Richmond Valley Flood Study

Volume 1: Flood Study Report (Final)

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Synopsis	Technical report documenting the modelling and mapping undertaken for the Richmond Valley Flood Study.

Amendment Record

The Amendment Record below records the history and issue status of this document.

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00	25 February 2022	Richmond Valley Council	Draft Report
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Executive Summary

BMT was commissioned by Richmond Valley Council (Council) to undertake the ‘Richmond Valley Flood Study’. The study has been delivered with financial support from the New South Wales (NSW) State Floodplain Management Program and has been undertaken in accordance with the principles of the NSW Government’s Flood Prone Land Policy.

The study is presented in two volumes; Volume 1 is the technical report (this document) and Volume 2 contains the design flood mapping.

In February 2022, the study was released as a Draft Report. In the days following, northern NSW experienced record breaking rainfall which led to severe and damaging flooding across northern NSW including within the Richmond River catchment. Given the significance of this flood event, the Draft Report and associated analyses were updated to account for this event within the assessment.

As part of the study a new catchment wide hydrology model has been developed along with a new hydraulic model that defines flood behaviour across the Richmond Valley LGA. These models will replace multiple models previously used by Council within different parts of the LGA.

The hydrologic and hydraulic models were calibrated to the historic flood events of January 2008, May 2009, March/April 2017 and February/March 2022. The calibration exercise involved applying recorded rainfall data in the models and comparing modelled flood levels with recorded data at river gauges and surveyed debris marks.

The hydraulic model provides a good calibration across the catchment for the events under consideration. The modelled peak flood levels generally meet or exceed the desired tolerances against recorded levels of +/- 0.3m in urban areas and +/-0.5m elsewhere. The hydraulic model also reproduced the timing and shape of the floods at the gauges and is therefore considered to provide a reliable representation of the flood propagation speed.

The calibrated flood models were used to determine design floods, expressed in terms of their annual exceedance probabilities (AEP). Design floods for the 5%, 2%, 1% and 0.2% AEPs are assessed as well as the probable maximum flood (PMF). Furthermore, these events with allowances for climate change are also assessed. The design floods have been derived in accordance with the Australian Rainfall and Runoff 2019 guideline which represents current best practice across Australia.

Flood frequency analyses (FFA) were undertaken as a second, independent method to derive design flood estimates. These take into account the full historical record of flooding, including the February/March 2022 event, and were undertaken at gauges with a sufficient length of record and which had a reliable rating curve to convert recorded flood levels into estimates of peak flow.

Peak FFA derived flows at Casino (Richmond River) and East Gundurimba (Wilsons River) indicated that modelled flows derived from design rainfall inputs understated the peak flows. Adjustments were therefore made to the modelled design flows to reconcile results with the more reliable FFA derived peak flow estimates.

The design flood results are mapped across the LGA for the following outputs:
- Peak flood levels
- Peak flood depths
- Peak flood velocities.

The model results were also processed to generate additional information to inform emergency management and land use planning. These additional outputs are summarised as follows:

- Flood Hazard mapping which categorises the floodplain into six classes of hazard and is based on current best practice guidelines (Australian Institute for Disaster Resilience (AIDR), 2017).
- Flood Planning Level output. This uses the 1% AEP event with climate change as a basis for the mapping and applies a 500mm freeboard allowance.
- Floodplain Function mapping which considers the hydraulic function of the floodplain in accordance with the NSW Floodplain Development Manual. This categorises the floodplain into floodway, flood storage and flood fringe.
- Mapping to support flood warning and emergency management comprising the delineation of flood islands which are areas that become isolated during floods.
- Counts of properties predicted to have above floor inundation during design floods. This is tabulated by suburb.
- Mapping showing locations where key roads are vulnerable to being cut by floodwater.

An analysis has also been undertaken where the new flood levels have been compared to previous flood levels used by Council. In general, this showed that the new flood levels are typically similar or higher to those derived in previous assessments.

The sensitivity of the model to the following aspects was also assessed:

- Future climate change
- Structure blockage considerations
- Assumed hydraulic roughness
- Timing of storm surge peak relative to catchment flood peak.

The climate change scenarios, undertaken on all events, incorporate a 10% increase in rainfall intensity and a 0.9m allowance for sea level rise as requested by Council. The lower parts of the catchment were most sensitive to climate change (showed greater increases in flood level). This is driven by the sea level rise allowance. Elsewhere, increases in peak level under a future climate were more modest, typically between 0.1m and 0.3m.

In summary, the updated Richmond Valley Flood Study delivers Council a hydraulic model based on the latest advancements in computer modelling and for which good calibration has been demonstrated, including to the significant February/March 2022 event. The model has enabled the flood behaviour across the LGA to be mapped across a greater scale and at a higher resolution then was previously possible.

The outputs of the study will allow for a consistent set of assumptions and standards to be applied across the Richmond Valley LGA with regards to flooding. It is recommended that the flood study assessment underpins the basis of any future revisions to floodplain risk management plans.

Table 1 summarises the peak design flood levels at key gauge locations within the catchment. Given the significance of the February/March 2022 event, recorded levels at gauges are also provided in

Table 1. Where a gauge has failed and there is no nearby surveyed flood level, the modelled result has been quoted for the February/March 2022 event.

Table 1. Peak Design Flood Levels (m AHD)

Location	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF	Feb/Mar 2022 (recorded)
Casino (Irving Bridge)	22.36	23.89	24.29	24.63	25.86	24.39^
Casino (Gauge)	21.39	22.68	22.96	23.30	24.31	22.89*
Yorklea	23.50	23.70	23.80	23.99	24.77	24.03
Tatham Bridge	11.63	12.06	12.35	12.92	15.09	12.71^
Codrington	8.45	8.56	8.64	8.89	10.76	8.85^
Coraki (MHL gauge)	6.18	6.32	6.45	6.73	10.23	6.83
Bungawalbin Junction	4.76	5.33	5.70	6.17	10.16	6.51
Woodburn	3.58	4.18	4.77	5.89	10.03	6.36
Broadwater	2.22	3.40	4.07	5.23	9.37	5.41^
Rappville Rail Bridge	46.66	46.77	46.87	47.05	48.01	47.11^
Rappville Summerland Way (Myrtle Creek)	37.55	37.65	37.73	37.89	38.62	38.03
Neileys Lagoon Road	9.33	9.83	10.10	10.57	12.77	11.31#
Rocky Mouth Creek Floodgates	4.11	4.55	4.97	5.96	10.06	5.12
Iron Gates	1.58	2.64	3.15	4.26	8.70	4.42
Evans Head (Fishing Co Op)	1.25	2.21	2.47	3.23	7.34	2.66/2.82#

*** WaterNSW adjusted level based on flood mark**
^ Modelled result
 # Surveyed peak flood mark

Contents

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Glossary

Term	Description
Annual Exceedance Probability (AEP)	The chance of a flood of a given size (or larger) occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (i.e. a 1 in 20 chance) of a peak discharge of 500 m^3/s (or larger) occurring in any one year. This report uses AEP terminology to express the likelihood of occurrence of a flood event (see also Average Recurrence Interval).
Areal Reduction Factor	A factor to convert a point rainfall estimate into an area averaged estimate.
Attenuation	Weakening in force or intensity.
Australian Height Datum (AHD)	Common national survey datum corresponding approximately to mean sea level.
Australian Rainfall and Runoff (ARR)	Engineers Australia publication pertaining to rainfall and flooding investigations in Australia
Average Recurrence Interval (ARI)	A statistical estimate of the average number of years between the occurrence of a flood of a given size or larger than the selected event. For example, floods with a flow as great as or greater than the 20-year ARI (5% AEP) flood event will occur, on average, once every 20 years. This report uses AEP terminology to express the likelihood of occurrence of a flood event. (See also Annual Exceedance Probability).
Breaklines	Survey strings used to define continuous linear features
Calibration	The adjustment of model configuration and key parameters to best fit an observed data set
Catchment	The area of land draining through the main stream (as well as tributary streams) to a particular site. It always relates to an area upstream of a specific location.
Critical duration	The critical duration is the design storm duration which provides the highest peak water levels (or flows) for a given design flood (e.g. 1% AEP) at a given location. For example, if the following design durations were modelled - 2-hour, 6-hour, 9-hour and 12-hour – and the 9-hour duration resulted in the highest peak water level at a given location then the critical duration for that location would be 9-hours.
Depth	The height or the elevation of floodwaters above ground level (in metres). Not to be confused with water level, which is the height of the water relative to a datum (not ground level).
Design flood/event	Hypothetical floods used for planning and floodplain management investigations. They may be comprised of a single design event or multiple events grouped into an ensemble. A design flood is defined by its probability of occurrence, for example the 1 in 100 Annual Exceedance Probability (AEP).
Ensemble	In the context of this study, an ensemble is a group of 10 temporal rainfall patterns which are used to determine a representative average flow/level.
Digital Elevation Model (DEM)	A three dimensional (3D) model of the ground surface elevation
Event (Flood)	Used (in the context of this study) to describe a flood occurrence. It can be a historical flood event or a design flood event.
Flood	Relatively high river or creek flows, which overtop the natural or artificial banks, and inundate floodplains and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
Flood behaviour	The pattern, characteristics and nature of a flood, including flood levels, velocities and flows.
Flood Frequency Analysis (FFA)	A statistical analysis technique used to estimate the magnitude or frequency of flooding.
Flood fringe	Flood prone land that is not designated as floodway or flood storage areas. These areas are within the floodplain, but generally recognised as suitable for in-fill development. (See also flood conveyance areas and flood storage areas).
Flood hazard	The potential for damage to property or risk to persons during a flood. Flood hazard is a key tool used to determine flood severity and is used for assessing the suitability of future types of land use.
Flood level	The height or elevation of floodwaters relative to a datum (typically the Australian Height Datum).
Flood Planning Levels (FPL)	Combination of flood levels derived from floods of specific AEPs (or historical flood events) plus freeboard selected for floodplain risk management purposes, as determined in management studies and incorporated in floodplain risk management plans. Selection of these levels should be based on an understanding of the full

Term	Description
	range of flood behaviour and the associated flood risk. It should also take into account the social, economic and ecological consequences associated with floods of different severities. Different FPLs may be appropriate for different categories of land use and for different flood plans. The concept of FPLs supersedes the "standard flood event". As FPLs do not necessarily extend to the limits of flood prone land, floodplain risk management plans may apply to flood prone land beyond that defined by the FPLs.
Flood prone land	Land susceptible to inundation by the probable maximum flood (also called flood liable land). Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Risk Management Plans should encompass all flood prone land (i.e. the entire floodplain).
Flood storage	Floodplain areas that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity. Loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. (See also flood fringe areas and flood conveyance areas).
Floodway	Those areas of the floodplain where a significant flow of water occurs during floods.
Freeboard	A factor of safety usually expressed as a height above the adopted flood level which helps determine the flood planning level. Selection of freeboard values consider factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
Hydraulic model	A computer model that uses data about the flow in streams and the terrain of a particular area to estimate flood heights, velocities and flow over time. In order to do this, the hydraulic model solves the equations for the conservation of mass and momentum / energy.
Hydrologic model	A computer model that uses rainfall data and estimates of the proportion of the rainfall which turns into runoff and the time which the runoff from each part of the catchment takes to flow into the stream to estimate flow in the stream over time.
Intensity Frequency Duration (IFD) Curve	A statistical representation of rainfall showing the relationship between rainfall intensity, storm duration and frequency (probability) of occurrence.
LiDAR (light detection and ranging)	Is a remote sensing (airborne) technology that measures distance by illuminating a target with a laser from a fixed wing aircraft and analysing the reflected light.
Probable Maximum Flood (PMF)	An extreme flood deemed to be the largest flood that could conceivably occur at a specific location. It is generally not physically or economically possible to provide complete protection against this flood event, but should be considered for emergency response etc. The PMF defines the extent of flood prone land (i.e. the floodplain).
Probable Maximum Precipitation (PMP)	The (theoretically) greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of year.
Rating curve	Is a graph of discharge versus stage (level) at a given location on a watercourse, usually at gauging stations.
Storm surge	The increases in coastal water levels above predicted astronomical tide level (i.e. tidal anomaly) resulting from a range of location dependent factors including the inverted barometer effect, wind and wave setup and astronomical tidal waves, together with any other factors that increase tidal water level.
TUFLOW	1D and 2D hydraulic modelling software used in this study. It simulates the complex hydrodynamics of floods and tides using the full 1D St-Venant equations and the full 2D free-surface shallow water equations.
Two dimensional (2D) flood model	Is a flood model which is able to simulate flood behaviour in two directions [such as across the floodplain or in wide waterways]. These models are capable of providing a detailed description of the flow in urban or rural floodplains and overbank areas.
URBS	Unified River Basin Simulator. A rainfall runoff routing hydrologic model (Carroll, 2020)
Validation	A test of the appropriateness of the adopted model configuration and parameters (through the calibration process) for other observed events.
Velocity	The speed at which floodwaters are moving (in metres per second). A flood velocity predicted by a 2D computer flood model is quoted as the depth averaged velocity, i.e. the average velocity throughout the depth of the water column. A flood velocity predicted by a 1D or quasi-2D computer flood model is quoted as the depth and width averaged velocity, i.e. the average velocity across the whole river or creek section.

Abbreviations

AEP	Annual Exceedance Probability
AHD	Australian Height Datum
AIDR	Australian Institute of Disaster Resilience
ARF	Areal reduction factor
ARI	Average Recurrence Interval
ARR1987	Australian Rainfall and Runoff 1987
ARR2019	Australian Rainfall and Runoff 2019
ARR Data Hub	Australian Rainfall and Runoff Data Hub
BoM	Bureau of Meteorology
CC3	Climate Change (Scenario 3)
CL	Continuing loss (rainfall)
CPU	Central Processing Unit
DAF	Decay Amplitude Factor
DEM	Digital Elevation Model
DIPNR	Department of Infrastructure, Planning and Natural Resources
DPIE	Department of Planning and Environment
EPW	Extreme Precipitable Water
FFA	Flood Frequency Analysis
FPL	Flood Planning Level
GEV	Generalised Extreme Value
GPU	Graphics Processing Unit
GTSMR	Generalised Tropical Storm Method Revised
HPC	Heavily Parallelised Compute
IFD	Intensity Frequency Duration
IL	Initial loss (rainfall)
LGA	Local Government Area
LiDAR	Light Detection and Ranging
LP3	Log Pearson 3
MAF	Moisture Adjustment Factor
MHL	Manly Hydraulics Laboratory
NSW	New South Wales
OEH	Office of Environment and Heritage
PHU	Pacific Highway Upgrade
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SGS	Sub Grid Sampling
TAF	Topographic Adjustment Factor
TIN	Triangular irregular network
URBS	Unified River Basin Simulator
WAE	Works as Executed
WBNM	Watershed Bounded Network Model

1 Introduction

1.1 Background

BMT was commissioned by Richmond Valley Council (Council) to undertake the 'Richmond Valley Flood Study'. The study has been delivered with financial support from the New South Wales (NSW) State Floodplain Management Program and has been undertaken in accordance with the principles of the NSW Government's Flood Prone Land Policy.

The overarching aim of the study is to provide updated flood mapping across Council's Local Government Area (LGA) for a range of design flood magnitudes, expressed in terms of the annual exceedance probability (AEP).

To achieve this, new hydrologic and hydraulic models have been developed taking advantage of the latest advancements in modelling software and the latest available topographic data. This has allowed the Richmond River floodplain to be mapped at a higher resolution and greater extent than that in prior flood studies.

A key part of the study has been the update of catchment hydrology to conform with the Australian Rainfall and Runoff 2019 (ARR2019) guideline (Ball et al, 2019) which represents current best practice in flood estimation within Australia. Previous flood assessments on the Richmond River were prepared in accordance with the now superseded Australian Rainfall and Runoff 1987 (ARR1987) guideline (Institution of Engineers, 1987).

This report documents the study along with the updated modelling and mapping outputs.

1.2 Floodplain Risk Management Process

Flooding in NSW is managed in accordance with the NSW Government's Flood Prone Land Policy (the Policy). The Policy is directed towards providing solutions to existing flooding problems in developed areas, understanding potential future increases in flood risk and ensuring that new development is compatible with its flood risk exposure and does not create additional flooding problems in other areas.

The NSW Government's 'Floodplain Development Manual' (DIPNR, 2005) supports the Policy by defining the responsibilities, roles and processes for the management of flood prone land in NSW. Under the Policy, the management of flood prone land is the responsibility of the local authority, in this case Richmond Valley Council, with technical and financial support from the NSW Government. This includes the development and implementation of local flood studies and floodplain risk management studies and plans to define and manage flood risk. These are prepared through the staged approach defined by the NSW Floodplain Management process shown in Figure 1.1.

The Richmond Valley Flood Study represents Stages 1 and 2 of the process and aims to compile relevant data in order to provide an improved understanding of flood behaviour in the study area. The study has been conducted under the NSW Floodplain Management Program and has received NSW Government financial support.

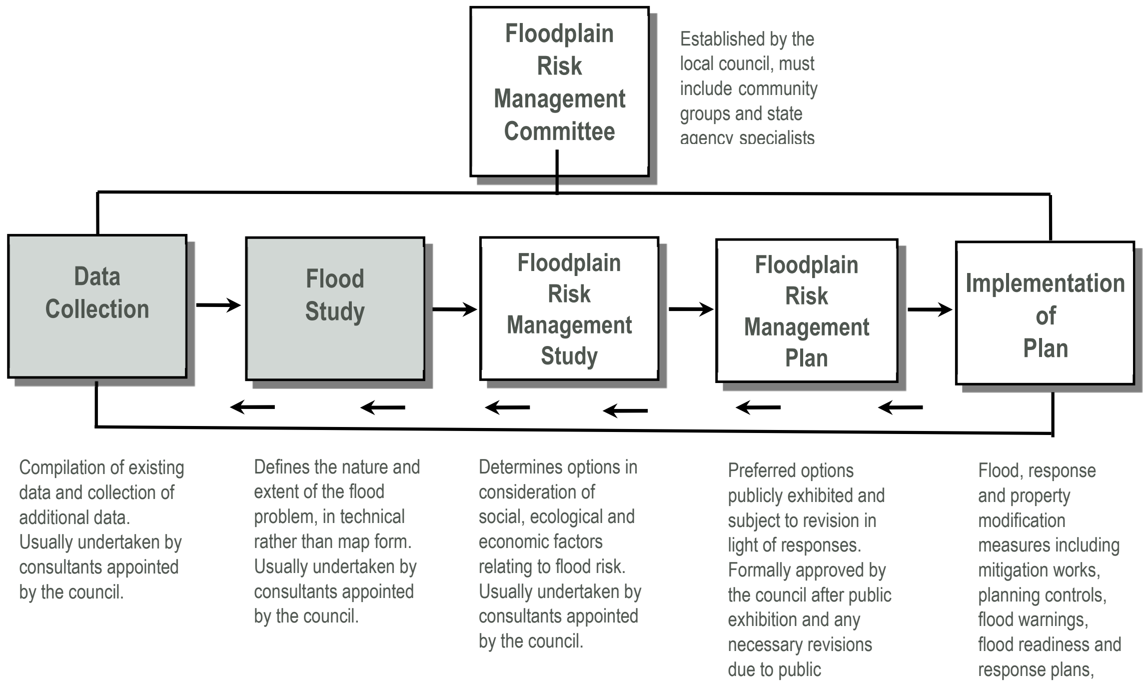


Figure 1.1 Stages of the Floodplain Management Process
(Source: ‘Floodplain Development Manual’ (2005))

1.3 Purpose of Study

The objective of the study is to improve understanding of flood behaviour and impacts and to better inform management of flood risk in the study area. This is achieved through modernisation of Council’s existing flood models. The updated modelling will resolve issues of inconsistency across previous models which were developed at different points in time and for different parts of the catchment.

The key deliverables from this study are:

- A calibrated hydrologic model covering the entire Richmond River catchment.
- A calibrated hydraulic model of the Richmond River and its key tributaries from upstream of Casino to the ocean at Ballina and Evans Head.
- An updated set of maps showing areas at risk from inundation in the 5%, 2%, 1% and 0.2% AEPs, with and without allowances for future climate change, as well as the PMF (Probable Maximum Flood).

1.4 Previous Studies

Since the devastating floods in 1954, the Richmond Valley has been the subject of numerous flood investigations and floodplain management studies. Key studies of most relevance to this current study are briefly summarised below.

Richmond River 2022 Post-event Analysis (BMT, 2023)

Following the record flooding across northern NSW in February/March 2022, the NSW Department of Planning and Environment (DPE), engaged BMT to undertake a post event flood behaviour analysis for the Richmond River catchment. The study utilised the draft flood models being prepared for Richmond Valley Council for the updated flood study and involved an extension of the hydraulic model upstream to Kyogle. The February/March 2022 event was simulated in the model and the results were analysed to understand the magnitude of the event along with an estimate of the flood damage caused. A number of useful datasets were captured for the post-event analysis report including a dataset of surveyed debris marks and additional bathymetry data downstream of Broadwater. These datasets were made available for the Richmond Valley Flood Study (this study).

Richmond River Flood Mapping Study (BMT WBM, 2010)

The most recent relevant study of the Richmond River within the Richmond Valley Council LGA is the Richmond River Flood Mapping Study (BMT WBM, 2010) which was prepared for the former Richmond River County Council (now Rous County Council) and Richmond Valley Council. This comprehensive study developed a catchment wide hydrologic (WBNM) model and a hydraulic (TUFLOW) model of the floodplain between Casino and Broadwater including the lower reaches of the Wilsons River and Bungawalbin Creek. This study is the pre-cursor to the current study and much of the data collected for this study has informed the current study. For ease of reference herein, the current study refers to the Richmond River Flood Mapping Study as the '2010 Flood Study'.

Casino Flood Study (WBM Oceanics, 1998a)

This study was one of the first studies to use computer modelling to determine design flood extents within the Richmond River catchment. It included development of a hydrologic model for the catchment area upstream from the Eden Creek confluence with the Richmond River and a quasi 2D, Mike 11 hydraulic model through Casino. Design flood flows were estimated using modelling and through flood frequency analyses (at Casino, Wiangaree and Toonumbar Dam) with the final estimated design flows based on a critical assessment of results from both methods.

Casino Floodplain Risk Management Study (WBM Oceanics, 2002)

The original intention of this study was to use the Casino Flood Study model results as the basis for further investigation of flood risk management options. Due to advancements in computer modelling between the Flood Study and the Floodplain Risk Management Study, the opportunity was taken to update the hydraulic modelling to use a fully 2D model developed in TUFLOW. The results from the updated model were used within the study and inform the currently adopted flood planning level across Casino.

Evans River Flood Study (BMT WBM, 2014)

The Evans River Flood Study developed a 2D hydraulic (TUFLOW) model of the Evans River including the Tuckombil Canal and Rocky Mouth Creek. The model was used to assess local events resulting from localised rainfall within the Evans River catchment and regional events where significant Richmond River flows entered into the Evans River via Rocky Mouth Creek and the Tuckombil Canal. The study provided high resolution flood mapping of the Evans River floodplain which was not modelled in 2D as part of the Richmond River Flood Mapping Study and was therefore a missing part of flood mapping within the overall Richmond River catchment.

Lismore Floodplain Risk Management Study (Engeny, 2020)

The Lismore Floodplain Risk Management Study was prepared for Rous County Council and developed hydrologic (URBS) and hydraulic (TUFLOW) models for the Wilsons catchment, one of the major tributaries of the Richmond River. Although largely upstream of the current study area, the Lismore study is relevant as the hydrologic modelling in the current study includes the Wilsons River catchment. Furthermore, the Lismore study has been developed in accordance with the techniques contained within the ARR2019 guideline and so the hydrologic model outcomes (peak flows) on the Wilsons River are expected to be similar to those derived in the present study.

1.5 Design Flood Terminology

Design flood events are hypothetical flood events with a given probability of occurrence. This probability of occurrence is the chance that the flood may occur or be exceeded in any one year and is termed the Annual Exceedance Probability (AEP). A 1% AEP flood is a flood that statistically has a 1% chance of occurring or being exceeded in any given year. This is also sometimes stated as a '1 in 100' chance of occurrence. Prior to ARR2019, design floods were typically referred to by their Average Recurrence Interval (ARI) with this terminology being phased out in ARR2019. Table 1.1 lists the AEPs considered in this study and their equivalent ARIs. In this report the AEP terminology, expressed as a percentage, has been used to describe probability of occurrence.

Table 1.1 Design Flood Terminology

AEP %	AEP 1 in Y	ARI (years)
5	20	19.5
2	50	50
1	100	100
0.2	500	500

1.6 Structure of this Report

The remainder of this report is structured as follows:

Section 2	provides an overview of the study area and historical flood risk.
Section 3	documents the data collection and review process.
Section 4	describes the hydrologic model development.
Section 5	describes the hydraulic model development.
Section 6	presents the hydrologic and hydraulic model calibration and validation.

Section 7	presents flood frequency analyses used to obtain independent estimates of design flood flows.
Section 8	presents the design flood methodology and outcomes.
Section 9	details sensitivity tests that have been undertaken.
Section 10	presents an analysis of results in terms of affected properties and roads.
Section 11	describes the post processing of model outputs to generate additional output beneficial to the floodplain management process.
Section 12	lists the key conclusions from the study.

2 Study Area

2.1 Catchment Overview

The Richmond River catchment is shown in Figure 2.1 and the study area is shown in greater detail in Figure 2.2. It is located within the Northern Rivers region of NSW and is one of NSW's largest coastal rivers with an overall catchment area of approximately 6,900km^2 which drains to the ocean at Ballina. The upper reaches of the catchment drain the steep mountainous Richmond and McPherson Ranges near the NSW/Queensland border where elevations exceed 1,000m. Lower reaches of the catchment are characterised by extensive, low-lying floodplains where watercourses remain tidal for significant distances inland[1]. Notable tributaries of the Richmond River include the Wilsons River and Bungawalbin Creek, both significant catchments in their own right.

The Richmond River catchment is located within Bundjalung Country and includes the city of Lismore and the towns of Casino, Coraki, Kyogle, Woodburn, Broadwater, Ballina, Evans Head and Rappville along with numerous smaller settlements. The majority of the towns are located adjacent to the catchment's rivers and creeks and are heavily dependent on the rivers and its floodplains for agricultural industries such as cattle (dairy and beef), and sugarcane as well as the fishing and tourism industries.

The Richmond River catchment contains a second outlet to the ocean at Evans Head via the Tuckombil Canal and the Evans River. The Tuckombil Canal was constructed for flood mitigation purposes in 1895 following significant floods in the late 1800's and was deepened and widened in 1965 to its present dimensions. It is approximately 1.5km long connecting Rocky Mouth Creek with the Evans River with a concrete weir separating Rocky Mouth Creek from the more saline waters of the Tuckombil Canal and Evans River.

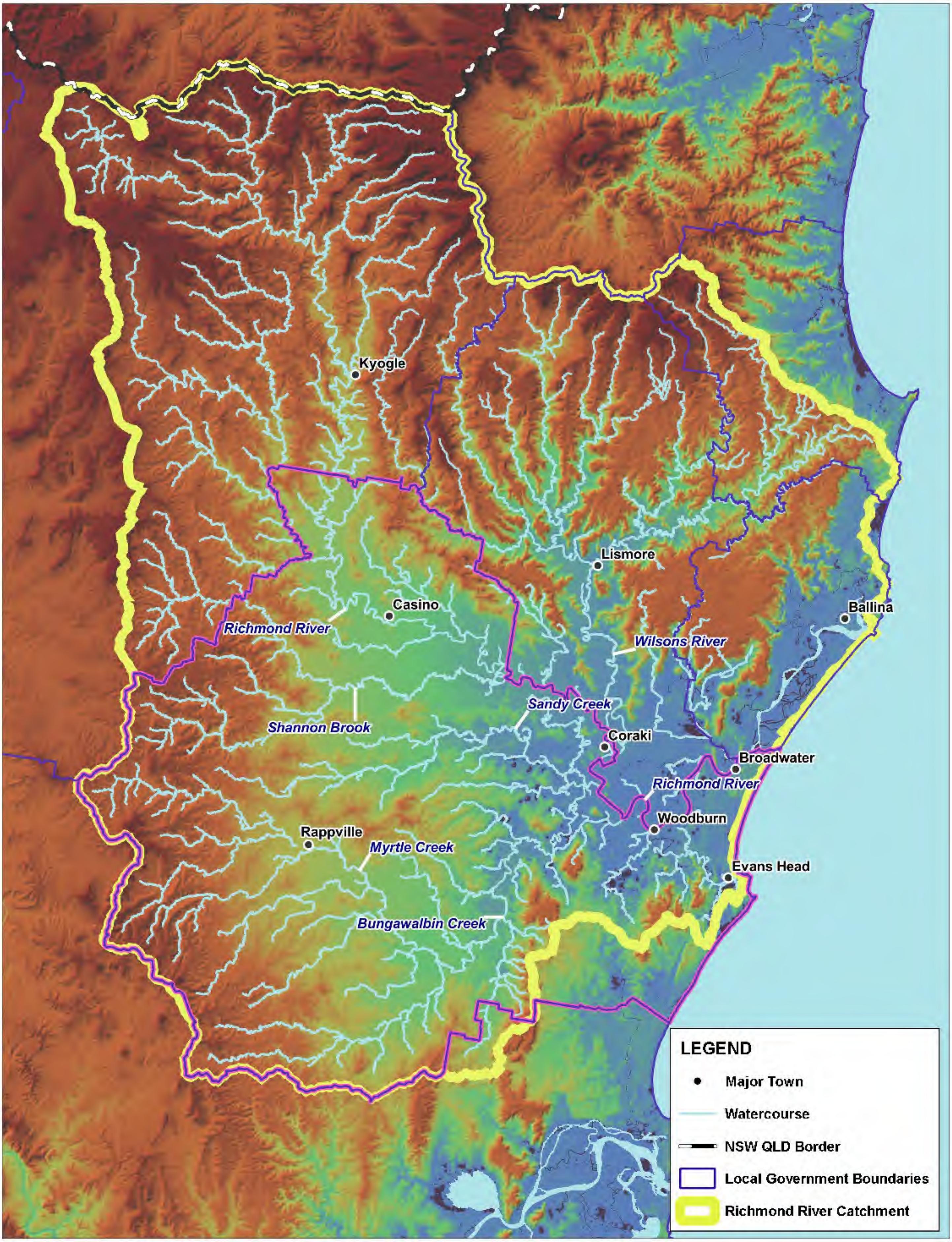
A natural constriction in the floodplain at Broadwater acts to hold floodwaters within the extensive low lying floodplain between Broadwater, Woodburn and Coraki. This floodplain includes the Tuckean Swamp, a coastal floodplain containing a series of artificial drains to facilitate agriculture. Part of the Tuckean Swamp is a protected nature reserve which is a remnant of the original swamp prior to drainage and land clearance. The Tuckean Swamp and Tuckean Nature Reserve are connected to the Richmond River via the Tuckean Broadwater, a tidal tributary of the Richmond River. The Bagotville Barrage was constructed in 1971 and provides a physical barrier between the upstream Tuckean floodplain and the tidal inflows from the Tuckean Broadwater. It contains a series of one-way flood gates to allow drainage from the Tuckean Swamp.

In addition to the Tuckombil Canal and the Baggotville Barrage, there are several other notable hydraulic structures that have an influence on catchment flows. These include levees at Lismore and Tuckurimba, floodgates on Rocky Mouth Creek and dams at Toonumbar, Rocky Creek and Emigrant Creek. The Tuckurimba Levee runs along the eastern bank of the Wilsons River between Baxters Lane and Coraki. It provides protection for properties to the east of the levee during frequent to moderate flood events.

Roads and railways which cross the Richmond River's floodplains are typically raised to provide a degree of flood immunity. This has resulted in embankments which can contain and redirect floodplain flows.

[1] The tidal limit of the Richmond River is just downstream of Casino, a distance of 110km from the ocean.

A significant recent development within the lower Richmond River catchment has been the Pacific Highway Upgrade. This has included the construction of new bridges across the Tuckombil Canal and the Richmond River near Broadwater and was opened to traffic in 2020. The highway also crosses significant lengths of the Richmond River floodplain and includes numerous culverts and smaller bridges to preserve cross drainage flow.



Title:
Richmond River Catchment

Figure: 2-1

Rev: A

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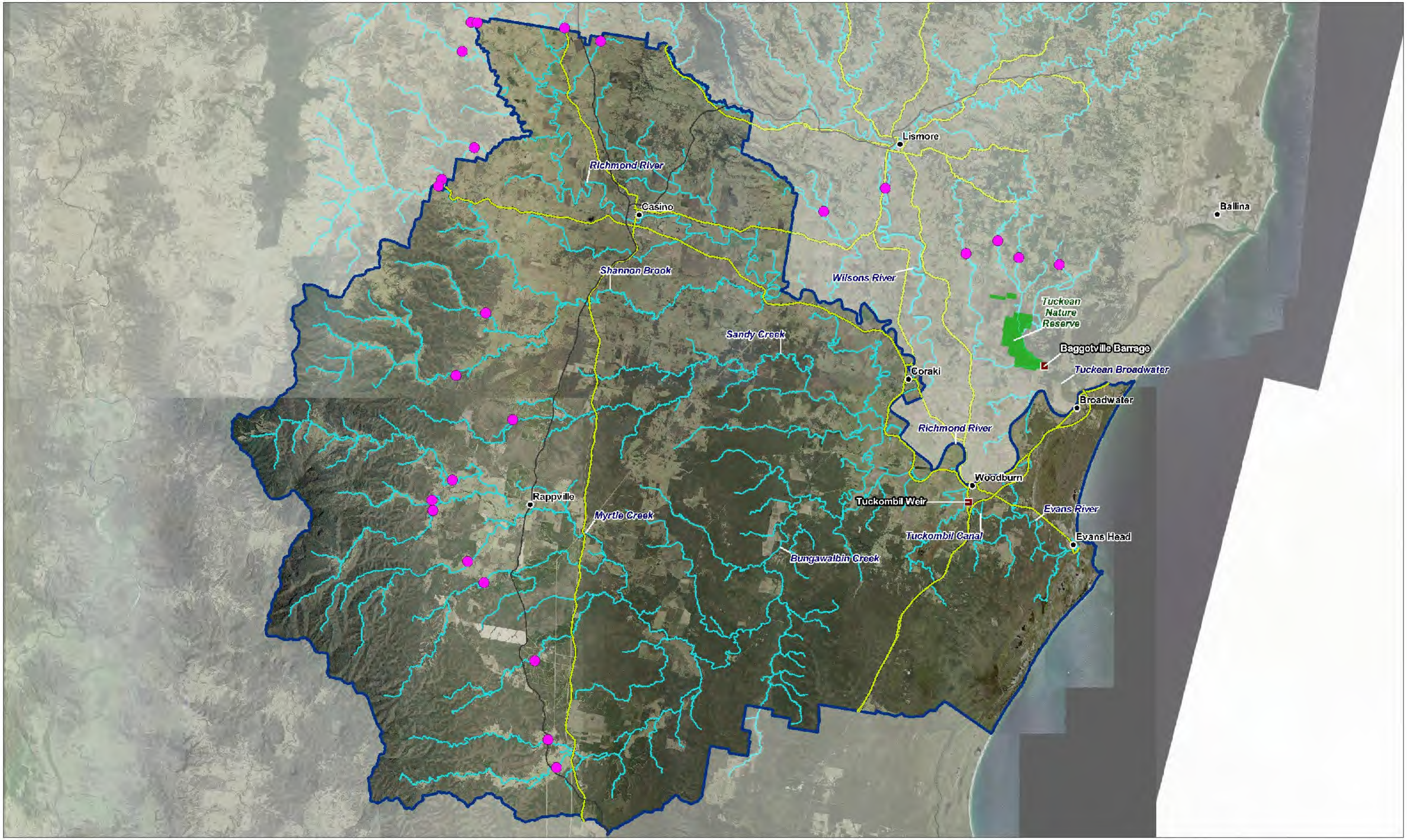

LEGEND

- Major Town
- Weir/Barrage
- Upstream Limit of Mapping
- Main Watercourse
- Railway
- Major Road
- Tuckean Nature Reserve
- Richmond Valley Council Boundary

Title:

Study Area

Figure: 2-2

Rev: A

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2.2 Flood History

With its large catchment area, multiple watercourses and high annual average catchment rainfall, the Richmond River and its tributaries inundate its floodplains on a fairly frequent basis. Large flood events can be very destructive and cause significant damage to townships located within the catchment. For many parts of the catchment, the largest flood on record occurred in February/March 2022. Other notable significant floods occurred in February 1954, March 1974, January 2008 and May 2009, with the 1954 flood causing major damage in the Casino area including the washing away of part of the old Irving Bridge (Figure 2.3).

The earliest recorded flood event occurred in 1857 and anecdotal data also points to a number of very large floods which occurred later in the 1800's but for which little information is known. A flood which occurred in January 1887 was perhaps one of the largest of these floods on the Richmond River and reportedly broke over the banks into Casino at several places causing severe damage in Casino and throughout the catchment. This was likely of a similar magnitude to the February 1954 and February/March 2022 floods in Casino[2].

Where available, peak flood levels of different events at key locations with the study area are summarised in Table 2.1.

Table 2.1 Historic flood levels (m AHD)

Flood Event	Casino	Coraki	Woodburn
February 1954	24.35*	6.1	4.4
March 1974	17.3	6.2	4.2
April 1988	17.4	5.8	3.9
January 2008	21.2	5.8#	3.1
May 2009	18.9	5.8#	3.2
April 2017	18.0	6.0	3.2
February/March 2022	22.8^	6.8	6.4

***Level taken at Casino Bridge. Other events are quoted at Casino Gauge located further downstream**

#Level from simulated results (failed gauges)

^Manually surveyed level at the gauge

[2] Based on a comparison of anecdotal accounts of the 1887 flooding with photos from the 1954 event.

Source: Richmond Valley Council

Figure 2.3 The February 1954 Flood in Casino (section of Irving Bridge shown washed away)

3 Data Collection and Review

3.1 Introduction

Many of the datasets used in the current study were previously collated as part of the 2010 Flood Study (BMT WBM, 2010) and they are still relevant to the current study. These include surveyed flood levels of the January 2008 and May 2009 flood events, river bathymetry and details of hydraulic structures.

During preparation of the current flood study the February/March 2022 event occurred. Council and DPE undertook an extensive data collection exercise to capture the event peak flood levels and bathymetric survey at Broadwater, Wardell and Ballina. These datasets have been used in this study.

Additional datasets sourced for the current study include more recent and extensive topography (LiDAR) data and rainfall and river level data for the flood events of February/March 2022 and March/April 2017.

The data collection process also included obtaining new data via the following means:

- BMT undertook site visits to key locations within the catchment and have compiled a georeferenced database of photos (typically at structures) along with site visit observations.
- Notification on Council's website regarding the project and an invitation for the public to supply any relevant information.

The remainder of this section summarises the key datasets used in this study. Use of the datasets within the study is documented within subsequent sections on model development and calibration.

3.2 Aerial Photography

Aerial imagery of the study area was supplied by Council for the whole LGA and was captured in 2009. More recent and higher resolution aerial images (captured in 2018) were supplied by Council for the townships of Casino, Woodburn, Coraki, Broadwater and Evans Head. This imagery has been used as a background for the flood mapping presented herein.

The aerial imagery was also used in combination with freely available online aerial datasets to inform the land use mapping when developing the hydraulic model.

3.3 Topographical Survey

3.3.1 Aerial Survey

Aerial topographic survey was available in the form of multiple LiDAR datasets captured across large swathes of the catchment at different points in time. LiDAR stands for 'Light detection and ranging' and the datasets available for this study were captured from a sensor mounted to a light aircraft. Table 3.1 lists the LiDAR datasets available across the study area. When combining the datasets to construct a digital elevation model (DEM) across the study area (see Section 5.3), preference is given to the more recent dataset when datasets overlap. The datasets in Table 3.1 are listed in order of most study area coverage. Figure 5.2 in Section 5.3 shows the locations of the datasets as applied within the hydraulic model.

Table 3.1 LiDAR Datasets

Dataset Title/Region	Capture Date	Resolution	% of study area
Lismore	June 2010	1m	32%
Woodburn	June 2010	1m	24%
Ballina	June 2010	1m	9%
Woodburn	July 2017	1m	9%
Bonalbo	September 2017	2m	5%
Bonalbo	November 2016	1m	4%
Coaldale	July 2017	2m	4%
Woodburn	March 2010	1m	4%
Lismore	November 2016	1m	3%
Coaldale	July 2017	1m	3%
Bonalbo	July 2017	1m	1%
Coaldale	November 2016	1m	<1%
Woodburn	November 2016	1m	<1%
Lismore	July 2017	1m	<1%

3.3.2 Ground Survey

Ground survey captured as part of previous studies was collated and reviewed. These surveys typically comprised spot heights along hydraulic controls such as road embankments and levees. The main source of ground survey was from a dataset termed 'Michel Survey' captured in 1998.

Where available, the 'Michel Survey' elevations have been cross checked against more recent LiDAR data. A number of inconsistencies were found, including survey points not always being positioned at the highest points of the levees. Furthermore, the density of surveyed point locations (generally 100m apart) was not sufficient to define the feature along its length for use in a 2D model. In such cases, the surveyed datasets were replaced with elevations extracted from the LiDAR datasets. An example showing the elevations and density of Michel Survey points against elevations from LiDAR is provided in Figure 3.1.

The 2010 Flood Study hydraulic model also contained a number of other breaklines along embankment features which were sampled from a DEM prepared by Fugro from photogrametric survey in 2007. Given that more accurate LiDAR data is now available, these breaklines were not applied in the updated model with new breaklines instead created from LiDAR data.

Ground survey has also been obtained for the Pacific Highway Upgrade. The details were contained within hydraulic models developed to inform the design of the Pacific Highway Upgrade and were supplied to BMT[3]. The highway has been represented in the model in the form of DEMs, TINs and breaklines. The new highway structures (bridges and culverts) have also been included in the model (see Section 3.7).

[3] Supplied to BMT by Pacific Complete (N. Geale, Personal Communication, March 2021).

Additional ground elevations in the form of proposed landform designs have been supplied by Council for the upgrade to the Woodburn-Coraki Road and for a large development at Broadwater (George Street).

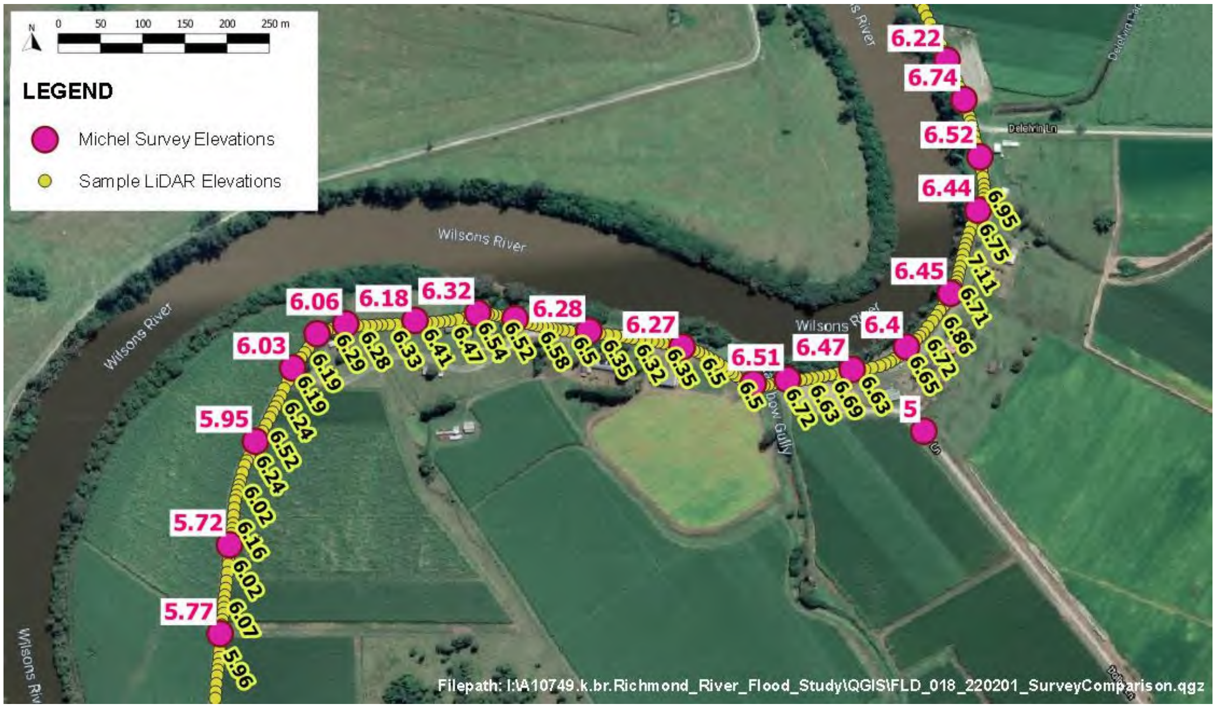


Figure 3.1 Sample showing locations and elevations from Michel Survey (highlighted) versus elevations from LiDAR.

3.4 Bathymetric Survey

Bathymetric survey, also referred to as hydrographic survey, was captured for the tidal extents of the Richmond River catchment in 2004 by the NSW Department of Natural Resources (now the Department of Planning, Industry and Environment). This was used in the 2010 Flood Study in the form of cross sections at discrete intervals along the watercourses. For this study, the dataset has been converted to a continuous bathymetric elevation model suitable for use in a 2D model.

Following anecdotal accounts of scour occurring at the Richmond River mouth during the 2022 event, additional survey was collected by DPE for discrete areas at Broadwater, Wardell and Ballina.

Bathymetry for the weir pool upstream of Jabour Weir in Casino was also provided by Council. This covers a 25km length of the Richmond River. The survey was captured in 2010.

Further details including the extent of bathymetric data used in the model are provided in Section 5.3.

3.5 Hydrometric Data

Hydrometric data includes historic rainfall and river level data at gauge locations and is used in the process of model calibration/verification. Four historic flood events have been modelled, these being January 2008, May 2009, March/April 2017 and February/March 2022. Comprehensive datasets of rainfall and river levels for the January 2008 and May 2009 events were collated as part of the 2010 Flood Study. Data for the March/April 2017 and February/March 2022 events have been collated from a number of different sources as follows:

- Manly Hydraulics Laboratory (MHL) river/tide levels at gauges within the tidal reaches of the Richmond River
- WaterNSW river levels at gauges
- Bureau of Meteorology (BOM) recorded river and rainfall data at gauges pre-processed into hourly intervals.

3.6 Flood Level Survey

Following the February/March 2022 flood event, various agencies including local Councils and DPE surveyed peak flood level marks (typically debris marks) throughout the Richmond River catchment. The surveys included flood level indicators at a number of gauges which failed, including Casino, Bungawalbin Creek (Neileys Lagoon Road), East Gundurimba and Evans Head. In total 437 peak flood levels were identified and surveyed within the extent of the hydraulic model with 131 of these within the Richmond Valley LGA and the remaining majority on the Lower Richmond River downstream of Broadwater.

The flood events of January 2008 and May 2009 occurred during the development of the 2010 Flood Study. As part of that study, two field data collection exercises were undertaken. This included the identification and survey of peak flood levels with 78 levels captured for the January 2008 event and 46 levels captured for the May 2009 event. The surveyed flood peak levels include the flood elevation, the location, additional relevant commentary and an assessment of the degree of confidence of the surveyed level.

3.7 Structures

Details of all major hydraulic structures (bridges, culverts and weirs) were primarily sourced from the 2010 Flood Study. These in turn were based upon work as executed (WAE) drawings.

A number of significant structures have been constructed since 2010. Most of these are associated with the Pacific Highway Upgrade. Details of the highway structures (bridges and culverts) were contained in the hydraulic models developed for the Pacific Highway upgrade and which were supplied to BMT.

3.8 Anecdotal Flood Data

A significant amount of anecdotal flood data in the form of photographs and videos was captured for the February/March 2022 event with much of this data available for viewing on the internet. Whilst the river level gauges and surveyed flood levels are primarily relied on for model calibration, anecdotal data serves a useful purpose for verifying flood extents or flow behaviour, noting that this may not be at the peak of the flood.

Data in the form of photographs and videos (including drone footage) showing a flood event in the Bungawalbin Creek catchment in March 2021 was supplied to BMT from the following sources:

- A local resident within the lower Bungawalbin Creek catchment who provided photos, and videos (including drone footage) of a levee breach on Bungawalbin Creek.
- Richmond Valley Council who provided video captured by drone of the flooding in the vicinity of Rappville.

Whilst the 2021 flood was not a calibration event for this study, the supplied data was studied and used as a check against the modelled flood behaviour in these respective parts of the Bungawalbin Creek catchment. Samples of anecdotal photographs are included in Figure 3.2 and Figure 3.3.

Figure 3.2 Breach and subsequent erosion of levee on lower Bungawalbin Creek (March 2021)

Figure 3.3 Aerial Image of the lower Richmond River Floodplain during the February/March 2022 Flood

4 Hydrologic Model Development

4.1 Background

A hydrologic model used in flood studies estimates the rate of runoff for a given storm event. Rainfall is applied as an input to the model and the outputs are flow hydrographs at points of interest across the catchment. These flow hydrographs can then be applied to a hydraulic model to estimate flood levels and flood extents.

Numerous hydrologic models have been previously developed within the Richmond River catchment. The two most relevant ones to this assessment are summarised below.

4.1.1 Richmond River Flood Mapping Study (BMT WBM, 2010)

The 2010 Flood Study developed a 'whole of catchment' hydrologic model and used the Watershed Bounded Network Model (WBNM). It consisted of 431 sub-areas split into five component 'sub-models' as follows:

- Upper Richmond (upstream of Casino)
- Mid Richmond (Casino to Coraki)
- Wilsons River (upstream of Coraki)
- Bungawalbin Creek
- Lower Richmond (downstream of Coraki).

An analysis on flood wave lag times supported use of different hydrologic lag parameters for the sub-models. This was due to the effects of significant floodplain storage on catchments such as the Wilsons River and Bungawalbin Creek. The model was calibrated to historical events and used to provide inputs to a hydraulic TUFLOW model.

4.1.2 Lismore Floodplain Risk Management Study (Engeny, 2020)

The Lismore Floodplain Risk Management Study used a hydrologic URBS model of the Wilsons River catchment. The model consisted of 124 sub-areas and covered all catchments upstream of Lismore, extending to approximately 10km upstream of Coraki.

4.2 Hydrologic Modelling Approach

The current study requires a whole of catchment hydrologic model which extends as far downstream as Broadwater. URBS was selected as the modelling software due to its functionality when implementing ARR2019 and noting that it will likely have ongoing benefits for flood forecasting purposes. Use of URBS also provides consistency with modelling developed for the Lismore Floodplain Risk Management Study.

4.2.1 URBS Software

URBS is a runoff-routing networked model of sub-areas based on centroidal inflows. The primary focus of its development has been flood forecasting and design flood hydrology (Carroll, 2016) and it is commonly used in flood studies and for operational flood forecasting models across Australia.

The URBS 'SPLIT' approach has been used whereby the model routes catchment and channel storages separately, with each component governed by separate equations. Rainfall on a sub-area is routed through that sub-area to the main watercourse (assumed to be at the sub-area centroid). The flow is then routed along a reach using a non-linear Muskingum method. The main routing variables used are catchment area (catchment storage) and reach length (channel storage). Two main routing parameters used are a catchment storage (or lag) parameter (β or beta) and a channel storage (or lag) parameter (α or alpha).

The alpha and beta parameters are used as calibration parameters in the runoff-routing component of the model. 'Alpha' is a measure of channel travel time with lower values of alpha representing reduced (quicker) travel times. 'Beta' is a measure of sub-catchment storage with high values of beta increasing storage and therefore increasing runoff attenuation within the catchment.

URBS also allows for the inclusion of baseflow and dam/reservoir storages.

4.2.2 Model Design

The hydrologic modelling approach is similar to that employed in the 2010 Flood Study except the sub-models are incorporated into a single overall URBS model rather than stand-alone component models. This has allowed for efficient implementation of ARR2019 design hydrology.

The schematisation of the URBS model for the Wilsons catchment as used in the Lismore Floodplain Risk Management Study, has been incorporated into the whole of catchment URBS model developed for this study.

The hydrologic model is divided into four catchment regions. These are shown in Figure 4.1 along with the sub-areas. The model regions are as follows:

- Upper Richmond (to Casino);
- Mid-Richmond (Casino to Broadwater, including Rocky Mouth Creek and Evans River);
- Wilsons; and
- Bungawalbin.

The 'Lower Richmond' sub-model from the 2010 Flood Study, has been apportioned into the Mid-Richmond and Bungawalbin regions with the remaining downstream area, downstream of the Richmond Valley LGA, not included in the model.

The sub-areas for model regions representing Upper Richmond, Mid-Richmond and Bungawalbin are based on the original sub-areas from the 2010 Flood Study with minor modifications made to ensure topographic consistency with the most recent topographic LiDAR data (see Figure 5.2 for LiDAR coverage). This resulted in the sub-division of some sub-areas to better suit the terrain. The sub-area linkages (stream lengths) have been recalculated using LiDAR data. Sub-areas and sub-area linkages for the Wilsons region have been adopted from the URBS model developed for the Lismore Floodplain Risk Management Study in order to maintain consistency.

In total the number of sub-areas is 566, which compares to 431 originally used for the 2010 Flood Study.

The hydrologic model includes Toonumbar Dam within the Upper Richmond region. The stage-storage-discharge relationship has been extracted from the original WBNM model and is shown in Table 4.1

Table 4.1 Toonumbar Dam Stage-Storage-Discharge Relationship

Stage (mAHD)	Storage (ML)	Discharge (m^3/s)
129.62 (Full Supply Level)	11,000	0
131.46	13,613	205
134.50	18,692	890
137.55	24,938	1,843
140.60	32,798	3,001
143.65	42,443	4,334
146.70	53,871	5,821

The parameterisation of the hydrologic URBS model was undertaken during the process of model calibration (see Section 6).

4.3 Rating Curve Updates

Rating curves relate water levels to estimates of flow. They are useful to understand the flow magnitudes of historical flood events for use in hydrologic model calibration and flood frequency analyses.

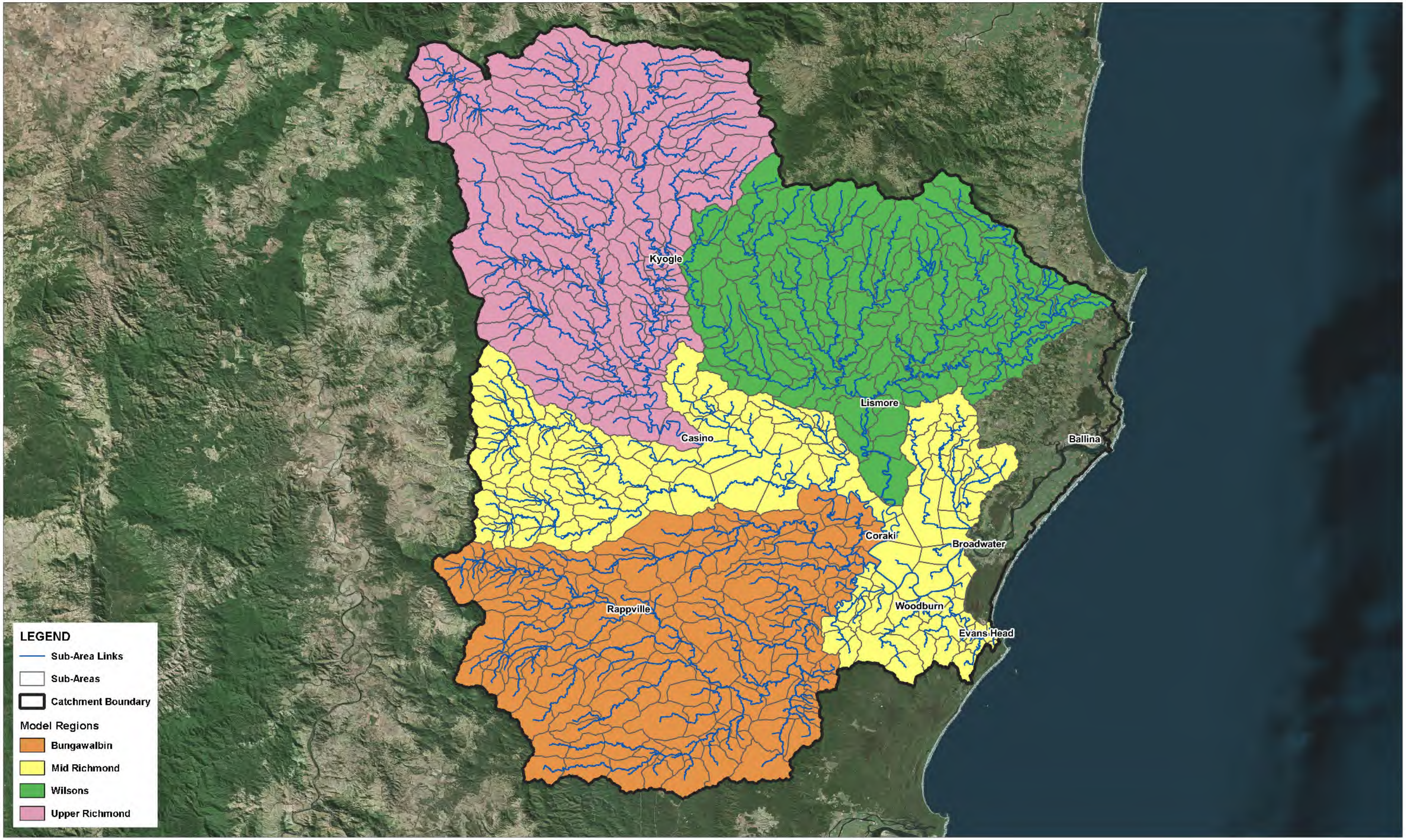
Considerable work has previously been undertaken by BMT in reviewing rating curves across the Richmond River catchment. In particular, this was recently undertaken for the Wilsons Catchment (BMT WBM, 2018) and for the gauge at Kyogle (BMT WBM, 2004).

As part of the current study, BMT has undertaken further investigations for rating curves at key locations within the model. Gauges at Wiangaree, Doubtful and Casino within the upper Richmond catchment, East Gundurimba on the Wilsons River and Rappville within the Bungawalbin Creek catchment were reviewed. The locations of these gauges are useful for checking the estimated flows passing into the study area.

The investigations involved developing localised hydraulic models and simulating a series of steady state flows for a wide range of flow magnitudes. For gauges at Wiangaree, Doubtful and Casino, the modelling confirmed that the existing rating curves[4] showed good general agreement with model results and so the existing rating curves were retained for use in determining rated flows for the modelled historical events. The existing rating curve for East Gundurimba[5] showed a notable mismatch to model results. The modelled rating was therefore adopted (see Section 7.7).

A reliable rating curve could not be developed for the gauge at Rappville within the Bungawalbin Creek catchment. The floodplain in the vicinity of the gauge is complex and includes a network of breakout channels. Hydrologic modelling in the Bungawalbin Creek catchment has therefore relied upon the sequential output from the hydraulic model to inform calibration.

[4] Existing rating curves were published by Water NSW for Wiangaree, Doubtful and Casino.
[5] The source of the existing rating at East Gundurimba is unknown.



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URBS Schematisation

Figure: 4-1

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5 Hydraulic Model Development

5.1 General Modelling Approach

The software used for the hydraulic modelling is TUFLOW (build 2020-10-AD), which is an industry leading hydraulic modelling software used extensively across Australia and internationally.

A 2D hydraulic model has been developed and takes advantage of TUFLOW's Quadtree feature which allows for the model grid resolution to be varied across the model domain. This enables key areas, such as areas of interest or hydraulically complex areas, to be modelled at a finer grid resolution whilst retaining a coarser resolution in areas that do not benefit from a fine resolution.

TUFLOW's Heavily Parallelised Computation (HPC) solver has been used which enables 2D models to be simulated on computers' Graphical Processing Units (GPU) rather than the traditional approach of using the Central Processing Units (CPU). This allows for large catchments, such as the Richmond, to be developed as a single model whilst retaining practical simulation times.

TUFLOW's Sub-Grid-Sampling (SGS) feature is also employed which allows the model to make maximum use of the underlying terrain data.

The extent of the model has been designed to cover the main catchment watercourses of the Richmond River and its key tributaries within the Richmond Valley Council LGA. The hydraulic model extent is shown in Figure 5.1 and includes the following key watercourses:

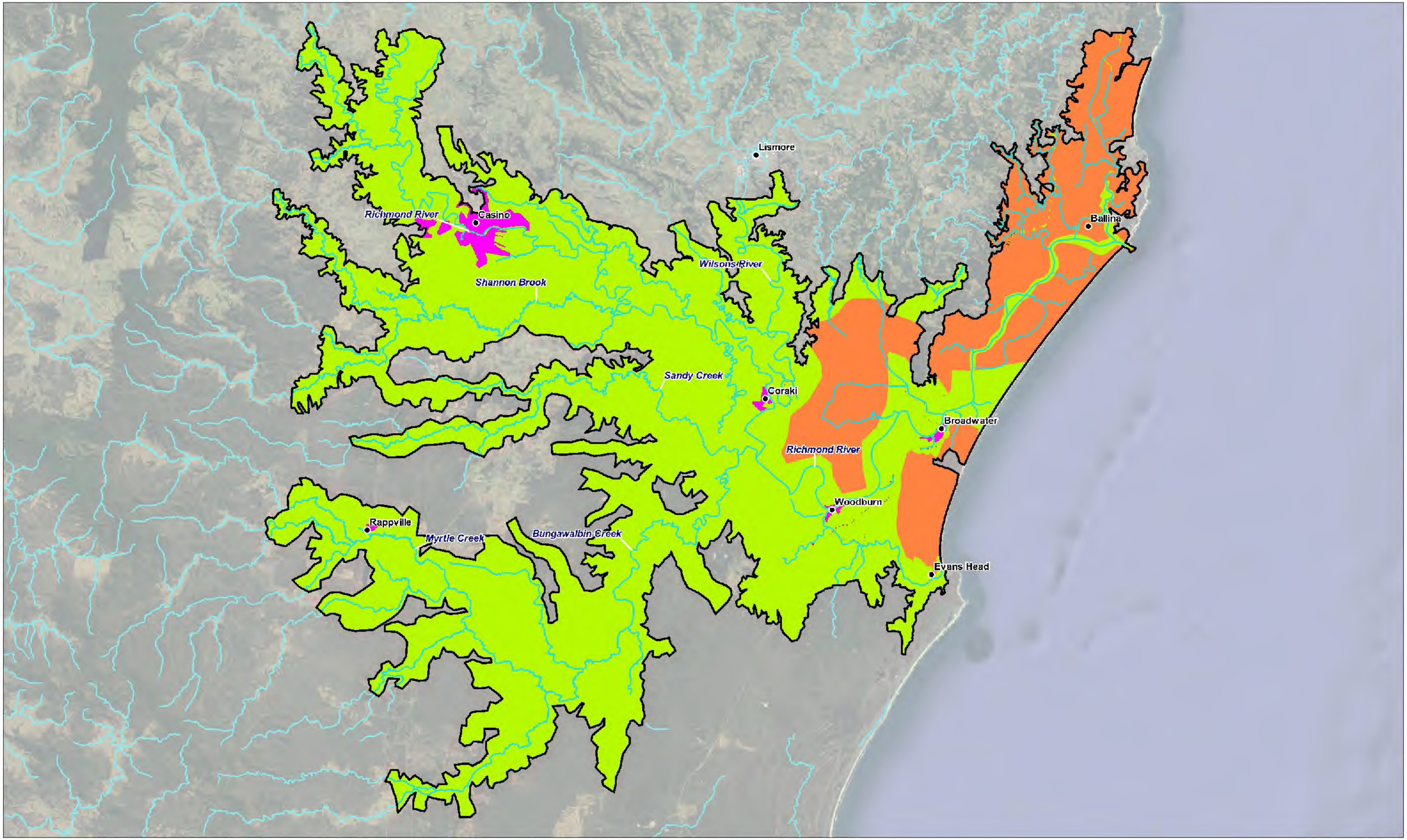
- Richmond River from Fairy Hill (upstream of Casino) to the ocean at Ballina;
- Shannon Brook at Piora to its confluence with the Richmond River at Tatham;
- Bungawalbin/Myrtle Creek from Wyan and Whiporie to the Richmond River;
- Wilsons River from Lismore Airport to the Richmond River;
- Rocky Mouth Creek from New Italy to the Richmond River; and
- Tuckombil Canal and Evans River from Rocky Mouth Creek to the ocean at Evans Head.

The extent of the hydraulic model is notably larger than that modelled for the 2010 Flood Study, due to a combination of available terrain data and advancements in the modelling software making such an extent feasible.

5.2 Model Resolution

TUFLOW's Quadtree feature has been used in order to vary the cell size across the model domain. A significant component of model development has been in optimising the cell sizes within the model whilst ensuring that practical simulation times are maintained. This process has prioritised allocating higher resolution to urban areas and for areas that benefit hydraulically from a high resolution. The adopted cell size configuration across the model extent is shown in Figure 5.1 and is summarised as follows:

- 40m for large floodplains that effectively become large floodwater storage areas during flood events. Also used for floodplain downstream of Broadwater which lies outside of Council's LGA and for areas within the model domain but which are at high relative elevation and so will not be inundated.
- 20m resolution across majority of modelled floodplain and for large waterways.
- 10m for some narrower creeks that are key hydraulic considerations (such as Tuckombil Canal).
- 5m for urban areas including Casino, Coraki, Woodburn, Broadwater, Rappville and Evans Head.



LEGEND

Model Extent

Main Watercourses

Aerial image: Google Satellite

Nest Levels

1 (40m Resolution)

2 (20m Resolution)

3 (10m Resolution)

4 (5m Resolution)

Title:

Hydraulic Model Configuration - Extent and Resolution

Figure: 5-1

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5.3 Topography

The base topography used in the 2D model is sampled from LiDAR data. LiDAR datasets were sourced from the Geoscience Australia website and consist of various datasets captured at different points in time. The datasets used are shown in Figure 5.2. These datasets have been compiled into a Digital Elevation Model (DEM) covering the whole of the Richmond catchment. Where datasets join, checks were performed to ensure there were no step changes which could result in artificial restrictions to flow. The DEM was then applied within the hydraulic model.

The model topography has been supplemented with bathymetry data. This bathymetry data is predominantly based on data captured by DIPNR in 2004 for the majority of the tidally affected parts of the catchment. In the 2010 Flood Study, the river channels were typically modelled in 1D using cross sections. For the current study, the river channels are modelled in 2D and so the bathymetry has been converted to a gridded dataset.

Additional bathymetry was supplied by:

- Council for an approximate 25km length of the Richmond River upstream of Jabour Weir in Casino. This data was captured in 2010 for a project considering potential weir raising options. This dataset has been incorporated into the model.

- DPE for discrete lengths of the Richmond River at Broadwater, Wardell and Ballina, to inform the magnitude of scour from the 2022 event.

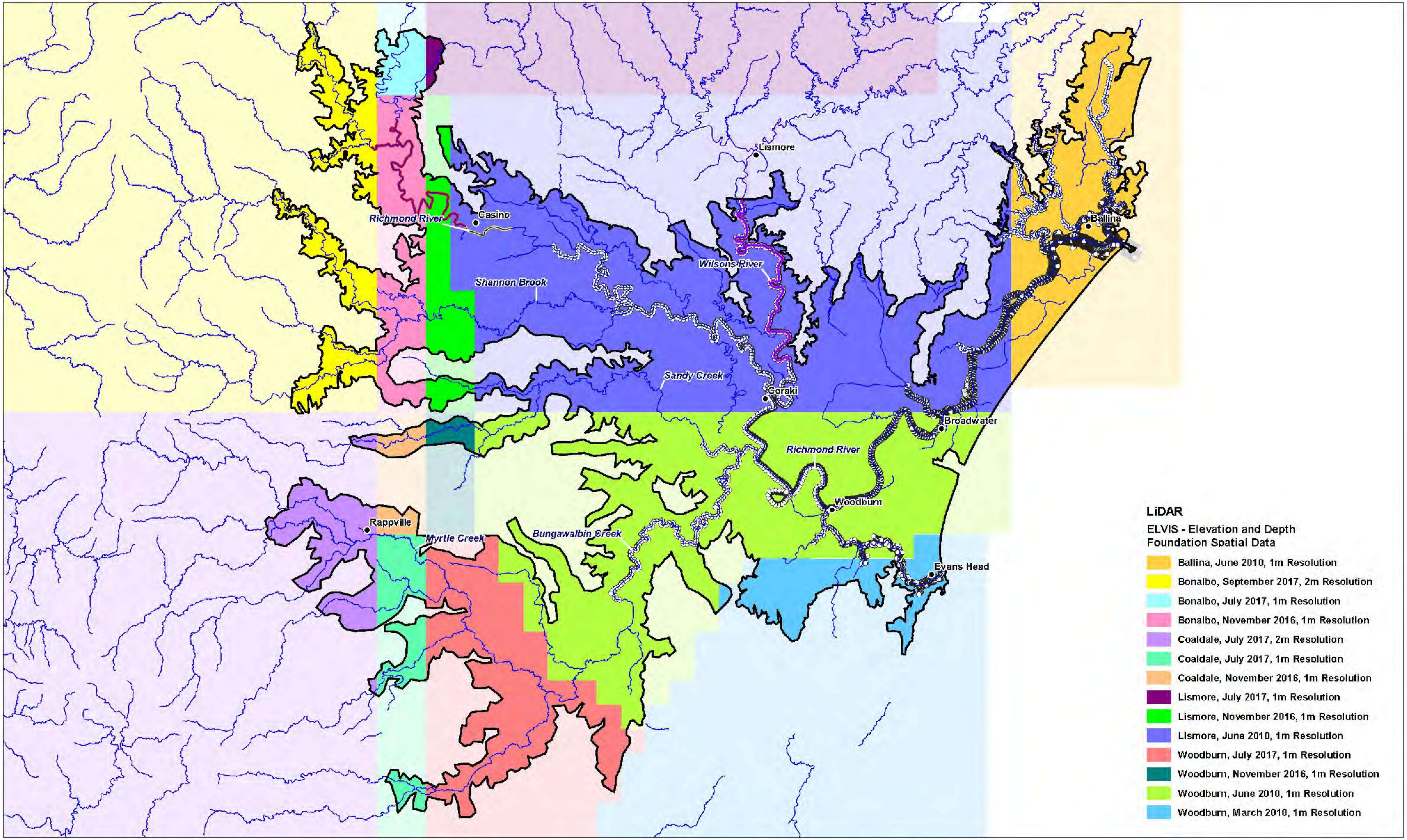
The extents of the available bathymetry datasets are shown in Figure 5.2.

Key levees, embankments and other ridge features have been incorporated into the model as breaklines. This includes survey data of levees used in the original model along with other embankment crest heights digitised from LiDAR data.

The Woolgoolga to Ballina Pacific Highway Upgrade is a significant infrastructure development which traverses sections of the Lower Richmond floodplain. The upgrade was completed in 2020 and includes new bridges over the Richmond River near Broadwater and over the Tuckombil Canal. As constructed designs of the highway upgrade have been obtained and incorporated into the model, although these are not included in the model calibration events which pre-date the highway upgrade.

5.4 Surface Roughness

Land use layers, used to assign Manning's n roughness values to the hydraulic model, have been informed from land use mapping applied in the 2010 Flood Study. These layers were reviewed and updated to cover the larger model extent of the current study. Initial values of Manning's n were based on a combination of industry standard values and those determined through previous model calibration in the 2010 Flood Study. These initial Manning's n values were then refined through the model calibration process (see Section 6). Figure 5.3 shows the land use types and final Manning's n values after model calibration.



LEGEND

- Model Extent
- 2004 DIPNR Surveyed Cross-Sections
- Bathymetry - Richmond River, 15 March 2010, 1m Resolution
- Bathymetry - Richmond River, May 2005, 5m Resolution
- Bathymetry - Sourced from Engeny Model, Survey DS4, 27 November 2018, 1m Resolution
- Main Watercourses

Title:

Hydraulic Model Configuration - Topography and Bathymetry Sources

Figure: **5-2**

Rev: **A**

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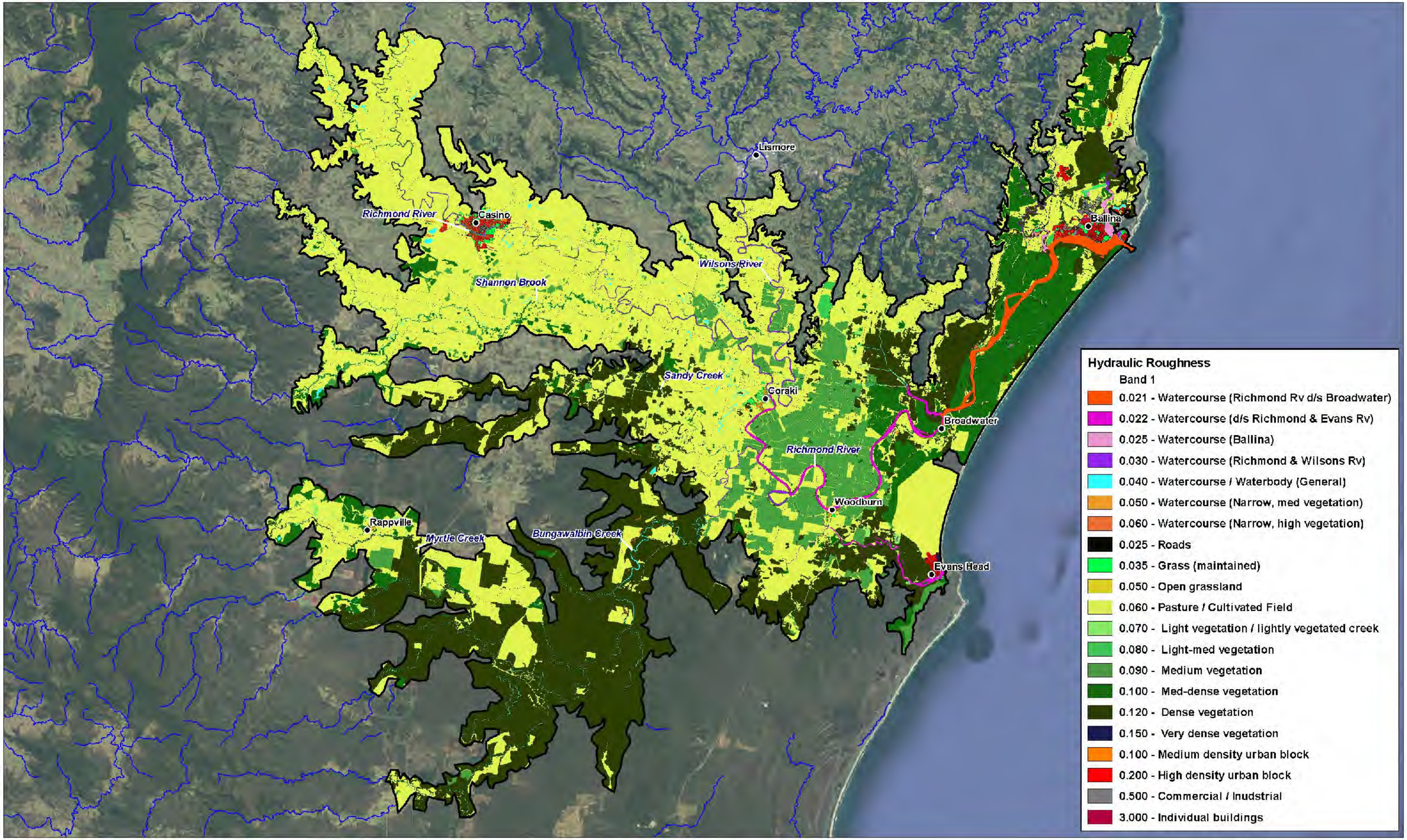

LEGEND

Model Extent

Main Watercourses

Aerial image: Google Satellite

Title:

Hydraulic Model Configuration
Hydraulic Roughness - Manning's n

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Figure: 5-3

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5.5 Boundary Conditions

Inflows to the TUFLOW model are flow hydrographs taken from the URBS hydrologic model. They are applied using a combination of TUFLOW’s flow-time (type QT) boundary and a source area (type SA) approach as follows:

- Cumulative catchment flows (total inflows) for catchments draining to the upstream extents of the hydraulic model were applied as QT boundaries at 47 locations; and
- Runoff from hydrologic sub-areas located within, or partially within, the hydraulic model extent were applied in a distributed manner across the model domain as ‘local inflows’. Local inflows were applied at 187 locations across the hydraulic model extent.

Downstream boundaries are included in the model representing the two outlets of the Richmond River catchment to the ocean. These are head versus time (type HT) boundaries located at Ballina and Evans Head. Four additional HT boundaries are also included to allow flow to pass into the ocean over sand dunes during very large (rare) design floods.

Figure 5.4 shows the locations of the model inflows and downstream boundaries.

5.6 Structures

Bridges along the major waterways have been included in the model using TUFLOWs layered flow constriction feature. This allows for separate layers to be specified for the sub-structure, superstructure and any railings or safety barriers.

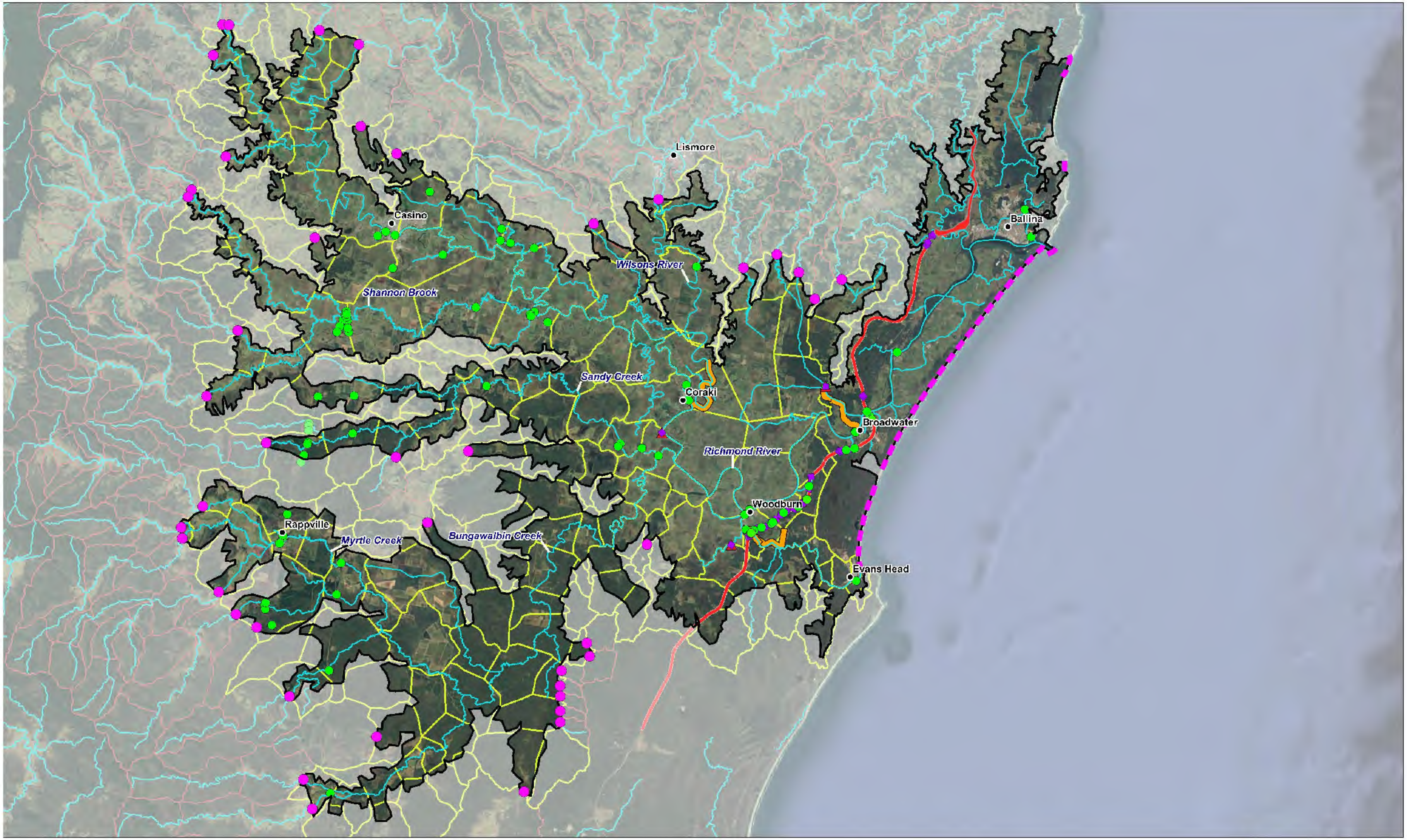
In total, 73 bridges are included in the model of which 13 form part of the Pacific Highway Upgrade. The Pacific Highway and its structures were removed for calibration events which occurred prior to the construction of the highway upgrade.

Other key hydraulic structures included in the TUFLOW model are:

- Tuckombil Weir (modelled as a breakline with an elevation at 0.94m AHD)
- Jabour Weir (modelled as a breakline)
- Rocky Mouth Creek Floodgates (modelled as a nested 1D element)
- Bagotville Barrage (modelled as a nested 1D element)
- West Coraki Canal floodgate (modelled as a nested 1D element).

Floodgate structures associated with minor drainage channels have typically not been included.

Figure 5.4 shows the locations of modelled structures.



LEGEND

- Main Watercourses
- Model Extent
- Inflow: Total u/s Flow-Time 'QT'
- Inflow: Local Source-Area 'SA' extent
- Culverts (1D)
- Floodgates (1D)
- Bridge/Overpass (2D layered flow constrictions)
- Pacific Highway
- Significant Levees
- Outflow: Level-Time 'HT'

Aerial image: Google Satellite

Title:

Hydraulic Model Configuration - Boundaries and Structures

Figure: 5-4

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6 Model Calibration and Validation

6.1 Overview

6.1.1 Introduction

Model calibration is the process by which model parameters are adjusted within acceptable bounds until the model is deemed to adequately represent real world behaviour. Historical flood events are used as the basis for achieving this with the goal of having modelled flood behaviour matching closely with recorded flood data.

6.1.2 Historical Event Selection

Calibration events (historical flood events) are typically selected as being events of significant magnitude (i.e. significant enough to cause flooding) and that have good availability of calibration data.

Calibration data is the data available on historical flood events that can be used to compare against model results. Such data can include recorded rainfall depths and river/ocean levels at gauges, surveyed peak flood levels e.g. debris marks, and anecdotal data such as eyewitness accounts and photos.

Four historic events have been used for model calibration/validation purposes. The events of February/March 2022, January 2008 and May 2009 have been used to calibrate the hydrologic and hydraulic models and the March/April 2017 event has been used to validate the models. These events were selected as they are some of the largest events to occur within recent years and each has a good coverage of recorded data, particularly the February/March 2022 event.

Table 6.1 summarises approximate period of the contributing rainfall for each selected event. The hydrologic and hydraulic models were modelled for a period greater than this to allow for the peak of the flood to pass through the catchment.

Table 6.1 Calibration Event Durations (Contributing Rainfall)

Event	Start Time	End Time	Rain Event Duration (Days)
January 2008	30/12/2007 09:00	07/01/2008 09:00	8
May 2009	19/05/2009 09:00	24/05/2009 09:00	5
March/April 2017	30/03/2017 00:00	01/04/2017 00:00	2
February/March 2022	23/02/2022 00:00	01/03/2022 00:00	6

Table 6.2 compares the peak flood levels recorded at key gauges within the catchment for each calibration event. Gauge locations are included in Figure 6.1. The highest level across the four events occurred during the 2022 event at each gauge.

Table 6.2 Peak Recorded Water Levels (m AHD) at Gauges for Selected Events

Gauge	Watercourse	2008	2009	2017	2022
Casino	Richmond River	21.2	18.9	18.0	22.9^
Yorklea	Shannon Brook	23.7	23.7	22.8	24.0
East Gundurimba	Wilsons River	8.7	9.4	10.3	12.9^
Rappville	Myrtle Creek	37.4	37.8	37.3	38.0
Coraki	Richmond River	5.8*	5.8*	6.0	6.8
Woodburn	Richmond River	3.1	3.2	3.2	6.4
Irongates	Evans River	1.1	1.5	1.3	4.4
Evans Head	Evans River	1.1	1.5	1.2	2.7 / 2.8^

***Based on calibrated hydraulic model results due to gauge failure or missing data**

^ WaterNSW adjusted level based on flood mark

6.1.3 Calibration Approach

A joint calibration approach has been undertaken, calibrating both hydrologic and hydraulic models together. This ensures that flows are comparable between the two models which is an important outcome if the hydrologic model is to be used for undertaking design event selection later in the study.

The URBS hydrologic model takes rainfall data as its main input. The rainfall data is composed of data from available daily rainfall stations and pluviograph (sub-daily) stations. The rainfall was allocated to sub-areas using URBS's SUBRAIN utility. The SUBRAIN utility applies a rainfall event depth to each sub-area using inverse distance weighting based on recorded rainfall depths at surrounding gauges. The rainfall temporal pattern is allocated based on the closest pluviograph station to each sub-area. Details of the stations used for each event are described in the event specific sections below.

An initial and continuing rainfall loss (IL-CL) approach was used for the URBS model. This is widely used for flood modelling and is considered by ARR2019 to be the most suitable rainfall excess technique for design flood estimation (ARR2019 Book 5, Chap 3).

The URBS initial and continuing loss values, along with the catchment routing parameters (alpha and beta), were adjusted for each event until a satisfactory fit was achieved to recorded (rated) flows at key gauges. Due to the complexity of flow interactions and tidal influences, gauge locations downstream of Casino (Richmond River), Yorklea (Shannon Brook), East Gundurimba (Wilsons River) and Rappville (Bungawalbin Creek) were not verified in the hydrologic model and the hydraulic model results were relied upon to inform the hydrologic calibration.

Flows output by URBS were then input into the TUFLOW hydraulic model and the modelled flood levels were compared against the recorded levels at gauges and other datapoints. The TUFLOW Manning's n values for hydraulic roughness were adjusted within an acceptable range to improve the calibration.

A key outcome of the calibration is a consistent set of parameters (URBS: alpha, beta and continuing loss[6] , TUFLOW: Manning's n) across all events which can then be used in design event modelling.

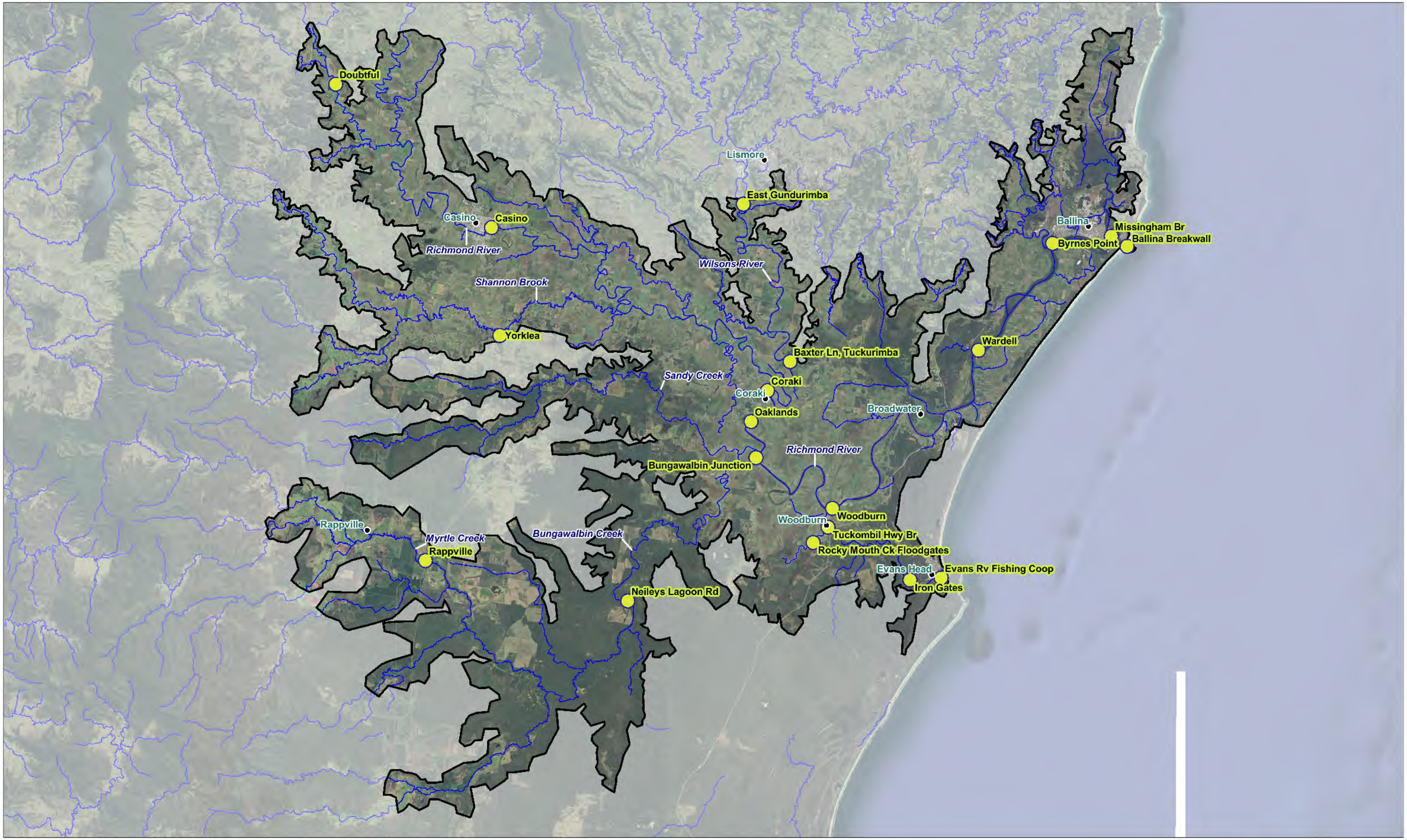
[6] Initial loss can vary significantly for different events as it is highly dependent on the antecedent wetness of the catchment prior to the event. A consistent initial loss across multiple events is therefore not sought.

6.1.4 Presentation of Calibration/Verification Results

Calibration results are presented in two main ways as follows:

- A series of hydrograph plots contained in Annex A The hydrograph plots compare modelled flood levels against recorded flood levels at gauging stations (where available). Modelled flows (both URBS and TUFLOW) are shown on the plots at relevant gauges.
- Figures (included below) showing comparisons between the modelled and recorded peak flood levels at flood marks for the events of 2008, 2009 and 2022. The symbols and colours applied to the flood marks indicate whether the model achieved the desired tolerance.
- A series of drawings contained in Annex B. The drawings show the peak flood extent, flood depths and peak level contours at 1m intervals.

Section 6.2 to Section 6.5 present an overview of each modelled historical event along with details of the model calibration and validation outcomes. Figure 6.1 shows the locations of the river level gauges used in the assessment.



LEGEND

Model Extent

Main Watercourses

Calibration_Gauges

Aerial image: Google Satellite

Title:

Calibration Gauges

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Figure:

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6.2 February/March 2022 Event

6.2.1 Event Overview

The February/March 2022 event resulted from a deep low pressure system that led to intense rainfall falling across the catchment between 23 February and 1 March 2022 with the majority of the rain falling on the 27 and 28 February. Rainfall within the Richmond River catchment exceeded 7-day average rainfalls by 40% to 60% (BoM, 2022). The rain fell onto a catchment which was already wet from previous rainfall events leading to rapid catchment saturation and significant runoff generation.

The runoff caused devastating flooding, particularly to the town of Lismore (Wilsons River), Coraki and Woodburn where it exceeded the previous record flood levels by a significant margin.

Event rainfall data was compiled from 44 pluviograph rainfall stations. A plot of the cumulative rainfall at the 44 pluviograph stations is shown in Figure 6.2. The locations of the pluviograph rainfall stations used in the assessment are shown in Figure 6.3. Figure 6.3 also shows the distribution of rainfall across the catchment as applied in URBS. A full list of the rainfall stations used along with the event rainfall depths is provided in Annex C. Figure 6.2 shows that up to 200mm initially fell early in the event (23 to 24 February) with the majority of rainfall falling in the 48 hours spanning the 27 and 28 February. The rainfall totals across this 48 hour period were particularly high in the Wilsons catchment with 48 hour totals above 1000mm being recorded at a number of gauges.

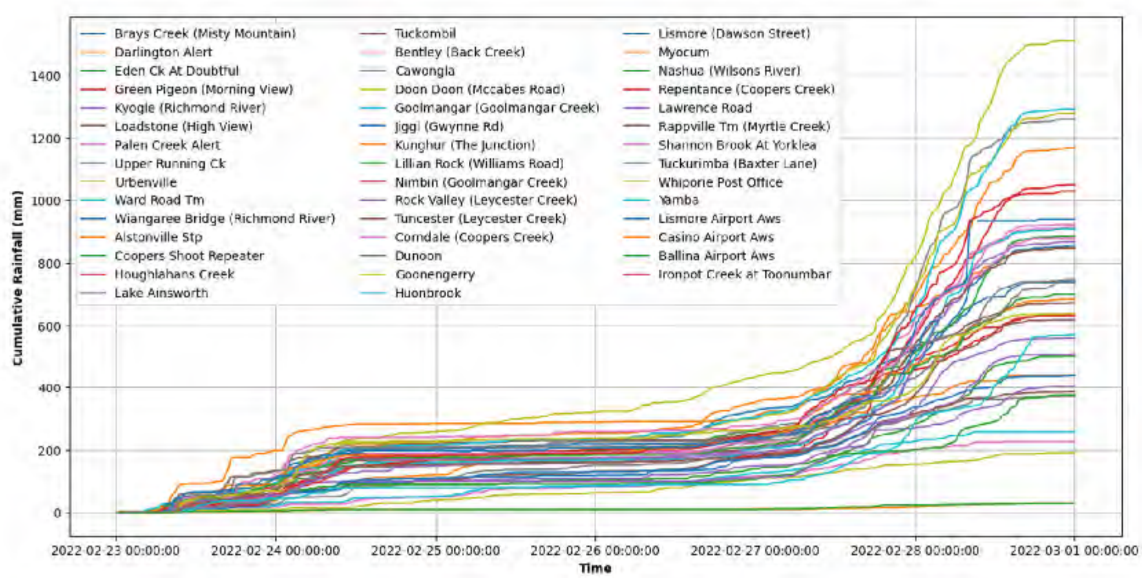
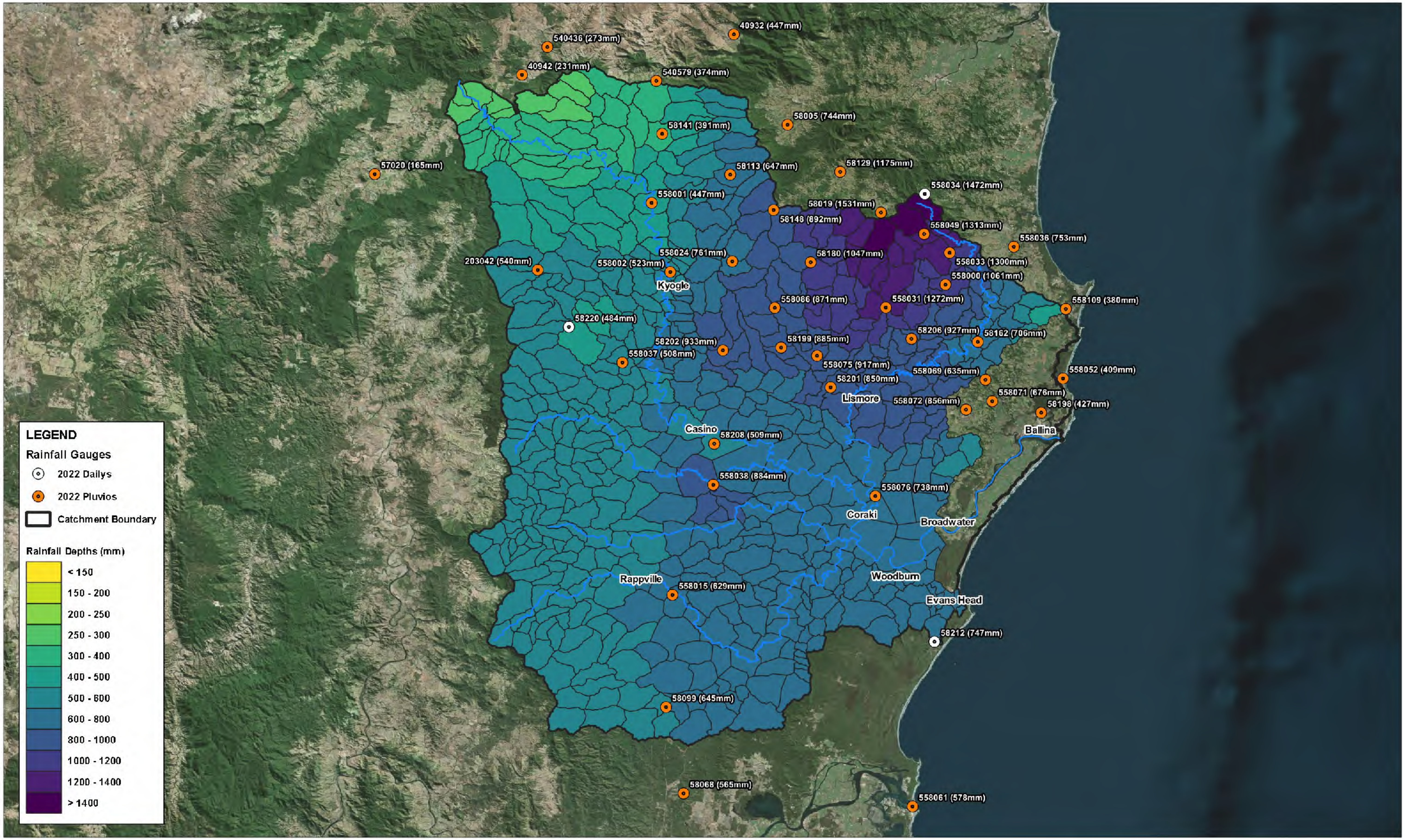


Figure 6.2 February/March 2022 Event: Cumulative Rainfall



Title:

February 2022 Event: Rainfall Gauge Locations and Modelled Rainfall

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6.2.2 Model Calibration

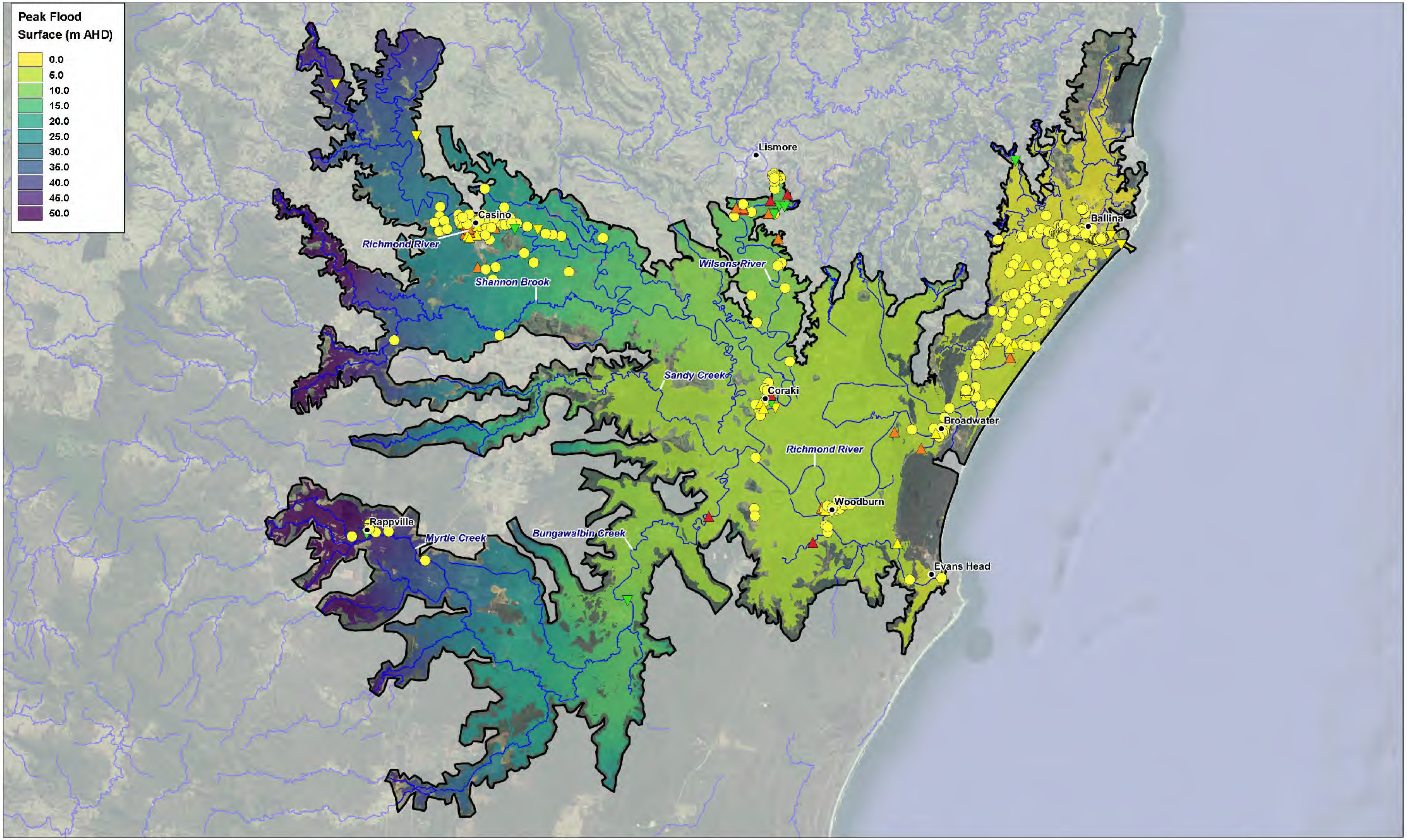
Figure 6.4 presents the calibration performance against peak levels recorded at the 437 flood marks within the hydraulic model extent. Annex A contains plots in which model hydrographs are compared to recorded hydrographs at each gauge location. Annex B contains a full set of drawings showing modelled flood extent, depth and peak level contours for the 2022 event.

At all available gauges, the model shows a good match to the overall hydrograph shape and the peak flood levels are also replicated well by the model. The event includes an initial smaller flood peak prior to the main peak. This smaller peak is slightly overpredicted by the model on the Wilsons River although provides a good match to the recorded peak further downstream at Coraki and Woodburn.

A good match of modelled peak flood levels to recorded flood marks is achieved as shown in Figure 6.4 with the majority of flood marks (346 out of 437) being within the desired 0.3m tolerance of the recorded flood level. Two areas showing general exceptions to this are for a short section of the Richmond River through Casino where a cluster of flood marks indicate that the modelled flood levels tend to be high and for an area on the Wilsons River where different floodmarks, in close proximity, indicate that the modelled level is both too high and too low (this is likely due to uncertainties in the recorded peak levels as such differences in water level over a short distance would not be expected).

Where the modelled flood levels are shown to be too high on the Richmond River at Casino it is noted that the model shows a reasonable match to peak flood levels both upstream and downstream of this area. It is possible that the high velocities of this event have resulted in the stripping of vegetation (as evidenced by aerial imagery taken after the event) which in turn has lowered the hydraulic roughness. This stripping of vegetation would have contributed to locally lower flood levels and would not be represented in the model. This is discussed further in Section 6.6.

Overall the February/March 2022 calibration is considered to show good representation of event behaviour.



LEGEND

Model Extent

Main Watercourses

Peak Flood level Difference
(Modelled minus Observed)

- < -1.0m
- -1.0m - -0.5m
- -0.5m - -0.3m
- -0.3m - 0.3m
- 0.3m - 0.5m
- 0.5m - 1.0m
- > 1.0m
- Outside of Model Domain

Aerial image: Google Satellite

Title:
February/March 2022 Event: Calibration to Flood Marks

Figure: 6-4

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6.3 January 2008 Event

6.3.1 Event Overview

The January 2008 event resulted from heavy and persistent rainfall across much of the catchment. The heaviest falls were recorded across parts of the Upper Richmond in the Border Ranges, north-east of Kyogle where 650mm was recorded over an 8-day period from 30 December 2007. Heavy rainfall also occurred across the Wilsons catchment with over 400mm recorded across the same 8-day period.

The rainfall caused major flooding in Kyogle and the area surrounding Casino on 5 and 6 of January. Moderate flooding occurred further downstream towards Coraki and minor flooding was evident at Woodburn.

Event rainfall data was compiled from 27 daily and 11 pluviograph rainfall stations. A plot of the cumulative rainfall at the 11 pluviograph stations is shown in Figure 6.5. The locations of the pluviograph and daily rainfall stations used in the assessment are shown in Figure 6.6. Figure 6.6 also shows the distribution of rainfall across the catchment as applied in URBS. A full list of the rainfall stations used along with the event rainfall depths is provided in Annex C.

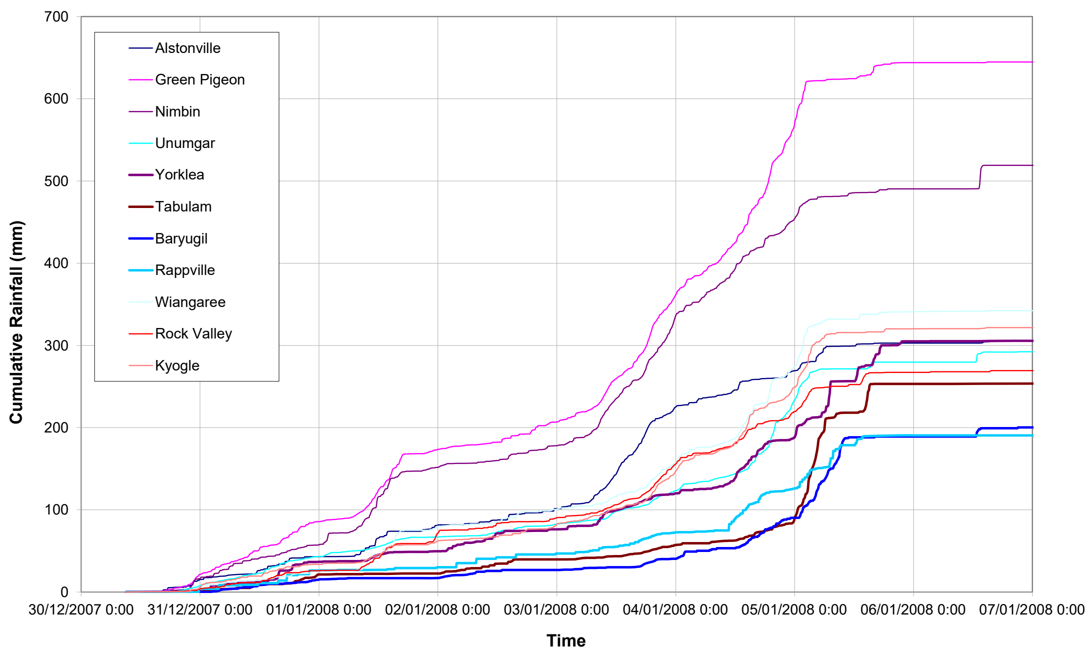
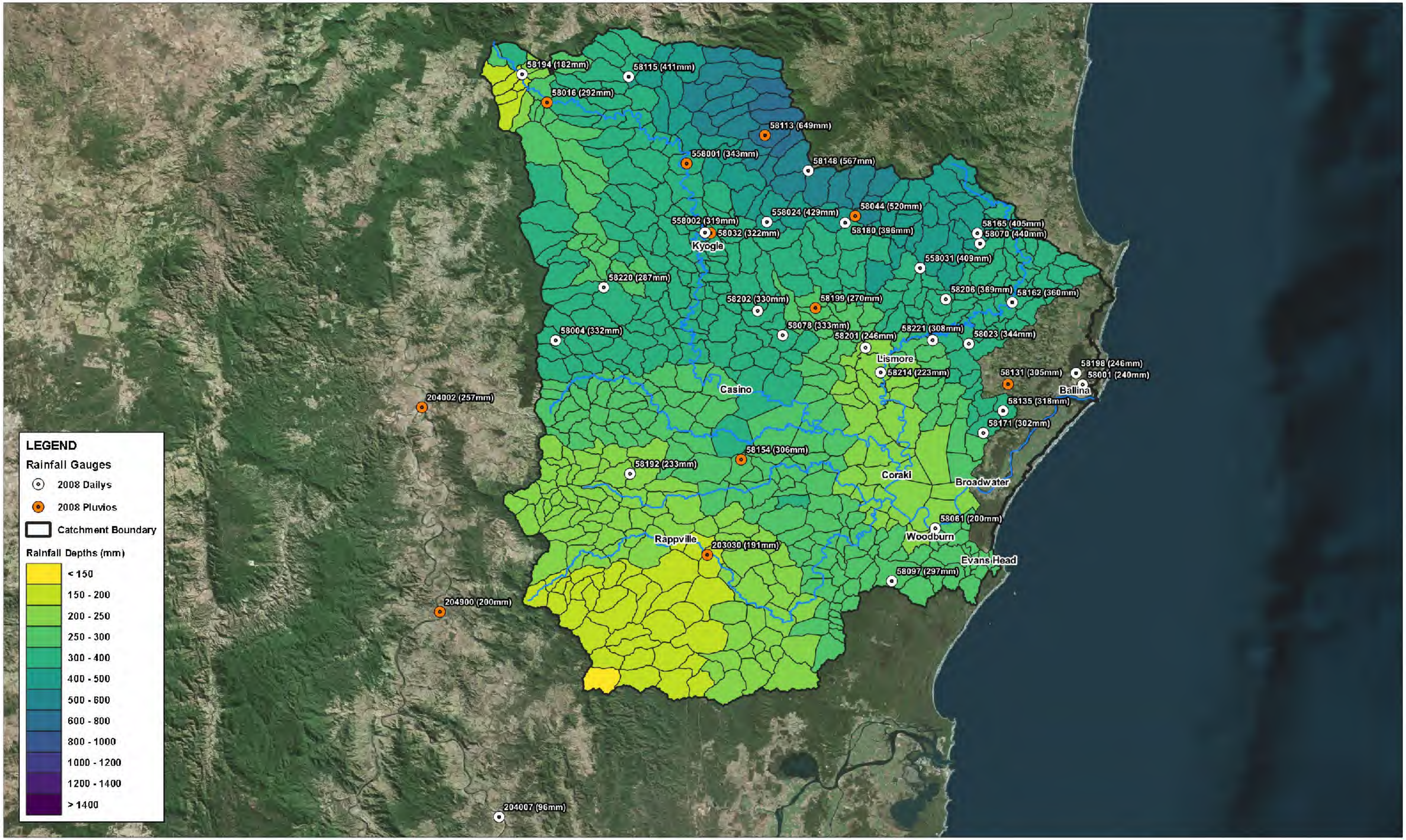


Figure 6.5 January 2008 Event: Cumulative Rainfall



Title:

January 2008 Event: Rainfall Gauge Locations and Modelled Rainfall

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6.3.2 Model Calibration

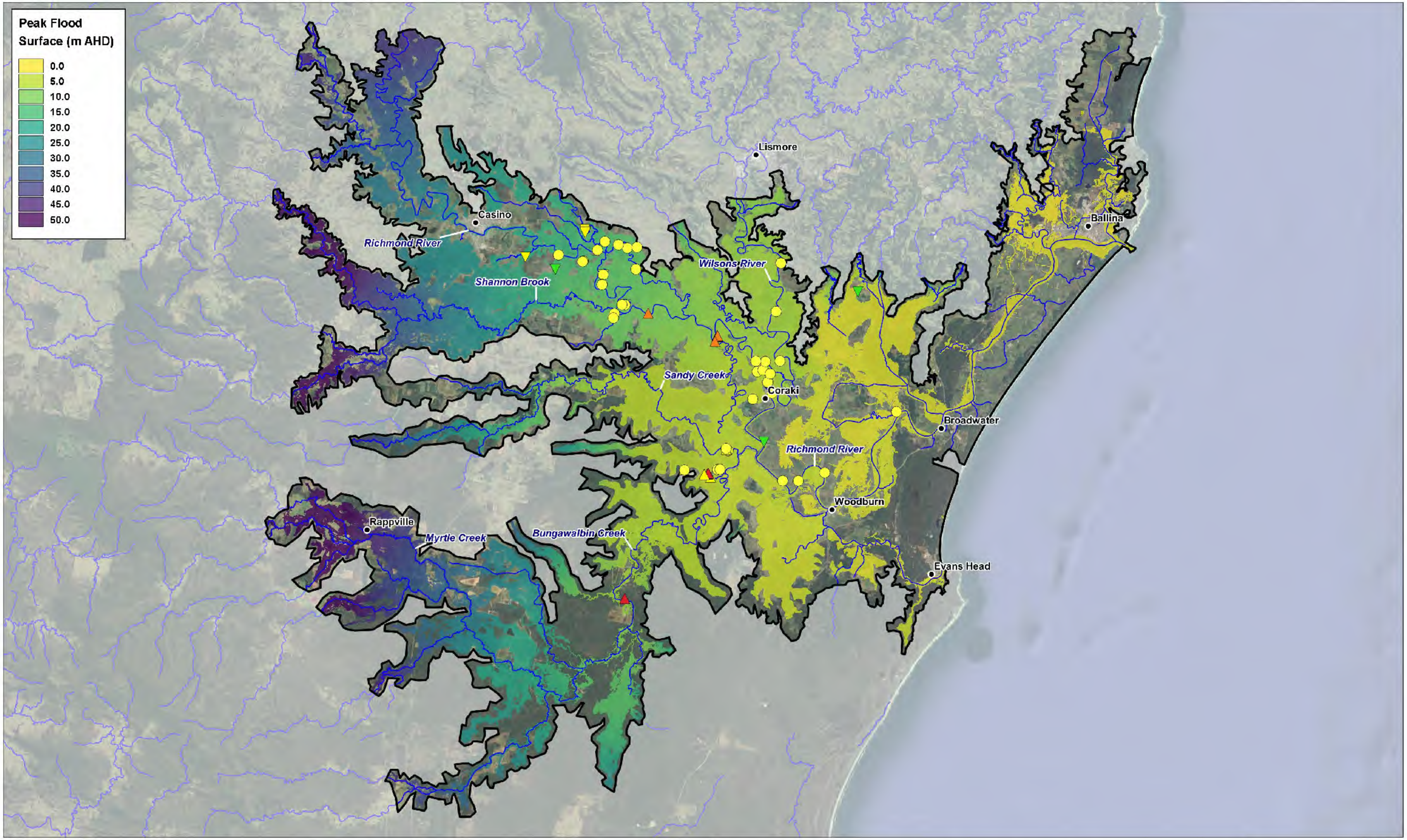
Figure 6.7 presents the calibration performance against peak levels recorded at flood marks and Annex A contains plots in which model hydrographs are compared to recorded hydrographs at each gauge location. Annex B contains a full set of drawings showing modelled flood extent, depth and peak level contours for the 2008 event.

Overall, the modelled results show a good match to the recorded hydrograph peaks, shapes and timings. The model is underpredicting peak flood levels in the Upper Richmond between the gauge at Doubtful but provides a good match to the peak level timing of the flood at Casino (see 2008 Sheet 1 Annex A). A good match to flood marks is generally achieved as shown in Figure 6.7. There is only one recorded flood mark in the Bungawalbin catchment and this indicates the model overpredicting the peak level. It is difficult to draw any firm conclusions from this noting that a good match has been made to the recorded flood timing and peak at the Rappville gauge. The 2010 Flood Study allocated this flood mark a reliability rating of medium suggesting there is some uncertainty associated with it.

The gauge at Rocky Mouth Creek Floodgates shows notable differences between modelled and recorded levels with the modelled flood peak around 0.8m higher than recorded. This was investigated further (see discussion in Section 6.6) and it was found that the recorded data for this event was most likely unreliable.

Elsewhere the calibration is considered to provide a good match to peak flood levels as evidenced by the flood marks and gauges. Overall, out of 67 flood marks, 38 are within 0.3m, with 30 of those within 0.2m, When compared back to the 2010 Flood Study there is an overall improvement in the calibration at all gauges, most notably at East Gundurimba, Bungawalbin Junction and Woodburn.

Overall the January 2008 calibration is considered to show good representation of event behaviour.



LEGEND

Model Extent

Main Watercourses

Peak Flood level Difference (Modelled minus Observed)

- < -1.0m
- -1.0m - -0.5m
- -0.5m - -0.3m
- -0.3m - 0.3m
- 0.3m - 0.5m
- 0.5m - 1.0m
- > 1.0m
- Outside of Model Domain

Aerial image: Google Satellite

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January 2008 Event: Calibration to Flood Marks

Figure: 6-7

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6.4 May 2009 Event

6.4.1 Event overview

Between 20 and 22 May 2009 heavy rainfall fell across the Richmond River catchment. The most intense rainfall fell across the upper parts of the Wilsons catchment with intense rainfall also falling across the majority of the Bungawalbin Creek catchment. The combined effect of large flows within the Wilsons River, Richmond River and Bungawalbin Creek resulted in extensive flooding across the Mid-Richmond area around Coraki. A king tide also contributed to elevated levels on the Lower Richmond and minor flooding was experienced at Woodburn.

Event rainfall data was compiled from 22 daily and 22 pluviograph rainfall stations. A plot of the cumulative rainfall at the 22 pluviograph stations is shown in Figure 6.8. The locations of the pluviograph and daily rainfall stations used in the assessment are shown in Figure 6.9. Figure 6.9 also shows the distribution of rainfall across the catchment as applied in URBS. A full list of the rainfall stations used along with the event rainfall depths is provided in Annex C.

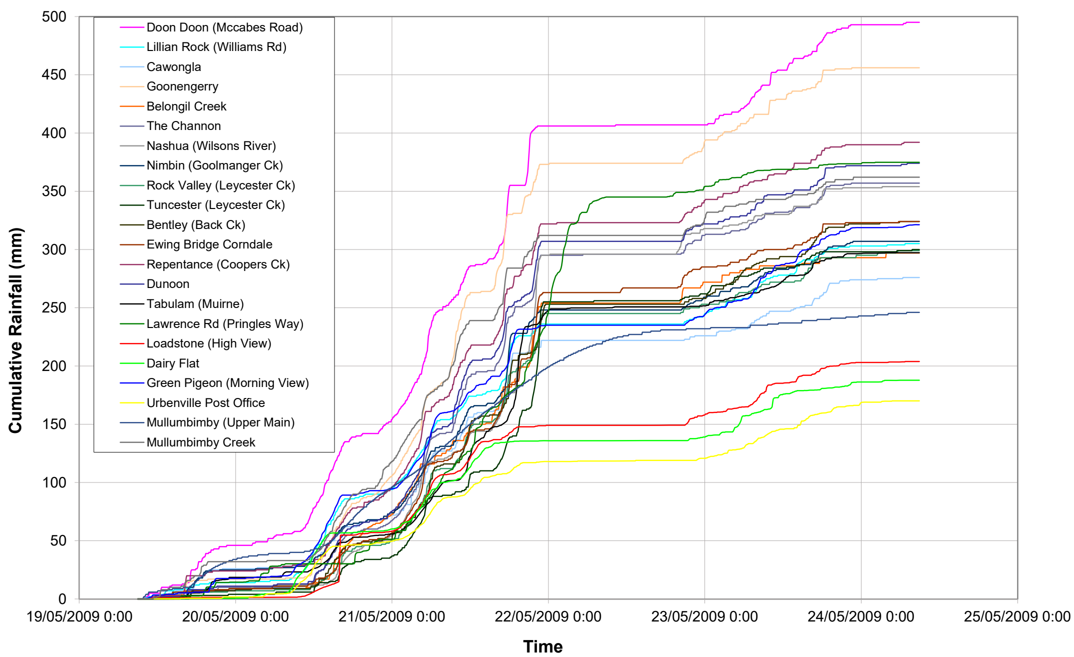
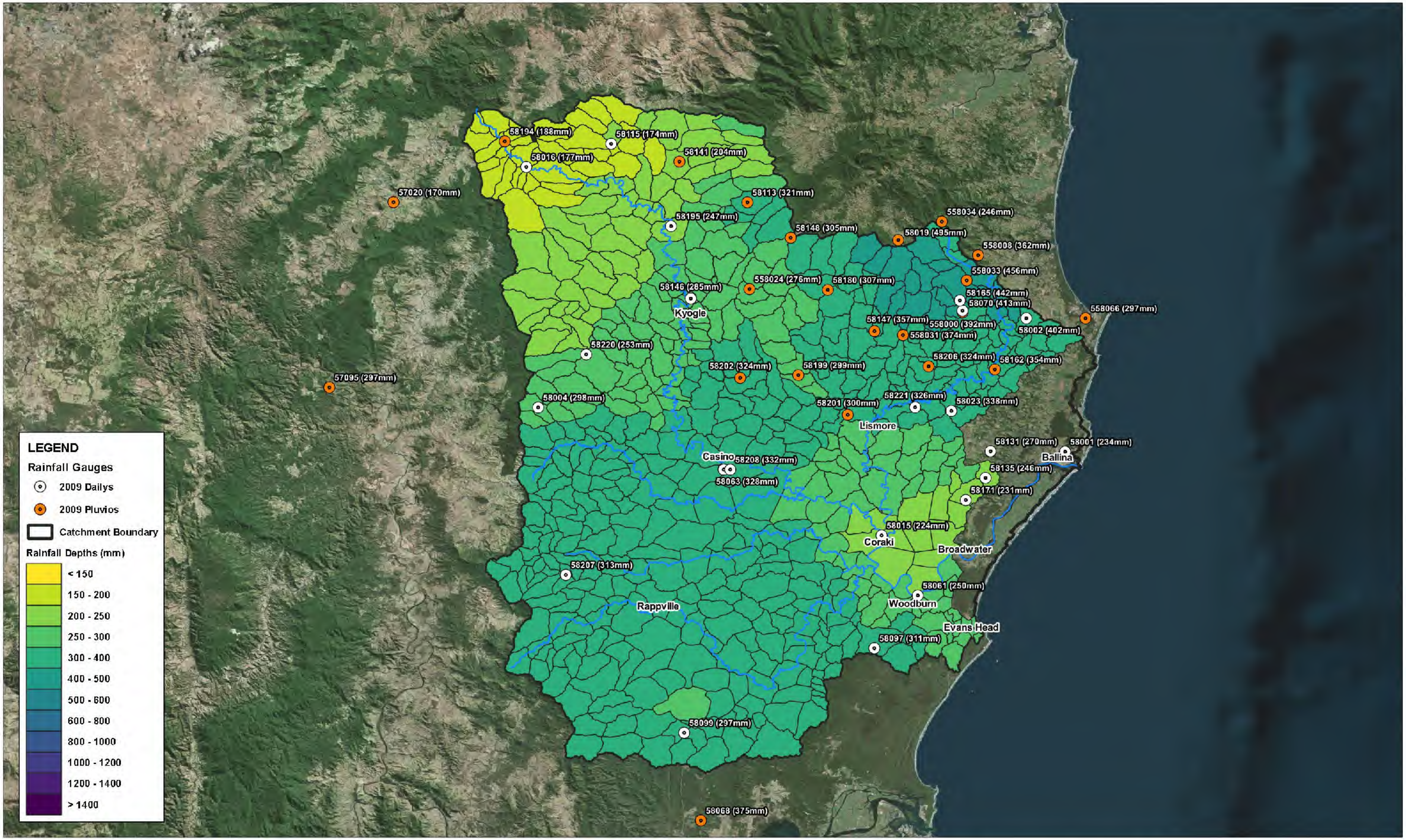


Figure 6.8 May 2009 Event: Cumulative Rainfall



Title:

May 2009 Event: Rainfall Gauge Locations and Modelled Rainfall

Figure: 6-9

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6.4.2 Model Calibration

Figure 6.10 presents the calibration performance against peak levels recorded at flood marks and Annex A contains plots in which model hydrographs are compared to recorded hydrographs at each gauge location. Annex B contains a full set of drawings showing modelled flood extent, depth and peak level contours for the event.

The calibration plots in Annex A show a good match between modelled results and recorded levels at the majority of gauge locations. In particular, the results at the Casino gauge show a very good match to the recorded data, capturing the shape, magnitude and timing of both peaks.

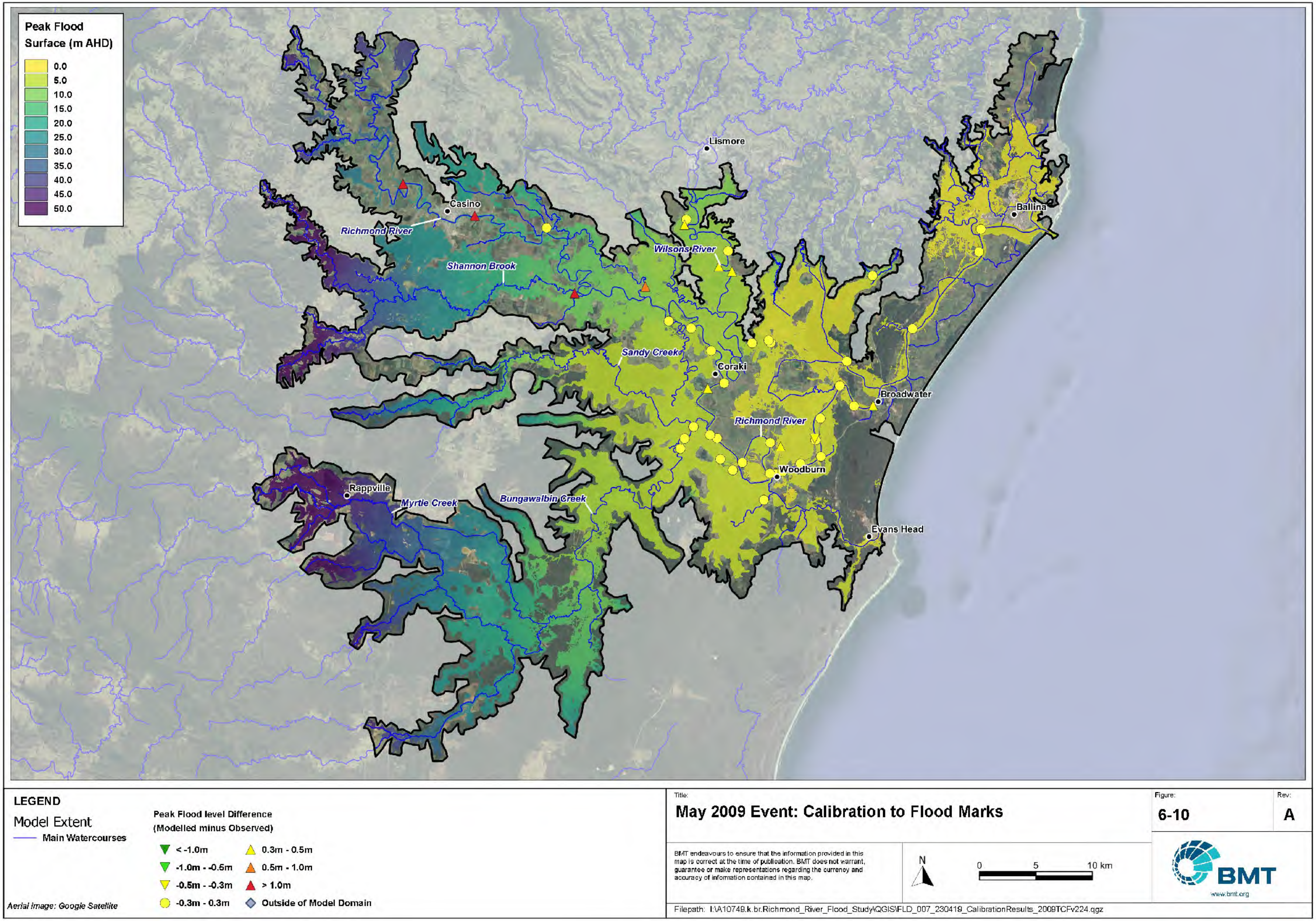
The model results show some additional volume coming through after the flood peak at the East Gundurimba (Wilsons) and Rappville (Bungawalbin) gauges. However, this doesn't appear to affect the Mid to Lower Richmond with the modelled hydrographs matching very well against recorded hydrographs at Bungawalbin Junction and Woodburn.

The calibration against flood marks (Figure 6.10) shows that the modelled flood levels were within 0.3m of the vast majority of flood marks, in particular for the area around Coraki and downstream. This includes the lower Bungawalbin and Rocky Mouth Creek where the 2009 event was the largest event modelled. Overall, out of 47 flood marks, 35 were within 0.3m, with 28 of those within 0.2m.

Whilst the model shows a good match to the gauge at Casino, the flood marks indicate that the modelled levels are too high both upstream and downstream of Casino. The limited number of floodmarks in this location mean it is difficult to place too much confidence in them and greater confidence is placed in the recorded levels at the Casino gauge, which show a very good match.

Where gauge comparisons can be made, the calibration compares similarly with the 2010 Flood Study. The 2010 Flood Study provided a slightly better match at the East Gundurimba gauge but the current calibration provides a better match to the hydrograph shape and timing at Woodburn. The 2009 event was also a calibration event for the Evans River Flood Study (BMT WBM, 2014). It is noted that the model performance in the current study is significantly improved at the Iron Gates and Evans Head gauges on the Evans River compared to that of the 2014 Flood Study.

Overall the 2009 calibration is considered to show a very good representation of event behaviour.



Title:

May 2009 Event: Calibration to Flood Marks

Figure: 6-10

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6.5 March/April 2017 Event

6.5.1 Event Overview

On the 30 and 31 March 2017, ex Tropical Cyclone Debbie caused heavy rainfall over parts of the Richmond catchment with most of the rainfall falling within a 24 hour period. Much of the Wilsons catchment recorded falls of between 500mm and 600mm. Notable falls also occurred near Evans Head with 326mm recorded at the Evans Head RAAF gauge.

The high intensity rainfall caused major flooding in Lismore and the CBD levee was consequently overtopped. The event was less severe elsewhere in the catchment although the flows coming down from the Wilsons catchment still resulted in high Richmond River levels at Coraki and Woodburn, causing flooding.

Event rainfall data was compiled from 41 pluviograph rainfall stations. A plot of the cumulative rainfall at the 41 pluviograph stations is shown in Figure 6.11. The locations of the pluviograph rainfall stations used in the assessment are shown in Figure 6.12. Figure 6.12 also shows the distribution of rainfall across the catchment as applied in URBS. A full list of the rainfall stations used along with the event rainfall depths is provided in Annex C. Figure 6.11 shows that the majority of rain fell in 24 hours with a relatively consistent intensity.

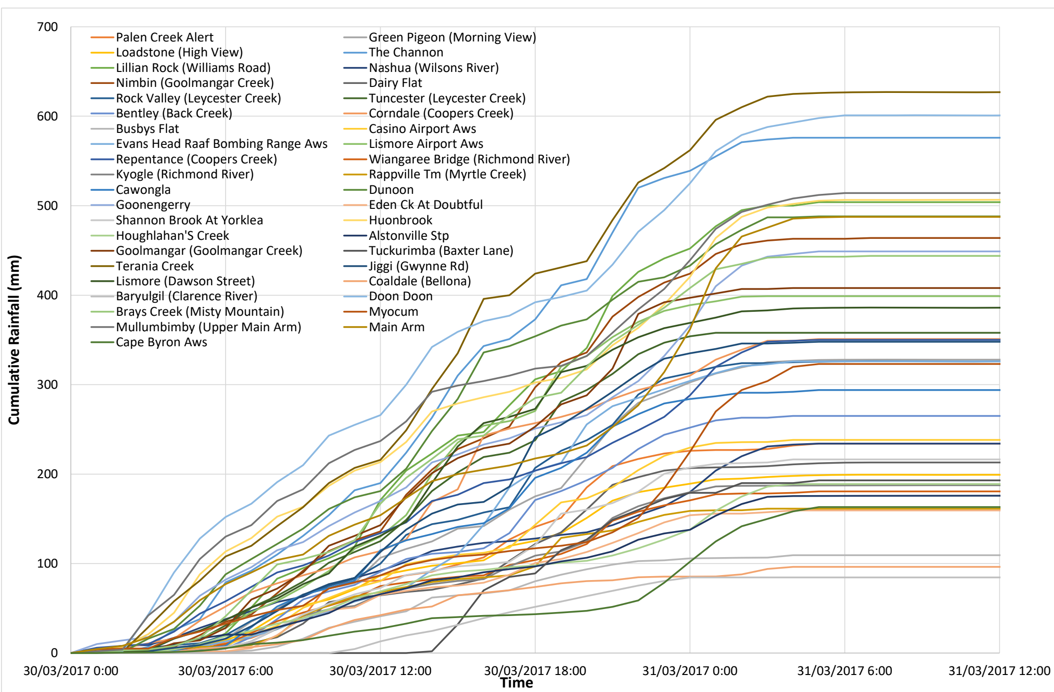
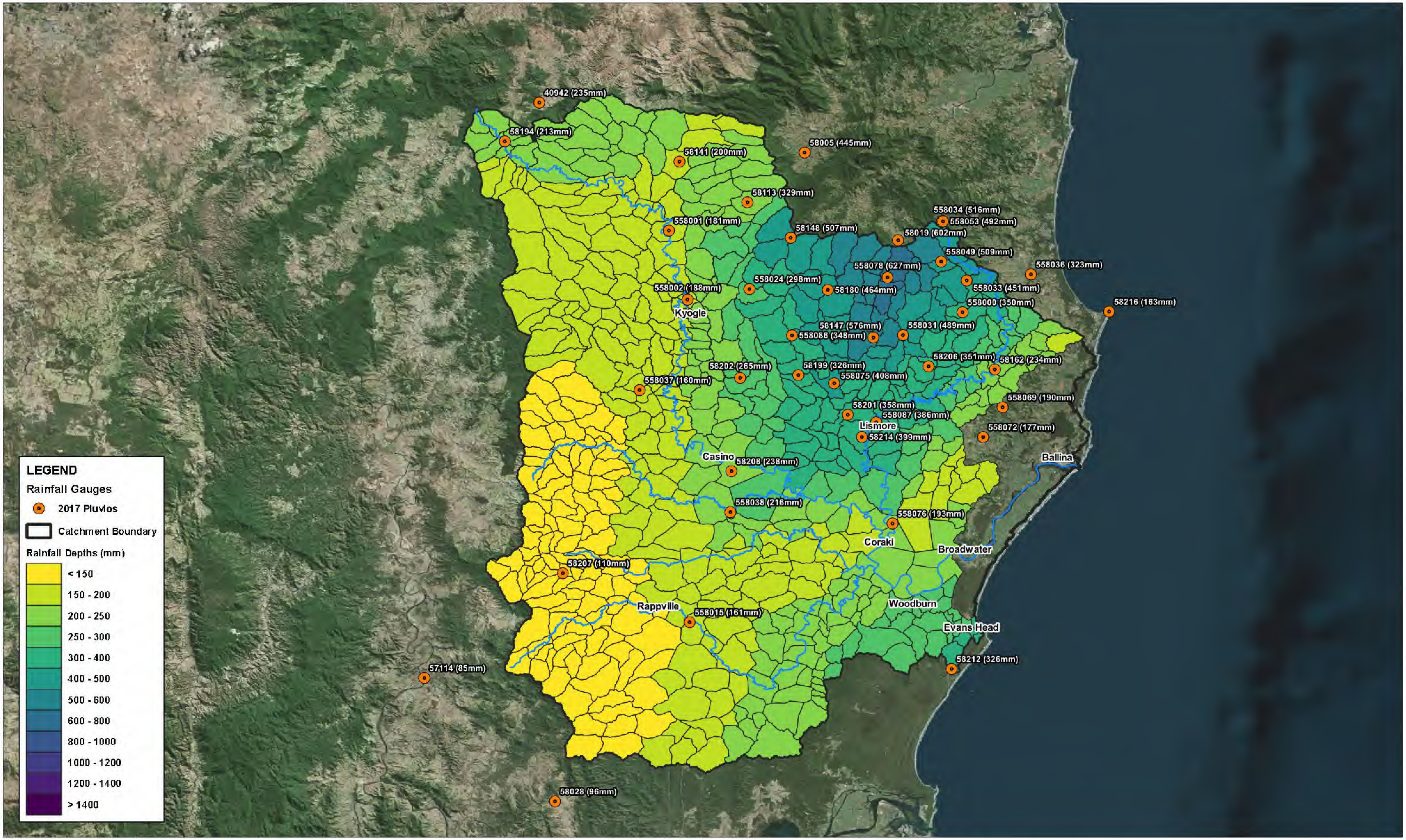


Figure 6.11 March/April 2017 Event: Cumulative Rainfall



Title:

March/April 2017 Event: Rainfall Gauge Locations and Modelled Rainfall

Figure: 6-12

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6.5.2 Model Validation

The March/April 2017 event was treated as a validation event, whereby the model parameters derived through calibrating the January 2008, May 2009 and February/March 2022 events were applied, unadjusted, to the March/April 2017 event. The results provided a good match to recorded data and this was improved upon slightly by applying a minor increase to the continuing rainfall loss in the Wilsons catchment to better match the hydrograph at East Gundurimba (from 2mm/h to 3mm/h). This was done as the flows from the Wilsons catchment are the dominant cause of downstream flooding and so a good match at East Gundurimba was important for the overall event simulation. The revised continuing loss also matched with the continuing loss used for this event in the Lismore Floodplain Risk Management Study.

Annex A contains plots in which model hydrographs are compared to recorded hydrographs at each gauge location. Annex B contains a full set of drawings showing modelled flood extent, depth and peak level contours for the event. There were no available peak flood level marks to which modelled levels could be compared for this event.

Overall the validation shows a good match to recorded data at the gauges with the shape magnitude and timing generally matching well for the gauges on the Richmond River. As for the 2008 event, the model underpredicts the flood peak at the Doubtful gauge. There is also a slight underprediction in the peak flood level and earlier model peak timing at the Neileys Lagoon Road gauge on Bungawalbin Creek. In turn, this causes less breakout flow to enter Rocky Mouth Creek and so there is a slight underprediction in flood levels at the Rocky Mouth Creek and Tuckombil Highway Bridge gauges.

The model slightly overpredicts the flood peak at Casino but the shape of the modelled hydrograph is notably different. The recorded shape is somewhat unusual as it has a very flat, extended peak which does not accord with the nature of the event of gauges elsewhere. Some further investigation was undertaken into the flood levels at Casino for the 2017 event and this is discussed in more detail in 6.6.

In summary, the model is considered to have been validated by replicating the March/April 2017 event flood behaviour to a satisfactory level. This provides confidence in the overall calibration of the hydrologic and hydraulic models.

6.6 Discussion on Calibration Performance

6.6.1 Overview

Overall it is considered that the model provides a good calibration across the catchment for the events under consideration. The modelled peak flood levels generally meet or exceed the desired tolerances against recorded levels of +/- 0.3m in urban areas and +/-0.5m elsewhere.

The model reproduces the timing and shape of the floods at the gauges and is therefore considered to provide reliable representation of the flood propagation speeds.

In summary, the performance of the model to be able to reproduce historical events is considered to be of a high standard, with the vast majority of the model not requiring any special attention or investigation during the calibration process. Aspects of the calibration that warrant further discussion are given in the following sections.

6.6.2 Casino

The hydraulic model replicates the recorded water levels at Casino very well for the January 2008 and May 2009 events. For the February/March 2022 event and the March/April 2017 event there are some differences between modelled and recorded peak levels that are warrant further discussion.

February/March 2022 Event

In the February/March 2022 event the model predicts a slightly higher flood peak level on the Richmond River at the Casino gauge (by 0.1m) than that recorded. This results in the model showing additional water spilling from the river into Casino than occurred, as evidenced by higher modelled peak flood levels at a number of surveyed flood marks within Casino. The model is typically around 0.2m higher than the recorded level at these flood marks. This difference remains within the calibration tolerance but was further investigated.

Aerial imagery taken before and after the February/March 2022 event (Figure 6.13) shows that the event resulted in notable stripping of in-bank vegetation on the Richmond River through Casino. A modelled sensitivity test for the February/March 2022 event, in which the Manning's n roughness values were reduced along the river banks, resulted in slightly lower river levels which matched more closely to the recorded level at the gauge. Furthermore, an improved match was also obtained against the surveyed flood marks within Casino.

Given that a consistent set of Manning's n roughness values are required for design event simulations the Manning's n values representing the denser vegetation have been retained, noting that this will give a result that has some conservatism with regards to peak flood levels in Casino.

Figure 6.13 Aerial imagery of Casino before (left) and after (right) the February/March 2022 flood

March/April 2017 Event

For the March/April 2017 event, the recorded hydrograph shape is notably different than the modelled hydrograph shape. The modelled peak level is also 0.4m higher than the recorded peak flood level at the Casino gauge. The flat extended nature of the recorded hydrograph at the Casino gauge appears dubious given the nature of the causative storm. It was found during the model calibration that a better match to the falling limb of the hydrograph could be achieved but only at the expense of a higher modelled peak level. Furthermore, no combination of model parameters appeared able to generate the flat peak shown in recorded data. It is BMT's suspicion that the 2017 peak flood level at the Casino gauge was higher than that recorded and that there was a partial gauge failure meaning the true flood peak was not captured.

There were no available flood debris marks to corroborate peak flood levels within the vicinity of Casino. Council supplied BMT with a photo of the Richmond River at Irving Bridge taken approximately at the time of peak water level during the 2017 event (Figure 6.14). A gauge board is located on the bridge (not visible in the photo). A photo taken from the opposite bank of the river shows the gauge board (Figure 6.15). It was possible to approximately estimate the peak of the 2017 event by using the concrete blocks as a guide reading off the level at the gauge for the estimated water level. This resulted in a peak flood level at Irving Bridge of 10.8m. Once converted to Australian Height Datum (AHD), the peak flood level was estimated to be 19.3mAHD.

The peak modelled flood level for the 2017 event at Irving Bridge is 19.2mAHD which indicates that the model is slightly lower (by 0.1m) than the peak of the flood. At the Casino gauge, the model was showing flood levels 0.4m higher than recorded levels. Given the proximity of Irving Bridge and Casino Gauge it is BMTs opinion that the gauge suffered a minor malfunction during the event and did not capture the peak. It is estimated that the flood peak was approximately 0.5m higher than that recorded at the Casino gauge.

It is likely therefore that the model underestimates the flood level at the Casino gauge during the 2017 event rather than overestimates it. To obtain a higher flood peak the continuing rainfall losses could be lowered and this would add additional volume into the model resulting in slightly improved fits to the tail of downstream hydrographs at gauges. However, given that the 2017 event was a validation event and that a reasonable match has been obtained, continuing losses were kept unchanged.

Figure 6.14 Approximate Peak of 2017 Event at Irving Bridge (photo taken 11:52am, 1/4/2017)

Figure 6.15 Estimated March/April 2017 Flood Peak at Casino (Irving Bridge) of 10.8m Gauge Level.

6.6.3 Shannon Brook

The match between the recorded and modelled stage hydrograph at the Yorklea gauge on Shannon Brook is considered reasonably good for all four modelled events.

Whilst the modelled stage hydrographs show a good agreement with the recorded data there is a notable difference between the modelled URBS flow and the modelled TUFLOW flow at Yorklea, with the TUFLOW flow being consistently more attenuated. This difference is less pronounced for the February/March 2022 event which was also significantly larger than the other three calibration events at this location. Whilst greater attenuation in the hydraulic model versus a hydrologic model is fairly typical, the degree of difference for the smaller events warrants some discussion.

The floodplain at Shannon Brook is complex and flowpaths within it are highlighted in Figure 6.16. This shows the flood depth and velocity product (flood hazard) which highlights areas of greater conveyance using brighter colours. It can be seen that a large proportion of flow bypasses the Yorklea gauge in a channel approximately 600m north of the gauge. Numerous breakouts also occur upstream of the gauge into the northern floodplain where the flow also bypasses the Yorklea gauge. The flow in these breakout channels will exhibit different timings and be subject to energy losses associated with the complex flow path interactions. In addition, the embanked railway line at Leeville also has a partial damming effect on flows which will increase the flood attenuation.

The hydrology model is unable to replicate this complex flow behaviour although it can be modified, in a limited way, to account for it. Given the very good match to the recorded stage hydrographs at the Yorklea gauge, it was considered not worthwhile to try and improve the hydrologic model any further.

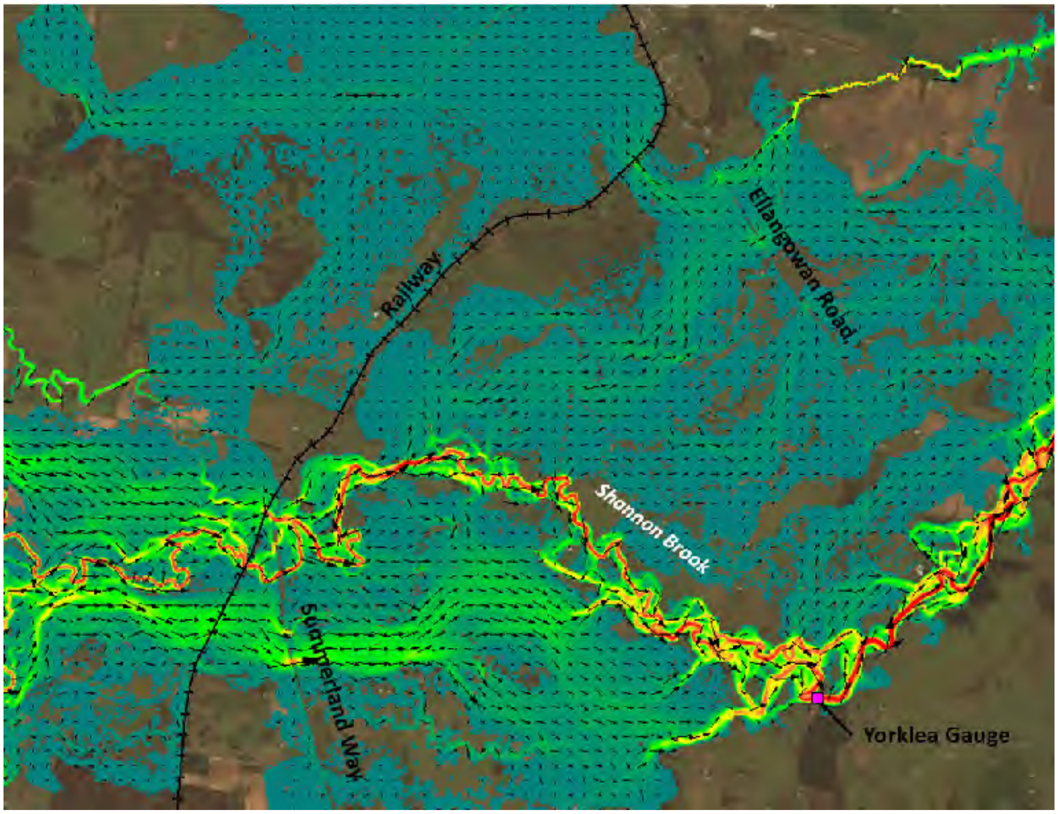


Figure 6.16 May 2009 event: Shannon Brook near Yorklea Gauge

6.6.4 Rocky Mouth Creek/Tuckombil Canal

The peak flood levels on Rocky Mouth Creek, which then enter the Tuckombil Canal, are driven by both backwater from the Richmond River and by breakout flow from Bungawalbin Creek (and Richmond River) which bypasses to the west of Swan Bay. Figure 6.17 uses the depth and velocity product (flood hazard) from the modelled February/March 2022 event to highlight the main flow paths across the floodplain in this area.

The flood levels are therefore dependent on a number of factors which can interact to varying degrees and which make it a challenging location for model calibration. As such the peak level and timing of hydrographs in Rocky Mouth Creek can be very sensitive to the representation of breakout flows in the wider floodplain.

In the May 2009 and March/April 2017 events, the model provided a very good match to the recorded data at Rocky Mouth Creek Floodgates gauge, both in terms of timing and the respective peak levels. In the February/March 2022 event, this gauge failed but the nearby gauge at Tuckombil Highway Bridge showed a very good match. However, for the January 2008 event there was a notable difference between the modelled and recorded data at the Rocky Mouth Creek Floodgates gauge with the model being around 0.8m higher.

The modelled 2008 event peak flood level of approximately 2.9mAHD compares well to both recorded and modelled peak flood levels at Woodburn. Given that the lower reaches of Rocky Mouth Creek experience significant backwater from the Richmond River it seems unlikely a recorded level of around 2.1mAHD at Rocky Mouth Creek Floodgates would occur when then recorded peak flood level at Woodburn was 3.0mAHD. This suggests that the recorded gauge data at Rocky Mouth Creek in the 2008 event is unreliable.

Of note, both the 2008 and 2009 recorded peak levels at Woodburn were similar (3.0mAHD). However, the modelled level at Rocky Mouth Creek Floodgates is around 0.5m higher in the 2009 event than in the 2008 event. The May 2009 event was a large event in Bungawalbin Creek and so the proportion of breakout flow which contributes to Rocky Mouth Creek is greater for this event resulting in a higher peak flood level at the Rocky Mouth Creek gauge compared to the 2008 event.

Whilst it is challenging to get a very good match to recorded levels at the Rocky Mouth Creek and Tuckombil Highway Bridge gauges, the shape and timing of the runoff response is matched reasonably well for all four modelled events, particularly for the 2009 event at Rocky Mouth Creek Floodgates and the 2022 event at Tuckombil Highway Bridge. The 2022 event was also the largest modelled event at this location by a significant margin. It is noted that there is a reasonably good match for all four events between the recorded and modelled stage hydrographs downstream on the Evans River at the Iron Gates gauge. This appears to be a significant improvement over the calibration presented in the Evans River Flood Study (BMT, 2014) and is attributed in part to the regional model's ability to model the complex and interacting flood behaviour from multiple regions of the Richmond River catchment.

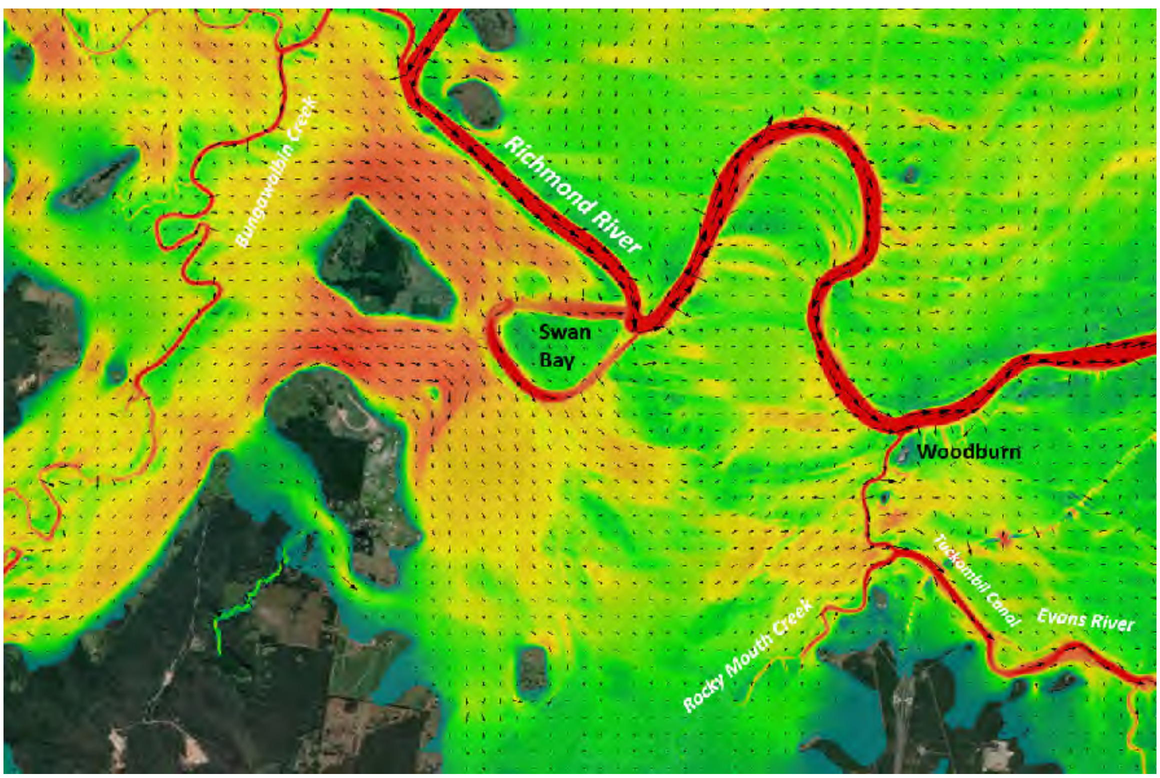


Figure 6.17 February/March 2022 Event showing interaction of flow in floodplain shown using velocity-depth product

6.6.5 Validity of Model Calibration

The model is considered to be well-calibrated for the range of flood magnitudes encompassed by the historical events modelled. For events with flow magnitudes in excess of these historical events, such as some of the design events, the uncertainty is greater. This uncertainty is managed by using model parameters that fall within accepted industry ranges. Furthermore, where possible the design flood modelling results are compared against independent flow estimates derived through flood frequency analyses with results reconciled between the two approaches.

6.7 Summary of Adopted Calibration Parameters

6.7.1 Hydrologic Parameters

The final parameters used in the URBS modelling are summarised in Table 6.3 for each modelled event. Baseflow parameters and Toonumbar Dam starting water levels are provided in Table 6.4 and Table 6.5 respectively. The following is noted with regard to the URBS calibration parameters:

- The routing parameters (alpha and beta) are consistent across the events for each model region. The Bungawalbin region required higher values (greater attenuation).
- Continuing loss values were typically 3mm/hr for the Wilsons region and 2mm/hr for other regions. The 2022 and 2008 events required slightly lower values on the Upper and Wilsons regions respectively.
- The initial loss shows significant variation as would be expected across different events. Lower initial losses were generally required for the March/April 2017 event and February/March 2022 events. It is noted that there was significant rainfall across the catchment in the lead up to these events.
- The continuing loss, alpha and beta values determined for each modelled region will be applied for design event simulations. The initial loss will be informed by analysing both calibration losses and information available on the ARR2019 datahub.
- The default value of 0.8 for the URBS sub-catchment routing exponent (m) was adopted for all events/regions.
- Baseflow parameters were consistent across all modelled events/regions. These parameters have minimal effect on the flood peak.

Table 6.3 URBS Calibration Parameters

Event	Model Region	IL (mm)	CL (mm/hr)	alpha	beta
2008	Upper Richmond	60	1.5	0.2	3.0
	Mid-Richmond	80	2	0.2	3.0
	Wilsons	100	2.5	0.2	3.0
	Bungawalbin	100	2	0.4	7.5
2009	Upper Richmond	60	2	0.2	3.0
	Mid-Richmond	120	2	0.2	3.0
	Wilsons	80	3	0.2	3.0
	Bungawalbin	100	2	0.4	7.5
2017	Upper Richmond	70	2	0.2	3.0

Event	Model Region	IL (mm)	CL (mm/hr)	alpha	beta
	Mid-Richmond	30	2	0.2	3.0
	Wilsons	60	3	0.2	3.0
	Bungawalbin	30	2	0.4	7.5
2022	Upper Richmond	50	1	0.2	3.0
	Mid-Richmond	50	2	0.2	3.0
	Wilsons	50	1	0.2	3.0
	Bungawalbin	50	2	0.4	7.5

Table 6.4 URBS Baseflow Parameters

Event	Model Region	Initial value (B0 (m^3/s))	Recession value (BR (%))	Quick flow proportion (BC (%))	Linearity
All	All	0	0.9	0.05	1

Table 6.5 Toonumbar Dam Initial Water Level

Event	Initial Stage (mAHD)*
2008	129.65
2009	129.70
2017	128.27
2022	129.70

***Full Supply Level is 129.62mAHD**

6.7.2 Hydraulic Parameters

The key hydraulic parameters adjusted for model calibration are the Manning's n values for different land use types. A consistent set of values was adopted across all modelled events and their values are summarised in Table 6.6. These Manning's n values are used for design flood modelling (Section 8), and their application spatially is shown in Figure 5.3.

Table 6.6 Adopted Manning's n values

Land Use Description	Manning's n value
Watercourse (Richmond River downstream of Broadwater and Evans River)	0.021
Watercourse (Richmond River between Coraki and Broadwater, and Evans River)	0.022
Watercourse (Richmond and Wilsons Rivers)	0.03
Watercourse (General)	0.04
Watercourse (Narrow, medium density vegetation)	0.05

Land Use Description	Manning's n value
Watercourse (Narrow, high density vegetation)	0.06
Roads	0.025
Grass (maintained)	0.035
Open grassland	0.05
Pasture / cultivated field	0.06
Light vegetation / lightly vegetated creek	0.07
Light-medium density vegetation	0.08
Medium density vegetation	0.09
Medium-dense vegetation	0.1
Dense vegetation	0.12
Very dense vegetation	0.15
Medium density urban block	0.1
High density urban block	0.2
Commercial / Industrial	0.5

6.8 Suitability of Model for Design Flood Modelling

6.8.1 Overview

The calibrated hydrologic/hydraulic models are used to simulate design flood events, including events of a greater magnitude than events used for calibration. The number of design events requiring simulation when applying ARR2019 methodology can also be substantial. The model build therefore needs to take these factors into consideration. The following sections briefly summarise how these considerations were addressed.

6.8.2 Hydrologic Model

The hydrologic model has been designed so that it can be simulated as a single model encompassing the various catchment regions within it. Where the routing parameters vary between the regions, factors have been used within the model to account for this variation.

6.8.3 Hydraulic Model

Two key considerations with regards to design event simulation in the hydraulic model are:

- Model extent
- Simulation times.

The suitability of the model extent was assessed by simulating the probable maximum flood (PMF) through the model using the 2010 Flood Study hydrologic inputs. The extent of the model was generally found to be suitable to capture the full extent of flooding.

A practical simulation time is a key consideration when applying ARR2019 design hydrology and to ensure that the model has ongoing use as a functional tool for floodplain risk management. The deeper and more extensive inundation for large design floods has the potential to significantly slow down model simulations. It is noted however that design simulations will likely run for a shorter simulation period that the historical calibration events. Table 6.7 summarises simulation times for the four calibration events and tests performed on preliminary design hydrology.

As shown, simulation times are less than 24 hours and may be 10 hours or less for design floods up to the 1% AEP. The testing demonstrates that the model simulation times are not excessive and are suitable for simulating design floods.

Table 6.7 Approximate Model Simulation Times

Event	Event Length (h)	Approximate Simulation Time (h)*
2008	290	20
2009	200	15
2017	160	16
2022	216	20
1% AEP	100	13

*simulations performed on a single GTX2080Ti graphics card

7 Flood Frequency Analysis

7.1 Introduction

There are a number of river gauges within the Richmond River catchment which have a long records of peak flood levels. These historic flood records can be statistically assessed using flood frequency analyses (FFA) to derive probabilistic estimates of peak flows such as the 1% AEP. This provides an independent means of estimating peak flows which can then be compared against peak flows derived from flood modelling. If there is a long and relatively continuous record length along with a reliable rating curve, then the results from the FFA can be considered with a reasonably high degree of confidence. These peak flow estimates would then take precedence over peak flow estimates derived from rainfall-based procedures.

7.2 Previous FFAs

A review has been undertaken of FFAs previously undertaken within the Richmond River catchment. The most relevant to the current study are summarised below.

7.2.1 Mid Richmond Flood Study (1998)

The Mid Richmond Flood Study (WBM Oceanics, 1998b) documented FFA on peak flood levels at Coraki, Woodburn and Broadwater. A peak level analysis (as opposed to peak flow) was undertaken due to the uncertainty in converting recorded flood levels to flow estimates at these gauges. Table 7.1 summarises the FFA derived peak flood levels at the locations assessed in the study.

Table 7.1 Peak FFA Flood Levels Estimates (m AHD) from Mid Richmond Flood Study

AEP	Coraki	Woodburn	Broadwater
1%	6.2	4.5	3.8
2%	6.1	4.2	3.4
5%	6.0	4.0	2.8
10%	5.9	3.7	2.3

There is considerable uncertainty when undertaking an FFA on peak flood levels and results should generally be treated with caution. As such, this technique has not been applied in the current study.

7.2.2 Casino Flood Study (1998)

The Casino Flood Study (WBM Oceanics, 1998a) undertook FFA on peak flows at Casino, Wiangaree and Toonumbar Dam with record lengths of 54, 26 and 28 years respectively. Various estimates of flow were derived utilising different skew parameter values. The results are summarised in Table 7.2 and are based on the skew estimate that most closely approximated the adopted result.

Table 7.2 Peak FFA Flow Estimates (m³/s) from Casino Flood Study

AEP	Casino*	Wiangaree**	Toonumbar***
1%	4,730	3,860	460
2%	3,860	3,210	412
5%	2,820	2,370	332
10%	2,120	1,760	258

*results shown for a skew of -0.3

** results shown for the reduced dataset with a skew of -0.7

*** results shown for a skew of -1.2

7.2.3 Kyogle Flood Study (2004)

The Kyogle Flood Study (WBM Oceanics, 2004), undertook at FFA at Kyogle. The study derived a rating curve based on a 2D model and used this to convert peak recorded levels to flow estimates. An extended period of record dating from 1928 to 2001 was used in the assessment. This contained some gaps and so the total record length applied was 53 years. Table 7.3 summarises the resulting peak flow estimates.

Table 7.3 Peak FFA Flow Estimates (m³/s) from Kyogle Flood Study

AEP	Kyogle
1%	2,280
2%	1,991
5%	1,611

7.2.4 Richmond Flood Mapping Study (2010)

FFA were not undertaken as part of the assessment and reference was made to the FFA previously undertaken for the Mid Richmond Flood Study (WBM Oceanics, 1998b).

7.2.5 Lismore Floodplain Risk Management Study (2019)

The Lismore Floodplain Risk Management Study (Engeny, 2020), undertook FFA at the following four gauges within the Wilsons River catchment: Lismore Rowing Club, Tuncester, Woodlawn College and East Gundurimba.

The FFA was undertaken on the annual maximum flood level series and so conversion of the historic levels to flow estimates using a rating curve was not undertaken. The results of the FFA are summarised in Table 7.4.

Table 7.4 Peak FFA Flood Levels Estimates (m AHD) from Lismore Floodplain Risk Management Study

AEP	Lismore Rowing Club	Woodlawn College	Tuncester	East Gundurimba
1%	12.2	12.6	15.0	10.5
5%	11.0	11.7	13.1	10.0
10%	10.2	11.0	11.8	9.5

7.3 Selection of Gauges for FFA

The following criteria were considered when identifying gauges at which to undertake FFA:

- A reasonably long and continuous record length of historic flood levels (considered in this study to approximately 40 years or more).
- Suitability of gauge location for establishment of a reliable rating curve.
- Proximity of gauge location to study area.

Table 7.5 summarises the gauges used in the assessment. The record length includes extensions of the record length for Casino and Kyogle based on datasets that preceded the installation of the current automatic gauge.

Table 7.5 Summary of FFA Gauge Selection

Gauge	Watercourse	Record Length (years)	Rating Curve	FFA Undertaken	Comment
Wiangaree	Richmond River	51	Yes (good)	Yes	Rating checked with modelling
Toonumbar	Iron Pot Creek	47	Yes (not assessed)	No	Small upstream catchment. Not significant for study
Doubtful	Eden Creek	21	Yes (not assessed)	No	Record length not suitable
Kyogle	Richmond River	95	Yes (good)	Yes	Rating checked with modelling
Casino	Richmond River	78	Yes (good)	Yes	Good location to assess flows into study area
Yorklea	Shannon Brook	43	Yes (poor)	No	Significant bypassing of gauge
Coraki	Richmond River	34*	No	No	Problematic for rating curve development due to backwater, bypassing and tide
Woodburn	Richmond River	36*	No	No	Problematic for rating curve development due to bypassing and tide
Broadwater	Richmond River	Non-continuous	No	No	No current automated gauge and patchy historic record.
East Gundurimba	Wilsons River	43	Yes (good)	Yes	Good location to assess flows into study area. Rating updated with modelling

Gauge	Watercourse	Record Length (years)	Rating Curve	FFA Undertaken	Comment
Rappville	Myrtle Creek	43	Yes (poor)	No	Problematic for rating curve development due to significant bypassing

***Both Coraki and Woodburn have extended records of peak flood levels based on manual recorded observations but these can not reliably be converted to flow estimates.**

Gauges on the lower Richmond river including Coraki, Woodburn and Broadwater do not lend themselves to development of reliable rating curves due to issues of backwater, bypassing and tidal influences. Furthermore, the development of floodplain levees at different stages in time will add to the uncertainty of converting historic levels to flows. For this reason, these gauges were not included in the assessment.

The four gauges at which an FFA has been undertaken are Casino, Kyogle, Wiangaree and East Gundurimba. These assessments are summarised below including any work undertaken on reviewing or developing rating curves.

7.4 Casino

7.4.1 Overview

The present gauge at Casino is located approximately 1.3km downstream of Irving Bridge. The gauge was moved to this location, from Irving Bridge, in 1970 and it has a complete record to the present day. Prior to 1970 flood levels at Casino were recorded at the pumping station gauge and Irving Bridge gauge, which were in close proximity. A relatively continuous record of peak flood levels exists for the Bridge/Pumping Station from 1945 to the late 1970's. Rating curves for both the bridge/pump station and the present gauge location have been developed and this has allowed the flood records to be combined to generate a near continuous annual peak flow record from 1945 to the present day.

7.4.2 Rating Curve Development

The location of the current Casino gauge (203004) is shown in Figure 6.1. The maximum level recorded at this gauge is 22.82mAHD during the February/March 2022 event. The maximum gauged level is 18.89mAHD which was gauged during the May 2009 flood event. A rating curve has been developed by Water NSW which extends to 21.51mAHD (a maximum flow of approximately 2,120m^3/s).

A localised hydraulic model of Casino was developed for this study and was used to simulate a series of steady state flows through Casino. This was done to both check and extend the Water NSW rating curve. The following was noted regarding flood behaviour for given flows:

- Flows of up to 2,000m^3/s are largely contained within bank through Casino.
- At a flow of 3,000m^3/s there is a notable back up of flow from the Richmond River into Black Gully on the southern bank of the Richmond River
- At a flow of 3,200m^3/s there is significant backwater within Black Gully and this begins to join with Oaky Creek and then rejoins the Richmond River downstream of Casino.
- At a flow of 3,500m^3/s, there is significant breakout flow to the south via Black Gully and this joins with Oaky Creek and Shannon Brook. There is also a break out of flow into Casino itself. This breakout occurs just downstream of the railway bridge and follows a notable depression in the terrain in a north-eastern direction before rejoining the Richmond River at the eastern end of Casino.

- At a flow of 4,000m^3/s there is widespread inundation across Casino.

The model has been used to develop two rating curves; one for the current gauge location and one for the former gauge at Irving Bridge. Both ratings relate levels at the respective gauge sites to total flow on the Richmond River entering Casino. This includes the breakout flow to the south via Black Gully which occurs for flows in excess of 3,000m^3/s.

The Water NSW rating curve at the gauge location is informed from gaugings up to a level of 18.89mAHD (flow of approximately 1,280m^3/s). As such, this rating is considered more reliable than that from the hydraulic model up to this elevation (which is within bank). For greater elevations, the Water NSW and modelled rating curves were broadly similar and so the Water NSW derived flow series has been adopted for use. This also accounts for changes in rating over time so is considered more appropriate for use.

The flood level at Casino for the February/March 2022 event exceeds the Water NSW rating curve and so the modelled rating curve for the Casino Gauge was used to derive the peak flow estimate.

A published existing rating curve was not available for the former gauge site at Irving Bridge. Therefore the rating curve developed using the hydraulic model has been used for the complete range of flows.

7.4.3 FFA

The FFA was undertaken using FLIKE software. Flows below a threshold of 200m^3/s were censored which removed 18 records. Both GEV and LP3 distributions were assessed with LP3 providing the better fit to data.

A sensitivity test was undertaken by incorporating the 1887 flood into the analysis. Whilst the level and therefore flow of this flood is not known, historical evidence points to it being one of the largest experienced in Casino it is likely comparable to the flood of 1954 and 2022 in terms of its flow. The 1887 flood preceded the construction of Toonumbar Dam which would have had a minor influence (reduction) on its flow if it was present at the time. This 1887 flood event has been incorporated as censored data assuming the flow was equal to, or larger, than that in the 1954 event which is considered a reasonable assumption for the sensitivity assessment.

The resulting peak flow estimates are summarised in Table 7.6 and shown as a flood frequency plot in Figure 7.1. When comparing the FFA estimates with those previously derived for Casino (also shown in Table 7.6), the new estimates are notably lower. The updated assessment is based on a longer record length and improved techniques. As such it is considered to be an improved estimate.

Table 7.6 FFA Results at Casino (Flows in m^3/s)

AEP	FFA	FFA with 1887 event	Previous FFA (1998)
1%	3,980	4,390	4,730
2%	3,320	3,600	3,860
5%	2,460	2,610	2,820
10%	1,840	1,920	2,120

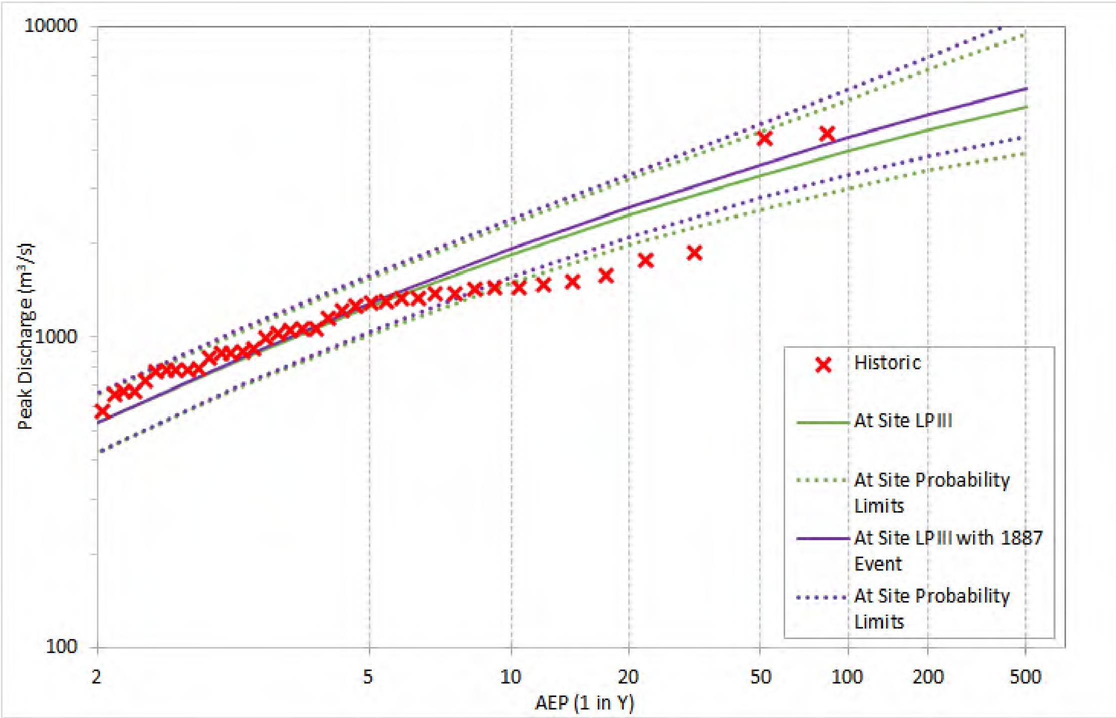


Figure 7.1 Flood Frequency Plot for Casino

7.5 Wiangaree

The Wiangaree gauge (203005) is a WaterNSW gauge located approximately 60km upstream of Casino and is the most upstream gauge on the Richmond River for which an FFA has been undertaken in this study. A local TUFLOW hydraulic model was developed and used to check the rating curve available from WaterNSW. It was found that the Water NSW rating curve was in general agreement with the hydraulic model and no refinement to the rating curve was required.

The largest flood on the current gauge record is the January 2008 flood and the largest gauged flood was in January 2013 with a gauged flow of approximately 770m^3/s.

An FFA has been undertaken in FLIKE software using the 51 years of available record. It is understood that the gauge at Wiangaree was moved to its present location in 1970. Older flood records are available prior to this (dating back to 1944) but the Casino Flood Study considered these data to be inconsistent or unreliable and so they have not been incorporated into this updated assessment. The multiple Grubbs Beck test was used to censor low flows with three records being censored in this way.

The FFA has fitted the data to the LP3 probability model using Bayesian inference techniques.

The resulting peak flow estimates are summarised in Table 7.7 and shown as a flood frequency plot in Figure 7.2. Table 7.7 also includes the peak flow estimates derived from the FFA undertaken for Wiangaree in the 1998 Casino Flood Study. Similar to the finding at the Casino gauge, the revised Wiangaree peak flows are notably lower than the previously derived flows. The updated assessment is based on a longer record length and improved techniques. As such it is considered to be an improved estimate.

Table 7.7 FFA at Wiangaree (Flows in m³/s)

AEP	FFA	Previous FFA (1998)
1%	2,440	3,860
2%	2,140	3,210
5%	1,740	2,370
10%	1,400	1,760

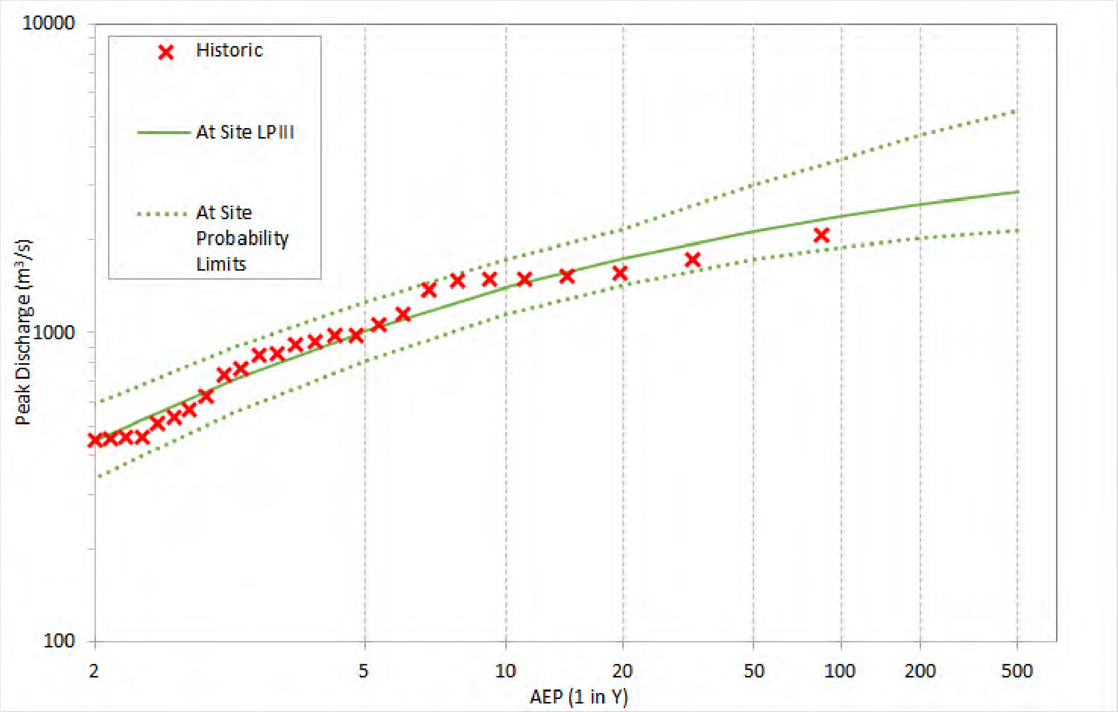


Figure 7.2 Flood Frequency Plot for Wiangaree

7.6 Kyogle

The gauge at Kyogle (203900) is a Water NSW gauge located approximately 50km upstream of Casino. The current gauge has a record extending back to 1985 but an extended record is available that dates back to 1928. The Kyogle Flood Study (WBM Oceanics, 2004) derived a rating curve for the gauge based on a hydraulic model which was used to derive a flow record for the majority of years between 1928 and 2001. This flow record has been extended to the present day (2022) using the same rating curve.

The FFA was undertaken on 63 years of data with a further 32 values censored for being below a nominated low flow threshold of 200m³/s. Missing years in the older part of the record were assumed to be years with no floods of note and so have been incorporated into the censored low flows.

The FFA has fitted the data to the LP3 probability model using Bayesian inference techniques.

The resulting peak flow estimates are summarised in Table 7.8 and shown as a flood frequency plot in Figure 7.3. Table 7.8 also includes the peak flow estimates derived from the FFA undertaken in the

2004 Kyogle Flood Study. The updated flow estimates are slightly higher than the previous estimates which is in contrast to the upstream gauge at Wiangaree and downstream gauge at Casino where the updated flows were significantly lower. The Kyogle peak flow estimates lie between the peak flow values at Wiangaree and Casino which is expected given that Kyogle is located between these two locations on the Richmond River. This provides confidence in the consistency of the estimates at these three locations.

Table 7.8 FFA at Kyogle (Flows in m^3/s)

AEP	FFA	Previous FFA (2004)
1%	2,610	2,280
2%	2,350	1,990
5%	1,940	1,610
10%	1,550	1,320

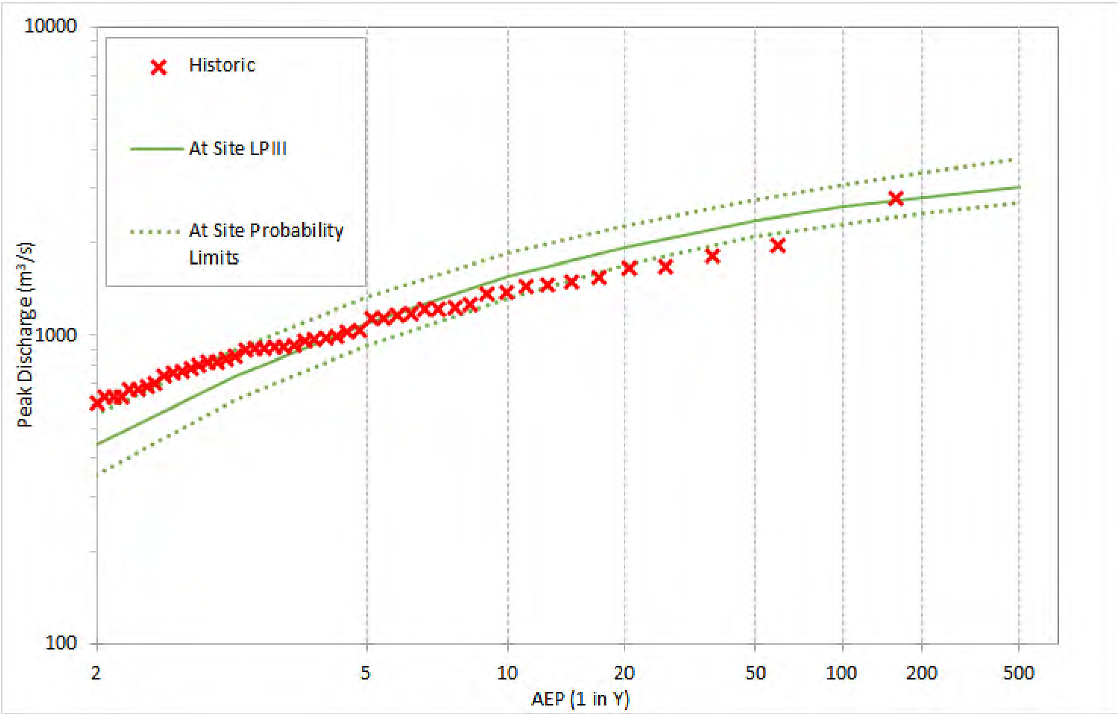


Figure 7.3 Flood Frequency Plot for Kyogle

7.7 East Gundurimba

The gauge at East Gundurimba is a Manly Hydraulics Lab (MHL) gauge with a 42 year continuous record dating back to 1981. The largest flood within this record by some margin is the flood of February/March 2022. The peak flow of the 2022 event at East Gundurimba was determined to be over 9,000m^3/s, which is over 3x the magnitude of the previous largest flood recorded at the gauge in 2017 (2,700m^3/s). The floods of 1954 and 1974 are not included within the gauged record but are known to be larger than the 2017 flood at Lismore, a short distance upstream from East Gundurimba.

The 1954 and 1974 floods have been incorporated into the assessment with their flows being defined as being larger than the peak flow of the March/April 2017 event. Whilst this approach has a number of limitations it is considered an improvement over not including these notable floods.

7.7.1 Rating Curve Development

A rating curve was available from previous modelling and this was assessed against a localised hydraulic model developed for the current study using calibrated Manning's n values. Notable differences between the rating curves were found as shown in Figure 7.4, with the existing rating resulting in a lower flow estimate for a given level. The flow-level relationship was noted to be subject to some degree of hysteresis during model calibration for flows up to around 1,500m^3/s which may explain the differences in the two ratings up to that flow. The modelled rating was adopted for use as it is considered to provide superior flow estimates for larger events, i.e. those typically of interest for FFA.

The East Gundurimba gauge failed during the February/March 2022 event before the peak level was reached. A nearby debris mark indicated that the flood level at the gauge was 12.95mAHD. Using the modelled rating curve, this level corresponds to a peak flow estimate of 9,370m^3/s.

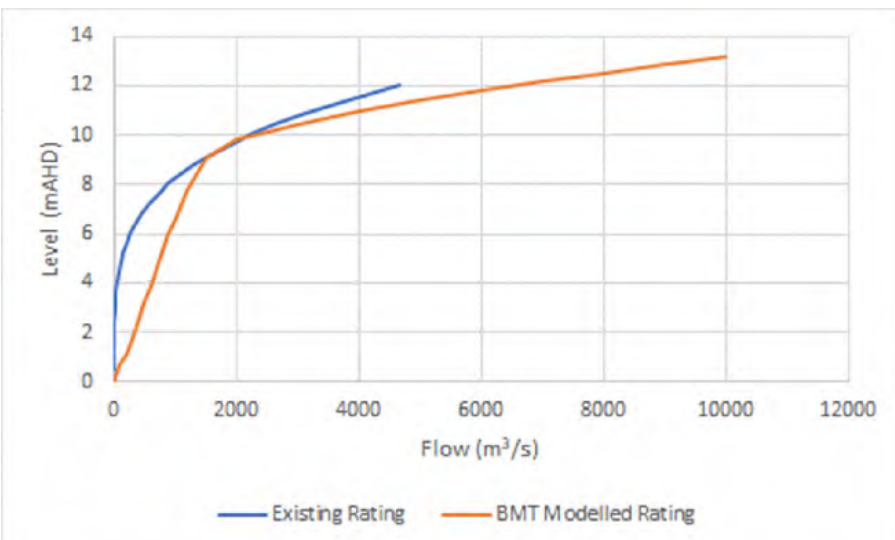


Figure 7.4 East Gundurimba Rating Curves

7.7.2 FFA

When incorporating the extraordinarily large flood of February/March 2022 into the 42 year record, the plotting position of the February/March 2022 event is understated in the analysis i.e. the plotting position indicates the event is more frequent than it likely is. Similarly, this large flow occurring within the relatively short period of record has the effect of biasing the fit at the rare end of the probability curve.

Based on an analysis of rainfall data across the Wilsons catchment (BMT, 2022), the AEP of the rainfall for the February/March 2022 event is typically greater than a 1 in 1,000 AEP and exceeds a 1 in 2,000 AEP at a number of gauges. This suggests that actual AEP of the flood event at East Gundurimba is significantly rarer than that indicated by the plotting position and will bias the fitting of the probability distribution to the flow record.

To overcome this limitation of inserting a significant flow into a relatively short period of record an attempt has been made to consider the February/March 2022 event within a longer historical context. To do this, the 2022 event has been removed from the annual series of peak flows and has instead been incorporated as a threshold value which is only exceeded once in a nominated period. This essentially treats the peak flow as a flood of record over this nominated period. Two nominated periods were considered based on the rainfall analysis; a 1,000 year period and a 2,000 year period.

In addition to the February/March 2022 event, the historical events of 1954 and 1974 have been incorporated into the analysis as threshold values. These floods occurred before the East Gundurimba gauge was installed but they were both larger than the 2017 event based on the recorded historic levels in Lismore.

The Grubbs-Beck test for low flows was also applied and resulted in the removal of 8 low flow events which were replaced by applying them as a threshold value.

Table 7.9 summarises the results of the assessment for the 1% AEP. The table includes estimates for which the February/March 2022 event is included within the record as well as estimates where it is treated as a threshold value. Both the Log Pearson 3 (LP3) and Generalised Extreme Value (GEV) distributions were applied.

Table 7.9 1% AEP Peak Flow Estimates (m^3/s) using Different FFA assumptions/distributions

Direct Inclusion of 2022 Event		2022 Threshold (1,000 years)		2022 Threshold (2,000 years)	
LP3	GEV	LP3	GEV	LP3	GEV
5,920	5,830	4,650	4,530	4,380	4,260

7.7.3 Adopted East Gundurimba Peak Flow Estimates

The FFA which applied the February/March 2022 event as a threshold value over a 1,000 year period has adopted for use. Both the GEV and LP3 distributions resulted in a similar magnitude 1% AEP peak flow.

It is recognised that the technique contains a significant amount of uncertainty but is considered an improvement over use of applying the 2022 event directly within the relatively short period of record and which would likely overstate the flows.

Table 7.10 summarises the resulting adopted FFA design flows at East Gundurimba derived by FFA for various AEPs. Figure 7.5 presents the flows in a flood frequency plot.

Table 7.10 FFA at East Gundurimba (Adopted LP3 Flows)

AEP	FFA Peak Flow (m^3/s)
1%	4,650
2%	3,800
5%	2,820
10%	2,180

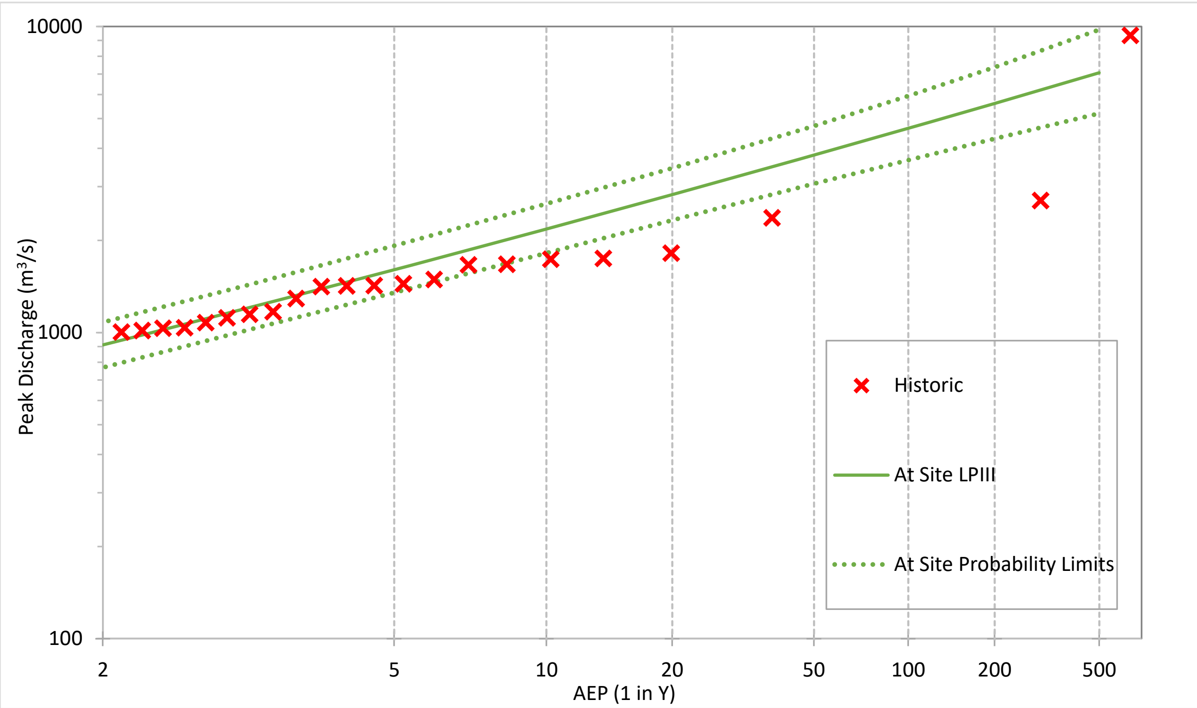


Figure 7.5 Flood Frequency Plot for east Gundurimba

No previous FFA results were available to compare against as the recently completed Lismore Floodplain Risk Management Study undertook the FFA on peak flood levels.

8 Design Flood Simulations

8.1 Introduction

The calibrated hydrologic and hydraulic models are used to simulate design floods for the following AEPs: 5%, 2%, 1% and 0.2% with and without an allowance for climate change. In addition, the Probable Maximum Flood (PMF)[7] is simulated. The design hydrology uses ARR2019 techniques which represents a notable change from the previous ARR1987 guideline which underpinned prior flood studies on the Richmond River.

A number of key inputs used in design flood simulations are obtained from the ARR Data Hub[8]. Where single, location specific inputs were required, a mid catchment location was used to select these inputs. This was determined on the basis of preliminary sensitivity testing of areal temporal pattern sets and areal reduction factors for a mid catchment location (Casino) and a lower catchment location (Broadwater).

The critical duration and critical event selection process undertaken in this study has used hydraulic modelling for event selection purposes. This contains a number of advantages over hydrologic modelling for complex floodplains like the Richmond River. The peak flows from the selected events were then reconciled with the peak flow estimates derived from the flood frequency analyses (FFA).

The remainder of this section documents the adopted design inputs, event selection process, reconciliation to FFA and results.

8.2 Design Inputs

8.2.1 Rainfall Depth

Intensity-Frequency-Duration (IFD) relationships are used to determine the design rainfall depths for a given storm duration and AEP. Design rainfall depths are available from the Bureau of Meteorology (BoM) and have been obtained for each URBS sub-area centroid from the 2016 IFD dataset. The rainfall depths have been obtained for all required AEPs and for durations ranging from 12 hours to 96 hours which was sufficient to encompass the range of durations critical across the study area for the main rivers and creeks.

Rainfall depths for the Probable Maximum Precipitation (PMP) were based on the Bureau of Meteorology's Generalised Tropical Storm Method Revised (GTSMR) methodology (Walland et al., 2003).

The raw PMP rainfall depths for the 24, 36, 48, 72, 96 and 120 hour durations were obtained from the depth-area curve/table for the GTSMR coastal zone and summer season. These raw depths were then modified by catchment adjustment factors. The modification factors were defined as follows:

- Moisture Adjustment Factor (MAF) - The MAF is the ratio of the extreme precipitable water of the catchment (EPWcatchment) to the standard extreme precipitable water (EPWstandard), which is 120 mm. That is, MAF = EPWcatchment/120mm. A catchment average value of EPWcatchment was extracted from the published EPW grid for this calculation.
- Decay Amplitude Factor (DAF) - The catchment average DAF was obtained from the published grid of DAF values.

[7] For this study the Probable Maximum Flood is the flood derived from the Probable Maximum Precipitation
[8] https://data.arr-software.org/

- Topographic Adjustment Factor (TAF) - The catchment average TAF was obtained from the published grid of TAF values.

The calculated catchment adjustment factors are presented in Table 8.1.

Table 8.1 GTSMR PMP Adjustment Factors

PMP Factors	Value
EPWcatchment	85.02
MAF	0.71
DAF	0.93
TAF	1.28

Table 8.2 provides an example of the design rainfall depths at Casino for relevant durations. It should be noted that rainfall depths applied in the modelling will vary across the catchment. The whole of catchment adjusted PMP rainfall depths are also included. The non-PMP rainfall depths shown in Table 8.2 exclude any adjustment for areal reduction factors or rainfall losses.

Table 8.2 IFD Design Rainfall Depths (mm) at Casino

AEP (%)	24 hour	48 hour	72 hour
5%	198	258	292
2%	238	307	344
1%	270	344	383
0.2%	328	430	474
PMP (adjusted)	840	1120	1360

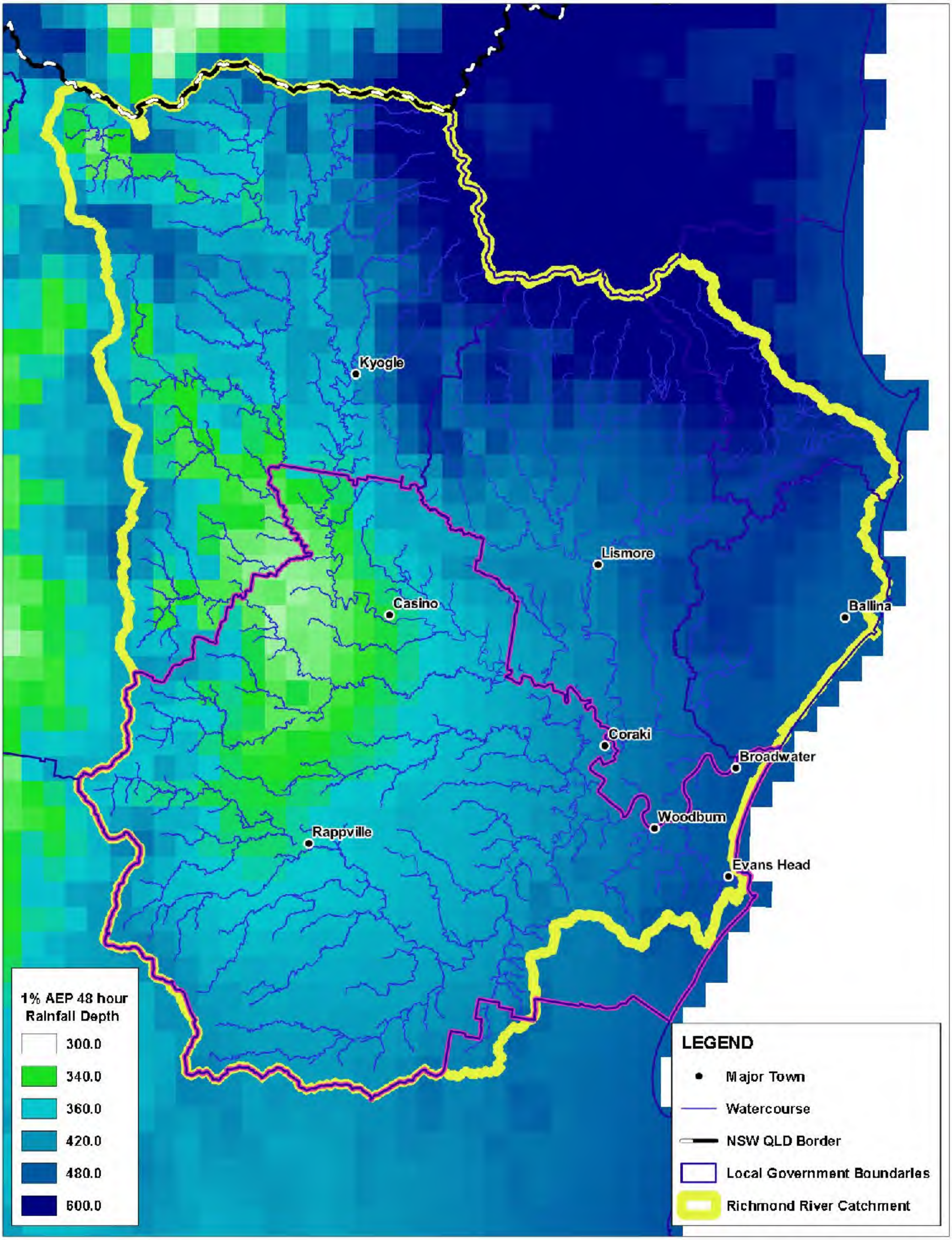
Lat -28.8688, Long 153.0457

Design rainfall depths derived from the IFD relationships were compared against depths derived from at site gauge analyses for 15 rainfall stations located across the Richmond River catchment. The analysis and supporting plots are included in Annex D. Overall it was concluded that the IFDs generally compared well against the at-site analyses although the significant event of February/March 2022 may result in higher at-site estimates at a number of stations.

8.2.2 Rainfall Spatial Pattern

Rainfall depths using published IFDs were extracted for each required AEP/duration combination at the centroids of each URBS sub-area. The rainfall depths vary across the catchment and show a significant range in depths. An example of the spatial distribution of design rainfall depths is shown in Figure 8.1 for the 1% AEP, and the 48 hour duration storm. The greatest depths are shown for the north-eastern parts of the catchment along the catchment divide with the Tweed Basin. Here, orographic effects and proximity to the coast lead to higher rainfall depths. The lowest depths are across an area west of Casino. For other AEPs and durations, the rainfall depths differ but the general spatial pattern of rainfall depths is consistent.

For the PMP, the adjusted rainfall was spatially distributed to each sub-area using a ratio of the average sub-area TAF to the catchment TAF as outlined in Section 3 of GTSMR guideline.



Title:

IFD Rainfall Depth – 1% AEP, 48 hour

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8.2.3 Rainfall Temporal Patterns

ARR2019 sets out an ensemble approach to design hydrology whereby, for each storm duration of a given AEP, an ensemble of 10 rainfall temporal patterns is simulated. A temporal pattern from each ensemble is then selected which results in a representative average value for the output of interest (typically peak flow, level or volume).

In accordance with ARR2019, this study uses the areal temporal patterns as the catchment areas of interest are greater than $75km^2$.

Different areal temporal pattern sets are available from ARR2019 and the selection of the appropriate set/s depends on the catchment area of interest. As discussed in Section 8.1, preliminary investigations concluded that a mid-catchment location would be appropriate for use in this study. The adopted temporal sets were selected from the East Coast South region and correspond to the areal category of $1,600km^2$ to $3,500km^2$. The areal temporal sets vary with duration but not with AEP. Each temporal pattern set comprises 10 ensemble patterns and cover a mix of front, mid and rear loaded storms.

The temporal rainfall pattern ensemble applied to the PMP were the AVM patterns available from the GTSMR guideline based on the $5,000km^2$ standard area category.

For modelling purposes, URBS labels the 10 temporal patterns within an ensemble as E0 to E9 and this labelling has been retained in the hydraulic modelling when referencing temporal patterns.

8.2.4 Areal Reduction Factors

Areal reduction factors (ARF) are a ratio between the catchment average rainfall depth and point specific (gauge location) rainfall depth for the same AEP/duration. The reduction in rainfall depth with the ARF is based on larger catchments being less likely to experience high intensity storms simultaneously over the whole of the catchment area.

ARF values are derived from regionalised parameters available from the ARR Data Hub for the East Coast North region. An ARF value of 1 means no reduction in rainfall. For the Richmond River catchment, the ARF will vary depending on the location under consideration. Table 8.3 compares the ARFs for a mid-catchment location (Casino) and a whole of catchment location (Broadwater) for different storm durations for the 1% AEP event.

Table 8.3 Variation in ARF between Casino and Broadwater

Location	Upstream Area (km^2)	24 hour	48 hour	72 hour
Casino	1800	0.87	0.90	0.92
Broadwater	6600	0.81	0.85	0.87

An ARF representative of a mid-catchment location was selected for use in the study. These ARF values are similar to those shown in Table 8.3 for Casino and were selected as a compromise between higher ARFs applicable to smaller catchments and lower ARFs applicable to larger catchments, noting that results are also reconciled to FFA and so selection of the ARF was of reduced importance.

8.2.5 Design Rainfall Losses

Design rainfall losses for a given simulation are specified as an initial loss (IL) and a continuing loss (CL). A design CL of 2mm/hr was adopted for all events except the PMF as this was a typical loss value determined through model calibration. Use of this value also meant the resulting design flows compare well to the flood frequency estimates. For the PMF, a continuing loss of 1mm/hr was applied in accordance with the ARR2019 guideline.

For the initial loss, ARR2019 makes a distinction between the initial loss for a complete storm (such as those modelled in calibration) (IL_S) and the initial loss for a design rainfall burst such as those used when applying IFD data in design flood modelling (IL_B). For a complete storm, the initial loss is the depth of rainfall prior to the commencement of surface runoff whereas for the burst it is the portion of the storm initial loss that occurs within the burst. This is illustrated in Figure 8.2 below which is replicated from Book 5, Chapter 3 of ARR2019. Burst loss values are therefore smaller than complete storm loss values.

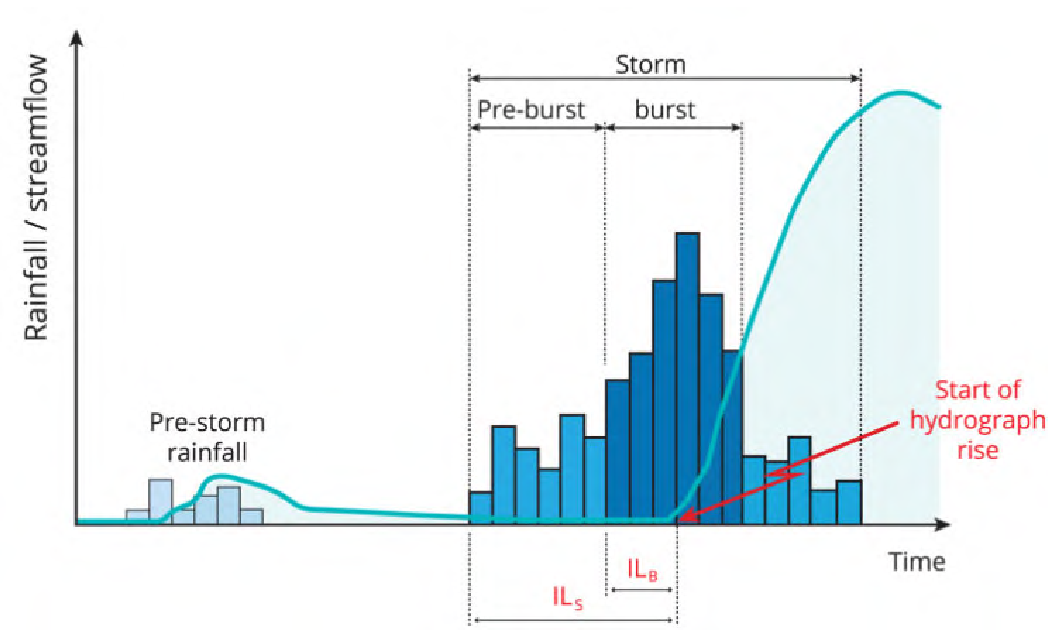


Figure 8.2 Distinction between Storm and Burst Initial Loss (from ARR2016 Book 5, Chapter 3)

For NSW, ARR2019 includes specific guidance when deriving IL_B which recommends the use of probability neutral initial loss in five hierarchal approaches (WMA Water, 2019). This study has used the preferred approach (Approach 1) where the IL_S was informed from model calibration and IL_B was calculated using by multiplying the IL_S by the ratio of the ARR Datahub IL_B and IL_S values.

To achieve this, the probability neutral losses were extracted from the ARR Data Hub at a central point within each of the four URBS modelled regions. The average probability neutral loss was then derived and divided by the ARR Datahub IL_S value to obtain the ratio.

The applied initial losses (IL_B) are summarised in Table 8.4 and rounded to the nearest 5mm. These were applied across the catchment.

Table 8.4 Rainfall Initial Losses (IL_B) Applied for Design

AEP	24 hour	36 hour	48 hour	72 hour
5%	15	20	25	35
2%	15	20	25	25
1%	5	5	10	10
0.2%	0	0	0	0
PMF	0	0	0	0

8.2.6 Baseflow

Variable baseflow is applied within the hydrologic model in accordance with the URBS baseflow model which is defined by two baseflow parameters[9] BR and BC. BR is the daily recession constant and a value of 0.9 is adopted as determined through the model calibration process. The BC is a baseflow constant proportion (a proportion of the quick runoff). Its value has been inferred from the ARR2019 Base Flow Peak Factor which is approximately 0.05 across much of the Richmond River catchment.

The simplified baseflow model equation is:

$$Qb_i = BR.Qb_{i-1} + BC.Qr$$

where Qb_i and Qb_{i-1} are the base flow values for time step i and time step i-1 respectively and Qr is the quick runoff.

Overall the baseflow has a relatively minor contribution to the peak flow but provides an improved fit on the recession limb of the flood event.

When modelling the PMF event, baseflow has been disabled as baseflow would only be expected to make a minor contribution to extreme floods.

8.2.7 Dam Initial Water Levels

Design event modelling assumes that Toonumbar Dam is at its full supply level of 129.62mAHD at the start of the simulation.

8.2.8 Routing Parameters and Manning's n values

Hydrologic routing parameters and the hydraulic Manning's n values have been retained from those used in model calibration.

8.2.9 Structure Blockage Considerations

Blockage of hydraulic structures for design events has been based on the guidelines in Book 6, Chapter 6 of ARR2019. This results in a 'design blockage' which is then applied to structures.

The hydraulic model of the Richmond River is a regional scale model with limited representation of smaller structures such as culverts. The main type of hydraulic structure represented are bridges. Consideration of blockage has therefore been applied only to bridges included in the model.

[9] A third exponent parameter 'BM' can be specified but has been set to the default value of 1.

The catchment wide debris potential was assessed as being ‘medium’ which is adjusted to ‘high’ for the 0.2% AEP and PMF events.

A generalised assumption has been made that the clear width of bridge spans lies between 1x and 3x the average length of the longest 10% of debris reaching the site (the L_{10} parameter).

This results in a design blockage factor applied to the bridge sub-structures as follows:

- 5%, 2% and 1% AEPs – design blockage factor 10% blocked
- 0.2% AEP and PMF – design blockage factor 20% blocked.

A sensitivity scenario of an ‘all clear’ scenario is also undertaken in which no additional design blockage is applied (see Section 9.2).

8.3 Selection of Downstream Boundary Condition

A study of elevated ocean water levels was carried out for the Richmond River entrance in 1994 (Lawson & Treloar, 1994). The study considered the probability of elevated ocean water levels due to low pressure systems (i.e. from cyclones and east coast lows) and wave forces. Extended investigations of that study in 1995 produced a set of water level hydrographs over the duration of a flood event for various exceedance probabilities. The peaks of those water level hydrographs are summarised in Table 8.5 and were applied in both the 2010 Flood Study and the 2014 Evans River Flood Study. Both of these studies assumed that the peak of the storm surge coincided with the peak of the event rainfall. As such, the storm surge passed through the tidal reaches of the model before the catchment runoff peak arrived.

Table 8.5 Peak Ocean Water Levels used in Currently Adopted Flood Studies

AEP	Peak Ocean Level (mAHD)
5%	1.8
2%	1.9
1%	2.0
0.2%	2.1
PMF	2.2

In 2015, the NSW Office of Environment and Heritage (now DPIE), released a floodplain risk management guide titled ‘Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways’ (Office of Environment and Heritage (OEH), 2015). The guide provides advice on approaches that can be used to derive ocean boundary conditions for flood investigations in coastal waterways including design ocean levels and the relative timing of catchment flooding and oceanic events.

Under this guide the Richmond River estuary falls under ‘Group 3 Wave dominated estuaries’. The guide then provides a further categorisation into ‘simplified waterway entrance types’ of which it is considered that the Richmond at Ballina and Evans River at Evans Head are classed as Type B entrances (see Annex E).

The guide provides dynamic ocean boundaries which have a peak ocean water level of 2.1mAHD for a 1% AEP and 2.0mAHD for a 5% AEP. The guide recommends the combinations of catchment flood and ocean water level boundary shown in Table 8.6. The peaks of the ocean water level boundaries are also given in Table 8.6 and should coincide with the peak of the catchment flood.

Table 8.6 Combinations of Catchment Flooding and Oceanic Inundation Scenarios

Design AEP for peak levels/velocities	Catchment Flood Scenario AEP	Ocean Water Boundary	
		Scenario AEP	Peak Level (mAHD)
5%	5%	HHWS(SS)*	1.13
2%	2%	5%	2.0
1% Envelope level	5%	1%	2.1
1% Envelope level	1%	5%	2.0
0.2%	0.2%	1%	2.1
PMF	PMF	1%	2.1

*High High Water Springs (Solstice Spring)

For the 1% AEP catchment flood, the ocean water levels recommended in the NSW guideline are similar to those used previously. i.e. an ocean water level boundary that peaks at 2.0mAHD. The main difference in approach compared to the 2010 Flood Study is with regard to the assumptions on timings with the guideline recommending that the respective peaks coincide.

For this study the combinations of catchment flooding and oceanic inundation as stated in the NSW guideline (and replicated in Table 8.6) have been applied. It has been assumed that the catchment runoff and ocean peaks coincide at Woodburn and the timing of the ocean peak has been adjusted to achieve this. Preliminary modelling showed that the enveloping of the two 1% AEP component events was not required as the 1% AEP catchment flood (with a 5% AEP ocean level) dominated the study area with the other 1% AEP component only being dominant at Ballina.

Section 9.4 documents a sensitivity test where the timing of the storm tide peak has been offset against that of the catchment flood peak.

8.4 Event Selection Methodology

Under ARR2019 guidelines, ensembles of ten temporal rainfall patterns are simulated for each duration/AEP combination. For each ensemble and at a given location, an event using a temporal pattern which is considered to give a representative (average) result is then selected.

Peak flow is typically the measure used to determine the average pattern as this is a direct output from hydrologic models which can simulate ensembles very rapidly compared to hydraulic models. The assumption is that the representative 'average' flow results in representative 'average' flood level.

For the Richmond catchment with its complex floodplain interactions which include bypassing of a number of key gauge locations, it is considered that selecting representative temporal patterns based on flow is not a suitable approach for much of the lower floodplain.

This study has therefore taken advantage of some key features in the TUFLOW software and uses the hydraulic model for event selection purposes. Ensembles are simulated in a coarser resolution hydraulic model with a significantly faster run time than the full resolution model. The events with representative temporal patterns for each duration/AEP are selected based on flood level.

The event selection hydraulic model was modified to use a coarser 80m grid across the model domain. This has also required the removal of a limited number of 1D modelled hydraulic structures. As SGS is enabled in the model, broadly similar flow hydrographs result at key gauge locations between the 80m model and the full resolution model (see Annex F). Whilst the resulting flood levels will differ between the coarse and full resolution models, the ability of the models to rank events in order of peak level magnitude remains largely consistent.

The validity of the approach was verified by simulating an ensemble of 10 temporal patterns for the 1% AEP, 24 hour duration event for both the coarse and full resolution models. The relative rankings of the ensemble members in terms of peak level were then compared for each model. The 24 hour duration is critical at Casino and Yorklea in the 1% AEP event. Table 8.7 and Table 8.8 present the resulting ensemble rankings for Casino and Yorklea respectively.

For Casino the rankings are the same in both the full resolution model and the coarse resolution model. For Yorklea the rankings show some minor variations between the two models. However the 6th ranking event for the full resolution model (event E5) has a flood level only 0.01m different from the 6th ranking event from the coarse resolution model.

As a general comment, the range in peak levels across all 10 ensemble events is relatively small (0.33m for Casino and 0.14m for Yorklea) meaning that the modelled flood levels at these locations are not particularly sensitive to the selected event.

Table 8.7 Ensemble Ranking: Casino Gauge (Richmond River) 1% AEP, 24h

Ensemble ID	Rank (Detailed Model)	Rank (80m model)	Elevation (Detailed Model) (m AHD)
E0	3	3	22.44
E1	2	2	22.41
E2	9	9	22.56
E3	10	10	22.62
E4	4	4	22.46
E5	8	8	22.52
E6	1	1	22.28
E7	6	6	22.50
E8	7	7	22.51
E9	5	5	22.48

Table 8.8 Ensemble Ranking: Yorklea Gauge (Shannon Brook) 1% AEP, 24h

Ensemble ID	Rank (Detailed Model)	Rank (80m model)	Elevation (Detailed Model) (m AHD)
E0	7	6	23.79
E1	2	2	23.74
E2	8	9	23.80
E3	10	10	23.85
E4	4	3	23.76
E5	6	7	23.78
E6	1	1	23.71
E7	5	5	23.76
E8	9	8	23.80
E9	3	4	23.74

This check provides confidence that use of the coarse resolution model for event selection is a valid and practical approach given the greater simulation times of the full resolution model.

Overall, the following process has been followed for each AEP under consideration:

1. The coarse resolution model was used to simulate ensembles of ten temporal patterns for 7 storm durations (12, 18, 24, 36, 48, 72, 96 hours) giving 70 simulations per AEP. A fixed ocean water level was assumed.
2. For each ensemble of a given duration/AEP a mean peak water surface grid was produced based on an analysis of levels in each model grid cell.
3. The mean peak water surface grids were compared and a critical storm duration map was produced identifying the storm durations which produced the maximum flood levels ('max of the means').
4. A subset of simulations (typically 2 or 3) was selected to be representative of the overall 'max of means' flood surface. The component simulations which comprise the subset were typically selected from different storm durations shown to be dominant in the critical duration mapping.
5. A maximum peak water surface grid was created from the subset of results grids.
6. The maximum peak water surface grid was compared back against the max of the means grid to ensure there were no significant differences in peak flood level. A tolerance of 0.1m was applied. If there were extensive differences greater than 0.1m then the subset was revised, incorporating additional component simulations if required.

The resulting subsets of simulations were then simulated in the full resolution hydraulic model and results were compared and reconciled to the FFA estimates (see Section 8.6).

This approach of using a coarse resolution hydraulic model to rank component ensemble events based on peak flood level is considered superior to use peak flows from the hydrologic model. It negates the need to select hydrologic focus locations at which to analyse events at discrete points. Instead, the whole model domain can be analysed and appropriate subsets of events selected based on these analyses. However, the hydrologic model is still required to generate inflows which are applied in the hydraulic model.

8.5 Event Selection Results

8.5.1 1% AEP

Event selection was based on a visual inspection of the critical temporal patterns for each duration across the area where that duration is critical. Figure 8.3 summarises the findings graphically by showing dominant critical durations and dominant temporal patterns within those durations across the model domain. Figure 8.4 maps the critical durations. It can be seen that three durations dominate; the 24, 48 and 72 hour durations. The temporal patterns which dominate each of these three respective durations are E7 for 24 hours, E6 for 48 hours and E2 for 72 hours.

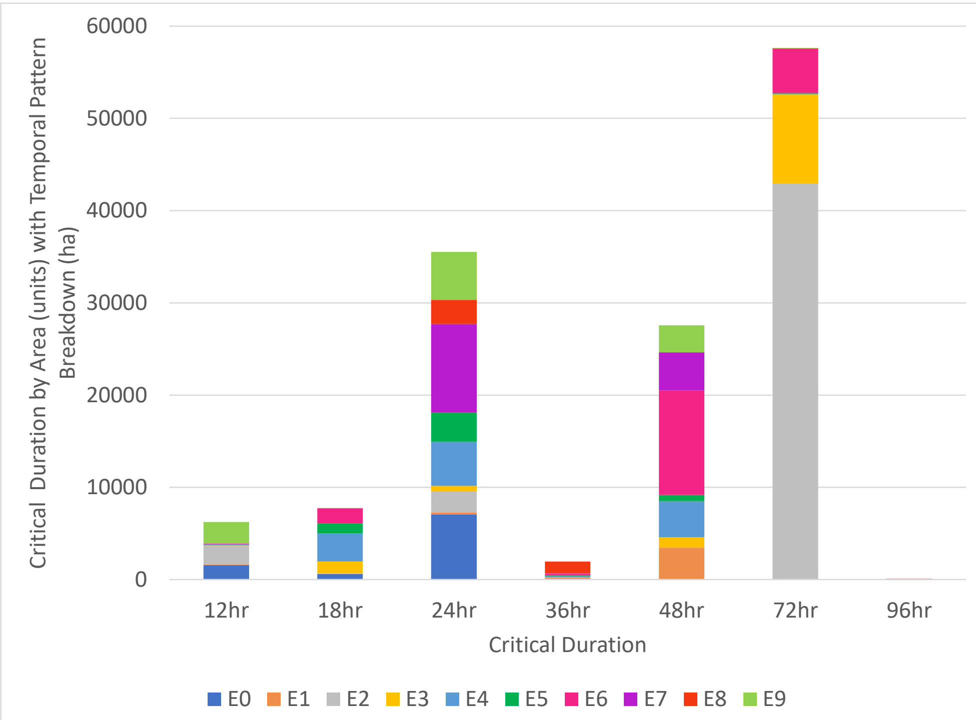


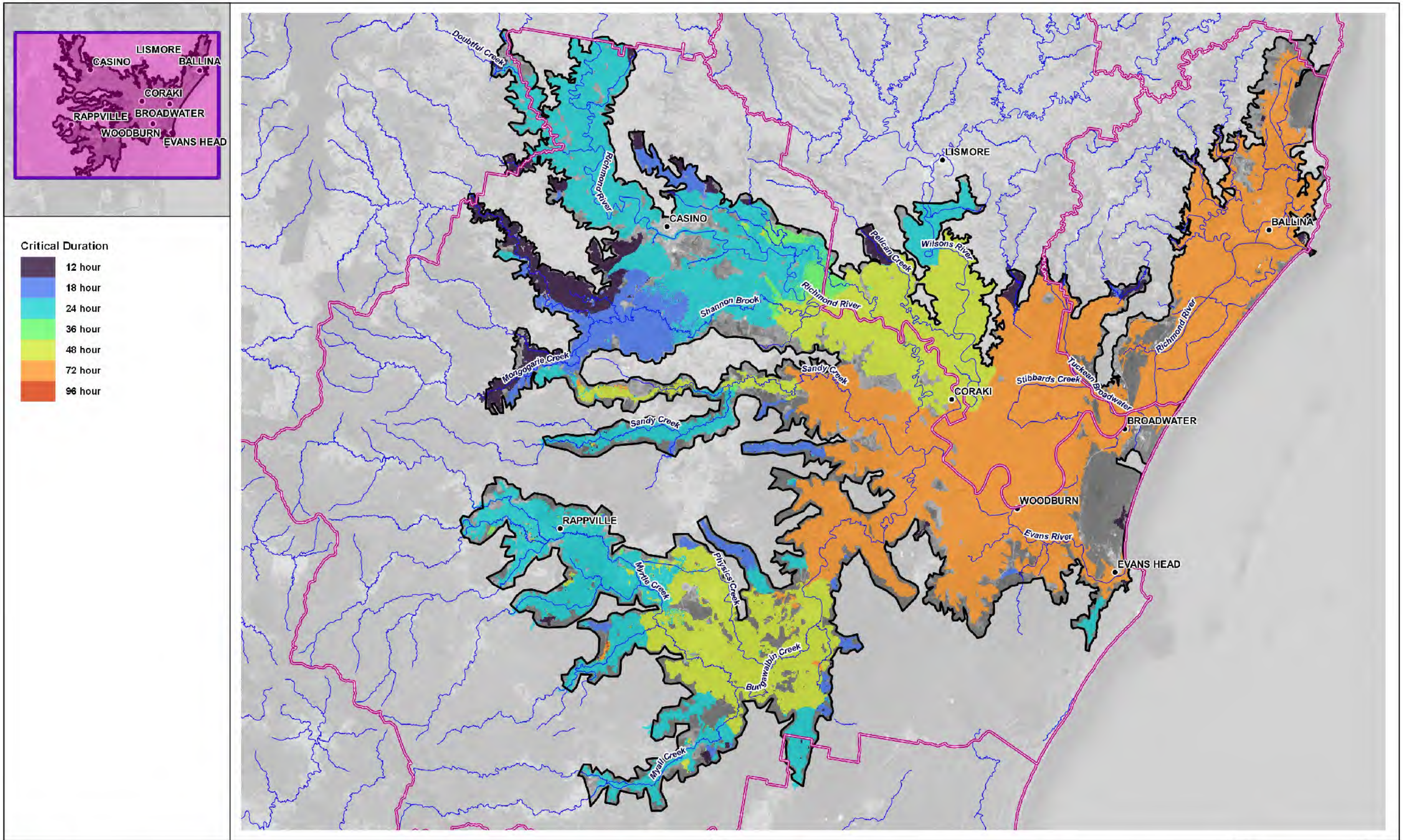
Figure 8.3 Graphical Representation of Event Selection: 1% AEP

Table 8.9 summarises the durations and patterns selected. A critical duration map using the subset of the three selected events is included in the Design Results Mapping Addendum.

Table 8.9 Selected Events: 1% AEP

Duration (h)	Selected Ensemble Event	ARR Event TP ID*
24	E7	228
48	E6	407
72	E2	493

*East Coast (South) Region



Legend

LGA Boundaries • Towns

Model Extent — Watercourses

Title:

1% AEP - Critical Duration for Event Selection

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8.5.2 5% AEP

Figure 8.5 graphically summarises results for the 5% AEP in the same format as for the 1% AEP. Figure 8.6 maps the critical durations. The 24, 48 and 72 hour critical durations dominate the model domain. Both the 24 hour and 48 hour durations have no clear dominant temporal pattern. Selection of patterns E0 for the 24 hour and E9 for the 48 hour durations provided the best overall outcome in terms of being within the peak level tolerance of 0.1m. For the 72 hour duration, initially pattern E7 was selected but this was replaced with pattern E5 after examining mapping tolerances. Table 8.10 summarises the durations and patterns selected. A critical duration map using the subset of the three selected events is included in the Design Results Mapping Addendum.

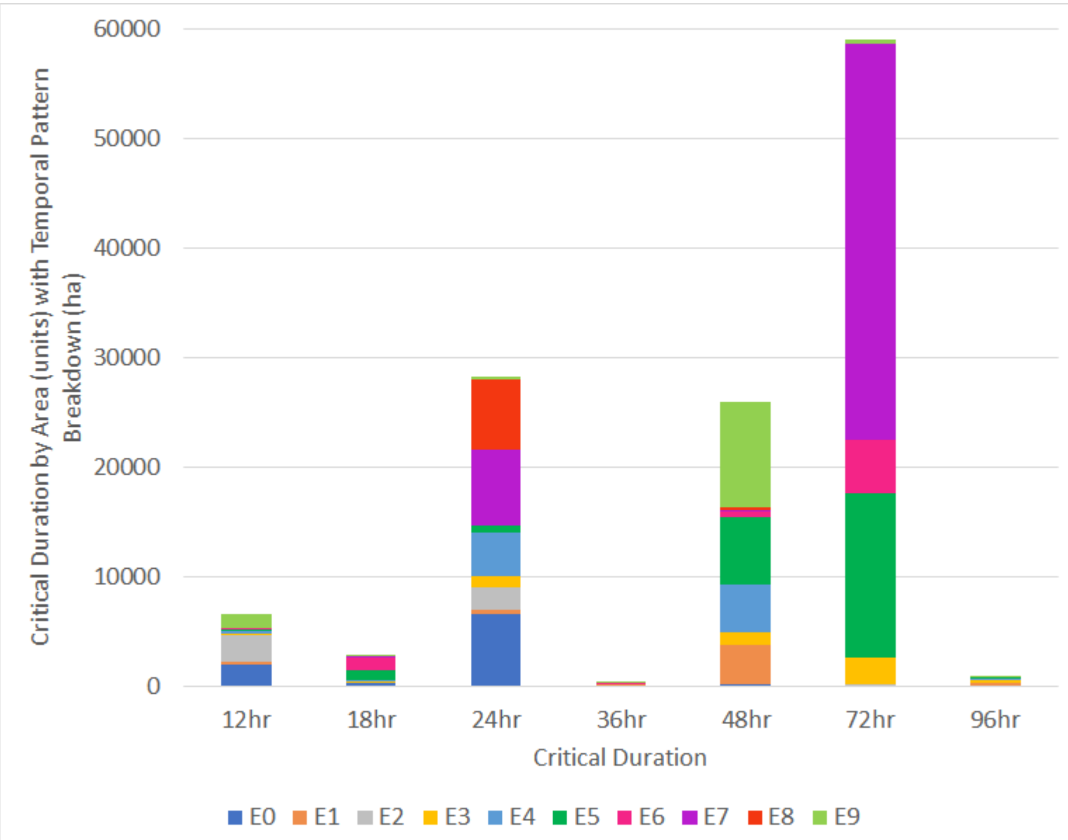
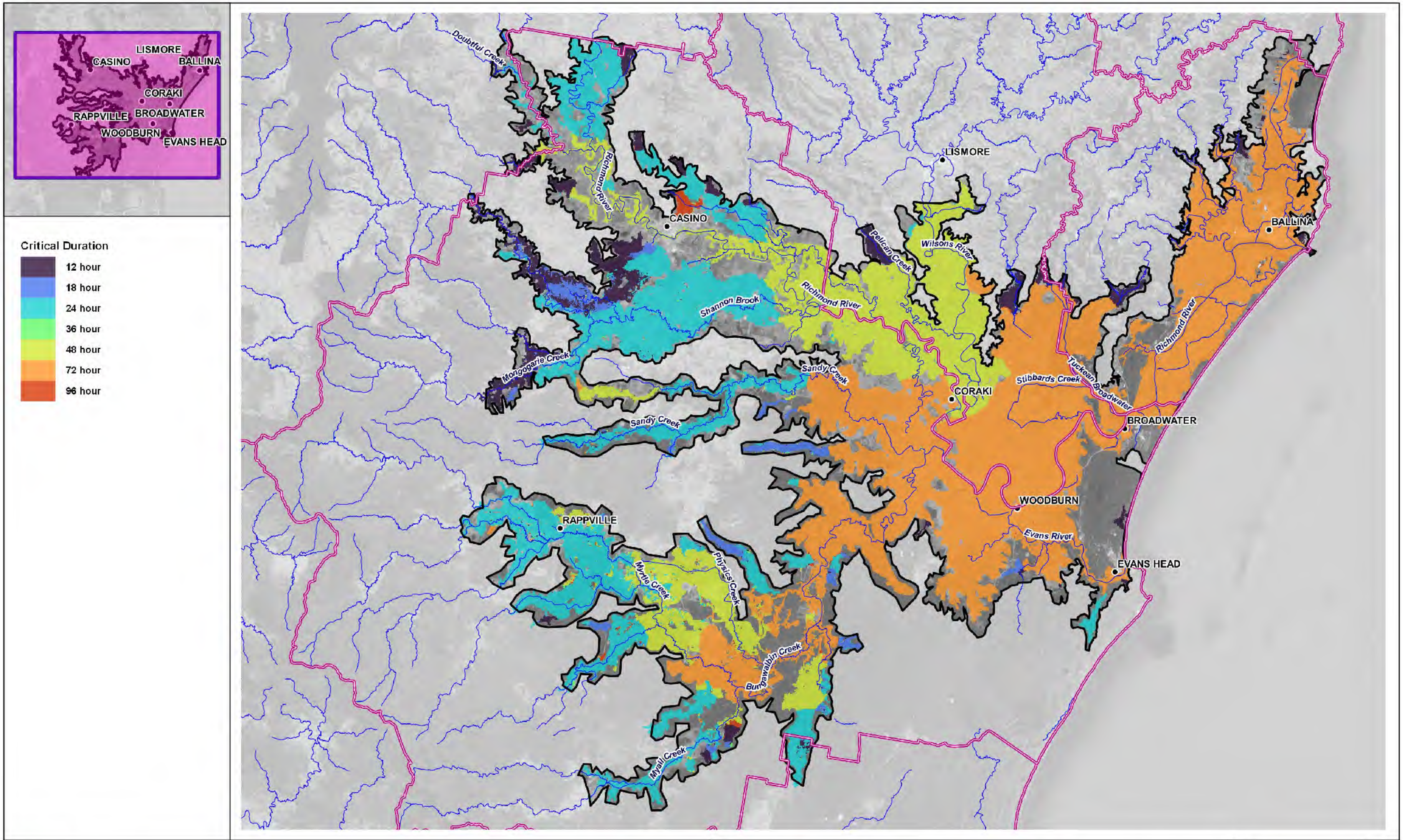


Figure 8.5 Graphical Representation of Event Selection: 5% AEP

Table 8.10 Selected Events: 5% AEP

Duration (h)	Selected Ensemble Event	ARR Event TP ID*
24	E0	221
48	E9	410
72	E5	496

* East Coast (South) Region



Title:

5% AEP - Critical Duration for Event Selection

Drawing: 8-6

Rev: A

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8.5.3 2% AEP

Figure 8.7 graphically summarises results for the 2% AEP in the same format as for the 1% AEP. Figure 8.8 maps the critical durations. The 24, 48 and 72 hour critical durations dominate the model domain. Upon inspection of the 48 hour results, there was very little difference in peak flood level from that of the 24 hour result with differences within the threshold of 0.1m i.e. the inclusion of the 48 hour event made minimal difference to the overall peak water surface. Therefore the 24 hour and 72 hour events were selected and the 48 hour event was not selected. For the 24 hour event temporal pattern E8 was selected and for the 72 hour event, E3 was selected. Whilst these temporal patterns are not the most dominant, they provided the best overall match within the specified tolerances. Table 8.11 summarises the durations and patterns selected. A critical duration map using the subset of the two selected events is included in the Design Results Mapping Addendum.

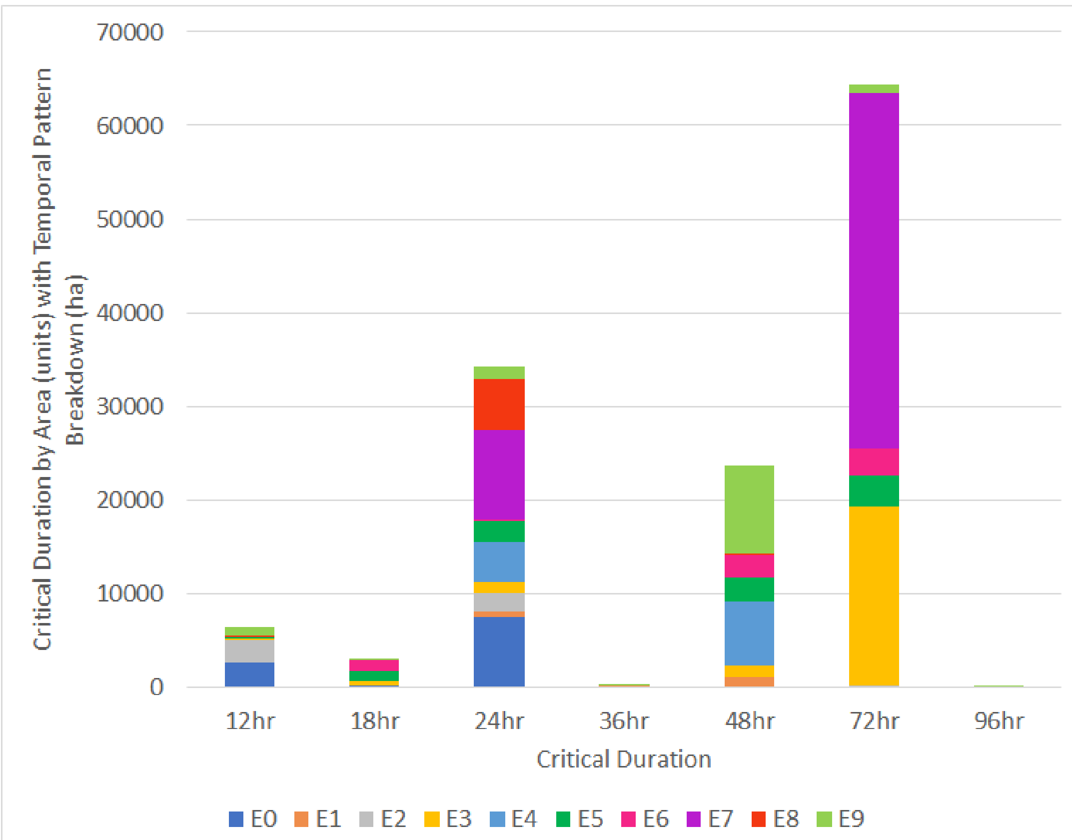


Figure 8.7 Graphical Representation of Event Selection: 2% AEP

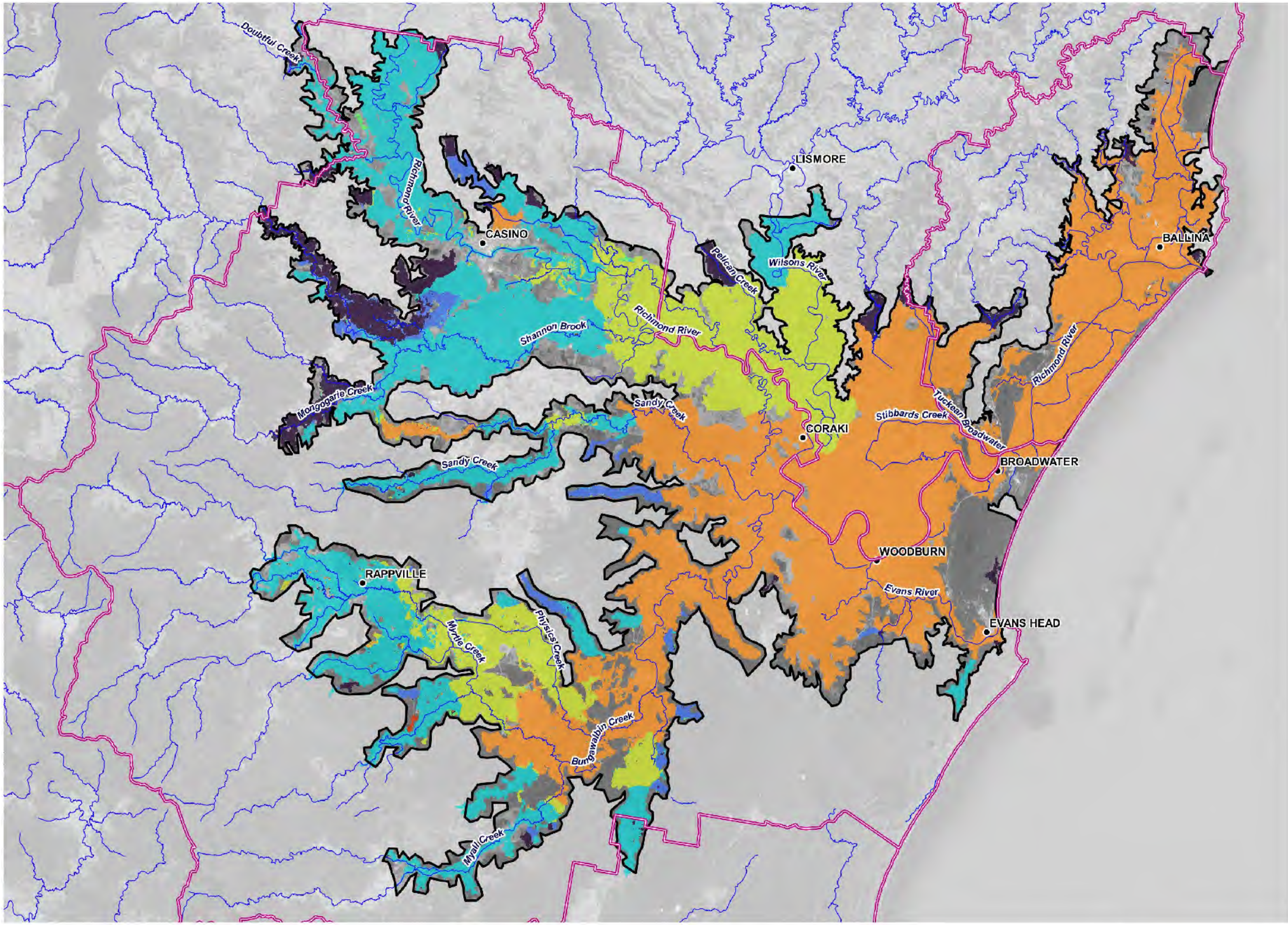
Table 8.11 Selected Events: 2% AEP

Duration (h)	Selected Ensemble Event	ARR Event TP ID*
24	E8	229
72	E3	494

*** East Coast (South) Region**

Critical Duration

- 12 hour
- 18 hour
- 24 hour
- 36 hour
- 48 hour
- 72 hour
- 96 hour



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

2% AEP - Critical Duration for Event Selection

Drawing: 8-8

Rev: A

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8.5.4 0.2% AEP

Figure 8.9 graphically summarises results for the 0.2% AEP in the same format as for the 1% AEP. Figure 8.10 maps the critical durations. The 48 and 72 hour critical durations dominate the model domain. Whilst the 24 hour critical duration was less dominant across the model domain, it still covered some key locations within the study area and so was retained.

Table 8.10 summarises the durations and patterns selected. Temporal pattern E5 was selected for the 24 hour duration as it provided the best outcome when considering the mapping tolerances. For the 48 hour and 72 hour durations the dominant temporal was selected (E6 for 48 hours and E3 for 72 hours). A critical duration map using the subset of the three selected events is included in the Design Results Mapping Addendum.

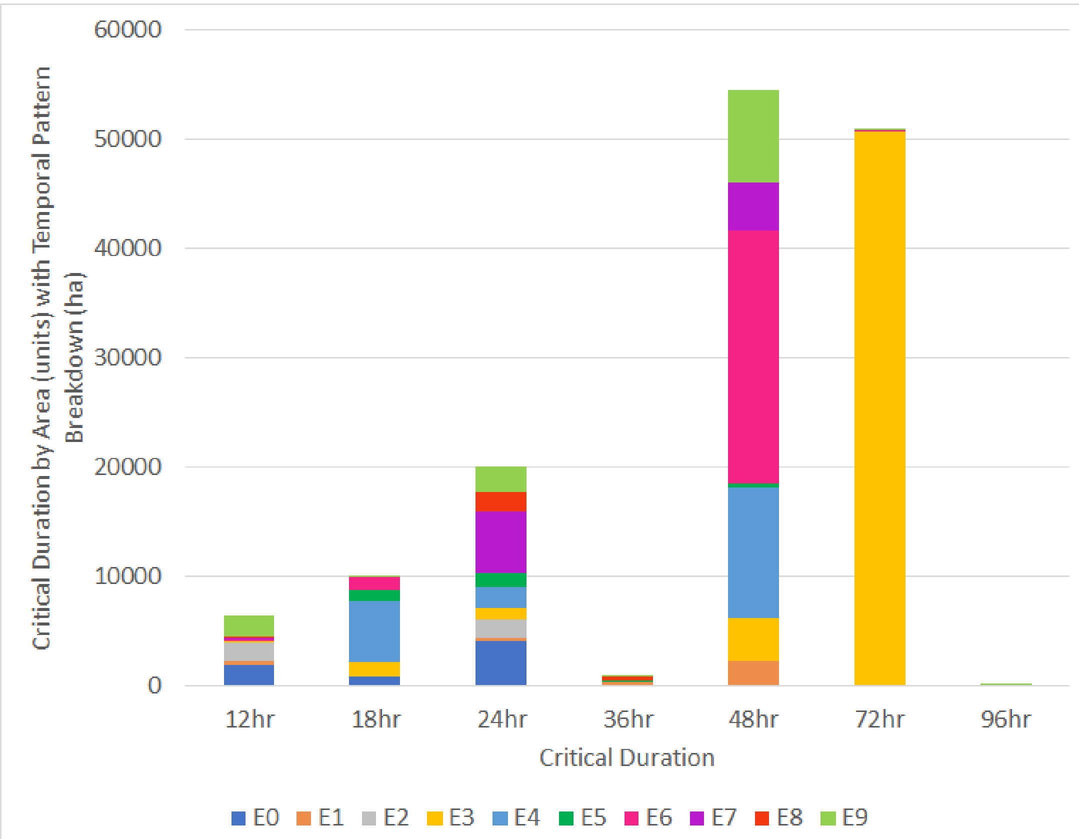


Figure 8.9 Graphical Representation of Event Selection: 0.2% AEP

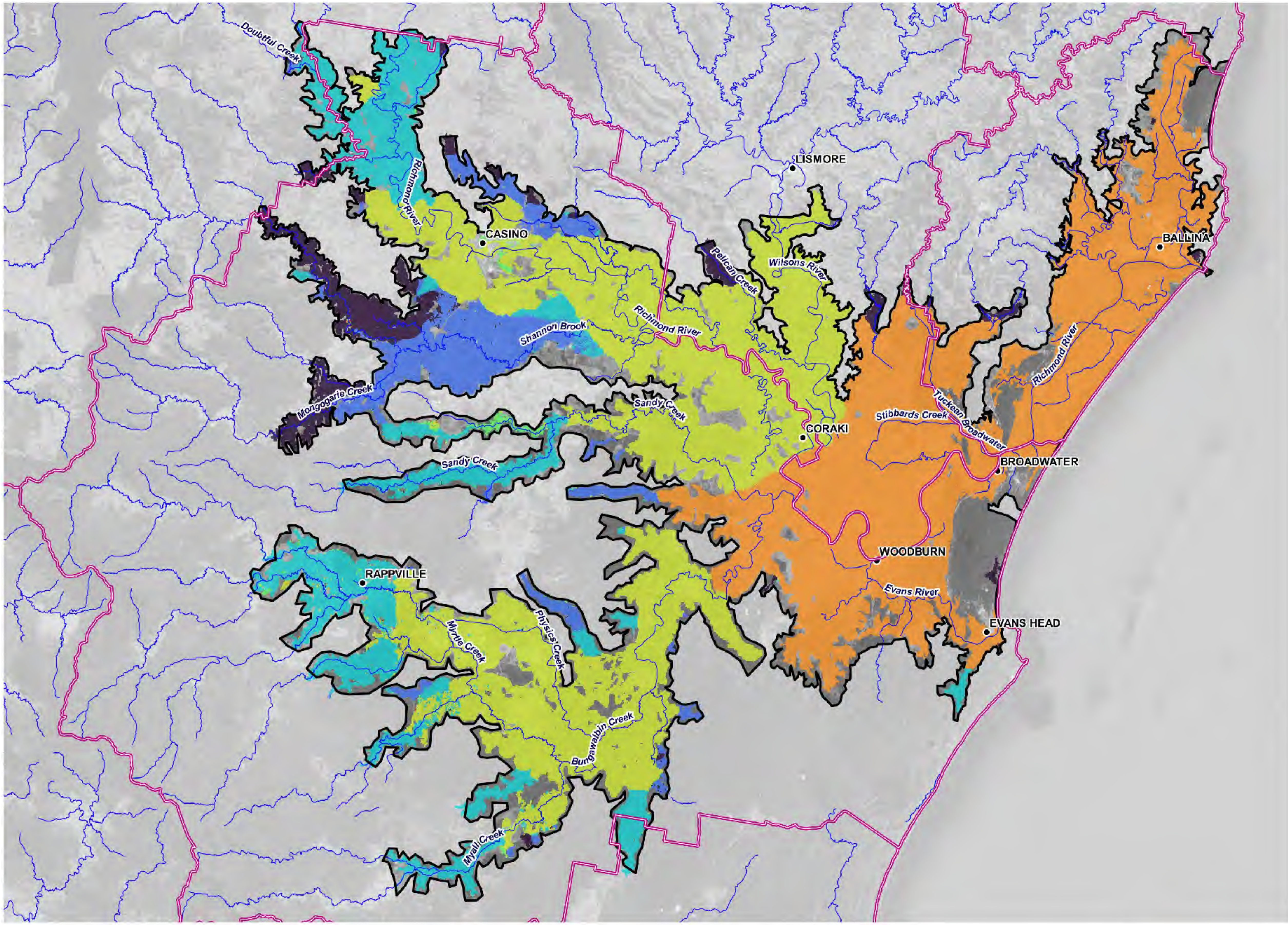
Table 8.12 Selected Events: 0.2% AEP

Duration (h)	Selected Ensemble Event	ARR Event TP ID
24	E5	226
48	E6	407
72	E3	494

* East Coast (South) Region

Critical Duration

- 12 hour
- 18 hour
- 24 hour
- 36 hour
- 48 hour
- 72 hour
- 96 hour



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

0.2% AEP - Critical Duration for Event Selection

Drawing: 8-10

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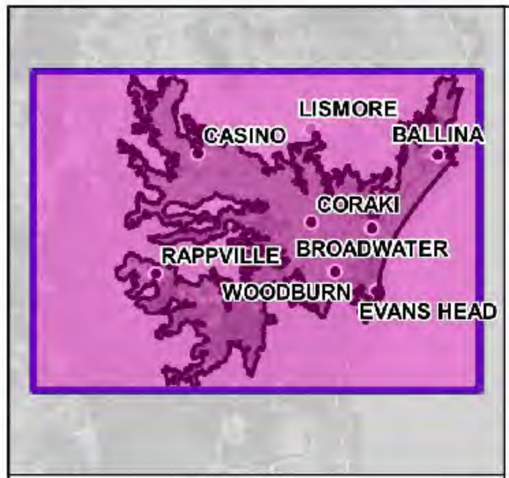
8.5.5 PMF

Critical durations for peak flood level for the PMF are mapped in Figure 8.11. As the AVM patterns were used, there is a single temporal pattern per duration and so a graphical representation of results has not been presented. Figure 8.11 shows that two durations dominate the study area with those being the 36 and 96 hour durations. The 96 hour duration is critical in the lower parts of the catchment and is of a longer duration than those found to be critical for the smaller design floods. This is due to the constriction in the terrain near Broadwater having a much greater influence on an extreme flood than a more moderate flood. As a consequence, the area upstream of the construction effectively becomes a large storage basin and peak flood levels within the basin have a greater dependence of the flood volume than the peak of the flood hydrograph entering the basin. Table 8.13 summarises the durations.

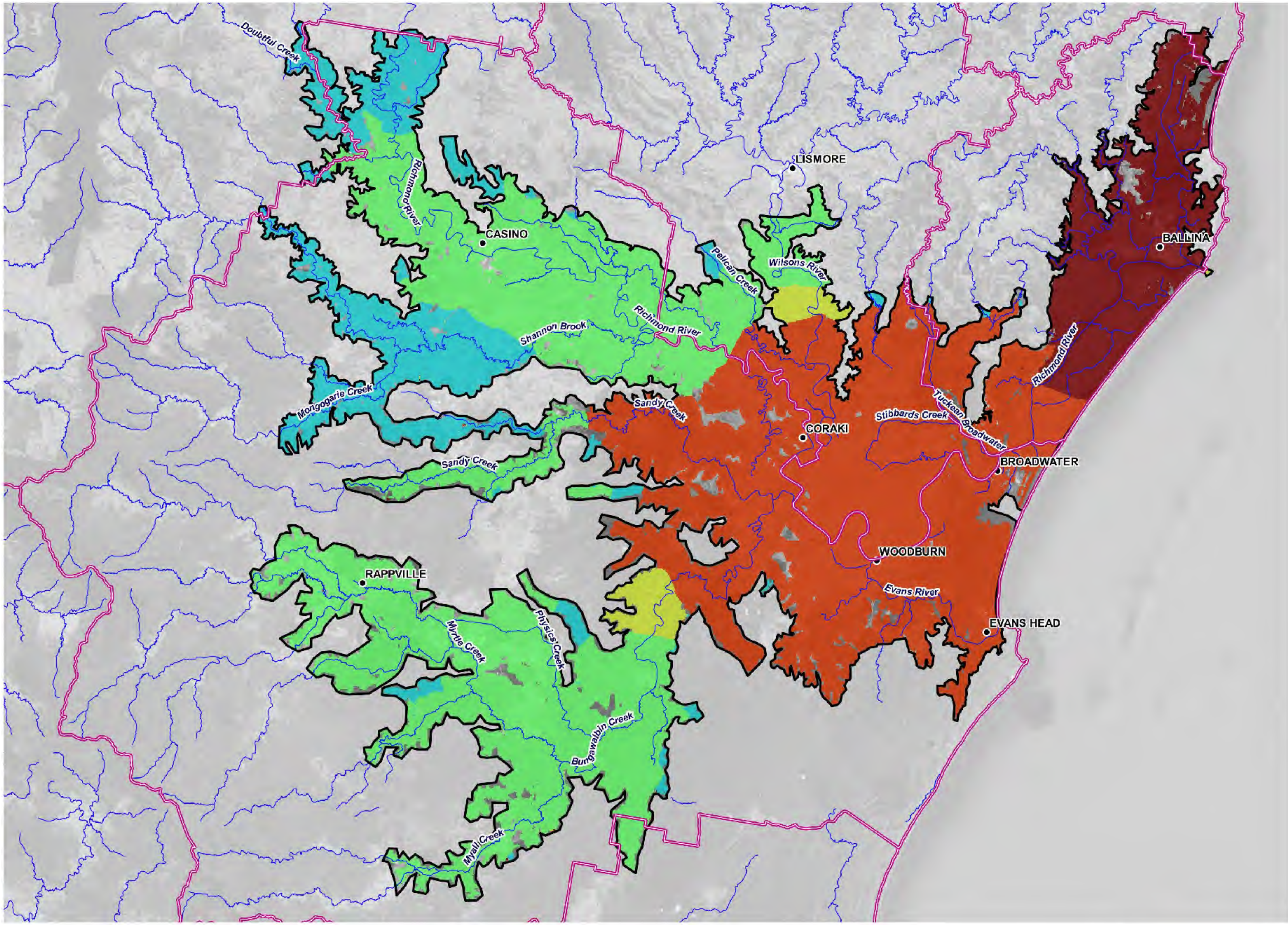
Table 8.13 Selected Events: PMF

Duration (h)	Selected Ensemble Event	ARR Event TP ID
36	AVM	n/a
96	AVM	n/a



Critical Duration

- 24 hour
- 36 hour
- 48 hour
- 72 hour
- 96 hour
- 120 hour



Legend

- LGA Boundaries
- Model Extent
- • Towns
- Watercourses

Title:

PMF - Critical Duration for Event Selection

Drawing: 8-11

Rev: A

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8.6 Reconciling Results With FFA

Table 8.14 and Table 8.15 present comparisons of the initial rainfall-based modelled peak flows with those derived independently using FFA for Casino and East Gundurimba respectively. These are two key locations in the model for assessing flows on the Richmond River and Wilsons River respectively. It can be seen that the FFA results give consistently higher flow estimates than the rainfall-based modelling techniques.

Table 8.14 Modelled Flow Comparisons with FFA (Casino*)

AEP	Modelled Peak Flow	FFA Peak Flow	Difference (from FFA)
5%	2,174	2,615	-20%
2%	2,889	3,600	-25%
1%	3,604	4,390	-22%

***Taken as total flow entering Casino from upstream i.e. before any breakouts**

Table 8.15 Modelled Flow Comparisons with FFA (East Gundurimba)

AEP	Modelled Peak Flow	FFA Peak Flow	Difference (from FFA)
5%	2,430	2,825	-16%
2%	3,039	3,800	-25%
1%	3,640	4,650	-27%

Where a reasonably long and reliable record of peak flows is available, FFA is considered to be the more accurate technique for design peak flow estimation as it incorporates all of the catchment variability due to being based on recorded data at the location of interest. Casino and East Gundurimba both have reasonable record lengths of 78 and 43 years respectively. The FFA estimates at these gauges have also accounted for the significant event of February/March 2022 along with other notable historic events. Therefore, the FFA peak flow estimates are considered to be the better estimate of peak flows at Casino and East Gundurimba compared to the rainfall-based modelling techniques.

To reconcile the rainfall-based modelled design flows with the FFA derived design flows, an appropriate scaling factor was applied to the modelled flows in order to provide a closer outcome to the FFA derived flows. The scaling factors are applied to the hydraulic model inflows upstream of the Casino and East Gundurimba gauges along with the resulting modelled peak flows once the scaling factors were applied. For ease of reference, the FFA derived flow estimates are also shown.

It can be seen that the reconciled modelled peak flows are now in general agreement with the FFA flow estimates with the exception of the 0.2% AEP event where the modelled flows are in the order of 10% lower than the FFA derived peak flow. Given the greater uncertainty associated with rarer events and that the reconciled modelled flow is within the confidence limits of the FFA estimate, the scaled 0.2% AEP flows were considered appropriate.

Table 8.16 Reconciled Design Peak Flows: Casino

AEP	FFA Peak Flow	Scaling Factor	Reconciled Peak Flow	Difference (from FFA)
5%	2615	1.1	2420	-7.4%

AEP	FFA Peak Flow	Scaling Factor	Reconciled Peak Flow	Difference (from FFA)
2%	3600	1.2	3590	-0.2%
1%	4390	1.2	4400	0.1%
0.2%	6325	1.2	5670	-10.3%

Table 8.17 Reconciled Design Peak Flows: East Gundurimba

AEP	FFA Peak Flow	Scaling Factor	Reconciled Peak Flow	Difference (from FFA)
5%	2825	1.1	2680	-5.1%
2%	3800	1.2	3670	-3.4%
1%	4650	1.3	4750	2.1%
0.2%	7060	1.3	6200	-12.2%

8.7 Design Flood Mapping

For each of the five design events modelled, the following mapping is presented:

- Peak flood elevations;
- Peak flood depths;
- Peak flood velocity;
- Peak classified flood hazard (AIDR classification, see Section 11.1).

These maps are presented in the Volume 2 Design Results Mapping Addendum which accompanies this Volume 1 report.

8.8 Design Flood Level Summary

Table 8.18 summarises the design flood elevations at key gauges within the study area.

Table 8.18 Peak Design Flood Levels (m AHD)

Location	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
Casino (Irving Bridge)	22.36	23.89	24.29	24.63	25.86
Casino (Gauge)	21.39	22.68	22.96	23.3	24.31
Yorklea	23.50	23.70	23.80	23.99	24.77
Tatham Bridge	11.63	12.06	12.35	12.92	15.09
Codrington	8.45	8.56	8.64	8.89	10.76
Coraki (MHL Gauge)	6.18	6.32	6.45	6.73	10.23
Bungawalbin Junction	4.76	5.33	5.70	6.17	10.16

Location	5% AEP	2% AEP	1% AEP	0.2% AEP	PMF
Woodburn	3.58	4.18	4.77	5.89	10.03
Broadwater	2.22	3.40	4.07	5.23	9.37
Rappville Rail Bridge	46.66	46.77	46.87	47.05	48.01
Rappville Summerland Way (Myrtle Creek)	37.55	37.65	37.73	37.89	38.62
Neileys Lagoon Road	9.33	9.83	10.10	10.57	12.77
Rocky Mouth Creek Floodgates	4.11	4.55	4.97	5.96	10.06
Iron Gates	1.58	2.64	3.15	4.26	8.70
Evans Head (Fishing Co Op)	1.25	2.21	2.47	3.23	7.34

8.9 Comparison with Previous Studies

8.9.1 Background

Design flood levels that inform current flood planning levels across the Richmond Valley LGA are based on the following three studies:

- Casino Floodplain Risk Management Study (2002)
- Richmond Flood Mapping Study (2010)
- Evans River Flood Study (2014).

All of these studies used ARR1987 techniques which are now superseded by ARR2019.

Figure 8.12 and Figure 8.13 present maps showing the change in peak flood level of this current study when compared to the previous studies for the 5% AEP and 1% AEP respectively. Where there is overlap between results from the Evans River Flood Study and the 2010 Flood Study, the results from the more recent Evans River Flood Study have been used for the comparison. Increases in peak flood level are where the current study gives higher results compared to previous studies and decreases are where it gives lower results.

A tabulated summary of peak flood levels and changes in levels is provided for key gauge locations in Table 8.19. In some cases, the change in level at the gauge (within the river) is different to that in the neighbouring floodplain. This is typically due to the updated model better representing floodplain levees and embankments. This is most notable at Coraki in the 1% AEP where the flood level in the river has increased by 0.05m but the flood levels across much of the town are around 0.6m lower than the previous study.

Table 8.19 Peak Flood Level Comparison Against Previous Studies

	5% AEP			1% AEP		
Gauge Location	Current Study (mAHD)	Previous Study (mAHD)	Difference (m)	Current Study (mAHD)	Previous Study (mAHD)	Difference (m)
Casino (Irving Bridge)	22.36	22.56	-0.20	24.29	24.30	-0.01
Casino (Gauge)	21.39	21.62	-0.23	22.96	22.80	0.16
Yorklea	23.50	n/a	n/a	23.8	n/a	n/a
Tatham Bridge	11.63	11.38	0.25	12.35	12.09	0.26
Codrington	8.45	8.26	0.19	8.64	8.38	0.26
Coraki (MHL Gauge)	6.18	5.75	0.43	6.45	6.21	0.24
Bungawalbin Junction	4.76	4.76	0	5.7	5.49	0.21
Woodburn	3.58	3.74	-0.16	4.77	4.68	0.09
Broadwater	2.22	2.77	-0.55	4.07	4.01	0.06
Rappville Rail Bridge	46.66	n/a	n/a	46.87	n/a	n/a
Rappville Summerland Way (Myrtle Creek)	37.55	n/a	n/a	37.73	n/a	n/a
Neileys Lagoon Road	9.33	n/a	n/a	10.1	n/a	n/a
Rocky Mouth Creek Floodgates	4.11	4.20	-0.09	4.97	4.93	0.04
Iron Gates	1.58	2.25	-0.67	3.15	3.09	0.06
Evans Head (Fishing Co Op)	1.25	1.80	-0.55	2.47	2.05	0.42

8.9.2 5% AEP Comparison

In the 5% AEP it can be seen that there are both areas of increases and decreases in peak flood level where the previous results overlap with the new results. Between Casino and Coraki the overall trend is for an increase in peak flood levels whereas in the lower floodplain downstream of Coraki the overall trend is for a decrease in peak level.

Notable reductions occur at Broadwater (0.55m lower), Iron Gates (0.67m lower) and Evans Head (0.55m lower). There are localised areas where the new results give higher flood levels including at Tatham, in the floodplain east of Coraki and in parts of Bungawalbin Creek. The increase noted at Coraki in Table 8.19 is confined to the river and the town itself actually shows a decrease of around 0.17m. On lower reaches of the Richmond and Evans Rivers, the lower levels in the updated study are largely attributed to the adoption of a tailwater condition which is 0.67m lower than the previous study ocean level.

8.9.3 1% AEP Comparison

In the 1% AEP the overall trend is for an increase in peak flood levels in the updated study, most notably on sections of the floodplain between Casino and Coraki.

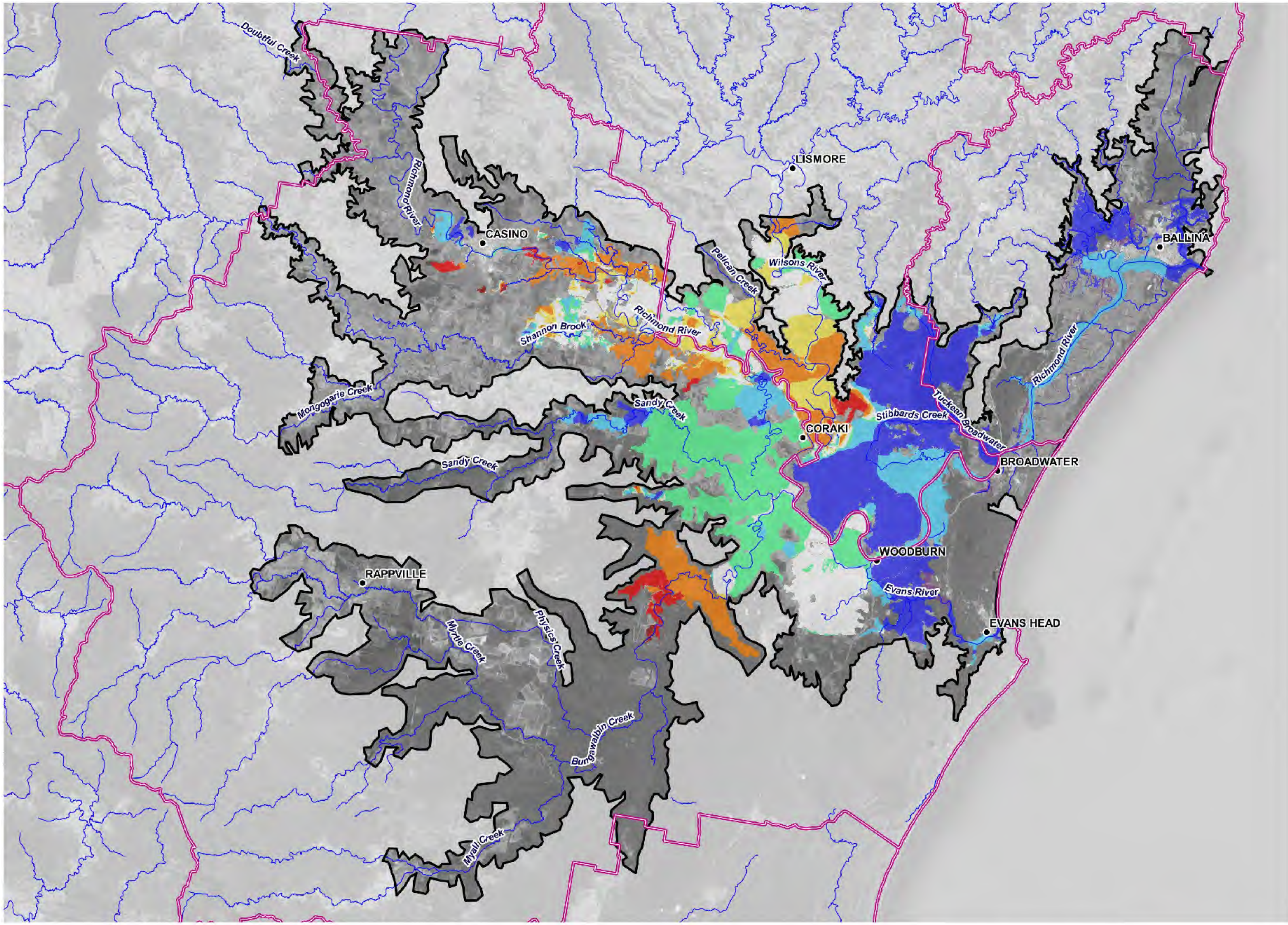
In Casino there is an overall increase in peak flood levels through the town. There is a large range in the differences in flood level with some areas showing decreases of 0.3m and other areas showing increases of up to 0.9m. This range reflects the improved representation of topography in the updated model which allows more accurate modelling of out of bank flows. In the Richmond River through Casino, the increases are more moderate and typically range between 0m and 0.2m.

Downstream of Coraki there is an overall minor increase in peak water level, for example by approximately 0.1m at Woodburn and Broadwater. The downstream section of the Evans River, including Evans Head, shows higher flood levels in the updated study (0.42m higher at the Evans Head gauge). Whilst the updated 1% AEP ocean water level boundary adopts the same storm surge peak level, the shift in the timing of this peak in relation to the riverine flood peak is likely to contribute to the change in peak flood level.

8.9.4 Comments and Conclusion

Both the 5% and 1% AEP peak flood level comparisons show a general trend for increased flood levels upstream of Coraki in the updated flood study. Downstream of Coraki, the 5% AEP event shows a general lowering of peak flood level and the 1% AEP shows a general minor increase in peak flood level. Whilst there are many differences between the studies, including improved representation of terrain in the updated model, the major differences are due to the hydrology including the use of ARR2019 design hydrology and the reconciling of peak flows with the flood frequency analyses. The differences and the somewhat scattered nature of these differences is therefore to be expected given the many factors involved.

Further comparisons between the use of ARR1987 and ARR2019 are provided in Section 8.10.



Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0

Legend

- LGA Boundaries
- Towns
- Model Extent
- Watercourses

Title:

5% AEP Peak Flood Level Difference - Current Study minus Previous Studies

Drawing: 8-12

Rev: A

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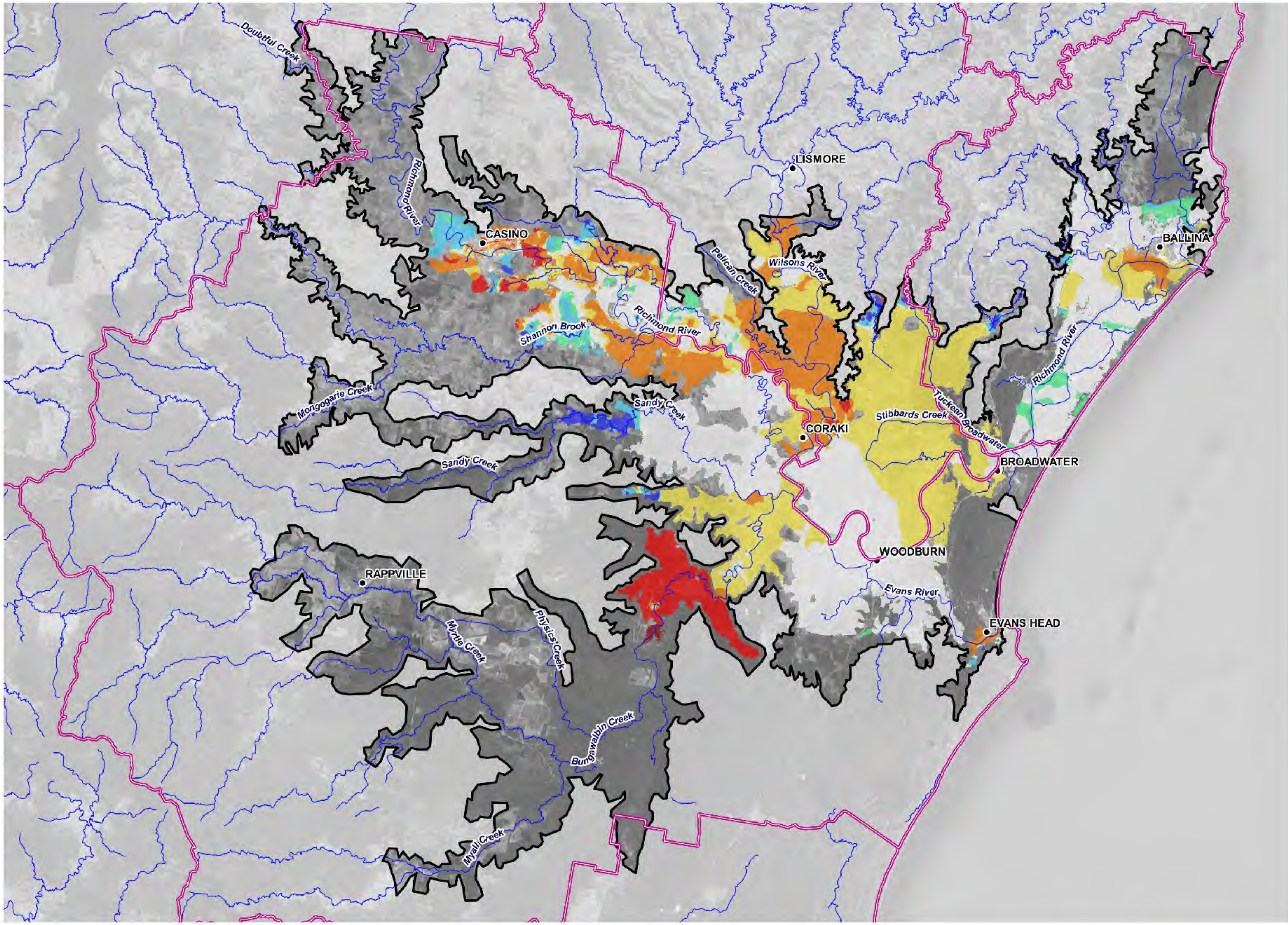
Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

1% AEP Peak Flood Level Difference - Current Study minus Previous Studies

Drawing: 8-13

Rev: A

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8.10 Comparison between use of ARR1987 and ARR2019

The current study has been undertaken following the guidance and design inputs based on ARR2019 guideline. To understand how results may have differed if the ARR1987 guideline was utilised, the hydrologic and hydraulic models developed and calibrated for this study have been simulated using ARR1987 design rainfall depths and temporal patterns.

Other inputs and assumptions used in the current study, for example the storm tide, baseflow and areal reduction factor assumptions, have been retained. The results between use of ARR2019 and ARR1987 have then been compared.

This comparison differs from that detailed in Section 8.9 as, whilst the 2010 Flood Study applied ARR1987 techniques, the models and modelling assumptions relating to ARFs, baseflow and storm tide assumptions differed to those in the present study. This study maintains consistency in those assumptions with the only differences being the rainfall depths and temporal/spatial patterns.

When using ARR1987, the Zone 3 temporal patterns have been applied as the Richmond River catchment is located within the Zone 3 region of ARR1987[10]. The models were then simulated with the ARR1987 design rainfall depths and temporal patterns (Zone 3) for the 24, 36, 48 and 72 hour storms. The peak flood level results from the hydraulic model were then combined using the maximum level for any given duration.

Figure 8.14 and Figure 8.15 present maps showing the change in peak flood level depending on whether an ARR1987 or ARR2019 approach is applied for the 5% and 1% AEP events respectively. Higher levels indicate that ARR1987 would give higher results than the present study and lower levels show were ARR1987 would be lower.

Note that for the purposes of allowing a like for like comparison, the ARR2019 results prior to being reconciled with FFA results were used in the comparison.

8.10.1 5% AEP Comparison

It can be seen from Figure 8.14 that the upper parts of the model (Richmond River to Codrington and much of the upper part of Bungawalbin Creek) show minimal difference in peak flood levels regardless of whether ARR2019 or ARR1987 rainfall depths and temporal patterns are used. At the Casino Gauge, the peak levels are only 0.03m different. In the lower floodplain there are significant differences with the ARR2019 results being up to 1.3m lower within the Tuckean Nature Reserve and 0.5m lower at Woodburn.

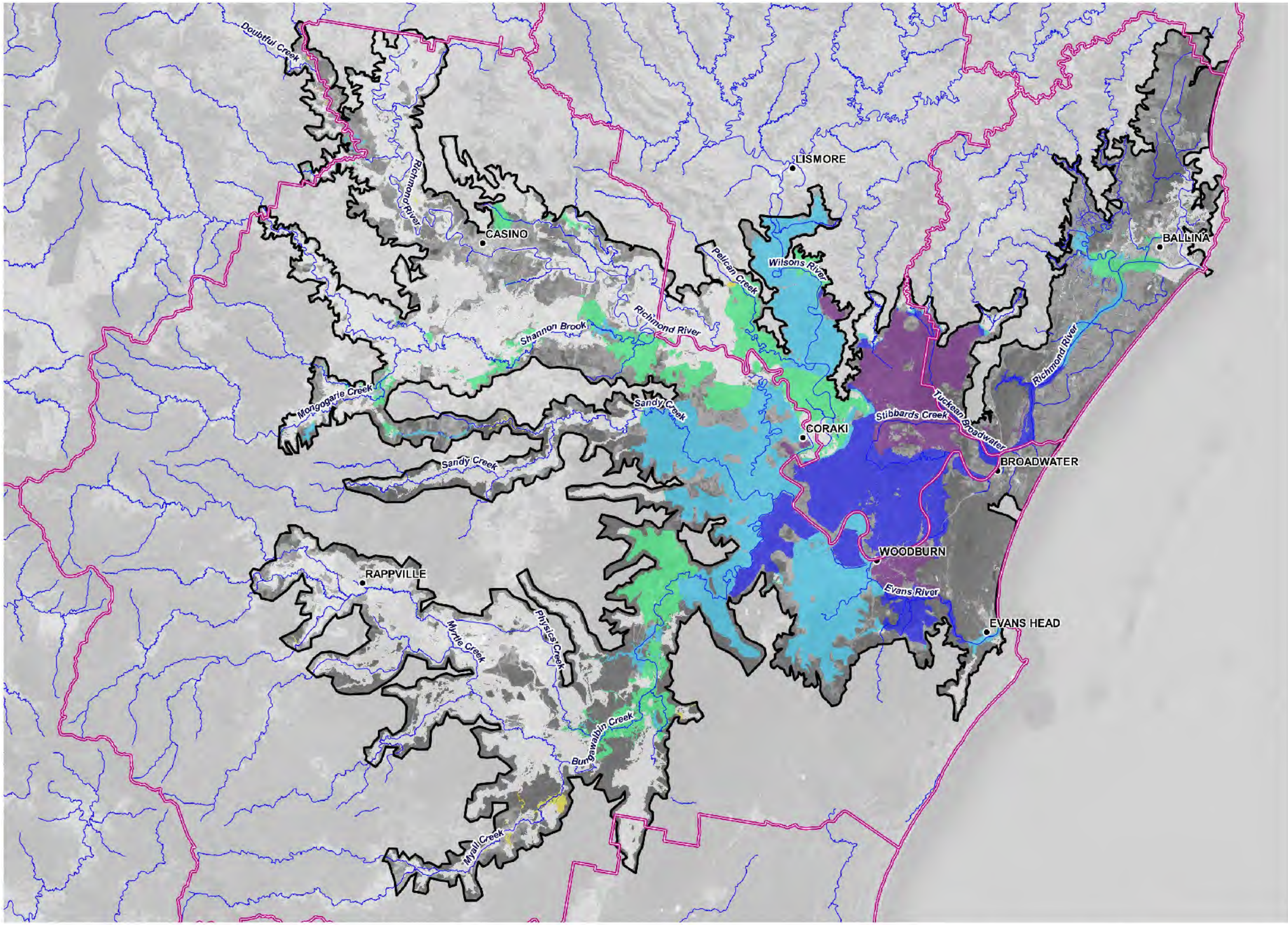
8.10.2 1% AEP Comparison

It can be seen from Figure 8.15 that the pattern of differences is similar to the 5% AEP comparison. Upper parts of the model show minimal difference in peak flood levels (at the Casino Gauge, the peak levels are only 0.01m different) with larger differences in the lower floodplain (the ARR2019 results being 0.9m lower at Woodburn).

[10] The 2010 Flood Study applied Zone 1 temporal patterns which generally resulted in significantly higher flows that if Zone 3 patterns were used. This was done to remain consistent with the Casino Flood Study and to achieve a better match with flow estimates derived at Casino through FFA. Revised FFA estimates developed for this study are notably lower at Casino and it is unlikely that Zone 1 patterns would be adopted for use had the ARR1987 guideline been applied.

Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

5% AEP Peak Flood Level Difference - ARR 2019 minus ARR 1987

Drawing: 8-14

Rev: A

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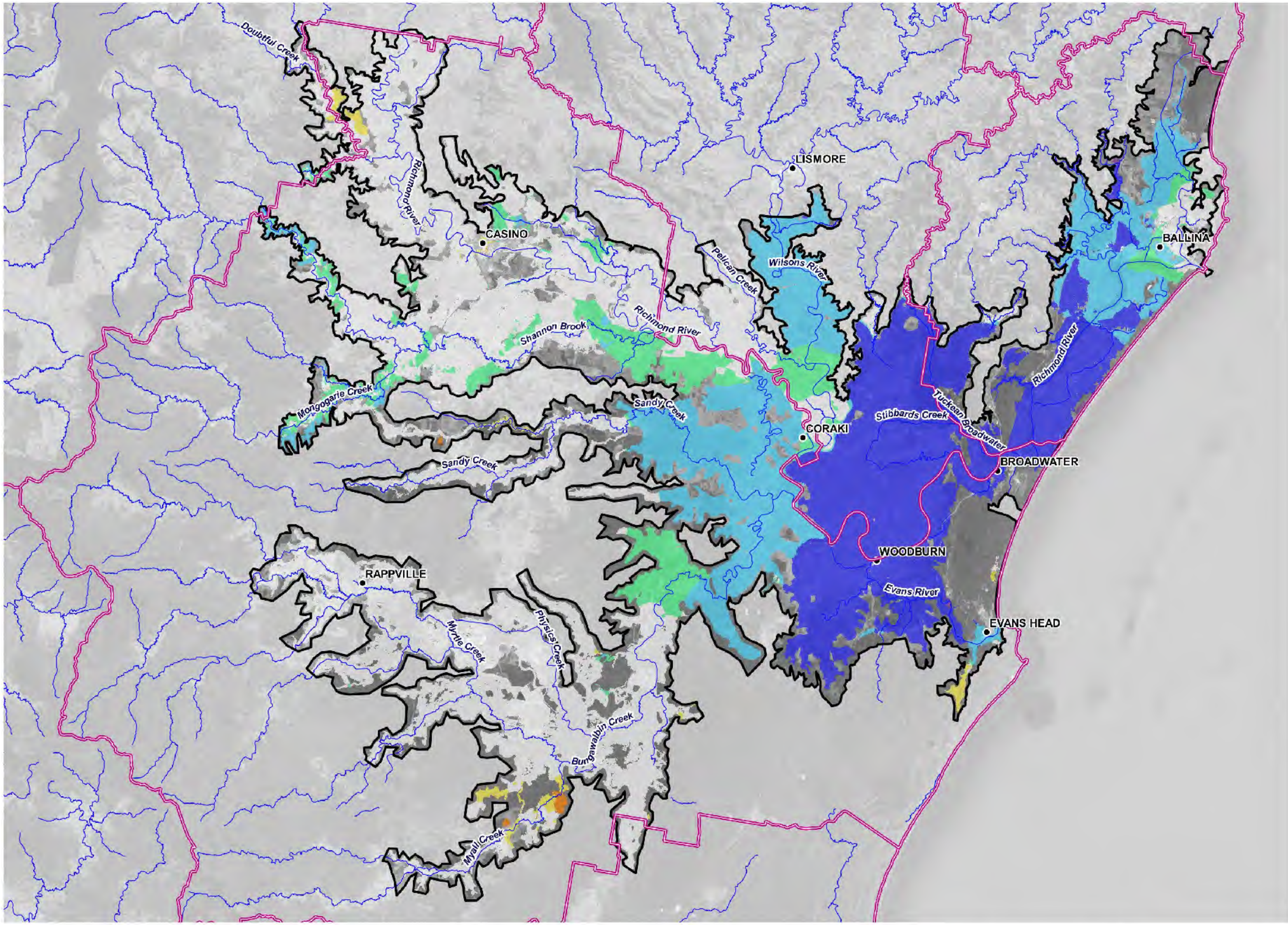
Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

1% AEP Peak Flood Level Difference - ARR 2019 minus ARR 1987

Drawing: **8-15**

Rev: **A**

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9 Sensitivity Scenarios

9.1 Climate Change

A climate change sensitivity scenario has been undertaken for the 1% AEP event. The modelled climate change scenario is in accordance with Richmond Valley Council's Scenario 3 (CC3) whereby the 1% AEP has rainfall depths increased by 10% and the downstream tidal boundary has been increased by 0.9m to account for future sea level rise. This has involved simulating both the hydrologic model to generate inflow hydrographs and then the hydraulic model to generate the revised flood levels.

The results of the climate change sensitivity scenario are tabulated for key gauge locations in Table 9.1 and are presented as a figure showing change in peak flood level (future climate 1% AEP minus current climate 1% AEP).

It can be seen that the most significant changes occur in the lower, tidal reaches of the catchment and this is attributed to the 0.9m rise in sea level. In upper parts of the catchment the change in peak level is dominated by the 10% increase in rainfall depths and increases in peak flood level typically range between 0.1m and 0.3m.

Table 9.1 Change in Peak Flood Levels with Climate Change at Key Locations

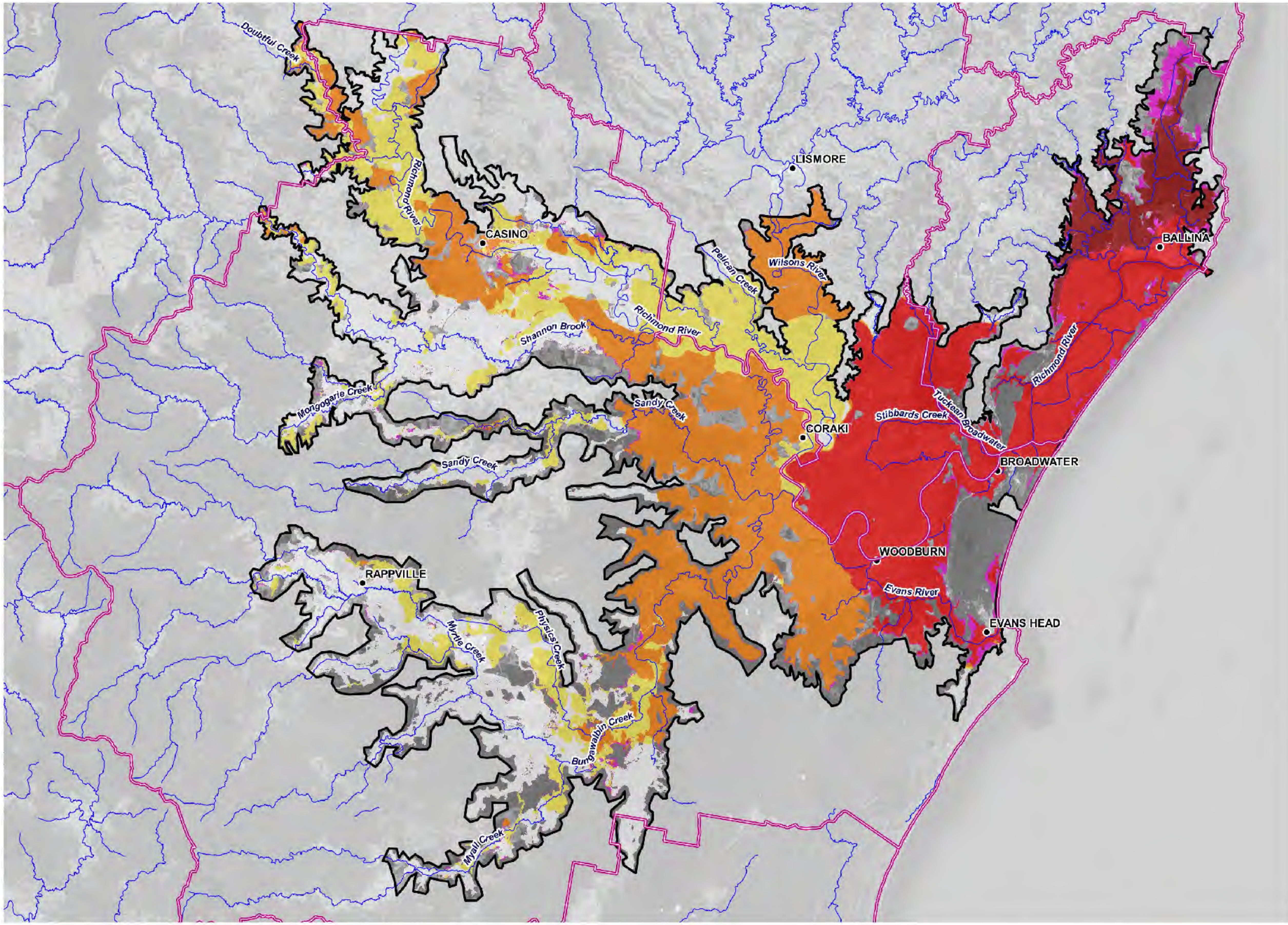
Location	1% AEP (2100)	1% AEP	Difference (m)
Casino (Irving Bridge)	24.50	24.29	0.21
Casino (Gauge)	23.12	22.96	0.16
Yorklea	23.88	23.80	0.08
Tatham Bridge	12.58	12.35	0.23
Codrington	8.73	8.64	0.09
Coraki (MHL Gauge)	6.57	6.45	0.12
Bungawalbin Junction	5.90	5.70	0.20
Woodburn	5.34	4.77	0.57
Broadwater	4.70	4.07	0.63
Rappville Rail Bridge	46.94	46.87	0.07
Rappville Summerland Way (Myrtle Creek)	37.79	37.73	0.06
Neileys Lagoon Road	10.29	10.10	0.19
Rocky Mouth Creek Floodgates	5.46	4.97	0.49
Iron Gates	3.88	3.15	0.73
Evans Head (Fishing Co Op)	3.32	2.47	0.85

Change in Inundation Extent

- Was Wet Now Dry
- Was Dry Now Wet

Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Towns
- Model Extent
- Watercourses

Title:

Change in 1% AEP Peak Flood Level due to Climate Change (CC3)

Drawing: 9-1

Rev: A

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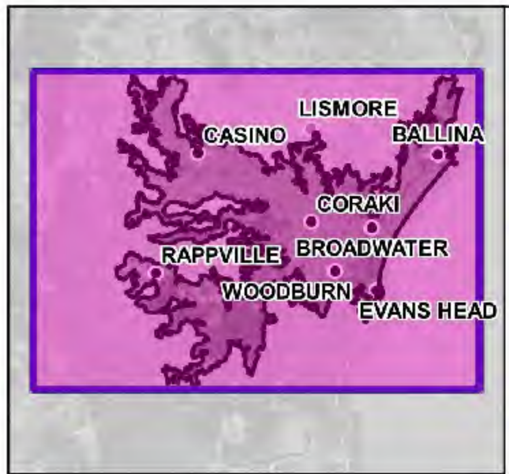
9.2 Blockage (All Clear)

Design flood simulations have all included a design blockage factor applied to bridges which represents the potential for those bridges to trap an amount of debris during large flood events (see Section 8.2).

An 'All Clear' sensitivity scenario has been undertaken in which the design blockage factors have been removed and peak flood level results have been compared back to the results which include the design blockage. This sensitivity scenario has been undertaken for the 1% AEP, 24 hour component event using the hydraulic model.

The results are shown in Figure 9.2 as changes in peak flood levels from the adopted design case. It can be seen that the flood levels and extents show no notable difference with the design blockage factors removed.

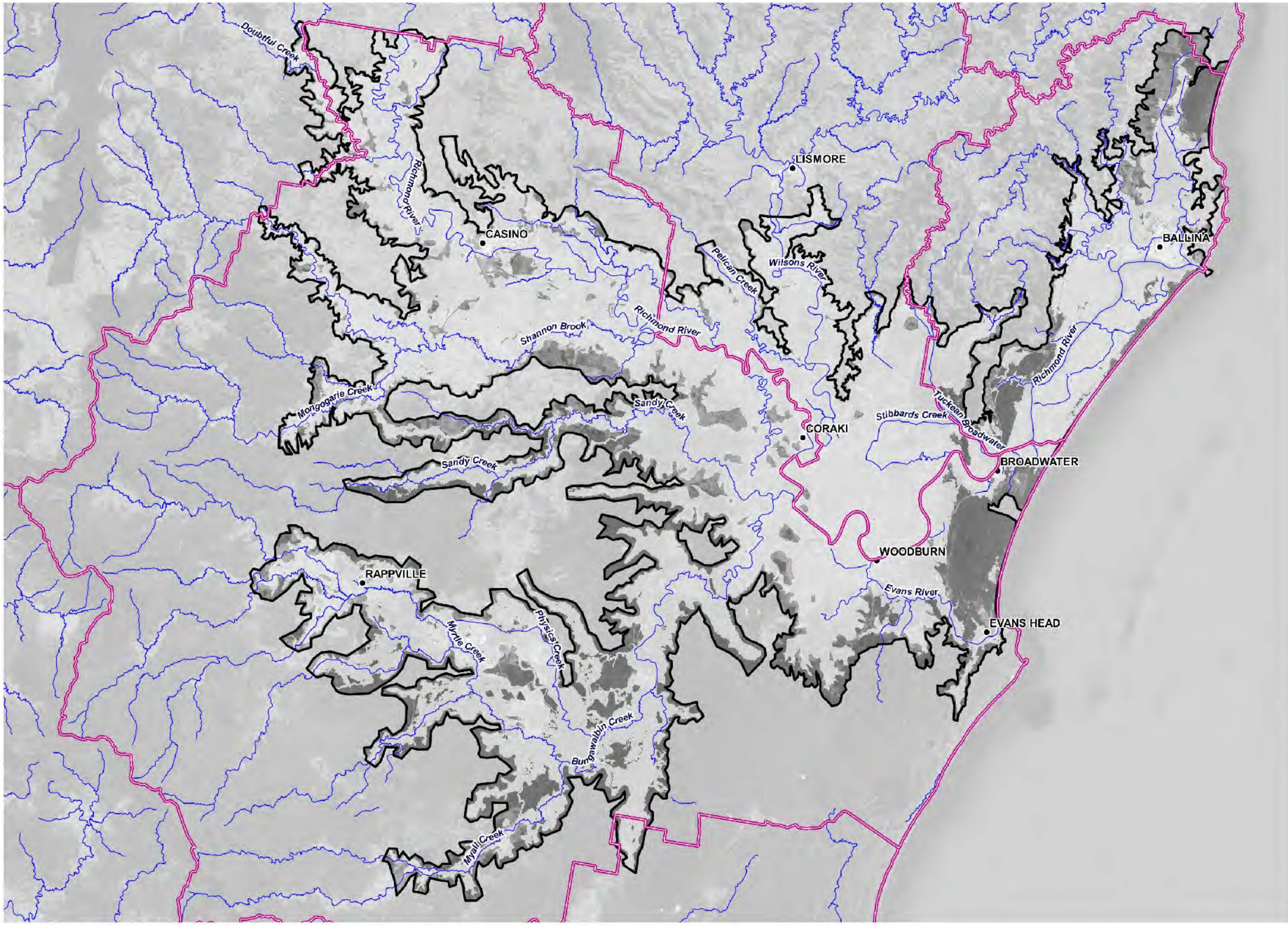


Change in Inundation Extent

- Was Wet Now Dry
- Was Dry Now Wet

Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

Change in 1% AEP Peak Flood Level – Design Case Blockage minus All Clear

Drawing: 9-2

Rev: A

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9.3 Manning's n

Two hydraulic model sensitivity scenarios were undertaken in which the Manning's n values, representing hydraulic roughness in the floodplain, have been increased and decreased by 20% respectively. The sensitivity scenarios have been undertaken for the 1% AEP event. The results are presented as mapped changes in peak level from the adopted 1% AEP flood surface. Figure 9.3 shows the results for the decreased Manning's n scenario and Figure 9.4 shows results for the increased Manning's n scenario.

Overall the results are as expected with the decreased Manning's n values resulting in a general lowering of flood levels and the increased Manning's n values resulting in higher flood levels. On the lower Richmond River downstream of Broadwater, the opposite results with higher flood levels under the decreased Manning's n values and lower flood levels with increased Manning's n values. This is attributed to the upper parts of the modelled catchment either slowing or increasing the speed at which floodwaters reach the ocean. The higher Manning's n values will increase upstream levels, slow the overall speed of the flood and thereby delay the timing of floodwater into the lower Richmond River. This can then offset the timing of the flood with the modelled storm tide.

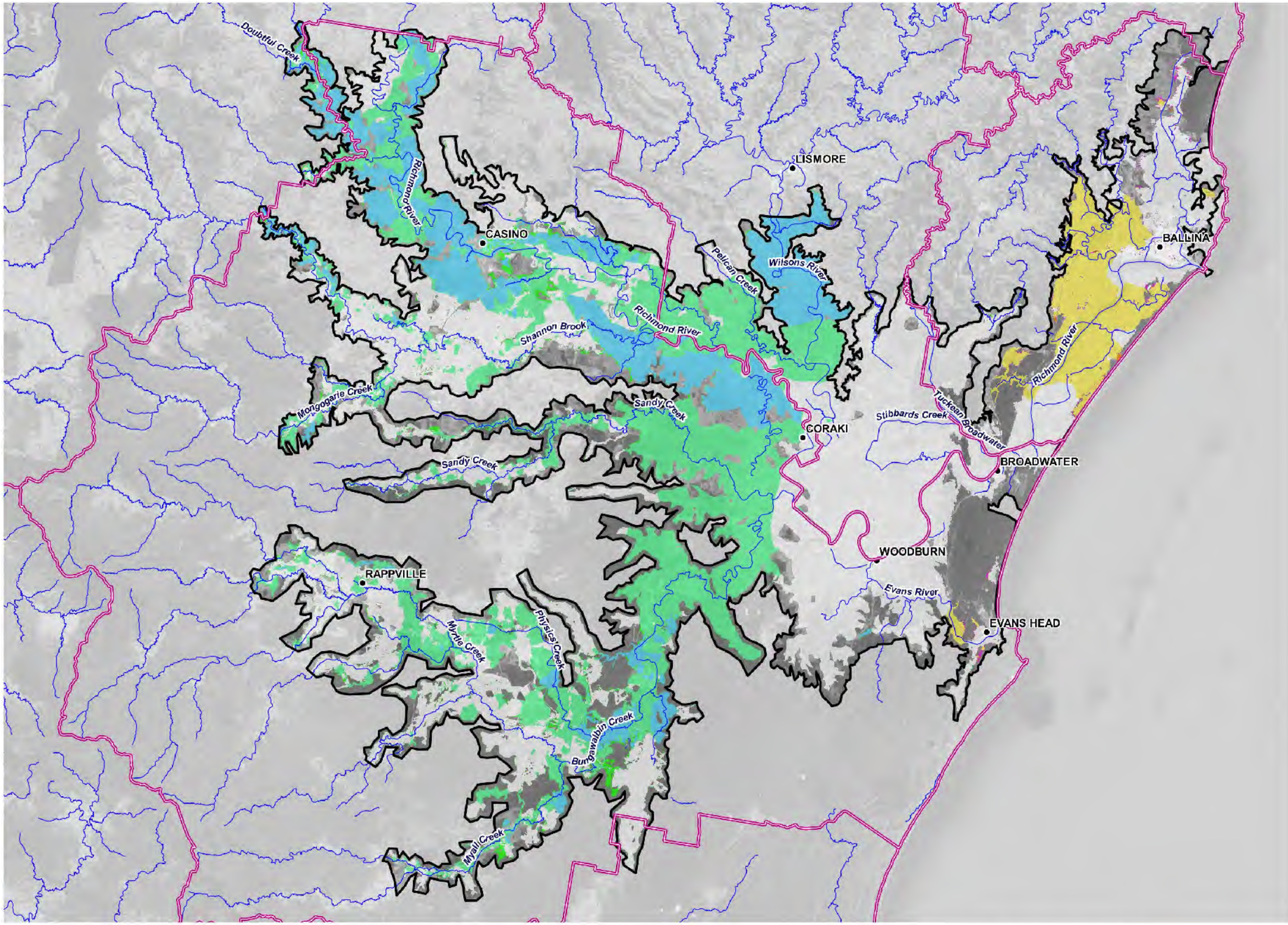
For much of the floodplain, the respective increases and decreases in flood level are only slight and within 0.1m. Casino was identified as the urban area where the sensitivity to Manning's n values is greatest.

Change in Inundation Extent

- Was Wet Now Dry
- Was Dry Now Wet

Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

Change in 1% AEP Peak Flood Level – Decreased Manning's n (20%)

Drawing: 9-3

Rev: A

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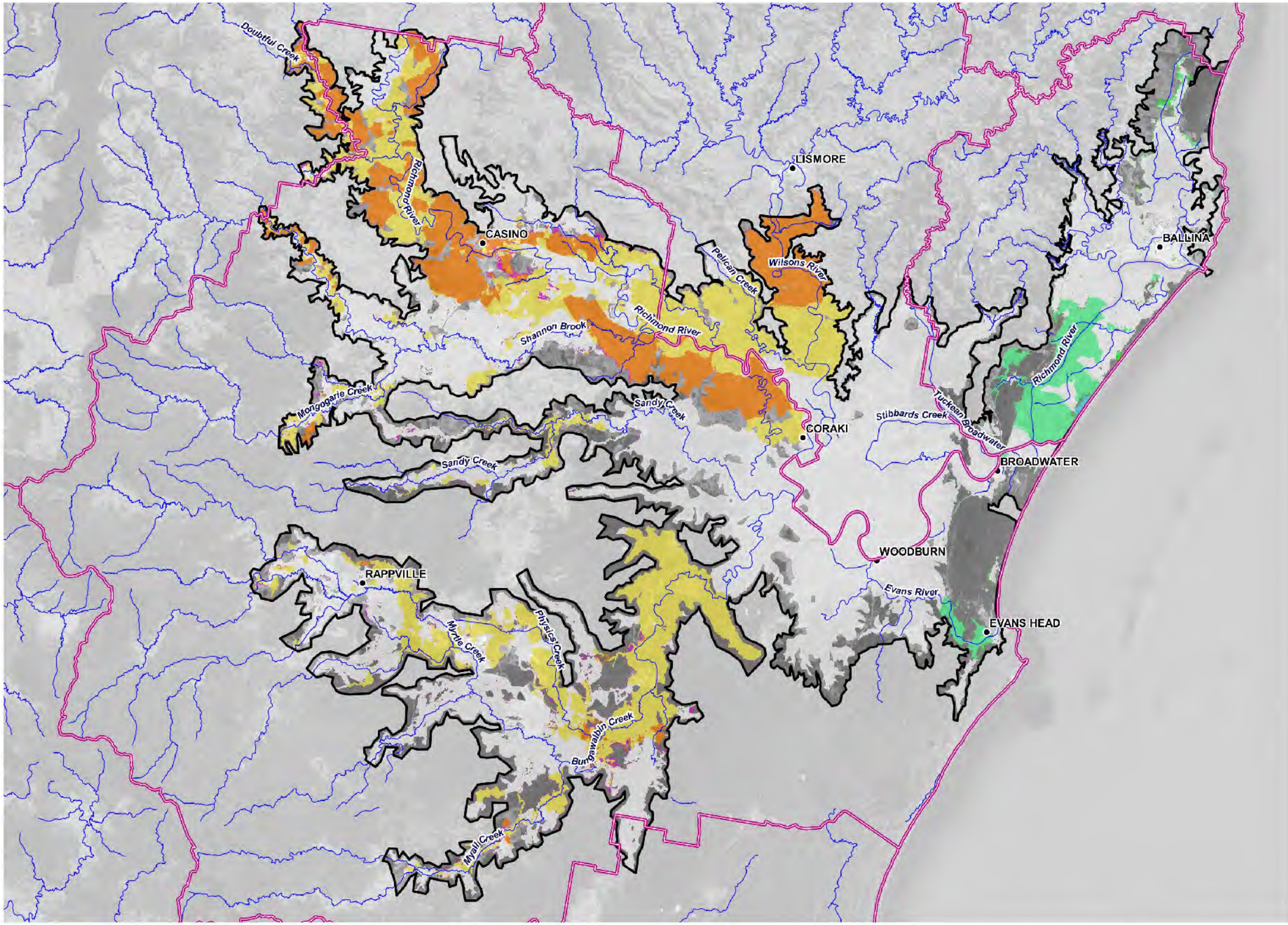
Change in Inundation Extent

- Was Wet Now Dry
- Was Dry Now Wet

Peak Flood Level Difference (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.2
- -0.2 to -0.1
- -0.1 to 0.1
- 0.1 to 0.2
- 0.2 to 0.5
- 0.5 to 2.0
- >1.0



Legend

- LGA Boundaries
- Model Extent
- Towns
- Watercourses

Title:

Change in 1% AEP Peak Flood Level – Increased Manning's n (20%)

Drawing: 9-4

Rev: A

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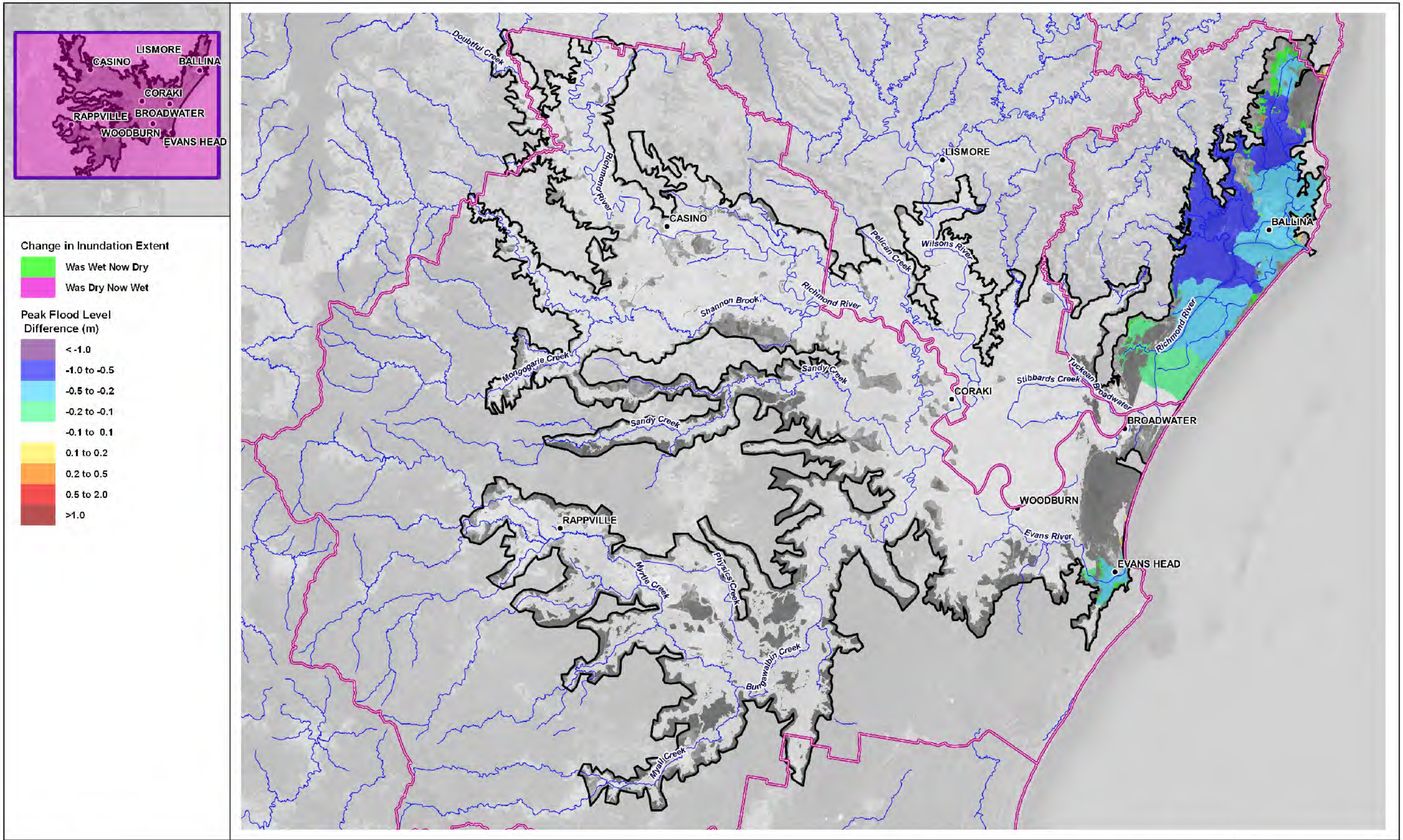
9.4 Timing of Storm Surge

A sensitivity scenario has been undertaken on the sensitivity of flood levels within the catchment to the timing of the peak of the storm surge. As described in Section 8.3, a 1% AEP catchment flood applies a 5% AEP storm surge with a peak that coincides with that of the catchment flood on the lower Richmond River (Woodburn). A sensitivity test has been undertaken where the peak of the storm surge is timed to occur before the onset of catchment generated flooding.

The sensitivity scenario has been performed for the 1% AEP with climate change event as this represents the most severe storm surge in the modelling assessment. Figure 9.5 presents the results by showing the resulting change in peak flood level due to having the storm tide peak before the catchment flood.

It can be seen from Figure 9.5 that the majority of the study area is not sensitive to assumptions around the timing of the storm surge. The exception to this is on the lower Evans River where the peak flood level reduces by approximately 0.3m at Evans Head.

The adopted approach to application of the storm tide follows current industry guidelines. If the study was to adopt a storm tide boundary that occurred prior to the catchment flood then a 1% AEP storm tide as opposed to a 5% AEP storm tide would need to be applied to ensure an overall 1% AEP flood level is captured across the lower floodplain. As a 1% AEP storm tide is higher than a 5% AEP storm tide then this would reduce further the differences in peak flood level shown in this sensitivity scenario.



Legend

LGA Boundaries • Towns

Model Extent — Watercourses

Title:

Change in 1% AEP Climate Change Peak Flood Level – Earlier Storm Tide

Drawing: 9-5

Rev: A

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10 Consequences of Flooding on the Community

10.1 Introduction

The design flood results have been used in combination with readily available information to provide additional information on the consequences of flooding on the community.

10.2 Property Counts

Table 10.1 summarises the number of properties inundated above habitable floor level for each modelled event. The property counts are listed by suburb and are based on Council's floor level survey database. The database may not capture all properties within the flood extent. Furthermore, properties which do not have a surveyed floor level were not included. These may be properties at which owners were not present at the time of the survey. It was outside the scope of the current study to verify details contained in the database or to update or infill any missing data. Despite these limitations, it is considered that this assessment provides a useful overview of those suburbs where the consequences of flooding in terms of property damage will be greatest.

From Table 10.1 the following points are noted with regards to numbers of inundated properties above floor level.

- The suburb of Coraki is the most affected for the most frequent modelled event (5% AEP) followed by Woodburn.
- In the 2% AEP event, the suburbs of Casino, Broadwater, Coraki and Woodburn are the most affected.
- In the 1% AEP event, there is notable overtopping of the river bank into Casino and a significant number of properties (700) are affected in the suburb. The suburbs of Broadwater, Coraki and Woodburn are also significantly affected.
- In the 0.2% AEP and the PMF event large parts of Casino are inundated. Due to its larger population it has significantly more inundated properties than other suburbs.

Table 10.1 Number of properties with above floor level inundation*

Suburb	5% AEP	2% AEP	1% AEP	1%CC AEP	0.2% AEP	PMF
Backmede	0	0	0	0	0	0
Bora Ridge	0	2	2	2	2	13
Broadwater	2	78	120	156	180	203
Bungawalbin	0	2	7	9	14	22
Casino	3	230	700	1099	1533	2333
Clovass	0	5	6	7	8	9
Codrington	3	4	6	6	13	23
Coombell	0	0	0	0	0	1
Coraki	24	86	160	194	240	374
Dobies Bight	1	1	2	3	5	12
Doonbah	0	3	8	16	21	37
East Coraki	1	2	3	6	12	23
Ellangowan	0	0	1	1	1	5
Evans Head	0	0	2	14	14	33
Fairy Hill	1	2	2	3	5	7

Suburb	5% AEP	2% AEP	1% AEP	1%CC AEP	0.2% AEP	PMF
Gibberagee	0	0	1	2	2	13
Greenridge	1	2	3	4	6	41
Irvington	2	7	7	8	10	18
Leeville	2	2	2	3	3	12
McKees Hill	1	2	2	2	2	2
Mongogarie	0	0	0	0	0	0
Myrtle Creek	0	0	1	1	2	7
New Italy	0	0	0	2	3	8
North Casino	0	0	0	0	0	2
Rileys Hill	1	3	4	8	12	22
Shannon Brook	0	0	1	1	1	8
Spring Grove	0	0	0	0	0	1
Stratheden	0	1	1	1	2	7
Swan Bay	1	6	17	32	39	57
Tatham	7	13	19	27	35	72
The Gap	0	0	0	0	0	2
Tomki	0	2	4	5	9	17
West Bungawalbin	0	0	0	0	0	0
West Coraki	0	0	0	0	3	8
Whiporie	0	0	0	0	0	0
Woodburn	15	63	131	219	262	300
Woodview	0	0	0	0	0	2
Yorklea	0	0	1	3	4	31

***Properties and floor levels based on Richmond Valley Councils Floor Level Database**

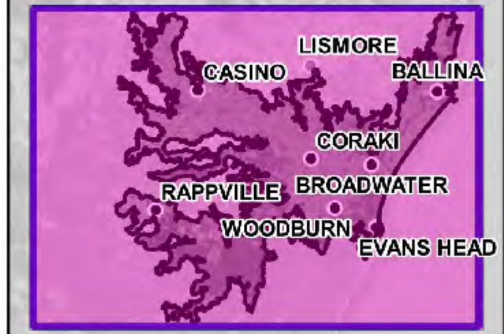
10.3 Major Arterial Roads

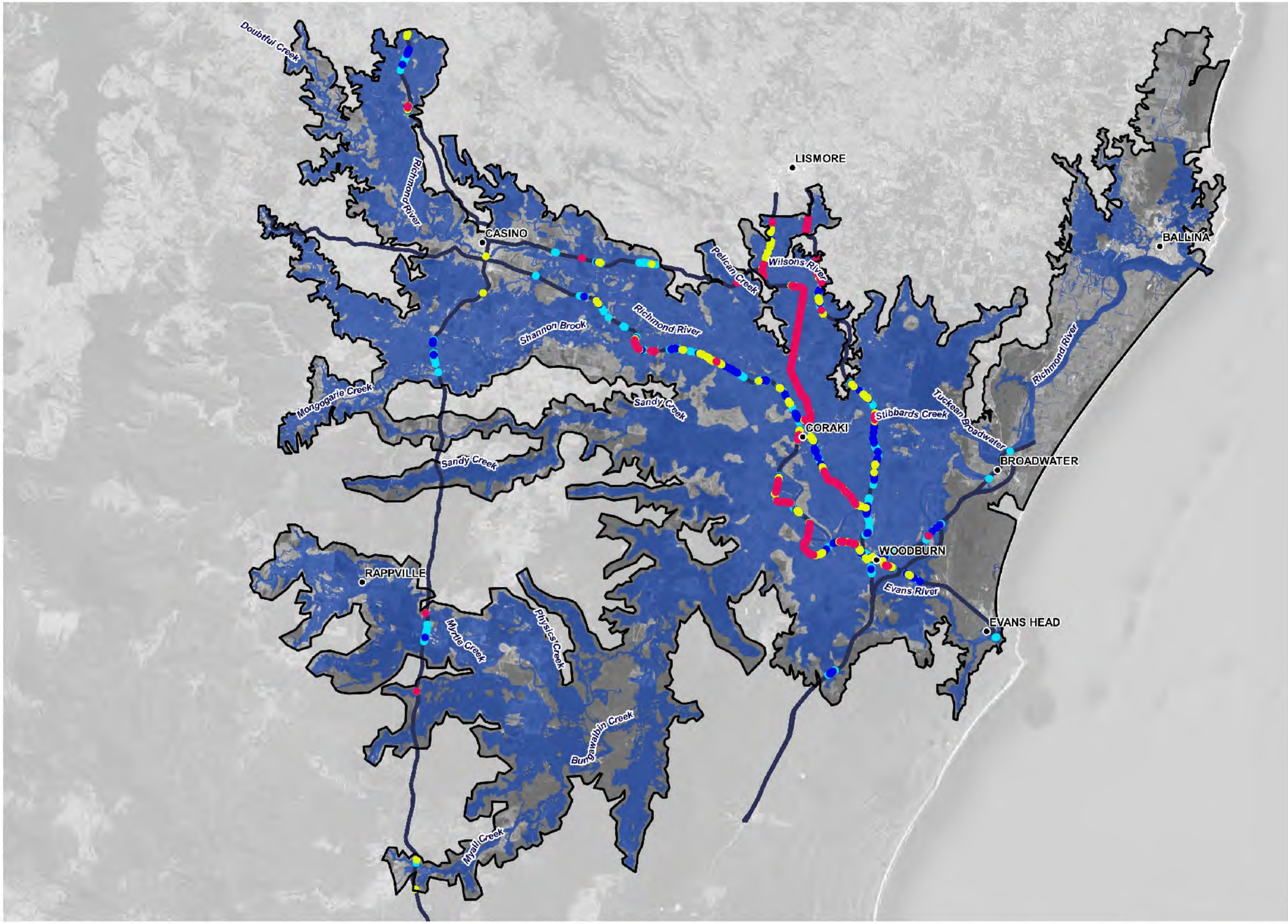
Estimated depths of flooding over key arterial roads within the study area is provided in Figure 10.1, Figure 10.2 and Figure 10.3 for the 5%, 2% and 1% AEPs respectively. The figures provide an indication as to which roads could be expected to become closed during these flood events. A threshold of 0.1m has been used to filter out shallow flooding. The assessment is indicative and relies on the accuracy of the road centreline and the representation of the road elevations in the hydraulic model. The analysis provides a useful first pass to identify potential problem areas for further investigation.

It can be seen that in a 5% AEP flood significant disruption would be caused with localised lengths of road connecting Casino with Lismore and Woodburn being inundated. This includes sections of Casino-Coraki Road at Tatham and numerous sections of Woodburn-Coraki Road. The results indicate that at least one carriageway of the Pacific Highway through the study area remains flood free in the 5% AEP.

In the 2% AEP the affected locations are generally the same as for the 5% AEP albeit with deeper and more extensive inundation. Results indicate that there is potential for limited inundation of the Pacific Highway south of the Tuckombil Canal and to the east of Woodburn.

In the 1% AEP the majority of key roads are affected by deep inundation. In the vicinity of Casino, most of the key roads remain dry or show shallow inundation. However, there are localised locations where the highways become cut. For example, to the south near Leeville, north of Fairy Hill and to the east at Tatham.





Depth (m)

- Dry or less than 0.1m
- 0.1m to 0.3m
- 0.3m to 0.5m
- 0.5m to 1.0m
- >1.0m

Flood Extent

- 5% AEP

Legend

Model Extent • Towns

Title:

5% AEP - Indicative Depths of Flooding on Key Roads

Drawing: 10-1

Rev: A

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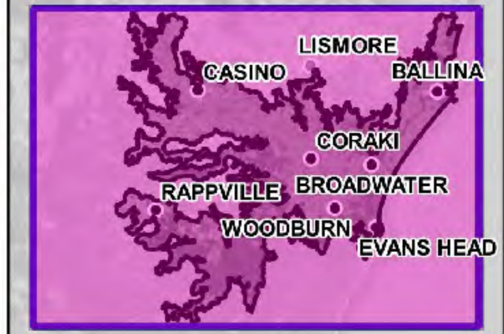
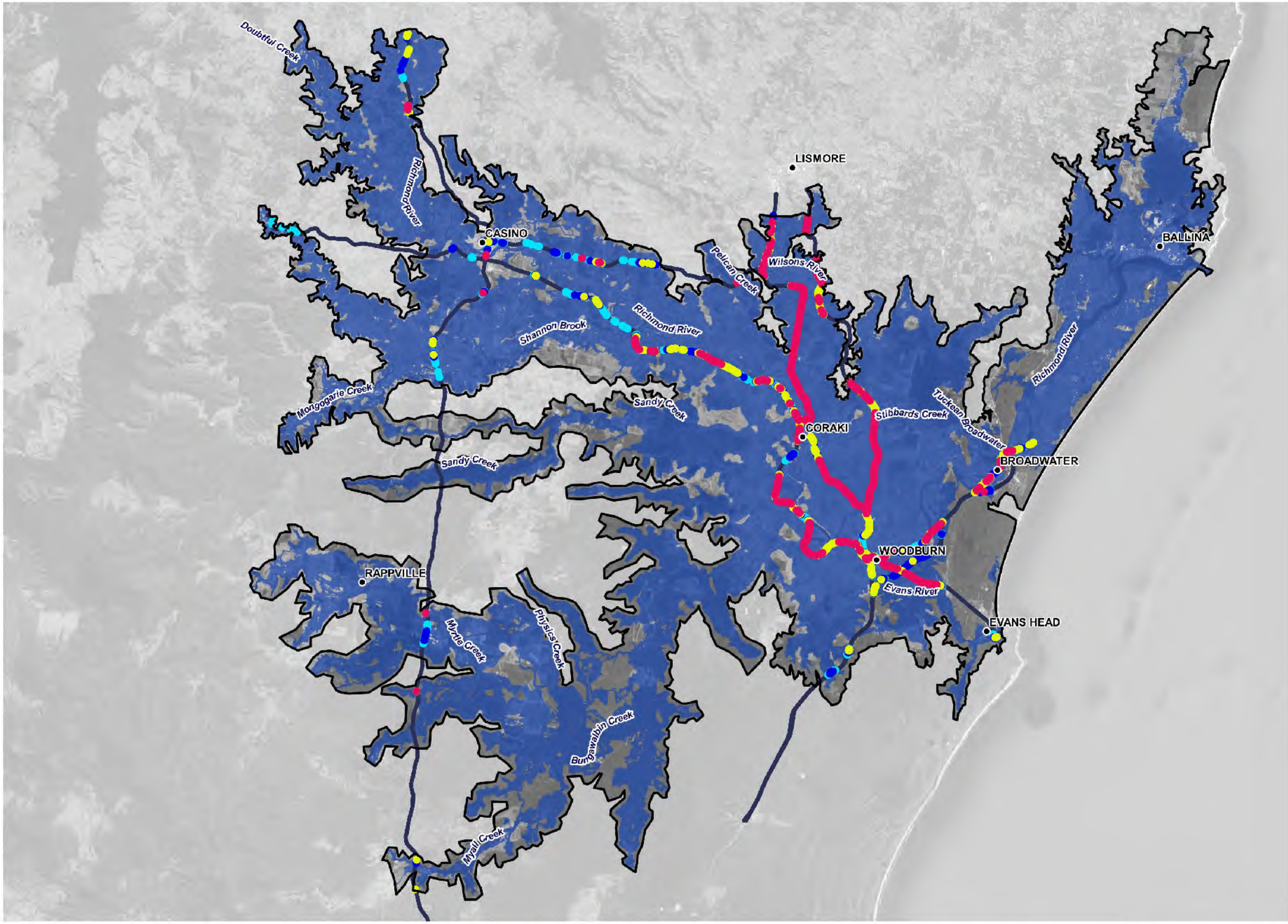


Depth (m)

- Dry or less than 0.1m
- 0.1m to 0.3m
- 0.3m to 0.5m
- 0.5m to 1.0m
- >1.0m

Flood Extent

- 2% AEP



Legend

- Model Extent
- Towns

Title:

2% AEP - Indicative Depths of Flooding on Key Roads

Drawing: 10-2

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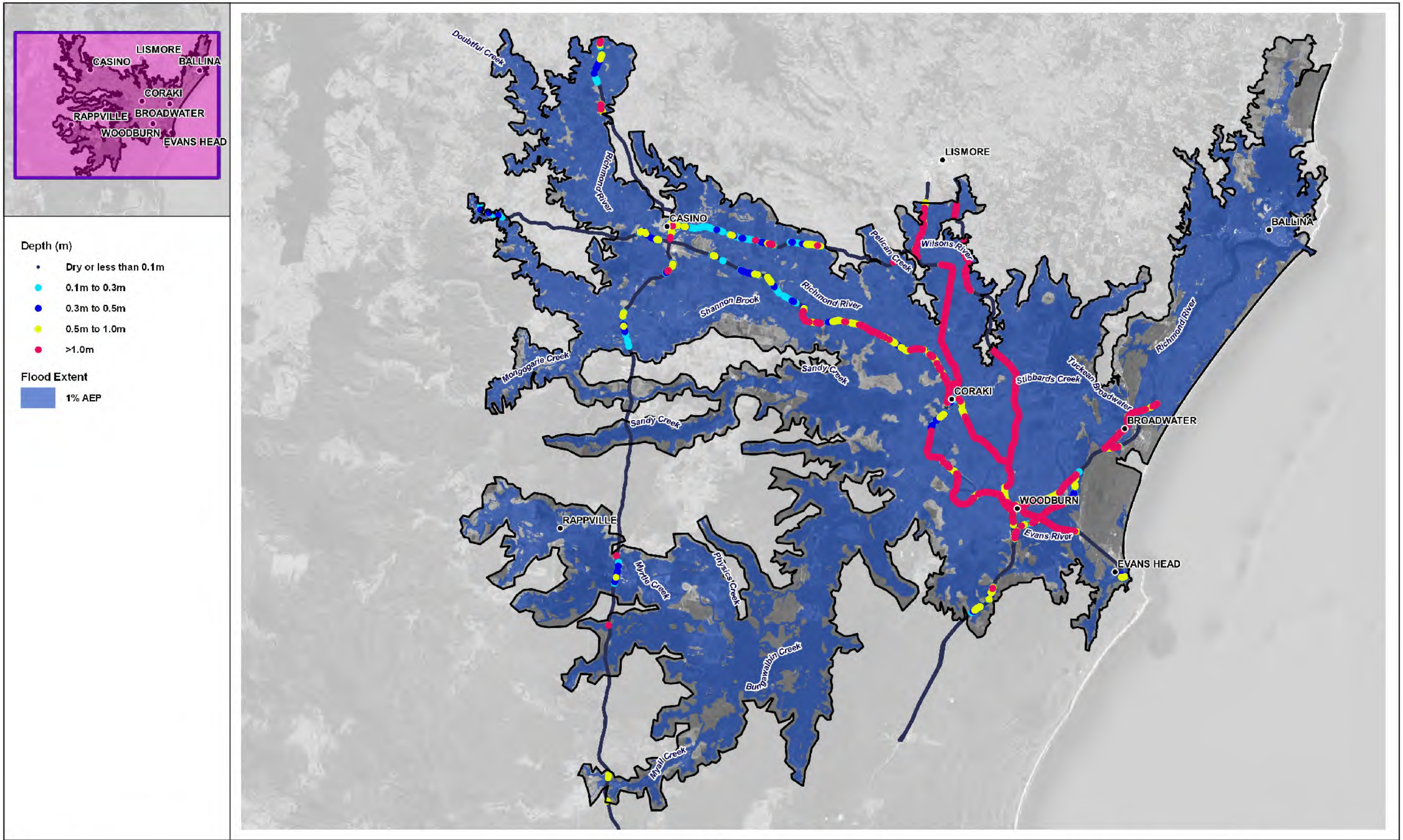

Legend

Model Extent • Towns

Title:

1% AEP - Indicative Depths of Flooding on Key Roads

Drawing: 10-3

Rev: A

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11 Post Processed Outputs

11.1 Flood Hazard

Flood hazard mapping has been prepared in accordance with general guidance from the Australian Institute for Disaster Resilience (AIDR, 2017a). This identifies flood hazard by considering the flood depth and velocity in combination and then classifies the hazard based on meaningful hazard thresholds. The categorised hazard can then be mapped to provide a general classification of flood hazard across the floodplain.

The flood hazard is classified into six categories of increasing severity based on the depth and velocity combinations shown in Figure 11.1 (replicated from AIDR 2017b) and originally from Smith et al (2014).

Descriptions of the flood hazard categories in terms of vulnerability thresholds are included in Figure 11.1 and are listed in Table 11.1. All mapping of flood hazard is included in the Design Results Mapping Addendum.

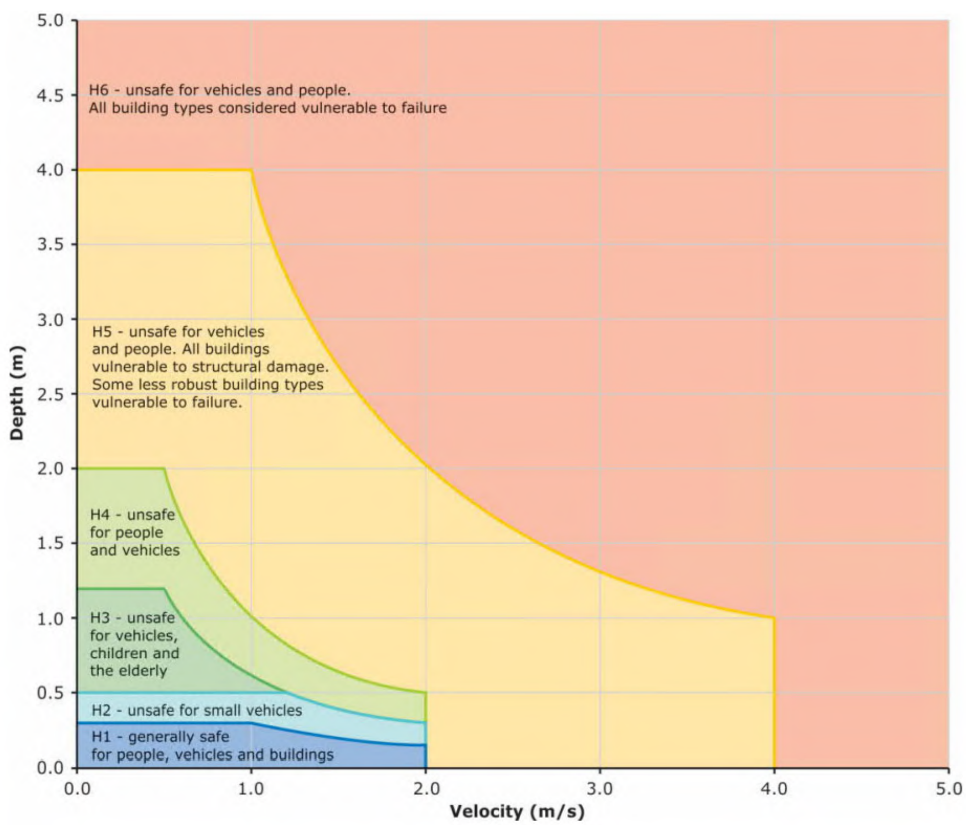

Figure 11.1 General Flood Hazard Vulnerability Curves (AIDR, 2017b)

Table 11.1 Hazard - vulnerability thresholds

Hazard Classification	Description
H1	Generally safe for vehicles, people and buildings.
H2	Unsafe for small vehicles.
H3	Unsafe for vehicles, children and the elderly.
H4	Unsafe for vehicles and people.
H5	Unsafe for vehicles and people. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure.
H6	Unsafe fir vehicles and people. All building types considered vulnerable to failure.

11.2 Flood Function

The NSW Floodplain Development Manual (DIPNR, 2005) recognises the following three hydraulic categories of flood prone land based on their flood function:

- Floodways;
- Flood Storage; and
- Flood Fringe.

There is no quantitative definition of these three categories or standard approach to differentiate between the classifications. As such, the delineation of these areas can be subjective and is based on knowledge of the catchment and previous experience in categorising flood function.

The criteria used to determine the hydraulic categories for this study are based on BMT's experience of similar catchments. These criteria are set out in Table 11.2 and have been used to map the floodplain in terms of its flood function for the 1% AEP and the 0.2% AEP events. The resulting maps are included in the Design Results Mapping Addendum.

Table 11.2 Criteria for Hydraulic Categories (flood function)

Hydraulic Category	Criteria
Floodway	1. Velocity x depth > 0.25m^2/s AND velocity > 0.25m/s; OR 2. Velocity > 1m/s; OR 3. Depth > 1m
Flood Storage	Areas outside of floodway where depth > 0.5m
Flood Fringe	Remaining area of floodplain not defined as Floodway or Flood Storage

11.3 Flood Warning and Emergency Management

The Australian Disaster Resilience (ADR) Handbook 7 (AIDR, 2017a) recommends the classification of the floodplain in a way that informs emergency response management. Three levels of floodplain classification are set out in the guideline based on evacuation/isolation considerations. These are summarised in Table 11.3.

Table 11.3 Floodplain Classifications for Emergency Response

Classification Category	Consideration
Primary	Is an area flooded in the PMF?
Secondary	Does an area have a flood free exit to community evacuation facilities during a flood
Tertiary	Six further classifications based on factors including consideration if whether all of the isolated area becomes fully submerged in a PMF

The full tertiary classification requires details and analysis of community evacuation facilities and evacuation routes and is beyond the scope of a flood study[11]. Outputs from this study have been processed to inform the basis of the tertiary classification by identifying land which can become isolated during flood events due to becoming surrounded by floodwater. These 'flood islands' have then been further categorised as a low flood island or high flood island. A low flood island is an island which eventually becomes fully inundated if floodwaters continue to rise. A high flood island is where part of that island remains dry in a PMF event.

Mapped outputs from the assessment showing the low and high flood islands are contained within the accompanying Design Results Mapping Addendum. The assessment uses all modelled design events (5% AEP to the PMF) when determining flood islands. The extent of each flood island shown in the mapping is based on that resulting from the smallest (most frequent) flood event at which the area becomes an island.

11.4 Support for Land Use Planning

The flood study results have been used to revise the Flood Planning Level (FPL) across the region. The Richmond Valley LEP 2012 (clause 6.5) adopts the 1% (1 in 100) AEP flood event plus a 500mm freeboard as the FPL. In accordance with Council's instruction, the revised FPL is based on the updated 1% AEP flood with an allowance for climate change. The freeboard of 500mm is also retained.

FPL mapping is provided in the Design Results Mapping Addendum accompanying this report. When using this mapping it is important to note that the addition of the 500mm freeboard is not hydraulically modelled. It is instead added onto the hydraulically modelled 1% AEP with climate change peak flood level. This approach will not capture any change in hydraulic behaviour which may result from higher flood levels. A buffering technique was used to push out the raised flood surface so that it tied in with higher ground.

Overall it is recommended that use of a constant 500mm freeboard allowance is reviewed as part of any future floodplain management study. A constant value will not factor in the sensitivity of flood levels to uncertainties. For example, in constricted parts of the floodplain the water level might increase significantly with increases in flow whereas in broad open floodplains a large increase in flow may only result in a modest increase in flood level.

[11] These are typically undertaken as part of a floodplain risk management study.

12 Conclusions

The Richmond Valley Flood Study has developed new hydrologic and hydraulic models which have allowed the Richmond River catchment floodplains to be modelled and mapped to a greater extent and with more detail than in previous studies. The models were jointly calibrated to the flood events of January 2008, May 2009, March/April 2017 and February/March 2022. Overall a good calibration was achieved with the model replicating the timing and peak of the flood events.

Design flood modelling was undertaken for the 5%, 2%, 1% and 0.2% AEP events and for the probable maximum flood. An assessment of the 1% AEP event under a future climate was also undertaken.

Key outcomes from the study are summarised under the following sections.

Data Collection

- A digital elevation model (DEM) was developed for the study and used in the modelling. This DEM uses the latest available LiDAR datasets across the study area and also incorporates significant road upgrades such as the Pacific Highway Upgrade and the Woodburn to Coraki Road upgrades. The DEM includes bathymetry data for the majority of the tidal reaches of the Richmond River catchment.

- Key embankments such as levees and road and rail embankments were enforced in the model to ensure the crest elevations of these features are represented.

Hydrologic Modelling

- A new hydrologic model was developed of the whole Richmond River catchment. It was found that generally consistent hydrologic model parameters could be applied across the catchment with a notable exception being on Bungawalbin Creek where higher ‘lag’ values were required compared to other tributaries. In part this was expected due to the significant flood storage within the Bungawalbin Creek catchment and its attenuating effect on floods. It is difficult to investigate this further as the Bungawalbin Creek catchment has a sparse coverage of pluviographs.

- The hydrologic model was used to simulate design hydrology in accordance with ARR2019 methodology.

Hydraulic Modelling

- The new hydraulic model uses the Quadtree feature of the TUFLOW software which has allowed the model resolution to be varied across the floodplain. As such, the resolution varies from 40m to 5m with the majority of the floodplain at 20m and low lying urban areas at 5m. This is a significant improvement on the 60m grid used across much of the floodplain in the 2010 Flood Study.
- The extension of the model into the Bungawalbin Creek catchment and upstream of Casino to Fairy Hill has allowed for floodplain mapping of these areas when this previously did not exist.
- Two independent methods of design flow estimation were used as follows:
  - Application of design rainfall depths to the calibrated models
  - Flood frequency analyses that consider the full historical available record of flood peaks
- The flood frequency design flow estimates were found to be higher than those derived from modelling based on design rainfalls at Casino and East Gundurimba. The modelled flows were therefore reconciled with the flood frequency estimates though the application of scaling factors.

- The model downstream (ocean) boundary for design events was updated to reflect current state guidance. The 1% AEP catchment flood event is modelled in combination with a 5% AEP storm surge. The timing of the storm surge peak was set to coincide with the timing of the flood on the lower catchment. Overall it was found that the resulting flood levels within the Richmond Valley LGA were not sensitive to the assumptions of the storm surge timing except at Evans Head under a climate change scenario (with sea level rise).
- The hydraulic model was used to identify a subset of events to represent the design flood AEPs. Use of the hydraulic model (as opposed to the hydrologic model) for this purpose is considered a robust approach and one that was required for the complex Richmond River floodplain. The finalised subset of events representing different AEPs ensures the model remains a practical tool for future floodplain assessments.
- Design flood levels are tabulated at key gauge locations and mapped across the study area. An overall finding is that design flood levels are typically similar to, or slightly higher, than those previously adopted.
- The climate change assessment incorporates sea level rise of 0.9m in accordance with state guidelines. Rainfall intensity is also increased by 10%. This corresponds to Councils ‘CC3’ scenario. The sea level rise and 10% increase in rainfall were applied to the 1% AEP event as part of a climate change scenario. The most significant increases in 1% AEP flood levels under the climate change scenario are at Evans Head (0.85m increase), Broadwater (0.62m increase) and Woodburn (0.58m increase).

Additional Mapping and Outputs

- The flood study outputs were post processed to provide additional information beyond flood levels, depths and velocities. Additional post processed map outputs include:
  - Flood Hazard mapping which categorises the floodplain into six classes of hazard and is based on current best practice guidelines.
  - Flood Planning Level output. This uses the 1% AEP event with climate change as a basis for the mapping and applies a 500mm freeboard allowance.
  - Floodplain Function mapping which considers the hydraulic function of the floodplain in accordance with the NSW Floodplain Development Manual. This categorises the floodplain into floodway, flood storage and flood fringe.
  - Mapping to support flood warning and emergency management comprising of the delineation of flood islands which are areas that become isolated during floods.
  - Counts of properties predicted to have above floor inundation during design floods. This is tabulated by suburb.
  - Mapping showing locations where key roads are vulnerable to being cut by floodwater.

In summary, the modelling and resulting mapping allows for a consistent set of assumptions and standards to be applied across the Richmond Valley LGA with regards to flooding. It is recommended that the flood study results and post processed outputs underpin the basis of any future revisions to floodplain risk management plans.

13 References

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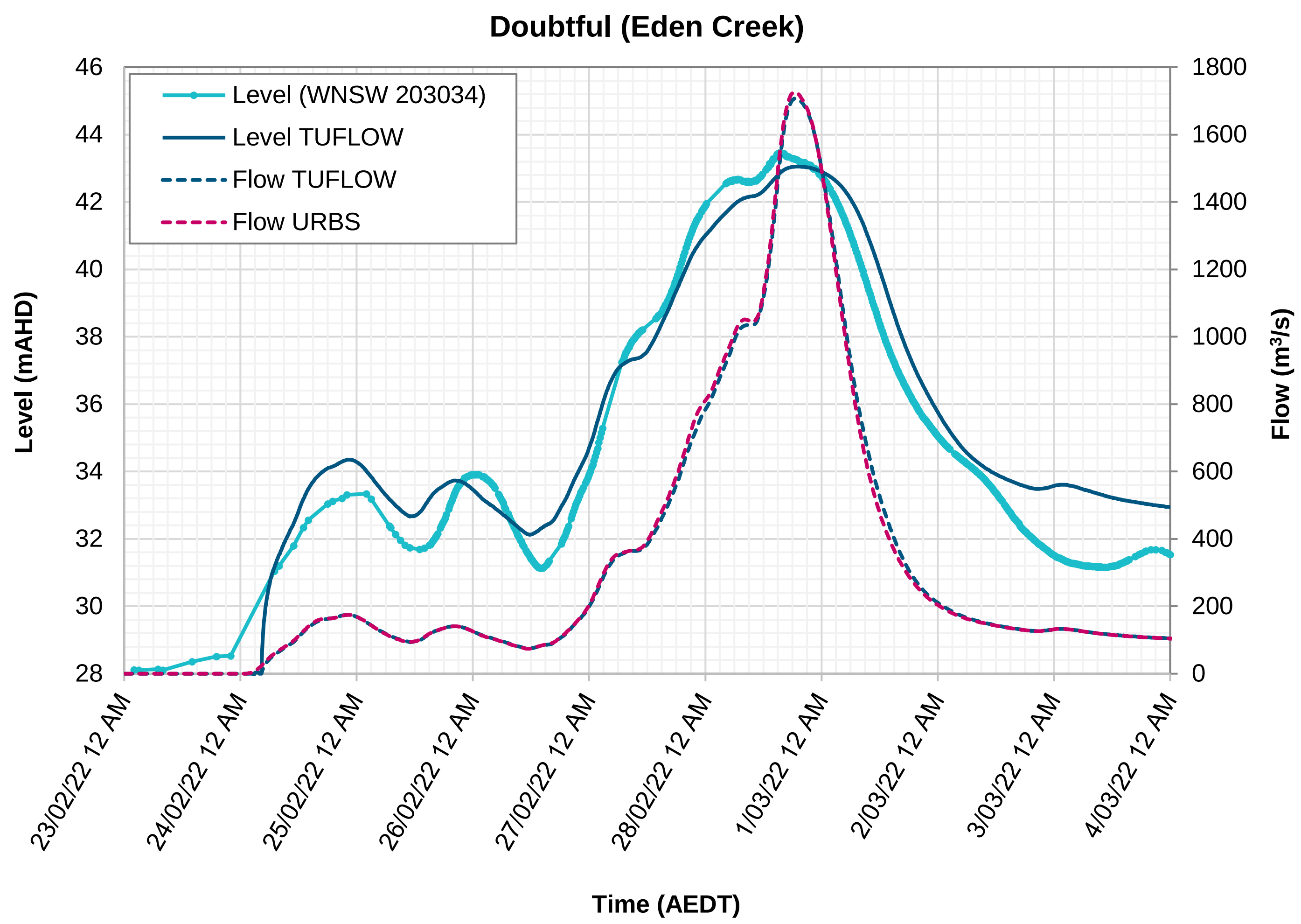
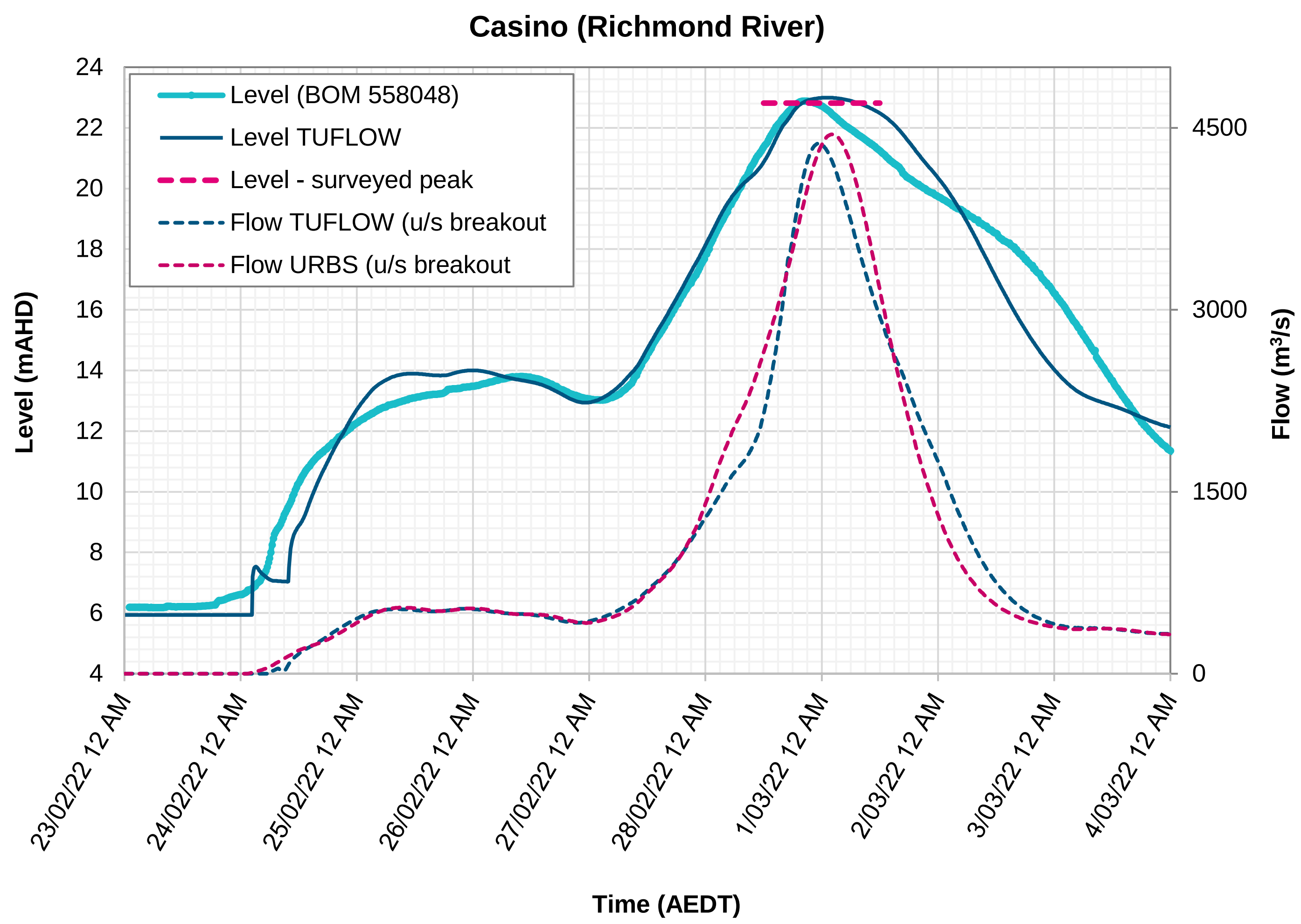
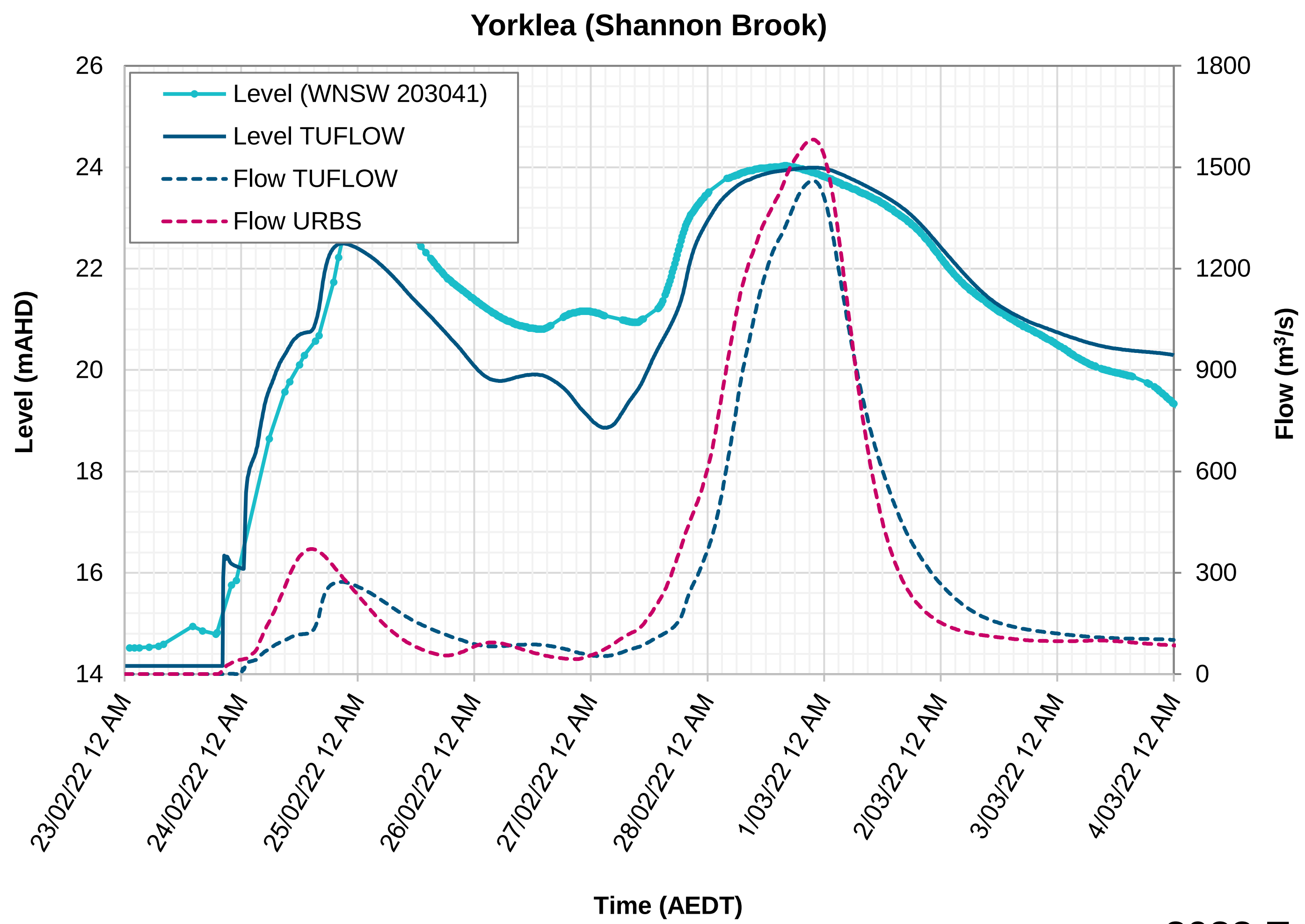
Annex A Calibration Plots

A.1 February-March 2022

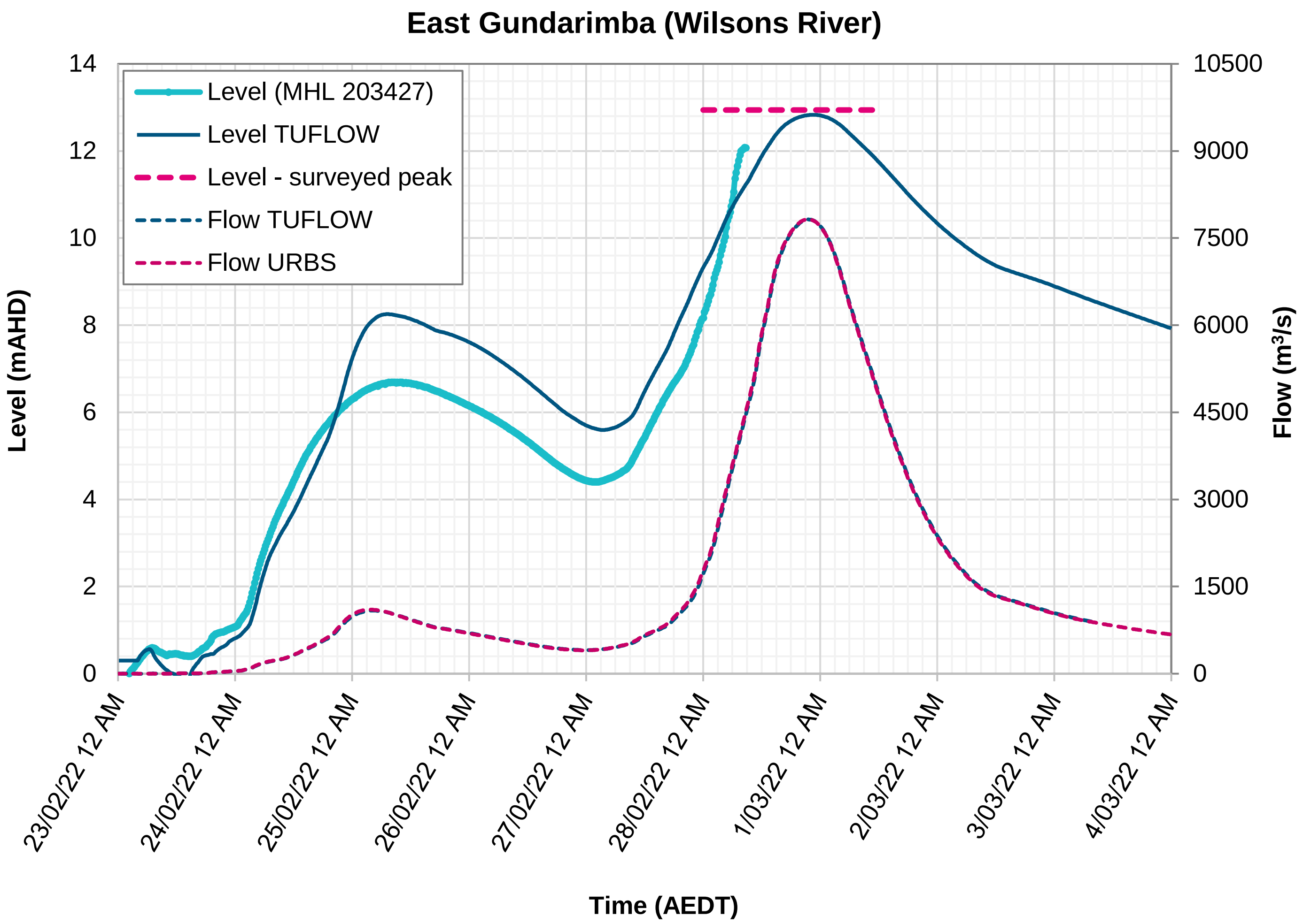


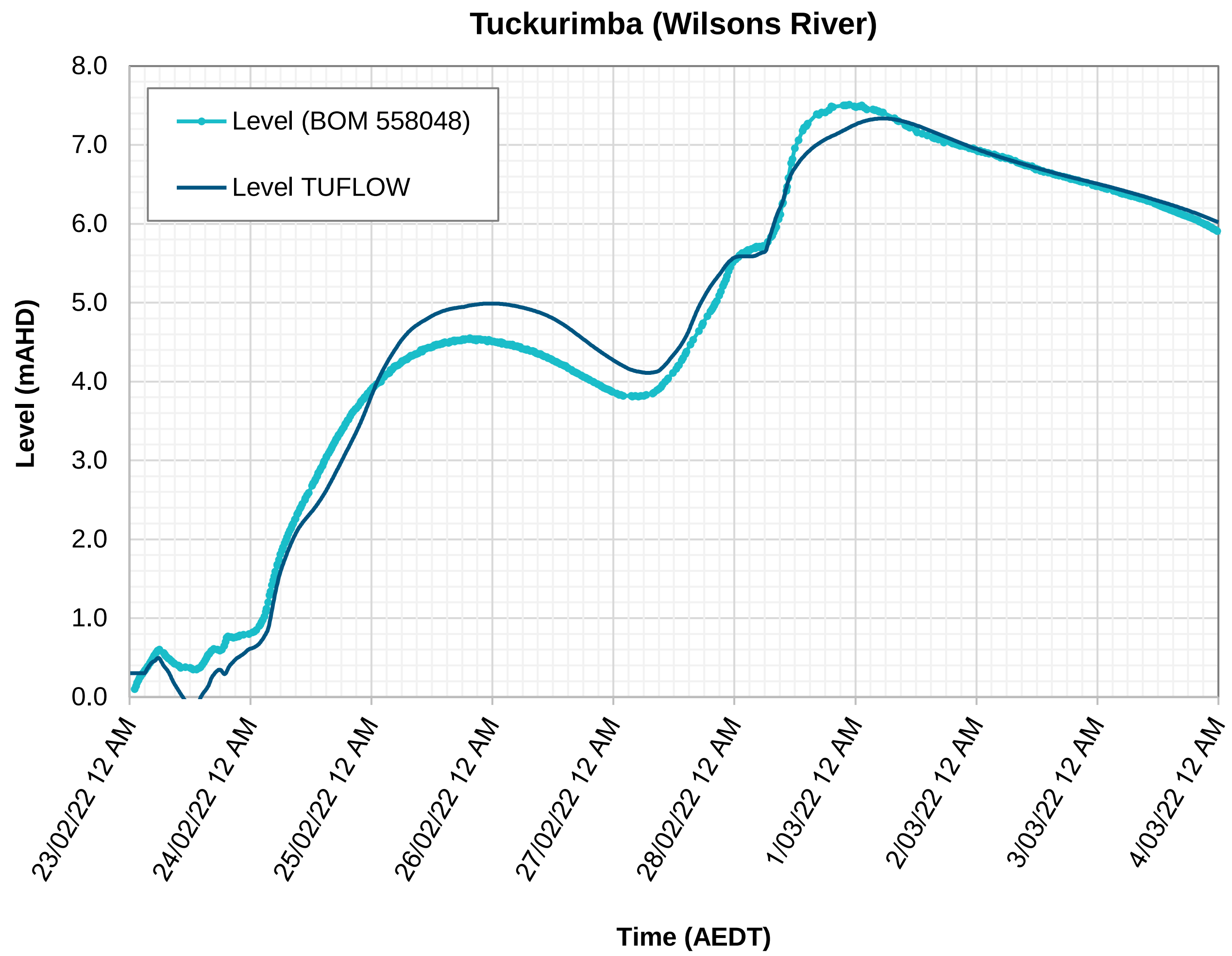


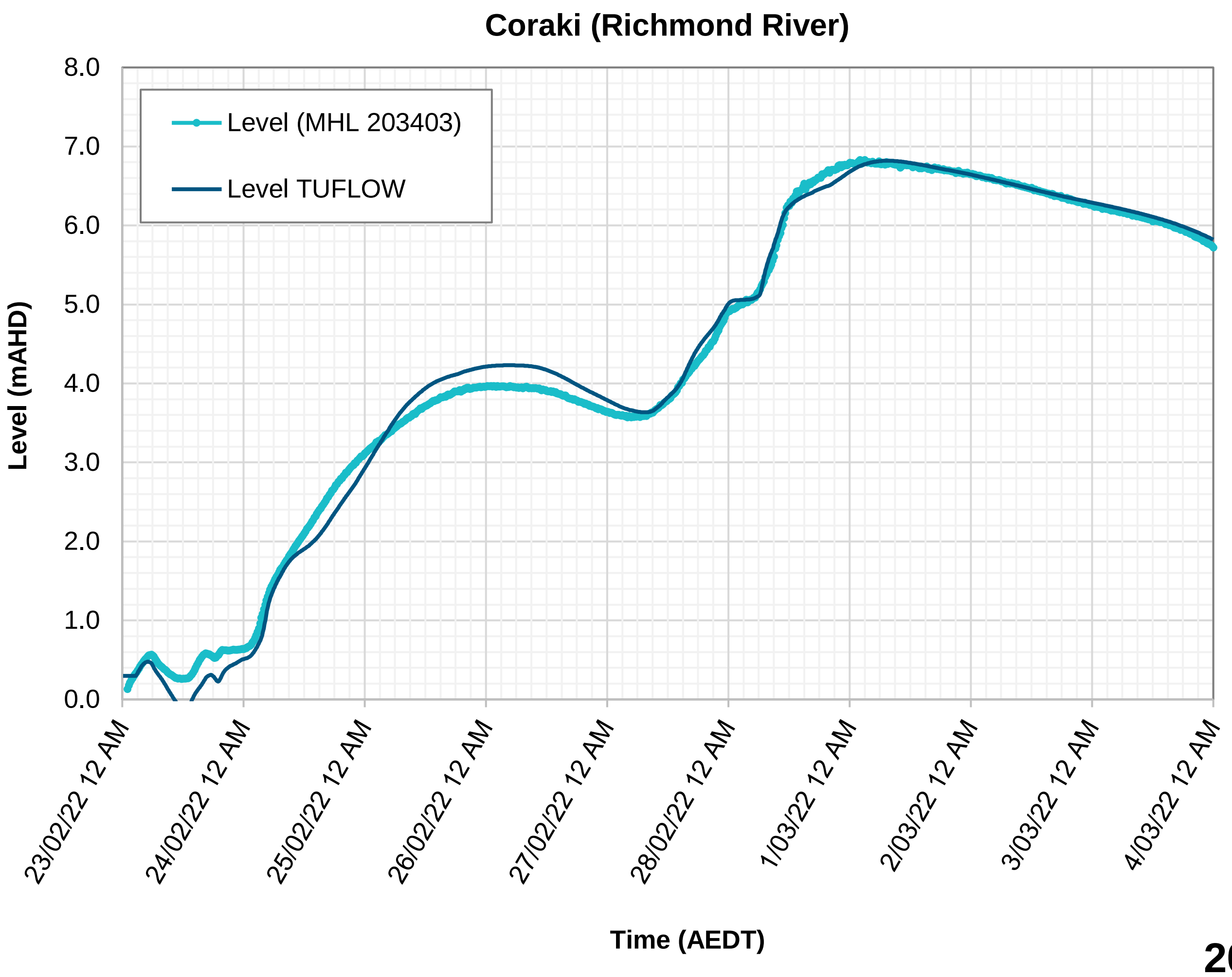


2022 Event Sheet 1

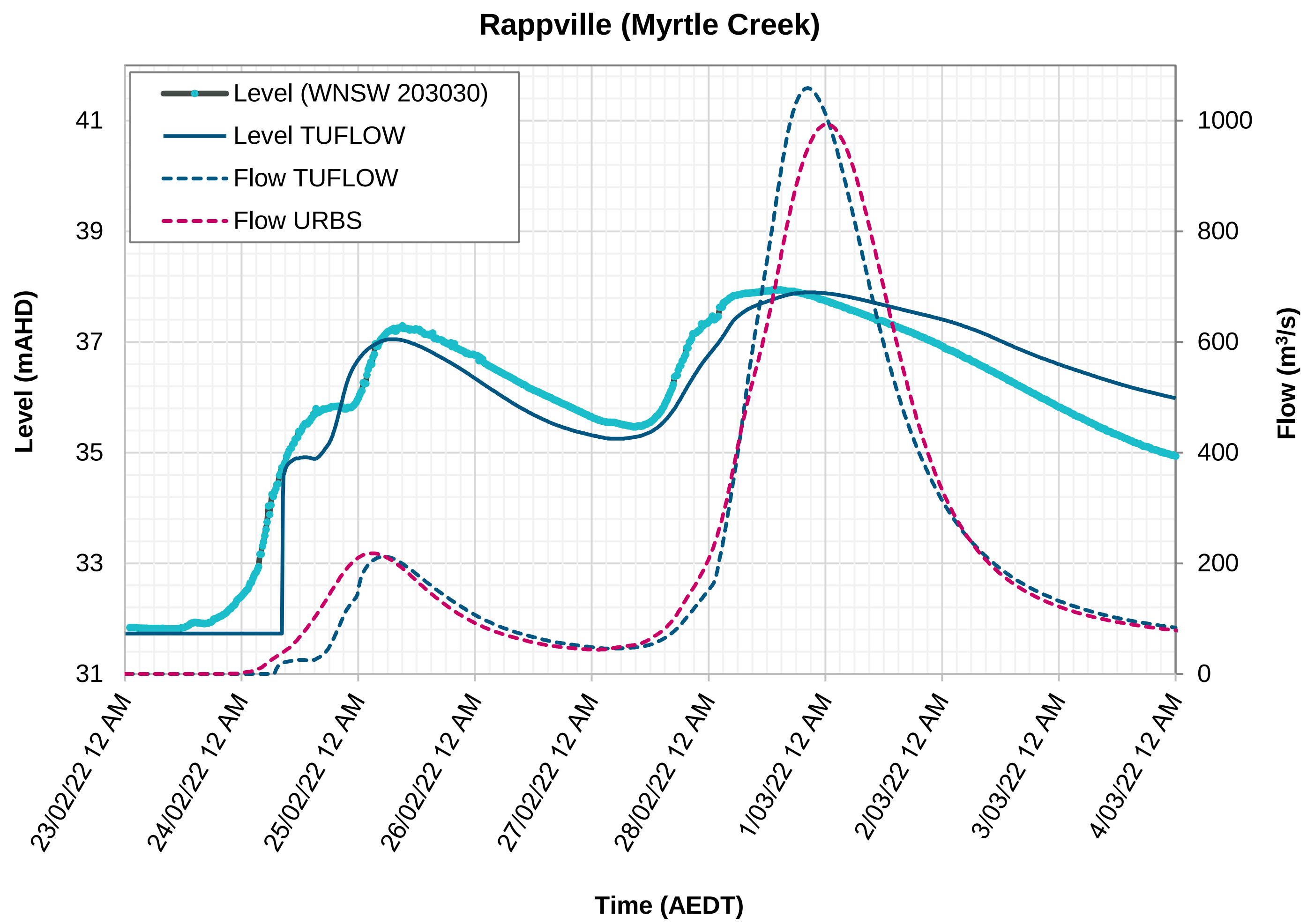
BMT

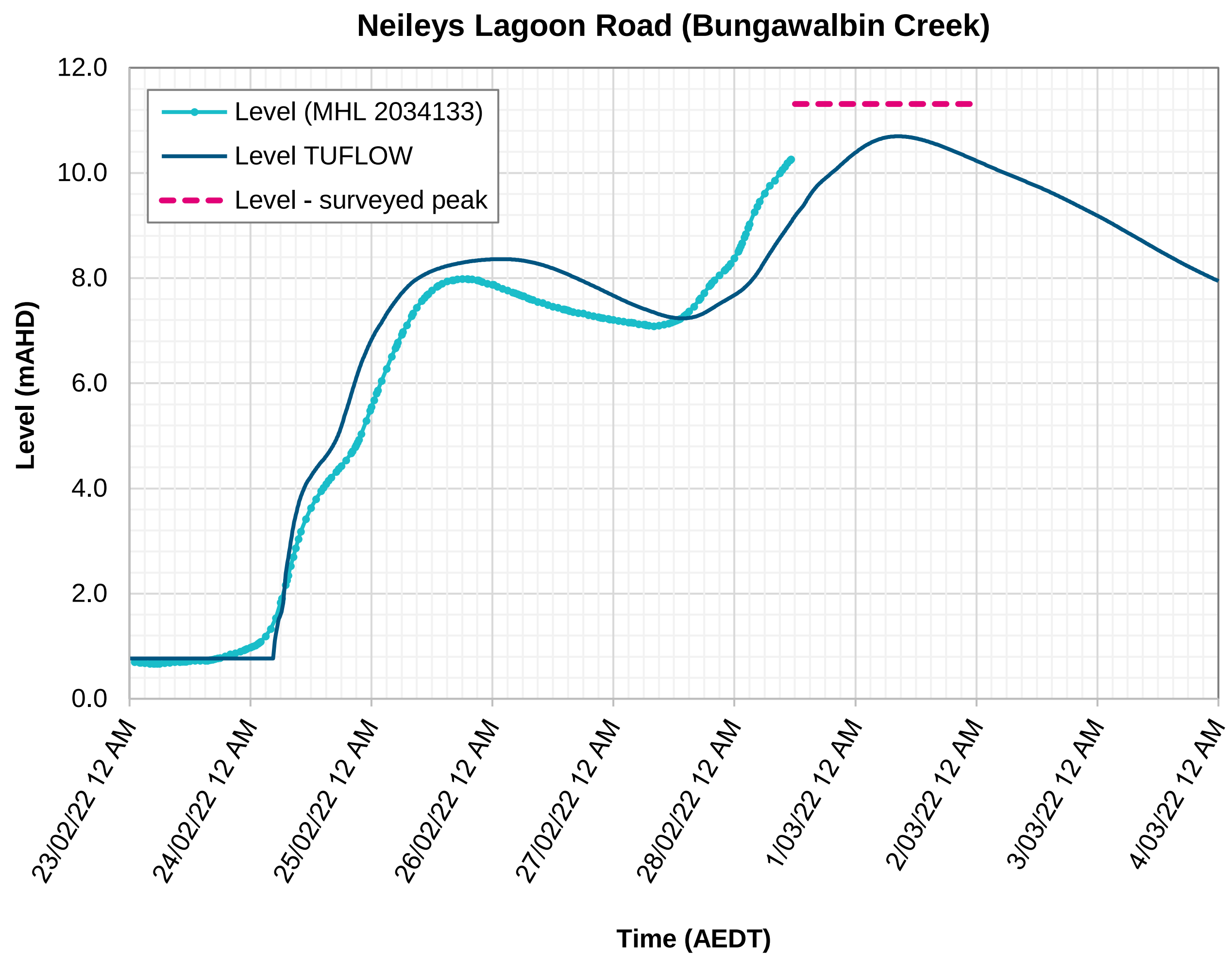


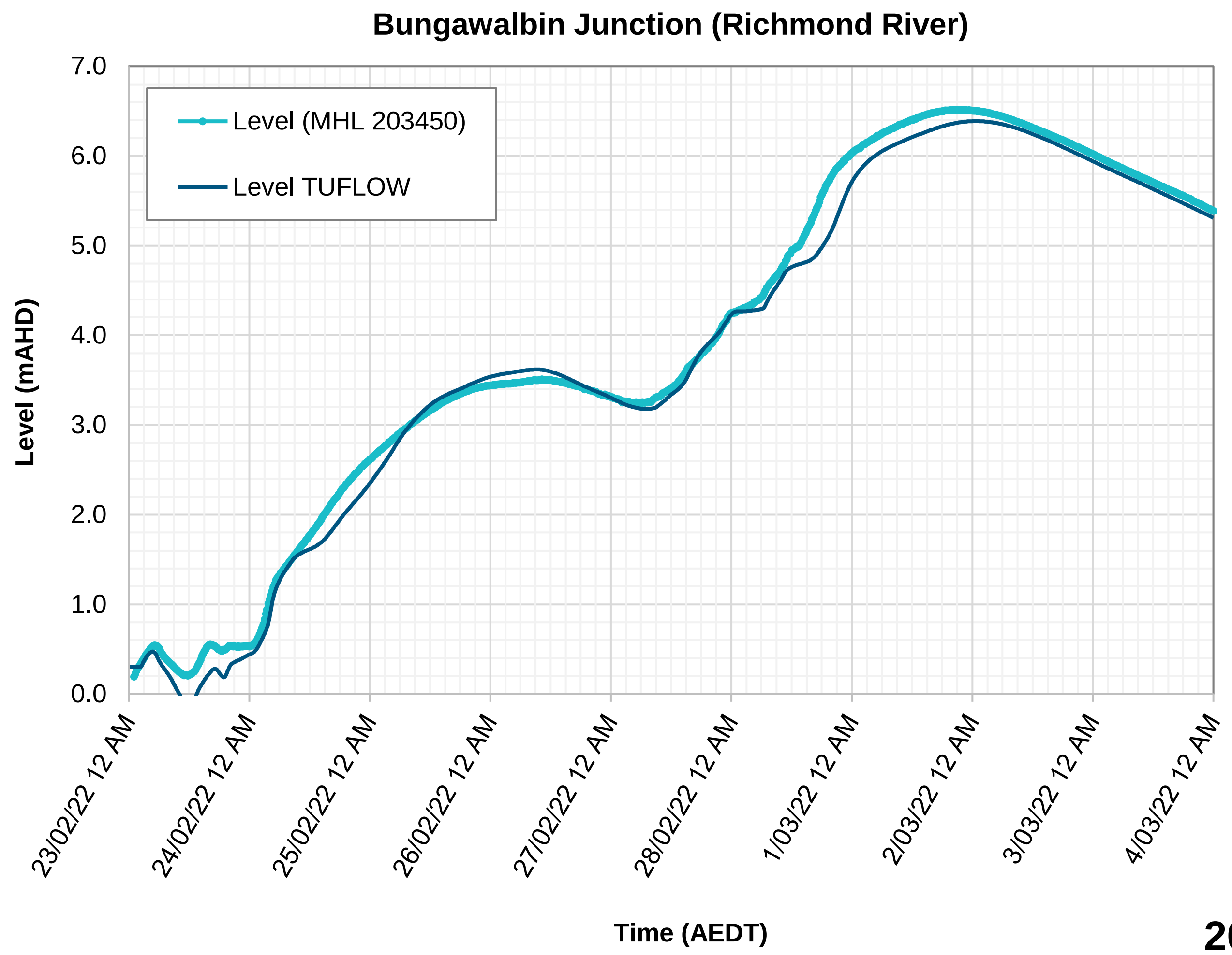


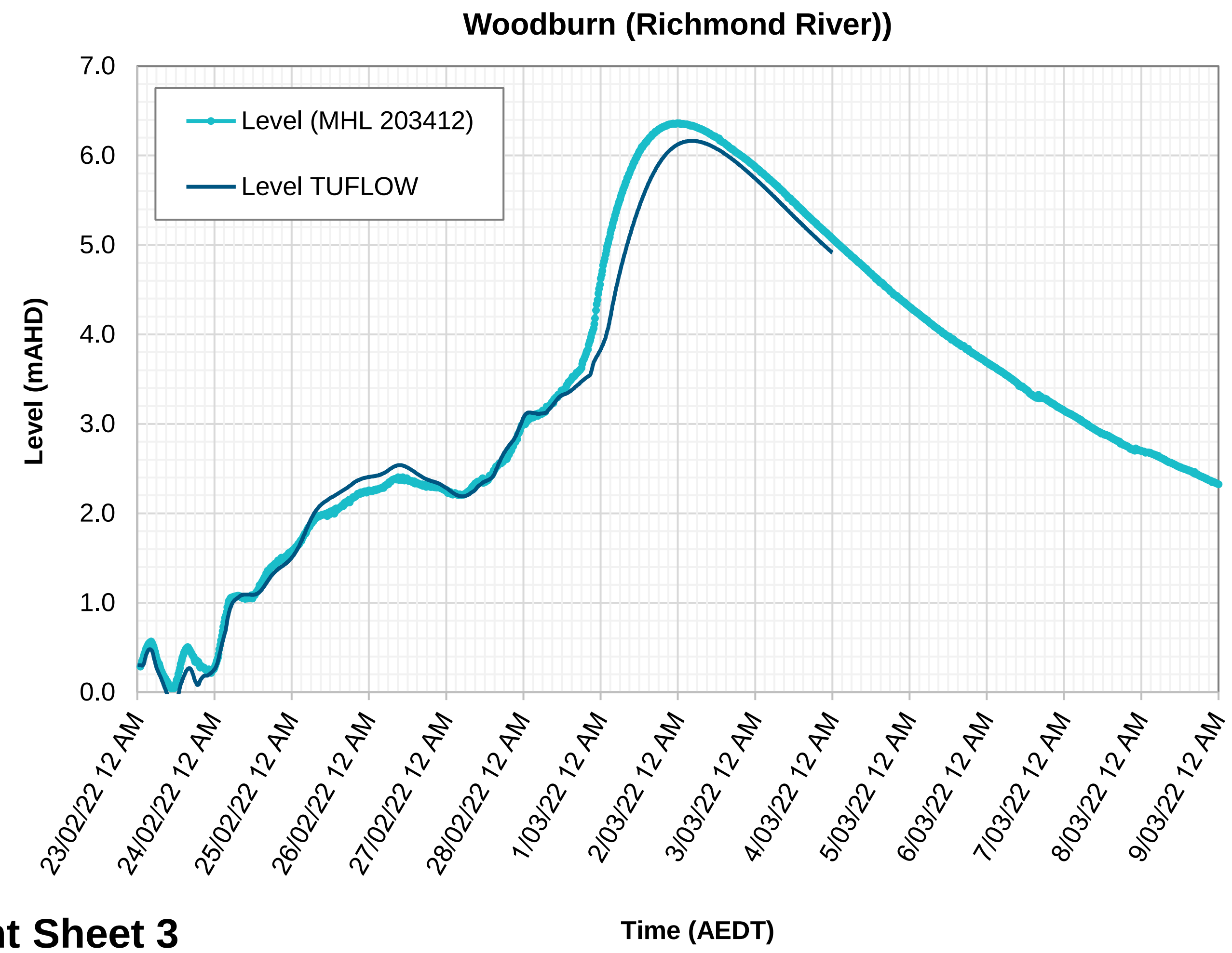


2022 Event Sheet 2

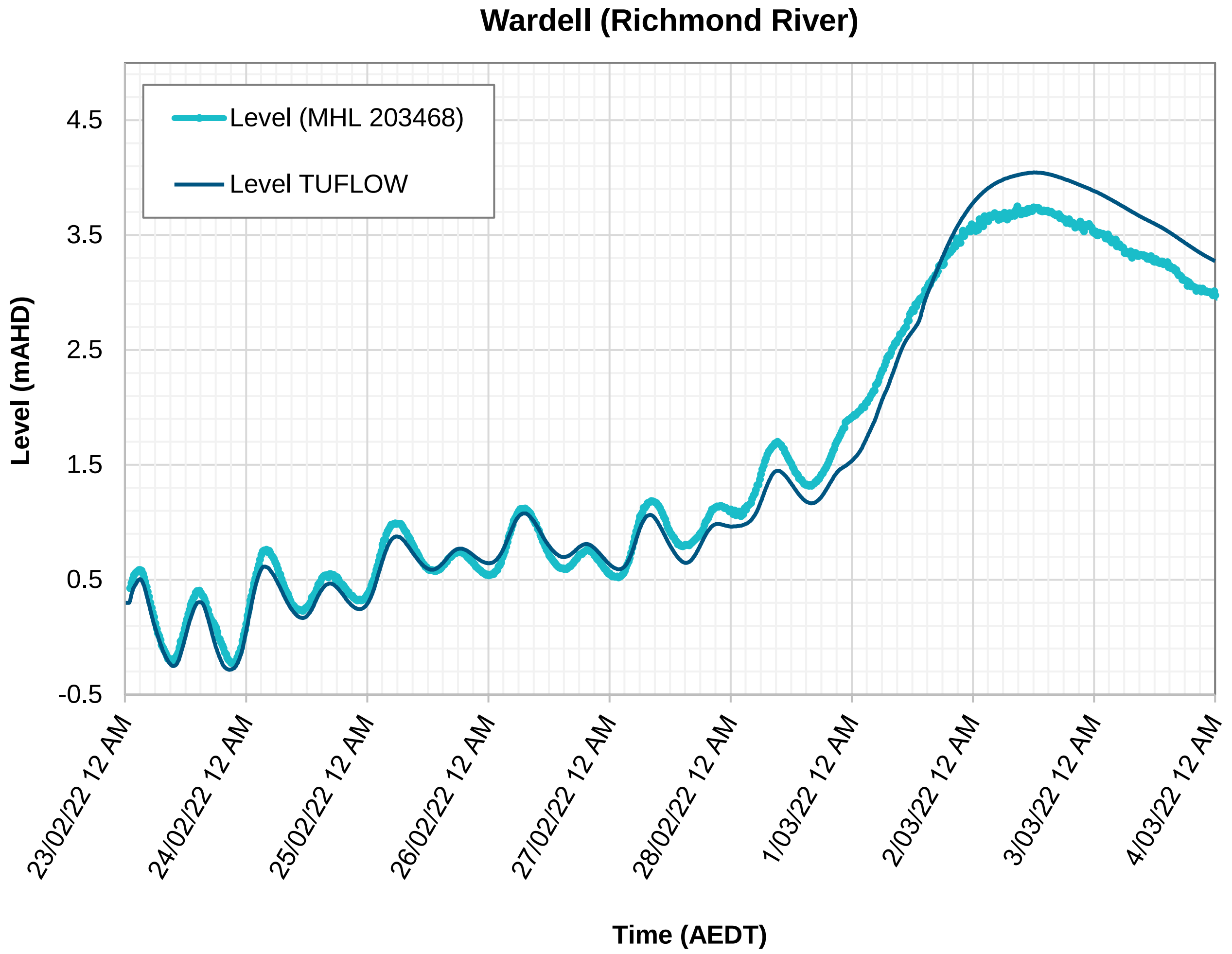


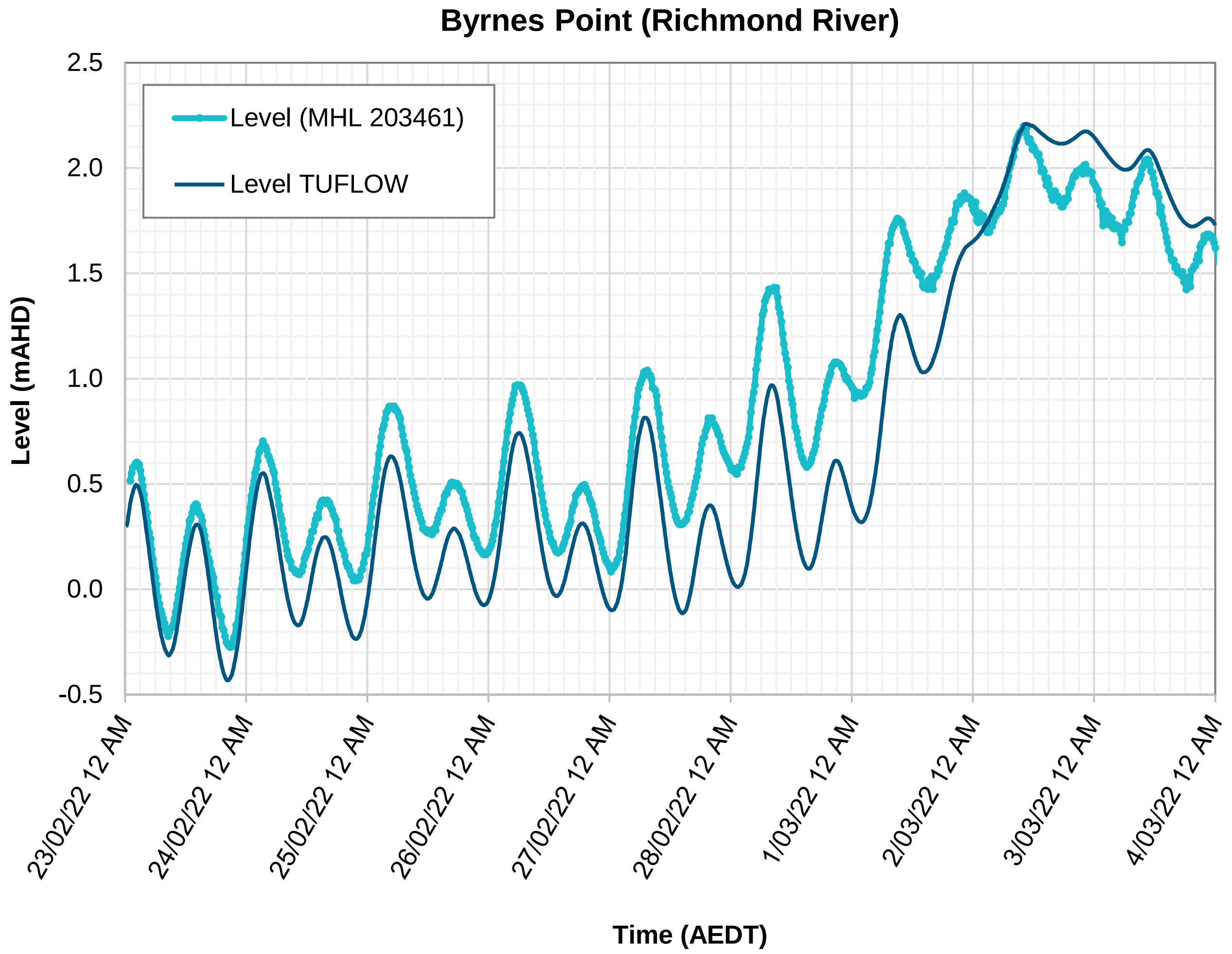


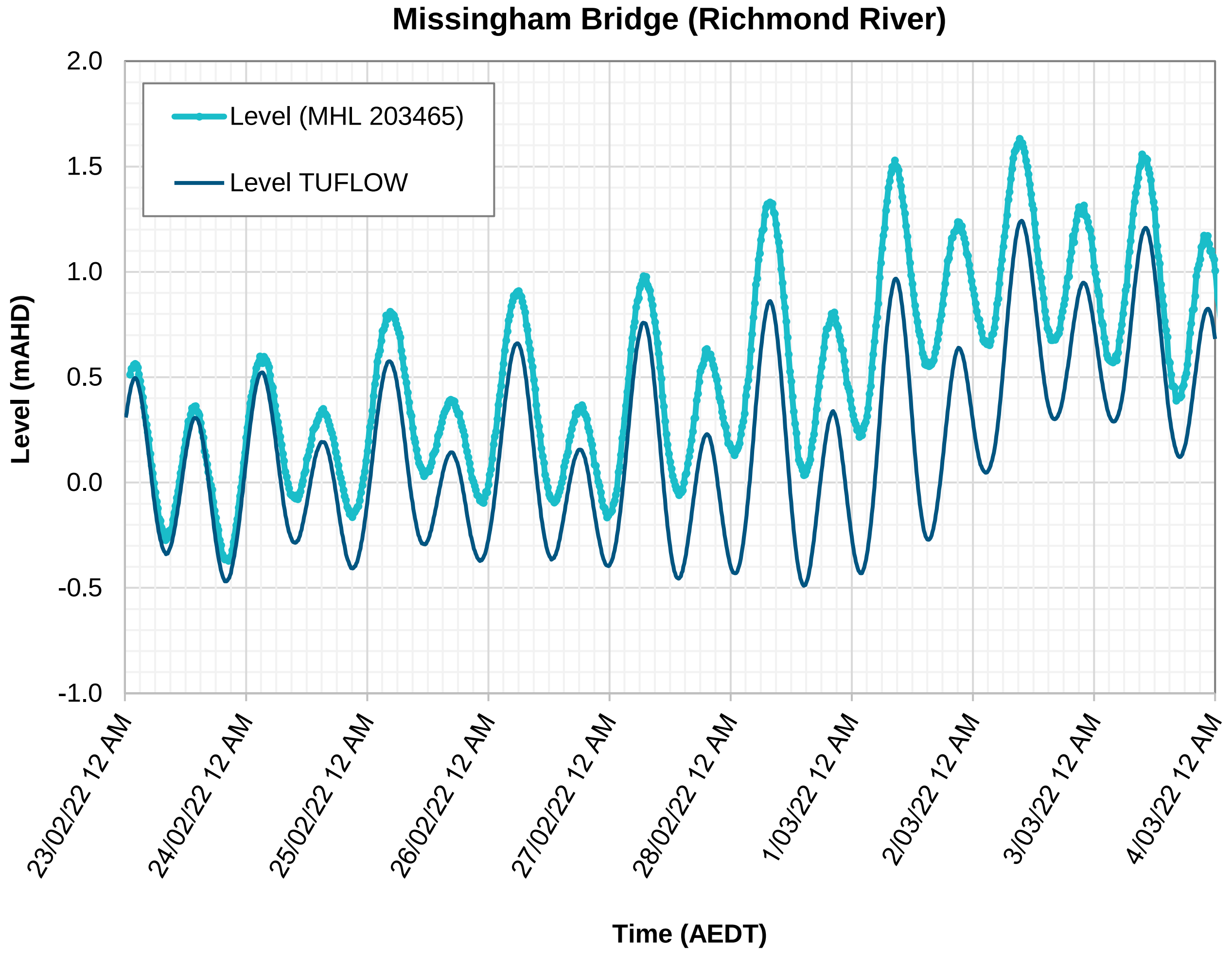


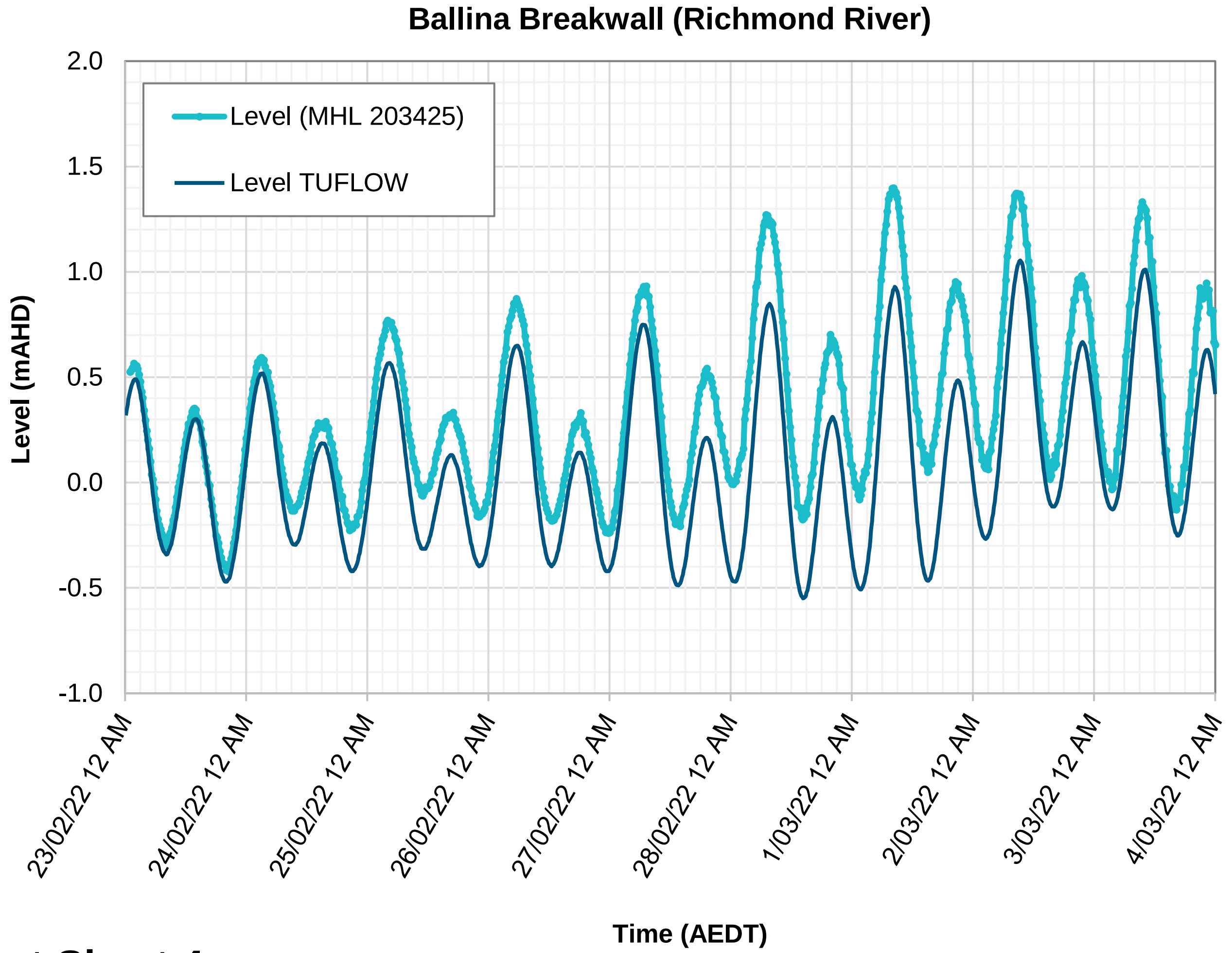


2022 Event Sheet 3

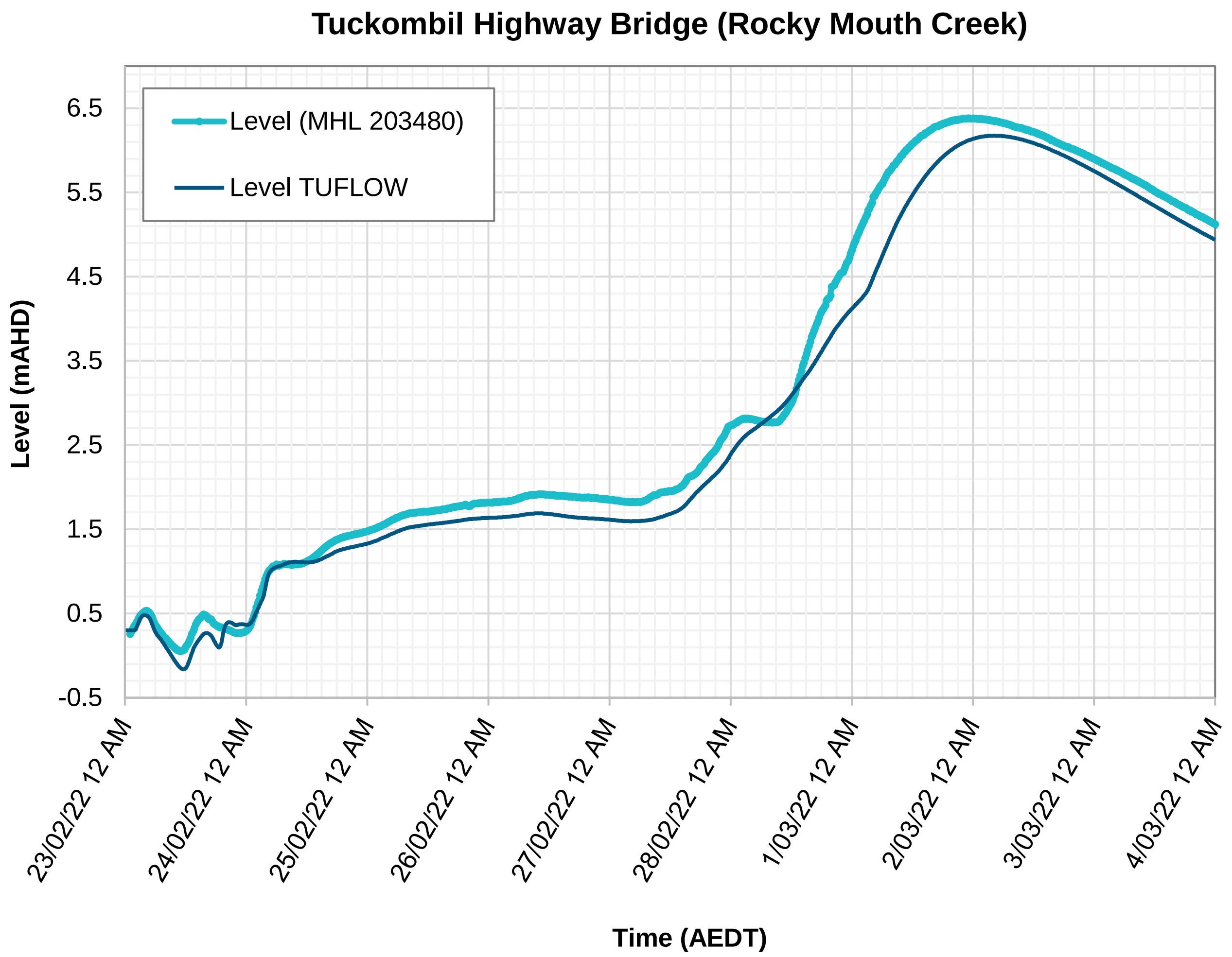


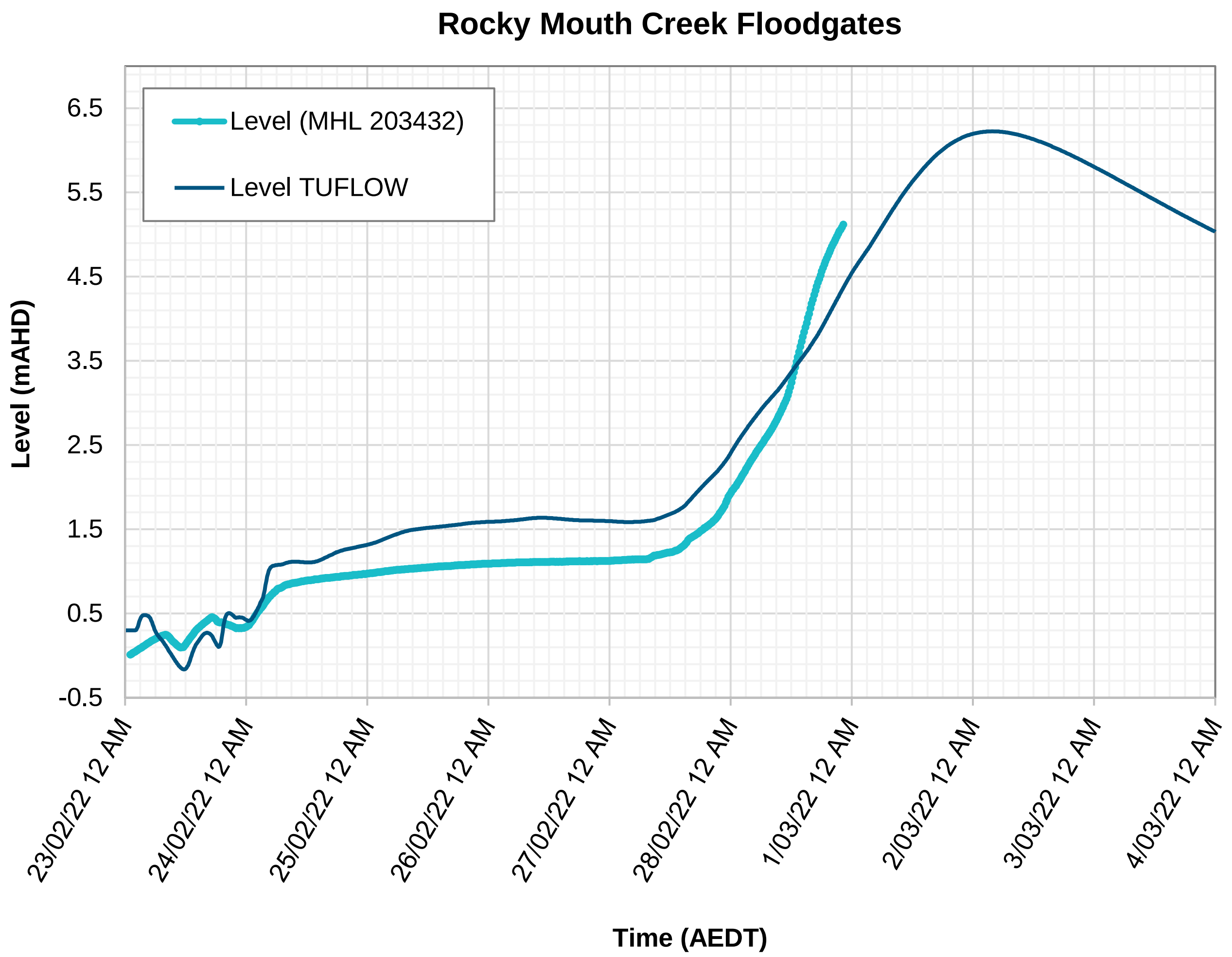


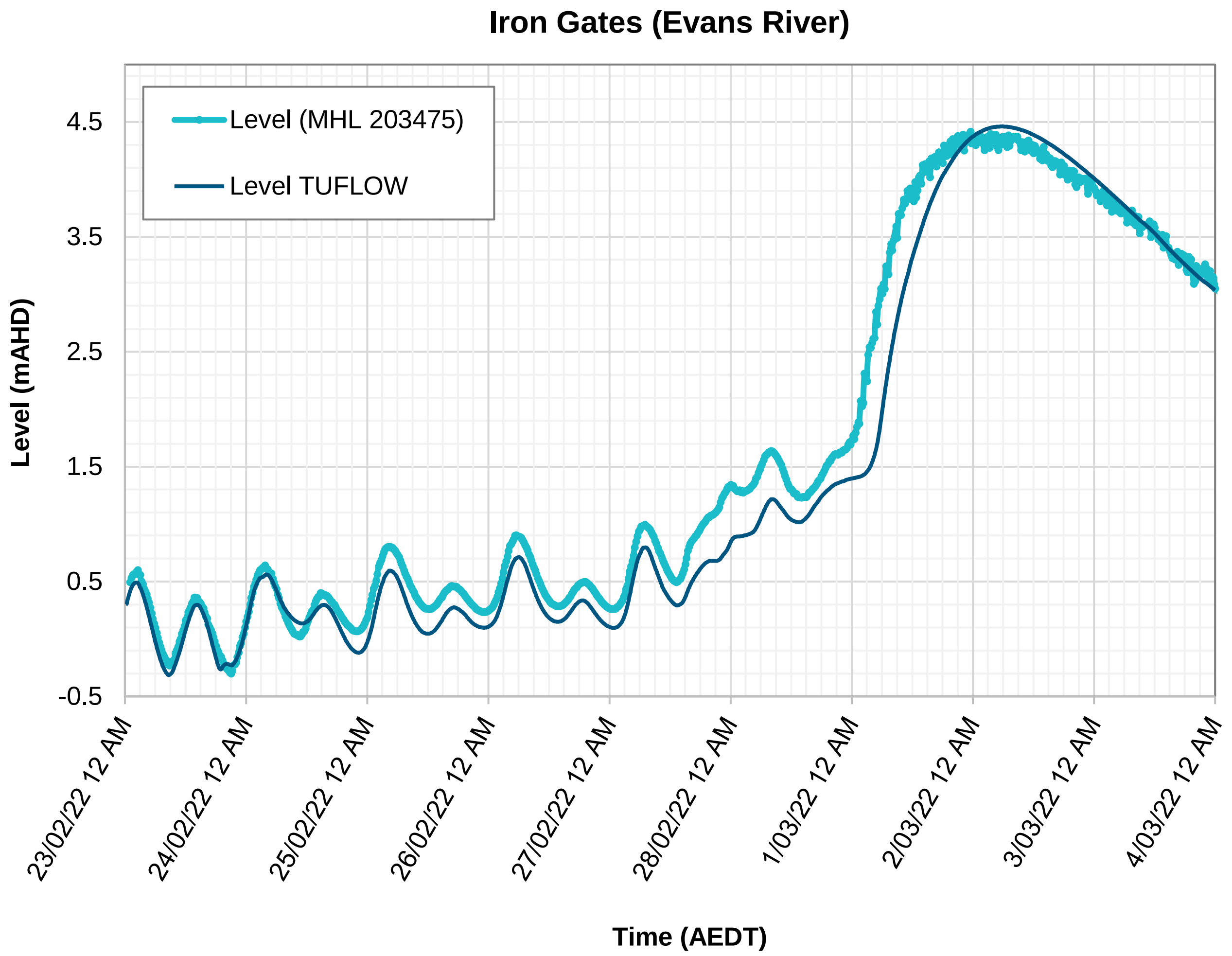


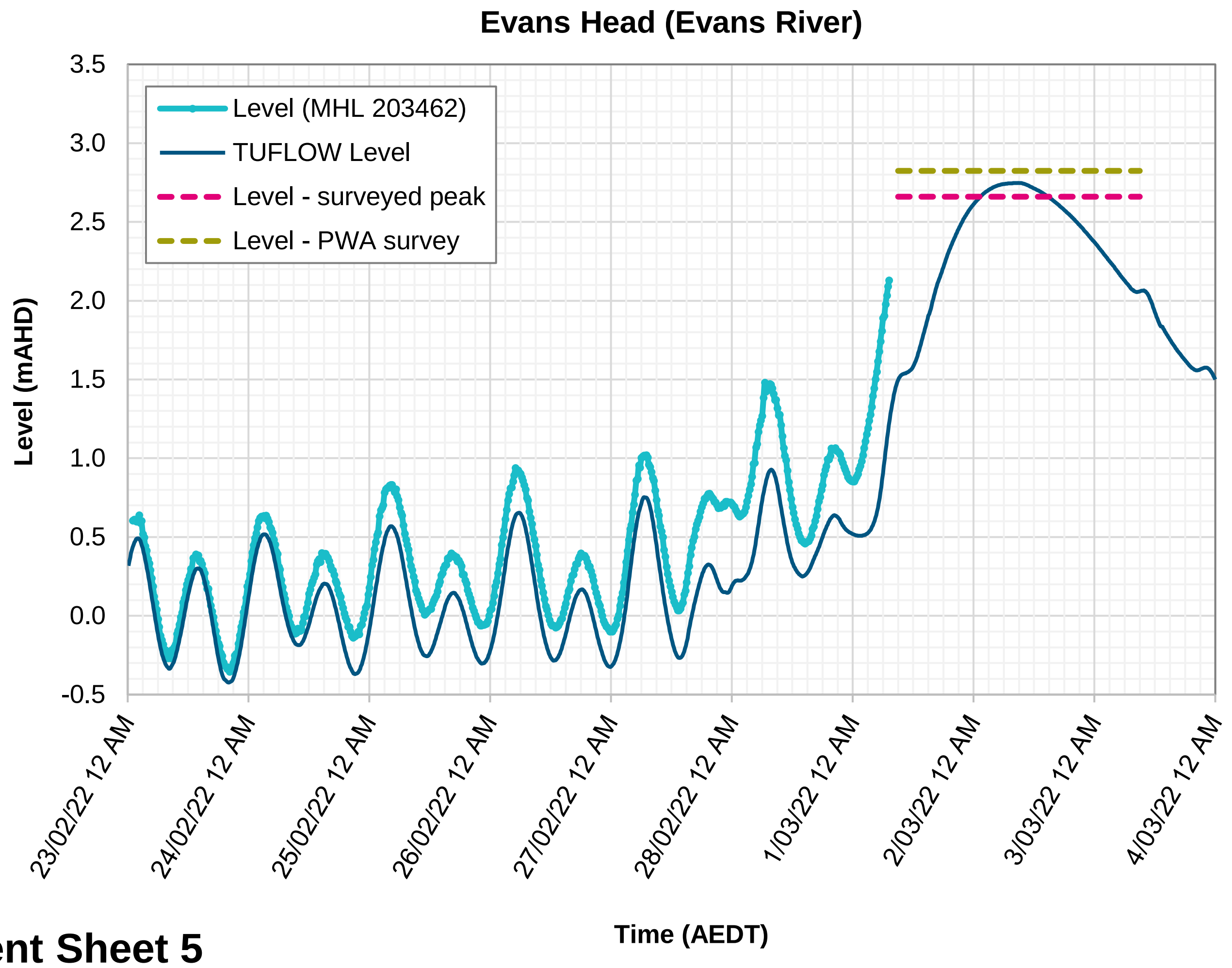


2022 Event Sheet 4









2022 Event Sheet 5

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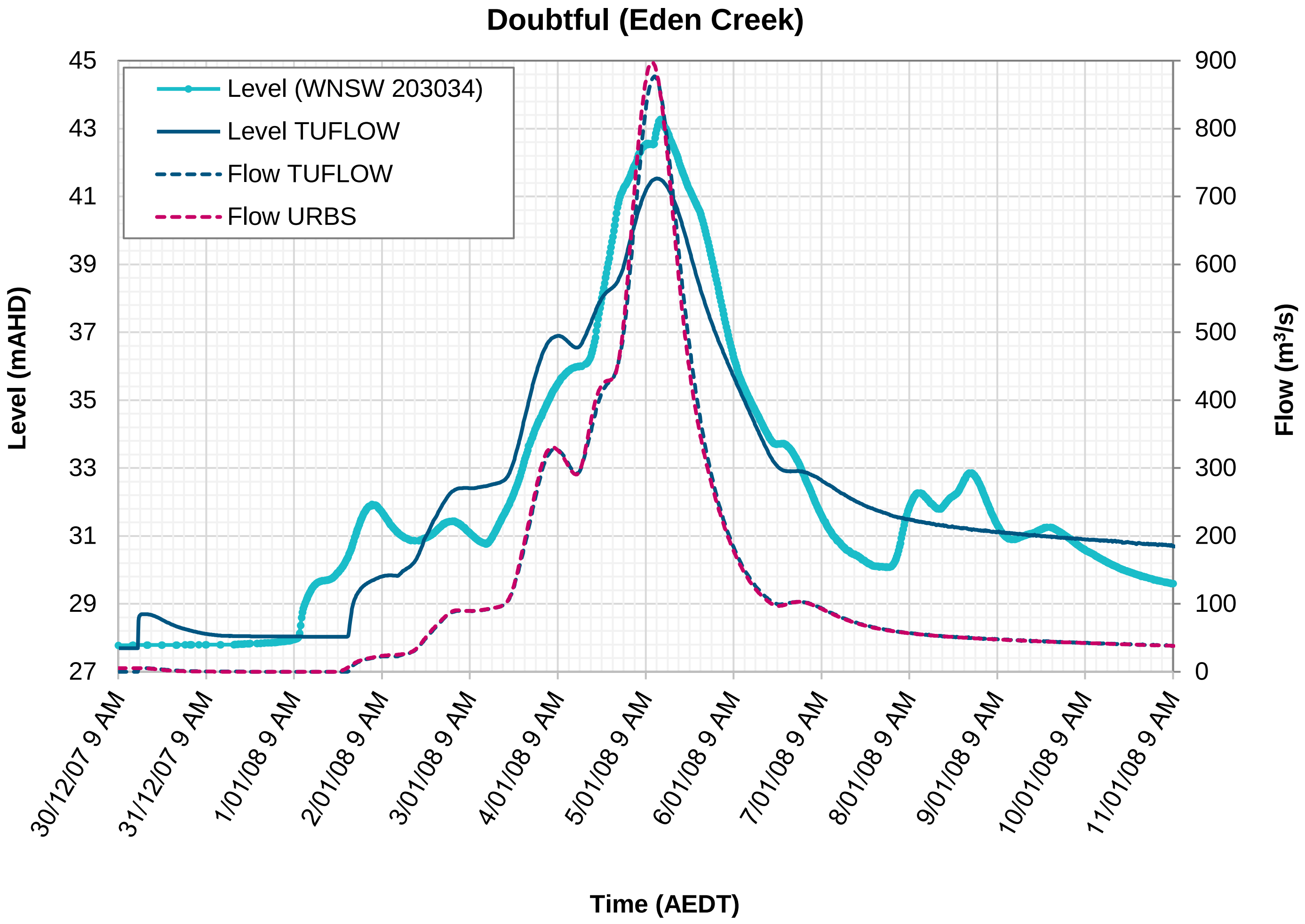

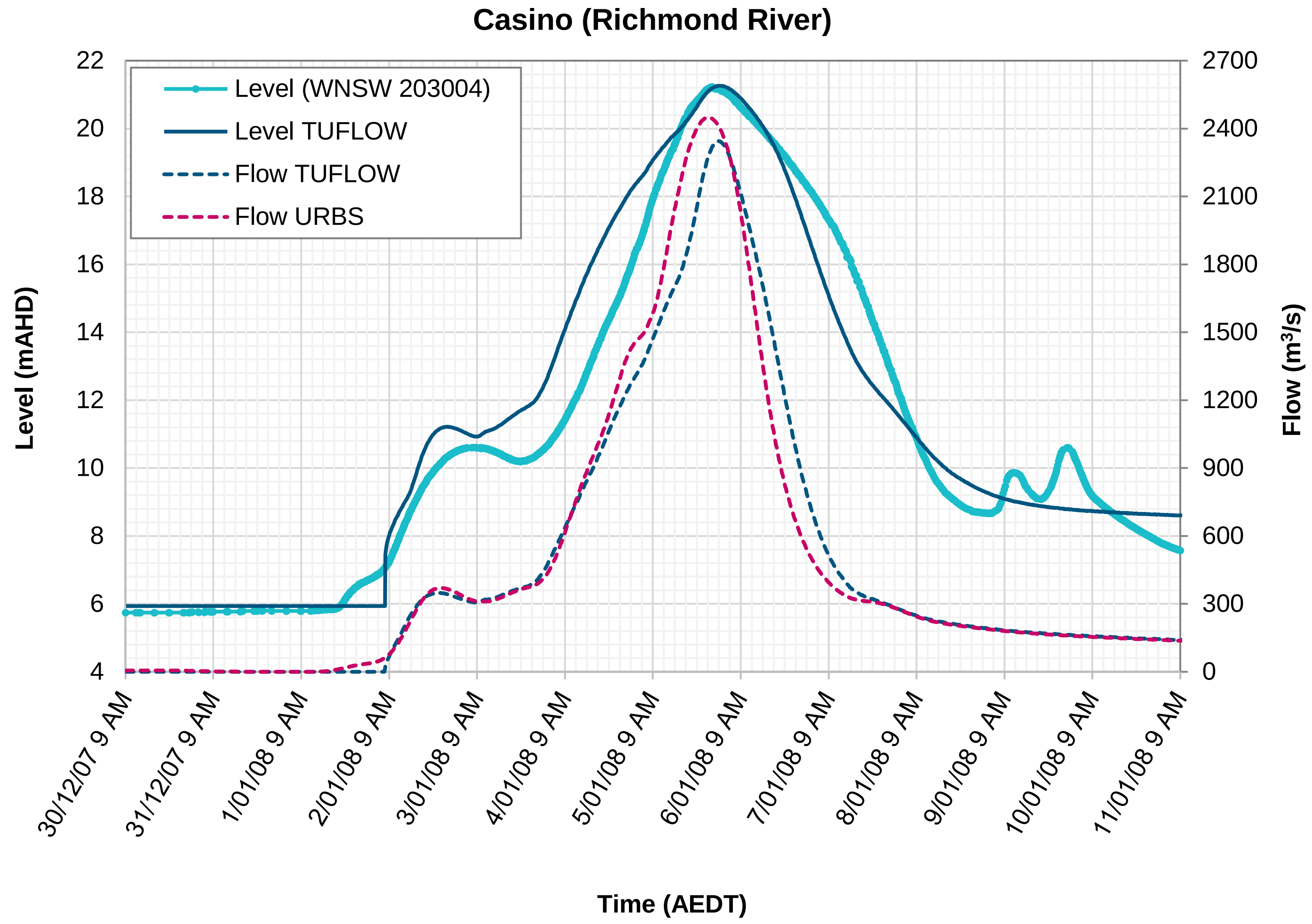

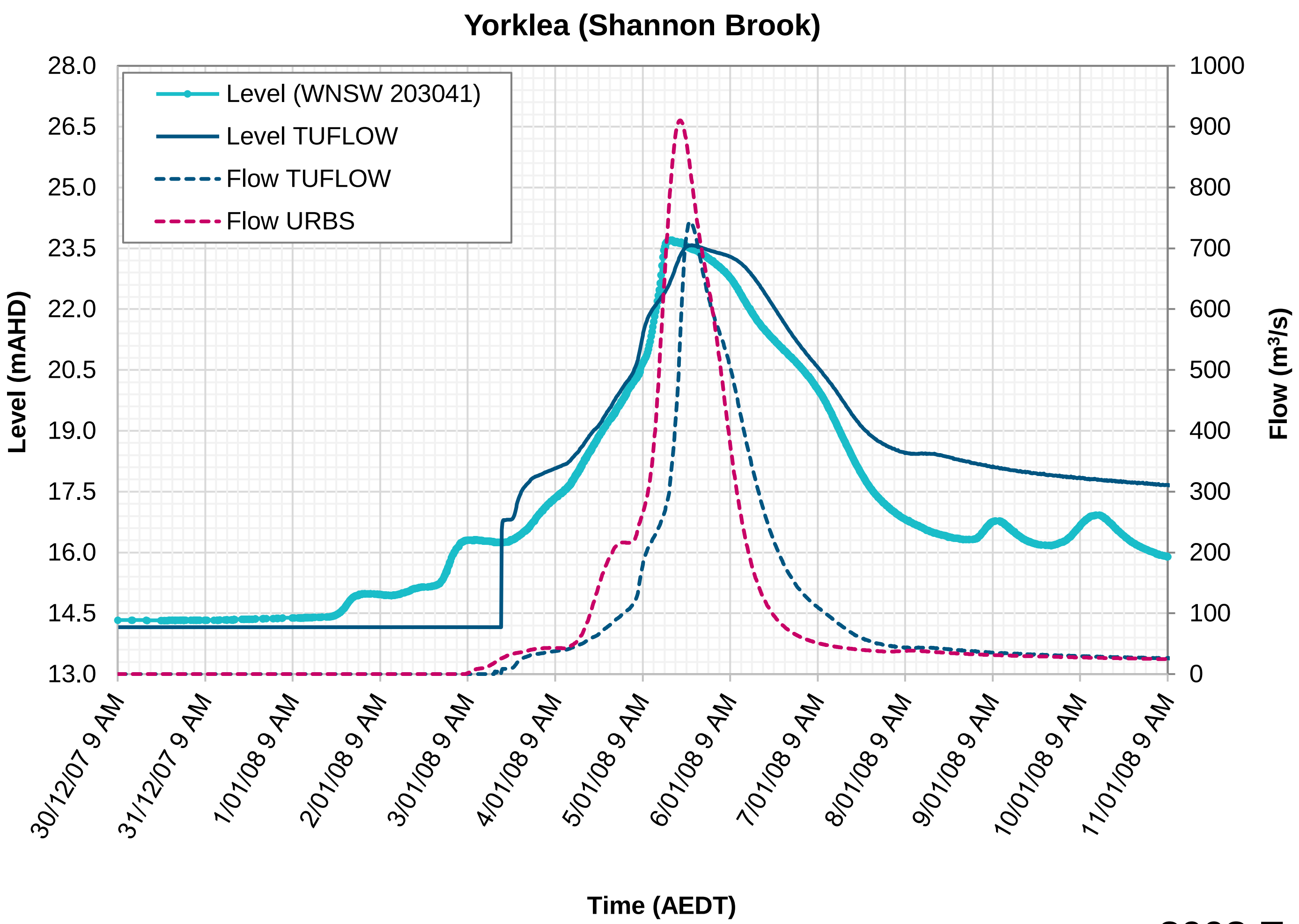

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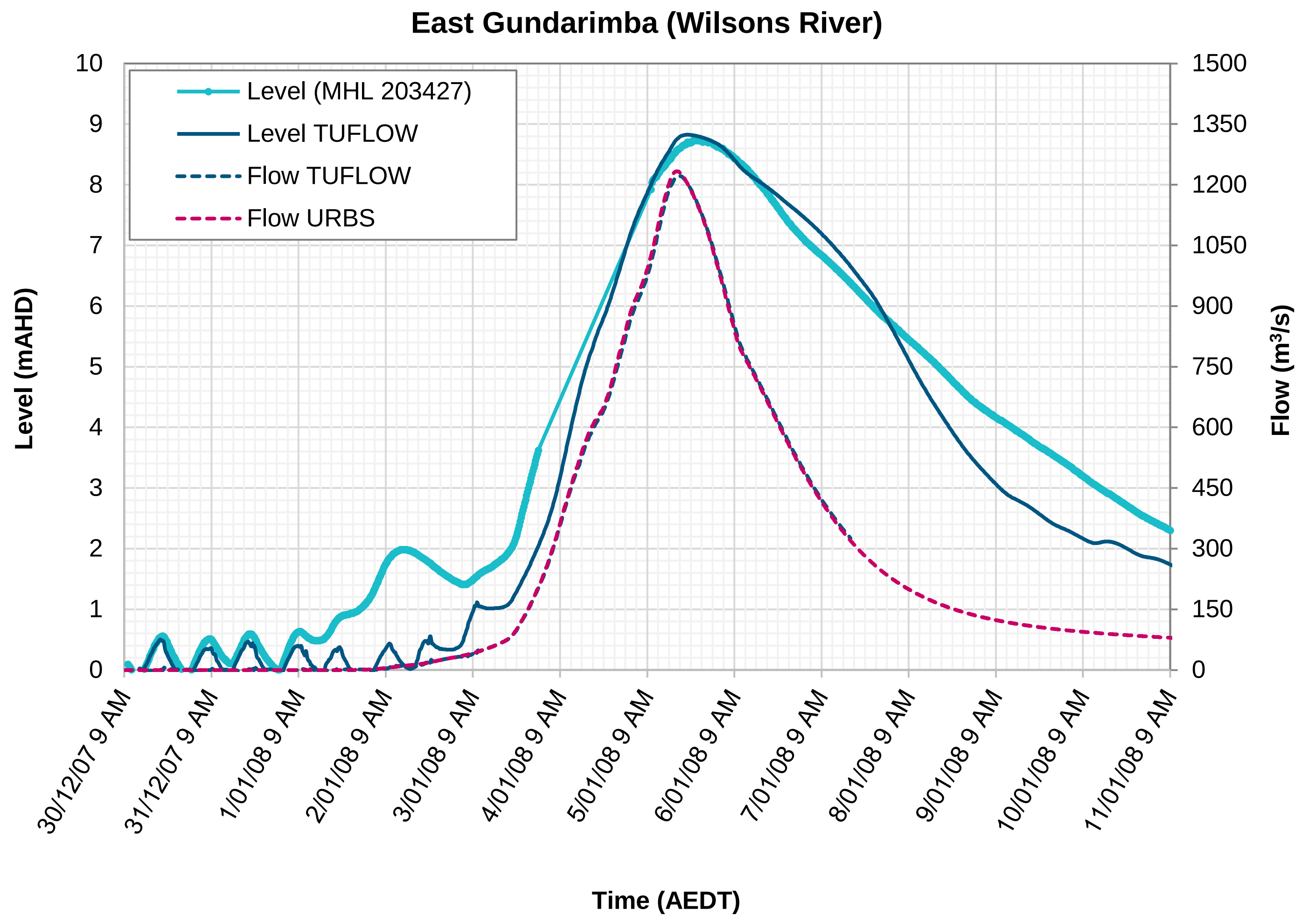
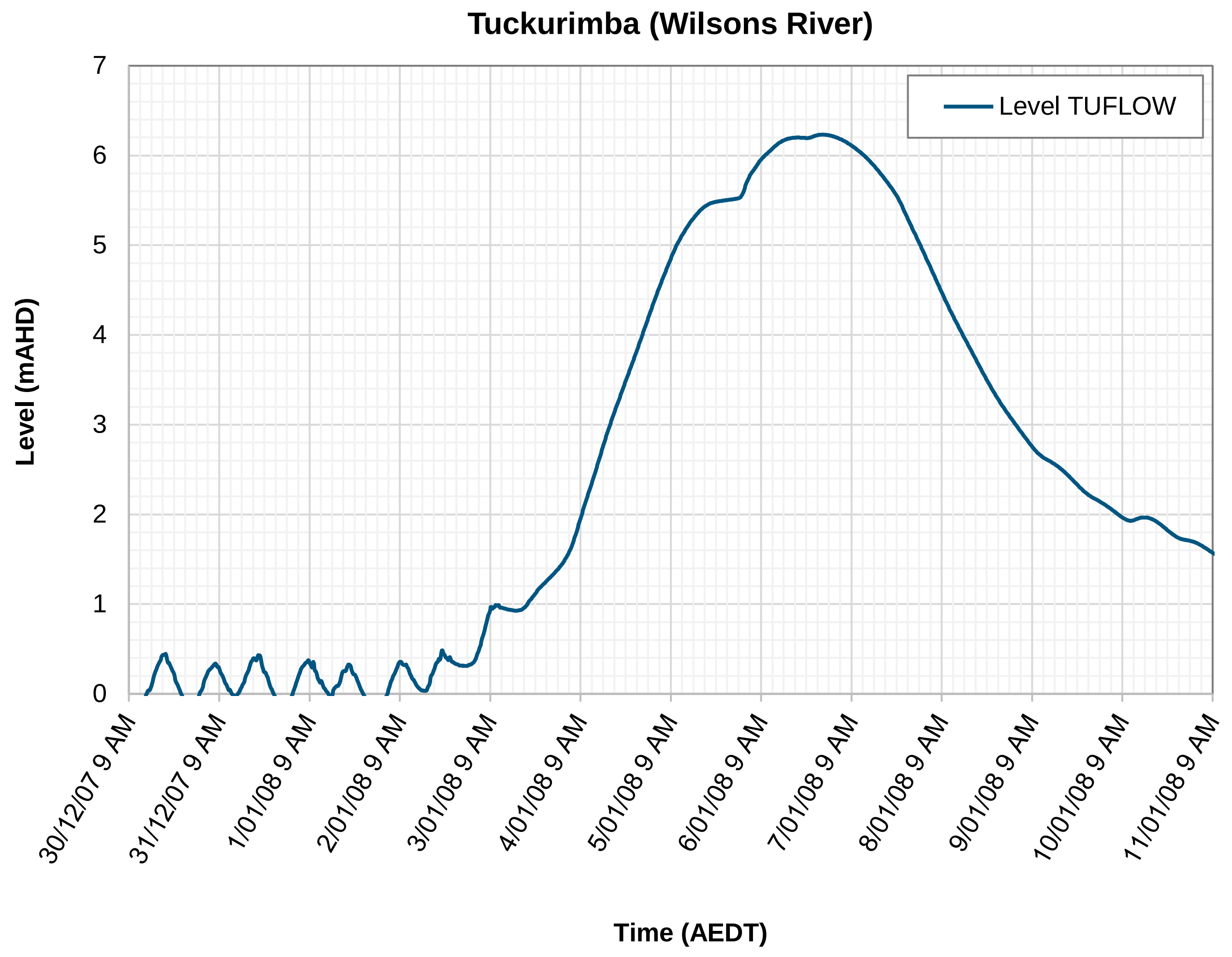
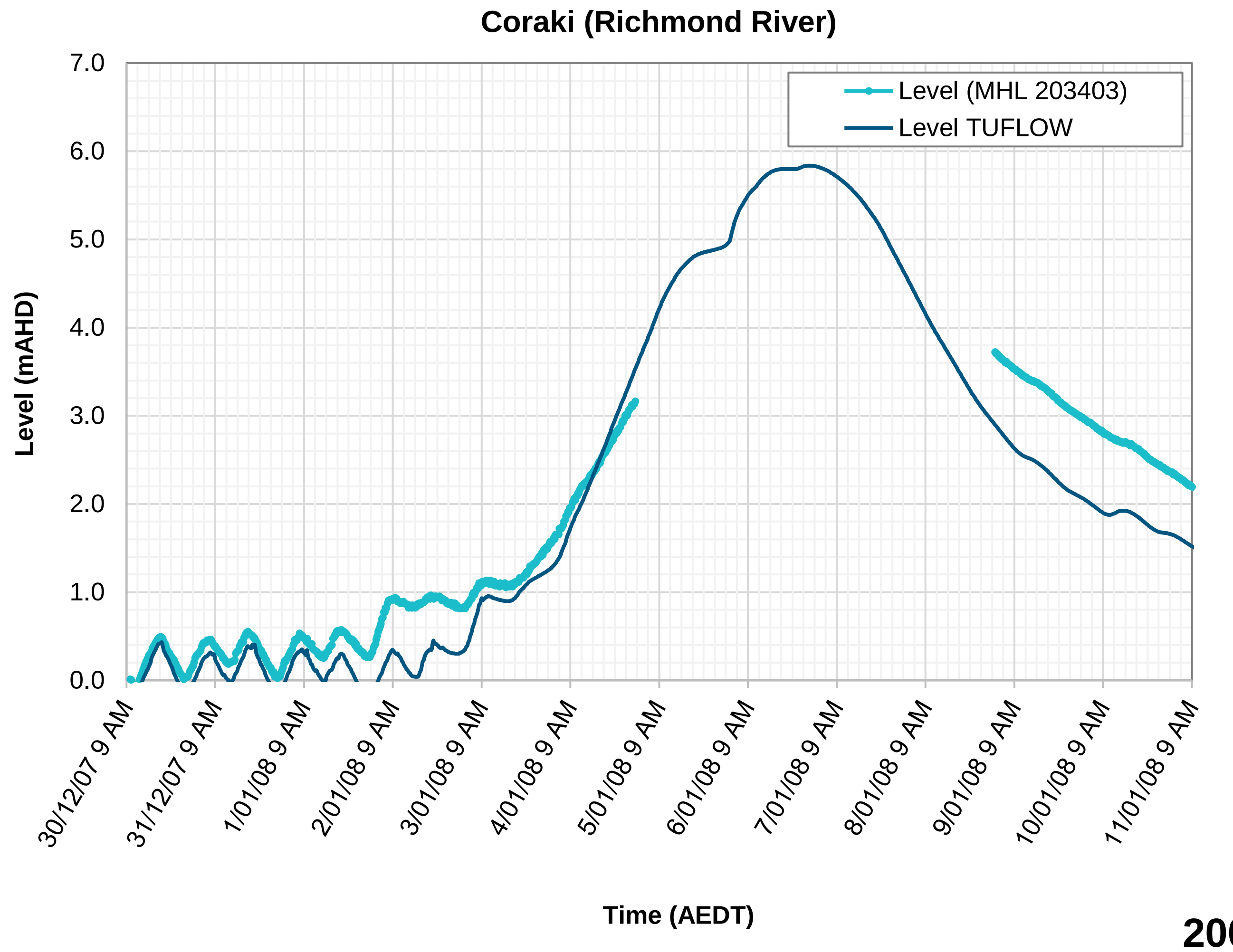
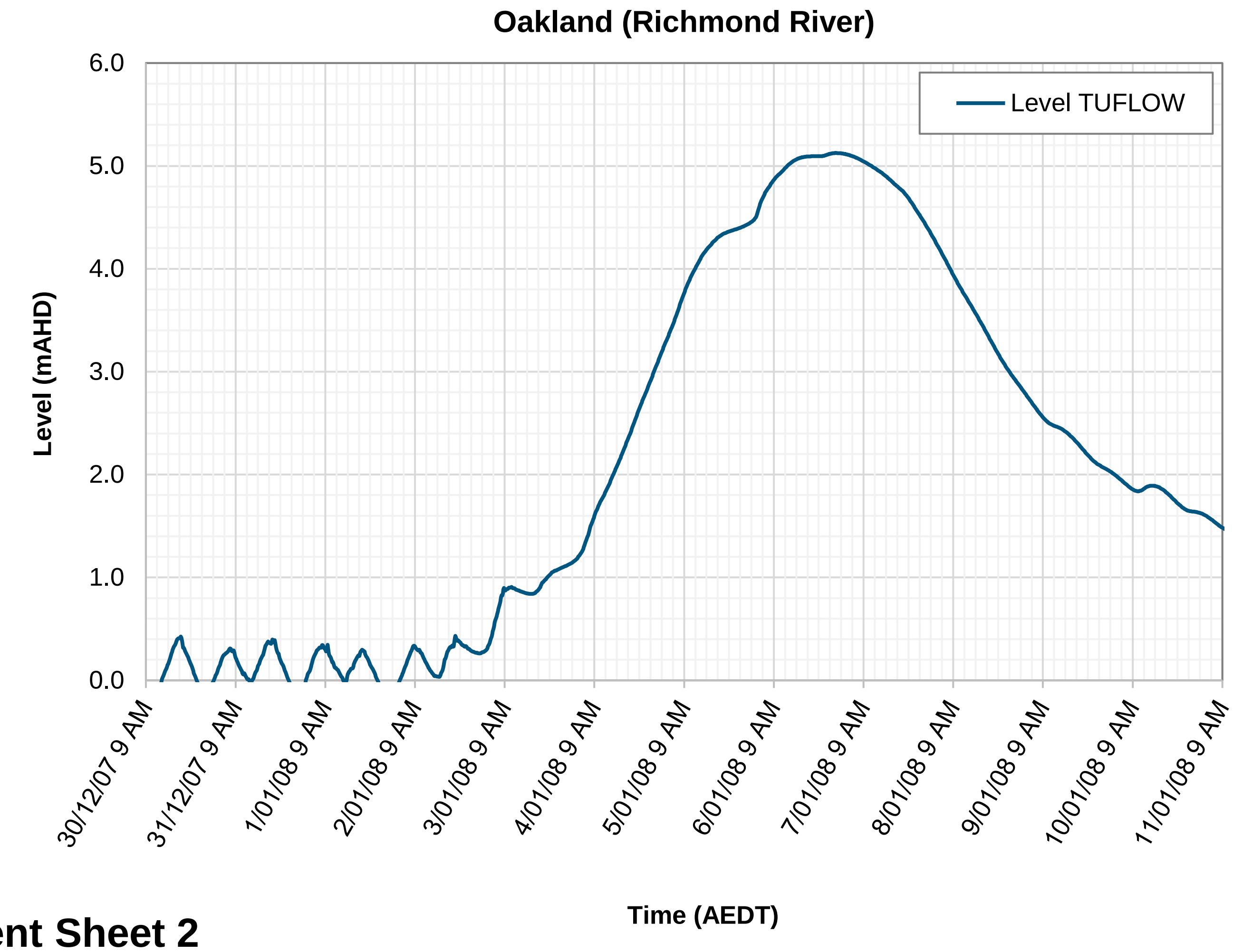
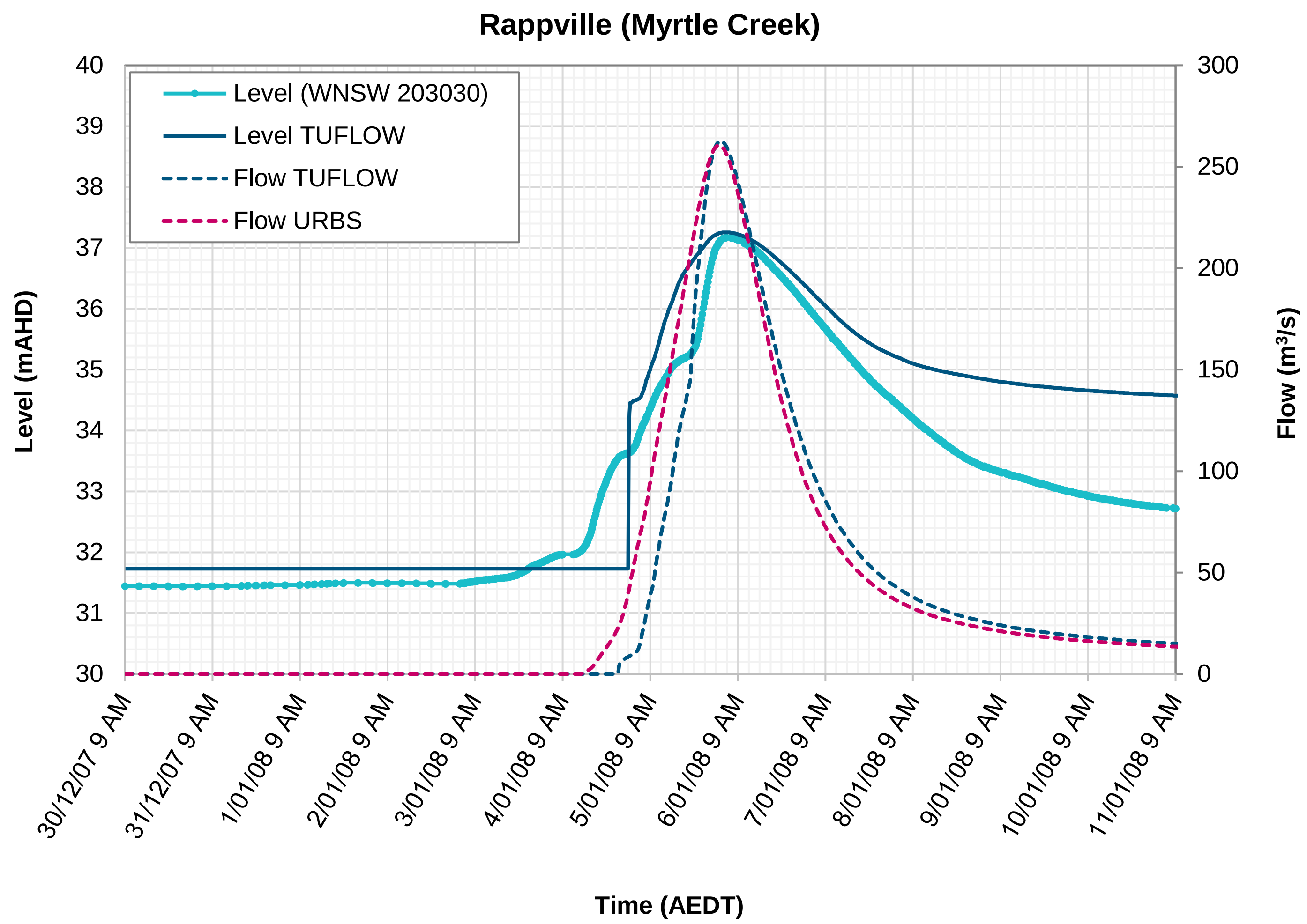


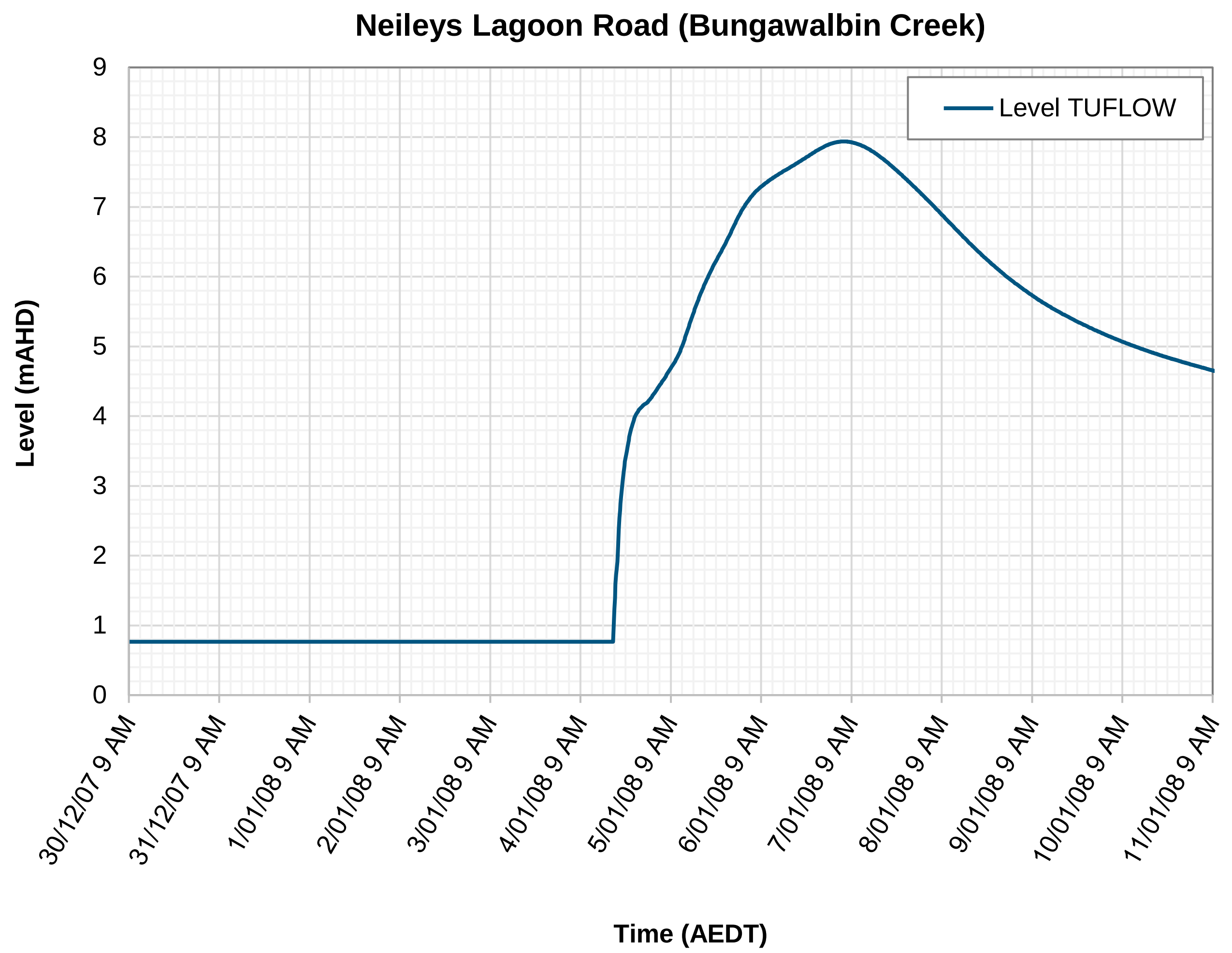


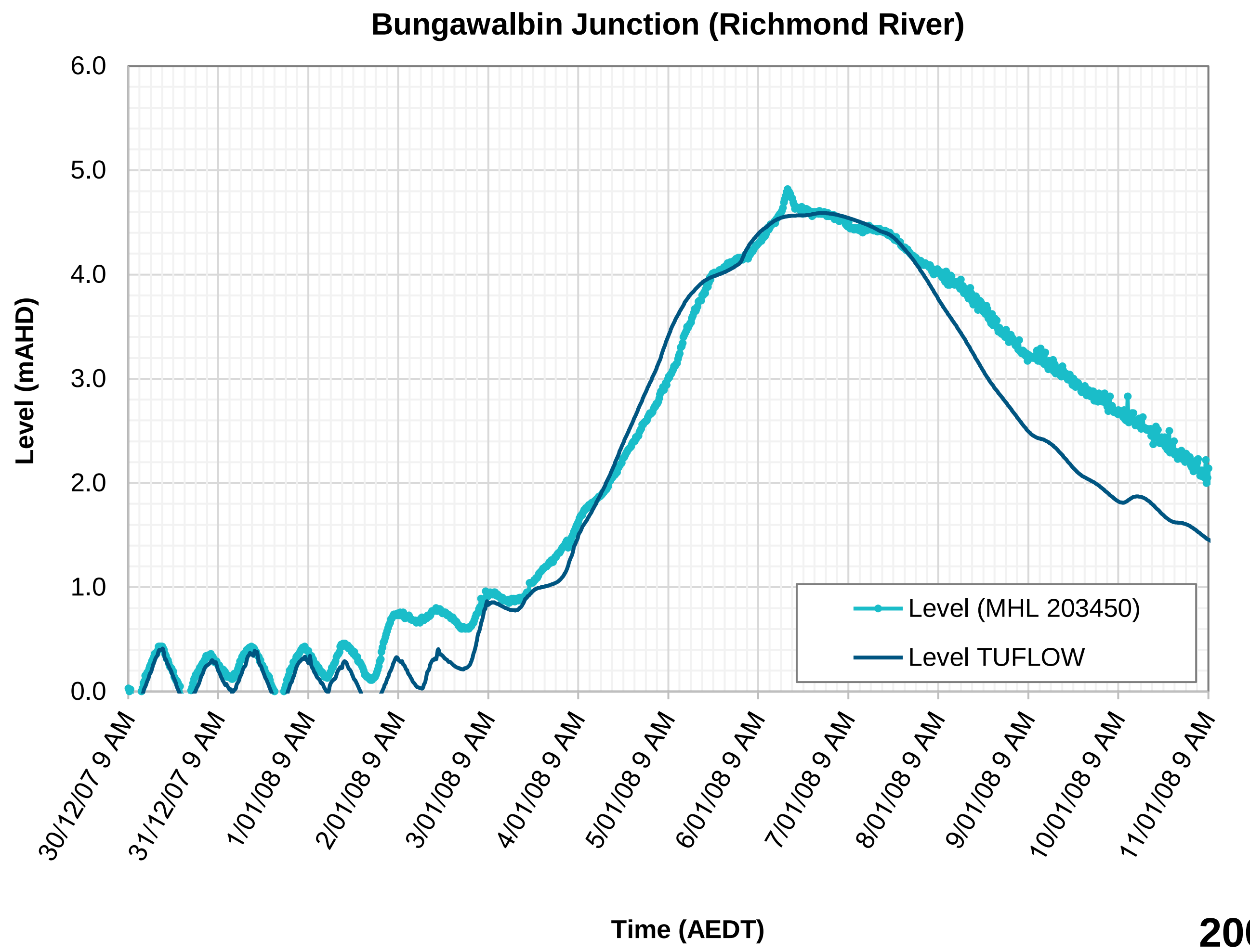


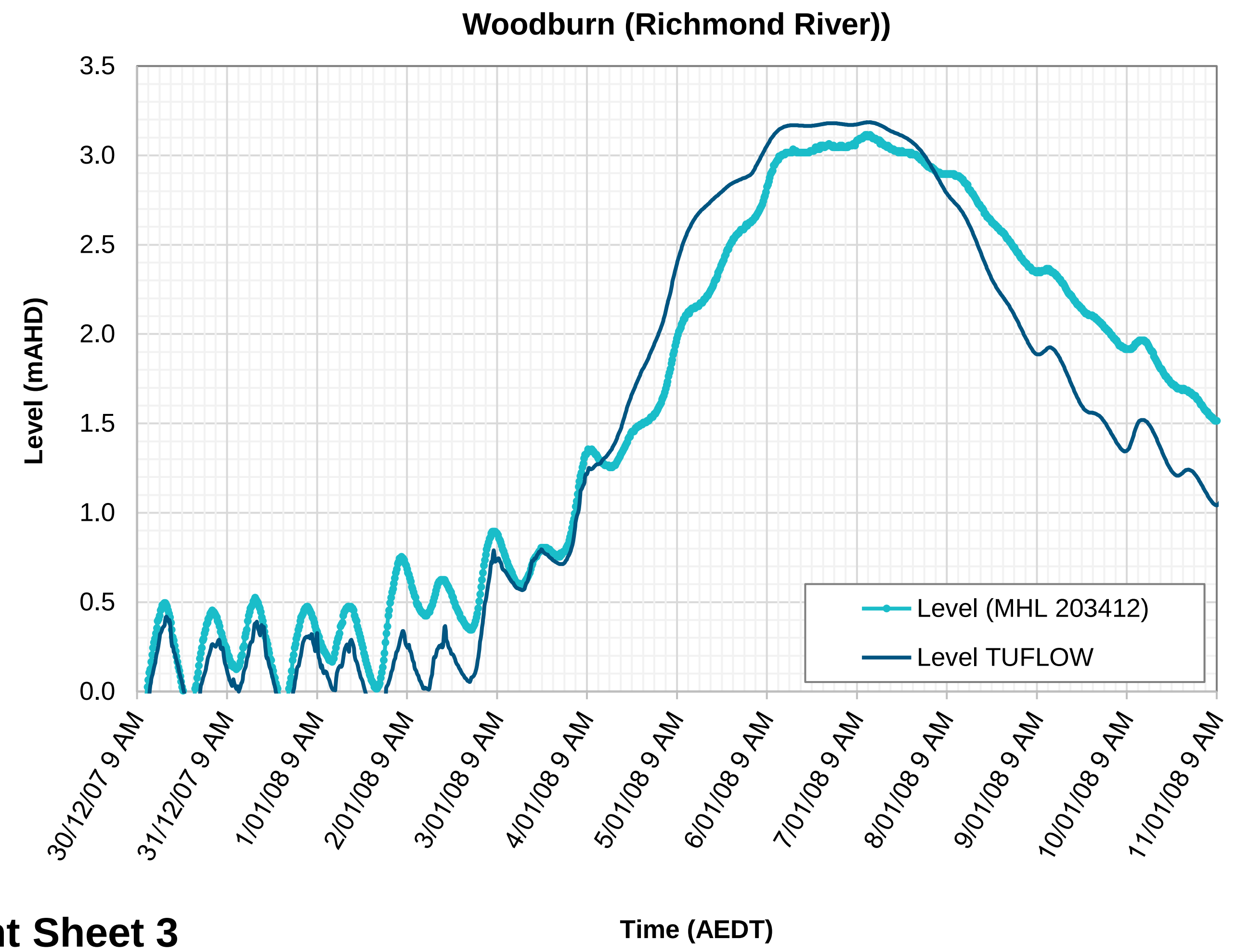


2008 Event Sheet 2









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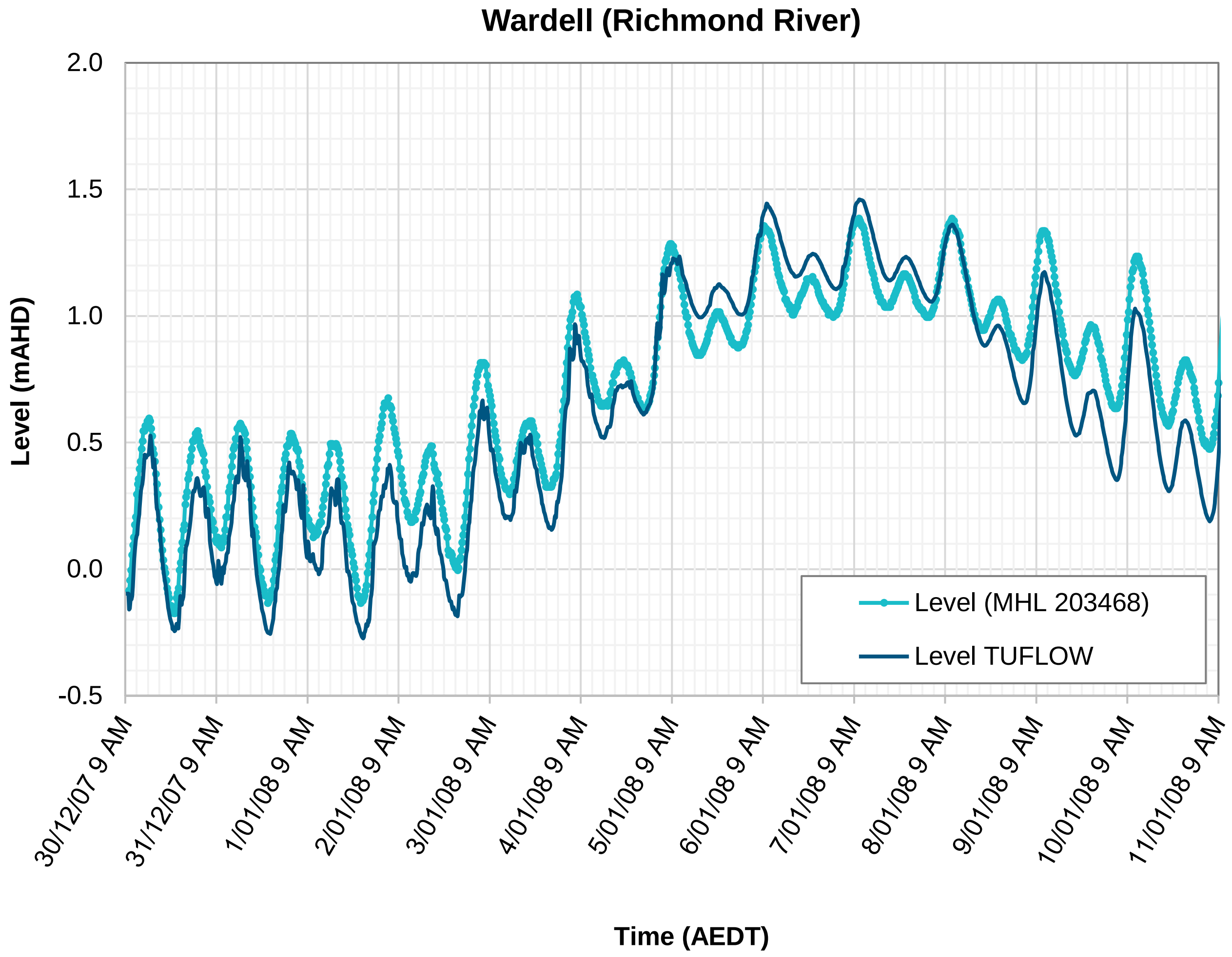
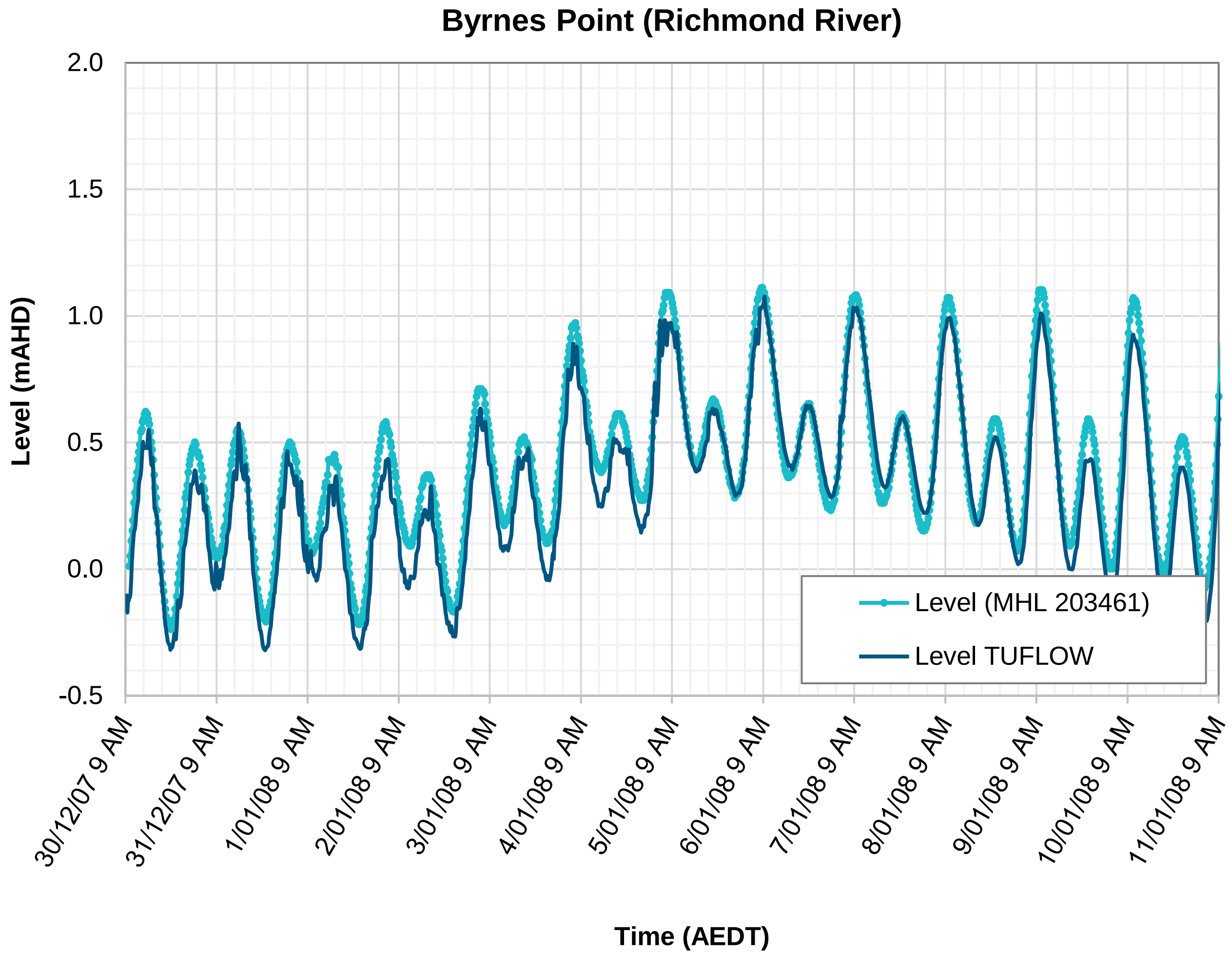
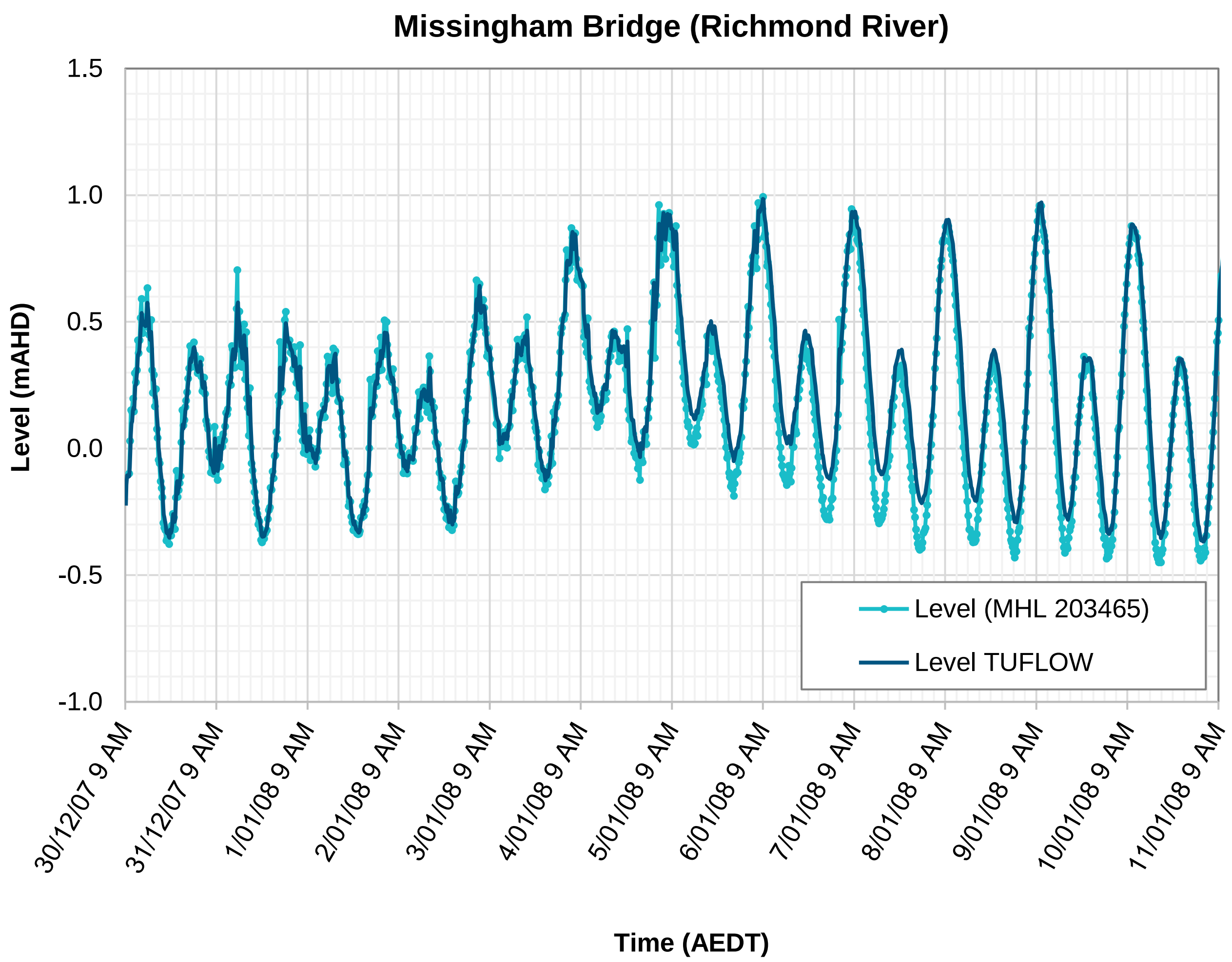
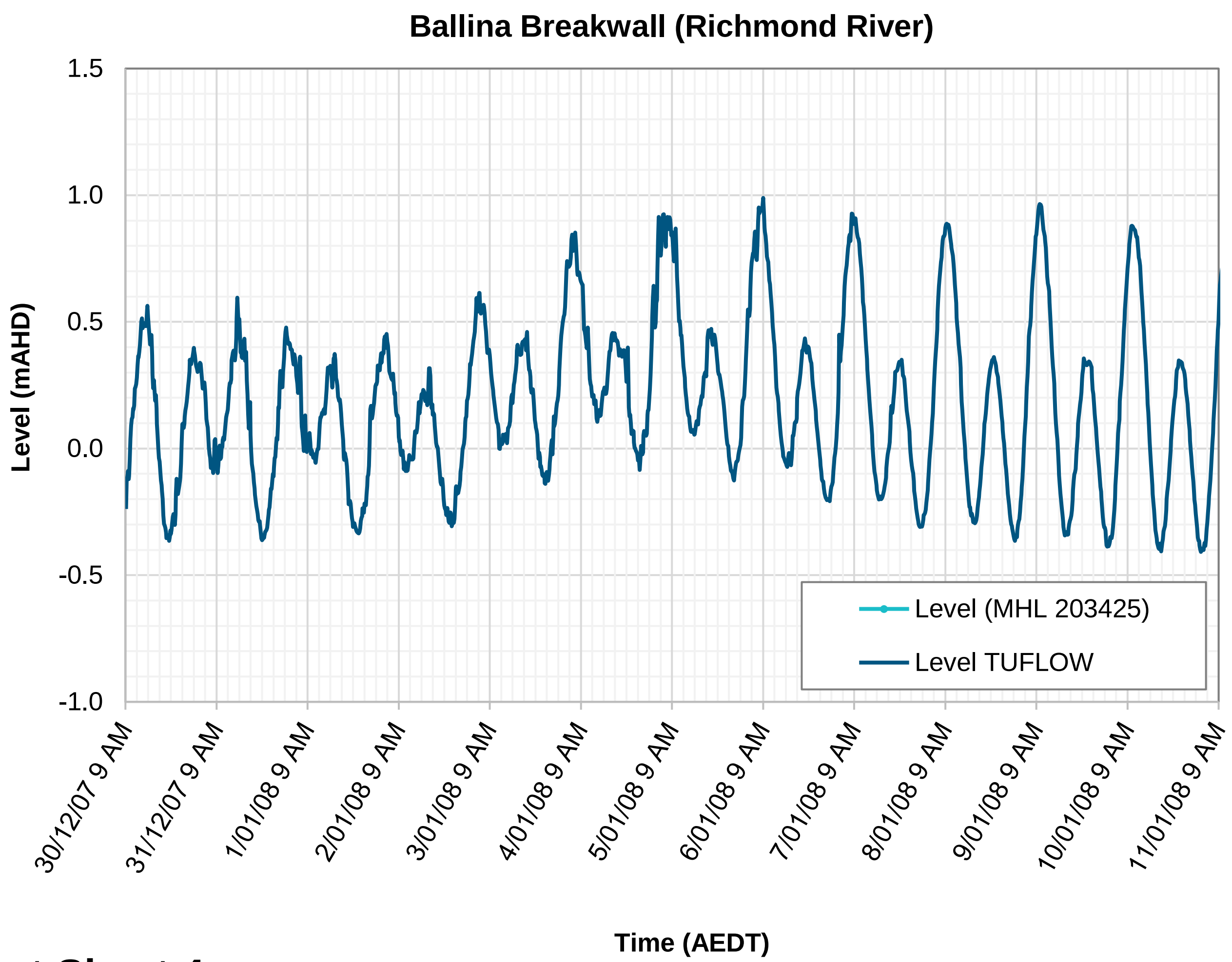
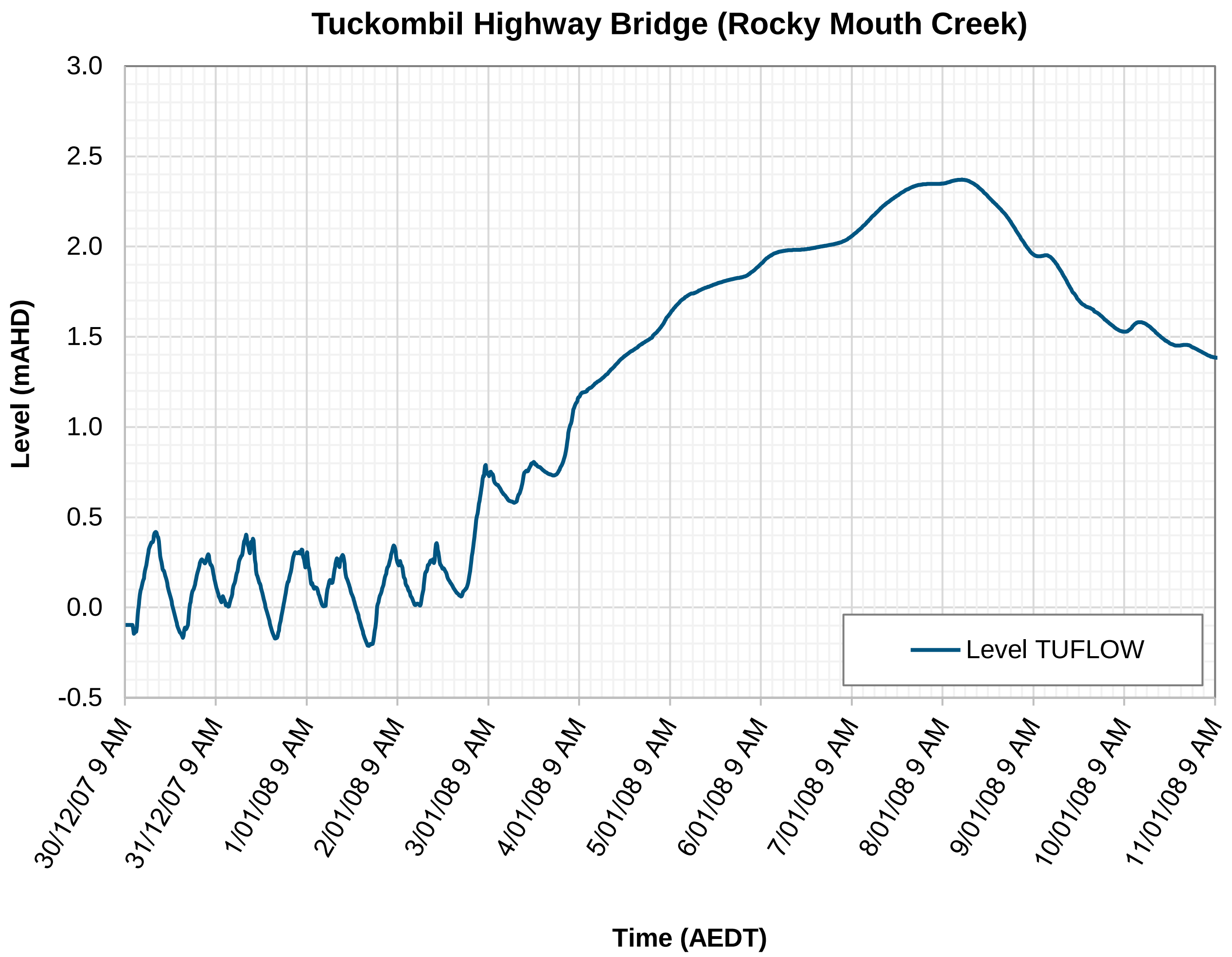


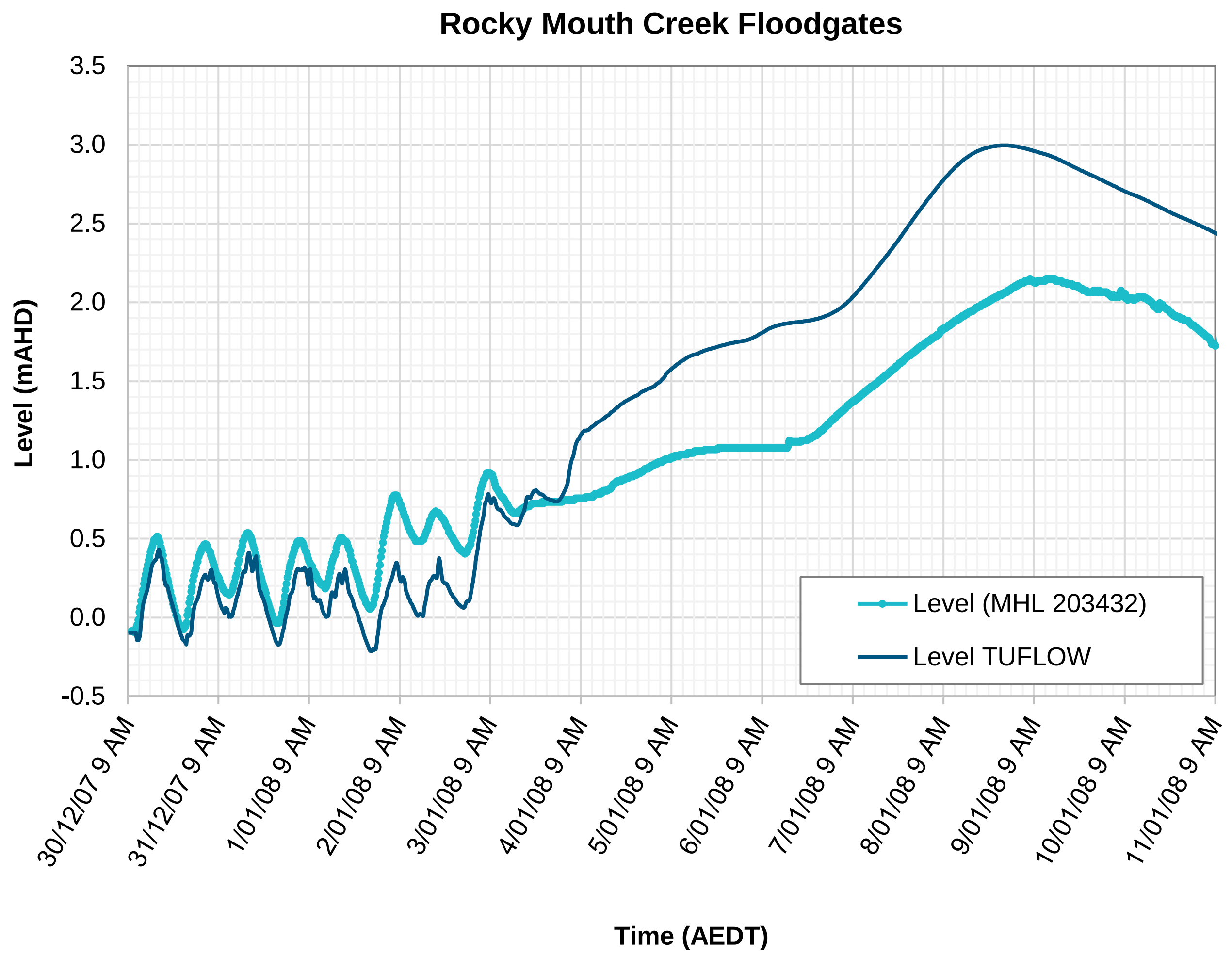


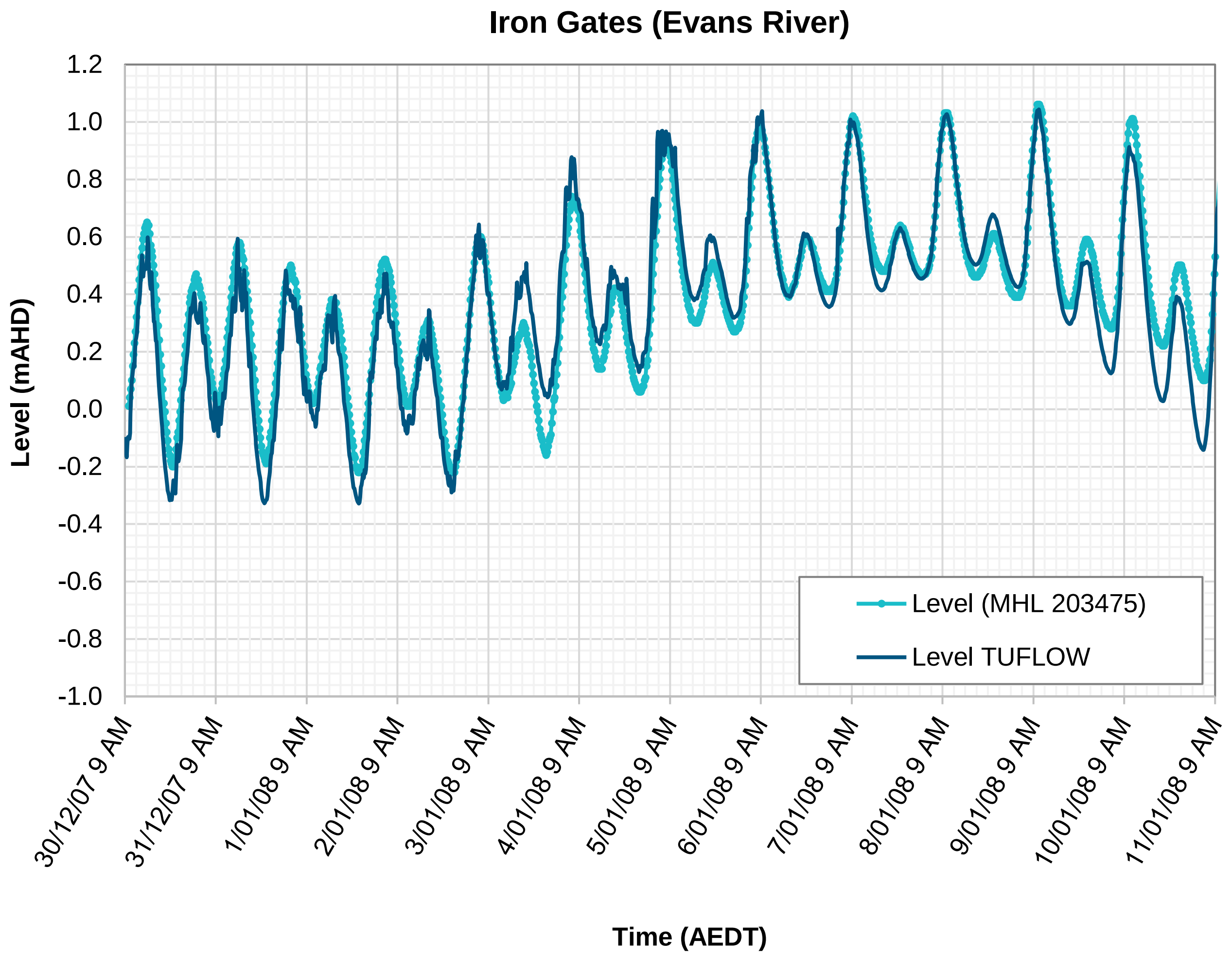


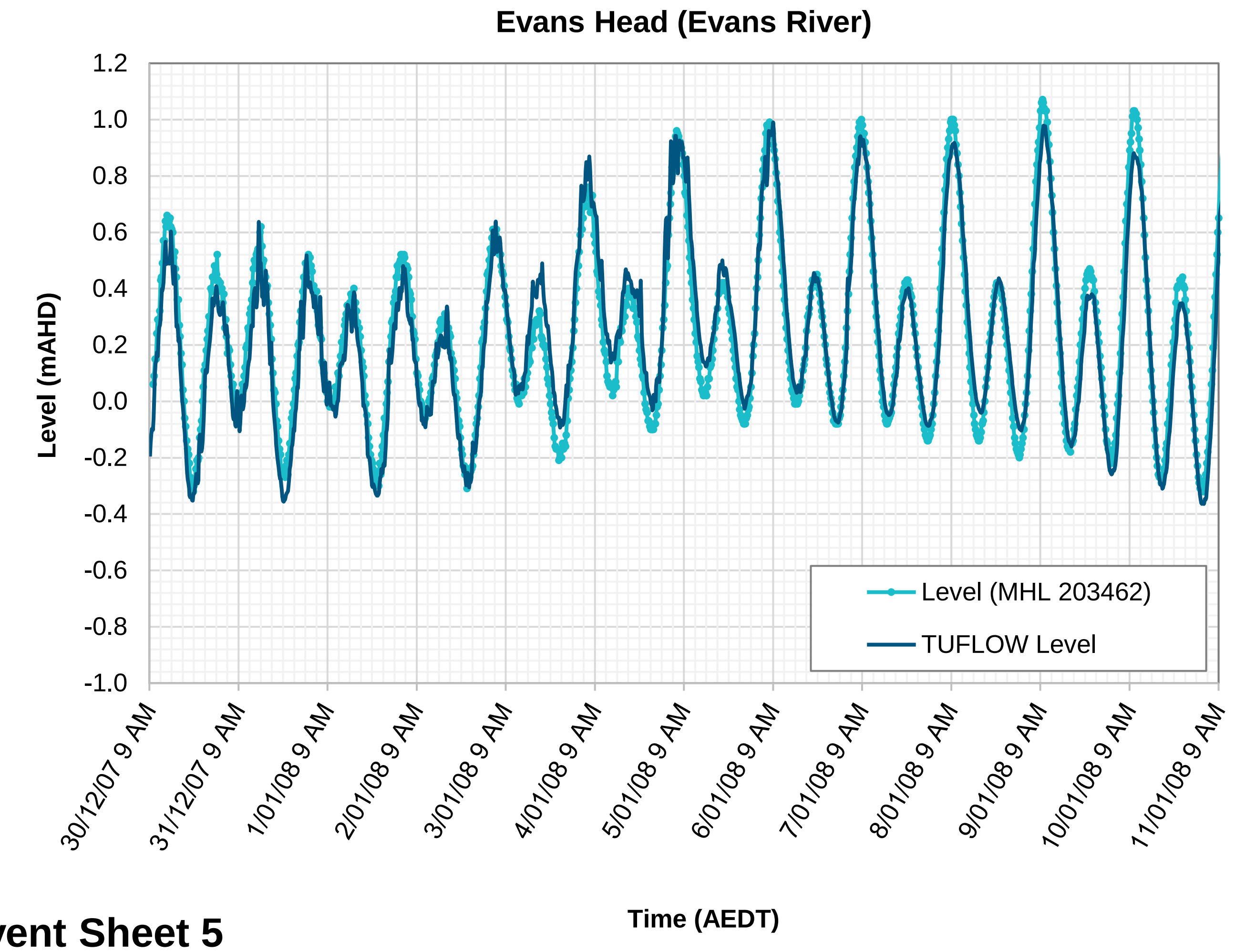


2008 Event Sheet 4









2008 Event Sheet 5

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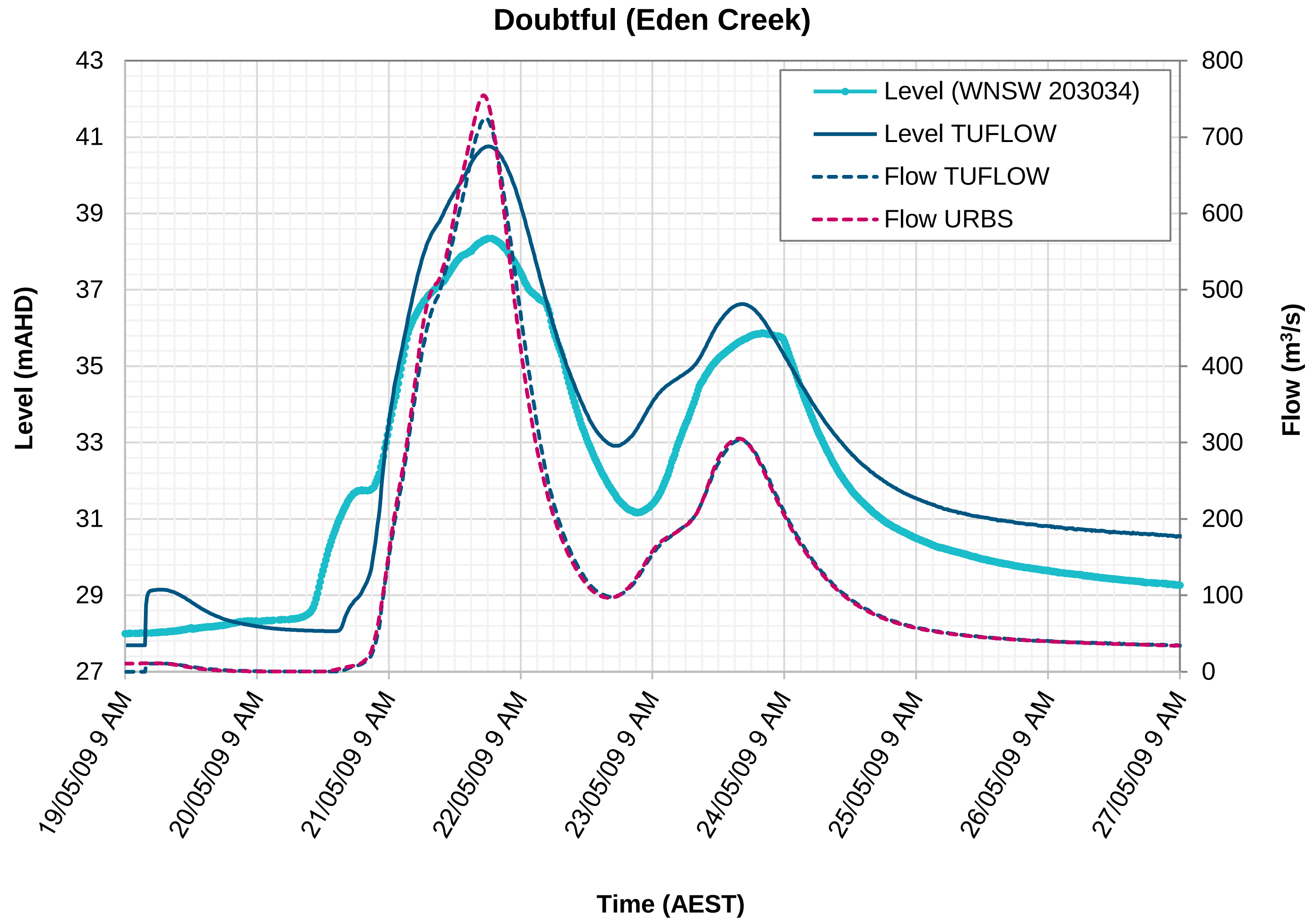
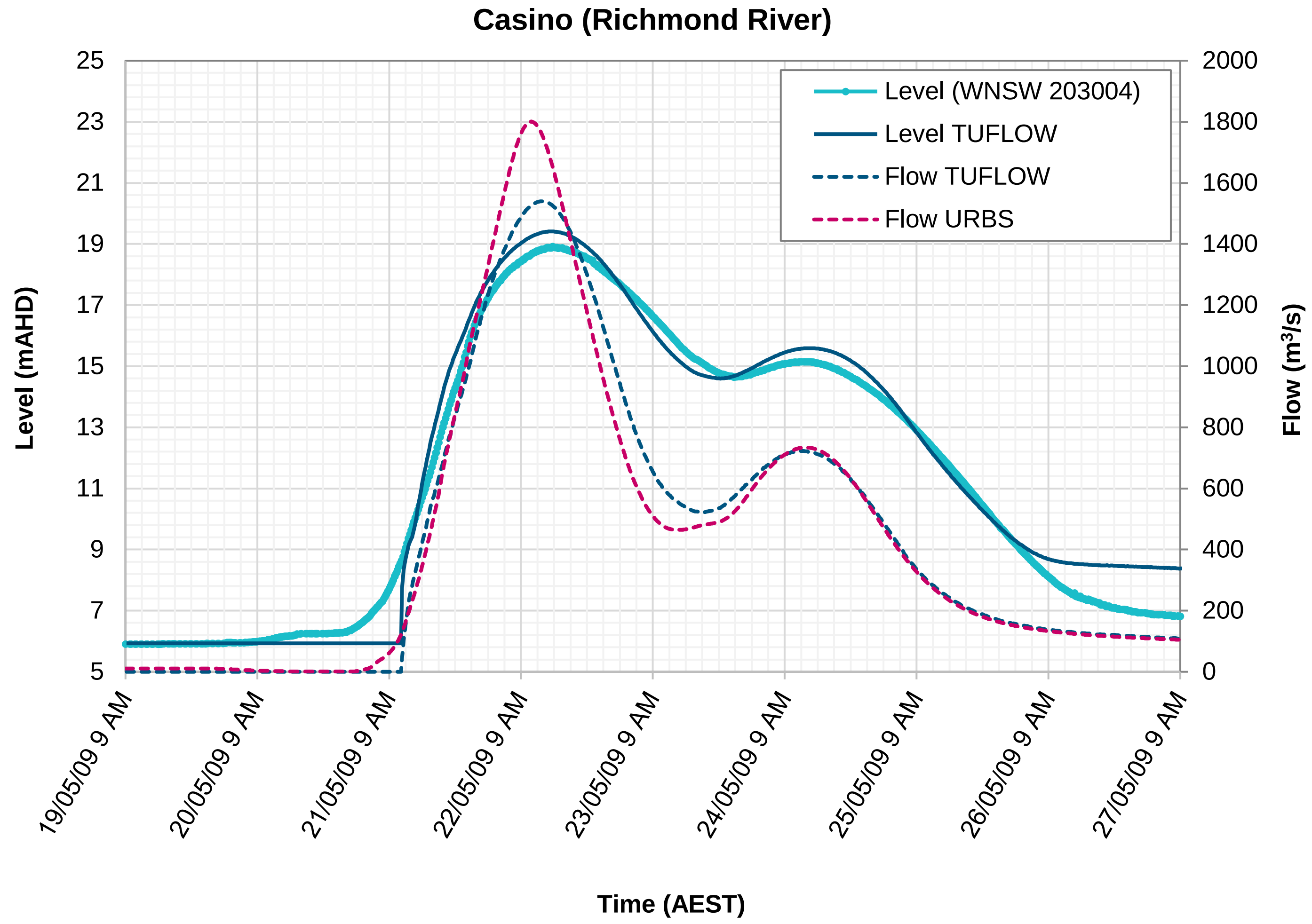
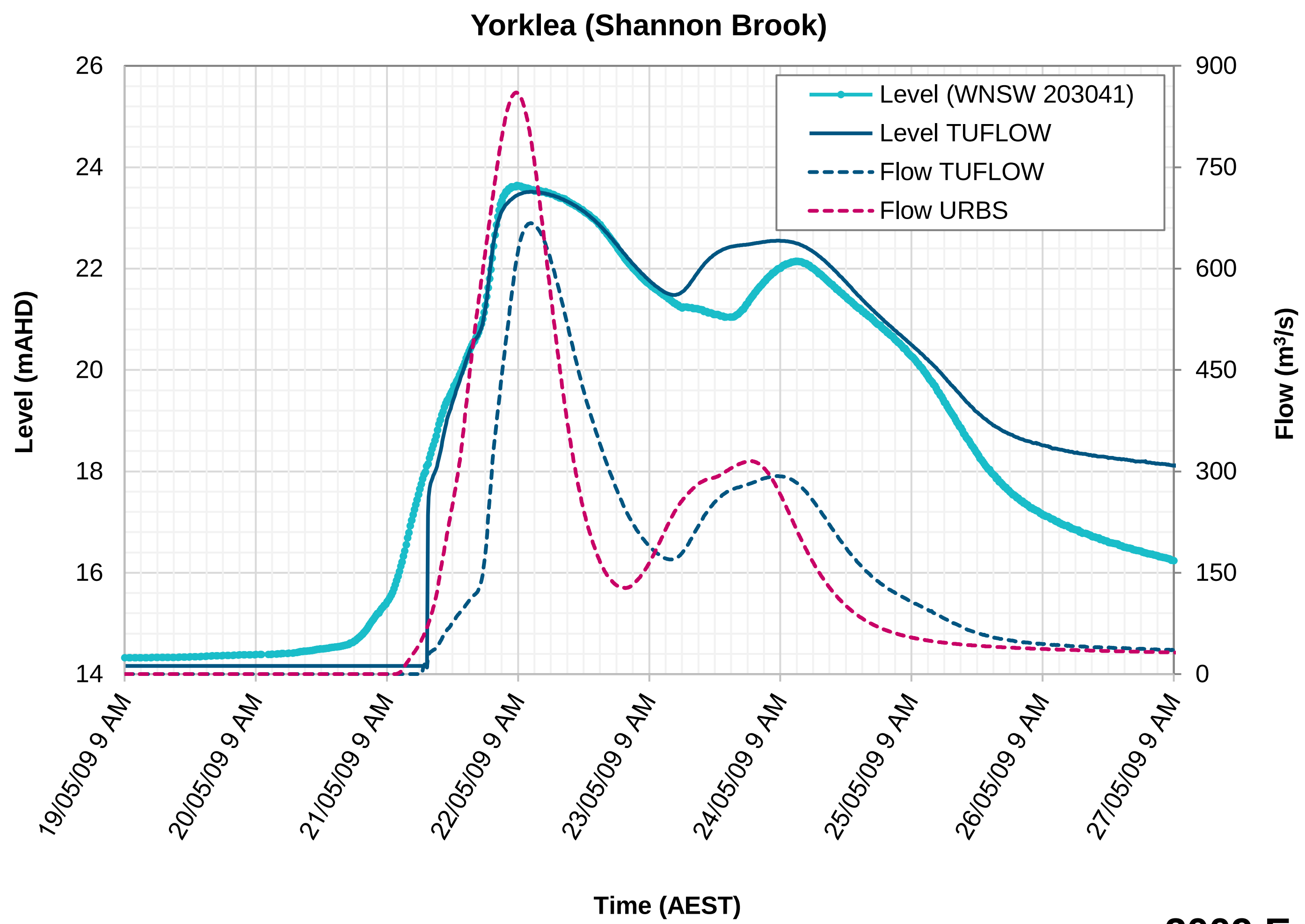
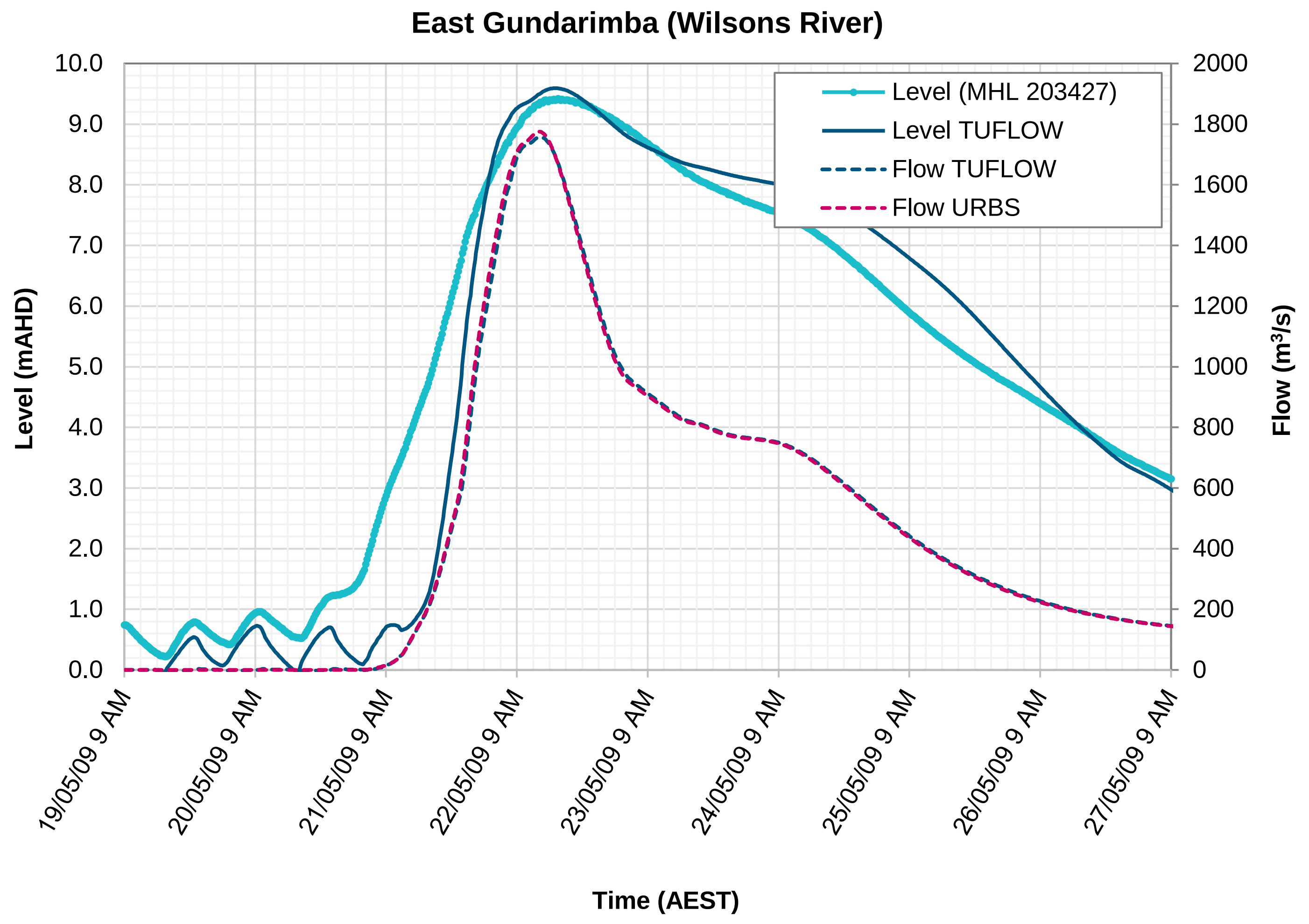
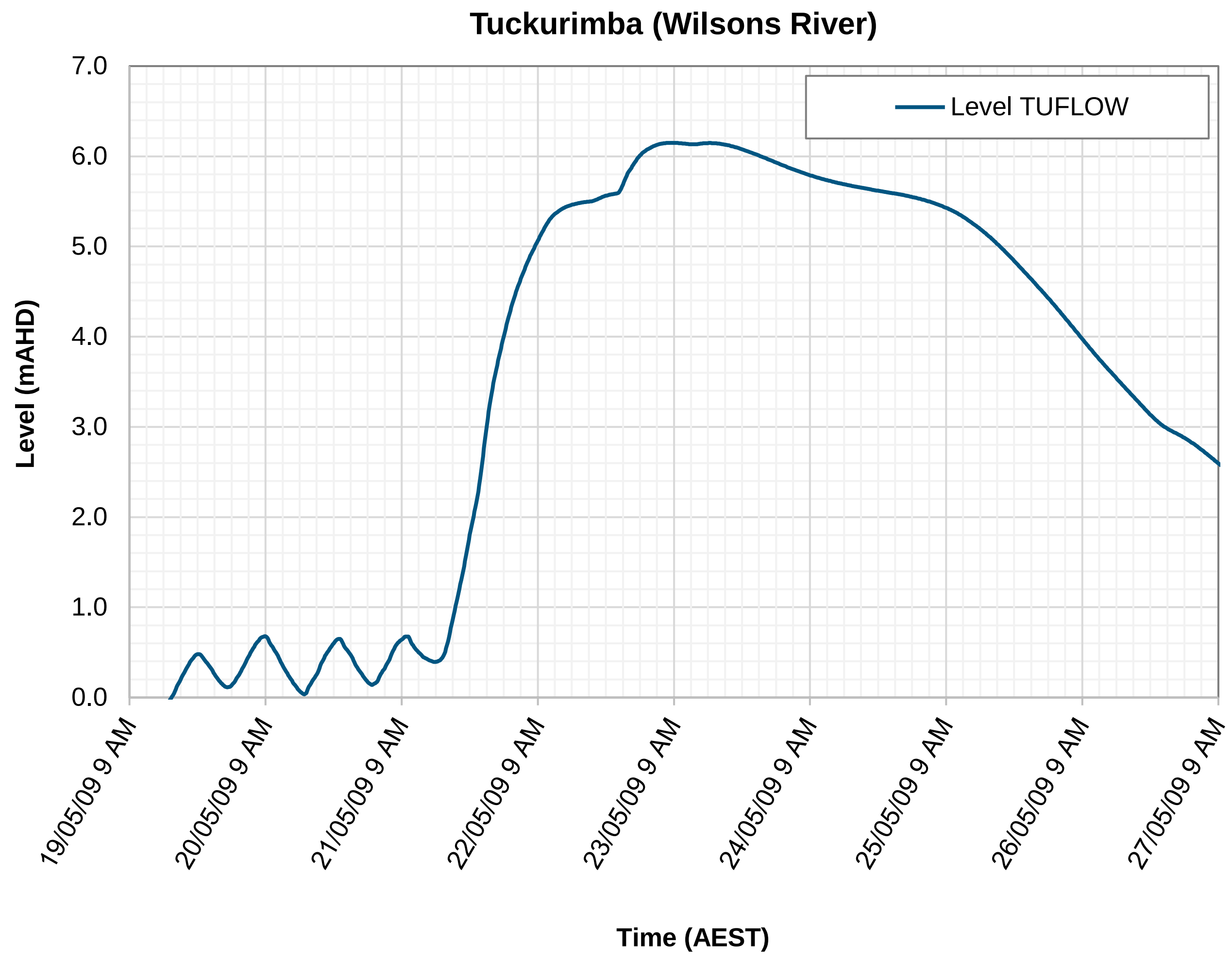


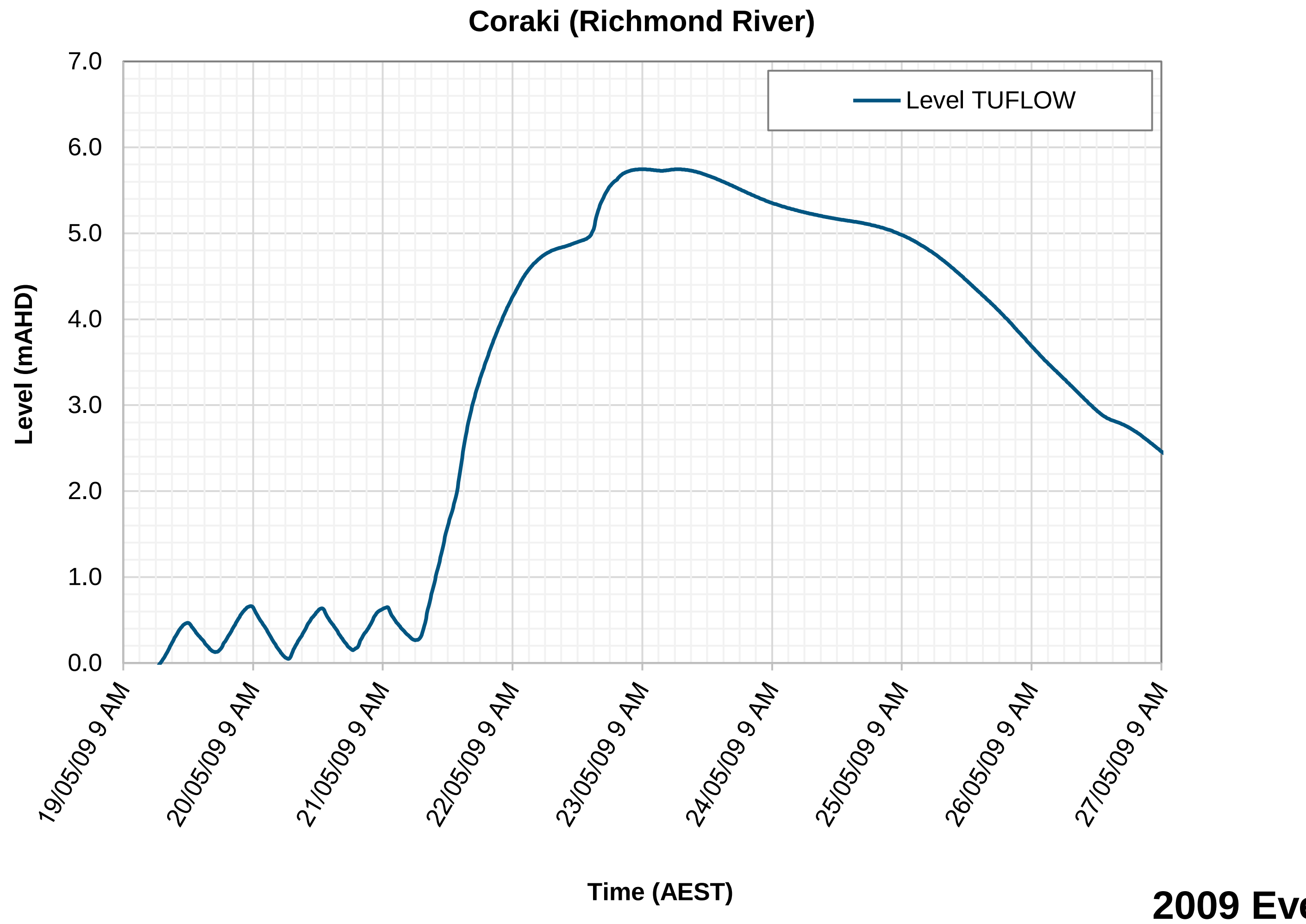


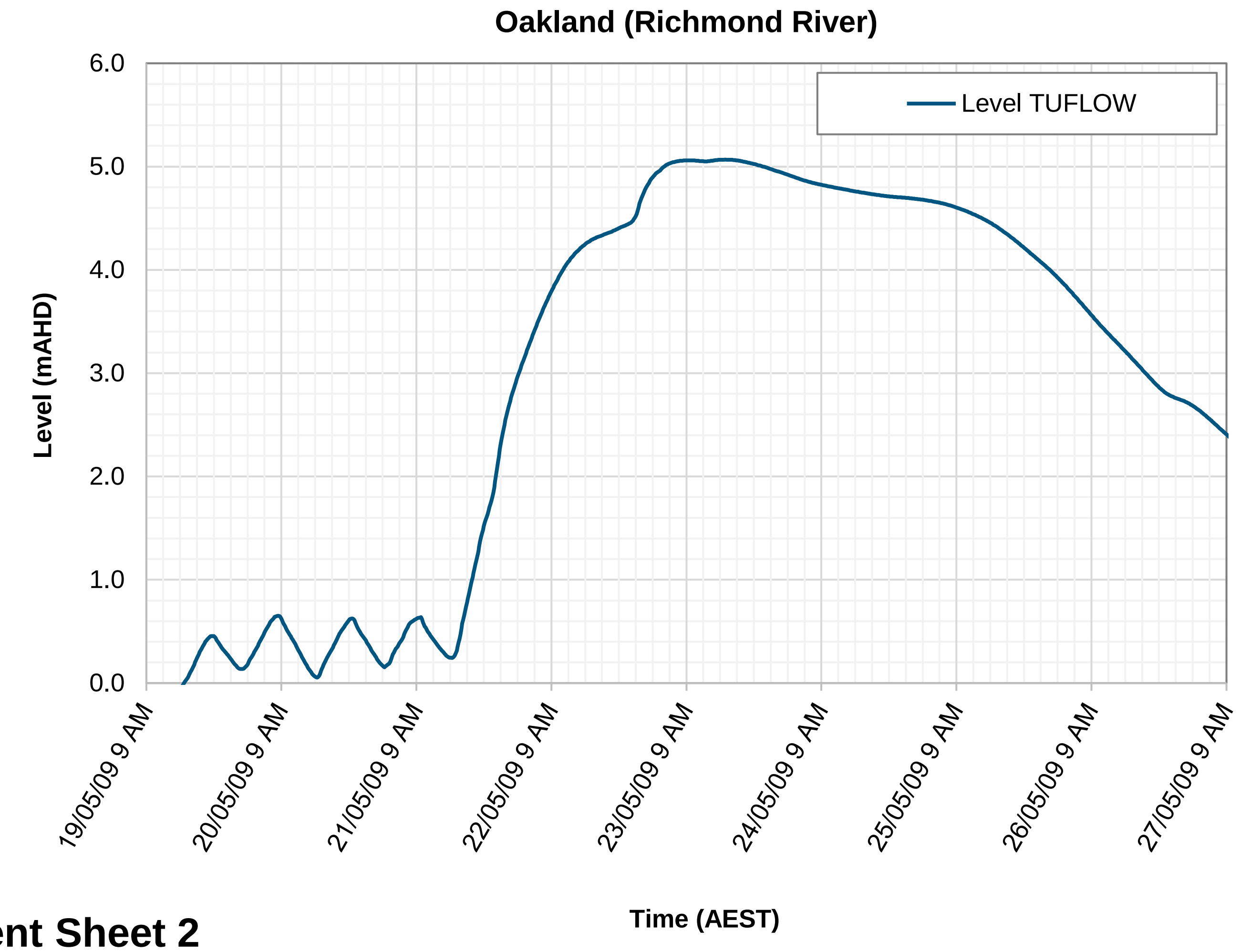


2009 Event Sheet 1

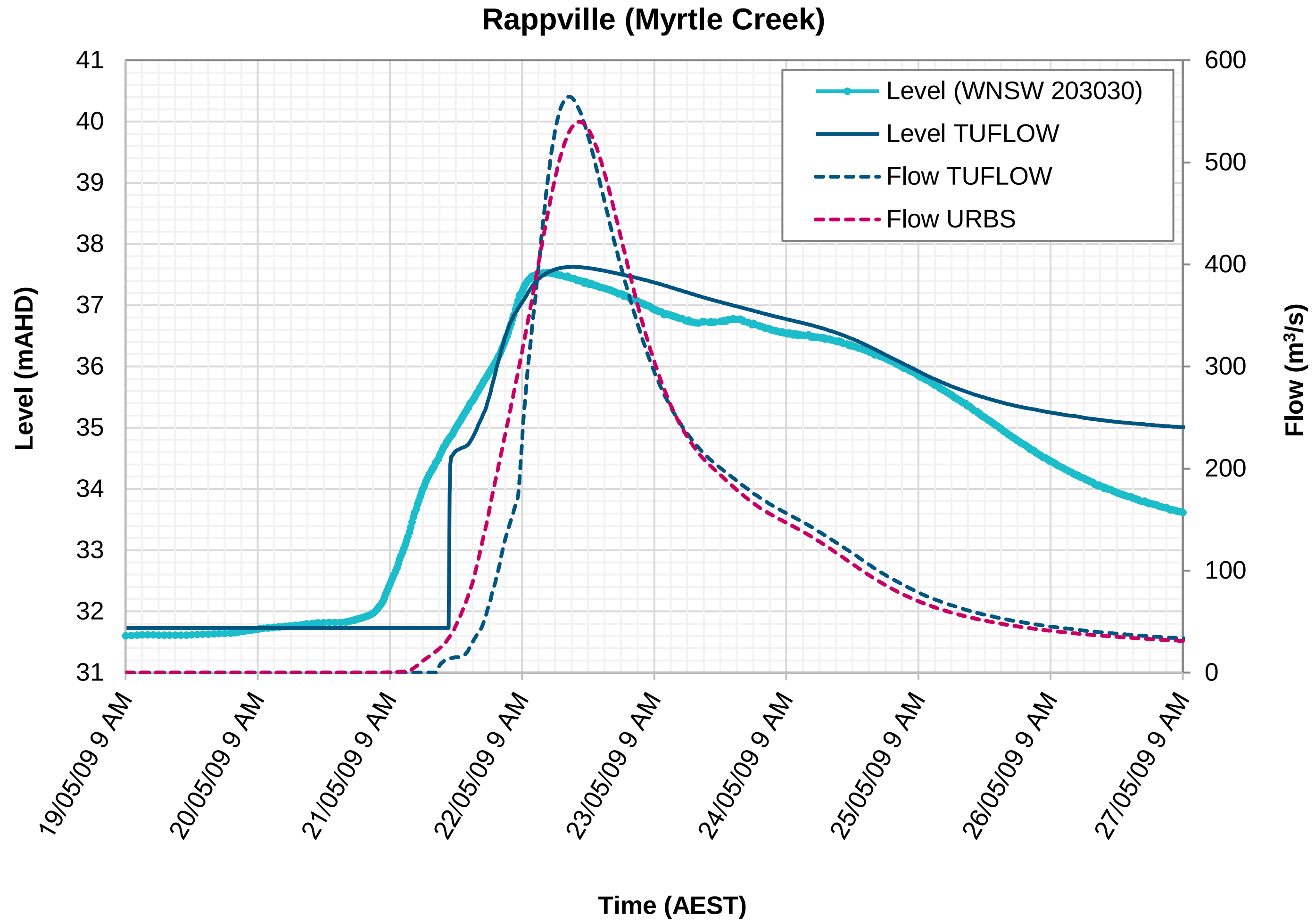


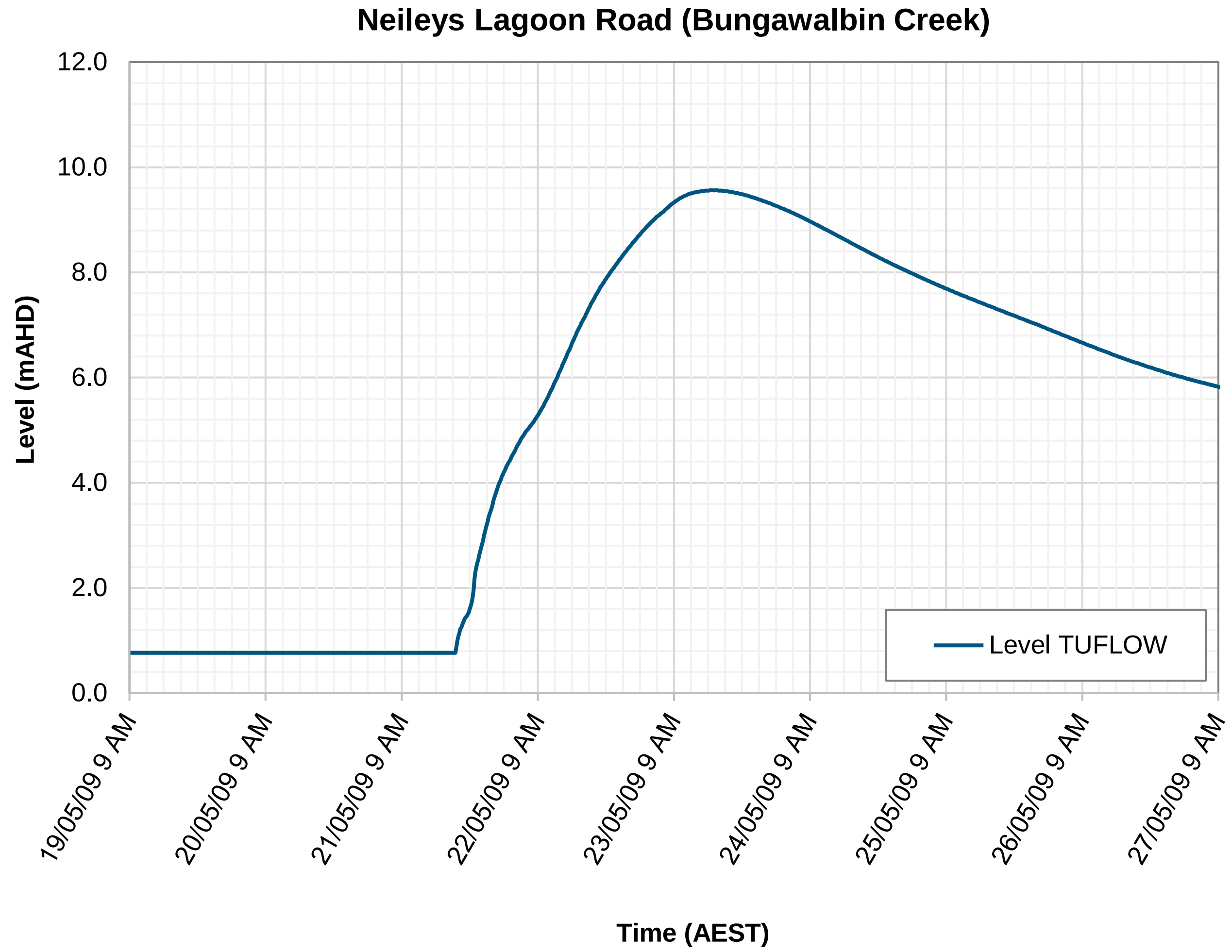


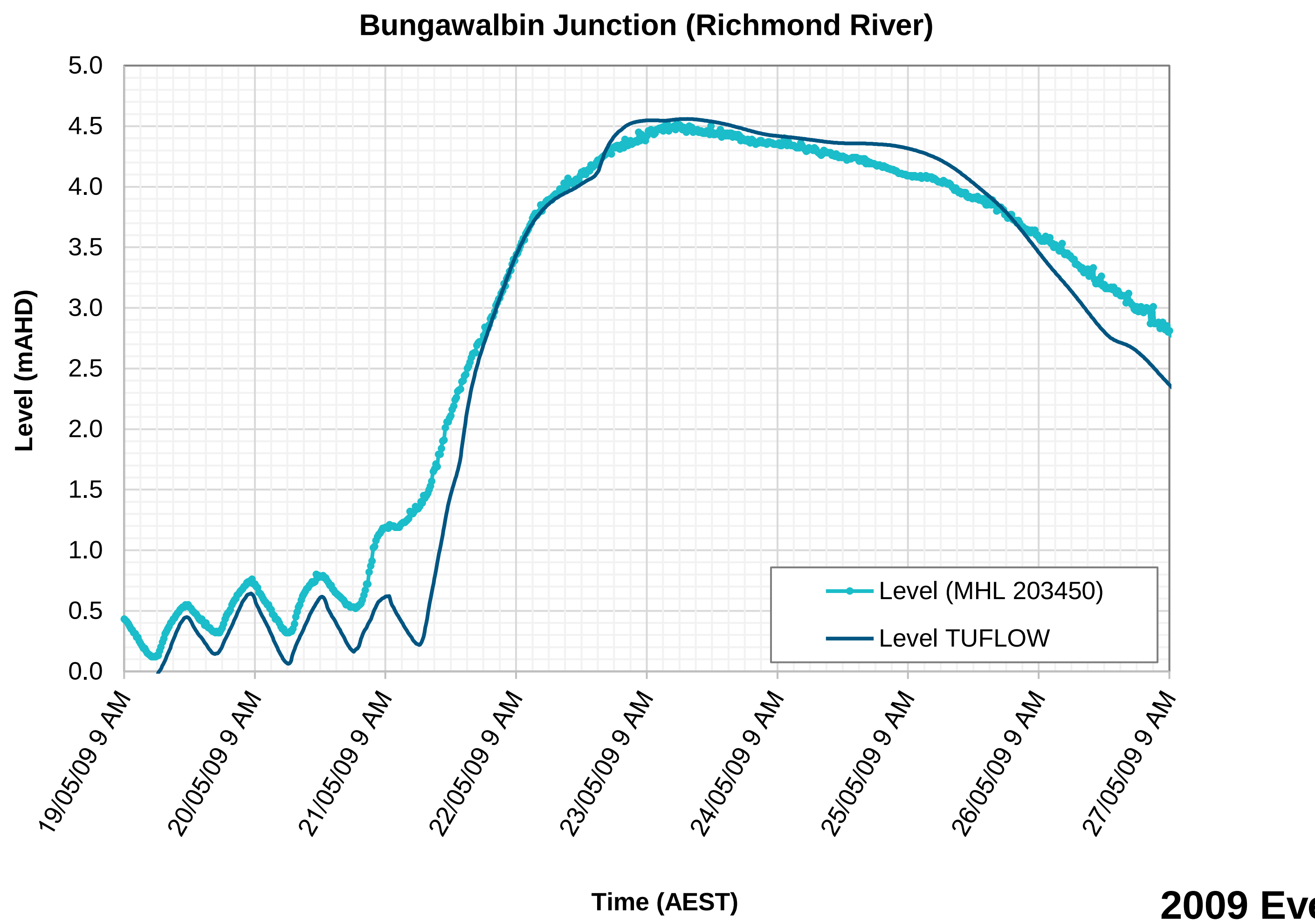


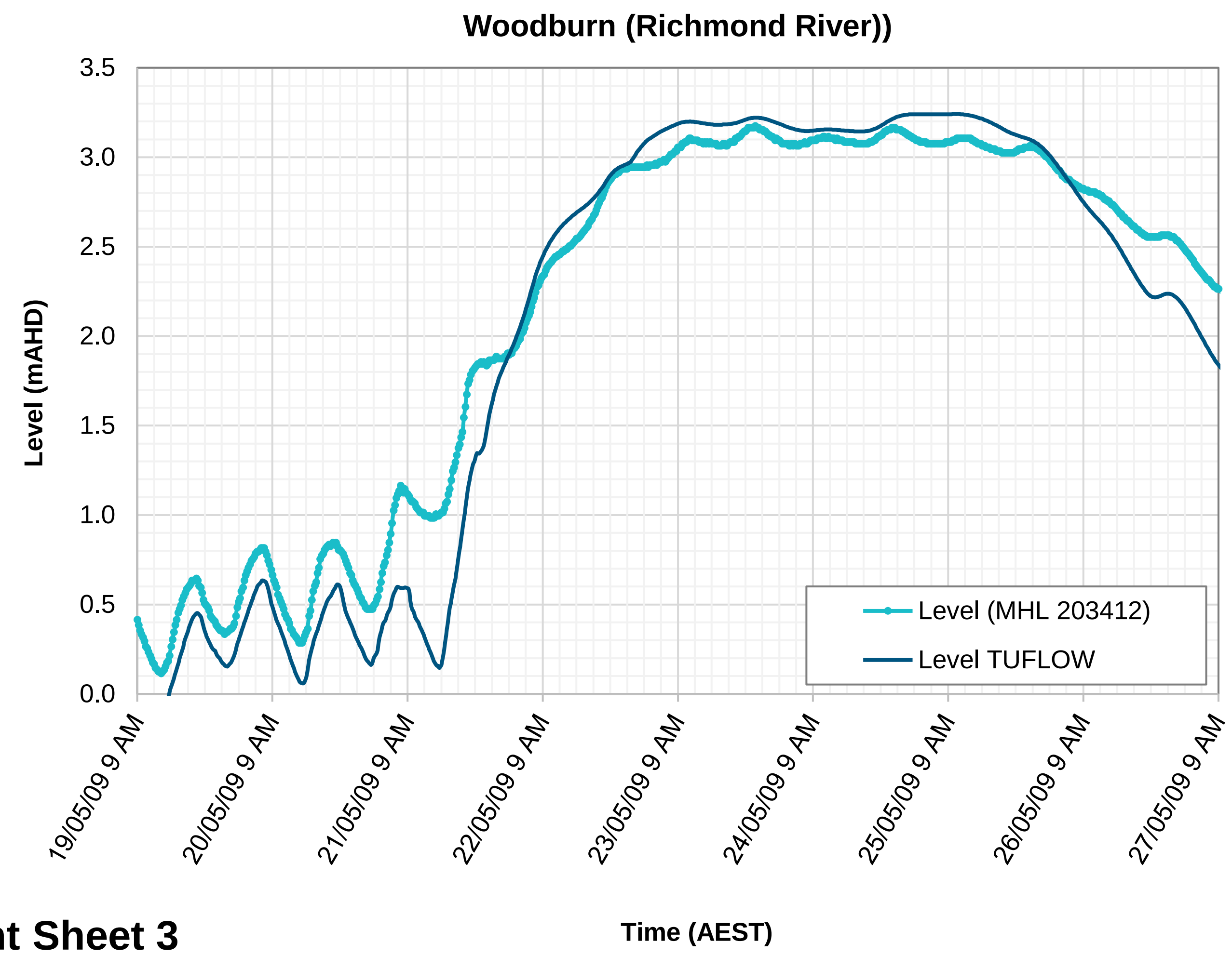


2009 Event Sheet 2

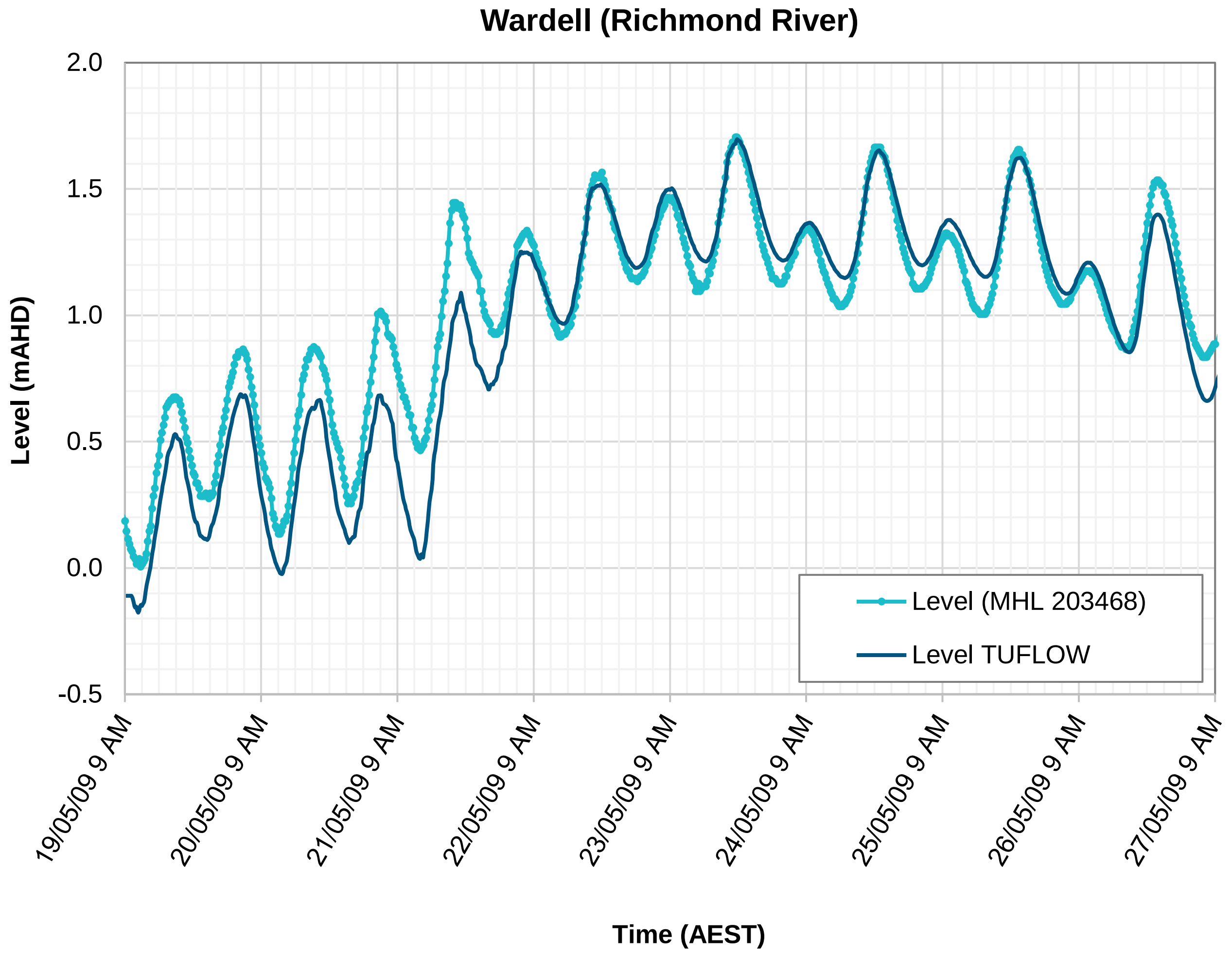


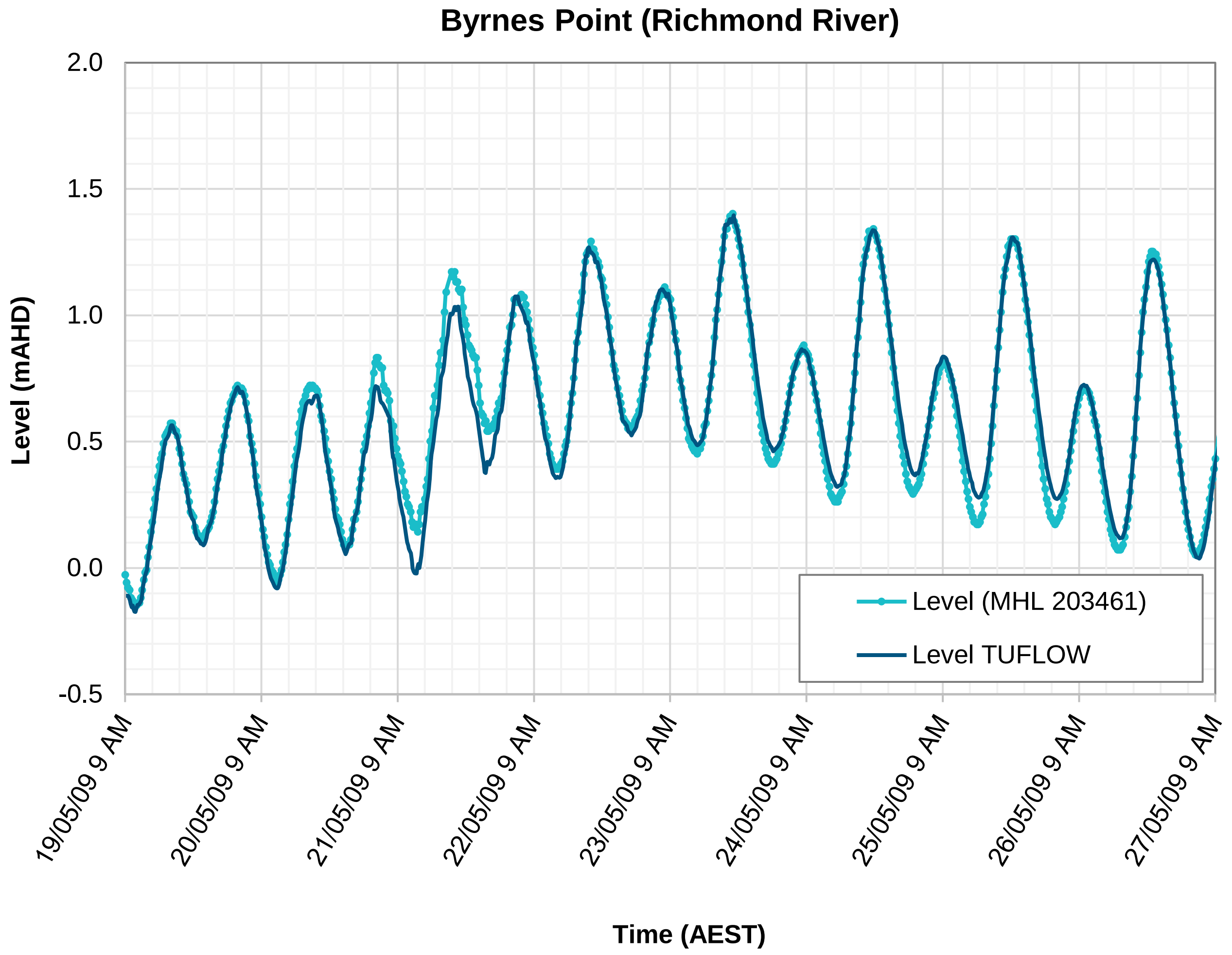


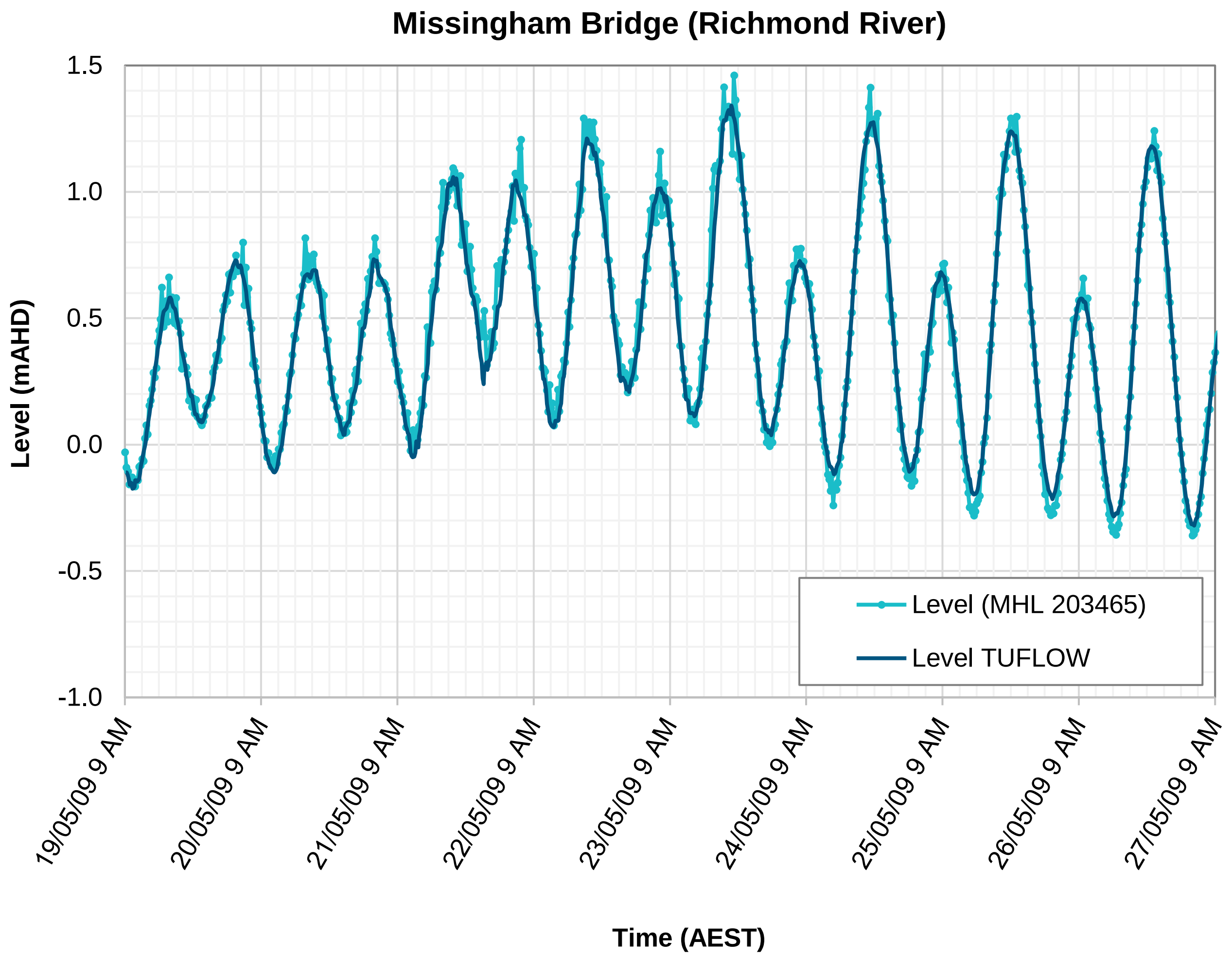


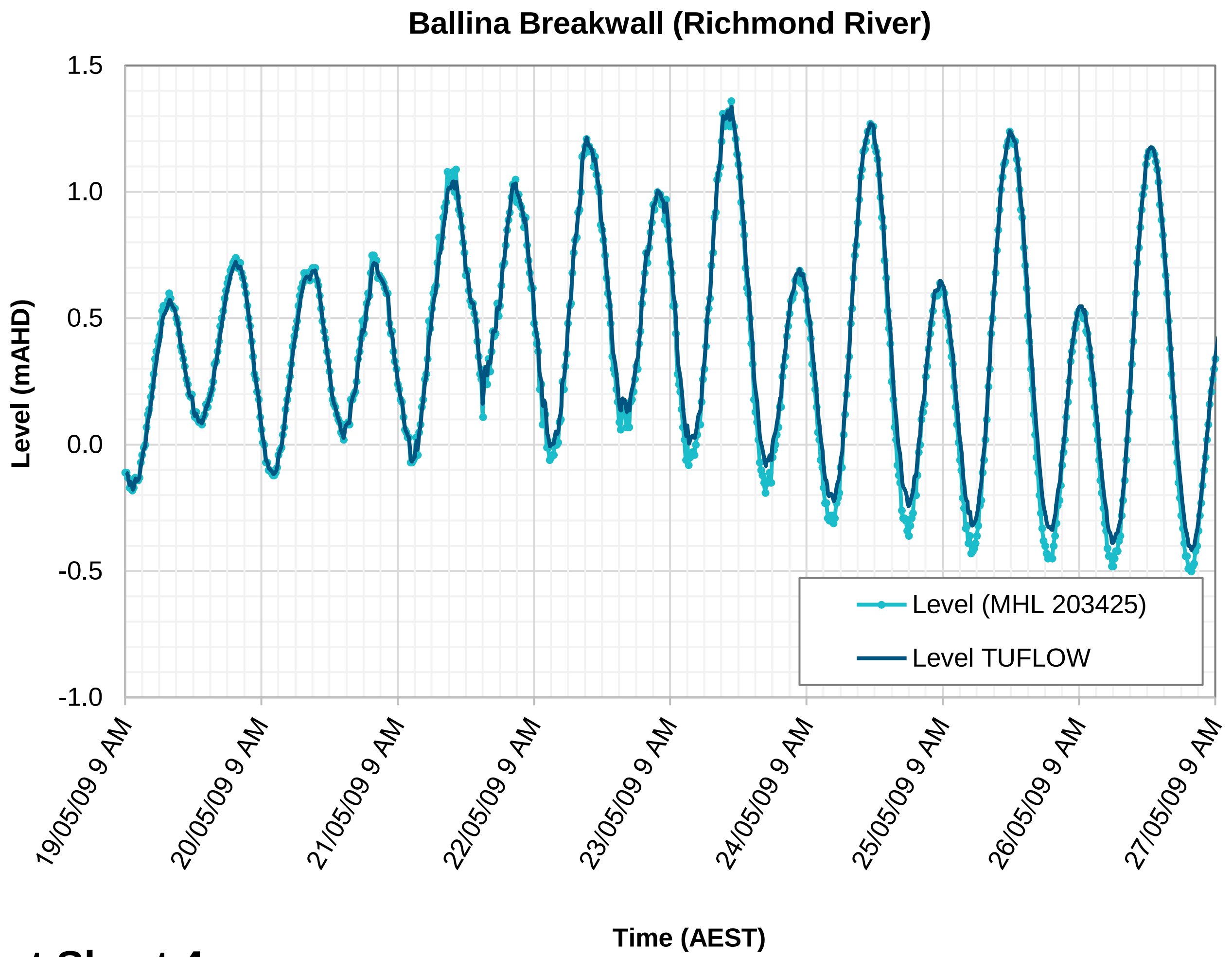


2009 Event Sheet 3

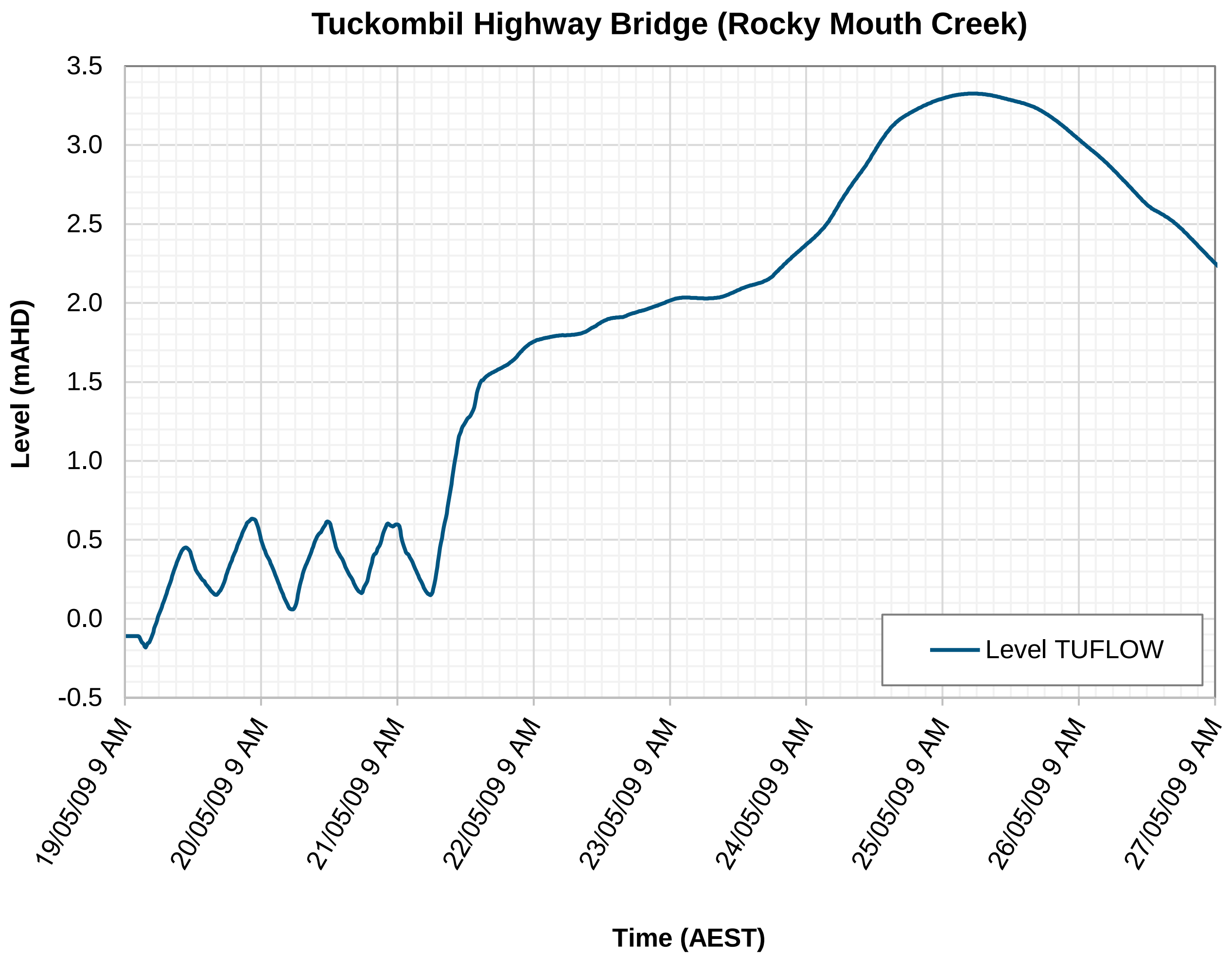


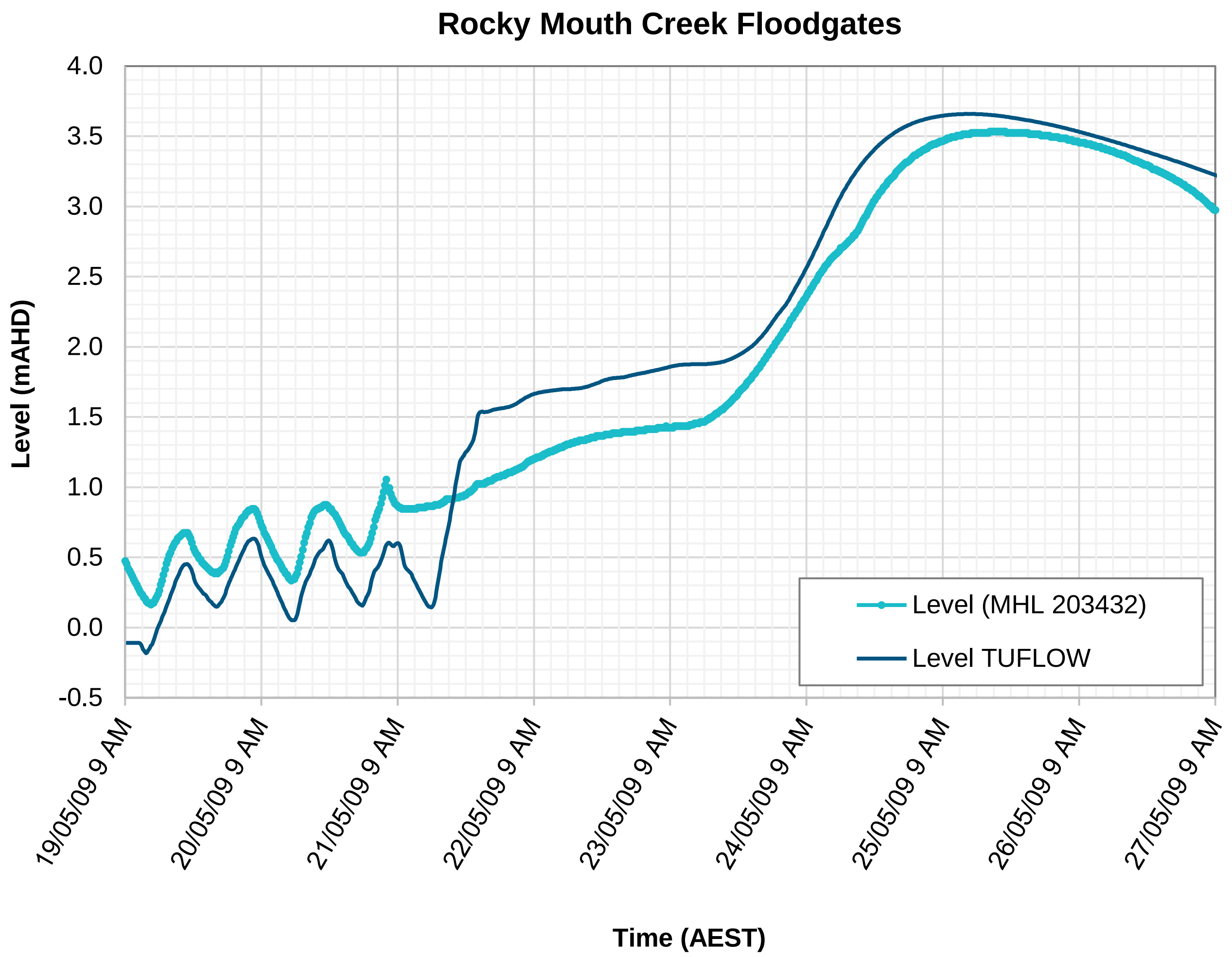


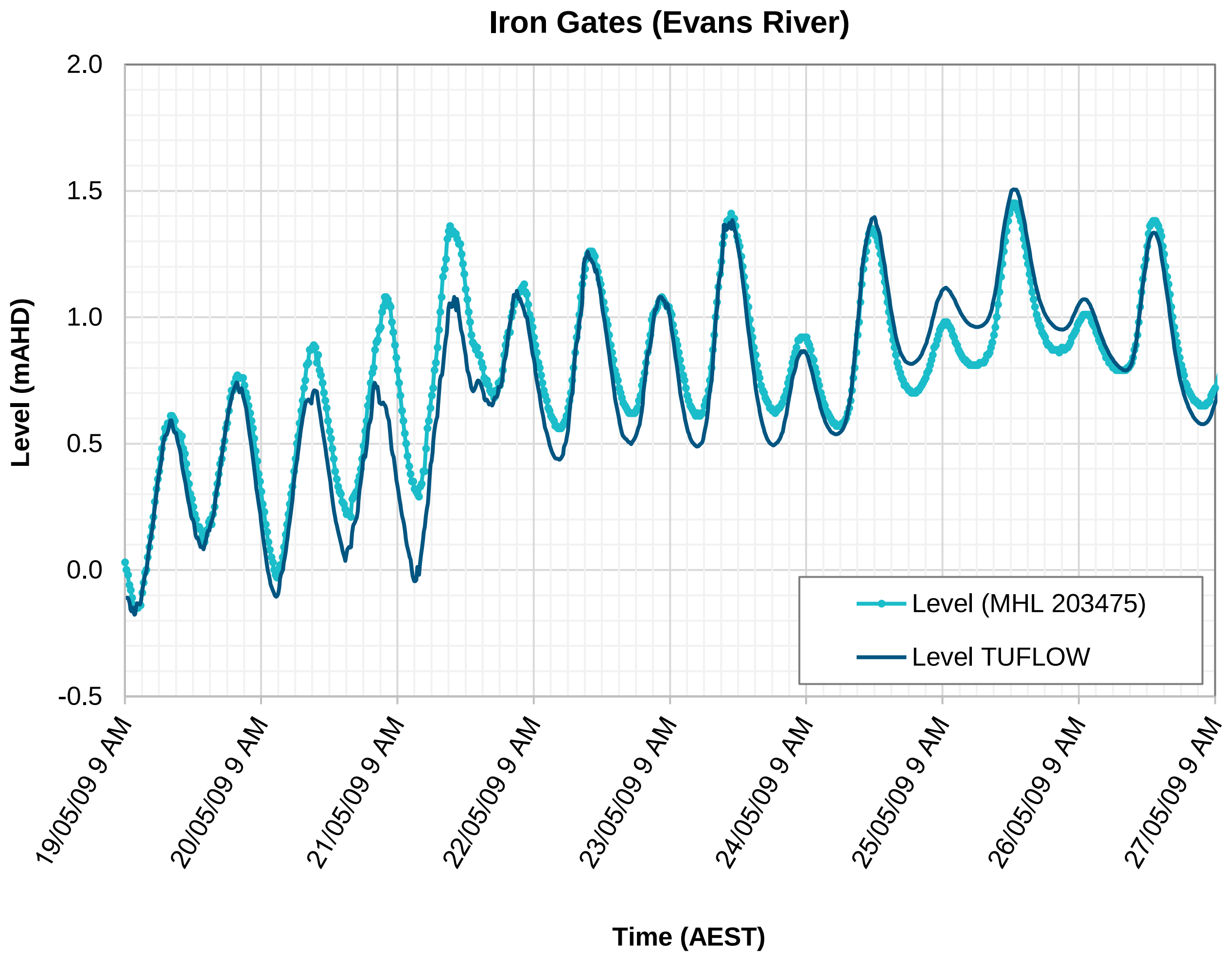


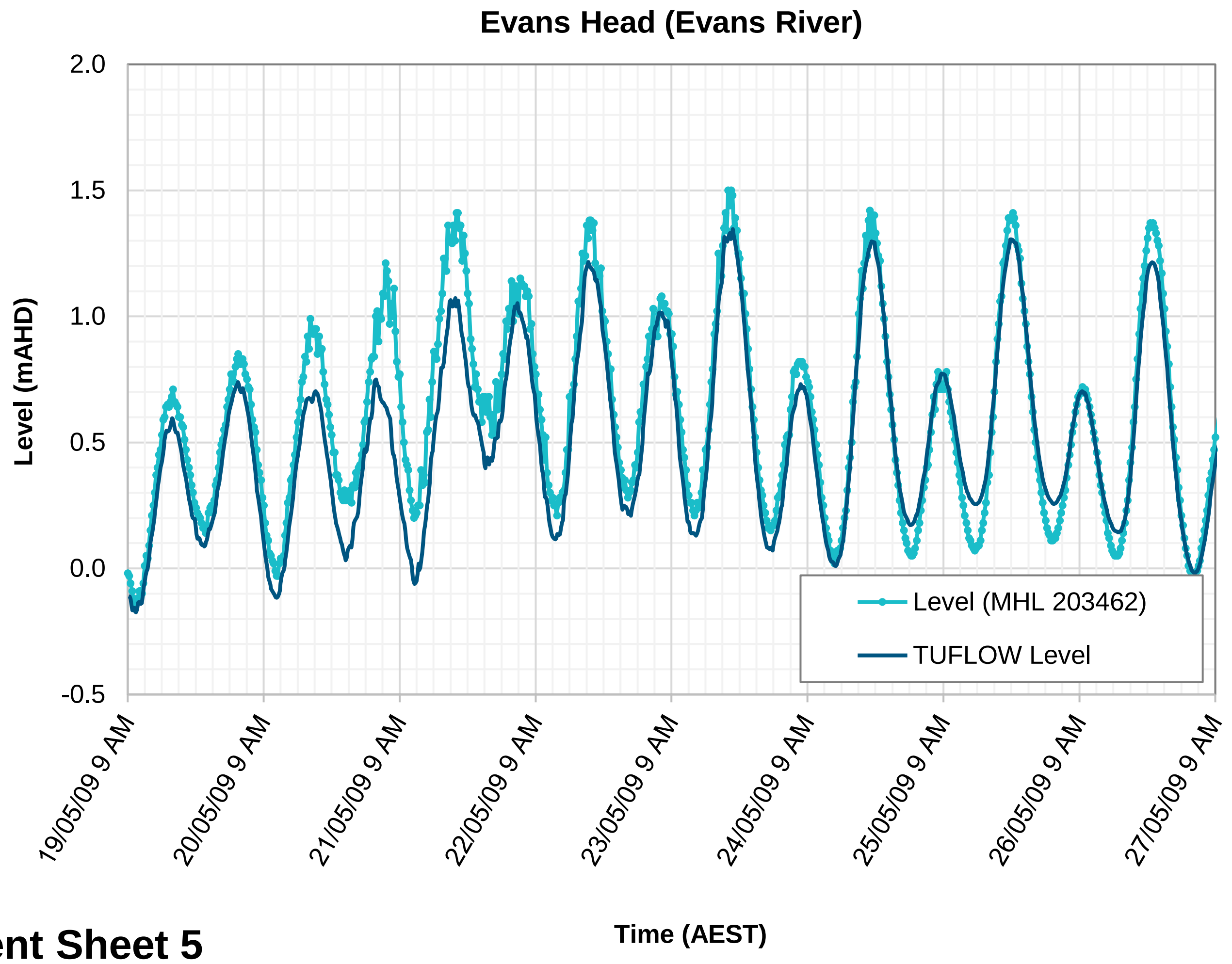


2009 Event Sheet 4









2009 Event Sheet 5

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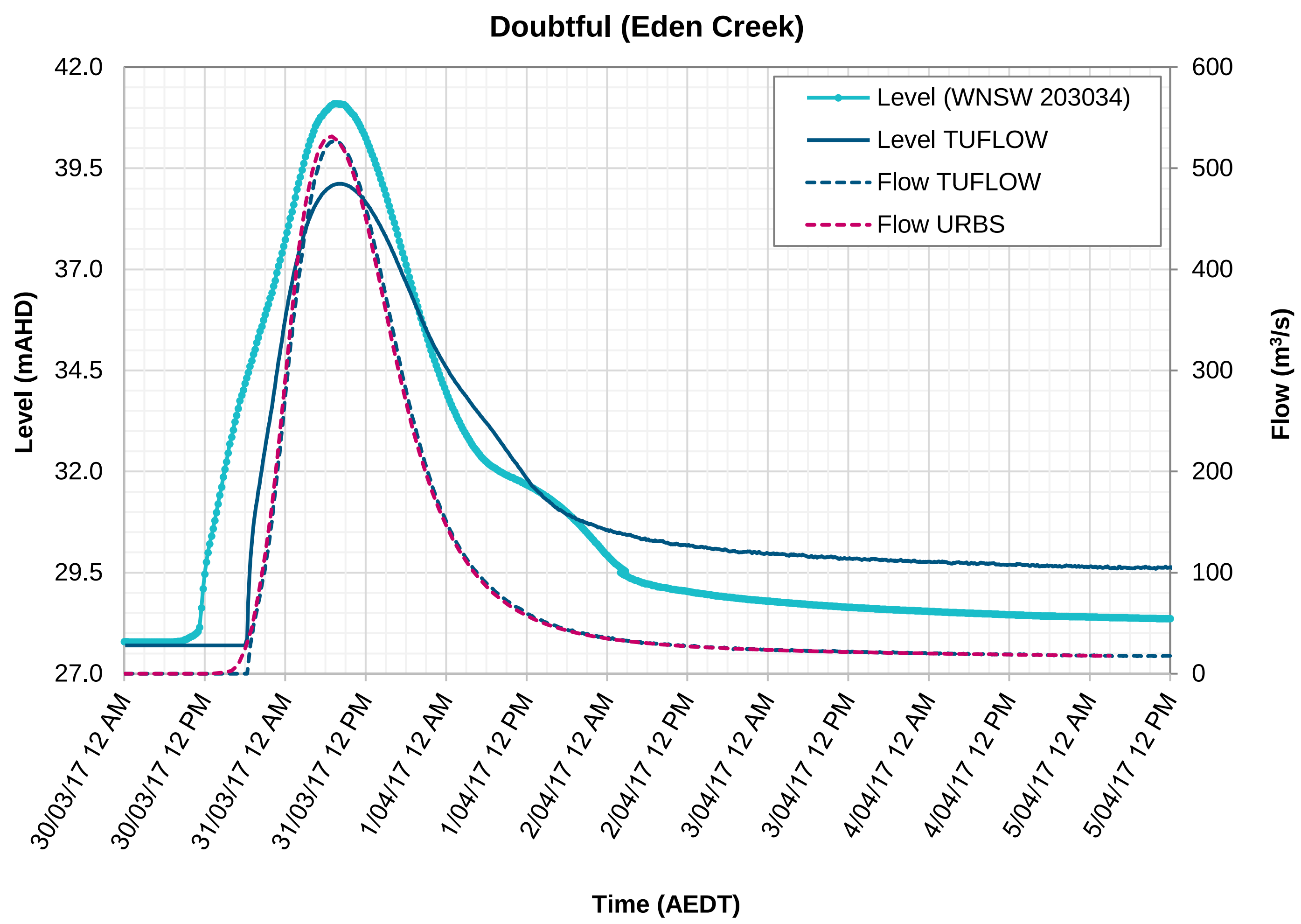

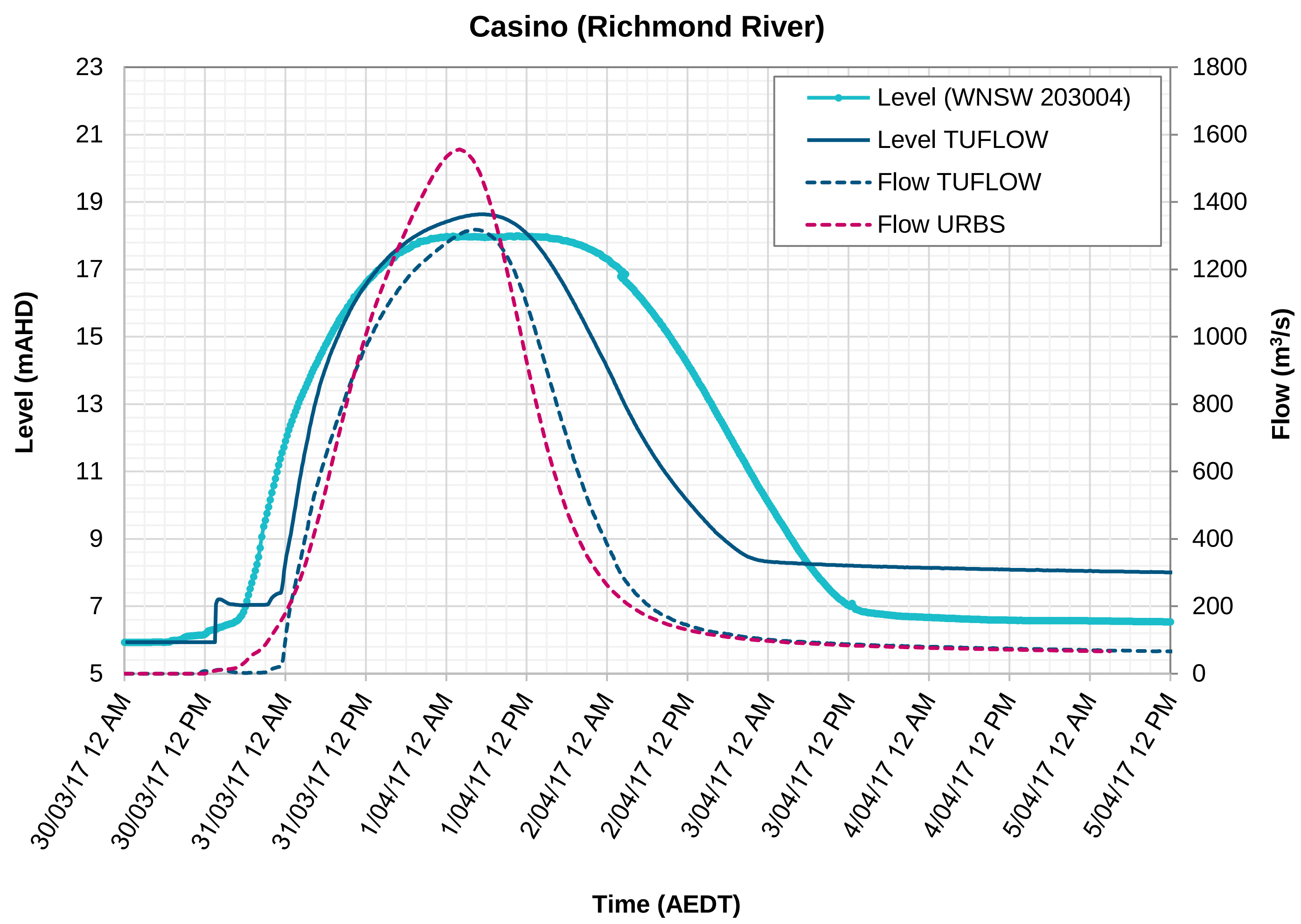

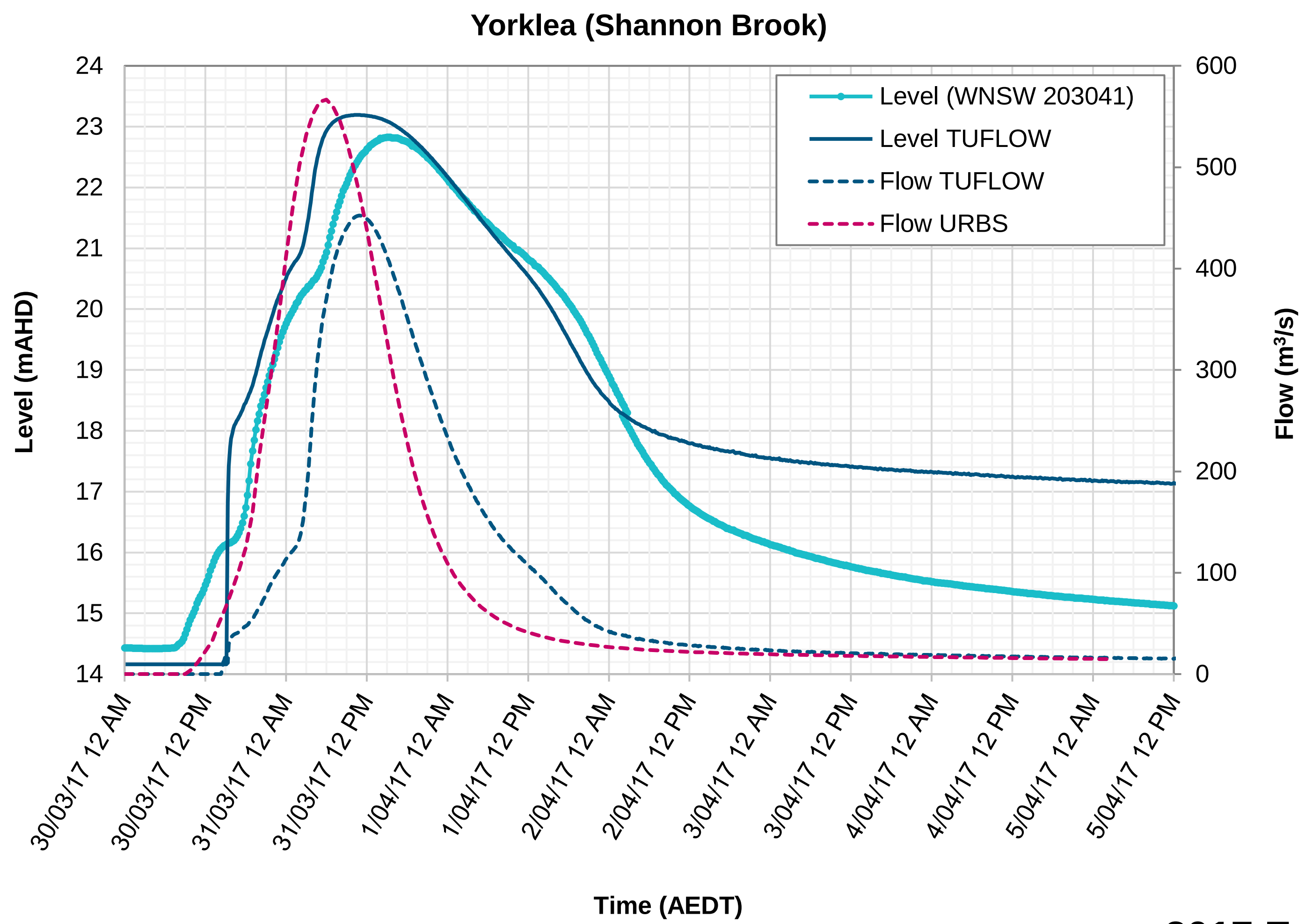

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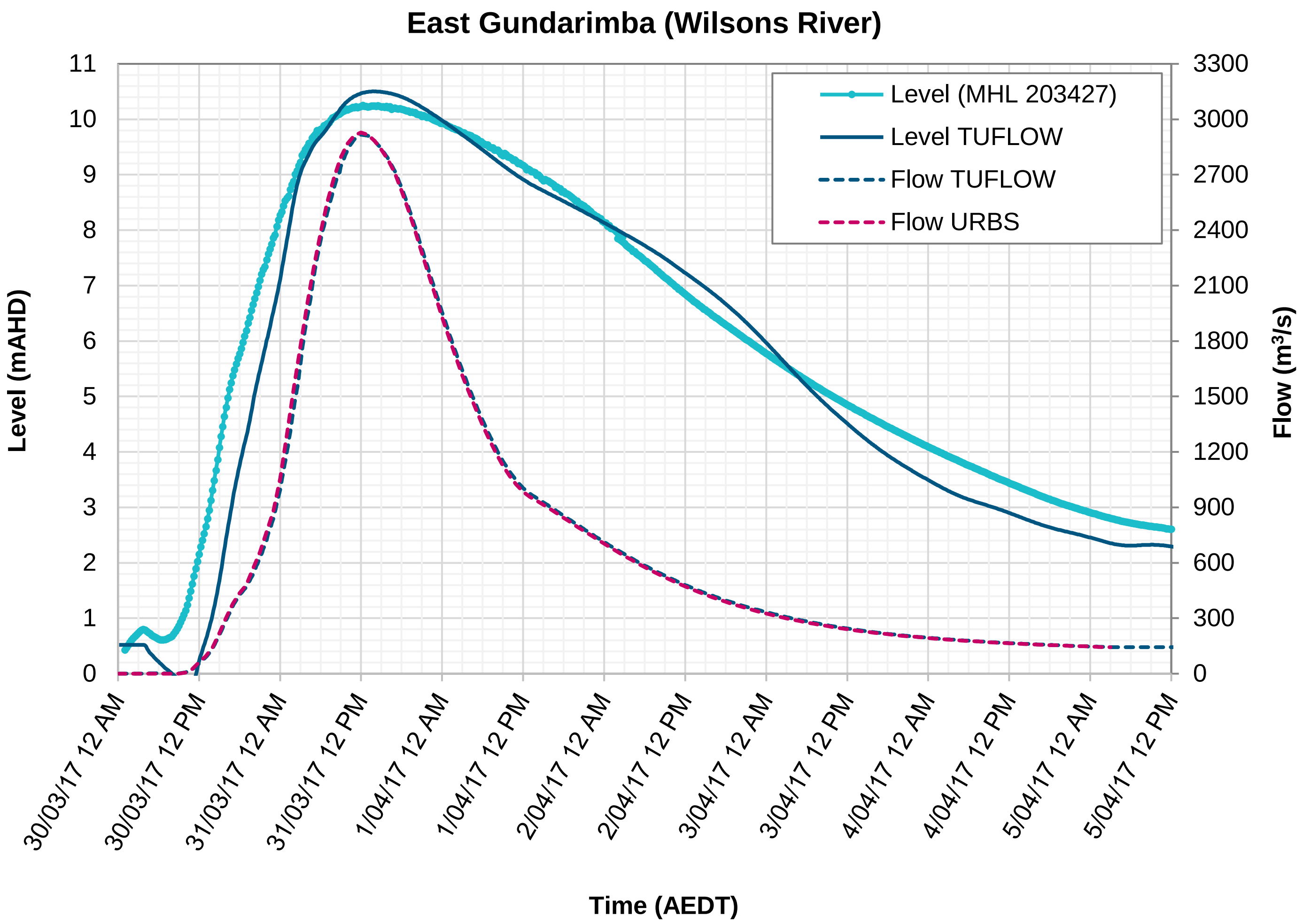

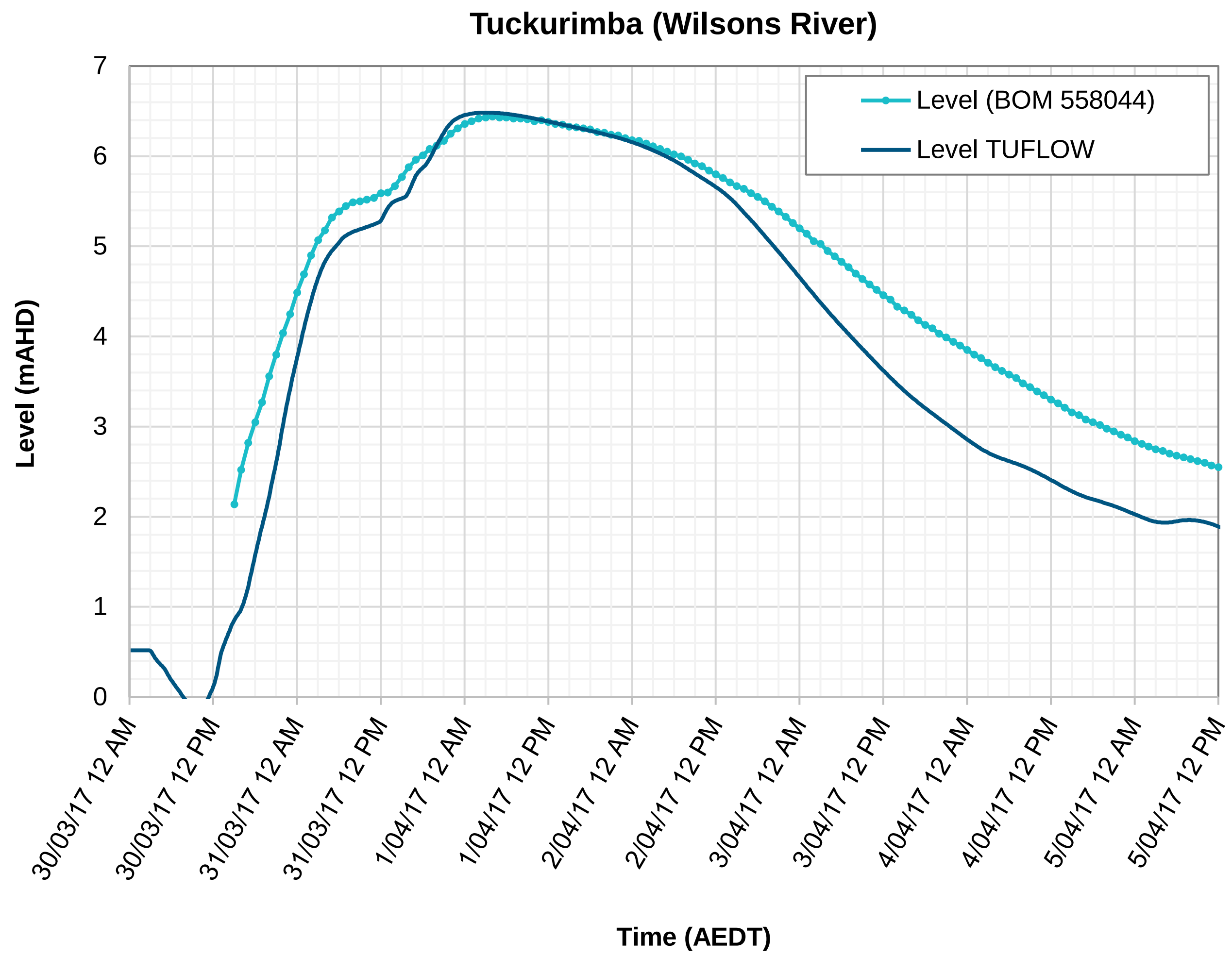

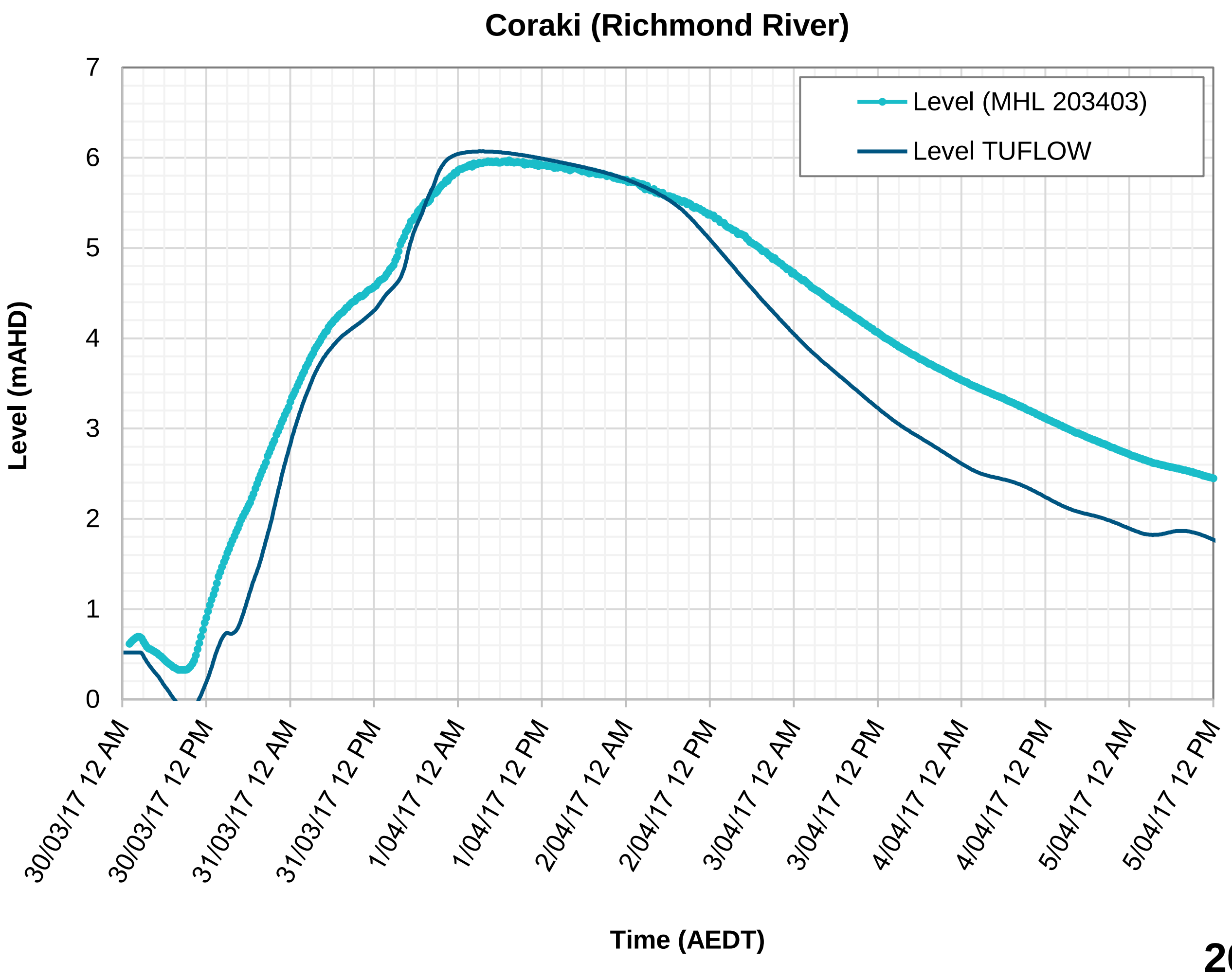

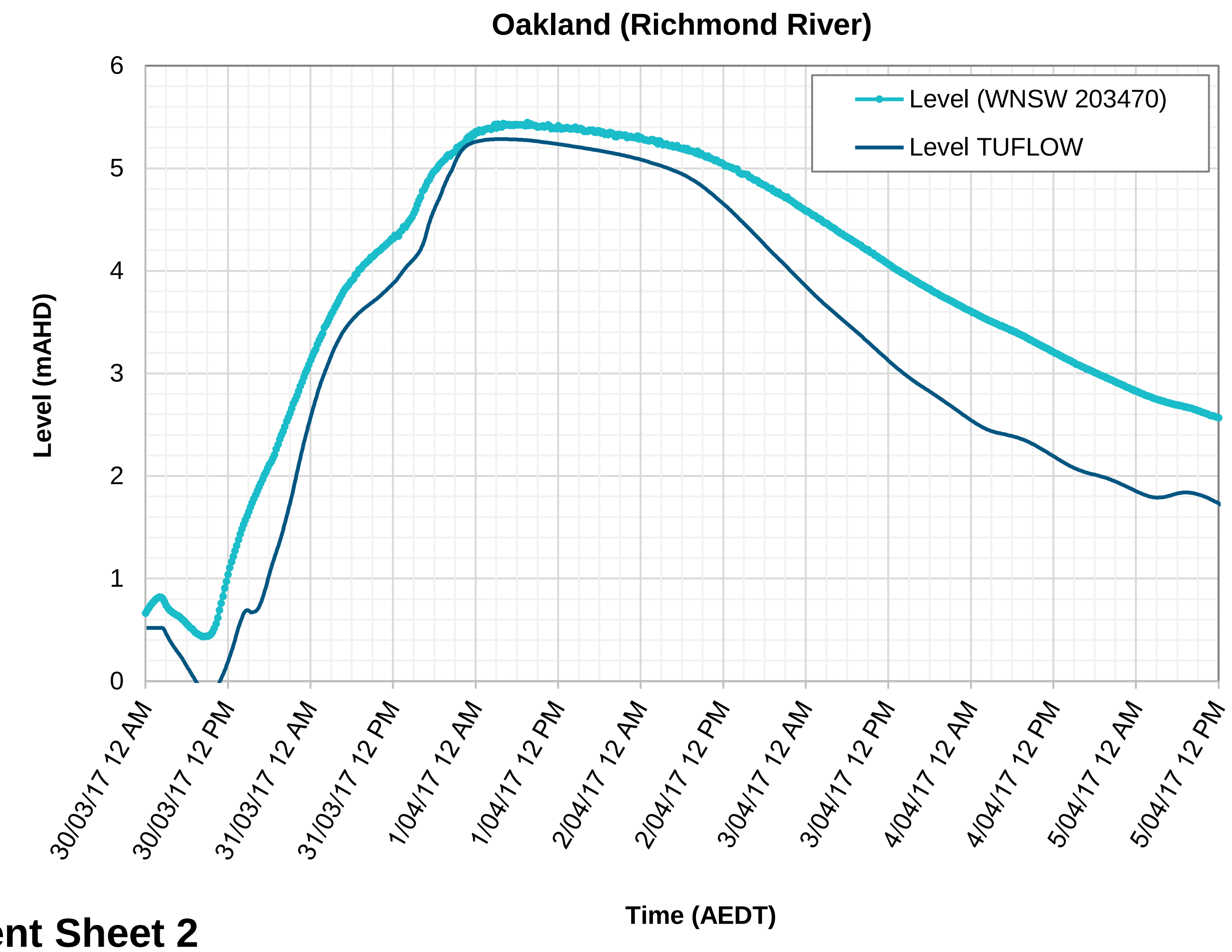

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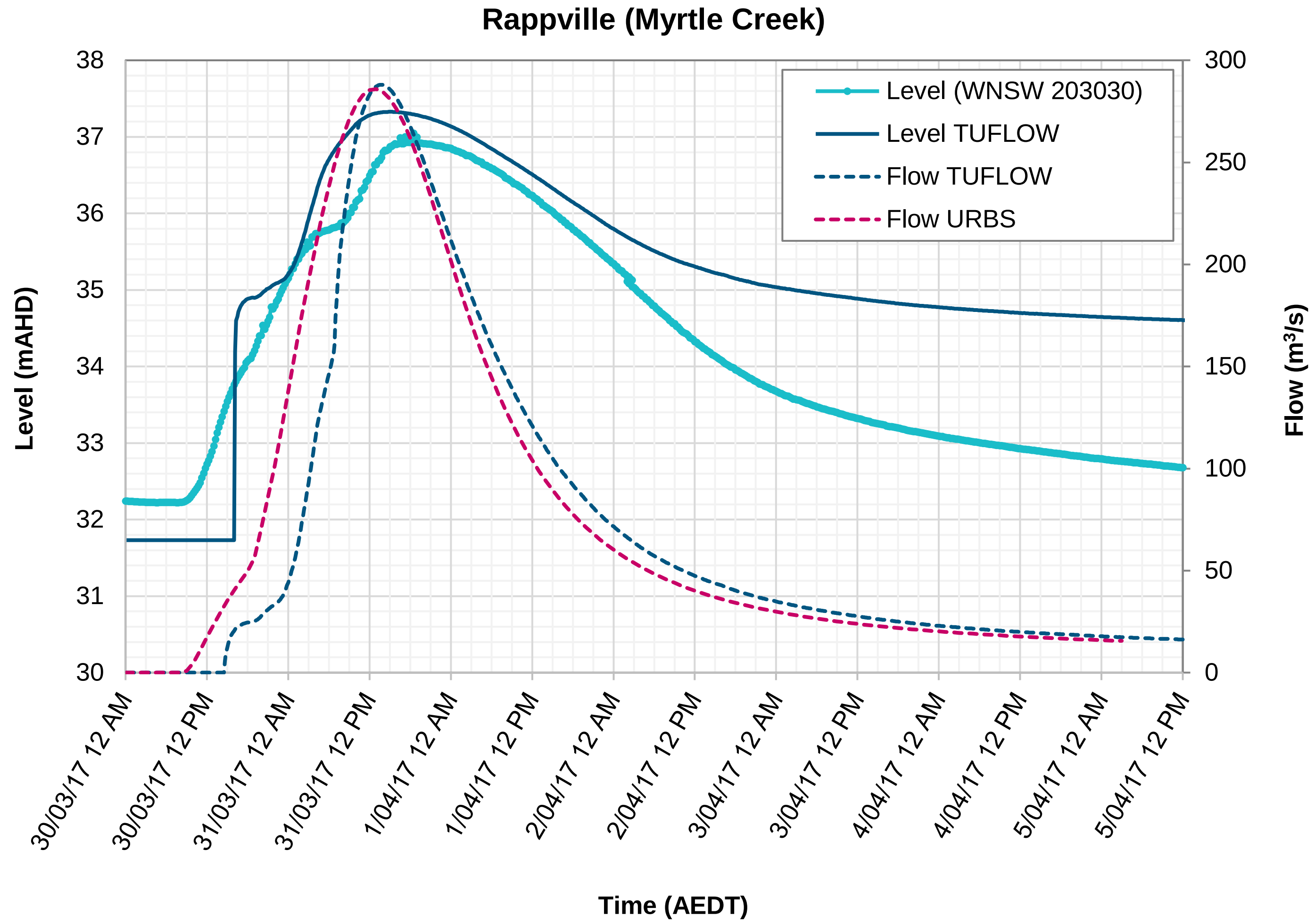
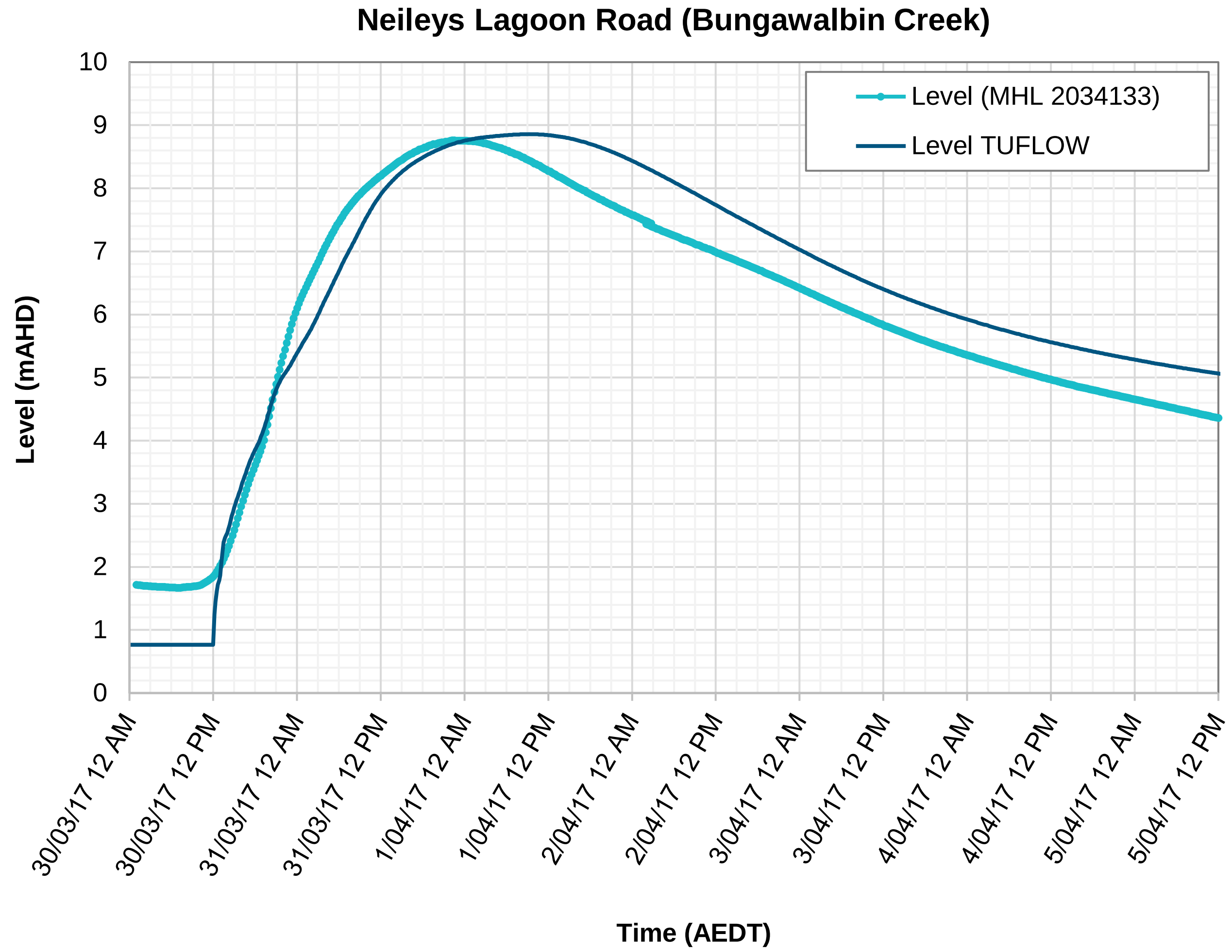
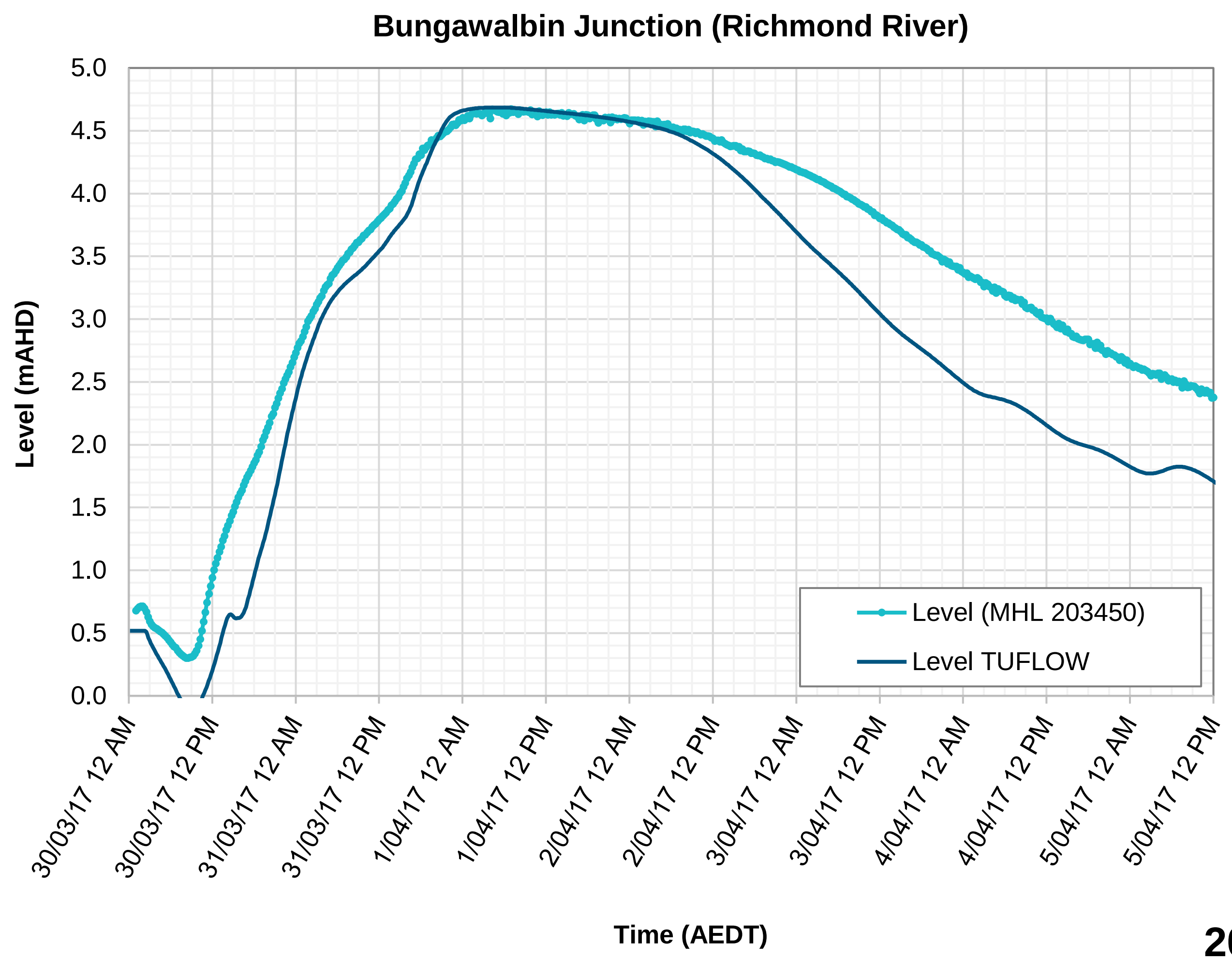
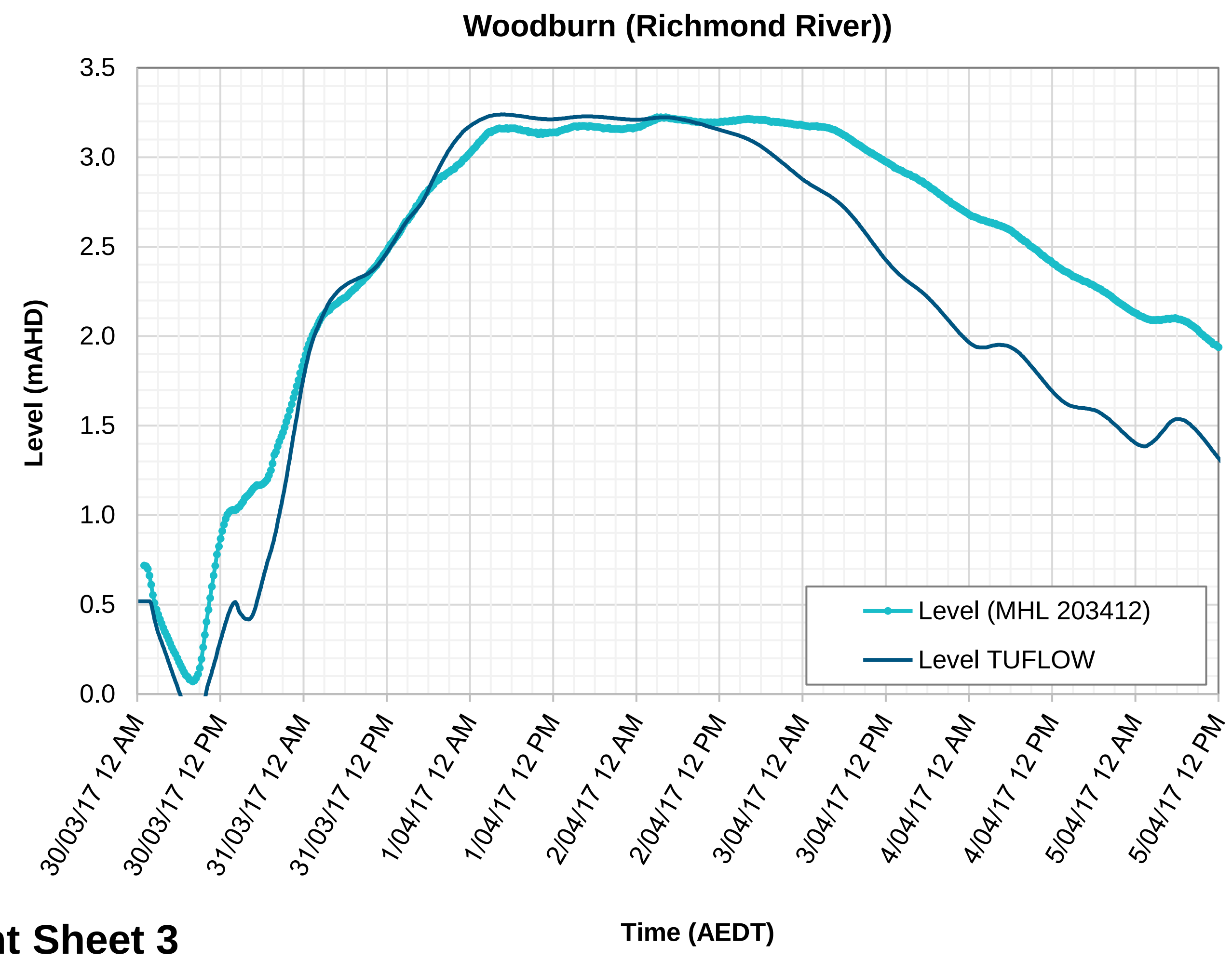
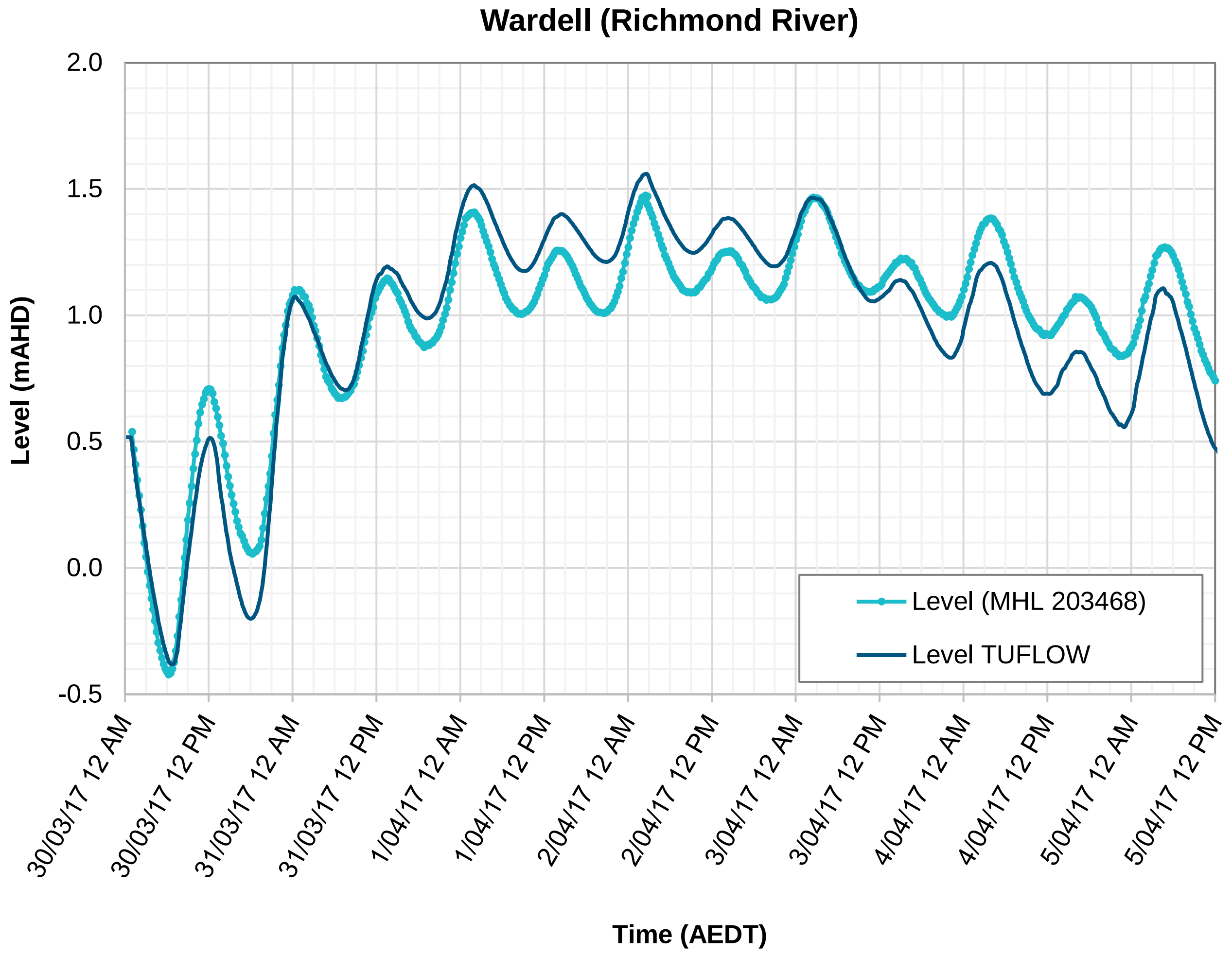


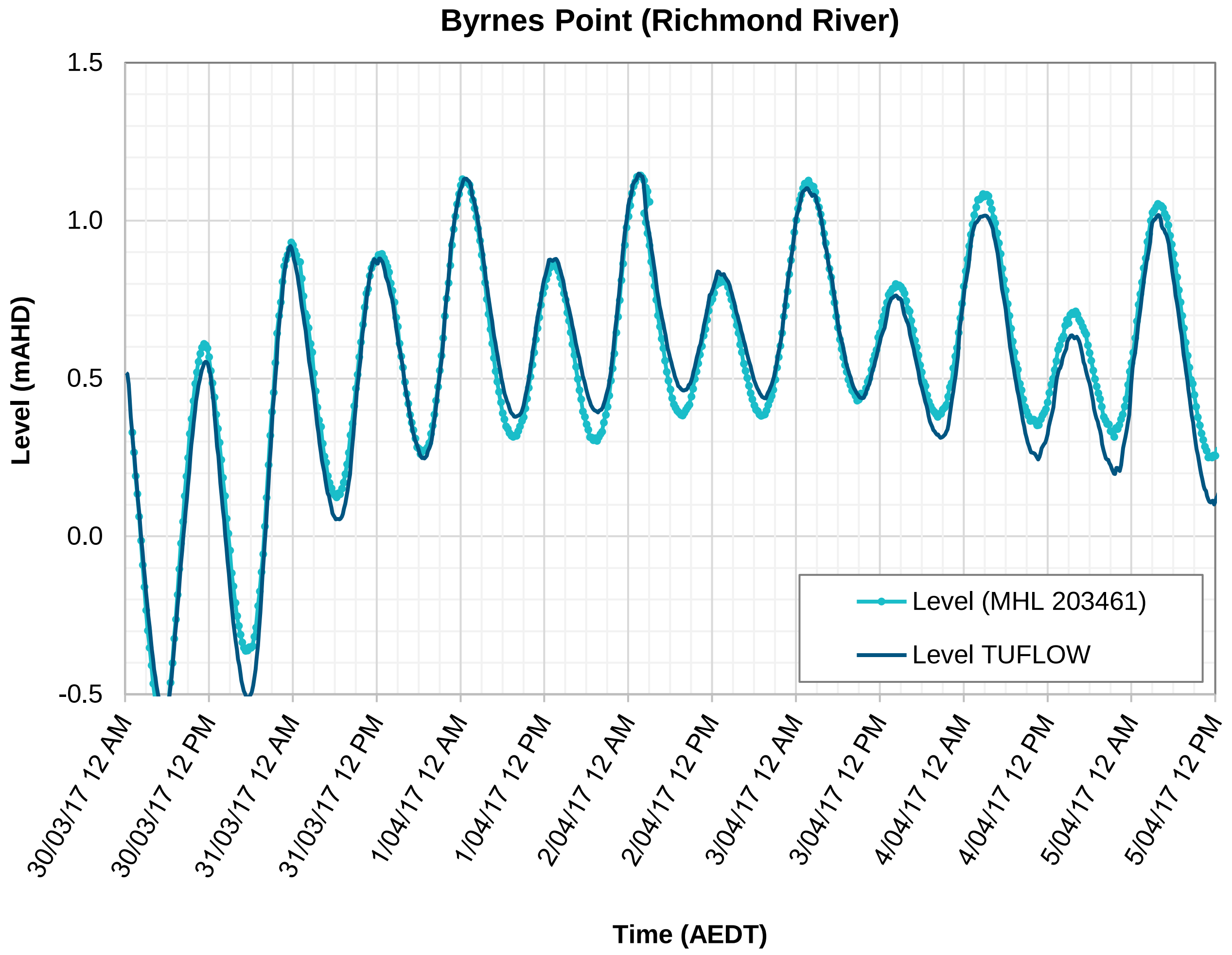


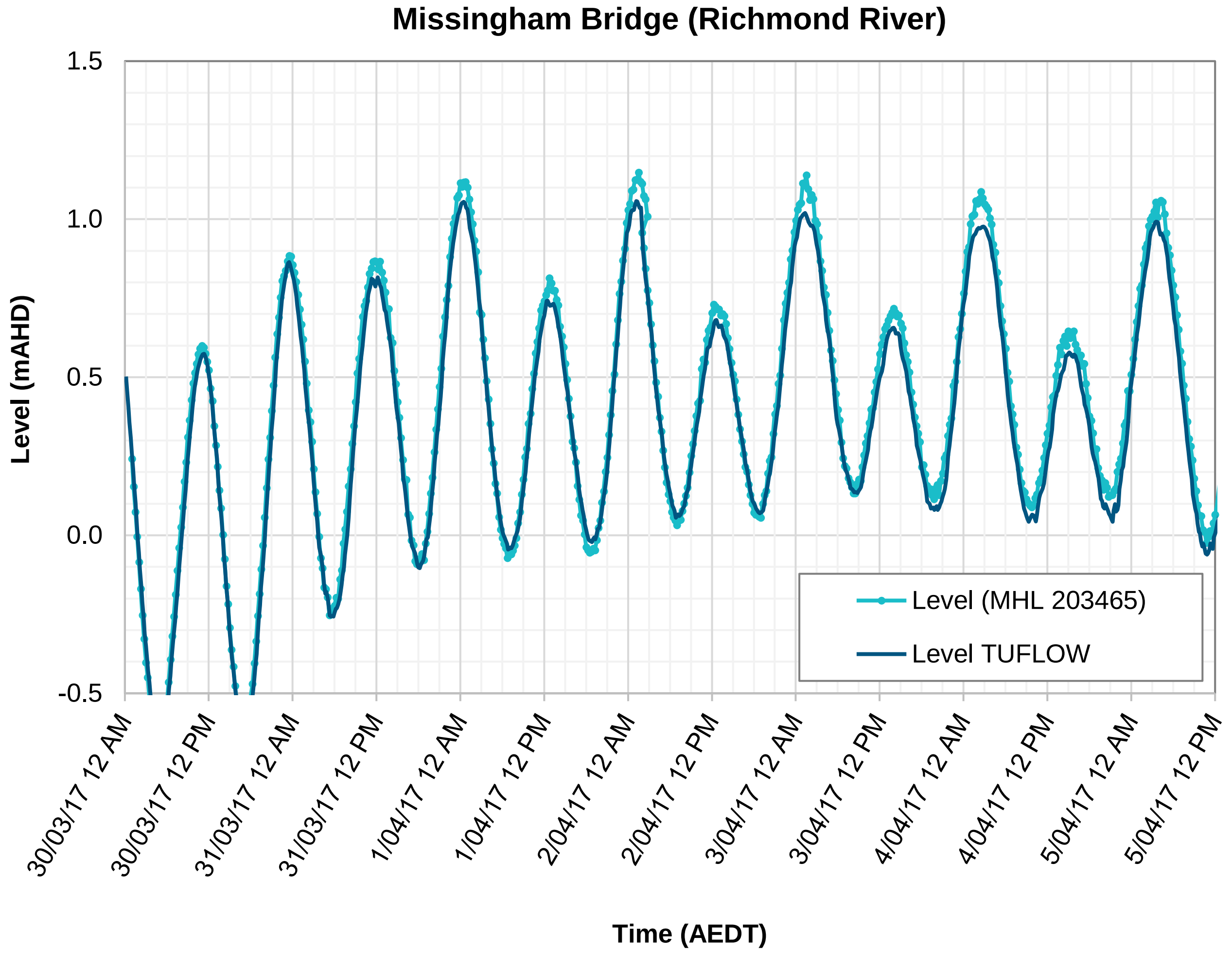


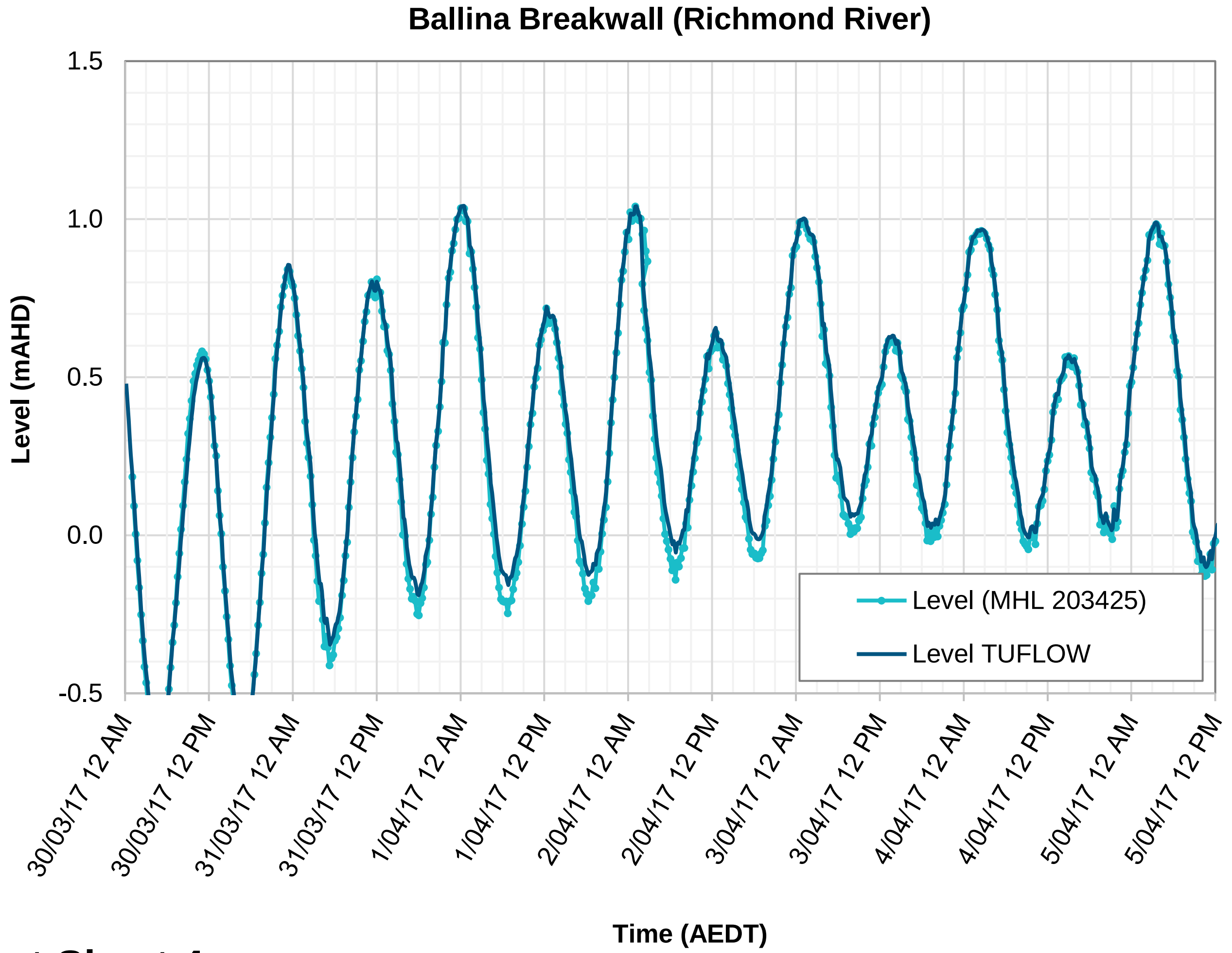


2017 Event Sheet 3

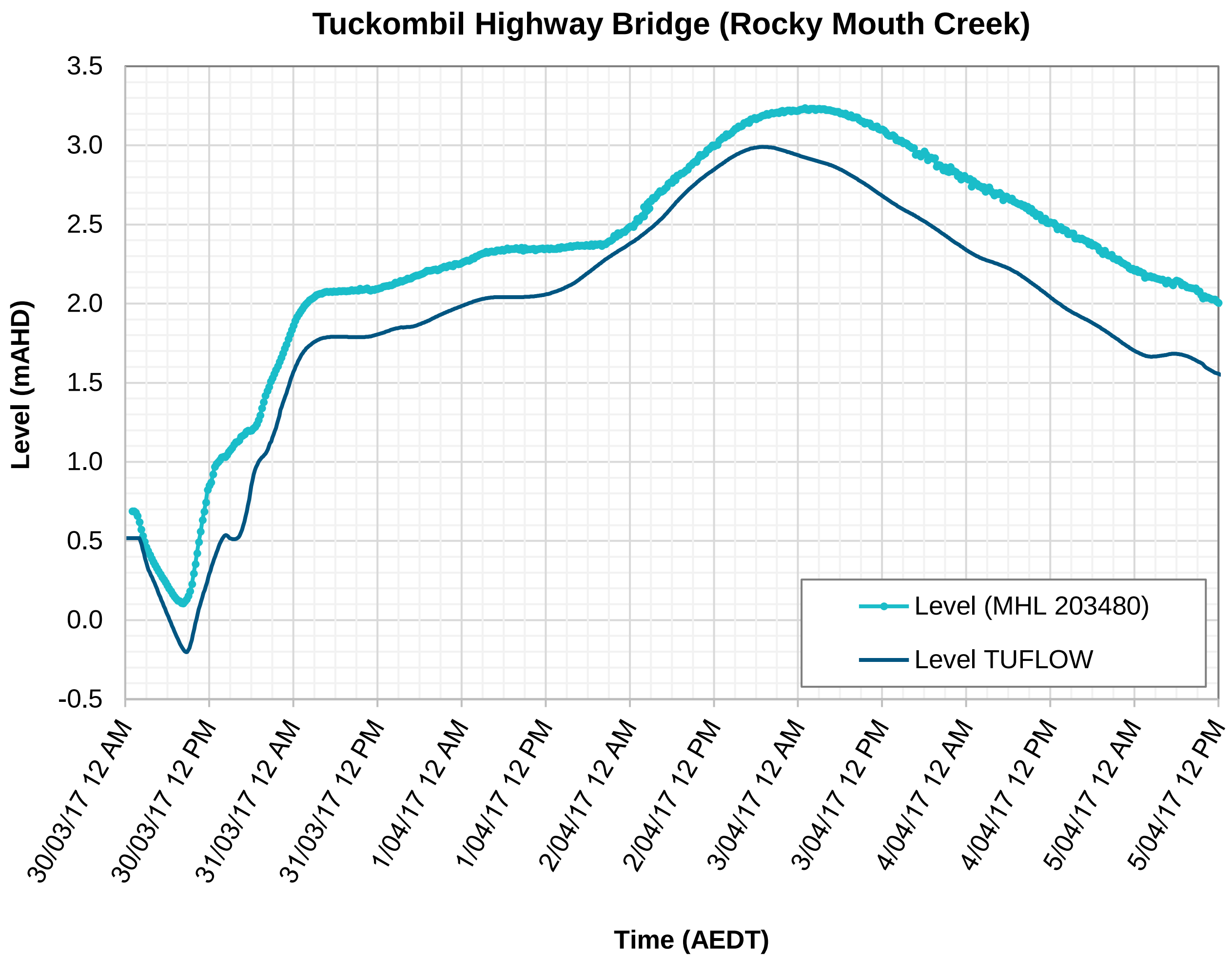


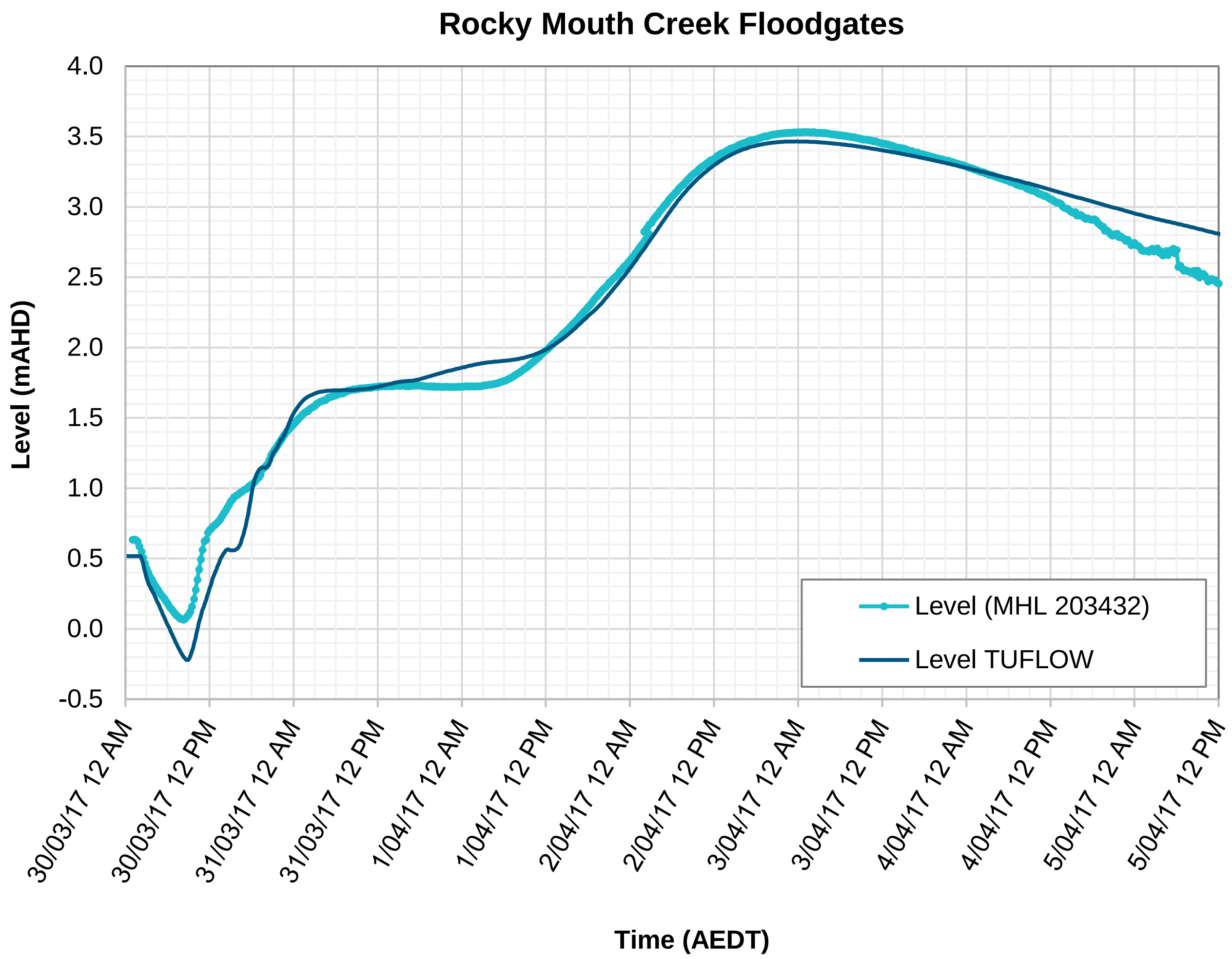


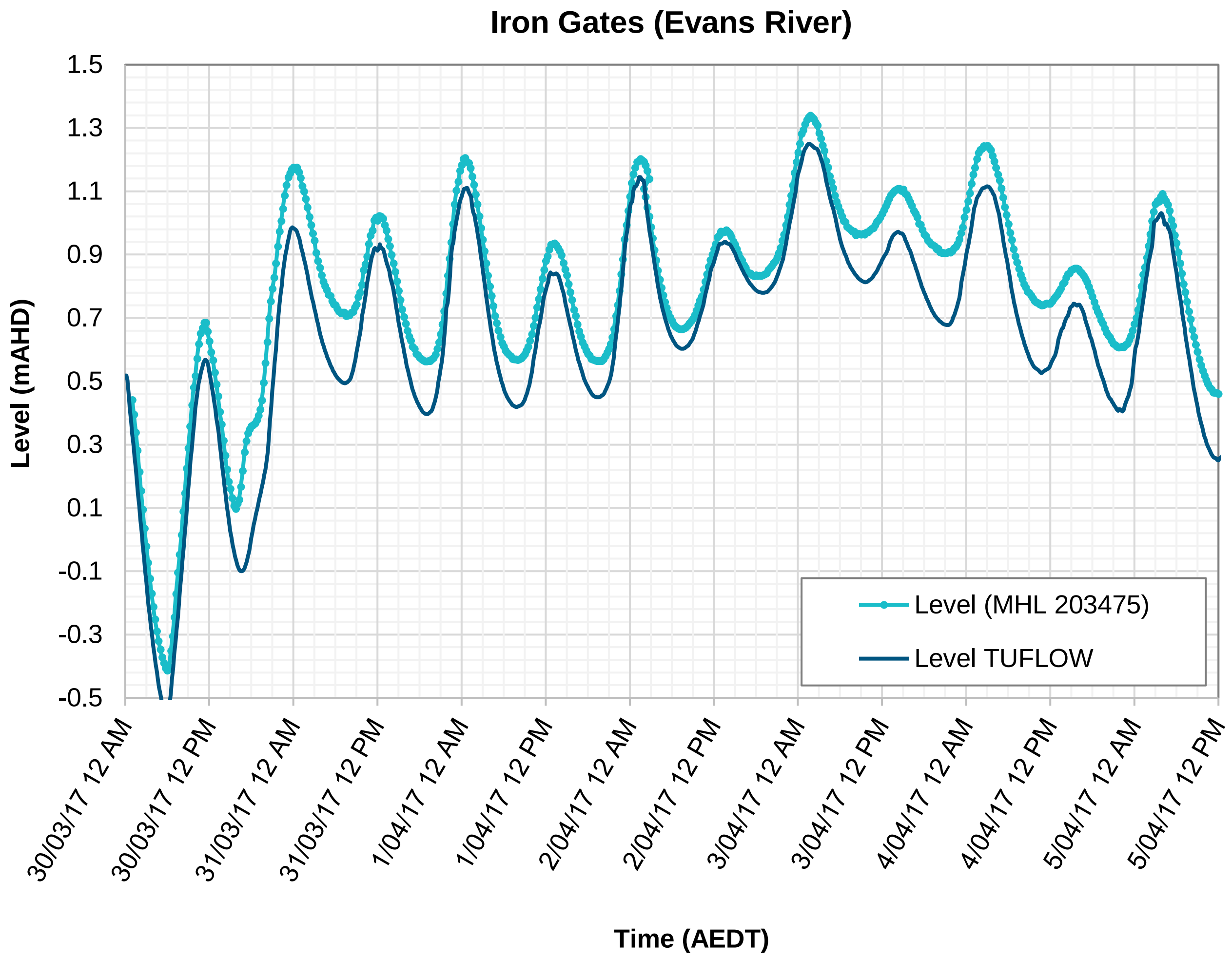


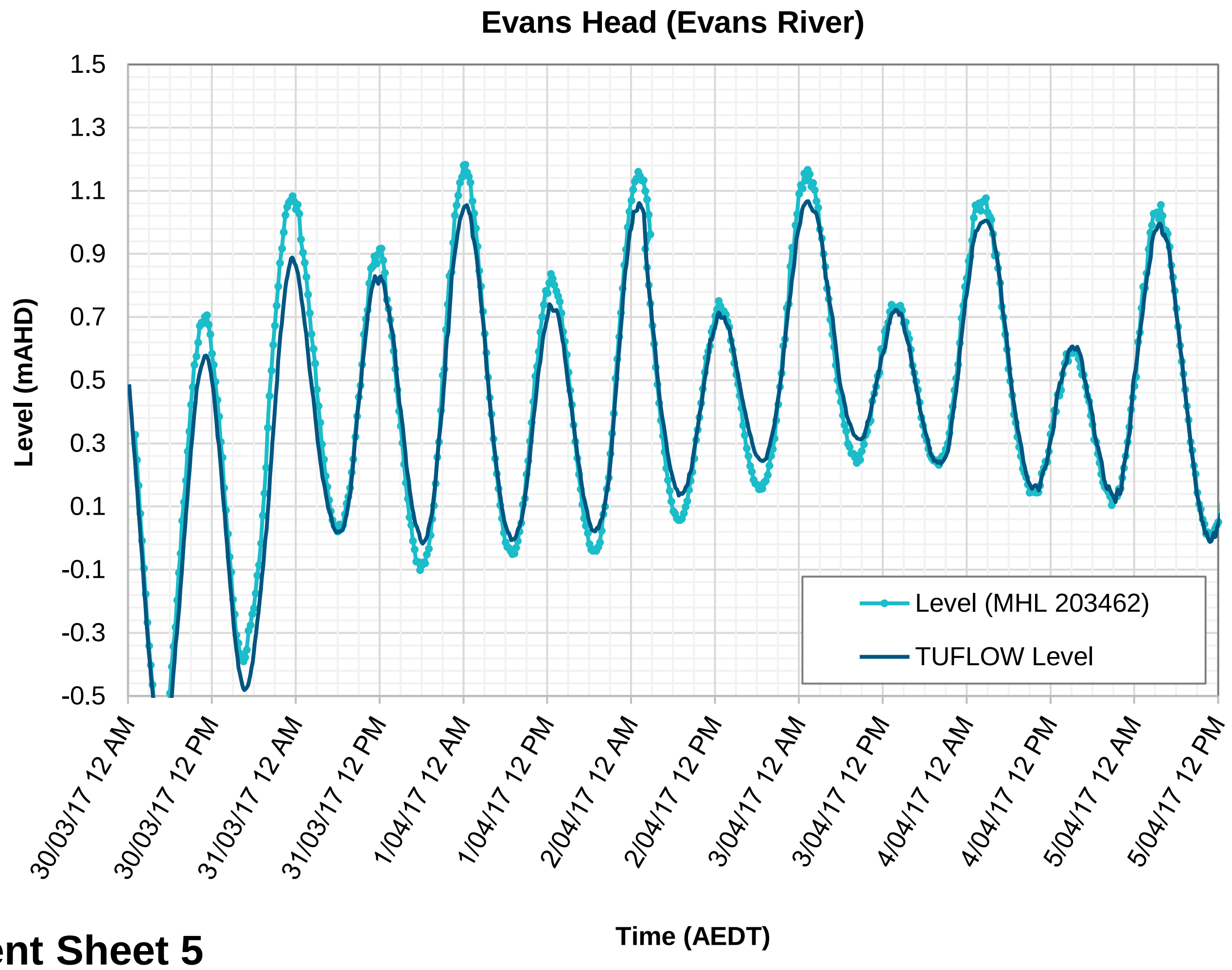


2017 Event Sheet 4



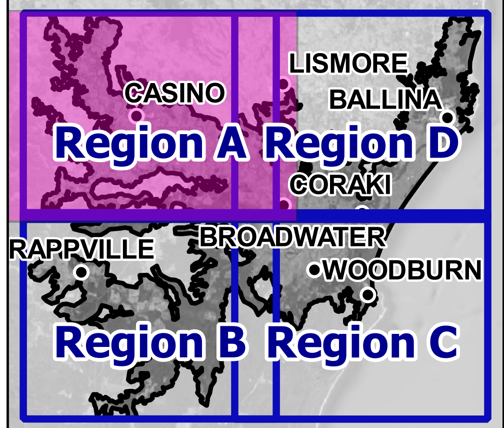






2017 Event Sheet 5

Annex B Calibration Drawings

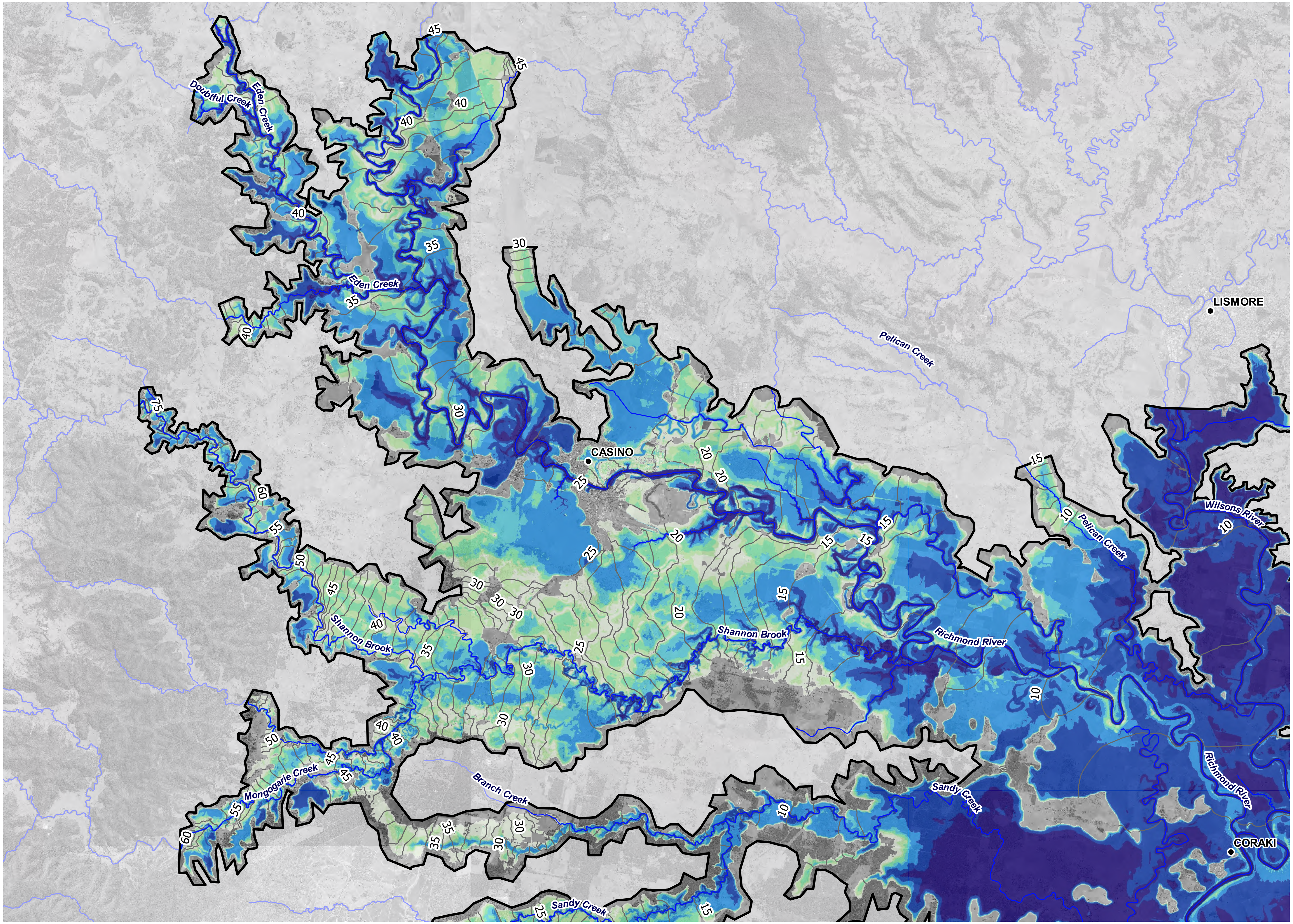


Peak flood level contours (mAHD)

February-March 2022 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region A - February-March 2022 Event Peak Flood Depth

Drawing: B-1

Rev: A

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N 0 3,000 6,000 m

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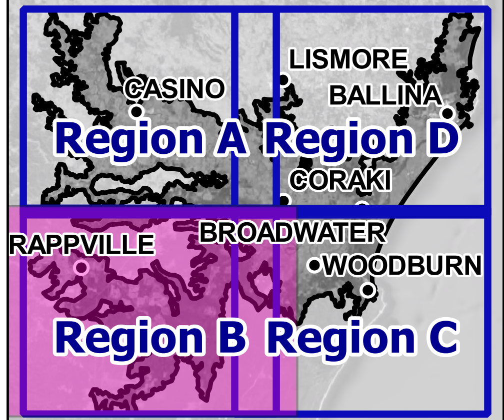
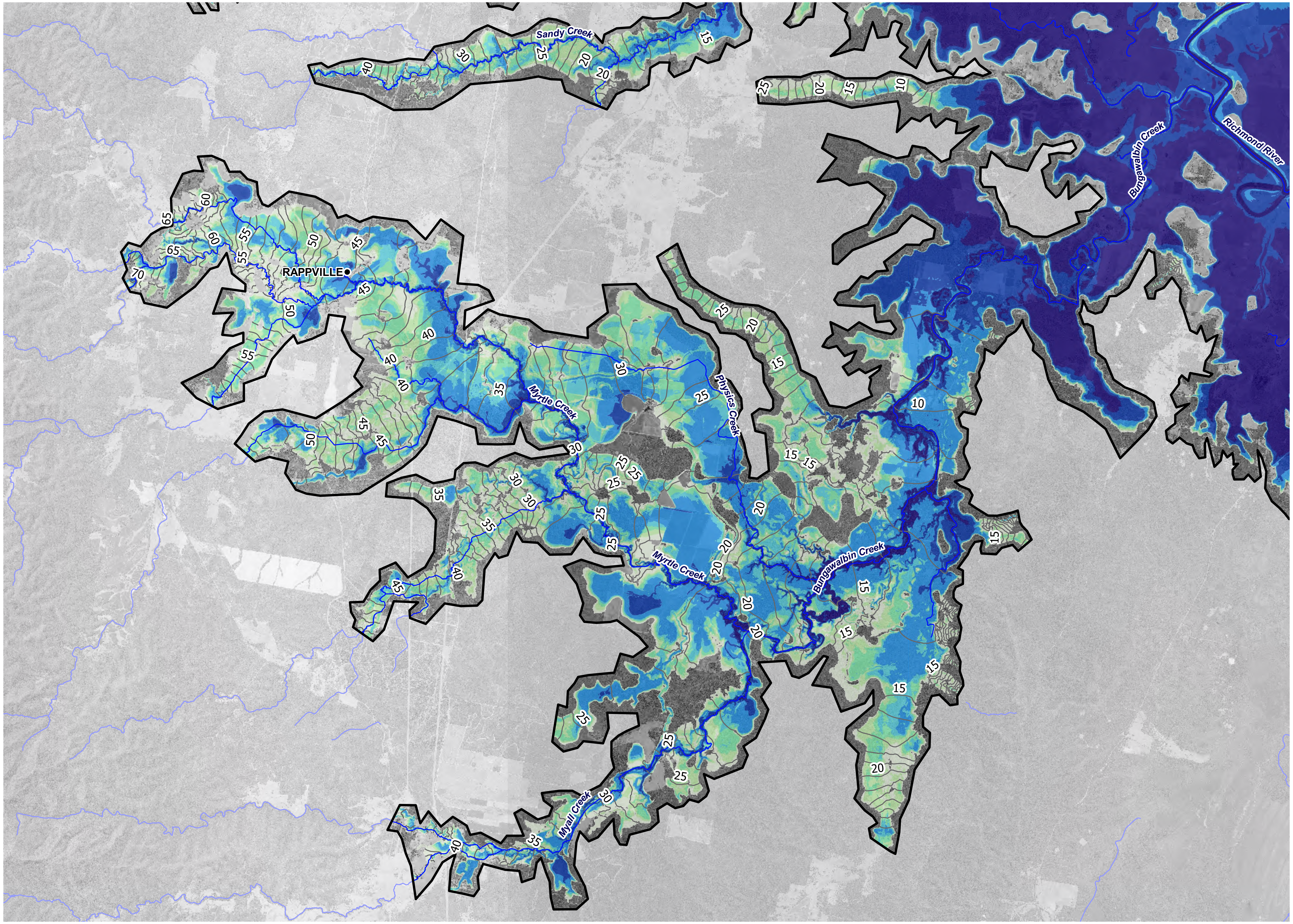


Peak flood level contours (mAHD)

February-March 2022 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region B - February-March 2022 Event Peak Flood Depth

Drawing: B-2

Rev: A

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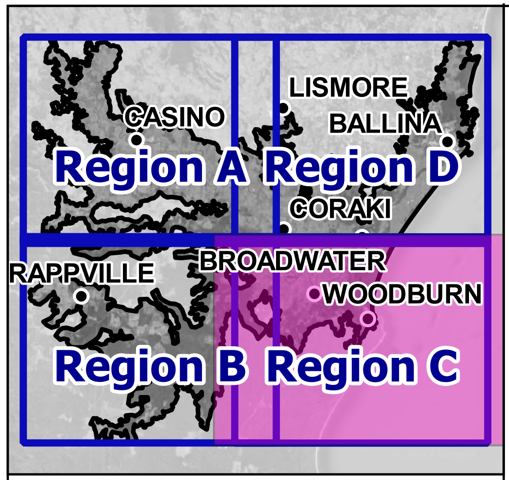
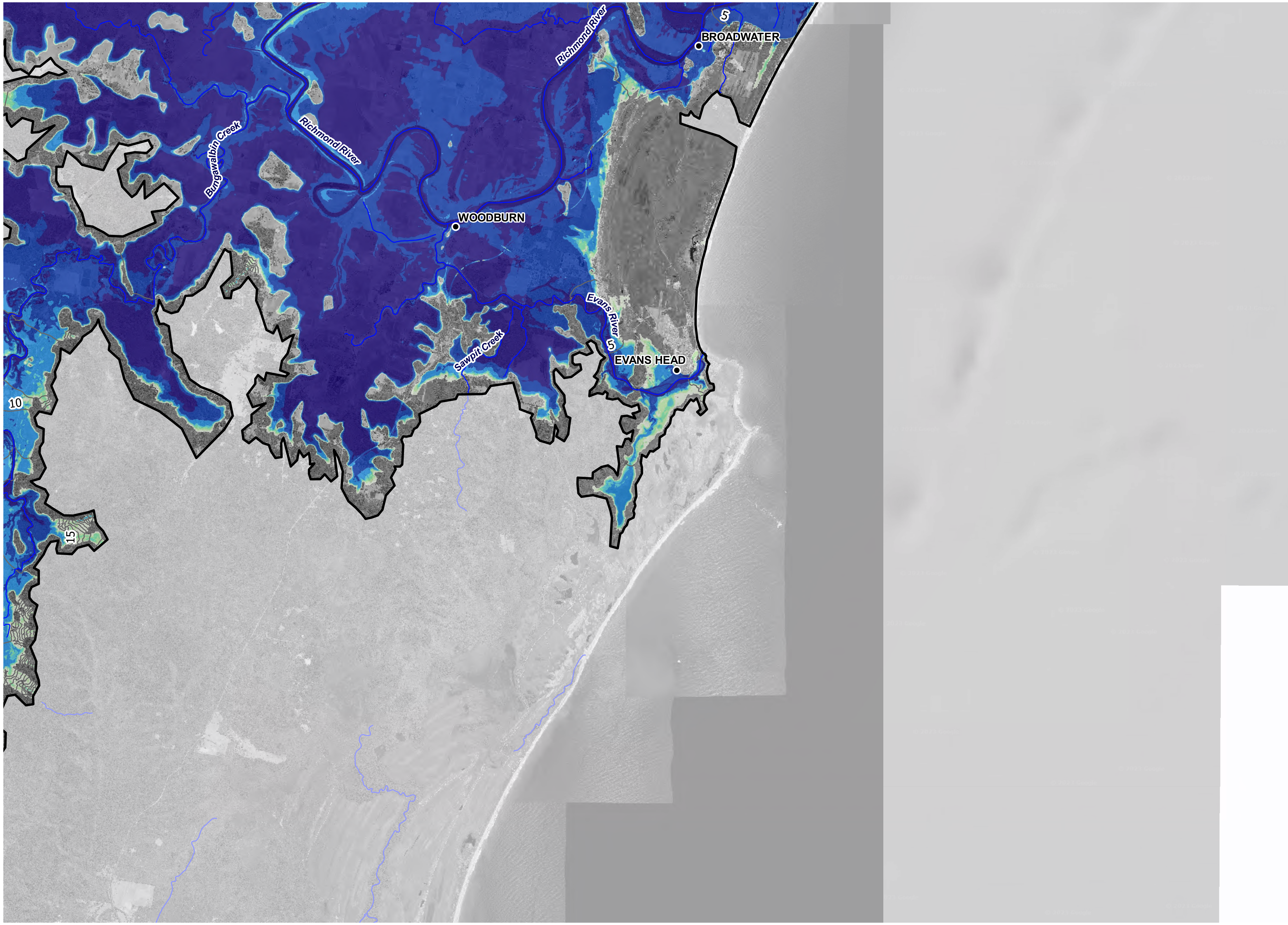


Peak flood level contours (mAHD)

February-March 2022 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region C - February-March 2022 Event Peak Flood Depth

Drawing: B-3

Rev: A

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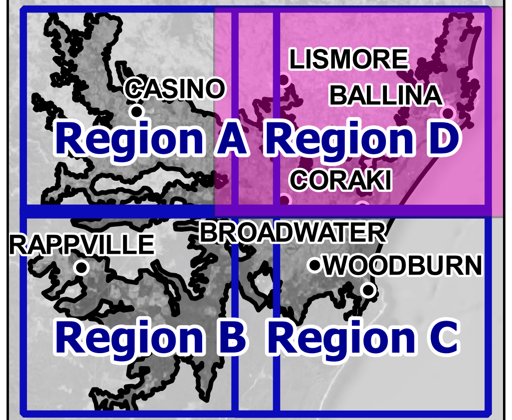
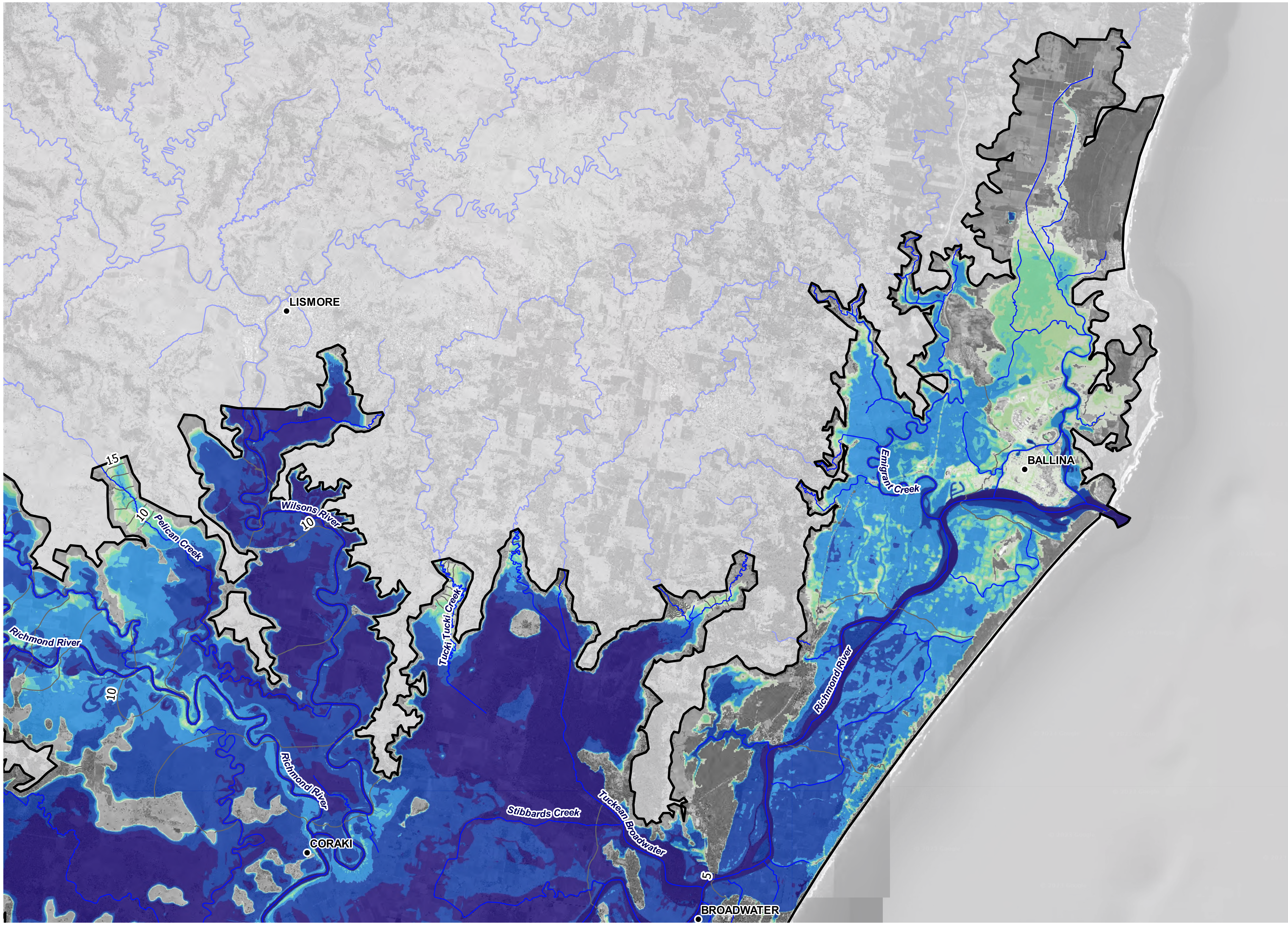


Peak flood level contours (mAHD)

February-March 2022 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region D - February-March 2022 Event Peak Flood Depth

Drawing:	Rev:
B-4	A

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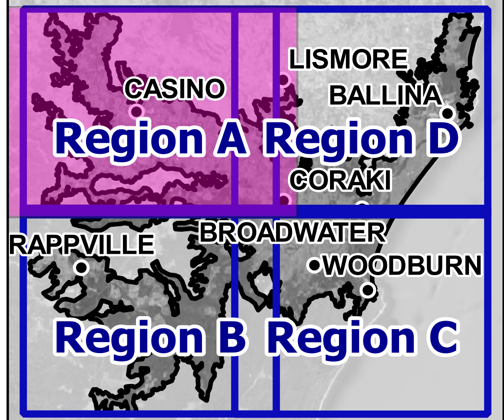
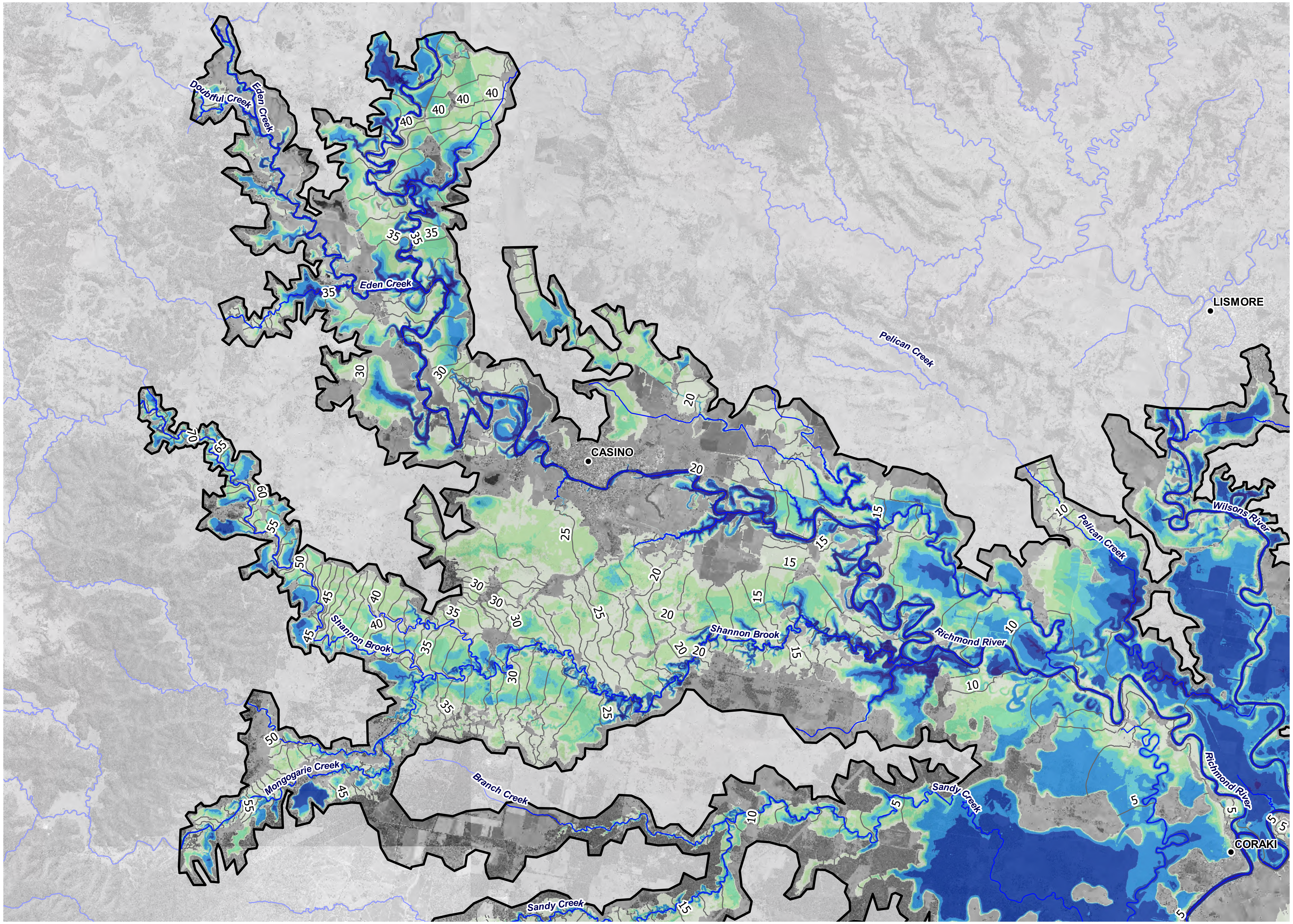


Peak flood level contours (mAHD)

January 2008 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- • Towns
- Model Extent
- Watercourses

Title:

Region A - January 2008 Event Peak Flood Depth

Drawing: **B-5**

Rev: **A**

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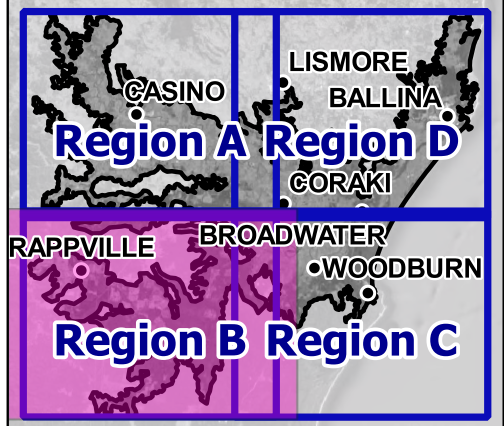
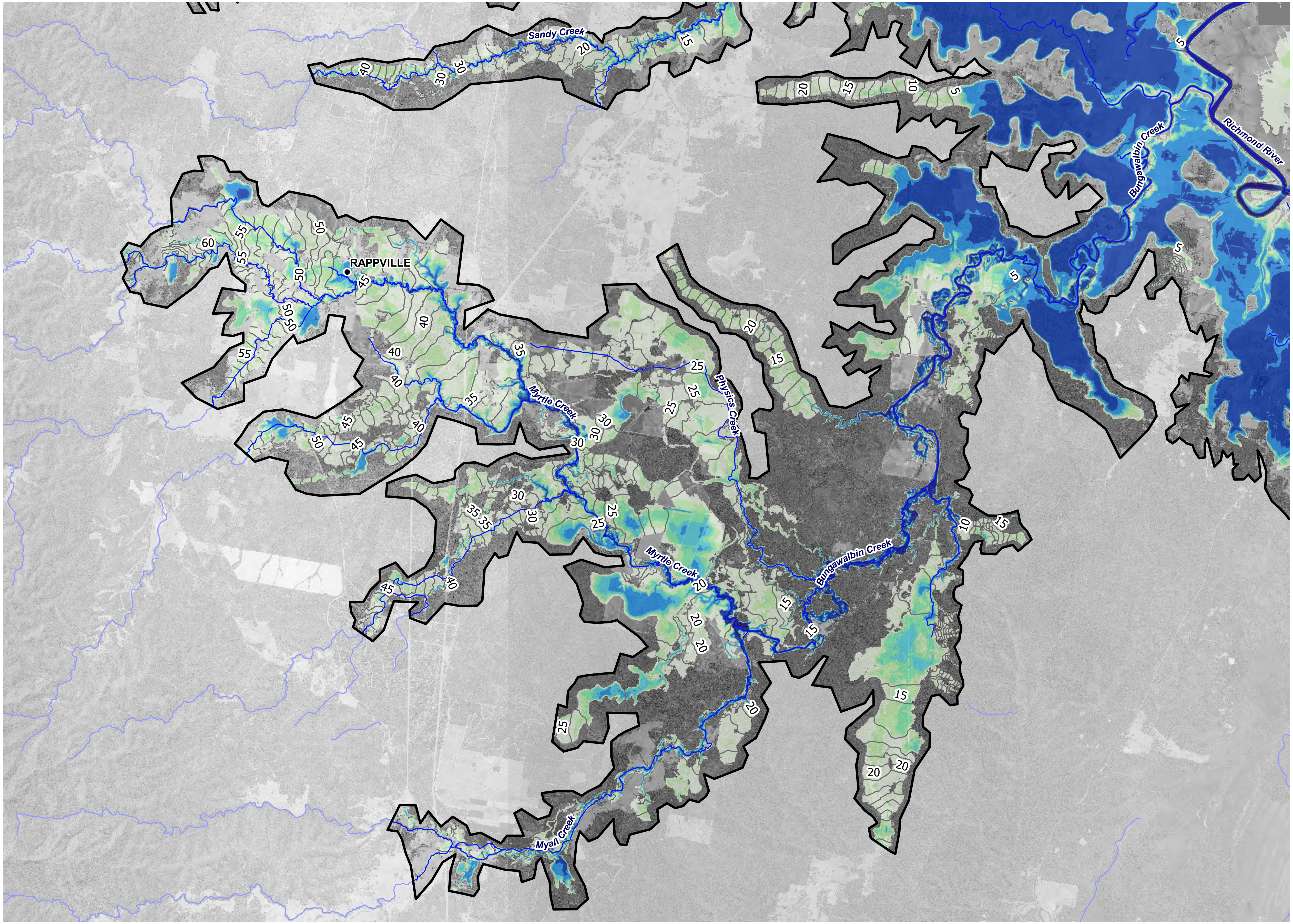


Peak flood level contours (mAHD)

January 2008 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region B - January 2008 Event Peak Flood Depth

Drawing: B-6

Rev: A

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N 0 3,000 6,000 m

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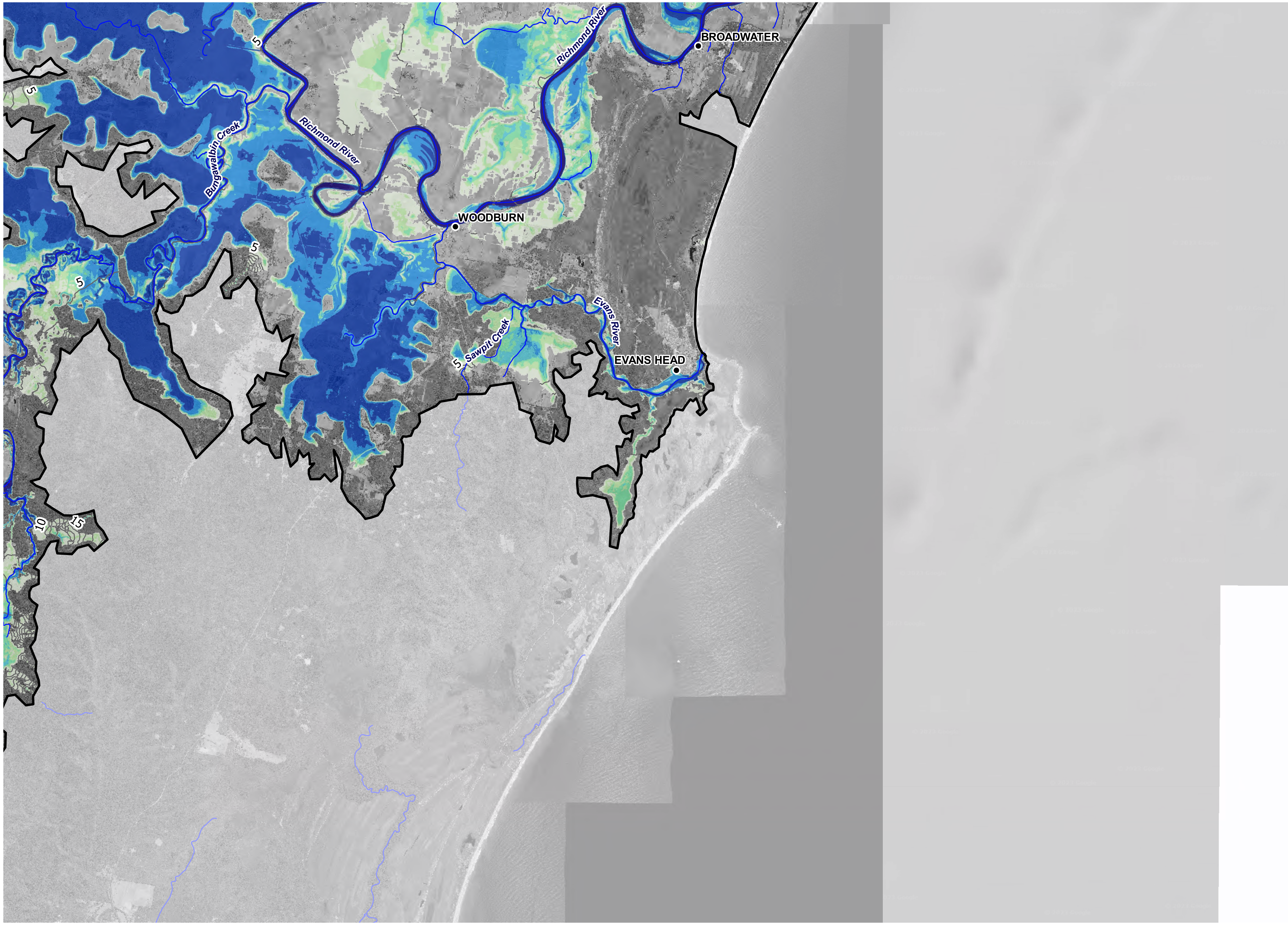

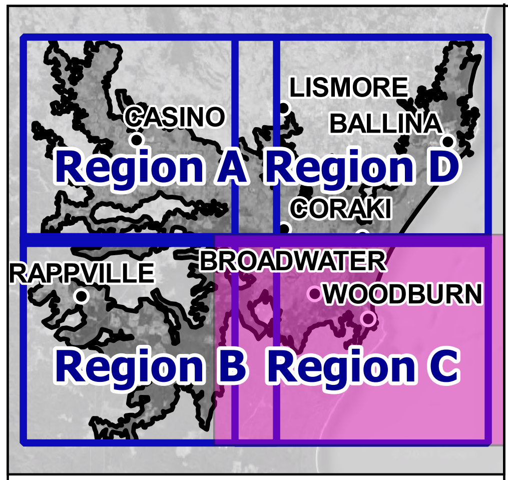

Peak flood level contours (mAHD)

January 2008 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4

Legend

- Towns
- Model Extent
- Watercourses

Title:

Region C - January 2008 Event Peak Flood Depth

Drawing: B-7

Rev: A

N 0 3,000 6,000 m

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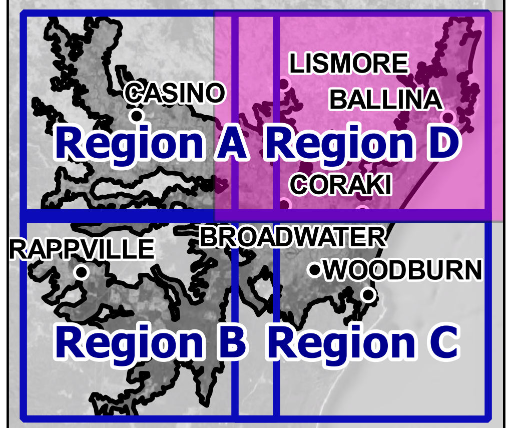
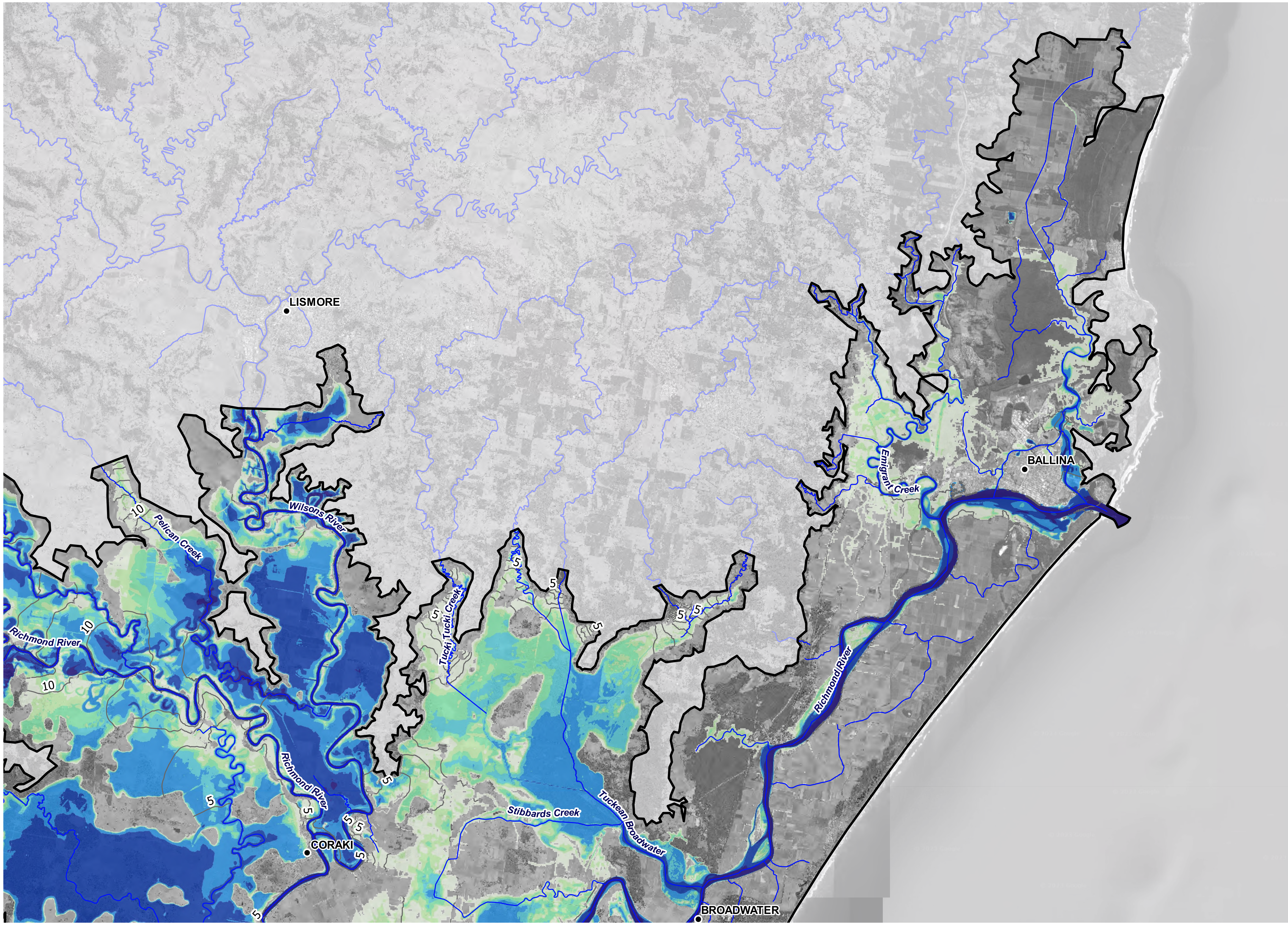


Peak flood level contours (mAHD)

January 2008 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- • Towns
- Model Extent
- Watercourses

Title:

Region D - January 2008 Event Peak Flood Depth

Drawing: B-8

Rev: A

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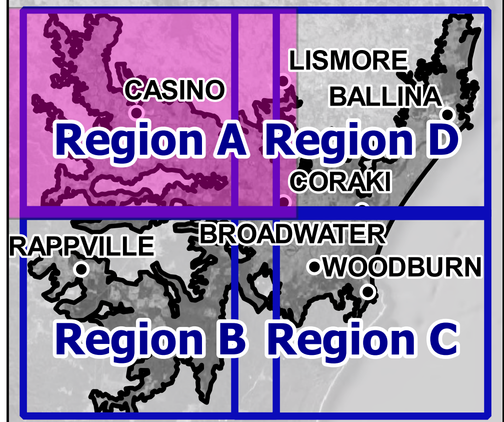
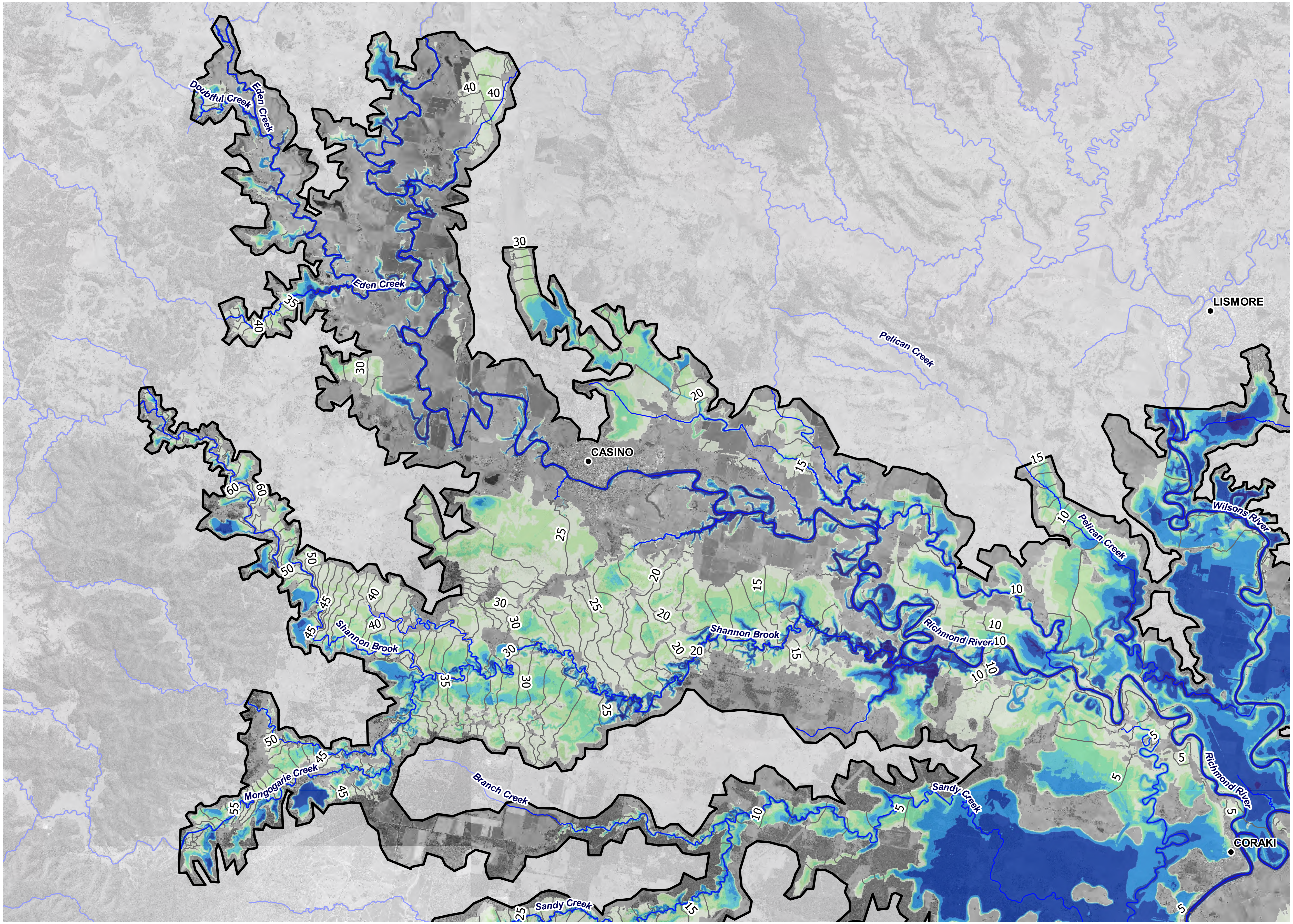


Peak flood level contours (mAHD)

May 2009 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region A - May 2009 Event Peak Flood Depth

Drawing: **B-9**

Rev: **A**

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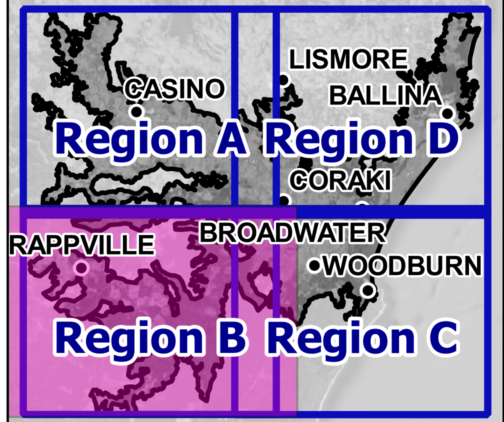
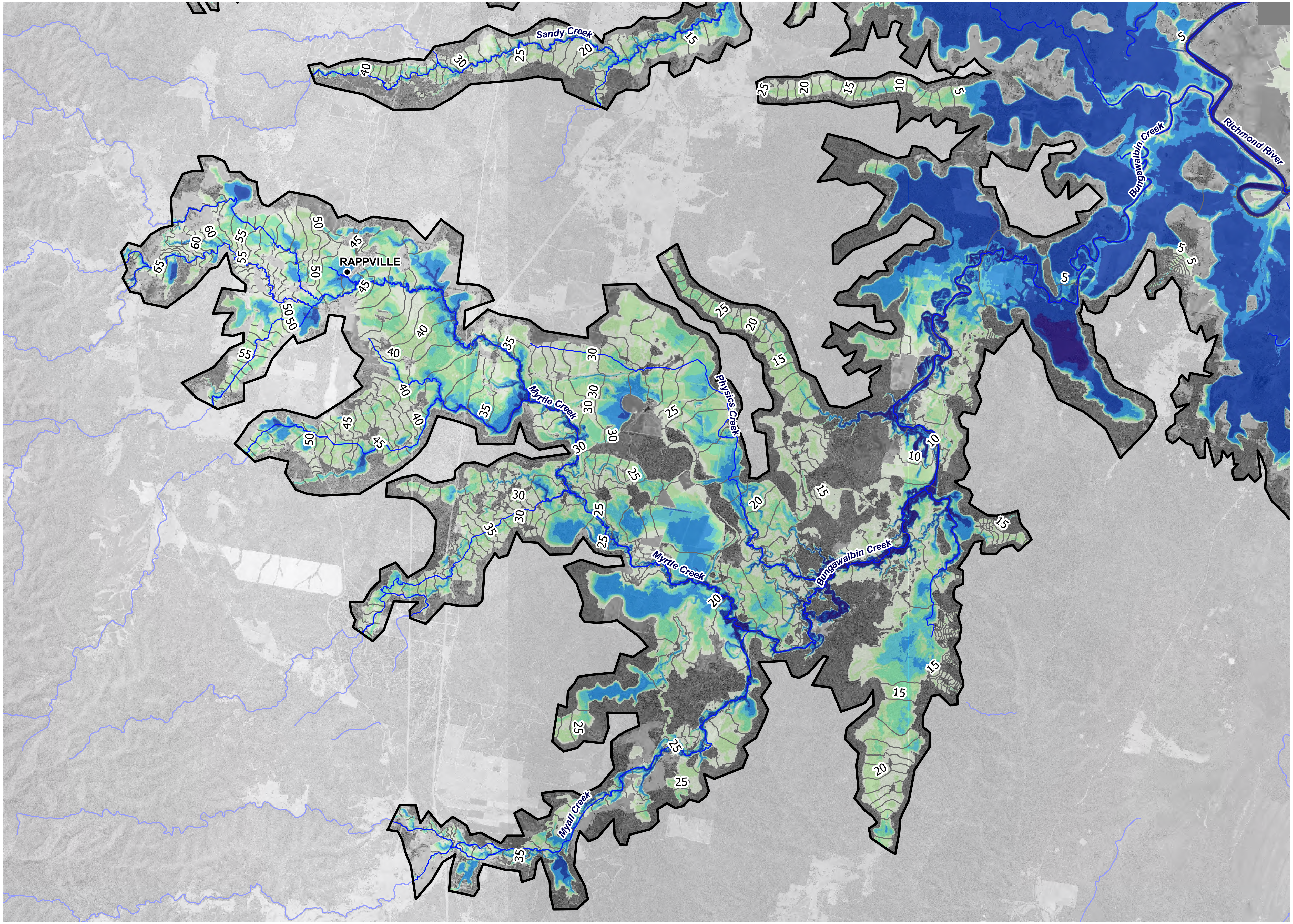


Peak flood level contours (mAHD)

May 2009 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- • Towns
- Model Extent
- Watercourses

Title:

Region B - May 2009 Event Peak Flood Depth

Drawing: B-10

Rev: A

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N

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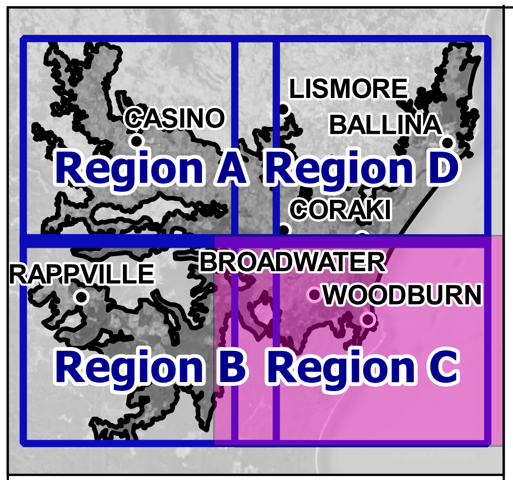
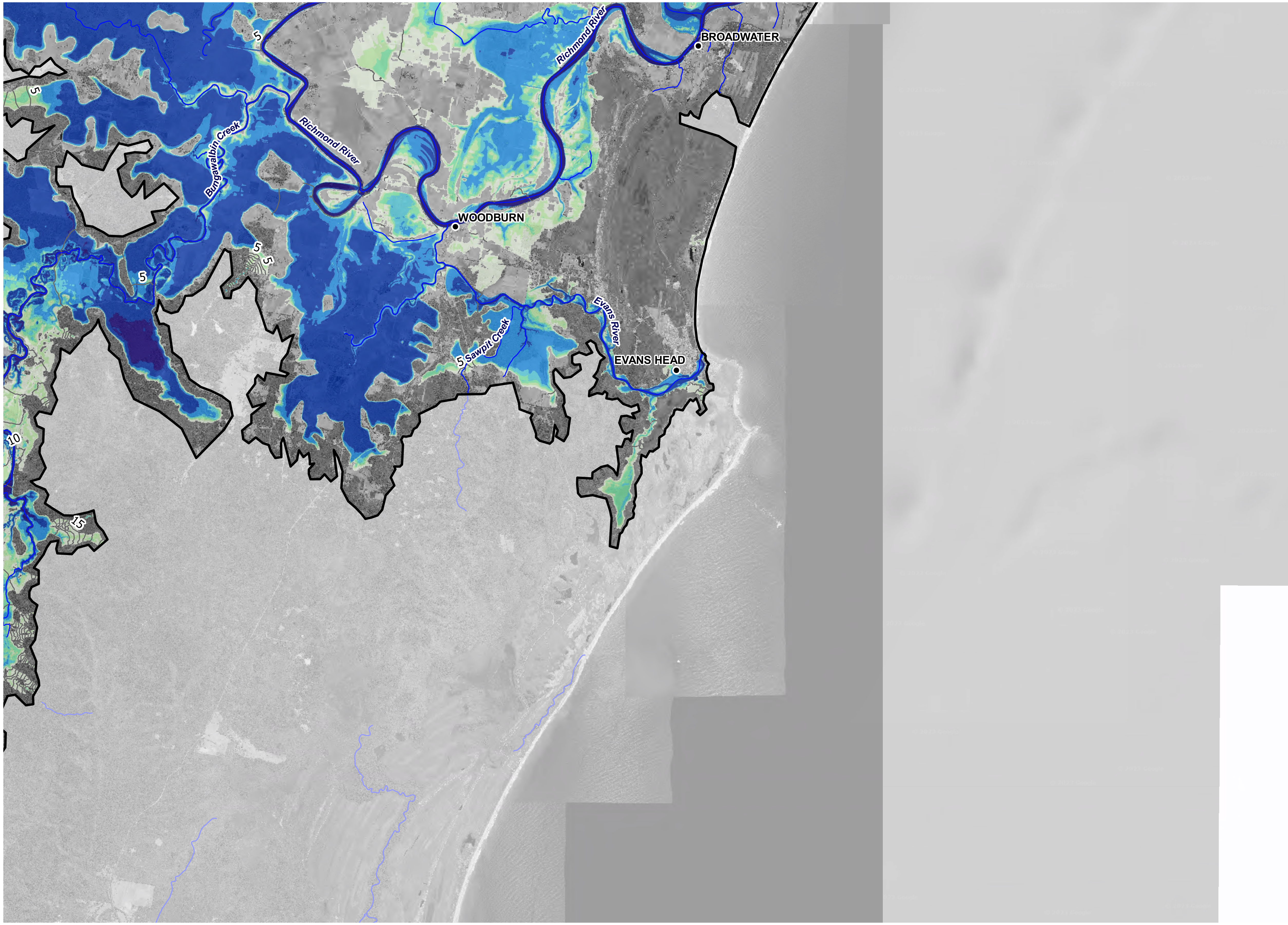


Peak flood level contours (mAHD)

May 2009 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region C - May 2009 Event Peak Flood Depth

Drawing: B-11

Rev: A

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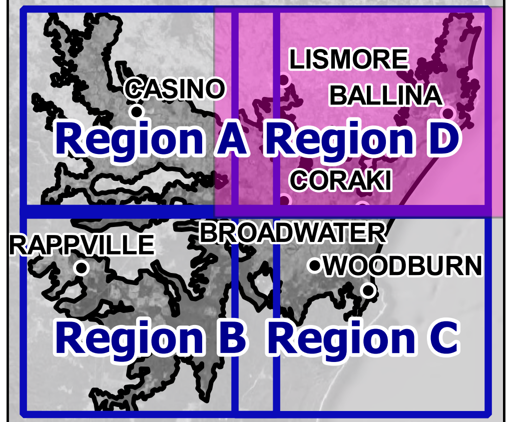
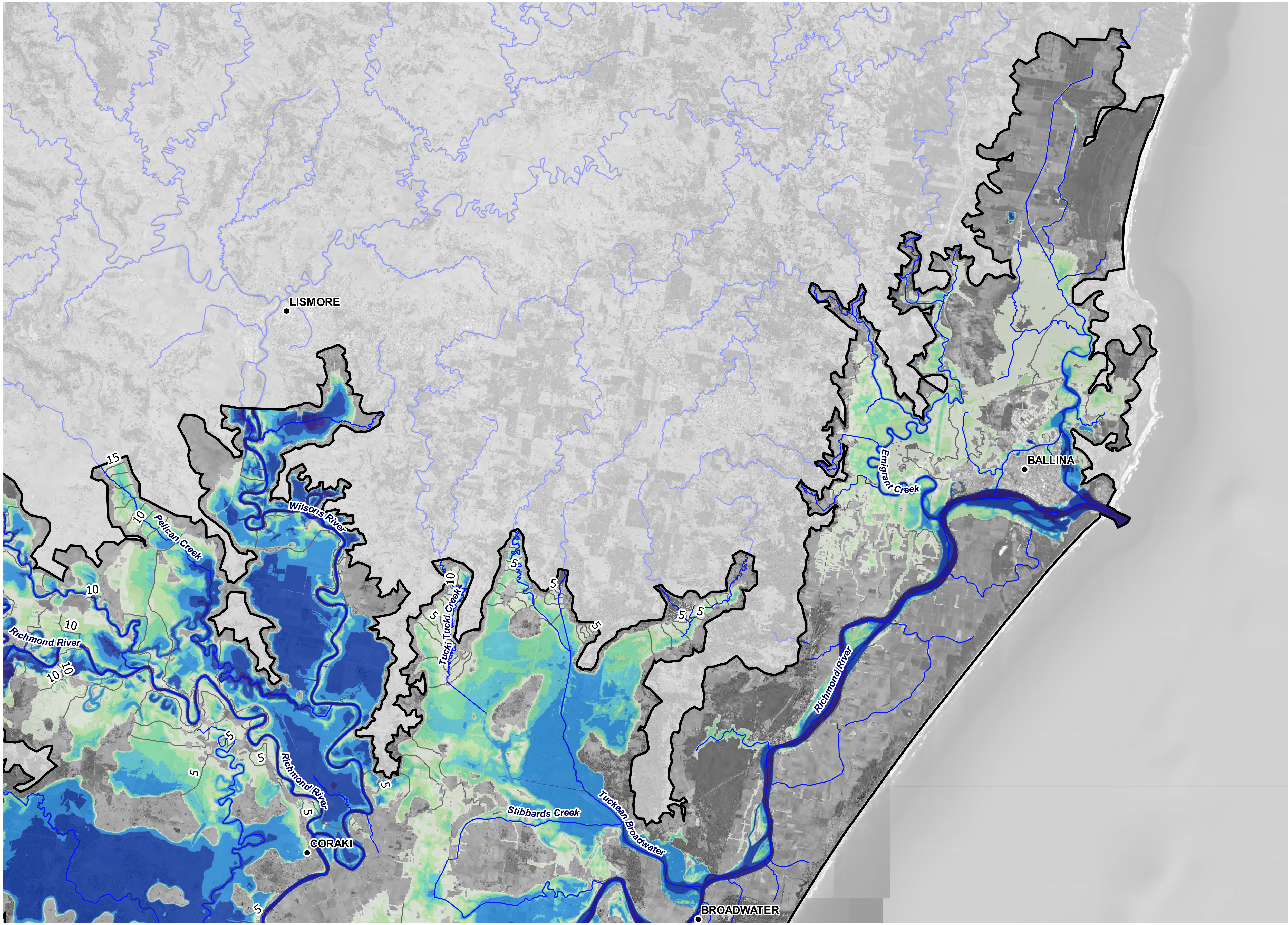


Peak flood level contours (mAHD)

May 2009 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- • Towns
- Model Extent
- Watercourses

Title:

Region D - May 2009 Event Peak Flood Depth

Drawing: B-12

Rev: A

N

0 3,000 6,000 m

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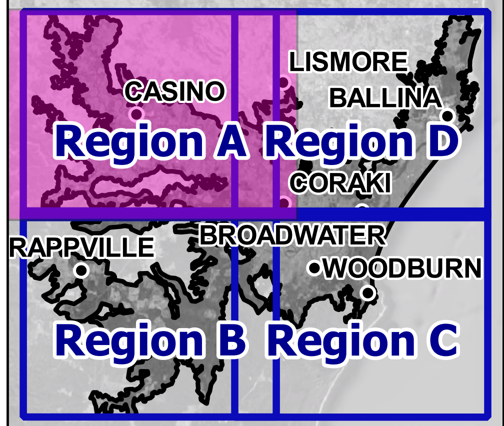
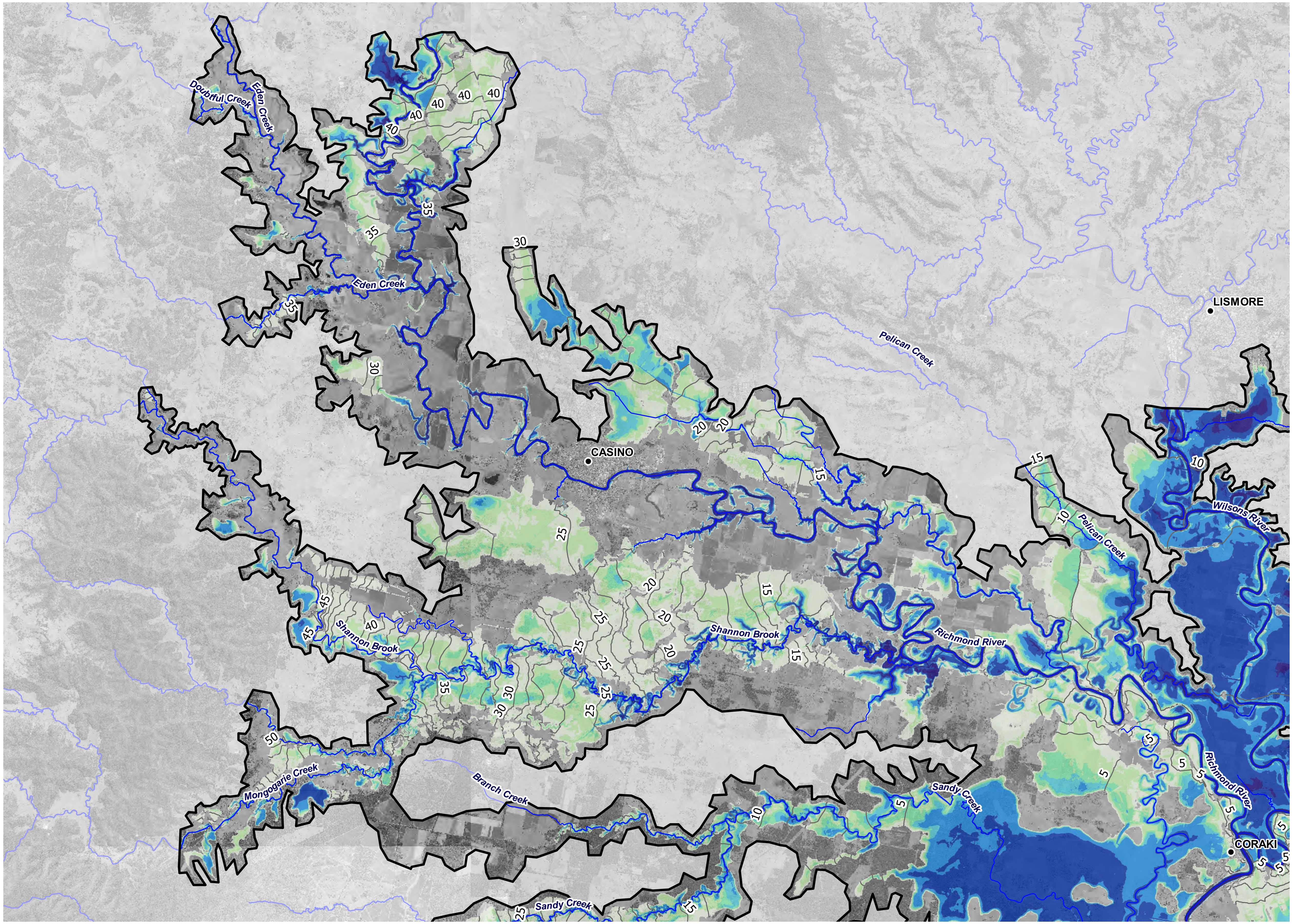


Peak flood level contours (mAHD)

March-April 2017 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- • Towns
- Model Extent
- Watercourses

Title:

Region A - March-April 2017 Event Peak Flood Depth

Drawing: B-13

Rev: A

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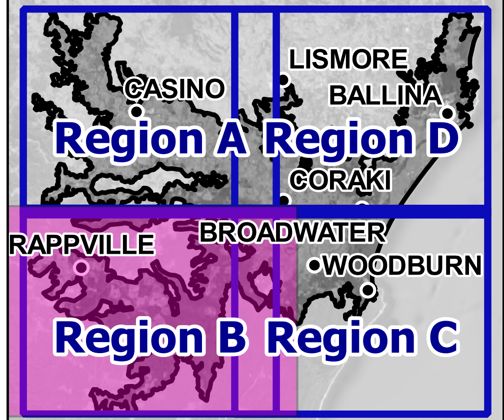
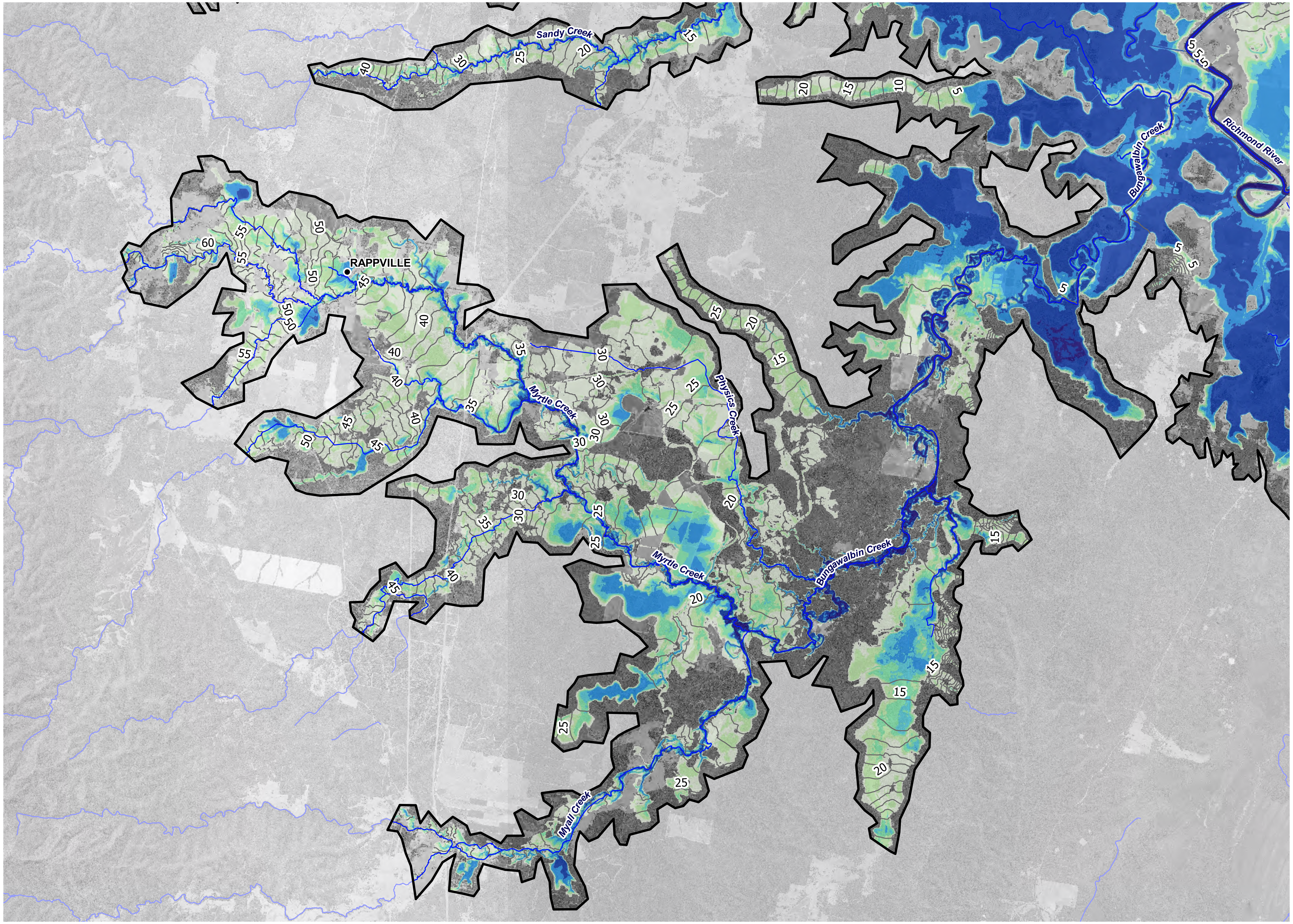


Peak flood level contours (mAHD)

March-April 2017 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region B - March-April 2017 Event Peak Flood Depth

Drawing: B-14

Rev: A

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N

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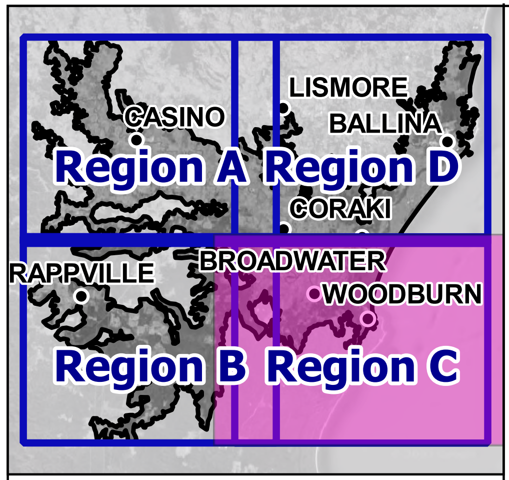
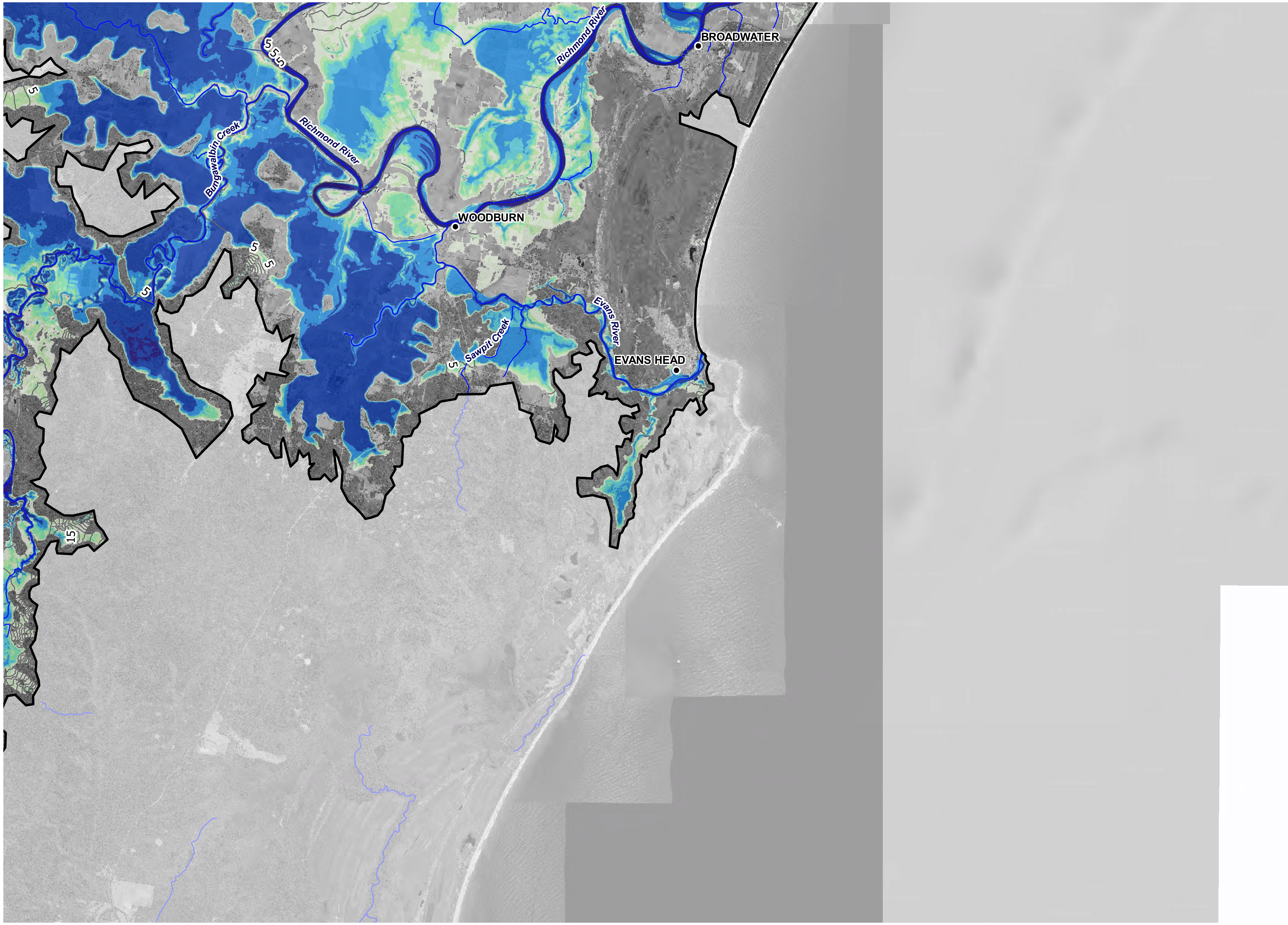


Peak flood level contours (mAHD)

March-April 2017 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region C - March-April 2017 Event Peak Flood Depth

Drawing: B-15

Rev: A

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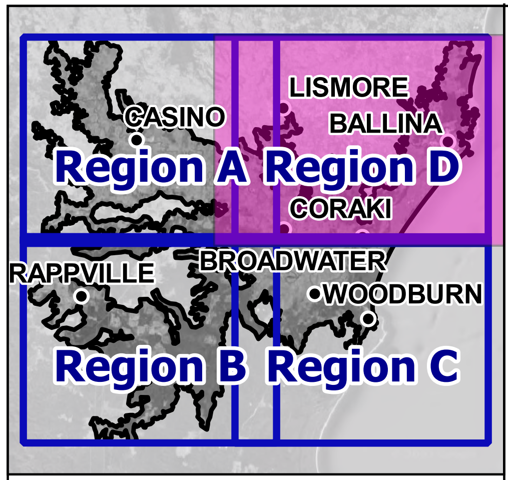
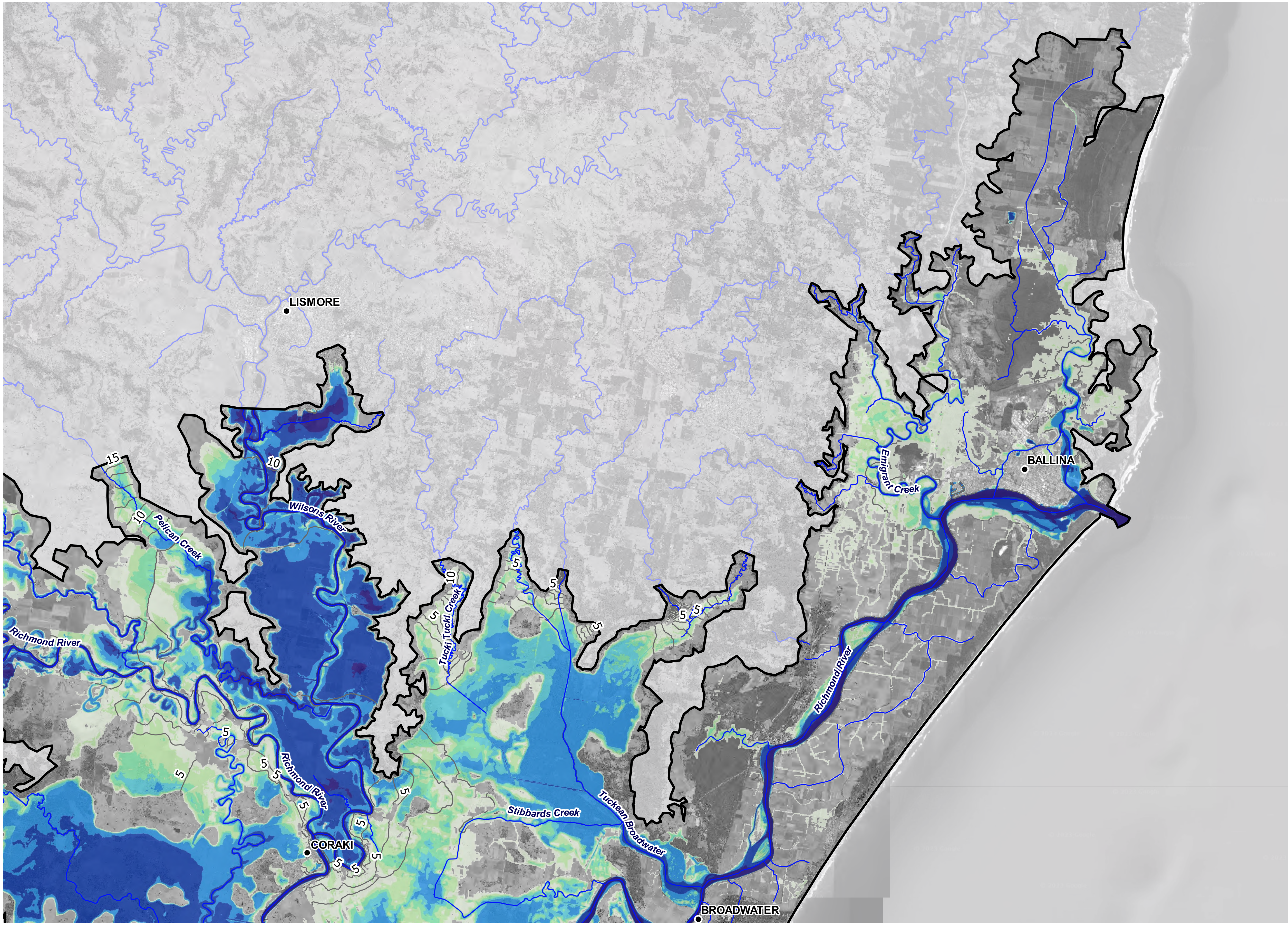


Peak flood level contours (mAHD)

March-April 2017 1m

Peak Flood Depth (m)

- <0.25
- 0.25 to 0.5
- 0.5 to 0.75
- 0.75 to 1.0
- 1.0 to 2.0
- 2.0 to 4.0
- >4



Legend

- Towns
- Model Extent
- Watercourses

Title:

Region D - March-April 2017 Event Peak Flood Depth

Drawing: B-16

Rev: A

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N 0 3,000 6,000 m

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Annex C Rainfall Stations Used in Assessment

C.1 February/March 2022 Event

Table C.1. February/March 2022 Event Pluviograph Stations

Station Number	Station Name	Event Depth (mm) 23 Feb 2022 to 1 Mar 2022
58005	Brays Creek (Misty Mountain)	740
40932	Darlington Alert	441
558037	Eden Ck at Doubtful	502
58113	Green Pigeon (Morning View)	637
203042	Ironpot Creek at Toonumbar	526
558002	Kyogle (Richmond River)	521
58141	Loadstone (High View)	386
40942	Palen Creek Alert	226
540579	Upper Running Ck	371
57020	Urbenville	191
540436	Ward Road Tm	257
558001	Wiangaree Bridge (Richmond)	440
58202	Bentley (Back Creek)	913
558024	Cawongla	750
58019	Doon Doon (Mccabes Road)	1512
558075	Goolmangar (Goolmangar Creek)	909
558086	Jiggi (Gwynne Rd)	856
58129	Kunghur (The Junction)	1168
58148	Lillian Rock (Williams Road)	886
58180	Nimbin (Goolmangar Creek)	1030
58199	Rock Valley (Leycester Creek)	868
58201	Tuncester (Leycester Creek)	863
58206	Corndale (Coopers Creek)	923
558031	Dunoon	1261

Station Number	Station Name	Event Depth (mm) 23 Feb 2022 to 1 Mar 2022
558033	Goonengerry	1279
558049	Huonbrook	1293
558087	Lismore (Dawson Street)	935 (to 08.45am 28 February)
58214	Lismore Airport Aws	624 (to 02:40am 28 February)
558036	Myocum	685
58162	Nashua (Wilsons River)	700
558000	Repentance (Coopers Creek)	1050
58208	Casino Airport Aws	503
58068	Lawrence Road	559
558015	Rappville Tm (Myrtle Creek)	618
558038	Shannon Brook At Yorklea	879
558076	Tuckurimba (Baxter Lane)	737
58099	Whiporie Post Office	637
558061	Yamba	571
558072	Alstonville Stp	846
58198	Ballina Airport Aws	419
558109	Coopers Shoot Repeater	375
558069	Houghlahan's Creek	631
558052	Lake Ainsworth	404
558071	Tuckombil	672

C.2 January 2008 Event

Table C.2. January 2008 Event Daily Rainfall Stations

Station Number	Station Name	Event Depth (mm) 30 Dec 2007 to 6 Jan 2008
58148	Lillian Rock (Williams Rd)	567
58070	Repentance Ck	440
558024	Cawongla (Alert)	429
58115	Grevillia (Lindesay View)	411

Station Number	Station Name	Event Depth (mm) 30 Dec 2007 to 6 Jan 2008
558031	Dunoon Alert Repeater	409
58165	Upper Coopers Ck	405
58180	Nimbin (Goolmangar Ck)	396
58206	Ewing Br Corndale (Coopers Ck)	369
58162	Nashua (Wilsons River)	360
58023	McLeans Ridges (Lascott Drive)	344
58078	Bentley	333
58004	Mummulgum (Bingeebeebra)	332
58202	Bentley (Back Ck)	330
558002	Kyogle (Richmond River)	319
58135	Meerschaum Vale (Barden)	318
58221	Lismore (Richmond Hill)	308
58171	Meerschaum Vale (Jenbetdaph)	302
58097	New Italy	297
58220	Woolners Arm	287
58198	Ballina Airport AWS	246
58201	Tuncester (Leycester Ck)	246
58001	Ballina (Crowley Village)	240
58192	Upper Mongogarie (Marangaroo)	233
58214	Lismore Airport AWS	223
58061	Woodburn Post Office	200
58194	Dairy Flat	182
204007	Clarence R @Lilydale (Newbold Crossing)	96

Table C.3. January 2008 Event Pluviograph Stations (8-day period from 30 December 2007)

Station Number	Station Name	Event Depth (mm) 30 Dec 2007 to 6 Jan 2008
58113	Green Pigeon (Morning View)	649
58044	Nimbin Post Office*	520
558001	Wiangaree Br (Richmond River)	343
58032	Kyogle Post Office	322
58154	Yorklea	306
58131	Alstonville Tropical Fruit Research Station	305

Station Number	Station Name	Event Depth (mm) 30 Dec 2007 to 6 Jan 2008
58016	Unumgar (Summerland Way)	292
58199	Rock Valley (Leycester Creek)	270
204002	Tabulam	257
204900	Baryulgil	200
203030	Rappville (Myrtle Ck)	191

***The rainfall depth but not the temporal pattern was used at Nimbin Post Office to allow Tuncester (Leycester Ck) gauge temporal pattern to capture more sub-catchments. This allowed for a better calibration**

Table C.4. May 2009 Event Daily Rainfall Stations

Station Number	Station Name	Event Depth (mm) 20 to 22 May 2009
58165	Upper Coopers Ck	442
58070	Repentance Ck	413
58002	Bangalow (Fowlers Lane)	402
58023	McLeans Ridges (Lascott Drive)	338
58208	Casino Airport AWS	332
58063	Casino Airport	328
58221	Lismore (Richmond Hill)	326
58207	Busbys Flat	313
58097	New Italy	311
58004	Mummulgum (Bingeebeebra)	298
58099	Whiporie Post Office	297
58146	Kyogle (Larkin Street)	285
58131	Alstonville Tropical Fruit Research Station	270
58220	Woolners Arm	253
58061	Woodburn Post Office	250
58195	Wiangaree Post Office	247
58135	Meerschaum Vale (Barden)	246
58001	Ballina (Crowley Village)	234
58171	Meerschaum Vale (Jenbetdaph)	231
58015	Coraki (Richmond Terrace)	224
58016	Unumgar (Summerland Way)	177

Station Number	Station Name	Event Depth (mm) 20 to 22 May 2009
58115	Grevillia (Lindesay View)	174

Table C.5. May 2009 Event Pluviograph Stations

Station Number	Station Name	Event Depth (mm) 20 to 22 May 2009
58019	Doon Doon (McCabes Rd)	495
558033	Goonengerry (Alert)	456
558000	Repentance (Coopers Ck)	392
58068	Lawrence Rd (Pringles Way)	375
558031	Dunoon Alert Repeater	374
558008	Mullumbimy Creek	362
58147	The Channon	357
58162	Nashua (Wilsons River)	354
58206	Ewing Br Corndale (Coopers Ck)	324
58202	Bentley (Back Ck)	324
58113	Green Pigeon (Morning View)	321
58180	Nimbin (Goolmangar Ck)	307
58148	Lillian Rock (Williams Rd)	305
58201	Tuncester (Leycester Ck)	300
58199	Rock Valley (Leycester Ck	299
57095	Tabulam (Muirne)	297
558066	Belongil Ck	297
558024	Cawongla (Alert)	276
558034	Mullumbimby (Upper Main)	246*
58141	Loadstone (High View)	204
58194	Dairy Flat	188
57020	Urbenville Post Office	170

*** The rainfall depth but not the temporal pattern was used at Mullumbimby Upper Main**

Table C.6. March/April 2017 Event Daily Stations

Station Number	Station Name	Event Depth (mm) 30 to 31 March 2017
558078	Terania Creek	627

Station Number	Station Name	Event Depth (mm) 30 to 31 March 2017
58019	Doon Doon (McCabes Road)	602
58147	The Channon	576
558034	Mullumbimby (Upper Main Arm)	516
558049	Huonbrook	509
58148	Lillian Rock (Williams Road)	507
558053	Main Arm	492
558031	Dunoon	489
58180	Nimbin (Goolmangar Creek)	464
558033	Goonengerry	451
58005	Brays Creek (Misty Mountain)	445
558075	Goolmangar (Goolmangar Creek)	408
58214	Lismore Airport Aws	399
558087	Lismore (Dawson Street)	386
58201	Tuncester (Leycester Creek)	358

Table C.7. March/April 2017 Event Pluviograph Stations

Station Number	Station Name	Event Depth (mm) 30 to 31 March 2017
558078	Terania Creek	627
58019	Doon Doon (McCabes Road)	602
58147	The Channon	576
558034	Mullumbimby (Upper Main Arm)	516
558049	Huonbrook	509
58148	Lillian Rock (Williams Road)	507
558053	Main Arm	492
558031	Dunoon	489
58180	Nimbin (Goolmangar Creek)	464
558033	Goonengerry	451
58005	Brays Creek (Misty Mountain)	445
558075	Goolmangar (Goolmangar Creek)	408
58214	Lismore Airport Aws	399

Station Number	Station Name	Event Depth (mm) 30 to 31 March 2017
558087	Lismore (Dawson Street)	386
58201	Tuncester (Leycester Creek)	358
58206	Corndale (Coopers Creek)	351
558000	Repentance (Coopers Creek	350
558086	Jiggi (Gwynne Rd)	348
58113	Green Pigeon (Morning View)	329
58212	Evans Head RAAF Bombing Range Aws	326
58199	Rock Valley (Leycester Creek)	326
558036	Myocum	323
558024	Cawongla	298
58202	Bentley (Back Creek)	265
58208	Casino Airport Aws	238
40942	Palen Creek Alert	235
58162	Nashua (Wilsons River)	234
558038	Shannon Brook At Yorklea	216
58194	Dairy Flat	213
58141	Loadstone (High View)	200
558076	Tuckurimba (Baxter Lane)	193
558069	Houghlahan's Creek	190
558002	Kyogle (Richmond River)	188
558001	Wiangaree Bridge (Richmond River)	181
558072	Alstonville Stp	177
58216	Cape Byron Aws	163
558015	Rappville Tm (Myrtle Creek)	161
558037	Eden Ck At Doubtful	160
58207	Busbys Flat	110
58028	Coaldale (Bellona)	96
57114	Baryulgil (Clarence River)	85

Annex D At Site Rainfall Frequency Assessments

D.1 Introduction

At-site rainfall frequency analyses (RFA) were performed at 15 rainfall gauges listed in Table D.1. and with locations shown in Figure D.1. Gauges were selected if they had a long record length (greater than 50 years). The analyses were performed on daily rainfall totals for a 24 hour and 48 hour duration.

The approach taken is similar to that documented in Book 2 of ARR2019 whereby a factor is applied to convert the daily rainfall to ‘unrestricted’ rainfall. The GEV probability distribution was applied using LH moments. A water year from 1 July to 30 June was assumed when compiling the annual maxima series.

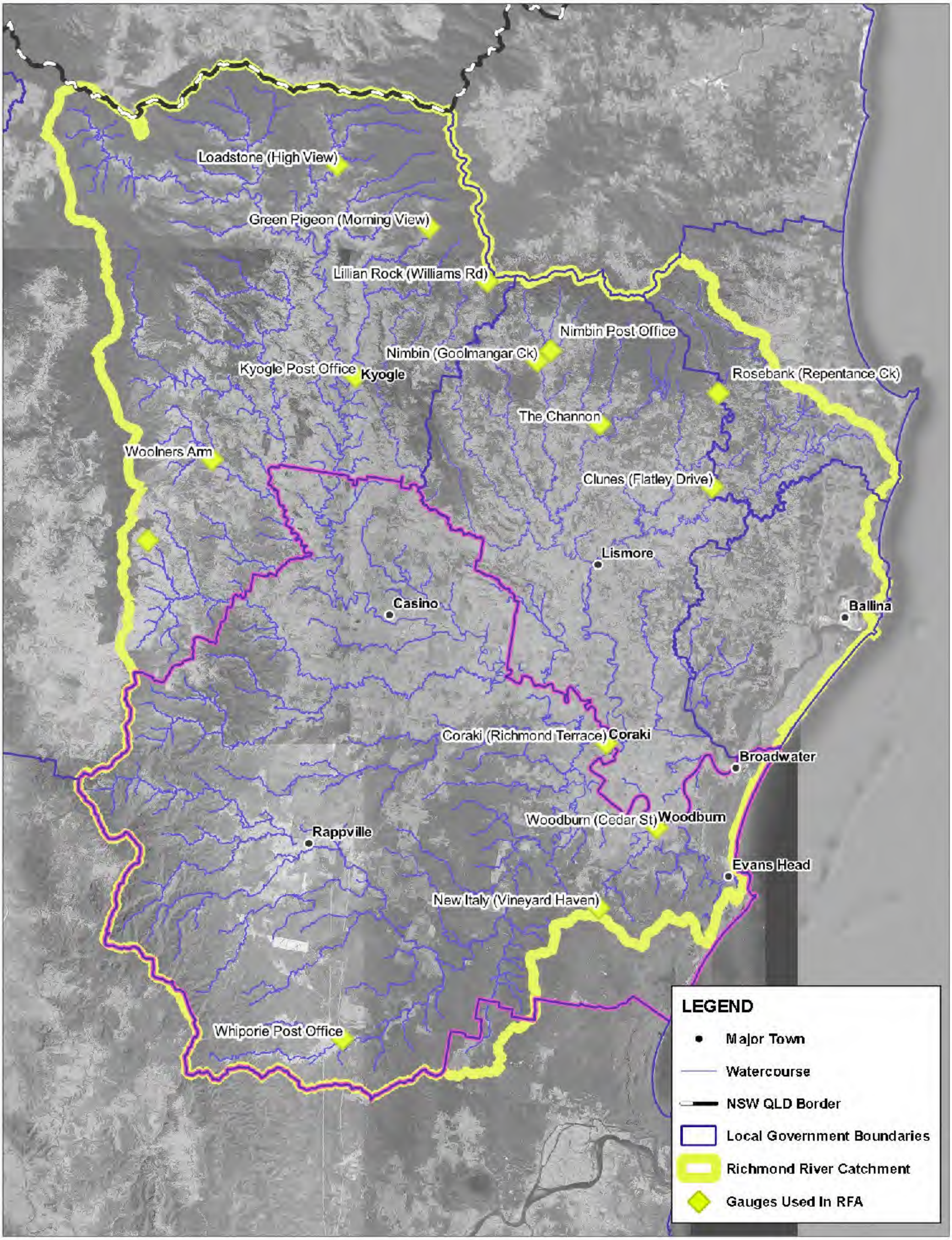
The assessment was undertaken prior to the February/March 2022 event which exceeded rainfall records at many stations. Inclusion of this significant event in the analysis may have resulted in some bias to the fit at the rare end of the rainfall frequency plot and so it was not undertaken.

Figure D.2 to Figure D.31 present the resulting rainfall frequency curves derived from the at-site analyses and plots these alongside the published IFDs. For the majority of gauges, the at-site assessments match reasonably closely with the published IFDs. There is a slight trend for the at-site IFDs to result in greater 1% AEP rainfall depths but not significantly so. There are also a number of sites where the IFDs result in higher 1% AEP rainfall depths. Overall it was concluded that there was no obvious and significant trend for the IFDs to overstate or understate rainfall based on the at-site assessments. As stated, this assessment excludes the February/March 2022 rainfall event which will likely increase the at-site estimates. Given that the design flood estimates from the models have been reconciled with the FFA peak flow estimates, then this does not have any notable bearing on the study outcomes.

Table D.1. At Site Rainfall Frequency Analyses: Gauges used

Gauge ID	Gauge Name	Record Length (years)
58061	WOODBURN (CEDAR ST)	134
58180	NIMBIN (GOOLMANGAR CREEK)	127
58015	CORAKI (RICHMOND TERRACE)	125
58044	NIMBIN POST OFFICE	117
58032	KYOGLE POST OFFICE	115
58097	NEW ITALY (VINEYARD HAVEN)	115
58147	THE CHANNON	94
58220	WOOLNERS ARM	93
58004	MUMMULGUM (BINGEEBEEBRA)	84
58070	ROSEBANK (REPENTANCE CREEK)	63
58127	CLUNES (FLATLEY DRIVE)	58
58148	LILLIAN ROCK (WILLIAMS ROAD)	57

Gauge ID	Gauge Name	Record Length (years)
58099	WHIPORIE POST OFFICE	56
58113	GREEN PIGEON (MORNING VIEW)	55
58141	LOADSTONE (HIGH VIEW)	51



Title:

Rain Gauges Used in RFA

Figure: D-1

Rev: A

BMT endeavours to ensure that the information provided in this map is correct at the time of publication. BMT does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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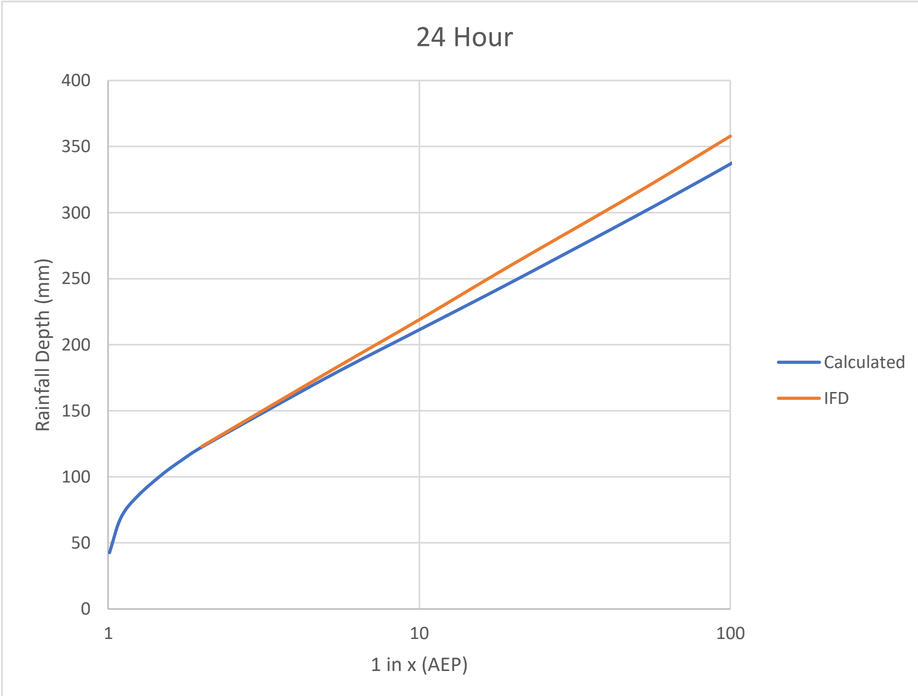

Figure D.2 Gauge 58061 (Woodburn, Cedar Street), 24 hour duration

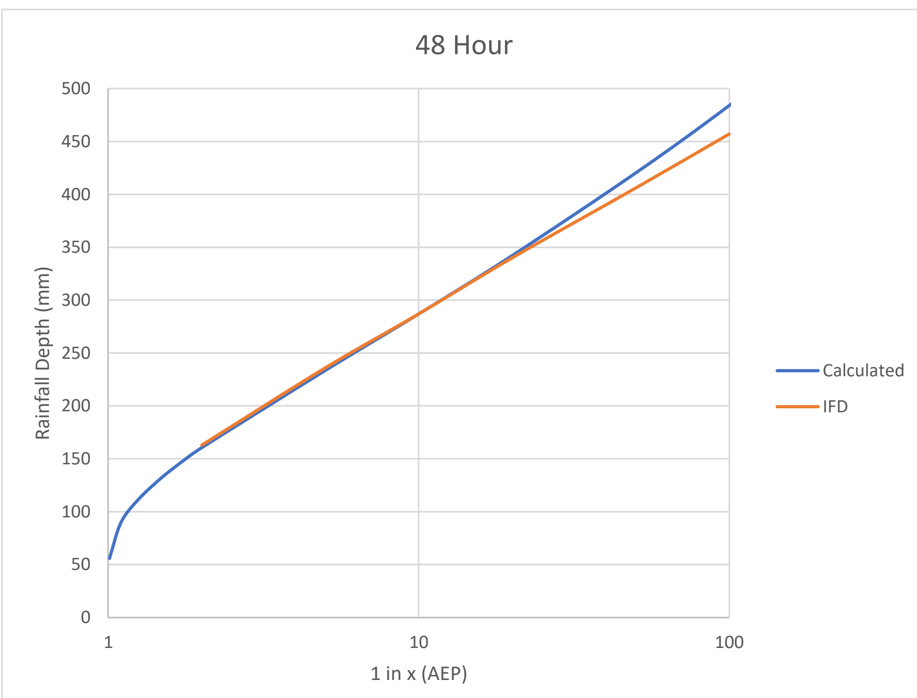

Figure D.3 Gauge 58061 (Woodburn, Cedar Street), 48 hour duration

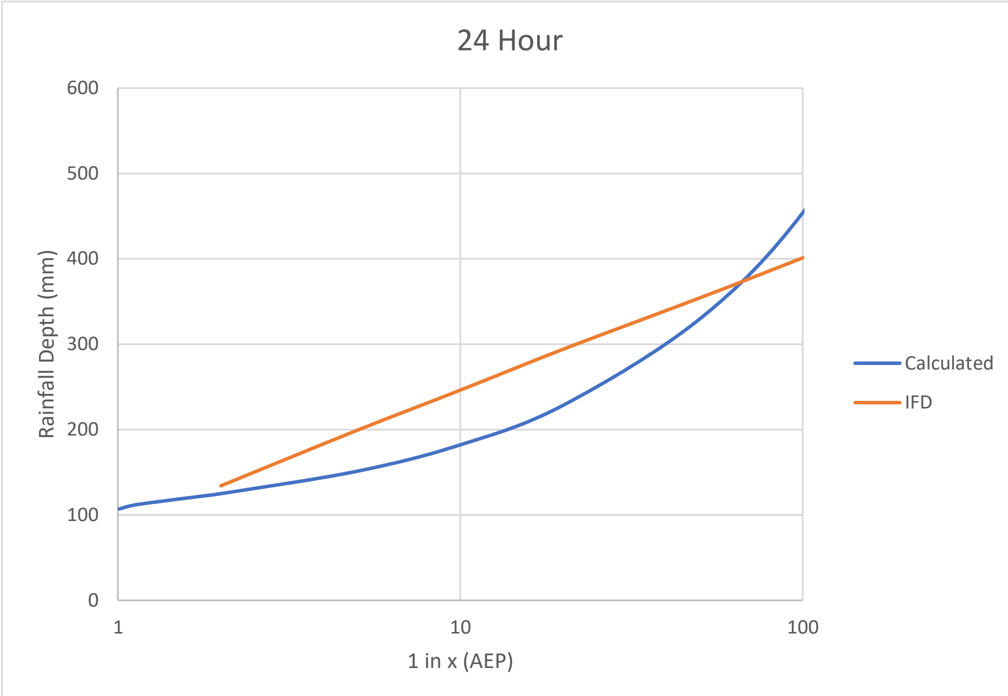

Figure D.4 Gauge 58180 (Nimbin, Goolmangar Creek), 24 hour duration

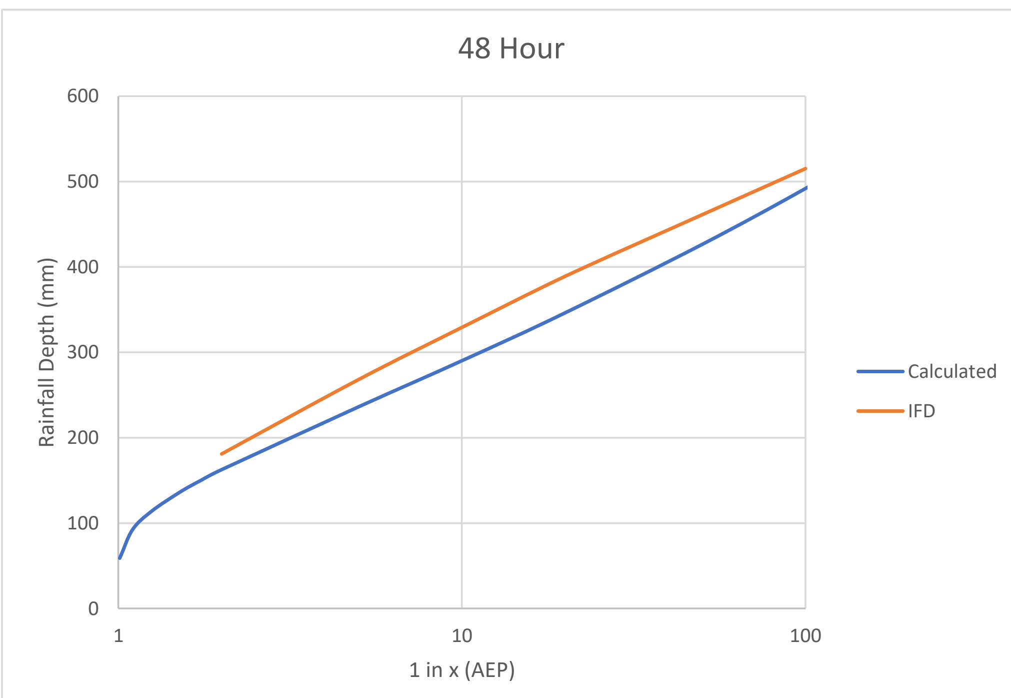

Figure D.5 Gauge 58180 (Nimbin, Goolmangar Creek), 48 hour duration

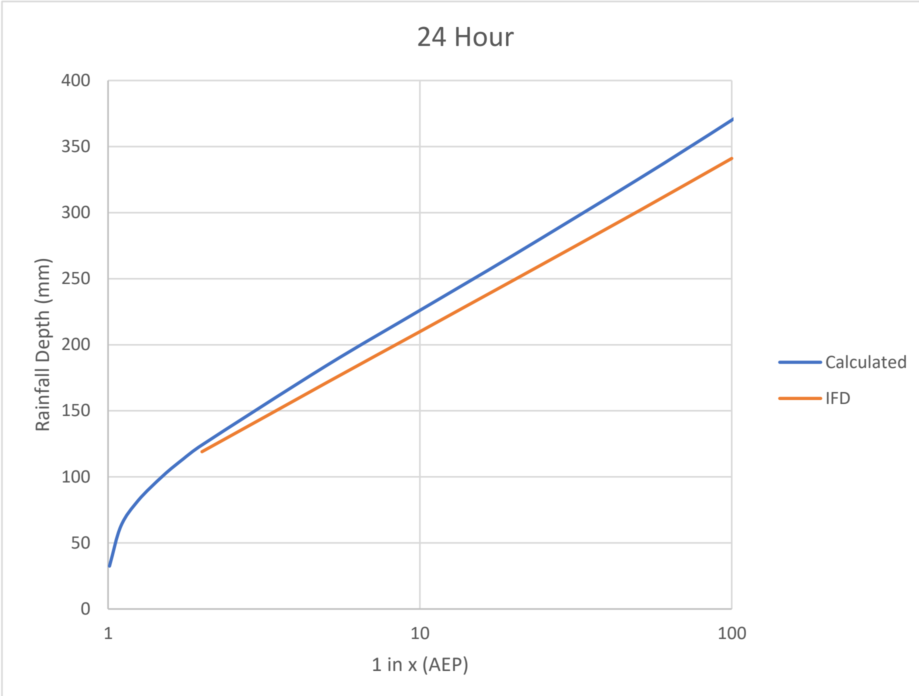

Figure D.6 Gauge 58015 (Coraki, Richmond Terrace), 24 hour duration

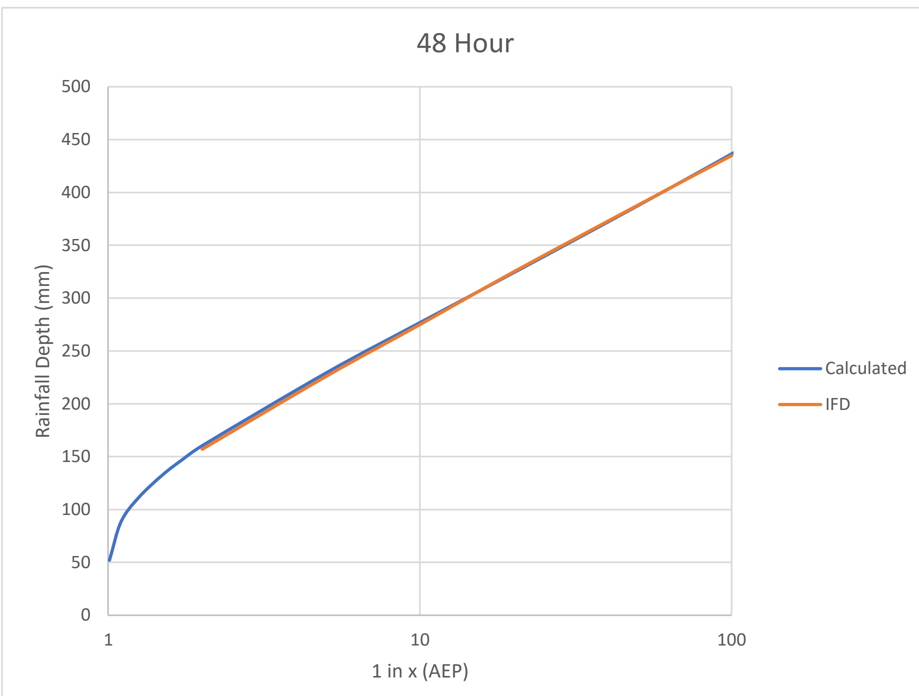

Figure D.7 Gauge 58015 (Coraki, Richmond Terrace), 48 hour duration

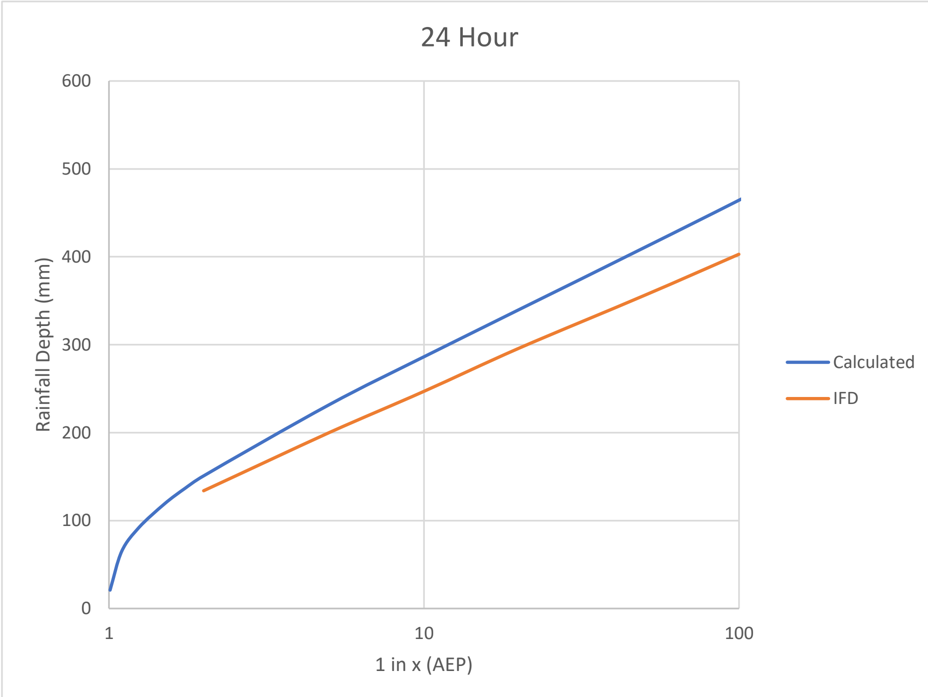

Figure D.8 Gauge 58044 (Nimbin Post Office), 24 hour duration

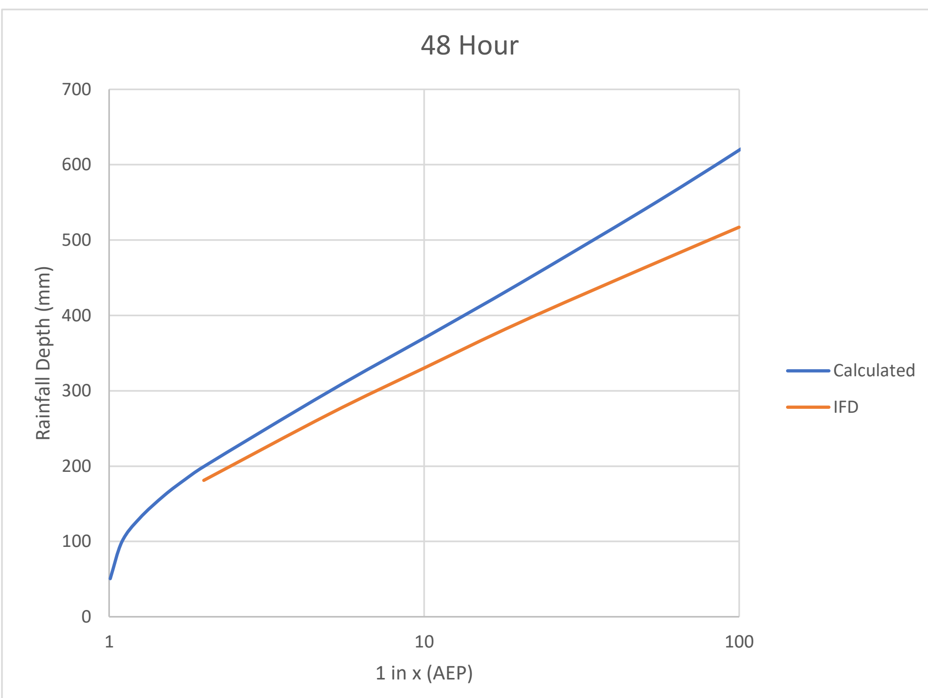

Figure D.9 Gauge 58044 (Nimbin Post Office), 48 hour duration

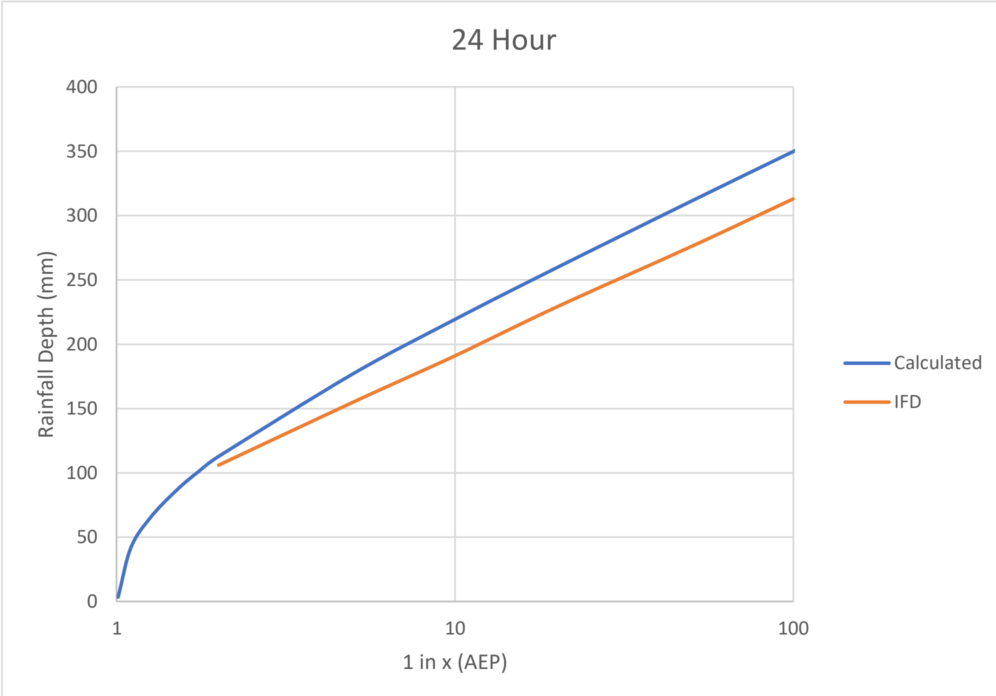

Figure D.10 Gauge 58032 (Kyogle Post Office), 24 hour duration

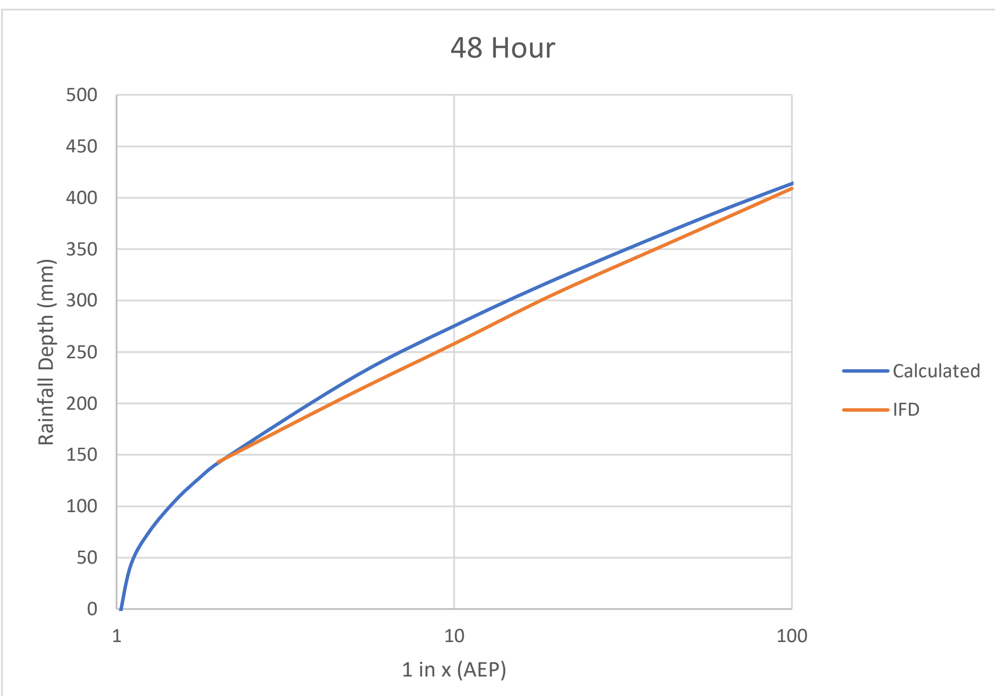

Figure D.11 Gauge 58032 (Kyogle Post Office), 48 hour duration

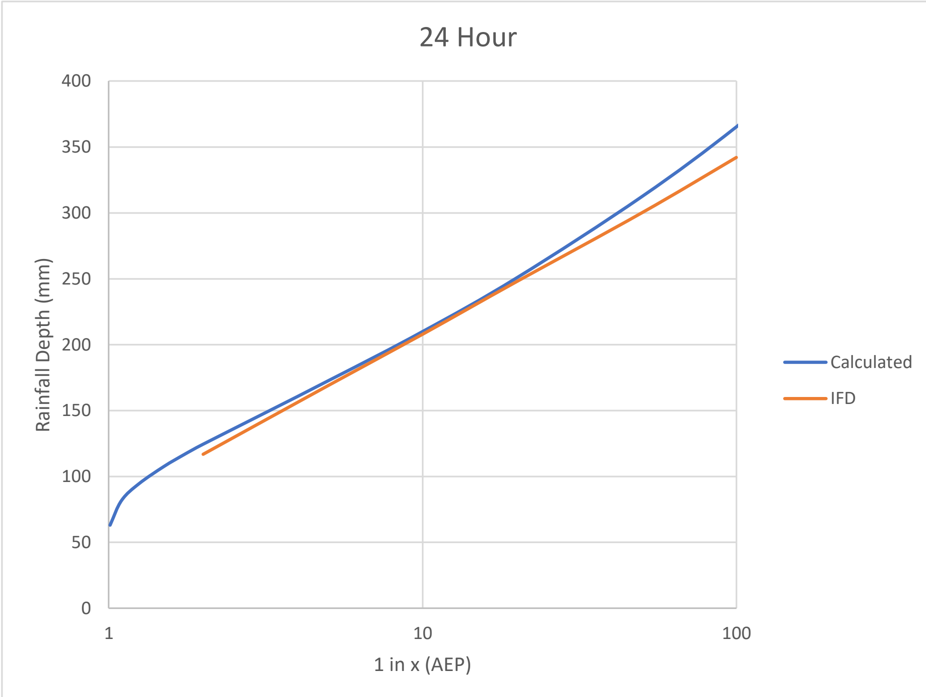

Figure D.12 Gauge 58097 (New Italy Vineyard Haven), 24 hour duration

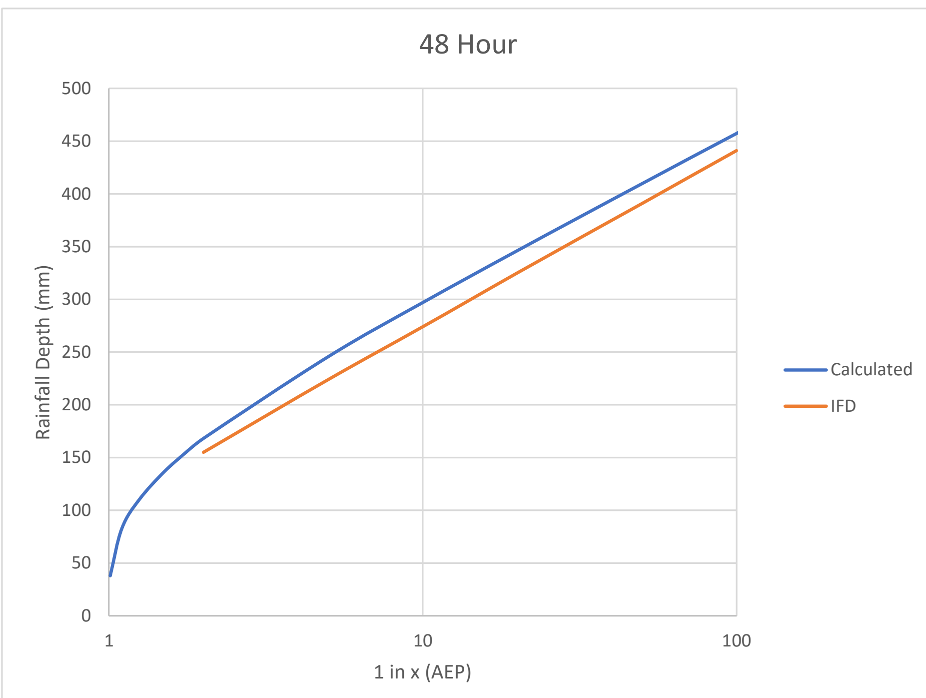

Figure D.13 Gauge 58097 (New Italy Vineyard Haven), 48 hour duration

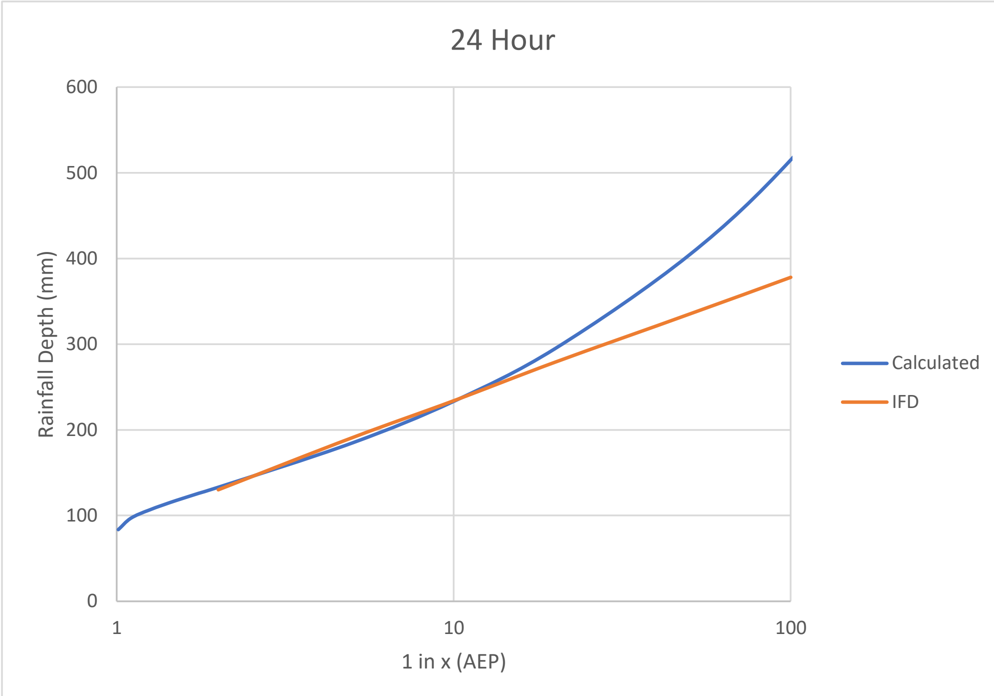

Figure D.14 Gauge 58147 (The Channon), 24 hour duration

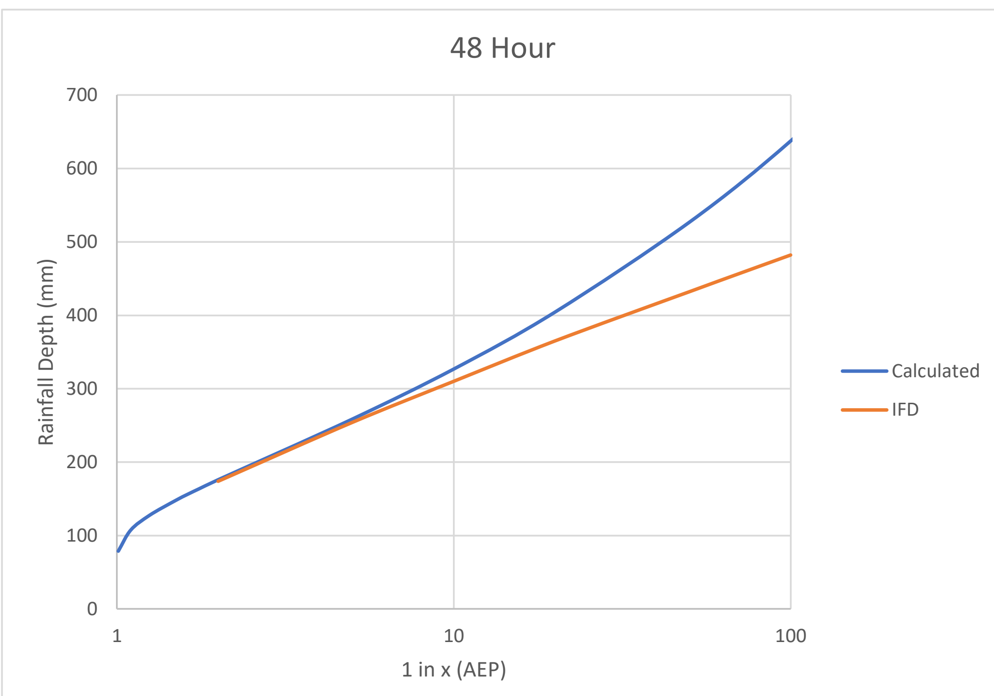

Figure D.15 Gauge 58147 (The Channon), 48 hour duration

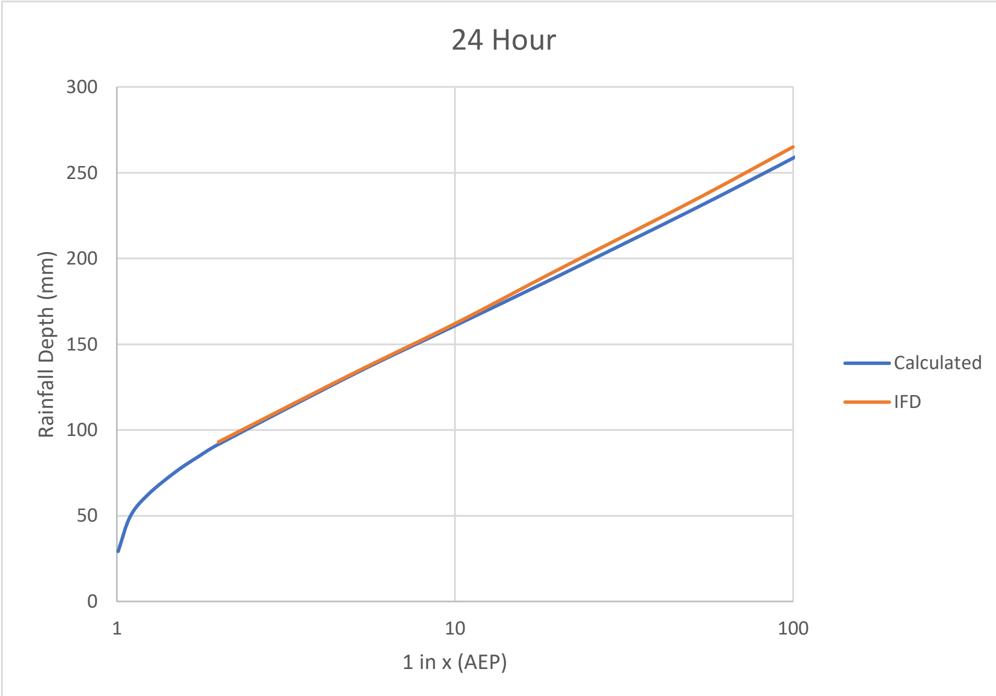

Figure D.16 Gauge 58220 (Woolners Arm), 24 hour duration

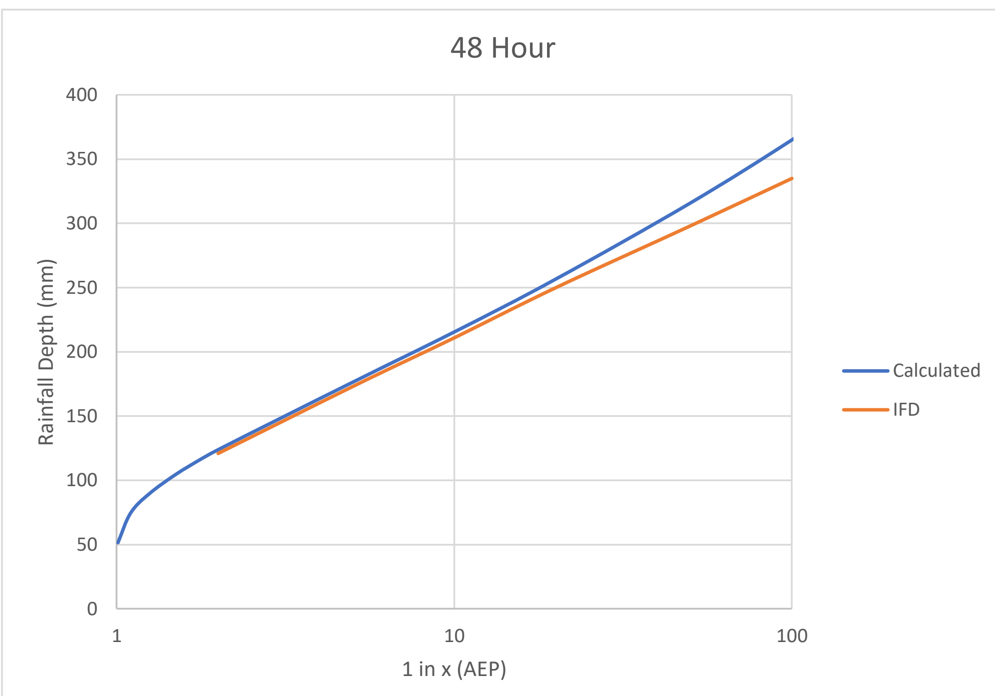

Figure D.17 Gauge 58220 (Woolners Arm), 48 hour duration

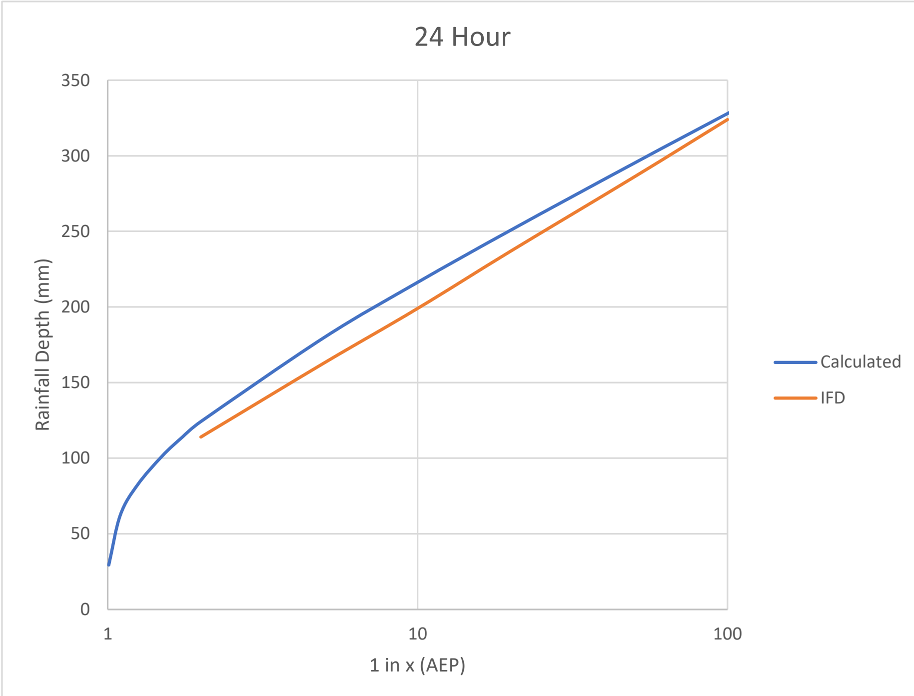

Figure D.18 Gauge 58004 (Mummulgum, Bingeebeebra), 24 hour duration

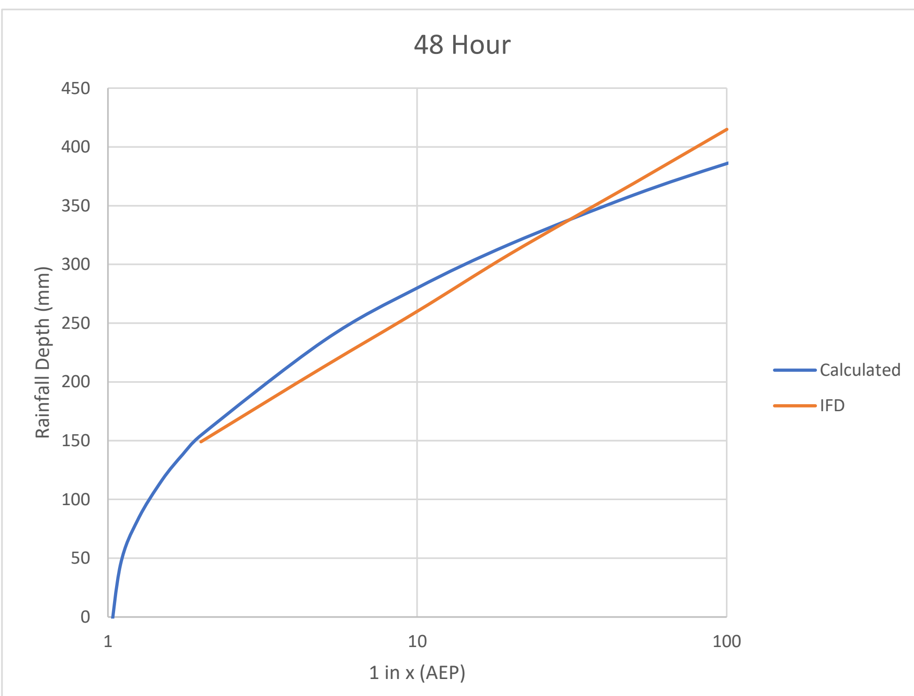

Figure D.19 Gauge 58004 (Mummulgum, Bingeebeebra), 48 hour duration

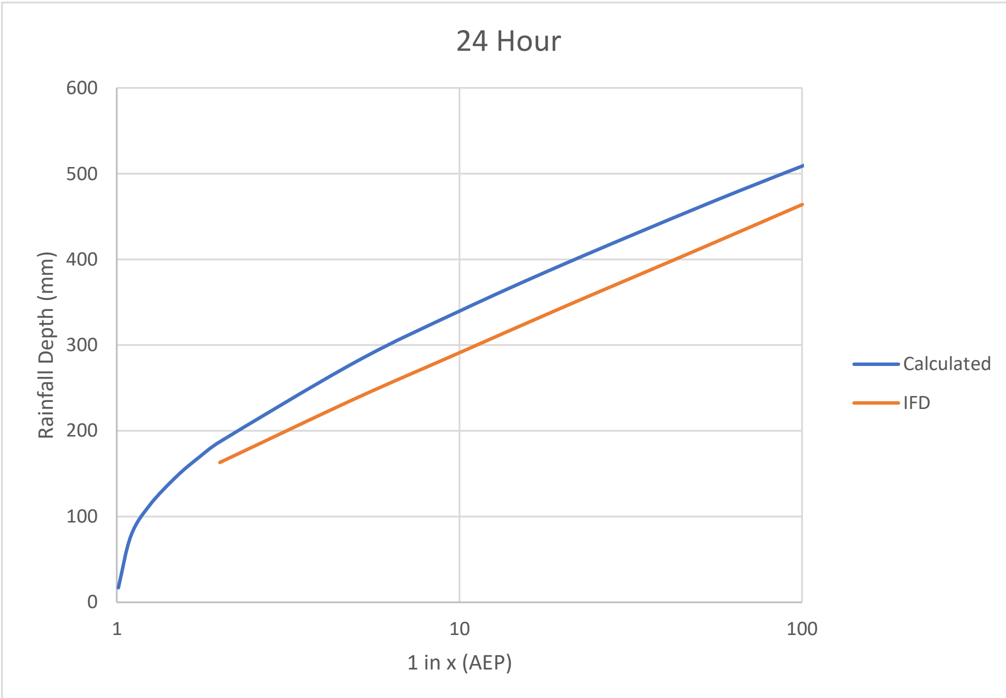

Figure D.20 Gauge 58070 (Rosebank, Repentance Creek), 24 hour duration

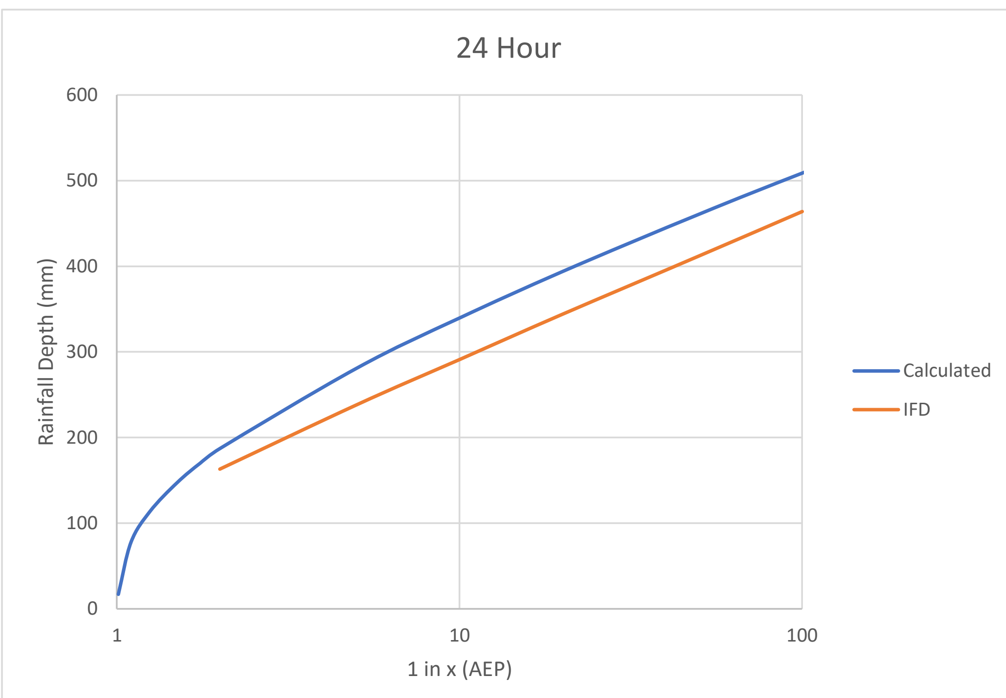

Figure D.21 Gauge 58070 (Rosebank, Repentance Creek), 48 hour duration

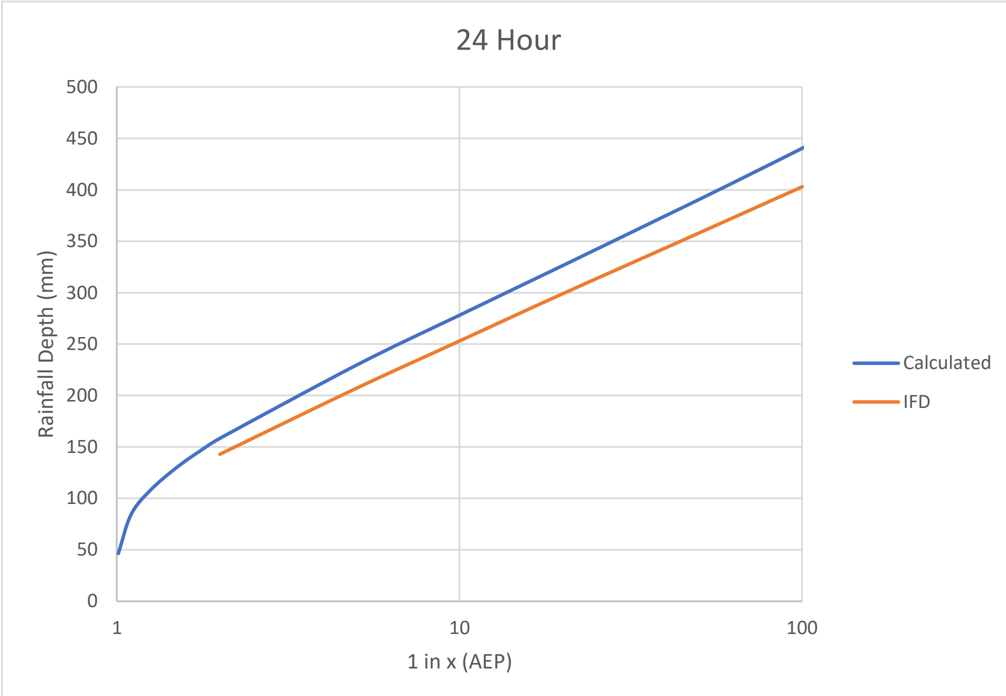

Figure D.22 Gauge 58127 (Clunes, Flatley Drive), 24 hour duration

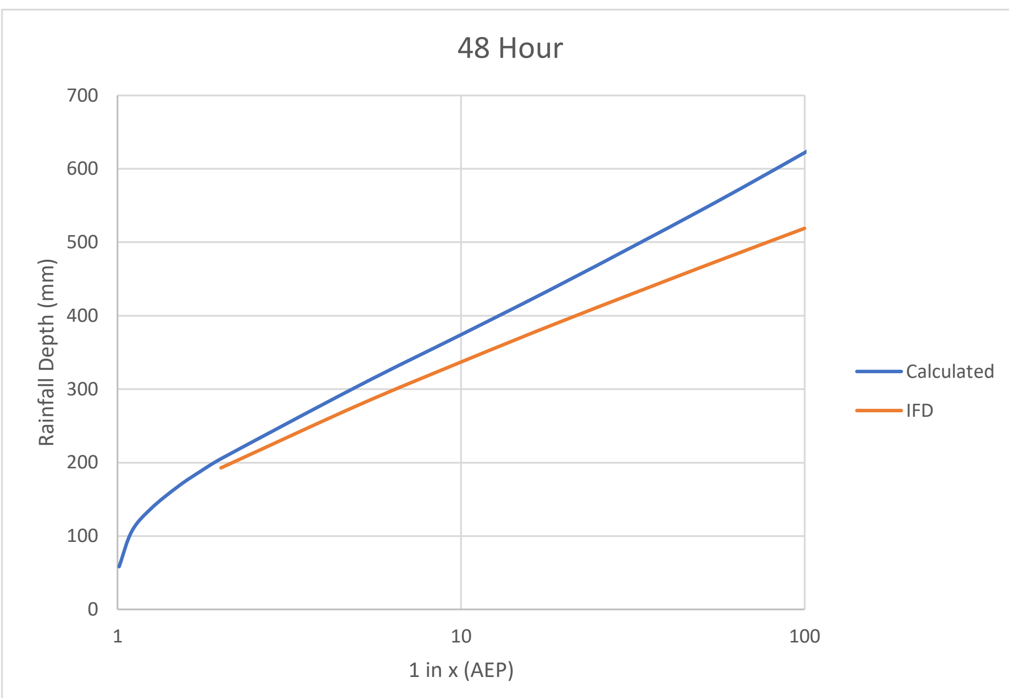

Figure D.23 Gauge 58127 (Clunes, Flatley Drive), 48 hour duration

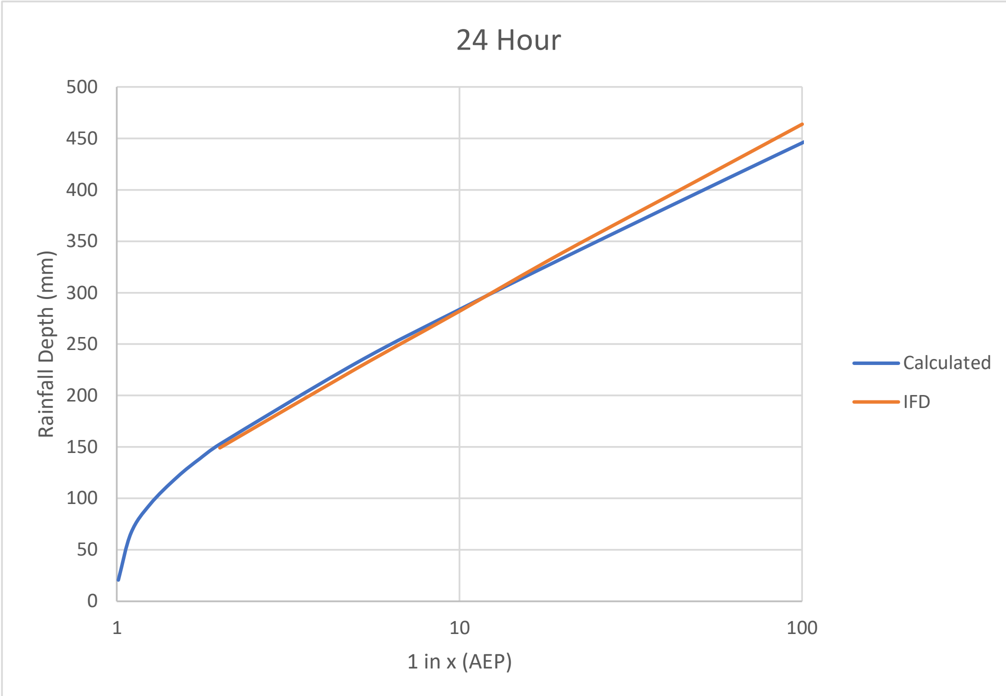

Figure D.24 Gauge 58148 (Lillian Rock, Williams Road), 24 hour duration

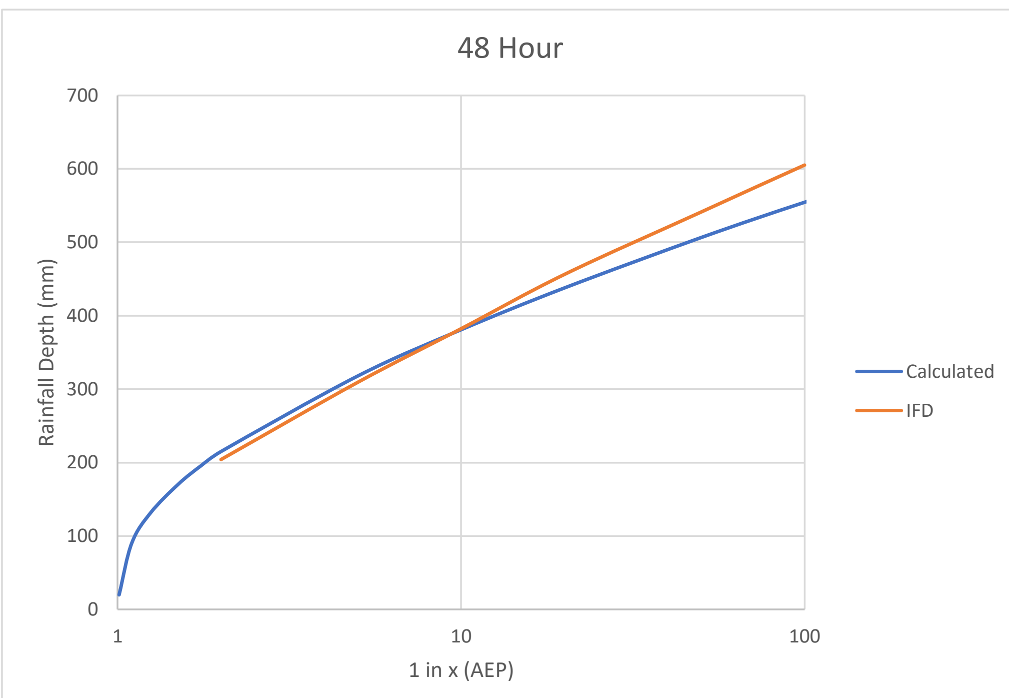

Figure D.25 Gauge 58148 (Lillian Rock, Williams Road), 48 hour duration

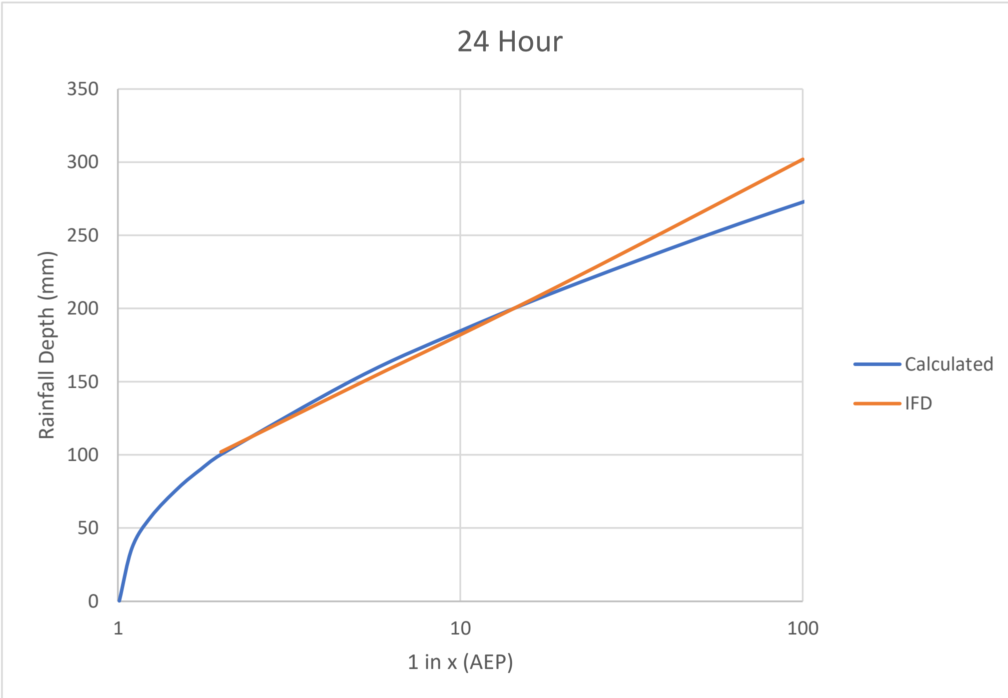

Figure D.26 Gauge 58099 (Whiporie Post Office), 24 hour duration

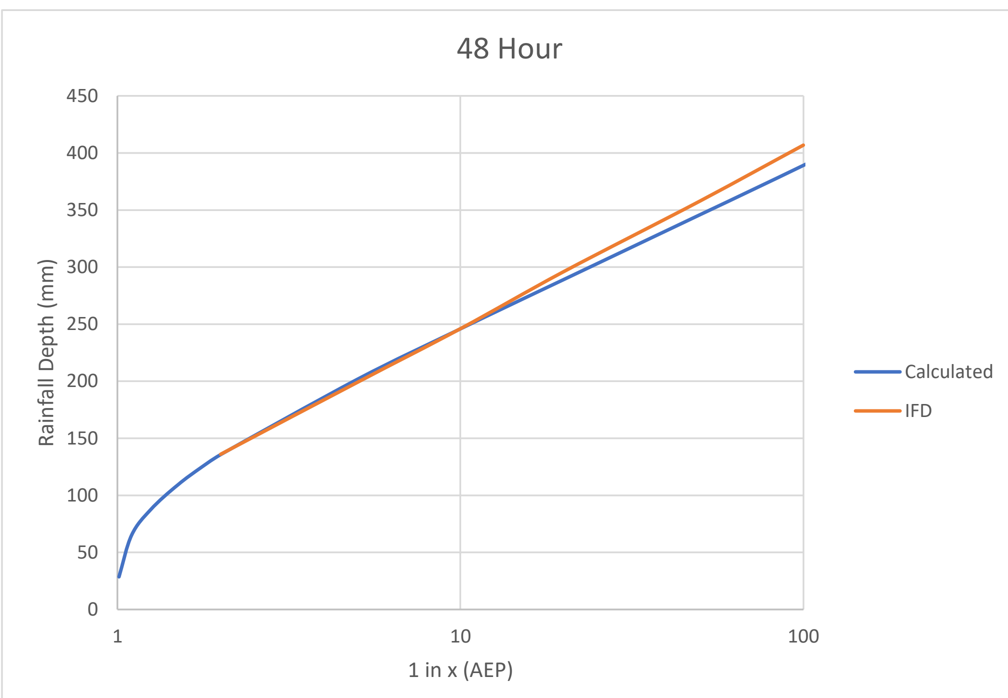

Figure D.27 Gauge 58099 (Whiporie Post Office), 48 hour duration

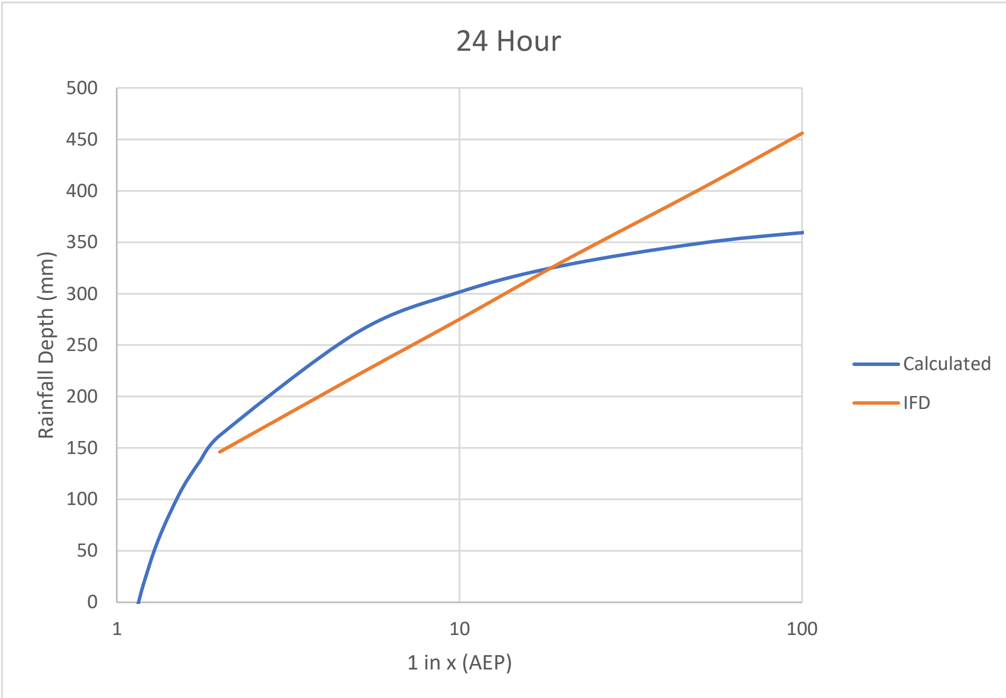

Figure D.28 Gauge 58113 (Green Pigeon, Morning View), 24 hour duration

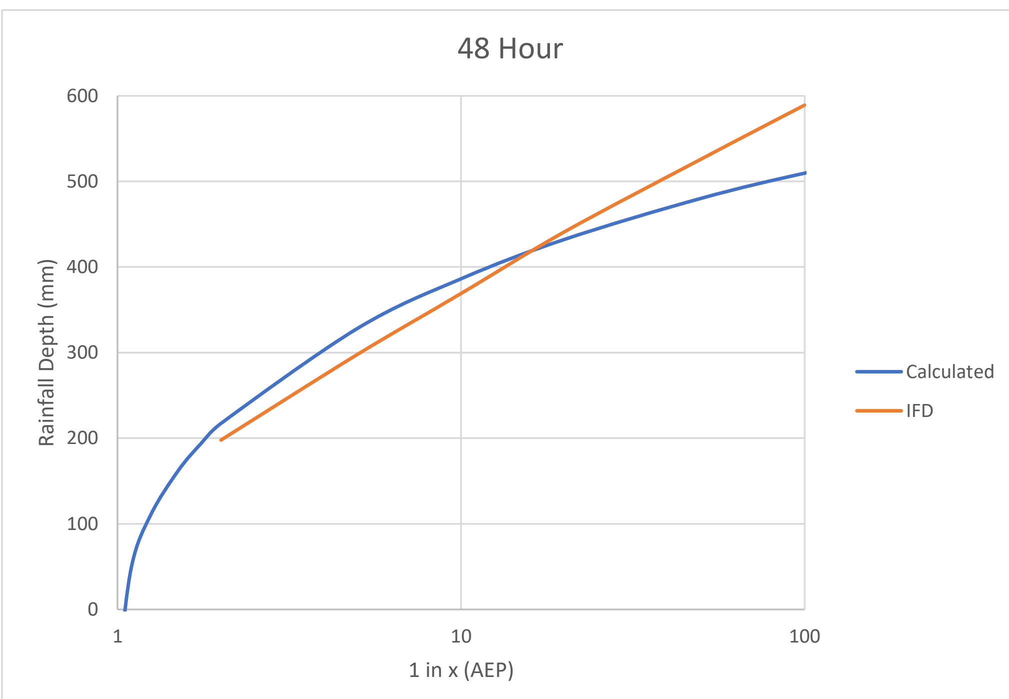

Figure D.29 Gauge 58113 (Green Pigeon, Morning View), 48 hour duration

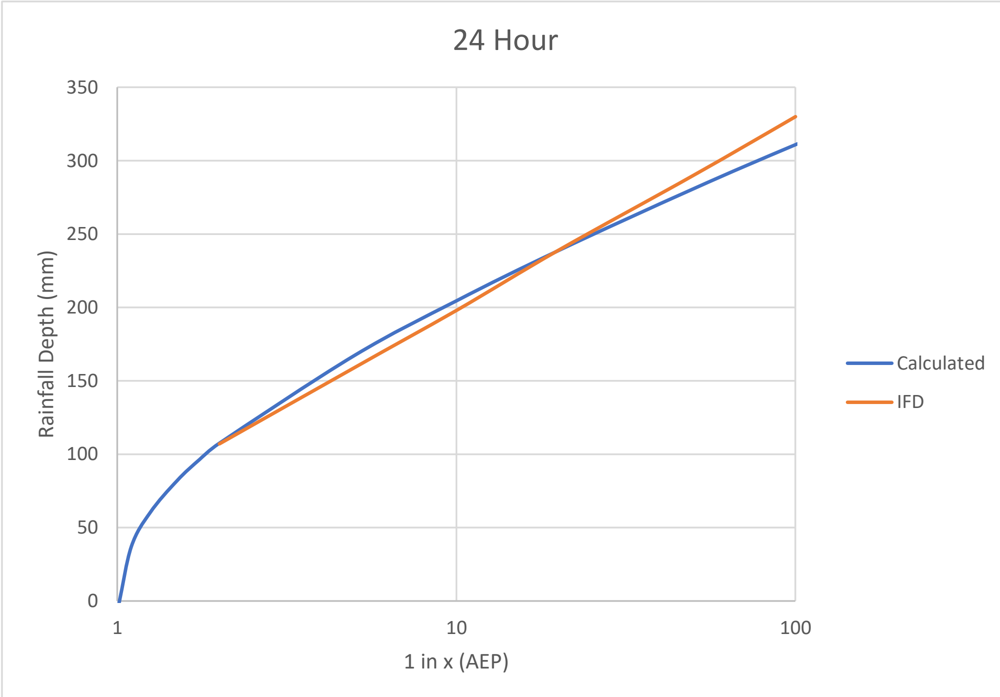

Figure D.30 Gauge 58141 (Loadstone, High View), 24 hour duration

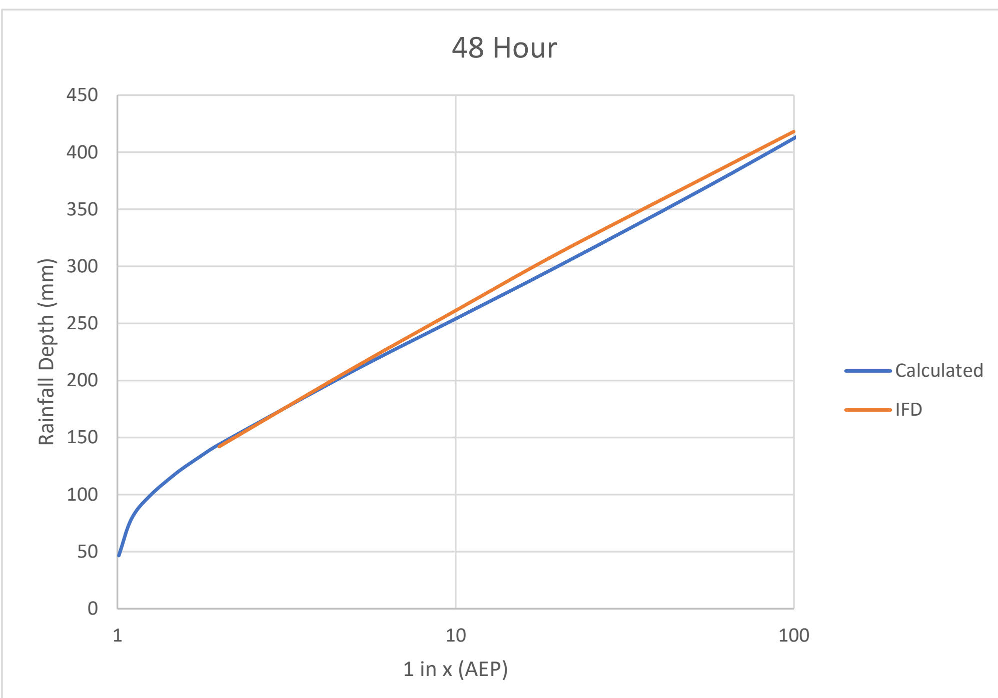

Figure D.31 Gauge 58141 (Loadstone, High View), 48 hour duration

Annex E Ocean Boundary

The NSW Floodplain Risk Management Guide *Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways* (OEH, 2015) provides advice on approaches that can be used to derive ocean boundary conditions and design flood levels for flood investigations in coastal waterways.

The guide first requires that the waterway entrance type is determined based on the five groups of estuary shown in Table E.1. . Under this classification both the Richmond River at Ballina and the Evans River at Evans Head are considered to be Group 3 estuaries.

Table E.1. Classification of Estuaries*

Group	Description
1 – Oceanic embayments	Marine waters with little influence of freshwater inflow eg Botany Bay, Jervis Bay
2 – Tide dominated estuaries	Large, deep entrances with tidal ranges similar to the open ocean, also known as drowned river valleys eg Port Stephens, the Hawkesbury River
3 – Wave dominated estuaries	Entrances that are constricted by wave deposited beach sand and flood-tidal deltas but are permanently open eg Tweed River, Lake Illawarra. Within this group there is a significant variation based upon whether the waterway discharges into a bay, port, or harbour, whether the entrance is trained (and the degree of training and stability), the relative size of the entrance and potential for the entrance to shoal.
4 – Intermittently closed estuaries	Also known as intermittently closed and open lakes and lagoons (ICOLLs). These are coastal water bodies that become isolated from the sea for extended periods eg Dee Why Lagoon, Lake Conjola
5 – Freshwater bodies	Coastal water bodies that rarely, if ever, are brackish but have occasional connection to the ocean eg Cudgen Lake, Myall Lakes

Based on the work of Roy et al (2001)

The OEH guide then simplifies the classification into three types as shown in Table E.2. Under this simplified classification, both the Richmond River and Evans River are considered to have Type B waterway entrance types. The general approach contained within the guide is then used for determining the magnitude and timing of the storm tide as described in Section 8.3 of this report.

Table E.2. Simplified Waterway Entrance Types (reproduced from OEH, 2015)

Type A	Type B	Type C
all *Group 1* open oceanic embayment all *Group 2* tide dominated estuaries *Group 3* estuaries: • draining directly to the ocean which have trained entrances and are maintained as navigable ports (e.g. Newcastle Harbour), excludes entrances maintained for small boat craft. • with trained entrances which drain to bays including the Brisbane Water, Tilligerry Creek and Cullendulla Creek. These entrance types result in little ocean tide attenuation and negligible wave set-up.	*Group 3* estuaries with fully (both sides of entrance) trained entrances which are not maintained as navigable ports. These entrances result in little ocean tide attenuation but have some potential for wave setup	*Group 4* Intermittently Closed Estuaries or ICOLLS *Group 3* estuaries with untrained or partially trained entrances which are likely to have very shallow flow depths across the entrance or may fully close from time to time. In these cases discharge to the ocean will be controlled by outlet berm characteristics (height, width and breadth). Design flood assessment for this classification needs to take into account the berm history and any entrance berm management strategy. The ocean boundary condition determined for the entrance type and approach (see Section 5) should be used as a downstream boundary for modelling, which should start at an appropriate location downstream of the controlling berm

*Richmond (at Ballina and Evens Head) is a Group 3 class estuary and is considered as Type B under the descriptions provided in this table.

Annex F Comparison of 80m Model and Adopted Model Results

A selection of hydrograph plots at key locations are provided below where flows from the hydraulic model are compared for the adopted model versus the 80m model used for event selection purposes. The results are shown prior to any scaling used to reconcile flows to the FFA. The plot locations were selected for capturing the full floodplain flow along the three major tributary systems of the Richmond River catchment.

It is not expected that the results should be identical given the significant change in model resolution. The aim is to show that the results are broadly comparable and therefore the 80m model is fit for the purpose of the relative assessment of ranking events by magnitude as part of the event selection process.

It can be seen from the plots that the overall shape and timing of the floods are similar in both models. The differences shown are not significant in the context of using the 80m model for event selection purposes. Section 8.4 provides an additional comparison on event rankings using both the adopted model and the 80m model which further validates the use of the 80m model for these purposes.

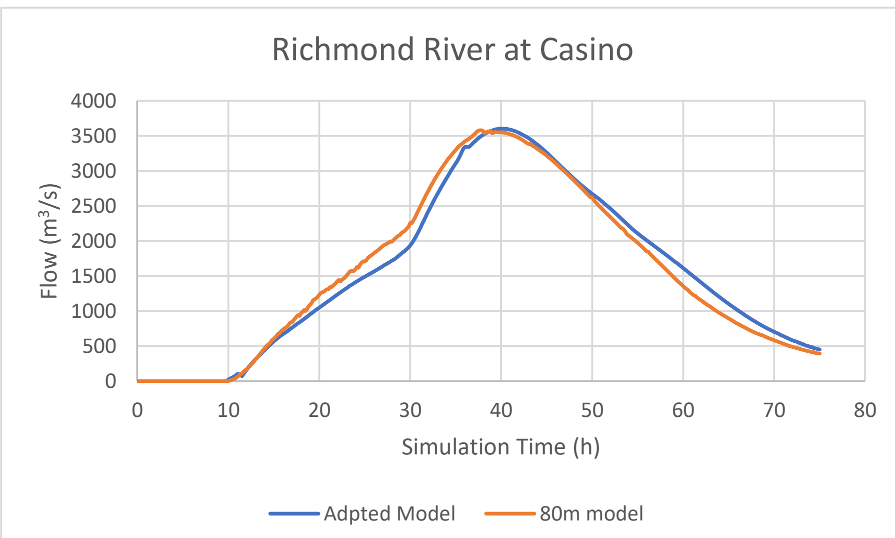

Figure F.1 Comparison at Casino (24h storm duration)

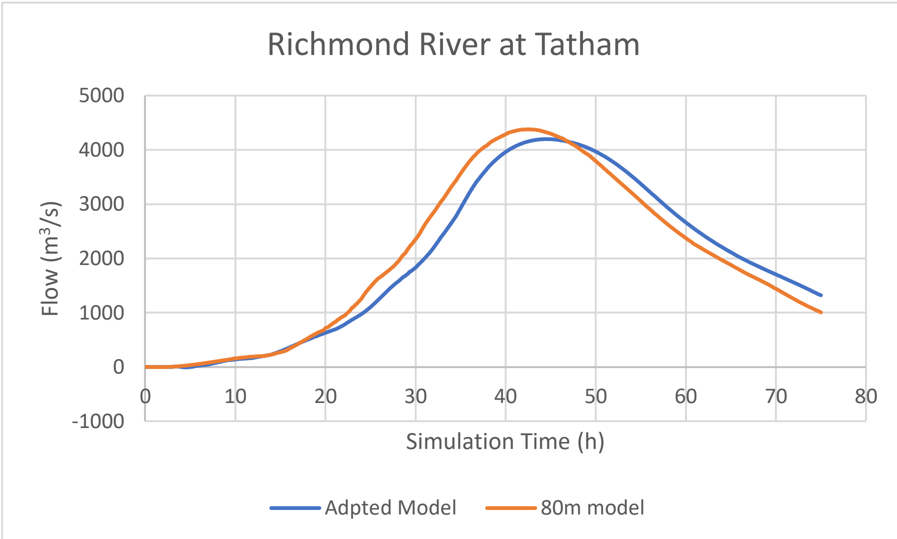

Figure F.2 Comparison at Tatham (24h storm duration)

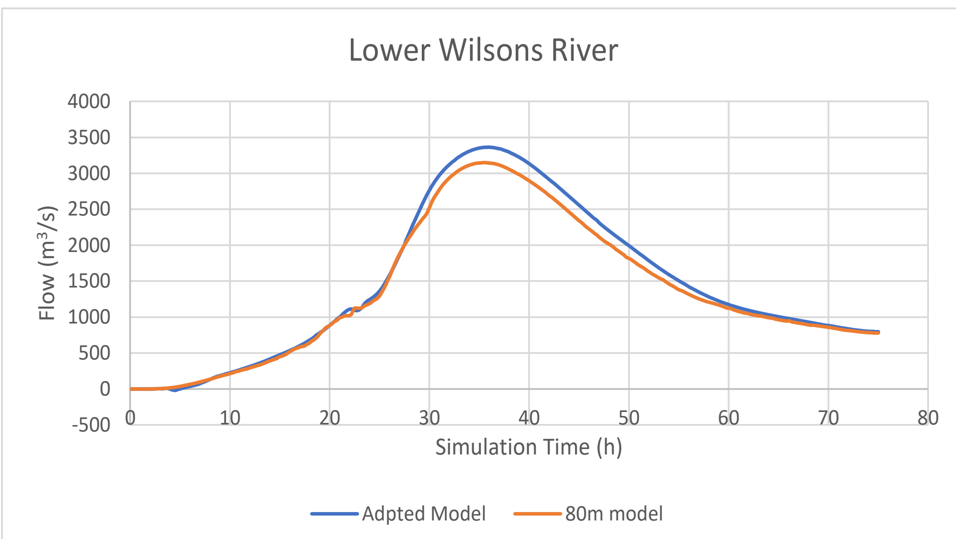

Figure F.3 Comparison on Lower Wilsons River (24h storm duration)

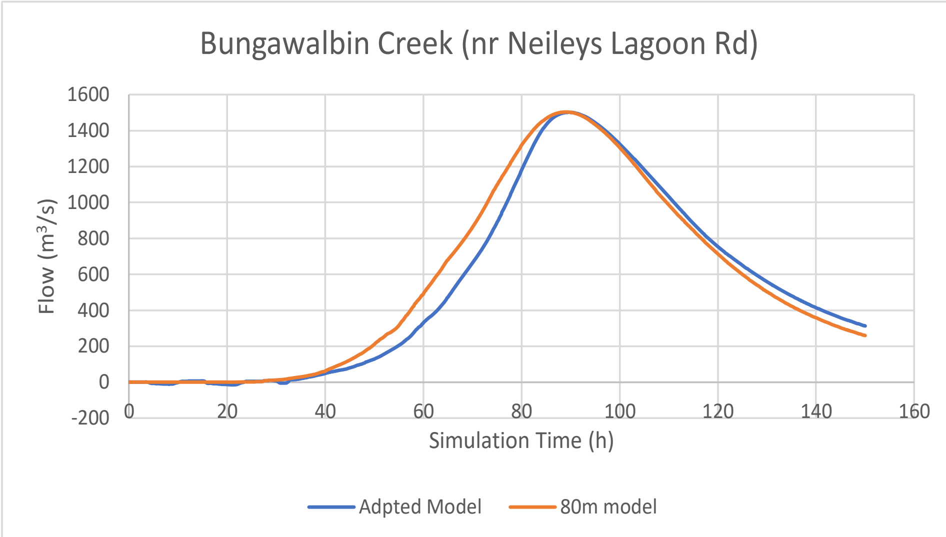

Figure F.4 Comparison on Bungawalbin Creek (72h storm duration)

Annex G Peer Review

G.1 Introduction

The Richmond Valley Flood Study has been subject to continual internal peer review. This has included formal reviews at key stages of the assessment and informal reviews through frequent communication across the project team.

Where the peer reviews noted issues or inconsistencies, then these were investigated and addressed. The end result is a study that has been subject to rigorous quality assurance. This section summarises the formal peer review process, both internal and external.

G.2 Formal Peer Review

Formal peer reviews were conducted at the key stages of the assessment as shown in Table G.1.

Table G.1. Formal Peer Reviews

Study Stage	Review Date	Review Type	Subject of Review	Description
Data Collection	Mar 2021	Internal	Topography	Review of available LiDAR and proposed use in hydraulic model
Data Collection	Mar 2021	Internal	Topography	Review of existing ground survey (Michael Survey) against LiDAR
Hydrologic Model Build	Apr 2021	Internal	Hydrologic Model	Main outcome of review was recommendation that component models consolidated into a single model.
Hydraulic Model Build	Apr 2021	Internal	Hydraulic Model	Complete review of model. Resulted in adjustment of finer resolution domains to improve model stability and simulation times.
Model/Build Calibration	Jun 2021	External (RVC)	Memo documenting model build and preliminary calibration	Review of model extents/ proposed resolutions and preliminary calibration. BMT advised that will experiment with achieving higher model resolution.
Model Calibration	Aug 2021	External (RVC, DPIE)	Calibration	Presentation of draft calibration results. External feedback that calibration generally good but see if any improvements can be made at Casino.
Model Calibration	Dec 2021	External (RVC)	Calibration	Updated calibration results presented. General improvement at all locations, particularly at Casino.
Design Modelling	Dec 2021	Internal	Event Selection	Event selection approach discussed. It was determined to use a course

Study Stage	Review Date	Review Type	Subject of Review	Description
				resolution hydraulic model for event selection purposes.
Design Modelling	Dec 2021	Internal	Hydraulic Model	Review resulted in modifications made to model to better accommodate PMF flood including additional outlets to the ocean near Broadwater.
Draft Report	Feb 2022	Internal	Report	Full internal review of draft report (technical and grammatical review)
Draft Report	Feb 2022	External (RVC, DPIE)	Report	Review of Draft Report
Draft Report	May 2023	Internal	Report	Full internal review of draft report (technical and grammatical review)

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