

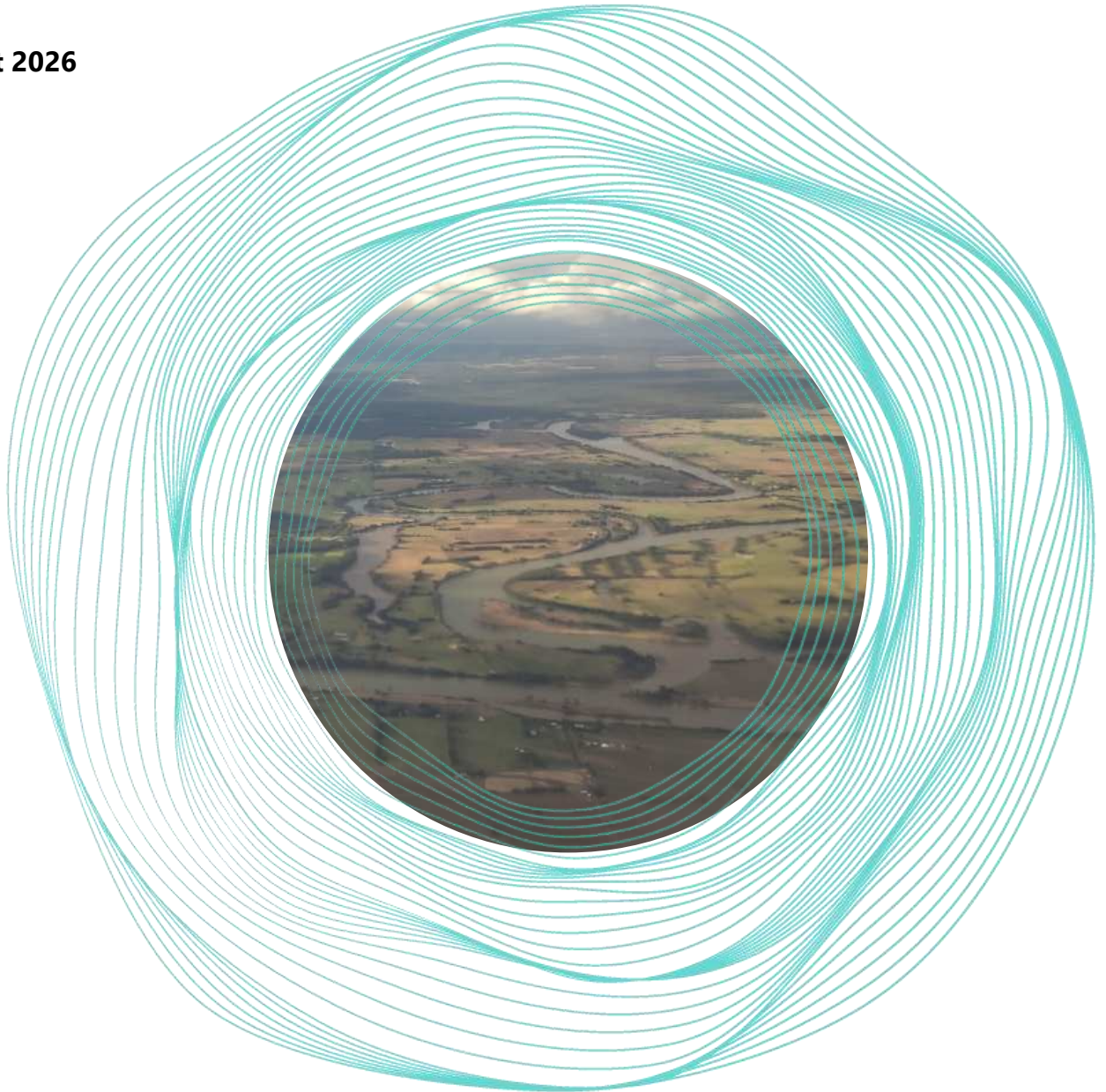
PORT MACQUARIE-HASTINGS COUNCIL

Hastings River Flood Study Extension

Updated Exhibition Draft

August 2026

Rev F



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Appendix D – Comparison Between Updated and Previously Adopted Design Event Flow Hydrographs

Appendix E – Extended RMA-2 Model Design Flood Event Mapping

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Appendix I – Flood Function Mapping

1 Introduction

The Hastings River catchment is located within the Port Macquarie Hastings Council Local Government Area (LGA) in the Mid North Coast region of New South Wales (NSW). The catchment comprises an area of approximately 3700 km² draining from the Great Dividing Range approximately 120 kilometres from the coast and discharges to the Tasman Sea at Port Macquarie, roughly 300 kilometres north of Sydney (refer **Figure 1-1**).

The major tributaries draining the catchment include the Hastings River, Forbes River, Ellenborough River, Pappinbarra River, Morton's Creek, Maria River, and the Wilsons River.

The catchment has a history of flooding, with extensive damage caused to private and public property in proximity to the Hastings River and major tributaries in events in 1963, 1968, 1978, 1995 and more recently in 2013 and 2021. The 1963 and 1968 floods were previously the largest on record in the lower Hastings River and have been used as benchmarks for development in the region.

Port Macquarie Hastings Council (Council) is responsible for local planning and land management within its LGA, including the management of flood prone land. Council engaged Advisian (*part of the Worley Group*) to update and extend the Hastings River Flood Study. The update was initiated in response to the significant flooding in 2021 which led to significant flooding across the catchment, including in areas not covered by Council's existing flood studies. This recent flood event and growing development pressure has led to the need for an expansion of the study area.

The Extended Hastings River Flood Study includes modelling for additional watercourses upstream of Wauchope and Telegraph Point along the Hastings and Wilson Rivers, respectively. The main components of the extension are shown in **Figure 1-2** and include:

- The Hastings River floodplain between Wauchope and Kindee Bridge near Long Flat;
- The Wilsons River floodplain upstream of Telegraph Point;
- The Pappinbarra River and Mortons Creek floodplains; both major tributaries of Hastings River; and
- Floodplains of Bril Bril Creek and other major tributaries of the Wilson River.

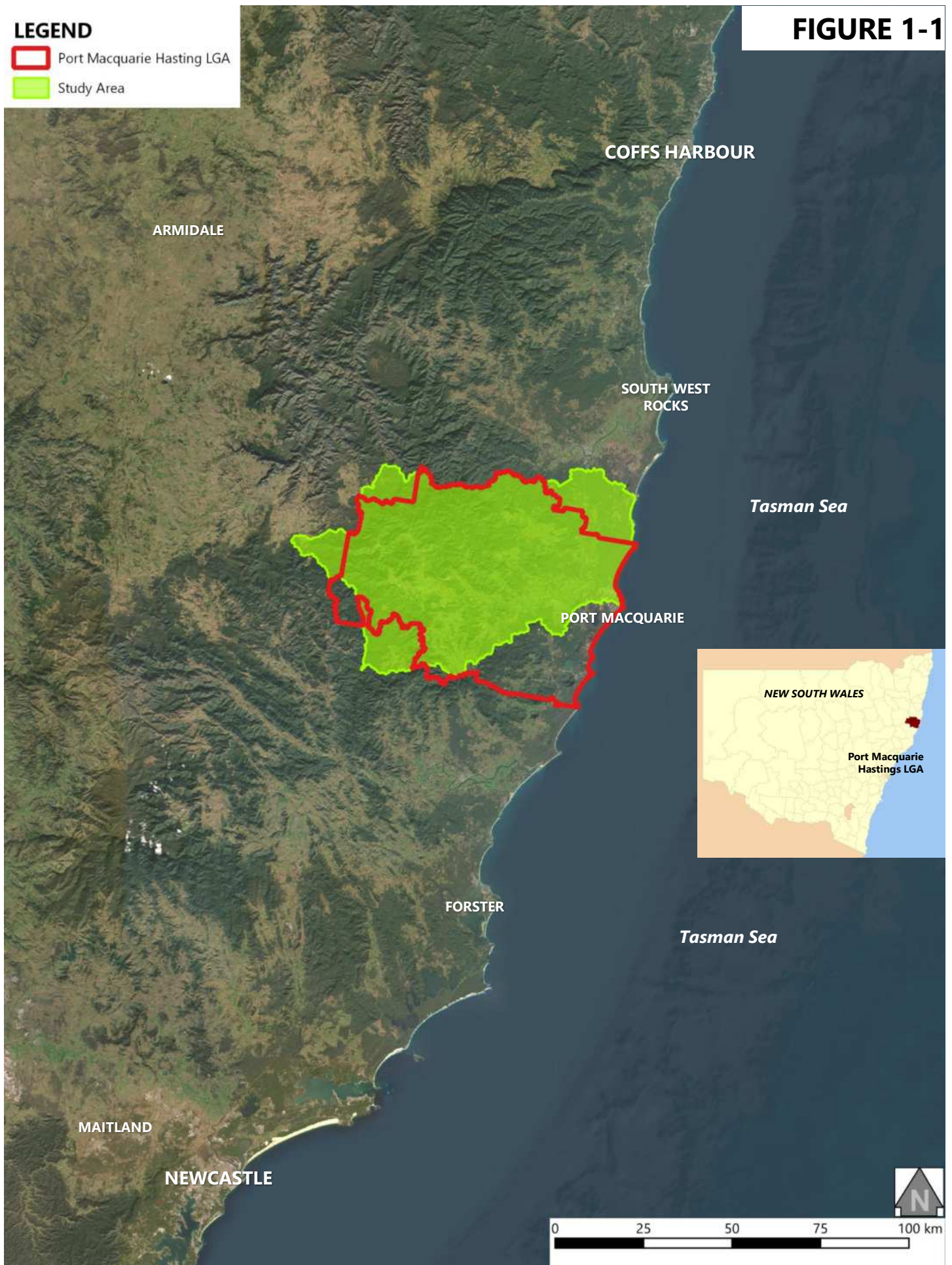
Objectives of this study include:

- Extension of Council's flood mapping data set into additional areas upstream of the study area of the Hastings River Flood Study (2006) and subsequent flood study updates.
- Develop and calibrate a hydrologic model for the Hastings River catchment in accordance with Australian Rainfall and Runoff 2019.
- Extend the existing two-dimensional RMA-2 flood model along the Hastings, Wilson and Maria Rivers.
- Develop a new TUFLOW hydraulic model to cover the major tributaries of the Hastings and Wilson Rivers (refer **Figure 1-2**).
- Validate the RMA-2 and TUFLOW models to the March 2021, May 2025 and February 2013 historic floods.
- Generate flood mapping for existing catchment conditions for a range of design events and Council's adopted climate change scenario.
- Predict flood conditions for a Probable Maximum Flood (**PMF**) event to replace existing mapping generated for a nominal 'Extreme' event.

LEGEND

-  Port Macquarie Hasting LGA
-  Study Area

FIGURE 1-1

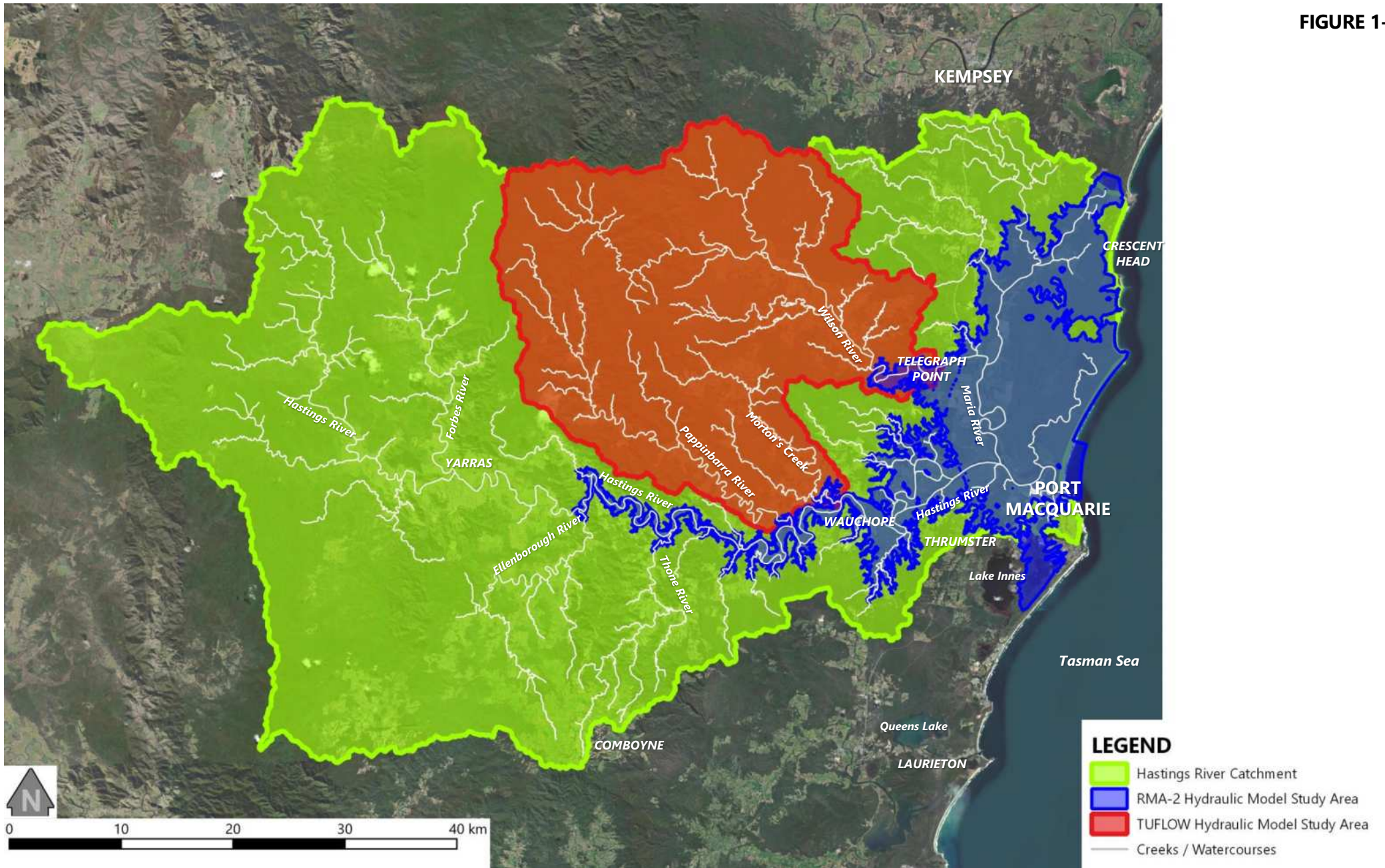


Prepared by:

Advisian
Working Smarter

HASTINGS RIVER STUDY LOCATION

FIGURE 1-2



2 Background

2.1 The Study Area

The study area of the Hastings River catchment has a total area of about 3693 km² (refer **Figure 2-1**). The study catchment is located on the NSW Mid North Coast and drains from the Great Dividing Range to the Tasman Sea at Port Macquarie. The ocean entrance is an open and trained intermediate wave dominated barrier estuary.

A two-model approach has been adopted for the hydraulic modelling to effectively model the Hastings River and its major tributaries. This involved development of a catchment wide model that focused on the Hastings, Wilson and Maria Rivers, and a major tributaries model that focused on the Pappinbarra River, Morton's Creek and the upper reaches of the Wilsons River. This approach uses the strengths of different modelling software to accurately model the area. These modelling areas are shown in (refer **Figure 1-2**).

The tidal influence extends up the Hastings River for approximately 32 kilometres to Bains Bridge, which crosses the river about 3 kilometres west of Wauchope. Downstream of Wauchope, the river meanders through a wide floodplain which has been cleared for agricultural use. It is soon after Wauchope that the floodplain widens substantially, before narrowing again for the Pacific Highway crossing at Dennis Bridge. Below this point, the floodplain widens to form an expansive coastal floodplain that extends north from the Oxley Highway to Crescent Head Road.

Further upstream of Wauchope in the middle catchment, the river narrows and meanders through small floodplains cleared for agricultural use and small residential areas. In the upper catchment, the river is incised within narrow floodplains. These areas are relatively steep and characterised by dense forest. The major tributaries have similar characteristics of the upper and middle catchment.

The topography of the study catchment is shown in **Figure 2-1**. The highest point in the catchment is located at the top end of Fenwick's Creek, at the western extent of the catchment area near Yarrowitch. This reaches a peak of 1,224 mAHD. The upper catchments of the major tributaries of the Forbes River, the Pappinbarra River and the Ellenborough River also begin at an elevation of above 1,000 metres. The terrain along the major watercourse quickly drop to below an elevation of 100 metres as the rivers begin to flatten in the middle catchment. Areas downstream of the North Coast Railway, downstream of Wauchope and Telegraph Point, as well as the catchment area of Maria River are relatively flat, as elevations across significant parts of the floodplain remain below 10 mAHD.

2.2 Historic Floods

The Hastings River catchment has experienced many significant floods over its history since European settlement in the early 19th Century, occurring in 1864, 1894, 1929, 1950, 1954, 1963, 1968, 1978, 2013, 2021 and 2025.

The *Hastings River Flood Study (2006)* provides a record of historic floods in Table 5 based on stream flow records at major gauges in the catchment. The record is updated in **Table 2-1** to include the additional nineteen (18) years of records available since the 2006 Flood Study.

FIGURE 2-1

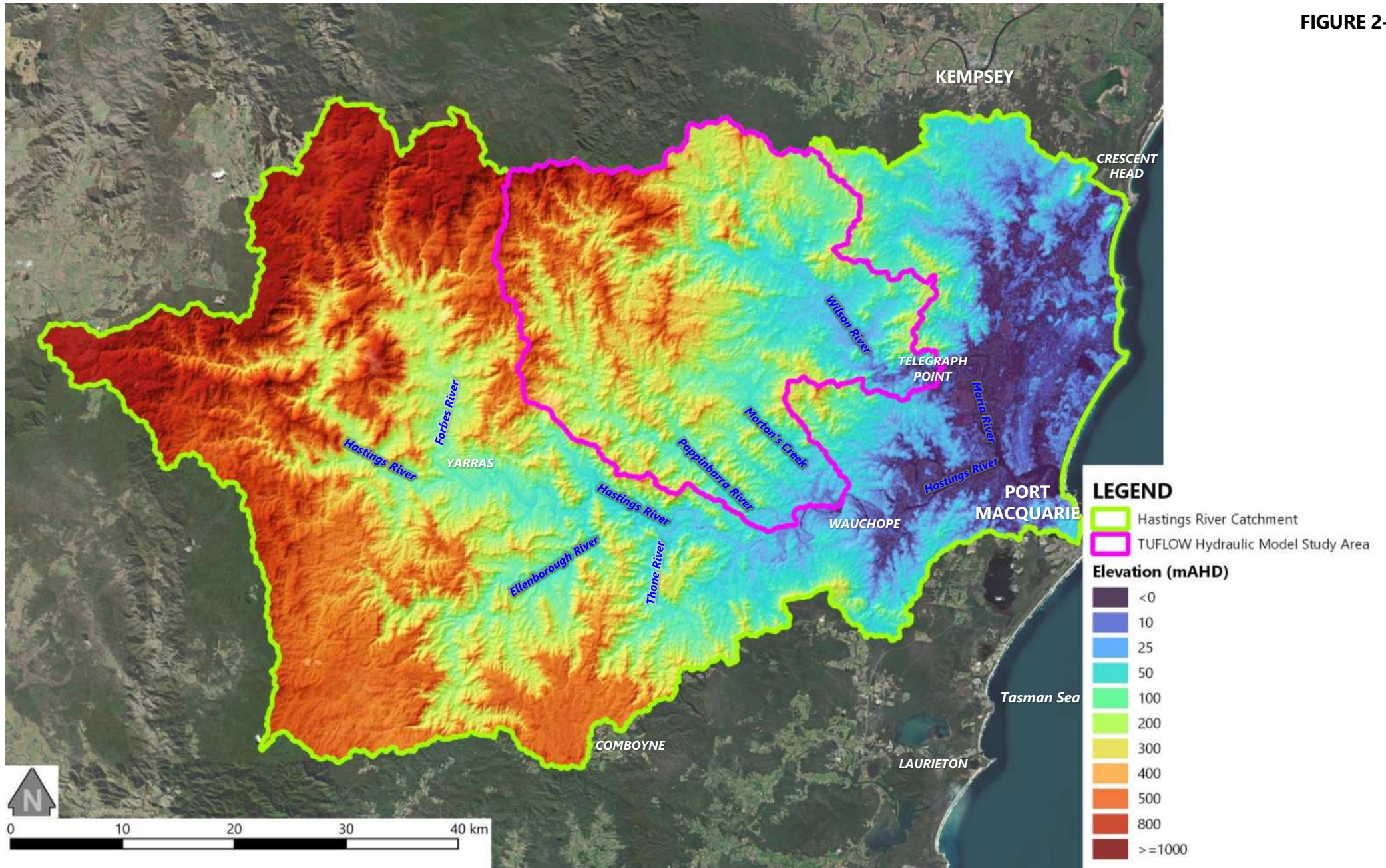


Table 2-1 Historic Record of Peak Flows at Stream Level Gauges

Gauge Number	Gauge Name	Duration of Records	Flood Date	Peak Flow (m ³ /s)	Ranking (1 to 3)
207013	Ellenborough River downstream Bunnoo River	1975 - 2024	March 1978	1,028	3
			May 1980	470	
			October 1982	743	
			July 1985	795	
			February 1990	842	
			March 1995	1,690	1
			February 2013	1,024	
			March 2021	1,507	2
			May 2025	941	
207004	Hastings River at Kindee Bridge	1945 - 2024	October 1972	1,533	
			March 1974	2,138	
			March 1977	1,318	
			March 1978	3,293	3
			May 1980	1,769	
			October 1982	1,601	
			July 1985	1,575	
			March 1989	1,563	
			February 1990	2,207	
			March 1995	2,585	
			February 2013	3,689	2
			March 2021	4,328	1
			May 2025	2,996	1
207014	Wilson River at Avenal	1984 - 2024	July 1985	981	3
			March 1989	788	
			February 1990	886	
			March 1995	331	
			February 2013	1,449	2
			March 2021	Incomplete	1
			May 2025	835	
207010	Pappinbarra Creek at Beechwood	1973 - 2024	July 1985	242	
			February 2013	606	
			March 2021	Incomplete	
			May 2025	473	

2.3 Previous Studies

The characteristics of flooding of the Hastings River and its tributaries are documented in a number of studies prepared for Port Macquarie-Hastings Council over the last 17 years. These studies include:

- *'Hastings River Flood Study' (August 2006)*
- *'Hastings River Floodplain Risk Management Study' (2012)*
- *'Hastings River Floodplain Risk Management Plan' (2014)*
- *'Hastings River Flood Study Update (Climate Change Assessment)' (2018)*
- *'Hibbard Precinct Flood Study' (2019)*
- *'Maria River Road, North Shore - Flood Impact Assessment' (2022)*

All of these reports have relied on the results of flood modelling undertaken using a two-dimensional flood model that has evolved over the 15-year period. This evolution has involved progressive modifications to the flood model to incorporate updated and more detailed topographic data as it has become available as well as the inclusion of major infrastructure that has been constructed in the floodplain including the upgrade to the Oxley Highway and construction of the new Pacific Highway between the Oxley Highway and Telegraph Point.

The *'Updated Hastings River Flood Study' (2018)* documents modelling results for the 1% AEP flood and a range of climate change scenarios using the updated version of the flood model. The climate change modelling considered various climate change scenarios which were identified in the 2012 and 2014 studies to provide Council with an understanding of the potential future changes to flood characteristics along the Hastings River downstream of the Bains Bridge crossing near Beechwood, as well as along the Wilson and Maria Rivers which drain the northern section of the valley.

The computer model was created using the RMA-2 software. RMA-2 is a finite element modelling software package that employs a variable grid geometry in which elements with irregular and curved boundaries can be modified as required without the need for regeneration of the entire grid.

Therefore, any proposed development on the floodplain can be incorporated into the existing model and the associated impacts can be quantified by comparing model results from simulations of "pre-development" and "post-development" scenarios.

In addition to these topographic changes, the updated modelling included modifications to selected model parameters and an overall refinement of the RMA-2 model network to better utilise the processing and modelling capabilities of present-day computers and the RMA-2 modelling software.

Following completion of the *'Updated Hastings River Flood Study' (2018)*, Council commissioned Advisian to undertake detailed flood modelling and investigations for the Hibbard Precinct for the purpose of better defining the floodway between Fernbank Creek and the Hibbard Precinct. This work led to further refinements being incorporated into the RMA-2 model that were focused across Hibbard, between Tuffins Lane to the west and Kemp Street to the east.

Most recently, the RMA-2 model was extended to include the Maria River and Connection Creek floodplains as part of investigations completed for the *'Maria River Road, North Shore - Flood Impact Assessment' (2022)*.

2.4 Study Objectives

Objectives of this study include:

- Extension of Council's flood mapping data set into additional areas upstream of the study area of the Hastings River Flood Study (2006) and subsequent flood study updates.

- Generate flood results, including hazards and hydraulic categories, for areas impacted by the February 2021 event and which are currently outside of the established model extents.
- Develop and calibrate a hydrologic model for the Hastings River catchment in accordance with Australian Rainfall and Runoff 2019.
- Extend the existing two-dimensional RMA-2 flood model along the Hastings, Wilson and Maria Rivers.
- Develop a new TUFLOW hydraulic model to cover the major tributaries of the Hastings and Wilson Rivers, including the Pappinbarra River, Mortons Creek, Frazers Creek, and Bril Bril Creek.
- Validate the RMA-2 and TUFLOW models to the March 2021, May 2025 and February 2013 historic floods.
- Generate flood mapping for existing catchment conditions for a range of design events and Council's adopted climate change scenario.
- Predict flood conditions for a Probable Maximum Flood (PMF) event to replace existing mapping generated for a nominal 'Extreme' event.

2.5 Relevant Manuals and Guidelines

2.5.1 Floodplain Development Manual, 2005

The *Floodplain Development Manual 2005* (the Manual) incorporates the NSW *Flood Prone Land Policy* and guides its implementation in the floodplain risk management process. It aims to reduce the impacts of flooding and flood liability on individual owners and occupiers of flood prone property and to reduce private and public losses resulting from floods.

The Manual outlines a merit-based framework to assist with floodplain risk management. It confirms that responsibility for management of flood risk remains with local government and provides guidance for councils in the development and implementation of local floodplain risk management plans.

A series of floodplain risk management guidelines were developed by the former OEH (now DPE) to complement the *Floodplain Development Manual*, providing additional technical information to councils and consultants to support the preparation and implementation of floodplain risk management plans. Guidelines referenced in the preparation of this study include:

- Floodway definition
- Flood emergency response classification of communities
- Residential flood damage and supporting calculation spreadsheet
- SES requirements from floodplain risk management process.

2.5.2 Australian Rainfall and Runoff, 1987

Australian Rainfall and Runoff: A Guide to Flood Estimation 1987 (ARR 1987) provides a national guideline document and data used for the estimation of design flood characteristics in Australia. It has been widely used in flood studies throughout Australia since its release.

ARR 1987 has historically been adopted for hydrologic and hydraulic modelling along the Hastings River to guide the determination of appropriate design rainfall and flood hydrographs. This includes the recent *Hibbard Precinct Flood Study (2019)* and *Maria River Road, North Shore – Flood Impact Assessment (2022)*.

More localised flood studies and development assessments that are not influenced by the wider Hastings River catchment, have in recent years started to shift to *Australian Rainfall and Runoff: A Guide to Flood Estimation 2019* (ARR 2019). This includes studies completed for Yippin Creek near Wauchope in 2019 and Karikaree Creek near Thrumster in 2022/2023.

2.5.3 Australian Rainfall and Runoff, 2019

Australian Rainfall and Runoff: A Guide to Flood Estimation 2019 (ARR 2019) was issued for use by practitioners in draft form in November 2016 and was finalised in May 2019. It provides an updated national guideline document, data, and software suite for the estimation of design flood characteristics in Australia.

The guidelines update previous editions of ARR in light of recent advances in knowledge regarding flood processes, the increased computational capacity available to hydrologists and flood engineers, expanding knowledge and application of hydro-informatics, improved information about climate change and the use of more detailed hydrological methods.

The guidelines also incorporate new Intensity-Frequency-Duration (IFD) design rainfall estimates developed by the Bureau of Meteorology (BoM), using 30 years of additional observations from over 10,000 rainfall gauging stations and improved statistical analysis techniques.

ARR 2019 has gradually been implemented into studies completed for localised watercourses and tributaries of the Hastings River. The Extended Hastings River Flood Study represents the first catchment wide study undertaken for the Hastings River that will exclusively adopt ARR 2019.

2.5.4 Australian Disaster Resilience Handbook 7, 2017

Australian Disaster Resilience Handbook 7 Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia (AIDR 2017) provides guidance on best practice principles as presently understood in Australia. It provides information on the underlying principles that need to be considered when managing flood risk and formulating floodplain management plans and how to apply it, with the aim of promoting effective, equitable and sustainable land use across Australia's floodplains. A number of supporting documents are provided in conjunction with Handbook 7 and have been referenced in the preparation of this study.

2.5.5 Flood Risk Management Manual, 2023

The *Flood Risk Management Manual* (DPE 2023) updates the Floodplain Development Manual (2005) and several of the existing technical guides. It considers lessons learnt from floods and the application of the flood risk management process and manual since 2005.

Associated guides referenced in the preparation of this study include:

- FB01 Understanding and Managing Flood Risk
- FB02 Flood Function
- FB03 Flood Hazard
- LU01 Flood Impact and Risk Assessment

3 Data Collection and Review

3.1 Topographic and Bathymetric Survey Data

3.1.1 Light Detection and Ranging (LiDAR)

Topographic Digital Elevation Models (DEMs) as derived from triangulation of Airborne Light Detection and Ranging (LiDAR) survey of the study area were sourced from the online ELVIS system by Geoscience Australia. The primary LiDAR data sets and types included the following:

- NSW Spatial Services 1 metre LiDAR Digital Elevation Model (DEM) for Camden-Haven (2012-2017), Kempsey (2009-2016), Wingham (2017) and Cowarral (2017).
- NSW Spatial Services 2 metre LiDAR Digital Elevation Model (DEM) for Kempsey (2017), Wingham (2018) and Cowarral (2018).

The above LiDAR data sets were combined and adopted as the primary topographic data set for this study for use in the set-up of the hydrologic and hydraulic models. The combined topography as compiled from the above sources is presented in **Figure 2-1**.

3.1.2 Bathymetric Survey

Section 3.3.1 of the *Hastings River Flood Study (2006)* documents the bathymetric/hydrographic survey that was incorporated into the RMA-2 model during initial model development. Additional hydrographic data had not been acquired over the years that followed and as part of subsequent studies.

The *Australian Ocean Data Network* (<https://portal.aodn.org.au/search>) was searched as part of this study to determine whether any new hydrographic survey was available. Hydrographic survey that covered the extended study area where of particular interest.

The extent of available hydrographic survey is shown in **Figure 3-1** relative to the extent of the extended RMA-2 model. The two available hydrographic data sets are:

- Hydrographic survey collected by the Department of Land and Water Conservation in October 2000 covering parts of the Hastings, Wilson and Maria Rivers and Kings and Limeburners Creeks.
- Offshore hydrographic survey collected by the NSW Public Works Department in May-October 1991.

As shown in **Figure 3-1**, the available hydrographic survey covered a small extent of the extended study area. This includes several cross-sections along the Hastings River upstream of the Mortons Creek confluence, Wilsons River cross-sections upstream of Telegraph Point, and cross-sections along the Maria River and Connection Creek.

3.1.3 Stormwater Asset Survey

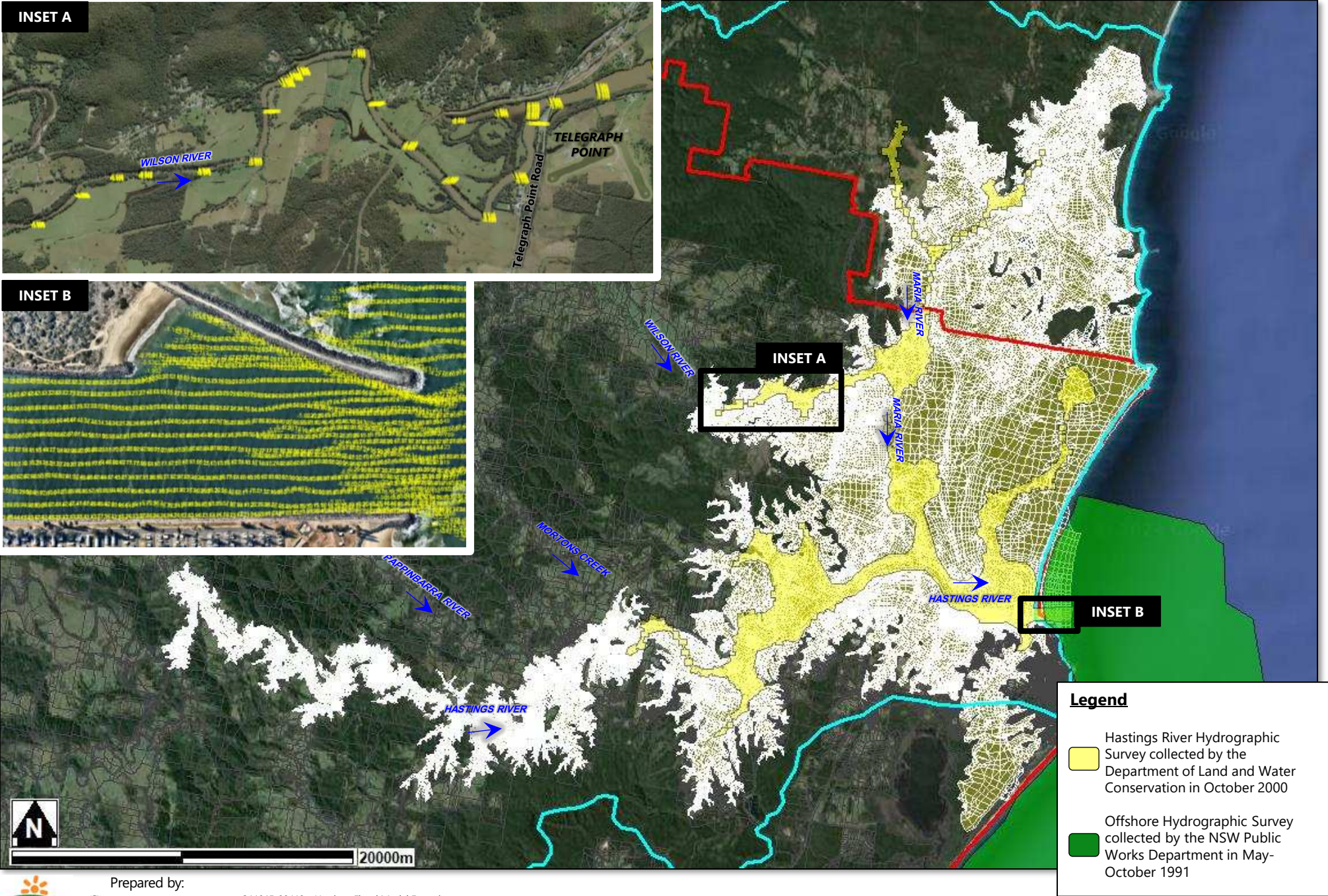
No stormwater asset data was provided by Council or collected for the purposes of this study.

3.1.4 Bridge and Culvert Data

Council provided GIS layers for the LGA that included information for:

- Bridges;
- Canal Structures;
- Pipe and box culverts;
- Flood mitigation drains and structures; and
- Stormwater open drains, end structures, Stormwater Quality Improvement Devices (SQID) and pits.

FIGURE 3-1



3.2 Historic Flood Data

A thorough search of available databases was undertaken to identify hydrometric data stations within or near the Hastings River catchment. This involved discussions with relevant agencies including the Bureau of Meteorology (BoM) and Manly Hydraulics Laboratory (MHL).

The search focused on pluviometers (*gauges that record continuous sub-daily rainfall*) and stations which were operating during the February 2013, March 2021 and May 2025 flood events that would be appropriate for use in flood model calibration. Relevant stations are discussed in the following.

3.2.1 Stream Level Data

Available stream level records for the Hastings River catchment were obtained from Manly Hydraulics Laboratory (MHL) and the Bureau of Meteorology (BOM). Stream level gauges utilised as part of this study are listed in **Table 3-1** along with the gauge identification/number according to MHL and the BOM.

The location of the streamflow gauges is shown in **Figure 3-2**.

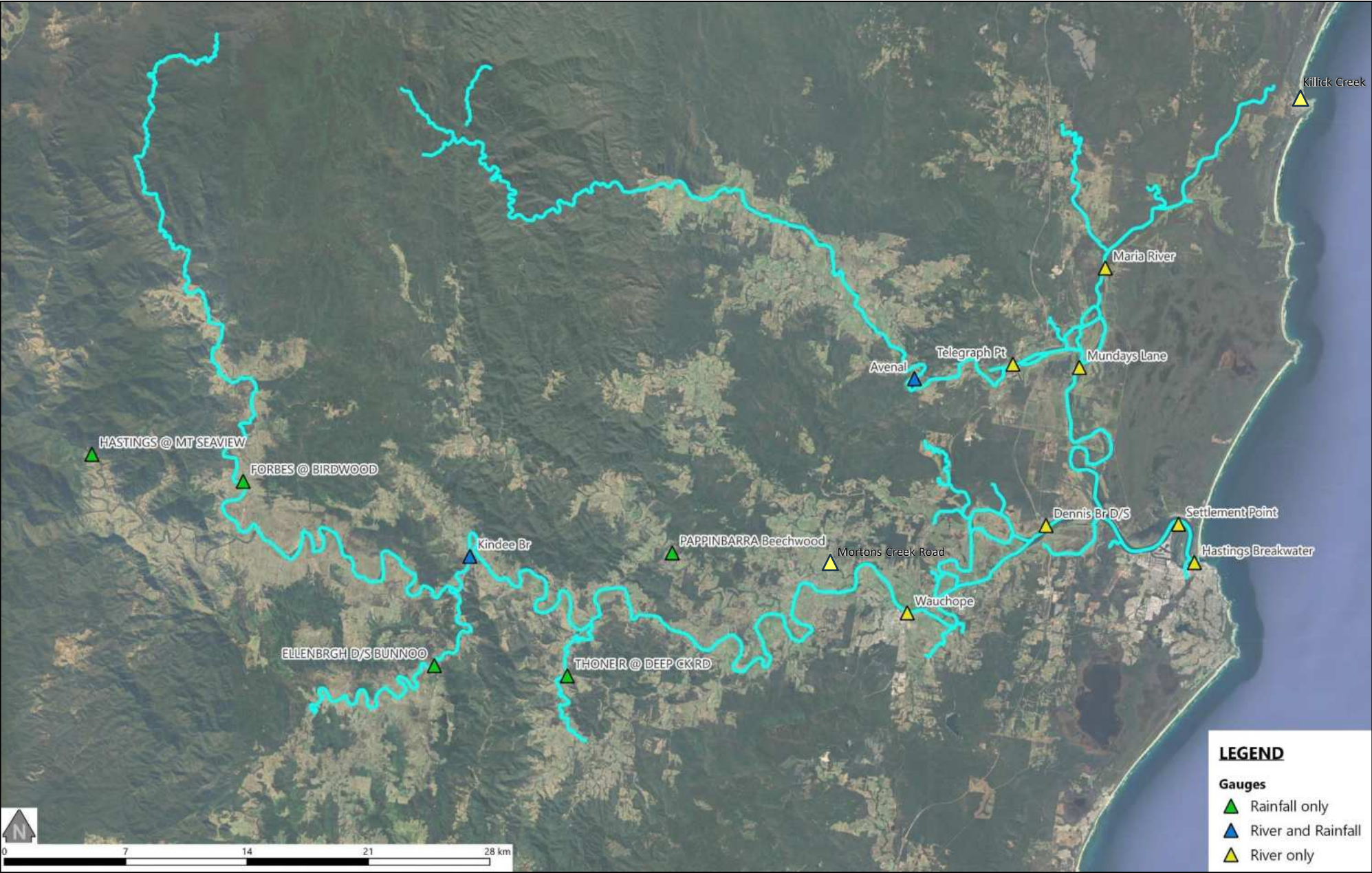
Table 3-1 Stream Level Gauges

Gauge (refer Figure 3-1)	BOM ID	MHL or WaterNSW ID
Hastings River at Kindee Bridge	60116	207004
Hastings River at Wauchope Railway Bridge	60124	207401
Hastings River downstream Dennis Bridge	560042	207444
Hastings River at Settlement Point	60133	207418
Hastings River at Entrance	560050	207420
Wilson River at Avenal	560011	207014
Wilson River at Telegraph Point	560011	207415
Maria River at Green Valley	/	207406
Maria River at Munday Lane	560067	2074127
Ellenborough River downstream Bunnoo River	560000	207013
Forbes River at Birdwood	560031	207006
Pappinbarra Creek at Beechwood	60138	207010
Mortons Creek at Mortons Creek Road	/	207017
Killick Creek at Crescent Head	/	207452

3.2.2 Rainfall Data

Rainfall data was sought from stations within and surrounding the Hastings River catchment. A particular focus was placed on obtaining data from pluviometers (*gauges that record continuous sub-daily rainfall*) to be able to resolve the temporal pattern of rainfall across the catchment during past flood events.

FIGURE 3-2



Prepared by:
Advisian

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**RMA-2 AND WBNM
GAUGES FOR VALIDATION
[STUDY AREA OVERVIEW]**

A list of all rainfall stations identified within and surrounding the Hastings River catchment is included in **Table 3-2**. The table also identifies whether the gauges were daily read or pluviometer (continuous read) for each of the recent weather events.

Table 3-2 Rainfall Gauges

Gauge	Status of the Recorded Data		
	February 2013	March 2021	May 2025
60052 Upper Rollands Plains	Pluviometer	Pluviometer	Pluviometer
560011 Telegraph Point (Wilson River)	Pluviometer	Pluviometer	Pluviometer
207406 (MHL) Green Valley (Maria River)	Pluviometer	Pluviometer	/
560016 Long Flat (Hastings River)	Pluviometer	Pluviometer	/
60168 Port Macquarie Airport AWS	Pluviometer	Pluviometer	Pluviometer
560020 Koree Island (Hastings River)	Pluviometer	Pluviometer	Pluviometer
60130 Forbes River (Birdwood)	Daily	Daily	/
60067 Upper Pappinbarra (Hillcrest)	Daily	Daily	/
60085 Yarras (Mt Seaview)	Daily	Daily	Pluviometer
60121 Elands Black Sands Creek	Daily	Daily	Pluviometer
60161 Comboyne Public School	Daily	Daily	Pluviometer
60163 Yarras Mountain	Daily	Daily	Pluviometer
60164 Yarrowitch	Daily	Daily	Pluviometer
560014 Port Macquarie Golf Club	/	/	Pluviometer
6016 Port Macquarie (Koala Street)	/	/	Pluviometer

The location of gauges containing rainfall data for the February 2013, March 2021 and May 2025 events are shown in **Figure 3-3**, **Figure 3-4** and **Figure 3-5**, respectively. Cumulative rainfall totals for each event are also superimposed on the figures.

3.2.3 Ocean Level Data

Data from the following ocean level stations was obtained for use in the study:

- Port Macquarie Tide Gauge (560050)
- Coffs Harbour Tide Gauge (205470)
- Crowdy Head Fishermans Wharf (208471)

3.2.4 Historical Flood Marks

At the commencement of this study, Council provided a GIS layer containing historic flood levels for the March 2021 flood event. It is understood that the flood levels have been compiled from survey of flood marks, debris lines, observed levels and High Water Marks (HWMs). As shown in **Figure 3-6**, the HWMs are scattered predominately across the lower Hastings River floodplain and across the extent covered by the RMA-2 model. In that regard, there are no HWMs surveyed within the extent of the proposed TUFLOW model.

FIGURE 3-3

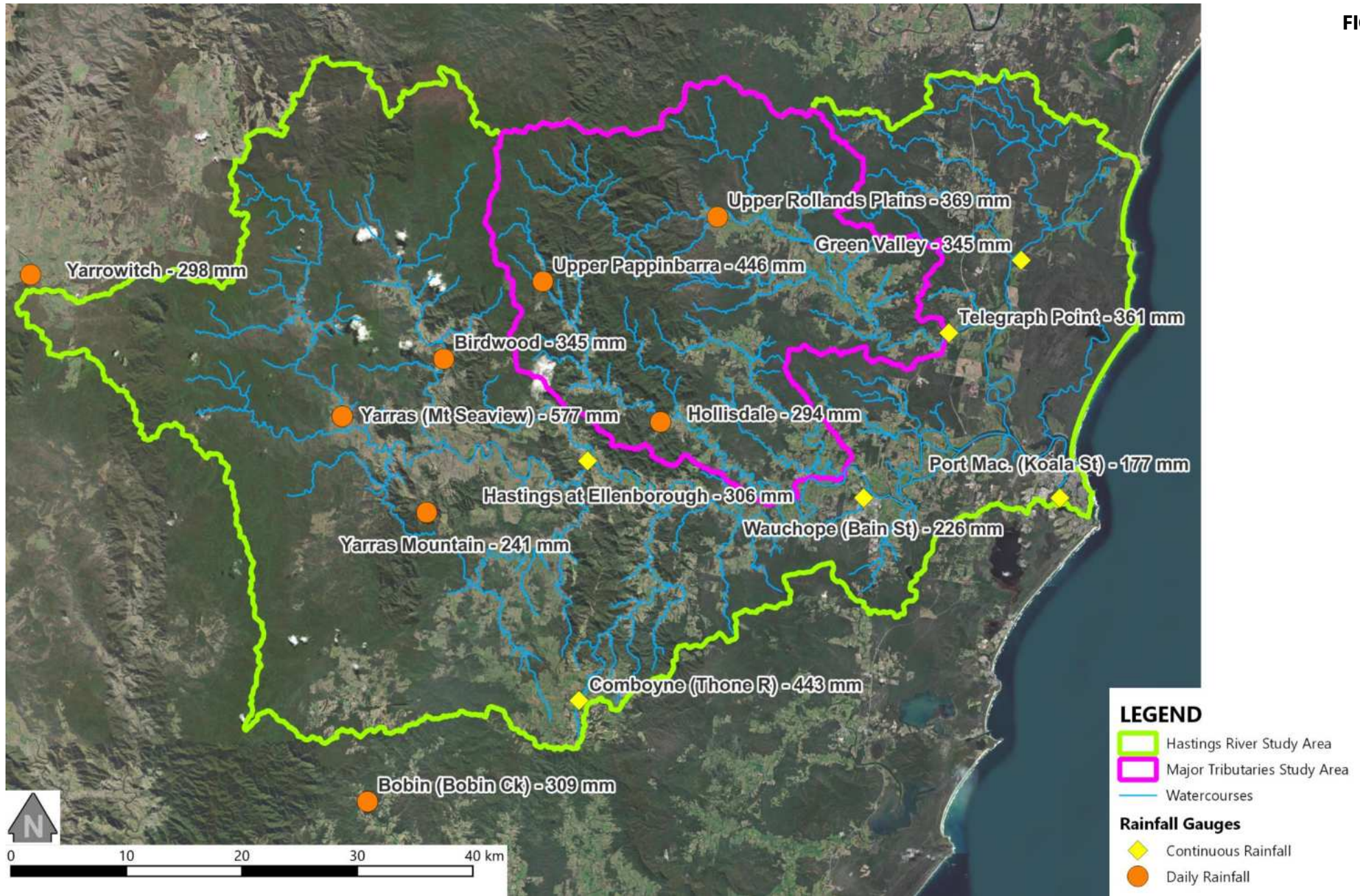


FIGURE 3-4

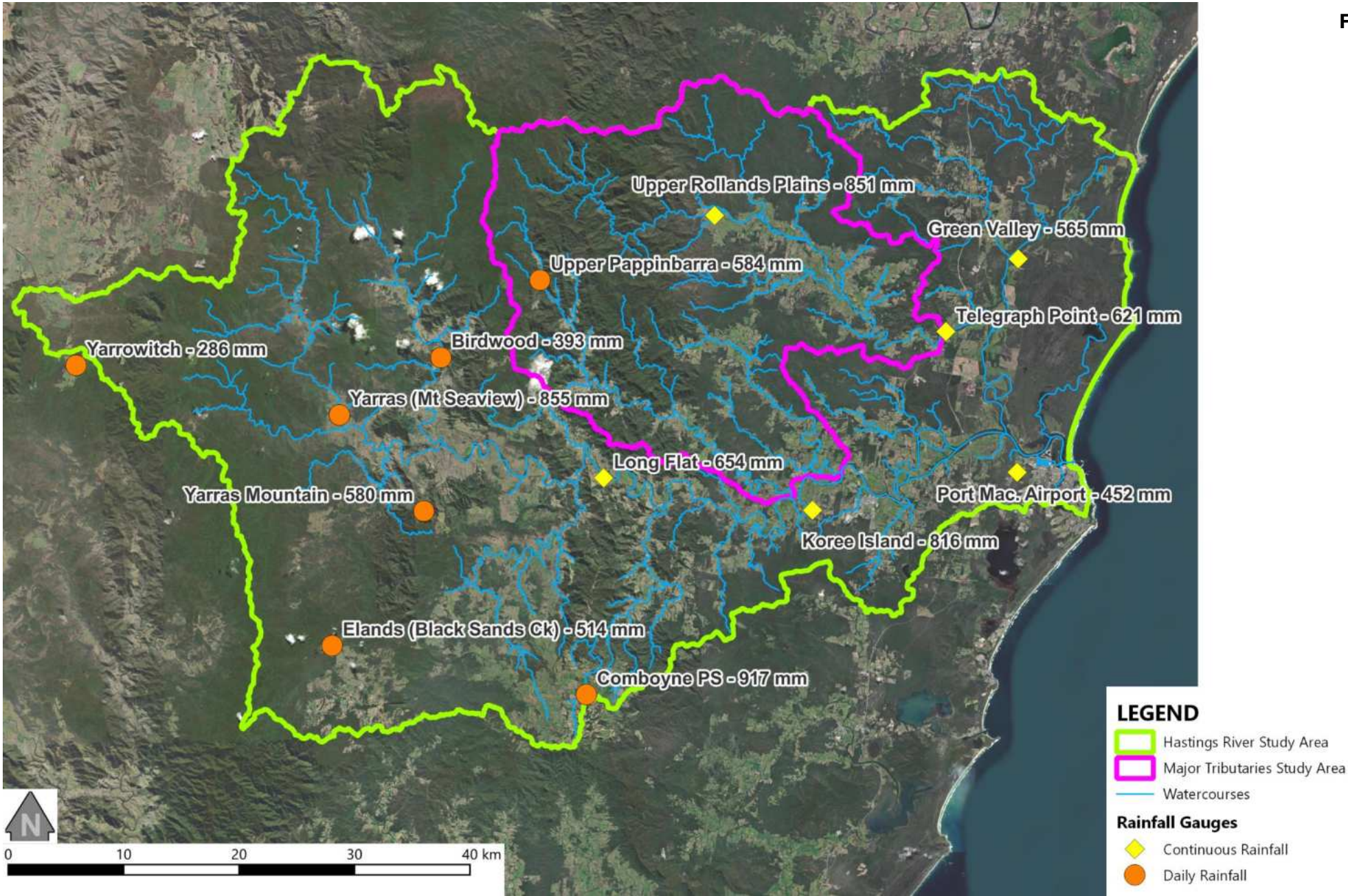


FIGURE 3-5

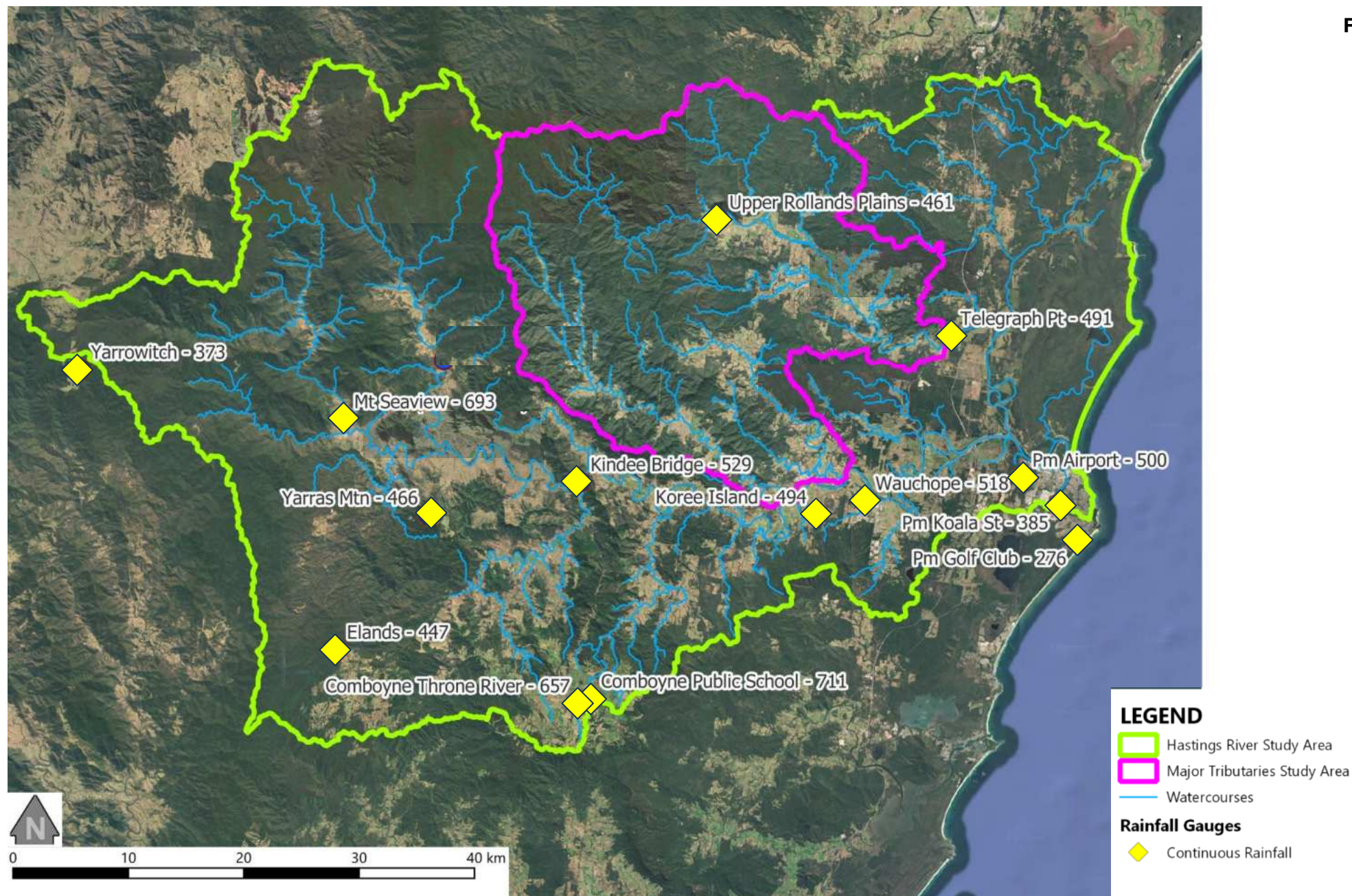
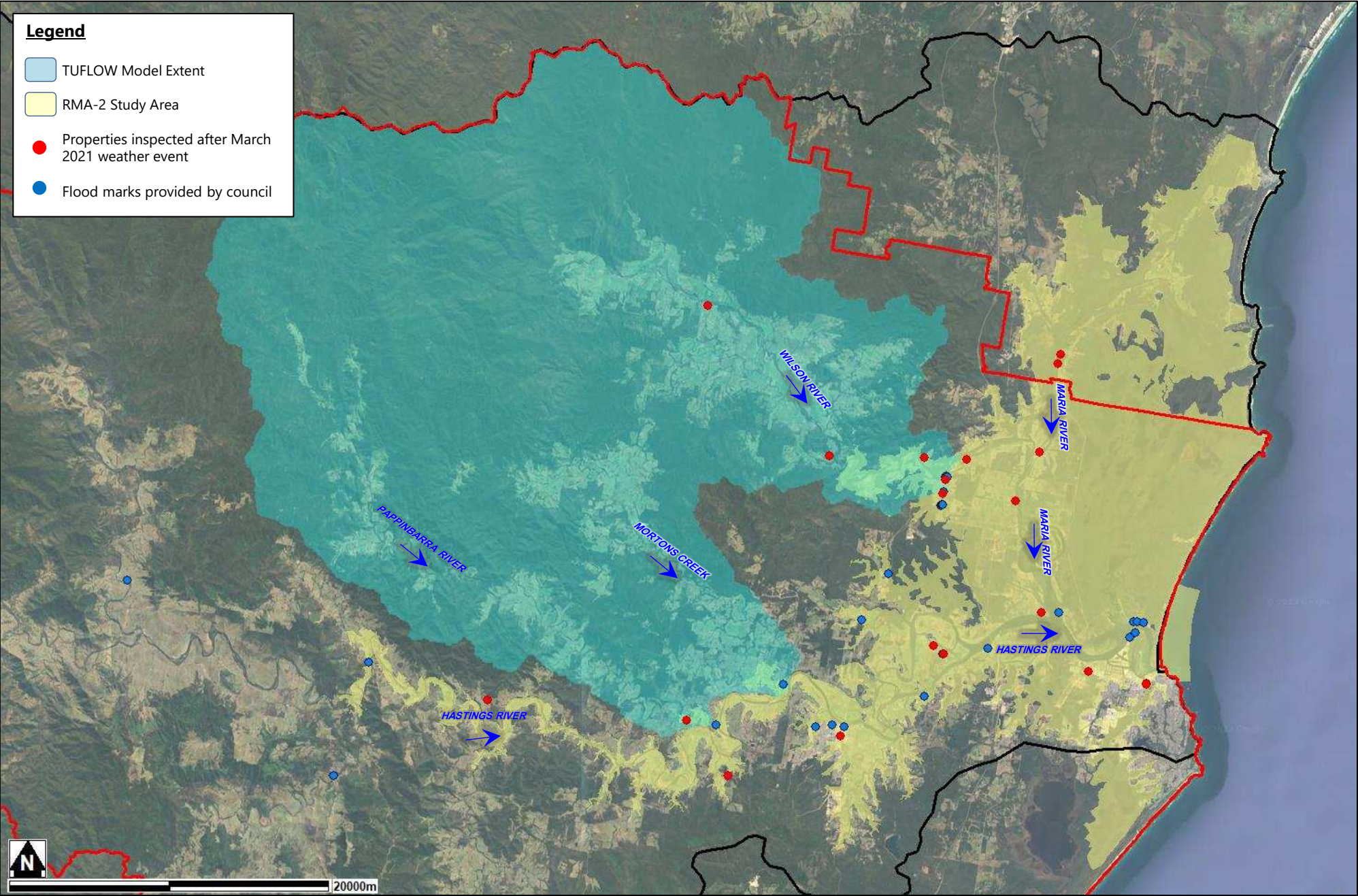


FIGURE 3-6



Prepared by:



311015-00418 - Hastings Flood Model Extension
fg311015-00418_230718_Hastings_A4L.pdf

**AVAILABLE HIGH WATER MARKS (HWMs)
FOR THE MARCH 2021 FLOOD EVENT**

The HWM data set provided by Council was further supplemented by flood records collected by Advisian as part of insurance inspections conducted in the aftermath of the 2021 flood. This additional HWM data is also superimposed on **Figure 3-6**.

Over forty (40) HWMs are available for the 2021 flood event based on the information received from Council and the information digitised following insurance inspections.

Council had also previously provided flood marks for the February 2013 event as part of the *Hibbard Precinct Flood Study (2019)*. Only three (3) HWMs were available for the 2013 event with all located within Hibbard.

No flood marks were identified for the May 2025 weather event at the time of model validation.

4 Model Updates and Development

4.1 Modelling Approach

As outlined in **Section 2.4**, the objective of this study is to extend Council's knowledge of expected flood characteristics in the upper Hastings River catchment which to-date has not been covered by past iterations of model development and flood studies. This includes the Pappinbarra, Mortons and upper Wilson River catchments which were significantly impacted during the February 2021 event. The study also seeks to utilise the recorded flood data from the February 2013 and March 2021 events for the purposes of model validating modelling results.

To achieve this, an approach has been implemented which utilises two separate hydraulic models as follows.

- The existing calibrated RMA-2 model has been adopted to define flood characteristics along the Hastings and Wilson Rivers and across the lower tidal reaches of the catchment. The RMA-2 model is to be extended along the Hastings and Wilson Rivers to incorporate recent studies completed across the catchment such as the '*Maria River Road, North Shore – Flood Impact Assessment*' (2022).
 - The RMA-2 model is considered an appropriate tool for this area as it has previously been successfully calibrated, and its flexible mesh allows for the model resolution to vary as appropriate to represent the various river and creek channels, lakes and lagoons, urban areas, and undeveloped areas of the floodplain.
- A new TUFLOW model has been developed to define flood characteristics above the tidal limit along the Hastings and Wilson Rivers, and the major tributaries including the Pappinbarra River and Morton's Creek.
 - TUFLOW is considered an appropriate tool for this area as it allows large areas to be efficiently and reliably modelled with tributaries and flow paths identified based on the catchment topography rather than interpretation by the modeler.

The existing RMA-2 flood model was developed as part of work undertaken in preparing the '*Hastings River Flood Study*' (Patterson Britton and Partners, 2006). This model covers the lower reaches of the catchment, extending from the ocean entrance at Port Macquarie upstream along the Hastings River beyond Wauchope, and along the Wilson River beyond Telegraph Point. It effectively covers the tidal reaches of the rivers, creeks, and lakes system.

A new TUFLOW hydraulic model has been developed to cover parts of the upper catchment and tributaries of the Hastings and Wilson Rivers. This model has a total area of 920 km² and covers the entire catchments of Pappinbarra River and Morton's Creek and parts of the Wilson River upstream of Telegraph Point. The TUFLOW model also includes the major tributaries of the Wilson River upstream of Telegraph Point. The extent of the TUFLOW model includes some overlap with the RMA-2 model. This allowed reliable downstream boundary conditions to be applied.

The extents of the RMA-2 and TUFLOW hydraulic models are shown in **Figure 1-2**.

Additionally, a new WBNM hydrologic model has been developed encompassing the entire catchment and will be used to derive inflow hydrographs to be applied to both hydraulic models. The WBNM model is to replace the existing XP-RAFTS hydrologic model that was developed as part of the original Hastings River Flood Study in 2006. WBNM was selected as it is a very robust software that has been validated against numerous catchments in NSW, while the XP-RAFTS software has been superseded by InfoWorks ICM and is no longer supported by the developer.

Further information detailing the development and updates made to the WBNM, RMA-2 and TUFLOW models is included in the following sections.

4.2 WBNM Hastings River Catchment Hydrologic Model

The existing XP-RAFTS hydrologic model of the Hastings River catchment is to be replaced as part of this study based on the XP-RAFTS software being no longer supported by the developers Innovyze. Development of a new WBNM hydrologic model represents a better investment based on the following considerations:

- The study and associated model extensions proposed presented an opportune time to develop a new model.
- WBNM is equipped for adoption and analysis of ARR2019 data and techniques.
- Structures, storages, and flow diversions can be added to the model to improve the stream routing behaviour and model calibration, and therefore the model's appropriateness for use in assessment of critical storm duration and 'average' ARR2019 rainfall temporal pattern.
- It is often expected to produce more reliable results than the 'direct rainfall' (rainfall-on-the-grid) approach in TUFLOW and be more computationally efficient, while the benefits of the direct rainfall approach in terms of identification of overland flow paths can be replicated by adopting an appropriately fine sub-catchment resolution.
- WBNM results files are easily converted for input as hydrographs into TUFLOW and RMA-2 hydraulic models.
- WBNM can be readily used by Council or other consultants as the software is license-free and models can be built and edited using typical GIS software, text files and/or the 'iWBNM' Excel interface.

Development of the WBNM hydrologic model is discussed further in the following sections.

4.2.1 Model Layout

The WBNM hydrologic modelling software will be used to simulate rainfall-runoff processes to determine flow hydrographs for input into the two hydraulic models. The sub-catchment delineation and linkage form the foundation of the WBNM hydrologic structure.

The hydrologic model extent for the Hastings River was determined from topographic data using the CatchmentSIM hydrologic and terrain analysis software. This was further delineated into 1,815 sub-catchments based on consideration of the catchment topography, watercourses, and the location of stream gauges and hydraulic structures such as bridges and culverts. The linkage between the sub-catchments was also determined by CatchmentSIM and was cross checked using GIS.

A higher resolution of sub-catchments was created within the TUFLOW hydraulic model area. The delineation was informed by initial TUFLOW direct rainfall modelling and was tailored to enable appropriate representation of key flow paths potentially posing flood risk to the community, property, or infrastructure to be achieved during the subsequent hydraulic modelling phase.

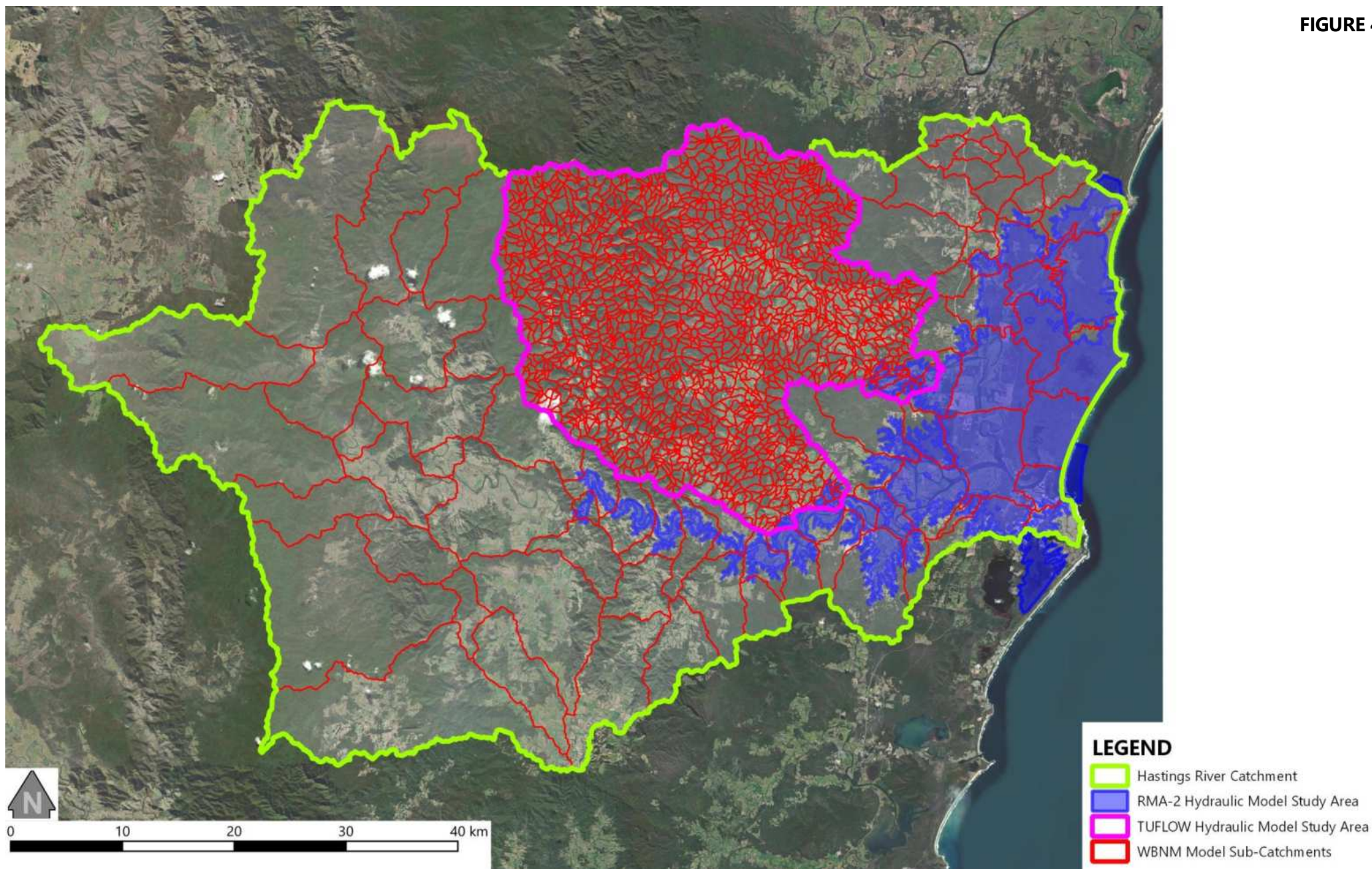
The original XP-RAFTS sub-catchment delineation was utilised as the base delineation across the remainder of the Hastings River catchment, before it was further refined to improve accuracy across the catchment.

The resulting WBNM hydrologic model sub-catchment delineation is presented in **Figure 4-1**.

4.2.2 Runoff Lag and Stream Routing Parameters

The primary parameters required by the WBNM model are the runoff lag factor 'C' and the stream routing factor 'F'.

FIGURE 4-1



The runoff lag factor 'C' controls the timing of locally generated runoff from each sub-catchment. A low C value represents a rapid runoff response, while a high value represents a slow runoff response. WBNM documentation recommends a runoff lag parameter value between 1.3 and 1.8, with a value close to 1.6 generally appropriate. A lag factor for impervious areas is also defined, with a default value of 0.1 recommended.

The stream routing factor 'F' determines the time it takes to travel along streams. WBNM documentation recommends a value of 1.0 to represent natural streams and flow paths. Lower values can be adopted to define stream modification such as clearing or straightening. Higher values can also be adopted to represent slower, flatter or particularly meandering flow paths.

The final WBNM parameters are to be determined through the model calibration and validation process.

4.2.3 Catchment Imperviousness

The degree of imperviousness of a catchment influences both the quantity and timing of runoff generated by a rainfall event.

The effective impervious percentage of each sub-catchment was determined through analysis of the surface material delineation developed for use in the RMA-2 and TUFLOW hydraulic models (refer **Figure 4-2**).

An effective percentage imperviousness was assigned to each surface material type as presented in **Table 4-1**. A specific area-averaged imperviousness was then assigned to each sub-catchment, resulting in values ranging from 0% in forested areas to 59% in areas with a high proportion of development and roads.

Table 4-1 Effective Impervious Percentage for different material types

Material	Effective Impervious Percentage
Watercourses	100%
Open Space, Medium and Heavy Vegetation	0%
Roads	100%
Residential and Commercial areas	50%

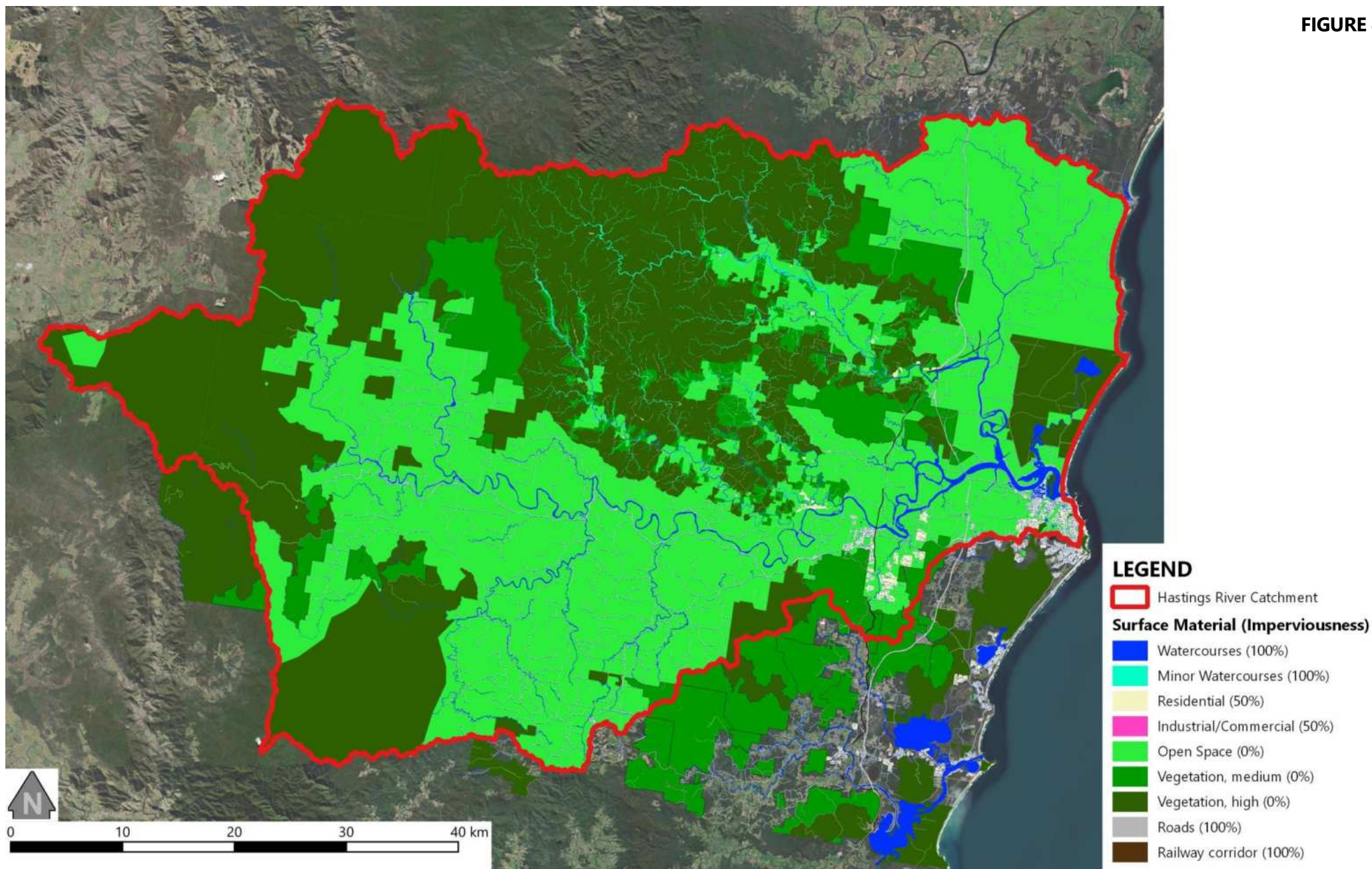
4.2.4 Rainfall Loss Rates

The term 'rainfall losses' refers to precipitation that does not contribute to direct runoff. During a storm such losses occur primarily due to the processes of interception by vegetation, and infiltration into the soil. The initial loss-continuing loss (IL-CL) approach is typically used in Australia to account for losses in the rainfall-runoff process and has been adopted in this study.

4.3 Extended Hastings River RMA-2 Hydraulic Model

The RMA-2 model that was updated as part of the *Hibbard Precinct Flood Study (2019)* was used as the base model for this study. This version of the model is currently adopted by Council for defining flood behavior and extents across the floodplain of the lower Hastings River.

FIGURE 4-2



4.3.1 RMA-2 Model Extension

The following major changes we made to the Hibbard Precinct Flood Study RMA-2 model as part of this study.

- Incorporation of the Maria River and Connection Creek floodplains originally added to the RMA-2 model as part of the '*Maria River Road, North Shore – Flood Impact Assessment*' (2022). This component of the RMA-2 model extension is shown in **Figure 4-3**.
- Extension of the RMA-2 network along the Hastings River from Wauchope to the confluence of the Ellenborough and Hastings Rivers. As shown in **Figure 4-3**, this leads to the RMA-2 model extending beyond Ellenborough Bridge.
- Inclusion of the lower reaches of Pappinbarra River and Morton's Creek to the north of the Hastings River.
- Extension of the RMA-2 network along the Wilson River to the Avenal Streamflow Gauge (*refer Figure 4-3*).

The changes outlined above led to an increase in the number of model nodes and elements from 49,300 and 57,800 in 2019, to 137,014 and 165,837, respectively. In that regard, the version of the RMA-2 model developed for this study has three times the number of nodes and elements to define the topography and roughness compared to the Hibbard Precinct Flood Study RMA-2 model.

The model extensions were predominantly incorporated based on topographic elevations defined by the 2012 LiDAR data set (*refer Section 3.1*). Isolated areas of the model were further refined based on additional survey data or design information such as along the new Pacific Highway and John Oxley Drive.

The extent of the extended RMA-2 network is shown in **Figure 4-3** with previous model versions superimposed. The comparison shows the extent to which additional detail has been incorporated into the RMA-2 model across the entire model domain, including the new areas along the Wilson River, Maria River, Connection Creek, and the extensions along the Hastings River upstream of Wauchope to Beechwood, Pipeclay, and Long Flat.

The final RMA-2 model network is shown in **Figure 4-4** to **Figure 4-8**. The upgraded network includes a much finer network spacing, particularly at hydraulic controls such as roads, impervious fences, buildings, and channels. In that regard, where the topography is generally flat, the network spacing can be larger without affecting the reliability of the flood model predictions.

Model Topography

Topographic elevations within the RMA-2 model extent were assigned to each node based on the combined 2012 LiDAR data set (*refer Section 3.1.1*). An example of the assigned RMA-2 model elevations is shown in **Plate 4-2**. The example shows the model topography in the vicinity of Koree Island and the Pappinbarra River outlet.

LiDAR may capture a lower resolution of ground points in heavily vegetated areas and does not penetrate water surfaces. As such, LiDAR data will not always provide a satisfactory topographic representation of watercourses for hydraulic modelling purposes. In light of this, node elevations within river and creek channels were manually adjusted to incorporate more reliable bed elevations. These changes were ideally made based on bathymetric survey and/or river cross-sections. Extensive bathymetry for the lower Hastings and Wilson Rivers was already available and incorporated into the model as part of the *Hastings River Flood Study* (2006).

Additional bathymetric survey was sourced from the Australian Ocean Data Network (<https://portal.aodn.org.au/search>) to cover as much of the extension areas as possible. This led to hydrographic survey being obtained for the Maria River north of the Wilson River confluence, and for the Wilson River upstream of Telegraph Point.

FIGURE 4-3

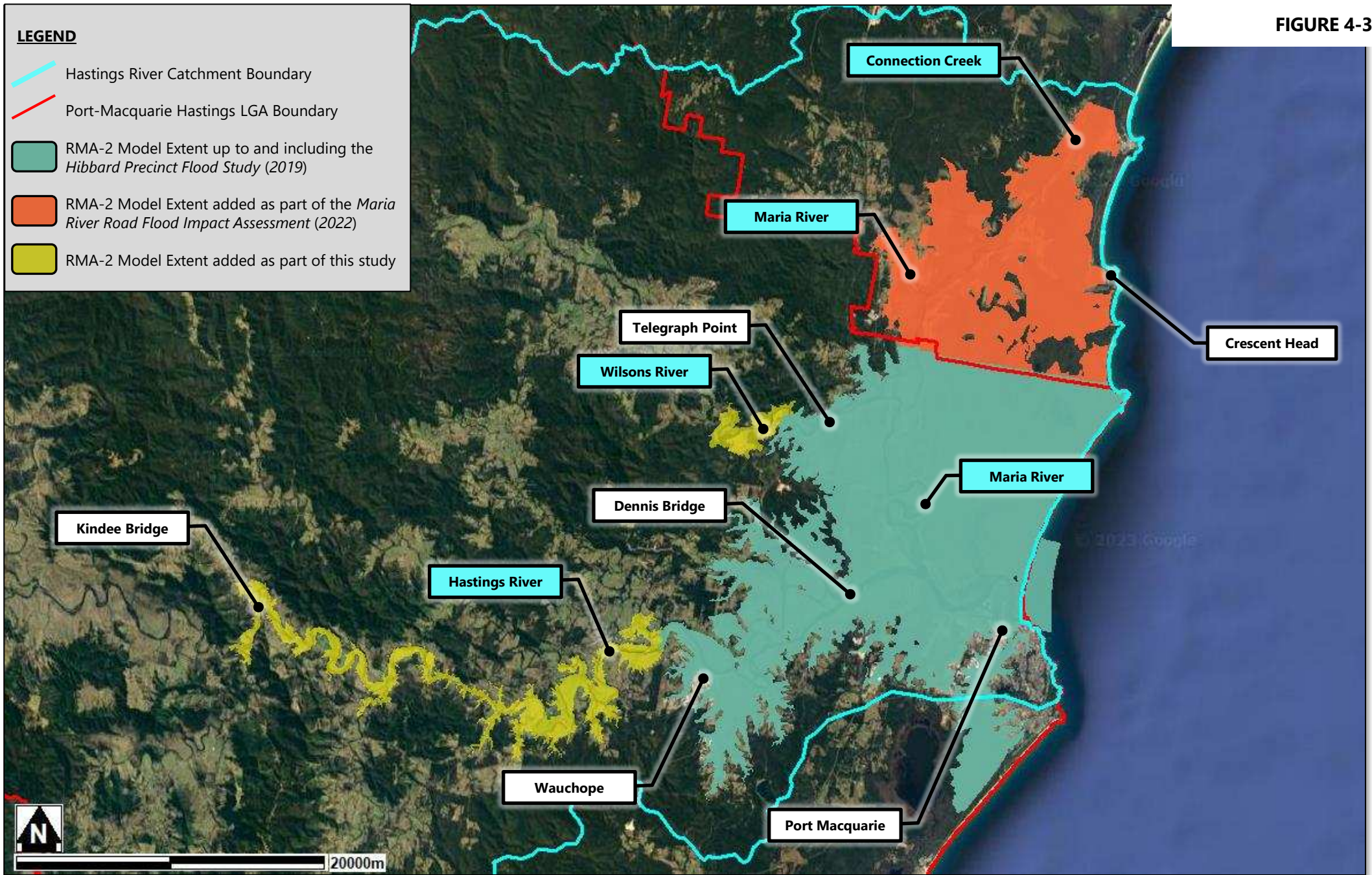


FIGURE 4-4

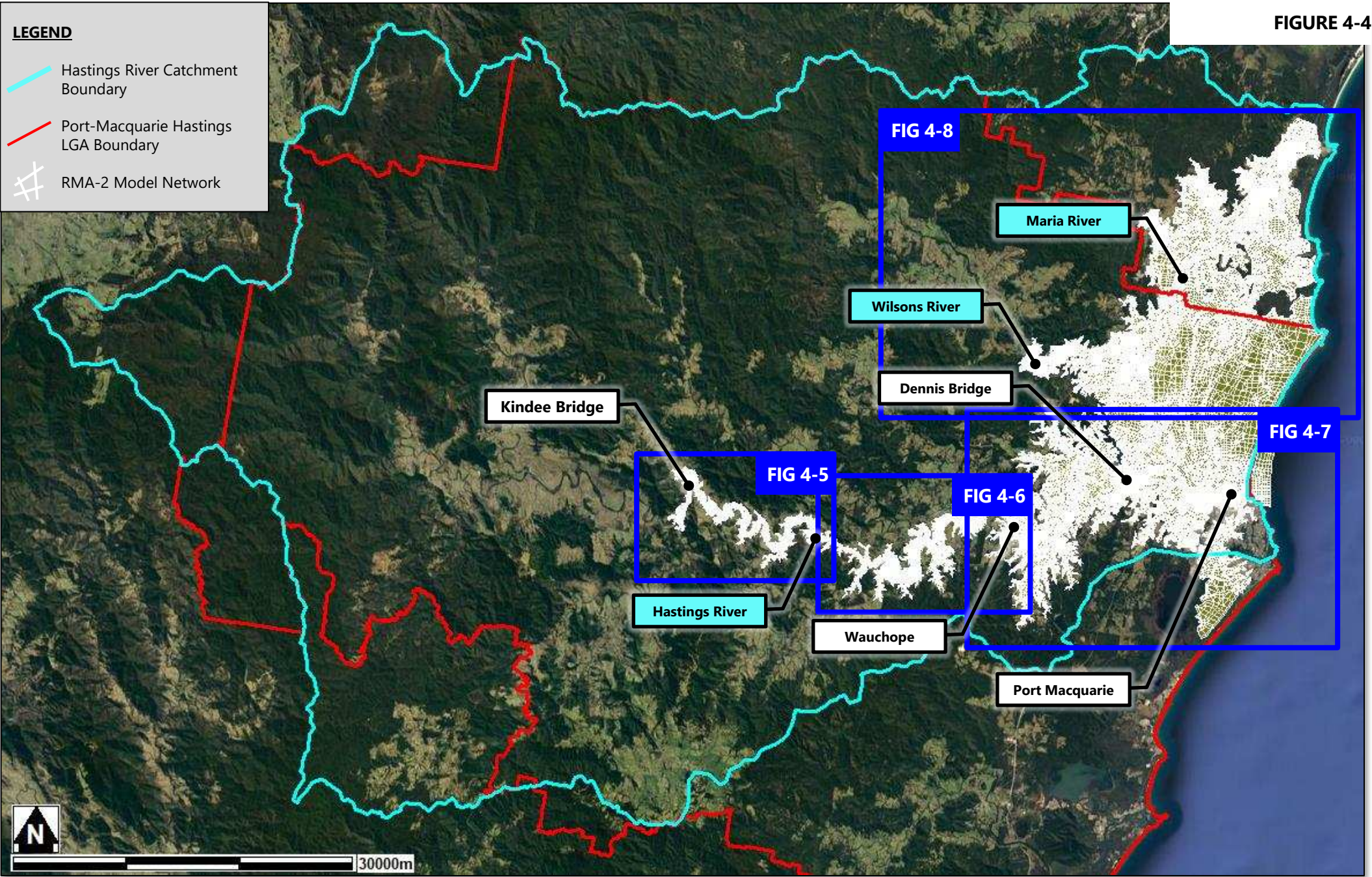


FIGURE 4-5

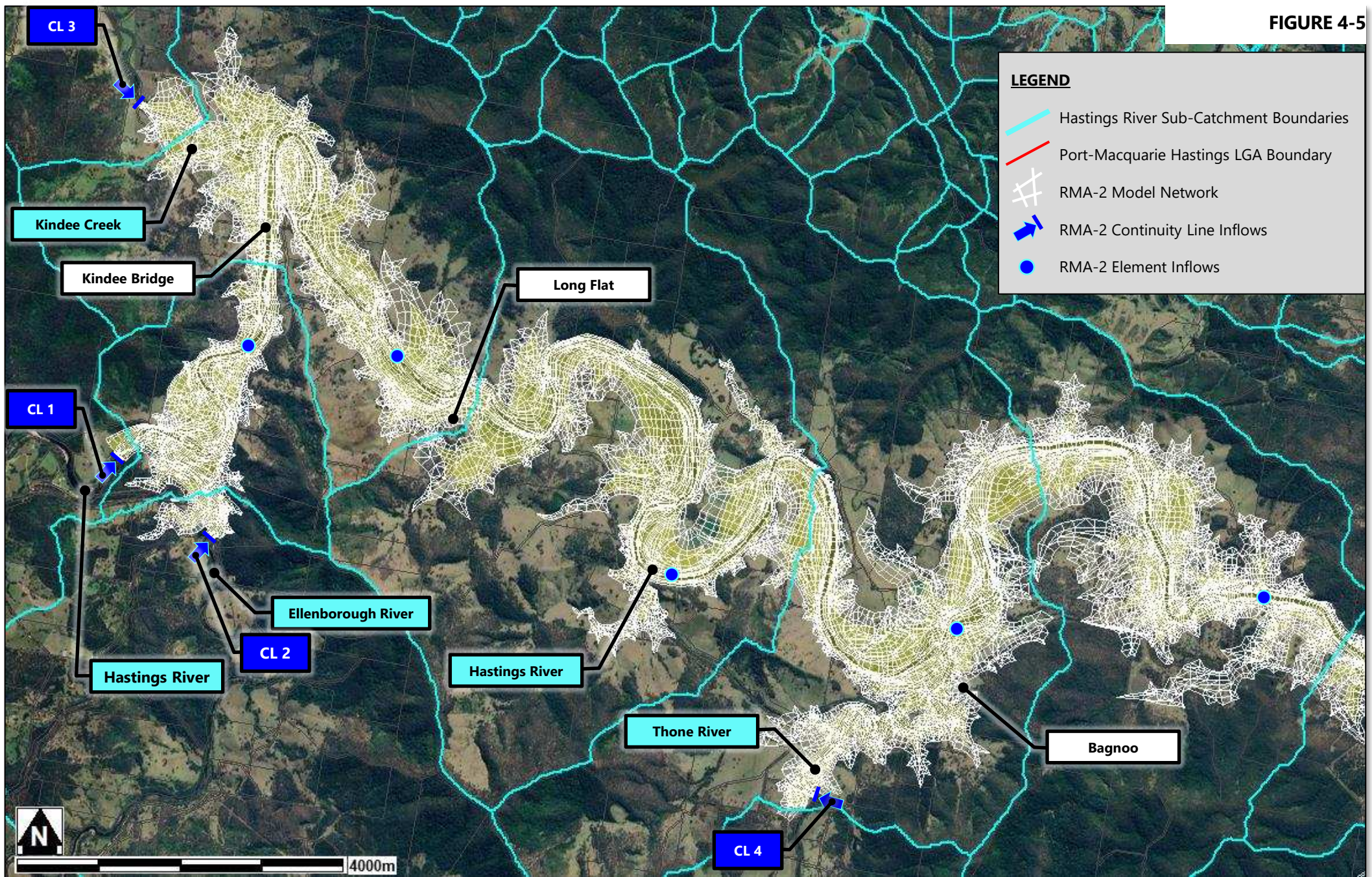


FIGURE 4-6

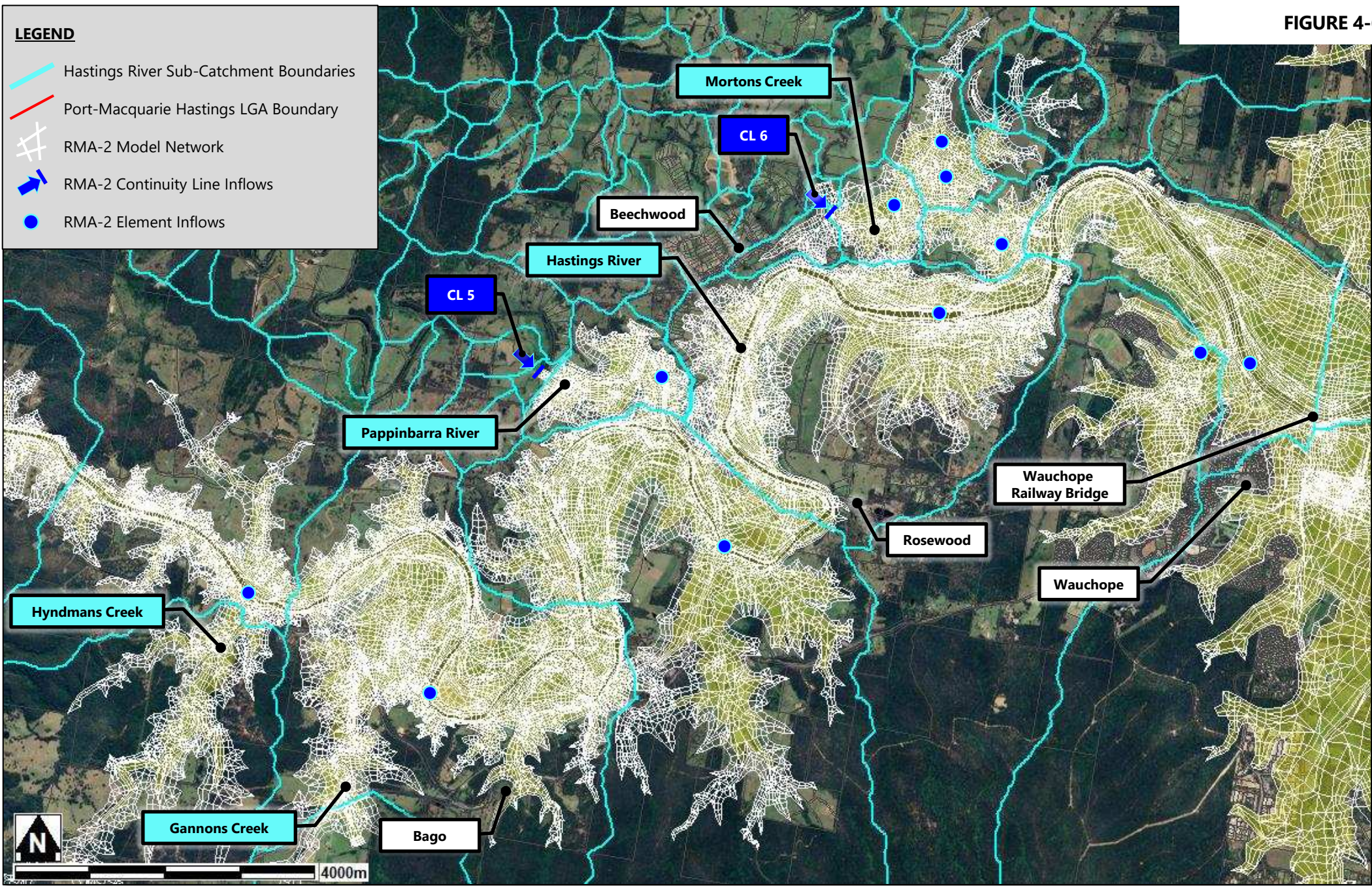


FIGURE 4-7

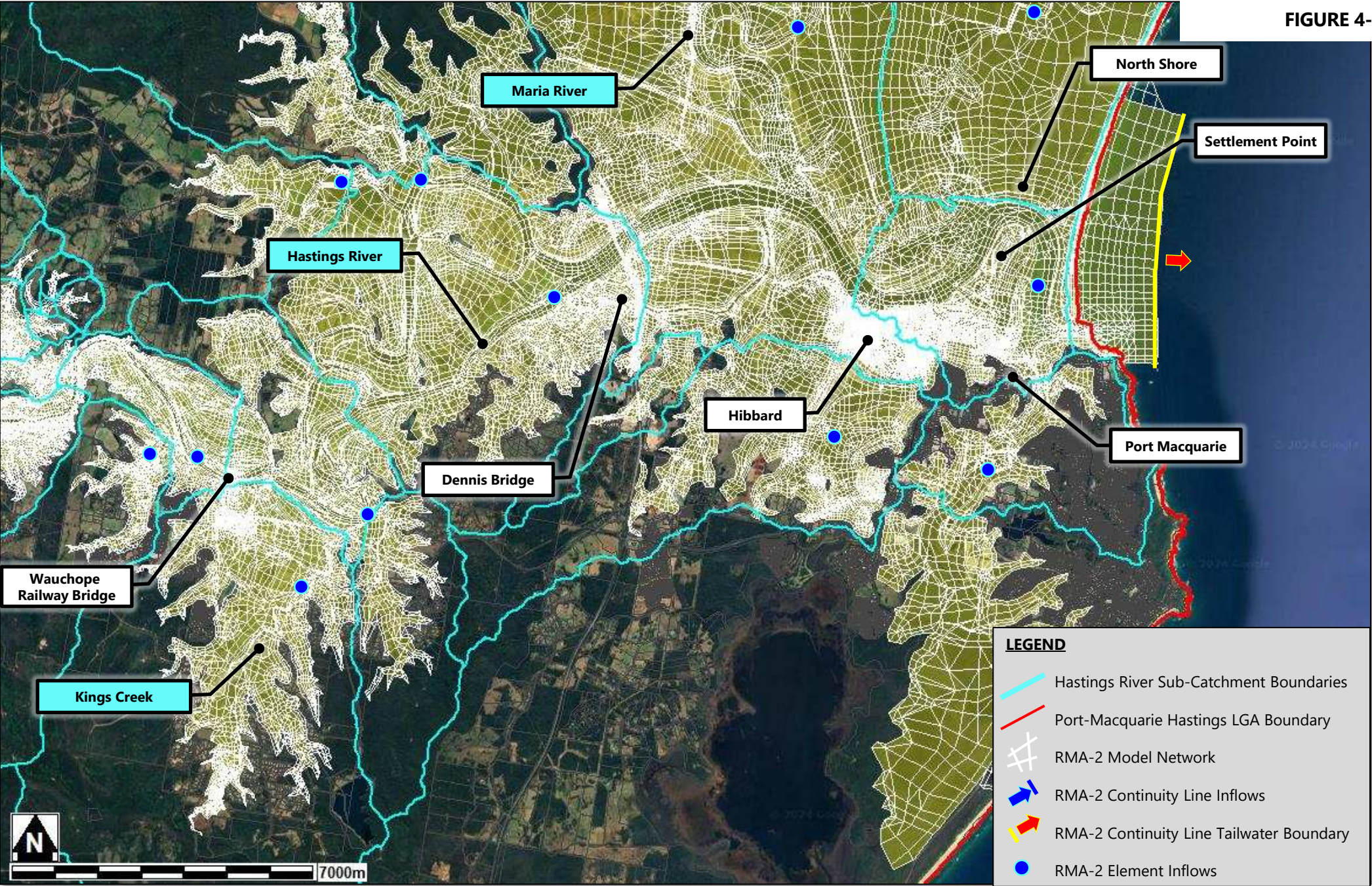
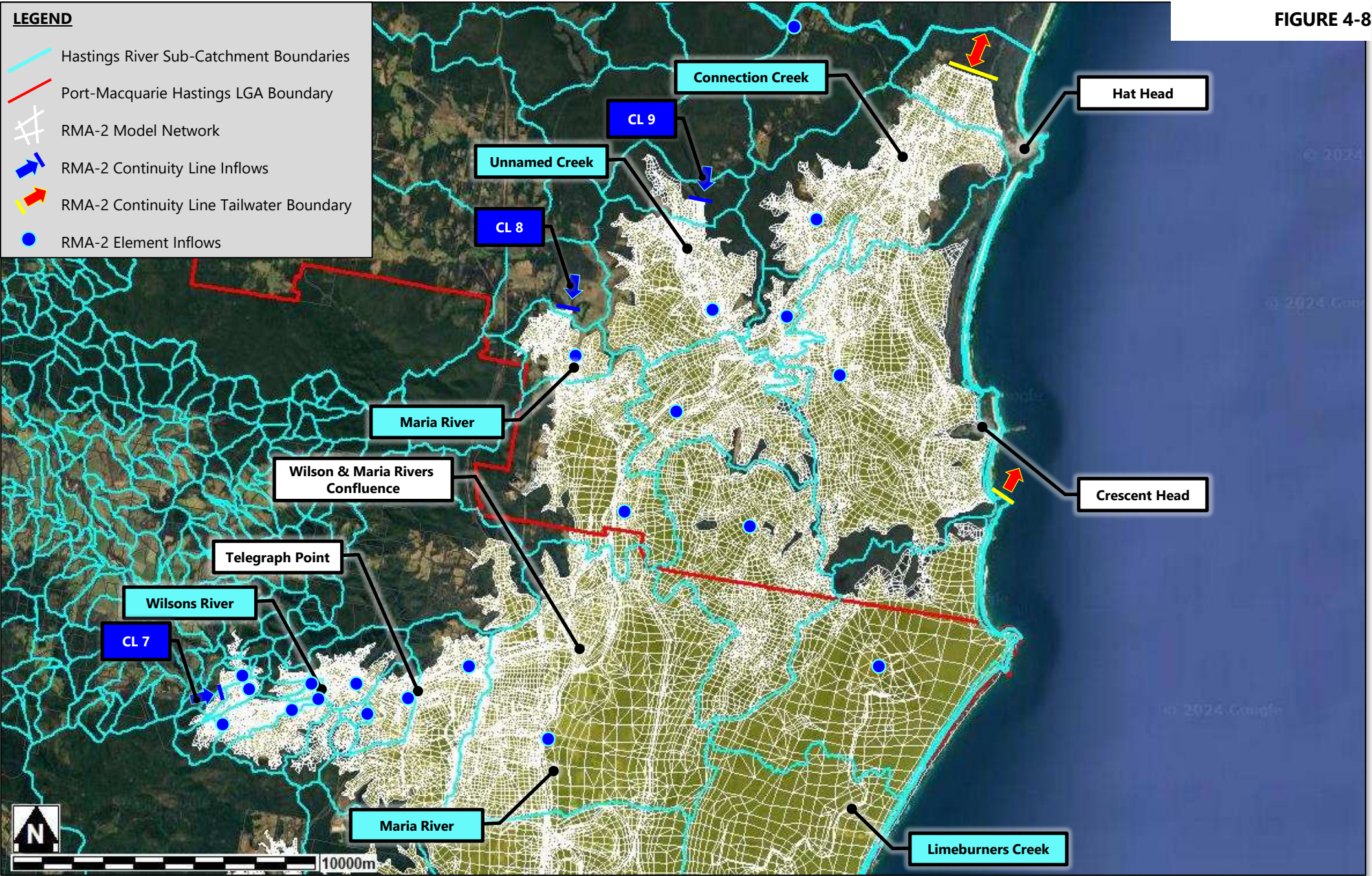


FIGURE 4-8



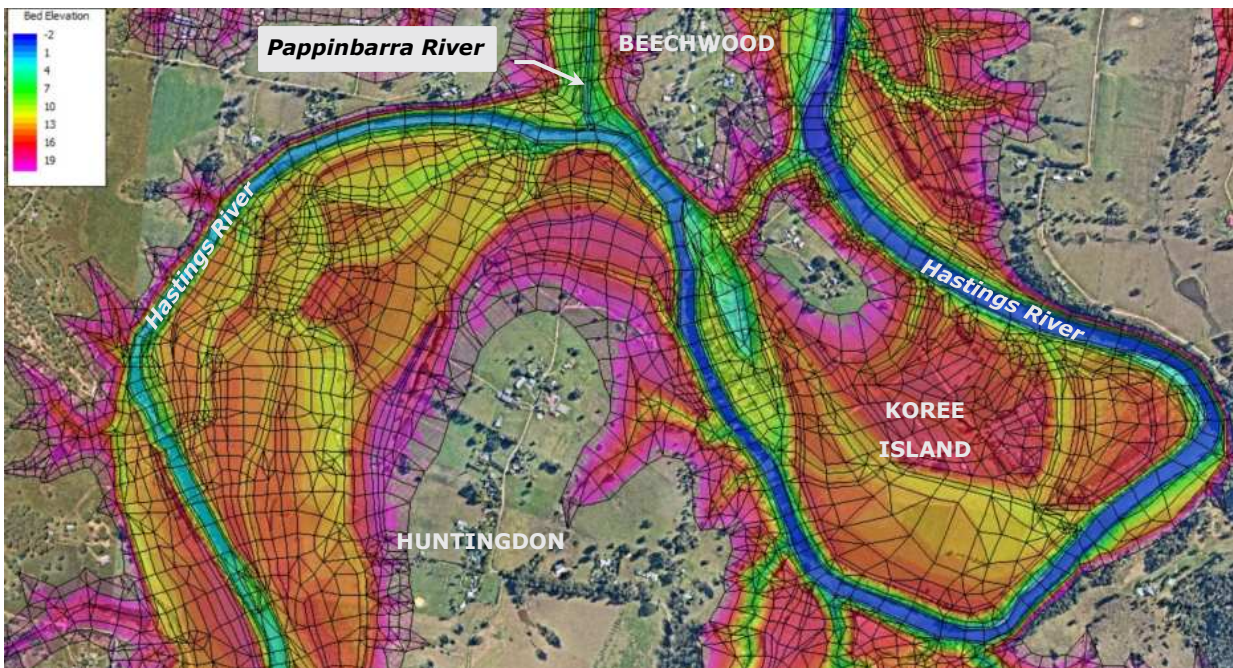


Plate 4-2 Example of RMA-2 Model Topography near Koree Island

Bathymetric survey was not located for the Hastings Rivers upstream of Morton's Creek. Bed elevations along this reach of the Hastings River were manually added based on assigning elevations that were 1.5 metres lower than the local LiDAR minima. This value was selected based following a comparison between channel cross-sectional areas and depths based on LiDAR and available survey. An example of this approach is shown in **Plate 4-2** for a cross-section along the Hastings River near Long Flat.

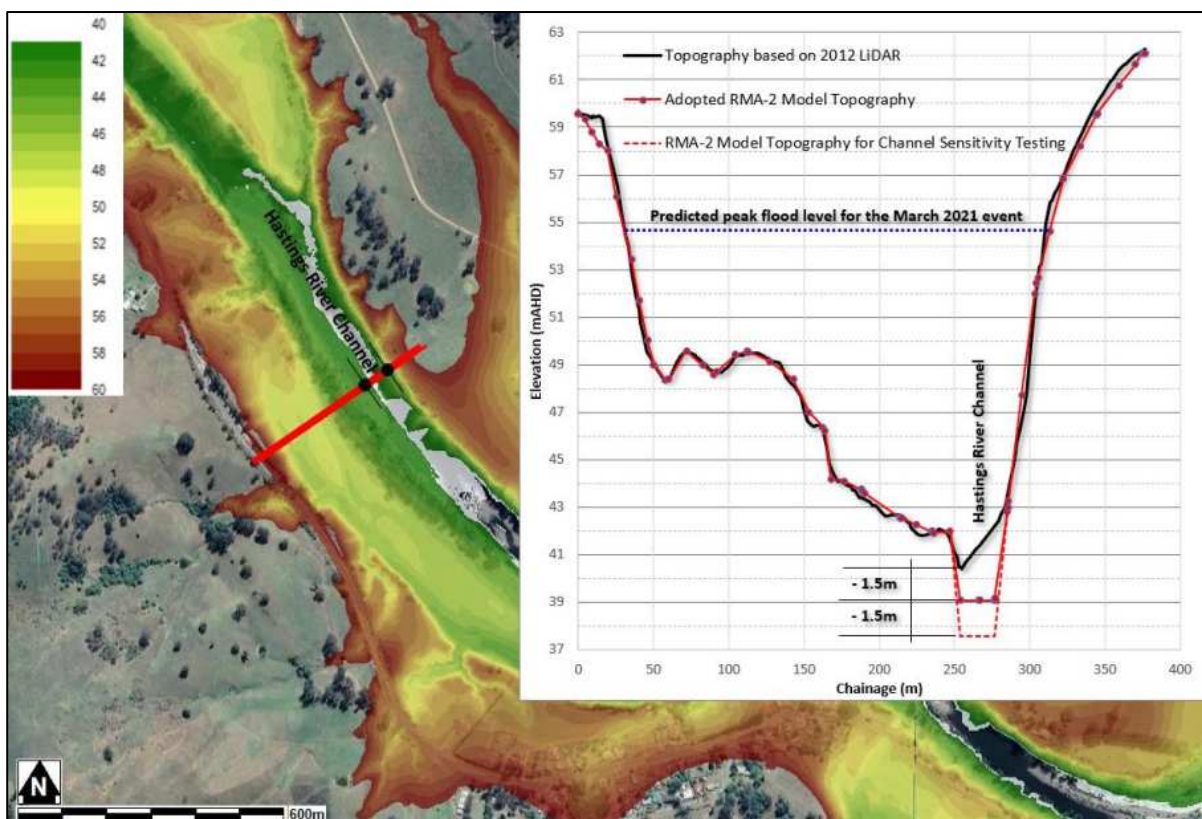


Plate 4-2 Manually Adjusted Bed Elevations Along the Hastings River – RMA-2

A sensitivity scenario was set-up to assess the impact on peak flood levels should bed elevations along the Hastings River upstream of Morton's Creek be manually lowered by a further 1.5 metres; i.e., 3 metres in total below the local LiDAR minima. The manual change was incorporated at all locations upstream of Morton's Creek including along rapids where this would not be appropriate. Notwithstanding, as this was to assess model sensitivity the further analysis required to identify these locations was not undertaken.

The sensitivity scenario predicted that the increased channel capacity could result in a reduction in peak flood levels for a March 2021 sized flood by between 0.2 to 0.5 metres. These reductions in levels were isolated to the extent of the channel modifications, with flood levels downstream of the Yippin Creek confluence changing by less than 0.03 metres. The difference reduces to 0.01 metres at Rawdon Island. Accordingly, the sensitivity scenario showed that although changes to bed elevations could have localised impacts, there would be minimal change across the wider Hastings River floodplain.

Model Roughness Values and Distribution

The adopted element types and associated roughness parameter values are listed in **Table 4-2**. The element types and roughness values are consistent with those adopted for the *Hibbard Precinct Flood Study* (2019). These roughness parameters were selected following validation of the RMA-2 model to the February 2013 flood.

Table 4-2 Adopted RMA-2 Material Types and Roughness Parameters

RMA-2 ELEMENT NUMBER [^]	DESCRIPTION	ROUGHNESS PARAMETER VALUE
1	Waterway Clear	0.03
2	Waterway Overgrown	0.08
3	Bridges & Culverts	0.1
4	Grassed Floodplain	0.035
5	Light Trees / Foliage	0.055
6	Moderate Trees / Foliage	0.075
7	Dense Trees / Foliage	0.095
8	Urban Area – Open and Unobstructed	0.04
9	Urban Area – Clutter and Fences	0.06
10	Buildings – Blocked to Flow	N/A
11	Roadways and Hardstand Areas	0.015
12	MDST Flow Control Element ^{^^}	N/A

[^] The listed element numbers and roughness types are only applicable to the RMA-2 model network across the Hibbard Precinct study area. Accordingly, the element numbers and types does not include those adopted elsewhere across the remainder of the model domain.

^{^^} MDST Flow Control Elements are used at critical flow locations (such as weirs, fences, and road embankments) to reduce the potential for sub-surface flows.

An example of the material type distributions adopted in the extended RMA-2 model is shown in **Plate 4-3**. The example shows the variation in material types assigned to the RMA-2 model in the vicinity of Korea Island and the Pappinbarra River outlet.

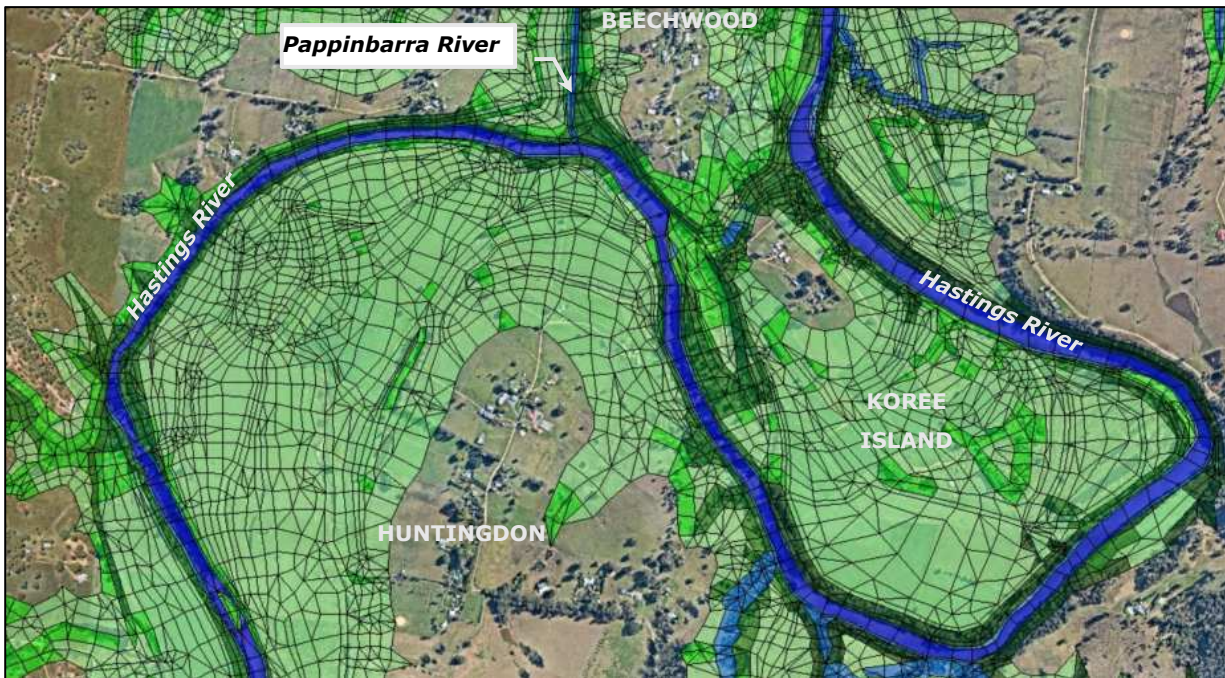


Plate 4-3 Example of RMA-2 Material Types Distributions near Koree Island

4.3.2 Upstream Boundary Conditions

The extended Hastings River RMA-2 model includes ten upstream boundary conditions that are assigned to the model as Continuity Lines. This is compared to the *Hibbard Precinct Flood Study (2019)* RMA-2 model which had three.

Each upstream boundary represents an inflow location where flow hydrographs extracted from the WBNM hydrologic model can be input. The location of each upstream boundary is shown in

Figure 4-5 to 4-8 and described below.

- Continuity Line 1 - Hastings River approximately 500 metres upstream of the confluence with the Ellenborough River.
- Continuity Line 2 - Ellenborough River approximately 1.2 river kilometres upstream of the confluence with the Hastings River.
- Continuity Line 3 - Thones River approximately 3.5 river kilometres upstream of the confluence with the Hastings River.
- Continuity Line 4 - Gannons Creek approximately 300 metres upstream of Comboyne Rd Bridge (over Gannons Creek).
- Continuity Line 5 - Kindee Creek approximately 1 kilometre west of the confluence with the Hastings River.
- Continuity Line 6 - Pappinbarra River approximately 50 metres downstream of the Pipeclay Rd bridge (over Pappinbarra River).
- Continuity Line 7 - Mortons Creek approximately 1.3 kilometres upstream of Mortons Creek Rd Bridge (over Mortons Creek).
- Continuity Line 8 - Wilson River approximately 2 kilometres upstream of Rollands Plains Rd Bridge (over Wilson River).
- Continuity Line 9 - Pipers Creek approximately 800 metres upstream of the confluence with Barrys Creek.
- Continuity Line 10 - Maria River approximately 4.5 kilometres north of the confluence with Connection Creek.

4.3.3 Downstream Boundary Conditions

The extended Hastings River RMA-2 model includes three downstream boundary conditions that are assigned to the model as Continuity Lines. Each downstream boundary represents a potential outflow that is controlled by either a stage hydrograph or time varying tailwater condition.

The location of each downstream boundary is shown in **Figure 4-5** to **4-8** and described below.

Continuity Line 11 - Macleay River at the confluence with Killick Creek and Connection Creek.

Continuity Line 12 - Port Macquarie Ocean boundary located under the road intersection of Point Plomer Rd and Melaleuca Rd.

Continuity Line 13 - Port Macquarie Ocean boundary at the Hastings River estuary near North Shore Beach.

Continuity Line 11 varies to Continuity Lines 12 and 13 based on its location on the floodplain rather than the ocean. Continuity Line 11 is used as the juncture between the Hastings River and Macleay River and allows floodwaters to flow between the two river systems, i.e., floodwaters can both escape and enter the Hastings River RMA-2 model at this continuity line. For design event modelling, Continuity Line 11 adopts time varying tailwater flood levels that are based on modelling completed for the *Macleay River Flood Study (2019)*.

Continuity Lines 12 and 13 adopt time varying ocean levels that are based on recorded data for historic events extracted from offshore tide level gauges. For design events, both continuity lines adopt tailwater conditions that are consistent with the *Hibbard Precinct Flood Study (2019)*.

Continuity Line 13 at the Hastings River estuary is incorporated into the RMA-2 model such that floodwaters can reach it and 'escape' the model by either of the following:

- Discharge to the ocean and Continuity Line 13 via the Hastings River channel, and
- Discharge to the ocean by overtopping Northwall Road and Plomer Road to the north of the Hastings River estuary.

Floodwaters escaping to Continuity Line 3 by overtopping Northwall Road and Plomer Road is only predicted to occur during extreme floods events such as the Probable Maximum Flood (PMF). Elevations along Northwall Road and Plomer Road, and along the adjacent sand dunes, were incorporated into the model based on the LiDAR data (refer **Section 3.1.1**).

4.3.4 Local Catchment Inflows

Forty-five (45) local inflows were incorporated into the model down as local element inflows. These inflows are input into the RMA-2 model through the floodplain at the locations indicated on **Figure 4-5** to **4-8**.

Previous versions of the Hastings River RMA-2 model did not include local catchment inflows. This reflected the coarser sub-catchment delineation within the XP-RAFTS model that resulted in only one sub-catchment covering the entire RMA-2 model domain.

4.4 TUFLOW Hydraulic Model

4.4.1 Software

The TUFLOW 2D/1D hydraulic modelling software package has been adopted to simulate flood hydraulics of several major tributaries of the Hastings River Basin. The TUFLOW software was determined to be a suitable tool for replicating the complex 2D nature of flooding in the area based on consideration of the following.

- Allows accurate representation of catchment topography and bathymetry to be developed in 2D from various sources (e.g. a combination of LiDAR and detailed survey).
- Allows large areas to be efficiently and reliably modelled with tributaries and flowpaths identified based on the catchment topography rather than interpretation by the modeler.
- Allows integrated investigation and interaction of overland, mainstream, tidal and ocean driven components of flooding.
- Solves the full 2D surface water equations.
- Produces high quality, GIS compatible flood mapping outputs.

The latest version of the TUFLOW hydraulic modelling software available at the time of the model calibration was adopted (2023-03-AA, released 16 March 2023). Given the significant area to be modelled, the GPU-based 'TUFLOW HPC' software was selected to maintain manageable model simulation times.

4.4.2 2D Model Domain

The 2D TUFLOW hydraulic model domain covers the entire catchments of several major tributaries of the Hastings River, the Wilson River, the Pappinbarra River and Morton's Creek (refer **Figure 4-9**). This comprises a total model area of 920 km².

The TUFLOW model domain overlaps the existing RMA-2 model in areas along the lower reaches of the Wilson River between Telegraph Point and Avenal, as well as the base of Morton's Creek to Beechwood and the Pappinbarra River to Pipeclay Road. This allowed for more reliable downstream boundary conditions to be applied from the Hastings River and Telegraph Point gauge recordings. It will also allow the TUFLOW model to be calibrated to data from the WaterNSW gauges located in the lower catchment of these tributaries, in addition to anecdotal reports of flooding.

It is expected that RMA-2 model results will be given precedence in the areas of overlap.

4.4.3 2D Model Grid Size

A model grid size of 10 metres was adopted to adequately resolve flood characteristics in the study area while maintaining manageable model simulation times, resulting in over 9.2 million computational grid cells.

Each square grid cell contains information on ground surface elevation, hydraulic roughness and other parameters as necessary (e.g. cell blockage and energy losses to represent the hydraulic effects of bridges). The ground surface elevation is sampled at the centre, mid-sides and corners of each cell from a specified Digital Elevation Model (DEM). For a 10 m grid this results in DEM elevations being sampled at 5 m centres.

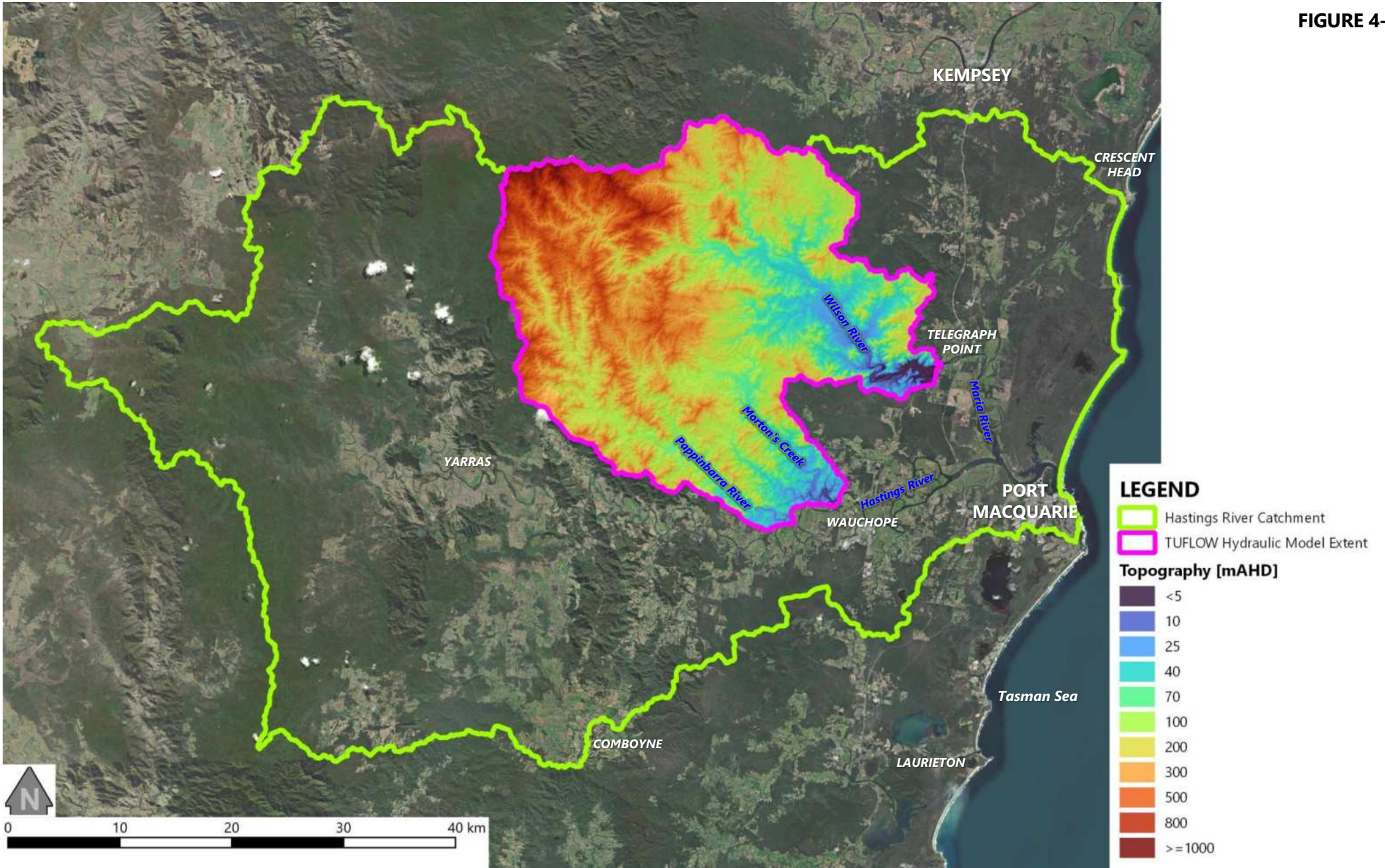
4.4.4 2D Model Terrain

The 2D TUFLOW model terrain was constructed from a combination of the latest LiDAR DEM and available bathymetric survey data as described in **Section 3.1.2** of this report.

LiDAR may capture a lower resolution of ground points in heavily vegetated areas and does not penetrate water surfaces. As such, LiDAR data will not always provide a satisfactory topographic representation of watercourses for hydraulic modelling purposes.

In order to improve the 2D TUFLOW representation of rivers, creeks and major tributaries, topographic modification techniques have been applied that use cross-sectional survey data and/or local LiDAR minima to enhance channel cross-sectional area, conveyance and flow continuity. One of the following approaches was adopted depending on the channel width and availability of survey data:

FIGURE 4-9



- **Triangulation:** Where sufficient bathymetric survey data points are available 'triangulation lines' were digitised along the watercourse, creating interpolated elevation points between the survey points. The survey points and interpolated points are then triangulated to create a continuous terrain surface. A polygon region is also digitised to define the boundary of the triangulation zone, and elevations are extracted from the underlying TUFLOW terrain (*generally based on LiDAR*) along the edge of the polygon to ensure that the resulting triangulated surface is seamless with the surround TUFLOW terrain.
- **Enforced thalwegs:** Where insufficient cross-sectional survey data points are available or the width of the channel does not warrant the use of the 'triangulation' approach described above, an alternative approach has been applied that enforces a continuous thalweg along watercourses. The thalweg is digitised as a continuous line and elevations are defined at regular intervals using the lowest data point from available survey or local LiDAR minima. A width is also assigned to the thalweg (or 'GULLY') line as selected based on inspection of aerial photographs and LiDAR DEMs.

The extent over which each of these approaches was applied is indicated in **Figure 4-10**.

4.4.5 Boundary Conditions

The TUFLOW hydraulic boundary conditions consist of the following:

- Local inflow hydrographs were applied to the 2D hydraulic model domain at each hydrologic model sub-catchment using the TUFLOW 'surface area' approach. This has been coupled with the 'streamline' approach which defines more explicitly where the flows are to be applied and results in improved definition of flow paths in the upper sub-catchments.
- Downstream water level boundaries applied at the base of each tributary.

The locations of these boundary conditions are shown in **Figure 4-11**.

4.4.6 Hydraulic Roughness

Hydraulic roughness coefficients (Manning's 'n') are used to represent the resistance to flow of different surface materials. Hydraulic roughness has a major influence on flow behaviour and is one of the primary parameters that may be altered to achieve calibration of hydraulic models.

Spatial variation in hydraulic roughness is represented in TUFLOW by delineating the catchment into zones of similar hydraulic properties. The hydraulic roughness zones adopted in this study have been delineated based on aerial photography and cadastral data. Manning's 'n' values assigned to each zone were determined based on previous work in the catchment, previous experience in calibrating TUFLOW models, and with reference to values recommended in the literature (*e.g., Chow 1959*). As resistance to flow due to surface and form roughness varies with depth (*e.g., Chow 1959, ARR 2019*), variable depth-dependent hydraulic roughness values have been adopted.

Manning's 'n' roughness coefficients applied in the TUFLOW model are listed in **Table 4-3**, with the delineation of hydraulic roughness zones shown in **Figure 4-12**. Below 'Depth 1' the first Manning's 'n' value is applied, while above 'Depth 2' the second Manning's 'n' value is applied. At depths between 'Depth 1' and 'Depth 2' Manning's values are determined by linear interpolation.

FIGURE 4-10

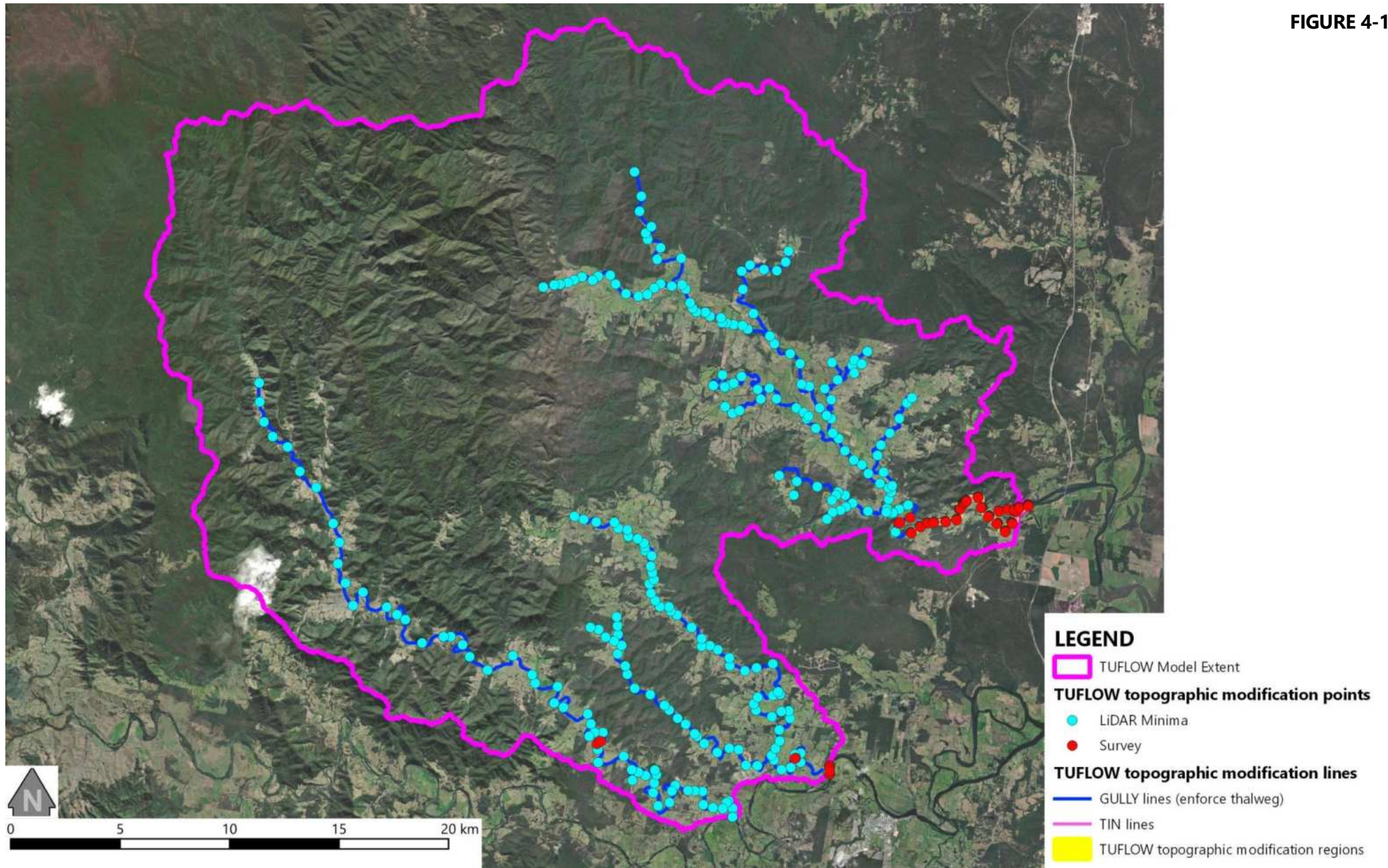


FIGURE 4-11

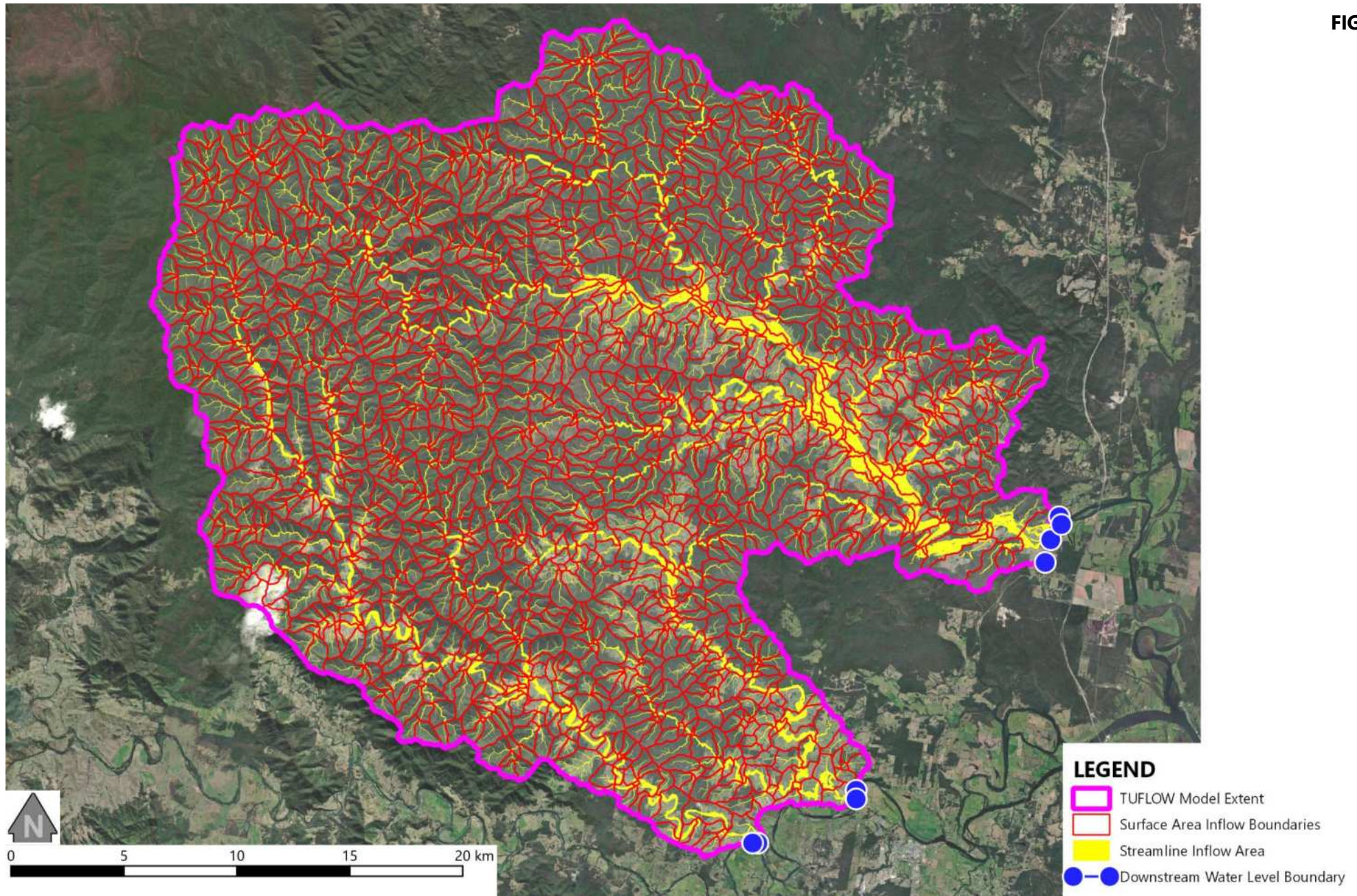


FIGURE 4-12

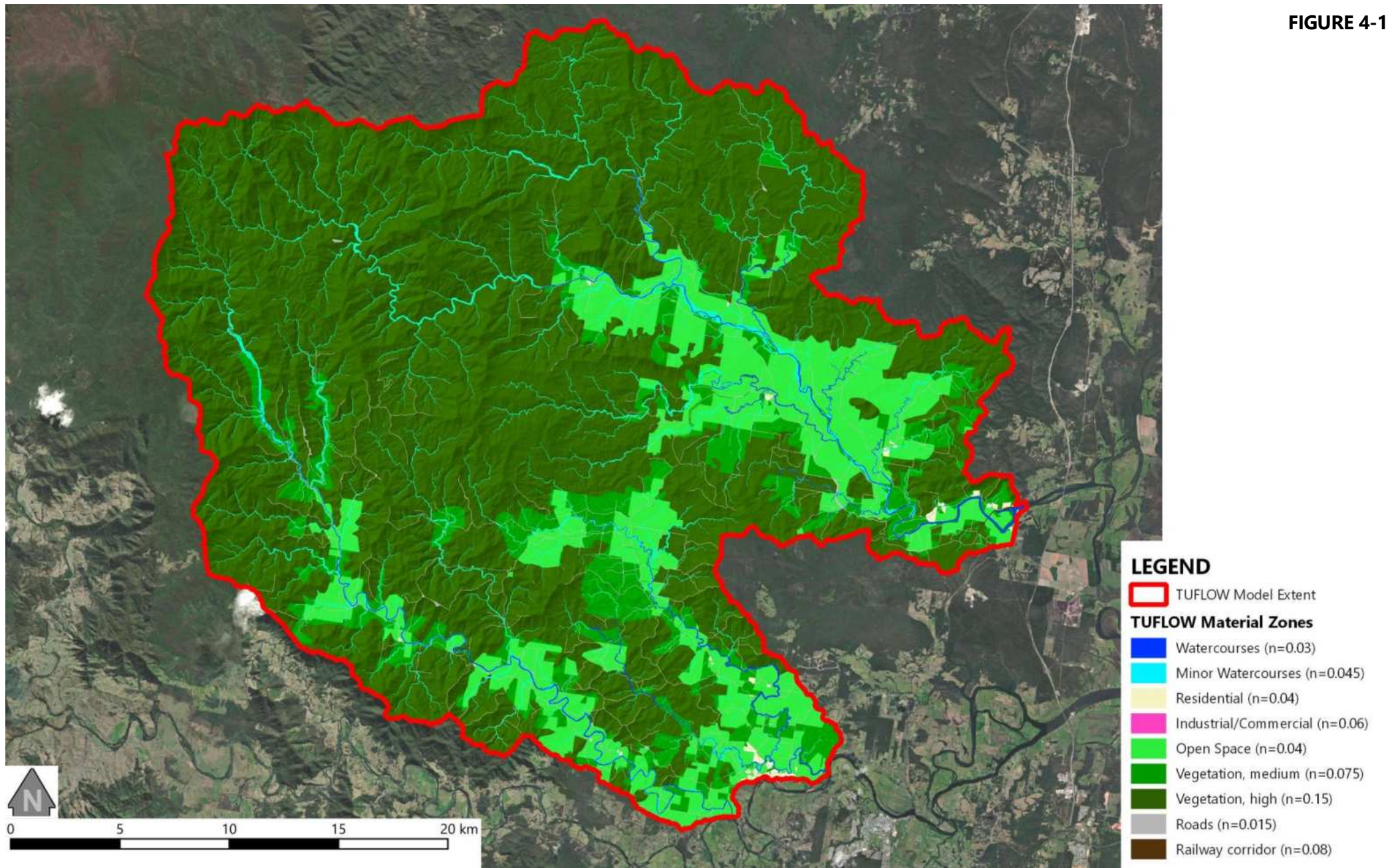


Table 4-3 TUFLOW Manning's 'n' values by depth for delineated Materials

Material	Depth 1 (m)	Manning's 1	Depth 2 (m)	Manning's 2
Watercourses	0.3	0.1	1.5	0.03
Minor Watercourses	0.05	0.06	0.15	0.045
Open Space	0.1	0.06	0.3	0.04
Medium Vegetation	0.15	0.15	0.5	0.075
Heavy Vegetation	0.3	0.2	1.0	0.15
Residential	0.3	0.08	1.5	0.04
Commercial	0.1	0.1	0.3	0.06
Roads	0.05	0.03	0.15	0.015
Railway Corridor	0.1	0.16	0.3	0.08

4.4.7 Bridges

The influence of bridges on flood behaviour has been represented in 2D using 'layered flow constrictions' which assign blockages and energy losses that simulate the hydraulic effects of bridge piers, the bridge deck and handrails.

For many of the bridges in the model domain no detailed survey or design drawings were available. Reasonable assumptions were thus made to approximate the geometry of such bridges including pier arrangement, span, deck thickness and level, and detail of handrails as follows. These assumptions were informed based on typical values found in available survey.

- *Bridge pier(s) width:* 0.7 metres
- *Bridge deck level:* estimated from the LiDAR DEM
- *Bridge deck thickness:* 0.7 metres
- *Height/blockage of railings:* estimated from Google Street View (where possible).

A total of 29 bridges were incorporated into the TUFLOW model. The locations of modelled bridges are shown in **Figure 4-13**.

4.4.8 Major Culverts

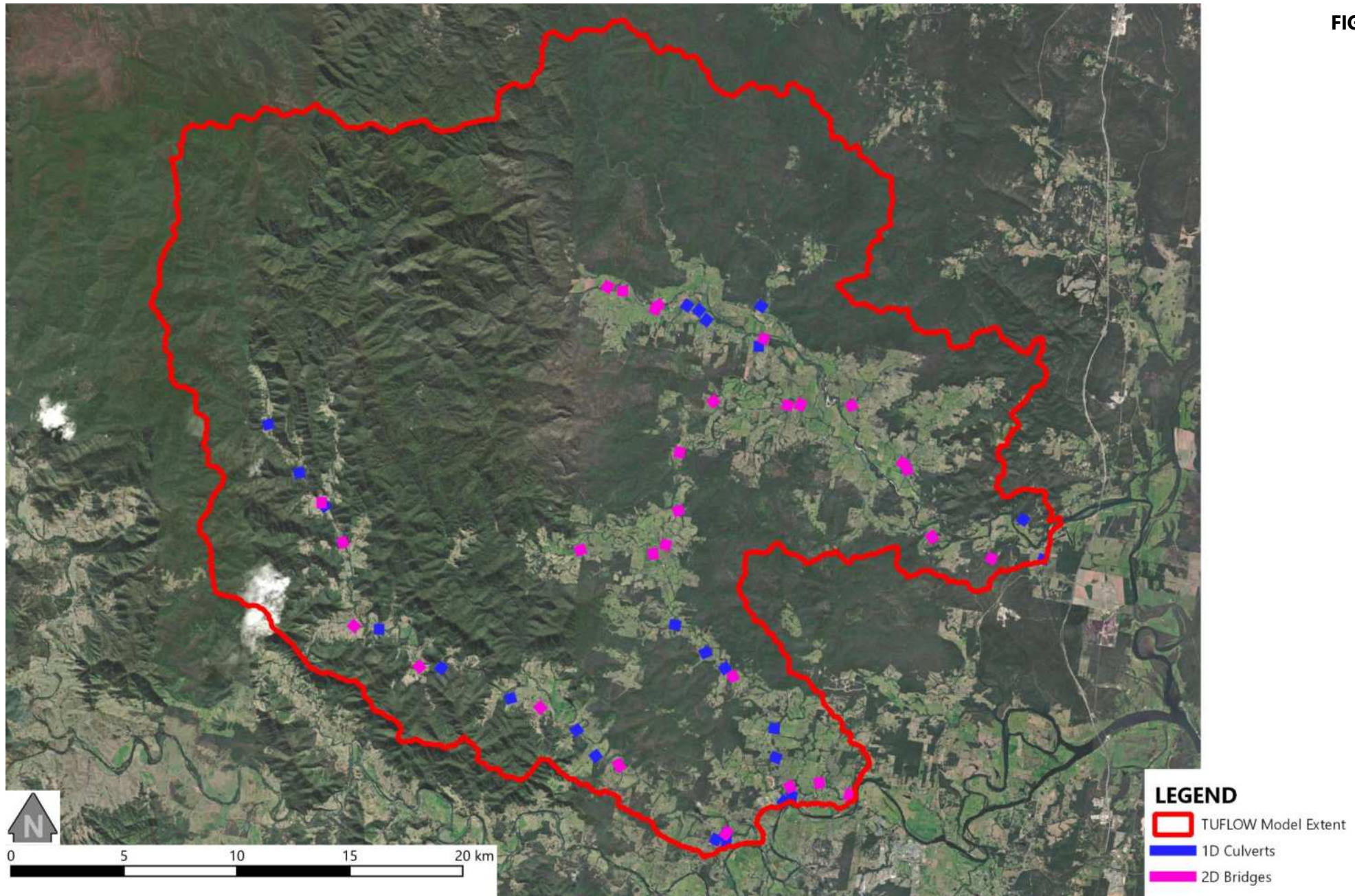
Major culverts were represented in the TUFLOW model by using 1D elements which are dynamically linked to the 2D grid surface to allow the transfer of flows.

A number of major culverts were identified in the study area from GIS layers provided by Council, and by reviewing flow paths predicted by an initial direct rainfall hydraulic model simulation. A total of 25 culverts were included in the model.

Culvert dimensions were obtained from GIS data supplied by Council. Information regarding invert levels was not provided. Invert levels were thus estimated by interrogating the minimum LiDAR ground elevations in the vicinity of the culvert inlets and outlets.

The locations of culverts and stormwater pipes included in the TUFLOW model is shown in **Figure 4-13**.

FIGURE 4-13



5 Calibration and Validation

5.1 Overview

Calibration and validation of any hydrologic and hydraulic models is an important step in the model development process. If an acceptable calibration of the model to recorded events can be achieved, it ensures the reliability of the results of design flood simulations such as the 1% Annual Exceedance Probability (AEP) flood. The approach in the current study was to undertake model calibration and validation to recorded data from flood events which occurred in February 2013, March 2021 and May 2025.

The results provide confidence in the ability of the developed hydrologic and hydraulic models to realistically simulate observed flood behaviour across the study area.

The process of calibration may also identify areas of uncertainty and/or limitations in available data. These factors can limit the potential success of model calibration.

5.2 Selection of Model Calibration and Validation Events

The suitability of historical flood events for use in model calibration and validation is generally dependent on the availability, completeness and quality of recorded rainfall, flood level and stream flow data. It is also preferable to use a number of events of variable flood size including at least one major flood.

The flood events in March 2021, February 2013 and May 2025 were selected for calibration and validation, respectively. Both events have considerable flood data, in comparison to previous flood events calibrated to in previous studies. These events were also identified as being significant to the local community.

5.3 Calibration – March 2021 Flood

5.3.1 Event Overview

The Bureau of Meteorology (BOM) provided an overview of the event in a Special Climate Statement titled, *'Special Climate Statement 74 – extreme rainfall and flooding in eastern and central Australia in March 2021'*.

Significant rainfall commenced along parts of the New South Wales coast on 17th and 18th March. The heaviest rain occurred along the Mid North Coast on 19th March, but significant falls covered much of the coast from the Illawarra northwards (BOM 2021).

One of the most significant aspects of this event in coastal New South Wales was its persistence, which resulted in many very high multi-day rainfall totals. Rainfall totals for the week to 23rd March exceeded 400 mm along a vast stretch of the NSW coast (refer **Plate 5.1**). A number of sites on the Mid North Coast had four (4) consecutive days with 100 mm or more from 19th to 22nd March. Comboyne, in the hills south-west of Port Macquarie, had three (3) consecutive days with 200 mm or more from 19th to 21st March. Comboyne also had a four-day total of 853 mm from 19th to 22nd March (*a record for this location*), and a combined total of 943 mm for the week ending 24th March (BOM 2021).

The highest daily rainfall for the Mid North Coast region occurred over the 24 hours leading up to 9am on 20th March, with the heaviest rainfall centred between Port Macquarie and Taree where over 200 mm was recorded across a significant area (refer **Plate 5.2**). The highest daily total of 405.5 mm was recorded in the Camden Haven River catchment at a flood warning gauge at Kendall (Delward).

The Hastings River catchment had its second-wettest period on record for timescales from 3 to 7 days ranking only behind the February 1929 event (BOM 2021).

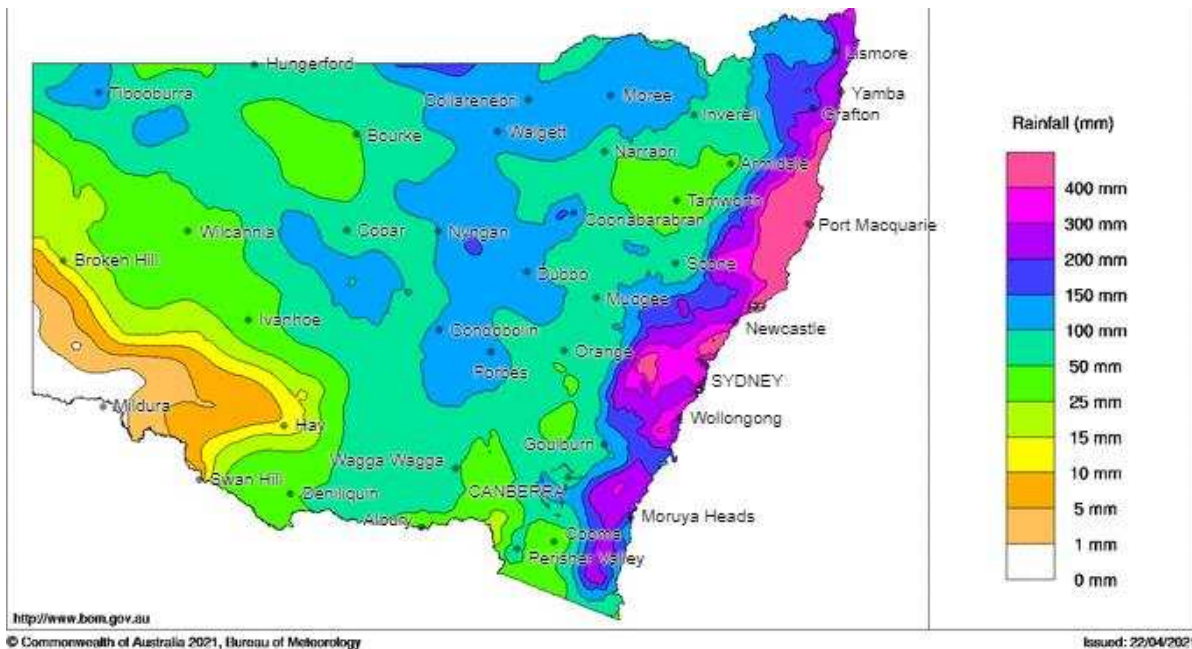


Plate 5.1 NSW rainfall totals for the week ending 24th March 2021 (Source: BOM)

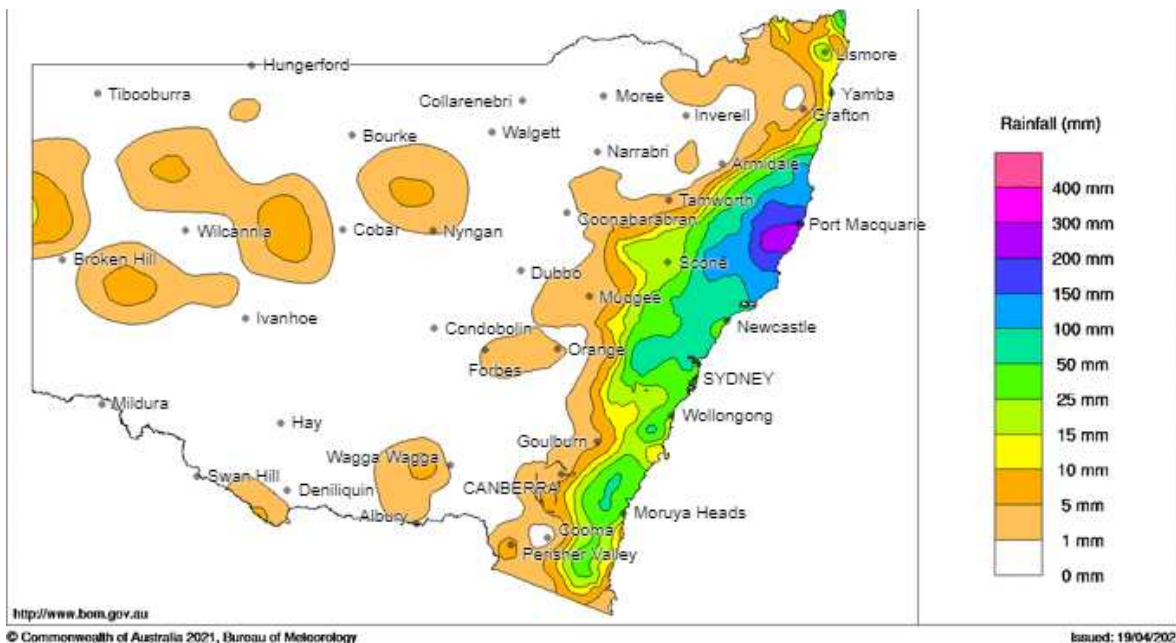


Plate 5.2 NSW rainfall totals for the 24-hour period to 9am 20th March 2021 (Source: BOM)

The heavy rainfall, which mostly fell on relatively wet catchments, contributed to significant and widespread flooding. The most significant flooding occurred in the Hastings, Camden Haven, and Manning Rivers. Record flood heights were observed at Kindee Bridge on the Hastings River and Logans Crossing on the Camden Haven River (BOM 2021).

5.3.2 River Level Data

A number of river level gauges across the catchment were in operation during the March 2021 event. These were spread throughout the catchment, on the Hastings River and a number of major tributaries. River level data recorded during the March 2021 flood is presented in **Plate 5-3**.

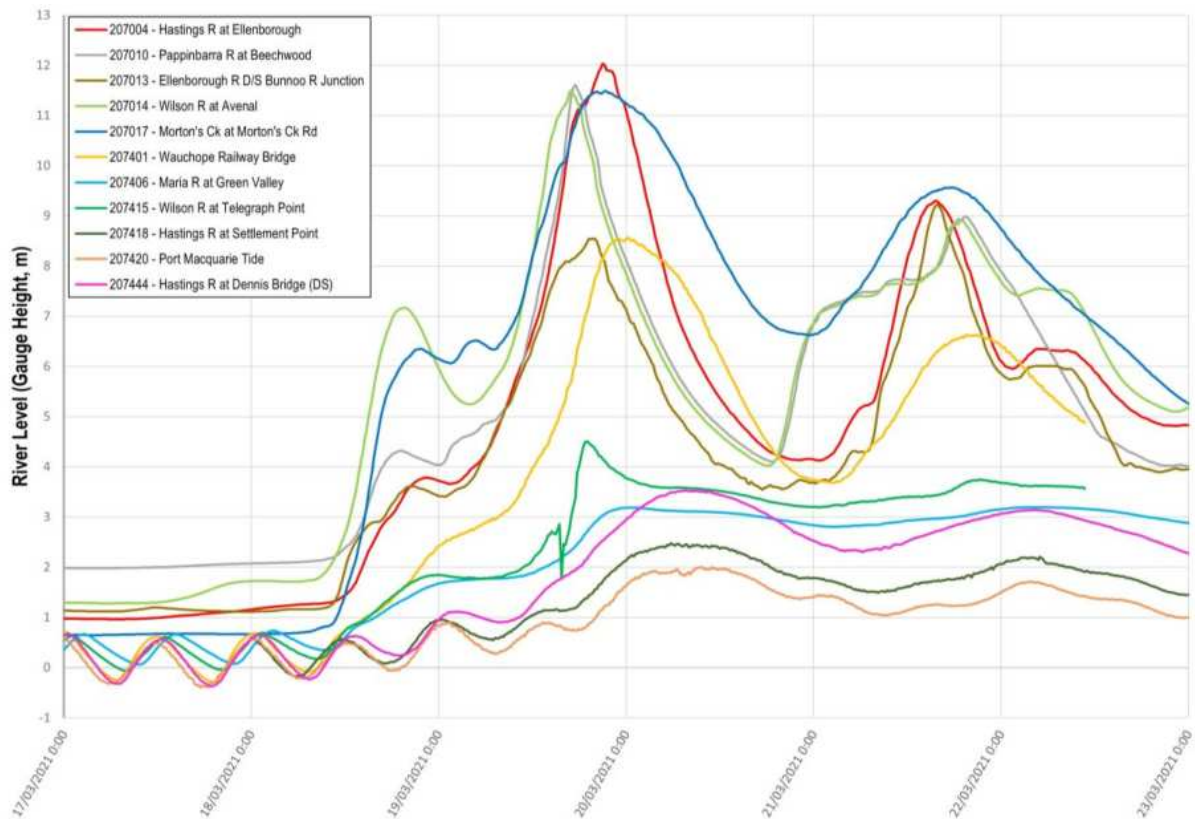


Plate 5-3 Recorded River level data for the Hastings River and Major Tributaries – March 2021

5.3.3 Hastings River WBNM Model Calibration

The WBNM software determines rainfall depths across each model sub-catchment from input rainfall gauges using an inverse-distance-weighting algorithm, with the temporal pattern across each sub-catchment taken from the nearest input rainfall gauge.

In general, there was high spatial variation in rainfall across the catchment, in terms of timing and volume, and a high percentage of daily read rainfall gauges, particularly in the western half of the catchment.

Initial calibration simulations found that use of rainfall data from the few available pluviometers alone did not achieve an appropriate representation of the spatial and temporal variation of rainfall across the study area in the March 2021 storm. To improve this, additional gauges were added, including additional 24-hour rainfall totals. These were applied to the WBNM hydrologic model by applying temporal patterns derived from nearby pluviometer(s). The location of these gauges varied from within to outside of the catchment boundary. To further improve the calibration, multiple temporal patterns were tested at these daily-read rainfall gauges to try obtain a representative rainfall.

The final adopted rainfall data for the March 2021 Hastings River calibration simulation is summarised below:

- 60052 Upper Rollands Plains – *Pluviometer*
- 560011 Telegraph Point (Wilson River) – *Pluviometer*
- 207406 Green Valley (Maria River) – *Pluviometer*
- 560016 Long Flat (Hastings River) – *Pluviometer*
- 60139 Port Macquarie Airport AWS – *Pluviometer*
- 560020 Korie Island (Hastings River) – *Pluviometer*

- 60067 Upper Pappinbarra (Hillcrest) – *Daily Read*
- 60085 Yarras (Mt Seaview) – *Daily Read*
- 60121 Elands Black Sands Creek – *Daily Read*
- 60161 Comboyne Public School – *Daily Read*
- 60163 Yarras Mountain – *Daily Read*
- 60164 Yarrowitch – *Daily Read*

A cumulative rainfall plot of the adopted rainfall data for the period from 00:00 on 18th March 2021 to 00:00 on 23rd March 2021 is presented in **Plate 5-4**, and the locations of these gauges are shown in **Figure 3-3**.

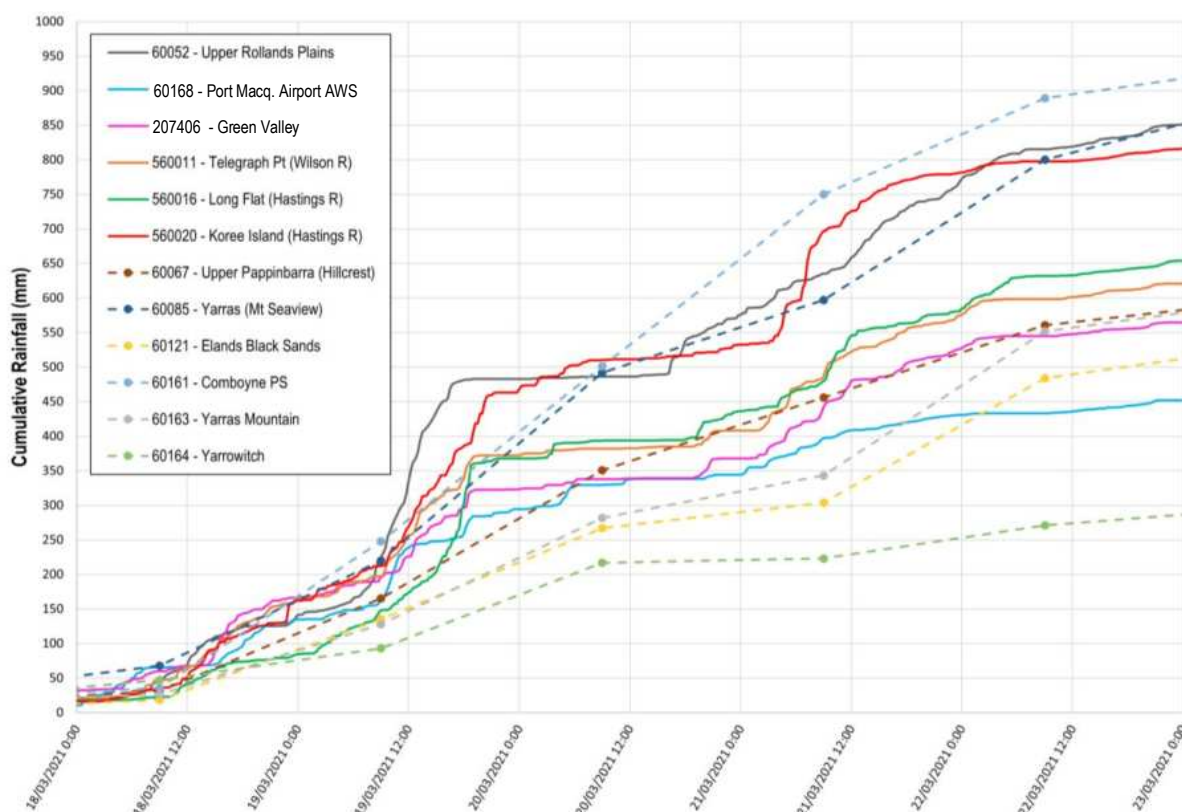


Plate 5-4 Cumulative Rainfall Plot for the March 2021 Event

The calibration of the WBNM hydrologic model was completed by comparing flows against recorded flow rate data obtained from river gauges across the catchment. This was undertaken by comparing WBNM total runoff hydrographs. Where differences arose, adjustment of WBNM hydrologic model parameters was done to try match the hydrologic model to recorded levels.

This involved adjustment of the WBNM runoff lag factor 'C' and stream routing factor 'F', as well as initial loss (IL) and continuing loss (CL) rates. The parameters used for the calibrated WBNM model is presented in **Table 5-1**.

In order to select an appropriate value for the WBNM runoff lag parameter 'C' a range of values from 1.3 to 1.8 were simulated. This was undertaken for both the March 2021 and February 2013 flood events and a 'C' value of 1.6 was determined to provide the best fit to recorded data.

Table 5-1 WBNM parameters for March 2021 calibration event

Parameter	Parameter Value
Runoff lag factor 'C'	1.6
Impervious runoff lag factor 'C'	0.1
Stream routing factor 'F'	Variable (1.0 to 1.3)
Initial Loss (pervious)	20
Continuing Loss (pervious)	5.0
Initial Loss (effective impervious)	0
Continuing Loss (effective impervious)	0

With the runoff parameter 'C' determined, refinement of the WBNM stream lag parameter 'F' was then undertaken to achieve improved matches between WBNM hydrographs and recorded data. A value of 1.0 was used for almost all catchments, where only 2 sub-catchments underwent alteration. These catchments were identified as being flatter and meandering more so than surrounding sub-catchments and given a value of 1.2 and 1.3, respectively.

A few issues arose throughout the hydrologic model calibration process, where the WBNM model couldn't generate hydrographs that fit both the shape and magnitude of those recorded at various gauges. These issues were largely attributed to the lack of continuous rainfall gauges in the catchment leading to a high level of uncertainty in the temporal distribution of rainfall on which the rainfall totals for daily gauges can be apportioned. This was particularly problematic for catchments upstream, or to the west of, Mortons Creek (refer **Figure 3-4**) including the Pappinbarra River, upper Hastings River and Ellenborough River catchments.

To confirm that the calibration difficulties were not limited to the new WBNM model, the same recorded rainfall data was simulated through the 2006 XP-RAFTS model. Flows predicted using the 2006 XP-RAFTS were found compare well to the WBNM model indicating that the issues are likely attributed to the uncertainty in the rainfall data used for flow estimation.

A further check was completed by comparing the timing, shape and peak of flow hydrographs generated by the WBNM model to those predicted by TUFLOW at gauge locations. This check showed that the runoff-routing in WBNM was comparable to that in TUFLOW further supporting the performance of the WBNM model.

Graphs comparing the final flow hydrographs generated for the March 2021 event using the WBNM model and those recorded at various streamflow gauges are included as **Appendix A**. Comparisons are included for the following streamflow gauges.

- **Figure A-1** for the Ellenborough River at Downstream Bunnoo River Junction (207013)
- **Figure A-2** for the Forbes River at Birdwood (Filly Flat) (207006)
- **Figure A-3** for the Hastings River at Mount Seaview (207015)
- **Figure A-4** for the Hastings River at Ellenborough (Kindee Bridge) (207004)
- **Figure A-5** for the Pappinbarra River at Beechwood (207010)
Note: Comparison was not complete due to the gauge rating curve not translating recorded levels to flows above a gauge height of 8.3 metres.
- **Figure A-6** for the Thone River at Deep Creek Road (207018)
- **Figure A-7** for the Wilson River at Avenal (207014)
Note: Comparison was not complete due to the gauge rating curve not translating recorded levels to flows above a gauge height of 8.0 metres.

It is important to note that the comparisons to the Pappinbarra River at Beechwood (207010) and Wilson River at Avenal (207014) were incomplete due to limitations in the rating curves adopted for the gauges used to convert recorded levels to flows. As shown in **Plates 5-5** and **5-6**, the rating curves for the gauges are derived to maximum gauge heights of approximately 8.3 metres and 8.0 metres, respectively. This translates to peak flood levels of approximately 50.76 mAHD and 18.4 mAHD, respectively. As shown in **Plate 5-3**, recordings at both gauges reached at least 3.3 metres above the maximum rating curve height.

WaterNSW

HYGPLOT V173 Output 05/07/2024

207010 PAPPINBARRA RIVER AT BEECHWOOD

Gaugings from 28/02/1985 to 13/06/2024

Rating Table 133.03 133.03 06/07/2022 to Present

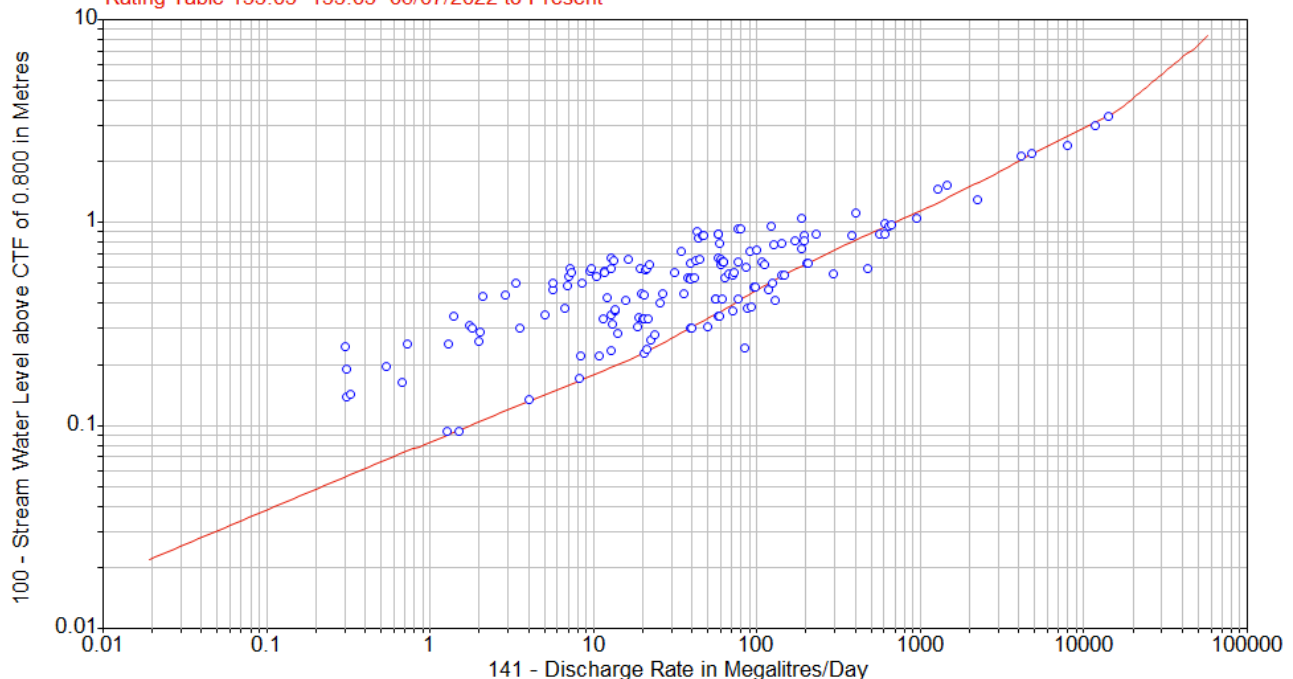


Plate 5-5 WaterNSW Rating Curve for the Pappinbarra River at Beechwood Gauge (207010)

Source: <https://realtime.data.watersnsw.com.au/water.stm>

5.3.4 Hastings River RMA-2 Hydraulic Model Calibration

The extended RMA-2 model was calibrated using the inflow hydrographs extracted from the WBNM model at all upstream and local catchment inflow locations (*refer Sections 4.3.1 and 4.3.2*). A comparison of predicted flood level hydrographs to recorded levels at gauges located within the RMA-2 model domain are included in **Appendix B**:

- **Figure B-1** for the Hastings River at Ellenborough (Kindee Bridge) Gauge (207004)
- **Figure B-2** for the Hastings River at Wauchope Railway Bridge Gauge (60124)
- **Figure B-3** for the Hastings River at Dennis Bridge Gauge (207444)
- **Figure B-4** for the Hastings River at Settlement Point Gauge (207418)
- **Figure B-5** for the Hastings River at Port Macquarie Gauge (207420)
- **Figure B-6** for the Wilson River at Avenal Gauge (207414)
- **Figure B-7** for the Wilson River at Telegraph Point Gauge (207415), and
- **Figure B-8** for the Maria River at Green Valley (207406)

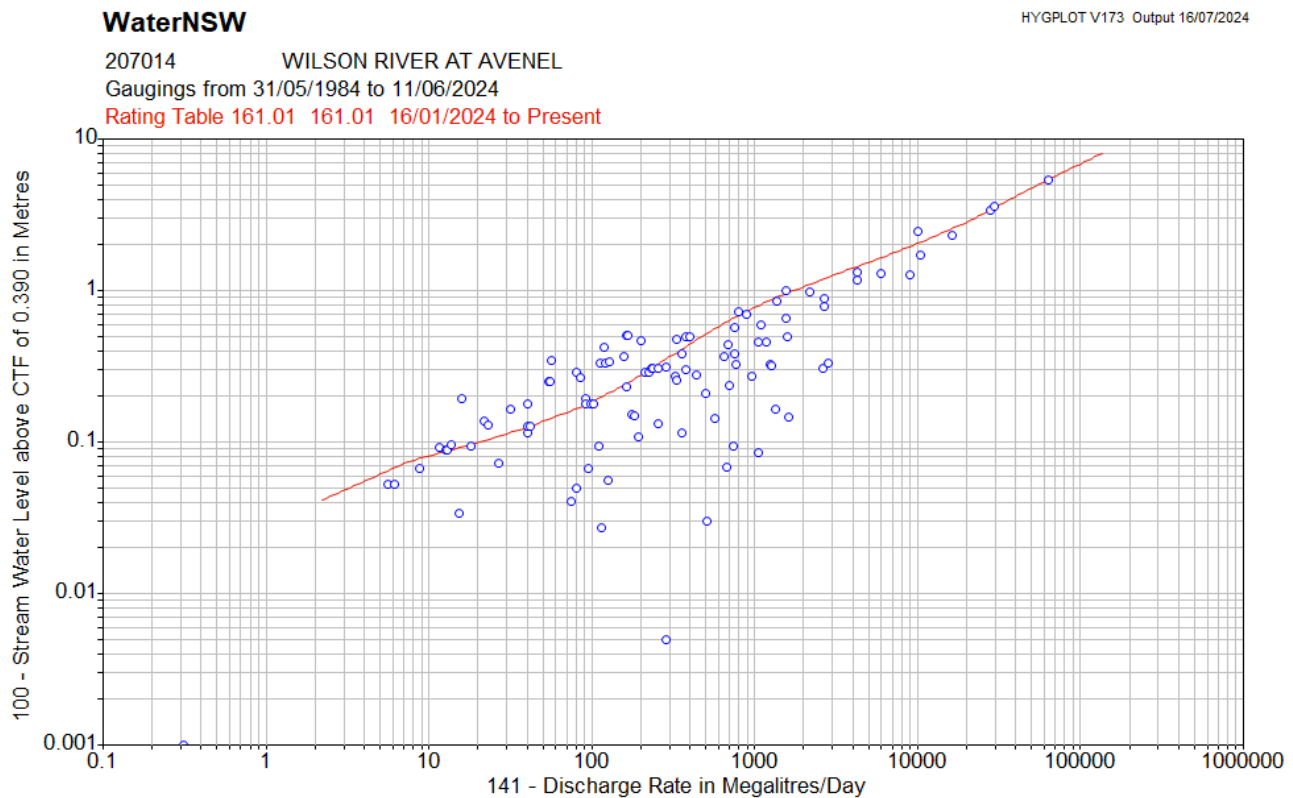


Plate 5-6 WaterNSW Rating Curve for the Wilson River at Avenal Gauge (207014)

Source: <https://realtime.data.watarnsw.com.au/water.stm>

Flood levels predicted by the RMA-2 model for the March 2021 event are considered to match reasonably well to the recorded data at Kindee Bridge, Wauchope Railway Bridge, Dennis Bridge and Settlement Point along the Hastings River, and at Green Valley along the Maria River. In particular, the shape of the flood level hydrographs are well replicated and differences in peak levels were generally within 0.2 to 0.4 metres, or better.

The flood level comparison at Kindee Bridge suggests that the RMA-2 model is overpredicting flood levels locally by up to 0.5 metres (refer **Figure B-1**). Review of **Figure A-4** indicates that the WBNM model predicts flows that are almost 15% or 600 m³/s higher than those recorded at the Kindee Bridge gauge. With flows in the RMA-2 model based on WBNM, it follows that the increased flows could be the cause for all, or a portion, of the higher flood level. Accordingly, the higher flood levels may not be reflective of any issue with the RMA-2 model itself.

The flood level comparison at the Telegraph Point gauge along the Wilson River suggests that the RMA-2 model is underpredicting flood levels locally by roughly 0.6 metres (refer **Figure B-7**). The HWM comparison included in **Figure 5-7** shows differences between recorded and predicted flood levels that are slightly lower and in the range of 0.12 to 0.75 metres, based on comparison to five HWMs. It is possible the discrepancy between predicted and recorded flood levels is attributed to either of, or a combination of the following:

- Blockages of culverts along Telegraph Point Road and/or the Pacific Highway that led to a reduction in conveyance capacity resulting in higher flood levels upstream of the Pacific Highway during the event.
- RMA-2 inflows generated by the WBNM model are underpredicted leading to lower flood levels. As discussed in **Section 5.3.3** and shown in **Plate 5-6**, the WaterNSW rating curve for the Wilson River at Avenal Gauge (207014) (refer **Figure A-7**) does not cover gauge heights above 8.0 metres. This height is over 3 metres below the peak flood height reached at the gauge and thus conditions near the peak were not translated to a flow estimate. Accordingly, the flows predicted by the WBNM model could not be validated at the gauge.

Figure 5-6 shows that the RMA-2 model is in good agreement with the recorded HWM data for the floodplain between Dennis Bridge and the ocean entrance. Across the area, the RMA-2 model was able to predict flood levels that were within 0.32 metres to the ten (10) available HWMs. The average difference is considerably lower at 0.18 metres.

The good calibration in these lower reaches of the study area is further shown in **Figure B-4** for the Settlement Point gauge. As shown, the RMA-2 model predicts flood levels that are in good agreement with the shape and peak flood level reached at the gauge.

The recorded HWM data indicates that the RMA-2 model could be underpredicting flood levels in the vicinity at the ocean entrance based on the comparison to recorded data at the Port Macquarie Tide gauge. As shown in **Figure B-5**, the simulated flood levels are generally lower than those recorded, including at the peak by up to 0.4 metres. It is noted that the issue could be localised with the results showing a steep floodwater gradient 50-100 metres upstream of the gauge. As shown in **Figure B-5**, flood levels extracted 100m upstream of the gauge show a much better fit to the recorded data with peak flood levels matching to within 0.10 metres. This is therefore likely to be a localised discrepancy that will have minimal impact to flood levels predicted further upstream.

A comparison of peak March 2021 flood levels predicted using the extended RMA-2 flood model against available flood marks is presented in **Figure 5-1 to 5-9**. **Table 5-2** presents a summary of the HWM calibration and shows that the RMA-2 model resulted in a good validation to the March 2021 event. This is reflected by the absolute average and median difference of 0.25m and 0.16m, respectively.

5.3.5 Major Tributaries Hydraulic TUFLOW Model Calibration

To calibrate the TUFLOW hydraulic model to the March 2021 event, the inflow hydrographs from the WBNM model were applied across the catchment. Relevant downstream boundary conditions were also applied at the base of all three tributaries. For the Wilson River, the model extends to MHL's Telegraph Point (207415) gauge. The recorded levels throughout the event were applied at this boundary.

For Morton's Creek, the downstream boundary was identified as having a significant impact on the recorded flood levels. This was problematic, as MHL's Wauchope Railway Bridge (MHL Gauge No. 207401) is the closest gauge on the Hastings River was located 4.8 km downstream and not representative of conditions at the confluence of the Hastings River and Morton's Creek during the event. Accordingly, an appropriate time and level shift was applied based on the flood surface gradient from the 2006 Hastings Flood Study.

For the Pappinbarra River, the downstream boundary conditions were less influential as the comparison gauge is located further up the catchment and influenced less by backwater flooding like the two other major tributaries.

The hydraulic model calibration was then assessed through the comparison of flood levels simulated by the TUFLOW model against available recorded water levels at WaterNSW gauges located along each of the tributaries. A comparison of predicted flood level hydrographs to recorded levels at gauges located within the TUFLOW model domain are included in **Appendix C**.

- **Figure C-1** for the Wilson River at Avenal Gauge (207014)
- **Figure C-2** for the Morton's Creek at Morton's Creek Road Gauge (207017)
- **Figure C-3** for the Pappinbarra River at Beechwood Gauge (207010)

A comparison of predicted flow rates between the recorded data, the WBNM model and the TUFLOW model is also included in **Appendix C**.

- **Figure C-4** for the Pappinbarra River at Beechwood Gauge (207010)

FIGURE 5-1

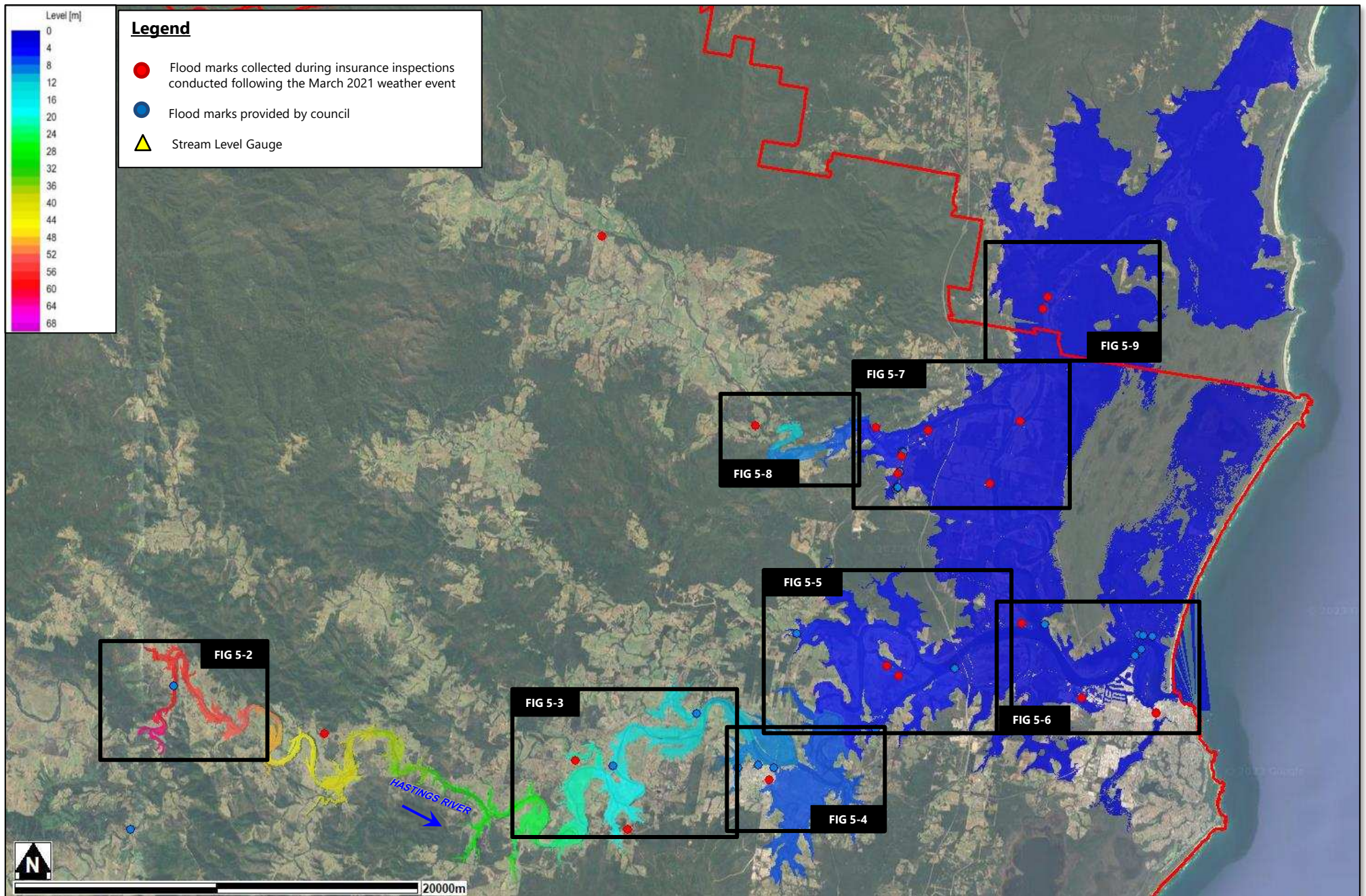


FIGURE 5-2

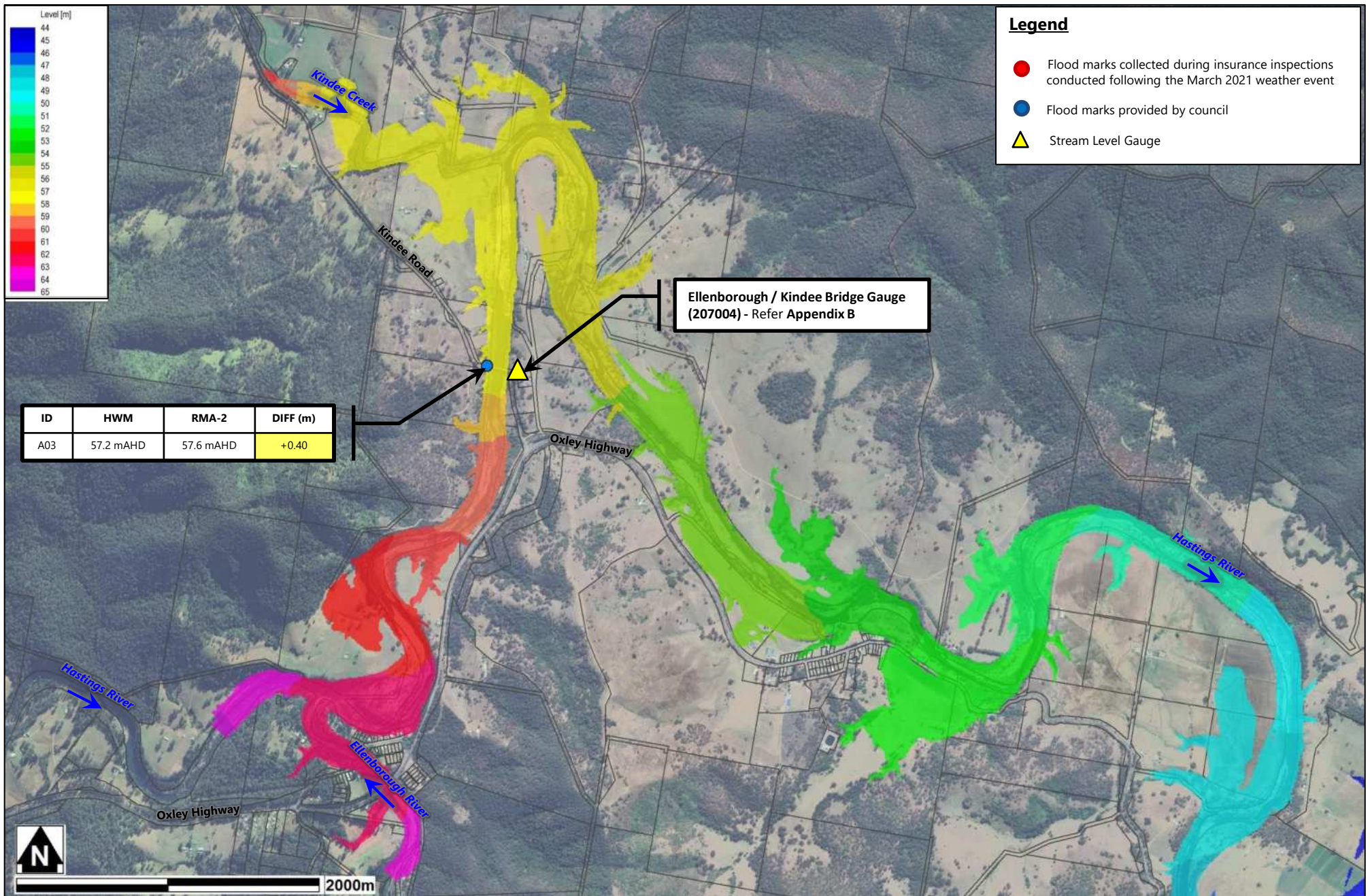


FIGURE 5-3

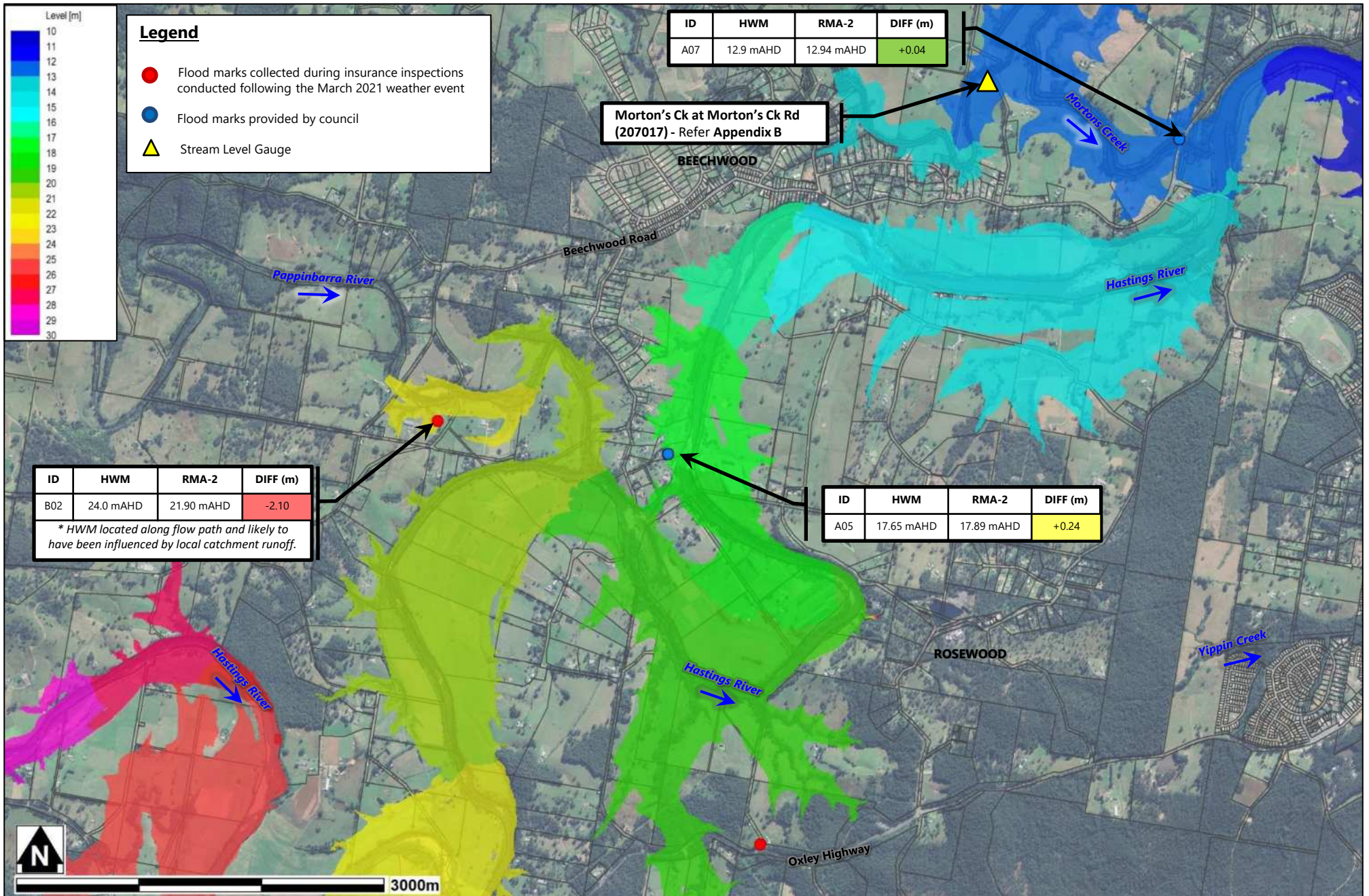


FIGURE 5-4

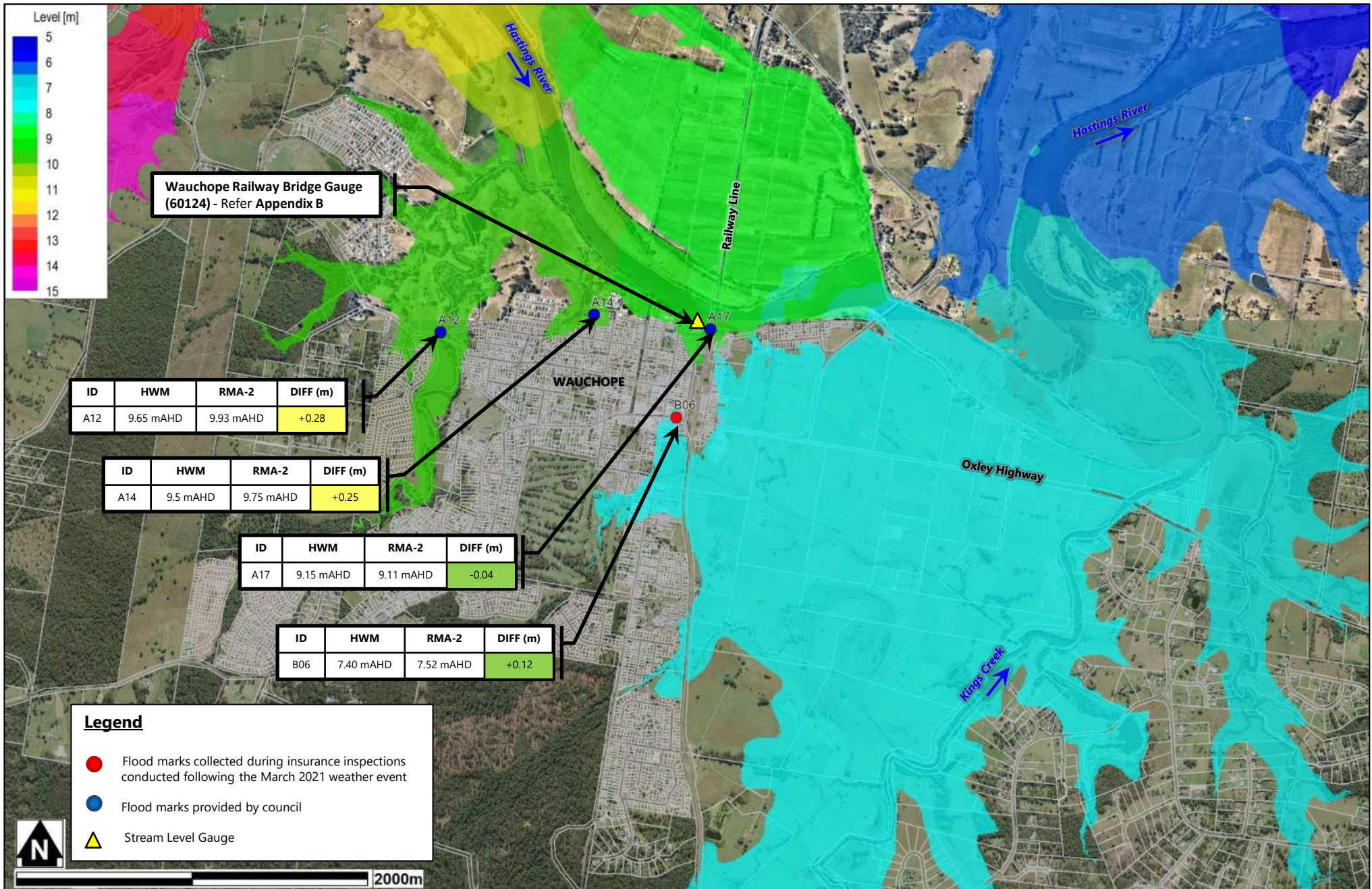


FIGURE 5-5

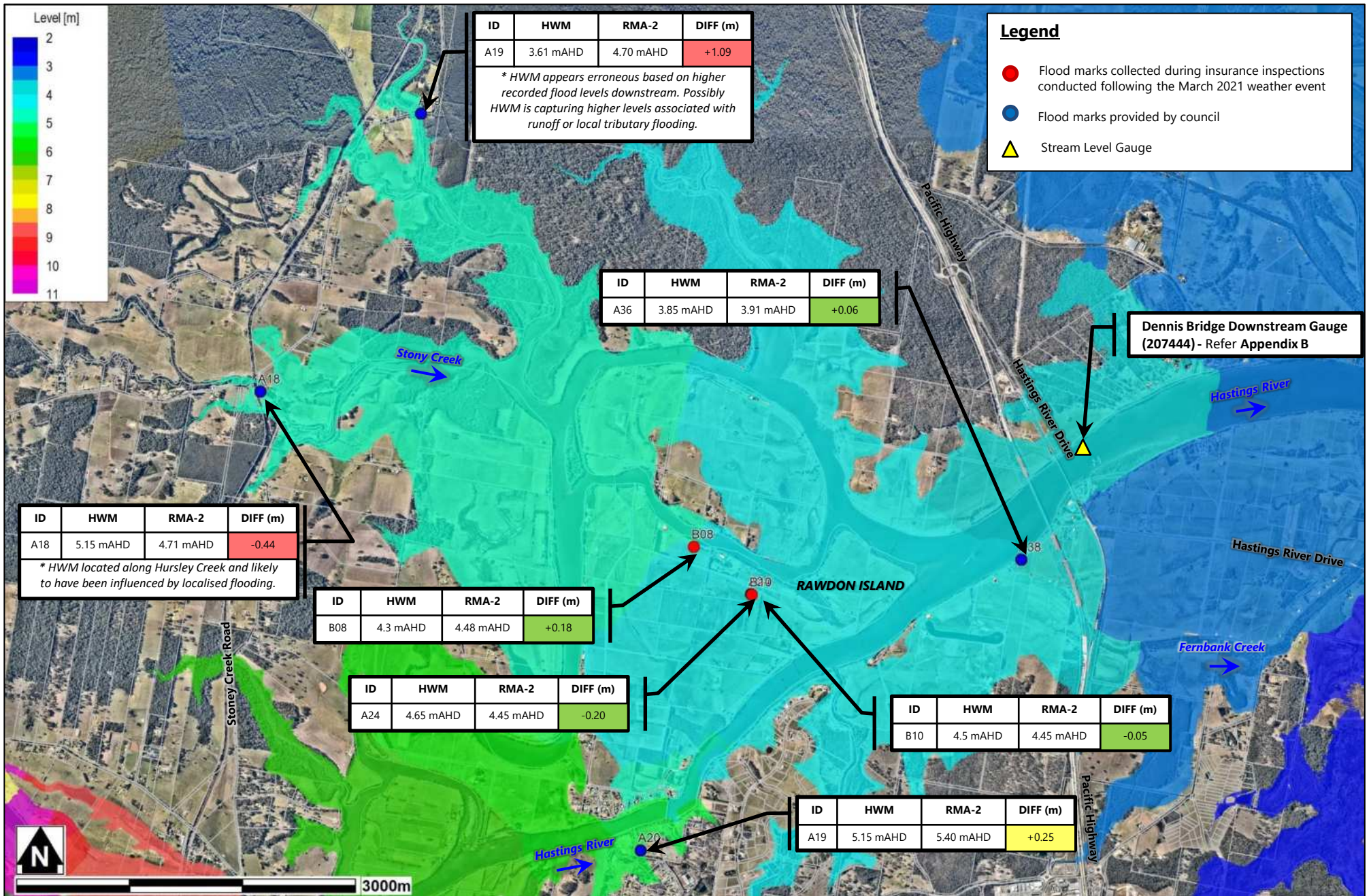


FIGURE 5-6

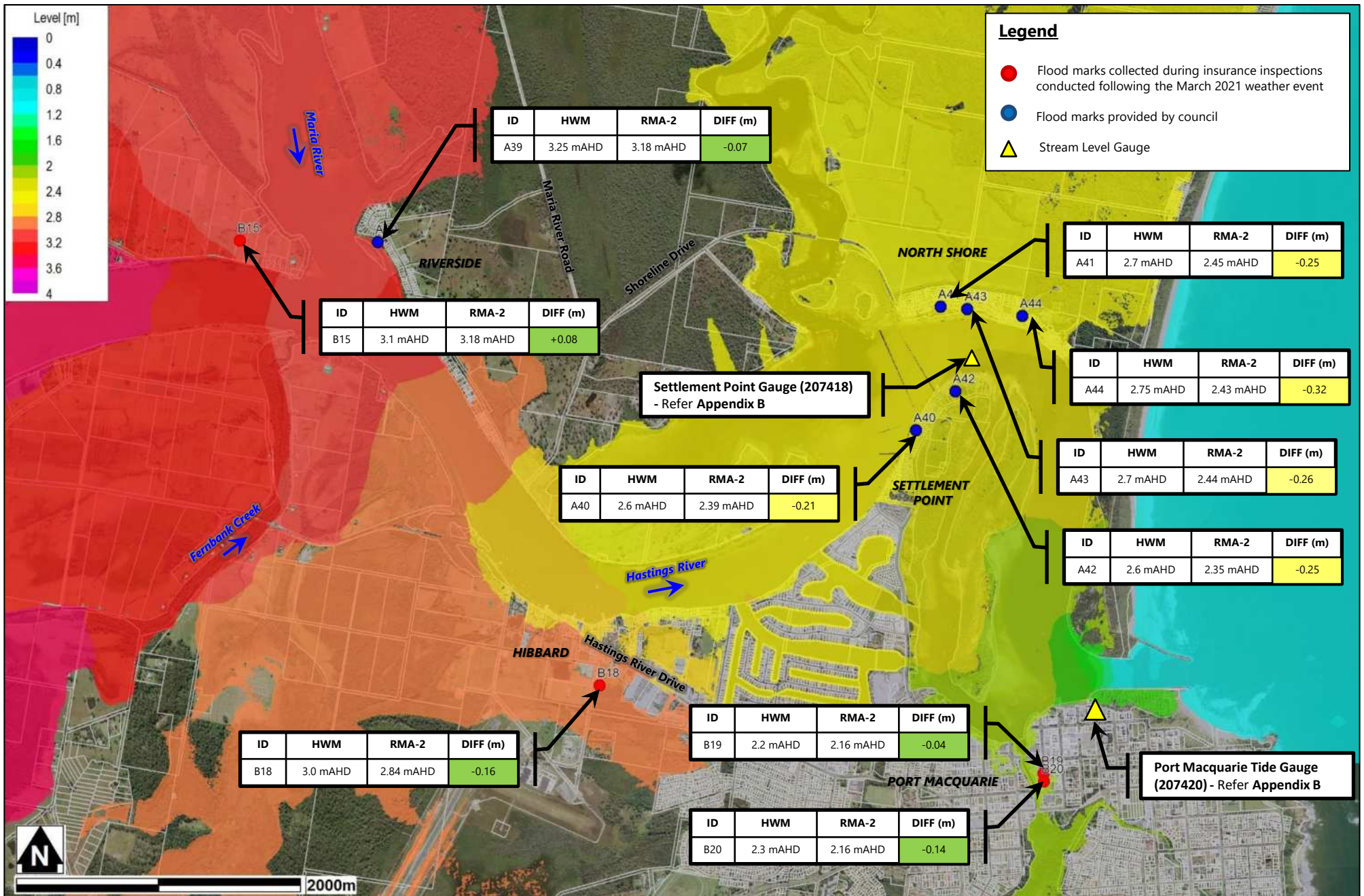


FIGURE 5-7

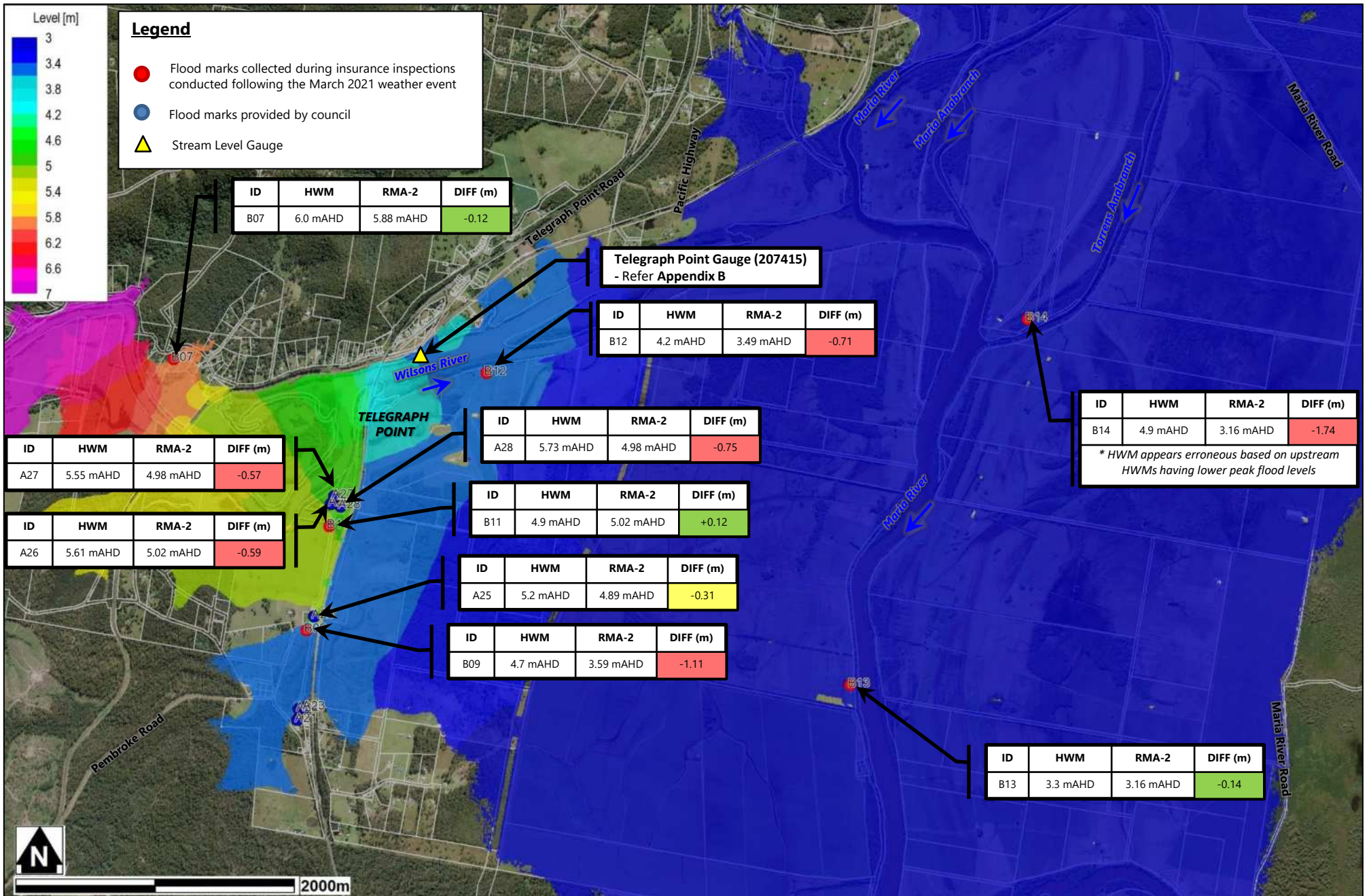


FIGURE 5-8

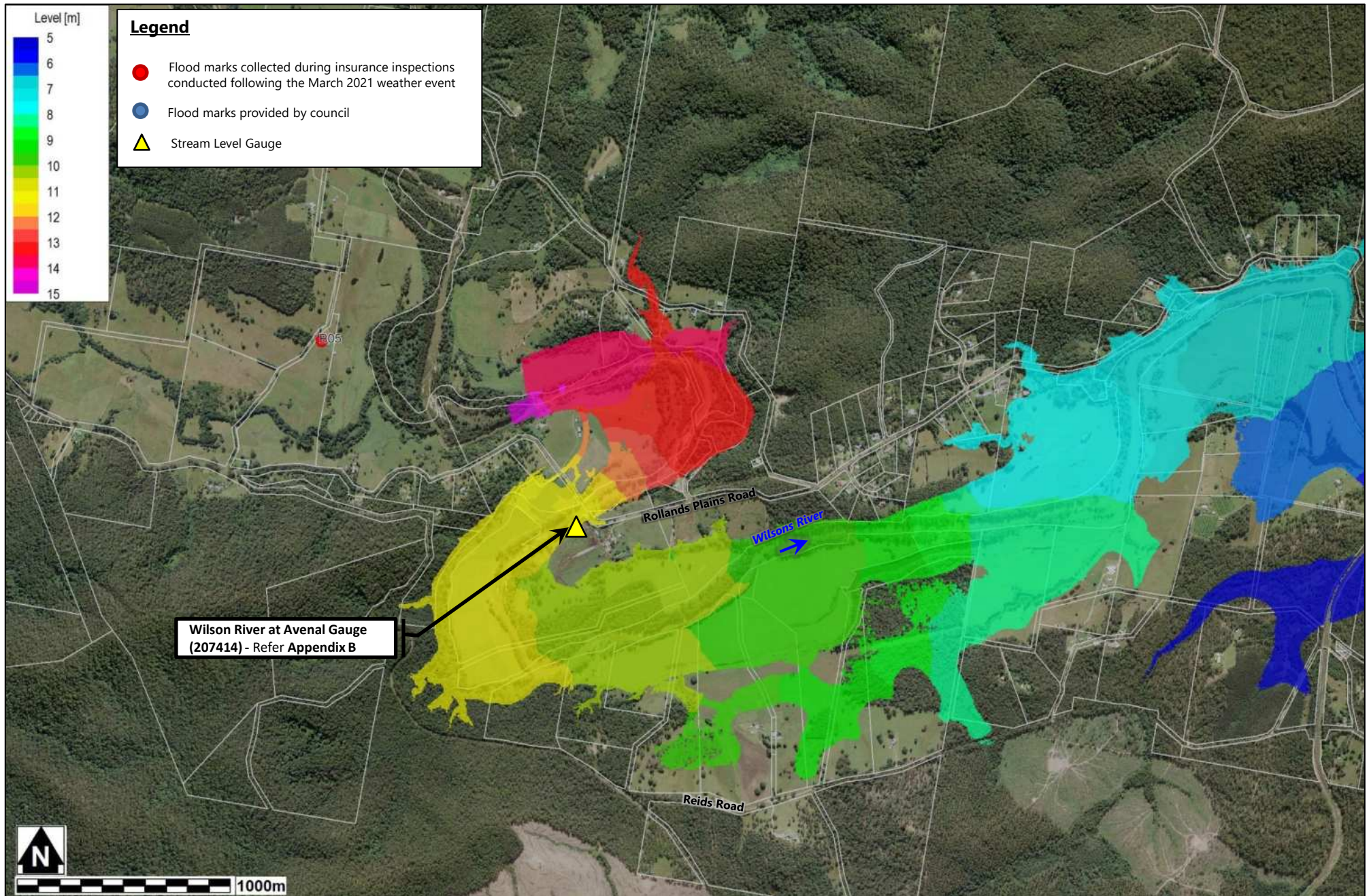


FIGURE 5-9

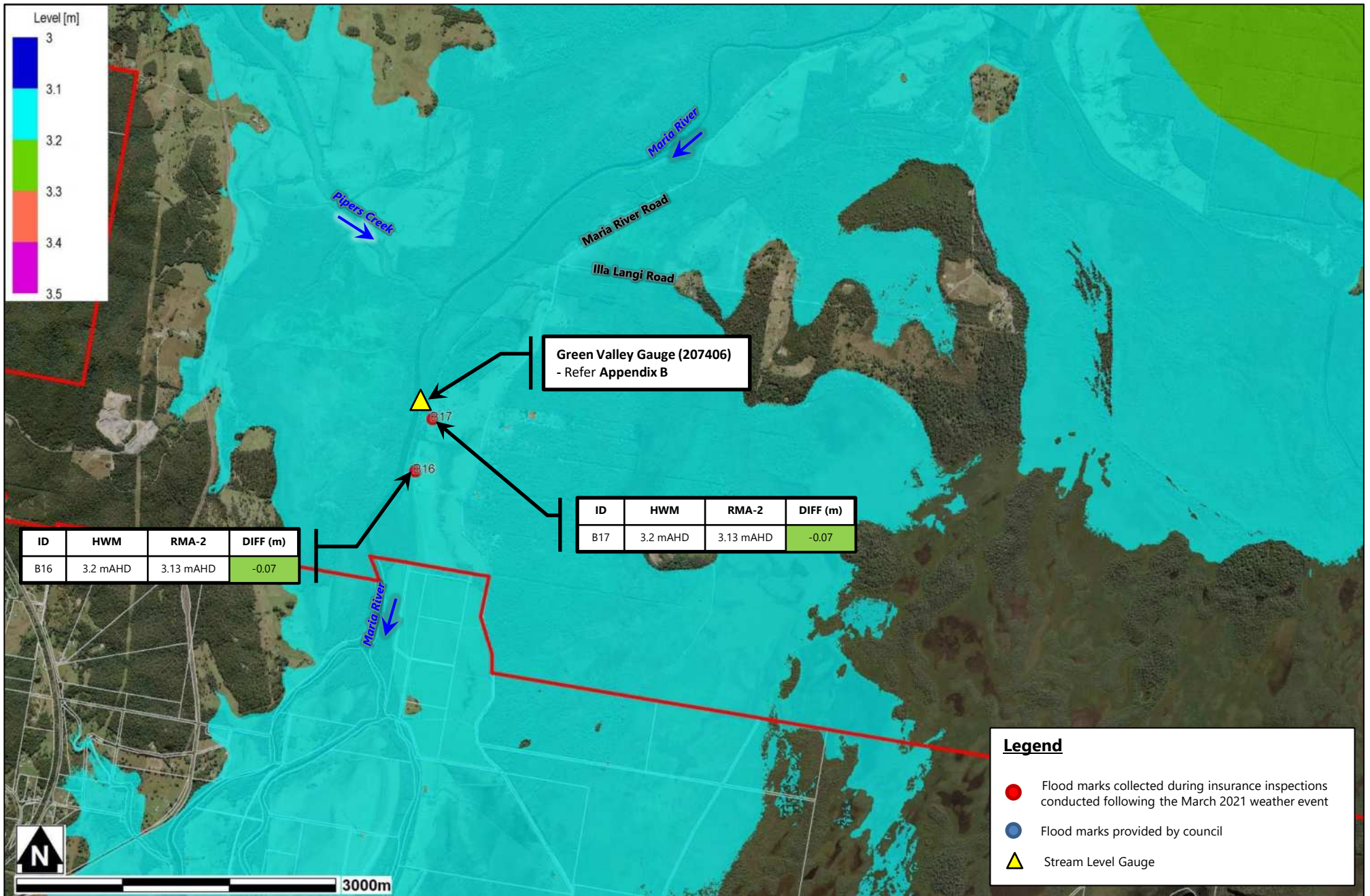


Table 5-2 Overview of RMA-2 Simulated and Recorded Flood Level Differences

HWM ID	Figure	Recorded Flood Levels (mAHD)	RMA-2 Simulated Flood level (mAHD)	Difference (m)	Location of HWM
A03	Figure 5-2	57.20	57.60	0.40	Long Flat
A05	Figure 5-3	17.65	17.89	0.24	Beechwood
A07	Figure 5-3	12.90	12.94	0.04	Beechwood
A12	Figure 5-4	9.65	9.93	0.28	Yippin Creek
A14	Figure 5-4	9.50	9.75	0.25	Wauchope
A17	Figure 5-4	9.15	9.11	-0.04	Wauchope
B06	Figure 5-4	7.40	7.52	0.12	Wauchope
B08	Figure 5-5	4.30	4.48	0.18	Rawdon Island
A24	Figure 5-5	4.65	4.45	-0.20	Rawdon Island
A36	Figure 5-5	3.85	3.91	0.06	Sancrox
B10	Figure 5-5	4.50	4.45	-0.05	Rawdon Island
A19	Figure 5-5	5.15	5.40	0.25	Sancrox
B15	Figure 5-6	3.10	3.18	0.08	Blackmans Point
A39	Figure 5-6	3.25	3.18	-0.07	Riverside
B18	Figure 5-6	3.00	2.84	-0.16	Hibbard
A40	Figure 5-6	2.60	2.39	-0.21	Settlement Point
A42	Figure 5-6	2.60	2.35	-0.25	Settlement Point
B19	Figure 5-6	2.20	2.16	-0.04	Port Macquarie
B20	Figure 5-6	2.30	2.16	-0.14	Port Macquarie
A41	Figure 5-6	2.70	2.45	-0.25	North Shore
A43	Figure 5-6	2.70	2.44	-0.26	North Shore
A44	Figure 5-6	2.75	2.43	-0.32	North Shore
A27	Figure 5-7	5.55	4.98	-0.57	Telegraph Point
A26	Figure 5-7	5.61	5.02	-0.59	Telegraph Point
A28	Figure 5-7	5.73	4.98	-0.75	Telegraph Point
B11	Figure 5-7	4.90	5.02	0.12	Telegraph Point
A25	Figure 5-7	5.20	4.89	-0.31	Telegraph Point
B09	Figure 5-7	4.70	3.59	-1.11	Telegraph Point
B13	Figure 5-7	3.30	3.16	-0.14	The Hatch
B16	Figure 5-8	3.20	3.13	-0.07	Limeburners Creek
B17	Figure 5-8	3.20	3.13	-0.07	Limeburners Creek
Average Difference (m) -				-0.12	Standard
				0.25	Absolute Values
Median Difference (m) -				-0.07	Standard
				0.16	Absolute Values

Flood Levels predicted by the TUFLOW hydraulic model for the March 2021 event are in good agreement to the recorded data along the Wilson River and Morton's Creek with peak levels replicated to within 200 mm at both gauges (refer **Figures C-1** and **C-2**). This takes into consideration the limited availability of recorded rainfall data on which to replicate the event. The modelled flow hydrographs also match the shape of the flood level hydrograph well with the timing of the peaks and shape of the rising and falling limbs replicated.

Figure C-3 indicates that the TUFLOW model is underpredicting flood levels along the Pappinbarra River at Beechwood gauge by up to 1.5 metres. As discussed in **Section 5.3.3** and shown in **Plate 5-5**, peak flows predicted by the WBNM model could not be validated against the Pappinbarra River gauge due to the rating curve being incomplete for gauge heights above 8.0 metres, which is approximately 3.3 metres below the peak flood height recorded in March 2021. The challenges in validating the TUFLOW model along the Pappinbarra River is therefore attributed to uncertainties in the magnitude of flows generated during the flood event, combined with a lack of daily read or continuous rainfall gauges in the catchment to reliably replicate the rainfall conditions. Accordingly, the poor calibration to flood levels recorded at the Pappinbarra River gauge is not considered to be a direct reflection of the reliability of the TUFLOW model.

The TUFLOW hydraulic results were then compared to high-water marks across the catchment. These marks were sourced from surveyed high-water marks provided by PMHC and independent inspections of flood affected properties for insurance reports for the March 2021 East Coast Weather Event.

In that regard, PMHC provided 44 surveyed high-water marks across the Hastings River Catchment, of which six (6) fall within the TUFLOW model boundaries. In addition, Worley (acting at the time as Advisian) conducted independent inspections of flood effected properties for insurance reports for the '*March 2021 East Coast Weather Event*' within the study area. From this, nine (9) additional flood marks were obtained for use in the calibration of the TUFLOW hydraulic model.

A comparison of high-water marks and peak modelled levels is shown in **Figure 5-10** to **5-16**. These high-water marks are overlayed on peak water level mapping for the study area as modelled by TUFLOW.

Table 5-3 also presents a summary of the HWM calibration.

The comparison shows that there were difficulties calibrating the TUFLOW model to the available HWMs with the majority of locations showing difference in flood levels of over +/- 0.4m. This is reflected in the average and median statistics showing differences ranging between -0.28m to 0.41 metres, depending on whether absolute values are adopted (refer **Table 5-3**).

It is noted however, that the differences are not considered to be a good indication of the reliability of the TUFLOW model given that the simulated levels are based on flows derived using the WBNM hydrologic model. As discussed in **Section 5.3.3**, issues were encountered when calibrating the WBNM model due to the lack of rainfall gauges available throughout the catchment. This was more prevalent in catchments upstream of Mortons Creek.

The calibration difficulties along the Pappinbarra River for example, were influenced by the availability of only one rainfall gauge in the catchment which was daily read (refer **Figure 3-3**). This results in significant uncertainty in the generated flows which in turn impacts the ability of the TUFLOW model to reliably reproduce the recorded flood levels.

In addition, a number of the HWMs located within the TUFLOW model extent are located along minor tributaries with small catchments. HWMs A38 and A40 are two such examples (refer **Figure 5-15**) with peak flood levels that are attributed to localised creek flows instead of from a larger watercourse, such as the nearby Wilson River. For these small catchments the issues associated with a lack of rainfall data were more severe resulting in a high level of uncertainty in the flows produced by the WBNM model. This in turn impacts the peak flood levels predicted by the TUFLOW model. Accordingly, the inability of the TUFLOW simulation to replicate the recorded HWMs is not a direct reflection of the model's reliability.

Table 5-3 Overview of TUFLOW Simulated and Recorded Flood Level Differences

HWM ID	Figure	Recorded Flood Levels (mAHD)	RMA-2 Simulated Flood level (mAHD)	Difference (m)	Location of HWM	Estimated Accuracy of HWM Elevation
A35	Figure 5-11	109.5	108.90	-0.60	Pappinbarra	Moderate (+/- 0.3m)
A36	Figure 5-12	46.00	46.40	0.40	Lower Pappinbarra	Low (+/- 0.5m)
A07	Figure 5-13	12.90	12.78	-0.12	Beechwood	Council Provided
A37	Figure 5-14	61.00	60.82	-0.18	Bril Bril	High (+/- 0.2m)
A40	Figure 5-15	55.50	55.50	0.00	Upper Rollands Plains	High (+/- 0.2m)
A39	Figure 5-15	55.80	56.30	0.50	Upper Rollands Plains	Low (+/- 0.5m)
A38	Figure 5-15	39.20	38.06	-1.14	Rollands Plains	High (+/- 0.2m)
A40	Figure 5-15	67.60	66.90	-0.70	Marlo Merrican	Moderate (+/- 0.3m)
A43	Figure 5-16	13.00	12.65	-0.35	Ballengarra	Council Provided
B07	Figure 5-16	6.00	5.59	-0.41	Telegraph Point	High (+/- 0.2m)
A27	Figure 5-16	5.55	5.10	-0.45	Telegraph Point	High (+/- 0.2m)
Average Difference (m) -				-0.28	Standard	
				0.44	Absolute Values	
Median Difference (m) -				-0.35	Standard	
				0.41	Absolute Values	

Overall, the 2021 calibration showed that the TUFLOW model was able to predict flood levels that were within a reasonable magnitude of recorded HWMs, noting the varying degree of uncertainty in the flows input into the model.

5.4 Validation – February 2013 Flood

5.4.1 Event Overview

A comprehensive event overview for the February 2013 flood is documented in the *Hibbard Precinct Flood Study (2019)*. The following information is reproduced from the 2019 Flood Study.

- On the 18th February 2013, a low-pressure system formed off the east coast of Australia. Over the next few days, the system tracked in a westerly direction, making landfall on the north coast of New South Wales on the 22nd February (refer **Plate 5-7**). This resulted in widespread, persistent and heavy rainfall across the Hastings River catchment.
- Rainfall across the Hastings River catchment was well above average during February 2013 (refer **Plate 5-8**). Heavy rain and thunderstorms affected large parts of the New South Wales east coast between the 20th and 25th of February.
- Severe thunderstorms affected parts of the Hastings River catchment between the 22nd and 23rd February, with multiple rainfall gauges recording over 150 mm in a 24-hour period across the two days. The Bureau of Meteorology rainfall gauge at Yarras recorded 415 mm in the period between 9am on the 22nd February and 9am on the 23rd February.
- Cumulative rainfall totals for the month of February 2013 as recorded at various daily-read gauges and pluviographs throughout the catchment are presented in **Plate 5-9**. The data shows that significant rainfall was recorded between the 22nd and 24th of February.

FIGURE 5-10

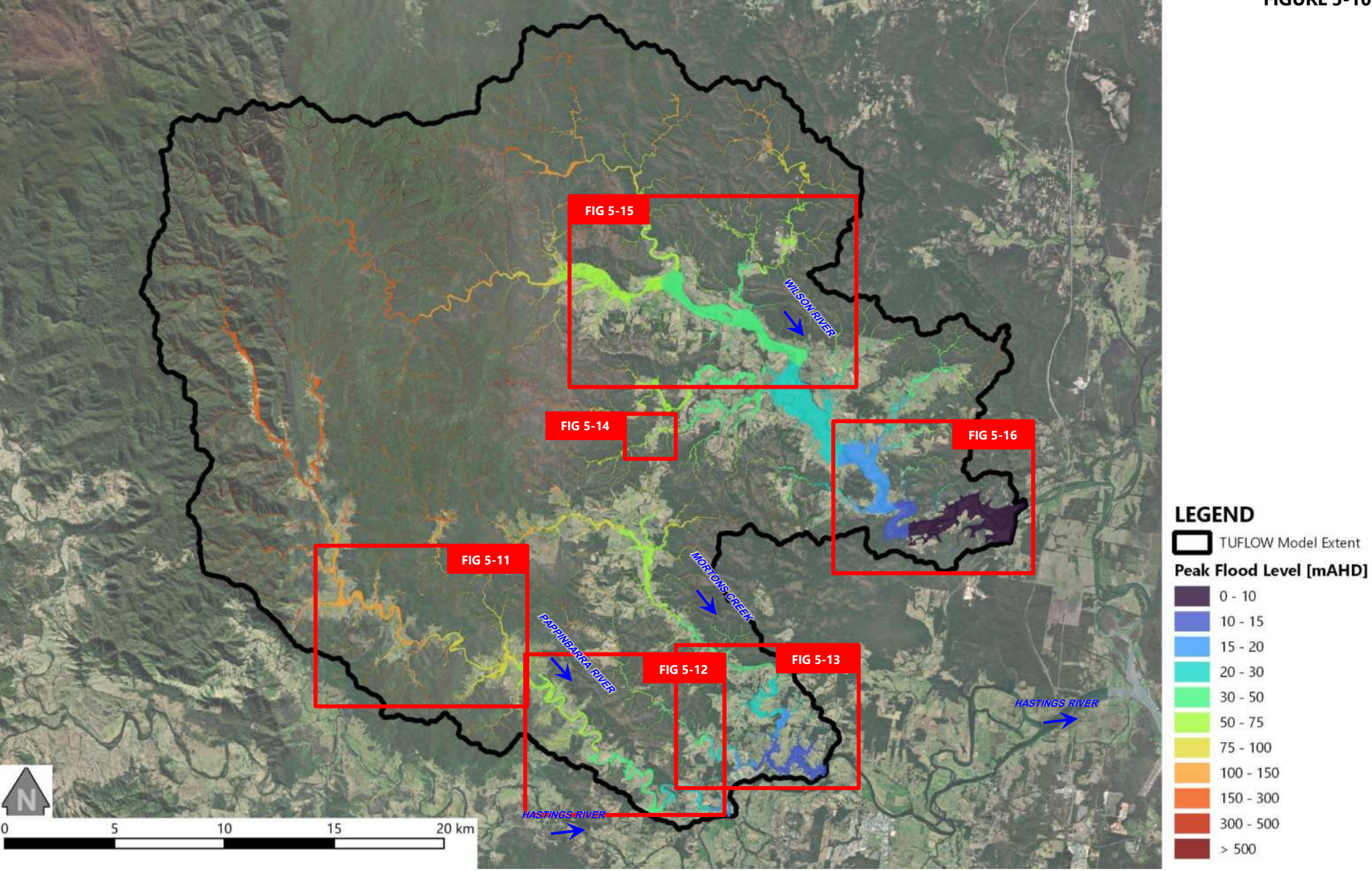
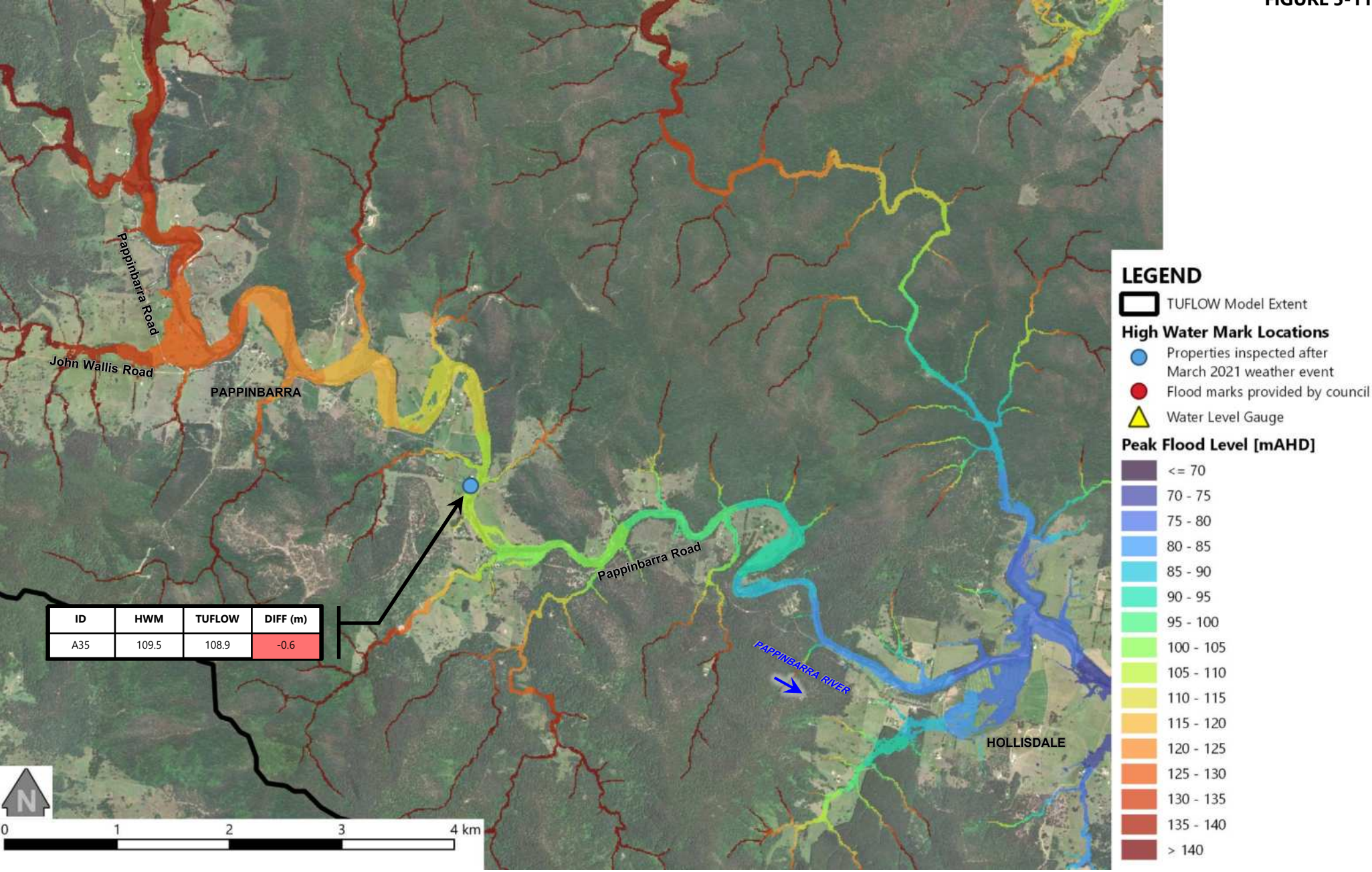


FIGURE 5-11



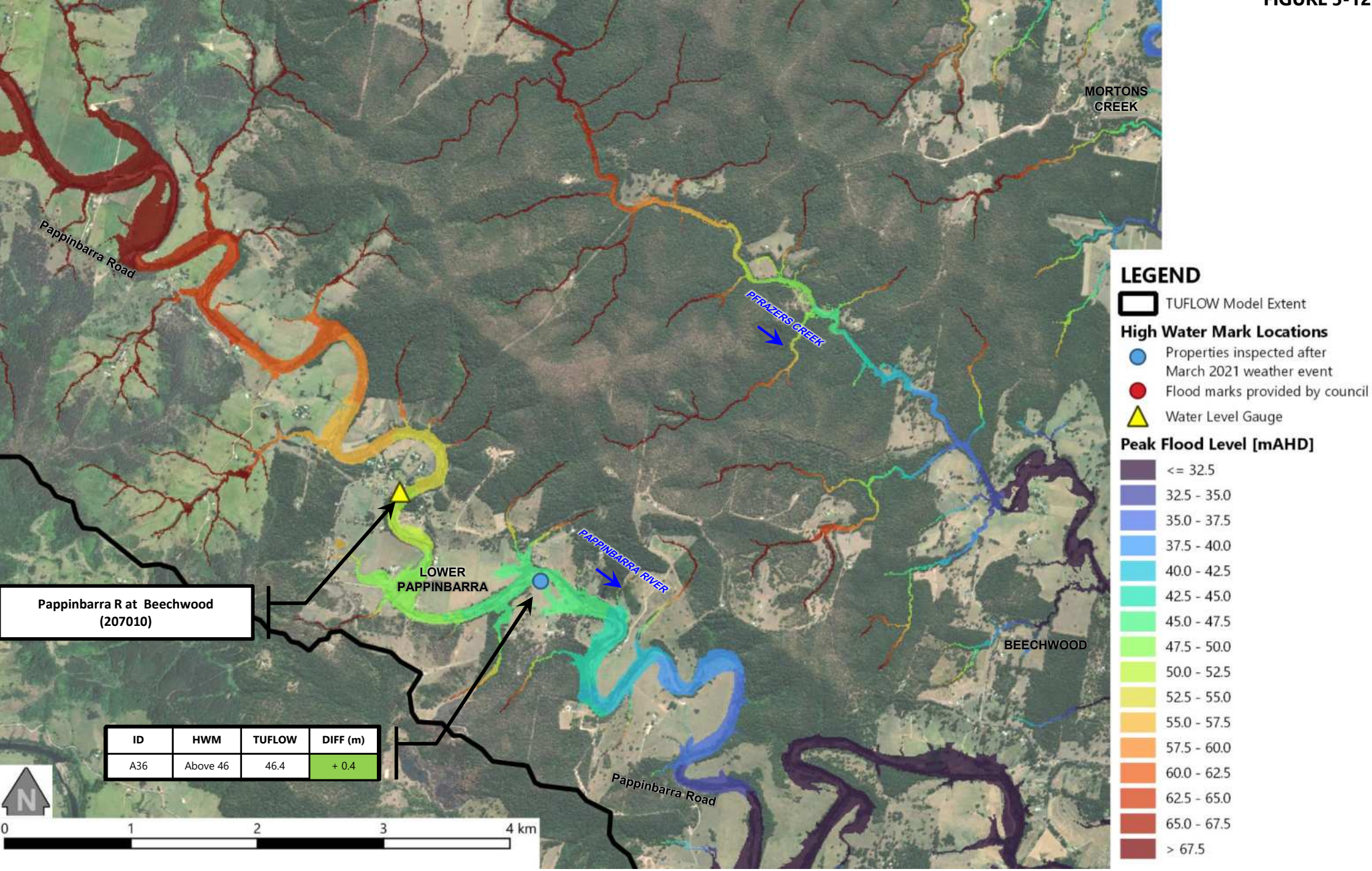
Note: Shading within HWM differences is representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Yellow = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF TUFLOW MODELLED LEVELS FOR THE MARCH 2021 FLOOD TO SURVEYED HWM (EXTENT 1 OF 6)



311015-00418 - Hastings Flood Model Extension
fg311015-00418_230718_Hastings_A4L.pdf

FIGURE 5-12



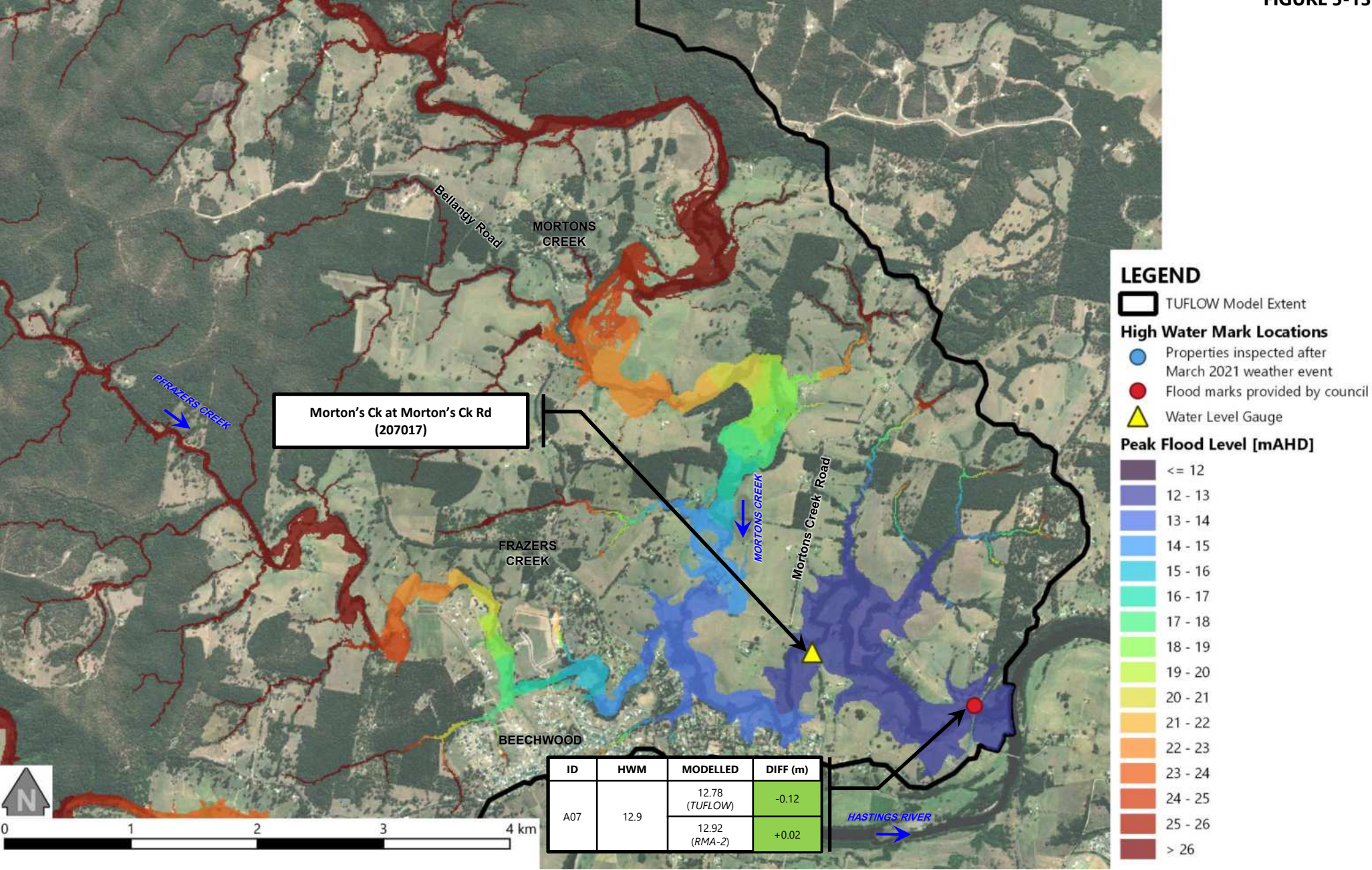
Note: Shading within HWM differences is representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Yellow = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF TUFLOW MODELLED LEVELS FOR THE MARCH 2021 FLOOD TO SURVEYED HWM (EXTENT 2 OF 6)



311015-00418 - Hastings Flood Model Extension
fg311015-00418_230718_Hastings_A4L.pdf

FIGURE 5-13



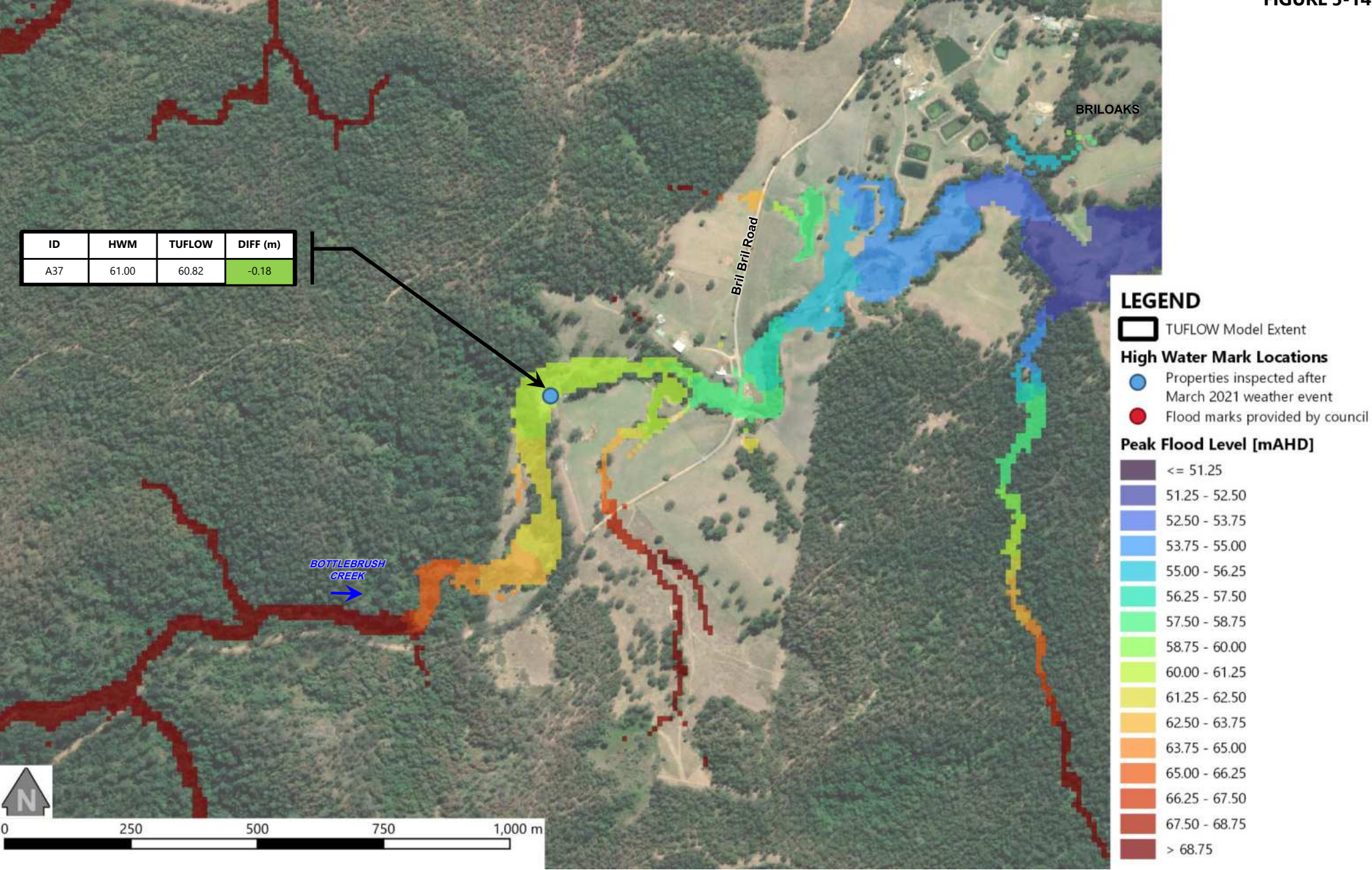
Note: Shading within HWM differences is representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Yellow = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF TUFLOW MODELLED LEVELS FOR THE MARCH 2021 FLOOD TO SURVEYED HWM (EXTENT 3 OF 6)



311015-00418 - Hastings Flood Model Extension
fg311015-00418_230718_Hastings_A4L.pdf

FIGURE 5-14

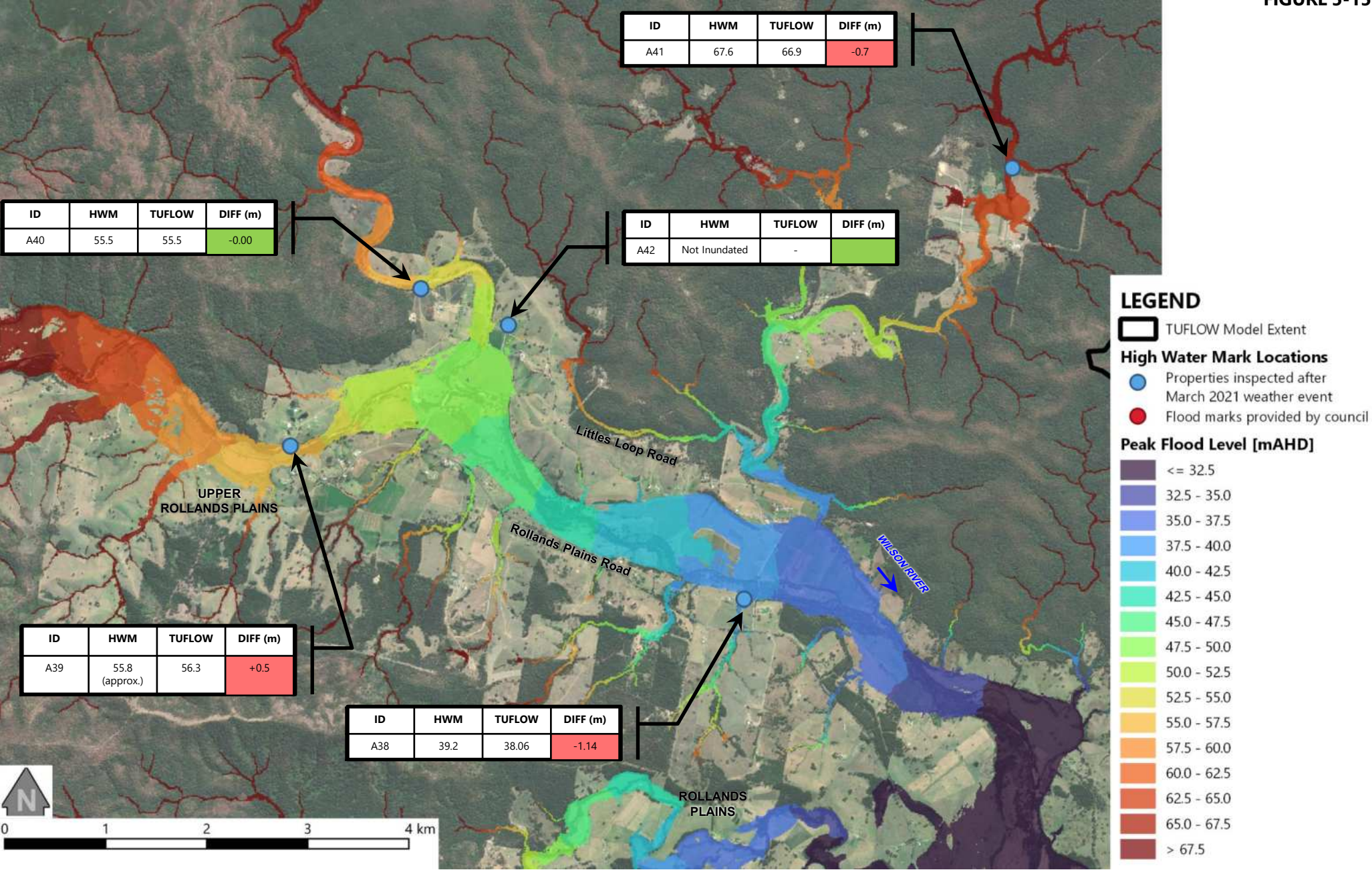


Note: Shading within HWM differences is representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Yellow = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF TUFLOW MODELLED LEVELS FOR THE MARCH 2021 FLOOD TO SURVEYED HWM (EXTENT 4 OF 6)



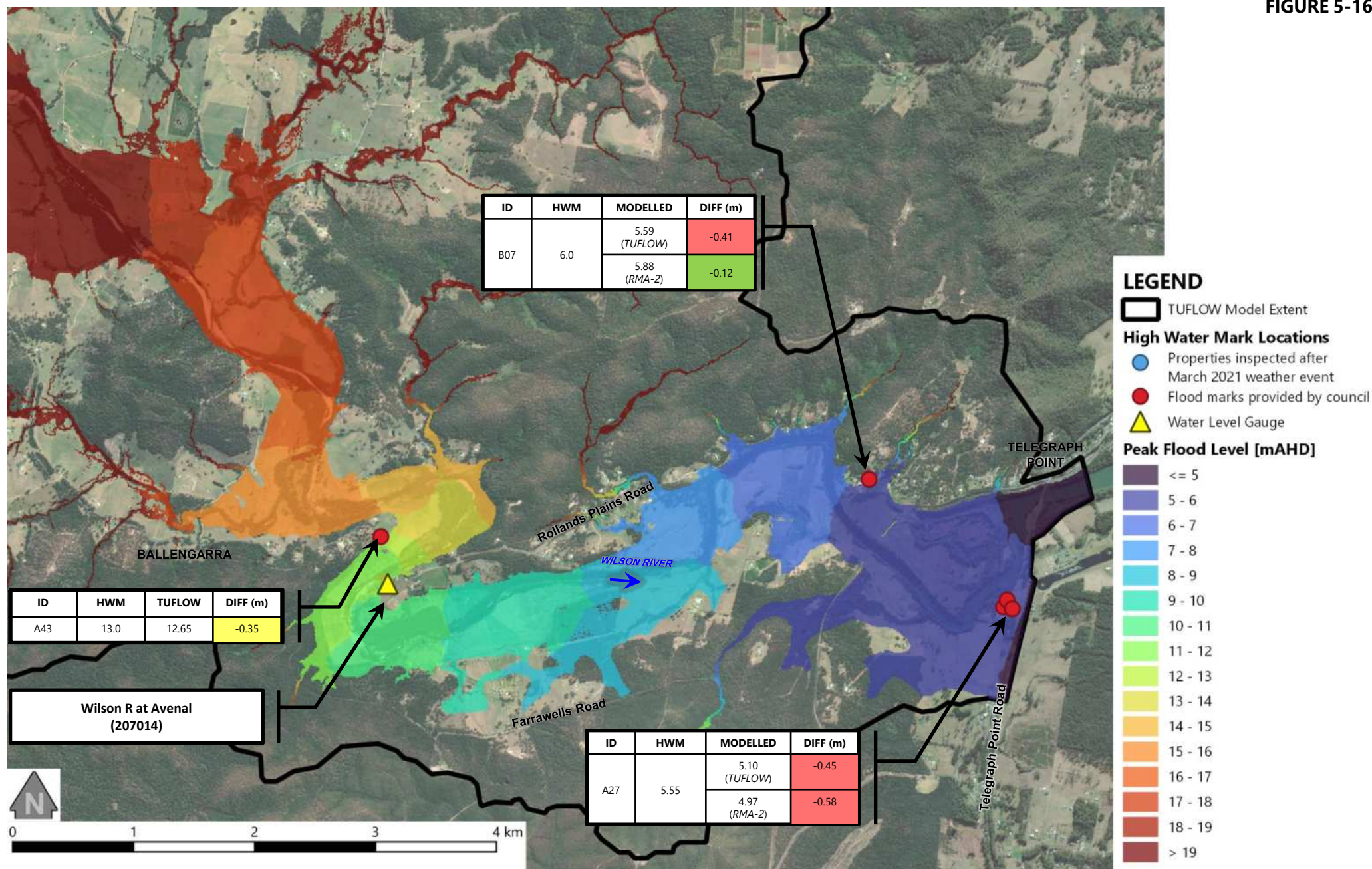
FIGURE 5-15



Note: Shading within HWM differences is representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Yellow = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF TUFLOW MODELLED LEVELS FOR THE MARCH 2021 FLOOD TO SURVEYED HWM (EXTENT 5 OF 6)

FIGURE 5-16



Note: Shading within HWM differences is representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Yellow = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF TUFLOW MODELLED LEVELS FOR THE MARCH 2021 FLOOD TO SURVEYED HWM (EXTENT 6 OF 6)

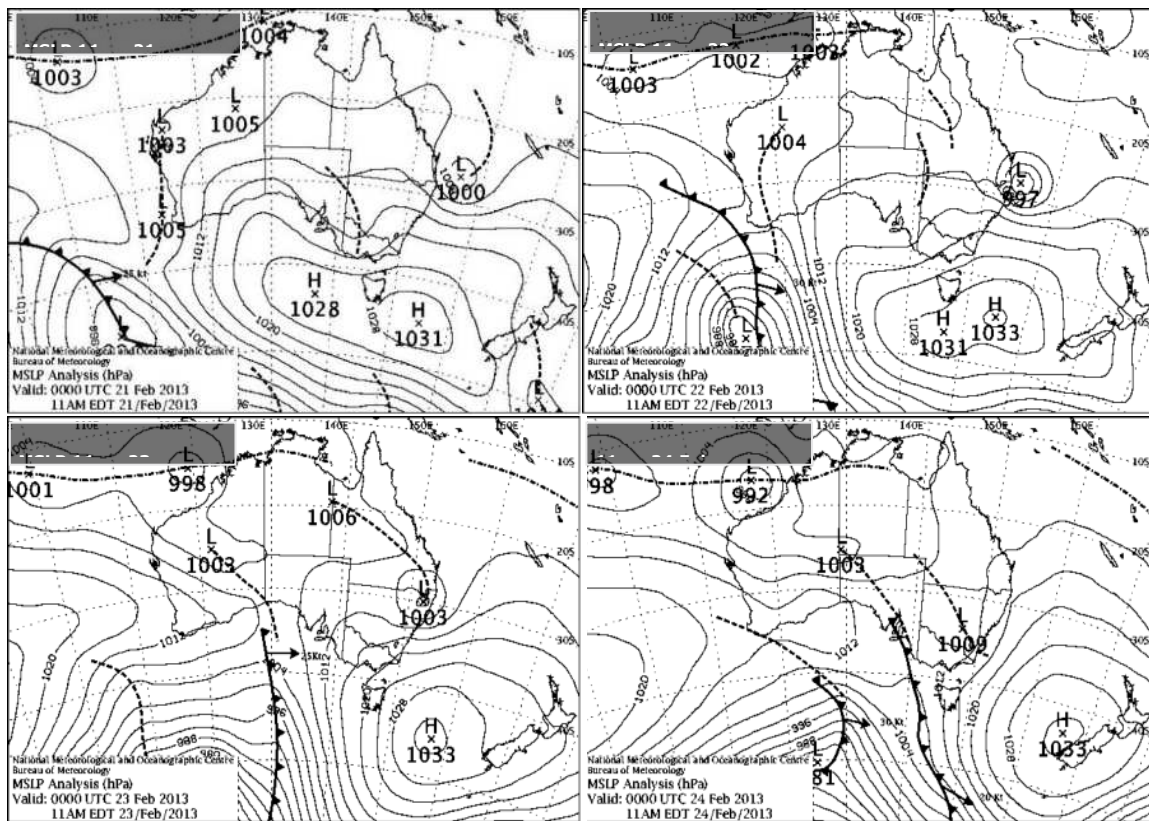


Plate 5-7 Mean Sea Level Pressure (MSLP) Plots for the February 2013 Event

Source: Hibbard Precinct Flood Study (2019)

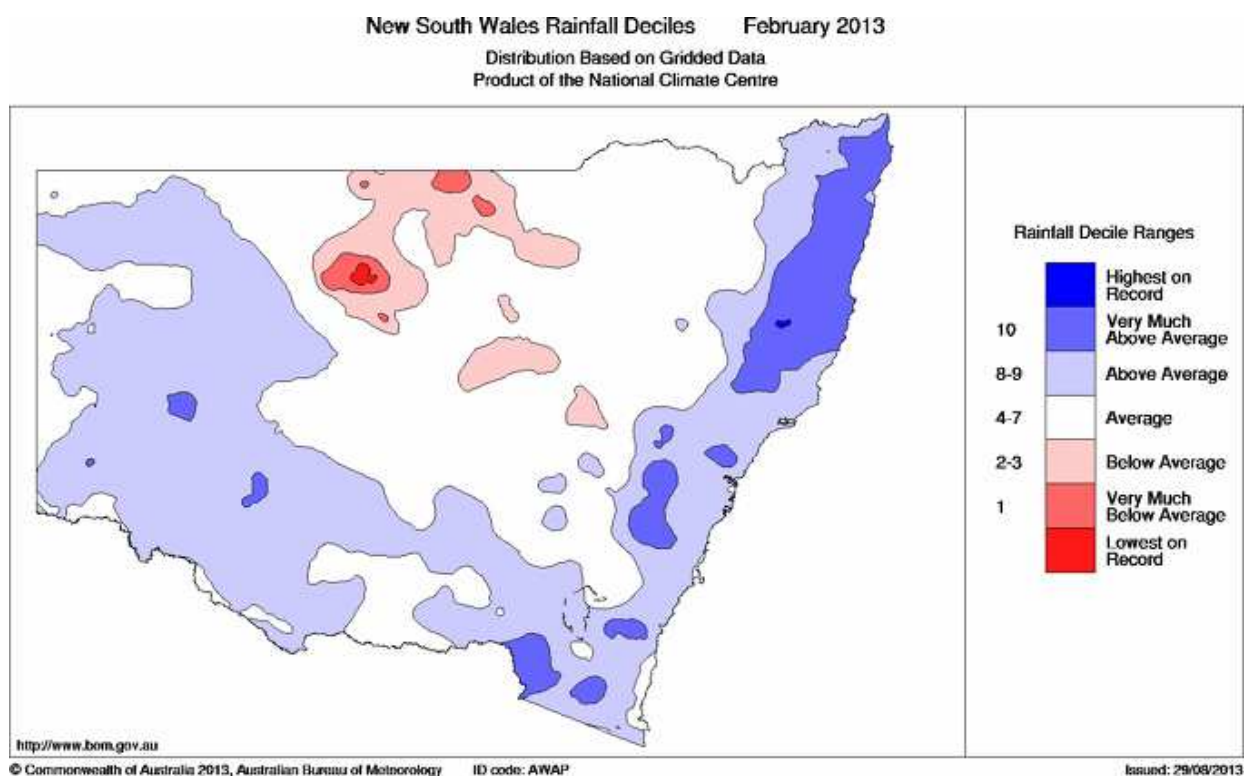


Plate 5-8 Monthly Rainfall Totals across New South Wales

Source: Hibbard Precinct Flood Study (2019)

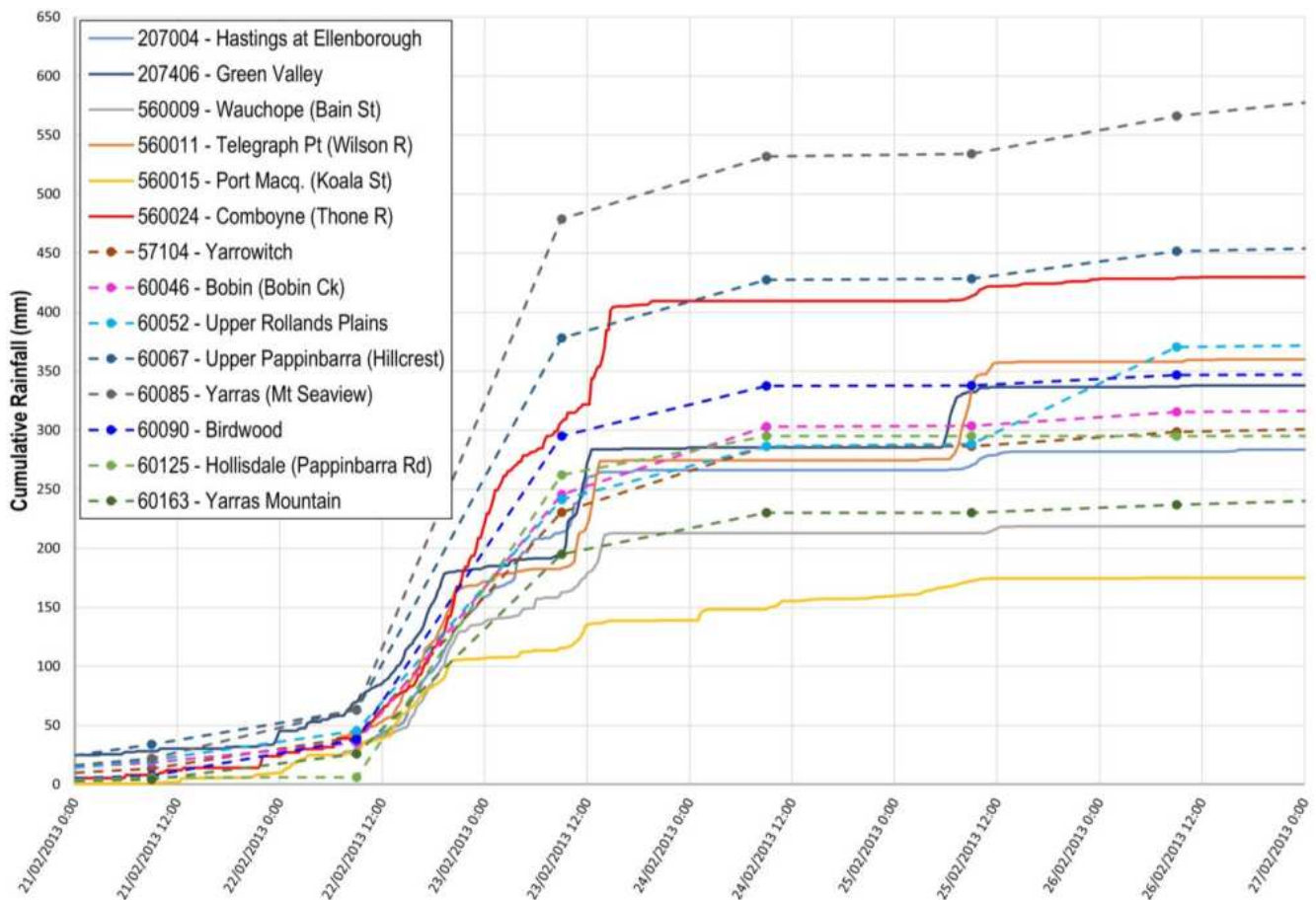


Plate 5-9 Cumulative Rainfall Totals across the Hastings River Catchment for 21st to 27th February 2013

- The Hibbard Precinct Flood Study (2019) calculated the following estimates of the Annual Exceedance Probability (AEP) of recorded rainfall at various gauges based on ARR87 Intensity-Frequency-Duration (IFD) data for various durations.
 - The rainfall recorded at the Yarras Gauge over a 24-hour period was in the order of a 1.2% AEP event; which is approximately equivalent to an average recurrence interval of 83 years.
 - The Comboyne Gauge recorded the second largest rainfall with 325 mm over a 24-hour period. This equates to a 5% AEP event for this duration, i.e., equivalent to an average recurrence interval of once every 20 years. The gauge is in the southern section of the Upper Hastings River Catchment.
 - All other gauges across the upper and middle areas of the catchment recorded rainfall that was approximately equivalent to the ARR 87 estimate for the 20% AEP event, i.e., equivalent to an average recurrence interval of once every 5 years.

5.4.2 River Level Data

A number of river level gauges across the catchment were in operation during the February 2013 event. These were spread throughout the catchment, on the Hastings River and a number of major tributaries. River level data recorded during the February 2013 flood is presented in **Plate 5-10**.

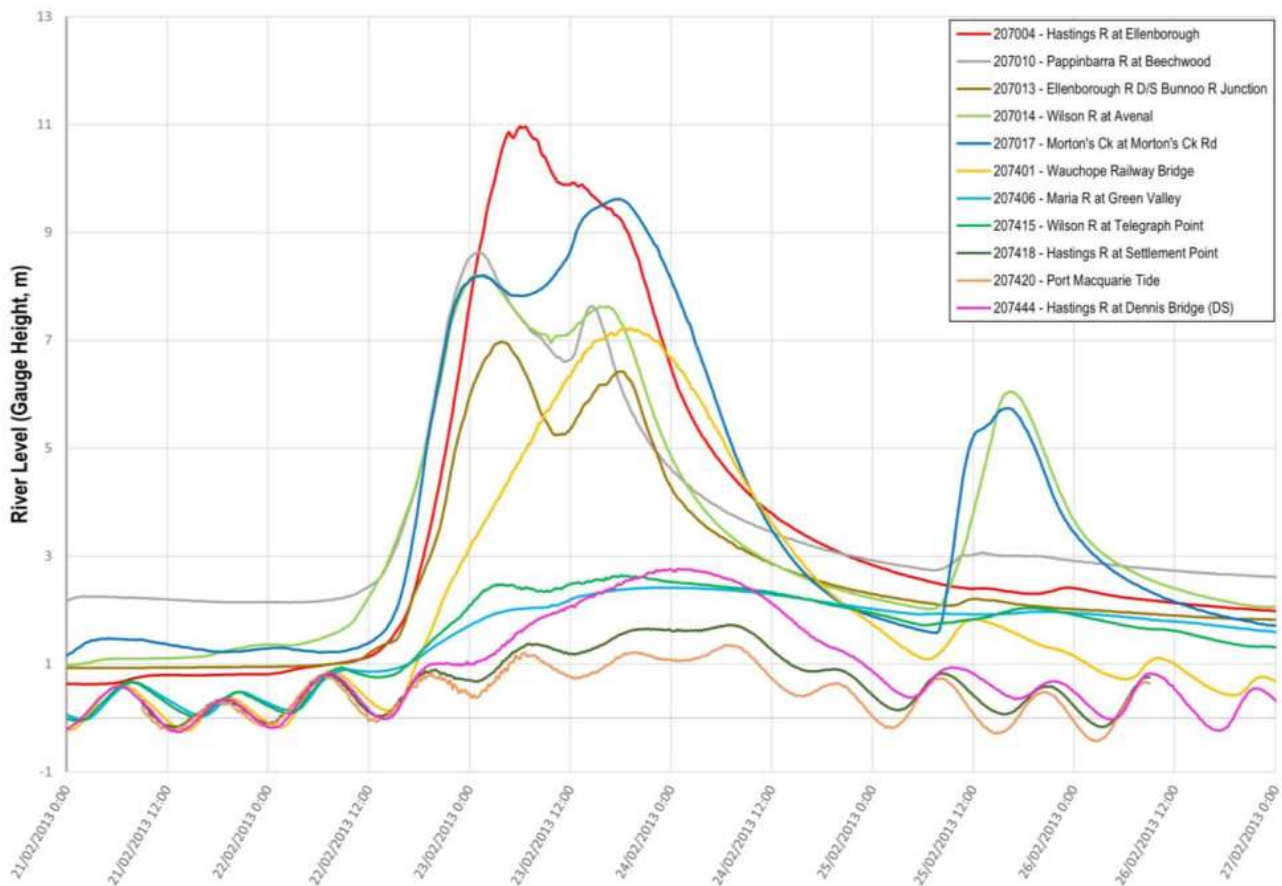


Plate 5-10 Recorded River level data for the Hastings River and Major Tributaries – February 2013

5.4.3 Rainfall Data

Cumulative rainfall totals at various daily-read and continuous-read (pluviometer) gauges throughout the catchment are presented in **Plate 5-9** for the period between the 21st to 27th of February 2013.

The data shows that there was high spatial variation in rainfall across the catchment, in terms of timing and total volume. The variation in timing is particularly problematic based on the limited number of pluviometer gauges across the catchment. This is particularly an issue in the upper catchment.

5.4.4 Hastings River WBNM Model Validation

Similar to the calibration process for the WBNM hydrologic model to the March 2021 event, the high spatial variation in rainfall across the catchment, in terms of timing and volume, as well as a high percentage of daily read rainfall gauges, particularly in the western half of the catchment, created significant uncertainties that limited the reliability of the calibration.

Initial calibration testing utilised the few available pluviometers as well as additional 24-hour rainfall totals with temporal patterns derived from nearby pluviometer(s). The location of these gauges varied from within to outside the catchment boundary. To further improve the validation, multiple temporal patterns were tested at each of the daily read gauges to try and obtain a representative rainfall.

The final adopted rainfall data for the February 2013 Hastings River validation simulation is summarised in the following:

- 207004 Hastings River at Ellenborough – *Pluviometer*
- 207406 Green Valley (Maria River) – *Pluviometer*

- 560009 Wauchope (Bain Street) – *Pluviometer*
- 560011 Telegraph Point (Wilson River) – *Pluviometer*
- 560015 Port Macquarie (Koala Street) – *Pluviometer*
- 560024 Comboyne (Thone River) – *Pluviometer*
- 57104 Yarrowitch – *Daily Read*
- 60046 Bobin (Bobin Creek) – *Daily Read*
- 60052 Upper Rollands Plains – *Daily Read*
- 60067 Upper Pappinbarra (Hillcrest) – *Daily Read*
- 60085 Yarras (Mt Seaview) – *Daily Read*
- 60090 Birdwood – *Daily Read*
- 60125 Hollisdale (Pappinbarra Road) – *Daily Read*
- 60163 Yarras Mountain – *Daily Read*

A cumulative rainfall plot of the adopted rainfall data is presented in **Plate 5-9**, and the locations of these gauges are shown in **Figure 3-5**.

The validation of the WBNM hydrologic model was completed by comparing flows against recorded flow rate data obtained from river gauges across the catchment. This was undertaken by comparing WBNM total runoff hydrographs. Calibrated WBNM parameters in **Table 5-1** were adopted for the validation process. Where differences arose, adjustment of the Initial and Continual loss rates was done to try match the hydrologic model to recorded levels. The validation process resulted in an IL-CL combination of 50 mm and 4.0 mm/hr for pervious surfaces.

Similar to the calibration process, a few issues arose where the WBNM model was unable to generate hydrographs that fit both the shape and magnitude of those recorded at various gauges. This was mainly due to the lack of temporal patterns that were available throughout the catchment to which the relevant daily-read gauges, particularly in the western half of the catchment, could be applied (refer **Figure 3-5**).

Graphs comparing the final flow hydrographs generated by WBNM model for the March 2013 event and those recorded at various streamflow gauges are included as **Appendix A**. Comparisons are included for the following streamflow gauges.

- **Figure A-8** for the Ellenborough River at Downstream Bunnoo River Junction (207013)
- **Figure A-9** for the Forbes River at Birdwood (Filly Flat) (207006)
- **Figure A-10** for the Hastings River at Mount Seaview (207015)
- **Figure A-11** for the Hastings River at Ellenborough (Kindee Bridge) (207004)
- **Figure A-12** for the Pappinbarra River at Beechwood (207010)
- **Figure A-13** for the Thone River at Deep Creek Road (207018)
- **Figure A-14** for the Wilson River at Avenal (207014)

5.4.5 Hastings River RMA-2 Hydraulic Model Validation

The extended RMA-2 model that was calibrated to the March 2021 event (refer **Section 5.3.4**) was used for further validation against the February 2013 event. The February 2013 event was simulated with the RMA-2 model by using the inflow hydrographs extracted from the WBNM model at all upstream and local catchment inflow locations (refer **Sections 4.3.1 and 4.3.1**).

A comparison of predicted flood level hydrographs to recorded levels at gauges located within the RMA-2 model domain are included in **Appendix B**.

- **Figure B-9** for the Hastings River at Ellenborough (Kindee Bridge) Gauge (207004)
- **Figure B-10** for the Hastings River at Wauchope Railway Bridge Gauge (60124)
- **Figure B-11** for the Hastings River at Dennis Bridge Gauge (207444)
- **Figure B-12** for the Hastings River at Settlement Point Gauge (207418)
- **Figure B-13** for the Hastings River at Port Macquarie Gauge (207420)
- **Figure B-14** for the Wilson River at Telegraph Point Gauge (207415), and
- **Figure B-15** for the Maria River at Green Valley (207406)

Figures B-10 to B-15 show that the RMA-2 model predicts flood levels that are in good agreement to those recorded at available gauges. The predicted flood levels match well to the recorded shape and peak flood levels to generally within 0.10 to 0.30 metres.

One exception to the above is for the Hastings River at Dennis Bridge gauge. As shown in **Figure B-11**, the RMA-2 model predicts flood levels that are 0.4 metres higher at the peak. The comparison also indicates that there is additional flow volume present in the RMA-2 simulation shown by the additional area present beneath the RMA-2 flood level hydrograph, compared to the recorded data. Given flood levels downstream of Wauchope are affected by volume it is possible that the higher flood levels are partly attributed to this.

High Water Marks (HWMs) for the February 2013 event were collected as part of the Hibbard Precinct Flood Study (2019). The available HWMs and the resulting validation of predicted RMA-2 flood levels is presented in **Figure 5-17**. The RMA-2 model is in good agreement with differences of between 0.02 and 0.08 metres predicted across all four HWMs.

5.4.6 Major Tributaries Hydraulic TUFLOW Model Validation

The validation for the Major Tributaries hydraulic model was also assessed against recorded water level data. The recorded water levels were compared at the same gauges as the 2021 calibration event.

To replicate February 2013 conditions, new relevant downstream water levels were applied to the model. These originated from recorded water level data from the same gauges as the calibration event, by applying MHL's Telegraph Point (207415) gauge recordings to the downstream end of the Wilson River and applying modified Wauchope (207401) gauge recordings to the downstream ends of the Pappinbarra River and Morton's Creek. This was used to simulate the conditions in the Hastings River at the time.

The hydraulic model validation was then assessed through the comparison of flood levels simulated by the TUFLOW model against available recorded water levels at WaterNSW gauges located along each of the tributaries. A comparison of predicted flood level hydrographs to recorded levels at gauges located within the TUFLOW model domain are included in **Appendix C**.

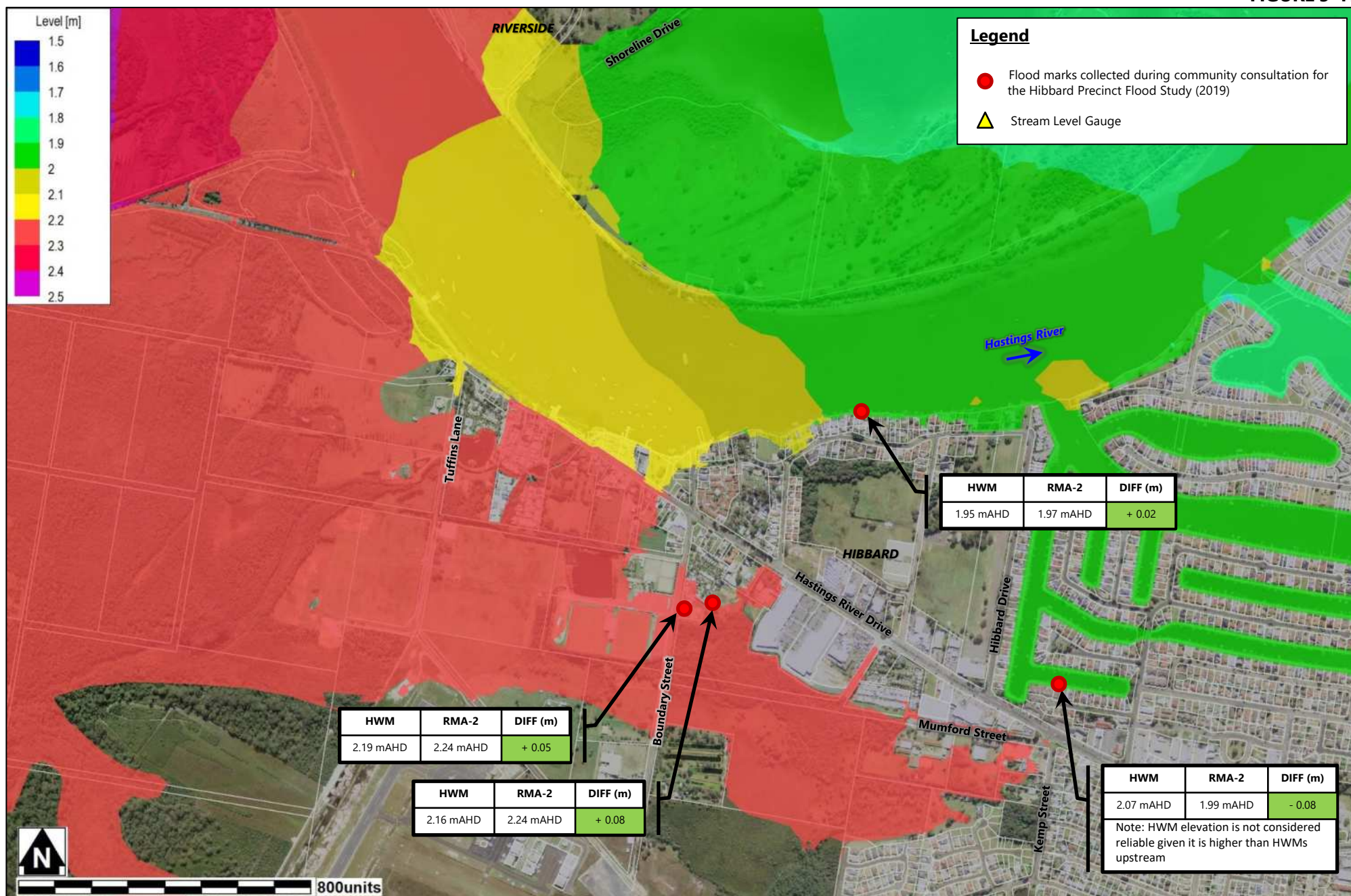
- **Figure C-5** for the Wilson River at Avenal Gauge (207014)
- **Figure C-6** for the Morton's Creek at Morton's Creek Road Gauge (207017)
- **Figure C-7** for the Pappinbarra River at Beechwood Gauge (207010)

A comparison of predicted flow rates between the recorded data, the WBNM model and the TUFLOW model is also included in **Appendix C**.

- **Figure C-8** for the Pappinbarra River at Beechwood Gauge (207010)

Overall, the TUFLOW results at the Morton's Creek gauge was able to reasonably resemble the recorded data at the gauge, in particular the shape and timing of the peaks.

FIGURE 5-17



Comparison at the Wilson River at Avenal gauge (207014) shows the TUFLOW model is overpredicting peak water levels locally by up to 1.8 metres (refer **Figure C-5**). Review of **Figure C-8** indicates that the WBNM model predicts flows that are around 25% or 400 m³/s higher than those recorded at the gauge. As the TUFLOW model is based on the flow inputs derived from the WBNM model, it follows that the increased flows would be the cause for all, or a portion of the higher flood level.

Similarly, the results at the Pappinbarra River gauge (207010) suggest that the TUFLOW model could be underpredicting the peak flood level by up to 0.6 metres (refer **Figure C-7**). A review of **Figure C-8** shows that the WBNM hydrologic model and TUFLOW hydraulic model simulate flow hydrographs that align well, with peak flows that are within 50 m³/s of each other.

The significant difference in the magnitude of flood levels vs flows predicted by TUFLOW to those recorded at the Pappinbarra River Gauge (207010) could indicate an issue with the adopted rating curve. For example, the TUFLOW model predicts a peak flood level of 50.4 mAHD at the gauge for a peak flow of 930 m³/s. The rating curve by comparison estimates a peak flow of 605 m³/s for a recorded flood level of 51.05 mAHD. This is a significant discrepancy that would warrant further investigation.

Peak flood level mapping for the February 2013 validation event as modelled by the TUFLOW hydrologic model is presented in **Figure 5-18**.

5.5 Validation – May 2025 Flood

5.5.1 Event Overview

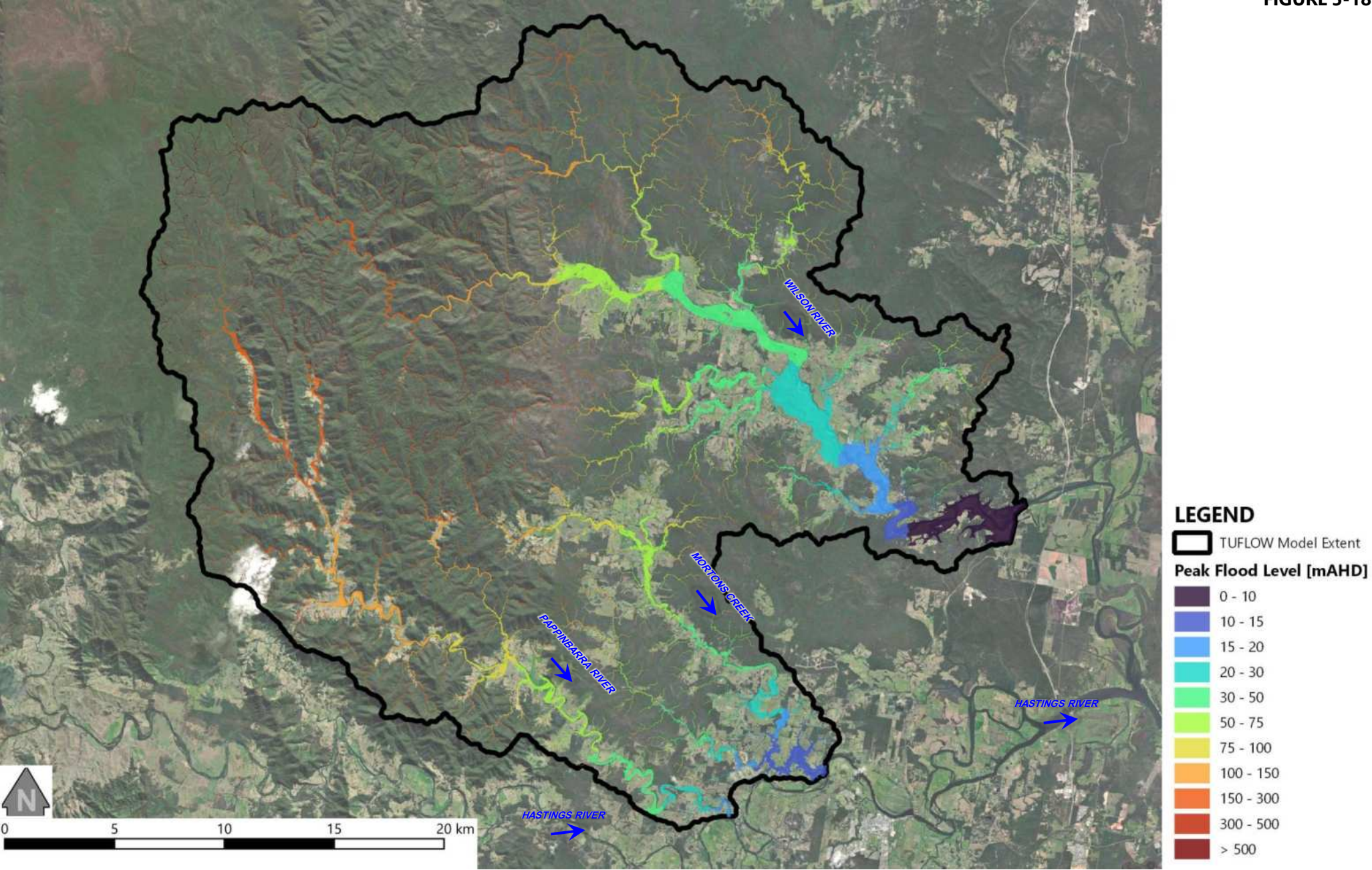
A major rain event affected significant areas of eastern Australia in mid-May 2025. The highest rainfall totals occurred in the mid north coast region of New South Wales (NSW) including the catchments of the Hastings, Manning, Wallamba and Hunter Rivers (refer **Plate 5-11**).

The Bureau of Meteorology (BOM) has provided an overview of the event in its regular monthly summaries which is titled, 'Australia in May 2025' and 'New South Wales in May 2025'. The following bullet points are drawn from these summaries and outline the key synoptic and hydrologic features of the event.

- A low-pressure trough developed off the coast of New South Wales on 18 May 2025 and combined with moist air from the Tasman Sea that was directed by a near-stationary high-pressure system. The trough brought heavy rainfall and damaging winds from the Sydney Coast to the Coffs Coast over several days, exceeding daily and multi-day rainfall records in the areas affected, including the Hastings River Catchment.
- Rainfall across the New South Wales coast in May 2025 were above to very much above average. Many sites recorded their highest May daily rainfall on record, and their highest total May rainfall on record for at least 20 years.
- Yarras (Mount Seaview) in the upper Hastings River catchment, recorded a monthly total of 926.8 mm – a record total for the site. Yarras also set a new May daily rainfall record of 316.2 mm on the 21st (previous record 161.0 mm in 1980) (refer **Plate 5-12**).
- Other stations in the Hastings River Catchment, namely, Kindee Bridge, Upper Rollands Plains and Yarrowitch also recorded a record May daily total.

The persistence of the event was a defining feature, with the majority of rainfall falling over a three-to four-day period between the 20th and 23rd May 2025. Nationally, BoM noted that the low-pressure trough that developed off the NSW coast on 18 May lingered for days, producing record-high rainfall throughout the Hunter and Mid North Coast, with multiple stations recording daily falls exceeding 200 mm.

FIGURE 5-18



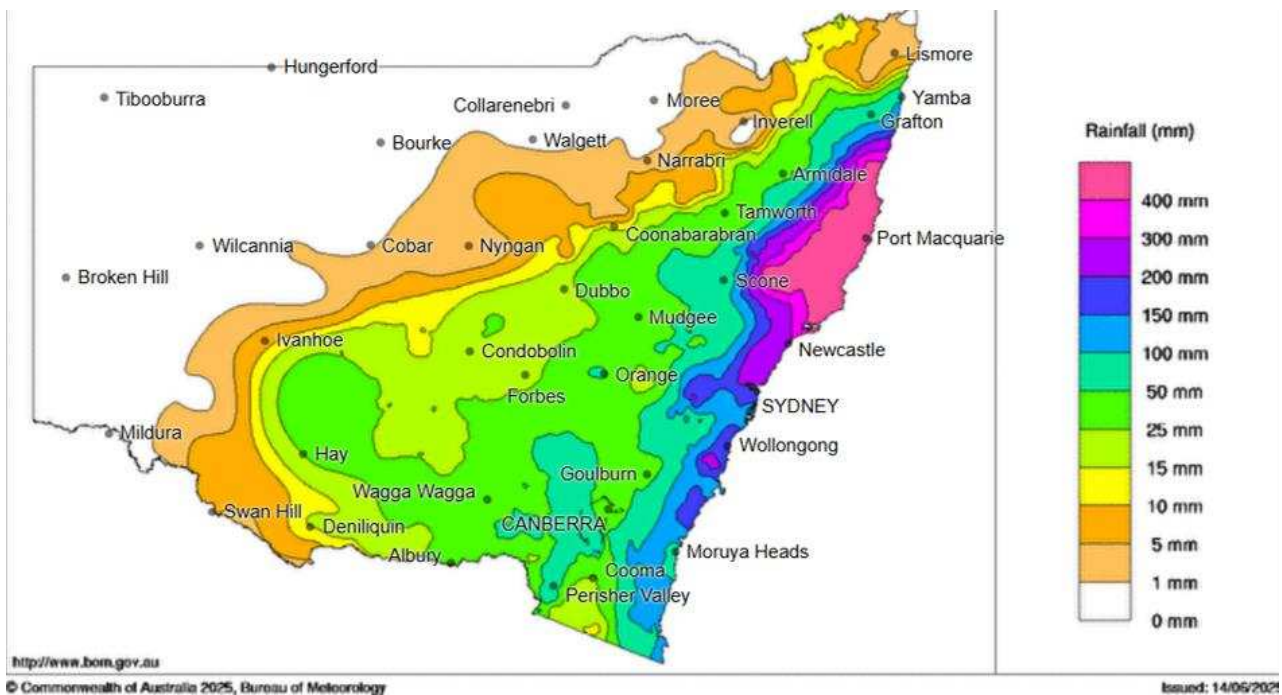


Plate 5-11 NSW rainfall totals for the week ending 25th May 2025 (Source: BOM)

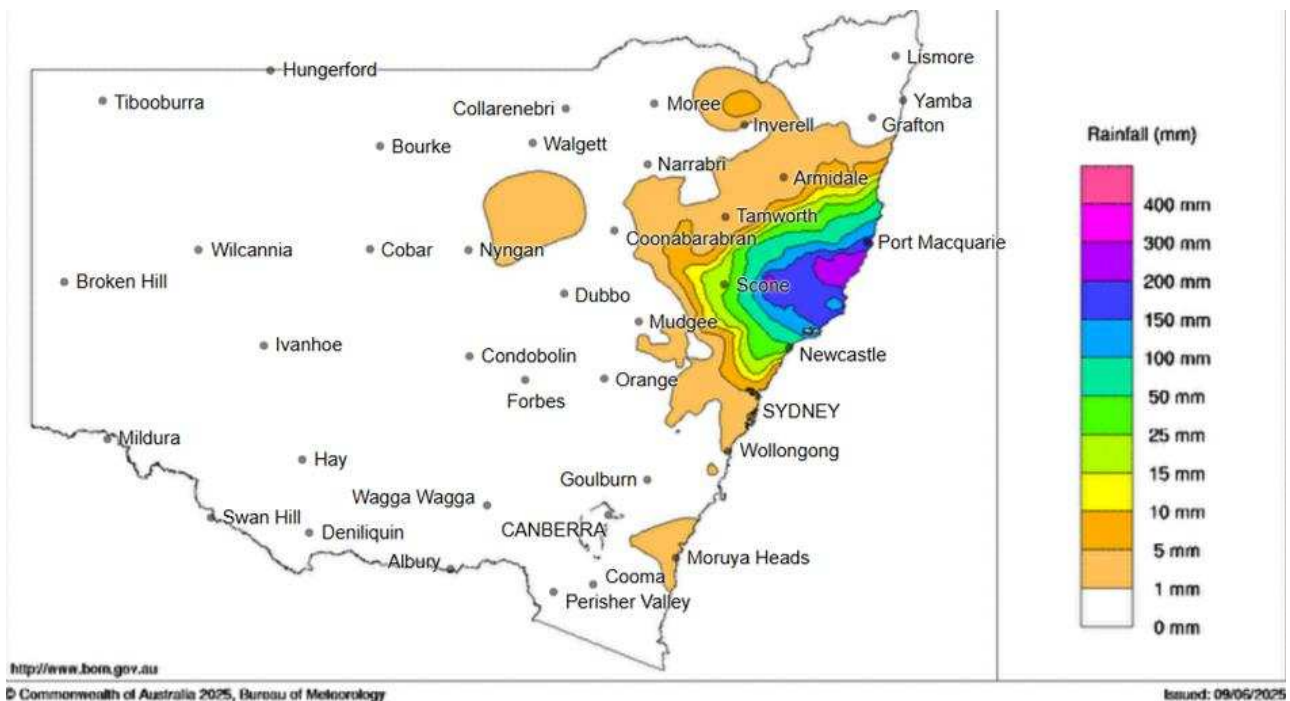


Plate 5-12 NSW rainfall totals for the 24-hour period on 20th May 2025 (Source: BOM)

5.5.2 River Level Data

Several river level gauges across the catchment were in operation during the May 2025 event. These were spread along the Hastings River and several of the major tributaries. River level data recorded during the May 2025 flood is presented in **Plate 5-13**.

The Annual Exceedance Probability (AEP) of the May 2025 event is estimated to range between a 5% and 2% AEP flood based on the recorded flood peaks at river level gauges.

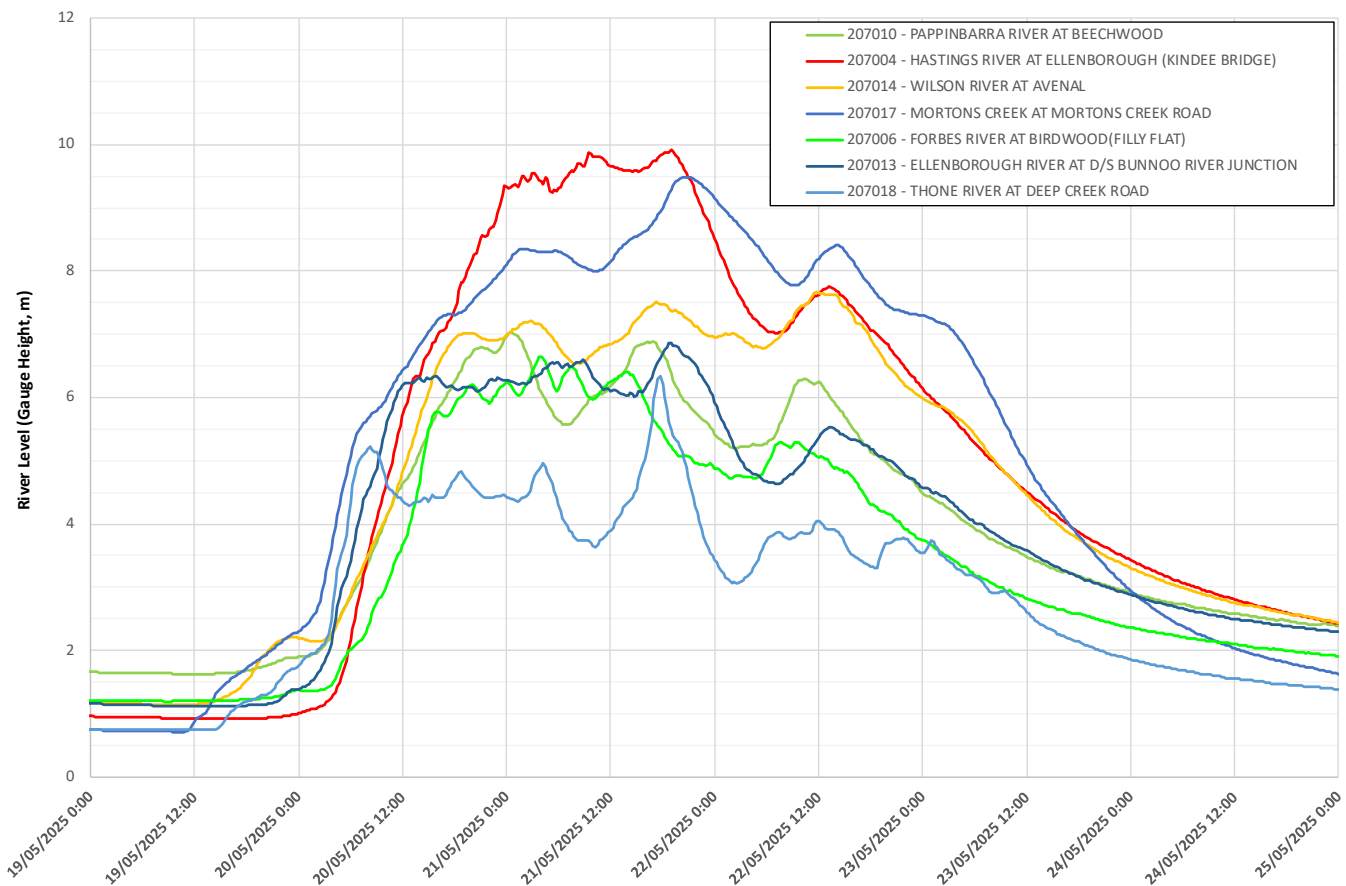


Plate 5-14 Recorded River level data for the Hastings River and Major Tributaries – May 2025

5.5.3 Hastings River WBNM Model Validation

The WBNM hydrologic model was also validated to recorded data collected for the May 2025 event. The recorded rainfall data showed a high spatial variation in rainfall across the catchment in terms of timing and total volume. Rainfall gauges to the south and east of the catchment recording significant rainfall on 18th May, compared to the north and west which did not see any substantial rainfall until midday on 19th May.

Initial simulations found that the use of rainfall data from the available gauges alone did not achieve an appropriate representation of the spatial and temporal variation of rainfall across the study area in the May 2025 storm event. To improve this, seven additional dummy gauges were added to better represent the spatial variation of rainfall in the catchment. The location of these gauges varied from within to outside of the catchment boundary. To further improve the validation, multiple rainfall records from nearby gauges were tested at these dummy gauges to try obtain a representative rainfall.

The final adopted rainfall data for the May 2025 Hastings River calibration simulation is summarised below:

- 60052 Upper Rollands Plains – *Pluviometer*
- 560011 Telegraph Point (Wilson River) – *Pluviometer*
- 560014 Port Macquarie Golf Club – *Pluviometer*
- 60139 Port Macquarie Airport AWS – *Pluviometer*
- 560020 Koree Island (Hastings River) – *Pluviometer*
- 60163 Yarras Mountain – *Pluviometer*
- 60085 Yarras (Mt Seaview) – *Pluviometer*

- 60121 Elands Black Sands Creek – *Pluviometer*
- 60161 Comboyne Public School – *Pluviometer*
- 560024 Comboyne Thone River – *Pluviometer*
- 60164 Yarrowitch (Rushbrook) – *Pluviometer*
- 560009 Wauchope (Bain Street) – *Pluviometer*
- 6016 Port Macquarie (Koala Street) – *Pluviometer*
- 60116 Kindee Bridge – *Pluviometer*

It is interesting to note that all gauges relied upon for the May 2025 event were pluviometer gauges that recorded rainfall at least at 15-minute intervals. This is different to the validation undertaken for the February 2013 and March 2021 events which relied on a combination of daily read and continuous (pluviometer) gauges (refer **Section 5.3** and **5.4**).

A cumulative rainfall plot of the adopted rainfall data for the period from 00:00 on 19th May 2025 to 00:00 on 24th May 2025 is presented in **Plate 5-14**, and the locations of these gauges are shown in **Figure 5-19**.

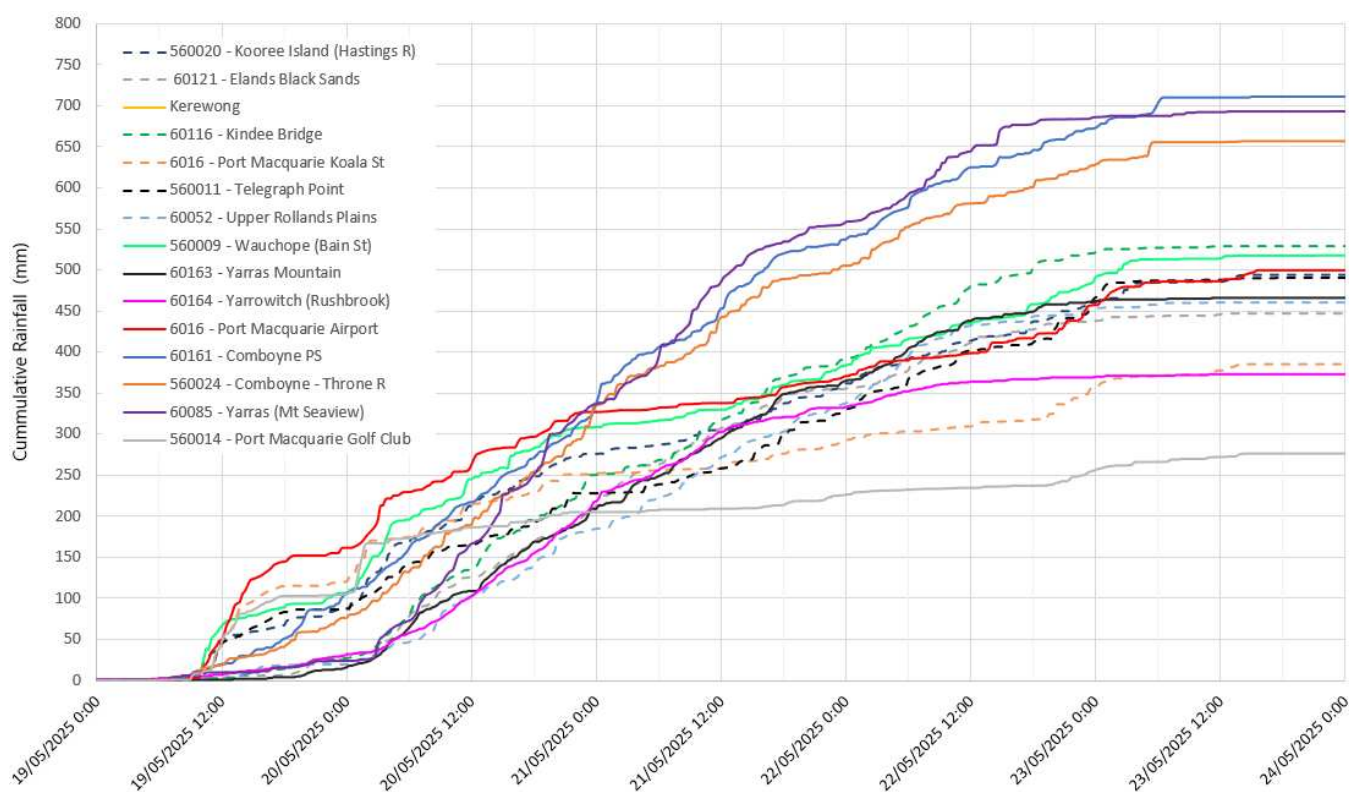


Plate 5-14 Cumulative Rainfall Plot for the May 2025 Event

The AEP of the rainfall recorded at each gauge for storm durations of 24, 36 and 48 hours is shown in **Table 5-4**. The data indicates that the gauges that recorded rainfall with the rarest AEP, that is, the rainfall with the lowest probability of occurring in any given year, were:

- Yarras (Mt Seaview) with rainfall recorded having an AEP of 1 in 500 for a 48-hour duration or over 1 in 500 for a 36-hour storm duration.
- Yarrowitch with rainfall recorded to have an AEP in the order of 1 in 200 for the 36- and 48-hour storm durations.

Rainfall for the rest of the gauges has an estimated AEP of between 20% (1 in 5) and 2% (1 in 50).

FIGURE 5-19

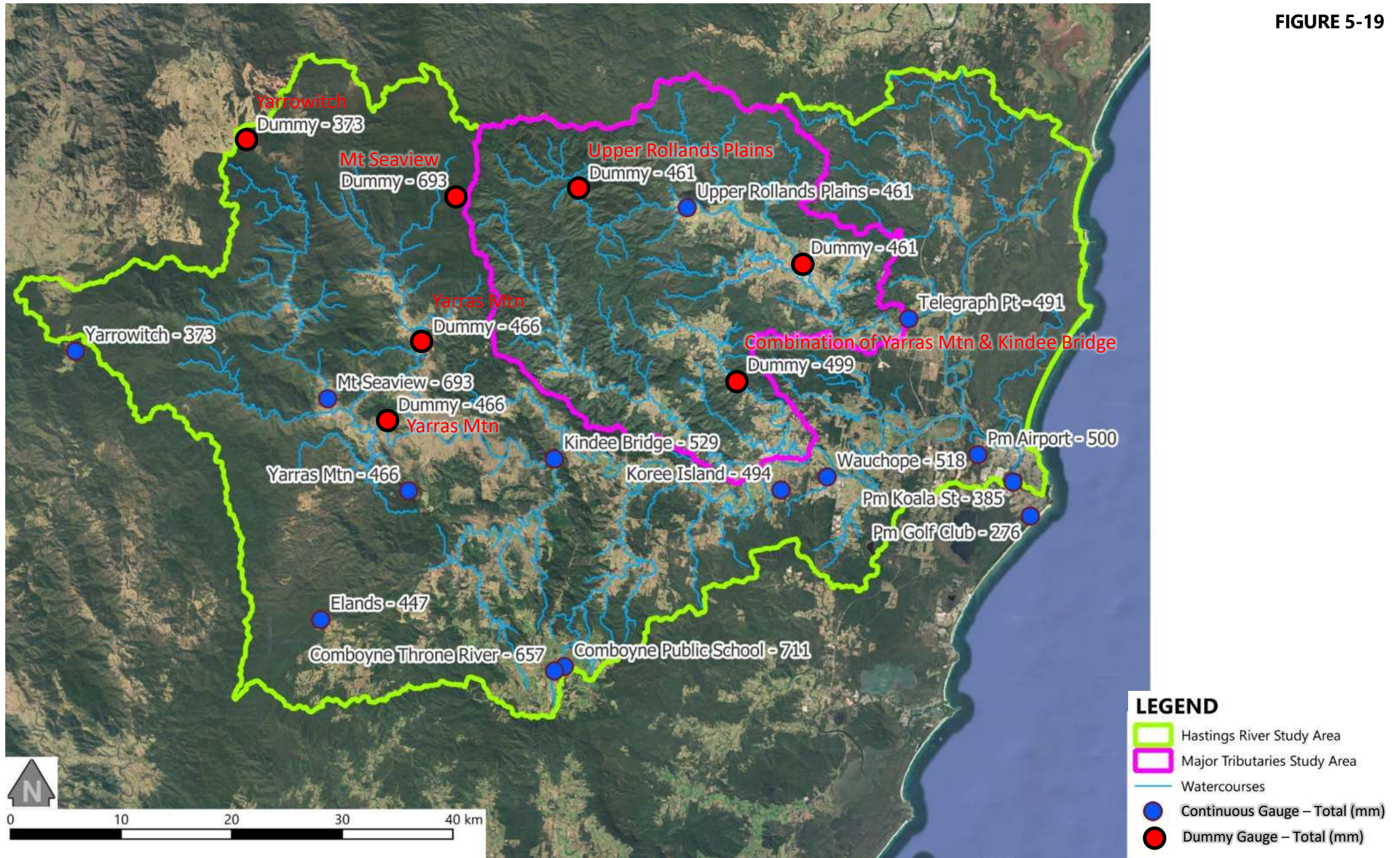


Table 5-4 Estimated AEP for the May 2025 event for various storm durations

Gauge ID and Name	Storm Duration		
	24-hr	36-hr	48-hr
6016 Port Macquarie (Koala Street)	20% AEP	10 - 20% AEP	20% AEP
560014 Port Macquarie Golf Club	< 20% AEP	< 20% AEP	< 20% AEP
60139 Port Macquarie Airport AWS	10 – 5% AEP	5% AEP	5% AEP
560011 Telegraph Point (Wilson River)	< 20% AEP	20% AEP	20% AEP
560009 Wauchope (Bain Street)	10% AEP	5% AEP	10 – 5% AEP
560020 Koree Island (Hastings River)	20% AEP	10% AEP	10% AEP
60052 Upper Rollands Plains	20% AEP	10% AEP	> 5% AEP
60161 Comboyne Public School	20 - 10% AEP	10 – 5% AEP	> 5% AEP
60116 Kindee Bridge	5% AEP	> 5% AEP	5 - 2% AEP
60121 Elands Black Sands Creek	10% AEP	10 – 5% AEP	5% AEP
60085 Yarras (Mt Seaview)	1 in 200 AEP	> 1 in 500 AEP	1 in 500 AEP
60163 Yarras Mountain	20 – 10% AEP	10 – 5% AEP	5% AEP
60164 Yarrowitch (Rushbrook)	1% AEP	1 in 200	1% – 1 in 200 AEP

Note: Red and yellow shading identifies the rainfall gauges and periods estimated to have AEP rainfall that exceeds a 1% AEP and 5% flood, respectively.

The dummy gauges and the corresponding rainfall records that were applied to the WBNM model are shown in **Figure 5-19**. Iterative modelling of different dummy gauge locations and rainfall record combinations found that the dummy gauges were required to minimise the amount of rainfall input into the model on the 19th May 2025 (refer **Plate 5-14**). This was supported when comparing simulated and recorded flow hydrographs, particularly for the Kindee Bridge and Pappinbarra River level gauges. Rainfall ‘heat maps’ downloaded from the Bureau of Meteorology (BoM) website were also used to guide the selection of dummy node locations and the selection of appropriate gauge records to apply.

The validation of the WBNM hydrologic model was completed by comparing flows against recorded flow rate data obtained from river gauges across the catchment. This was undertaken by comparing WBNM total runoff hydrographs. The final calibrated WBNM parameters adopted for simulation of the May 2025 weather event are shown in **Table 5-5**. Values adopted for the March 2021 and February 2013 events also listed. As shown, the only value modified for the May 2025 simulation was a reduction to the continuing losses from 5.0 mm/hr or 4.0 mm/hr to 2.5 mm/hr.

Table 5-5 WBNM parameters for May 2025 validation event

Parameter	Parameter Value		
	March 2021	February 2013	May 2025
Runoff lag factor 'C'	1.6	1.6	1.6
Impervious runoff lag factor 'C'	0.1	0.1	0.1
Stream routing factor 'F'	Variable (1.0 to 1.3)	Variable (1.0 to 1.3)	Variable (1.0 to 1.3)
Initial Loss (pervious)	20	20	20
Continuing Loss (pervious)	5.0	4.0	2.5
Initial Loss (effective impervious)	0	0	0
Continuing Loss (effective impervious)	0	0	0

Graphs comparing the final flow hydrographs generated for the May 2025 event using the WBNM model and those recorded at various streamflow gauges are included as **Appendix A**. Comparisons are included for the following streamflow gauges.

- **Figure A-15** for the Ellenborough River at Downstream Bunnoo River Junction (207013)
- **Figure A-16** for the Forbes River at Birdwood (Filly Flat) (207006)
- **Figure A-17** for the Hastings River at Ellenborough (Kindee Bridge) (207004)
- **Figure A-18** for the Pappinbarra River at Beechwood (207010)
- **Figure A-19** for the Thone River at Deep Creek Road (207018)
- **Figure A-20** for the Wilson River at Avenal (207014)
- **Figure A-21** for the Mortons Creek at Mortons Creek Road (207017)

It is noted that the comparison at Mortons Creek (refer **Figure A-21**) is not a correct representation of flows simulated and recorded along Mortons Creek. This is because the gauge is located near the Hastings River and as a result flood levels back-up along Mortons Creek and influence flood levels. Accordingly, only the initial rise in flows can be attributed to flow arriving from rainfall across the Mortons Creek catchment. Subsequent changes to the flow hydrograph would be heavily influenced by the Hastings River and whilst this is taken into consideration in the hydraulic modelling (refer **Sections 5.5.4** and **5.5.5**) it is not captured by the WBNM hydrologic modelling.

5.5.4 Hastings River RMA-2 Hydraulic Model Calibration

The extended RMA-2 model was validated using the inflow hydrographs extracted from the WBNM model at all upstream and local catchment inflow locations. A comparison of predicted flood level hydrographs to recorded levels at gauges located within the RMA-2 model domain are included in **Appendix B**:

- **Figure B-16** for the Hastings River at Ellenborough (Kindee Bridge) Gauge (207004)
- **Figure B-17** for the Mortons Creek at Mortons Creek Road Gauge (207017)
- **Figure B-18** for the Hastings River at Wauchope Railway Bridge Gauge (60124)
- **Figure B-19** for the Hastings River at Dennis Bridge Gauge (207444)
- **Figure B-20** for the Hastings River at Settlement Point Gauge (207418)

- **Figure B-21** for the Hastings River at Port Macquarie Gauge (207420)
- **Figure B-22** for the Wilson River at Avenal Gauge (207414)
- **Figure B-23** for the Wilson River at Telegraph Point Gauge (207415), and
- **Figure B-24** for the Maria River at Mundays Lane (207406)

The validation outcomes are summarised in **Table 5-5**.

The RMA-2 model was successful in simulating peak flood levels for the May 2025 event that were on average within 0.04 metres of those recorded at the water level gauges (refer **Table 5-5**). Review of the graphs shows that the shape of the predicted flood level hydrographs was in close agreement for the majority of the gauges. The most common deviation was a rising limb that was too early or steep, an issue that was reflective of the substantial difference observed in the recorded rainfall data, with approximately half of the gauges recording at least 50-100 mm of rainfall on the 18th May 2025 compared to less than 20 mm for the other gauges. This variability was particularly evident in the validation plots in the upper reaches of the catchment, i.e., refer validation plots for Hastings River at Kindee Bridge, Pappinbarra River, Wauchope Railway Bridge and Wilson River at Avenal.

Table 5-5 Overview of RMA-2 Simulated and Recorded Flood Level Differences for the May 2025 event and for available water level gauges

Figure ID	Gauge Name	Recorded Flood Level (mAHD)	RMA-2 Simulated Flood level (mAHD)	Difference (m)
B-16	Hastings River at Kindee Bridge	54.90	55.21	+0.31
B-17	Mortons Creek at Mortons Creek Road	10.97	10.82	-0.15
B-18	Hastings River at Wauchope Railway Bridge	7.35	7.32	-0.03
B-19	Hastings River downstream Dennis Bridge	3.19	3.20	+0.01
B-20	Hastings River at Settlement Point	2.09	2.13	+0.04
B-21	Hastings River at Entrance	1.79	1.73	-0.05
B-22	Wilson River at Avenal	7.67	8.13	+0.47
B-23	Wilson River at Telegraph Point	3.16	3.03	-0.13
B-24	Maria River at Mundays Lane	2.98	2.91	-0.07
Average Difference (m) -				+0.04
Median Difference (m) -				-0.03

Note: Green, yellow and red shading denotes differences between simulated and recorded flood levels of up to 0.20 metres, 0.40 metres and above, respectively.

The validation was not as successful for the Kindee Bridge and Avenal gauges. At the Hastings River at Kindee Bridge gauge the challenges were attributed to the high spatial variability of rainfall recorded for the

gauges located upstream, combined with a lack of gauges upstream and to the north across the Forbes River catchment. With the exception of the steeper rise in levels and higher peak reached, simulated levels in the later stages match very closely to the recorded data (refer **Figure B-16** and **Table 5-5**).

Similar issues are attributed to the poor validation at the Avenal gauge, with only one rainfall gauge located upstream across the Wilson River catchment. It is therefore highly possible that the rainfall temporal pattern and totals for this catchment are not accurately represented in the hydrologic model (refer **Figure B-22** and **Table 5-5**). The differences between simulated and recorded flood levels are therefore attributed to difficulties with the input flows rather than the RMA-2 model itself.

5.5.5 Major Tributaries Hydraulic TUFLOW Model Calibration

To validate the TUFLOW hydraulic model to the May 2025 event, the inflow hydrographs from the WBNM model were applied across the model domain. Relevant downstream boundary conditions were also applied at the base of all three tributaries according to flood levels extracted from the RMA-2 model (refer **Section 5.5.4**). This was particularly important for the Mortons Creek at Mortons Creek Road gauge which records peak flood levels that are heavily influenced by flood levels along the Hastings River which back-up along Mortons Creek. The Wilson River at Avenal and Pappinbarra gauges are not impacted by flood levels at the downstream boundaries.

The hydraulic model calibration was then assessed through the comparison of flood levels simulated by the TUFLOW model against available recorded water levels at WaterNSW gauges located along each of the tributaries. A comparison of predicted flood level hydrographs to recorded levels at gauges located within the TUFLOW model domain are included in **Appendix C**.

- **Figure C-9** for the Wilson River at Avenal Gauge (207014)
- **Figure C-10** for the Morton's Creek at Morton's Creek Road Gauge (207017)
- **Figure C-11** for the Pappinbarra River at Beechwood Gauge (207010)

Flood Level hydrographs predicted by the TUFLOW hydraulic model for the May 2025 event are in good agreement to the shape and timing of the peaks recorded at the three gauges. The TUFLOW model was not able to replicate the peak flood levels recorded with differences of +/- 0.5 metres observed at all three gauges. As discussed in **Section 5.5.4** for the RMA-2 validation, the difficulties are largely attributed to a lack of rainfall gauges across the Pappinbarra River, Mortons Creek and upper Wilson River catchments. In that regard, for the May 2025 event there were no rainfall gauges, daily or continuous, in the Pappinbarra River and Mortons Creek catchments. The nearest gauges were at least 20 km away from the catchments. This leads to significant uncertainty in the rainfall data input into the WBNM hydrologic model, particularly when there was a high spatial variability in the recorded rainfall during the event (refer **Plate 5-14**).

It is expected that a more favorable validation could be achieved if rainfall had been recorded across the Pappinbarra River, Mortons Creek and upper Wilson River catchments.

6 Design Flood Estimation

6.1 Overview

Design flood conditions are estimated from hypothetical design rainfall events that have a particular statistical probability of occurrence. The assessment of design flood conditions presented in this report has been based on the guidance and techniques outlined in *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Geoscience Australia 2019) (ARR 2019).

The probability of a design event occurring can be expressed in terms of percentage Annual Exceedance Probability (AEP) and provides a measure of the relative frequency and magnitude of the flood event. Flood conditions for the 5%, 2%, 1%, 1 in 200, 1 in 500 and 1 in 2000 AEP design events have been investigated in this study along with the Probable Maximum Flood (PMF).

It is important to note that the adoption of ARR 2019 is new for the Hastings River catchment with previous catchment wide flood studies having been based on Australian Rainfall and Runoff 1987 (ARR 1987). These studies include:

- 'Hastings River Flood Study' (August 2006)
- 'Hastings River Floodplain Risk Management Study' (2012)
- 'Hastings River Floodplain Risk Management Plan' (2014)
- 'Hastings River Flood Study Update (Climate Change Assessment)' (2018)
- 'Hibbard Precinct Flood Study' (2019)

In addition to ARR 2019, this study varies to those above based on:

- Inclusion of the 1 in 500 and 1 in 2000 AEP design events.
- Simulation of the Probable Maximum Flood (PMF) in lieu of an 'extreme' event. The extreme flood is based on the adoption of inflows that are three (3) times those determined for the 1% AEP flood.
- Extension of the modelling domain to include a greater length of the Hastings and Wilson Rivers, Maria River, and Connection Creek and the Pappinbarra River and Morton's Creek (refer **Section 4.4.2**).

An important component of the ARR 2019 guidelines is the recommendation that design event flood hydrology be based on observed data and Flood Frequency Analysis (FFA) where possible and available. To facilitate this, the FFA previously prepared for the Hastings River at Ellenborough (Kindee Bridge) Gauge (207004) as part of the 2006 Hastings River Flood Study was updated to include recent annual maxima records up to 2024. This was important to ensure that the validation of design flows against FFA was up to date and included the recent and sizeable March 2021 flood event.

6.2 Updated Flood Frequency Analysis

6.2.1 General

Book 3 Chapter 2 of ARR 2019 describes FFA as 'procedures that use recorded and related flood data to identify underlying probability model of flood peaks, at a particular location in the catchment, which can then be used to perform risk-based design and flood risk assessment, while providing input to regional flood estimation methods.

An overview of the available FFA for the Hastings River catchment is provided in **Table 6-1**. As shown, FFA was originally prepared for the Hastings River at Ellenborough (Kindee Bridge) Gauge (207004) and the Hastings River at Wauchope Railway Bridge Gauge (207401) as part of the 2006 Flood Study. Additional FFA was subsequently completed for other gauges as part of the 'Review of ARR Design Inputs for NSW' (OEH, February 2019) project and further updated by WMAwater and uploaded to the <https://ffa.wmawater.com.au/> portal.

Table 6-1 Overview of FFA Available within Catchment

Gauge	Source	Period of Data	Type
Hastings River at Ellenborough (Kindee) Bridge (207004)	'Hastings River Flood Study' (2006)	1945-2003 (58 years)	Flows
	https://ffa.wmawater.com.au/	1972-2019 (47 years)	
Hastings River at Wauchope Railway Bridge Gauge (207401)	'Hastings River Flood Study' (2006)	1870-2004 (134 years) [^]	Levels
Wilson River at Avenal (207014)	'Review of ARR Design Inputs for NSW' (2019)	1984-2019 (35 years)	Flows
	https://ffa.wmawater.com.au/	1985-2020 (35 years)	
Ellenborough River D/S Bunnoo River Junction (207013)	'Review of ARR Design Inputs for NSW' (2019)	1975-2019 (44 years)	Flows
	https://ffa.wmawater.com.au/	1975-2019 (44 years)	
Hastings at Mount Seaview (207015)	'Review of ARR Design Inputs for NSW' (2019)	1984-2019 (35 years)	Flows
	https://ffa.wmawater.com.au/	1984-2020 (36 years)	
Birdwood (Lilly Flat (207006)	'Review of ARR Design Inputs for NSW' (2019)	1971-2019 (48 years)	Flows
	https://ffa.wmawater.com.au/	1971-2019 (48 years)	

[^] Period of data includes continuous gauge records between 1985 and 2004. Prior to 1985 only anecdotal records of major flood events are available.

Note: Shading indicates whether the FFA was recommended for use. Where gauges are shaded red, these were often not recommended for use due to a 'high negative skew'.

As shown in **Table 6-1**, many of the FFA prepared following the 'Review of ARR Design Inputs for NSW' (2019) project were not recommended for use in the estimation of design flood flows. This was due to a high negative skew in the FFA curve that results in only very small increases in flow magnitudes for floods rarer than a 1% AEP event.

As discussed in the 2016 Flood Study, the stage FFA for the Wauchope Railway Bridge was undertaken to account for 'the lack of adequate streamflow data for flood frequency analysis in the lower reaches of the Hastings River'.

6.2.2 FFA Updates

Hastings River at Ellenborough (Kindee Bridge) gauge (207004)

The FFA for the Hastings at Ellenborough (Kindee Bridge) gauge was updated based on the use of the following three annual data series:

- Continuous annual series from 1972 to present, obtained from WaterNSW.
- Continuous annual series between 1945 to 1971, obtained from the 2006 report (Table 7).
- Zero flood events above a threshold of the 1968 peak flood flow in the period from 1828 to 1944.

The above resulted in the input of 80 annual data points, and 117 years of record with floods no larger than the 1968 event.

The flood frequency analysis was undertaken using the FLIKE software with a Log-Pearson III distribution found to produce the best fit for the data. Low flow data points were censored according to the Multiple Grubbs-Beck low outlier test. This resulted in the censorship of 27 annual series points, all with flows estimated to be below 546 m³/s. The FFA results are summarised in **Table 6-2** and the frequency distribution curve is presented in **Plate 6-1**.

As shown in **Table 6-2** and **Plate 6-1**, the updated FFA results in an increase to the estimate flow values for design events of around 2% AEP and above. For example, the updated FFA estimates a peak flow for the 1% AEP event that is 299 m³/s higher than the 2020 FFA, i.e., a 7% increase. For lower events such as the 5% AEP flood, the 2024 FFA predicts a small decrease in flow of approximately 2%.

Table 6-2 Hastings River at Ellenborough (Kindee Bridge) Updated FFA Results

Annual Exceedance Probability (AEP)	Estimated Flow Values (m ³ /s)		
	2006 Flood Study FFA	2020 FFA	Updated FFA
5% AEP	2835	3062	2995 (-67)
2% AEP	3490	3692	3800 (+108)
1% AEP	3910	4082	4381 (+299)
1 in 200 AEP	4213	4409	4936 (+527)

Ellenborough River Downstream Bunnoo River Junction gauge (207013)

The FFA for the Ellenborough River downstream Bunnoo River Junction gauge (207013) was updated based on the use of the following annual data series:

- Continuous annual series from 1975 to present, obtained from WaterNSW.

This produced 50 data points for input to the distribution analysis. Following a similar approach to the Hastings at Ellenborough gauge, the FLIKE software using a Log-Pearson III distribution gave the best fit. The Multiple Grubbs-Beck low outlier test censored 12 annual records, all with annual peak flows of less than 152 m³/s. The FFA results are summarised in **Table 6-3** and the frequency distribution curve is presented in **Plate 6-2**.

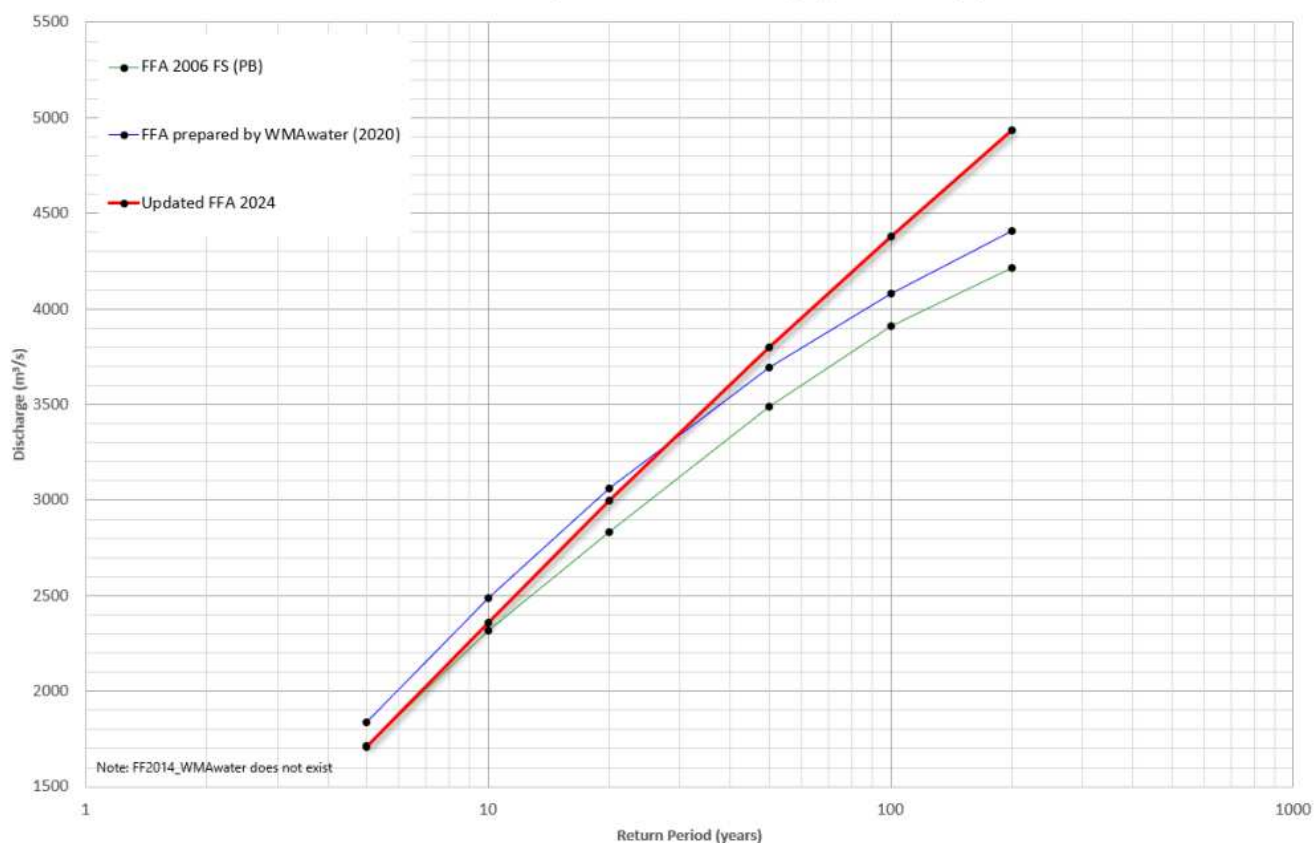


Plate 6-1 Hastings River at Ellenborough (Kindee Bridge) gauge Frequency Distribution Curve

Table 6-3 Ellenborough River downstream Bunnoo River Junction Updated FFA Results

Annual Exceedance Probability (AEP)	Estimated Flow Values (m ³ /s)		
	2006 Flood Study FFA	2020 FFA	Updated FFA
5% AEP	Not prepared	1192	1284 (+92)
2% AEP	Not prepared	1546	1697 (+151)
1% AEP	Not prepared	1811	2015 (+204)
1 in 200 AEP	Not prepared	2072	2335 (+263)

The updated FFA shows an increase to flow values for all design events ranging in magnitude between 7 and 13%.

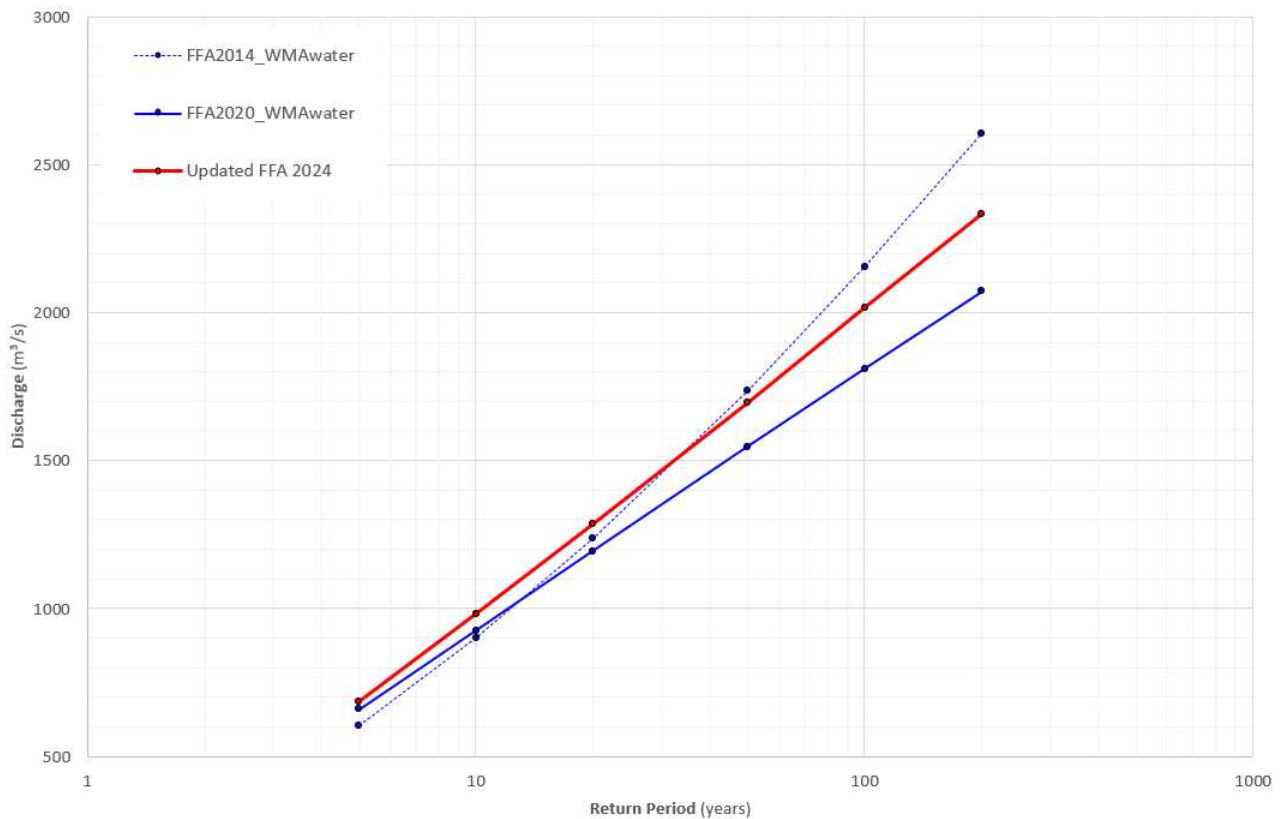


Plate 6-2 Ellenborough River downstream Bunnoo River Junction Frequency Distribution Curve

Hastings River at Wauchope Railway Bridge gauge (60124)

The FFA for the Hastings River at Wauchope Railway Bridge gauge (60124) was updated based on the use of the following three annual data series:

- Water level data reported in Table 11 of the Hastings River Flood Study (2006). This dataset includes:
 - Annual data from 1985 to 2004
 - Large event data between 1860 and 1984 (20 points)
- Zero other events above 6.0 mAHD between 1860 to 1984
- Continuous annual series from 2005 to present, obtained from BOM.

The above produced 38 data points for analysis. The FLIKE software was used, and a Log-Pearson III distribution gave the best fit. The multiple Grubbs-Beck low outlier test resulted in 23 records being censored with below peak water levels of below 5.38 mAHD.

The FFA results are summarised in **Table 6-4** and the frequency distribution curve is presented in **Plate 6-2**.

Table 6-4 Hastings River at Wauchope Railway Bridge Updated FFA Results

Annual Exceedance Probability (AEP)	Peak Flood Level (mAHD)		
	2006 Flood Study FFA	2020 FFA	Updated FFA
5% AEP	7.8	Not investigated	7.5 (-0.3)
2% AEP	8.5	Not investigated	8.5 (+0.0)
1% AEP	8.95	Not investigated	9.05 (+0.10)
1 in 200 AEP	9.1	Not investigated	9.47 (+0.37)

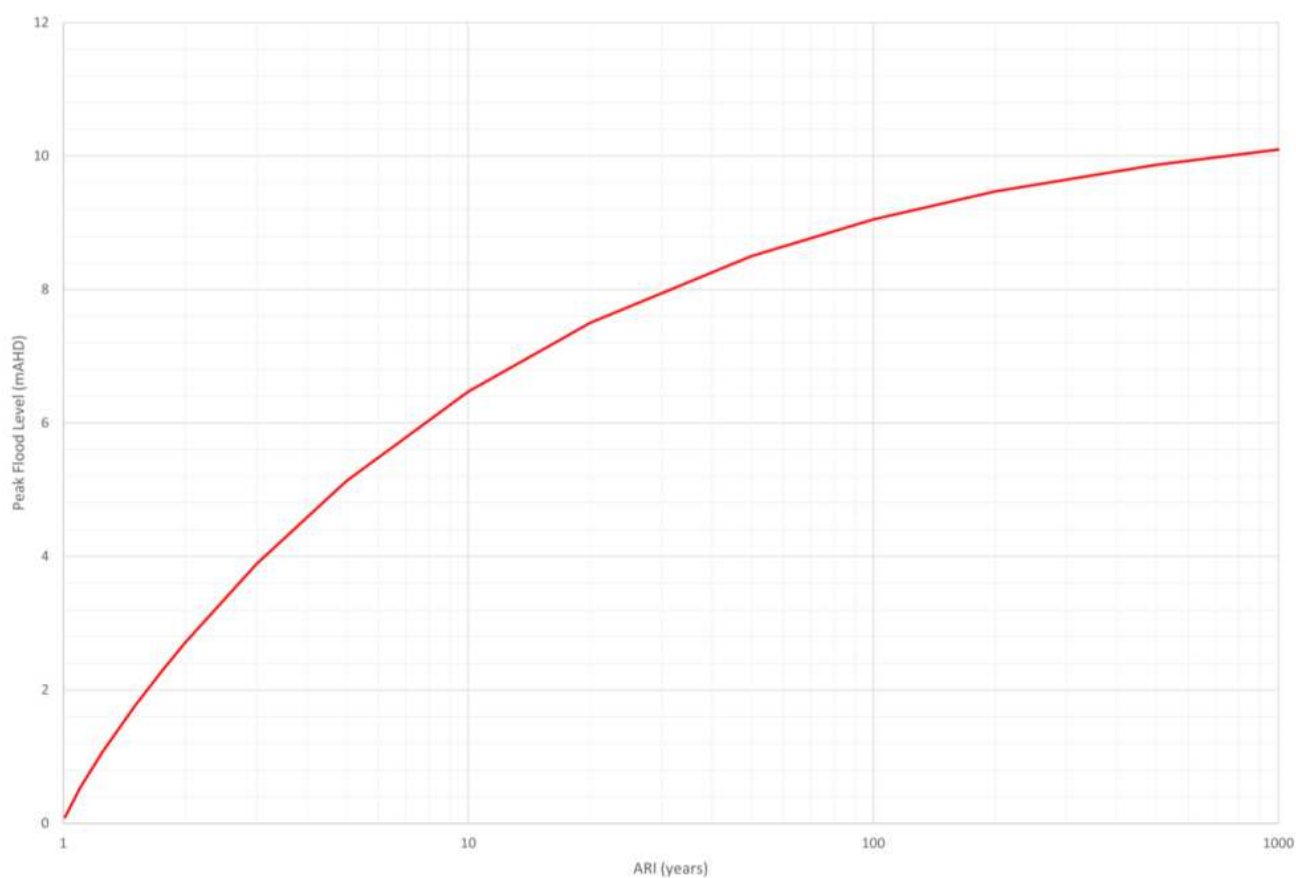


Plate 6-3 Hastings River at Wauchope Railway Bridge Frequency Distribution Curve

6.3 Design Rainfall

6.3.1 Design Rainfall Depths

Design rainfall depths for the 5% AEP to 1 in 2000 AEP design events were obtained online from the Australian Rainfall and Runoff Data Hub. As discussed in the following section, IFD data sampled from nine locations was used to resolve spatial variation in design rainfall depths across the catchment.

6.3.2 Design Rainfall Spatial Pattern

As discussed in Book 2, Chapter 6 of ARR 2019, it is recommended that a non-uniform spatial pattern be applied to catchments with an area of greater than 1000 km². This is required to replicate the systematic spatial variability in rainfall determined from the Topographic Adjustment Factor (TAF) derived from the PMP method. This is particularly important in the Hastings River catchment as the rainfall is known to change significantly across the catchment, owing to the large catchment area and orographic effect of the varying terrain.

To account for the spatial variability, gridded IFD data and the TAF for GTSMR PMP calculations were reviewed to assess the variation in design rainfall depths across the study area. Based on an agreement in spatial variation between the two methods, variation by IFD grid was adopted as it was considered that the use of IFD data from 9 locations – in a 3 x 3 grid across the catchment – was appropriate to replicate the spatial variation observed in the grids.

6.3.3 Design Rainfall Temporal Patterns

To estimate a design flood hydrograph a temporal pattern must be applied to the design rainfall depths to describe how rain falls over time. Traditionally a single burst temporal pattern has been applied for each design rainfall event and duration; however, this approach has been questioned as a wide variety of temporal patterns is possible.

The ARR 2019 guidelines now recommend that 'ensembles' of 10 temporal rainfall patterns that have been derived to represent variability in observed patterns be analysed for each design storm magnitude and duration.

ARR 2019 states that the 10 patterns within an ensemble provide a range of plausible answers, with testing demonstrating that peak flows for a number of the patterns tend to cluster around the mean for most catchments. For the purposes of selecting a single representative design rainfall pattern, the average of the 10 resulting peak flows is taken to be the actual peak design flood flow at a given location, and the temporal pattern resulting in a peak flow nearest to (but not more than 5% less than) this average would typically be adopted to determine the design flood hydrograph.

6.3.4 Rainfall Losses

The term 'rainfall losses' refers to precipitation that does not contribute to direct runoff. During a storm such losses occur primarily due to the processes of interception by vegetation, and infiltration into the soil. The initial loss-continuing loss (IL-CL) approach is typically used in Australia to account for losses in the rainfall-runoff process and has been adopted in this study.

Initial losses for pervious surfaces adopted in this study are in line with the approach documented in NSW specific advice provided by NSW Department of Climate Change, Energy the Environment and Water (NSW DCCEEW). This approach adjusts the average calibration losses (20mm) with the Probability Neutral Burst Initial Losses (PNBIL). This results in a variable initial loss depending on AEP and duration of storm.

A final continuing loss rate of 4.7 mm/hr was adopted which aligned with the value adopted for the March 2021 calibration and which resulted in a good fit to the FFA prepared for the Hastings at Ellenborough gauge (refer **Section 6.2**).

For the PMF, initial and continuing loss rates of 0 mm and 1 mm/hr were adopted in accordance with guidance outlined in Book 8 Chapter 6 of ARR 2019.

6.3.5 Assessment of Critical Storm Duration and Temporal Patterns

Critical storm duration refers to the duration of design storm that will result in the highest peak flood flows or levels at a particular location. The critical duration is influenced by various factors including upstream catchment area and may vary between locations of interest throughout a catchment or study area. With the introduction of ARR 2019 a representative temporal pattern must also be identified which produces a peak flow closest to but not less than the design peak flow (*that being the average of peak flows from an ensemble set of 10 temporal patterns*).

For the purposes of this study, definition of design flood conditions is required at various locations of interest which have varying catchment sizes and properties (e.g. slope, degree of urbanisation, stream type and size etc.), and therefore may have varying critical storm durations and applicable temporal rainfall patterns.

Given the run time of the developed RMA-2 and TUFLOW hydraulic models, it is not practical to simulate multiple temporal patterns for multiple durations for each design flood (i.e. AEP). A more practical approach was thus adopted, as follows:

- The WBNM hydrologic model was used to determine critical storm durations, associated temporal patterns and average peak design flows at 35 assessment locations as shown in **Figure 6-1**.
- From this a number of critical storm durations and associated temporal patterns of interest were identified for further investigation for each flood magnitude
- From the investigated storms, two durations were selected for each flood magnitude that in combination provided the overall best match to 'average peak design flows' across the assessment locations.

A summary of the selected critical storm durations and temporal patterns for each design event are presented in **Table 6-5**.

FIGURE 6-1

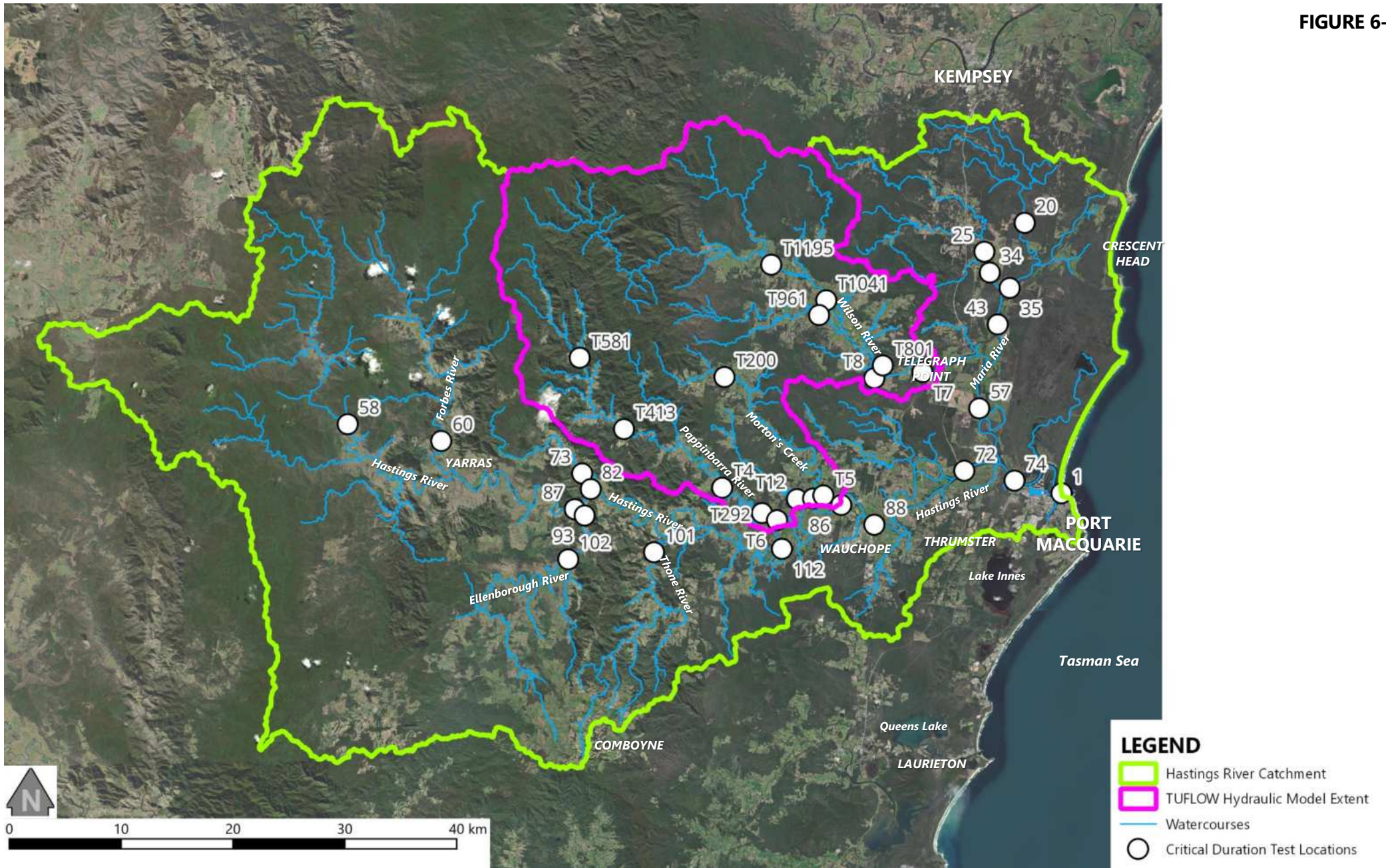


Table 6-5 Critical design storm durations and selected representative temporal patterns

Design Event	Selected critical storm durations and representative temporal patterns		
	Critical Duration (min)	Pattern Set	'Average' Pattern No.
5% AEP	1080	East Coast South – 5000 km ²	145
	2880	East Coast South - 5000 km ²	415
2% AEP	1080	East Coast South - 5000 km ²	145
	2880	East Coast South - 5000 km ²	415
1% AEP	1080	East Coast South - 5000 km ²	145
	2880	East Coast South - 5000 km ²	415
1 in 200 AEP	1080	East Coast South - 5000 km ²	145
	2880	East Coast South - 5000 km ²	415
1 in 500 AEP	1080	East Coast South - 5000 km ²	145
	2880	East Coast South - 5000 km ²	415
1 in 2000 AEP	1080	East Coast South - 5000 km ²	145
	2880	East Coast South - 5000 km ²	415
PMF	1440	Temporal Pattern for Standard area of 2500 km ² from 'Guidebook of the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method' (BoM, 2005)	

A comparison of peak design flood flows from the selected storm duration and temporal pattern combinations above with the average peak flow from the temporal pattern ensemble at each site are presented for the 5% AEP to 1 in 2000 AEP design events in **Table 6-6** to **Table 6-11**. The results presented are based on a 'no blockages' scenario.

Resulting peak flood flows are generally comparable to the averaged peak flood flows, within a range of percentage difference that is typical of the ARR 2019 temporal pattern ensemble approach (i.e. 5 to 10%). It is considered that the selected storm durations and temporal patterns are the most suitable of those available to provide an appropriate balance of peak design flood flows across all assessment locations, and therefore that the selected design rainfall hyetographs and parameters are appropriate for determining design flood hydrographs for the study catchments.

There was a focus to include a longer duration event to account for the significant volume of flood storage that exists in the lower reaches of the study area. These large storages have the potential to absorb a significant portion of a short duration event leading to lower peak flood levels. Longer duration storms also typically result in higher flows for large catchment due to the closer relative timing between main channel and tributary flows.

As with a comparison with 35 locations, there is never a complete agreement, and the aim of the critical duration analysis is to determine the most appropriate duration and temporal pattern to demonstrate the average peak flow.

It is considered that the selected storm durations and temporal patterns are the most suitable of those available to simulate an appropriate balance of peak design flood flows across all assessment locations. The selected design rainfall hyetographs and parameters are therefore considered appropriate for determining design flood hydrographs for the study catchments.

TABLE 6-3

Location	WBNM Subarea	Upstream Catchment Area (km²)	All Durations, All Patterns					Selected Durations and Patterns 5% AEP 1080min TP 145 + 5% AEP 2880min TP 415			
			Critical Duration (min)	Averaged Peak Flow (m³/s)	'Average' Temporal Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg) /Avg	Duration (min)	Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg)/Avg
Ocean Outlet	1	3695	1440	6301	239	6368.0	1.1%	1080	145	6461	2.5%
Thone River Gauge	101	125	720	463	51	474.5	2.4%	1080	145	524.0	13.1%
Ellenborough River Gauge	102	498	1440	1144	239	1161.7	1.5%	1080	145	1223.4	6.9%
Hastings River before Pappinbarra	112	1914	1440	2997	239	2997.8	0.0%	1080	145	3000.9	0.1%
Maria River RMA inflow	20	86	720	322	51	328.4	2.0%	1080	145	326.7	1.5%
Pipers Creek RMA inflow	25	122	720	402	51	412.2	2.5%	1080	145	421.2	4.7%
Pipers Creek	34	135	720	443	51	454.1	2.6%	1080	145	462.6	4.5%
Upper Maria River	35	211	720	706	60	716.6	1.6%	1080	145	720.5	2.1%
Original Maria River RMA inflow	43	369	720	1155	60	1167.4	1.1%	1080	145	1190.5	3.1%
Maria River d/s Wilson River	57	974	720	2901	51	2878.3	-0.8%	1080	145	2967.7	2.3%
Hastings River @ Mt Seaview Gauge	58	348	720	433	60	432.4	-0.1%	1080	145	457.7	5.7%
Forbes R Gauge	60	334	720	640	51	641.8	0.2%	1080	145	698.5	9.1%
Hastings River - Dennis Bridge	72	2556	1440	3942	234	3949.6	0.2%	1080	145	3936.7	-0.1%
Kindee Creek RMA inflow	73	46.2	720	146	51	147.2	0.6%	1080	145	156.0	6.6%
Hastings Maria Confluence	74	3606	1440	6215	239	6294.7	0.7%	1080	145	6396.0	2.9%
Hastings @ Ellenborough Gauge	82	1582	1440	2559	239	2577.4	1.0%	1080	145	2646.3	3.4%
Bain Bridge - Previous RMA inflow	86	2197	1440	3563	239	3598.6	1.2%	1080	145	3619.8	1.6%
Hastings River RMA inflow	87	1051	1440	1409	239	1425.5	1.1%	1080	145	1457.7	3.4%
Wauchope Rail Bridge	88	2358	1440	3803	239	3845.6	0.9%	1080	145	3845.4	1.1%
Ellenborough River RMA inflow	93	526	1440	1174	239	1183.9	-0.2%	1080	145	1234.0	5.1%
Mid Wilson River	T1041	352	720	1270	60	1267.1	-0.1%	1080	145	1327.0	4.5%
Upper Wilson River	T1195	269	720	984	60	983.6	1.1%	1080	145	1032.9	5.0%
Frazers Creek upstream of Beechwood	T12	32.6	720	133	51	134.1	0.9%	1080	145	138.2	4.2%
Mortons Creek RMA inflow	T14	133	720	481	51	485.8	4.3%	1080	145	495.1	2.9%
Upper Morton's Creek	T200	27.4	720	114	55	119.2	-1.5%	1080	145	118.2	3.5%
Lower Pappinbarra River	T292	247	720	771	51	760.2	-0.8%	1080	145	791.0	2.5%
Pappinbarra River Gauge	T4	238	720	767	51	760.4	0.4%	1080	145	790.2	3.1%
Mid Pappinbarra River	T413	144	720	515	60	517.3	0.7%	1080	145	540.4	4.8%
Mortons Creek Gauge	T5	135	720	484	51	486.7	1.0%	1080	145	496.7	2.7%
Upper Pappinbarra River	T581	84.6	720	331	51	333.9	-1.7%	1080	145	348.6	5.4%
Pappinbarra River RMA inflow	T6	251	720	773	51	759.5	0.5%	1080	145	790.7	2.3%
Telegraph Point Rail Bridge	T7	522	720	1816	51	1825.7	-0.4%	1080	145	1880.8	3.6%
Wilson @ Avenal Gauge	T8	509	720	1806	60	1799.3	-0.2%	1080	145	1878.2	4.0%
Wilson River RMA inflow	T801	503	720	1795	60	1791.2	1.5%	1080	145	1867.9	4.1%
Bril Bril Creek	T961	68.8	720	273	51	277.4	3.7%	1080	145	284.5	4.1%

TABLE 6-4

Location	WBNM Subarea	Upstream Catchment Area (km²)	All Durations, All Patterns					Selected Durations and Patterns 2% AEP 1080min TP 145 + 2% AEP 2880min TP 415			
			Critical Duration (min)	Averaged Peak Flow (m³/s)	'Average' Temporal Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg) /Avg	Duration (min)	Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg)/Avg
Ocean Outlet	1	3695	1440	8578	239	8638.8	0.7%	1080	145	8823	2.9%
Thone River Gauge	101	125	720	597	51	612.2	2.5%	1080	145	671.4	12.4%
Ellenborough River Gauge	102	498	1440	1523	239	1551.4	1.9%	1080	145	1641.8	7.8%
Hastings River before Pappinbarra	112	1914	1440	4133	239	4125.8	-0.2%	1080	145	4165.2	0.8%
Maria River RMA inflow	20	86	720	415	51	422.5	1.9%	1080	145	420.9	1.5%
Pipers Creek RMA inflow	25	122	720	523	51	535.9	2.5%	1080	145	546.8	4.6%
Pipers Creek	34	135	720	575	51	590.3	2.6%	1080	145	601.1	4.5%
Upper Maria River	35	211	720	920	60	937.6	1.9%	1080	145	939.3	2.1%
Original Maria River RMA inflow	43	369	720	1513	60	1533.0	1.3%	1080	145	1562.1	3.2%
Maria River d/s Wilson River	57	974	720	3817	51	3797.9	-0.5%	1080	145	3895.2	2.0%
Hastings River @ Mt Seaview Gauge	58	348	720	604	60	611.2	1.3%	1080	145	650.9	7.8%
Forbes R Gauge	60	334	720	868	60	869.1	0.1%	1080	145	954.3	9.9%
Hastings River - Dennis Bridge	72	2556	1440	5430	239	5448.5	0.3%	1080	145	5444.3	0.3%
Kindee Creek RMA inflow	73	46.2	720	189	51	190.7	0.7%	1080	145	201.0	6.1%
Hastings Maria Confluence	74	3606	1440	8455	239	8534.3	0.9%	1080	145	8732.1	3.3%
Hastings @ Ellenborough Gauge	82	1582	1440	3516	239	3522.9	0.2%	1080	145	3665.3	4.2%
Bain Bridge - Previous RMA inflow	86	2197	1440	4891	239	4927.1	0.7%	1080	145	4999.0	2.2%
Hastings River RMA inflow	87	1051	1440	1979	239	1991.7	0.6%	1080	145	2056.2	3.9%
Wauchope Rail Bridge	88	2358	1440	5223	239	5266.8	0.8%	1080	145	5312.6	1.7%
Ellenborough River RMA inflow	93	526	1440	1566	239	1580.0	0.9%	1080	145	1659.3	6.0%
Mid Wilson River	T1041	352	720	1641	60	1641.8	0.1%	1080	145	1710.7	4.3%
Upper Wilson River	T1195	269	720	1270	60	1273.6	0.3%	1080	145	1329.2	4.6%
Frazers Creek upstream of Beechwood	T12	32.6	720	169	51	170.5	0.7%	1080	145	175.5	3.6%
Mortons Creek RMA inflow	T14	133	720	622	51	630.0	1.3%	1080	145	638.0	2.6%
Upper Morton's Creek	T200	27.4	720	145	55	152.4	5.2%	1080	145	148.7	2.6%
Lower Pappinbarra River	T292	247	720	1007	51	994.9	-1.2%	1080	145	1029.6	2.3%
Pappinbarra River Gauge	T4	238	720	998	51	994.0	-0.4%	1080	145	1027.4	3.0%
Mid Pappinbarra River	T413	144	720	664	60	668.5	0.7%	1080	145	693.6	4.5%
Mortons Creek Gauge	T5	135	720	625	51	631.7	1.1%	1080	145	641.0	2.6%
Upper Pappinbarra River	T581	84.6	720	423	51	425.0	0.5%	1080	145	443.7	4.9%
Pappinbarra River RMA inflow	T6	251	720	1009	51	994.4	-1.5%	1080	145	1029.6	2.0%
Telegraph Point Rail Bridge	T7	522	720	2356	51	2372.8	0.7%	1080	145	2433.7	3.3%
Wilson @ Avenal Gauge	T8	509	720	2340	60	2335.3	-0.2%	1080	145	2426.4	3.7%
Wilson River RMA inflow	T801	503	720	2324	60	2324.3	0.0%	1080	145	2411.8	3.8%
Bril Bril Creek	T961	68.8	720	350	51	353.9	1.0%	1080	145	362.6	3.5%

TABLE 6-5

Location	WBNM Subarea	Upstream Catchment Area (km²)	All Durations, All Patterns					Selected Durations and Patterns 1% AEP 1080min TP 145 + 1% AEP 2880min TP 415			
			Critical Duration (min)	Averaged Peak Flow (m³/s)	'Average' Temporal Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg) /Avg	Duration (min)	Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg)/Avg
Ocean Outlet	1	3695	1440	10448	239	10514.1	0.6%	1080	145	10813	3.5%
Thone River Gauge	101	125	720	707	51	723.9	2.4%	1080	145	786.2	11.2%
Ellenborough River Gauge	102	498	1440	1829	239	1862.6	1.8%	1080	145	1976.3	8.1%
Hastings River before Pappinbarra	112	1914	1440	5064	239	5052.5	-0.2%	1080	145	5136.4	1.4%
Maria River RMA inflow	20	86	720	493	51	501.0	1.6%	1080	145	495.4	0.5%
Pipers Creek RMA inflow	25	122	720	626	51	640.2	2.2%	1080	145	646.5	3.2%
Pipers Creek	34	135	720	689	51	705.0	2.4%	1080	145	711.1	3.3%
Upper Maria River	35	211	720	1104	60	1123.3	1.8%	1080	145	1112.7	0.8%
Original Maria River RMA inflow	43	369	720	1821	60	1841.9	1.2%	1080	145	1856.5	2.0%
Maria River d/s Wilson River	57	974	720	4594	51	4566.2	-0.6%	1080	145	4632.8	0.8%
Hastings River @ Mt Seaview Gauge	58	348	720	755	60	768.2	1.8%	1080	145	808.8	7.2%
Forbes R Gauge	60	334	720	1067	60	1069.8	0.2%	1080	145	1162.1	8.9%
Hastings River - Dennis Bridge	72	2556	1440	6654	239	6670.8	0.3%	1080	145	6717.0	1.0%
Kindee Creek RMA inflow	73	46.2	720	226	51	227.7	0.6%	1080	145	237.4	4.8%
Hastings Maria Confluence	74	3606	2550	10292	239	10380.6	0.9%	1080	145	10700.6	4.0%
Hastings @ Ellenborough Gauge	82	1582	1440	4293	239	4297.0	0.1%	1080	145	4506.7	5.0%
Bain Bridge - Previous RMA inflow	86	2197	1440	5977	239	6015.2	0.6%	1080	145	6153.8	3.0%
Hastings River RMA inflow	87	1051	1440	2445	239	2450.1	0.2%	1080	145	2555.6	4.5%
Wauchope Rail Bridge	88	2358	1440	6387	239	6433.1	0.7%	1080	145	6546.9	2.5%
Ellenborough River RMA inflow	93	526	1440	1882	239	1898.6	0.9%	1080	145	2002.8	6.4%
Mid Wilson River	T1041	352	720	1946	60	1946.5	0.0%	1080	145	2014.4	3.5%
Upper Wilson River	T1195	269	720	1505	60	1508.7	0.3%	1080	145	1564.0	3.9%
Frazers Creek upstream of Beechwood	T12	32.6	720	200	51	200.2	0.2%	1080	145	205.2	2.7%
Mortons Creek RMA inflow	T14	133	720	739	55	746.7	1.1%	1080	145	752.3	1.9%
Upper Morton's Creek	T200	27.4	720	170	55	180.8	6.0%	1080	145	173.1	1.5%
Lower Pappinbarra River	T292	247	720	1205	51	1191.1	-1.2%	1080	145	1223.5	1.5%
Pappinbarra River Gauge	T4	238	720	1193	51	1188.9	-0.4%	1080	145	1219.7	2.2%
Mid Pappinbarra River	T413	144	720	788	60	793.6	0.7%	1080	145	816.6	3.6%
Mortons Creek Gauge	T5	135	720	743	55	749.3	0.9%	1080	145	756.0	1.8%
Upper Pappinbarra River	T581	84.6	720	499	51	498.6	0.0%	1080	145	518.8	4.0%
Pappinbarra River RMA inflow	T6	251	720	1209	55	1195.0	-1.2%	1080	145	1223.9	1.2%
Telegraph Point Rail Bridge	T7	522	720	2802	51	2817.8	0.6%	1080	145	2871.7	2.5%
Wilson @ Avenal Gauge	T8	509	720	2780	60	2771.8	-0.3%	1080	145	2859.7	2.9%
Wilson River RMA inflow	T801	503	720	2760	60	2758.6	-0.1%	1080	145	2841.9	3.0%
Bril Bril Creek	T961	68.8	720	414	51	415.8	0.5%	1080	145	424.5	2.6%

TABLE 6-6

Location	WBNM Subarea	Upstream Catchment Area (km²)	All Durations, All Patterns					Selected Durations and Patterns 1 in 200 AEP 1080min TP 145 + 1 in 200 AEP 2880min TP 415			
			Critical Duration (min)	Averaged Peak Flow (m³/s)	'Average' Temporal Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg) /Avg	Duration (min)	Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg)/Avg
Ocean Outlet	1	3695	1440	12421	239	12498.2	0.6%	1080	145	12882	3.7%
Thone River Gauge	101	125	720	814	51	833.4	2.4%	1080	145	904.2	11.1%
Ellenborough River Gauge	102	498	1440	2143	239	2183.7	1.9%	1080	145	2320.1	8.3%
Hastings River before Pappinbarra	112	1914	1440	6043	239	6021.2	-0.4%	1080	145	6154.6	1.8%
Maria River RMA inflow	20	86	720	573	51	582.1	1.6%	1080	145	574.6	0.3%
Pipers Creek RMA inflow	25	122	720	730	51	746.1	2.2%	1080	145	752.8	3.1%
Pipers Creek	34	135	720	803	51	821.8	2.4%	1080	145	828.3	3.2%
Upper Maria River	35	211	720	1290	60	1315.8	2.0%	1080	145	1298.8	0.7%
Original Maria River RMA inflow	43	369	720	2132	60	2159.7	1.3%	1080	145	2171.2	1.8%
Maria River d/s Wilson River	57	974	720	5370	51	5344.6	-0.5%	1080	145	5417.2	0.9%
Hastings River @ Mt Seaview Gauge	58	348	720	909	60	929.9	2.3%	1080	145	985.0	8.3%
Forbes R Gauge	60	334	720	1260	60	1267.8	0.6%	1080	145	1377.4	9.3%
Hastings River - Dennis Bridge	72	2556	1440	7938	239	7949.4	0.1%	1080	145	8034.0	1.2%
Kindee Creek RMA inflow	73	46.2	720	261	51	262.5	0.6%	1080	145	273.6	4.8%
Hastings Maria Confluence	74	3606	1440	12229	239	12329.7	0.8%	1080	145	12743.6	4.2%
Hastings @ Ellenborough Gauge	82	1582	1440	5116	239	5121.5	0.1%	1080	145	5395.5	5.5%
Bain Bridge - Previous RMA inflow	86	2197	1440	7118	239	7151.9	0.5%	1080	145	7352.6	3.3%
Hastings River RMA inflow	87	1051	1440	2943	239	2938.7	-0.2%	1080	145	3092.5	5.1%
Wauchope Rail Bridge	88	2358	1440	7607	239	7652.0	0.6%	1080	145	7823.9	2.8%
Ellenborough River RMA inflow	93	526	1440	2208	239	2229.2	1.0%	1080	145	2357.3	6.8%
Mid Wilson River	T1041	352	720	2247	60	2253.6	0.3%	1080	145	2329.9	3.7%
Upper Wilson River	T1195	269	720	1735	60	1745.4	0.6%	1080	145	1807.1	4.1%
Frazers Creek upstream of Beechwood	T12	32.6	720	229	51	228.8	-0.1%	1080	145	235.4	2.8%
Mortons Creek RMA inflow	T14	133	720	851	51	863.6	1.5%	1080	145	869.1	2.1%
Upper Morton's Creek	T200	27.4	720	195	55	207.1	6.3%	1080	145	197.5	1.4%
Lower Pappinbarra River	T292	247	720	1393	55	1385.2	-0.5%	1080	145	1418.5	1.9%
Pappinbarra River Gauge	T4	238	720	1377	51	1376.6	-0.1%	1080	145	1413.0	2.6%
Mid Pappinbarra River	T413	144	720	906	51	914.4	0.9%	1080	145	940.7	3.8%
Mortons Creek Gauge	T5	135	720	856	51	867.3	1.3%	1080	145	873.5	2.1%
Upper Pappinbarra River	T581	84.6	720	572	51	570.3	-0.3%	1080	145	596.1	4.2%
Pappinbarra River RMA inflow	T6	251	720	1398	55	1390.3	-0.5%	1080	145	1419.3	1.5%
Telegraph Point Rail Bridge	T7	522	720	3243	51	3264.6	0.7%	1080	145	3329.8	2.7%
Wilson @ Avenal Gauge	T8	509	720	3215	60	3212.7	-0.1%	1080	145	3313.2	3.1%
Wilson River RMA inflow	T801	503	720	3191	60	3196.8	0.2%	1080	145	3291.3	3.2%
Bril Bril Creek	T961	68.8	720	476	51	477.0	0.3%	1080	145	461.0	-3.1%

TABLE 6-7

Location	WBNM Subarea	Upstream Catchment Area (km²)	All Durations, All Patterns					Selected Durations and Patterns 1in 500 AEP 1080min TP 145 + 1 in 500 AEP 2880min TP 415			
			Critical Duration (min)	Averaged Peak Flow (m³/s)	'Average' Temporal Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg) /Avg	Duration (min)	Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg)/Avg
Ocean Outlet	1	3695	1440	15123	239	15220.1	0.6%	1080	145	15743	4.1%
Thone River Gauge	101	125	720	963	51	985.6	2.3%	1080	145	1066.6	10.7%
Ellenborough River Gauge	102	498	1440	2581	239	2631.6	2.0%	1080	145	2795.6	8.3%
Hastings River before Pappinbarra	112	1914	1440	7392	239	7362.0	-0.4%	1080	145	7560.7	2.3%
Maria River RMA inflow	20	86	720	678	51	688.2	1.5%	1080	145	679.6	0.2%
Pipers Creek RMA inflow	25	122	720	868	51	886.4	2.1%	1080	145	894.0	3.0%
Pipers Creek	34	135	720	954	51	976.3	2.3%	1080	145	983.6	3.1%
Upper Maria River	35	211	720	1534	51	1567.6	2.2%	1080	145	1544.9	0.7%
Original Maria River RMA inflow	43	369	720	2542	60	2578.8	1.4%	1080	145	2586.7	1.7%
Maria River d/s Wilson River	57	974	720	6413	51	6393.9	-0.3%	1080	145	6465.7	0.8%
Hastings River @ Mt Seaview Gauge	58	348	720	1122	60	1153.7	2.8%	1080	145	1217.2	8.5%
Forbes R Gauge	60	334	720	1530	60	1543.8	0.9%	1080	145	1668.2	9.0%
Hastings River - Dennis Bridge	72	2556	1440	9718	239	9719.5	0.0%	1080	145	9867.4	1.5%
Kindee Creek RMA inflow	73	46.2	720	310	51	311.7	0.6%	1080	145	323.8	4.5%
Hastings Maria Confluence	74	3606	1440	14880	239	15003.4	0.8%	1080	145	15569.5	4.6%
Hastings @ Ellenborough Gauge	82	1582	1440	6244	239	6257.8	0.2%	1080	145	6613.4	5.9%
Bain Bridge - Previous RMA inflow	86	2197	1440	8688	239	8722.3	0.4%	1080	145	9013.7	3.7%
Hastings River RMA inflow	87	1051	1440	3618	239	3603.7	-0.4%	1080	145	3819.7	5.6%
Wauchope Rail Bridge	88	2358	1440	9289	239	9334.5	0.5%	1080	145	9597.9	3.3%
Ellenborough River RMA inflow	93	526	1440	2661	239	2689.3	1.1%	1080	145	2850.2	7.1%
Mid Wilson River	T1041	352	720	2658	60	2676.2	0.7%	1080	145	2751.7	3.5%
Upper Wilson River	T1195	269	720	2053	60	2072.1	0.9%	1080	145	2133.3	3.9%
Frazers Creek upstream of Beechwood	T12	32.6	720	269	51	268.2	-0.4%	1080	145	276.7	2.8%
Mortons Creek RMA inflow	T14	133	720	1006	60	998.8	-0.7%	1080	145	1029.3	2.3%
Upper Morton's Creek	T200	27.4	720	229	55	243.8	6.5%	1080	145	231.0	1.0%
Lower Pappinbarra River	T292	247	720	1655	55	1661.0	0.3%	1080	145	1688.4	2.0%
Pappinbarra River Gauge	T4	238	720	1636	51	1640.4	0.3%	1080	145	1680.1	2.7%
Mid Pappinbarra River	T413	144	720	1071	51	1080.4	0.8%	1080	145	1110.3	3.6%
Mortons Creek Gauge	T5	135	720	1012	51	1027.9	1.5%	1080	145	1034.7	2.2%
Upper Pappinbarra River	T581	84.6	720	674	51	669.3	-0.6%	1080	145	699.9	3.9%
Pappinbarra River RMA inflow	T6	251	720	1663	55	1668.1	0.3%	1080	145	1689.7	1.6%
Telegraph Point Rail Bridge	T7	522	720	3844	55	3843.7	0.0%	1080	145	3943.8	2.6%
Wilson @ Avenal Gauge	T8	509	720	3808	60	3816.4	0.2%	1080	145	3920.2	2.9%
Wilson River RMA inflow	T801	503	720	3778	55	3794.9	0.4%	1080	145	3891.6	3.0%
Bril Bril Creek	T961	68.8	720	560	51	560.0	0.0%	1080	145	574.7	2.6%

TABLE 6-8

Location	WBNM Subarea	Upstream Catchment Area (km²)	All Durations, All Patterns					Selected Durations and Patterns 1 in 2000 AEP 1080min TP 145 + 1 in 2000 AEP 2880min TP 415			
			Critical Duration (min)	Averaged Peak Flow (m³/s)	'Average' Temporal Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg) /Avg	Duration (min)	Pattern No.	Patt. Peak Flow (m³/s)	% Difference (Patt. - Avg)/Avg
Ocean Outlet	1	3695	1440	19509	239	19626.6	0.6%	1080	145	20429	4.7%
Thone River Gauge	101	125	720	1203	51	1229.5	2.2%	1080	145	1327.8	10.4%
Ellenborough River Gauge	102	498	1440	3286	239	3352.4	2.0%	1080	145	3564.1	8.5%
Hastings River before Pappinbarra	112	1914	1440	9583	239	9536.5	-0.5%	1080	145	9869.1	3.0%
Maria River RMA inflow	20	86	720	847	51	858.2	1.3%	1080	145	847.2	0.0%
Pipers Creek RMA inflow	25	122	720	1091	51	1112.4	1.9%	1080	145	1119.3	2.6%
Pipers Creek	34	135	720	1200	51	1225.7	2.1%	1080	145	1232.1	2.7%
Upper Maria River	35	211	720	1929	51	1970.1	2.1%	1080	145	1938.3	0.5%
Original Maria River RMA inflow	43	369	720	3207	60	3261.2	1.7%	1080	145	3252.0	1.4%
Maria River d/s Wilson River	57	974	720	8098	51	8091.4	-0.1%	1080	145	8153.6	0.7%
Hastings River @ Mt Seaview Gauge	58	348	720	1468	59	1508.4	2.7%	1080	145	1599.3	8.9%
Forbes R Gauge	60	334	720	1967	60	1991.3	1.2%	1080	145	2140.6	8.8%
Hastings River - Dennis Bridge	72	2556	1440	12612	239	12589.5	-0.2%	1080	145	12876.0	2.1%
Kindee Creek RMA inflow	73	46.2	720	389	51	390.4	0.5%	1080	145	403.4	3.8%
Hastings Maria Confluence	74	3606	1440	19182	239	19328.4	0.8%	1080	145	20198.1	5.3%
Hastings @ Ellenborough Gauge	82	1582	1440	8071	239	8102.8	0.4%	1080	145	8607.4	6.6%
Bain Bridge - Previous RMA inflow	86	2197	1440	11236	239	11263.5	0.2%	1080	145	11732.9	4.4%
Hastings River RMA inflow	87	1051	1440	4719	239	4698.0	-0.4%	1080	145	5015.2	6.3%
Wauchope Rail Bridge	88	2358	1440	12019	239	12058.5	0.3%	1080	145	12504.0	4.0%
Ellenborough River RMA inflow	93	526	1440	3394	239	3429.5	1.1%	1080	145	3651.6	7.6%
Mid Wilson River	T1041	352	720	3316	60	3352.7	1.1%	1080	145	3430.1	3.5%
Upper Wilson River	T1195	269	720	2557	51	2585.4	1.1%	1080	145	2657.0	3.9%
Frazers Creek upstream of Beechwood	T12	32.6	720	334	51	330.6	-0.9%	1080	145	342.3	2.6%
Mortons Creek RMA inflow	T14	133	720	1256	60	1255.1	-0.1%	1080	145	1284.9	2.3%
Upper Morton's Creek	T200	27.4	720	283	60	301.6	6.5%	1080	145	284.3	0.4%
Lower Pappinbarra River	T292	247	720	2076	51	2074.5	-0.1%	1080	145	2120.2	2.1%
Pappinbarra River Gauge	T4	238	720	2049	51	2063.5	0.7%	1080	145	2106.6	2.8%
Mid Pappinbarra River	T413	144	720	1334	51	1342.5	0.6%	1080	145	1381.2	3.5%
Mortons Creek Gauge	T5	135	720	1264	60	1258.4	-0.5%	1080	145	1292.0	2.2%
Upper Pappinbarra River	T581	84.6	720	834	51	824.7	-1.1%	1080	145	865.0	3.7%
Pappinbarra River RMA inflow	T6	251	720	2086	51	2076.3	-0.5%	1080	145	2123.0	1.8%
Telegraph Point Rail Bridge	T7	522	720	4806	60	4794.8	-0.2%	1080	145	4930.3	2.6%
Wilson @ Avenal Gauge	T8	509	720	4757	60	4787.3	0.6%	1080	145	4892.4	2.9%
Wilson River RMA inflow	T801	503	720	4719	60	4761.5	0.9%	1080	145	4855.2	2.9%
Bril Bril Creek	T961	68.8	720	695	51	692.1	-0.5%	1080	145	711.6	2.3%

6.3.6 Probable Maximum Precipitation (PMP)

The Probable Maximum Precipitation (PMP), as used to determine the Probable Maximum Flood (PMF), was derived using the methods outlined in in ARR 2019. The Hastings River Catchment is located within the "GSAM-GTSMR Coastal Transition Zone". Both the Generalised Southeast Australia Method (GSAM) and Generalised Tropical Storm Method Revised (GTSMR) therefore apply as potential PMP calculation methods for the region. **Table 6-12** presents the PMP estimates for the Hastings River catchment derived from each method for durations from 24 to 120 hours. GTSMR PMP estimates were found to generate higher rainfall totals and were therefore critical and adopted for the estimation of the PMF event.

Table 6-12 PMP Storm Depths (mm)

Duration (hr)	GTSMR	GSAM	Maximum
24	1010	850	1010
36	1170	970	1170
48	1320	1060	1320
72	1600	1180	1600
96	1820	1280	1820
120	1930	NA	1930

Following the instructions outlined in 'Guidebook of the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method' (BoM, 2005), spatial variety was incorporated into the hydrologic model based on the TAF grid for GTSMR calculation. This incorporated the spatial variety observed based on the orographic effect of the catchment terrain.

The temporal patterns for the application of GTSMR PMP rainfalls are prescribed within the guidebook. These are associated with standard catchment areas, where the 2500 km² standard area is prescribed for size of the Hastings River catchment.

Loss rates for the PMF simulations were adopted in accordance with guidelines outlined in Book 8, Chapter 4 of ARR 2019. This describes an initial loss and continuing loss rates of 0 mm and 1 mm/hr, respectively, were suitable values to adopt for a rural catchment such as the Hastings River catchment.

6.4 Downstream Boundary Conditions

6.4.1 RMA-2 Model

As discussed in **Section 4.3.3**, the extended Hastings River RMA-2 model includes three downstream boundaries which represent a potential outflow that is controlled by either a stage hydrograph or time varying tailwater condition. The downstream boundaries are located along Connection Creek (to the north of the Maria River), and ocean boundaries at Point Plomer and at the Hastings River outlet near North Shore Beach. The location of these downstream boundaries is shown in **Figure 4-7** and **Figure 4-8**.

Flood levels in the lower parts of the RMA-2 model may be influenced by the coinciding ocean water levels and coincident flooding across the Macleay River catchment to the north. A time varying tidal boundary with a peak ocean level of 2.2 mAHD was selected for the downstream boundaries at Point Plomer and North Shore Beach. This is consistent with modelling undertaken for the Hibbard Precinct Flood Study (2019).

The adopted boundary conditions along Connection Creek reflect coincident flooding across the Macleay River catchment. Following careful consideration of coincident flood scenarios, the design event combinations listed in **Table 6-13** were selected. The selected combinations are generally consistent with those adopted as part of the *Macleay River Flood Study (2019)*.

Table 6-13 Coincident Hastings River and Macleay River Design Events Adopted for the RMA-2 Design Event Modelling

Hastings River Design Event (Applied as model inflows)	Macleay River Design Event (Applied at the Connection Creek D/S Boundary)
5% AEP	5% AEP
2% AEP	5% AEP
1% AEP	5% AEP
1% AEP with Climate Change	5% AEP
1 in 200 AEP	5% AEP
1 in 500 AEP	1% AEP
1 in 2000 AEP	1% AEP
PMF	1% AEP

The ocean outlet downstream of Port Macquarie is consistent with the RMA-2 model set-up adopted as part of the *'Hibbard Precinct Flood Study'* (2019). In that regard, during events up to and including a 1 in 500 AEP flood, flows along the Hastings River discharge to the ocean via the river mouth and the training wall. During larger events such as the Probable Maximum Flood (PMF), floodwaters break the banks between North Shore and the ocean outlet leading to widespread overtopping of Northwall Road and the sand dunes. Accordingly, the downstream boundary condition is set-up to allow floodwaters to reach the ocean via this route.

6.4.2 TUFLOW Model

Flood levels in the lower parts of the TUFLOW model are influenced by coincident flooding along the adjoining watercourses. For example, the Pappinbarra River and Mortons Creek both discharge to the Hastings River which, could coincidentally be at various stages of flooding. As we are simulating a catchment wide design flood event, the use of accurate downstream water level boundaries will provide further confidence in the peak flood levels in the lower catchment of each of these tributaries.

Accordingly, the downstream water levels for the TUFLOW model are based on flood levels predicted using the RMA-2 hydraulic model. This creates a time varying water level boundary that replicates the regional flooding conditions predicted using the extended Hastings River RMA-2 model. This process was applied to each of the tributaries, Mortons Creek, the Pappinbarra River and the Wilson River, for each design event and duration storm.

6.5 Hydrology

Design flood hydrographs determined using the WBNM hydrologic model were used to define inflows for the RMA-2 and TUFLOW hydraulic models.

Summaries of peak discharges at key locations along the Hastings and Wilson Rivers and along their major tributaries are provided in **Table 6-14** and **Table 6-15**. **Table 6-14** lists peak discharges for events up to and including the 1% AEP flood with climate change. **Table 6-15** includes all larger events up to and including the PMF.

Shaded cells in the tables indicate the critical duration; that is, the storm duration that produces the highest peak discharge. It is important to note that these critical durations are based on flow magnitudes only and

may not generate the highest flood levels when routed through the hydraulic models. For example, factors that will influence flooding include the coincident timing of flows from other watercourses and the attenuation of flows via flood storages.

As shown in **Table 6-14** and **Table 6-15**, the 18-hour storm duration was found to be result in the highest peak flow magnitudes across the entire catchment. Although the 48-hour storm produced lower peak flows, the longer duration event resulted in larger flow volumes which are critical in the lower reaches of the catchment, downstream of Wauchope.

6.5.1 Comparison to Previous Studies

As discussed in **Section 6.1**, previous catchment wide flood studies for the Hastings River catchment have been based on procedures and data outlined in *Australian Rainfall and Runoff 1987* (ARR 1987). This includes the currently adopted *Hastings River Flood Study* (2006) and the more recent *Hibbard Precinct Flood Study* (2019) which considers the lower reaches of the Hastings River downstream of the Pacific Highway bridge crossing.

The use of procedures outlined in ARR 2019 and the application of a new WBNM hydrologic model of the catchment is a major modification to the hydrologic modelling that had been undertaken previously for the catchment. It is therefore warranted to undertake a comparison between the design flows and flow hydrographs predicted as part of previous studies to this flood study extension. The outcomes of this comparison are discussed below.

- A comparison between peak flow magnitudes predicted at the upstream boundaries of the original Hastings River RMA-2 model is presented in **Table 6-16**.
- The comparison shows that peak discharges have decreased along the Hastings River for design events up to and including the 1% AEP climate change scenario. The decrease is largest for the 5% AEP event with a reduction of 21% predicted. Reductions in the order of 9-13% are predicted for the other events (refer **Table 6-16**).
- Peak flows along the Hastings River for the PMF event are predicted to be 58% higher than those predicted for the previously adopted 'extreme' flood event. The 'extreme' event was derived by multiplying the 1% AEP flood hydrographs by three (3) whilst the PMF for this study is based on the Generalised Southeast Australia Method (GSAM) and Generalised Tropical Storm Method Revised (GTSMR) calculation methods (refer **Section 6.3.6**).
- Peak flows along the Wilsons River have increased by up to 10% for events up to and including the 1% AEP climate change event. Peak flows are predicted to increase by 33% when comparing flows generated for the PMF to those adopted for the previous 'extreme' event (refer **Table 6-16**).

Comparison plots between the updated and previously adopted flow hydrographs are included within **Appendix D**. Flow hydrographs are provided for the same comparison locations and design events listed in **Table 6-16**, i.e., at the upstream boundaries of the original RMA-2 flood model.

The changes to peak flows and hydrograph shapes are heavily attributed to the use of updated Intensity-Frequency-Duration (IFD) data and grids, and temporal patterns recommended for by ARR 2019.

Table 6-14 Comparison of Predicted Peak Discharges at Key Locations for Events up to the 1% AEP with Climate Change

Location	WBNM sub- catchment name	Peak Discharge (m³/s)							
		5% AEP		2% AEP		1% AEP		1% AEP with CC	
		18hr	48hr	18hr	48hr	18hr	48hr	18hr	48hr
Locations along the Hastings River									
U/S of Ellenborough River Confluence / RMA-2 Inflow	93	1234	935	1659	1284	2003	1559	2315	1805
Kindee Road Bridge	82	2646	1927	3665	2785	4507	3467	5270	4083
Thone River Confluence	110	2966	2236	4112	3222	5064	4004	5927	4716
Gannons Creek Confluence / Hartys Plains	112	3001	2295	4165	3312	5136	4124	6015	4861
Pappinbarra River / Koree Island	96	3628	2767	5010	3955	6166	4903	7206	5763
Mortons Creek / Bains Bridge	86	3620	2766	4999	3957	6154	4906	7193	5767
Wauchope Railway Bridge	88	3845	2970	5313	4244	6547	5261	7654	6183
Dennis Bridge / Old Pacific Highway Crossing	72	3937	3102	5444	4438	6717	5511	7859	6485
Ocean Outlet	1	6461	4916	8823	6954	10813	8574	12591	10044
Locations along the Wilsons River									
Rollands Plains Road Crossing (Upstream of Rollands Plans)	T1275	504	328	650	434	766	515	869	590
Littles Loop Road Crossing / Rollands Plains	T1223	589	382	761	506	898	602	1020	690
Glencoe Creek Confluence	T1225	1018	652	1309	860	1540	1021	1745	1167
Clarefield Dungay Ck Rd Crossing	T1153	1045	672	1347	890	1586	1056	1800	1210
Bril Bril Ck Confluence	T815	1685	1068	2174	1420	2561	1690	2906	1936
Banglow Ck Confluence	T793	1802	1138	2326	1513	2740	1803	3111	2065
Rollands Plains Road Crossing / Ballengarra	T8	1878	1187	2426	1581	2860	1881	3248	2159
Telegraph Point Road Crossing	T1	1881	1200	2434	1602	2873	1911	3267	2194
Outlets of Major Tributaries – Discharging to the Hastings and Wilson Rivers									
Kindee Ck	73	156	95	201	127	237	153	269	176
Pappinbarra Ck	T3/T6	791	532	1030	719	1224	863	1396	994
Mortons Ck U/S Frazers Ck	T20	354	228	457	303	540	361	613	413
Mortons Ck Outlet	T2	505	325	653	433	771	515	877	591
Bril Bril Ck	T870	303	189	387	248	454	292	514	333

Table 6-15 Comparison of Predicted Peak Discharges at Key Locations for Events up to the PMF

Location	WBNM sub- catchment name	Peak Discharge (m³/s)						
		1 in 200 AEP		1 in 500 AEP		1 in 2000 AEP		PMF
		18hr	48hr	18hr	48hr	18hr	48hr	24hr
Locations along the Hastings River								
U/S of Ellenborough River Confluence / RMA-2 Inflow	93	2357	1842	2850	2234	3652	2357	8645
Kindee Road Bridge	82	5396	4180	6613	5154	8607	5396	23818
Thone River Confluence	110	6065	4824	7442	5945	9702	6065	27011
Gannons Creek Confluence / Hartys Plains	112	6155	4971	7561	6134	9869	6155	27923
Pappinbarra River / Koree Island	96	7367	5879	9029	7225	11751	7367	32440
Mortons Creek / Bains Bridge	86	7353	5884	9014	7228	11733	7353	32486
Wauchope Railway Bridge	88	7824	6304	9598	7741	12504	7824	34563
Dennis Bridge / Old Pacific Highway Crossing	72	8034	6608	9867	8123	12876	8034	36157
Ocean Outlet	1	12882	10171	15743	12405	20429	12882	52387
Locations along the Wilsons River								
Rollands Plains Road Crossing (Upstream of Rollands Plans)	T1275	885	591	1046	697	1303	885	3153
Littles Loop Road Crossing / Rollands Plains	T1223	1039	691	1229	816	1533	1039	3678
Glencoe Creek Confluence	T1225	1778	1169	2099	1379	2612	1778	6072
Clarefield Dungay Ck Rd Crossing	T1153	1834	1213	2166	1433	2699	1834	6290
Bril Bril Ck Confluence	T815	2964	1943	3504	2297	4369	2964	9671
Banglow Ck Confluence	T793	3173	2072	3751	2452	4678	3173	10211
Rollands Plains Road Crossing / Ballengarra	T8	3313	2167	3920	2566	4892	3313	10628
Telegraph Point Road Crossing	T1	3333	2202	3949	2611	4939	3333	10843
Outlets of Major Tributaries – Discharging to the Hastings and Wilson Rivers								
Kindee Ck	73	274	178	324	212	403	274	899
Pappinbarra Ck	T3/T6	1419	1002	1690	1196	2123	1419	5131
Mortons Ck U/S Frazers Ck	T20	624	416	739	495	924	624	1916
Mortons Ck Outlet	T2	893	596	1058	707	1323	893	2715
Bril Bril Ck	T870	524	335	618	396	767	524	1571

Table 6-16 Comparison of Peak Design Flows at Previous Continuity Lines

Location	Peak Discharge (m ³ /s)									
	5% AEP		2% AEP		1% AEP		1% AEP with CC		Extreme / PMF	
	XP-RAFTS	WBNM	XP-RAFTS	WBNM	XP-RAFTS	WBNM	XP-RAFTS	WBNM	XP-RAFTS	WBNM
Hastings River upstream of Wauchope	4565	3620 (-21%)	5715	5000 (-13%)	6850	6155 (-10%)	7896	7195 (-9%)	20550	32440 (+58%)
Wilson River upstream of Telegraph Point	1740	1880 (+8%)	2210	2435 (+10%)	2705	2870 (+6%)	3122	3265 (+5%)	8115	10780 (+33%)

Notes: 1. Peak flows predicted using XP-RAFTS have been adopted for all previous flood studies (refer **Section 2.3**). Previous hydrologic modelling was based on ARR 87 procedures and IFD data (refer **Section 6.2**).

2 Peak flows predicted using WBNM adopt 18 and 48 hour storm durations for all design events excluding the PMF. The PMF is based on a 24 hour duration.

6.5.2 Comparison to Flood Frequency Analysis

Table 6-17 shows a comparison between flows predicted using the WBNM model and those estimated based on the updated FFA prepared for the Hastings River at Ellenborough (Kindee Bridge) gauge (refer **Section 6.2**).

Table 7-4 Hastings River at Ellenborough (Kindee Bridge) Updated FFA Results

Annual Exceedance Probability (AEP)	Estimated Flow Values (m ³ /s)		
	Updated FFA	2024 WBNM Model	Difference
5% AEP	2995	2646	- 349 (-11%)
2% AEP	3800	3665	- 135 (-4%)
1% AEP	4381	4507	+ 126 (+3%)
1 in 200 AEP	4936	5395	+459 (+9%)

The comparison in **Table 6-17** shows that the WBNM model incorporating ARR 2019 produces peak flows that are within 11% of those predicted by the updated FFA for the Hastings River at Ellenborough (Kindee Bridge) gauge. This is a good fit that supports the use of the 2024 WBNM model for design event flow estimation.

A further comparison between the updated FFA and peak flows predicted as part of the 2006 Hastings River Flood Study is included in **Plate 6-4**. The comparison shows that the WBNM model predicts flows that are in closer agreement to the FFA for the Hastings River at Ellenborough (Kindee Bridge) gauge for events between 5% and 1 in 200 AEP.

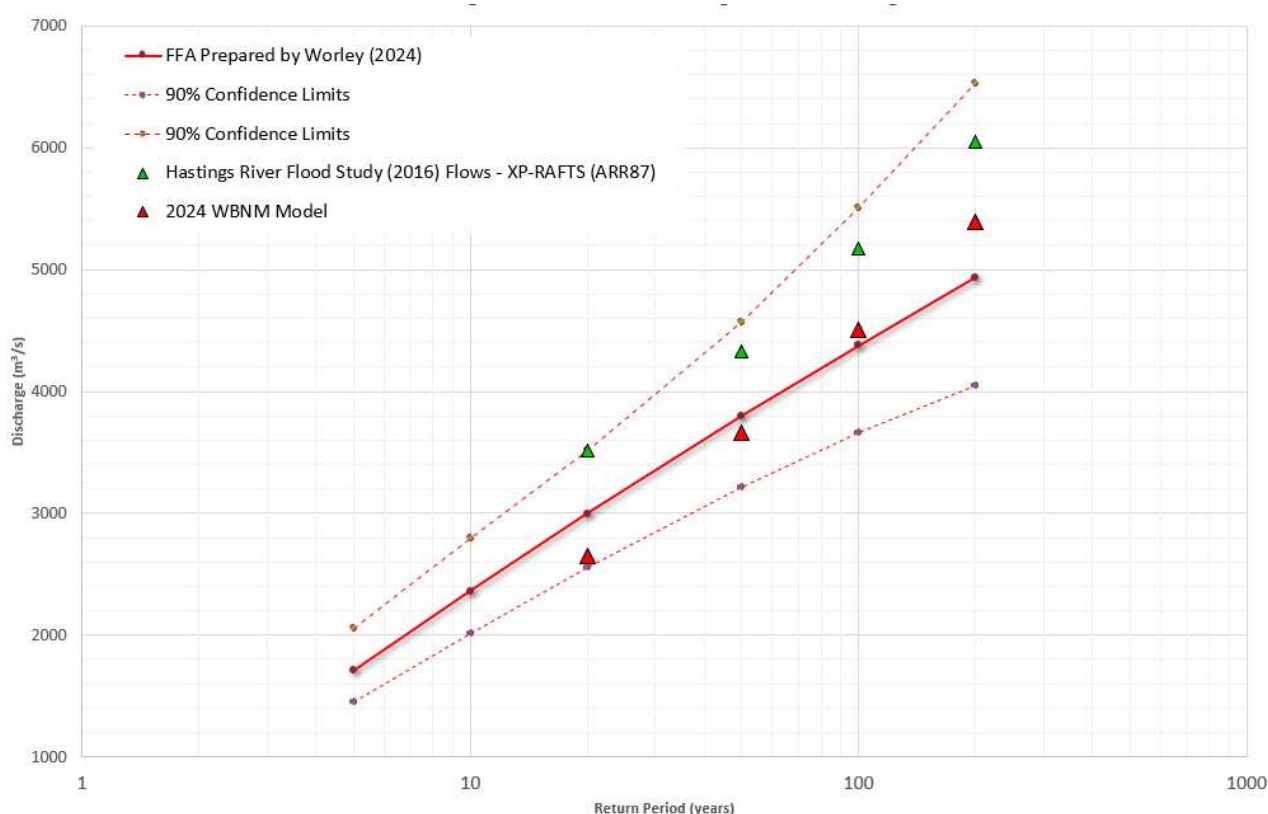


Plate 6-4 Comparison between Hastings River at Ellenborough (Kindee Bridge) gauge FFA and Simulated Design Flows

6.6 Dune Breach and Riverbed Scour During PMF

Dune breach and riverbed scour have not previously been included in the RMA-2 model geometry when modelling design events up to and including the previously adopted nominal extreme flood. The proposed replacement of the previously adopted extreme event approach with a contemporary calculation of the PMF in accordance with procedures outlined in ARR 2019 (refer **Section 6.3.6**) raised concerns for local industry professionals during public exhibition of Rev E of this study. The local industry professionals are of the view that the new PMF levels for the valley are overly conservative and would unduly restrict development. During an industry professionals panel session held on 10 April 2025, it was suggested that further modelling should be undertaken to incorporate a breach of the sand dunes to the north of the Hastings River outlet, i.e., along Northwall Road and Plomer Road, and addition of bed scour along the Hastings River.

The following sections outline the dune breach and riverbed scour scenarios considered for inclusion when modelling the PMF event. The recommended dune breach and riverbed scour scenario is also detailed.

6.6.1 Considered Dune Breach and Riverbed Scour Scenarios

The DEM in the RMA-2 flood model includes terrain data that defines elevations along the dune system that extends to the north of the river entrance. This terrain was based on elevations defined by Light Detection and Ranging (LiDAR) survey flown in 2012. The RMA-2 flood model also includes riverbed elevations that are based on bathymetric survey (refer **Section 3.1.2**).

The RMA-2 flood model has in the past assumed that there is no breach of the sand dunes or scour of the riverbed during the full range of design events. As discussed above, this position has been challenged recently by industry professionals for modelling of the PMF event, for which initial model results showed that the sand dunes would be overtopped and that flow velocities along the Hastings River would exceed 2 m/s at many locations downstream of Dennis Bridge at Sancro, and reach up to 9.5 m/s through the river entrance. The RMA-2 flood model without inclusion of any dune breach or riverbed scour is referred to herein as the base case model.

It is noted that one of the primary purposes of simulating the PMF event is to estimate the largest possible flood conditions. This information is required to assist flood emergency response management.

The following dune breach and riverbed scour scenarios were designed recognising the limited data defining the geological strata in the vicinity of the river entrance. The riverbed scour scenarios were therefore approached from a sensitivity analysis perspective, with varying extents and depths of scour modelled to assess the impact on peak PMF levels.

The dune breach and riverbed scour scenarios considered are described in the following bullet points.

- **Dune Breach Scenario 1** – Includes multiple breaches of the sand dune to the north of the river entrance. The dune breaches were all modelled with dynamic (time varying) crest elevations that were scoured (crest elevations lowered) gradually only once overtopping at each location had commenced. Conditions along the sand dunes were assessed at hourly intervals during the rising limb of the PMF event.

The evolution of Dune Breach Scenario 1 is shown in **Figures 6-2, 6-3** and **6-4**, and is summarised in the following:

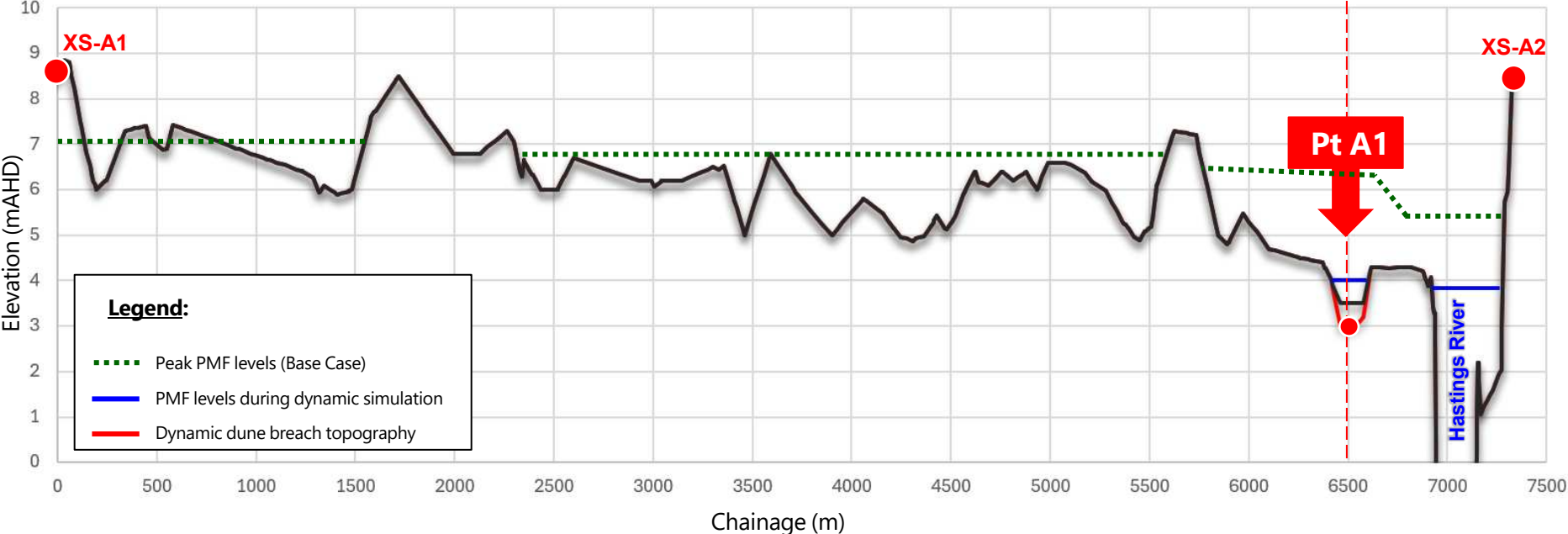
1. Initial overtopping of the sand dunes occurs approximately 500 metres north of the river entrance (refer breach location A1 on **Figure 6-2**).
2. The initial breach is added to the RMA-2 model network 1-hour after overtopping commences at a point where the crest is overtopped by 0.5 metres. A depth of scour of 0.5 metres is applied over a length of approximately 100 metres.
3. At 2-hours after initial overtopping, and 1-hour after initial scour, the breach depth is increased by a further 0.5 metres. At this stage, floodwaters have not overtopped other dune low-points.
4. At 3-hours after initial overtopping, and 2-hours after initial scour, two new breach locations are formed further north of Breach A1 (refer Points A2 and A3 on **Figure 6-3**). The initial scour depths applied at both locations are between 0.2 and 0.4 metres. Concurrently, Breach location A1 expands in width and depth to approximately 800 metres and 1.5 metres, respectively.
5. At 4-hours after initial overtopping, and 3-hours after initial scour, two additional breach locations are formed further north of Breach A3 (refer Points A4 and A5 on **Figure 6-3**). Based on the more acute nature of the low-point at both locations, the initial scour depths are higher than previous locations at 1-metre. The depth of the breach at all other locations is increased by between 0.2 and 1.0 metres.
6. At 5-hours after initial overtopping, and 4-hours after initial scour, no additional breach locations are formed. The breach depths are increased at all locations except at Point A1, where it is considered to have reached the minimum adopted scour crest elevation of 2 mAHD (refer **Figure 6-4**).
7. At 6-hours after initial overtopping, and 5-hours after initial scour, no further breach locations are formed. The breach depths at Points A3, A4 and A5 increase by up to 1 metre reaching the minimum adopted scour crest elevation of 2 mAHD (refer **Figure 6-4**).

The rationale for the adopted scouring extents and depths is based on empirical values documented in '*Coastal Lagoon Entrance Dynamics*' (Gordon 1990) for the volume of sand removal during overtopping of a lagoon entrance barrier. Gordon determined that the rate of sand scour from an entrance barrier varies with the stage of overtopping and the width of the breach. The stages are described below:

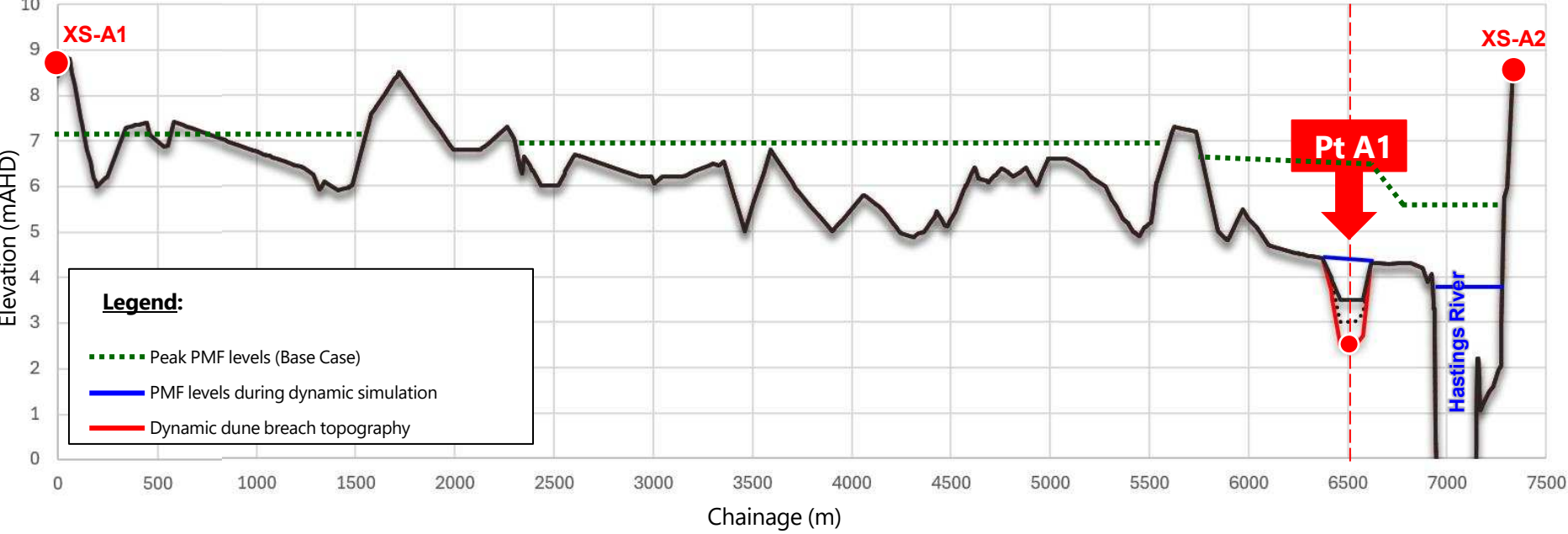
FIGURE 6-2



Status of the dynamic dune breach 1-hour after initial overtopping

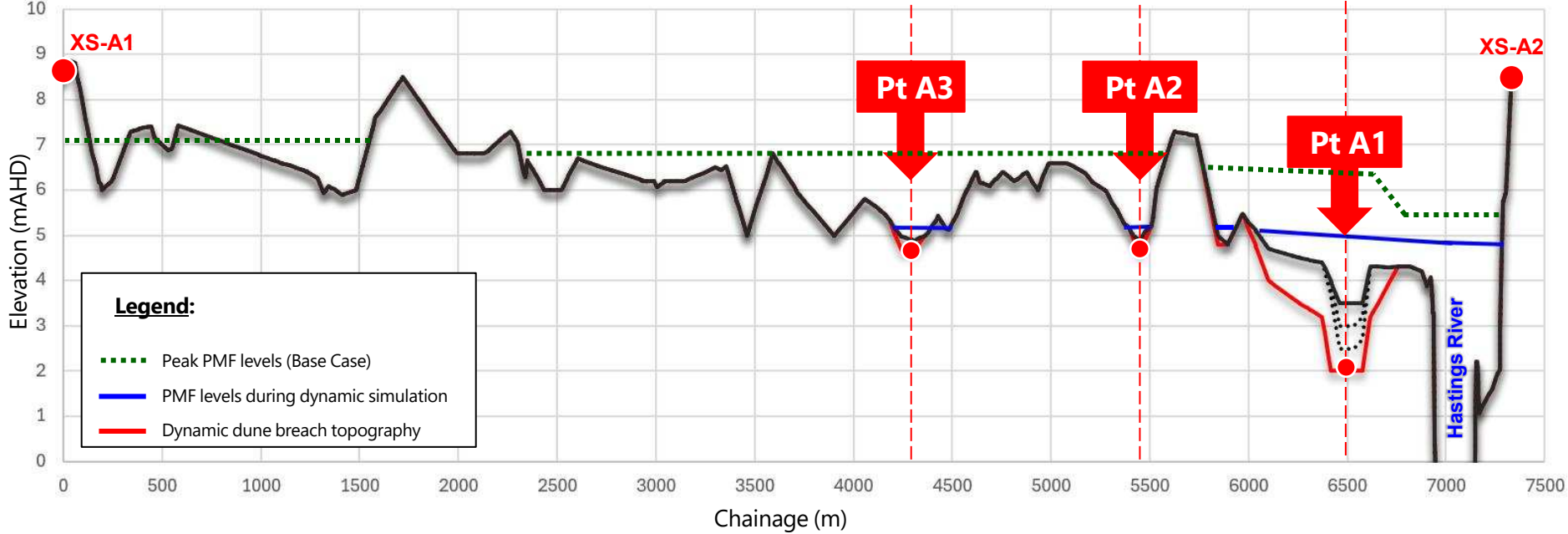


Status of the dynamic dune breach 2-hours after initial overtopping

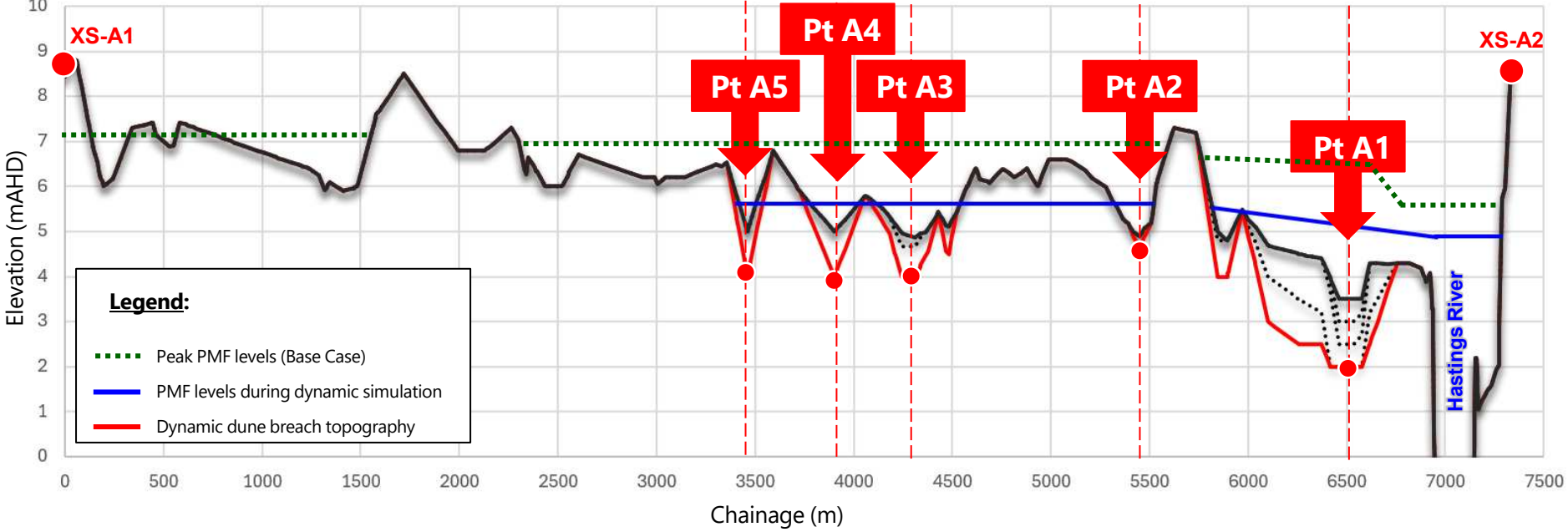




Status of the dynamic dune breach 3-hours after initial overtopping

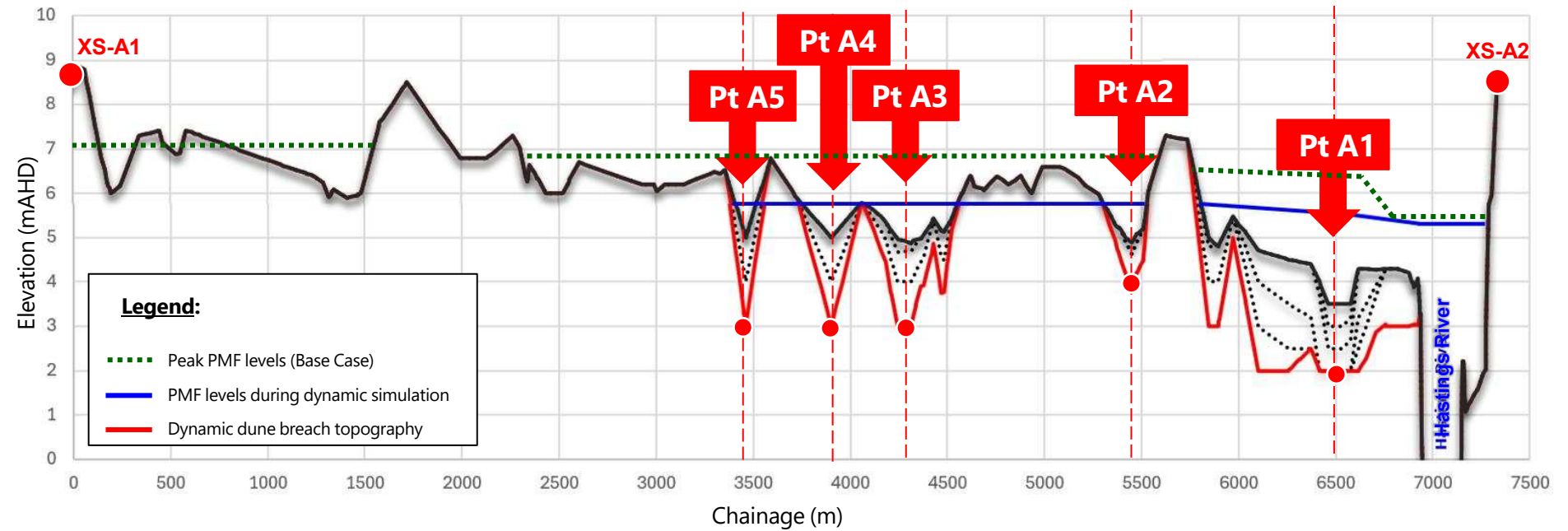


Status of the dynamic dune breach 4-hours after initial overtopping

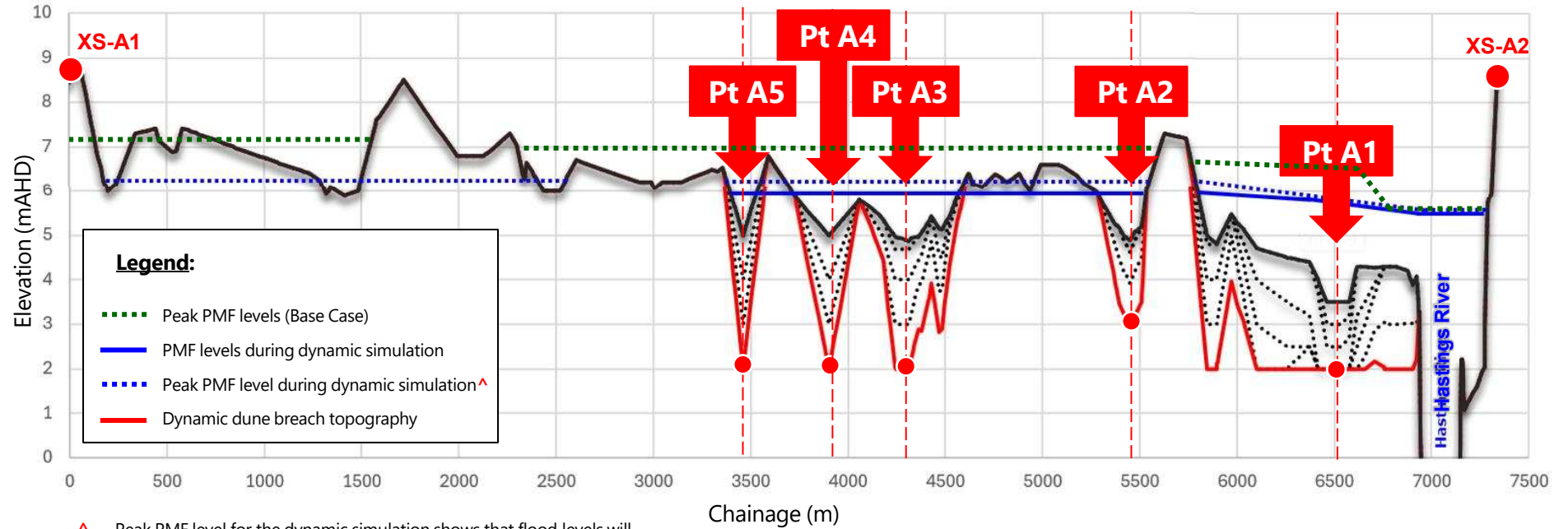




Status of the dynamic dune breach 5-hours after initial overtopping



Status of the dynamic dune breach 6-hours after initial overtopping



[^] Peak PMF level for the dynamic simulation shows that flood levels will continue to rise beyond addition of the final dune breach modification.

- Stage 1 Initial overtopping and development of a channel
- Stage 2 The weir/hydraulic jump stage, and
- Stage 3 the river flow stage.

Given the high rate of rise and velocities of floodwaters in the PMF, it was considered that Stages 1 and 2 would occur very quickly and thus scour rates based on the river flow stage (Stage 3) are most appropriate for this dune scour assessment. As shown in **Plate 6-5**, there is estimated to be approximately 3,500 m³ of sand removed during a 40-minute period during the river flow stage. The average channel 'width' (*orthogonal to the direction of flow*) over this period is 52 metres.

Based on the rates derived from **Plate 6-5**, the volume of sand removed per hour and per metre width of channel is calculated to be 101 m³/hr/m. The depth of scour per hour is also influenced by the thickness/breadth of the breach in the direction of flow. This varies substantially across the sand dunes to the north of the Hastings River entrance with the dunes spanning anywhere between 50 to 250 metres. The range of scour depths that could be applied per hour is therefore between 0.40 to 2.02 metres.

The adopted depths of scour for Dune Breach Scenario 1, which range between 0.2 to 1.0 metres per hour, are therefore considered to be reasonable and justifiable for adoption in the modelling.

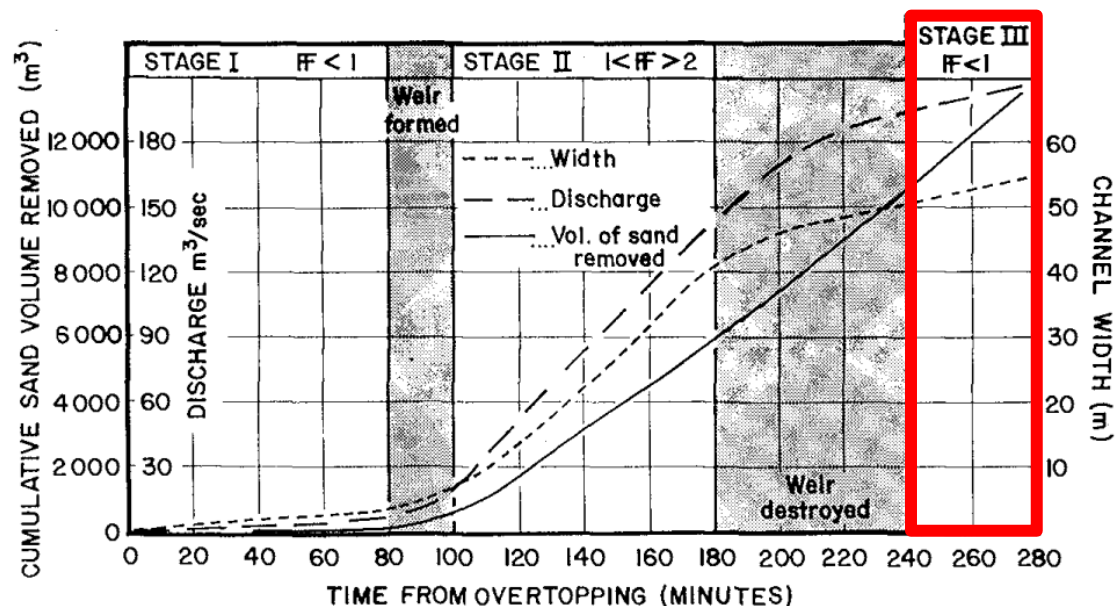
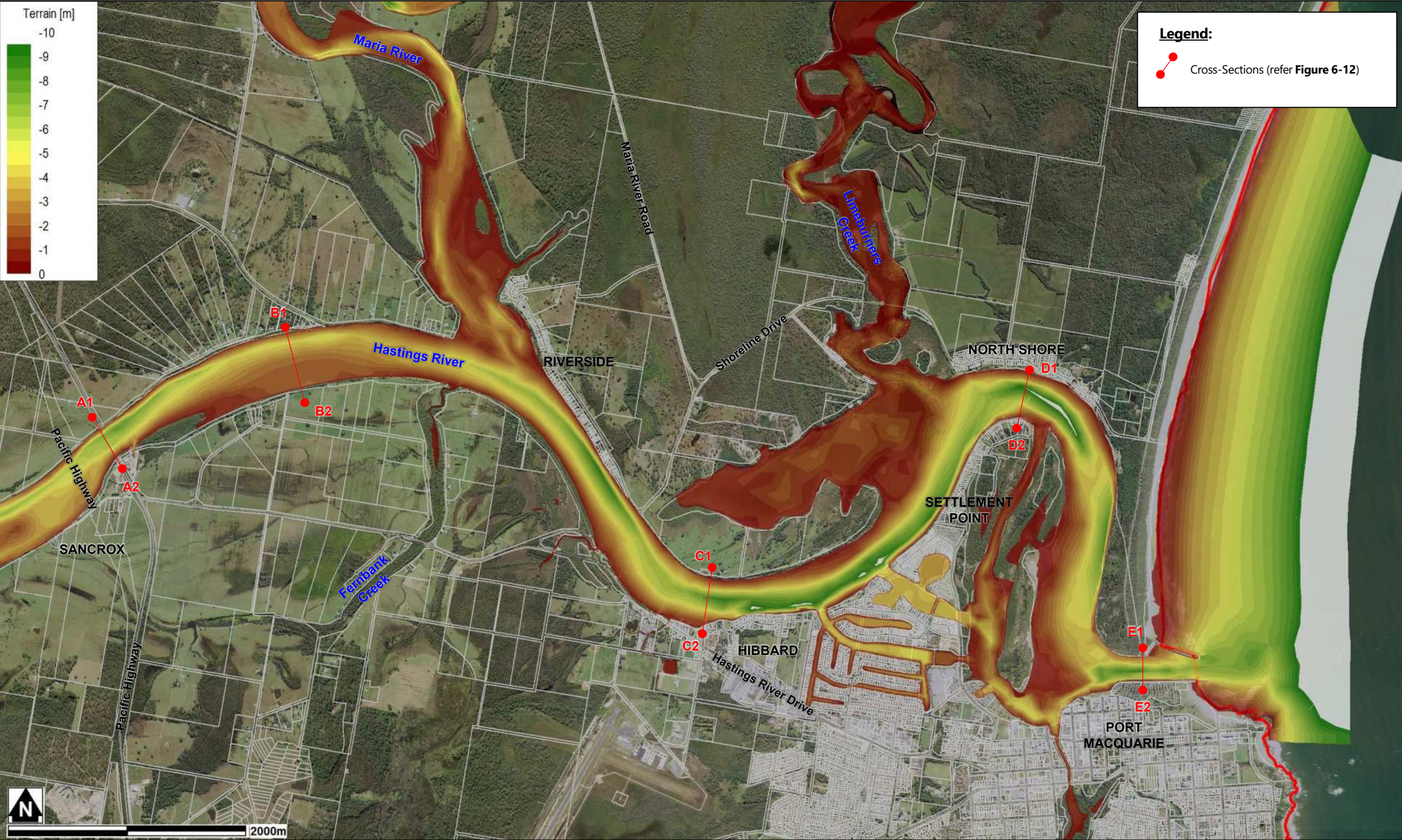


Plate 6-5 Volume of Sand Removed during the Three Stages of a Breakout (Gordon 1990).

- **River Scour Scenario 1** – River Scour scenario 1 was focused on the river entrance where depth averaged flow velocities are predicted to reach up to 9.5 m/s during the rising limb of a PMF event. Like Dune Breach Scenario 1, scouring of the riverbed elevations were incorporated into the RMA-2 model at hourly intervals during the rising limb of the event. Scour was first applied by reducing bed elevations by up to 0.5 metres per hour over a 6-hour period. This resulted in a reduction in average bed elevations at the river entrance from around -7.0 mAHD to -9.5 mAHD.

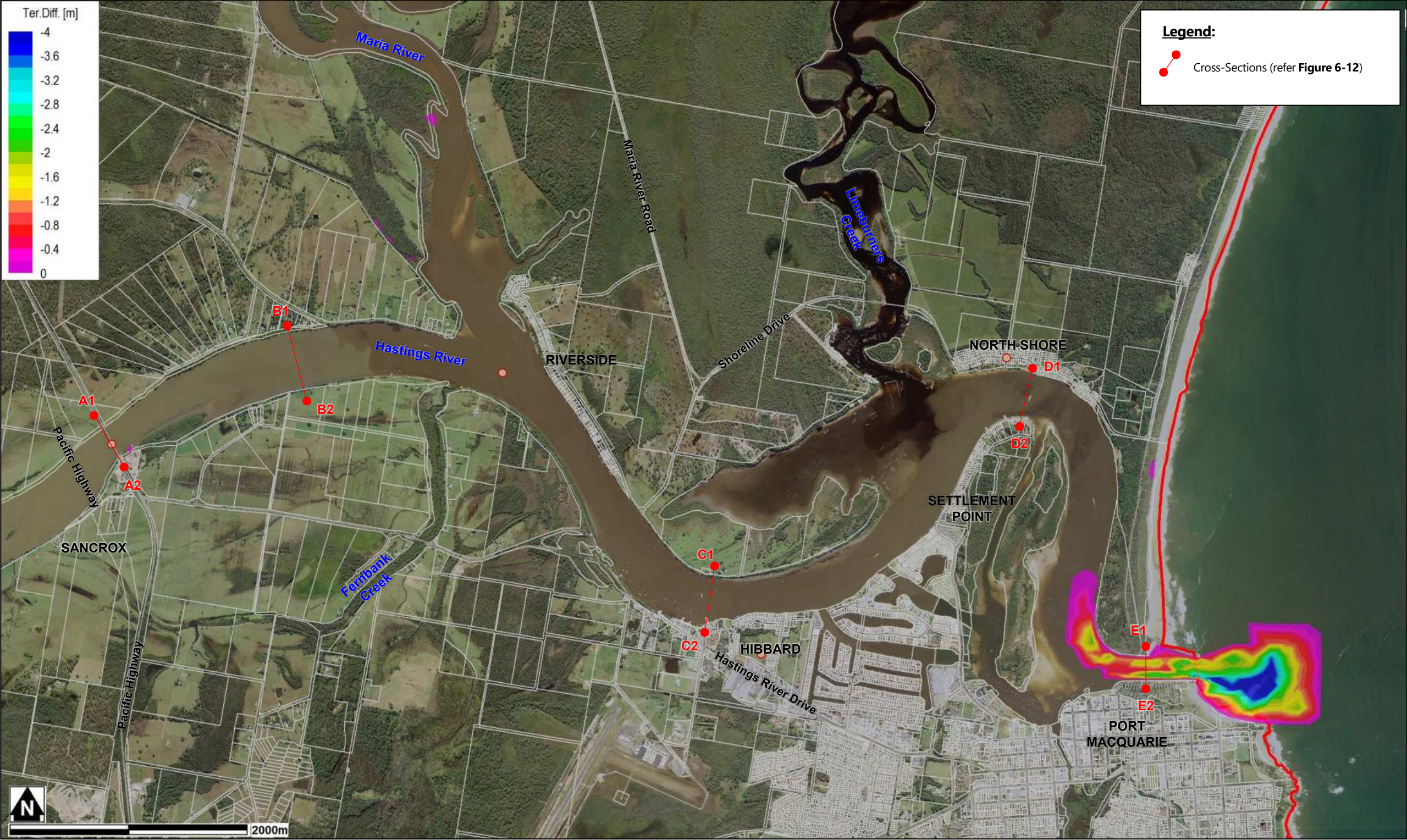
Riverbed elevations for the base case scenario are presented in **Figure 6-5**. The extent of scour incorporated in River Scour Scenario 1 is shown in **Figure 6-6**. It indicates that the scenario was focused only on the river entrance, with only minor scour introduced in the approach to the entrance. Final scoured riverbed elevations between Sancrox and the river entrance are shown on **Figure 6-7** for Scenario 1.

FIGURE 6-5



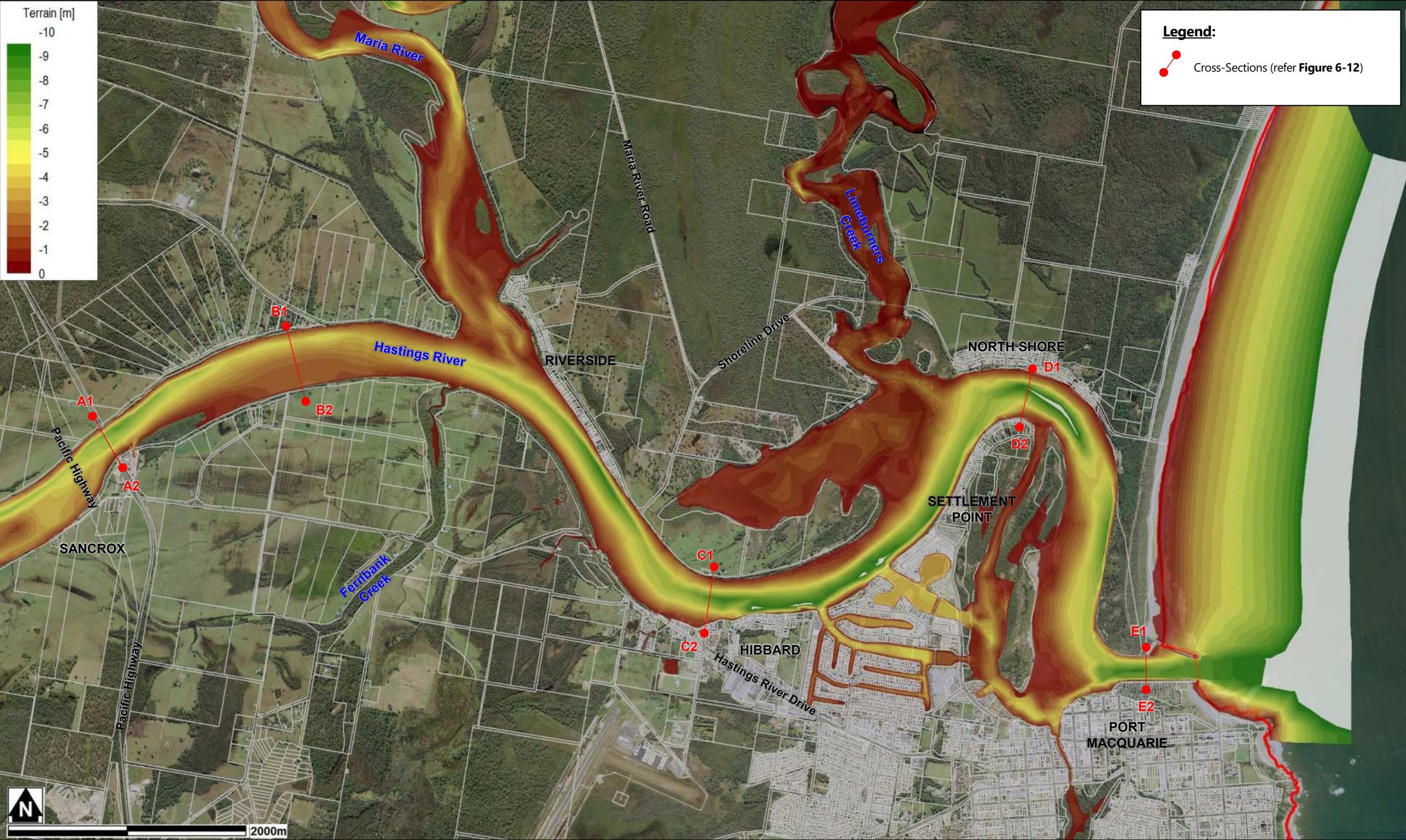
^ Base case scenario refers to the bed elevations adopted in the Hastings River RMA-2 model used for the 'Hastings River Flood Study Extension' (in draft, 2025)

FIGURE 6-6



^ Changes are based on a topographic difference map generated by subtracting elevations adopted for the 'base case' RMA-2 model (refer **Figure 4-4**) from the 'Scour Scenario 1' RMA-2 model.

FIGURE 6-7



During a presentation to Council and DCCEEW representatives of the results for Dune Breach 1 and River Scour Scenario 1, it was requested that consideration be given to incorporating scour further upstream along the Hastings River. Reference was made to the higher flow velocities of up to 4.4 m/s that extend through the Pacific Highway and Hastings River Drive (Dennis Bridge) bridge crossings through to Hibbard. Along this reach, in bank flow velocities are predicted to be typically higher than 2.4 m/s at the peak of the PMF. These values are based on the PMF modelling results pre-inclusion of any dune breach and riverbed scour.

Accordingly, the following additional river scour scenarios were subsequently tested.

- **River Scour Scenario 2** – Expanded scour scenario modelled with a static riverbed surface. The scour scenario adopts riverbed elevations from River Scour Scenario 1 (i.e., scour at the river entrance) and applies up to 1 metre of scour as far upstream as Dennis Bridge and the Pacific Highway crossing at Sancrox.

The extent of scour incorporated under River Scour Scenario 2 is shown in **Figure 6-8**. Scoured riverbed elevations between Sancrox and the river entrance are shown on **Figure 6-9**.

- **River Scour Scenario 3** – River Scour Scenario 3 incorporates a further 1 metre of scour in addition to the scour already incorporated for Scour Scenario 2. An exception to this is along the thalweg where bed elevations were not lowered further than what had been incorporated for Scenario 2. The extent and depths of the incorporated scour is shown in **Figure 6-10**. Scoured riverbed elevations between Sancrox and the river entrance are shown on **Figure 6-11**.

The consistent elevation along the thalweg (between River Scour Scenarios 2 and 3) is better indicated by the cross-sections included in **Figure 6-12**. Cross-sections A-A to D-D all show how the additional 1-metre of scour was limited to the edges of the riverbed. Cross-section E-E shows that scour at the river entrance was modelled consistently between all three scenarios.

6.6.2 RMA-2 Modelling Results

Dune Breach and Riverbed Scour Scenarios in Isolation

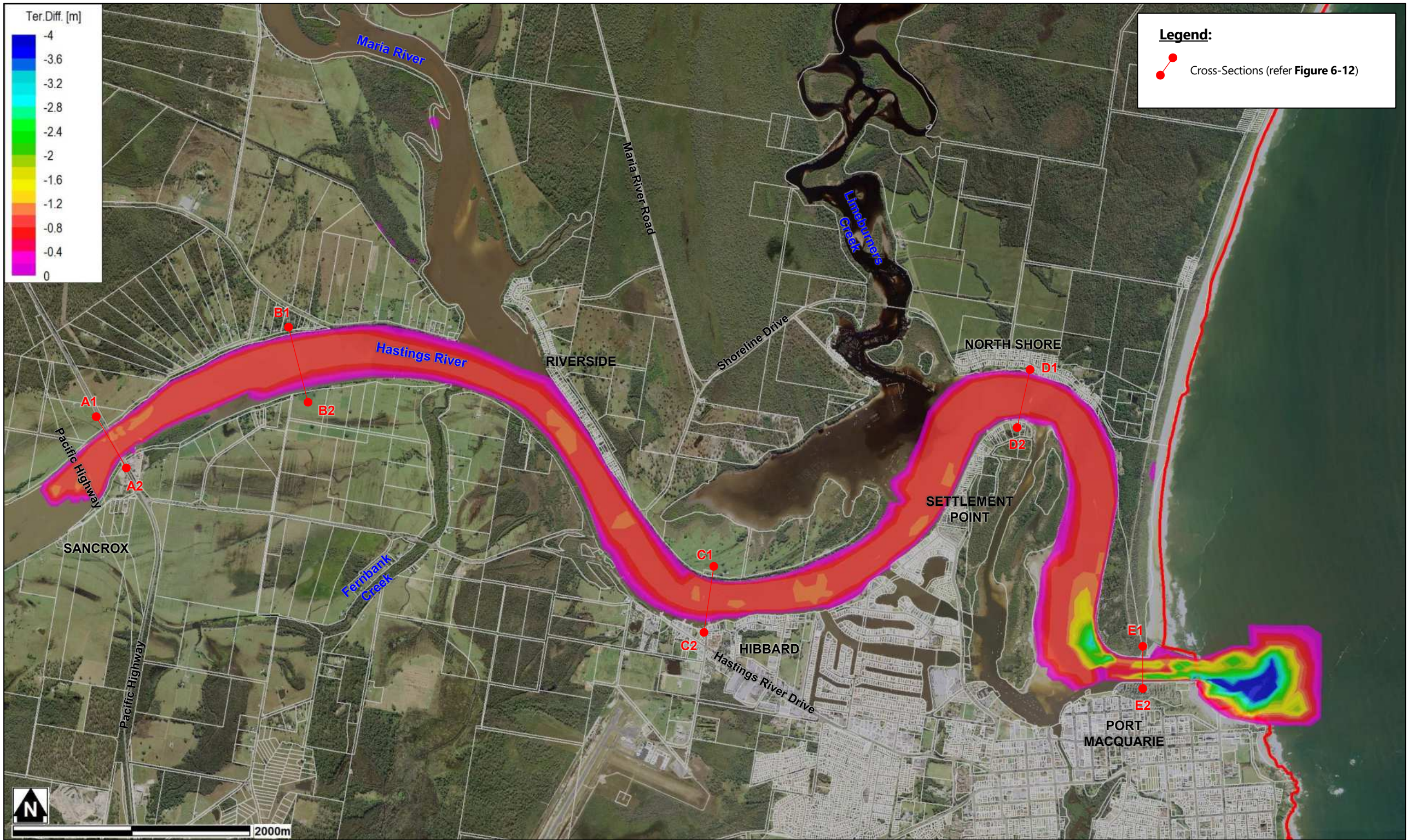
Each of the scour scenarios described in **Section 6.6.1** were incorporated into the Hastings River RMA-2 flood model. It is noted that two versions of Dune Breach Scenario 1 and River Scour Scenario 1 were modelled, one with the breach and scour added dynamically as described in **Section 6.6.1**, and another with the ultimate scoured state modelled from the start of the simulation, i.e., as a static condition. This was undertaken to assess the sensitivity of peak flood levels to the set-up of the scour scenario.

Peak flood levels for the PMF event are listed in **Table 6-18** for key locations and for each of the dune breach and river scour scenarios. Values in the blue brackets show the difference in peak flood levels when compared to the results of a PMF simulation without any dune breach and riverbed scour (i.e., the base case). The values in the red brackets are a comparison between peak PMF levels predicted using a dynamic or static modelling approach.

Flood level difference mapping is presented in **Figures 6-13 to 6-18**. The key findings from the modelling are summarised in the following bullet points.

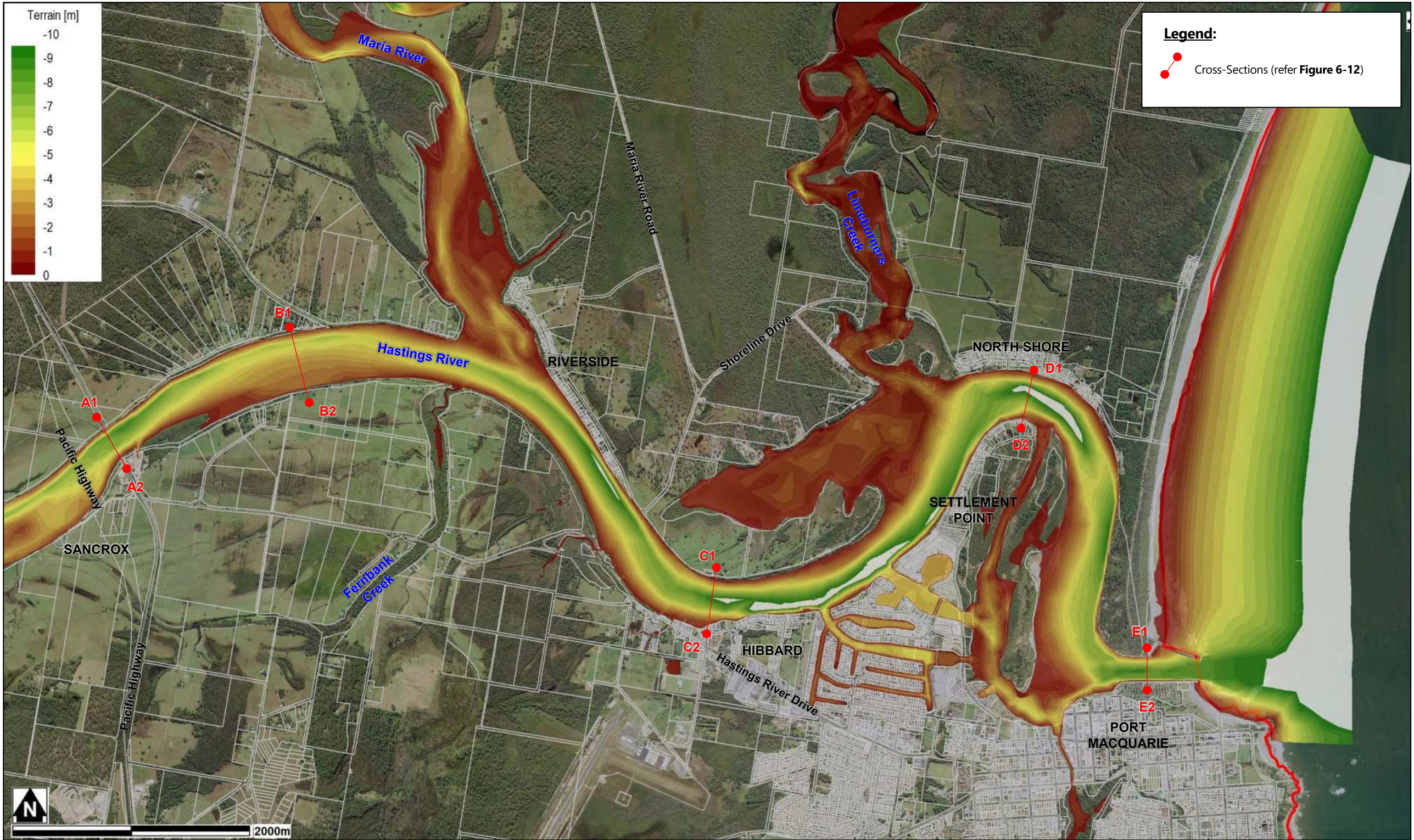
- The dune breach scenario (both dynamic and static) is predicted to cause a larger reduction in peak PMF flood levels than either entrance or riverbed scour. As shown in **Table 6-5**, Dune Breach Scenario 1 is predicted to reduce peak flood levels for the PMF by up to 0.56 metres at North Shore, 0.57 metres across Kooloonbung Creek and Kooloonbung Swamp, 0.43 metres at Hibbard and 0.18 metres near Riverside and the Maria and Hastings River confluence.
- Upstream of the Maria River confluence, the entrance and riverbed scour scenarios start to dominate, with River Scour Scenarios 2 and 3 predicted to lower flood levels by a higher margin. The reduction in peak PMF flood levels at the Dennis Road Bridge at Sancrox is predicted to be 0.22 metres for River Scour Scenario 2, compared to 0.09 metres for Dune Breach Scenario 1 (refer **Table 6-5**).

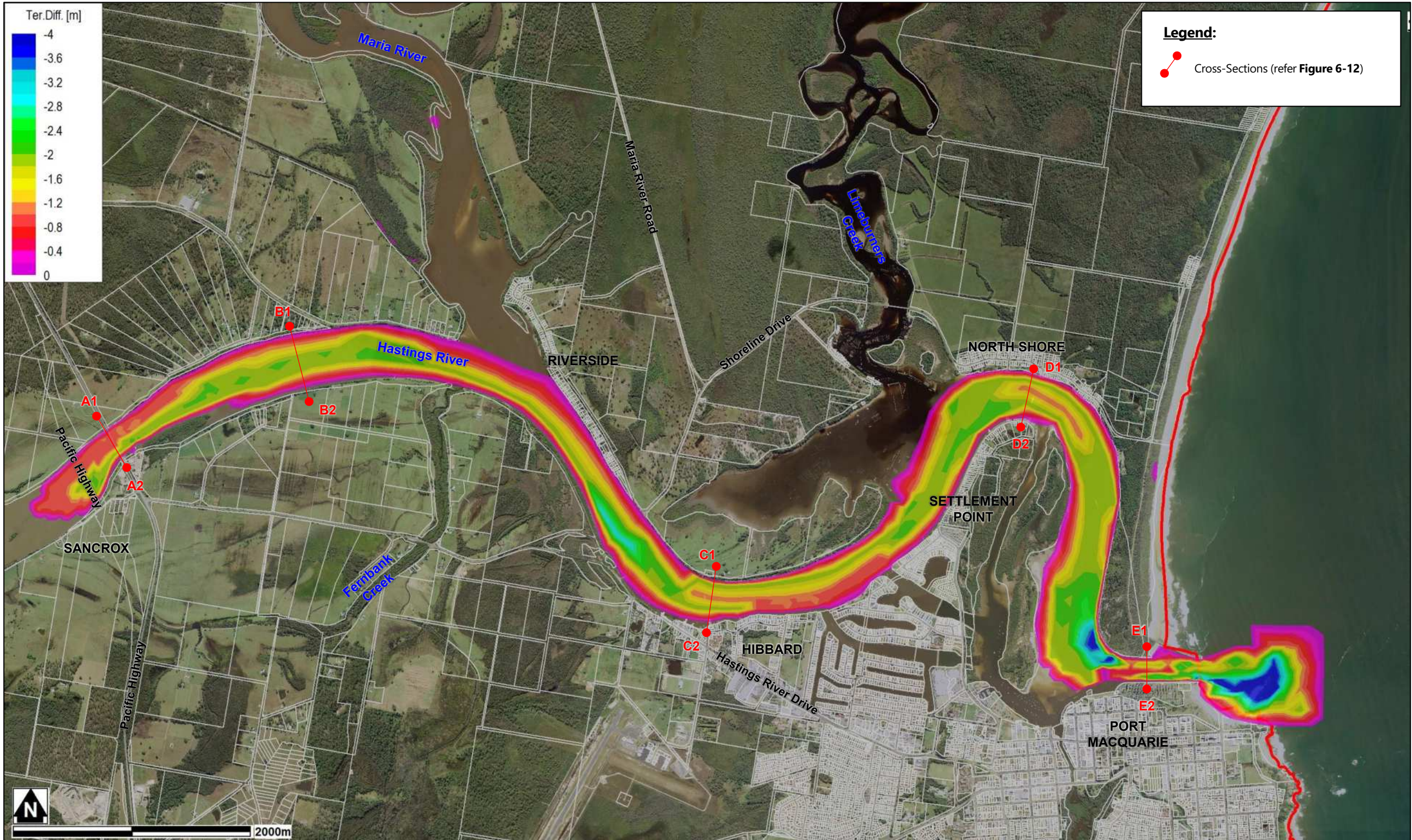
FIGURE 6-8



^ Changes are based on a topographic difference map generated by subtracting elevations adopted for the 'base case' RMA-2 model (refer **Figure 4-4**) from the 'Scour Scenario 2' RMA-2 model.

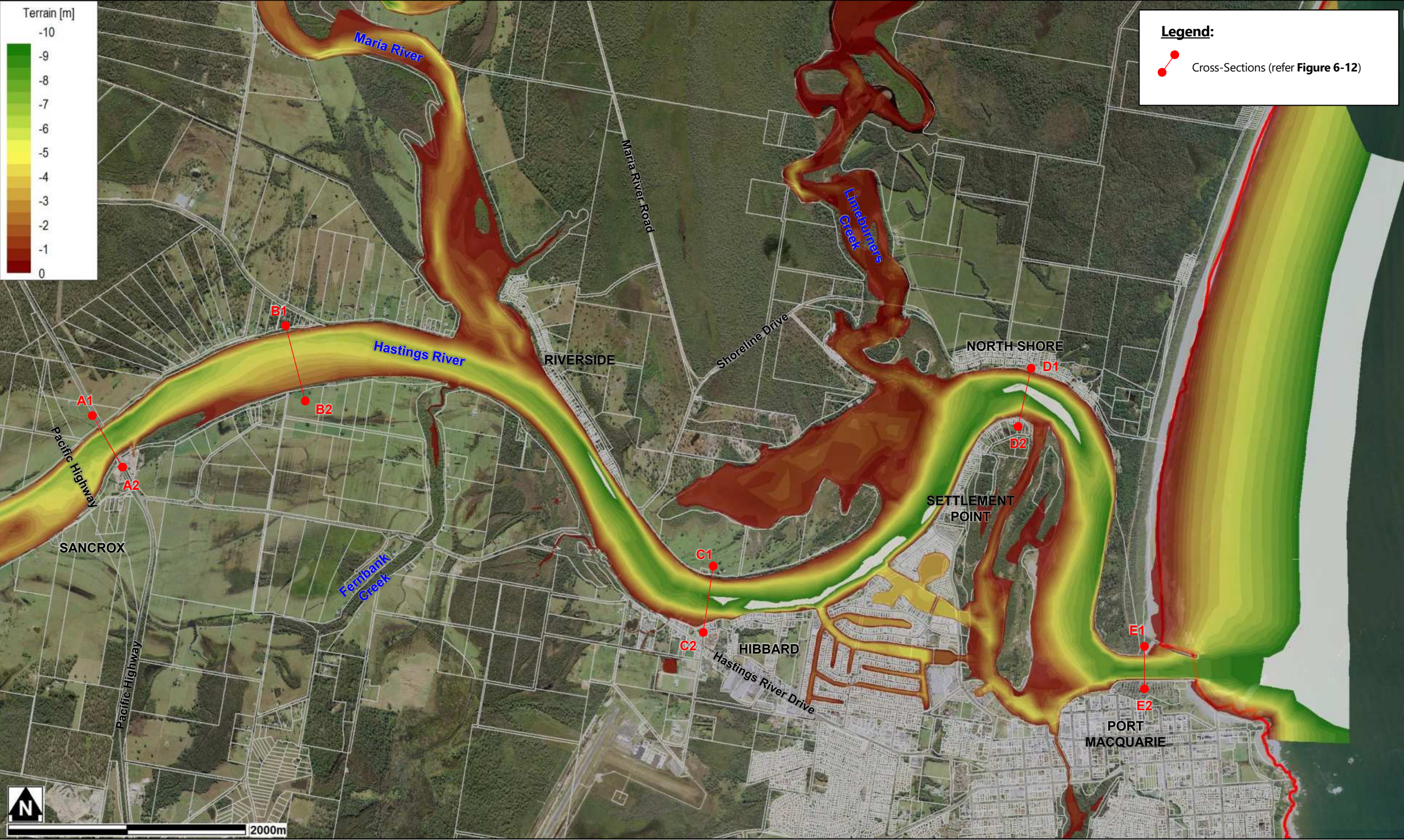
FIGURE 6-9



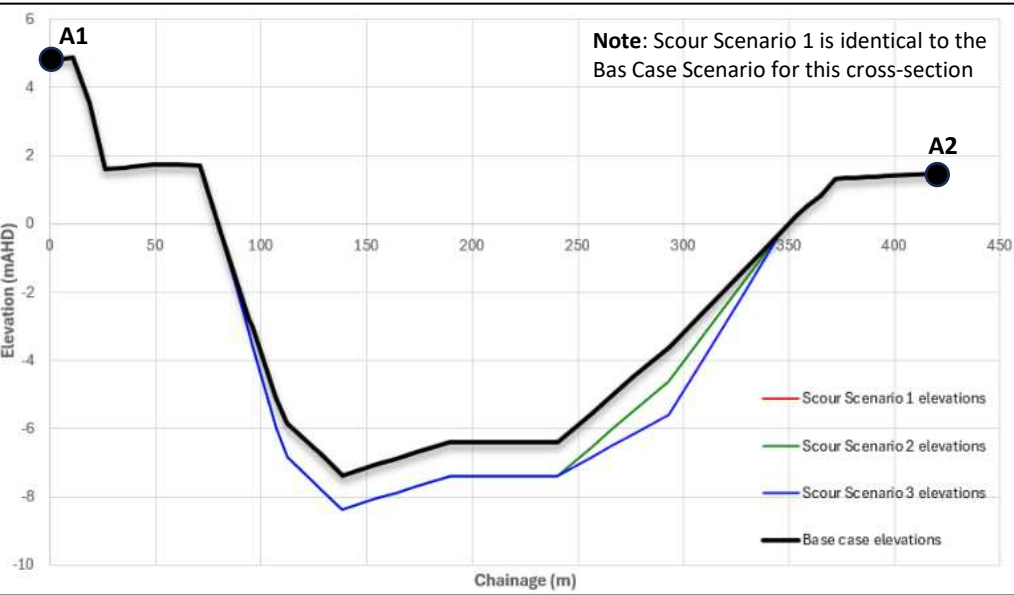


^ Changes are based on a topographic difference map generated by subtracting elevations adopted for the 'base case' RMA-2 model (refer **Figure 4-4**) from the 'Scour Scenario 3' RMA-2 model.

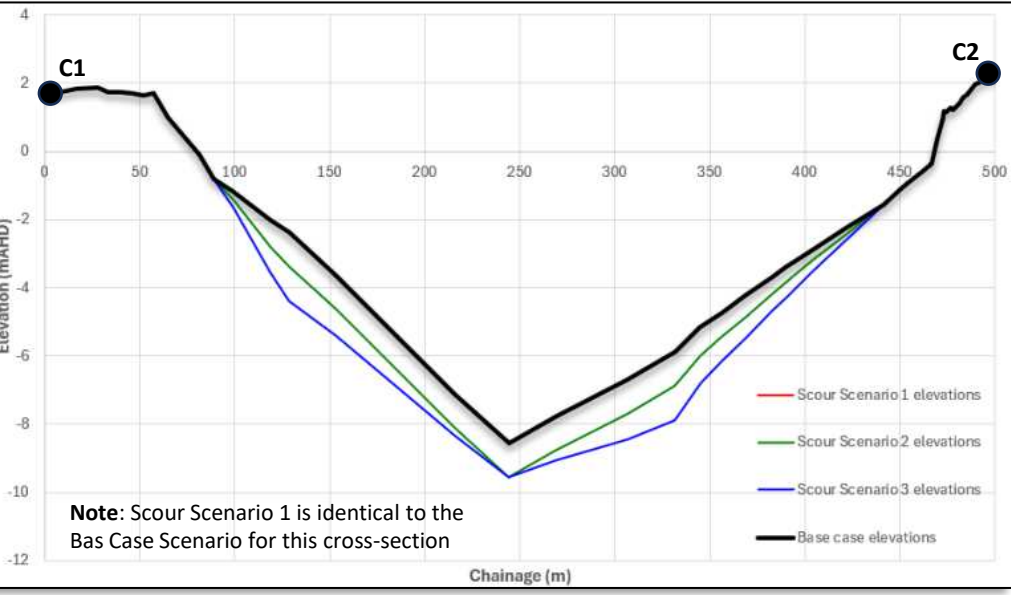
FIGURE 6-11



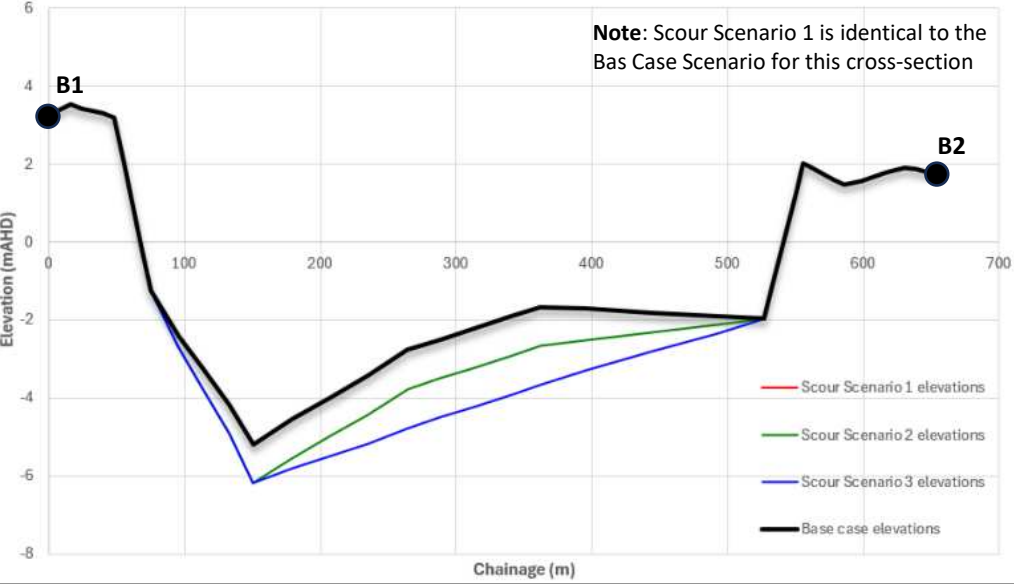
Cross-Section A1-A2 – Dennis Bridge



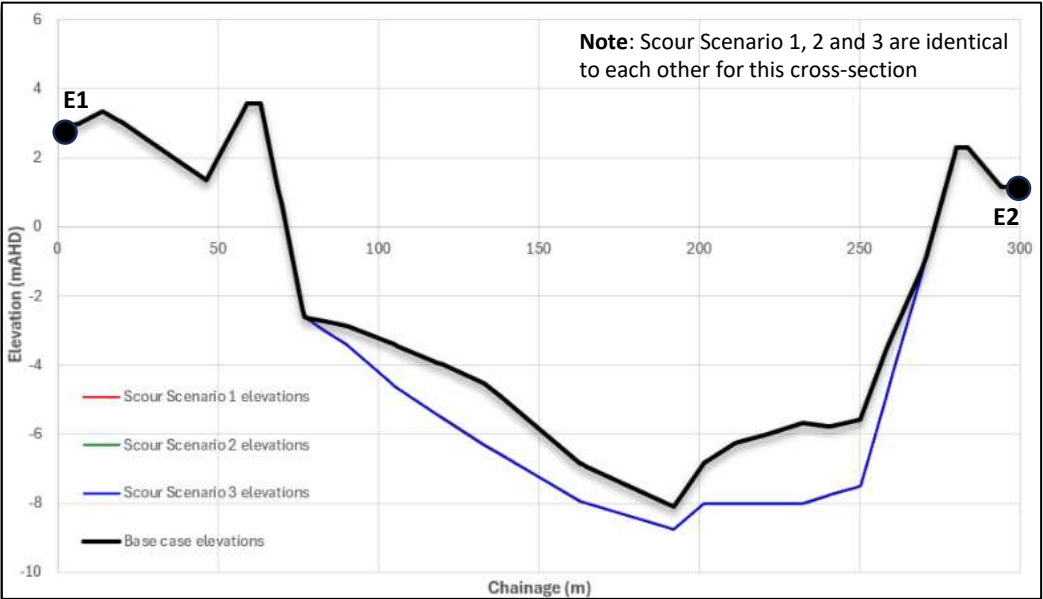
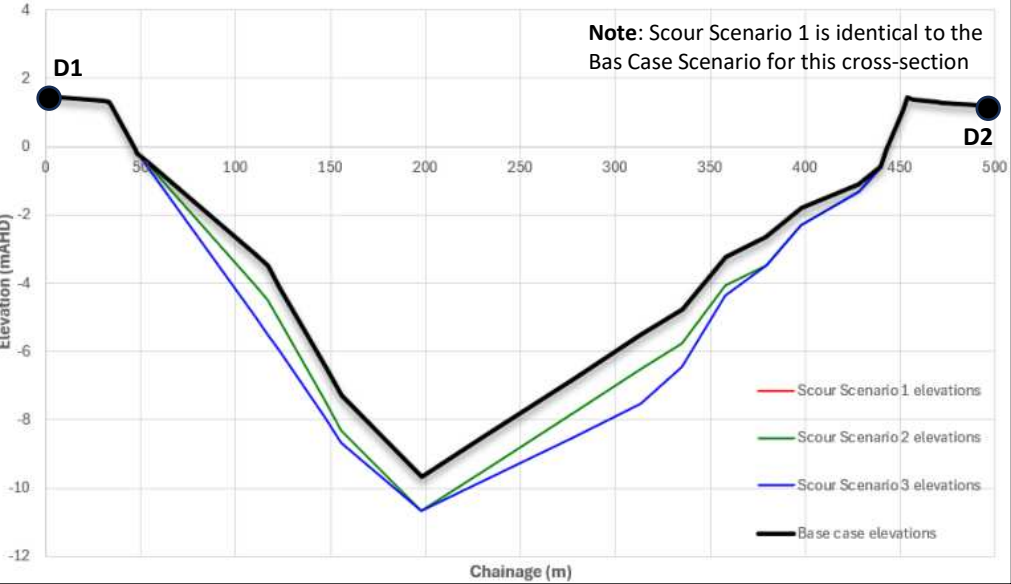
Cross-Section C1-C2 – Hibbard / Boundary Street



Cross-Section B1-B2 – Between Sancrox and Maria River



Cross-Section D1-D2 – North Shore Ferry



Cross-Section E1-E2
– Port Macquarie / River Entrance

CROSS-SECTION PLOTS COMPARING
HASTINGS RIVER-BED ELEVATIONS FOR
THE BASE CASE AND SCOUR SCENARIOS

FIGURE 6-13

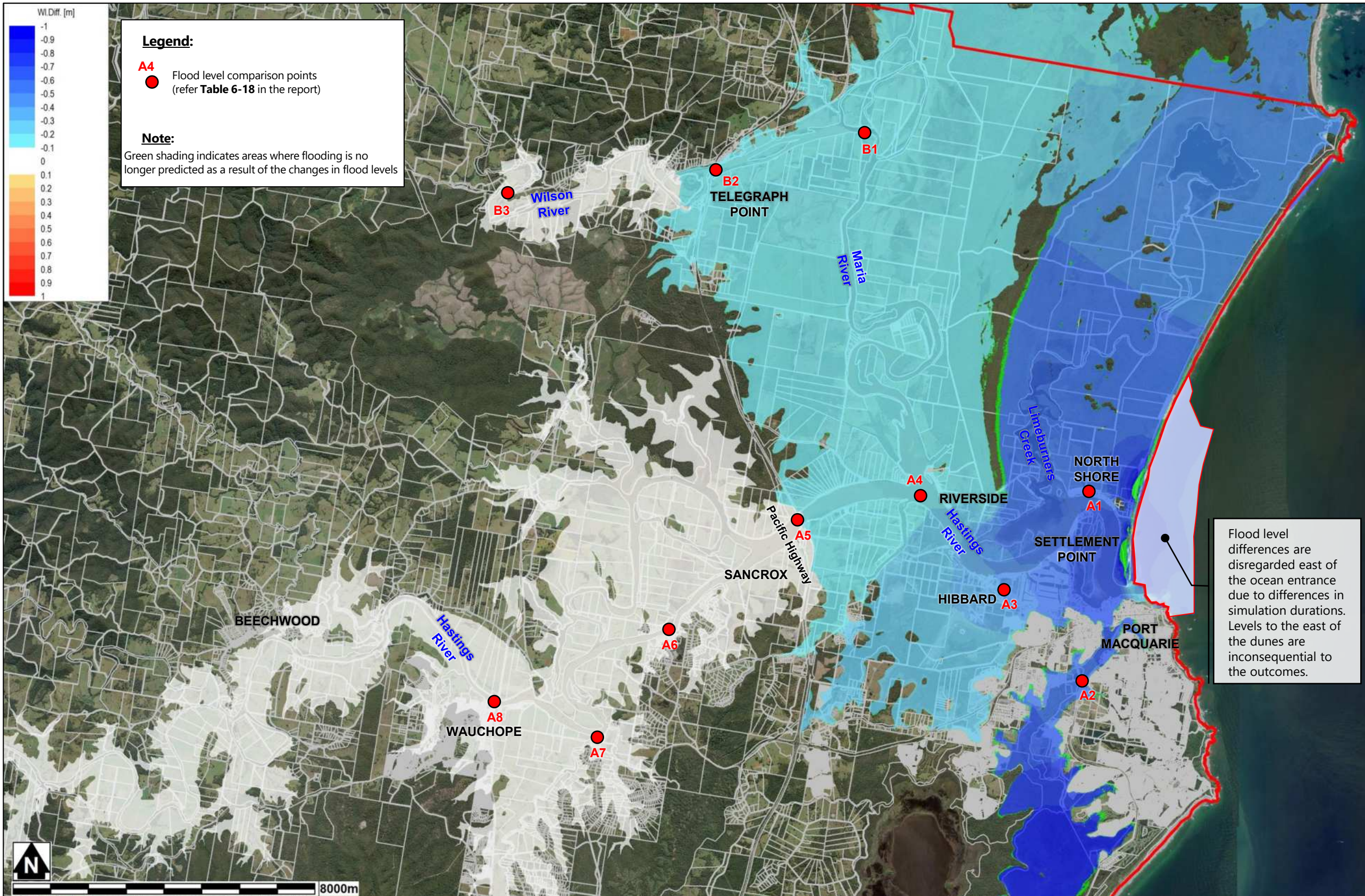


FIGURE 6-14

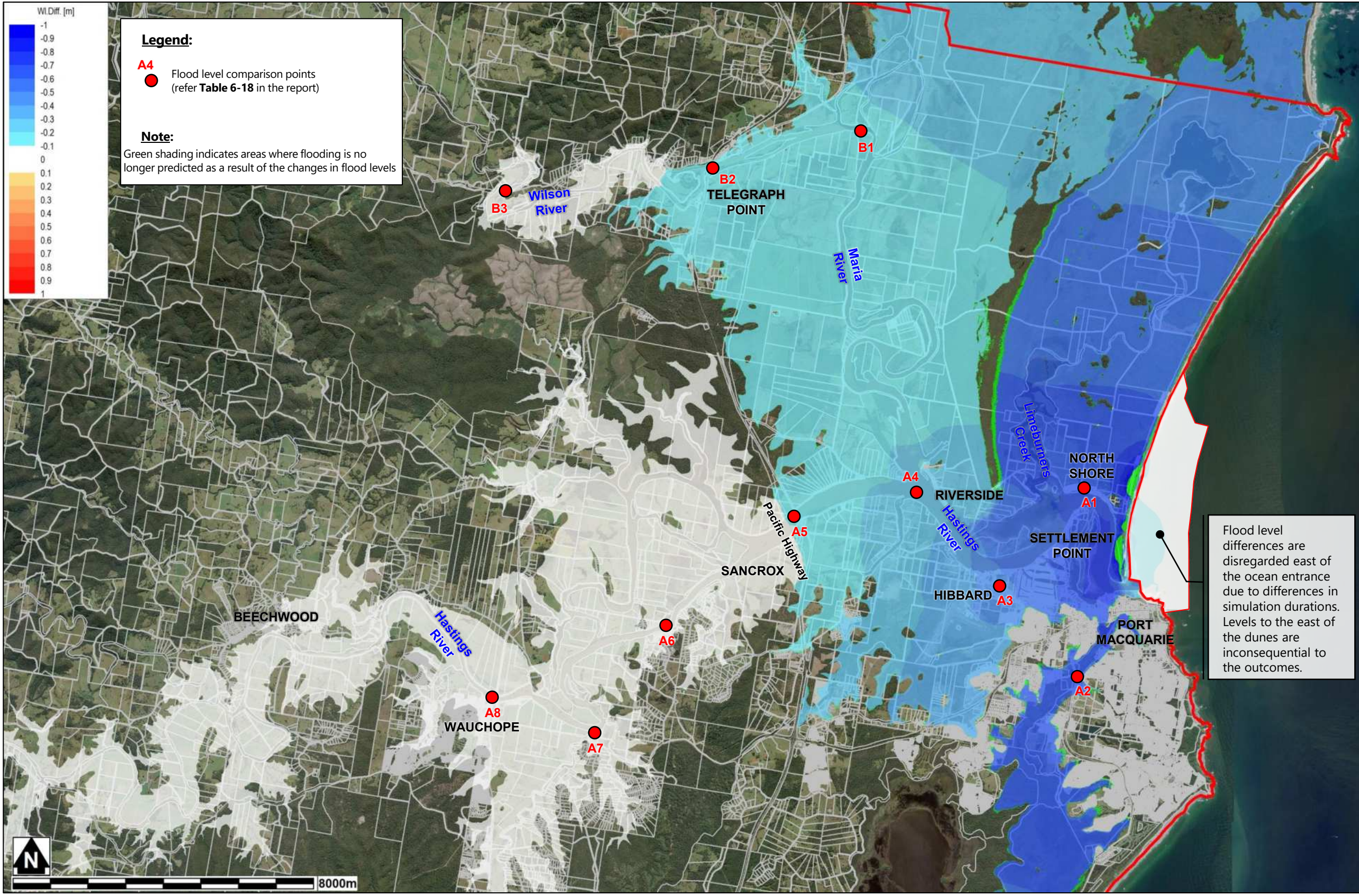


FIGURE 6-15

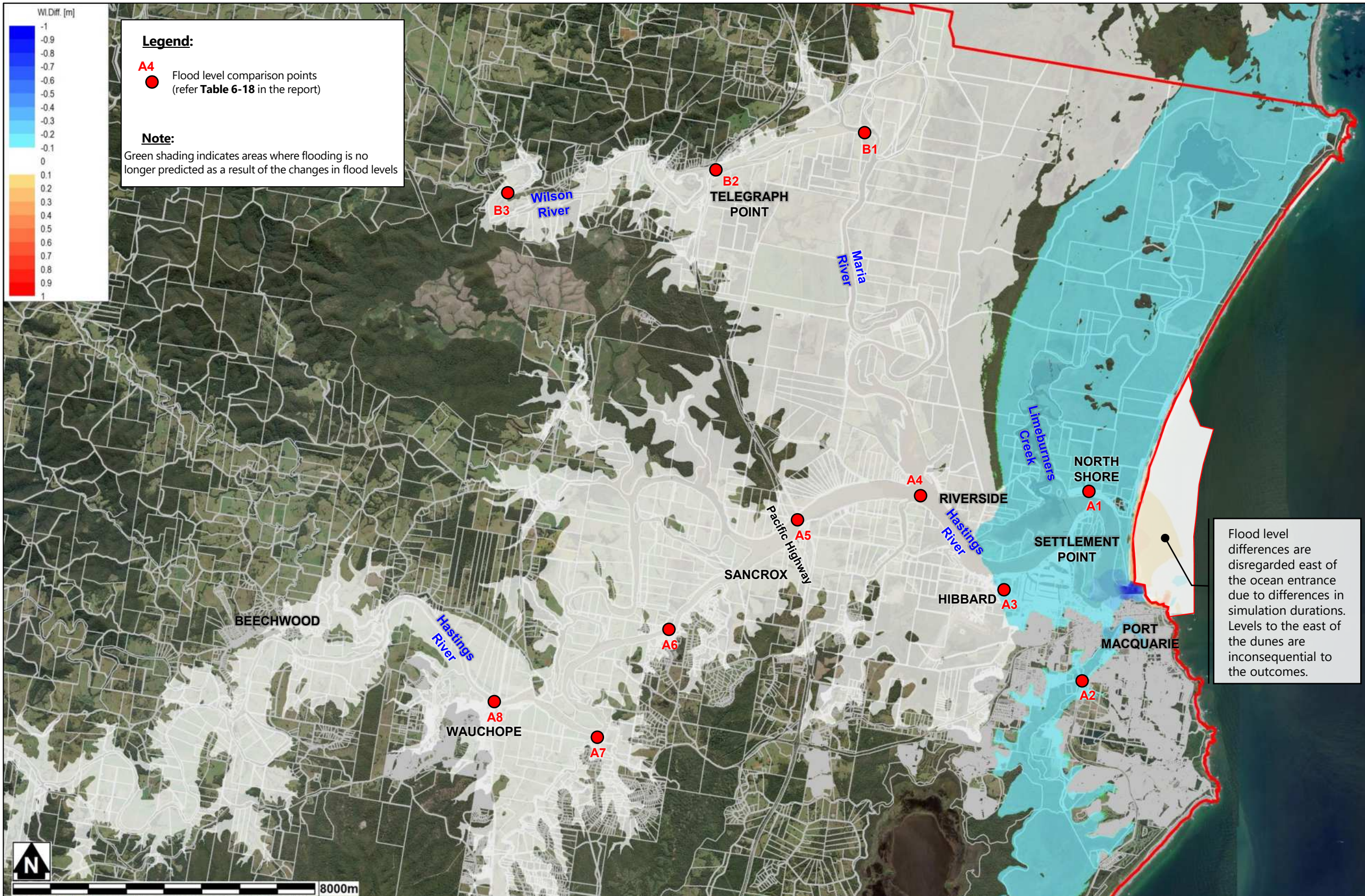


FIGURE 6-16

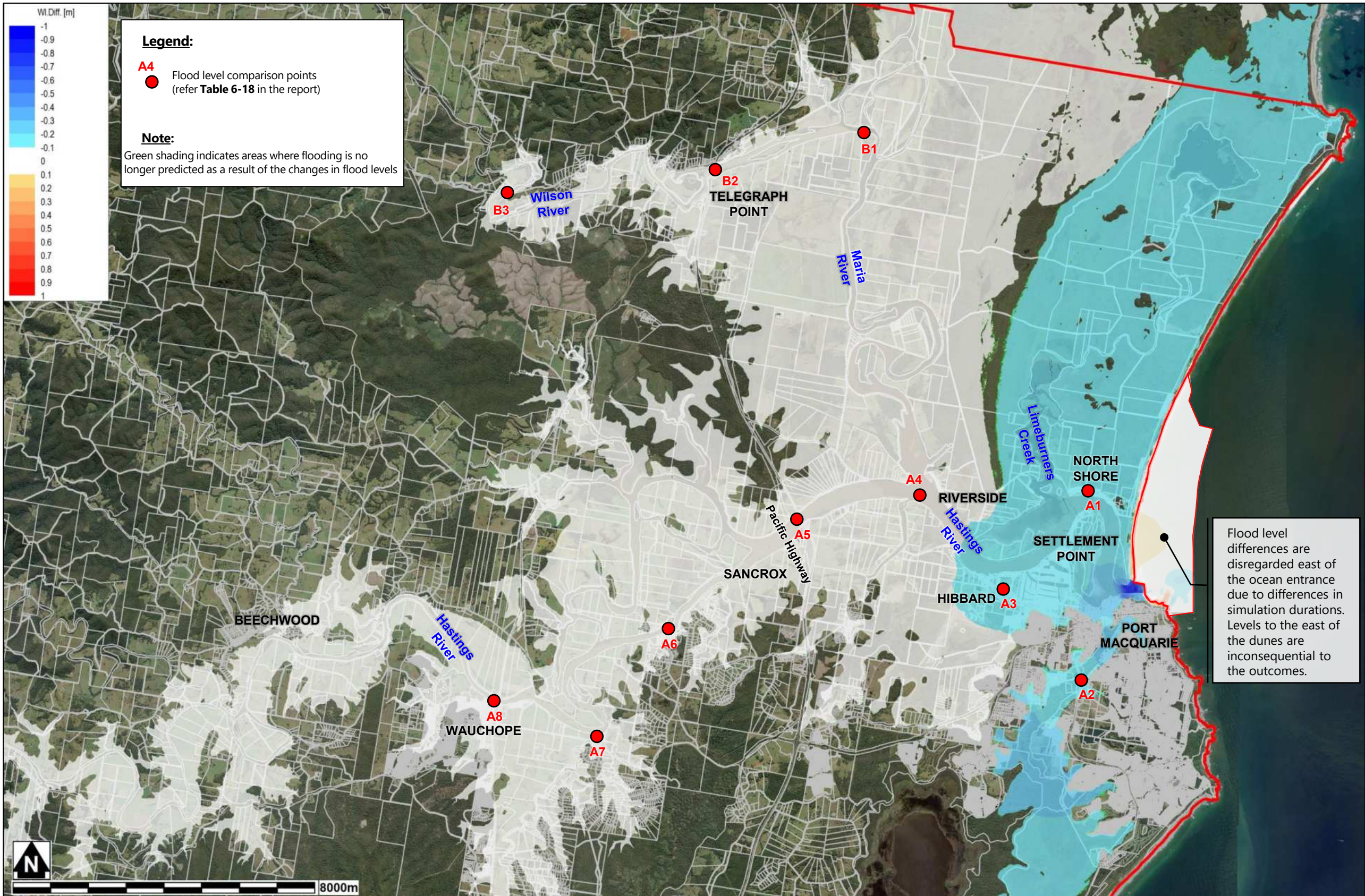


FIGURE 6-17

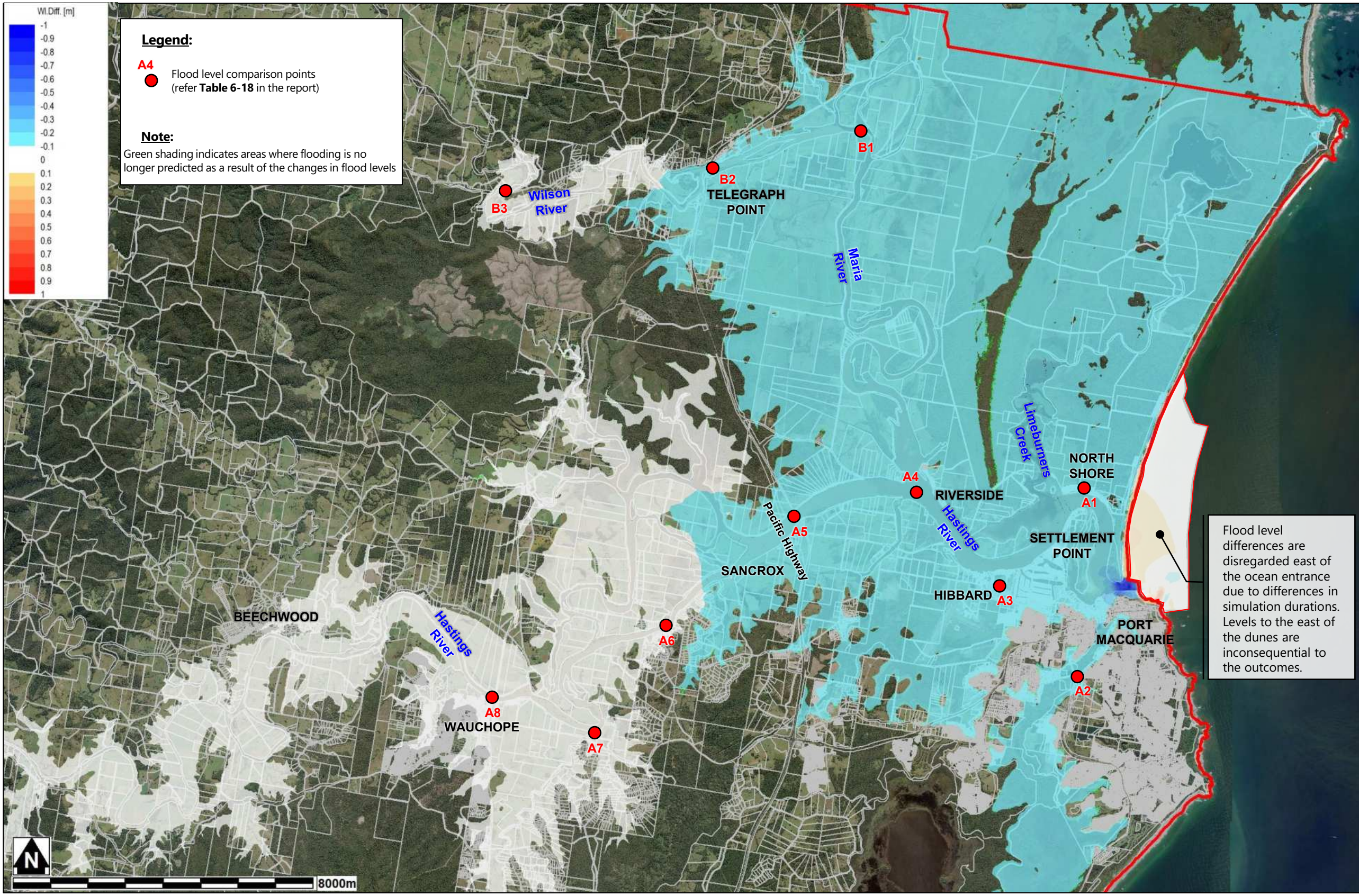


FIGURE 6-18

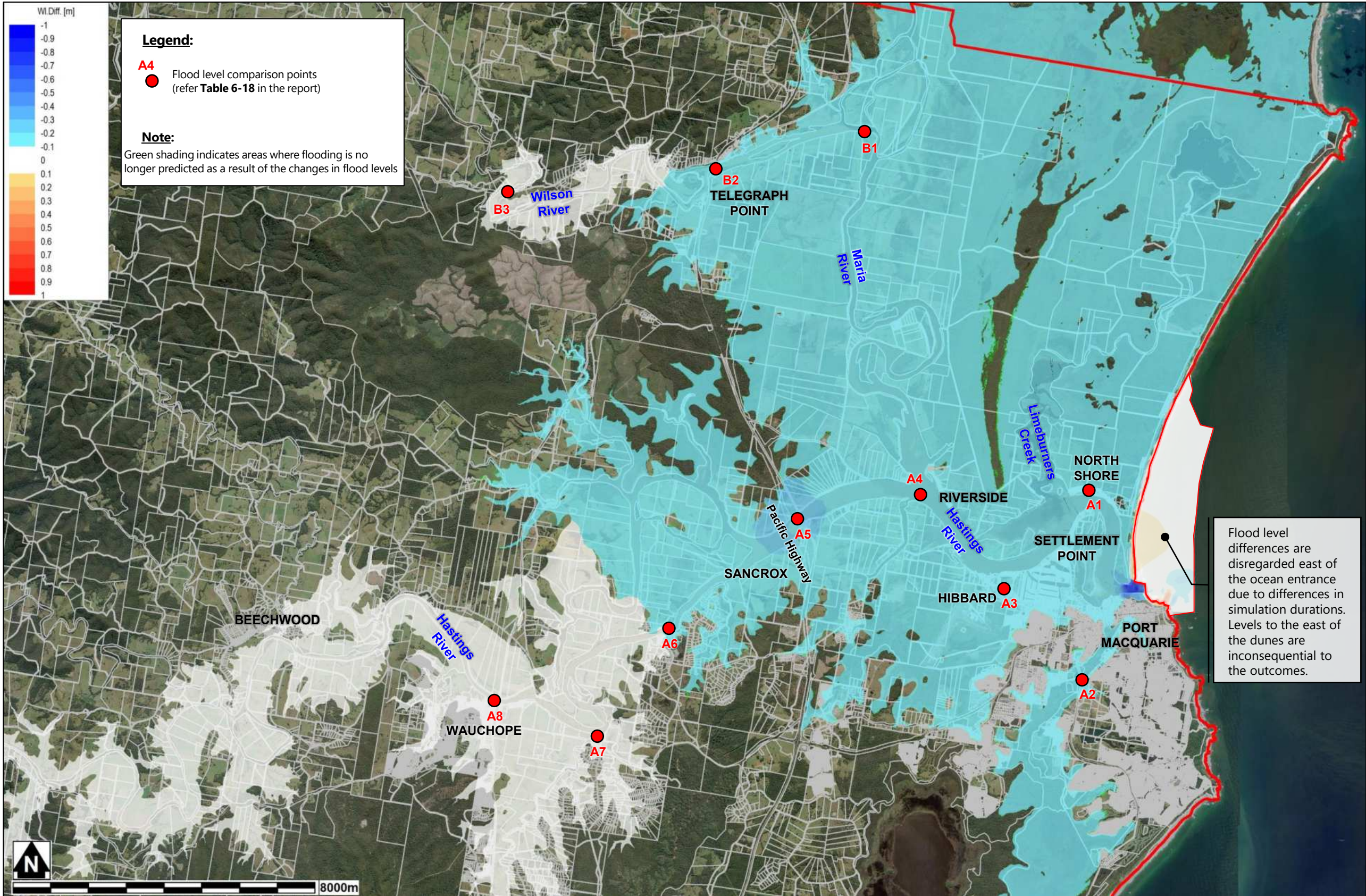


Table 6-5 Comparison of Predicted Peak PMF Levels at Key Locations for Each of the Dune Breach and Riverbed Scour Scenarios

Location	Point ID (Refer Fig 6-13)	Predicted Peak Flood Level (mAHD)						
		Base Case [^]	Dune Breach 1		Scour Scenario 1		Scour Scenario 2	Scour Scenario 3
			Dynamic	Static	Dynamic	Static	Static	Static
Locations along the Hastings River								
North Shore	A1	6.96	6.40 (-0.56)	6.35 (-0.61) (-0.05)	6.84 (-0.12)	6.83 (-0.13) (-0.01)	6.82 (-0.14)	6.81 (-0.15)
Kooloonbung Creek / Lake Road	A2	5.50	4.93 (-0.57)	4.88 (-0.62) (-0.05)	5.32 (-0.18)	5.30 (-0.20) (-0.02)	5.33 (-0.17)	5.34 (-0.16)
Hibbard	A3	7.18	6.75 (-0.43)	6.70 (-0.48) (-0.05)	7.07 (-0.11)	7.05 (-0.13) (-0.02)	7.03 (-0.15)	7.02 (-0.16)
Riverside / Hastings and Maria River Confluence	A4	7.79	7.61 (-0.18)	7.57 (-0.22) (-0.04)	7.73 (-0.06)	7.71 (-0.08) (-0.02)	7.66 (-0.13)	7.63 (-0.16)
Sancrox / Dennis Bridge	A5	8.32	8.23 (-0.09)	8.20 (-0.12) (-0.03)	8.28 (-0.04)	8.27 (-0.05) (-0.01)	8.17 (-0.15)	8.10 (-0.22)
Rawdon Island Road Bridge over the Hastings River	A6	10.33	10.32 (-0.01)	10.31 (-0.02) (-0.01)	10.32 (-0.01)	10.32 (-0.01) (-0.00)	10.26 (-0.07)	10.23 (-0.10)
Hastings River and Kings Creek Confluence	A7	13.72	13.72 (-0.00)	13.72 (-0.00) (-0.00)	13.72 (-0.00)	13.72 (-0.00) (-0.00)	13.71 (-0.01)	13.70 (-0.02)
Wauchope Railway Bridge	A8	14.81	14.81 (-0.00)	14.81 (-0.00) (-0.00)	14.81 (-0.00)	14.81 (-0.00) (-0.00)	14.80 (-0.01)	14.80 (-0.01)
Locations along the Wilson and Maria Rivers								
Wilson and Maria River confluence	B1	7.64	7.49 (-0.15)	7.45 (-0.19) (-0.04)	7.59 (-0.05)	7.57 (-0.07) (-0.02)	7.52 (-0.12)	7.49 (-0.15)
Telegraph Point Road crossing	B2	7.73	7.60 (-0.13)	7.57 (-0.16) (-0.03)	7.68 (-0.05)	7.66 (-0.07) (-0.02)	7.61 (-0.12)	7.58 (-0.15)
Rollands Plains Road crossing	B3	15.19	15.19 (-0.00)	15.17 (-0.02) (-0.02)	15.19 (-0.00)	15.19 (-0.00) (-0.00)	15.19 (-0.00)	15.18 (-0.01)

[^] Peak PMF levels pre-inclusion of any dune breach and/or riverbed scour

(-0.00) Difference in flood level compared to the base case

(-0.00) Difference in flood level compared to the dynamic modelling scenario

Note: Highlighted cells identify the scenario that produced the largest reduction in peak PMF levels

- Changes to peak PMF flood levels are negligible from Wauchope upstream with a maximum difference of only 0.01 metres for all modelled dune breach and river scour scenarios.
- Peak PMF levels were found to have a low sensitivity to the magnitude of the riverbed scour applied. As shown in **Table 6-5**, the difference in peak PMF levels were typically less than 0.1 meters between the three river scour scenarios. This includes comparison between River Scour Scenario 1 and Scenario 3, which between the river entrance and Dennis Bridge have up to 2 metres difference in the scour depth. This suggests that further refinement of the river scour scenarios would result in very little difference in the predicted peak PMF levels.
- Peak PMF levels predicted based on the dynamic and static modelling approaches were very similar and within +/- 0.05 metres at all locations. The difference was closest for River Scour Scenario 1 with peak flood levels varying by only +/- 0.02 metres (refer **Table 6-5**).

It is concluded that the river scour scenario can be modelled using a static approach without any significant impact on peak flood levels. Further investigation of the dynamic and static dune breach scenarios found that differences in peak flood levels could be reduced to +/- 0.03 metres if the minimum crest elevation for each dune breach were raised from 2 mAHD to 3 mAHD.

Raising the minimum crest elevation also has the benefit of limiting the additional conveyance afforded by the breaches before the sand dunes would be overtopped under existing conditions. In that regard, and as shown in **Figure 6-4**, the sand dunes have an existing low-point at an elevation of 3.5 mAHD.

Combined Dune Breach and Riverbed Scour Scenario

A combined dune breach and river scour scenario appropriate for the Hastings River during a PMF event was developed based on the modelling results for each in isolation which showed that one scenario would not prevent the other from being triggered. For example, inclusion of River Scour Scenarios 1 to 3 is predicted to reduce peak PMF levels along the sand dunes by up to 0.15 metres. The low-points across which Dune Breach Scenario 1 was modelled would still be overtopped by depths of 1.85 to 2.85 metres under this scenario. Accordingly, inclusion of the river scour scenarios would not prevent breach of the sand dunes.

Similarly, modelling results for Dune Breach Scenario 1 indicate that velocities through the river entrance would decrease by less than 0.8 m/s resulting in peak flow velocities of up to 8.7 m/s. Accordingly, entrance scour would still be expected.

Inclusion of the dune breach was also found to cause increased flow velocities along the Hastings River between Hibbard and the river entrance. These increases occur as floodwaters discharge more freely toward the sand dunes and through the modelled dune breach locations. Peak PMF flow velocities are predicted to increase by up to 0.4 m/s as far upstream as Hibbard.

The following scenarios were adopted for the combined dune breach and riverbed scour scenario:

- Modified version of Dune Breach Scenario 1 incorporated as a static breach with minimum crest elevations for all breach locations set to 3 mAHD, and
- River Scour Scenario 3.

Peak PMF flood levels for the base case and combined dune breach and riverbed scour scenario are listed in **Table 6-19** for key locations in the study area. Values in the blue brackets show the difference in peak flood levels when compared to the base scenario without any dune breach or riverbed scour. Flood level difference mapping is also presented in **Figure 6-19**.

Table 6-19 Comparison of Predicted Peak PMF Levels at Key Locations for the base case scenario and the combined dune breach and riverbed scour scenario

Location	Point ID (Refer Fig 6-19)	Predicted Peak Flood Level (mAHD)	
		Base Case ^	Breach + Scour Scenario
Locations along the Hastings River			
North Shore	A1	6.96	6.37 (-0.59)
Kooloonbung Creek / Lake Road	A2	5.50	4.90 (-0.60)
Hibbard	A3	7.18	6.67 (-0.51)
Riverside / Hastings and Maria River Confluence	A4	7.79	7.47 (-0.32)
Sancrox / Dennis Bridge	A5	8.32	7.98 (-0.34)
Rawdon Island Road Bridge over the Hastings River	A6	10.33	10.20 (-0.13)
Hastings River and Kings Creek Confluence	A7	13.72	13.70 (-0.02)
Wauchope Railway Bridge	A8	14.81	14.80 (-0.01)
Locations along the Wilson and Maria Rivers			
Wilson and Maria River confluence	B1	7.64	7.34 (-0.30)
Telegraph Point Road crossing	B2	7.73	7.45 (-0.28)
Rollands Plains Road crossing	B3	15.19	15.19 (-0.00)

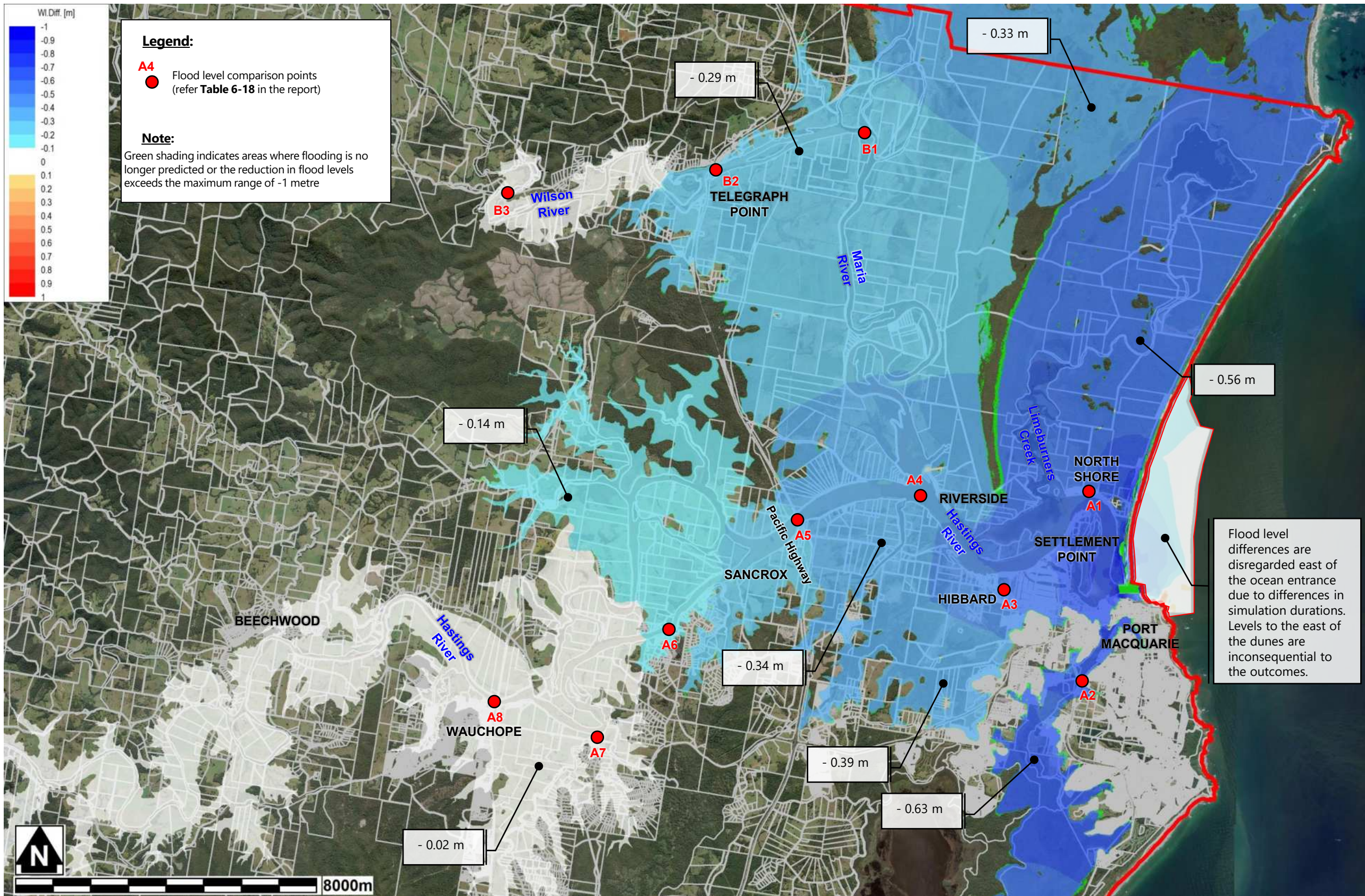
[^] Peak PMF levels pre-inclusion of any dune breach and/or riverbed scour
(-0.00) Difference in flood level compared to the base case

6.6.3 Conclusion

The combined dune breach (Dune Breach Scenario 1) and river scour scenario (River Scour Scenario 3) is to be included in modelling of the PMF event for the Hastings River catchment. The scenario reflects a reasonable estimate of the conditions that would be expected to manifest during a PMF event. Modelling for alternate riverbed scour scenarios has shown that the differences to peak flood levels during a PMF event would be minor and within +/- 0.10 metres. Further modelling or refinement of the adopted scenario is not considered warranted as it would result in only minor changes to flood levels at the peak of the PMF.

Modelling has also confirmed that there are only minimal differences between the dune breach and river scour scenarios being modelled dynamically or as a static surface. By raising minimum crest elevations for the dune breach scenario from 2.0 mAHD to 3.0 mAHD the impact on peak flood levels is expected to be in the order of +/- 0.02 metres for the PMF event.

FIGURE 6-19



7 Design Event Results

7.1 Major Rivers RMA-2 Hydraulic Model

7.1.1 Design Flood Mapping

Design flood mapping produced using the RMA-2 hydraulic model is provided in **Appendix E**. This includes mapping of peak flood levels and depths and flow velocities in accordance with the following figure breakdown:

- Predicted Peak 5% AEP Flood Levels (refer **Figure E-1** to **Figure E-8**)
- Predicted Peak 1% AEP Flood Levels (refer **Figure E-9** to **Figure E-16**)
- Predicted Peak 1% AEP with Climate Change Flood Levels (refer **Figure E-17** to **Figure E-24**)
- Predicted Peak PMF Flood Levels (refer **Figure E-25** to **Figure E-32**)
- Predicted Peak 5% AEP Depths & Velocities (refer **Figure E-33** to **Figure E-40**)
- Predicted Peak 1% AEP Depths & Velocities (refer **Figure E-41** to **Figure E-48**)
- Predicted Peak 1% AEP with Climate Change Depths & Velocities (refer **Figure E-49** to **Figure E-56**)
- Predicted Peak PMF Depths & Velocities (refer **Figure E-57** to **Figure E-64**)

The modelling for design events up to and including the 1 in 2000 AEP flood was based on the use of the RMA-2 model network adopted for calibration to the March 2021 event combined with flow hydrographs described in **Section 6.5**. Modelling for the PMF event was based on a version of the RMA-2 model with the final recommended dune breach and riverbed scour conditions included (refer **Section 6.7**).

The flood mapping represents peak flood conditions produced by a process of 'flood enveloping'. For each design AEP, this process combines maximum flood level results from the adopted storm durations to produce a 'design flood envelope'. Peak flow magnitudes for these design event durations are listed in **Tables 6-14** and **6-15** for key locations throughout the RMA-2 model extent.

The storm durations used to produce the peak design flood envelopes are summarised in **Table 7-1**. Mapping for the 1% AEP event showing where each critical duration applies is shown as **Figure 7-1**.

7.1.2 Peak Flood Levels

Peak flood levels for the full range of design events are listed below in **Table 7-2** on the following page. The selected locations for comparison are indicated on **Figures E1** to **E-64**.

7.1.3 Comparison to Previous Studies

Flood level difference maps have been prepared to compare the extended Hastings River Flood RMA-2 results to those generated as part of the *Hibbard Precinct Flood Study (2019)*. The comparison plots have been prepared to cover the entire model domain for the 5% and 1% AEP floods and the 1% AEP with climate change event.

Flood level difference maps have also been prepared to compare the extended RMA-2 results for the PMF event to the Extreme event documented in the *Hastings River Flood Study (2006)* which is currently adopted by Council.

The flood level difference mapping is presented in **Figure F1** to **F12** in **Appendix F**.

Table 7-1 Summary of scenarios used to produce peak design flood envelopes

Design Flood Event	Storm Duration for Local Catchment Flows	Downstream Boundary Conditions
5% AEP	18 & 48 hours	Macleay River Flood – 5% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd
2% AEP	18 & 48 hours	Macleay River Flood – 5% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd
1% AEP	18 & 48 hours	Macleay River Flood – 5% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd
1% AEP Climate Change	18 & 48 hours	Macleay River Flood – 5% AEP [^] Ocean Boundary – Tidal varying with peak of 3.1 mAHd
1 in 200 AEP	18 & 48 hours	Macleay River Flood – 5% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd
1 in 500 AEP	18 & 48 hours	Macleay River Flood – 1% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd
1 in 2000 AEP	18 & 48 hours	Macleay River Flood – 1% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd
PMF	24 hours	Macleay River Flood – 1% AEP [^] Ocean Boundary – Tidal varying with peak of 2.2 mAHd

[^] Flood level at the Connection Creek RMA-2 boundary adopt time varying flood levels extracted from flood modelling produced as part of the 'Macleay River Flood Study' (2019).

The followings comments are made regarding the comparison figures.

- The changes to flood levels are predominantly attributed to changes to the inflow hydrographs and to the RMA-2 model extent.
- Changes to the peak magnitude of inflows throughout the model domain (refer **Table 6-16**). As shown, the updated hydrology based on WBNM with ARR19 results in a reduction to peak flows along the Hastings River of between 10% to 21% for events up to and including the 1% AEP flood. Reductions to peak flows of these magnitudes result in a lowering of flood levels around Wauchope, Kings Creek, and Rawdon Island.
- Increased flood levels are predicted downstream of Hibbard, i.e., at Settlement Point, North Shore and Port Macquarie, during the 5% and 1% AEP events, 1% AEP with climate change and the PMF (refer **Figures F-2, F-5, F-8 and F-11**).

The increase in flood levels is a direct outcome of the RMA-2 model calibration, which showed that the original RMA-2 model was underpredicting levels downstream of Hibbard during the 2021 and 2013 events when compared to recorded gauge data and HWMs. This led to a review of riverbed elevations along the Hastings River and at the ocean outlet, combined with the introduction of a salinity component resulting in a higher water density at the ocean entrance and beyond to the tailwater boundary. The introduction of these changes led to an improved calibration to both events resulting in the same parameters being adopted for modelling of design flood events.

- **Table 6-16** also indicates that peak flows for the PMF event are 58% higher along the Hastings River when compared to the previously adopted Extreme flood. As shown in **Figure F-11**, the increased flows are predicted to result in flood levels to rise by up to 2.87 metres around Wauchope and 2.30 metres at Rawdon Island.
- **Figure F-11** shows that flood levels for the PMF event are predicted to increase by between 0.75 and 1.3 metres downstream of Dennis Bridge when compared to the 2006 Extreme event.

FIGURE 7-1

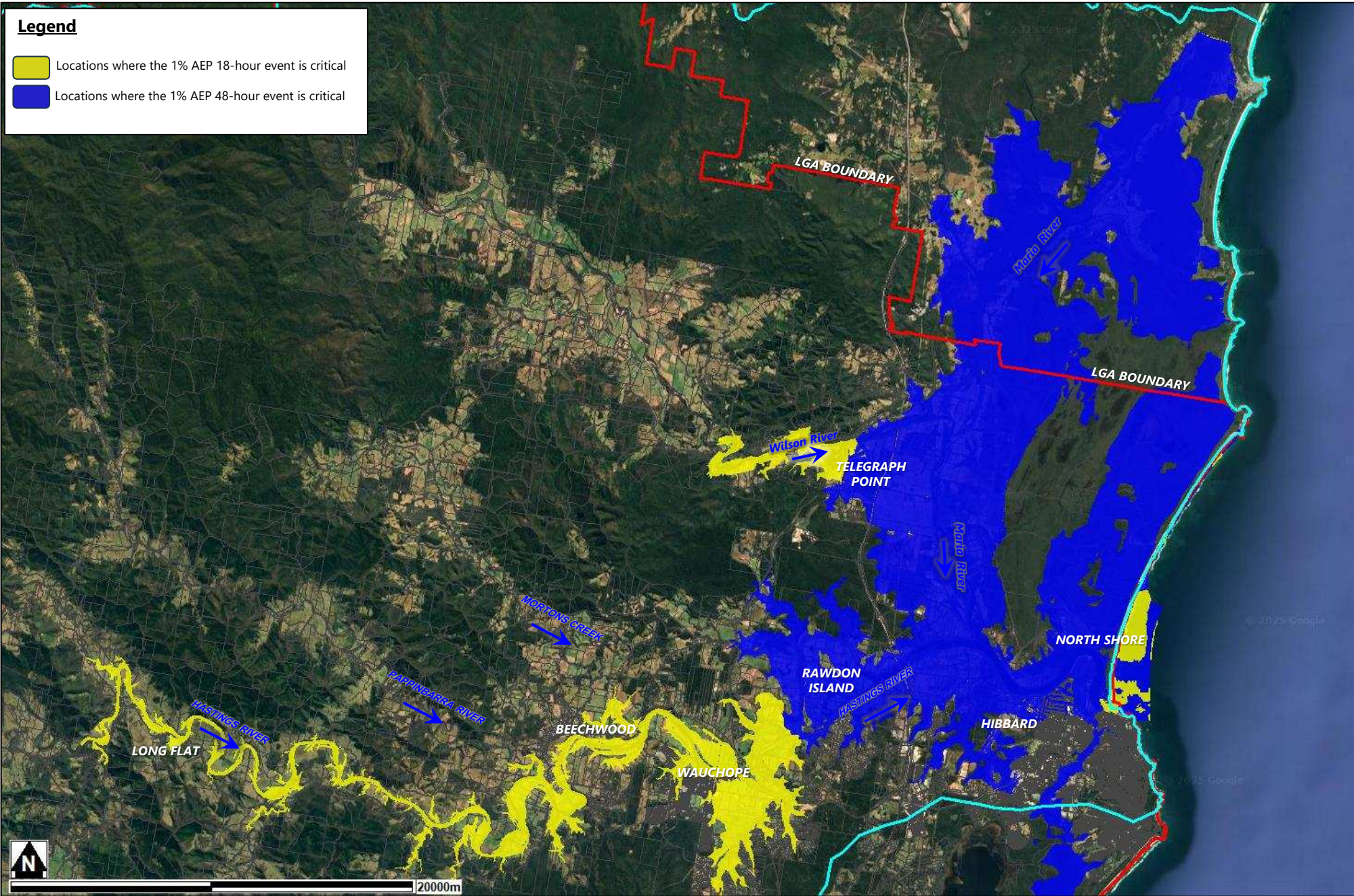


Table 7-2 Predicted Flood Levels at Key Locations throughout the RMA-2 Model Extent

Location	Point Identifier (refer App E)	Predicted Peak Flood Level (mAHD)							
		5% AEP	2% AEP	1% AEP	1% AEP with CC	1 in 200 AEP	1 in 500 AEP	1 in 2000 AEP	PMF
Locations along the Hastings River									
Ellenborough River Confluence	H1	58.68	60.51	61.88	63.04	63.23	64.93	67.15	76.06
Kindee Bridge Gauge	H2	54.06	55.73	56.9	57.83	57.97	59.15	60.93	70.54
Kindee Creek Confluence	H3	53.03	54.75	56.01	57.05	57.2	58.51	60.45	70.59
North of Longflat / Henry St	H4	49.44	51.36	52.73	53.85	54.02	55.52	57.42	65.46
Thone River Confluence	H5	36.77	38.79	40.23	41.29	41.43	42.61	44.05	52.14
Hyndmans Creek Confluence	H6	26.08	27.69	28.78	29.6	29.71	30.87	32.39	39.33
Gannons Creek Confluence	H7	21.89	23.2	24.15	24.94	25.05	26.23	27.57	33.38
Pappinbarra River Confluence	H8	17.22	18.55	19.39	20.03	20.12	21.05	22.42	30.01
Mortons Creek / Bains Bridge	H9	9.84	11.2	12.11	12.82	12.91	13.96	15.61	22.88
U/S Wauchope Railway Bridge	H10	6.46	7.69	8.54	9.16	9.22	9.98	10.95	14.88
Kings Creek Confluence	H11	5.6	6.47	7.08	7.56	7.6	8.25	9.18	13.70
U/S Dennis Bridge / Old Pacific Highway Crossing	H12	2.77	3.22	3.79	4.07	3.85	4.24	4.84	8.00
Maria River Confluence	H13	2.56	2.89	3.46	3.84	3.49	3.89	4.49	7.47
Locations along the Wilsons River									
U/S Railway Line Crossing	W1	4.46	4.89	5.21	5.46	5.48	5.81	6.28	8.25
U/S Telegraph Point Rd Crossing	W2	3.28	3.63	3.88	4.16	4.1	4.36	4.73	7.45
U/S Pacific Highway Crossing	W3	2.6	2.85	3.51	3.77	3.46	3.86	4.46	7.40
Tributaries and Floodplain									
U/S Oxley Highway Crossing of Ellenborough River	E1	59.64	61.3	62.58	63.67	63.85	65.35	67.34	76.05
Kings Creek Flood Storage	K1	5.55	6.5	7.1	7.58	7.62	8.3	9.3	14.03
Rawdon Island Flood Storage	R1	3.19	3.79	4.39	4.75	4.66	5.18	5.94	9.90
South of Hibbard Drive	HB1	2.48	2.7	3.04	3.65	3.17	3.51	4.05	6.67
Wetland east of Boundary St	HB2	2.5	2.77	3.19	3.7	3.28	3.62	4.13	6.82
Flood storage south of Port Macquarie Airport	HB3	2.41	2.77	3.32	3.78	3.38	3.78	4.37	7.12
North Shore	N1	2.45	2.65	2.95	3.6	3.08	3.39	3.9	6.37
Koolunbung Creek Storage	K1	2.39	2.54	2.75	3.27	2.85	3.07	3.41	4.52

- Changes to peak flow magnitudes are also predicted along the Wilson River. As shown in **Table 6-16**, peak flows are predicted to increase for design events rarer than a 2% AEP flood by between 5% and 33%; including for the PMF. As shown in **Figure F-1 to F-9** in **Appendix F**, the increased flows are predicted to result in higher flood levels along much of the Wilson River floodplain upstream of the Pacific Highway.
- The RMA-2 extensions along the Maria River and parts of Connection Creek to the north of the Port Macquarie-Hastings LGA boundary are also predicted to result in significant changes to the results compared to the Hibbard Flood Study. These areas to the north of the Port Macquarie LGA boundary represent a significant flood storage that can act to attenuate the rise of flood levels in the Port Macquarie LGA. This results in flood level reductions across the Maria River floodplain and in areas downstream of the Pacific Highway.

7.2 Major Tributaries TUFLOW Hydraulic Model

7.2.1 Design Flood Mapping

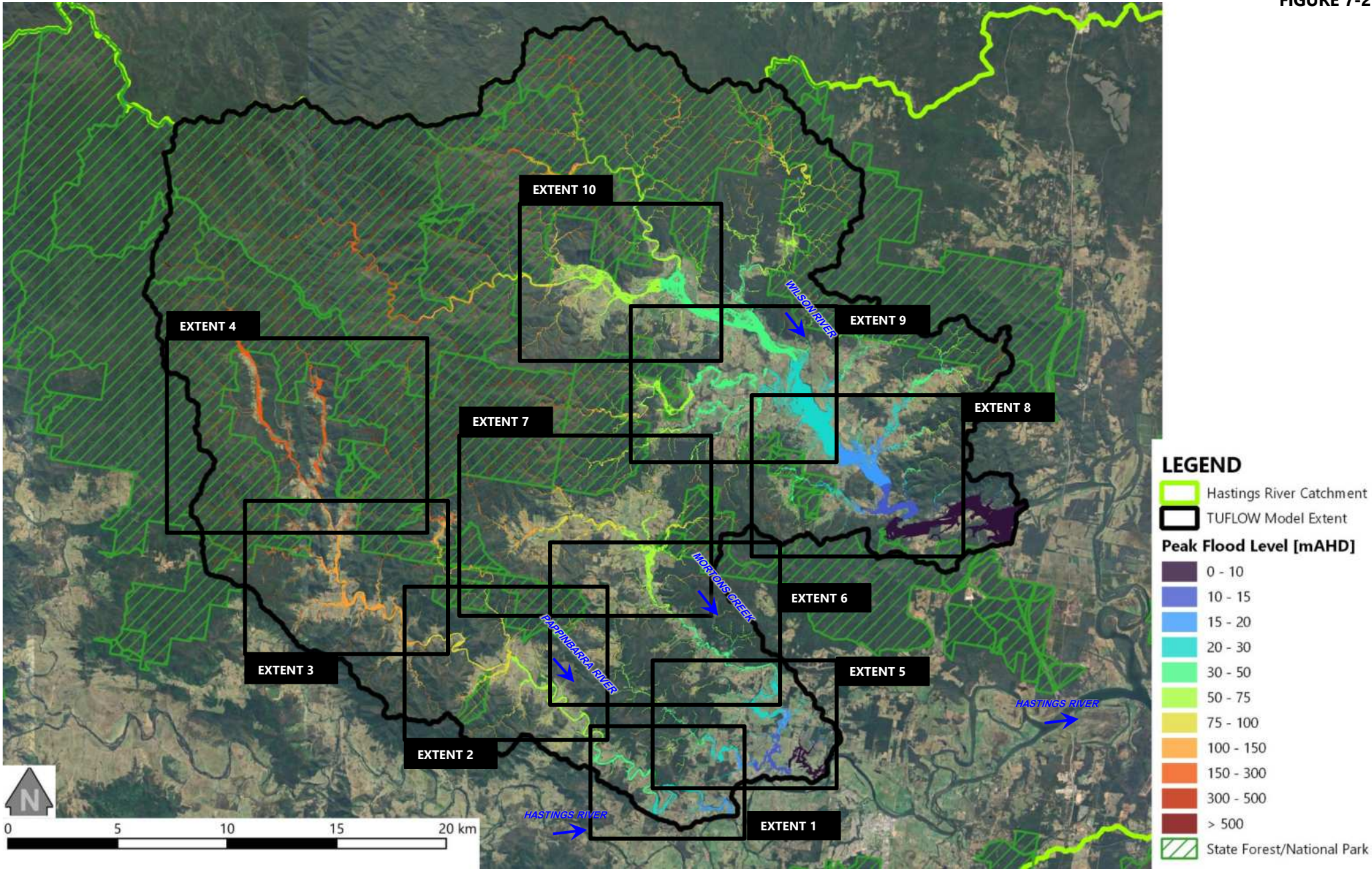
Design flood mapping produced using the TUFLOW hydraulic model is provided in **Appendix G**. This includes mapping of peak flood levels and depths and flow velocities (as velocity vectors) in accordance with the following study area breakdown shown in **Figure 7-2** and the figure sets listed below:

- Predicted Peak 5% AEP Flood Levels (refer **Figure G-1** to **Figure G-10**)
- Predicted Peak 1% AEP Flood Levels (refer **Figure G-11** to **Figure G-20**)
- Predicted Peak 1% AEP with Climate Change Flood Levels (refer **Figure G-21** to **Figure G-30**)
- Predicted Peak PMF Flood Levels (refer **Figure G-31** to **Figure G-40**)
- Predicted Peak 5% AEP Depths & Velocities (refer **Figure G-41** to **Figure G-50**)
- Predicted Peak 1% AEP Depths & Velocities (refer **Figure G-51** to **Figure G-60**)
- Predicted Peak 1% AEP with Climate Change Depths & Velocities (refer **Figure G-61** to **Figure G-70**)
- Predicted Peak PMF Depths & Velocities (refer **Figure G-71** to **Figure G-80**)

The flood mapping represents peak flood conditions produced by a process of 'flood enveloping'. For each design AEP, this process combines maximum flood level results from the adopted storm durations to produce a 'design flood envelope'. The 18 hour and 48-hour storm durations were used to produce the peak design flood envelopes.

Review of the design envelope results showed that the 18-hour storm duration was critical for approximately 95% of the TUFLOW model domain. The remaining 5% that was critical for the 48-hour duration was limited to the downstream areas of the major tributaries where flood levels were influenced by backwater flooding from the Hastings River. As discussed in **Section 6.4.2**, downstream model boundaries for these locations were assigned based on flood levels extracted from the RMA-2 modelling results for the same event and storm duration.

FIGURE 7-2



7.2.2 Peak Flood Levels

Peak flood levels for the full range of design events are listed below in **Table 7-3**. The selected locations for comparison are indicated on **Figures G-1 to G-80** included as **Appendix G**.

Table 7-3 Predicted Flood Levels at Key Locations throughout the TUFLOW Model Extent

Location	Point Identifier (refer App G)	Predicted Peak Flood Level (mAHD)					
		5% AEP	1% AEP	1% AEP with CC	1 in 500 AEP	1 in 2000 AEP	PMF
Bril Bril Ck U/S of Rollands Plains Rd Br	B1	27.92	28.59	28.74	28.95	29.16	29.98
Frazers Ck U/S of Bellangry Rd	F1	13.17	13.92	14.17	14.69	15.60	22.79
Upper Mortons Ck U/S of Bellangry Road Br	M1	72.41	72.61	72.67	72.78	72.91	73.52
Mortons Creek U/S of Slippery Ck Rd	M2	42.46	43.20	43.41	43.81	44.28	46.40
Lower Mortons Ck U/S of Mortons Ck Rd	M3	9.88	12.13	12.82	13.94	15.57	22.77
Pappinbarra R near John Wallis Rd	P1	125.56	126.16	126.38	126.68	127.12	129.45
Pappinbarra R near Hollisdale	P2	80.64	81.59	81.91	82.38	83.02	85.54
Pappinbarra R U/S of Pappinbarra Rd	P3	50.73	52.09	52.55	53.25	54.18	57.94
Pappinbarra R U/S of Pipeclay Rd Bridge	P4	19.17	20.63	21.19	22.09	23.25	30.28
Upper Rollands Plains - Wilson R U/S of Rollands Plains Rd	W1	66.67	67.07	67.18	67.34	67.49	68.17
Upper Rollands Plains - Wilson R U/S of Little Loops Rd	W2	55.06	55.89	56.16	56.62	57.24	60.42
Wilson R U/S of Clarefield Dungay Ck Rd	W3	37.00	37.55	37.71	37.96	38.30	40.37
Wilson R after confluence with Bril Bril Ck	W4	19.04	19.59	19.77	20.05	20.43	22.68
Wilson R U/S of Rollands Plains Rd (near Avenal Gauge)	W5	10.05	11.31	11.75	12.41	13.20	16.47

7.2.3 Comparison to Previous Studies

The TUFLOW hydraulic model domain covers major tributaries that have not previously been modelled as part of any flood study. A comparison to previous studies was therefore not possible.

7.3 Provisional Flood Hazard Mapping

The personal danger and physical property damage caused by a flood varies both in time and place across the floodplain. Accordingly, the variability of flood patterns across the floodplain over the full range of floods, needs to be understood by flood prone landholders and by floodplain risk managers.

Representation of the variability of flood hazard across the floodplain provides floodplain risk managers with a tool to assess the existing flood risk and to determine the suitability of land use and future development.

The hazard associated with a flood is represented by the static and dynamic energy of the flow, which is in essence, the depth and velocity of the floodwaters. Therefore, the flood hazard at a particular location within the floodplain, is a function of the velocity and depth of the floodwaters at that location.

Guideline 7.3 – Flood Hazard of ‘*Handbook 7 – Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia*’ of the Australian Disaster Resilience Handbook Collection (2017) presents a set of hazard curves which assess the vulnerability of people, vehicles and buildings to flooding based on the velocity and depth of flood flows. These curves have been adopted to define flood hazard in this study and are reproduced in **Plate 7-2**.

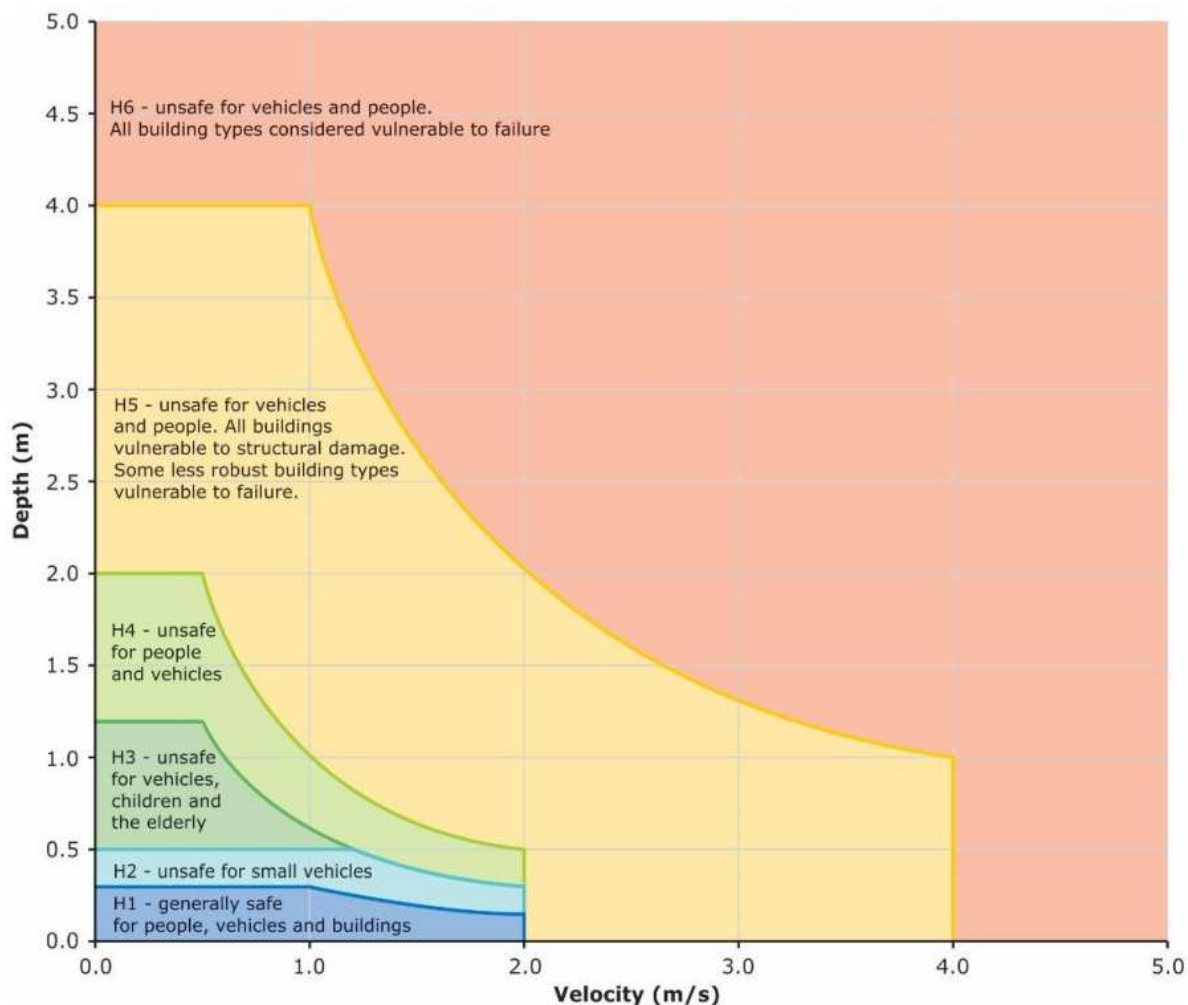


Plate 7-2 Flood Hazard Hydraulic Criteria (Handbook 7 – Managing the Floodplain 2017)

The modelling results generated using the extended RMA-2 model and the major tributaries TUFLOW model were used to prepare provisional flood hazard mapping for the study area. Provisional flood hazard mapping for the 1% AEP event is presented in **Figures H-1 to H-18** in **Appendix H** for the RMA-2 and TUFLOW model extents.

The mapping is based on a ‘peak-of-peaks’ design envelope in accordance with **Table 7-1** and the discussion in **Section 7.2.1**.

7.4 Flood Function Mapping

7.4.1 General

The hydraulic category or flood function for a site identifies the potential for development to impact on existing flood behaviour. The NSW Government's *'Floodplain Development Manual' (2005)* divides flood prone land into three hydraulic categories; namely Floodway, Flood Storage and Flood Fringe. The 2005 Manual defines the three categories as described below:

- **Floodway areas** are those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels, and even their partial blockage would cause a significant redistribution of flood flow or a significant increase in flood level.
- **Flood storage areas** are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. Loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation.
- **Flood fringe areas** are the remaining area of the floodplain after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

The latest advice for the delineation of floodway corridors is documented in *Floodplain Risk Management Guideline FB02* titled *'Flood Function'* included in the NSW Government's *Flood Risk Management Manual: the policy and manual for the management of flood liable land* (2023). FB02 replaces the FRM guideline on floodway definition (DECC 2007).

FRM Guideline FB02 outlines the following three methodologies that can be used to delineate floodway corridors.

- Indicator techniques are in most cases only suitable to provide an estimate of the floodway extent with further testing and manual assessment required.
- Encroachment techniques are generally only undertaken when using 1D models and is considered unsuitable for identifying floodways on its own.
- Conveyance techniques which rely on the identification of floodways based on a review of flow distributions.

Review of the *'Conveyance Technique'* shows that it is similar to the approach first adopted by WorleyParsons in the delineation of floodway corridors as part of the *'Hastings River Floodplain Risk Management Study'* (2012) without the additional verification undertaken through blockage analysis/modelling. A similar technique was then adopted to review the floodway corridor through Hibbard as part of the *'Hibbard Precinct Flood Study'* (2019). Similarly to the methodology adopted as part of the 2012 FRMS, the Hibbard Precinct detailed investigations relied on an assessment of existing flood modelling results, including flow distributions, and encroachment/blockage analysis.

A description of the methodology adopted as part of the *'Hibbard Precinct Flood Study'* (2019) is included in Section 8 of the report.

7.4.2 Extended Floodway Mapping

The conveyance technique was adopted to delineate floodway corridors for those new areas of the floodplain covered by the extended RMA-2 flood model and the major tributaries TUFLOW model. This involved the following steps:

1. Review of flow distributions at regular intervals along watercourses to identify width of the floodplain that conveys 80% of the total flow. Although this focused on the 1% AEP event, the 1 in 500 AEP flood was also interrogated in locations where new breakouts, or flood runners, appeared to form.

2. Velocity x Depth (VxD) and flow velocity mapping was prepared to assess flood behaviour at the edges of the identified 80% flow extents. Representative values of VxD and flow velocities were identified and used to 'map' the floodway extent between locations of flow analysis (refer point 1 above). The following ranges of VxD thresholds were identified to apply to the various watercourses:
 - Typically 15-25 m²/s along the extended length of the Hastings River, but as high as 35 m²/s in very incised sections or as low as 5 m²/s in areas where floodwaters are distributed over a wider extent.
 - Typically 10-20 m²/s along the extended length of the Wilson River.
 - Typically 5-8 m²/s along the Pappinbarra River.
 - Typically 1-5 m²/s along the other watercourses where the floodplain is not as incised and flow magnitudes are smaller. This includes Mortons, Frazers, Bril Bril and Bottle Brush Creeks.
3. Preparation of mapping of areas where VxD exceeds 10 m²/s and velocities exceed 2 m/s. This step was undertaken to identify any additional areas that are predicted to convey significant flows and velocities, but which may have fallen outside of the 80% flow corridor identified during Step 1 (refer above).
4. Extension of the floodway corridor to include areas mapped during Step 3 (refer above).
5. Manual review and editing of GIS layers against aerial photography and cadastral boundaries. This final step was undertaken to streamline the extent of the floodway and minimise minor encroachment into properties. Manual refinement was typically limited to changes to the floodway width of no more than 1 to 2 metres.

7.4.3 Extended Mapping of Flood Storage and Fringe

Flood storage and fringe was mapped based on the same criteria adopted as part of the 'Hastings River Floodplain Risk Management Study' (2012) and the 'Hibbard Precinct Flood Study' (2019). Accordingly, flood storage and flood fringe were defined as:

- Flood Storage - those parts of the floodplain outside of the floodway corridor and with depths of over 0.3 metres at the peak of the 1% AEP flood.
- Flood Fringe - those parts of the floodway outside of the floodway corridor and with depths of up to 0.3 metres at the peak of the 1% AEP flood.

7.4.4 Flood Function Mapping

Flood function mapping for areas covered by the major rivers RMA-2 model and the major tributaries TUFLOW model are presented in **Figures I-1 to I-18** in **Appendix I**.

8 References

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9 Glossary and Abbreviations

The following glossary and abbreviations have been sourced from the *Flood Risk Management Manual* (DPE 2023a).

Term	Short form	Definition	Context for use/additional information
Annual exceedance probability	AEP	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage	AEP is generally the preferred terminology. ARI is the historical way of describing a flood event, for example, a 1% AEP flood has a 1% or 1 in 100 chance of being reached or exceeded in any given year. Further information on the preferred terminology of design events of varying magnitudes is available online through the Bureau of Meteorology website: http://www.bom.gov.au/water/designRainfalls/index.shtml#faq
Australian height datum	AHD	A common national surface level datum often used as a referenced level for ground, flood, and flood levels	0.0 m AHD corresponds approximately to mean sea level
Average recurrence interval	ARI	The long-term average number of years between the occurrence of a flood equal to or larger in size than the selected event	ARI is the historical way of describing a flood event. AEP is generally the preferred terminology, for example, a 100-year ARI flood that has 1 in 100 chance of being reached or exceeded in any given year. It is equivalent to a 1% AEP flood
Catchment		The area of land draining to a specific location	It includes the catchment of the primary waterway as well as any tributary streams and flowpaths
Catchment flooding		Flooding due to prolonged or intense rainfall (e.g. severe thunderstorms, monsoonal rains in the tropics, tropical cyclones)	Types of catchment flooding include riverine, local overland and groundwater flooding
Chance		The likelihood of something happening that will have adverse or beneficial consequences	In FRM this generally relates to the adverse consequences of floods with chance being related to AEP, for example, 1% chance or 1 in 100 chance per year is equivalent to 1% AEP
Consequence		The outcomes of an event or situation affecting objectives, expressed qualitatively or quantitatively	Consequences can be adverse (e.g. death or injury to people, damage to property and disruption of the community) or beneficial
Continuing flood risk		Risk to existing and future development that may be reduced by EM measures	Flood risk to the existing development and future development may be reduced by EM measures depending on flood constraints, however, these measures cannot remove all risk and a residual risk will remain

Term	Short form	Definition	Context for use/additional information
Defined flood event	DFE	The flood event selected as a general standard for the management of flooding to development	Aims to reduce the frequency of flooding but does not remove all flood risk, for example, in selecting a 1% AEP flood as a DFE you are accepting that there is a 1 in 100 chance that a larger event will occur in any year. This risk is being built into the decision
Design flood		The flood selected as part of the FRM process that forms the basis for physical works to modify the impacts of flooding	The design flood may be considered the flood mitigation standard, for example, a levee may be designed to exclude a 2% AEP flood, which means that floods rarer than this may breach the structure and impact upon the protected area. In this case, the 2% AEP flood would not equate to the crest level of the levee, because this generally has a freeboard allowance, but it may be the level of the spillway to allow for controlled levee overtopping
Development		May be treated differently depending on the following categorisation: - infill development: the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under current land zoning - new development: development of a completely different nature to that associated with the former land-use (e.g. the urban subdivision of a previously rural area) - redevelopment: rebuilding in an area (e.g. as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale)	New developments involve rezoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power Redevelopment generally does not require either rezoning or major extensions to urban services
Development control plan	DCP	See Environmental Planning and Assessment Act 1979	
Emergency management	EM	A comprehensive approach to dealing with risks to the community arising from hazards. It is a systematic method for identifying, analysing, evaluating, and managing these risks	May include measures to reduce flood frequency or consequences through prevention and mitigation, measures, and preparation, as well as response and recovery should a flood occur (see PPRR)
Existing flood risk		The risk an existing community is exposed to as a result of its location on the floodplain	Existing flood risk may be reduced by existing or proposed FRM measures leaving a residual flood risk to the existing community. Residual flood risk may be further reduced by addressing continuing risk
Flood		A natural phenomenon that occurs when water covers land that is normally dry. It may result from coastal inundation (excluding tsunamis) or catchment flooding, or a combination of both	Flooding results from relatively high stream flow that overtops the natural or artificial banks in any part of a stream, river, estuary, lake, or dam,

Term	Short form	Definition	Context for use/additional information
			and/or local overland flowpaths associated with major drainage, and/or oceanic inundation resulting from super-elevated ocean levels
Flood (hydrologic and hydraulic) modelling		Hydrologic and hydraulic computer models to simulate catchment processes of rainfall, run-off, stream flow and distribution of flows across the floodplain or similar	They typically involve consideration of the local flood history, available collected data, and the development of models that are calibrated and validated, where possible, against historic flood events and extended to determine the full range of flood behaviour
Flood affected land		Equivalent to flood prone land	See the definition of flood prone land
Flood awareness		An appreciation of the likely effects of flooding, and a knowledge of the relevant flood warning, response and evacuation procedures facilitating prompt and effective community response to a flood threat	In communities with a low degree of flood awareness, flood warnings may be ignored or misunderstood, and residents confused about what they should do, when to evacuate, what to take with them and where to go
Flood constraints		Key constraints that flooding place on land	These include flood function, flood hazard, flood range, and flood emergency response classification. These can be used to inform FRM including consideration of options such as mitigation works, EM and land-use planning
Flood damage		The tangible (direct and indirect) and intangible costs (financial, opportunity costs, clean-up) of flooding	Tangible costs are quantified in monetary terms (e.g. damage to goods) Intangible damages are difficult to quantify in monetary terms and include the increased levels of physical, emotional and psychological health problems suffered by flood affected people that are attributed to a flood
Flood evacuation		The movement of people from a place of danger to a place of relative safety, and their eventual return	People are usually evacuated to areas outside of flood prone land with access to adequate community support Livestock may be relocated to areas outside of the influence of flooding
Flood fringe areas		That part of the flood extents for the event remaining after the flood function areas of floodway and flood storage areas have been defined	
Flood function		The flood related functions of floodways, flood storage and flood fringe within the floodplain	Flood function is equivalent to hydraulic categorisation
Flood hazard		A flood that has the potential to cause harm or conditions with the potential to result in loss of life, injury, and economic loss	The degree of hazard varies with the severity of flooding and is affected by flood behaviour (extent, depth, velocity, isolation, etc.)

Term	Short form	Definition	Context for use/additional information
Flood impact and risk assessment	FIRA	A study to assess flood behaviour, constraints, and risk, understand offsite flood impacts on property and the community resulting from the development, and flood risk to the development and its users	These studies are generally undertaken for development and are to be prepared by a suitably qualified engineer experienced in hydrological and hydraulic analysis for FRM
Flood liable land		Equivalent to flood prone land	See the definition of flood prone land
Flood plan (local or state)	Local (LFP)	A sub-plan of an EM plan that deals specifically with flooding; they can exist at state, zone, and local levels	The NSW Government develops flood plans as a legislative responsibility to determine how best to respond to floods. These community-based plans describe the risk to the community, outline agency roles and responsibilities, the agreed community emergency response strategy and how floods will be managed
Flood planning area	FPA	The area of land below the FPL	The FPA is generally developed based on the FPL for typical residential development. Different types of development may have different FPLs applied within the FPA. In addition development controls will vary across the FPA due to varying flood constraints
Flood planning level	FPL	The combination of the flood level from the DFE and freeboard selected for FRM purposes	Different FPLs may apply to different types of development Determining the FPL for typical residential development should generally start with a DFE of the 1% AEP flood plus an appropriate freeboard (typically 0.5 m). This assists in determining the FPA
Flood prone land		Land susceptible to flooding by the PMF event	Flood prone land is also known as the floodplain, flood liable land and flood affected land
Flood risk		Risk is based on the consideration of the consequences of the full range of flood behaviour on communities and their social settings, and the natural and built environment	See also risk. The degree of risk varies with circumstances across the full range of floods. It is affected by factors including flood behaviour and hazard, topography, and EM difficulties
Flood risk management	FRM	The management of flood risk to communities	
Flood risk management manual: the policy and manual for the management of flood liable land	the manual	This manual	

Term	Short form	Definition	Context for use/additional information
Flood storage areas		Areas of the floodplain that are outside floodways which generally provide for temporary storage of floodwaters during the passage of a flood and where flood behaviour is sensitive to changes that impact on temporary storage of water during a flood	See also flood function, floodways, and flood fringe areas
Flood study		A comprehensive technical investigation of flood behaviour undertaken in accordance with the principles in this manual and consistent with associated guidelines A flood study defines the nature of flood behaviour and hazard across the floodplain by providing information on the extent, level, and velocity of floodwaters, and on the distribution of flood flows considering the full range of flood events up to and including extreme events, such as the PMF	A flood study is undertaken in accordance with the FRM process outlined in this manual to support the understanding and management of flood risk. It is different from a flood impact and risk assessment (FIRA)
Flood warnings		Warnings issued when there is more certainty that flooding is expected, are more targeted and are issued for specific catchments	Flood warnings include more specific predictions of the severity of expected flooding and may give quantitative figures such as expected river water heights at gauge stations
Floodplain		Equivalent to flood prone land	See the definition of flood prone land
Floodways		Areas of the floodplain which generally convey a significant discharge of water during floods and are sensitive to changes that impact flow conveyance. They often align with naturally defined channels or form elsewhere in the floodplain	See also flood function, floodways and flood fringe areas Floodways are sometimes known as flow conveyance areas
Flow		The rate of flow of water measured in volume per unit time, for example, cubic metres per second (m ³ /s)	Flow is different from the speed or velocity of flow, which is a measure of how fast the water is moving
Freeboard		A factor of safety typically used in relation to the setting of minimum floor levels or levee crest levels	Freeboard aims to provide reasonable certainty that the risk exposure selected in deciding on a specific event for development controls or mitigation works is achieved. Freeboards for development controls and mitigation works will differ. In addition freeboards for development control may vary with the type of flooding and with the type of development
Frequency		The measure of likelihood expressed as the number of occurrences of a specified event in a given time	For example, the frequency of occurrence of a 20% AEP or 5-year ARI flood is once every 5 years on average

Term	Short form	Definition	Context for use/additional information
Future flood risk		The risk future development and its users are exposed to as a result of its location on the floodplain	Future flood risk may be reduced by existing or proposed FRM measures and land-use planning controls that consider the flood constraints on the land. This leaves a residual flood risk to the new development and its users. This residual flood risk may be further reduced by addressing continuing flood risk
Gauge height		The height of a flood level at a particular water level gauge site related to a specified datum	The datum or may not be the AHD
Hazard		A source of potential harm or conditions that may result in loss of life, injury, and economic loss due to flooding	
Hydraulics		The study of water flow in waterways and flowpaths; in particular, the evaluation of flow parameters such as water level and velocity	
Hydrology		The study of the rainfall and run-off process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods	
Likelihood		A qualitative description of probability and frequency	See also frequency and probability
Likelihood of occurrence		The likelihood that a specified event will occur	With respect to flooding, see also AEP and ARI
Local environmental plan	LEP	See Environmental Planning and Assessment Act 1979	
Local government area	LGA		The area serviced by the local government council
Local overland flooding	LOF	Inundation by local run-off on its way to a waterway, rather than overbank flow from a waterway	
Local strategic planning statement	LSPS		Local strategic planning statements assist councils to implement the priorities set out in their community strategic plan and actions in regional and district plans
Loss		Any negative consequence or adverse effect, financial or otherwise	

Term	Short form	Definition	Context for use/additional information
NSW Floodplain Management Program	the program	The NSW Government's program of technical support and financial assistance to local councils to enable them to understand and manage their flood risk	The program, manual and FRM guides support the delivery of the policy through a partnership across governments
NSW Flood prone land policy	the policy	The NSW Flood prone land policy included in this document	
Probability		A statistical measure of the expected chance of a flood	For example, AEP
Probable maximum flood	PMF	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation (PMP), and where applicable, snow melt, coupled with the worst flood-producing catchment conditions	This is equivalent to the probable maximum precipitation flood in Australian Rainfall and Runoff (ARR) The PMF in ARR is used for estimating dam design floods
Probable maximum precipitation	PMP	The greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long term climatic trends (World Meteorological Organization 1986)	PMP is the primary input to PMF estimation
Rainfall intensity		The rate at which rain falls, typically measured in millimetres per hour (mm/h)	Rainfall intensity varies throughout a storm in accordance with the temporal pattern of the storm
Residual flood risk		The risk to the existing and future community that remains with FRM, EM and land-use planning measures in place to address flood risk	FRM measures cannot remove all flood risk, but rather they reduce residual flood risk
Risk		The effect of uncertainty on objectives' (ISO 2018)	See also flood risk. Note 4 of the definition in ISO31000: 2018 also states that 'risk is usually expressed in terms of risk sources, potential events, their consequences and their likelihood'
Run-off		The amount of rainfall that ends up as streamflow, also known as rainfall excess	
State environmental planning policy	SEPP	See Environmental Planning and Assessment Act 1979	
Scenario		A scenario may relate to current, historical, or assumed future floodplain, catchment and climate conditions	Flood behaviour varies over time with changes in key catchment and floodplain (such as the scale of development) and climatic conditions (including climate change), and due to the implementation of FRM measures. A range of scenarios are generally needed to understand and assess flood behaviour

Term	Short form	Definition	Context for use/additional information
Stage		Equivalent to water level ; measured with reference to a specified datum	Measurement may relate to AHD, a local datum, or a local water level gauge
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	These factors may include the inverted barometer effect, wind and wave setup and astronomical tidal waves, together with any other factors that increase tidal water level
Velocity		The speed of floodwaters, measured in metres per second (m/s)	
Vulnerability		The degree of susceptibility and resilience of a community, its social setting, and the built environment to flooding	Vulnerability is assessed in terms of ability of the community and environment to anticipate, cope, and recover from flood events