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WALLAMBA RIVER FLOOD STUDY

FINAL

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Project **Wallamba River Flood Study**	**Project Number** **111028**
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EXECUTIVE SUMMARY

The NSW Government's Flood Policy provides for:

- a framework to ensure the sustainable use of floodplain environments,
- solutions to flooding problems,
- a means of ensuring new development is compatible with the flood hazard.

Implementation of the Policy requires a four stage approach, with the Floodplain Risk Management Study constituting the second stage. The first stage, the Wallamba River Flood Study, was completed in 1985 and established design flood levels from Tuncurry upstream to the Pacific Highway at Nabiac. The Flood Study results were later upgraded as part of the Wallamba River Floodplain Risk Management Study in 2004.

Due to the significant time since completion of the Flood Study a review of design flood levels along the Wallamba River was undertaken as part of this Nabiac Floodplain Risk Management Study. As a result of this review it was determined that the use of more sophisticated hydraulic models (2D as opposed to 1D) coupled with better survey data (use of ALS survey and higher quality bathymetry) would provide greater definition of the design flood extents. However the hydrologic modelling approach using a WBNM model has not been changed.

Reasons for Updating the Hydraulic Modelling Approach

The main reasons for updating the hydraulic modelling approach are as follows:

- the use of a two-dimensional (2D) hydraulic model,
- availability of detailed bathymetric data to better describe the bed of the Wallamba River rather than the use of cross sections,
- availability of airborne laser scanning (ALS) survey that provides a very accurate definition of the topography of the floodplain,
- a more detailed appraisal of design ocean level conditions and resulting design flood levels in Wallis Lake.

Adopted Hydraulic Modelling Approach

The adopted approach was to establish a TUFLOW 2D hydraulic model based on the available bathymetric and ALS survey with inflows from a WBNM hydrologic model. A calibration/verification was undertaken for the March 1978 event but this of limited value due to the relatively small magnitude of this event and the lack of quality rainfall (pluviometer) data.

Sensitivity analysis was undertaken to assess the impacts of various model parameters and the model was used for design flood estimation.

Coincidence of Ocean Levels and Runoff

Flood levels in Wallis Lake are affected by runoff from the upper catchment into the lake as well as inflows from the Pacific Ocean due to elevated ocean levels. However whilst these two flooding mechanisms are associated with each other, it is incorrect to assume that a (say) 100 year ARI (Average Recurrence Interval) ocean event will occur in conjunction with a 100 year ARI rainfall event. Such an event would have an ARI of greater than 100 year (say

500 year ARI or greater).

Elevated ocean levels occur due to a combination of tides (the high tide varies from approximately 0.5 m to 1.1 mAHD during the year) and what are known as ocean anomalies. The main components of ocean anomalies (difference between the predicted and the recorded tide) are storm surge and wave setup at the entrance to Wallis Lake. The storm surge component is the increase in ocean water level that occurs during storms as a result of the inverse barometric pressure effect and wind stress. Barometric pressure causes a localised rise in ocean water levels of about 0.1 m for each 10hPA drop in pressure and strong onshore winds produce surface currents that cause a build up of water against the coastline.

The oceanographic component of the tidal anomaly covers a range of other factors that can affect ocean water levels. The most important of these are the shelf waves generated by large storms remote from the NSW coast.

Together these components can raise ocean levels by up to 1m. As part of the Wallis Lake Flood Study Review in 2011 ocean anomalies were investigated and two runoff/ocean scenarios were adopted to determine design flood levels in Wallis Lake. A modified normal tide (peak level of 1 mAHD) was adopted in conjunction with the design rainfall event (termed a rainfall dominated event) and the design ocean level in conjunction with a 5 year ARI event (termed an ocean dominated event).

The following conditions were adopted for the design flood analysis in Wallis Lake:

- 0 mAHD initial water level in Wallis Lake,
- 36 hour critical rainfall storm duration inflows in conjunction with a modified normal tide (peak at 1 mAHD) (rainfall dominated event),
- design ocean levels based on the design ocean levels in Fort Denison/Sydney harbour plus a wave setup component of 0.35 m in the 100 year ARI, reducing to 0.25 m in the 5 year ARI. This scenario was run in conjunction with the 5 year ARI 36 hour critical rainfall storm duration inflows (ocean dominated event),
- upstream of the Forster/Tuncurry road bridge the rainfall dominated event produced the greater design flood levels and for this reason was been adopted as the design “tailwater” condition in Wallis Lake for use in this Wallamba River Flood Study.

1. INTRODUCTION

1.1. General

The Wallamba River catchment is located on the mid-north coast of NSW. It is one of the major tributaries to Wallis Lake which enters the Pacific Ocean at Forster/Tuncurry (Figure 1 and Figure 2) with a catchment area of approximately 500 km^2.

A previous Wallamba River Flood Study was completed in 1985 (Reference 1) in which a hydrologic/hydraulic modelling process was established, calibrated and used to determine design flood levels. This study was subsequently updated as part of the Wallamba River Floodplain Risk Management Study (Reference 2) in 2004. This present study updates this 2004 study.

Updating of the hydrologic/hydraulic modelling approach for the Wallamba River in this present study was considered necessary for the following reasons:

1. Since the completion of the Wallamba River Floodplain Risk Management Study (Reference 3) in 2004 there have been significant advances in hydraulic modelling software which now include two-dimensional models (2D). These models have the advantage over the previously used 1D models of calculating direction as well as magnitude. This is particular advantageous for the Wallamba River as more accurate determination of flow paths around the shoals and across the meander loops can be obtained. They also allow for more accurate representation of the considerable floodplain storage in the lower parts of the Wallamba River.
2. 2D models are more data intensive, requiring detailed topographic data. This data has become available since 2000 (Figure 3) with provision of a detailed bathymetric survey and overbank survey (from ALS provided in 2009). A 2D model provides better utilisation of this information rather than a 1D approach.
3. The Wallis Lake Flood Study (Reference 3) has been updated in 2011 to incorporate the use of ALS in a 2D hydraulic model of Wallis Lake and has analysed the effects of climate change (sea level rise and rainfall intensity increase). The results from Reference 3 have therefore changed the assumed design “tailwater” levels of the Wallamba River catchment and may thus impact on design flood levels upstream.

1.2. Objectives

The primary objective of this Flood Study is to define flood behaviour (5, 10, 20, 100, 200 and 500 year ARI design storms and the Probable Maximum Flood) for the Wallamba River downstream of the Pacific Highway at Nabiac to Tuncurry and to:

- define flood behavior in terms of flood levels, depths, velocities, flows and flood extents within the study area,
- prepare flood hazard and flood extent mapping, and
- consider the potential effects of a climate change induced increase in design rainfall intensities and sea level rise for the 5, 20 and 100 year ARI events.

This Appendix details the results and findings of the investigations. The key elements include:

- a summary of available historical flood related data,
- establishment of the hydrologic and hydraulic models,
- verification of the hydrologic and hydraulic models,
- definition of the design flood behaviour for existing conditions through the analysis and interpretation of model results, and
- sensitivity analysis and the assessment of the potential effects of climate change on flooding.

1.3. Description of Study Area

1.3.1. Settlements

The study area is defined as the floodplain of the Wallamba River upstream of the confluence with Wallis Lake (at Tuncurry) to the Pacific Highway at Nabiac. Upstream of the Pacific Highway at Nabiac the land is within the Greater Taree City Council local government area. The floodplain of the Wallamba River within the study area is predominantly used for rural activities with some rural residential housing. The only large urban community adjacent to the Wallamba River is Nabiac although there are camping parks and isolated rural homes along the river banks.

1.3.2. Catchment Description

The upper catchment is generally steep with slopes greater than 20 degrees and heavily forested. Approximately 6 kilometres upstream of Nabiac the Wallamba River joins with Khoribakh Creek. The Wallamba River is generally confined to an incised channel until approximately 3 kilometres upstream of Nabiac. Downstream of Nabiac the floodplain extends rapidly forming an extensive low lying, relatively poorly drained area.

The main tributaries to this lower floodplain are Pipeclay Creek, Bungwahl Creek and Darwakh Creek. Downstream of Failford Road there are a number of named and un-named islands in the river channel, including Gowack Island, Gereeba Island and Wallamba Island. It is likely that in very large flood events the floodwaters from the Wallamba and Coolongolook Rivers will merge downstream of Chapmans Road.

The Wallamba River is tidal to upstream of Nabiac with the low flow channel expanding in width from less than 100 m wide at Nabiac to over 600 m wide (including the islands) at Chapmans Road.

1.3.3. Flood History

According to the flood records (Reference 1) major flooding on the Wallamba River occurred in April 1927, February 1929, 1947 and March 1978. Minor flooding was recorded in February 1957, and March 1983. Interestingly, all these floods occurred in late summer/early autumn. There are peak heights for all of the above events but the quantity and quality of the records is unknown. These flood heights have been reported in several studies and it is assumed that the

Wallamba River Flood Study (Reference 1) provides the most accurate account and has been reproduced as Appendix A. The locations of these levels are shown on Figure 4.

According to the available peak height data the April 1927 event was the largest event on the Wallamba River and this event also produced the highest recorded level in Wallis Lake (approximately 2.2 to 2.3 m AHD). However, there is uncertainty whether this was caused by some form of blockage at the mouth of Wallis Lake as this was prior to construction of the present breakwaters. For many of the historic events there is conflicting peak height data (range of levels at or near the same location). It is assumed that this has been investigated as part of previous studies and there is no resolution regarding what is the most accurate record for each event. It is noted that some very high levels are termed “unreliable” in some references.

The town of Nabiac experiences more extensive flooding due to runoff from the local upstream catchment, rather than from the Wallamba River (Reference 4). The events that have caused flooding in Nabiac (as opposed to the Wallamba River) in approximate order of severity since the year 2000 include:

- June 2007,
- February 2002,
- October 2004,
- March 2000,
- February 2001,
- February 2003,
- October 2004,
- December 2005,
- January and March 2006.

None of the above events at Nabiac produced significant flooding on the Wallamba River. Historical flood levels around Wallis Lake and in the lower part of the Wallamba River are provided in Reference 5 (many of the levels are copies of those provided in Reference 1).

2. AVAILABLE DATA

The first stage in the investigation of flooding matters is to establish the nature, size and frequency of the problem. On large river systems there are generally stream height and historical records dating back to the early 1900's or in some cases even further. On the Wallamba River, historical water levels have been observed for a number of events as listed in Reference 1 and Appendix A. However there is no continuous and accurate record of flood heights that could, for example, be used for flood frequency analysis to determine design flood levels.

2.1. Data Sources

Data utilised in the Wallamba River modelling has been sourced from a variety of organisations/references as listed in Table 1.

Table 1: Data Sources

Type of Data	Format provided (Source)	Format Stored
Ground levels (ALS)	GIS and ASCII (GLC)	GIS and TUFLOW model
Detailed Bathymetry Survey	GIS (Dept of Land and Water Conservation)	GIS and TUFLOW model
GIS Information (cadastre)	GIS (GLC)	GIS
Design Rainfall	ASCII (AR&R)	WBNM model
Historical Rainfall Records	Reference 1, BoM	Report, MS Excel
Historical Flood Data	Reference 1	GIS, Report

2.2. Topographic Survey

The establishment of a hydraulic model requires survey of the river channel (bathymetry) as well as the overbank floodplain. Survey data includes bathymetric survey of Wallis Lake and the Wallamba River that was surveyed in October to November 1998 by the Department of Land and Water Conservation as well as Light Detection and Ranging (LiDAR) aerial survey obtained in 2007 and provided by Great Lakes Council in 2009.

A combination of the ALS and bathymetric survey was used to create the Digital Elevation Model (DEM) used in the hydraulic modelling. Where bathymetric survey was available it was used in preference to the ALS information. The accuracy of the ALS data is of the order of ± 0.2 m on hard surfaces but is much greater on non hard surfaces and particularly in heavily vegetated areas which are a feature of the floodplain areas downstream of Nabiac.

2.3. Wallamba River Bridge at Nabiac

Prior to 1870 there was a causeway across the Wallamba River called Clarkson's Crossing which was located just south of Nabiac (Photo 1) and set at the limit of the tidal reaches of the Wallamba. The causeway was originally stone however was later upgraded to two lanes of concrete to accommodate motor vehicle traffic along the Pacific Highway.

Photo 1: Clarkson's Crossing around 1914-1915 (source: www.nabiac.com)

In March 1959 Clarkson's Crossing was replaced with a truss bridge with two lanes downstream of the original crossing (Photo 2 and Photo 3). In 2005/2006 the truss bridge was replaced as part of the duplication of the Pacific Highway and in its place two bridges were constructed along the same alignment (Photo 4 and Photo 5).

Photo 2: Looking north across the Wallamba River at Nabiac, Aug 2004 (source: www.nabiac.com)

Photo 3: Wallamba River bridge, 1959-2005 (source: www.nabiac.com)

The effect of each bridge on flood behaviour is discussed in Section 5.3.2.


Photo 4: Wallamba River bridge at Nabiac looking east, Oct 2012


Photo 5: Wallamba River bridge at Nabiac looking north, Oct 2012

2.4. Historical Data

2.4.1. Recorded Flows and Levels

The only flow records available are for “The Old Sawmill” gauging station on the Wallamba River. The station is located approximately 5.5 kilometres upstream of Nabiac and was operated from April 1968 until September 1978 (Reference 1). A number of small flood records are available for that period including the March 1978 flood event. The March 1978 flood peak exceeds the maximum recorded height of the gauge by 2 metres and the highest flow gauging undertaken at the gauge corresponds to a flow of less than one third the estimated March 1978 peak flow. The data available for this station is therefore only of limited value. Peak flood levels were not available for the station from either the OEH website or PINNEENA, although gauging data was available from PINNEENA.

The Wallamba River Flood Study of 1985 undertook an extensive search of available flood records and these are reproduced as Appendix A. In 1982 a network of maximum height recorders was installed as well as a water level recording gauge. Since 1982 there have been no major flood events and thus this data is of no use for model calibration.

2.4.2. Historical Rainfall

Long term rainfall records are available from a number of gauges (Figure 2). However these daily read stations are of limited value for model calibration as they only indicate the 24 hour total rainfall and not the temporal pattern which is essential to define the rainfall intensity and duration.

The maximum rainfall during the 1927 and 1978 events at the Bulby Bush rainfall gauge within the catchment is described in Table 2 and isohyetal maps (from Reference 2) are shown in Appendix A.

Table 2: Daily Rainfall Totals at Bulby Bush (060003) – Reference 2

Flood Event	1 day	2 day	3 day
1927	225	257	282
1978	149	255	294

There are no pluviometer records (continually record rainfall) in the catchment for any of the major floods (for 1983 the gauge at Nabiac was incomplete) and thus reliance has to made on the surrounding pluviometers (Monkerai, Upper Johnsons Creek, Chichester Dam, Karuah Forest and Taree) for the March 1978 event.

In the absence of any other suitable flood event the March 1978 event was the only historical event available for model calibration. Daily read rainfalls for the 1978 event are shown in Table 3.

Table 3: Daily Rainfall Totals for the March 1978 Flood Event

Station	Name	Date of Reading			48h Total
		18-Mar	19-Mar	20 Mar	
60003	Bulby Brush	39	106	149	255
60013	Forster Beach	-	-	234 (1)	156 (1)
60015	Gloucester	-	-	311 (1)	207 (1)
60021	Krambach Post Office	56	126	145	271
60030	Taree Radio Station	25	87	181	268
60033	Krambach Bellevue	41	100	129	229
60062	Waukivory (The Ranch)	12	-	292 (2)	292
60087	Tinonee	24	97	134	231
60103	Krambach Tipperary	38	196	150	346

Note (1) – rainfall data only available as a total over 3 days and as such the 48 hour rainfall total is estimated only
Note (2) – rainfall data only available as a total over 2 days

Table 4: Peak Rainfall Intensities during the March 1978 Flood Event

Station		Duration						
		1 hour	2 hour	6 hour	9 hour	12 hour	24 hour	48 hour
Monkerai	Intensity (mm/hr)	46	32	18	13	11	8	5
	(Approx. ARI)	*(5)*	*(5-10)*	*(10)*	*(5-10)*	*(5-10)*	*(10-20)*	*(10)*
Upper Johnsons Creek	Intensity (mm/hr)	22	17	13	12	11	9	6
	(Approx. ARI)	*(<1)*	*(<1)*	*(2-5)*	*(2-5)*	*(5-10)*	*(20)*	*(20-50)*
Chichester Dam	Intensity (mm/hr)	21	18	15	14	13	9	6
	(Approx. ARI)	*(<1)*	*(<1)*	*(2-5)*	*(10)*	*(10-20)*	*(20)*	*(20-50)*
Karuah	Intensity (mm/hr)	24	18	14	13	11	9	6
	(Approx. ARI)	*(<1)*	*(<1)*	*(2-5)*	*(5-10)*	*(5-10)*	*(20)*	*(20-50)*
Taree	Intensity (mm/hr)	19	17	15	15	14	9	6
	(Approx. ARI)	*(<1)*	*(<1)*	*(2-5)*	*(10-20)*	*(20-50)*	*(20)*	*(20-50)*

Rainfall totals for various durations are shown for the 1978 Flood Event at nearby pluviometer stations in Table 4 and cumulative rainfall totals for the three rainfall stations closest to the catchment (Monkerai, Upper Johnsons Creek and Taree) are presented on Figure 5 and in

Appendix . It should be noted that all rainfall stations are a considerable distance from the catchment and may not be representative of rainfall within the catchment. The recorded rainfalls at the gauges have very different temporal patterns and rainfall events do not necessarily produce flooding with a similar recurrence interval.

2.5. Design Rainfall

Design rainfall intensities were based on AR&R 1987 (Reference 6). Uniform depths of rainfall with zero areal-reduction factors were applied across the entire catchment. Design rainfall depths used in the study are provided in Table 5.

Table 5: Design Rainfall Intensities (mm/h)

Storm Duration	1Y ARI	2Y ARI	5Y ARI	10Y ARI	20Y ARI	50Y ARI	100Y ARI	200Y ARI	500Y ARI
1 hour	28.9	36.9	46.4	52	59	68	75	83	92
1.5 hour	22.6	29.0	36.5	40.9	46.7	54	60	66	74
2 hour	19.0	24.3	30.8	34.5	39.5	46.0	51	56	63
3 hour	14.7	18.9	24.1	27.1	31.0	36.3	40.2	44.2	49.7
4.5 hour	11.4	14.7	18.8	21.2	24.4	28.6	31.7	35.0	39.4
6 hour	9.52	12.3	15.8	17.8	20.6	24.1	26.8	29.6	33.4
9 hour	7.39	9.54	12.3	14.0	16.2	19.0	21.2	23.4	26.5
12 hour	6.18	7.99	10.4	11.8	13.6	16.1	18.0	19.9	22.5
18 hour	4.81	6.24	8.15	9.28	10.8	12.7	14.2	15.8	17.9
24 hour	4.02	5.22	6.84	7.81	9.07	10.7	12.0	13.3	15.1
30 hour	3.49	4.54	5.96	6.82	7.93	9.40	10.5	11.7	13.3
36 hour	3.10	4.04	5.32	6.08	7.08	8.41	9.43	10.5	11.9
48 hour	2.56	3.33	4.41	5.06	5.90	7.01	7.88	8.76	9.97
72 hour	1.91	2.50	3.32	3.82	4.47	5.33	6.00	6.68	7.62

Probable Maximum Flood (PMF) design rainfall depths were calculated using Reference 7.

2.6. Previous Studies

2.6.1. Wallamba River Flood Study – 1985 (Reference 1)

This study was the first comprehensive study that established design flood levels for the Wallamba River and covered the reach from 1 kilometre upstream of Nabiac to 1 kilometre downstream of Failford. The study sourced all available data and established a hydrologic model (Cordery-Webb unit hydrograph – refer Reference 6 for details and copied in Appendix A) and a hydraulic model (HEC2 – refer Reference 6 for details).

The models were jointly calibrated to the March 1978 event and subsequently used for design flood estimation. It was based on the 1977 ARA&R and used a critical duration of 24 hours. The study extends from the Pacific Highway to the confluence of the Wallamba River with Darawakh Creek. The results from this study are discussed in Section 5 of the present report.

The hydraulic model was based on surveyed cross sections but these obviously do not define the floodplain study to the same extent as ALS, thus it is likely that a considerable amount of floodplain storage was not accounted for in the modelling approach.

2.6.2. Forster/Tuncurry Flood Study Report – 1989 (Reference 2)

This study undertook a Flood Study for Wallis Lake (subsequently updated in Reference 3) which included the lower part of the Wallamba River. A WBNM hydrologic model was established, which replaced the Cordery-Webb unit hydrograph method used previously. A Wallingford hydraulic model of the Wallis Lake catchment, including the Wallamba River as far upstream as Nabiac, was also established.

This study is of interest as it provides some historical flood level data (though most would appear to be the same as provided in Reference 1) and provides peak flows for the Wallamba River.

The study used design rainfall from the 1987 AR&R and design flows were based on a critical duration of 24 hours. The main emphasis of the 1989 Flood Study was on Wallis Lake and the lower reaches for the Wallamba River to Failford and takes into consideration both riverine flooding and tidal inundation and backwater effects.

2.6.3. Wallamba River Floodplain Risk Management Study for Nabiac, Failford and Minimbah Areas – 2004 (Reference 3)

The 2004 Flood Study Review was undertaken as part of the Wallamba River Floodplain Risk Management Study for Nabiac, Failford and Minimbah areas to provide more reliable design flood levels downstream of Nabiac, using MIKE-11 hydraulic modelling. The WBNM hydraulic model from the 1989 study was refined to provide improved definition of runoff from the Wallamba River catchment, in particular the catchment downstream of the Pacific Highway. Design rainfall was based on AR&R (1987) and used a critical duration of 36 hours.

This modelling approach was again calibrated to the March 1978 flood using the same rainfall and flood height data as for Reference 1. The 36 hour duration was determined as the critical storm duration and the WBNM hydrologic model produced a 100 year ARI peak flow at Nabiac within 4% of the flows determined in Reference 5. In general the results indicate that the 100 year ARI flood level is 0.4 m higher at Nabiac and 0.2 m higher at Failford than the 1985 study (Reference 1). The main reason for the difference was the use of the 1987 edition of Australian Rainfall and Runoff (Reference 6) in the 2004 study as opposed to the use of the 1977 edition of Australian Rainfall and Runoff in the 1985 study.

2.6.4. Nabiac Flood Study – 2010 (Reference 4)

The 2010 Flood Study was commissioned to assess local catchment flooding from a number of local creeks which flow through Nabiac using a 2D SOBEK model. The previous WBNM used in the 2004 study was refined for the local Nabiac Study while the larger catchment upstream of

Nabiac Bridge was unaltered providing peak discharges similar to the 2004 study.

The SOBEK model used to determine design flood levels uses two grid resolutions, one 5m by 5m and the more refined 2.5m by 2.5m focusing on the Nabiac Township.

The hydraulic model was calibrated to recorded flood levels during the June 2007 flood event and verified through the February 2002 and October 2004 events. Flood contours and extents were provided for the 5 year, 10 year, 20 year, 100 year and 200 year ARI and Probable Maximum Flood (PMF) design events.

2.6.5. Wallis Lake Flood Study Review – 2011 (Reference 5)

The 2011 Flood Study review was undertaken as part of the Wallis Lake Foreshore (Floodplain) Risk Management Study and Plan to provide more reliable design flood levels of the floodplain using SOBEK 2D.

The Flood Study Review updates the hydraulic modelling for Wallis Lake using the original WBNM hydrology from the Forster/Tuncurry Flood Study (Reference 2). The hydraulic model was calibrated to the May 2003 and March 2005 flood events and used to define design flood levels for existing conditions and future climate change scenarios. Procedures and assumptions to define ocean water levels were updated from the 1989 Study.

Water levels from the Wallis Lake Flood Study Review were used to define downstream boundary conditions for design events in the current study.

3. APPROACH ADOPTED

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.). In the absence of an extensive historical flood record, a flood frequency approach cannot be undertaken for the Wallamba River and must rely on the use of design rainfalls and establishment of a hydrologic/hydraulic modelling system. A diagrammatic representation of the flood study process is shown below.

For the current Wallamba River Study a hydrologic model (WBNM) and a hydraulic model (TUFLOW) were established, calibrated to historical data (March 1978) and used for design flood estimation.

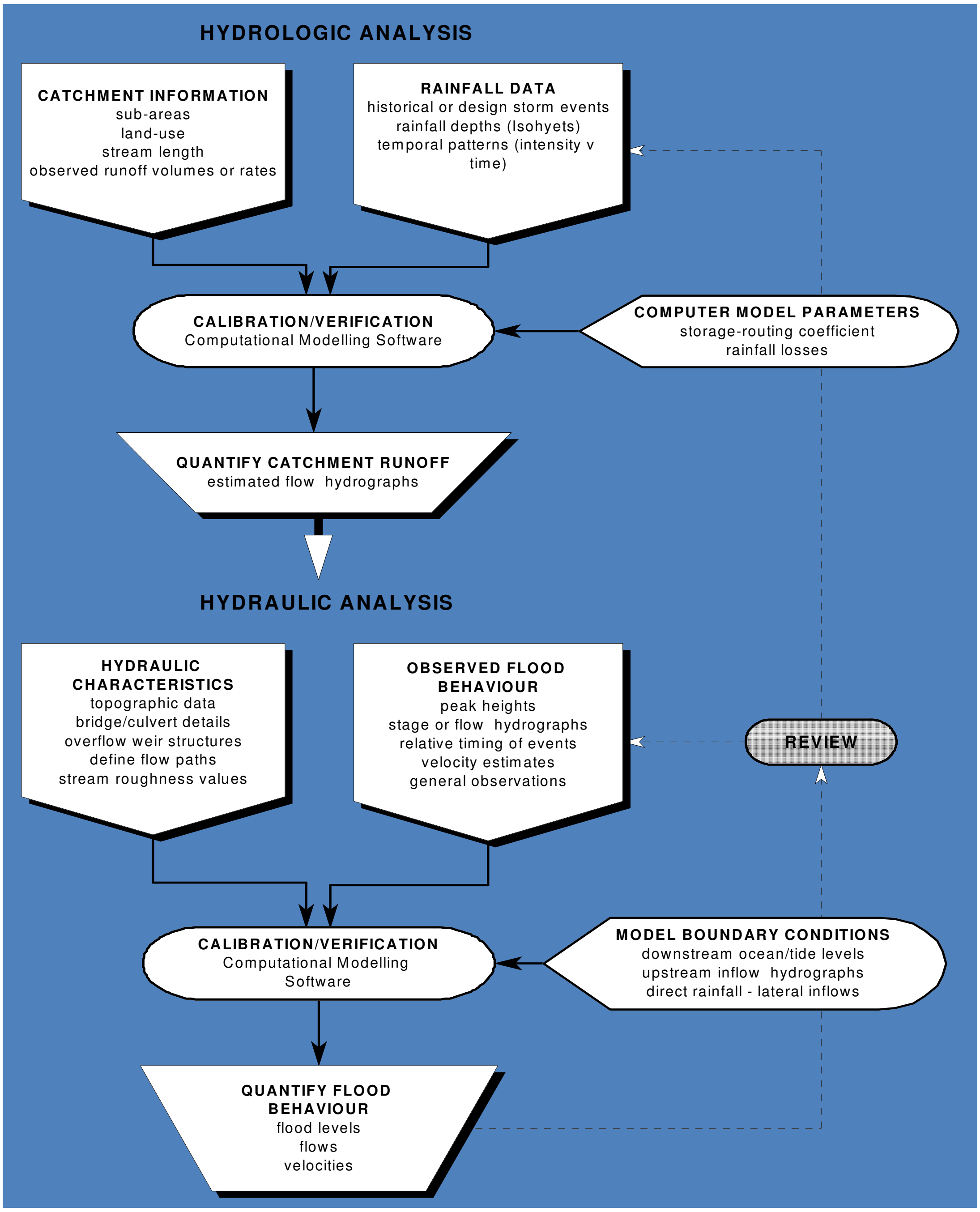

Diagram 1 Flood Study Process

4. HYDROLOGIC MODELLING

4.1. WBNM Background

Techniques suitable for design flood estimation in an urban environment are described in AR&R 1987 (Reference 6). These techniques range from simple procedures to estimate peak flows (e.g. Probabilistic Rational Method), to more complex rainfall-runoff routing models that estimate complete flow hydrographs and can be calibrated to recorded flow data.

The rainfall-runoff routing model WBNM was used to estimate the hydrologic response of the catchment and the model layout is shown in Figure 6. The model was used to generate flow hydrographs for the March 1978 flood using the limited amount of available rainfall data and then used to generate discharge hydrographs for the design flood events. The model input parameters are a storage lag factor (termed C) and the rainfall initial and continuing losses.

4.2. Model Verification

4.2.1. General

If data are available the WBNM model can be “calibrated” to historical flow records by including the historical rainfall data and adjusting the model parameters until a good match to the recorded flow and height data is achieved. The main issue with this approach for the Wallamba River is the limited amount of pluviometer records available and the absence of flow data.

Pluviometer data is required to provide a temporal pattern to be applied to the daily rainfall records. It is known that the rainfall temporal patterns can vary greatly across even a small area and thus over the Wallamba River sub-catchments the availability of only a few pluviometers outside the actual Wallamba River catchment means that the resulting “accuracy” of the calibrated model is low.

Ideally models are calibrated and validated against observed flood information, however for the study area the insufficient quality and quantity of historical data means that this process is not possible. Thus the approach taken is most appropriately termed verification and is where a limited and not definitive calibration is undertaken using the limited data available for the March 1978 event as well as comparison with results from previous studies.

4.2.2. March 1978 Calibration

After a review of available rainfall data, the available pluviometer records from rainfall stations at Upper Johnsons Creek, Taree and Monkerai were used to determine the temporal pattern for the March 1978 event. Daily totals from pluviometer stations and daily read rainfall stations were used to determine the total rainfall depth for each sub-catchment for the hydrologic model. Three calibration scenarios were tested, each using a different temporal pattern from each gauge.

The timing of the peak storm burst in March 1978 varies across the catchment with the peak storm burst at the Taree pluviometer occurring approximately 12 hours after the Upper Johnsons Creek gauge. Due to this difference in timing between the rainfall gauges and as the catchment response time is less than this, applying more than one of the three temporal patterns to the catchment hydrology will result in reduced catchment flows. It is impossible to determine the correct spatial and temporal pattern of the storm event over the Wallamba River catchment without either reliable stream flow data or additional rainfall gauge information within the catchment. As discussed in Section 2.4.1, the only available gauge data on the Wallamba River for the 1978 event is not reliable enough for use in calibration. As a result the temporal pattern from the Upper Johnsons Creek gauge was given more weight as it is closer to the catchment upstream of Nabiac.

In the absence of any other information the recommended value of 1.29 for the C – storage routing parameter in the WBNM model was adopted. Given the high level of uncertainty in rainfall adopted for the 1978 event the initial and continuing loss parameters were the same as adopted in Reference 2. The adopted hydrologic model parameters are given in Table 6.

Table 6: Adopted WBNM Hydrologic Model Parameters

Parameter	Value
C	1.29
Initial Loss (mm)	21
Continuing Loss (mm/h)	2.5

A comparison between results (March 1978 and design) from the following hydrologic modelling approaches is given in Table 7:

- Cordery-Webb method (Reference 1),
- WBNM model used in Reference 2,
- WBNM model used in Reference 4,
- Probabilistic Rational Method (PRM) as outlined in Reference 6,
- current WBNM hydrologic model.

Table 7: Hydrologic Model Flow Comparisons (m^3/s) at Nabiac Bridge

Method	March 1978	20 Year 36h ARI	50 Year 36h ARI	100 Year 36h ARI	100 Year 30h ARI	100 Year 48h ARI
WBNM (present study)	1143 (Taree) 729 (Monkerai) 776 (UJC)	1345	1575	1817	1724	1748
Cordery-Webb (Reference 1)	691	937	1098	1260	-	-
WBNM (Reference 2)	794	1296	1515	1740	-	-
WBNM (Reference 4)	-	1301	1531	1771	-	-
PRM	-	1115	1521	1904	-	-

The above results indicate that the present study assumes a greater peak flow for all design events than Reference 1 but very similar peak flows from References 2 and 4 (this is to be expected given these use the same hydrologic model). The PRM method produces lower peak flows for the 20 year ARI but greater for the 100 year ARI event. Note that the values in **Table 7** are from the hydrologic model at Nabiac Bridge, and differ slightly from the values in **Table 9**, which are inflows to the hydraulic model at a location slightly further upstream.

Additionally sensitivity testing was undertaken to assess the effects of changing model parameters and is presented in Section 5.4.

5. HYDRAULIC MODELLING

5.1. TUFLOW Background

The TUFLOW modelling package (Reference 8) includes a finite difference numerical model for the solution of the depth averaged shallow water equations in two dimensions (2D). The TUFLOW software has been widely used for a range of similar floodplain projects both internationally and within Australia and is capable of dynamically simulating complex overland flow regimes. The TUFLOW model build used in this study is 2012-05-AB-w64, further details regarding TUFLOW software can be found in Reference 8.

For the hydraulic analysis of overland flow paths across large river floodplains, a two-dimensional (2D) model such as TUFLOW provides several key advantages when compared to a traditional one-dimensional (1D) model (such as the Mike-11 model used in Reference 2). For example, in comparison to a 1D approach, a 2D model 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,
- inherently represent the available floodplain storage within the 2D model geometry (assuming suitable ALS data is available). This is a significant improvement upon a 1D approach where floodplain storage is difficult to accurately represent.

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 in detail across the model extent. This information can then be easily integrated into a GIS based environment enabling the outcomes to be incorporated into Council's planning activities.

5.2. Model Establishment

5.2.1. Topography and Model Extent

Given the objectives of the study and the availability of ALS and bathymetric data a 2D overland flow hydraulic model is the most suitable model to effectively assess flood behaviour. The TUFLOW hydraulic model of the study area includes the area approximately 1 kilometre upstream of Nabiac Bridge to 0.5 kilometres downstream of Tuncurry. The upstream limit was defined by the availability of bathymetric data whilst the downstream limit was the point at which the Wallamba River merges into Wallis Lake. The total area included in the 2D model is approximately 110 km^2 and the model extent is shown in Figure 7.

Within the 2D model domain, the topography was defined using a regular grid of 10 m x 10 m cells. This spatial resolution was adopted to sufficiently define the channel of the Wallamba River, which is on average 100 m wide.

Available ALS Survey does not include the areas north of Failford. In order to provide a more accurate estimate of available floodplain storage ground elevations within the Bungwahl and the Frogalla Swamps were approximated from 10 m ortho-photographic contours.

5.2.2. Downstream Water Levels

The downstream or tailwater boundary at the downstream limit of the Wallamba River at Tuncurry was defined using:

- observed levels for the March 1978 historical event (as reported in Reference 1 – Wallamba River Flood Study) and
- reported design flood levels from the Wallis Lake Flood Study Review (Reference 3).

For the March 1978 event the boundary was specified as a constant tailwater level (refer Table 8). For other events the boundary was specified as a dynamic stage hydrograph (refer Table 8) as derived from Reference 3.

Table 8: Adopted Peak Downstream Water Levels

Event	Peak Tailwater Level in Wallis Lake (mAHD)
March 1978	1.04
5 year ARI	1.25*
10 year ARI	1.39*
20 year ARI	1.58
50 year ARI	1.80
100 year ARI	1.99
PMF	4.48

Note: * level taken from rainfall dominated event rather than ocean dominated event which produces a greater level in Wallis Lake

5.2.3. Inflows

An inflow hydrograph for the Wallamba River upstream of Nabiac was extracted from the WBNM model as the combined flow from the upstream sub-catchments and defined as a flow versus time boundary condition at the upstream limit of the TUFLOW model domain.

For local sub-catchments draining within the TUFLOW model domain, local runoff hydrographs were extracted from the WBNM model and specified as source over area inflow boundaries defined within the 2D domain of the TUFLOW model. Nine of these inflow hydrographs were applied, and included the Bungwahl Creek tributary. Table 9 details peak inflows from WBNM into the TUFLOW model for all design events with the locations shown on Figure 7.

Note that the values in **Table 7** are from the hydrologic model at Nabiac Bridge, and differ slightly from the values in **Table 9**, which are inflows to the hydraulic model at a location slightly further upstream.

Table 9: Peak Design Inflows from WBNM into TUFLOW (m^3/s)

Location	5y ARI	10y ARI	20y ARI	50y ARI	100y ARI	200y ARI	PMF
WallUS9	896	1080	1322	1550	1789	2036	8243
NABDS2	3	3	4	4	5	6	24
TOWN4	1	1	2	2	2	2	9
NABDS4	5	6	7	8	9	10	41
NABDS5	8	9	11	12	13	15	64
NABDS6	3	3	4	4	5	6	24
NABDS7	2	2	2	2	3	3	12
Wallamba01	92	110	134	155	179	203	869
BUNGWAHL	191	228	277	322	369	418	1816
Wallamba02	224	268	327	380	437	495	2069

5.3. Calibration

5.3.1. Background

Where possible the performance of the hydrologic/hydraulic models should be “tested” against observed flood behaviour from past events within the catchment to ensure the accuracy of results. In this way the assumed model parameters can be adjusted so that the modelling behaviour best reproduces the historical patterns to flooding (generally replication of recorded levels). The process of adjusting model parameters to best reproduce observed flood behaviour is known as model calibration. Usually, the models are calibrated to a single flood event for which there is sufficient flood data available (e.g. flow records, peak flood levels or flood extents etc.). The performance of the calibrated model can then be tested by simulating other historical floods (without adjusting the model parameters) and comparing the ability of the calibrated models to reproduce the observed behaviour for these events. This is known as model validation or verification.

To calibrate/verify the models require a sufficient amount of flood data (quality and quantity) within the modelling extent. Although many major floods are known to have occurred within the catchment, only a few events have flood height data available (1927, 1929, 1947, 1957, 1978, 1983) with only the March 1978 event having pluviometer data (necessary to define the temporal pattern of rainfall) and none have flow records. Pluviometer records for the March 1978 event are not available within the catchment area, with the nearest located at Taree (10.8 km to the north east) and Upper Johnsons Creek (13.8 km south west of the catchment).

Given the lack of available data, the hydrologic/hydraulic models could only be calibrated to the March1978 event with no verification possible.

When flooding occurs within the catchment in the future, it is recommended that Council collect any available information (rainfall data, flows, flood heights etc) as soon as practicable after the event.

5.3.2. March 1978 Flood Event

The only significant and known change to the Wallamba River waterway since 1978 was the re-construction of Nabiac Bridge. Given that peak flood levels in the 1978 event are below the bridge deck and that the changes to the bridge piers appear unlikely to have changed flow conditions below the bridge, 1978 conditions were assumed to be similar to those used in present day design scenarios.

Flows from the hydrologic model were used as input into the hydraulic model and resulting flood levels compared to historical records (Figure 8). Peak flood levels produced for the 1978 flood event using the Upper Johnsons Creek rainfall pattern were given preference due to the proximity to the catchment upstream of Nabiac. Comparisons of the three peak height profiles for March 1978 against recorded flood levels are made on Figure 8. Given the uncertainty of peak height used as the downstream boundary condition, sensitivity analysis was undertaken with varying tailwater levels and the results of which are shown on Figure 9.

The above calibration approach is not rigorous enough to define with certainty the peak design flows, given the relatively poor quality of the calibration data (rainfall and peak height data). Greater certainty will only be possible when more extensive and higher quality calibration data becomes available.

5.3.3. Comparison to Design Flood Results

The model was calibrated to the March 1978 event and comparisons of peak flood height data was made against design flood levels in Table 10 and Figure 10. The 1978 event was modelled using three different temporal patterns, derived from the three available pluviometers.

Tailwater levels in Wallis Lake during the March 1978 event are unknown. The nearest available recorded flood mark to the downstream boundary of the model was 1.04 mAHD at Chapman Road. Three different tailwater scenarios at Wallis Lake were tested:

- 0.0 mAHD;
- 0.5 mAHD; and
- 1.0 mAHD.

The results of the tailwater sensitivity analysis are shown on Figure 9. A tailwater of 0.0mAHD was found to produce the best match to flood marks in the lower reaches of the Wallamba River (including at Chapman Road. Using a tailwater of 1.0 mAHD resulted in peak flood levels approximately 0.3 m to 0.4 m higher at Chapman Road, with the influence diminishing further upstream. Based on these findings, a tailwater of 0.0 mAHD was adopted for the 1978 calibration.

Table 10: Comparison of Historic Flood Levels to Design Results

Chainage	1927	1929	1947	1957	1963	1977	1978	1983	5y ARI	10y ARI	20y ARI	100y ARI	200y ARI	PMF
-7500	-	-	-	-	-	-	13.2 13.0	10.94	-	-	-	-	-	-
-4000	-	-	-	-	-	-	-	5.49	-	-	-	-	-	-
470	-	-	-	-	-	-	5.8	-	5.8	6.3	6.9	7.9	8.3	11.4
700	**Nabiac Bridge**													
940	7.3	7.86	7.15	-	-	-	5.5	3.34	5.4	5.9	6.5	7.4	7.8	10.8
2800	-	-	-	4.81	-	-	-	-	4.3	4.8	5.2	5.9	6.3	9.4
3000	5.93	-	-	5.63	-	-	-	-	4.1	4.6	5.0	5.6	5.9	9.0
3300	-	-	-	-	-	-	-	2.43	4.1	4.5	5.0	5.7	6.0	9.2
3800	-	-	-	-	-	-	4.05 3.75	-	3.7	4.1	4.6	5.5	5.8	9.0
5600	5.84	-	-	-	-	-	2.78	-	3.5	3.9	4.4	5.3	5.6	8.9
6200	5.56	-	-	4.96	-	-	2.5	1.79	3.2	3.7	4.2	5.0	5.4	8.7
6600	-	-	-	-	-	-	4.24 3.15	-	3.3	3.8	4.3	5.1	5.5	8.8
10100	-	-	-	-	-	-	2.75	-	2.6	3.0	3.4	4.1	4.3	7.8
11400	-	-	-	-	-	-	-	1.68	-	-	3.2	3.8	4.1	7.7
12700	-	-	3.35	-	-	-	-	-	2.3	2.6	3.0	3.6	3.9	7.3
14400	5.1	-	-	-	-	-	2.36 2.1	1.37	2.1	2.4	2.7	3.2	3.4	5.9
16600	-	-	-	-	-	-	1.67	2.06 1.23	2.0	2.2	2.5	2.9	3.1	5.2
19700	2.23	-	-	-	2.09	-	-	1.17	1.4	1.5	1.6	1.9	2.2	2.4
20400	-	1.92 1.88	-	-	-	1.75	1.39	1.09	1.7	1.8	2.1	2.4	2.6	4.4
23100	-	1.74	-	-	-	-	-	-	1.5	1.7	1.9	2.3	2.5	4.4
24500	-	1.84	-	-	-	-	1.04	1.16	1.4	1.6	1.8	2.2	2.4	4.4

Peak flood levels for the 1927, 1929 and 1947 events indicate the events were significant, and are close to the modelled 100 year ARI design flood levels near Nabiac Bridge. It is possible that around the Nabiac Bridge flood levels were influenced by the Clarkson's Crossing causeway which may have resulted in higher flood levels than for present day conditions. Additionally, the river geomorphology may have changed considerably since these floods occurred.

5.3.4. Manning's "n" Roughness Co-efficient

The Manning's "n" values for each grid cell were based on calibration to the March 1978 recorded data and comparison of available peak height data and design flood levels. The adopted values are shown in Table 11 and categories are in Figure 11.

Table 11: Manning's "n" values adopted in TUFLOW

Category	Manning's "n"	Description
1	0.08	Thick vegetation
2	0.05	Light urban
3	0.06	General floodplain
4	0.07	Upstream in-bank
5	0.05	Middle in-bank
6	0.02	Downstream in-bank
7	0.08	Vegetated floodplain

5.4. Sensitivity Analysis

In order to further establish the accuracy of the modelling approach and suitability for defining flood levels, a range of sensitivity runs were undertaken. These runs demonstrate how design flood results vary based on reasonable range of key model parameters. In this case the sensitivity runs have sought to test the following model inputs.

- Roughness. Input values of roughness to the 2D models have been increased and reduced by 20%;
- Rainfall. Local rainfall intensities have been increased and reduced by 20%.

Results are provided in Table 12 and Table 13 at various locations within the study area. See Figure 7 for the location of result and extraction points.

Table 12: Hydraulic Model Sensitivity – 100 Year ARI Flows (m^3/s)

Location	Manning's -20%	Manning's +20%	Rainfall -20%	Rainfall +20%
Nabiac Bridge	0%	0%	-24%	25%
200m U/S Nabiac Street	-3%	4%	-19%	19%
Mill Road, Failford	-3%	3%	-26%	26%
Chapmans Road, Tuncurry	-6%	6%	-29%	16%

Table 13: Hydraulic Model Sensitivity – 100 Year ARI Peak Flood Levels (m)

Location	Chainage (km)	Manning's -20% Difference (m)	Manning's +20% Difference (m)	Rainfall -20% Difference (m)	Rainfall +20% Difference (m)
U/S Nabiac Bridge	0.64	-0.5	0.4	-0.9	0.7
D/S Nabiac Bridge	0.76	-0.5	0.4	-0.8	0.7
Nabiac Street	3,43	-0.3	0.3	-0.7	0.6
Glen Ora Road Track	7.75	-0.3	0.3	-0.7	0.6
Elliots Road	10.34	-0.2	0.2	-0.6	0.5
Mill Road	14.37	-0.1	0.1	-0.4	0.4
Aquatic Road	16.47	-0.1	0.1	-0.3	0.3
Elliots Rd / Gowack Island	18.16	-0.1	0.1	-0.3	0.3
U/S Gereeba Island	22.20	-0.1	0.1	-0.2	0.3
Chapmans Road	24.52	-0.1	0.0	-0.1	0.3

The results indicate the following:

- **Roughness** – roughness results indicate a high level of sensitivity to roughness values used in upper region of the study area near Nabiac. Closer to Frogalla Swamp and Wallis Lake, there is less sensitivity due to lower mainstream velocities. Varying roughness by ±20% has an impact of -0.5m to +0.4m near Nabiac Bridge. In the lower areas of the catchment near Chapmans Road, the impact is only ± 0.1m.
- **Rainfall** – the study area shows a high level of sensitivity to rainfall intensity, with flood levels at Nabiac Bridge varying by -0.9m to +0.7m. In the lower areas of the catchment, the impact is reduced due to the larger storage volumes with levels varying by -0.1m to +0.3m near Chapmans Road.

A change in Manning's "n" of +20% results in a change in flood level at Nabiac Bridge of 0.4 m. Changing Manning's "n", particularly at Nabiac Bridge near the upstream boundary of the model, will not change flow as the inflows remain the same, but it will change conveyance and therefore water levels significantly. In downstream areas with lower flood velocities the effect of varying Manning's "n" is less significant.

Assuming a higher Manning's "n" reduces the conveyance of the river and, reducing velocities, increasing attenuation and slowing the timing of the flood peak. As a result peak flows at the downstream end of the model near Chapmans Road are reduced by 11%. On the converse side, by decreasing Manning's "n" we are increasing the speed of the flood wave, reducing attenuation of flow and therefore peak flows increase towards the more downstream areas near Chapmans Road by 13%.

On the other hand a 20% change in rainfall results in a 24% to 25% change in flow and results in a -0.9 m to +0.7 m change in flood level. In this case the model is using the same conveyance assumptions and therefore any increase in flood level must correspond with an increase in flow (or vice versa).

5.5. Design Event Modelling

5.5.1. Overview

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

- *flood frequency* analysis – based on 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. No such records were available within the catchment. For this reason a *rainfall and runoff routing* approach using WBNM model results was adopted for this study to derive inflow hydrographs for input to the TUFLOW hydraulic model, which determines design flood levels, flows and velocities. This approach reflects current engineering practice and is consistent with the quality and quantity of available data.

5.5.2. Rainfall

Design rainfall data were calculated according with Australian Rainfall and Runoff (AR&R) in Reference 6. The 100 year ARI critical storm duration was determined as the 36 hour event (refer Table 4) and this duration was adopted for all design events apart from the PMF. For the PMF event design rainfall was obtained from Reference 7 and the critical duration was taken as the 6 hour rainfall event.

5.5.3. Design Results

The results from the design event modelling provide a description of the design flood behaviour within the study area. Information such as peak flood levels; flows and depths were extracted from the TUFLOW model and have been documented as part of this report. In addition, the model results have also been produced in digital format that can be readily imported into Council's GIS systems.

Table 14 and Table 15 provide a summary of design flood levels and flows at key locations for each event and peak height profiles for design events are provided on Figure 10. Design flood hydrographs at the Nabiac Bridge, Failford near Mill Road and Chapmans Road are shown on Figure 12. Design flood extents and depths are provided on Figure 13 to Figure 16 for the 10, 20 and 100 year ARI events and the PMF. Design flood levels for the PMF event were taken from the envelope of the 6 hour and the 24 hour duration events, the latter resulting in peak flood levels in Wallis Lake.

Table 14: Peak Design Flood Levels (mAHD)

Location	Chainage (km)	5 Year ARI	10 Year ARI	20 Year ARI	100 Year ARI	200 Year ARI	PMF
Upstream Nabiac Bridge	0.64	5.7	6.2	6.8	7.7	8.2	11.3
Downstream Nabiac Bridge	0.76	5.6	6.1	6.7	7.6	8.0	11.0
Nabiac Street	3.425	4.0	4.4	4.9	5.6	6.0	9.1
Glen Ora Road Track	7.751	3.1	3.5	4.0	4.8	5.2	8.5
Elliots Road	10.34	2.6	3.0	3.4	4.1	4.4	7.8
Mill Road	14.37	2.1	2.4	2.7	3.2	3.4	6.0
Aquatic Road	16.47	2.0	2.2	2.5	3.0	3.1	5.2
Elliots Road/Gowack Island	18.16	1.8	2.0	2.2	2.6	2.8	4.5
Upstream Gereeba Island	22.20	1.6	1.7	2.0	2.4	2.5	4.4
Chapmans Road	24.52	1.4	1.6	1.8	2.2	2.4	4.4

Table 15: Peak Design Flows (m^3/s)

Location	5 Year ARI	10 Year ARI	20 Year ARI	100 Year ARI	200 Year ARI	PMF
Nabiac Bridge	880	1070	1310	1771	2010	5050
200m U/S Nabiac Street, Nabiac	860	1020	1200	1510	1670	4070
Mill Road, Failford	760	930	1150	1590	1820	6760
Chapmans Road, Tuncurry	700	860	1070	1440	1570	5810

Note that the values in Table 15 are results from the hydraulic model at Nabiac Bridge, and differ slightly from the values in Table 7, which are extracted from the hydrologic model.

5.5.4. Provisional Hazard Mapping

The provisional hazard maps for the 20 and 100 year ARI events and the PMF are presented as Figure 17 to Figure 19. Provisional hazard has been calculated as the product of peak depth and peak velocity in accordance with Figure L2 of the NSW Floodplain Development Manual (Reference 9).

5.5.5. Preliminary Hydraulic Categorisation

There are no definite criteria for carrying out hydraulic categorisation work and thus preliminary hydraulic categorisation was carried out using criteria utilised in other similar studies. Reference 9 provides the following qualitative definitions for the three hydraulic categories:

- Floodway – areas where a significant portion of flow is transported during flood events and areas which if blocked, even only partially, would lead to significant afflux and redistribution of flow;
- Flood Storage – areas of low velocity flow important for temporary storage of floodwaters during the passage of a flood. If a flood storage area is removed from the floodplain flood levels in nearby areas will increase and peak flow downstream could be expected to increase; and
- Flood Fringe – those areas not either floodway or flood storage.

There is no technical definition of hydraulic categorisation and different approaches are used by different consultants and authorities. For this study hydraulic categorisation was defined as:

- Floodway = Velocity * Depth > 1.0 m^2/s AND velocity > 0.1 m/s **OR** Velocity > 1.0 m/s. The remainder of the floodplain outside of the Floodway becomes either Flood Storage or Flood Fringe,
- Flood Storage is defined where the depth is greater than 1.0 m outside the Floodway,
- Flood Fringe where the depth is less than 1.0 m outside the Floodway.

Hydraulic categorisation for the 20 and 100 Year and PMF events are provided on Figure 20 to Figure 22.

As can be seen floodway areas are generally limited to in-bank Wallamba River flow although some floodway is defined in areas within the floodplain where high velocities are present after flood flows break out of the main channel and into the Frogalla and Bungwahl Swamps.

5.5.6. Comparison of Results from Previous Studies

Table 16 and Figure 10B compares the peak design 20 and 100 Year ARI flood levels estimated in this study with those presented in Reference 1 and 2 for a number of locations along the Wallamba River. Flow comparisons at Nabiac Bridge are provided in Table 7.

Table 16: Comparison of Peak Flood Level Results to Previous Studies

Location	20 Year ARI			100 Year ARI		
	Current Study	1985 FS (Ref 1)	2004 FRMS (Ref 2)	Current Study	1985 FS (Ref 1)	2004 FRMS (Ref 2)
Nabiac Bridge	6.7	6.1	6.7	7.7	7.2	7.5
Failford	2.7	2.4	2.7	3.2	3.1	3.3
Darawakh Bridge	2.0	1.8	2.2	2.4	2.3	2.6

The hydraulic model used in the 1985 Wallamba River FS (Reference 1) was HEC-2, whilst Mike-11 was used in the 2004 FRMS (Reference 2). Both previous hydraulic models were based on surveyed cross-sections which do not define the floodplain as well as ALS, disregarding a significant amount of floodplain storage especially within Frogalla and Bungwahl Swamps and other downstream areas.

Peak Flood levels around chainage 2000 m to 4000 m are lower than those in Reference 3 due to the inclusion of additional floodplain area which increases the conveyance capacity. Downstream of this, between 4000 m and 8000 m, the channel becomes more constricted and this results in an increased flood level.

6. CLIMATE CHANGE ASSESSMENT

6.1. Overview

The 2005 Floodplain Development Manual (Reference 9) requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of climate change on flood behaviour.

Since completion of the 1989 Flood Study (Reference 1), current best practice for considering the impacts of climate change (sea level rise and rainfall increase) have been evolving rapidly. Key developments in the last four years have included:

- release of the Fourth Assessment Report by the Inter-governmental Panel on Climate Change (IPCC) in February 2007 (Reference 10), which updated the Third IPCC Assessment Report of 2001 (Reference 11);
- preparation of Climate Change Adaptation Actions for Local Government by SMEC Australia for the Australian Greenhouse Office in mid 2007 (Reference 12);
- preparation of Climate Change in Australia by CSIRO in late 2007 (Reference 13), which provides an Australian focus on Reference 10;
- release of the Floodplain Risk Management Guideline Practical Consideration of Climate Change by the NSW Department of Environment and Climate Change in October 2007 (Reference 14 - referred to as the DECC Guideline 2007);
- Hunter, Central and Lower North Coast Regional Climate Change Project — Report 3: Climate Change Impact for the Hunter, Lower North Coast and Central Coast Region of NSW (Hunter and Central Coast Regional Environmental Strategy, 2009 (Reference 15).

In October 2009 the NSW Government issued its Policy Statement on Sea Level Rise (Reference 16) which states: "*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 the current trends will be reversed.*

Sea level rise is an incremental process and will have medium to long-term impacts. The best national and international projections of sea level rise along the NSW coast are for a rise relative to 1990 mean sea levels of 40 cm by 2050 and 90 cm by 2100. However, the 4th Intergovernmental Panel on Climate Change in 2007 also acknowledged that higher rates of sea level rise are possible";

In August 2010, the former NSW Department of Environment, Climate Change and Water issued the following:

- Flood Risk Management Guide (Reference 16): Incorporating sea level rise benchmarks in flood risk assessments,
- Coastal Risk Management Guide (Reference 17): Incorporating sea level rise benchmarks in coastal risk assessments.

In addition an accompanying document *Derivation of the NSW Government's sea level rise planning benchmarks* (Reference 18) provided technical details on how the sea level rise assessment was undertaken.

As a result of the information provided in the above and other documents, and to keep up-to-date with current best practice, this study incorporates an assessment of climate change. Although there are some minor variations in the sea levels predicted in these studies, policies, and guides, they all agree on an ocean level rise on the NSW coast of around 0.9 m by the year 2100 relative to 1990 levels.

The most recent guideline, the NSW Sea Level Rise Policy Statement (2009) (Reference 19) and associated guides, indicates a 0.9 m sea level rise by the year 2100 and a 0.4 m rise by the year 2050. It should be noted that climate change and the associated rise in sea levels will continue beyond 2100.

The climate change scenarios in the earlier DECC Guideline 2007 (Reference 14) suggested for undertaking rainfall sensitivity analysis in flood studies are indicated below.

increase in peak rainfall and storm volume:

low level rainfall increase	=	10%,
medium level rainfall increase	=	20%,
high level rainfall increase	=	30%.

A high level rainfall increase of up to 30% is recommended for consideration in the DECC Guideline 2007 (Reference 14) due to the uncertainties associated with this aspect of climate change and to apply the "precautionary principle". A 30% rainfall increase is probably overly conservative. The Hunter & Central Coast Regional Environmental Management Strategy 2009 (Reference 15) climate change study of the Hunter, for example, predicted an increase of spring rainfall of about 15% by 2080, and a drop in the other three seasons, although this does not predict the intensity of individual design events. A timeframe for the provision of definitive predictions of the actual increase is unknown. The DECC Guideline 2007 (Reference 14) is currently the only NSW reference providing guidelines for rainfall increases for design flood analysis due to climate change.

6.2. Results

Table 17 provides an assessment of various climate change scenarios for the 5 year, 20 year and 100 year ARI events. Locations indicated in Table 17 are as follows, and may be seen in Figure 7.

- Nabiac Bridge
- 200m U/S Nabiac Street, Nabiac
- Mill Road, Failford
- Chapmans Road, Tuncurry

Table 17: Climate Change Impacts on Peak Design Flow (m^3/s)

5 Year ARI Event				
Location	**Nabiac Bridge**	**200m U/S Nabiac St**	**Mill Road**	**Chapmans Road**
Base Case	881	861	760	698
Rain+10%	15%	13%	15%	16%
Rain+20%	29%	25%	31%	32%
Rain+30%	44%	36%	46%	48%
Ocean+0.5m	0%	0%	1%	-5%
Ocean+0.9m	0%	0%	3%	-3%
Rain+10%, Ocean+0.5m	15%	13%	17%	10%
Rain+10%, Ocean+0.9m	15%	13%	19%	10%
Rain+20%, Ocean+0.5m	29%	25%	32%	24%
Rain+20%, Ocean+0.9m	29%	24%	34%	21%
Rain+30%, Ocean+0.5m	44%	35%	47%	40%
Rain+30%, Ocean+0.9m	44%	35%	49%	36%

20 Year ARI Event				
Location	**Nabiac Bridge**	**200m U/S Nabiac St**	**Mill Road**	**Chapmans Road**
Base Case	1309	1196	1149	1068
Rain+10%	13%	10%	14%	14%
Rain+20%	26%	19%	28%	27%
Rain+30%	39%	29%	42%	36%
Ocean+0.5m	0%	0%	1%	-6%
Ocean+0.9m	0%	-1%	2%	-8%
Rain+10%, Ocean+0.5m	13%	10%	15%	12%
Rain+10%, Ocean+0.9m	13%	8%	23%	52%
Rain+20%, Ocean+0.5m	26%	19%	29%	23%
Rain+20%, Ocean+0.9m	26%	19%	30%	5%
Rain+30%, Ocean+0.5m	39%	29%	43%	34%
Rain+30%, Ocean+0.9m	39%	29%	44%	8%

100 Year ARI Event				
Location	**Nabiac Bridge**	**200m U/S Nabiac St**	**Mill Road**	**Chapmans Road**
Base Case	1771	1506	1591	1440
Rain+10%	12%	10%	13%	9%
Rain+20%	25%	19%	27%	14%
Rain+30%	36%	27%	41%	17%
Ocean+0.5m	0%	0%	0%	-9%
Ocean+0.9m	0%	0%	2%	-17%
Rain+10%, Ocean+0.5m	12%	9%	14%	-14%
Rain+10%, Ocean+0.9m	12%	9%	15%	-16%
Rain+20%, Ocean+0.5m	25%	19%	28%	-13%
Rain+20%, Ocean+0.9m	25%	19%	28%	-14%
Rain+30%, Ocean+0.5m	35%	27%	42%	-6%
Rain+30%, Ocean+0.9m	35%	27%	42%	-3%

The impact of Climate Change on design flood levels may be seen in Table 18. Locations are as indicated:

- A = Upstream Nabiac Bridge
- B = Downstream Nabiac Bridge
- C = Nabiac Street
- D= Glen Ora Road Track
- E = Elliots Road
- F = Mill Road
- G = Aquatic Road
- H = Elliots Road / Gowack Island
- I = Upstream Gereeba Island
- J = Chapmans Road

As can be seen generally the sensitivity to increased rainfall is more pronounced near Nabiac, with flood levels increasing by up to 1.06 m at Nabiac Bridge. Ocean level rise scenarios become more significant in the lowest sections of the Wallamba River, closest to Wallis Lake, with flood levels increasing up to 0.54 m near Chapmans Road.

The change in peak height profiles due to rainfall increases for existing ocean levels, 2060 ocean levels and 2100 ocean levels may be seen in Figure 23, Figure 24 and Figure 25 respectively.

Table 18: Climate Change Impacts on Peak Flood Levels (m)

5 Year ARI Event										
Location	A	B	C	D	E	F	G	H	I	J
Base Case (2010)	5.69	5.60	4.03	3.11	2.63	2.13	1.98	1.79	1.57	1.44
Rain+10%	0.36	0.35	0.28	0.30	0.25	0.18	0.16	0.13	0.10	0.09
Rain+20%	0.69	0.68	0.53	0.58	0.49	0.35	0.32	0.26	0.23	0.21
Rain+30%	1.00	0.98	0.77	0.85	0.72	0.52	0.48	0.40	0.36	0.34
Ocean+0.5m	0.01	0.01	0.03	0.06	0.08	0.11	0.12	0.15	0.22	0.29
Ocean+0.9m	0.03	0.03	0.08	0.18	0.23	0.29	0.32	0.38	0.51	0.61
Rain+10%, Ocean+0.5m	0.37	0.36	0.30	0.36	0.33	0.28	0.28	0.28	0.35	0.40
Rain+10%, Ocean+0.9m	0.38	0.37	0.35	0.45	0.45	0.44	0.47	0.50	0.62	0.72
Rain+20%, Ocean+0.5m	0.70	0.68	0.56	0.63	0.56	0.44	0.43	0.41	0.47	0.52
Rain+20%, Ocean+0.9m	0.71	0.69	0.60	0.72	0.67	0.59	0.61	0.62	0.74	0.83
Rain+30%, Ocean+0.5m	1.01	0.98	0.79	0.89	0.77	0.59	0.56	0.52	0.55	0.60
Rain+30%, Ocean+0.9m	1.02	0.99	0.83	0.96	0.87	0.72	0.72	0.71	0.81	0.89

20 Year ARI Event										
Location	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**
Base Case (2010)	6.79	6.67	4.87	4.04	3.41	2.69	2.50	2.23	1.97	1.81
Rain+10%	0.38	0.36	0.30	0.32	0.27	0.19	0.18	0.15	0.13	0.12
Rain+20%	0.74	0.70	0.57	0.61	0.51	0.37	0.34	0.31	0.28	0.27
Rain+30%	1.06	1.01	0.83	0.87	0.74	0.54	0.49	0.44	0.42	0.42
Ocean+0.5m	0.00	0.00	0.02	0.04	0.05	0.07	0.08	0.12	0.19	0.25
Ocean+0.9m	0.01	0.01	0.06	0.11	0.14	0.20	0.23	0.31	0.44	0.54

20 Year ARI Event										
Location	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**
Rain+10%, Ocean+0.5m	0.38	0.37	0.31	0.35	0.31	0.25	0.24	0.24	0.26	0.30
Rain+10%, Ocean+0.9m	0.41	0.40	0.42	0.51	0.50	0.43	0.43	0.41	0.50	0.64
Rain+20%, Ocean+0.5m	0.75	0.71	0.58	0.63	0.54	0.41	0.39	0.37	0.38	0.40
Rain+20%, Ocean+0.9m	0.75	0.72	0.61	0.68	0.61	0.52	0.52	0.56	0.70	0.82
Rain+30%, Ocean+0.5m	1.06	1.01	0.84	0.89	0.76	0.56	0.52	0.47	0.47	0.49
Rain+30%, Ocean+0.9m	1.07	1.02	0.86	0.93	0.82	0.66	0.64	0.67	0.82	0.96

100 Year ARI Event										
Location	A	B	C	D	E	F	G	H	I	J
Base Case (2010)	7.74	7.58	5.63	4.84	4.10	3.19	2.96	2.64	2.36	2.20
Rain+10%	0.37	0.35	0.30	0.31	0.26	0.19	0.16	0.13	0.14	0.15
Rain+20%	0.69	0.66	0.56	0.58	0.51	0.37	0.31	0.26	0.27	0.31
Rain+30%	0.96	0.91	0.79	0.82	0.74	0.53	0.45	0.37	0.43	0.49
Ocean+0.5m	0.00	0.00	0.01	0.02	0.03	0.04	0.05	0.08	0.17	0.24
Ocean+0.9m	0.01	0.01	0.03	0.06	0.09	0.13	0.15	0.22	0.39	0.51
Rain+10%, Ocean+0.5m	0.38	0.36	0.32	0.35	0.32	0.28	0.27	0.32	0.52	0.65
Rain+10%, Ocean+0.9m	0.38	0.36	0.33	0.35	0.33	0.29	0.29	0.35	0.57	0.69
Rain+20%, Ocean+0.5m	0.70	0.67	0.58	0.60	0.55	0.44	0.41	0.49	0.72	0.84
Rain+20%, Ocean+0.9m	0.70	0.67	0.58	0.61	0.56	0.45	0.42	0.55	0.78	0.89
Rain+30%, Ocean+0.5m	0.96	0.91	0.80	0.84	0.77	0.59	0.54	0.71	0.93	1.04
Rain+30%, Ocean+0.9m	0.96	0.91	0.80	0.85	0.78	0.61	0.57	0.76	0.98	1.09

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- Office of Environment and Heritage,
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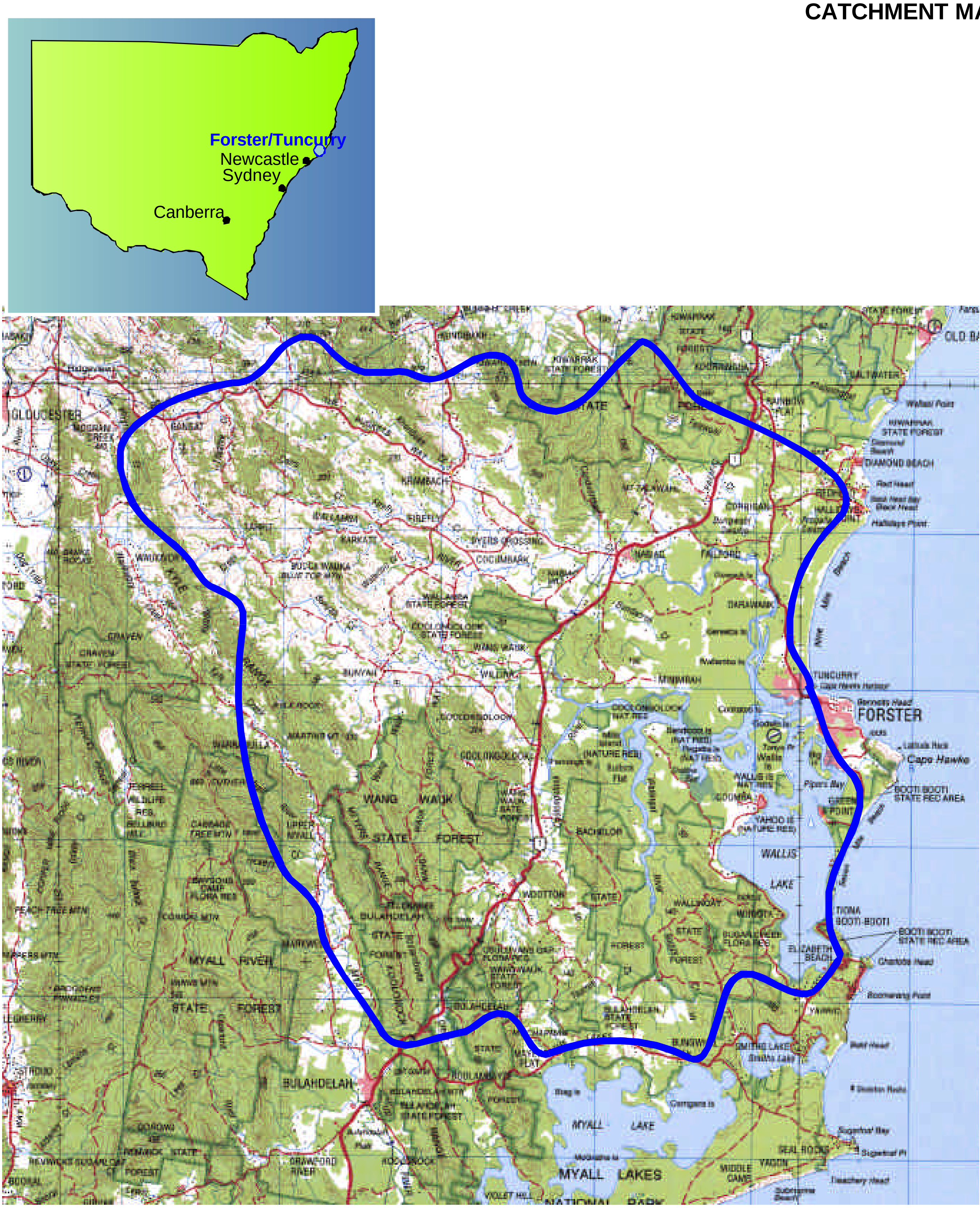
Figures

FIGURE 1
CATCHMENT MAP



5 0 5 10 15 20 25km

SCALE

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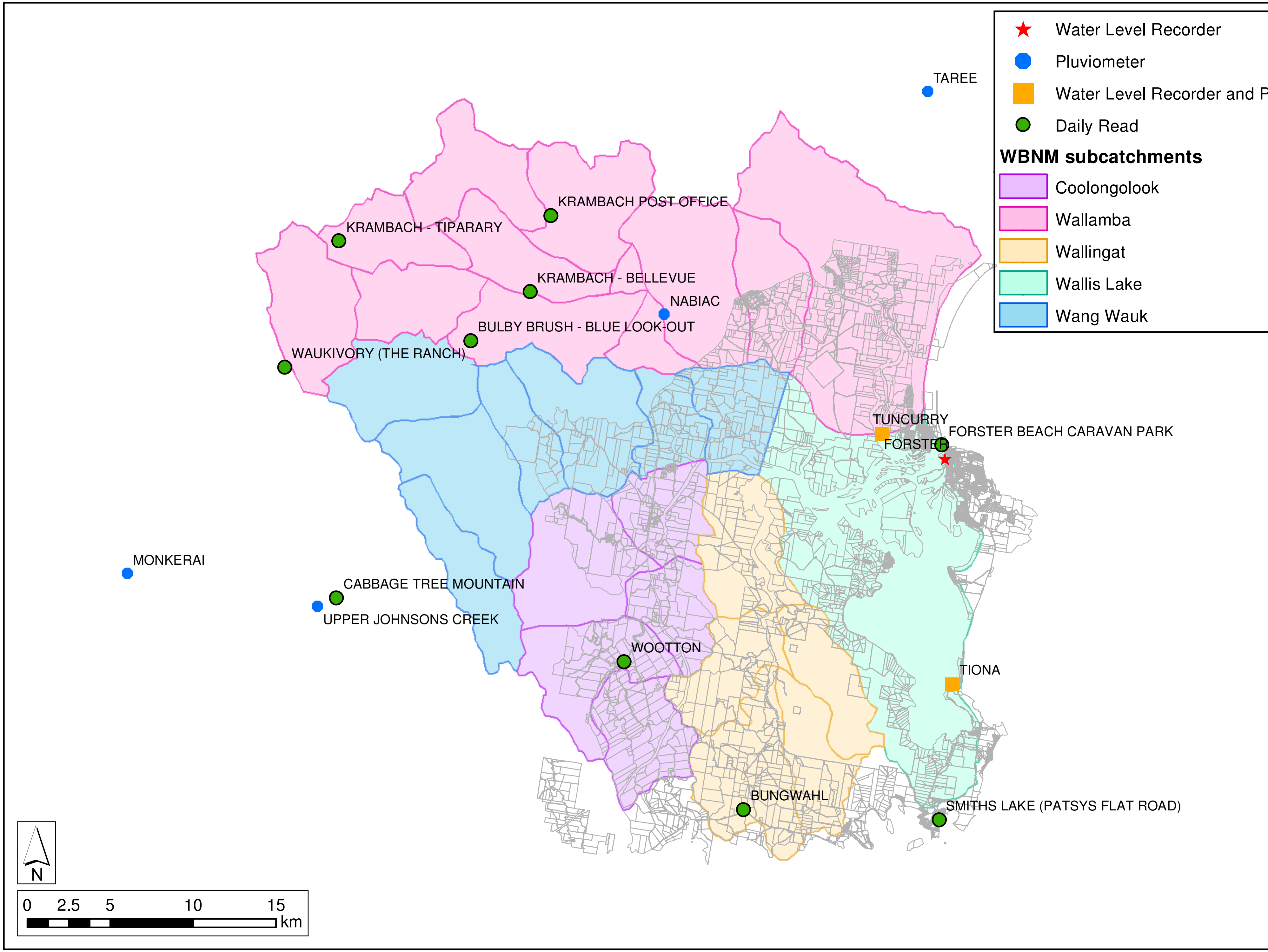

FIGURE 2
WALLIS LAKE
SUBCATCHMENT LAYOUT AND GAUGE LOCATIONS

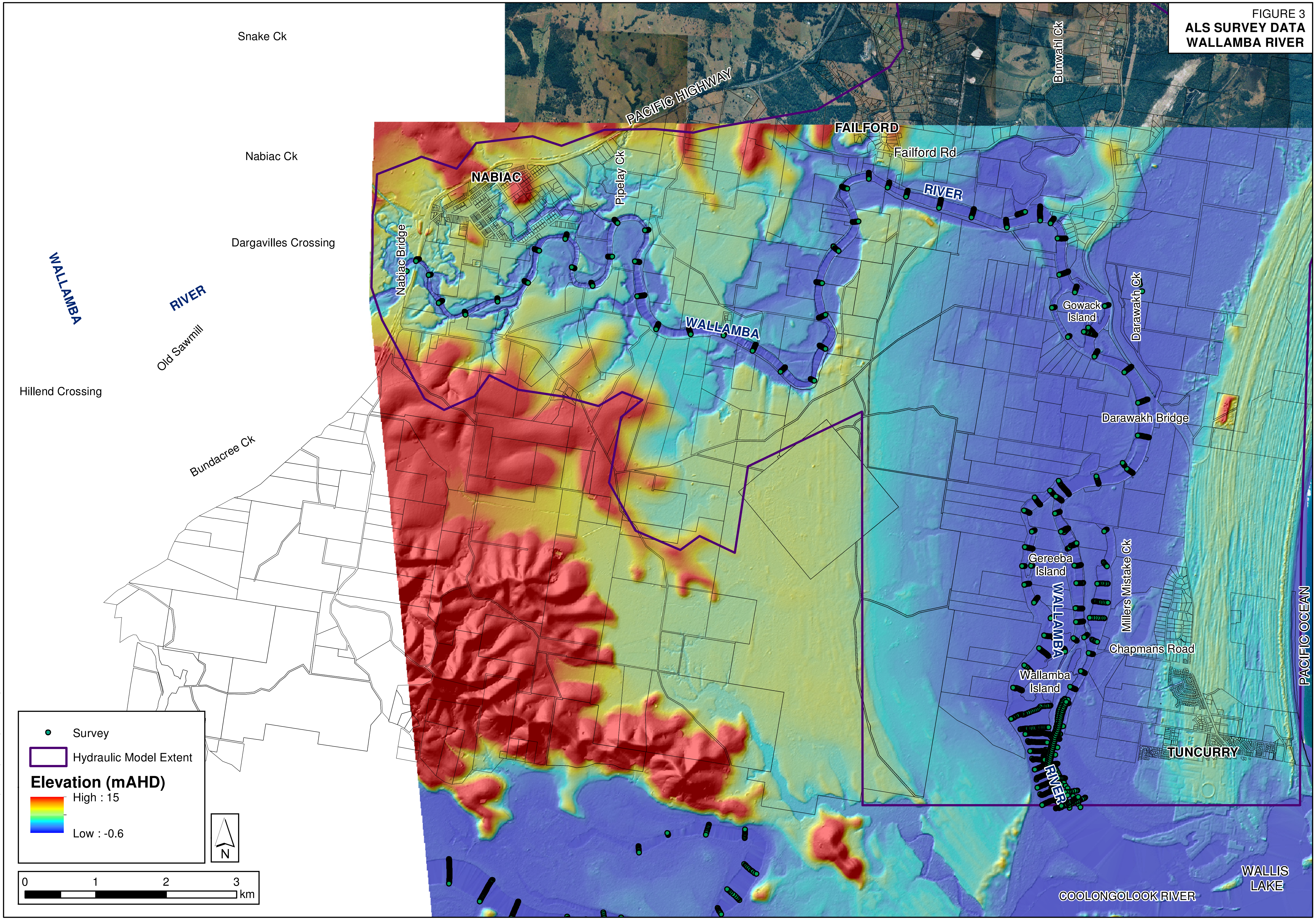

FIGURE 3
ALS SURVEY DATA
WALLAMBA RIVER
Snake Ck
Nabiac Ck
Dargavilles Crossing
WALLAMBA
RIVER
Old Sawmill
Hillend Crossing
Bundacree Ck
PACIFIC HIGHWAY
FAILFORD
Failford Rd
Bunwahl Ck
NABIAC
Pipelay Ck
Nabiac Bridge
RIVER
WALLAMBA
Gowack Island
Darawakh Ck
Darawakh Bridge
Gereeba Island
Millers Mistake Ck
WALLAMBA
Chapmans Road
Wallamba Island
RIVER
TUNCURRY
PACIFIC OCEAN
WALLIS LAKE
COOLONGOLOOK RIVER
Survey
Hydraulic Model Extent
Elevation (mAHD)
High : 15
Low : -0.6
N
0
1
2
3
km
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FIGURE 4
HISTORICAL FLOOD DATA
NABIAC
FAILFORD
TUNCURRY
1983 Flood Level
1978 Flood Level
1957 Flood Level
1947 Flood Level
1929 Flood Level
1927 Flood Level
N
0 1 2 3 km
Note: All historical flood locations are approximate
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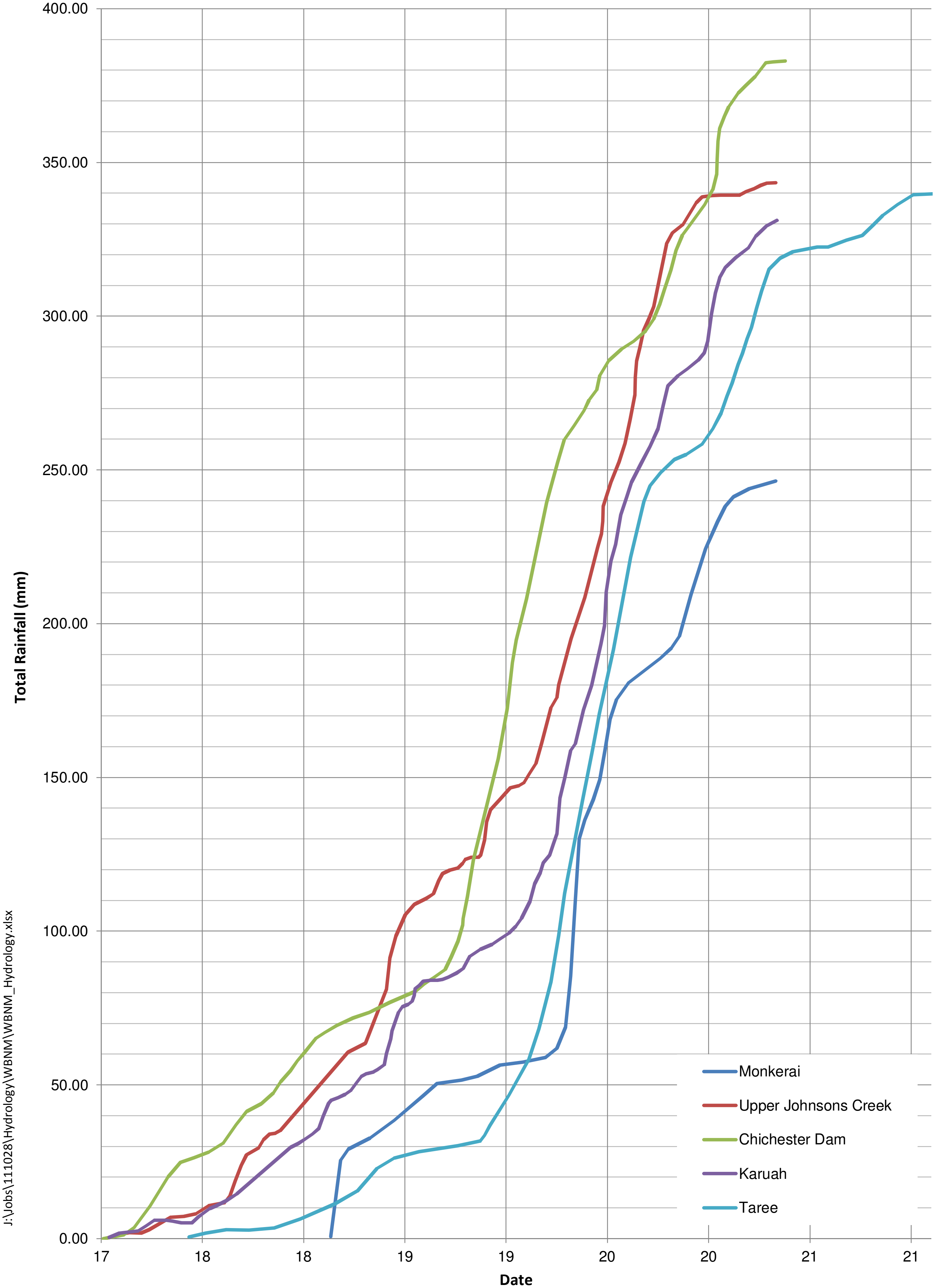
FIGURE 5
CUMULATIVE RAINFALL
MARCH 1978



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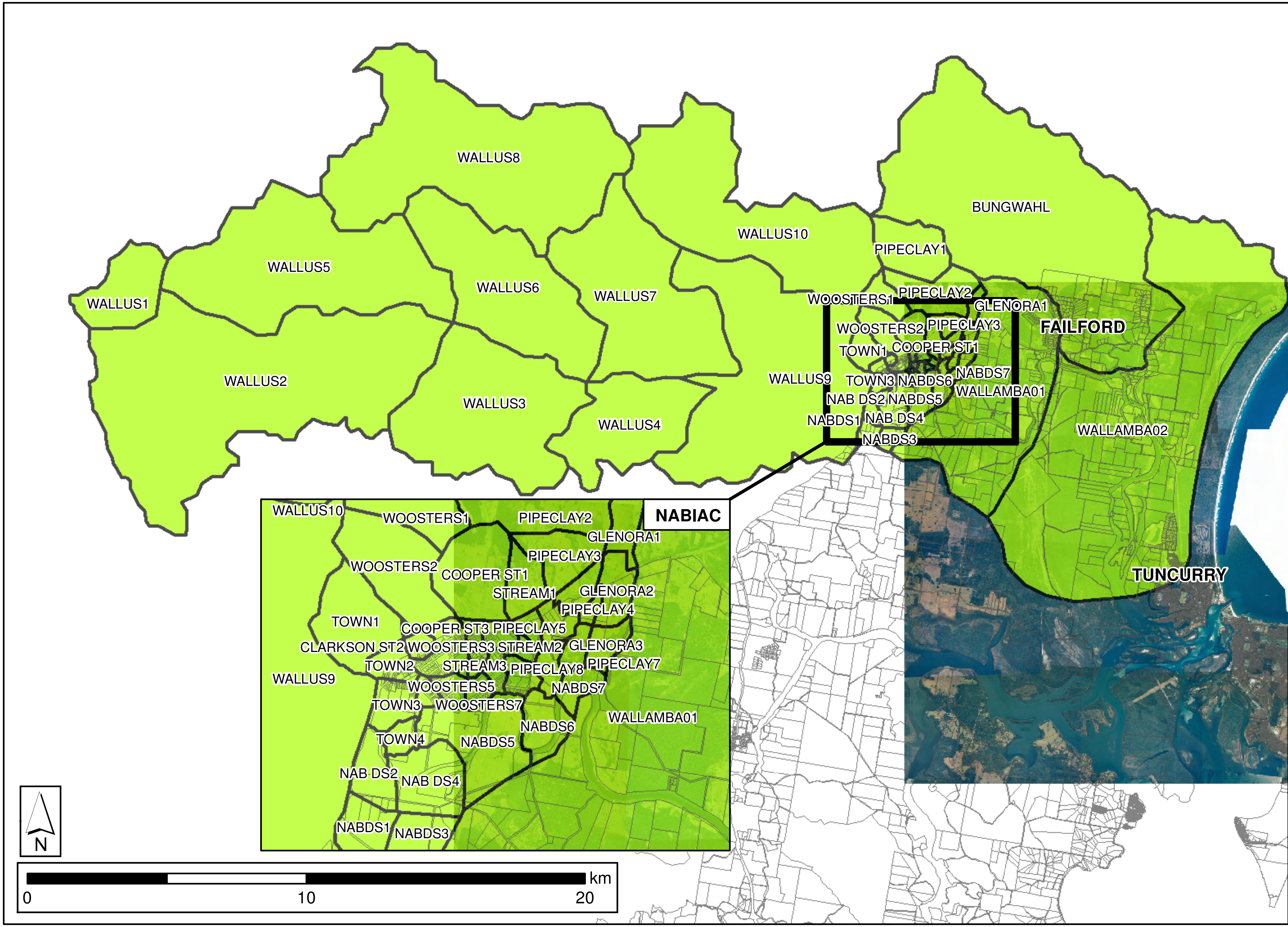
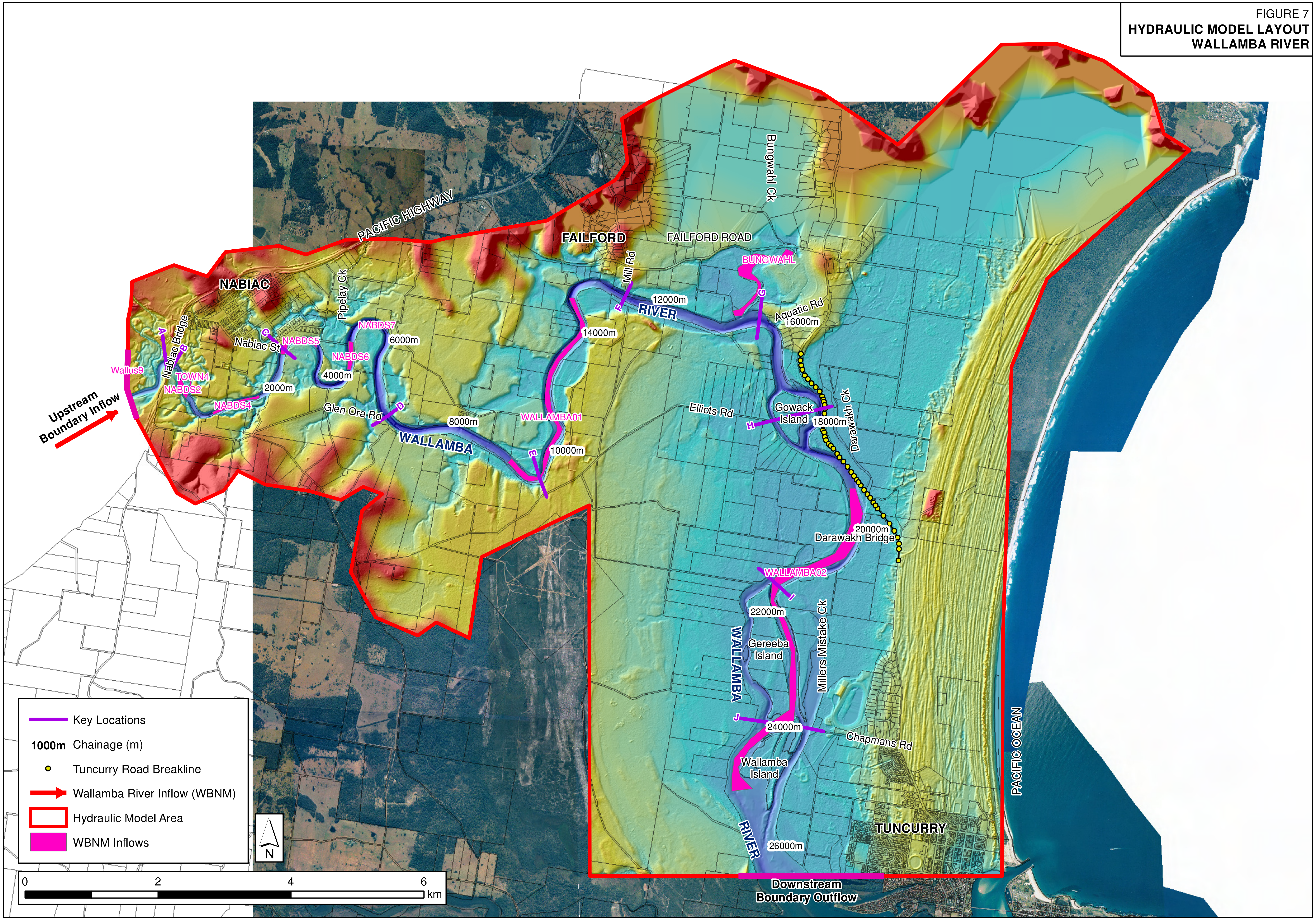


FIGURE 6
HYDROLOGIC MODEL LAYOUT
WALLAMBA RIVER

FIGURE 7
HYDRAULIC MODEL LAYOUT
WALLAMBA RIVER



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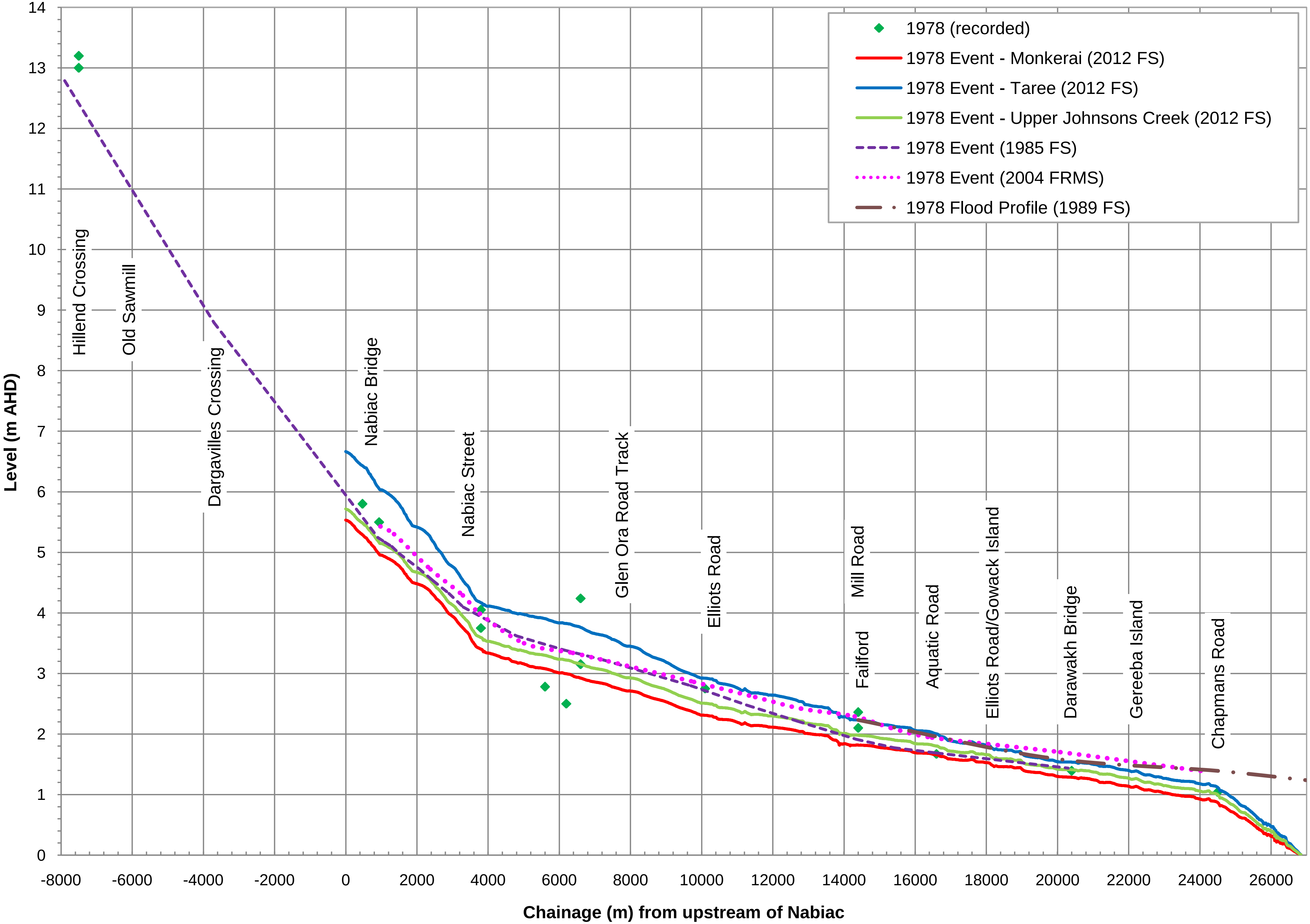

FIGURE 8
MARCH 1978 PEAK FLOOD PROFILE

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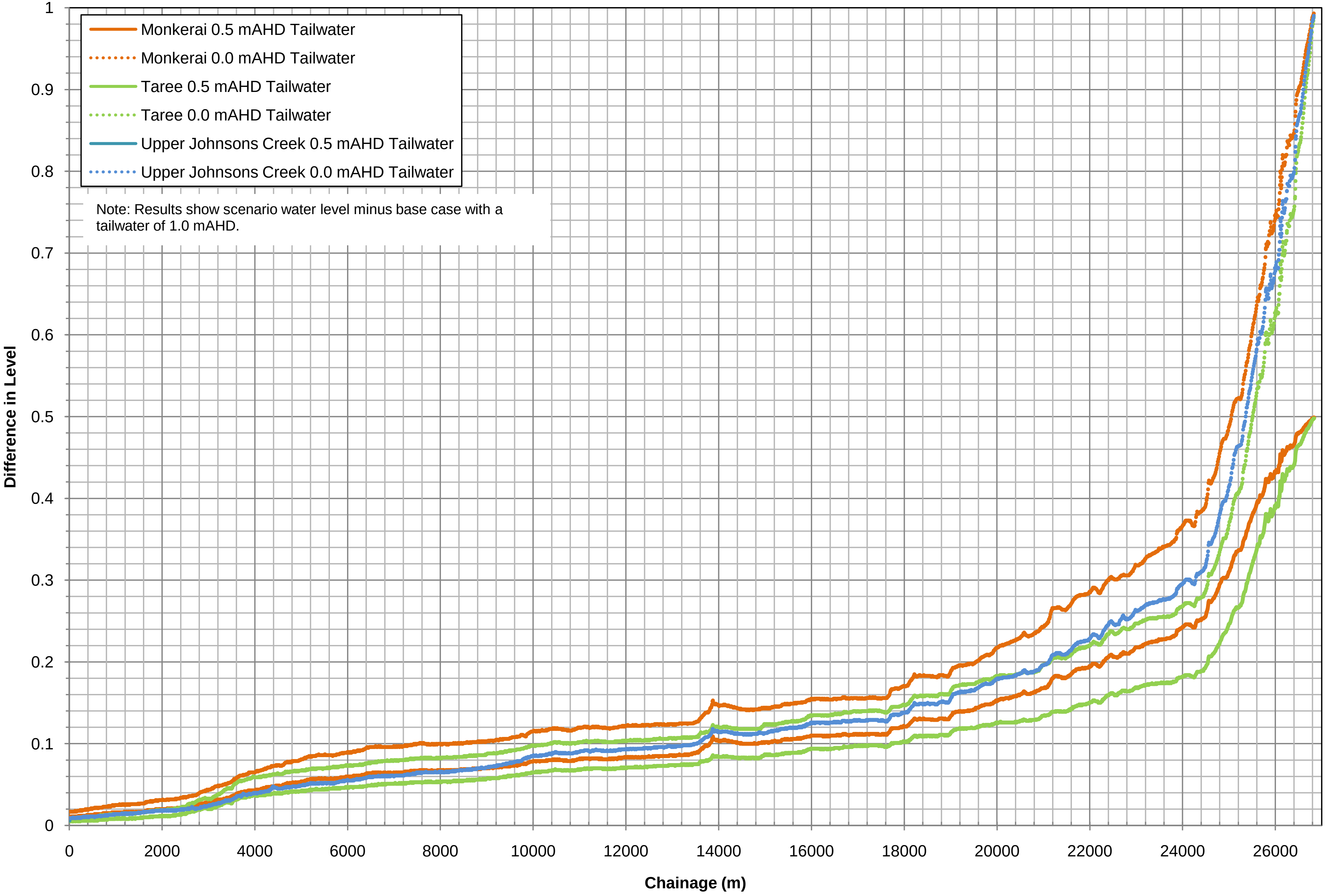

FIGURE 9
TAILWATER SENSITIVITY ANALYSIS
MARCH 1978

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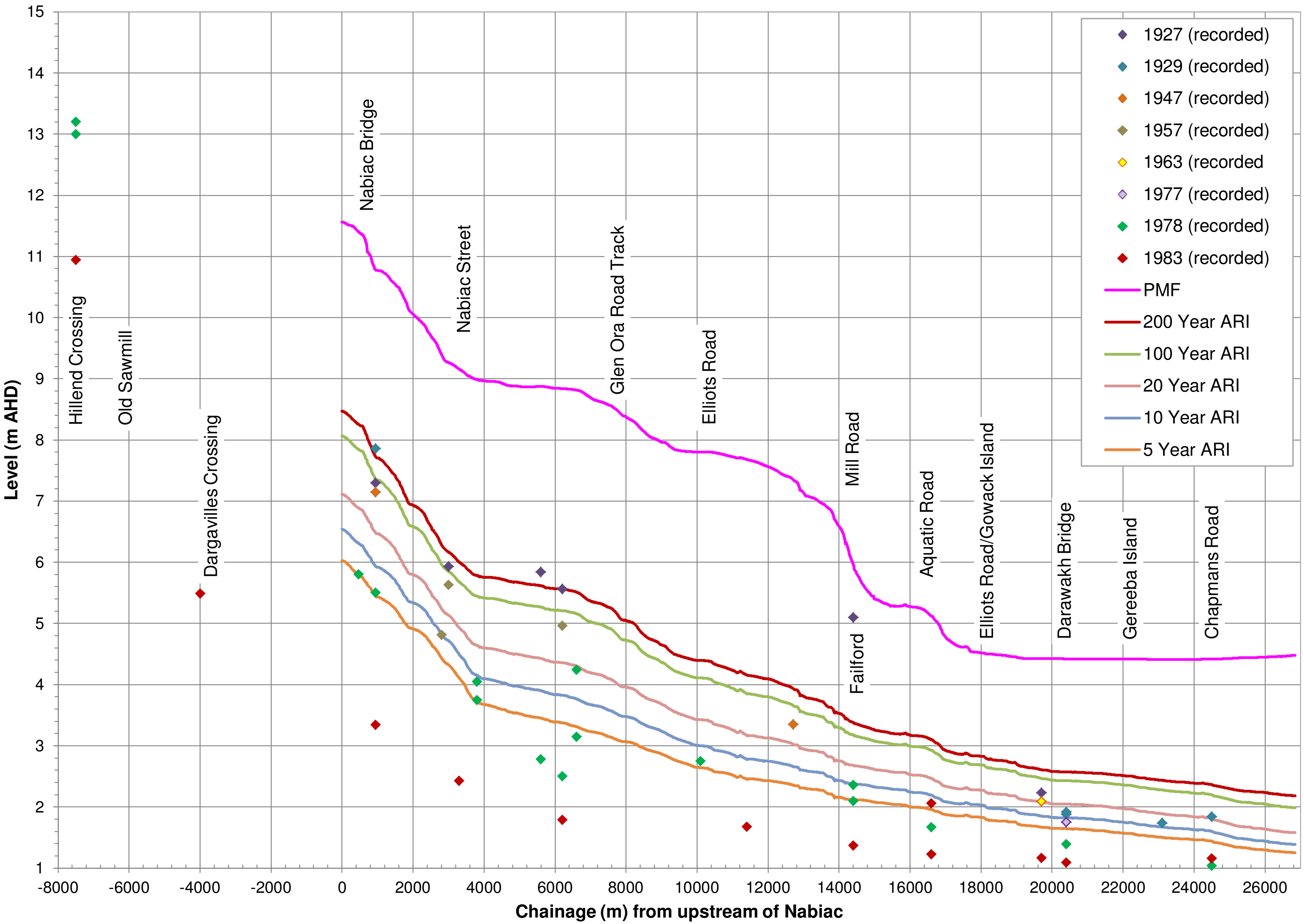

FIGURE 10A
DESIGN PEAK FLOOD PROFILES
COMPARISON TO HISTORIC LEVELS

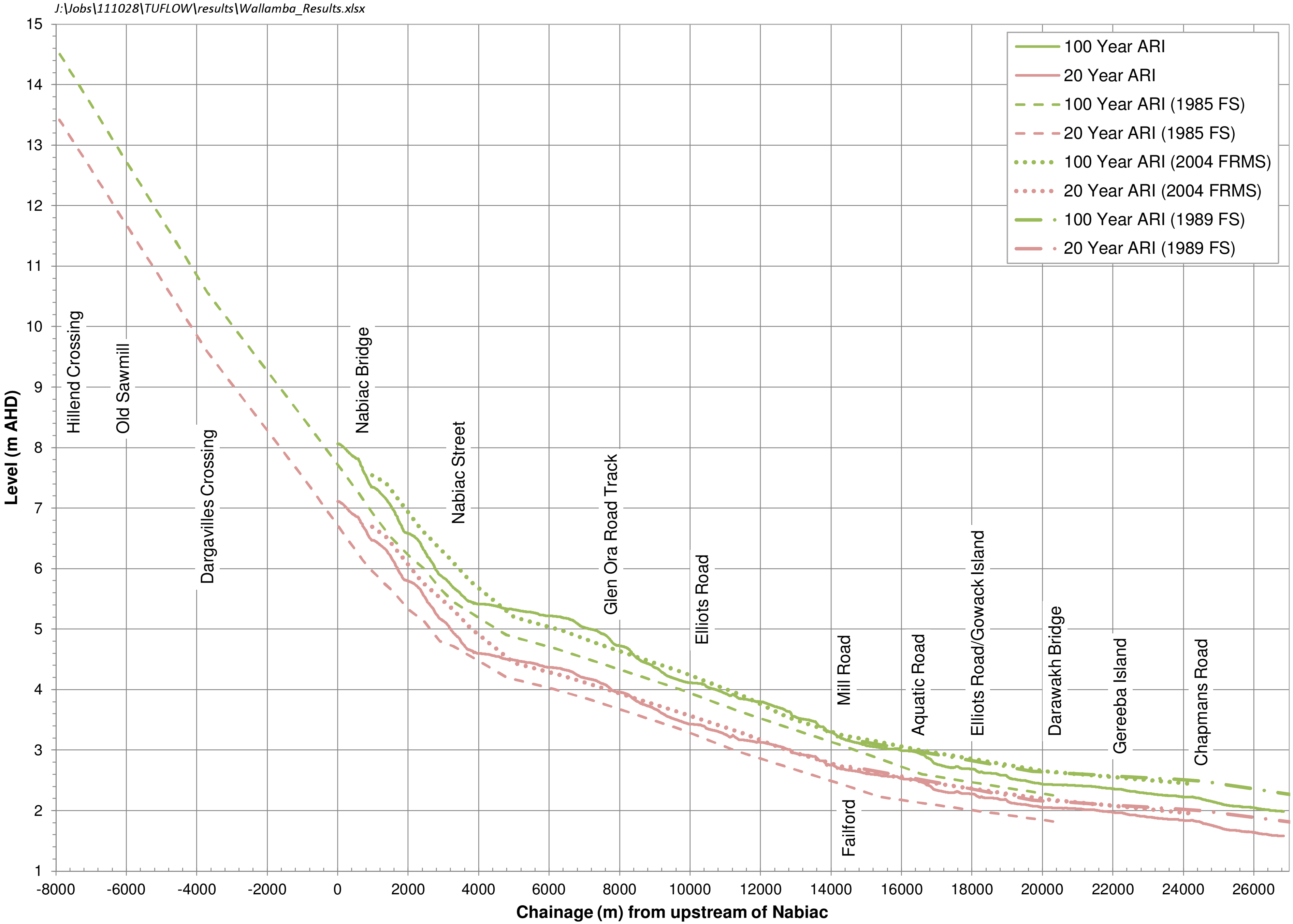

FIGURE 10B
DESIGN PEAK FLOOD PROFILES COMPARISON TO PREVIOUS STUDIES

FIGURE 11
CLASSIFICATION OF FLOODPLAIN TOPOGRAPHY

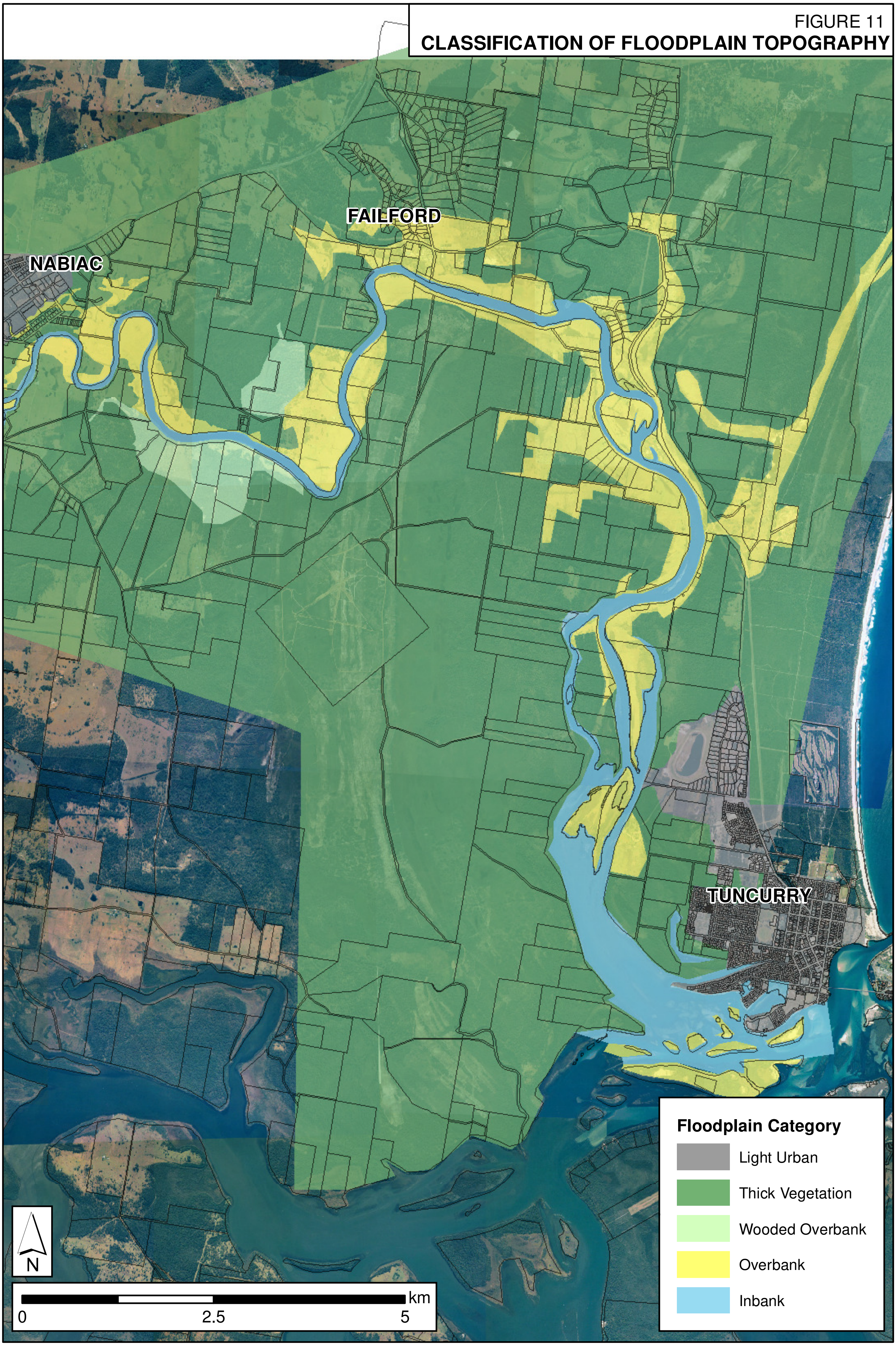

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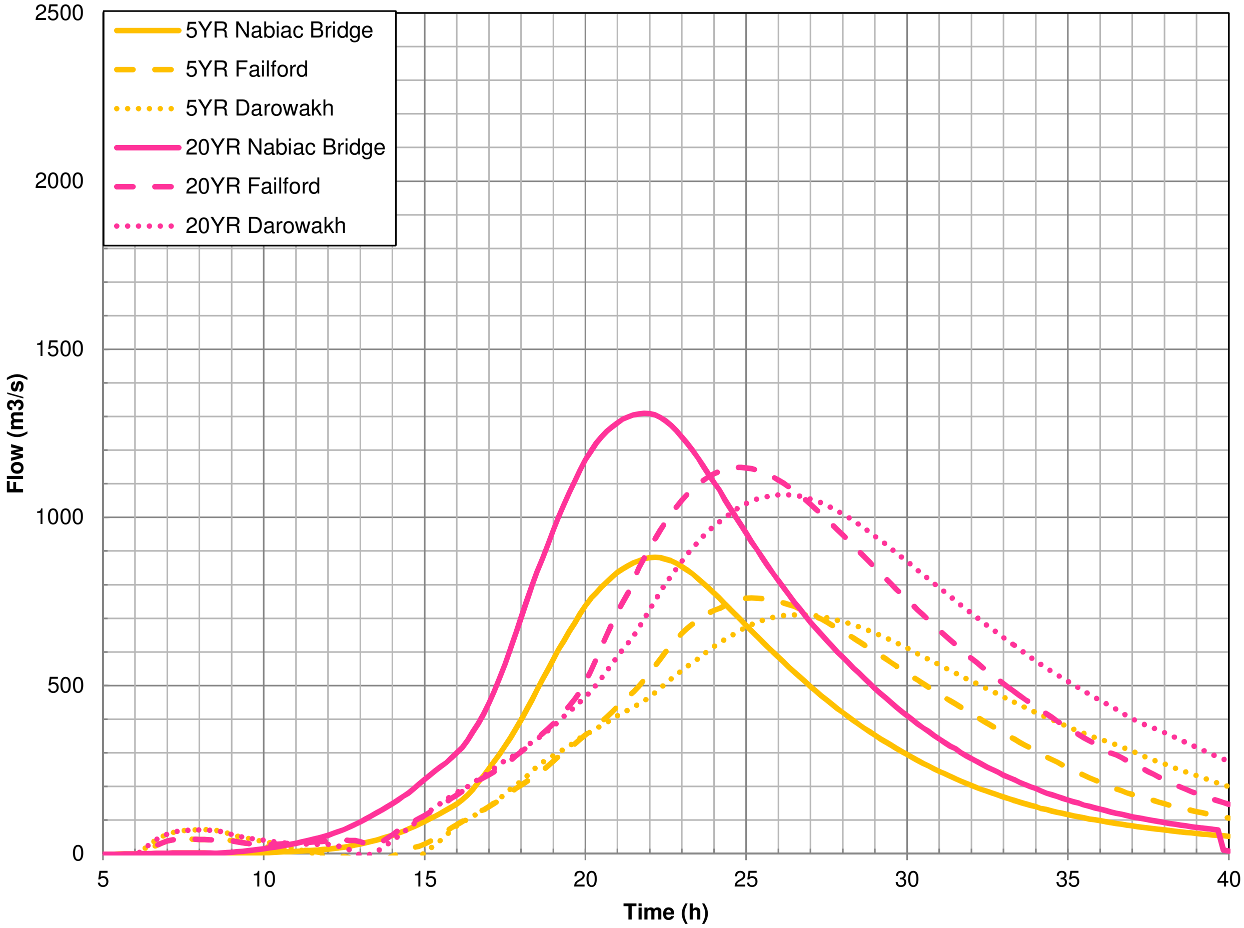
FIGURE 12A
DESIGN FLOOD HYDROGRAPHS



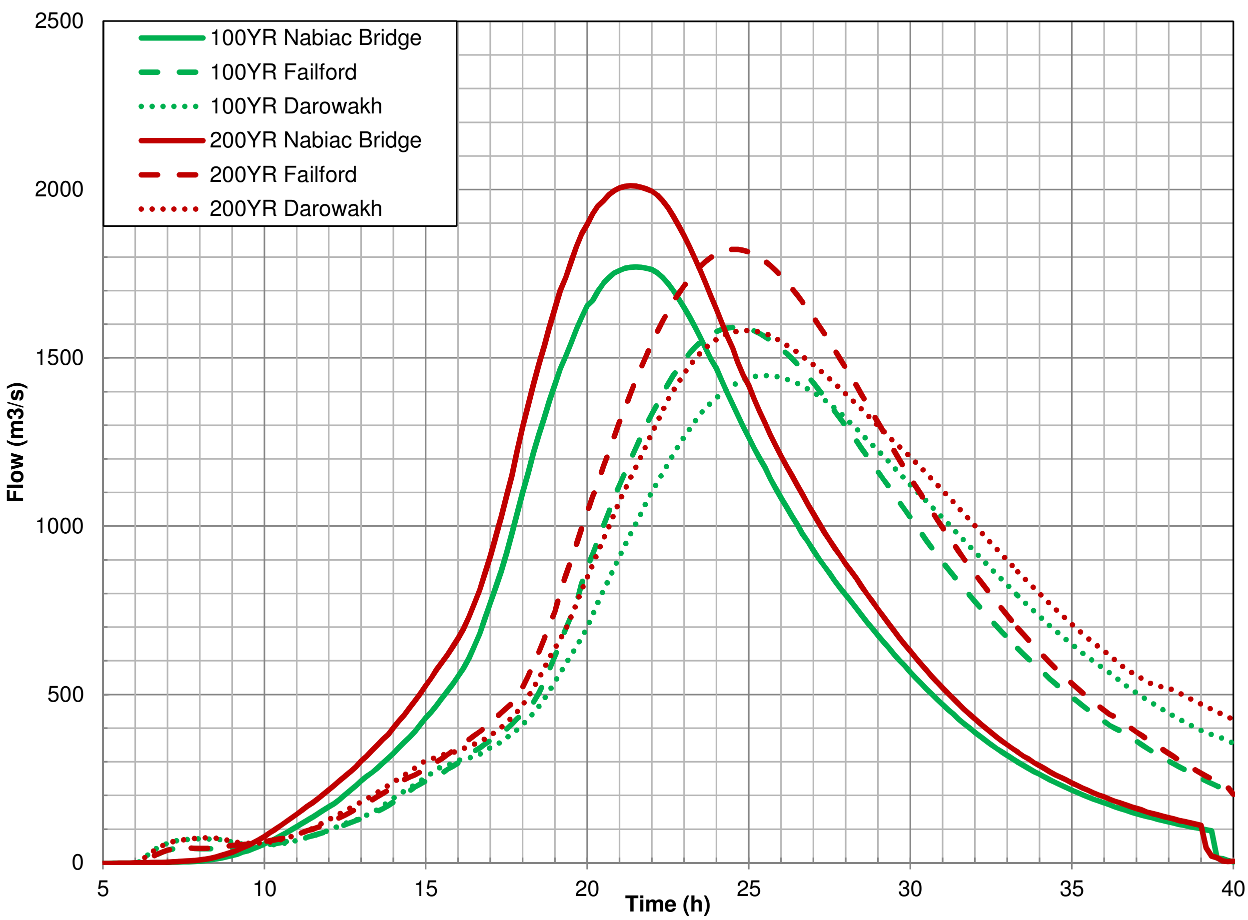


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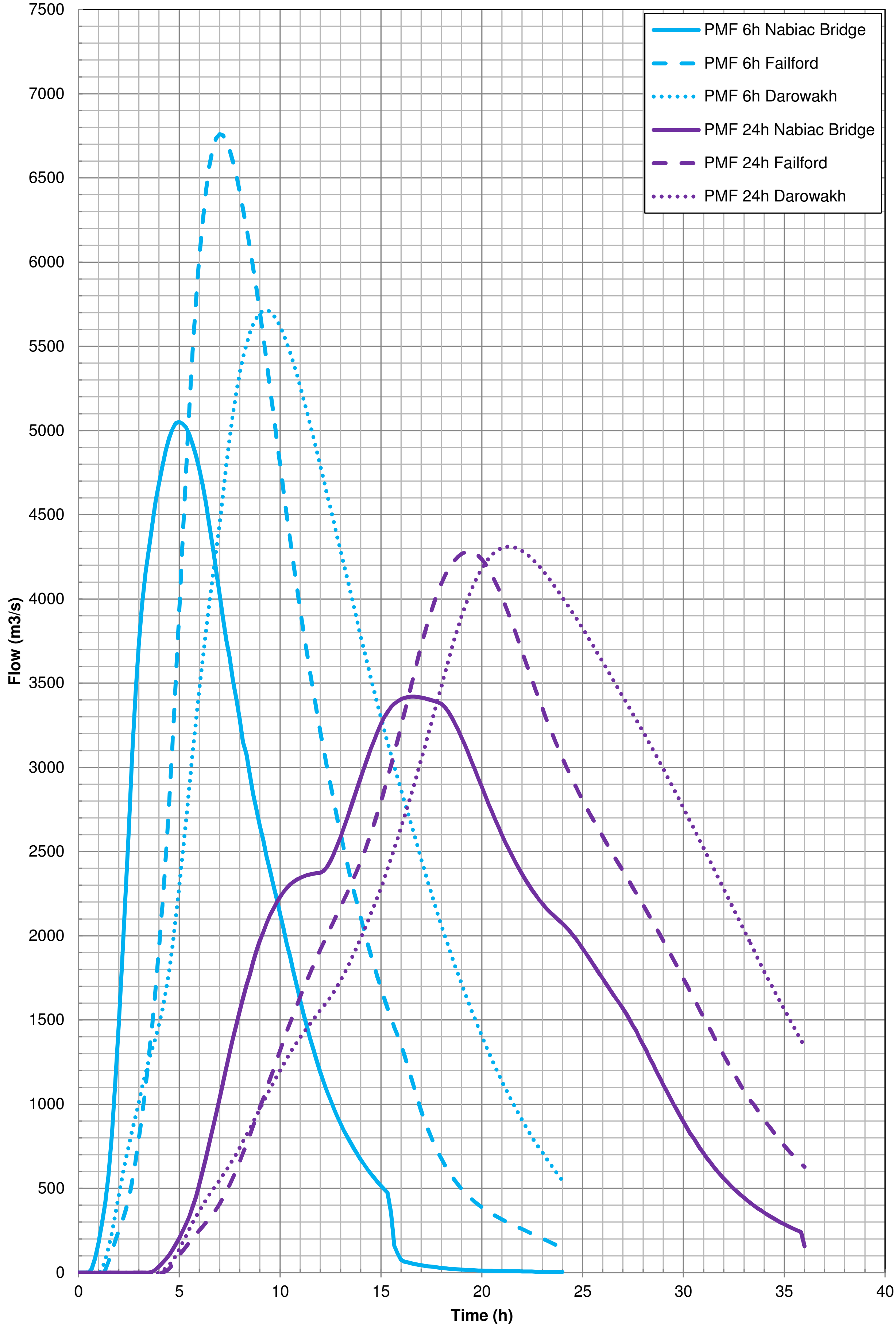
FIGURE 12B
DESIGN FLOOD HYDROGRAPHS



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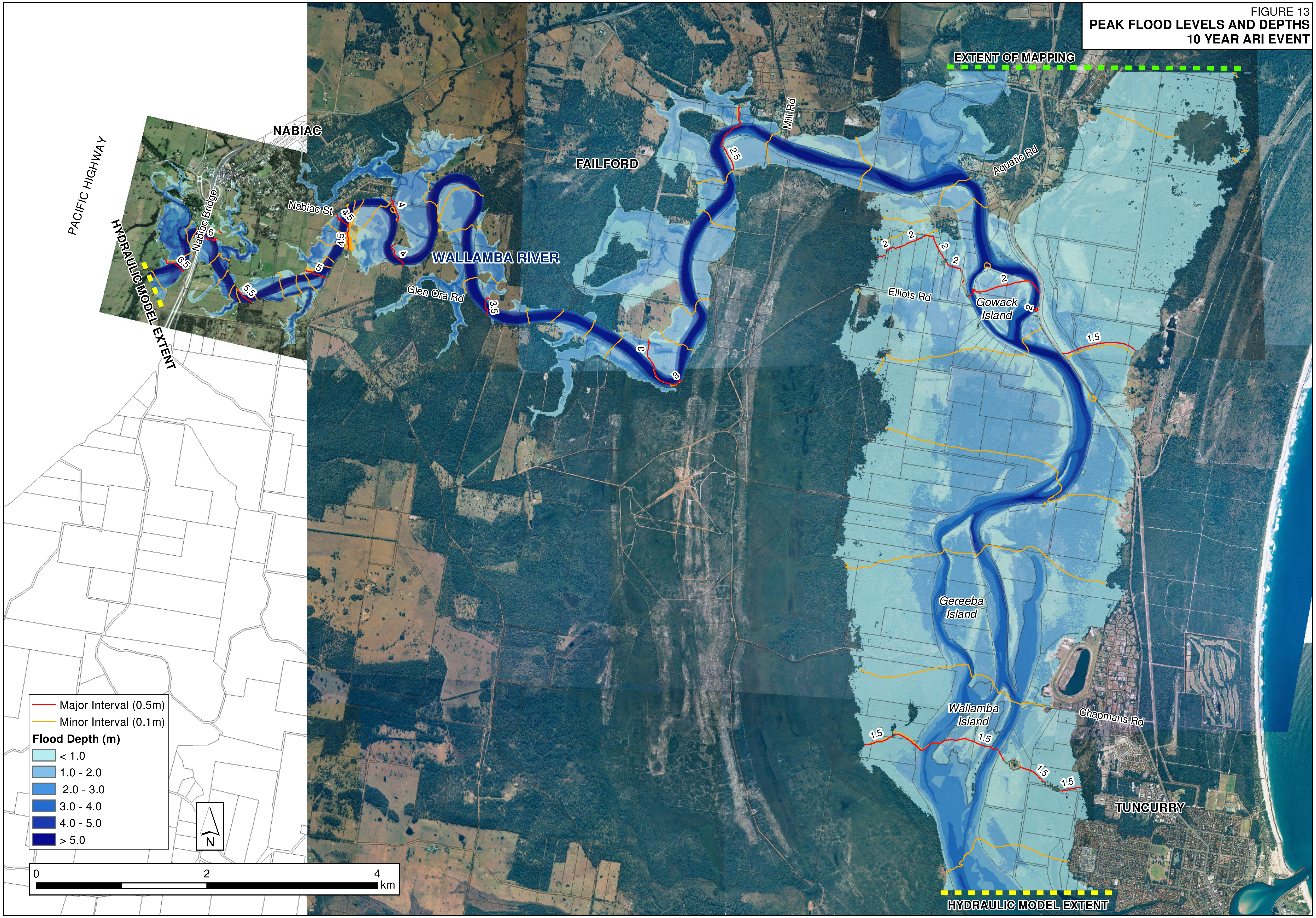

FIGURE 13
PEAK FLOOD LEVELS AND DEPTHS
10 YEAR ARI EVENT
EXTENT OF MAPPING
HYDRAULIC MODEL EXTENT
HYDRAULIC MODEL EXTENT
PACIFIC HIGHWAY
NABIAC
FAILFORD
WALLAMBA RIVER
Nabiac Bridge
Nabiac St
Glen Ora Rd
Mill Rd
Aquatic Rd
Elliots Rd
Gowack Island
Gereeba Island
Wallamba Island
Chapmans Rd
TUNCURRY
Major Interval (0.5m)
Minor Interval (0.1m)
Flood Depth (m)
< 1.0
1.0 - 2.0
2.0 - 3.0
3.0 - 4.0
4.0 - 5.0
> 5.0
N
0
2
4
km
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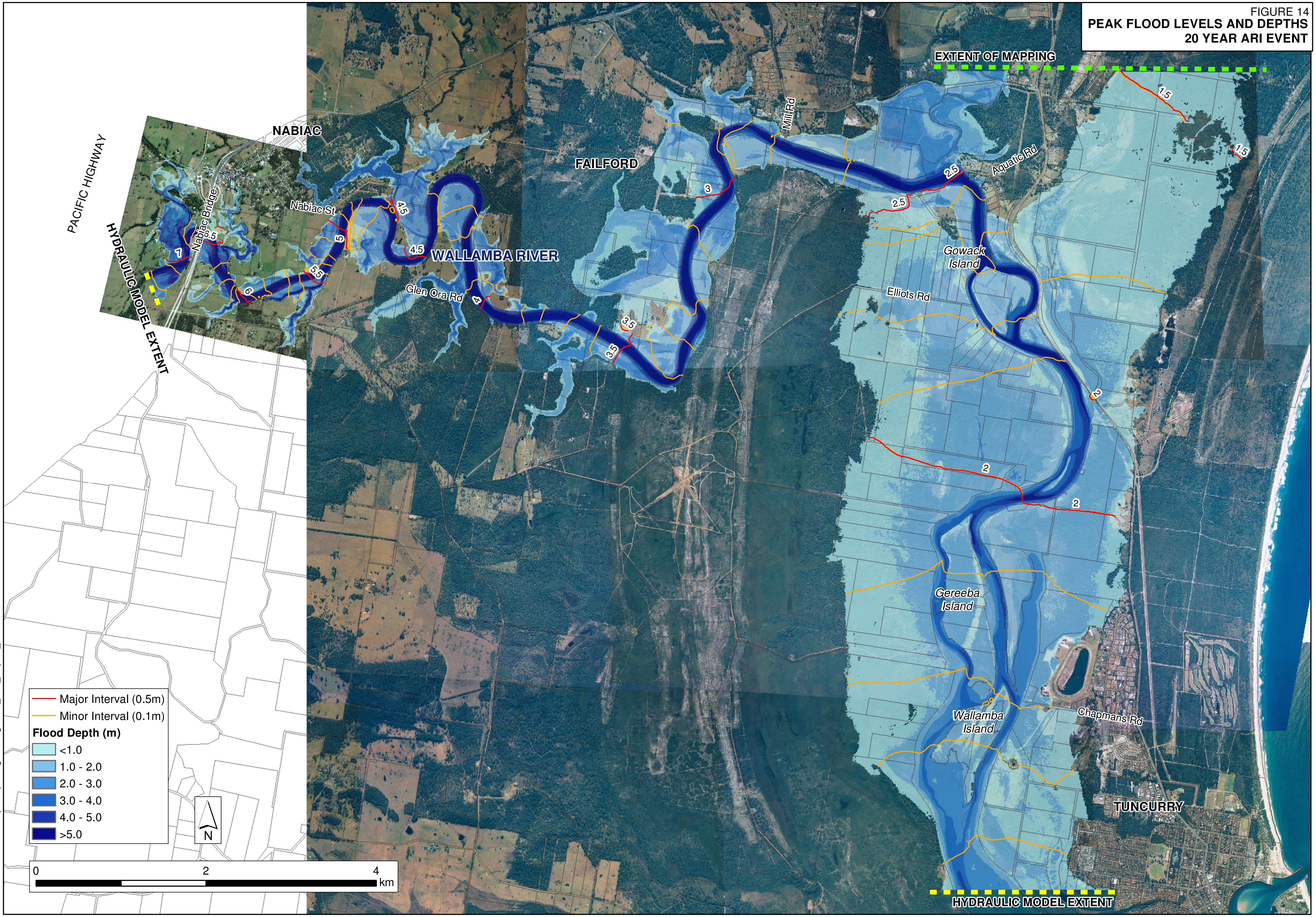
FIGURE 14
PEAK FLOOD LEVELS AND DEPTHS 20 YEAR ARI EVENT



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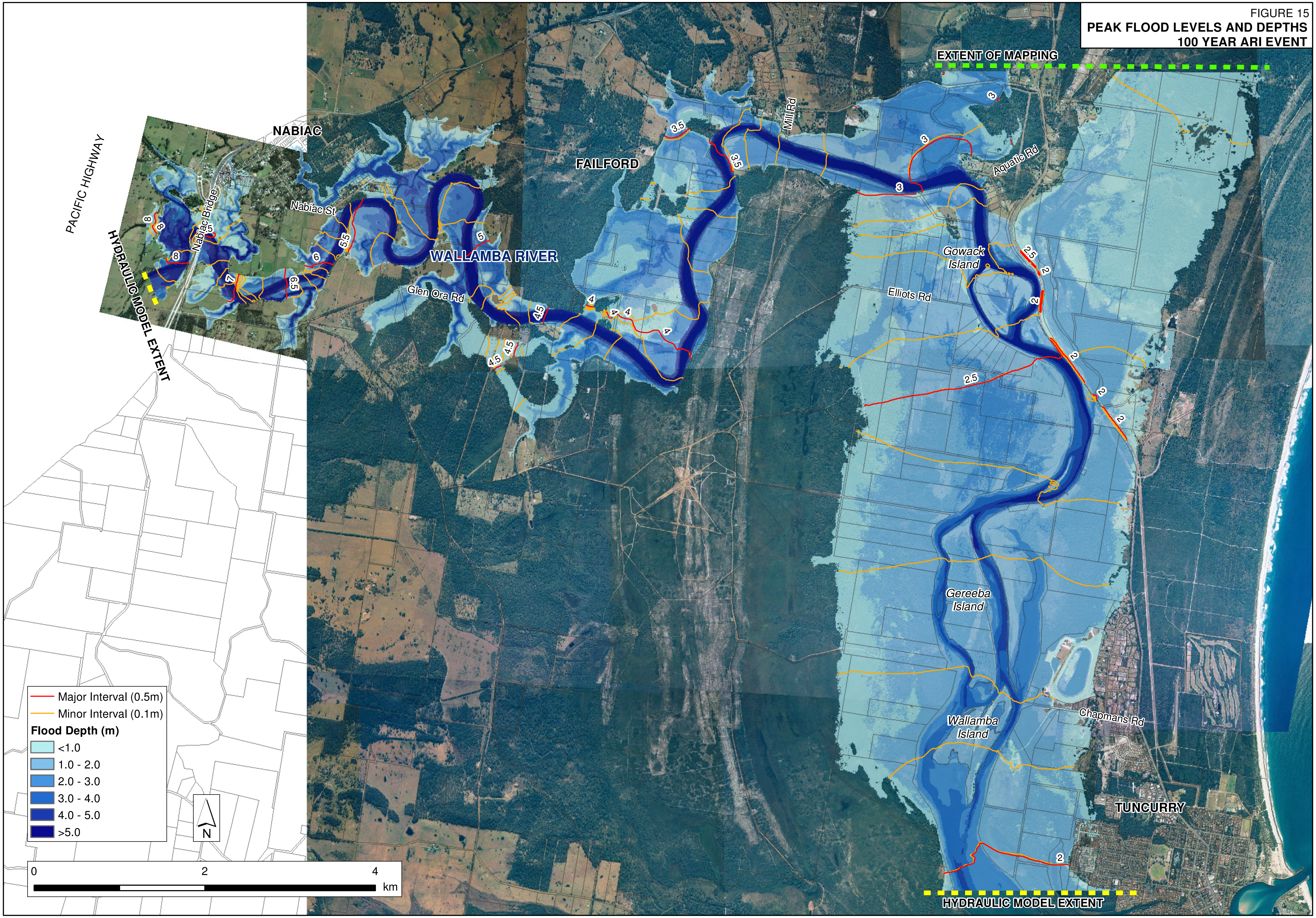

FIGURE 15
PEAK FLOOD LEVELS AND DEPTHS
100 YEAR ARI EVENT

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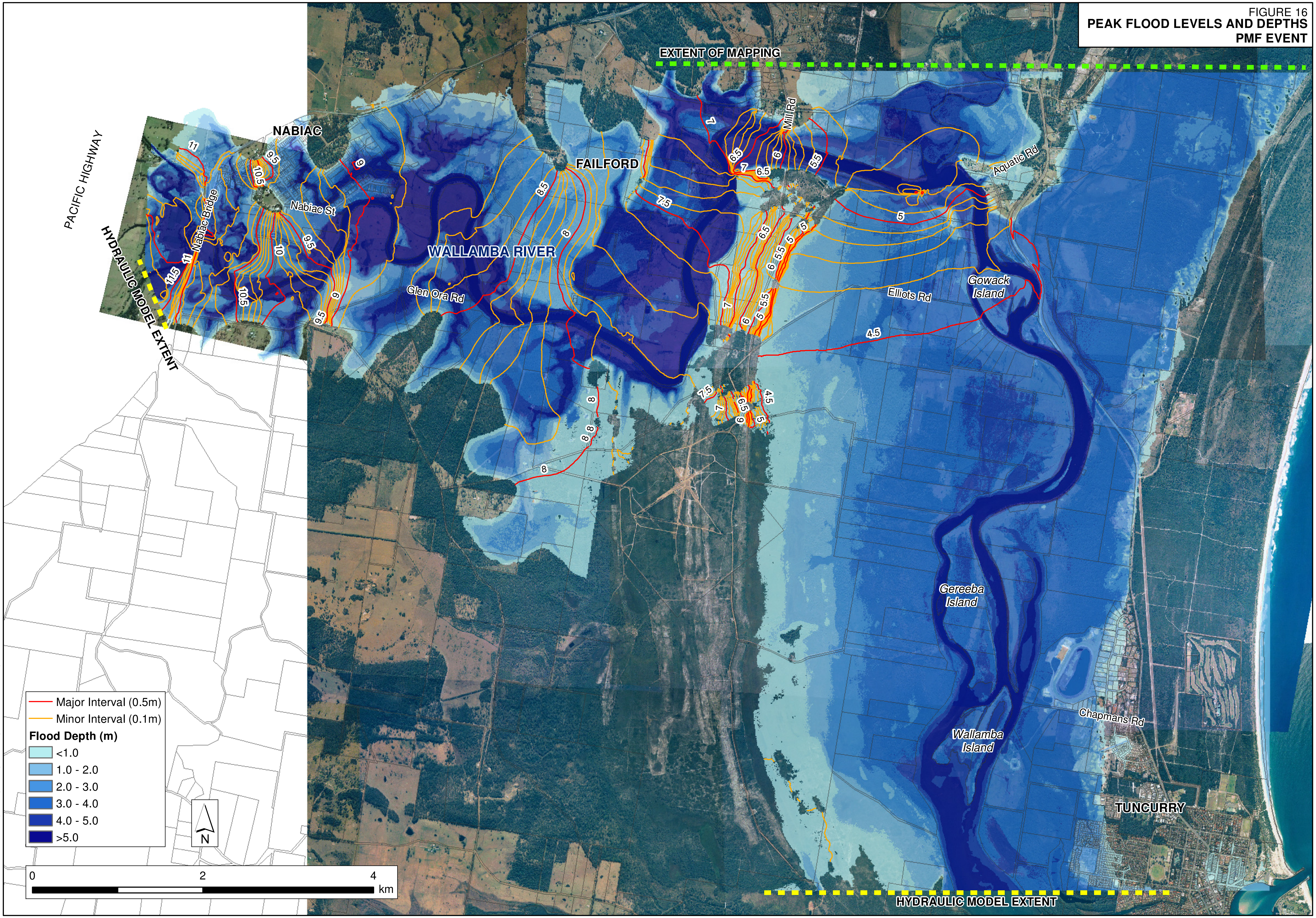
FIGURE 16
PEAK FLOOD LEVELS AND DEPTHS
PMF EVENT
EXTENT OF MAPPING
HYDRAULIC MODEL EXTENT
HYDRAULIC MODEL EXTENT
PACIFIC HIGHWAY
NABIAC
FAILFORD
TUNCURRY
WALLAMBA RIVER
Nabiac Bridge
Nabiac St
Glen Ora Rd
Mill Rd
Aquatic Rd
Elliots Rd
Chapmans Rd
Gowack Island
Gereeba Island
Wallamba Island
Major Interval (0.5m)
Minor Interval (0.1m)
Flood Depth (m)
<1.0
1.0 - 2.0
2.0 - 3.0
3.0 - 4.0
4.0 - 5.0
>5.0
N
0
2
4
km
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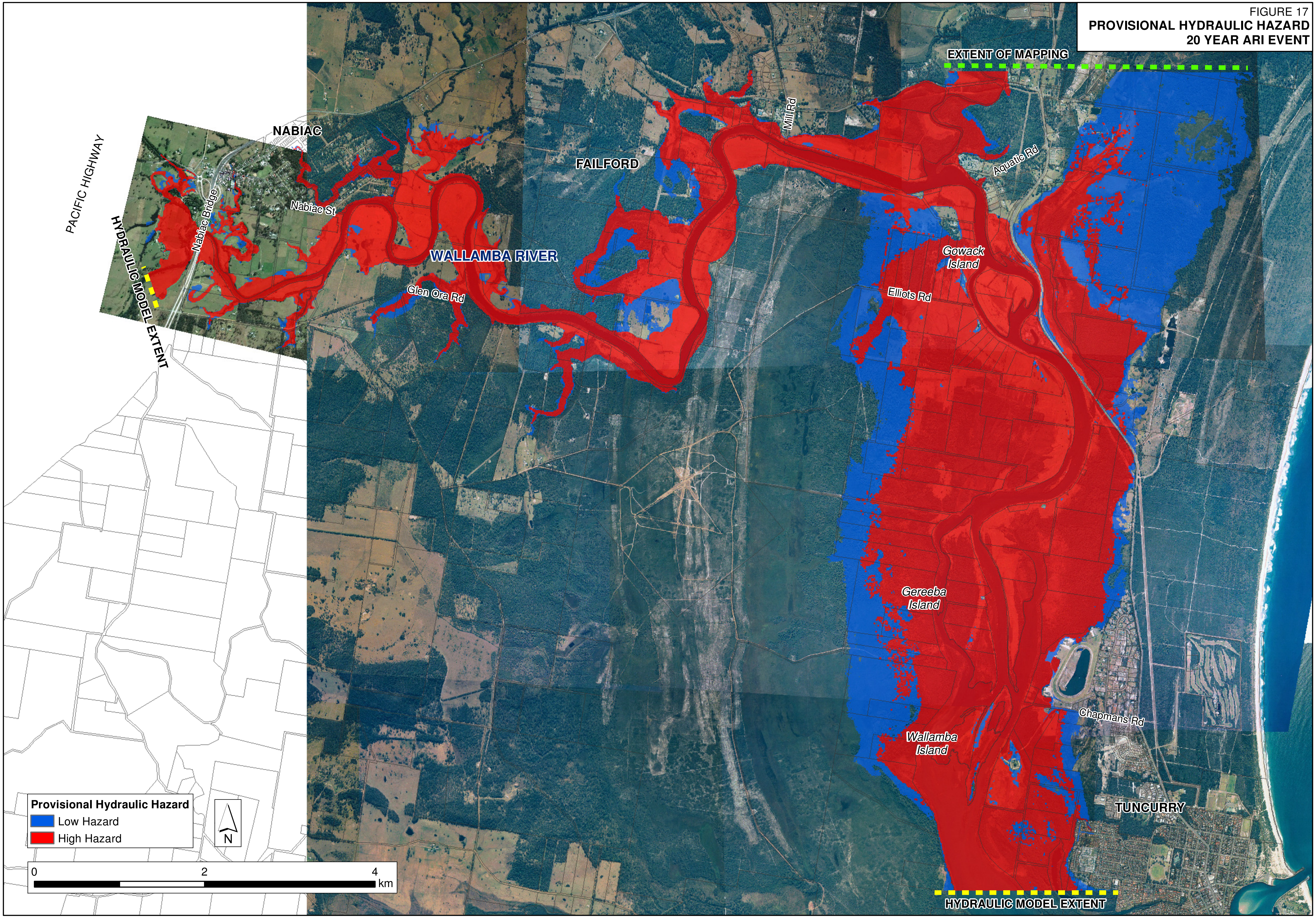
FIGURE 17
**PROVISIONAL HYDRAULIC HAZARD
20 YEAR ARI EVENT**



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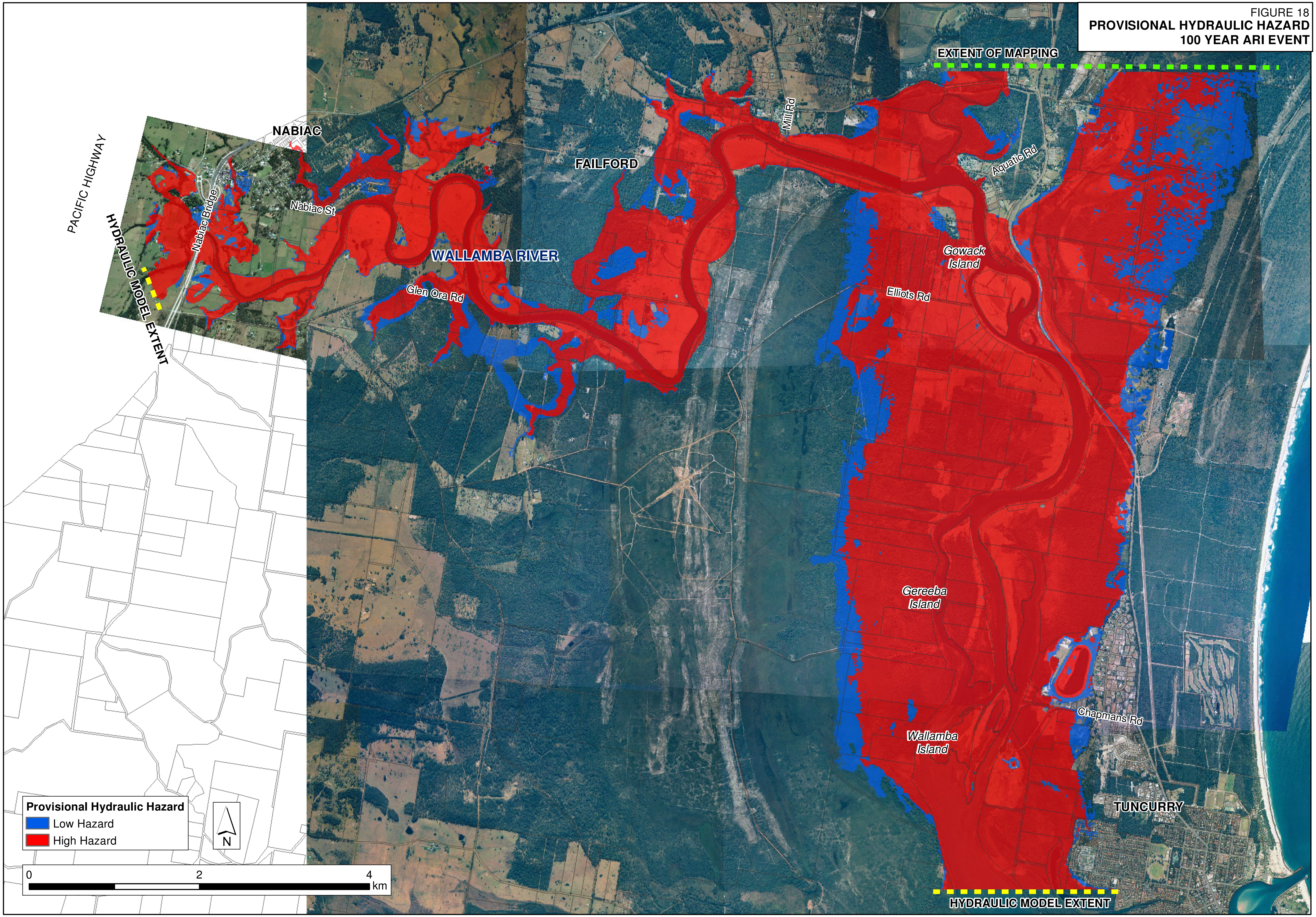

FIGURE 18
PROVISIONAL HYDRAULIC HAZARD
100 YEAR ARI EVENT

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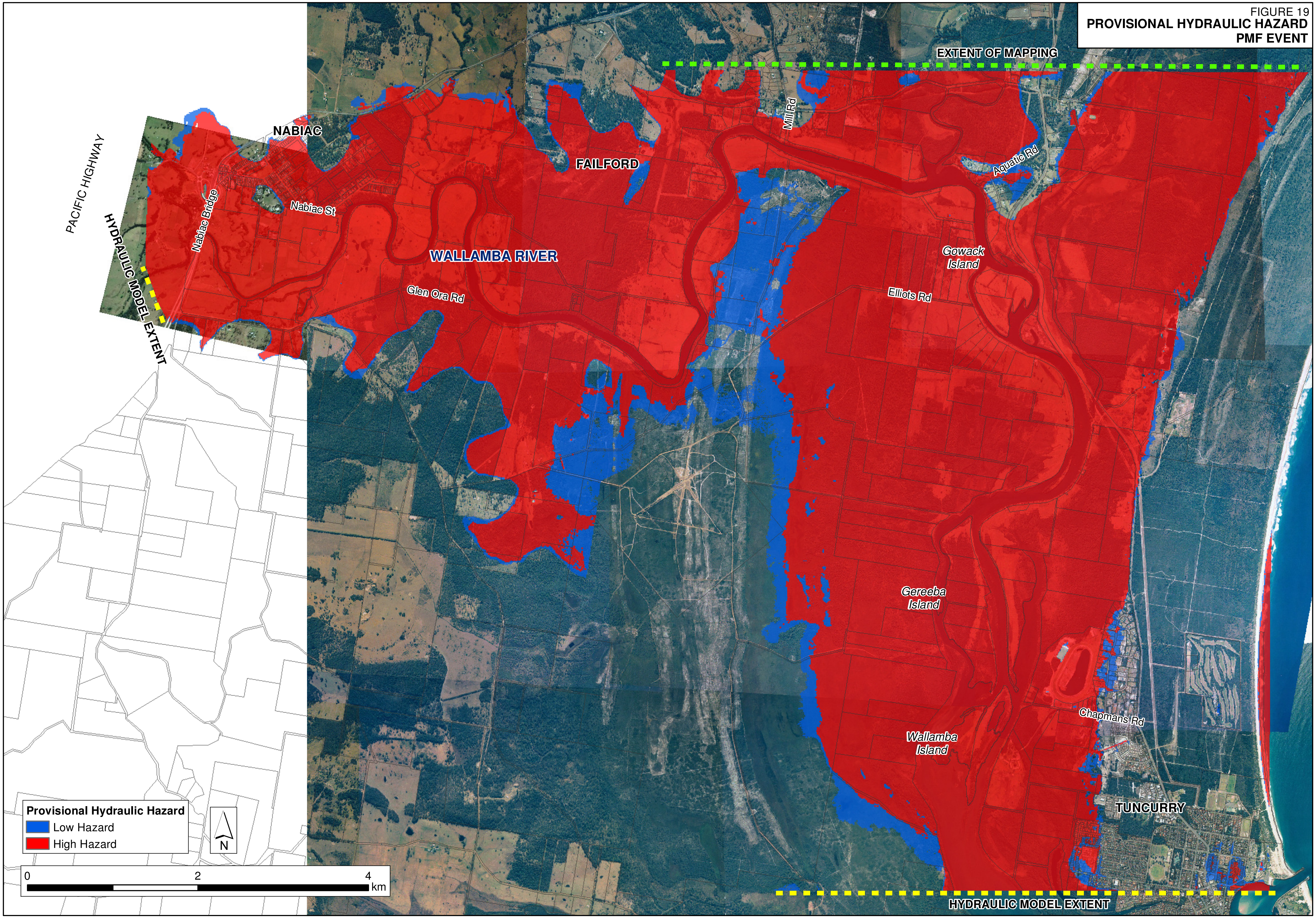

FIGURE 19
PROVISIONAL HYDRAULIC HAZARD
PMF EVENT

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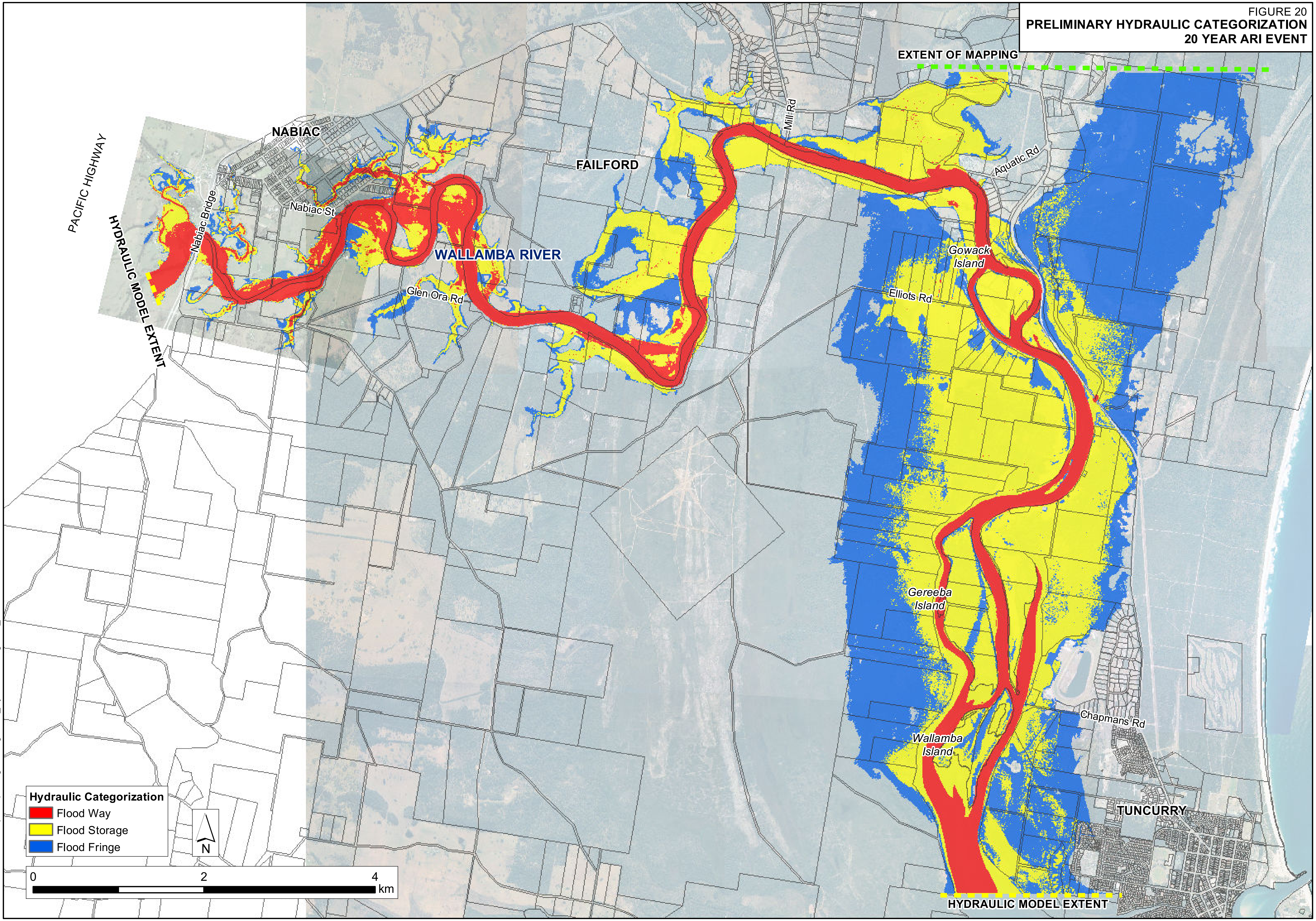
FIGURE 20
PRELIMINARY HYDRAULIC CATEGORIZATION
20 YEAR ARI EVENT
EXTENT OF MAPPING
NABIAC
FAILFORD
PACIFIC HIGHWAY
HYDRAULIC MODEL EXTENT
Nabiac Bridge
Nabiac St
WALLAMBA RIVER
Glen Ora Rd
Mill Rd
Aquatic Rd
Gowack Island
Elliots Rd
Gereeba Island
Chapmans Rd
Wallamba Island
TUNCURRY
HYDRAULIC MODEL EXTENT
Hydraulic Categorization
Flood Way
Flood Storage
Flood Fringe
N
0
2
4
km
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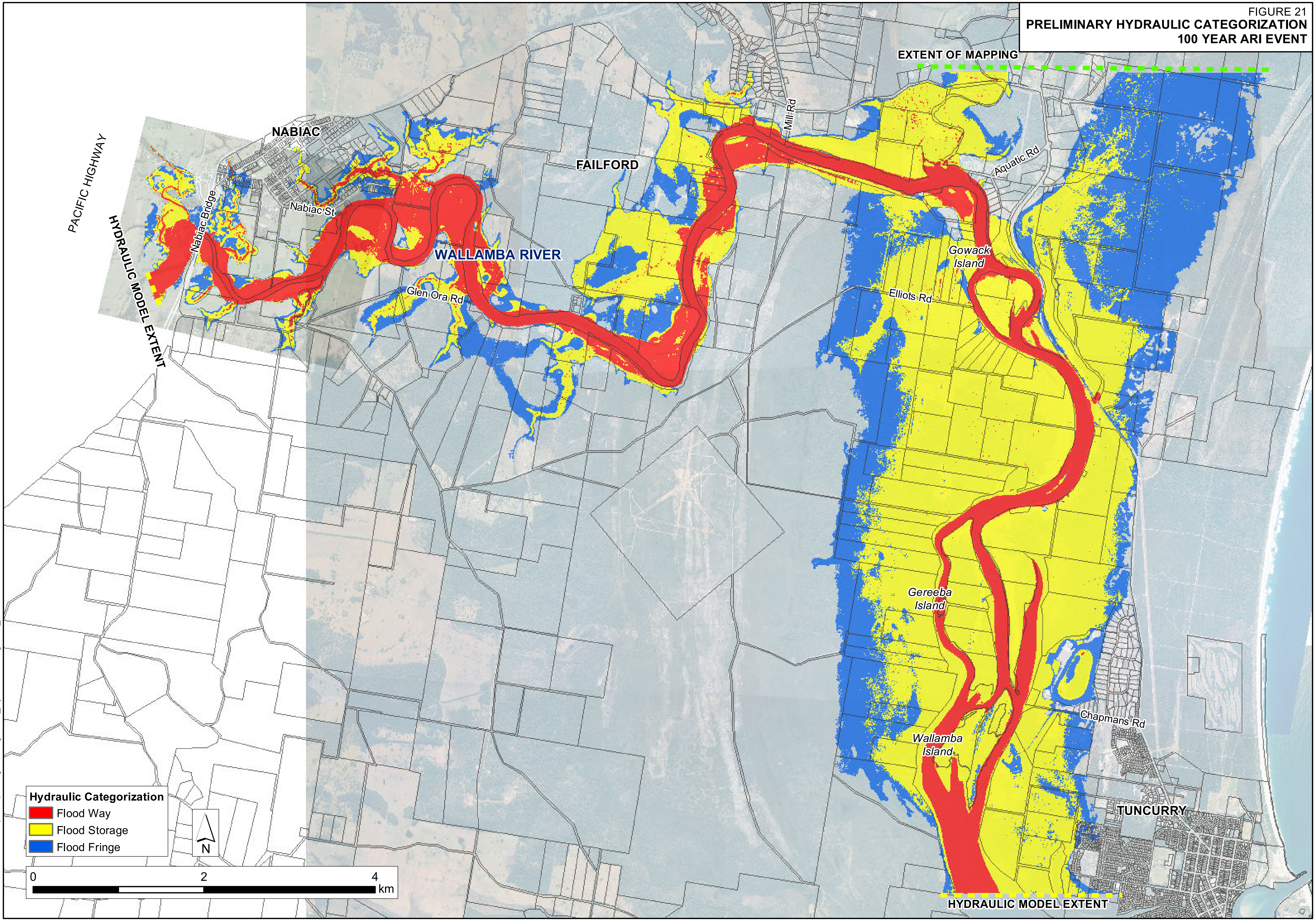

FIGURE 21
PRELIMINARY HYDRAULIC CATEGORIZATION
100 YEAR ARI EVENT
EXTENT OF MAPPING
PACIFIC HIGHWAY
HYDRAULIC MODEL EXTENT
NABIAC
Nabiac Bridge
Nabiac St
FAILFORD
Mill Rd
Aquatic Rd
WALLAMBA RIVER
Glen Ora Rd
Gowack Island
Elliots Rd
Gereeba Island
Chapmans Rd
Wallamba Island
TUNCURRY
HYDRAULIC MODEL EXTENT
Hydraulic Categorization
Flood Way
Flood Storage
Flood Fringe
N
0
2
4
km
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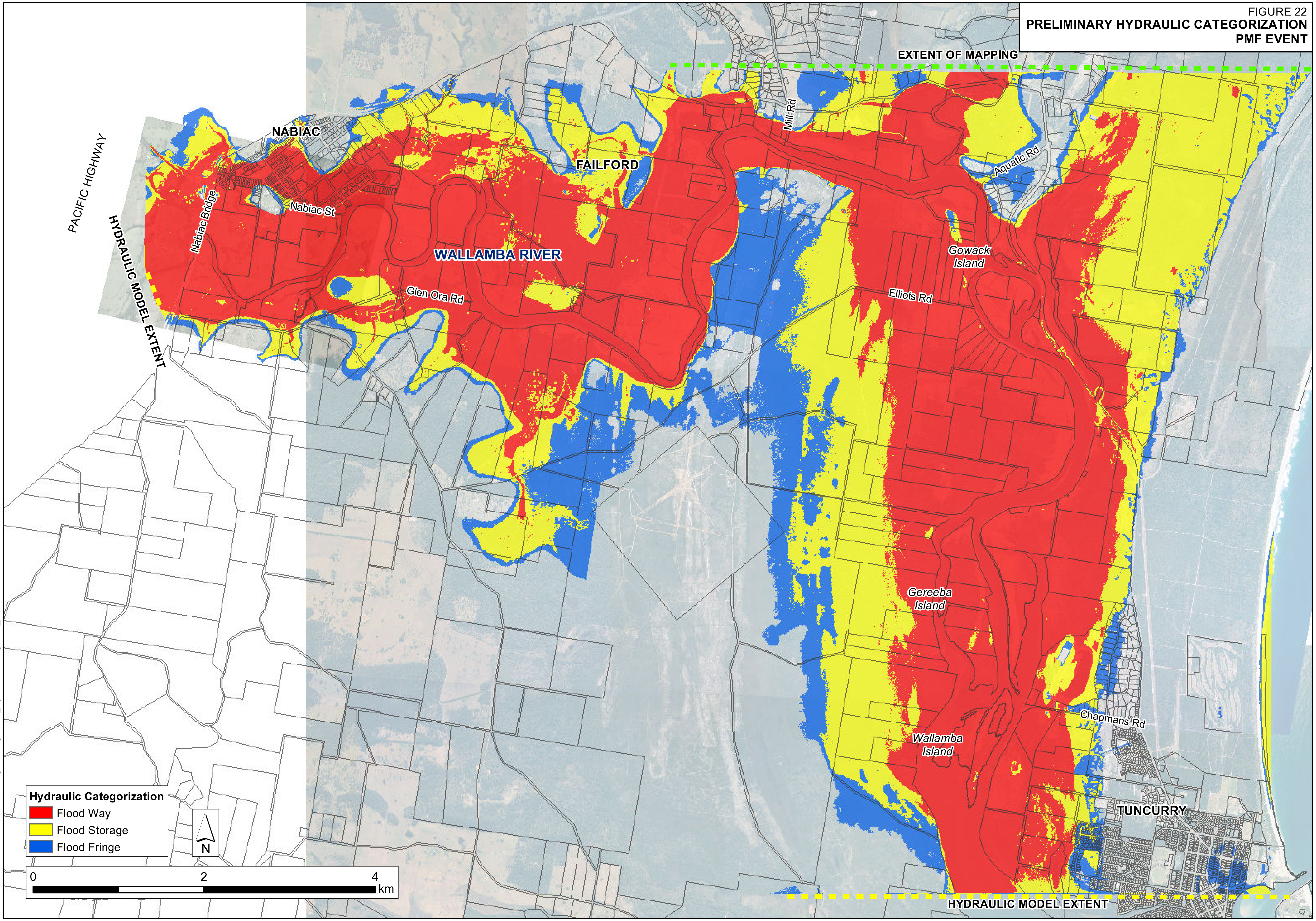
FIGURE 22
PRELIMINARY HYDRAULIC CATEGORIZATION PMF EVENT



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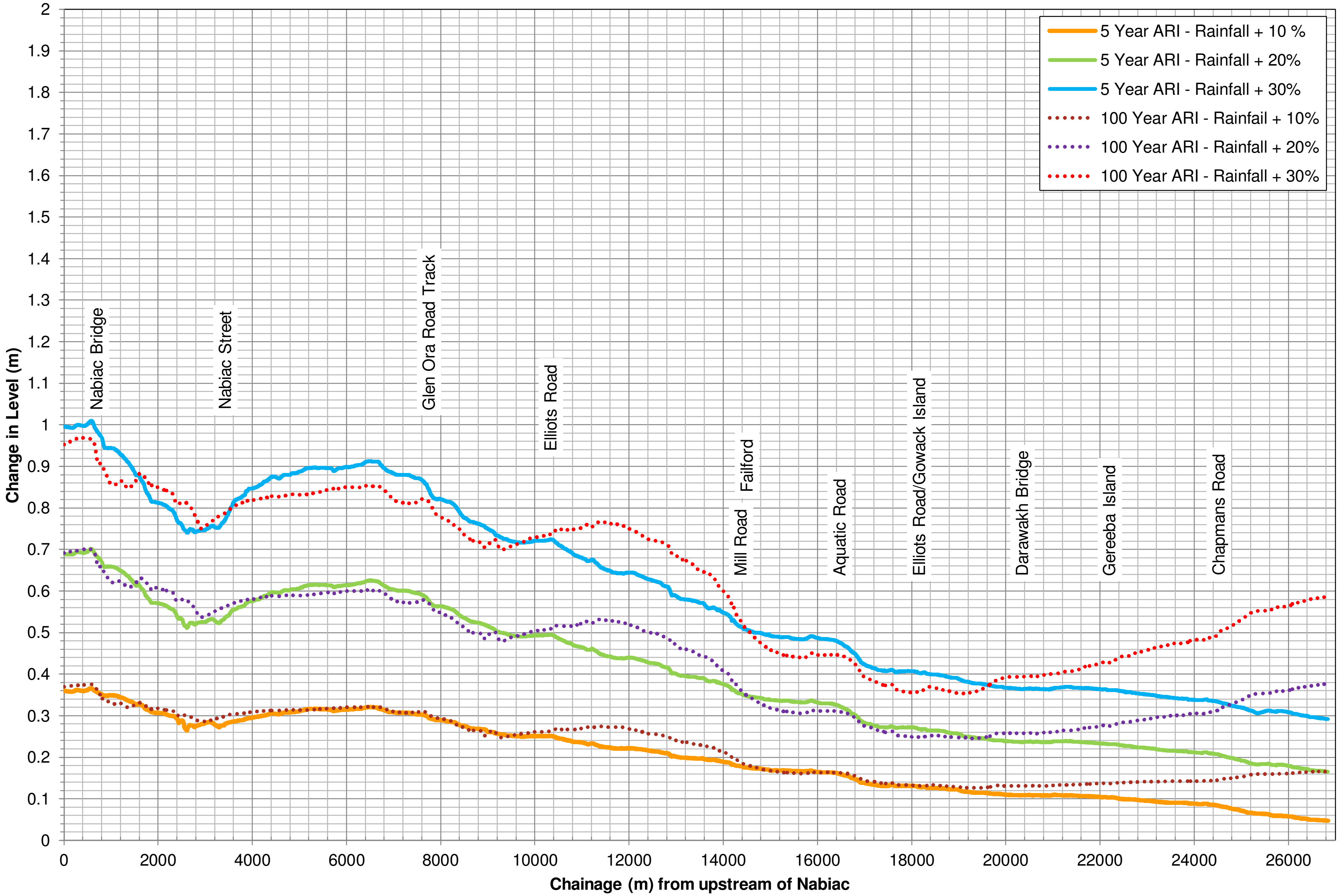

FIGURE 23
CLIMATE CHANGE PROFILES
EXISTING CONDITIONS

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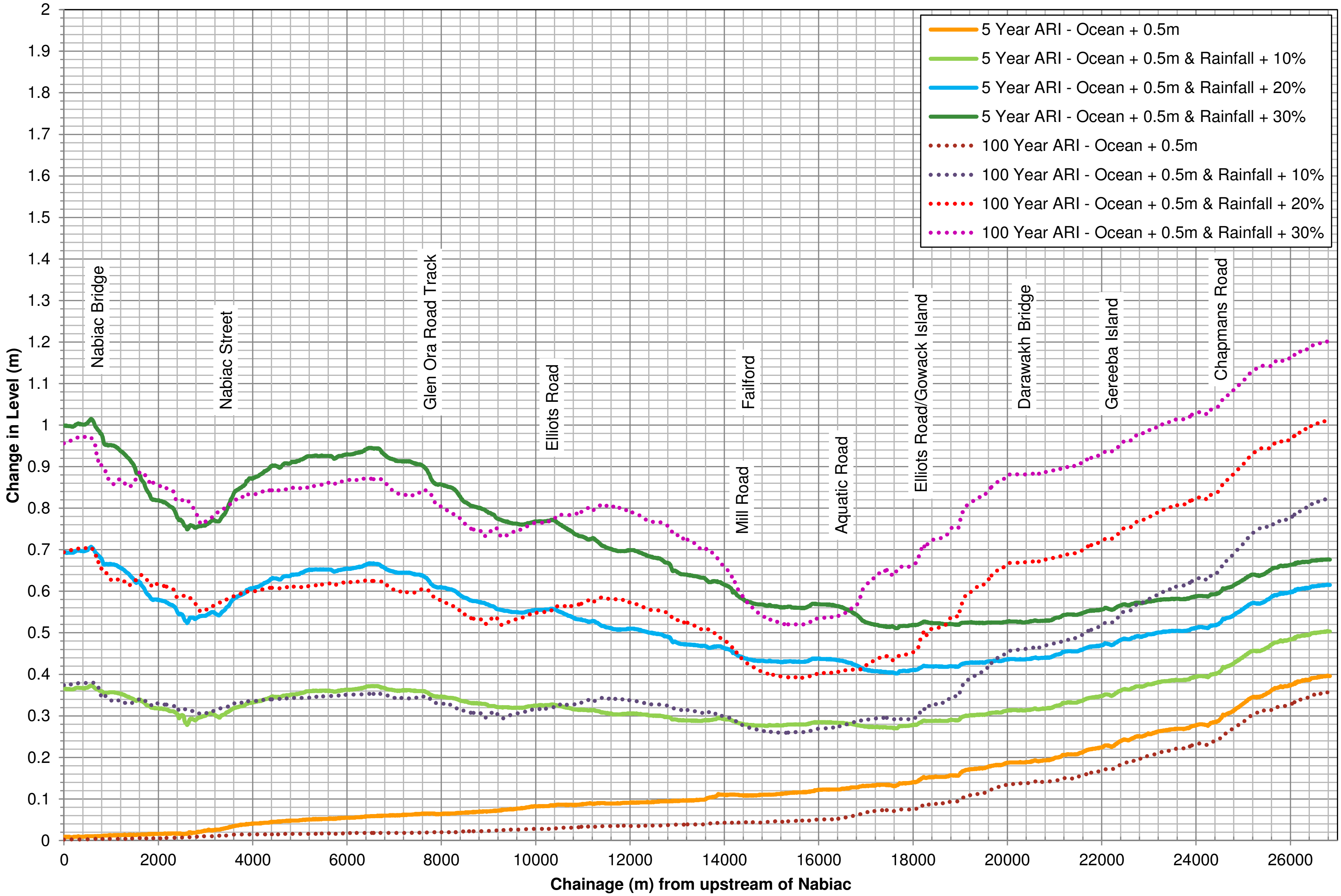

FIGURE 24
CLIMATE CHANGE PROFILES
2060 CONDITIONS

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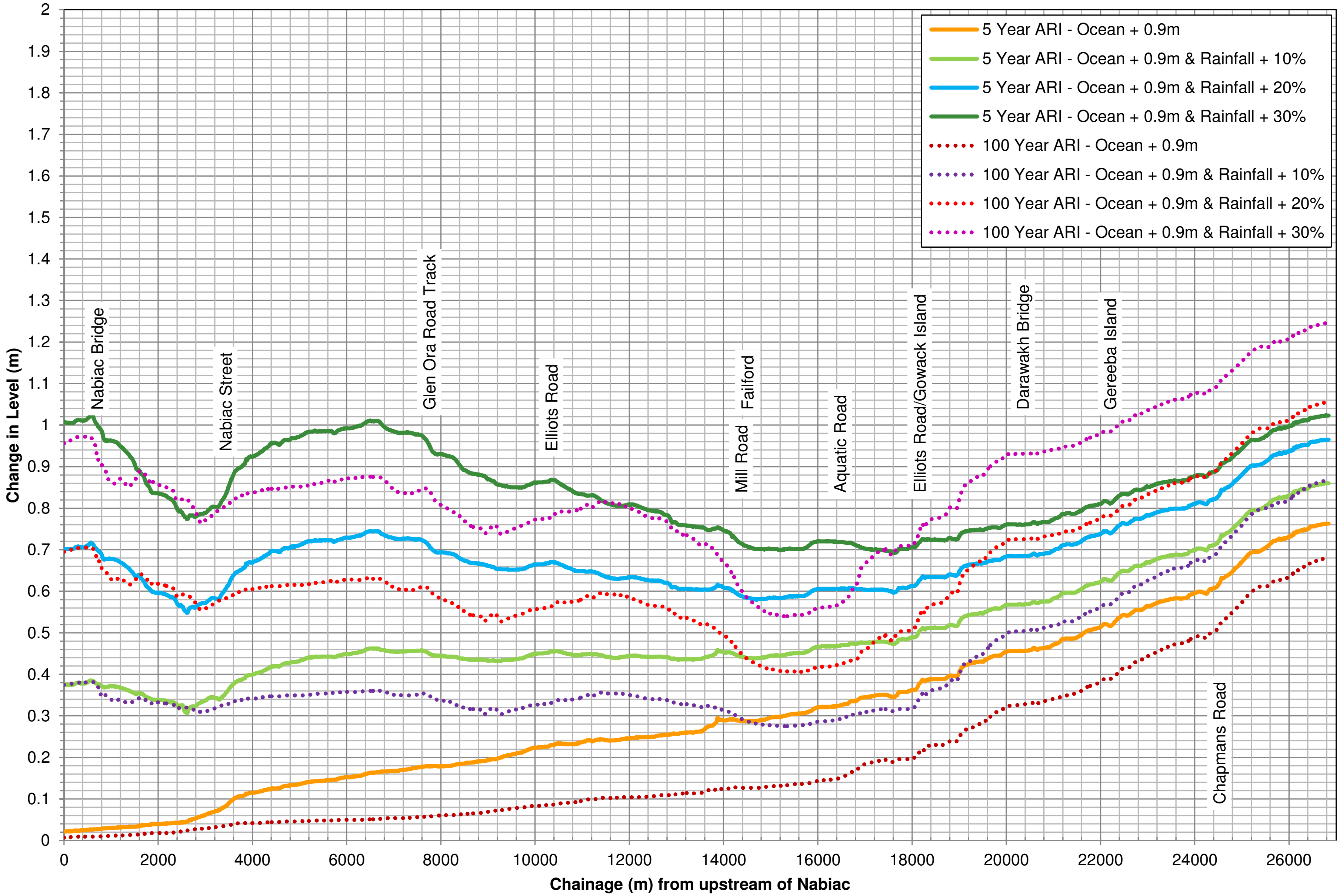


FIGURE 25
CLIMATE CHANGE PROFILES
2100 CONDITIONS

Appendix A

TABLE A1 - HISTORICAL FLOOD INFORMATION

1927 FLOOD

LOCATION NO.	FLOOD LEVEL (m AHD)	SOURCE	COMMENT
11	5.56	Mr Elliot McMaster, occupier "Glen Ora"	
12	5.84	Mr Elliot McMaster, occupier "Glen Ora" as told to Mr Hodges, house occupier	
15	5.93	Mr Bob Campbell, Nabiac (former occupier)	Reliability uncertain
17	7.30	Mr Norman Lulham, Nabiac	Local creek level

1929 FLOOD

LOCATION NO.	FLOOD LEVEL (m AHD)	SOURCE	COMMENT
1	1.84	(Ref 5)	
2	1.74	(Ref 5)	
3	1.92	(Ref 5)	
3	1.88	(Ref 5)	
17	7.86	Great Lakes Shire Council	Local creek level ?

1947 FLOOD

LOCATION NO.	FLOOD LEVEL (m AHD)	SOURCE	COMMENT
7	3.35	Mr W Saxby, Willow Point Rd Failford	
17	7.15	Mr Norman Lulham, Nabiac	

1957 FLOOD

LOCATION NO.	FLOOD LEVEL (m AHD)	SOURCE	COMMENT
11	4.96	Mr Elliot McMaster, occupier "Glen Ora"	
15	5.63	Mr Bob Campbell, Nabiac (former occupier)	Reliability uncertain
16	4.81	Mr Clayton Everingham, Nabiac	Considered reliable

1978 FLOOD

LOCATION NO.	FLOOD LEVEL (m AHD)	SOURCE	COMMENT
1	1.04		
3	1.39		
5	1.67	Mr Peter Johnson, resident Wallamba Ski Park	
5	2.69	Proprietor, Shalimar Caravan Park	Considered to be too high
6	2.10		
9	2.75	Great Lakes Shire Council	
10	3.15/4.24?	Bayley, occupier "Belmont" (Great Lakes Shire Council)	
11	2.50	Mr Elliot McMaster, occupier "Glen Ora"	Considered reliable
12	2.78	Mr David Hodges, occupier	Considered doubtful
13	3.75/4.05?	Mr Colman occupier (Great Lakes Shire Council)	
17	5.5	Mr Northam, occupier (Great Lakes Shire Council)	
18	5.8	Mr Abbott, occupier (Great Lakes Shire Council)	
20	13.0-13.2	Water Resources Commission gauge (209005) "The Old Sawmill"	

1983 FLOOD

LOCATION NO.	FLOOD LEVEL (m AHD)	SOURCE	COMMENT
1	1.16	Chapmans Road PWD MHR	
3	1.09	Darawank Bridge PWD MHR	
4	1.17	Gowack Island PWD MHR	
5	1.23	Wallamba Ski Park	Considered doubtful
5	2.06	Shalimar Caravan Park	
6	1.37	Bullocky Way PWD MHR	
8	1.68	Willow Point Road PWD MHR	
11	1.79	Mr Elliot McMaster, occupier "Glen Ora"	
14	2.43	Nabiac Street PWD MHR	
17	3.34	Nabiac Bridge PWD MHR	
19	5.49	Dargavilles Crossing PWD MHR	
20	10.94	The Old Sawmill PWD MHR	

CORDERY-WEBB METHOD PARAMETERS

TABLE B1 - CATCHMENT DETAILS

1. Chapmans Road :

Catchment Area	=	500 km^2
Mainstream Length	=	71.2 km
Average Slope	=	0.00109 m/m
C value	=	16.3 hours
K value	=	7.5 hours

2. Failford :

Catchment Area	=	427 km^2
Mainstream Length	=	61.2 km
Average Slope	=	0.00141 m/m
C value	=	13.6 hours
K value	=	6.9 hours

3. Nabiac Bridge :

Catchment Area	=	328.4 km^2
Mainstream Length	=	47.7 km
Average Slope	=	0.00229 m/m
C value	=	10.0 hours
K value	=	6.0 hours

4. Old Sawmill :

Catchment Area	=	259 km^2
Mainstream Length	=	41.7 km
Average Slope	=	0.00252 m/m
C value	=	9.1 hours
K value	=	5.5 hours

TABLE B2 : DERIVED UNIT HYDROGRAPHS

TIME INTERVAL (T)	WALLAMBA RIVER - LOCATIONS			
	THE OLD SAWMILL (T = 2 hours)	NABIAC (T = 2 hours)	FAILFORD (T = 2 hours)	CHAPMANS ROAD (T = 3 hours)
1	0.359	0.363	0.469	0.372
2	1.446	1.457	1.865	1.464
3	3.161	3.180	4.015	3.137
4	5.313	5.353	6.651	5.197
5	6.628	7.842	7.919	7.439
6	5.728	7.875	6.406	8.221
7	3.967	5.659	4.147	6.568
8	2.761	4.045	2.678	4.395
9	1.921	2.891	1.730	2.940
10	1.337	2.067	1.117	1.968

DAILY RAINFALL DATA

MARCH 1978 FLOOD

TOTALS (mm) RECORDED AT 0900

DATE	STATION (STATION NUMBER)									
	BOOMERANG BEACH (060088)	BULBY BUSH (060003)	BUNGWAHL (BURRADUC) (060047)	FORSTER (060013)	GLOUCESTER (060015)	KRAMBACH (060021)	TINONEE (060087)	TIPARARY (VIA KRAMBACH) (060103)	SUGARLOAF POINT LIGHTHOUSE (060028)	WAUKIVORY (060062)
18.03.1978	34	39	28			56		38	33	
19.03.1978	62	106	83			126		196	35	
20.03.1978	65	149	116	234**	311**	145	230	150	55	292*

* two day total

** three day total

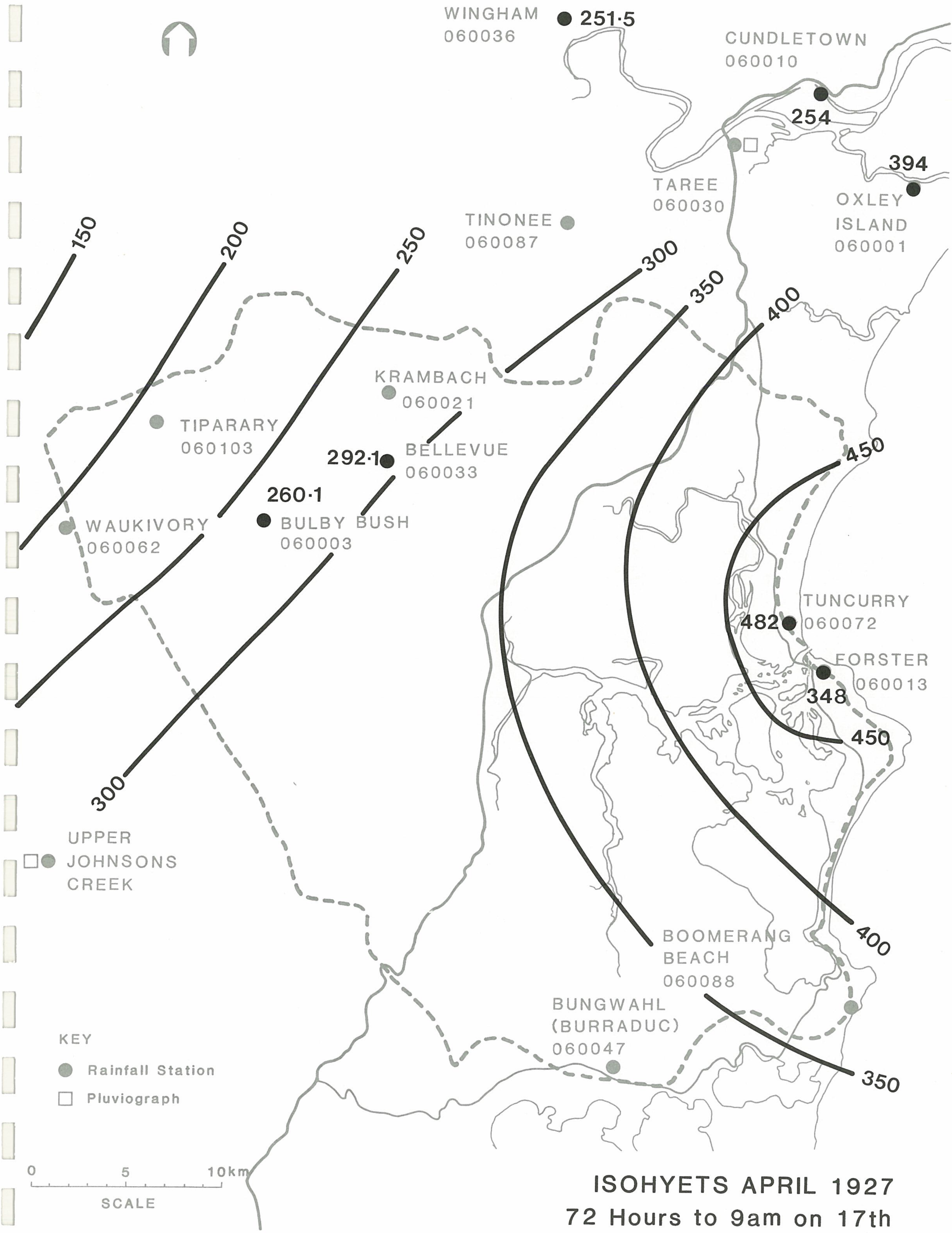
DAILY RAINFALL DATA

APRIL 1927 FLOOD

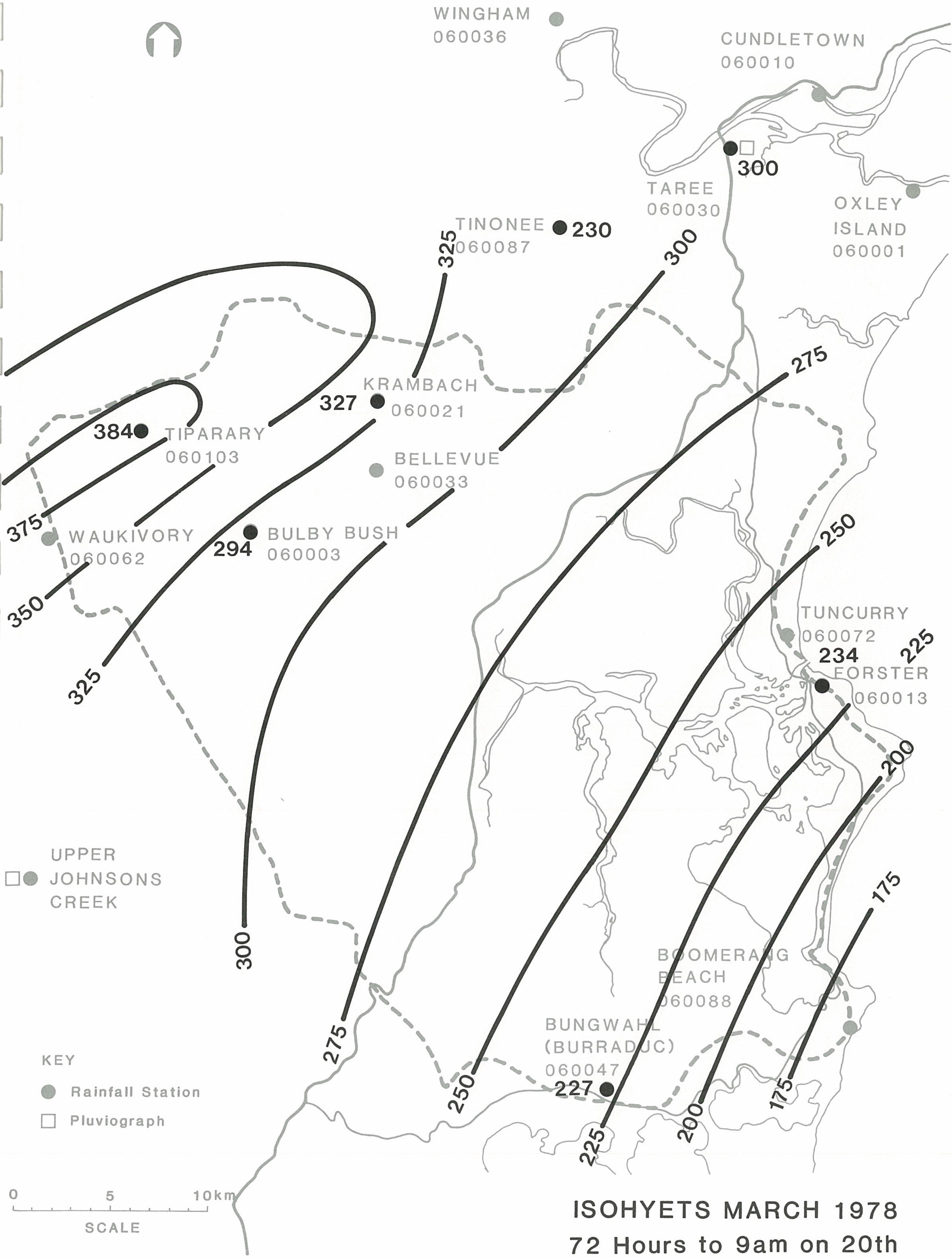
TOTALS (mm) RECORDED AT 0900

DATE	STATION (STATION NUMBER)									
	BELLEVUE (060033)	BULBY BUSH (060003)	CUNDLETOWN (060010)	FORSTER (060013)	GLOUCESTER (060015)	OXLEY IS. (LYNDHURST) (060001)	SUGARLOAF PT. LIGHTHOUSE (060028)	STROUD (061071)	TUNCURRY (060072)	WINGHAM (060036)
12.04.1927				44		4	5		23	
13.04.1927						6				
14.04.1927	20	25	5	8	7		47	53	12	40
15.04.1927	36	32	25	85		57	86		82	
16.04.1927	236	225	224	216 *	83	337	160	260	363	241
17.04.1927		3	3	47		15	62		37	
18.04.1927	3	33			14		3	3		2
19.04.1927	25	20	13	25	14	18	20	27	17	9
20.04.1927	42	43	27	50	20	36	32	74	15	16
21.04.1927	5		9	6	1	6	5	19	9	

* Gauge probably overflowed. Maximum gauge capacity 200mm.



ISOHYETS APRIL 1927
72 Hours to 9am on 17th
Figure B1



ISOHYETS MARCH 1978
72 Hours to 9am on 20th
Figure B2

CUMULATIVE RAINFALL DEPTHS ESTIMATED GRAPHICALLY FROM REFERENCE 1 (MARCH 1978)

Time (days)	Monkerai	Time (days)	Taree	Time (days)	Chichester Dam	Time (days)	Upper Johnsons Creek	Time (days)	Karuah
17.63	1	16.93	1	16.51	0	16.52	0	16.54	0
17.68	25	17.02	2	16.61	1	16.59	1	16.59	2
17.72	29	17.12	3	16.66	4	16.64	2	16.68	3
17.83	33	17.23	3	16.74	11	16.70	2	16.72	4
17.95	38	17.35	3	16.83	20	16.74	3	16.76	6
18.09	46	17.48	6	16.89	25	16.84	7	16.79	6
18.16	50	17.65	11	16.96	26	16.91	7	16.82	6
18.28	52	17.76	16	17.03	28	16.97	8	16.85	6
18.36	53	17.86	23	17.10	31	17.03	11	16.90	5
18.47	56	17.95	26	17.17	37	17.11	12	16.95	5
18.58	57	18.07	28	17.22	41	17.14	14	16.98	7
18.70	59	18.26	30	17.29	44	17.16	19	17.00	8
18.75	62	18.37	32	17.35	47	17.19	24	17.03	10
18.79	69	18.39	34	17.39	51	17.22	27	17.07	11
18.82	85	18.42	37	17.43	55	17.28	29	17.10	12
18.84	110	18.51	47	17.47	58	17.30	32	17.12	12
18.85	123	18.61	58	17.51	61	17.33	34	17.17	15
18.86	130	18.66	68	17.56	65	17.36	34	17.44	30
18.89	136	18.72	84	17.60	67	17.39	35	17.47	31
18.93	143	18.76	99	17.66	69	17.72	61	17.51	32
18.96	149	18.79	112	17.74	72	17.81	64	17.54	34
18.99	159	18.88	143	17.82	74	17.85	71	17.57	36
19.01	169	18.92	158	17.92	77	17.91	81	17.60	40
19.05	175	18.96	171	18.05	80	17.93	91	17.62	44
19.10	181	19.03	192	18.09	83	17.96	98	17.64	45
19.26	189	19.12	222	18.14	85	18.00	105	17.67	46
19.31	192	19.18	240	18.20	88	18.05	109	17.71	47
19.36	196	19.21	245	18.23	92	18.10	111	17.73	48
19.41	210	19.26	249	18.26	97	18.14	112	17.76	51
19.48	224	19.33	253	18.29	102	18.17	116	17.78	53
19.54	233	19.39	255	18.29	104	18.19	119	17.81	54
19.58	238	19.47	258	18.31	111	18.22	120	17.84	54
19.62	241	19.52	263	18.34	123	18.26	121	17.87	55
19.70	244	19.56	268	18.46	156	18.29	122	17.90	57
19.83	246	19.59	274	18.51	172	18.30	123	17.91	60
		19.62	278	18.53	187	18.33	124	17.93	65
		19.65	284	18.55	195	18.36	124	17.94	68
		19.67	288	18.60	208	18.37	125	17.97	74
		19.69	293	18.70	239	18.39	130	17.99	75
		19.71	296	18.75	253	18.40	136	18.02	76
		19.74	303	18.79	260	18.42	140	18.04	77
		19.76	308	18.84	264	18.47	143	18.05	79
		19.80	315	18.88	269	18.52	147	18.05	81
		19.85	319	18.91	273	18.56	147	18.07	82
		19.92	321	18.95	276	18.59	148	18.09	84
		19.98	322	18.96	281	18.61	151	18.13	84
		20.04	323	19.01	286	18.65	155	18.16	84
		20.09	322	19.07	289	18.67	161	18.18	84

Time (days)	Monkerai	Time (days)	Taree	Time (days)	Chichester Dam	Time (days)	Upper Johnsons Creek	Time (days)	Karuah
		20.18	325	19.13	292	18.72	173	18.21	85
		20.26	326	19.19	295	18.75	176	18.26	86
		20.31	330	19.23	299	18.76	180	18.29	88
		20.36	333	19.26	304	18.82	195	18.32	92
		20.43	336	19.28	309	18.89	208	18.37	94
		20.51	339	19.31	315	18.95	225	18.43	96
		20.60	340	19.34	321	18.97	229	18.47	98
				19.37	326	18.98	233	18.51	99
				19.40	329	18.98	238	18.55	102
				19.48	336	19.02	246	18.57	104
				19.52	341	19.06	253	18.62	110
				19.54	346	19.09	259	18.64	115
				19.54	351	19.11	266	18.67	119
				19.55	357	19.14	274	18.68	122
				19.55	361	19.14	280	18.71	125
				19.58	365	19.14	285	18.73	128
				19.60	368	19.16	290	18.75	132
				19.64	373	19.18	295	18.77	143
				19.68	375	19.21	299	18.79	150
				19.73	378	19.23	303	18.82	159
				19.77	381	19.27	315	18.84	161
				19.78	382	19.29	323	18.88	172
				19.82	383	19.32	327	18.92	180
				19.88	383	19.37	330	18.97	194
						19.44	337	18.98	199
						19.47	339	18.99	210
						19.51	339	19.02	220
						19.56	339	19.04	226
						19.60	339	19.07	235
						19.65	339	19.09	241
						19.69	340	19.12	246
						19.72	341	19.16	252
						19.76	343	19.21	258
						19.79	343	19.25	263
						19.83	343	19.27	271
								19.30	277
								19.35	281
								19.40	283
								19.45	286
								19.48	288
								19.50	292
								19.52	301
								19.53	308
								19.55	312
								19.58	316
								19.63	319
								19.70	322
								19.73	326
								19.78	329
								19.84	331