook site; however, modelling indicates that the storage volume of the existing drain will need to be increased. Refer to Appendix E, which indicates that the basin within the Edenbrook Drain requires partial widening, with the proposed park area integrated as part of the basin. As shown in Table 13, the post-development peak flow rates are lower than the pre-development flows for all storm events. This outcome is achieved using a basin with a total storage capacity of **31,600 m³**, and **2 × 825 mm** pipes to control the **63.2% AEP** event flows. A **20 m weir at RL 25.25** manages the **5% AEP** event flows, conveying flows to **3 × 900 mm** pipes that control the **2% AEP** event flows, with a final **40 m emergency weir at RL 25.78** designed to manage the **1% AEP plus climate change** event flows. The top water level during the **1% AEP + climate change** event reaches **RL 25.96**; therefore, surrounding road levels have been set to at least **RL 26.26** to maintain minimum **300 mm freeboard**.

As Saltwater Creek currently presents a high risk of flooding to downstream residential properties, all storm events have been compared against pre-development peak flow rates (see Table 14). The storms highlighted in yellow represent the critical durations for each event. As shown, the critical storms are generally of longer duration, and post-development peak flows are lower than pre-development across all events—except for the shorter-duration storms (30–60 minutes) for the **39.40%** and **63.20% AEP** events. These minor exceedances are considered acceptable, as the shorter events generate limited total runoff volume which will generally contained within the waterway corridor, drain quickly, and do not contribute significantly to downstream flood risk. Further, the 39.40% and 63.20% AEP flow rates are similar across most storm events. This is because, in both cases, the top water level remains below the 20 m weir crest and the outflow is controlled solely by the two 825 mm pipes, which operate at similar capacity under similar head conditions.

4.3.5 Pressler South

Table 15: Pressler South Site Post-Dev Comparison

Pressler South Site

AEP	Pre - DRAINS	Post - DRAINS	TWL	Peak Storm
1.00% + CC	4.27	4.14	28.96	120
1.00%	3.55	2.82	28.89	90
2.00%	3.07	2.57	28.76	90
5.00%	2.41	2.23	28.65	180
10.00%	2.78	1.76	28.61	180
18.10%	2.11	0.964	28.47	120
39.40%	1.4	0.832	28.32	90
63.20%	0.952	0.712	28.28	120

Table 16: Pressler South Culvert (Point C) Post-Dev Comparison

Pressler South Culvert (Point C)

AEP	Location	Pre	Post
1.00% + CC	Culv0445	22.7	24.6
	C1	9.15	9.02
	Bypass	14.9	14.8
1.00%	Culv0445	19.9	21.7
	C1	7.61	6.88
	Bypass	13.5	13.1
2.00%	Culv0445	19.2	20.7
	C1	6.59	6.06
	Bypass	9.8	9.41
5.00%	Culv0445	18.5	19.5
	C1	5.16	4.97
	Bypass	6.65	6.17
10.00%	Culv0445	16.4	16.7
	C1	5.95	4.92
	Bypass	5.99	5.3
18.10%	Culv0445	14.9	14.4
	C1	4.52	3.37
	Bypass	3.3	2.78
39.40%	Culv0445	11.9	11
	C1	3	2.43
	Bypass	1.19	1.06
63.20%	Culv0445	8.47	8.2
	C1	2.04	1.812
	Bypass	0.923	0.887

4.3.6 Pressler South Summary

For the discharge from the Pressler South site, detention is proposed to reduce post-development peak flows to match pre-development conditions. As the South Pressler post-development site occupies only a portion of the overall pre-development catchment (C1 in the pre-development catchment plan from the DRAINS Calibration Report), the pre-development peak flows that the site is required to match were determined by subtracting the undeveloped upstream catchment (31.8 ha, as shown in Appendix D) from the total South Pressler pre-development catchment area (59.6 ha, Catchment C1 as shown in the pre-development catchment plan from the DRAINS Calibration Report). Accordingly, the effective pre-development area for calculation of pre-development peak flows is 27.8 ha ($59.6 - 31.8 = 27.8$ ha).

The peak flows from the pre-development area calculated above are shown in Table 7, which also demonstrates that the post-development peak flow rates from South Pressler Site are lower than the pre-development flows for all storm events. This is achieved using a basin with a total storage capacity of 94,100 m³, and 2 × 750 mm pipes to control the 63.2% AEP event flows. A 20 m weir at RL 28.53 manages the 5% AEP event flows, which then conveys flows to 3 × 750 mm pipes controlling the 2% AEP event flows, and a final 40 m emergency weir at RL 28.90 manages the 1% AEP plus climate change event flows. The top water level during the 1% AEP + climate change event reaches RL 28.96; therefore, surrounding road levels have been set to at least RL 29.26 to achieve a minimum 300 mm freeboard.

The post-development site within South Pressler will displace floodwater currently ponding during pre-development storm events. During DRAINS modelling, it was observed that reducing this ponding area increased the flow through the culvert under Bundaberg Ring Road (Point C) and the bypass along Bundaberg Ring Road. To mitigate this, earthworks have been proposed to lower (cut) portions of the undeveloped area within the South Pressler site to create a flood basin (refer to Appendix D). Table 8 presents a comparison of post-development and pre-development peak flows through the culvert under Bundaberg Ring Road (Point C). As shown, the proposed cut in the undeveloped area effectively reduces post-development peak flows to pre-development levels for the 18.10% AEP to 63.2% AEP storm events. However, for the less frequent storm events, post-development flows remain slightly higher than pre-development flows at Point C.

To achieve lower flows through Point C, a larger flood basin would be required; however, this may not be necessary at this stage given the limitations of the DRAINS modelling due to its hydrologic modelling limitations. Further modelling will be undertaken in TUFLOW during the detailed design phase when proper design features of the precinct area are available, to confirm and ensure that the final adopted detention system for this area will be sufficient to mitigate all post-development peak flows back to pre-development levels for all storm events.

TUFLOW modelling in the detailed design phase will also be required to confirm that the displacement of ponding within the site does not adversely affect upstream properties. If impacts are identified, redesign of detention system proposed and/or further mitigation measures will need to be considered.

5.0 TUFLOW Developed Precinct Assessment

5.1 Methodology

As noted in Section 2.1, Council provided TTW with an updated version of the Bundaberg Citywide Overland Flow Study TUFLOW model (.tcf version 003) with guidance notes on the parameters, storm events, durations and temporal patterns to be used for the assessment.

While Council's existing model was used to inform the DRAINS modelling and catchment delineation (as detailed in Section 2.0 and 3.0), amendments were made to the model in order to assess the functionality of the concept design.

Two scenarios have been modelled by TTW in this assessment:

- Pre-development scenario (existing base case)
- Post-development scenario (following Precinct Development)

To assess how the proposed precinct development can reduce flood behaviour across the Precinct and meet hydraulic performance criteria, the model must be modified to incorporate the precinct stormwater concept via DRAINS inputs alongside other features of the design (design surface, detention basins and associated structures).

As the existing Council model adopts a rainfall-on-grid methodology, the existing Council model must be amended to extract the precinct model from the wider catchment, and to incorporate the pre-development DRAINS inputs – then forming the updated 'pre-development' scenario. This allows for a like-for-like comparison and ensures the impact of the concept can be effectively assessed.

This section details the updates that were made to Council's existing v003 model, alongside the subsequent amendments to the pre-development scenario to produce a post-development model incorporating the concept design.

5.2 Pre-development Model Updates

A rainfall on grid (ROG) hydrology approach was adopted in Council's model, in which design rainfall is applied to each cell within the 2D mesh. Hydrologic losses and runoff are therefore calculated for each cell and routed through downstream cells to simulate overland flow paths and evaluate flood depths and velocities across the model domain.

For the purposes of this assessment, a two-tier approach was adopted to appropriately represent flood behaviour across the broader catchment while providing enhanced resolution within the precinct:

- **Wider catchment:** the ROG approach was retained for the wider catchment to ensure that regional overland flow contributions to the precinct are appropriately represented.
- **Precinct:** a higher-resolution representation was adopted within the precinct itself by applying runoff via 2D source areas. The source inflows applied to the precinct area were derived from the pre-development DRAINS hydrologic model (detailed in Section 3.0). The resulting design flow hydrographs were applied to the corresponding source areas within the hydraulic model.

Figure 20 provides an overview of the hydrological approach adopted across the model extent, alongside the sub catchments and 2D source area locations within the precinct.

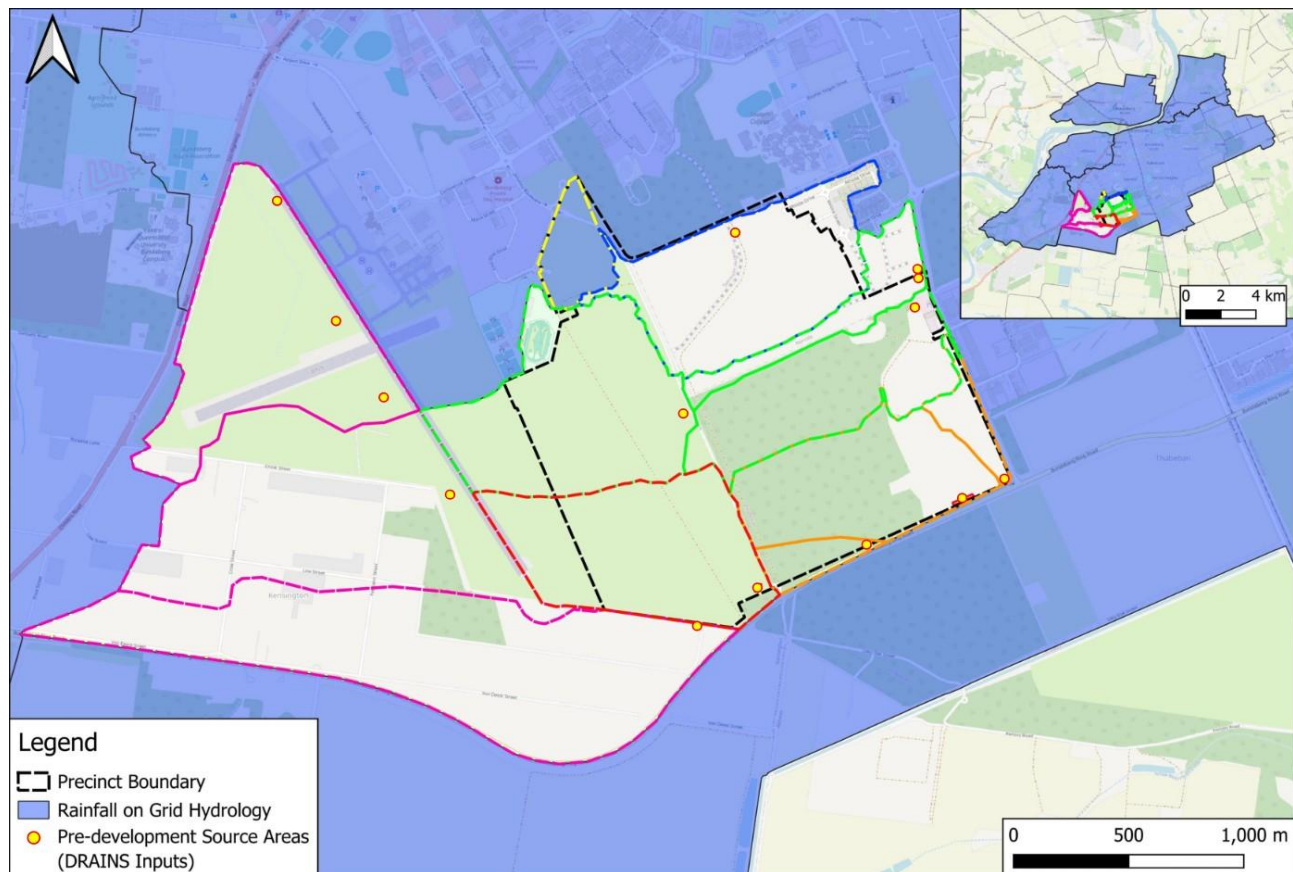


Figure 20: Overview of hydrological approach and source area locations across the model domain

5.3 Post-Development Updates

The pre-development model was further modified to produce a post-development model. Changes made to form the post-development model are discussed below, which include:

- Surface roughness representation for areas outside the precinct have remained similar to that of Council's original model. For areas within the precinct, surface roughness has been applied to the model based on the type of land use proposed and adopted for the precinct plan. The surface roughness adopted for each proposed land use type within the precinct in the post-development TUFLOW model is summarised as follow. It is worth noting that higher Manning's n value of 0.1 has been adopted in the TUFLOW model for drainage corridors, recreational and detention areas assuming low frequency of maintenance, as suggested by Council.

Figure 21 shows the extent of these surface roughness categories adopted in the TUFLOW model for the precinct area.

- Road – n of 0.025
- Low-Medium residential development – n of 0.08
- Medium-High residential development – n of 0.12
- Health Precinct and industrial development – n of 0.17
- Drainage Corridor, recreational and detention area – n of 0.1

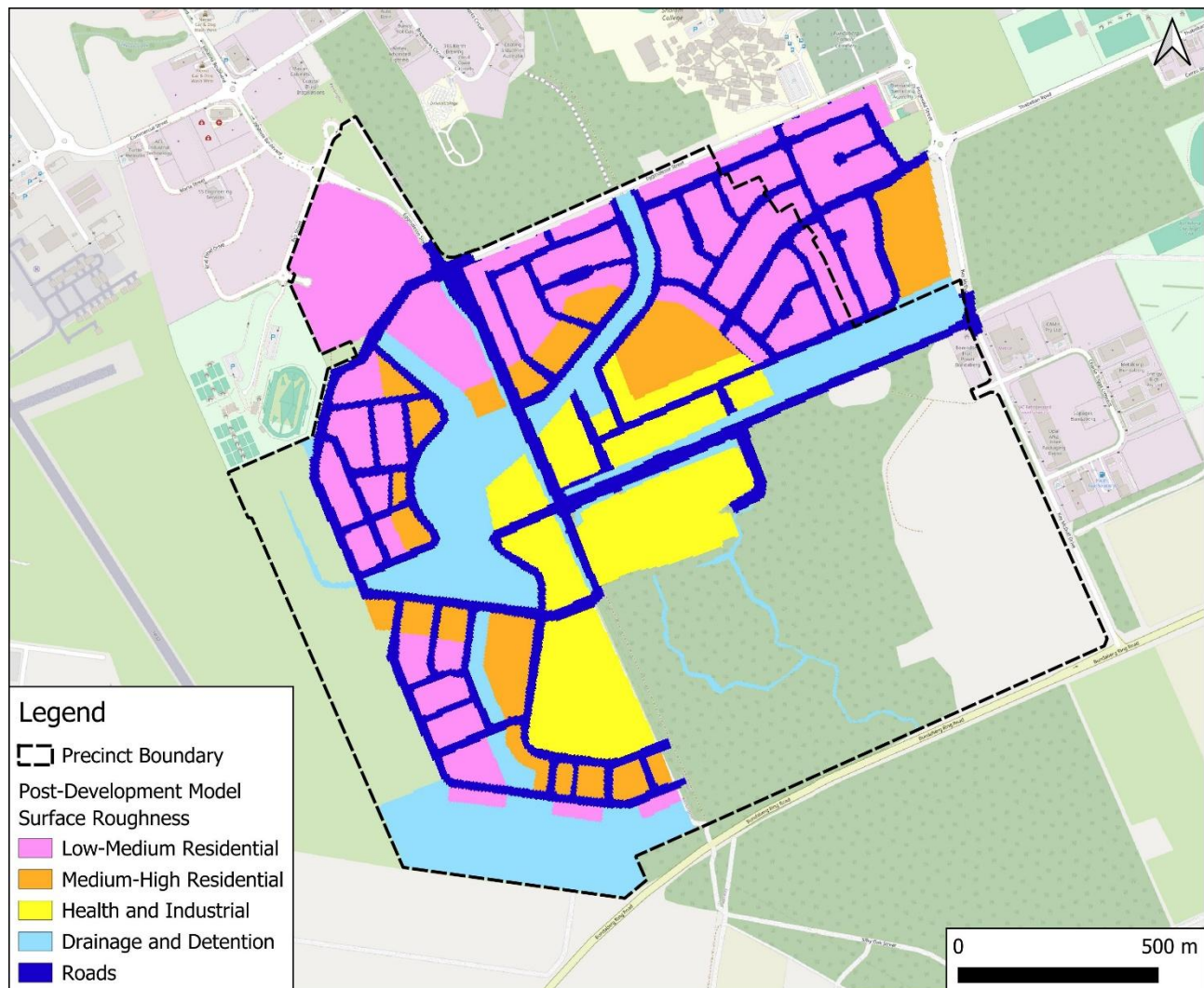


Figure 21: Surface roughness adopted for the precinct area in the post-development model

In addition to the above, the following amendments were made to form the post-development model:

- Preliminary precinct grading based on the 12d design TIN, including concept development pad levels, roads, detention basin footprints and overland flow paths. The updated ground levels are shown in Figure 22;
- Precinct stormwater concept (including basin and trunk drainage network). The stormwater infrastructure incorporated into the post-development model is presented in Figure 23;
- Updated hydrological input data based on the post-development concept design flows (refer to Section 4.0 for detail on the post-development DRAINS model). The 2D source areas were modified and applied to the appropriate low point of each sub-catchment.

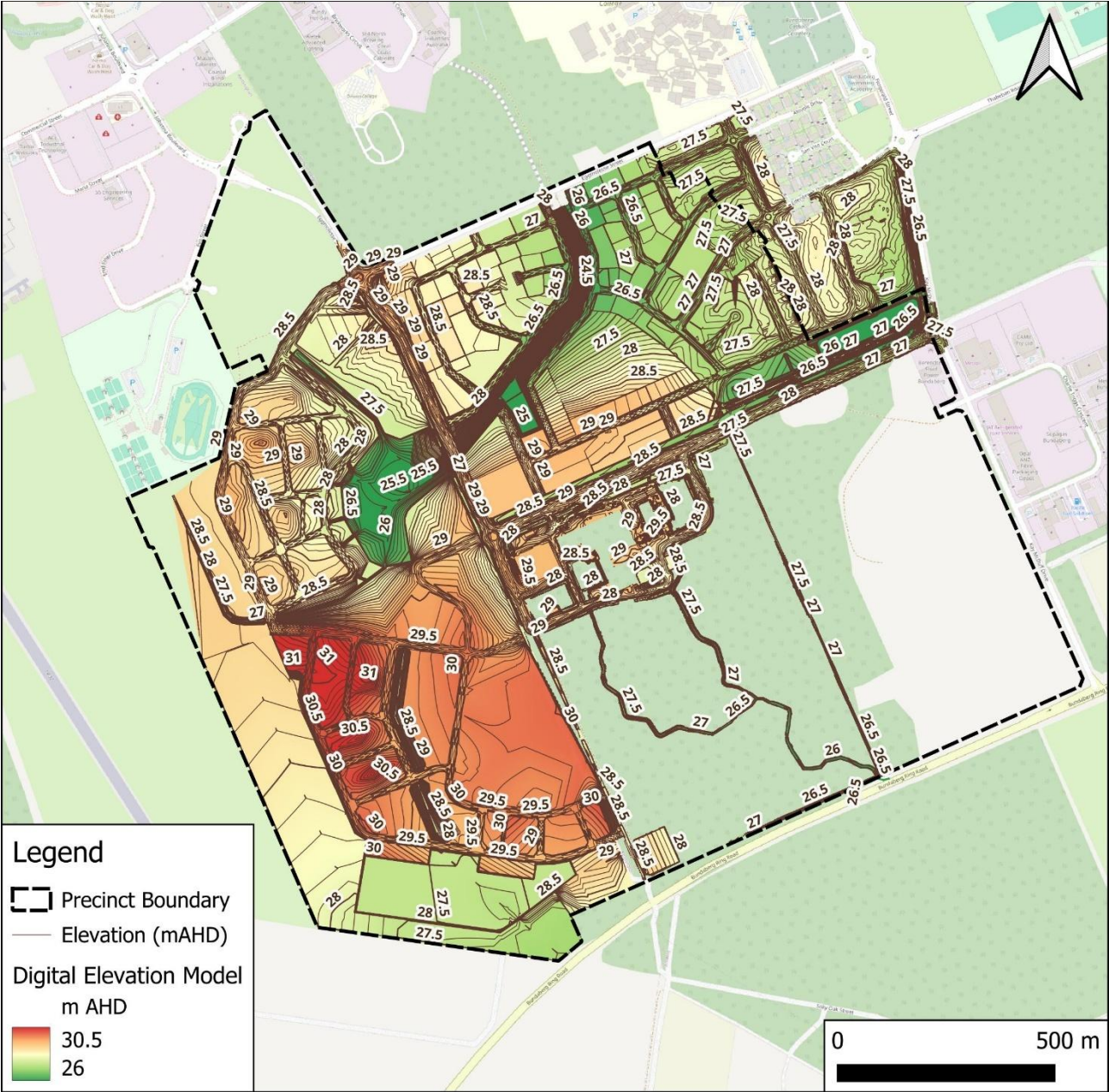


Figure 22: Elevation data incorporated into the post-development model (Source: TTW concept design TIN)

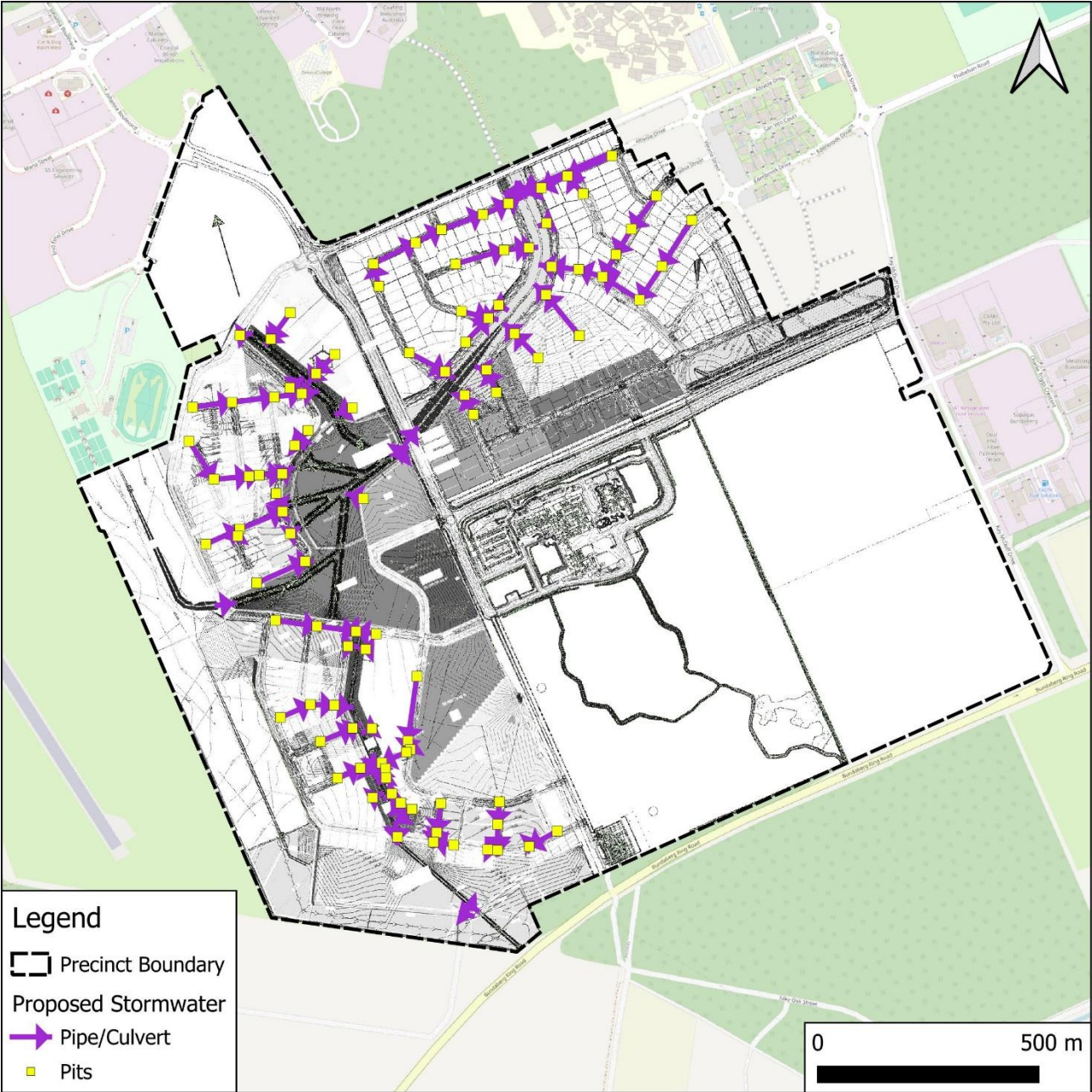


Figure 23: Stormwater infrastructure incorporated into the post-development model

5.4 Results

Mapping of peak flood levels and depths, flood velocities and flood hazard has been prepared for the pre- and post-development scenario for all the AEP events assessed between 1% AEP+CC event and 63.2% AEP event. This mapping is presented in Appendix G.

5.4.1 Pre-Development Scenario

The updated pre-development modelling (i.e. using the source area inflows) shows that the estimated flooding regime for the precinct area in the 1% AEP with climate change consideration event is generally producing results consistent with that estimated by the original Council's model (i.e. using the ROG rainfall approach). In particular, updated pre-development modelling show less wide-spread flood inundation for the precinct area when compared to that of the original Council's ROG modelling. This is mainly due to adoption of source area inflows in the updated pre-development modelling where inflows are applied to certain areas in the model instead of the entire precinct area (i.e. as adopted by the ROG modelling approach). Nonetheless, the main and well-defined flow paths estimated by the original Council's ROG modelling (refer to Figure 5) have generally being replicated in the updated pre-development modelling. This includes the areas in Saltwater Creek at reporting Point E, near Puie Street.

Similarly, flood hazards for these main and well-defined flow paths estimated by the updated pre-development modelling are also consistent with those estimated by the original Council's ROG modelling (refer to Figure 7). The estimated peak flood levels and depths, velocities and hazards for the 1% AEP with future climate change consideration event of the updated pre-development scenario are summarised in Figure 24, Figure 25 and Figure 26, respectively.

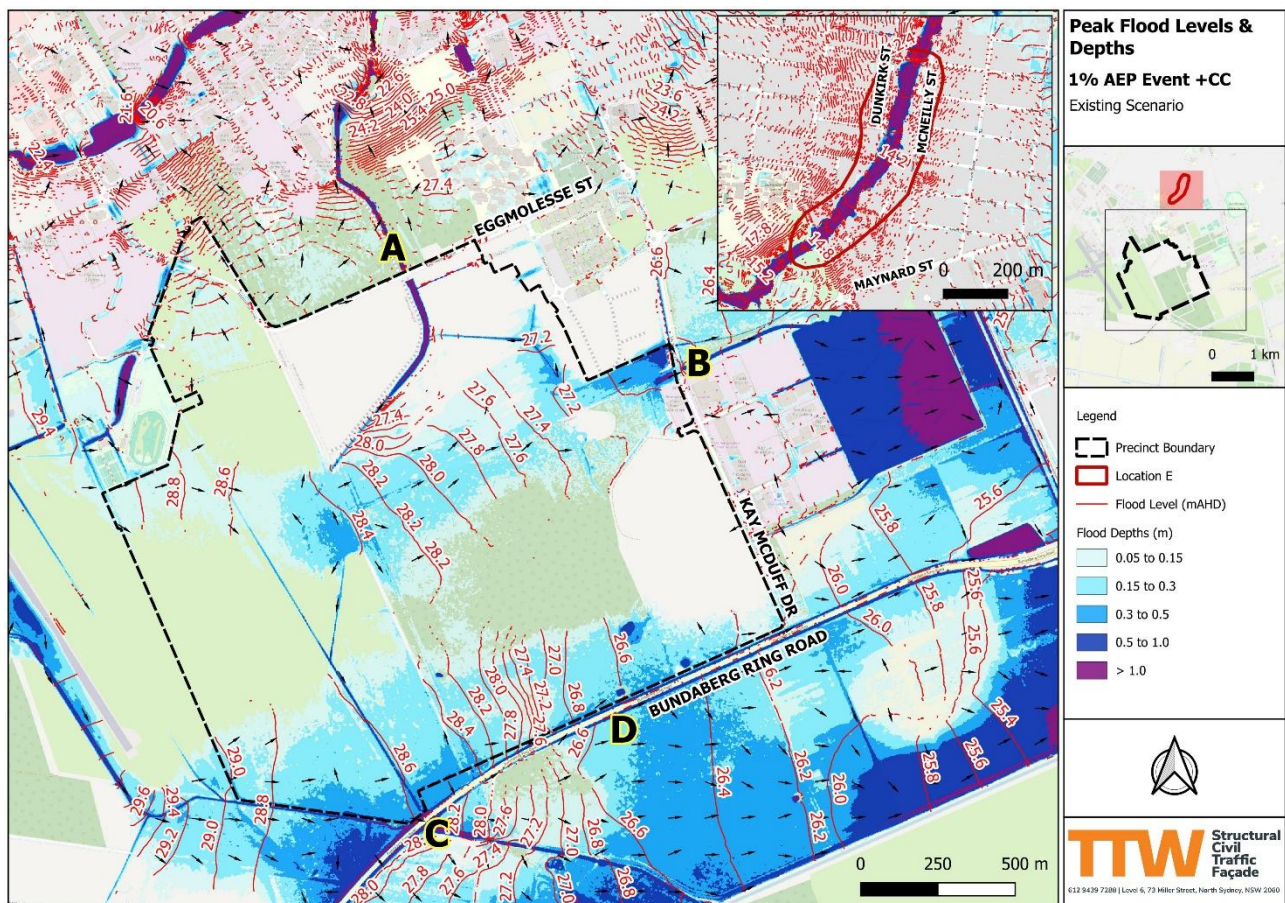


Figure 24: Pre-development flood depths and levels in the 1% AEP event with future climate change

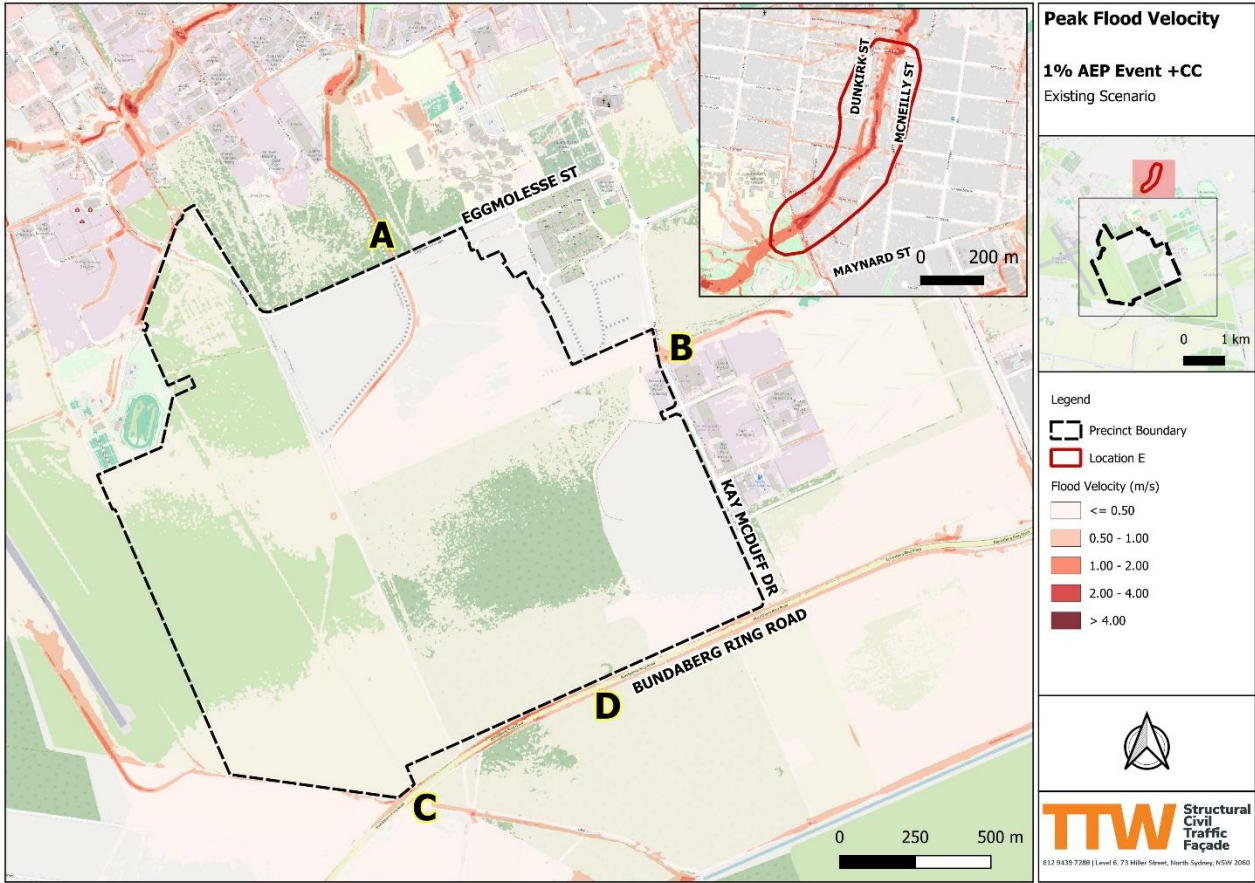


Figure 25: Pre-development flood velocity in the 1% AEP event with future climate change

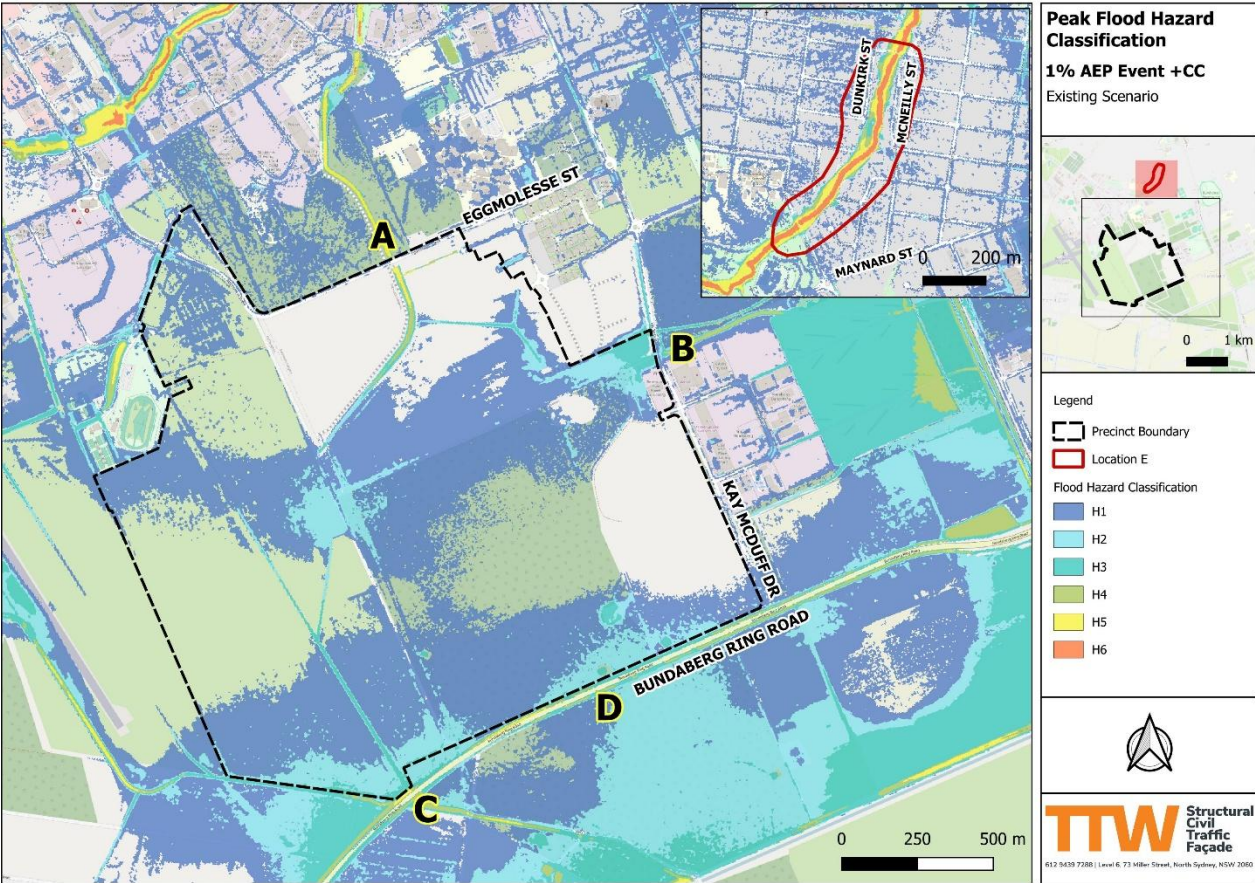


Figure 26: Pre-development flood hazard classification in the 1% AEP event with future climate change

5.4.2 Post-Development Scenario

The estimated peak flood levels and depths, velocities and hazards for the 1% AEP with future climate change consideration event of the post-development scenario are summarised in Figure 27, Figure 28 and Figure 29, respectively.

The modelling results show that flooding regime for the areas outside of the precinct boundary (i.e. downstream of reporting points A, B, C and D) remained generally consistent with that of the pre-development scenario discussed in Section 5.4.1. However, changes to flooding regime within the precinct area is observed for the post-development scenario when compared to that of the pre-development scenario. In particular, majority of the proposed development allotments and areas within the precinct are shown to be flood free in the 1% AEP with climate change consideration event, and flooding are mainly restricted to the designated drainage corridors, recreational and detention areas adopted as part of the precinct planning.

Flooding also being observed along internal roads adopted in the TUFLOW model, showing that the modelling provides a practical flooding conditions whereby roads are often designed to convey excess stormwater runoff as overland flows into nearby waterways. The modelling results show that deep flooding areas are observed within the detention areas at the north and south Pressler development, the Edenbrook drain as well as the stormwater detention system located south-east of the Edenbrook development (i.e. north of the East-West Channel), demonstrating that temporary storage of stormwater runoff occur in these designated areas as anticipated.

While results show generally similar flooding regime for areas outside the precinct, it is necessary to carry out detailed analysis of the key flooding parameters at the five reporting locations between the post-development and pre-development scenarios to confirm the performance and adequacy of the detention measures adopted in the current modelling. This is discussed in Section 5.5. While the modelling of the post-development scenario generally shows flooding regime as anticipated, there are some constraints and areas that can further improved in the subsequent modelling works during the actual design phase of the precinct. This is discussed further in Section 6.0.

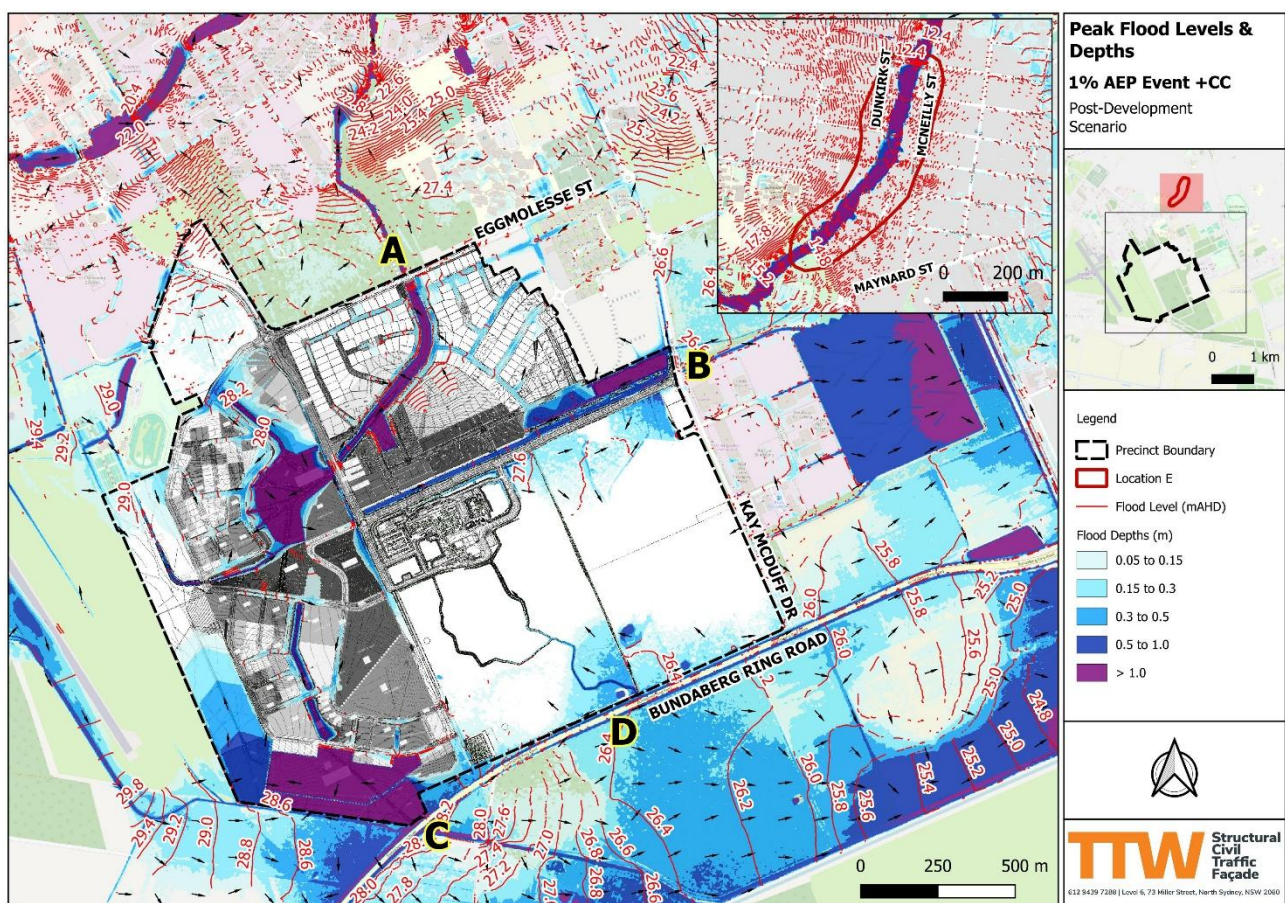


Figure 27: Post-development flood depths and levels in the 1% AEP event with future climate change

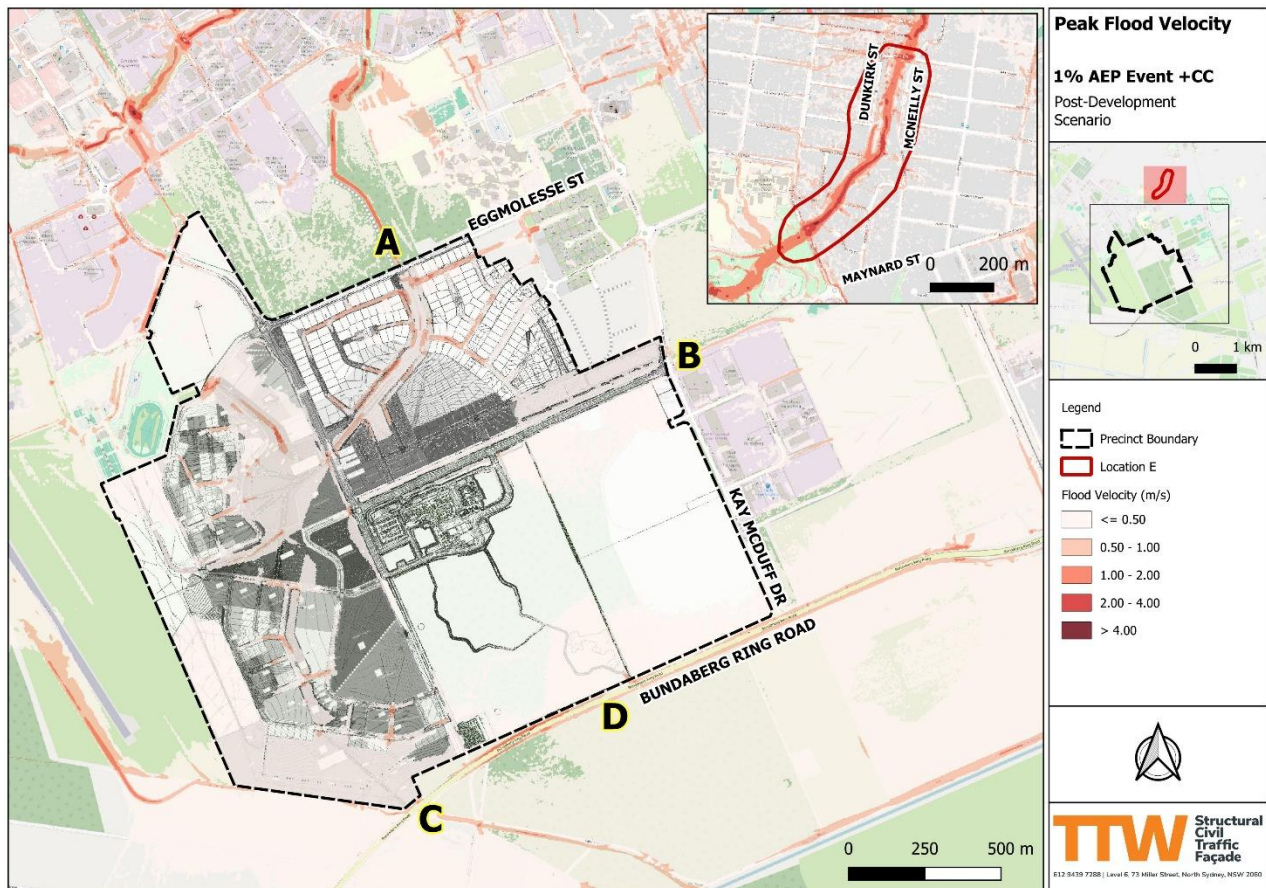


Figure 28: Post-development flood velocity in the 1% AEP event with future climate change

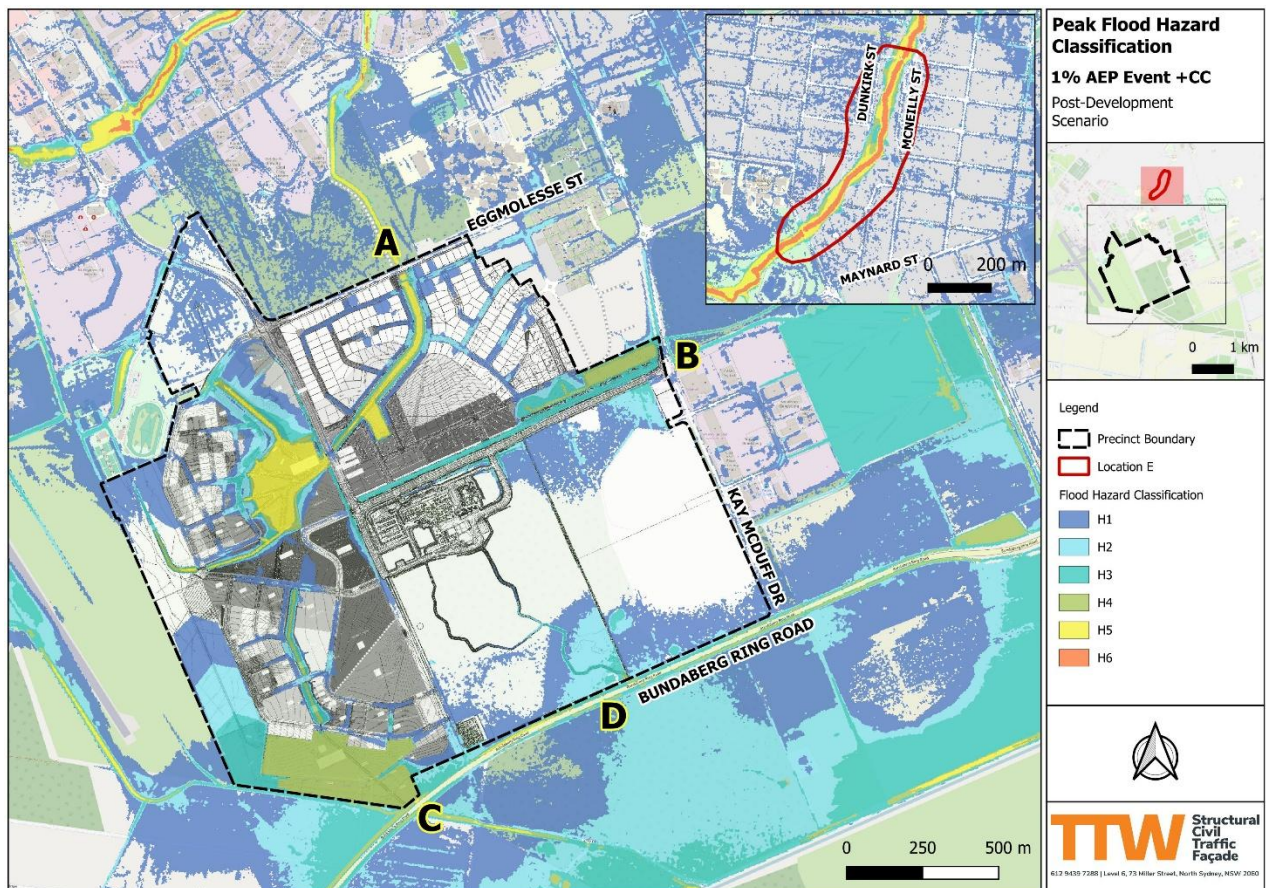


Figure 29: Post-development flood hazard categorisation in the 1% AEP event with future climate change

5.5 Impact Assessment

As noted in Section 1.3.2, an impact assessment of the precinct concept must demonstrate no increase in peak flow, level, velocity at the key assessment locations around the precinct.

Section 5.5.1 assesses change in flow at each assessment location; Section 5.5.2 provides flood level afflux mapping denoting the change in levels as a result of the precinct concept; and Section 5.5.3 provides an analysis of change in flow velocities.

5.5.1 Flow Assessment

An assessment of the like-for-like change in flow from the pre-development to the post-development scenario in each storm duration for the 1% AEP+CC, 10% AEP and 63.2% AEP events is presented in Figure 30 overleaf. The results for all AEP events assessed are included in Appendix H.

Table 17 to Table 21 outline the change in the overall peak flow for each AEP (i.e. the 'peak-peak flow') from the pre-development scenario to the post-development scenario at Locations A to E.

Post vs Pre Peak Flow Comparison						Green = Peak											
Post	Point A (Culv0211)			Point B (HP5)			Point C (Culv0445)			Point D (Culv0600)			Point E (Salt_Puie_Line)				
1%CC	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing Q	Post-Dev Q	Diff Q		
30min	14.73	8.87	-5.86	3.88	3.03	-0.85	13.50	6.75	-6.75	1.14	1.12	-0.02	91.87	88.18	-3.69		
60min	16.29	14.21	-2.08	6.75	3.68	-3.07	17.56	15.08	-2.48	1.05	1.46	0.41	100.34	98.22	-2.11		
90min	16.64	16.83	0.18	7.96	4.40	-3.57	19.36	19.43	0.07	1.01	1.56	0.54	106.13	107.02	0.89		
120min	17.32	18.86	1.54	9.47	4.98	-4.49	20.76	22.26	1.50	1.44	1.82	0.38	113.93	115.69	1.76		
270min	16.82	18.78	1.97	13.14	4.72	-8.42	22.22	24.52	2.30	2.33	1.99	-0.34	112.98	115.40	2.42		
360min	17.18	19.03	1.85	11.79	4.69	-7.11	21.73	24.03	2.30	2.05	2.04	-0.01	125.54	125.39	-0.15		
Post	Point A (Culv0211)			Point B (HP5)			Point C (Culv0445)			Point D (Culv0600)			Point E (Salt_Puie_Line)				
10% AEP	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing Q	Post-Dev Q	Diff Q		
30min	5.96	3.93	-2.03	1.78	1.55	-0.24	5.04	2.55	-2.49	0.86	0.85	-0.01	43.77	43.19	-0.58		
60min	7.34	6.20	-1.14	2.40	2.29	-0.11	8.25	3.55	-4.70	0.99	1.07	0.08	50.66	50.40	-0.25		
120min	8.15	7.92	-0.23	3.97	2.91	-1.06	12.22	11.52	-0.70	0.95	1.08	0.13	59.99	60.33	0.34		
180min	8.44	9.01	0.56	5.44	3.16	-2.28	15.06	14.64	-0.42	1.03	1.15	0.11	65.39	66.90	1.51		
270min	7.29	9.79	2.49	5.67	3.12	-2.55	14.60	14.98	0.38	1.07	1.18	0.11	57.06	59.82	2.76		
360min	6.90	10.00	3.10	6.17	3.15	-3.02	15.19	15.38	0.19	1.01	1.07	0.06	61.45	62.78	1.34		
Post	Point A (Culv0211)			Point B (HP5)			Point C (Culv0445)			Point D (Culv0600)			Point E (Salt_Puie_Line)				
63.2% AEP	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing	Post-Dev	Difference	Existing Q	Post-Dev Q	Diff Q		
30min	1.70	2.71	1.01	0.49	0.38	-0.11	1.37	0.61	-0.76	0.60	0.55	-0.05	16.17	19.73	3.56		
90min	2.96	3.16	0.20	1.17	1.23	0.06	3.95	1.25	-2.70	0.77	0.78	0.01	23.57	23.88	0.31		
180min	3.19	3.34	0.15	1.61	1.78	0.17	5.73	3.74	-2.00	0.98	0.77	-0.21	34.47	35.40	0.93		
270min	2.77	3.27	0.51	1.96	1.94	-0.02	6.57	5.21	-1.36	0.76	0.97	0.21	31.35	31.93	0.58		
360min	2.30	3.09	0.79	2.10	1.73	-0.37	6.89	5.86	-1.03	0.72	0.81	0.09	23.37	23.85	0.49		
540min	2.48	3.32	0.84	1.84	1.60	-0.24	5.83	5.26	-0.57	0.85	0.85	0.00	28.84	29.36	0.52		

Figure 30: Comparison of pre-development and post-development peak flows for each storm duration in the 1% AEP+CC, 10% AEP and 63.2% AEP events

Table 17: Difference in pre and post peak flow for the AEP events assessed at Location A

Change in peak flow at Location A (Culv0211) (cu.m/s)			
Event	Pre-development flow	Post-development flow	Difference
1% AEP+CC	17.32	19.03	1.71
1% AEP	13.90	16.73	2.83
2% AEP	11.81	15.12	3.31
5% AEP	10.74	12.21	1.47
10% AEP	8.44	10.00	1.56
18.1% AEP	6.77	8.36	1.60
39.4% AEP	4.28	4.88	0.60
63.2% AEP	3.19	3.34	0.15

Table 18: Difference in pre and post peak flow for the AEP events assessed at Location B

Change in peak flow at Location B (HP5) (cu.m/s)			
Event	Pre-development flow	Post-development flow	Difference
1% AEP+CC	13.14	4.98	-8.16
1% AEP	16.73	4.36	-12.36
2% AEP	15.12	3.43	-11.69
5% AEP	12.21	3.49	-8.72
10% AEP	6.17	3.16	-3.00
18.1% AEP	8.36	2.81	-5.56
39.4% AEP	4.88	0.73	-4.15
63.2% AEP	3.34	1.01	-2.33

Table 19: Difference in pre and post peak flow for the AEP events assessed at Location C

Change in peak flow at Location C (Culv0445) (cu.m/s)			
Event	Pre-development flow	Post-development flow	Difference
1% AEP+CC	22.22	24.52	2.30
1% AEP	20.91	24.25	3.34
2% AEP	19.70	21.44	1.73
5% AEP	17.27	18.12	0.85
10% AEP	15.19	15.38	0.19
18.1% AEP	13.55	12.96	-0.60
39.4% AEP	9.92	8.82	-1.10
63.2% AEP	6.89	5.86	-1.03

Table 20: Difference in pre and post peak flow for the AEP events assessed at Location D

Change in peak flow at Location D (Culv0600) (cu.m/s)			
Event	Pre-development flow	Post-development flow	Difference
1% AEP+CC	2.33	2.04	-0.29
1% AEP	1.64	1.74	0.11
2% AEP	1.26	1.34	0.08
5% AEP	1.20	1.34	0.14
10% AEP	1.07	1.18	0.11
18.1% AEP	1.09	1.10	0.01
39.4% AEP	1.15	0.95	-0.20
63.2% AEP	0.98	0.97	-0.01

*Note that the post development flows for the hospital site are unmitigated. Proposed stormwater, swales and basins will need to be incorporated to reduce post development flows during the detailed design stage.

Table 21: Difference in pre and post peak flow for the AEP events assessed at Location E

Change in peak flow at Location E (Puie_Street) (cu.m/s)			
Event	Pre-development flow	Post-development flow	Difference
1% AEP+CC	125.54	125.39	-0.15
1% AEP	92.02	97.03	5.02
2% AEP	79.08	80.53	1.45
5% AEP	74.15	75.90	1.75
10% AEP	65.39	66.90	1.51
18.1% AEP	59.96	61.54	1.59
39.4% AEP	45.86	46.13	0.27
63.2% AEP	34.47	35.40	0.93

5.5.2 Flood Level Afflux Assessment

Flood level afflux mapping has been prepared to assess the change in flood levels around the precinct as a result of the precinct concept.

The impact of the concept development on pre-development flood levels in the 1% AEP+CC event is presented in Figure 31, on the 10% AEP event in Figure 32, and for the 63.2% AEP event in Figure 33. Flood level afflux mappings for all AEP events assessed are included in Appendix G.

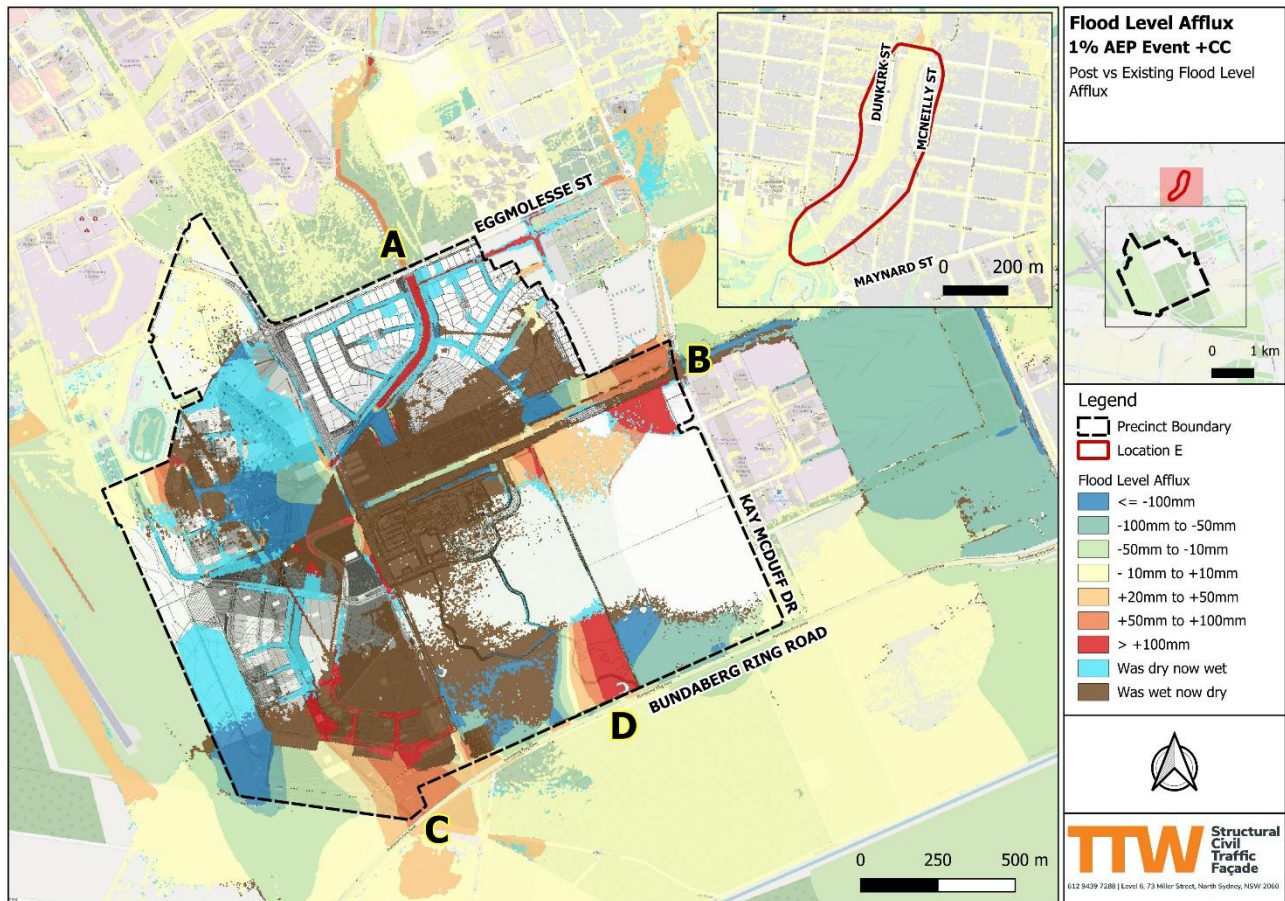


Figure 31: Impact of development on pre-development flood levels in the 1% AEP event with climate change

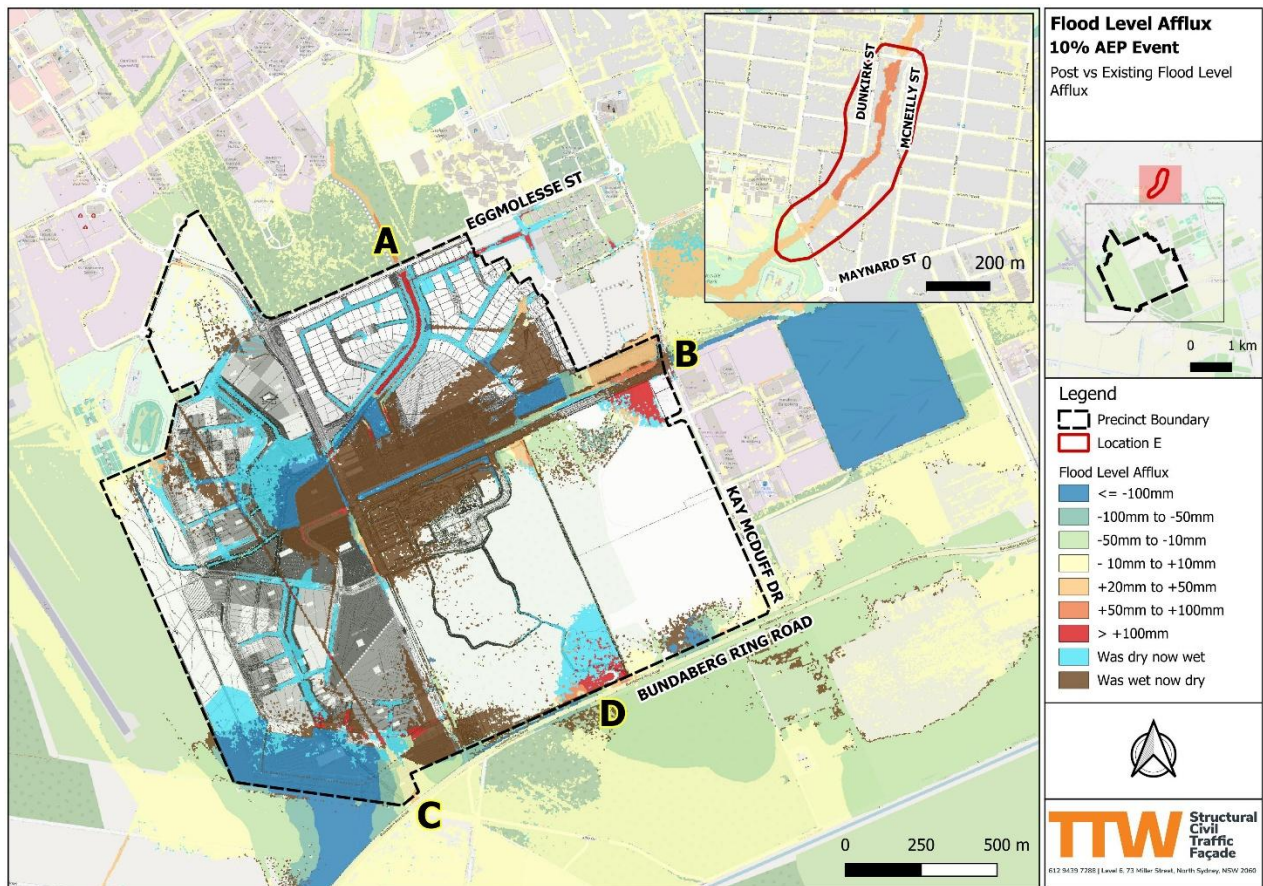


Figure 32: Impact of development on pre-development flood levels in the 10% AEP event

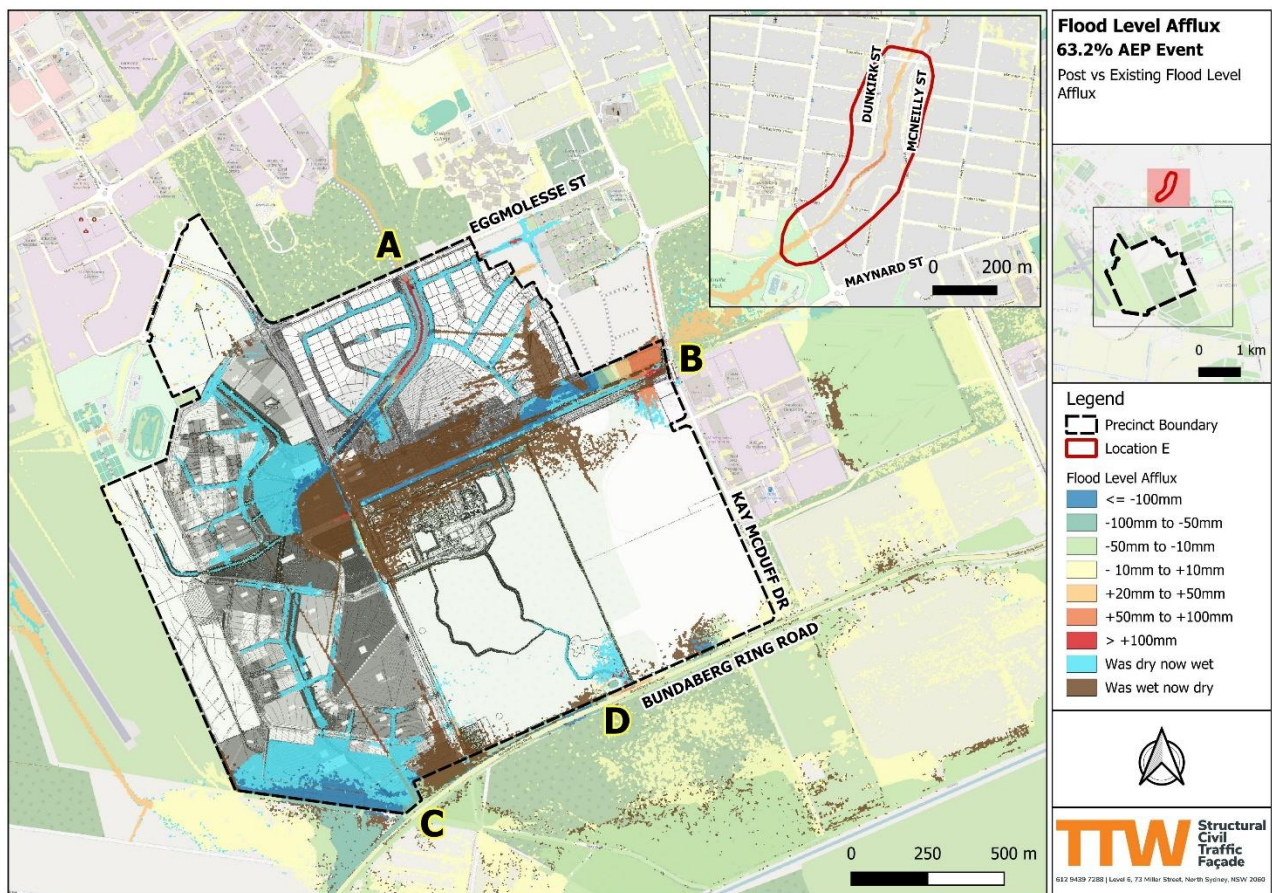


Figure 33: Impact of development on pre-development flood levels in the 63.2% AEP event

5.5.3 Velocity Assessment

Table 22 to Table 26 outline the change in the overall peak velocity for each AEP (i.e. the 'peak-peak velocity') from the pre-development scenario to the post-development scenario at Locations A to E.

Table 22: Difference in pre and post peak velocity for the AEP events assessed at Location A

Change in peak velocity at Location A (Culv0211) (m/s)			
Event	Pre-development velocity	Post-development velocity	Difference
1% AEP+CC	2.27	2.17	-0.10
1% AEP	2.25	2.13	-0.12
2% AEP	2.13	2.10	-0.04
5% AEP	2.12	2.07	-0.05
10% AEP	2.05	2.04	-0.01
18.1% AEP	1.95	2.03	0.08
39.4% AEP	1.75	1.98	0.22
63.2% AEP	1.57	1.90	0.33

Table 23: Difference in pre and post peak velocity for the AEP events assessed at Location B

Change in peak velocity at Location B (HP5) (m/s)			
Event	Pre-development velocity	Post-development velocity	Difference
1% AEP+CC	0.76	0.50	-0.25
1% AEP	0.73	0.47	-0.26
2% AEP	0.71	0.44	-0.27
5% AEP	0.70	0.41	-0.29
10% AEP	0.69	0.39	-0.30
18.1% AEP	0.65	0.38	-0.26
39.4% AEP	0.48	0.35	-0.13
63.2% AEP	0.37	0.31	-0.06

Table 24: Difference in pre and post peak velocity for the AEP events assessed at Location C

Change in peak velocity at Location C (Culv0445) (m/s)			
Event	Pre-development velocity	Post-development velocity	Difference
1% AEP+CC	1.83	2.02	0.19
1% AEP	1.72	2.00	0.28
2% AEP	1.62	1.76	0.14
5% AEP	1.42	1.49	0.07
10% AEP	1.25	1.27	0.02
18.1% AEP	1.12	1.07	-0.05
39.4% AEP	0.82	0.73	-0.09
63.2% AEP	0.70	0.62	-0.08

Table 25: Difference in pre and post peak velocity for the AEP events assessed at Location D

Change in peak velocity at Location D (Culv0600) (m/s)			
Event	Pre-development velocity	Post-development velocity	Difference
1% AEP+CC	0.77	0.67	-0.09
1% AEP	0.54	0.57	0.04
2% AEP	0.54	0.56	0.01
5% AEP	0.53	0.57	0.04
10% AEP	0.57	0.52	-0.05
18.1% AEP	0.56	0.52	-0.04
39.4% AEP	0.52	0.49	-0.03
63.2% AEP	0.52	0.51	-0.01

Table 26: Difference in pre and post peak velocity for the AEP events assessed at Location E

Change in peak velocity at Location E (Puie_Street) (m/s)			
Event	Pre-development velocity	Post-development velocity	Difference
1% AEP+CC	2.56	2.55	-0.01
1% AEP	2.54	2.54	0.00
2% AEP	2.59	2.59	0.00
5% AEP	2.53	2.56	0.03
10% AEP	2.51	2.53	0.02
18.1% AEP	2.47	2.49	0.03
39.4% AEP	2.43	2.49	0.06
63.2% AEP	2.26	2.29	0.03

5.5.4 Duration of Inundation at Location E

As noted in Section 1.3.2, an impact assessment of the precinct concept must demonstrate no increase in duration of above-floor inundation at Location E along Saltwater Creek.

This study assessed duration of inundation for the 360-minute storm (highest flood level) and the 270-minute (highest duration) for the 1% AEP+CC event. The lowest property floor level around Puie Street is approximately 13.5m, this is the inundation level taken as a reference.

A time-stage graph showing 1% AEP+CC flood level at Puie Street in the pre- and post-development scenarios is depicted in Figure 34 (360-minute duration) and Figure 35 (270-minute), while Table 27 and Table 28 outline the change in level and inundation time for the two design storms respectively.

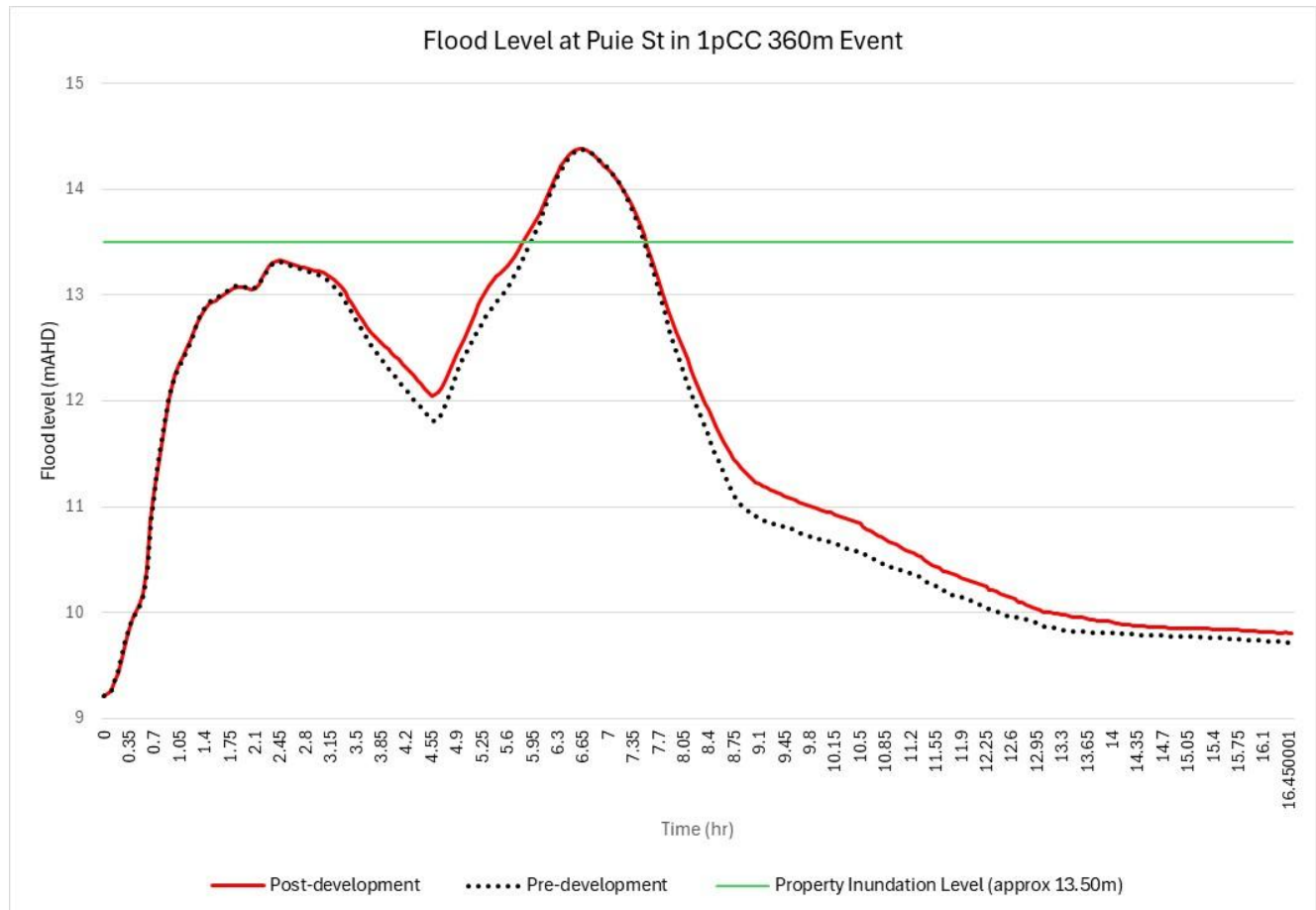


Figure 34: Time-stage graph outlining the pre- and post-development flood level at Puie Street in the 1% AEP+CC event, 360-minute storm duration

Table 27: Pre- and post-development above floor inundation in the 1% AEP+CC event, 360-minute storm

Above Floor Inundation Assessment 360 minute	
Pre-Development Peak Flood Level	14.379 m
Post-Development Peak Flood Level	14.382 m
Peak flood level difference	2.9 mm
Pre-Development Inundation Time	93 mins
Post-Development Inundation Time	99 mins
Additional inundation time	6 mins

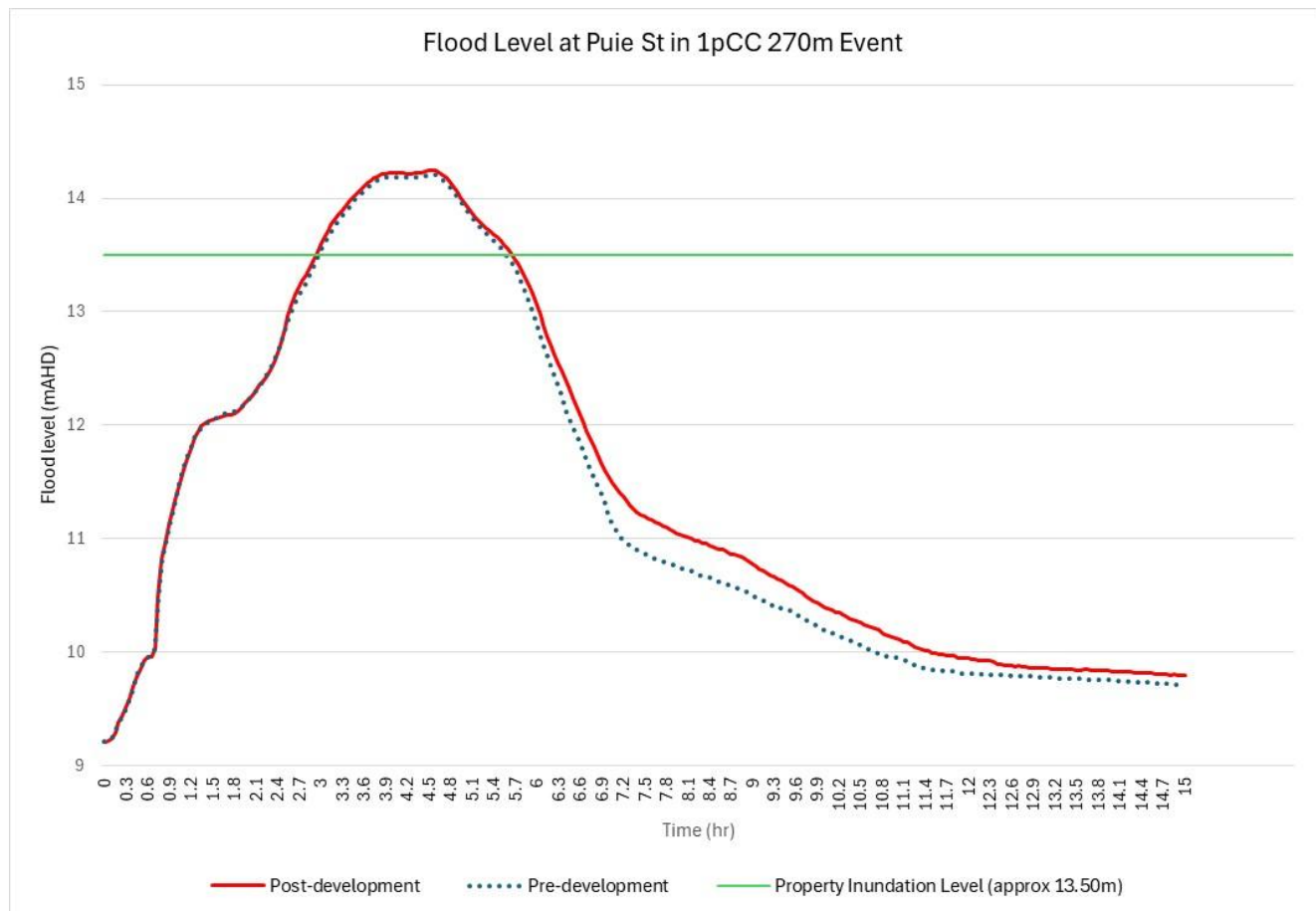


Figure 35: Time-stage graph outlining the pre- and post-development flood level at Puie Street in the 1% AEP+CC event, 270-minute storm duration

Table 28: Pre- and post-development above floor inundation in the 1% AEP+CC event, 270-minute storm

Above Floor Inundation Assessment 270 minute	
Pre-Development Peak Flood Level	14.208m
Post-Development Peak Flood Level	14.245m
Peak flood level difference	36.7mm
Pre-Development Inundation Time	153 mins
Post-Development Inundation Time	162 mins
Additional inundation time	9 mins

The above analysis found that the additional inundation time of flood affected properties in the 1% AEP+CC event is between 6 to 9 minutes. This can be further improved with some adjustment to the outlet configuration of the upstream detention system as well as the provision of additional storage. It is worth noting that the current modelling has adopted a very preliminary (i.e. concept in nature) precinct wide development design, hence it is likely that proper design might yield additional storage capacity in the upstream detention system. Additional modelling will be required in the detailed design phase and care should be taken to ensure that the ultimately adopted outlet configuration of the upstream detention system should result in acceptable outcome across the range of AEP events assessed, not only the 1% AEP+CC event. Given the existing inundation time (between 1.5 to 2.5 hours), this is not considered a substantial or unreasonable increase and therefore meets the criteria of no actionable nuisance.

6.0 Conclusion

This report presents a comprehensive precinct-wide hydraulic and hydrologic assessment of the proposed Bundaberg Health and Enterprise Precinct, undertaken to support future development of the precinct while meeting Bundaberg Regional Council's flood and stormwater performance requirements. The assessment incorporated a full range of design storm events, from 63.2% AEP to 1% AEP including climate change, with particular emphasis on downstream flood risk within the Saltwater Creek catchment.

A combined modelling approach was adopted using Council's TUFLOW Citywide Overland Flow Model and a precinct-scale DRAINS model. The DRAINS model was calibrated against TUFLOW for pre-development conditions and demonstrated good agreement, with peak flows generally within $\pm 10\%$. This provided confidence in the use of the DRAINS model for testing post-development stormwater strategies, detention basin sizing, and grading concepts prior to detailed hydraulic verification.

The assessment confirms that the proposed stormwater and grading concept can be implemented across the precinct while broadly maintaining or improving existing flood behaviour. Local precinct detention basins, controlled discharge structures, and coordinated site grading have been shown to effectively manage increased runoff associated with urbanisation, limit peak discharges, and reduce the coincidence of peak flows across sub-catchments. In most cases, post-development peak flows from the precinct are lower than pre-development conditions across the full range of storm events. However, some increases are observed especially for the more frequent AEP events assessed. This can be further improved with some adjustments to the detention systems' outlet configuration adopted in the current model to throttle flows leaving the detention system (i.e. storing more stormwater runoff) for the more frequent AEP events.

An impact assessment was undertaken at five key locations around and downstream of the precinct, including the sensitive residential area along Saltwater Creek. Results indicate that the proposed development does not result in substantial or unreasonable increases in peak flood levels, peak flows, flood velocities, or duration of inundation and therefore meet the criteria of no actionable nuisance. Minor increases were identified at certain locations and storm events, including small increases in peak flow and flood level during extreme events; however, these increases are considered negligible in magnitude and do not constitute actionable nuisance as defined by Council.

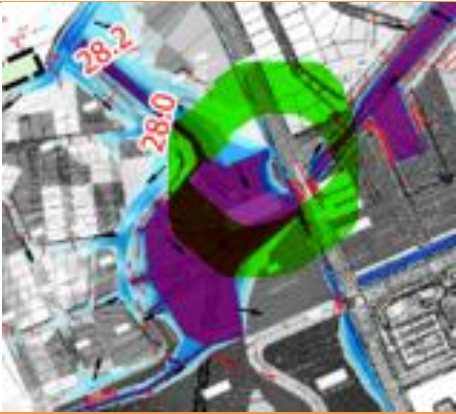
At Saltwater Creek, where flood risk to existing residential properties is a key concern, modelling demonstrates that post-development run off is generally restricted to pre-development flows for the 1% AEP+CC event. Very small increases in peak flood level (in the order of millimetres) and short increases in inundation duration were identified for the most extreme events; however, these changes are not considered significant and are unlikely to materially alter flood risk or property impacts. Further refinement of outlet structures during detailed design provides additional opportunity to further minimise these effects if required.

Localised areas of post-development inundation within the precinct were also identified in the concept modelling. These are largely attributable to the preliminary nature of the design surface, incomplete representation of internal road networks, and conservative placement of hydrologic inflows within the hydraulic model. These matters are considered resolvable during the detailed design stage through refinement of grading, drainage layouts, detention basin geometry, and hydraulic controls. Further detailed modelling will be required to finalise basin batters, storage areas, vegetation treatment, and flood immunity levels for roads, lots, and public open space areas. Further, the modelling results have shown that the timing of flows also play an important role in not causing increases in peak flood levels especially along the Saltwater Creek. Therefore, additional modelling carry out in the detailed design phase should take this into consideration and assess against each AEP event.

Overall, the assessment demonstrates that the Bundaberg Health and Enterprise Precinct can be developed in a manner that is consistent with Council's flood and stormwater objectives, while appropriately managing downstream flood risk under both current and future climate conditions. Subject to further refinement during detailed design and final verification modelling, the proposed stormwater and grading strategy provides a robust and practical framework to support the staged delivery of the precinct.

Further refinement during detailed design stage

Location	Description	Potential Solution/Cause	Location Plan
1. Health and knowledge lot west of hospital	Slight flooding on eastern edge of the health and knowledge lot adjacent to roadside drain	Roadside drain adopted in the modelling may not have sufficient capacity. This can further look at and design accordingly in design phase to ensure flows will be fully contained in the drain.	
2. Pressler residential lots at the north-east	Flood inundation occurs on part of these residential lots	Some of the inundation is due to the location of the inflows being placed in the model as a result of lack of internal roads in the design DEM adopted. Inundation of some middle part of lots is due to incomplete triangulation of tin resulting in some gullies being form within the lots. The western side of these lots (i.e. adjacent to waterway) should also potential be lifted to ensure the required immunity is achieved.	
3. Edenbrook stormwater treatment and detention basin	Basin looks full and overflowing onto Kay McDuff Dr in the 1% CC event, potentially causing the impacts observed further downstream on the north-east	The capacity and design of the basin should be further looked at to allow additional flows anticipated for a future fully developed catchment scenario. This is to ensure that the system has the adequate capacity to accommodate the 1% CC design flows anticipated from the contributing catchments into the future.	
4. Vacant lot east of proposed Bundaberg hospital site and south of East-West Channel	Wide-spread flooding is currently observed on the lot	The lot has been assumed undeveloped in the modelling and modelled with no changes to surface elevations, hence resulting in wide-spread flooding on the lot. A post development DEM can be included in the modelling to minimise flooding area on the lot at the alter design phase and hydrology for the area should be adjusted accordingly to reflect the ultimate land use density and adopted in the TUFLOW modelling.	

5. North Pressler Detention Area	Flood free within the north-eastern corner of the detention area	This is mainly due to the incomplete triangulation of the area and resulting in loss of additional storage in the model. Adjustment to the DEM can be made to fix the issue and allow additional storage area and volume to be modelled in the design phase. This should provide further attenuation of flows to the downstream area.	
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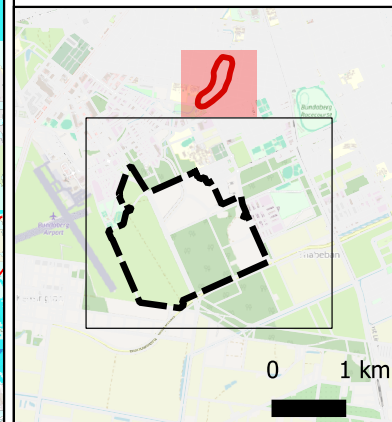
Associate Director (Flood)
Eirian Crabbe

Appendix A - 1% AEP +CC Flood Maps (Council Model)

Peak Flood Levels & Depths

1% AEP Event +CC

Existing Scenario

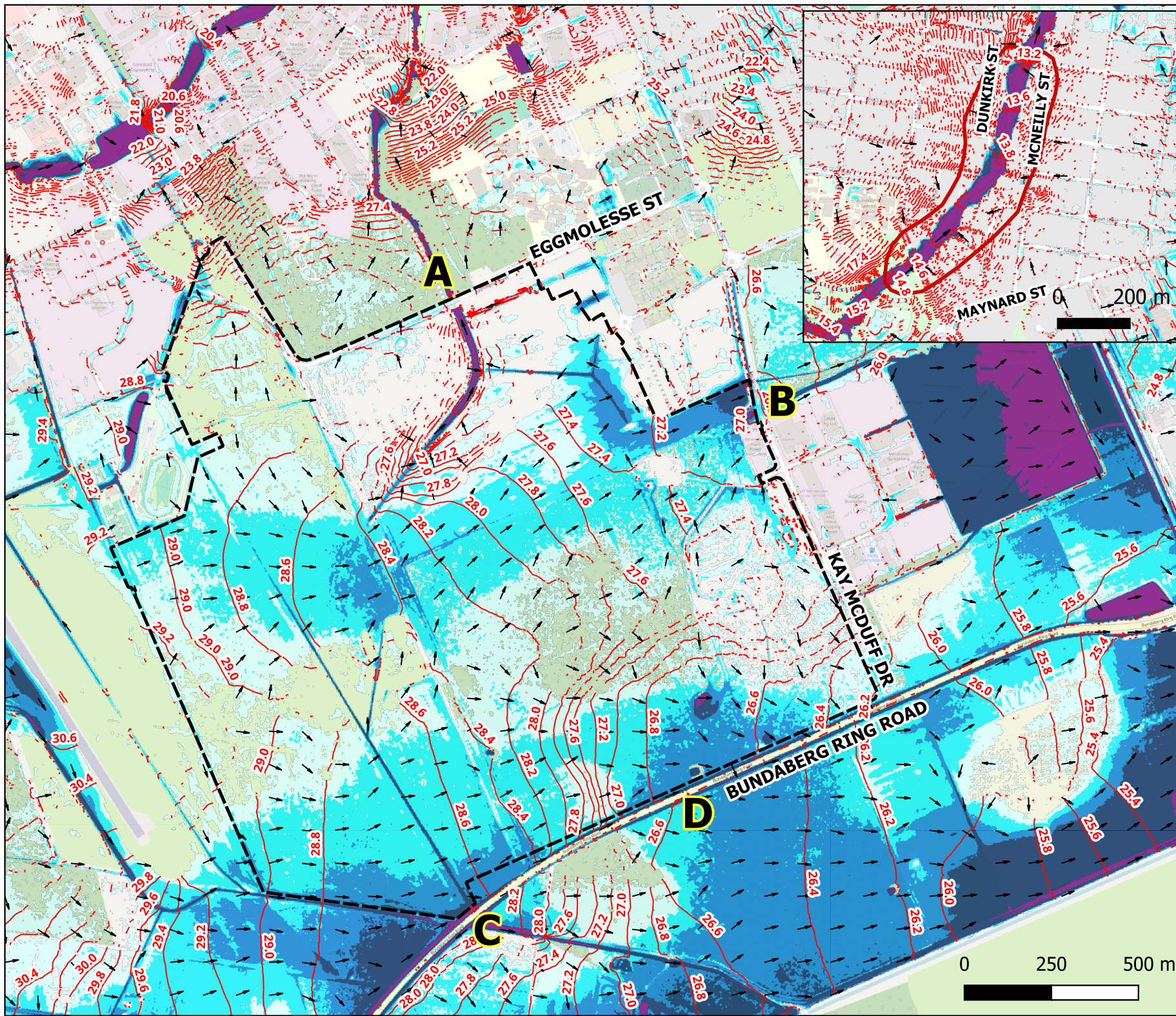


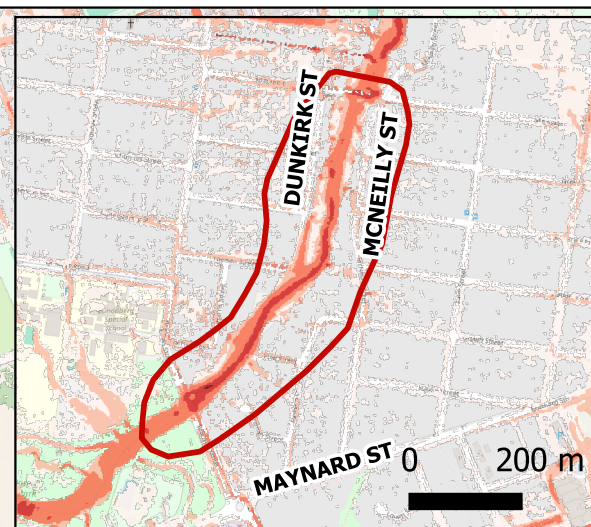
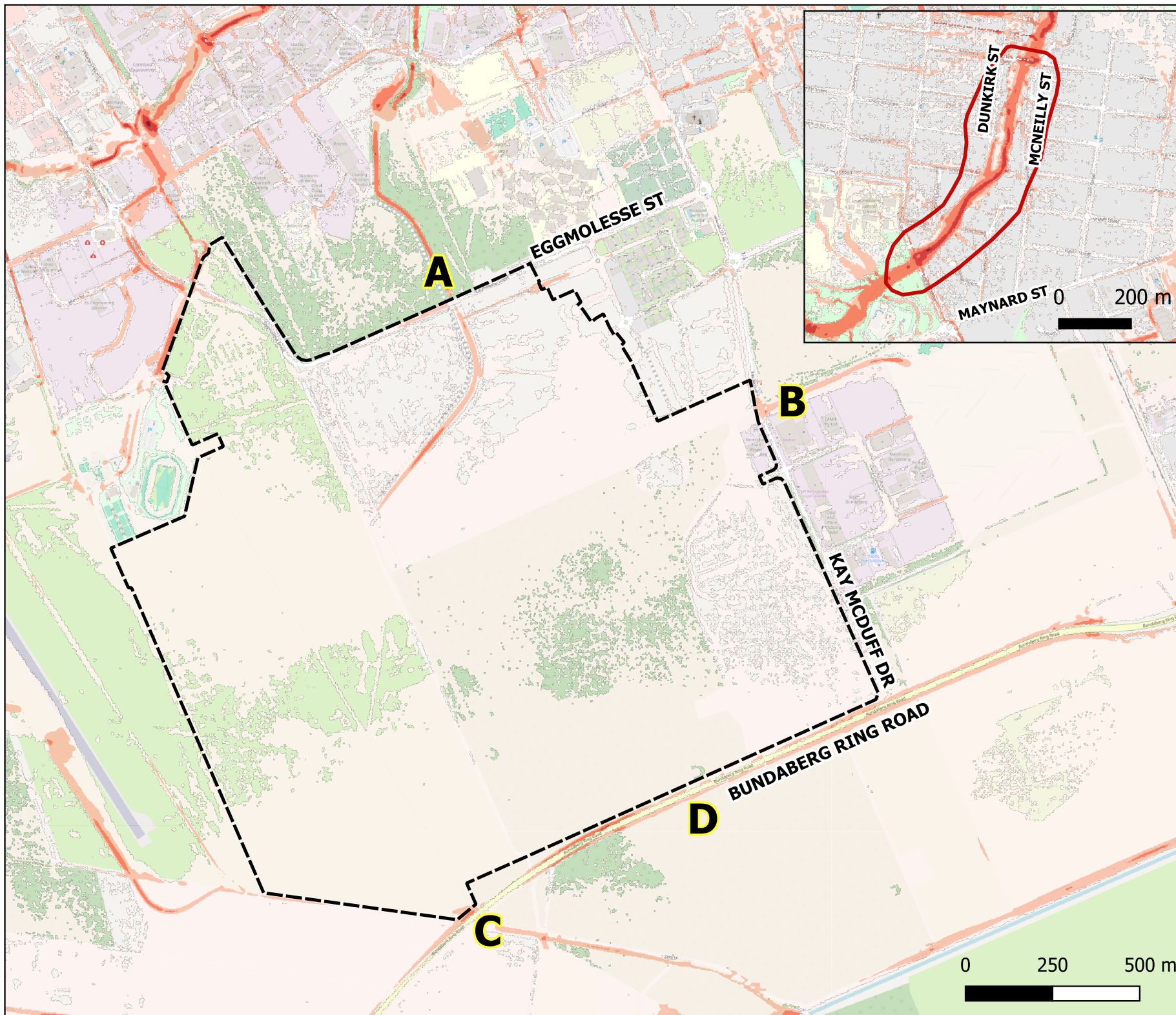
Legend

- Precinct Boundary
- Location E
- Flood Level (mAHD)

Flood Depths (m)

- 0.05 - 0.15
- 0.15 - 0.3
- 0.3 - 0.5
- 0.5 - 1.0
- > 1.0

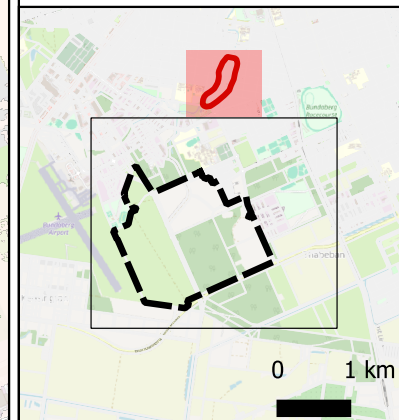




Peak Flood Velocity

1% AEP Event + CC

Existing Scenario



Legend

 Precinct Boundary

 Location E

Flood Velocity (m/s)

 ≤ 0.50

 0.50 - 1.00

 1.00 - 2.00

 2.00 - 4.00

 > 4.00

