“Where will our knowledge take you?”

Coffs Creek and Park Beach Flood Study

Final Report

May 2018

COFFS HARBOUR CITY COUNCIL

Coffs Creek and Park Beach Flood Study

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

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Executive Summary

Introduction

The Coffs Creek and Park Beach Flood Study has been prepared for Coffs Harbour City Council (Council) to define the existing flood behaviour in the catchment and establish the basis for subsequent floodplain management activities. Review of previously defined flood behaviour was required due construction of recent flood mitigation works, including multiple detention basins, across the catchment.

The primary objective of the Flood Study is to define the flood behaviour within the Coffs Creek 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 and acquisition of additional data including survey as required;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design event including the 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.

Catchment Description

The Coffs Creek catchment has a relatively small area of around 25km^2 and is located on the eastern Australian coast. The flat coastal floodplain rises steeply to an escarpment in the west. Elevations rapidly increase from below 10m AHD to more than 400m AHD over just a few kilometres. The Coffs Creek estuary forms the downstream limit of the catchment.

The catchment is bound to the north by densely vegetated ranges of state forest and national parkland. Much of the low lying floodplain area is urban development, consisting of residential, commercial and industrial properties. The upper catchment is primarily used for agriculture and horticulture purposes.

Coffs Creek consists of many branching streamlines and can be divided into three sections; Coffs Creek, including the main arm and minor tributaries to the north west; the Northern Tributaries of Coffs Creek, running adjacent to Bray Street and Argyll Street; and the area located east of the railway line, draining the low-lying areas of Park Beach.

The topography of the Coffs Creek catchment is conducive to extreme weather events. During the formation of a low pressure system off the coast known as an east coast low (ECL), the steep terrain located very close to the coastline is exposed. In the presence of strong onshore wind, moisture filled air masses are pushed towards the hills, where they rapidly rise facilitating intense rainfall over the upper catchment. The phenomenon of increased rainfall across the upper catchment was found to be consistent across a number of historic rainfall events.

The Coffs Creek catchment is prone to severe flash flooding as it is a relatively small catchment with steep upper slopes, a high level of urban development on the floodplain and the tendency for high rainfall.

Following from recommendations in the Floodplain Risk Management Study and Plan (Bewsher Consulting, 2005), multiple detention basins have been constructed in the catchment in recent years. These include the Bakers Road basin located upstream of William Sharp Drive (constructed 2010), the Bennetts Road basin (constructed 2012-2013) and the Spagnolos Road (constructed 2015).

Historical Flooding

A number of floods are known to have occurred in Coffs Harbour since the late 1800s. However, detailed information surrounding events prior to the 1970s is scarce. Newspaper clippings indicate that significant flood events were experienced in November 1917 and February 1938. Since rainfall records commenced, floods are known to have occurred in June 1950, April 1962 and April 1963. The April 1963 event was the largest of these.

More information is available for floods experienced in the latter part of the 20th century. This includes photographs, flood levels and other evidence relating to the number of properties inundated by floodwaters. Large flood events occurred in March 1974 and May 1977 and a smaller flood occurred in April 1989.

In recent years, extreme floods occurred in 1991, 1996 and 2009. The floods of November 1996 and March 2009 are the largest on record in Coffs Harbour were of similar magnitude. The rainfall gradient phenomenon (“orographic rainfall”) was observed across the catchment for both the 1996 and 2009 events, with rainfall recorded over the upper catchment equivalent to design rainfall estimates rarer than the 0.2% AEP. During the 1996 event, recorded flood levels were up to 1.0m higher than previously defined 1% AEP design flood levels.

Serious flooding has also occurred within Park Beach in recent years, resulting from heavy, intense rainfall over the lower catchment in November 2009 and February 2015.

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 Coffs Creek catchment 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.

The floodplain topography is defined using a digital elevation model (DEM) derived from topographic, hydrographic and topographic survey data provided by Council. To supplement the available data, additional channel cross section survey of the Argyll Street branch of the Northern Tributaries of Coffs Creek was acquired during the course of the study.

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.

In recent years, both the March 2009 and November 1996 events were major flood events in the Coffs Creek catchment. The 2009 event has been selected as the principle calibration event for the model for the following reasons:

- More comprehensive coverage of rainfall records during the event;
- Catchment topography during 2009 will be closer to 2013 LiDAR data given that extensive development within the catchment has occurred since 1996;
- Better coverage of surveyed flood marks within Park Beach; and
- Official MHL stream gauge recorded the entire event.

Due to the uncertainty surrounding catchment topography as a result of development between the 1996 and 2009 events, the November 1996 event will be used to validate the model.

In March 2015, Park Beach and areas along the Northern Tributaries of Coffs Creek were flood affected due to localised heavy rainfall. This event was therefore used to validate the models performance in Park Beach.

Design Event Modelling and Output

The developed models have been applied to derive design flood conditions within the Coffs Creek catchment. In order to account for the rainfall gradient observed across the catchment in extreme flood events, scaling factors have been applied to design rainfall estimates which were calculated in accordance with the procedures Australian Rainfall and Runoff (IEAust, 2001). A range of storm durations using standard AR&R (2001) temporal patterns, were modelled in order to identify the critical storm duration for design event flooding in the catchment.

The impact of the recently constructed detention basins on design flood levels and the potential benefit of construction of a fourth detention basin at Upper Shephards Lane were assessed. The performance of the existing levees within the catchment was also reviewed.

A range of design flood conditions were modelled. The simulated design events included the 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF event. The model results for the design events considered have been presented in a detailed flood mapping series for the catchment (see Mapping Compendium). The flood data presented includes design flood inundation, peak flood water levels and depths and peak flood velocities.

Hydraulic categories (floodway, flood fringe and flood storage) and provisional flood hazard categories (in accordance with Figure L2 of the NSW Floodplain Development Manual (2005)) have been mapped for flood affected areas within the catchment. True hazard categories, as defined in the Coffs Creek Floodplain Risk Management Study (Bewsher Consulting, 2005), have also been mapped.

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;
- Structure and stormwater pipe blockages;
- Changes in the adopted roughness parameters; and
- Alternate design rainfall gradient scaling factors.

Conclusions

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

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

- Compilation and review of existing information pertinent to the study and acquisition of additional data including survey;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Calibration of the developed models using the available flood data, including the recent events of 1996, 2009 and 2015; and
- 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 different design flood conditions within the catchment, particularly peak flood levels and inundation extents. This is largely due to construction of detention basins within the catchment, but also due to:

- Changing from a 1D to almost entirely 2D model representation; and
- Revising the design rainfall scaling factors and lowering the sea level boundary in accordance with OEH guidelines (2015).

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1 Introduction

The Coffs Creek and Park Beach Flood Study has been prepared for Coffs Harbour City Council (Council) to define the existing flood behaviour in the catchment and establish the basis for subsequent floodplain management activities. Review of previously defined flood behaviour was required due construction of recent flood mitigation works, including multiple detention basins, across the catchment.

1.1 Study Location

Coffs Harbour is situated on the NSW mid-north coast around 430km north of Sydney and 320km south of Brisbane. Typical of catchments in the region, the Coffs Creek catchment is bounded by mountainous ranges to the west with a narrow floodplain area extends some 70km along the coast. The Coffs Creek catchment drains an area of around 25km^2 into the Tasman Sea.

Most of the City is located within the catchment of Coffs Creek as shown in Figure 1-1. Coffs Creek consists of many branching streamlines and can be divided into three sections; Coffs Creek, including the main arm and minor tributaries to the north west; the Northern Tributaries of Coffs Creek, running adjacent to Bray Street and Argyll Street; and the area located east of the railway line, draining the low-lying areas of Park Beach. The catchment consists of mixed land uses including land cleared for agriculture, areas of remnant vegetation and urban development.

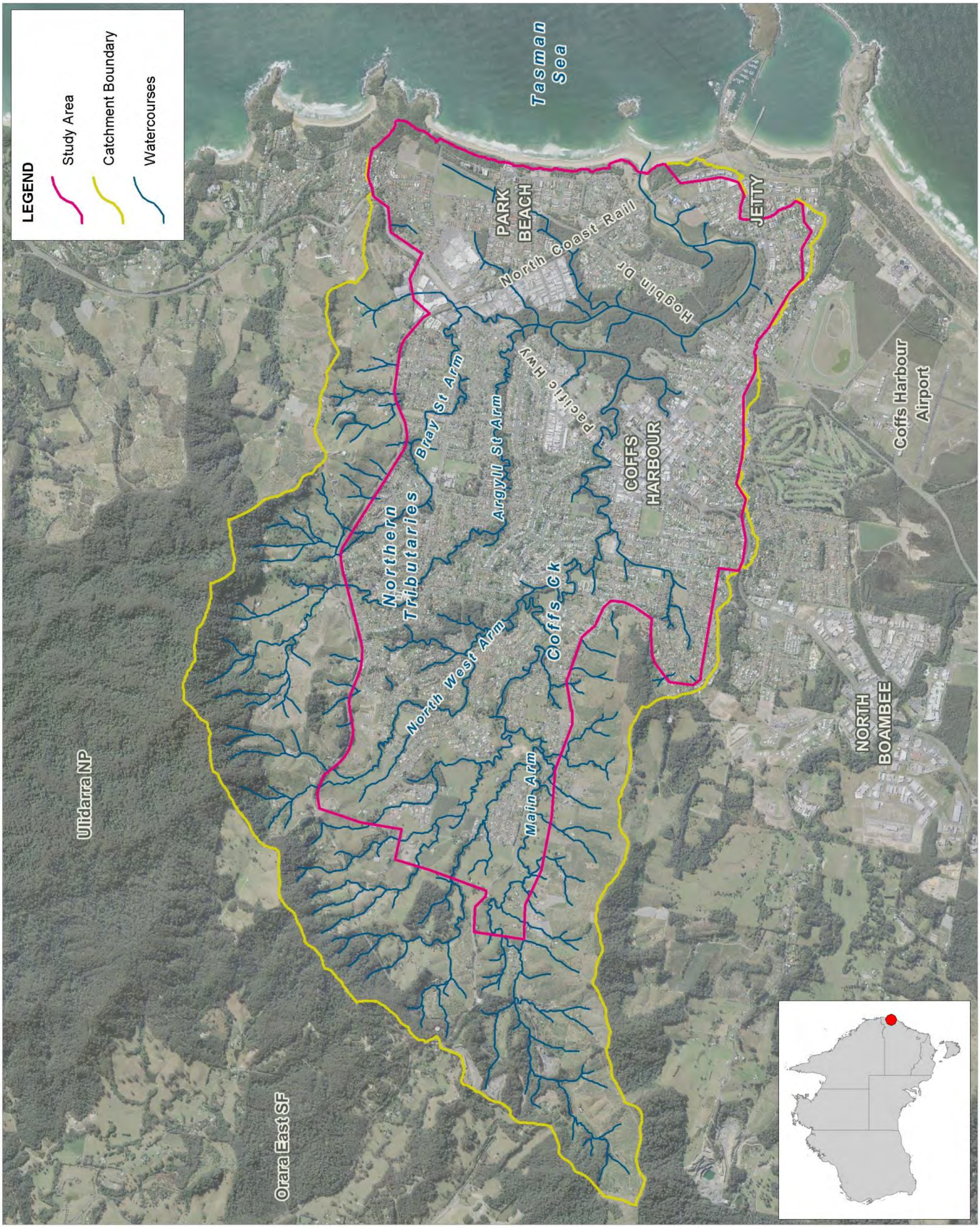
1.2 Study Background

Detailed studies of the flood behaviour within the Coffs Creek catchment have previously been undertaken, and extensive flood mitigation works have been completed over the past 25 years to manage the flood risk in Coffs Harbour. This current study is to be the first to investigate flooding within the entire Coffs Creek catchment collectively, combining Coffs Creek Main Arm, the Northern Tributaries and Park Beach. It defines the flood behaviour under historical and existing conditions, utilising additional information available from the recent major flood event occurring in March 2009 and a smaller flood occurring in March 2015.

Coffs Harbour has a long history of flooding. The floods of March 2009 and November 1996 are among the largest on record. A second flood occurred in November of 2009, with most serious flood impacts recorded in the Park Beach area. Smaller floods occurred in the Coffs Creek catchment in February 1992, December 1991, April 1989, May 1977 and March 1974.

The earliest flood investigation was completed in the late 1980s and facilitated the construction of bypass channels and modification to the natural creek alignment, to alleviate the flood risk along the reach of Coffs Creek downstream of Robin Street to Grafton Street (now the Pacific Highway). Since then, there have been many other flood investigations focusing on different areas within the catchment. A summary of previous studies is detailed in Section 2.2.1. The repeated occurrence of major floods has indicated that the nature of flooding within the catchment was more serious than initially thought.

The topography of the catchment has been identified as being a major contributor to the number of flood producing rainfall events experienced in Coffs Harbour. The Coffs Creek Flood Study (Webb, McKeown and Associates, 2001) included an assessment of historic east coast low rainfall events



Title:

Study Locality

Figure: **1-1**

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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

0 0.75 1.5km

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Filepath : K:\N20327_Coffs_Creek\MapInfo\Workspaces\Figures\DRG_001_1501XX_Locality.WOR by Professor Leslie of UNSW. This established relationships between rainfall recorded at the principal continuous gauge at Coffs Harbour Airport and that recorded at higher elevations in the catchment. Increased rainfall across the upper catchment was found to be consistent across a number of historic rainfall events. To account for this, the 2001 study recommended a scaling of design rainfall estimates was appropriate, and redefined design flood conditions in the catchment, to be reviewed in this current study.

The following Floodplain Risk Management Study and Plan (Bewsher Consulting, 2005) recommended the construction of four detention basins along the main Coffs Creek alignment, as well as assessment of the Loaders Lane levee that was constructed in the early 1990s. Local channel clearing and a levee constructed along the Bray Street arm of the Northern Tributaries were completed, as recommended in the Northern Tributaries Floodplain Risk Management Study (Patterson Consultants, 1997).

In 2008, BMT WBM was engaged by Council to undertake a review of the Coffs Creek Flood Study (WMA, 2001) which involved extending the modelled study area, and investigation into the merits of the proposed flood mitigation options put forward in the 2005 FRMS&P. The major flood event of March 2009 placed the project on hold as data was compiled and a detailed rainfall assessment was completed.

Three of the proposed detention basins have since been constructed (Bakers Road in 2010, Bennetts Road in 2012-13 and Spagnolos Road in 2015).

A floodplain risk management options assessment for Park Beach will be completed concurrently with this study. It is envisaged that an overall Coffs Creek Floodplain Risk Management Plan will follow the conclusion of the current study, to produce a consistent Plan for the entire Coffs Creek catchment.

1.3 The Need for Floodplain Risk Management at Coffs Harbour

The flooding of Coffs Harbour in recent years has highlighted the risk of developed areas situated within the floodplain of the many tributaries of Coffs Creek.

Floodplain risk management considers the consequences of flooding on the community, and aims to develop appropriate floodplain management measures to minimise and mitigate the impact of flooding. This incorporates the existing flood risk associated with current development, and future flood risk associated with future development, changes in land use and impacts of climate change.

Continued urbanisation within the catchment and construction of flood mitigation works since the completion of previous studies, necessitates the need for a revision of design flood conditions in the catchment to accurately define the existing flood risk. Alterations to design flood conditions and flood risk mapping, as a result of the constructed detention basins at Bakers Road, Bennetts Road and Spagnolos Road, will become apparent. The potential impact of the fourth detention basin suggested for Upper Shephards Lane will also be assessed. The performance of existing flood mitigation works will be reviewed, including the Loaders Lane levee located along the upper reach of the Coffs Creek Main Arm, and levee walls and benching around Collice Place and Langker Place along the Bray Street Arm of Northern Tributaries of Coffs Creek.

The outputs of this Flood Study will provide Council with updated information to allow for more informed planning decisions within the floodplain of Coffs Creek, in regards to effectively managing the flood risk. Combining the Coffs Creek Flood Study and Park Beach Flood Study will allow for one consistent Floodplain Risk Management Plan to be developed for the whole catchment area, with the ability to clearly prioritise the need for mitigation measures across the catchment.

1.4 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 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 Coffs Creek catchment.

1.5 Study Objectives

The primary objective of the Flood Study is to define the flood behaviour within the Coffs Creek 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 and acquisition of additional data including survey as required;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design event including the 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.

1.6 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 provides information on the available survey data for this study.

Section 4 details the development of the computer models.

Section 5 details the model calibration and validation process.

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

Section 7 presents design flood simulation results, including sensitivity testing, and associated flood mapping.

2 Study Approach

2.1 The Study Area

2.1.1 Catchment Description

The Coffs Creek catchment has a relatively small area of around 25km^2 and is located on the eastern Australian coast. The topography of the catchment is shown in Figure 2-1. The flat coastal floodplain rises steeply to an escarpment in the west. Elevations rapidly increase from below 10m AHD to more than 400m AHD over just a few kilometres. The Coffs Creek estuary forms the downstream limit of the catchment. The estuary is predominantly open but has closed in the past under low flow conditions along Coffs Creek, during periods of beach accretion, or after large ocean storm events (GeoLink, 2014). The tidal limit of Coffs Creek extends upstream along the main arm to just beyond the Pacific Highway Bridge.

The catchment is bound to the north by densely vegetated ranges of state forest and national parkland. Much of the low lying floodplain area is urban development, consisting of residential, commercial and industrial properties. The upper catchment is primarily used for agriculture and horticulture purposes.

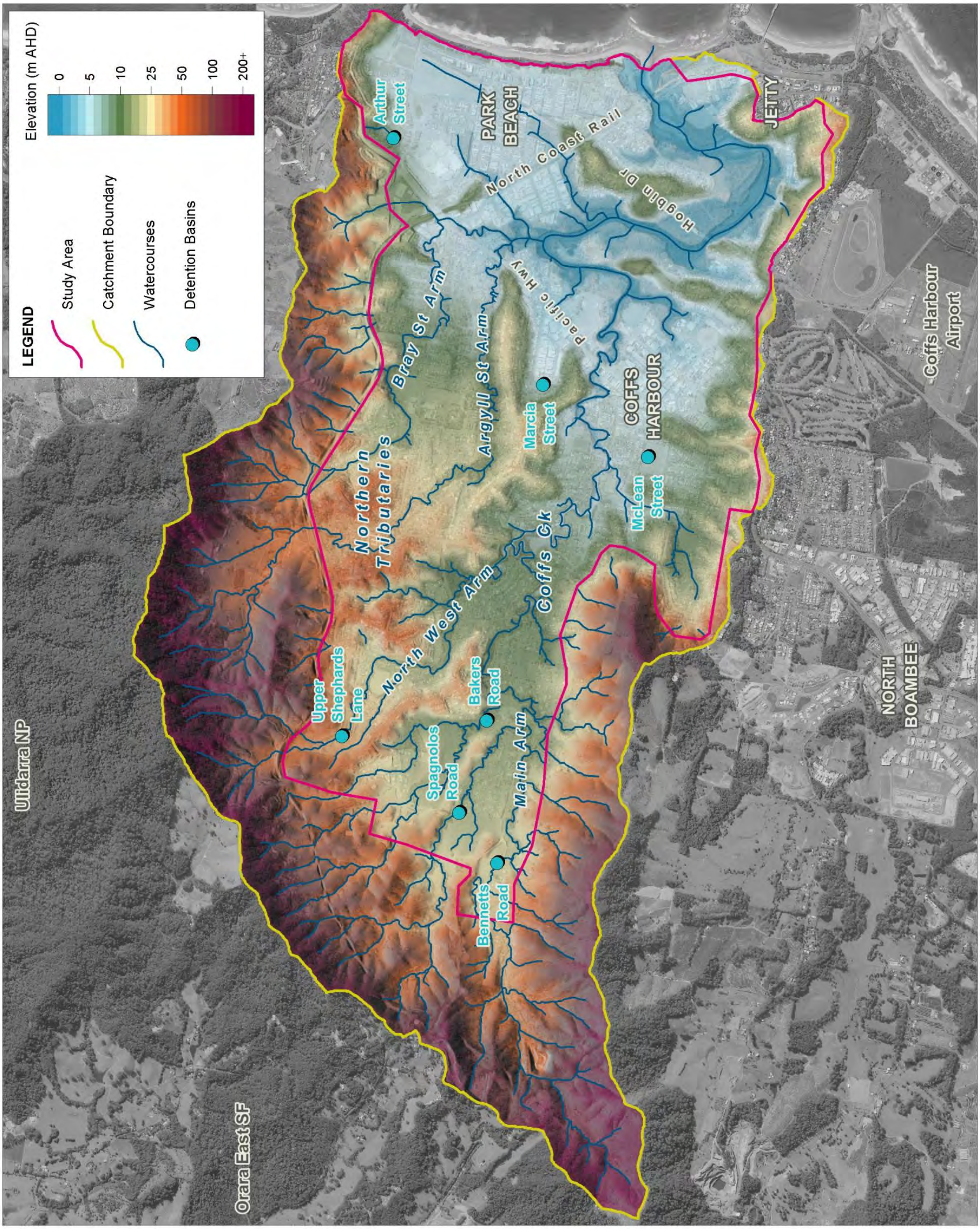
There are many major transport routes traversing the catchment. The Pacific Highway cuts across the catchment and provides a clear divide between the creek and estuary. Many tightly meandering, vegetated tributaries drain through the catchment upstream of the highway, before merging into a wider and clearer channel flowing into the ocean. The North Coast Rail crosses Coffs Creek near the ocean mouth, before heading to the west adjacent to the northern catchment boundary. Located between the railway line and the beach is the suburb of Coffs Harbour known as Park Beach. The Park Beach area is very flat with poor drainage. Due to the basin-like topography in Park Beach, the area will fill with floodwater during periods of high rainfall. The majority of the area drains to the west across the North Coast Railway line through a system of culverts. However, the northern most sub-catchment known as Macauleys Headland drains east to the beach, and the southern sub-catchments, including the Park Beach Caravan Park, discharge to the south directly into Coffs Creek.

The topography of the Coffs Creek catchment is conducive to extreme weather events. During the formation of a low pressure system off the coast known as an east coast low (ECL), the steep terrain located very close to the coastline is exposed. In the presence of strong onshore wind, moisture filled air masses are pushed towards the hills, where they rapidly rise facilitating intense rainfall over the upper catchment. The phenomenon of increased rainfall across the upper catchment was found to be consistent across a number of historic rainfall events.

The Coffs Creek catchment is prone to severe flash flooding as it is a relatively small catchment with steep upper slopes, a high level of urban development on the floodplain and the tendency for high rainfall.

2.1.2 History of Flooding

A number of floods are known to have occurred in Coffs Harbour since the late 1800s. However, detailed information surrounding events prior to the 1970s is scarce. Evidence sourced from



Title:

Catchment Topography

Figure:

2-1

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Filepath : K:\N20327_Coffs_Creek\MapInfo\Workspaces\Figures\DRG_002_1501XX_Topography.WOR newspaper clippings reproduced in the Coffs Creek Flood Study (WMA, 2001) indicates that significant flood events were experienced in November 1917 and February 1938. Anecdotal evidence suggests that February 1938 was the larger of the two, with floodwaters observed to be around two feet higher than during the 1917 event.

Since rainfall records commenced, floods are known to have occurred in June 1950, April 1962 and April 1963. The April 1963 event was the largest of these, where approximately 570mm of rainfall was recorded over a 48-hour period. Any further detail regarding these events is limited.

More information is available for floods experienced in the latter part of the 20th century. This includes photographs, flood levels and other evidence relating to the number of properties inundated by floodwaters. Large flood events occurred in March 1974 and May 1977 and a smaller flood occurred in April 1989. These events initiated the numerous investigations into the nature and severity of flooding in Coffs Harbour. Resulting from these investigations, various physical flood mitigation measures have been constructed across the catchment throughout the years. Measures include channel modifications (re-alignment, widening and clearing), levees and detentions basins. The location of detention basins is shown on Figure 2-1.

The 1991 event was a result of heavy rainfall across the entire catchment. Around 200mm was recorded in the 24 hours to 9am on the 13th December, by numerous rainfall gauges across the catchment. The most intense rainfall occurred in a nine hour period, with Coffs Harbour Airport recording 195mm over this time. This storm duration was equivalent to between a 10% AEP and 5% AEP design rainfall and resulted in significant flooding in the Coffs Harbour CBD. A minor flood also occurred in February 1992. This event was a short duration storm focused over the coast. Coffs Harbour Airport recorded 86mm in 75 minutes.

The flood of November 1996 is the largest on record in Coffs Harbour. A rainfall gauge located at the Catholic Club upstream of the Pacific Highway Bridge recorded 239mm over 12 hours. Review of private rainfall gauges indicated that around three times the rainfall recorded at the airport was found to have occurred in the upper catchment. A private gauge in the Northern Tributary catchment recorded 388mm over a 4.5 hour period. Rainfall of this intensity is equivalent to design rainfall rarer than the 0.2% AEP. The worst affected areas along the Coffs Creek main arm included residential developments around Goodenough Terrace and Loaders Lane, the CBD and Gundagai Street. The Bray Street arm of the Northern Tributaries was badly affected, as was the industrial area along Orlando Street. Upstream of the Pacific Highway Bridge, recorded flood levels were up to 1.0m higher than previously defined 1% AEP design flood levels. Flooding in Park Beach was not as severe, with flood levels comparable to around 5% AEP design levels.

The March 2009 event was of a similar magnitude to the November 1996 event, with slightly higher flood levels recorded upstream of the Pacific Highway. The rainfall gradient phenomenon was again observed across the catchment, with rainfall depths recorded in the upper catchment up seven times those recorded at the Airport for the 3-hour critical storm duration. The Red Hill Reservoir gauge recorded rainfall more intense than design estimates for the 0.2% AEP, for durations between three to nine hours. As a result of considerably less rainfall along the coast, flooding within Park Beach was not to the same magnitude as flooding along Coffs Creek and its tributaries.

Serious flooding within Park Beach resulted from heavy, intense rainfall in November 2009 and more recently in February 2015. The November 2009 event was the more extreme of the two, with a total of 361mm recorded in the 24 hours to 9am on the 6th at the Coffs Harbour Airport gauge. Around 184mm of this was recorded in just three hours. Rainfall of this intensity is equivalent to design rainfall of around a 1% AEP for durations less than six hours, and around a 2% AEP for the 24 hour total. Both events particularly highlighted the severity of flooding affecting properties in low-lying areas of San Francisco Avenue and York Street.

2.2 Compilation and Review of Available Data

2.2.1 Previous Studies

The nature of flooding in Coffs Harbour has been the subject of numerous investigations over recent years. Previous studies available include flood studies, design reports for flood mitigation measures and data compilation exercises following large flood events within the catchment. There have also been a number of smaller, independent flood studies conducted for the development of commercial and residential premises, which have not been included in the following summary.

2.2.1.1 Report on Coffs Creek Flood Investigation (Laurie Montgomerie and Pettit Pty Ltd, August 1978)

The first of many flood studies to be completed for the catchment area of Coffs Harbour following two large floods in the 1970s. Approximate water level gradients and flood inundation extents were mapped for a 1% AEP event.

2.2.1.2 Coffs Creek Flood Mitigation Robin to Grafton Street (Laurie Montgomerie and Pettit Pty Ltd, January 1982)

The report details flood mitigation works to improve the creek channel downstream of Robin Street to Grafton Street. An EIS for the works was completed in the Coffs Creek Flood Mitigation Works (Kinhill Engineers Pty Ltd, 1990). Staged construction of the channel works was completed between June 1991 and June 1993. Channel modifications downstream of Azeala Ave were completed before the flood of December 1991.

2.2.1.3 April 1989 Flood Coffs Creek (Webb, McKeown and Associates, December 1989)

Rainfall records and flood height data were compiled following the April 1989 event. The study was completed to supplement the Lower Coffs Creek Flood Study (Public Works Department, 1992) for the purpose of providing additional flood data for model calibration.

2.2.1.4 Lower Coffs Creek Flood Study (Public Works Department, January 1992)

The study was commissioned to redefine design flood estimates along Coffs Creek downstream of Robin Street, with channel modification work resulting from the Flood Mitigation Scheme to be included in the updated hydraulic model. A RORB hydrologic model and a MIKE-11 (1D) hydraulic model were developed and calibrated/verified against the May 1977 and April 1989 events. Cross section survey of the creek was collected to supplement existing survey gathered for earlier studies. The surveyed cross-sections used to construct the hydraulic model formed the basis of the channel representation of the lower Coffs Creek reach in this current study.

Design rainfall was calculated in accordance with the methods released by AR&R (1987).

2.2.1.5 13 December 1991 Flood Coffs Creek (Webb McKeown and Associates Pty Ltd, December 1992)

All available rainfall, flood and tidal data for the December 1991 flood event were compiled into the report. The performance of the flood mitigation works constructed upstream of Grafton Street during the flood was assessed. Modifications to improve the structural integrity and performance of the system were recommended.

The additional flood information was used to verify the hydrologic and hydraulic model parameters adopted in the Lower Coffs Creek Flood Study (1992). The existing hydraulic model was converted into a 1D RUBICON model and incorporated all of the Flood Mitigation Scheme channel works. Modelling results indicated that flood mitigation works in place during the storm were effective and reduced flood levels upstream of Gundagai Street by more than 0.9m. Had the complete works been in place, levels would have been reduced by a further 0.25m at Gundagai Street and would have been approximately 1.6m lower in the vicinity of Robin Street.

2.2.1.6 Coffs Harbour Flood of 23 November 1996 Data Collection (Webb, McKeown and Associates, June 1997)

The flash flood that hit Coffs Harbour on 23rd November 1996 was declared a natural disaster. Immediately following the flood, Webb, McKeown and Associates compiled a database of rainfall records, flood heights, ocean water levels, damages and costs incurred from across the flood affected area. The report summarises the data collection process, presents all the compiled information and includes details of the economic implications, social implications and environmental impacts resulting from the flood event.

As well as interviewing most flood affected residents, the data collection process involved the distribution of questionnaires to around 1200 residents/business owners along Coffs Creek and the Northern Tributaries. Of these, 256 responses were returned, many of which contained detailed information regarding flood damages. Approximately 460 residential and commercial properties were inundated above floor level during this event. Flood levels exceeded the 1% AEP level previously defined in the Lower Coffs Creek Flood Study (Public Works Department, 1992) by up to 1.0m upstream of Grafton Street, and by 0.8m and 0.4m along the Bray Street and Argyll Street branches of the Northern Tributaries respectively. It is estimated that the total damages from insurance claims was in excess of $30M.

A significant output from this study was the detailed compilation of rainfall records. This information proved valuable in future work completed as part of the Coffs Creek Flood Study (WMA, 2001 – see Section 2.2.1.9) to determine appropriate design rainfall estimates applicable to Coffs Harbour.

2.2.1.7 Northern Tributaries of Coffs Creek Flood Study (Paterson Consultants Pty Ltd, November 1997)

Paterson Consultants completed the first flood study focused on the Northern Tributaries of Coffs Creek. Prior to the study only smaller flood investigations have been completed for development purposes.

A community survey was distributed to residents and provided around 20 flood levels within the study area, primarily from the December 1991 event. Some records were provided for the smaller event of February 1992. These two events were used for the calibration of the hydrologic and hydraulic models.

The RORB hydrologic model was largely based on the model developed in the Lower Coffs Creek Flood Study (1992), adopting similar model parameters. Extensive cross section survey along the Bray Street tributary, the Argyll Street tributary and drainage channel alignments in Park Beach was collected to define channel cross sections for use in the MIKE-11 hydraulic model.

Design flood modelling was completed for the 20% AEP, 5% AEP, 1% AEP and PMF events, for durations ranging from two to nine hours. For the 1% AEP, a peak flood level of 3.3m AHD was calculated at the confluences of the Bray Street and Argyll Street tributaries, rising to 15.6m AHD along the Bray Street tributary at Apollo Drive, and 15.0m AHD along the Argyll Street tributary at Joyce Street. Along the main Coffs Creek arm, the 1% AEP flood level was expected to rise to 2.75m AHD just downstream of the Pacific Highway. Within Park Beach, a peak flood level of 4.3m AHD was calculated at Park Beach Road.

A Flood Hazard Assessment based on the depth and velocity relationship of floodwaters was completed. Areas of High Hazard were largely contained to creek channels – the caravan park situated at the confluence of the Bray Street and Argyll Street tributaries being the only residential or commercial premise located within a High Hazard area.

2.2.1.8 Northern Tributaries of Coffs Creek Floodplain Risk Management Study (Paterson Consultants Pty Ltd, November 1997)

The Northern Tributaries of Coffs Creek Floodplain Risk Management Study was undertaken by Patterson Consultants in conjunction with the Northern Tributaries of Coffs Creek Flood Study. As a result of the extreme flood that occurred in November 1996. A major component of the study involved extending and recalibrating the hydraulic model to the 1996 event and consequently revising design flood levels.

During the 1996 event, floodwaters overtopped the creek bank at locations where overland flow links had not been allowed for within the 1D modelling. To accurately simulate expected overland flow paths, the hydraulic model was extended upstream to Apollo Drive, with overland flow links added where required. In order to replicate flood levels recorded during the 1996 event, roughness parameters in the hydraulic model were increased along most of the Bray Street arm and the lower reach of the Argyll Street arm. Peak flood levels were recalculated using the updated hydraulic model. Compared to design flood levels presented in the Northern Tributaries of Coffs Creek Flood Study, the estimated 1% AEP flood level along the Bray Street tributary increased by 0.2-0.4m. Peak flood levels along the Argyll Street tributary increased by less than 0.1m.

The average annual cost of flood damages for the Northern Tributaries and Park Beach study area was estimated to be over $320K.

Some flood mitigation works were completed as an outcome from the study. A levee was constructed across the back of Collice Place and Langker Place along the Bray Street arm. The creek capacity was also increased along the middle reaches. Other options such as an additional detention basin on the Bray Street oval, and increasing capacity of culverts providing cross drainage from Park Beach under the North Coast Rail did not eventuate following further investigation.

2.2.1.9 Coffs Creek Flood Study (Webb, McKeown and Associates, May 2001)

Following from the 1996 event, where recorded flood waters exceeded previously estimated 1% AEP flood levels by up to 1.0m, WMA were commissioned by Council to complete a revised Flood Study for Coffs Creek. The potential underestimation of design flood levels was considered likely due to the strong orographic effect of rainfall observed during major flood events within the catchment. The study contained a detailed assessment of historic rainfall patterns. It was concluded that the variation in intensity of rainfall across the catchment observed in major flood events is not represented in current methods for deriving design rainfall estimates (AR&R, 1987). Scaling factors were applied to design rainfall totals based on elevation, to simulate findings from the investigation.

A RORB hydrological model and a 1D RUBICON hydraulic model were used. The hydraulic model expands on the area modelled in the previous Lower Coffs Creek Flood Study (1992), extending from the ocean entrance to just downstream of Coramba Road and Polwarth Drive along the Main Arm and North-West Arm respectively. All constructed flood mitigation measures were included in the model.

The hydrologic and hydraulic models were calibrated to the November 1996 event, and verified against the December 1991, April 1989, May 1977 and March 1974 events.

Design flood conditions were calculated for the 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and the PMF event. Applying the “best estimate” scaling factor to design rainfall estimates resulted in design flood levels generally around 0.3-0.4m higher than results derived using methods as presented in AR&R (1987). In comparison to revised design flood level profiles, flood levels recorded in the 1996 event are equivalent to around 0.5% AEP levels upstream of the Pacific Highway, and around 1% AEP levels closer to the entrance.

The study also involved numerous sensitivity tests, including the impact of variation in design rainfall estimates within upper and lower bounds, the impact of catchment development and the influence of ocean entrance and tidal conditions.

2.2.1.10 Floodplain Risk Management Study (Bewsher Consulting, October 2005)

The Coffs Creek Flood Study updated by WMA suggested that flooding within the Coffs Creek catchment was much worse than originally thought. Bewsher Consulting were engaged by Council to complete a Floodplain Risk Management Study and Plan. The Study involves investigation into measures to reduce the flood risk in the lower Coffs Creek catchment. The Northern Tributaries of Coffs Creek and the Park Beach area were not included in the study area.

A flood damages assessment details the number of homes inundated above floor level for 1% AEP and PMF events. The average annual cost of flood damage was predicted at $2.2M. The total cost of damages if the 1% AEP flood was to occur was estimated at $28M.

The potential management options identified include physical works and other measures. Physical works options included widening bridge openings of Orlando Street, the Railway and Grafton Street; dredging of the estuary and lower reaches; clearing channel vegetation and the construction of detentions basins, levees and floodways. Other management measures included zoning and development controls, flood warning and emergency planning, voluntary house purchase, floor level raising and improved community awareness/preparedness.

2.2.1.11 Floodplain Risk Management Plan, Draft (Bewsher Consulting, October 2005)

The Floodplain Risk Management Plan was completed by Bewsher Consulting in conjunction with the Floodplain Risk Management Study. The Plan details the recommended measures from the Study and outlines a funding and implementation plan. The document was the precursor to this current study.

Specific measures recommended in the Plan include:

- Construction of four detention basins in the upper catchment;
- Floodway and channel improvements adjacent to Ann Street;
- Review of the Loaders Lane levee;
- Continuing the implementation of the independent Central Business District Drainage Scheme;
- Implementation of planning and development controls;
- Updating the SES Local Flood Plan for Coffs Creek;
- Implementing a public awareness program;
- Flood proofing and flood action plans by individual home owners; and
- Continued maintenance of creek corridors.

Assessment of any flood mitigation options using hydraulic modelling was advised. Administration of all suggested measures was estimated to have a total cost of $9.3M and would reduce the average annual cost of flood damage by around $7.4M and reduce the total cost of damages from a 1% AEP event by around $17.7M.

2.2.1.12 Park Beach Floodplain Management Study (de Groot and Benson, 2010)

Following from the two large flood events of March and November 2009, this study was completed by de Groot and Benson to assess the flood risk and investigate potential mitigation options in Park Beach. The study revised the design flood modelling and flood damages assessment undertaken in the Northern Tributaries of Coffs Harbour Floodplain Risk Management Study (Paterson

Consultants Pty Ltd, 1997) using a DRAINS model calibrated to the November 2009 event. The DRAINS model produced higher design flood levels than those estimated in the earlier Northern Tributaries Study, largely resulting from the Coffs Creek tailwater boundary condition having increased by around 0.55m, as an output of the Coffs Creek Flood Study (WMA, 2001).

A flood damages assessment was completed using existing property floor level survey for the area. Limited data availability was identified as a shortfall of the assessment, as only half of flood affected properties had surveyed floor levels. Taking this into consideration, a total Net Present Value of flood damages to residential and commercial properties over the next 50 years was estimated to be in the order of $5M.

The study recommended that further investigation into drainage works to redirect additional catchment area to the Macauleys Headland outlet, or to a new beach outlet may alleviate flooding of San Francisco Avenue and York Street.

2.2.1.13 Coffs Harbour Region March/April 2009 Flood Event (WMAwater, August 2013)

The project considered the major rainfall event that occurred at Coffs Harbour on the 31st March and 1st April 2009, in terms of improving rainfall gradient estimates for the Coffs Creek and Boambee-Newports Creek catchments, as well as advising on appropriate design rainfall estimation methods for NSW east coast catchments with similar topography.

Analysis of recent major orographic (November 1996 and March/April 2009) and non-orographic (November 2009) rainfall events for a range of durations up to 24 hours found that the magnitude of the rainfall ratio between the Airport and upper catchment varied significantly across different durations within the same storm for the two orographic rainfall events, where shorter durations displayed a more pronounced gradient. The non-orographic event exhibited almost the reverse effect, where upper catchment areas recorded around half the rainfall recorded at the Airport. It also found that the presences of an easterly (on-shore) wind may influence the magnitude of the rainfall gradient present across the catchment. Investigation into incorporating the probability of an easterly wind when deriving rainfall gradient ratios was suggested.

A comprehensive dataset was compiled by Council following the event and consists of photographs, 249 flood level marks across the Coffs Creek and Boambee-Newports Creek catchments, and privately read rainfall data. Most of the information was sourced from a widely distributed community questionnaire. Recorded flood levels indicated that the 2009 event displayed a similar level of inundation to the 1996 flood downstream of Pacific Highway, and was of higher magnitude in the upper catchment.

The hydrological and hydraulic models developed for the Coffs Creek Flood Study were used for an assessment of the 2009 event. Simulation of the March 2009 event generally modelled flood levels within ±0.4m of recorded levels. The model tended to underestimate peak flood levels – possibly attributed to hydraulic model schematisation or structural blockages.

2.2.2 Water Level Data

Manly Hydraulics Laboratory (MHL) operates one water level recorder in the Coffs Creek catchment. The gauge is located on Coffs Creek just upstream of the Pacific Highway Bridge and has been operational since December 1980. The gauge malfunctioned during the 1996 flood and did not provide accurate water level recordings.

Recorded flood levels upstream of the Pacific Highway Bridge (previously Grafton Street) are presented in Table 2-1. Where gauge recordings were not available, approximate levels have been gathered from other sources. Only flood events occurring since the 1970s are presented.

Table 2-1 Recorded Peak Flood Levels at the Pacific Highway Bridge

Rank	Flood Event	Peak Flood Level (m AHD)
1	23rd November 1996	5.4‘
2	31st March 2009	5.1
3	6th November 2009	4.3
4	12th March 1974	3.9 **
5	13th December 1991	3.1
6	19th May 1977	3.1 *
7	27th April 1989	2.5 *

‘ Surveyed debris mark

** Recorded peak flood level estimated (Figure 15 and Figure 16, Coffs Creek Flood Study, 2001)*

*** Peak flood level estimated from model verification long section profile (Figure 17, Coffs Creek Flood Study, 2001)*

The relative magnitude of each flood event is difficult to quantify, as flooding across the catchment is largely dependent on local rainfall, and rainfall intensity is known to vary significantly across the catchment during extreme events. For example, floods of March/April 2009 and November 1996 resulted from intense rainfall over the upper catchment with flood impacts upstream of the Pacific Highway being the worst on record. The November 2009 flood resulted from heavy rainfall over the coast, with flooding being most severe within the Park Beach area.

MHL also operates an ocean tide recorder located on the Coffs Harbour Jetty. This gauge has been operational since July 1980.

In addition to the MHL gauges, Council operates four stream gauges in the catchment. These located at Bray Street, Gundagai Street, Loaders Lane and Bennetts Road and were installed in 2010. The location of the official water level gauges is shown on Figure 2-2.

As of February 2017, SES is trailing a DipStik flood warning instrument on the Bray Street Arm of the Northern Tributaries at Orlando Street.

2.2.3 Historical Flood Levels

Since the 1970s a number of large floods have been recorded in Coffs Harbour. The quantity and quality of flood levels captured after each event has improved in time. After the events of April 1989, November 1996 and March 2009 various flood information was sourced and compiled by WMA. Limited flood records are available for events occurring in the 1970s. For floods occurring prior to this, even anecdotal evidence is scarce.

Twelve flood levels were documented for the 1991 event and around 930 marks were compiled for the 1996 event. Of the 242 flood levels collected following the March/April 2009 event, 42 are located within the Coffs Creek catchment. Council also gathered surveyed peak flood marks in Park Beach following the smaller events of November 2009 and March 2015. Peak flood levels are typically surveyed from debris/water level marks or deduced from photographic and anecdotal evidence.

The Coffs Creek Flood Study (WMA, 2001) obtained CCTV footage of rising floodwaters at The Promenade (located at the intersection of Harbour Drive and Mildura Street) during the 1996 event. From this an approximate stage hydrograph has been estimated. Although the exact peak water level may be uncertain, the timing of the flood wave is useful in terms of model verification.

2.2.4 Rainfall Data

Historically, there has been relatively few rainfall gauges located within the catchment of Coffs Creek. A more extensive network of both continuous and daily read gauges exists in the broader region, which can be used to provide insight into rainfall behaviour during flood events. The rainfall gauges in the vicinity of Coffs Harbour are detailed in Table 2-2. The distribution of these gauges is shown in Figure 2-2.

The absence of rainfall data available within the catchment, specifically of continuous rainfall recordings, was identified as a significant data gap following the 1996 event. This prompted the installation of three additional pluviographs operated by MHL across the upper catchment. After the 2009 event, Council installed an additional eight continuously read rainfall gauges, six of which are located within the catchment boundary.

Comprehensive data compilation, including collation of rainfall records, was a completed after the large flood events of April 1989, December 1991, November 1996 and March 2009 (see Section 2.2.1 for further information).

Further discussion on recorded rainfall data for historical events is presented with the calibration and validation of the models developed for the study in Section 5.

2.2.5 Council Data

A number of spatial datasets were provided by Council for use in the study. These included Light Detection and Ranging (LiDAR) collected in 2007 and aerial photography taken in 2009, both covering the entire study area. Modelled flood behaviour is inherently dependent on the ground topography and for this study an accurate representation of the floodplain is essential. Advanced GIS analysis allows the LiDAR imagery to be assessed in concert with spatial 2D flood model data, facilitating mapping and flood risk categorisation, and overall flood management. Local topographic controls such as roads embankments in the modelled floodplain are accurately represented in the LiDAR dataset.

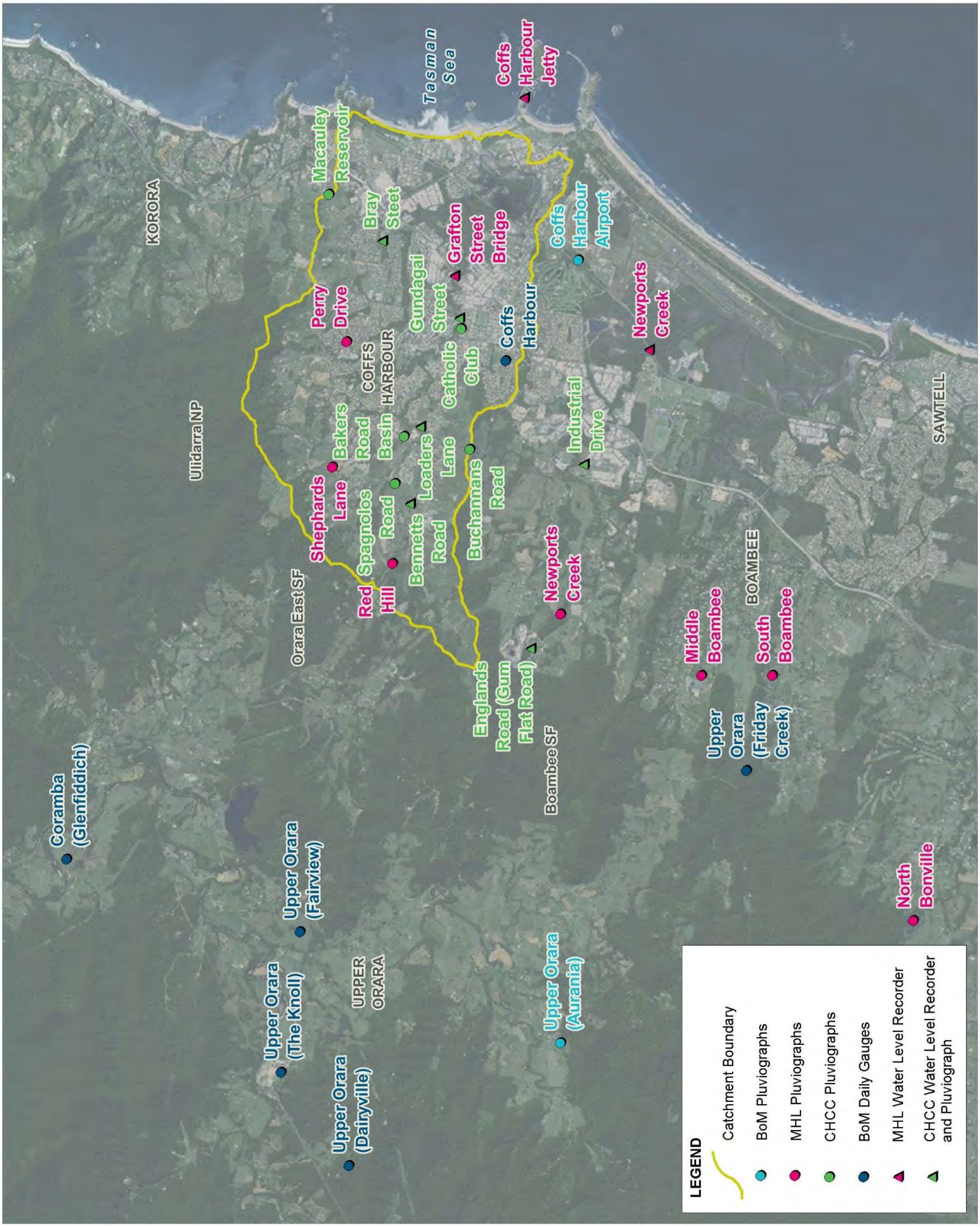
Topographic survey cross section of the channel and floodplain utilised in the development of previous hydraulic models were accessible.

Other GIS layers provided by council include cadastre, water courses, LEP zonings, asset mapping, and details of the stormwater drainage system, bridges and culverts.

Design drawings and reports detailing flood mitigation works completed within the catchment were also available. They included levees (Loaders Lane and Collice Place / Langker Place), channel modification works from Robin Street to Grafton Street and detention basins constructed at Bakers Road and Bennetts Road. Design plans for the detention basin constructed at Spagnolos Road and the proposed Upper Shephards Lane detention basin were also provided, along with drawings of drainage within the CBD including the Maclean Street detention basin.

Table 2-2 Rainfall Gauges in the Coffs Creek Catchment Locality

Station Number	Station Name	Operator	Data Type	Opened	Closed
059040	Coffs Harbour Airport*	BoM	Pluviometer	1960	-
059026	Upper Orara (Aurania)	BoM	Pluviometer	1970	-
-	Newports Creek	MHL	Pluviometer	1990	-
-	Middle Boambee	MHL	Pluviometer	1990	-
-	North Bonville	MHL	Pluviometer	1990	-
-	South Boambee	MHL	Pluviometer	1991	-
-	Red Hill	DPI	Pluviometer	1989	post 1996
7820	Red Hill	MHL	Pluviometer	1999	-
7822	Shephards Lane	MHL	Pluviometer	1999	-
7824	Perry Drive	MHL	Pluviometer	1999	-
-	Catholic Club	CHCC	Pluviometer	1996	pre 2009
-	Bray Street	CHCC	Pluviometer	2010	-
-	Macauley’s Reservoir	CHCC	Pluviometer	2010	-
-	Gundagai Street	CHCC	Pluviometer	2010	-
-	Loaders Lane	CHCC	Pluviometer	2010	-
-	Bakers Road Basin	CHCC	Pluviometer	2010	-
-	Bennetts Road	CHCC	Pluviometer	2010	-
-	Buchannans Road	CHCC	Pluviometer	2010	-
-	Spagnolos Road	CHCC	Pluviometer	2010	-
-	Industrial Drive	CHCC	Pluviometer	2010	-
-	Englands Road (Gum Flat Road)	CHCC	Pluviometer	2010	-
059009	Coramba (Glenfiddich)	BoM	Daily	1891	-
059010	Coffs Harbour	BoM	Daily	1899	1965
059011	Upper Orara (The Knoll)	BoM	Daily	1908	1974
059040	Coffs Harbour Airport	BoM	Daily	1943	-
059097	Upper Orara (Friday Ck)	BoM	Daily	1970	1983
059095	Upper Orara (Dairyville)	BoM	Daily	1970	-
059101	Upper Orara (Fairview)	BoM	Daily	1971	2002
059151	Coffs Harbour Airport Comparison	BoM	Daily	2013	-



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Rainfall and Water Level Gauges in the Vicinity of Coffs Harbour

Figure: **2-2**

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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

0 2 4km

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Due to the numerous flood studies previously completed for the catchment, a large quantity of flood information exists. Council gave access to hydrologic and hydraulic modelling files developed in previous studies and GIS databases of flood extent mapping, historic flood levels and floor level survey of the previously identified flood affected properties.

2.3 Site Inspections

A site inspection was undertaken in the initial stages 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 road and rail crossing and associated embankments;
- General nature of Coffs Creek and its tributaries and the floodplains noting river plan form, vegetation type and coverage and the presence of significant flow paths; 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:

- LiDAR survey data (from 2008 and 2013) for the entire study area;
- Topographic ground survey cross sections of Coffs Creek and its tributaries;
- Hydrographic survey cross sections of the estuary mouth (1979 and following the 1996 event); and
- Cross sections representing the channel and floodplain extracted from previously developed hydraulic models of Coffs Creek and its tributaries.

Extensive analysis was undertaken to assess the adequacy of the available survey datasets for flood modelling purposes. This analysis, which identified the potential benefit of collecting additional structure and channel cross section survey, is detailed in Section 3.

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.

Extensive community consultation was undertaken as part of the Coffs Creek Flood Study (Webb, McKeown and Associates, 2001) and the Floodplain Risk Management Study and Plan (Bewsher Consulting, 2005).

The Draft Coffs Creek and Park Beach Flood Study was placed on public exhibition from 28th February to 30th March 2018. The exhibition sought public comments and feedback on the study. No comments were received during the public exhibition period.

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 catchments to the downstream study limits at the entrance into the Tasman Sea.

2.6.2 Hydraulic Model

The hydraulic model (discussed in Section 4.2) developed for this study includes:

- Two-dimensional (2D) representation of the channel and floodplain of Coffs Creek and its tributaries including Park Beach, covering an area of 15km^2 (approximately 60% of total catchment area); and
- One-dimensional (1D) representation of:
  - Key trunk drainage systems including in the CBD, drainage lines just downstream of the North Coast Railway line (around Taloumbi Road, Antaries Avenue, Pearce Drive), around Gundagai Street / Galliploi Road, around Red Cedar Drive / Robin Street, along Wilton Place, around Combine Street and along Marcia Street;
  - Stormwater network in Park Beach;
  - Culvert structures providing outflow from detention basins; and
  - Other key hydraulic structures (bridges and culverts) along creek alignments providing cross drainage at road crossings.

The hydraulic model is applied to determine flood levels, velocities and depths across the study area for historical and design 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 flood events of November 1996 and March/April 2009 were used for calibration of the developed models. The flood of March 2015 has also been considered as validation for flood behaviour in Park Beach. Assessment of the model performance also incorporated a range of sensitivity tests of key variables/model assumptions. Sensitivity testing was undertaken for the design flood events and has been reported in Section 7.7.

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.

The approach adopted in the Coffs Creek Flood Study (WMA, 2001) was applied when determining design rainfall estimates for the Coffs Creek catchment. Design rainfall estimates were derived for the catchment in accordance with the procedures outlined in Australian Rainfall and Runoff (IEAust, 2001), with weighting factors then applied based on catchment topography.

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. Flood hazard categories (provisional and true hazard) and hydraulic categories are derived from the hydrodynamic model results and are also mapped. The mapping outputs are described in Section 7 and are presented in the accompanying flood mapping compendium.

3 Survey Data

One of the initial tasks when undertaking a flood study is to identify the need for any additional survey requirements. This section outlines the analytical process that was undertaken in order to assess the adequacy of the available data. This process determined that the available datasets provided an adequate representation of floodplain and channel topography, but that some structural details were absent. It identified that there was added benefit in acquiring additional structure and channel survey data to provide a complete data set across the study area.

As previously mentioned, two LiDAR aerial survey data sets were available. They both provided detailed representation of the floodplain for the entire study area. In addition, channel cross section survey data and cross section information extracted from previous studies were available for various reaches of Coffs Creek and its tributaries. Each of the data sets containing channel cross section data is detailed in Table 3-1. The coverage of the data is shown on Figure 3-1.

Table 3-1 Summary of Avaliable Channel Cross Section Data

Source	Data Collected	Details	Data Type
Lower Coffs Creek Flood Study (Public Works, 1992)	1978 - 1992	Cross sections surveyed specifically for the study, as well as for prior investigations. Other studies include: • Coffs Creek Flood Mitigation Robin Street to Grafton Street (Laurie Montgomerie and Pettit, 1982); • Coffs Creek Flood Mitigation Works EIS (Kinhill Engineers, 1990); • Shephards Field Estate Flood Study (Bruce Fidge and Associates, 1990); • McCarthy Park Estate Stage 1 - Coffs Creek Flood Study (De Groot and Benson, 1998); and • Design cross sections for channel mitigation works.	RUBICON model cross sections
Flood Study for Coffs Creek West (GHD, 1991)	1991	Small scale flood investigation for development in the area.	HEC-RAS model cross sections
North West Coffs Creek (GHD, 1994)	1994	Small scale flood investigation for development in the area.	HEC-RAS model cross sections
Northern Tributaries of Coffs Creek Flood Study (Patterson Consultants, 1997)	1997	Survey collected specifically for the study, including sections in Park Beach.	Surveyed spot elevations
Coffs Creek Flood	1999	Additional cross sections	RUBICON

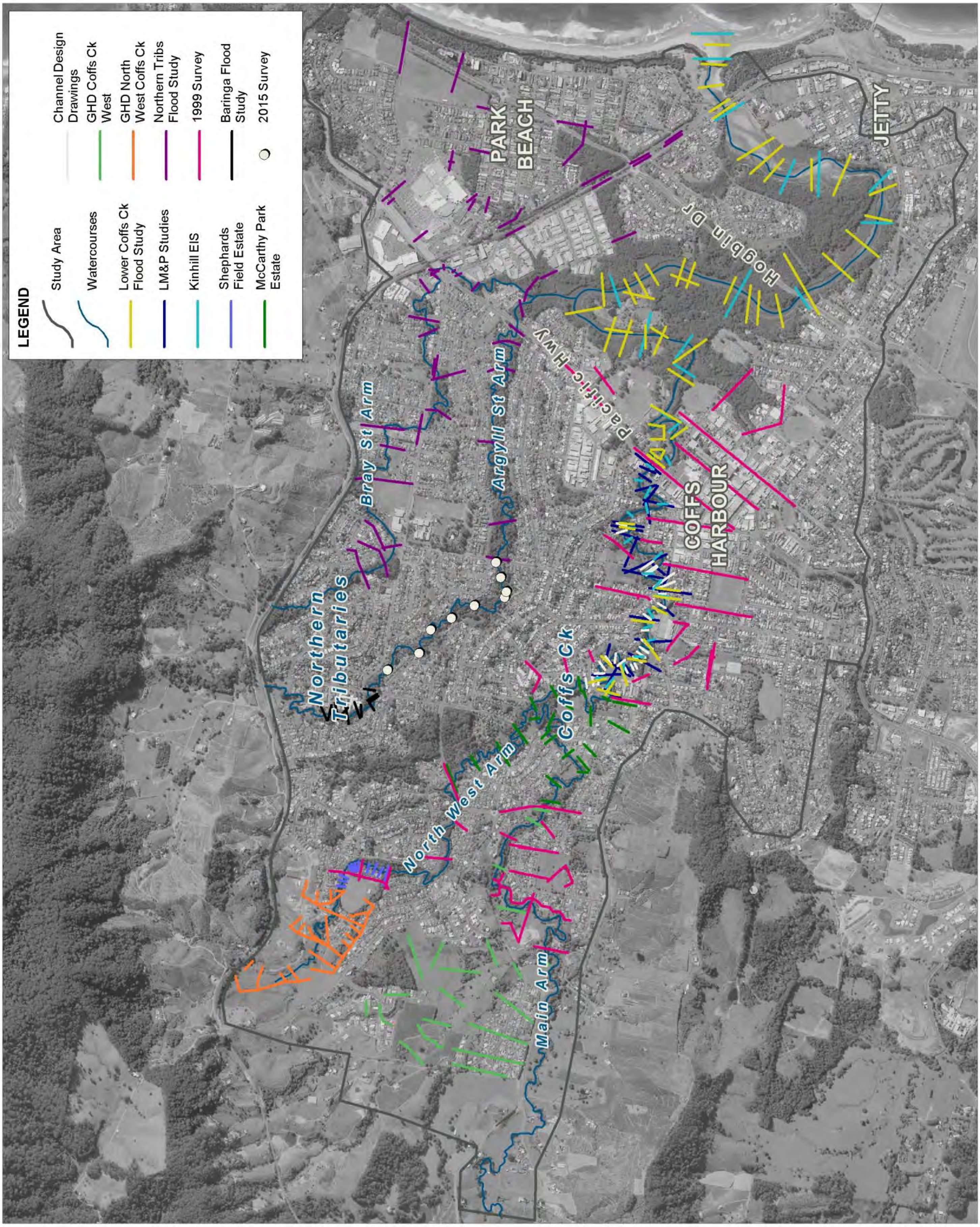
Source	Data Collected	Details	Data Type
Study (Webb, McKeown and Associated, 2001)		surveyed to supplement existing survey used in the Lower Coffs Creek Flood Study.	model cross sections
Baringa Hospital Flood Study (Lewis Ford and Associates, unknown)	Unknown	Small scale flood investigation for expansion of Baringa Hospital.	HEC-RAS model cross sections
Bennets Road Detention Basin WAE (Newnham Karl Weir, 2013)*	2013	Works as executed drawings for Bennetts Road detention basin.	Plan drawings

** Location of cross section data not shown on Figure 3-1.*

An accurate centreline of the Coffs Creek watercourses was digitised utilising both the LiDAR survey DEM and high resolution aerial photography. The lowest LiDAR elevations in the vicinity of the centrelines were extracted to produce long profiles of the channel topography. Available survey elevation data for the channel bed and structure inverts was extracted and compared to the LiDAR long profiles. The results of this comparison for the modelled lengths of the various tributaries of Coffs Creek are presented in Figure 3-2 to Figure 3-5. The chainage reference locations for these and subsequent long sections in the report are presented in Figure 3-6. All long sections have been reproduced at A3 size in Appendix B.

The long profiles show that interference from riparian vegetation has resulted in the LiDAR survey failing to adequately capture the low-flow channel topography. The high degree of spatial variation in the LiDAR elevations is evident, particularly along the Coffs Creek Main Arm. It can be seen that the channel bed elevation consistently lines below the LiDAR long section, across all channel reaches presented. A channel long section line of best fit that removed some of this variability by tracking along the average LiDAR elevations was derived. This channel bed elevation line was then shifted as appropriate to provide a good fit to surveyed channel bed elevations. The resulting adopted channel elevation is also presented on the figures. Downstream of the Pacific Highway the available cross sections were used to define the channel profile, as the deep water results in the LiDAR elevations becoming unrepresentative. Further detail on this process is contained in Section 4.2.4.

It was identified that structural details and channel cross section data was unavailable for a reach of the Northern Tributaries Argyll Street Arm between Joyce Street and Mackays Rd (approximately 1.5km). Given the good coverage of channel cross section data available across the remainder of the study area and constant results of the analysis, the requirement for additional channel cross sections was not vital. However, as missing structural details were required to be surveyed along this reach it was decided that obtaining some additional channel cross sections would be worthwhile to provide a comprehensive coverage of data across the study area.



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Location of Channel Cross Section Data

Figure: 3-1

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Approx. Scale

0 0.5 1km

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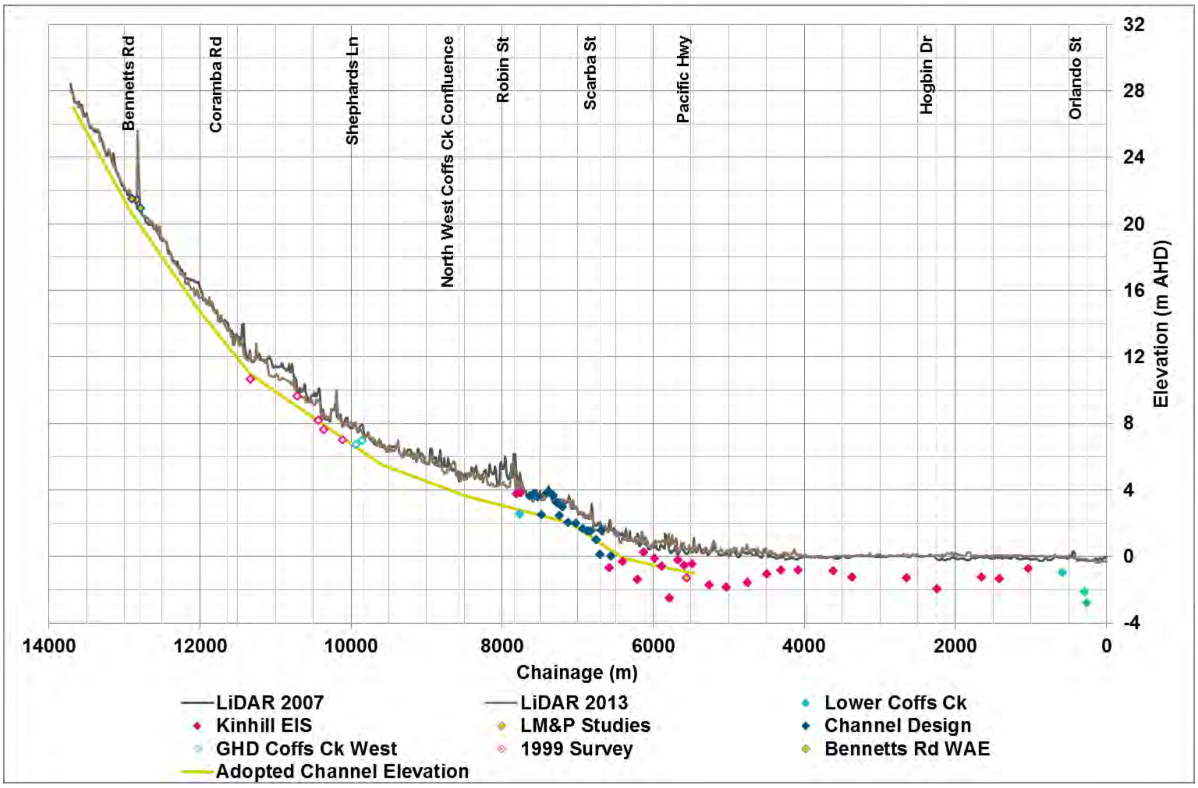


Figure 3-2 Assessment of LiDAR Survey Representation of Coffs Creek (Main Arm) Channel Topography

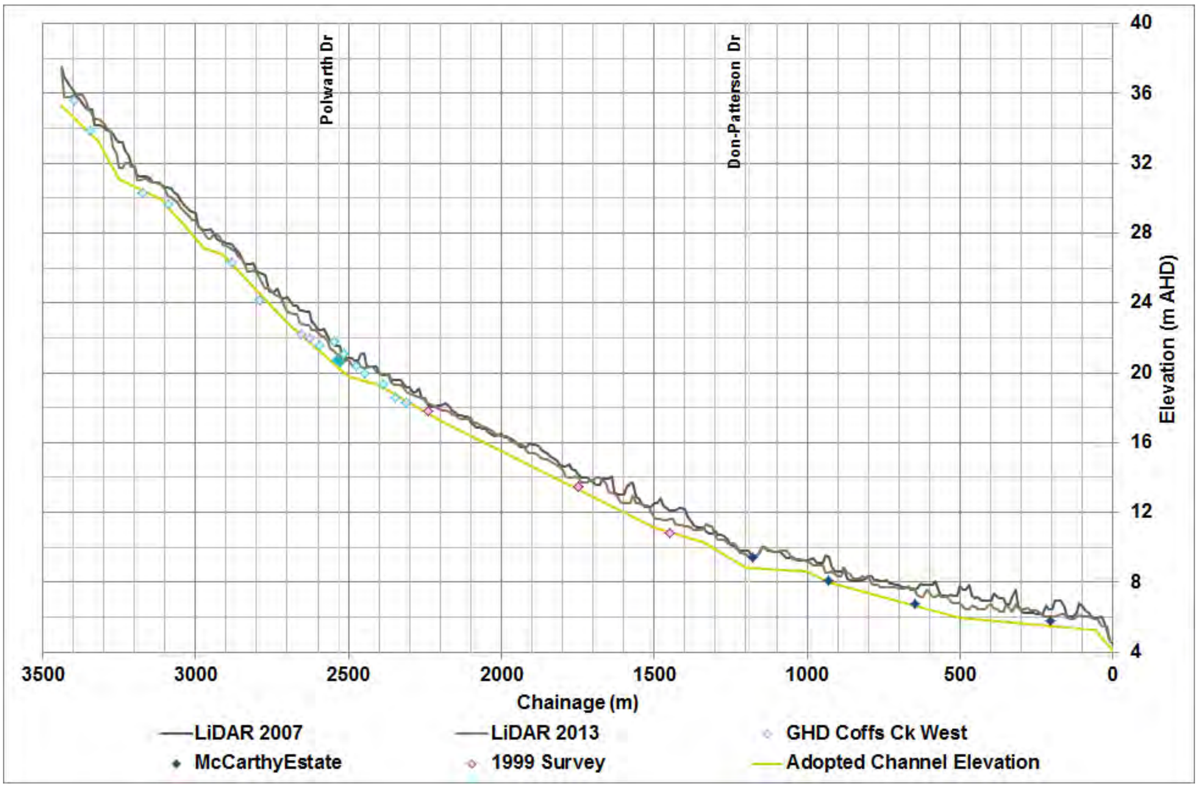


Figure 3-3 Assessment of LiDAR Survey Representation of Coffs Creek (North West Arm) Channel Topography

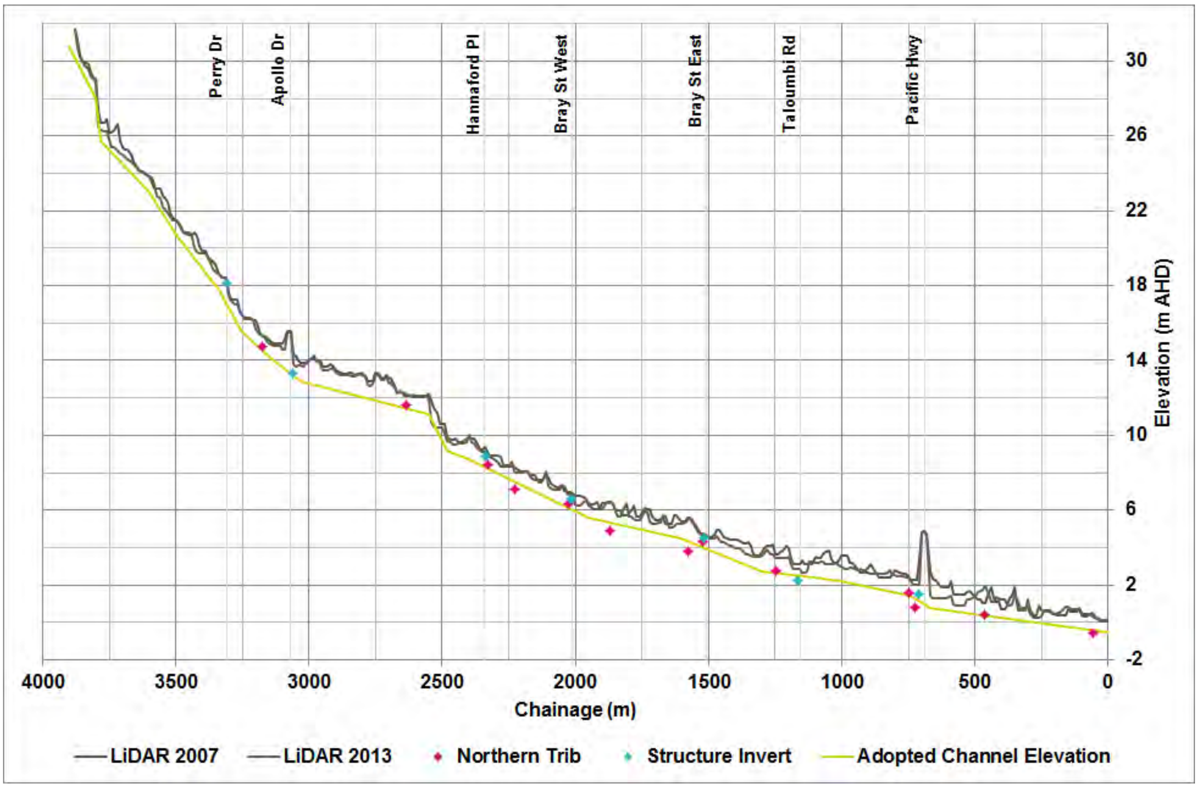


Figure 3-4 Assessment of LiDAR Survey Representation of Coffs Creek Northern Tributaries (Bray Street Arm) Channel Topography

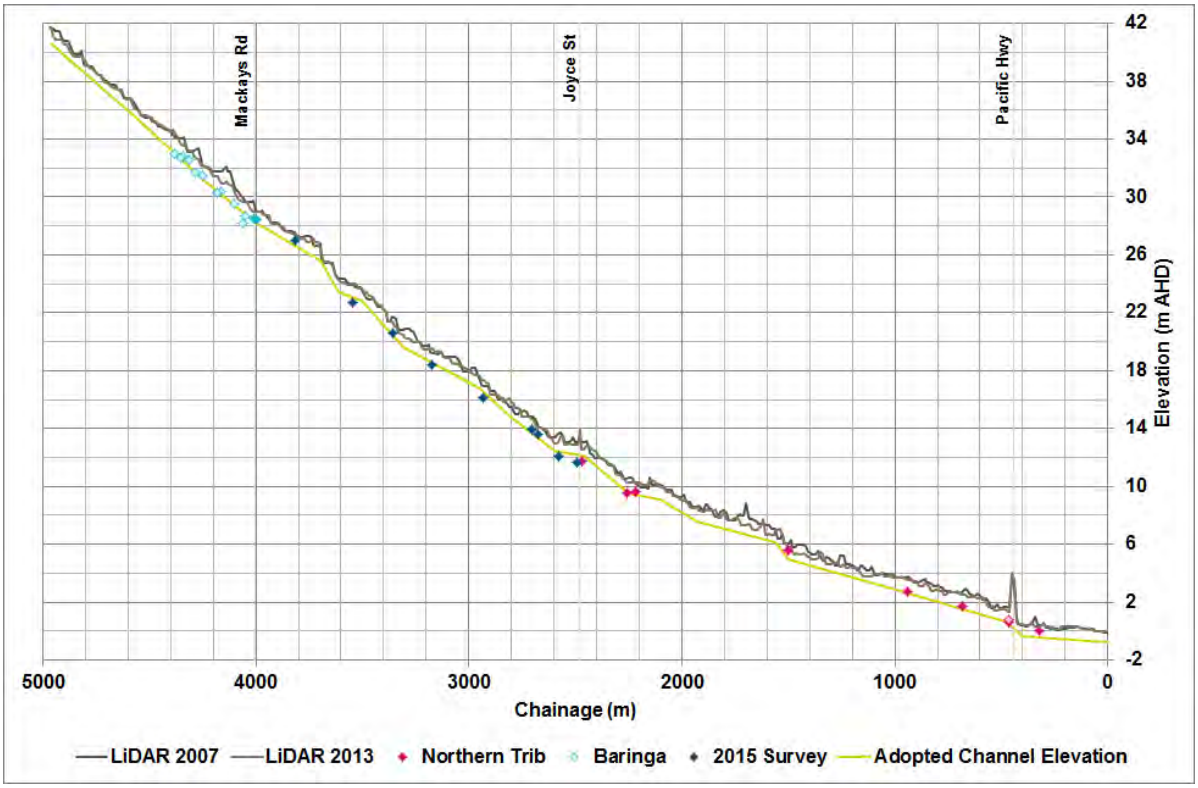
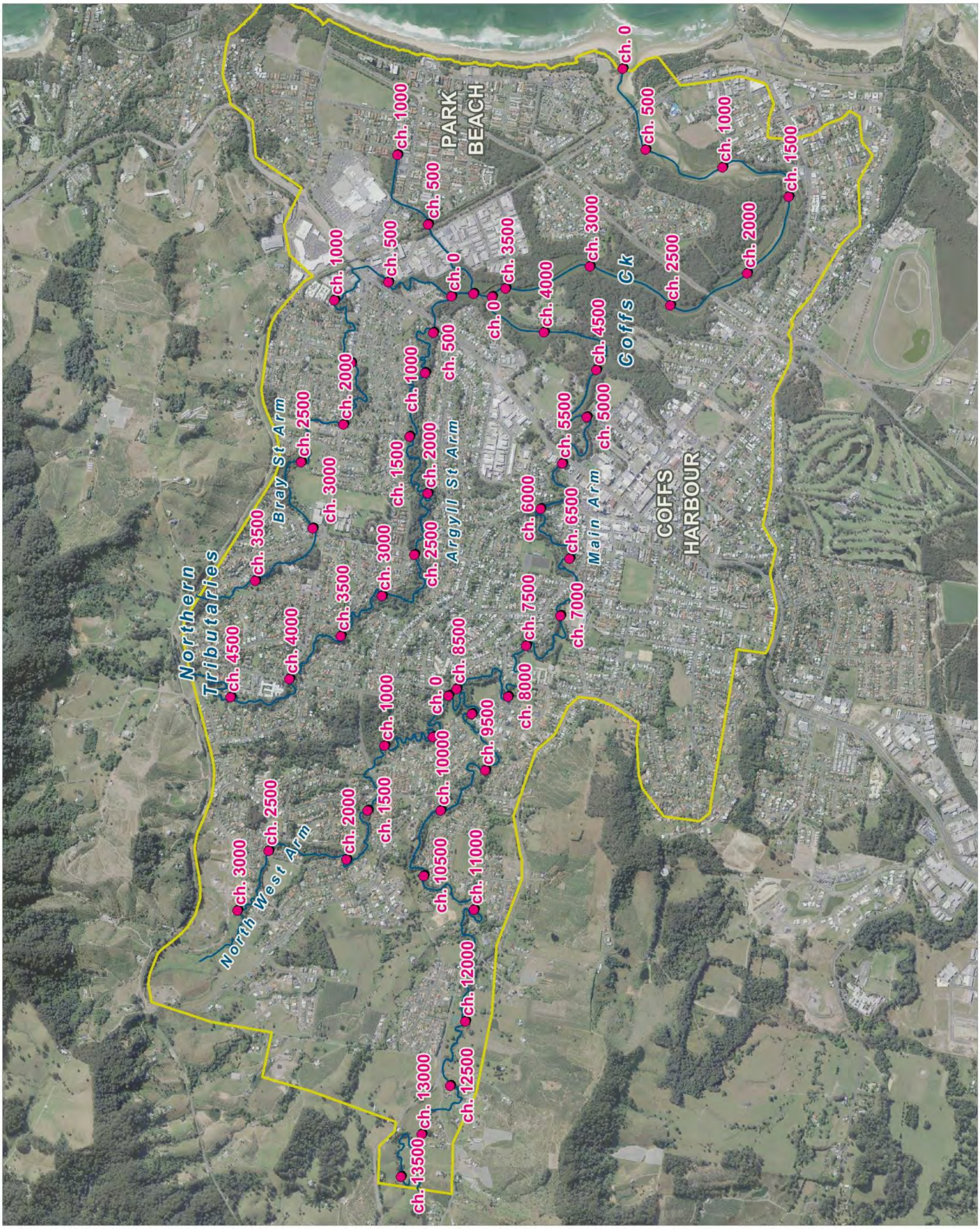


Figure 3-5 Assessment of LiDAR Survey Representation of Coffs Creek Northern Tributaries (Argyll Street Arm) Channel Topography



Title:

Chainage Reference Locations

Figure:

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Approx. Scale

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4 Model Development

4.1 Hydrological Model

4.1.1 Flow Path Mapping and Catchment Delineation

The Coffs Creek catchment drains an area of approximately 25km^2 to the Tasman Sea. For the hydrological model this area has been delineated into 97 sub catchments as shown in Figure 4-1. The sub catchment delineation provides for generation of flow hydrographs at key confluences or inflow points to the hydraulic model.

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 consider the proportion of catchment that consists of dense vegetation, cleared land and urban development. As indicated in the table and evident from aerial photography, the majority of the mid to lower catchment is urban development. The upper catchment is primarily cleared land use, with generally only the upmost catchment fringes being forested. Urban sub catchments have been modelled using a second sub catchment approach, where the impervious areas are treated separately. The PERN value for these impervious areas has been set to 0.015 accordingly.

Given that the town of Coffs Harbour has undergone extensive development since the late 1970s, the hydrological model will need to reflect the level of urbanisation within the catchment during each calibration event. Parameters were adjusted where appropriate to represent catchment conditions for the 1996 event. For the 2009 and 2015 calibration and verification events, the current level of catchment development was assumed. The sub catchment parameters in Table 4-1 represent current catchment conditions.

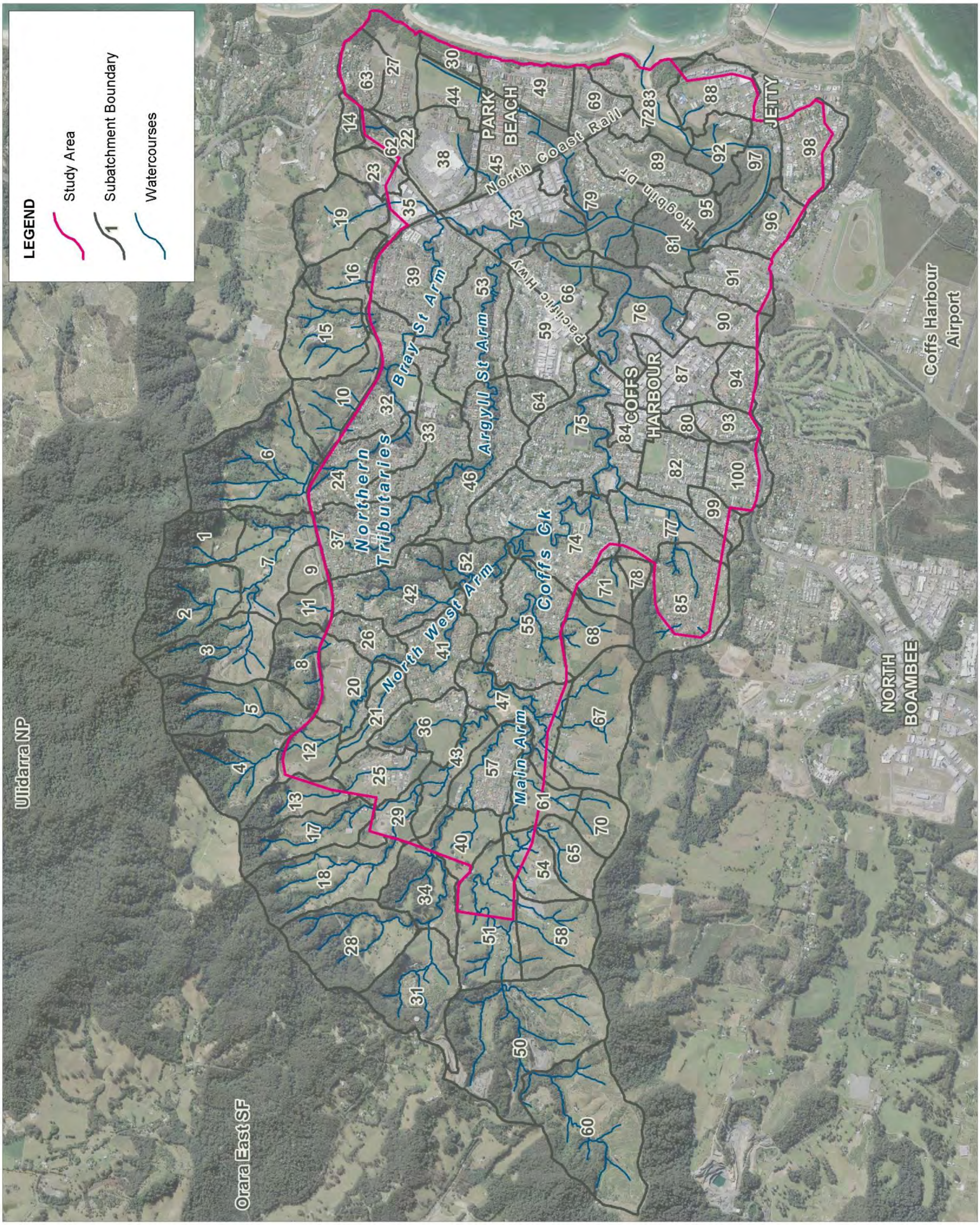
4.1.2 Rainfall Data

Rainfall information is the primary input and driver of the hydrological model, which simulates the catchments response in generating surface run-off. 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 36 hours 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.



Title:

RAFTS Model Subcatchment Layout

Figure: **4-1**

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Approx. Scale

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Table 4-1 RAFTS Sub Catchment Properties

Catchment ID	Area (ha)	Slope (%)	PERN	Catchment ID	Area (ha)	Slope (%)	PERN	Catchment ID	Area (ha)	Slope (%)	PERN
1	20.9	25.5	0.09	34	26.5	2.9	0.09	68	17.2	16.7	0.06
2	34.6	20.3	0.10	35	9.9	2.3	0.04	69	14.0	0.3	0.05
3	33.5	13.5	0.08	36	23.4	2.3	0.06	70	18.7	18.3	0.06
4	34.3	13.2	0.08	37	38.3	1.6	0.05	71	12.5	17.6	0.06
5	37.7	12.9	0.08	38	30.6	1.6	0.05	72	4.3	0.7	0.05
6	57.3	7.1	0.08	39	49.6	0.5	0.05	73	41.4	0.6	0.07
7	33.5	2.8	0.07	40	19.2	1.8	0.06	74	59.0	0.5	0.05
8	19.2	18.4	0.08	41	25.4	1.7	0.05	75	54.2	0.1	0.05
9	7.1	9.0	0.06	42	38.4	2.3	0.06	76	42.8	2.5	0.08
10	24.9	3.5	0.06	43	18.6	0.9	0.07	77	30.1	1.0	0.05
11	9.2	13.5	0.06	44	15.7	0.2	0.05	78	4.5	24.3	0.08
12	16.1	6.0	0.06	45	30.4	0.4	0.06	79	36.7	1.0	0.08
13	18.0	8.8	0.07	46	41.8	1.3	0.06	80	4.9	0.6	0.04
14	6.0	21.7	0.08	47	29.3	0.7	0.06	81	20.3	0.4	0.10
15	32.9	7.9	0.06	49	23.1	0.3	0.05	82	19.8	0.6	0.04
16	17.6	9.2	0.06	50	83.5	7.6	0.08	83	26.1	4.7	0.05
17	27.0	7.2	0.08	51	39.4	1.6	0.06	84	9.7	0.2	0.04
18	41.5	6.3	0.07	52	24.0	1.1	0.07	85	44.4	4.8	0.06
19	29.4	5.6	0.07	53	37.9	0.8	0.06	87	26.9	1.0	0.04
20	23.4	2.1	0.05	54	17.8	4.7	0.06	88	25.0	1.1	0.05
21	13.0	2.2	0.06	55	45.2	0.7	0.05	89	24.5	0.7	0.06
22	5.9	1.7	0.04	57	23.4	0.7	0.06	90	18.4	1.4	0.05
23	11.3	6.0	0.06	58	29.4	15.3	0.06	91	22.0	1.2	0.05
24	29.9	2.0	0.05	59	31.2	1.1	0.04	92	16.9	0.6	0.09
25	19.8	2.6	0.05	60	67.4	13.0	0.07	93	11.0	2.2	0.04
26	13.0	2.8	0.05	61	19.2	1.3	0.06	94	13.4	2.2	0.04
27	19.7	1.2	0.07	62	5.7	1.2	0.06	95	10.6	0.5	0.09
28	44.0	6.6	0.09	63	15.1	4.6	0.06	96	16.7	3.6	0.05
29	11.9	2.5	0.06	64	10.0	3.9	0.05	97	24.0	1.9	0.09
30	7.6	0.3	0.09	65	15.7	11.9	0.06	98	26.0	3.9	0.04
31	37.7	7.7	0.08	66	25.6	0.3	0.07	99	9.0	2.2	0.04
32	30.9	0.9	0.05	67	46.1	9.6	0.06	100	20.6	0.9	0.04
33	31.0	1.5	0.05								

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.

Coffs Harbour has experienced a number of extreme rainfall events and the nature of the local topography has been identified as being a major contributor. To reflect this, the 2001 Flood Study recommended a scaling of design rainfall based on the AR&R 1987 IFDs with a rainfall gradient applied based on elevation. A modified version of this approach was adopted for this study.

The rainfall inputs for the historical calibration/validation events are discussed in Section 5. Further detail concerning design rainfall estimates, including the background to and implementation of rainfall scaling factors is described in Section 6.

4.2 Hydraulic Model

4.2.1 Topography

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. In addition to the 2007 LiDAR survey provided by Council, BMT WBM had previously purchased LiDAR data covering the study area collected in 2013. Of particular benefit to this study is that in the years between each data set, numerous detention basins and other local works (levees and channel modifications) have been constructed across the catchment. For the Coffs Creek catchment, a 2m resolution gridded DEM was principally derived from the 2013 LiDAR data set, with components of the 2007 LiDAR utilised for calibration events.

As discussed in Section 3, cross section survey of the watercourses was required to supplement the LiDAR data and provide the necessary detail on channel shape and dimensions for representation in the hydraulic model. 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. detention basins, levees, bridges);
- Desired accuracy to meet the study’s objectives; and
- Computational limitations.

With consideration to the available survey information and local topographical and hydraulic controls, a 2D model was developed extending from the Coffs Creek entrance at the downstream limit, upstream along the major tributary routes. An important consideration in defining the hydraulic model extent was the location of detention basins within the catchment. As defined in the Study Brief, the 2D model domain extends a sufficient distance upstream of each detention basin, so the influence of each could be assessed within the hydraulic model. The model incorporates the Coffs Creek main arm to just upstream of the Bennetts Road detention basin, some 13.7km in length. The northern boundary of the hydraulic model is defined by the embankment of the North Coast railway line. The North West arm of Coffs Creek extends 3.4km upstream from the confluence with the main arm of Coffs Creek to the railway, encompassing the location of the proposed Upper Shephards Lane detention basin. The modelled length of the Argyll Street arm and Bray Street arm of the Northern Tributaries are 5km and 3.9km respectively. The area modelled within the 2D domain comprises a total area of some 14.8km^2 which represents the lower 60% of the Coffs Creek catchment. The extent of the hydraulic model within the Coffs Creek catchment was presented in Figure 2-1 along with the location of each detention basin.

A TUFLOW 2D domain model resolution of 4m was adopted for study area. It should be noted that TUFLOW samples elevation points at the cell centres, mid-sides and corners, so a 4m cell size results in DEM elevations being sampled every 2m. This resolution was selected to give necessary detail required for accurate representation of floodplain and channel topography and its influence on overland flows.

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

The LiDAR data provides an accurate representation of floodplain topography, but does not capture channel details below the water surface. To accurately represent channel dimensions and flow capacity, the channel network must be further represented within the hydraulic model. The approach adopted in this study involved embedding the channel topography within the 2D model domain. This provides several advantages over a 1D channel representation, including:

- A smoother transition between channel and floodplain conveyance;
- A more spatially rich representation of the high-flow in-channel flood conveyance, taking account of local topographic controls both at and beneath bank-full level;
- An inherent representation of the channel sinuosity;
- Spatial variation of velocities across the width of the channel; and
- Improved flood mapping output for in-channel areas.

Upstream of the Pacific Highway four distinct watercourses were modelled: the main arm of Coffs Creek, the North West arm of Coffs Creek and the two branches of the Northern Tributaries: the Argyll Street arm and the Bray Street arm.

Due to the different nature of the creek channel upstream and downstream of the Pacific Highway, two different methods were adopted to define the width of the channel bed. Upstream of the Pacific Highway, the channel was lowered by one cell width (4m) to allow for a continuous flow path along the creek alignment. Bed elevations were determined from the data analysis presented in Section 3. A sample cross sections of the modelled topography, derived from LiDAR data is provided in Figure 4-2.

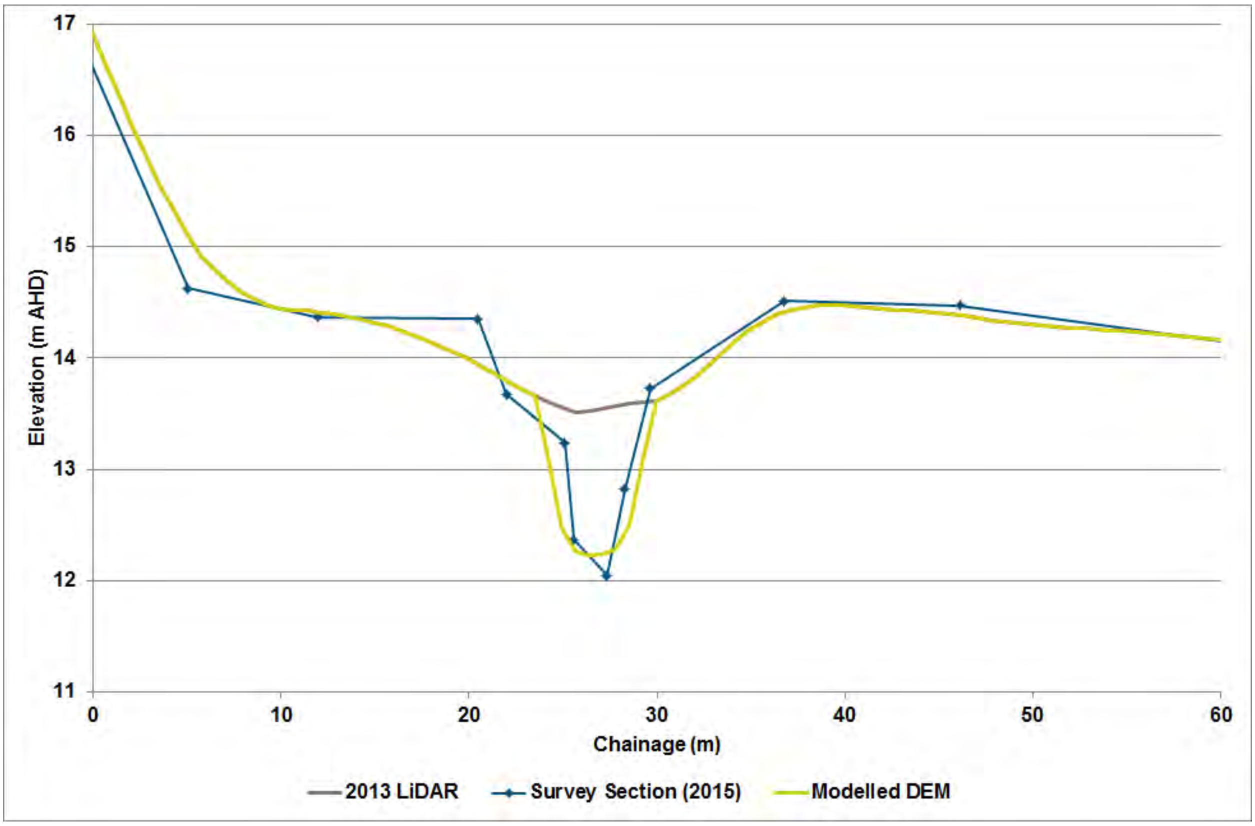


Figure 4-2 Sample Model Channel Section Derived from LiDAR Data Assessment (Upstream of Pacific Highway)

Downstream of the Highway, bed width is generally wider and more varied. Channel cross section survey information extracted from the RUBICON model developed for the Coffs Creek Flood Study (WMA, 2001) was used to define key elevation points (e.g. 0m AHD and -1m AHD) at each location. Interpolating between the points provided a smooth, continuous transition of channel width along the reach. A sample cross section of the modelled topography, derived from the cross section information is provided in Figure 4-3.

It can be seen that the flow areas of the modelled channel profiles are similar to those represented in the available cross section data.

4.2.5 Flood Mitigation Works

Flood mitigation works constructed across the catchment include levees, channel modification and detention basins. The model DEM was built from LiDAR data collected in 2013, which accurately defines topography of all existing features within the catchment. The crest height of the Loaders

Lane levee and the Collice/Langker Place levee were extracted from design drawings and has been reinforced in the 2D domain as 3d breaklines.

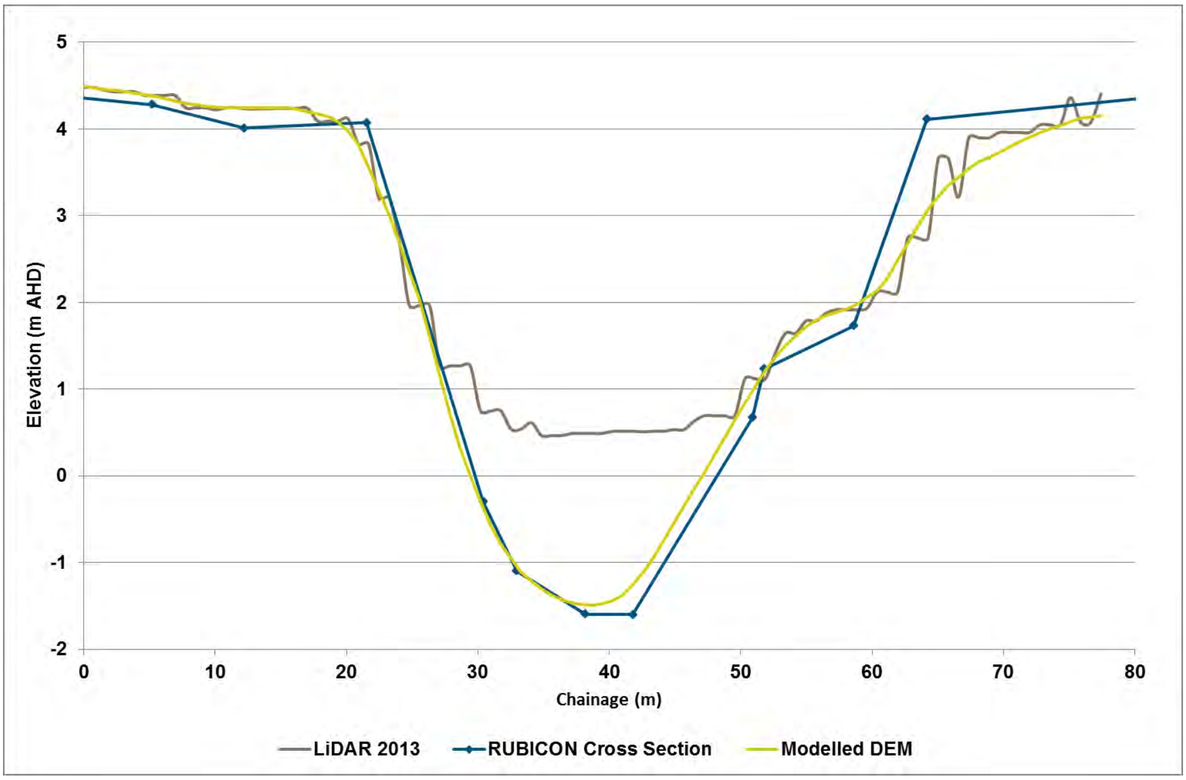


Figure 4-3 Sample Model Channel Section Derived from RUBICON Cross Section Data (Downstream of Pacific Highway)

There are multiple detention basins located within the study area. Detention basins constructed prior to 2013, including the basin located upstream of the Marcia Street industrial district, the Bakers Road basin located upstream of William Sharp Drive (constructed 2010) and the Bennetts Road basin (constructed 2012-2013) were surveyed in the LiDAR data set. Design drawings of the Spagnolos Road detention basin (constructed 2015) were provided by Council and allowed for the basin to be included within the topography. The stage-storage relationships for these basins are represented by the topography of the 2D model domain. The outlet structures are represented in the modelled 1D stormwater drainage network and are detailed in Table 4-2. The location of each detention basin was shown on Figure 2-1.

Table 4-2 Detention Basin Outlet Details

Basin Location	Status	Outlet Configuration
McLean Street	Constructed	Discharges into stormwater drainage network: • Three grated surface inlet pits draining to a 1.5m diameter RCP • One grated surface inlet pit draining to a 1.8m diameter RCP
Arthur Street	Constructed	Discharges into stormwater drainage network: • One 0.6m diameter RCP (with surface inlet pit), joining to one 1.05m diameter RCP (with surface

Basin Location	Status	Outlet Configuration
		inlet pit)
Marcia Street	Constructed	Discharges into stormwater drainage network: • One 0.525m diameter RCP (with headwall) • Invert level assumed
Bakers Road	Constructed	Low flow outlet: • One 3000 x 2400 RCBC (entrance plate reduces opening to 0.9m high) • Invert 11.2m AHD, grade 0% High flow outlet: • Three 3000x3000 RCBC on side • Invert at 17.1m AHD
Bennetts Road	Constructed	• One 3300 x 1200 RCBC, approx 54m long • US invert 21.46m AHD, DS invert 20.92m AHD
Spagnolos Road	Constructed	• Two 1.05m diameter RCP, 76.5m long • US invert 18.0m AHD, DS invert 16.4m AHD
Upper Shephards Lane	Future construction unconfirmed	• 2700 x 900 RCBC, approx 66m long • Invert levels assumed

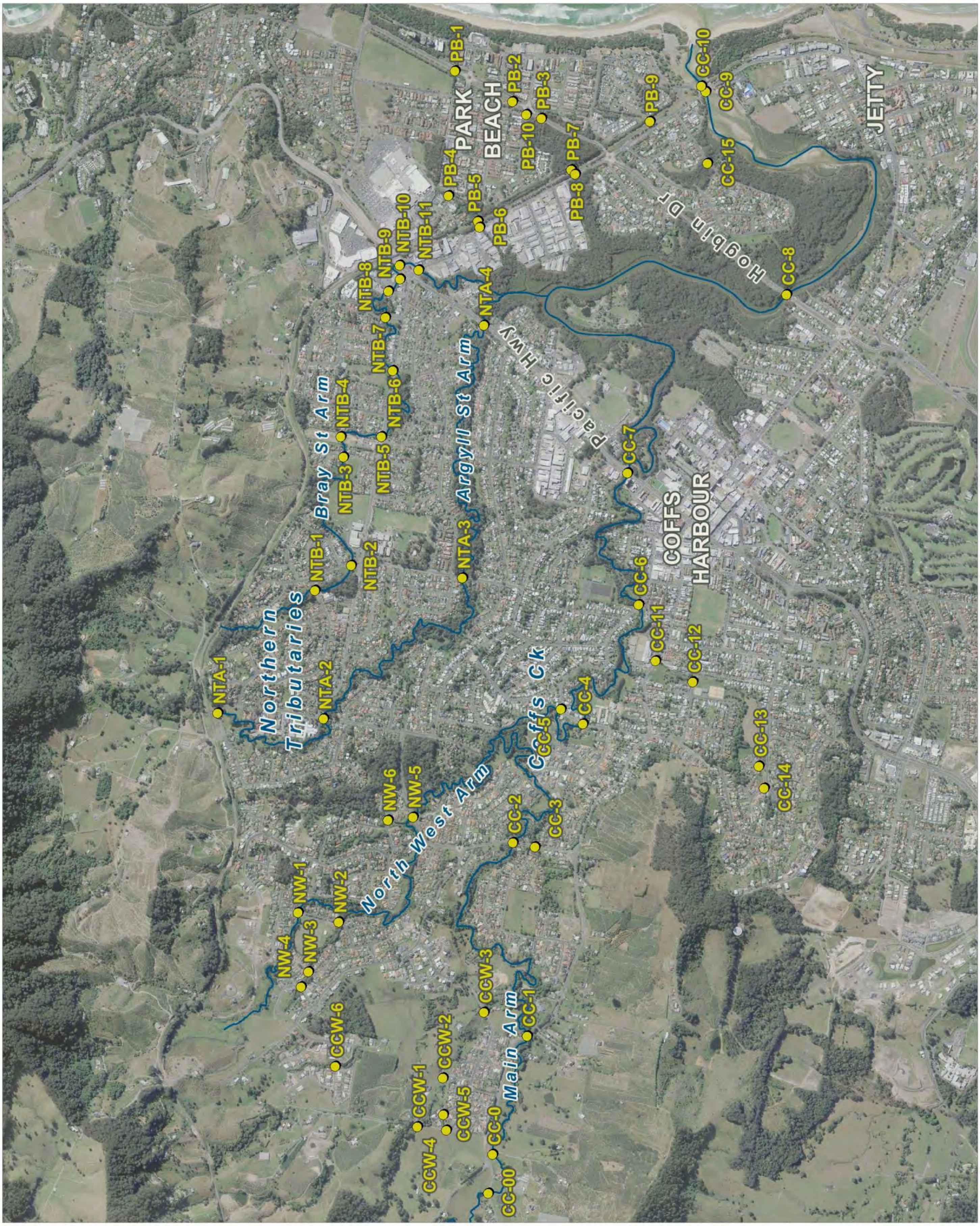
4.2.6 Structures

There are a number of bridge and culvert crossings over the main channel alignments and tributaries within the model extents, as shown in Figure 4-4 and detailed in Table 4-3. These structures vary in terms of construction type and configuration, with varying degrees of influence on local hydraulic behaviour. Incorporation of these hydraulic structures in the models provides for simulation of the hydraulic losses associated with these structures and their influence on peak water levels within the study area.

Bridge structures and larger culvert crossings have been modelled as flow constrictions within the 2D domain. Smaller culverts, where the flow width is typically less than one cell wide, have been modelled using 1D structures to provide flow through embankments represented within the 2D domain.

The North Cost railway line traverses the northern boundary of the study area. There are numerous culverts providing drainage through the railway embankment. The potential for the embankment to attenuate flood flows from the upper catchment into the study area was assessed in the XP-RAFTS hydrological model using the retarding basin functionality. The relationship between storage volume and water level behind the rail embankment was calculated from the DEM. Culvert details were entered as the discharge outlet and the rail embankment was represented as a spillway within the XP-RAFTS sub-catchment node.

The total catchment upslope of sub catchment ID7 (refer to Figure 4-1) has an area of around 0.9km^2 and is drained by two large arch-shaped culverts (one 1.97m x 1.97m and one 2.1m x 2.1m). The analysis indicated that culverts can convey the peak flow rate for the 1% AEP event without any attenuation affects behind the embankment. The other culverts have a similar or



Title:
Modelled Structure Locations

Figure: **4-4**

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Approx. Scale

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Filepath : K:\N20327_Coffs_Creek\MapInfo\Workspaces\Figures\DRG_00X_1501XX_Structures.WOR greater flow area to catchment ratio and were assumed to also freely discharge flows from the upper catchment. Accordingly, the railway culverts were not included in either the hydrological or hydraulic model for calibration or design events.

Table 4-3 Modelled Hydraulic Structures

Structure ID	Street Crossing	Description	Dimensions
CC-00	Coramba Rd (west)	Concrete box culvert	1 x 3.3x1.2m (WxH)
CC-0	Unsealed road	Concrete box culvert and concrete pipes	1 x 3.0x1.8m (WxH) + 2 x 1.8m diameter
CC-1	Coramba Rd (east)	Concrete box culverts	2 x 3.5x3.6m (WxH)
CC-2	Shephards Ln	Bridge	
CC-3	Shephards Ln	Concrete pipes	1 x 1.35m + 2 x 1.5m diameter
CC-4	Robin St (south west)	Bridge	
CC-5	Robin St (north east)	Concrete box culverts	3 x 3.6x2.7m (WxH)
CC-6	Scarba St	Bridge	
CC-7	Pacific Hwy	Bridge	
CC-8	Hogbin Dr	Bridge	
CC-9	Orlando St	Bridge	
CC-10	Railway (adj Orlando St)	Bridge	
CC-11	West High St	Concrete pipes	3 x 1.5m diameter
CC-12	King St	Concrete pipe	1 x 1.5m diameter
CC-13	Airlie Cl	Concrete box culvert	1 x 3.0x1.2m (WxH)
CC-14	Archer Cl	Concrete pipe	1 x 1.35m diameter
CC-15	Brodie Dr	Concrete pipe	1 x 1.2m diameter (assumed)
CCW-1	Spagnolos Rd (north)	Concrete pipe	1 x 1.8m diameter
CCW-2	Rural crossing	Concrete pipes	6 x 1.05m diameter
CCW-3	William Sharp Dr	Concrete box culverts	2 x 3.1x1.8m (WxH)
CCW-4	Spagnolos Rd (south)	Concrete pipes	5 x 0.825m diameter
CCW-5	Roselands Dr (south)	Concrete pipes	5 x 0.825m diameter
CCW-6	Roselands Dr (north)	Concrete pipes	4 x 0.9m diameter
NW-1	Polwarth Dr (north)	Concrete box culverts	3 x 2.15x2.15m (WxH)
NW-2	Polwarth Dr (south)	Concrete pipe	1 x 1.5m diameter
NW-3	Sunnyside Cl	Concrete pipe	1 x 0.6m diameter
NW-4	Coriedale Dr	Concrete pipe	1 x 0.75m diameter
NW-5	Donn-Patterson Dr	Concrete box culverts	3 x 4.25x1.4m (WxH)
NW-6	Griffith Ave	Concrete pipe	2 x 1.5m diameter

Structure ID	Street Crossing	Description	Dimensions
NTA-1	Mackays Rd (north)	Concrete pipe	1 x 1.5m diameter
NTA-2	Mackays Rd (south)	Concrete box culverts	2 x 3.35x2.4m (WxH)
NTA-3	Joyce St	Bridge	
NTA-4	Pacific Hwy	Bridge	
NTB-1	Perry Dr	Concrete pipe	5 x 1.35m diameter
NTB-2	Apollo Dr	Concrete pipes	2 x 1.2m diameter
NTB-3	Hannaford Pl	Concrete box culverts	3 x 2.4x1.5m (WxH)
NTB-4	Unsealed road	Bridge	
NTB-5	Bray St (west)	Concrete box culverts	2 x 3.6x1.8m (WxH)
NTB-6	Bray St (east)	Concrete box culverts	4 x 3.0x1.2m (WxH)
NTB-7	Taloumbi Rd	Concrete box culverts	3 x 3.2x3.2m (WxH)
NTB-8	Cinema Carpark (north west)	Concrete box culverts	2 x 3.6x3.5m (WxH)
NTB-9	Cinema Carpark (south east)	Concrete pipes	2 x 3.6x3.5m (WxH)
NTB-10	Pacific Hwy	Concrete box culverts	2 x 3.0x2.4m (WxH)
NTB-11	Orlando St	Bridge	
PB-1	Hogbin Dr	Concrete box culverts	2 x 1.2m diameter
PB-2	Hogbin Dr	Concrete pipes	5 x 0.6m diameter
PB-3	Hogbin Dr	Concrete pipes	3 x 1.2m diameter
PB-4	Park Beach Rd	Concrete pipes	4 x 1.8m diameter
PB-5	North Coast Railway (north west)	Concrete box culvert	1 x 3.0x2.7m (WxH)
PB-6	Orlando St (north west)	Bridge	
PB-7	North Coast Railway (south east)	Concrete box culvert	1 x 1.2x0.9m (WxH)
PB-8	Orlando St (south east)	Concrete pipe	1 x 1.2m diameter
PB-9	North Coast Railway (east)	Concrete pipe	1 x 1.2x0.8m (WxH)
PB-10	Prince St	Concrete pipes	5 x 0.6m diameter

4.2.7 Drainage Network

The study requires the modelling of the drainage system in some of the urban sub catchments. Council provided information on the existing drainage system where modelling was required. This data comprised GIS data detailing pit/pipe locations, pit inlet type/dimensions and pipe sizes.

Trunk drainage lines with diameter greater than 750mm were considered for inclusion in the modelling and were incorporated (along with smaller pipes if required) where deemed to be of significant influence on the modelled mainstream flooding conditions. Invert levels were taken from design or as-constructed drawings. Where invert levels were not available, they were estimated from the DEM, by assuming a minimum cover of 600mm from the known pipe size.

The pipe network, represented as a 1D layer in the model, is dynamically linked to the 2D domain at specified pit locations for inflow and surcharging.

4.2.8 Boundary Conditions

The catchment runoff is determined through the hydrological model and is applied to the TUFLOW model as flow vs. time inputs. These are applied at the upstream modelled watercourse limits and also as distributed inflows along the modelled watercourse reaches. In the urban sub catchments with modelled stormwater drainage, the hydrological model inflows are applied directly to the 1D pipe network and will surcharge to the 2D surface representation when pipe full capacity is exceeded.

The downstream model limit corresponds to the water level in the Tasman Sea. The adopted water levels for the downstream boundary condition for the calibration and design events are discussed in Section 5 and Section 6 respectively.

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.

In recent years, both the March 2009 and November 1996 events were major flood events in the Coffs Creek catchment. Following both events, extensive data collection was carried out resulting in a comprehensive coverage of surveyed flood levels, photographs and personal recounts to be used for the calibration of both events. The 2009 event has been selected as the principal calibration event for the model for the following reasons:

- More comprehensive coverage of rainfall records during the event;
- Catchment topography during 2009 will be closer to 2013 LiDAR data given that extensive development within the catchment has occurred since 1996;
- Better coverage of surveyed flood marks within Park Beach; and
- Official MHL stream gauge recorded the entire event.

Due to the uncertainty surrounding catchment topography as a result of development between the 1996 and 2009 events, the November 1996 event will be used to validate the model.

In March 2015, Park Beach and areas along the Northern Tributaries of Coffs Creek were flood affected due to localised heavy rainfall. Flood marks collected by Council following the 2015 event and the installation of additional continuous rainfall gauges in 2010 mean there is sufficient calibration data for Park Beach for the March 2015 event. This event was therefore used to validate the models performance in Park Beach.

5.2 March 2009 Model Calibration

5.2.1 Rainfall Data

Three pluviographs operated by MHL recorded the March 2009 event within the Coffs Creek catchment. The Council operated gauge at the Catholic Club ceased operations following the November 1996 event. The Coffs Harbour Airport gauge operated by BoM recorded the event; however, the gauge displays some uncertainties.

Rainfall radar data was acquired for the March 2009 event from the Grafton radar station, located around 70km to the north-west. Analysis of the radar data against reliable pluviograph records presents a close match at the Red Hill, Shephards Lane and Perry Drive gauges. The comparison between recorded and radar derived temporal patterns at the Coffs Harbour Airport gauge is shown in Figure 5-1. The consistent rainfall depths recorded at the pluviograph gauge beyond 4pm look questionable. Given the closer match observed to the radar data in the hours prior and that the radar was comparable at the other gauge locations, uncertainty surrounds the accuracy of these recordings. Although total rainfall depths derived from radar data are not entirely accurate, the total rainfall depth for the 24 hours to 9am on the 1st April at Coffs Harbour Airport estimated from the radar data sums to over 280mm compared to the official recorded total of 161mm, which is more consistent with the private gauge rainfall totals recorded in the vicinity of the Airport (seen in Figure 5-2).

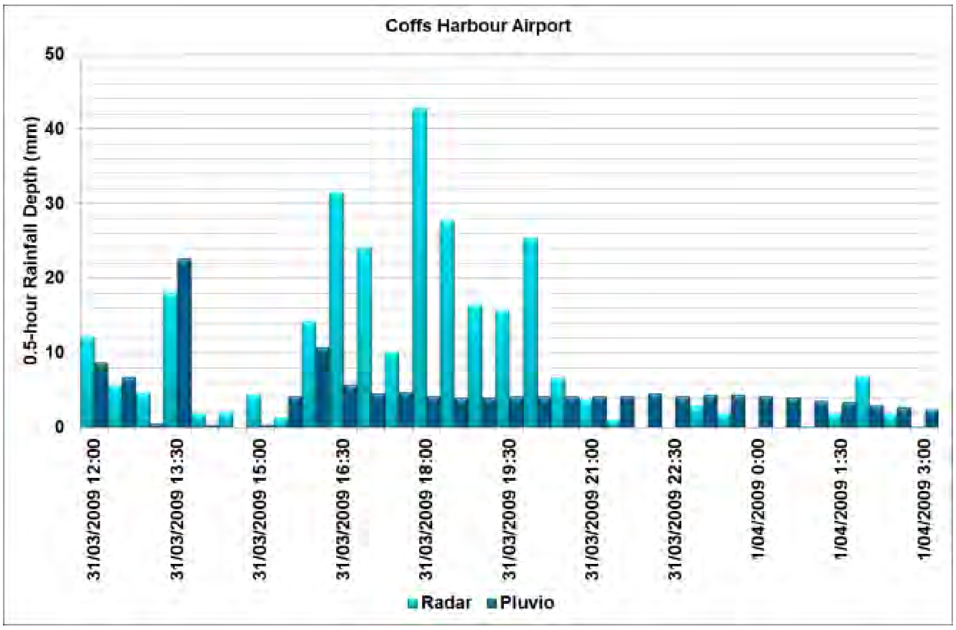


Figure 5-1 Comparison Between Recorded and Radar Derived Temporal Pattern

Following the 31st March – 1st April 2009 event, data collection was carried out by Council. Community questionnaire responses provided 24 rainfall records inside and within close proximity to the Coffs Creek catchment. Most records were reported as daily totals and were read on the morning of the 1st April, typically between 7-10am. Although the data may not be entirely accurate, it can be used as a guide to estimate the spatial distribution of rainfall intensity across the catchment during the event.

An analysis of the available rainfall data was completed by WMAwater (2013). The official pluviograph data was supplemented with private gauge rainfall totals to produce an isoheytal map for the 24 hours to 9am on the 1st April. The private rainfall records were reviewed in this current study. It is believed that nine reported rainfall depths are ambiguous or erroneous. Along with the four official continuous rainfall records in and around the catchment, there are 16 reliable private rainfall records providing comprehensive data coverage across the mid-lower catchment.

Rainfall depth isohyets for the March 2009 event are shown on Figure 5-2 along with the location of official rainfall gauges and private rainfall records. The historical rainfall analysis completed for the rainfall gradient revision (see Section 6.2.2) was drawn on to inform the allocation of the total rainfall zones presented in the figure, particularly for the upper catchment area. Across the lower catchment, there is sufficient coverage of privately read rainfall totals able to be used to approximate the rainfall distribution.

As uncertainty lies in the intensity of rainfall on the escarpment of the upper catchment, an alternate rainfall distribution was also simulated, to assess model sensitivity to spatial distribution of rainfall. The previous study (WMA, 2001) adopted a relationship between topographical elevation and expected rainfall depths relative to the Coffs Harbour Airport. The alternate simulation extrapolated recorded rainfall totals to upper catchment areas based on this previous relationship.

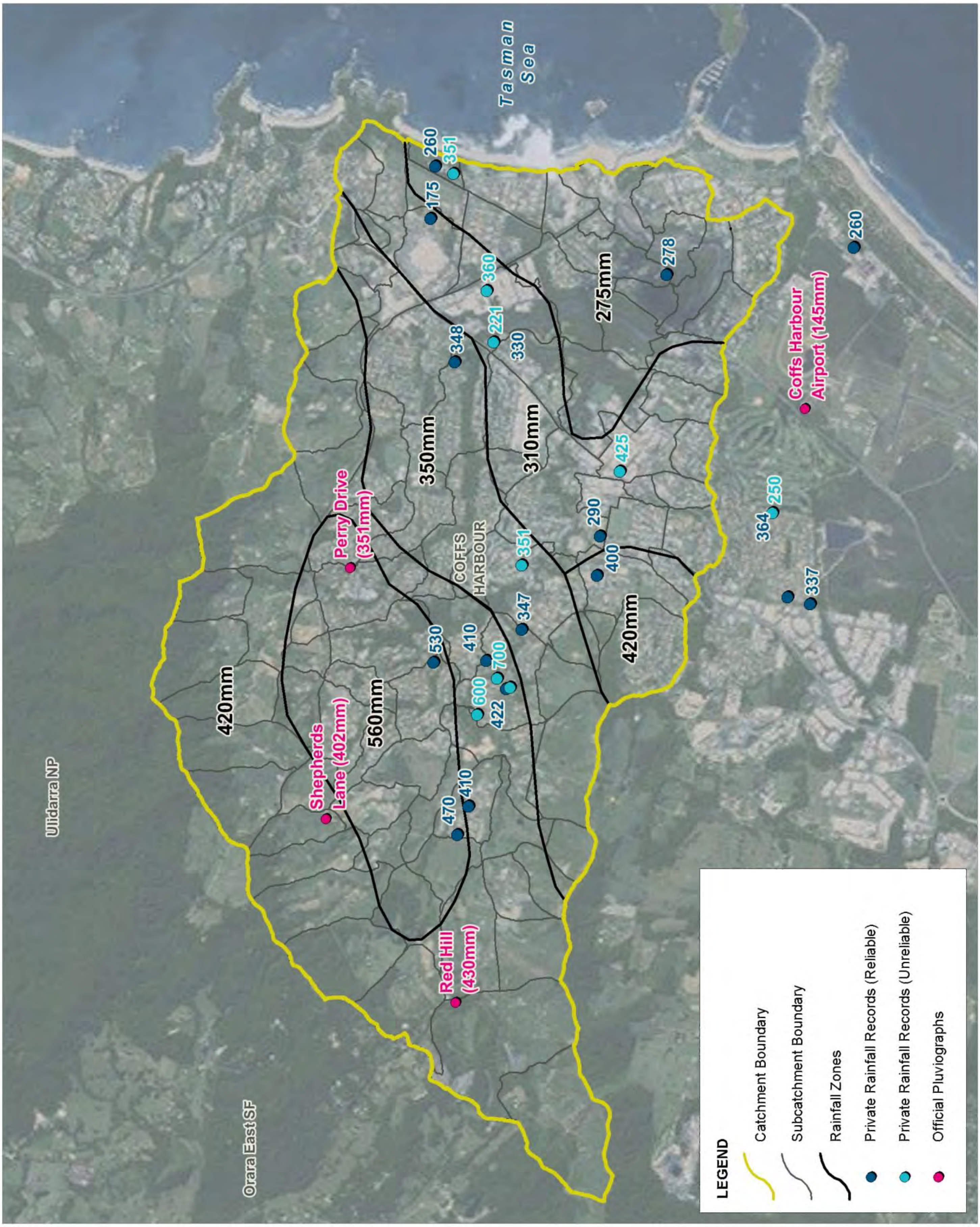
The half-hourly rainfall hyetograph recorded at each gauge in the catchment on the 31st March 2009 is presented in Figure 5-3. Red Hill and Shephards Lane are located within the main arm catchment of the Coffs Creek and Perry Drive is located in the Northern Tributaries catchment. The spatial variation of rainfall across the catchment is quite significant given the close proximity of each gauge (i.e. all lie within less than 4km of each other). In the Coffs Creek main arm catchment, rainfall intensities peak over a 1 hour period in the second hour of the storm before fairly constantly decreasing over the remaining seven hours. Higher rainfall intensities were recorded at Red Hill, particularly in the 1.5 hours between 2:00-3:30pm. The temporal pattern recorded at the Perry Drive gauge shows two peaks of rainfall. In comparison to the gauges at Red Hill and Shephards Lane, less rainfall was recorded in the first half of the storm, with more rainfall recorded in the second half. Two bursts of high intensity rainfall were recorded between 1:00-1:30pm and 4:00-4:30pm. The onset of the first burst occurred half an hour after heavy rainfall was first recorded at Red Hill and Shephards Lane. The second burst in the Northern Tributaries catchment occurred when rainfall within the Coffs Creek main arm catchment was easing.

In order to gain an appreciation of the relative intensity and magnitude of the March 2009 event, the recorded rainfall depth for various durations within the storm is compared with the Intensity Frequency Duration (IFD) data for the catchment. The AR&R is in the process of revising the design flood estimate guidelines, and have released updated 2013 IFDs based on the extended history of rainfall records available since they were first developed in 1987. 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 and design losses (the revision of which is still underway). Design IFD rainfall curves were obtained from AR&R (2001) based on the 1987 and 2013 datasets. Figure 5-4 presents the recorded March 2009 rainfall intensities against both the 1987 IFDs and 2013 IFDs, for comparison.

With reference to Figure 5-4, the increase in short duration (less than 12 hour events) rainfall depths for higher frequency events (the 1% AEP, 0.5% AEP and 0.2% AEP design events) between the 1987 IFDs and 2013 IFDs can be attributed to two of the largest rainfall events on record in Coffs Harbour having occurred since original estimates were developed in 1987.

Based on the 1987 IFDs, the rainfall depth recorded at Shephards Lane was consistently equal to the 0.2% AEP depth for durations up to 6 hours. Red Hill follows a similar curve up to around 3 hour duration. The critical duration at Red Hill is between a 3 and 9 hour duration, where rainfall exceeding the 0.2% AEP design depths were recorded. Rainfall recorded at Perry Drive was lower than the 5% AEP for durations shorter than 3 hours, steadily increasing to just below the 0.5% AEP for the 9 hour duration. Based on 2013 IFDs, the March 2009 event is more aligned with a 0.5% AEP event at Red Hill and Shephards Lane and a 1% AEP event at Perry Drive.

The total rainfall applied to each sub catchment within the hydrological model was approximated from the estimated distribution presented in Figure 5-2. The Red Hill, Shephards Lane or Perry Drive temporal pattern was adopted for each sub catchment, based on distance to the gauge and similar topographic characteristics.



Title:
Rainfall Analysis, 24hrs to 9AM 1st April 2009

Figure:
5-2

Rev:
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Approx. Scale
0 0.75 1.5km

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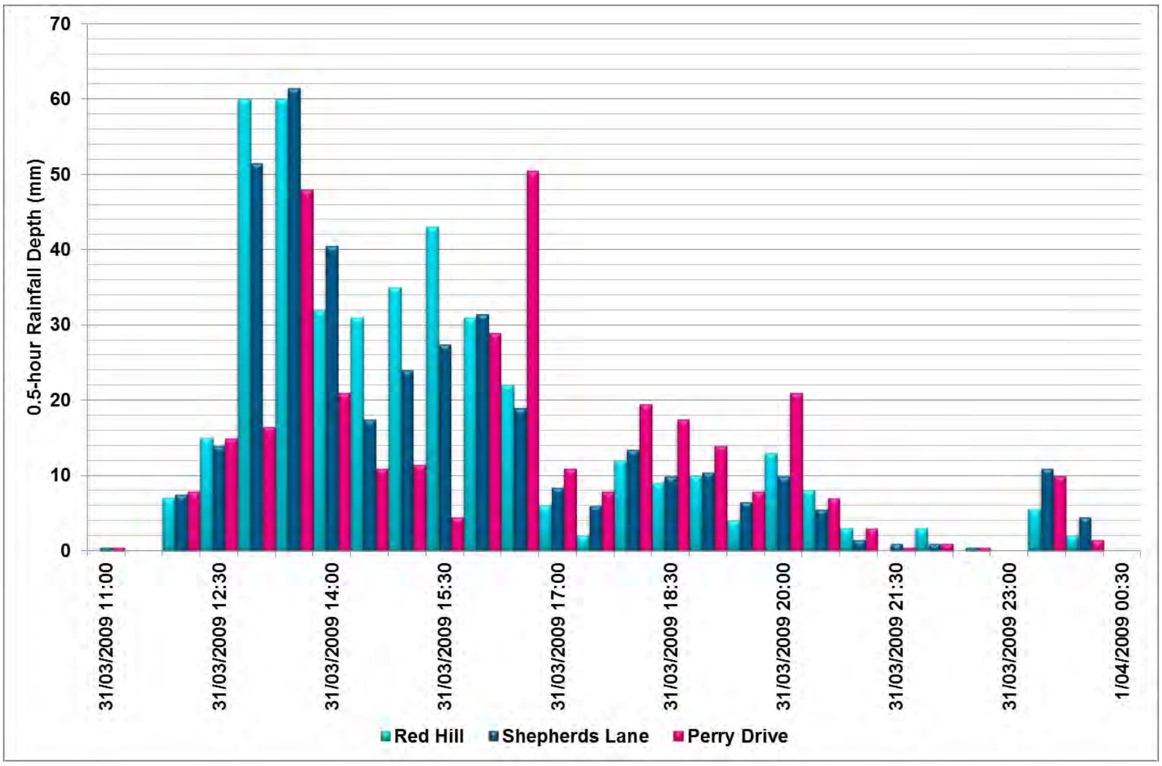


Figure 5-3 Rainfall Hyetograph for the March 2009 Calibration Event

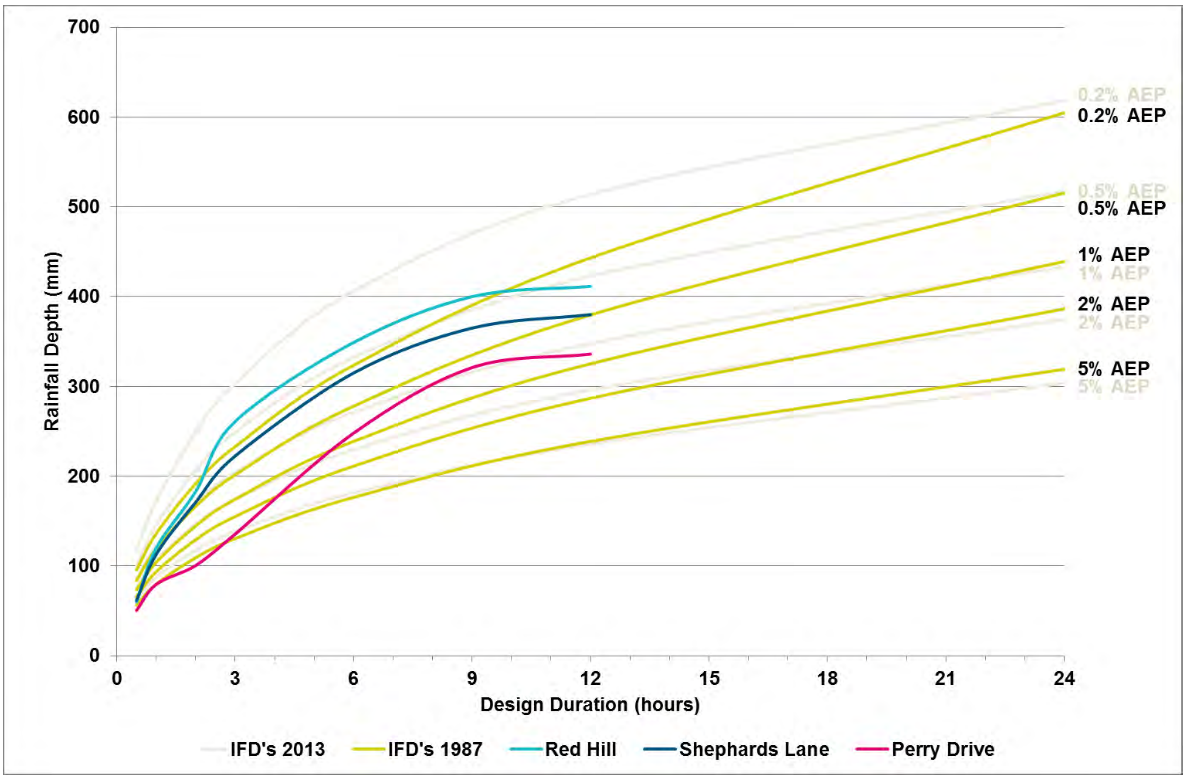


Figure 5-4 Comparison of Recorded March 2009 Rainfall with IFD Relationships

5.2.2 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 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/h) for the duration of the runoff event.

Typical design loss rates applicable for eastern NSW catchments are initial loss of 10mm to 35mm 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.

Rainfall records indicate that in the 24 hours to 9am on the 31st March (i.e. the day before the event) the gauges at Red Hill, Shephards Lane and Perry Drive recorded 101mm, 89mm and 77mm respectively. An initial loss of 0mm has been adopted for the model calibration, as heavy rainfall on the day prior would have accounted for the initial loss of this rainfall event.

5.2.3 Downstream Boundary Conditions

Ocean tide (water level) data was available for the March 2009 event from a continuous tide gauge maintained by MHL at the Coffs Harbour Jetty. This water level data (as presented in Figure 5-5) was used as the downstream boundary for the March 2009 event.

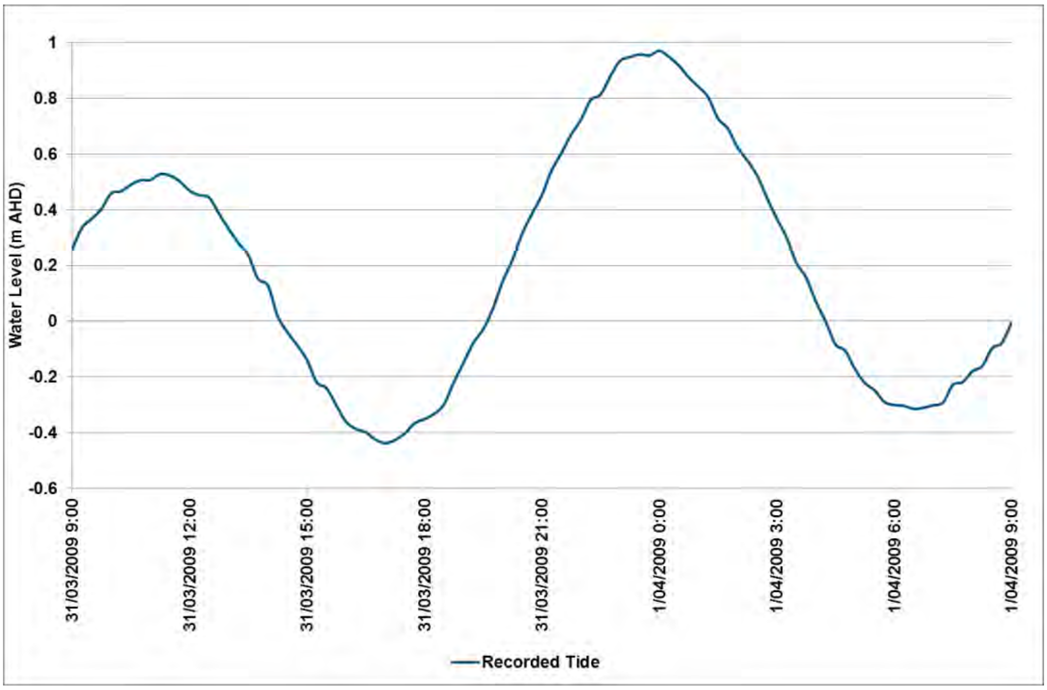


Figure 5-5 Recorded Tidal Water Level at the Coffs Harbour Jetty for the March 2009 Event

5.2.4 Adopted Model Parameters

The model calibration centred around the adjustment of the sub-catchment PERN values and Bx storage routing factor (hydrological model parameters) and the Manning's 'n' values for the floodplain and channel (hydraulic model parameter). The hydrologic and hydraulic model were calibrated together, by changing parameters within reasonable bounds until the best fit against recorded peak flood levels was achieved.

The final values adopted, as shown in Table 5-1, were found to give a good result in representing the recorded water level hydrograph at the Pacific Highway gauge. The adopted parameters also provided a good match to the surveyed flood marks located across the catchment.

Table 5-1 Adopted Model Parameters

Parameter	Value	Comment
Initial Loss	0mm	Adopted to reflect catchment conditions resulting from the large rainfall depth preceding the main storm burst.
Continuing Loss	2.5mm/hr	Adopted continuing loss rate as recommended in AR&R (2001) for design events.
PERN *Forested* *Cleared* *Urban (pervious)* *Urban (impervious)*	 0.12 0.06 0.04 0.015	The PERN factors are used to adjust the catchment routing factor to allow for catchment roughness. Catchment average values were estimated based on representative land use/ground coverage.
Bx (storage routing parameter)	1.0	The adopted value was applied globally for the entire catchment and provided the best fit of catchment response in terms of flow magnitude and timing in the hydrological model.
Manning's n (channel) *Entrance* *Upper reaches*	 0.03 0.12	In channel roughness transitions from a lower value at the entrance to a higher value in the upper reaches. Variability reflects degree of channel vegetation, channel size and sinuosity.
Manning's n (floodplain) *Road easements* *Industrial areas* *Urban lots* *Pasture* *Light vegetation* *Dense vegetation* *Buildings*	 0.03 0.04 0.06 0.06 0.08 0.12 1.00	Different land uses on the floodplain were delineated based on aerial photography and Council GIS layers.

5.2.5 Observed and Simulated Flood Behaviour

The recorded water level hydrograph at the MHL gauging station on the Coffs Creek at the Pacific Highway provides the principal calibration data for the hydrological model for this event. Following the event, over 200 flood levels were surveyed by Council through community questionnaire responses. The remaining calibration was therefore completed based on comparison of modelled flood levels against recorded flood marks across the catchment.

A comparison of the recorded and modelled water level hydrographs at the Coffs Creek gauge is shown in Figure 5-6. The results indicate a good agreement between the initial response, timing and shape of the modelled hydrograph to observed conditions at the Pacific Highway Bridge. The modelled peak water level is within 0.1m of the recorded peak of 5.14m AHD.

There are a number of uncertainties in the simulated hydrological conditions, particularly relating to the assumed spatial and temporal distribution of rainfall, which may have a significant influence on the catchment generated runoff. To demonstrate the model sensitivity to this, an alternate rainfall distribution across the upper catchment was also modelled, as detailed in Section 5.2.1, with results presented on Figure 5-6 for comparison. It shows that the adopted rainfall produces a better match to the shape of the recorded flood peak, than does the alternative sensitivity test.

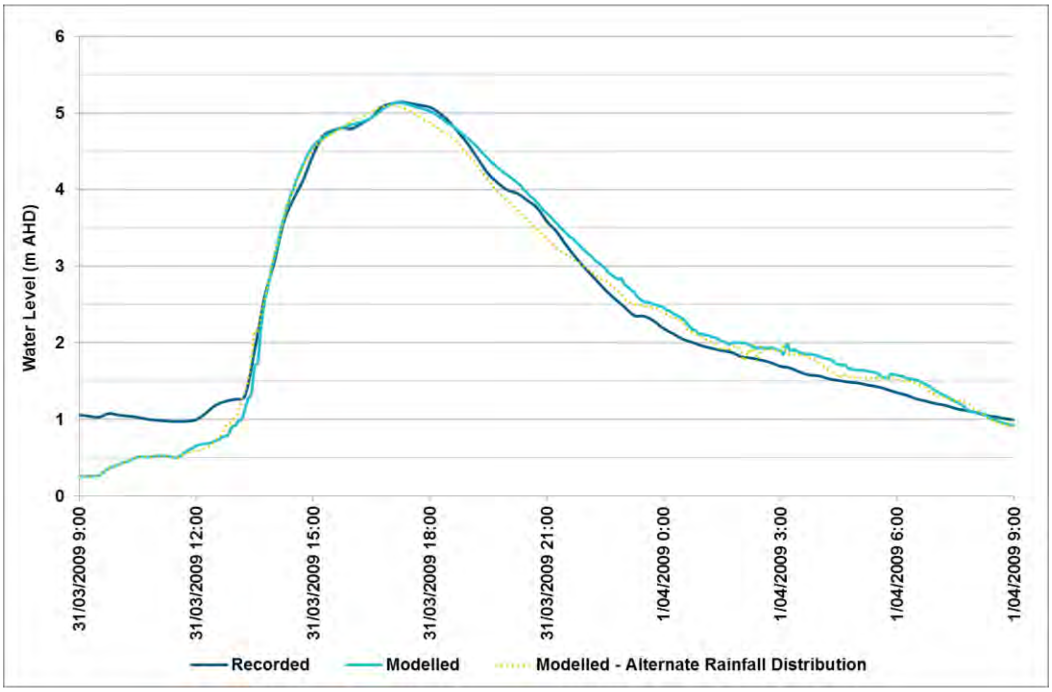


Figure 5-6 Recorded and Modelled Peak Water Level Hydrograph at Pacific Highway Bridge for the March 2009 Event

Most of the surveyed flood marks across the study area are related to flooding of the main arm of Coffs Creek and are located on the floodplain area between the Pacific Highway and Loaders Lane. There are also a number of flood marks located along the Bray Street arm of the Northern Tributaries, as well as a few located in Park Beach.

Coffs Creek

The modelled peak flood level profile for the Coffs Creek Main Arm is presented in Figure 5-7, against the available flood marks relating to Coffs Creek flooding. The modelled bed elevation is also included for reference. A similar flood level profile for the North West Arm of Coffs Creek (from chainage of 0m at the confluence with the Coffs Creek Main Arm) is presented in Figure 5-8. In both figures, it can be seen that the modelled flood profile matches well to the observed data.

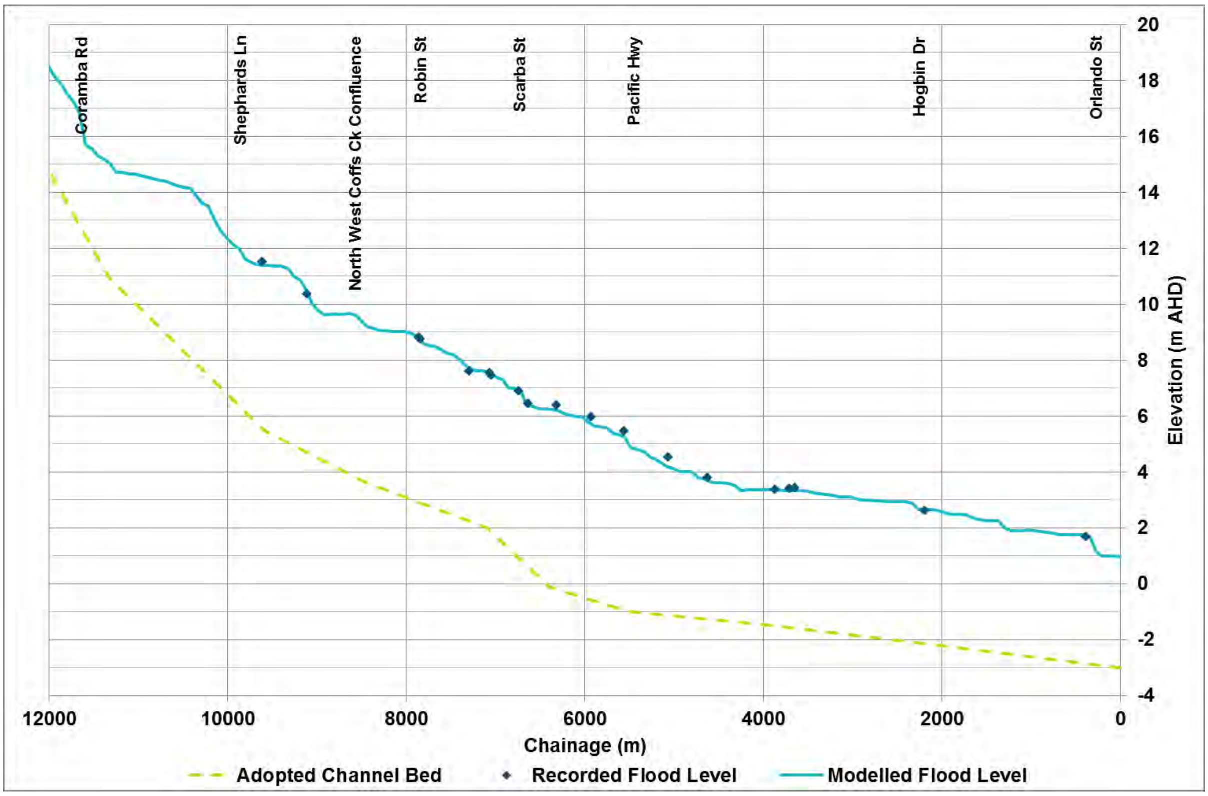


Figure 5-7 Long Section along the Coffs Creek Main Arm for the March 2009 Calibration Event

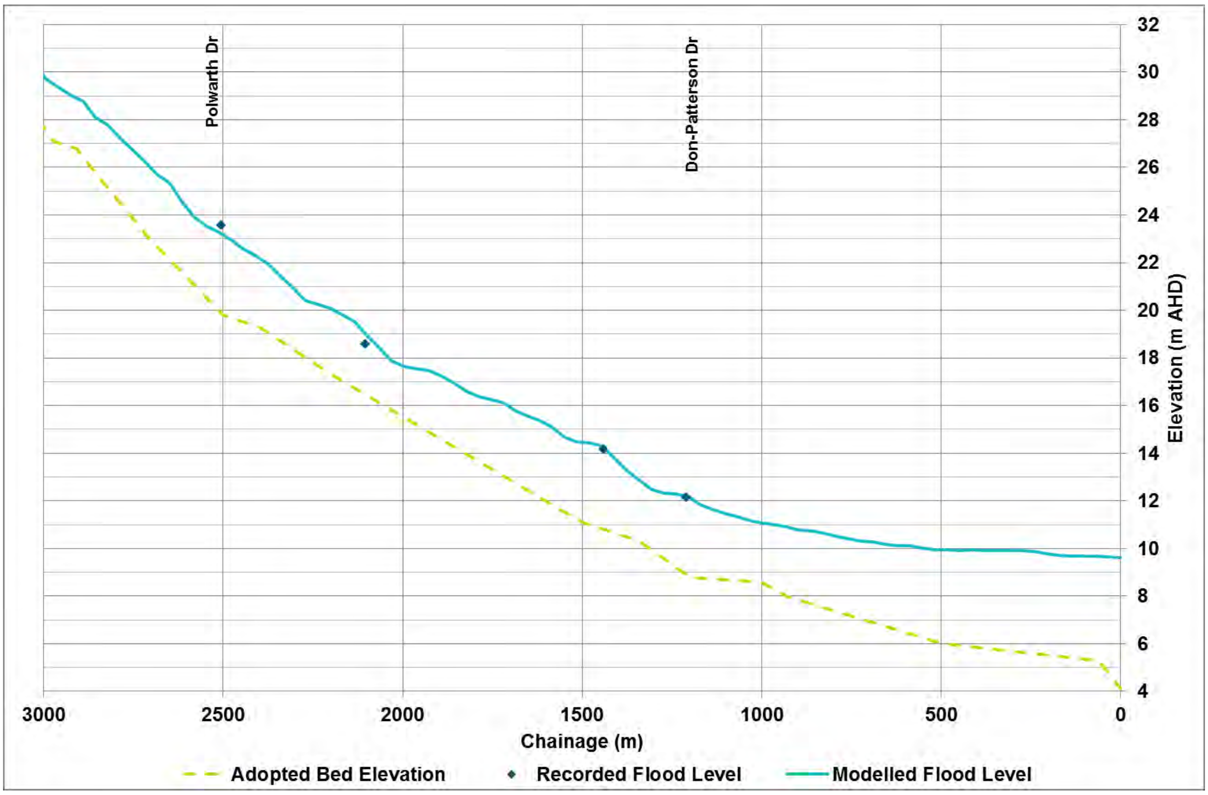


Figure 5-8 Long Section along the Coffs Creek North West Arm for the March 2009 Calibration Event

In the area between Loaders Lane and the Pacific Highway there are a number of overland flood flow paths that become active during large flood events. These include across the Pacific Highway down Harbour Drive, through the CBD (West High Street / Moonee Street / Scarba Street area), down Gundagai Street and across Frances Street, through the Naranga Gardens Estate (Adelines Way and Moreton Bay Ave) and though the Loaders Lane / Goodenough Terrace area. Local topographic controls determine the flood flow distribution, with flood levels displaying significant spatial variability across the floodplain. Figure 5-9 and Figure 5-10 show the modelled peak flood depths and peak water level contours across the floodplain downstream and upstream of Robin Street, respectively. The locations of survey flood marks are also displayed. A comparison of the modelled peak flood levels with the surveyed flood marks is presented in Table 5-2.

Table 5-2 Flood Mark Survey Locations for the March 2009 Event - Coffs Creek Main Arm

ID	Flood Level (m AHD)		ID	Flood Level (m AHD)		ID	Flood Level (m AHD)	
	Surveyed	Modelled		Surveyed	Modelled		Surveyed	Modelled
1	3.4	3.4	**29**	6.9	6.9	**57**	10.8	11.3
2	3.4	3.4	**30**	7.4	7.2	**58**	12	11.9
3	3.8	3.7	**31**	7	7.1	**59**	11.5	11.4
4	3.8	3.6	**32**	6.9	7.1	**60**	11.3	11.4
5	3.8	3.6	**33**	7.5	7.7	**61**	11.6	11.5
6	3.7	3.6	**34**	7.5	7.5	**62**	11.5	11.4
7	3.9	3.7	**35**	7.5	7.6	**63**	11.8	11.7
8	3.8	3.7	**36**	7.6	7.7	**64**	11.9	11.6
9	3.8	3.8	**37**	8.3	8.2	**65**	11.9	11.8
10	4.2	4.2	**38**	8.1	8.3	**66**	12	11.9
11	3.5	4.2	**39**	8.3	8.3	**67**	12.6	12.3
12	4.7	4.8	**40**	8.6	8.5	**68**	12.7	12.4
13	4.9	4.9	**41**	8.2	8.2	**69**	12.5	12.4
14	4.7	4.8	**42**	8.1	8.2	**70**	12.6	12.5
15	4.5	4.2	**43**	8.8	8.8	**71**	12.8	12.5
16	5.3	5.3	**44**	8.8	8.8	**72**	14.4	14.0
17	5.5	5.3	**45**	8.5	8.6	**73**	14.1	14.1
18	5.5	5.2	**46**	8.8	8.6	**74**	14.9	14.5
19	5.6	5.7	**47**	8.5	8.6	**75**	15	14.7
20	5.7	5.9	**48**	8.8	8.9	**76**	16.1	15.8
21	5.9	5.9	**49**	9.0	9.0	**77**	15.6	15.6
22	5.9	6	**50**	9.1	9.0	**78**	11.9	12.1
23	6.0	6.1	**51**	9.3	9.1	**79**	12.3	12.2
24	6.0	5.7	**52**	9.1	8.9	**80**	12.2	12.2
25	6.4	6	**53**	10.4	10.4	**81**	14.2	14.2
26	6.4	6.2	**54**	10.6	10.7	**82**	18.6	19.0
27	6.4	6.4	**55**	11.2	11.2	**83**	18.7	18.7
28	6.9	6.9	**56**	11.2	11.3			

It can be seen from Table 5-2 that there is a good match between the modelled peak flood levels and surveyed flood marks. Almost 70% of the observation points are within 0.1m, of which 25% provide an exact match. Only twelve of the points show a difference of 0.3m or more and it is likely that some of the larger discrepancies may be a result of uncertainties associated with surveyed flood levels. Given the complex nature of the floodplain in this area the model calibration is better than would typically be expected to be achieved.

The results of the March 2009 model calibration for all available flood marks within the study area are shown in Appendix A.

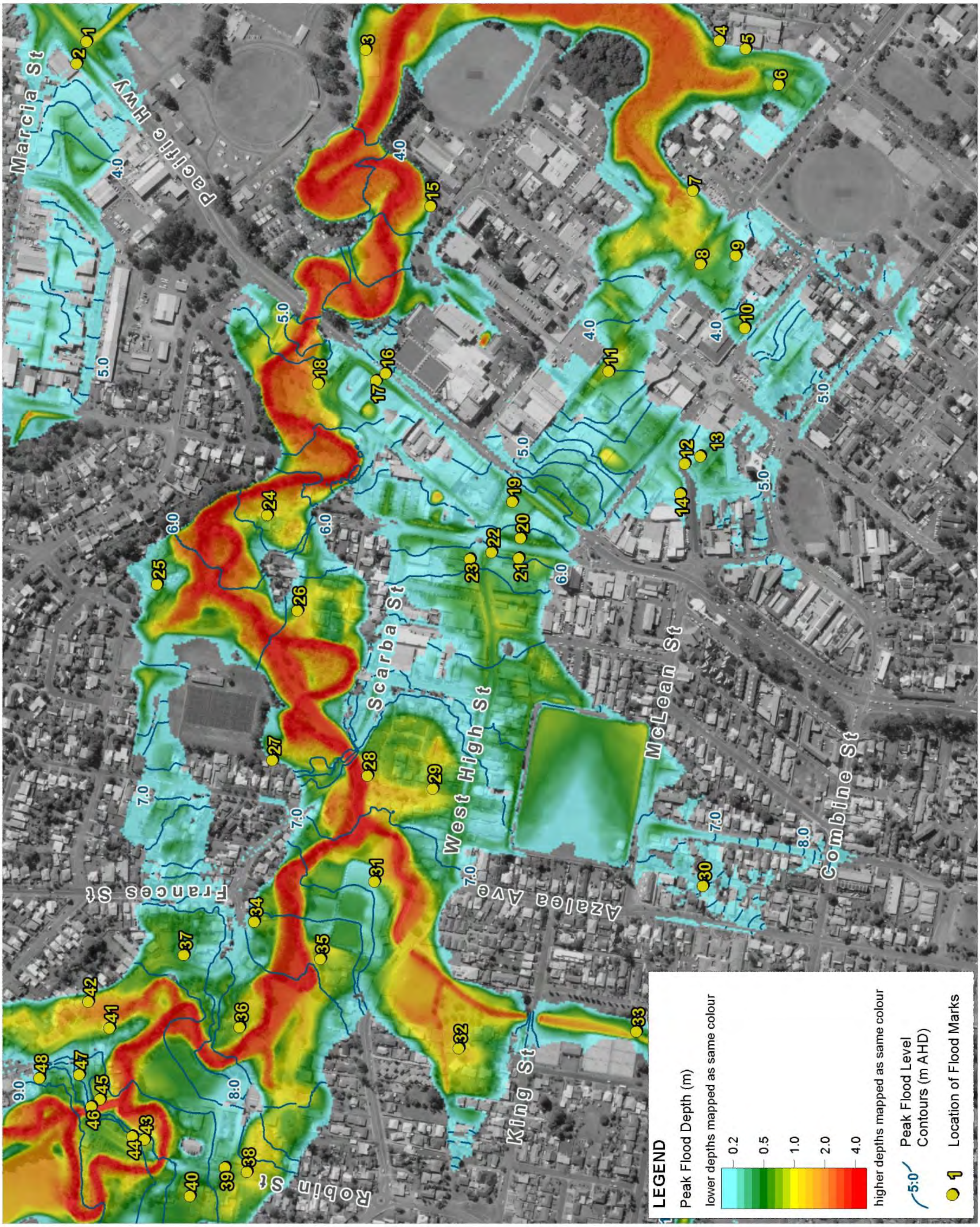
Northern Tributaries of Coffs Creek

The nature of flooding along the Northern Tributaries is similarly characterised by overland flow paths though urban areas that readily become active during large flash flood events. Floodwater spills from the Bray Street tributary at various locations between the Pacific Highway and Frederick Street. Just downstream of Hannaford Place, local flood mitigation works including levee walls and benching have been constructed since 1996 to alleviate the flood impact to properties along Langker Place and Collice Place. Further upstream, floodwater has a tendency to breach the banks at the Perry Drive crossing inundating properties along Apollo Drive. Flooding also occurs along a naturally occurring gully line through Antaries Avenue and Polaris Close.

The modelled peak flood level profile for the Bray Street Arm of the Northern Tributaries is presented in Figure 5-11 against the available surveyed flood marks. Unfortunately, there are no survey flood marks available along the Argyll Street Arm to compare the channel long section profile for this event.

Generally, there is a good match between recorded and modelled flood levels along the Bray Street Arm for the March 2009 event. There are a number of culvert crossings along both the Bray Street and Argyll Street branches of the Northern Tributaries that could become blocked with debris during flood events. Potential culvert blockage could possibly account for minor discrepancies in levels modelled between Taloumbi Road and Bray Street East on Figure 5-11.

There are 20 surveyed flood marks available along the lower reaches of Northern Tributaries of Coffs Creek which can be used as calibration points for out-of-bank flooding for this event. Figure 5-12 shows the modelled peak flood depths and peak water level contours across the floodplain of the Bray Street and Argyll Street Arms downstream of Joyce Street. The locations of survey flood marks are also displayed. A comparison of the modelled peak flood levels with the surveyed flood marks is presented in Table 5-3.



Title:

March 2009 Model Calibration
Coffs Creek Main Arm, Downstream of Robin Street

Figure: **5-9**

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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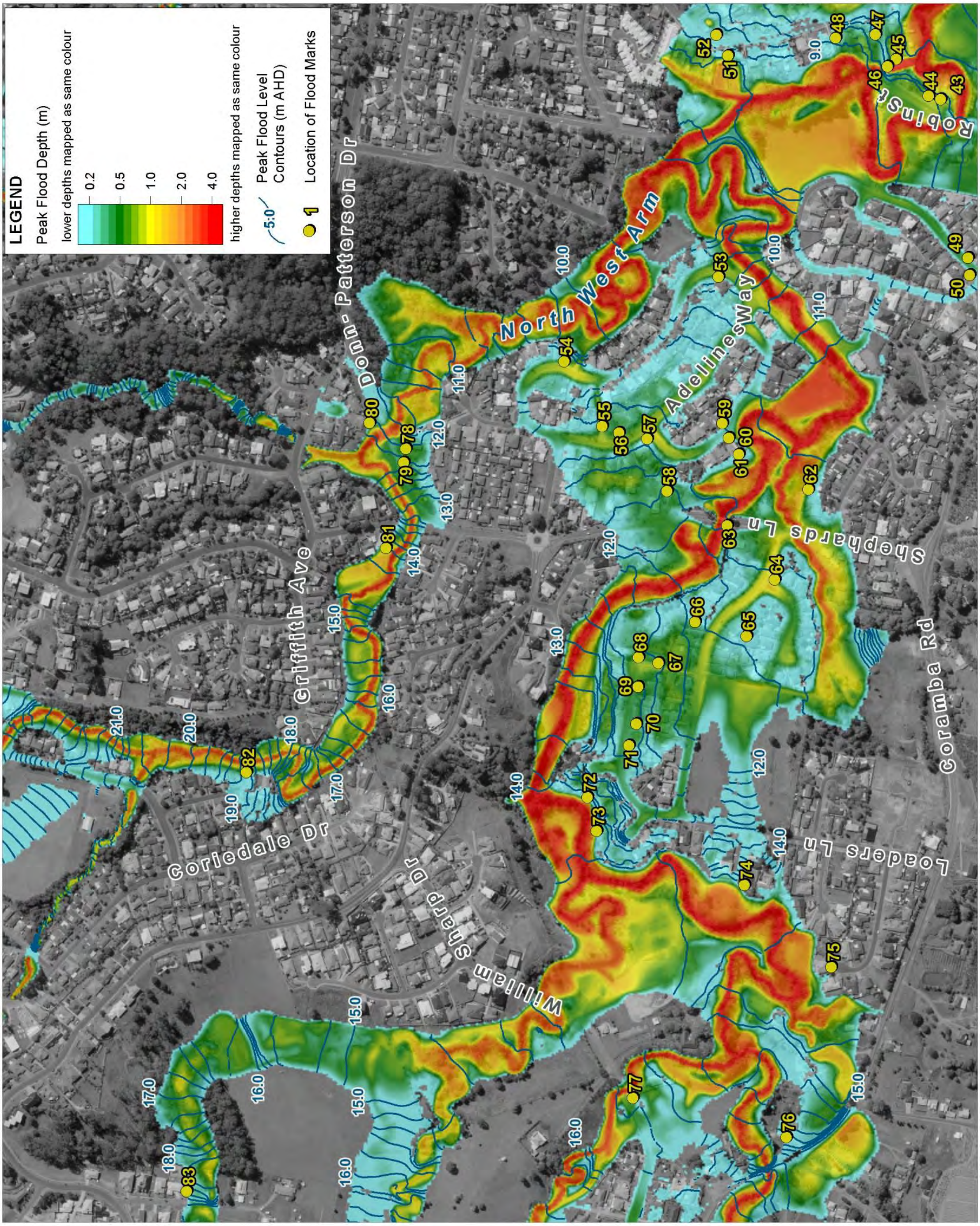

Title:

March 2009 Model Calibration
Coffs Creek Main Arm, Upstream of Robin Street

Figure: **5-10**

Rev: **A**

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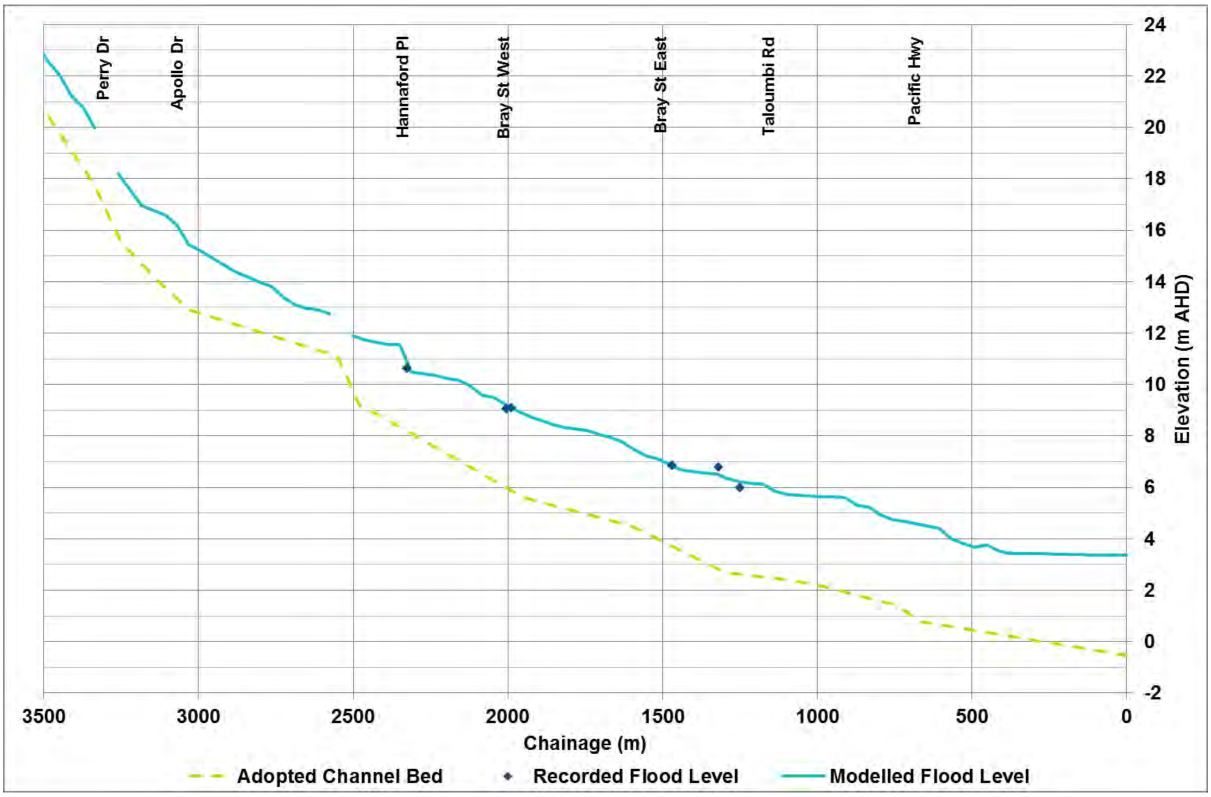
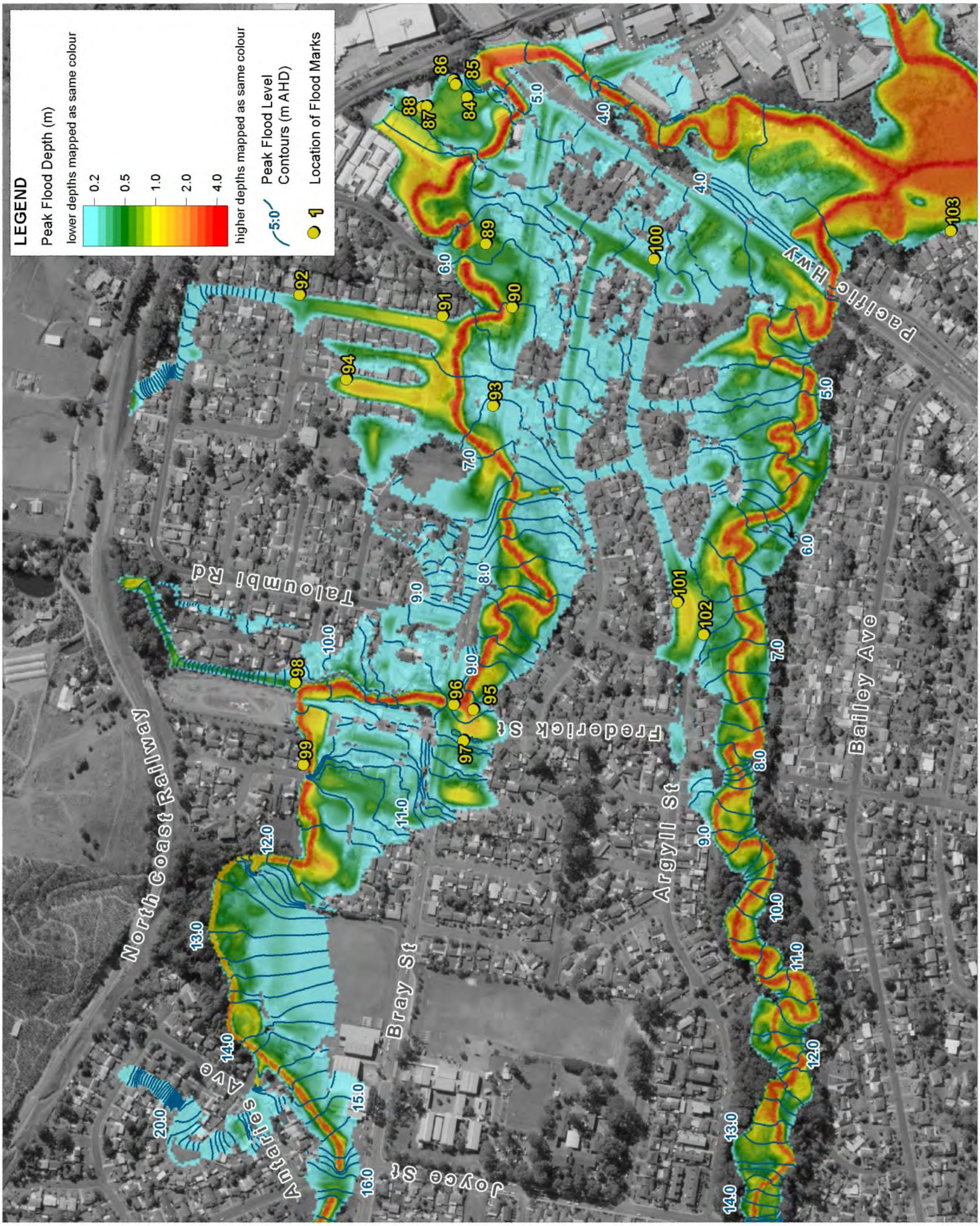


Figure 5-11 Long Section along the Bray Street Arm of the Northern Tributaries for the March 2009 Calibration Event

Table 5-3 Flood Mark Survey Locations for the March 2009 Event – Northern Tributaries of Coffs Creek

ID	Flood Level (m AHD)		ID	Flood Level (m AHD)	
	Surveyed	Modelled		Surveyed	Modelled
84	5.2	5.3	**94**	6.5	6.6
85	5.3	5.4	**95**	9.1	9.1
86	5.2	5.4	**96**	9.1	9.1
87	5.5	5.4	**97**	9.1	9.2
88	5.5	5.4	**98**	10.6	10.5
89	5.8	5.9	**99**	10.6	10.5
90	6.0	6.2	**100**	4.7	4.8
91	6.8	6.5	**101**	6.8	6.6
92	6.8	6.5	**102**	6.8	6.6
93	6.8	6.9	**103**	3.4	3.3

There is a good match between the modelled peak flood levels and surveyed flood marks in the lower Northern Tributaries of Coffs Creek, with all but one modelled level within 0.2m of surveyed peak flood levels. Three of the 20 levels provide an exact match. Therefore, it was concluded that the model can reproduce flood profiles and overland flow paths representative of actual flood behaviour along the Northern Tributaries of Coffs Creek.



Title:

March 2009 Model Calibration
Northern Tributaries of Coffs Creek

Figure: **5-12**

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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Park Beach

There is little flood data available for the calibration of the model to the March 2009 event in the Park Beach area. There are eight surveyed flood levels located along the drainage channel adjacent to the southern Park Beach Plaza car park, downstream to the railway crossing/Orlando Street Bridge and into Coffs Creek. Figure 5-13 shows the modelled peak water level profile against surveyed flood marks at this location. As seen in Figure 5-13, the modelled flood level was consistently lower than recorded flood marks by around 0.2-0.4m.

There is considerable uncertainty in spatial variation, temporal pattern and total depth of rainfall over Park Beach, as the three available rainfall gauges are located higher in the catchment. The Coffs Harbour Airport gauge is located some 2.5km away from the Park Beach catchment, in the wider Boambee/Newport Creeks catchment and is separated by a steep ridge. Due to the orographic rainfall effects known to occur in the region, it is likely that the Airport gauge, although at similar elevation, could have recorded significantly different rainfall than that which occurred in Park Beach.

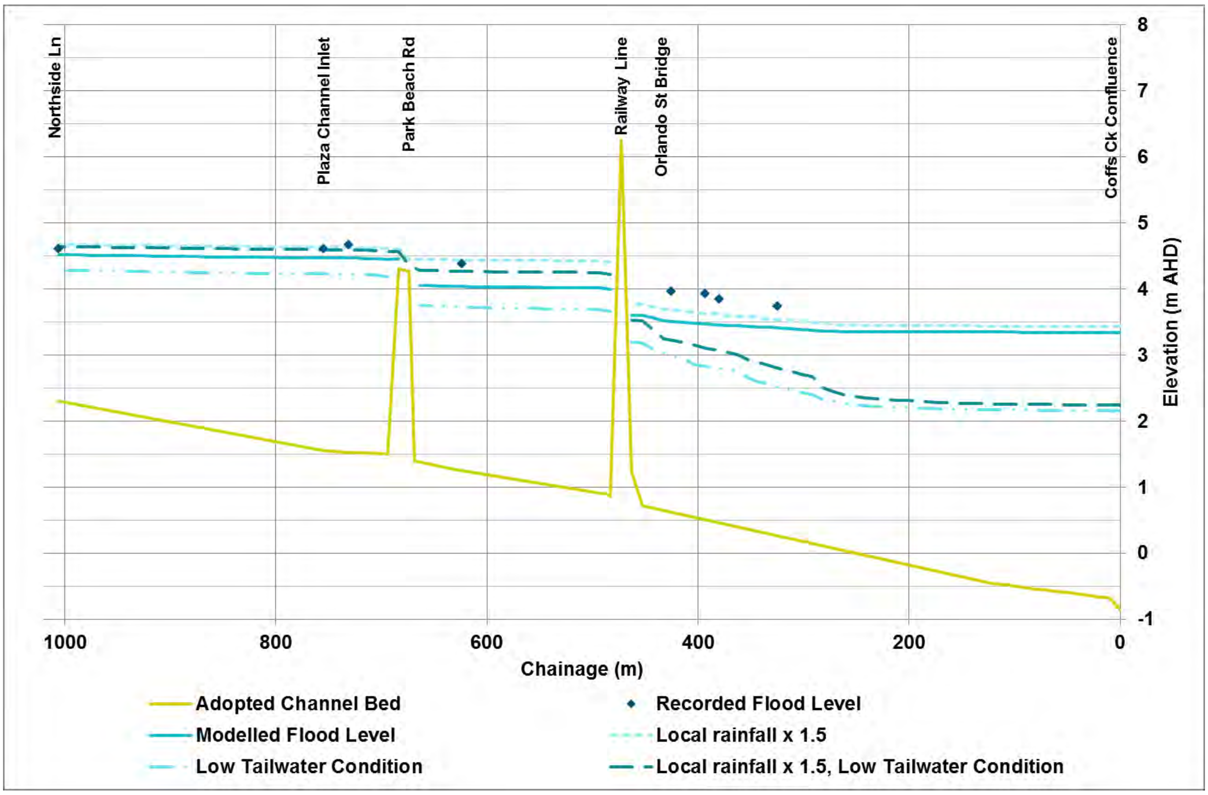


Figure 5-13 Long Section along the Park Beach Plaza/Orlando Street Channel for the March 2009 Calibration Event

As detailed in Section 5.2.1, estimates for rainfall isohyets across the lower catchment were determined from privately read totals. Some of these values were ambiguous and could be incorrect. Additionally, the temporal pattern of rainfall over Park Beach could be significantly different to that recorded at the Perry Drive, Shephards Lane and Red Hill gauges. If the timing of intense rainfall over Park Beach was different to that recorded over the upper catchment by a few hours, the influence of a lower tail water condition in Coffs Creek could also impact peak levels modelled along this channel.

Sensitivity tests were carried out to assess the influence of total rainfall depth and Coffs Creek tail water conditions on peak flood levels in Park Beach. Local rainfall applied to Park Beach sub catchments in the RAFTS hydrological model were increased by 50% (i.e. multiplied by 1.5) to produce a total rainfall depth of around 400mm. Rainfall depths over 400mm were recorded across most of the Coffs Creek catchment, so are within reason for this event. To assess a lower Coffs Creek tail water condition, rainfall inputs to the remaining sub catchments in the RAFTS model were removed to ensure the Coffs Creek water level remained sufficiently low through the model simulation.

The results of the sensitivity tests are also shown in Figure 5-13. Simulations with rainfall increased by 50% best replicated recorded flood marks upstream of the railway line. Downstream of the Orlando Street bridge, the model is consistently underestimating the observed flood level profile for the March 2009 event. As the gradient of the flood level profile displayed in the low tail water simulations is comparable to that observed in the recorded flood marks, it can be assumed that given the correct combination of tail water level and local rainfall inflow, a better match could be achieved.

The discrepancies between recorded and modelled peak flood levels are within acceptable bounds, considering uncertainties in the rainfall data such as spatial and temporal variations across the catchment.

5.3 November 1996 Model Verification

5.3.1 Rainfall Data

The compilation of rainfall data for the 1996 event completed by Webb, McKeown and Associates (1997) was utilised for this study. There were six pluviographs and two daily read gauges located in the vicinity of the Coffs Creek catchment that were operational during the event. These gauges were operated by BoM, MHL and Council. The continuously read gauge at the Catholic Club operated by Council was the only official gauge located within the catchment boundary at the time. Private rainfall records from 83 privately read gauges, of which 17 were located within the catchment boundary, were obtained.

The November 1996 storm was localised and intense, with rainfall depths exhibiting significant spatial variation. The isohyetal map produced by Webb, McKeown and Associates showing the total 24 hour rainfall to 9am on 24th November is reproduced in Figure 5-14 along with the location of rainfall records. The magnitude of rainfall assumed to have fallen on the upper catchment during this event was extreme. A 24 hour total rainfall depth of 500mm was assumed to have fallen on around 40% of the Coffs Creek catchment area. However, considerable uncertainty surrounds this estimation due to the lack of reliable gauge recordings over the upper catchment.

Temporal patterns recorded at Newports Creek, Middle Boambee and South Boambee displayed similar variation in rainfall intensity with time. These gauges are located within the Boambee/Newport Creeks catchment to the south of the Coffs Creek catchment. The main burst of rainfall occurred from around 3pm to 10pm on the 23rd November with intensity steadily increasing over this time. Approximately 394mm, 378mm and 472mm was recorded at each gauge respectively over the 7 hour period. The Catholic Club gauge indicated a slightly delayed onset and shorter duration of intense rainfall. Around 226mm was recorded over 6 hours from 4:30pm. Each gauge recorded the most intense rainfall between 8pm and 9pm.

Comparably, the Coffs Harbour Airport and Upper Orara (Aurania) gauges each displayed quite different temporal patterns. As the Upper Orara (Aurania) gauge is located west of the escarpment it was deemed to be unrepresentative of rainfall within the Coffs Creek catchment. The Coffs Harbour gauge recorded two intense 30 minute bursts of 45mm and 38mm between 5:30-6pm and 8-8:30pm respectively, the second of which aligned with the peak recorded across the other gauges. The half-hour rainfall hyetographs recorded at Newports Creek, the Catholic Club and Coffs Harbour Airport are shown in Figure 5-15. Although 24 hour totals were often reported for the storm, it can be seen that almost all of the rain fell within a 7 hour period.

Figure 5-16 presents the recorded November 1996 rainfall intensities against both the 1987 IFDs and 2013 IFDs, to gain an appreciation of the relative intensity and magnitude of the storm across the catchment.

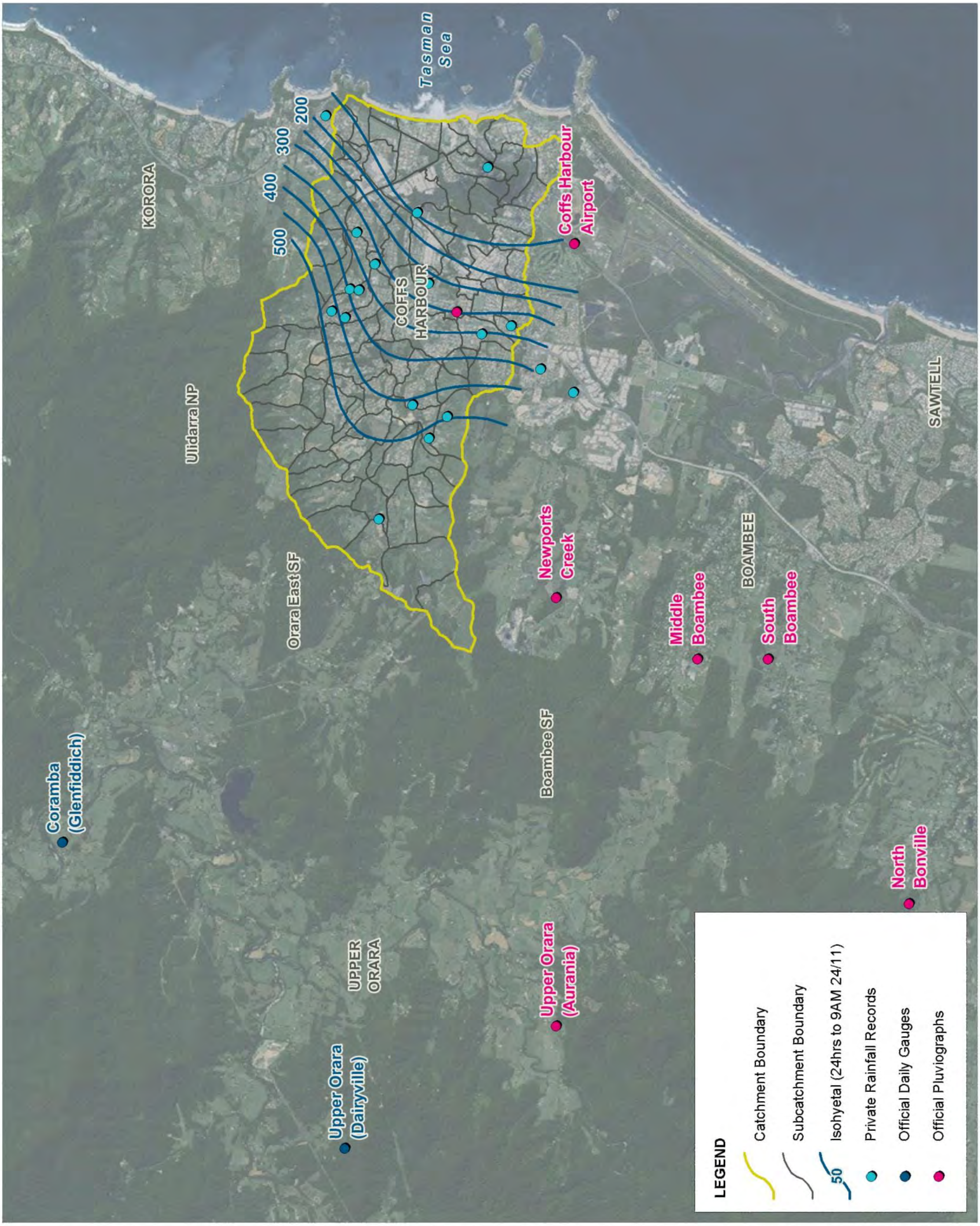
Rainfall recorded at the Catholic Club tracks approximately along a 2% AEP event for durations up to 6 hours. Assuming a similar depth vs. duration trend was evident for the upper catchment (where a total of around rainfall depth of 500mm was observed) this equates to rainfall well above the 0.2% AEP for the 6 hour duration. That is, rainfall of this intensity is expected to occur less frequently than once every 500 years.

In developing the hydrological model, the approach adopted in the Coffs Creek Flood Study (2001) was used to assign total rainfall depth and temporal pattern to each sub catchment. Rainfall totals were applied to each sub catchment based on the isohyetal distribution seen in Figure 5-15. Due to the absence of multiple reliable rainfall records of the upper catchment, there is considerable uncertainty surrounding actually rainfall intensity during this event.

The temporal pattern recorded at the Catholic Club or Newports Creek was applied to each sub catchment, determined by distance. This method is appropriate given the similarity of the two temporal patterns. There is some uncertainty regarding the most representative temporal pattern for the lower sub catchments, arising from the difference observed between temporal patterns at the Catholic Club and the Coffs Harbour Airport gauge. However, given that the local inflow generated within the hydrological model for the downstream sub catchments will have a minor influence on flood behaviour within the broader catchment, the temporal pattern recorded at the Catholic Club gauge was adopted.

5.3.2 Antecedent Conditions

Rainfall records indicate that in the 24 hours to 9am on the 23rd November (i.e. the day before the event) 88mm and 73mm were recorded at Coffs Harbour Airport and the Catholic Club respectively. Privately read gauges indicated that more rainfall fell across upper catchment areas in this same period, with 114mm reported over the Northern Tributaries. An initial loss of 0mm has been adopted for the model calibration, as heavy rainfall on the day prior would have accounted for the initial loss of this rainfall event.



Title:
Rainfall Analysis, 24hrs to 9AM 24th November 1996

Figure: 5-14

Rev: A

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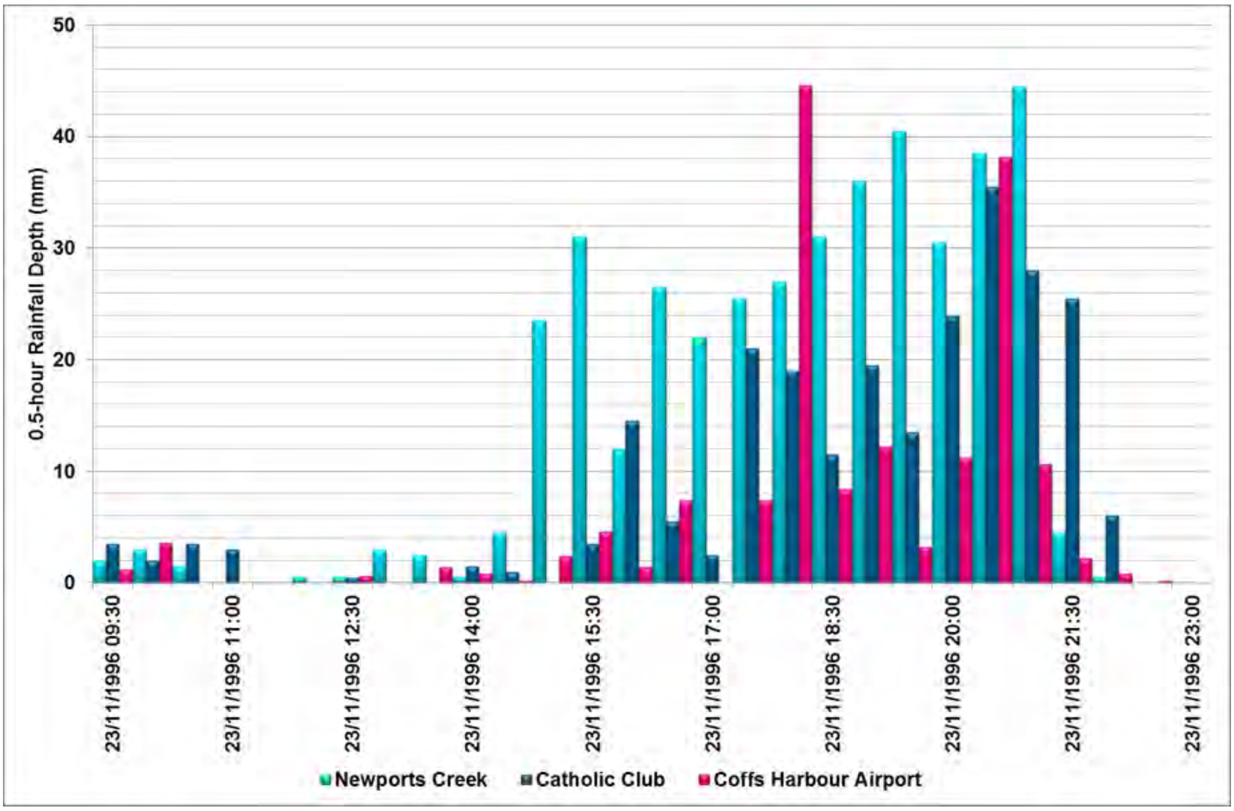


Figure 5-15 Rainfall Hyetograph for the November 1996 Verification Event

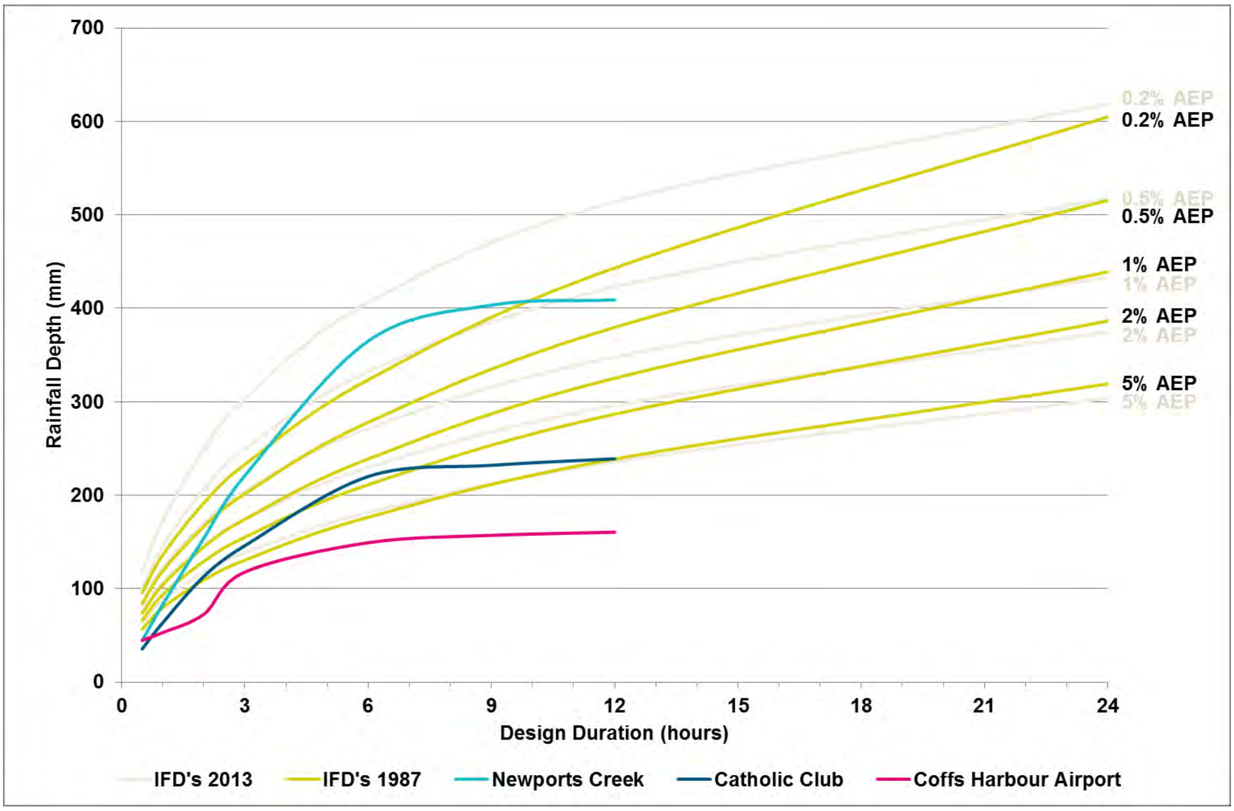


Figure 5-16 Comparison of Recorded November 1996 Rainfall with IFD Relationships

5.3.3 Downstream Boundary Conditions

Ocean tide (water level) data was available for the November 1996 event from a continuous tide gauge maintained by MHL at the Coffs Harbour Jetty. This water level data (as presented in Figure 5-17) was used as the downstream boundary for the November 1996 event.

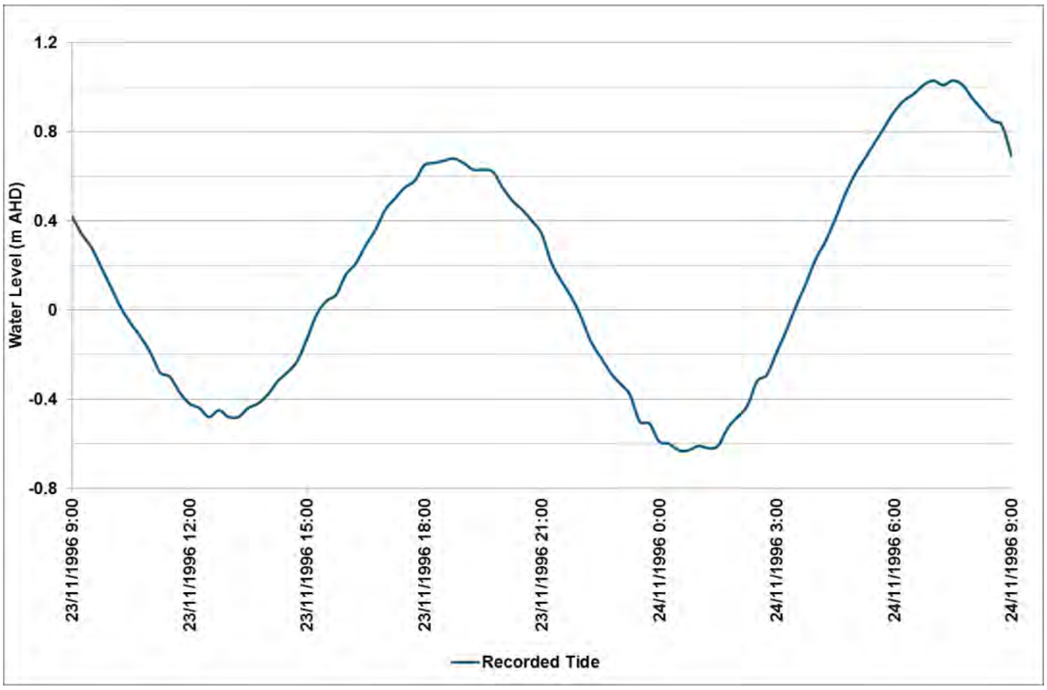


Figure 5-17 Recorded Tidal Water Level at the Coffs Harbour Jetty for the November 1996 Event

5.3.4 Changes to the Model Configuration

For verification against the November 1996 event, it was necessary to alter the application of some parameters within both the hydrological and hydraulic models, to represent the level of urbanisation in the upper catchment at the time. Vegetated areas have higher rainfall losses and lower rate of runoff than urbanised areas. In the hydrological model, PERN values were increased and impervious areas decreased for relevant sub catchments. Within the hydraulic model, material roughness values were changed from urban land use type to vegetated or cleared/pasture land use type.

For simulation of the 1996 event, the Hogbin Drive embankment and bridge structure (constructed in 2007) has been removed from the model domain.

Considerable residential development downstream of Shephards Lane/Don-Patterson Drive has occurred in the years after the 1996 flood, including the Naranga Gardens Estate (Adelines Way and Moreton Bay Avenue) and Red Cedar Drive developments. The model configuration in this location was altered to reflect actual catchment conditions during the 1996 event. This was assisted by comparing current LiDAR survey against 1d model cross sections used in the previous study.

5.3.5 Observed and Simulated Flood Behaviour

Unfortunately, the water level gauge located upstream of the Pacific Highway bridge failed during the November 1996 event. Data compilation following the event established an approximate water level hydrograph from CCTV footage at The Promenade shopping centre located on the corner of Harbour Drive and Mildura Street. Although the data may not be entirely accurate, it allows for the general timing and shape of the hydrograph to be validated. The hydrograph estimated by Webb, McKeown and Associates is reproduced in Figure 5-18 and is compared against the modelled water levels at the same location.

The shape and timing of the modelled peak water level hydrograph is a close match to the estimated hydrograph at The Promenade. There is uncertainty surrounding the actual peak height at this location, given surveyed debris mark on the floodplain in the same location gave a peak flood level of 2.85m AHD. This location is a significant choke point as all flow is contained within the Coffs Creek channel at this location, due to the topography. There is potential for inconsistencies between modelled and recorded peak water levels here depending on the entrance condition and level of scour that occurred during the event. With this in mind, a modelled peak water level at The Promenade of 2.70m AHD is considered a reasonable match to available flood records.

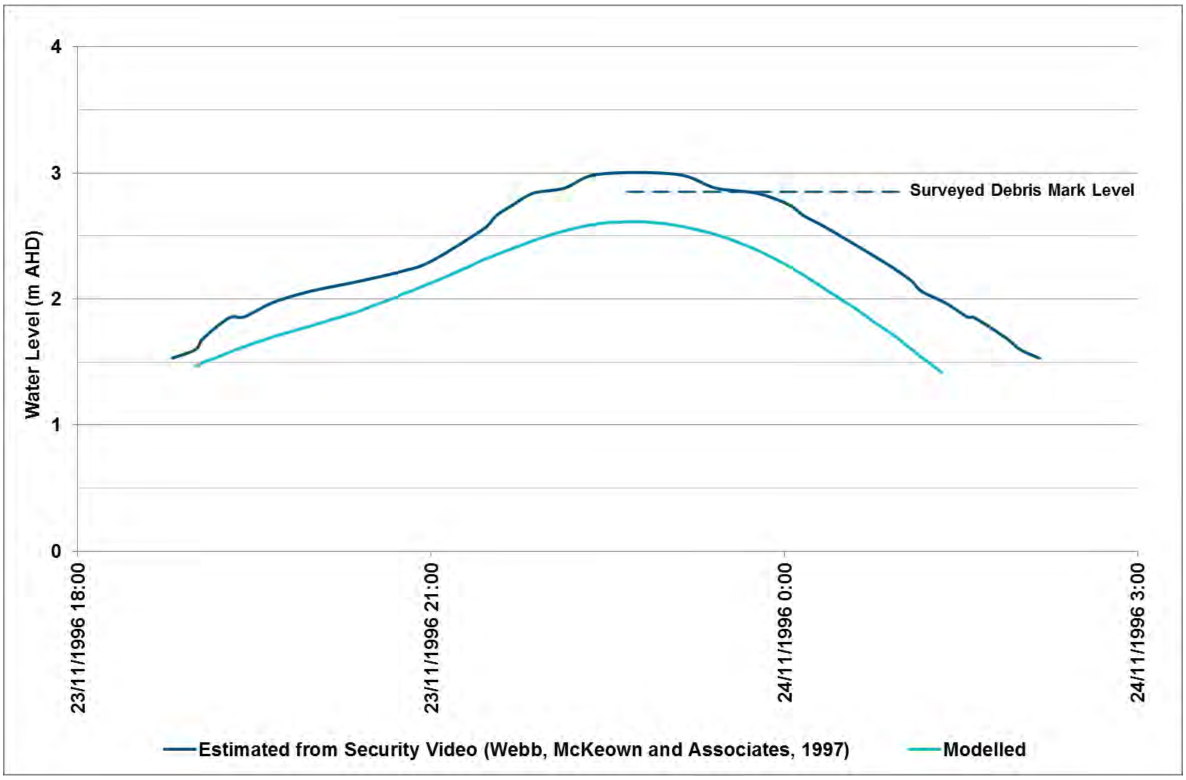


Figure 5-18 Recorded and Modelled Peak Water Level Hydrograph at The Promenade for the November 1996 Event

Coffs Creek

The modelled peak flood level profile for the Coffs Creek Main Arm is presented in Figure 5-19 against the available flood marks relating to Coffs Creek flooding. It can be seen that the modelled flood profile matches well to the observed data.

There are over 100 surveyed flood marks available to assess the models performance for floodplain flows along the Coffs Creek Main Arm for the November 1996 event. Figure 5-20 shows the modelled peak flood depth and peak water level contours across the floodplain downstream of Robin Street. The locations of survey flood marks are also displayed. A comparison of the modelled peak flood levels with the surveyed flood marks is presented in Table 5-4.

It can be seen from Table 5-4 that there is a good match between the modelled peak flood levels and surveyed flood marks. Around 80% of the observation points are within 0.2m, of which 36% provide an exact match. Only 11 locations show a difference of 0.3m or more.

Upstream of Robin Street, there are 16 recorded flood marks. These are predominantly located in the Loaders Lane / Goodenough Terrace area. Four are located just downstream of Shephards Lane. The location of flood marks in this area is shown in Figure 5-21. Modelled water levels through the Loaders Lane estate are consistently an average of 0.5m higher than recorded flood levels. However, there is much uncertainty associated with most of the surveyed flood marks in this area, due to significant flood gradients across individual lots, which is often up to and above 1.0m. The spatial location of the flood marks does not appear to be entirely accurate (often positioned in the centre of the lot). For locations where the flood gradient across the lot is limited, the model is producing a much better match to the observed levels, i.e. often within 0.2m. Surveyed peak flood levels are compared against modelled results in Table 5-5. The survey points with unreliable locations have been highlighted.

Given the good match between modelled and recorded flood levels downstream of Robin Street and the considerable uncertainty in distribution of rainfall totals across the upper catchment during this event, the model parameters adopted for calibration to the March 2009 event are deemed to provide satisfactory results for the November 1996 event on the main arm of Coffs Creek.

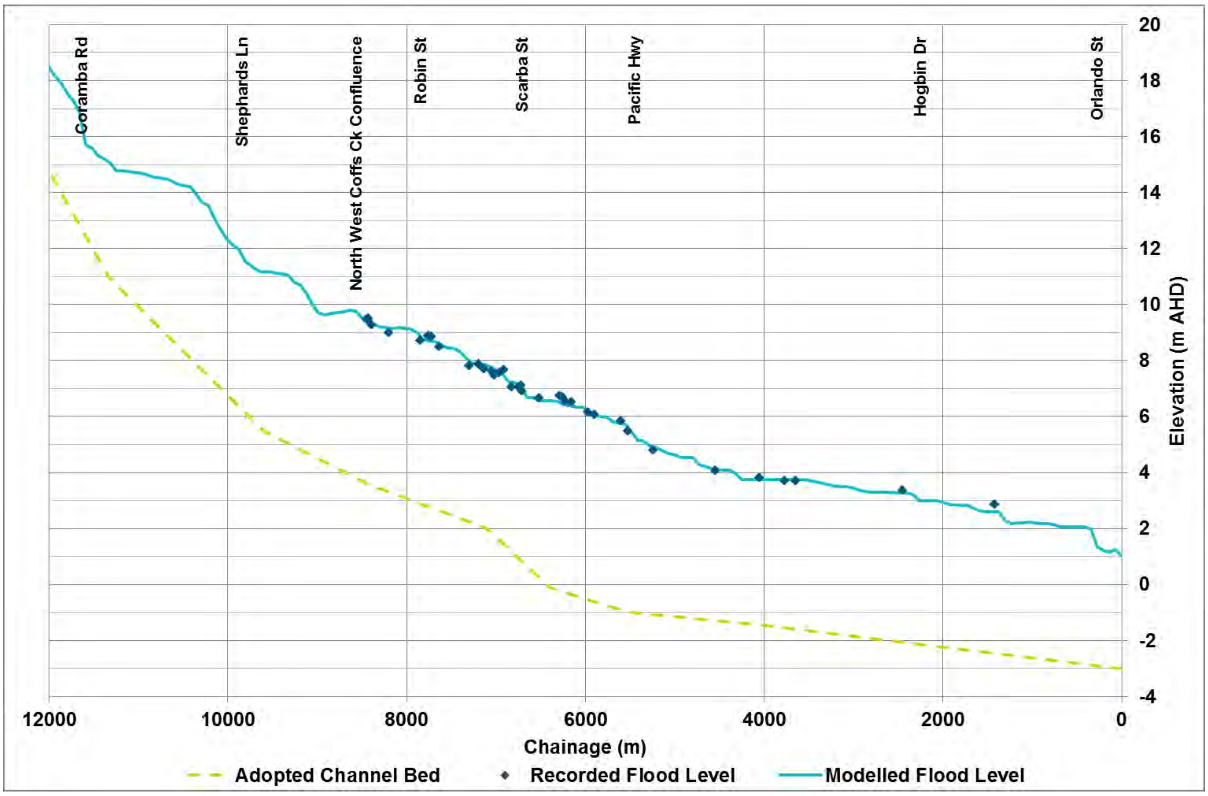
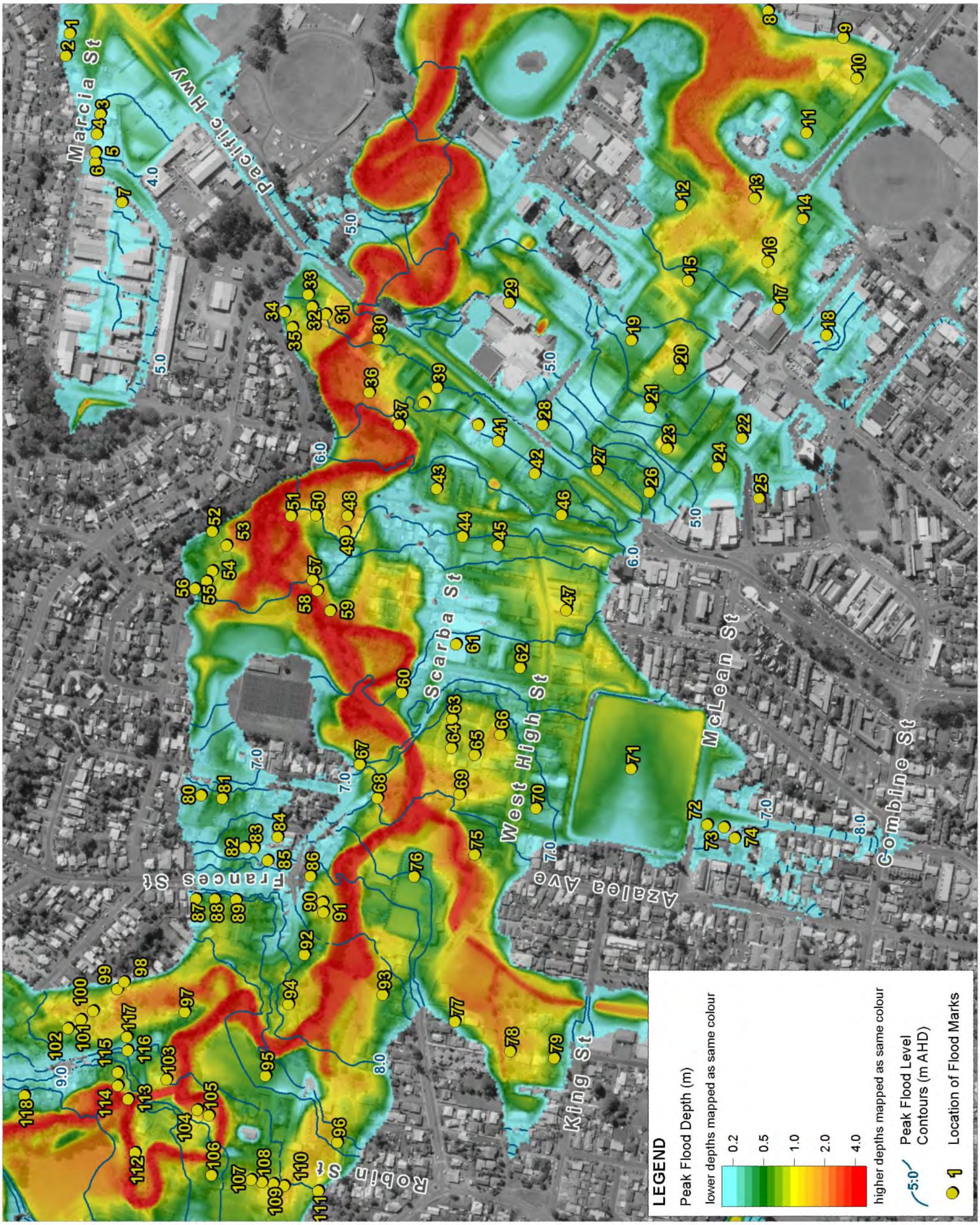


Figure 5-19 Long Section along the Coffs Creek Main Arm for the November 1996 Verification Event



Title:

March 1996 Model Calibration
Coffs Creek Main Arm, Downstream of Robin Street

Figure: **5-20**

Rev: **A**

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0 125 250m

Approx. Scale

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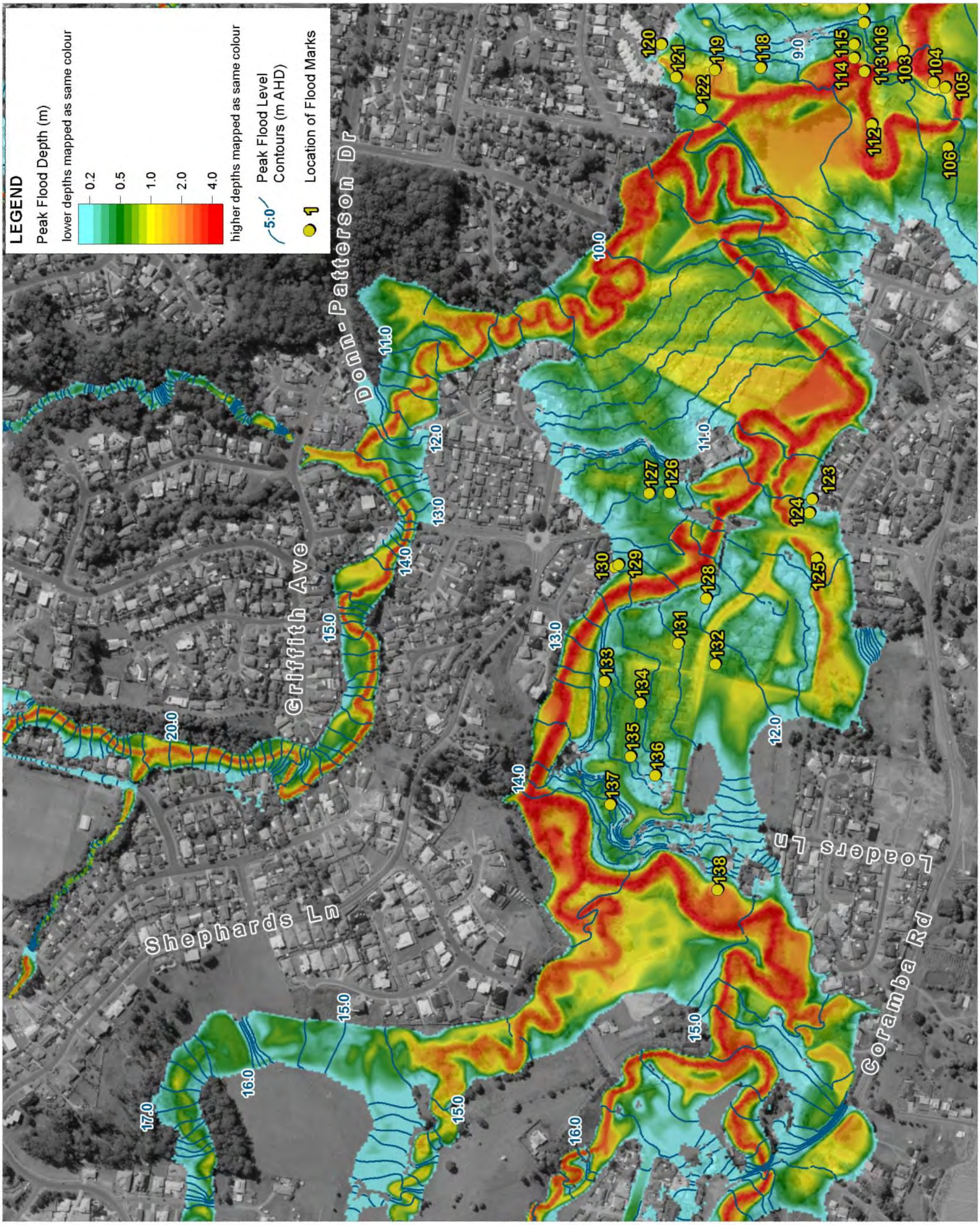

Title:

March 1996 Model Calibration
Coffs Creek Main Arm, Upstream of Robin Street

Figure: **5-21**

Rev: **A**

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Approx. Scale

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Table 5-4 Flood Mark Survey Locations for the November 1996 Event - Coffs Creek Main Arm, Downstream of Robin Street

ID	Flood Level (m AHD)		ID	Flood Level (m AHD)		ID	Flood Level (m AHD)	
	Surveyed	Modelled		Surveyed	Modelled		Surveyed	Modelled
1	3.7	3.8	40	5.9	5.9	79	7.4	7.3
2	3.8	3.8	41	5.7	5.8	80	7.3	7.3
3	3.8	3.8	42	5.7	5.9	81	7.2	7.3
4	3.9	3.9	43	6.1	6.1	82	7.8	7.5
5	4.0	3.9	44	6.0	6.2	83	7.8	7.5
6	4.2	4.1	45	6.1	6.4	84	7.8	7.5
7	4.3	4.2	46	5.9	6.0	85	8.1	7.6
8	4.1	4.1	47	6.4	6.6	86	7.6	7.6
9	4.1	4.1	48	6.2	6.1	87	8.2	8.3
10	4.0	4.1	49	6.3	6.1	88	8.3	8.3
11	4.1	4.1	50	6.2	6.2	89	8.2	8.2
12	4.1	4.2	51	6.2	6.3	90	7.5	7.7
13	4.1	4.2	52	6.5	6.4	91	7.6	7.7
14	4.2	4.2	53	6.5	6.4	92	7.9	7.9
15	4.2	4.3	54	6.6	6.4	93	7.7	7.8
16	4.1	4.2	55	6.5	6.4	94	7.8	8.0
17	4.2	4.2	56	6.6	6.4	95	8.2	8.3
18	4.5	4.3	57	6.6	6.4	96	8.2	8.3
19	4.4	4.5	58	6.7	6.4	97	7.9	8.5
20	4.3	4.5	59	6.7	6.5	98	8.2	8.5
21	4.6	4.6	60	6.7	6.6	99	8.3	8.5
22	4.8	4.9	61	6.7	6.7	100	8.4	8.6
23	5.0	4.9	62	6.7	6.7	101	8.4	8.6
24	4.8	4.9	63	6.9	7.0	102	8.7	8.6
25	5.0	4.9	64	7.1	7.1	103	8.5	8.6
26	5.4	5.4	65	6.8	7.1	104	8.9	8.7
27	5.5	5.7	66	7.0	7.1	105	8.7	8.7
28	5.3	5.4	67	6.8	6.9	106	8.7	8.9
29	4.8	4.9	68	6.9	7.1	107	8.6	8.7
30	5.9	5.6	69	7.0	7.2	108	8.4	8.7
31	5.5	5.6	70	7.1	7.1	109	8.4	8.7
32	5.5	5.7	71	6.9	6.9	110	8.3	8.6
33	5.6	5.6	72	7.2	6.9	111	8.3	8.6
34	5.7	5.7	73	7.2	6.9	112	9.0	9.2
35	6.0	5.7	74	7.3	7.0	113	9.0	9.2
36	5.8	5.7	75	7.0	7.3	114	8.8	9.2
37	5.9	5.9	76	7.7	7.4	115	8.6	9.1
38	5.8	5.7	77	7.4	7.3	116	8.7	8.7

Table 5-5 Flood Mark Survey Locations* for the November 1996 Event - Coffs Creek Main Arm, Upstream of Robin Street

ID	Flood Level (m AHD)		ID	Flood Level (m AHD)	
	Surveyed	Modelled		Surveyed	Modelled
118	*9.3*	*9.3*	***129***	*12.0*	*12.3*
119	*9.4*	*9.3*	***130***	*12.2*	*12.3*
120	*9.4*	*9.3*	**131**	11.8	12.3
121	*9.1*	*9.3*	**132**	11.8	12.1
122	*9.4*	*9.4*	***133***	*12.2*	*13.0*
123	11.6	11.4	***134***	*12.3*	*12.5*
124	11.4	11.4	***135***	*12.3*	*12.7*
125	11.5	11.6	***136***	*12.2*	*12.6*
126	11.8	11.9	***137***	*13.5*	*13.8*
127	11.9	11.7	***138***	*13.6*	*14.6*
128	11.2	12.1			

** Italics indicate survey location is unreliable*

Northern Tributaries of Coffs Creek

The modelled peak flood level profile is presented against the available surveyed flood marks for the Argyll Street Arm and the Bray Street Arm of the Northern Tributaries in Figure 5-22 and Figure 5-23 respectively.

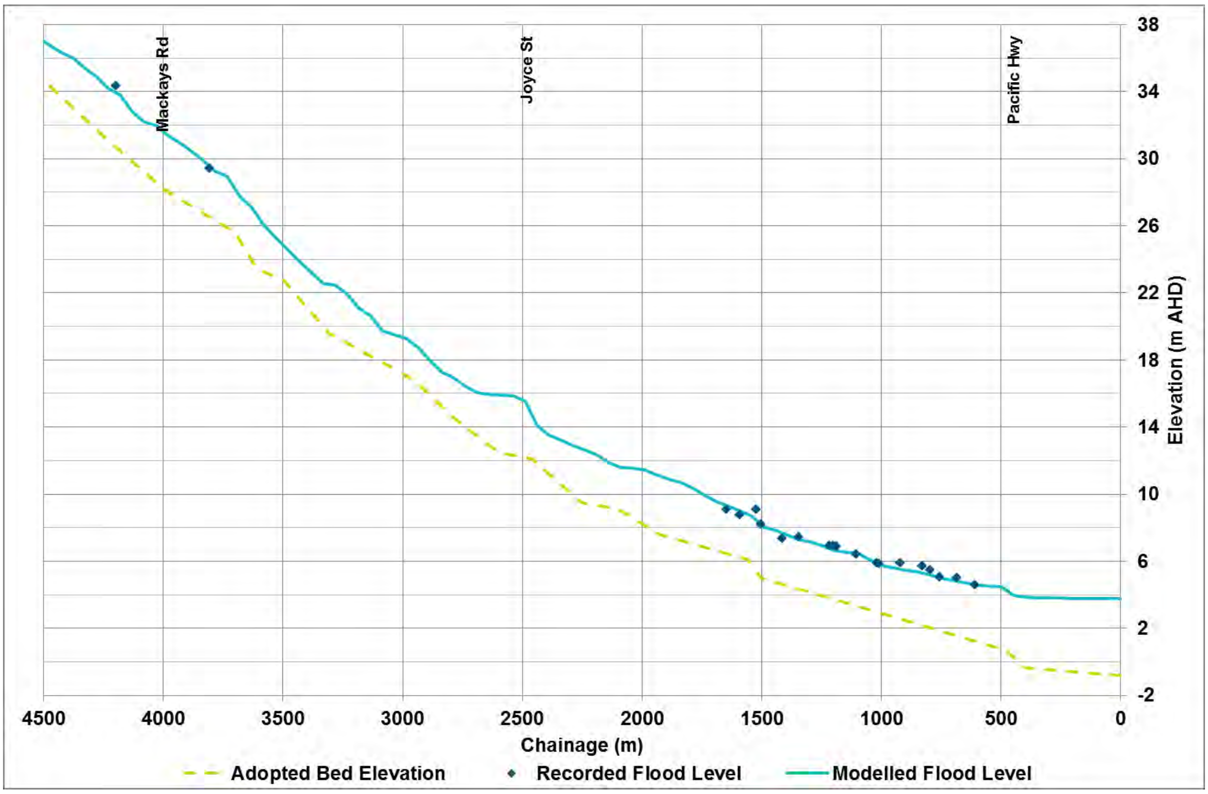


Figure 5-22 Long Section along the Argyll Street Arm of the Northern Tributaries for the March 2009 Calibration Event

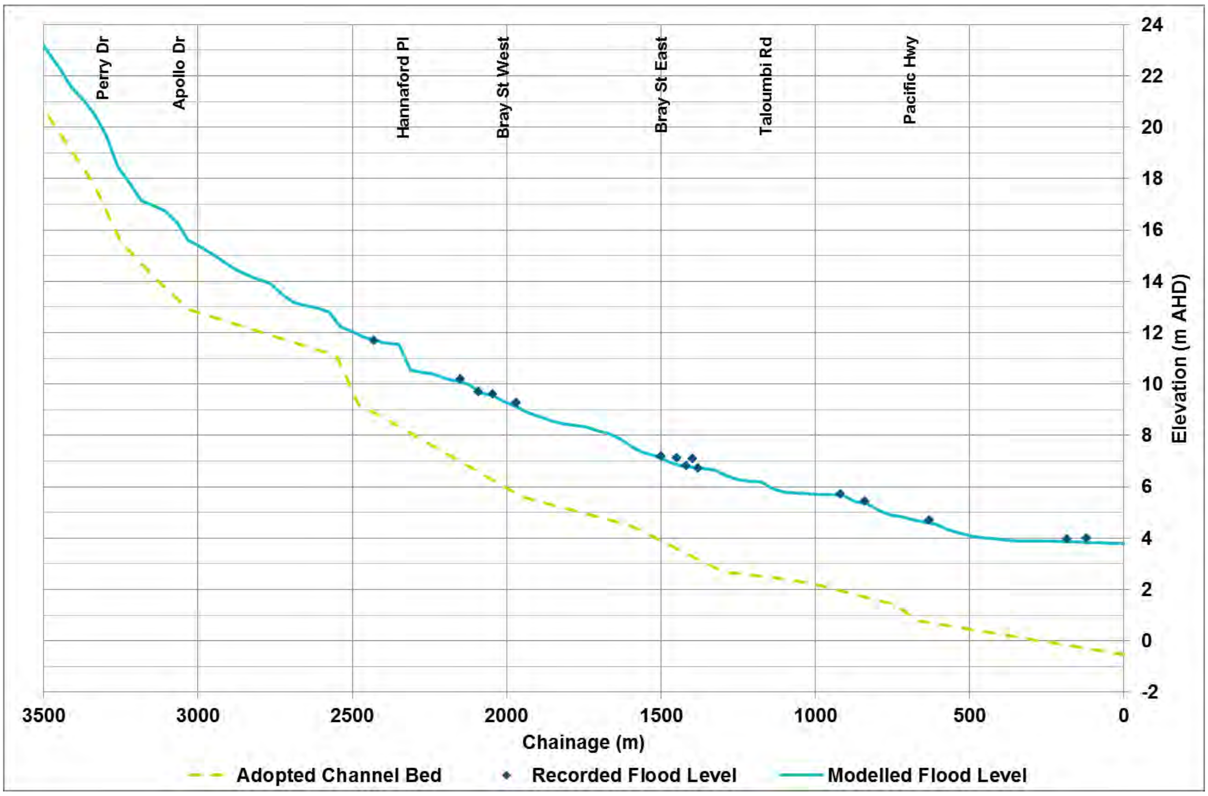


Figure 5-23 Long Section along the Bray Street Arm of the Northern Tributaries for the November 1996 Verification Event

The modelled flood level profile is comparable to most of the recorded flood marks for both the Bray Street and Argyll Street branches.

In Figure 5-22, the modelled flood profile upstream of Mackays Road falls approximately 0.5m below the recorded flood mark. The railway culvert upstream of Mackays Road has since been upgraded, but was previously an undersized 1.8m culvert. During the 1996 event, the railway overtopped and a section of the embankment washed away (Patterson Consultants Pty Ltd, 1997). Due to attenuation of the flood wave behind the embankment as a result of an undersized culvert, failure of the embankment could have released a large volume of water. As the channel is quite well defined in the location of the flood mark, the lower modelled flood level could be attributed to underestimation of the peak flow rate.

There are surveyed flood levels for properties along the Bray Street Arm, just downstream of Apollo Drive, in the area of Antaries Way / Apollo Street. These flood marks are located out of bank and there is uncertainty surrounding the accuracy of the location of these levels. Figure 5-24 shows the inundation extent and peak flood level contours modelled for the 1996 event against the recorded flood levels.

Uniform placement of recorded marks in the centre of lots indicates that this may not be the exact location that the flood level was surveyed on the lot, or that the exact location is unknown. The elevation of the flood marks recorded along Apollo Drive indicate that they are likely to have been recorded at the street-front of the property, where flooding would be resulting from spill-over from Perry Drive. Potential blockage of the Perry Drive crossing could have resulted in additional overland flooding in this location. Similarly, for the properties located along Antaries Ave adjacent to the creek line, the flood level displays a significant gradient across the lot and the location of the surveyed mark is very important in terms of achieving appropriate calibration.

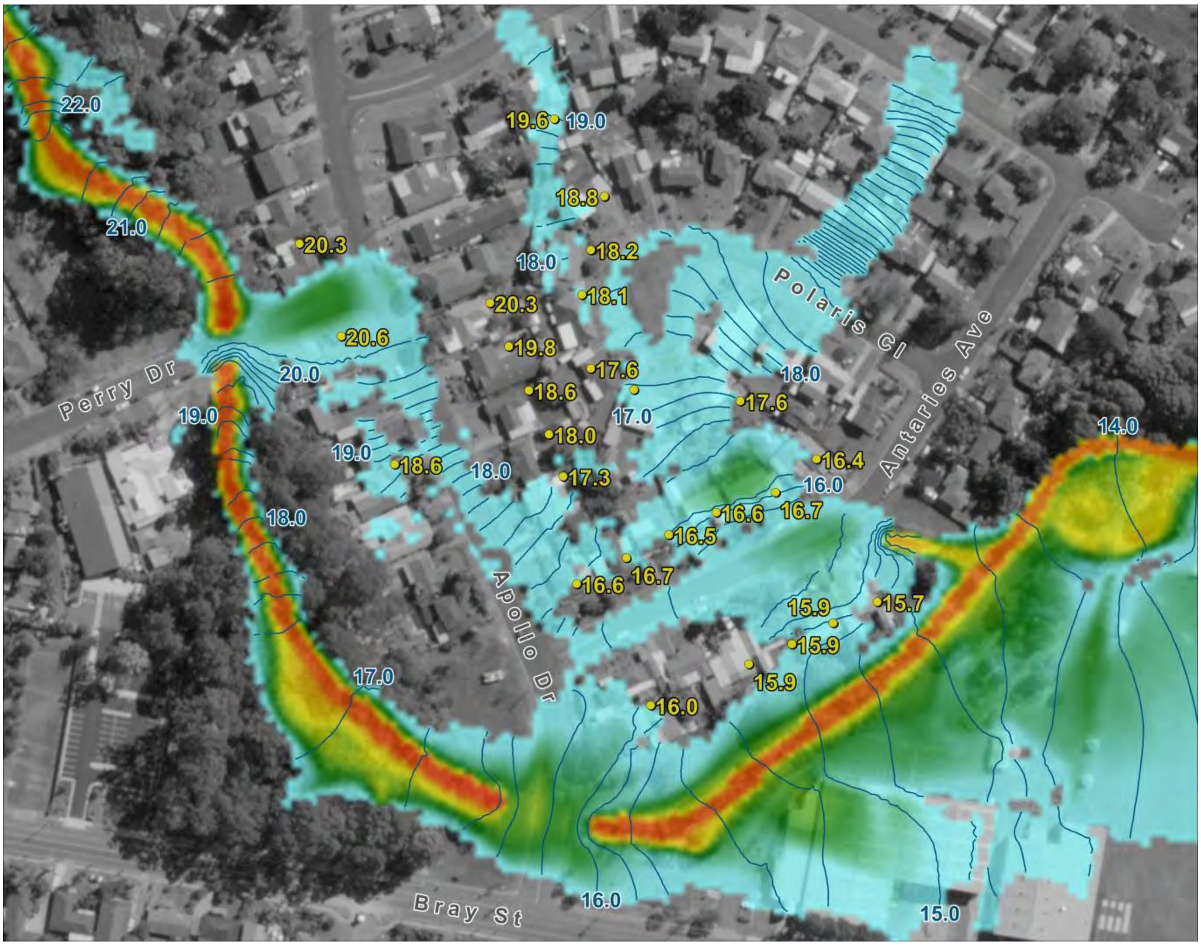


Figure 5-24 Model Calibration November 1996 Event, Bray Street

Park Beach

The November 1996 event lacked flood level marks for calibration in the Park Beach area. In addition, there is uncertainty regarding rainfall over Park Beach during the 1996 event as rainfall records are scant.

Figure 5-25 shows a long section along the Park Beach Plaza/Orlando Street drainage channel for the November 1996 event. The good match between modelled and recorded flood levels downstream of the Orlando Street bridge indicates that the downstream water level in Coffs Creek is representative of conditions during the event.

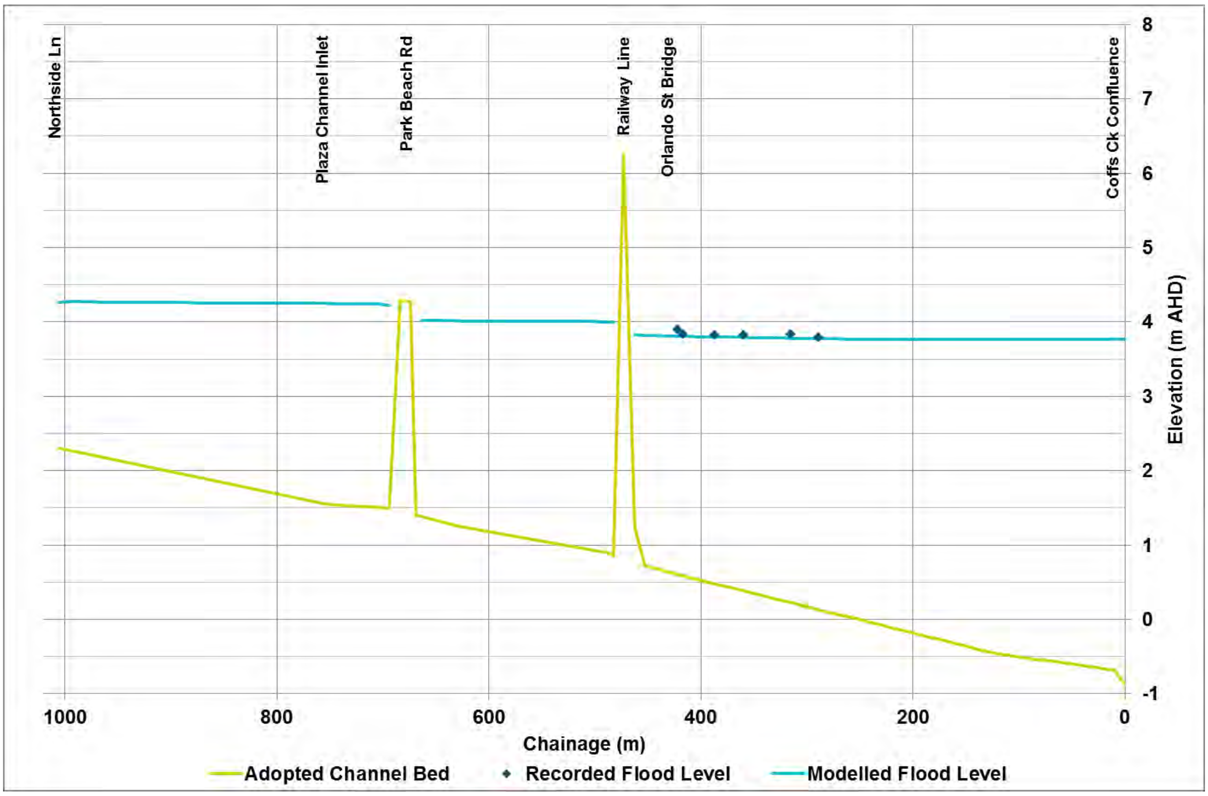


Figure 5-25 Long Section along the Park Beach Plaza/Orlando Street Channel for the November 1996 Calibration Event

5.4 March 2015 Model Verification

As previously mentioned, the March 2015 event was used to verify the model performance within the Park Beach catchment.

5.4.1 Rainfall Data

Following the 2009 event, Council installed eight additional pluviographs within the Coffs Creek catchment. Continuous rainfall data from the Macauley's Reservoir, Bray Street, Gundagai Place and Perry Drive gauges (location of gauges seen on Figure 2-2) was provided by Council for 13th March 2015. The March 2015 event resulted from short duration, high intensity rainfall over Park Beach and the lower catchment of the Northern Tributaries of Coffs Creek. The Bray Street gauge recorded just over 170mm, Macauley's Reservoir and Gundagai Place recorded just less than 140mm and around 120mm was observed further up the catchment at Perry Drive.

The temporal pattern recorded at each gauge is presented in Figure 5-26. The Bray Street and Macauley's Reservoir gauges are located in closest proximity to Park Beach and were assumed to provide the most representative conditions for the Park Beach catchment. Both gauges recorded rainfall totals increasing over the first 1.5 hours, with the highest half hourly intensity recorded at 2:00pm. As flooding within the catchment was concentrated to Park Beach (with minor flooding experienced in some areas of the Northern Tributaries), it was assumed that Park Beach would have experienced the highest total rainfall within the Coffs Harbour area. A total rainfall depth of

170mm was therefore assigned to the Park Beach sub catchments in the RAFTS hydrological model.

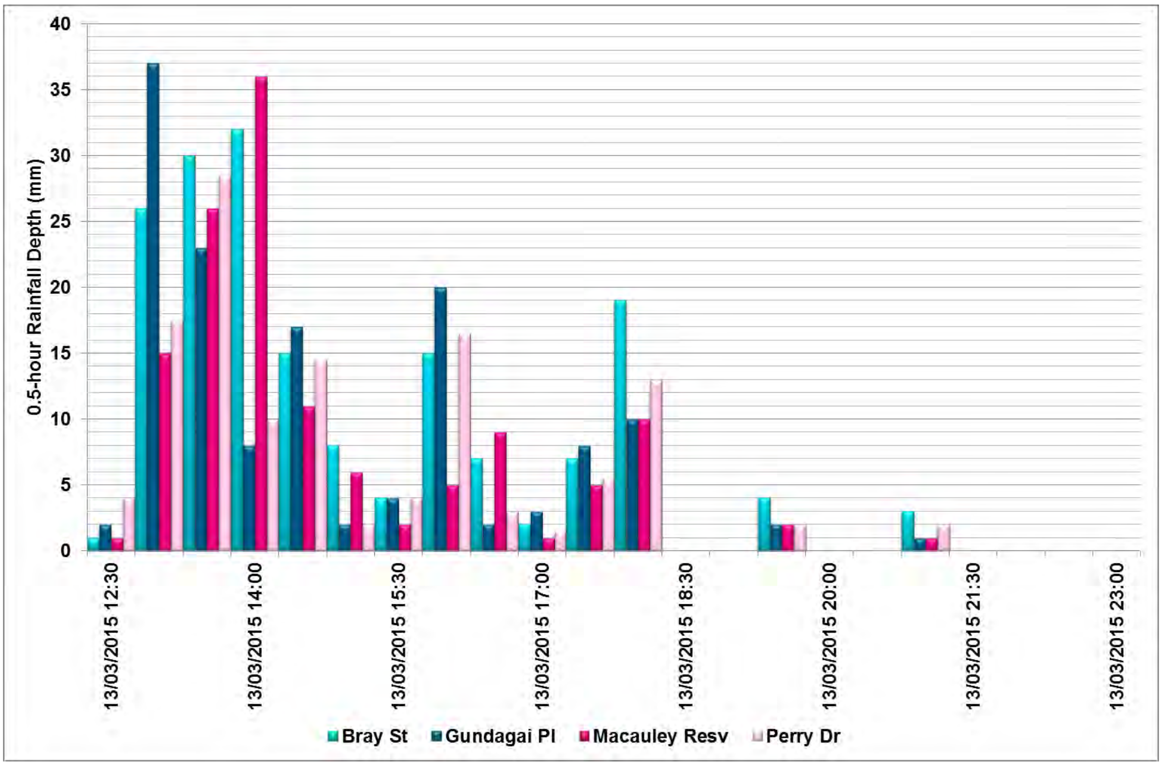


Figure 5-26 Rainfall Hyetograph for the March 2015 Verification Event

The March 2015 rainfall intensities are compared against IFD relationships for the Coffs Creek catchment in Figure 5-27. At the Bray Street gauge, the event was just below a 5% AEP (20 year ARI) for the 2-6 hour duration. Rainfall observed at the other gauges was relatively minor, and could be expected to occur around once every 5 years.

5.4.2 Antecedent Conditions

The main storm burst occurred during a period of little other rainfall. Therefore, an initial loss of 35mm/hr was adopted. This is the upper limit recommended for use in AR&R (2001), but is reasonable as there was negligible rainfall recorded in the days leading up to the event.

5.4.3 Downstream Boundary Conditions

The event was very minor on the main alignment of Coffs Creek, with water levels in Coffs Creek rising by around 1.0m. A water level hydrograph provided by Council recorded on Coffs Creek at the Scarbra Street/Gundagai Place Bridge was used to confirm tail water levels in Coffs Creek.

Rainfall was only applied over the lower Coffs Creek catchment and Park Beach area within the RAFTS hydrological model. Comparison of recorded and simulated water levels at the Scarbra Street bridge indicated that modelled levels were within 0.1m of recorded levels at the Scarba Street bridge.

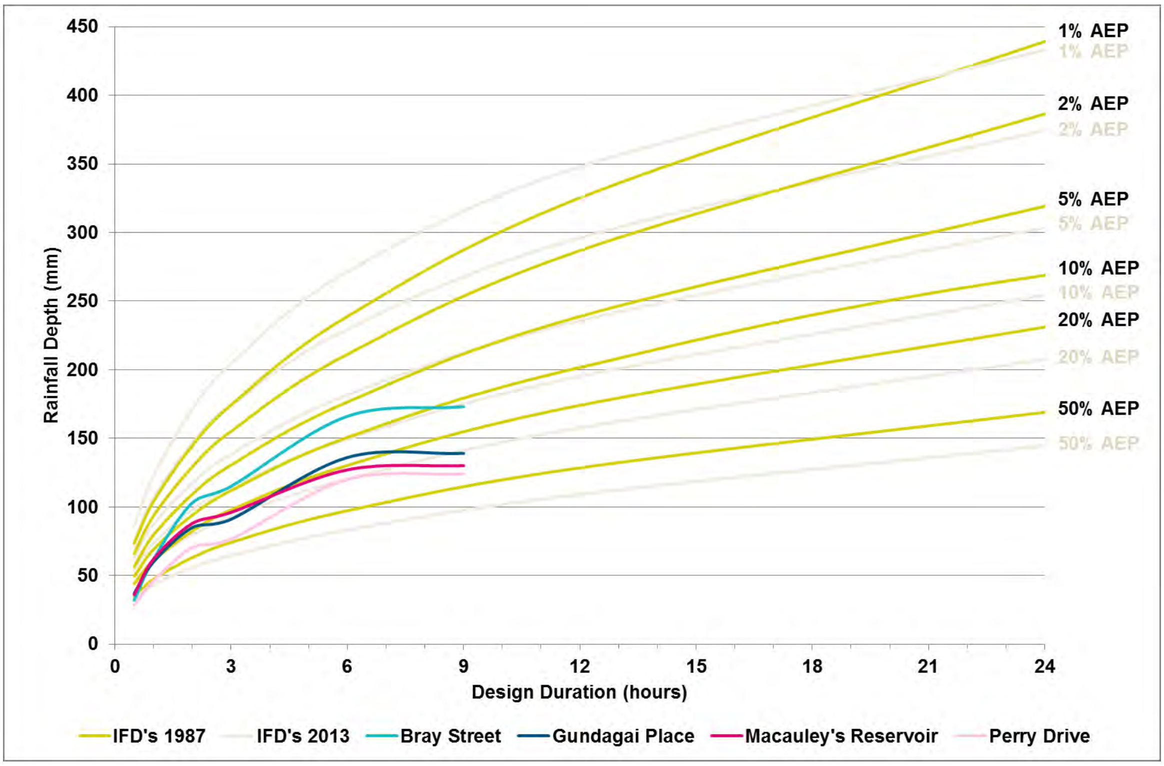


Figure 5-27 Comparison of Recorded March 2015 Rainfall with IFD Relationships

5.4.4 Observed and Simulated Flood Behaviour

Following the event, Council surveyed debris marks and flood levels in the area. Eighteen spot flood levels are available; however a few are thought to be unreliable. A Facebook page set up by community members shared information, including personal observations, photographs and video footage, during and after the event. The flood marks and photographic evidence were used for the model verification to the March 2015 event.

The Park Beach catchment is very flat, and drainage is controlled by the culverts under the railway. The channel downstream of the railway culvert closest to the Park Beach Plaza and the Orlando Street Bridge drains flows from the majority of the catchment. Figure 5-28 depicts the modelled peak flood level profile along this channel section against reliable surveyed flood marks for the March 2015 event. As seen in Figure 5-28, the model is accurately replicating observed flood behaviour during the Marc 2015 event.

The model performance over the remaining catchment area was also assessed. Figure 5-29 shows the modelled peak flood depth and peak water level contours across the entire Park Beach catchment. Location of the survey flood marks is displayed. A comparison of the modelled peak flood levels with the surveyed flood marks is presented in Table 5-6. The accuracy of survey point 15 is known to be questionable. With this in mind, of the 17 reliable surveyed flood marks, 13 points are within 0.1m. The remaining four points show differences of less than 0.2m.

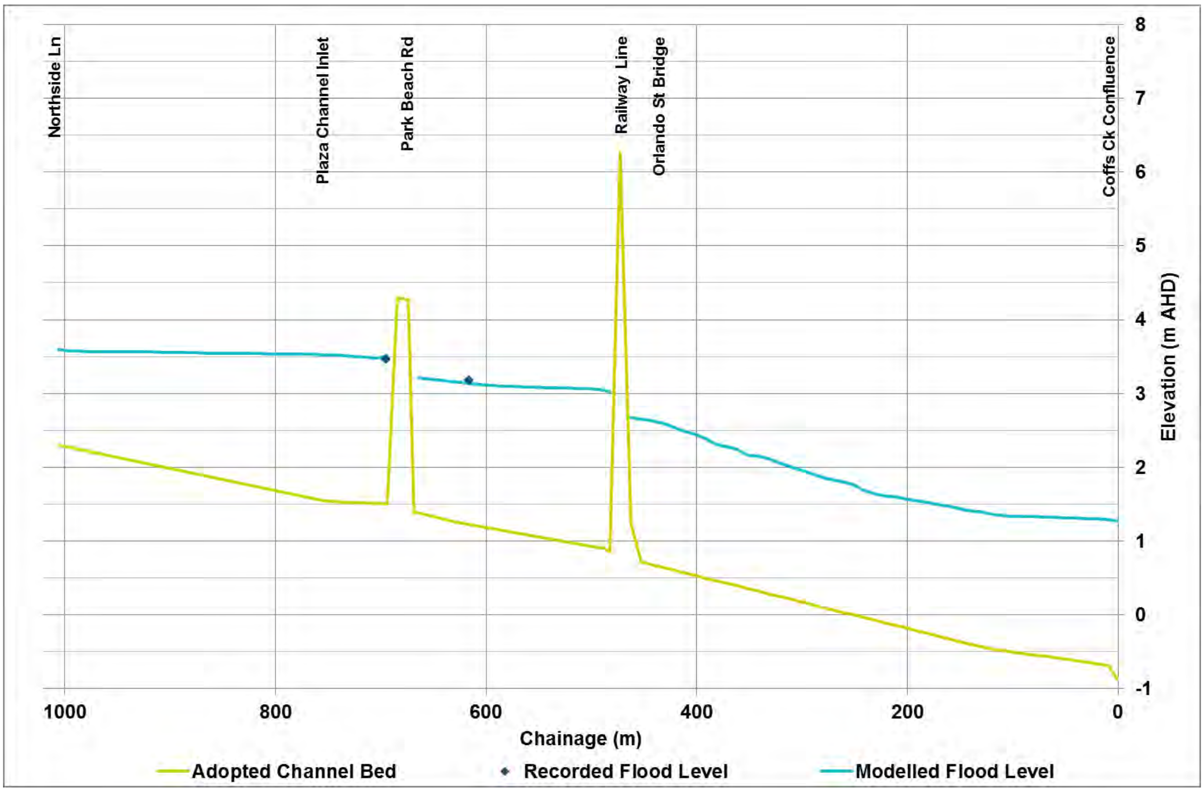
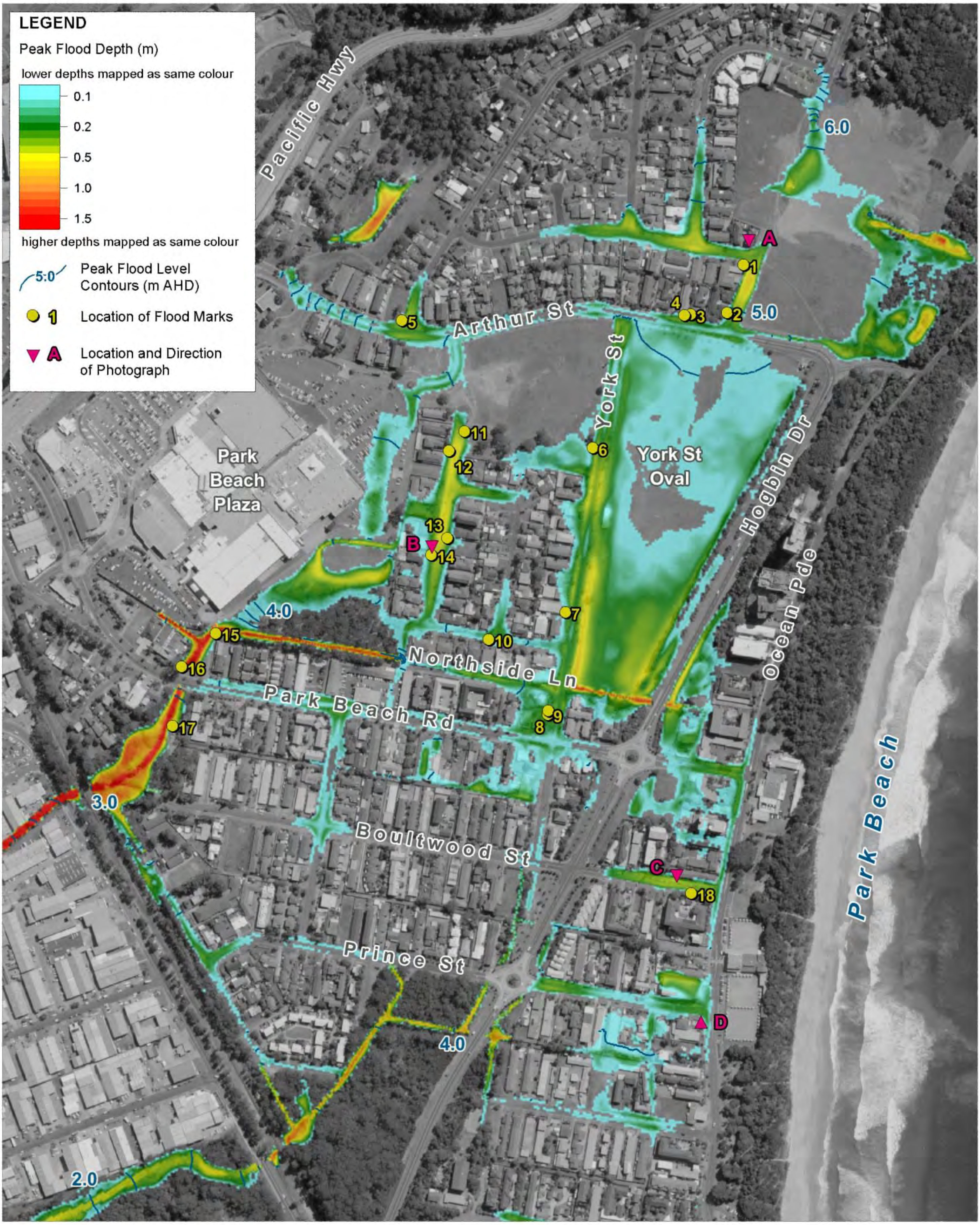


Figure 5-28 Long Section along the Park Beach Plaza/Orlando Street Channel for the March 2015 Calibration Event

Table 5-6 Flood Mark Survey Locations for the March 2015 Event – Park Beach

ID	Flood Level (m AHD)		ID	Flood Level (m AHD)	
	Surveyed	Modelled		Surveyed	Modelled
1	5.0	5.0	**10**	4.4	4.5
2	4.8	5.0	**11**	4.2	4.4
3	4.9	4.9	**12**	4.3	4.4
4	4.8	4.9	**13**	4.3	4.4
5	5.1	5.3	**14**	4.4	4.4
6	4.6	4.7	**15**	3.1	3.5
7	4.5	4.6	**16**	3.5	3.5
8	4.5	4.6	**17**	3.2	3.2
9	4.5	4.6	**18**	4.6	4.6

The location of a selection of the photographs taken from the community ‘Flood Watch’ Facebook Page is also included on Figure 5-29. These photographs are reproduced in Figure 5-30 and Figure 5-31. Although photographs provide useful flood information for the event, they may not be representative of exact peak flood conditions if there were not taken at the peak of the flood event.



Title:

March 2015 Model Calibration Park Beach

Figure: 6-29

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The flood photograph Location A has been taken on the corner of Karuah Avenue upstream of the York Street Oval, looking south down Richmond Drive. Cars are still negotiating the road so it is assumed flood depths are less than 0.5m, and look to be around 0.3m. A similar depth and pattern of inundation up and around the corner has been predicted by the model. San Francisco Avenue is a location frequently affected by flooding in Park Beach. Location B depicts a photograph taken in front of 11 San Francisco Avenue, looking to the south. Flood inundation extends up to the front of properties; however inundation beyond the road reserve appears to be less than around 0.1m above ground level. The model produces comparable results, as seen in Figure 5-29.

Anecdotal reports by community members on the Facebook page indicated that the eastern ends of Boultwood Street and Prince Street (toward Ocean Parade) experienced significant flooding and were not passable by car. Photo Location C shows flooding to the eastern end of Boultwood Street, looking across to Sandcastles Holiday Apartments. Photo Location D was taken from the corner of Prince Street and Ocean Parade, looking North toward Park Beach Bowling Club. Again, the depth of flood water and the extent of inundation evident in the photographs is replicated by the model.

As the nature of the catchment topography provides significant flood storage, flooding within the broader Park Beach area is largely a function of the total volume of rainfall. Due to the uncertainty surrounding the accuracy of the surveyed flood marks and the high variation in spatial distribution of rainfall evident during the March 2015 event, the model is assumed to provide good calibration over Park Beach.

Figure 5-30 March 2015 Photo Locations A and B

Figure 5-31 March 2015 Photo Locations C and D

6 Design Flood Conditions

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.

Refer to Table 6-1 for a definition of AEP.

Table 6-1 Design Flood Terminology

AEP	Comment
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.
10%	As for the 0.2% AEP flood but with a 10% probability.
20%	As for the 0.2% AEP flood but with a 20% probability.
50%	As for the 0.2% AEP flood but with a 50% 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.

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

- 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 inputs to the hydraulic model;
- Design downstream ocean boundary levels; and
- The impact of future climate change on ocean levels and catchment inflows.

In accordance with Council's brief, the design events to be simulated include the 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP and PMF event. The 1% AEP flood is generally used as a reference flood for development planning and control for residential development.

The adopted storm durations are discussed in Section 6.2.5. The adopted ocean downstream boundary conditions are discussed in Section 6.3.

6.1 Changes to the Model Configuration

The construction of three additional detention basins has occurred in the catchment since the 2009 event. As LiDAR survey used in this study was captured in 2013, the Bakers Road and the Bennetts Road detention basins (constructed 2010 and 2012-13 respectively) already exist within the 2D topography. Note that both of these basins were removed for simulation of the 2009 event. Design drawings for the Spagnolos Lane detention basin (constructed 2015) were provided by Council and were incorporated into the model geometry for all design runs. Plans for the proposed basin at Upper Shephards Lane were also provided to be included in alternate design scenarios to assess its potential for reducing the flood risk within the catchment.

Civil works are planned to upgrade the road surface and culvert crossing at William Sharp Drive (structure CCW-3). Proposed design plans were provided by Council. For all design simulations, the model DEM has been modified to include the proposed road geometry and the existing twin 3.1m x 1.8m culvert has been replaced with a twin 3.0m x 2.1m culvert.

6.2 Design Rainfall

Design rainfall parameters are derived from standard procedures defined in AR&R (2001) which are based on statistical analysis of recorded rainfall data across Australia. The methods were first presented in 1987 and therefore only consider rainfall data available up to this time. The derivation of location specific design rainfall parameters (e.g. rainfall depth and temporal pattern) for the study catchment is presented below.

6.2.1 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.

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. The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology.

Rainfall depths estimated from the IFD curves are point intensities and are in theory not applicable to large areas, as it is unlikely that the intensity of rainfall would be sustained over entire catchments. For catchments larger than 4km^2, areal reduction factors (ARF’s) are usually applied in accordance with methods presented in AR&R (2001). However, research specific to the Coffs Creek catchment has shown that rainfall intensity can vary significantly across the catchment. The scaling factors applied to design rainfall (discussed in Section 6.2.2) account for the variation in intensity across the catchment, relative to rainfall totals recorded at a point and negate the need to apply an ARF.

Table 6-2 shows the design rainfall intensities calculated at the centre of the Coffs Creek catchment from the methods first presented by AR&R in 1987. Discussion regarding current use of the 2013 IFDs and their differences against the 1987 IFDs was included in Section 5.2.1.

Table 6-2 Average Design Rainfall Intensities (mm/hr)

Duration (hrs)	Design Rainfall Intensities (mm/hr)				
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP
0.5	87.9	98.8	113	132	147
1	61.2	69.2	79.9	94.0	105
2	41.5	47.4	55.1	65.3	73.2
3	32.9	37.8	44.1	52.5	59
6	22.1	25.6	30.1	36.1	40.9
12	14.9	17.4	20.5	24.9	28.2
24	10.0	11.8	14.0	17.0	19.4

6.2.2 Rainfall Gradients

The rainfall gradient typically observed across the catchment during large events is so localised that it is not captured by existing official gauges used to derive IFD estimates.

The Coffs Creek Flood Study (WMA, 2001) reviewed historical rainfall events and determined that IFD data for the catchment should be scaled to reflect the observed increase in rainfall intensity at higher elevations within the catchment. The rainfall gradient ratios adopted for simulation of design events in the previous study is presented in Table 6-3.

Table 6-3 Rainfall Gradient Ratios Adopted in Previous Study (WMA, 2001)

Elevation (m AHD)	Gradient Ratio (Relative to Airport)		
	Lower Bound	Best Estimate	Upper Bound
20	1.00	1.00	1.00
80	1.05	1.30	1.60
140	1.10	1.60	2.20
200	1.15	1.90	2.80
400	1.20	2.25	3.30

As part of this current study, these rainfall gradients were reviewed. The 2009 event provided key additional data for the historical rainfall assessment, due to the installation of three additional continuous rainfall gauges located in the upper catchment in the early 2000s (Perry Drive, Shephards Lane and Red Hill).

The rainfall gradient assessment completed for this study utilised rainfall records (private and official) available for the 1996 and 2009 events. Analysis of the 2009 event was the primary focus, as a larger data set containing more official/reliable records was available. For both the 1996 and 2009 events, a 'base' rainfall was observed to be present across the middle catchment area. From this, a total of four rainfall zones were identified to exist across the catchment. Each zone exhibited a consistent rainfall ratio relative to the base zone for both the 1996 and 2009 events. The four zones identified are:

- **Rainfall Zone A:** Coastal zone where there is no clear pattern to the rainfall distribution, other than that rainfall totals were typically lower than the base rainfall total. Rainfall in this zone was observed to be highly variable. For calibration events, the use of a scaling factor was not deemed appropriate – rather, sufficient coverage of recorded rainfall data enabled spatial distribution of total rainfall to be reasonably well estimated. For design rainfall estimates, a scaling factor of 1.0 will apply – to ensure rainfall totals in the lower catchment and Park Beach are not underestimated for events that do not exhibit this type of rainfall gradient pattern (e.g. the November 2009 event and March 2015 event).
- **Rainfall Zone B:** Areas across the centre of the catchment that exhibit a fairly consistent rainfall total, identified as the 'base depth'. The base rainfall depth identified for the 1996 event and the 2009 event was 300mm and 350mm, respectively. For design rainfall estimates, the base depth will be as determined from IFD data calculated for the centre of the catchment.
- **Rainfall Zone C:** Elevated areas within the catchment that exhibit consistently higher rainfall depths than the adopted base. Scaling factor of 1.2 identified as suitable to apply to the base depth for both calibration events and design events.
- **Rainfall Zone D:** Area at the foot of the escarpment that is likely to receive the heaviest rainfall during events, as was observed to occur during both the 1996 and 2009 events. A scaling factor of 1.6 was identified as being suitable to apply to the base depth for both calibration events and design events.

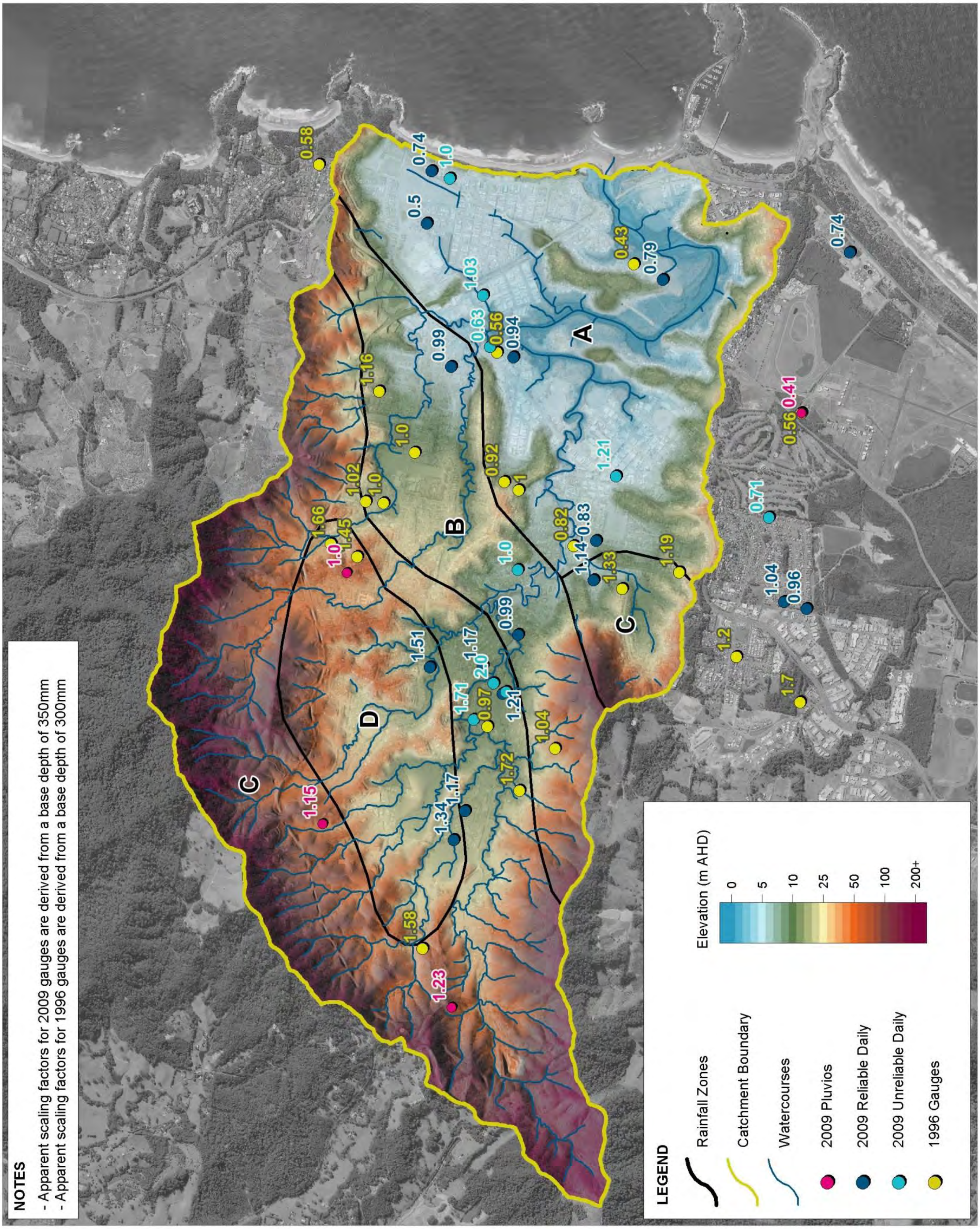
The distribution of each Rainfall Gradient Zone across the catchment is presented in Figure 7-1. Also shown is the apparent scaling factor for each rainfall gauge used in the analysis, which have been derived from the base depth identified for each event.

The allocation of Rainfall Gradient Zones in this way is supported by literature sources detailing orographic rainfall patterns. Rainfall behaviour that is influenced by the underlying topography is known as orographic rainfall. Mountainous topography can affect rainfall in numerous ways, depending on the type of storm, where it forms and how it moves over the terrain (Houze, 2012).

Investigation into the meteorological aspects of the March 2009 event concluded that severe rainfall was 'anchored over the hills' for several hours. Houze (2012) describes this as 'blocking' or 'damming' – a behaviour that often occurs when low-level air is present ahead of the system, as was the case for the 2009 event (Speer, Philips and Hanstrum, 2011). This type of orographic rainfall, where the most intense rainfall is observed at the base of the escarpment, is shown simply in Figure 6-2.

6.2.3 Temporal Pattern

The IFD data presented in Table 6-2 provides 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



Title:
Rainfall Gradient Zones

Figure: 6-1

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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0 0.75 1.5km
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Filepath : K:\N20327_Coffs_Creek\MapInfo\Workspaces\Figures\DRG_00X_160217_Rainfall_Gradient.WOR patterns adopted in the current study are based on the standard patterns presented in AR&R (2001).

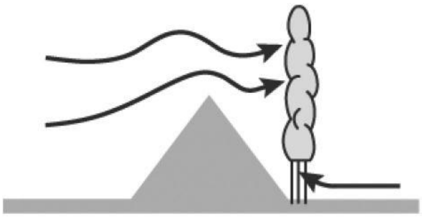


Figure 6-2 One example of orographic rainfall

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.

6.2.4 Rainfall Losses

The hydrologic model parameters adopted for the design floods were based on the initial and continuing loss model, with a continuing loss of 2.5mm/h as recommended in AR&R (2001). For the initial loss AR&R recommends values between 10mm and 35mm for eastern NSW.

An initial loss of 0mm was used in the Coffs Creek Flood Study (2001) for historical and design events. With this in mind, an initial loss of 10mm was adopted for use in this study as the lower bound recommended by AR&R.

6.2.5 Critical Duration

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 range of storm durations were modelled in order to identify the critical storm duration for design event flooding in the catchment. The duration producing the highest flow rate out of the hydrological model may not necessarily result in the peak flood level in the hydraulic model as catchment characteristic come into play. Storage effects of floodplain topography may attenuate the flood wave as it moves down the catchment. Durations producing a greater volume of floodwater may result in higher flood levels, as opposed to the duration that produces the peak flow rate.

The 1% AEP flood event was run for all durations to determine the critical duration for each location in the study area. Durations ranging from 90 minutes to 24 hours were assessed. The scaling factors calculated for design rainfall estimates were only applied for durations up to 24 hours and given the size of the catchment, a duration of less than 24 hours is expected to be critical. Park Beach is an exception, where flooding is more volume driven, as discharge from the catchment is driven by drainage under the railway.

The critical duration for the upper reaches was found to be the 2 hour storm, whereas for lower catchment areas the 9 hour storm was critical. Adopting both the 2 hour and 9 hour storm durations provided the critical condition across most of the modelled area. In locations where the 2 hour or 9 hour storm was not the critical duration (e.g. Park Beach and the upper reach of the Argyll Street arm), the peak flood level of the critical duration was typically less than 10mm greater than that of the peak flood level for the 2 hour or 9 hour storm duration.

The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology. The critical storms using this method were found to be the 2, 3 and 6 hour durations for the upper Creek reaches, middle Creek reaches and lower catchment areas (including Park Beach), respectively.

6.2.6 Climate Change

Current research predicts that a likely outcome of future climatic change will be an increase in flood producing 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 Coffs Creek 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.

6.3 Design Ocean Boundary

Design ocean boundaries for use in flood risk assessments are recommended by the Flood 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 different design ocean peak water levels are to be adopted based on the type of ocean 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. Coffs Creek. 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 north of Crowdy Head are presented in Table 6-4. The different peak levels reflect the degree of influence of wave set up applicable to the various types of entrances.

Table 6-4 Design Peak Ocean Water Levels (OEH, 2015) for Various Entrance Types, located North of Crowdy Head

Ocean Event	Peak Ocean Water Level (m AHD)		
	Entrance Type A	Entrance Type B	Entrance Type C
5% AEP	1.5	2.0	2.45
1% AEP	1.55	2.1	2.65

The entrance at Coffs Creek is 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 6-3 presents the design ocean water level time series for entrance Type B along with the High High Water Springs (Solstice Spring) (HHWS(SS)) time series, as applicable for locations north 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=8 hours.

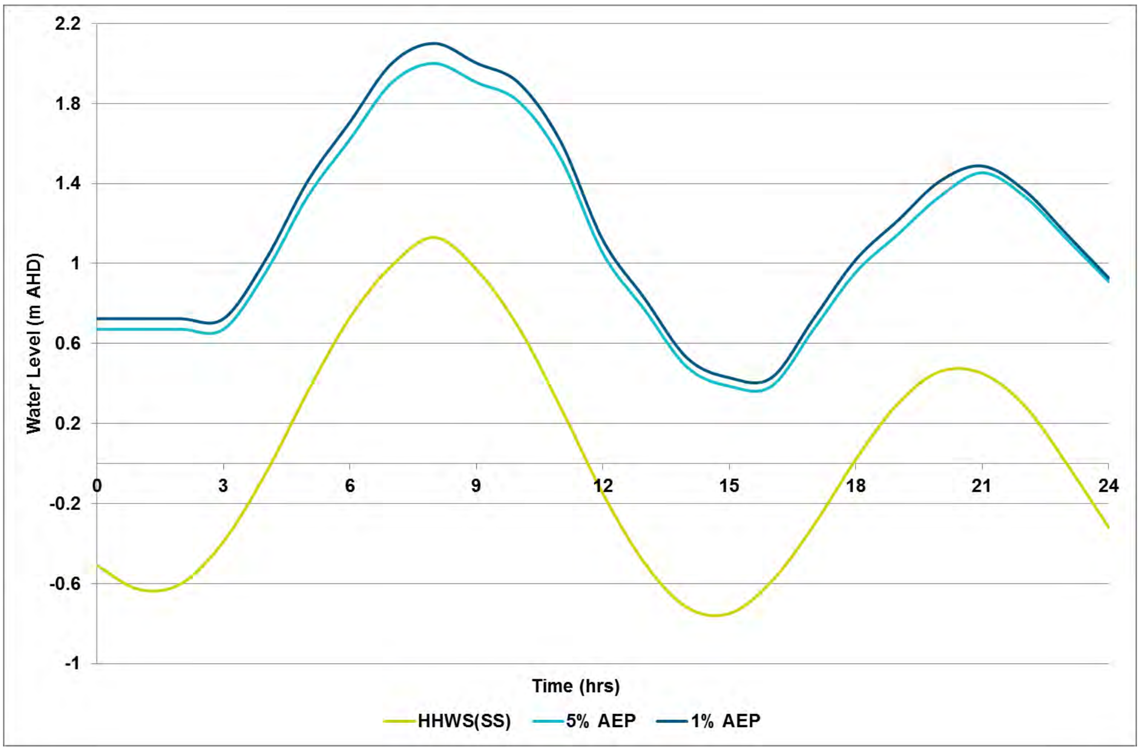


Figure 6-3 Design Ocean Water Level Time Series for Entrance Type B, located North of Crowdy Head (OEH, 2015)

The ocean boundary level recommended by OEH (2015) for each design catchment flood scenario is presented in Table 6-5 and has been adopted for design simulations in this study.

Table 6-5 Design Peak Ocean Water Levels

Catchment Event	Ocean Event	Peak Ocean WL (m AHD)
5% AEP	HHWS	1.13
2% AEP	5% AEP	2.0
1% AEP	5% AEP	2.0
0.5% AEP	1% AEP	2.1
0.2% AEP	1% AEP	2.1
PMF	1% AEP	2.1

6.3.1 Climate Change

The NSW Sea Level Rise Policy Statement (DECCW, 2009) provided projected increases in mean sea level for NSW of 0.4m and 0.9m, by the years 2050 and 2100 respectively. These increases are no longer prescribed by the state government but have been adopted for the purpose of this study in the absence of other suitable recommendations. Therefore, design ocean boundaries have been raised by 0.4m and 0.9m to assess the potential impact of sea level rise on flood behaviour in the Coffs Creek catchment.

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 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and 0.2% AEP. The PMF event has also been modelled. The impact of future climate change on flooding in the study catchment was also considered for the 1% AEP design flood event.

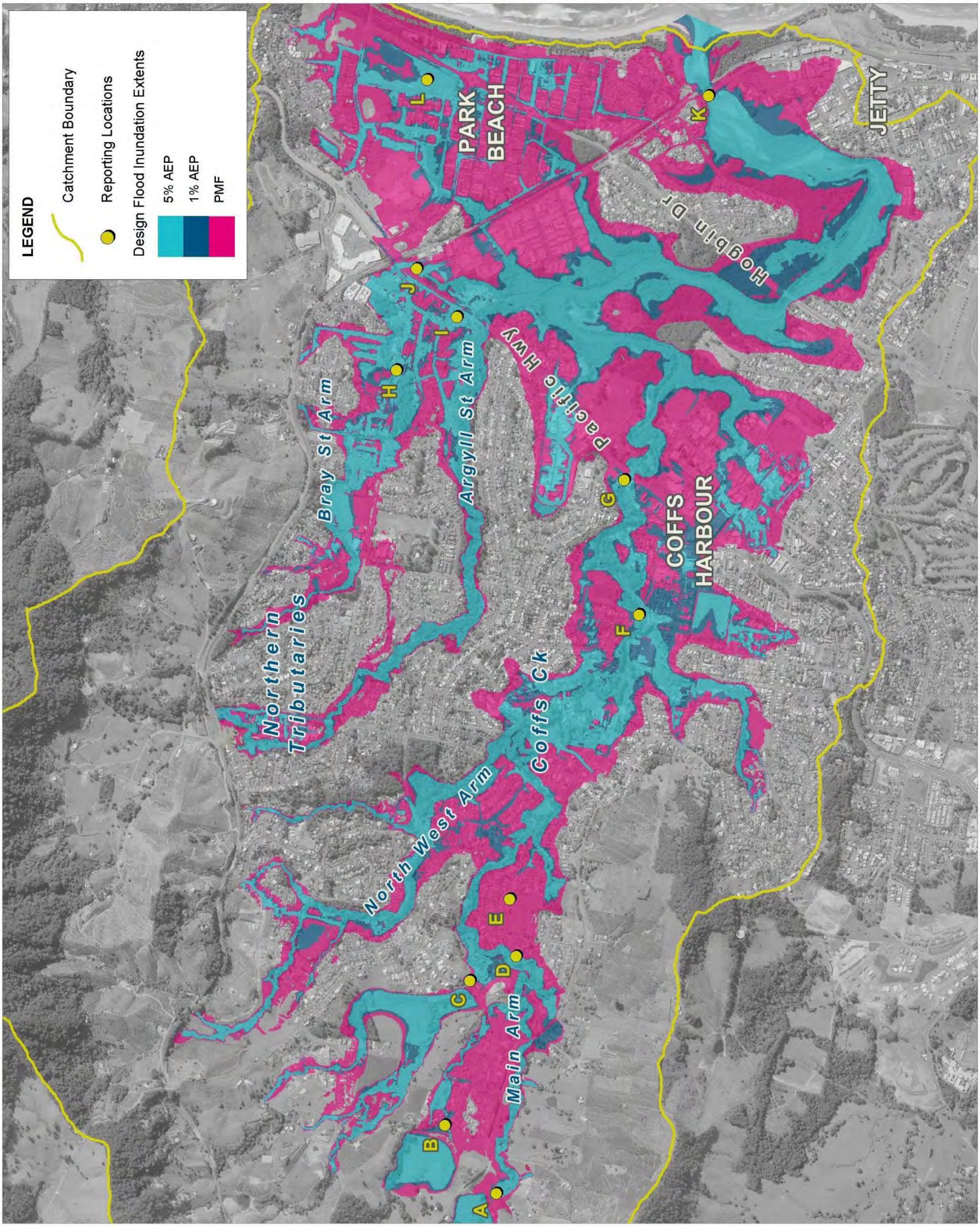
The design flood results are presented in a separate mapping compendium. For the simulated design events including the 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF events, a map of peak flood level, depth and velocity is presented covering the modelled area.

7.1 Peak Flood Conditions

Predicted flood levels at selected locations (as presented in Figure 7-1) are shown in Table 7-1 for the full range of design flood events considered. Longitudinal profiles showing predicted flood levels along Coffs Creek (Main Arm and North West Arm) are shown in Figure 7-2 and Figure 7-3. Similar longitudinal profiles along the Northern Tributaries (Argyll Street Arm and Bray Street Arm) are shown in Figure 7-4 and Figure 7-5.

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

ID	Location	Flood Event Frequency					
		5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
A	Bennetts Rd detention basin U/S (CHCC water level gauge site)	27.1	27.9	28.6	28.9	29.1	29.8
	Bennetts Rd detention basin D/S	22.3	22.3	22.3	22.6	22.7	23.9
B	Spagnolos Rd detention basin U/S	22.4	23.2	23.6	23.8	24.0	24.8
	Spagnolos Rd detention basin D/S	18.1	18.2	18.4	18.7	18.8	19.4
C	Bakers Rd detention basin U/S	16.2	16.7	17.1	17.4	17.7	18.7
	Bakers Rd detention basin D/S	13.4	13.6	13.7	14.2	14.4	15.4
D	Loaders Ln (CHCC water level gauge site)	13.8	13.9	14.0	14.4	14.5	15.5
E	Loaders Ln	-	-	-	11.6	12.3	13.6
F	Gundagai St (CHCC water level gauge site)	6.2	6.6	6.8	6.9	7.0	8.3
G	Pacific Hwy, Coffs Ck (MHL water level gauge site)	4.4	4.7	4.9	5.1	5.4	6.7
H	Bray Street (CHCC water level gauge site)	7.0	7.0	7.1	7.1	7.2	7.7
I	Pacific Hwy, NT’s	4.3	4.4	4.4	4.4	4.5	6.0
J	Orlando St, NT's (SES DipStik site)	4.4	4.5	4.5	4.6	4.7	6.0
K	Orlando St, Coffs Ck	1.2	2.0	2.0	2.1	2.2	4.5
L	York St Playing Field	4.7	4.7	4.7	4.8	4.8	6.0



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Design Flood Inundation Extents and Reporting Locations

Figure: 7-1

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Approx. Scale

0 0.5 1km

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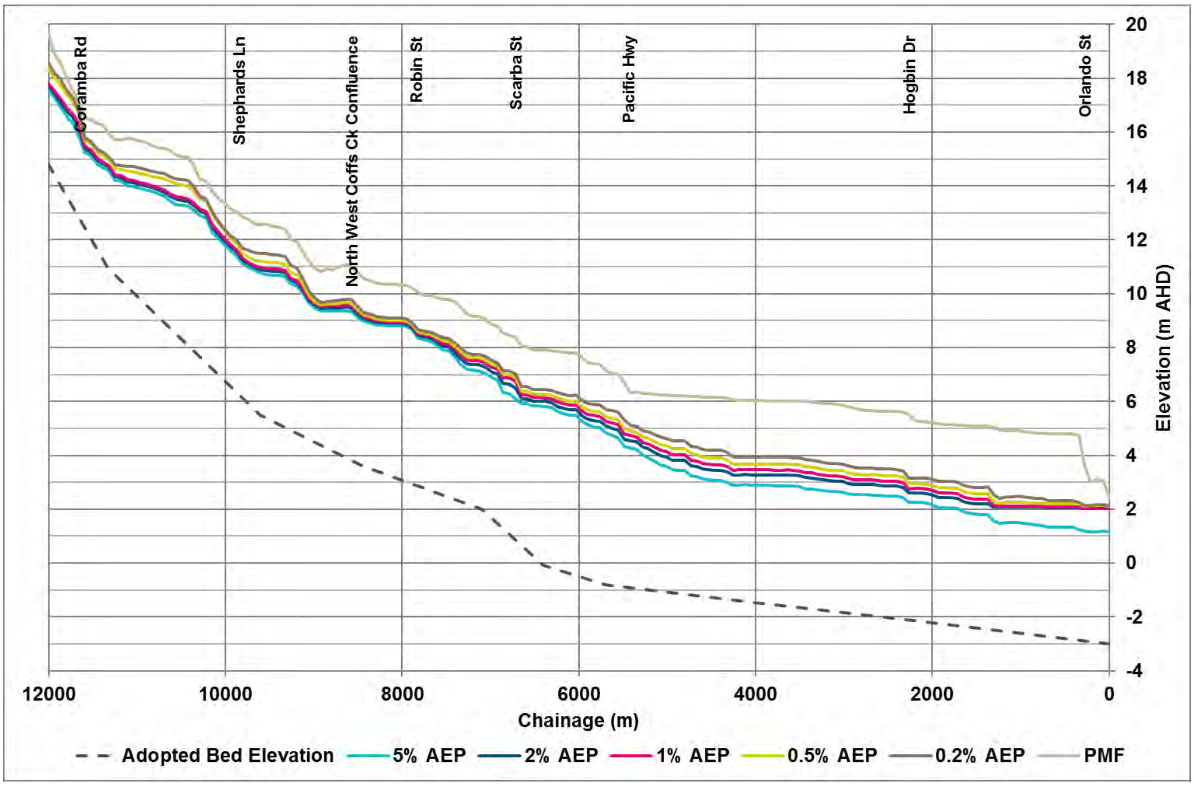


Figure 7-2 Long Section along the Coffs Creek Main Arm for Design Flood Events

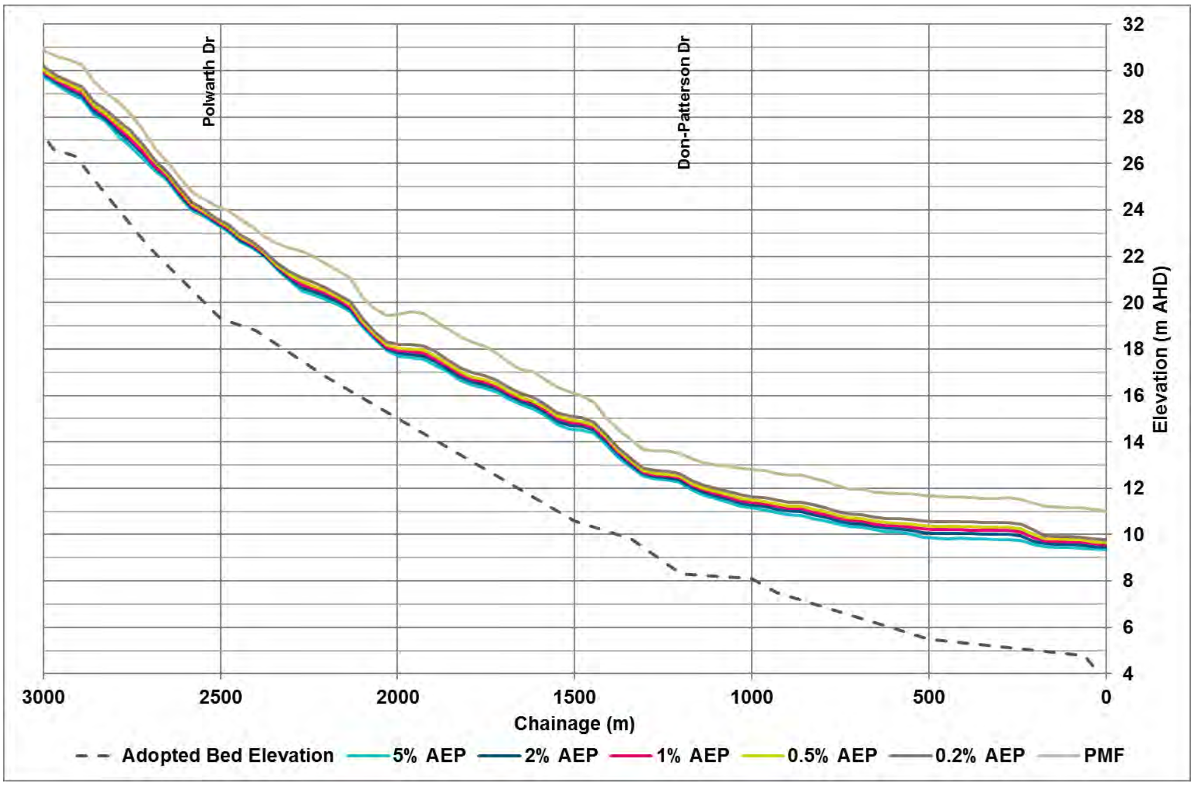


Figure 7-3 Long Section along the Coffs Creek North West Arm for Design Flood Events

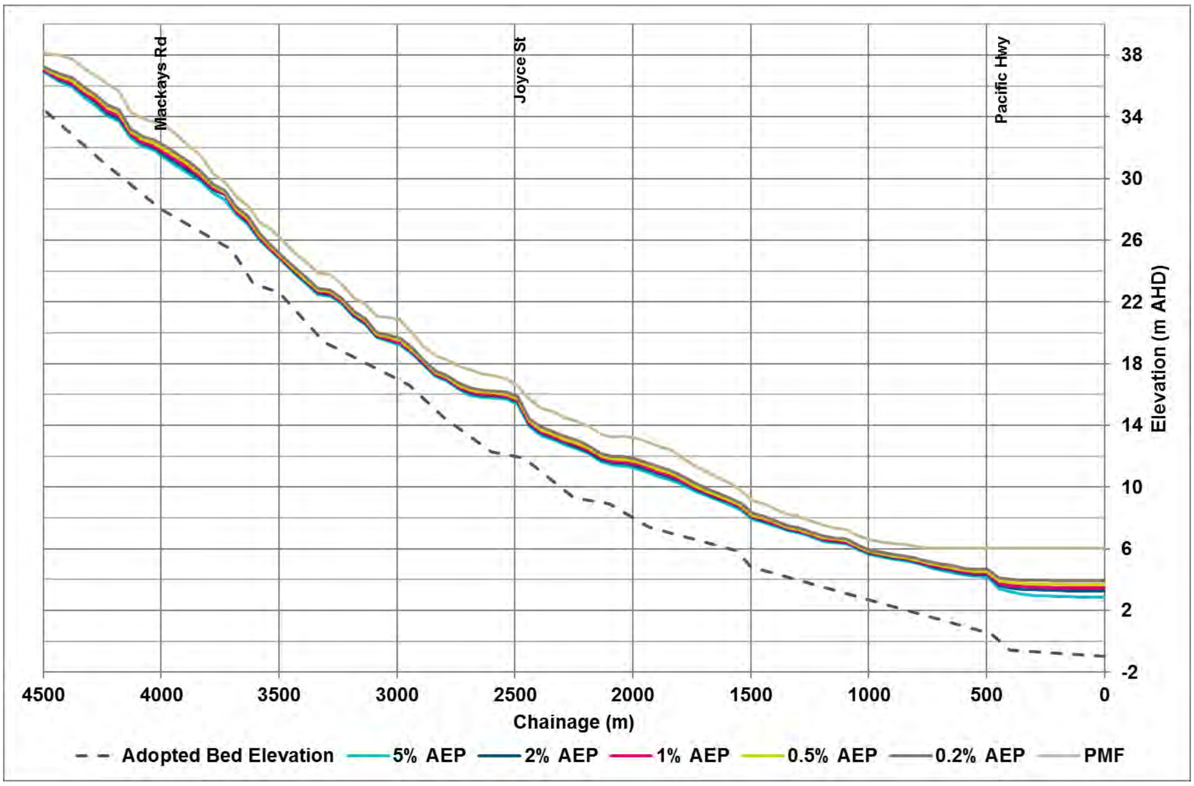


Figure 7-4 Long Section along the Argyll Street Arm of the Northern Tributaries for Design Flood Events

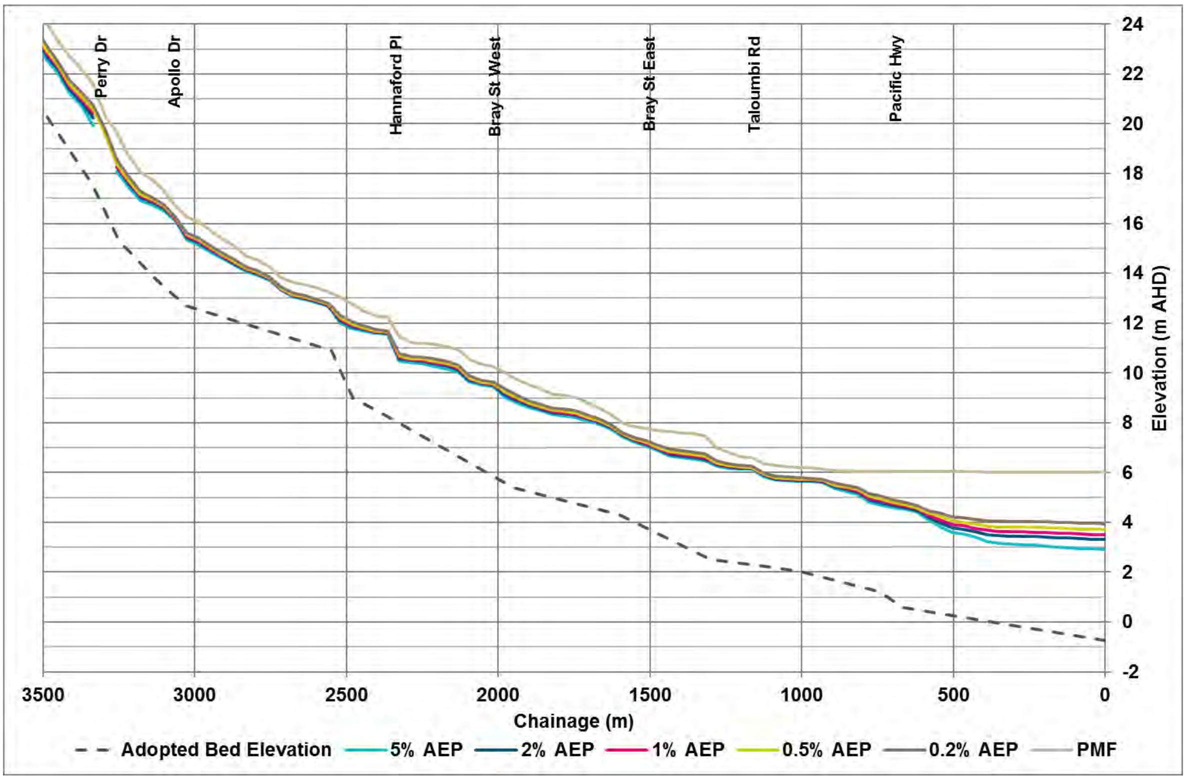


Figure 7-5 Long Section along the Bray Street Arm of the Northern Tributaries for Design Flood Events

Figure 7-1 shows the design flood inundation extents for the 5% AEP, 1% AEP and PMF events. Significant areas of urban development, including residential and commercial properties are flood affected at the 5% AEP event. The flood extents for the 5% AEP event and 1% AEP event are broadly similar, albeit with some additional flood flow paths becoming active, especially in the following locations:

- Just upstream of the confluence of the Coffs Creek Main Arm and the North West Arm, across Shephards Lane through to Adelines Way;
- The CBD, particularly around West High Street upstream of Scarbra Street; and
- Across the Pacific Highway, to the south of the Coffs Creek bridge along Harbour Drive and Park Ave.

The inundation extents for the PMF event show a much increased area at risk to flooding. Extents of inundation in the lower catchment areas significantly increase, with almost all of Park Beach, the industrial area west of Orlando Street and the CBD downstream of the Pacific Highway (Coff Street and Harbour Drive) flood affected. During the PMF, typical flood depths of around 1.2m would be expected in Park Beach. In the upper reaches of Coffs Creek, a large number of additional residential properties will become flood affected during the PMF than compared with the 1% AEP event. Areas under increasing risk include around the North West Arm confluence (Adelines Way and Moreton Bay Ave and further downstream to Red Cedar Drive) and properties between Bakers Road and Spagnolos Road. Along the Northern Tributaries, an additional flow path becomes active between the Argyll Street Arm and Bray Street Arm, across Oxley Place and Joyce Street.

Peak in-channel flood velocities are typically around 1.0m/s to 2.0m/s for the 1% AEP event, being lower in the floodplain areas. Flood velocities on the developed floodplain areas are typically less than 0.5m/s, but may be locally high around control structures and on roadways. Floodwater in Park Beach is slow moving, with typical velocities of around 0.1m/s to 0.2m/s expected.

7.2 Flood Flows

Predicted peak flood flows at selected locations (as presented in Figure 7-1) are shown in Table 7-2 for the full range of design flood events considered.

The flood flow hydrographs for the modelled events at the Coffs Creek gauge are presented in Figure 7-6. The hydrographs at the creek gauge are taken from the 9 hour duration storm, as this is the critical event at that location. They peak at around 6 hours after the onset of the storm. The PMF event that resulted in the largest flow rate at the Coffs Creek gauge was the 6 hour event. This storm has a rapid response, with a peak flow rate of just over 800m^3/s occurring around 3 hours after the onset of the storm.

Table 7-2 Modelled Peak Flood Flows (m^3/s)

ID	Location	Flood Event Frequency					
		5% AEP	2% AEP	1% AEP	0.5% AEP	0.2% AEP	PMF
A	Bennetts Rd detention basin D/S	24	26	30	54	67	272
B	Spagnolos Rd detention basin D/S	9	10	16	29	44	177
C	Bakers Rd detention basin D/S	18	19	20	27	38	192
D	Loaders Ln (CHCC water level gauge site)	49	57	63	92	119	426
E	Loaders Ln (residential area)	63*	84*	89*	130	170	568
F	Gundagai St (CHCC water level gauge site)	138	160	181	205	237	699
G	Pacific Hwy, Coffs Ck (MHL water level gauge site)	142	160	182	210	246	834
H	Bray Street (CHCC water level gauge site)	45	55	63	72	89	261
I	Pacific Hwy, NT's	86	104	120	139	170	519
J	Orlando St, NT's (SES DipStik site)	30	35	39	43	47	88
K	Orlando St, Coffs Ck	239	300	327	362	401	817

**peak flow rate is within channel (i.e. no overland flow across Loaders Ln)*

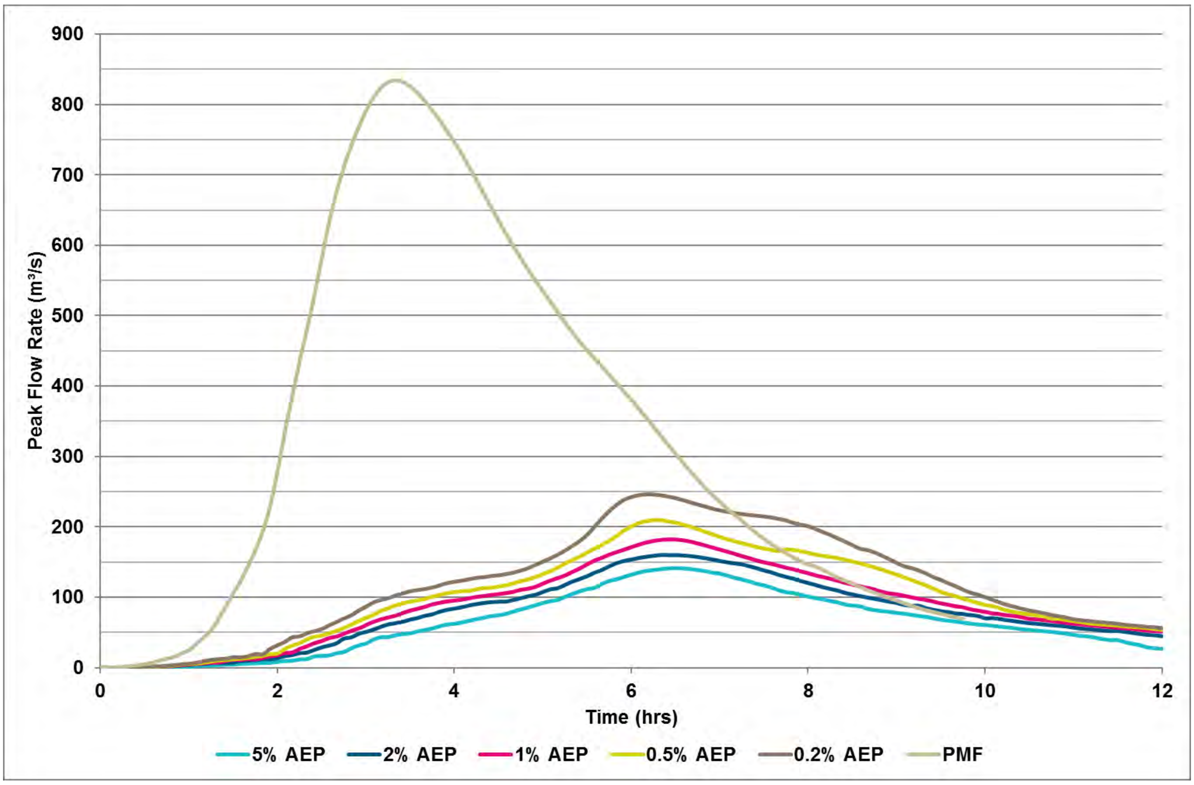


Figure 7-6 Modelled Design Event Hydrographs at Coffs Creek Gauge

7.3 Flood Mitigation Works

Since completion of the previous Flood Study in 2001, a number of flood mitigation measures have been constructed across the catchment. These include the Bennetts Road, Bakers Road and Spagnolos Road detention basins. The basins will have altered flood behaviour in terms of inundation extents, peak flood levels and peak flows across the catchment from design flood results defined in previous studies. The impact of these basins on design flood levels is reviewed in this Section. The potential benefit of construction of a fourth detention basin at Upper Shephards Lane is also assessed.

This section also reviews the performance of the following two levees within the catchment:

- Loaders Lane on the Coffs Creek Main Arm (constructed prior to 1996); and
- Langker and Collice Place on the Bray Street Arm of Northern Tributaries (constructed in the early 2000s).

7.3.1 Detention Basins

In order to assess the performance of the detention basins, an alternate model scenario was simulated where the three detention basins were removed from the model domain. This involved replacing the 2D DEM in the vicinity of the basins to be derived from 2007 LiDAR (surveyed prior to their construction) and removing outflow structures from the 1D domain. The impact of removing the detention basins on the extent of inundation and peak flood depths is shown on Figure 7-7 to Figure 7-10.

The detention basins provide a significant reduction in peak flood levels and inundation extent during the 1% AEP event along the main Coffs Creek alignment. Numerous overland flow paths known to become active during large events have been completely alleviated as a result of basin construction. Such locations include:

- North of Coramba Road in the location of Roselands Dr and William Sharp Drive;
- Loaders Lane / Goodenough Terrace along the Coffs Creek Main Arm; and
- Adelines Way and Moreton Bay Avenue.

The most significant reductions in peak flood level are noted just upstream of the confluence of the Coffs Creek Main Arm and North West Arm. Peak flood levels around the Loaders Lane area typically reduce by between 0.4m and 0.6m. Downstream of the contribution from the Donn-Patterson Dr catchment, the reduction in flood levels reduces to less than 0.2m. This level of reduction holds until downstream of Scarba St, after which they increase to between 0.2m and 0.3m. This level of reduction then holds until downstream of Collingwood St, where the flows no longer influence the peak flood level but are instead driven by the sea level boundary.

The impact of constructing the proposed Upper Shephards Lane detention basin on peak design flood levels is presented in Figure 7-11. A reduction in flood levels is noted along the North West Arm of Coffs Creek. Upstream of Polwarth Drive, levels reduce by up to 1.0m. Downstream of Polwarth Dr, levels reduce by around 0.2m to 0.4m.

Longitudinal profiles showing the impact of each detention basin scenario along the Coffs Creek main arm are shown in Figure 7-12 and Figure 7-13. Results along the Coffs Creek North West Arm are shown in Figure 7-14. Peak modelled flood levels across the catchment are presented in Table 7-7 at the end of this Section.

The basins also work to reduce peak flow rates and extend response time. Table 7-3 summarises the peak flow at the downstream location of each basin, with and without basins constructed. For each location, the timing of peak flow will be delayed by approximately one hour from the onset of the storm as a result of basin construction.

Table 7-3 Impact of Detention Basins - Summary of Peak Flow Rates

Location	Peak Flow Rate (m^3/s)		
	Without Basin	Basin Constructed	Percent Reduction
Downstream Bennetts Rd Basin	55.6	30.1	46%
Downstream Bakers Rd Basin	38.5	20.1	48%
Downstream Spagnolos Rd Basin	33.9	16.2	52%

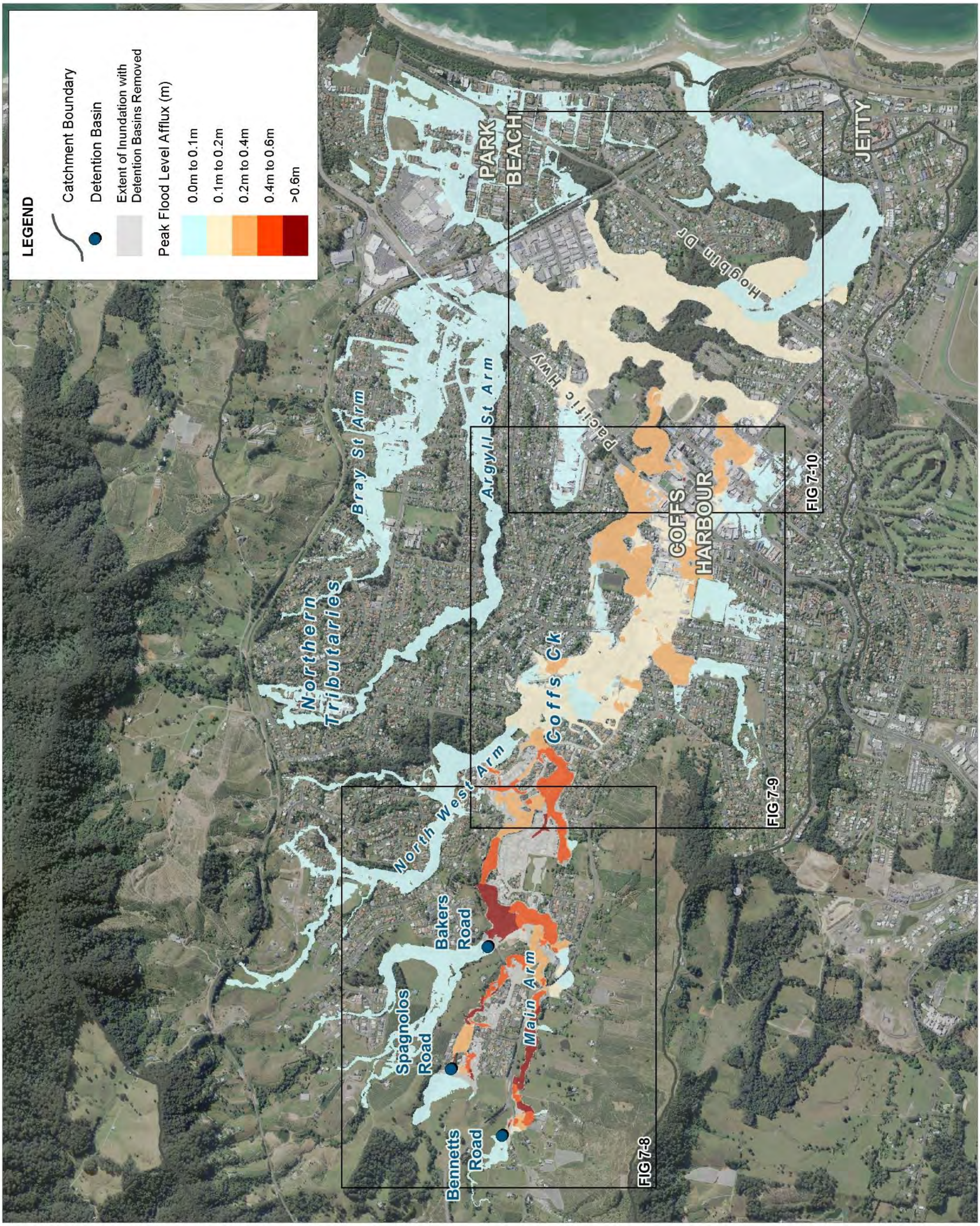
Flow hydrographs extracted at various locations within the catchment are shown in Figure 7-15 for the alternate detention basin model simulations. For the scenario where the detention basins are removed, the increase in peak flow at Loaders Lane is significant. Peak flow at this location reduces by 45% as a result of basin construction. The influence of other contributing catchment inflows lessens the impact further downstream in the catchment.

By the time the flood wave reaches the Pacific Highway, the reduction in peak flow rate as a result of basin construction is 16%. Including the proposed Upper Shephards Lane basin will reduce peak flow rates at Polwarth Drive by 26% and 5%, respectively.

7.3.2 Levees

The levee at Loaders Lane is no longer overtopped during the 1% AEP design flood event due to the effect of the detention basins. Levee breaching would be initiated near the driveway of 28 Loaders Lane, where the levee crest is elevated to around 14.1m AHD. Under existing design flood conditions, the peak flood level at this location is currently just over 14.0m AHD. The levee is currently just providing protection to the 1% AEP level; however, no additional allowance is made for freeboard. The levee currently overtops during the 0.5% AEP event, with a peak flood level of 14.3m AHD modelled at the levee.

Properties located behind the Collice Place / Langker Place levee become inundated by flood water during the 1% AEP event with a typical depth of flooding of up to 0.5m being modelled. Flood waters currently spill out of the drainage channel north of the levee, through the backyard of 8 Gillies Close and 7 Langker Place. A 900mm diameter pipe discharges flows from under the rail embankment into the Bray Street Arm of Coffs Creek. This pipe alignment has been included in the modelling, but only has capacity for approximately 30% of the 1% AEP design flow rate discharging under the rain embankment at this location. Just north of the levee, a peak flood level of 11.2m AHD is modelled for the 1% AEP event. Design drawings indicate that the levee is elevated to 10.4m AHD at this location.



Title:

Impact of Removing Bennetts Rd, Bakers Rd and Spagnolos Rd Detention Basins on the 1% AEP Event

Figure: 7-7

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Approx. Scale

0 0.5 1km

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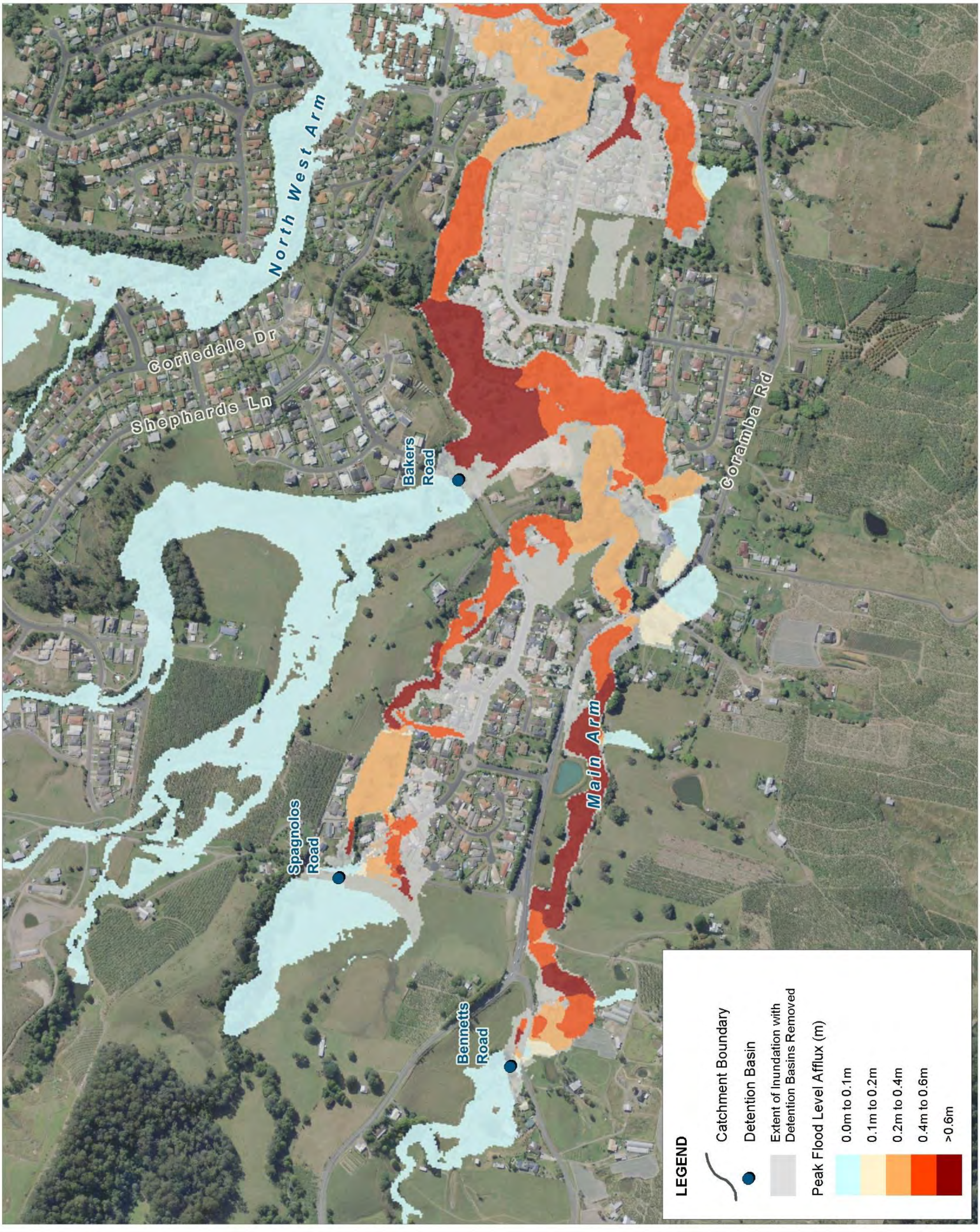

Title:

Impact of Removing Detention Basins on the 1% AEP Event Upper Coffs Creek

Figure: 7-8

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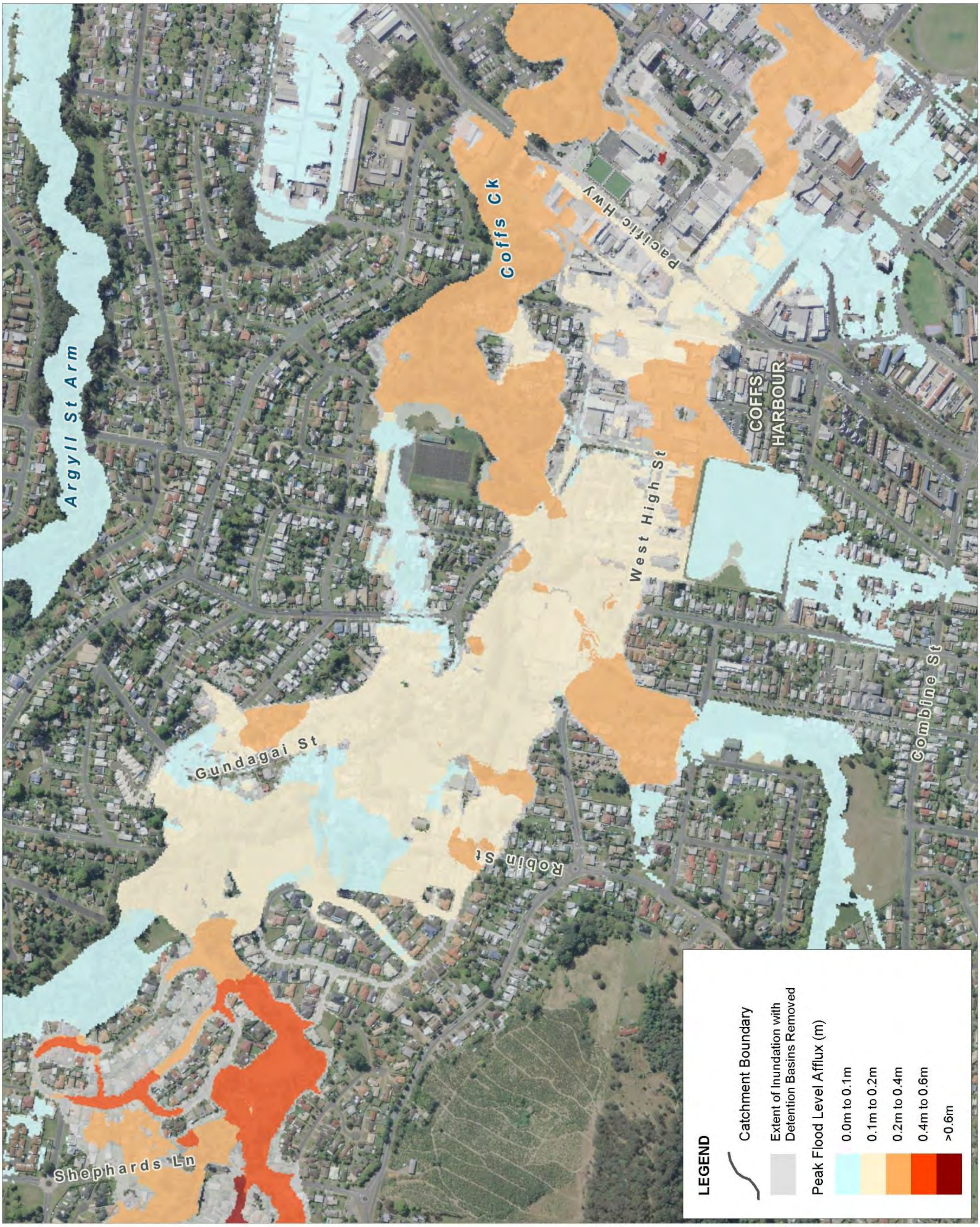

Title:
Impact of Removing Detention Basins on the 1% AEP Event Coffs Creek City Centre

Figure: 7-9

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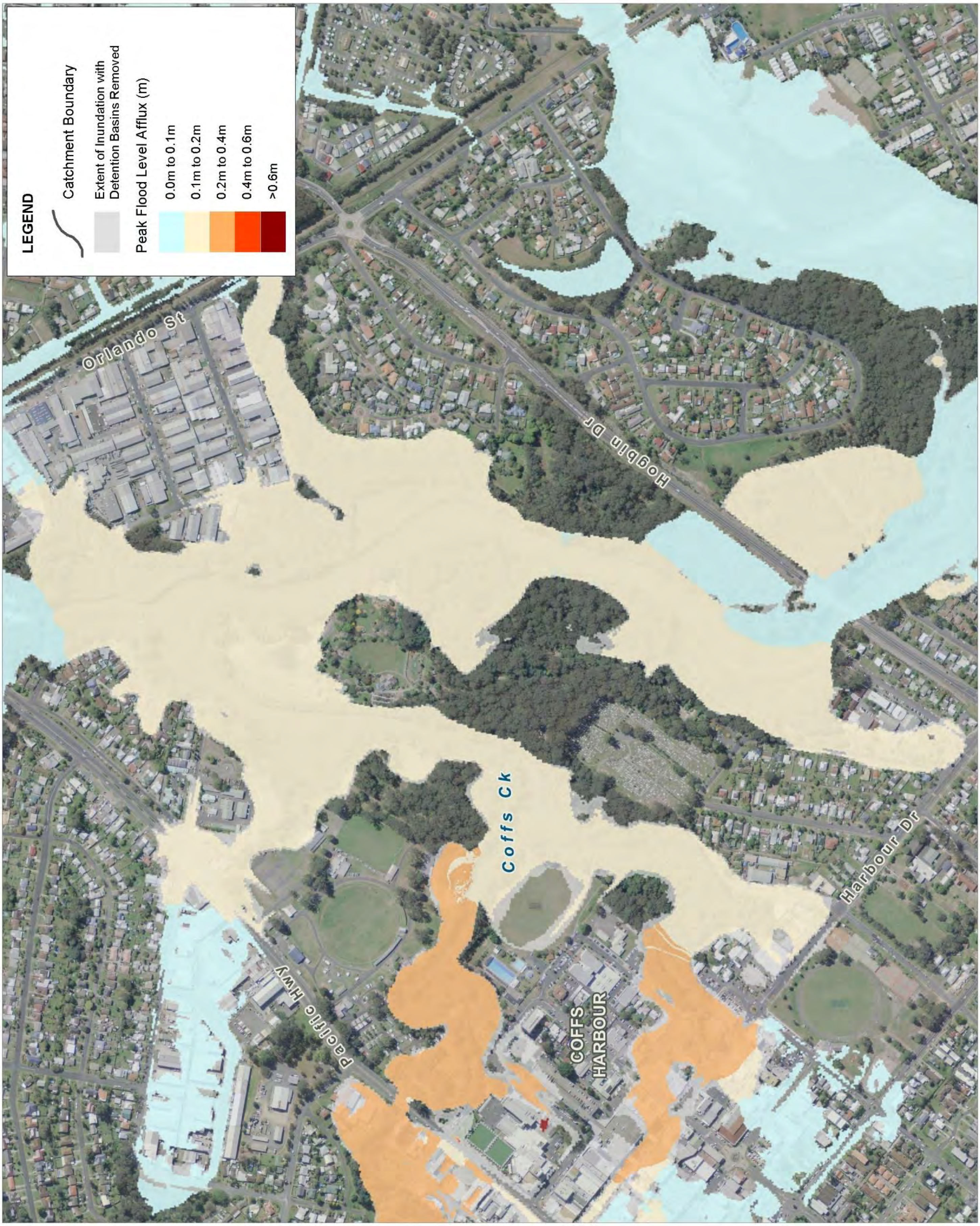

Title:

Impact of Removing Detention Basins on the 1% AEP Event Lower Coffs Creek

Figure: 7-10

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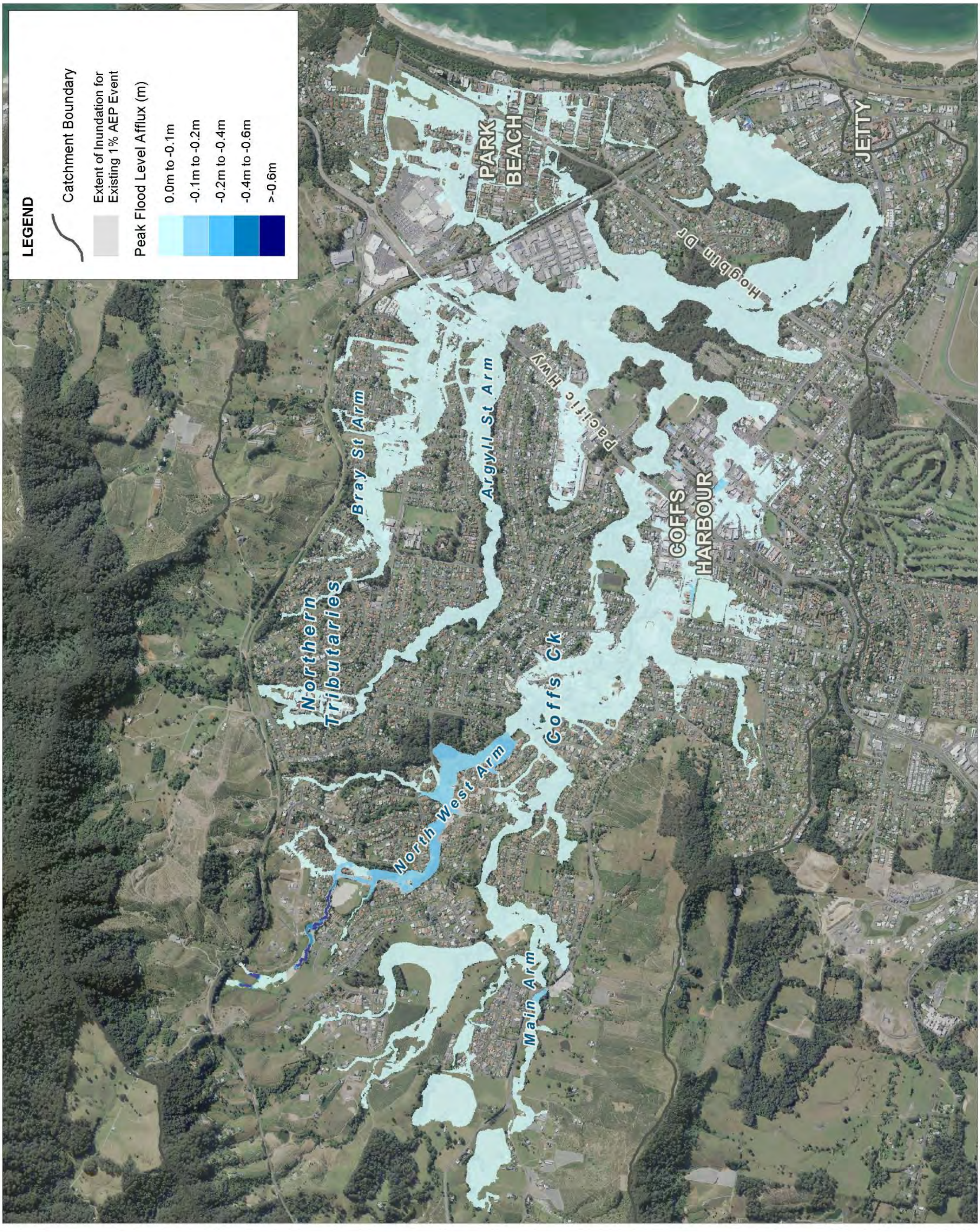

Title:
Impact of Constructing Upper Shephards Ln Detention Basin on the 1% AEP Event

Figure: 7-11

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Approx. Scale
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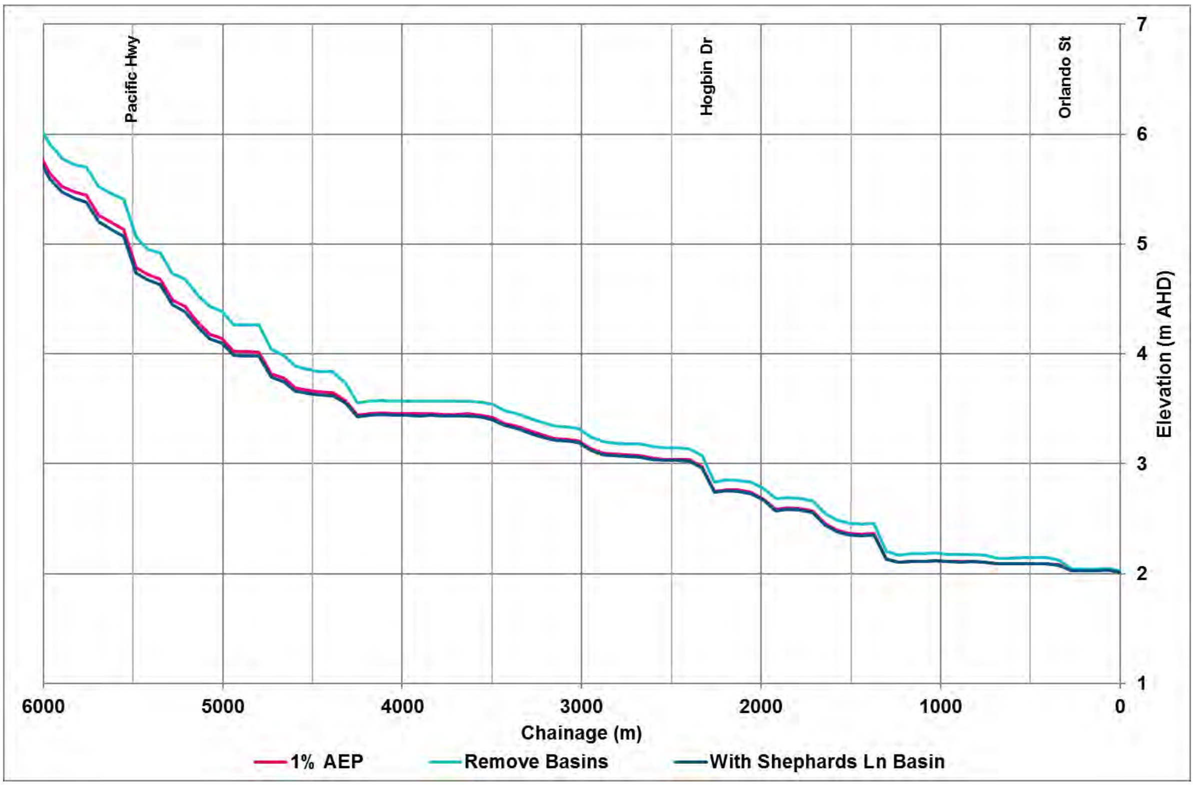


Figure 7-12 Long Section along the Coffs Creek Main Arm (Downstream) for Detention Basin Scenarios

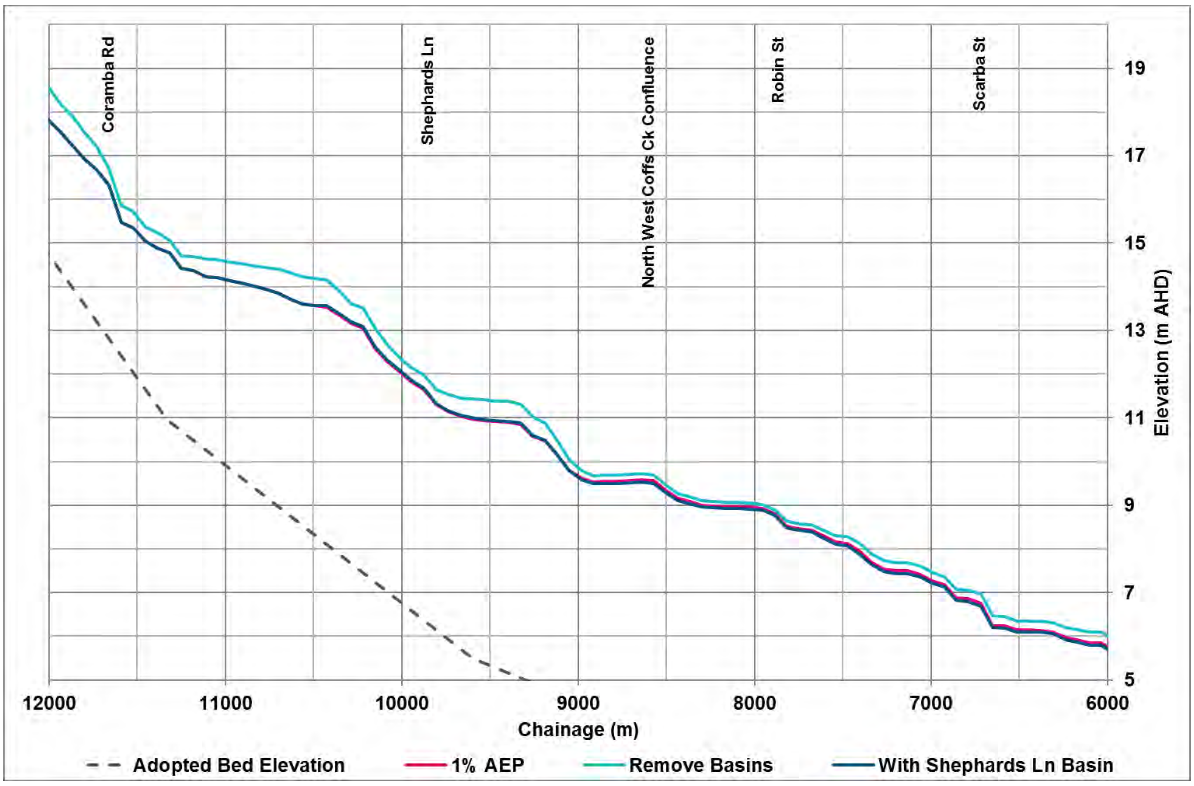


Figure 7-13 Long Section along the Coffs Creek Main Arm (Upstream) for Detention Basin Scenarios

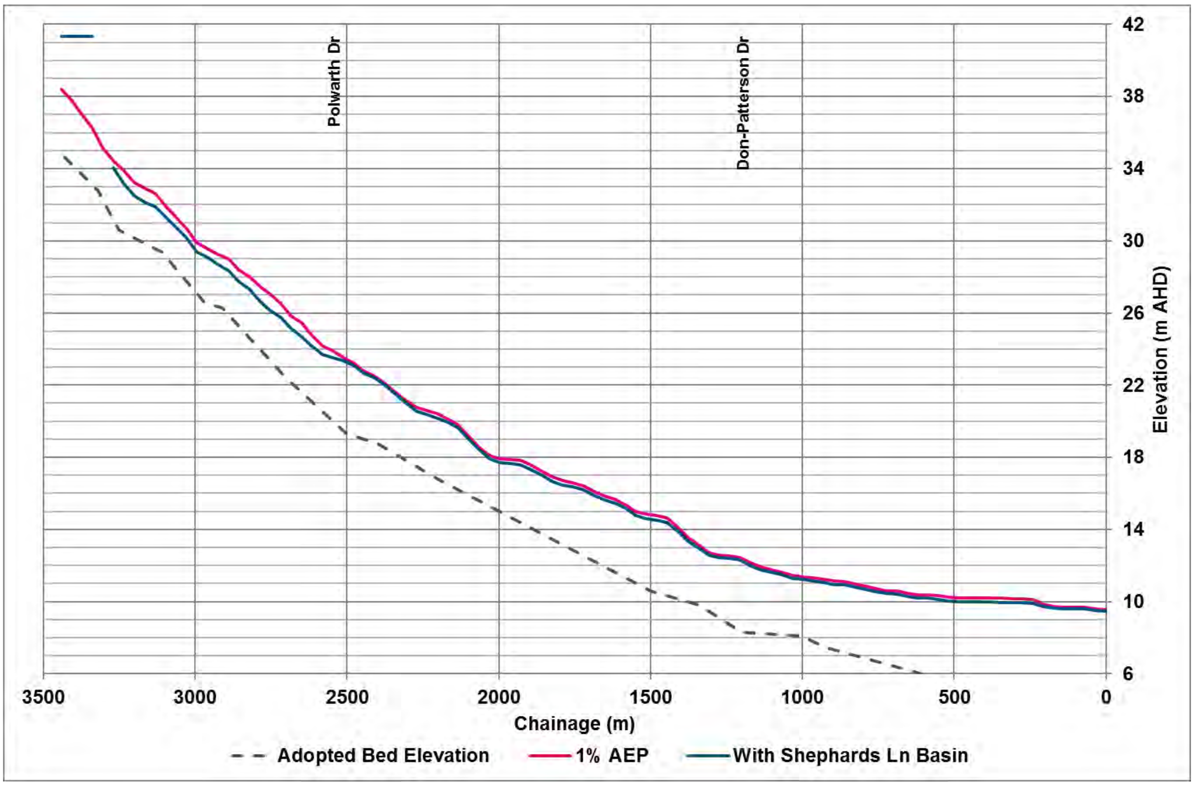


Figure 7-14 Long Section along the Coffs Creek North West Arm for Detention Basin Scenarios

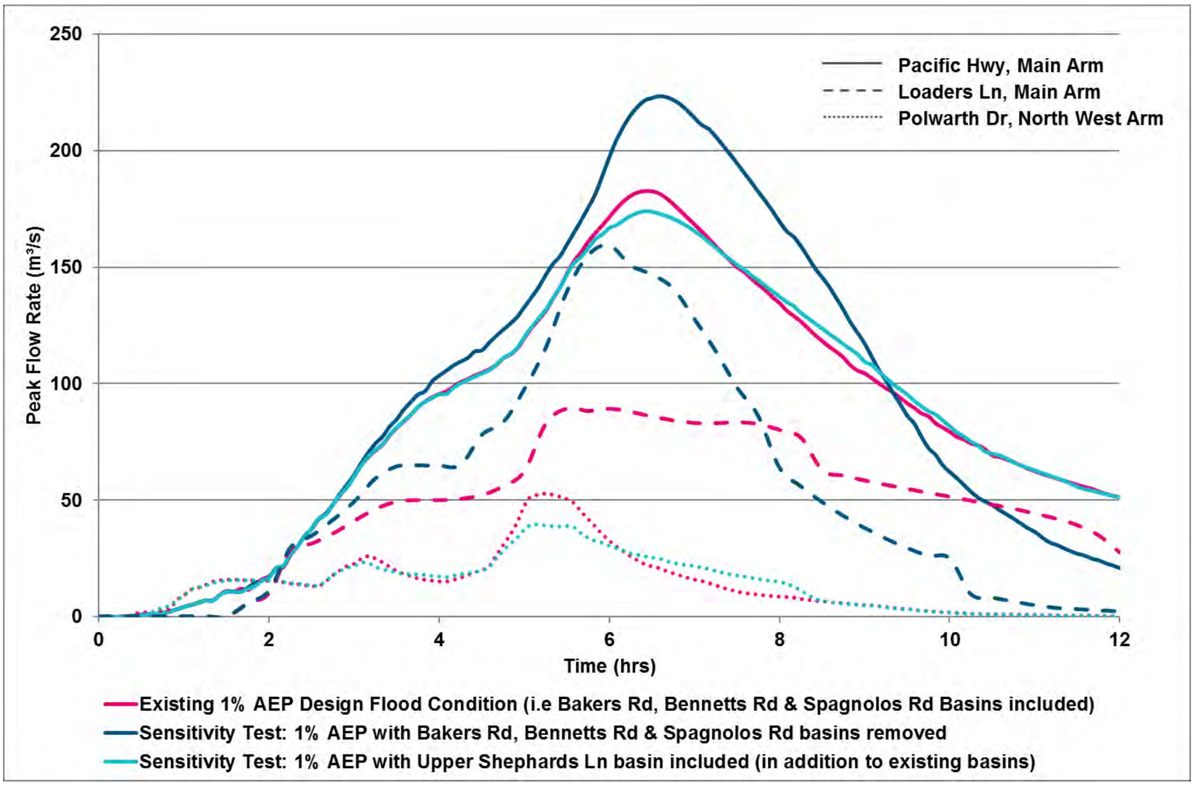


Figure 7-15 1% AEP Peak Flows for Alternate Detention Basin Simulations

7.4 Hydraulic Categorisation

There are no prescriptive methods for determining what parts of the floodplain constitute flood ways, 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 7-4.

Table 7-4 Hydraulic Categories

Floodway	Velocity * Depth > $0.3m^2/s$ 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 < $0.3m^2/s$ and Depth > 0.5m 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.

Hydraulic category mapping is included in the Mapping Compendium, and is presented for the 1% AEP design event.

7.5 Provisional 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 7-16.

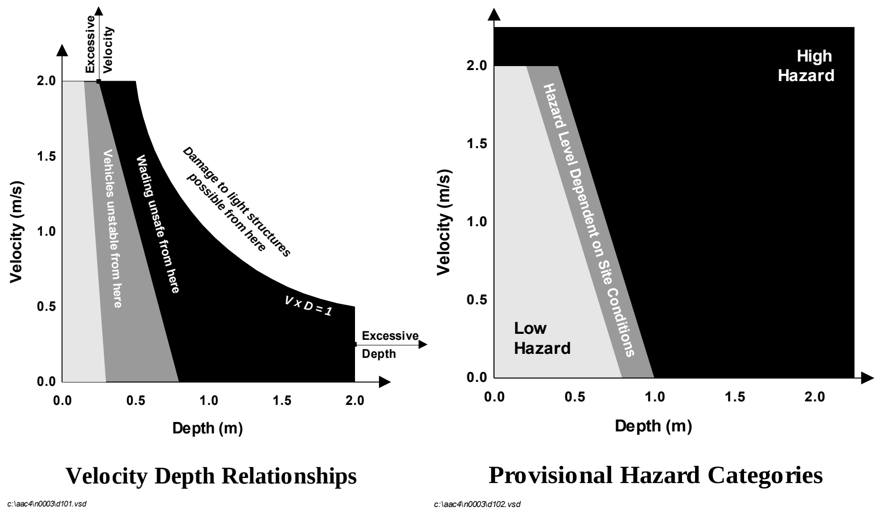


Figure 7-16 Provisional Flood Hazard Categorisation

Provisional hazard category mapping is included in the Mapping Compendium, and is presented for the 1% AEP design event.

7.6 True Hazard

The true hazard categorisation is typically based on the hydraulic hazard categorisation discussed in Section . The Coffs Creek Floodplain Risk Management Study (Bewsher Consulting, 2005) specifically defined four categories of true hazard or flood risk, with guidance to the appropriate level of planning control applicable to each category.

The true hazard categories, as defined by Bewsher (2005), are as follows:

- **High Flood Risk –** Area within the 1% AEP event flood extent that is classified as high hydraulic hazard (see Section 7.4) and/or where there are significant evacuation difficulties. The high flood risk area is where high flood damages, potential risk to life, or evacuation problems are anticipated. Most development should be restricted with stringent development controls within this area.
- **High Flood Risk Flow Corridor –** A high flow corridor exists within the high flood risk area. It is defined as the area between the main creek banks and/or where the velocity * depth product exceeds 1.0.
- **Medium Flood Risk –** Area within the 1% AEP event flood extent that is not classified as high hydraulic hazard and where there are no significant evacuation difficulties. The potential for damages can be minimised by the application of appropriate development controls.
- **Low Flood Risk –** Area within the PMF flood extent that is not classified as high or medium flood risk. The risk of damage is low and most land uses would be permitted within this area.

True hazard category mapping is included in the Mapping Compendium.

7.7 Sensitivity Tests

7.7.1 Climate Change

The potential impacts of future climate change were considered for the 1% AEP design event. There are potential impacts associated with both an increase in rainfall intensities and an increase in sea level rise. Table 7-5 summarises the climate change scenarios modelled.

The impact of potential sea level rise extends as far upstream along the Coffs Creek Main Arm as the Pacific Highway bridge.

Longitudinal profiles showing the climate change assessment along the Coffs Creek main arm is shown on Figure 7-17. Peak modelled flood levels at the reporting locations are presented in Table 7-7 at the end of this Section.

Table 7-5 Climate Change Scenarios

Modelled Simulation	Boundary Conditions
Adopted 1% AEP Design Event	1% AEP rainfall 5% AEP ocean event
1% AEP + 2050 SLR	1% AEP rainfall 5% AEP ocean event + 0.4m
1% AEP + 2100 SLR	1% AEP rainfall 5% AEP ocean event + 0.9m
1% AEP + 10% rainfall	0.5% AEP rainfall 1% AEP ocean event (i.e. Adopted 0.5% AEP Design Event)
1% AEP + 10% rainfall + 2050 SLR	0.5% AEP rainfall 1% AEP ocean event + 0.4m
1% AEP + 10% rainfall + 2100 SLR	0.5% AEP rainfall 1% AEP ocean event + 0.9m
1% AEP + 30% rainfall	0.2% AEP rainfall 1% AEP ocean event (i.e. Adopted 0.2% AEP Design Event)
1% AEP + 30% rainfall + 2050 SLR	0.2% AEP rainfall 1% AEP ocean event + 0.4m
1% AEP + 30% rainfall + 2100 SLR	0.2% AEP rainfall 1% AEP ocean event + 0.9m

7.7.2 Structure and Pipe Blockage

The assessment of the impact of structure blockages on peak flood levels is a key consideration for floodplain management. The design flood conditions assumed that all hydraulic structures and stormwater pipes were free from blockage. For the blockage sensitivity test the 1% AEP design event was simulated adopting a 25% blockage for structures with openings larger than 6m and a 100% blockage for all other structures and stormwater pipes. Figure 7-18 shows the increase in peak flood levels and inundation extents resulting from the blockage of structures.

A structure blockage of 100% is unlikely, but provides an upper limit for potential blockage impacts. Park Beach is particularly sensitive to blockage of structures, both in terms of flood level and flood extent. Under normal conditions, the entire Park Beach catchment is drained through the culverts under the railway embankment. When the stormwater pipes are blocked, flood water pools within the area, significantly increasing the flood risk to the local area. Peak flood levels increase by just over 1.0m behind the rail embankment in the Park Beach Plaza/Orlando Street channel when structures are assumed to be blocked.

Elsewhere in the catchment, impacts are typically localised to just upstream of blocked culvert structures and to areas where significant trunk drainage has been included in the model. It should be noted that expected increases in peak flood levels as a result of structure and pipe blockage is typically within the order of 0.2m. The Perry Drive crossing on the Bray Street Arm of the Northern Tributaries is particularly sensitive to blockage. In the scenario where this culvert is assumed to be completely blocked, an additional flow path is generated along Apollo Drive, with levels upstream of the culvert increasing in the order of 0.5m. Another location sensitive to blockage is the stormwater drainage and Brodie Drive culvert discharging into Coffs Creek, just upstream of Orlando Street. Results of the sensitivity testing indicate that inundation of three residential properties (modelled depths in the order of 0.1-0.2m) and increased inundation of the Brodie Drive road reserve can be expected.

As there are significant impacts in the form of increased peak flood levels and additional overland flow paths with potential to inundate further properties, it is important for council to maintain structure openings clear from debris, particularly the detention basin outlets.

Longitudinal profiles showing the result of the structural blockage assessment along the various tributaries of Coffs Creek are shown in Figure 7-19 to Figure 7-26. Peak modelled flood levels are presented in Table 7-7 at the end of this Section.

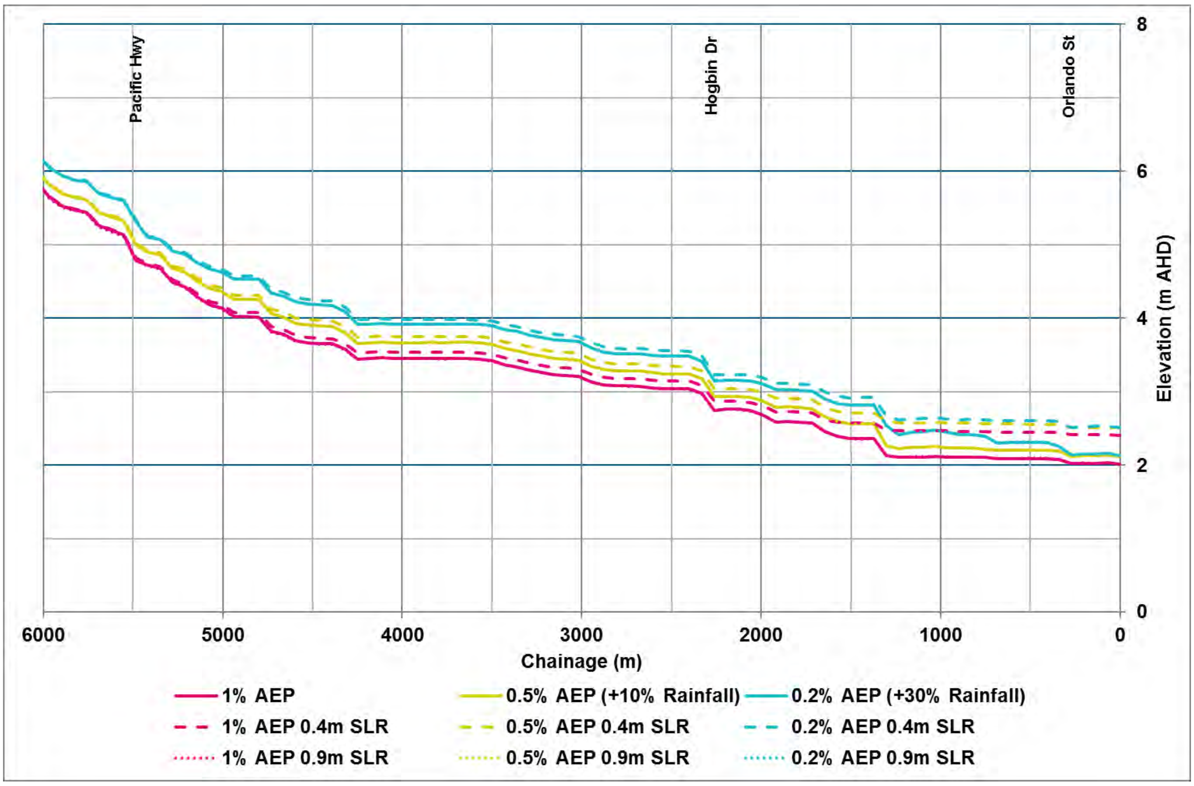
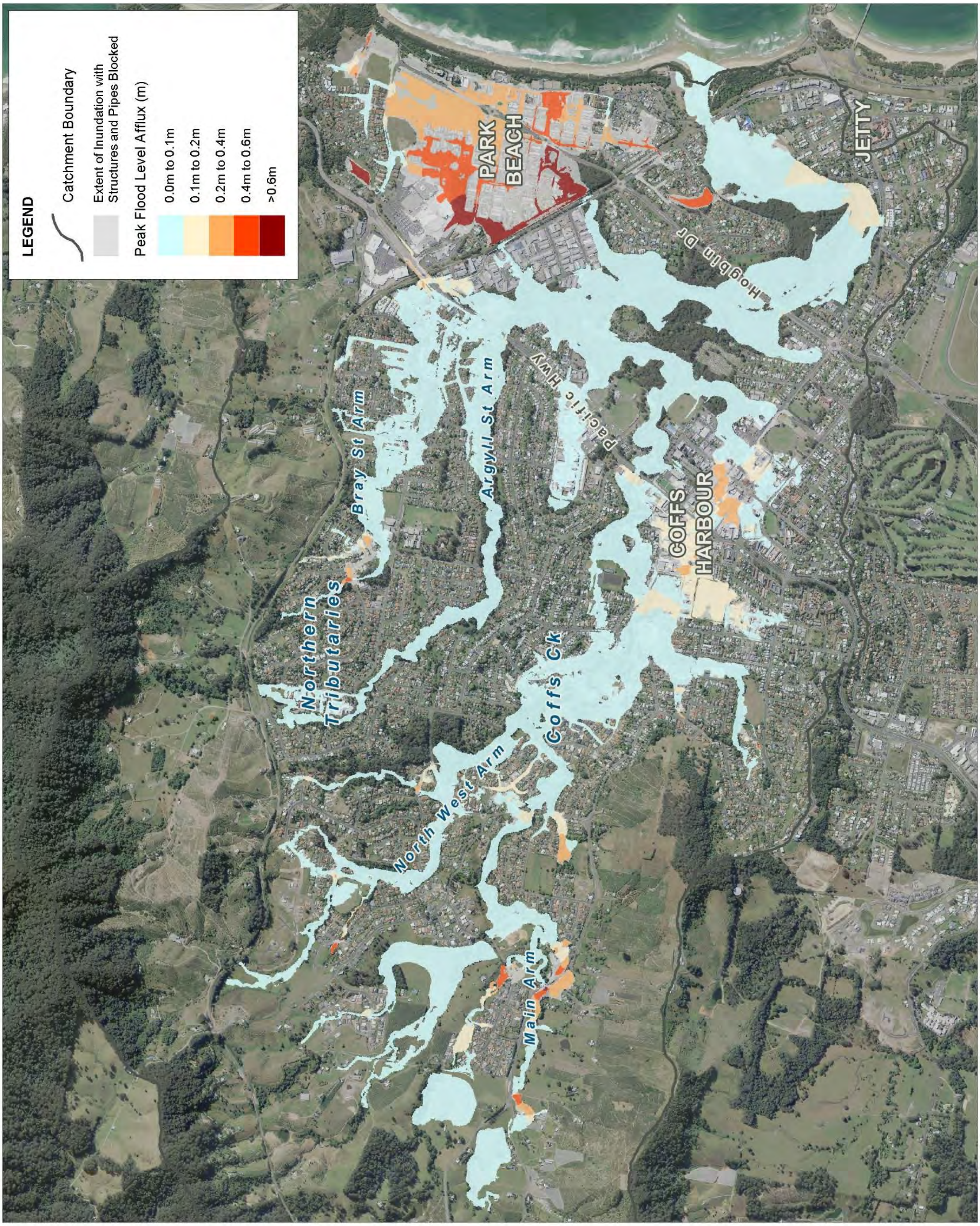


Figure 7-17 Long Section along the Coffs Creek Main Arm for Climate Change Scenarios

7.7.3 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 catchment event. The impact of increasing the adopted manning's 'n' values typically raises peak flood levels by up to 0.3m. Reducing the adopted manning's 'n' values typically lowers peak flood levels by up to 0.2m.

Longitudinal profiles showing the result of this assessment along the various tributaries of Coffs Creek are shown in Figure 7-19 to Figure 7-26. Peak modelled flood levels are presented in Table 7-7 at the end of this Section.



Title:

Impact of Structure and Pipe Blockage on the 1% AEP Event

Figure: 7-18

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Approx. Scale

0 0.5 1km

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7.7.4 Alternate Design Rainfall Gradients

Whilst the rainfall gradients adopted in this study are deemed most appropriate for design rainfall estimates based on available information, inherent uncertainty remains. It was therefore decided to develop upper and lower bound gradient ratios to assess the potential range of impact on peak flood levels. Upper and lower bound ratios presented in the Coffs Creek Flood Study (WMA, 2001) were used to provide guidance as to appropriate values. The adopted upper and lower bound ratios are presented in Table 7-6.

Table 7-6 Upper and Lower Bound Design Rainfall Gradients

Gradient Ratio (Relative to Centre of Catchment)	Rainfall Zone			
	A	B	C	D
Lower Estimate	1.0	1.0	1.0	1.1
Best Estimate	1.0	1.0	1.2	1.6
Upper Estimate	1.0	1.0	1.65	2.2

Longitudinal profiles showing the result of applying upper and lower bound design rainfall estimates along the various tributaries of Coffs Creek are shown in Figure 7-19 to Figure 7-26. Peak modelled flood levels are presented in Table 7-7 at the end of this Section.

7.8 Comparison with Previous Studies

Design flood conditions are considerably different to those presented in previous studies. This is largely due to construction of detention basins within the catchment, but also due to:

- Changing from a 1D to almost entirely 2D model representation; and
- Revising the design rainfall scaling factors and lowering the sea level boundary (in accordance with OEH ((2015)).

Figure 7-27 presents a long section of the 1% AEP peak flood level along the main alignment of Coffs Creek determined from this current study against that defined in the previous study (Webb, McKeown and Associates, 2001).

The main difference between the 1D and 2D representation is that the 1D model provides generally consistent hydraulic gradients, which is a function of the simpler model representation. The only hydraulic controls that are evident in the 1D flood gradient are those which have been explicitly modelled (i.e. bridge structures), whereas the 2D representation captures the less obvious controls such as local constrictions in the floodplain conveyance. These would only have been captured by the 1D model if a surveyed cross section happened to be taken at the right location and incorporated into the model accordingly.

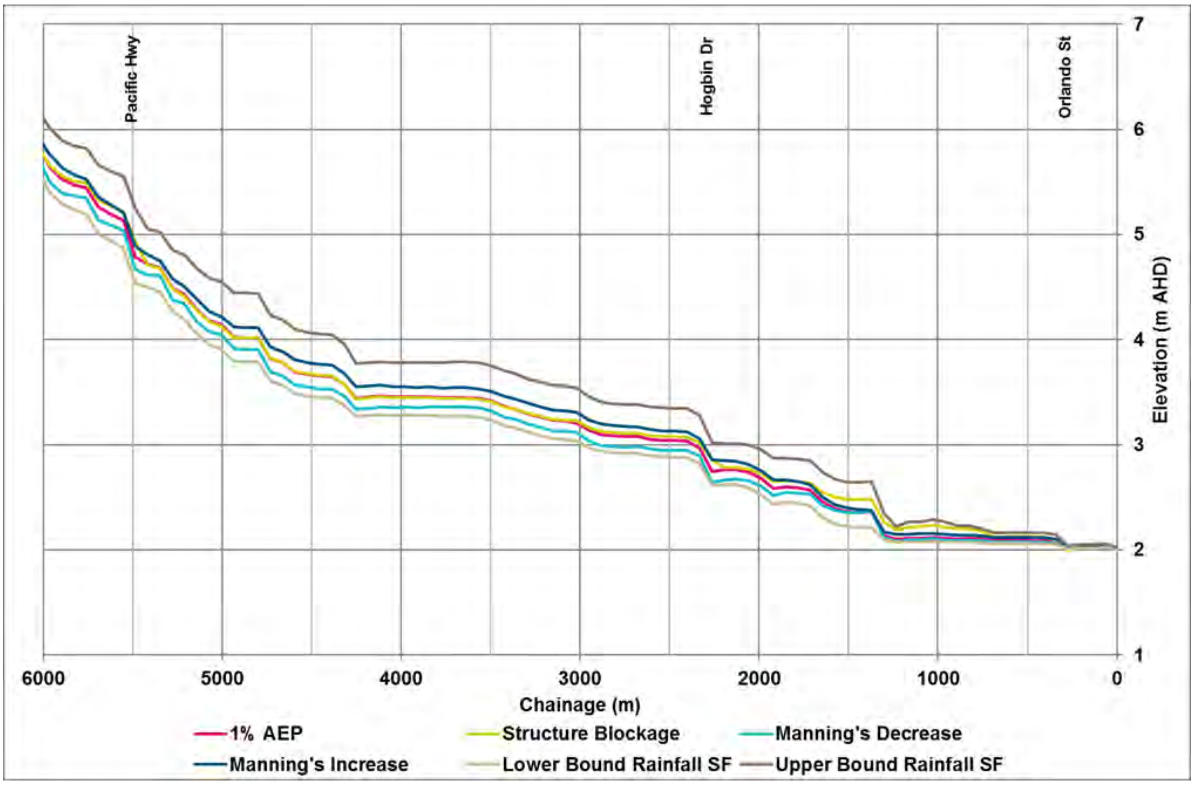

Figure 7-19 Long Section along the Coffs Creek Main Arm (D/S) for Sensitivity Assessment

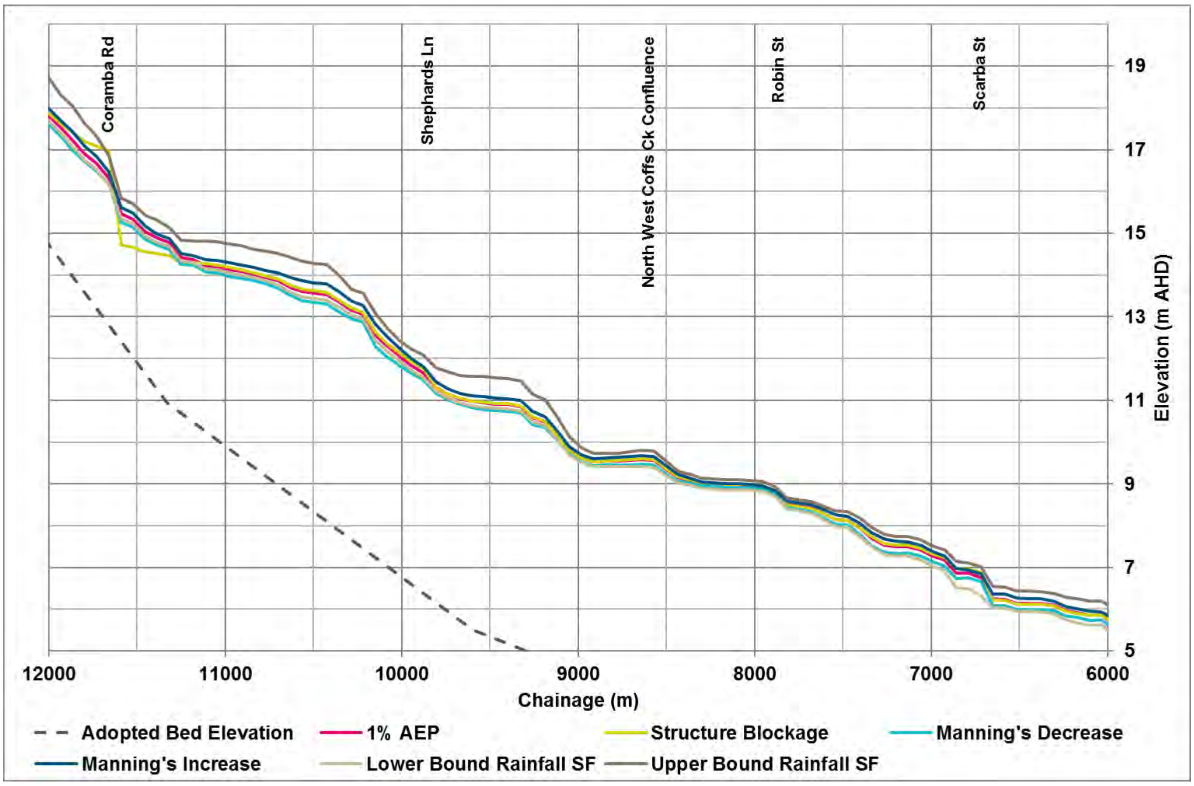

Figure 7-20 Long Section along the Coffs Creek Main Arm (U/S) for Sensitivity Assessment

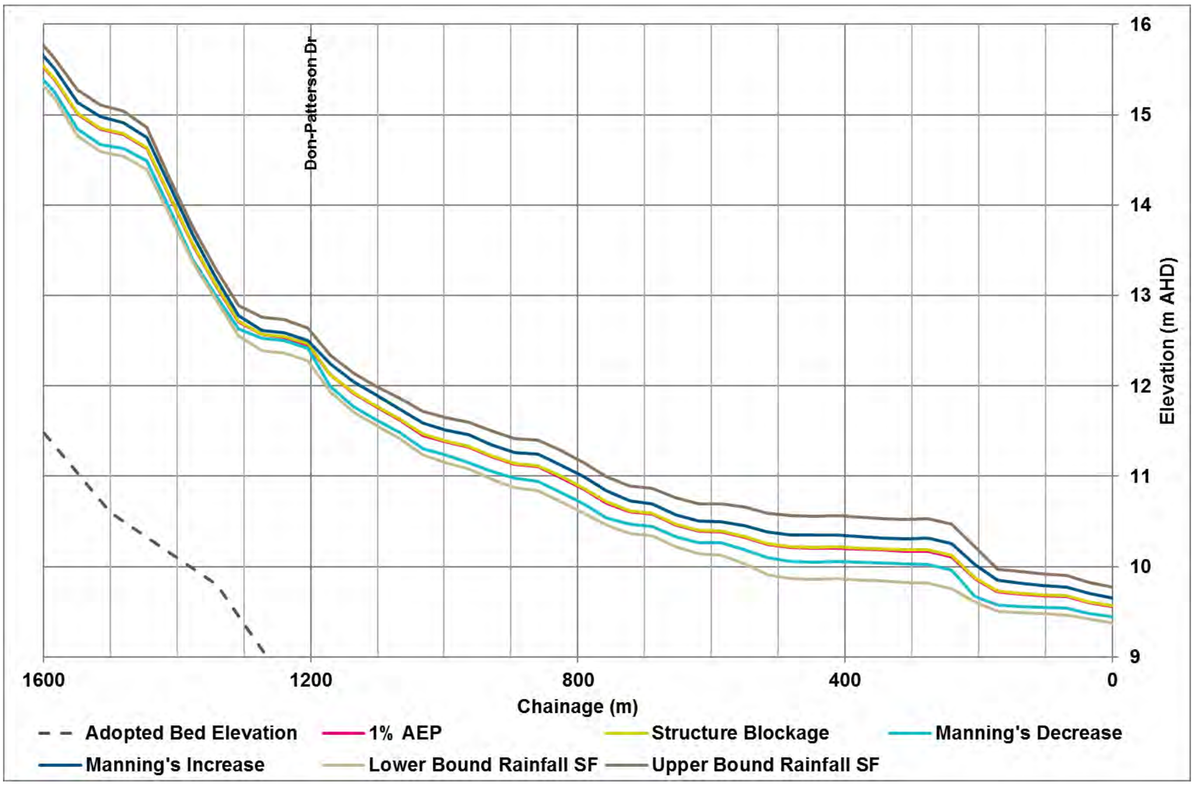

Figure 7-21 Long Section along the Coffs Creek North West Arm (D/S) for Sensitivity Assessment

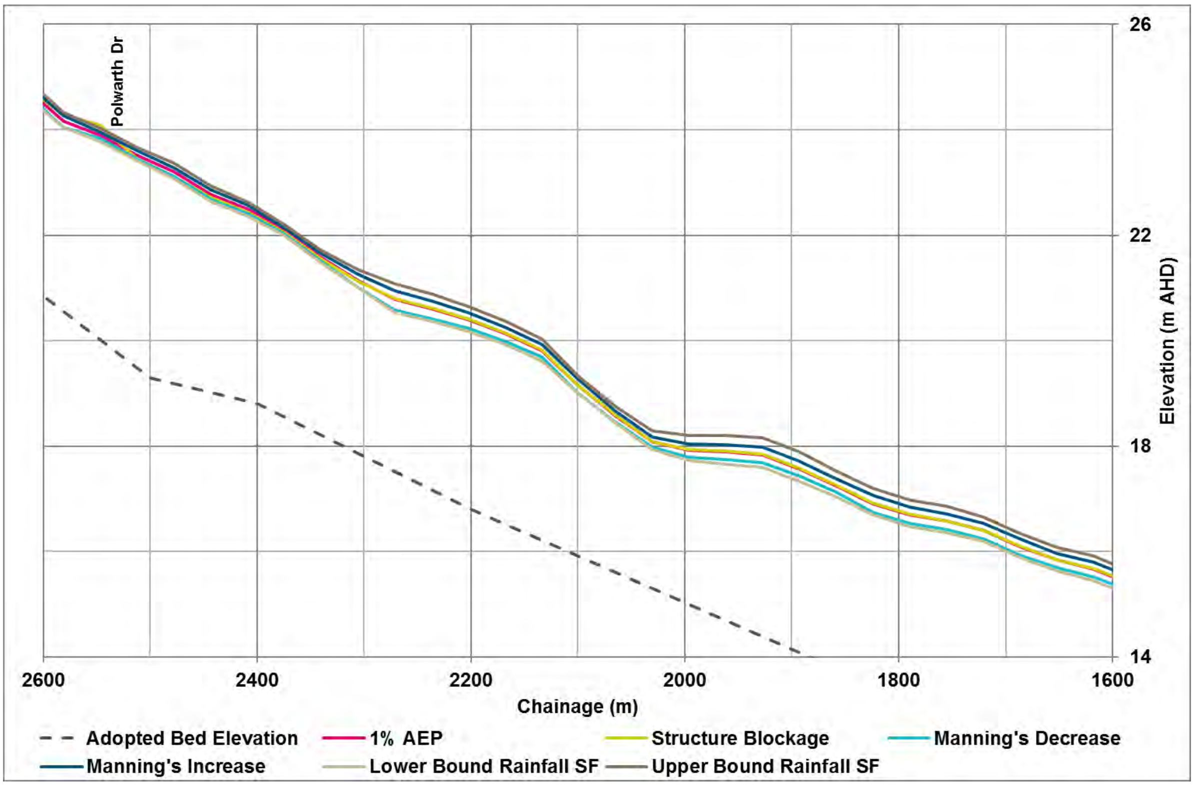

Figure 7-22 Long Section along the Coffs Creek North West Arm (U/S) for Sensitivity Assessment

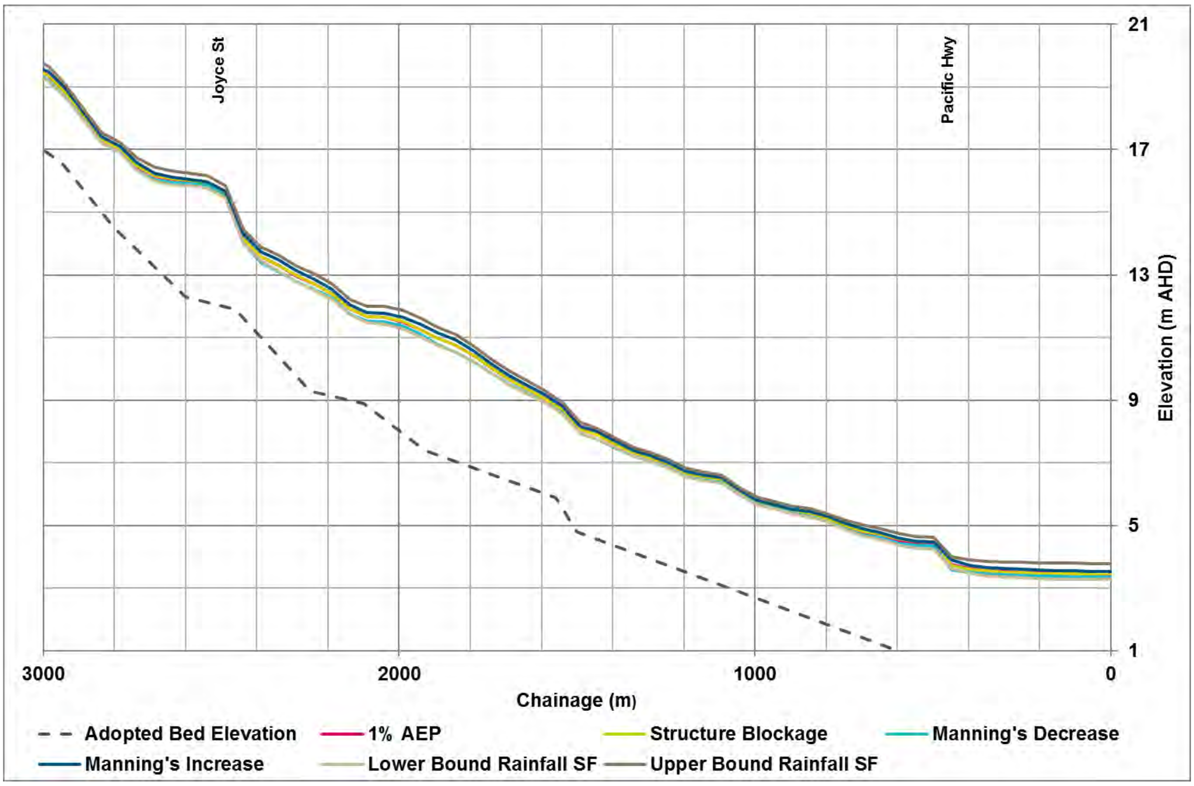

Figure 7-23 Long Section along the Argyll Street Arm of the Northern Tributaries (D/S) for Sensitivity

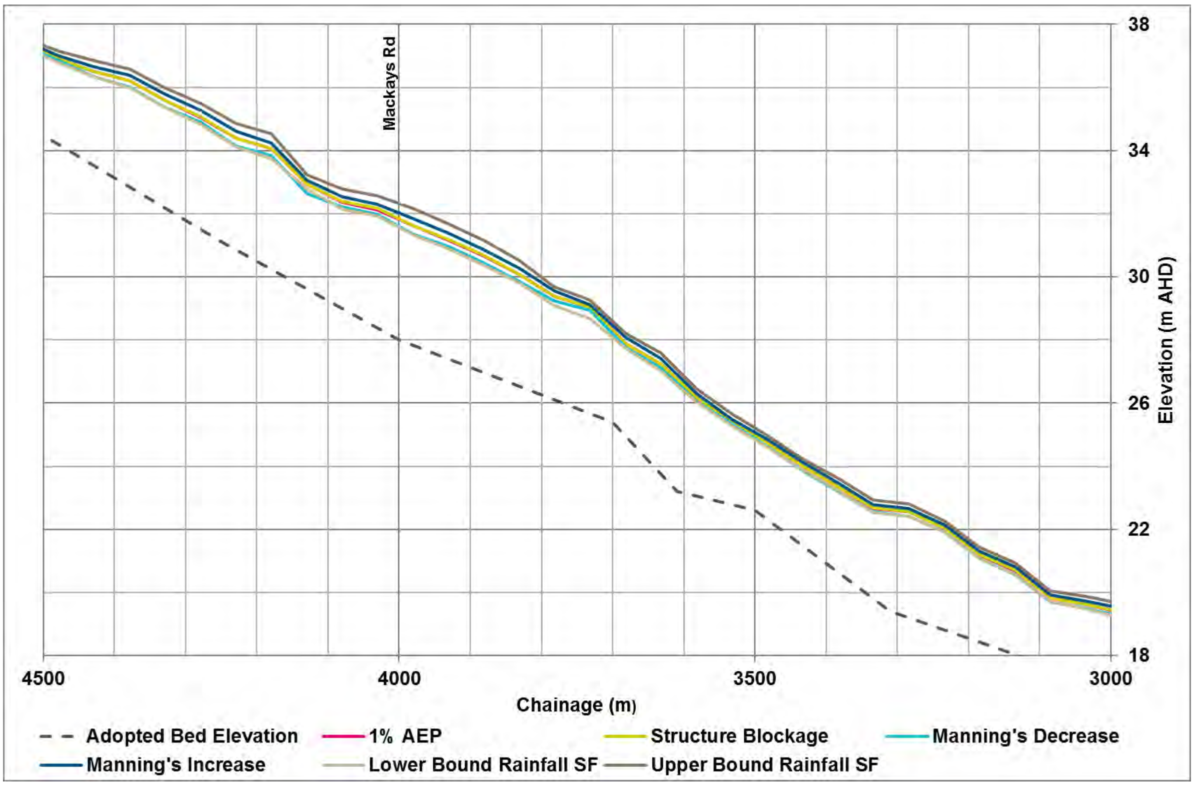

Figure 7-24 Long Section along the Argyll Street Arm of the Northern Tributaries (U/S) for Sensitivity

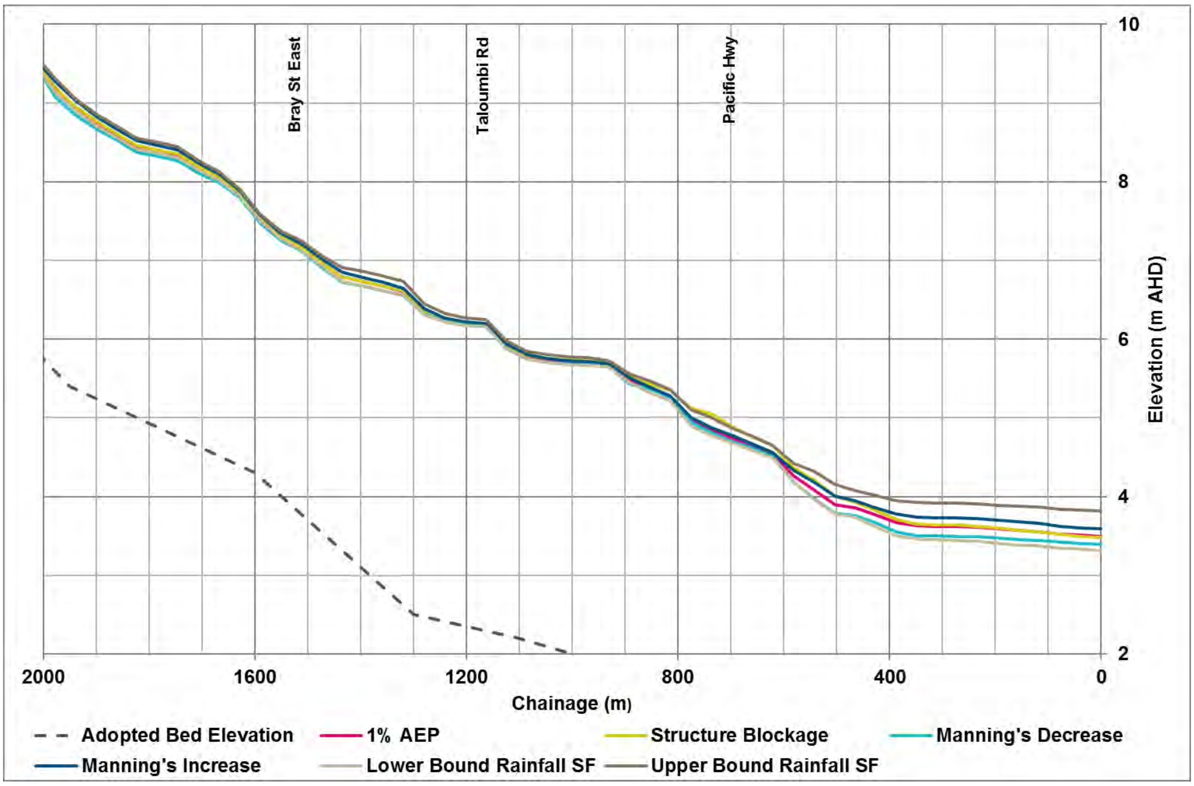

Figure 7-25 Long Section along the Bray Street Arm of the Northern Tributaries (D/S) for Sensitivity

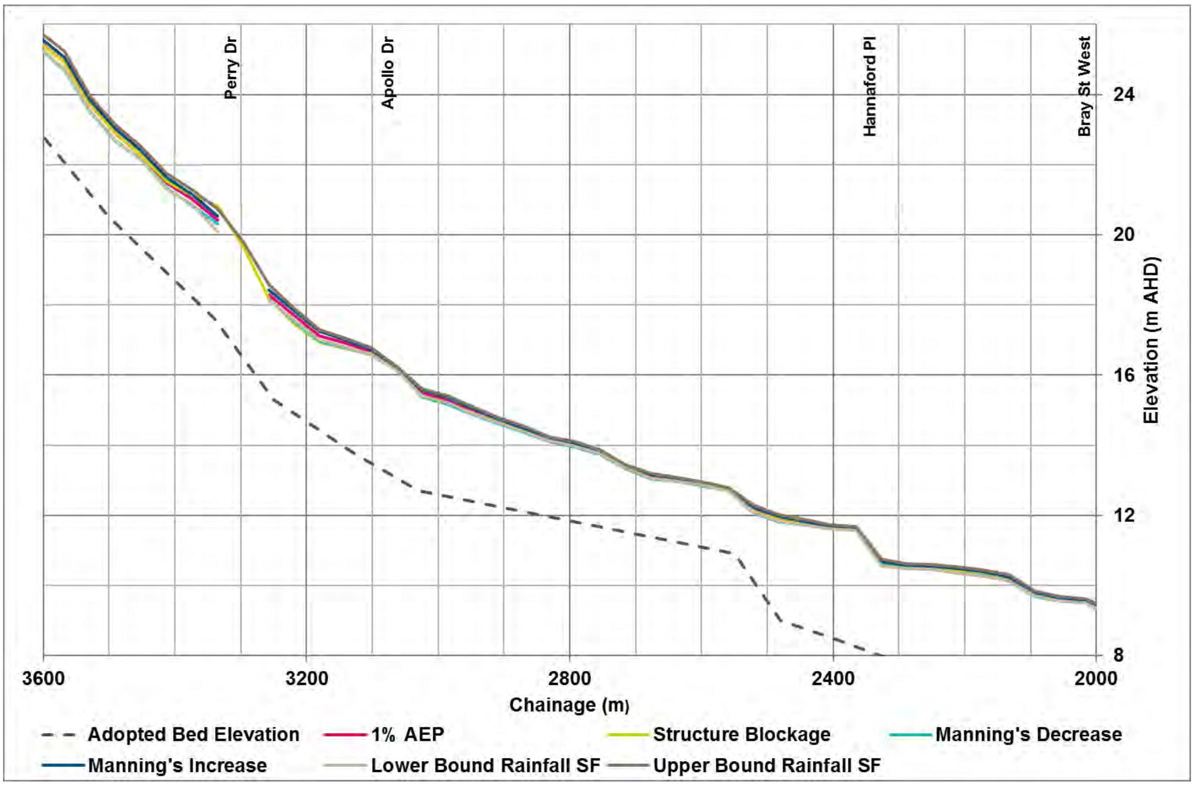

Figure 7-26 Long Section along the Bray Street Arm of the Northern Tributaries (U/S) for Sensitivity

Table 7-7 Summary of Model Sensitivity Results

ID	Location	Modelled Peak Flood Level (m AHD)															
		1% AEP	Remove Detention Basins	With Upper Shephard Ln Basin	1% AEP w 0.4m SLR	1% AEP w 0.9m SLR	1% AEP +10% RF (0.5% AEP)	0.5% AEP w 0.4m SLR	0.5% AEP w 0.9m SLR	1% AEP +30% RF (0.2% AEP)	0.2% AEP w 0.4m SLR	0.2% AEP w 0.9m SLR	Structure Blockage	Manning Increase 25%	Manning Decrease 25%	Lower Bound RF Scale Factor	Upper Bound RF Scale Factor
A	Bennetts Rd detention basin U/S	28.6	23.7	28.6	28.6	28.6	28.9	28.9	28.9	29.1	29.1	29.1	28.6	28.6	28.6	27.6	29.0
	Bennetts Rd detention basin D/S	22.3	22.5	22.4	22.3	22.4	22.6	22.6	22.6	22.7	22.7	22.7	22.3	22.4	22.4	22.4	22.8
B	Spagnolos Rd detention basin U/S	23.6	20.1	23.6	23.6	23.6	23.8	23.8	23.8	24.0	24.0	24.0	23.6	23.6	23.6	22.6	24.0
	Spagnolos Rd detention basin D/S	18.4	18.9	18.4	18.4	18.4	18.7	18.7	18.7	18.8	18.8	18.8	18.4	18.4	18.4	18.2	18.8
C	Bakers Rd detention basin U/S	17.1	14.4	17.1	17.1	17.1	17.4	17.4	17.4	17.7	17.7	17.7	17.1	17.2	17.0	16.3	17.8
	Bakers Rd detention basin D/S	13.7	14.4	13.7	13.7	13.7	14.2	14.2	14.2	14.4	14.4	14.4	13.7	13.9	13.5	13.6	14.5
D	Loaders Ln (gauge site)	14.0	14.5	14.0	14.0	14.0	14.4	14.4	14.4	14.5	14.5	14.5	14.0	14.2	13.8	13.9	14.6
E	Loaders Ln	-	12.1	-	-	-	11.6	11.6	11.7	12.3	12.3	12.3	-	-	-	-	12.4
F	Gundagai St	6.8	6.9	6.7	6.8	6.8	6.9	6.9	6.9	7.0	7.0	7.0	6.9	6.8	6.7	6.4	7.0
G	Pacific Hwy, Coffs Ck	4.9	5.2	4.8	4.9	4.8	5.1	5.1	5.1	5.4	5.4	5.4	5.0	5.0	4.8	4.6	5.3
H	Bray Street	7.1	7.1	7.1	7.1	7.1	7.1	7.1	7.1	7.2	7.2	7.2	7.1	7.1	7.0	7.0	7.1
I	Pacific Hwy, NT's	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.4	4.4	4.4	4.4	4.5
J	Orlando St, NT's	4.5	4.5	4.5	4.5	4.5	4.6	4.6	4.6	4.7	4.7	4.7	4.6	4.6	4.5	4.5	4.6
K	Orlando St, Coffs Ck	2.0	2.1	2.0	2.4	2.0	2.1	2.5	2.1	2.2	2.5	2.2	2.1	2.1	2.0	2.0	2.1
L	York St Playing Field	4.7	4.7	4.7	4.7	4.7	4.8	4.8	4.8	4.8	4.8	4.8	5.1	4.8	4.7	4.7	4.7

BMT WBM

An example of the improved representation of local hydraulic controls in the 2D TUFLOW model developed for this study is evident at the McCann Court and Tulipwood Close localities. The complex nature of some highly sinuous reaches is also difficult to represent in a 1D model and can result in an over-estimation of floodplain conveyance. This effect is evident around the North Coast Holiday Park. The TUFLOW model generates a hydraulic gradient through this reach compared to the simple backwater control in the 1D model, which underestimates the downstream level at the Pacific Highway. Locations within simpler reaches of the channel match well between the 1D and 2D models.

The revision of design rainfall scaling factors has had a minor impact on peak flood conditions along the main alignment of Coffs Creek. Although design flow rates in the elevated upper catchment have been reduced from those previously calculated, they have increased a little further down the catchment. This has effectively ‘balanced out’ further down the catchment, with the peak flow at the Pacific Highway (223m^3/s) being comparable to that calculated in the previous study (~230m^3/s), for the model scenario that does not include the existing three detention basins in the upper catchment. Downstream of the Pacific Highway, peak flood levels are lower than those defined in the previous study due to the different ocean boundary levels adopted for design events. For the 1% AEP event, the previous study adopted a peak ocean level of 2.4m AHD. For this study, 2.1m AHD was adopted in accordance with recommendations provided by OEH (2015).

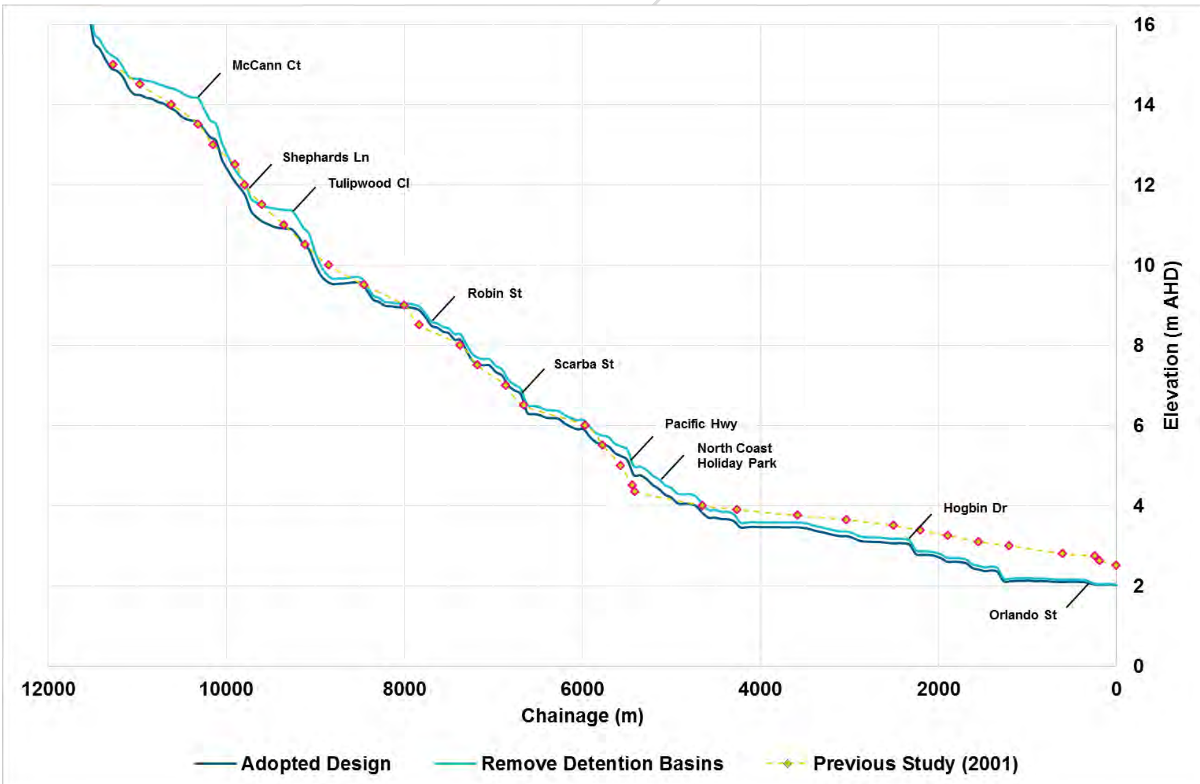

Figure 7-27 Comparison of 1% AEP Design Flood Level to Previous Study – Coffs Creek Main Arm

8 Conclusions

The primary objective of the study was to undertake a detailed flood study of the Coffs Creek catchment and to establish models as necessary for design flood level prediction

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

- Compilation and review of existing information pertinent to the study and acquisition of additional data including survey;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Calibration of the developed models using the available flood data, including the recent events of 1996, 2009 and 2015;
- Prediction of design flood conditions in the catchment and production of design flood mapping series; and
- Assessment of the possible impacts of climate change including different sea level rise scenarios.

The principal outcome of the flood study is the understanding of flood behaviour in the catchment and, in particular, design flood level information. The study provides updated flooding information from the previous Coffs Creek Flood Study (Webb, McKeown and Associates, 2001) and is to be used to inform floodplain risk management within the study area.

Design flood conditions are considerably different to those presented in previous studies. This is largely due to construction of detention basins within the catchment, but also due to:

- Changing from a 1D to almost entirely 2D model representation; and
- Revising the design rainfall scaling factors and lowering the sea level boundary in accordance with OEH guidelines (2015).

Combining the Flood Studies for Coffs Creek and Park Beach into one document will provide for consistent floodplain risk management in the region with the ability to clearly prioritise the need for mitigation measures across the catchment.

Typically, a Floodplain Risk Management Plan is regarded as a dynamic instrument requiring review and modification over time. The catalyst for change may include new flood events and experiences, legislative change, alterations in the availability of funding, or changes to the area's planning strategies. Due to the updated design flood conditions presented in this study, it is recommended that the existing Floodplain Risk Management Study for Coffs Creek (Bewsher Consulting, 2005) be reviewed.

9 References

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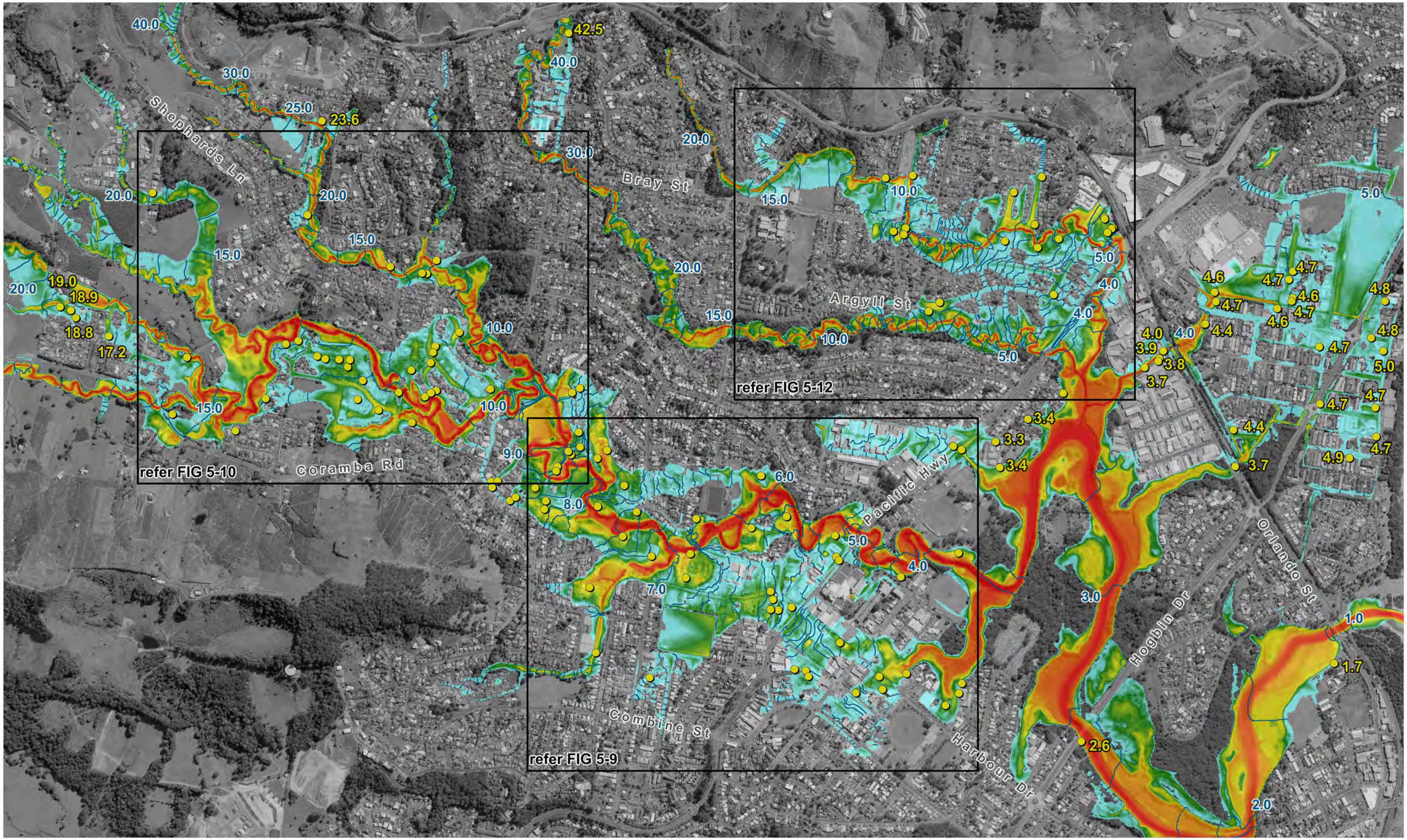
Webb, McKeown and Associates (2001) *Coffs Creek Flood Study*.

WMAwater (2013) *Coffs Harbour Region March/April 2009 Flood Event.*

Appendix A March 2009 Model Calibration (All Calibration Points)



LEGEND

Peak Flood Depth (m)

- 0.2 (lower depths mapped as same colour)
- 0.5
- 1.0
- 2.0
- 4.0 (higher depths mapped as same colour)

1.5 Peak Flood Level Contours (m AHD) (0.2m interval up to 10m AHD, 1.0m interval for 10m AHD and over)

1.5 Location and Level of Flood Marks

Title:

Coffs Creek & Park Beach Flood Study
March 2009 Model Calibration - All Flood Marks

Figure: **A-1**

Rev: **A**

N

0 250 500m
Approx. Scale

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Filepath : K:\N20327_Coffs_Creek\MapInfo\Workspaces\Figures\DRG_A01_170412_Mar09_Calibration_all_A3.WOR

Appendix B A3 Long Sections

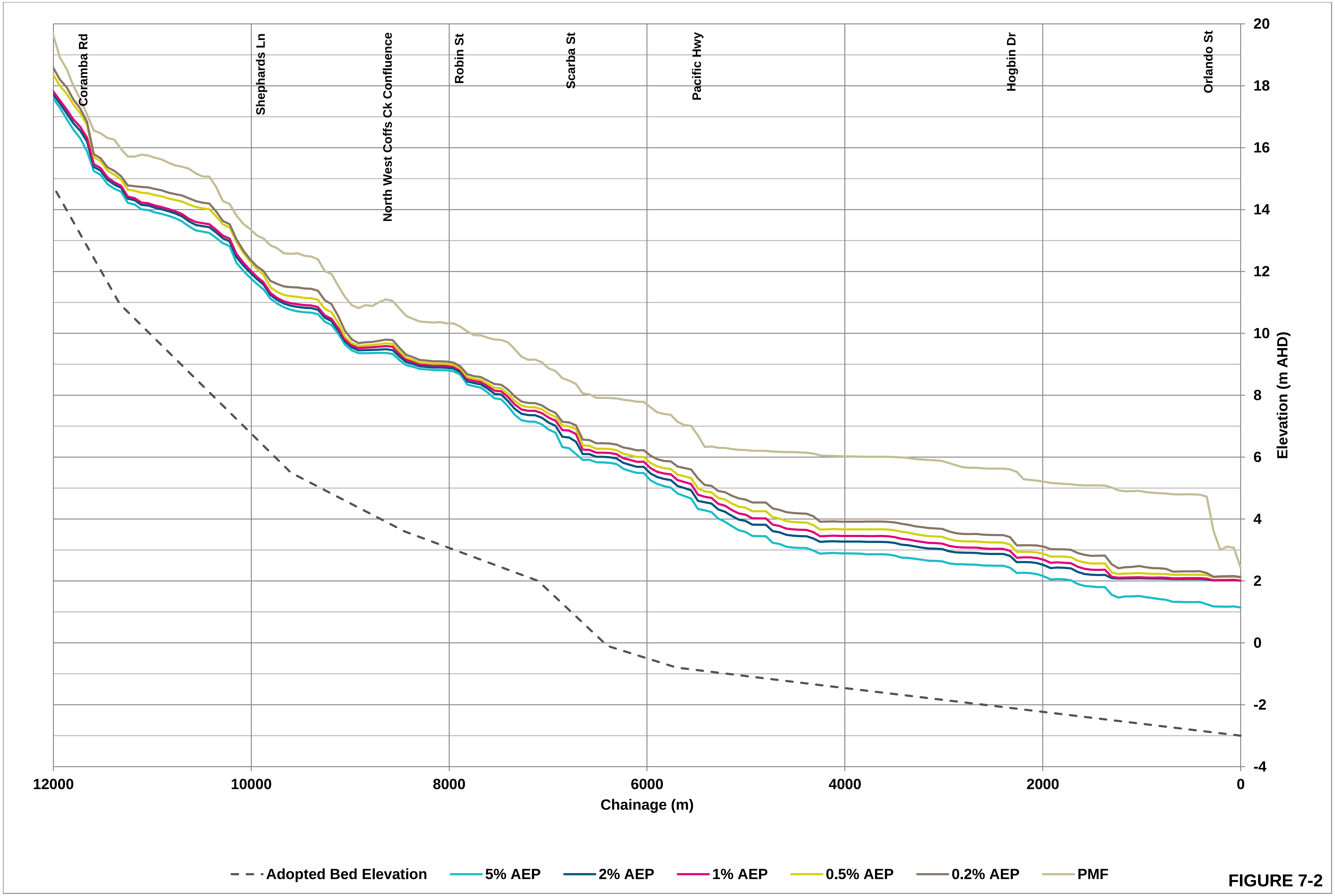
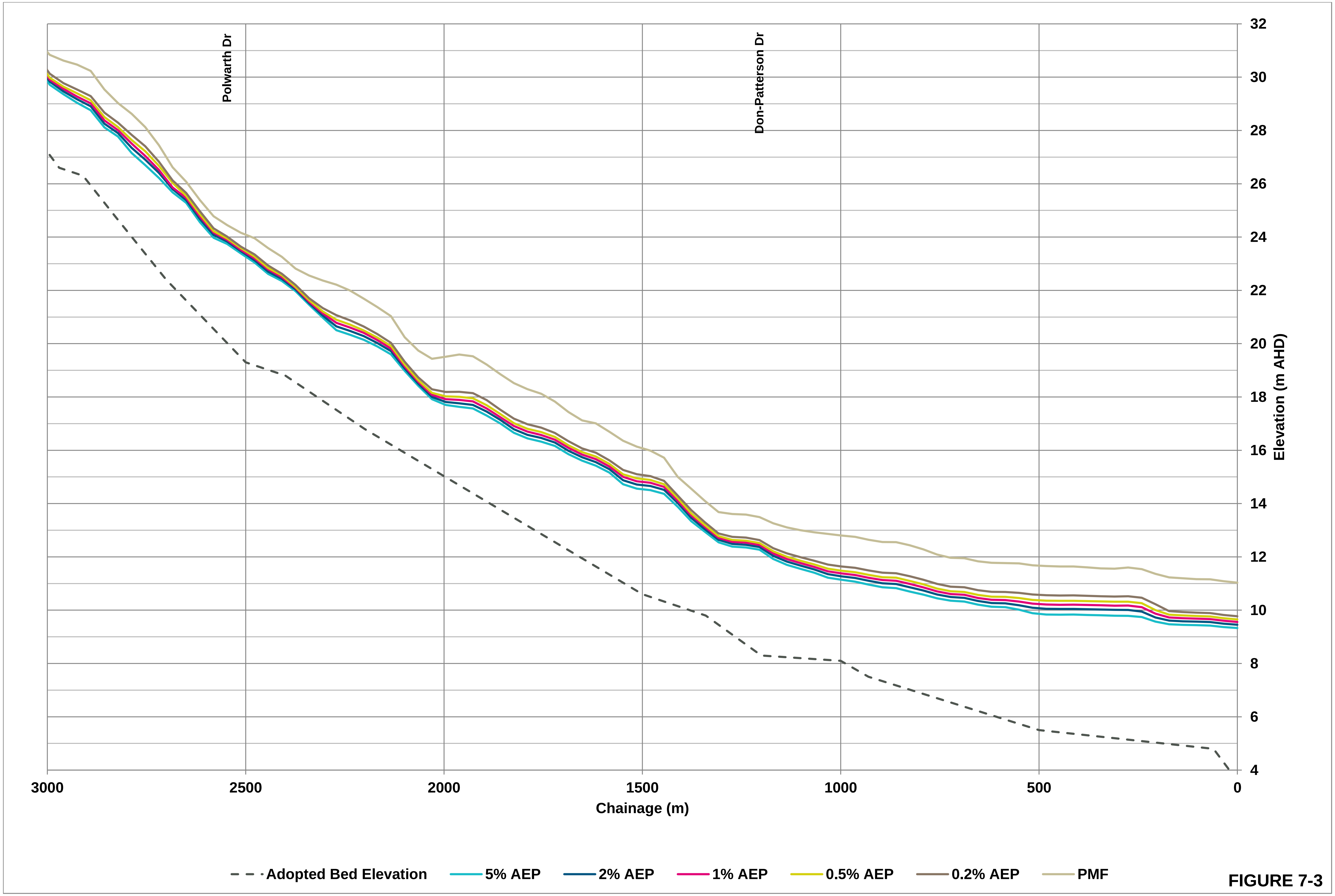
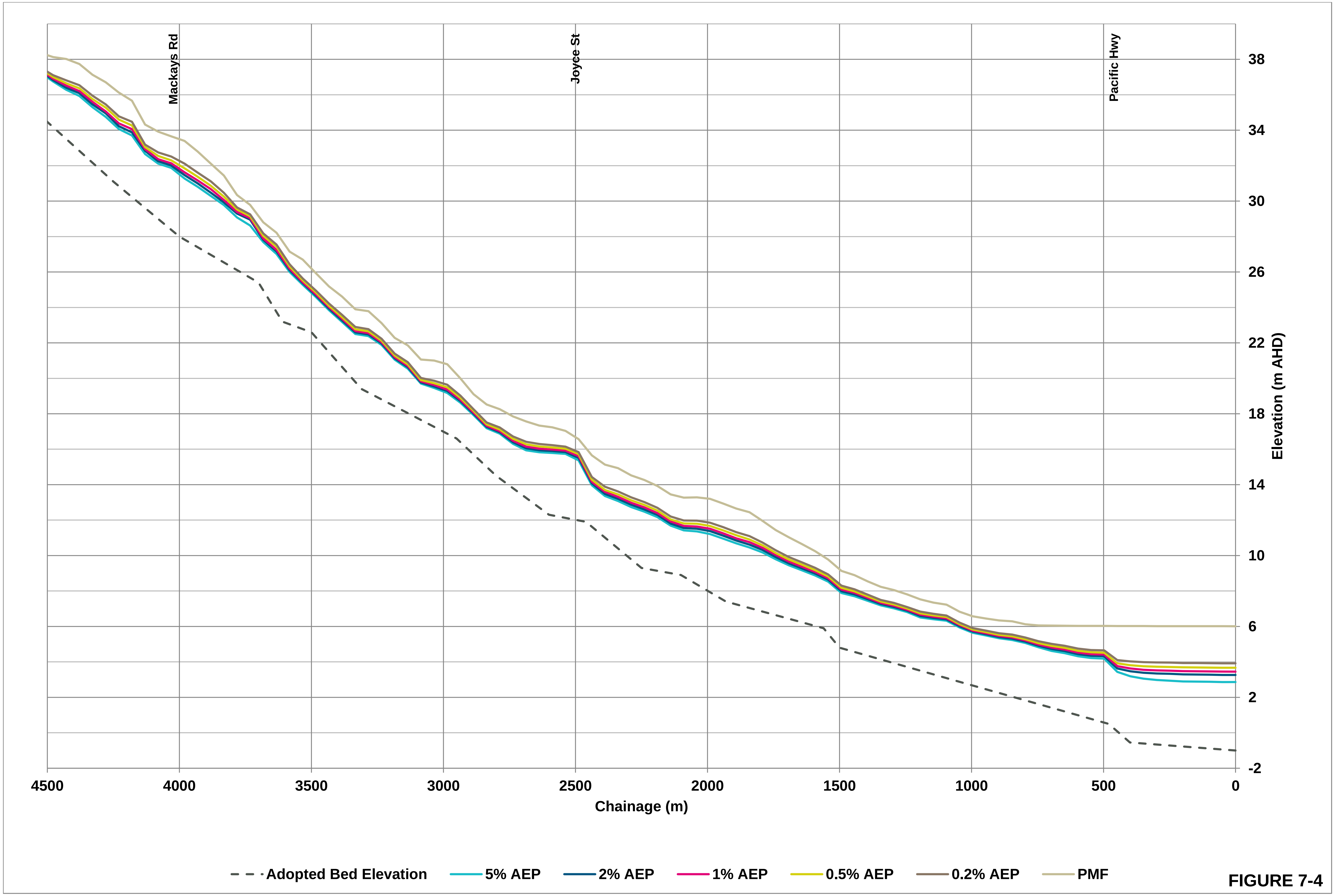

Coramba Rd
Shephards Ln
North West Coffs Ck Confluence
Robin St
Scarba St
Pacific Hwy
Hogbin Dr
Orlando St
20
18
16
14
12
10
8
6
4
2
0
-2
-4
Elevation (m AHD)
12000
10000
8000
6000
4000
2000
0
Chainage (m)
Adopted Bed Elevation
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF

FIGURE 7-2



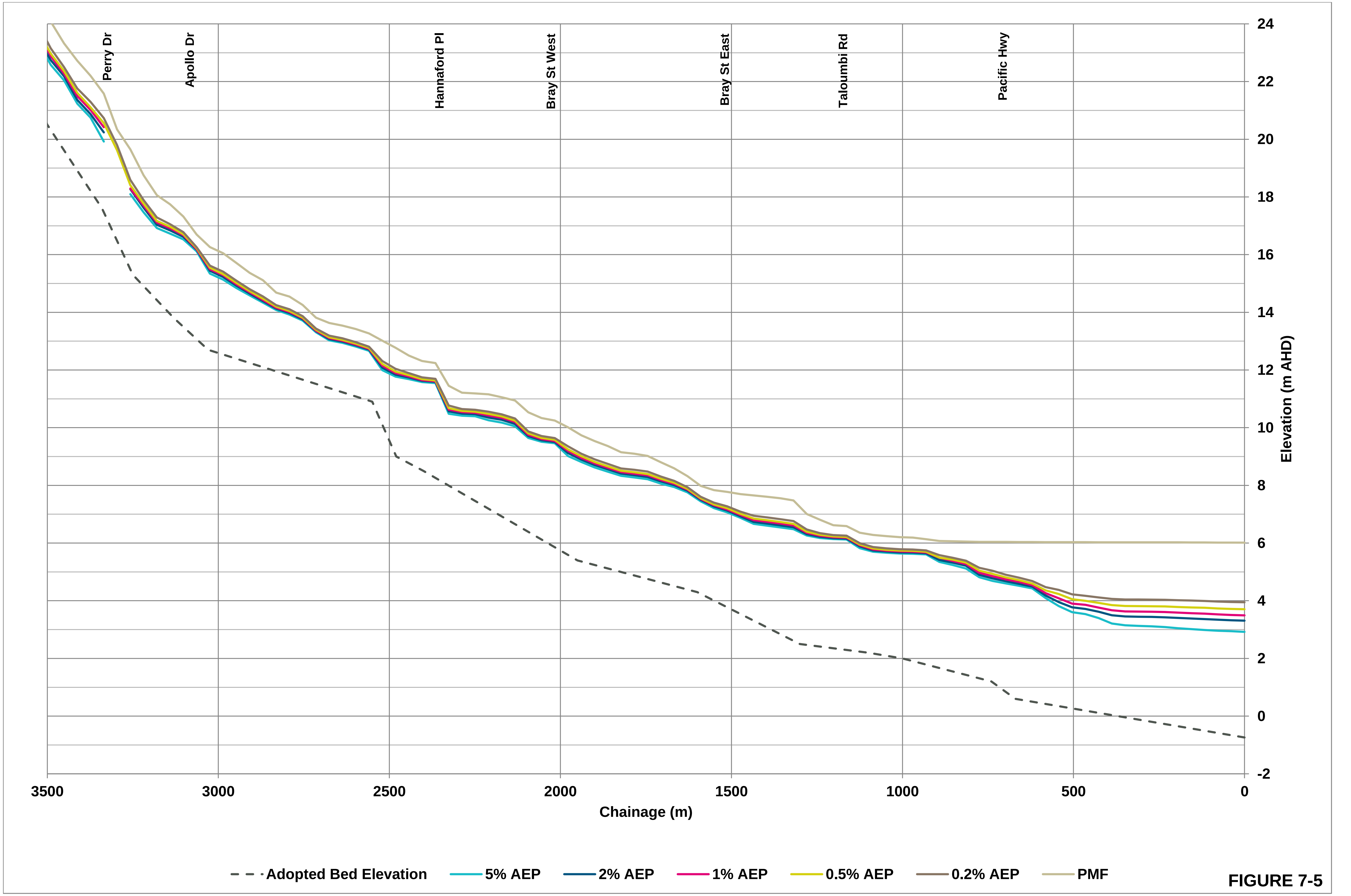
Polwarth Dr
Don-Patterson Dr
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
Elevation (m AHD)
3000
2500
2000
1500
1000
500
0
Chainage (m)
Adopted Bed Elevation
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF

FIGURE 7-3



Mackays Rd
Joyce St
Pacific Hwy
38
34
30
26
22
18
14
10
6
2
-2
Elevation (m AHD)
4500
4000
3500
3000
2500
2000
1500
1000
500
0
Chainage (m)
Adopted Bed Elevation
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF

FIGURE 7-4



Perry Dr
Apollo Dr
Hannaford Pl
Bray St West
Bray St East
Taloumbi Rd
Pacific Hwy
24
22
20
18
16
14
12
10
8
6
4
2
0
-2
Elevation (m AHD)
3500
3000
2500
2000
1500
1000
500
0
Chainage (m)
Adopted Bed Elevation
5% AEP
2% AEP
1% AEP
0.5% AEP
0.2% AEP
PMF

FIGURE 7-5

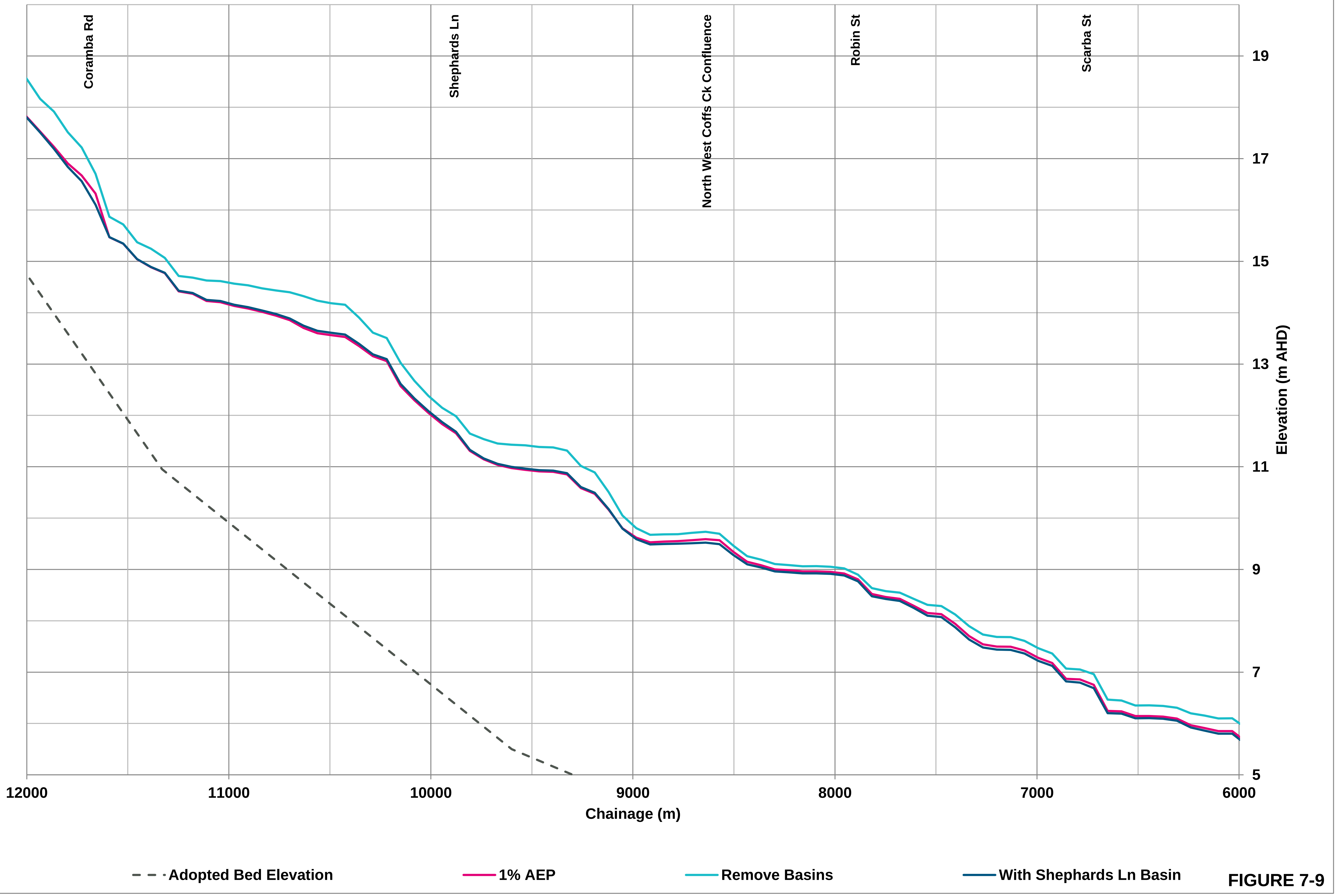

Coramba Rd
Shephards Ln
North West Coffs Ck Confluence
Robin St
Scarba St
19
17
15
13
11
9
7
5
Elevation (m AHD)
12000
11000
10000
9000
8000
7000
6000
Chainage (m)
Adopted Bed Elevation
1% AEP
Remove Basins
With Shephards Ln Basin

FIGURE 7-9

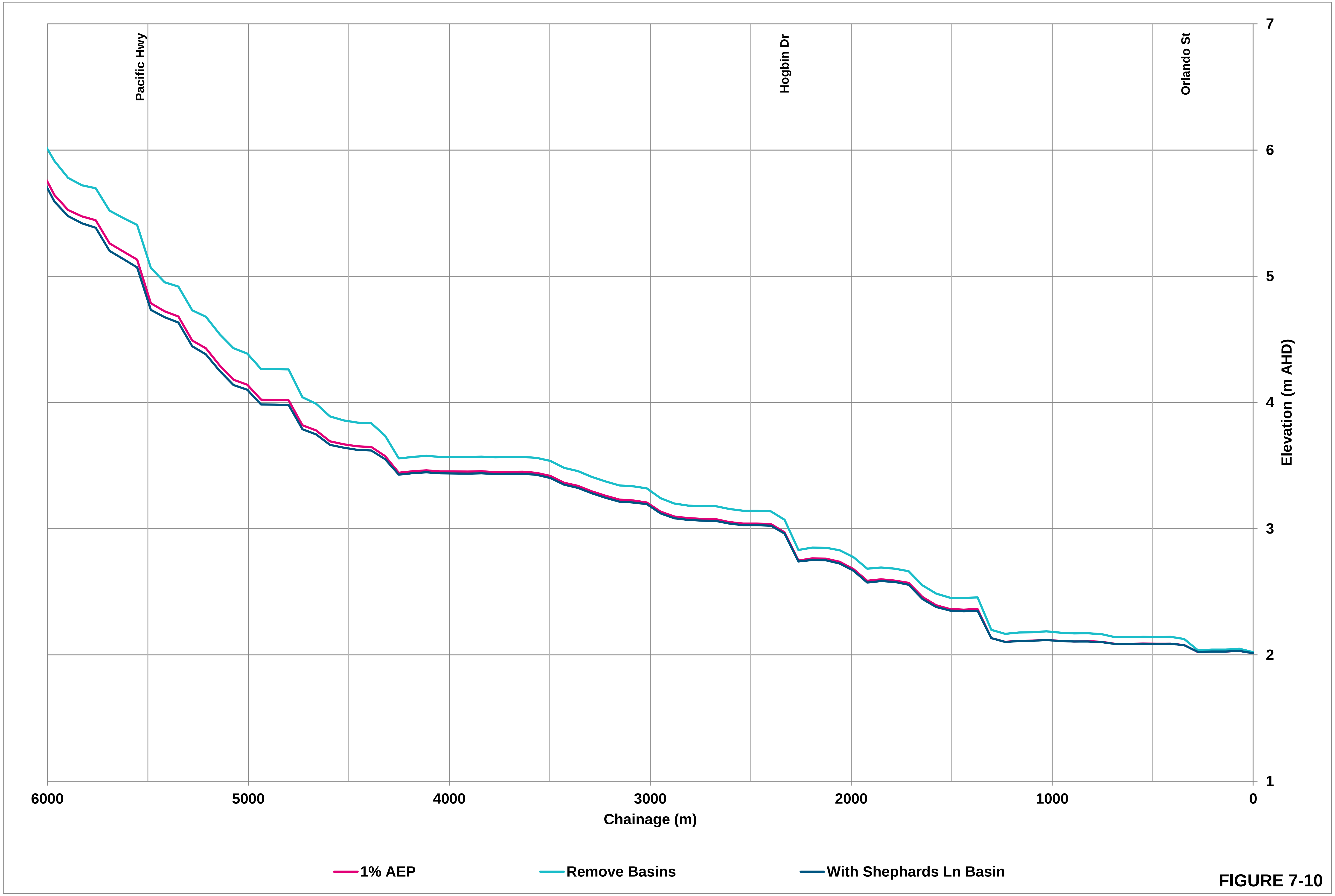
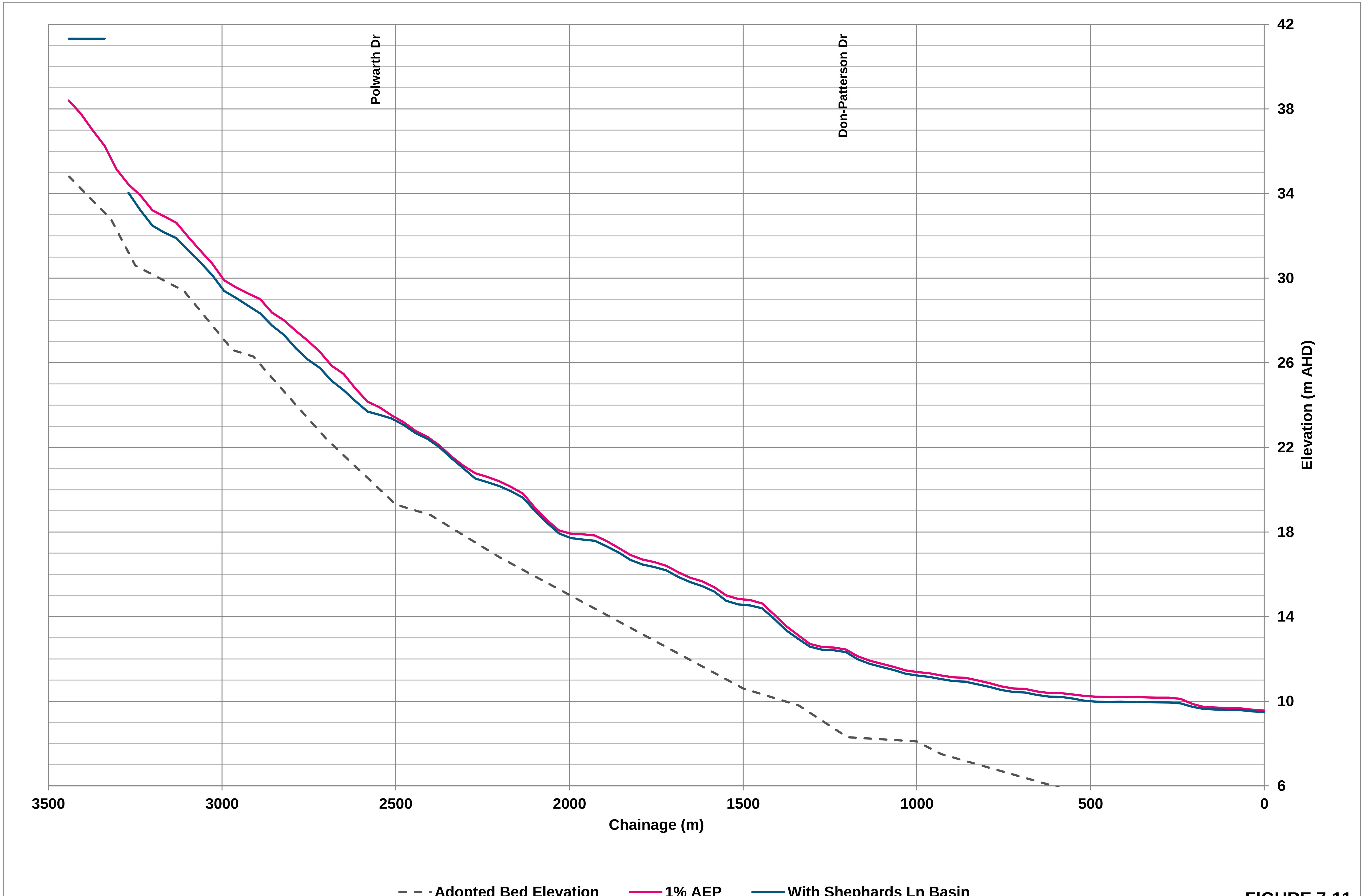
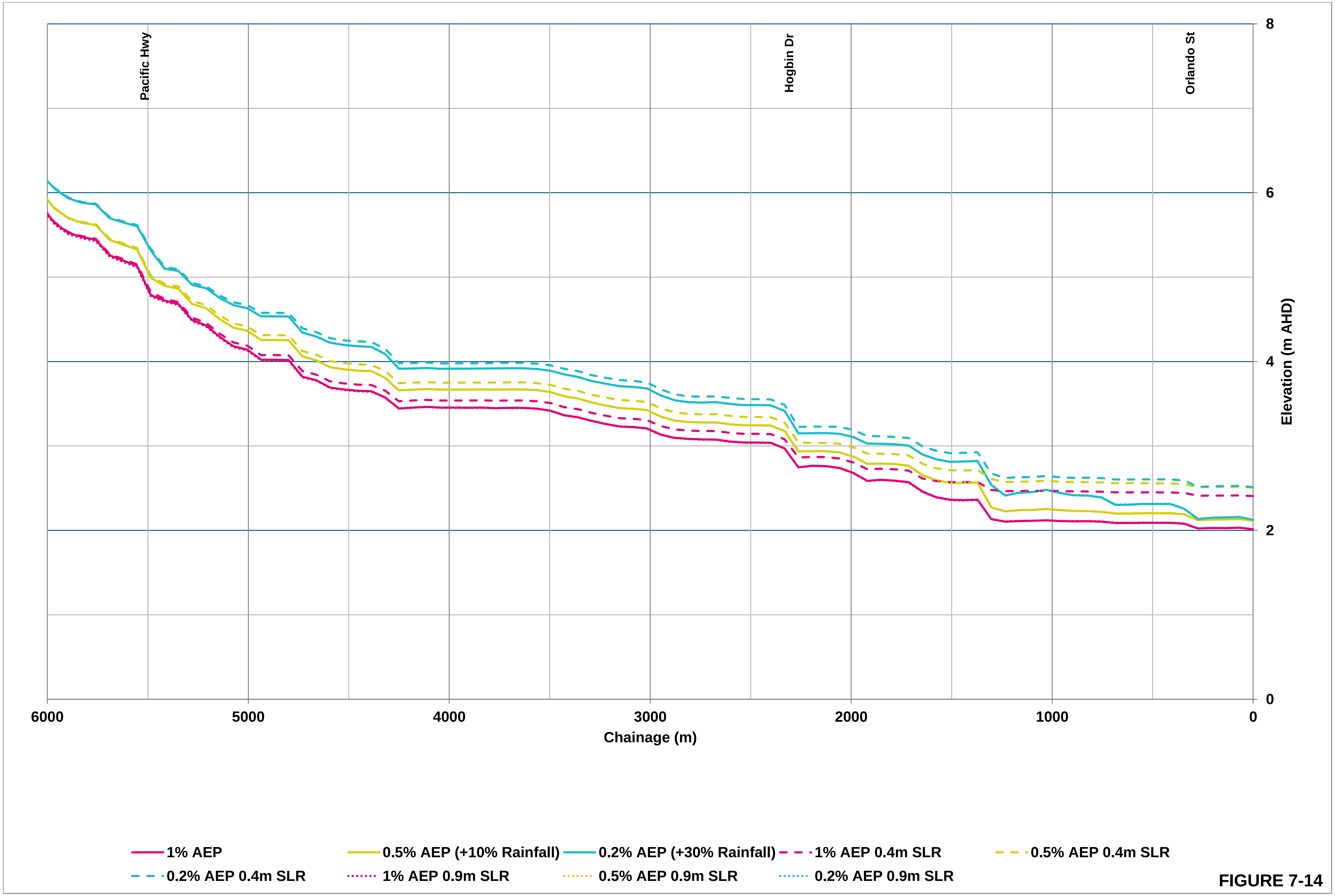

Pacific Hwy
Hogbin Dr
Orlando St
7
6
5
4
3
2
1
Elevation (m AHD)
6000
5000
4000
3000
2000
1000
0
Chainage (m)
1% AEP
Remove Basins
With Shephards Ln Basin

FIGURE 7-10



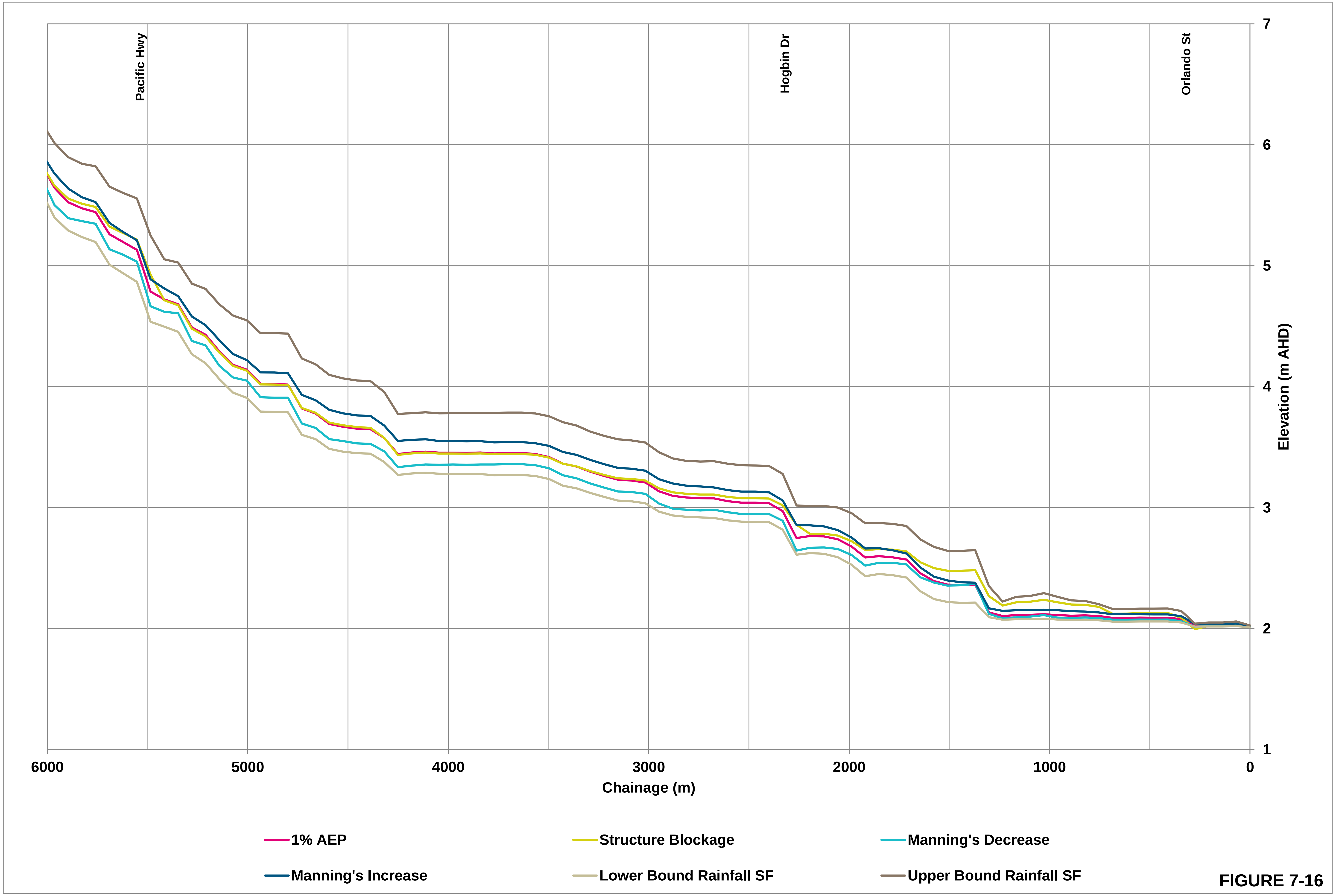
Polwarth Dr
Don-Patterson Dr
42
38
34
30
26
22
18
14
10
6
Elevation (m AHD)
3500
3000
2500
2000
1500
1000
500
0
Chainage (m)
Adopted Bed Elevation
1% AEP
With Shephards Ln Basin

FIGURE 7-11



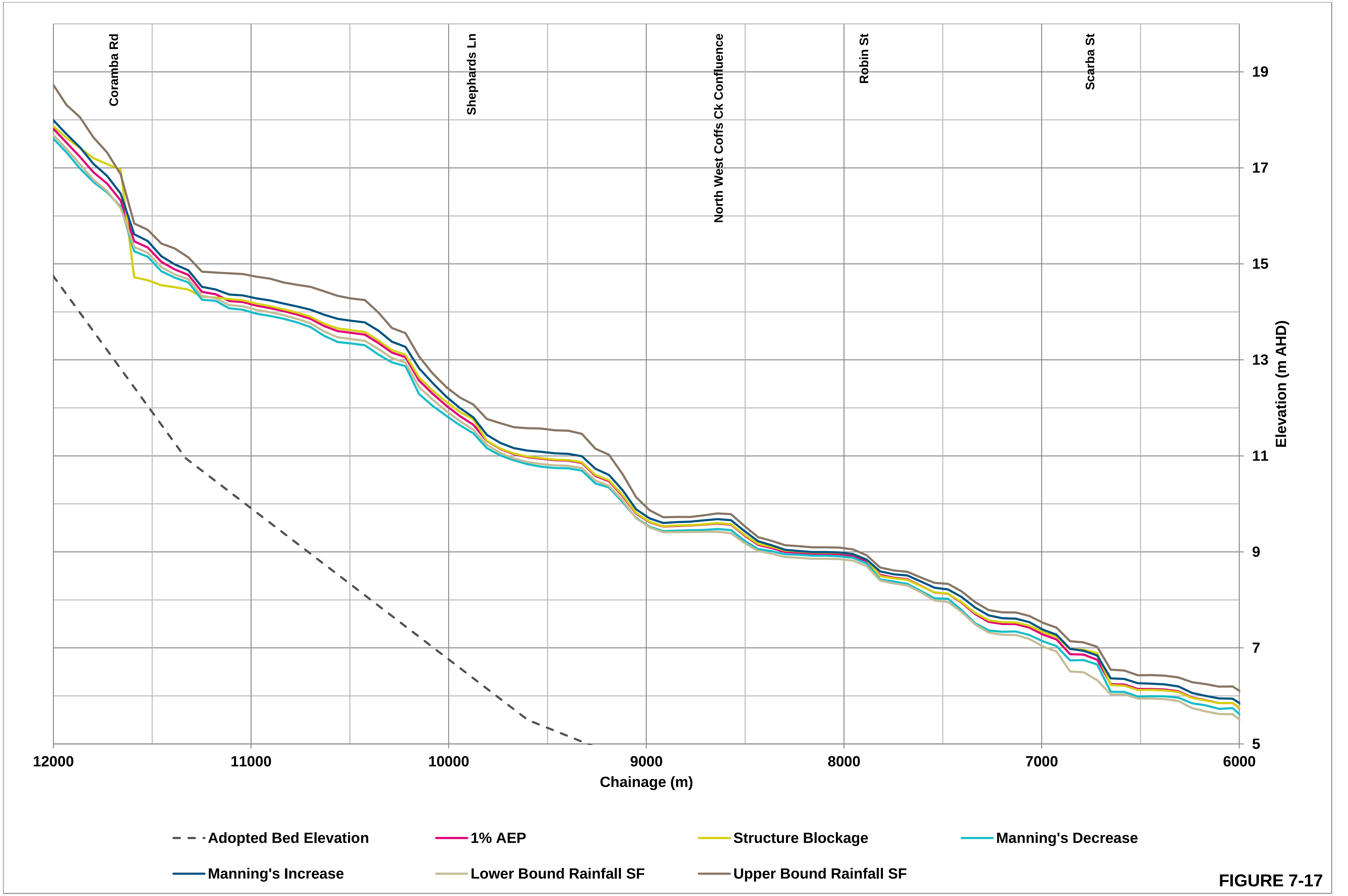
Pacific Hwy
Hogbin Dr
Orlando St
8
6
4
2
0
Elevation (m AHD)
6000
5000
4000
3000
2000
1000
0
Chainage (m)
1% AEP
0.5% AEP (+10% Rainfall)
0.2% AEP (+30% Rainfall)
1% AEP 0.4m SLR
0.5% AEP 0.4m SLR
0.2% AEP 0.4m SLR
1% AEP 0.9m SLR
0.5% AEP 0.9m SLR
0.2% AEP 0.9m SLR

FIGURE 7-14



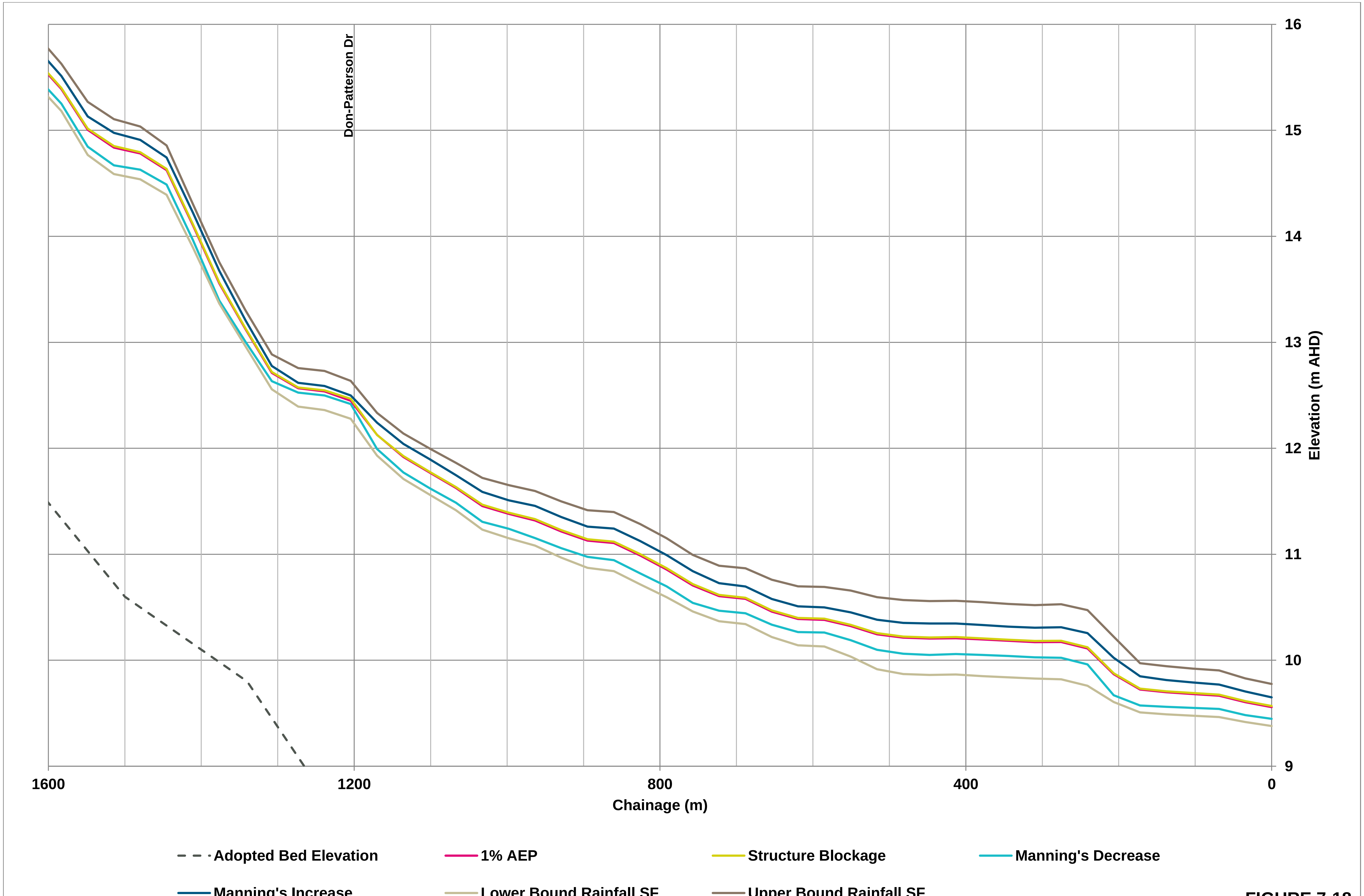
Pacific Hwy
Hogbin Dr
Orlando St
7
6
5
4
3
2
1
6000
5000
4000
3000
2000
1000
0
Chainage (m)
Elevation (m AHD)
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-16



Coramba Rd
Shephards Ln
North West Coffs Ck Confluence
Robin St
Scarba St
19
17
15
13
11
9
7
5
Elevation (m AHD)
12000
11000
10000
9000
8000
7000
6000
Chainage (m)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-17



Don-Patterson Dr
16
15
14
13
12
11
10
9
Elevation (m AHD)
1600
1200
800
400
0
Chainage (m)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-18

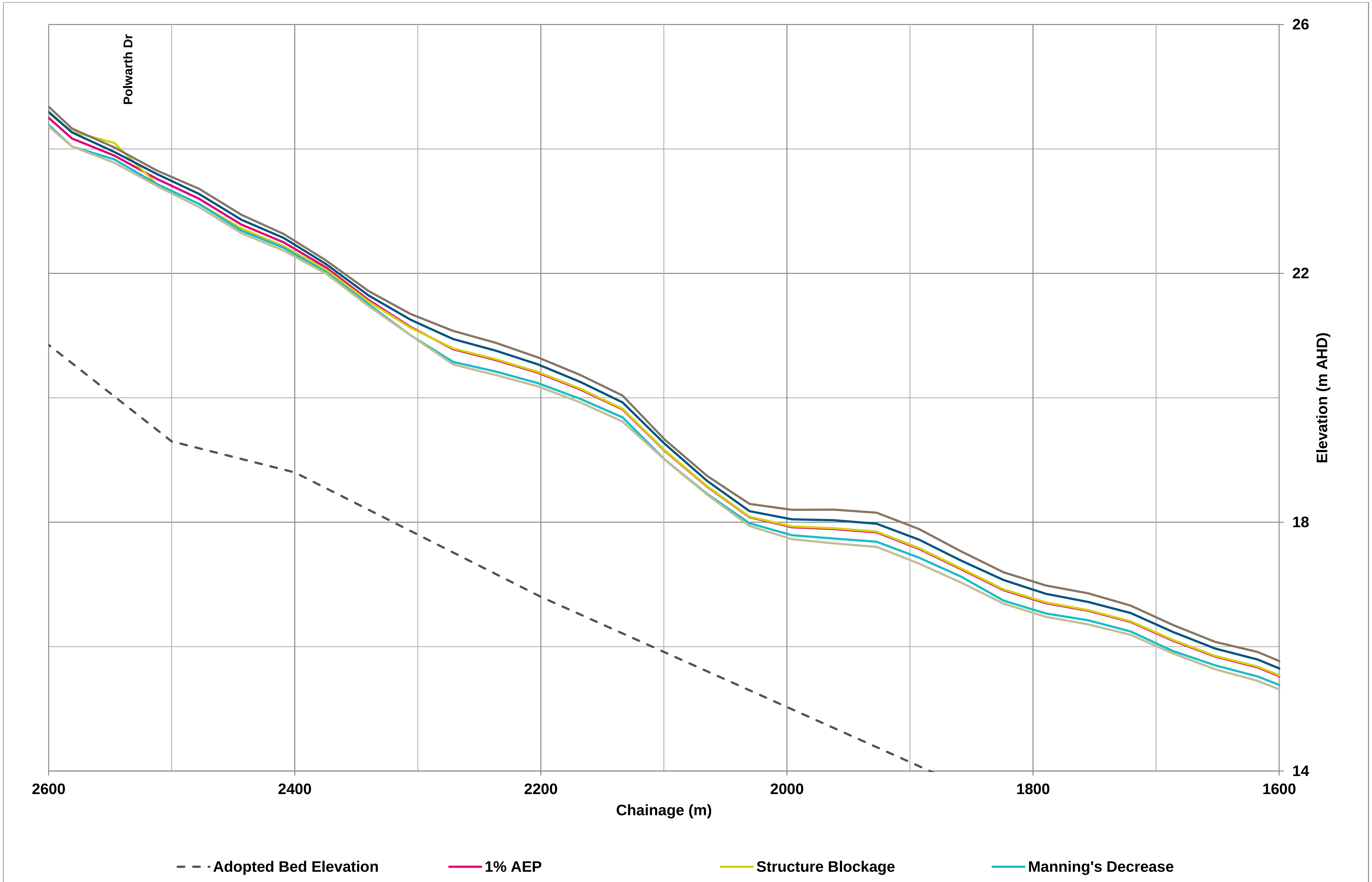
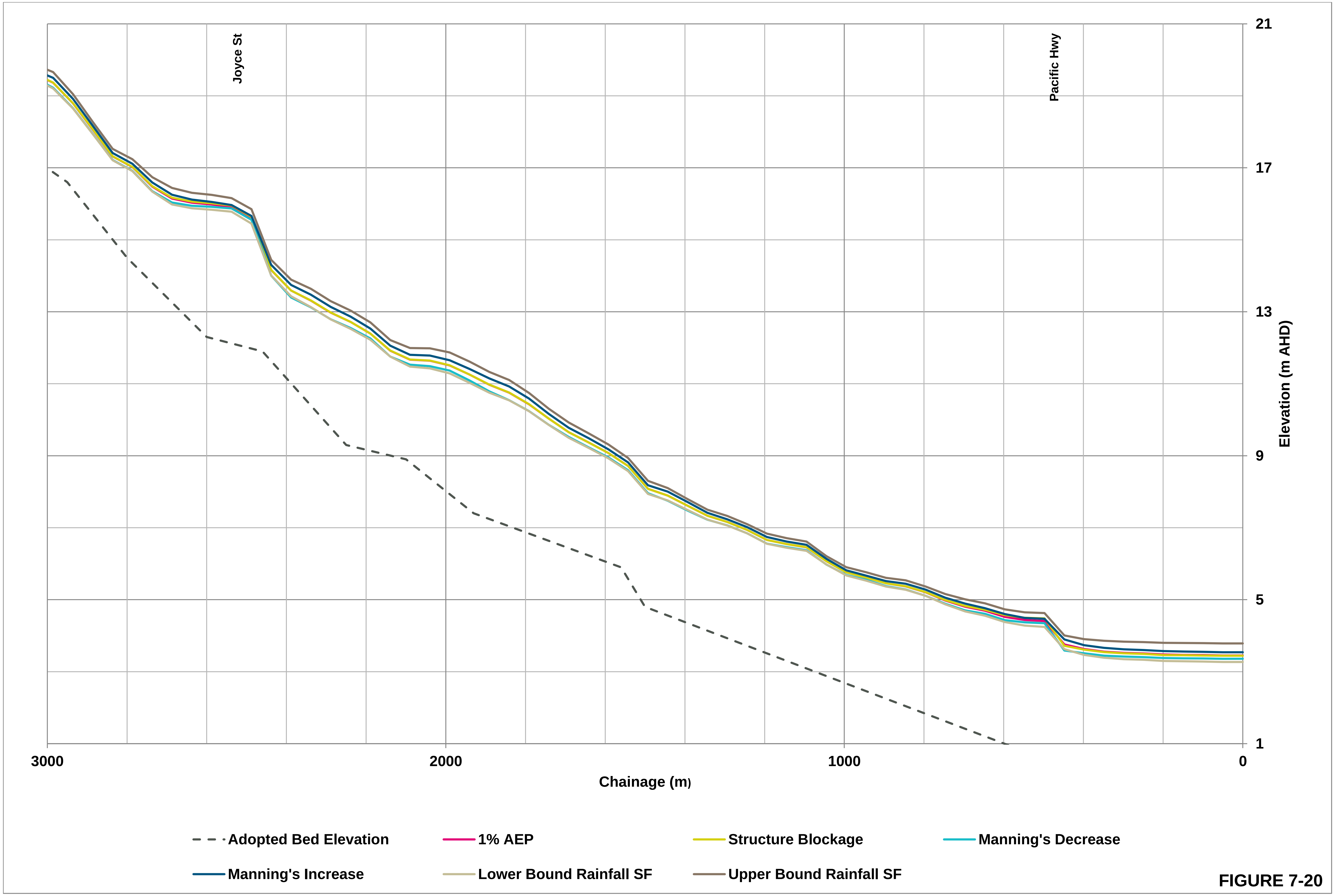

Polwarth Dr
26
22
18
14
Elevation (m AHD)
2600
2400
2200
2000
1800
1600
Chainage (m)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-19



Joyce St
Pacific Hwy
21
17
13
9
5
1
3000
2000
1000
0
Chainage (m)
Elevation (m AHD)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-20

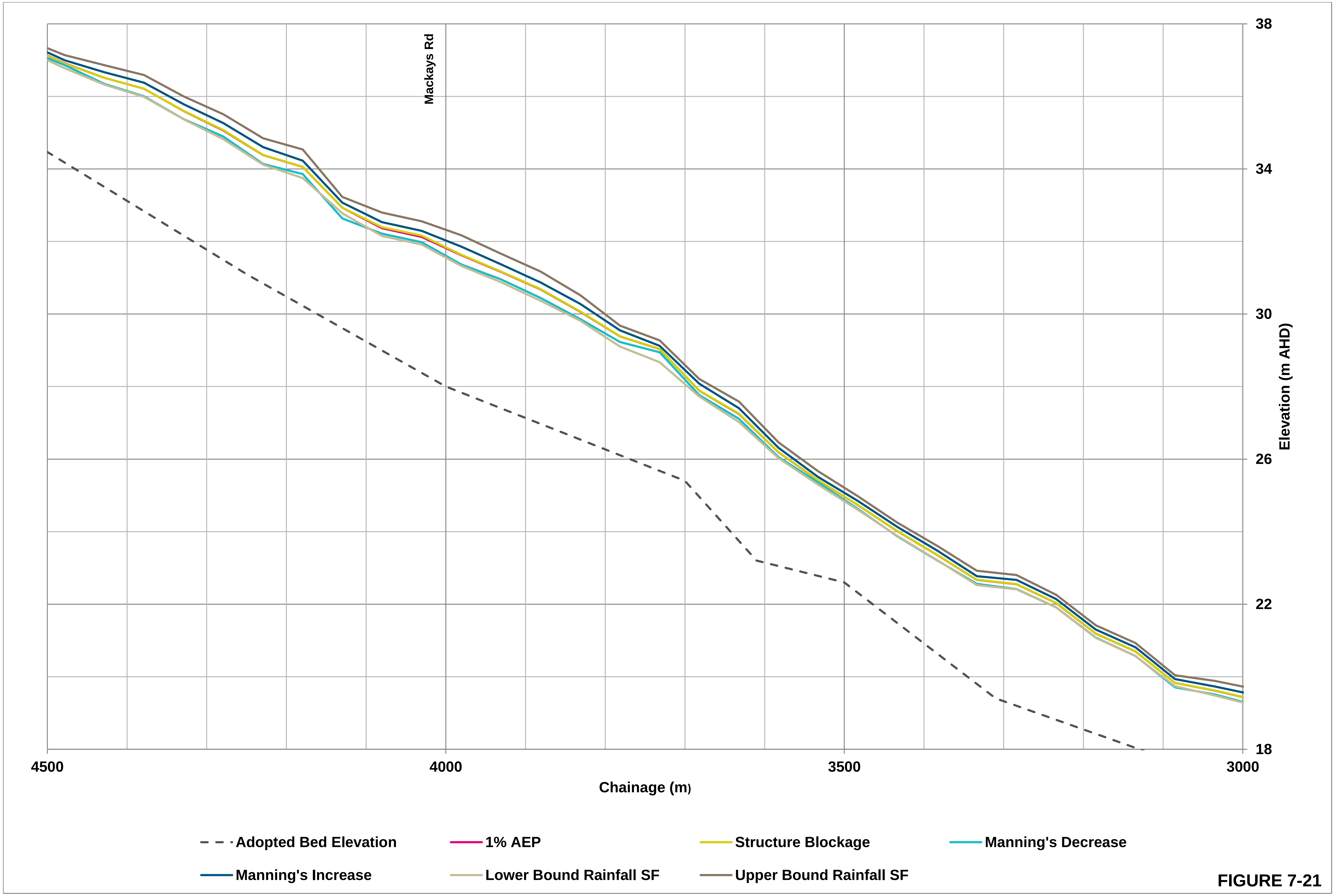

Mackays Rd
38
34
30
26
22
18
Elevation (m AHD)
4500
4000
3500
3000
Chainage (m)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-21

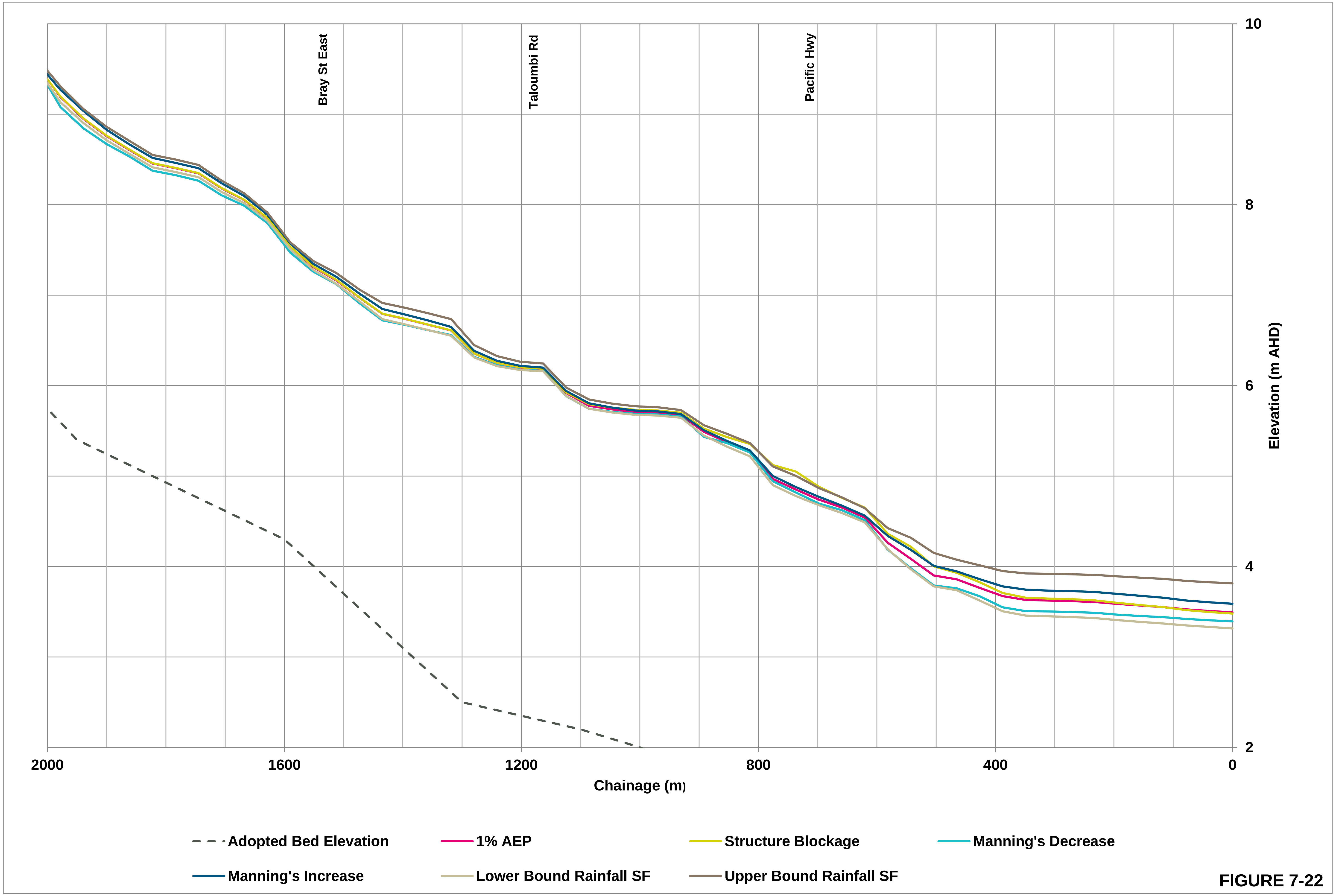

Bray St East
Taloumbi Rd
Pacific Hwy
10
8
6
4
2
2000
1600
1200
800
400
0
Chainage (m)
Elevation (m AHD)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-22

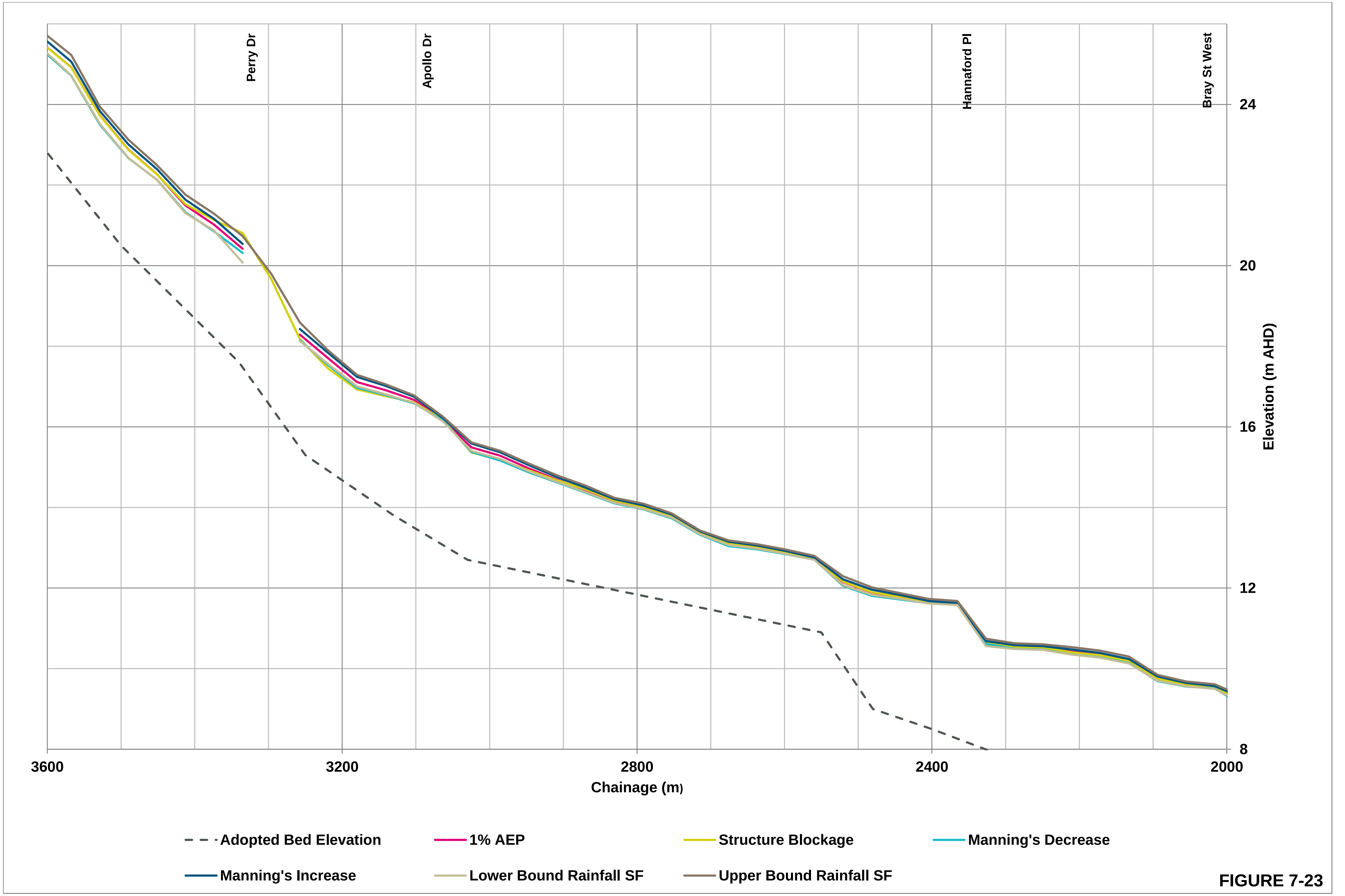

Perry Dr
Apollo Dr
Hannaford Pl
Bray St West
24
20
16
12
8
Elevation (m AHD)
3600
3200
2800
2400
2000
Chainage (m)
Adopted Bed Elevation
1% AEP
Structure Blockage
Manning's Decrease
Manning's Increase
Lower Bound Rainfall SF
Upper Bound Rainfall SF

FIGURE 7-23

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BMT WBM Melbourne	Level 5, 99 King Street, Melbourne 3000 PO Box 604, Collins Street West VIC 8007 Tel +61 3 8620 6100 Fax +61 3 8620 6105 Email melbourne@bmtwbm.com.au Web www.bmtwbm.com.au
BMT WBM Newcastle	126 Belford Street, Broadmeadow 2292 PO Box 266, Broadmeadow NSW 2292 Tel +61 2 4940 8882 Fax +61 2 4940 8887 Email newcastle@bmtwbm.com.au Web www.bmtwbm.com.au
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