EUROBODALLA SHIRE COUNCIL

TOMAKIN, MOSSY POINT, BROULEE AND MOGO FLOOD STUDY

FINAL REPORT

FEBRUARY 2017

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DRAFT REPORT

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Project Tomakin/Mossy Point/Broulee/Mogo Flood Study	**Project Number** 114088
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LIST OF ABBREVIATIONS

1D	One (1) Dimensional
2D	Two (2) Dimensional
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ARI	Average Recurrence Interval
AR&R	Australian Rainfall and Runoff
ALS	Airborne Laser Scanning
DEM	Digital Elevation Model
E/Y	Exceedances per Year
GSAM	Generalised South-East Australia Method (for PMP calculations)
GSDM	Generalised Short-Duration Method (for PMP calculations)
ICOLL	Intermittently Closed and Open Lake or Lagoon
IFD	Intensity-Frequency-Duration
LGA	Local Government Area
LiDAR	Airborne Light Detection and Ranging Survey
NPWS	National Parks and Wildlife Services
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
TIN	Triangular Irregular Network
UTC	Coordinated Universal Time

FOREWORD

The NSW Government's Flood Prone Land Policy provides a framework to ensure the sustainable use of floodplain environments. The Policy is specifically structured to provide solutions to existing flooding problems in rural and urban areas. In addition, the Policy provides a means of ensuring that any new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The NSW Government provides technical and financial assistance to Councils in the discharge of their floodplain management responsibilities.

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

1. Flood Study
- Determine the nature and extent of the flood problem.

2. Floodplain Risk Management
- Evaluates management options for the floodplain in respect of both existing and proposed development.

3. Floodplain Risk Management Plan
- Involves formal adoption by Council of a plan of management for the floodplain.

4. Implementation of the Plan
- Construction of flood mitigation works to protect existing development, use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

This document forms the first stage of the floodplain risk management process, i.e. the Flood Study.

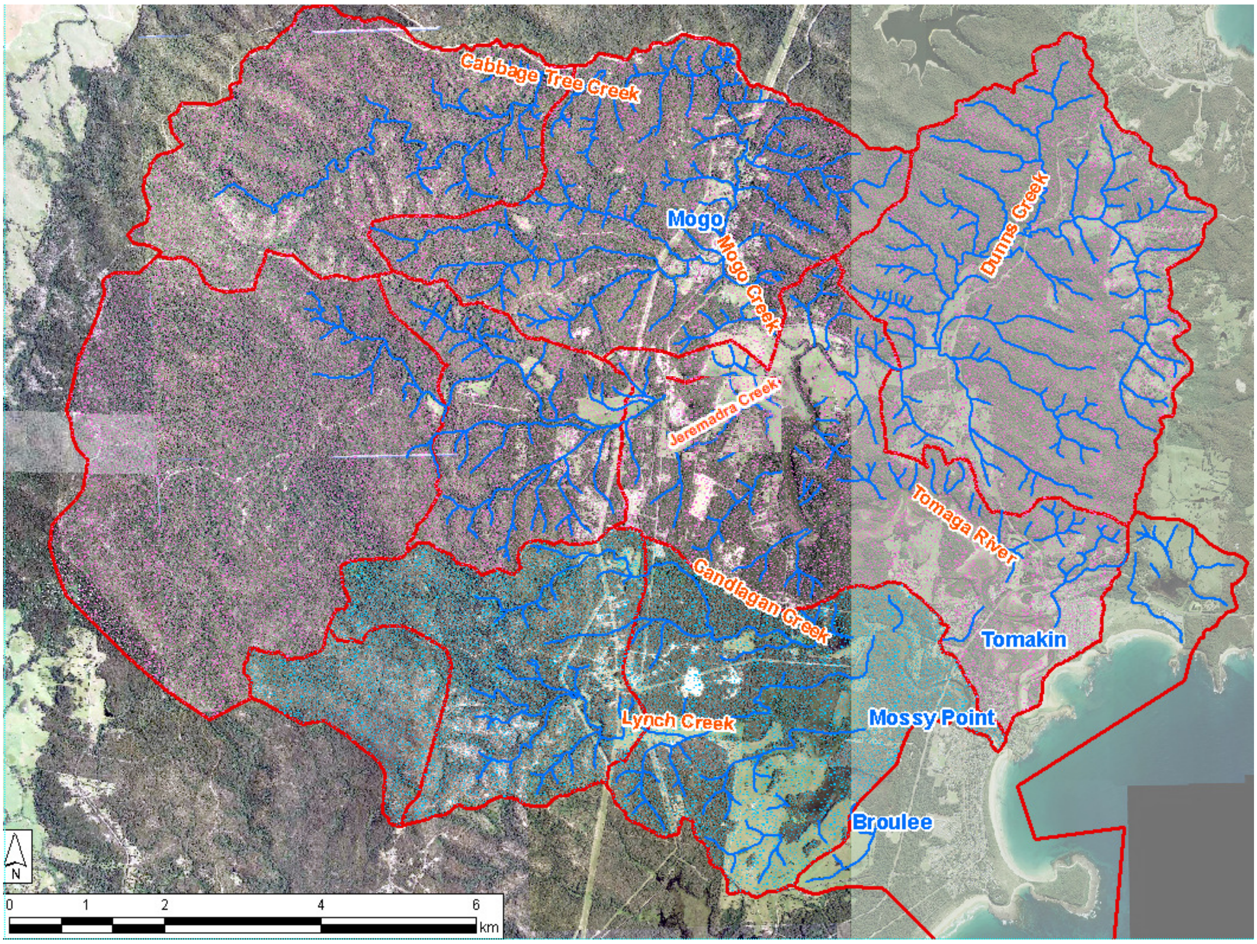
1. INTRODUCTION

1.1. Background

This flood study has been prepared on behalf of Eurobodalla Shire Council (ESC), on the South Coast of New South Wales. It covers the areas of Tomakin, Mossy Point, Broulee and Mogo (Figure 1) over two major catchments. The first catchment has an approximate area of 94 km^2 which drains to the Tomaga River while the second catchment has an approximate area of 26 km^2 draining to Candlagan Creek.

Tomakin, Mossy Point and Broulee are mostly residential while Mogo is characterised by commercial areas with some residential land-use. Mogo with an upstream catchment area of approximately 27 km^2 is subject to both local flooding from Mogo Creek and flooding from Cabbage Tree Creek. Dunns Creek and Jeremadra Creek with approximate catchment areas of 19 km^2 and 30 km^2 respectively are two other major tributaries of the Tomaga River. Lynch Creek is a tributary of Candlagan Creek. Tomakin is located beside the Tomaga River mouth (catchment area of 94 km^2), Broulee is located beside the Candlagan Creek mouth (catchment area of 26 km^2) while Mossy Point is situated in between the two.

Diagram 1: Major Creeks Catchments



1.2. Objectives

The purpose of this Flood Study is to define the flood behaviour under current catchment conditions. This objective is achieved through the development of a suite of hydrologic and hydraulic models that can also be used as the basis for a future Floodplain Risk Management Study and Plan for the study area, and to assist Eurobodalla Shire Council (ESC) when undertaking flood-related planning decisions for existing and future developments.

Following endorsement of the calibration report, assessment of the 20%, 10%, 5%, 2%, 1% and 0.5% AEP design events as well as the Probable Maximum Flood (PMF) has been carried out. The primary objectives of the study are:

- to determine the flood behaviour including design flood levels and velocities over a range of flooding events, from storm runoff in the catchment and from tidal influences;
- to determine provisional residential flood planning areas and flood planning levels;
- to undertake provisional flood emergency response planning classification of communities;
- to provide a model that can establish the effects of flood behaviour of future development; and
- to assess the sensitivity of flood behaviour to potential climate change effects such as increases in rainfall intensities and sea level rise.

The flood study report will detail the results and findings of the Flood Study investigations. The key elements include:

- a summary of available flood related data;
- establishment and validation of the hydrologic and hydraulic models;
- sensitivity analysis of the model results to variation of input parameters;
- the estimation of design flood behaviour for existing catchment conditions;
- preliminary hydraulic categories and provisional hazard mapping;
- preliminary residential flood planning areas and flood planning levels;
- flood emergency response classification of communities; and
- potential implications of climate change projections.

A glossary of flood related terms is provided in Appendix A.

2. AVAILABLE DATA

The data utilised in this study has been sourced from a variety of organisations or references.

2.1. Topographic Data

The catchment topography was defined by Airborne Light Detection and Ranging (LiDAR) survey, bathymetric hydrosurvey and cross-sectional levels from design drawings. Using only ground strikes and water strikes a Triangular Irregular Network (TIN) was generated. Note, the ground strike resolution for Broulee was insufficient consequently for that area, all available strikes were used for generating the TIN as detailed in Section 2.1.1. The resulting TIN was sampled at a regular spacing of 2 m by 2 m and creeks/rivers cut out utilising bathymetric survey and cross-sectional information to create a Digital Elevation Model (DEM). The DEM (discussed further in Section 6.3 and shown in Figure 2) constitutes the basis for the two-dimensional hydraulic model utilised for the study.

2.1.1. LiDAR Survey

LiDAR survey of the catchment and its immediate surroundings was provided for the study by Eurobodalla Shire Council. The LiDAR collected in 2012 originates from the NSW Department of Land and Property Information (LPI). A description of the strike types and their respective classification is shown in Table 1.

The metadata description sheet for the Batemans Bay area LiDAR data indicates an average point density of 1.61/m^2 corresponding to an accuracy in the order of:

- +/- 0.3 m in the vertical direction (to one standard deviation); and
- +/- 0.8 m in the horizontal direction (to one standard deviation).

The accuracy of the LiDAR data can be influenced by a number of factors. LiDAR strike penetration is limited through water and consequently any deeper water areas were supplemented with bathymetric survey and cross-sectional information. Similarly, vegetation (tree or shrub canopy) and structures (buildings or bridges) artificially elevate ground levels and therefore these strikes were discounted with the exception of the Broulee area where true ground strike resolution was insufficient. For the Broulee area, a large concentration of points was unclassified or classified as medium to high vegetation and inclusion of these categories was required to obtain sufficient resolution for the TIN in that area. On the ground verification of apparent anomalies in the resulting TIN by WMAwater engineers found the features to be in fact similar to that observed in the generated TIN. Overall, no observed discrepancy was conclusive enough to justify manually manipulating grid levels for the Broulee area.

Table 1: LiDAR Point Cloud Classification Scheme

Number	Point Class	Description
0	Unclassified	Created, never classified
1	Default	Unclassified
2	Ground	Bare ground
3	Low Vegetation	0-0.3m (essentially sensor 'noise')
4	Medium Vegetation	0.3-2m
5	High Vegetation	>2m
6	Buildings, Structures	Buildings, houses etc.
7	Low/High Points	Spurious point return (unusable)
8	Model Key Points	Reserved for 'model key points' only
9	Water	Any point in water
10	Bridge	Any bridge overpass
11	Not used	Reserved for future definition
12	Overlap Points	Flight line overlap points
13-31	Not used	Reserved for future definition

2.1.2. Bathymetric Survey

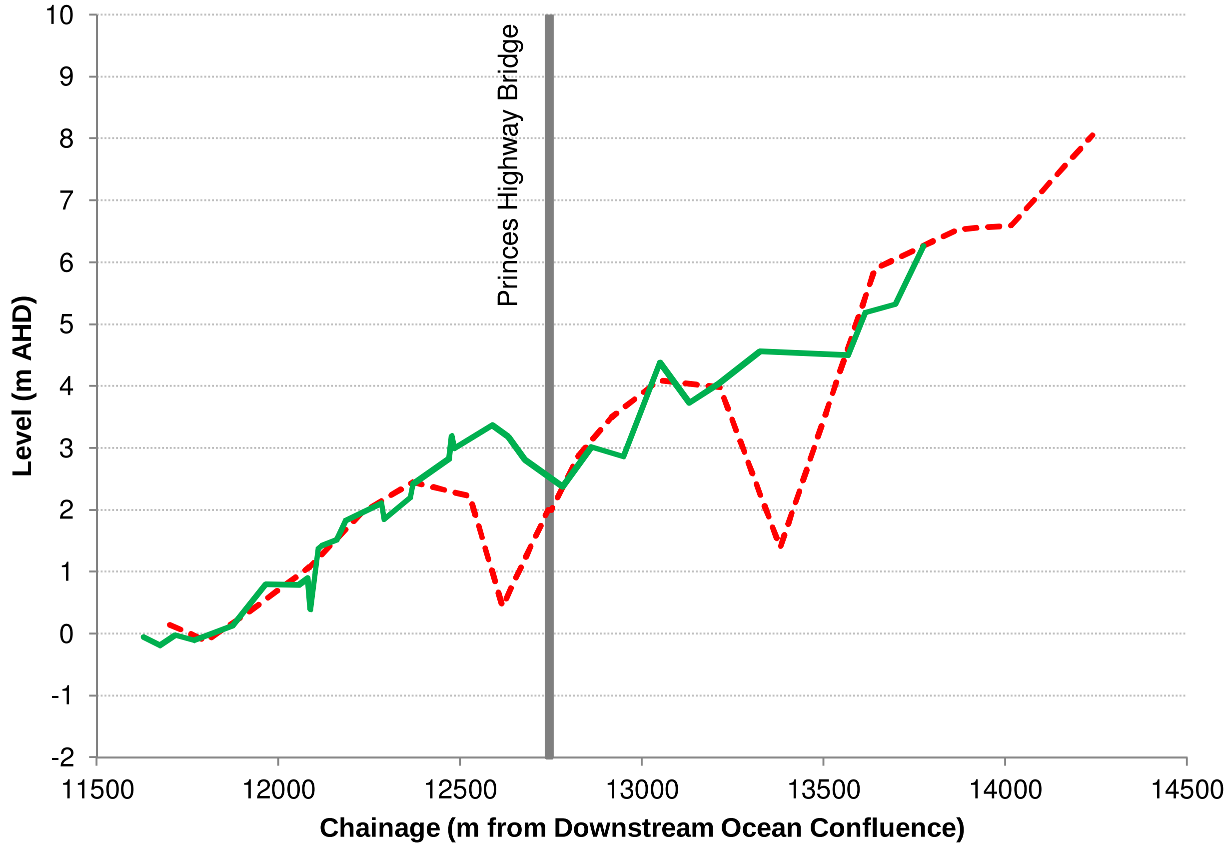
The bathymetric survey for the Tomaga River is available from the Office of Environment and Heritage (OEH) website. The website indicates that the data (shown in Figure 2) was collected in December 1998. High resolution data is available for the river mouth as far as the George Bass Drive Bridge. Cross-sectional data at regular 200-300 metre intervals is available further upstream as far as approximately where Tomakin Road crosses Dunns Creek.

2.1.3. Ground Survey

Ground survey of the Mogo Creek invert levels were undertaken by Eurobodalla Shire Council in December 2016. The survey extended from 1,140 m downstream of the Princes Highway and 1,043 m upstream of the Princes Highway.

The surveyed invert levels were compared to the invert levels surveyed in 1986/1987 as part of the Mogo Flood Study (discussed in Section 2.8.1). From this comparison, the general slope of the creek invert was similar; however two significant depressions (of approximately 3.1 m and 2.4 m) that appeared in the 1987 data was not present in the 2016 data, shown in Chart 1.

Chart 1: Creek invert survey undertaken in 2016 (shown as a green solid line) compared to the creek invert survey used in the 1987 Mogo Flood Study (shown as a red dotted line)



2.1.4. Cross-sectional Levels

Available bridge and road drawings for the study area were provided by Eurobodalla Shire Council. Drawings including site survey were used to inform levels particularly for Candlagan Creek.

2.2. Culvert and Bridge Data

Roads and Maritime Services provided GIS data for drainage assets within the study area particularly culverts traversing the Princes Highway and Mogo Bridge.

Additionally, Eurobodalla Shire Council provided Culvert and Bridge data. All major bridges shown in Figure 3 were independently verified by site visit and where not provided, bridge parameters were estimated from visual inspection. Similarly, where culvert dimensions were not available, diameters were estimated from visual inspection.

Bridge and culvert structures included in the hydraulic model are shown in Figure 4.

2.3. Pit and Pipe Data

Eurobodalla Shire Council provided an asset database that included pit and pipe data for the stormwater network, the sewage network and the potable water network. The stormwater network was included in the hydraulic modelling process as shown in Figure 4.

The stormwater pipe data detailed the dimensions of the ESC-owned structures across the study areas. The invert level of the upstream and downstream end of the pipes were provided for the most part and these were used to inform pit invert levels. Where invert levels were not available, levels were estimated by subtracting an assumed cover and the pipe diameter from the TIN levels.

2.4. Historic Water Level Data (Continuous)

A water level recorder is available within the Tomaga River catchment situated at George Bass Drive. The gauge is operated by Manly Hydraulics Laboratory (MHL) and was commissioned in August 1996. The water level gauge is summarised in Table 2 and shown in Figure 5.

Table 2: Water Level Stations Operated by MHL within the Study Area

Station Number	Station Name	Operating Authority	Date Opened
216455	George Bass Drive	MHL	28/08/1996

The water level data supplied is reported as having an accuracy range in the order of +/- 0.02 m and is tidally affected. There are no other publicly available water level records for the Tomaga River and Candlagan Creek catchments.

2.5. Historic Ocean Tide Datum (Continuous)

The ocean tide stations closest to the study area are summarised in Table 3. The gauges are operated by Manly Hydraulics Laboratory (MHL).

Table 3: Ocean Tide Level Stations

Station Number	Station Name	Operating Authority	Distance from centre of catchment (km)	Date Opened
216410	Batemans Bay Clyde River at Princess Jetty	MHL	13	01/12/1985
216471	Ulladulla Harbour	MHL	59	06/12/2007
219470	Bermagui	MHL	68	29/07/1987
216470	Jervis Bay	MHL	93	01/09/1989

Data was provided in 15 minute increments in Australian Eastern Standard Time (AEST). The vertical datum of the Princess Jetty data and Ulladulla Harbour data is AHD. The Bermagui data was provided in Bermagui Local Hydro Datum (BLHD = -0.714 m AHD) and the Jervis Bay data was in Chart Datum (CD = -1.070 m AHD).

2.5.1. NSW Tidal Planes Analysis

Manly Hydraulics Laboratory prepared the *NSW Tidal Planes Analysis: 1990-2010 Harmonic Analysis* report on behalf of the NSW Office of Environment and Heritage. It was released in October 2012 and was based on data from 188 tidal monitoring stations from the 1st July 1990 to the 30th June 2010. Data from the Ulladulla Harbour station is shown in Table 4.

Table 4: Tidal Planes Analysis Results for Ulladulla Harbour Gauge (MHL, 2012)

Tidal Planes	Annual Average Amplitude (m AHD)
High High Water Solstices Springs (HHWSS)	0.960
Mean High Water Springs (MHWS)	0.617
Mean High Water (MHW)	0.510
Mean High Water Neaps (MHWN)	0.403
Mean Sea Level (MSL)	0.040
Mean Low Water Neaps (MLWN)	-0.325
Mean Low Water (MLW)	-0.431
Mean Low Water Springs (MLWS)	-0.538
Indian Spring Low Water (ISLW)	-0.783

2.6. Historic Rainfall Data

There are a number of rainfall stations close to the study area. This includes daily read stations and continuous pluviometer stations. The daily read stations record total rainfall for the 24 hours to 9am of the day being recorded. For example, the rainfall received for the period between 9:00am 28th January to 9:00am 29th January 1999 would be recorded on the 29th January 1999.

The continuous pluviometer stations record rainfall in sub-daily increments. These records are typically used to create the rainfall temporal distribution used to model the historical events, against which the hydrologic and hydraulic models are calibrated.

Table 5 presents a summary of the official rainfall gauges located close to or within the catchment. These gauges are operated by the Bureau of Meteorology (BOM) for the most part and Eurobodalla Shire Council (ESC) operates the tipping bucket located at Deep Creek Dam. As shown in Figure 5, while these stations are situated proximate the catchment area, none are actually located within the Tomaga River and Candlagan Creek catchments.

Table 5: Rainfall Stations Proximate the Tomaga River/Candlagan Catchments

Station Number	Station Name	Operating Authority	Latitude	Longitude	Height (m AHD)	Distance from Catchment Boundary (km)	Distance from Centre of Catchment (km)	Date Opened	Date Closed	Type
69006	Bettowynd (Condry)	BOM	-35.7	149.79	165	25.7	33.1			Daily
69018	Moruya Heads Pilot Station	BOM	-35.91	150.15	17	5.9	11.6	1/01/1875	-	Daily
69023	Nelligen (Thule Rd)	BOM	-35.65	150.15	5	12.1	17.5	1/01/1898	-	Daily
69033	Moruya (Burra Creek)	BOM	-35.9	149.96	20	12	18.7	1/01/2001	-	Daily
69035	Bettowynd (Nobbys Hill)	BOM	-35.76	149.82	240	21.1	28.6	1/01/2001	-	Daily
69042	Moruya (The Lagoon)	BOM	-35.77	149.94	70	10.3	17.8	1/01/1960	-	Daily
69048	Upper Deua (Warawitcha)	BOM	-35.76	149.82	166	21.2	28.6	1/01/2011	-	Daily
69052	Batemans Bay - Buckenbowra	BOM	-35.73	150.05	30	4	11.3	1/01/1943	-	Daily
69127	**Araluen Lower (Araluen Rd)**	**BOM**	**-35.69**	**149.84**	**145**	**22**	**29.4**	**1/01/1980**	**-**	**Continuous**
69134	Batemans Bay (Catalina Country Club)	BOM	-35.72	150.19	11	4.4	11	1/01/1985	-	Daily
69142	Moruya (Kiora)	BOM	-35.92	150.04	20	9.3	15.1	1/01/1969	-	Daily
69148	**Moruya Airport AWS**	**BOM**	**-35.9**	**150.14**	**4**	**4.8**	**10.4**	**1/01/1999**	**-**	**Continuous**
D	**Deep Creek Dam**	**ESC**	**-35.76**	**150.18**	**44**	**1.3**	**6.4**	**3/12/1996**	**-**	**Continuous**

2.6.1. Analysis of Pluviometer Data

Continuous pluviometer stations provide a more detailed description of temporal variations in rainfall. As shown in Table 5, three continuous stations are situated close to the Tomaga River and Candlagan Creek catchments. The Moruya Airport and Araluen Lower pluviometers are operated by the BOM and were established in January 1999 and January 1980 respectively while the Deep Creek Dam tipping bucket is operated by Eurobodalla Shire Council and was established in December 1996. Table 6 summarises the largest events on record for the three respective pluviometers. The highest rainfall total over 24 hours of any gauge was recorded at Moruya Airport for the 15/02/10 event. The same event ranked sixth on the Deep Creek Dam gauge which is located north of the catchment (while Moruya Airport Gauge is located south). Araluen gauge is located to west of the catchment but it is substantially further away from the study area than the other two gauges and is on the other side of the low mountain range delimiting the Tomaga River/Candlagan Creek Catchments.

Table 6: Maximum Recorded Storm Depths at Pluviometers (in mm)

Moruya (69148)		
	Start of Event	24 hr Rainfall (mm)
1	**15/02/2010 2:30**	**193.4**
2	13/04/2002 19:30	141.97
3	20/10/2004 6:00	121.6
4	11/11/2013 6:00	120.2
5	6/02/2002 16:00	118.2
6	16/08/2014 19:00	117
7	5/08/2008 9:30	114.6
8	17/01/2001 19:00	110
9	26/08/2001 19:00	107.6
10	30/10/2005 14:30	101.8

Deep Creek Dam (D)		
	Start of Event	24 hr Rainfall (mm)
1	30/10/2005 16:00	176
2	21/10/2004 2:00	166
3	24/10/1999 4:00	162
4	28/01/1999 5:00	150.5
5	17/08/2014 7:00	145.5
6	**14/02/2010 22:00**	**133**
7	10/11/2012 1:00	133
8	30/10/2005 22:00	132
9	18/08/1998 2:00	118
10	8/07/1998 1:00	114.5

Araluen (69127)		
	Start of Event	24 hr Rainfall (mm)
1	29/04/1988 14:00	167.57
2	5/07/1988 9:00	164.43
3	27/06/1997 9:30	163.87
4	10/07/1991 9:30	160.55
5	2/04/1981 9:00	160.07
6	16/09/2013 15:00	150.2
7	23/06/2013 14:30	144.6
8	1/08/1990 9:00	128.58
9	23/10/1999 20:00	123.28
10	14/06/2007 22:00	122.6
138	**14/02/2010 22:00**	**33.2**

2.6.2. Analysis of Daily Read Data

An analysis of the daily records for the nearest daily rainfall stations was undertaken to identify and provide some context for past storm events. As per the pluviometer gauges, no daily read gauge is located within the Tomaga River/Candlagan Creek catchments. However as illustrated in Figure 5, a number of gauges are distributed around the catchments' periphery and these are summarised in Table 7. The daily totals from these gauges provide the means by which a total rainfall depth surface can be triangulated across the study area to facilitate a tentative calibration exercise.

Pluviometric information shows that the February 2010 event commenced prior to the 9:00 gauge recording of the 15th of February and contributed to some/most of the rainfall recorded at 9:00 on the 16th of February. The ratio of the total depth recorded by the Moruya pluviometer for the most intense 24 hour period during the event over the sum of the two reading recorded by the Moruya daily read gauge was applied to each respective gauge. This normalised the total rainfall for each gauge while taking account of the fact that the event occurred on either side of the 9:00am reading time.

Table 7: Rainfall Stations Proximate the Tomaga River/Candlagan Catchments used to Derive Rainfall Depth for 15/02/2010 Event

Station Details		**Daily Read Depth (mm at 9:00)**		**Cumulative Depth**	**Normalised Depth for Event**
Station Number	Station Name	15/02/2010	16/02/2010	15/02/2010 + 16/02/2010	24hr Total
	Moruya Pluviometer*	67.2	149.8		
69018	Moruya Heads Pilot Station	78.3	158	236.3	193.40
69023	Nelligen (Thule Rd)	80.2	53.2	133.4	109.18
69033	Moruya (Burra Creek)	56	57	113	92.48
69042	Moruya (The Lagoon)	80.4	43.4	123.8	101.32
69052	Batemans Bay - Buckenbowra	70.4	53	123.4	101.00
69134	Batemans Bay (Catalina Country Club)	92.6	64.6	157.2	128.66
69142	Moruya (Kiora)	84	124	208	170.24
69148	Moruya Airport AWS	69	153	222	181.70
***Note Moruya Pluviometer shows less than Moruya Heads Pilot Station because the totals only include the 24 hr event rainfall and not rainfall that occurred that day but outside of event**					

There was insufficient pluviometer data for the 1934, 1974 and 1991 events (for which anecdotal evidence was provided and discussed in Section 3.3) and so these events were not modelled for calibration purposes. However, to provide some context for these events, the daily read data was analysed for the largest daily total for the years specified at the Moruya Heads Pilot Station (69018). From this, an approximate ARI was calculated from the design rainfall intensity-frequency-duration (IFD) data corresponding to each daily read gauge location.

For the 1934 event, two gauges were in operation and the data is shown in Table 8. However, the Nelligen gauge (69023) had a gap in the data available spanning December 1933 and January 1934. Of the 1934 data available at the Nelligen gauge, the largest daily total was approximately half the largest daily total recorded at Moruya Heads Pilot Station in 1934, and was therefore not analysed.

Table 8: Rainfall Depth and Approximate ARI for the 8th January 1934

Station Number	Station Name	Daily Read Depth (mm at 9:00)	Approximate ARI
69018	Moruya Heads Pilot Station	206.8	10 – 20 year ARI event
69023	Nelligen (Thule Rd)	N/A	N/A
***Note: the Nelligen gauge (69023) had a gap in the data available, spanning December 1933 and January 1934.**			

Four gauges were in operation in 1974 and the data is shown in Table 9. However, the Nelligen gauge (69023) had a gap in the data available spanning 1966 through to 1999. By comparison, the Moruya gauge (69042) had no gap in data for that year, however recorded a value of 0 mm of rainfall on the 20th April 1974.

Table 9: Rainfall Depth and Approximate ARI for the 20th April 1974

Station Number	Station Name	Daily Read Depth (mm at 9:00)	Approximate ARI
69018	Moruya Heads Pilot Station	115.4	1 – 2 year ARI event
69023	Nelligen (Thule Rd)	N/A	N/A
69042	Moruya (The Lagoon)	0	N/A
69052	Batemans Bay - Buckenbowra	136.6	1 – 2 year ARI event
***Note the Nelligen gauge (69023) had a gap in the data available, spanning 1966 to 1999.**			

For the 1991 event, five gauges were in operation and the data is shown in Table 10. Similar to the 1974 event, the Nelligen gauge (69023) had a gap in the data available. The Batemans Bay gauge (69134) also had a gap in the data available spanning 1987 to October 1991.

Table 10: Rainfall Depth and Approximate ARI for the 9th June 1991

Station Number	Station Name	Daily Read Depth (mm at 9:00)	Approximate ARI
69018	Moruya Heads Pilot Station	131.4	2 – 5 year ARI event
69023	Nelligen (Thule Rd)	N/A	N/A
69042	Moruya (The Lagoon)	190.0	5 – 10 year ARI event
69052	Batemans Bay - Buckenbowra	247.0	10 – 20 year ARI event
69134	Batemans Bay (Catalina Country Club)	N/A	N/A
***Note the Nelligen gauge (69023) had a gap in the data available, spanning 1966 to 1999. The Batemans Bay gauge (69134) had a gap in the data available, spanning 1987 to October 1991.**			

2.7. Design Rainfall Data

The design rainfall intensity-frequency-duration (IFD) data, for events up to and including the 1% AEP event, were obtained from the Bureau of Meteorology's online design rainfall tool. The input parameters for these calculations were sourced from AR&R (1987).

Table 11: Rainfall IFD Data at the George Bass Drive Water Level Gauge (216455)

DURATION	Design Rainfall Intensity (mm/hr)						
	1 yr ARI	2 yr ARI	5 yr ARI	10 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI
5Mins	93.1	121	157	179	208	247	277
6Mins	87.2	113	148	168	195	232	260
10Mins	71.5	93	122	140	163	195	219
20Mins	52.4	68.5	91.7	106	124	149	169
30Mins	42.7	56	75.6	87.7	103	125	142
1Hr	29	38.2	52	60.7	71.9	87.2	99.2
2Hrs	19.1	25.2	34.3	40	47.3	57.4	65.3
3Hrs	14.9	19.5	26.5	30.8	36.5	44.1	50.1
6Hrs	9.62	12.6	17	19.6	23.1	27.9	31.6
12Hrs	6.23	8.16	10.9	12.6	14.9	17.9	20.2
24Hrs	4.02	5.28	7.13	8.27	9.77	11.8	13.4
48Hrs	2.52	3.33	4.58	5.36	6.38	7.76	8.86
72Hrs	1.87	2.47	3.42	4.01	4.78	5.83	6.68

2.8. Previous Reports

Little historical flooding has been reported in Tomakin, Mossy Point and Broulee. To date, studies have focused on Mogo which has experienced a number of flood events. Development pressures in Tomakin, Mossy Point and Broulee as well as elsewhere in the Tomaga River catchment (specifically along Dunns Creek Road) provides the impetus for the catchment-wide flood study.

2.8.1. Report on Mogo Flood Study

Residential, commercial and light industrial development in Mogo accelerated in the mid-1980's creating pressure to develop potentially flood prone land adjacent Cabbage Tree Creek. Consequently the Mogo Flood Study (Reference 1) was commissioned by Eurobodalla Shire Council to clarify the existing flood affection of Mogo. The study established the flood extent and levels of the 20% AEP, 5% AEP and 1% AEP events within and in the vicinity of the town of Mogo.

2.8.2. Mogo Floodplain Management Study

Subsequent to the completion of the Mogo Flood Study, the next phase of the Floodplain management process was undertaken comprising of the Mogo Floodplain Management Study (Reference 2). Both structural and non-structural measures to reduce the flood risk were considered. The study considered filling and channel upgrade to be the two most appropriate structural measures. However preference toward non-structural measures such as zoning and development controls which do not permit new building on land affected by flooding was expressed and no structural measure has been actioned.

2.8.3. Mogo Commercial Area Drainage Study

The Mogo Commercial Area Drainage Study (Reference 3) identified the preferred works for formalising a depression drain located on the east side of the Princes Highway, north of Tomakin Road.

The depression drain with a catchment area of 41.4 hectares discharges into Cabbage Tree Creek via two existing 0.9 m diameter culverts across the Princes Highway immediately north of Tomakin Road. The assessment considered the 1 year, 5 year, 20 year and 100 year ARI events and design flood flows were computed using the RAFTS-XP rainfall-runoff model. For the 100 year event, peak local flows of 14.3 m^3/s and peak Cabbage Tree Creek of 195.7 m^3/s were obtained.

The EXTRAN-XP hydraulic model was used to assess a number of pipe arrangement options to mitigate peak flood levels consisting of:

- A 1.35 m diameter pipe capable of discharging the 5 year ARI event into Cabbage Tree Creek;

- 0.45 m low flow pipe located along the centreline of the 10 m drainage easement and discharging into one of the existing 0.9 m pipes; and
- A trapezoidal shaped grassed open drain within the 10 m easement capable of discharging the 20 year ARI event.

While the proposed measures assist in alleviating flooding from the local catchment, the ability of a large event from Cabbage Tree Creek to backwater through the enhanced drainage system as well as the influence of an elevated sea level requires further consideration. The above works require further consideration and have not been actioned.

3. COMMUNITY CONSULTATION

3.1. Online Media

Following approval by the state government for a grant to assist in funding the flood study, the Bay Post – Moruya Examiner published details of the project advising the community that their input would be desired and that community consultation as well as a public exhibition period would be part of the study.

The article is available online at http://www.batemansbaypost.com.au/story/2148834/tomago-river-flood-study-funded/ and similar notice was provided on the Eurobodalla Shire Council website.

3.2. Community Questionnaire and Information Sheet

In collaboration with Eurobodalla Shire Council, a questionnaire and information sheet were distributed to residents and business owners within the study areas. The information sheet described the Floodplain Risk Management Process and provided information on the current flood study. The questionnaire requested information on flooding that residents and business operators may hold. This could be based upon photographs or observations of previous floods. Both the questionnaire and the information sheet directed the community to an online questionnaire (on the Survey Monkey platform), should they wish to complete the questionnaire via an alternative method. The information sheet also informed the community of a drop-in session held on the 15th of April 2015 (see Section 3.3).

The community questionnaire and information sheet that were distributed by Eurobodalla Shire Council can be found in Appendix B.

3.3. Drop-in Session

Eurobodalla Shire Council and WMAwater organised a drop-in session that was held at the Tomakin Community Hall between 4:00pm and 7:00pm on the 15th of April 2015. Present were representatives from Eurobodalla Shire Council, OEH and WMAwater as well as the wider social network. The community was informed of this meeting via the community information sheet.

The community could attend on an individual basis at any time that was convenient for them during the hours that representatives were present. The objective of this being that attendance would not be unreasonably hindered by restrictive hours that would have been the case in a collective meeting rather than individualised (“drop-in”) meetings.

The drop-in session proved to be popular with over 30 attendees being present including two previous shire engineers, members of the Mossy Point Association as well as members of the Mogo Business Association.

Anecdotal evidence suggests that the largest event to take place in living memory was in 1934. Other significant events took place in 1974 and 1991. Subsequent to the 1974 event, the shire engineer (present at the drop-in session) marked telegraph poles on Elizabeth Drive approximately 0.3 metre above the peak flood level for the event and these are shown in Figure 6. Reports of more recent flooding were used to verify flood extents as part of the hydraulic model calibration.

These reports were characterised by shallow overland flow runoff for the majority of the catchment with the exception of Mogo where more significant and regular flooding was documented. Consequently, a further meeting was scheduled where WMAwater engineers met with Mogo Business owners and were shown local landmarks that have been historically flood affected.

3.4. Consultation – Public Exhibition

Eurobodalla Shire Council carried out the public exhibition of the Tomakin, Mossy Point, Broulee and Mogo Flood Study over the period of the 23rd May to the 24th June 2016. The public exhibition period was communicated to the community via the Council website, a media release in the local newspaper, and a community information newsletter posted to residents and business owners in the area. Community information sessions were also held at the Tomakin Community Hall on the 7th June 2016 from 6:00pm to 8:00pm and on the 8th June 2016 from 11:00am to 2:00pm. These community information sessions were attended by Council and WMAwater representatives.

The owners of 50 properties attended the community information sessions, four phone enquiries were received by Council and two written submissions were received during the public exhibition period.

One submission was concerned with erosion risk for beaches in the study area and expressed the desire for the coastal risk and possible mitigation strategies to be investigated within this or related studies. Although coastal erosion is not investigated within the Flood Study, the community interest in this issue was duly noted.

The other submission was concerned with the Mogo commercial area, the impact upon Development Applications (DA) and a desire for mitigation options to be investigated. The second submission was also interested in the differences between the 1987 Mogo Flood Study (discussed in Section 2.8.1) and the current Flood Study. This prompted additional creek invert level survey to be collected and further investigated, as discussed in Section 2.1.3).

The Flood Study is the first stage of the floodplain risk management process and aims to determine the extent of flooding within the study area. The next stage of the process is the Floodplain Risk Management Study (FRMS), whereby possible mitigation options are investigated.

4. STUDY METHODOLOGY

The estimation of flood behaviour in a catchment is often conducted as a two-stage process, consisting of:

1. hydrologic modelling to convert rainfall estimates to overland flow and stream runoff; and
2. hydraulic modelling to estimate flow distributions, flood levels and velocities.

When historical flood data are available they can be used to allow calibration of the models, and increase confidence in the estimates. The calibration process is undertaken by altering model input parameters to improve the reproduction of observed catchment flooding. Recorded rainfall and stream-flow data are required for calibration of the hydrologic model, while historic records of flood levels, velocities and inundation extents can be used for the calibration of hydraulic model parameters.

Following model calibration the design rainfall is modelled. The approach adopted in flood studies to determine design flood levels largely depends upon the objectives of the study and the quantity and quality of the data (survey, flood, rainfall, flow etc.).

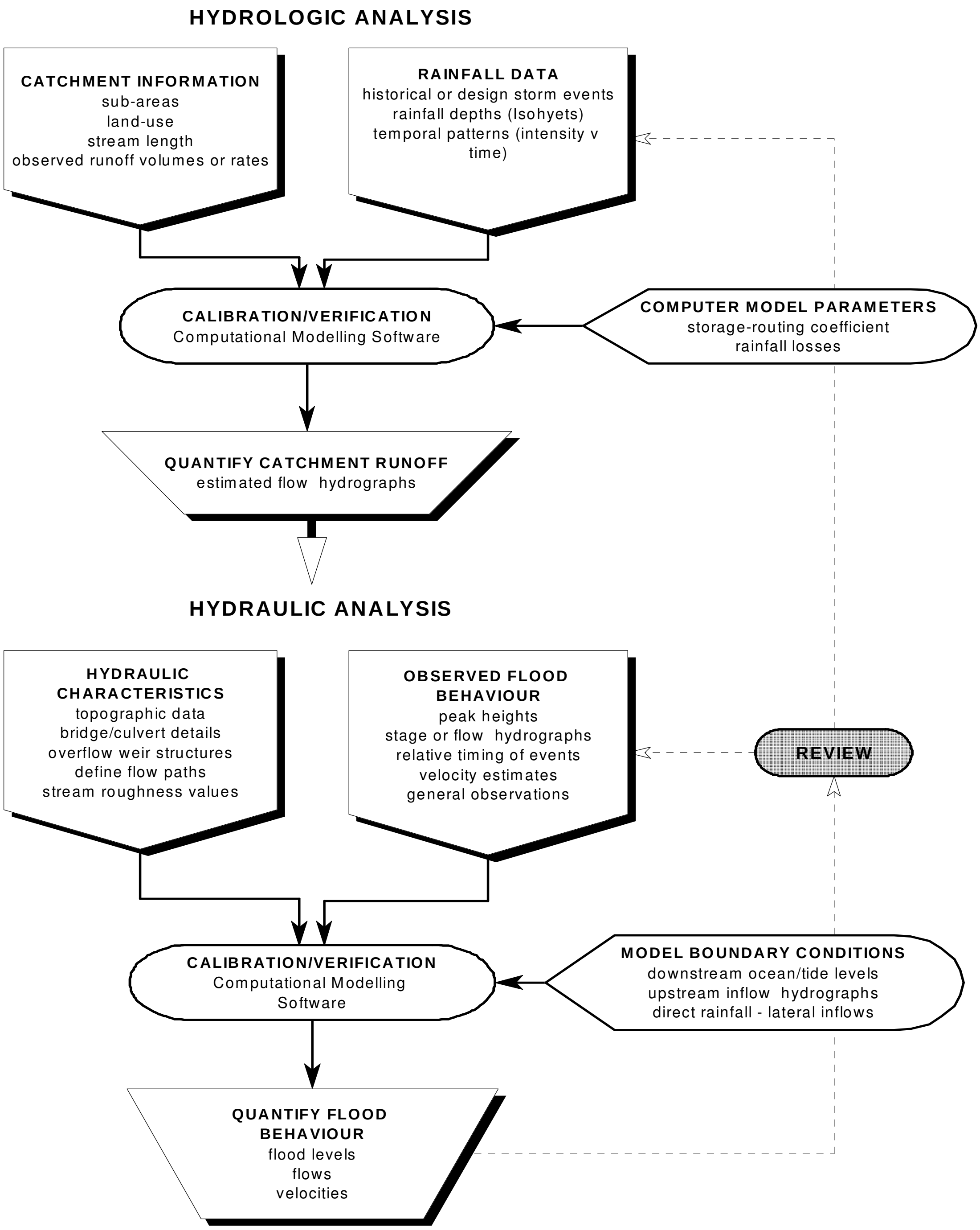
Flood estimation in urban catchments generally presents challenges for the integration of the hydrologic and hydraulic modelling approaches, which have been treated as two distinct tasks as part of traditional flood modelling methodologies. As the main output of a hydrologic model is the flow at the outlet of a catchment or sub-catchment, it is generally used to estimate inflows from catchment areas upstream of an area of interest. The hydrological model can also be useful to conceptually model hydrologic processes within the study area (such as runoff from roof and gutter systems, and On-site Stormwater Detention (OSD) systems). The aim of identifying the full extent of flood inundation can therefore be complicated by the separation of hydrologic and hydraulic processes into separate models, and these processes are increasingly being combined in a joint modelling approach.

The broad approach adopted for this study was to use a widely utilised and well-regarded hydrologic model to conceptually model the rainfall concentration phase, and for steep catchment areas upstream of the hydraulic model study area. The runoff hydrographs from the hydrologic model were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area. This joint modelling approach was verified against flooding reported by the community and flow estimates from the Regional Flood Frequency Estimate method.

This approach reflects current engineering best practice and is consistent with the quality and quantity of available data.

A diagrammatic representation of the Flood Study process is shown in Diagram 2.

Diagram 2: Flood Study Process



5. HYDROLOGIC MODEL DEVELOPMENT

5.1. Introduction

AR&R (1987) describes various techniques suitable for design flood estimation in rural and urban catchments. These techniques range from simple procedures to estimate peak flows (such as the Probabilistic Rational Method), to flood frequency analysis and more complex rainfall-runoff routing models that estimate complete flow hydrographs. Determination of which technique to employ is often based on the availability of data. For the present study, the rainfall and runoff routing approach was adopted. In current Australian engineering practice, examples of the more commonly used runoff routing models include RORB, RAFTS and WBNM. These models allow the rainfall depth to vary both spatially and temporally over the catchment, and have parameters governing runoff volume/shape that can be calibrated against recorded data.

For the present study, the Watershed Bounded Network Model (WBNM) was used. The WBNM model is an event-based, lumped-catchment conceptual model that is based on an extensive empirical dataset of rainfall-runoff relationships for Australian catchments. The model requires very few parameters to describe the physical aspects of the catchment, and is therefore less sensitive than other models to assumptions about catchment characteristics such as shape, steepness, and ground cover. WBNM was therefore considered a suitable tool for this study. WBNM has been widely adopted in Australia for use in similar studies.

5.2. Sub-catchment Delineation

The catchment boundary was determined by the ridges that create the natural drainage division. Precipitation falling on the other side of these boundaries would flow into other catchments and so was not modelled within these study areas.

Within the Tomaga River and Candlagan Creek catchments, smaller sub-catchment areas were delineated based on LiDAR survey and contours where LiDAR survey was not available. The sub-catchment layout ensures that where hydraulic controls exist that these are accounted for and able to be appropriately incorporated into hydraulic routing. The catchment layout for the hydrologic model is shown on Figure 7.

5.3. Model Parameters

The WBNM hydrologic runoff-routing model was used to determine hydraulic model inflows, both from catchment areas upstream of the hydraulic model extent, and for the local sub-catchments within the hydraulic model domain of the study.

The model input parameters for each sub-catchment are:

- a lag factor (termed C), which can be used to accelerate or delay the runoff response to rainfall;
- a stream-flow routing factor, which can speed up or slow down concentrated flows

occurring through each catchment;

- rainfall initial and continuing losses to represent infiltration and filling of depression storage; and
- the percentage of catchment area with a pervious/impervious surface.

5.3.1. Lag Parameter

Lag times for runoff depend on several physical catchment characteristics, including area, shape and steepness (among others) for natural catchments. Experimental data for natural catchments in Australia has demonstrated that the dominant factor affecting lag is catchment area, with other characteristics showing strong correlation with area such that there is a strong case for catchment lag to be determined on area alone.

Experimental derivation of the Lag Parameter for 129 storms on 10 catchments in eastern NSW found that a value of 1.68 gave a good fit to all the data. A value of 1.7 was adopted for historical and design flood modelling in this study, in agreement with the NSW data and the value adopted in the nearby catchments from the Wagonga Inlet, Kianga and Dalmeny Flood Study (2016).

5.3.2. Stream-flow Routing Parameter

WBNM provides the option to route upstream flows to the bottom of a sub-catchment via nonlinear routing, time-delay routing and Muskingum routing. This routing is required to estimate the attenuation and timing of flows from sub-catchments in the steep upper catchment areas that are not included in the hydraulic model extent. The nonlinear method was adopted for this study. For this method, Boyd et. al. (2007) recommends values of 1.0 for natural channels and 0.67 for gravel beds. Therefore, for this study, a value of 1.0 was adopted.

Where the hydrologic sub-catchment area coincided with the hydraulic sub-catchment area, these were applied as local inflows (the location of which are sown in Figure 7) with no routing of upstream flows.

5.3.3. Rainfall Losses

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in AR&R (1987). The methods are of varying complexity, with the more complex options only suitable if sufficient data are available (such as detailed soil properties). The method most typically used for design flood estimation is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur, and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

Initial and continuing losses are often used as the primary parameters for calibrating hydrologic models when observational data are available. For this study, typical values are adopted based on available data in similar nearby catchments. Table 6.2 of ARR (1987) recommends that for catchments east of the dividing range in New South Wales, in the absence of calibration data, an initial loss of 10 mm to 35 mm is appropriate, with a continuing loss of 2.5 mm/hr.

For this study, the initial loss of 20 mm was adopted with a continuing loss of 3.5 mm/hr.

5.3.4. Impervious Areas

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occur significantly faster than from vegetated surfaces. This results in a faster concentration of flow within the downstream area of the catchment, and increased peak flow in some situations. It is therefore necessary to estimate the proportion of the catchment area that is covered by such surfaces.

The impervious surfaces within the study areas were determined through digitisation of the road surfaces (used in the hydraulic model to specify Manning's 'n' roughness coefficients, see Section 6.4) and building footprints (used in the hydraulic model to simulate impermeable obstructions to the flood flow, see Section 6.3) through visual inspection of aerial photography. The discretisation of layers considered impermeable, namely roads and buildings, is shown in Figure 8. The proportion of these impervious surfaces within the sub-catchment area was adopted as the impervious percentage of each respective sub-catchment area.

5.3.5. Summary of Model Parameters

The key modelling parameters adopted for the historic hydrologic modelling are summarised as follows:

- Lag Parameter (C) – 1.7
- Pervious Area Initial Rainfall Loss – 20 mm
- Pervious Area Continuing Rainfall Loss – 3.5 mm/hour
- Impervious Area Initial Rainfall Loss – 1 mm
- Impervious Area Continuing Rainfall Loss – 0 mm/hour

The key modelling parameters adopted for the design hydrologic modelling are summarised as follows:

- Lag Parameter (C) – 1.7
- Pervious Area Initial Rainfall Loss – 20 mm
- Pervious Area Continuing Rainfall Loss – 3.5 mm/hour
- Impervious Area Initial Rainfall Loss – 1 mm
- Impervious Area Continuing Rainfall Loss – 0 mm/hour

6. HYDRAULIC MODEL DEVELOPMENT

6.1. Introduction

The availability of high quality LiDAR data and flow behaviour present means that the study area is suitable for two-dimensional (2D) hydraulic modelling of major flowpaths. Various 2D software packages are available, such as SOBEK, TUFLOW and Mike FLOOD, among others. The TUFLOW package was adopted for this study as it is widely used in Australia and WMAwater have extensive experience in the use of the TUFLOW model.

The TUFLOW modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in two dimensions. The 2D model is capable of dynamically simulating complex overland flow regimes and interactions with sub-surface drainage systems.

For the hydraulic analysis of complex overland flow paths an integrated 1D/2D model such as TUFLOW provides several key advantages when compared to a 1D only model. For example, a 2D approach can:

- provide localised detail of any topographic and /or structural features that may influence flood behaviour,
- better facilitate the identification of the potential overland flow paths and flood problem areas,
- dynamically model the interaction between hydraulic structures such as culverts and complex overland flowpaths, and
- inherently represent the available flood storage within the 2D model geometry.

Importantly, a 2D hydraulic model can better define the spatial variations in flood behaviour across the study area. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped across the model extent. This information can then be easily integrated into a GIS based environment enabling the outcomes to be readily incorporated into Council's planning activities. The model developed for the present study provides a flexible modelling platform to properly assess the impacts of any management strategies within the floodplain (as part of the ongoing floodplain management process).

In TUFLOW the ground topography is represented as a uniformly-spaced grid with a ground elevation and a Manning's "n" roughness value assigned to each grid cell. The grid cell size is determined as a balance between the model definition required and the computer run time (which is largely determined by the total number of grid cells).

6.2. Model Extent

The Tomaga River and Candlagan Creek catchments are largely rural and development is concentrated around four areas, namely Tomakin, Mossy Point, Broulee and Mogo. Typically, developed areas require a grid resolution of no more than 3 metres to capture the various flow mechanisms characteristic of a built-up environment. However, such a grid resolution over the 120 km^2 covered by the Tomaga River and Candlagan Creek would result in excessive runtime and while splitting the two catchments into separate models would reduce run-times, the proximity of the respective river mouths makes this difficult as flood levels from both catchment can be inter-dependent. A more elegant solution that is particularly suited to the study area is to take a nested approach to the hydraulic modelling. The upper parts of the catchment which are outside the study area are routed in the hydrologic model and applied as boundary inflows to the hydraulic model domain. The overall hydraulic model extent is shown on Figure 9 and has a 10 m x 10 m grid resolution which is refined to a resolution of 2.5 m x 2.5 m for the areas where development is concentrated.

6.3. Digital Elevation Model

The model grid was established by sampling from a 2 m x 2 m DEM. This DEM was generated from a triangulation of filtered ground points from the 2012 LiDAR dataset discussed in Section 2.1.1 and bathymetric survey discussed Section 2.1.2 was used to cut out the Tomaga River channel geometry. Figure 2 presents the two respective datasets as well as the cross-sectional information derived from bridge construction drawings implemented in the cutting out Candlagan Creek.

Permanent buildings and other significant structures likely to act as significant flow obstructions were incorporated into the terrain model. These features were identified from the available aerial photography and modelled as impermeable obstructions to the flood flow.

6.4. Roughness Coefficient

The TUFLOW model used for this study utilises the Manning's formulation to determine the energy loss from friction and other sources. The roughness coefficient, '*n'*, is an empirically derived parameter which represents the retarding force applied to flowing water by the channel bed or ground surface. In the computational modelling of real systems, this parameter often also incorporates other sources of energy loss such as turbulence and flow expansion/contraction from non-uniform cross sections.

The value of '*n'* represents the resistance to flow in a given channel which depends on a number of factors such as:

- surface roughness;
- vegetation;
- channel irregularity and alignment;
- obstructions;
- silting and scouring;
- the size and shape of the channel; and
- the stage and discharge.

Inspection of the aerial photography was used to classify various land-uses categories, such as urban areas and vegetated areas. From this, spatially varying roughness values were applied to the model, based upon these differing categories. The roughness values adopted for the hydraulic model are shown in Table 12 and Figure 8.

The values are consistent with typical values in the literature (Chow, 1959 and Henderson, 1966), industry guidelines (*AR&R Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, Engineers Australia, 2012) and previous experience with modelling similar catchment conditions. The sensitivity of model results to changes in the roughness values is discussed in Section 9.4.

Table 12: Manning's 'n' Values

Surface Type	Manning's 'n' Value
Concrete-lined pipes	0.015
Roads and paved surfaces	0.02
Waterways – Rivers, Lakes, Estuaries and Ocean	0.03
Dirt areas	0.03
Light density vegetation (very short grass or sparse vegetation)	0.04
Medium density vegetation	0.07
Heavy density vegetation	0.12
Swamp areas	0.06
Default	0.08

6.5. Hydraulic Structures

The behaviour of hydraulic structures like culverts, fences, channels and bridges can have a significant influence on flood behaviour. When culverts are flowing near capacity or become blocked, backwater upstream of the culvert can flood properties or cause the road to be overtopped. The piers and deck of bridges over creeks can present an obstruction to flow, resulting in afflux (increased water level) upstream of the structure. It is therefore important to pay particular attention to the modelling of these features.

Key hydraulic structures were included in the hydraulic model, as shown on Figure 4. Culverts were generally modelled as 1D features embedded in the 2D model, since the majority of the culverts of interest have dimensions smaller than the grid resolution. For the bridges, where the main flow width exceeds the grid resolution, modelling was undertaken in the 2D domain using a TUFLOW software feature specifically designed for this purpose, whereby the energy losses and blockage caused by the piers, deck and above deck structure can be applied directly to the grid cells.

The modelling parameter values for the culverts and bridges were based on the geometrical properties of the structures, which were obtained from records of structures held by the authorities responsible for them, photographs taken during site inspections (Figure 3), and previous experience modelling similar structures. The Roads and Maritime Services provided data on the dimensions of structures underneath the Princes Highway. Eurobodalla Shire Council provided data on the location of drainage structures within their jurisdiction and where details of dimensions were not available, larger asset dimensions were estimated during site inspection and less critical assets were assumed blocked. Sensitivity analysis of the effect of the hydraulic structure parameters is presented in Section 9.4.

Smaller localised obstructions within private property, such as fences, were not explicitly represented within the hydraulic model, due to the difficulty of identifying and characterising these structures from aerial photographs, and the relative impermanence of these features. The cumulative effect of fences on flow behaviour is implicitly contained within the roughness parameter discussed in Section 6.4.

6.6. Blockage Assumptions

Blockage of hydraulic structures can occur with the transportation of a number of materials by flood waters. This includes vegetation, garbage bins, building materials, cars, and even houses in extreme cases as witnessed during the recent flooding of Dungog in April 2015. However, the disparity in materials that may be mobilised within a catchment can vary greatly.

Debris availability and mobility can be influenced by factors such as channel shear stress, height of floodwaters, severity of winds, storm duration and seasonal factors relating to vegetation. The channel shear stress and height of floodwaters that influence the initial dislodgment of blockage materials are also related to the average exceedance probability (AEP) of the event. Storm duration is another influencing factor, with the mobilisation of blockage materials generally increasing with increasing storm duration (Barthelmess and Rigby 2009, cited in Engineers Australia 2013).

The potential effects of blockage include:

- decreased conveyance of flood waters through the blocked hydraulic structure or drainage system;
- variation in peak flood levels;
- variation in flood extent due to flows diverting into adjoining flow paths; and
- overtopping of hydraulic structures.

Existing practices and guidance on the application of blockage can be found in:

- the Queensland Urban Drainage Manual (Department of Natural Resources and Water, 2008);
- AR&R Revision Project 11 Blockage of Hydraulic Structures (Engineers Australia, 2013); and
- the policies of various local authorities and infrastructure agencies.

Current modelling has been undertaken assuming 25 percent blockage of pipes and culverts greater than or equal to 450 mm in diameter. Pipes less than 450 mm in diameter are conservatively assumed to be completely blocked. The sensitivity of model results to changes in the blockage assumptions is discussed in Section 9.4.

It is worth noting that for large structures upstream of Tomakin, Mossy Point, Broulee and Mogo, assuming blockage while increasing peak levels upstream of the structure, may potentially decrease peak flood levels in the areas of interest for this study. Therefore, assuming blockage is not always inherently conservative. As a result no blockage was assumed for large bridges.

7. HISTORIC FLOOD MODELLING

7.1. Introduction

Modelling of known historic flood events is carried out to calibrate and validate the hydrologic and hydraulic models. This process is important to ensure that the models are sufficiently representing flood behaviour within acceptable limits. Calibration involves modifying (within an acceptable range) the model parameter values to replicate observed flood behaviour or levels. Validation is undertaken to ensure that the model parameter values determined in the calibration phase are acceptable in other flood events with no need for additional alteration of values.

The model parameters that are typically adjusted include (as detailed within the *ARR Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, 2012):

- Hydraulic roughness parameters;
- Energy losses at structures/bends;
- Inflow hydrographs (parameters involved include temporal rainfall patterns and spatial rainfall distribution);
- Downstream boundary location and assumptions, particularly stage-discharge boundaries; and
- Blockage of inlets and hydraulic structures.

Selection of calibration and validation events is based upon data availability and magnitude of the storm or flood event. Ideally, the rainfall calibration events span a range of magnitudes with a preference for the more significant events, such as those near the 1% AEP event.

It is ideal to have historical rainfall (daily and pluviographic) and historical streamflow (daily and instantaneous) data to calibrate the hydrologic model, independent of the hydraulic model. As streamflow data is not available within the study areas, the hydrologic model has been calibrated in tandem with the hydraulic model in this flood study. This is in accordance with guidelines produced by Engineers Australia (within the *AR&R Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, 2012) that recommends that the two models be jointly calibrated.

To calibrate the hydrologic and hydraulic models it is necessary to have data on historical rainfall, historical boundary conditions and historical flood records or observations.

The historic rainfall conditions can be determined from daily and pluviometer gauging stations. The pluviometer data provides information on the temporal pattern of the rainfall (as in, the variation in the rainfall amount across a period of time). The combination of the daily and pluviometer data provides information on the possible spatial distribution of the rainfall (as in, the variation in the rainfall depth across the catchment area). Generally, historic boundary conditions may be a stage-discharge relationship or tidal data for catchments discharging into ocean-influenced waterways.

Historic records or observations that can be used to define historical flood behaviour, and thereby calibrate the model against, include:

- *Rain Gauges*: pluviometers provide rainfall intensities which permit modelling of the rainfall pattern for the event while daily rain gauges assist in mapping total rainfall over the event across the subcatchments;
- *Continuous Water Level Recorders*: gauges that record the complete hydrograph enable calibration of not just the peak flood level but also the timing of the rise and fall of the flood;
- *Maximum Height Gauges*: gauges that record the peak flood level reached during a specific event;
- *Peak Level Records*: markers placed (usually by government agencies) after the event to indicate the peak flood level or maximum flood extent reached;
- *Debris Marks*: where floating debris remains on an object from the receding flood waters, resulting in a line indicating the flood level reached;
- *Watermarks on Structures*: residual watermarks on structures can indicate the flood level reached; and
- *Anecdotal Information*: descriptions of flood levels or behaviour, as well as photographs or videos.

The study area contains very little of the abovementioned data recorders:

- No pluviometers are located within the Tomaga River or Candlagan Creek Catchments;
- No daily rain gauges are located within the Tomaga River or Candlagan Creek Catchments;
- The closest ocean tide station is Ulladulla Harbour located 70 km away;
- The continuous water level recorder located on the Tomaga River is tidally affected;
- No large events have occurred in recent years;
- Community consultation feedback mostly anecdotal.

The paucity of information does not facilitate an extensive calibration and validation exercise. Nevertheless, the study considers a tentative calibration based on the little data available within the catchment and supplemented with data from neighbouring catchments. Adjustment and checks of modelling inputs undertaken include:

- Tidal event hydraulic model calibration;
- Rainfall event hydraulic model calibration;
- Validation of modelled flows against those obtained using Regional Flood Frequency Estimates; and
- Verification of modelled flood extents against flood affected areas reported by the community.

7.2. Tidal Calibration

7.2.1. Description

In addition to rainfall-derived calibration events, it is recommended that tidal calibration be undertaken in catchments where the interaction between the tidal inundation and the rainfall runoff is important, as is the case in the catchments investigated in this flood study. Tidal calibration ensures that the model can reproduce tidal amplification and isolate the mechanisms that may be responsible for variations in the modelled and recorded hydrographs.

Tidal calibration is of particular significance for this study as the majority of the areas of interest are within the tidal affectation zone. Tomakin, Mossy Point and Broulee are situated on the coast and the tidal signature can be observed as far up as Mogo.

7.2.2. Methodology

Tidal calibration is undertaken by modelling the recorded hydrograph produced by a tide level gauge during a period with no recorded precipitation. The resulting hydraulic model hydrograph is compared against the recorded hydrograph produced by a continuous water level gauge.

The tide level recorded at Jervis Bay was applied to the downstream boundary of the hydraulic model, thereby prompting the filling and emptying of the model domain within the tidally affected zone. The George Bass Drive continuous water level recorder is 3 km upstream of the applied tailwater boundary and within the tidal zone.

Two tidal calibration events were selected, where gauges around the Tomaga River and Candlagan Creek catchments recorded no precipitation. The 3 day period between the 6th and 8th of May 2012 was selected due to the chronological proximity to the rainfall calibration event and the current catchment conditions. The 3 day period between the 22nd and 25th of December 1998 was selected due to the correlation with the bathymetric survey period (Section 2.1.2).

7.2.3. Calibration Results

Results for the tidal calibration of the hydraulic model for the 1998 and 2012 events are shown in Figure 10 and Figure 11. The timing of the tidal oscillations was replicated by the model across both events. The 1998 event displayed a stronger correlation between the hydraulic model behaviour and the recorded hydrograph; in terms of both the amplitude and the rate of the rise and fall. The hydraulic model results for 2012 event exhibited slightly higher water levels at the crests and lower water levels at the troughs. Both the ascending and descending limbs displayed some acceleration, with the latter somewhat more than the former. As there is a period of time separating the 2012 event from the 1998 bathymetric survey, the variation in hydraulic model behaviour could be attributed to variations in the morphology of the waterway (i.e. scouring).

Generally, from observation of the tidal levels comparative to the water levels recorded at

George Bass Drive, it can be seen that tidal flows are subject to attenuation such that:

- the water level crest at George Bass Drive is lower than the tidal crest and occurs on the descending limb of the tide; and
- the water level trough at George Bass Drive is higher than the tidal trough and occurs on the ascending limb of the tide.

7.3. Rainfall Calibration – February 2010 Event

7.3.1. Description

As discussed in Section 2.6.1, three continuous stations are situated close to the Tomaga River and Candlagan Creek catchments. The Moruya Airport and Araluen Lower pluviometers were established in January 1999 and January 1980 respectively while the Deep Creek Dam tipping bucket was established in December 1996. The highest rainfall total over 24 hours of any gauge was recorded at Moruya Airport on February 2010. The same event ranked sixth on the Deep Creek Dam gauge which is located north of the catchment (while Moruya Airport Gauge is located south). Araluen gauge is located to west of the catchment but it is substantially further away from the study area than the other two gauges and is on the other side of the low mountain range delimiting the Tomaga River/Candlagan Creek Catchments and the event ranked 138th.

While larger events have occurred in the past (Section 3.3), temporal patterns are required to undertake a thorough calibration exercise; consequently only events that occurred subsequent to the installation of pluviometers can be considered. Furthermore, the George Bass Drive water level gauge, which is the only source of recorded water levels within the catchment, was opened in 1998. Consequently the February 2010 event presents the only viable option for a potential rainfall calibration without widespread assumptions.

7.3.2. Methodology

Analysis of pluviometer data, the rainfall distribution derived from rainfall gauges proximate the catchments (no rainfall gauges were located within), and radar data originating from the Canberra (Captains Flat) radar station permits confirmation of the February 2010 storm behaviour.

The 2010 storm event occurred over a 24 hour period approximately; straddling two days of daily read rainfall data (hence the rainfall distribution is derived from the 48 hour period prior to 9am on the 16th February 2010). Linear triangulation of the total depths for the 48 hour period creates an interpolated surface from which total rainfall depths can be interrogated for each respective subcatchment shown in Figure 12. Applying the most relevant temporal pattern to each subcatchment and running the hydrologic model provides the flow hydrographs for each subcatchment which is then applied as an inflow into the hydraulic model as shown in Figure 9.

Tidal levels recorded at the Jervis Bay Gauge (216470) during the event are applied as the downstream tailwater level for the hydraulic model. Similarly, initial water level in both the 1D and 2D hydraulic model domain is taken as the recorded level at Jervis Bay at the time the event simulation is commenced.

7.3.3. Calibration Results

From observation of the tidal levels comparative to the water levels recorded at George Bass Drive, it can be seen that during the 2010 event:

- Runoff began arriving at George Bass Drive between 17:00 hrs and 20:00 hrs on the 14th February. This was determined from the crest of the water level occurring at a similar height to the crest of the tide level (rather than the water level recording a lower height to the tide level, discussed in Section 7.2.3). This runoff was found to originate from the rainfall that occurred prior to 15:00 hrs on the 14th February.
- Runoff arriving at George Bass Drive peaked during the low tide around 15:00 hrs on the 15th February. This resulted in a plateau of the water levels recorded at George Bass Drive and the largest difference between the tide level and the water level at George Bass Drive.
- Runoff continued to arrive at George Bass Drive until after 00:00 hrs on the 17th February. This was again determined from the crest of the water level occurring at a similar height to the crest of the tide level.

It should be noted that rainfall that descends on the upper areas of the catchment will result in runoff at a downstream location (such as George Bass Drive) several hours after the rainfall occurred due to the flow needing to traverse the distance between the upper and lower areas of the catchment.

Results for the rainfall calibration of the hydraulic model against the February 2010 event are shown in Figure 13. The peak water level modelled was found to be within 0.01 m of the peak water level recorded at George Bass Drive. However, the modelled results displayed a somewhat earlier peak than what was recorded. The water level peak that was recorded at around 22:00 hrs on the 15th February was found to occur within the model at around 17:00 hrs on the 15th February. This would likely indicate that the rainfall depths applied to the model are a reasonable representation of what occurred; however that the rainfall temporal pattern experienced within the catchment may have differed slightly from the temporal patterns recorded within the adjacent catchments. This is a constraint of not having any pluviometric data within the catchment. Nevertheless, replication of the peak water level means that some confidence can be derived for the hydraulic model schematisation, including but not limited to the selected roughness values within the hydraulic model domain.

7.4. Flow Validation – Regional Flood Frequency Estimates

7.4.1. Description

The paucity of streamflow data in the study area is typical of many small to medium sized catchments in Australia. In these cases, peak flow estimates can be obtained using a Regional Flood Frequency Estimation (RFFE) approach, which transfers flood frequency characteristics from a group of gauged catchments to the location of interest. Even in cases where there is recorded streamflow data it is beneficial to pool the information in the gauged record with the RFFE information. The RFFE technique used in this study is being developed as part of the ARR Project 5 (Reference 7) and information is derived from a national database consisting of 853 gauged catchments.

7.4.2. Methodology

Flood Frequency Analysis refers to procedures that use recorded and related flood data to identify the underlying probability model of flood peaks at a particular location in the catchment. While the methodology is designed to be robust and widely applicable, catchment attributes such as catchment storage and steep impervious gradients requires consideration as they can significantly affect discharge volumes. This is particularly applicable to the swamp area south of Candlagan Creek which is likely to significantly attenuate flood peaks. Consequently flood estimates from the RFFE method will overestimate discharge volumes downstream of that location.

7.4.3. Validation Results

The peak discharges obtained from the RFFE method were compared to that computed by the hydraulic model. For the upper parts of the Tomaga River catchment, discharge values obtained from the RFFE method are 15% - 20% lower than that computed from the hydraulic model. The discrepancy arises as a result of the steep gradient characteristic of the upper catchment not being explicitly contained in the RFFE method. The steeper upper part of the Candlagan Creek catchment led to a larger discrepancy.

Values from the RFFE method are in good agreement with that obtained from the hydraulic model further downstream. For the catchment areas as far as the Princes Highway approximately, the bias introduced by the steep slopes is reduced by the more typical topography characterising the middle part of the catchment.

Downstream of the highway, the Tomaga River floodplains and swamp area north of Candlagan Creek significantly attenuate flows explaining some of the underestimation by the RFFE method to that obtained by the hydraulic model. Furthermore, the 1 m AHD downstream tailwater conservatively applied to the hydraulic model leads to backwatering of the flows therefore augmenting the difference between discharges obtained from the respective methods.

7.5. Flood Extent Verification –Reported Flooding

7.5.1. Description

As part of the community consultation a number of areas that get wet were reported (discussed in Section 3.3). While levels such as those shown in Figure 6 subsequent to the 1974 storm cannot be used in a full calibration due to the absence of rainfall data, they still provide information regarding flood mechanism and extents.

7.5.2. Verification Results

Figure 14 identifies areas reported as previously flood affected against the 1% AEP modelled extent. Areas identified as flood affected in Tomakin, Broulee and Mogo are shown to be wet in the 1% AEP event. It is worth noting that flood affectation in Tomakin and Broulee is characterised by overland flow whereas Mogo is more mainstream and consequently more likely to scale in larger events. No flooding has been reported in Mossy Point and modelled flood extents indicate that the elevation of the area leads to low flood risk there.

The floodmarks shown in Figure 6 were surveyed; with the marks found to be at 4.9 m AHD. Anecdotally, the markings were said to be 0.3 m above the 1974 peak flood level (discussed in Section 3.3), and so the flood level was adjusted accordingly (i.e. 4.6 m AHD). Table 13 compares the surveyed marks to the design flood levels (modelled as per Section 8). The equivalent design levels were found to be generally around a 0.2 EY event (or 5 year ARI event), which corresponds to the ARI estimated from the daily read rainfall data (shown in Table 9).

Table 14 approximates the depths from anecdotal descriptions and compares these to the design flood depths (modelled as per Section 8). The equivalent design depths were found to be between the 10% AEP – 5% AEP event (or the 10 year – 20 year ARI event), which corresponds to the ARI estimated from the daily read rainfall data (shown in Table 10).

Table 13: Elizabeth Drive, Broulee – 1974 event flood marks compared to design levels (in m AHD)

Location	Surveyed level of floodmark minus 0.3 m	Average Equivalent Design Event	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
90	4.6	~ 0.2 EY	4.65	4.76	4.88	5.02	5.14	5.25	5.51
86	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
72	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
62	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
56	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
52	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
36	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
34	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51
28	4.6		4.65	4.76	4.88	5.02	5.14	5.25	5.51

Table 14: Princes Highway, Mogo – 1991 event flood depths (approximate) compared to design depths (in meters)

Location	Approx. depth	Average Equivalent Design Event	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
52	1.60	10% AEP – 5% AEP	1.45	1.58	1.92	2.29	2.63	2.80	4.10
48	1.60		1.36	1.53	1.88	2.24	2.58	2.75	4.06
42	1.60		1.35	1.48	1.83	2.19	2.53	2.70	4.03

8. DESIGN FLOOD MODELLING

8.1. Introduction

There are two basic approaches to determining design flood levels, namely:

- *flood frequency analysis* – based upon a statistical analysis of the flood events, and
- *rainfall and runoff routing* – design rainfalls are processed by hydrologic and hydraulic computer models to produce estimates of design flood behaviour.

The *flood frequency* approach requires a reasonably complete homogenous record of flood levels and flows over a number of decades to give satisfactory results. The George Bass Drive gauge has was unsuitable for this purpose due to tidal affectation and insufficient length of operation (gauge established in 1996).

For this reason a *rainfall and runoff routing* approach was used. The rainfall intensities and patterns from AR&R 1987 were used in the computing of the design rainfall to be input into the WBNM model. The WBNM model derived inflow hydrographs that were input to the TUFLOW hydraulic model. The TUFLOW hydraulic model in turn determines design flood levels, flows and velocities.

The key modelling parameters adopted for the design hydrologic modelling are summarised as follows:

- Lag Parameter (C) – 1.7
- Pervious Area Initial Rainfall Loss – 20 mm
- Pervious Area Continuing Rainfall Loss – 3.5 mm/hour
- Impervious Area Initial Rainfall Loss – 1 mm
- Impervious Area Continuing Rainfall Loss – 0 mm/hour

8.2. Oceanic Coincidence

Flooding in tidal waterways may occur due to a combination of oceanic inundation and catchment flooding derived from the same storm cell. The combined impact of these two sources on overall flood risk varies significantly with distance from the ocean and the degree of ocean influence, which is in turn affected by the estuary's entrance conditions. The *Development of Practical Guidance for Coincidence of Catchment Flooding and Oceanic Inundation*, hereon in referred to as the guide, presents a multivariate approach to translating the real-world environment for hydraulic modelling purposes. A sequential road-map is provided quantifying a number of parameters likely to affect flood mechanisms particularly in the context of peak flood levels and velocities. Parameters include the waterway entrance type, degree of accuracy required in the results and geographical location. The approach facilitates an optimum solution between the conflicting constraints of maintaining consistency in the modelling methodology while avoiding over-conservativeness in results.

The guide recognises the differing requirements of studies. Consequently, it accommodates three approaches to deriving ocean boundary conditions and design flood levels for flood modelling investigations in coastal waterways. A simplistic approach, a general approach and a detailed approach are proposed. The simplistic approach is considered suitable for analysis of small scale site specific developments where a cost effective but conservative method is warranted. The guide recommends either the general or detailed approaches for strategic studies undertaken for local government or with state government funding unless agreed to in writing by the local council and the funding provider, if state government. For general or detailed approaches, the combination of catchment flooding and ocean inundation scenarios is shown in Table 15.

Table 15: Combinations of Catchment Flooding and Oceanic Inundation Scenarios (Table 8.1 within *Modelling the interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways – OEH Draft 2014)*

Design AEP for peak levels/velocities	Catchment Flood Scenario	Ocean Water Level Boundary Scenario
50% AEP	50% AEP	HHWS
20% AEP	20% AEP	HHWS
10% AEP	10% AEP	HHWS
5% AEP	5% AEP	HHWS
2% AEP	2% AEP	5% AEP
1% AEP Envelope Level	5% AEP	1% AEP
1% AEP Envelope Level	1% AEP	5% AEP
1% AEP Envelope Velocity	1% AEP	Neap
0.5% AEP	0.5% AEP	1% AEP
0.2% AEP	0.2% AEP	1% AEP
PMF	PMF	1% AEP

Report No. MHL 1881 (*NSW Ocean Water Levels – Manly Hydraulics Laboratory, 2011*) documents a consistent tidal water level increase from south to north along the NSW coastline. Consequently, the guide splits the coastline into two regions based on whether the study area is north or south of Crowdy Head. Design ocean still water levels are obtained from the Fort Denison gauge in Sydney Harbour. This provides peak elevated ocean levels for design purposes (rounded up to nearest 0.05 m) and these levels are adjusted with an additional 0.1 m for regions situated north of Crowdy Head. The study area is located to the south of Crowdy Head.

The guide provides a framework within which the interaction of catchment flooding and oceanic inundation for the various classes of estuary waterways found in NSW (as well as associated ocean boundary conditions) can be assessed. The degree of influence of coastal processes on flooding within a waterway depends on the connectivity of the waterway to the ocean. This in turn depends on the type of estuary linked to the coastal waterway, the morphology and training of the waterway entrance and any management intervention. The guide classifies waterways into five Groups which are in turn simplified in three types, namely: Type A, Type B and Type C. Type A includes open oceanic embayments, tide dominated estuaries and trained entrances draining directly to the ocean or to bays. Type B includes fully trained wave dominated entrances and Type C includes ICOLLS and estuaries with untrained entrances. The categorisation is catchment specific and can be guided by the NSW Government 'Estuaries of NSW' website (http://www.environment.nsw.gov.au/estuaries/list.htm), which provided classifications based on Roy *et al* (2001) (Reference 18).

Tomago River and Candlagan Creek were classified as wave-dominated, barrier estuaries with open entrance conditions. However, the entrances are untrained and the study area was therefore determined to be Type C, as summarised in the table below.

Table 16: Summary of Decision Making

Name of Waterway	Tomaga River and Candlagan Creek	
Location		
Purpose of Assessment	Flood Study	
Local Council	Eurobodalla Shire Council	
1. Available Information Informing this assessment		
	Adopted Methodology / Figures	**Reasoning / Reference / Source of Information**
2. Waterway Entrance Type	C	Group 3 Wave Dominated Estuaries
3. Selected Approach	General	Develop downstream boundary for catchment wide flood study
4. Entrance Condition and Management	N/A	Waterway Entrance Type C
5. Modelling the Ocean Water Level Boundary		
North or South of Crowdy Head	South	Eurobodalla – location south of Crowdy Head
Peak Design Ocean Boundary Water Level	1% AEP – 2.55 m AHD 5% AEP – 2.35 m AHD	Eurobodalla – location south of Crowdy Head for Type C Waterway
Static or Dynamic Analysis	Dynamic	
Initial water level conditions in estuary	Based upon dynamic ocean boundary water levels aligned to start of the simulation	
6. Translating the Ocean Boundary to Study Boundary		
Adjustment	N/A	
Method Used / Source	N/A	
7. Relative timing of catchment flooding and oceanic inundation		
Peak Catchment with Static / Dynamic Ocean	Dynamic catchment flooding and oceanic inundation – peaks aligned	Aligned at downstream boundary of study area
8. Determining design flood levels		
Design AEP	PMF 0.5% AEP 1% AEP 2% AEP 5% AEP 10% AEP 0.2 EY	Project Brief

Design Flood Envelope	1%	Envelope derived from combinations as per Table 8.1 in guide
9. Sensitivity Testing		
Ocean boundary condition	Ocean boundary level increased by 0.3 m	
Peak Timing	Offset of peak ocean boundary to flood peaks by +/- 3hrs	Time of concentration 6-24hrs
Efficiency of Entrance	N/A	
10. Incorporating sea level rise		
Councils adopted Projections	Available	Council's adopted SLR projections
Adjustment made to: Boundary condtions Initial water levels Starting entrance conditions	Add Council's SLR projection to these factors.	Project Brief

8.3. Rainfall Critical Duration

To determine the critical storm duration for various parts of the catchment and inform the adopted design flood modelling, modelling of the 1% AEP rainfall event with a constant 2.35 m AHD ocean level was undertaken for a range of design storm durations from 25 minutes to 72 hours, using temporal patterns from AR&R (1987). An envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

It was found that the 72 hour, 48 hour, 36 hour, 9 hour and 4.5 hour design storm durations were critical across the whole catchment for the 1% AEP event. The 36 hour storm duration was critical across a larger area of the catchment than the other storm durations; covering the Tomago River and Candlagan Creek, from the ocean outlet up to Tomakin Road and the Princes Highway. The 72 hour storm duration was critical within the township of Broulee, bounded by George Bass Drive to the west, Candlagan Creek to the north and the ocean to the east. The peak flood level difference between the two durations was 0.03 m in favour of the 72 hour duration (within Broulee) and 0.62 m in favour of the 36 hour duration. The 48 hour storm duration was critical along Mogo Creek (a tributary to Tomakin River) from downstream of the Princes Highway up to Burkes Lane. The peak flood level difference between the two durations was 0.03 m in favour of the 48 hour duration (within Mogo) and 0.10 m in favour of the 36 hour duration. The 9 hour storm duration was critical along the tributaries into Tomago River and Candlagan Creek, and within the township of Tomakin. The peak flood level difference between the two durations was 0.1 m in favour of the 9 hour duration (along Tomago River from Tomakin Road up to Dunns Creek Road) and 0.4 m in favour of the 36 hour duration. Within Tomakin, the peak flood level difference between the 9 hour and 36 hour durations was 0.02 m in favour of the 9 hour duration. The 4.5 hour storm duration was critical along the tributaries upstream of Dunns Creek Road. The peak flood level difference between the two durations was 0.1 m in favour of the 4.5 hour duration and 0.7 m in favour of the 36 hour duration. Therefore it was determined appropriate to adopt the 36 hour design storm duration for the design storm events ranging from the 20% AEP to the 0.5% AEP event.

Additionally, the critical storm duration was determined for the PMF event for a range of storm durations, ranging from 30 minutes to 6 hours using the Generalised Short-Duration Method (GSDM) and from 24 hours to 96 hours using the Generalised South-East Australia Method (GSAM). Similarly, an envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

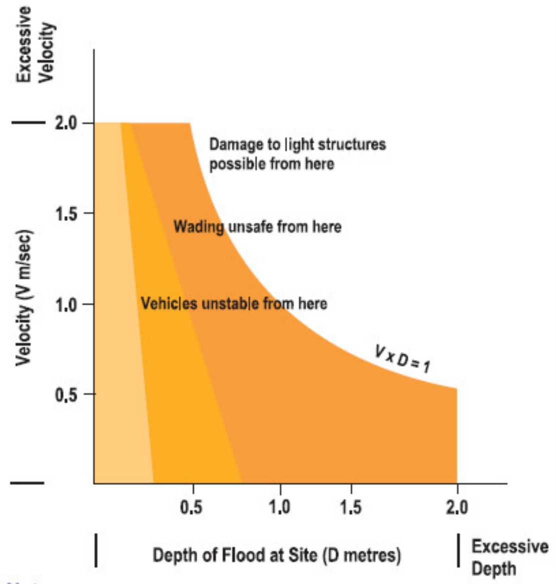
It was found that the 36 hour, 6 hour, 3 hour, 2 hour and 1.5 hour design storm durations were critical across the catchment for the PMF event. The 6 hour storm duration was critical across a larger area of the catchment than the other storm durations; covering the township of Tomakin, the Tomago River and Candlagan Creek, from the ocean outlet up to Tomakin Road and the Princes Highway. The 36 hour storm duration was critical within the township of Broulee; with the peak flood level difference of 0.4 m in favour of the 36 hour duration and 1.7 m in favour of the 6 hour duration. The 3 hour storm duration was critical along the Jeremadra Creek and Mogo Creek, including the township of Mogo. The peak flood level difference was 0.3 m in favour of the 3 hour duration and 0.6 m in favour of the 6 hour duration. The 2 hour storm duration was critical along Tomago River from Tomakin Road up to Dunns Creek Road; with the peak flood level difference of 0.2 m in favour of the 2 hour duration and 1.2 m in favour of the 6 hour duration. The 1.5 hour storm duration was critical along the tributaries upstream of Dunns Creek Road; with the peak flood level difference of 0.03 m in favour of the 1.5 hour duration and 1.6 m in favour of the 6 hour duration. Therefore it was determined appropriate to adopt the 6 hour design storm duration for the PMF event.

8.4. Analysis

8.4.1. Provisional Hydraulic Hazard

Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual, the relevant section of which is shown in Diagram 3. For the purposes of this report, the transition zone presented in Diagram 3 (L2) was considered to be high hazard.

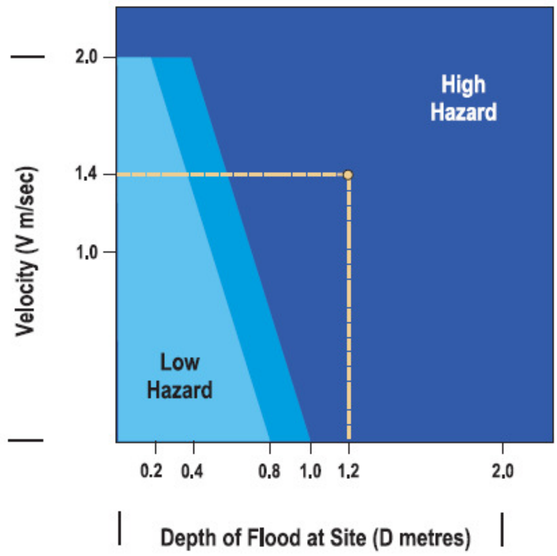
Diagram 3: (L1) Velocity and Depth Relationship; (L2) Provisional Hydraulic Hazard Categories (NSW State Government, 2005)



Notes

1. At velocities in excess of 2.0 m/s, the stability of foundations and poles can be affected by scour. Also, grass and earth surfaces begin to scour and can become rough and unstable
2. The velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only the average velocity is considered. For instance, the velocity of floodwaters in a model test has risen from an average of 1 m/sec to 3 m/sec between houses.
3. Vehicle instability is initially by buoyancy.
4. At floodwater depths in excess of 2.0 meters and even at low velocities, there can be damage to light-framed buildings from water pressure, flotation and debris impact.

Derived from laboratory testing and flood conditions which caused damage.



Notes

The degree of hazard may be either -

- reduced by establishment of an effective flood evacuation procedure.
- increased if evacuation difficulties exist.

In the transition zone highlight by the median colour, the degree of hazard is dependant on site conditions and the nature of the proposed development.

__Example:__

If the depth of flood water is **1.2 m**
and the velocity of floodwater is **1.4 m/sec**
then the provisional hazard is **high**

8.4.2. Provisional Hydraulic Categorisation

The hydraulic categories, namely floodway, flood storage and flood fringe, are described in the Floodplain Development Manual (NSW State Government, 2005). However, there is no technical definition of hydraulic categorisation that would be suitable for all catchments, and different approaches are used by different consultants and authorities, based on the specific features of the study area.

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells et. al. (2003):

- <u>Floodway</u> is defined as areas where:
  - o the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s **AND** peak velocity > 0.25 m/s, **OR**
  - o peak velocity > 1.0 m/s **AND** peak depth > 0.15 m

  The remainder of the floodplain is either Flood Storage or Flood Fringe:
- <u>Flood Storage</u> comprises areas outside the floodway where peak depth > 0.5 m; and
- <u>Flood Fringe</u> comprises areas outside the Floodway where peak depth < 0.5 m.

8.4.3. Preliminary Flood Emergency Response Classification of Communities

The Floodplain Development Manual, 2005 requires flood studies to address the management of continuing flood risk to both existing and future development areas. As continuing flood risk varies across the floodplain so does the type and scale of emergency response problem and therefore the information necessary for effective Emergency Response Planning (ERP). Classification provides an indication of the vulnerability of the community in flood emergency response and identifies the type and scale of information needed by the SES to assist in emergency response planning (ERP).

Criteria for determining flood ERP classifications and an indication of the emergency response required for these classifications are provided in the Floodplain Risk Management Guideline, 2007 (Flood Emergency Response Planning: Classification of Communities). Table 17 summarises the response required for areas of different classification. However, these may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas.

Table 17: Response Required for Different Flood ERP Classifications

Classification	Response Required		
	Resupply	Rescue/Medivac	Evacuation
High Flood Island	Yes	Possibly	Possibly
Low Flood Island	No	Yes	Yes
Area with Rising Road Access	No	Possibly	Yes
Area with Overland Escape Routes	No	Possibly	Yes
Low Trapped Perimeter	No	Yes	Yes
High Trapped Perimeter	Yes	Possibly	Possibly
Indirectly Affected Areas	Possibly	Possibly	Possibly

8.5. Results

8.5.1. Peak Flood Depths and Levels

The peak flood depths and peak flood levels are summarised in the table below. In the 0.2 EY event, inundation occurs on approximately a dozen properties on the north side of Connells Close in Mossy Point, roadways in Broulee and properties along Veitch Street, Creek Street, Charles Street and the Princes Highway in Mogo. In the 2% AEP event, the affectation increases to include more properties on Connells Close (to the south and east of the roadway), River Road and Hilmer Avenue in Mossy Point and properties in Broulee from Heath Street to Train Street.

Table 18: Peak Flood Depths (m) and Peak Flood Levels (m AHD) at Key Locations

Location	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
Depth (m)							
Candlagan Creek - Upstream of Coronation Dr	1.89	2.11	2.34	2.75	2.88	3.08	4.02
Candlagan Creek - Upstream of George Bass Dr	1.74	1.97	2.22	2.62	2.80	3.02	4.01
Broulee - Crn of Angle St and Elizabeth Dr	0.61	0.72	0.84	0.98	1.10	1.21	1.48
Tomago River - at the mouth	2.96	3.01	3.11	4.32	4.50	4.56	5.14
Mossy Point - Connells Cl	0.28	0.38	0.60	1.49	1.63	1.91	3.61
Tomakin - Crn of Ainslie Pde and Parks Pde	0.29	0.32	0.36	0.41	0.44	0.66	2.37
Tomago River - Downstream of George Bass Dr	3.68	3.91	4.18	4.95	5.12	5.40	7.12
Jeremadra Creek - Upstream of the Princes Hwy	2.68	2.98	3.32	3.64	4.03	4.63	5.59
Mogo Creek - Upstream of the Princes Hwy	2.37	2.69	3.06	3.43	3.79	3.96	5.28
Mogo Creek - Goba St	2.66	2.91	3.20	3.44	3.69	3.88	5.34
Level (m AHD)							
Candlagan Creek - Upstream of Coronation Dr	1.70	1.92	2.15	2.56	2.69	2.89	3.83
Candlagan Creek - Upstream of George Bass Dr	1.80	2.03	2.27	2.68	2.85	3.08	4.07
Broulee - Crn of Angle St and Elizabeth Dr	4.65	4.76	4.88	5.02	5.14	5.25	5.51
Tomago River - at the mouth	1.02	1.08	1.17	2.39	2.57	2.63	3.21
Mossy Point - Connells Cl	1.43	1.54	1.76	2.64	2.79	3.07	4.76
Tomakin - Crn of Ainslie Pde and Parks Pde	2.76	2.79	2.83	2.88	2.91	3.13	4.84
Tomago River - Downstream of George Bass Dr	1.46	1.69	1.96	2.73	2.90	3.18	4.90
Jeremadra Creek - Upstream of the Princes Hwy	5.74	6.03	6.37	6.69	7.09	7.69	8.65
Mogo Creek - Upstream of the Princes Hwy	7.68	8.00	8.37	8.74	9.10	9.27	10.58
Mogo Creek - Goba St	8.79	9.03	9.32	9.56	9.82	10.01	11.46

8.5.2. Peak Flow

The peak flood flows are summarised in the table below. In the smaller events, such as the 0.2 EY and 10% AEP event, the tidal flats between George Bass Drive and Coronation Drive attenuates the flow resulting in lower peak flow at Coronation Drive. In larger events, the attenuation properties of the tidal flats are exceeded by the flow along Candlagan Creek. Along Tomago River between George Bass Drive and the confluence with the ocean, the flow exceeds the attenuation capacity of the tidal flats in all events investigated; with the flow width across George Bass Drive extending further to the east in the PMF event.

Table 19: Peak Flows (m^3/s) at Key Locations

Location	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
Flow (m^3/s)							
Candlagan Creek - Crossing Coronation Dr	27.91	37.54	49.99	73.55	85.62	100.73	249.44
Candlagan Creek - Crossing George Bass Dr	28.31	38.02	49.64	68.71	80.48	95.00	248.40
Tomago River - at the mouth	212.51	265.65	337.18	452.28	522.00	618.45	1558.90
Mossy Point - Connells Cl	0.82	1.03	1.52	2.12	2.40	3.12	6.84
Tomago River - Crossing George Bass Dr (between Annetts Pde and Tomakin Rd)	203.37	260.12	335.51	415.69	505.72	604.08	1588.28
Jeremadra Creek - Crossing the Princes Hwy	114.87	145.81	184.53	221.29	260.03	300.58	857.59
Mogo Creek - Goba St	74.11	93.58	117.94	142.19	168.58	196.30	507.54

8.5.3. Provisional Hydraulic Hazard

During the 5% AEP event, the high hazard areas are mostly contained within the creeks and river; including the creek through Mogo and small sections of roadway in Broulee. In the 1% AEP event, the high hazard areas extend out from the creeks and rivers more. The township of Tomakin was determined to be mostly low hazard in the 5% AEP and 1% AEP event, transitioning to mostly high hazard in the PMF event.

8.5.4. Provisional Hydraulic Categorisation

During the 5% AEP event, the floodway areas are mostly contained within the creeks and rivers, including the creek through Mogo. The flood storage areas in the tidal flats area are adjacent to the creeks and river, and small sections of roadway in Broulee. In the 1% AEP event, the floodway areas extend out from the creeks and rivers more. The township of Tomakin was determined to be mostly flood fringe in the 5% AEP and 1% AEP event, transitioning to mostly flood storage in the PMF event.

8.5.5. Preliminary Flood Emergency Response Classification of Communities

Mapping of the preliminary flood emergency response classification of communities is shown on Figure 31. Mogo is classified as Rising Road Access as the properties are inundated from the creek at the rear of the properties, allowing evacuation by road. Many parts of Broulee and Tomakin were classified as Low Flood Island, with roads cut before properties are inundated. Mossy Point was classified as either Overland Refuge Area on High Flood Island (to the north) or High Flood Island. The former classification allows evacuation to the adjacent High Flood Island area, which is above the PMF extent but isolated by floodwater.

9. DESIGN FLOOD MODELLING – SENSITIVITY ANALYSIS

9.1. Introduction

The following sensitivity analyses were undertaken for the 1% AEP event to establish an understanding of the variability of design flood levels that may occur if different conditions or parameters were adopted:

- *Climate Change (Sea Level Rise) (See Section)*: Sea level rise scenarios of 0.10m, 0.23m, 0.39m and 0.72m were assessed;
- *Climate Change (Rainfall Increase) (See Section)*: Sensitivity to rainfall/runoff estimates were assessed by increasing the rainfall intensities by 10%, 20% and 30%;
- *Time of Concentration*: Sensitivity to the coincidence between the rainfall flood hydrograph and the ocean flood hydrograph were assessed by varying the coincidence by ± 3 hours;
- *Manning's 'n' Roughness Value*: The hydraulic roughness values were increased and decreased by 20% across the catchment; and
- *Blockage*: Sensitivity to blockage of pipes and culverts was assessed for 0% and 100% blockage.

It should be noted that the parameters are not independent and adjustment of one parameter (such as the Manning's *n* value) would generally require adjustment of other values (such as impervious percentage) in order for the model to produce the same level at a given location. The aim of the sensitivity analysis is to give an estimate of the potential variability of design flood levels.

9.2. Background to Sea Level Rise

The *NSW Sea Level Rise Policy Statement* was released by the NSW Government in October 2009. This Policy Statement was accompanied by the *Derivation of the NSW Government's sea level rise planning benchmarks* (NSW State Government, 2009) which provided technical details on how the sea level rise assessment was undertaken. Additional guidelines were issued separately by OEH, including the *Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments 2010.*

The 2009 Policy Statement says that:

> *"Over the period 1870-2001, global sea levels rose by 20 cm, with a current global average rate of increase approximately twice the historical average. Sea levels are expected to continue rising throughout the twenty-first century and there is no scientific evidence to suggest that sea levels will stop rising beyond 2100 or that current trends will be reversed… The 4th Intergovernmental Panel on Climate Change in 2007 also acknowledged that higher rates of sea level rise are possible"* (NSW State Government, 2009)

Subsequent to the commencement of this Flood Study (and in progress), the NSW Government announced its Stage One Coastal Management Reforms on the 8th September 2012. As part of these reforms, the NSW Government no longer recommends state-wide sea level rise benchmarks for use by local councils, with councils having the flexibility to consider local conditions when determining local future hazards.

Accordingly, ESC, in partnership with Shoalhaven City Council, commissioned Whitehead and Associates (Environmental Consultants) Pty Ltd and Coastal Environment Pty Ltd to undertake the *South Coast Regional Sea-level Rise Planning and Policy Response Framework* Report. The exhibition draft was completed in July 2014.

The key scientific findings were summarised as:

- *There is no compelling reason to not adopt the projections of the Intergovernmental Panel on Climate Change (IPCC) as the most widely accepted and competent information presently available.*
- *Recent sea level rise trends offshore of New South Wales are similar to the global average.*
- *Recent changes in sea level have been very similar between Sydney and the Shoalhaven and Eurobodalla coasts.*
- *Future NSW sea-level rise will likely be similar to the global average with only minor variation.*

The report provided locally adjusted projections of sea level rise derived from the IPCC's Assessment Report 5. Within this framework four Representative Concentration Pathway (RCP) scenarios were prescribed. These were based upon pathways for atmospheric greenhouse gas and aerosol concentrations, combined with land use changes. The RCP's were denoted as RCP8.5, RCP6.0, RCP4.5 and RCP2.6 that were consistent with the W/m^2 of the radiative forcing increase comparative to the conclusion of the 21st century.

Table 20 shows the locally adjusted projections of sea level rise as extracted from the *South Coast Regional Sea-level Rise Planning and Policy Response Framework* Report.

Table 20: Locally Adjusted Projections of Sea-level rise for Shoalhaven and Eurobodalla

Year	RCP2.6			RCP4.5			RCP6.0			RCP8.5		
	Low	Middle	High	Low	Middle	High	Low	Middle	High	Low	Middle	High
2015	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2020	0.02	0.02	0.03	0.02	0.02	0.03	0.02	0.02	0.03	0.02	0.02	0.03
2030	0.05	0.07	0.10	0.05	0.07	0.10	0.05	0.06	0.10	0.06	0.07	0.10
2040	0.10	0.12	0.16	0.09	0.12	0.16	0.08	0.12	0.15	0.11	0.14	0.17
2050	0.13	0.17	0.23	0.14	0.18	0.24	0.13	0.17	0.23	0.16	0.20	0.26
2060	0.15	0.21	0.30	0.18	0.24	0.32	0.16	0.22	0.30	0.21	0.29	0.37
2070	0.18	0.27	0.37	0.22	0.31	0.41	0.21	0.29	0.39	0.29	0.39	0.50
2080	0.21	0.31	0.44	0.25	0.38	0.51	0.25	0.36	0.50	0.35	0.49	0.64
2090	0.23	0.36	0.51	0.30	0.44	0.60	0.31	0.44	0.61	0.44	0.61	0.79
2100	0.25	0.41	0.58	0.34	0.50	0.69	0.36	0.53	0.72	0.53	0.74	0.98

ESC adopted the RCP6.0 High scenario at the Ordinary Council Meeting on the 25 November 2014.

Herein, the 2030, 2050, 2070 and 2100 projections were investigated as they relate to strategic planning horizons, to assess the sensitivity to projected sea level rise on the catchment's flood behaviour. The projected sea level rise values were 0.10m, 0.23m, 0.39m and 0.72m respectively.

9.3. Background to Increased Rainfall

The Bureau of Meteorology has indicated that there is no intention at present to revise design rainfalls to take account of the potential climate change, as the implications of temperature changes on extreme rainfall intensities are presently unclear, and there is no certainty that the changes would in fact increase design rainfalls for major flood producing storms. There is some recent literature by CSIRO that suggests extreme rainfalls may increase by up to 30% in parts of NSW (in other places the projected increases are much less or even decrease); however this information is not of sufficient accuracy or certainty as yet (NSW State Government, 2007).

Any change in design flood rainfall intensities will increase the frequency, depth and extent of inundation across the catchment. It has also been suggested that the cyclone belt may move further southwards. The possible impacts of this on design rainfalls cannot be ascertained at this time as little is known about the mechanisms that determine the movement of cyclones under existing conditions.

Projected increases to evaporation are also an important consideration because increased evaporation would lead to generally dryer catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions. The influence of dry catchment conditions on river runoff is observable in climate variability using the Indian Pacific Oscillation (IPO) index (Westra et. al., 2009). Although mean daily rainfall intensity is not observed to differ significantly between IPO phases, runoff is significantly reduced during periods with fewer rain days.

The combination of uncertainty about projected changes in rainfall and evaporation makes it extremely difficult to predict with confidence the likely changes to peak flows for large flood events within the catchments under warmer climate scenarios.

In light of this uncertainty, the NSW State Government (2007) advice recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime. Specifically, it is suggested that increases of 10%, 20% and 30% to rainfall intensity be analysed.

9.4. Results

9.4.1. Tidal Inundation

The extent of the HHWS tidal inundation (without rainfall) does not vary significantly for the 2030 and 2050 tidal horizons, with a slight extension within the tidal flats located to the north of Mossy Point and south of George Bass Drive. The 2070 and 2100 tidal horizons extend further into the tidal flats and further along Lynch Creek and Candlagan Creek.

9.4.2. Sea Level Rise

The constricted channel width of Candlagan Creek and the Tomago River at the confluence with the ocean resulted in peak flood level increases less than the corresponding sea level rise increase. In the 2030 scenario the peak flood levels increased by 0.06 m (in which sea levels were increased by 0.10 m), in the 2050 scenario the peak flood levels increased by 0.15 m (in which sea levels were increased by 0.23 m), in the 2070 scenario the peak flood levels increased by 0.26 m (in which sea levels were increased by 0.39 m) and in the 2100 scenario the peak flood levels increased by 0.51 m (in which sea levels were increased by 0.72 m).

9.4.3. Rainfall Increase

A rainfall increase of 10% resulted in increases to peak flood levels by up to 0.2 m along Candlagan Creek, Tomago River and through the township of Mogo. The peak flood levels within Broulee were found to increase by up to 0.1 m, although within Tomakin the peak flood level impact was less than 0.01 m.

A rainfall increase of 20% resulted in increases to peak flood levels by up to 0.4 m along Tomago River and increases of up to 0.3 m along Candlagan Creek and through the township of Mogo. The peak flood levels within Broulee were found to increase by up to 0.2 m and within

Tomakin the peak flood level impact was up to 0.1 m.

A rainfall increase of 30% resulted in increases to peak flood levels by up to 0.6 m along Tomago River and increases of up to 0.4 m along Candlagan Creek and through the township of Mogo. The peak flood levels within Broulee were found to increase by up to 0.2 m and within Tomakin the peak flood level impact was up to 0.1 m.

It should be noted that increases in rainfall are such that the 1% AEP event with a rainfall increase of 30% results in runoff approximately equivalent to a 0.2% AEP event under present day conditions.

9.4.4. Time of Concentration

Varying the time of concentration by ± 3 hours resulted in decreases in peak flood levels of up to 0.2 m within Tomago River (from the mouth of the river to upstream of the George Bass Drive Bridge). From upstream of the George Bass Drive Bridge to the junction of Tomago River with Jeremadra Creek, the peak flood levels were found to decrease by up to 0.1 m. Along Candlagan Creek, the peak flood levels decreased by up to 0.1 m.

9.4.5. Manning's Roughness

Peak flood levels were found to decrease across the catchment with decreased Manning's Roughness values. Within Candlagan Creek and Tomago River the peak flood levels decreased by up to 0.3 m.

Increased Manning's Roughness values were found to increase peak flood levels across the catchment. Within Candlagan Creek and Tomago River the peak flood levels increased by up to 0.3 m.

9.4.6. Blockage Assumptions

The hydraulic model was relatively insensitive to the assumption of no blockage of the culverts and pipes. Upstream of Dunns Creek Road, the no blockage scenario resulted in small sections of decreased peak flood levels, up to 0.1 m. Within the township of Broulee, the no blockage scenario resulted in decreased peak flood levels up to 0.02 m.

The 100% blockage scenario resulted in minor impacts across the catchment and slightly more impacts in the vicinity of the blocked infrastructure. Upstream of Dunns Creek Road and along Lynch Creek and Candlagan Creek where the Princes Highway crosses, the peak flood level was found to increase by up to 0.2 m. Within the township of Broulee, the 100% blockage scenario was found to increase peak flood levels by up to 0.1 m.

10. DISCUSSION – FLOOD BEHAVIOUR

A number of flood mechanisms have been investigated; including mainstream flooding, overland flooding and tidal inundation. Tidal inundation or storm surge occurs when atmospheric conditions result in higher sea levels, such as king tides. Mainstream flooding is when water levels rise up from rivers and creeks that have reached capacity. Overland flooding is the rainfall runoff as it travels downward to either a creek/river or underground drainage network.

10.1. Tomakin

The township of Tomakin is subject to different flood mechanisms across different areas. The area to the west of Sunpatch Parade and north of Parks Parade is predominantly subject to tidal inundation and mainstream flooding. The area to the east of Sunpatch Parade is subject to overland flooding.

10.2. Mossy Point

Similar to Tomakin, Mossy Point has a variety of flood mechanisms present. North of River Road is predominantly tidal inundation and mainstream flooding. The remainder of Mossy Point is subject overland flooding.

10.3. Broulee

Broulee drains in two directions; to the north and to the south. From Iluka Avenue overland flow travels north to Candlagan Creek, with properties adjacent to Candlagan Creek subject to tidal inundation and mainstream flooding.

South of Iluka Avenue, the township of Broulee is subject to overland flooding that drains south. This area is relatively flat, with slopes less than 0.5%, which results in flood water not draining away as fast as it would on a steeper slope. Additionally, the vegetated dunes that border the township to the east and south present a hindrance to overland flow discharging into the ocean.

10.4. Mogo

The township of Mogo is predominantly subject to mainstream flooding. Areas downstream of Tomakin Road, including Mogo Zoo, are also within the tidal affectation area.

11. PRELIMINARY FLOOD PLANNING AREAS

11.1. Background

Land use planning is considered to be one of the most effective means of minimising flood risk and damages from flooding. The Flood Planning Area (FPA) identifies land that is subject to flood related development controls and the Flood Planning Level (FPL) is the minimum floor level applied to new developments within the FPA.

The process of defining FPA's and FPL's is somewhat complicated by the variability of flow conditions between mainstream and local overland flow, particularly in urban areas. The more traditional approaches typically having been developed for riverine environments and mainstream flow.

Defining the area of flood affectation due to overland flow (which by its nature includes shallow flow) often involves determining at which point it becomes significant enough to classify as "flooding". The difference in peak flood level between events of varying magnitude may be minor in areas of overland flow, such that applying the typical freeboard can result in a FPL greater than the Probable Maximum Flood (PMF) level.

The FPA should include properties where future development would result in impacts on flood behaviour in the surrounding area and areas of high hazard that pose a risk to safety or life. Further to this, the FPL is determined with the purpose to decrease the likelihood of over-floor flooding of buildings and the associated damages.

The Floodplain Development Manual suggests that the FPL generally be based on the 1% AEP event plus an appropriate freeboard. The typical freeboard cited in the manual is that of 0.5 m; however it also recognises that different freeboards may be deemed more appropriate due to local conditions. In these circumstances, some justification is called for where a lower value is adopted.

Further consideration of flood planning areas and levels are typically undertaken as part of the Floodplain Management Study where council decides which approach to adopt for inclusion in their Floodplain Management Plan.

11.2. Methodology and Criteria

The methodology used in this report was as follows:

- 2070 sea level rise scenario peak flood levels trimmed to exclude areas with peak flood depths less than 0.15 m;
- Freeboard of 0.5 m applied;
- waterRIDE software used to calculate extent of 2070 scenario plus freeboard;
- Properties with greater than 10% of the cadastral lot (total land area of a property) inundated selected.

11.3. Results

The results from the aforementioned process identified 1,609 properties for inclusion in the preliminary flood planning area.

A sensitivity analysis was undertaken to determine how many additional properties were identified under the 2070 sea level rise scenario comparative to the existing sea level scenario. In the existing sea level scenario, 1,605 properties were identified, which is four less than the 2070 sea level rise scenario.

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- Residents of Eurobodalla Shire Council within the study area;
- Roads and Maritime Services.

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Figures

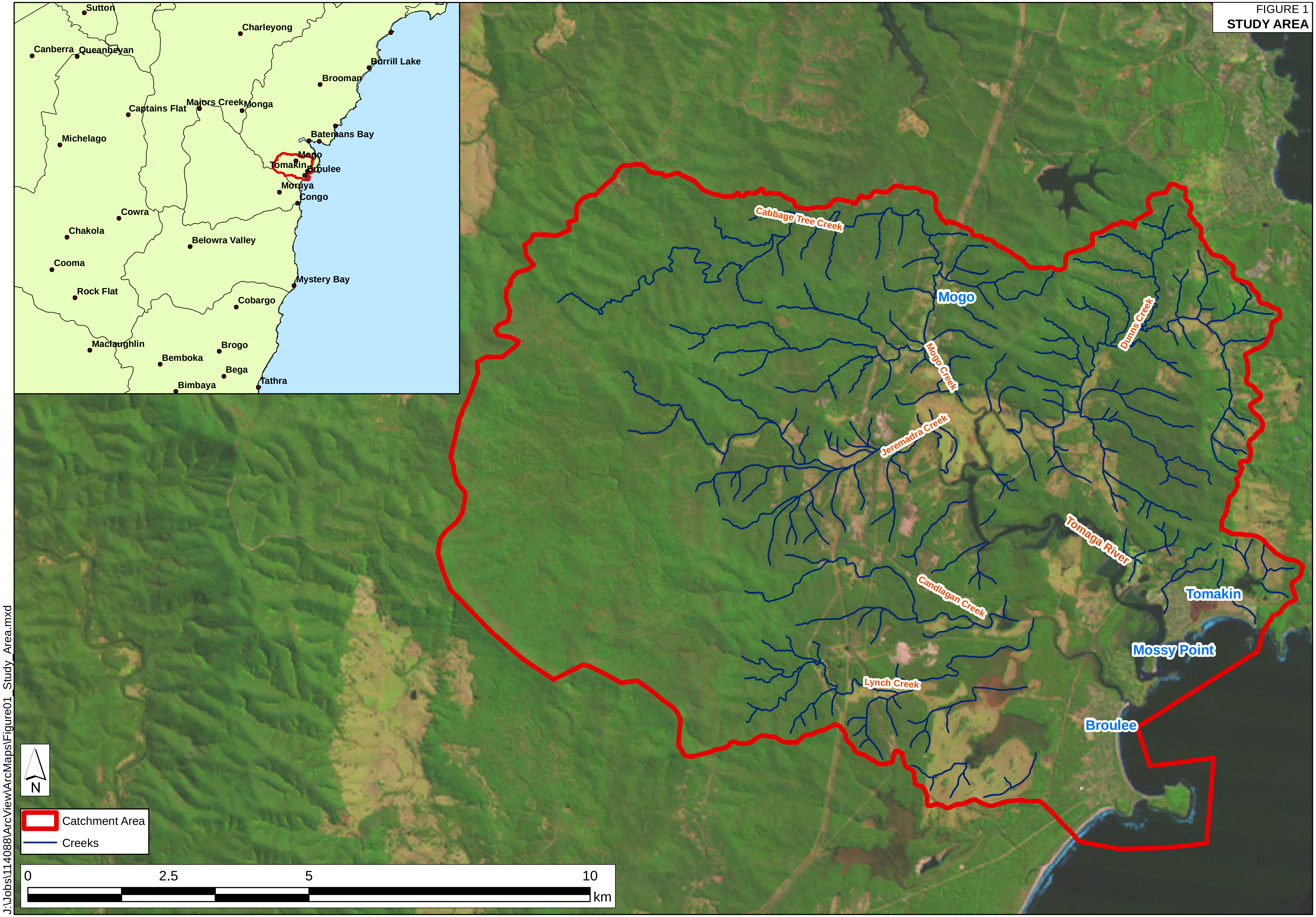

FIGURE 1
STUDY AREA
Sutton
Charleyong
Canberra
Queanbeyan
Burrill Lake
Brooman
Captains Flat
Majors Creek
Monga
Michelago
Batemans Bay
Mogo
Tomakin
Broulee
Moruya
Congo
Cowra
Chakola
Belowra Valley
Cooma
Mystery Bay
Rock Flat
Cobargo
Maclaughlin
Brogo
Bemboka
Bega
Tathra
Bimbaya
Cabbage Tree Creek
Mogo
Dunns Creek
Mogo Creek
Jeremadra Creek
Tomaga River
Candlagan Creek
Tomakin
Mossy Point
Lynch Creek
Broulee
N
Catchment Area
Creeks
0
2.5
5
10
km
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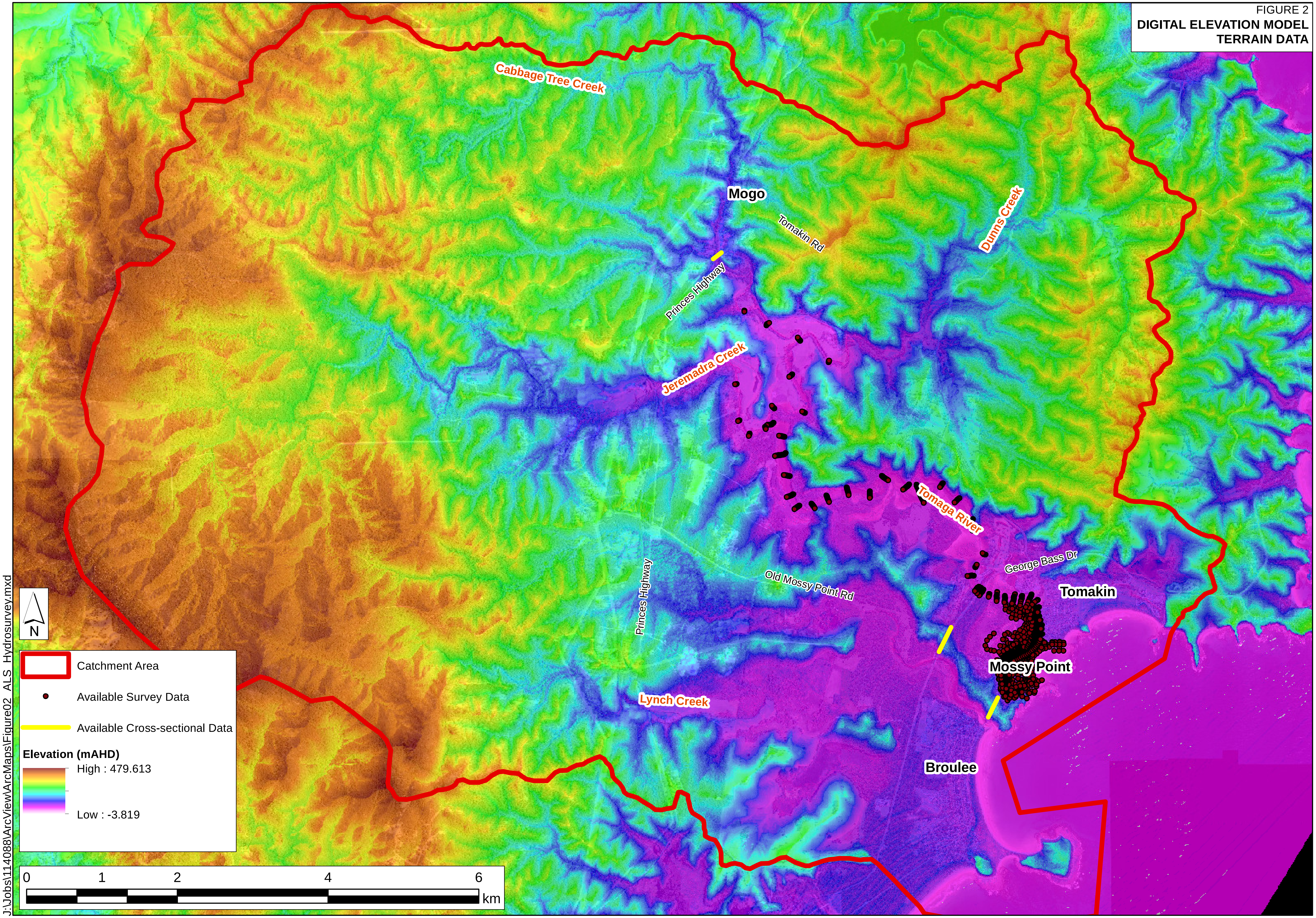

FIGURE 2
DIGITAL ELEVATION MODEL
TERRAIN DATA
Cabbage Tree Creek
Mogo
Tomakin Rd
Dunns Creek
Princes Highway
Jeremadra Creek
Tomaga River
George Bass Dr
Old Mossy Point Rd
Tomakin
Princes Highway
Mossy Point
Lynch Creek
Broulee
Catchment Area
Available Survey Data
Available Cross-sectional Data
Elevation (mAHD)
High : 479.613
Low : -3.819
0
1
2
4
6
km
N
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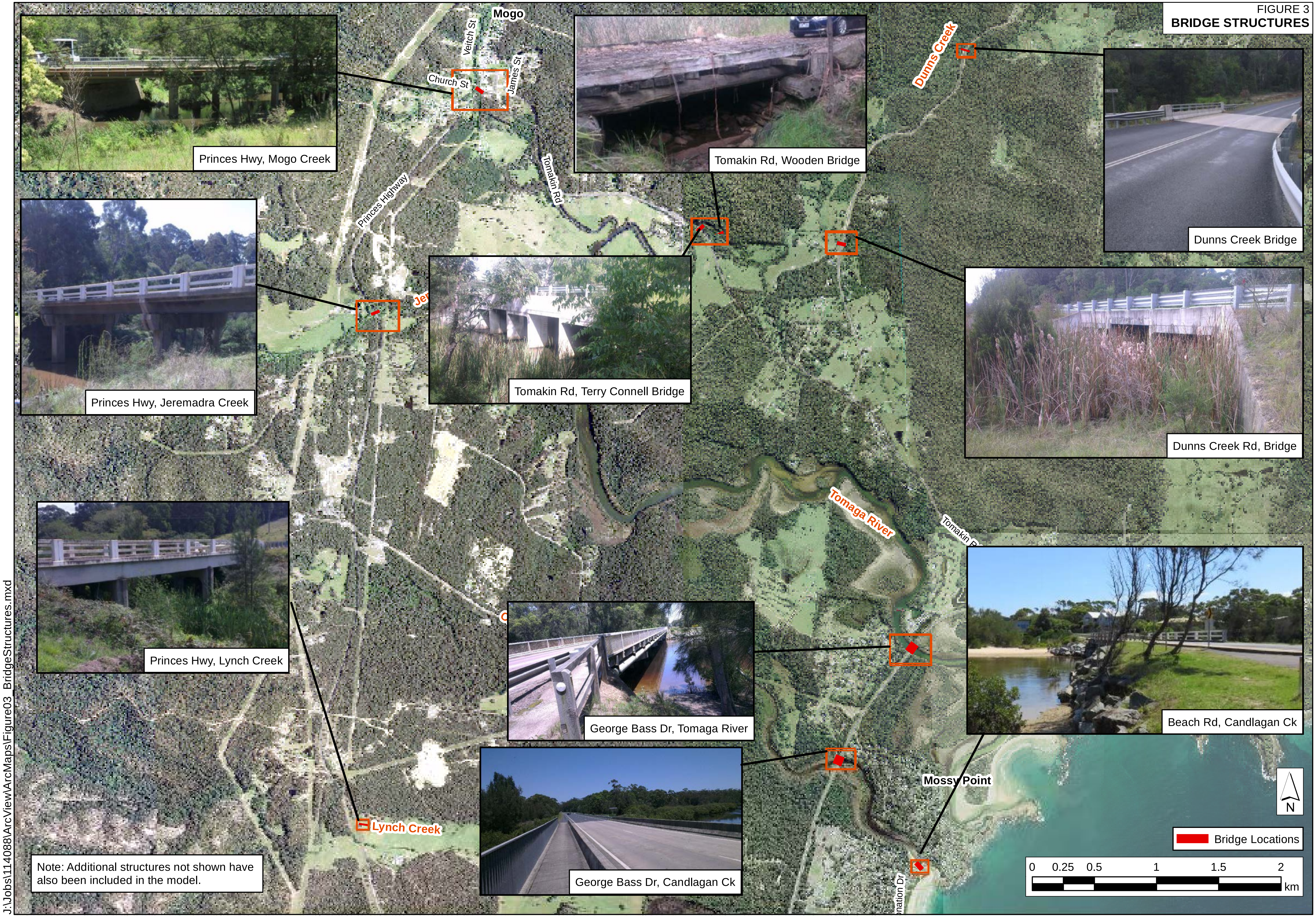

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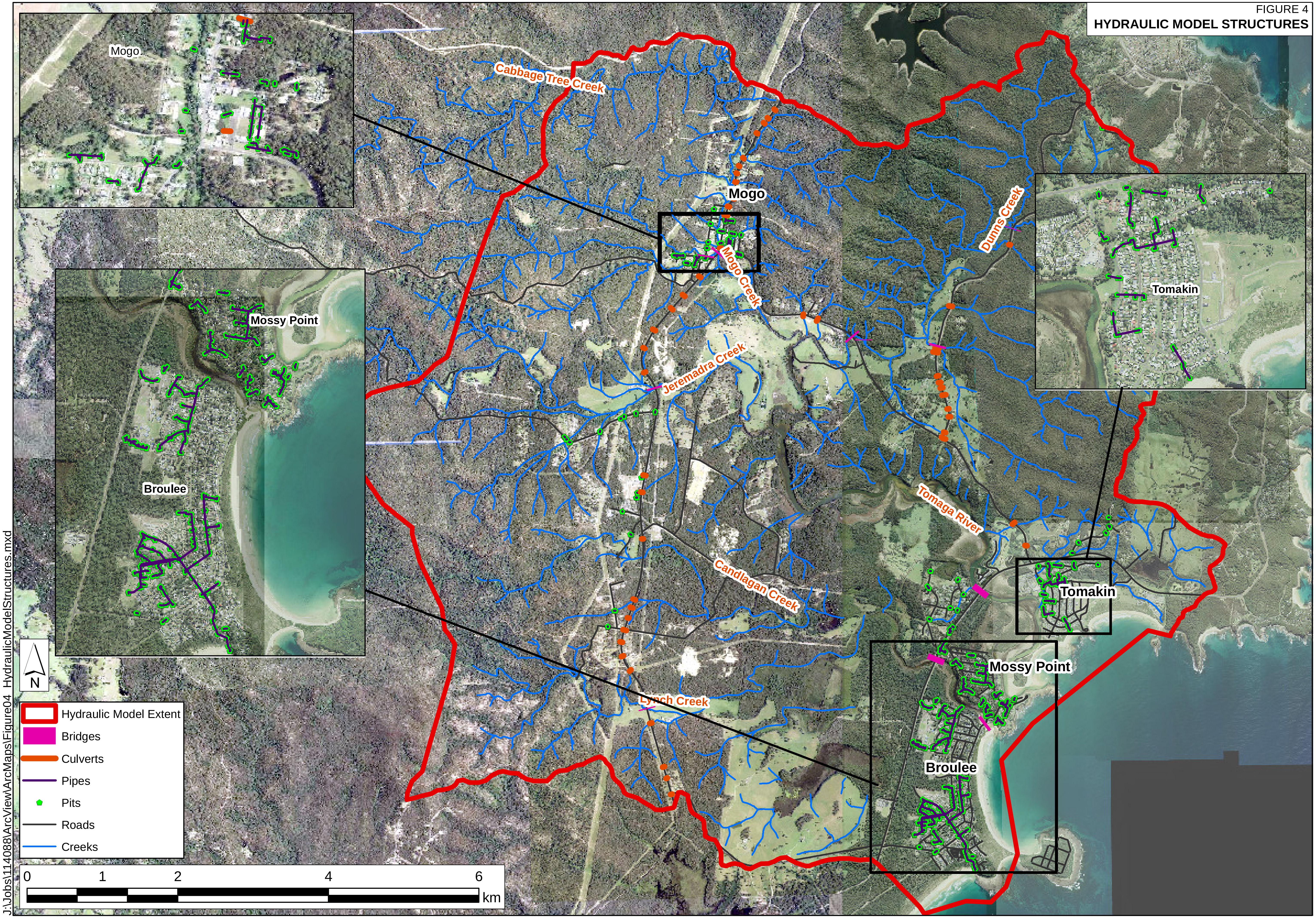

FIGURE 4
HYDRAULIC MODEL STRUCTURES
Mogo
Cabbage Tree Creek
Mogo Creek
Jeremadra Creek
Dums Creek
Tomakin
Tomaga River
Candlagan Creek
Lynch Creek
Mossy Point
Broulee
Hydraulic Model Extent
Bridges
Culverts
Pipes
Pits
Roads
Creeks
0
1
2
4
6
km
N
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FIGURE 5
GAUGE LOCATIONS

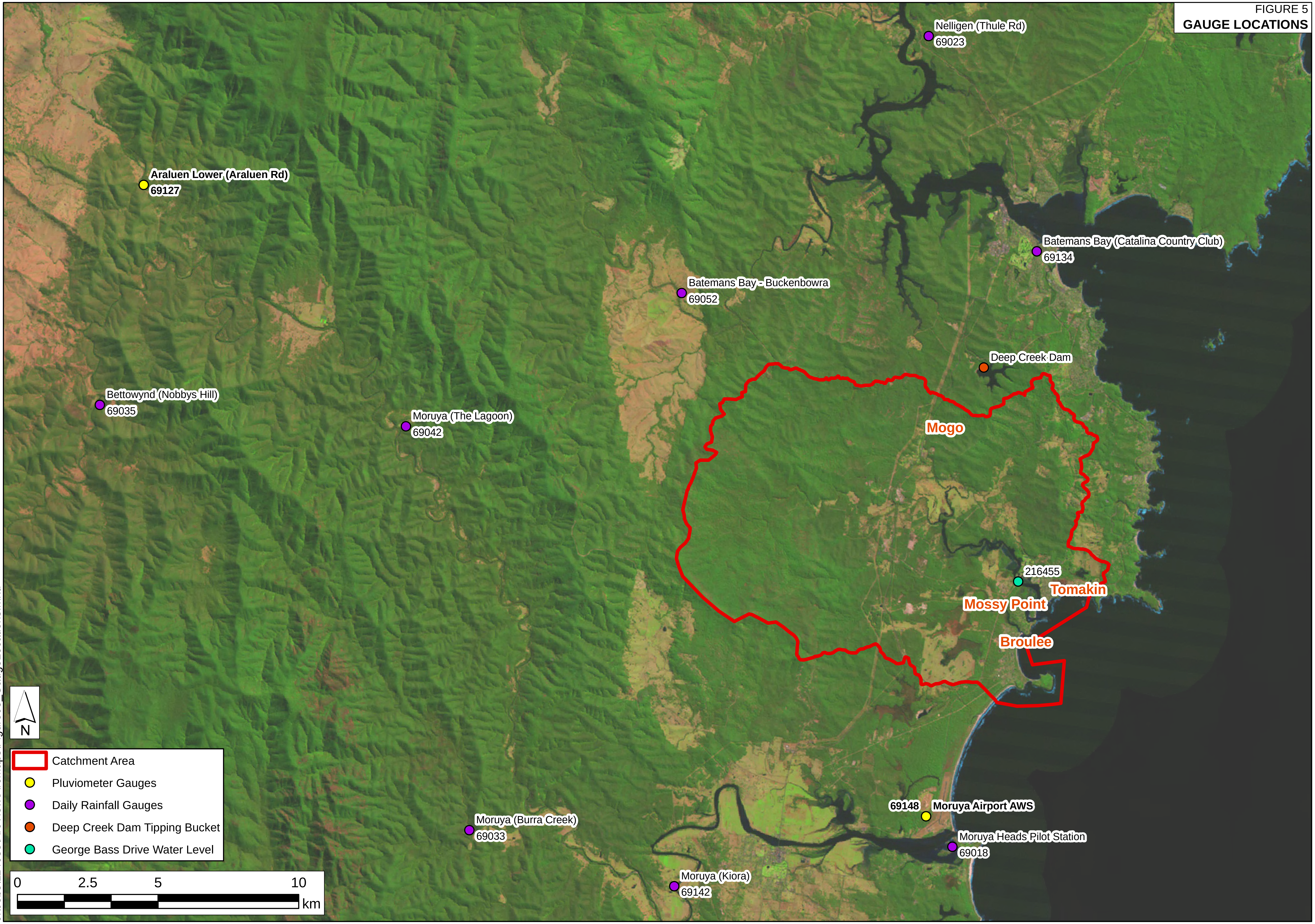

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FIGURE 6
HISTORICAL FLOOD MARKS
ELIZABETH DRIVE 1974 EVENT
28 Elizabeth Drive
18 Elizabeth Drive
34 Elizabeth Drive
36 Elizabeth Drive
52 Elizabeth Drive
56 Elizabeth Drive
62 Elizabeth Drive
72 Elizabeth Drive
86 Elizabeth Drive
90 Elizabeth Drive
Broulee
George Bass Dr
Elizabeth Dr
Smith St
Coronation Dr
Adams Way
Clarke
N
Flood Marks
Properties Floodmarked
0 50 100 200 300 400
m
Note: Floodmarks shown are approximately 0.3m above the actual flood level
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