“Where will our knowledge take you?”

Review and Update Manning River Flood Study

Final Report

April 2016

Manning River Flood Study

Prepared for:	Greater Taree City Council
Prepared by:	BMT WBM Pty Ltd (Member of the BMT group of companies)

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Executive Summary

Introduction

The Manning River Flood Study has been prepared for Greater Taree City Council (Council) to define the existing flood behaviour in the catchment and establish the basis for subsequent floodplain management activities.

The primary objective of the Flood Study is to define the flood behaviour within the Manning River catchment through the establishment of appropriate numerical models. The study has produced information on flood flows, velocities, levels and extents for a range of flood event magnitudes under existing catchment and floodplain conditions. Specifically, the study incorporates:

- Compilation and review of existing information pertinent to the study;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design event including the 20% AEP, 5% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF event; and
- Presentation of study methodology, results and findings in a comprehensive report incorporating appropriate flood mapping.

Catchment Description

This study focuses on the Manning River catchment area downstream of Wingham.

Downstream of Taree, the Manning River splits into two arms and enters the ocean at two locations; Harrington and Farquhar Inlet, which is located just north of the Old Bar township. Both entrances are dynamic. Farquhar Inlet can become severely restricted and is known to have closed on many occasions historically. The entrance at Harrington is a permanently open but can become significantly shoaled, particularly in periods between large floods.

The Great Dividing Range forms the upper limit of the Manning River catchment, where elevations of around 1200m AHD are typical. The Barrington Tops, located in the south-west of the catchment, peaks at just below 1600m AHD. The Manning River spills onto a vast, low-lying floodplain (elevated to less than 2m AHD) area downstream of Taree.

Land use within the catchment largely consists of forested areas or pastureland and other cultivated areas. There is little urban development within the catchment.

The towns of Tinonee, Taree, Cundletown, Harrington, and Manning Point, among others, are located within the study area. Taree is the largest of these and has a population of around 20,000.

Historical Flooding

Significant flooding has occurred in the catchment since records began some 185 years ago. The latter end of the 19th century saw numerous large floods occurring in the catchment, with half of the largest ten floods on record occurring between 1866 and 1895. The following flood event in February 1929, with notable flood events also occurred in 1930, 1956, 1978 and 1990. After a relatively flood-free period throughout the remainder of the 1990s and the early 2000s, two large events occurred recently in June 2011 and March 2013.

Due to the large size of the catchment and spatial variation in rainfall, the relative magnitude of historical flood events is not necessarily the same across the whole catchment area. At Taree (Macquarie Street), 1929 event resulted in the highest flood on record, with a peak level of 5.6m AHD. Peak flood levels of 5.45m AHD and 5.15m AHD were recorded during the 1978 and 1866 events respectively, and make up the second and third highest levels on record.

Community Consultation

Community consultation has been an important component of the current study. The consultation has aimed to inform the community about the development of the flood study and its likely outcome as a precursor to subsequent floodplain management activities. It has provided an opportunity to collect information on their flood experience and their concerns on flooding issues.

The key elements of the consultation process have been as follows:

- Questionnaire available to be completed by landowners, residents and businesses within the study area;
- An information session for the community to present information on the progress and objectives of the flood study and obtain feedback on historical events in the catchment and other flooding issues; and
- Public exhibition of the draft Flood Study.

Model Development

Development of hydrologic and hydraulic models has been undertaken to simulate flood conditions in the catchment. The hydrological model developed using XP-RAFTS software provides for simulation of the rainfall-runoff process using the catchment characteristics of the Manning River and historical and design rainfall data. The hydraulic model, simulating flood depths, extents and velocities utilises the TUFLOW two-dimensional (2D) software developed by BMT WBM. The 2D modelling approach is suited to model the complex interaction between channels and floodplains and converging and diverging of flows through structures and urban environments.

Two hydraulic models were developed for this study:

- A TULFOW Classic model was developed to provide a two-dimensional (2D) representation of the channel and floodplain of the Manning River extending from Killawarra to the ocean; and
- A TUFLOW-FV model was developed to provide a 2D representation of the ocean entrances at Harrington and Old Bar, with the purpose of simulating the sediment transport processes occurring during flood events.

The floodplain topography is defined using a digital elevation model (DEM) derived from aerial survey data. Bathymetric (hydrographic) survey from the NSW Office of Environment and Heritage (OEH) is available for the tidal reaches of Manning River and its tributaries Custom GIS tools were utilised to interpolate between the cross sections to provide a continuous river bathymetry. This data was integrated with the LiDAR to provide a composite DEM of the channel and floodplain.

With consideration to the available hydrographic survey information and local topographical and hydraulic controls, a 2D model was developed extending from the ocean, through the entrances at

Harrington and Farquhar Inlet and upstream along the north and south arms of the Manning River past Wingham. The upstream extent of the model terminates just upstream of the stream flow gauge location on the Manning River at Killawarra. The hydraulic model incorporates the entire lower Manning River floodplain - a total area of some $580km^2$ which represents around 38% of the total Manning River catchment area.

Model Calibration and Validation

The selection of suitable historical events for calibration of computer models is largely dependent on available historical flood information. Ideally the calibration and validation process should cover a range of flood magnitudes to demonstrate the suitability of a model for the range of design event magnitudes to be considered.

The model calibration is largely based on the June 2011 event, due to the volume of data available. The March 1978 flood event was also used for calibration of the hydraulic model to a larger flood event. The February 1929, February 1990 and March 2013 events were used for model verification.

The focus of the June 2011 model calibration process was essentially to determine the most appropriate set of flow and roughness conditions, in order for the model to be able to reasonably reproduce observed flood behaviour within the catchment. The process adopted for this study involved defining an appropriate rating curve at the Killawarra streamflow gauge to determine the likely peak flow rate during the June 2011 event, which when used in combination with channel roughness, should achieve a good match to water level time series recorded at other downstream locations.

The model parameters adopted for the June 2011 event were used for all other calibration/verification events to confirm their ability to replicate observed flood behaviour in the catchment for a range of historic flood events.

Design Event Modelling and Output

The developed models have been applied to derive design flood conditions within the Manning River catchment. For the study catchments, design floods were based on flood frequency and design rainfall estimates in accordance with the procedures Australian Rainfall and Runoff (IEAust, 2001).

The design events considered in this study include the 20% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF events. The model results for the design events considered have been presented in a detailed flood mapping series for the catchment (see separate Mapping Compendium). The flood data presented includes design flood inundation, peak flood water levels and depths and peak flood velocities.

Provisional flood hazard categorisation in accordance with Figure L2 of the NSW Floodplain Development Manual (2005) has been mapped in addition to the hydraulic categories (floodway, flood fringe and flood storage) for flood affected areas.

Sensitivity Testing

A number of sensitivity tests have been undertaken to identify the impacts of the adopted model conditions on the design flood levels. Sensitivity tests included:

- The impact of potential future climate change, including projected sea level rises and increased rainfall intensities;
- Changes in the adopted roughness parameters;
- Alternate initial entrance geometries; and
- An alternate entrance breakout scenario (entrance geomorphology).

Conclusions

The objective of the study was to undertake a detailed flood study of the Manning River catchment and establish models as necessary for design flood level prediction.

In completing the flood study, the following activities were undertaken:

- Collation of historical and recent flood information for the study area;
- Development of computer models to simulate hydrology and flood behaviour in the catchment;
- Calibration of the developed models using the available flood data, including the recent events of 2011, 2013 and 1990 and the historic events of 1929 and 1978;
- Prediction of design flood conditions in the catchment and production of design flood mapping series.

The main departure of this study from the previous work is the reduction in design peak flood flows. This difference can be attributed to the model parameters adopted for calibration of the hydraulic model and the resulting rating curve at Killawarra that was used to convert recorded levels to flows as inputs into the updated design flood frequency analysis. As a result, this study determined design peak flood levels in the Lower Manning that are typically around 0.2m to 0.5m lower than that of the previous study. For the 1% AEP design flood event, this study determined the peak flood level at the Martin Bridge, Taree, to be 5.5m AHD – around 0.3m lower than that defined by Public Works (1991).

The current flood warning trigger levels at the Martin Bridge, Taree, are presented in the following table against design flood levels and historic flood levels for context.

Table 1 Flood Warning Levels, Design Flood Levels and Historic Flood Levels at Taree (Martin Bridge)

Flood Classification	Peak Flood Level (m AHD)
Minor Flood Warning	*1.8*
Moderate Flood Warning	*2.4*
20% AEP	2.9
2013	**3.37**
Major Flood Warning	*3.7*
1990	**4.37**
5% AEP	4.4
2011	**4.5**
2% AEP	5.1
1% AEP	5.5
1978	**5.75**
0.5% AEP	5.8
1929	**5.9**
0.2% AEP	6.3
PMF	9.4

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Glossary

annual exceedance probability (AEP)	AEP (measured as a percentage) is a term used to describe flood size. It is a means of describing how likely a flood is to occur in a given year. For example, a 1% AEP flood is a flood that has a 1% chance of occurring, or being exceeded, in any one year. It is also referred to as the '100 year ARI flood' or '1 in 100 year flood'. The term 100 year ARI flood has been used in this study. See also average recurrence interval (ARI).
Australian Height Datum (AHD)	National survey datum corresponding approximately to mean sea level.
attenuation	Weakening in force or intensity
average recurrence interval (ARI)	ARI (measured in years) is a term used to describe flood size. It is the long-term average number of years between floods of a certain magnitude. For example, a 100 year ARI flood is a flood that occurs or is exceeded on average once every 100 years. The term 100 year ARI flood has been used in this study. See also annual exceedance probability (AEP).
catchment	The catchment at a particular point is the area of land that drains to that point.
design flood	A hypothetical flood representing a specific likelihood of occurrence (for example the 100yr ARI or 1% AEP flood).
development	Existing or proposed works that may or may not impact upon flooding. Typical works are filling of land, and the construction of roads, floodways and buildings.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
flood	A 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 flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
flood behaviour	The pattern / characteristics / nature of a flood.
flood fringe	Land that may be affected by flooding but is not designated as floodway or flood storage.
flood hazard	The potential for damage to property or risk to persons during a flood. Flood hazard is a key tool used to determine flood severity and is used for assessing the suitability of future types of land use.The degree of flood hazard varies with circumstances across the full range of floods.

flood level	The height of the flood described either as a depth of water above a particular location (eg. 1m above a floor, yard or road) or as a depth of water related to a standard level such as Australian Height Datum (eg the flood level was 7.8 mAHD). Terms also used include flood stage and water level.
flood liable land	See flood prone land.
floodplain	Land susceptible to flooding up to the probable maximum flood (PMF). Also called flood prone land. Note that the term flood liable land now covers the whole of the floodplain, not just that part below the flood planning level.
floodplain risk management study	Studies carried out in accordance with the Floodplain Development Manual (NSW Government, 2005) that assesses options for minimising the danger to life and property during floods. These measures, referred to as ‘floodplain risk management measures / options’, aim to achieve an equitable balance between environmental, social, economic, financial and engineering considerations. The outcome of a Floodplain Risk Management Study is a Floodplain Risk Management Plan.
floodplain risk management plan	The outcome of a Floodplain Risk Management Study.
flood planning levels (FPL)	The combination of flood levels and freeboards selected for planning purposes, as determined in Floodplain Risk Management Studies and incorporated in Floodplain Risk Management Plans. The concept of flood planning levels supersedes the designated flood or the flood standard used in earlier studies..
flood prone land	Land susceptible to inundation by the probable maximum flood (PMF) event. Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Risk Management Plans should encompass all flood prone land (i.e. the entire floodplain).
flood stage	See flood level.
flood storage	Floodplain area that is important for the temporary storage of floodwaters during a flood.
flood study	A study that investigates flood behaviour, including identification of flood extents, flood levels and flood velocities for a range of flood sizes.
floodway	Those areas of the floodplain where a significant discharge of water occurs during floods. Floodways are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.

freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determing the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
high flood hazard	For a particular size flood, there would be a possible danger to personal safety, able-bodied adults would have difficulty wading to safety, evacuation by trucks would be difficult and there would be a potential for significant structural damage to buildings.
hydraulics	The term given to the study of water flow in rivers, estuaries and coastal systems.
hydrology	The term given to the study of the rainfall-runoff process in catchments.
low flood hazard	For a particular size flood, able-bodied adults would generally have little difficulty wading and trucks could be used to evacuate people and their possessions should it be necessary.
m AHD	Metres Australian Height Datum (AHD).
m/s	Metres per second. Unit used to describe the velocity of floodwaters.
m^3/s	Cubic metres per second or ‘cumecs’. A unit of measurement for creek or river flows or discharges. It is the rate of flow of water measured in terms of volume per unit time.
overland flow path	The path that floodwaters can follow if they leave the confines of the main flow channel. Overland flow paths can occur through private property or along roads. Floodwaters travelling along overland flow paths, often referred to as ‘overland flows’, may or may not re-enter the main channel from which they left; they may be diverted to another water course.
peak flood level, flow or velocity	The maximum flood level, flow or velocity that occurs during a flood event.
probable maximum flood (PMF)	The largest flood likely to ever occur. The PMF defines the extent of flood prone land or flood liable land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with the PMF event are addressed in the current study.
probability	A statistical measure of the likely frequency or occurrence of flooding.
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of this study, it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek.

stage	See flood level.
topography	The shape of the surface features of land
velocity	The term used to describe speed of floodwaters, usually in m/s.
water level	See flood level.

1 Introduction

The Manning River Flood Study has been prepared for Greater Taree City Council (Council) to define the existing flood behaviour in the catchment and establish the basis for subsequent floodplain management activities.

1.1 Study Location

The Manning River basin encompasses an area of just over 8,100km^2 and drains to the Tasman Sea on the NSW mid-north coast. The Gloucester River, Barnard River and Nowendoc River join the Manning River upstream of Mount George, with their catchments contributing 1,930km^2, 1,830km^2 and 1,650km^2 respectively to the total Manning River catchment area. The lower Manning River floodplain is some 2,060km^2 in size and includes the catchments of Dingo Creek and the Lansdowne River.

The townships of Gloucester, Wingham, Taree, Harrington and Old Bar are the largest communities within the Manning River catchment. The upper catchment is predominantly densely vegetated forest and the lower floodplain is occupied by rural pasture lands. The study catchment is shown in Figure 1-1.

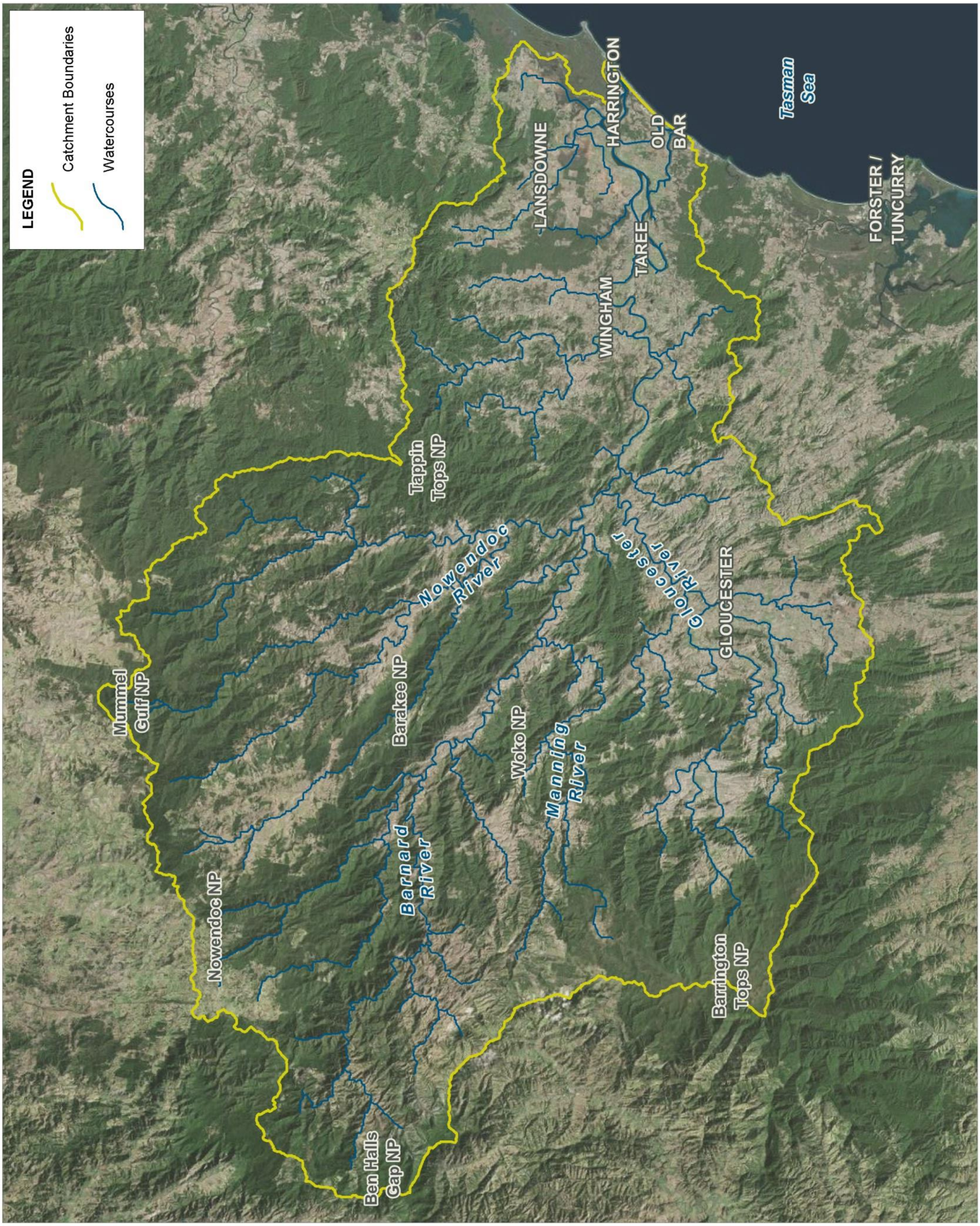
This study will focus on the flood behaviour in the lower Manning River catchment area, downstream of Wingham and Lansdowne.

1.2 Study Background

A number of investigations into the flood behaviour within the study area have previously been undertaken. The most recent study encompassing the same area of interest as this current study was the Manning River Flood Study which was completed by Public Works in 1991. A Floodplain Management Study followed in 1996.

More recently, the Wingham Flood Study and Floodplain Risk Management Study were completed by WorleyParsons in May 2011. These studies focussed on the township of Wingham only, with hydraulic modelling extending from the Killawarra Bridge to approximately 5km downstream of Wingham. A flood study of the Lansdowne River is currently been undertaken. The area addressed in these recent studies is not required to be included in the hydraulic assessment as part of this current study.

Significant flooding has occurred in the catchment since records began some 185 years ago. The latter end of the 19th century saw numerous large floods occurring in the catchment, with half of the largest ten floods on record occurring between 1866 and 1895. The following flood event in February 1929 is the largest on record for the lower Manning River. Notable flood events also occurred in 1930, 1956, 1978 and 1990. After a relatively flood-free period throughout the remainder of the 1990s and the early 2000s, two large events occurred recently in June 2011 and March 2013.



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Study Locality

Figure: 1-1

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Since the completion of the previous study in 1991, there have been significant developments in hydraulic modelling. The opportunity to undertake a new study will provide improvements to the existing flooding information, particularly with regards to the flood mapping outputs. These will help guide both the floodplain risk management and emergency response management processes.

1.3 The Floodplain Risk Management Process

The State Government's Flood Prone Land Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the Floodplain Development Manual.

Under the Policy the management of flood liable land remains the responsibility of Local Government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the State Government through the following four sequential stages:

Table 1-1 Stages of Floodplain Risk Management

	Stage	Description
1	Formation of a Committee	Established by Council and includes community group representatives and State agency specialists.
2	Data Collection	Past data such as flood levels, rainfall records, land use, soil types etc.
3	**Flood Study**	**Determines the nature and extent of the flood problem.**
4	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed developments.
5	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of risk management for the floodplain.
6	Implementation of the Floodplain Risk Management Plan	Construction of flood mitigation works to protect existing development. Use of environmental plans to ensure new development is compatible with the flood hazard.

This study represents Stage 3 of the above process and aims to provide an understanding of flood behaviour within the lower Manning River catchment.

1.4 Study Objectives

The primary objective of the Flood Study is to define the flood behaviour within the Manning River catchment through the establishment of appropriate numerical models. The study has produced information on flood flows, velocities, levels and extents for a range of flood event magnitudes under existing catchment and floodplain conditions. Specifically, the study incorporates:

- Compilation and review of existing information pertinent to the study;

- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design event including the 20% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5%, AEP 0.2% AEP and PMF event; and
- Presentation of study methodology, results and findings in a comprehensive report incorporating appropriate flood mapping.

The principal outcome of the flood study is the understanding of flood behaviour in the catchment and in particular design flood level information that will be used to set appropriate flood planning levels for the study area.

1.5 About this Report

This report documents the Study's objectives, results and recommendations.

Section 1 introduces the study.

Section 2 provides an overview of the approach adopted to complete the study.

Section 3 outlines the community consultation program undertaken.

Section 4 details the development of the computer models.

Section 5 details the model calibration and validation process including sensitivity tests.

Section 6 presents the adopted design flood inputs and boundary conditions.

Section 7 presents design flood simulation results and associated flood mapping.

2 Study Approach

2.1 The Study Area

2.1.1 Catchment Description

This study focuses on the Manning River catchment area downstream of Wingham.

Downstream of Taree, the Manning River splits into two arms and enters the ocean at two locations; Harrington and Farquhar Inlet, which is located just north of the Old Bar township. Both entrances are dynamic. Farquhar Inlet can become severely restricted and is known to have closed on many occasions historically. The entrance at Harrington is a permanently open but can become significantly shoaled, particularly in periods between large floods. A break wall was constructed along the northern channel bank in 1984 to offer protection to ships passing through.

The topography of the Manning River catchment is shown in Figure 2-1.

The Great Dividing Range forms the upper limit of the Manning River catchment, where elevations of around 1200m AHD are typical. The Barrington Tops, located in the south-west of the catchment, peaks at just below 1600m AHD. The Manning River spills onto a vast, low-lying floodplain (elevated to less than 2m AHD) area downstream of Taree.

Land use within the catchment largely consists of forested areas or pastureland and other cultivated areas. There is little urban development within the catchment.

The towns of Tinonee, Taree, Cundletown, Croki, Coopernook, Harrington and Manning Point are located within the study area. Taree is the largest of these and has a population of around 20,000.

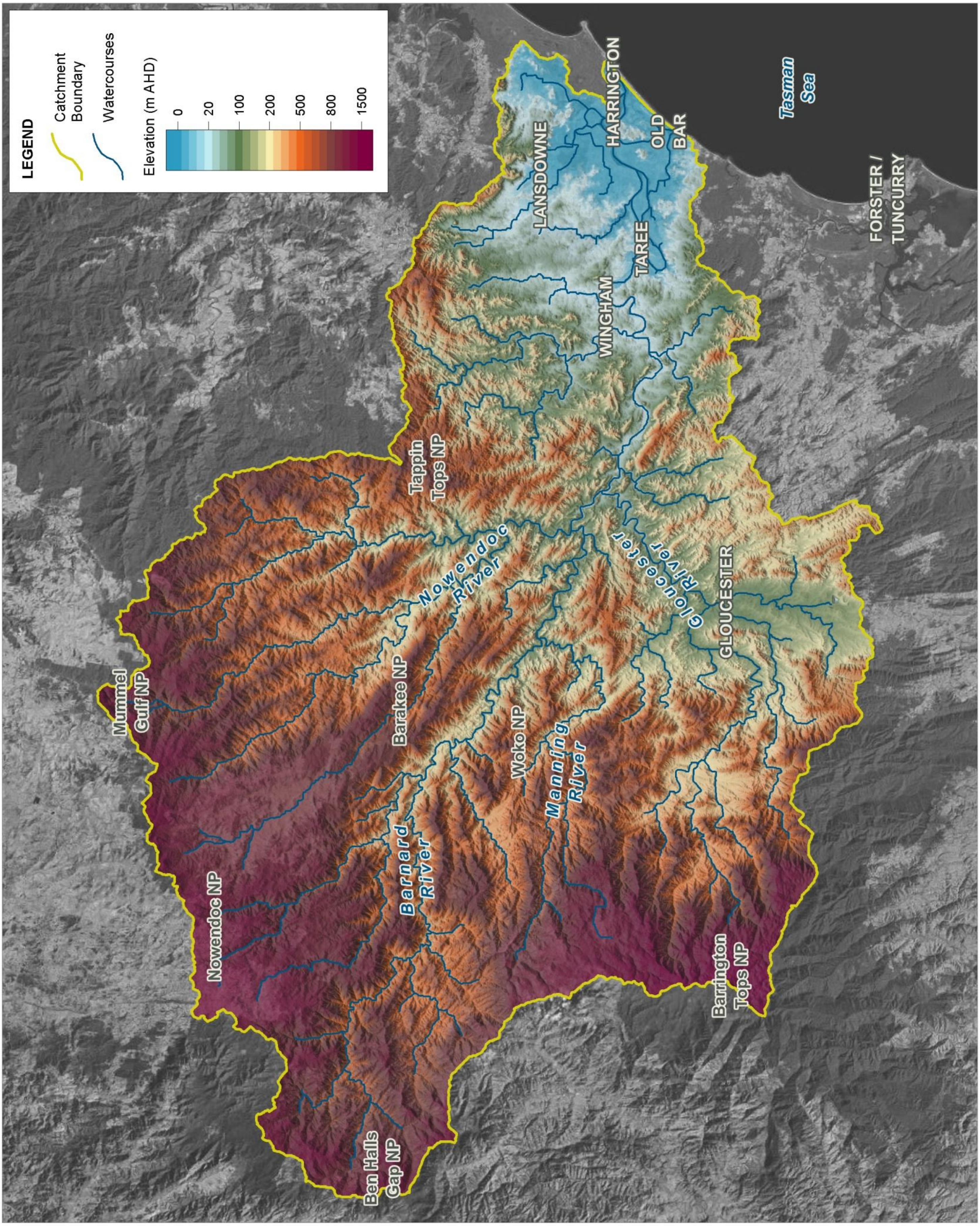
There are several major transport routes through the catchment including the Pacific Highway, Thunderbolts Way, Bucketts Way and the North Coast Railway Line. The Pacific Highway Taree Bypass was constructed between 1993 and 2000. It crosses both the north and south arms of the Manning River just downstream of Cundletown.

2.1.2 History of Flooding

There is a long and relatively frequent history of flooding within the lower Manning River catchment. The three largest floods on record occurred in 1866, 1929 and 1978. In more recent years, large flood events have occurred in 1990 and 2011, with a smaller event in 2013.

Flooding in the catchment is known to cause extensive flood damages and considerable disruption to residents. Access roads readily become inundated, isolating people and properties. Helicopters have been required to assist in the safe evacuation of residents in the past. Flooding has resulted in significant damage to residential properties and commercial businesses, with substantial loss of livestock a major impact of past flood events. Two lives were lost in the Manning River catchment during the 1929 flood.

Due to the large size of the catchment and spatial variation in rainfall, the relative magnitude of historical flood events is not necessarily the same across the whole catchment area. At Taree (Macquarie Street), the 1929 event resulted in the highest flood on record, with a peak level of 5.6m AHD. Peak flood levels of 5.45m AHD and 5.15m AHD were recorded during the 1978 and



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1866 events respectively, and make up the second and third highest levels on record.

A sample of flood photographs from the 1978, 1990 and 2011 flood events have been sourced from the internet and have been reproduced in Figure 2-2 to Figure 2-9.

Figure 2-2 March 1978 Flood - Martin Bridge, Taree

Figure 2-3 March 1978 Flood - Pulteney Street, Taree

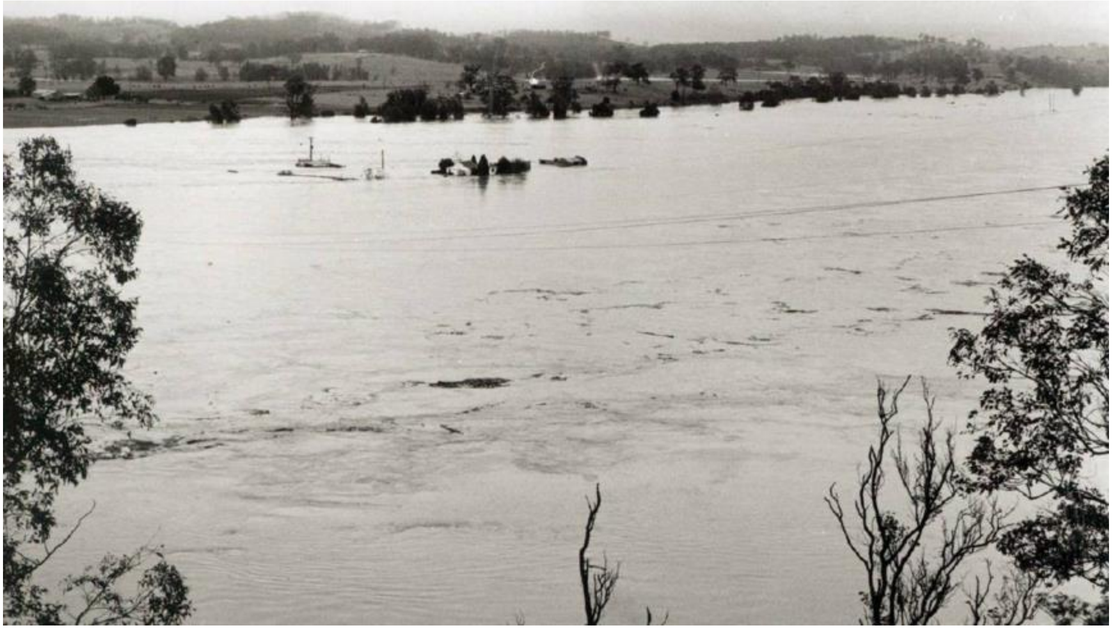


Figure 2-4 March 1978 Flood - Mondrook Point

Figure 2-5 February 1990 Flood - Former Swimming Pool, River Street, Taree

Figure 2-6 February 1990 Flood - Manning River Rowing Club, Taree

Figure 2-7 June 2011 Flood - Manning River Rowing Club, Taree

Figure 2-8 June 2011 Flood - Fothering Park and Council Offices from Victoria Street, Taree

Figure 2-9 June 2011 Flood – Coopernook Road, from Pacific Highway

2.2 Compilation and Review of Available Data

2.2.1 Previous Studies

There have been numerous studies into the nature and behaviour of flooding within the catchment in the past. The first major investigation into determining design flood levels, for the purpose of planning and flood management, was completed in 1991.

2.2.1.1 Manning River Flood History 1831-1979 (Public Works, 1981)

This report compiles information relevant to all flood events known to have occurred in the catchment since records began, with focus on the Wingham and Taree townships, as well as the lower floodplain area.

The report compiled significant flood events only - classified as flood events producing levels of over 3.6m AHD at Taree (Macquarie Street) and 10.7m AHD at the historic Wingham gauge. Both of these levels were calculated to be within the order of a one in 5 year event at their respective locations.

2.2.1.2 Manning River Flood Study (Public Works, 1991)

The Manning River Flood Study is the most recent flood study investigating the same area of interest as this current study. It focuses on the floodplain area downstream from Wingham and includes all the major tributaries to the ocean entrances at Harrington and Farquhar Inlet. A Floodplain Management Study followed in 1996 and was completed by Willing and Partners.

A RORB hydrological model and an ESTRY 1D hydraulic model were developed for the study.

Flood frequency analyses were performed from long-term historic flood levels available at Killawarra, Wingham and Taree. At Killawarra, discharge frequency relationships (using the Log Pearson III distribution) were determined from two methods – a series of historical peaks only (a set of 26 values) and an annual maxima series recorded at the gauge from 1945 to 1990. An average of the two discharges for each design event was adopted for simulation of design flood events.

At Wingham and Taree, the flood frequency analyses were completed based on peak flood levels. The 1% AEP flood level at Taree was determined to be 5.5m AHD, and was derived from analysis of all known major floods up to and including the 1990 event.

As the Killawarra gauge failed during the 1978 event, the study estimated a likely hydrograph based on recorded hydrographs at upstream locations along the Gloucester and Nowendoc Rivers. The report also contains recorded water level time series at Wingham and Taree for the 1978 flood event.

2.2.1.3 Wingham Flood Study Review and Upgrade (WorleyParsons, 2011)

This study was completed with the aim of developing a 2D hydraulic model to simulate flooding in and around the township of Wingham. Design flows calculated in the Public Works (1991) were adopted for use in the study. The downstream limit of the hydraulic model is Mondrook Creek, located just north of Tinonee.

2.2.1.4 Lansdowne Flood Study Review, Upgrade and Extension (WorleyParsons, 2014)

This study involved reviewing and upgrading the hydrologic and hydraulic modelling completed in the Manning River Flood Study (Public Works, 1991) around the township of Lansdowne. This was necessary as the original study focussed on mainstream flooding of the Manning River rather than local catchment flooding of the Lansdowne River.

A refined hydrological model was developed using the WBMN software to incorporate local inflows along smaller tributaries to the Lansdowne River that were not included in the coarser sub catchment representation of the original study. A 2D hydraulic model was developed using the finite element program RMA-2 and extended from around 5.5km north-west of Lansdowne to the confluence with the Manning River. Both the hydrologic and hydraulic models were calibrated and verified against events occurring in 1999 and 1995. The 1978 event was also used but to a lesser degree due to the lack of stream flow and water level records.

The 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF design flood events were simulated. Resulting from the Lansdowne Flood Study Review, Upgrade and Extension, the 1% AEP peak flood level in Lansdowne was revised to be just over 9m AHD downstream of the railway bridge. In the original Flood Study, the 1% AEP design flood level cited for Lansdowne was around 3m AHD and was driven by backwater inundation from the Manning River.

2.2.2 Water Level Data

There are a number of locations within the catchment at which water levels have been historically recorded. The current continuous water level gauging locations are presented in Figure 2-10 and are summarised in Table 2-1.

Table 2-1 Current Stream Gauges in the Study Area

Gauge #	Operator	Location	Period of Record
208425	MHL	Harrington	1987 – current
208415	MHL	Farquhar Inlet	1987 – current
208404	MHL	Croki	1992 – current
208430	MHL	Dumaresq Island	2001 – current
208410	MHL	Taree	2010 – current
208420	MHL	Taree West	2010 – current
208400	MHL	Wingham[1]	2009 – current
208004	DPI	Killawarra	1945 – current

[1]A stream gauge has been in operation at the Bight Bridge, Wingham, since 1947 (refer to text for further detail).

In recent years, flood levels at Taree are recorded at the telemetric gauge located on the downstream, eastern side of the Martin Bridge (gauge was installed in February 2010). Although the gauge failed during the June 2011 event, a peak flood level of 4.5m AHD was recorded manually by SES staff at this location. A smaller event occurred in March 2013 where a peak flood level of 3.6m AHD was recorded at the Martin Bridge gauge.

Historically, the Wingham gauge was operated by the Bureau of Meteorology and was located in a different position at the bridge. It is known that after the 1978 event, it was moved from the northern bank of the Manning River on Wingham-Tinonee Road to its present day location at the second pier from the south of the Wingham Bridge (WorleyParsons, 2011). There is a significant flood gradient present across the channel at the location of the gauge site. This topic is discussed further in Section 5.4.1.

The gauges at Harrington and Farquhar Inlet are of high value to this study due to their close proximity to the ocean entrances. Water levels recorded at these locations provides critical information into entrance condition before, during and after flood events. The Harrington gauge is located approximately 1.6km from the break wall entrance, on the southern side of the wall near the bridge. It is around 600m from the ocean when the entrance is completely open. The gauge at Farquhar Inlet is located at the edge of the estuary near Oxley Island, approximately 1.3km from the ocean entrance.

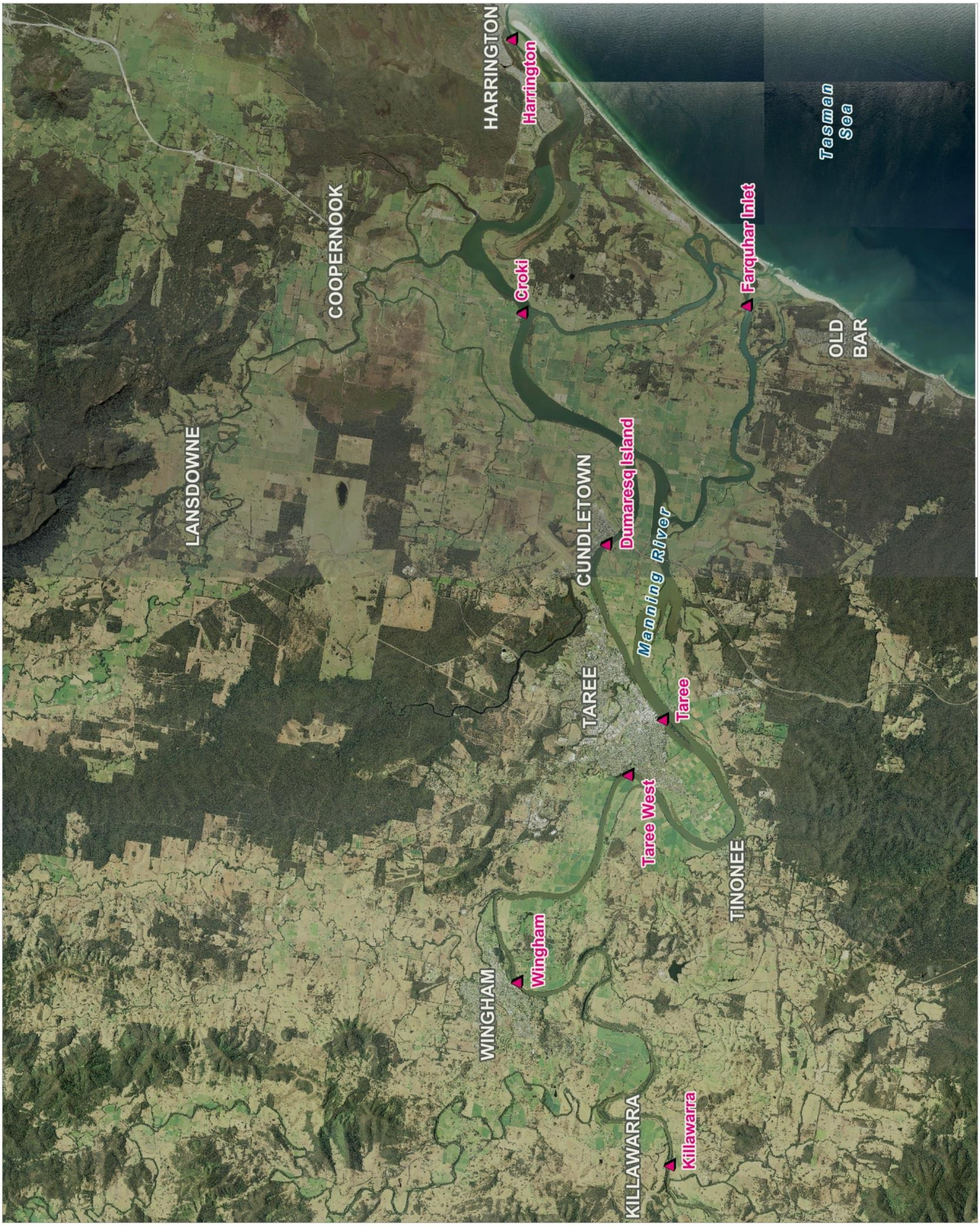
Continuous gauge records were obtained where available for the selected calibration events, discussed in Section 5. The Killawarra gauge record is discussed further in Section 6.2.1, which provides flood frequency analysis at the site.

2.2.3 Historical Flood Levels

In addition to the gauge sites discussed in Section 2.2.1.4, historic flood levels have been recorded at a number of locations since European settlement of the area in 1830. Flood records are available at Wingham and Killawarra for the period prior to installation of stream gauges at each site. Water levels have also been recorded historically at various locations in Taree. A gauge located at the Taree Aquatic Club on Macquarie Street has been in operation since 1973 (operated by Public Works). Historic levels in Wingham and Taree were compiled in the Manning River Flood History 1831-1979 (Public Works, 1981) as described in Section 2.2.1.1.

Care must be taken when assessing peak flood level records, as changes within the catchment over the years (clearing of catchment vegetation, topographic changes associated with urban development, construction of arterial roads, bank stability works etc.) may mean levels cannot be directly compared. Adding to the uncertainty, it is also known that some historic flood levels for Taree cited in previous reports were referenced to an incorrect datum.

Of particular importance to this study is the long and complete history of flood levels at Killawarra, and to a lesser degree, at Taree. The ten highest peak water levels recorded at Killawarra are presented in Table 2-2. Of the ten highest flood levels on record, seven occurred prior to the gauge installation in 1945. The peak flood levels cited for the 1978, 1990 and 2011 event were recorded by the continuous stream gauge. The ten highest peak flood levels on record for Taree are presented in Table 2-3.



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Current Stream Gauges in the Study Area

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Table 2-2 Peak Flood Levels at Killawarra

Rank	Year	Flood Level (m AHD)
1	1866	19.1
2	1929	18.5
3	1978	18.1
4	1895	17.6
5	1875	17.1
6	1894	16.9
7	1930	16.8
8	1990	16.5
9	1870	16.4
10	2011	16.1

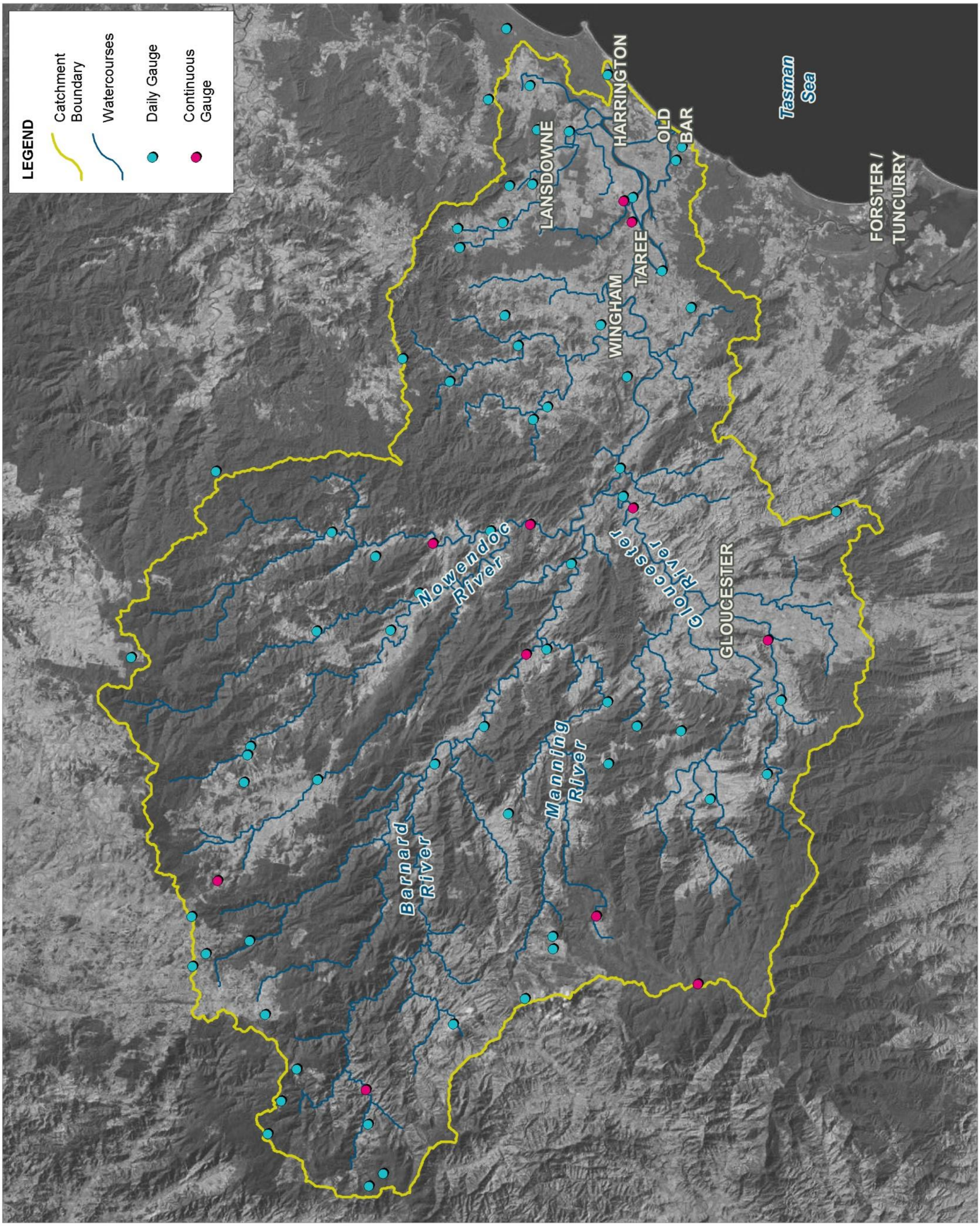
Table 2-3 Peak Flood Levels at Taree, Macquarie Street

Rank	Year	Flood Level (m AHD)
1	1929	5.6
2	1978	5.45
3	1866	5.15
4	1930	5.1
5	1895	4.85
6	1875	4.85
7	1870	4.65
8	1956	4.55
9	1867	4.5
10	1894	4.3

2.2.4 Rainfall Data

Daily and continuous rainfall records were obtained from BoM and NSW Office of Water. Within the Manning River catchment, daily records are available dating back to the 1929 event, and continuous records are available for the 1978 event onwards. Figure 2-11illustrates the coverage of all daily and continuous gauges across the study catchment, including historic gauges that are no longer in operation. It should be noted that a number of additional continuous gauges exist within the catchment. However, these gauges are used as part of the flood warning system and the data is difficult to obtain, so they have not been included on the figure.

Details of the continuous gauges in the study catchment that were of use for this study are contained in Figure 2-11.



Title:
Rainfall Gauges within the Manning River Catchment

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Table 2-4 Continuous Rainfall Gauges within the VIcinity of the Study Catchment

Station Number	Location	Flood Events Available
60030	Taree (Patanga Cl)	1978, 1990
60104	Nowendoc (Green Hills)	1978, 1990, 2011, 2013
60112	Gloucester (Hiawatha)	2011, 2013
60141	Taree Airport AWS	2011, 2013
208002	Manning River at Tomalla (Campbells No.2)	1978

Analysis of rainfall data for the calibration events is detailed in Section 5.

2.2.5 Council Data

A number of spatial datasets were provided by Council for use in the study. These included aerial photography from 2009 and 2010, previous flood investigations reports, SES documents and flood photographs of the 1978 event.

2.3 Site Inspections

Site inspections were undertaken during the course of the study to gain an appreciation of local features influencing flooding behaviour. Some of the key observations to be accounted for during the site inspections included:

- Presence of local structural hydraulic controls including the road and rail bridges and associated embankments;
- General nature of the Manning River, the tributary channels and associated floodplains noting river plan form, vegetation type and coverage and the presence of significant flow paths;
- Nature of the Harrington and Farquhar Inlet entrances and current entrance configuration; and
- Location of existing development and infrastructure on the floodplain.

This visual assessment was useful for defining hydraulic properties within the hydraulic model and ground-truthing of topographic features identified from the survey datasets.

2.4 Survey Requirements

A number of datasets containing topographic information were available from Council and are summarised as follows:

- Hydrographic survey of the Manning River from 1999, covering the Manning River from Harrington and Farquhar Inlet to upstream past Wingham (including tributaries); and
- LiDAR survey data of the catchment from 2012.

As hydrographic survey and high resolution LiDAR data is available for the entire study area, additional survey was not required. Processing of the hydrographic survey data is detailed in Section 4.2.4.

2.5 Community Consultation

The success of a Floodplain Management Plan hinges on its acceptance by the community and other stake-holders. This can be achieved by involving the local community at all stages of the decision-making process. This includes the collection of their ideas and knowledge on flood behaviour in the study area, together with discussing the issues and outcomes of the study with them.

The key elements of the consultation process in undertaking the flood study have included:

- Issue of a questionnaire to obtain historical flood data and community perspective on flooding issues; and
- Public exhibition of Draft Report and community information session.

These elements are discussed in further detail in Section 3.

2.6 Development of Computer Models

2.6.1 Hydrological Model

For the purpose of the Flood Study, a hydrologic model (discussed in Section 4.1) was developed to simulate the rate of storm runoff from the catchment. The model predicts the amount of runoff from rainfall and the attenuation of the flood wave as it travels down the catchment. This process is dependent on:

- Catchment area, slope and vegetation;
- Variation in distribution, intensity and amount of rainfall; and
- Antecedent conditions of the catchment.

The output from the hydrologic model is a series of flow hydrographs at selected locations such as at the boundaries of the hydrodynamic model. These hydrographs are used by a hydrodynamic model to simulate the passage of a flood through the study catchment.

2.6.2 Hydraulic Models

The hydraulic model is applied to determine flood levels, velocities and depths across the study area for historical and design events. Two hydraulic models (discussed in Section 4.2 and 4.3) were developed for this study:

- A TULFOW Classic model was developed to provide a two-dimensional (2D) representation of the channel and floodplain of the Manning River extending from Killawarra to the ocean, covering some 565 km^2 (approximately 38% of catchment area downstream of Killawarra).

- A TUFLOW-FV model was developed to provide a 2D representation of the ocean entrances at Harrington and Old Bar, with the purpose of simulating the sediment transport processes occurring during flood events.

The technical aspects of each model are discussed below.

TUFLOW Classic Model

BMT WBM has applied the fully 2D software modelling package TUFLOW. The 2D model has distinct advantages over 1D and quasi-2D models in applying the full 2D unsteady flow equations. This approach is necessary to model the complex interaction between watercourses and floodplains and converging and diverging of flows through structures. The channel and floodplain topography is defined using a high resolution DEM for greater accuracy in predicting flows and water levels and the interaction of in-channel and floodplain areas.

TUFLOW-FV Model

The TUFLOW-FV modelling software (a flexible mesh, finite volume numerical model that simulates hydrodynamic, sediment transport and water quality processes in oceans, coastal waters, estuaries and rivers) was used to model the Manning River ocean entrances at Harrington and Old Bar.

The spatial domain (or study area extent) is discretised using contiguous, non-overlapping irregular triangular and quadrilateral “cells”. The solution scheme is explicit and uses a varying Courant dependent time step. Compared to a fixed grid approach, this has significant benefits for applications of complex geometry, or sharply varying flow and concentration gradients. The flexible mesh gives the modeller more scope to design a model domain that best suits the problem to be solved.

The TUFLOW-FV model was developed to provide a 2D representation of the Manning River ocean entrances at Harrington and Farquhar Inlet, with the purpose of simulating the sediment transport processes occurring during flood events.

2.7 Calibration and Sensitivity Testing of Models

The hydrologic and hydraulic models were calibrated and verified to available historical flood event data, to establish the values of key model parameters and confirm that the models were capable of adequately simulating real flood events.

The following criteria are generally used to determine the suitability of historical events to use for calibration or validation:

- The availability, completeness and quality of rainfall and flood level event data;
- The amount of reliable data collected during the historical flood information survey; and
- The variability of events – preferably events would cover a range of flood sizes.

The major historical flood events of February 1929, March 1978 and June 2011 were identified as suitable events for calibration/validation of the developed models. Assessment of the model performance also incorporated a range of sensitivity tests of key variables/model assumptions, including:

- The influence of adopted model roughness;
- Entrance breakout at Harrington and Old Bar; and
- Increases in rainfall intensities and increased ocean water level conditions to assess the impact of predicted climate change.

Sensitivity testing was undertaken for the design flood events and has been reported in Section 7.5.

2.8 Establishing Design Flood Conditions

Design floods are statistical-based events which have a particular probability of occurrence. For example, the 1% Annual Exceedance Probability (AEP) event is the best estimate of a flood with a peak discharge that has a 1% (i.e. 1 in 100) chance of occurring in any one year. For the study catchments, design floods were based on a combination of flood frequency and design rainfall estimates, in accordance with the procedures Australian Rainfall and Runoff (IEAust, 2001). In accordance with Council's brief, the simulated design events include the 20% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF event.

The design flood conditions form the basis for floodplain management in the catchment and in particular design planning levels for future development controls. The adopted design flood conditions are presented in Section 6.

2.9 Mapping of Flood Behaviour

Design flood mapping is undertaken using output from the hydraulic model. Maps are produced showing water level, water depth and velocity for each of the design events. The maps present the peak value of each parameter. Provisional flood hazard categories and hydraulic categories are derived from the hydraulic model results and are also mapped. The mapping outputs are described in Section 7 and presented in the accompanying flood mapping compendium.

3 Community Consultation

3.1 The Community Consultation Process

Community consultation has been an important component of the current study. The consultation has aimed to inform the community about the development of the flood study and its likely outcome as a precursor to subsequent floodplain management activities. It has provided an opportunity to collect information on their flood experience and their concerns on flooding issues.

The key elements of the consultation process have been as follows:

- Questionnaire available to be completed by landowners, residents and businesses within the study area;
- An information session for the community to present information on the progress and objectives of the flood study and obtain feedback on historical events in the catchment and other flooding issues; and
- Public exhibition of the draft Flood Study.

These elements are discussed in detail below. The community information brochure and questionnaire are also provided in Appendix A.

3.2 Community Questionnaire

An online community questionnaire was advertised in the Manning River Times and on Council's website along with some background information about the Flood Study. Hard copies were also available in libraries for residents to complete. The questionnaire sought to collect information on previous flood experience and flooding issues. The focus of the questionnaire was historical flooding information that may be useful for correlating with predicted flooding behaviour from the modelling.

In total 54 questionnaire returns were received, of which only 2 were hard copies. Almost 90% of the respondents indicated that they or someone they knew had been affected by flooding in the past. For the 1978 event, respondents were equally affected by flooding related to disruption of traffic, flooding to front/back yards and flooding to home or business contents. For the 2011 event, most of those affected by flooding related to disruption of traffic and nuisance yard flooding, with only a few whose home or business content had been flooded.

The majority of responses suggested that heavy rains were the principal driver of flooding problems within the catchment, with some residents citing that blockage of drains was also a contributing factor.

Calibration data was obtained for the 1978 and 2011 events at 12 and 9 locations respectively from recorded flood levels mentioned in the responses.

3.3 Community Information Session

A community information session was held during the public exhibition period on the evening of Wednesday 16th March from 4pm to 7pm at Council's administration building in Taree. There were no attendees to the community information session.

3.4 Public Exhibition

The Draft Flood Study Report was placed on public exhibition for a four week period between 1st March and the 28th March 2016. The exhibition sought public comments and feedback on the study. No comments were received during the public exhibition period.

4 Model Development

4.1 Hydrological Model

The hydrologic model simulates the rate at which rainfall runs off the catchment. The amount of rainfall runoff and the attenuation of the flood wave as it travels down the catchment is dependent on:

- The catchment slope, area, vegetation and other characteristics;
- Variations in the distribution, intensity and amount of rainfall; and
- The antecedent conditions (dryness/wetness) of the catchment.

These factors are represented in the model by:

- Sub-dividing (discretising) the catchment into a network of sub-catchments inter-connected by channel reaches representing the watercourses. The sub-catchments are delineated, where practical, so that they each have a general uniformity in their slope, landuse, vegetation density, etc;
- The amount and intensity of rainfall is varied across the catchment based on available information. For historical events, this can be very subjective if little or no rainfall recordings exist.
- The antecedent conditions are modelled by varying the amount of rainfall which is “lost” into the ground and “absorbed” by storages. For very dry antecedent conditions, there is typically a higher initial rainfall loss.

The output from the hydrologic model is a series of flow hydrographs at selected locations such as at the boundaries of the hydraulic model. These hydrographs are used by the hydraulic model to simulate the passage of the flood through the catchment.

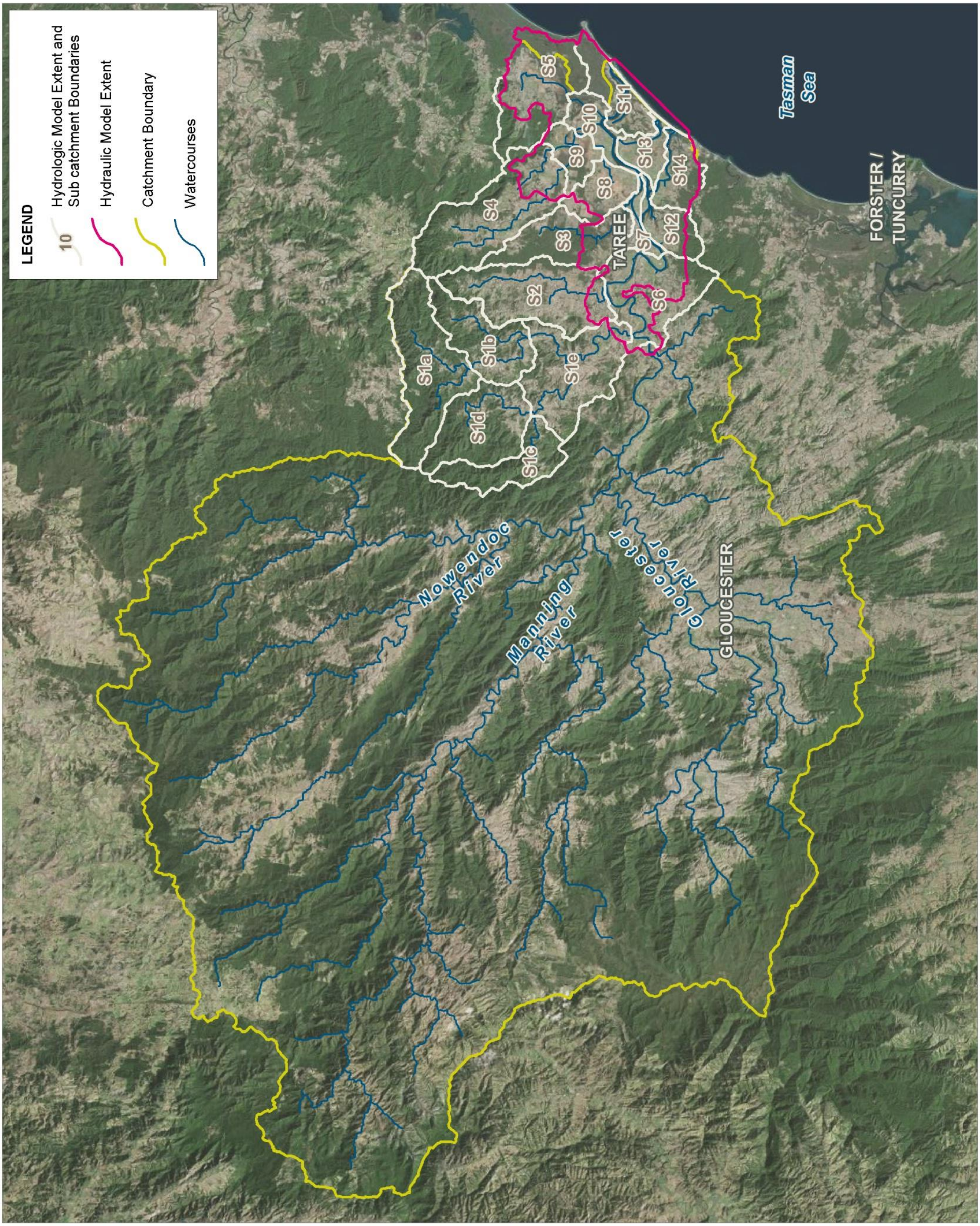
The XP-RAFTS software was used to develop the hydrologic model using the physical characteristics of the catchment including catchment areas, ground slopes and vegetation cover as detailed in the following sections.

4.1.1 Flow Path Mapping and Catchment Delineation

The Manning River catchments drain approximately 6630km^2 upstream of Killawarra. The catchment area downstream of Killawarra is 1530km^2. This includes the Dingo Creek catchment (560km^2) and the Lansdowne River as well as the lower Manning River floodplain.

Due to the availability of historical streamflow data to generate inflows at the upstream extent of the hydraulic model, the hydrological model is only required to provide local inflows into the model downstream of Killawarra.

In order to accurately represent the rate and volume of runoff generated from the catchment to be fed into the hydraulic model, it was important to delineate the catchments appropriately. The hydrological model was split into a network of sub-catchments to provide sufficient detail of local catchment inflows as shown in Figure 4-1.



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Hydrologic and Hydraulic Model Extent

Figure: 4-1

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Table 4-1 summarises the key catchment parameters adopted in the XP-RAFTS model, including catchment area, vectored slope and PERN (roughness) value estimated from the available topographic information and aerial photography. The adopted PERN values considered if the majority of the sub-catchment could be described as either forested area (PERN of 0.12) or cleared/pasture area (PERN of 0.06).

Table 4-1 RAFTS Sub-catchment Properties

ID	Area (km^2)	Slope (%)	PERN
S1a	149.5	1.44	0.12
S1b	92.5	0.81	0.1
S1c	69.4	1.67	0.12
S1d	108.6	1.11	0.1
S1e	136.8	0.24	0.06
S2	143.3	0.33	0.06
S3	91.8	0.26	0.12
S4	167.7	0.33	0.08
S5	126.5	0.25	0.1
S6	117.4	0.14	0.06
S7	21.2	0.12	0.06
S8	47.3	0.07	0.06
S9	36.4	0.09	0.06
S10	39	0.19	0.06
S11	55.6	0.14	0.1
S12	42.1	0.13	0.1
S13	31.5	0.05	0.06
S14	46.3	0.19	0.1

As indicated in the table and evident from aerial photography, most of the upper Manning catchment is densely vegetated. The lower catchment and floodplain areas are predominantly cleared/pasture land use.

4.1.2 Rainfall Data

Rainfall information is the primary input and driver of the hydrological model, which simulates the catchments response in generating surface runoff. Rainfall characteristics for both historical and design events are described by:

- Rainfall depth – the depth of rainfall occurring across a catchment surface over a defined period (e.g. 270mm in 36hours or average intensity 7.5mm/h); and
- Temporal pattern – describes the distribution of rainfall depth at a certain time interval over the duration of the rainfall event.

Both of these properties may vary spatially across the catchment.

The procedure for defining these properties is different for historical and design events. For historical events, the recorded hyetographs at continuous rainfall gauges provide the observed rainfall depth and temporal pattern. Where only daily read gauges are available within a catchment, assumptions regarding the temporal pattern may need to be made.

For design events, rainfall depths are most commonly determined by the estimation of intensity-frequency-duration (IFD) design rainfall curves for the catchment. Standard procedures for derivation of these curves are defined in AR&R (2001). Similarly AR&R (2001) defines standard temporal patterns for use in design flood estimation.

The rainfall inputs for the historical calibration/validation events are discussed in further detail in Section 5.

4.2 Hydraulic Model

BMT WBM has applied the fully 2D software modelling package TUFLOW Classic. The 2D model has distinct advantages over 1D and quasi-2D models in applying the full 2D unsteady flow equations. This approach is necessary to model the complex interaction between watercourses and floodplains and converging and diverging of flows through structures. The channel and floodplain topography is defined using a high resolution DEM for greater accuracy in predicting flows and water levels and the interaction of in-channel and floodplain areas.

4.2.1 Topography and Bathymetry

The ability of the model to provide an accurate representation of the flow distribution on the floodplain ultimately depends upon the quality of the underlying topographic model. For this study, a 2m by 2m gridded DEM was derived from the NSW LPI LiDAR survey datasets.

Bathymetric survey data (NSW Office of Environment and Heritage, 1999) is available for the Manning River. It extends from the ocean entrance at Old Bar and Harrington to just downstream of Killawarra and includes major and minor tributaries. The channel topography has been incorporated into the 2D model representation and is discussed further in Section 4.2.4.

4.2.2 Extents and Layout

Consideration needs to be given to the following elements in constructing the model:

- Topographical data coverage and resolution;
- Location of recorded data (e.g. levels/flows for calibration);
- Location of controlling features (e.g. dams, levees, bridges);
- Desired accuracy to meet the study’s objectives; and
- Computational limitations.

With consideration to the available hydrographic survey information and local topographical and hydraulic controls, a 2D model was developed extending from the ocean, through the entrances at Harrington and Farquhar Inlet and upstream along the north and south arms of the Manning River past Wingham. The upstream extent of the model terminates just upstream of the stream flow gauge location on the Manning River at Killawarra. The hydraulic model incorporates the entire lower Manning River floodplain. The area modelled within the 2D domain comprises a total area of some 580km^2 which represents around 38% of the Manning River catchment area downstream of Killawarra. The extent of the hydraulic model is shown in Figure 4-1

A TUFLOW 2D domain model resolution of 20m was adopted for study area. It should be noted that TUFLOW samples elevation points at the cell centres, mid-sides and corners, so a 20m cell size results in DEM elevations being sampled every 10m. This resolution was selected to give necessary detail required for accurate representation of floodplain and channel topography and its influence on flood flows. It also considers the need to largely restrict modelled depths as being less than the cell width and to achieve model simulations within a reasonable run time.

A 20m grid model resolution may not pick up topographical features at a finer scale than 10m (e.g. the crest of a roadway embankment or ridgeline of a channel bank). These features have been reinforced into the 2d model with "z-shapes" (3D topographical breaklines)..

4.2.3 Hydraulic Roughness

The development of the TUFLOW model requires the assignment of different hydraulic roughness zones. These zones are delineated from aerial photography and cadastral data identifying different land-uses (e.g. forest, cleared land, roads, urban areas, etc.) for modelling the variation in flow resistance.

The hydraulic roughness is one of the principal calibration parameters within the hydraulic model and has a major influence on flow routing and flood levels. The roughness values adopted from the calibration process is discussed in Section 5.

4.2.4 Channel Network

Bathymetric (hydrographic) survey from the NSW Office of Environment and Heritage (OEH) is available for the tidal reaches of Manning River and its tributaries.

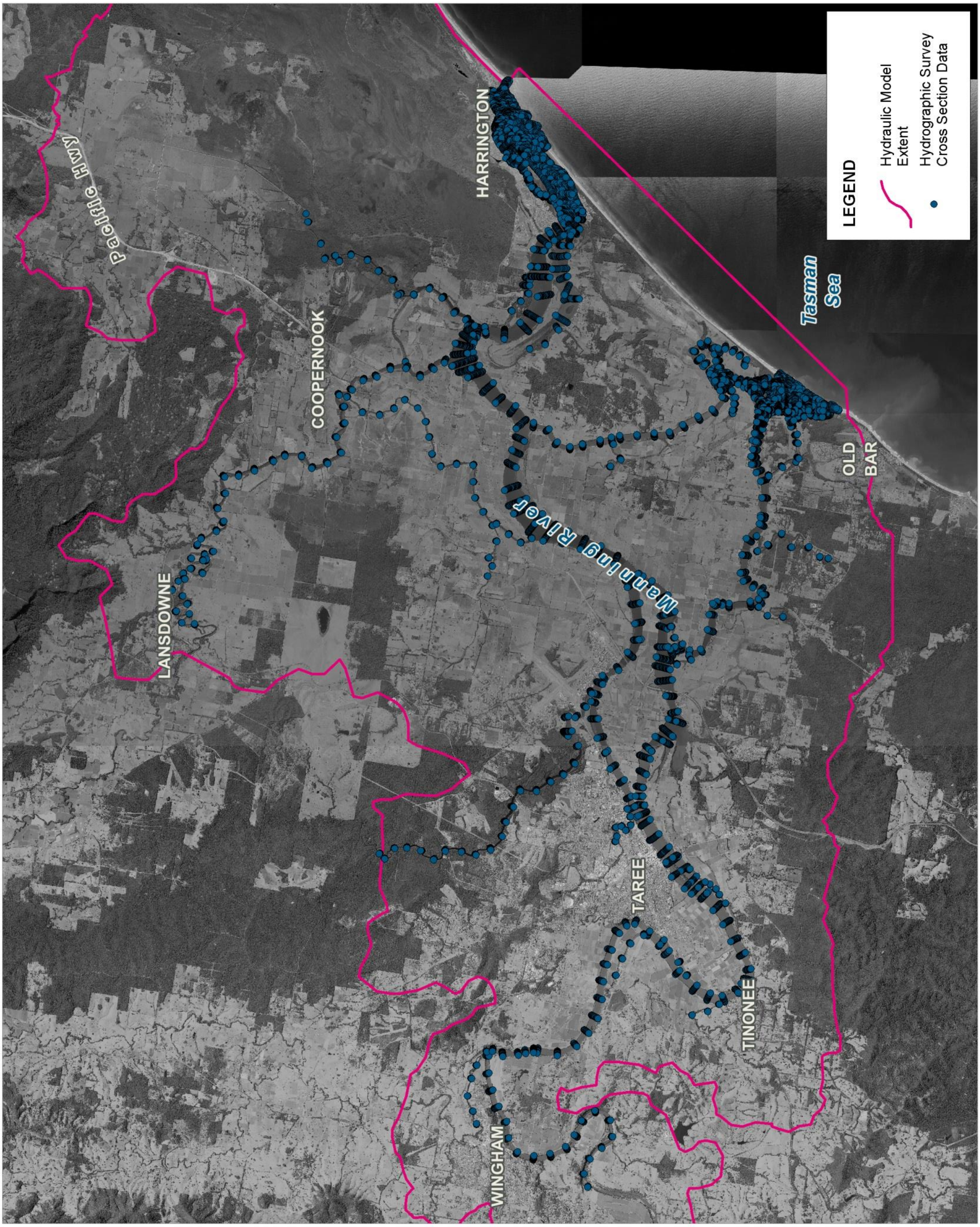
Figure 4-2 presents the coverage of the available hydrographic survey sections.

Custom GIS tools were utilised to interpolate between the cross sections to provide a continuous river bathymetry, dependent on the river flow direction. Channel cross sections were interpolated into a mesh, which was converted into a 2m by 2m gridded DEM, as indicated on Figure 4-3.

This is of high value to the study, as it provides detailed representation of channel capacity. This data was integrated with the LiDAR to provide a composite DEM of the channel and floodplain.

4.2.5 Structures

There are a number of large bridge crossings over the watercourses within the model extents. These structures vary in terms of construction type and configuration, with varying degrees of influence on local hydraulic behaviour. Incorporation of these major hydraulic structures in the model provides for simulation of the hydraulic losses associated with these structures and their influence on peak water levels within the study area.



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Bathymetric Survey Coverage	**4-2**	**A**

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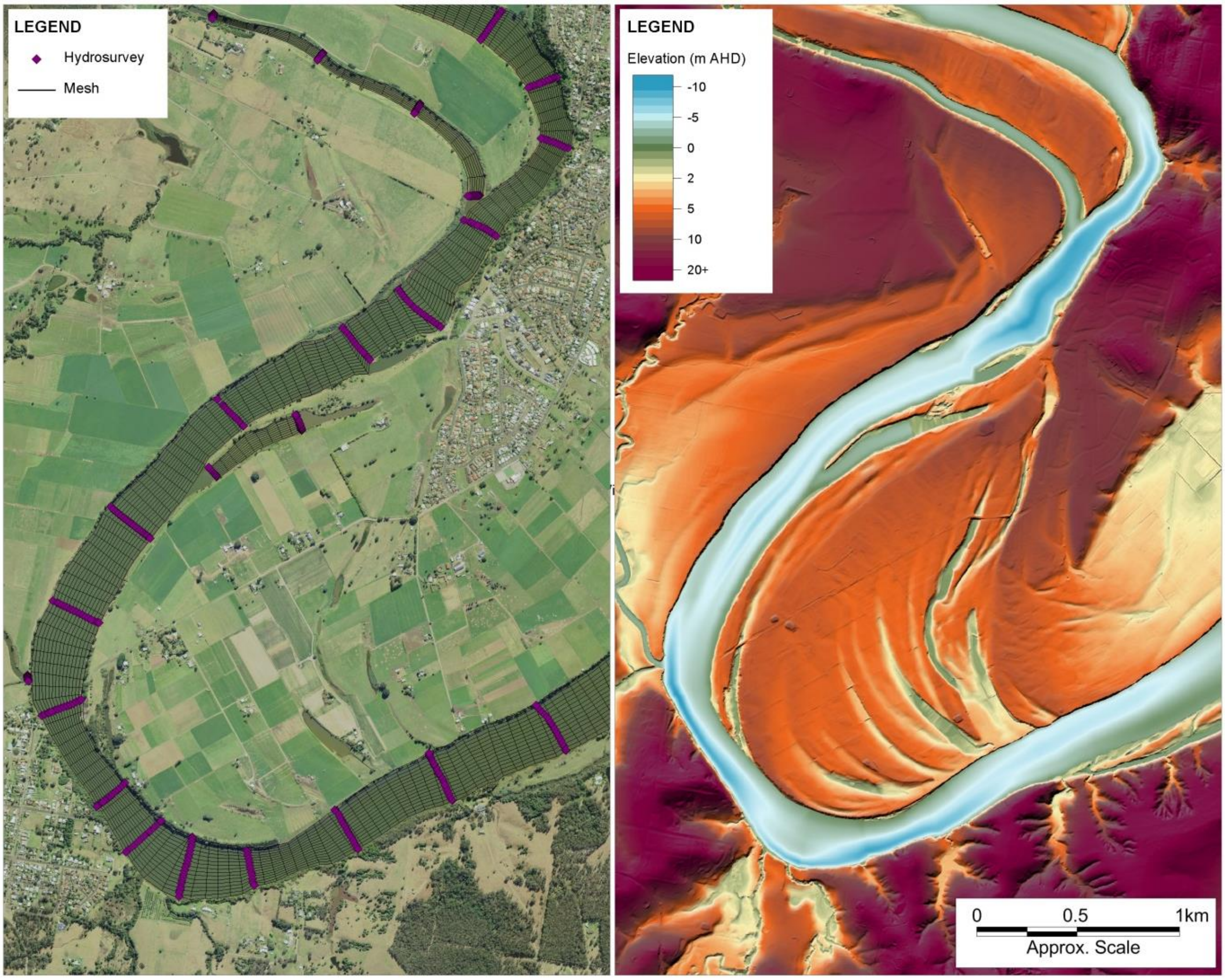


Figure 4-3 Example of Manning River Bathymetry Interpolation at Taree West

The following bridges have been included in the hydraulic model:

- Gloucester Road bridge, Killawarra;
- Bight Bridge, Wingham-Tinonee Road, Wingham;
- Martin Bridge, Commerce Street/Manning River Drive, Taree;
- Dumaresq Island Road, Cundletown; and
- Pacific Highway Taree Bypass (three bridge spans).

These hydraulic structures have been modelled as flow constrictions within the 2D domain. This utilises the layered flow constriction option available in TUFLOW, which represents the bridge superstructure and losses. Obvert levels, road crests and hand rail obstruction details are entered along with additional form losses.

There are a number of smaller bridges and culverts allowing for drainage under road embankments. To allow for the backwater influence from the Manning River to fill storage areas behind these embankments, minor flow connections have been provided through the embankment within the 2D domain.

4.2.6 Boundary Conditions

The upstream model limit corresponds to input flow hydrographs on the Manning River at the Killawarra stream gauge site.

The local catchment runoff is determined through the hydrological model and is applied to the TUFLOW model as flow vs. time inputs. These are applied as distributed inflows along the modelled watercourse reaches.

The downstream model limit corresponds to the water level in the Tasman Sea.

The adopted boundary conditions for the calibration and design events are discussed in Section 5 and Section 6 respectively.

4.3 Entrance Dynamics Model

The Manning River has two entrances that discharge flows to the Tasman Sea. The main entrance at Harrington is permanently open, but the location of the river channel and extent of shoaling in the entrance are dynamic. The second entrance at Farquhar Inlet is highly dynamic and subject to intermittent periods of opening and closure. The dynamic nature of the entrances is influenced by coastal sediment transport processes and scour during significant catchment flood events. During decades that are relatively flood-free the entrances will exhibit a period of increased shoaling through a build-up of sediment. Following periods of frequent flooding large volumes of material will be washed from the entrances into the sea, resulting in a relatively open condition that will gradually shoal again over time. These processes also result in the location of the entrance channel migrating along the dune.

In order to assess the impact that the entrance dynamics may have on catchment flood events a representative entrance condition and breakout dynamic was incorporated into the TUFLOW Classic flood model. The TUFLOW-FV modelling software was used to derive an appropriate level of entrance scour from catchment flood events. TUFLOW-FV is a flexible mesh, finite volume numerical model that simulates hydrodynamic, sediment transport and water quality processes in oceans, coastal waters, estuaries and rivers.

The TUFLOW-FV model was developed to provide a 2D representation of the Manning River ocean entrances at Harrington and Farquhar Inlet, with the purpose of simulating the sediment transport processes occurring during flood events. The approach adopted in this study is an iterative process whereby the level of entrance scour resulting from various magnitude flows is determined in TUFLOW-FV to be simplistically represented within the TUFLOW Classic model.

A geomorphologic module compatible with TUFLOW Classic, developed by BMT WBM, was considered for use in this study to simulate entrance scour. Use of this module would have eliminated the need to develop two separate models; however, the software has not yet been commercially released and inclusion of the module can compromise efficient computational run times. TUFLOW-FV was selected in preference to the geomorphologic module as it is commercially available, allowing for ease of handover and future application of the model.

Incorporating a dynamic entrance breakout representation into the flood modelling enables the sensitivity of upstream flood conditions to the entrance conditions to be determined.

4.3.1 Topography and Bathymetry

The 2m by 2m gridded DEM (including bathymetry) developed for the TUFLOW Classic model was used to sample cell elevations for the TUFLOW –FV model.

The final composite DEM used to assign bed elevations within the model is shown in Figure 4-4. The bathymetry was extended offshore to an assumed depth of -10m AHD near the ocean boundary. Localised adjustment of cell elevations was undertaken to adequately represent topographical features with finer resolution than was picked up in the cell-centre sampling.

4.3.2 Extents and Layout

The TUFLOW-FV model includes the entrance of Harrington and Farquhar Inlet. The mesh extends upstream along the north and south arms of the Manning River, to include the floodplain between that becomes active under large flows through the system. The extent and layout of the mesh is included in Figure 4-4.

4.3.3 Model Parameters

The adopted model parameters, as shown in Table 4-2, are all within reasonable bounds and have been adopted for calibrated models for other studies in similar environments. Sensitivity testing of adopted parameters was undertaken.

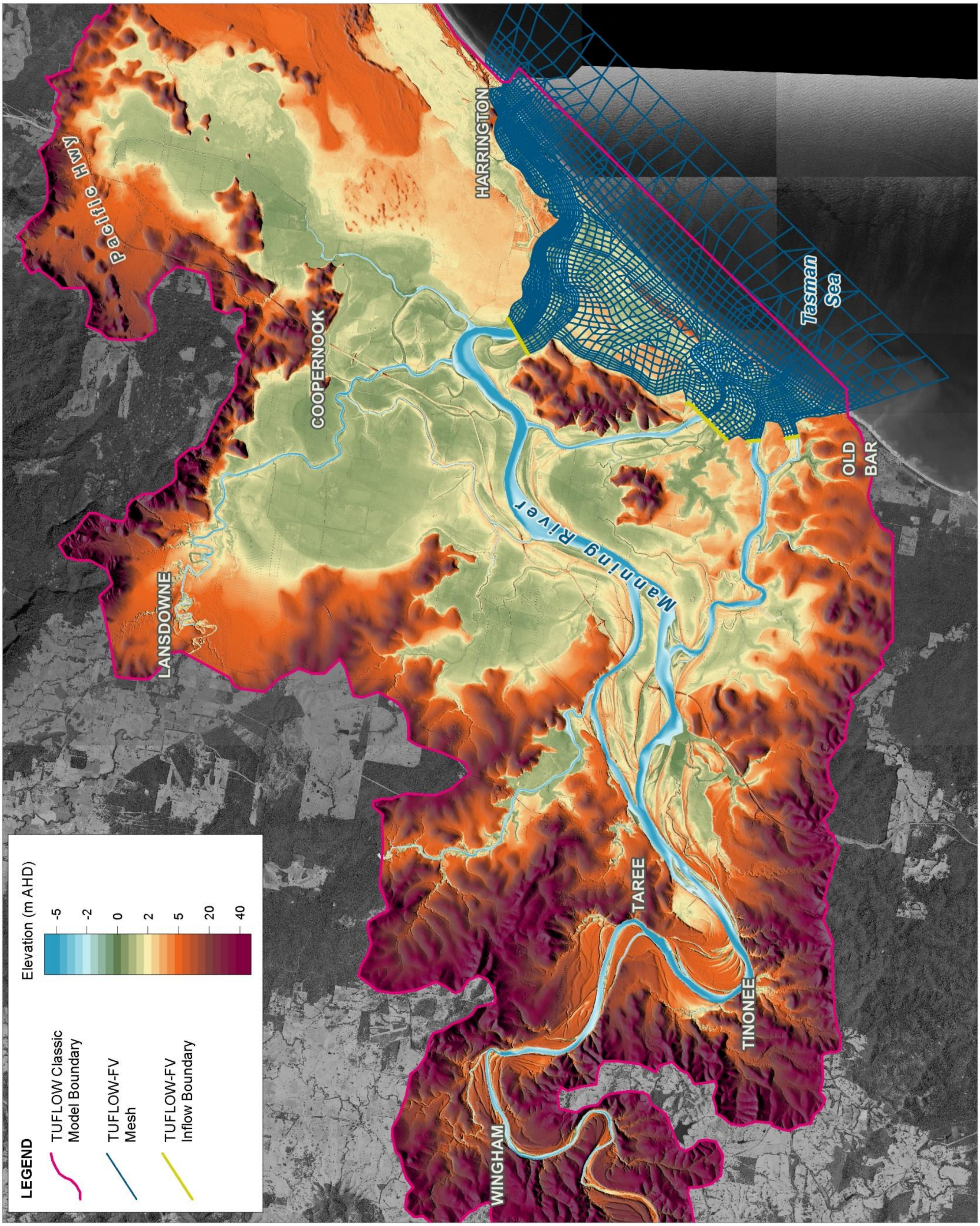
Table 4-2 Adopted Parameters for TUFLOW-FV Modelling

	Parameter	Value
Breakout Processes	Under Water Slump Slope	30°
	Dry Slump Slope	11°
	Bed Load Scaling	1.0
	Manning's 'n' (channel) *Tidal waterways* *Harrington entrance* *Farquhar Inlet entrance*	 0.015 0.01 0.02
Sediment Characteristics	d_{10}	0.160 mm
	d_{50}	0.230 mm
	d_{90}	0.375 mm

4.3.4 Boundary Conditions

As per the TUFLOW Classic model, the downstream model limit corresponds to the water level in the Tasman Sea.

The upstream model limit corresponds to flow rates along the Manning River. Flows were extracted from intermediate iterations of the TUFLOW Classic model to be applied as flow vs. time input boundaries at the upstream model extents.



Title: **TUFLOW-FV Model Extent and Geometry**	Figure: **4-4**	Rev: **A**

N

0 2.5 5km

Approx. Scale

BMT WBM

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5 Model Calibration

5.1 Selection of Calibration Events

The selection of suitable historical events for calibration of computer models is largely dependent on available historical flood information. Ideally the calibration and validation process should cover a range of flood magnitudes to demonstrate the suitability of a model for the range of design event magnitudes to be considered.

The model calibration is largely based on the June 2011 event, due to the volume of data available. The March 1978 flood event was also used for calibration of the hydraulic model to a larger flood event. The February 1929, February 1990 and March 2013 events were used for model verification.

The available data, modelling approach and model results for each of these events are discussed in further detail in the following sections. A number of other potential verification events were available, such as 1968, 1976, and 1977 but were not chosen in preference to the above events given their smaller magnitude and limited data availability.

5.2 June 2011 Model Calibration

5.2.1 Model Inflow and Channel Roughness

The focus of the June 2011 model calibration process was essentially to determine the most appropriate set of flow and roughness conditions, in order for the model to be able to reasonably reproduce observed flood behaviour within the catchment.

As the observed flood levels are a function of both flows and roughness there are a number of combinations of the two that will produce similar levels. Spot gaugings (measured combinations of flow rate and water level) recorded at streamflow gauge sites are a useful dataset for determining appropriate model roughness values. A large number of these at high flow rates will provide a good rating curve (flow vs. level relationship), which can be matched within the model by selecting an appropriate roughness value. Fortunately, the Killawarra gauge has been operational since 1945 and has a relatively good set of larger magnitude spot gaugings. As spot gaugings are manually calculated they can be inaccurate, so care must be taken when analysing the data set.

The process adopted for this study involved defining an appropriate rating curve at the Killawarra streamflow gauge to determine the likely peak flow rate during the June 2011 event, which when used in combination with channel roughness, should achieve a good match to water level time series recorded further downstream at Wingham, Taree West and Taree.

The PINNEENA database released by the Office of Water contains historical information for streamflow gauges across NSW. Figure 5-1 presents the available spot gaugings and the various high flow rating curves that have been adopted for the gauge site at Killawarra.

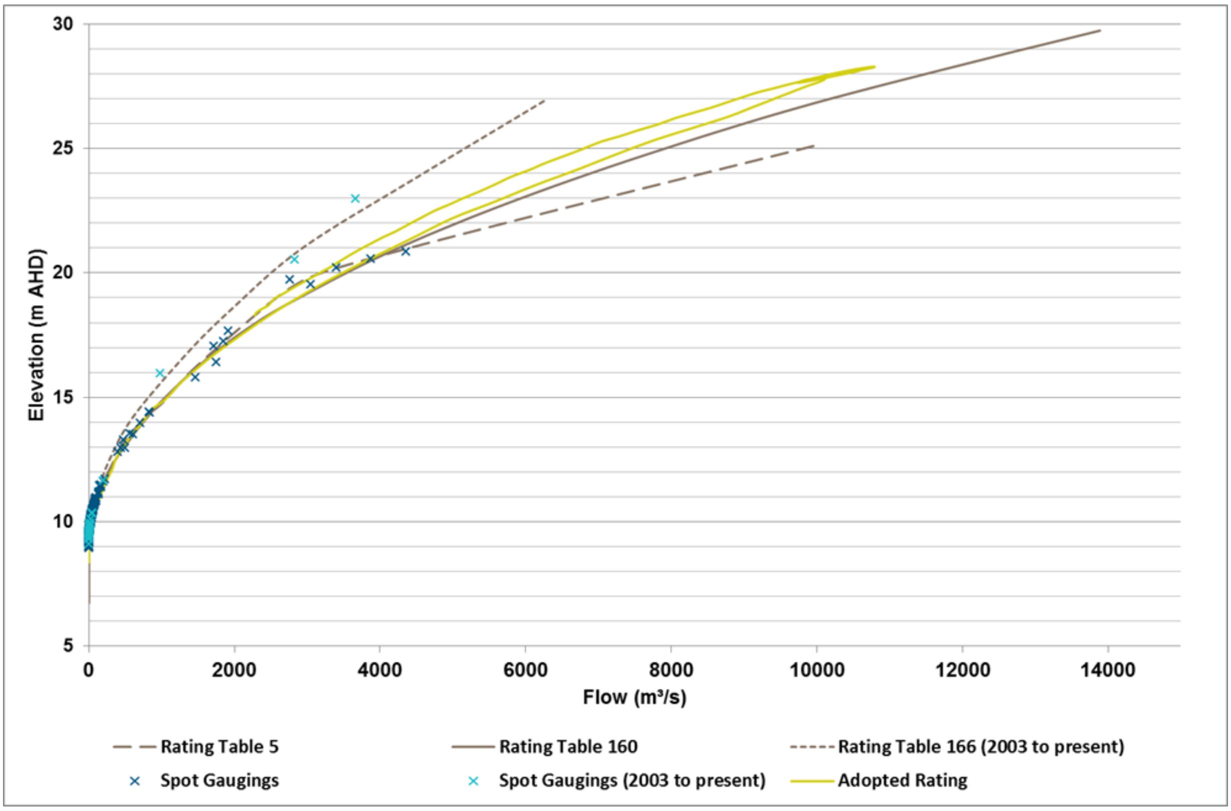


Figure 5-1 Rating Curve Analysis for the Killawarra Gauge

"Rating Table 166" is the most recently derived curve for the site and utilises spot gaugings recorded post-2003. It can be seen that it follows a noticeably different alignment to the previous rating curves adopted over the history of the gauge. It is likely that these latest spot gaugings are unreliable. This notion is supported by anecdotal evidence from Council and SES staff that indicated the gauging instrumentation was damaged during the 2011 event, potentially resulting in erroneous recordings.

Manning's 'n' values of 0.013 and 0.025 were adopted for the channel roughness in tidal (downstream of Taree) and non-tidal areas (upstream of Wingham) respectively. This was based on the most appropriate values from the available literature and previous experience with channels of a similar nature. A transitional roughness value of 0.02 was adopted for the Manning River channel passing through Taree West and Taree. The Manning's 'n' value assigned to entrance areas is discussed in Section 5.3. Inclusion of densely vegetated channel banks and channel islands was found to be necessary in order to achieve a good model calibration to recorded peak water levels within the study area.

The rating curve generated at the Killawarra site from the TUFLOW model based on these adopted channel roughness values (also shown on Figure 5-1) falls more in line with the historical rating curves. As such, the 'recorded' flow hydrograph at the Killawarra gauge location was scaled to match the peak flow rate required to generate the observed peak water level for the June 2011 event. A peak flow rate of just over 7,000m^3/s was required to generate the recorded peak water level of 24.85m AHD – almost 40% more flow than estimated using "Rating Table 166." The adopted peak flow rate at Killawarra for the June 2011 event is presented in Figure 5-2. This flow hydrograph forms the upstream boundary condition of the hydraulic model.

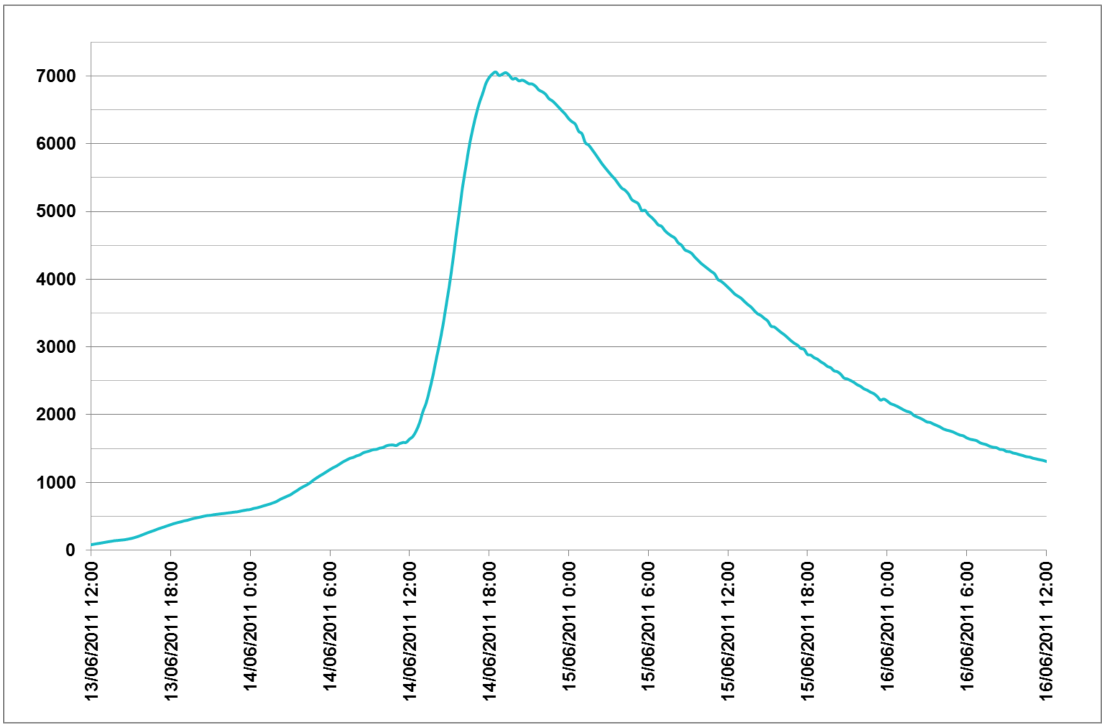


Figure 5-2 Adopted Flow Hydrograph for the June 2011 Event at the Killawarra Gauge

5.2.2 Entrance Geomorphology

The performance of the model at the downstream extent is influenced by the entrance geomorphology of the Harrington and Farquhar Inlets. Both entrances are dynamic, with significant scouring known to occur during flood events due to the high velocities associated with large magnitudes of flow.

In order to achieve a closer match between modelled and observed water level hydrographs at the Harrington and Farquhar Inlet streamflow gauges, it was necessary to represent the initial entrance geometry and expected entrance breakout conditions within the hydraulic model. A TUFLOW-FV model was developed to simulate the level of sediment transport and magnitude of scour associated with the flood flows of the June 2011 event. The development and configuration of the TUFLOW-FV model is detailed in Section 5.3.

5.2.3 Rainfall Data

Given the relative size of the catchment upstream of Killawarra and the rainfall observed across the catchment, the flow through Killawarra will be driving the peak flow rate through the study area during this event, with minor influence from local inflows downstream. The XP-RAFTS hydrological model was used to simulate runoff from local catchments downstream of Killawarra

The BoM pluviograph at Taree Airport AWS (060141) recorded continuous rainfall data during the June 2011 event and is presented as a one hourly rainfall hyetograph in Figure 5-3. The temporal variation of rainfall recorded at this gauge was deemed to be representative of rainfall over the entire lower catchment areas and was applied to the relevant sub-catchments within the XP-RAFTS hydrological model.

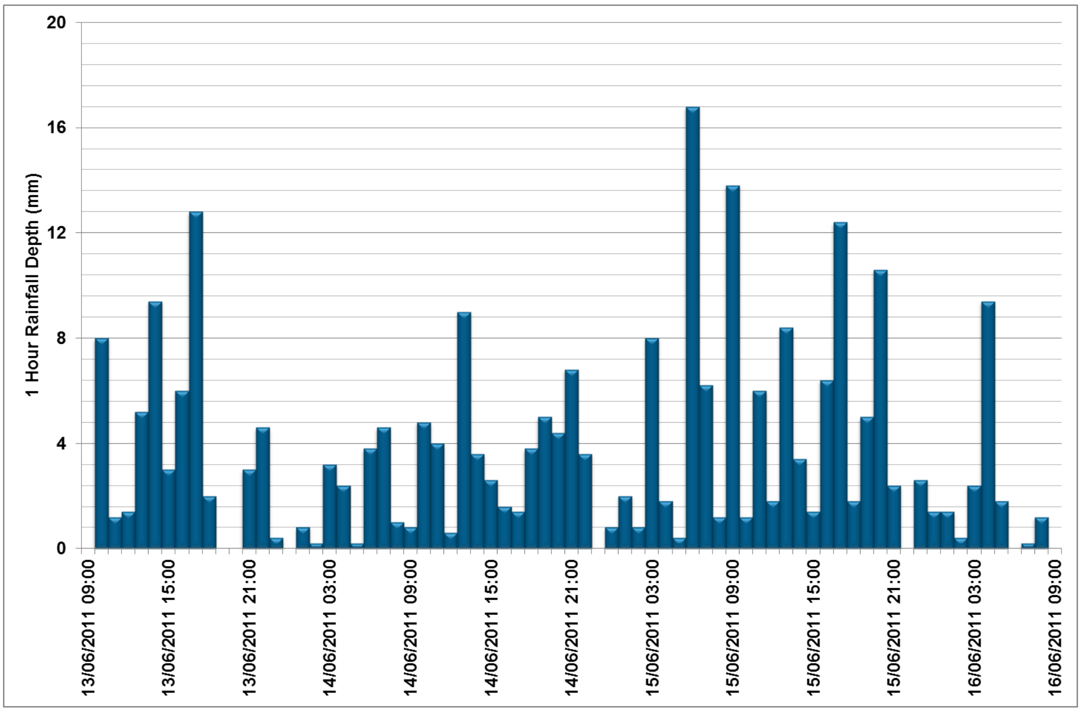
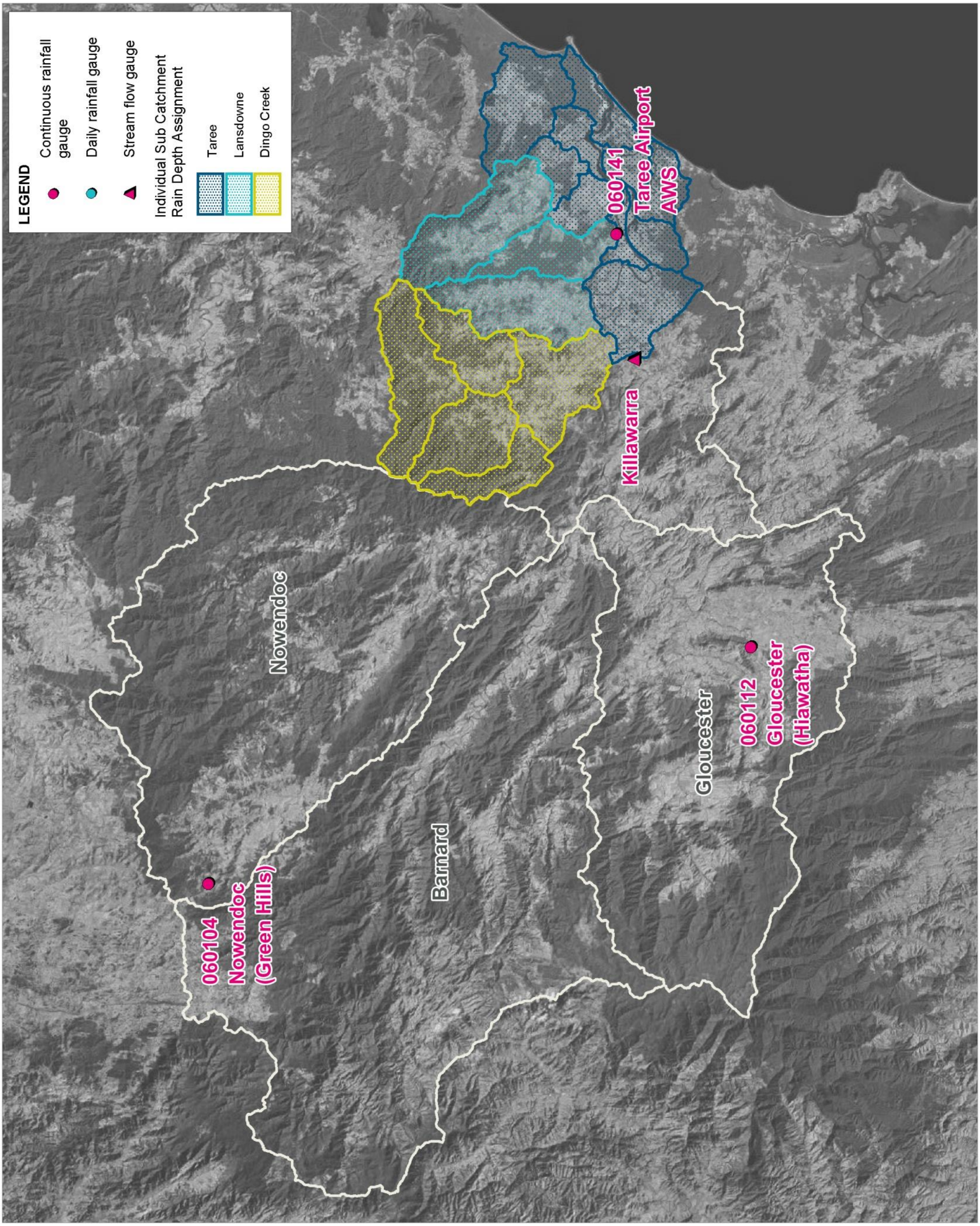


Figure 5-3 Rainfall Hyetograph for the June 2011 Event at the Taree Gauge

Analysis of daily rainfall records indicated that rainfall depth varied across the lower sub-catchments over the three duration of the 2011 event. From 9am on the 13th to 9am on 16th of June, a total of 260mm was recorded at the Taree gauge, with more rainfall observed over the Dingo Creek and Lansdowne catchment areas. Consequently, the total rainfall depth was increased to around 350mm and 325mm for these areas, respectively, for the 72 hour period. The distribution of total rainfall across the lower sub-catchments is shown in Figure 5-4. The relative size of the catchment upstream and downstream of Killawarra is evident on the figure.

In order to gain an appreciation of the relative intensity and magnitude of the June 2011 event, the recorded rainfall depth for various durations within the storm is compared with the Intensity Frequency Duration (IFD) data across the catchment. The AR&R is in the process of revising the design flood estimate guidelines, and have released updated 2013 IFDs. However, these are currently to be used for sensitivity purposes only and not adopted for design flood estimation, as their appropriate use is linked to the adopted design temporal rainfall patterns (the revision of which is still underway).

Design IFD rainfall curves were calculated based on the methods presented in AR&R for the 1987 and 2013 datasets, as discussed later in Section 6.3. IFD data varies spatially and with catchment size. As the total Manning River catchment covers a vast area, design rainfall intensities will vary across the catchment. The IFD data shown on the figure is representative of storms across the entire Manning River catchment, with Aerial Reduction Factors (ARFs - base on a catchment area of 8,200km^2) applied, as per AR&R guidelines. Figure 5-5 presents the recorded June 2011 rainfall intensities against both the 1987 IFDs and 2013 IFDs, for comparison.



Title:

June 2011 Sub Catchments and Rainfall Distribution

Figure: 5-4

Rev: A

N

0 12.5 25km
Approx. Scale

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In addition to the pluviograph at Taree, BoM operated rainfall gauges at Nowendoc (Green Hills) (060104) and Gloucester (Hiawatha) (060112) were reviewed. The rainfall depth vs. duration curves for these gauges are also presented on Figure 5-5. As the gauges recorded rainfall at a point, the depth vs. duration curves from the gauge recordings have also been presented against IFD data applicable to a point within the Manning River catchment (i.e. the ARF's are not applied) and can be seen in Figure 5-6.

With reference to Figure 5-5, when assessing rainfall across the catchment as a whole, the June 2011 event falls somewhere between a 20% AEP and 5% AEP event based on the 1987 IFD curves.

If assessing rainfall recorded at each of the gauges individually, the recorded depth vs. duration profile for the lower floodplain area at Taree is below a 50% AEP event for durations up to 12 hours. Rainfall depths steadily increase for larger durations, placing the 72 hour recorded total just short of the estimated 10% AEP rainfall event. The variation in rainfall intensity across the catchment is evident. The Gloucester and Nowendoc gauges recorded a similar depth vs. duration profile, with the exception of the 24 hours duration where the Gloucester gauge recorded the largest total rainfall.

The difference between the two IFD comparison figures demonstrates the lower likelihood that all areas within the wider Manning River catchment will experience rainfall of consistent magnitude during the same event.

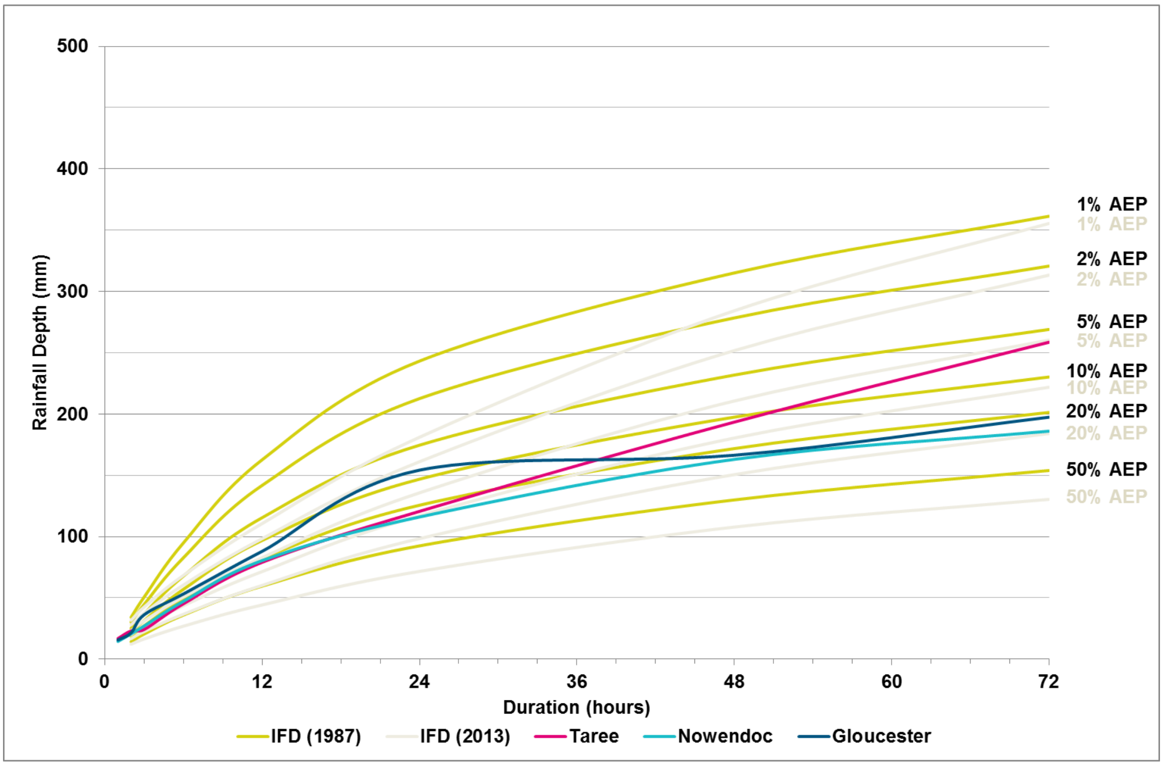


Figure 5-5 Comparison of Recorded June 2011 Rainfall with IFD Relationships (Entire Catchment Intensities)

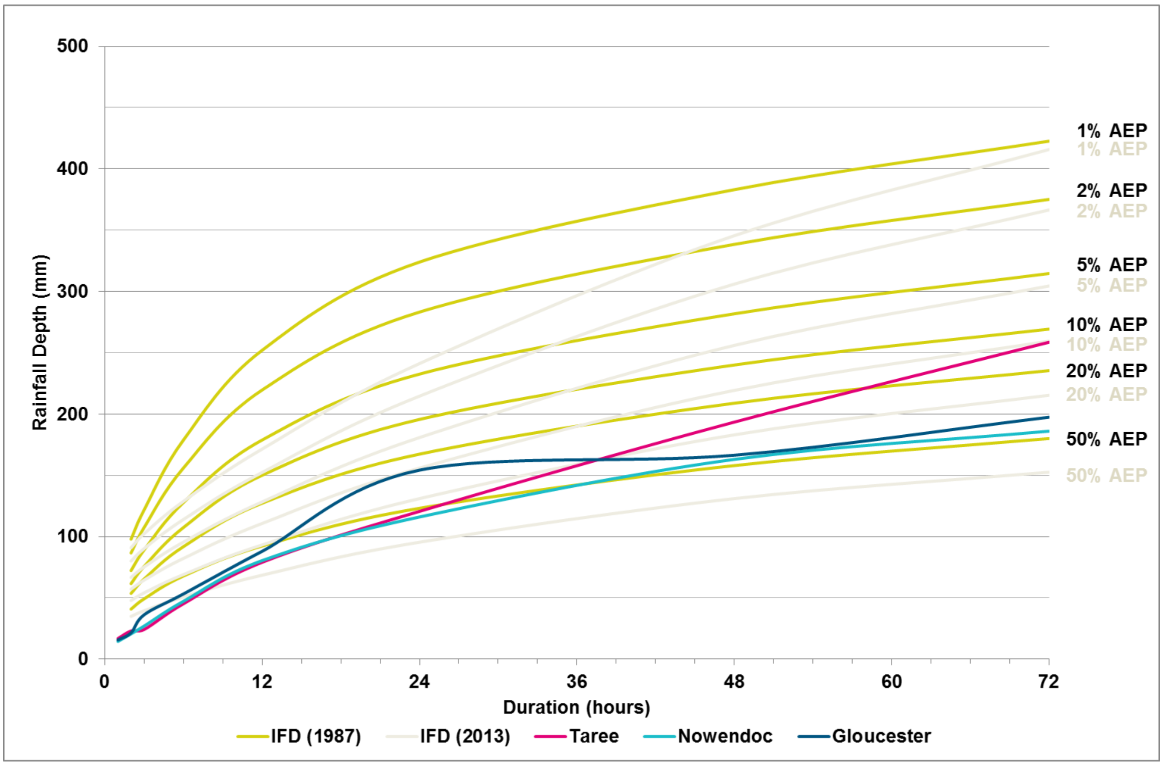


Figure 5-6 Comparison of Recorded June 2011 Rainfall with IFD Relationships (Point Intensities)

5.2.4 Antecedent Conditions

The antecedent catchment condition, reflecting the degree of wetness of the catchment prior to a major rainfall event, directly influences the magnitude and rate of runoff. The initial loss-continuing loss model has been adopted in the XP-RAFTS hydrologic model developed for this study. The initial loss component represents a depth of rainfall effectively lost from the system and not contributing to runoff, and simulates the wetting up of the catchment to a saturated condition. The continuing loss represents the rainfall lost through soil infiltration once the catchment is saturated and is applied as a constant rate (mm/hr) for the duration of the runoff event.

Typical design loss rates applicable for eastern NSW catchments are initial loss of 10 to 35 mm and continuing loss of 2.5mm/h (AR&R, 2001). For historical events however, the initial loss is indicative of the catchment wetness and prior rainfall to the modelled storm burst.

Daily rainfall records indicate that between 10-50mm of rainfall was typically recorded in the 24 hours to 9am on both the 12th and 13th of June at various gauge locations across the Manning River catchment. An initial loss 10mm was adopted to account for the expected level of wetness of the catchment for this event.

5.2.5 Adopted Model Parameters

The final values adopted, as shown in Table 5-1 were found to give a good result in representing the recorded water level hydrographs at the stream flow gauges located at Wingham, Taree West, Taree, Dumaresq Island, Croki, Farquhar Inlet and Harrington.

Table 5-1 Adopted Model Parameters for the June 2011 Event

Parameter	Value
Initial Loss (mm)	10
Continuing Loss (mm/hr)	2.5
PERN *Forested* *Cleared*	 0.12 0.06
Bx (storage routing parameter)	1.0
Manning's n (channel) *Tidal waterways (including Harrington entrance)* *Farquhar Inlet entrance* *Transition from tidal to non-tidal waterways* *Non-tidal waterways*	 0.013 0.02 0.02 0.025
Manning's n (floodplain) *Pasture / cleared land* *Urban areas* *Dense vegetation*	 0.04 0.06 0.12

5.2.6 Downstream Boundary Condition

Ocean tide data measured at Port Stephens - located approximately 100 km south along the coast from the study area - was obtained from MHL. Although measured at a different location, the timing and maximum/minimum level of the tidal signature will be close enough to that at the Harrington and Old Bar entrances that data is suitable for use.

The Port Stephens tide data is shown in Figure 5-7. However, it is also necessary to include consideration of wind and wave setup, due to the highly shoaled entrance conditions of the Manning River. Although not appropriate to apply the full wave setup typical for the open coast (i.e. ~15% of the significant wave height) a reduced amount is still required.

Observed data from Harrington suggests that a wave setup component of around 0.2m is typically evident during normal conditions. This is comparable to the wave setup on the open coast and is reflective of the shoaled nature of the entrance. However, during a flood event the entrance would open considerably and the effect of wave setup would become less significant. Therefore a wave setup of 0.2m has been applied to the Port Stephens tidal recording throughout the duration of the event. This is representative of around a full application of wave setup at the onset of the event, diminishing to around a 20% application during the event peak (around 3% of significant wave height during a storm event).

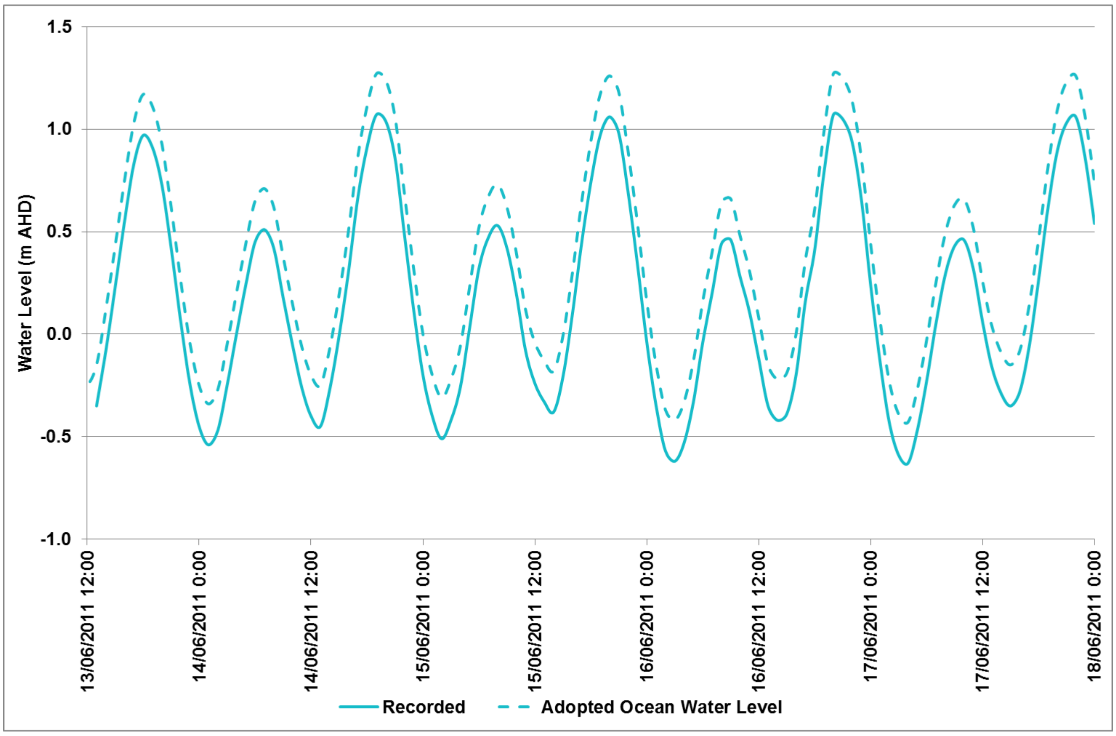


Figure 5-7 Tidal Data for the June 2011 Event Measured at Port Stephens and the Adopted Ocean Boundary Condition (to include Wind and Wave Setup)

5.2.7 Observed and Simulated Flood Behaviour

The effectiveness of the model representation of the catchment response to the assumed 2011 inflow at Killawarra and the local rainfall inflows can be assessed through comparison of the recorded and modelled hydrographs at the various water level gauging stations within the catchment. Given the uncertainty surrounding appropriate rating curves at the gauges, this comparison has been undertaken using water levels.

There is a good number of streamflow gauges along the Manning River within the study area which have continuously recorded water level data for the June 2011 event. In addition to the NSW Office of Water operated gauge at Killawarra, Manly Hydraulics Labouratory (MHL) operates seven gauges located at Wingham, Taree West, Taree, Dumaresq Island, Croki, Harrington and Farquhar Inlet.

The following figures (Figure 5-8 to Figure 5-14) present the modelled water level against the recorded data for the June 2011 at each gauge location.

The model accurately replicates the peak flood levels within around 0.1m of the recorded level at all gauge locations. The overall shape of the flood hydrograph is also well represented in all instances.

For the purpose of demonstrating the significance of the entrance breakout and its influence on modelled flood levels, the results of an alternate scenario are also presented where the entrance breakout is not represented, i.e. the adopted initial entrance geography is constant throughout the simulation.

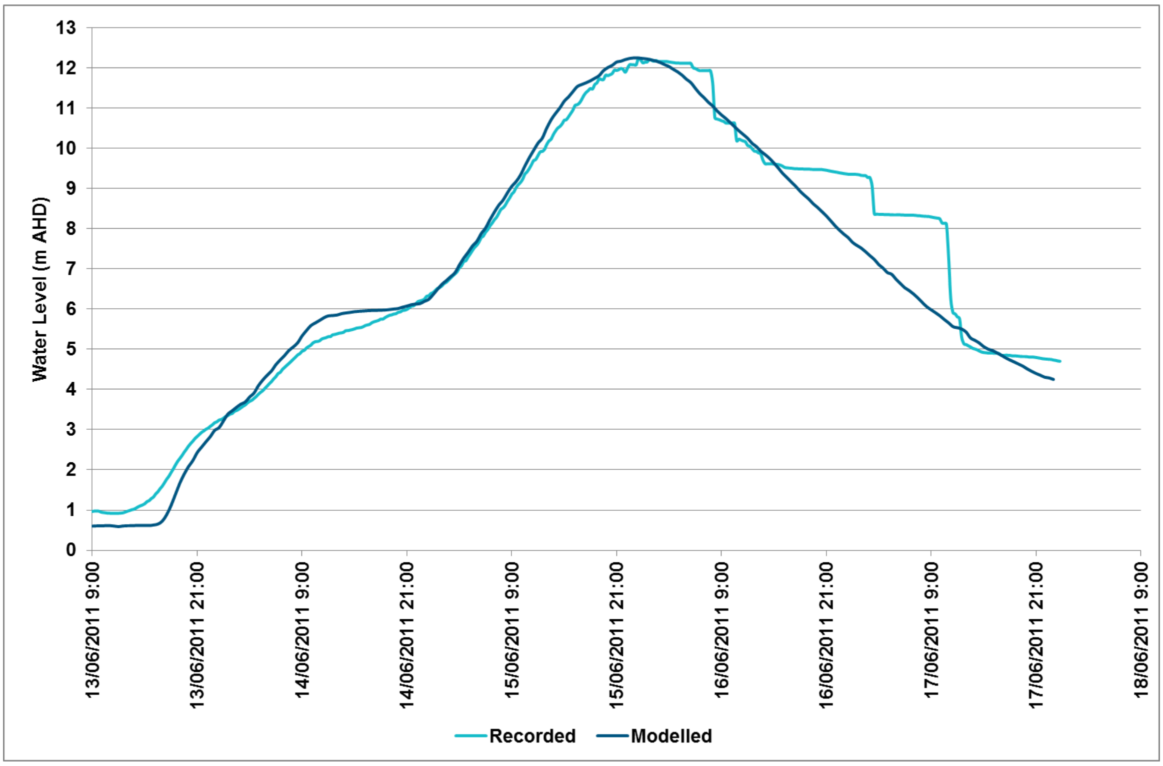


Figure 5-8 Comparison of Recorded and Modelled Water Levels at the Wingham Gauge for the June 2011 Event

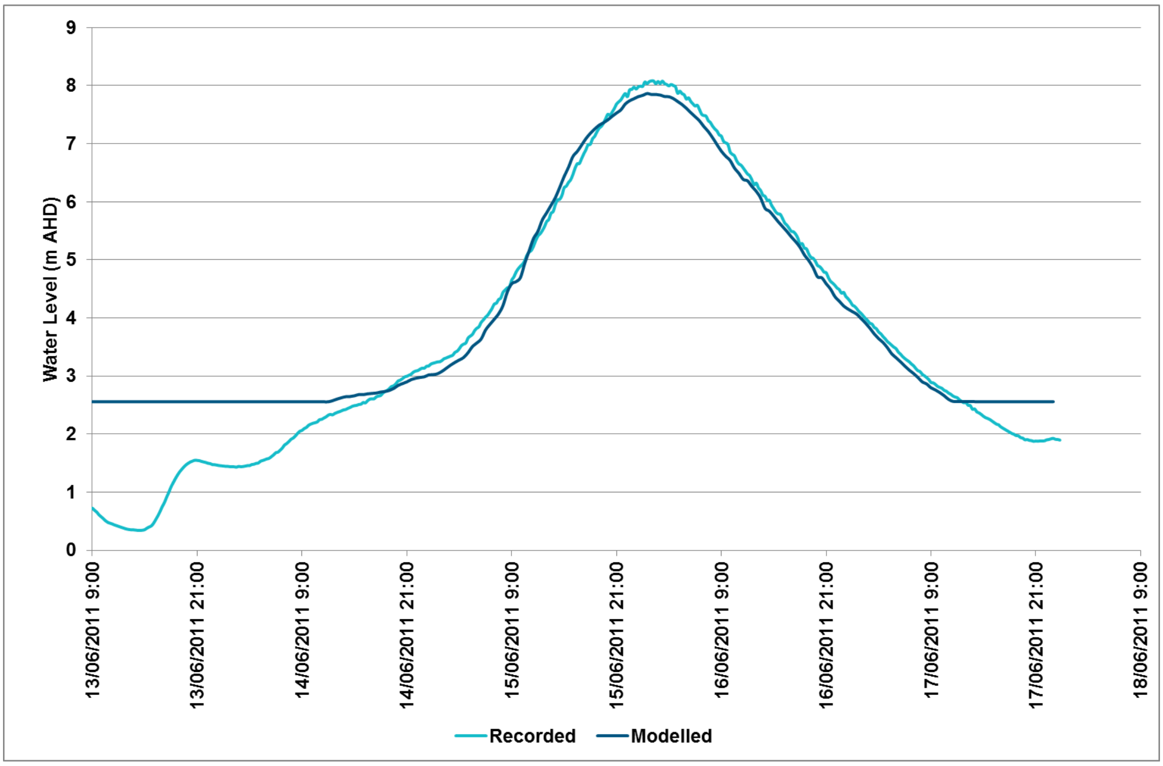


Figure 5-9 Comparison of Recorded and Modelled Water Levels at the Taree West Gauge for the June 2011 Event

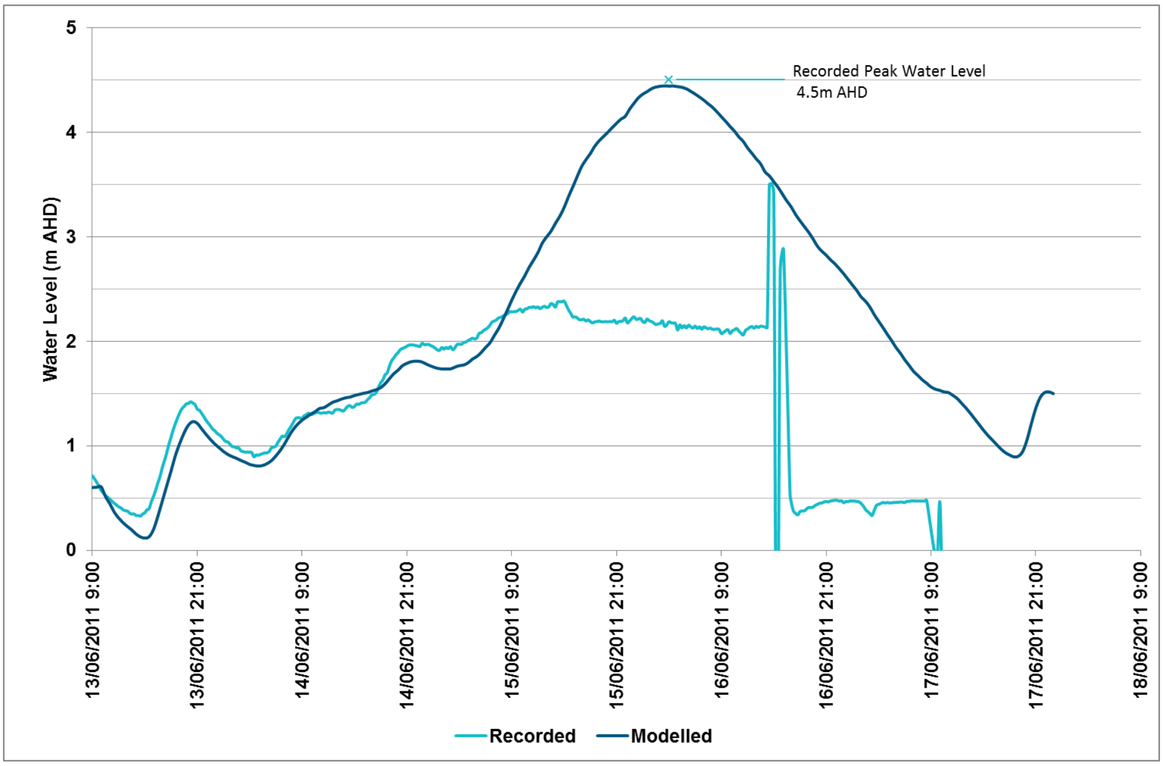


Figure 5-10 Comparison of Recorded and Modelled Water Levels at the Taree Gauge for the June 2011 Event

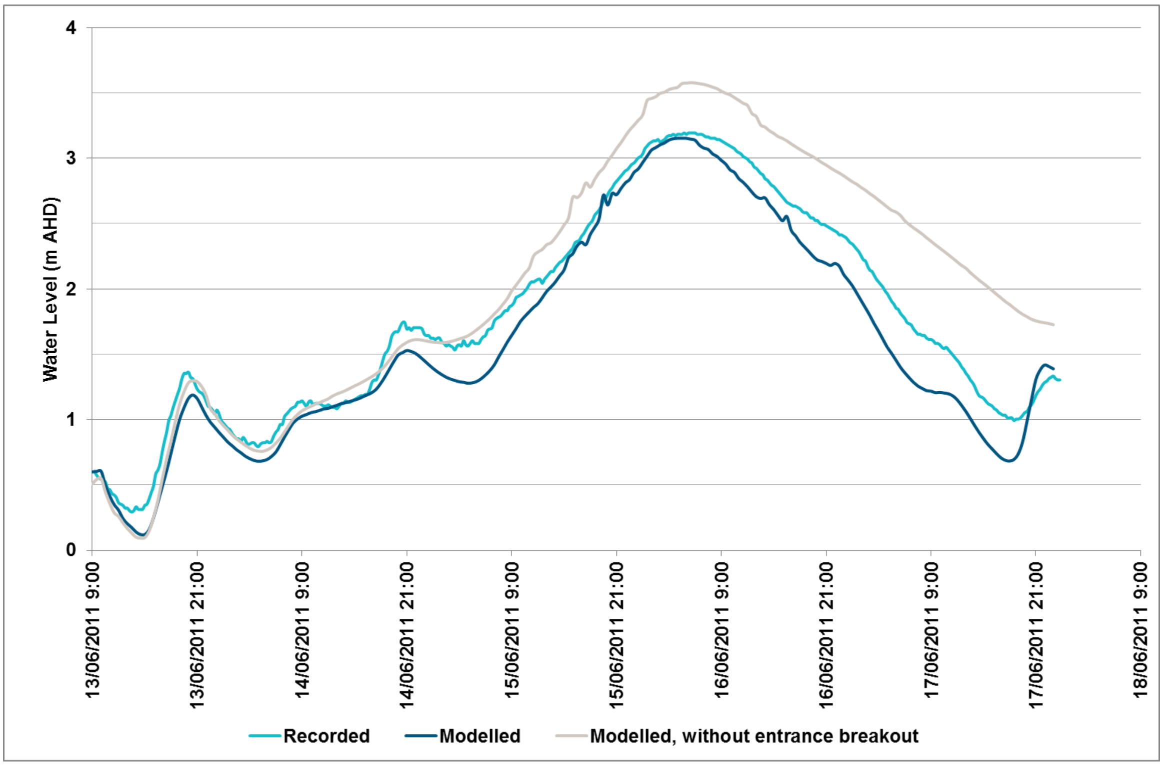


Figure 5-11 Comparison of Recorded and Modelled Water Levels at the Dumaresq Island Gauge for the June 2011 Event

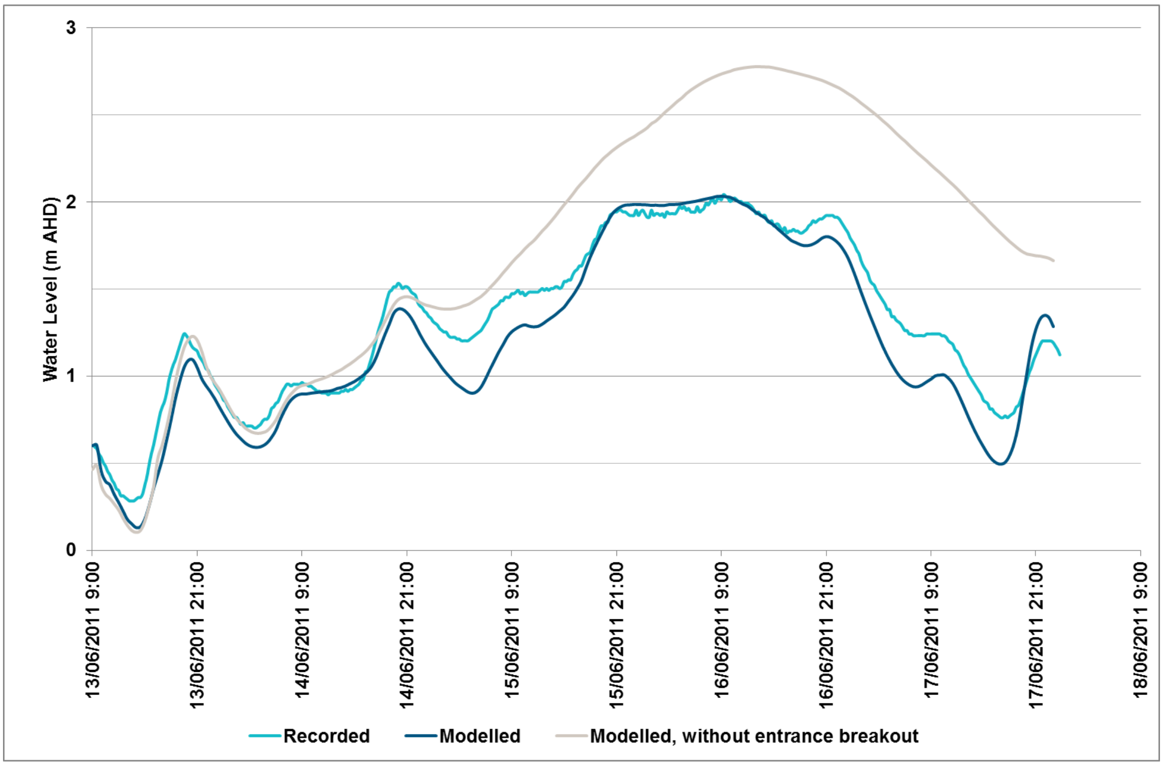


Figure 5-12 Comparison of Recorded and Modelled Water Levels at the Croki Gauge for the June 2011 Event

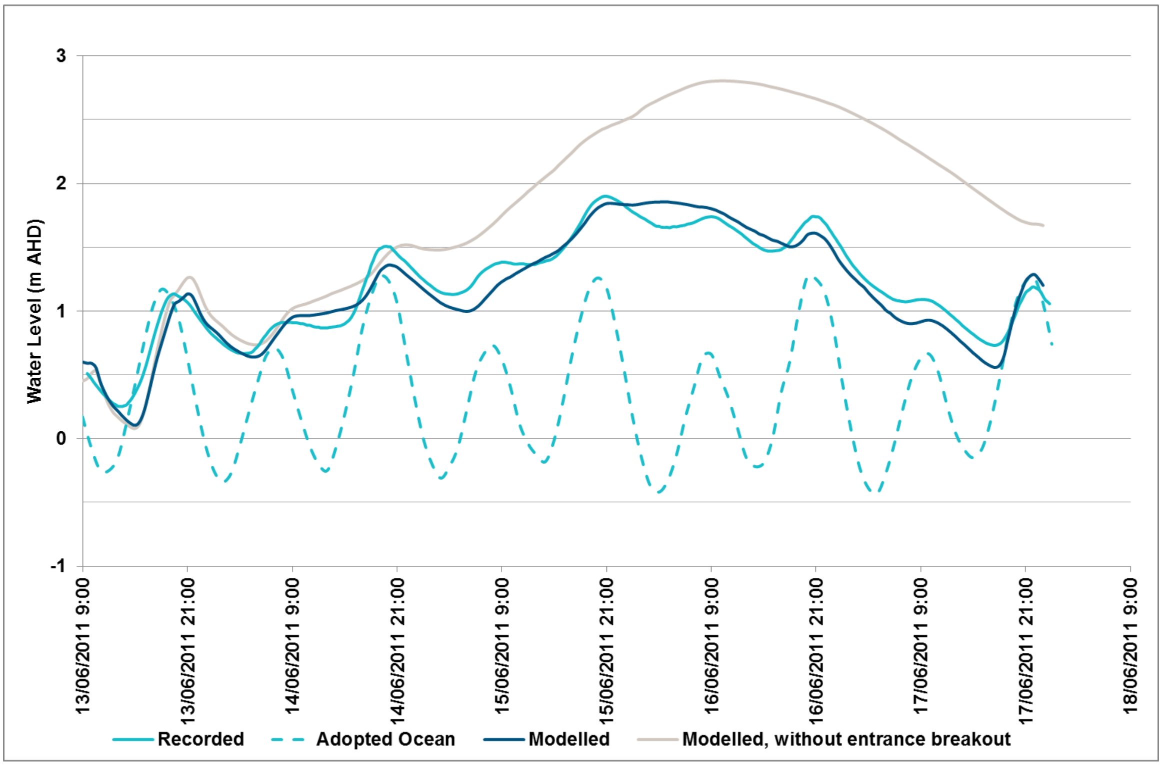


Figure 5-13 Comparison of Recorded and Modelled Water Levels at the Farquhar Inlet Gauge for the June 2011 Event

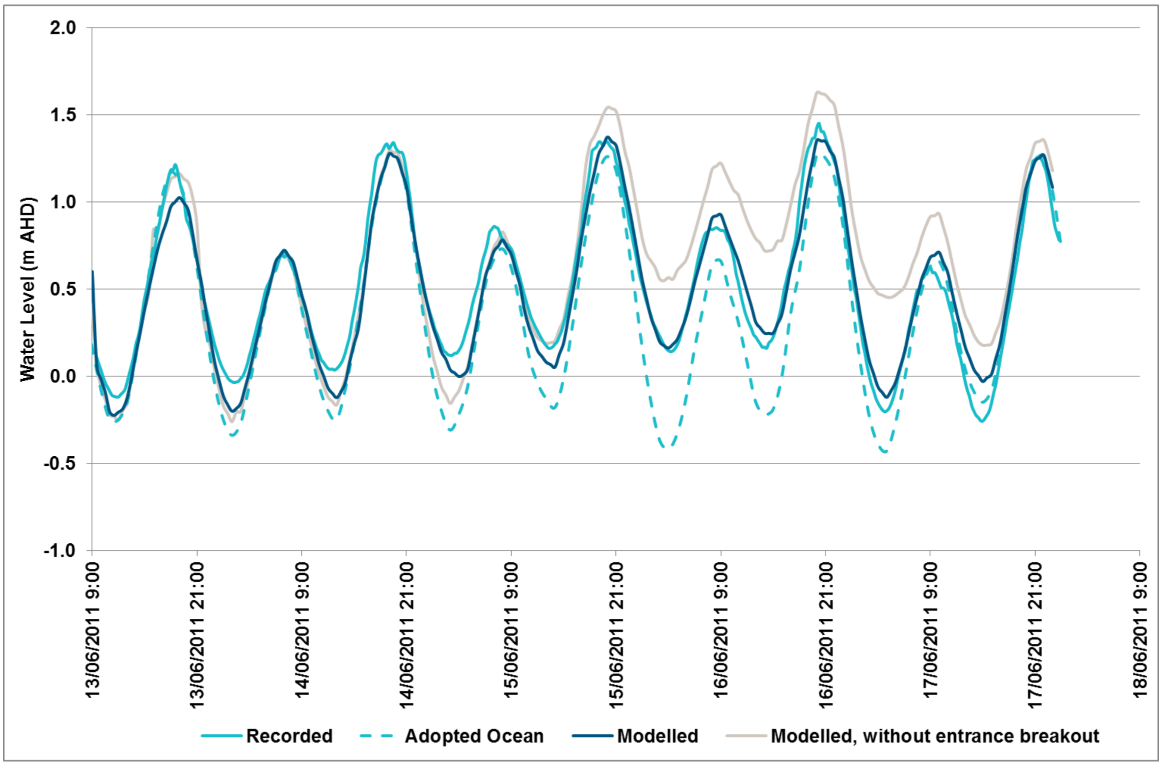


Figure 5-14 Comparison of Recorded and Modelled Water Levels at the Harrington Gauge for the June 2011 Event

Comparing modelled water levels for the scenarios that do and do not include the entrance breakout demonstrates the influence the breakout has on peak flood levels in the lower catchment area. The tidal signature observed during the peak flow at the downstream gauge locations of Farquhar Inlet, Croki and Dumaresq Island is completely removed and modelled peak water levels are elevated above recorded peak levels by up to 0.9m when initial entrance conditions are maintained throughout the flood event. Discrepancies noted in the peak of the tidal signature in the lead-up to the main flood wave could be attributed to the adopted timing of the breakout and that the increasing rate of sediment transport with higher flow has been simply represented as a linear change between initial and final breakout geometry.

Given the complexity of the entrance breakout regime and the simplistic way in which it was represented within the TUFLOW Classic model, the results provide a very good match between observed and simulated conditions.

5.3 June 2011 Entrance Dynamics Calibration

The TUFLOW-FV model developed for this study simulates the sediment transport resulting from the hydrodynamic behaviour (i.e. flood flows) from the 2011 flood event. Entrance geomorphology is actually much more complex and other processes, such as wind and wave action, will also influence the movement of sediment. Although wind and wave action has been considered in terms of generating a higher downstream ocean boundary (detailed in Section 5.2.6), its dynamic action is not included in the modelling of the entrance breakout.

At Farquhar Inlet the entrance breakout is similar to that experienced within the Intermittently Closed and Open Lakes and Lagoons (ICOLL) systems that are a feature of the NSW coast. The entrance breakout is largely unaffected by the wind and wave actions of the sea as the flood flows are typically concentrated through a single location along the dune, resulting in high velocities that serve to "push" sediment away from the entrance.

The location of the breakout channel will migrate along the dune, depending on the relative location of the low point in the dune crest at the onset of a flood event. Within the TUFLOW-FV model the location of the breakout will occur where the channel is positioned in the initial model topography. However, from a flooding perspective the location of the entrance breakout has a minimal impact on the upstream flood conditions. The resultant upstream flood conditions are principally driven by the cross-sectional area of channel that is eroded through the dune during the entrance scour of the flood event. The objective of the TUFLOW-FV model calibration is therefore to drive an appropriate relationship between the incoming flood flow rates and the resultant size of channel that is eroded through the dune.

At Harrington significant flood flows will result in extensive overtopping of the dune, which runs parallel to the entrance channel. This results in a distributed scouring of the dune, with lower velocities than the concentrated scouring typical of Farquhar Inlet. This would result in a general slumping of the dune into the sea. The combined energy of the flood flows and wind/wave action of a stormy sea would then serve to "break up" the entrance sediment, effectively resulting in a large open entrance condition. This is evident in Figure 5-14, which showed very little impact of the June 2011 flood on the recorded tidal signature at Harrington during the peak of the flood wave, particularly on the ebb tides. Following the flood event the coastal sediment transport processes would see the relatively quick re-forming of the dune, as the entrance establishes a more typical configuration.

The extent of entrance scour during flood events can be seen in Figure 5-15, which presents aerial photography of the Manning River entrances from 2009. The areas of vegetated dune are stable and will have built to a height of around 4m or more, becoming vegetated through time as their stable nature has enabled the establishment of mature trees and bushes. The areas of un-vegetated dune are dynamic in nature and therefore mature vegetation cannot take hold due to the dune being periodically "washed away". At Farquhar Inlet the entrance breakout would typically form a relatively narrow channel such as those evident in Figure 5-15. The extensive area of dynamic dune is due to the changing location of the entrance channel with each flood event. At Harrington the extensive dynamic dune would mostly be destroyed during each significant flood. It is likely that the entrance channel would re-form further south along the dune following a flood event and would then slowly migrate northwards towards the breakwater.

The different nature of Harrington and Farquhar Inlet was considered when determining appropriate Manning's "n" channel roughness values for each of the entrances. The Harrington entrance experiences significant breakout during flood events and becomes almost completely open to the ocean, so the same roughness value adopted for tidal channels was considered appropriate for the Harrington entrance. As Farquhar Inlet remains relatively closed, a higher value was adopted for Farquhar Inlet to account for the "rougher" nature of the channel bed due to the extensive sediment depositions.

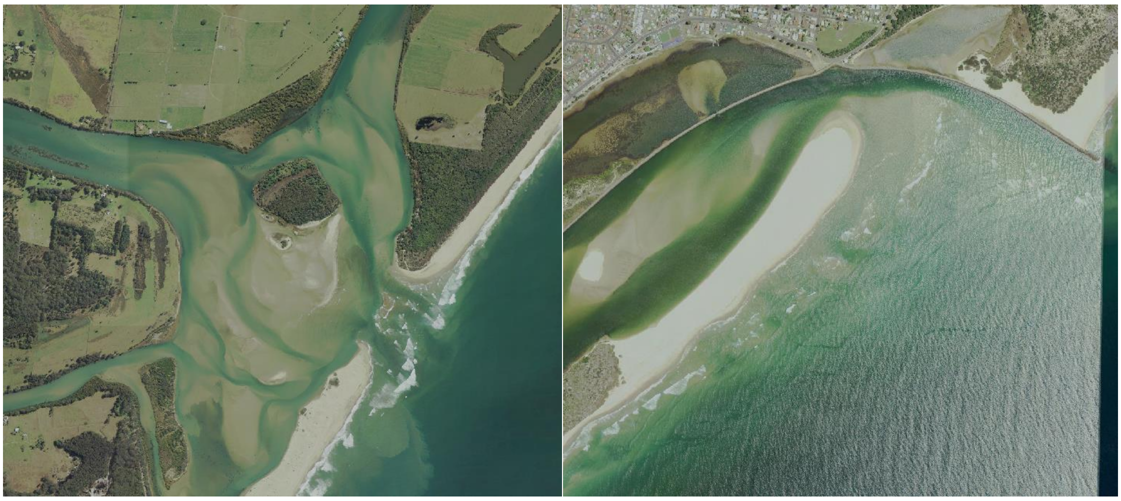


Figure 5-15 2009 Aerial Photography of the Farquhar Inlet (L) and Harrington (R) entrances of the Manning River

The TUFLOW-FV model was used to simulate the flows through the two entrances for the June 2011 flood event. Standard model parameters for roughness, particle size and slump slopes were adopted. The model parameters were then adjusted to achieve a good calibration to the recorded water levels at Harrington and Farquhar Inlet, as presented in Figure 5-13 and Figure 5-14.

The entrance breakout regime simulated in the TUFLOW-FV modelling of the 2011 event was incorporated into the TUFLOW Classic model as a series of “time-varying z-shapes” that vary the 2d model cell elevations over time. The timing, extent and depth of entrance scour modelled in TUFLOW-FV were used in conjunction with the recorded water levels to ensure an appropriate representation was provided in TUFLOW Classic. Comparison of recorded water level time series at the Harrington and Farquhar Inlet gauges against the modelled results is detailed in Section 5.2.7.

This approach to modelling the entrance dynamics within a separate model provides a detailed understanding as to the nature of the entrance dynamics at Harrington and Farquhar Inlet, whilst then enabling the modelled behaviour to be represented within the standard TUFLOW platform, maintaining accessibility to potential future users of the model. The results presented in Figure 5-11 to Figure 5-14 show that the TUFLOW Classic model is providing a sound representation of the entrance breakout of the June 2011 event, resulting in a good model calibration at the two entrance locations and the gauges further upstream. The impact of not including an entrance breakout representation is also presented and demonstrates the importance of this mechanism on the resultant modelled flood behaviour.

5.4 March 1978 Model Calibration

The hydraulic modelling approach for the March 1978 event was to input the flow hydrograph at the Killawarra gauge and assess the TUFLOW hydraulic model performance at Wingham, Taree and downstream to the ocean entrance.

All model parameters are as determined for the 2011 calibration event, as detailed in Section 5.2.5.

5.4.1 Model Inflow

A peak flood level of 26.8m AHD at the Killawarra gauge is cited for the March 1978 event. This peak flood level is estimated from a debris mark, following failure of the gauge recording instrumentation at 23:00 on the 19th March (Public Works, 1991). Due to the uncertainty surrounding the accuracy of this peak level, it was not used to establish a flow rate for the upstream model boundary. Peak levels recorded at Wingham and Taree were therefore assessed.

At the Wingham gauge, the peak water level recorded for 1978 corresponds to the old Wingham gauge location on the northern, elevated side of the Wingham bridge. Given the super-elevation of flood levels around the outside of the bend, it is likely that the peak level recorded at the old gauge location is misrepresentative of typical flood levels in the wider vicinity of the site. An equivalent level at the current gauge location would be more suitable for this assessment, as the peak flood level there is not influenced by this effect. However, it is difficult to convert historical levels to equivalent levels at the new gauge location as there is a significant water level gradient running perpendicular to the direction of flow (i.e. across the channel) due to the tight bend in the channel at this location and the high flood velocities. The gradient varies with flow rate and the relationship is not linear.

At Taree, further difficulty is encountered as the peak flood levels reported in historical documents (5.45m AHD) are referenced to Macquarie Street. With reference to other available surveyed flood levels on the floodplain during the 1978 event, it was found that the level at the bridge is approximately 0.3m higher, so a peak flood level in the order of 5.75m AHD is appropriate for the current gauge location. In addition, the Taree gauge site is not as well suited to rating curve analysis as the site at Killawarra. This is due to the difference between the rising and recession limbs, which is a function of the backwater influence in broader floodplain areas. Nevertheless, from the TUFLOW generated rating curve at Taree, a flow rate of around 11,500m^3/s is required to achieve a peak flood level of 5.75m AHD at Taree. With the local inflow contribution considered (see rainfall data analysis in Section 5.4.2), this equates to an inflow of 10,800m^3/s at Killawarra to achieve the target peak flood level at Taree.

As the Killawarra gauge failed to record the event, the shape and timing of the flow hydrograph at Killawarra that was estimated as part of the previous Flood Study (Public Works, 1991) was adopted for use in this study. The estimated Killawarra hydrograph was also re-defined in the Wingham Flood Study (WorleyParsons, 2011). The 1978 event was simulated using both versions of the hydrograph. The Public Works (1991) hydrograph, when scaled to the required peak flow rate provided the best match in terms of shape and timing of recorded water levels at both the Wingham and Taree gauge sites, so was adopted for use in this study. The modelled flow hydrograph at Killawarra for the March 1978 is shown in Figure 5-16 and forms the upstream inflow boundary condition to the hydraulic model.

5.4.2 Rainfall Data

Similarly to the 2011 calibration event, utilising an inflow hydrograph at Killawarra negates the need for a detailed rainfall analysis across the entire catchment area. The only rainfall inputs required for the hydrological model to be input to the hydraulic model are for the lower sub-catchment areas.

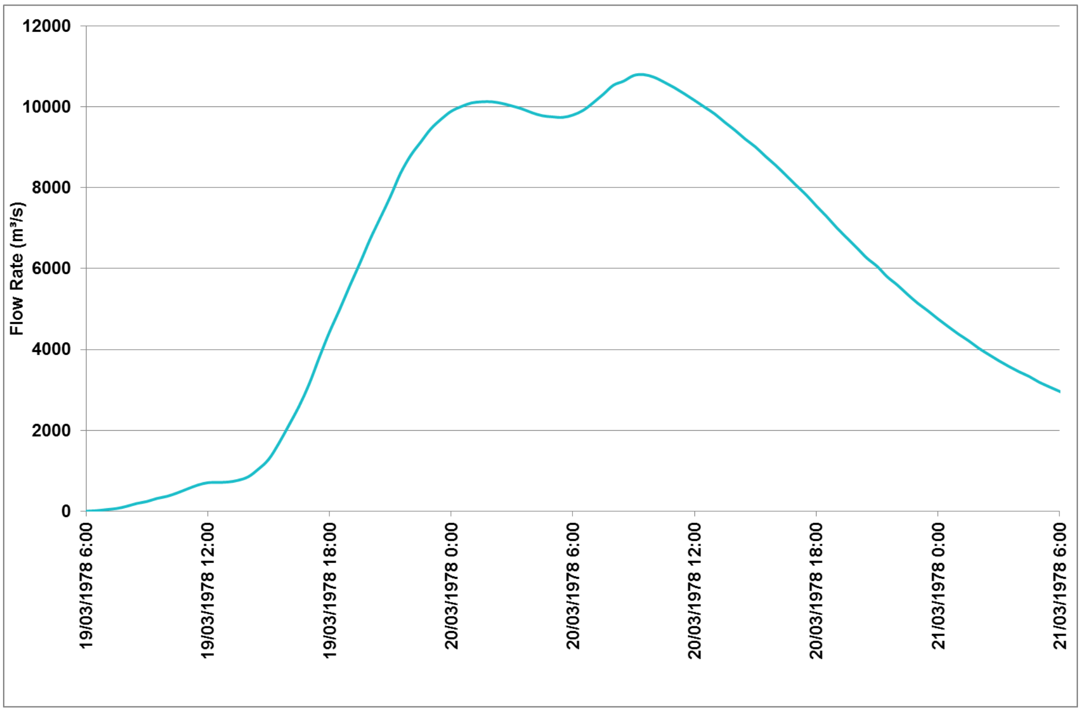


Figure 5-16 Adopted Flow Hydrograph for the March 1978 Event at the Killawarra Gauge

There were a number of continuous rainfall gauges operating within the catchment during the March 1978 event. There are three BoM operated gauges located at Taree (060030), Nowendoc (Green Hills) (060104) and Gloucester (Hiawatha) (060112). The NSW Office of Water also operates a gauge on the Manning River at Tomalla (208003). One hour rainfall hyetogrpahs for the Taree gauge is shown in Figure 5-17. The location of each gauge is presented in Figure 5-18.

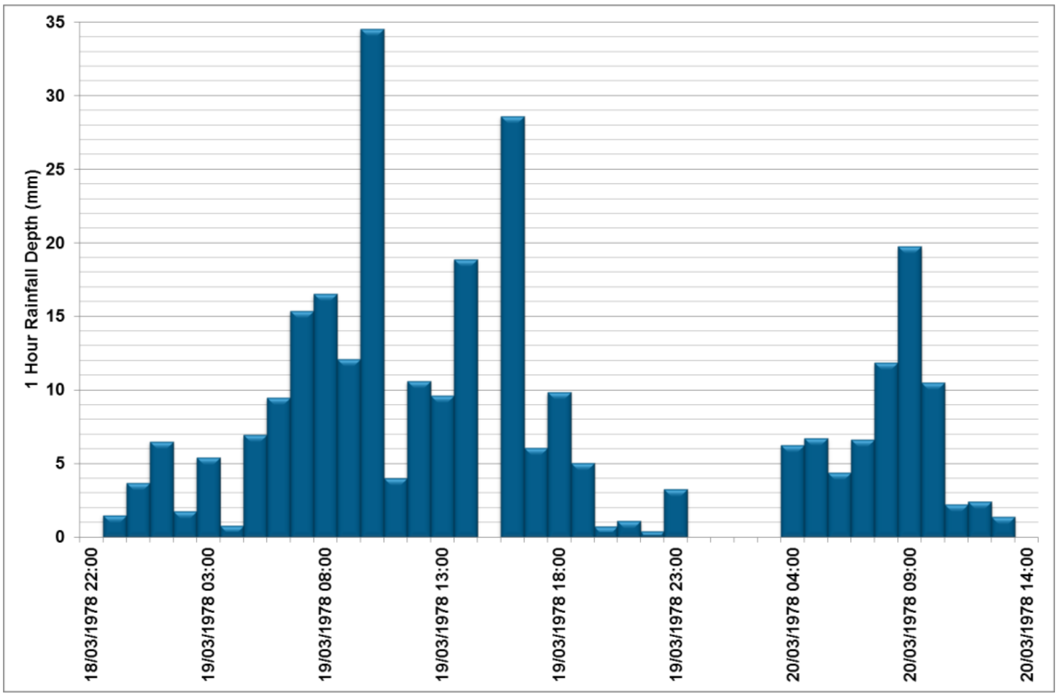
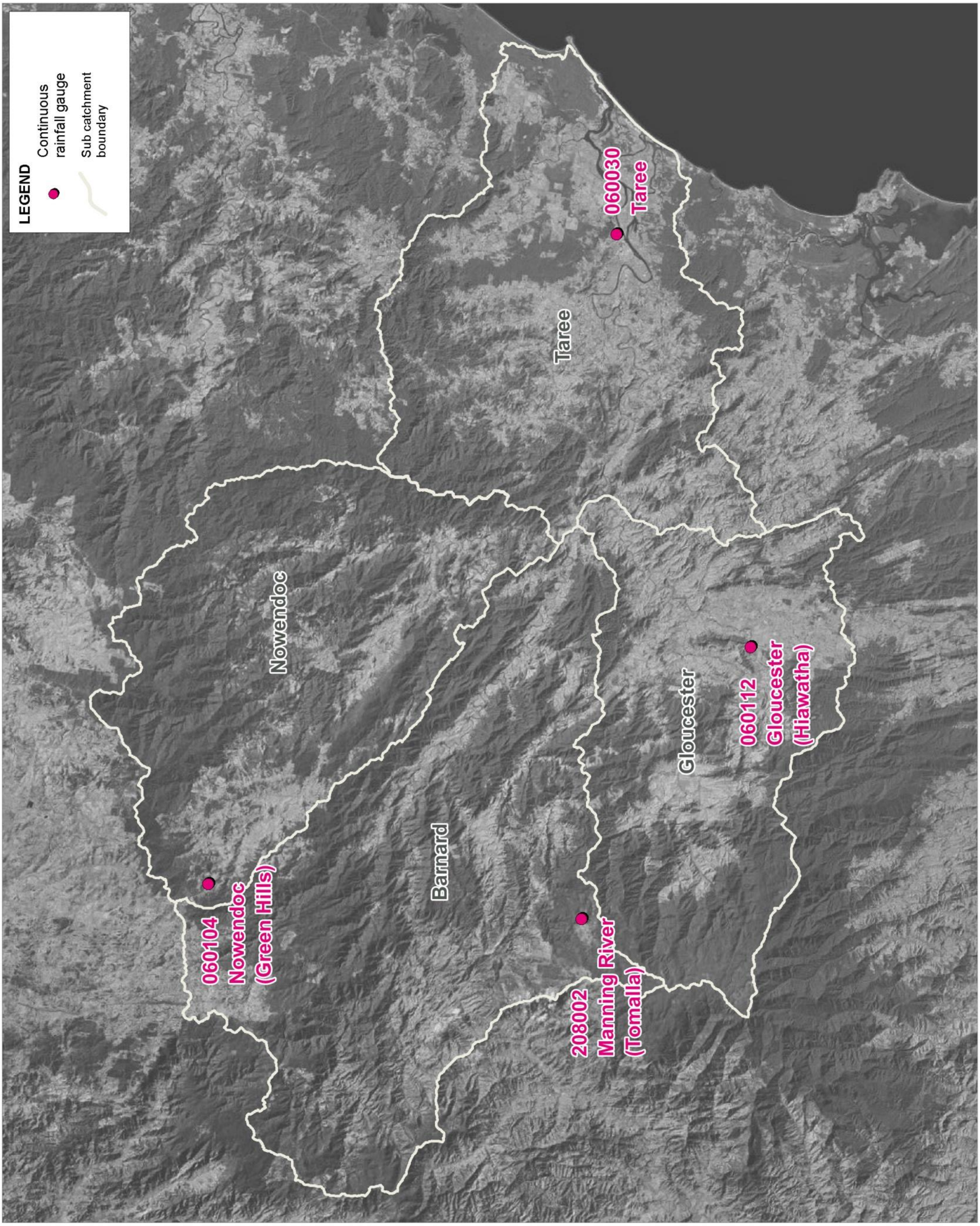


Figure 5-17 Rainfall Hyetograph for the March 1978 Event at the Taree Gauge



Title:
March 1978 Rainfall Distribution

Figure: **5-18**

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The temporal pattern recorded at each gauge was similar. The total depth at each gauge was deemed to be representative of rainfall falling within each major sub catchment area. Therefore, the temporal pattern recorded at the Taree gauge was assumed to be representative of rainfall over the lower catchment area (sub catchment labelled “Taree” on Figure 5-15).

In order to gain an appreciation of the relative magnitude of the rainfall recorded during the March 1978 event, depth vs. duration curves recorded at Taree, Nowendoc and Tomalla are again presented against IFD data. Figure 5-19 and Figure 5-20 show the IFD data applicable both to the whole Manning River catchment (ARF’s applied) and to a point.

The trajectory of each curve closely follows the alignment of the design rainfall curves up to durations of 48 hours. Beyond 48 hours durations, the rainfall depths at each gauge location plateau which indicates that the March 1978 event was a 48 hour event. Each curve presents a similar alignment as the temporal pattern recorded at each gauge was similar for the March 1978 event.

Figure 5-19 shows that the probability of rainfall as intense at that recorded at Tomalla occurring simultaneously over the entire Manning River catchment is well over a 1% AEP event and would be even rarer than a 0.2% AEP (1 in 500 year) design rainfall event.

With reference to Figure 5-20 and the 1987 IFD curves, the Taree gauge recorded rainfall equivalent to around a 10% AEP to 5% AEP design event. Less rainfall was recorded in the Nowendoc catchment, where total rainfall depths of around 130mm are expected to occur more frequently than once every 2 years (i.e. a 50% AEP event). The Tomalla gauge recorded extreme rainfall, equivalent to design magnitudes in excess of 1% AEP. If the IFD was to be extrapolated to events with even less likelihood of occurring, the 48 hour rainfall recorded at Tomalla in March 1978 is likely to be roughly similar to a 0.5% AEP (1 in 200 year) design flood event at Taree.

5.4.3 Downstream Boundary Condition

The tidal boundary adopted for simulation of the March 1978 event in the previous Flood Study (Public Works, 1991) was also used here for the model verification. The water level time series was calculated from an astronomical tide with additional components of wave set-up and storm surge included. The ocean water level is reproduced in Figure 5-21 and forms the downstream boundary condition of the hydraulic model.

In light of the uncertainties surrounding entrance conditions at the onset of the event, the 1978 ocean water level has been scaled to match the recorded peak flood levels at the downstream model limit, as peak levels here are generated by the backwater influence of elevated ocean levels.

5.4.4 Model Topography

The Taree Bypass was constructed from 1993 to 2000. For the March 1978 model run, the bypass embankment was removed from the DEM. The modelled flow constriction of any associated bridge structures was also removed.

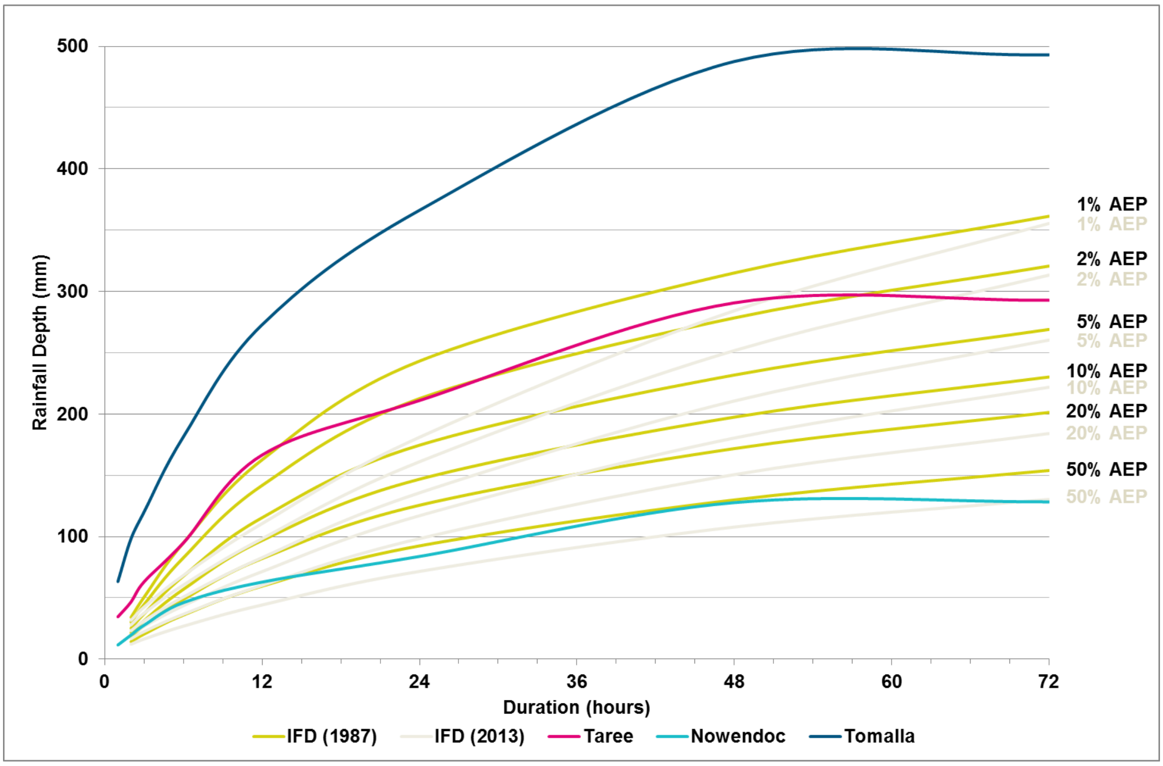


Figure 5-19 Comparison of Recorded March 1978 Rainfall with IFD Relationships (Entire Catchment Intensities)

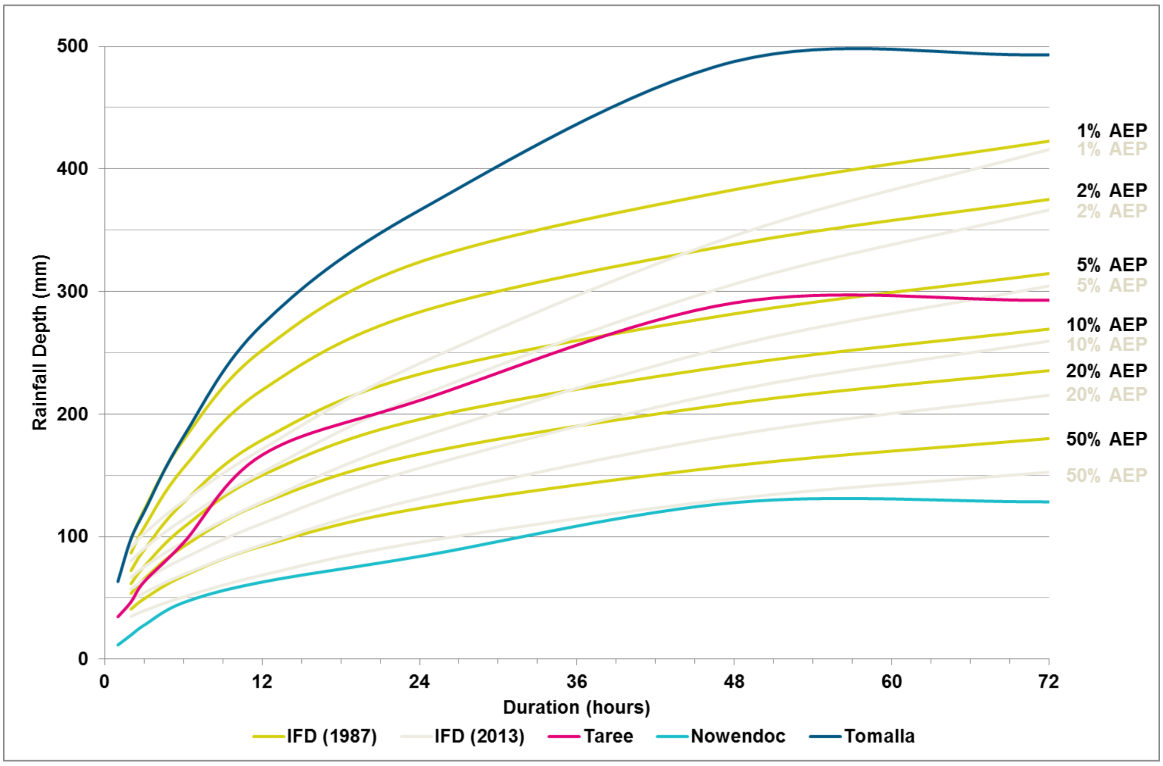


Figure 5-20 Comparison of Recorded March 1978 Rainfall with IFD Relationships (Point Intensities)

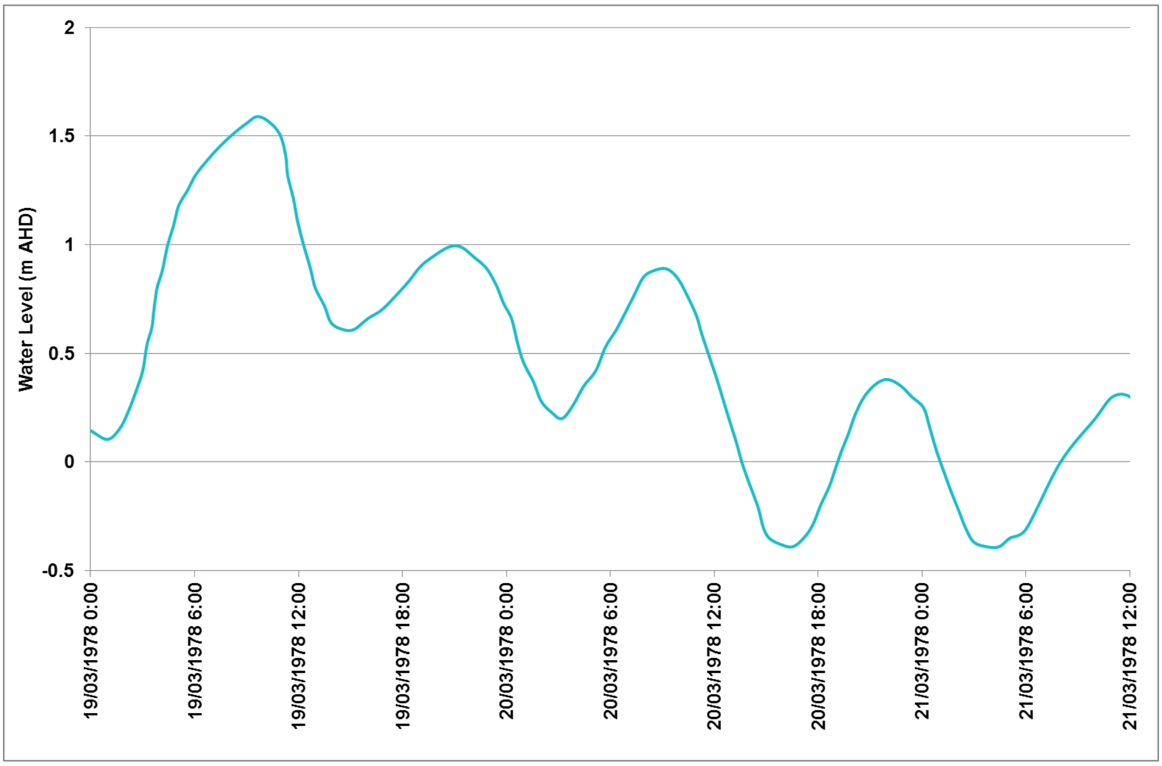


Figure 5-21 Adopted Ocean Water Level for the March 1978 Event

5.4.5 Observed and Simulated Flood Behaviour

The MHL gauges were not operational during the 1978 event. Given the uncertainties around the accuracy of the peak flood level recorded for this event at Killawarra, the gauges at Wingham and Taree form the basis for the model calibration. Comparison of the recorded and modelled peak flood level time series at the three gauges are shown on Figure 5-22, Figure 5-23 and Figure 5-24. Note that for the 1978 event, the water level time series at Taree was recorded at the Macquarie Street location. It has been converted to an equivalent height at the present day gauge location (Martin Bridge) for presentation purposes.

Due to the issues associated with the location of the historical Wingham gauge, achieving a good fit at the Taree gauge was the main focus of the model verification to this event.

The modelled peak flood level profile for the Manning River is presented in Figure 5-25 against the available recorded flood marks for this event. The modelled bed elevation is also included for reference. Figure 5-25 is reproduced at A3 size in Appendix B.

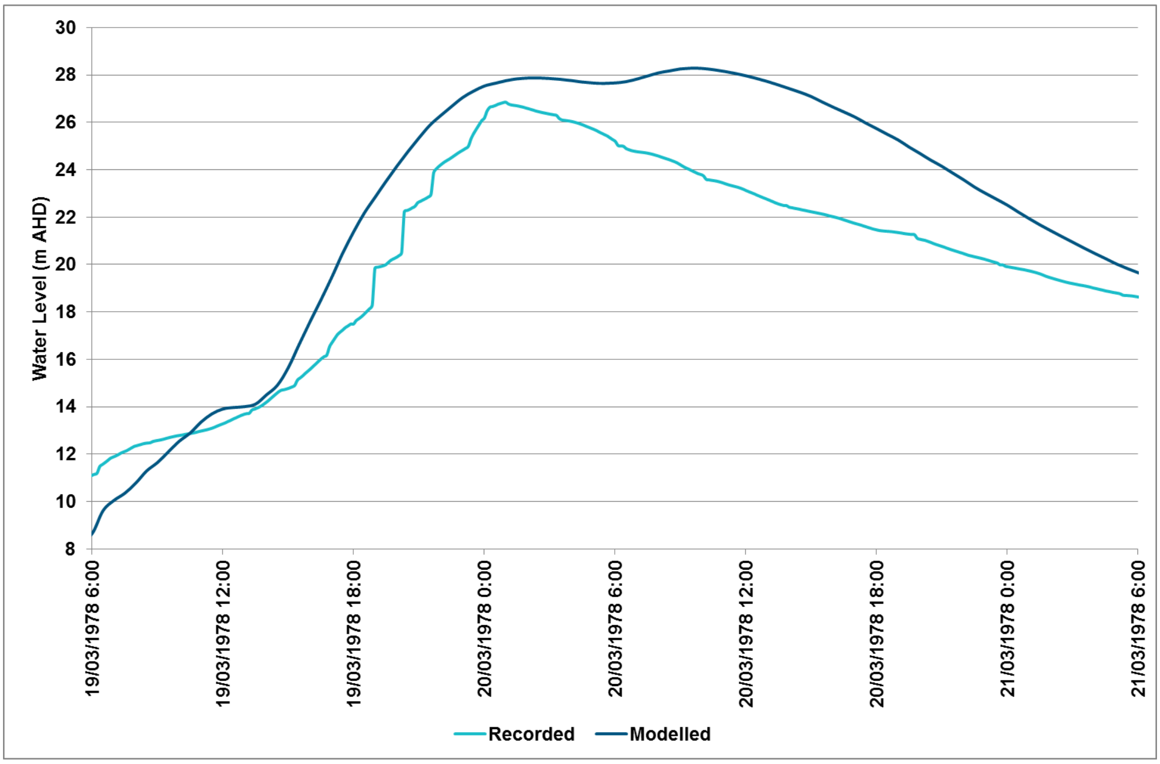


Figure 5-22 Comparison of Recorded and Modelled Water Levels at the Killawarra Gauge for the March 1978

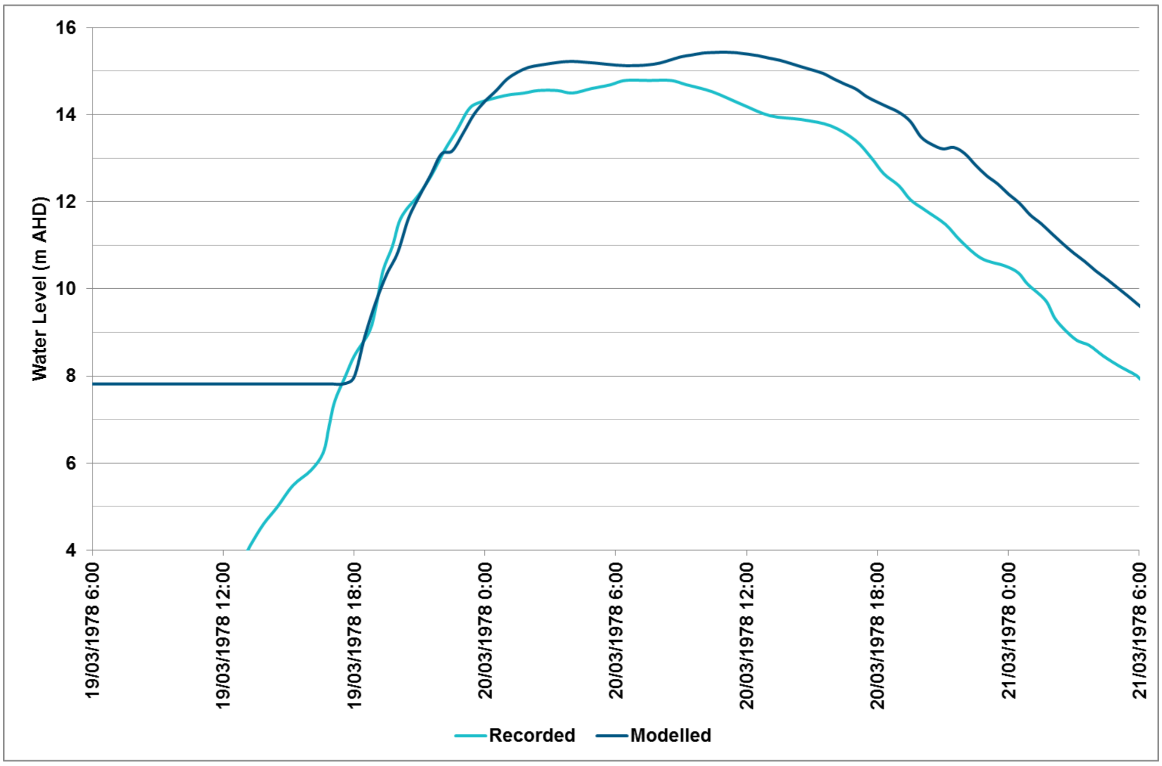


Figure 5-23 Comparison of Recorded and Modelled Water Levels at the Wingham Gauge for the March 1978 Event

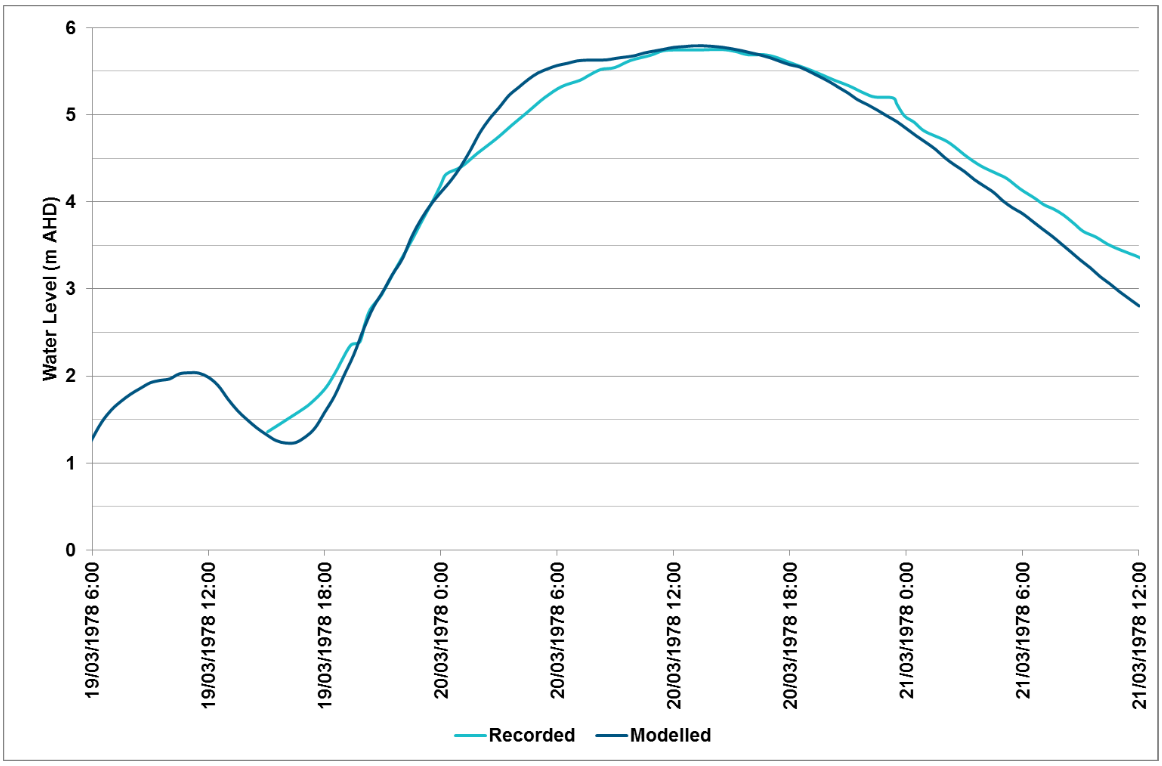


Figure 5-24 Comparison of Recorded and Modelled Water Levels at Taree (Martin Bridge) for the March 1978 Event

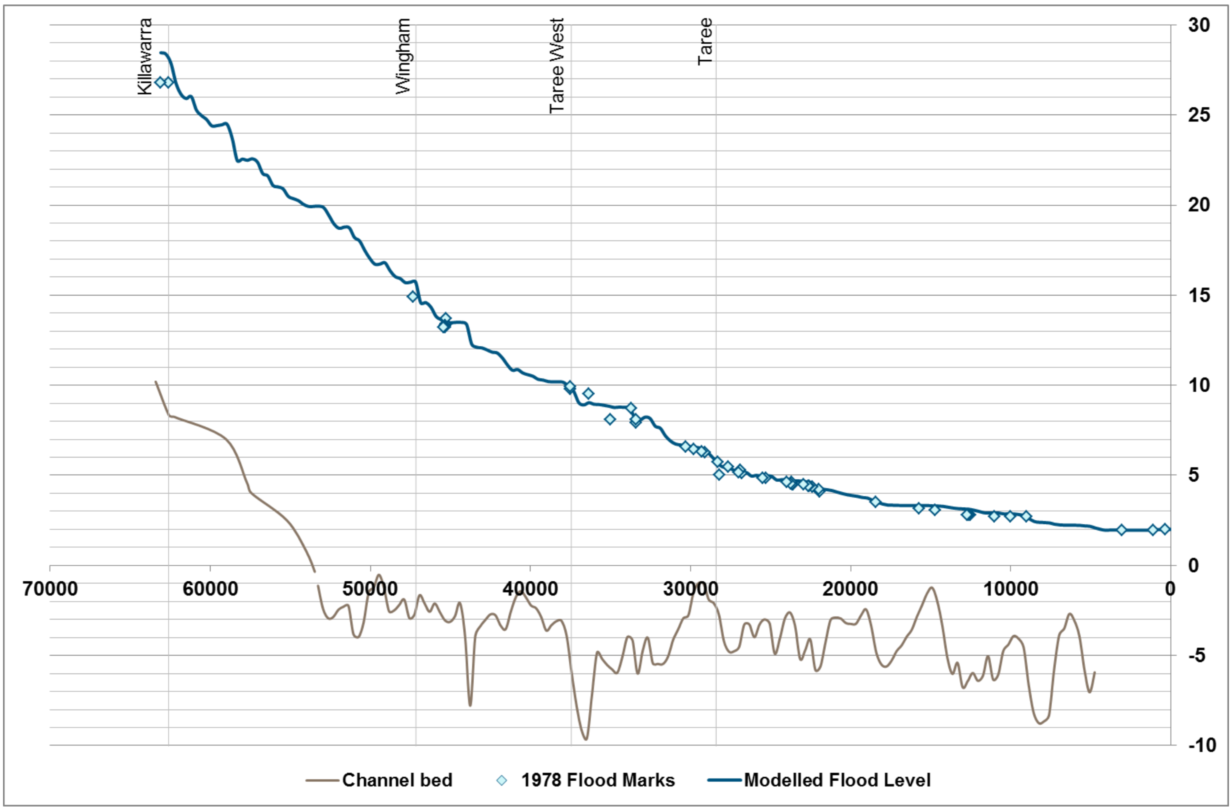


Figure 5-25 Long Section along the Manning River for the March 1978 Event

5.5 March 2013 Model Verification

The March 2013 event was modelled to verify entrance dynamics under lower flows. This event was suitable due to the availability of water level records at Harrington and Farquhar Inlet. Calibration of the hydraulic model to the June 2011 event indicated that the entrance at Harrington would breakout to an open condition for flows similar in magnitude to a 5% AEP event at Taree. Uncertainty surrounding the Harrington breakout for more frequent events can be verified through simulation of the March 2013 event (between a 20% AEP and 5% AEP).

All model parameters, model topography and initial entrance geometry are as determined for the 2011 calibration event, as detailed in Section 5.2.

5.5.1 Model Inflow

The recorded water level hydrograph at the Killawarra gauge location was converted into a flow hydrograph using the TUFLOW generated rating curve. The inflow hydrograph is presented in Figure 5-26 and forms the upstream model boundary condition. A peak flow rate of just over 4,200m^3/s is estimated at Killawarra for the March 2013 event.

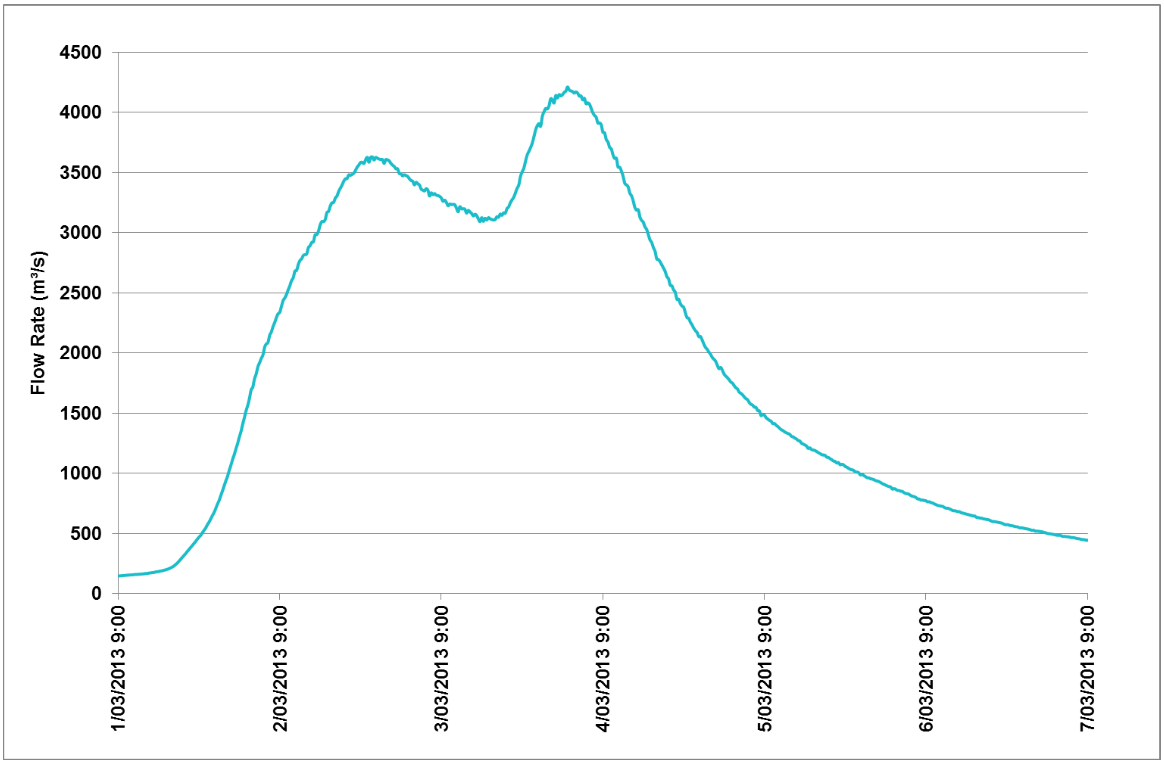


Figure 5-26 Adopted Flow Hydrograph for the March 2013 Event at the Killawarra Gauge

5.5.2 Entrance Geomorphology

As the March 2013 event is between a 20% AEP and 5% AEP based on peak flood levels at Taree, representation of entrance dynamics for the 20% AEP design event were adopted for the model simulation. Development of design entrance geomorphology is detailed in Section 6.6.

A model scenario where entrance breakout was not included at Harrington was also simulated to support the assumed breakout conditions for lower magnitude events.

5.5.3 Rainfall Data

Similarly to the June 2011 event, daily rainfall records indicated that total depth of rainfall varied significantly across the lower Manning River catchment. From 9am on the 1st March to 9am on the 4th of March, a total of 222mm was recorded at the Taree gauge, with more rainfall observed over the Dingo Creek and Lansdowne catchment areas (around 350mm and 388mm, respectively, for the 72 hour period).

The same hydrological modelling approach used for the June 2011 calibration event was adopted for the March 2013 event, whereby Taree, Dingo Creek and Lansdowne sub catchments were assigned different total rainfall depths. The temporal variation of rainfall recorded at the Taree Airport AWS (060141) pluviograph was deemed to be representative of rainfall over the entire lower catchment area and was applied to all sub catchments. The one hourly rainfall hyetograph recorded at the gauge is presented in Figure 5-27.

Although rainfall over the lower Manning River catchment was of a similar magnitude to the June 2011 event, the upper catchment received significantly less rainfall. This resulted in a peak flow rate at the Killawarra gauge location that was around 60% of that recorded during the 2011 event.

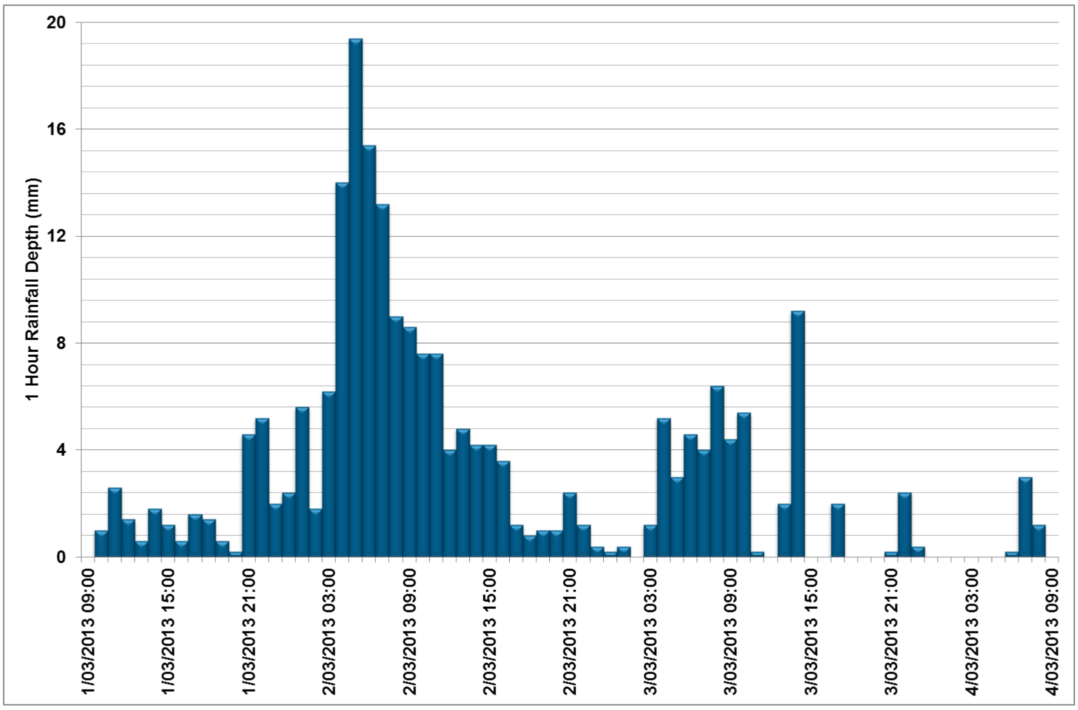


Figure 5-27 Rainfall Hyetograph for the March 1978 Event at the Taree Gauge

5.5.4 Downstream Boundary Condition

An ocean tidal signature was derived from tidal data detailing the timing of maximum/minimum levels (NSW Transport Maritime, 2012). As per the 2011 calibration, a wave setup component of 0.2m was added.

5.5.5 Observed and Simulated Flood Behaviour

Adopting the entrance breakout representation developed for design flood modelling provided a good match to flood levels at the Harrington and Farquhar Inlet gauges. Figure 5-28 and Figure 5-29 present the modelled water level against the recorded data for the March 2013 at the Farquhar Inlet and Harrington gauge locations, respectively. It can be seen that at both gauge locations, modelled flood levels are within 0.1m of recorded peak flood levels.

Discrepancy between the troughs of the tidal signature in the first 72 hours of the model simulation could be attributed to adopted initial entrance conditions or rate of socure during the event. As the priority is to achieve comparable peak flood levels, the entrance breakout representation adopted is sufficent.

If it is assumed that the Harrington entrance will not completely break out, peaks and troughs of the tidal signature are consistently over-estimated by up to 0.3m as the flood wave passes through the entrance (see Figure 5-29). The results of the March 2013 model simulation support the assumption that the Harrington entrance will breakout to an open entrance condition under design flows equivalent to a 20% AEP or greater.

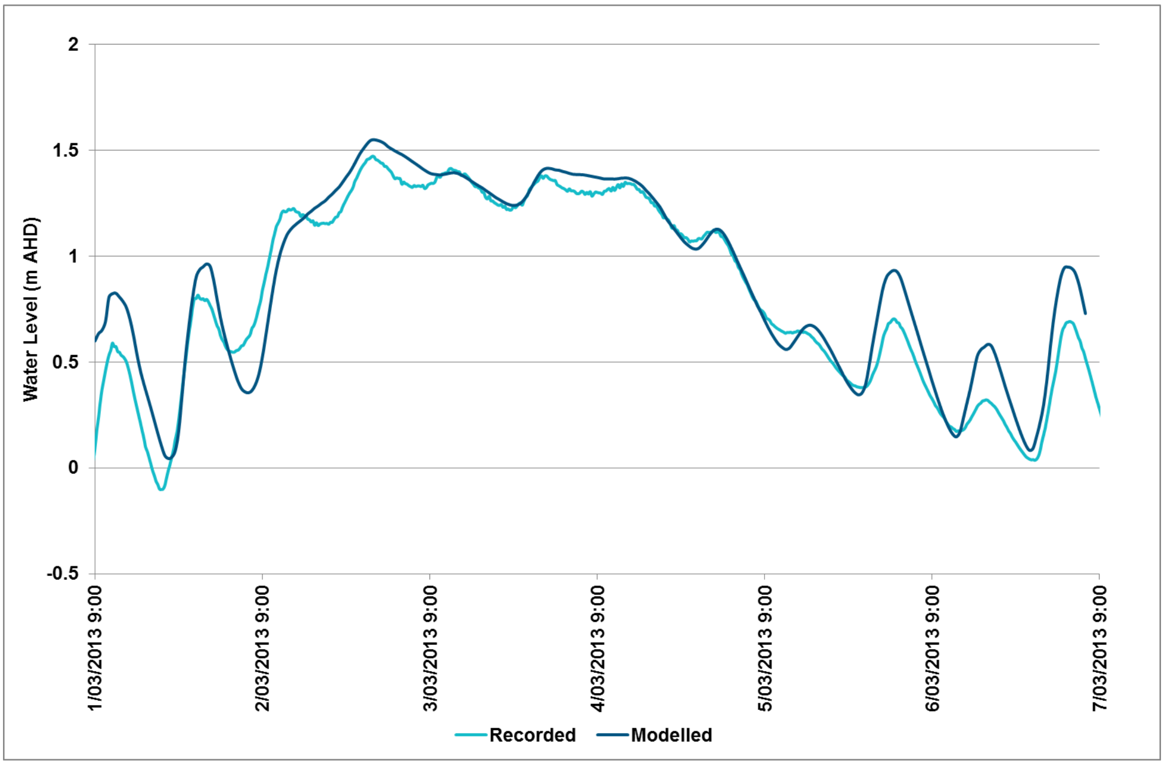


Figure 5-28 Comparison of Recorded and Modelled Water Levels at the Farquhar Inlet Gauge for the March 2013 Event

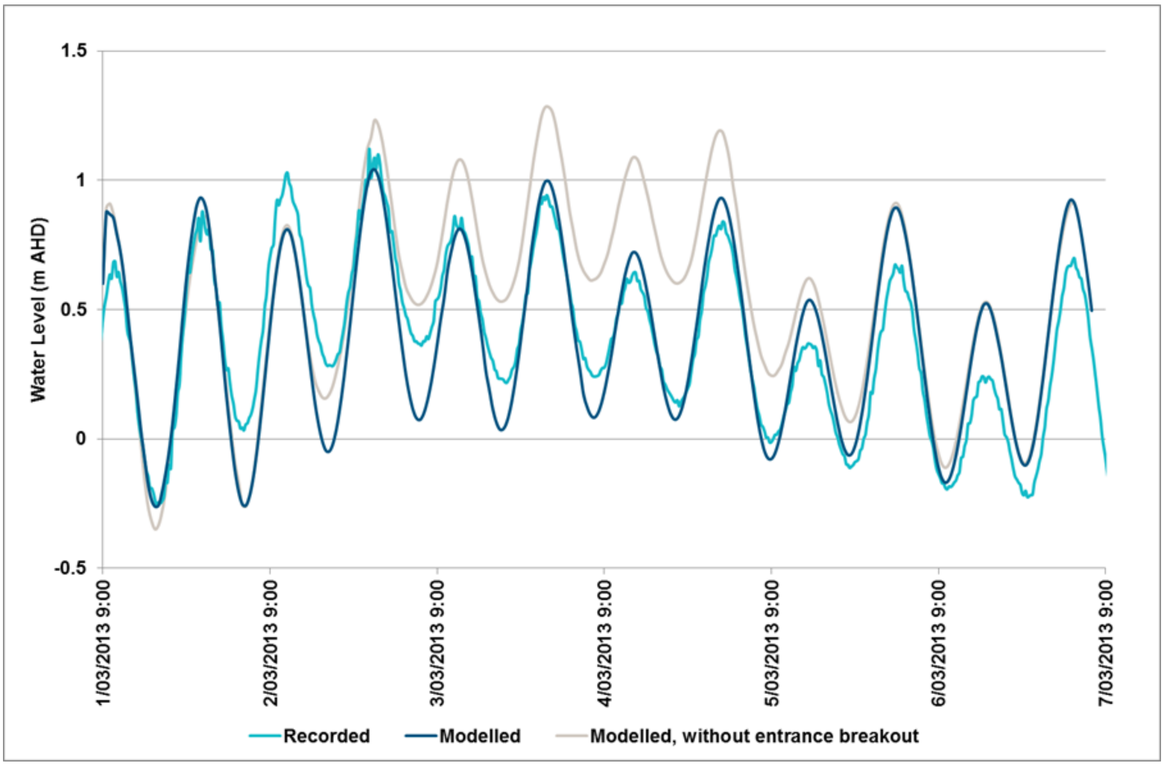


Figure 5-29 Comparison of Recorded and Modelled Water Levels at the Harrington Gauge for the March 2013 Event

5.6 February 1990 Model Verification

The February 1990 event was simulated to assess the performance of the hydraulic model for an additional historic event. The Harrington and Farquhar Inlet gauges recorded water level time series during this event. The Taree gauge was not in operation but a peak flood level of 4.37m AHD was recorded at the Martin Bridge.

All model parameters, model topography and initial entrance geometry are as determined for the 2011 calibration event, as detailed in Section 5.2.

5.6.1 Model Inflow

The recorded water level hydrograph at the Killawarra gauge location was converted into a flow hydrograph using the TUFLOW generated rating curve. The inflow hydrograph is presented in Figure 5-30 and forms the upstream model boundary condition. A peak flow rate of just over 7,200m^3/s is estimated at Killawarra for the February 1990 event.

5.6.2 Entrance Geomorphology

The February 1990 is similar in magnitude to the March 2011 event, in terms of both flow rate at Killawarra and peak flood level recorded at Taree. The final entrance breakout geometry adopted for the March 2011 simulation was therefore used for the February 1990 event, with the rate of scour altered to align with the timing of the flood wave through the lower catchment.

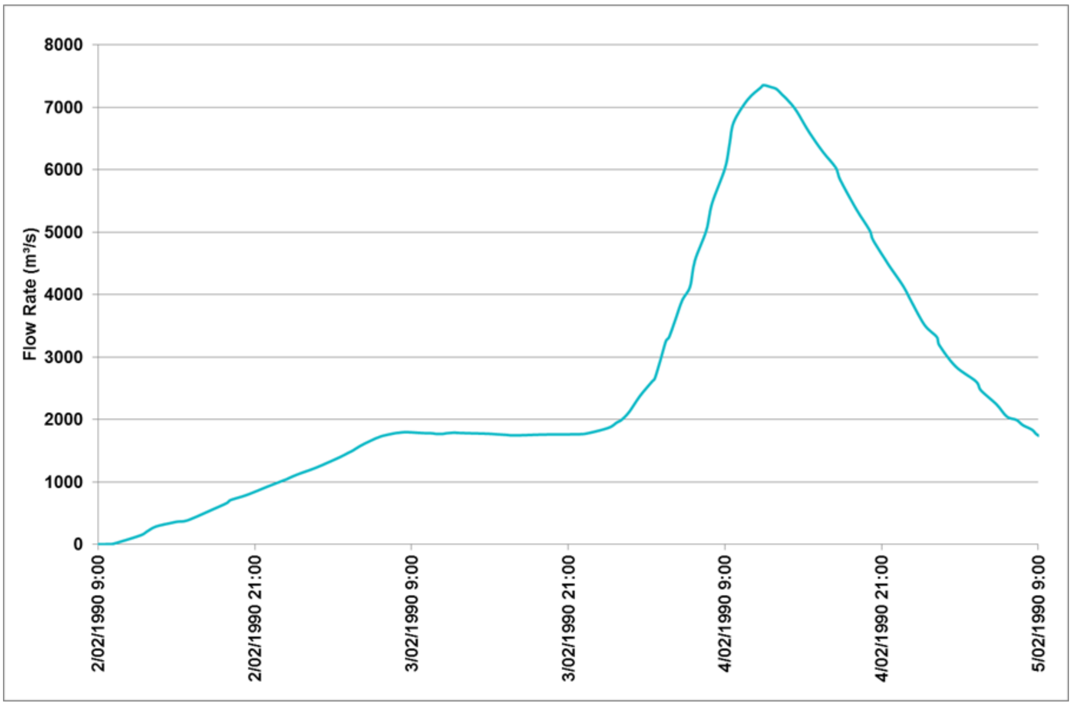


Figure 5-30 Adopted Flow Hydrograph for the February 1990 Event at the Killawarra Gauge

5.6.3 Rainfall Data

The temporal variation of rainfall recorded at the Taree (Patanga Cl) (060030) pluviograph was deemed to be representative of rainfall over the entire lower catchment area and was applied to all sub catchments downstream of Killawarra within the hydrological model. The one hourly rainfall hyetograph recorded at the gauge is presented in Figure 5-31.

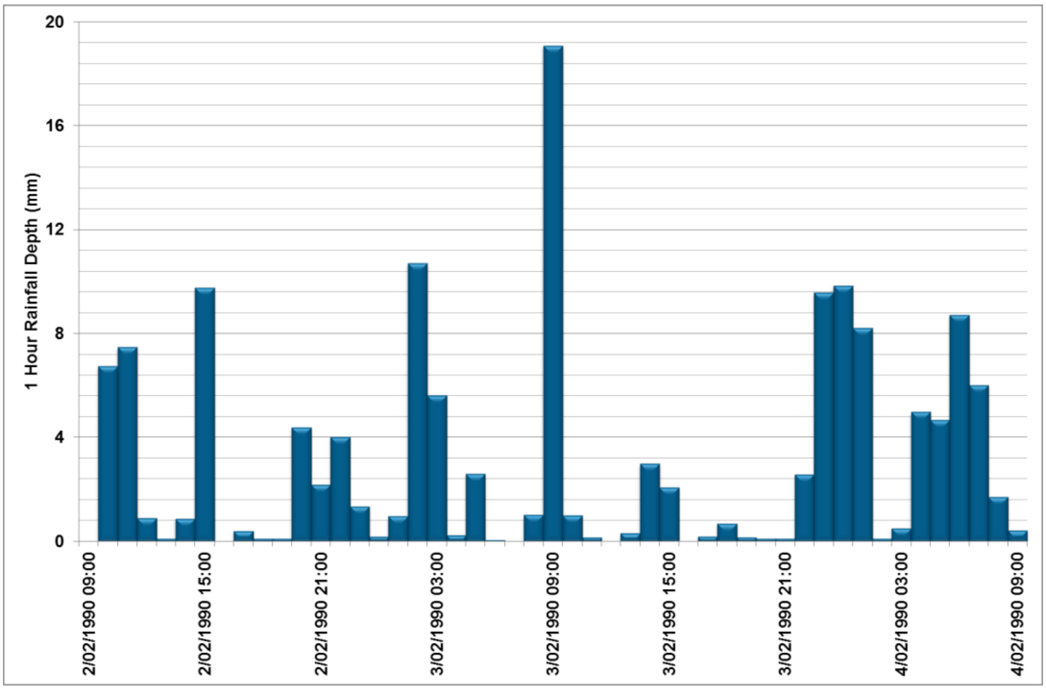


Figure 5-31 Rainfall Hyetograph for the February 1990 Event at the Taree Gauge

5.6.4 Downstream Boundary Condition

Ocean tide data measured at Port Stephens was applied as the downstream model water level boundary. As per the 2011 simulation, although measured at a different location, the timing and maximum/minimum level of the tidal signature will be close enough to that at the Harrington and Old Bar entrances that data is suitable for use.

The Port Stephens tide data and the adopted downstream water level boundary for simulation of the February 1990 event is shown in Figure 5-32.

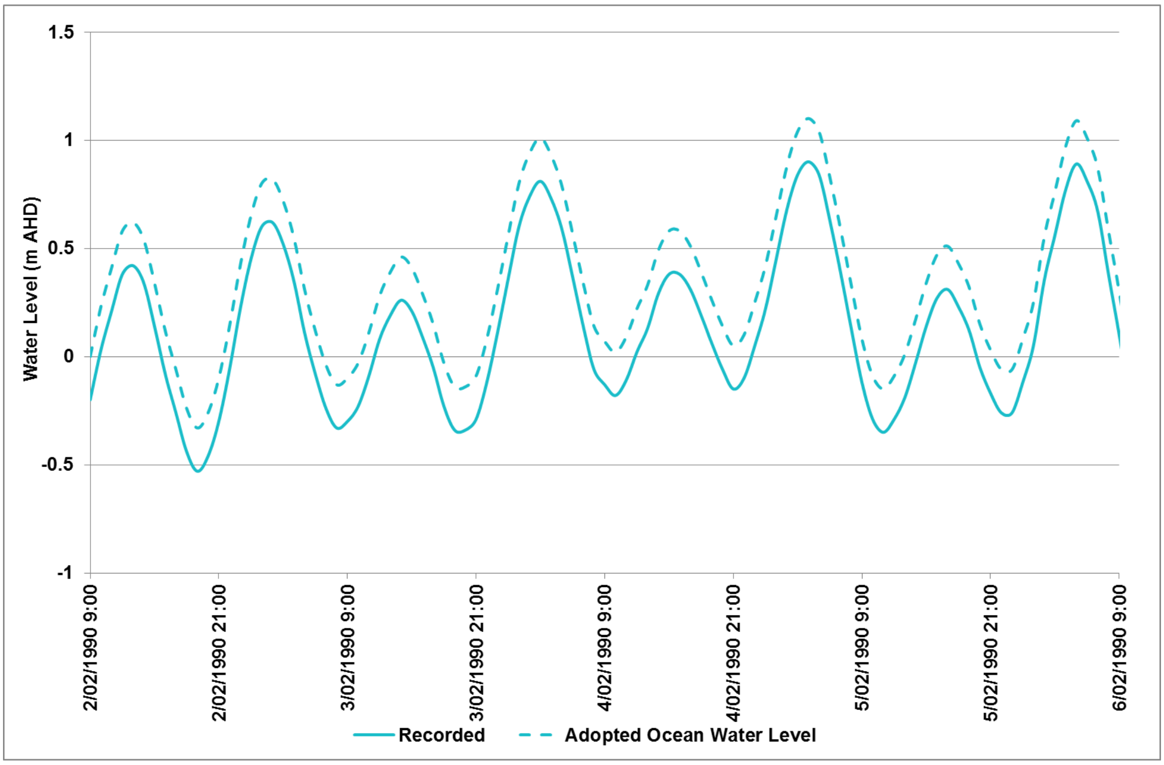


Figure 5-32 Tidal Data for the February 1990 Event Measured at Port Stephens and the Adopted Ocean Boundary Condition (to include Wind and Wave Setup)

5.6.5 Observed and Simulated Flood Behaviour

The model simulation provided a good match against recorded flood levels for the February 1990 event. At the Martin Bridge, Taree, the modelled peak flood level was approximately 0.15m lower than the observed level of 4.37m AHD.

Comparison of the recorded and modelled peak flood level time series at the Farquhar Inlet and Harrington gauges are shown in Figure 5-33 and Figure 5-34, respectively. Peak flood levels are within 0.1m. Again, given the complexity of the dynamic nature of the entrances, the model simulation provides a very good representation of observed flood conditions during the event.

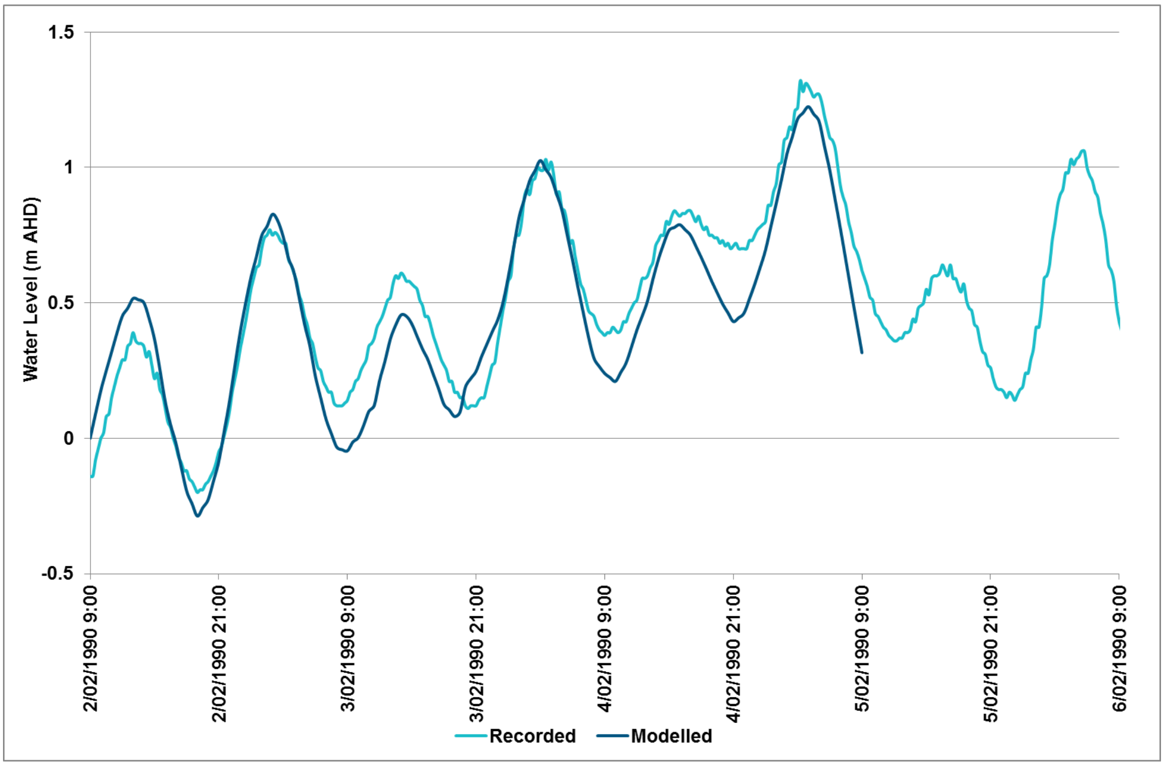


Figure 5-33 Comparison of Recorded and Modelled Water Levels at the Farquhar Inlet Gauge for the February 1990 Event

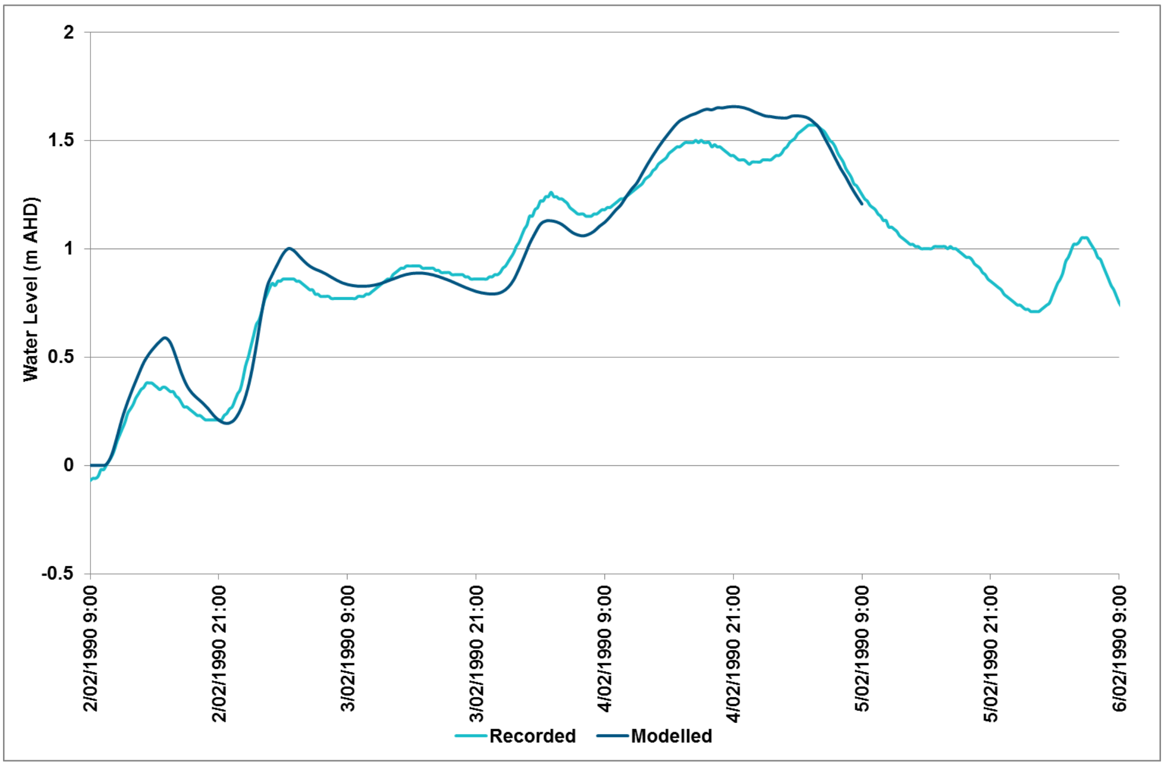


Figure 5-34 Comparison of Recorded and Modelled Water Levels at the Harrington Gauge for the February 1990 Event

5.7 February 1929 Model Verification

5.7.1 Model Inflow

To determine appropriate model inflows for the February 1929 event, the same approach used for the 1978 event was adopted.

The peak flood levels recorded at Killawarra are very similar between the 1978 and 1929 events. A peak flood level of 5.6m AHD was recorded on the floodplain at Taree, in the location of the present day Aquatic Club on Macquarie Street. Based on expected flood gradients along the floodplain in this location, this roughly correlates to a peak flood level of 5.9m AHD on the floodplain where the Martin Bridge now crosses the Manning River. Therefore, the 1929 event adopted the same inflow hydrograph at Killawarra that was used for the March 1978 event – with a peak flow rate of 10,800m^3/s. A peak flow rate of just under 11,000m^3/s is required to achieve the peak flood level observed at Taree. Based on the rainfall analysis in Section 5.7.2, the higher rainfall recorded across the lower Manning River floodplain will account for the higher flows required to generate this level at Taree.

5.7.2 Rainfall Data

The 1929 event lasted for three days from the 7th - 9th February. There are no continuous rainfall gauges available from which to establish a temporal rainfall pattern for the event. However, daily rainfall totals recorded at various locations across the catchment are available.

Recorded daily totals have been processed and interpolated into a gridded dataset (SILO) by BoM. This data set was analysed over the three day period, with an average total rainfall depth determined for each major catchment area. The results of the analysis are shown in Table 5-2. The catchments referred to in the table are the same as those presented in Figure 5-18 unless otherwise indicated.

Table 5-2 Total Rainfall Depths for Various Sub Catchment Areas for the February 1929 and March 1978 Events

Catchment	3-day Total Rainfall Depth (mm)	
	Feb 1929	Mar 1978
Nowendoc	356	293
Barnard	275	235
Gloucester	368	347
Taree	441	328
Killawarra*	326	284
Manning**	355	295

**Combined catchment area upstream of the Killawarra gauge*

***Entire Manning River catchment area*

Three-day total rainfall depths for the March 1978 event (19th - 21st) are also presented for reference. It should be noted that although daily totals were summed to calculate the values presented in the table, most of the rainfall in the March 1978 event fell within a 24 hour period. In the absence of pluviograph data, the storm duration is unknown but is likely to be somewhere between a 24 hour and 72 hour storm. For this reason, the relative magnitude of rainfall for the 1929 event cannot be directly compared to the March 1978 event or the IFD design curves.

As the February 1929 model simulation is largely based on the March 1978 input parameters, the 1978 local inflows downstream of Killawarra were increased by approximately 35%, based on the difference in the average total rainfall over the “Taree” catchment between the two events.

5.7.3 Downstream Boundary Condition

The downstream boundary adopted for the 1929 model scenario is the same as the 1978 event, and has again been scaled to replicate observed peak flood levels at the downstream model limit.

5.7.4 Model Topography

In addition to removal of the Taree Bypass, the Martin Bridge at Taree (constructed 1940) was removed for simulation of the 1929 event. The embankment of the southern approach to the bridge was also removed from the DEM.

There is much uncertainty regarding the state of the floodplain during the 1929 event. There could have been local topographic controls that are not represented in the current catchment topography. There is also potential that the large floods of 1929 and 1978 resulted in alterations to the catchment topography into its present day state. The performance of the hydraulic model for the 1929 verification event is assessed with these factors in mind.

5.7.5 Observed and Simulated Flood Behaviour

There are no recorded water level time series available for the February 1929 event so verification of the model performance has been assessed using recorded spot peak flood levels located across the Manning River floodplain. The modelled peak flood level profile for the Manning River is presented in Figure 5-35, against the available recorded flood marks for this event. The modelled bed elevation is also included for reference. Figure 5-35 is reproduced at A3 size in Appendix B.

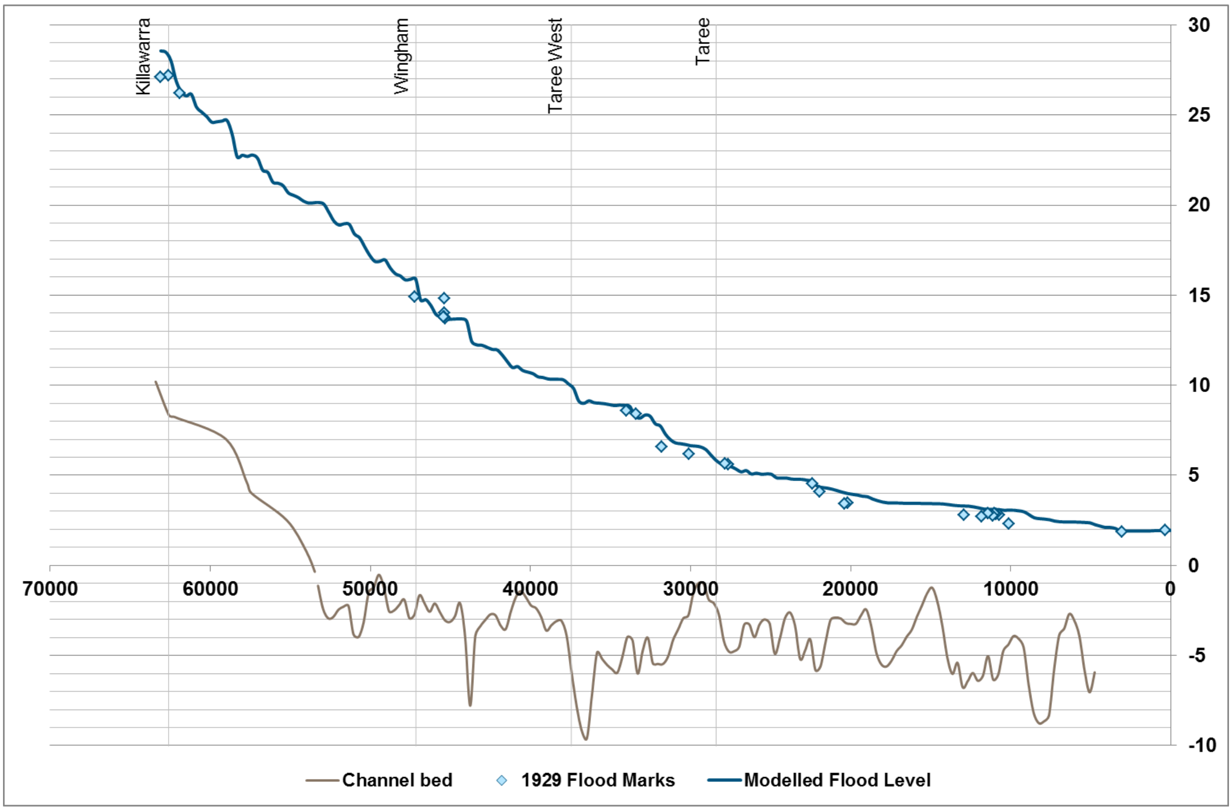


Figure 5-35 Long Section along the Manning River for the February 1929 Event

6 Design Flood Conditions

6.1 Simulated Design Events

Design floods are hypothetical floods used for planning and floodplain management investigations. They are based on having a probability of occurrence specified as Annual Exceedance Probability (AEP) expressed as a percentage. Definition of an AEP is contained in Table 6-1.

Table 6-1 Design Flood Terminology

AEP	Comments
0.2%	A hypothetical flood or combination of floods which represent the worst case scenario with a 0.2% probability of occurring in any given year.
0.5%	As for the 0.2% AEP flood but with a 0.5% probability.
1%	As for the 0.2% AEP flood but with a 1% probability.
2%	As for the 0.2% AEP flood but with a 2% probability.
5%	As for the 0.2% AEP flood but with a 5% probability.
20%	As for the 0.2% AEP flood but with a 20% probability.
Extreme Flood / PMF[1]	A hypothetical flood or combination of floods which represent an extreme scenario.

1 *A PMF (Probable Maximum Flood) is not necessarily the same as an Extreme Flood*.

The design events to be simulated include the 20% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF events. The 1% AEP flood is generally used as a reference flood for development planning and control for residential development.

In determining the design floods it is necessary to take into account:

- Flood frequency analyses at locations of historic flood records. These provide a statistical estimate of design peak flow conditions from the available recorded data and are used to in conjunction with the design rainfall outputs from the hydrological model to establish appropriate design flood conditions, particularly as the major inflow at the upstream extent of the hydraulic model.
- Design rainfall parameters (rainfall depth, temporal pattern and spatial distribution). These inputs drive the hydrological model, from which design flow hydrographs will be extracted as local inputs to the hydraulic model.
- Design downstream ocean boundary levels.

- Design entrance channel initial geometry and geomorphology.
- The potential impact of future climate change on catchment inflows and design ocean levels.

As discussed, the entrance condition is a significant controlling feature in terms of flood water levels observed in lower floodplain areas. The Department of Environment, Climate Change and Water's (DECCW's) Flood Risk Management Guide: Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways (2015), provides guidance on selecting appropriate initial geometry and breakout regimes for shoaled entrances, design ocean boundary levels and coincident catchment and ocean flood conditions.

The considerations outlined above are detailed in the following sections.

6.1.1 Coincident Catchment and Ocean Flood Events

A range of design events was defined to model the behaviour of coincident flooding from both catchment and ocean sources within the Manning River catchment. An overview of adopted model conditions for these design events is presented in Table 6-2, as recommended in the Flood Risk Management Guide (DECCW, 2015). The adopted ocean boundary conditions are discussed in Section 6.5.

Table 6-2 Design Model Runs for Coincident Flood Events

Design Flood Event	Killawarra Boundary Peak Inflow (m^3/s)	Local Rainfall	Ocean Boundary Peak Water Level (m AHD)
20% AEP	4,100 (20% AEP)	20% AEP 48h duration	1.03 (HHWS(SS))
5% AEP	6,700 (5% AEP)	5% AEP 48h duration	1.03 (HHWS(SS))
2% AEP	8,100 (2% AEP)	2% AEP 48h duration	1.90 (5% AEP)
1% AEP	9,200 (1% AEP)	1% AEP 48h duration	1.90 (5% AEP)
0.5% AEP	10,300 (0.5% AEP)	0.5% AEP 48h duration	2.00 (1% AEP)
0.2% AEP	11,900 (0.2% AEP)	0.2% AEP 48h duration	2.00 (1% AEP)
PMF	27,600 (3 x 1% AEP)	3 x 1% AEP 48 duration	2.00 (1% AEP)

6.1.2 Tidal Inundation

As per Council's brief, existing tidal inundation is to be mapped. The High High Water Spring (Solstice Spring) tidal signature provided in the Flood Risk Management Guide (DECCW, 2015) for locations south of Crowdy Head (peak water level at 1.03m AHD) was adopted as the ocean water level boundary. To assess tidal inundation independently from flood events along the Manning River, model inflows were not applied for this simulation.

6.1.3 Climate Change

The potential impact of climate change on flood behaviour within the lower Manning River catchment has also been considered. The impact of sea level rise on flood levels and tidal inundation extents was assessed, as was the potential for increases in design rainfall intensities.

Increase in flood producing rainfall events due to climate change can be assessed by undertaking sensitivity analyses on design events for up to 30% increase in flow rates. For this study, the 0.5% AEP and 0.2% AEP events were adopted for the rainfall intensity assessment, as representative of an approximate 10% and 30% increase in flows respectively. The baseline ocean boundary condition will remain at the 5% AEP peak water level, as adopted for the coincident 1% AEP design flood event. Further detail is contained in Section 6.3.5.

To assess the impacts of sea level rise, projected sea level rises of 0.28m by 2050 and 0.98m by 2100 were added to ocean boundary water levels. Further detail surrounding adopted sea level rise projections is contained in Section 6.5.1.

Scenarios involving increased river flows coincident with increased sea level rise have also been included to provide a range of potential climate change flood conditions. An overview of adopted model conditions for the climate change scenarios is presented in Table 6-3.

Table 6-3 Design Model Runs for Climate Change Assessment – Increase in Design Rainfall Intensities and Sea Level Rise

Design Flood Event	Killawarra Boundary Peak Inflow (m^3/s)	Local Rainfall	Ocean Boundary Peak Water Level (m AHD)
1% AEP + 10% Flow	10,300 (0.5% AEP)	0.5% AEP 48h duration	1.90 (5% AEP)
1% AEP + 30% Flow	11,900 (0.2% AEP)	0.2% AEP 48h duration	1.90 (5% AEP)
1% AEP w/ 2050 SLR	9,200 (1% AEP)	1% AEP 48h duration	2.30 (5% AEP +028.m to 2050)
1% AEP w/ 2100 SLR	9,200 (1% AEP)	1% AEP 48h duration	2.80 (5% AEP +0.98m to 2100)
1% AEP + 10% Flow w/ 2050 SLR	10,300 (0.5% AEP)	0.5% AEP 48h duration	2.30 (5% AEP +0.28m to 2050)
1% AEP + 10% Flow w/ 2100 SLR	10,300 (0.5% AEP)	0.5% AEP 48h duration	2.80 (5% AEP +0.98m to 2100)
1% AEP + 30% Flow w/ 2050 SLR	11,900 (0.2% AEP)	0.2% AEP 48h duration	2.30 (5% AEP +0.28m to 2050)
1% AEP + 30% Flow w/ 2100 SLR	11,900 (0.2% AEP)	0.2% AEP 48h duration	2.80 (5% AEP +0.98m to 2100)
Tidal Inundation w/ 2050 SLR	Nil	Nil	1.43 (HHWS(SS) +0.28m to 2050)
Tidal Inundation w/ 2100 SLR	Nil	Nil	1.93 (HHWS(SS) +0.98m to 2100)

6.2 Flood Frequency Analysis

The long history of flood records within the lower Manning River catchment allow for a flood frequency analysis to be undertaken. The two sites selected for analysis are:

- Killawarra – This site was selected for a flood frequency analysis due to the long history of recorded flood levels at the gauging site. In addition, the site is well suited to rating analysis as the rising and receding limb of the rating curve both follow a very similar path, allowing for confidence in the water level to flow rate conversion. Although outside of the area of interest of this study, it forms the upstream extent of the hydraulic model and inflows at this location account for the majority of flood flow within the lower Manning River catchment. A flood frequency analysis at a site with a long history of data can provide greater certainty in the design flood flow rates when compared to deriving them from a hydrological model of the entire catchment area.
- Taree – There is also a long history of recorded peak flood levels available in Taree, although the gauging site has only been operational since 2010. Deriving design flood flows at Taree will allow the modelling to provide the best estimate of consistent magnitude design flood conditions across the wider study area, i.e. the 1% AEP design flood event will produce the 1% AEP design flood flows and levels determined from the flood frequency analysis at both Killawarra and Taree.

The hydraulic model was used to derive a rating curve at each site, from which the recorded flood levels were converted to flows.

The TUFLOW FLIKE extreme value analysis package was used to undertake the flood frequency analyses. Developed by Professor George Kuczera from the School of Civil Engineering at the University of Newcastle Australia, TUFLOW FLIKE is compliant with the recent major revision of industry guidelines for flood estimation, documented in the draft update of Australian Rainfall and Runoff (ARR).

The FLIKE analyses used a Bayesian inference method with the Gumbel probability model. The FLIKE package has the capability to perform probabilistic analysis with other models, including Log-normal, Log Pearson III, Generalised Extreme Value and Generalised Pareto. However, the Gumbel distribution was selected as it provided the best fit against high-flow historic thresholds.

6.2.1 Manning River at Killawarra

The water level gauge located on the Manning River at Killawarra has been in operation since 1945 and as such offered sufficient data to undertake a flood frequency analysis at the site. Annual maxima water levels were extracted from the available data, which were then converted to flow rates based on the rating curve derived from the TUFLOW model at this location.

The Killawarra analysis had a total of 71 annual maxima available, of which the lowest two were excluded from the analysis. There were also seven significant floods on record having occurred prior to installation of the stream gauge in 1945. These floods (detailed in Table 6-4) were included beyond the period of gauge record, assuming two occurrences of floods with a threshold flow of above 9,000m^3/s and seven occurrences above a threshold flow of 7,000m^3/s in the years between

1830 and the start of the gauge record in 1945 (representative of the 1978 and 2011 floods respectively).

The fitted Gumbel distribution is presented on Figure 6-1 along with the 90% confidence limits and plotting positions of the observed annual maxima and historic records.

Table 6-4 Historic Threshold Floods having Occurred Prior to 1945 - Manning River at Killawarra

Year	Stage (m AHD)	Flow (m^3/s)
1866	19.1	10,000
1929	18.5	9,300
1895	17.6	8,400
1875	17.1	7,900
1894	16.9	7,700
1930	16.8	7,600
1870	16.4	7,300

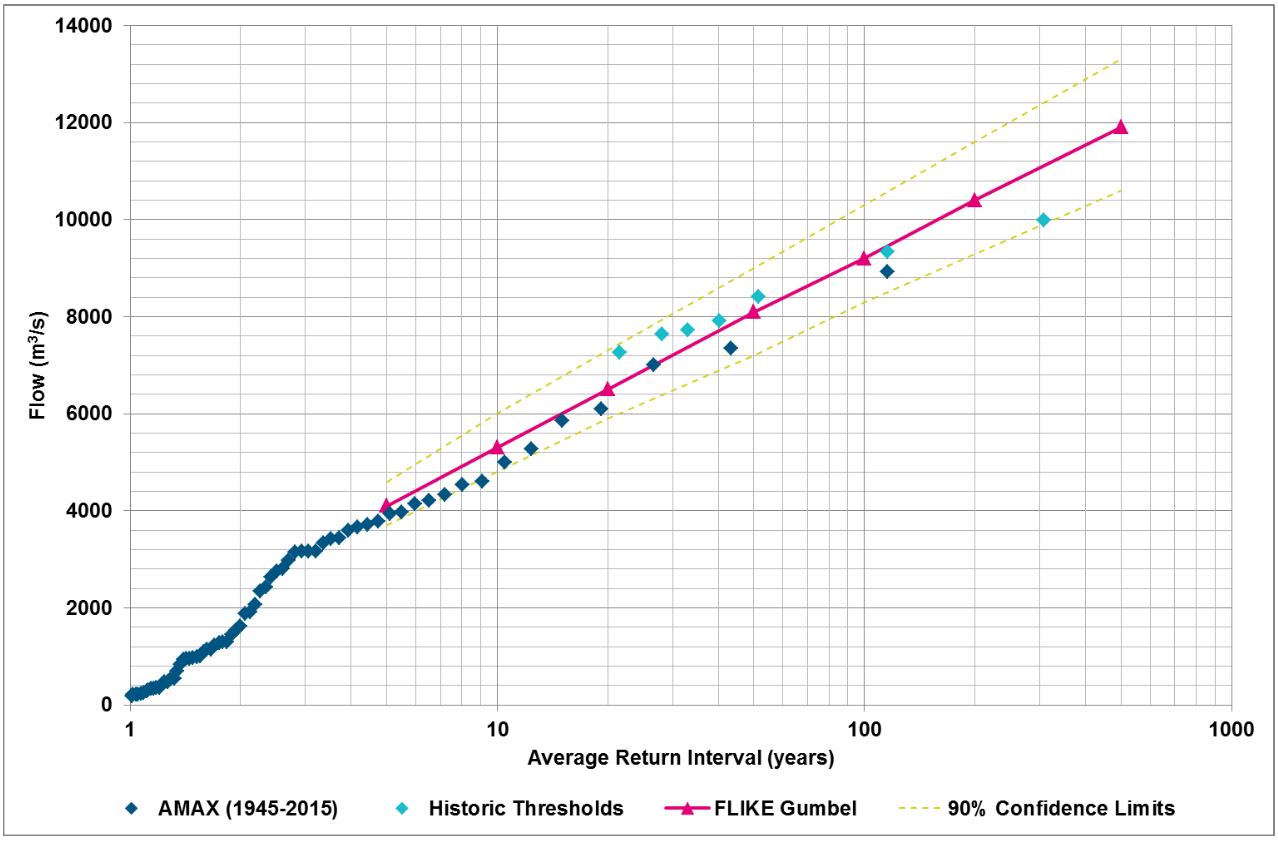


Figure 6-1 Flood Frequency Analysis for the Manning River at Killawarra

6.2.2 Manning River at Taree

As the water level gauge located on the Manning River at Taree (Martin Bridge) has only been in operation since 2010, it did not offer sufficient data to undertake a flood frequency analysis at the site based on annual maxima water levels, as was undertaken for the Killawarra site.

It can be assumed that all floods of significance since 1830 are known and are included in the record of historical peak levels for Taree (Public Works, 1981). Using the Cunnane method to determine plotting positions for each flood event resulted in a frequency distribution that appeared to overestimate expected flow rates for low-mid range order events.

Given the length of record available at the Killawarra gauging sites it is expected that the frequency analysis estimates should be reasonably reliable. Rainfall runoff inputs downstream of Killawarra (calculated from the hydrological model) would be required to be doubled to achieve flows at Taree in line with the Cunnane derived distribution.

An alternative approach was therefore adopted, where the historical peak levels were supplemented with the annual maxima data recorded at the Killawarra gauge. The annual maxima flow series (converted from levels using the TUFLOW model rating) from the Killawarra gauge was used as a surrogate for representative annual maxima flow statistics at Taree.

In some years the maximum recorded flow at Taree may be significantly higher than at Killawarra, when driven by a flood event with the highest intensity rainfall occurring over the lower Manning River catchment. However, in other years the maximum recorded flow at Tarre may be lower than that at Killawarra, when driven by a flood event occurring principally across the upper catchment of the Manning River, as the flood flow hydrograph attenuates when progressing downstream through the lower catchment. It was assumed that in a long-term statistical dataset the annual maxima flow statistics would be reasonably similar at the two sites.

A FLIKE Flood Frequency Analysis was undertaken at Taree using the annual maxima flow data at Killawarra from 1945. Years for which recorded flood levels were available at Taree were used to provide an appropriate peak flow estimate in the analysis. Recorded peak flood levels at Taree for historic events preceding 1945 were incorporated as threshold exceedances, as was utilised for Killawarra.

The FLIKE approach provides a more robust statistical analysis of data and calculated a frequency distribution of flows at Taree that was more consistent to that determined at Killawarra. Flood flows increased by an average of 15% between Killawarra and Taree and provided a close match to additional local inflows downstream of Killawarra determined from rainfall-runoff modelling utilising design rainfall IFDs. This is consistent with the increase in upstream catchment areas between the two gauge locations, which is around a 13% increase (around 7,480km^2 compared to 6,640km^2).

As discussed, the Taree analysis used the Killawarra continuous record as a surrogate (replaced with Taree data, where available) and had a total of 71 annual maxima available, of which the lowest two were excluded from the analysis. There were an additional ten significant floods on record having occurred prior to installation of the Killawarra stream gauge in 1945. These floods (detailed in Table 6-5) were included beyond the period of gauge record, assuming one occurrence of a flood with a threshold flow of above 11,700m^3/s and nine occurrences above a threshold flow of 7,700m^3/s in the years between 1830 and the start of the gauge record in 1945 (representative of the 1978 and 2011 floods respectively).

It should be noted that floods having occurred prior to the installation of the gauge on the Martin Bridge in 2010 are assumed to have been recorded at the Taree Aquatic Club on Macquarie Street, approximately 740m downstream of the bridge. Rating curves were derived for both the Martin Bridge and Macquarie Street location from the TUFLOW hydraulic model. This allowed for peak flood levels to be ascertained at both locations based on the peak flow through Taree.

The fitted Gumbel distribution is presented on Figure 6-2 along with the 90% confidence limits and plotting positions of the observed annual maxima and historic records. The Killawarra flows used to supplement the historic records are clearly indicated on the figure. It can be seen that for Average Return Intervals of interest for design purposes (over a 5 year ARI) is largely determined from data recorded at Taree.

Table 6-5 Historic Threshold Floods having Occurred Prior to 1945 - Manning River at Macquarie St, Taree

Year	Stage (m AHD)	Flow (m^3/s)
1929	5.6	12,300
1866	5.15	10,600
1930	5.1	10,400
1895	4.85	9,500
1875	4.85	9,500
1870	4.65	8,900
1867	4.5	8,400
1894	4.3	7,900
1857	4.25	7,700

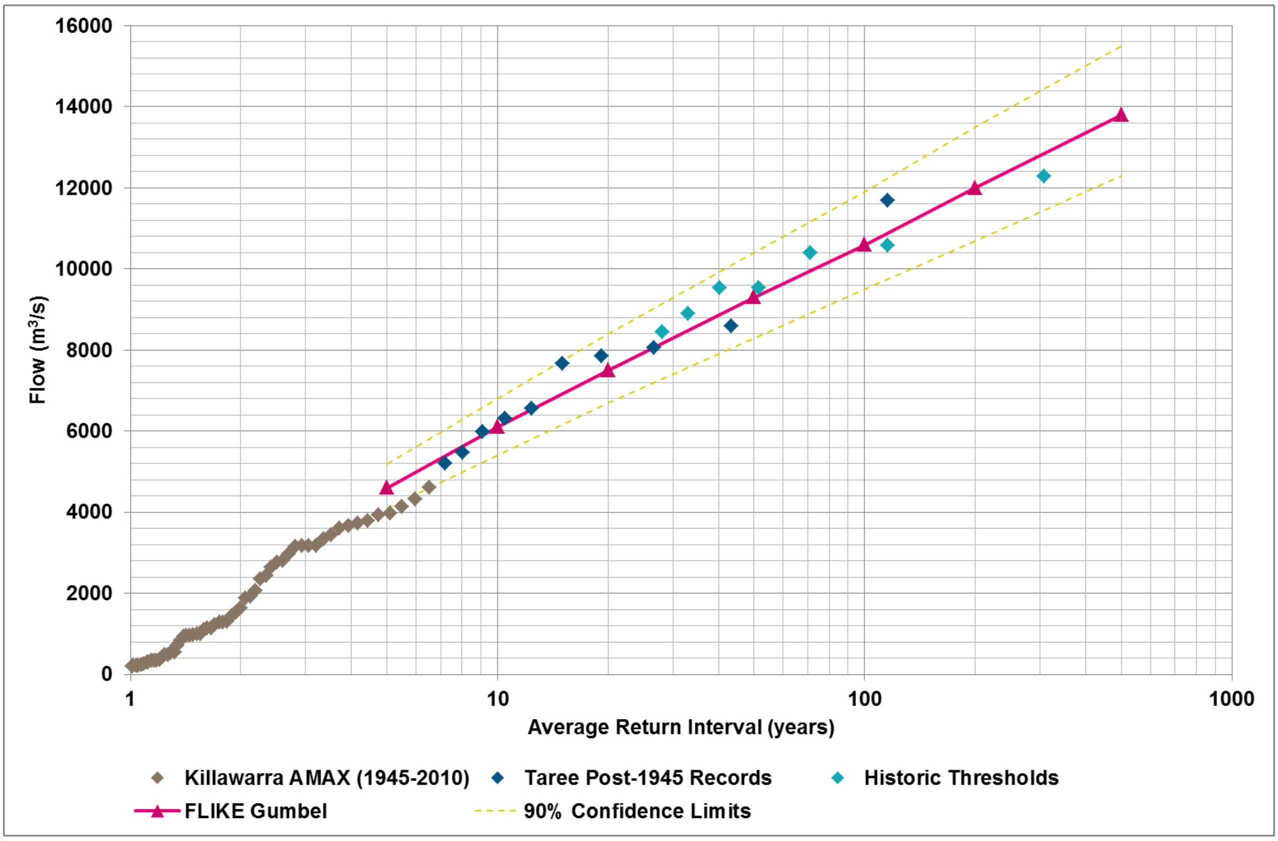


Figure 6-2 Flood Frequency Analysis for the Manning River at Taree

6.3 Design Rainfall

Inflows to the hydraulic model at Killawarra will largely drive the flood behaviour for the design flood events. However, local inflows are required for the lower floodplain area downstream of Killawarra. Design rainfall parameters are input into the RAFTS-XP hydrological model was used to calculate local inflow hydrographs.

Design rainfall parameters are location specific and are derived from standard procedures defined in AR&R (2001) which are based on statistical analysis of recorded rainfall data across Australia.

6.3.1 Rainfall Depths

Design rainfall depth is based on the generation of intensity-frequency-duration (IFD) design rainfall curves utilising the procedures outlined in AR&R (2001). These curves provide rainfall depths for various design magnitudes (up to the 1% AEP) and for durations from 5 minutes to 72 hours.

Design rainfall parameters for input into the hydrological model only need to be determined for the lower Manning River catchment (see Figure 5-4). However, as part of the historical rainfall event analysis presented in Section 5.2.3 and Section 5.4.2, average design rainfall intensities applicable to the centre of the entire Manning River catchment area were used for comparative purposes.

Table 6-6 shows the average design rainfall intensities applicable to the centre of the lower Manning River catchment (downstream of Killawarra), as based on the 1987 AR&R IFDs.

Table 6-6 Average Design Rainfall Intensities (mm/hr) (AR&R, 1978)

Storm Duration (hr)	Design Event Frequency			
	20% AEP	5% AEP	2% AEP	1% AEP
12	11.3	14.8	17.5	19.6
24	7.58	10.0	11.8	13.2
48	4.95	6.50	7.68	8.57
72	3.78	4.97	5.86	6.55

The Probable Maximum Precipitation (PMP) is used in deriving the Probable Maximum Flood (PMF) event. The theoretical definition of the PMP is "the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of year" (AR&R, 2001). The ARI of a PMP/PMF event ranges between 10^4 and 10^7 years and is beyond the "credible limit of extrapolation". That is, it is not possible to use rainfall depths determined for the more frequent events (1% AEP and less) to extrapolate the PMP. For this study, the PMP has been estimated as three times the 1% AEP design flood flows.

The catchment size also required an Areal Reduction Factor (ARF) to be applied to the design point rainfall depths. This was undertaken using the recommended approach in the AR&R Revision Project 2 – Spatial Patterns of Rainfall (Sinclair Knight Merz, 2013). The areal reduction factors determined for each storm duration are presented in Table 6-7.

Table 6-7 Design Rainfall Areal Reduction Factors

Storm Duration (hr)	Lower Manning River Catchment, Downstream of Killawarra (1520km^2)
12	0.775
18	0.814
24	0.839
36	0.871
48	0.889
72	0.911

6.3.2 Temporal Patterns

The IFD data presented in Table 6-6 provides for the average intensity (or total depth) that occurs over a given storm duration. Temporal patterns are required to define what percentage of the total rainfall depth occurs over a given time interval throughout the storm duration. The temporal patterns adopted in this study are based on the standard patterns presented in AR&R (2001).

The same temporal pattern has been applied across the whole catchment. This assumes that the design rainfall occurs simultaneously across each of the modelled sub-catchments. The direction of a storm and relative timing of rainfall across the catchment may be determined for historical events if sufficient data exists, however, from a design perspective the same pattern across the catchment is generally adopted.

6.3.3 Rainfall Losses

Standard initial and continuing loss values of 10mm and 2.5mm/hr were adopted, as recommended in AR&R for coastal NSW. These are consistent with those adopted for the calibration and validation events.

6.3.4 Critical Durations

The critical duration is the storm duration for a given event magnitude that provides for the peak flood conditions at the location of interest. For example, small catchments are more prone to flooding during short duration storms, while for large catchments longer durations will be more critical.

A critical duration of 48 hours was adopted for design rainfall events and is expected for a catchment of this size. For this study, the critical duration for the catchment was determined based on analysis of flow hydrographs observed at Killawarra for historical flood events. The critical duration analysis is detailed further in Section 6.4.2.

6.3.5 Climate Change

Current guidelines predict that a likely outcome of future climatic change will be an increase in extreme rainfall intensities. Climate Change in New South Wales (CSIRO, 2004) provides projected regional changes in rainfall intensities for each season and annually for the years 2030 and 2070. The Manning River catchment falls into the North-East region of NSW where compared to other regions in the state, projected increases are not as significant. It has been projected that 2.5% AEP 24 hour duration annual rainfall depths will increase by more than 5% by the year 2030 and 2070 in the study catchment. The 2.5% AEP 72 hour duration annual rainfall depth projections are increases by 5% for the year 2070.

The NSW Government has also released a guideline (DECCW, 2007) for Practical Consideration of Climate Change in the floodplain management process that advocates consideration of increased design rainfall intensities of up to 30%.

In line with this guidance note, additional tests incorporating a 10% increase to design rainfall at 2050 and a 30% increase to design rainfall at 2100 have been undertaken. The design flows for the 0.5% AEP and 0.2% AEP event are around 10% and 30% higher, respectively, than those of the 1% AEP, so comparison of these two events provides an appropriate assessment for potential impacts of increased design rainfall depths. Results of the sensitivity testing are contained in Section 7.7.

6.4 Adopted Design Flows

6.4.1 Design Peak Flows

The peak design flows determined from the flood frequency analyses at Killawarra and Taree are detailed in Section 6.2.

Design flow input at Killawarra achieved a very close match to required design flows at Taree. As this study focuses on the lower Manning floodplain, it was decided that matching the flood frequency flows at Taree would be the primary aim. For simplicity, the Killawarra inflow was therefore adjusted where necessary to hit the required peak flow at Taree. Alternatively, local rainfall inputs could be modified.

Table 6-8 Design Peak Flow Rates

Design Event	Killawarra		Taree
	FFA Flow Rate (m^3/s)	Adopted Flow Rate (m^3/s)	FFA Flow Rate (m^3/s)
20% AEP	4,100	4,100	4,600
5% AEP	6,500	6,700	7,500
2% AEP	8,100	8,150	9,300
1% AEP	9,200	9,150	10,600
0.5% AEP	10,400	10,250	12,000
0.2% AEP	11,900	11,900	13,800

6.4.2 Design Inflow Hydrograph Shape

The critical duration was determined based on an analysis of the hydrograph shape observed at Killawarra during historical flood events. The following three hydrographs were used in the analysis:

- The estimated hydrograph adopted for the 1978 model calibration event;
- The recorded water level time series for the 1990 event, converted into a flow hydrograph using the rating curve at Killawarra generated for this study; and
- The hydrograph adopted for the 2011 model calibration event.

Each hydrograph was converted into a factored flow where peak flow rate was assigned a factor of 1.0. The peaks were aligned and a composite hydrograph calculated. The adopted hydrograph is presented in Figure 6-3, scaled to match the peak flow for each design event.

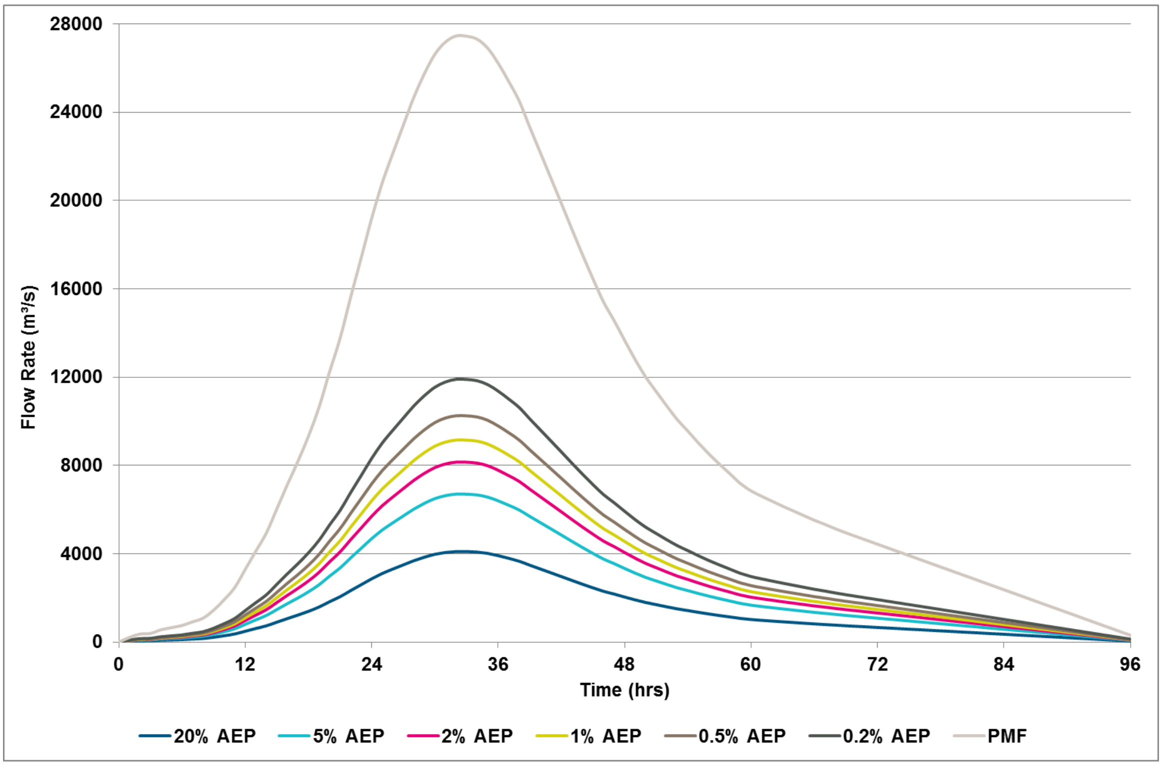


Figure 6-3 Composite Design Flow Hydrographs at Killawarra

6.5 Design Ocean Boundary

Design ocean boundaries for use in flood risk assessments are recommended by the Floodplain Risk Management Guide (OEH, 2015), where the recommended design ocean water levels have been determined based on long term records from Fort Denison in Sydney Harbour. The design levels include the following considerations:

- Barometric pressure set up of the ocean surface due to the low atmospheric pressure of the storm;
- Wind set up due to strong winds during the storm "piling" water upon the coastline;
- Astronomical tide, particularly the HHWS(SS); and
- Wave set up.

OEH (2015) recommends difference design ocean peak water levels be adopted based on the type of entrance. Type A entrances are subject to little ocean tide attenuation and are not influenced by wind and wave set up, e.g. Newcastle Harbour. Type B estuaries are typically open but may be affected by shoaling and may have some potential for wave set up e.g. Harrington. Type C estuaries are prone to heavy shoaling and often close completely (also known as Intermittently Closed and Open Lakes and Lagoons (ICOLLS)). Peak design ocean water levels for each of the different entrance types for locations south of Crowdy Head are presented in Table 6-9. The different peak levels reflect the degree of influence of wave set up applicable to the various types of entrances.

Table 6-9 Design Peak Ocean Water Levels (OEH, 2015) for Various Entrance Types, located South of Crowdy Head

Ocean Event	Peak Ocean Water Level (m AHD)		
	Entrance Type A	Entrance Type B	Entrance Type C
5% AEP	1.4	1.9	2.35
1% AEP	1.45	2.0	2.55

The entrances of Harrington and Farquhar Inlet are best characterised as Entrance Type B; therefore, peak ocean water levels for Entrance Type B have been adopted for simulation of design flood events in this study.

The temporal pattern of the ocean water level boundaries for design flood events was based on the time series provided by OEH (2015). Figure 6-4 presents the design ocean water level time series for entrance Type B along with the HHWS(SS) time series, as applicable for locations south of Crowdy Head. For design events, the timing of the peak water level was adjusted to coincide with the peak catchment inflow, which occurs at around T=32 hours.

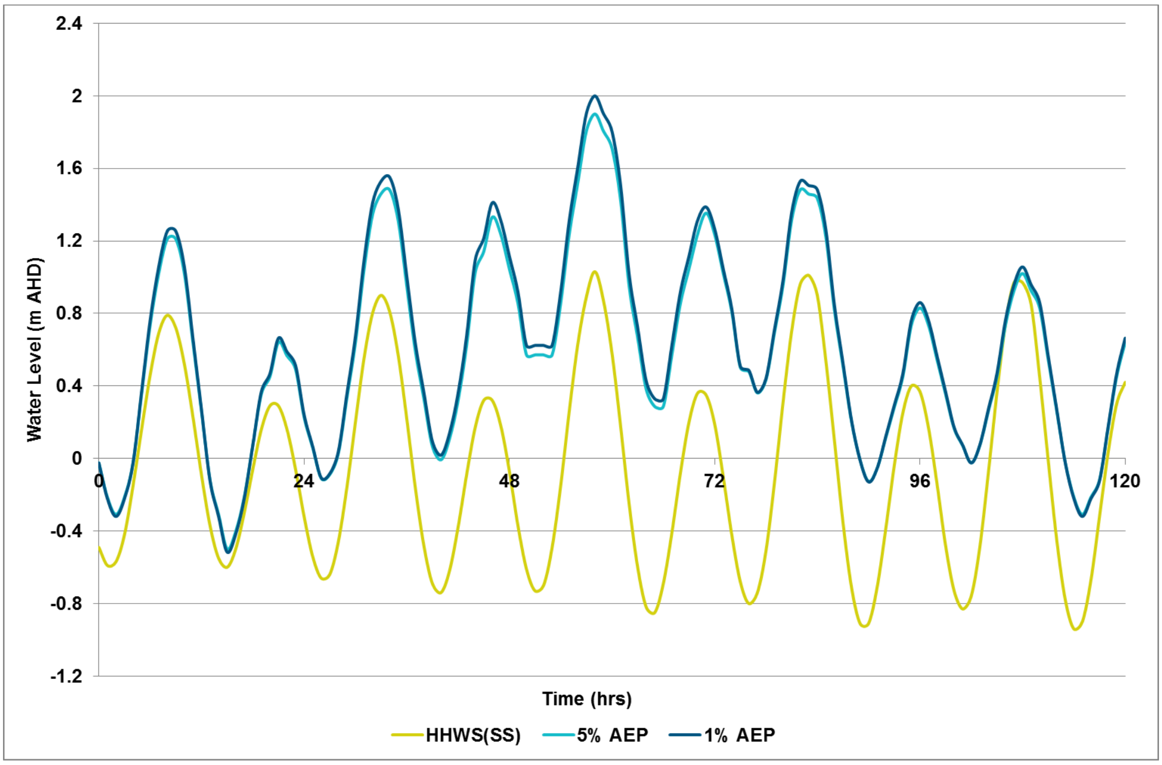


Figure 6-4 Design Ocean Water Level Time Series for Entrance Type B, located South of Crowdy Head (OEH, 2015)

6.5.1 Climate Change

Current guidelines predict that a likely outcome of future climatic change will be an increase in mean sea level. Council does not have an adopted policy on sea level rise projections; however, their acting position is that sea level rise increases of 0.28m by 2050 and 0.98m by 2100 shall be used in this study.

Climate change may also result in an increase in the frequency and intensity of storms, further exacerbating the effects of sea level rise on coastal flood behaviour. The data provided in Projected Changes in Climatological Forcing for Coastal Erosion in NSW (CSIRO, 2007) indicates that a conservative approach would be to adopt around a 10% increase in significant wave heights for the 50 year planning horizon and around a 30% increase for the 100 year planning horizon. An increase in significant wave heights for ocean events would result in an increased wave set up. However, this component has not been incorporated into the climate change assessment for this study.

6.6 Design Entrance Geomorphology

The design berm geometry has a significant influence on modelled flood levels in the lower Manning River floodplain area. In defining the entrance condition for the design flood analysis, consideration has been given to typical initial entrance geometry and expected entrance breakout conditions at Harrington and Farquhar Inlet.

The initial entrance condition adopted for the June 2011 calibration event has been adopted for all design events. At Harrington, this involved the main channel following a north-east alignment along the break wall with a 1.2m AHD berm across the remainder of the entrance. Farquhar Inlet was modelled as a single channel approximately 180m wide.

The TUFLOW-FV hydrodynamic model was used to establish expected entrance breakout conditions for each design flood event. The findings of the June 2011 calibration were used to make an informed decision in regard to the expected breakout conditions at Harrington. The June 2011 event is just over a 10% AEP design event at Killawarra and it was assumed that for a flow of this magnitude, the final breakout scenario at Harrington was almost entirely open. This same breakout regime was therefore adopted for all design events. For the design event simulations, breakout at Harrington was initiated near the onset of the storm and occurred over a period of 24 hours.

The breakout regime at Farquhar Inlet is variable and depends on the flow rate through the system. Based in initial design runs, a relationship between inflow at Killawarra and expected flow upstream of Harrington and Farquhar Inlet could be estimated. For each design event, the flow was distributed to the north and south arm of the Manning River, as the upstream flow input into the TUFLOW-FV model, to determine the expected breakout condition.

A cross section of final bed elevation at the Farquhar Inlet entrance is presented in Figure 6-5 for each design event. It can be seen that the channel becomes progressively wider with increasing flows. For the PMF event, a second entrance breaks out along the sand dune to the south. The final bed elevation output from the TUFLOW-FV model was used to develop a simplified representation of channel breakout for the TUFLOW Classic modelling. For the design event simulations, breakout at Farquhar Inlet was initiated near the onset of the storm and occurred over a period of 48 hours.

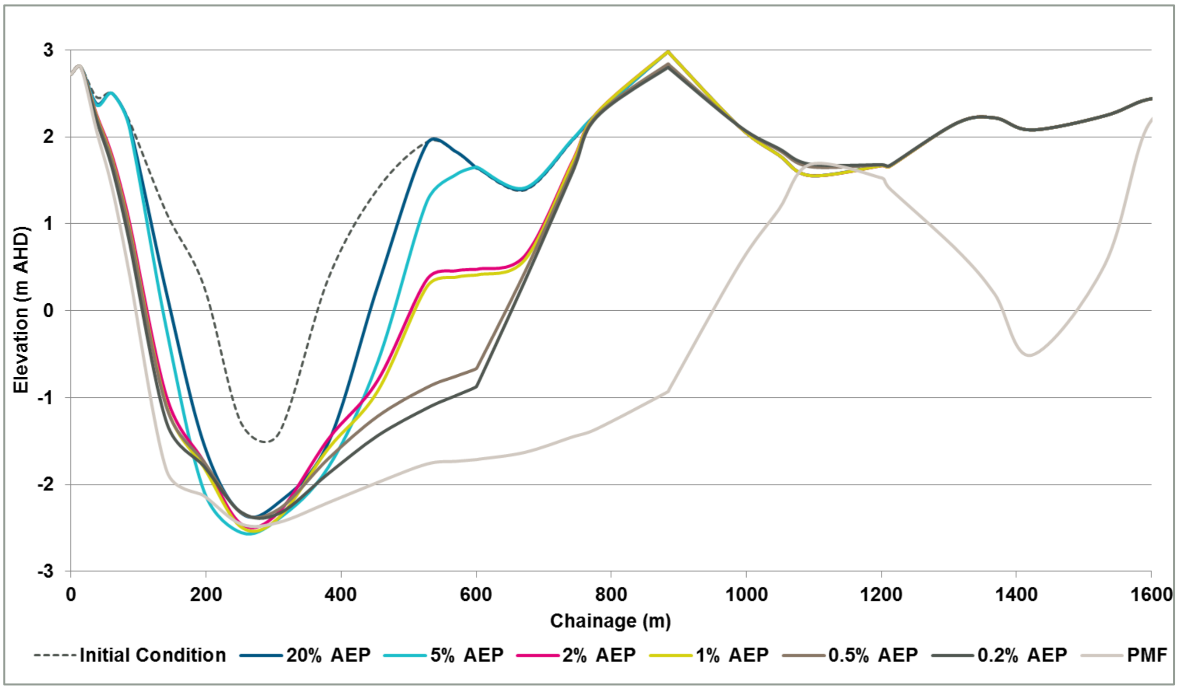


Figure 6-5 Cross Section of Farquhar Inlet Breakout for Design Events

7 Design Flood Results

A range of design flood conditions were modelled, the results of which are presented and discussed below. The simulated design events included the 20% AEP, 5% AEP, 2% AEP, 1% AEP 0.5% AEP and 0.2% AEP for flooding resulting from coincident catchment and ocean events. The PMF flood event has also been modelled.

The impact of future climate change on flooding was also considered, focussing on the 1% AEP flood event.

The design flood results are presented in a separate flood mapping compendium. For the simulated design events including the 20% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF events, a map of peak flood level, depth and velocity is presented covering the modelled area. The model results represent the flood conditions of the Manning River and its floodplain downstream of Wingham.

7.1 Flood Behaviour

The Manning River channel is wide and deep and has considerable flow conveyance capacity. On exceeding flow capacity, flood water spills onto the floodplain. The resulting flood behaviour is dependent on the nature of the floodplain and is consequently vastly different upstream and downstream of Taree.

Between Killawarra and Wingham, the floodplain is typically broad (approximately 300m wide) and well-defined with steep valley sides. Flood behaviour here is characterised by deep and rapidly rising flood waters.

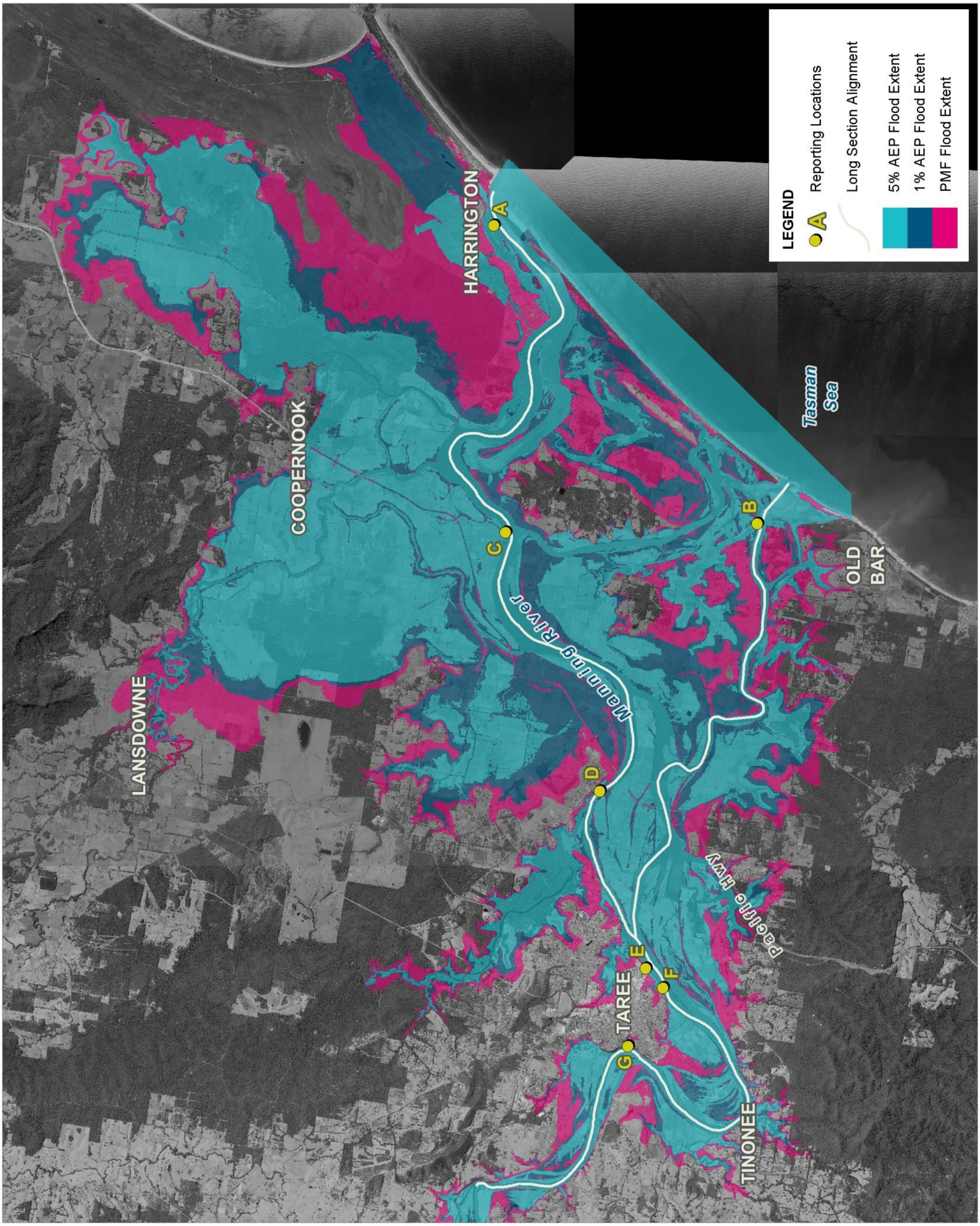
Downstream of Taree, the topography opens into a vast, flat, low-lying floodplain area. Flood waters readily break the southern bank of the channel at Taree during the 5% AEP event, almost completely inundating Dumaresq Island. Inundation of the floodplain results from the backwater influence from the Manning River. Due to the considerable storage area offered by the vast floodplain, the depth of flood water generally is much lower and slower moving. For the 1% AEP, depth of water on the floodplain is typically less than 2m. InterconnectiOn across the floodplain between the entrances at Harrington and Farquhar Inlet is initiated during the 2% AEP event.

Many of the smaller tributaries of the Manning River are perched above the floodplain. The in-channel capacity of the Lansdowne River is breached upstream of Coopernook during the 20% AEP event and significant floodplain inundation occurs downstream of Lansdowne and around Coopernook and Moorland.

7.2 Peak Flood Conditions

Modelled peak flood levels at selected locations (as presented Figure 7-1) are shown in Table 7-1 for the full range of design flood events considered.

Longitudinal profiles showing modelled peak flood levels for the Manning River are shown in Figure 7-2 and Figure 7-3, with the channel bed profile also shown for reference. Both figures are reproduced at A3 size in Appendix B.



Title:
Design Flood Inundation Extents and Reporting Locations

Figure: **7.1**

Rev: **A**

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

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Approx. Scale

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Table 7-1 Modelled Peak Flood Levels (m AHD) for Design Flood Events

ID	Location	Design Event Frequency						
		20% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
A	Harrington	1.1	1.2	2.0	2.0	2.1	2.1	2.4
B	Farquhar Inlet	1.4	1.8	2.1	2.2	2.3	2.5	4.4
C	Croki	1.4	1.9	2.5	2.8	3.0	3.4	5.5
D	Dumaresq Island	2.0	3.1	3.9	4.1	4.5	4.9	7.3
E	Taree, Macquarie St	2.7	4.1	4.8	5.2	5.5	6.0	8.6
F	Taree, Martin Bridge	2.9	4.4	5.1	5.5	5.8	6.3	9.4
G	Taree West	5.3	7.8	8.9	9.5	9.9	10.7	15.5

There is an extremely flat flood gradient between the entrance and Cundletown (Dumaresq Island). For reference, the distance between Harrington and Dumaresq Island reporting location is around 22.7km. For the 1% AEP event, this equates to a flood grade of less than 0.01% and is indicative of the vast floodplain storage area becoming active during large flood events.

7.3 Hydraulic Categorisation

There are no prescriptive methods for determining what parts of the floodplain constitute floodways, flood storages and flood fringes. Descriptions of these terms within the NSW Floodplain Development Manual (DIPNR, 2005) are essentially qualitative in nature. Of particular difficulty is the fact that a definition of flood behaviour and associated impacts is likely to vary from one floodplain to another depending on the circumstances and nature of flooding within the catchment.

The hydraulic categories as defined in the Floodplain Development Manual are:

- **Floodway -** Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.
- **Flood Storage -** Areas that are important in the temporary storage of the floodwater during the passage of the flood. If the area is substantially removed by levees or fill it will result in elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.
- **Flood Fringe -** Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant effect on the flood pattern or flood levels.

A number of approaches were considered when attempting to define flood impact categories across the study catchment. The approach that was adopted derived a preliminary floodway extent from the velocity * depth product (sometimes referred to as unit discharge). The peak flood depth was used to define flood storage areas. The adopted hydraulic categorisation is defined in Table 7-2.

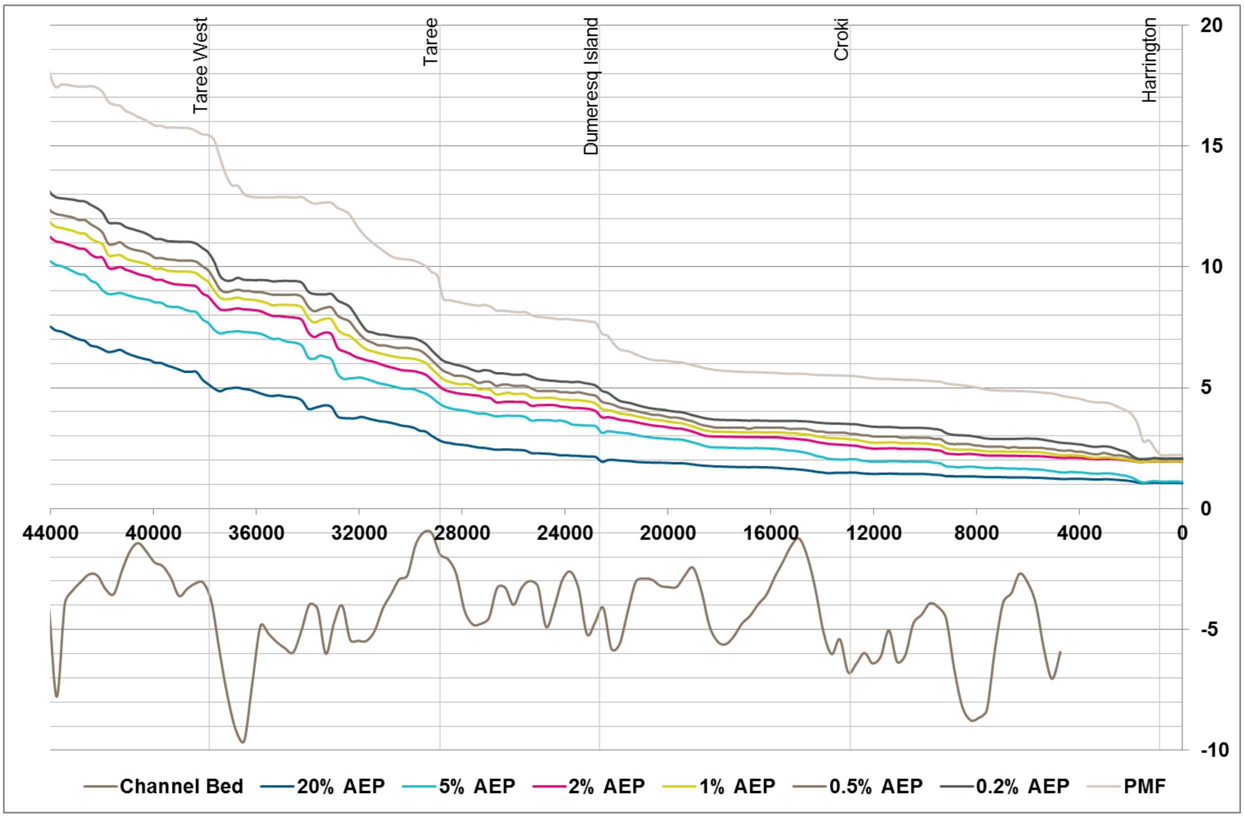


Figure 7-2 Manning River Peak Flood Level Profiles, Main Alignment from Harrington

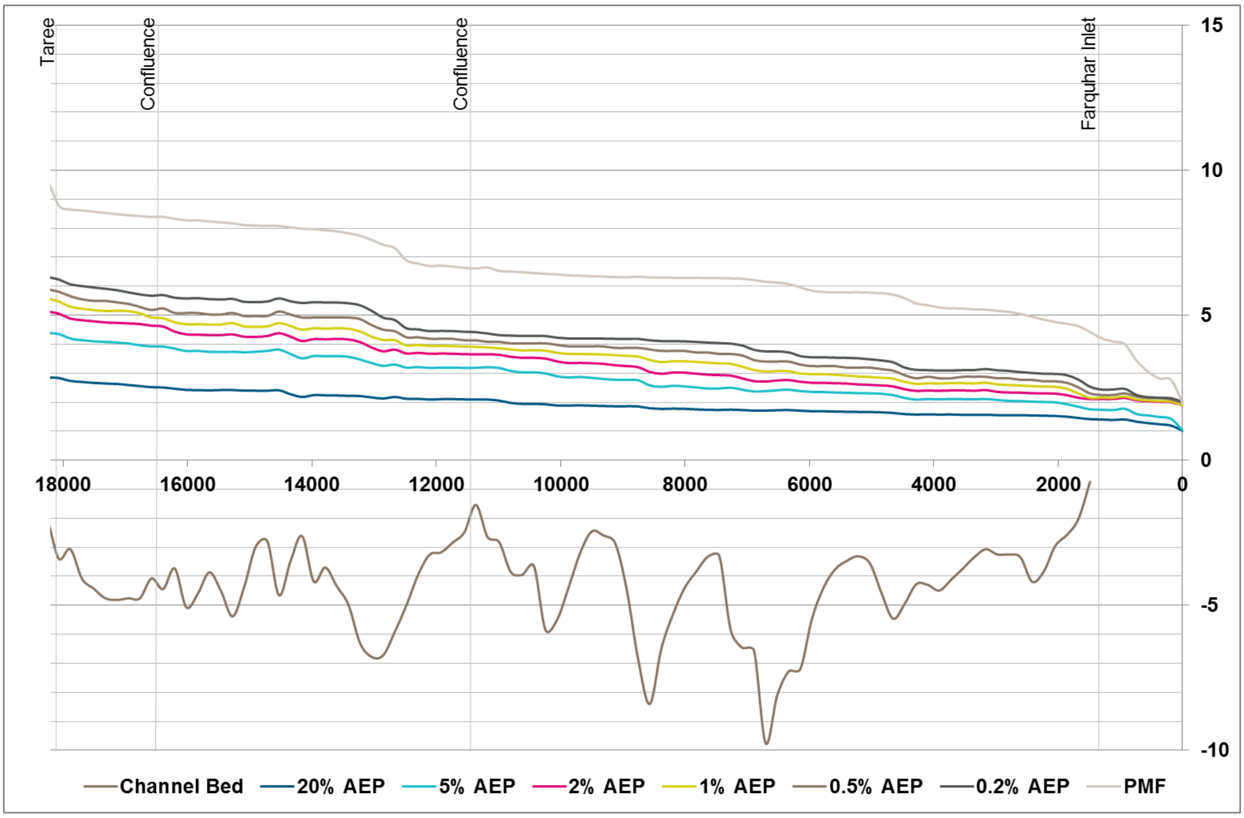


Figure 7-3 Manning River Peak Flood Level Profiles, South Alignment from Farquhar Inlet to Taree

Table 7-2 Hydraulic Categories

Floodway	Velocity * Depth > 0.3 at the 1% AEP event	Areas and flow paths where a significant proportion of floodwaters are conveyed (including all bank-to-bank creek sections).
Flood Storage	Velocity * Depth < 0.3 and Depth > 0.5 metres at the 1% AEP event	Areas where floodwaters accumulate before being conveyed downstream. These areas are important for detention and attenuation of flood peaks.
Flood Fringe	Flood extent of the PMF event	Areas that are low-velocity backwaters within the floodplain. Filling of these areas generally has little consequence to overall flood behaviour.

Preliminary hydraulic category mapping is included in the Mapping Compendium, and is presented for the 1% AEP event, the 1% AEP event with sea level rise projections to 2050 and 2100 and the PMF.

For the 1% AEP event, the floodway area is extensive. Upstream of Taree, most of the floodplain is classed as floodway. Dumaresq Island becomes inundated with flood water and is almost entirely classed as floodway. Much of the lower floodplain area that becomes inundated during the 1% AEP event is classed as flood storage. Flood ways are largely contained to channels and major spillways. For the PMF event, almost the entire floodplain is classed as floodway, as there is typically around 2m-4m depth of flood water on the floodplain.

7.4 Provisional Flood Hazard

The NSW Floodplain Development Manual (DIPNR, 2005) defines flood hazard categories as follows:

- **High hazard** – possible danger to personal safety; evacuation by trucks is difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings; and
- **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 key factors influencing flood hazard or risk are:

  - Size of the Flood
  - Rate of Rise - Effective Warning Time
  - Community Awareness
  - Flood Depth and Velocity
  - Duration of Inundation

- Obstructions to Flow
- Access and Evacuation

The provisional flood hazard level is often determined on the basis of the predicted flood depth and velocity. This is conveniently done through the analysis of flood model results. A high flood depth will cause a hazardous situation while a low depth may only cause an inconvenience. High flood velocities are dangerous and may cause structural damage while low velocities generally have no major threat.

Figures L1 and L2 in the Floodplain Development Manual are used to determine provisional hazard categorisations within flood liable land. These figures are reproduced in Figure 7-4. The provisional hydraulic hazard is included as mapping series H of the Mapping Compendium and is based on the 1% AEP design event.

Provisional hazard category mapping is included in the Mapping Compendium, and is presented for the 1% AEP event, the 1% AEP event with sea level rise projections to 2050 and 2100 and the PMF.

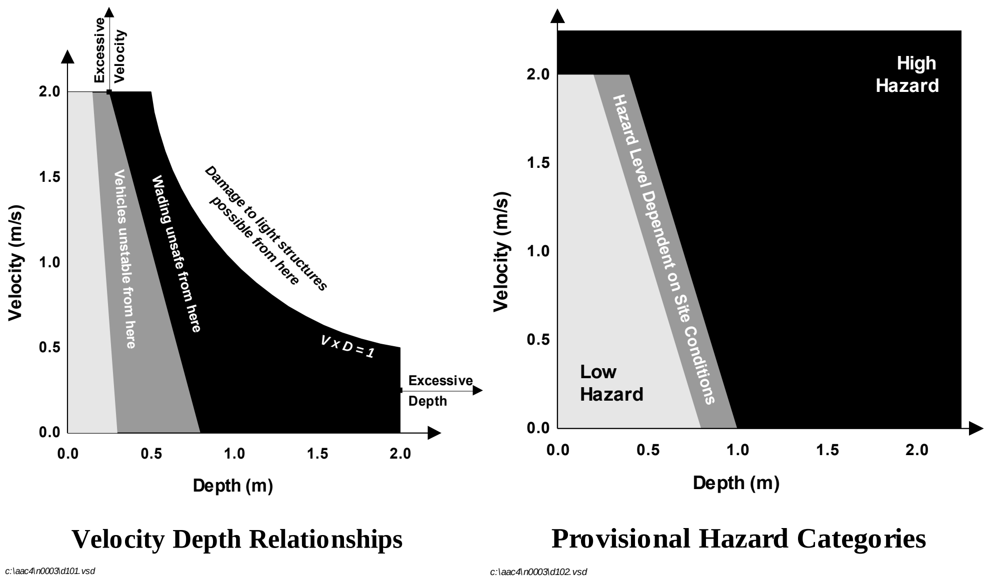


Figure 7-4 Provisional Flood Hazard Categorisation

7.5 True Hazard

The true hazard categorisation is typically based on the hydraulic hazard categorisation discussed in Section 7.3. However, it also takes into consideration other flood risks, particularly those relating to personal safety and evacuation. Due to the long critical duration of flooding in the lower Manning River catchment, rural properties may become isolated for extended periods of time.

Given the potential for rising floodwaters to isolate areas of Low Hazard within High Hazard areas, the provisional hazard has been modified to reclassify islands of Low Hazard as High Hazard.

True hazard category mapping is included in the Mapping Compendium, and is presented for the 1% AEP event, the 1% AEP event with sea level rise projections to 2050 and 2100 and the PMF.

7.6 Flood Planning Level

Flood Planning Levels (FPLs) are used for planning purposes, and directly determine the extent of the Flood Planning Area (FPA), which is the area of land subject to flood-related development controls. The FPL is the level below which a Council places restrictions on development due to the hazard of flooding.

It is typical for the flood planning level to be derived from a designated design flood event plus a 0.5m freeboard allowance, to account for a number of underlying uncertainties. The 1% AEP event is usually adopted as the designated flood, however the FPL and FPA can include allowances for future climate change conditions (i.e. sea level rise and rainfall intensity increase). The adopted FPL and associated FPA will be used by Council for flood planning purposes in the Lower Manning.

7.7 Sensitivity Tests

7.7.1 Climate Change

7.7.1.1 Increased Rainfall Intensity

The potential impacts of future climate change in the form of increased rainfall intensities were considered for the 1% AEP design event. The projected increases in rainfall intensities expected for the study area and the approach adopted to incorporate these into the modelling is detailed in Section 6.3.5. Increases in rainfall intensities by around 10% result in increase in peak flood levels of 0.3m for the 1% AEP event at Taree. For around 30% increases in rainfall intensity, peak flood levels increase by 0.8m at Taree.

7.7.1.2 Sea Level Rise

A sensitivity test was undertaken for the 1% AEP design event to assess the impact of the adopted sea level boundary for the catchment derived flood events. Council's acting position of sea level rise is detailed in Section 6.5.1. Sea level rise impacts largely diminish upstream of Dumaresq Island. At Taree, a difference of 0.1m is observed between the base line 1% AEP peak flood levels and the 2100 sea level rise scenario.

7.7.1.3 Coincident Increased Rainfall Intensity and Sea Level Rise

Scenarios involving increased river flows coincident with increased sea level rise were also assessed to provide a range of potential climate change flood conditions.

Longitudinal profiles showing the impacts of increased rainfall intensity, sea level rise and coincident climate change scenarios for the Manning River are shown in Figure 7-5 and Figure 7-6. Peak modelled flood levels are presented in Table 7-3 at the end of this Section.

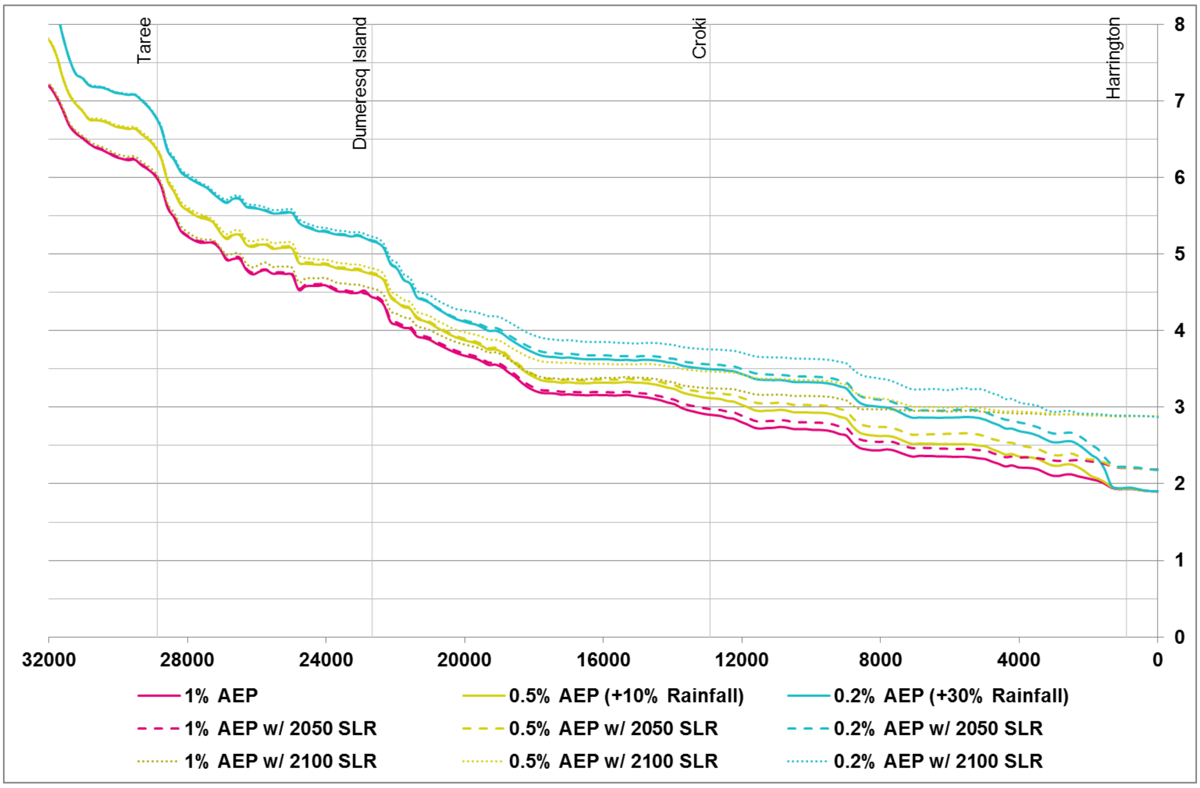


Figure 7-5 Manning River Peak Flood Levels for Climate Change Scenarios, Main Alignment from Harrington

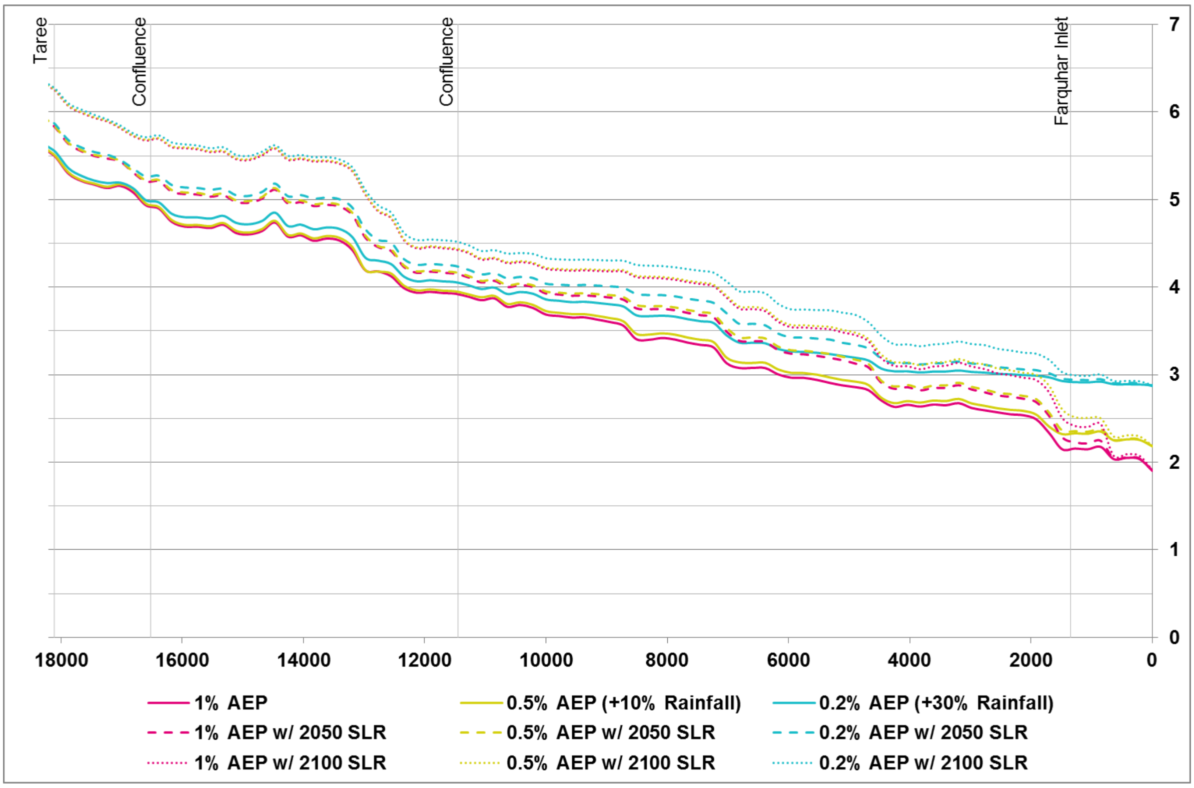


Figure 7-6 Manning River Peak Flood Levels for Climate Change Scenarios, Main Alignment from Harrington

7.7.2 Channel and Floodplain Roughness

The sensitivity of modelled peak flood levels to the adopted Manning's 'n' roughness values were tested for the 1% AEP design event. Roughness values for all materials types within the channel and floodplain were increased and decreased by 25%. Longitudinal profiles showing the result of this assessment for the Manning River are shown in Figure 7-9 and Figure 7-10. Peak modelled flood levels are presented in Table 7-3 at the end of this Section.

7.7.3 Entrance Geometry

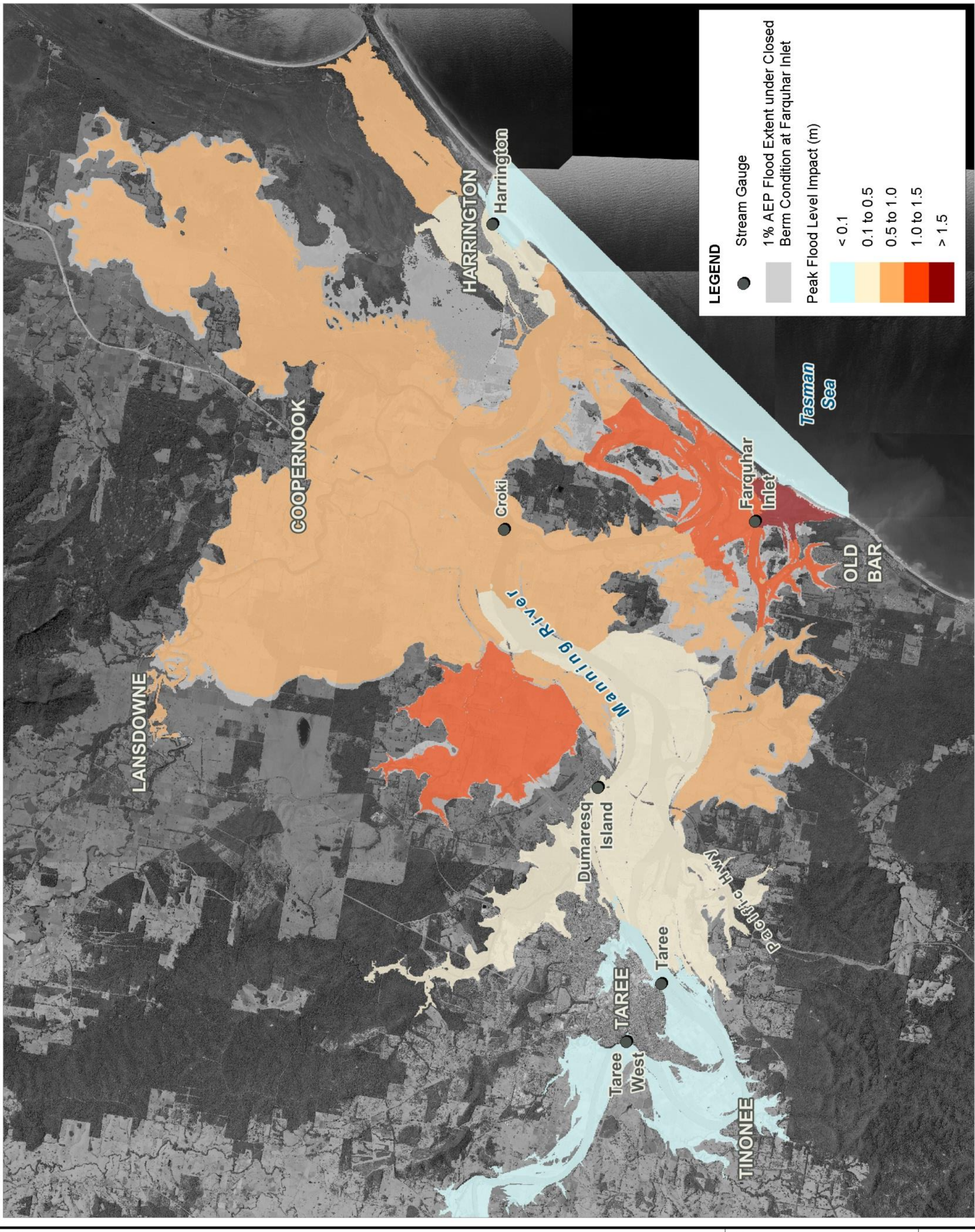
Design flood modelling adopted the same initial entrance geometry that was assumed for the 2011 calibration event where both entrances were relatively open at the onset of the event (as detailed in Section 5.3). As the Harrington entrance is permanently open, scenarios that involved a more closed entrance condition at Farquhar Inlet were modelled, to assess the sensitivity of peak flood levels to the initial entrance configuration (i.e. channel depth/width and dune elevation).

For design events, the adopted initial condition at Farquhar Inlet consisted of a 2.0m AHD berm, with the main channel defined at -1.4m AHD (approximately 180m wide). For the entrance sensitivity scenarios, the impact of a more restrictive berm geometry at Farquhar Inlet is considered. The simulated 'closed' berm scenarios ranged from increases to the channel bed elevation through to raising the entire berm to an elevation of 4m AHD. To model these scenarios, the TUFLOW-FV model was extended to cover the entire lower Manning River floodplain from the entrances to Taree, due to the significant backwater effect on the relative flow distribution between the two entrances. Design flow rates at Taree were extracted from the TUFLOW Classic model and applied as the upstream inflow boundary. The downstream ocean water level boundary was as per TUFLOW Classic design simulations. Initial entrance geometry at Farquhar Inlet was altered to represent the range of berm scenarios considered.

For the worst case scenario (an initial berm height of 4m AHD), Farquhar Inlet did not break out (i.e. remained completely closed throughout the simulation). The extent of influence on peak flood levels resulting from a completely closed entrance at Farquhar Inlet is presented on Figure 7-7. Compared against the baseline 1% AEP design event, the impact of increased flood levels extends to just downstream of Taree.

For the range of berm scenarios modelled, peak flood levels at the Farquhar Inlet and Croki gauges were extracted. The impact on peak flood levels at the Harrington gauge was largely negligible, as water levels at this location are driven by ocean tides through the open entrance conditions. Impacts are noticed slightly upstream of the Harrington gauge, where the channel width is more constricted. Increases in peak flood level at Croki will be representative of the upper range of impacts likely to be observed along the northern arm of the lower Manning River.

Figure 7-8 presents the relationship between berm height at Farquhar Inlet and the modelled peak flood levels at the Farquhar Inlet and Croki gauge sites. Results are presented for the 20% AEP, 5% AEP and 1% AEP design flood events. As expected, the peak flood level modelled at the gauge locations increases with higher berm scenarios. At the Farquhar Inlet gauge, peak flood levels would be expected to increase by up to 1.5m (from baseline design flood levels) for the 1% AEP event, under a completely closed entrance at Farquhar Inlet.



Title:

Extent of Influence of a Closed Berm at Farquhar Inlet for the 1% AEP Event

Figure: 7-7

Rev: A

N

0 2.5 5km

Approx. Scale

BMT WBM

www.bmtwbm.com.au

Filepath : DWG_010_1%AEP_FI_Closed_Influence.WOR

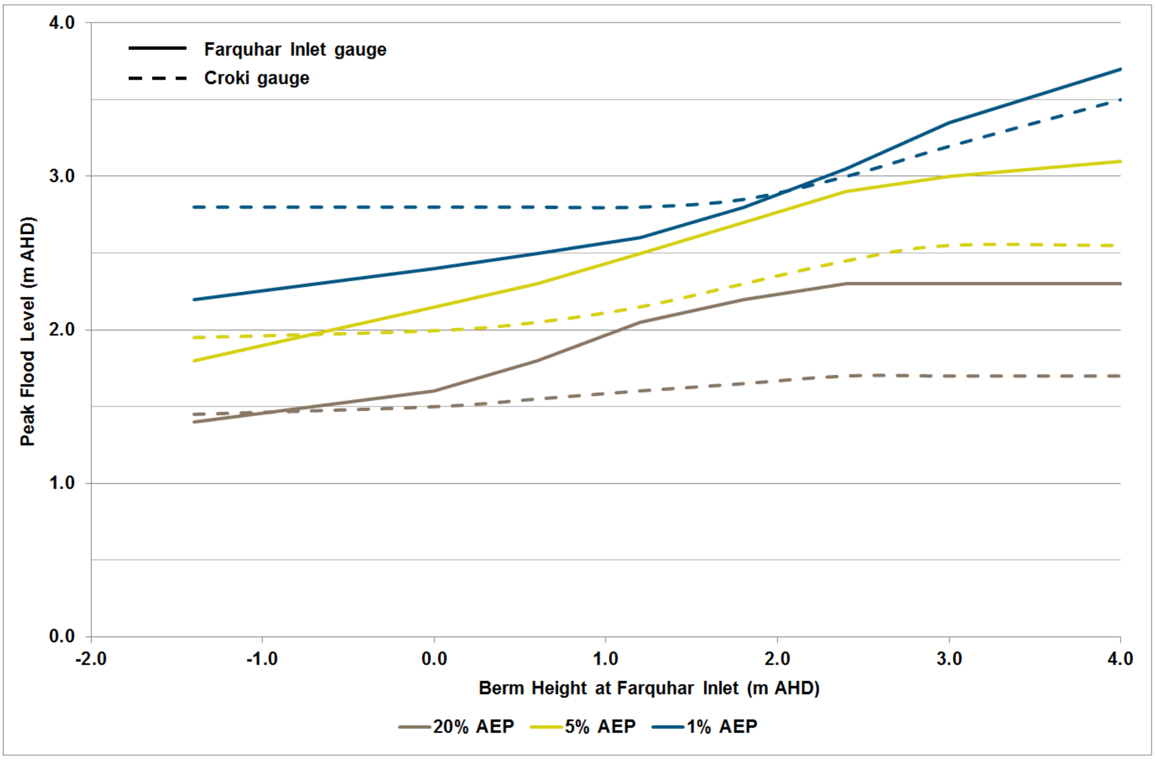


Figure 7-8 Farquhar Inlet Initial Berm Height vs. Modelled Peak Flood Level at Gauge Sites

A completely closed entrance at Farquhar Inlet represents the worst case scenario. It is highly unlikely that the berm would be at this level (~4m AHD) at the onset of a major flood. For context, the typical elevation of closed berm saddles for NSW ICOLLs range from around 2m AHD to 3m AHD.

The current entrance management plan for Farquhar Inlet (WorleyParsons, 2010) involves maintaining the berm at a maximum elevation of 2.5m AHD, with a 50m wide pilot channel at 2m AHD. Under this type of entrance opening regime, the maximum modelled water level at both the Farquhar Inlet and Croki gauges is expected to be just under 3.0m AHD for the 1% AEP event. This corresponds to a 0.7m and 0.1m increase (from baseline 1% AEP design flood conditions, where Farquhar Inlet is open at the onset of the storm) at the Farquhar Inlet and Croki gauges respectively.

The longitudinal profile showing the impact of a completely closed entrance at Farquhar Inlet is shown in Figure 7-9 and Figure 7-10. Peak modelled flood levels are presented in Table 7-3 at the end of this Section.

7.7.4 Entrance Geomorphology

For the purpose of demonstrating the significance of the entrance breakout modelling and its influence on peak flood levels, sensitivity testing included an alternate scenario where the entrance breakout is not modelled, i.e. the adopted initial entrance geometry is constant throughout the simulation.

Just upstream of Harrington, peak flood levels increase by up to 0.8m from the baseline 1% AEP condition, as a result of a more closed entrance scenario. The resulting impact is more noticeable at Farquhar inlet, where peak flood levels increase by almost 1m.

The longitudinal profile showing the impact of omitting a dynamic entrance breakout is shown in Figure 7-9 and Figure 7-10. Peak modelled flood levels at various locations within the study area are presented in Table 7-3 at the end of this Section.

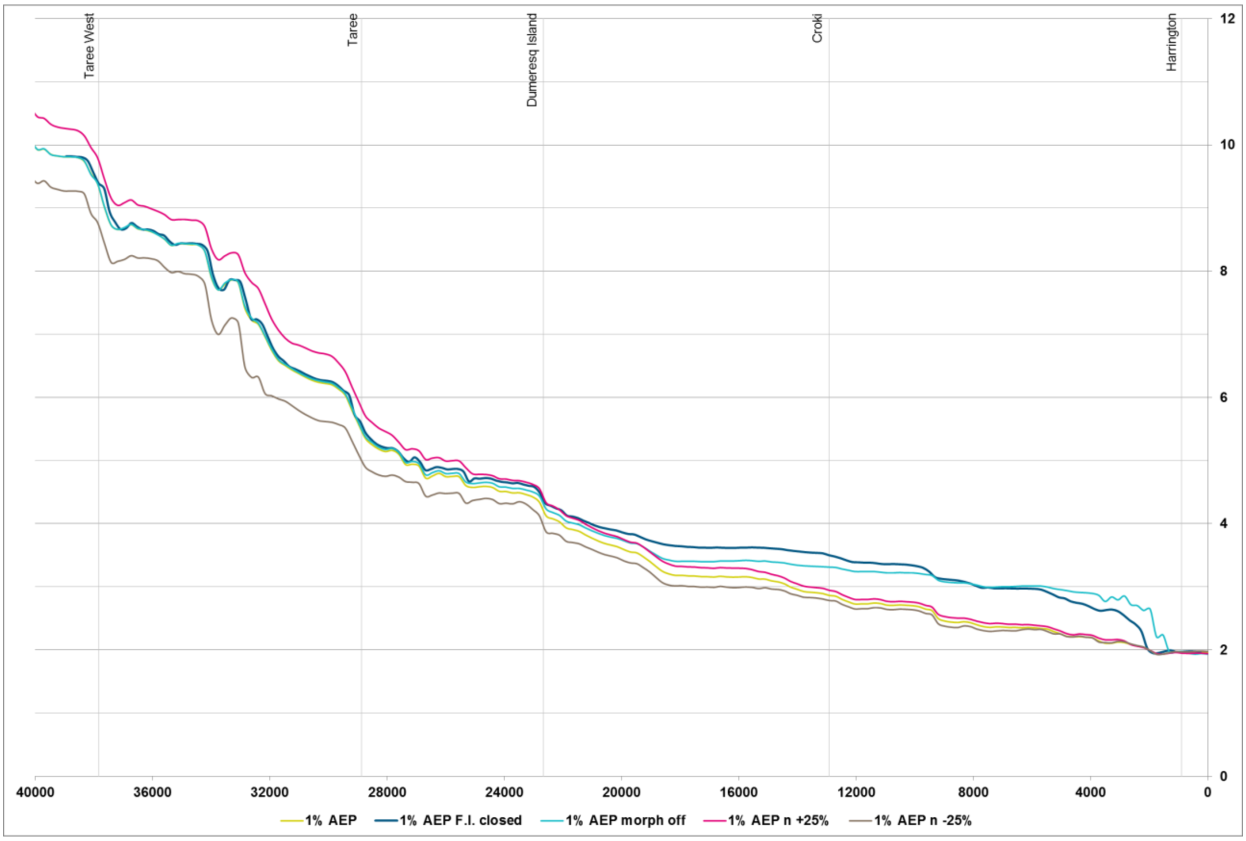


Figure 7-9 Manning River Peak Flood Level Sensitivity, Main Alignment from Harrington

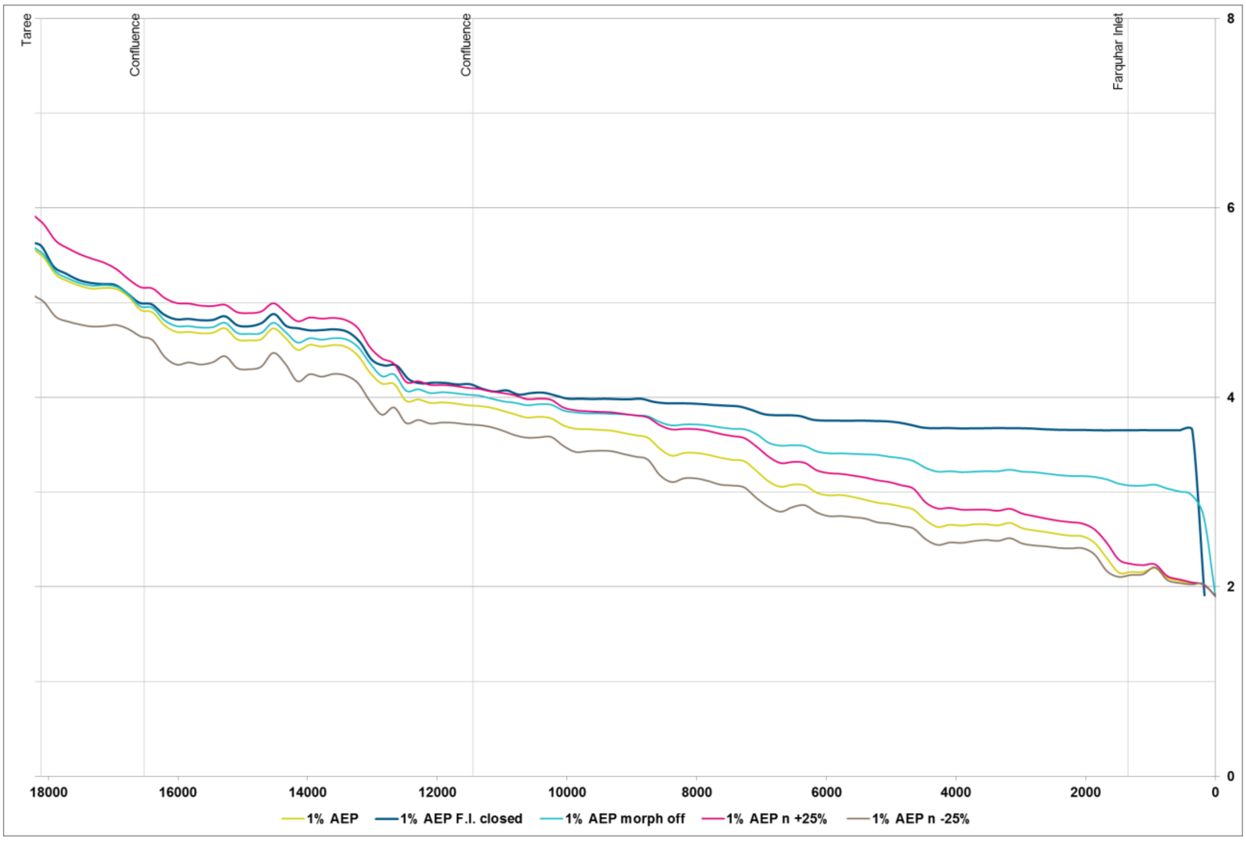


Figure 7-10 Manning River Peak Flood Level Sensitivity, South Alignment from Farquhar Inlet to Taree

Table 7-3 Summary of Model Sensitivity Results

ID	Location	Modelled Peak Flood Level (m AHD) – Sensitivity to Adopted 1% AEP Design Condition												
		Adopted Design	Farquhar Inlet Entrance Closed	No Entrance Breakout	+25% 'n'	-25% 'n'	2050 SLR	2100 SLR	+10% Flow	+30% Flow	+10% Flow w/ 2050 SLR	+30% Flow w/ 2050 SLR	+10% Flow w/ 2100 SLR	+30% Flow w/ 2100 SLR
A	Harrington	2.0	2.0	2.0	2.0	2.0	2.2	2.9	2.0	2.0	2.2	2.9	2.3	2.9
B	Farquhar Inlet	2.2	3.7	3.1	2.3	2.2	2.3	2.9	2.3	2.4	2.4	2.9	2.5	3.0
C	Croki	2.8	3.5	3.3	2.9	2.7	2.8	3.2	3.0	3.4	3.1	3.4	3.5	3.7
D	Dumaresq Island	4.1	4.4	4.2	4.4	3.9	4.2	4.3	4.5	4.9	4.5	4.6	4.9	4.9
E	Taree, Macquarie St	5.2	5.3	5.3	5.5	4.8	5.2	5.3	5.6	6.0	5.6	5.6	6.0	6.0
F	Taree, Martin Bridge	5.5	5.6	5.5	5.8	5.0	5.5	5.6	5.8	6.3	5.8	5.8	6.3	6.3
G	Taree West	9.5	9.5	9.5	9.9	8.9	9.5	9.5	10.0	10.7	10.0	10.0	10.7	10.7

7.8 Comparison with Previous Studies

7.8.1 Design Flood Levels

Peak design flood levels modelled at Taree are presented in Table 7-4 against the peak flood levels determined for the previous Flood Study (Public Works, 1991). Levels have been rounded to the nearest 100mm.

Table 7-4 Comparison of Design Peak Flood Levels at Taree with Previous Study

Design Event	Peak Flood Level (m AHD)			
	This Study		Public Works (1991)	
	Macquarie St	Martin Bridge	Macquarie St	Martin Bridge
20% AEP	2.7	2.9	-	-
5% AEP	4.1	4.4	4.4	4.8
2% AEP	4.8	5.1	5.1	5.5
1% AEP	5.2	5.5	5.5	5.8
0.5% AEP	5.5	5.8	5.9	6.3
0.2% AEP	6.0	6.3	-	-
PMF	8.6	9.4	9.6	10.2

This current study determined design peak flood levels in the Lower Manning that are typically around 0.2m to 0.5m lower than that of the previous study. This can largely be attributed to the difference in design flow estimation between the two studies, as detailed in Section 7.8.2.

The Lansdowne Flood Study Review, Upgrade and Extension (WorleyParsons, 2014) utilised water levels at Manning River and Lansdowne River confluence determined in the original Flood Study (Public Works, 1991) as the downstream boundary for design flood events. Peak design flood levels modelled on the Lansdowne River just upstream of the Pacific Highway are presented in Table 7-5 against the peak flood levels determined for the Lansdowne Flood Study. Levels have been rounded to the nearest 100mm.

Table 7-5 Comparison of Design Peak Flood Levels at Lansdowne River (upstream of Pacific Hwy) with Previous Study (WorleyParsons, 2014)

Design Event	Peak Flood Level (m AHD)	
	This Study	WorleyParsons (2014)
20% AEP	1.4	1.4
5% AEP	1.9	2.4

Design Event	Peak Flood Level (m AHD)	
2% AEP	2.5	2.6
1% AEP	2.8	2.9
0.5% AEP	3.0	3.1
0.2% AEP	3.4	-
PMF	5.4	6.0

It can be seen that peak flood levels derived in this current study are similar to those determined in the Lansdowne Flood Study and are typically within 0.2m.

7.8.2 Design Flood Flows

Table 7-4 presents the design peak flow rates calculated by the Flood Frequency Analysis at Killawarra from this study and the previous Flood Study (Public Works, 1991).

This study consistently calculated peak flow rates at Killawarra that were lower than those calculated as part of the previous study. The 20% AEP and 0.5% AEP are around 20% and 30% lower, respectively, compared to the original study. This discrepancy can in part be attributed to an additional 14 years of data and a more robust statistical analysis used as part of the current Flood Frequency Analysis. In addition, for the 2011 calibration event there were a number of key water level time series where a good match between recorded and modelled levels was achieved. This provided confidence in the calibration and the adopted hydraulic model parameters, and therefore confidence in the rating curve adopted for the Killawarra gauge site.

Table 7-6 Comparison of Design Peak Flows at Killawarra with Previous Study

Design Event	Peak Flow Rate (m^3/s)	
	This Study	Public Works (1991)
20% AEP	4,100	-
5% AEP	6,500	7,900
2% AEP	8,100	10,300
1% AEP	9,200	12,000
0.5% AEP	10,400	14,500
0.2% AEP	11,900	-
PMF	27,600	45,000

8 Conclusions

The objective of the study was to undertake a detailed flood study of the Manning River catchment downstream of Wingham and to establish models as necessary for design flood level prediction.

In completing the flood study, the following activities were undertaken:

- Collation of historical and recent flood information for the study area;
- Development of computer models to simulate hydrology and flood behaviour in the catchment;
- Calibration of the developed models using the available flood data, including the recent events of 2011, 2013 and 1990 and the historic events of 1929, 1956 and 1978; and
- Prediction of design flood conditions in the catchment and production of design flood mapping series.

The study provides updated and more detailed flooding information than the previous Manning River Flood Study (Public Works, 1991) to be used to inform floodplain risk management within the study area. One aspect of floodplain risk management is flood warning and emergency response.

The State Emergency Service (SES) has formal responsibility for emergency management operations in response to flooding. Other organisations normally provide assistance, including the Bureau of Meteorology, Council, police, fire brigade, ambulance and community groups. Emergency management operations are usually outlined in a Local Flood Plan.

SES actions during the event of a flood in the lower Manning River floodplain are guided by Flood Intelligence Cards. Within the lower Manning River catchment, cards exist for Wingham, Taree, Croki, Harrington and Lansdowne. These contain information on key flood heights at river gauges, flooding consequences and required actions. Details contained within this study report and design flood mapping will provide useful information with which to update the Flood Intelligence Cards – particularly for Taree, Croki and Harrington.

Flood classifications in the form of locally-defined flood levels are used in flood warnings to give an indication of the severity of flooding (minor, moderate or major) expected. The SES classifies major, moderate and minor flooding according to the gauge height values at the Martin Bridge, Taree.

The flood classification levels are described by:

- **Minor:** flooding which causes inconvenience such as closing of minor roads and the submergence of low-level bridges. The lower limit of this class of flooding, on the reference gauge, is the initial flood level at which landholders and/or townspeople begin to be affected in a significant manner that necessitates the issuing of a public flood warning by the BoM.
- **Moderate:** flooding which inundates low-lying areas, requiring removal of stock and/or evacuation of some houses. Main traffic routes may be flooded.
- **Major:** flooding which causes inundation of extensive rural areas, with properties, villages and towns isolated and/or appreciable urban areas flooded.

The current flood warning trigger levels at the Martin Bridge, Taree, are presented in Table 8-1 against design flood levels and historic flood levels for context.

Table 8-1 Flood Warning Levels, Design Flood Levels and Historic Flood Levels at Taree (Martin Bridge)

Flood Classification	Peak Flood Level (m AHD)
Minor Flood Warning	*1.8*
Moderate Flood Warning	*2.4*
20% AEP	2.9
2013	**3.37**
Major Flood Warning	*3.7*
1990	**4.37**
5% AEP	4.4
2011	**4.5**
2% AEP	5.1
1% AEP	5.5
1978	**5.75**
0.5% AEP	5.8
1929	**5.9**
0.2% AEP	6.3
PMF	9.4

A number of major access roads are subject to flood inundation and have the potential isolate individual residents or townships during flood events. These include Tinonee Road, The Bucketts Way, Manning River Drive, Old Bar Road, Harrington Road, Crowdy Street, Manning Point Road Rd (at various locations on Mitchells Island, Oxley Island and toward Old Bar Rd) and the Pacific Highway. Modelled design flood conditions at these locations have been summarised in Table 8-2.

Table 8-2 Summary of Major Access Road Inundation (d = Peak Flood Depth, V = Peak Flood Velocity)

Flood Event	Tinonee Rd		The Bucketts Way		Manning River Dr		Old Bar Rd		Harrington Rd		Crowdy St		Manning Point Rd (Old Bar)		Manning Point Rd (Mtchells Island)		Manning Point Rd (Oxley Island)		Pacific Hwy	
	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)	d (m)	V (m/s)
20% AEP	0.7	0.7	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
5% AEP	3.0	0.9	1.4	0.8	0.3	0.1	-	-	0.7	1.0	-	-	0.1	0.3	0.3	0.5	-	-	-	-
2% AEP	4.0	1.0	2.5	1.0	0.9	0.2	-	-	1.2	0.9	-	-	0.4	0.9	1.1	0.8	0.5	1.0	0.6	0.7
1% AEP	4.5	1.0	3.1	1.0	1.3	0.2	-	-	1.4	0.9	-	-	0.6	0.8	1.4	0.8	0.6	1.1	0.8	0.8
0.5% AEP	5.0	1.0	3.5	1.0	1.7	0.3	0.3	0.7	1.7	0.9	-	-	0.9	0.7	1.7	0.8	0.8	1.3	1.1	0.8
0.2% AEP	5.5	1.0	4	1.1	2.3	0.7	0.6	0.9	2.0	0.9	-	-	1.3	0.6	2.0	1.0	1.2	1.3	1.5	0.8
PMF	8.9	1.0	7.8	1.3	5.6	2.6	2.8	1.0	4.0	1.1	2.6	0.6	3.6	0.9	4.3	1.3	3.2	1.3	3.5	1.0

9 References

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2011 Flood Photos, http://www.abc.net.au/local/photos/2011/06/17/3246424.htm

1990 Flood Photos, http://www.manningrivertimes.com.au/story/2194760/throwback-thursday-1990-taree-flood

Appendix A Community Information Brochure and Questionnaire

Manning River Flood Survey

We have commissioned a new flood study for the Manning River catchment, as many things have changed since our last study was conducted in the mid 1990's.

BMT WBM, an independent company specialising in flooding and floodplain risk management, will undertake the study.

The flood study is the first step in assisting us to better understand, plan and manage the risk of flooding across the catchment.

The information that you provide in this survey will prove invaluable in developing flood modelling for the Manning River catchment. It will also provide us with an understanding of existing flooding problems and areas where measures to reduce flood damage should be investigated in the future.

The specific major floods that we would like information about are March 1978, February 1990 and June 2011.

The following survey should only take around 10 minutes to complete. If you would prefer, we also have a copy of this survey on our website.

1. **How long have you lived and/or worked in the area?**

Years ☐

Months ☐

2. **Have you or someone you know been affected by flooding in the past or do you have any information about flooding in the Manning Catchment?**

☐ Yes

☐ No

3. **Were you or someone you know affected by, or have any information about any of the below floods? If you have been affected by more than one flood, please complete questions 3-7 again on an additional survey.**

☐ March 1978 ☐ February 1990 ☐ June 2011

4. Where did this flooding occur? (street address if possible)

5. How were you or someone you know affected?

- [ ] Traffic was disrupted
- [ ] Front/back yard was flooded
- [ ] House/business and its contents were flooded
- [ ] Sewer or water was turned off at my property
- [ ] Other

Please provide a detailed description of how you were affected

6. Can you provide details of how high floodwaters reached?

- [ ] Yes
- [ ] No

If yes, please give as much detail as possible (location, dates, times, description of water movement, depth of water, flood mark location, high water mark on building, and level of flood depth indicator).

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7. **What do you think may have been the main source/cause of the flooding?**

☐ Creek/river banks overtopping

☐ Blockage of bridges

☐ Blockage of drains

☐ Other (please specify)

8. **Are you concerned that your property could be flooded in the future?**

☐ Yes

☐ No

If yes, why are you concerned?

9. **Do you have any other comments or information that you think would be useful for this investigation?**

10. Can you please provide contact details in case we need to contact you for additional information? This information is optional, will remain confidential and will not be published unless you give us permission to do so.

Name:	
Address:	
Town:	
Postcode:	
Email:	
Phone:	

Thank you for taking the time to complete this survey!

If you have any questions, additional information or flood photos/videos please contact:

Roshan Khadka

Greater Taree City Council

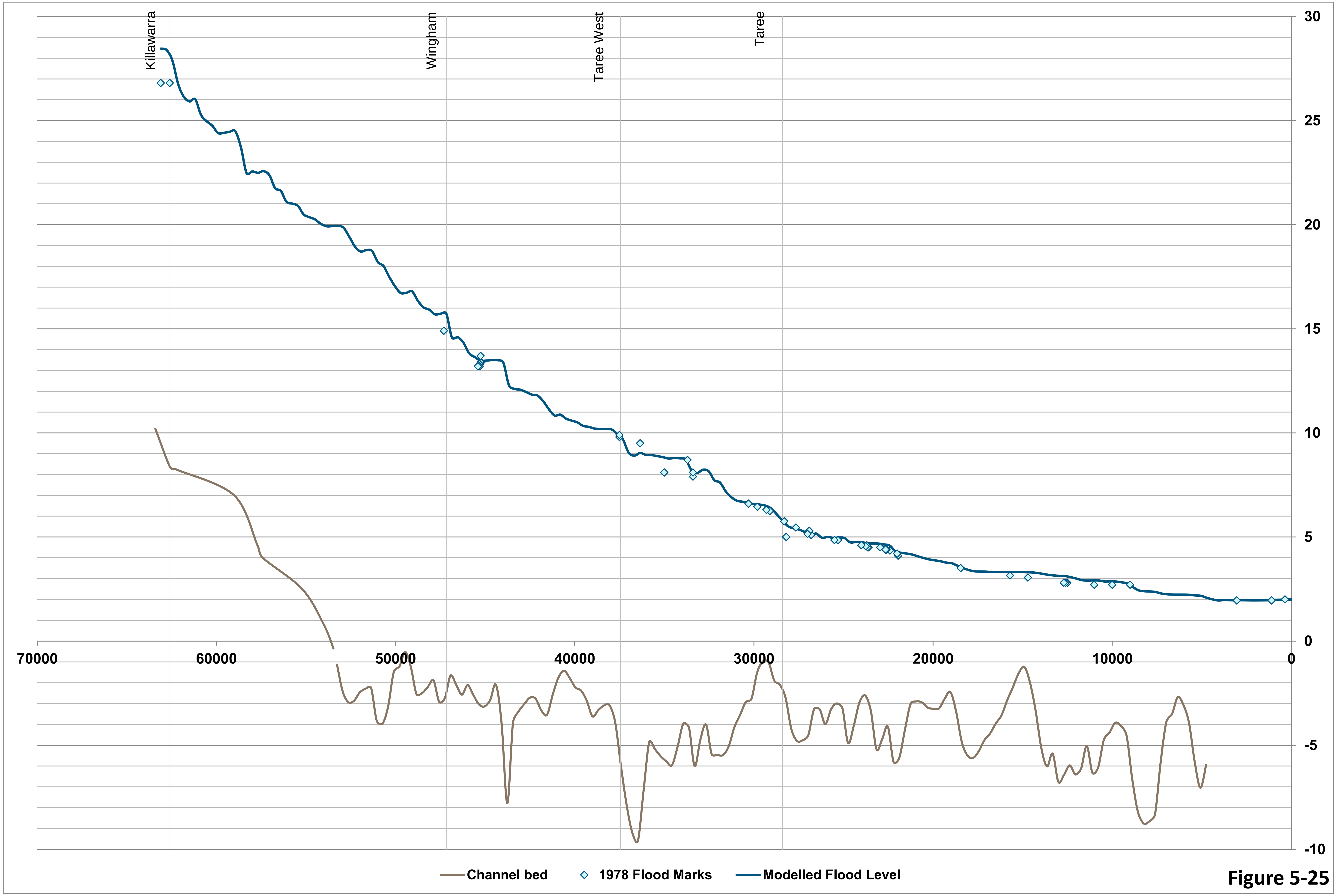
2 Pulteney Street, Taree NSW 2430

Phone: 6592 5399

Email: tareecouncil@gtcc.nsw.gov.au

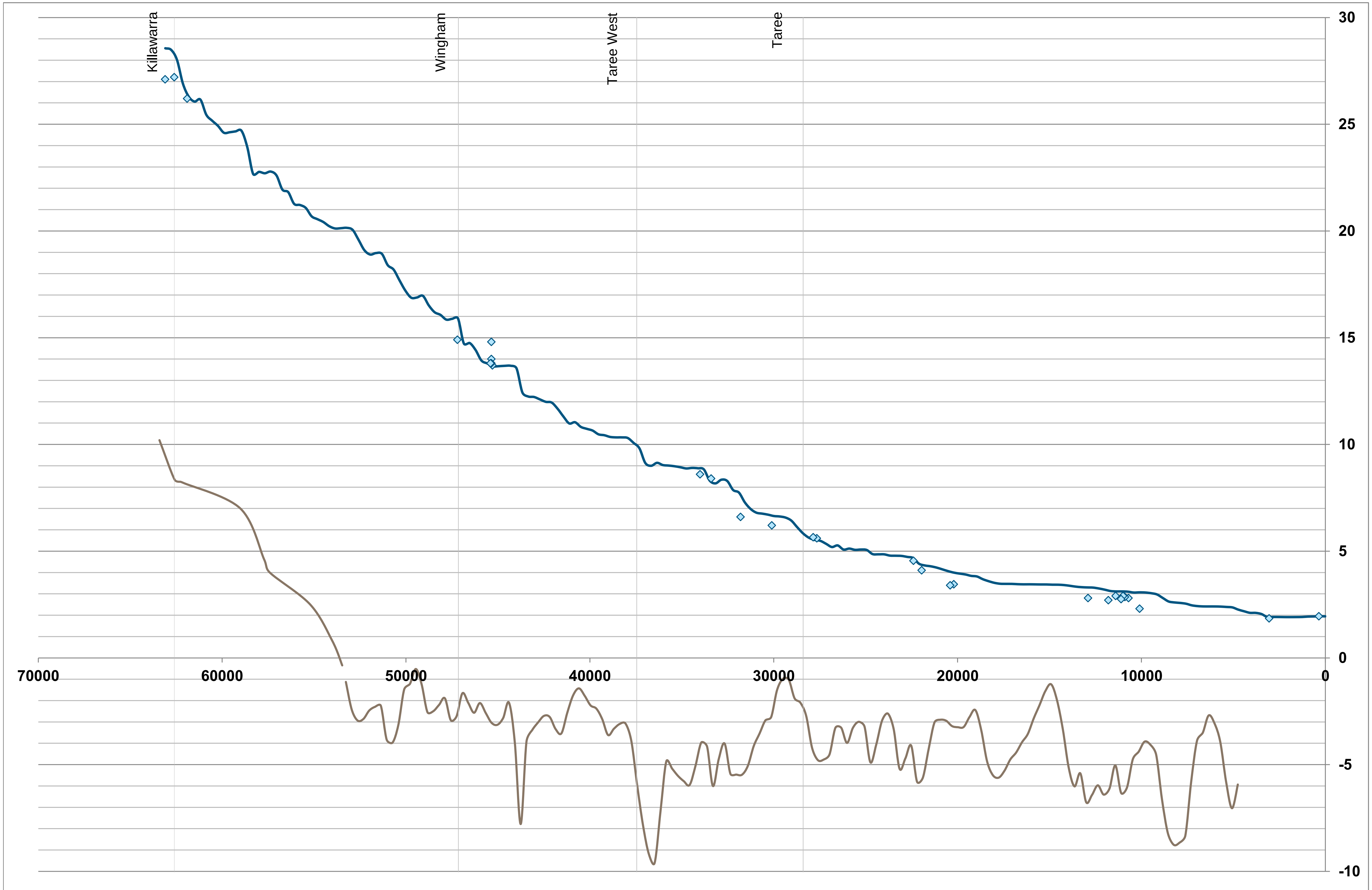
Website: www.gtcc.nsw.gov.au

Appendix B Long Sections



Killawarra
Wingham
Taree West
Taree
30
25
20
15
10
5
0
-5
-10
70000
60000
50000
40000
30000
20000
10000
0
Channel bed
1978 Flood Marks
Modelled Flood Level

Figure 5-25



Killawarra
Wingham
Taree West
Taree
30
25
20
15
10
5
0
-5
-10
70000
60000
50000
40000
30000
20000
10000
0
Channel bed
1929 Flood Marks
Modelled Flood Level

Figure 5-35

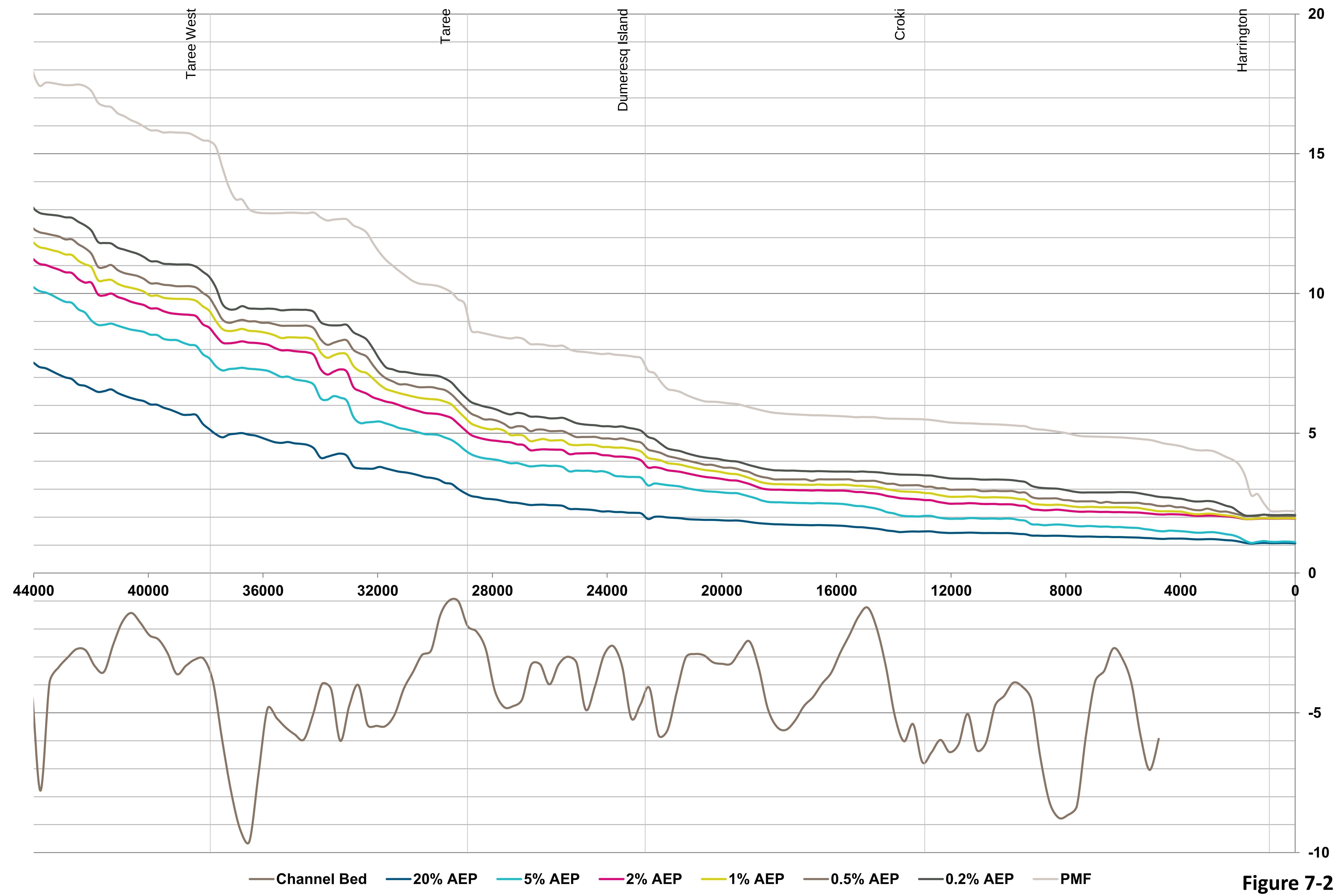
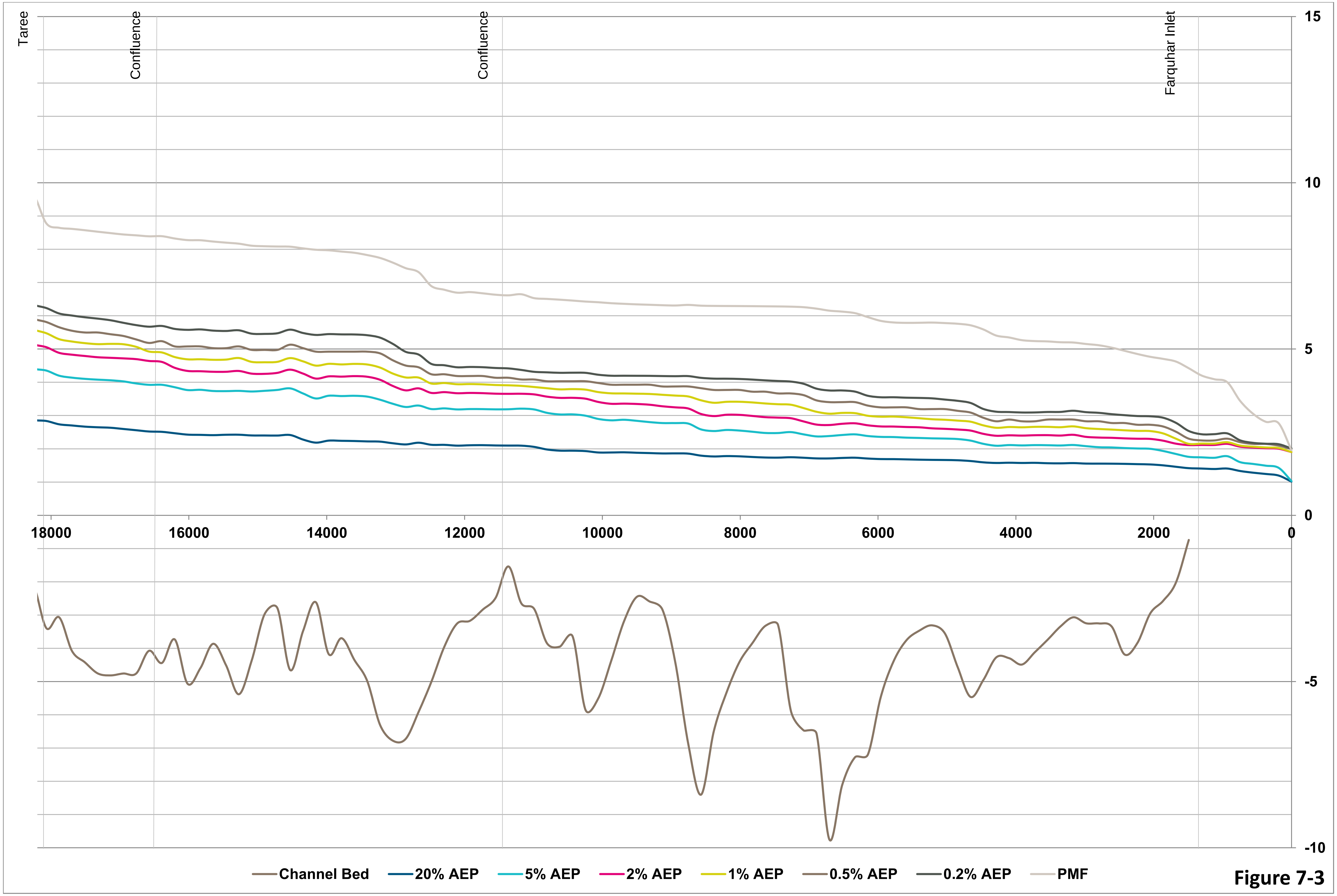
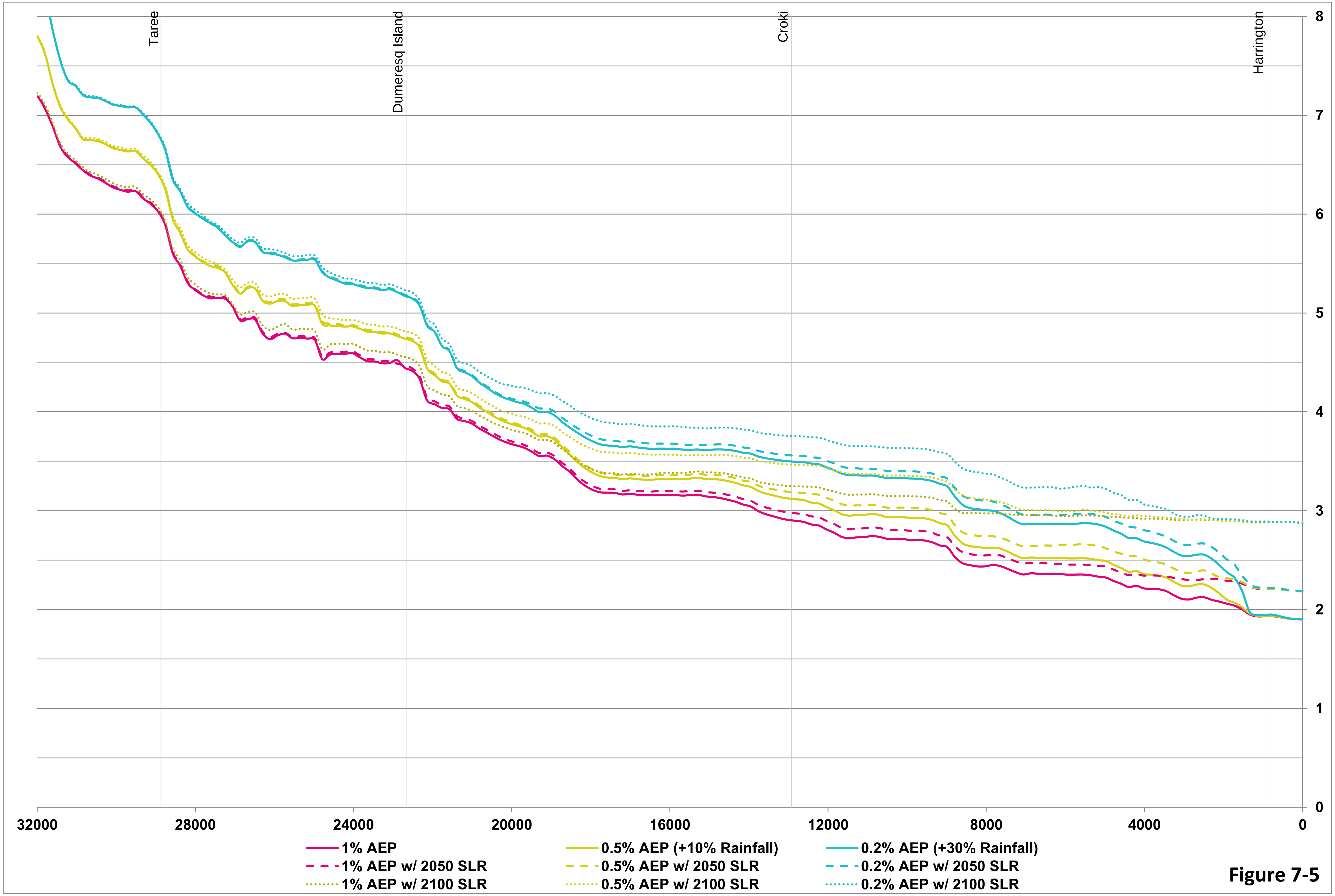

Taree West
Taree
Dumeresq Island
Croki
Harrington
20
15
10
5
0
-5
-10
44000
40000
36000
32000
28000
24000
20000
16000
12000
8000
4000
0
Channel Bed
20% AEP
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF

Figure 7-2



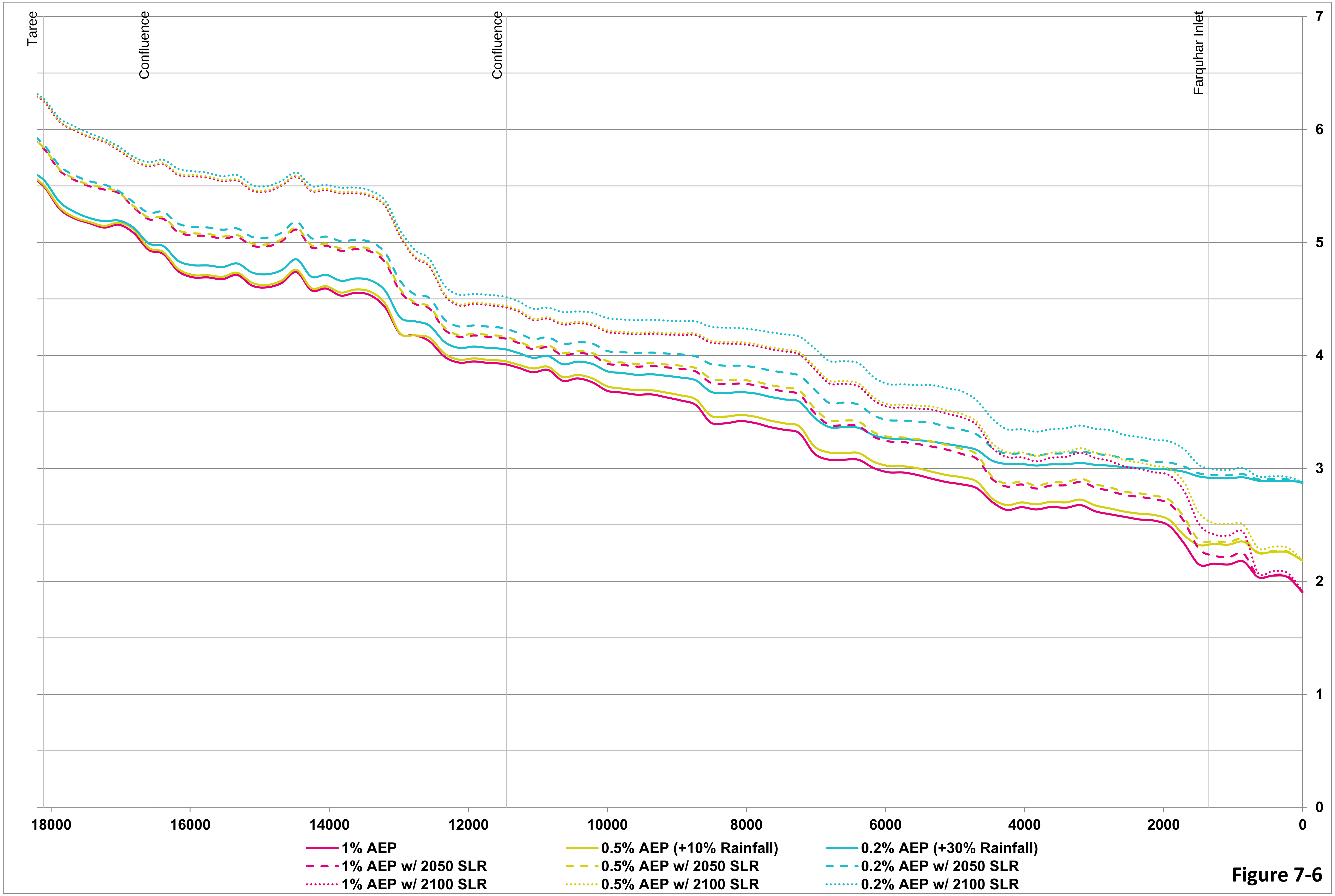
Taree
Confluence
Confluence
Farquhar Inlet
15
10
5
0
-5
-10
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
Channel Bed
20% AEP
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF

Figure 7-3



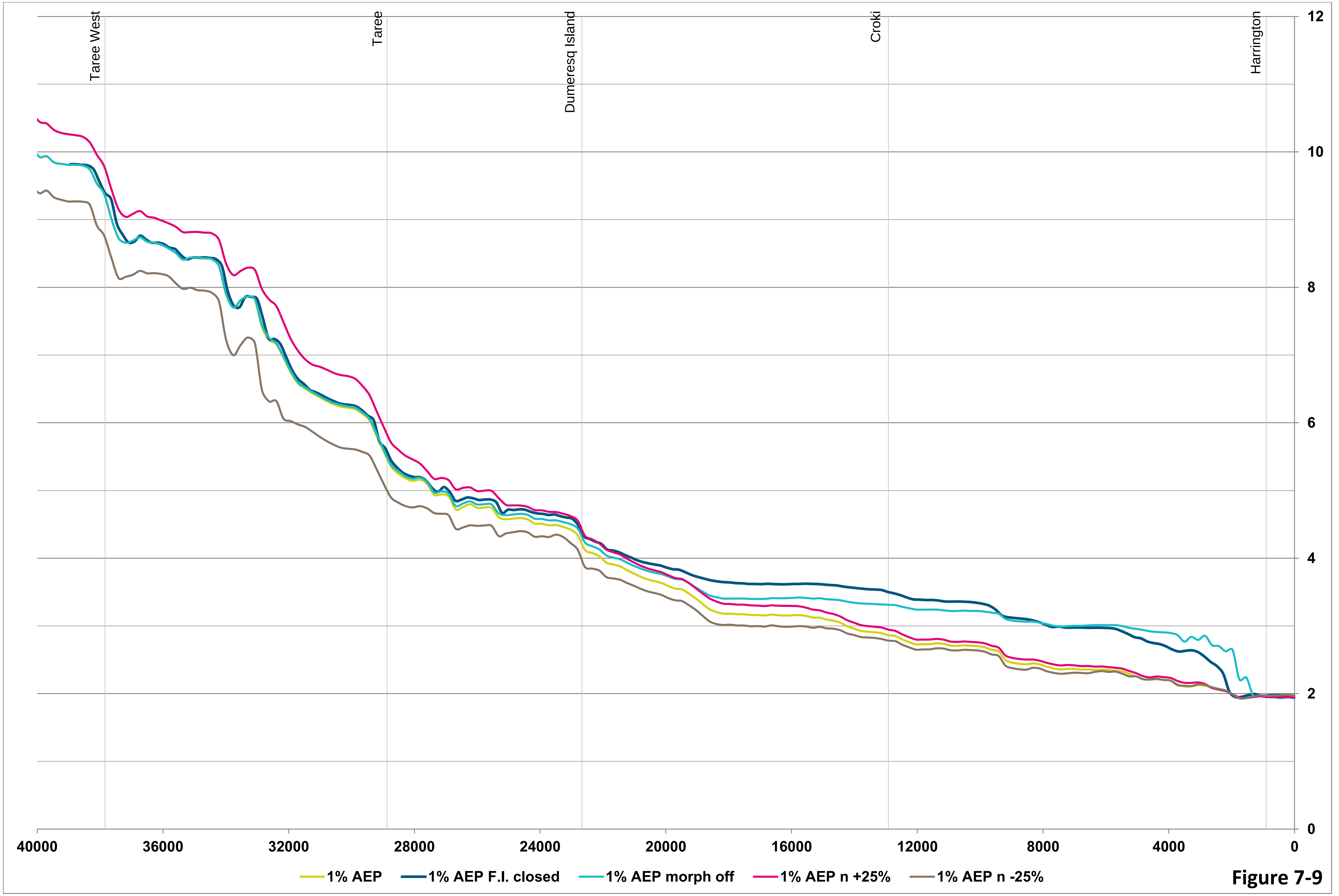
Taree
Dumeresq Island
Croki
Harrington
8
7
6
5
4
3
2
1
0
32000
28000
24000
20000
16000
12000
8000
4000
0
1% AEP
1% AEP w/ 2050 SLR
1% AEP w/ 2100 SLR
0.5% AEP (+10% Rainfall)
0.5% AEP w/ 2050 SLR
0.5% AEP w/ 2100 SLR
0.2% AEP (+30% Rainfall)
0.2% AEP w/ 2050 SLR
0.2% AEP w/ 2100 SLR

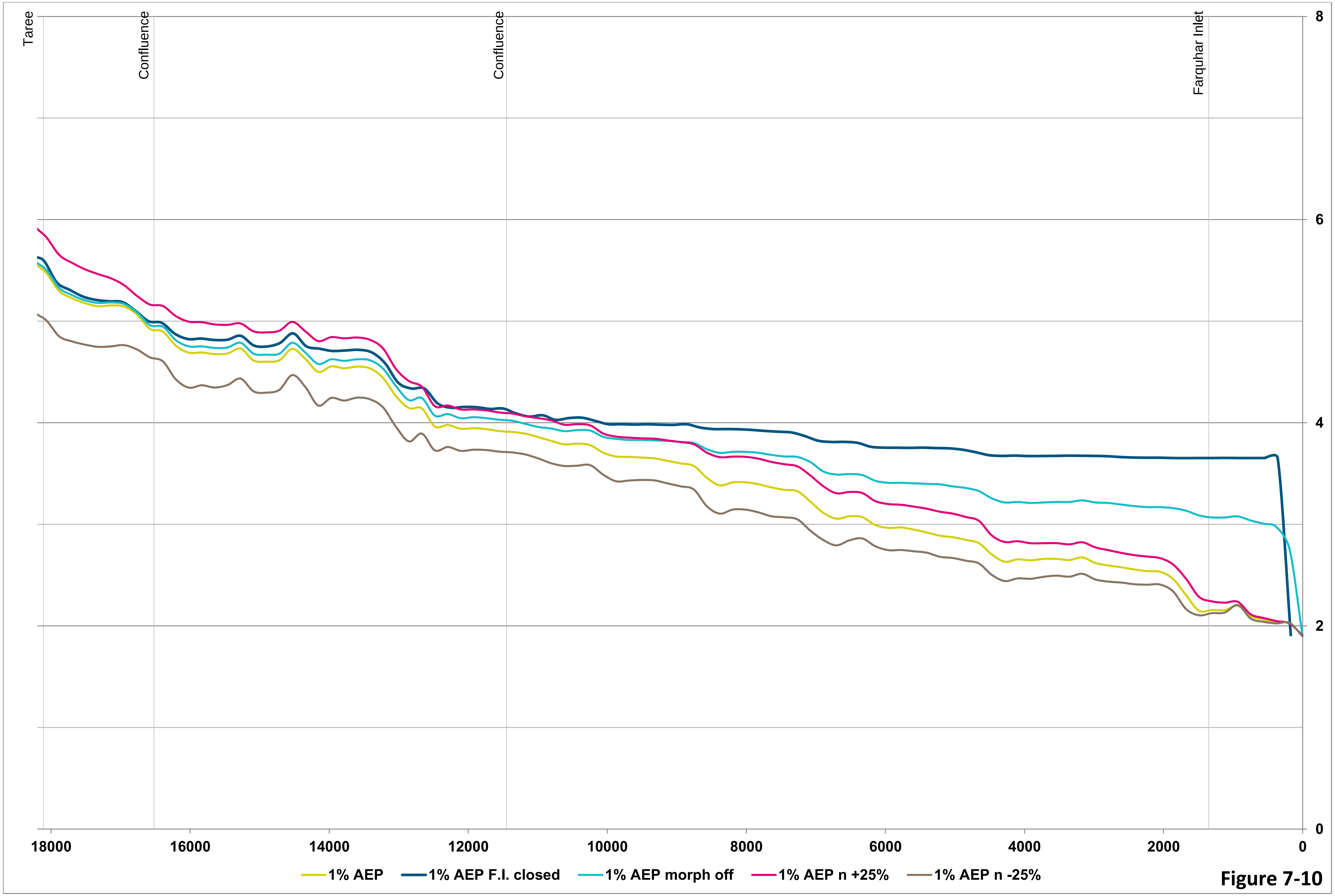
Figure 7-5



Taree
Confluence
Confluence
Farquhar Inlet
7
6
5
4
3
2
1
0
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
1% AEP
1% AEP w/ 2050 SLR
1% AEP w/ 2100 SLR
0.5% AEP (+10% Rainfall)
0.5% AEP w/ 2050 SLR
0.5% AEP w/ 2100 SLR
0.2% AEP (+30% Rainfall)
0.2% AEP w/ 2050 SLR
0.2% AEP w/ 2100 SLR

Figure 7-6


Taree West
Taree
Dumeresq Island
Croki
Harrington
12
10
8
6
4
2
0
40000
36000
32000
28000
24000
20000
16000
12000
8000
4000
0
1% AEP
1% AEP F.I. closed
1% AEP morph off
1% AEP n +25%
1% AEP n -25%
Figure 7-9



Taree
Confluence
Confluence
Farquhar Inlet
8
6
4
2
0
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
1% AEP
1% AEP F.I. closed
1% AEP morph off
1% AEP n +25%
1% AEP n -25%

Figure 7-10

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