BYRON SHIRE COUNCIL

Byron Bay Drainage Scheme Concept Design Flooding Assessment Report

QC2003_002-REP-003-1

7 NOVEMBER 2023

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Rev	Date	Description	Author	Reviewer	Project Mgr.	Approver
0	28/09/2023	75% Draft	Sean Willis Jesse Hunter	Mark Page Tim Randell	Tim Randell	Mark Page
1	07/11/2023	100% Final	Jesse Hunter	Tim Randell	Mark Page	Mark Page
Signatures:						

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1. INTRODUCTION

1.1 Scope of Works

Engeny Australia (Engeny) has completed a flood assessment as part of the concept design phase of the Byron Bay Drainage Upgrade Design Project. The scope of works for the flood assessment included:

(1) Reviewing the existing preferred drainage strategy, which was last modified in 2015 (BMT WBM, 2015) *(Completed May 2023).*

(2) Undertaking detailed ground survey across the catchments of interest to inform the design. *(Completed July 2023).*

(3) Developing concept design of the schemes to inform Council's next steps for the works. *(This report).*

A concept engineering report containing concept design drawings (Engeny, September 2023) were also prepared to support the proposed drainage upgrade strategy and should also be refereed to when reading this flooding assessment (QC2003_002--REP-004. Engeny, September 2023).

1.2 Purpose of this Report

This report addresses the flooding assessment (BMT, 2015) and concept design of the Byron Bay drainage upgrade design project, which has comprised a comprehensive update of Byron Shire Council's (Council's) Preferred Byron Bay drainage strategy. This was based on the preferred drainage strategy (BMT WBM 2015), which was an amended version of a strategy proposed in 2010 (SMEC, 2010).

This report should be read in conjunction with the concept engineering report (ref. QC2003_002-REP-004-1) which investigates in more detail the engineering requirements of the drainage works and pump stations to achieve the flooding outcomes described in this report.

The objective of the concept design was to develop the preferred drainage strategy in more detail to inform decision making regarding the following:

- Identify infrastructure upgrade requirements to achieve a 10% AEP flood immunity standard of service.
- Overall project feasibility in terms of flooding, and engineering limitations.
- Project objectives and impact areas.
- Project benefits.
- Flood damage costs.
- Capital costs of construction.

This report outlines items (a) to (g) as listed below. The concept engineering report (QC2003_002-REP-004) outlines items (i) to (k) as listed below.

1.2.1 Concept Design Approach

A summary of the adopted approach and tasks completed for the drainage strategy review are as follows:

(a) Topographic survey update.

(b) Update hydrological model.

(c) Update hydraulic model incorporating detailed drainage survey.

(d) Hydraulic model validation.

(e) Design event 1D/2D flood modelling.

(f) Drainage scheme implementation modelling and design refinement (12d / TUFLOW).

(g) Selection of concept drainage scheme.

(h) Flood damages assessment.

(i) Engineering assessment on preferred drainage scheme.

(j) Identification of constraints, issues and further work on the proposed scheme.

(k) Costing of the scheme.

Considerable design iterations were completed for key components of the scheme such as the drainage networks and proposed pump stations. The objective of the concept design is to demonstrate the potential benefits and level of service that can be achieved, and simple refinement of key components. It is expected that significant additional optimisation, refinement, and potential staging/prioritisation of the scheme will be undertaken in future design phases.

1.3 Background Information

Byron Bay township is susceptible to inundation from:

- Localised overland flooding resulting from intense, short-duration storms over the low lying and relatively flat town catchment,
- Longer duration regional flooding over the Belongil Creek catchment,
- Ocean storm tide (storm surge plus high tide) events (which influence both the Belongil Creek estuary and Clarkes Beach stormwater outfall), and
- Belongil Creek opening (ICOLL) outlet issues.

Peak flood levels across the township can occur within 1 to 6 hours of commencement of significant rainfall and is significantly influenced by high tides coinciding with intense rainfall events and Belongil Creek flooding.

Byron Bay has old underground drainage infrastructure which has two principal points of discharge which are:

1) West to Belongil Creek via the Byron Bay town drain (Butler Street Drain), and

2) North to the Clarkes Beach outlet (1200mm diameter pipe).

Rainfall runoff within the township of Byron Bay, east of the North Coast Railway line does not have direct overland escape and water levels can be independent of the flooding from Belongil Creek, due to the constrained stormwater drainage system. However, high creek levels due to either Belongil Creek catchment flooding, creek mouth closure or high ocean levels can also back up into the town drainage system which can significantly impact stormwater discharge from the township.

Previous studies have determined that the local stormwater network in Byron Bay has limited capacity (estimated capacity of less than 63% AEP) and is often unable to adequately manage local stormwater runoff. Stormwater discharge is limited by a flat hydraulic gradient between the low-lying township, Belongil Creek floodplain and beach outlets. The Belongil Creek catchment and key drainage features are illustrated in Figure 2.2 and Figure 2.3.

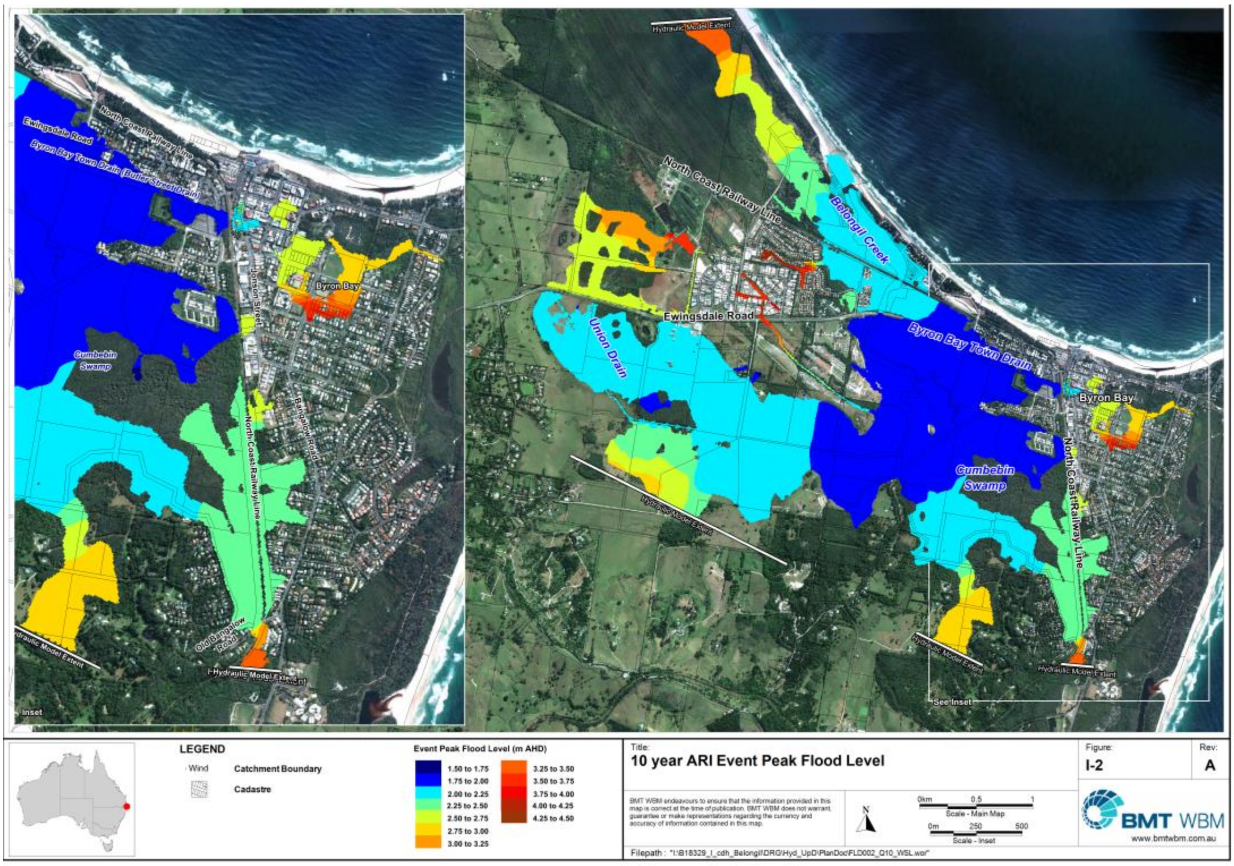

FIGURE 1.1: 10% AEP EVENT PEAK FLOOD LEVEL (BMT, 2015) (BELONGIL CREEK FLOODPLAIN RISK MANAGEMENT STUDY AND PLAN)

In order to improve drainage to the most impacted areas of the township, the strategy identified a variety of stormwater upgrades across several areas of the town. These schemes focused on improving drainage in the following areas:

Belongil Creek Catchment

- Shirley Street and Byron Street adjacent to the Butler Street Drain.
- Jonson Street and Byron Street in the town centre, which drains to the Butler Street Drain.

Clarke's Beach Outfall Catchment

- Middleton Street area.
- Cowper Street area.

Due to the low ground levels and gradients through these areas relative to ocean and Belongil Creek levels, the scheme relies on flood levees and stormwater pumping systems to manage trapped stormwater. Additionally, the Cowper Street catchment scheme mitigation recommended a combined detention basin/wetland and drainage upgrade as well as duplication of the existing 1200mm diameter stormwater outfall pipe to Clarkes Beach.

The holistic drainage scheme was last updated in 2015 and had high level feasibility investigated from a flooding perspective. The scheme requires robust review and update in context of engineering feasibility and cost, Councils adopted strategic plans and engineering standards and general community acceptance.

2. PROJECT INPUT DATA

2.1 Project Data

BSC provided electronic data that has been used to create a GIS database (using QGIS software) of the catchment, which included:

- Cadastral Data and road reserve layout.
- Potable and recycled water Infrastructure.
- Floor level data base (2016) Infrastructure.
- Drainage Infrastructure.
- Significant tree register.

The LiDAR and aerial photography adopted for the study was:

- 1m digital elevation model (DEM) of existing topography sourced from ELVIS, dated October 2010.
- Aerial photography sourced from Nearmaps, dated November 2022.

2.1.1 Detailed Topographic Survey

Detailed survey of the drainage scheme areas was required to inform the concept design. A wide area across the township was surveyed over a period of approximately 3 months from April to July 2023 by Bennett and Bennett Surveyors. The survey consisted of a three-dimensional scan, traditional ground pickup and drainage survey to provide a detailed model of the road reserves and crown land relevant to the drainage upgrade. No survey of private property was included in the scope.

Surface features are well defined in the survey data, however due to the age and construction of some of Council's underground drainage assets, there were limitations on the survey detail of some sewer stormwater systems due to access restrictions.

Generally, survey detail is high and appears to be of good quality. Areas where pipe levels and sizes were not able to be fully surveyed which may require further survey in future are:

- Cowper Street Trunk Drainage (inaccessible junction pits with no access openings)
- Middleton to Cowper Trunk Drainage (inaccessible or buried junction pits with no access openings)
- Area south of Middleton Street Courthouse (low lying area with overgrown swampy conditions and standing water unable to comprehensively be surveyed)
- Byron Street trunk drainage (some inaccessible junction pits with no access openings.

In these areas, assumptions on connectivity, levels and pipe sizes were made with considerable confidence that the assumptions will not have a major impact on the outcome of the strategic approach.

Survey was undertaken to MGA2020 coordinate system and to Australian height datum.

2.1.2 Tide Levels

A summary of the tide levels in the study area are outlined in Table 2.1.

TABLE 2.1: TIDAL PLANES FOR THE TWEED-BYRON REGION DERIVED FROM TIDAL CONSTITUENTS (BMT WBM, 2013)

Tidal plane	Level (m AHD)
Highest Astronomical Tide (HAT)	Approx. 1.0 – 1.1
Mean High Water Springs (MHWS)	0.66
Mean High Water Neaps (MHWN)	0.37
Mean Sea Level (MSL)	0.0
Mean Low Water Neaps (MLWN)	-0.37
Mean Low Water Springs (MLWS)	-0.66
Lowest Astronomical Tide (LAT)	-1.0

2.1.3 Ocean Storm Tide Levels

Storm tide levels were defined and consistently adopted through both the SMEC (2010) study and BMT (2015) update for the Belongil Creek Flood Study. The peak tailwater levels are summarised in Table 2.2 and these have been adopted at the boundary of the new local model developed for the concept design. The levels consist of the combination of highest astronomical tide, barometric setup, wind and wave setup with significant wave.

The previous regional flood model (BMT, 2015) adopted a time varying boundary condition accounting for astronomical tide variation through a 40-hour simulation time as shown in Figure 2.1. As the regional model critical storm is a 12-hour duration, the tidal variation has a significant impact on simulated flood levels based on the timing of the tidal sequence at the start of the simulation.

Due to the significantly shorter critical durations associated with small urban catchments the timing of high tide can significantly impact on local flood conditions. Therefore a conservative approach to the design has been taken by adopting the ocean storm tide level (defined as the storm surge allowance plus high tide level), as the design boundary condition. This is particularly conservative in Belongil Creek, where in some cases flood planning levels are lower than the ocean storm tide levels adopted (refer Section 2.1.4.1) due to the hydrodynamics of the Belongil Creek estuary.

Opportunity exists to refine and reduce the tailwater levels slightly in Belongil Creek if further information or advice is agreed with Council on the design standard of the system.

TABLE 2.2: ADOPTED STORM TIDE LEVELS

Event	Peak Level
1% AEP	2.29m AHD
5% AEP	2.19m AHD
10% AEP	2.12m AHD
18% AEP	2.04m AHD

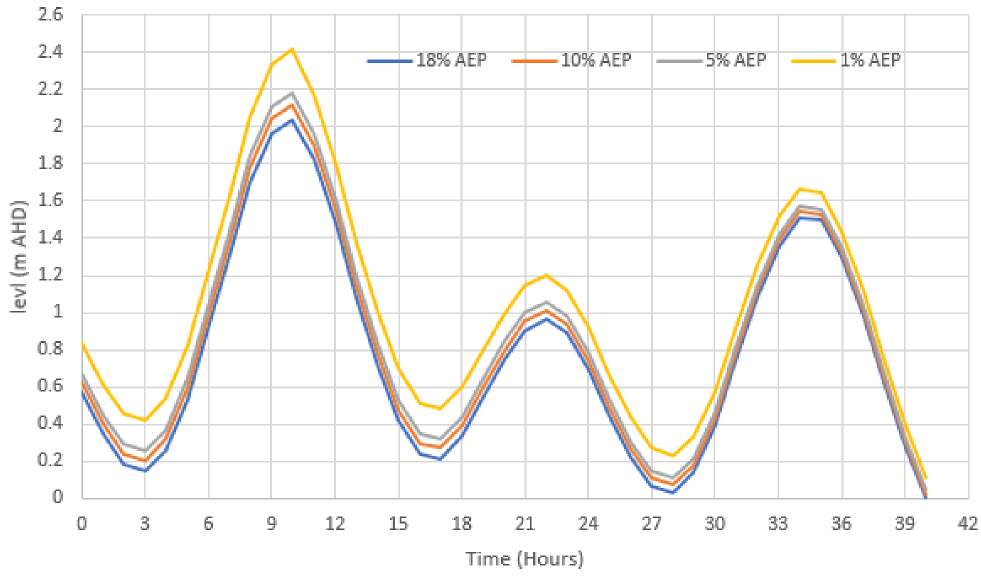

Figure 2.1: Model Boundary Condition (BMT 2015, SMEC 2010)

2.1.4 Belongil Creek Flood Levels

Belongil Creek estuary is defined as an intermittent closed and open lake or lagoon (ICOLL) system. The Belongil Creek outlet to Belongil Beach often closes due to accumulated sand from coastal processes. Due to various water quality issues from contaminants within the Belongil Creek catchment such as to Byron Bay STP treated effluent and urban pollutants, creek flows are often contaminated and discharge to the ocean has known environmental and community impacts.

Council holds a NSW marine park permit (MEAA22/32) for management of artificial opening of the ICOLL to minimise environmental impacts of creek flows regularly flowing to the beach. Council is permitted to artificially excavate a creek outlet to the ocean when the level in the estuary is 1.0m AHD or higher as measured at the Ewingsdale Road bridge provided there is forecast or actual rainfall of 20mm within the Belongil Creek catchment within 24 hours following the expected opening time.

The status of the creek outlet opening has a significant influence on water levels in the Butler Street drain and local drainage adjacent to Belongil Creek. This is a critical consideration to the design of flood protection systems, as under certain circumstances when license conditions are not met, Belongil Estuary levels can be independent of and much higher than typical tidal variations. This can occur during days of repeated small rainfall totals below the 20mm threshold and therefore the estuary level can be high in relatively dry conditions independent of both ocean influence and immediate storm influences.

Belongil Creek ICOLL has been reported to operate in a range of levels from 0.2m AHD at low tide / open outlet conditions to as high as 2.0 – 2.5m AHD during closed outlet and small rainfall events that prohibit artificial mouth opening (Alluvium 2021). It is not unusual for relatively fixed levels of approximately-1.5m AHD to occur within the estuary for days at a time based on the monitoring data at Ewingsdale bridge. This requires consideration in design of appropriate design tailwater levels, levee design height and sensitivity testing of the system under high ICOLL levels.

2.1.4.1 Flood Planning Levels

The regional flood model (BMT, 2015) has identified a range of flood planning levels in Belongil Creek using the calibrated flood model. These are summarised at the Butler Street drain point of interest in Table 2.3.

TABLE 2.3: BUTLER ST FLOOD PLANNING LEVELS IN METRES AHD (BMT 2015)

Location	10% AEP	1% AEP	1% AEP 2050 Climate	1% AEP 2100 Climate	1% AEP 2050 Horizon FPL 1	1% AEP 2100 Horizon FPL 1
Butler Street Drain *	1.99	2.32	2.32	2.61	2.82	3.11
Jonson Street	2.24	2.36	2.35	2.60	2.86	3.10
Middleton Street	2.65	2.94	2.94	2.94	3.44	3.44
Cowper Street	2.94	3.18	3.18	3.18	3.68	3.68

** Note this Flood Planning Level is 0.13m lower than the design storm tide tailwater level adopted in the concept design for the 1:10 AEP design event*

3. HYDROLOGIC MODELLING

3.1 Introduction

To inform the strategy review, a new hydrological model has been developed for the Byron Bay township. This is required to better capture the detailed drainage behaviour within the township and its interaction with the Belongil Creek floodplain and ocean levels. The existing XP-RAFTS hydrological model had only received minor updates since it was originally developed by SMEC (2009) as part of BMT WBM (2011) update. Since this time, the industry standards for hydrological modelling have changed significantly, the software that was originally used for the Belongil Creek hydrological study has become outdated and the focus area of this study no longer aligns with the modelling of the entire Belongil Creek catchment. A summary of the modelling history is contained in the strategy review report (Engeny, 2023).

The following sections describe the modelling approach and the key tasks and assumptions adopted in the development of a new hydrological model used for this assessment.

3.2 Modelling Approach

As the existing hydrological models were originally developed for the purpose of the regional Belongil Creek Flood Study, they lack sufficient detail to accurately model the Byron Bay Township for the purpose of drainage design. A new hydrological model was developed to represent small local drainage catchments.

The runoff – routing software Watershed Boundary Network Model (WBNM) was used to replace the XP-RAFTS software used in previous studies due to XP-RAFTS becoming no longer supported as a standalone software package. The software provider ended updates for XPRAFTS in 2020 and could not be used for this study.

The existing catchment conditions for the Byron Bay township were modelled to provide runoff hydrographs for the local 50%, 18%, 10%, 5%, 2%, 1% AEP and PMF design events, as well as the March 2022 rainfall event. The WBNM was built in accordance with the Australian Rainfall and Runoff (ARR) guidelines (Ball et. al, 2019), as well as NSW's Office of Environment and *Flood Risk Management Manual* (2023).

The purpose of the initial model build documented in the following sections is to provide a robust modelling platform to allow design development and assessment of the drainage scheme to occur in future phases of the project. The model results presented in this report are based on the proposed concept design strategy and may be further refined in the detailed design phase of the project.

3.3 Model Inputs

3.3.1 Catchment and Sub-Catchment Delineation

A total of 166 sub-catchments were delineated for the Byron Bay Township and adjacent external catchments. This results in a total model catchment area of 10.27 km^2, with the Byron Township accounting for approximately 2.50 km^2. Figure 3.1 provides the overall model sub-catchment delineation and Figure 3.2 provides Byron Township specific sub-catchment delineation extent and catchment areas are summarised in Appendix A.

The catchment delineation for this assessment was limited to include only local catchments that relate directly to the drainage upgrades proposed in the strategy. Belongil Creek regional catchment hydrology was not considered in the new hydrological model. Regional flooding of Belongil Creek has been considered through selection of conservative Belongil Creek tailwaters in hydraulic modelling only.

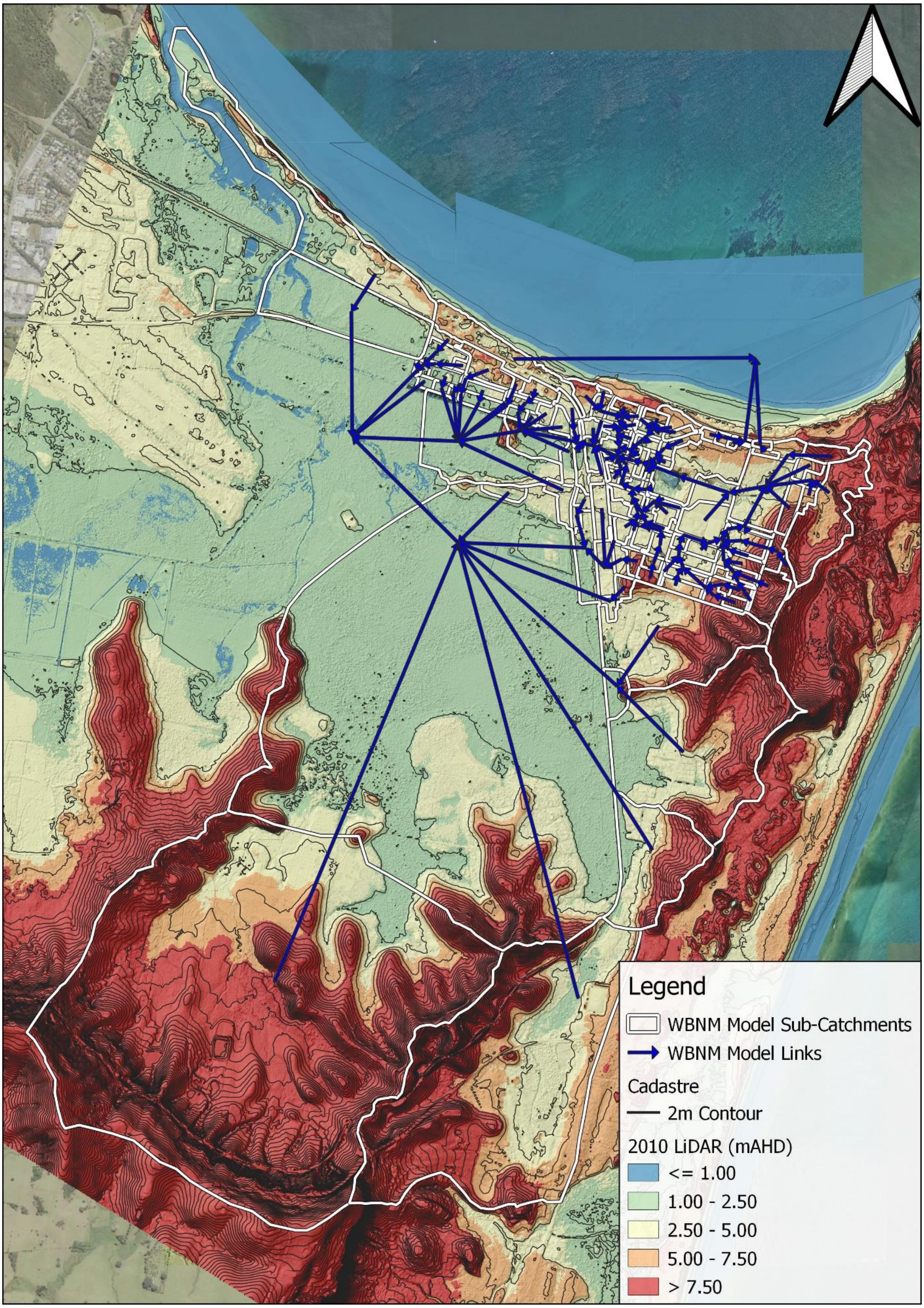

FIGURE 3.1: BYRON BAY REGIONAL SUB-CATCHMENT DELINEATION EXTENT

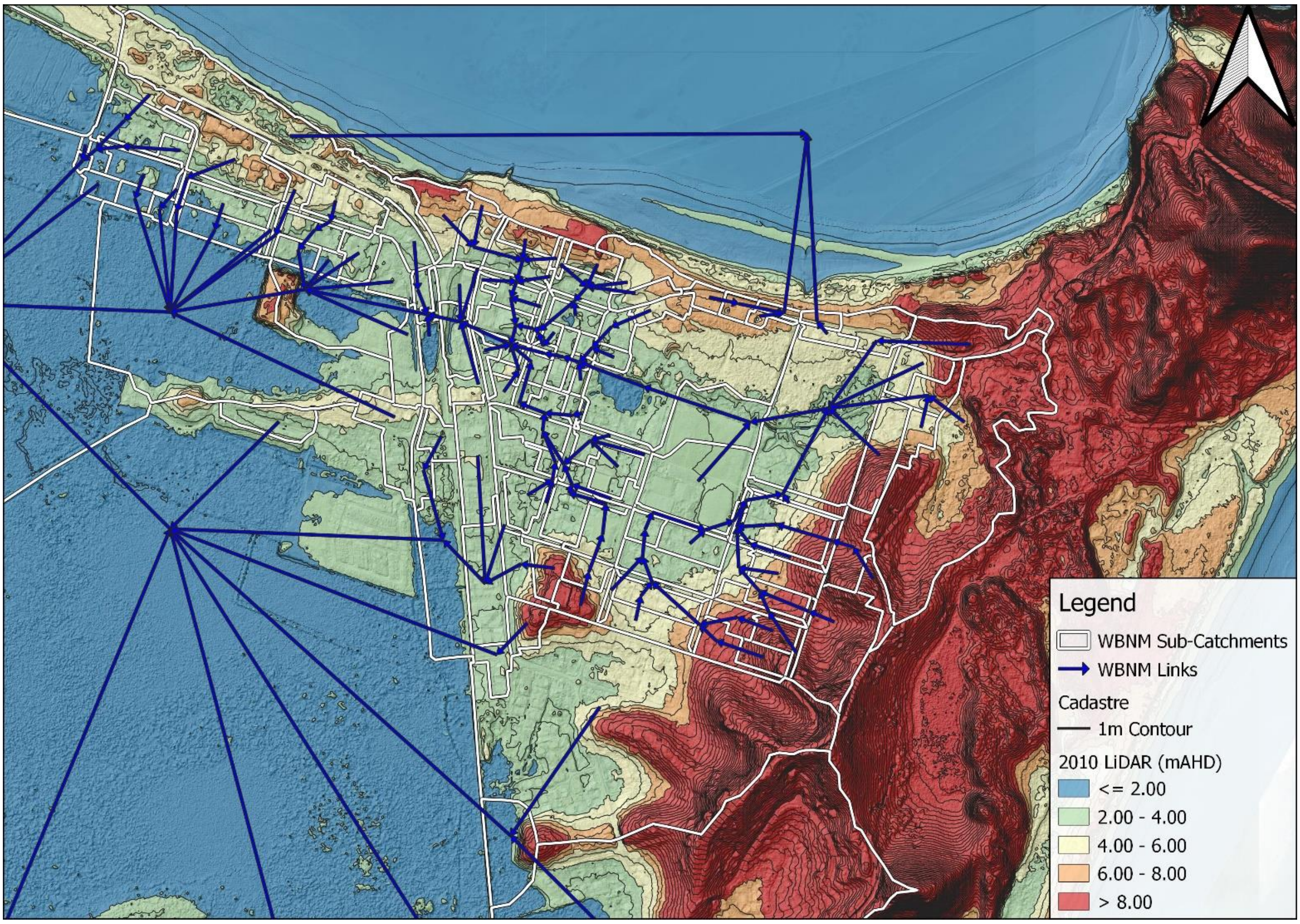

FIGURE 3.2: BYRON BAY TOWNSHIP SUB-CATCHMENT DELINEATION EXTENT

3.3.1.1 Fraction Impervious

The fraction impervious percentage was calculated for each catchment based of the Byron Local Environmental Plan 2014 (BSC, 2014), which has collated seven land use types corresponding to planning zones outlined in Table 3.1. This approach was adopted to ensure the drainage upgrades are sized to reflect potential future increases in fraction impervious within the catchment resulting from future development.

TABLE 3.1: LANDUSE TYPES AND FRACTION IMPERVIOUS

Land Use Type	Fraction Impervious (%)
Recreation and Open Space	0
Intensive agriculture / livestock	30
Rural Residential	20
Urban Residential	70
Industrial	80
Commercial	90
Waterbodies	100

The existing hydrological models for the Belongil Creek used aerial inspection derived impervious fractions, and the approach adopted for this study results in higher impervious fraction values overall. The impact of this approach was reviewed for areas around Shirley Street and the Town Centre to compare the impact on fraction impervious from alternative fraction impervious value from the aerial inspection approach against the ultimate land use approach. The changes are shown in Table 3.2, and demonstrate that the land use fraction impervious approach results in up to 19% higher values than the aerial inspection in areas that are not fully developed yet.

Note that since this assessment was originally completed, the hydrological catchments have been reshaped, however the relevant underlying fraction impervious values have remained consistent.

TABLE 3.2: LANDUSE TYPES AND FRACTION IMPERVIOUS

Catchment	Aerial Inspection Fraction Impervious (Current day)	Land Use Fraction Impervious (Ultimate developed per planning scheme)	Change
7A (Shirley Street)	45%	64%	19% increase
7C (Shirley Street)	48%	67%	19% increase
9B (Town Centre)	89%	89%	No change
9L (Town Centre)	76%	87%	11% increase

The finalised sub-catchment information for WBNM is provided in Table A.1.

3.3.2 WBNM Model Inputs

The following section below provide further information regarding rainfall hydrological inputs used for this study.

3.3.2.1 ARR 2019 Data Hub Parameters

3.3.2.1.1 Intensity-Frequency-Duration (IFD) Data

ARR 2019 IFD data for Byron was sourced from the Bureau of Meteorology using the online 2019 Rainfall IFD request system. Data was requested for the catchment centroid, represented by the coordinates of -28.6581, 153.6048. Refer to Table A.1 in Appendix A for the default IFD depths for the study area.

3.3.2.1.2 IFD data comparison ARR 1987 vs ARR 2019

To date, all modelling for the Belongil Creek study utilised the Australian Rainfall & Runoff 1987 (ARR 1987) guidelines, which have since been superseded by the ARR 2019 guidelines. One of the more significant updates introduced in this revision was the revision of IFD data based off more recent information.

Table 3.3 provides a breakdown of the percentage difference between the ARR 1987 and 2019 rainfall depths for the Byron Bay township. ARR2016 both on face value and following application of ARR pre burst and losses the ARR2016 rainfall data has generally lower values than the historic ARR1987 values (used by SMEC in the development of the original strategy).

TABLE 3.3: PERCENTAGE DIFFERENCE IN ARR 2019 AND ARR 1987 IFD VALUES (NEGATIVE VALUE INDICATES LOWER ARR 2019 VALUE)

Duration – AEP	63%	50%	18%	10%	5%	2%	1%	1:1000	1:2000
5 min	-9%	-10%	-11%	-8%	-8%	-8%	-8%	4%	0%
10 min	-9%	-11%	-12%	-9%	-9%	-9%	-9%	6%	3%
15 min	-10%	-12%	-13%	-10%	-10%	-10%	-9%	6%	3%
20 min	-11%	-13%	-14%	-11%	-11%	-10%	-10%	5%	2%
25 min	-12%	-14%	-15%	-12%	-12%	-11%	-10%	-1%	-3%
30 min	-13%	-15%	-16%	-13%	-12%	-11%	-9%	-7%	-10%
45 min	-13%	-15%	-16%	-12%	-11%	-9%	-7%	-2%	-5%
1 hour	-13%	-15%	-15%	-11%	-10%	-7%	-5%	-1%	-4%
1.5 hour	-12%	-13%	-13%	-9%	-6%	-4%	-1%	-3%	-6%
2 hour	-10%	-11%	-10%	-5%	-3%	0%	3%	1%	-2%
3 hour	-8%	-9%	-7%	-3%	0%	3%	6%	2%	-2%
4.5 hour	-9%	-10%	-11%	-8%	-8%	-8%	-8%	4%	0%
6 hour	-9%	-11%	-12%	-9%	-9%	-9%	-9%	6%	3%

3.3.2.1.3 ARR2019 Design Depth Modifications

It is understood that further work may be pending on the development of site-specific IFD rainfall for the region based on the March 2022 flood post event analysis, which may result in increases to the Byron Bay design IFD rainfall similar to those determined for other Northern Rivers locations (such as Lismore). There is a risk that the proposed drainage infrastructure is under-sized if the future site-specific IFD rainfall is greater than the design rainfall outlined in ARR 2019. Based on discussions with Council and DPE it was agreed that a 10% uplift is applied to the design rainfall to account for this uncertainty in the concept design This increased rainfall is observed to be generally consistent with the ARR 1987 intensities that were superseded in ARR 2019 ; however, these may require revision as future work is completed.

The adoption 10% rainfall uplift factor roughly aligns with:

- The original ARR 1987 data set used in the scheme development both before and after consideration of losses (SMEC, 2010).
- The IPCC RCP4.5 intermediate climate change scenario based on ARR guidance. (http://www.bom.gov.au/water/designRainfalls/document/Bates-et-al-2015b.pdf).
- The sensitivity 1 climate change scenario per the Byron Bay LEP.

The adopted IFD values ranging from the 63.2% (1-year ARI) up to the 1:2000 AEP (2000-year ARI) have been provided in Table 3.4.

TABLE 3.4: ADOPTED IFD VALUES (ARR2019 + 10%)

Duration – AEP	63%	50%	39%	20%	18%	10%	5%	2%	1%	1:1000	1:2000
5 min	10.10	11.33	12.54	14.96	15.29	17.49	19.91	23.21	25.74	33.44	35.86
10 min	16.39	18.26	20. 24	24.09	24.64	27.94	31.68	36.30	39.71	51.37	55.11
15 min	20.68	23.10	25.63	30.47	31.02	35.31	39.82	45.65	49.83	64.46	69.08
20 min	23.98	26.73	29.70	35.31	35.97	40.92	46.20	53.02	57.97	75.13	80.52
25 min	26.62	29.70	33.00	39.27	40.04	45.54	51.48	59.18	64.90	84.15	90.20
30 min	28.82	32.23	35.75	42.68	43.45	49.50	56.10	64.68	71.06	92.29	98.89
45 min	34.10	38.06	42.35	50.60	51.70	59.07	67.32	78.21	86.57	112.20	121.00
1 hour	38.06	42.57	47.30	56.87	58.08	66.66	76.34	89.21	99.33	129.80	138.60
1.5 hour	44.22	49.61	55.11	66.99	68.31	78.98	90.97	107.47	121.00	157.30	168.30
2 hour	49.17	55.33	61.38	75.13	76.67	89.21	103.18	123.20	138.60	179.30	192.50
3 hour	57.20	64.68	71.72	88.88	90.64	106.26	124.30	148.50	168.30	217.80	233.20
4.5 hour	67.10	76.12	84.48	105.93	108.02	127.60	149.60	179.30	203.50	262.90	282.70
6 hour	75.35	85.80	95.26	119.90	123.20	145.20	170.50	205.70	233.20	301.40	322.30

3.3.2.1.4 Temporal Rainfall Patterns

Point temporal patterns were used per AR2019 guidance as the Byron Bay model catchment is less than 75 km^2. East Coast (South) ARR 2019 temporal patterns, were adopted for the study.

3.3.2.1.5 Initial and Continuing Loss Model

AR&R Data Hub provides estimates for an initial loss of 27 mm and continuing loss of 2.1 mm/hr for pervious areas. These values were revised as detailed in Section 3.3.2.2.

An initial loss of 1 mm and continuing loss of 0 mm/hr for impervious areas was adopted for this study.

3.3.2.1.6 Pre-Burst Application

No standard ARR pre-burst rainfall depths have been used for this assessment as they are not available.

3.3.2.2 NSW Specific Guidelines

With the introduction of ARR 2019, NSW DPE reviewed the changes introduced and identified several issues as documented in their *Floodplain Risk Management Manual (2023)*. The department recommended making several modifications to the base ARR Data Hub outputs to address the issues. A hierarchical approach to loss and pre-burst estimation is required per OEH guidance, from most (1) to least (5) preferred, which is as follows:

(1) Use the average of calibration losses from the actual study on the catchment if available.

(2) Use the average calibration losses from other studies in the catchment, if available and appropriate for the study.

(3) Use the average calibration losses from other studies in the similar adjacent catchments, if available and appropriate for the study.

(4) Use the NSW FFA-reconciled losses available through the ARR Data Hub. These losses may be used within the catchment in which they were derived (available through the ARR Data Hub) or similar adjacent catchments with appropriate scrutiny. This is used with the unmodified ARR Data Hub initial losses which requires the application of additional scrutiny to the balance between initial loss and pre-burst to ensure it is reflective of flood history and observations for the catchment being investigated in the lead-up to events. This is particularly important in catchments of 100 km2 or less.

(5) Use default ARR data hub continuing losses with a multiplication factor of 0.4. This is used with the unmodified ARR Data Hub initial losses which requires the application of additional scrutiny to the balance between initial loss and pre-burst to ensure it is reflective of flood history and observations for the catchment being investigated in the lead-up to events. This is particularly important in catchments of 100 km^2 or less.

For the initial model simulations for the Byron Bay Township, Approach 5 has been utilised, as it provided the best initial model inputs for this assessment. The losses that were previously calibrated for the Belongil Creek Study were not adopted, as the focus of that assessment was on the catchment wide conditions. This requires the modification to the previously referenced AR&R 2019 Data Hub as discussed in the following sections.

3.3.2.3 Initial and Continuing Loss Model

New initial loss information was calculated in line with the previously discussed modification, to keep in line with the NSW guidelines. the adopted initial losses are provided in Table 3.5. Finally, the ARR 2019 continuing loss value of 2.1 mm/hr was factored by 0.4 as recommended by the NSW guidelines, resulting in a continuing loss value of 0.84 mm/hr being applied to the Byron Bay Township model.

TABLE 3.5: NSW PROBABILITY NEUTRAL BURST LOSS RAINFALL DEPTHS (MM)

Duration	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	1:200 AEP	1:500 AEP	1: 1000 AEP	1:2000 AEP
1 hour	24.2	13.2	12.3	12	10.8	7.7	3.8	1.5	0.8	0.4
1.5 hour	25.1	14.7	13.3	12	10.2	8.3	4.1	1.7	0.8	0.4
2 hours	22.4	13.4	13	11.1	10.3	6.1	3.0	1.2	0.6	0.3
3 hours	21.9	13.7	12.6	10.2	9.6	5.2	2.6	1.0	0.5	0.3
6 hours	19.9	12.8	12.2	10.9	11.1	3.5	1.7	0.7	0.3	0.2
12 hours	21.9	15.3	15	12.2	13.8	4.5	2.2	0.9	0.4	0.2
18 hours	25.8	19.1	19.6	14.9	16.7	5.4	2.7	1.1	0.5	0.3
24 hours	**29.9**	**21.7**	**21.4**	**16.5**	**14.3**	**5.9**	**2.9**	**1.2**	**0.6**	**0.3**

3.4 March 2022 Flood Event Analysis

From the morning of the 28th March to the 1st April, the Byron Bay LGA experienced significant rainfall that resulted in flooding within the Byron Bay Township area. The scale of the rainfall event was believed to have exceeded any event in previous years, even those used in the calibration of the original Belongil Creek Flood Study.

It is noted that DPE have commissioned a post-event flood study that was not completed or available at the time of writing of this report.

This event result in widespread damages to the Byron Bay Township, with flood survey markers collected by Council following the storm event. As a method of model validation, an analysis of the event has been conducted for this study, with the storm being first assessed within the hydrological model, to provide inflows for the hydraulic model to assess flood damages. No hydrological review of results has been completed, as the survey markers within the township have been used as the main form of review.

The Belongil Creek Bridge Alert Gauge (558099), the main rainfall gauge for the township, recorded a total rainfall depth of 596.5 mm between 12am 24 March to 12am 1 April 2022. This data was available in 15-minute increments and was used to assess the annual exceedance probability of the recorded rainfall for the March 2022 event. The analysis identifies the maximum rainfall depths for selected intervals (30 minutes, 1 hour, 1.5 hour, etc) taken from the recorded rainfall. A plot of the IFD depths for the Belongil Creek Bridge Alert Gauge versus the March 2022 event have been provided in Figure 3.3.

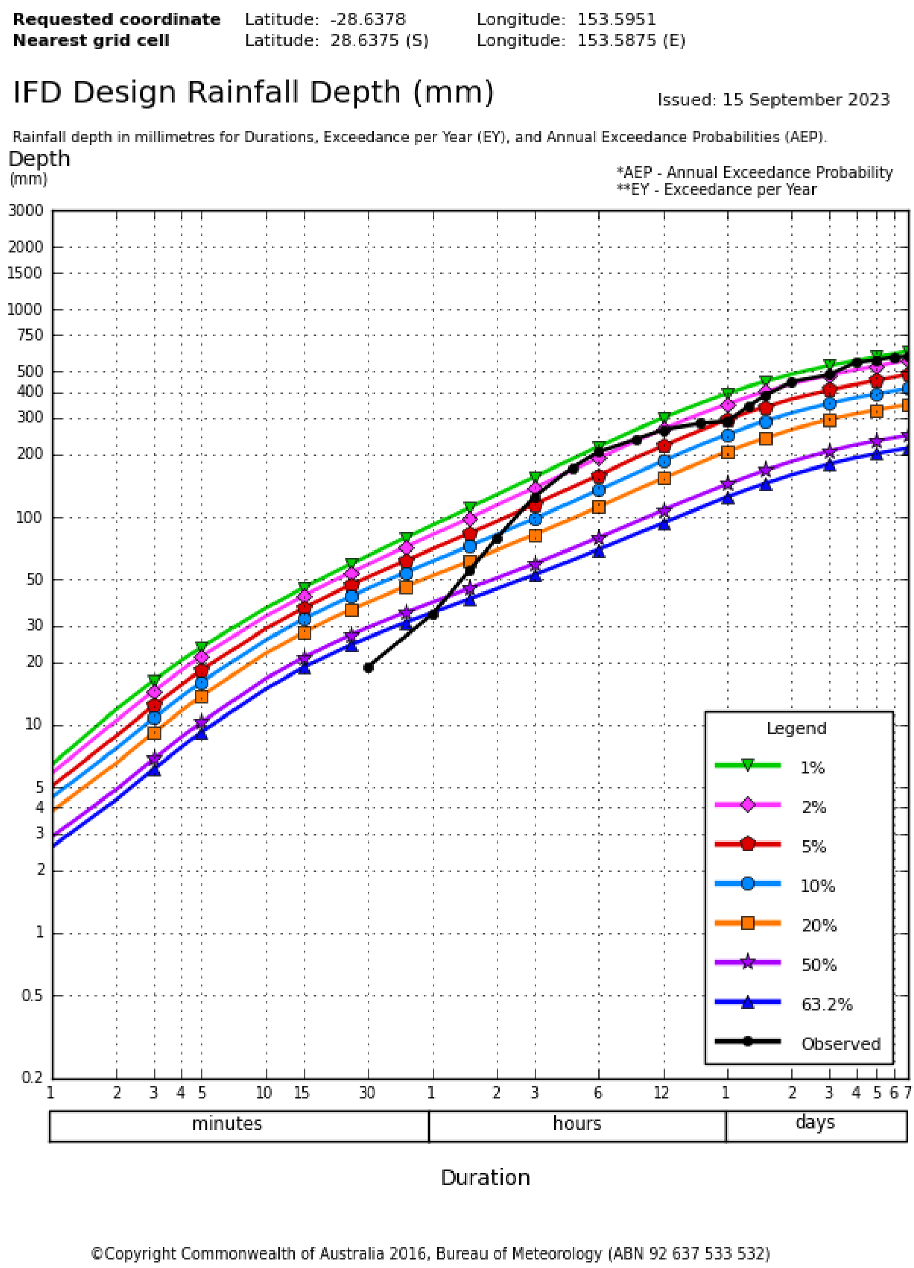

FIGURE 3.3: BELONGIL CREEK BRIDGE ALERT GAUGE (558099) ARR2019 IFD DEPTH VS MARCH 2022 RECORDED DEPTH

Pluviography data available for the March 2022 event was limited to 15-minute rainfall intervals, with the main rainfall burst event provided in Figure 3.4. The entire rainfall event was not simulated, as the surveyed debris data only started being collected from the 1st of April onwards, and as the second burst was bigger, negated the need to include the earlier event. This 24-hour burst event has a total rainfall of

289.5 mm and was simulated with no initial loss conditions. According to Figure 3.3, the 24-hour event corresponds to be between a 10% and 5% AEP event.

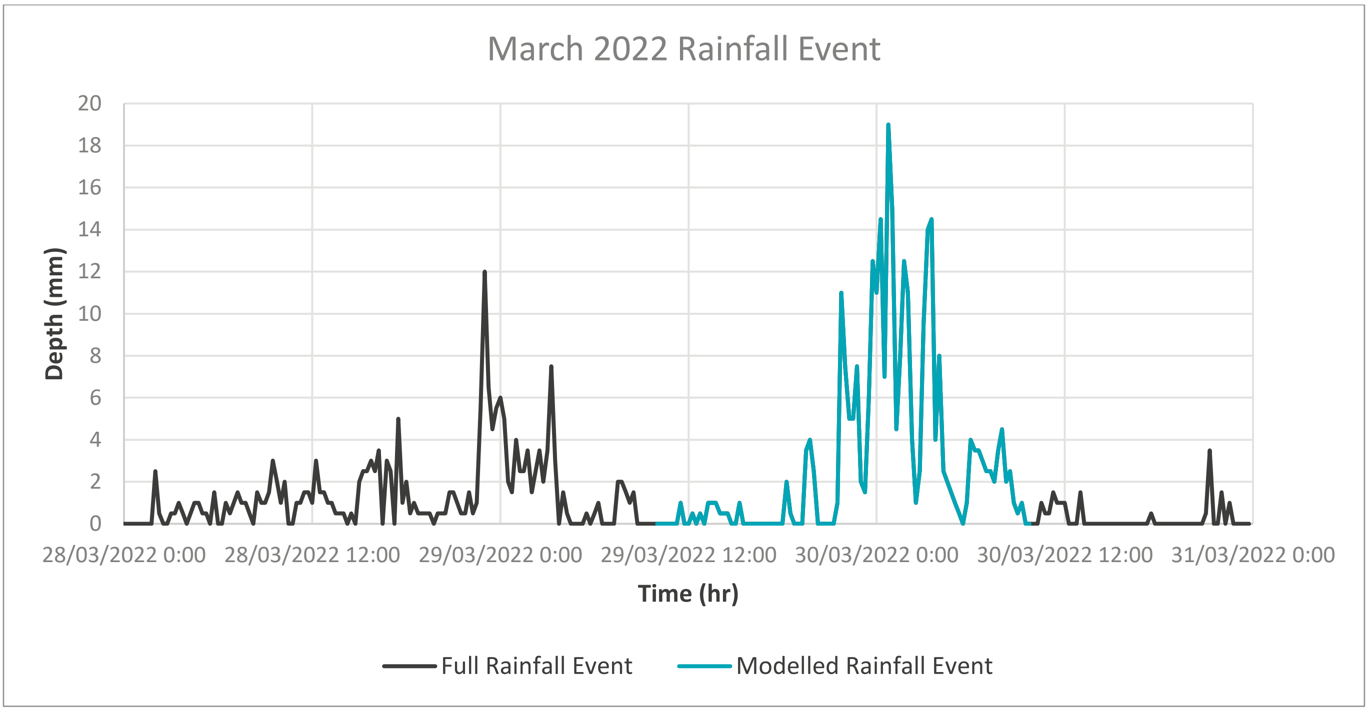

FIGURE 3.4: BYRON BAY (BELONGIL CREEK BRIDGE) (558099) MARCH 2022 GAUGED RAINFALL

3.5 Results

As previously discussed, the review of the March 2022 event is contained within the hydraulic portion of this report, as flood height survey markers have been used for the review. The same goes for the design hydrological outputs, as the purpose of the hydrological model is only to generate local hydrographs for each catchment that is to be applied directly to the hydraulic model. Due to the nature of the Byron Bay catchment, and how flat the topography is, significant ponding of water is expected, with flow breaking out in multiple different directions, without any consistency to the hydrological direction. Reporting on the hydraulic results will ensure that these factors are considered.

3.6 Hydrology Discussion

Based on the commentary of the model development above, it should be noted that the design hydrology adopted for the study is considered a robust, conservative model with forward looking assumptions on land use and design rainfall. The model is not truly representative of existing 2023 conditions for the following reasons:

- AR&R design rainfall assumptions scaled up 10%
- Ultimate land use fraction impervious adopted per Byron Bay planning scheme.

These assumptions were considered necessary to model an appropriate baseline hydrological scenario for design purposes and allow like for like comparison with hydrology for the baseline and design upgrade scenarios to map impacts and calculate flood damages.

Therefore, the design hydrology is not directly comparable for historical event validation or detailed flood damages costing done in post event analysis studies or the Belongil Creek regional flood study. The main purpose of the modelling to provide a robust basis for design of drainage upgrades and allow an understanding of the relative benefits of the Byron Bay drainage scheme.

The design basis for the Byron Bay drainage scheme does not include specific design measures for climate change at this stage as agreed with Council. For this reason, hydrologic and hydraulic climate change scenarios have not been tested or reported in the modelling as climate change scenarios does not influence the current concept design outcomes, costs or engineering. Climate change consideration in the design is further discussed in section 7 of this report.

4. HYDRAULIC MODELLING

4.1 Introduction

A new local hydraulic model using the TUFLOW 1D/2D solver has been developed to assess the performance of the proposed drainage upgrade strategy. The existing regional TUFLOW hydraulic model of the Belongil Creek catchment (BMT, 2015) is too coarse and is not considered to be fit for purpose for the detailed drainage modelling required for this project.

Flood modelling was undertaken for the following scenarios:

- **Design Baseline Scenario:** Existing configuration with ultimate future land use hydrology (note this is not "existing conditions").
- **Design Upgrade Scenario:** Proposed drainage configuration with ultimate future land use hydrology.
- **Alternate Upgrade Scenario:** Design upgrade scenario but with a 10% AEP levee height.
- **Pump Failure Upgrade Scenario:** Design upgrade scenario with complete pump system failure.
- **Outlet Blockage Upgrade Scenario:** Design upgrade scenario with 100% blockage of culvert outlets.

To ensure consistency with the hydrologic modelling the design baseline scenario was prepared assuming full catchment development in accordance with the Byron Bay planning scheme to account for potential future development. It is imperative that any proposed drainage upgrades are assessed in hydraulic scenarios adopt the same underlying hydrologic parameters, lest the hydraulic comparison not be representative of the proposed change. It is considered that a baseline scenario adopting full catchment development is an appropriate design assumption and therefore any demonstrated flood improvements may be further accentuated when applied to the current development within the catchment.

The following sections summarise the inputs and assumptions adopted in the preparation of the new local hydraulic model.

4.2 Hydraulic Model Development

4.2.1 Model Topography, Cell Size and Extent

The Baseline Scenario Digital Elevation Model (DEM) was developed based on the following data:

- 1m DEM LiDAR dated 2010 (sourced from publicly available Elevation and Depth Foundation Spatial Data).
- Detailed ground survey dated July 2023 (supplied by Bennet and Bennet).
- Butler Street As-Constructed survey (supplied by BSC).
- Carlyle Street 3D design surface (supplied by BSC).
- Wordsworth Street 3D design surface (supplied by BSC).
- Sandhills Wetland 3D design surface (supplied by BSC).

The Upgrade Scenario Digital Elevation Model (DEM) was developed using the above model with the inclusion of the following additional data designed by Engeny:

- Shirley Street Levee 3D design surface.
- Butler Street Drain 3D design surface.
- Town Centre Sump 3D design surface.
- Middleton Street Basin 3D design surface.
- Lawson Street 3D design surface (supplied by BSC).

A 1m grid cell resolution was adopted for the Study and selected to ensure that flooding and overland flow paths were represented in the highest possible detail. It was determined that sub-grid sampling (SGS) was not required given the fine cell resolution and would significantly increase model simulation time with no material improvement to output quality.

Due to poor definition in the DEM or other data inaccuracies, modifications to the model terrain have been implemented using TUFLOW z-shape tools to represent the following:

- Road crest lines using TUFLOW break lines.
- Butler Street Drain bed and banks using assumed channel geometry.

The adopted model extent and topography for the Baseline Scenario is shown below in Figure 4.1.

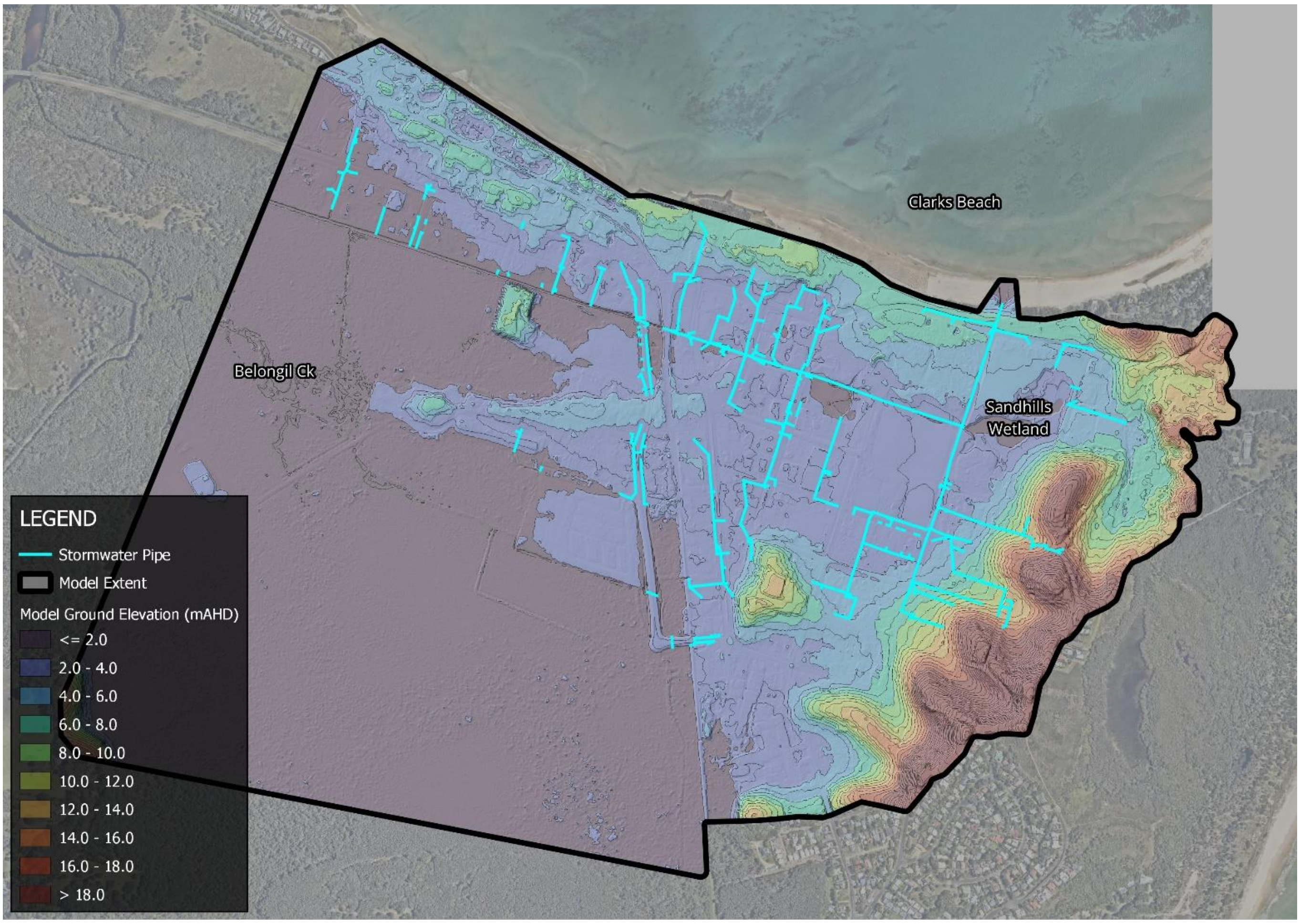

FIGURE 4.1: TUFLOW MODEL EXTENT AND TERRAIN

4.2.2 Boundary Conditions

Inflows to the TUFLOW model were applied as local catchment hydrographs as determined by the WBNM hydrologic model described in Section 3. Local catchment hydrographs were applied using a combination of various approaches to best represent flow accumulation for each sub-catchment, the flow application methods adopted are summarised below:

- Flows applied to the 2D cells connected to a stormwater inlet pit using the SA PITS approach for sub-catchments discharging at the top of a stormwater network,
- Flows applied to swale drains and kerb gutters using the SA Streamlines approach for urban sub-catchments where future upgrades proposed; and
- Flows applied to the 2D domain using the standard SA approach for rural catchments with no stormwater network or urban influence.

Outflow boundaries were applied using the HT approach which allows TUFLOW to calculate a level-discharge outflow relationship based on a defined tailwater level. Outflow boundaries were applied to the model to represent discharge into Belongil Creek and Clarks Beach based on a range of storm surge and wind/wave barometric setup events, these are summarised in Table 4.1.

Time-varying tailwater levels were adopted for the March 2022 event based on data extracted from the Rainbow Bridge on Ewingsdale Rd (Gauge Station H558099). There are no other gauges within the vicinity that could be used to inform historical event tailwater conditions and therefore a tailwater correlating to the Highest Astronomical Tide (HAT) was adopted at the Clarkes Beach outlet for the March 2022 event.

The TUFLOW Model boundary setup is shown in Figure 4.2.

TABLE 4.1: ADOPTED MODEL TAILWATER CONDITIONS

Storm Event	Belongil Creek Tailwater Level (mAHD)	Clarkes Beach Outlet Tailwater Level (mAHD)
March 2022	Time-varying (starting at 1.45 and peak at 2.01)	1.10 (Highest Astronomical Tide)
0.5% AEP to PMP	2.29 (1% AEP storm tide)	2.29 (1% AEP storm tide)
2% AEP to 1% AEP	2.19 (5% AEP storm tide)	2.19 (5% AEP storm tide)
5% AEP (envelope approach)	2.29 (1% AEP storm tide)	2.29 (1% AEP storm tide)
10% AEP to 5% AEP	2.12 (10% AEP storm tide)	2.12 (10% AEP storm tide)
50% AEP to 18% AEP	2.04 (18% AEP storm tide)	2.04 (18% AEP storm tide)

FIGURE 4.2: TUFLOW STORMWATER NETWORK AND BOUNDARIES

4.2.3 Hydraulic Roughness

The hydraulic roughness has been selected based on the inspection of aerial imagery and applied using the boundary cadastre data as supplied by BSC. The adopted Manning's 'n' Roughness values are summarised below in Table 4.2 and the spatial distribution of the materials layer is provided in Figure 4.3.

The selection of hydraulic roughness values has considered the impact of typical urban furniture and buildings and the change to overland flow behaviour that they generate. Buildings have not been applied as 2D obstructions as this artificially reduces the available flood storage within the model and is therefore considered to produce unrealistically high flood levels. Given the high density of buildings and commercial structures present in the model, especially within the Town Centre, it is considered unreasonable to adopt 2D building obstructions and an increased hydraulic roughness has been adopted.

TABLE 4.2: ADOPTED MODEL HYDRAULIC ROUGHNESS

Material	Hydraulic Roughness
Road Reserve	0.025
Open Water Body / Ocean	0.030
Low-Density Residential	0.150
Medium-Density Residential	0.200
Commercial	0.250
Open Space	0.040
Light Vegetation	0.060
Medium-Dense Vegetation	0.080
Swamp	0.050
Community Facilities	0.100

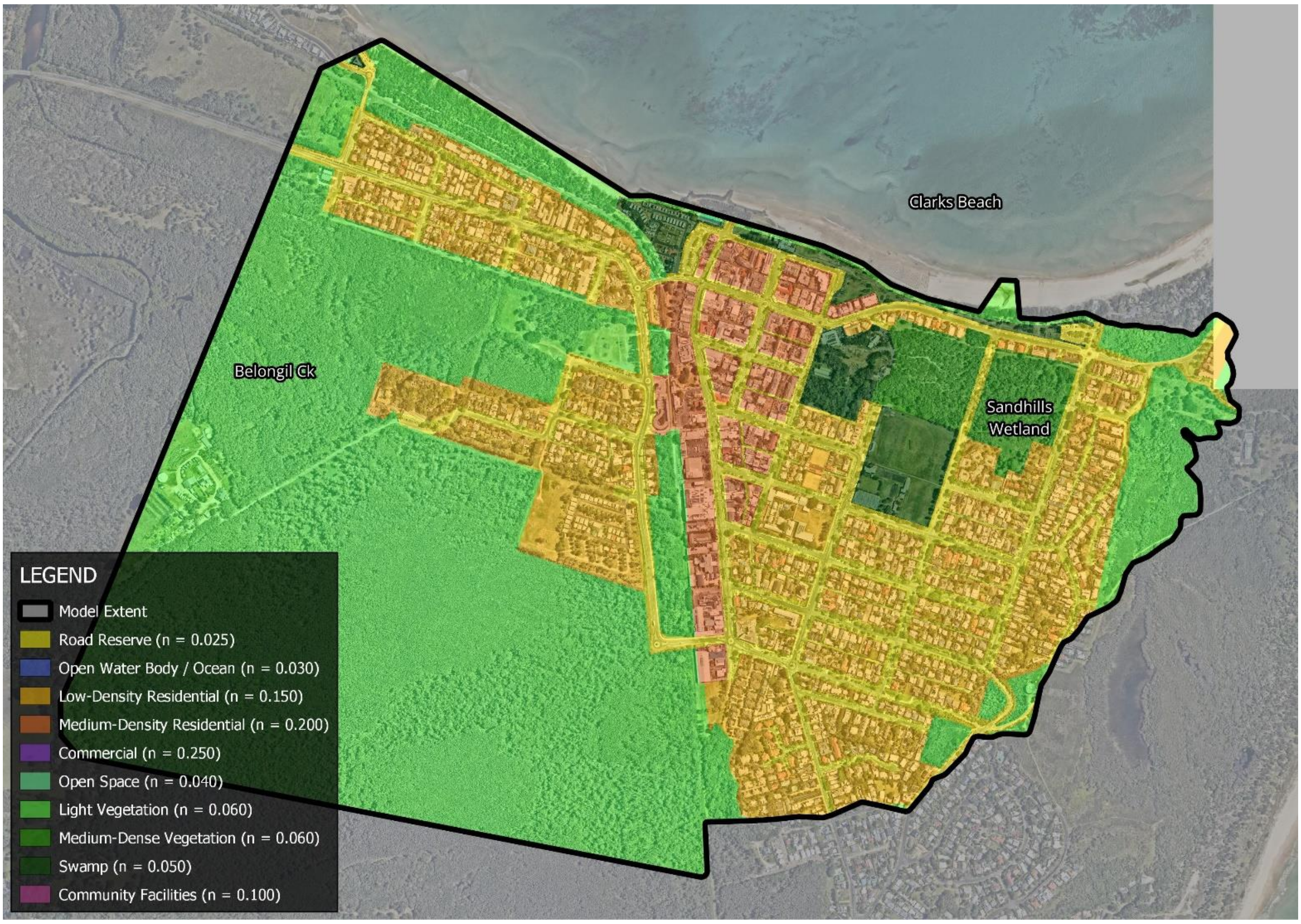

FIGURE 4.3: TUFLOW HYDRAULIC ROUGHNESS

4.2.4 Drainage Network

The stormwater network adopted in the TUFLOW model is based on the following information:

- Belongil Creek Floodplain Risk Management Plan (BMT, March 2015).
- Council's GIS Stormwater Network.
- Detail survey dated 2023 (supplied by Bennett and Bennett).

A preliminary review of the above data indicated that significant additional network exists beyond what was included with the Belongil Creek Floodplain Risk Management Plan, based on the BSC GIS Stormwater Network provided. The adopted network was therefore developed using original BMT stormwater network and extended to include the missing features as included in the BSC GIS database. It is noted that a significant amount of key structural information required to represent the stormwater network was missing from the BSC GIS database, such as:

- Manhole and pipe inverts.
- Pipe sizes.
- Inlet pit sizes.

Missing pit and pipe information was in-filled in the model stormwater network based on the following assumptions:

- Minimum pipe grades.
- Minimum 0.6m cover to reinforced concrete pipes.

- Minimum 0.3m cover to reinforced concrete box culverts.
- Pipe sizes based on upstream or surrounding network.

Structure sizes recorded in the detail survey dated 2023 replaced any of the stormwater network details extracted from the original BMT model and any of the assumed pipe sizes.

Existing stormwater pits were assumed to be either 2.4m kerb lintels with 600x900 grate or a dome-grated 600x900 field inlet, blockages have been applied as per QUDM recommendations for design summarised below:

- Kerb lintels: 100% blockage of inlet grate.
- Field Inlets: 50% blockage.

Proposed stormwater pits were modelled with unlimited capture capacity to ensure the proposed drainage upgrades are not pit limited. Exact pit specifications will be defined at detailed design using the headwater level and capture capacity relationship.

The stormwater network is shown schematically in Figure 4.1.

4.2.5 Simulated Events

The simulated temporal patterns have been selected based on the pattern that most closely represents mid-loaded behaviour. ARR 2019 provides guidance around burst-loading which is referred to as the distribution of rainfall within a storm burst and is a defining characteristic of a rainfall event. Burst-loading is characterised as one of three options, being front- mid- or back-loaded, and is defined based on when the first 50% of rainfall occurs within a burst, in accordance with the criteria below:

- Front-loaded – 0 to 40%
- Mid-loaded – 40 to 60%
- Back-loaded – 60 to 100%

Simulation of the middle-loaded temporal pattern allows for a set of representative flood results to be produced without compromising model simulation time by including the full suite of 10 temporal patterns for each storm duration. At this stage of the project where high-level testing of the preferred drainage strategies is being completed, it is considered suitable to adopt the mid-loaded temporal pattern only for each storm duration. Additional temporal pattern simulations will be considered at the Detailed Design phase.

The full summary of simulated events is summarised in Table 4.3.

TABLE 4.3: MODEL SIMULATION SUMMARY

Storm Event	Storm Duration	Storm Temporal Pattern	Tailwater Condition	Drainage Scenario
1% AEP	30-minute	9	2.19 (5% AEP Storm tide)	Baseline, Upgrade
	60-minute	6		
	120-minute	1		
	180-minute	10		
2% AEP	30-minute	9	2.19 (5% AEP Storm tide)	Baseline, Upgrade
	60-minute	6		
	120-minute	1		
	180-minute	10		
5% AEP	30-minute	8	2.12 (10% AEP Storm tide)	Baseline, Upgrade
	60-minute	7		
	120-minute	6		
	180-minute	4		

Storm Event	Storm Duration	Storm Temporal Pattern	Tailwater Condition	Drainage Scenario
10% AEP	30-minute	8	2.12 (10% AEP Storm tide)	Baseline, Upgrade
	60-minute	7		
	120-minute	6		
	180-minute	4		
	360-minute	1		
18% AEP	30-minute	7	2.04 (18% AEP Storm tide)	Baseline, Upgrade
	60-minute	5		
	120-minute	4		
	180-minute	5		
	360-minute	5		
50% AEP	30-minute	7	2.04 (18% AEP Storm tide)	Baseline, Upgrade
	60-minute	5		
	120-minute	4		
	180-minute	5		
	360-minute	5		
PMP	30-minute	4	2.29 (1% AEP Storm tide)	Baseline, Upgrade
	60-minute	4		
	120-minute	4		
	180-minute	4		
0.05% AEP	30-minute	4	2.29 (1% AEP Storm tide)	Baseline, Upgrade
	60-minute	4		
	120-minute	4		
	180-minute	4		
0.2% AEP	30-minute	4	2.29 (1% AEP Storm tide)	Baseline, Upgrade
	60-minute	4		
	120-minute	4		
	180-minute	4		
0.5% AEP	30-minute	4	2.29 (1% AEP Storm tide)	Baseline, Upgrade
	60-minute	4		
	120-minute	4		
	180-minute	4		
March 2022	1440-minute		Time-Varying at Belongil Creek and constant 1.1m (HAT) at Clarkes Beach	Baseline, Upgrade

4.3 Hydraulic Model Verification

4.3.1 Limitations

As discussed in Section 3.6, the design models developed for the study are forward looking for design purposes with conservative topographic and fraction impervious assumptions. Therefore, the model inputs for the verification assessment are likely to have more conservative results than if a true current day scenario were assessed. Based on this it is considered unlikely the verification analysis will achieve good alignment in simulated and actual measured results. However, it was considered worthwhile to undertake the verification exercise to demonstrate the results show reasonable level of accuracy and are not significantly different to the measured March 2022 event values.

4.3.2 March 2022

Hydraulic model validation was undertaken by simulating the March 2022 historical event and comparing the estimated flood levels to recorded debris marks collected following the rainfall event. As discussed above in Section 3, The second burst of the storm event which occurred in late March was simulated, and debris survey is understood to have been collected between 1 April and 4 April 2022.

The locations of the flood debris survey marks are provided in Figure 4.4 and the flood level comparison outcomes are summarised in Table 4.4. The flood depth map for the simulated March 2022 event is provided in Appendix F.

The flood level comparisons for the March 2022 event were observed to range between 300mm below and 170mm above the surveyed flood heights. Investigation into the provided flood heights has flagged that two of the recorded flood heights, at the Little Burns Street and Lawson-Jonson Street data points, are located below the input terrain data and outside of the simulated flood extents. Furthermore, the Great Northern and The Corner data points record an inconsistent hydraulic grade. That is, whilst the ''The Corner'' data point is located upstream of the Great Northern and at a ground elevation 10mm higher, the Great Northern data point records a flood height 200mm lower than The Corner.

This brings into question the adequacy and accuracy of the supplied flood debris data as the level discrepancy is not considered possible given the terrain. Using debris marks to validate peak flood levels from a hydraulic model can be limited where debris marks are influenced by physical factors such as moving vehicles causing waves which artificially elevate the flood level. There is a high likelihood of this occurring in the Byron Bay, particularly given the flood depths are generally below 500mm and may have been trafficable during the event. In addition to these observations, it is also noted that comparisons to the historical March 2022 event are being completed using flood extents based on outdated 2010 LiDAR as this is the most recent terrain data available at the time of the study.

It is considered that the flood model developed for Byron Bay provides a reasonable fit to the limited flood level data provided and is suitable to assess the relative benefit provided by the proposed drainage upgrades across the Township.

TABLE 4.4: MARCH 2022 FLOOD LEVEL COMPARISON

Debris Marker ID	Debris Marker Location	Surveyed Flood Level (mAHD)	Modelled Flood Level (mAHD)	Modelled Flood Depth (m)	Flood Level Difference (m)
26	LITTLE BURNS ST	2.14	Out of flood extent	Out of flood extent	Out of flood extent
27	RAILWAY PARK	2.99	Out of flood extent	Out of flood extent	Out of flood extent
28	SOMERSET ST	2.16	Out of flood extent	Out of flood extent	Out of flood extent
29	FUNDIES	2.76	2.67	0.20	-0.09
30	THE CORNER	2.64	2.67	0.31	0.03
31	BB LIBRARY	2.73	2.83	0.13	0.10
32	GREAT NORTHERN	2.88	2.67	0.42	-0.21
33	BAY WHALERS	2.72	2.81	0.33	0.09
34	THE CELLAR	2.71	2.72	0.49	0.01

Debris Marker ID	Debris Marker Location	Surveyed Flood Level (mAHD)	Modelled Flood Level (mAHD)	Modelled Flood Depth (m)	Flood Level Difference (m)
35	LAWSON-FLETCHER	2.63	2.80	0.21	0.17
36	LAWSON-JOHNSON	2.70	Out of flood extent	Out of flood extent	Out of flood extent
37	SHIRLEY-DRYDEN ST	2.32	2.01	0.48	-0.30
38	KENDALL ST-EWNGS 1	2.27	2.01	0.37	-0.26
40	KENDALL ST-EWNGS 2	2.19	**2.01**	**0.48**	**-0.18**

FIGURE 4.4: FLOOD DEBRIS SURVEY LOCATIONS

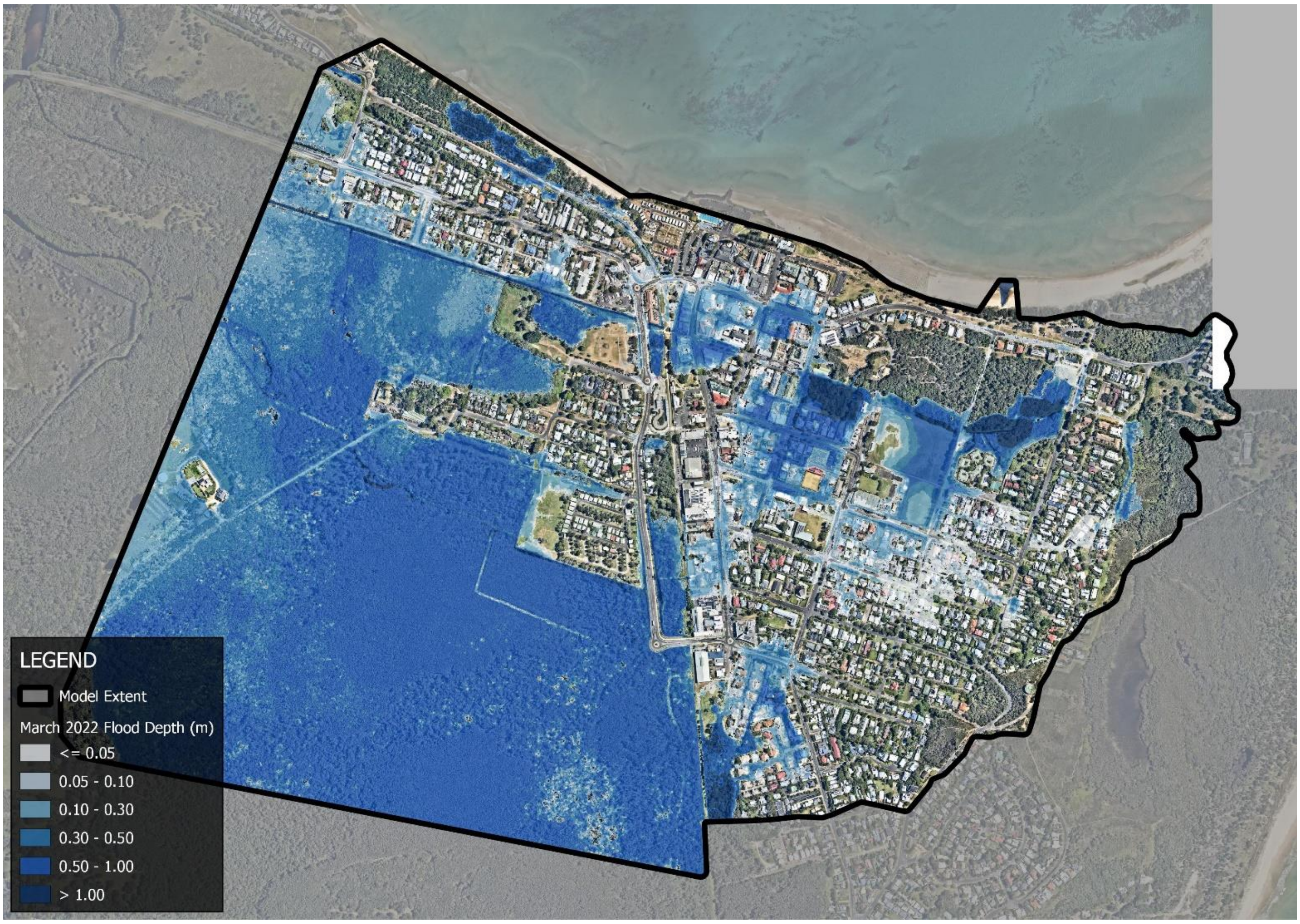

FIGURE 4.5: MARCH 2022 MODELLED FLOOD DEPTH

4.3.3 Design Event Validation to Previous Modelling

A review and comparison of the design flood levels produced by the local hydraulic model for Byron Bay indicates that modelled flood heights are higher than the previous BMT flood model within the township and lower within Belongil Creek. The previous BMT flood model was simulated for the 12-hour critical storm duration only using a time varying storm tide boundary condition. Modelling of a single duration in the previous model indicates that critical storm durations for local catchments within the township were not captured in the regional model, therefore it is anticipated that the local hydraulic model would produce higher flood levels within the Township. Regional Belongil Creek flood levels were considered in the setting of the levee heights for the Upgrade Scenario.

In addition to the above, the local hydraulic model had delineated sub-catchments to a much finer scale to better represent key overland flow paths within Byron Bay. Therefore, the selection of inflow locations is anticipated to generate a significantly different flood extent to a regional model. It is considered that the local hydraulic model developed for this study provides a much better method for prediction of local and overland flow flood behaviour within the Township due to:

- A range of simulated storm durations,
- Sub-catchments delineated to a much finer resolution within the Township,

Representation of underground drainage asset details as outcomes from the detailed survey,

- Representation of key overland flow paths through selection of inflow points; and
- A much finer model resolution.

A comparison of the flood extents is provided in Figure 4.6.

Flood depth mapping for the baseline design and historical events is provided in Appendix C.

Flood level afflux mapping for the design and historical events is provided in Appendix D.

FIGURE 4.6: 1% AEP FLOOD EXTENT COMPARISON

4.4 Baseline Scenario Flood Results

A summary of the number of flooded properties for the 10% AEP and 1% AEP design events as well as the March 2022 for each of the study areas is provided in Table 4.5. The study areas are defined by the red polygons in Figure 4.7 and a flooded property was identified in accordance with the following methodology:

- Extract building envelopes used for the flood damages assessment for each study area.
- Adopt surveyed FFL or, calculate building ground levels based on average terrain across the building polygon, if survey does not exist.
- Investigate peak flood level results at each dwelling to determine peak flood level at each structure.
- Classify a property as affected by flooding if the peak flood level exceeds ground level.

It is noted that this approach is consistent with the results presented in the *Trade-Off Assessment – Town Pump Station removal and Byron Street to Cowper Street Trunk Drainage Connection (Engeny, October 2023)*.

The 10% AEP flood depth is shown below in Figure 4.7 and the minimum and maximum property flood depth in the 10% AEP for each study area was:

- Shirley Street
  - Minimum: 0.07m
  - Maximum: 1.1m

- Town Centre
  - Minimum: 0.07m
  - Maximum: 1.7m
- Cowper Street
  - Minimum: 0.15m
  - Maximum: 2.8m

TABLE 4.5: DESIGN BASELINE SCENARIO NUMBER OF FLOODED PROPERTIES SUMMARY

Event	Shirley Street	Town Centre	Middleton / Cowper Street	Total
10% AEP	64	45	86	195
1% AEP	69	53	116	238
March 2022	**62**	**49**	**102**	213

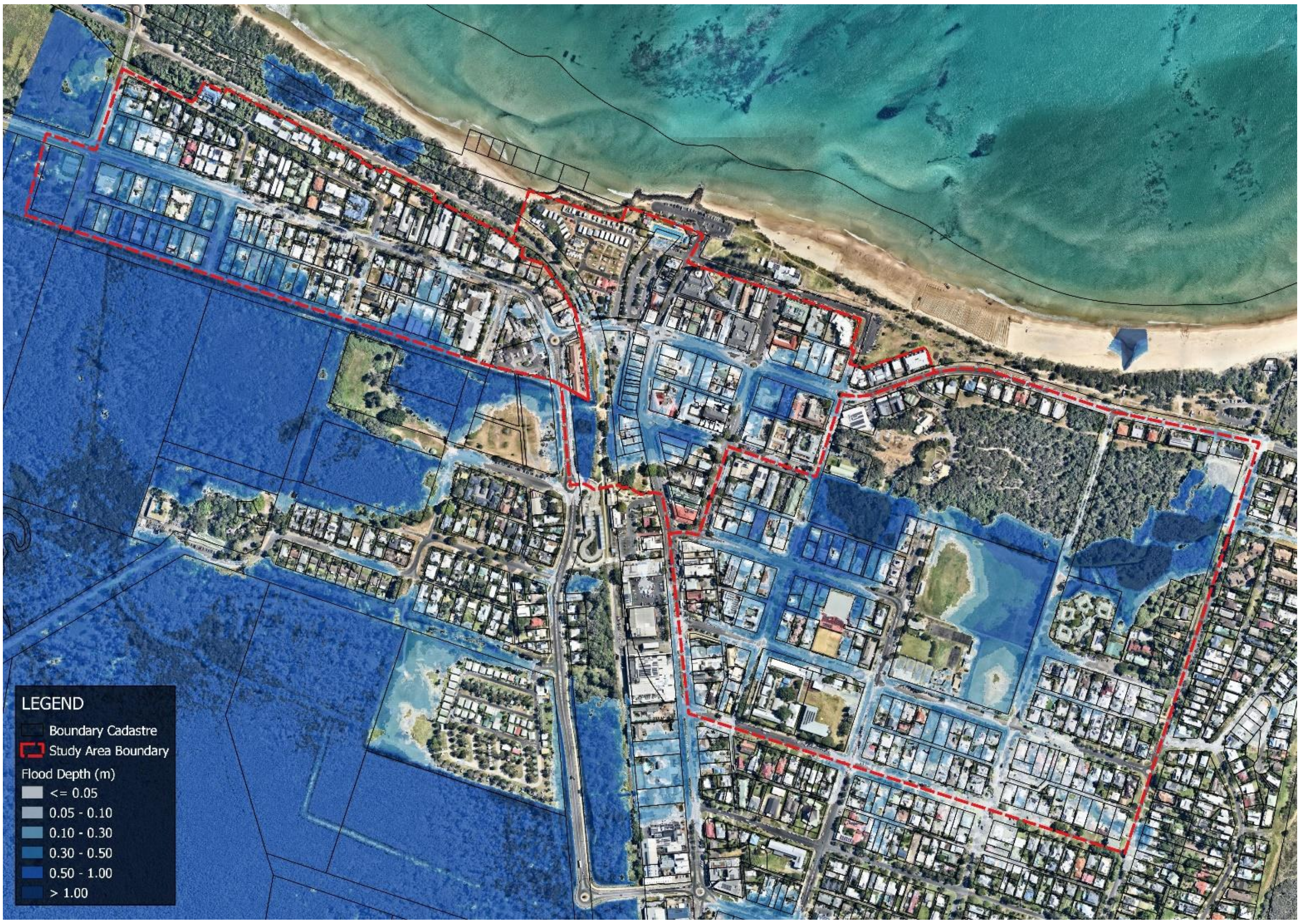

FIGURE 4.7: 10% AEP BASELINE SCENARIO FLOOD DEPTH

4.4.1 Shirley Street (Area 1)

Flood levels over the Shirley Street Study Area are primarily driven by the tidal and regional flood levels within Belongil Creek and therefore the flood levels within Shirley Street in the 10% AEP are generally 2.12m AHD. As this 2.12m AHD level is a conservative assumption within the hydraulic model, number of flooded properties may reduce significantly if alternate regional levels were selected in the model.

Overland flow impacts are present on Shirley Street between Milton Street and Wordsworth Street due to inadequate capacity within the stormwater network. Properties within the Shirley Street study area were observed to be inundated in the 50% AEP and are therefore immune to an unknown storm event AEP more frequent than the 50% AEP.

4.4.2 Town Centre (Area 2)

Flooding within the Town Centre is observed to pond at several local low points across the Study Area due to inadequate network capacity. Ponding levels along Lawson Street in the 10% AEP are observed to be approximately 2.7m AHD producing a peak ponding depth of 0.3m. Ponding at the intersection of Jonson Street and Byron Street are observed to be at approximately 2.5m AHD producing a peak ponding depth of 0.6m. There are no overland flow relief points from the Town Centre to allow for overland flow to discharge to the Belongil Creek floodplain as overland flow is significantly restricted by the rail line embankment.

34 properties within the Town Centre are predicted to be flooded in the 50% AEP event and are therefore have a lower flood immunity than the 50% AEP event (likely in the range 0.5 to 1 flood exceedances per year). These properties are considered to experience flooding of some degree in most years and are amongst the most severely affected properties in the study area.

Peak flows in the culvert underneath the railway line ranges from 1.3m^3/s in the 50% AEP to 2.6m^3/s in the 1% AEP.

4.4.3 Middleton and Cowper Street (Area 3)

The Cowper Street system is characterised by two low lying basins at Cowper Street and Middleton Street. The first basin at Cowper Street is represented by a low point of 1.8m AHD roughly halfway between Carlyle Street and Lawson Street. The Middleton Street basin is immediately east of Middleton Street and north of Marvell St within the existing swamp with LiDAR levels captured at approximately 1.5m AHD. The Cowper Street network currently discharges to a headwall at Clarkes Beach and is also hydraulically connected via pipes to the Byron Street pipe network, however the pipe grade and levels are inconsistent across this connection and is not hydraulically efficient.

Depending on the nature of a given flood event and where the outlet capacity is available (i.e. Belongil Creek vs Clarkes Beach) there is potential for hydraulic grade to flow either way between the Town Centre and Cowper Street. The Middleton Street network connects to the Byron Street pipe network and ultimately discharges to the Belongil Creek floodplain.

The modelled flow rates and directions in the 10% and 1% AEP events of this pipe connection and overland flow path are shown in Table 4.6, noting that the pipe and overland flows are in the direction of east to west.

TABLE 4.6: COWPER STREET TO BYRON STREET PEAK FLOWS

Event	Pipe Peak Flow (m^3/s)	Overland Peak Flow (m^3/s)
50% AEP	0.14	0.01
20% AEP	0.14	0.12
10% AEP	0.14	0.78
5% AEP	0.12	0.93
2% AEP	0.12	2.63
1% AEP	0.12	4.03

When the existing pipe network capacity is exceeded the stored water levels at the Cowper Street sag are observed to overtop the local topography of 3.2m AHD and discharge into the swamp adjacent Middleton Street which is a secondary low point at base level of approximately 1.5m AHD, the 10% AEP stored water level at this location is approximately 2.9m AHD. The existing pipe network layout and pipe longitudinal section from the Cowper Street chamber to the Town Centre outlet into the Belongil Creek floodplain is shown below in Figure 4.8 and Figure 4.9.

Both the Cowper Street and Middleton Street sag points do not have any overland flow relief points to Clarkes Beach or the Belongil Creek floodplain. Like other parts of the Study Area that are severely flood prone, the Cowper Street system is not able to provide 50% AEP flood immunity to road and properties in the area.

FIGURE 4.8: EXISTING SCENARIO PIPE NETWORK LAYOUT

FIGURE 4.9: COWPER STREET TO RAIL LINE PIPE LONGITUDINAL SECTION

4.5 Design Upgrade Scenario Hydraulic Model

The Design Baseline Scenario hydraulic model was updated with the proposed drainage upgrades for each Study Area, these are summarised below:

- Shirley Street:
  - Realignment of the Butler St drain.
  - levee at 2.89m AHD adjacent the Butler Street drain (1% AEP level plus 0.6m freeboard).
  - Pump station on Kendall St with wet well storage and design pump system capacity of 1550 L/s.
  - Pump station on Dryden St with wet well storage and design pump system capacity of 1300 L/s.
  - Pump station on Milton St with wet well storage and design pump system capacity of 1350 L/s.
  - Stormwater pipe network capacity upgrades discharging to each of the pump stations.
  - New gravity fed stormwater outlets into Belongil Creek for each of the stormwater pipe networks with flap gates to prevent tidal ingress.
- Town Centre
  - Levee at 2.89m AHD along Butler Street (1% AEP level plus 0.6m freeboard).
  - Pump station at the Butler Street sump with wet well storage and design pump system capacity of 5000 L/s.
  - Stormwater pipe network capacity upgrades discharging to the Butler St pump station with large trunk upgrade.
  - Installation of flood gates on the Butler St drain culverts, Town Centre pipe network outlet and Butler St pipe network outlets.
- Cowper Street
  - Overland flow path and flood storage between Middleton Street sag and Cowper Street sag at 0.8m AHD with a flood gate outlet connecting to the Cowper Street stormwater pipe network.
  - Trunk stormwater pipe network capacity upgrades along Cowper Street and Middleton Street.
  - Duplication and upgrade of Cowper Street outlet to Clarkes beach.
  - Installation of flood gates at the Clarkes Beach outlet.

Additional sensitivity models were undertaken for the below:

- 10% AEP levee height (2.26m AHD)
- Pump failure.
- Outlet blockage.

Additional detail of the proposed drainage scheme is provided in the engineering report.

4.6 Design Upgrade Scenario Flood Results

A summary of the number of flooded properties for the design upgrade scenario 10% AEP and 1% AEP design events as well as the March 2022 for each of the study areas is provided in Table 4.7 with a comparison to the number of flooded properties in the design baseline scenario. These results indicate that 53 and 49 properties are anticipated to be no longer flooded by the 10% AEP design storm and March 2022 event, respectively, in the Shirley Street study area. The Town Centre and Cowper Street study areas show 32 fewer properties and 14 fewer properties flooded in the 10% AEP design storm, as well as 38 fewer properties and 26 fewer properties in the March 2022 event.

Of the properties that are still flooded, flood depths reduce significantly. The 10% AEP flood depth is shown below in Figure 4.10 and the maximum and minimum property flood depth in the 10% AEP for each study area are:

- Shirley Street
  - minimum: 0.05m
  - maximum: 0.3m
- Town Centre
  - minimum: 0.05m
  - maximum: 1.2m
- Cowper Street
  - minimum: 0.05m
  - maximum: 2.7m

TABLE 4.7: DESIGN UPGRADE SCENARIO NUMBER OF FLOODED PROPERTIES SUMMARY

Event	Shirley Street	Town Centre	Middleton / Cowper Street	TOTAL	Shirley Street Reduction from Baseline	Town Centre Reduction from Baseline	Middleton / Cowper Street Reduction from Baseline	TOTAL
10% AEP	11	13	72	96	-53	-32	-14	-99
1% AEP	25	23	106	154	-44	-30	-10	-84
March 2022	**13**	**11**	**76**	**100**	**-49**	**-38**	**-26**	**-113**

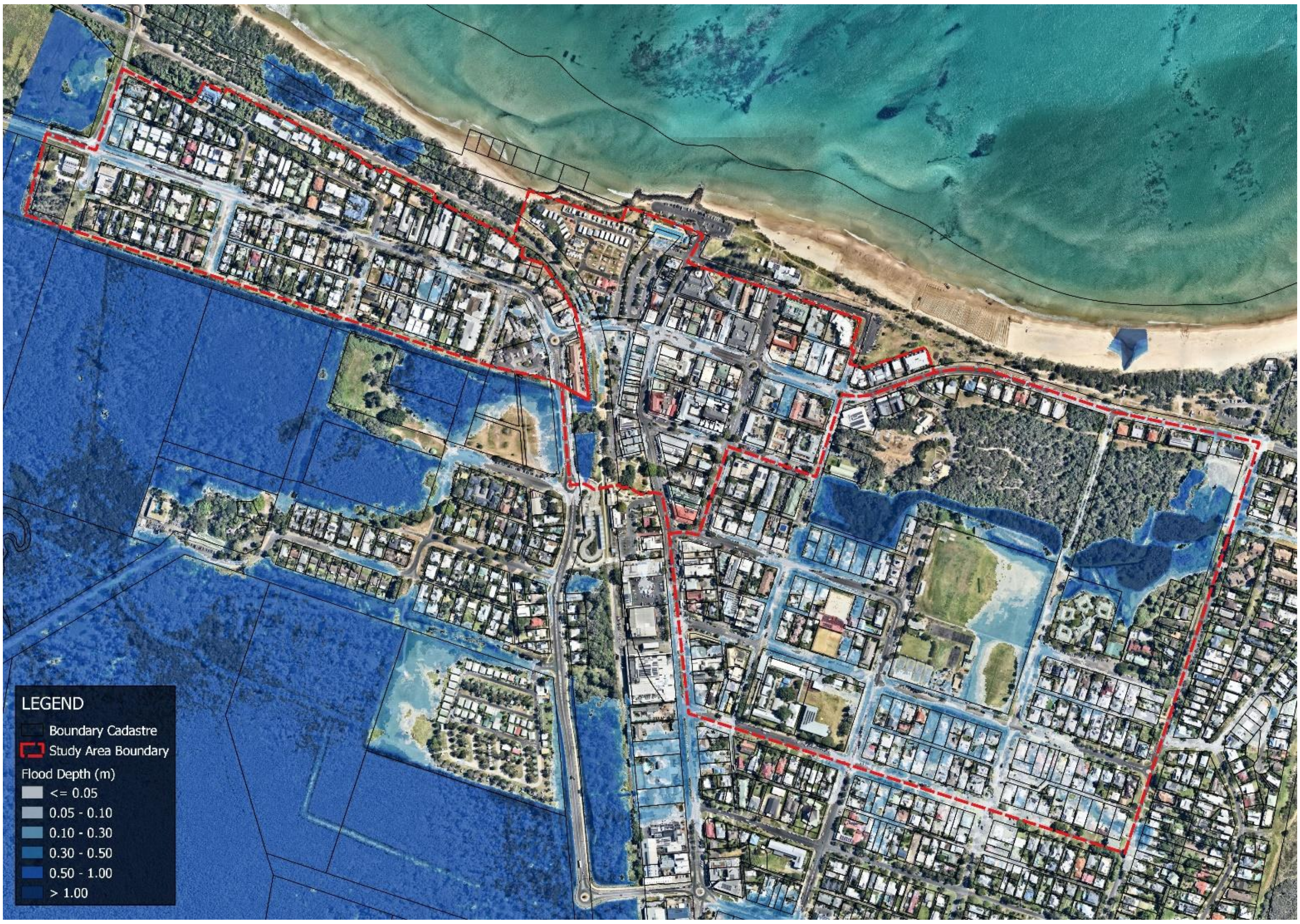

FIGURE 4.10: 10% AEP DESIGN UPGRADE SCENARIO FLOOD DEPTH

4.6.1 Shirley Street

The proposed drainage upgrades provide significant flood benefit to the Shirley Street Study Area with 11 properties no longer flood impacted in the 10% AEP as shown in Figure 4.11. The full set of flood results and flood afflux are provided in Appendix B.

A summary of the pump performance for Kendall Street, Dryden Street and Milton Street are provided in Table 4.8 and Table 4.9.

The results show flood level afflux up to 0.1m in the Belongil Creek floodplain downstream of the Butler Street culverts. These results are mostly caused by the reduction in flow area and new constriction along the Butler Street drain following construction of the Shirley Street levee. These impacts are not observed to cause actionable nuisance, however, could be relieved by additional earthworks excavation to open up the flow constriction along the Butler Street drain, which is believed to be a capped landfill. Whilst the pumps are discharging up to 6.0m^3/s additional flow to the floodplain, hydraulic storage within Belongil Creek is significant and the increased flows do not result in impacts exceeding 0.1m.

FIGURE 4.11: 10% AEP DESIGN UPGRADE SCENARIO FLOOD LEVEL AFFLUX

TABLE 4.8: 10% AEP DESIGN UPGRADE SCENARIO PUMP PERFORMANCE

Location	Peak Pump Inflow (m^3/s)	Peak Stored Water Level at Pump Station (mAHD)
Kendall Street	1.68	0.78
Dryden Street	1.18	0.10
Milton Street	1.08	0.18

TABLE 4.9: 1% AEP DESIGN UPGRADE SCENARIO PUMP PERFORMANCE

Location	Peak Pump Inflow (m^3/s)	Peak Stored Water Level at Pump Station (mAHD)
Kendall Street	1.83	1.06
Dryden Street	1.65	0.91
Milton Street	1.77	0.62

Investigation of the flood results has indicated that for:

- The 10% AEP:
  - 53 fewer properties impacted by flooding.
  - Flood level reductions of up to 0.75m within road reserve.
  - Flood level reductions of up to 0.84m within private property.
- The 1% AEP:
  - 44 fewer properties impacted by flooding.
  - Flood level reductions of up to 0.72m within road reserve.
  - Flood level reductions of up to 0.90m within private property.
  - No overtopping of the levee.

Overland flow is generally contained to the road corridor except for a breakout point between Milton Street and Wordsworth Street. Investigation into capturing this flow was undertaken, however without significant upgrades provided on Shirley Street it was determined that this could not be captured appropriately. It was decided that upgrades downstream of this breakout would be proposed which are shown to provide some benefit at this location.

The levee heights are proposed to be constructed to a levee design crest level of 2.89m AHD. This levee height is immune to the 1% AEP storm surge levels of 2.29m AHD and the Probable Maximum Flood levels of 2.6m AHD estimated by the local Byron Bay hydraulic model, however the levee is overtopped by 0.25m in the PMF event which is consistent with the results of the original 2015 BMT study.

4.6.2 Town Centre

The proposed drainage upgrades provide significant flood benefit to the Town Centre study area generally consistent with what is observed at Shirley Street as majority of properties are now immune to the 10% AEP flood extent as shown in Figure 4.12.

A summary of the pump performance for the pump station is provided in Table 4.10 and Table 4.11.

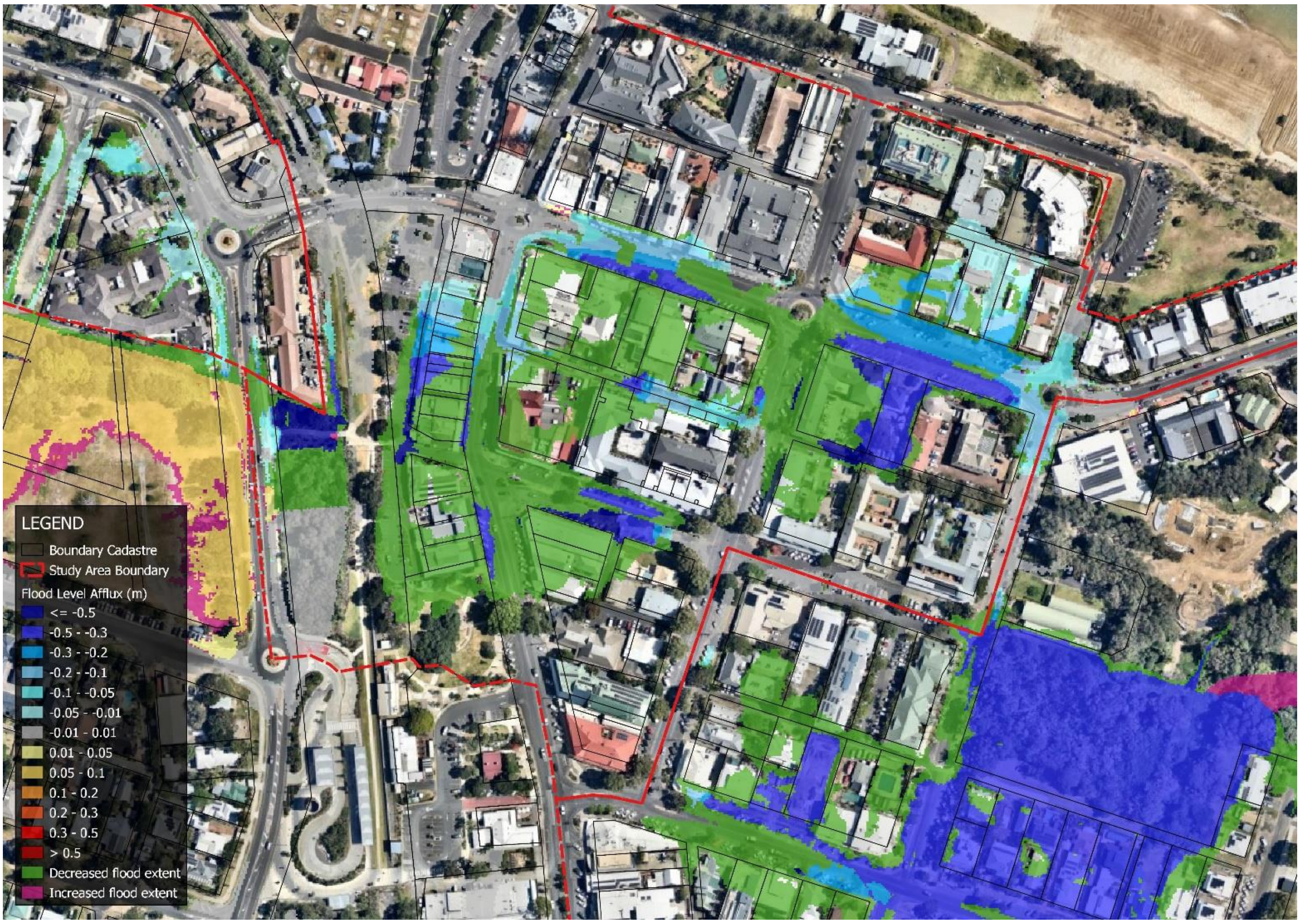

FIGURE 4.12: 10% AEP DESIGN UPGRADE SCENARIO FLOOD LEVEL AFFLUX

TABLE 4.10: 10% AEP DESIGN UPGRADE SCENARIO PUMP PERFORMANCE

Location	Peak Pump Inflow (m^3/s)	Peak Stored Water Level (mAHD)
Butler Street	5.90	0.82

TABLE 4.11: 1% AEP DESIGN UPGRADE SCENARIO PUMP PERFORMANCE

Location	Peak Pump Inflow (m^3/s)	Peak Stored Water Level (mAHD)
Butler Street	6.05	1.64

Investigation of the flood results has indicated that for:

- The 10% AEP:
  - 32 fewer properties impacted by flooding,
  - Flood level reductions of up to 0.71m within road reserve,
  - Flood level reductions of up to 0.58m within private property,
  - Remnant flooding depth of up to 0.39m present at Lot 1 on DP338416 south of Lawson Street at the eastern, and
  - Remnant flooding depth of up to 0.16m present over commercial lots on the western side of Jonson Street north of Byron Street.
- The 1% AEP:
  - 38 fewer properties impacted by flooding,
  - Flood level reductions of up to 0.87m within road reserve,
  - Flood level reductions of up to 0.68m within private property,
  - No overtopping of the levee,
  - Remnant flooding depth of up to 0.70m present at Lot 1 on DP338416 south of Lawson Street at the eastern, and
  - Remnant flooding depth of up to 0.22m present over commercial lots on the western side of Jonson Street north of Byron Street.

The flooding over Lot 1 on DP338416 is due to the trapped sag point with insufficient capacity in the stormwater pipe network as well as the property being located below the crown of Lawson Street. Significant upgrades to the proposed drainage network would be required, within the order of an additional box culvert bringing the total up to 3 barrels extending to the outlet, to achieve the design target flood immunity for this property which is not considered feasible. It is understood that the dwelling on this property sustained damage in the 2022 event and is now undergoing repair and renovations to lift the dwelling. It is considered that the infrastructure cost associated with achieving 10% AEP flood immunity at this location significantly outweighs the benefits and is therefore not proposed to be included in the overall drainage scheme.

Flooding over the commercial properties fronting Jonson Street is due to insufficient capacity in the network to the rear of the buildings. Whilst upgrades are proposed at this location which provide flood level reductions and reduced flood extents at this location, an additional box culvert under the rail line at the outlet into the Butler Street pump station is required to allow the upgrades to freely drain and any further upgrades to achieve the design immunity at these lots would require additional barrels. The properties are observed to be low lying and additional barrels is considered to provide limited benefit.

4.6.3 Cowper Street

The proposed drainage upgrades for the Cowper Street and Middleton Street systems are intended to reduce flood levels through additional conveyance to the ocean and additional flood storage which ultimately discharges to the Clarkes Beach outlet. The flood modelling has shown that stormwater network capacity upgrades in isolation are unable to provide any material benefit and therefore a detention basin is required. This is due to the low-lying areas around Middleton Street and Cowper Street which are observed to be as low as 2.15m AHD, only marginally higher than the design storm tide level of 2.12m AHD at the outlet. This severely limits the available hydraulic grade from Cowper and Middleton Street to the beach, as there is inadequate hydraulic grade to drive a significant outlet flow to the beach without major, infeasible large pipe sizing. This makes trunk pipe upgrades an ineffective mitigation strategy as the flow capacity rapidly declines and cost rapidly increases when additional pipes are provided.

Flood storage is the preferred approach to reducing flooding in the area (given pumping is the only alternative to storage), and this requires excavated flood storage below property flooding levels of approximately 2.1 m AHD. As the flood storage is also below the ocean storm tide level, the solution is reliant on effective backflow prevention through flap gates or similar devices. Additionally, as flood storage is contingent on sizing the storage for a given event, once the event is exceeded (i.e. the 10% AEP 6 hour critical duration), the flood storage has limited benefits.

The proposed drainage scheme provides significant flood benefit over the properties on Middleton Street, as shown in Figure 4.13, however is unable to achieve the 10% AEP design flood immunity within the constraints of the current scheme as flood levels in the 10% AEP are observed to be at 2.6m AHD over the properties backing onto the swamp and fronting Marvell Street.

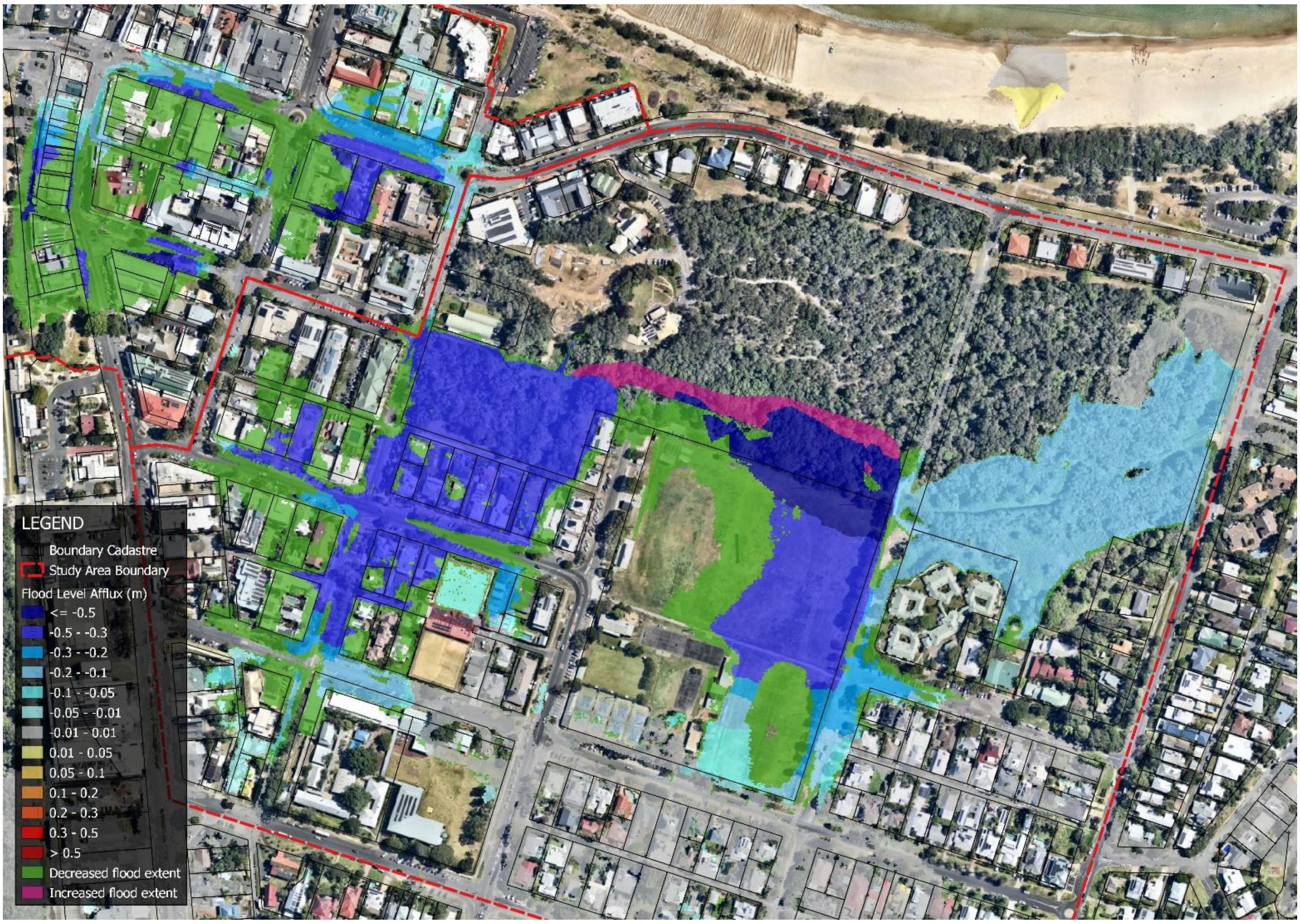

FIGURE 4.13: 10% AEP DESIGN UPGRADE SCENARIO FLOOD LEVEL AFFLUX

Investigation of the flood results has indicated that for:

- The 10% AEP:
  - 14 fewer properties are impacted by flooding.
  - Flood level reductions of up to 0.34m at the Middleton St sag within Crown land.
  - Flood level reductions of up to 0.35m within private property on Marvell Street.
  - Flood level reductions of up to 0.54m at the Cowper St sag within Crown land.
  - Flood level reductions of up to 0.25m at the Cowper St sag within the road reserve.
  - Flood level reductions of up to 0.1m within the Sandhills Wetland.
  - Remnant flooding of depths up to 0.38m at the properties along Marvell Street.
  - Remnant flooding of depths up to 0.21m at Lot 9 on DP758207 (7 Marvell Street).
- The 1% AEP:
  - 26 fewer properties are impacted by flooding.
  - Flood level reductions of up to 0.14m at the Middleton St sag within Crown land.
  - Flood level reductions of up to 0.14m within private property on Marvell Street.
  - Flood level reductions of up to 0.5m at the Cowper St sag within Crown land.
  - Flood level reductions of up to 0.41m at the Cowper St sag within the road reserve.
  - Flood level reductions of up to 0.17m within the Sandhills Wetland.

The remnant flooding at 7 Marvell Street in 10% AEP and greater events is due to the property being beneath the crown level of the road and the insufficient pipe network capacity. It is understood that this property has since been developed since the capture of the LiDAR and is therefore assumed that any flooding issues associated with the site have been resolved throughout the development application process. No additional stormwater upgrades are therefore proposed at this location.

Remnant flooding in the 10% AEP over the properties fronting Marvell Street is due to requiring additional flood storage to reduce the ponded flood levels further at the Middleton Street and Cowper Street sag. It is noted that the proposed detention basin, shown below in Figure 4.14 and Figure 4.15, shows that the ponded water levels at the Middleton St sag and Cowper St sag are shown to be hydraulically connected in the 10% AEP Upgrade Scenario but are not connected in the 10% AEP Baseline Scenario. Therefore, a further reduction in flood levels can be achieved by expanding the eastern extent of the basin further to the north. It is understood that environmental and land tenure constraints are present at the Middleton Street sag swamp limiting the amount of excavation that can be undertaken in this area. It is considered that the current basin arrangement optimises the flood benefits within the known constraints of the proposed drainage strategy.

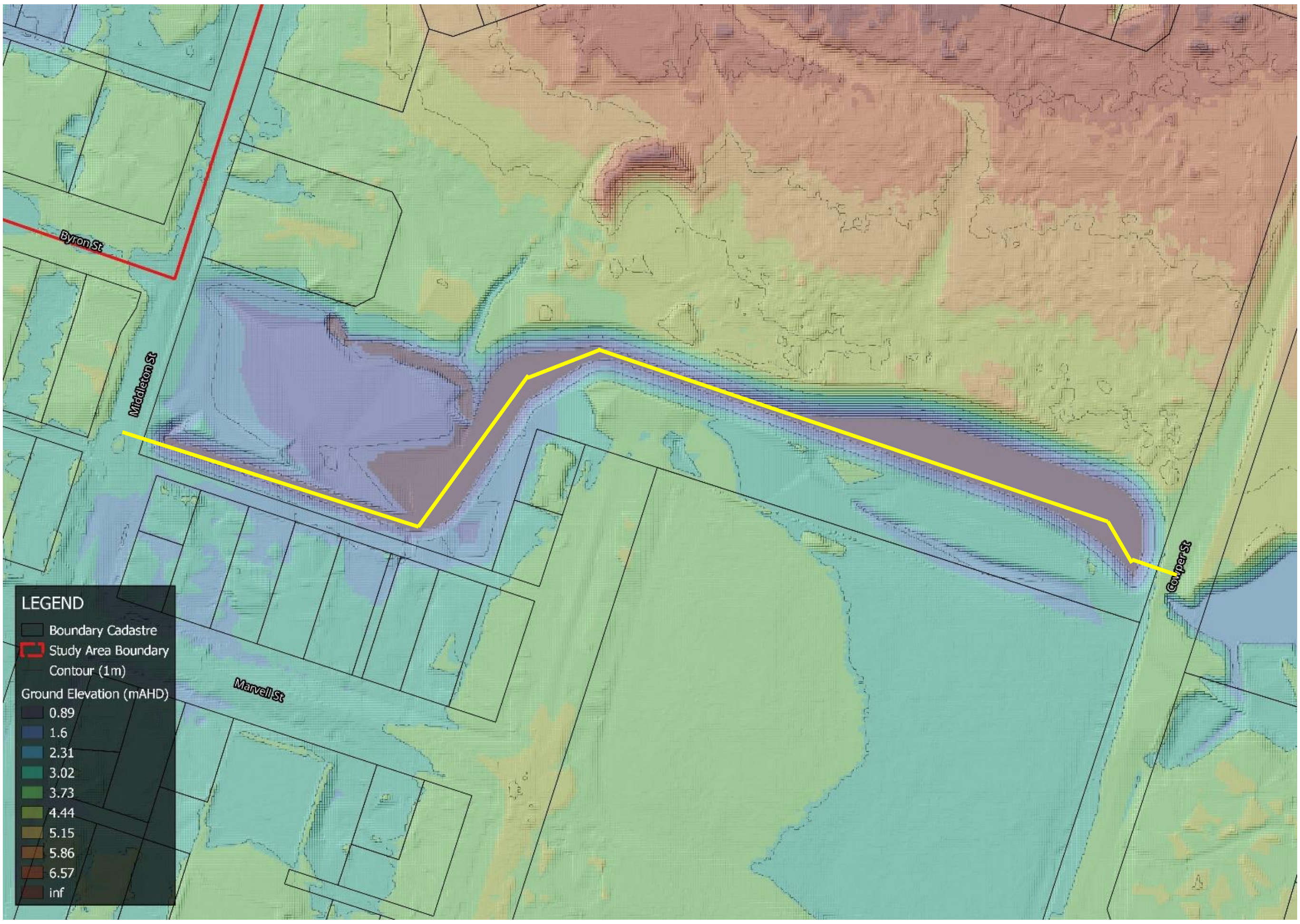

FIGURE 4.14: PROPOSED MIDDLETON ST AND COWPER ST BASIN

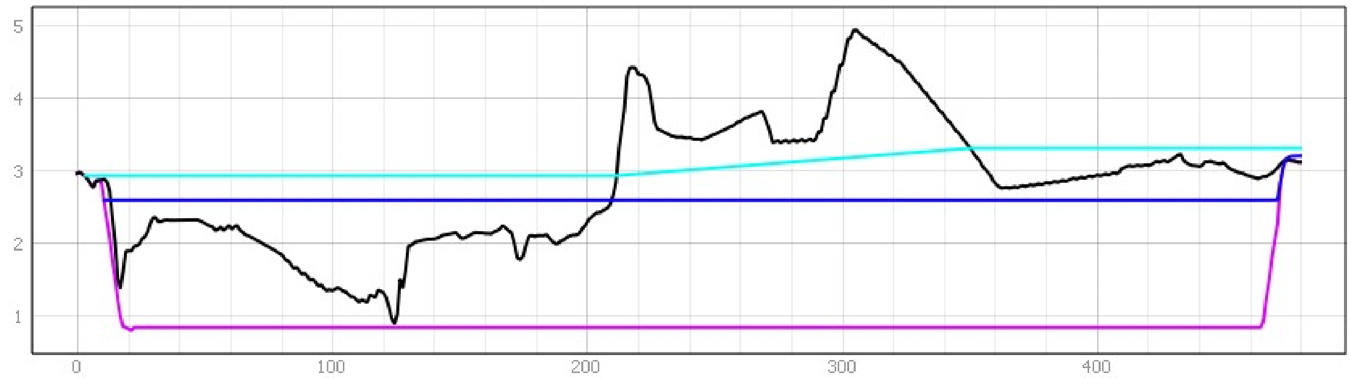

FIGURE 4.15: PROPOSED MIDDLETON ST AND COWPER ST BASIN CROSS-SECTION ALONG DRAIN INVERT (CYAN = BASELINE 10% AEP WSL, COBALT = UPGRADE 10% AEP WSL, BLACK = BASELINE TERRAIN & MAGENTA = UPGRADE TERRAIN)

4.6.4 Cowper Street Outlet

Figure 4.16 and Figure 4.17 show the 10% AEP and 1% AEP HGL from the Cowper Street basin to the Clarkes Beach outlet through the existing 1200mm diameter pipe. It shows the HGL along the trunk pipe is relatively consistent at approximately 0.2% in the 10% AEP and 0.25% in the 1 % AEP. Flow rates through the single 2100mm diameter pipe outlet are summarised in Table 4.12 and are slightly below the target 4m^3/s in the 10% AEP.

Additional pipe capacity at the outlet would provide additional benefit, however, requires significant trunk pipe upgrades and cost. At 0.2% hydraulic grade an additional 1050mm diameter pipe would achieve an additional 1m^3/s capacity. It is considered box culvert outlets to the beach would have theoretical greater capacity due to higher cross-sectional area at high water levels, however, are undesirable because they are generally less effective at self-cleansing than pipes.

TABLE 4.12: CLARKES BEACH PEAK OUTLET FLOW RATE

Location	10% AEP flow (m^3/s)	1% AEP flow (m^3/s)
2100mm RCP	3.3	4.4

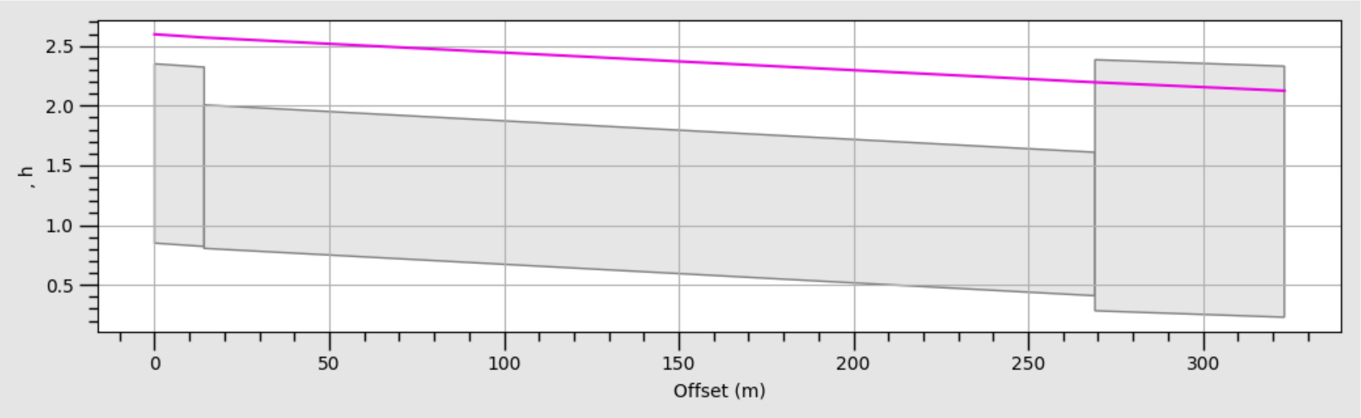

FIGURE 4.16: 10% AEP PIPE HGL

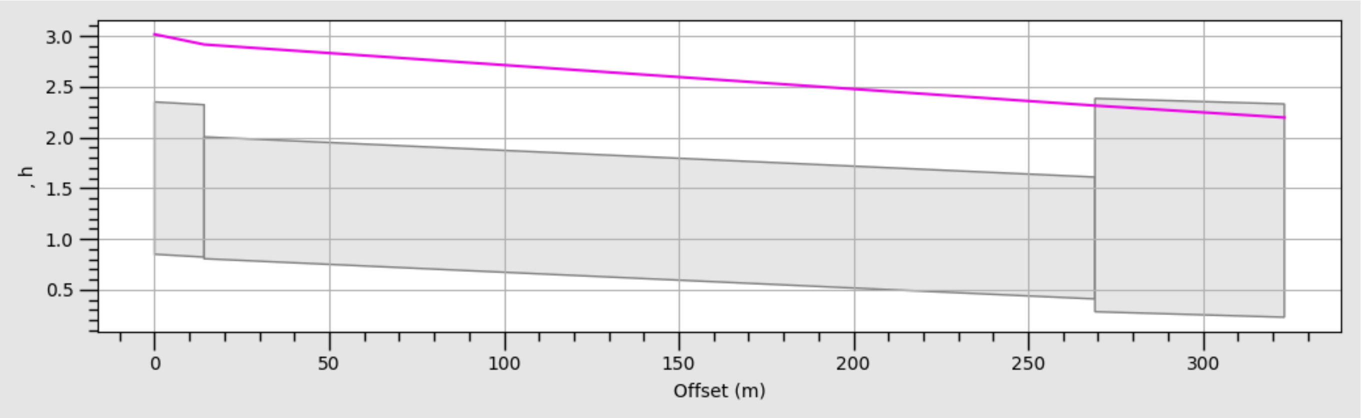

FIGURE 4.17: 1% AEP PIPE HGL

4.6.5 Massinger Street

A series of pipe upgrades were proposed on Massinger Street and Lawson Street to capture overland flow from east of Massinger Street. The network upgrades would divert the major flows away from the Sandhills Wetland along Massinger Street and Lawson Street. This would discharge at the Clarkes Beach outlet to divert inflows away from the known Cowper Street problem area, as shown in Figure 4.18. Diversion of the major flows would allow for additional storage within the Sandhills Wetland to be engaged by the Cowper Street sag.

The option was modelled in detail with a trunk 1500mm diameter pipe to divert flows down Massinger and Lawson Street to Clarkes beach. The proposed upgrades were observed to:

- Provide no material flood level reduction at the Cowper Street sag.
- Create flood level increases on Lawson Street and Massinger Street as well as one of the adjacent properties due to the reduction in performance of the local road drainage network.

Whilst there are significant flood level reductions within the Sandhills Wetland, these were observed to be limited to the wetland and did not translate to additional hydraulic storage at the Cowper Street sag west of Cowper Street. The road levels of Cowper Street at the junction chamber of the main line is at 3.1m AHD, any flood level reductions below this level on the east side of Cowper Street will not influence levels on the western side. Therefore, the proposed pipe diversion and associated flood level reductions only serve to reduce the flows entering the wetland which is anticipated to negatively impact vegetation growth and wetland performance.

The proposed Massinger Street upgrades were discussed with council and removed from the proposed drainage scheme based on the unacceptable flood impacts over Lawson Street and adjacent properties, and the potentially small benefit compared to costs of the trunk pipe infrastructure.

FIGURE 4.18: PROPOSED MASSINGER STREET TRUNK DRAINAGE 10% AEP FLOOD AFFLUX

4.7 Alternate Upgrade Scenario Flood Results (10% AEP Levee)

The flood mapping for the Alternate Upgrade Scenario is provided in Appendix D. The afflux results show that a 10% AEP levee height of 2.26m AHD can achieve similar drainage performance and flood benefits of the Design Upgrade Scenario in the 10% AEP event.

It is noted that the peak 1% AEP flood level is observed to be 2.31m AHD immediately downstream of the Butler Street culverts and therefore represents up to 0.04m of overtopping from Belongil Creek floodplain into the Butler Street sump, there is no overtopping of the 10% AEP at Butler Street.

It is observed that the peak flood levels downstream of Butler Street adjacent the full length of the Shirley Stret levee is observed to remain immune to the 1% AEP with no freeboard.

The model scenario with a reduced levee height of 2.26m AHD can achieve the same drainage improvements in the 10% AEP and is observed to overtop in the 5% AEP by 0.01m and therefore significantly impacts Town Centre pump station performance in events rarer than the 5% AEP.

The alternate upgrade scenario does not include adequate freeboard in the design and exposed to additional risk of overtopping due to:

- Wave action,
- Settlement,
- Construction defects,
- Construction tolerances; and
- Change in future rainfall and tidal patterns.

It is recommended that Council consider whether the additional risk exposure of a lower levee height is a suitable trade-off with reduced construction and maintenance costs.

4.8 Sensitivity Testing Scenarios - Flood Results

4.8.1 Pump Failure Upgrade Scenario

A pump failure scenario was modelled to represent the change to flood behaviour should all operational and standby pumps cease to function, therefore relying on the floodgate culvert outlets into the Belongil Creek floodplain to manage flooding.

The flood level afflux results for the 10% AEP and 1% AEP are provided in Appendix C.

Table 4.13 summarises the peak flood levels behind the Butler Street levee in the pump failure scenario as well as the flood level afflux when compared to the design baseline scenario. The results indicate that under pump failure conditions the proposed drainage scheme is still able to provide an approximate flood level reduction of 0.15m in the 10% and 1% AEP design events. This is typically due to the levee preventing regional flood ingress across the area. The afflux results show a small impact of approximately 0.02m at the levee between Milton Street and Wordsworth Street which is observed to extend over a shed and no habitable dwellings. It is noted that a low tailwater scenario combined with a large local event occurring in tandem with pump failure may cause increased flood levels when compared to the design baseline scenario. It is recommended that additional low tail water runs should be considered at detailed design to inform risk exposure.

Table 4.14 summarises the peak flood levels behind Butler Street and the adjacent portion of the levee in the Pump Failure scenario, as well as the flood level afflux when compared to the Design Baseline Scenario. The flood level increases are observed to extend over 'The Swell' hotel located on Lot 1 on DP781101 by up to 0.10m in the 1% AEP and 0.65m in the 10% AEP. The flood level increases materially impact trafficability along the southward land such that the flood hazard has increased from H1 to H2 (AR&R 2016 flood hazard classification) in the 1% AEP, representing flood hazard unsafe for small vehicles. There is no change to road trafficability on Butler Street in the 10% AEP. Flood levels in the Town Centre east of the rail line in the Pump Failure Scenario are observed to reduce by 0.12m and 0.21m in the 10% and 1% AEP design events, respectively, when compared to the Design Baseline Scenario.

It is recommended that the impacts are investigated at the detailed design phase to determine whether further design modifications can be implemented to reduce Councils risk exposure.

TABLE 4.13: SHIRLEY STREET PUMP FAILURE FLOOD LEVELS

Design Event	Pump Failure Peak Flood Level (m AHD)	Pump Failure Flood Level Afflux to Baseline (m)	Pump Failure Flood Level Afflux to Design Upgrade (m)
10% AEP	1.99	-0.14	0.42
1% AEP	2.04	-0.16	**0.32**

TABLE 4.14: BUTLER STREET (TOWN CENTRE) PUMP FAILURE FLOOD LEVELS

Design Event	Pump Failure Peak Flood Level (mAHD)	Pump Failure Flood Level Afflux to Baseline (m)	Pump Failure Flood Level Afflux to Design Upgrade (m)
10% AEP	2.29	0.590	1.051
1% AEP	2.45	0.186	**0.690**

4.8.2 Outlet Blockage Upgrade Scenario

A culvert outlet blockage scenario was modelled to understand the change to flood behaviour under fully blocked conditions for any pipes discharging to the Belongil Creek floodplain or Clarkes Beach. Under this scenario it is assumed that all pumps continue to operate.

TABLE 4.15 AND

Table 4.16 summarise the flood levels and flood level afflux at the Middleton Street and Cowper Street sag points for the 10% and 1% AEP design events. The afflux mapping is provided in Appendix C.

The flood level afflux results indicate that properties surrounding Middleton Street are anticipated to experience up to 70mm additional flooding in the 10% AEP under fully blocked conditions when compared to the design baseline scenario. The Cowper Street sag is observed to experience flood level reductions even under outlet blockage conditions. The proposed detention basin has created a new hydraulic connection between the Cowper Street sag and Middleton Street sag that was not present in the Design Baseline Scenario where the stored water levels at the Cowper Street sag would be higher than the Middleton Street sag stored water levels, this is resulting in increased flood levels over Middleton Street when the detention basin is unable to free drain.

The Town Centre and Shirley Street systems are unaffected by outlet blockage conditions as the floodgate outlets into the Belongil Creek floodplain are not observed to flow in the Design Upgrade Scenario model as the stored water levels behind the level are not sufficient to drive flow through the flap gate.

It is recommended that the impacts generated by outlet blockage are investigated at the detailed design phase to determine whether any modifications or design improvements can be implemented to reduce the flood level increases associated with full outlet blockage conditions.

Consideration of the likelihood of a blockage and pump failure scenario should also be made as this may potentially cause worsening of flooding across the town centre area.

TABLE 4.15: MIDDLETON STREET SAG OUTLET BLOCKAGE FLOOD LEVELS

Design Event	Outlet Blockage Peak Flood Level (mAHD)	Outlet Blockage Flood Level Afflux to Baseline (mAHD)	Outlet Blockage Flood Level Afflux to Design Upgrade (mAHD)
10% AEP	3.01	0.07	0.18
1% AEP	3.21	0.05	**0.60**

TABLE 4.16: COWPER STREET SAG OUTLET BLOCKAGE FLOOD LEVELS

Design Event	Outlet Blockage Peak Flood Level (mAHD)	Outlet Blockage Flood Level Afflux to Baseline (mAHD)	Outlet Blockage Flood Level Afflux to Design Upgrade (mAHD)
10% AEP	3.01	-0.30	0.40
1% AEP	3.22	-0.30	**0.20**

4.9 Flooding Discussion

There are several areas that warrant significant further analysis in refinement of the scheme in detailed design. These are:

- Alternative upgrade options for Cowper Street and further increasing outlet size.
- Low tailwater sensitivity.
- Impacts behind levees over dwellings.

4.9.1 Cowper Street Alternative Options

As outlined in the hydraulic results section the proposed drainage upgrades for the Cowper Street study area are not sufficient to provide 10% AEP immunity to all surrounding properties affected by flooding in the Design Baseline Scenario due to the low-lying topography. Alternative options could be investigated to confirm whether other cost-effective solutions are available. The current design has been prepared generally in accordance with the original strategy, noting that changes to the Sandhills wetland have resulted in less flood storage than originally recommended by the strategy. The current strategy provides approximately 13ML of additional storage and it is estimated that a further 11ML may be required to achieve the 10% AEP desired standard of service (24ML total required below approximately RL 2.1m AHD).

Other solutions may include increased flood storage and/or pumping arrangements, it is noted that at the time of this report Council have requested alternative options to be investigated following completion of the concept design such as gravity drainage and pumping considerations. Council have advised that pursuing additional flood storage between Middleton and Cowper Street is not feasible due to space and land tenure constraints.

4.9.2 Sensitivity to Low Tailwater Boundary Conditions (Normal Tide levels)

The pump failure sensitivity results demonstrated that the flood levels associated with pump failure are lower than the baseline scenario flood levels. Therefore, the failure of the pumps does not expose residents to increased flood inundation risk. However, the modelling scenarios undertaken assumed tidal ingress under storm tide conditions which produce significant flood inundation over the Shirley Street area. This is mitigated by the proposed levee, and the local runoff trapped by the levee is not significant enough to cause flood level impacts, it is recommended that this is considered and assessed as part of the detailed design phase.

However, it was identified that under normal tide conditions during an intense local storm, the trapped local runoff may be significant enough to cause increased local flood risk for residents compared to the baseline, noting that this would be dependent on the performance of the flood gate culverts discharging into the Belongil Creek floodplain. Detailed design should address this impact and consider additional sensitivity modelling of normal tide conditions (HAT of approximately 1.10m AHD) for the 50% AEP, 10% AEP and 1% AEP to confirm if any stormwater network outlets require size increase to mitigate any impacts caused by trapped drainage behind the levee.

4.9.3 Levee Impacts – Pump Failure

The pump failure flood afflux results show minor impacts, generally characterised by an extended floodplain rather than increases to existing inundation depths, over the buildings located between Wordsworth Street and Butler Street because of the proposed levee. In addition, the afflux shows impacts between Milton Street and Wordsworth Street however these are observed to not conflict with any habitable dwellings. It is recommended that the detailed design phase investigate opportunities and design modifications such that these impacts are no longer present in the design upgrade scenario.

Like the Shirley Street levee impacts noted above, the pump failure sensitivity also showed increased flood levels in the 1% AEP extending over the property immediately north of the Town Centre pump station. Impacts also exist in the 10% AEP storm however these do not interact with the building in this event. It is recommended that the detailed design phase consider any opportunities or modifications to the design to resolve these predicted flood impacts.

5. FLOOD DAMAGE COSTS ASSESSMENT

5.1 Background

5.1.1 Flood Damages Assessment

This flood damage assessment has been developed to establish an estimation of the economic costs associated with a flood event and inform a cost-benefit analysis (CBA) of any proposed drainage upgrade works.

The New South Wales Department of Planning and Environment (DPE) has developed a Flood Damage CBA Tool to aid in the estimation of flood damages (NSW DPE July 3 2023 Draft Version). Within this spreadsheet, flood damages are classified into direct and indirect damages, with different costs associated to each. Direct costs are further classified into structural, internal and external damages. Indirect costs include clean-up costs and loss of trading income, these are typically more difficult to quantify than the direct costs associated with the inundation level of an individual building. A breakdown of the inputs and parameters included within this Flood Damages Assessment Tool is described in following sections.

The determinations made within this report are an estimation of damages. It is difficult to directly quantify damages from a known flood event, as such there is an expected level of inaccuracy to estimations from a predicted event. There are numerous assumptions that have been made to determine these values which are discussed throughout this report.

Within the flood damages assessment spreadsheet, the inflation rate is calculated to accurately reflect the damages at the time of analysis. The Average Weekly Earnings (AWE) and the Consumer Price Index (CPI) inflation rate were determined to be 9.00% and 10.25% respectively. At the time of analysis, 2023 data was unavailable, thus the analysis has been undertaken in 2022-dollar values.

Properties that lie within the flood model boundary have been included within the Flood Damages Assessment, as shown in Figure 5.1

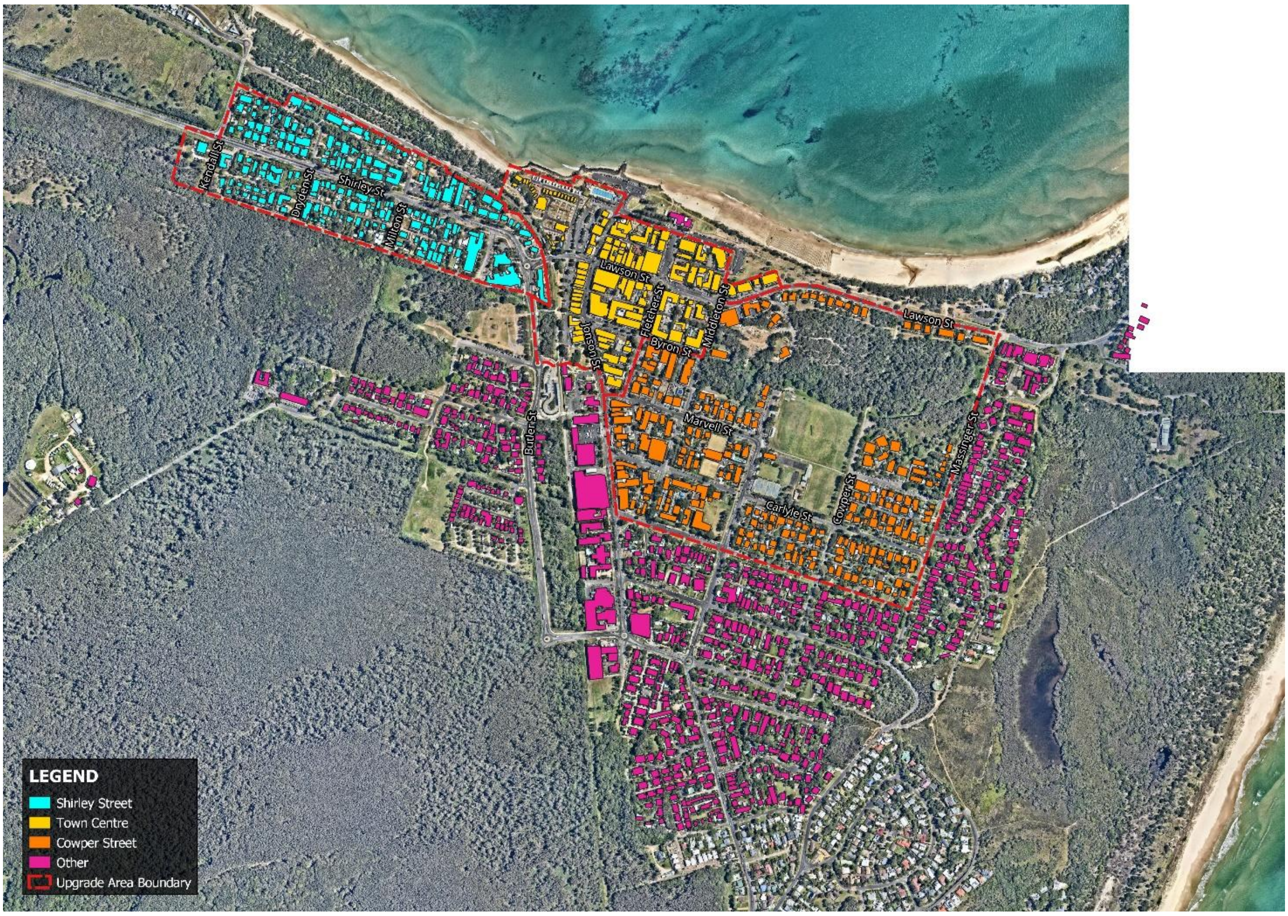

Figure 5.1: Flood Assessment Boundary and Buildings Footprint

5.2 Input Data

5.2.1 Property and Ground Level Data

To estimate flood damages the following property inputs are required:

- Storeys.
- Floor Level.
- Ground Level.
- Property Type.
- Area.

The spatial buildings layer was compared to aerial imagery and modified to ensure an appropriate representation of floor area is present, it is noted that the buildings spatial layer was developed using automatic processing tools and therefore will inherently contain errors unless appropriate quality reviews are undertaken. Floor level survey data supplied by Byron Shire Council included the floor level, ground level, storeys and property type for approximately 50% of properties within the Study Area. Google Maps and Street View were used to identify any information required for the flood damages estimate that was not included in the supplied data from Council. Floor levels were assigned using assumed depths above ground levels for different dwelling types, these are summarised below:

- Slab – 0.3m on ground level.
- Short Stumps – 1m on ground level.
- High Stumps – 2m on ground level.

The ground level for non-surveyed properties was adopted as the average ground level across the footprint of the associated building or dwelling. Any building footprints with an area less than 50sqm were removed from the set. A total of 1,177 properties were included in the flood damages estimate covering the full Byron Bay township, it is noted that these buildings may lie outside the modelled flood extent and does not represent the number of inundated properties.

5.2.2 Flood Level Data

The flood damage estimate is calculated based on the flood depths and levels at each structure for the range of modelled AEPs including the PMF, 0.05%, 0.1%, 1%, 2%, 5%, 10%, 20% and the 50% AEP design events.

5.3 Damage Curves

5.3.1 Residential

The residential components of the flood damages estimate are categorised as single storey, double storey, multi-unit and townhouse structures. The damage curves for single and double storey structures are provided in Figure 5.2 and Figure 5.3, respectively. In the absence of property areas, the default is to assign a "size" to each property, the corresponding areas are provided in Table 5.1.

The average area for the single storey properties included in this assessment was observed to be approximately 195m². This falls between the 'medium' and 'recommended' damage curves on Figure 5.2 and represents an average economic cost of $215,000 per single storey residential dwelling for a structure inundation depth of 1.0m.

The average area for a double story property within the assessment area was observed to be approximately 225m². Thus, a typical double storey residential property with a structure inundation depth of 1m would incur total damages of approximately $110,000, as shown in Figure 5.3.

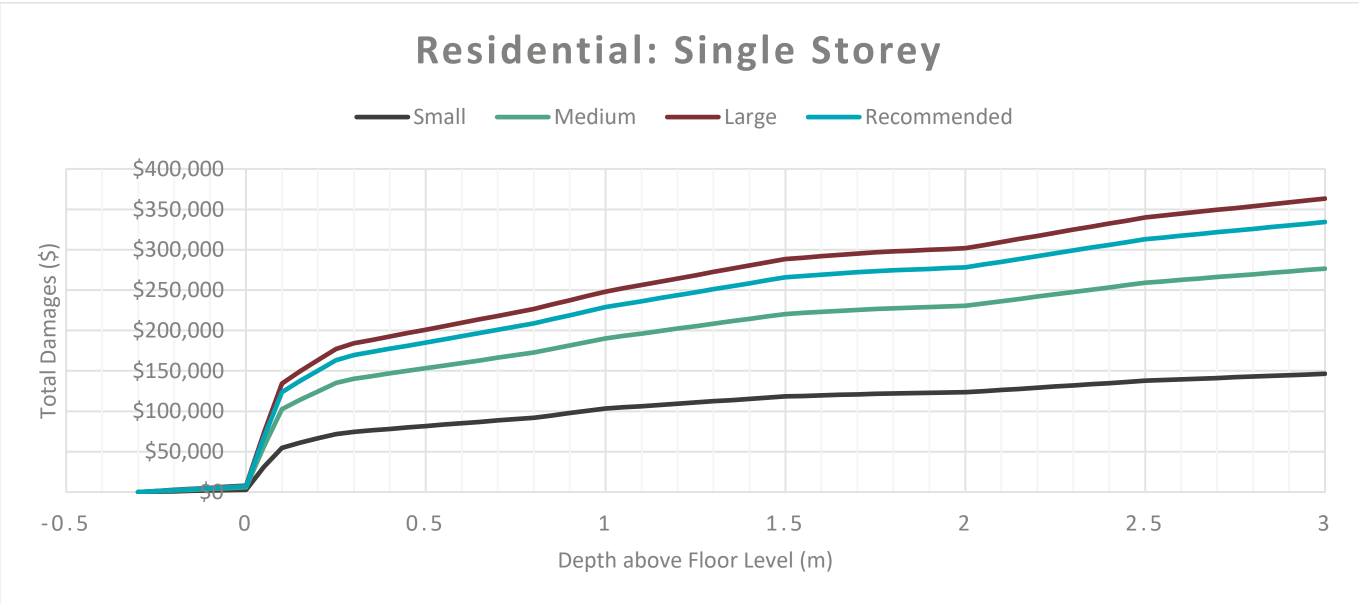

FIGURE 5.2: RESIDENTIAL DAMAGE CURVE – SINGLE STOREY

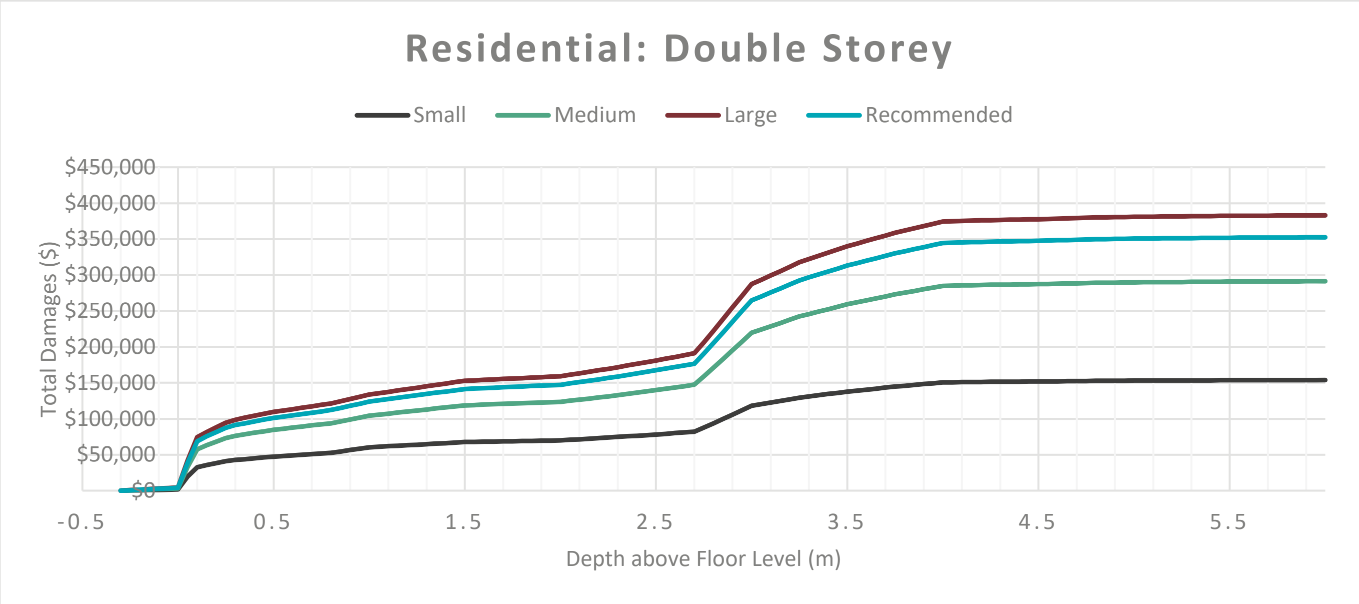

FIGURE 5.3: RESIDENTIAL DAMAGE CURVE – DOUBLE STOREY

TABLE 5.1: AREA ASSIGNED TO PROPERTY TYPE - RESIDENTIAL

Size	Small	Medium	Large	Default	Units	Townhouse
Area (m^2)	90	180	240	220	100	160

The Flood Damages Estimation Tool includes structural, internal, external and indirect costs within these residential damage curves. The structural component describes a replacement value per m^2 for each property type. Internal costs, depicting the loss of contents, are assigned a value of $490/$m^2$. It is assumed that at a flood depth of 0.3m, external damages of $15,000 are incurred. Indirect costs, typically used to depict the required clean-up costs, are allocated at $15,000 per property where above floor flooding exists.

5.3.2 Commercial

The commercial properties damage curve is provided in $/$m^2$, as shown in Figure 5.4. The areas assigned to each property type are shown in Table 5.2. The average commercial property within the assessment area was 645m^2, this falls in the medium-to-high category. As such it would be expected that a typical commercial property, with structure inundation depth of 1m would incur total damages of $1,061,025.

The structural, internal and external costs associated with the commercial damage curve remain the same as the residential. The indirect costs are defined as 30% of direct actual damages to account for clean-up and loss of trading costs.

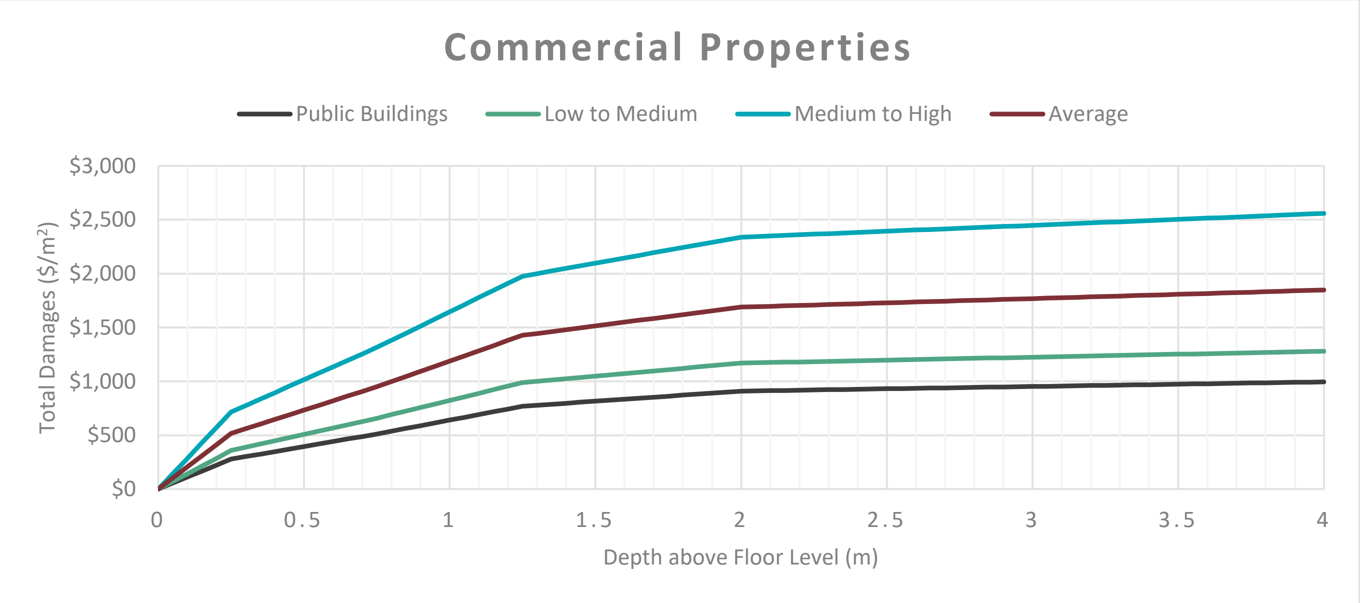

FIGURE 5.4: COMMERCIAL DAMAGE CURVE

TABLE 5.2: AREA ASSIGNED TO PROPERTY TYPE - COMMERCIAL

Size	Public Buildings	Low to Medium	Average	Medium to High
Area (m^2)	418	186	418	650

5.4 Exclusions

The flood damage estimation method does not consider the following:

- Risk to life factors. To include a risk to life factor within a flood damages assessment a "Hazard Category" must be assigned to each property within the assessment area. This has not been undertaken for this damage's estimation, as such, risk to life considerations are not included within this estimate.
- In many flood damage assessments, an actual vs potential ratio is utilised to highlight the differences in damages between an inexperienced flooding community and a flood-prone community. Factors such as warning time, warning systems and flooding education can be considered to develop a ratio of potential damages that could be expected in an actual setting. The purpose of the flood damages assessment conducted within this report is to compare the damages between the existing case and the proposed development. A potential vs actual ratio has not been used in either case.

5.5 Design Baseline Scenario Flood Damages

5.5.1 Total Damages

The total flood damage estimate for the entire township by AEP is provided in Table 5.3.

TABLE 5.3: DESIGN BASELINE SCENARIO TOTAL FLOOD DAMAGE ESTIMATE BY AEP

AEP	No. Properties Flooded Above Ground	No. Properties Flooded Above Floor	Average Over Floor Depth (m)	Structural	Internal	External	Total
PMP	554	321	0.54	$88,265,000	$11,073,000	$4,778,000	$104,116,000
0.05%	416	149	0.34	$31,925,000	$2,746,000	$2,398,000	$37,070,000
0.1%	401	143	0.32	$29,026,000	$2,565,000	$2,379,000	$33,970,000
1%	348	96	0.23	$12,909,000	$1,707,000	$1,813,000	$16,429,000
2%	326	86	0.21	$9,585,000	$1,464,000	$1,719,000	$12,767,000
5%	290	62	0.22	$6,096,000	$1,044,000	$1,435,000	$8,575,000
10%	272	54	0.22	$7,180,000	$1,035,000	$1,341,000	$9,556,000
20%	234	36	0.23	$5,664,000	$687,000	$944,000	$7,294,000
50%	179	25	0.25	$5,067,000	$541,000	$755,000	$6,363,000

5.5.2 Average Annual Damages Total

The Average Annual Damages Total (AADT) represents an annualised economic cost associated with each event, this leverages off the probabilistic nature of different flood events occurring. As the likelihood of a storm event occurring increases (AEP decreasing) the influence on the AADT is generally observed to decrease.

A summary of the AADT and the contribution to the AADT estimate for each design event AEP is provided in Table 5.4 and the total flood damage estimate for each study area is summarised in Table 5.5. The 'Other' classification in Table 5.5 is used to represent buildings within the township but external to the areas of influence within each Study Area as shown in Figure 5.1.

The results indicate that the 20% AEP design storm event has the greatest influence on the AADT estimate, therefore, by mitigating events up to the 10% AEP is likely to have a high benefit to the AADT.

TABLE 5.4: DESIGN BASELINE SCENARIO - AADT ESTIMATE BY AEP

AEP	Contribution to AADT ($)	Contribution to AADT (%)	Structural	Internal	External
PMP	$35,000	0.6%	$30,000	$3,000	$2,000
0.05%	$18,000	0.3%	$15,000	$1,000	$1,000
0.1%	$227,000	4.0%	$189,000	$19,000	$19,000
1%	$146,000	2.6%	$112,000	$16,000	$18,000
2%	$324,000	5.7%	$235,000	$38,000	$47,000
5%	$456,000	8.0%	$332,000	$52,000	$69,000
10%	$849,000	14.9%	$642,000	$86,000	$114,000
20%	$2,051,000	35.9%	$1,610,000	$184,000	$255,000
50%	$1,603,000	28.1%	$1,267,000	$135,000	$189,000

TABLE 5.5: DESIGN BASELINE SCENARIO - AADT BY STUDY AREA

Location	AADT Estimate
Shirley Street	$1,047,000
Town Centre	$3,203,000
Cowper Street	$888,000
Other township (out of study area)	$571,000

5.6 Design Upgrade Scenario Flood Damages

5.6.1 Total Damages

The total flood damage estimate for the Design Upgrade Scenario is summarised in Table 5.7. These values represent both residential and commercial properties, the breakdown of direct and indirect costs is also provided.

The total flood damage estimate for the entire township by AEP is provided in Table 5.6.

TABLE 5.6: DESIGN UPGRADE SCENARIO TOTAL FLOOD DAMAGE ESTIMATE BY AEP

AEP	No. Properties Flooded Above Ground	No. Properties Flooded Above Floor	Average Over Floor Depth (m)	Structural	Internal	External	Total
PMP	556	319	0.53	$85,409,000	$11,287,000	$4,778,000	$101,474,000
0.05%	382	95	0.22	$10,733,000	$1,654,000	$1,341,000	$13,728,000
0.1%	362	76	0.23	$8,545,000	$1,535,000	$1,284,000	$11,364,000
1%	264	40	0.27	$5,633,000	$855,000	$850,000	$7,338,000
2%	239	35	0.24	$4,961,000	$608,000	$793,000	$6,362,000
5%	191	23	0.22	$3,506,000	$306,000	$472,000	$4,284,000
10%	173	21	0.30	$4,954,000	$271,000	$397,000	$5,621,000
20%	140	15	0.36	$4,260,000	$74,000	$264,000	$4,599,000
50%	90	12	0.37	$4,000,000	$74,000	$189,000	$4,263,000

5.6.2 Average Annual Damages Total

A summary of the AADT and the contribution to the AADT estimate for each design event AEP is provided in Table 5.7 and the total flood damage estimate for each study area is summarised in Table 5.8.

Similar to the Design Baseline Scenario the results indicate that the 20% AEP design storm event has the greatest influence on the AADT estimate.

TABLE 5.7: DESIGN UPGRADE SCENARIO - AADT BY AEP

AEP	Contribution to AADT ($)	Contribution to AADT (%)	Structural	Internal	External
PMP	$29,000	0.8%	$24,000	$3,000	$2,000
0.05%	$6,000	0.2%	$5,000	$1,000	$1,000
0.1%	$84,000	2.4%	$64,000	$11,000	$10,000
1%	$69,000	2.0%	$53,000	$7,000	$8,000
2%	$164,000	4.7%	$127,000	$14,000	$19,000
5%	$249,000	7.1%	$211,000	$14,000	$22,000
10%	$515,000	14.6%	$461,000	$17,000	$33,000
20%	$1,330,000	37.8%	$1,239,000	$22,000	$68,000
50%	$1,071,000	30.5%	$1,000,000	$18,000	$47,000

TABLE 5.8: DESIGN UPGRADE SCENARIO - AADT BY STUDY AREA

Location	Average Annual Flood Damages Estimate
Shirley Street	$16,000
Town Centre	$2,720,000
Cowper Street	$209,000
Other	$571,000

5.6.3 Average Annual Damage Benefits

The reduction in annual average damages costs based on the methods outlined in previous sections are summarised in Table 5.9.

The Shirley Street drainage upgrades are anticipated to provide the greatest reduction in theoretical damages costs associated with the range of modelled flood events with an annual return of approximately $2.3M and represents approximately 47% of the reductions across the entire Byron Bay project area.

The Town Centre drainage upgrades provide the least benefit to reduced flood damages to the order of approximately $0.5M.

The Cowper Street drainage upgrades provides a significant equivalent reduction of flood damages for the Study Area however provides a reduction value of approximately $0.7M which is within the same order of magnitude as the Town Centre improvements.

TABLE 5.9: DESIGN UPGRADE SCENARIO – AADT BY STUDY AREA

Location	Annual Average Flood Damages Estimate - Baseline case	Annual Average Flood Damages Estimate - Design case	Cost Reduction from Baseline	% AADT reduction
Shirley Street	$1,047,000	$16,000	-$1,031,000	-98%
Town Centre	$3,203,000	$2,720,000	-$483,000	-15%
Cowper Street	$888,000	$209,000	-$679,000	-76%
Other	$571,000	$571,000	$0	0%
TOTAL	**$5,709,000**	**$3,516,000**	**-$2,193,000**	**-38%**

6. PRELIMINARY COST BENEFIT ANALYSIS

A simplified cost benefit analysis of the different components of the scheme was undertaken to determine the relative benefits and value of the three different flood mitigation areas across the township. The cost benefit analysis has evaluated benefit cost ratios (BCR) using the following assumptions:

- Discount rate of 7%
- 30-year discount period.
- Consideration of capital cost only (operating cost, sustaining capital and maintenance costs are ignored).
- Consideration of annual average damage cost reduction as benefits (baseline AAD minus upgraded AAD).

Based on the reduction in AADT (benefits), and comparison of the capital costs estimated in the concept engineering report (QC2003_002-REP-004) has been made to inform a BCR. This is summarised in Table 6.1.

This analysis provides an indication of the relative benefits of the upgrades across each of the areas for comparison purposes. Based on the simple cost benefit comparison, the area with the greatest benefit to cost ratio is the Middleton/Cowper Street area (area 3). This is driven by the high cost of implementation of the other areas which is ultimately due to the capital cost of pump stations. The next best area is Shirley Street (area 1), and this has a better benefit to cost ratio than the town centre due to the number of properties benefited by a reduction in regional flooding achieved by the flood levee. The town centre area (area 1) has the lowest benefit to cost ratio due to the high-cost infrastructure compared to the number of properties that are benefitted.

TABLE 6.1: DESIGN UPGRADE SCENARIO - AADT BY STUDY AREA

Location	Reduction in AADT	Estimated Capital Cost	BCR	Internal Rate of Return
Shirley Street (area 1) with 1% AEP Levee	$1,031,000	$24.9M	0.55	1.6 %
Shirley Street (area 1) with 10% AEP levee	$1,031,000	22.4M	0.68	2.5%
Town Centre (area 2)	$483,000	$15.8M	0.41	-0.6
Middleton /Cowper Street (area 3)	$679,000	$7.3M	1.12	8.6%
TOTAL	$2,193,000	$48.0M (1% AEP levee) $45.5M (10% AEP levee)	0.61 0.64	2.3% 2.8%

The benefit cost analysis is a simple metric for comparison purposes against the different areas of the scheme only. It does not consider the following intangible items:

Intangible items that may improve BCR

- Likely climate change impacts over time would increase damage cost and net benefits.
- Potential increases to land values or residual value of the scheme beyond the 30 year assessment period (noting 80 year design life).
- Social and community values.
- Public health impacts.
- Mental health.
- Risk to life and limb.
- Tourism.

Intangible items that worsen BCR

- Maintenance costs.
- Operating costs (this may be a significant exclusion for pump stations).
- Sustaining capital for infrastructure renewal in the design life.

Another metric that has been used to assess the relative performance of the upgrade works across the three areas is cost versus number of properties flood free. This is summarised in Table 6.2 for the 10% AEP event. This yields a similar outcome to the benefit cost ratio and demonstrates the area 3 comparison cost per property is the lowest, flowed by area 2 and area 1 being the highest cost.

TABLE 6.2: DESIGN UPGRADE SCENARIO COMPARISON COST PER PROTECTED PROPERTY (10% AEP)

Location	Estimated Capital cost	10% AEP number of properties protected	Comparison Cost per flood free property
Shirley Street (area 1 – 1% AEP levee)	$24.9M	53	$470,000
Shirley Street (area 1 – 10% AEP levee)	$22.4M	53	$423,000
Town Centre (area 2)	$15.8M	32	$494,000
Middleton /Cowper Street (area 3)	$7.3M	14	$521,000
TOTAL	**$48.0M (1% AEP levee)** **$45.5M (10% AEP levee)**	**99**	**$485,000** **$460,000**

These results demonstrate that the Middleton Street and Cowper Street upgrades (area 3) have the highest value of the three areas and should be progressed as a priority, followed by Shirley Street and the Town Centre upgrades.

It is noted the benefit cost assessment is dominated by the capital costs, and particularly pump station costs. If significant optimisation or consolidation of pump arrangements can be made to reduce costs, the relative values may change.

7. DISCUSSION

The flooding assessment of the baseline is considered conservative and rigorous to provide a robust basis for sizing of the scheme and to determine the costs and benefits of the scheme.

Further work could be undertaken to develop a more rigorous cost benefit analysis for the scheme; however, it is considered the simple methods used herein demonstrate the relative benefits of the various areas of the scheme and enable decisions on area prioritisation. It is noted the preliminary benefit cost ratio of the entire scheme is significantly less than 1.0 and it is considered unlikely to achieve a BCR greater than 1 upon further, more detailed analysis.

Additionally, the assessment demonstrates the pump stations are a significant component of the cost and feasibility of the scheme and further work at the detailed design phase to rationalise or reduce pump station costs would significantly benefit the project.

7.1 Climate Change

Climate change has been considered in developing the design basis of the upgrade and the approach to the concept design. The current design basis makes allowances for future development and climate influences such as rainfall intensity increases, without directly designing for sea level rise or other possible climate change factors. It is acknowledged in the concept design the upgrade is a significant improvement in providing climate change resilience to the township and provides a foundation for further future upgrades to address other climate change factors if they arise.

Two climate change horizons are currently in the Byron Bay LEP, a 30-year horizon (2050) and 80-year horizon (2100) which have increasingly significant predictions relating to flooding in Byron Bay over time. The two key focuses for future design that incorporates and accounts for additional impacts of climate change:

(1) An increase to baseline rainfall intensity due to more intense/ frequent storms (10% and 30% increase); and

(2) Sea level rise (0.4m and 0.9m increase).

Climate change flood model sensitivity analysis per the Byron Bay LEP (2014) for the 1% AEP event is not considered to influence the design outcomes of the scheme at this stage, and as such the modelling of these results is not necessary for the design analysis, however can be evaluated in future to assess resilience to various climate change conditions.

7.1.1 Baseline Rainfall Intensity Increase

The LEP requires consideration of 10% and 30% increase in baseline rainfall in the 30-year (2050) and 80-year (2100) horizons for climate change scenarios.

As previously discussed in the hydrology model section, there is in built conservatism in the concept design hydrology model base case which indirectly accounts for a potential 10% intensity increase in rainfall intensity in a 30-year horizon. This conservatism may already address the 30-year climate change horizon; however, this requires confirmation with further work on design IFD's which is being undertaken by Council.

If a 30% intensity increase were adopted looking toward the 80-year horizon this is likely to have significant impact on the design performance of the scheme, and the 10% AEP design standard would not be met if the higher rainfall estimates were adopted.

7.1.2 Sea Level Rise

The LEP requires consideration of 0.4m and 0.9m increase in base sea levels in the 30-year (2050) and 80-year (2100) horizons for climate change scenarios. This would increase the design tailwater levels significantly and profoundly change the design approach for the scheme.

The key impacts of sea level rise on the scheme would be:

- Flood levee's will be required to be significantly higher depending on the climate change scenario adopted for design.
- Pumps may require re-sizing to adjust for the additional lift required in the design event.
- Gravity drainage outlets and particularly the Clarkes beach outlet and Middleton/Cowper Street system may be dysfunctional in the design storm tide events.
- Additional pump stations may be required as more of the township will be considered 'low lying' and have no functional hydraulic grade to allow gravity drainage during daily high tide periods.

8. SUMMARY AND RECOMMENDATIONS

The flooding assessment of the preferred Byron Bay drainage strategy has provided a thorough, comprehensive evaluation of the scheme and has defined the flooding benefits achieved from implementation of the various system components across the township. New flood models have been developed that focus on the local catchments of the township and consider influence from regional flooding and ocean tide levels. The modelling adopts forward looking assumptions on land use and rainfall inputs and has been validated to the March 2022 flood event, which provided confidence that the modelled results generally align with the recorded flooding of this event.

Climate change has been considered in developing the design basis and the approach to the concept design. The current design basis makes allowances for future development and climate influences such as rainfall intensity increases, without directly designing for sea level rise or other possible climate change factors. It is acknowledged in the concept design the upgrade is a significant improvement in providing climate change resilience to the township and provides a foundation for further future upgrades to address other climate change factors if they arise in the future.

The flood assessment outcomes of the upgrade are summarised in Table 8.1and Table 8.2 which demonstrates significant improvements in flooding across the study area.

TABLE 8.1: DESIGN UPGRADE SCENARIO – NUMBER OF PROPERTIES PROTECTED

Location	10% AEP number of protected properties	1% AEP number of protected properties	March 2022 Event
Shirley Street (area 1)	-53	-44	-49
Town Centre (area 2)	-32	-30	-38
Middleton /Cowper Street (area 3)	-14	-10	-26
TOTAL	**-99**	**-84**	**-113**

TABLE 8.2: DESIGN UPGRADE SCENARIO - AADT BY STUDY AREA

Location	AADT Estimate - Baseline case	AADT Estimate - Design case	Reduction in AADT	% AADT reduction
Shirley Street	$1,047,000	$16,000	-$1,031,000	-98%
Town Centre	$3,203,000	$2,720,000	-$483,000	-15%
Cowper Street	$888,000	$209,000	-$679,000	-76%
Other areas	$571,000	$571,000	$0	0%
TOTAL	**$5,709,000**	**$3,516,000**	**-$2,193,000**	**-38%**

Furthermore, using capital costs of the scheme developed in the Engineering report, a simple benefit cost assessment has been undertaken for the three project areas and has identified that the highest value area for flood mitigation is area 3 (Middleton and Cowper Street) followed by area 2 (Shirley Street) then area 1 (the Town Centre) as shown in Table 8.3.

TABLE 8.3: COMPARISON COST PER FLOOD FREE PROPERTY (10% AEP)

Location	Estimated Capital cost	10% AEP number of protected properties	Scheme Cost per property protected (10% AEP)	Preliminary BCR
Shirley Street (area 1)	$24.9M	53	$470,000	0.55
Shirley Street (area 1)	$22.4M	53	$423,000	0.68
Town Centre (area 2)	$15.8M	32	$494,000	0.41
Middleton /Cowper Street (area 3)	$7.3M	14	$521,000	1.12
TOTAL	**48.0M (1% AEP levee)** **45.5M (10% AEP levee)**	**99**	**$485,000** **$460,000**	**0.61** **0.64**

Design sensitivity analysis was undertaken on various aspects of the scheme. Two levee heights were evaluated (1% AEP and 10% AEP) and it was found the 10 % AEP levee achieved most of the performance outcomes of the 1% AEP levee. The analysis does not capture the additional risks associated with the lower levee height such as reduced freeboard and the potentially greater future upgrades required.

Other sensitivity analysis evaluated were as follows:

- Pump Failure: In the event of pump failure, it is predicted that flood levels will still be lower than the existing flood levels such that there is generally no worsening across the township when compared to the regional flooding. It is noted that there are some minor localised increases around the Shirley Street levee and Town Centre pump station that should be investigated further at the Detailed Design phase and any local impacts mitigated.
- Stormwater Outlet Blockage: In the event of total outlet blockage, it is predicted most areas do not experience worsening from baseline and the upgrades provide a net benefit to the catchment. Properties in the Middleton Street area may experience up to 70mm additional flooding in the 10% AEP due to the new hydraulic connection between the Cowper Street sag and Middleton Street sag. It is recommended that potential strategies to resolve these impacts are investigated at the detailed design phase. It is also noted that whilst the change to hydraulic performance is a result of the earthworks at the Middleton Street sag, none of the proposed drainage works will increase the risk of blockage at the Clarkes Beach outlet.

Some flooding challenges remain in various parts of the study area and some property flooding is not feasible to mitigate due to low lying properties with trapped road sag points (Town Centre) or overland flow paths (Middleton and Carlyle Street) from upstream of the study area.

Furthermore, additional design development and optioneering may be necessary to achieve the target 10% AEP flood immunity standard of service for the Middleton to Cowper Street overland flow path and wetland, which is limited by land available for the upgrade and the design strategy in the area. A flood pump station in the area may also be required to achieve the desired 10% AEP flood performance for flood prone houses along Marvell and Middleton Street. The feasibility of the overland flow path/wetland located adjacent Middleton Street may be marginal based on the low levels and reliance on backflow prevention.

Based on the flooding assessment and simplified cost benefit analysis, it is apparent the scheme costs are significant, and the high cost of stormwater pump stations may be cost prohibitive as they outweigh the economic benefits if considered in isolation. Further rationalisation and optimisation of pumps stations is required to determine if the areas relying on pump stations are feasible.

Overall, it is considered the preferred Byron Bay drainage strategy provides major improvement to flooding across the study area. Additional engineering feasibility analysis, costing and planning for detailed design has been documented in the Engineering assessment report.

9. REFERENCES

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10. QUALIFICATIONS

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