SHOALHAVEN CITY COUNCIL

Willinga Lake Flood Study and Floodplain Risk Management Study & Plan

Volume 1 – Flood Study Report

November 2024

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PROJECT 311015-00067440: Willinga Lake FRMS&P, Volume 1: Flood Study Report

Rev	Description	Originator	Reviewer	Worley Approver	Revision Date	Customer Approver	Approval Date
Rev A	Draft	LC	CT		12/06/2024		
		L Collins	C Thomas	NA		NA	
Rev B	Revised Draft	LC	CT		23/10/2024		
		L Collins	C Thomas	NA		NA	
Rev C	Draft for	LC	CT		07/11/2024		
	Public Exhibition	L Collins	C Thomas	NA		NA	

Acknowledgement *of Country*

Walawaani (welcome),

Shoalhaven City Council recognises the First Peoples of the Shoalhaven and their ongoing connection to culture and country. We acknowledge Aboriginal people as the Traditional Owners, Custodians and Lore Keepers of the world's oldest living culture and pay respects to their Elders past, present and emerging.

Walawaani njindiwan (safe journey to you all)

This acknowledgment includes Dhurga language. We recognise and understand that there are many diverse languages spoken within the Shoalhaven.

Acknowledgements

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Shoalhaven City Council would further like to acknowledge the community of Bawley Point and Kioloa who have given their time in responding to project surveys, attending workshops and providing feedback on reports and materials.

Shoalhaven City Council has prepared this document with financial support from the Australian Government through the National Recovery and Resilience Agency's *Preparing Australian Communities Program – Local Stream*. Technical assistance has also been received from the NSW Government through the Department of Climate Change, Energy, the Environment and Water (DCCEEW) and the NSW State Emergency Service (SES). This document does not necessarily represent the opinions of the Australian Government, NSW Government or DCCEEW.

Report Structure

The reporting for the Willinga Lake Flood Study and Floodplain Risk Management Study and Plan is separated into four report volumes as follows:

- Volume 1: Flood Study Report
  - This report details the development of flood model development, calibration, design flood estimation and mapping for the study area.
- Volume 2: Floodplain Risk Management Study & Plan (FRMS&P)
  - This report details investigations into flood affectation, risk and impacts, the assessment of potential management measures, and the selection of recommended management measures to form a Floodplain Risk Management Plan.
- Volume 3: Flood Mapping Compendium.
- Volume 4: Flood Emergency Response Plan.

This document comprises **Volume 1: Flood Study Report**.

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Executive Summary

Introduction

Willinga Lake is located within the Shoalhaven City Council Local Government Area (LGA) in the South Coast Region of New South Wales. It is an Intermittently Closed and Open Lake or Lagoon (ICOLL) which drains to the Tasman Sea at the southern end of North Beach, near the village of Bawley Point.

The Willinga Lake catchment has an area of about 14 km^2 and is sparsely developed. It is predominantly zoned RU2 Rural Landscape, with significant areas of C2 Environmental Conservation, W1 Natural Waterways, and C1 National Park, and small areas of RE1 Public Recreation, R5 Large Lot Residential, and RU5 Village.

The study area also includes several smaller local catchments which drain across Bawley Point Road and Murramarang Road toward the Tasman Sea. Bawley Point Road and Murramarang Road provide the only reliable vehicular access to Bawley Point, Kioloa and Pretty Beach. Inundation of these roads has the potential to isolate the local communities including more than 1,000 properties and 11 businesses, as well as three holiday parks that contribute to a significant increase in population during holiday periods.

Shoalhaven City Council (Council) is responsible for land use planning within its LGA, including the management of flood risk. Council engaged Worley Consulting to undertake the *Willinga Lake Flood Study and Floodplain Risk Management Study and Plan* (FRMS&P). This document comprises Volume 1: Flood Study Report.

The study has been undertaken in accordance with the NSW Government's *Flood Prone Land Policy*, the primary objective of which is to reduce the impact of flooding on individual owners and occupiers of flood prone land, and to reduce private and public losses caused by flooding.

The study provides an improved understanding of the potential impacts of floods on the local community and will inform the ongoing management of flood risk in the Willinga Lake catchment. This includes the provision of detailed flood intelligence to the NSW State Emergency Service (SES) and the public regarding flood risk and how to respond to floods.

Flood Model Development and Calibration

New hydrologic (WBNM) and hydraulic (TUFLOW) flood models have been developed using the latest available data for the catchment and up-to-date guidelines, modelling software and techniques.

The models underwent calibration and verification to historic flood data for the November 2023 and February 2023 flood events to confirm their ability to reliably simulate catchment flood behaviour.

The models and their outputs will help inform the subsequent preparation of the FRMS&P for Willinga Lake including the assessment of potential floodplain risk management measures.

Design Flood Modelling and Mapping

Design flood conditions are estimated from hypothetical design rainfall events that have a particular statistical probability of occurrence. These design floods are used by Council and other agencies to understand flood risk and help plan for the occurrence of flooding.

The probability of a design event occurring can be expressed in terms of percentage Annual Exceedance Probability (AEP), which provides a measure of the relative frequency and magnitude of the flood event. The new WBNM hydrologic and TUFLOW hydraulic models were used to simulate a range of design flood events including the 50%, 20%, 10%, 5%, 2%, 1%, 1 in 200 and 1 in 500 AEP floods and the Probable Maximum Flood (PMF).

Resultant flood mapping is presented in **Volume 3**.

General Description of Flood Behaviour

Willinga Lake is bordered by a considerable area of flat, low-lying land with elevations primarily between about 1.2 and 1.8 mAHD. On the eastern bank of the lake, to the south of Bawley Point Road (*i.e., Skylark Close and surrounds*), there is a slightly higher area of land with elevations in the range of 1.8 to 2.1 mAHD. Beyond this the land rises steeply out of the floodplain. The low point of Bawley Point Road in this area has an elevation of 1.92 mAHD.

As a function of this terrain, a significant area of land is inundated even in a 50% AEP event, with the flood extent continuing to increase rapidly up to the 5% AEP event. The extent and depth of inundation in these relatively frequent floods is strongly influenced by the condition of the Willinga Lake entrance berm which typically has a crest elevation in the range of 1.6 to 1.9 mAHD when closed (*a level of 1.7 mAHD was adopted for estimation of design flood conditions*).

The steeper terrain approaching the edge of the floodplain means that in flood events rarer than the 2% AEP the incremental increase in flood extent is quite small, even in the PMF. Flood levels rise in relatively small increments with increase flood magnitude before a more significant jump in flood levels in the PMF event.

Bawley Point Road would first be inundated by floodwaters from Willinga Lake in the 5% AEP event, though it would experience minor flooding in the 10% AEP event driven by runoff from the local catchment in the vicinity of Skylark Cl.

Peak flood velocities in the Willinga Lake floodplain are low across the full range of events, typically only exceeding 1 m/s within the entrance channel, beneath the Bawley Point Road bridge and locally along the tributaries which drain into Willinga Lake.

Critical storm durations of 270-minutes to 720-minutes (*4.5 to 12 hours*) were found to drive peak flood conditions in the lake. This is typically indicative of moderate rates of rise and moderate to long durations of flooding (*e.g., Bawley Point Road is inundated for 9 hours in the 1% AEP*).

Given the considerable area of land below 2.1 mAHD, the Willinga Lake floodplain and Bawley Point Road are also susceptible to inundation driven by elevated ocean conditions such as the 5% AEP (*2.35 mAHD*) and 1% AEP (*2.55 mAHD*) oceanic inundation events.

Along the watercourses and flowpaths draining to Willinga Lake, a critical storm duration of 60 to 180-minutes was found, indicative of relatively ‘flashy’ catchments where flooding occurs in response to short durations of intense rainfall, resulting in flood levels that quickly rise and fall over the course of a few hours. Velocities are higher, with peaks in the order of 1 m/s in the 1% AEP and 2 m/s in the PMF. Willinga Road adjacent to the bridge is first inundated in a 2% AEP event, and Forster Drive is first inundated in a 10% AEP event affecting access to Willinga Park.

Bawley Point Road Local Catchments

Local catchment flooding of Bawley Point Road is also relatively 'flashy', driven by short durations of intense rainfall. Model results indicate that a location along Bawley Point Road about 300 to 400 m south-east of the Princes Highway ('BPR-1') would be inundated in events as frequent as the 50% AEP, with the hazard and duration of inundation increasing with event magnitude.

Along with Willinga Lake driven flooding of Bawley Point Road near Skylark Close, hazardous flood conditions at this location would result in the most extensive, frequent, and longest duration isolation of local communities from the Princes Highway and associated services and emergency responders.

Murramarang Road Local Catchments

There are several local catchments which drain across Murramarang Road toward the Tasman Sea via minor creeks and lagoon-like watercourses which lie behind the sand dunes of the various beaches from Cormorant Beach in the north through to Merry Beach in the south.

It was found that peak flood levels in these various lagoons, which are driven by longer duration storms, did not back up to cause flooding of Murramarang Road. Rather, inundation of Murramarang Road is generally driven by relatively 'flashy' shorter duration storms.

Inundation along Murramarang Road can be quite widespread and frequent, however flood depth, hazard and duration are generally lower than along Bawley Point Road. While this may result in the compartmented isolation of the various pockets of development and tourist facilities from one another, and from the main settlement of Bawley Point, inundation of Murramarang Road is unlikely to be critical in isolating these areas from the Princes Highway.

Potential Impacts of Sea Level Rise

Shoalhaven City Council has adopted seal level rise (SLR) benchmarks of 0.23 m by 2050 and 0.85 m by 2100. These benchmarks have been adopted to assess the potential impact of sea level rise on flooding in the study area for the 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 1 in 500 AEP, and PMF events. For ICOLLs, it is expected that the elevation of the entrance berm will increase in kind with SLR.

The findings are summarised as follows.

- In the Willinga Lake catchment:
  - In the 20% AEP and 5% AEP events flood level increases in Willinga Lake were slightly lower than the 2050 and 2100 SLR benchmarks of 0.23 m and 0.85 m respectively. These increases are associated with the equivalent increase in the initial entrance berm elevation rather than the coinciding HHWS(SS) ocean level.
  - In the 1% AEP event flood level increases in Willinga Lake were slightly higher than the 2050 and 2100 SLR benchmarks, driven primarily by the increase in coinciding ocean tailwater level (*rather than initial berm elevation*). The impact of the 2050 SLR scenario did not propagate as far upstream as the Willinga Road bridge, while in the 2100 SLR scenario there was an increase in peak flood level of 0.2 m at the bridge.
  - In the PMF event the 2050 and 2100 SLR scenarios caused flood level increases in Willinga Lake of only about 0.05 m and 0.35 m respectively. These increases are considerably less than the increase

in initial berm level and ocean tailwater, indicating that their influence is reduced in a flood event of this magnitude.

- In the Bawley Point Road local catchments road inundation would not be sensitive to the 2050 SLR benchmark, and only BPR-2 would be sensitive to the 2100 SLR benchmark.
- In the Murramarang Road local catchments road inundation would not be sensitive to the 2050 SLR benchmark, and only Butlers Creek would be sensitive to the 2100 SLR benchmark.

Potential Impacts of Climate Change Including Increased Rainfall and Sea Level Rise

The recently updated ARR 2019 Book 1 Chapter 6 presents climate change projections for five potential Shared Socioeconomic Pathways (SSPs) up to 2100 consistent with IPCC 2023. SSP3-7.0 has been selected for use in this study. It represents a high greenhouse gas emissions scenario where CO_2 emissions double by 2100. The associated rainfall increases vary with storm duration, ranging from 18% to 27% for the 2050 planning horizon and 32% to 49% for 2100.

The findings are summarised as follows.

- 2050 SSP3-7 rainfall increase and sea level rise of 0.23 m
  - In events from the 20% AEP up to and including the 2% AEP flood level increases in Willinga Lake are in the order of 0.2 to 0.3 m. These increases are about 0.05 to 0.10 m higher than under the SLR only scenario.
  - Flood level increases in Willinga Lake are about 0.3 m, 0.23 m and 0.4 m in the 1% AEP, 1 in 500 AEP and PMF events respectively. The increase is only about 0.05 m higher than under the 1% AEP SLR only scenario but is 0.35 m higher in the PMF event.
  - Along the watercourses and flowpaths draining to Willinga Lake, flood level increases were predominantly in the range of 0.02 to 0.14 m across all events and locations. A larger increase of 0.34 m was observed at the Willinga Road bridge in the PMF event.
  - At 'BPR-1' in the Bawley Point Road local catchments, flood level increases ranged from 0.02 m in the 20% AEP event to 0.07 m in the 1 in 500 AEP event. In the PMF event a larger increase of 0.26 m was observed.
  - At the Butlers Creek bridge in the Murramarang Road local catchments, flood level increases were in the order of 0.2 m in the 2% AEP to 1 in 500 AEP events. In the PMF event a larger increase of 0.45 m was observed.
- 2100 SSP3-7 rainfall increase and sea level rise of 0.85 m
  - In events from the 20% AEP up to and including the 1% AEP flood level increases in Willinga Lake are in the range of 0.8 to 0.9 m. These increases are about 0.02 to 0.15 m higher than under the SLR only scenario.
  - Flood level increases in Willinga Lake are about 0.75 m, and 0.9 m in the 1 in 500 AEP and PMF events respectively. For the PMF event this increase is almost 0.6 m higher than under the PMF SLR only scenario.
  - Along the watercourses and flowpaths draining to Willinga Lake, flood level increases were predominantly in the range of 0.05 to 0.2 m across all events and locations. At the Willinga Road bridge larger increases were observed of about 0.35 m in the 1% AEP and 1 in 500 AEP events and 0.7 m in the PMF event.

- At 'BPR-1' in the Bawley Point Road local catchments, flood level increases ranged from 0.04 m in the 20% AEP event to 0.12 m in the 1 in 500 AEP event. In the PMF event a larger increase of 0.47 m was observed.
- At the Butlers Creek bridge in the Murramarang Road local catchments, flood level increases were in the order of 0.3 m in the 2% AEP to 1 in 500 AEP events. In the PMF event a larger increase of 0.75 m was observed.

Overall, the investigated rainfall and sea level rise projections indicate that climate change would be expected to have a significant adverse impact on flooding in the study area.

Sensitivity Analysis

A series of analyses were carried out using the hydrologic and hydraulic models both to inform the selection of appropriate conditions for adoption in design flood estimation, and to understand the relative influence of selected parameters on design flood model results. This included investigation of entrance berm conditions, ocean tailwater levels, hydraulic roughness, culvert blockage, rainfall temporal pattern, alternative rainfall losses, and a comparison of ARR 1987 and ARR 2019 based results.

Recommendations

It is recommended that the new hydrologic and hydraulic models developed and calibrated as part of this Flood Study along with the associated design flood estimates and mapping be adopted for use in the Floodplain Risk Management Study & Plan.

1 Introduction

Willinga Lake is located within the Shoalhaven City Council Local Government Area (LGA) in the South Coast Region of New South Wales (*refer* **Figure 1-1**). It is an Intermittently Closed and Open Lake or Lagoon (ICOLL) which drains to the Tasman Sea at the southern end of North Beach, near the village of Bawley Point (*refer* **Figure 2-2**).

The Willinga Lake catchment has an area of about 14 km^2 and is sparsely developed. It is predominantly zoned RU2 Rural Landscape, with significant areas of C2 Environmental Conservation, W1 Natural Waterways, and C1 National Park, and small areas of RE1 Public Recreation, R5 Large Lot Residential, and RU5 Village.

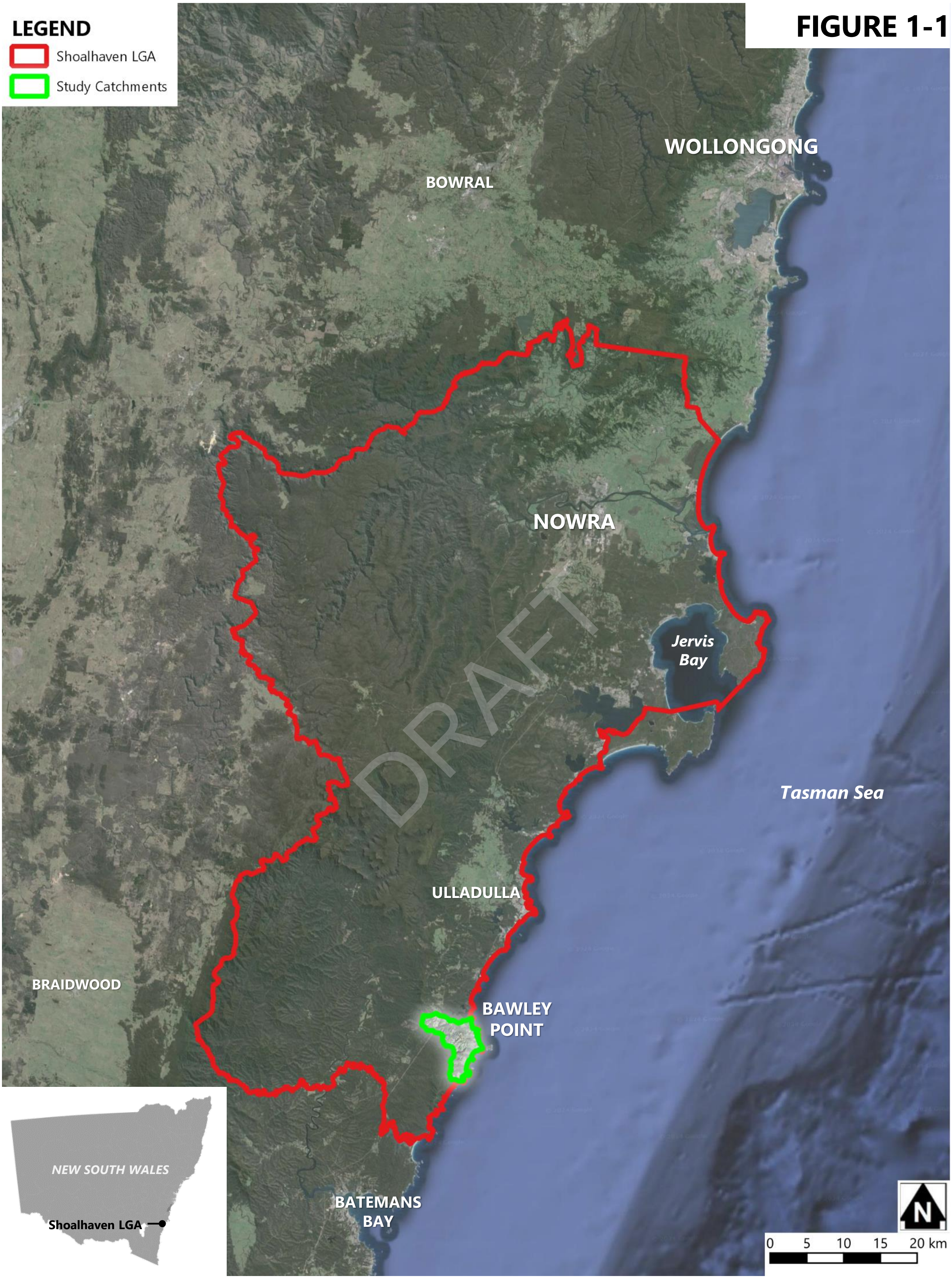
The study area also includes several smaller local catchments which drain across Bawley Point Road and Murramarang Road toward the ocean. Bawley Point Road and Murramarang Road provide the only reliable vehicular access to Bawley Point, Kioloa and Pretty Beach. Inundation of these roads has the potential to isolate the local communities including more than 1,000 properties and 11 businesses, as well as three holiday parks that contribute to a significant increase in population during holiday periods.

Shoalhaven City Council (Council) is responsible for land use planning within its LGA, including the management of flood risk. Council engaged Worley Consulting to undertake the *Willinga Lake Flood Study and Floodplain Risk Management Study and Plan* (FRMS&P).

The study has been undertaken in accordance with the NSW Government's *Flood Prone Land Policy*, the primary objective of which is to reduce the impact of flooding on individual owners and occupiers of flood prone land, and to reduce private and public losses caused by flooding.

The study provides an improved understanding of the potential impacts of floods on the local community and will inform the ongoing management of flood risk in the Willinga Lake catchment. This includes the provision of detailed flood intelligence to the NSW State Emergency Service (SES) and the public regarding flood risk and how to respond to floods.

FIGURE 1-1



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WILLINGA LAKE STUDY LOCATION

2 The Study Area

2.1 Overview

The overall study area includes the Willinga Lake catchment and several smaller local catchments which drain across Bawley Point Road and Murramarang Road toward the Tasman Sea (*refer* **Figure 2-2**). This encompasses the villages of Bawley Point and Kioloa and their access roads from the Princes Highway at Termeil in the north-west through to Pretty Beach in the south. Meroo Lake lies immediately to the north of the study area and can potentially have a backwater influence on the inundation of low points along Bawley Point Road.

The Willinga Lake catchment constitutes the primary study area, while the local catchments draining across Bawley Point Road and Murramarang Road represent secondary study areas. The primary focus in the secondary study areas is to understand the frequency and duration of inundation which may affect road safety and contribute to isolation of the local communities. The **Volume 3** Flood Mapping Compendium includes mapping for the primary Willinga Lake study area only.

The topography of the study area is shown in **Figure 2-3**, as defined by Light Detection and Ranging (LiDAR) survey captured in 2011. The highest point in the study area lies at Bundle Hill at the southern edge of the Willinga Lake catchment and has an elevation of about 218 mAHD. Low-lying land surrounds Willinga Lake, Meroo Lake and their tributaries, as well as several minor watercourses and wetlands which lie behind the sand dunes of the various beaches from Cormorant Beach in the north through to Merry Beach in the south.

2.2 Willinga Lake Catchment

The Willinga Lake catchment has an approximate area of 14 km^2, with a typical lake surface area of 0.3 km^2.

Willinga Lake is classified as an Intermittently Closed and Open Lake or Lagoon (ICOLL). The ocean entrance to the lake lies at the southern end of North Beach, adjacent to Willinga Point. The entrance is often closed off from the ocean by a sand berm which is built up by the action of waves, tides and winds. During periods of rain, water levels in the lake rise behind the closed berm until the water eventually spills over the berm, quickly scouring an entrance channel and re-opening the lake to the ocean (*refer* **Figure 2-1**). The lake then becomes tidal until the waves and tide push enough sand into the entrance channel to close it again. This process repeats naturally in a constant but irregular cycle.

There have been reports from the community of historic occurrences of Bawley Point Road being inundated by floodwater when the Willinga Lake entrance is closed. On several occasions, the entrance berm has been surveyed at an elevation above that of the Bawley Point Road low point and a rainfall event during such conditions could undoubtedly result in inundation of the road and isolation of Bawley Point and Kioloa. However, during most recent significant rainfall events the lake entrance berm has begun to open as water levels reached the edge of Bawley Point Road, thus preventing the road from overtopping. Anecdotal reports suggest that local residents or visitors may occasionally dig a small pilot channel to encourage opening of the entrance before inundation of Bawley Point Road can occur.

The Willinga Lake catchment is generally sparsely developed. Land use is predominantly rural with significant areas of Environmental Conservation, Natural Waterways, and National Park. A small area of medium density residential development lies at Bawley Point in the east of the catchment and is home to most of the residents in the catchment. This includes several low-lying properties at Skylark Close where ground elevations are predominantly between 2 and 5 mAHD. The longer established properties further to the east are generally on higher ground (10 to 20 mAHD), though elevations at a handful of properties on the western side of Shearwater Crescent range between about 2.5 and 6 mAHD.

The Willinga Park equestrian and events centre lies on the southern side of Willinga Lake near the centre of the catchment. The site has undergone considerable development since 2009 including clearing, earthworks, and the construction of internal roads, equine facilities, conference facilities, a restaurant and accommodation. Most buildings on the property appear to be located at elevations above about 9 mAHD.

Figure 2-1 Willinga Lake entrance in closed (left) and open (right) conditions

2.3 Bawley Point Road Local Catchments

There are two local catchments in the north-west of the study area which drain across Bawley Point Road into Reedy Creek and subsequently into Meroo Lake (*refer* **Figure 2-2**). Like Willinga Lake, Meroo Lake is an ICOLL. Available topographic LiDAR data sets from 2018 and 2011 both captured the entrance in a closed condition with minimum berm crest elevations of 2.6 mAHD and 2.93 mAHD, respectively.

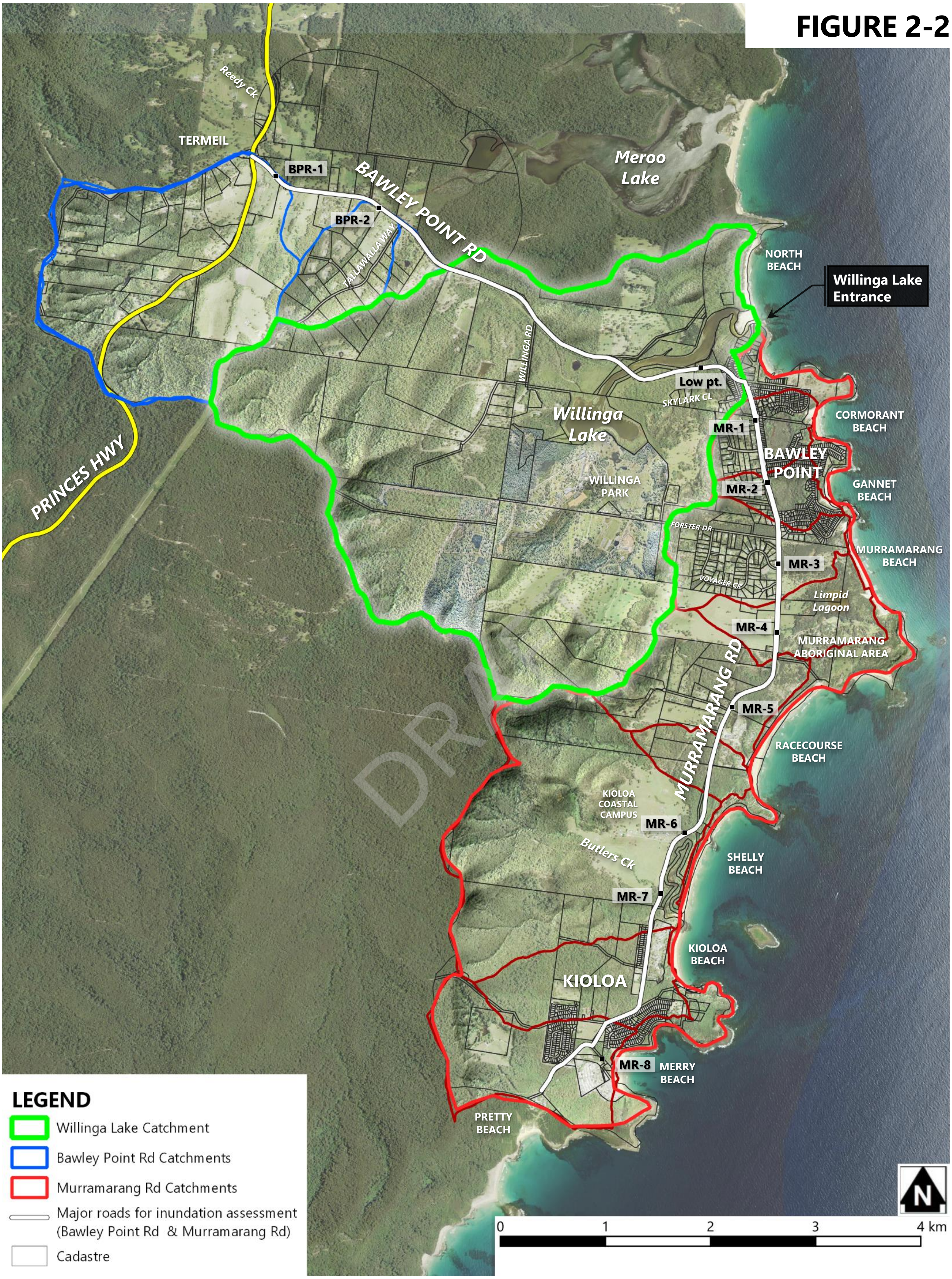
In the western-most local catchment (BPR-1), the minimum crest elevation of Bawley Point Road is about 6.32 mAHD. Anecdotal information from local residents indicates that this location has become inundated on several occasions in the past.

In the local catchment to the east near Tallawalla Way (BPR-2), the minimum crest elevation of Bawley Point Road is about 3.32 mAHD. This is only 0.4 to 0.7 m above LiDAR berm elevations for Meroo Lake and, accordingly, road inundation may be influenced by the Meroo Lake entrance condition and lake water levels which have not been investigated in detail in this study.

2.4 Murramarang Road Local Catchments

There are several local catchments which drain across Murramarang Road toward the ocean (*MR-1 to MR-8*) via minor creeks and lagoon-like watercourses which lie behind the sand dunes of the various beaches from Cormorant Beach in the north through to Merry Beach in the south (*refer* **Figure 2-2**). Named watercourses in this area are Butlers Creek and Limpid Lagoon, while the others are unnamed.

FIGURE 2-2



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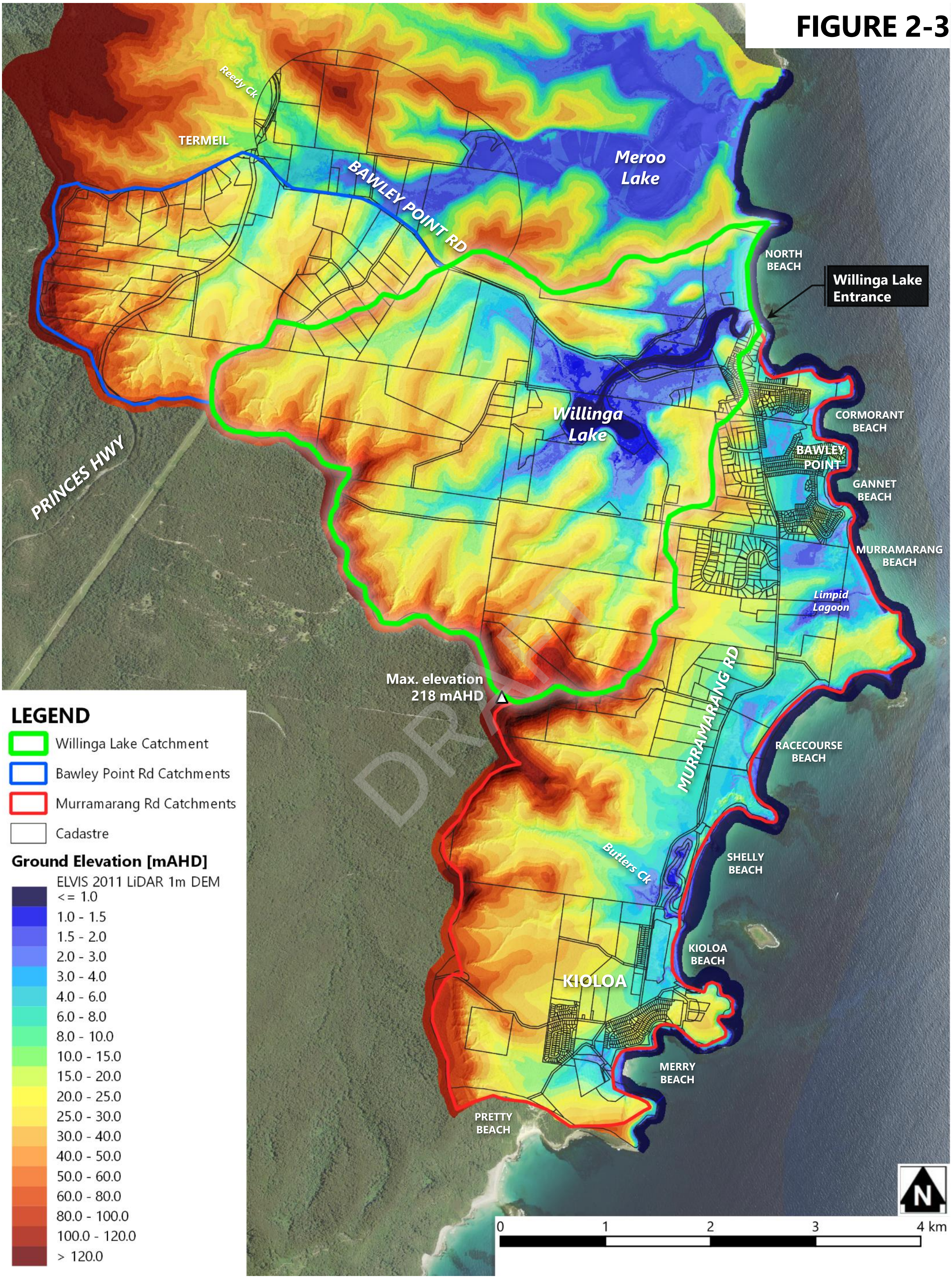
OVERVIEW OF THE STUDY AREA

FIGURE 2-3



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TOPOGRAPHY OF THE STUDY AREA

3 Background

3.1 The Need for Floodplain Risk Management

Floods are part of the Australian landscape. They occur in many parts of Australia, and their severity and causative mechanisms may vary widely between locations.

While floods have positive impacts such as providing inflows to water supplies, sustaining flood-dependent ecosystems and improving soil moistures and fertility for farming, where humans have occupied the floodplain they pose significant risk to life and property. Negative impacts of flooding include human fatalities and injuries, economic damage, environmental damage, and disruption of individuals' lives and the function of communities (AIDR 2017a).

Historically, flood damage in Australia is greater than that of any other natural hazard, and flood-related deaths are a continuing occurrence. There were reportedly 178 fatalities attributed to flooding in Australia between 2000 and 2015 (Haynes et al. 2016).

Despite the hazard posed, flooding is the most manageable natural disaster, as its behaviour and potential extent can be estimated and considered in decision making. In New South Wales, the management of flood liable land is governed by the NSW Government's Flood Prone Land Policy, the main objective of which is to reduce the impact of flooding and flood liability on owners and occupiers of flood-prone property and reduce public and private losses from flooding. The policy also recognises the benefits of the appropriate and sustainable use, occupation and development of flood-prone land.

Studies such as the Willinga Lake Flood Study and Floodplain Risk Management Study and Plan are undertaken to help local government make informed decisions about managing flood risk by using detailed flood models to quantify flood characteristics and investigating options to manage and alleviate flood risk including potential property, flood and response modification measures.

3.2 The NSW Floodplain Risk Management Process

Policy and practice are defined in the *Flood Risk Management Manual* (DCCEEW 2023a), which was gazetted in June 2023 to replace the *Floodplain Development Manual* (NSW Government 2005).

Under the Policy, the management of flood risk remains the responsibility of local government. The State Government provides financial and technical assistance to local government through its Floodplain Management Program which is administered by the NSW Department of Climate Change, Energy, the Environment and Water (DCCEEW).

The NSW Floodplain Risk Management Process consists of a number of stages as defined in the *Flood Risk Management Manual* and reproduced in **Figure 3-1**. The process is cyclical, and reviews may be triggered by various instances, for example the occurrence of significant flood events which provide additional data on flood behaviour, or the occurrence of significant changes to the catchment condition over time.

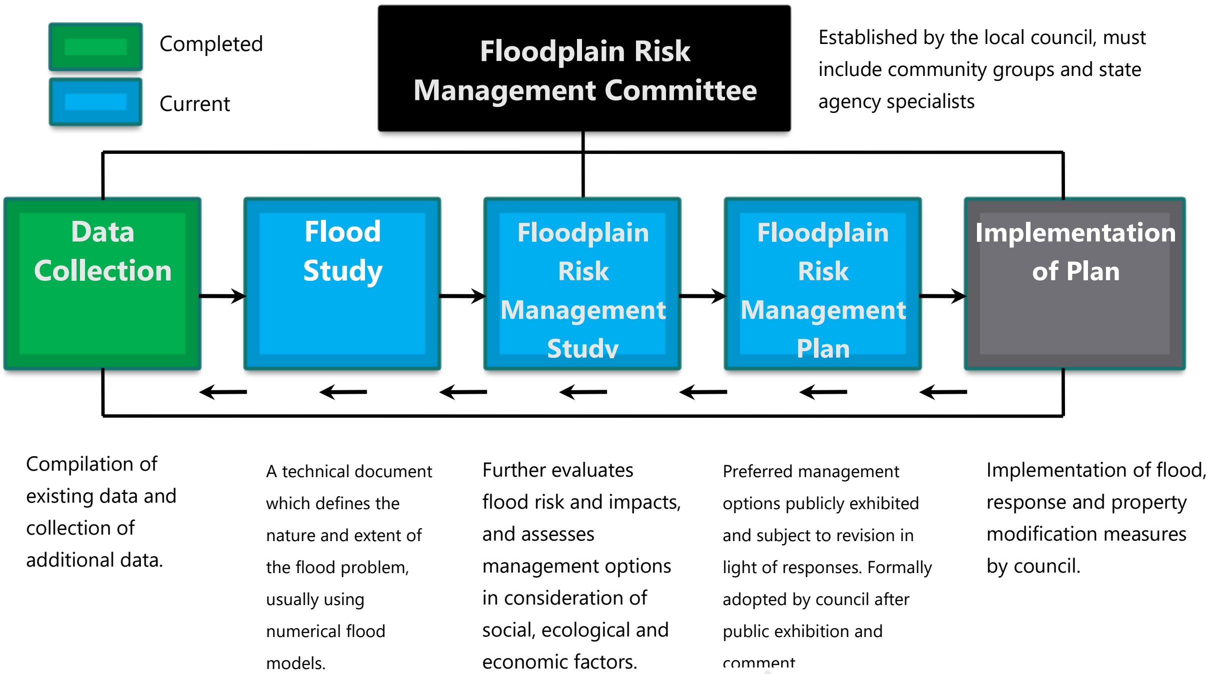


Figure 3-1 Stages of the NSW Floodplain Risk Management Process

3.3 Study Objectives

The overall objective of this study is to complete a comprehensive Flood Study and Floodplain Risk Management Study & Plan for the study area, and thereby provide an improved understanding of the potential impacts of floods on the local community and how flood risk may be better managed.

Phase 1 of the study defines flood behaviour in the study area based on the latest topographic data, modelling techniques and technology, and guidance provided in ARR 2019. This includes the provision of design flood levels, depths, discharges, velocities, hazard, hydraulic categories and other information relevant to the management of flood risk.

Phase 2 of the study assesses the potential impacts of flooding on the community and investigates options to improve management of flood risk including flood, property and response modification options.

Phase 3 of the study comprises of the development of a plan recommending implementation of the preferred measures as determined by a multi-criteria analysis including economic assessment and community feedback.

Phase 1 is detailed in this report (**Volume 1: Flood Study**) while phases 2 and 3 are detailed in the **Volume 2: Floodplain Risk Management Study & Plan** report.

3.4 Management of Coastal Lakes and Lagoons in NSW

Shoalhaven City Council is responsible for the management of several river and lake entrances in the Shoalhaven LGA in accordance with authorisations provided by the NSW Government. Due to the historical development of some of these floodplains, low-lying properties are at risk of flooding – the severity and impacts of which may be sensitive to entrance conditions. These coastal rivers and lakes are diverse, complex ecosystems and comprise highly dynamic areas which are constantly changing, whether they are open or closed to the ocean.

The Shoalhaven region has a number of coastal lakes, including Willinga Lake, that are perched behind a barrier of sand between the lake and ocean when they are closed. These sand barriers can be breached naturally by overtopping caused by heavy rainfall or heavy seas. However, prior to this water may build up behind the sand barriers which can lead to flooding in areas where historic development of low-lying private properties and infrastructure has occurred. These coastal lakes are known as Intermittently Closed and Open Lakes and Lagoons (or ICOLLs) and are common on the south coast of NSW where catchments are generally smaller in size and average rainfall is lower.

Shoalhaven City Council recognises that ICOLLs are governed by natural processes and should be maintained with respect to these unless assets are at risk of being flooded. Council and the NSW Government aim to maintain sand barriers and coastal lakes in as close to a natural condition as possible. In doing so, managing lake entrances is a balance between maintaining natural process and minimising flooding of assets including private property.

Shoalhaven City Council manages various ICOLL entrances in accordance with an Entrance Management Policy developed in partnership with NSW Government agencies and in consultation with local communities.

All river and lake entrances are located on land owned by NSW Crown Lands. Council is only permitted to artificially open entrances in accordance with the levels and conditions contained within Entrance Management Policies and any additional conditions imposed on Council in the relevant NSW Crown Lands licence. Council is required to not unnecessarily intervene with ICOLL entrances, allowing the natural processes to occur. However, when the water level within the ICOLL reaches a certain pre-determined 'trigger level' Council will artificially open the lake entrance to the ocean in accordance with the conditions of the Entrance Management Policy. Such works are undertaken following consultation with State Government Agencies.

Generally, an ICOLL entrance is closed by waves washing sand into the entrance channel and restricting tidal flow. When tidal flow is restricted over time this leads to the gradual build-up of sand in the entrance channel until the point of complete closure. A series of ocean storms can accelerate this process.

At the time of writing, there is no Entrance Management Policy in place for Willinga Lake and, as such, Council cannot legally artificially open the entrance.

3.5 Relevant Manuals and Guidelines

3.5.1 Flood Risk Management Manual, 2023

The *Flood Risk Management Manual* (DCCEEW 2023a) supersedes the Floodplain Development Manual (2005), the NSW Flood Prone Land Policy and several of the previous 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
- EM01 Support for Emergency Management Planning
- MM01 Flood Risk Management Measures.

3.5.2 Floodplain Development Manual, 2005

Prior to the release of the *Flood Risk Management Manual* (DCCEEW 2023a), the *Floodplain Development Manual* 2005 (the Manual) incorporated the *NSW Flood Prone Land Policy* and guided its implementation in the floodplain risk management process. It aimed 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 outlined 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.

3.5.3 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 2017a) 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.

3.5.4 Australian Rainfall and Runoff, 1987

Australian Rainfall and Runoff: A Guide to Flood Estimation 1987 (*ARR 1987*) provided a national guideline document and data used for the estimation of design flood characteristics in Australia. It was widely used in flood studies throughout Australia up until the release of ARR 2019.

ARR 1987 has been used in the flood study to provide a comparison to design flood conditions determined using the ARR 2019 guidelines.

3.5.5 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.

3.5.6 ARR 2019 Updated Climate Change Considerations Chapter, 2024

On 27 August 2024 an update of the Climate Change Considerations chapter of ARR 2019 (Book 1 Chapter 6) was published by DCCEEW and Engineers Australia.

The chapter provides up to date guidance aligned with the latest relevant science such as IPCC 2023 and Wasko et al. (2023). This includes new estimates of increased rainfall intensity under a range of climate change scenarios which are typically higher than those presented in the original chapter.

Climate change scenario modelling presented in this flood study was undertaken prior to the release of the final chapter and is based on the draft version which was published in December 2023. The final chapter incorporated marginally lower global mean surface temperature projections than the draft, resulting in estimated rainfall increases that are about 10% lower. The impact on Willinga Lake peak flood levels will be very minor due to the major influence of the berm level and/or tailwater but would be greater along other local watercourses and tributaries.

3.5.7 OEH Floodplain Risk Management Guidelines

A series of floodplain risk management guidelines were developed by the former OEH (*now DCCEEW*) 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.

A number of these guidelines document remain relevant and have been considered in the preparation of this study including:

- Modelling the interaction of catchment flooding and oceanic inundation in coastal waterways
- Floodway definition
- Practical consideration of climate change
- Flood emergency response classification of communities.

4 Data Collection and Review

4.1 Light Detection and Ranging (LiDAR) Topographic Data

Light Detection and Ranging (LiDAR) is an airborne remote sensing technology which uses the pulse from a laser to collect measurements of the ground surface. Topographic Digital Elevation Models (DEMs) as derived from LiDAR survey of the study area were sourced from the Geoscience Australia ELVIS data portal. The following datasets cover the study area:

- 'BatemansBay201106-LID1-AHD' and 'Ulladulla201105-LID1-AHD' 1 metre DEMs derived from LiDAR captured between May and June 2011 with a reported accuracy of 0.3 m in the vertical and 0.8 m in the horizontal; and,
- 'LkTburC2018-C3-AHD' classified point clouds captured in 2018 as part of the NSW Marine LiDAR project with reported accuracy of 0.15 m in the vertical and 0.17 m in the horizontal. Elevation points classified as 'ground' were triangulated to create a 1 metre DEM which covers areas within about 1 to 2 kilometres of the coast.

It is noted that the LiDAR data may be of lower resolution and accuracy in areas of dense vegetation and does not penetrate water surfaces.

The topography of the study area as captured by the 2011 LiDAR DEM is presented in **Figure 2-3**. The 2018 LiDAR DEM is shown in **Figure 4-1**.

It was found that the 2018 LiDAR often exhibits a better representation of watercourses and lagoon areas, with typically lower minimum ground levels and improved continuity (*i.e., less triangulation*) along channels (*e.g., refer inset in* **Figure 4-1**). This may be due to a combination of factors including the use of more recent LiDAR and filtering technology, and the presence of less surface water at the time of the survey. The 2018 LiDAR also offers a more recent representation of the terrain for part of the Willinga Park development.

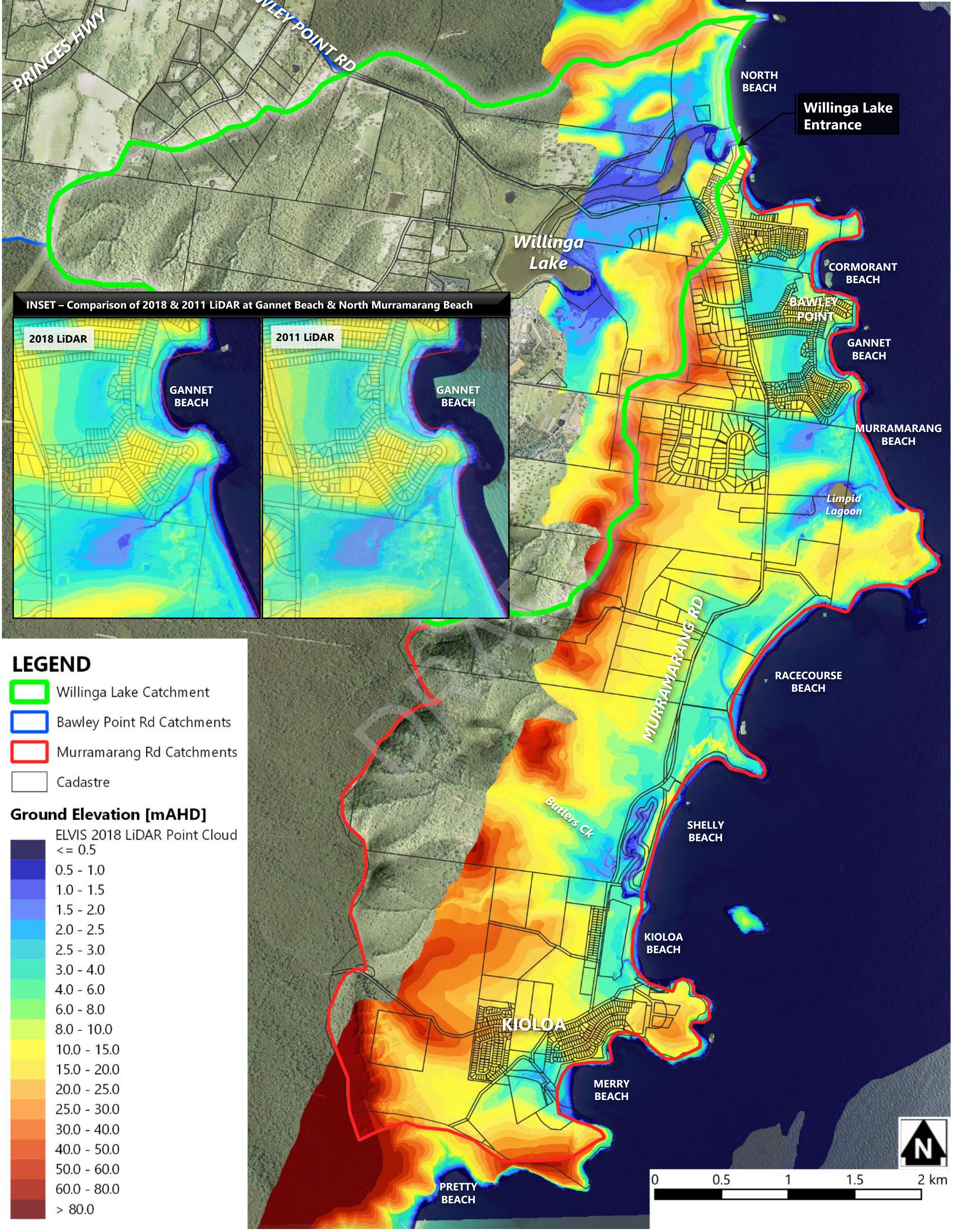
The 2011 LiDAR DEM will be adopted as the primary topographic data set for use in the study including for flood modelling purposes. This will be supplemented by the 2018 LiDAR where it is considered to provide a better topographic representation. This may include along watercourses, within Willinga Park, and at selected hydraulic features or roads.

4.2 Ground and Structural Survey Data

Ground and structural survey data, design plans and measurements provided by Council for use in the study include the following:

- Design Plans for Bawley Point Road Bridge.
- Detailed survey of Bawley Point Road to the east of the bridge.
- GIS and CAD files with limited details for 5 hydraulic structures along Murramarang Road.
- Site inspection notes, photographs and measurements for 7 minor hydraulic structures.
- Survey of 9 minor culverts in the vicinity of Skylark Close
- Detailed survey of 25 culverts and bridges identified during the data review phase of the project.

FIGURE 4-1



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2018 LIDAR DEM

4.3 Bathymetric Survey

Bathymetric survey of Willinga Lake was captured by the Office of Environment and Heritage (*OEH, now DCCEEW*) in August 2002. The extent of the survey and bed elevations are indicated in **Figure 4-2**.

Bed elevations in the lake and channel were predominantly in the range of about 0.0 to 0.6 mAHD, with deeper elevations of -0.5 to -1.0 mAHD approaching the ocean entrance. The entrance was in a closed condition at the time of the survey with a minimum berm crest elevation of 1.83 mAHD.

Greater depths in the -0.6 to -1.6 mAHD range were surveyed at the Bawley Point Road bridge. This would be associated with scour driven by a significant constriction in the channel width at the bridge which causes higher flow velocities, as well as localised turbulence around the bridge piers.

While bed elevations would vary naturally over time, the 2002 bathymetric survey is considered representative of typical conditions for the purposes of this study.

Several surveys of the Willinga Lake entrance were also undertaken by Council between August 2022 and January 2024 and are discussed in **Chapter 6**.

Figure 4-2 Bathymetric Survey Data (OEH, 2002)

4.4 Hydrometric and Historic Flood Data

In preparation for this study, and to support flood response by the SES, Council installed a new water level gauge and pluviometer (*i.e., a gauge that records continuous sub-daily rainfall*) at Willinga Lake at the Bawley Point Road Bridge. The gauges were installed in December 2022; however, the water level gauge experienced some datum and telemetry issues until about July 2023. The datum was able to be corrected using survey data for the February 2023 event which enabled its use in model verification.

A thorough search of available databases was undertaken to identify additional hydrometric data stations within or near the study area. This involved conducting searches with the Bureau of Meteorology (BoM), WaterNSW and Manly Hydraulics Laboratory (MHL). Relevant stations are mapped in **Figure 4-3** and are discussed in the following sections.

4.4.1 Stream Level Data

Relevant available stream level records were obtained, comprising data from the following stations:

- 569045 Willinga Lake (Bawley Point) – Shoalhaven City Council, December 2022 to present.
- 216008 Butlers Creek at Kioloa – WaterNSW, 2012 to 2014.

4.4.2 Rainfall Data

Rainfall data was sought from stations within and surrounding the study catchments. A particular focus was placed on obtaining data from pluviometers to resolve the temporal pattern of rainfall across the catchment during past flood events.

Rainfall data was obtained for the following rainfall stations:

- 569045 Willinga Lake (Bawley Point) – Shoalhaven City Council, December 2022 to present
- 069040 Kioloa Old Post Office – BoM daily read, 1957 to 2021
- 216008 Butlers Creek at Kioloa – WaterNSW, 1981 to 1988
- 569036 Lake Tabourie – Shoalhaven City Council, current, 6.5 km north of study area
- 569037 Burrill Lake – Shoalhaven City Council, January 2006 to present, 12.5 km north of study area
- 569024 Ulladulla – Shoalhaven City Council, April 2001 to present, 17 km north of study area
- 069121 Brooman (Carisbrook) – BoM daily read, 1979 to present, 11 km north-west of study area
- 216002 Clyde River at Brooman – WaterNSW, 2015 to present, 10 km west of study area.

4.4.3 Ocean Level Data

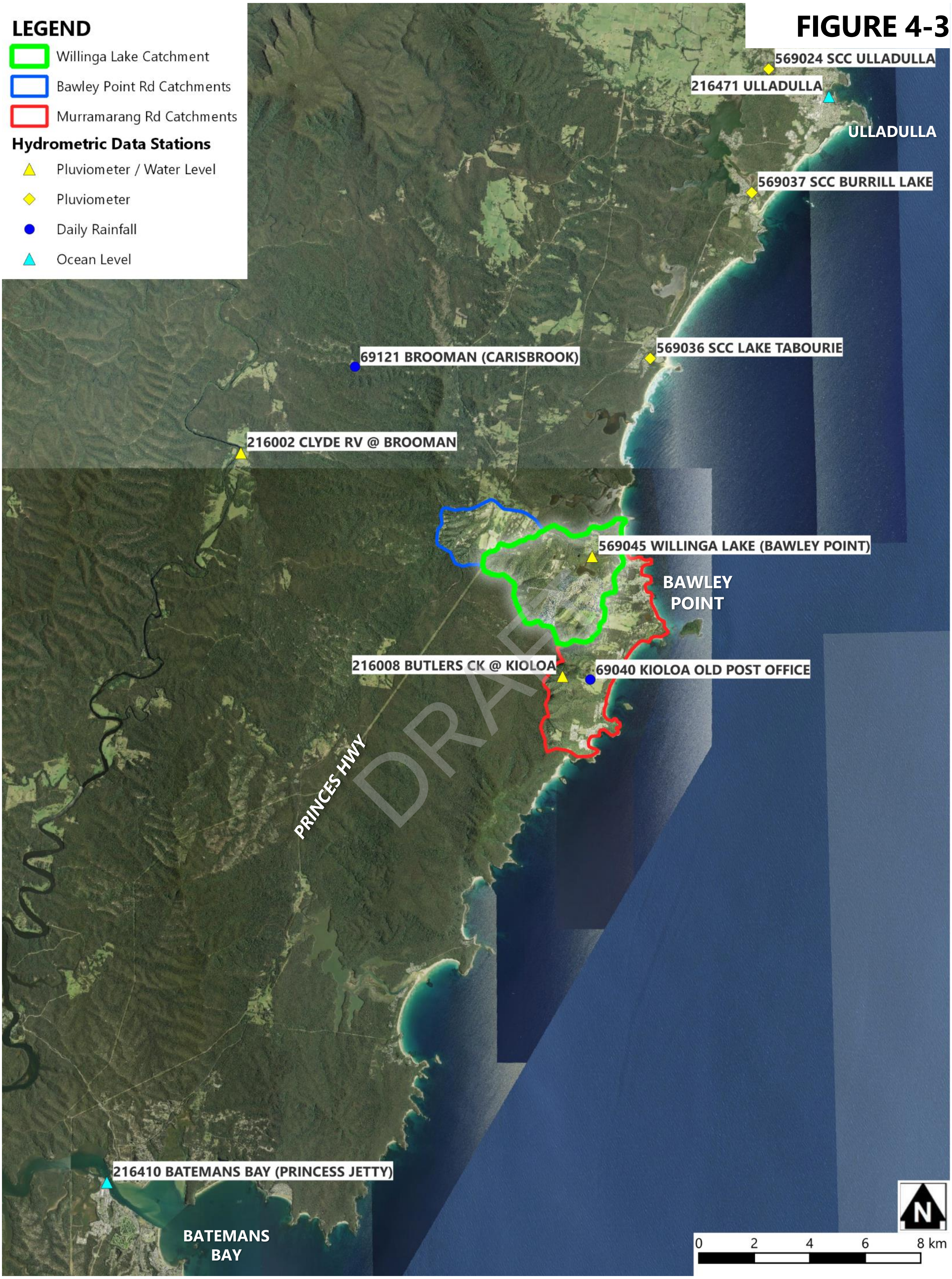
Data from the following ocean level stations was obtained for use in the study:

- 216471 Ulladulla – Manly Hydraulics Laboratory (MHL), 2008 to present, 17 km north of study area.
- 216410 Batemans Bay – MHL, 1970 to present, 22 km south of study area.

4.5 Review of Willinga Lake Entrance Condition

A review of available data relating to the historic condition of the Willinga Lake entrance is presented in **Chapter 6**. This includes review of aerial photography, LiDAR topography, detailed survey data and anecdotal information.

FIGURE 4-3



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HYDROMETRIC DATA STATIONS

4.6 Review of Local Catchment Beach Berm and Road Levels

One of the aims of this study is to investigate the frequency and duration of inundation of Bawley Point Road and Murramarang Road due to local catchment flooding.

The Bawley Point Road local catchment watercourses drain to Meroo Lake. Accordingly, inundation of Bawley Point Road may be influenced by the Meroo Lake entrance condition and berm elevation.

The Murramarang Road local catchment watercourses drain across the beach into the Tasman Sea. The elevation of beach berms at the downstream end of these watercourses thus affects water levels within them and may influence inundation of Murramarang Road.

LiDAR elevation data has been interrogated to identify hydraulically controlling berm elevations for each local catchment watercourse and extract relevant road elevations to determine how sensitive road inundation may be to the beach berm condition. This information is compiled in **Table 4-1**. The key findings are summarised as follows:

- Differences in beach berm elevations between the 2011 and 2018 LiDAR data sets are relatively small ranging from just 0.03 m at Limpid Lagoon to a maximum of 0.35 m at Cormorant Beach.
- There are only two locations where road elevations are low enough that berm conditions are considered likely to have any significant influence on road inundation – these are an internal bridge within the Tasman Holiday Park Racecourse Beach, and a low point along Bawley Point Road 170 metres to the west of Tallawalla Way.

Similarly to an ICOLL, when water levels in the local watercourses rise in response to heavy rainfall the beach berms would overtop, and a discharge channel would begin to scour across the beach into the ocean. No topographic data is available indicating the geometry of such discharge channels for any of the local catchments.

Aerial photography from December 2021 shows the various local watercourses containing considerable standing water despite the presence of a scoured beach channel through which water can still be seen discharging (*refer* **Figure 4-4**). This suggests that, even when scoured, discharge across the beach to the ocean remains constrained and does not lead to the rapid and complete drainage of the watercourses.

Figure 4-4 December 2021 aerial photos of local watercourses (Source: Google Earth)

Table 4-1 Comparison of local catchment beach berm and road elevations from LiDAR

Watercourse / Beach	Controlling Beach Berm RL (mAHD)		Road Low Points		
	2011 LiDAR	2018 LiDAR	Road	Approximate Location	Crest elevation at low point (mAHD)
Meroo Lake	2.93	2.60	Princes Highway	80m north of Old Princes Hwy	12.62
				50m south of Bawley Point Rd	10.81
			Bawley Point Road	400m south-east of Princes Hwy	6.32
				600m east of Princes Hwy	7.99
				600m west of Tallawalla Way	7.85
				170m west of Tallawalla Way	3.32
				90m south-east of Tallawalla Way	8.21
Cormorant Beach	2.21	2.56	Murramarang Road	40m south of Binnowee Place	8.67
				Near Weemala Cres	5.41
			Tingira Dr	Near Kywong Ave	4.48
			Lurnea Ave	150m east of Murramarang Rd	4.21
Gannet Beach	2.13	1.95	Murramarang Road	Near Wonnawong place	10.98
				90m north of Malibu Dr	9.16
			Rosemary Avenue	70m west of Mailbu Dr	5.35
			Malibu Drive	Malibu Dr culverts, 50m north of Northaven Ave	4.35
			Northaven Ave	In front of 25 Northaven Ave	4.46
Murramarang Beach North	2.06	1.94	Murramarang Road	60m south of Forster Dr	4.20
				90m north of Voyager Cres	3.89
			Malibu Drive	65m south-east of Sunseeker Dr	3.34
			Forster Drive	Near Murramarang Rd	4.72
			Voyager Cres	Near Murramarang Rd	3.93
Limpid Lagoon (Murramarang Beach South)	2.11	2.08	Murramarang Road	Upstream of Limpid Lagoon	6.01
Racecourse Beach North	1.92	2.18	Murramarang Road	160m north of Bundle Hill Rd	5.08
				Near Bundle Hill Rd	6.59
			Tasman Holiday Park	Internal bridge	2.40
Racecourse Beach South	2.03	2.19	Murramarang Road	Adjacent to Tasman Holiday Park	6.09
Butlers Creek	1.53	1.79	Murramarang Road	Northern tributary, 130m north of Moores Rd	3.94
				Northern tributary, 120m south of Moores Rd	3.15
				Butlers Creek Bridge	3.05
Kioloa Beach	1.67	2.00	Murramarang Road	115m south of O'Hara Street	6.46
				90m north of Scerri Dr	6.99
Merry Beach	1.86	1.59	Merry Beach Road	Northern tributary culverts	2.59
			Merry Beach Road	Bridge to Ingenia Holidays	3.00
			Pretty Beach Road	160m north of Pretty Beach Campground	9.21

Road inundation considered potentially sensitive to berm level

4.7 Anecdotal Flood Information from the Community

The community consultation process has provided an opportunity for the community and other stakeholders to collaborate in the development of the FRMS&P. As well as informing the community about the study and increasing flood awareness, engagement throughout the process has provided an opportunity to garner information on past flooding, community attitudes and concerns regarding flooding, flood preparedness, and views on potential flood risk management options.

The initial phases involved the distribution of a newsletter and questionnaire to the community, and a community 'drop-in' session. Anecdotal information on flooding received from the community identified Bawley Point Road near the Princes Highway ('BPR-1') as the primary concern with regard to road inundation in the study area (*e.g. refer* **Figure 7-1**), while there were differing views as to whether Willinga Lake driven inundation of Bawley Point Road near Skylark Close has occurred in recent decades.

Additional details of community consultation activities and findings are presented in the **Volume 2** report.

5 Flood Model Development

5.1 Modelling Approach

Numerical computer models have been adopted as the primary means of investigating flood behaviour for the study area. When used carefully, modern computer models allow simulation of flood behaviour over large areas in a cost efficient and reliable manner.

For this study, the traditional approach of coupling separate hydrologic and hydraulic models has been selected. The hydrologic model simulates the catchment rainfall-runoff processes, with resulting flow hydrographs input to the hydraulic model. The hydraulic model simulates the physical behaviour of the flow as it passes through the catchment, producing information on flood levels, flood extents and flow velocities.

The WBNM hydrologic and TUFLOW 2D/1D hydraulic modelling software packages have been adopted.

5.2 Hydrologic Model Development

5.2.1 Overview

The purpose of the WBNM hydrologic model is as follows.

- Assess critical design storm duration(s) and ensemble modelling, including assessing the 'average' ARR 2019 temporal pattern(s); and,
- Simulate catchment rainfall-runoff processes and generate local runoff hydrographs for input to the TUFLOW hydraulic model.

The development of the WBNM model is described in the following.

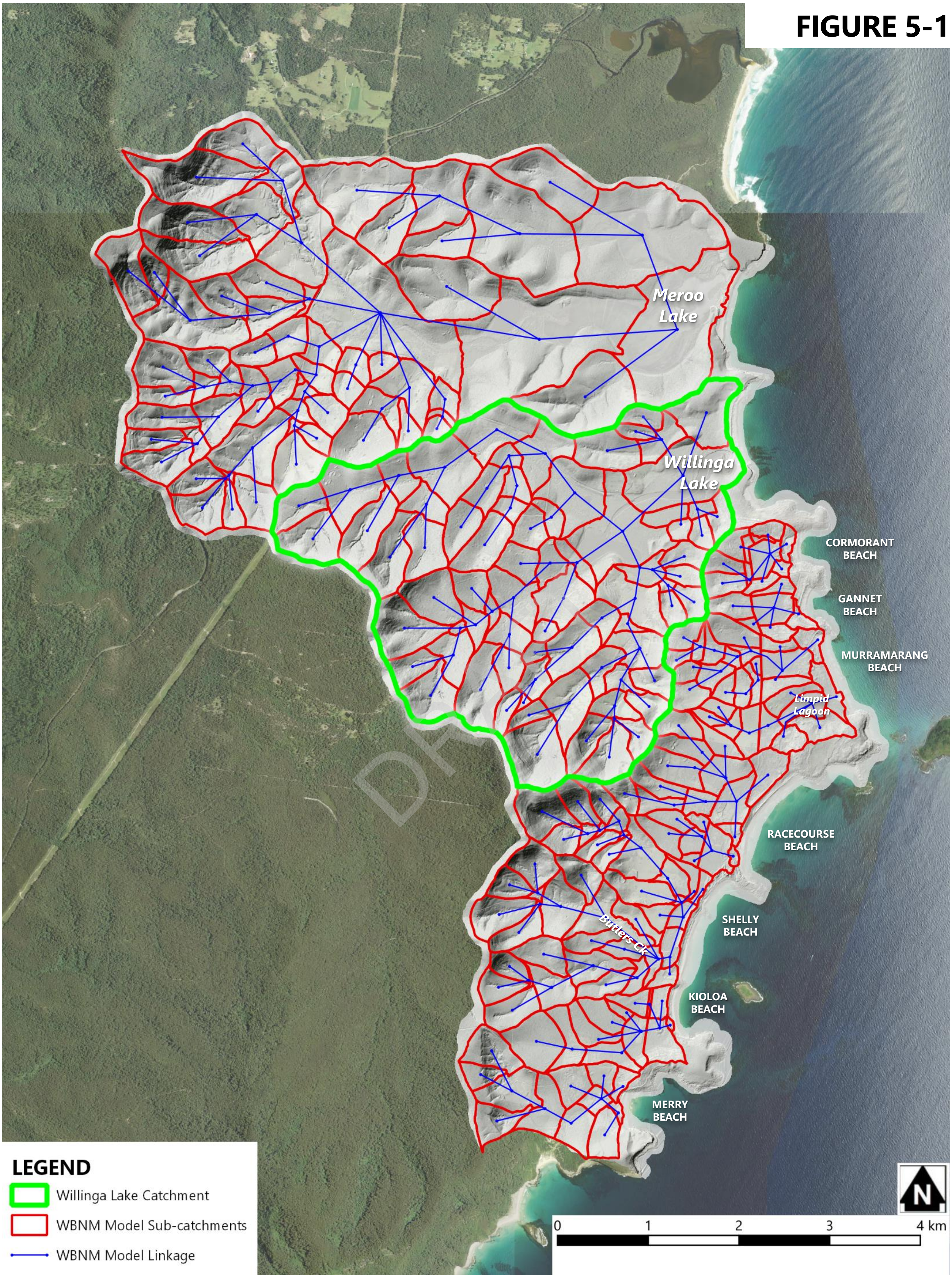
5.2.2 Model Layout

The sub-catchment delineation and linkage form the foundation of the WBNM hydrologic model structure and are presented in **Figure 5-1**.

The sub-catchment delineation process used to develop the model layout is described as follows.

- The extent of the Willinga Lake catchment was determined from the 2011 LiDAR 1m DEM using the CatchmentSIM hydrologic and terrain analysis software, along with that of the local catchments and Meroo Lake.
- These were further delineated into 273 sub-catchments based on consideration of the catchment topography, flood flow paths (*as identified through initial direct rainfall hydraulic modelling*), and the location of roads, culverts and bridges. Some manual refinement of sub-catchment boundaries was undertaken in QGIS to improve consistency in sub-catchment shape and alignment with relevant topographic, cadastral and hydraulic features.
- A relatively fine sub-catchment resolution was adopted within the study area that was tailored to achieve appropriate representation of all flow paths potentially posing flood risk to the community, property, or infrastructure. A coarser delineation was adopted in the Meroo Lake catchment.
- The resulting sub-catchment areas are in the order of 1 to 10 ha in developed areas, and 10 to 70 ha in rural and undeveloped areas.

FIGURE 5-1



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WBNM HYDROLOGIC MODEL LAYOUT

5.2.3 Runoff Lag and Stream Routing Parameters

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

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

The stream routing lag factor 'F' determines the time it takes for flows to travel along streams. WBNM documentation recommends a default stream lag factor of 1.0 to represent natural streams. Lower values can be adopted if the stream has undergone modifications such as clearing, straightening, or concrete lining.

Recommended default parameters were initially adopted and were confirmed to be appropriate through the model calibration and verification process. The final values are presented in **Table 5-1**.

Table 5-1 Adopted WBNM runoff lag and stream routing parameters

WBNM Model Parameter	Parameter Value
Runoff lag factor 'C'	1.6
Impervious runoff lag factor 'C'	0.1
Stream routing factor 'F'	Variable (0.8 to 1.0)

5.2.4 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 of the WBNM model was determined through analysis of the detailed surface material delineation developed for use in the TUFLOW hydraulic model (*refer* **Figure 5-3**). Each surface type was converted to a percentage imperviousness, and then a specific area-averaged imperviousness assigned to each model sub-catchment. The effective percentage impervious assigned to each surface material type is presented in **Table 5-2**.

Table 5-2 Effective percentage impervious by land surface type

Surface Type	Effective Percentage Impervious
Watercourses and concrete open channels	100%
Buildings	100%
Residential / Commercial (excl. buildings)	40%
Vegetation	0%
Road Corridors (incl. road reserve)	70%

Impervious percentages presented in Table 1 of Shoalhaven DCP 2014 Chapter G2 were considered for adoption in this study. However, the presented imperviousness values are intended for use in stormwater management in urbanised areas and were found to be somewhat high for use in calibration to recorded flood events in the largely rural and sparsely developed Willinga Lake catchment.

5.2.5 Hydrologic Model Structures and Storages

5.2.5.1 Relevance of Hydrologic Model Stream Routing

The definition of flood levels and flow velocities which are the primary objective of this study have been determined by applying 'local runoff' hydrographs from each sub-catchment of the WBNM model to the TUFLOW hydraulic model which then computes its own, far more complex stream routing processes.

Stream routing within the hydrologic model is not, therefore, explicitly required to define flood behaviour with the hydraulic model, but is of importance to the study for other purposes as follows:

- The proper representation of stream routing in the hydrologic model allows cross-comparisons to be made between flow hydrographs from the WBNM and TUFLOW models (*refer* **Section 7.5**). Agreement in the hydrograph shape and peak flows shows that the differing principals and numerical approaches behind each model are converging on a common result, providing additional confidence in the modelling process. These comparisons also provide a form of quality check to confirm that water volume is being properly applied and conserved within the TUFLOW model.
- The hydrologic model is used to assess 'critical storm duration' at key locations throughout the catchment. It is therefore important that the total flows (i.e. routed flows) calculated by the hydrologic model be reliable at these locations.

Stream routing behaviour in the WBNM hydrologic model can be improved through the addition of 'structures' to represent the attenuation and/or diversion of flows.

5.2.5.2 WBNM Structures and Storages

The movement of flood flows through the catchment can be influenced by the presence of structures or natural floodplain features that detain flows (*i.e. temporarily store water and release it at a slower rate*). Accordingly, key flood storages and structures affecting flow behaviour in the catchment were identified and added to the WBNM hydrologic model.

Sixteen 'structures' were added to the WBNM hydrologic model to represent their effect on downstream flood flow hydrographs. The primary purpose of including these structures is to improve the WBMN hydrologic model's ability to reliably represent the routing of flows through the catchment and subsequently ensure that the WBNM model is appropriate for use in the assessment of 'critical storm duration' at key locations throughout the catchment.

The added structures were either known to have a significant influence on the passage of flood flows through the catchment (*e.g. Willinga Lake and other lagoon-like watercourses*) or were identified as having a significant impact on flows through comparisons made between WBNM and TUFLOW simulated hydrographs (*e.g. several locations where road embankments detain flows*).

Each structure is comprised of a level-storage-discharge (HSQ) relationship. Level vs storage relationships were extracted from the TUFLOW model DEM at regular intervals. Initially discharges were determined using built-in WBNM broad-crested weir calculations, but the resulting comparisons to TUFLOW hydrographs (*the* *best available source of information on flow behaviour given a lack of flow gauging data*) were generally relatively poor. Accordingly, several TUFLOW simulations were completed to extract more reliable level vs discharge relationships. The ARR 1987 based 5% AEP and 1% AEP 120 minute and 360 minute storms and the GSDM PMF 60 minute and 180 minute storms were used for this purpose to include results across a range of storm magnitudes and durations.

After addition of the HSQ structures, the agreement between WBNM and TUFLOW in terms of hydrograph shape, timing and peak flow was generally excellent throughout the catchment (*refer* **Section 7.5**). This shows that the differing principals and numerical approaches behind each model are converging on a common result, thus providing additional confidence in the modelling process and ability of the hydrologic model to assess ARR 2019 critical storm durations and temporal patterns.

5.2.6 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 loss and continuing loss values adopted in this study for model calibration and design flood estimation are discussed in **Chapter 7** and **Chapter 8** respectively.

5.3 Hydraulic Model Development

5.3.1 2D Model Domain and Grid Size

The 2D TUFLOW hydraulic model domain comprises the entire 14 km^2 Willinga Lake catchment, 12 km^2 of land that drains to Meroo Lake (i.e. the Bawley Point Road local catchments), and 15 km^2 of land that drains across Murramarang Road into the Tasman Sea (i.e. the Murramarang Road local catchments). This results in a total area of about 41 km^2 as shown in **Figure 5-2**.

A model grid size of 3 m was adopted to adequately resolve flood characteristics in the study area, resulting in over 4.5 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 3 m grid this results in DEM elevations being sampled at 1.5 m centres.

5.3.2 2D Model Terrain

The 2D TUFLOW model terrain was constructed from a range of data sources in order to achieve an appropriate representation of the current condition of the floodplain and watercourses of the study area. The primary data sets and types include the following:

- 2011 NSW LPI LiDAR DEM ('Ulladulla201105' and 'BatemansBay201106')
  - Adopted as the primary topographic data set for the study
- 2018 DCCEEW marine LiDAR classified point cloud ('LkTburC2018')

  - Converted to a 1m DEM and used to supplement the 2011 LiDAR, particularly along watercourses and at Willinga Park

- Bathymetric survey
  - Used to improve the representation of Willinga Lake (OEH 2002 survey) and other watercourses where available
- Willinga Lake entrance surveys
  - Used to identify and define appropriate 'closed' and 'open' entrance conditions
- Ground survey
  - Used to improve the topographic representation of roads and other features at key watercourse crossings.
- Works As Executed and Design Plans
  - Used to improve the representation of certain structures including the Bawley Point Road bridge.

Details of model representation of the geometry of the Willinga Lake entrance berm at the beginning of a flood and its morphology during a flood are presented in **Section 6.2** of this report.

5.3.3 Boundary Conditions

The TUFLOW hydraulic model boundary conditions consist of the following:

- 'Surface area' application of 'local' flow hydrographs from each WBNM hydrologic model sub-catchment to the 2D hydraulic model domain.
- A downstream ocean water level boundary applied at the Tasman Sea along the eastern edge of the model.
- A normal-depth outflow boundary applied at Reedy Creek with a minimum level of 2.6 mAHD, equal to the Meroo Lake berm crest elevation evident in the 2011 LiDAR DEM.

The locations of these boundary conditions are shown in **Figure 5-2**.

5.3.4 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 consideration of aerial photography, LiDAR point cloud classifications, cadastral data, and site observations. Manning's 'n' values assigned to each zone were determined based on site observations and previous experience, 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 5-3**, with the delineation of hydraulic roughness zones shown in **Figure 5-3**. 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.

Table 5-3 Adopted Manning's 'n' hydraulic roughness coefficients

Material	Depth 1 (m)	Manning's 'n' Value 1	Depth 2 (m)	Manning's 'n' Value 2
Minor watercourses with vegetation	0.2	0.1	1.0	0.05
Lake and watercourses with clean sandy bed	0.1	0.06	0.3	0.03
Beach	0.1	0.08	0.3	0.04
Buildings	-	3.0	-	-
Medium density development	0.3	0.2	1.5	0.1
Low density development	0.2	0.1	0.6	0.06
Open Space	0.1	0.06	0.3	0.04
Vegetation	0.15	0.16	0.5	0.08
Road Corridor	0.05	0.06	0.15	0.03

5.3.5 Bridges

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

The geometry of the bridges including pier arrangement, span, deck thickness and level, and detail of handrails or safety barriers were taken from available survey and plans. Associated form losses were estimated using procedures detailed in *Waterway Design* (*AustRoads 1994*).

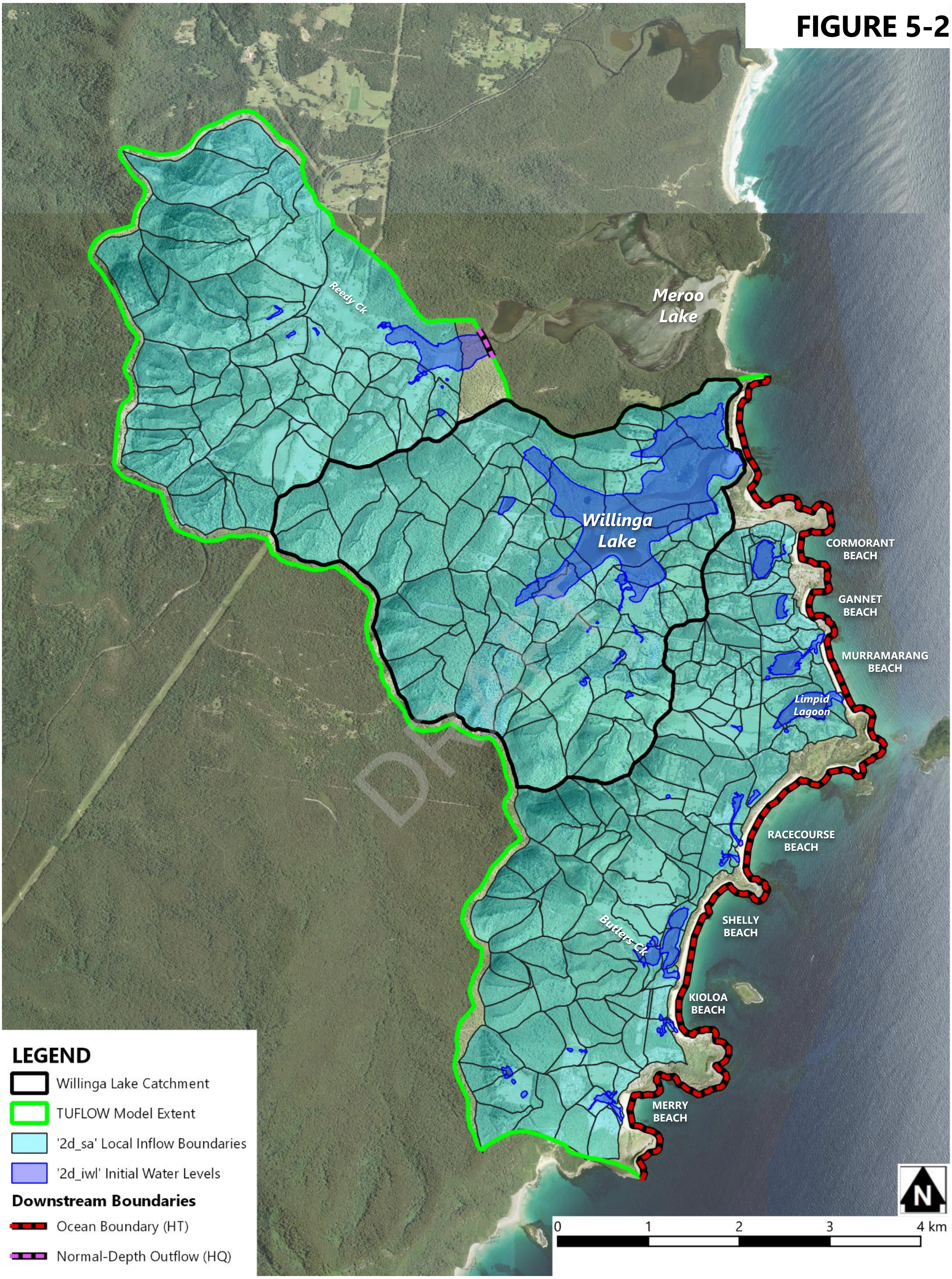
The locations of modelled bridges are shown in **Figure 5-4**. Structural details adopted in the TUFLOW model are presented in **Appendix B**.

5.3.6 Culverts

Culverts beneath major roads were represented in TUFLOW model using 1D elements which are dynamically linked to the 2D surface grid to allow the transfer of flows. Selected stormwater pits and pipes draining to these culverts were also added to the model. The extent of the 1D culverts and stormwater pipes included in the TUFLOW model is shown in **Figure 5-4**. The details of culverts and pipes were compiled from detailed survey data where available. This was supplemented by approximate measurements made by Council for several minor structures.

Structural details adopted in the TUFLOW model for culverts with 600 mm diameter and larger are presented in **Appendix B**.

FIGURE 5-2



Prepared by:

Shoalhaven City Council

worley consulting

TUFLOW HYDRAULIC MODEL EXTENT AND BOUNDARY CONDITIONS

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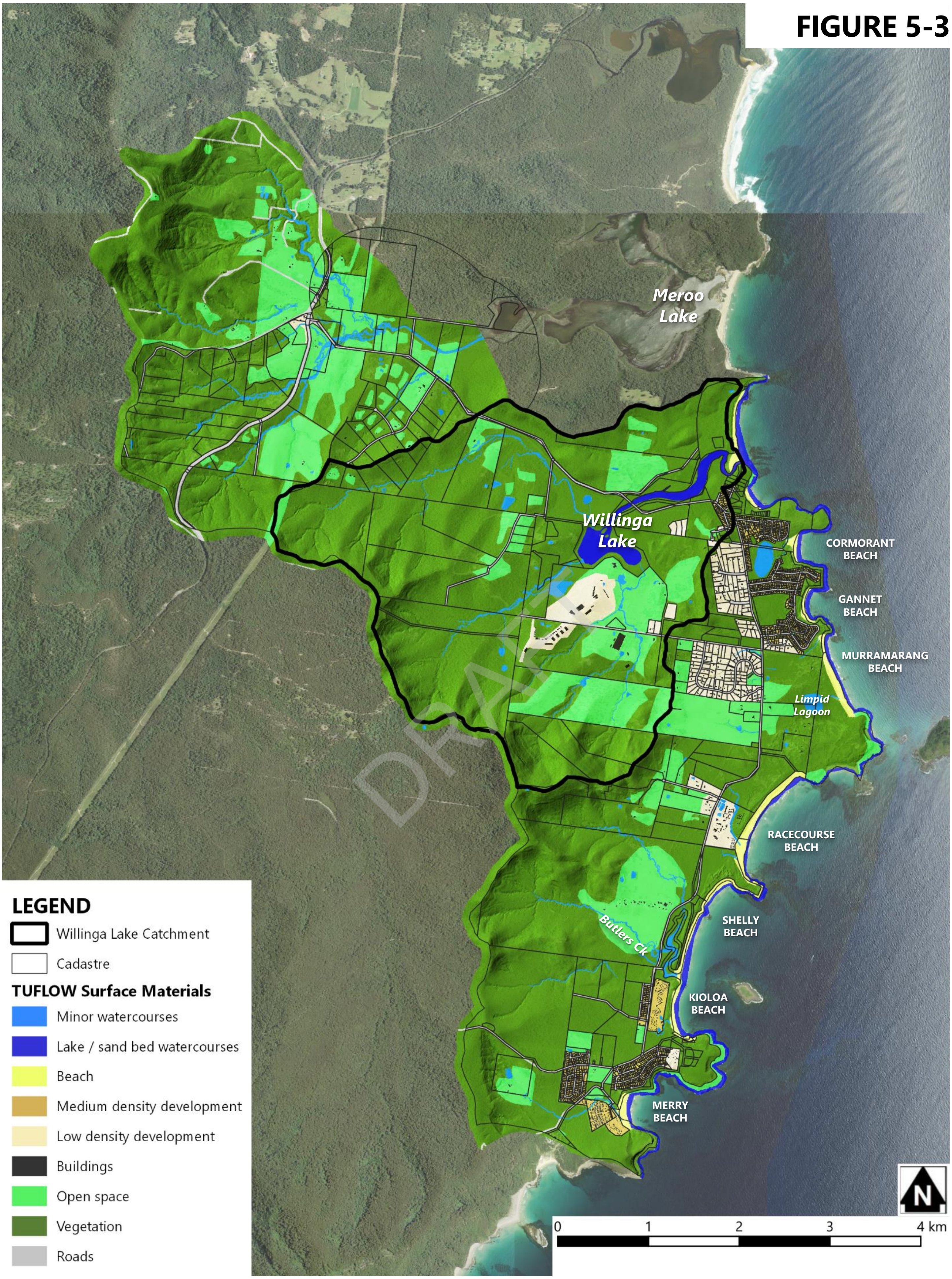
FIGURE 5-3



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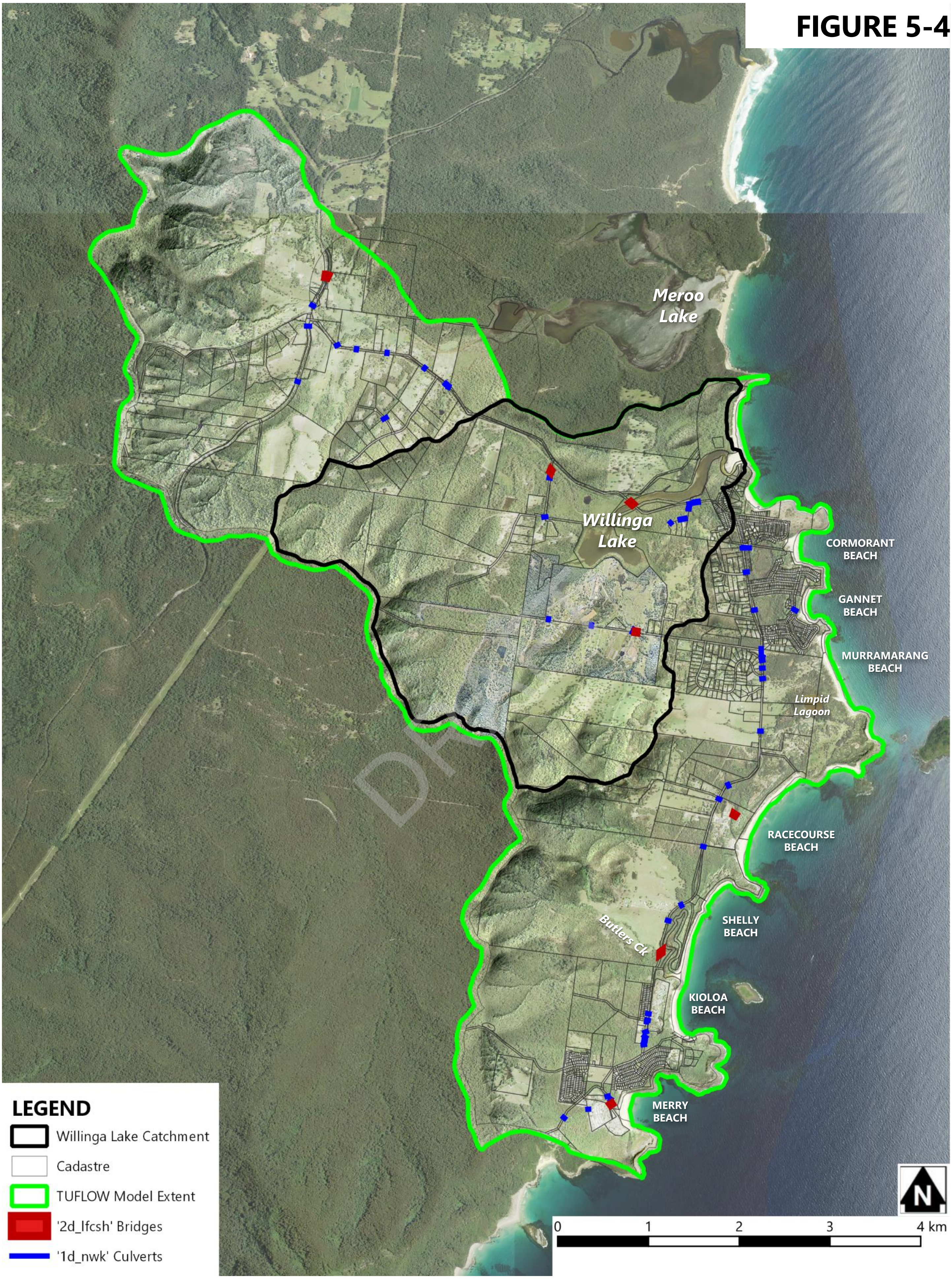
TUFLOW HYDRAULIC ROUGHNESS DELINEATION

FIGURE 5-4



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TUFLOW CULVERTS AND BRIDGES

6 Entrance Condition Analysis

Flood levels in Willinga Lake are influenced by the geometry of the entrance berm at the beginning of a flood, and by its morphology during a flood. To assist in understanding the influence of berm conditions on flooding and to select an appropriate condition for design flood estimation, a review of available data and model sensitivity testing were undertaken as described in the following sections.

6.1 Willinga Lake Entrance Condition Data Review

6.1.1 Aerial Photography

A review of Google Earth aerial photography of the Willinga Lake entrance between November 2004 and December 2022 has been undertaken, with a selection of images presented in **Figure 6-1**.

The condition of the entrance in each of the photographs is as follows:

- November 2004 – entrance open
- August 2013 – entrance open
- November 2015 – entrance closed
- December 2015 – entrance closed
- September 2018 – entrance closed
- April 2020 – entrance closed
- January 2021 – entrance closed
- December 2021 – entrance closed
- December 2022 – entrance closed.

Additional observations include the following:

- The aerial photography suggests that the entrance was predominantly in a closed condition from November 2015 to December 2022. It is noted that the entrance has historically closed up relatively quickly following opening, and that several openings would have occurred during this period.
- When in an open state, the channel was relatively narrow (*about 20 to 25 metres wide*) and hugged the rocks at Willinga Point.
- In some cases when the entrance was closed, the sand colouring suggested that the berm may have been recently overtopped by high tides and/or wave run-up.
- The berm width when closed ranged from about 30 m to about 70 m, with the narrowest part of the berm located close to the rocks at Willinga Point.

6.1.2 2002 OEH Bathymetric Survey, 2011 LiDAR and 2018 LiDAR

The entrance condition as captured in the 2002 bathymetric survey, 2011 LiDAR and 2018 LiDAR is presented in **Figure 6-2**. In all three data sets the entrance was in a closed condition with minimum berm crest elevations of 1.83 mAHD, 1.96 mAHD and 1.92 mAHD, respectively. The overall berm geometry was also comparable between the data sets with the minimum crest elevation located close to Willinga Point at the southern end of North Beach.

It is notable that survey of Bawley Point Road indicates a minimum road crest elevation of 1.92 mAHD about 70 metres to the east of Skylark Close, while the elevation along the road shoulder adjacent to this is

1.81 mAHD. This is lower than the entrance berm at the time of the 2011 and 2018 LiDAR surveys and, accordingly, if significant flood events were to have occurred at these times, Bawley Point Road would have been overtopped prior to initiation of natural breakout of the lake entrance.

6.1.3 Council Entrance Surveys (August 2022 to January 2024)

In preparation for this study, Council began conducting regular monthly surveys of the lake entrance in August 2022 and carried these through to January 2024. A record of key details from each survey has been compiled in **Table 6-1**. Less frequent surveys continued to be completed for the duration of this investigation.

Table 6-1 Summary of Council Survey of Lake Entrance (August 2022-January 2024)

Date	Entrance Condition	Channel Bed Elevation (mAHD)	Approx. Channel Width (m)	Min. Berm Crest Elevation (mAHD)
22nd August 2022	Closed	-	-	1.66
20th September 2022	Closed	-	-	1.68
20th October 2022	Open	<0.4*	10-20	-
21st November 2022	Open	<0.32*	10-20	-
21st December 2022	Closed	-	-	1.76
18th January 2023	Closed	-	-	1.93
17th February 2023	Closed	-	-	1.97
16th March 2023	Closed	-	-	2.04
1st June 2023	Open	-0.07	15-20	-
30th June 2023	Open	<-0.05*	15-20	-
28th July 2023	Open	<0.06*	10-15	-
31st August 2023	Open	<0.31*	5-15	-
28th September 2023	Closed	-	-	1.38
6th November 2023	Closed	-	-	1.65
4th December 2023	Open	<-0.17*	20-25	-
12th December 2023	Open	-0.51	20	-
4th January 2024	Open	-0.45	15	-

**No survey captured below the water surface*

The surveys provide useful insights into typical berm heights, scoured entrance geometry and the nature of the ICOLL opening and closure cycle.

The surveyed minimum berm crest elevations range from about 0.5 m below Bawley Point Road when the entrance had recently closed in September 2023, to about 0.2 m higher than Bawley Point Road when the entrance had been closed for 3 months or more in March 2023.

6.1.4 Review of Environmental Factors for Artificial Opening of Willinga Lake

Essential repairs to the Bawley Point Road bridge piles were begun in May 2006. When the work commenced, the lake entrance was closed to the sea and the water level was low enough for the repair work to be done. Significant rainfall (>260mm at Ulladulla) in late May and early June caused the water level to rise to a height that submerged the piles and was preventing continuation of work. Accordingly, Council prepared a Review of Environmental Factors (REF) to support a one-off mechanical opening of the lake entrance to enable to essential bridge works to continue.

Information on the historical management and opening of the entrance was included in the REF as follows:

- In the past, a trigger level (measured at the bridge) of 1.28m AHD was set, above which Council would mechanically open the lake. A new level of 1.4m AHD was set in 1989 after raising of the road.
- There has been concern that if water levels remain elevated for long periods, Bawley Point Road would suffer degradation. However, opening of the lake to protect the road was not approved last time an application was made to the NSW Government in 2001 even though the water level was above 1.4 mAHD.
- Information on known openings of the lake is presented in **Table 6-2**. The lake also opened between 1999 and 2006 but no records were available for the compilation of the REF.

Table 6-2 Known openings of Willinga Lake as reported in 2006 REF

Date	Level (mAHD)	Natural/Artificial Opening
21/8/1987	1.32	Artificial
7/7/1988	1.59	Artificial
6/1/1989	1.38	Artificial
27/1/1989 – New opening trigger level of 1.4m AHD adopted after raising of Bawley Point Road		
5/12/1989		Natural
4/2/1990	0.78	Natural (observed 5/2/1990)
5/4/1990	1.45	Artificial
9/6/1991	1.32	Natural
10/2/1992	1.68	Artificial
16/6/1995	1.49	Natural
12/6/1998	1.77	Artificial (Backhoe)
15/6/1999	1.68	Artificial (Backhoe)

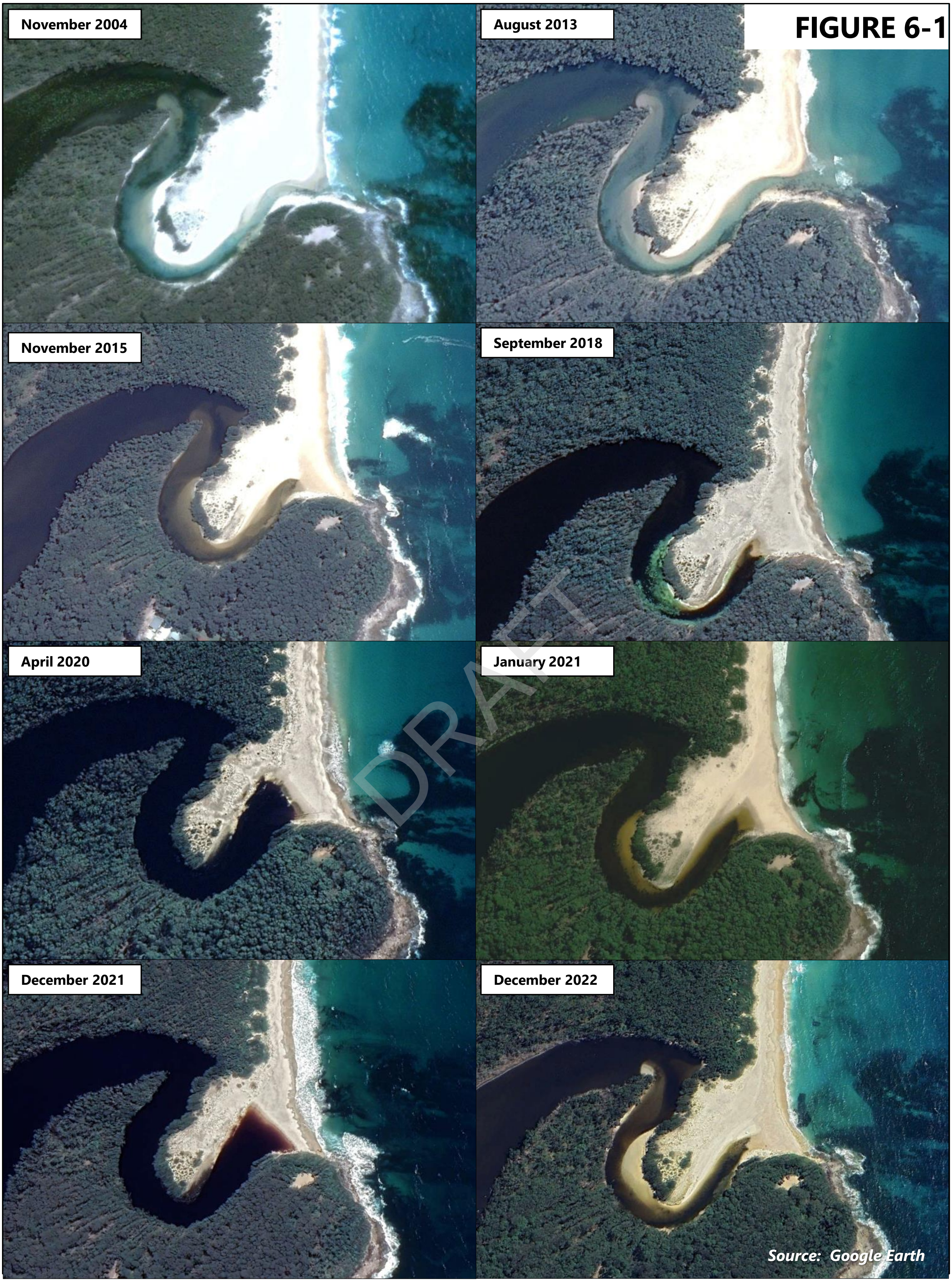
6.1.5 Summary

The available data and anecdotal information suggest that the Willinga Lake entrance is in a closed condition the majority of the time. Surveyed minimum berm crest elevations range from 1.38 mAHD (*when recently closed*) to 2.04 mAHD (*after more than 3 months of closure*), with the majority in the range of 1.66 mAHD to 1.97 mAHD.

One of the key objectives of the study is to investigate inundation of Bawley Point Road due to flooding of Willinga Lake. Detailed survey indicates that Bawley Point Road has a minimum crest elevation of 1.92 mAHD located about 70 metres to the east of Skylark Close. The minimum surveyed elevation along the road shoulder adjacent to this was 1.81 mAHD.

Given that the road elevation lies within the range of observed berm crest levels, inundation of Bawley Point Road is expected to be sensitive to the condition of the berm when a flood occurs.

FIGURE 6-1



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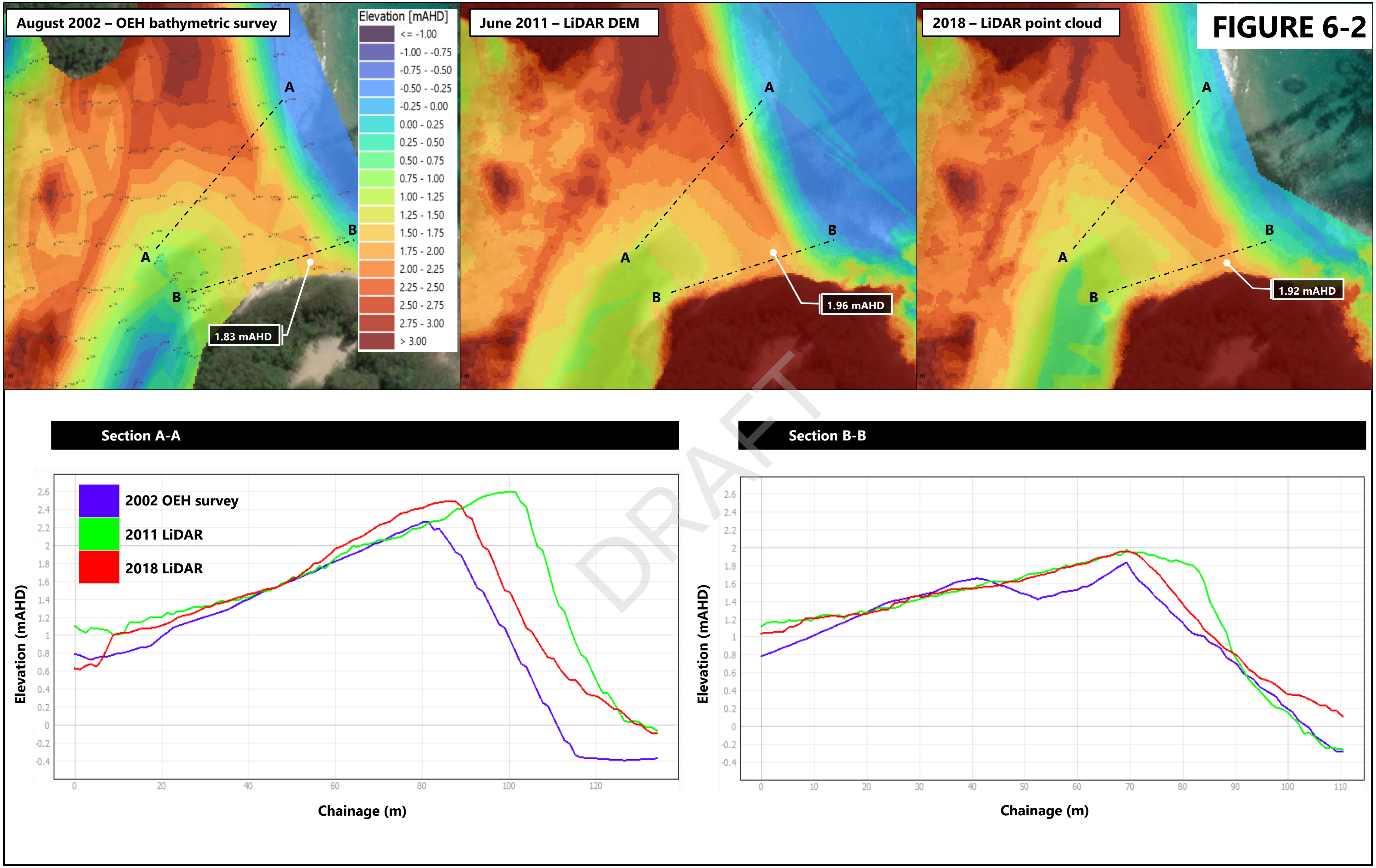
REVIEW OF AERIAL POTOGRAPHY OF THE WILLINGA LAKE ENTRANCE



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REVIEW OF OEH SURVEY & LIDAR AT WILLINGA LAKE ENTRANCE

6.2 Hydraulic Model Representation of Entrance Breakout

6.2.1 TUFLOW Variable Z-Shape Approach

To represent dynamic entrance breakout in the TUFLOW hydraulic model the '2d_vzsh' (time-varying elevation) technique was adopted. When a specified trigger is reached, the '2d_vzsh' approach changes the model terrain in the subject area from a specified initial condition to a specified final landform over a specified period of time (*refer* **Figure 6-3**).

Worley has utilised this approach effectively for other ICOLL systems such Bundeena Creek, Manly Lagoon and Fairy Lagoon, including successful calibration to recorded water level data and validation to more complex morphological modules.

An initial berm geometry was developed based on a Council entrance survey dated 6 November 2023 with a minimum berm crest elevation of 1.65 mAHD. The berm crest elevation was then adjusted as required using topographic modifications in TUFLOW.

The final scoured entrance geometry was based primarily on a Council entrance survey dated 12 December 2023 which represents the most scoured data set in terms of channel width but includes a sand bar in the mouth of the entrance. The sand bar was removed from the data by incorporating some slightly lower bed levels in that area based on the 4 December 2023 and 4 January 2024 surveys.

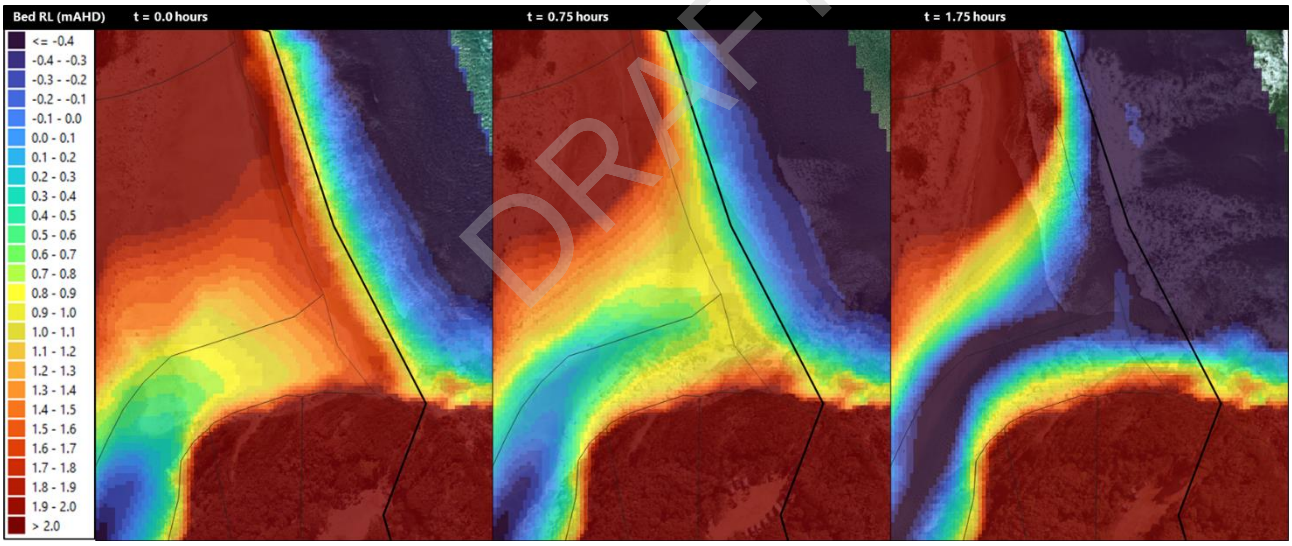


Figure 6-3 Progression of bed elevation using 2d_vzsh to represent entrance scour

6.2.2 Estimation of Entrance Scour Rate and Duration

With the initial berm and final scoured geometry selected, the final input required for the '2d_vzsh' approach is a duration over which the scour occurs.

Figure 6-4 presents a three staged model of ICOLL entrance breakout behaviour taken from *Coastal Lagoon Entrance Dynamics* (Gordon, 1990). It was derived based on detailed data collection and analysis of several breakouts at Dee Why Lagoon, Narrabeen Lagoon and Wollumboola Lake (in the Shoalhaven LGA). According to Gordon, although ICOLLs and their catchments vary in size and geometry their hydraulic behaviour, and entrance dynamics are remarkably similar. Accordingly, the data presented in **Figure 6-4** is considered a reliable estimation of the processes that are expected to occur at Willinga Lake.

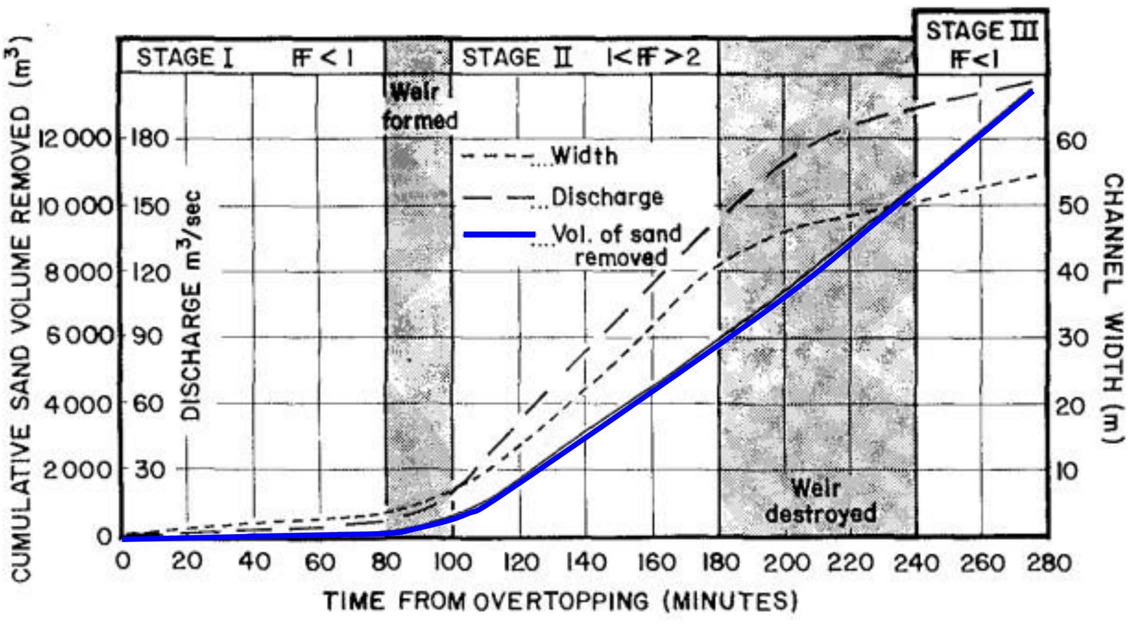


Figure 6-4 Entrance breakout behaviour model developed from studies of NSW ICOLLs

In terms of the amount of sand scoured from the entrance over time (*refer to the blue line on* **Figure 6-4**), sand removal is initially slow until a weir is formed, after which the rate of scour is close to linear at about 4,000 m^3 per hour.

Based on the selected initial and final model berm conditions the entrance plug was calculated to have a volume of approximately 7,000 m^3, which would take 1.75 hours to scour away at the identified rate. Initially 0.5 hours was added to this to allow for the formation of a weir giving a total scour duration of 2.25 hours. However, through the November 2023 calibration process it was found that lake water levels receded too slowly with a scour duration of 2.25 hours, and that a scour duration of 1.75 hours gave the best result.

Based on the model calibration findings a scour duration of 1.75 hours was adopted for design flood modelling. A trigger water level 0.1 m higher than the berm crest elevation was selected, allowing time for overtopping velocities to develop before initiating scour.

6.3 Sensitivity to Entrance Berm Conditions

To assist in understanding the influence of berm conditions on flooding and thereby select an appropriate condition for design flood estimation, the sensitivity of flood levels to three initial berm levels was investigated, both with and without simulation of entrance breakout.

The comparatively frequent 20% and 5% AEP events were selected for the analysis as the berm level was considered likely to have a major influence on inundation on Bawley Point Road in these events, and to have less influence on rare floods such as the 1% AEP.

Historic survey and anecdotal information on the Willinga Lake entrance berm elevation when closed range between about 1.3 and 2.0 mAHD, with most observations between about 1.65 and 1.95 mAHD (*refer* **Section 6.1**). Elevations at the lower end of the observed range were not considered for use in design modelling due to a lack of conservatism, while those at the upper end were not considered as they would theoretically result in frequent inundation of Bawley Point Road, which does not align with community observations. Accordingly, berm crest elevations of 1.6, 1.7 and 1.85 mAHD were selected for sensitivity testing purposes.

The results of entrance berm sensitivity analysis are presented in **Table 6-3** and are summarised as follows.

- A closed berm with no entrance breakout results in significantly higher peak water levels in Willinga Lake than with entrance breakout (*+0.34 to +0.45 m*), as would be expected.
- For the 20% AEP event
  - An increase in berm level from 1.6 mAHD to 1.85 mAHD <u>with</u> entrance breakout resulted in a 0.24 m increase in Willinga Lake water level. The depth of Bawley Point Road overtopping increased by only 20 mm, while the duration of inundation approximately doubled to about 3 hours (*refer* **Figure 6-5**).
  - An increase in berm level from 1.6 mAHD to 1.85 mAHD <u>without</u> entrance breakout resulted in a 0.13 m increase in Willinga Lake water level and in the depth of Bawley Point Road overtopping.
  - Elevated water levels in Willinga Lake drop away quickly following breakout as flows are not high enough to sustain them. This suggests shorter durations and front-loaded temporal patterns may be critical for frequent flood events.
  - Bawley Point Road adjacent to Willinga Lake is inundated in the 20% AEP across the range of berm elevations investigated. However, the mechanism and duration of the inundation differs. That is, for 1.6 and 1.7 mAHD berms the overtopping is driven by Skylark Close local catchment flooding (*refer* **Figure 6-7**), while for the 1.85 mAHD berm the peak depth is driven by Willinga Lake flooding.
- For the 5% AEP event
  - An increase in berm level from 1.6 mAHD to 1.85 mAHD <u>with</u> entrance breakout resulted in a 0.14 m increase in Willinga Lake water level. The depth of Bawley Point Road overtopping increased by only 30 mm, while the duration of inundation increased from about 5 hours to about 6 hours (*refer* **Figure 6-5**).
  - An increase in berm level from 1.6 mAHD to 1.85 mAHD <u>without</u> entrance breakout resulted in a 0.12 m increase in Willinga Lake water level and in the depth of Bawley Point Road overtopping.
  - For berm heights of 1.6 and 1.7 mAHD, Willinga Lake water levels continue to rise slightly for about 3 hours after breakout. This indicates that for larger floods flows can be high enough to sustain elevated lake levels and suggests that sensitivity to initial entrance berm level decreases with increase in flood magnitude.

The conclusions and recommendations from the analysis were as follows.

- Inclusion of dynamic entrance breakout in design flood estimation is recommended as application of a permanently closed entrance berm would be unrealistic and overly conservative.
- An initial berm elevation of 1.85 mAHD is not recommended for design flood estimation as even in the 50% AEP this would result in Willinga Lake driven flooding of Bawley Point Road. Notwithstanding the possible influence of human intervention, this would be difficult to reconcile with the limited historic observations of road flooding.
- The majority of available survey and anecdotal observations indicate berm crest elevations higher than 1.65 mAHD. Accordingly, an initial berm crest elevation of 1.7 mAHD was adopted for design flood modelling to avoid being under conservative (*refer* **Table 8-3**).

Table 6-3 Sensitivity of Willinga Lake 20% and 5% AEP peak flood levels to berm conditions

Design Flood Event	Initial Berm RL	Initial Water Level (mAHD)	Entrance Breakout Scenario	Peak WL at Gauge (mAHD)	Bawley Point Rd overtopping depth
20% AEP 9 hour storm TP4	1.6 mAHD	1.5 mAHD	Initiated at 1.7 mAHD, scour duration 1.75 hrs	1.73	0.04 m*
			No breakout	2.18	0.26 m
	1.7 mAHD	1.6 mAHD	Initiated at 1.7 mAHD, scour duration 1.75 hrs	1.83	0.04 m*
			No breakout	2.22	0.31 m
	1.85 mAHD	1.75 mAHD	Initiated at 1.7 mAHD, scour duration 1.75 hrs	1.97	0.06 m
			No breakout	2.31	0.39 m
5% AEP 6 hour storm TP6	1.6 mAHD	1.5 mAHD	Initiated at 1.7 mAHD, scour duration 1.75 hrs	1.91	0.05 m*
			No breakout	2.32	0.40 m
	1.7 mAHD	1.6 mAHD	Initiated at 1.7 mAHD, scour duration 1.75 hrs	1.95	0.06 m*
			No breakout	2.36	0.44 m
	1.85 mAHD	1.75 mAHD	Initiated at 1.7 mAHD, scour duration 1.75 hrs	2.05	0.09 m
			No breakout	2.44	0.52 m

**Driven by levels upstream (south) of Bawley Point Rd, i.e. predominantly Skylark Close local catchment flooding*

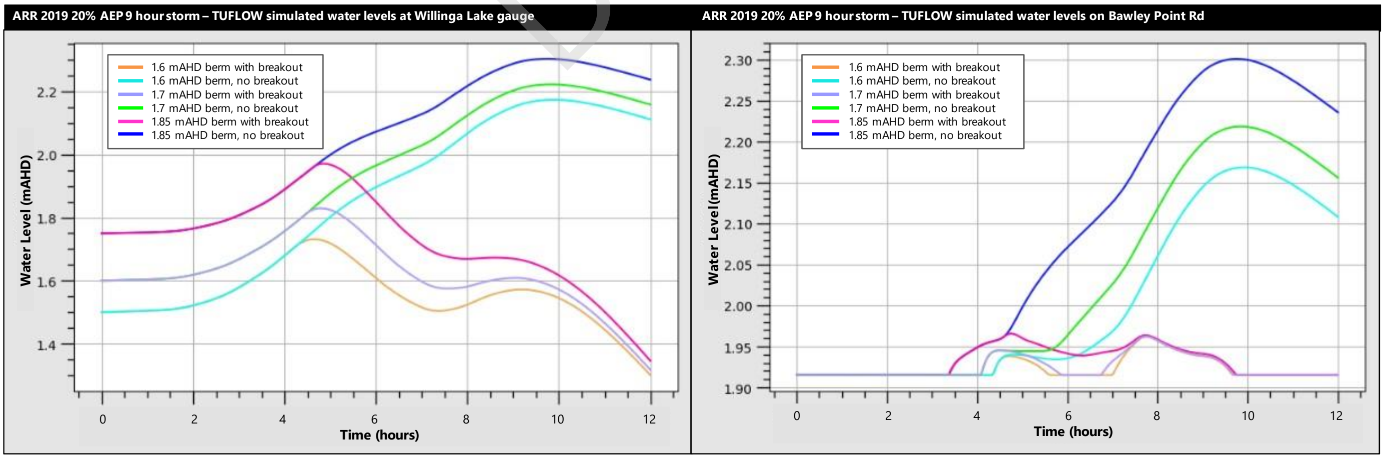


Figure 6-5 Sensitivity of water level hydrographs to berm level for the 20% AEP 9 hour storm

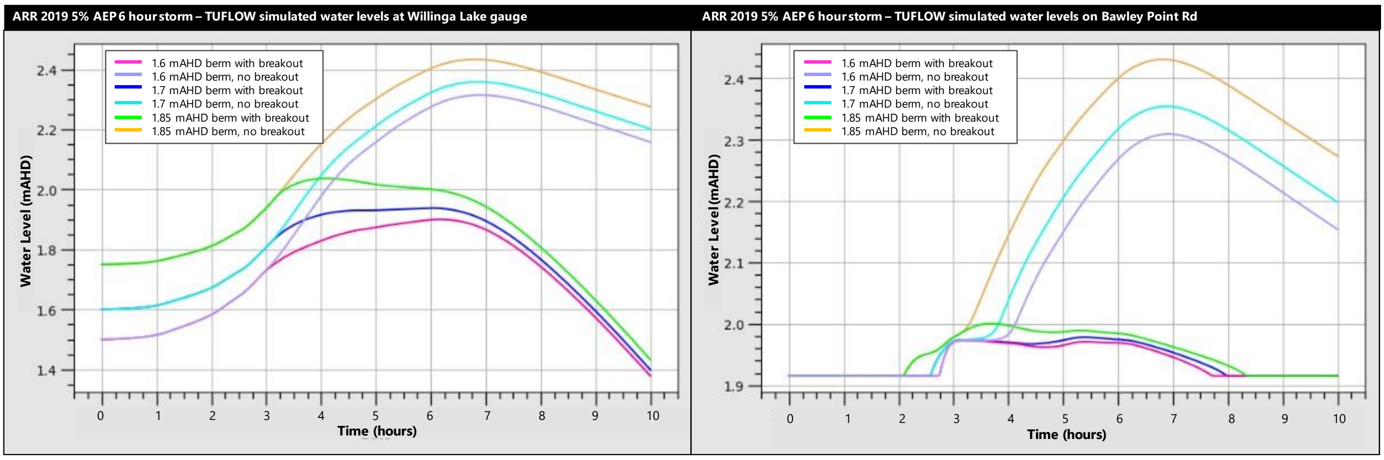


Figure 6-6 Sensitivity of water level hydrographs to berm level for the 5% AEP 6 hour storm

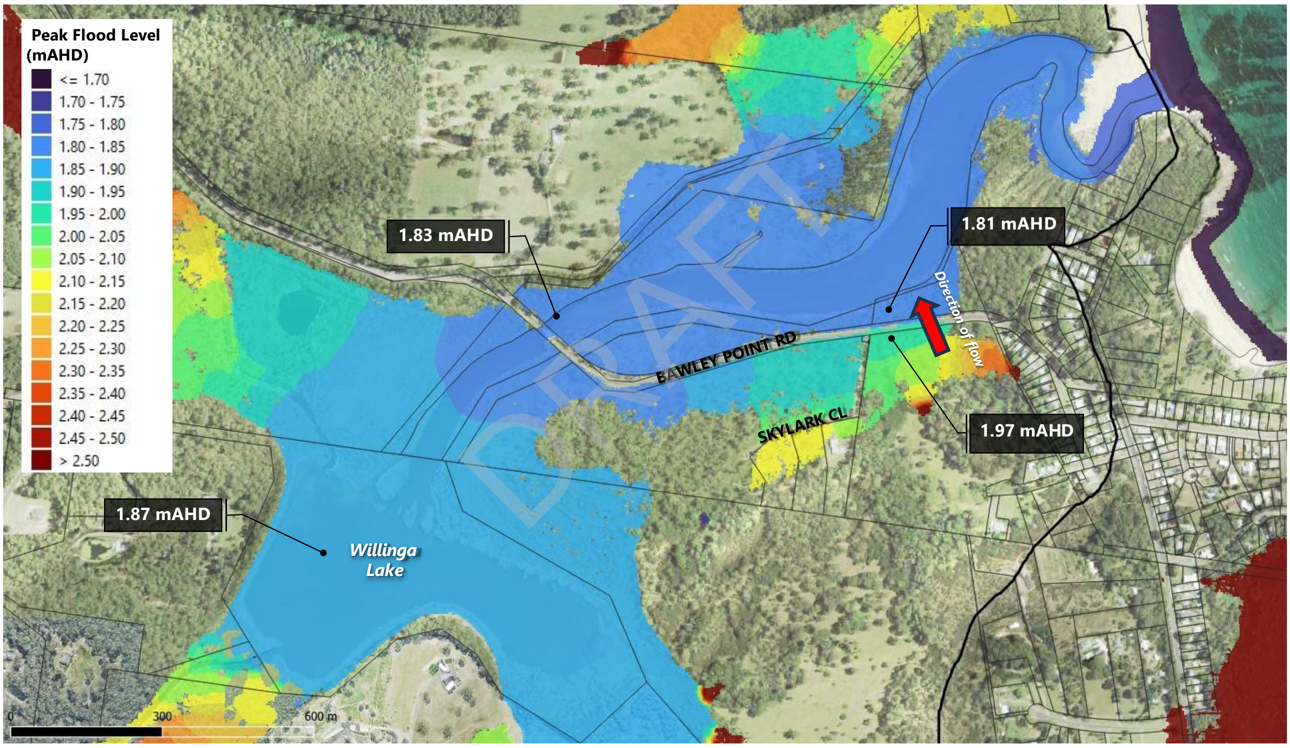


Figure 6-7 20% AEP 9 hr peak flood levels indicating local catchment flooding of Bawley Point Rd

7 Model Calibration, Verification and Validation

7.1 Overview

Model calibration and verification is an essential step in the flood modelling process. Confirmation that the models can reproduce observations and measurements from historical flood events is required to demonstrate their ability to reliably simulate expected flood behaviour in the study area.

7.2 Selection of Model Calibration and Verification Events

The suitability of historical flood events for use in model calibration and verification 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 magnitude including at least one major flood.

Calibration is particularly reliant on the availability of sub-daily rainfall and water level records, of which there is limited data available within the study area. The Willinga Lake pluviometer and water level gauge represent the best available data in the catchment but were only installed in December 2022. A pluviometer and stream flow gauge were previously located within the Butlers Creek catchment, however the readily available rainfall and water level records do not overlap temporally and are not of use for model calibration.

In order to identify significant rainfall events for potential use in model calibration and verification daily rainfall totals from the 069040 Kioloa Old Post Office and 569045 Willinga Lake gauges were ranked and consideration was given to the availability of pluviometer and flood level data for each event. Selected events are listed in **Table 7-1**. The majority of the largest daily rainfall totals from the Kioloa gauge did not have sufficient data available to be considered for use in calibration and were not listed.

Table 7-1 Selected significant storms identified from rainfall data

Date	24 Hour Rainfall Total (mm)	Ranking at 569045 Willinga Lake	Ranking at 069040 Kioloa	Available Calibration Data
November 2023	151.5	1	No Data	Willinga Lake pluvio & WLs
9 February 2023	~90	2	No Data	Willinga Lake pluvio & WLs
1-2 April 2023	~90	3	No Data	Willinga Lake pluvio & WLs
7 February 1971	278.6	No Data	1	No pluvio or WLs
18 December 2020	200	No Data	3	Ulladulla/Brooman pluvio, no WLs
12 October 2012	182	No Data	5	Ulladulla/Brooman pluvio, video at Northaven Ave
9 June 1991	123	No Data	34	No pluvio, 2006 REF cites natural breakout at 1.32 mAHD
16 June 1995	80.8	No Data	103	Ulladulla pluvio, 2006 REF cites natural breakout at 1.49 mAHD

From the events listed in **Table 7-1** the November 2023 storm was selected as the primary calibration event while the February 2023 storm was selected for use in model verification. These events were selected based primarily on the availability of data from the Willinga Lake water level gauge. It is noted that both events are considered fairly minor in terms of probability (*refer* **Section 7.3.3** *and* **7.4.1**).

7.3 Model Calibration – November 2023 Event

7.3.1 Event Overview

In late November 2023 a deep low-pressure system centred over southern New South Wales brought several days of widespread rain (*BoM, 2023*). On the 29th of November many sites had their highest November daily rainfall on record including the Ulladulla AWS gauge located about 17 km north of Willinga Lake.

In response to the rainfall, water levels in Willinga Lake began to gradually rise after 12 pm of the 28th of November, before beginning to rise more rapidly from about 6 am on the 29th. The Willinga Lake water level gauge recorded a peak level of 1.67 mAHD at 4:06 pm on the 29th after which water levels dropped rapidly indicating that the entrance berm had been overtopped and begun to scour out. By about 9 am on the 30th water levels had dropped back to 0.5 mAHD before rising with the incoming tide.

Anecdotal reports suggest that at its peak the water level in Willinga Lake came close to reaching the asphalt along the northern edge of Bawley Point Road near Skylark Close. Based on available survey data the edge of the roadway has an elevation of about 1.8 mAHD.

Elsewhere in the study area, Bawley Point Road was inundated by local catchment flooding at a location about 400 metres south-east of the Princes Highway (*refer* **Figure 7-1**).

Figure 7-1 Inundation of Bawley Point Road near Princes Highway, 29 November 2023

7.3.2 Rainfall Data

A cumulative rainfall plot for the period from 9 am 28 November 2023 to 9 pm 29 November 2023 is presented in **Figure 7-2** including available continuous rainfall gauge (pluviometer) data within and in closest proximity to the Willinga Lake catchment.

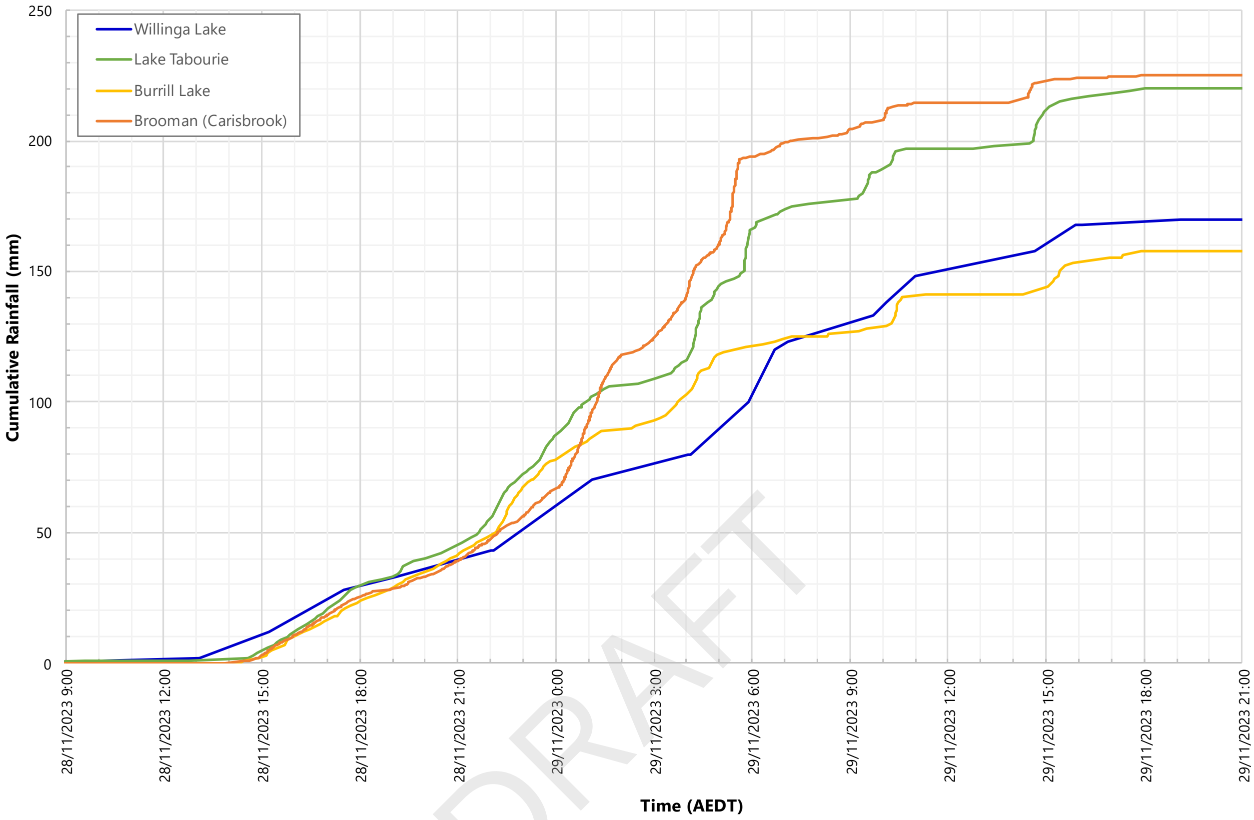


Figure 7-2 Cumulative rainfall plot for 9 am 28 November to 9 pm 29 November 2023

The Willinga Lake gauge recorded 168 mm of rain over a 30 hour period from 1:30 pm on 28 November 2023 including 151.5 mm over a 24 hour period. The heaviest period of rain began at about 4:15 am on 29 November with 43 mm recorded over the following 3 hours. Rainfall data from the Willinga Lake gauge was applied to flood models to enable their calibration to the November 2023 event.

Cumulative rainfall data for three other pluviometers nearest to Willinga Lake is also plotted in **Figure 7-2**. The total rainfall recorded at Burrill Lake (15 km north of Willinga Lake) was slightly lower than that at Willinga Lake, while Lake Tabourie (8 km north of Willinga Lake) and Brooman (11 km north-west of Willinga Lake) recorded considerably higher rainfall, particularly between about 10 pm 28 November and 6 am 29 November.

7.3.3 Review of Rainfall Return Frequency

In order to assess the relative intensity and probability of the rainfall that occurred at Willinga Lake during the November 2023 event, maximum recorded rainfall depths over durations from 15 minutes to 24 hours have been compared to Intensity-Frequency-Duration (IFD) data extracted from the Bureau of Meteorology Design Rainfall Data System (2016) at the location of the Willinga Lake gauge and as presented in **Figure 7-3**. Rainfall recorded at Lake Tabourie has been included for comparison.

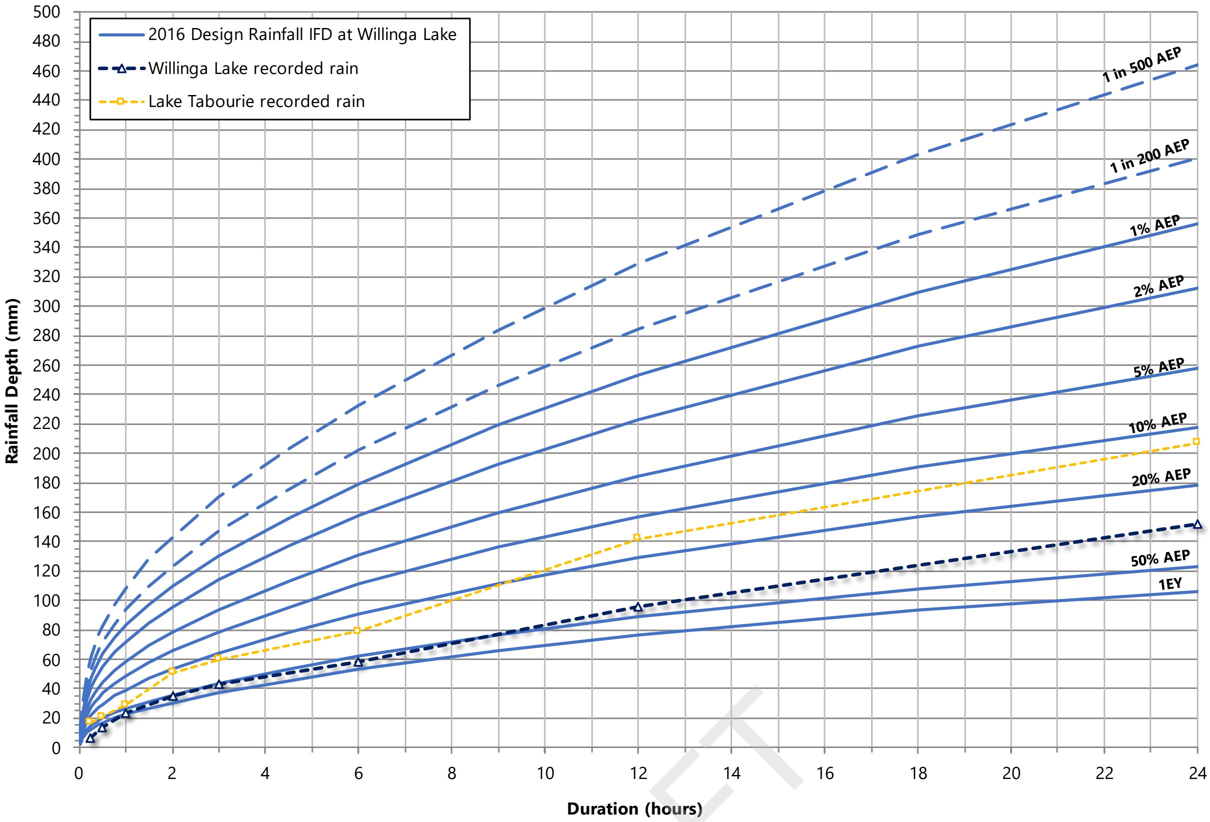


Figure 7-3 Comparison of November 2023 recorded rainfall to 2016 IFD at Willinga Lake

The return frequency of the Willinga Lake recorded rainfall is typically comparable to that of a 1 Exceedance per Year (EY) design rainfall event over durations from 15 minutes to 1 hour and to that of a 50% Annual Exceedance Probability (AEP) design rainfall event over durations from 2 hours to 12 hours. Over durations of 24 and 30 hours the recorded rainfall is about halfway between that of a 50% Annual Exceedance Probability (AEP) and 20% AEP design rainfall event. Rainfall at Lake Tabourie was higher across all durations and was in the order of a 20% AEP design rainfall event over durations from 2 hours to 12 hours, and slightly lower than a 10% AEP design rainfall event over a duration of 24 hours (*based on IFD at Willinga Lake*).

7.3.4 Water Level Data

7.3.4.1 Willinga Lake

Water level data recorded at Council's Willinga Lake (Bawley Point) gauge during the November 2023 event is presented in **Figure 7-4**. This data has been used as the primary means of calibrating flood models to the November 2023 event.

7.3.4.2 Ocean Level Data

Recorded data from MHL ocean level stations at Ulladulla Harbour and Princess Jetty (Batemans Bay) is also presented in **Figure 7-4**. Despite being slightly further away from Willinga Lake, data from the Princess Jetty gauge was selected to provide the downstream boundary condition for use in model calibration. The Princess Jetty data was selected as it exhibits a cleaner tidal signal with less low time-scale perturbations (likely related to wave seiching in Ulladulla Harbour). It is evident that the Princess Jetty level was influenced by catchment flows from the Clyde River during parts of the event but, importantly, this did not affect high tide on 30 November once the Willinga Lake entrance had opened.

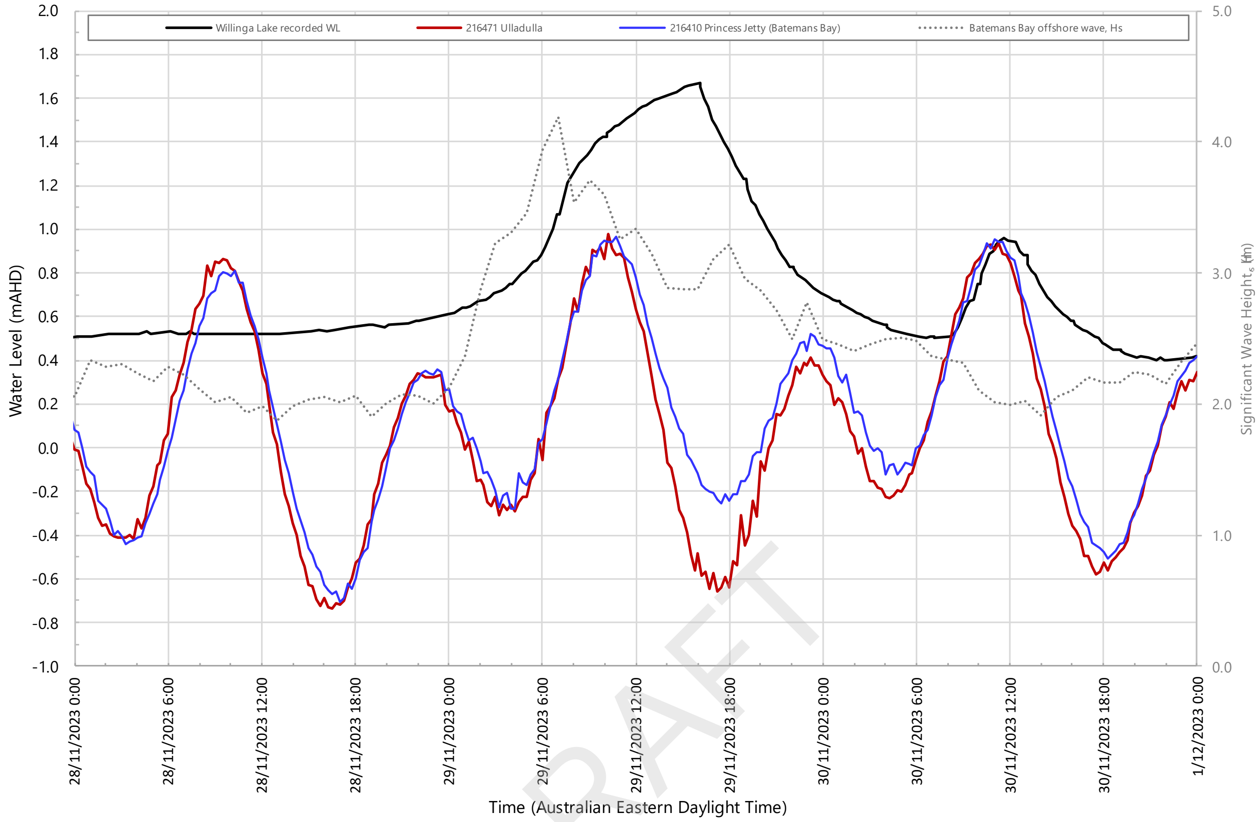


Figure 7-4 Recorded lake and ocean water level data for the November 2023 event

7.3.4.3 Wave Setup

Wave setup – *the super elevation of ocean levels in the surf zone caused by the breaking of waves* – is an important consideration in determining ocean tailwater levels at the entrances of ICOLLs and other shallow watercourses (e.g. OEH 2015). Data from the MHL Batemans Bay offshore wave buoy indicates that significant wave height, H_s, ranged from about 2.0 to 4.0 metres during the event (*refer* **Figure 7-4**), while the wave period, T, was around 8 seconds. Thus, some degree of wave setup at the lake entrance would have been expected.

Wave setup is a maximum at the shoreline where the depth, h, is zero. At the open lake entrance there is some depth, so setup is less than a maximum. The shoreline wave setup η_{max} can be estimated from the following equation derived from Goda (2000).

$$\eta_{max} = 0.043 H_0' m^{-0.116} \times \left({H_0'}/{L_0} \right)^{(0.0197 l_n m - 0.1592)}$$

Where H'_0 is the un-refracted deep-water significant wave height, m is the beach slope (typically 1/30) and L_0 the deep-water wavelength ($1.56T^2$).

Wave setup can be assumed to increase linearly with decreasing depth across the surf zone from the offshore break point (Goda 2000), assumed to be at a depth equal to H'_0. This gives wave setup at the lake entrance at $h_{entrance}$ as follows.

$$\eta_{h_{entrance}} = \left(1 - {h_{entrance}}/{H_0'}\right)\eta_{max}$$

Using the above equations with recorded values of Hs and T, and depth at the entrance taken from TUFLOW, gave a shoreline wave setup η_{max} of 0.31 metres and wave setup at the lake entrance of 0.09 metres. The wave setup of 0.09 m was added to the Princess Jetty tidal data for application at the November 2023 TUFLOW downstream boundary.

7.3.5 Hydrologic Model Calibration

7.3.5.1 The Hydrologic Model Calibration Process

Direct calibration of the WBNM hydrologic model was not possible as no recorded flow data is available in the study catchments for comparison to model results. Rather, calibration of the hydrologic model was undertaken in combination with that of the hydraulic model.

Representation of the stage-storage-discharge behaviour of Willinga Lake was included in the WBNM model using a 'structure' (*refer* **Section 5.2.5**). As a result, the WBNM model outputs an estimation of the Willinga Lake water level which can be compared to recorded water level data to provide an indication of the expected hydraulic model calibration. Thus, rainfall and loss rate inputs were tested and adjusted in the WBNM model to produce a desired result prior to their testing in the TUFLOW hydraulic model.

Pseudo-calibration of the hydrologic model was also undertaken by comparing simulated flood hydrographs from the WBNM model to those from the TUFLOW hydraulic model at a series of locations and refining hydrologic parameters to help improve the match. As described in **Section 5.2.5** of this report, this process helped identify storages and structures warranting representation in the hydrologic model, and showed that the differing principals and numerical approaches behind each model are converging on a common result, thus providing additional confidence in the modelling process.

7.3.5.2 Calibration of Hydrologic Model Parameters

Calibration of a WBNM hydrologic model generally involves modification of the WBNM runoff lag factor 'C' and stream routing lag factor 'F', as well as initial loss (IL) and continuing loss (CL) rates.

The runoff lag factor 'C' controls the timing of locally generated runoff from each model sub-catchment. A default C value of 1.6 is recommended as a starting point, with values of between 1.3 (*rapid runoff response*) and 1.8 (*slower runoff response*) generally appropriate.

A default C value of 1.6 was initially adopted and no evidence was found through the calibration process to warrant altering this value.

With the runoff parameter 'C' determined, refinement of the WBNM stream lag parameter 'F' was then undertaken to achieve improved matches between simulated November 2023 WBNM and TUFLOW hydrographs as described in the following section. This resulted in an F value of 1.0 for most of the study area, with a lower F value of 0.8 applied in model sub-catchments where land along the flood flow path is predominantly cleared and/or developed.

7.3.5.3 Pseudo-Calibration to TUFLOW Flood Hydrographs

Pseudo-calibration of the hydrologic model was undertaken by comparing simulated flood hydrographs from the WBNM model to those from the TUFLOW hydraulic model at a series of locations. This process was undertaken for the following reasons:

- Agreement in the hydrograph shape and peak flows shows that the differing principals and numerical approaches behind each model are converging on a common result, providing additional confidence in the modelling process.
- The hydrologic model is used to assess 'critical storm duration' at key locations throughout the catchment. It is therefore important that the total flows (i.e. routed flows) calculated by the hydrologic model be reliable at these locations. The detailed TUFLOW hydraulic model provides the best available source of information on stream routing behaviour throughout the study area.
- The comparisons also provide a form of quality check to confirm that water volume is being properly applied and conserved within the TUFLOW model.

After adding a series of stage-storage-discharge (HSQ) structures to the WBNM model (*refer* **Section 5.2.5**) the WBNM-TUFLOW hydrograph comparisons were generally very good. Further improvement was then achieved at some locations by lowering the stream lag 'F' value from 1.0 to 0.8 where land along the flood flow path is predominantly cleared and/or developed.

A comparison between November 2023 flood hydrographs simulated using the WBNM hydrologic model, and those extracted from the TUFLOW hydraulic model at equivalent locations is presented in **Figure 7-5**. The results of the hydrograph comparisons are summarised as follows:

- Agreement in hydrograph shape, timing and peak flow is generally excellent throughout the catchment. This includes results across a range of catchment sizes and stream types.
- It was found that WBNM could not replicate TUFLOW flow hydrographs at the Willinga Lake entrance when initial water levels are low relative to the berm height. This is because the HSQ relationship specifies a single flow for any given water level. For November 2023 the flow is zero from the initial level of 0.5 mAHD until the berm overtops at about 1.65 mAHD. At this stage the entrance berm begins to scour and flows can increase despite water levels dropping, a behaviour which cannot be replicated in WBNM. Accordingly, at this location the pseudo-calibration focused on matching the WBNM 'stage' to the recorded / TUFLOW water level up until the time of entrance breakout.
- Differences in peak flows are evident at two local road catchment locations during this relatively minor storm event. However, no such issues were evident for larger design storms of magnitudes of greater interest to this study (*refer* **Section 7.5**).

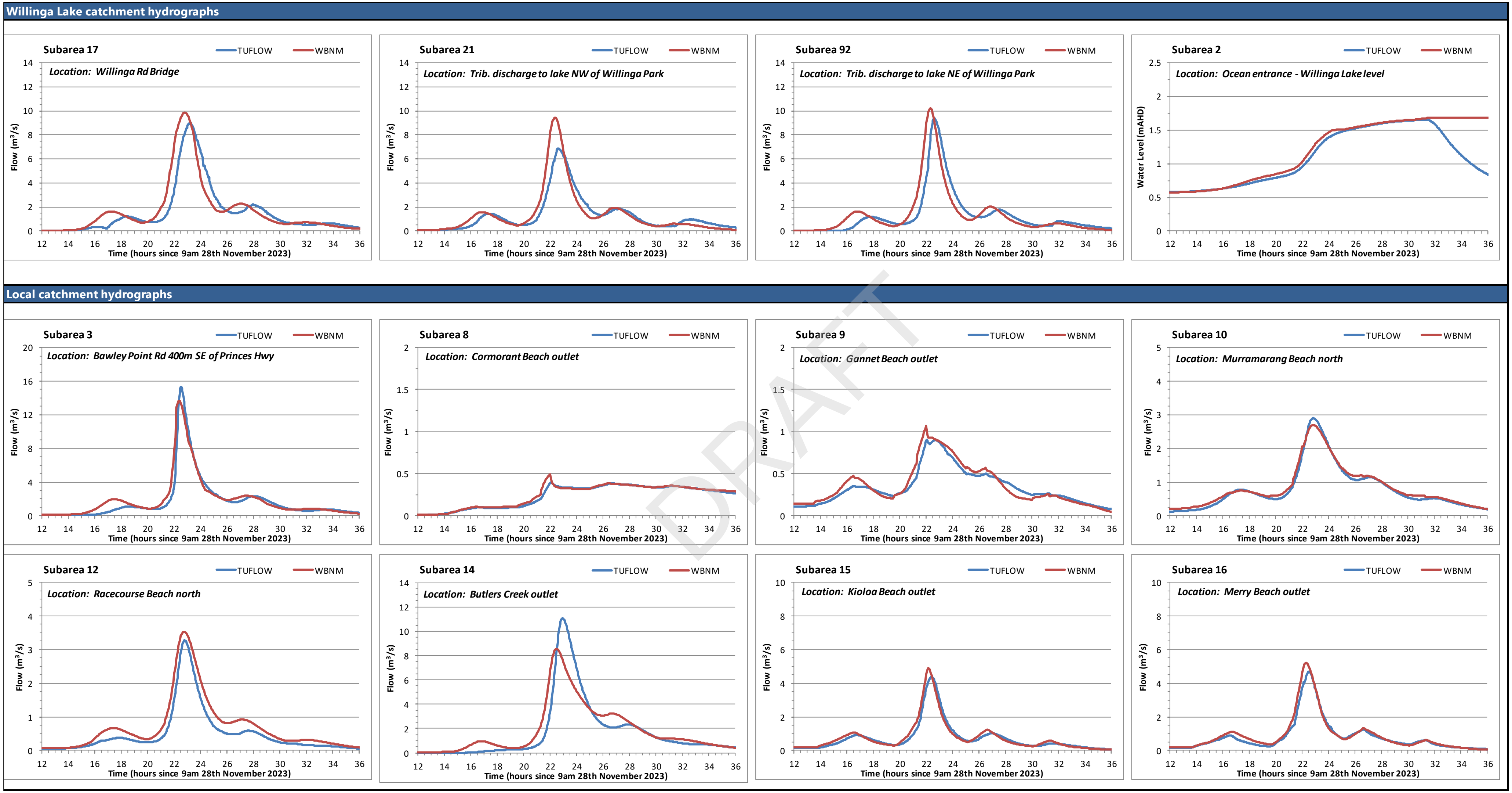


Figure 7-5 Comparison of WBNM and TUFLOW simulated November 2023 hydrographs

7.3.5.4 Calibration of Rainfall Loss Rates

A range of rainfall loss rates sourced from ARR 1987 and ARR 2019 were tested in the WBNM hydrologic model and adjusted until the stage hydrograph for Willinga Lake produced a result comparable to the recorded water level data. The identified loss rates were then tested in the TUFLOW hydraulic model to confirm the calibration result and appropriateness of the loss rates.

The above process resulted in an adopted IL of 50 mm and CL of 6.5 mm/hr for the November 2023 event.

It is noted that this initial loss rate is considered to be quite high. It is believed that this owes partly to the low initial water level in Willinga Lake, and thus low groundwater table in the surrounding sandy soils, for this event (0.5 mAHD). It is expected that rain would have continued to infiltrate into the sand until the groundwater table reached the ground surface. Initial loss would not be expected to be this high if the initial water level and groundwater table were relatively high at the start of a rainfall event, as is the case for design flood modelling.

The sensitivity of the calibration results to rainfall loss rates is investigated in **Section 7.3.8**.

7.3.6 Hydraulic Model Calibration

The hydraulic model calibration was assessed through the comparison of flood levels simulated by the TUFLOW model with water levels recorded by the Willinga Lake gauge. A comparison was also made to anecdotal information regarding local catchment driven inundation of Bawley Point Road.

7.3.6.1 Model Representation of November 2023 Conditions

Relevant conditions during the November 2023 flood event were represented in the TUFLOW hydraulic model as follows:

- The initial geometry of the Willinga Lake entrance berm was represented using survey undertaken by Council on 6 November 2023 with a minimum berm crest elevation of 1.65 mAHD.
- An initial water level of 0.5 mAHD was applied in Willinga Lake per the gauge record.
- A dynamic breakout of the Willinga Lake entrance berm was included in the simulation with the final scoured entrance condition informed by Council surveys undertaken on 4 December 2023, 12 December 2023 and 4 January 2024. The breakout took 1.75 hours to reach the fully scoured condition, reduced from an initial estimate of 2.75 hours (*refer* **Section 6.2.2**) to better replicate recorded water levels.

7.3.6.2 Comparison of Recorded and Simulated Water Levels

The results of the November 2023 calibration to recorded water level data is presented in **Figure 7-6**.

The match in the timing, shape and peak of recorded and simulated flood level hydrographs is excellent throughout the flood including the rising limb, peak, falling limb and subsequent tidal signal. The simulated flood peak of 1.66 mAHD provides an excellent match to the recorded flood peak of 1.67 mAHD, though this is heavily influenced by breakout of the entrance berm.

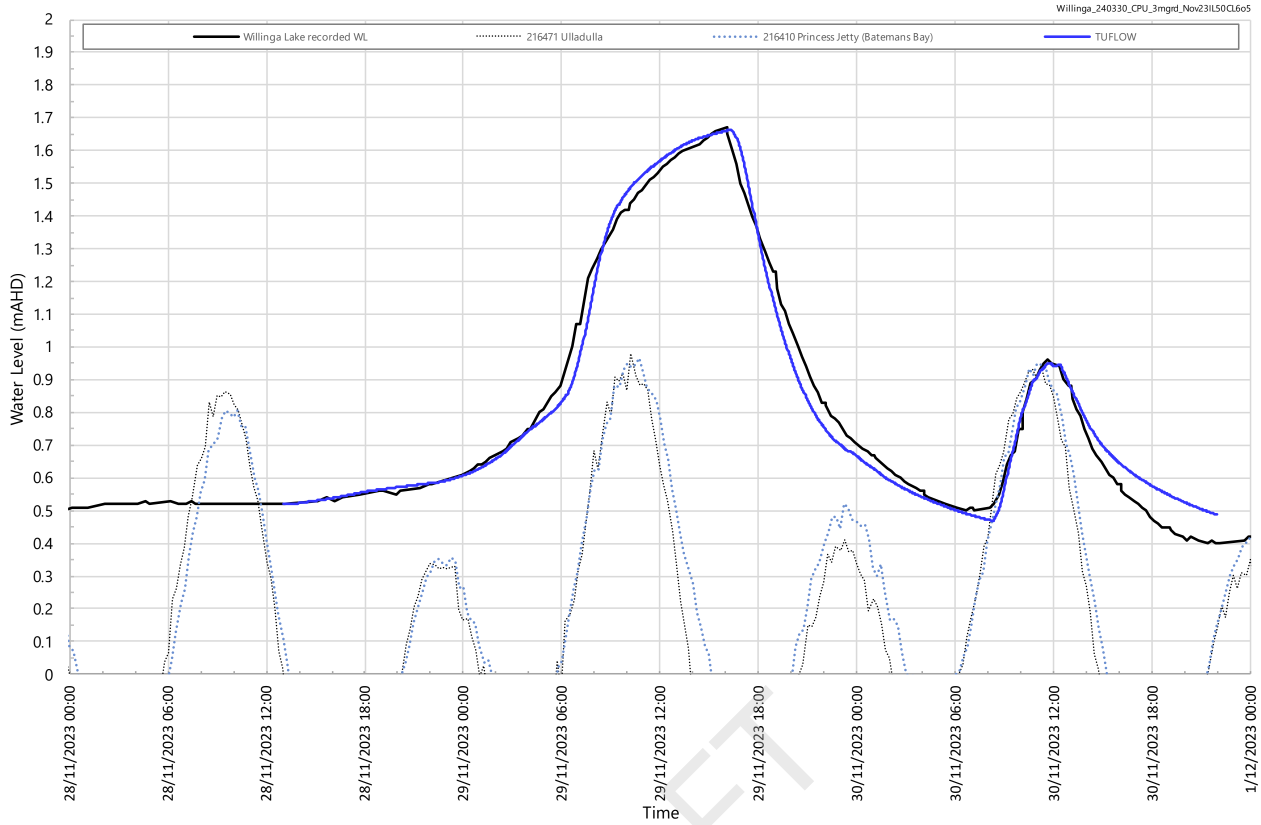


Figure 7-6 Comparison of recorded and simulated flood levels for the November 2023 event

7.3.6.3 Comparison to Anecdotal Information

Following the November 2023 storm Council received a photograph from a local resident showing Bawley Point Road being inundated by local catchment flooding at a location about 400 metres south-east of the Princes Highway (*refer* **Figure 7-1**).

The TUFLOW model simulation of the event indicates a maximum depth of inundation of 0.25 m along Bawley Point Road at this location with a 100 metre length of road inundated at the peak of the flood. Unfortunately, it is difficult to estimate any specifics from the photograph for comparison though it appears that a significant length of road was inundated.

Figure 7-7 November 2023 event comparison of TUFLOW results to photograph

7.3.7 Summary of November 2023 Model Calibration

The calibration of the WBNM hydrologic and TUFLOW hydraulic model is summarised as follows:

- A comparison of recorded and simulated flood levels at the Willinga Lake gauge for the November 2023 storm indicates that the timing, shape, magnitude and peaks of simulated flood level hydrographs closely match recorded water level data.
- A comparison of flood hydrographs extracted from the WBNM and TUFLOW models generally shows excellent agreement in terms of timing, shape, peak flows and volume.

The calibration results provide confidence in the ability of the developed WBNM hydrologic and TUFLOW hydraulic models to replicate actual flood behaviour in the study catchments. This includes representation of dynamic breakout of the Willinga Lake entrance berm and interaction with oceanic conditions.

The cross-comparison of hydrographs from the WBNM hydrologic and TUFLOW hydraulic models shows that the differing principals and numerical approaches behind each model are converging on a common result. This provides additional confidence in the model calibration and in the subsequent use of the WBNM hydrologic model to assess critical storm durations and estimate design flood flows.

It is noted that the November 2023 storm was of a relatively small magnitude (50% to 20% AEP), and it cannot be guaranteed that the models would perform equally well if calibrating to a rarer storm event (e.g. of a 1% AEP magnitude). However, Willinga Lake is a volume dominated system where the entrance berm condition also has a strong influence on flood behaviour. It is considered that the developed models reliably represent the storage volume of the floodplain and the influence of the entrance berm, and thus it is expected that they would ably replicate flood events rarer than the November 2023 storm.

7.3.8 Sensitivity Analysis

A series of sensitivity tests were undertaken for the November 2023 calibration event using the WBNM hydrologic model to assess the influence of alternative model parameters, rainfall loss rates and rainfall inputs on peak flow rates at a range of locations (*or estimated peak water level in the case of Willinga Lake*). The results are presented in **Table 7-2** and are discussed in the following sections.

7.3.8.1 WBNM Runoff Lag Parameter 'C'

The calibrated WBNM hydrologic model adopted a standard runoff lag value of 1.6. The influence of alternative values of 1.4 (*representing a faster runoff response*) and 1.8 (*representing a slower runoff response*) on peak flow rates was assessed with the findings as follows (*refer* **Table 7-2**).

- At Willinga Lake both the increase and decrease in runoff rate had no impact on the estimated peak water level. This is because levels in the lake are dominated by volume rather than the timing of inflows.
- Under a faster runoff response (C=1.4) simulated peak flow rates increased at all other locations. The increase ranged from 3% at watercourses where storage volume has an important influence, to 11% along flow dominated watercourses.
- Under a slower runoff response (C=1.8) simulated peak flow rates decreased at all other locations. The decrease ranged from 3% at watercourses where storage volume has an important influence (*i.e. where timing is less important*), to 9% along flow dominated watercourses.

7.3.8.2 WBNM Stream Routing Lag Parameter 'F'

The calibrated WBNM hydrologic model adopted a standard stream routing lag 'F' value of 1.0 for natural sub-catchments and a lower (*i.e. faster*) value of 0.8 in model sub-catchments where land along the flood flow path is predominantly cleared and/or developed. The influence of alternative catchment-wide 'F' values of 0.6 and 0.8 on peak flow rates was assessed with the findings as follows (*refer* **Table 7-2**).

- At Willinga Lake the reduction in stream routing lag had no impact on the estimated peak water level as levels in the lake are dominated by volume rather than the timing of inflows.
- At all other locations an 'F' value of 0.8 resulted in simulated peak flow rate increases ranging from 1% to 10%. The lower increases occurred at watercourses where part of the catchment is already cleared/developed (*i.e. F is already 0.8*) and/or where storage volume has an important influence on peak flow.
- At all other locations an 'F' value of 0.6 resulted in simulated peak flow rate increases ranging from 6% to 21%. In this case the lower increases generally occurred at watercourses where storage volume has an important influence on peak flow.

7.3.8.3 Alternative Rainfall Loss Rates

The calibrated WBNM hydrologic model adopted IL of 50 mm and CL of 6.5 mm/hr for the November 2023 event. Rainfall loss rates sourced from ARR 1987, ARR 2019 and the associated 'FFA reconciled losses' for nearby catchments were initially tested in the WBNM hydrologic model and adjusted until a desirable calibration was achieved.

The influence of the alternative rainfall loss rates on peak flow rates was assessed with the findings for Willinga Lake as follows (*refer* **Table 7-2**).

- ARR 1987 standard loss rates for NSW east of the western slopes (IL 10 mm, CL 2.5 mm/hr)
  - The estimated peak level in Willinga Lake is 0.10 m higher than for the adopted loss rates.
  - This owes to both a significantly lower IL and CL than those adopted.
- ARR 2019 'original' loss rates for the catchment (IL 17 mm, CL 5.5 mm/hr)
  - The estimated peak level in Willinga Lake is 0.02 m higher than for the adopted loss rates.
  - The IL is significantly lower than the adopted value, while the CL is only slightly lower than the adopted value. Given that the resulting difference in estimated peak water level is small it appears that peak water level is more sensitive to CL than IL for the November 2023 event (*notwithstanding that IL would influence the initial hydrograph shape and timing*).
- ARR 2019 'FFA-reconciled' loss rates for the Kadoona gauge (IL 35 mm, CL 2.4 mm/hr)
  - The estimated peak level in Willinga Lake is 0.08 m higher than for the adopted loss rates.
  - This IL is considerably higher than the ARR 2019 value which was tested yet resulted in a considerably higher estimated peak water level. This reinforces that peak water level is more sensitive to CL than IL for the November 2023 event.

Findings regarding the influence of the alternative rainfall loss rates on peak flow rates at all other locations are summarised as follows (*refer* **Table 7-2**).

- ARR 1987 standard loss rates for NSW east of the western slopes (IL 10 mm, CL 2.5 mm/hr)
  - Peak flow rates are higher at all locations owing to both a significantly lower IL and CL than those adopted, with increases at most locations ranging between 14% and 57%.
  - Higher increases of 125% and 241% occurred at Cormorant Beach and Murramarang Beach south (Limpid Lagoon) respectively. These two watercourses have large storage areas relative to their small catchments. As a result, the loss of volume associated with high rainfall loss rates has a pronounced impact on discharge.
- ARR 2019 'original' loss rates for the catchment (IL 17 mm, CL 5.5 mm/hr)
  - Peak flow rates are higher at all locations owing to both a significantly lower IL and CL than those adopted, with increases at most locations ranging between 3% and 15%.
  - A higher increase of 53% occurred at Murramarang Beach south (Limpid Lagoon) which has a large storage area relative to its small catchment.
  - It is evident that peak flows are more sensitive to CL than IL for the November 2023 event, and that some watercourses are more sensitive to IL rates than others.
- ARR 2019 'FFA-reconciled' loss rates for the Kadoona gauge (IL 35 mm, CL 2.4 mm/hr)
  - Peak flow rates are higher at all locations owing to both a lower IL and CL than those adopted, with increases at most locations ranging between 13% and 50%.

- Higher increases of 88% and 213% occurred at Cormorant Beach and Murramarang Beach south (Limpid Lagoon) respectively. These two watercourses have large storage areas relative to their small catchments.
- This IL is considerably higher than the ARR 2019 value which was tested yet resulted in considerably higher estimated peak flows. This reinforces that peak flows are more sensitive to CL than IL for the November 2023 event.

7.3.8.4 Alternative Recorded Rainfall

The November 2023 calibration adopted rainfall from the Willinga Lake gauge which recorded 168 mm of rain over a 30 hour period including 151.5 mm over 24 hours and 43 mm over 3 hours. The nearest alternative pluviometer is located about 8 km north at Lake Tabourie. Rainfall at Lake Tabourie was 30% to 45% higher than at Willinga Lake across a range of durations, with 219 mm of rain recorded over a 30 hour period including 207 mm over 24 hours and 60 mm over 3 hours.

The Lake Tabourie rainfall was applied to the WBMN hydrologic model to assess the implications had similar rainfall occurred at Willinga Lake. The findings are summarised as follows (*refer* **Table 7-2**).

- The peak water level in Willinga Lake could have been about 0.12 m higher had the Lake Tabourie rainfall occurred locally. This suggests that water would have reached the edge of the asphalt on Bawley Point Road near Skylark Lane.
- Elsewhere in the study area, peak flow rates are higher at all locations.
  - At most locations the percentage increase in peak flow was comparable to the percentage increase in rainfall (26% to 50%).
  - Higher percentage increases in flow occurred at some locations, particularly at Cormorant Beach (234%) and Murramarang Beach south (320%). The increases at these locations would have been driven primarily by the significant increase in runoff volume.

7.3.8.5 Conclusions

The findings of the November 2023 event sensitivity analysis can be summarised as follows:

- Willinga Lake
  - Peak flood levels and flows at Willinga Lake are volume dominated and are not particularly sensitive to changes affecting the rate of runoff and timing of stream flows from the catchment.
  - For this event, it was found that Willinga Lake flood levels were more sensitive to changes in the continuing loss rate than the initial loss rate (*notwithstanding that IL would influence the initial hydrograph shape and timing*).
  - The above provides confidence that the developed models and adopted loss rates are appropriate for estimating flood levels in Willinga Lake.
- At all other locations peak flood flows were more sensitive to changes affecting the rate of runoff and timing of stream flows. However, no calibration data was available in these areas to inform the potential refinement of the adopted parameters. Accordingly, confidence in the model results in these areas is lower than at Willinga Lake.

Table 7-2 Results of November 2023 event WBMN hydrologic model sensitivity testing

WBNM subarea	Location	Adopted WBNM Peak flow (m^3/s)	Runoff lag 'C'				Stream routing lag 'F'				Rainfall Loss Rates						Alternative Rain	
			C = 1.4		C = 1.8		F = 0.6		F = 0.8		ARR 1987 IL 10 mm CL 2.5 mm/hr		Original ARR 2019 IL 17 mm CL 5.5 mm/hr		FFA reconciled: Kadoona IL 35 mm CL 2.4 mm/hr		Lake Tabourie recorded rain	
2	Willinga Lake (level, mAHD)	1.69	1.69	(+0.00 m)	1.69	(+0.00 m)	1.69	(+0.00 m)	1.69	(+0.00 m)	1.79	(+0.10 m)	1.71	(+0.02 m)	1.77	(+0.08 m)	1.81	(+0.12 m)
17	Willinga Road Bridge	9.85	10.67	(+8%)	9.04	(-8%)	11.40	(+16%)	10.43	(+6%)	14.56	(+48%)	10.73	(+9%)	14.69	(+49%)	15.02	(+52%)
21	Tributary discharging to Willinga Lake north-west of Willinga Park	9.43	10.50	(+11%)	8.54	(-9%)	11.43	(+21%)	10.38	(+10%)	13.32	(+41%)	10.39	(+10%)	13.42	(+42%)	12.67	(+34%)
92	Tributary discharging to Willinga Lake north-east of Willinga Park	10.23	11.30	(+11%)	9.30	(-9%)	11.41	(+12%)	10.31	(+1%)	14.24	(+39%)	11.19	(+9%)	14.35	(+40%)	13.66	(+34%)
3	Bawley Point Road 400m south-east of Princes Hwy (BPR1)	13.63	14.91	(+9%)	12.52	(-8%)	15.46	(+13%)	14.06	(+3%)	18.97	(+39%)	14.86	(+9%)	19.11	(+40%)	18.15	(+33%)
8	Cormorant Beach outlet	0.49	0.51	(+3%)	0.48	(-3%)	0.51	(+4%)	0.50	(+2%)	1.11	(+125%)	0.53	(+8%)	0.93	(+88%)	1.65	(+234%)
9	Gannet Beach outlet	1.07	1.10	(+3%)	1.04	(-3%)	1.11	(+4%)	1.09	(+2%)	1.21	(+13%)	1.10	(+3%)	1.21	(+13%)	2.03	(+90%)
10	Murramarang Beach north	2.70	2.79	(+3%)	2.61	(-3%)	2.82	(+5%)	2.74	(+1%)	3.25	(+20%)	2.94	(+9%)	3.26	(+21%)	4.04	(+50%)
11	Murramarang Beach south	0.58	0.62	(+7%)	0.54	(-6%)	0.62	(+8%)	0.59	(+3%)	1.97	(+241%)	0.88	(+53%)	1.81	(+213%)	2.42	(+320%)
12	Racecourse Beach north	3.52	3.70	(+5%)	3.36	(-5%)	3.75	(+6%)	3.63	(+3%)	5.23	(+48%)	3.86	(+9%)	5.28	(+50%)	5.65	(+60%)
13	Racecourse Beach south	1.96	2.13	(+9%)	1.81	(-8%)	2.20	(+13%)	2.09	(+7%)	2.59	(+32%)	2.12	(+8%)	2.60	(+33%)	2.48	(+26%)
14	Butlers Creek outlet	8.57	9.33	(+9%)	7.92	(-8%)	9.78	(+14%)	9.00	(+5%)	13.44	(+57%)	9.83	(+15%)	13.50	(+58%)	14.79	(+73%)
15	Kioloa Beach outlet	4.91	5.31	(+8%)	4.49	(-9%)	5.64	(+15%)	5.21	(+6%)	6.33	(+29%)	5.27	(+7%)	6.37	(+30%)	6.43	(+31%)
16	Merry Beach outlet	5.21	5.63	(+8%)	4.84	(-7%)	5.81	(+12%)	5.45	(+5%)	7.15	(+37%)	5.64	(+8%)	7.20	(+38%)	7.42	(+42%)

7.4 Model Verification – February 2023 Event

7.4.1 Event Overview and Rainfall Data

Large parts of NSW experienced lines of storms on the 9 February 2023 which brought heavy rain and strong winds to parts of the state including the South Coast region.

The Willinga Lake gauge recorded 88 mm of rainfall over about 15 hours (*slightly lower than a 50% AEP storm*) including 42 mm over a 2-hour period (*slightly higher than a 50% AEP storm*). This resulted in a rise of about 0.6 metres at the Willinga Lake water level gauge but did not result in overtopping and breakout of the entrance berm.

A cumulative rainfall plot for 9 February 2023 is presented in **Figure 7-8** including available continuous rainfall gauge (pluviometer) data within and in closest proximity to the Willinga Lake catchment.

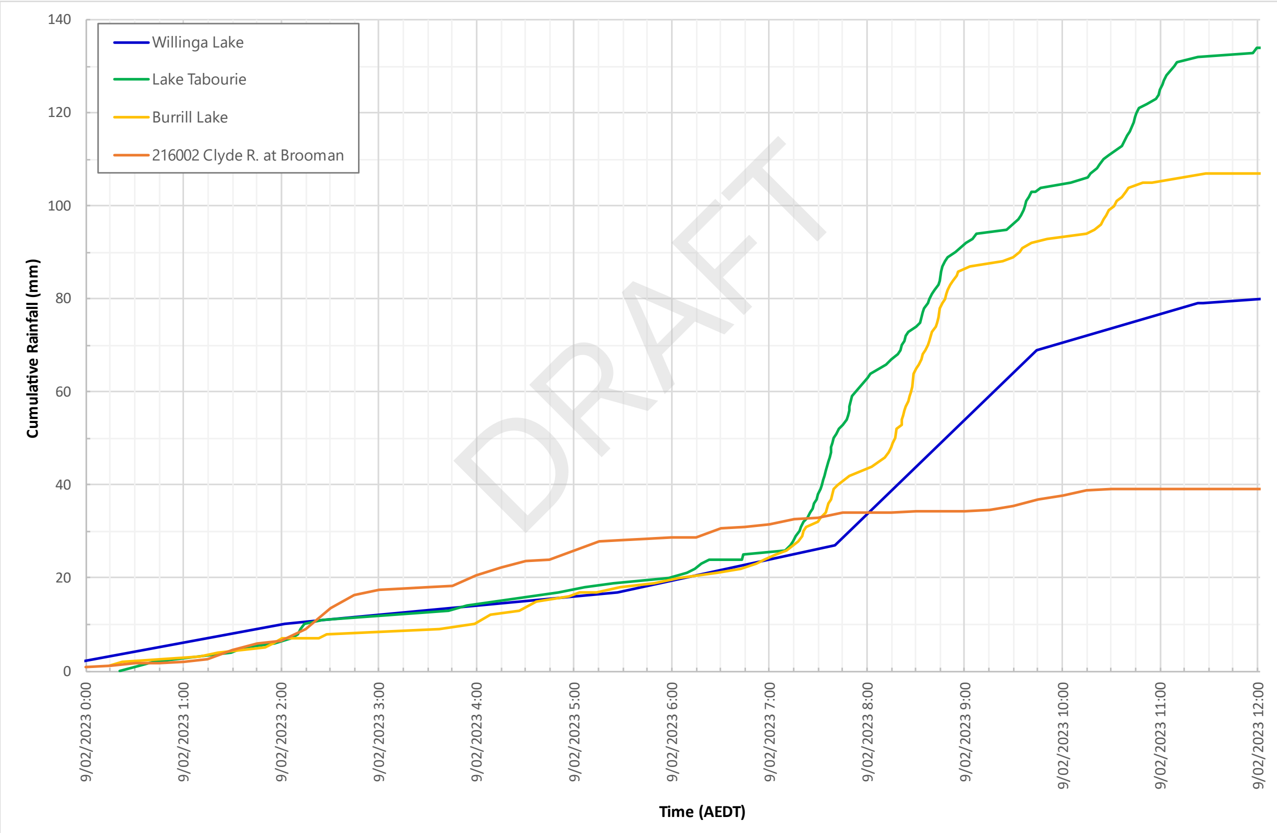


Figure 7-8 Cumulative rainfall plot for 9 February 2023

Oceanic conditions during this event were not of importance to the model verification as the entrance remained closed.

7.4.2 Hydraulic Model Verification

The hydraulic model verification was assessed through the comparison of flood levels simulated by the TUFLOW model with water levels recorded by the Willinga Lake gauge.

7.4.2.1 Calibration of Rainfall Loss Rates

As with the calibration event, a range of rainfall loss rates sourced from ARR 1987 and ARR 2019 were tested in the WBNM hydrologic model and adjusted until the stage hydrograph for Willinga Lake produced a result comparable to the recorded water level data. The identified loss rates were then tested in the TUFLOW hydraulic model to confirm the calibration result and appropriateness of the loss rates.

The above process resulted in an adopted IL of 50 mm and CL of 7.0 mm/hr for the February 2023 event. These loss rates are almost identical to those determined for the November 2023 event.

7.4.2.2 Model Representation of November 2023 Conditions

Relevant conditions during the November 2023 flood event were represented in the TUFLOW hydraulic model as follows:

- The geometry of the Willinga Lake entrance berm was represented using the 2011 LiDAR data with a minimum berm crest elevation of 1.96 mAHD, above the recorded peak flood level. Dynamic breakout of the Willinga Lake entrance berm was not required.
- An initial water level of 0.64 mAHD was applied in Willinga Lake per the gauge record.

7.4.2.3 Comparison of Recorded and Simulated Water Levels

The results of the February 2023 verification to recorded water level data is presented in **Figure 7-9**. The match in the timing, shape and peak of recorded and simulated flood level hydrographs is excellent throughout the event. The verification result provides additional confidence in the developed flood models.

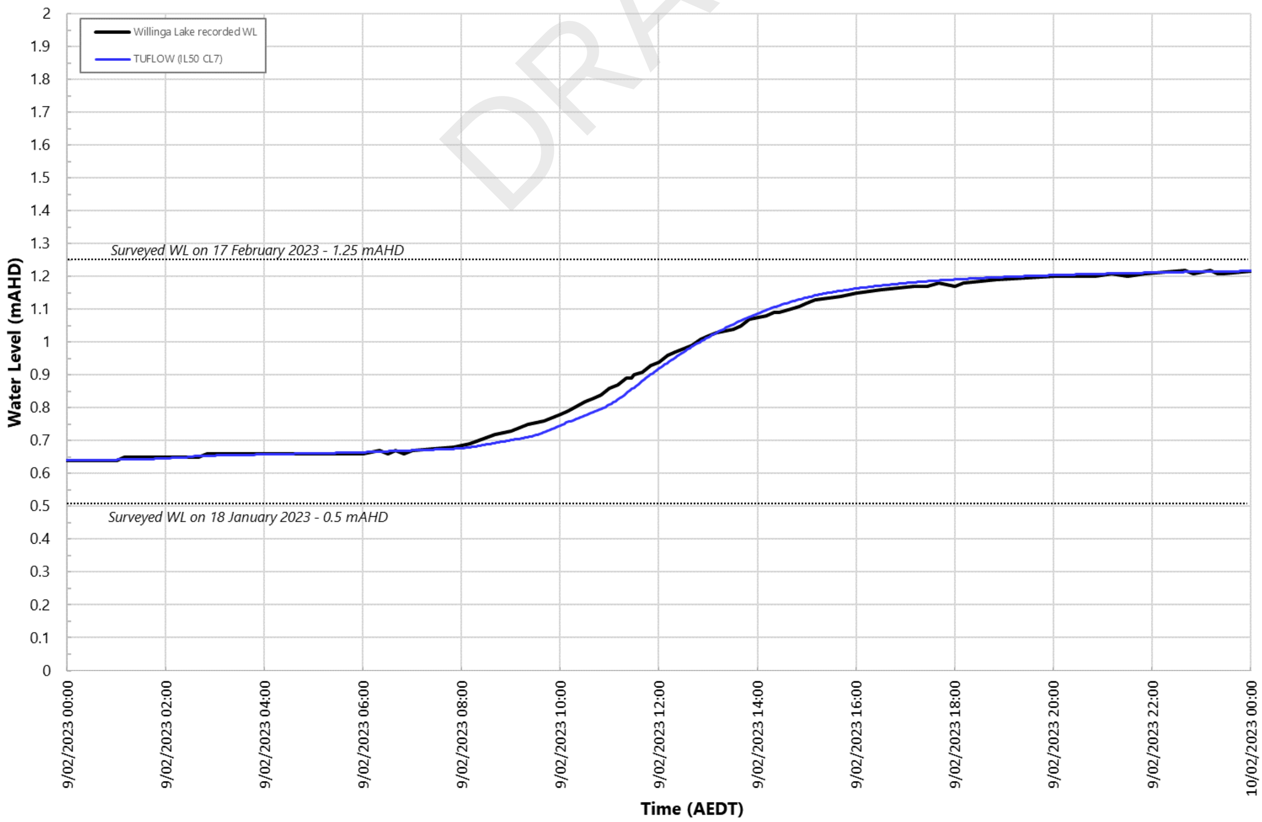


Figure 7-9 Comparison of recorded and simulated flood levels for the February 2023 event

7.5 Hydrologic Model Validation

Prior to undertaking ARR 2019 design flood modelling additional validation of the WBNM hydrologic model was undertaken to confirm its ability to reliably estimate peak flood flows across a range of storm durations and magnitudes.

The ARR 1987 based 5% AEP and 1% AEP 120 minute and 360 minute storms and the GSDM PMF 60 minute and 180 minute storms were simulated in the TUFLOW hydraulic model and hydrographs were extracted for comparison to the corresponding WBNM hydrologic model hydrographs (*as well as to inform stage-storage-discharge relationships in the model, refer* **Section 5.2.5**). The ARR 1987 design floods were used for this purpose rather than their ARR 2019 counterparts as this removed the complication of temporal pattern selection, and because likely critical storm durations for the study catchment were simpler to estimate without the influence of multiple temporal patterns.

The results are presented in **Figure 7-10**, **Figure 7-11**, **Figure 7-12** for the ARR 1987 5% AEP event, ARR 1987 1% AEP event, and PMF event respectively.

While there are minor differences evident, as would be expected, the agreement in hydrograph shape, timing and peak flow is generally excellent throughout the catchment. This includes results across a range of catchment sizes and types, as well as storm magnitudes and durations.

In some locations the WBNM hydrographs are less 'flashy' than those from TUFLOW. Some improvement was made by lowering the WBNM stream routing lag factor 'F' from 1.0 to 0.8 in cleared/developed areas, however lowering the F value further would have had unwarranted negative impacts in other locations.

Overall, the good agreement in the hydrograph shape and peak flows shows that the differing principals and numerical approaches behind each model are converging on a common result, thus providing additional confidence in the modelling process and ability of the hydrologic model to assess ARR 2019 critical storm durations and temporal patterns.

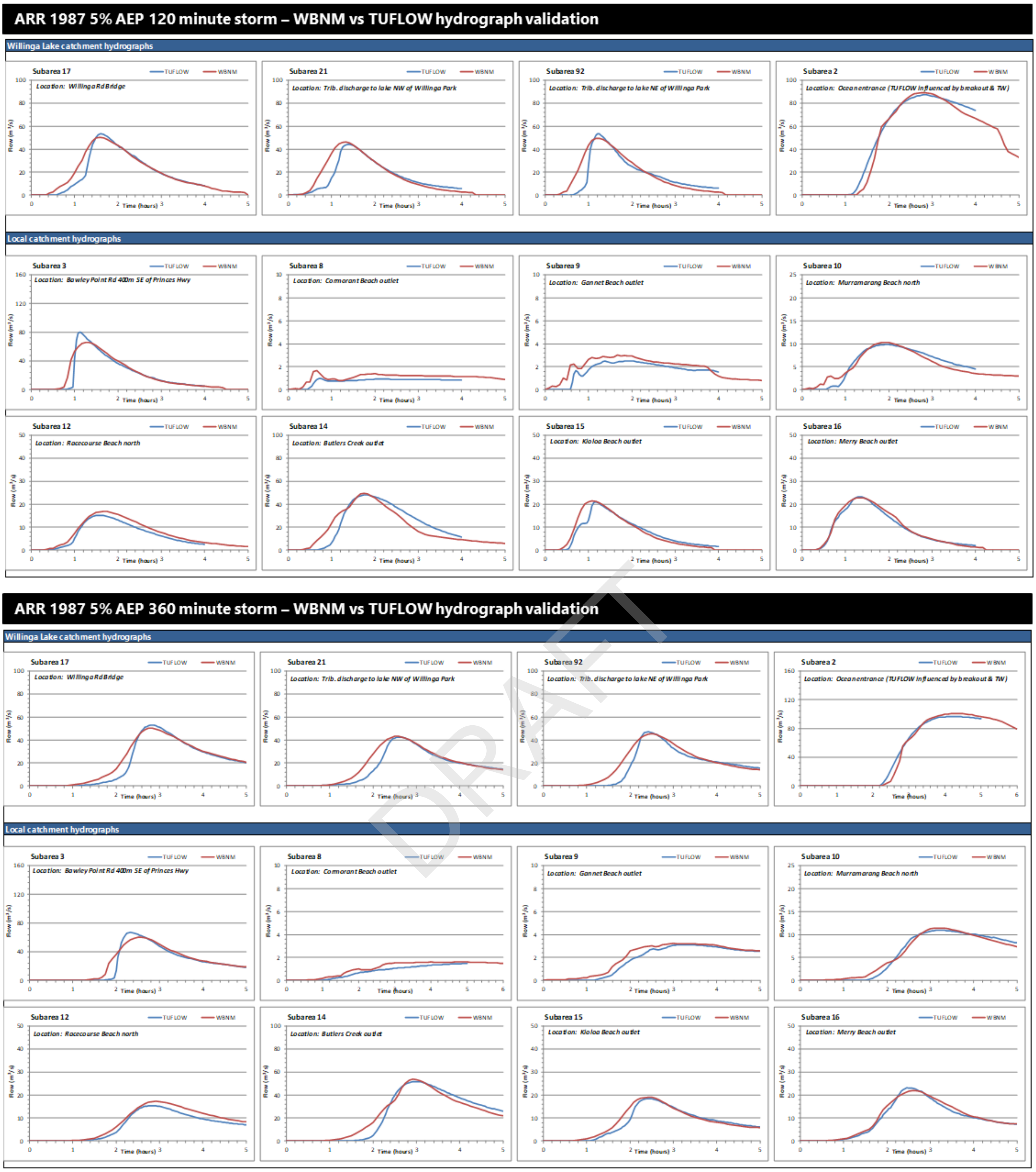
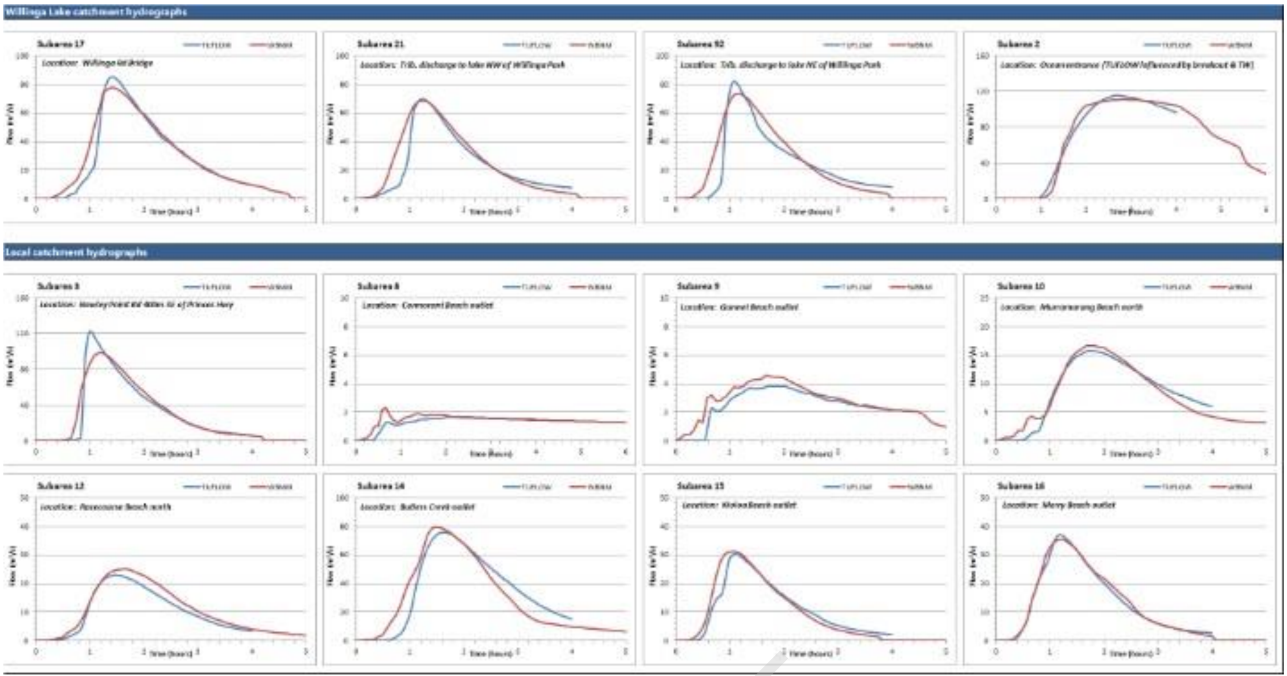


Figure 7-10 WBNM vs TUFLOW hydrograph validation for the ARR 1987 5% AEP event

ARR 1987 1% AEP 120 minute storm – WBNM vs TUFLOW hydrograph validation



ARR 1987 1% AEP 360 minute storm – WBNM vs TUFLOW hydrograph validation

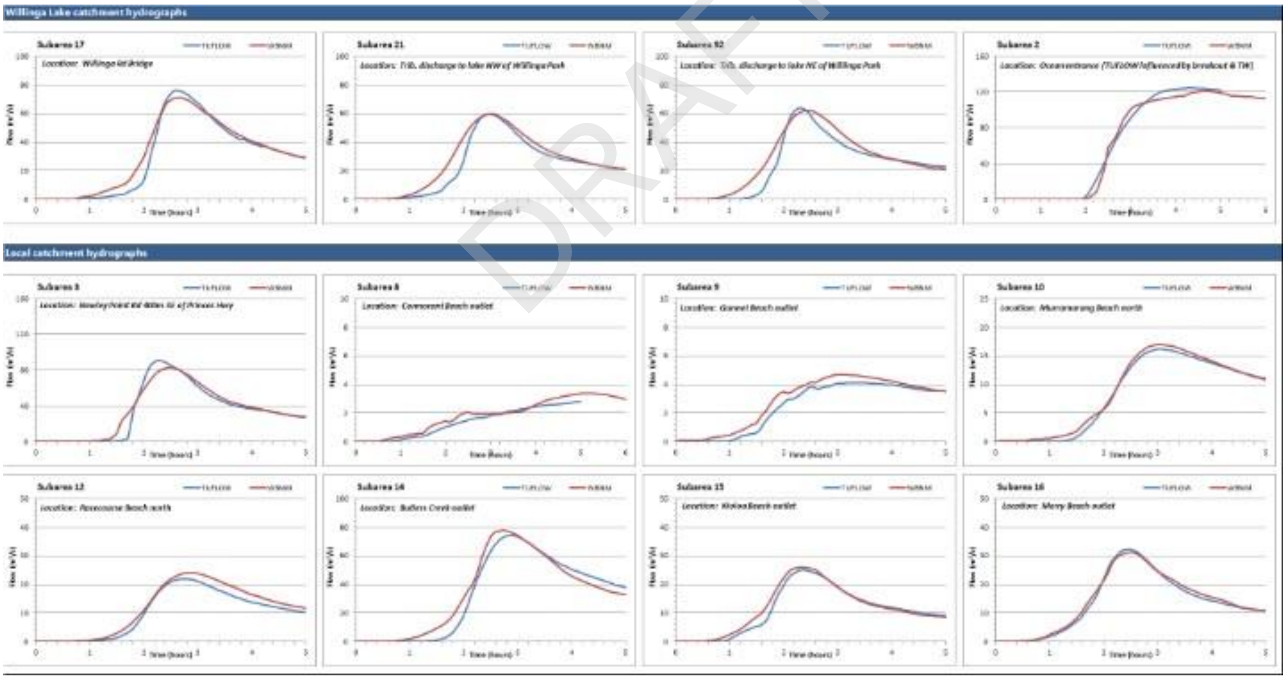
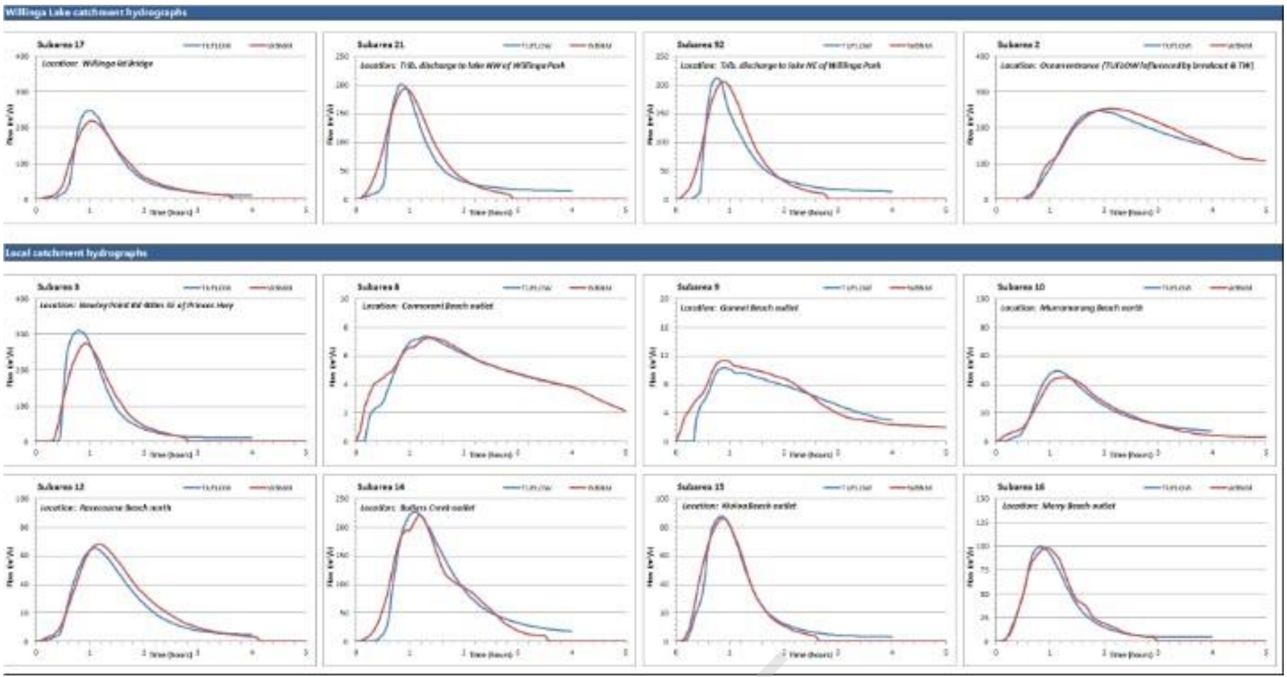


Figure 7-11 WBNM vs TUFLOW hydrograph validation for the ARR 1987 1% AEP event

GSDM PMF60 minute storm – WBNM vs TUFLOW hydrograph validation



GSDM PMF180 minute storm – WBNM vs TUFLOW hydrograph validation

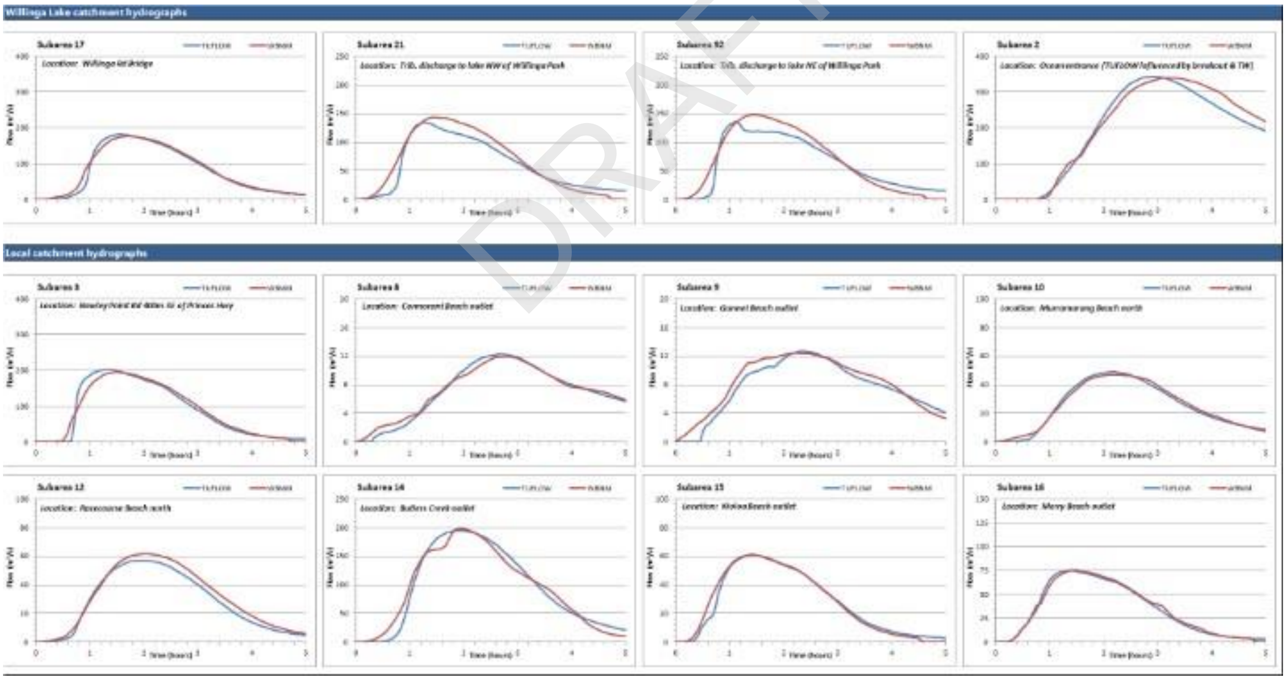


Figure 7-12 WBNM vs TUFLOW hydrograph validation for the PMF event

8 Design Flood Estimation

8.1 Introduction

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 guidance, data and techniques as provided 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) which provides a measure of the relative frequency and magnitude of the flood event. Flood conditions for the 50%, 20%, 10%, 5%, 2%, 1%, 1 in 200 and 1 in 500 AEP design events have been investigated in this study along with the Probable Maximum Flood (PMF).

8.2 Design Rainfall

8.2.1 Design Rainfall Depths

Design rainfall depths for the 50% AEP to 1 in 500 AEP design events were obtained online from the BoM Design Rainfall Data System (2016) (*accessed 20/04/2024*).

IFD data from several points across the study area were compared and showed minimal spatial variation in design rainfall depths. Accordingly, IFD from a single point at the centroid of the Willinga Lake catchment was adopted.

The Probable Maximum Precipitation (PMP), as used to determine the PMF, was calculated per the Generalised Short Duration Method (GSDM) defined by BoM (2003). The GSDM ellipse approach was used in deriving averaged PMP rainfall depths across the study area but was not applied as a spatially varying rainfall distribution as it is not considered representative of the study catchment.

An Areal Reduction Factor (ARF) of 1.0 was adopted for all simulations (i.e. no reduction was applied to design rainfall depths).

Adopted design rainfall depths for each design event are presented in **Table 8-1**.

8.2.2 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 in most catchments peak flows for a number of the patterns tend to cluster around the mean. 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.

8.2.3 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 loss rates for pervious surfaces adopted in this study are based on the 'Probability-Neutral Burst Initial Loss' values available from the ARR Data Hub. Continuing loss rates for pervious surfaces are based on ARR Data Hub values with a multiplication factor of 0.4 applied per NSW specific advice from DCCEEW. For the PMF, standard ARR 1987 based design loss rates for NSW east of the western slopes were adopted as the ARR Data Hub values were intended for use with ARR 2019 rainfall data and approaches.

Adopted initial loss (IL) and continuing loss (CL) values for pervious surfaces are presented in **Table 8-1**. No losses were applied to impervious surfaces.

8.2.4 Assessment of Critical Storm Duration and Temporal Pattern

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 generally 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 the FRMS&P, 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, storage behaviour etc.*), and therefore may have varying critical storm durations and applicable temporal rainfall patterns.

Given the run time of the developed TUFLOW two-dimensional hydraulic model, 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 several assessment locations of interest.
- 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 were selected for each flood magnitude that in combination provided the overall best match to 'average peak design flows' across the assessment locations.
- For the 5% AEP, 1% AEP and PMF events additional investigation of several storm durations of interest was undertaken using the TUFLOW hydraulic model to confirm the preferred combination of two storm durations. The findings of this analysis were then considered in the selection of storm durations for other events.

A summary of the selected critical storm durations and temporal patterns for each design event is presented in **Table 8-1**. Additional information on the assessment process is provided in **Appendix A**.

Table 8-1 Selected critical storm durations, design rainfall, temporal patterns and loss values

Design Flood Event	Critical Storm Duration (min)	Design Rainfall Depth (mm)	Temporal Pattern	Initial Loss (mm)	Continuing Loss (mm/hr)
50% AEP	180 360	43.1 61.7	6087 (TP4) 6152 (TP7	7.0 7.9	2.2
20% AEP	180 360	63.4 89.9	6094 (TP9) 6152 (TP7)	5.6 5.3	2.2
10% AEP	120 270	64.6 94.8	6043 (TP6) 6111 (TP5)	5.8 6.3	2.2
5% AEP	120 270	77.0 112.0	6043 (TP6) 6111 (TP5)	5.5 5.7	2.2
2% AEP	120 540	94.0 192.0	5864 (TP1) 6163 (TP5)	6.7 7.6	2.2
1% AEP	120 540	108.0 217.0	6036 (TP8) 6156 (TP4)	3.4 2.9	2.2
1 in 200 AEP	120 720	121.0 283.0	6036 (TP8) 6157 (TP4)	3.4 3.4	2.2
1 in 500 AEP	120 720	141.0 327.0	6036 (TP8) 6157 (TP4)	3.4 3.4	2.2
PMF	60 180	287.7 439.3	GSDM	10.0	2.5

8.3 Design Ocean Boundary Conditions

Flood levels in lower parts of the study area may be influenced by the coinciding ocean water level. Ocean water levels at the entrance to Willinga Lake and the other minor watercourses during a flood may consist of a combination of the following:

- Astronomical tide
- Tidal anomalies, most notably storm surge (*changes in ocean level driven by the combined effects of variations in air pressure and wind stress during storms*)
- Wave setup (*an increase in water level along the beach shoreline and at shallow waterway entrances associated with wave breaking*).

The latest advice regarding the selection of ocean boundary conditions for use in studies under the NSW Floodplain Management Program is provided in *Floodplain Risk Management Guide - Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways* (*OEH 2015*). The entrance to Willinga Lake intermittently becomes closed to the ocean by the build-up of a sand bar caused by the deposition of sand by various coastal processes. Accordingly, it is classified as an Intermittently Closed or Open Lake or Lagoon (ICOLL) and is treated as a 'Type C – Group 4' waterway entrance under these guidelines. This governs the relevant ocean water levels to be applied at the downstream model boundary for each design flood event.

For the purposes of this study dynamic, time-varying ocean water levels were applied. The timing of peak ocean water levels was adjusted to coincide approximately with peak catchment discharge from Willinga Lake for the ARR 2019 based 1% AEP 540 minute duration, temporal pattern 4 design storm.

Highest High Water Springs (Summer Solstice) (HHWS(SS)) is the highest purely tidal ocean water level (*i.e. not including anomalies or wave setup*) that occurs in any given year (*i.e. equivalent to annual highest astronomical tide*). The HHWS(SS) level was selected based on the Jervis Bay at HMAS Creswell gauge as reported in *OEH Tidal Planes Analysis 1990-2012 Harmonic Analysis* (*MHL 2012*). The Jervis Bay gauge was selected in favour of the Ulladulla Harbour and Batemans Bay Offshore gauges as it has a longer and more reliable data record.

Water levels applied at the downstream model boundary for each design flood event investigated are presented in **Table 8-2**. Tidal and oceanic inundation of Willinga Lake associated with each of the cited ocean levels was also investigated with an open entrance condition.

Table 8-2 Adopted design ocean boundary conditions for Type C estuary (ICOLLs)

Design Flood Event	Catchment Flood	Coinciding Ocean Level Event	Ocean Water Level (mAHD)
50% AEP	50% AEP	HHWS(SS) for Jervis Bay[1]	1.06
20% AEP	20% AEP	HHWS(SS) for Jervis Bay[1]	1.06
10% AEP	10% AEP	HHWS(SS) for Jervis Bay[1]	1.06
5% AEP	5% AEP	HHWS(SS) for Jervis Bay[1]	1.06
2% AEP	2% AEP	5% AEP ocean level for south of Crowdy Head[2]	2.35
1% AEP Flood Level Envelope	1% AEP	5% AEP ocean level, south of Crowdy Head[2]	2.35
	5% AEP	1% AEP ocean level, south of Crowdy Head[2]	2.55
1 in 200 AEP	1 in 200 AEP	1% AEP ocean level, south of Crowdy Head[2]	2.55
1 in 500 AEP	1 in 500 AEP	1% AEP ocean level, south of Crowdy Head[2]	2.55
PMF	PMF	1% AEP ocean level, south of Crowdy Head[2]	2.55

1. Source: *OEH Tidal* Planes Analysis 1990-2012 Harmonic Analysis (*MHL 2012*)
2. Source: *Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways* (*OEH 2015*)

8.4 Willinga Lake Entrance Conditions

Flood levels in Willinga Lake are influenced by the geometry of the entrance berm at the beginning of a flood and its morphology during a flood. Relevant conditions adopted for use in design simulations are summarised in **Table 8-3**. Further information on the approach and its development is provided in **Chapter 6**.

Table 8-3 Summary of Willinga Lake entrance conditions adopted for design simulations

Entrance Condition	Value	Source / Description
Minimum initial entrance berm RL	1.7 mAHD	Selected based on review of available data and the results of sensitivity testing *(refer* **Chapter 6***)*.
Initial Willinga Lake water level	1.6 mAHD	
Willinga Lake water level triggering entrance breakout	1.8 mAHD	Assumed that entrance breakout begins once the berm is overtopped by a water depth of 0.1 m, allowing time for appropriate velocities to develop and begin transporting sand.
Duration over which entrance breakout occurs	1.75 hours	Initially estimated based on the volume of sand to be scoured then refined through the model calibration process *(refer* **Section 6.2.2***)*.
Final entrance geometry	Variable	Based on 12 December 2023 survey with minor modifications from the 4 December 2023 and 4 January 2024 surveys.

9 Design Flood Results

9.1 General Description of Flood Behaviour

Willinga Lake

Willinga Lake is bordered by a considerable area of flat, low-lying land with elevations primarily between about 1.2 and 1.8 mAHD. On the eastern bank of the lake, to the south of Bawley Point Road (*i.e., Skylark Close and surrounds*), there is a slightly higher area of land with elevations in the range of 1.8 to 2.1 mAHD. Beyond this the land rises steeply out of the floodplain (*refer* **Figure 9-1**). The low point of Bawley Point Road in this area has an elevation of 1.92 mAHD.

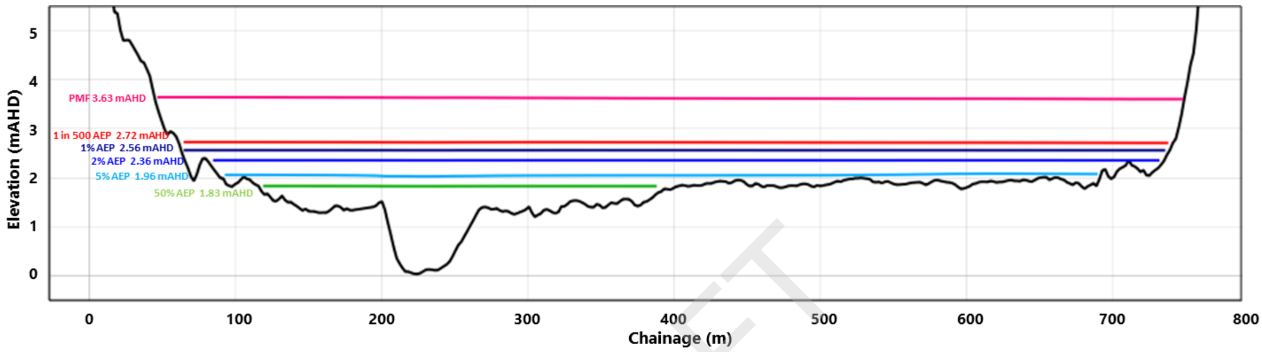


Figure 9-1 Terrain & peak flood level cross-section through the floodplain south of Bawley Point Rd

As a function of this terrain, a significant area of land is inundated even in a 50% AEP event with the flood extent continuing to increase rapidly up to the 5% AEP event (*refer* **Figure 9-2**). The extent and depth of inundation in these relatively frequent floods is strongly influenced by the condition of the Willinga Lake entrance berm which typically has a crest elevation in the range of 1.6 to 1.9 mAHD when closed (*a level of 1.7 mAHD was adopted for estimation of design flood conditions*).

The steeper terrain approaching the edge of the floodplain means that in flood events rarer than the 2% AEP the incremental increase in flood extent is quite small, even in the PMF. Flood levels rise in relatively small increments with increase flood magnitude before a more significant jump in flood levels in the PMF event.

Bawley Point Road would first be inundated by floodwaters from Willinga Lake in the 5% AEP event, though it would experience minor flooding in the 10% AEP event driven by runoff from the local catchment in the vicinity of Skylark Cl.

Peak flood velocities in the Willinga Lake floodplain are low across the full range of events, only exceeding 1 m/s within the entrance channel and locally elsewhere in the PMF.

Critical storm durations of 270-minutes to 720-minutes (*4.5 to 12 hours*) were found to drive peak flood conditions in the lake. This is typically indicative of moderate rates of rise (*e.g., refer* **Figure 9-5**) and moderate to long durations of flooding (*e.g., Bawley Point Road is inundated for 9 hours in the 1% AEP*).

Given the considerable area of land below 2.1 mAHD, the Willinga Lake floodplain and Bawley Point Road are also susceptible to inundation driven by elevated ocean conditions such as the 5% AEP (*2.35 mAHD*) and 1% AEP (*2.55 mAHD*) oceanic inundation events.

Along the watercourses and flowpaths draining to Willinga Lake a critical storm duration of 60 to 180-minutes was found, indicative of relatively 'flashy' catchments where flooding occurs in response to short durations of intense rainfall and flood levels quickly rise and fall over the course of a few hours. Velocities are higher, with peaks in the order of 1 m/s in the 1% AEP and 2 m/s in the PMF. Willinga Road adjacent to the bridge is first inundated in a 2% AEP event, and Forster Drive is first inundated in a 10% AEP event affecting access to Willinga Park.

Bawley Point Road Local Catchments

Local catchment flooding of Bawley Point Road is also relatively 'flashy', driven by short durations of intense rainfall. Model results indicate that a location along Bawley Point Road about 300 to 400 m south-east of the Princes Highway ('BPR-1') would be inundated in events as frequent as the 50% AEP, with the hazard and duration of inundation increasing with event magnitude.

Along with Willinga Lake driven flooding of Bawley Point Road near Skylark Close, hazardous flood conditions at this location would result in the most extensive, frequent, and longest duration isolation of local communities from the Princes Highway and associated services and emergency responders.

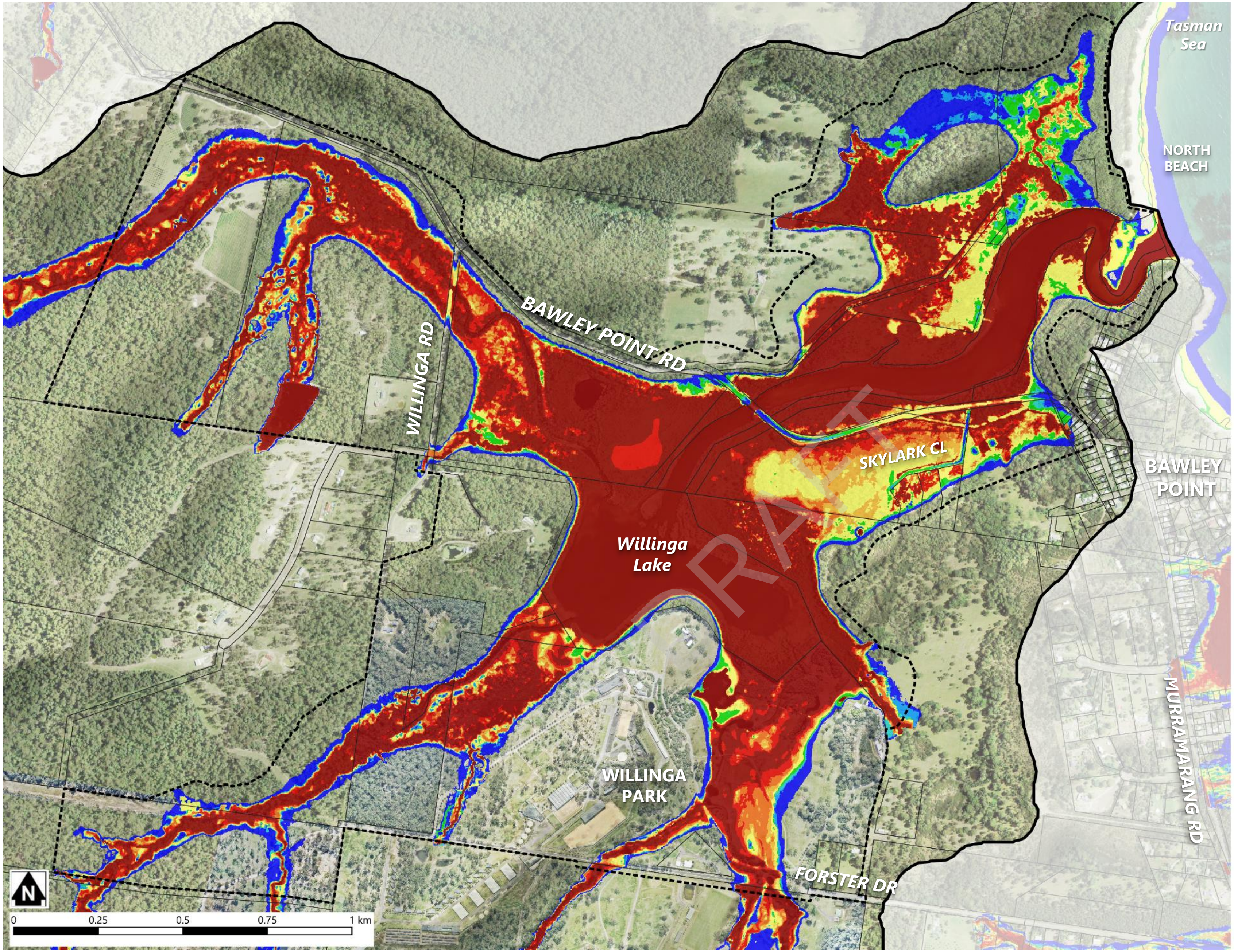
Murramarang Road Local Catchments

There are several local catchments which drain across Murramarang Road toward the ocean via minor creeks and lagoon-like watercourses which lie behind the sand dunes of the various beaches from Cormorant Beach in the north through to Merry Beach in the south.

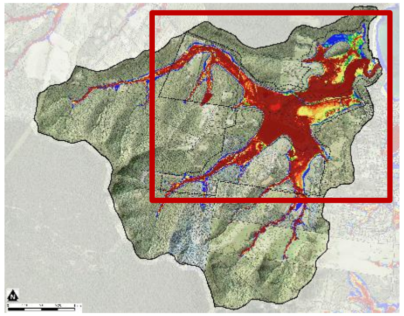
It was found that peak flood levels in the various lagoons, which are driven by longer duration storms, did not back up to cause flooding of Murramarang Road. Rather, inundation of Murramarang Road is generally driven by relatively 'flashy' shorter duration storms (*e.g., refer* **Figure A- 2**).

Inundation along Murramarang Road can be quite widespread and frequent, however flood depth, hazard and duration are generally lower than along Bawley Point Road. While this may result in the compartmented isolation of the various pockets of development and tourist facilities from one another, and from the main settlement of Bawley Point, inundation of Murramarang Road is unlikely to be critical in isolating these areas from the Princes Highway.

FIGURE 9-2



Map Extent



LEGEND

- Willinga Lake Catchment
- Willinga Lake Study Area
- Cadastre

Design Flood Extents

- 50% AEP
- 20% AEP
- 10% AEP
- 5% AEP
- 2% AEP
- 1% AEP
- 1 in 200 AEP
- 1 in 500 AEP
- PMF

Shoalhaven City Council

Prepared by: worley consulting

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SIMULATED DESIGN FLOOD EXTENTS FOR THE WILLINGA LAKE STUDY AREA

9.2 Design Flood Mapping

9.2.1 Existing Condition Design Flood Maps

Design flood mapping for 'existing conditions' within the Willinga Lake catchment is presented in **Volume 3**, **Figures 1-1 to 1-42**. No flood mapping has been produced for the Bawley Point Road and Murramarang Road local catchments. Information on road inundation in these catchments is presented in **Section 9.8**.

The existing condition design flood mapping presented in **Volume 3** includes the following.

- Peak flood depths and levels for nine design flood events from the 50% AEP to PMF.
- Peak flood depths and levels for the HHWS(SS), 5% AEP and 1% AEP oceanic inundation events.
- Peak flood velocities for the 1% AEP, 1 in 500 AEP and PMF design flood events.
- Flood Hazard per AIDR (2014) for nine design flood events from the 50% AEP to PMF.
- Flood Hazard per the FDM (2005) for the 1% AEP and PMF design flood events.
- Flood Function for the 1% AEP and PMF design flood events.
- Combined FDM Hazard and Flood Function for the 1% AEP and PMF design flood events.

Further information on the development of these flood maps is presented in the following.

9.2.2 Design Flood Envelopes

The flood mapping presented in **Volume 3** of this report comprises of peak flood conditions produced by a process known as 'flood enveloping'. For each design flood event and maximum flood model results from a number of storm durations and tailwater levels are combined to produce a 'design flood envelope'. The scenarios used to produce the peak design flood envelopes are summarised in **Table 9-1**.

Table 9-1 Summary of scenarios used to produce peak design flood envelopes

Design Flood Event	Catchment Flood	Storm Durations (min)	Ocean Tailwater
50% AEP	50% AEP	180 360	HHWS(SS)
20% AEP	20% AEP	180 360	HHWS(SS)
10% AEP	10% AEP	120 270	HHWS(SS)
5% AEP	5% AEP	120 270	HHWS(SS)
2% AEP	2% AEP	120 540	5% AEP ocean level
1% AEP	1% AEP	120 540	5% AEP ocean level
	5% AEP	270	1% AEP ocean level
1 in 200 AEP	1 in 200 AEP	120 720	1% AEP ocean level
1 in 500 AEP	1 in 500 AEP	120 720	1% AEP ocean level
PMF	PMF	60 180	1% AEP ocean level

9.2.3 Filtering of Model Results

In order to properly define flooding along minor tributaries and overland flowpaths, this study adopted a hydrologic method whereby local runoff hydrographs were applied to the TUFLOW hydraulic model at a fine spatial resolution. Consequently, model results required filtering to distinguish 'flooding' from areas of shallow catchment runoff and minor ponding.

The following filtering criteria were applied to the peak design flood envelopes prior to mapping.

- *Retain results where* 'Depth ≥ 0.15 m'
  - This criterion removes flood depths shallower than 0.15 m which are generally considered quite benign
- *Retain results where* 'Velocity x Depth ≥ 0.025 m^2/s'
  - This criterion was used to include depths shallower than 0.15 m where some flow conveyance is evident which may:
    - Occur near-bank along creeks and waterways
    - Form part of overland flood flow paths
    - Form important linkages between deeper areas of flooding and their source
  - Mapping of these shallower flows can be important as:
    - It provides a better understanding of flood behaviour
    - Obstruction of such flows may have adverse flood impacts
    - It provides confidence that discrete 'ponds' of inundation removed by the following mapping criterion are not associated with broader flood flowpaths
- *Retain results where* 'Area of inundation ≥ 100 m^2'
  - This criterion removes discrete areas of inundation with an area of less than 100 m^2. Such areas are generally expected to be related to local ponding in ground depressions rather than significant flood flowpaths.

9.3 AIDR Flood Hazard

Flood hazard provides a measure of the potential risk to life, well-being and property posed by a flood.

Figure 9-3 presents a set of curves which classify hazard based on the vulnerability of people, vehicles and buildings to various flood depth and velocity thresholds. The hazard curves were developed based on laboratory testing in 2014 (Smith et al.) and were subsequently published in *Australian Disaster Resilience Guideline 7-3 Flood Hazard* (AIDR 2017b) and in ARR 2019. As a result, these hazard classifications are known by various names. For the purposes of this study, they will be referred to as 'AIDR Flood Hazard'.

AIDR Flood Hazard maps for nine design flood events from the 50% AEP to PMF under existing catchment conditions are presented in **Volume 3 Figures 1-28 to 1-36**.

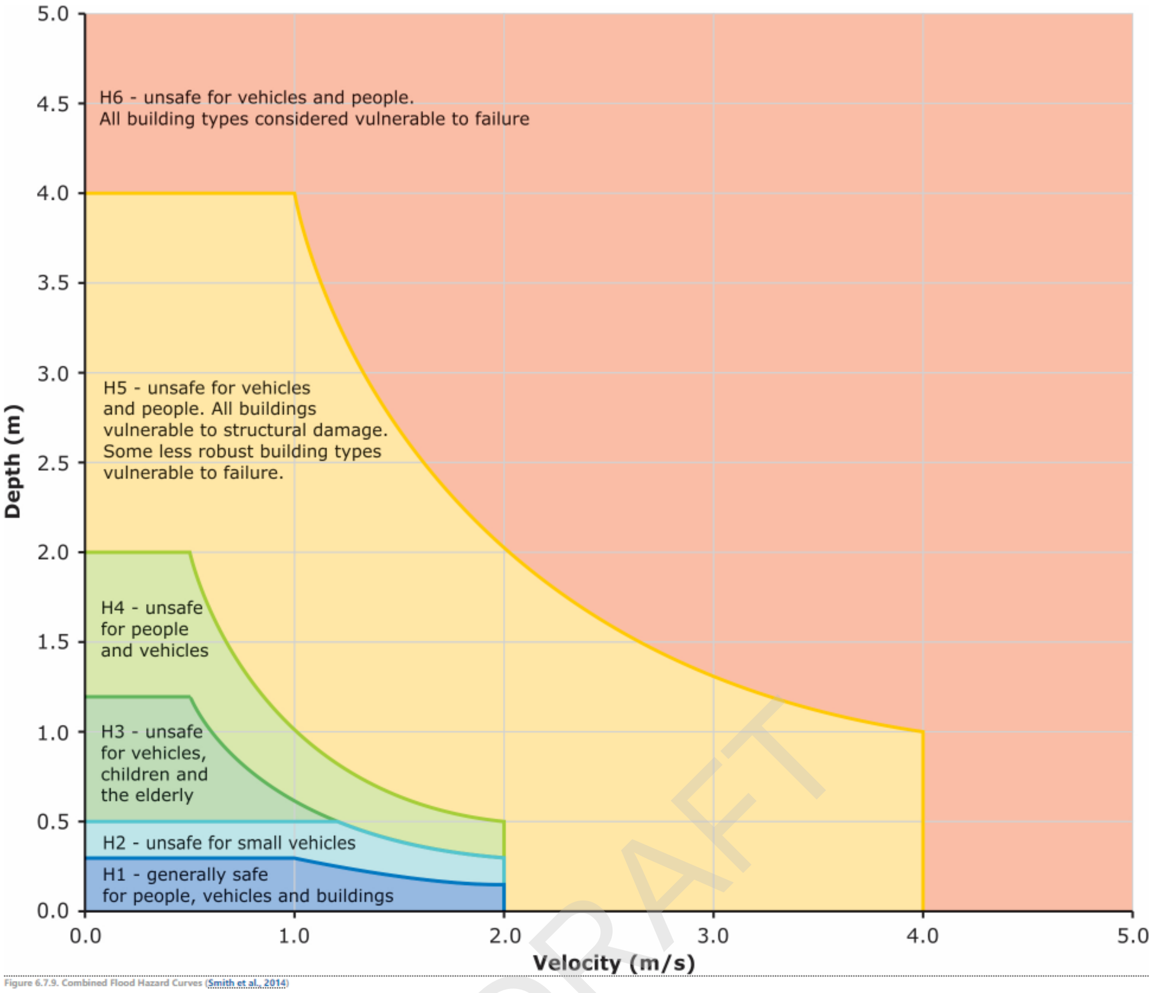


Figure 9-3 Flood Hazard Vulnerability Curves

9.4 FDM Flood Hazard

The *NSW Floodplain Development Manual* (2005) defines an alternative set of flood hazard as follows.

- High hazard – possible danger to personal safety; evacuation by trucks difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings.
- Low hazard – should it be necessary, trucks could evacuate people and their possessions; able-bodied adults would have little difficulty in wading to safety.

The FDM flood hazard categories are determined based on relationships between simulated flood depths and velocities defined in Figures L1 and L2 of the manual as reproduced in **Figure 9-4**. The 'transition zone' between high and low hazard is typically assigned to the high hazard category.

FDM Flood Hazard maps for the 1% AEP and PMF under existing catchment conditions are presented in **Volume 3 Figures 1-37 and 1-38**.

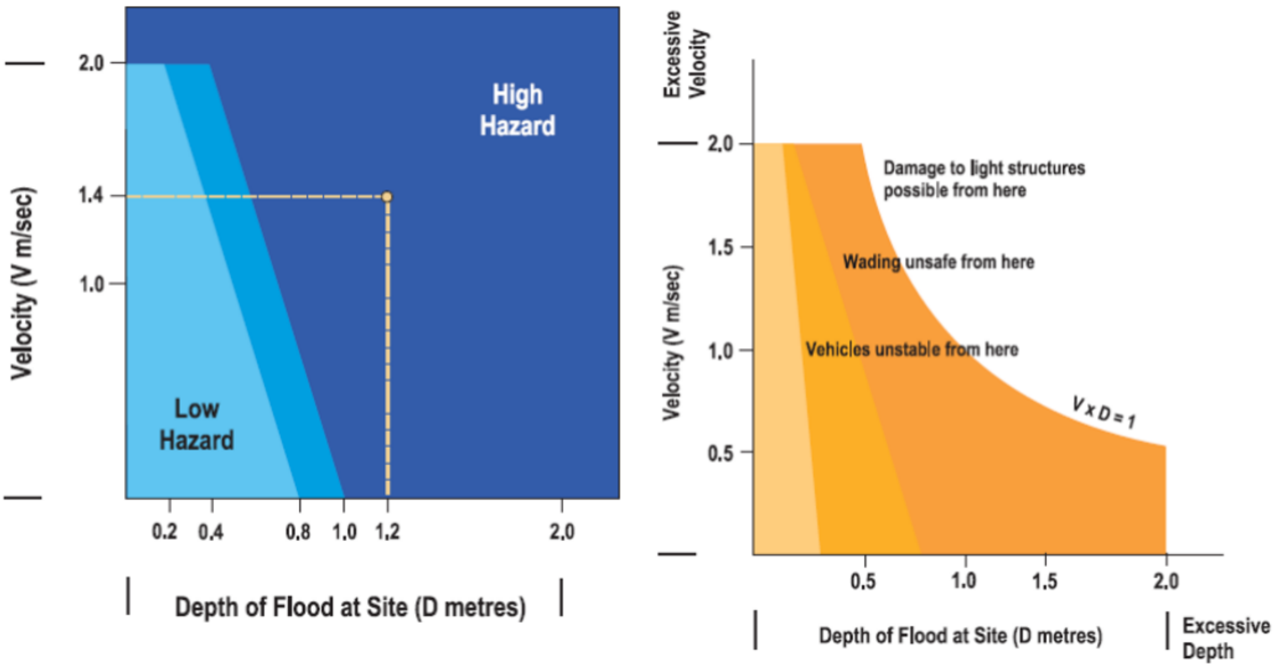


Figure 9-4 Velocity-depth relationships for FDM (2005) Hazard Categories

9.5 Flood Function

The delineation of the floodplain into 'flood function' or 'hydraulic categories' based on its function during floods is used as a tool to help inform what impact development activity within different areas of the floodplain may be expected to have on flood behaviour.

The *Flood Risk Management Manual* (DCCEEW 2023a) defines three hydraulic categories based on flood functions as described below.

- **Floodway areas** are those areas of the floodplain which convey a significant proportion of flows 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 areas of the floodplain outside of floodways 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 outside of floodway and flood storage areas. Development in flood fringe areas would generally not have any significant effect on the pattern of flood flows and/or flood levels.

There is no single specific procedure or uniform criteria for defining the extent of the floodway that can be applied across the wide range of floodplain types and flood behaviour encountered. However, *Flood Risk Management Guide FB02 Flood Function* (DPE 2023c) describes three types of approaches currently used by practitioners to define floodways in NSW, including 'indicator', 'encroachment' and 'conveyance' techniques.

Typically, the floodway is defined for the Defined Flood Event (DFE), in this case the 1% AEP design event, but may also take into consideration the variation in flood behaviour across a range of flood magnitudes. For the purposes of this study, the floodway extent has been defined using a combination of 'conveyance' and 'indicator' techniques as described in **Table 9-2** and derived in consideration of procedures outlined by

Thomas and Golaszewski (2012 and 2018) and *Flood Risk Management Guide FB02 Flood Function* (DPE 2023c). Resulting flood function mapping for the 1% AEP and PMF events is presented **Volume 3 Figures 1-39 and 1-40**.

Table 9-2 Criteria and Methodology for Definition of Flood Function

Flood Function	Description of Criteria and Methodology 1% AEP	PMF
Floodway	▪ The potential floodway corridor was reviewed using the waterRIDE software to examine the extent of the floodplain that conveys 80% of the peak flood flow during the 1% AEP design event. This provided an indication of relevant velocity-depth product (VxD) thresholds to derive the floodway (*generally 0.2 to 0.4 m^2/s*), however was complicated by low velocities in the vicinity of the lake, and multiple narrow braided channels upstream of Willinga Road bridge. ▪ A VxD threshold of 0.25 m^2/s was found to provide reasonable continuity along tributaries and flow paths and was adopted as the primary indicator for the 1% AEP floodway. Some minor improvement in floodway continuity were then made manually. ▪ The tidal extent of the estuary as defined by the HHWS(SS) level was also defined as floodway. ▪ The floodway was further refined by removing small, isolated areas of floodway less than 400 m^2 and converting 'holes' (*i.e. small areas of flood storage and flood fringe enclosed by floodway*) of less than 400 m^2 to floodway.	▪ The 1% AEP floodway extent plus those areas with velocity-depth product ≥ 0.6 m^2/s in the PMF. ▪ Engineering judgement was used in the removal of small, isolated areas of less than 400 m^2 and filling 'holes' of less than 400 m^2.
Flood Storage	▪ Non-floodway areas with a peak 1% AEP flood depth of 0.5 m or greater were defined as Flood Storage. ▪ Flood storage areas were further refined by removing small, isolated areas of flood storage less than 400 m^2 and converting 'holes' (*i.e. small areas of flood fringe enclosed by flood storage*) of less than 400 m^2 to flood storage.	▪ The 1% AEP flood storage extent plus non-floodway areas with a peak PMF flood depth of 0.75 m or greater.
Flood Fringe	▪ Remaining areas of the floodplain not classified as floodway or flood storage.	

9.6 Combined FDM Flood Hazard and Flood Function

Volume 3 Figures 1-41 and 1-42 present the 1% AEP and PMF flood extents classified based on the combination of FDM Flood Hazard and Flood Function. This results in the following six categories which are referenced in the Shoalhaven Development Control Plan (DCP 2014) to direct the application of flood-related development controls on flood prone land.

- Low Hazard Flood Fringe
- Low Hazard Flood Storage
- Low Hazard Floodway
- High Hazard Flood Fringe
- High Hazard Flood Storage
- High Hazard Floodway.

9.7 Peak Design Flood Level Profiles

Figure 9-7 presents peak flood level profiles from upstream of Willinga Road bridge, through Willinga Lake, and on to the Tasman Sea. The following is evident from the profiles.

- Peak flood levels are relatively flat from the entrance berm through to the Bawley Point Road bridge.
- The Bawley Point Road bridge and road embankment act as a slight hydraulic control, with peak flood levels 50 mm or more higher on the upstream side in most design flood events.
- Flood levels are, again, relatively flat through the body of Willinga Lake.
- The 50% AEP, 2% AEP and 1% AEP profiles are particularly flat from the entrance, beyond the bridge and through the body of Willinga Lake.
  - In the case of the 50% AEP event, peak flood levels are heavily influenced by the initial entrance berm elevation. As flood flows are very low relative to the storage volume and discharge capacity of the lake, flood levels reach only marginally above the level at which entrance breakout commences and begin dropping thereafter.
  - In the case of the 2% AEP and 1% AEP events, peak flood levels are heavily influenced by the coinciding elevated ocean levels applied for these events. The degree of influence is such that levels in the lake are essentially equivalent to the 5% AEP and 1% AEP ocean levels, respectively.
- Entering the unnamed watercourse there is a distinct increase in the flood surface gradient.
- The Willinga Road bridge and embankment act as a significant hydraulic control, with peak flood levels 150 mm or more higher on the upstream side in most design flood events.
- PMF peak flood levels are in the order of 1 metre higher than the 1% AEP across the length of the profile.

9.8 Tabulated Hydraulic Model Results and Road Inundation

Peak design flood levels, depths, velocities, AIDR Hazard and duration of inundation from the TUFLOW hydraulic model are presented in the following tables. This includes key locations within Willinga Lake, and key road overtopping locations in the Willinga Lake catchment, Bawley Point Road local catchments, and Murramarang Road local catchments.

- **Table 9-3: 50%, 20% and 10% AEP events**
- **Table 9-4: 5%, 2% and 1% AEP events**
- **Table 9-5: 1 in 200 AEP, 1 in 500 AEP and PMF events.**

Bawley Point Road and Murramarang Road provide the only reliable vehicular access routes from the Princes Highway to Bawley Point, Kioloa and Merry Beach. Inundation of these roads has the potential to isolate the local communities, businesses, and tourist facilities. A summary of road inundation findings is presented in the following sections. Additional detail is provided in the **Volume 2: Floodplain Risk Management Study & Plan** report.

All references to duration of inundation are based on a depth threshold of 50 mm and are rounded to the nearest 15 minutes.

9.8.1 Road Inundation in the Willinga Lake Catchment

The key road inundation location in the Willinga Lake Catchment affecting access to Bawley Point and areas further south is the low point along Bawley Point Road about 70 metres to the east of Skylark Close. Detailed survey in this location indicates a road crest elevation of 1.92 mAHD.

Findings regarding flooding of this location are summarised in the following.

- Bawley Point Road is flood free (*less than 20 mm depth*) in events up to and including the 20% AEP.
- In the 10% AEP event the road is overtopped to a peak depth of about 0.07 m and peak velocity of about 0.4 m/s, resulting in H1 hazard (*generally benign conditions*). The duration of inundation is about 1.5 hours. Peak flood depths in this event are driven by local catchment flooding passing from south to north across the road and on into Willinga Lake.
- In the 2% AEP event the depth of overtopping jumps to a peak of about 0.45 m resulting in H2 hazard (*unsafe for small vehicles*). The duration of inundation also increases to almost 10 hours. These significant increases are largely related to the elevated 5% AEP ocean tailwater applied for this event.
- In the 1% AEP event the depth of overtopping increases to about 0.66 m resulting in H3 hazard (*unsafe for all vehicles, children and the elderly*). The duration of inundation remains around 10 hours.
- The depth and duration of inundation increase to 0.78 m and 10.5 hours in the 1 in 500 AEP.
- In the PMF the depth, hazard and duration of inundation increase more markedly to 1.65 m, H4 (*unsafe for all vehicles and people*) and 10.75 hours.

Elsewhere in the catchment Willinga Road adjacent to the bridge is first inundated in a 2% AEP event, and Forster Drive is first inundated in a 10% AEP event affecting access to Willinga Park.

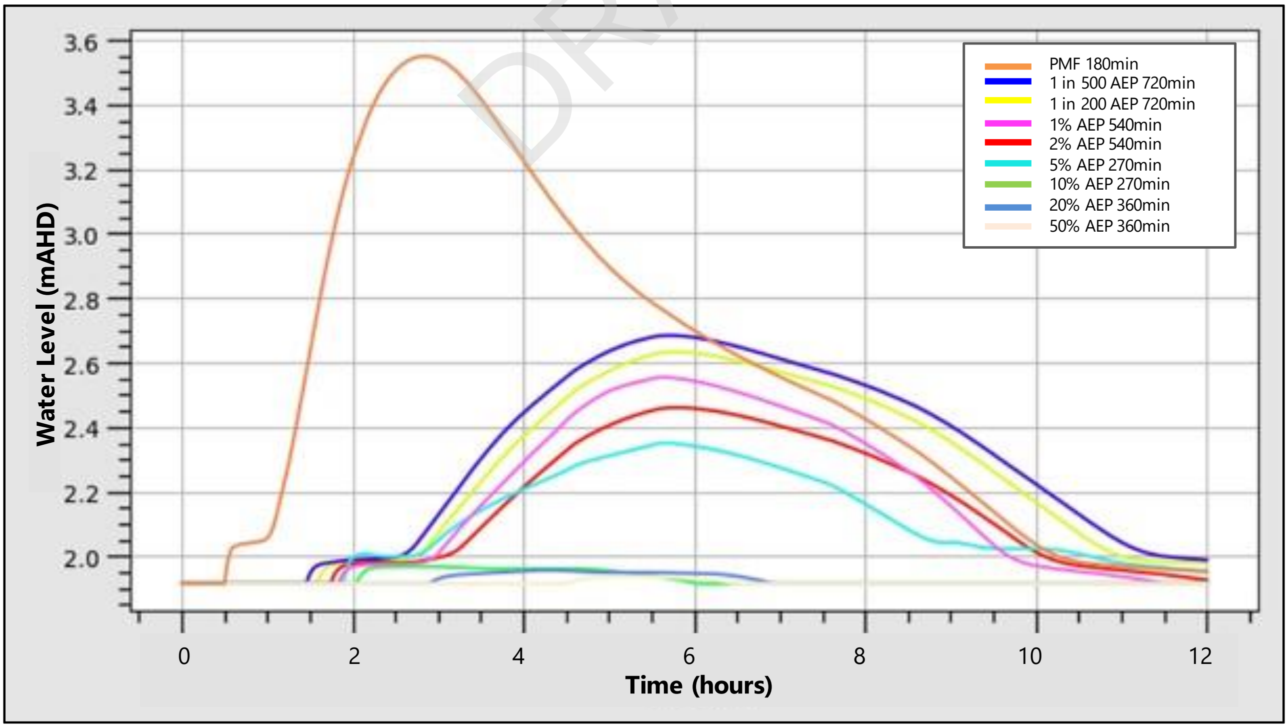


Figure 9-5 TUFLOW water level hydrographs at the low point of Bawley Point Rd near Skylark Cl

9.8.2 Road Inundation in the Bawley Point Road Local Catchments

Anecdotal information from the community identified a location along Bawley Point Road about 300 to 400 m south-east of the Princes Highway ('BPR-1') as the primary concern with regard to road inundation in the study area. Flood modelling results largely confirm this, with the findings as follows.

- Bawley Point Road is inundated in events as frequent as the 50% AEP, with peak depths of about 0.18 m and peak velocities of about 0.6 m/s, resulting in H1 hazard (*generally benign conditions*). The duration of inundation is about 2.25 hours.
- In the 20% AEP event the hazard increases to H2 (*unsafe for small vehicles*) and remains H2 up to and including the 5% AEP event. Peak depths in these events range from 0.3 m to 0.4 m and peak velocities in the range of about 1.2 to 1.7 m/s. The duration of inundation is up to 4.75 hours.
- H4 hazard conditions (*unsafe for all vehicles and people*) are reached in the 2% AEP event. The duration of inundation is about 6 hours, and is as high as 10.75 hours in the 1 in 500 AEP event.

Model results indicate that the BPR-1 location would be inundated more frequently than Bawley Point Road adjacent to Willinga Lake, would more frequently present flood conditions that are hazardous even to large vehicles (*i.e., ≥H3 hazard*), and in rare to very rare flood events would even experience durations of inundation similar to those at Willinga Lake. Accordingly, under existing catchment and climatic conditions, BPR-1 is the most critical location in the study area with regard to flood isolation.

Elsewhere in the Bawley Point Road local catchments, the road is inundated at a low point 170 m west of Tallawalla Way (BPR-2). This location is first inundated in a 20% AEP event for a duration of about 1 hour, with peak depths of about 0.16 m and peak velocities of about 0.5 m/s, resulting in H1 hazard (*generally benign conditions*). Hazard at this location remains low (H1) until the 1 in 200 AEP event where H2 hazard occurs (*unsafe for small vehicles*) and the duration of inundation is almost 4 hours. H4 hazard (*unsafe for all vehicles and people*) occurs in the PMF.

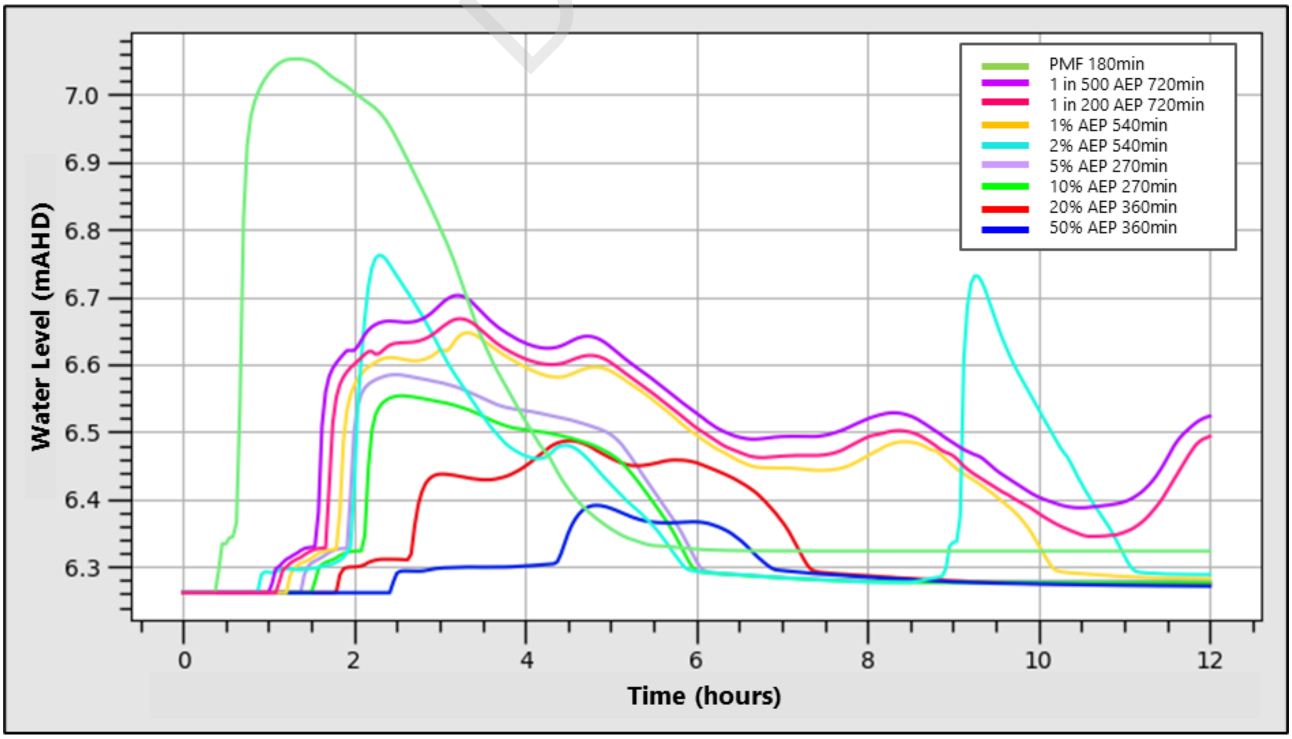


Figure 9-6 TUFLOW water level hydrographs at Bawley Point Rd near Princes Highway (BPR-1)

9.8.3 Road Inundation in the Murramarang Road Local Catchments

There are several local catchments which drain across Murramarang Road toward the Tasman Sea via minor creeks and lagoon-like watercourses which lie behind the sand dunes of the various beaches from Cormorant Beach in the north through to Merry Beach in the south. Prior to this assessment, little was known about road inundation in these areas.

There are numerous potential locations of inundation that would affect road access between developed areas and campgrounds in Pretty Beach, Merry Beach, Kioloa, Racecourse Beach and Gannet Beach through to Bawley Point. To analyse isolation, only the most critical location between each developed area requires assessment.

An overview of Murramarang Road inundation and isolation is presented in the following. A more detailed analysis will be undertaken in the subsequent **Volume 2** Floodplain Risk Management Study & Plan report.

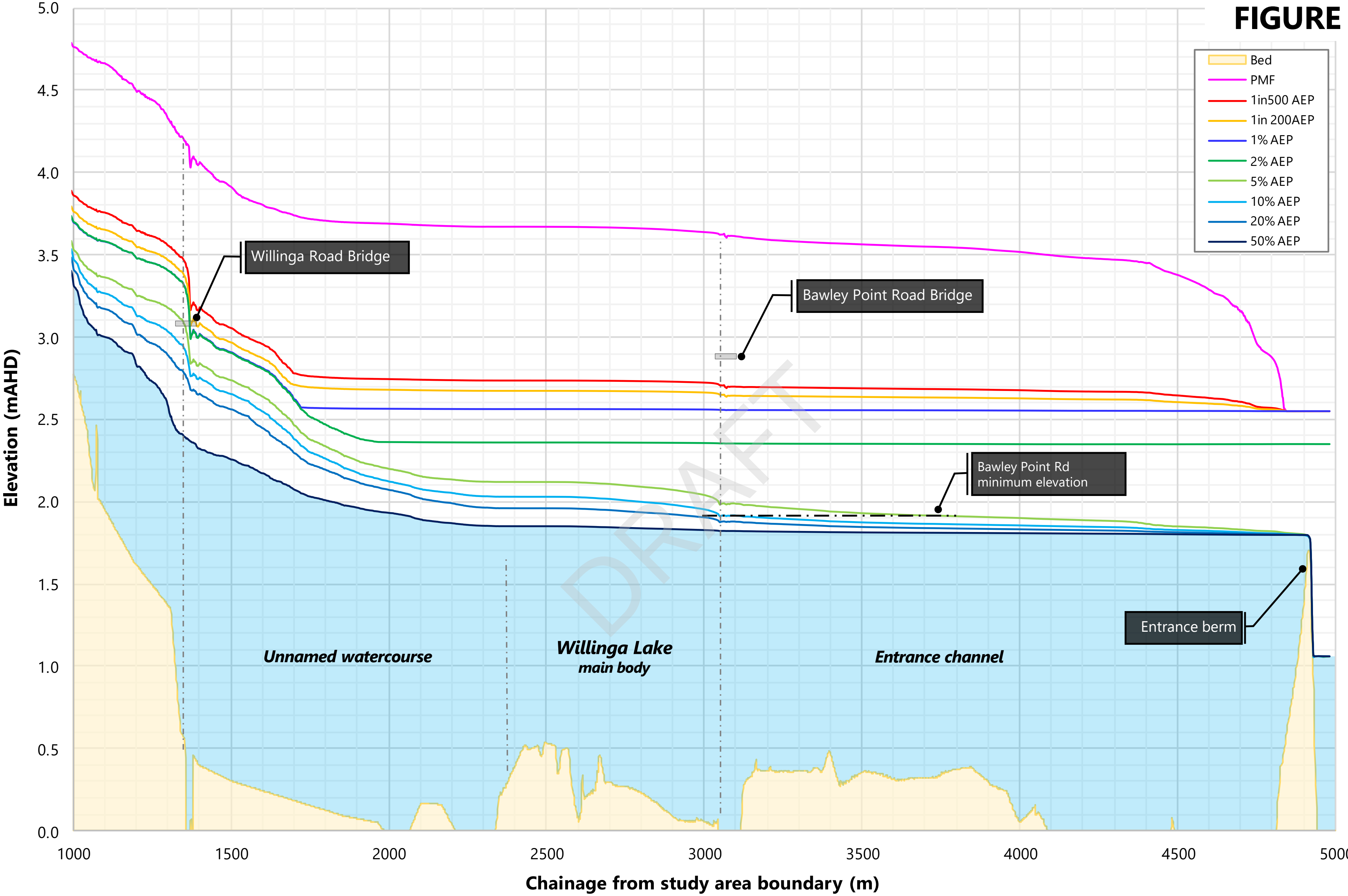
- There are a number of frequently inundated locations in the south of the study area that would isolate the Pretty Beach and Merry Beach areas in a 50% AEP event.
- A location immediately north of Bundle Hill Road would be inundated in a 20% AEP isolating the Racecourse Beach area.
- Additional locations would become inundated in the 10% AEP causing further isolation issues.
- The hazard of road inundation at locations north of Kioloa remains H1 (*generally benign*) in events up to and including the 2% AEP event meaning that, while not recommended, large vehicles could pass through floodwaters if required.
- H2 hazard (*unsafe for small vehicles*) first occurs near the Butlers Creek bridge in the 1% AEP event, and H3 hazard (*unsafe for all vehicles, children and the elderly*) at the same location in the 1 in 500 AEP event.
- Durations of inundation are as short as 1 hour at some locations, and as long as about 10 hours at others depending on the flood magnitude.

9.8.4 Summary of Road Inundation and Isolation

Flood modelling has confirmed that the critical road inundation locations that would result in isolation of the local communities, businesses and tourist facilities of the study area are Bawley Point Road about 400 metres south-east of the Princes Highway (BPR-1) and Bawley Point Road adjacent to Willinga Lake near Skylark Close. Hazardous flood conditions at these locations would result in the most extensive, frequent, and longest duration isolation from the Princes Highway and associated services and emergency responders.

Inundation along Murramarang Road can be quite widespread and frequent, however flood depth, hazard and duration are generally lower than along Bawley Point Road. While this may result in the compartmented isolation of the various pockets of development from one another, and from the main settlement of Bawley Point, inundation of Murramarang Road is unlikely to be critical in isolating these areas from the Princes Highway.

FIGURE 9-7



DESIGN PEAK FLOOD LEVEL PROFILES FROM WILLINGA ROAD BRIDGE THROUGH TO THE TASMAN SEA

Table 9-3 Tabulated 50%, 20% and 10% AEP hydraulic model results for the Willinga Lake catchment, Bawley Point Road and Murramarang Road

ID	Location	TUFLOW DEM (mAHD)	50% AEP					20% AEP					10% AEP				
			Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)	Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)	Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)
	Willinga Lake Catchment																
	Willinga Lake WL gauge	-0.62	1.82	2.44	0.6	H5	12:00	1.88	2.49	0.7	H5	12:00	1.91	2.53	0.8	H5	12:00
	Willinga Lake main body	0.31	1.85	1.54	0.1	H4	12:00	1.96	1.65	0.1	H4	12:00	2.03	1.72	0.1	H4	12:00
	Bawley Point Rd, 70m east of Skylark Close	1.90	-	-	-	-	-	-	-	-	-	-	1.97	0.07	0.4	H1	1:30
	Willinga Road Bridge	3.11	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	Forster Drive, 35m west of Willinga Park entrance gate	5.42	-	-	-	-	-	-	-	-	-	-	5.54	0.12	0.6	H1	0:30
	Forster Drive, 500m west of Willinga Park entrance gate	7.86	-	-	-	-	-	-	-	-	-	-	7.94	0.08	0.4	H1	0:15
	Bawley Point Road Local Catchments																
BPR-1	Bawley Point Road, 350m south-east of Princes Hwy	6.27	6.44	0.18	0.6	H1	2:15	6.57	0.30	0.9	H2	4:45	6.61	0.34	1.1	H2	4:00
BPR-1	Bawley Point Road, 400m south-east of Princes Hwy	6.15	6.24	0.09	0.6	H1	1:45	6.32	0.17	1.2	H1	4:15	6.36	0.21	1.5	H1	3:30
BPR-2	Bawley Point Road, 170m west of Tallawalla Way	3.30	-	-	-	-	-	3.46	0.16	0.5	H1	1:00	3.50	0.20	0.6	H1	1:30
	Murramarang Road Local Catchments																
MR-1	Murramarang Road, 40m south of Binnowee Place	8.71	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-1	Murramarang Road, Near Weemala Cres	5.38	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-2	Murramarang Road, Near Wonnawong place	10.93	-	-	-	-	-	-	-	-	-	-	11.06	0.13	0.2	H1	2:15
MR-2	Murramarang Road, 90m north of Malibu Dr	9.11	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-3	Murramarang Road, 60m south of Forster Dr	4.17	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-3	Murramarang Road, 90m north of Voyager Cres	3.79	-	-	-	-	-	-	-	-	-	-	3.83	0.05	0.6	H1	0:30
MR-4	Murramarang Road, Upstream of Limpid Lagoon	5.96	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-5	Murramarang Road, 160m north of Bundle Hill Rd	4.96	-	-	-	-	-	-	-	-	-	-	5.03	0.07	0.7	H1	0:15
MR-5	Murramarang Road, Near Bundle Hill Rd	6.53	-	-	-	-	-	6.59	0.06	0.5	H1	1:00	6.59	0.06	0.5	H1	2:00
MR-5	Murramarang Road, Adjacent to Tasman Holiday Park	6.08	-	-	-	-	-	-	-	-	-	-	6.13	0.05	0.4	H1	0:45
MR-6	Murramarang Rd, Butlers Ck trib, 130m nth of Moores Rd	3.95	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-6	Murramarang Rd, Butlers Ck trib, 120m sth of Moores Rd	3.13	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-7	Murramarang Road, Butlers Creek Bridge	2.94	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-7	Murramarang Road, 115m south of O'Hara Street	6.47	-	-	-	-	-	-	-	-	-	-	6.56	0.09	0.5	H1	0:45
MR-7	Murramarang Road, 90m north of Scerri Dr	6.96	7.09	0.12	0.8	H1	3:15	7.11	0.15	1.1	H1	4:45	7.12	0.16	1.2	H1	3:45
MR-8	Pretty Beach Rd, 160m north Pretty Beach Campground	9.17	9.40	0.22	0.3	H1	6:30	9.46	0.29	0.5	H1	7:15	9.49	0.32	0.5	H2	6:15
MR-8	Merry Beach Road, Northern tributary culverts	2.54	2.59	0.05	0.2	H1	1:00	2.73	0.20	0.4	H1	3:15	2.85	0.31	0.5	H2	3:30
MR-8	Merry Beach Road, Bridge to Ingenia Holidays	3.07	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	Tasman Holiday Park, Internal bridge	2.30	2.70	0.40	0.2	H2	5:30	2.91	0.61	0.3	H3	7:00	3.02	0.72	0.4	H3	6:15

Table 9-4 Tabulated 5%, 2% and 1% AEP hydraulic model results for the Willinga Lake catchment, Bawley Point Road and Murramarang Road

ID	Location	TUFLOW DEM (mAHD)	5% AEP					2% AEP					1% AEP				
			Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)	Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)	Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)
	Willinga Lake Catchment																
	Willinga Lake WL gauge	-0.62	1.98	2.60	0.9	H5	12:00	2.35	2.97	1.0	H5	12:00	2.56	3.17	1.0	H5	12:00
	Willinga Lake main body	0.31	2.12	1.81	0.1	H4	12:00	2.36	2.05	0.1	H5	12:00	2.56	2.25	0.1	H5	12:00
	Bawley Point Rd, 70m east of Skylark Close	1.90	1.98	0.08	0.4	H1	3:30	2.35	0.45	0.5	H2	9:45	2.56	0.66	0.3	H3	9:00
	Willinga Road Bridge	3.11	-	-	-	-	-	3.22	0.12	1.1	H1	0:45	3.22	0.12	1.1	H1	1:00
	Forster Drive, 35m west of Willinga Park entrance gate	5.42	5.63	0.21	0.9	H1	1:15	5.79	0.37	1.4	H2	2:00	5.77	0.35	1.4	H2	3:00
	Forster Drive, 500m west of Willinga Park entrance gate	7.86	7.98	0.12	0.7	H1	0:45	8.03	0.17	1.1	H1	1:45	8.02	0.16	1.1	H1	2:00
	Bawley Point Road Local Catchments																
BPR-1	Bawley Point Road, 350m south-east of Princes Hwy	6.27	6.65	0.38	1.2	H2	4:15	6.76	0.49	1.5	H4	6:00	6.74	0.47	1.4	H4	8:30
BPR-1	Bawley Point Road, 400m south-east of Princes Hwy	6.15	6.40	0.25	1.7	H2	3:45	6.51	0.35	2.0	H4	5:00	6.49	0.34	2.0	H4	7:45
BPR-2	Bawley Point Road, 170m west of Tallawalla Way	3.30	3.52	0.23	0.7	H1	1:30	3.58	0.28	0.8	H1	1:30	3.59	0.30	0.8	H1	3:30
	Murramarang Road Local Catchments																
MR-1	Murramarang Road, 40m south of Binnowee Place	8.71	-	-	-	-	-	8.78	0.07	0.3	H1	1:15	8.81	0.10	0.5	H1	0:45
MR-1	Murramarang Road, Near Weemala Cres	5.38	-	-	-	-	-	5.45	0.08	0.4	H1	-	5.47	0.10	0.6	H1	0:15
MR-2	Murramarang Road, Near Wonnawong place	10.93	11.07	0.14	0.2	H1	3:00	11.09	0.16	0.2	H1	2:15	11.12	0.19	0.3	H1	3:30
MR-2	Murramarang Road, 90m north of Malibu Dr	9.11	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
MR-3	Murramarang Road, 60m south of Forster Dr	4.17	4.25	0.08	0.7	H1	1:00	4.31	0.14	1.3	H1	1:30	4.31	0.14	1.3	H1	1:15
MR-3	Murramarang Road, 90m north of Voyager Cres	3.79	3.84	0.05	0.7	H1	1:00	3.86	0.07	0.8	H1	1:30	3.85	0.07	0.8	H1	2:00
MR-4	Murramarang Road, Upstream of Limpid Lagoon	5.96	-	-	-	-	-	6.06	0.11	0.5	H1	0:45	6.05	0.09	0.4	H1	0:30
MR-5	Murramarang Road, 160m north of Bundle Hill Rd	4.96	5.04	0.08	0.9	H1	1:00	5.07	0.11	1.3	H1	1:45	5.07	0.10	1.2	H1	2:00
MR-5	Murramarang Road, Near Bundle Hill Rd	6.53	6.60	0.07	0.6	H1	3:00	6.61	0.08	0.7	H1	2:15	6.61	0.08	0.7	H1	3:15
MR-5	Murramarang Road, Adjacent to Tasman Holiday Park	6.08	6.14	0.06	0.5	H1	1:30	6.15	0.07	0.7	H1	2:00	6.15	0.07	0.6	H1	2:45
MR-6	Murramarang Rd, Butlers Ck trib, 130m nth of Moores Rd	3.95	4.04	0.09	0.7	H1	0:45	4.10	0.16	1.3	H1	1:30	4.10	0.15	1.3	H1	1:15
MR-6	Murramarang Rd, Butlers Ck trib, 120m sth of Moores Rd	3.13	3.20	0.06	0.4	H1	0:45	3.22	0.08	0.6	H1	1:30	3.22	0.09	0.6	H1	1:45
MR-7	Murramarang Road, Butlers Creek Bridge	2.94	-	-	-	-	-	3.18	0.24	0.1	H1	1:00	3.20	0.25	0.1	H2	1:00
MR-7	Murramarang Road, 115m south of O'Hara Street	6.47	6.58	0.11	0.7	H1	1:15	6.63	0.16	1.2	H1	2:00	6.63	0.16	1.1	H1	3:15
MR-7	Murramarang Road, 90m north of Scerri Dr	6.96	7.13	0.17	1.3	H1	4:00	7.15	0.19	1.4	H1	5:45	7.15	0.19	1.4	H1	8:15
MR-8	Pretty Beach Rd, 160m north Pretty Beach Campground	9.17	9.52	0.34	0.6	H2	7:00	9.58	0.40	0.7	H2	10:00	9.57	0.39	0.7	H2	11:15
MR-8	Merry Beach Road, Northern tributary culverts	2.54	2.99	0.45	0.5	H3	4:00	3.25	0.72	0.8	H3	5:15	3.23	0.70	0.7	H3	7:15
MR-8	Merry Beach Road, Bridge to Ingenia Holidays	3.07	-	-	-	-	-	3.20	0.12	0.6	H1	1:00	3.18	0.11	0.5	H1	0:45
	Tasman Holiday Park, Internal bridge	2.30	3.13	0.83	0.4	H3	6:45	3.32	1.02	0.5	H3	10:00	3.33	1.03	0.5	H3	10:30

Table 9-5 Tabulated 1in 200 AEP, 1 in 500 AEP and PMF hydraulic model results for the Willinga Lake catchment, Bawley Point Road and Murramarang Road

ID	Location	TUFLOW DEM (mAHD)	1 in 200 AEP					1 in 500 AEP					PMF				
			Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)	Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)	Peak Water Level (mAHD)	Peak Depth (m)	Peak Velocity (m/s)	Hazard	Duration (hrs:mins)
	Willinga Lake Catchment																
	Willinga Lake WL gauge	-0.62	2.64	3.26	1.1	H5	12:00	2.69	3.31	1.2	H5	12:00	3.61	4.22	1.9	H6	12:00
	Willinga Lake main body	0.31	2.67	2.36	0.1	H5	12:00	2.74	2.43	0.1	H5	12:00	3.67	3.36	0.4	H5	12:00
	Bawley Point Rd, 70m east of Skylark Close	1.90	2.63	0.73	0.4	H3	10:15	2.68	0.78	0.6	H3	10:30	3.55	1.65	1.1	H4	10:45
	Willinga Road Bridge	3.11	3.25	0.14	1.4	H1	1:15	3.29	0.18	1.6	H2	3:00	4.02	0.91	2.1	H5	3:45
	Forster Drive, 35m west of Willinga Park entrance gate	5.42	5.81	0.39	1.5	H2	3:30	5.85	0.44	1.6	H4	3:45	6.20	0.78	2.4	H5	3:15
	Forster Drive, 500m west of Willinga Park entrance gate	7.86	8.04	0.18	1.2	H1	3:00	8.06	0.20	1.3	H1	3:30	8.30	0.43	2.1	H5	3:00
	Bawley Point Road Local Catchments																
BPR-1	Bawley Point Road, 350m south-east of Princes Hwy	6.27	6.78	0.51	1.5	H4	10:30	6.82	0.56	1.6	H4	10:45	7.27	1.01	2.4	H5	10:00
BPR-1	Bawley Point Road, 400m south-east of Princes Hwy	6.15	6.53	0.38	2.1	H5	9:00	6.58	0.43	2.2	H5	10:00	7.17	1.02	2.8	H5	5:30
BPR-2	Bawley Point Road, 170m west of Tallawalla Way	3.30	3.67	0.38	0.9	H2	3:45	3.80	0.50	1.0	H2	4:00	4.95	1.65	1.5	H4	4:00
	Murramarang Road Local Catchments																
MR-1	Murramarang Road, 40m south of Binnowee Place	8.71	8.82	0.11	0.6	H1	0:45	8.83	0.12	0.7	H1	1:15	8.90	0.18	0.8	H1	3:30
MR-1	Murramarang Road, Near Weemala Cres	5.38	5.48	0.11	0.6	H1	0:15	5.50	0.12	0.7	H1	0:30	5.55	0.17	1.1	H1	2:15
MR-2	Murramarang Road, Near Wonnawong place	10.93	11.14	0.21	0.3	H1	4:30	11.16	0.23	0.4	H1	5:45	11.23	0.30	0.5	H2	3:30
MR-2	Murramarang Road, 90m north of Malibu Dr	9.11	9.17	0.06	0.5	H1	-	9.17	0.06	0.6	H1	-	9.19	0.08	0.7	H1	0:45
MR-3	Murramarang Road, 60m south of Forster Dr	4.17	4.32	0.15	1.4	H1	2:00	4.34	0.17	1.6	H1	3:15	4.49	0.32	2.2	H5	3:00
MR-3	Murramarang Road, 90m north of Voyager Cres	3.79	3.86	0.07	0.9	H1	2:45	3.86	0.07	0.9	H1	3:30	4.03	0.24	1.4	H1	3:00
MR-4	Murramarang Road, Upstream of Limpid Lagoon	5.96	6.07	0.12	0.6	H1	0:45	6.10	0.14	0.8	H1	1:00	6.20	0.25	1.4	H2	2:45
MR-5	Murramarang Road, 160m north of Bundle Hill Rd	4.96	5.07	0.11	1.3	H1	2:45	5.08	0.12	1.5	H1	3:30	5.16	0.20	2.6	H5	3:00
MR-5	Murramarang Road, Near Bundle Hill Rd	6.53	6.61	0.09	0.7	H1	4:00	6.62	0.09	0.8	H1	5:15	6.66	0.13	1.4	H1	3:15
MR-5	Murramarang Road, Adjacent to Tasman Holiday Park	6.08	6.16	0.08	0.7	H1	3:00	6.17	0.09	0.8	H1	3:45	6.25	0.17	1.5	H1	3:15
MR-6	Murramarang Rd, Butlers Ck trib, 130m nth of Moores Rd	3.95	4.12	0.17	1.4	H1	2:15	4.14	0.20	1.5	H1	3:15	4.53	0.59	2.3	H5	3:00
MR-6	Murramarang Rd, Butlers Ck trib, 120m sth of Moores Rd	3.13	3.23	0.09	0.6	H1	2:30	3.23	0.10	0.7	H1	3:30	4.55	1.42	1.0	H4	3:15
MR-7	Murramarang Road, Butlers Creek Bridge	2.94	3.31	0.37	0.2	H2	2:15	3.44	0.49	0.4	H3	2:45	4.54	1.59	1.1	H5	3:15
MR-7	Murramarang Road, 115m south of O'Hara Street	6.47	6.64	0.17	1.2	H1	3:30	6.66	0.19	1.4	H1	4:00	6.80	0.33	2.1	H5	3:15
MR-7	Murramarang Road, 90m north of Scerri Dr	6.96	7.16	0.20	1.5	H2	10:15	7.17	0.21	1.6	H2	10:45	7.28	0.32	1.9	H3	5:30
MR-8	Pretty Beach Rd, 160m north Pretty Beach Campground	9.17	9.59	0.41	0.8	H2	11:15	9.61	0.44	0.8	H2	11:15	9.87	0.70	1.2	H4	11:45
MR-8	Merry Beach Road, Northern tributary culverts	2.54	3.33	0.79	0.7	H3	9:00	3.46	0.92	0.7	H3	10:15	4.35	1.82	1.4	H5	6:45
MR-8	Merry Beach Road, Bridge to Ingenia Holidays	3.07	3.25	0.17	0.9	H1	1:00	3.36	0.28	1.2	H2	1:30	4.24	1.17	1.4	H5	3:00
	Tasman Holiday Park, Internal bridge	2.30	3.41	1.10	0.5	H3	10:45	3.52	1.22	0.5	H4	10:45	4.46	2.16	0.7	H5	11:30

10 Assessment of the Potential Impacts of Climate Change

10.1 Climate Change and Flooding

The Intergovernmental Panel on Climate Change's *Fifth Assessment Report* (IPCC 2013) found that human influence on climate is clear and increasing, with impacts observed across all continents and oceans. While projections vary, there is a general consensus that climate change will alter the severity of flood impacts through sea level rise (SLR) and an increase in the intensity of heavy rainfall events.

Quantifying the potential impacts of climate change on flooding allows current decisions on proposed development and flood risk mitigation measures within the study area to be assessed in an informed manner that considers potential changes in flood risk in the future.

10.1.1 Sea Level Rise

Under climate change, increasing temperatures would cause mean sea levels to rise due to the melting of ice sheets and thermal expansion due to ocean warming. While measuring underlying long-term trends in mean sea level is complicated by the occurrence of cyclical processes, analysis by Couriel, Modra and Jacobs (2014) suggests that sea level rise of 1 to 3 mm per year is already occurring at Fort Denison in Sydney Harbour. This is consistent with the geocentric global average sea level change reported by IPCC (2013).

Shoalhaven City Council has adopted SLR benchmarks of 0.23 m by 2050 and 0.85 m by 2100. These benchmarks have been adopted to assess the potential impact of seal level rise on flooding in the study area.

For ICOLLs, it is expected that the elevation of the entrance berm will increase in kind with SLR, along with a landward shift. All simulations of SLR scenarios have incorporated the relevant increase in berm elevation. The same increase was also applied to the minimum scoured bed level at the entrance.

10.1.2 Increase in Rainfall Intensity

While climate models show uncertainty in quantifying the effect of climate change on rainfall intensity, the *Climate Change in Australia Technical Report* from CSIRO and BoM (2015) projects increased intensity of extreme rainfall events for the east coast with a high confidence.

ARR 2019 Updated Climate Change Considerations Chapter, 2024

On 27 August 2024 an update of the Climate Change Considerations chapter of ARR 2019 (Book 1 Chapter 6) was published by DCCEEW and Engineers Australia.

The chapter provides up to date guidance aligned with the latest relevant science such as IPCC 2023 and Wasko et al. (2023). This includes new estimates of increased rainfall intensity under a range of climate change scenarios which are typically higher than those presented in the original chapter.

Climate change scenario modelling presented in this flood study was undertaken prior to the release of the final chapter and is based on the draft version which was published in December 2023. The final chapter incorporated marginally lower global mean surface temperature projections than the draft, resulting in estimated rainfall increases that are about 10% lower. The impact on Willinga Lake peak flood levels would be very minor due to the major influence of the berm level and/or tailwater but would be greater along other local watercourses and tributaries.

Adopted Increases in Rainfall Intensity

ARR 2019 Book 1 Chapter 6 presents climate change projections for five potential Shared Socioeconomic Pathways (SSPs) up to 2100 consistent with IPCC 2023. The SSPs are used to derive greenhouse gas emissions scenarios with different climate policies and are often referred to as very low (SSP1-1.9), low (SSP1-2.6), medium (SSP2-4.5), high (SSP3-7.0) and very high (SSP5-8.5) emissions pathways.

SSP3-7.0 has been selected for use in this study. It represents a high greenhouse gas emissions scenario where CO_2 emissions double by 2100 due to an international political climate with a low priority for addressing environmental concerns that leads to strong environmental degradation in some regions.

Adopted mean warming projections and percentage increases in rainfall per degree of warming for 2050 and 2100 planning horizons under SSP3-7.0 are presented in **Table 10-1** along with the resulting percentage increase in rainfall and design rainfall depth. It is notable that projected rainfall increases are higher for shorter duration storms.

Table 10-1 Adopted rainfall increases and depths for SSP3-7 2050 and 2100 climate scenarios

Design Flood Event	Critical Storm Duration (min)	Current Design Rainfall Depth (mm)	Temporal Pattern	SSP3-7 2050 Scenario (Medium-Term)				SSP3-7 2100 Scenario (Long-Term)			
				Mean Warming Projection (°C)	% inc rainfall per °C	Total % rainfall increase	Design Rainfall Depth (mm)	Mean Warming Projection (°C)	% inc rainfall per °C	Total % rainfall increase	Design Rainfall Depth (mm)
20% AEP	180	63.4	6094 (TP9)	2.0	11.8%	**23.6%**	78.4	3.6	11.8%	**42.5%**	90.3
	360	89.9	6152 (TP7)		10.2%	**20.4%**	108.2		10.2%	**36.7%**	122.9
10% AEP	120	64.6	6043 (TP6)		12.8%	**25.6%**	81.1		12.8%	**46.1%**	94.4
	270	94.8	6111 (TP5)		10.8%	**21.6%**	115.3		10.8%	**38.9%**	131.7
5% AEP	120	77.0	6043 (TP6)		12.8%	**25.6%**	96.7		12.8%	**46.1%**	112.5
	270	112.0	6111 (TP5)		10.8%	**21.6%**	136.2		10.8%	**38.9%**	155.5
2% AEP	120	94.0	5864 (TP1)		12.8%	**25.6%**	118.1		12.8%	**46.1%**	137.3
	540	192.0	6163 (TP5)		9.5%	**19.0%**	228.5		9.5%	**34.2%**	257.7
1% AEP	120	108.0	6036 (TP8)		12.8%	**25.6%**	135.6		12.8%	**46.1%**	157.8
	540	217.0	6156 (TP4)		9.5%	**19.0%**	258.2		9.5%	**34.2%**	291.2
1 in 500 AEP	120	141.0	6036 (TP8)		12.8%	**25.6%**	177.1		12.8%	**46.1%**	206.0
	720	327.0	6157 (TP4)		9.0%	**18.0%**	385.9		9.0%	**32.4%**	432.9
PMF	60	287.7	GSDM		13.7%	**27.4%**	366.5		13.7%	**49.3%**	429.6
	180	439.3			11.8%	**23.6%**	543.0		11.8%	**42.5%**	625.9

**Global mean surface temperature projections based on December 2023 Draft. Final ARR 2019 Book 1 Chapter 6 values approx. 10% lower.*

10.1.3 Adopted Climate Change Scenarios

The following climate change scenarios have been adopted to assess the potential impacts of climate change on flooding in the Willinga Lake catchment. Associated flood mapping is presented in **Volume 3**.

- 2050 Sea Level Rise Only Scenario (*refer* **Volume 3 Figures 2-1 to 2-36**)
  - 20%, 10%, 5%, 2%, 1% and 1 in 500 AEP and PMF flood events with 0.23 m SLR
  - HHWS(SS), 5% AEP and 1% AEP Oceanic Inundation events with 0.23 m SLR
- 2100 Sea Level Rise Only Scenario (*refer* **Volume 3 Figures 3-1 to 3-36**)

   - 20%, 10%, 5%, 2%, 1% and 1 in 500 AEP and PMF flood events with 0.85 m SLR
   - HHWS(SS), 5% AEP and 1% AEP Oceanic Inundation events with 0.85 m SLR
- 2050 SSP3-7 Climate Change Scenario (*refer* **Volume 3 Figures 4-1 to 4-30**)
   - 20%, 10%, 5%, 2%, 1% and 1 in 500 AEP and PMF flood events with SSP3-7 medium-term rainfall increases and 0.23 m SLR
   - Refer to **Table 10-1** for the percentage rainfall increases applied
- 2100 SSP3-7 Climate Change Scenario (*refer* **Volume 3 Figures 5-1 to 5-30**)
   - 20%, 10%, 5%, 2%, 1% and 1 in 500 AEP and PMF flood events with SSP3-7 long-term rainfall increases and 0.85 m SLR
   - Refer to **Table 10-1** for the percentage rainfall increases applied.

10.2 Assessment of Potential Climate Change Impacts

10.2.1 Sea Level Rise Only Scenarios

A summary of the impacts of the investigated sea level rise only scenarios on peak flood levels in the Willinga Lake catchment is presented in **Table 10-2**. The findings are summarised as follows.

- 2050 sea level rise of 0.23 m
   - In the 20% AEP and 5% AEP events flood level increases in Willinga Lake are in the order of 0.2 and 0.15 m respectively. These increases are associated with the 0.23 m increase in the initial entrance berm elevation rather than the coinciding HHWS(SS) ocean level of 1.29 mAHD which remains considerably lower than peak flood levels.
   - In the 1% AEP event peak flood levels in Willinga Lake increase by about 0.28 m, driven primarily by the increase in coinciding ocean tailwater level (rather than initial berm elevation). The impacts of SLR on peak flood levels did not propagate as far upstream as the Willinga Road bridge.
   - In the PMF event peak flood levels in Willinga Lake increase by only about 0.05 m indicating that neither than initial berm elevation nor the coinciding ocean tailwater level are dominant influences for this magnitude of flood event. A minor increase of 0.02 m occurred as far upstream as the Willinga Road bridge.
- 2100 sea level rise of 0.85 m
   - In the 20% AEP and 5% AEP events flood level increases in Willinga Lake are in the order of 0.7 to 0.8 m. These increases are associated with the 0.85 m increase in the initial entrance berm elevation rather than the coinciding HHWS(SS) ocean level of 1.91 mAHD which remains considerably lower than peak flood levels.
   - In the 1% AEP event peak flood levels in Willinga Lake increase by 0.86 m, driven primarily by the increase in coinciding ocean tailwater level (*rather than initial berm elevation*). The impacts of SLR on peak flood levels propagate upstream as far as the Willinga Road bridge where there is increase in peak flood level of 0.21 m.
   - In the PMF event peak flood levels in Willinga Lake increase by about 0.35 m. While significant, this increase is considerably less than the increase in initial berm level and ocean tailwater, indicating

that their influence is reduced in a flood event of this magnitude. A minor increase of 0.12 m occurred as far upstream as the Willinga Road bridge.

Findings in the Bawley Point Road local catchments are summarised as follows.

- SLR was not simulated in the Bawley Point Road local catchments as this would have required detailed investigation and modelling of Meroo Lake and its entrance berm, which was not within the project scope. Based on elevations along Bawley Point Rd, it is expected that the 2050 SLR scenario would have negligible impact on road inundation, while the 2100 SLR scenario would have negligible impact on BPR-1 but would be expected to have a minor impact at BPR-2.

Findings in the Murramarang local catchments are summarised as follows.

- The 2050 SLR scenario did not impact road inundation. The 2100 SLR scenario caused increases in peak flood levels at Butlers Creek of about 0.2 m in the 1% AEP, and just 0.01 m in the PMF.

10.2.2 Combined Rainfall Increase and Sea Level Rise Scenarios

A summary of the impacts of the investigated combined SSP3-7 rainfall increase and sea level rise climate change scenarios on peak flood levels in the Willinga Lake catchment is presented in **Table 10-3**. One key location in each of the Bawley Point Road and Murramarang Road local catchments has also been included. The findings are summarised as follows.

- 2050 SSP3-7 rainfall increase and sea level rise of 0.23 m
  - In events from the 20% AEP up to and including the 2% AEP flood level increases in Willinga Lake are in the order of 0.2 to 0.3 m. These increases are about 0.05 to 0.10 m higher than under the SLR only scenario.
  - Flood level increases in Willinga Lake are about 0.3 m, 0.23 m and 0.4 m in the 1% AEP, 1 in 500 AEP and PMF events respectively. The increase is only about 0.05 m higher than under the 1% AEP SLR only scenario but is 0.35 m higher in the PMF event.
  - Along the watercourses and flowpaths draining to Willinga Lake, flood level increases were predominantly in the range of 0.02 to 0.14 m across all events and locations. A larger increase of 0.34 m was observed at the Willinga Road bridge in the PMF event.
  - At 'BPR-1' in the Bawley Point Road local catchments, flood level increases ranged from 0.02 m in the 20% AEP event to 0.07 m in the 1 in 500 AEP event. In the PMF event a larger increase of 0.26 m was observed.
  - At the Butlers Creek bridge in the Murramarang Road local catchments, flood level increases were in the order of 0.2 m in the 2% AEP to 1 in 500 AEP events. In the PMF event a larger increase of 0.45 m was observed.
- 2100 SSP3-7 rainfall increase and sea level rise of 0.85 m
  - In events from the 20% AEP up to and including the 1% AEP flood level increases in Willinga Lake are in the range of 0.8 to 0.9 m. These increases are about 0.02 to 0.15 m higher than under the SLR only scenario.
  - Flood level increases in Willinga Lake are about 0.75 m, and 0.9 m in the 1 in 500 AEP and PMF events respectively. For the PMF event this increase is almost 0.6 m higher than under the PMF SLR only scenario.

- Along the watercourses and flowpaths draining to Willinga Lake, flood level increases were predominantly in the range of 0.05 to 0.2 m across all events and locations. At the Willinga Road bridge larger increases were observed of about 0.35 m in the 1% AEP and 1 in 500 AEP events and 0.7 m in the PMF event.
- At 'BPR-1' in the Bawley Point Road local catchments, flood level increases ranged from 0.04 m in the 20% AEP event to 0.12 m in the 1 in 500 AEP event. In the PMF event a larger increase of 0.47 m was observed.
- At the Butlers Creek bridge in the Murramarang Road local catchments, flood level increases were in the order of 0.3 m in the 2% AEP to 1 in 500 AEP events. In the PMF event a larger increase of 0.75 m was observed.

10.2.3 Summary of Potential Climate Change Impacts

Overall, the investigated rainfall and sea level rise projections indicate that climate change would be expected to have a significant adverse impact on flooding in the study area.

The 2050 projections would increase peak flood levels by 0.25 to 0.4 m in Willinga Lake across a range of flood magnitudes, and would increase peak flood levels along minor watercourses by 0.05 to 0.45 m.

The 2100 projections would increase peak flood levels by 0.8 to 0.9 m in Willinga Lake across a range of flood magnitudes, and would increase peak flood levels along minor watercourses by 0.05 to 0.75 m.

Table 10-2 Summary of Willinga Lake peak flood levels and increases under sea level rise only scenarios

Location	TUFLOW DEM (mAHD)	20% AEP			5% AEP			1% AEP			PMF		
		Existing PFL (mAHD)	SLR PFL (mAHD)	SLR Diff. (m)	Existing PFL (mAHD)	SLR PFL (mAHD)	SLR Diff. (m)	Existing PFL (mAHD)	SLR PFL (mAHD)	SLR Diff. (m)	Existing PFL (mAHD)	SLR PFL (mAHD)	SLR Diff. (m)
		2050 Sea Level Rise Only Scenario (+0.23 m)											
Willinga Lake WL gauge	-0.62	1.88	2.08	+0.21	1.98	2.15	+0.17	2.56	2.82	+0.27	3.61	3.66	+0.06
Willinga Lake main body	0.31	1.96	2.14	+0.18	2.12	2.25	+0.13	2.56	2.84	+0.28	3.67	3.73	+0.05
Bawley Point Rd, 70m east of Skylark Close	1.90	-	2.06	-	1.98	2.11	+0.13	2.56	2.82	+0.26	3.55	3.61	+0.06
Willinga Road Bridge	3.11	-	-	-	-	-	-	3.22	3.22	+0.00	4.02	4.03	+0.02
Forster Drive, 35m west of Willinga Park entrance gate	5.42	-	-	-	5.63	5.63	-0.00	5.77	5.77	+0.00	6.20	6.20	0.00
Forster Drive, 500m west of Willinga Park entrance gate	7.86	-	-	-	7.98	7.98	0.00	8.02	8.02	0.00	8.30	8.30	0.00
		2100 Sea Level Rise Only Scenario (+0.85 m)											
Willinga Lake WL gauge	-0.62	1.88	2.69	+0.81	1.98	2.74	+0.76	2.56	3.42	+0.86	3.61	3.95	+0.34
Willinga Lake main body	0.31	1.96	2.70	+0.74	2.12	2.77	+0.65	2.56	3.42	+0.86	3.67	4.00	+0.33
Bawley Point Rd, 70m east of Skylark Close	1.90	-	2.68	-	1.98	2.73	+0.75	2.56	3.41	+0.86	3.55	3.90	+0.35
Willinga Road Bridge	3.11	-	-	-	-	-	-	3.22	3.43	+0.21	4.02	4.14	+0.12
Forster Drive, 35m west of Willinga Park entrance gate	5.42	-	-	-	5.63	5.63	-0.00	5.77	5.77	+0.00	6.20	6.20	0.00
Forster Drive, 500m west of Willinga Park entrance gate	7.86	-	-	-	7.98	7.98	0.00	8.02	8.02	0.00	8.30	8.30	0.00

Table 10-3 Summary of Willinga Lake peak flood levels and increases under combined climate change scenarios

Location	TUFLOW DEM (mAHD)	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			1 in 500 AEP			PMF		
		Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)	Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)	Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)	Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)	Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)	Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)	Existing PFL (mAHD)	CC PFL (mAHD)	CC Diff. (m)
		2050 Climate Change Scenarios (SSP3-7 rainfall & 0.23 m SLR)																				
Willinga Lake WL gauge	-0.62	1.88	2.12	+0.24	1.91	2.17	+0.25	1.98	2.24	+0.26	2.35	2.58	+0.23	2.56	2.85	+0.29	2.69	2.92	+0.23	3.61	4.01	+0.40
Willinga Lake main body	0.31	1.96	2.22	+0.26	2.03	2.28	+0.25	2.12	2.38	+0.26	2.36	2.59	+0.23	2.56	2.87	+0.31	2.74	2.96	+0.22	3.67	4.07	+0.40
Bawley Point Rd, 70m east of Skylark Close	1.90	-	2.08	-	1.97	2.13	+0.16	1.98	2.20	+0.22	2.35	2.58	+0.23	2.56	2.84	+0.29	2.68	2.91	+0.23	3.55	3.95	+0.40
Willinga Road Bridge	3.11	-	-	-	-	-	-	-	3.20	-	3.22	3.27	+0.05	3.22	3.28	+0.06	3.29	3.38	+0.09	4.02	4.36	+0.34
Forster Drive, 35m west of Willinga Park entrance gate	5.42	-	5.52	-	5.54	5.65	+0.11	5.63	5.74	+0.11	5.79	5.85	+0.06	5.77	5.85	+0.08	5.85	5.91	+0.06	6.20	6.34	+0.14
Forster Drive, 500m west of Willinga Park entrance gate	7.86	-	-	-	7.94	7.98	+0.04	7.98	8.00	+0.02	8.03	8.05	+0.02	8.02	8.06	+0.03	8.06	8.11	+0.05	8.30	8.38	+0.08
BPR-1: Bawley Point Road, 350m south-east of Princes Hwy	6.27	6.59	6.61	+0.02	6.62	6.66	+0.04	6.66	6.71	+0.04	6.76	6.81	+0.04	6.75	6.81	+0.06	6.83	6.90	+0.07	7.27	7.53	+0.26
MR-7: Murramarang Road, Butlers Creek Bridge	2.94	-	-	-	-	-	-	-	3.10	-	3.18	3.36	+0.18	3.20	3.41	+0.21	3.44	3.60	+0.16	4.54	4.99	+0.45
		2100 Climate Change Scenarios (SSP3-7 rainfall & 0.85 m SLR)																				
Willinga Lake WL gauge	-0.62	1.88	2.74	+0.87	1.91	2.80	+0.89	1.98	2.87	+0.89	2.35	3.20	+0.85	2.56	3.44	+0.88	2.69	3.47	+0.78	3.61	4.53	+0.92
Willinga Lake main body	0.31	1.96	2.79	+0.83	2.03	2.85	+0.82	2.12	2.93	+0.81	2.36	3.20	+0.84	2.56	3.44	+0.88	2.74	3.48	+0.74	3.67	4.58	+0.91
Bawley Point Rd, 70m east of Skylark Close	1.90	-	2.74	-	1.97	2.79	+0.82	1.98	2.86	+0.87	2.35	3.20	+0.85	2.56	3.43	+0.88	2.68	3.46	+0.78	3.55	4.47	+0.92
Willinga Road Bridge	3.11	-	-	-	-	3.21	-	-	3.25	-	3.22	3.34	+0.12	3.22	3.46	+0.24	3.29	3.55	+0.26	4.02	4.72	+0.70
Forster Drive, 35m west of Willinga Park entrance gate	5.42	-	5.59	-	5.54	5.73	+0.18	5.63	5.80	+0.17	5.79	5.87	+0.08	5.77	5.88	+0.11	5.85	5.96	+0.11	6.20	6.40	+0.20
Forster Drive, 500m west of Willinga Park entrance gate	7.86	-	7.97	-	7.94	8.00	+0.06	7.98	8.02	+0.04	8.03	8.07	+0.04	8.02	8.09	+0.06	8.06	8.14	+0.08	8.30	8.43	+0.14
BPR-1: Bawley Point Road, 350m south-east of Princes Hwy	6.27	6.59	6.63	+0.04	6.62	6.70	+0.08	6.66	6.75	+0.09	6.76	6.85	+0.08	6.75	6.86	+0.11	6.83	6.95	+0.12	7.27	7.74	+0.47
MR-7: Murramarang Road, Butlers Creek Bridge	2.94	-	-	-	-	-	-	-	3.29	-	3.18	3.50	+0.31	3.20	3.53	+0.34	3.44	3.73	+0.29	4.54	5.29	+0.75

11 Sensitivity Analysis

A series of analyses were carried out using the hydrologic and hydraulic models both to inform the selection of appropriate conditions for adoption in design flood estimation, and to understand the relative influence of selected parameters on flood model results.

The results of sensitivity testing for design flood events are presented in the following. Additional sensitivity testing undertaken for the November 2023 calibration event is presented in **Section 7.3.8**.

11.1 Sensitivity to Entrance Berm Conditions

Flood levels in Willinga Lake are influenced by the geometry of the entrance berm at the beginning of a flood, and by its morphology during a flood. To assist in understanding the influence of berm conditions on flooding and thereby select an appropriate condition for design flood estimation, the sensitivity of flood levels to three initial berms levels was investigated and is presented in **Section 6.3**.

11.2 Sensitivity to Ocean Tailwater Conditions

The sensitivity of 5% AEP and 1% AEP peak flood levels to coinciding tailwater levels was tested for the HHWS(SS), 5% AEP and 1% AEP oceanic conditions. The results are presented in **Table 11-1**.

Table 11-1 Results of ocean tailwater sensitivity testing

Design Flood Event	Tailwater Condition	Peak Ocean Level (mAHD)	Peak WL at lake centre (mAHD)	Peak WL at Gauge (mAHD)	Bawley Point Rd overtopping depth
5% AEP	HHWS(SS)	1.06	2.12	1.99	0.06 m
	5% AEP	2.35	2.36	2.36	0.44 m
	1% AEP	2.55	2.56	2.56	0.64 m
1% AEP	HHWS(SS)	1.06	2.35	2.17	0.19 m
	5% AEP	2.35	2.52	2.47	0.55 m
	1% AEP	2.55	2.65	2.62	0.70 m

It is evident that the elevated 5% AEP and 1% AEP ocean levels have a significant impact on peak flood levels. The 5% AEP and 1% AEP catchment driven peak flood levels in Willinga Lake with the lower HHWS(SS) tailwater are 2.12 and 2.35 mAHD respectively. It follows that the 5% AEP and 1% AEP ocean levels of 2.35 and 2.55 mAHD (*including wave setup*) would have a significant impact on flood levels and even be the dominant driver of flooding under some scenarios.

11.3 Sensitivity to Hydraulic Roughness

Hydraulic roughness, as defined using the Manning's 'n' parameter, is one of the primary parameters that can be refined through the hydraulic model calibration process. Whilst the calibration process has provided confidence in the adopted values, the inherent variability in potential hydraulic roughness values warrants consideration of their relative impact on adopted design flood conditions.

The sensitivity of 5% AEP and 1% AEP event peak flood levels to hydraulic roughness has been tested by applying a 25% increase and a 25% decrease in the adopted values for the baseline design flood conditions. The resulting flood impact mapping is presented in **Volume 3 Figures 4-1 to 4-4**.

Impacts of a 25% increase in hydraulic roughness are summarised as follows.

- 5% AEP event
  - Peak flood levels across Willinga Lake upstream of Bawley Point Road increased by about 47 mm, associated with a slight decrease in discharge rate through the bridge and entrance channel.
  - Peak flood levels along the various watercourses and flowpaths increased by about 40 to 70 mm.
- 1% AEP event
  - Peak flood levels across Willinga Lake increased by less than 10 mm. Peak lake levels in this event are heavily influenced by ocean levels and less so by hydraulic roughness.
  - Peak flood levels along the various watercourses and flowpaths increased by about 30 to 80 mm.

Impacts of a 25% decrease in hydraulic roughness are summarised as follows.

- 5% AEP event
  - Peak flood levels across Willinga Lake upstream of Bawley Point Road decreased by about 27 mm, associated with a slight increase in discharge rate through the bridge and entrance channel.
  - A localised increase of about 20 mm occurred on the outside a bend in the channel approaching the Willinga Lake entrance, associated with a slight increase in flow velocities.
  - Peak flood levels along the various watercourses and flowpaths decreased by about 30 to 100 mm.
- 1% AEP event
  - Peak flood levels across Willinga Lake decreased by about 5 mm or less. Peak lake levels in this event are heavily influenced by ocean levels and less so by hydraulic roughness.
  - Peak flood levels along the various watercourses and flowpaths decreased by about 30 to 100 mm.

Overall, the sensitivity of peak flood levels to changes in hydraulic roughness is considered low, particularly in Willinga Lake. The watercourses and flowpaths which drain to Willinga Lake have higher flood flow velocities and are thus more sensitive to changes in hydraulic roughness.

11.4 Sensitivity to Culvert Blockage

The sensitivity of flood levels to culvert blockage was investigated by applying blockage factors derived following guidance presented in ARR 2019 Book 6 Chapter 6 '*Blockage of Hydraulic Structures*' as follows.

- High debris availability assumed due to considerable forested land.
- Medium debris mobility assumed as source area slopes are generally moderate.
- Medium debris transportability assumed as stream bed slopes are moderate and streams shallow.
- This results in a medium debris potential (HMM).
- An L^{10} of 1.2 metres was assumed.

The resulting blockage factors are presented in **Table 11-2** according to design event and culvert size.

Table 11-2 Summary of Design Blockage Factors

Design Flood Event	Class 1 Opening < 1.2 m	Class 2 1.2 m ≤ Opening ≤ 3.6 m	Class 3 Opening > 3.6 m
50% AEP to 10% AEP	25%	0%	0%
5% AEP to 1 in 200 AEP	50%	10%	0%
1 in 500 AEP and PMF	100%	20%	10%

The resulting flood level difference mapping showing increases in peak flood levels caused by the structural blockages is presented in **Volume 3 Figures 4-5 and 4-6**. The findings are summarised as follows.

- 5% AEP event
  - A localised increase in peak flood level of 27 mm occurs at a set of culverts on Forster Drive, about 500 m west of the Willinga Park entrance gates.
  - Small, localised increases in peak flood level of up 9 mm occur in the vicinity of Skylark Close. It is expected that this location may be more sensitive to blockages in more frequent flood events where local catchment flooding is critical rather than Willinga Lake flooding.
  - No notable flood level increases occur elsewhere in the Willinga Lake study area due to blockage.
- 1% AEP event
  - A localised increase in peak flood level of 19 mm occurs at a set of culverts on Forster Drive, about 500 m west of the Willinga Park entrance gates.
  - No notable flood level increases occur elsewhere in the Willinga Lake study area due to blockage.

Overall, peak flood levels in the Willinga Lake study area are not sensitive to structural blockages.

11.5 Comparison of ARR 2019 and ARR 1987

A comparison of ARR 2019 and ARR 1987 based design rainfall depths, rainfall loss rates and resulting peak flood levels at the Willinga Lake water level gauge is presented in **Table 11-3**. **Volume 3 Figures 4-7 and 4-8** show associated differences in peak flood levels across the Willinga Lake study area.

Table 11-3 Comparison of ARR 2019 and ARR 1987 rainfall, loss rates and peak flood levels

Design Flood Event	Critical Storm Durations (min)	Design Rainfall Depth (mm)	Initial Loss (mm)	Continuing Loss (mm/hr)	PFL at Willinga Lake gauge (mAHD)
ARR 19 5% AEP	120 270	77.0 112.0	5.5 5.7	2.2	**1.98**
ARR 87 5% AEP	120 540	106.4 186.0	10.0	2.5	**2.15**
ARR 19 1% AEP	120 540	108.0 217.0	3.4 2.9	2.2	**2.56**
ARR 87 1% AEP	120 540	147.6 258.3	10.0	2.5	**2.72**

The following is evident from in **Table 11-3** and **Volume 3 Figures 4-7 and 4-8**.

- The ARR 1987 design rainfall depths are significantly higher than the corresponding ARR 2019 values, ranging from 19% higher for the 1% AEP 540 minute storm to 38% higher for the 5% AEP 120 minute storm.
- Despite the ARR 1987 rainfall loss rates being somewhat higher than their ARR 2019 counterparts, this resulted in ARR 1987 peak flood levels being 0.17 m and 0.16 m higher at the Willinga Lake gauge in the 5% AEP and 1% AEP events respectively.
- In the 5% AEP event ARR 1987 peak flood levels are about 0.22 m higher in Willinga Lake upstream of Bawley Point Road, 0.15 to 0.18 m higher downstream of Bawley Point Road, and generally in the order of 0.1 to 0.15 m higher along other watercourses and flowpaths.
- In the 1% AEP event ARR 1987 peak flood levels are about 0.25 m higher in Willinga Lake upstream of Bawley Point Road, 0.15 to 0.18 m higher downstream of Bawley Point Road, and generally in the order of 0.15 to 0.2 m higher along other watercourses and flowpaths.

11.6 Sensitivity to Temporal Rainfall Pattern

In estimating design flood conditions, the ARR 2019 guidelines recommend analysis of 'ensembles' of 10 temporal rainfall patterns that have been derived to represent observed variability for each design storm magnitude and duration.

The sensitivity of design flow estimations to changes in temporal rainfall pattern has been analysed by reviewing peak flow box charts across multiple storm durations and hydrographs for each of the ten rainfall temporal patterns in an ensemble for the selected critical duration storms. The box charts and ensemble hydrographs at two key locations are presented in **Figure 11-1** and **Figure 11-2** for the 5% AEP and 1% AEP events respectively. Key observations from the analysis are summarised in the following.

- **Willinga Lake**
  - For the 5% AEP event:
    - The **box chart** illustrates that for short duration storms (2 to 3 hours) variation in peak flows between temporal patterns is relatively small, as the attenuating nature of the lake dampens out the influence of the hyetograph shape.
    - For the selected critical duration of 270 minutes (4.5 hours) and longer durations, the variation in peak flow due to temporal pattern is considerable, with the highest being about 50% greater than the lowest. Nonetheless, there are several hydrographs bunched close together, with peaks within about ±15% of that of the adopted 'average' pattern.
    - The range of peak flows observed for temporal patterns within a single duration is typically greater than the variability between 'averaged' peak flows across several durations. For example, peak temporal pattern flows for the 4.5 hour event range from about 60 m^3/s to 90 m^3/s while 'average' peak flows across durations from 2 hours to 24 hours range only from about 60 m^3/s to 75 m^3/s.
    - The flow **hydrographs** for the 270 minute storm show a high degree of similarity in shape and duration for most temporal patterns. This is a product of the dampening effect of the Willinga Lake storage. The timing of the rising limb and peak can differ by up to about 2.5 hours.
  - For the 1% AEP event:
    - The **box chart** exhibits some similar trends to the that for the 5% AEP event, in that variation in peak flows between temporal patterns is smaller for shorter duration storms than longer durations, and that variability between temporal patterns within a single duration can be greater than the variability between 'averaged' peak flows across several durations.
    - For the selected critical duration of 540 minutes (9 hours) the variation in peak flow due to temporal pattern, while lower than for the 5% AEP event, remains considerable with the highest being about 28% greater than the lowest.
    - The flow **hydrographs** for the 540 minute storm generally have a similar 'double peaked' shape, with the timing of the peak varying between the front end and back end depending on the pattern. The adopted 'average' temporal pattern has its peak earlier in the storm, the highest peak flow pattern has its peak later in the storm, while both peaks of the pattern with the lowest peak flow are comparable.
- **Bawley Point Road near the Princes Highway (BPR-1)**
  - For the 5% AEP event:
    - The **box chart** exhibits some similar trends to those for Willinga Lake. Variation in peak flows between temporal patterns is generally smaller for shorter duration storms than longer durations, and variability between temporal patterns within a single duration can be greater than the variability between 'averaged' peak flows across several durations.
    - For the selected critical duration of 120 minutes (2 hours) the variation in peak flow due to temporal pattern is considerable, with the highest being about 60% greater than the lowest. Nonetheless, there are several hydrographs bunched close together, with peaks within about ±15% of that of the adopted 'average' pattern.
    - Of the flow **hydrographs** for the 120 minute storm about half are very similar in shape, with two notably 'peakier' shapes, and two that are notably more 'rounded'. The timing of the rising limb and peak can differ by up to about 1.25 hours.

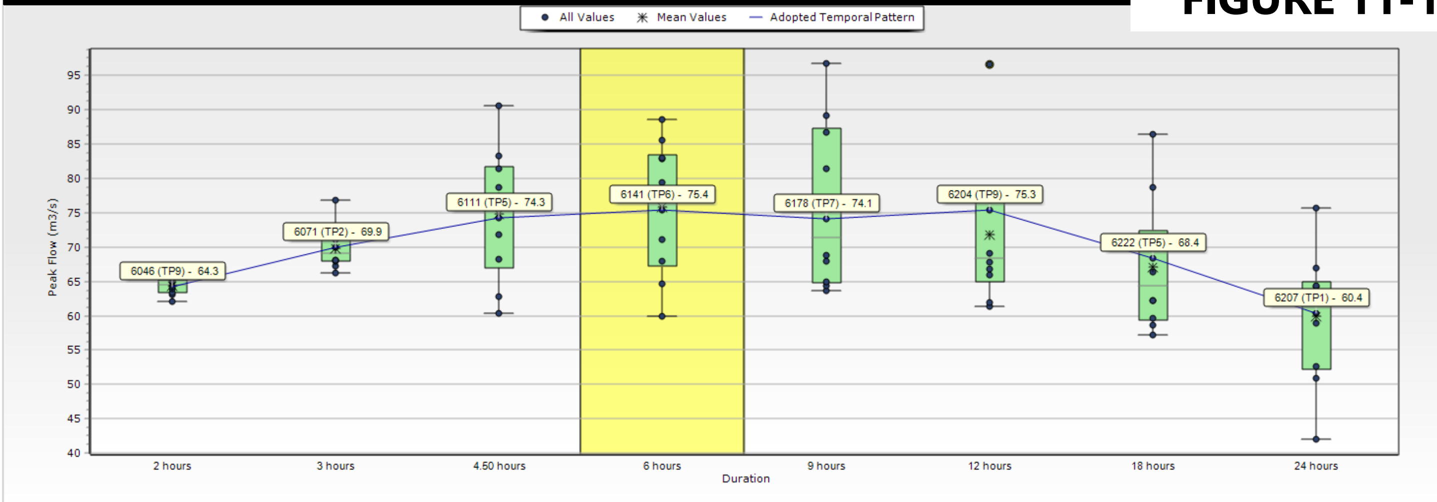
- For the 1% AEP event:
  - The **box chart** exhibits similar trends to the others, in that variation in peak flows between temporal patterns is smaller for shorter duration storms than longer durations, and that variability between temporal patterns within a single duration can be greater than the variability between 'averaged' peak flows across several durations.
  - For the selected critical duration of 120 minutes (2 hours) the variation in peak flow due to temporal pattern, while lower than for the 5% AEP event, remains considerable with the highest being about 38% greater than the lowest. Nonetheless, 8 out of the 9 other patterns have peaks within about ±10% of that of the adopted 'average' pattern.
  - The flow **hydrographs** for the 120 minute storm show significant variation in hydrograph shape, some having a 'front loaded' shape, some a 'back loaded' shape, and others that have a longer, flatter peak including that with the lowest peak flow.

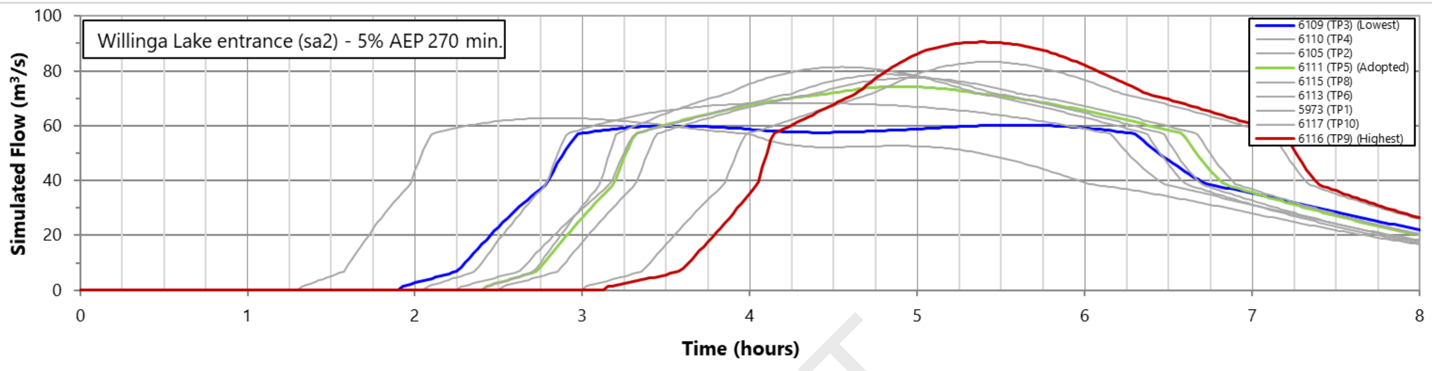
In summary, it was generally found that the 5% AEP event temporal pattern ensemble hydrographs exhibited greater variation that those for the 1% AEP event. While it is not clear whether there is greater variation inherent in the temporal rainfall patterns for the 5% AEP event, it appears that the hydraulics of the system begin to reduce the degree of variation in hydrographs for larger flood events such as the 1% AEP. It was also found that longer duration storms exhibit greater variability than short duration storms, which would be as expected.

Overall, the ARR 2019 ensemble approach exhibits large variations in peak flow between temporal patterns (*e.g. up to 60% at BPR-1 in the 5% AEP 120 minute storm*). The magnitude of this variation is typically much greater than the variation in 'average' peak flows between comparable storm durations, and even leads to considerable overlap in flow estimates between floods of different probabilities. For example, the 5% AEP 4.5 hour TP9 peak flow of 90.5 m^3/s at Willinga Lake is higher than the 1% AEP 9 hour TP6 peak flow of 87 m^3/s, yet according to the ARR 2019 guidelines it is said to be 5 times more likely to occur each year. This suggests that there may be a lack of AEP neutrality within temporal patterns ensembles and thus that there would be little value in investigating design flood conditions associated with temporal patterns that are not close to the mean for a particular AEP.

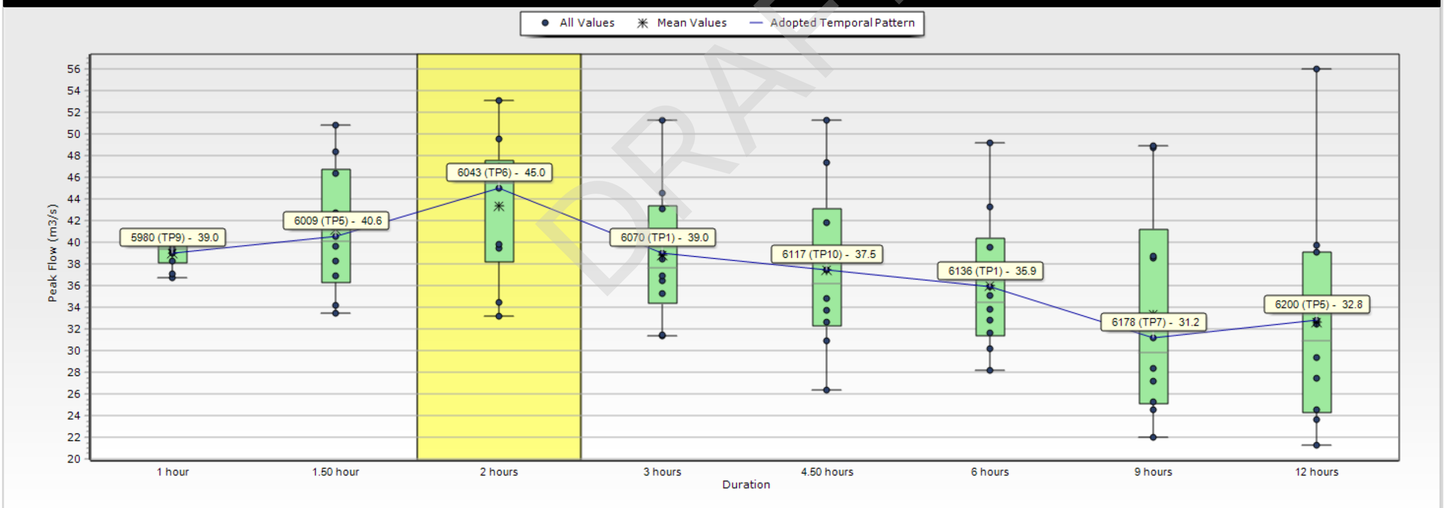
Sensitivity of 5% AEP flows to temporal pattern at Willinga Lake – Box Chart & Critical Dur'n Hydrographs

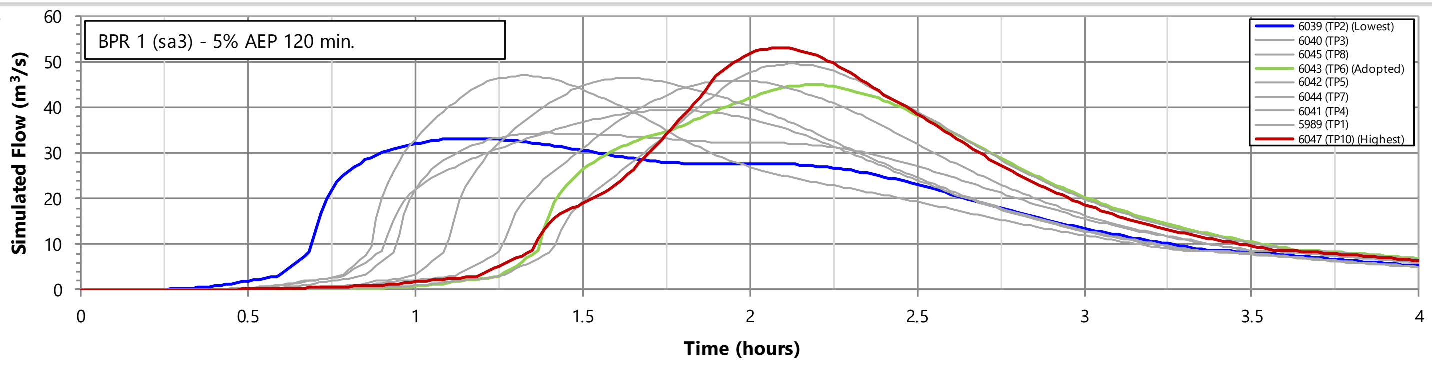
FIGURE 11-1





Sensitivity of 5% AEP flows to temporal pattern at BPR1 (Bawley Point Rd nr. Princes Hwy) – Box Chart & Critical Dur'n Hydrographs





Prepared by:

Shoalhaven City Council

worley consulting

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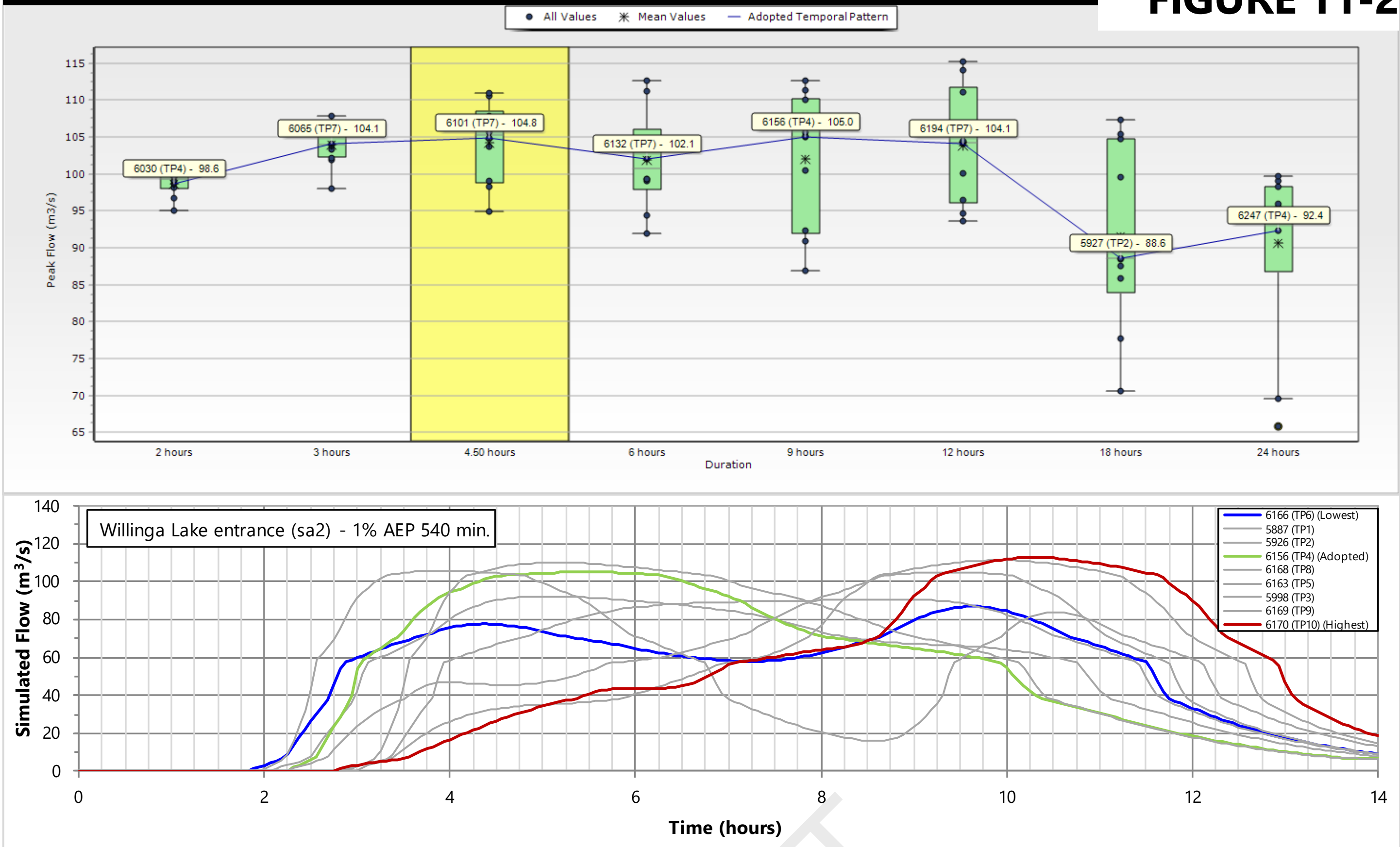
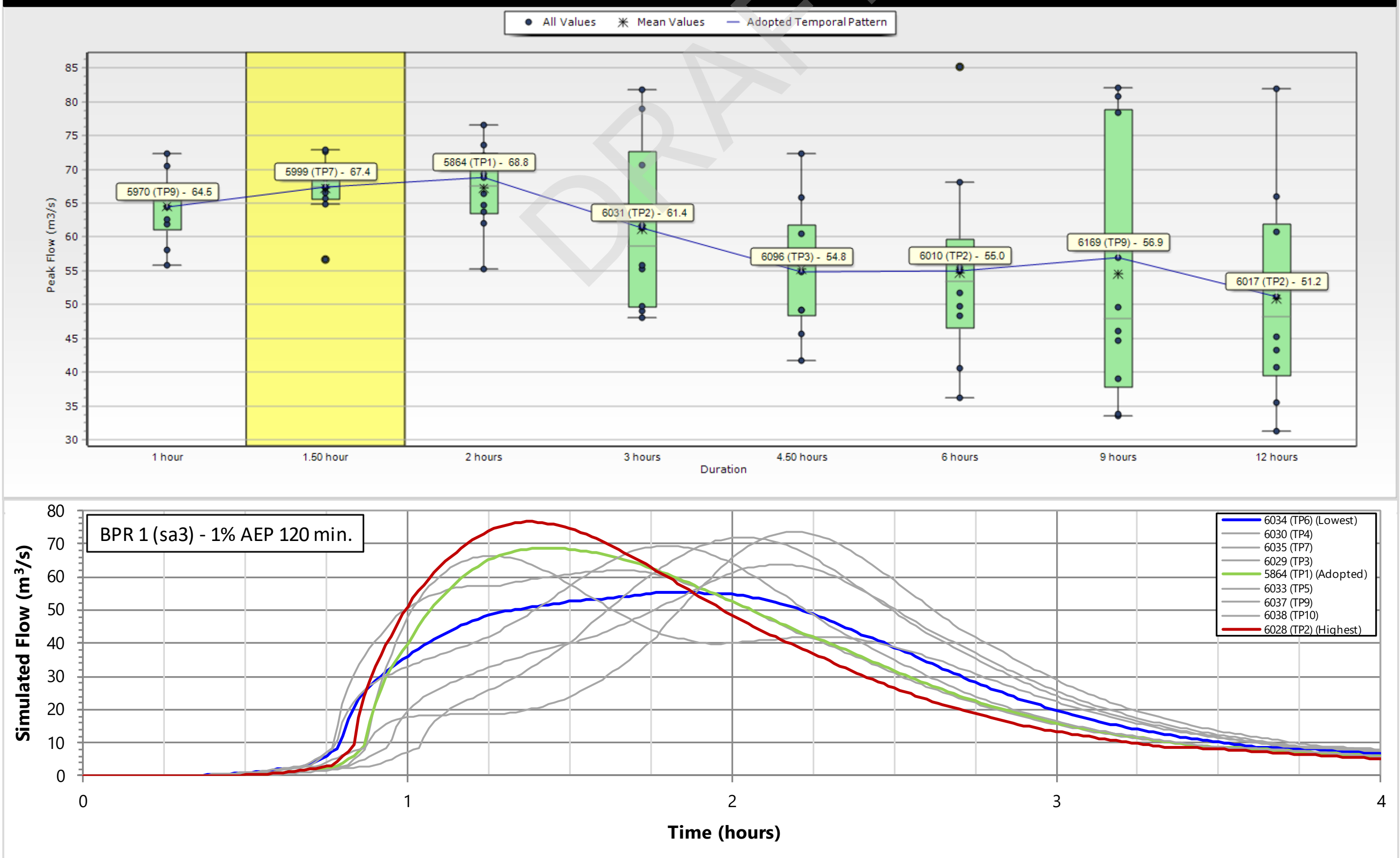
SENSITIVITY OF 5% AEP FLOWS TO TEMPORAL RAINFALL PATTERN

FIGURE 11-2

Sensitivity of 1% AEP flows to temporal pattern at Willinga Lake – Box Chart & Critical Dur'n Hydrographs



Sensitivity of 1% AEP flows to temporal pattern at BPR1 (Bawley Point Rd nr. Princes Hwy) – Box Chart & Critical Dur'n Hydrographs



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SENSITIVITY OF 1% AEP FLOWS TO TEMPORAL RAINFALL PATTERN

11.7 Sensitivity to Hydrologic Model Parameters and Loss Rates

A series of sensitivity tests were undertaken for the 5% and 1% AEP events using the WBNM hydrologic model to assess the influence of alternative model parameters and rainfall loss rates on peak flow rates at a range of locations. The results are presented in **Table 11-4** and **Table 11-5** and are discussed in the following.

11.7.1 WBNM Runoff Lag Parameter 'C'

The calibrated WBNM hydrologic model adopted a standard runoff lag value of 1.6. The influence of alternative values of 1.4 (*representing a faster runoff response*) and 1.8 (*representing a slower runoff response*) on peak flow rates was assessed with the findings as follows (*refer* **Table 7-2**).

- 5% AEP event (*refer* **Table 11-4**)
  - At Willinga Lake, a faster runoff response (C=1.4) had no impact on the estimated peak discharge rate, while the slower runoff response (C=1.8) resulted in a 6% decrease in estimated peak discharge rate. While discharge from the lake is dominated by volume rather than the timing of inflows, it appears that the high C value slowed inflows to the lake to such a degree that discharge rate was affected.
  - Under a faster runoff response (C=1.4) simulated peak flow rates increased at most other reporting locations. Increases were in the order of 0 to 5% at watercourses where storage volume has an important influence, and 5 to 10% along flow dominated watercourses.
  - Under a slower runoff response (C=1.8) simulated peak flow rates decreased at most other locations. Decreases were in the order of 0 to 5% at watercourses where storage volume has an important influence, and 5 to 9% along flow dominated watercourses.
- 1% AEP event (*refer* **Table 11-5**)
  - At Willinga Lake, both the increase and decrease in runoff rate resulted in a 1% change in the estimated peak discharge rate. This is because discharge from the lake is dominated by volume rather than the timing of inflows.
  - Under a faster runoff response (C=1.4) simulated peak flow rates increased at all other reporting locations. Increases were in the order of 1 to 4% at watercourses where storage volume has an important influence, and 5 to 7% along flow dominated watercourses.
  - Under a slower runoff response (C=1.8) simulated peak flow rates decreased at most other reporting locations. Decreases were in the order of 0 to 4% at watercourses where storage volume has an important influence, and 5 to 7% along flow dominated watercourses.
  - Changes in flow rates for the 1% AEP were typically lower than for the 5% AEP event.

11.7.2 WBNM Stream Routing Lag Parameter 'F'

The calibrated WBNM hydrologic model adopted a standard stream routing lag 'F' value of 1.0 for natural sub-catchments and a lower (*i.e. faster*) value of 0.8 in model sub-catchments where land along the flood flow path is predominantly cleared and/or developed. The influence of alternative catchment-wide 'F' values of 0.6 and 0.8 on peak flow rates was assessed with the findings as follows.

- 5% AEP event (*refer* **Table 11-4**)

- At Willinga Lake the reduction in stream routing lag to 0.8 had no impact on the estimated peak discharge rate, while further reduction to 0.6 resulted in an increase of 1%. This is because discharge from the lake is dominated by volume rather than the timing of inflows.
- At other reporting locations an 'F' value of 0.8 resulted in simulated peak flow rates increases ranging from 0% to 7%. The lower increases occurred at watercourses where part of the catchment is already cleared/developed (*i.e. F is already 0.8*) and/or where storage volume has an important influence on peak flow.
- At other reporting locations an 'F' value of 0.6 resulted in simulated peak flow rates increases ranging from 0% to 16%. In this case the lower increases occurred at watercourses where storage volume has an important influence on peak flow.

- 1% AEP event (*refer* **Table 11-5**)
  - At Willinga Lake the reduction in stream routing lag to 0.8 had no impact on the estimated peak discharge rate, while further reduction to 0.6 resulted in an increase of 1%. This is because discharge from the lake is dominated by volume rather than the timing of inflows.
  - At other reporting locations an 'F' value of 0.8 resulted in simulated peak flow rates increases ranging from 1% to 5%. The lower increases occurred at watercourses where part of the catchment is already cleared/developed (*i.e. F is already 0.8*) and/or where storage volume has an important influence on peak flow.
  - At other reporting locations an 'F' value of 0.6 resulted in simulated peak flow rates increases ranging from 1% to 12%. In this case the lower increases occurred at watercourses where storage volume has an important influence on peak flow.

11.7.3 Alternative Rainfall Loss Rates

Design flood estimation adopted ARR 2019 based probability-neutral initial losses ranging from 2.9 to 7.9 mm and a continuing loss of 2.2 mm/hr (*refer* **Table 8-1**). Significantly higher loss rates of 50 mm IL and 6.5 mm/hr CL were adopted for the November 2023 calibration event. The influence of the calibration rainfall loss rates on design peak flow rates was assessed with the findings as follows.

- 5% AEP event (*refer* **Table 11-4**)
  - The increased losses resulted in significant reductions in peak flows at all locations, predominantly in the range of 30% to 60%.
- 1% AEP event (*refer* **Table 11-5**)
  - The increased losses resulted in significant reductions in peak flows at all locations, ranging from 25% to 47%.

It is noted that these significant reductions in peak flows would not necessarily have a significant impact on peak flood levels at Willinga Lake across all design flood events. This is because peak flood levels in frequent floods are dominated by the initial berm elevation, while peak flood levels in the 2% and 1% AEP events are dominated by the elevated ocean tailwater level. Additionally, the increased losses may have resulted in increases in critical storm duration that would have lessened reductions in peak flows (*this was not investigated*).

Table 11-4 Results of 5% AEP event WBMN hydrologic model sensitivity testing

WBNM subarea	Location	Adopted WBNM Peak flow (m^3/s)	Runoff lag 'C'				Stream routing lag 'F'				Rainfall Loss Rates	
			C = 1.4		C = 1.8		F = 0.6		F = 0.8		Cali. Losses IL 50 mm CL 6.5 mm/hr	
2	Willinga Lake outlet	76.35	76.35	(0%)	71.78	(-6%)	77.35	(+1%)	75.98	(-0%)	35.28	(-54%)
17	Willinga Road Bridge	35.76	39.24	(+10%)	32.39	(-9%)	41.18	(+15%)	38.20	(+7%)	15.88	(-56%)
21	Tributary discharging to Willinga Lake north-west of Willinga Park	31.95	34.29	(+7%)	29.54	(-8%)	35.61	(+11%)	33.74	(+6%)	13.23	(-59%)
92	Tributary discharging to Willinga Lake north-east of Willinga Park	33.70	35.85	(+6%)	31.52	(-6%)	36.01	(+7%)	33.87	(+1%)	13.54	(-60%)
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	44.99	47.78	(+6%)	42.16	(-6%)	48.17	(+7%)	45.70	(+2%)	18.19	(-60%)
8	Cormorant Beach outlet	1.50	1.50	(0%)	1.47	(-2%)	1.50	(+0%)	1.49	(-0%)	0.71	(-52%)
9	Gannet Beach outlet	3.02	3.20	(+6%)	2.78	(-8%)	3.21	(+6%)	3.14	(+4%)	2.08	(-31%)
10	Murramarang Beach north	7.24	7.24	(0%)	7.23	(-0%)	7.32	(+1%)	7.23	(-0%)	3.73	(-48%)
11	Murramarang Beach south	4.00	4.00	(0%)	3.80	(-5%)	4.02	(+1%)	3.96	(-1%)	1.21	(-70%)
12	Racecourse Beach north	12.26	12.91	(+5%)	11.63	(-5%)	13.07	(+7%)	12.64	(+3%)	5.77	(-53%)
13	Racecourse Beach south	6.00	6.23	(+4%)	5.76	(-4%)	6.42	(+7%)	6.17	(+3%)	2.71	(-55%)
14	Butlers Creek outlet	32.12	35.58	(+11%)	29.77	(-7%)	37.28	(+16%)	34.35	(+7%)	12.89	(-60%)
15	Kioloa Beach outlet	15.03	15.90	(+6%)	14.10	(-6%)	16.50	(+10%)	15.68	(+4%)	6.70	(-55%)
16	Merry Beach outlet	16.89	17.74	(+5%)	16.02	(-5%)	17.97	(+6%)	17.31	(+3%)	7.18	(-57%)

Table 11-5 Results of 1% AEP event WBMN hydrologic model sensitivity testing

WBNM subarea	Location	Adopted WBNM Peak flow (m^3/s)	Runoff lag 'C'				Stream routing lag 'F'				Rainfall Loss Rates	
			C = 1.4		C = 1.8		F = 0.6		F = 0.8		Cali. Losses IL 50 mm CL 6.5 mm/hr	
2	Willinga Lake outlet	104.99	105.56	(+1%)	104.37	(-1%)	105.95	(+1%)	105.45	(+0%)	78.32	(-25%)
17	Willinga Road Bridge	54.29	58.09	(+7%)	50.45	(-7%)	60.39	(+11%)	56.80	(+5%)	29.03	(-47%)
21	Tributary discharging to Willinga Lake north-west of Willinga Park	46.29	49.44	(+7%)	43.61	(-6%)	51.67	(+12%)	48.73	(+5%)	27.01	(-42%)
92	Tributary discharging to Willinga Lake north-east of Willinga Park	48.65	51.95	(+7%)	45.86	(-6%)	52.00	(+7%)	48.93	(+1%)	28.85	(-41%)
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	64.71	68.78	(+6%)	61.15	(-6%)	70.06	(+8%)	66.27	(+2%)	38.67	(-40%)
8	Cormorant Beach outlet	2.30	2.32	(+1%)	2.27	(-1%)	2.34	(+2%)	2.31	(+1%)	1.61	(-30%)
9	Gannet Beach outlet	3.44	3.64	(+6%)	3.42	(-0%)	3.69	(+7%)	3.48	(+1%)	2.78	(-19%)
10	Murramarang Beach north	12.00	12.37	(+3%)	11.62	(-3%)	12.54	(+5%)	12.19	(+2%)	8.56	(-29%)
11	Murramarang Beach south	6.66	6.71	(+1%)	6.61	(-1%)	6.76	(+1%)	6.70	(+1%)	3.98	(-40%)
12	Racecourse Beach north	18.62	19.40	(+4%)	17.88	(-4%)	19.48	(+5%)	19.05	(+2%)	10.73	(-42%)
13	Racecourse Beach south	8.54	9.05	(+6%)	8.11	(-5%)	9.43	(+10%)	8.93	(+4%)	5.28	(-38%)
14	Butlers Creek outlet	60.02	63.24	(+5%)	56.69	(-6%)	63.78	(+6%)	62.06	(+3%)	32.87	(-45%)
15	Kioloa Beach outlet	20.64	21.93	(+6%)	19.44	(-6%)	23.00	(+11%)	21.56	(+4%)	12.89	(-38%)
16	Merry Beach outlet	23.38	24.95	(+7%)	22.25	(-5%)	25.66	(+10%)	24.15	(+3%)	14.50	(-38%)

11.8 Post-Bushfire Sensitivity

The Willinga Lake catchment comprises large areas of bushfire prone land, with some 50-75% of the catchment impacted by the 2019-20 Black Summer bushfires.

Bushfires have the potential to change the runoff response of an affected catchment through the destruction of vegetation and the alteration of soil properties. This may result in a faster runoff response and lower hydraulic roughness as well as changes to rainfall losses from canopy interception and soil infiltration. It is generally expected that the quantity of runoff would increase following a bushfire, before slowly decreasing back to pre-fire quantities.

Sensitivity testing presented in the previous sections has included investigation of the impacts of reduced runoff lag rates and reduced hydraulic roughness which, in combination, are representative of the physical catchment changes that could be expected post-bushfire. The relevant findings of these analyses are summarised as follows.

- Willinga Lake
  - A faster runoff response (C=1.4) had negligible impact on peak 5% AEP or 1% AEP discharge rates from Willinga Lake.
  - A 25% decrease in hydraulic roughness resulted in small decreases in 5% AEP and 1% AEP peak flood levels in Willinga Lake.
- All other watercourses in the study area
  - Under a faster runoff response (C=1.4) peak flow rates increased at most other locations in the study area by 0 to 5% at watercourses where storage volume has an important influence, and 5 to 10% along flow dominated watercourses in both the 5% AEP and 1% AEP events.
  - Under a 25% decrease in hydraulic roughness peak flood levels along the various watercourses and flowpaths decreased by about 30 to 100 mm in both the 5% AEP and 1% AEP events.

Based on the above it is expected that the combination of a faster runoff response and lower hydraulic roughness that could potentially occur under post-bushfire conditions would have negligible impact on peak flood levels in Willinga Lake and other volume dominated watercourses. In the more flow dominated watercourses and flowpaths it is expected that there would be small increases in peak flood levels and velocities as the increase in flow rate may outweigh the influence of reduced hydraulic roughness across the floodplain.

12 Conclusions and Recommendations

This report details the development and calibration of detailed flood models for the Willinga Lake catchment and the 'Bawley Point Road local catchment' and 'Murramarang Road local catchment' secondary study areas. It also details the subsequent estimation and mapping of design flood conditions and analysis of the potential impact of climate change on flooding.

The modelling and mapping were undertaken as the initial stage of the Willinga Lake Flood Study and Floodplain Risk Management Study and Plan (FRMS&P) to provide a contemporary definition of flood characteristics across the study area using the latest relevant guidelines including Australian Rainfall and Runoff: A Guide to Flood Estimation 2019 (ARR 2019). This includes the provision of design flood levels, depths, discharges, velocities, hazard, flood function, and other information relevant to the management of flood risk. A compendium of flood mapping is presented in **Volume 3** of this report.

The models and outputs produced provide a basis to inform the preparation of the FRMS&P for the study area including the assessment of potential floodplain risk management measures.

It is recommended that the design flood estimates and associated modelling and mapping described in this report be adopted for use in the FRMS&P.

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14 Glossary and Abbreviations

The following glossary and abbreviations has 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
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

Term	Short form	Definition	Context for use/additional information
Coastal inundation		Inundation due to tidal or storm-driven coastal events, including storm surges in lower coastal waterways. This can be exacerbated by wind-wave generation from storm events	
Consent authority		The authority or agency with the legislative power to determine the outcome of development and building applications	This may be the relevant local council or Minister
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
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 breech 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

Term	Short form	Definition	Context for use/additional information
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)
Ecologically sustainable development	ESD	As outlined in the Local Government Act 1993	Principles of ESD are outlined in the Local Government Act 1993
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

Term	Short form	Definition	Context for use/additional information
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, 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

Term	Short form	Definition	Context for use/additional information
Flood education		Seeks to provide information to raise community awareness of flooding so as to enable individuals to understand how to manage themselves and their property in response to flood warnings	
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.)
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

Term	Short form	Definition	Context for use/additional information
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	
Flood storage areas		Areas of the floodplain that are outside floodways which generally provide for temporary storage of floodwaters during the passage of a	See also flood function, floodways and flood fringe areas

Term	Short form	Definition	Context for use/additional information
		flood and where flood behaviour is sensitive to changes that impact on temporary storage of water during a flood	
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

Term	Short form	Definition	Context for use/additional information
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
FRM measures		Measures that can reduce flood risk	FRM measures may include FRM, flood mitigation, EM and land-use planning measures
FRM options		The FRM measures that might be feasible for the management of a particular area of the floodplain	Preparation of an FRM plan requires a detailed evaluation of FRM options
FRM plan		A management plan developed in accordance with the principles in this manual and its supporting guidelines	Previously known as a floodplain risk management plan or floodplain management plan. It may describe how particular areas of flood prone land are to be used and managed to achieve defined objectives
FRM study		A management study developed in accordance with the principles in this manual and its supporting guidelines	Previously known as a floodplain risk management study or floodplain management study
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

Term	Short form	Definition	Context for use/additional information
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	
Integrated planning and reporting framework	IP & R framework	The IP & R framework includes a suite of integrated plans that set out a vision and goals and strategic actions to achieve them. It involves a reporting structure to communicate progress to council and the community as well as a structured timeline for review to ensure the goals and actions are still relevant	Preparation of FRMS and plans and implementation and maintenance of works requires linkages to the IP & R framework
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	

Term	Short form	Definition	Context for use/additional information
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	
Merit-based approach		Weighs social, economic, ecological and cultural impacts of land-use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and wellbeing of the state's rivers and floodplains	The merit approach operates at 2 levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk, which are formulated into council plans, policy and environmental planning instruments At a site-specific level, it involves consideration of the merits of a development consistent with council LEPs, DCPs and local FRM policies, and consistent with FRM plans
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	
Prevention, preparedness, response and recovery	PPRR	Involves: ▪ prevention: to eliminate or reduce the level of the risk or severity of emergencies ▪ preparedness: enhances the capacity of agencies and communities to cope with the consequences of emergencies	In the flood context prevention involves FRM (including flood mitigation), EM and land-use planning measures

Term	Short form	Definition	Context for use/additional information
		▪ response: to ensure the immediate consequences of emergencies to communities. are minimised ▪ recovery: measures that support individuals and communities affected by emergencies in the reconstruction of physical infrastructure and restoration of physical, emotional, environmental and economic wellbeing	
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'

Term	Short form	Definition	Context for use/additional information
Risk analysis		The systematic use of available information to determine how often specified (flood) events occur and the magnitude of their likely consequences	
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
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
Technical working group	TWG		
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

Appendix A. Assessment of Critical Storm Duration

A.1 Overview of Critical Storm Duration and Temporal Pattern

For the purposes of the FRMS&P, 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, storage behaviour etc.*), and therefore may have varying critical storm durations and applicable temporal rainfall patterns.

Given the run time of the developed TUFLOW two-dimensional hydraulic model, 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 several assessment locations of interest for each design event.
- 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 were selected for each flood magnitude, that in combination, provided the overall best match to 'average peak design flows' across the assessment locations.
- For the 5% AEP, 1% AEP and PMF events additional investigation of several storm durations of interest was undertaken using the TUFLOW hydraulic model to confirm the preferred combination of two storm durations. The findings for these events were then considered in selecting storm durations for other events.

A summary of the final selected critical storm durations and temporal patterns for each design event is presented in **Table A- 2.** Additional details of the assessment process are provided in the following sections.

A.2 Additional Assessment of Critical Storm Durations using TUFLOW

While all due effort was made to build reliable stream routing and storage behaviour into the WBNM hydrologic model, due to the complexities of timing and entrance breakout there remained some uncertainty as to whether the critical storm durations identified by the model, based on peak flow rates, would also produce maximum TUFLOW flood levels in Willinga Lake. Accordingly, it was decided to simulate several storm durations of interest in TUFLOW to confirm the preferred critical duration for Willinga Lake.

The additional analysis was undertaken for the 5% AEP, 1% AEP and PMF events. Details are presented in the following section with a particular focus on the 1% AEP event.

TUFLOW Assessment of Critical Storm Durations for the 1% AEP Event

Hydrologic modelling using WBNM and Storm Injector identified critical storm durations of 90 minutes for Bawley Point Road local catchments, 120 minutes for most Murramarang Road local catchments, and 270 minutes for Willinga Lake (*refer* **Table A- 8**). Notably, average peak flows for Willinga Lake were comparable for durations from 3 hours to 12 hours (*refer* **Figure A- 1**).

Based on **Figure A- 1** and **Table A- 8** there was some uncertainty as to whether a storm duration of 90 minutes or 120 minutes would be best representative of critical durations across the various local catchments, and as to which storm duration from 3 hours to 12 hours would produce maximum flood levels in Willinga Lake.

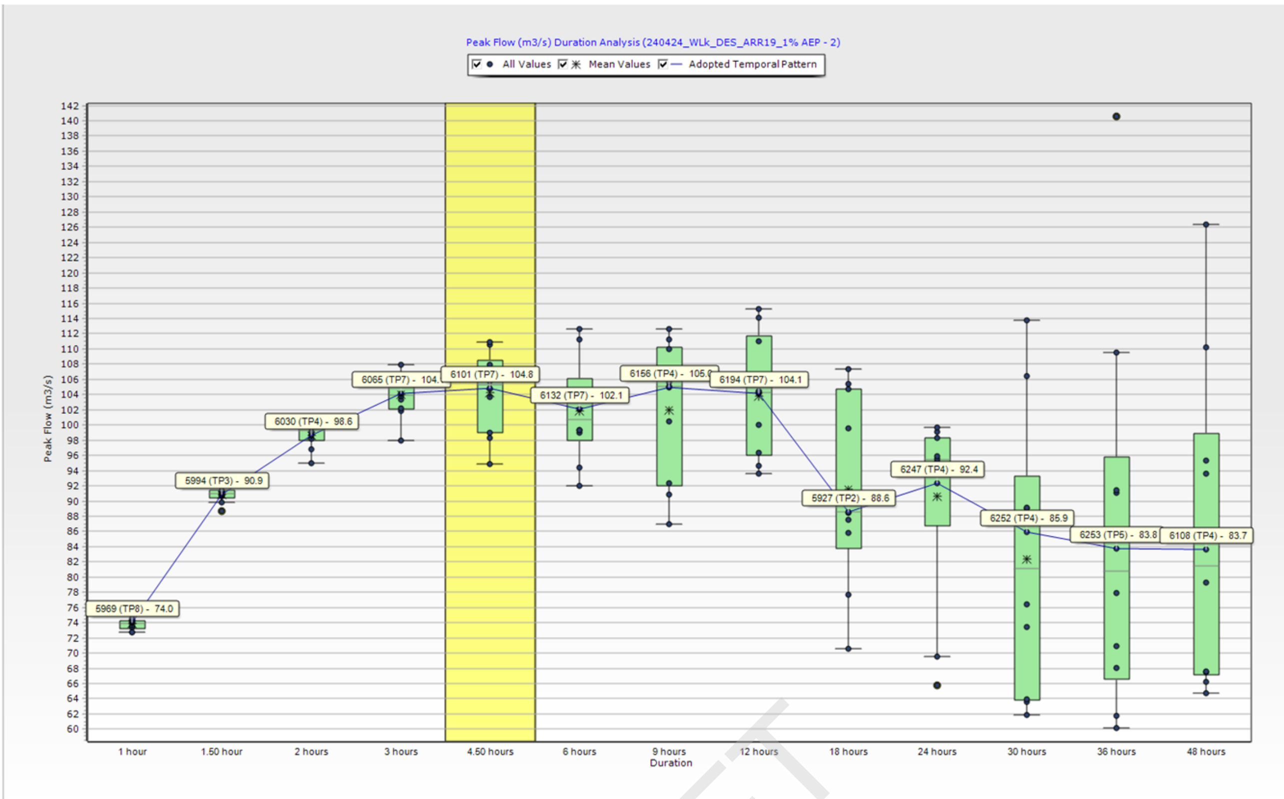


Figure A- 1 Box chart showing range of simulated peak 1% AEP flows by duration at the Willinga Lake outlet

Accordingly, and given the importance of the 1% AEP to floodplain management activities, all 7 standard storm durations from 90 minutes up to and including 720 minutes (12 hours) were simulated in the TUFLOW model for one temporal pattern of interest each. As the assessment was focused on catchment driven flooding a moderate ocean tailwater level (HHWS(SS)) was applied. **Figure A- 2** shows which storm duration resulted in maximum peak flood levels across the study area for the 1% AEP event.

Critical durations across the study area as indicated by the TUFLOW modelling are summarised as follows.

- Willinga Lake catchment
  - 180 minutes for Willinga Lake upstream of Bawley Point Road.
  - 540 minutes for Willinga Lake downstream of Bawley Point Road, including where the maximum depth of road overtopping occurs near Skylark Close.
  - 90 minutes for watercourses at Willinga Road Bridge and through Willinga Park.
- Bawley Point Road local catchments
  - 90 minutes at the critical overtopping location near the Princes Highway (BPR-1).
  - 120 minutes at other minor overtopping locations.
- Murramarang Road local catchments
  - 120 minutes at the majority of road overtopping locations.
  - 90 minutes at a few minor road overtopping locations.

- 270 minutes to 720 minutes in storage areas downstream of Murramarang Road, however flooding from these storages does not back up across the road and thus is not key to the study objectives.

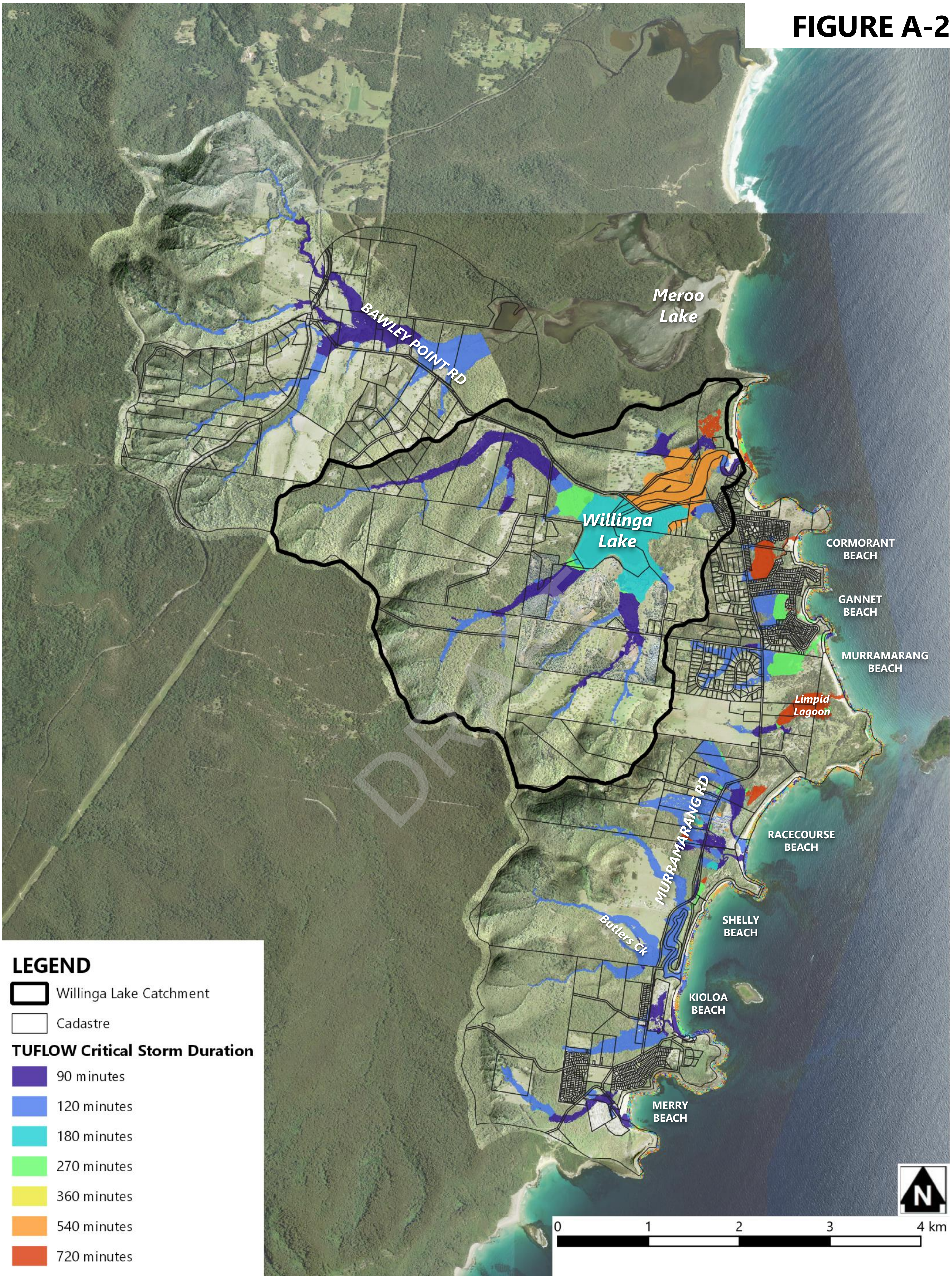
The WBNM and TUFLOW assessments of critical storm duration identified that the main durations of interest were the 90 minute, 120 minute, 270 minute and 540 minute storms. Peak flood levels from these storms were then reviewed at a range of locations of interest to analyse the magnitude of differences in peak flood level and to determine the combination of two durations which would best represent peak flood levels across the study area (*refer* **Table A- 1**). The key findings were as follows.

- While the 90 minute duration storm is critical at the key 'BPR-1' location, the difference in peak flood level to the 120 minute storm at this location is only 1 mm. The 120 minute storm is critical at several more locations than the 90 minute storm. Accordingly, it would be preferable to adopt the 120 minute duration storm.
- Overall, there would be minimal difference between adopting the 270 minute storm or 540 minute storm. However, at the low point of Bawley Point Road near Skylark Close the resulting peak flood level for the 540 minute storm is 22 mm higher than that for the 270 minute storm. Given that investigation of Willinga Lake driven flooding of Bawley Point Road is a primary objective of the study it would be preferable to adopt the 540 minute duration storm.

A comparison of the maximum flood level envelope from all seven durations with that created from the 90 minute and 270 minute storms only (*i.e., the key durations identified by the WBNM model*) is presented in side 'A' of **Figure A- 3**. The difference in peak flood level across the vast majority of the study area is less than 20 mm, indicating that adoption of the two critical storm durations identified by WBMN at the two key locations would result in an appropriate estimation of 1% AEP peak flood levels across the study area. The same comparison using the maximum flood level envelope from the 120 minute and 540 minute storms is presented in side 'B' of **Figure A- 3** and generally represents a slight improvement.

Accordingly, based on the TUFLOW modelling the preferred combination of storm durations for the 1% AEP event is the 120 minute and 540 minute storms. Nonetheless, the TUFLOW analysis demonstrated that the implications of adopting the 90 minute and 270 minute storms identified by the WBMN model would have been insignificant in terms of difference in peak flood levels.

FIGURE A-2



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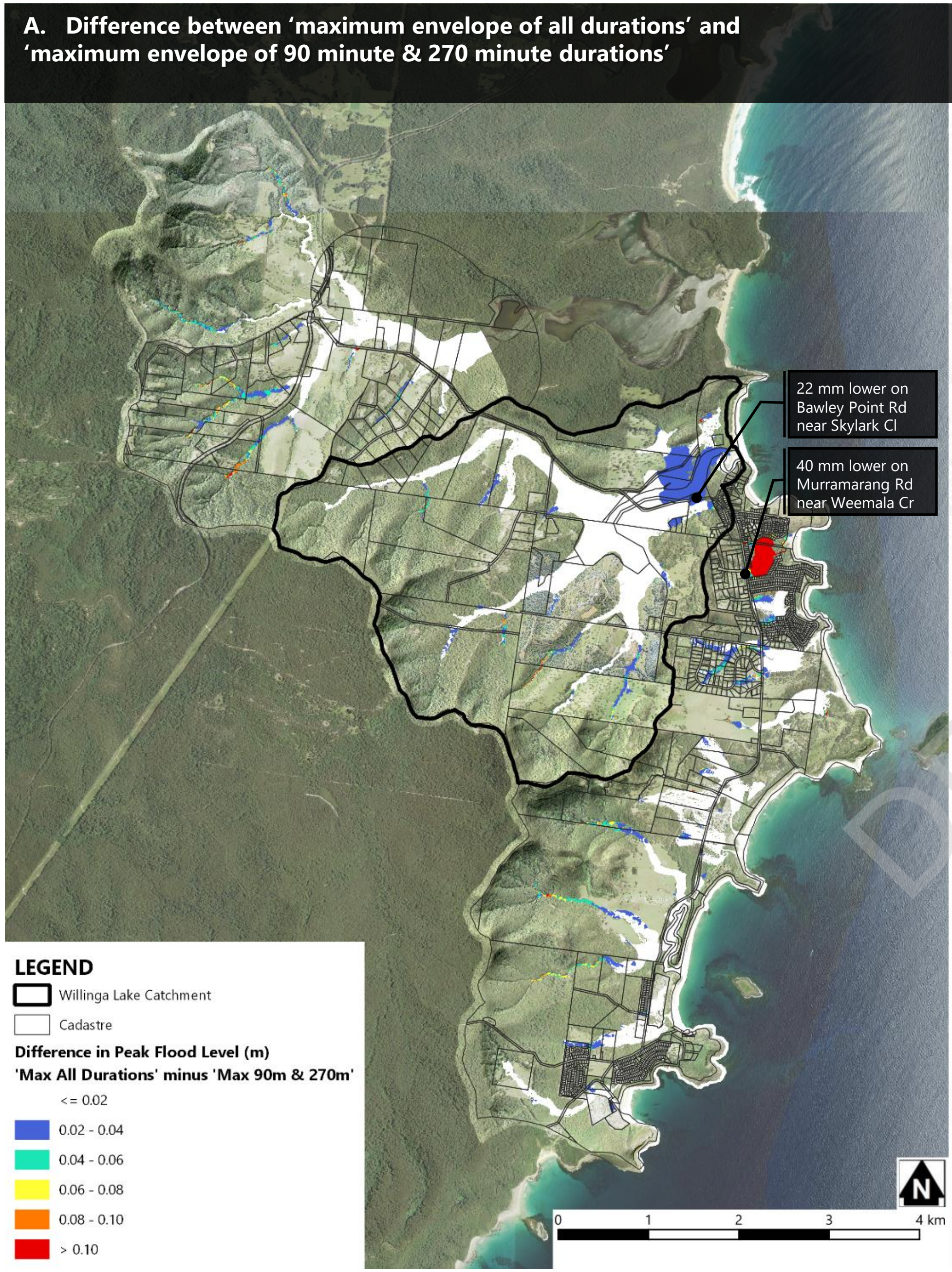
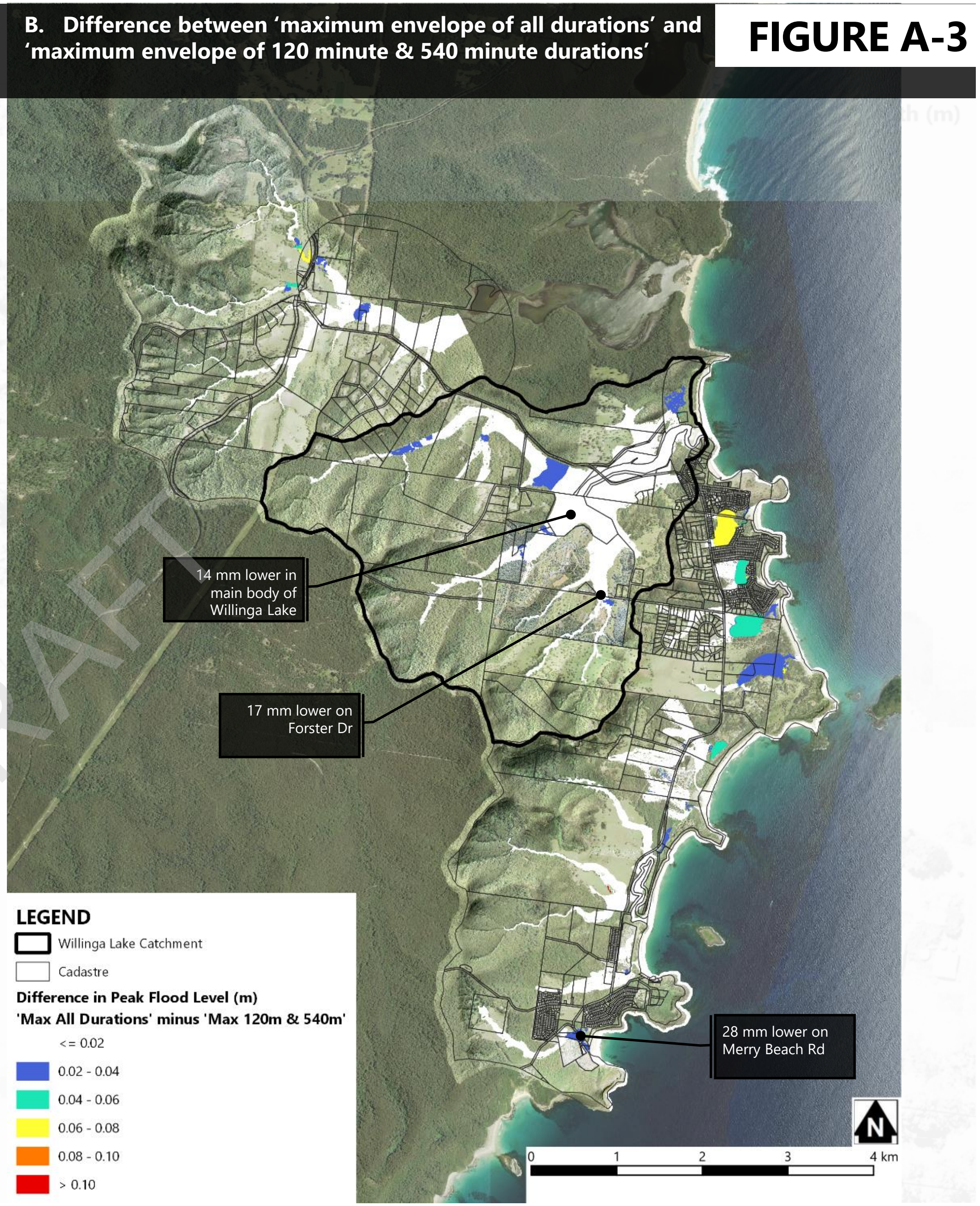
TUFLOW ASSESSMENT OF 1% AEP CRITICAL STORM DURATIONS ACROSS THE STUDY AREA

Table A- 1 Peak 1% AEP peak flood levels for a range of storm durations and combinations

Location	1pct90m	1pct120m	1pct180m	1pct270m	1pct360m	1pct540m	1pct720m	Max Envelope	Envelope 90m & 270m	Envelope 120m & 270m	Envelope 120m & 540m	Difference (m) MaxEnv - 90&270	Difference (m) MaxEnv - 120&270	Difference (m) MaxEnv - 120&540
Willinga Lake Catchment														
Willinga Lake WL gauge	2.110	2.152	2.168	2.155	2.110	2.171	2.135	2.171	2.155	2.155	2.171	0.016	0.016	0.000
Willinga Lake main body	2.267	2.327	2.359	2.355	2.284	2.346	2.322	2.359	2.355	2.355	2.346	0.005	0.005	0.014
Bawley Point Rd_70m east of Skylark Close	2.044	2.086	2.097	2.081	2.034	2.104	2.061	2.104	2.081	2.086	2.104	0.022	0.018	0.000
Willinga Road Bridge	3.224	3.223	3.171	3.200	0.000	3.152	3.173	3.224	3.224	3.223	3.223	0.000	0.001	0.001
Forster Drive_35m west of Willinga Park entrance gate	5.785	5.768	5.665	5.711	5.511	5.614	5.688	5.785	5.785	5.768	5.768	0.000	0.017	0.017
Forster Drive_500m west of Willinga Park entrance gate	8.019	8.023	7.989	8.000	7.921	7.976	7.995	8.023	8.019	8.023	8.023	0.004	0.000	0.000
Bawley Point Road Local Catchments														
Bawley Point Road_350m south-east of Princes Hwy	6.752	6.751	6.680	6.704	6.618	6.657	6.695	6.752	6.752	6.751	6.751	0.000	0.000	0.000
Bawley Point Road_400m south-east of Princes Hwy	6.507	6.506	6.435	6.458	6.374	6.412	6.449	6.507	6.507	6.506	6.506	0.000	0.001	0.001
Bawley Point Road_170m west of Tallawalla Way	3.575	3.593	3.544	3.553	3.491	3.522	3.542	3.593	3.575	3.593	3.593	0.018	0.000	0.000
Murramarang Road Local Catchments														
Murramarang Road_40m south of Binnowee Place	8.757	8.809	0.000	8.754	0.000	0.000	0.000	8.809	8.757	8.809	8.809	0.052	0.000	0.000
Murramarang Road_Near Weemala Cres	5.438	5.475	0.000	5.435	0.000	0.000	0.000	5.475	5.438	5.475	5.475	0.037	0.000	0.000
Murramarang Road_Near Wonnawong place	11.078	11.123	11.059	11.076	11.043	11.039	11.063	11.123	11.078	11.123	11.123	0.046	0.000	0.000
Murramarang Road_90m north of Malibu Dr	9.150	9.164	9.145	9.151	0.000	0.000	9.145	9.164	9.151	9.164	9.164	0.013	0.000	0.000
Murramarang Road_60m south of Forster Dr	4.293	4.305	4.264	4.280	4.227	4.247	4.275	4.305	4.293	4.305	4.305	0.012	0.000	0.000
Murramarang Road_90m north of Voyager Cres	3.852	3.854	3.843	3.846	3.831	3.837	3.846	3.854	3.852	3.854	3.854	0.002	0.000	0.000
Murramarang Road_Upstream of Limpid Lagoon	6.065	6.048	0.000	6.022	0.000	0.000	0.000	6.065	6.065	6.048	6.048	0.000	0.017	0.017
Murramarang Road_160m north of Bundle Hill Rd	5.065	5.066	5.047	5.054	5.028	5.039	5.052	5.066	5.065	5.066	5.066	0.001	0.000	0.000
Murramarang Road_Near Bundle Hill Rd	6.610	6.610	6.601	6.604	6.594	6.597	6.604	6.610	6.610	6.610	6.610	0.000	0.000	0.000
Murramarang Road_Adjacent to Tasman Holiday Park	6.154	6.153	6.140	6.147	6.124	6.136	6.145	6.154	6.154	6.153	6.153	0.000	0.001	0.001
Murramarang Road_Butlers Creek northern tributary, 130m north of Moores Rd	4.091	4.098	4.051	4.071	0.000	4.037	4.062	4.098	4.091	4.098	4.098	0.007	0.000	0.000
Murramarang Road_Butlers Creek northern tributary, 120m south of Moores Rd	3.209	3.222	3.202	3.207	3.190	3.194	3.204	3.222	3.209	3.222	3.222	0.013	0.000	0.000
Murramarang Road_Butlers Creek Bridge	3.189	3.196	3.095	3.144	0.000	2.985	3.036	3.196	3.189	3.196	3.196	0.007	0.000	0.000
Murramarang Road_115m south of O'Hara Street	6.627	6.629	6.598	6.608	6.562	6.584	6.605	6.629	6.627	6.629	6.629	0.002	0.000	0.000
Murramarang Road_90m north of Scerri Dr	7.150	7.154	7.134	7.140	7.121	7.129	7.138	7.154	7.150	7.154	7.154	0.003	0.000	0.000
Pretty Beach Road_160m north of Pretty Beach Campground	9.571	9.568	9.530	9.545	9.485	9.514	9.538	9.571	9.571	9.568	9.568	0.000	0.003	0.003
Merry Beach Road_Northern tributary culverts	3.261	3.234	3.060	3.171	2.882	3.011	3.079	3.261	3.261	3.234	3.234	0.000	0.028	0.028
Merry Beach Road_Bridge to Ingenia Holidays	3.201	3.182	0.000	3.134	0.000	0.000	0.000	3.201	3.201	3.182	3.182	0.000	0.019	0.019
Tasman Holiday Park_Internal bridge	3.331	3.328	3.252	3.316	3.131	3.204	3.285	3.331	3.331	3.328	3.328	0.000	0.003	0.003
											Max. Diff.	**0.052**	**0.028**	**0.028**
											Avg. Diff.	**0.009**	**0.005**	**0.004**

FIGURE A-3





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DIFFERENCE IN 1% AEP PEAK FLOOD LEVEL ENVELOPES FOR ALL DURATIONS AND REPRESENTATIVE DURATION COMBINATIONS

TUFLOW Assessment of Critical Storm Durations for the 5% AEP and PMF Events

The analysis was repeated for the 5% AEP and PMF events with the findings as follows.

5% AEP event

- WBNM hydrologic model assessment
  - A critical duration of 360 minutes was identified for Willinga Lake, with similar average peak flows for durations from 270 minutes to 540 minutes.
  - A critical duration of 120 minutes was identified for BPR-1 and the majority of other local watercourses.
- TUFLOW hydraulic model assessment
  - Willinga Lake
    - The 270 minute storm produced maximum peak flood levels upstream of Bawley Point Road and the 120 minute storm produced maximum peak flood levels downstream of Bawley Point Road.
    - Adopting the 360 minute storm rather than the 270 minute storm would have resulted in a peak flood level 20 mm lower in the main body of Willinga Lake and at Bawley Point Road. The implications of this would not have been significant.
    - It is evident that, in frequent to intermediate floods, water levels in Willinga Lake downstream of Bawley Point Road peak soon after entrance breakout begins and thus rainfall intensity and temporal pattern can be of greater importance than total rainfall volume.
  - Local catchments
    - The 120 minute storm produced maximum peak flood levels at all road over-topping locations in the Willinga Lake, Bawley Point Road and Murramarang Road local catchments.
    - The 540 minute storm was critical in a number of storage areas downstream of Murramarang Road, however flooding from these storages does not back up across the road and thus is not key to the study objectives.

PMF event

- WBNM hydrologic model assessment
  - A critical duration of 180 minutes was identified for Willinga Lake, with similar peak flows for durations from 120 minutes to 270 minutes.
  - A critical duration of 60 minutes was identified for BPR-1 and most local watercourses.
- TUFLOW hydraulic model assessment
  - Willinga Lake
    - The 180 minute storm produced maximum peak flood levels throughout the lake.
- Local catchments
    - The 60 minute storm produced maximum peak flood levels at the majority of road over-topping locations in the Willinga Lake, Bawley Point Road and Murramarang Road local catchments.

Additional analysis of the 50% AEP event was also undertaken using TUFLOW and confirmed that, for frequent to intermediate flood events, it is appropriate to adopt a slightly shorter critical storm duration for Willinga Lake than that indicated by the WBNM model.

Conclusions

While the TUFLOW analysis allowed for minor refinement of selected critical storm durations for the 5% and 1% AEP events, overall it confirmed that the storms identified by the WBNM hydrologic model would otherwise have produced an appropriate estimate of design peak flood levels across the study area. Any differences that resulted from assessment and refinement using the TUFLOW model were minor.

Nonetheless, the findings of the TUFLOW analysis were considered in the selection of critical storm duration combinations as indicated in **Table A- 2**. This involved selection of slightly shorter critical durations for Willinga Lake for the 50% to 5% AEP storms (*which coincide with the comparatively low HHWS(SS) tailwater and are heavily influenced by entrance breakout*), and selection of slightly longer critical durations the 2% and 1% AEP storms (*where the flood peak coincides with that of a 5% AEP ocean level and entrance breakout has less influence*).

Table A- 2 WBNM based and final selected critical storm durations and temporal patterns

Design Flood Event	WBNM Based Assessment		Final Selection in Consideration of TUFLOW Findings	
	Critical Storm Duration (min)	Temporal Pattern	Critical Storm Duration (min)	Temporal Pattern
50% AEP	180 720	6087 (TP4) 6212 (TP5)	180 360	6087 (TP4) 6152 (TP7)
20% AEP	180 540	6094 (TP9) 6186 (TP4)	180 360	6094 (TP9) 6152 (TP7)
10% AEP	120 540	6043 (TP6) 6178 (TP7)	120 270	6043 (TP6) 6111 (TP5)
5% AEP	120 360	6043 (TP6) 6141 (TP6)	120 270	6043 (TP6) 6111 (TP5)
2% AEP	120 270	5864 (TP1) 6101 (TP7)	120 540	5864 (TP1) 6163 (TP5)
1% AEP	90 270	5999 (TP7) 6101 (TP7)	120 540	5864 (TP1) 6156 (TP4)
1 in 200 AEP	90 720	5999 (TP7) 6157 (TP4)	120 720	6036 (TP8) 6157 (TP4)
1 in 500 AEP	90 720	5999 (TP7) 6157 (TP4)	120 720	6036 (TP8) 6157 (TP4)
PMF	60 180	GSDM	60 180	GSDM

A.3 Comparison of Peak Flows from Selected Representative Storms to 'At Site' Critical Duration Storms

A comparison of peak design flood flows from the selected storm duration and temporal pattern combinations in **Table A- 2** with the average peak flow from the temporal pattern ensemble at a range of assessment locations is presented in **Table A- 3** to **Table A- 11** from events from the 50% AEP to PMF.

These comparisons assisted in selecting two storm durations and temporal patterns for each flood magnitude that in combination provided the overall best match to 'average peak design flows' across the assessment locations, with consideration also given to the additional peak flood level analysis undertaken using TUFLOW.

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%).

The selected representative storms did result in underestimation of peak flows at minor watercourses / storage areas downstream of Murramarang Road where critical storm durations were longer than those selected, particularly at Cormorant Beach and Limpid Lagoon (Murramarang Beach south). However, the TUFLOW critical duration analysis found that long duration storm flooding from these storages does not back up across the road (*refer* **Figure A- 2**) and thus is not key to the study objectives. Rather, it was found that design event flooding of Murramarang Road is generally caused by shorter duration storms such as the 120 minute storm.

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 (and peak flood levels indicated by the TUFLOW analysis) 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.

Table A- 3 50% AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	WBNM At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	720	28.4	6212 (TP5)	30.4	+6.9%	720	6212 (TP5)	30.4	+6.9%	360	6152 (TP7)	25.7	-9.4%
17	Willinga Road Bridge	180	10.9	6087 (TP4)	11.3	+4.3%	180	6087 (TP4)	11.3	+4.3%	180	6087 (TP4)	11.3	+4.3%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	180	10.2	6087 (TP4)	11.1	+8.1%	180	6087 (TP4)	11.1	+8.1%	180	6087 (TP4)	11.1	+8.1%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	180	10.9	6087 (TP4)	11.7	+7.6%	180	6087 (TP4)	11.7	+7.6%	180	6087 (TP4)	11.7	+7.6%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	180	14.4	6087 (TP4)	15.5	+7.8%	180	6087 (TP4)	15.5	+7.8%	180	6087 (TP4)	15.5	+7.8%
8	Cormorant Beach outlet	2880	0.6	6377 (TP7)	0.5	-14.0%	720	6212 (TP5)	0.6	-3.5%	360	6152 (TP7)	0.4	-31.6%
9	Gannet Beach outlet	540	1.0	6188 (TP6)	1.0	0.0%	720	6212 (TP5)	1.0	-4.9%	360	6152 (TP7)	1.0	-1.9%
10	Murramarang Beach north	540	2.9	6186 (TP4)	3.0	+6.3%	720	6212 (TP5)	2.9	0.0%	360	6152 (TP7)	2.8	-0.7%
11	Murramarang Beach south	720	1.2	6214 (TP7)	1.3	+7.4%	720	6212 (TP5)	1.5	+22.1%	360	6152 (TP7)	0.9	-30.3%
12	Racecourse Beach north	540	4.0	6188 (TP6)	4.1	+2.8%	720	6212 (TP5)	3.7	-6.3%	360	6152 (TP7)	4.3	+8.1%
13	Racecourse Beach south	180	2.0	6095 (TP10)	2.2	+9.0%	180	6087 (TP4)	2.2	+7.5%	180	6087 (TP4)	2.2	+7.5%
14	Butlers Creek outlet	540	10.1	6186 (TP4)	10.1	+0.1%	720	6212 (TP5)	9.8	-2.9%	360	6152 (TP7)	9.6	-5.2%
15	Kioloa Beach outlet	180	4.8	6087 (TP4)	5.2	+7.1%	180	6087 (TP4)	5.2	+7.1%	180	6087 (TP4)	5.2	+7.1%
16	Merry Beach outlet	180	5.3	6087 (TP4)	5.6	+4.9%	180	6087 (TP4)	5.6	+4.9%	180	6087 (TP4)	5.6	+4.9%

Table A- 4 20% AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	WBNM At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	540	51.9	6186 (TP4)	55.7	+7.2%	540	6186 (TP4)	55.7	+7.2%	360	6152 (TP7)	52.0	+0.2%
17	Willinga Road Bridge	180	19.9	6094 (TP9)	20.9	+5.4%	180	6094 (TP9)	20.9	+5.4%	180	6094 (TP9)	20.9	+5.4%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	180	17.9	6094 (TP9)	19.0	+6.1%	180	6094 (TP9)	19.0	+6.1%	180	6094 (TP9)	19.0	+6.1%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	180	19.0	6094 (TP9)	20.3	+6.9%	180	6094 (TP9)	20.3	+6.9%	180	6094 (TP9)	20.3	+6.9%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	180	25.3	6094 (TP9)	27.0	+6.5%	180	6094 (TP9)	27.0	+6.5%	180	6094 (TP9)	27.0	+6.5%
8	Cormorant Beach outlet	720	1.3	6214 (TP7)	1.3	-2.2%	540	6186 (TP4)	1.3	-3.7%	360	6152 (TP7)	0.9	-35.8%
9	Gannet Beach outlet	270	2.0	6124 (TP5)	2.0	-0.5%	540	6186 (TP4)	1.8	-9.6%	360	6152 (TP7)	2.3	+13.6%
10	Murramarang Beach north	540	4.2	6186 (TP4)	4.1	-3.3%	540	6186 (TP4)	4.1	-3.3%	360	6152 (TP7)	4.0	-4.5%
11	Murramarang Beach south	540	2.6	6186 (TP4)	2.6	+2.7%	540	6186 (TP4)	2.6	+2.7%	360	6152 (TP7)	2.1	-18.0%
12	Racecourse Beach north	180	7.0	6087 (TP4)	7.4	+5.4%	540	6186 (TP4)	6.9	-0.6%	360	6152 (TP7)	7.2	+3.2%
13	Racecourse Beach south	90	3.5	6024 (TP7)	3.5	0.0%	180	6094 (TP9)	3.3	-4.6%	180	6094 (TP9)	3.3	-4.6%
14	Butlers Creek outlet	540	18.5	6188 (TP6)	19.3	+4.5%	540	6186 (TP4)	21.3	+15.2%	360	6152 (TP7)	17.2	-6.9%
15	Kioloa Beach outlet	180	8.2	6095 (TP10)	8.7	+5.5%	180	6094 (TP9)	8.5	+3.0%	180	6094 (TP9)	8.5	+3.0%
16	Merry Beach outlet	180	9.6	6087 (TP4)	10.3	+7.7%	180	6094 (TP9)	10.2	+7.2%	180	6094 (TP9)	10.2	+7.2%

Table A- 5 10% AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	WBNM At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	540	64.5	6178 (TP7)	63.3	-1.8%	540	6178 (TP7)	63.3	-1.8%	270	6111 (TP5)	62.8	-2.7%
17	Willinga Road Bridge	120	26.1	6046 (TP9)	25.7	-1.7%	120	6043 (TP6)	23.8	-8.8%	270	6111 (TP5)	26.4	+0.9%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	120	24.3	6041 (TP4)	25.3	+4.0%	120	6043 (TP6)	25.0	+2.8%	120	6043 (TP6)	25.0	+2.8%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	120	26.0	6043 (TP6)	27.1	+4.5%	120	6043 (TP6)	27.1	+4.5%	120	6043 (TP6)	27.1	+4.5%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	120	34.6	6043 (TP6)	36.4	+5.1%	120	6043 (TP6)	36.4	+5.1%	120	6043 (TP6)	36.4	+5.1%
8	Cormorant Beach outlet	720	1.5	6202 (TP7)	1.5	+1.3%	540	6178 (TP7)	1.5	-4.6%	270	6111 (TP5)	1.2	-23.0%
9	Gannet Beach outlet	180	2.5	6079 (TP7)	2.5	+0.4%	540	6178 (TP7)	2.3	-5.3%	270	6111 (TP5)	2.7	+8.9%
10	Murramarang Beach north	540	5.9	6178 (TP7)	5.9	-1.0%	540	6178 (TP7)	5.9	-1.0%	270	6111 (TP5)	6.2	+4.9%
11	Murramarang Beach south	540	3.5	6178 (TP7)	3.6	+2.9%	540	6178 (TP7)	3.6	+2.9%	270	6111 (TP5)	2.7	-22.6%
12	Racecourse Beach north	120	9.1	6046 (TP9)	8.9	-1.5%	540	6178 (TP7)	9.0	-0.8%	270	6111 (TP5)	9.7	+6.9%
13	Racecourse Beach south	120	4.8	6043 (TP6)	4.9	+2.5%	120	6043 (TP6)	4.9	+2.5%	120	6043 (TP6)	4.9	+2.5%
14	Butlers Creek outlet	360	25.8	6142 (TP7)	27.3	+6.0%	540	6178 (TP7)	25.3	-2.0%	270	6111 (TP5)	26.4	+2.4%
15	Kioloa Beach outlet	120	11.4	6042 (TP5)	11.9	+4.9%	120	6043 (TP6)	11.8	+3.7%	120	6043 (TP6)	11.8	+3.7%
16	Merry Beach outlet	120	13.1	6041 (TP4)	13.7	+4.7%	120	6043 (TP6)	13.1	+0.2%	120	6043 (TP6)	13.1	+0.2%

Table A- 6 5% AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	WBNM At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	360	75.8	6141 (TP6)	75.5	-0.5%	360	6141 (TP6)	75.5	-0.5%	270	6111 (TP5)	74.3	-2.0%
17	Willinga Road Bridge	120	33.8	6041 (TP4)	33.2	-1.6%	360	6141 (TP6)	35.9	+6.2%	270	6111 (TP5)	34.0	+0.6%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	120	30.6	6043 (TP6)	32.0	+4.5%	120	6043 (TP6)	32.0	+4.5%	120	6043 (TP6)	32.0	+4.5%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	120	32.6	6043 (TP6)	33.7	+3.3%	120	6043 (TP6)	33.7	+3.3%	120	6043 (TP6)	33.7	+3.3%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	120	43.4	6043 (TP6)	45.0	+3.6%	120	6043 (TP6)	45.0	+3.6%	120	6043 (TP6)	45.0	+3.6%
8	Cormorant Beach outlet	2880	1.8	6240 (TP2)	1.8	-0.6%	360	6141 (TP6)	1.7	-4.5%	270	6111 (TP5)	1.6	-11.3%
9	Gannet Beach outlet	180	2.8	6073 (TP4)	2.8	+1.4%	360	6141 (TP6)	2.8	+1.8%	270	6111 (TP5)	2.9	+4.7%
10	Murramarang Beach north	540	7.8	6181 (TP9)	8.0	+2.0%	360	6141 (TP6)	8.5	+8.9%	270	6111 (TP5)	8.1	+3.6%
11	Murramarang Beach south	540	4.6	6178 (TP7)	4.5	-1.1%	360	6141 (TP6)	3.9	-14.2%	270	6111 (TP5)	3.9	-14.8%
12	Racecourse Beach north	120	11.6	6046 (TP9)	11.4	-1.8%	360	6141 (TP6)	12.4	+6.7%	270	6111 (TP5)	11.8	+1.9%
13	Racecourse Beach south	120	6.0	6043 (TP6)	6.0	+0.7%	120	6043 (TP6)	6.0	+0.7%	120	6043 (TP6)	6.0	+0.7%
14	Butlers Creek outlet	180	33.8	6079 (TP7)	33.7	-0.4%	360	6141 (TP6)	42.5	+25.7%	270	6111 (TP5)	40.1	+18.6%
15	Kioloa Beach outlet	120	14.3	6042 (TP5)	14.9	+4.3%	120	6043 (TP6)	14.8	+3.4%	120	6043 (TP6)	14.8	+3.4%
16	Merry Beach outlet	120	15.9	6044 (TP7)	16.6	+4.8%	120	6043 (TP6)	16.4	+3.2%	120	6043 (TP6)	16.4	+3.2%

Table A- 7 2% AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	WBNM At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	270	94.4	6101 (TP7)	95.2	+0.9%	270	6101 (TP7)	95.2	+0.9%	540	6163 (TP5)	95.0	+0.7%
17	Willinga Road Bridge	120	44.1	6036 (TP8)	44.4	+0.9%	120	5864 (TP1)	44.3	+0.6%	120	5864 (TP1)	44.3	+0.6%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	120	39.4	5864 (TP1)	40.1	+1.8%	120	5864 (TP1)	40.1	+1.8%	120	5864 (TP1)	40.1	+1.8%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	120	41.9	5864 (TP1)	42.2	+0.8%	120	5864 (TP1)	42.2	+0.8%	120	5864 (TP1)	42.2	+0.8%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	120	55.8	5864 (TP1)	56.3	+0.9%	120	5864 (TP1)	56.3	+0.9%	120	5864 (TP1)	56.3	+0.9%
8	Cormorant Beach outlet	1440	2.5	6251 (TP6)	2.4	-2.0%	270	6101 (TP7)	1.8	-26.1%	540	6163 (TP5)	1.7	-31.0%
9	Gannet Beach outlet	180	3.2	6031 (TP2)	3.2	+0.3%	120	5864 (TP1)	3.3	+3.5%	120	5864 (TP1)	3.3	+3.5%
10	Murramarang Beach north	540	10.7	6167 (TP7)	10.7	+0.3%	270	6101 (TP7)	9.1	-15.4%	540	6163 (TP5)	10.2	-5.0%
11	Murramarang Beach south	720	6.3	6017 (TP2)	6.4	+1.1%	270	6101 (TP7)	5.4	-14.0%	540	6163 (TP5)	6.9	+9.7%
12	Racecourse Beach north	180	15.1	6031 (TP2)	14.8	-2.4%	120	5864 (TP1)	15.0	-1.1%	120	5864 (TP1)	15.0	-1.1%
13	Racecourse Beach south	90	7.7	5999 (TP7)	7.7	-0.5%	120	5864 (TP1)	7.8	+1.4%	120	5864 (TP1)	7.8	+1.4%
14	Butlers Creek outlet	180	48.4	6066 (TP8)	51.6	+6.6%	120	5864 (TP1)	47.8	-1.1%	120	5864 (TP1)	47.8	-1.1%
15	Kioloa Beach outlet	90	18.3	6002 (TP8)	18.4	+0.8%	120	5864 (TP1)	18.1	-1.1%	120	5864 (TP1)	18.1	-1.1%
16	Merry Beach outlet	120	20.1	5864 (TP1)	20.0	-0.3%	120	5864 (TP1)	20.0	-0.3%	120	5864 (TP1)	20.0	-0.3%

Table A- 8 1% AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	WBNM At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	270	104.2	6101 (TP7)	104.8	+0.6%	270	6101 (TP7)	104.8	+0.6%	540	6156 (TP4)	105.0	+0.7%
17	Willinga Road Bridge	120	54.4	6036 (TP8)	54.3	-0.2%	90	5999 (TP7)	51.8	-4.8%	120	6036 (TP8)	54.3	-0.2%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	120	47.7	5864 (TP1)	48.8	+2.3%	90	5999 (TP7)	47.2	-1.0%	120	6036 (TP8)	51.3	+7.6%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	120	50.5	6029 (TP3)	50.0	-1.0%	90	5999 (TP7)	50.5	-0.1%	120	6036 (TP8)	51.4	+1.7%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	90	67.3	5999 (TP7)	67.4	+0.2%	90	5999 (TP7)	67.4	+0.2%	540	6156 (TP4)	67.6	+0.5%
8	Cormorant Beach outlet	1440	3.4	6251 (TP6)	3.4	0.0%	270	6101 (TP7)	2.1	-38.8%	540	6156 (TP4)	4.2	+23.5%
9	Gannet Beach outlet	270	3.7	6099 (TP6)	3.7	+1.1%	270	6101 (TP7)	3.6	-2.2%	540	6156 (TP4)	3.5	-4.9%
10	Murramarang Beach north	180	12.8	6066 (TP8)	13.7	+6.8%	270	6101 (TP7)	12.0	-6.2%	540	6156 (TP4)	13.6	+6.1%
11	Murramarang Beach south	720	7.4	6017 (TP2)	7.5	+0.5%	270	6101 (TP7)	6.7	-9.6%	540	6156 (TP4)	8.3	+11.3%
12	Racecourse Beach north	120	18.6	6036 (TP8)	18.6	+0.3%	90	5999 (TP7)	17.7	-4.7%	120	6036 (TP8)	18.6	+0.3%
13	Racecourse Beach south	60	9.3	5970 (TP9)	9.2	-0.8%	90	5999 (TP7)	9.4	+1.5%	120	6036 (TP8)	9.9	+6.5%
14	Butlers Creek outlet	120	59.3	6036 (TP8)	60.0	+1.2%	90	5999 (TP7)	55.6	-6.3%	120	6036 (TP8)	60.0	+1.2%
15	Kioloa Beach outlet	90	22.2	5999 (TP7)	22.2	+0.0%	90	5999 (TP7)	22.2	+0.0%	120	6036 (TP8)	23.9	+7.8%
16	Merry Beach outlet	120	24.7	5864 (TP1)	24.4	-1.0%	90	5999 (TP7)	24.8	+0.5%	120	6036 (TP8)	25.4	+3.2%

Table A- 9 1 in 200 AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	720	114.2	6157 (TP4)	109.7	-3.9%	720	6157 (TP4)	109.7	-3.9%	720	6157 (TP4)	109.7	-3.9%
17	Willinga Road Bridge	120	63.1	6036 (TP8)	62.5	-1.0%	90	5999 (TP7)	62.5	-0.9%	120	6036 (TP8)	62.5	-1.0%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	90	54.7	6002 (TP8)	54.8	+0.2%	90	5999 (TP7)	54.7	+0.0%	120	6036 (TP8)	58.9	+7.7%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	90	58.3	5999 (TP7)	58.2	-0.2%	90	5999 (TP7)	58.2	-0.2%	120	6036 (TP8)	60.7	+4.2%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	90	78.1	5999 (TP7)	78.0	-0.2%	90	5999 (TP7)	78.0	-0.2%	120	6036 (TP8)	81.5	+4.3%
8	Cormorant Beach outlet	720	4.1	6017 (TP2)	4.0	-2.2%	720	6157 (TP4)	5.2	+26.6%	720	6157 (TP4)	5.2	+26.6%
9	Gannet Beach outlet	270	4.2	6101 (TP7)	4.3	+3.1%	720	6157 (TP4)	4.5	+7.4%	720	6157 (TP4)	4.5	+7.4%
10	Murramarang Beach north	180	14.9	6066 (TP8)	15.9	+6.9%	720	6157 (TP4)	15.8	+5.9%	720	6157 (TP4)	15.8	+5.9%
11	Murramarang Beach south	720	8.5	6017 (TP2)	8.5	0.0%	720	6157 (TP4)	9.5	+10.7%	720	6157 (TP4)	9.5	+10.7%
12	Racecourse Beach north	120	21.2	6036 (TP8)	21.2	+0.0%	90	5999 (TP7)	20.5	-3.4%	120	6036 (TP8)	21.2	+0.0%
13	Racecourse Beach south	60	10.8	5970 (TP9)	10.7	-0.7%	90	5999 (TP7)	11.0	+2.1%	120	6036 (TP8)	11.2	+4.3%
14	Butlers Creek outlet	120	68.6	6036 (TP8)	69.3	+1.0%	90	5999 (TP7)	67.1	-2.1%	120	6036 (TP8)	69.3	+1.0%
15	Kioloa Beach outlet	90	25.6	5999 (TP7)	25.5	-0.4%	90	5999 (TP7)	25.5	-0.4%	120	6036 (TP8)	27.7	+8.2%
16	Merry Beach outlet	90	29.0	5999 (TP7)	29.8	+2.7%	90	5999 (TP7)	29.8	+2.7%	120	6036 (TP8)	29.5	+1.5%

Table A- 10 1 in 500 AEP comparison of WBNM peak design flood flows from average of temporal pattern ensemble and selected representative temporal patterns

WBNM subarea	Location	At Site Critical Durations and Patterns					WBNM Representative Durations and Patterns				Final Representative Durations and Patterns in Consideration of TUFLOW Analysis			
		Critical Duration (min)	Averaged Peak Flow (m^3/s)	'Average' Pattern No.	Av. Patt. Peak Flow (m^3/s)	% Difference (Avg & Patt Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)	Selected Duration (min)	Selected Pattern No.	Selected Patt. Peak Flow (m^3/s)	% Difference (Avg & Selected Peak Flow)
2	Willinga Lake outlet	720	129.7	6157 (TP4)	126.9	-2.2%	720	6157 (TP4)	126.9	-2.2%	720	6157 (TP4)	126.9	-2.2%
17	Willinga Road Bridge	120	76.0	5864 (TP1)	77.8	+2.3%	90	5999 (TP7)	75.9	-0.2%	120	6036 (TP8)	80.9	+6.4%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	90	65.8	6002 (TP8)	65.8	-0.0%	90	5999 (TP7)	65.6	-0.3%	120	6036 (TP8)	70.9	+7.8%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	90	70.1	5999 (TP7)	69.8	-0.5%	90	5999 (TP7)	69.8	-0.5%	120	6036 (TP8)	75.9	+8.3%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	90	94.3	5999 (TP7)	93.9	-0.4%	90	5999 (TP7)	93.9	-0.4%	120	6036 (TP8)	102.2	+8.4%
8	Cormorant Beach outlet	720	4.9	6017 (TP2)	4.8	-2.4%	720	6157 (TP4)	5.6	+13.2%	720	6157 (TP4)	5.6	+13.2%
9	Gannet Beach outlet	270	5.2	6063 (TP2)	5.3	+1.2%	720	6157 (TP4)	5.5	+5.6%	720	6157 (TP4)	5.5	+5.6%
10	Murramarang Beach north	180	17.8	6066 (TP8)	18.8	+6.0%	720	6157 (TP4)	18.2	+2.3%	720	6157 (TP4)	18.2	+2.3%
11	Murramarang Beach south	720	10.1	6017 (TP2)	10.0	-0.8%	720	6157 (TP4)	10.9	+7.9%	720	6157 (TP4)	10.9	+7.9%
12	Racecourse Beach north	120	25.3	6036 (TP8)	25.2	-0.3%	90	5999 (TP7)	24.5	-3.3%	120	6036 (TP8)	25.2	-0.3%
13	Racecourse Beach south	60	12.9	5970 (TP9)	12.7	-1.4%	90	5999 (TP7)	13.5	+4.2%	120	6036 (TP8)	13.3	+3.1%
14	Butlers Creek outlet	120	82.2	6035 (TP7)	82.5	+0.3%	90	5999 (TP7)	81.1	-1.4%	120	6036 (TP8)	83.0	+1.0%
15	Kioloa Beach outlet	90	30.6	5999 (TP7)	30.4	-0.9%	90	5999 (TP7)	30.4	-0.9%	120	6036 (TP8)	33.3	+8.6%
16	Merry Beach outlet	90	35.5	5999 (TP7)	36.2	+1.9%	90	5999 (TP7)	36.2	+1.9%	120	6036 (TP8)	36.7	+3.2%

Table A- 11 PMF comparison of WBNM 'at site' peak design flood flows and selected critical durations

WBNM subarea	Location	WBNM At Site Critical Durations		Final Representative Durations in Consideration of TUFLOW Analysis		
		Critical Duration (min)	Peak Flow (m^3/s)	Selected Duration (min)	Selected Dur'n Peak Flow (m^3/s)	% Difference
2	Willinga Lake outlet	180	345.9	180	345.9	0.0%
17	Willinga Road Bridge	60	233.7	60	233.7	0.0%
21	Tributary discharging to Willinga Lake north-west of Willinga Park	60	210.5	60	210.5	0.0%
92	Tributary discharging to Willinga Lake north-east of Willinga Park	60	222.8	60	222.8	0.0%
3	Bawley Point Road 400m south-east of Princes Hwy (BPR-1)	60	298.7	60	298.7	0.0%
8	Cormorant Beach outlet	270	14.3	180	13.8	-3.8%
9	Gannet Beach outlet	120	13.7	180	13.5	-1.4%
10	Murramarang Beach north	120	52.6	180	50.0	-4.8%
11	Murramarang Beach south	120	34.5	180	33.0	-4.3%
12	Racecourse Beach north	90	81.2	60	77.4	-4.6%
13	Racecourse Beach south	60	40.0	60	40.0	0.0%
14	Butlers Creek outlet	90	265.4	60	252.2	-5.0%
15	Kioloa Beach outlet	60	89.9	60	89.9	0.0%
16	Merry Beach outlet	60	109.0	60	109.0	0.0%

Appendix B. TUFLOW Bridge and Culvert Details

Table B- 1 Details of TUFLOW 2D 'lfcsh' bridges

Road / Location	Catchment ID	Watercourse	Pier Arrangement	Pier Blockage (% of opening width)	Pier Form Loss Coefficient (ΔKp)	Bridge Deck Obvert (mAHD)	Bridge Deck Thickness (m)	Bridge Deck RL (mAHD)	Railing	Height (m)	Estimated Blockage (%)	Data Source
Princes Highway		Reedy Creek	Open span bridge	-	-	11.46	1.00	12.46	Permeable safety barrier	1.20	50.0	Estimated from Google Street View
Old Princes Highway		Reedy Creek	Open span bridge	-	-	11.30	1.00	12.30	Permeable safety barrier	1.20	60.0	Estimated from Google Street View
Bawley Point Road		Willinga Lake	Two rows of 0.36 m wide square piers	2.67	0.02	2.05	0.69	2.74	Permeable handrail	0.91	25.0	Details from layout plans
Willinga Road		Willinga Lake tributary	Open span bridge	-	-	2.56	0.69	3.25	Concrete safety notches	0.24	50.0	CEH survey 2024
Forster Road		Willinga Lake tributary	Open span bridge	-	-	5.11	0.28	5.39	Concrete barrier	0.11	95.0	CEH survey 2024
Tasman Holiday Park Racecourse Beach	MR-5	Racecourse Beach	Open span bridge	-	-	1.90	0.50	2.40	Permeable handrail	1.00	12.5	CEH survey 2024
Murramarang Road	MR-7	Butlers Creek	Open span bridge	-	-	2.34	0.80	3.14	Permeable handrail	1.02	12.5	CEH survey 2024
Ingenia Holidays Merry Beach	MR-8	Merry Beach	Open span bridge	-	-	2.5 to 2.8	0.45	2.95 to 3.25	Wooden barrier	0.15	90.0	CEH survey 2024

Table B- 2 Details of TUFLOW 1D culverts 600 mm diameter and larger

TUFLOW ID	SCC Asset ID	Road	Catchment ID	Watercourse / Beach	Box or Circular	No. of Barrels	Width or Diameter (m)	Height (m)	US Invert (mAHD)	DS Invert (mAHD)	Data Source
CEH01		Princes Highway		Reedy Creek	Box	2	2.55	2.10	8.84	8.71	CEH survey 2024
CEH02		Princes Highway		Reedy Creek tributary	Circular	1	1.00	-	8.95	8.58	CEH survey 2024
CEH03		Princes Highway		Reedy Creek tributary	Box	2	2.35	1.54	7.37	7.30	CEH survey 2024
CEH04		Bawley Point Road	BPR-1	Reedy Creek tributary	Circular	3	1.25	-	4.72	4.54	CEH survey 2024
CEH05		Bawley Point Road		Reedy Creek tributary	Circular	3	1.00	-	6.68	6.42	CEH survey 2024
CEH06		Bawley Point Road		Reedy Creek tributary	Circular	2	1.00	-	5.91	5.89	CEH survey 2024
CEH07		Bawley Point Road	BPR-2	Reedy Creek tributary	Circular	3	1.00	-	2.16	1.97	CEH survey 2024
CEH08a		Bawley Point Road		Reedy Creek tributary	Circular	2	0.80	-	6.92	6.56	CEH survey 2024
CEH08b		Bawley Point Road		Reedy Creek tributary	Circular	2	0.75	-	7.45	7.25	CEH survey 2024
CEH24b		Forster Drive		Willinga Lake tributary	Circular	3	0.60	-	4.80	4.26	Number of barrels from Nearmaps. Diameter from CEH25. Inverts from LiDAR minima.
CEH25		Forster Drive		Willinga Lake tributary	Circular	5	0.60	-	6.51	6.24	CEH survey 2024
CEH25b		Forster Drive		Willinga Lake tributary	Circular	3	0.60	-	15.21	14.30	Number of barrels from Nearmaps. Diameter from CEH25. Inverts from LiDAR minima.
CEH12a		Murramarang Road	MR-1	Cormorant Beach	Circular	1	1.00	-	3.97	4.07	CEH survey 2024
CEH13		Malibu Drive	MR-2	Gannet Beach	Circular	2	1.30	-	1.50	1.28	CEH survey 2024
CEH14		Murramarang Road	MR-3	Murramarang Beach	Circular	3	1.00	-	2.41	2.23	CEH survey 2024
CEH15		Murramarang Road	MR-3	Murramarang Beach	Circular	2	0.60	-	2.97	2.96	CEH survey 2024

TUFLOW ID	SCC Asset ID	Road	Catchment ID	Watercourse / Beach	Box or Circular	No. of Barrels	Width or Diameter (m)	Height (m)	US Invert (mAHD)	DS Invert (mAHD)	Data Source
CEH16	11028268	Murramarang Road	MR-4	Limpid Lagoon	Box	1	1.50	0.90	4.59	4.53	CEH survey 2024
11028269	11028269	Murramarang Road	MR-5	Racecourse Beach	Box	1	1.50	0.90	3.85	2.91	Dimensions based on SCC asset 11028268. Inverts from LiDAR minima.
BundleHill		Murramarang Road	MR-5	Racecourse Beach	Box	1	1.50	0.90	6.17	5.71	Dimensions based on SCC asset 11028268. Inverts from LiDAR minima.
CEH19	11028270	Murramarang Road	MR-6	Butlers Creek northern tributary	Circular	3	1.50	-	2.13	1.96	CEH survey 2024
CEH22		Murramarang Road		Kioloa Beach	Circular	2	0.75	-	4.75	4.75	CEH survey 2024