

Lower Macleay Flood Study

Kempsey Shire Council

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Flood Estimation Terminology

This flood study report discusses flooding events in relation to the probability of a certain magnitude flood event occurring during a year. The main terms used to describe design flooding are:

- **Exceedances per year (EY):** the number of times an event is likely to occur or be exceeded within any given year.
- **Annual exceedance probability (AEP):** the probability or likelihood of an event occurring or being exceeded within any given year, usually expressed as a percentage.

The table below lists the probability terminology used for the 2016 design rainfalls and shows in **bold** the standard EY and AEP values for which design rainfalls are available. Generally, EY terminology is used for Very Frequent design rainfalls, AEP (%) terminology is used for Frequent and Infrequent design rainfalls, and AEP (1 in x) terminology is used for Rare design rainfalls.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.50
	1	63.2	1.58	1.00
Frequent	0.69	50.00	2	1.44
	0.5	39.35	2.54	2.00
	0.22	20.00	5	4.48
	0.2	18.13	5.52	5.00
	0.11	10.00	10.00	9.49
Infrequent	0.05	5.00	20	20.0
	0.02	2.00	50	50.0
	0.01	1.00	100	100
	0.005	0.50	200	200
Rare	0.002	0.20	500	500
	0.001	0.10	1000	1000
	0.0005	0.05	2000	2000
	0.0002	0.02	5000	5000
Extremely Rare				
Extreme			PMP	

Note:

- The 50% AEP **does not** correspond to the 2 year Average Recurrence Interval (ARI). Rather it corresponds to the 1.44 ARI.
- The 20% AEP **does not** correspond to the 5 year Average Recurrence Interval (ARI). Rather it corresponds to the 4.48 ARI.

Executive Summary

Introduction

The Lower Macleay River floodplain occupies an approximately 500km² of coastal floodplain downstream of Kempsey, to the Macleay River's main outlet at South West Rocks. The floodplain drains an upstream catchment area of 11,500km² and also has several secondary outlets to the ocean at Hat Head, Crescent Head, Ryans Cut and South West Rocks and a hydraulic link to the Maria River and Hastings River to the south, all owing to improved channel linkages constructed during historic flood mitigation schemes.

Kempsey Shire Council engaged Jacobs to undertake a catchment-scale flood study of the Lower Macleay floodplain, downstream of Kempsey. Numerous flooding studies and assessments have been conducted for the Lower Macleay River in the past, with the recent studies including the *Kempsey CBD Floodplain Risk Management Study* in 2017. The recent studies are fragmented in their study extents.

This current study provides an expansive floodplain-wide assessment of flooding conditions according to the latest floodplain management guidance and policy. The purpose of this study is to undertake investigation to assess mainstream flood behaviour in the broader Lower Macleay floodplain including villages and urban areas. Key objectives of this study are to:

- Review available catchment, topographic, flood and drainage infrastructure data; flood study reports, overland flow path studies and associated hydrologic and hydraulic models
- Undertake hydrologic analysis of upstream catchment inflows to define the river inflow characteristics for historic and design flood events and establish hydrologic models to estimate flood event inflows for the study area downstream of Kempsey
- Establish and calibrate a TUFLOW hydraulic model for the study area to determine flood hydraulic behaviour, with consideration of interactions with flooding in adjacent river catchments and the ocean. There is a particular emphasis on floodplain villages, flood mitigation infrastructure and ocean outlets
- Prepare flood mapping including peak flood extents & levels, depths and velocities for all modelled events
- Assess the sensitivity of flooding to modelling assumptions
- Assess the sensitivity of flooding to climate change, including sea level rise and increased rainfall
- Undertake community consultation to communicate the outcomes of this study
- Prepare a report on the above.

Catchment Description

The Lower Macleay River floodplain covers an area of approximately 500km² downstream of Kempsey. The dominant water courses within the floodplain are the Macleay River, Macleay Arm, Clybucca Creek/Collombatti Creek, Kinchela Creek, Belmore River and Christmas Creek. The general topography of the floodplain includes well defined natural riverbank levees up to 7m AHD along the rivers and creeks formed by deposition of alluvial sediments over the course of numerous historic floods. The terrain grades down to low-lying areas typically less than 1m AHD, and down to 0m AHD. The Macleay River currently discharges to the ocean at South West Rocks via a fully trained river entrance. Historically the river discharged to the ocean at Grassy Head, approximately 10km to the north, until the 1893 flood broke out and formed a new entrance at the existing location. During flood events floodwaters may discharge to the ocean via secondary outlets including Korogoro Creek and Rows Cut at Hat Head, Killick Creek and Ryans Cut at Crescent Head, and South West Rocks Creek at South West Rocks.

Land use in the Lower Macleay is predominantly rural and agricultural. Within the rural landscape are areas of forest, wetlands and other environmentally significant lands. There are a number of small urban centres and villages, including Smihtown, Gladstone, Frederickton, Kinchela, Jerseyville, Fishermans Reach, Stuarts Point, South West Rocks, Crescent Head and Hat Head.

The upstream catchment covers an area of approximately 11,500km², stretching up to the Northern Tablelands region to townships including Armidale, Guyra and Walcha. Catchment elevations range from near sea level at Kempsey to over 1,000m AHD on the Northern Tablelands. The catchment is of a fan-shaped form, where the central catchment area is fed by a sprawling network of tributaries. The shape and generally steep terrain of the catchment tends to concentrate flood runoff from the upstream sub-catchments, influencing flood behaviour and producing floods with typically rapid rates of rise in the lower valley.

History of Flooding

Significant floods occurred in the valley in 1838, 1893, 1949, 1950, 1963, 2001, 2009, 2011 and 2013. A major flood mitigation scheme was implemented from 1955 to the mid-1970s consisting of numerous flood gates, drainage channels and levees to help protect Kempsey township and the agricultural area, and to provide flood warning and improved drainage. Recent infrastructure works in the floodplain include the Pacific Highway Kempsey Bypass, which crosses the river and floodplain via a 3.2km bridge to the east of Kempsey.

The largest flood on record is the 1949 event (1 in 90 Annual Exceedance Probability (AEP)) followed by the 1950 event (1 in 80 AEP). The 1949 flood reached 8.52m AHD at Kempsey Traffic Bridge resulting in the loss of six (6) lives and more than 7,000 head of stock and 600 buildings damaged or destroyed.

The March 2001 flood event is the largest flood event in the valley in recent years and was the largest to occur since the 1963 event. Over 1,000 residents were evacuated by air from the floodplain urban centres. This flood event was estimated at 1 in 13 AEP. Significant flooding also recently occurred during February 2013, which was a 1 in 7 AEP event.

Following the 1949 and 1950 floods, there was significant community reaction which demanded action to mitigate the impacts of flooding on the Lower Macleay Floodplain. This led to the formation of the Macleay Valley Flood Mitigation Committee which formulated a basis of the flood problem and developed options for the Lower Macleay Flood Mitigation Scheme, a series of works to address flooding problems. Works on the Scheme commenced in 1955 and were progressively implemented until the mid-1970's that included the construction of 47 flood-gated structures (of various sizes), 116km of drainage channels and 180km of levee.

The stated aims of the flood mitigation scheme were:

- 1) Protection of the Kempsey township up to a 1 in 10 year flood event
- 2) Protection for the lower Macleay agricultural area from a 1 in 2.5 year flood event
- 3) Rapid drainage of floodwaters
- 4) Provision for erosion protection
- 5) Improved flood mapping, proofing and data, and
- 6) Provision of flood warning.

Model Development

A TUFLOW combined one-dimensional (1D) and two-dimensional (2D) hydrodynamic model with 20m grid resolution was developed for this study. TUFLOW has capability in representing complex 2D flow patterns in the river and floodplains, bridges and levees, in addition to representing one-dimensional drainage elements including finer-scale drains and channels, culverts, floodgates and flood control structures where required. The model extends from upstream of Kempsey to the ocean, including the main Macleay River entrance, the secondary creek outlets to the ocean and down to the Maria River interface. The model terrain was based on LiDAR and river bathymetric survey data from the former Office of Environment and Heritage (OEH; now Department of Planning, Industry and Environment). Additional survey of levees and hydraulic structures were input into the model.

An objective of the study is to focus on flooding conditions in the villages on the floodplain. Hence, within the 20m parent grid a series of nested 5m fine resolution grids were defined for the selected villages.

Hydrologic assessment was undertaken, including review and adoption of the Macleay River flows upstream of Kempsey, which were established in previous flood studies for Council. Additional hydrologic modelling was undertaken in DRAINS using the RAFTS hydrologic module to define rainfall-runoff generated from on local tributary catchments and local catchments onto the floodplain. Analysis of design flood data from the Hastings River Flood Study (Patterson Britton and Partners, 2006, for Port Macquarie Hastings Council) was undertaken to define the inflow and outflow conditions at the Maria River interface. Time-varying tidal conditions were defined in the ocean boundaries based on guidelines from OEH.

Model Calibration and Verification

The TUFLOW model was calibrated and verified to the March 2001 flood and the February 2013 flood. These events were selected given their recent occurrence in addition to the large amount of data collected on observed flooding. The 2001 event represents the largest event in the last 20 years while the 2013 event is one of the most recent and significant events. There have been a number of significant flood events prior to 2001 including the 1949 (1 in 90 AEP), 1950 (1 in 80 AEP) and 1963 (1 in 17 AEP) events which were not considered for calibration in this study due to uncertainties in input data, in addition to significant changes in the floodplain since these events, including construction the Macleay Flood Mitigation Scheme, built between the 1950s and 1970s. Additionally, the quantity and quality of flood observations from these early events is relatively low.

The focus of the 2001 event model calibration was mainly to define appropriate hydraulic roughness conditions, in order for the model to be able to reasonably reproduce observed flood behaviour within the catchment. Model inputs included synthesised Macleay River inflows, validated to recorded stream flow upstream of Kempsey, synthesised local catchment inflows and recorded water levels in the Maria River and ocean. The process for the model calibration involved adjustment of the hydraulic roughness along the river channel and on the floodplain in addition to checks on hydraulic controls (topography, structures) to obtain good agreement with recorded water levels at Kempsey, Smithtown and South West Rocks water level gauges. Check against observed flood levels and depths was also undertaken. The 2013 event simulation confirmed the ability of the model to replicate flooding conditions based on the selected calibration parameters.

The model calibration uncovered the need to adjust the river bed elevations at the river entrance and secondary outlets. The Macleay entrance has a number of shallow shoals which would be eroded during high flood flows. Lowering of the bed in the model was necessary to obtain a good calibration particularly in the lower reaches. The lowered bed profiles for calibration and design events were confirmed based on detailed sediment transport and bed morphology modelling using Delft 3D coastal modelling software.

Design Flood Estimation

The calibrated model was used to derive design flood conditions within Lower Macleay. The design events considered in this study include the 0.2 Exceedances per Year (EY), 5%, 1% and 0.2% AEP and Probable Maximum Flood events. Flood mapping was produced including design flood inundation, peak flood water levels and depths and peak flood velocities. Provisional flood hazard categorisation in accordance with Australian Institute for Disaster Resilience (AIDR) guidelines was also prepared. Zoomed in flood mapping on each of the floodplain villages in the study area was also prepared. Design flood levels are summarised at key locations as shown in Table 1.

Table 1 Design Flood Levels at Selected Locations (m AHD)

Event AEP	AEP				
	0.2EY	5%	1%	0.2%	PMF
Frederickton	5.84	6.26	6.56	7.01	9.32
Smithtown (Nestle)	4.50	4.72	5.03	5.68	8.79
Smithtown (Gauge)	4.25	4.43	4.64	5.61	8.71
Kinchela	3.42	3.57	4.38	5.56	8.68
Jerseyville	1.95	2.67	4.02	5.35	8.40
South West Rocks (Gauge)	1.19	1.93	3.47	4.94	7.76
Fishermans Reach	1.19	2.04	3.47	4.88	7.51
Stuarts Point	1.18	2.05	3.47	4.82	6.31
Collombatti D/S of Pacific Highway	2.81	3.46	4.39	5.61	8.73
Clybucca Floodgates U/S	2.40	3.16	4.28	5.51	8.57
Upper Belmore	2.67	3.79	4.49	5.57	8.61
Upper Kinchela	2.26	3.56	4.47	5.58	8.68
Hat Head (Creek)	1.27	1.71	2.62	4.08	6.81
Crescent Head (Creek)	1.32	2.16	2.61	3.25	4.87

Flood behaviour at the villages on the floodplain is described below.

South West Rocks and Jerseyville

South West Rocks is affected by flooding from the Macleay River, South West Rocks Creek and Spencers Creek. Property yard flooding occurs along the river in the 5% AEP event and the Riverside Tavern, built on a filled pad, becomes surrounded by floodwaters but not submerged. Properties along South West Rocks Creek are not flood-affected in the 0.2EY event as flood levels are dictated mainly by ocean tide levels in up to the 5% AEP event.

Property flooding becomes more substantial in the 1% AEP event particularly along South West Rocks Creek when significant overflow occurs from the Macleay River into South West Rocks Creek, and on residential and industrial properties in southern South West Rocks from Spencers Creek, with depths of up to 1m. The ground around Riverside Tavern is submerged.

Significant flooding occurs in the 0.2% AEP and PMF as river overflows into South West Rocks Creek result in depths of over 2m in the PMF affecting the holiday park and floodwaters back up into the gully and marsh area south of Gordon Young Drive. Flood depths in southern South West Rocks are over 2m in the 0.2% AEP and over 4m in the PMF on properties.

In Jerseyville due to the low-lying nature of the properties the village is inundated in the 0.2EY event to depths of up to 1m, and 1 – 2m in the 5% AEP. Flood depths exceed 2m in the 1% AEP.

Flow velocities are relatively low (less than 0.5m/s) in all flooded properties in South West Rocks and Jerseyville in up to the 0.2% AEP event, with velocities up to 0.75m/s in Jerseyville in the PMF. There are localised areas

of higher velocity. However, due to the high depths of flooding Jerseyville is affected by high (H3 – H4) flood hazard rating in the 5% AEP event and very high (H5) flood hazard rating in the 1% AEP.

Fishermans Reach

Flooding is contained in the watercourse in up to the 5% AEP event, and overflows the banks affecting property in the 1% AEP generally to 0.5m depth with areas up to 2m. Depths are generally 2m in the 0.2% AEP and up to 4m in the PMF. Flooding around Fishermans Reach is dictated by backwater flooding from the Macleay River backing up northward up Macleay Arm. Flow velocities are generally benign in the 1% AEP, around 0.25m/s with areas up to 0.5m/s. In the PMF velocities are typically 0.5m/s with localised areas at 1m/s or more.

Flood hazard is generally up to high rating (H3) in the 1% AEP and very high to extreme (H5 – H6) in the PMF.

Stuarts Point

Flooding begins to affect Stuarts Point village in the 5% AEP when the caravan park becomes inundated to over 0.5m depth, with up to 2m flooding depths in the 1% AEP. The majority of the village is flood-free up to the 0.2% AEP event and is affected by depths of 1 – 2m in the PMF. Flow velocities are generally up to 0.5m/s in the PMF in the village with areas in roadways experiencing up to 2m/s velocities. Flood hazard is generally up to high rating (H4) in the PMF.

Similar to Fishermans Reach, flooding around Stuarts Point is dictated by backwater flooding from the Macleay River backing up northward up Macleay Arm. There is also localised flooding in the low points between the dune features to the west of the village, which affects some properties there in the 5% AEP and rarer.

Frederickton

Frederickton is affected by flooding from the Macleay River itself as well as from two gullies in the east and north of the village, which are caused by backwater flooding from Clybucca Wetlands and the northern floodplain. There is also flooding in Christmas Creek in the west of the village which is mainly caused by backwater from the Macleay River.

Properties in the south-east of the village experience yard flooding in the 0.2EY event from the river, and up to two properties are affected by flooding in the eastern gully. In the 5% AEP the river overtops its bank in the vicinity of the Frederickton Hotel and causes flooding of properties to depths of 0.5m. Backwater flooding in the eastern gully causes property flooding of 0.5 – 1m.

Most of the village is flood-free in up to the 0.2% AEP event. Some properties are affected by flood depths of 0.5 – 2m in the 1% AEP event and 1 – 3m in the 0.2% AEP event. In the PMF properties in the south-east and east of the village are affected by combined flooding from the Macleay River in addition to the eastern gully, with depths exceeding 4m on some properties.

Flow velocities on properties are generally up to 0.5m/s in up to the 0.2% AEP event and exceed 2m/s in some locations in the PMF. Flood hazard on affected properties is generally up to high (H3 – H4) in the 1% and 0.2% AEP with some very high (H5) hazard in deeper areas and flow paths, and very high to extreme (H5 – H6) in the PMF.

Some properties in Frederickton may also be affected by overland flooding in the gullies running through the village, however, this was not a focus of this study and the overland flooding is not shown on the flood mapping. Further assessment is recommended to define the overland flooding in this village.

Smithtown

Smithtown is affected in the 0.2EY event with the Macleay River overtopping the village levees and river banks at several locations, including on the western side north of Jeffery Street, at the southern end of the village and along the eastern side of the village. Flood depths are greatest in the middle of the village where there is a

natural low point, where floodwaters generally flow northward on the floodplain. Depths are in the range of 1m in the 0.2EY event, up to 2m in the 1% AEP, 2 – 3m in the 0.2% AEP and over 5m in the PMF. The highest point of the Smithtown bridge western approach ramp is above the PMF, as is the bridge deck itself if it is not damaged during the flood event.

The highest flow velocities and hazard are due to the river breakout on the western side of the village, where velocities are 1.5m/s in the 0.2EY and 5% AEP events, and over 2m/s in the 1% and 0.2% AEP events. high flow velocities are also present as floodwaters flow northward over the Smithtown Road embankment. Flow velocities exceed 2.5m/s in the PMF.

Flood hazard is up to high (H3 – H4) in the 0.2EY event, mostly high in the 5% AEP event and very high (up to H5) in the 1% AEP event.

Gladstone

Gladstone is affected by bank overflows from the Macleay River. Depths of this bank overflow flooding are shallow in the 0.2EY event, less than 0.2m although there are areas with depths up to 0.5m. In the 5% AEP the bank overflows are also relatively shallow on the higher ground along the west side of the village, with greater depths of around 1m as backwater from the floodplain to the east begins to contribute to flooding. Flood depths are up to 2m in the 1% AEP, up to 3m in the 0.2% AEP and over 5m in the PMF. The highest point of the Smithtown bridge eastern approach ramp is above the 0.2% AEP flood level.

Flow velocities in the village are generally low (less than 0.25m/s) in up to the 5% AEP with velocities in flow paths and roads around 1 - 1.5m/s. In the 1% and 0.2% AEP the maximum flow path velocities are up to 2m/s. PMF velocities are 1 – 2m/s. Floodwaters tend to flow from the river and towards the east onto the floodplain in all events.

Flood hazard rating is up to high (H3) in the 5% AEP event, high to very high (H4 – H5) in the 1% and 0.2% AEP events and extreme (H6) in the PMF.

Kinchela

Kinchela flooding is caused by bank overflows from the Macleay River and Kinchela Creek. Depths of flooding are up to 0.5m in the 0.2EY event and 1m in the 5% AEP event. Flood flows pond and build up against South West Rocks Road and Hat Head Road before overflowing onto the wider floodplain to the east. In the 1% AEP flood depths on properties are 0.5 – 2m, 2 – 3m in the 0.2% AEP event and over 6m in the PMF. The junction area of South West Rocks Road and Hat Head Road is above the 1% AEP flood, while the highest points of the bridge approaches over Kinchela Creek entrance are above the 0.2% AEP flood.

Flow velocities in the village are less than 0.5m/s in up to the 1% AEP. Higher flow velocities are present where floodwaters overtop and spill over the road embankments and river levee. Velocities exceed 2m/s in the 0.2% AEP and the PMF to the east of the village where flows break out of Kinchela Creek.

Flood hazard rating is generally low (H1 – H2) with some areas up to high (H3) in the 5% AEP event, high to very high (H3 – H5) in the 1% and 0.2% AEP events and extreme (H6) in the PMF.

Crescent Head

Crescent Head is affected by flooding from the Lower Macleay southern floodplain, on the western side of the village, in addition to flooding in Killick Creek which is influenced by flows from the southern floodplain into the creek combined with elevated ocean levels entering the creek entrance. In the 0.2EY event Killick Creek flooding is influenced mainly by high ocean tides with minor contribution of flows from the southern floodplain into the head of the creek. A few properties experience shallow yard flooding.

In the 5% AEP event floodplain inflows into Killick Creek increase and properties along the creek are flooded to depths of 0.5 – 1m. Flooding in the southern floodplain begins to encroach on the properties in the western side

of town. Depths increase to 1 – 2m in the 1% AEP event along Killick Creek properties and up to 0.5m on western side properties. Floodwater begins to overflow from the southern floodplain to Killick Creek through properties.

In the 0.2% AEP event maximum flood depths on Killick Creek properties is over 2m and 1 – 2m on western side properties. The southern floodplain overflows at two locations. In the PMF depths on affected properties of 3 – 4m are common, with overflows from the southern floodplain to depths of over 2m through the town. The southern half of town is located on elevated ground above the PMF.

Active flows are contained mainly in Killick Creek with velocities over 2m/s in the 5% AEP event and rarer. Flow velocities in the southern floodplain are generally low, less than 0.5m/s, affecting the western side of town in up to the 0.2% AEP event. There are localised higher velocity flows in streets in the 0.2% AEP event. In the PMF there numerous locations where velocities exceed 1.5m/s due to overflows from the southern floodplain.

Flood hazard is generally low – moderate (H1 – H2) on affected properties in the 1% AEP, with some low-lying properties along Killick Creek with high hazard (H4). The extent of low – moderate hazard increases through the centre of the village in the 0.2% AEP with maximum hazard on properties increasing to very high (H5). In the PMF there are extensive areas of very high – extreme (H5 – H6) hazard through the town. The sewage treatment plant to the north of the town appears to be mostly unaffected by the 0.2% AEP flood.

Hat Head

Hat Head is affected by flooding in Korogoro Creek, which is influenced by flows from the southern floodplain into the creek combined with elevated ocean levels entering the creek entrance. Levees along the northern and southern sides of the creek provide protection to village areas. The levees are overtopped in events rarer than the 5% AEP event.

Local catchment runoff contributes to flooding of the levee-protected areas in the northern and southern parts of the village. These flows pond in the low-lying area behind the levee on both sides of the creek. This is the dominant source of flooding in the village in the 0.2EY and 5% AEP events, when the levees are not overtopped and mainstream floodwaters are contained in the creek. Some properties to the south of the creek are affected by flooding to depths of up to 0.8m in the 0.2EY event and up to 1m in the 5% AEP in the low points of properties. Depths of flooding in the drainage low points in the village area to the north of the creek are generally shallow, up to 0.2m in the 0.2EY event and up to 0.3m in the 5% AEP event.

In the 1% AEP event the village on both sides of the creek are inundated typically to depths 0.6 – 1m and localised areas over 1m. Depths in the village increase to over 2m in the 0.2% AEP. This is predominantly due to floodwaters overtopping the levee and causing flooding of the village areas in both of these events.

Flood depths in the PMF are 4 – 5m in the village. Flows into Korogoro Creek from the southern Lower Macleay floodplain are limited in up to the 0.2% AEP event due to the constriction of the hydraulic connection through the sand hills between the floodplain and the head of the creek, to the north of the village. There are significant overflows over the sand hills separating the floodplain and Korogoro Creek which entirely fills the creek floodplain between the sand hills and the coastal beach dunes. The narrow creek outlet abutting Korogoro Point headland is the only outlet for these floodwaters. Most of the village is inundated in the PMF, with some properties to the south of the creek located on high ground above the PMF.

Flow velocities on the floodplain and through properties are low in up to the 1% AEP, generally less than 0.5m/s in the streets. These velocities increase to 0.75m/s in the 0.2% AEP. In the PMF maximum flow velocities are generally above 1m/s with higher velocities near Hat Head Road crossing where flows break out of the creek and near the boat ramp where there are steeper ground slopes and flows re-enter the creek. There are also high velocities where floodwaters spill over the sand dunes onto the beach.

There are areas of low to high (H1 – H3) flood hazard in the southern part of the village in the 0.2EY event. The northern part of the village is mainly not flooded in this event with only some nuisance flooding in low points. In the 5% AEP the flood hazard is generally high (H3) in flood-affected areas in the southern part of the village.

The hazard rating increases to high (H3) in the northern part of the village in the 1% AEP and is rated moderate to high (H2 – H3, some H4) in the southern side of the village. Flood hazard is very high (H5) in the 0.2% AEP and extreme (H6) in the PMF.

Note that further assessment of local catchment flooding inside the levees is required to confirm peak flooding, particularly in the 0.2EY and 5% AEP events.

Sensitivity Analysis

Sensitivity testing was undertaken in the TUFLOW model to assess the changes in flooding behaviour due to variation in model parameter values, removal of the works implemented as a part of the Lower Macleay flood mitigation scheme and climate change impacts. In summary:

- Flood levels in the 1% AEP event vary typically by +/- 0.2m and up to +/- 0.5m from design flood levels by varying the hydraulic roughness of the river and floodplain by +/- 20%. This indicates the modelled flood levels are fairly sensitive to global changes in the roughness values. While localised changes to land use on the floodplain are unlikely to result in large impacts to flood levels, widespread changes to land use, for example change from a pasture land cover to cropping land use, may have marked effects on the peak flood levels. Future changes in floodplain agriculture and vegetation cover may warrant revision of the design flood estimates.
- Flood levels across the floodplain reduce by -0.1m to -0.3m due to low tide conditions compared to the design ocean conditions. Separately, a coinciding high tide condition results in changes ranging from minor reductions up to increases in flood levels of up to +0.2m which affect developed areas near the river entrance. There are higher variances localised to the river outlets, which do not impact on existing developed areas. Minimal or no change upstream of Smithtown for both sensitivity conditions. These changes in flood levels do not significantly change the flood affectation of development, noting that the coinciding high tide condition is an unlikely occurrence. Hence it is not considered incorporate the coinciding high tide condition into the design flood estimate.
- Flood levels reduce by -0.3m in a fairly localised area due to a coastal dune breach in the PMF which forms an additional outlet for floodwaters. The PMF levels are relatively insensitive to this condition and it is considered prudent to assume non-breach of the coastal dunes in the PMF estimation.
- With no Lower Macleay flood mitigation scheme, in the 0.2EY event flood levels are increased by up to 0.4m on the northern floodplain and reduced by up to 0.4m in the southern floodplain. In the 1% AEP event there are up to 0.1m increases and up to 0.2m decreases on the northern and southern floodplains, respectively, with localised higher changes. In the PMF there are broadly 0.05m increases with localised increases up to 0.4m. These are the likely upper bound changes to flood conditions from any proposed future modifications to the form and operation of the flood mitigation scheme.
- In the climate change assessment the 1% AEP flood levels broadly increase on the Lower Macleay floodplain by 0.3 – 0.5m in the 2050 scenario, and 0.7 – 1m in the year 2100 scenario. Increases of up to 0.8m in the year 2050 and up to 1.2m in the year 2100 scenarios occur at the main river outlet and minor outlets, in addition to upstream of Kempsey. These impacts are due to the combination of increased rainfall and flood flows in addition to sea level rise. Given the sensitivity of flooding conditions in the climate change scenarios, it would be prudent to consider incorporation of climate change impacts into planning controls for development of the Lower Macleay floodplain.

Limitations of the Study

Several limitations of the study are acknowledged.

- Design flood levels in and around Kempsey CBD are analysed in greater detail in the *Kempsey CBD Floodplain Risk Management Study* (WMAwater, 2017b) using a 10m model resolution compared to the 20m resolution in this current study. Floodplain management actions have already been established and adopted based on the floodplain risk management study. It is considered appropriate to defer to the 2017 study for design flood levels and conditions in and around the Kempsey CBD, downstream to Frederickton. Refer to the hatched area around Kempsey CBD on the relevant flood maps in this report which denote the

area where the design flood levels from the Kempsey CBD flood modelling and mapping (WMAwater, 2017b) applies. The design flood levels for the remaining unhatched area, covering the Lower Macleay floodplain, are defined by this current flood study.

- This study focussed on the assessment of flooding in the Lower Macleay valley. While parts of the Maria River and Connection Creek floodplain were represented in the TUFLOW model, inclusion of the broader Maria River floodplain and catchment, which is within the Hastings River catchment, was outside the scope of the study. Parts of the Maria River floodplain are in Kempsey LGA. To assist Council with defining design flood levels in the Maria River floodplain, interpretation of the results from this study and from the *Hastings River Flood Study* (Patterson Britton and Partners, 2006) was undertaken. Design flood levels and extents were estimated and are presented for this area. Refer to Appendix J. As a part of the future Lower Macleay Flood Risk Management Study (Stage 2), extending the hydrologic and hydraulic modelling south into the Maria River floodplain system should be considered as a model enhancement initiative to better define the design flood conditions in this area.
- A number of minor watercourses of interest were incorporated in the 20m grid, including Belmore River and Kinchela Creek. These watercourses are relatively narrow. It is anticipated that the Lower Macleay TUFLOW model will be used in subsequent assessments to help refine operation of the Belmore River and Kinchela Creek flood control structures for management of frequent floods such as the 1 in 2 AEP event which will require model refinement for these creeks such as representation in 1D. It is recommended that any refinement of the model and further detailed investigation of the performance of the key flood control structures be undertaken in the subsequent floodplain risk management study for the Lower Macleay.
- Changes to the duration of inundation on the floodplain could not be ascertained from the flood modelling sensitivity assessment of removing the flood mitigation scheme. Major drains were represented in both the baseline and sensitivity scenarios as it was reasoned that these drains would be installed regardless of the flood mitigation scheme in order to improve agricultural lands. Further analysis could be undertaken subsequent to this study to estimate the changes in inundation times on the floodplain. This would require refinement of the modelling of major and minor agricultural drains and extending the model simulation time to the order of several weeks. The flooding assessment limited the simulated time to five days, which is sufficient to pass the flood wave through to the river outlet.
- A primary focus on the broader Lower Macleay floodplain means that less focus was placed in the assessment flooding in small local watercourses particularly those crossing the Pacific Highway and North Coast Railway. This may not reflect flooding and potentially cutting of the highway and railway by the PMF and possible disruption of the highway and railway as evacuation routes and transport links. It is recommended that this be investigated in the subsequent floodplain risk management study.
- Similarly to above, local catchment flooding in areas protected by levees was not a primary focus of this study. Local inflows from the Lower Macleay floodplain critical storm events (48 hour duration) were input for more sensitive locations including Frederickton and Hat Head. Assessment of additional, shorter storm durations should be undertaken to confirm peak local overland flooding conditions in these areas. It was agreed with Council that this should be deferred to a future local drainage study for Frederickton, for that location. A similar study is less critical for Hat Head, which is significantly affected by mainstream 1% AEP flooding which would dictate the flood planning levels for that village.
- The Macleay River is a highly dynamic river system which may undergo geomorphic change including shift in bed and channel shape and alignment as a result of, and also during, flooding events. While the flood modelling in this study does approximate the likely scouring of the waterway entrances during flood events for estimating flood behaviour, it assumes a static river and floodplain form in the rest of the river system during the simulation of flooding. This is an acceptable practice in flood studies given the uncertainty of how the river morphology may change during a flood. It should be noted that morphological changes may differ between different flood events.

Conclusions

This study has achieved its purpose in the development of appropriate flood modelling for estimating flooding conditions and behaviour for design flood events in the Lower Macleay floodplain, particularly for larger flood events such as the 5% and 1% AEP events and rarer. Additional assessment is required outside the scope of

this study to achieve a number of other floodplain management objectives of Council, including refined modelling of minor channel and watercourses for assessing drainage and flood control structure operation and estimation of PMF conditions in local watercourses crossing the Pacific Highway and North Coast Railway. Further, the design flood information from previous more detailed studies supersede this study in specific areas of the floodplain, specifically around Kempsey CBD.

Parts of Kempsey LGA fall in the Maria River floodplain outside of the Lower Macleay flood study and were excluded from the modelling in this study. Design flood levels for the Maria River floodplain were inferred from available information and estimated flood extents mapped for the 5% and 1% AEP and PMF events. These estimates should be refined with future more detailed assessment.

Recommendations

A number of recommendations are made for further assessment in subsequent studies and data collection to help enhance the knowledge on flood behaviour in the Lower Macleay and surrounds:

- 1) Further detailed modelling is recommended to better define design flood conditions in the Maria River floodplain area, between the Lower Macleay model southern boundary and the Kempsey local government area boundary with Port Macquarie Hastings local government area, in the vicinity of Point Plomer. While an estimate of the design flood levels and extents has been prepared based on available data (refer Appendix J), the future modelling assessment should provide more detailed information on flood behaviour, flood hazard etc. It is recommended that this further modelling be undertaken in the subsequent Lower Macleay Floodplain Risk Management Study (FRMS) which could include extension of the hydrologic and hydraulic modelling to cover the Maria River. As a part of the future modelling updates, flooding scenarios such as a rare flood event in the Maria River (e.g. 1% AEP) coinciding with a frequent flood event in the Macleay River (e.g. 0.2EY), and vice versa, should be analysed to maximise the flood gradient in this area to determine the sensitivity of flow velocities and flood hazard.
- 2) Hydrologic modelling enhancement and updates is recommended including update with ARR 2019 to align with the latest design rainfall and hydrologic guidance. This may result in revision of design flood flows in the Macleay River. It is recommended that this is undertaken in the subsequent Lower Macleay FRMS.
- 3) Further refinement of the TUFLOW hydraulic model developed in this study is recommended in the subsequent Lower Macleay FRMS, including:
 - a) Refinement of representation of smaller-scale watercourses in the study area, including Belmore River and Kinchela Creek, and including representation of these as 1D channel features. This would permit more reliable representation of channel conveyance for the purpose of flood mitigation options assessment in minor flood events smaller than the 0.2EY event as assessed in this study.
 - b) Assessment of very rare to extreme (0.2% AEP, PMF) flood events in local catchments. In particular this will assist with assessment of probable maximum flooding in minor watercourses and flood immunity of roads (Pacific Highway, etc.) for emergency management purposes.
 - c) Additional general model sensitivity testing should be considered in the subsequent FRMS including sensitivity of the hydrologic modelling to ARR 2019 parameters (design rainfalls, rainfall losses, areal reduction factors, hydrologic modelling procedures) and catchment routing parameter (Bx).
- 4) Further assessment of local catchment flooding is recommended in particular for Frederickton, where shorter duration storm events should be assessed to determine peak overland flooding conditions in Frederickton Drain. It is recommended that this be undertaken in a future local drainage study, as proposed by Council.
- 5) Development of a Strategic Water Level Gauging Network Plan that provides future consideration for locations to record data in future flood events to improve model calibration in the future. Locations for monitoring include those areas on the floodplain, such as Clybucca Wetlands, Upper Belmore and Upper Kinchela Floodplains, and on tributary watercourses such as at Belmore and Kinchela flood control structures, which are not currently monitored. Consideration should also be made for water level monitoring of ocean levels locally so there is less reliance on tidal monitoring at Coffs Harbour. While an oceanway

(i.e. at the Macleay entrance) site may not be suitable for ocean level monitoring, suitable sites in Trial Bay, to which the Macleay discharges, may be available.

- 6) Due to the influence that sedimentation and siltation has on flood conditions and the performance of the current mitigation scheme, updated detailed bathymetry survey is recommended for all major waterways in the Lower Macleay upstream to Greenhill. Additionally, since the oceanway acts as a hydraulic control and upstream water levels in the estuary are sensitive to actual scour depths, it is recommended that there is annual survey of the entrance bed and other locations identified as hydraulic controls or particularly dynamic bedforms. Survey should be undertaken in dry conditions and immediately after a significant flood event.
- 7) Consideration should be made for future update of the flood model and design flood estimates following substantial changes to floodplain conditions, including development, land use and river morphology.
- 8) Consideration should be made in the subsequent FRMS for incorporation of climate change impacts to flooding into Council's planning and development controls, given the projected significant changes to flooding conditions as a result of climate change.
- 9) Council and the flood risk management committee will be consulted on this draft flood study report. It is recommended that the final draft report subsequently be placed on public exhibition to consult the community and stakeholders on the findings of this study.

Foreword

The primary objective of the New South Wales Government's Flood Prone Land Policy is to reduce the impact of flooding and flood liability on individual owners and occupiers of flood prone property, and to reduce private and public losses resulting from floods, utilising ecologically positive methods, wherever possible. Under the Policy, Kempsey Shire Council is responsible for local land use planning in its service area, including management of both mainstream and overland flooding within the Kempsey Local Government Area.

The policy provides for a floodplain management system comprising the following five sequential stages:

- 1. Data Collection** Involves compilation of existing data and collection of additional data
- 2. Flood Study** Determines the nature and extent of the flood problem
- 3. Floodplain Risk Management Study** Evaluates management options in consideration of social, ecological and economic factors relating to flood risk with respect to both existing and future development
- 4. Floodplain Risk Management Plan** Involves formal adoption by Council of a plan of management for the floodplain
- 5. Implementation of the Plan** Implementation of flood, response and property modification measures (including mitigation works, planning controls, flood warnings, flood preparedness, environmental rehabilitation, ongoing data collection and monitoring by Council)

Council proposes to prepare a comprehensive flood study for Lower Macleay River floodplain in accordance with the NSW Government's "Floodplain Development Manual: the management of flood liable land", April 2005.

This study represents Stages 1 and 2 of the management process and has been prepared for Council by Jacobs. This report is the Final Draft Flood Study Report which documents the assessment and findings up to the completion of Stage 2.

Important note about this report

The sole purpose of this report and the associated services performed by Jacobs is to undertake a flood study for the Lower Macleay River floodplain, in accordance with the scope of services set out in the contract between Jacobs and Kempsey Shire Council (the Client). That scope of services, as described in this report, was developed with the Client.

In preparing this report, Jacobs has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by the Client and/or from other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

Jacobs derived the data in this report from information sourced from the Client, third parties, and/or available in the public domain at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose described above and by reference to applicable standards, guidelines, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report, to the extent permitted by law.

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1. Introduction

1.1 General

The Lower Macleay River floodplain (refer to Figure 1-1) occupies 500km² of coastal floodplain downstream of Kempsey, to the Macleay River's main outlet at South West Rocks. The floodplain drains an upstream catchment area of 11,500km² and also has several secondary outlets to the ocean at Hat Head, Crescent Head, Ryans Cut and South West Rocks and a hydraulic link to the Maria River and Hastings River to the south, all owing to improved channel linkages constructed during historic flood mitigation schemes.

The floodplain is generally low lying and contains numerous watercourses including Belmore River, Kinchella Creek, Collombatti Creek and Clybucca River, in addition to a number of wetland areas. The towns and villages of Smithtown, Gladstone, Kinchella, Jerseyville, Fishermans Reach and Stuarts Point are situated in the floodplain, with South West Rocks, Hat Head and Crescent Head accessed by roads through the floodplain.

Significant floods occurred in the valley in 1838, 1893, 1949, 1950, 1963, 2001, 2009, 2011 and 2013. A major flood mitigation scheme was implemented from 1955 to the mid-1970s consisting of numerous flood gates, drainage channels and levees to help protect Kempsey township and the agricultural area, and to provide flood warning and improved drainage. Recent infrastructure works in the floodplain include the Pacific Highway Kempsey Bypass, which crosses the river and floodplain via a 3.2km bridge to the east of Kempsey.

Council engaged Jacobs to undertake a catchment-scale flood study of the Lower Macleay floodplain, downstream of Kempsey. Numerous flooding studies and assessments have been conducted for the Lower Macleay River in the past, with the recent studies including the *Kempsey CBD Floodplain Risk Management Study* in 2017. The recent studies are fragmented in their study extents.

This current study provides an expansive floodplain-wide assessment of flooding conditions according to the latest floodplain management guidance and policy, using contemporary best-practice technology and analysis techniques. The design flood estimates and hydraulic models produced by the study will guide future floodplain planning and management on the Lower Macleay floodplain.

1.2 Purpose of This Study

The purpose of this study is to undertake investigation to assess mainstream flood behaviour in the broader Lower Macleay floodplain including villages and urban areas. Key objectives of this study are to:

- Review available catchment, topographic, flood and drainage infrastructure data; flood study reports, overland flow path studies and associated hydrologic and hydraulic models
- Undertake hydrologic analysis of upstream catchment inflows to define the river inflow characteristics for historic and design flood events and establish hydrologic models to estimate flood event inflows for the study area downstream of Kempsey
- Establish and calibrate a hydraulic model for the study area to determine flood hydraulic behaviour, with consideration of interactions with flooding in adjacent river catchments and the ocean. There is a particular emphasis on floodplain villages, flood mitigation infrastructure and ocean outlets
- Prepare flood mapping including peak flood extents & levels, depths and velocities for all modelled events
- Assess the sensitivity of flooding to modelling assumptions including the removal of the existing flood mitigation works
- Assess the sensitivity of flooding to climate change, including sea level rise and increased rainfall
- Undertake community consultation to communicate the outcomes of this study
- Prepare a report on the above.



Legend

- Study Area
- Macleay River Catchment
- Kempsey Local Government Area
- Main Road
- Drainage

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SHEET		1 of 1		A3	
		GDA 1994 MGA Zone 56			
TITLE		Study Area			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
LC	IA176800	Figure 1-1	1	1	
CHECK	DATE				
AH	3/09/2019				

1.3 Structure of the Report

The contents of this report are structured in the following manner:

Section 2: Background information on the study area is provided

Section 3: Relevant reports and data available to this study are described

Section 4: Hydrologic assessment including description of hydrologic modelling is discussed

Section 5: The development and configuration of the TUFLOW hydraulic model is described

Section 6: Calibration and verification of the TUFLOW model is discussed.

Section 7: The modelling assumptions made in estimating design flooding are discussed

Section 8: The design flooding conditions are described

Section 9: Sensitivity analysis of the modelling is outlined

Section 10: Limitations of this study are addressed

Section 11: Conclusions and recommendations are provided

Section 12: Acknowledges contributions received from the community and organisations to undertake this study

Section 13: References cited in the report are detailed

Section 14: A glossary of terms are provided.

2. Background on the Study Area

2.1 Description of Study Area

This study focusses on the Lower Macleay River floodplain, which covers an area of approximately 500km² downstream of Kempsey. The dominant water courses within the floodplain are the Macleay River, Macleay Arm, Clybucca Creek/Collombatti Creek, Kinchela Creek, Belmore River and Christmas Creek.

The general topography of the floodplain includes well defined natural riverbank levees up to 7m AHD along the rivers and creeks formed by deposition of alluvial sediments over the course of numerous historic floods. The terrain grades down to low-lying areas typically less than 1m AHD, and down to 0m AHD. The main floodplain and wetland areas are located at Clybucca Wetlands in the north and Belmore River and Kinchela Creek swamp areas, including Swan Pool. There are a number of other wetland areas connected to the estuary including the Yarrahapinni Wetlands.

Collombatti Creek, which drains into Clybucca Creek, and Christmas Creek are the main local watercourses with a contributing catchment area which produces flows into the watercourses. Belmore River and Kinchela Creek are both lowland waterways which historically transferred tidal and river flood flows into the attached coastal backswamp areas, with freshwater outflows only occurring following main river floods overflowing into the swamps.

The Macleay River currently discharges to the ocean at South West Rocks via a fully trained river entrance. Historically the river discharged to the ocean at Grassy Head, approximately 10km to the north, until the 1893 flood broke out and formed a new entrance at the existing location. During flood events floodwaters may discharge to the ocean via secondary outlets including Korogoro Creek and Rows Cut at Hat Head, Killick Creek and Ryans Cut at Crescent Head, and South West Rocks Creek at South West Rocks.

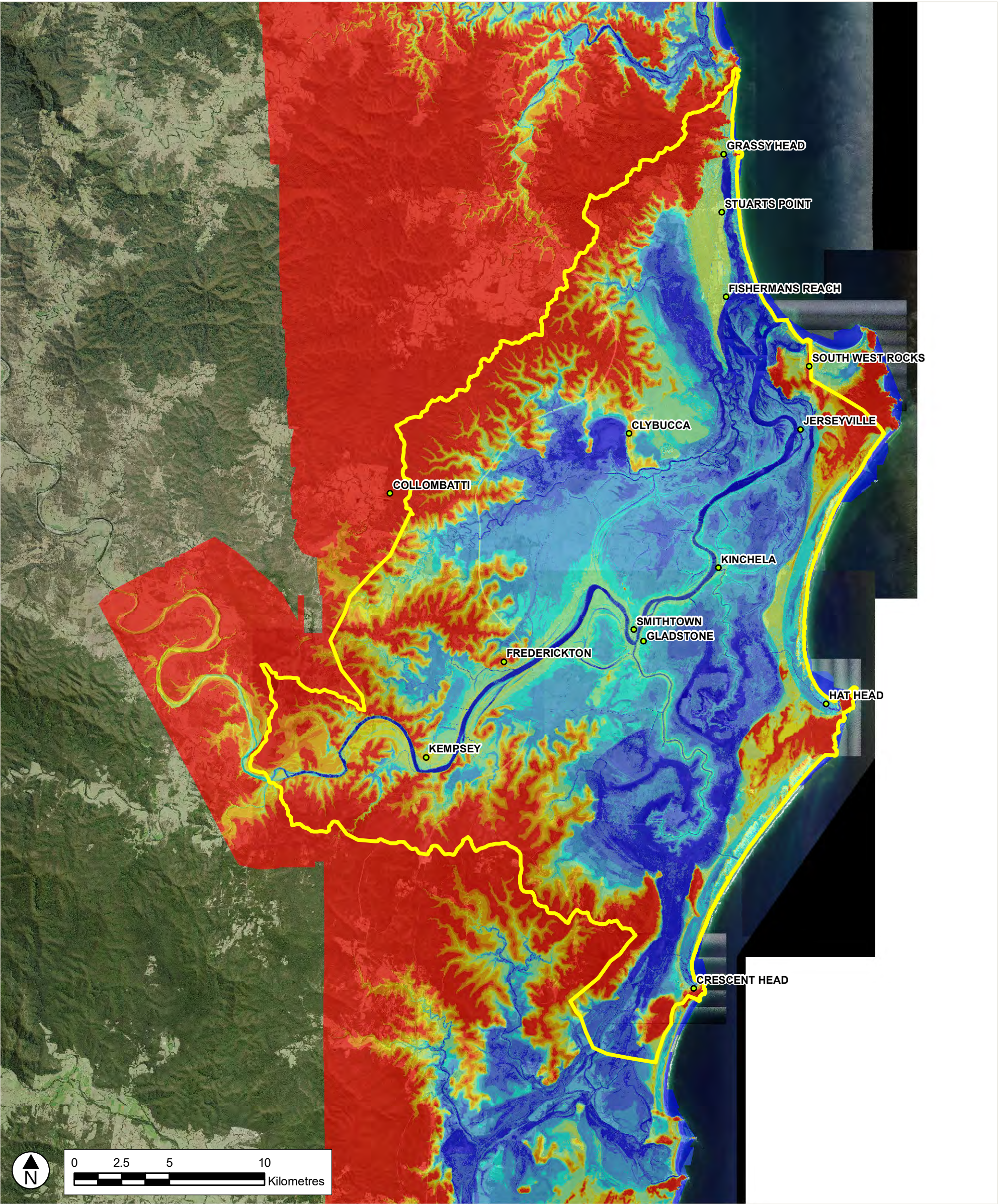
The floodplain in the vicinity of Belmore River and Kinchela Creek (broadly referred to in this report as the “southern floodplain”), to the south of the main river, are hydraulically connected to the Hastings River floodplain to the south, with flows occurring between the Macleay floodplain and the Hastings floodplain via Connection Creek and the Maria River. A floodgate on Connection Creek and the raised Crescent Head Road embankment are low-level barriers to this interflow. These are overtopped when floodwaters reach approximately 1.2m AHD, which may occur during frequent flood events.

The floodplain area to the north of the river, adjoining Collombatti Creek/Clybucca Creek, including the Clybucca Wetlands, is broadly referred to in this report as the “northern floodplain”.

2.2 The Upstream Catchment

The upstream catchment of the Macleay River, upstream of Kempsey, covers an area of approximately 11,500km², stretching up to the Northern Tablelands region to townships including Armidale, Guyra and Walcha. The catchment generally drains to the east where a significant portion of the middle catchment is dominated by gorge country. The upper and lower ends of the catchment are relatively flat, while the central two-thirds are extremely mountainous and rugged. Catchment elevations range from near sea level at Kempsey to over 1,000m AHD on the Northern Tablelands, as shown on Figure 2-2.

The catchment is of a fan-shaped form, where the central catchment area is fed by a sprawling network of tributaries. The main tributaries include the Apsley, Muddy and Chandler Rivers, fed by numerous smaller streams from very steep gorge country. These watercourses converge to the Macleay River around 35km upstream of Kempsey. In this area the catchment shape funnels down to a narrow valley. The shape and generally steep terrain of the catchment tends to concentrate flood runoff from the upstream sub-catchments, influencing flood behaviour and producing floods with typically rapid rates of rise in the lower valley. Beyond Kempsey, the floodplain broadens as the Lower Macleay River meanders towards the coast.



GIS MAP file : IA176800_R04_Fig2-1_StudyAreaTerrain_01

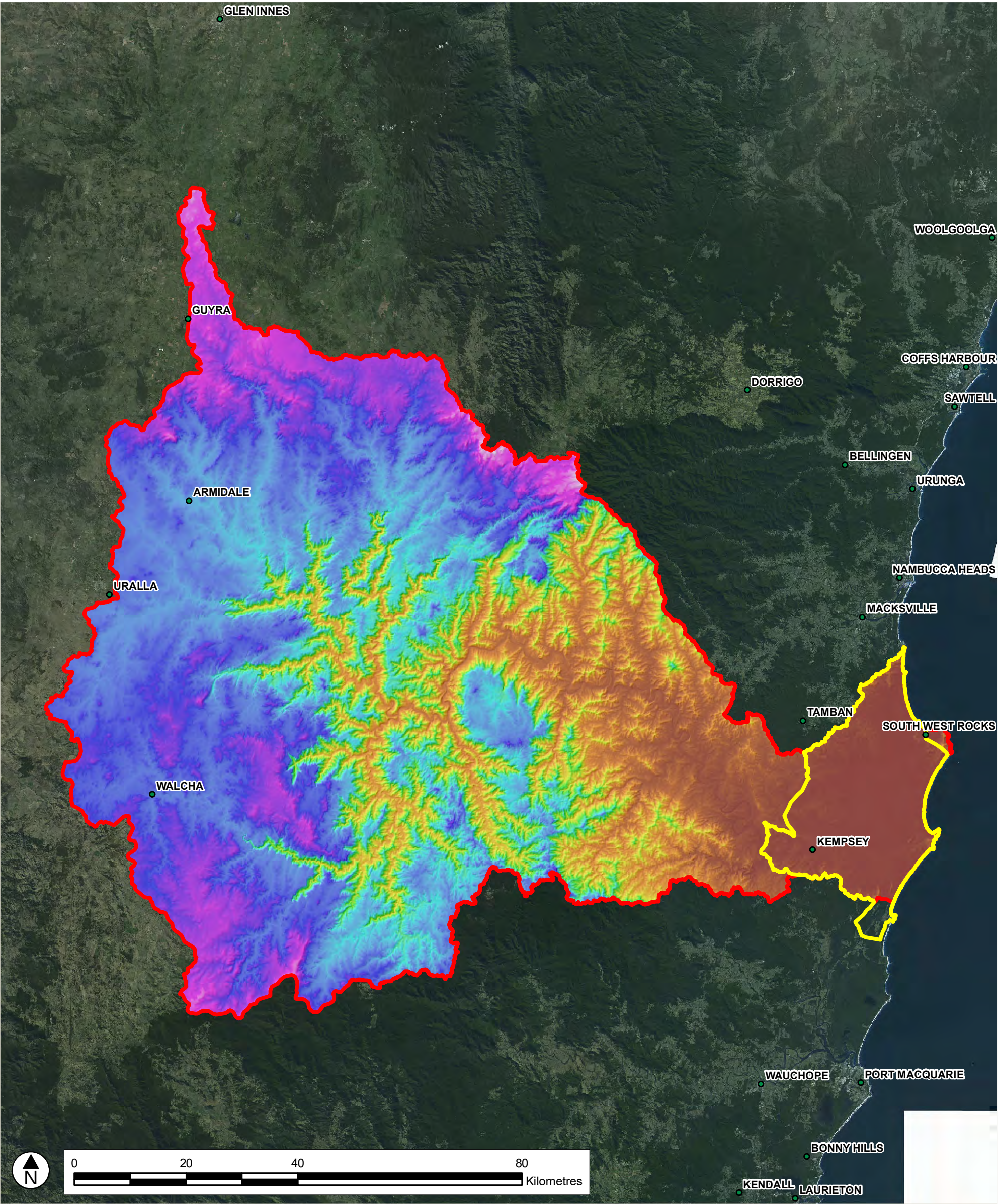
Legend

Ground Elevation (m AHD)			
	< 0.5		5 - 8
	0.5 - 1		8 - 10
	1 - 2		10 - 15
	2 - 3		15 - 20
	3 - 5		> 20
			Study Area

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SHEET		1 of 1		A3	
				GDA 1994 MGA Zone 56	
TITLE		Study Area Terrain			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
MR	IA176800	FIGURE 2-1	1	1	
CHECK	DATE				
LC	3/09/2019				



Legend

Value

High : 1575

Low : 0

Macleay River Catchment

Study Area



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A3				
SHEET	1 of 1		GDA 1994 MGA Zone 56	
TITLE		Macleay Upper Catchment Terrain		
PROJECT		Lower Macleay Flood Study		
DRAWN	PROJECT #	MAP #	REV	VER
MR	IA176800	FIGURE 2-2	1	1
CHECK	DATE			
LC	11/09/2019			

2.3 Existing Development and Land Use

Land use in the Lower Macleay is predominantly rural and agricultural, made up mostly of pastoral and grazing lands with some pockets of cropping land. Within the rural landscape are areas of forest, wetlands and other environmentally significant lands.

Within the floodplain are a number of small urban centres and villages, including Smithtown, Gladstone, Frederickton, Kinchela, Jerseyville, Fishermans Reach and Stuarts Point, which are located primarily on the banks of the Macleay River. Smithtown and Gladstone, supports a large milk product factory and further downstream the village of Jerseyville is a centre of the local fishing industry.

Other developed centres include South West Rocks, Crescent Head and Hat Head, which are situated along the coast and which are also affected by flooding in the Lower Macleay both from direct impacts as well as indirectly by having transport, communications and services cut-off during flood events.

2.4 Flood Behaviour

The configuration of the Macleay catchment is practically suited to the production of high peak flood flows. The major tributaries join the main stem of the river within a small distance of each another causing a rapid concentration of floodwater in the main stream below their confluences, and the large upstream catchment funnels to a narrow bottleneck upstream of Kempsey. Following major rain storms the Macleay River at Kempsey may rise from a normal dry weather discharge of about 56 m³/sec to 14,160m³/sec (the estimated discharge of the August 1949 flood) within a 30 hour period.

The main river flows are the dominant cause of flooding in the study area. Floodwaters tend to overflow from the river particularly between Kempsey and Smithtown, filling the adjacent floodplain areas. Floodwaters slowly drain through the floodplain, flowing in parallel to the main river, conveyed in natural and man-made floodplain channels and watercourses, converging on the main river in the lower reaches of the catchment.

During major flood events floodwaters may break out to the ocean at a number of minor creek outlets as mentioned in Section 2.1. There is also potential for overtopping of the coastal beach foredune system at a number of locations in rare flood events, scouring out additional discharge points in such events.

It is quite likely that there would be coinciding flooding in the adjacent Hastings River to the south of the Macleay, in addition to other river systems during a regional flooding event. Depending on the relative size of flood events in the Macleay River floodplain and Hastings River floodplain, floodwaters may flow into the Macleay floodplain or out to the Hastings floodplain via the low-lying area connecting the two floodplains to the south-west of Crescent Head. Flows may occur in both directions over the course of particular flood events.

2.5 History of Flooding

The Macleay valley has a history of flooding. The first recorded flood to cause damage was in 1838. As mentioned earlier the 1893 flood event significantly changed the course of the river in the lower reaches the breakout of a new river entrance at its current location.

Winter floods in 1949 where water levels recorded at Kempsey Traffic Bridge reached 8.52m AHD resulting in the loss of six (6) lives and more than 7,000 head of stock and 600 buildings damaged or destroyed. Winter floods again in 1950 recorded peak water levels at the Kempsey Traffic Bridge of 8.31m AHD. The 1949 flood was estimated at approximately a 1 in 90 AEP while the 1950 flood was estimated at 1 in 80 AEP.

The March 2001 flood event is the largest flood event in the valley in recent years and was the largest to occur since the 1963 event. Over 1,000 residents were evacuated by air from the floodplain urban centres. The estimated rural flood damages for that event have been assessed at greater than \$12 million dollars (Webb McKeown and Associates, 2002). This flood event was estimated at 1 in 13 AEP.

The March 2001 flood event followed a minor flood in early February 2001. Most upper river rain stations received 30 to 50mm on both 31 January and 1 February, with mid and lower catchment stations receiving 50 to 100mm on those two days. A low pressure system formed on 5 March with Bureau of Meteorology issuing wet weather alerts. Significant rainfall fell on an already wet upper catchment between 5 and 12 March, as summarised on Table 2-1.

Table 2-1 Rainfall on the Macleay River Catchment 5 – 11 March 2001 (in mm)

Location	Mon 5th	Tues 6th	Wed 7th	Thurs 8th	Fri 9th	Sat 10th	Sun 11th	TOTAL
Pt Lookout	15	84	71	233	170	78	23	674
Guyra	12	4	15	42	8	1	8	90
Blue Nobby	8	13	25	86	8	2	9	151
Walcha	1	4	1	4	21	4	2	37
Wollomombi	2	21	12	57	43	11	1	147
Tia	0	14	97	30	5	11	0	157
Boonanghi	1	66	31	72	105	72	25	372
Millbank	2	65	26	108	151	50	8	410
Bellbrook	2	52	22	104	144	27	8	359
Turners Flat	1	72	14	79	93	45	13	317
Wittittrin	1	3	3	2	9	5	4	27
Kempsey	4	122	30	80	42	11	3	292
Seven Oaks	1	24	4	31	18	2	1	81
Seale Road	12	81	12	49	25	7	1	187

Flooding peaked at Kempsey on 12 March at 7.44m on the Kempsey Traffic Bridge gauge, overtopping a number of levees in the CBD in addition to rural levees in the lower floodplain. Extensive inundation of the lower floodplain occurred. Gladstone, Smithtown, South West Rocks, Hat Head, Crescent Head and many upstream and downstream rural areas were isolated. Many food drops, or evacuation for medical reasons, were organised. The sewerage treatment system at Smithtown/ Gladstone was overloaded by floodwaters and surcharging occurred. Council staff were unable to bring the system back on line until late Tuesday. Both villages were voluntarily evacuated of some residents until the sewer was fully functional (reference: Kempsey Shire Council Report on Macleay Flood of March, 2001 River Levels, Rainfall and Recovery).

Significant flooding also recently occurred during February 2013. The flood peaked on 24 February at 7.11m AHD at the Kempsey Traffic Bridge gauge, slightly lower than the March 2001 event. Floodwaters overtopped levees at Kempsey causing inundation of parts of the CBD. Widespread flooding was experienced in the Lower Macleay floodplain including inundation of Smithtown and Gladstone, among other villages. This event was estimated as a 1 in 7 AEP event.

Photos from historic floods are presented below.

Figure 2-3 View of the floodplain looking downstream of Kempsey after the 1949 flood (source: Kempsey Shire Council)



Figure 2-4 Flooding at Kempsey railway bridge during the 1949 flood (source: Kempsey Shire Council)



Figure 2-5 Floodwaters overtopping the Eden Street levee in Kempsey CBD in the 2001 flood (source: Kempsey Shire Council)



Figure 2-6 Aerial photo of Smithtown and Gladstone, looking downstream, in the 2013 flood (source: Kempsey Shire Council)



Figure 2-7 Kempsey Bypass and new Pacific Highway bridge over Lower Macleay floodplain, 2013 flood (source: Kempsey Shire Council)



Figure 2-8 Belmore River Flood Control Structure discharging flood water from the Belmore River into the Belmore floodplain, 2013 flood (source: Kempsey Shire Council)



Figure 2-9 Kinchela Creek Left Bank Flood Control Structure discharging flood water from Kinchela Creek into the west Kinchela area, 2013 flood (source: Kempsey Shire Council)



Figure 2-10 Ryans Cut - Ocean floodwater outlet discharging flood water from the Belmore area into the ocean, 2013 event (source: Kempsey Shire Council)



Figure 2-11 Killick Creek - floodwater discharging into the ocean at Crescent Head, 2013 event (source: Kempsey Shire Council)



2.6 Lower Macleay Flood Mitigation Scheme

Following the severe flood events in 1949 and 1950 there was significant community reaction which demanded action to mitigate the impacts of flooding on the Lower Macleay Floodplain. This led to the formation of the Macleay Valley Flood Mitigation Committee which formulated a basis of the flood problem and developed options for the Lower Macleay Flood Mitigation Scheme, a series of works to address flooding problems (McDonald, 1967). The product of this assessment was the “Jacka Report” (Macleay Valley Flood Mitigation Committee, 1953) which documented the assessment and recommendations for the Scheme.

Works on the Scheme commenced in 1955 and were progressively implemented until the mid-1970’s that included the construction of 47 flood-gated structures (of various sizes), 116km of drainage channels and 180km of levee.

The stated aims of the flood mitigation scheme were:

- 1) Protection of the Kempsey township up to a 1 in 10 year flood event
- 2) Protection for the lower Macleay agricultural area from a 1 in 2.5 year flood event
- 3) Allow for rapid drainage of floodwaters
- 4) Provide for erosion protection
- 5) Improve flood mapping, proofing and data, and
- 6) Provision of flood warning.

Focussing on the Lower Macleay downstream of Kempsey CBD, a significant feature of the Scheme includes the system of levees, flood gates and flood control structures on the southern floodplain, including the areas of Upper Belmore and Upper Kinchela and wetland areas of Swan Pool. Extensive levees along both banks of

Belmore River and Kinchela Creek are linked with on-stream floodgates in the upper sections of each watercourse, which aim to prevent floodwaters from the Macleay River spilling into the southern floodplain and inundating the large tracts of productive agricultural land in minor flood events. Large off-line (lateral) flood control structures and floodways on Belmore River left bank and Kinchela Creek left and right banks can be opened to allow controlled flow of floodwaters into the floodplain in slightly larger flood events to help mitigate flooding to downstream villages. These parts of the Scheme are likely to be overtopped with minimal influence on flooding behaviour in moderate and major flood events.

Works related to the Scheme on the southern floodplain also include constructed drainage link channels between the floodplain and to a number of coastal creeks to assist with drainage of floodwaters following a flood event. These creeks included Korogoro Creek at Hat Head and Killick Creek at Crescent Head, and the coastal lagoon at Ryans Cut, to the north of Crescent Head. Each of these waterways were naturally separated from the southern floodplain by sand dune systems. Floodgate headworks and an excavated channel through the beach dune system were also constructed at Rowes Cut, on Korogoro Creek just north of Hat Head, as an additional coastal outlet. However, Rowes Cut was not maintained and over time was filled in by the beach dune system.

At the southern extremity of the southern floodplain a connecting channel (Connection Creek) and floodgates were constructed, linking the southern Lower Macleay floodplain to the Maria River and Hastings River floodplains to the south. An additional ocean outlet was constructed at Big Hill, connecting the Maria River floodplain to the ocean.

On the northern floodplain, Collombatti Creek/Clybucca Creek naturally drained the low-lying areas in this region. Drainage channels including Seven Oaks Drain and Andersons Inlet were constructed to improve floodwater drainage. Major headworks at Clybucca floodgates, were constructed on Clybucca Creek to prevent saltwater intrusion into the northern floodplain area to improve agricultural potential.

Numerous flood mitigation works were also implemented in and around Kempsey CBD, which is not the focus of this study. Refer to WMAwater (2015, 2017a, 2017b) for details.

2.7 Recent and Proposed Development in the Floodplain

A number of changes in the Lower Macleay floodplain have occurred in recent years:

- Pacific Highway Upgrade – Kempsey Bypass and new bridge crossing: as a part of the Pacific Highway upgrade program the Kempsey Bypass and a new dual-carriageway crossing of the Macleay River and floodplain was constructed between South Kempsey and to the east of Frederickton. The viaduct over the floodplain is 3.2km long making it one of Australia's longest bridges. The bridge crosses the river 7km downstream of Kempsey. To the north of Frederickton, the highway continues northwards to Eungai, crossing the Collombatti Creek floodplain on a raised embankment with a number of waterway crossings via bridges and culverts. The highway upgrade and bridge was completed in 2013.
- Yarrahapinni Wetlands restoration: these wetlands are on Borirgala Creek on the northern lower floodplain. Historically, a tidal barrage and floodgates were installed to stop tidal intrusion of estuarine water to improve upstream lands for agriculture. This led to poor environmental outcomes including degradation of the wetland health and problems with acid sulphate soils. Under a wetland restoration program by National Parks and Wildlife Service and Office of Environment and Heritage the tidal barrage was breached and floodgates removed to return tidal flushing to the wetlands, occurring in approximately 2015. The restoration works may have some influence on flooding behaviour in the northern lower floodplain during smaller flood events.
- Proposed Clybucca Wetlands restoration: there is a current proposal to restore the 10km² Clybucca Wetlands, located on the northern floodplain to the north of Frederickton to provide similar environmental outcomes to the Yarrahapinni Wetland restoration. This is likely to involve removal of the Clybucca floodgates along with other drainage modifications and is expected to alter flood behaviour in the northern floodplain. Works have not yet commenced.

3. Review of Available Information

3.1 Previous Studies and Reports

3.1.1 General

Numerous flood study and flood assessment reports have previously been prepared for localities in the Lower Macleay Valley, including Kempsey, the rural villages and floodplains and for the Pacific Highway Kempsey Bypass. A number of other reports have also been prepared and made available for this study including levee condition reports, geomorphology, hydrography, estuary management studies, etc. The reports most pertinent to this current flood study are summarised below.

3.1.2 Kempsey Hydraulic Model TUFLOW Update Final Report (WMAwater, 2015)

This report discusses the development of a TUFLOW model for the *Kempsey CBD Floodplain Risk Management Study and Plan*. The TUFLOW model (developed for the Kempsey bypass project for RMS) was updated with more recent LiDAR data to form the TUFLOW model used for this study. The adopted inflows were derived with the WBNM model from the *Macleay River Flood Study* (1989). The probable maximum flood (PMF) is based on a probable maximum precipitation (PMP) estimated using the Generalised Tropical Storm Method (GTSM; BOM, 2003).

3.1.3 Kempsey CBD Floodplain Risk Management Study and Plan (WMAwater, 2017)

Builds on the flood study documented in Kempsey Hydraulic Model TUFLOW Update Final Report (WMAwater, 2015). Outlines the existing flooding problem in Kempsey CBD to Frederickton area including hydraulic and hazard characterisation, flood damages, flood planning level; the catchment context (planning, social, environmental); and identifies and assesses a number of floodplain management measures.

3.1.4 Kempsey Flood Study Hydraulic Modelling Report (WMA, 2009)

Discusses the calibration and verification of the WBNM and RUBICON models from Webb McKeown and Associates (1997) to the March 2001 flood event and conversion of modelling to a 2D SOBEK model for the Macleay River from Belgrave Falls to Frederickton. The Kempsey CBD is modelled with a 10m grid, while areas outside the CBD are modelled with a 40m grid. Hydrologic details of the 2001 flood including rainfall isohyets and flood marks are provided.

3.1.5 Lower Macleay Floodplain Management Plan, Supplementary Report covering the Floodplain between Kempsey and Frederickton – Final (Webb McKeown & Associates, 2004a)

This study updated the RUBICON model from Webb McKeown & Associates (1997) with survey along No. 1 Floodway in Kempsey and cross section data of the main river between Aldavilla and Frederickton. Design flood levels and velocities were estimated for the entire floodplain from Kempsey to South West Rocks, including the ocean outlets at Hat Head and Crescent Head.

3.1.6 Lower Macleay Floodplain Management Study (Webb McKeown & Associates 1997)

This study investigates the flooding issues in the Lower Macleay floodplain at that time and presents options to improve the performance of the area during and after flooding and in non-flood periods. Hydrologic modelling - Watershed Bounded Network Model (WBNM) was used for the study based on the *Macleay River Flood Study* (Webb McKeown and Associates/NSW Public Works, 1989). Hydraulic modelling was undertaken of the floodplain downstream of Kempsey in RUBICON which is a one-dimensional hydraulic model, originally developed by Webb McKeown and Associates (1989). The design flood flows were revised higher compared to the 1989 study.

3.1.7 Macleay River Flood Study April (Webb McKeown and Associates/NSW Public Works, 1989)

This flood study estimated flood behaviour for the 5%, 2%, 1% AEP events and an extreme event for the floodplain downstream of Aldavilla and flood outlets at Ryan's Cut, Korogoro Creek, South West Rocks Creek and Killick Creek. Hydrologic modelling was undertaken in a WBNM model with the catchment divided into 35 sub-catchments for the entire Macleay River catchment to the outlet. An assumed spatial distribution of rainfall across the catchment was adopted for the design flood events. Hydraulic modelling was undertaken using a CELLS quasi-2D model.

The modelling was calibrated to the May 1963 event and verified with the August 1949, May 1977 and May 1980 events. A design 1% AEP peak flow of 15,300m³/s was estimated at Kempsey Bridge, and a peak of 13,400m³/s at Bellbrook, approximately 45km north-west of Kempsey. The latter estimate was validated using partial series flood frequency analysis of the Bellbrook stream gauge. The report stated that a hydrologic modelling approach is preferred over the flood frequency analysis for estimation of larger floods in this catchment.

The rainfall for the extreme event was estimated by BOM in 1982 based on the available techniques at the time, with a catchment-averaged 24 hour PMP depth of 464mm. The adopted extreme event flow was 40,000m³/s, which is significantly lower than the estimate of 67,500m³/s made by WMAwater in 2015. It is noted that the hydraulic modelling adopted an ocean tailwater level of 2.6m AHD for the 1% AEP design flood.

The report describes that the river entrance at its current location was significantly scoured out in the 1949 flood, as was outlets at Korogoro Creek, Killick Creek and Ryans Cut. Additionally, two flood breakouts through the beach sand dunes occurred between Crescent Head and Korogoro Point (it is not clear if these included Ryans Cut) and two to the north of Hat Head. Sensitivity to the condition of the minor ocean outlets (silted or scoured out) to flood levels was assessed, with the effects on flooding only extending up to Kinchela. Entrance and outlet conditions as surveyed in 1987 (i.e. no additional breakouts) were assumed for up to the 1% AEP design. Assumed 1949 breakout conditions were adopted for the extreme flood. This appears to involve souring and widening around the river entrance although the report does not specify the exact nature of the assumed breakout conditions.

The report also outlines a series of floodplain works which were constructed following the 1949 flood, from August 1949 to May 1980.

3.1.8 Kempsey Levee Sensitivity Assessment (Webb McKeown & Associates, 2008)

Assessment of the sensitivity of constructed levees in the Kempsey area to variations in flood gradient including the frequency of overtopping of each levee.

Compares the peak flood level and rate of rise at Kempsey Traffic Bridge for a number of historic flood events, design flood events and "modified" flood events

3.1.9 Kinchela Flood Channel EIS Stage 1 Report (Webb McKeown & Associates, 1994)

The report describes the capacity of the Kinchela floodway as being in the range of 41 to 47m³/s in a design flood of 2500m³/s at Fredrickton, depending on whether the flood lift gates are in place or removed, based on modelling in RUBICON. The capacity of the floodway assessed by the RUBICON model was approximately half of the design capacity of 71m³/s, derived using physical modelling.

The report describes the findings of a Public Works Department assessment of the Belmore River floodways to be about 400m³/s, about 70% of the design capacity of 560m³/s. The modelling also indicated that the Belmore River channel was the system constraint on flow conveyance, and not the floodway.

3.1.10 A Report on the Hydrological Impacts of Flood Mitigation Works on the Floodplain of the Macleay River Below Kempsey (University of New England, 1967)

This report describes the impacts on flooding of the flood mitigation works constructed in the floodplain starting 1955, in terms of flood levels, extent and times of inundation. While it did not expect peak flood levels to be markedly reduced by the mitigation scheme during major floods, the study concluded that there would be

significant reductions in the durations of inundation due to improved drainage. Further, the scheme would help to contain flooding within the river channel in minor flood events and reduce the degree of inundation of productive agricultural land on the floodplains in these events.

For areas of the floodplain including Clybucca – Seven Oaks, Pola Creek, Gladstone – Kinchela and Kinchela – Jerseyville areas, the study estimated the duration for floodwaters to recede to 2 feet depth (0.6m) would be reduced from up to three weeks without the mitigation scheme, down to about 1 – 1.5 weeks with the scheme.

3.1.11 Macleay River Flood March 2001 Flood Damages Data Collection (Webb McKeown & Associates 2004b)

Following the March 2001 flood event, staff from Webb McKeown & Associates undertook field reconnaissance and detailed inspection to ascertain the extent of flooding and damages. This included resident interviews and distribution of a questionnaire. The flood event was described as a 1 in 13 AEP event.

The report describes that Council undertook survey of flood and floor levels and mapped the flooding extent for Kempsey CBD, in addition to a presentation by Council on the flood behaviour, flood warnings and data collected, which was appended to the report. Observed point depths of flooding were mapped for Kempsey CBD and Smithtown. Observations of flooding in the vicinity of the Pacific Highway were reported by Roads and Traffic Authority (RTA; now Roads and Maritime Services) away from the main river including locations and timing of flooding a number of flood photographs during the event were presented.

The report also describes rainfall and river gauge data provided by BOM for the flood event. Cumulative rainfall depth and water level plots were presented for stations across the catchment. The report describes issues with the BOM data including malfunctioning of the automatic river gauge located at Kempsey Traffic Bridge some 10 – 12 hours before the peak of the flood. The report also summarises data on the flood damages and clean-up costs incurred during and after the flood event.

A review of computer modelling of the flood event was undertaken. The flow hydrograph generated by the WBNM hydrologic model at Kempsey Traffic Bridge was said in the report to have a good match to the recorded flows, although the modelled hydrograph had to be shifted by 9 hours to match the gauge records. The stage hydrographs at Kempsey Traffic Bridge from the RUBICON hydraulic model and gauge records were also compared and provided a good match.

3.1.12 Kempsey Shire Flood Emergency Sub Plan (NSW SES, 2017)

The flood emergency sub plan cover preparedness measures, the conduct of response operations and the coordination of immediate recovery measures from flooding within the Kempsey Shire area. They cover operations for all levels of flooding within the council area.

The flood emergency sub plan includes details of flooding behaviour in the Macleay Valley. It describes how the River originally discharged to the ocean in the vicinity of Grassy Heads, and during the 1893 flood the current entrance at South West Rocks was formed. The plan indicates that most of the larger floods at Kempsey have resulted from events in which significant rain has fallen over the whole of the catchment. Lesser events may occur after rain only falling over parts of the catchment. The 1949, 1950, 1963, 2001 floods all followed general catchment-wide rainfall. While the 2009 and 2011 floods had rainfall over the entire catchment area, the heaviest falls were in the lower part of the catchment.

The flood emergency sub plan describes the main mechanisms of flooding including flood inflows from the Macleay River catchment and from overflows from the Maria River/Hastings River catchment. It provides a flood history of the catchment and details of the 1949, 1950, 1963, 2001, 2009 and 2011 floods, including hydrologic conditions, flood behaviour and impacts on the community.

Details on the malfunctioning of the Kempsey river gauge are provided which may help to reconcile issues between the recorded data and the modelling results during model calibration. Timing of the flood peak and overtopping of Kempsey CBD levees is described. Gladstone, Smithtown, South West Rocks, Hat Head,

Crescent Head and many upstream and downstream rural areas are identified as being isolated during the flood event.

The levee systems throughout the Lower Macleay floodplain (including Korogoro Creek) are described. The operation of the Belmore River and Kinchela flood control structures are described, namely that when a flood warning is predicted for flood levels greater than 5.0m AHD at Kempsey Traffic Bridge gauge, Council will issue a preliminary notice that the flood control structures may need to be opened. The control gates are opened gradually in several stages as required or until full outflow is achieved. As soon as possible after the flood has receded off local roads and the level has dropped below 5.0m AHD at Kempsey gauge, the flood control structures will be progressively closed.

The figures and flood extent maps contained in the flood plan were not provided for this review.

3.1.13 Flood Levee Audit Reports (Kempsey Shire Council, 2015/2016)

Nine (9) levee audit reports are available for the flood levee system in the Lower Macleay floodplain, including around Kempsey. The reports indicate the surveyed crest levels and condition of the levee banks which can be used to verify the LiDAR data along the levees, or to help to accurately define the crest levels in the hydraulic modelling. Survey was undertaken in 2014.

3.2 Flood Models

As outlined in Section 3.1, a number of hydrologic and hydraulic models were developed over the years for the purposes of the various studies undertaken. The different models were developed based on the software and techniques available at the time of each study and were updated to provide better detail at various times. Hydraulic models developed include CELLS (1980s), RUBICON (early 1990s), SOBEK (c. 2009) and recently TUFLOW. Generally, details on the topography, waterways and hydraulic structures were transferred from one model to the next as upgrades occurred.

Key TUFLOW models which have been developed by consultants WMAwater for the Lower Macleay floodplain have been obtained from Council and RMS. These are summarised in Table 3-1 in order of priority, as determined by their date of development and adoption in Council's floodplain management program. The model extents and modelled hydraulic structures are indicated on Figure 3-1. Culverts and drainage structures represented in the models are shown on Figure 3-6 along with other culverts in Council's spatial layers outside the model domains. A number of the models have similar extents to the Kempsey CBD model, confirming that the model for that area was updated over time to suit the purposes of each separate flooding assessment.

There is an extensive gap along the F2E section of the Pacific Highway upgrade between Model 1 and Model 2 and covering the Collombatti Creek floodplain. The *Pacific Highway Upgrade Frederickton to Eungai Flood Study Report – Developed Concept Design 15%* (HAJV, 2013) states that as this area is largely influenced by the Macleay River backwater flooding and no detailed hydraulic modelling was undertaken as a part of the F2E project. Data on the hydraulic structures on the Pacific Highway was extracted from the RMS work-as-executed information for this study's modelling, refer to Section 3.9.

Table 3-1 Review of key existing TUFLOW models for Lower Macleay

ID/ Priority	Study Model and Date	Development Scenario	Comment
1	Kempsey CBD FRMS June 2017	Existing case and Proposed mitigation case	Adopted model and design flood estimates by Council. Includes Macleay River bridge with updated details compared to model ID 4 (model 4 has an embankment/break in the road viaduct halfway along the floodplain). Includes road cross drainage on major and some local roads. The modelled Macleay River bridge details are inconsistent with the 2016 LiDAR elevations of the bridge approaches (bridge deck uniform 7.5m AHD, soffit uniform 6.7m AHD; northern bridge approach 10.8m AHD, southern bridge approach 8.5m AHD). Hence there is inconsistency of 1 – 2.3m in the level of the deck. This will not affect the design flood estimates for up to the 0.5% AEP flood (flood level below soffit) but would impact on flood hydraulics for rarer floods. River bathymetry based on NSW Office of Environment and Heritage (OEH; now Department of Planning, Industry and Environment) data.
2	Pacific Highway Upgrade Frederickton to Eungai (F2E) – Model 1 and Model 2 February 2013	Developed case (Detailed Design)	Collombatti Creek floodplain not modelled. A number of culverts and bridges represented.
3	South West Rocks Road Upgrade 2010 – 2013	Developed case (3 road raising preliminary options)	Flood assessment involving uniform raising of South West Rocks Road by 100mm, 200mm and 300mm.
4	Pacific Highway Upgrade Kempsey to Frederickton (Macleay River bridge) June 2010	Developed case	Kempsey CBD model considered to supersede this model.



GIS MAP file : IA176800_R04_Figs-1_TUFLOW_Models_01

Legend

- Kempsey CBD FRMS Model
- Pacific Highway Upgrade F2E Model 1
- Pacific Highway Upgrade F2E Model 2
- SWR Road Upgrade and Kempsey to Fredrickton Models
- Study Area



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A3

SHEET	1 of 1	GDA 1994 MGA Zone 56		
TITLE		Existing TUFLOW Model Extents		
PROJECT Lower Macleay Flood Study				
DRAWN	PROJECT #	MAP #	REV	VER
LC	IA176800	Figure 3-1	1	1
CHECK	DATE			
AH	9/09/2019			

3.3 Flood Photography

Council provided oblique aerial photographs, captured by Council staff from a helicopter during the 2013 flood event, showing inundation of villages and the floodplain and flood behaviour in various parts of the floodplain.

Aerial photography of the 2001 flood is also presented in Webb McKeown and Associates (2004b) although the majority of this photography is focussed on Kempsey CBD area with a limited number of photos of the Lower Macleay floodplain.

3.4 Rainfall Data

Sub-daily rainfall gauges located in the vicinity of the study area are shown on Figure 3-2. Data from sub-daily rain gauges located in the vicinity of the study area have been obtained from Bureau of Meteorology (BOM) and Manly Hydraulics Laboratory (MHL). The data was collected for the March 2001 and February 2013 calibration events, and were used to generate local catchment runoff in the model calibration and verification.

Sub-daily rainfall has been obtained from BOM for the Kempsey (Wide Street) gauge for the period of record (December 1956 to September 2012).

The sub-daily rainfall for the historic storm events are supplemented by daily rainfall for BOM gauges at Collombatti, Fishermans Reach, South West Rocks (Smoky Cape), Eungai (Searles Road) and Crescent Head, also shown on Figure 3-2. The daily rainfall data was discretised with the pluviograph temporal patterns for the model calibration.

Temporal distribution of rainfall recorded in the Lower Macleay floodplain for the nominated calibration/verification flood events of 2001 and 2013 are shown in Appendix A.



GIS MAP file : IA176800_R04_Fig3-2_RainGauges_02

Legend

- Rain Gauges near Study Area**
- MHL Sub-daily
 - BOM Sub-daily
 - BOM Daily
 - Study Area



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A3

SHEET	1 of 1	GDA 1994 MGA Zone 56			
TITLE		Rain Gauges near Study Area			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
LC	IA176800	FIGURE 3-2	2	1	
CHECK	DATE				
AH	3/09/2019				

A3

3.5 Stream Gauge Data

3.5.1 Peak Flood Levels

Council provided a summary of peak flood levels recorded at BOM water level gauges, which were confirmed by SES. This is presented in Table 3-2. From the available data, it appears that single asterisk notation denotes that the telemetered gauge failed and a manual reading was taken. The March 2018 flood level was noted by Jacobs from the telemetered gauge readings on the BOM website during the event.

An additional spreadsheet file was provided by Council summarising the peak flood levels and time of peak at the various river gauges for the August 1949, May 1963, February 1997 and March 2001 floods and then numerous floods between January 2008 and March 2013. The reported flood peaks vary slightly from the BOM values below.

Table 3-2 Historical flood heights (m AHD) from BOM

Date	Georges Creek	Bellbrook	Kempsey	Smithtown
1838	-	-	7.2	-
1841	-	-	7.2	-
Aug 1848	-	-	6.3	-
Feb 1863	-	-	6.64	-
Feb 1864	-	-	7.94	-
July 1864	-	-	6.8	-
Aug 1864	-	-	7.84	-
July 1866	-	-	6.8	-
April 1867	-	-	7.2	-
March 1870	-	-	6.8	-
March 1875	-	-	8.14	-
June 1879	-	-	6.6	-
March 1890	-	-	6.6	-
April 1892	-	8.8	6.6	-
March 1893	-	-	7.2	-
June 1893	-	17.1	8.14	-
July 1921	-	16.16	7.84	-
Feb 1928	-	-	6.14	-
Feb 1929	-	12.20	6.84	-
March 1946	-	12.73	6.10**	-
Aug 1949	14.10	17.22	8.52	-
June 1950	-	18.06	8.31	-
Aug 1952	-	13.03	6.56	-
Feb 1954	-	11.23	6.36	-
Nov 1959	-	9.75	6.43	-
April 1962	-	8.15	6.08	-
May 1963	13.50	15.54	7.68	Estimated 4.50
June 1967	-	10.24	6.56	-

Jan 1968	-	8.84	6.31	-
Jan 1974	8.63	7.70	6.10	-
March 1974	6.78	7.11	6.23	-
Feb 1976	8.64	7.56	6.05	-
May 1977	7.60	6.75	6.11	-
May 1980	7.00	7.14	6.27	-
April 1989	6.72	6.34	6.11	-
March 2001	11.90	12.80	7.44*	Estimated 4.34
May 2009	10.77	11.40	6.90*	Estimated 4.29
June 2011	11.0*	10.65*	6.70*	Estimated 4.21
Feb 2013	12.14	11.30	7.11 ¹	
March 2018			3.85	

Note 1: Kempsey Traffic Bridge continuous water level gauge recorded a peak flood level of 6.92m AHD for February 2013 flood event.

3.5.2 Continuous Streamflow and Water Level Data

Available streamflow and water level recording stations in the vicinity of the study area are shown on Figure 3-3. PINEENA (a surface water and groundwater monitoring data base published by DPI Water) version 10.2 data base contains continuous water level and streamflow data for non-tidal river gauges on the Macleay River upstream of Kempsey. The data from the Turners Flat gauge was extracted and is shown in Figure 3-4 for the February 2013 event.

Water level data for river gauges in the tidal reaches of the Macleay River and Hastings River system were obtained from MHL for the following gauges:

- Aldavilla
- Borirgala Creek
- Crescent Head
- Dennis Bridge Downstream (Hastings River)
- Euroka Upstream
- Green Valley (Maria River)
- Hat Head
- Kempsey
- Smithtown
- South West Rocks

Telegraph Point (Wilson River). MHL also operates a number of additional water level gauges in and around the study area for the Kempsey Shire flood warning system (<https://mhlfit.net/users/KempseyShireCouncil/>). These additional gauges are also indicated on Figure 3-3

Tidal gauge data was obtained from MHL for the Coffs Harbour and Port Macquarie tide gauges for historic ocean water levels.

The streamflow, river and tidal water level data were extracted for the nominated calibration and verification flood events (March 2001 and February 2013).



GIS MAP file : IA176800_R02_Fig2-3_StreamGauges_01

Legend

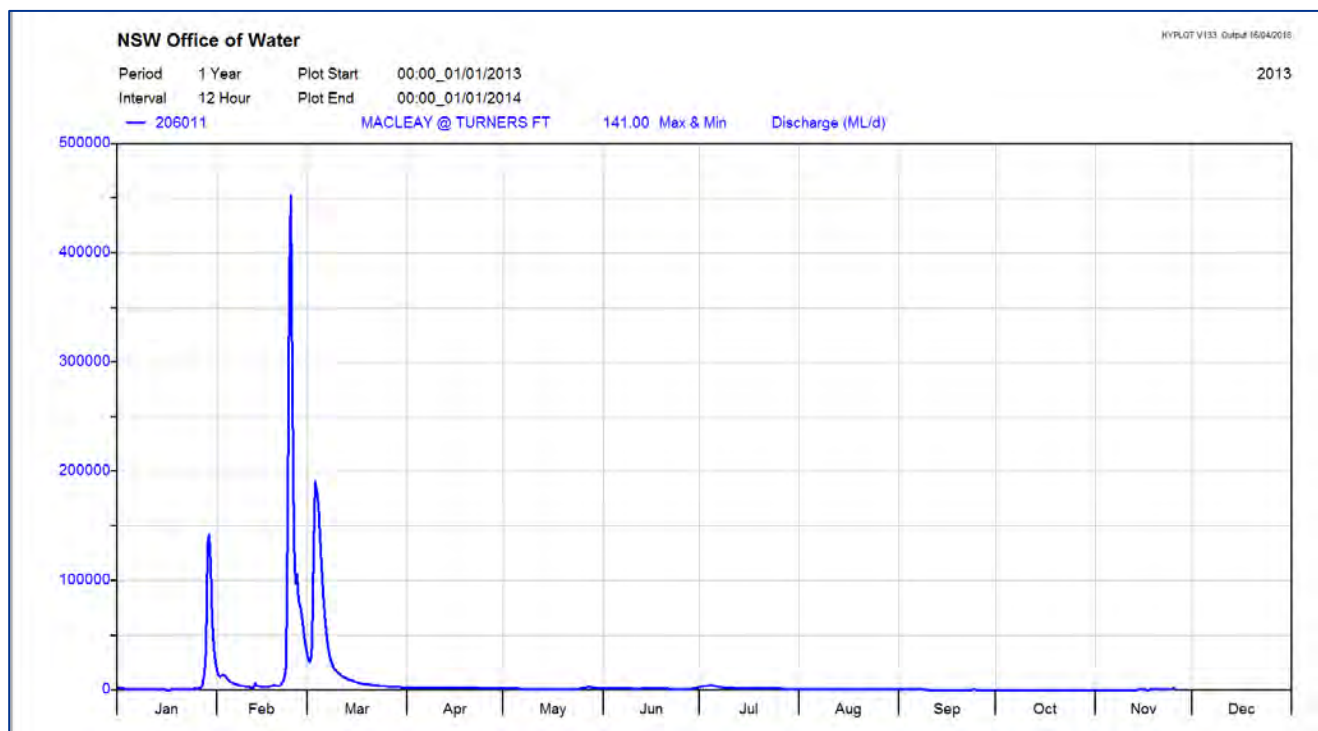
-  Water Level Gauges near Study Area
-  Study Area



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A3			
SHEET	1 of 1	GDA 1994 MGA Zone 56	
TITLE	Stream Gauges near Study Area		
PROJECT	Lower Macleay Flood Study		
DRAWN	PROJECT #	MAP #	REV VER
MR	IA176800	FIGURE 3-3	1 1
CHECK	DATE		
LC	13/10/2018		

Figure 3-4 Turners Flat Streamflow Data – February 2013 Flood Event



3.5.3 Note on Change of Stream Gauge Datum

Flood levels at Kempsey Traffic Bridge gauge were historically reported to the local gauge datum. From 12 November 2003 reporting of water levels at this gauge was changed to Australian Height Datum (AHD). All references to water levels and flood warnings following this data have been issued to AHD. Australian Height Datum is a common survey datum used throughout Australia with levels measured above mean sea level.

This change was made to provide consistency with flood modelling and development controls, provide a common reference for all gauges downstream of Kempsey, and to help eliminate confusion and errors in reporting and referencing water levels at this gauge.

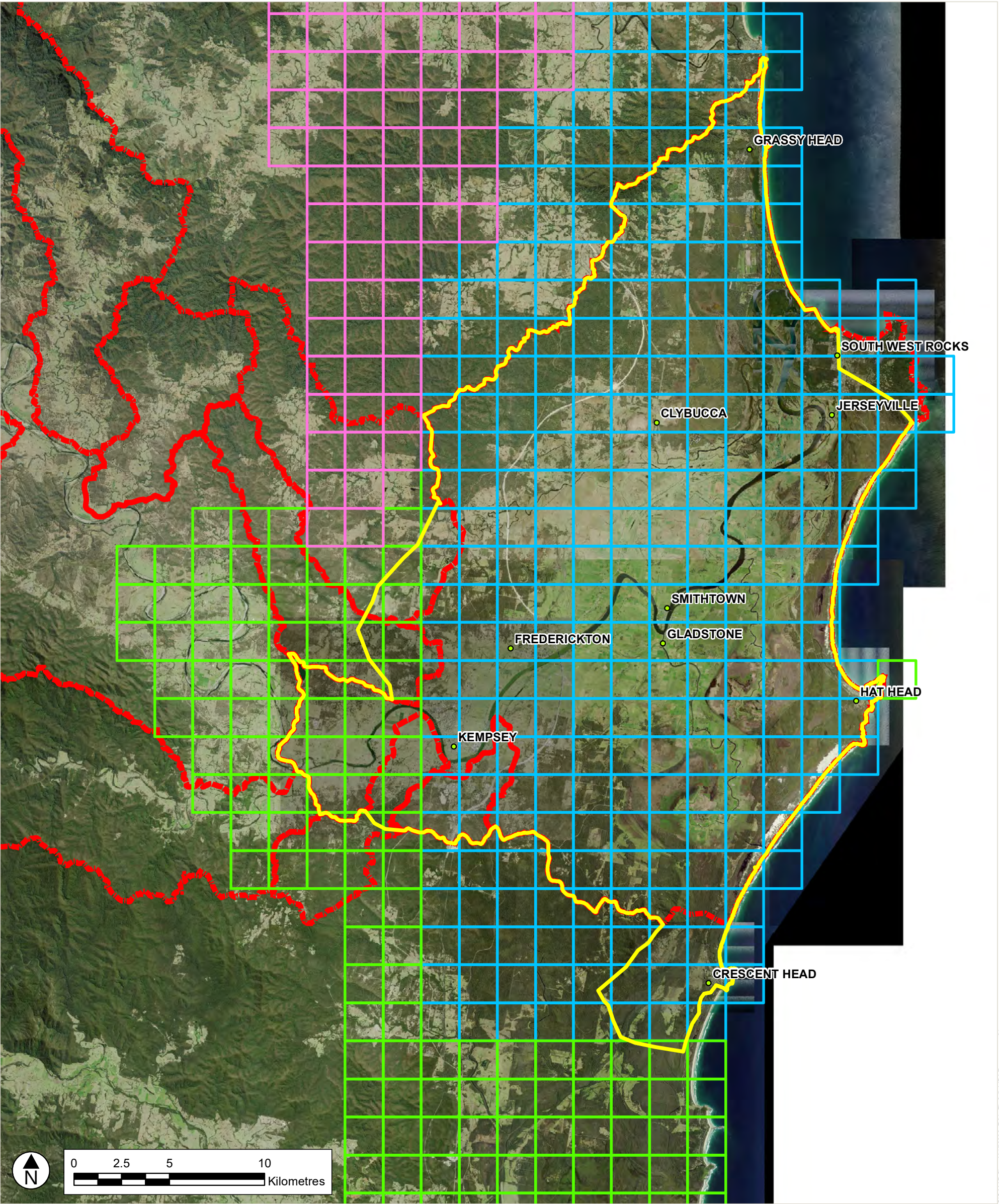
The old gauge zero is at 0.53m AHD. To obtain water levels to AHD, add 0.53m to levels referencing the old gauge datum.

3.6 Floodgate Management

Council provided a spreadsheet summarising the floodgate operation for flood events from February 2009 to March 2013 including opening and closing times and the coinciding flood level at Kempsey Traffic Bridge at that time. Structures covered include Belmore floodgate, Belmore flood control structure, Kinchela floodgates, Kinchela Left and Right Bank flood control structures, and Euroka Creek and Christmas Creek floodgates.

3.7 Topographic Data

Council provided the topographic data in the form of Digital Elevation Model (DEM) of 1m, 5m and 10m spatial resolution for the Kempsey, Macleay River and Nambucca areas based on LiDAR survey of the study area. Years of capture are 2009/2010, 2016 and 2009/2010, respectively. The data coverage is shown on Figure 3-5. The topographical data covers the entire study area (including the areas to the south of Crescent Head and extending into the Hastings River catchment). Survey data of the flood levees in the study area, collected for the levee audit reports, was provided by Council.



GIS MAP file: IA176800_R01_Fig2-5_LiDARcoverage_01

Legend

- Study Area - Draft_region
- LiDAR - Macleay River 2016
- LiDAR - Kempsey 2009/10
- LiDAR - Nambucca 2009/10
- Macleay River Sub-Catchments



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SHEET 1 of 1				GDA 1994 MGA Zone 56	
TITLE LiDAR Data Coverage					
PROJECT Lower Macleay Flood Study					
DRAWN	PROJECT #	MAP #	REV	VER	
MR	IA176800	FIGURE 3-5	1	1	
CHECK	DATE				
LC	16/04/2018				

3.8 Bathymetric Data

Bathymetric data held by the former Office of Environment and Heritage (OEH) was downloaded from their website <http://www.environment.nsw.gov.au/estuaries/list.htm>. The data set consists of spot soundings captured in 2001 to 2005. Waterways surveyed include Macleay River, Belmore River, Kinchela Creek, Spencers Creek, Clybucca Creek, Korogoro Creek and Killick Creek. On the Macleay River survey was undertaken along river cross sections at 100 – 250m intervals upstream of Jerseyville, and point-cloud data at typically 10m spacing in the lower river downstream of Jerseyville. The OEH bathymetric data was processed to form digital elevation models (DEMs) of the waterway channel beds for input into the TUFLOW hydraulic model.

3.9 Pacific Highway Upgrade Design Data

In addition to the TUFLOW hydraulic models for the various sections of the Pacific Highway upgrade as discussed in Section 3.2, additional detailed design and work-as-executed design data was obtained from RMS for F2E and Kempsey Bypass. Details of the road geometry and hydraulic structures from these sources supplements those in the existing TUFLOW models. Detailed design and WAE plans for the Pacific Highway Macleay River Bridge were not available for this study.

3.10 Aerial Imagery

Aerial imagery plates were provided by Council for various locations across the study area. The dates of capture range from 2010 to 2014 and are of varying resolution from 10cm to 50cm.

3.11 Topographic and Hydrographic Survey

Survey of selected culverts, floodgates and flood control structures was commissioned during July/August 2018 for this flood study to collect level and dimensions data where data gaps were identified.

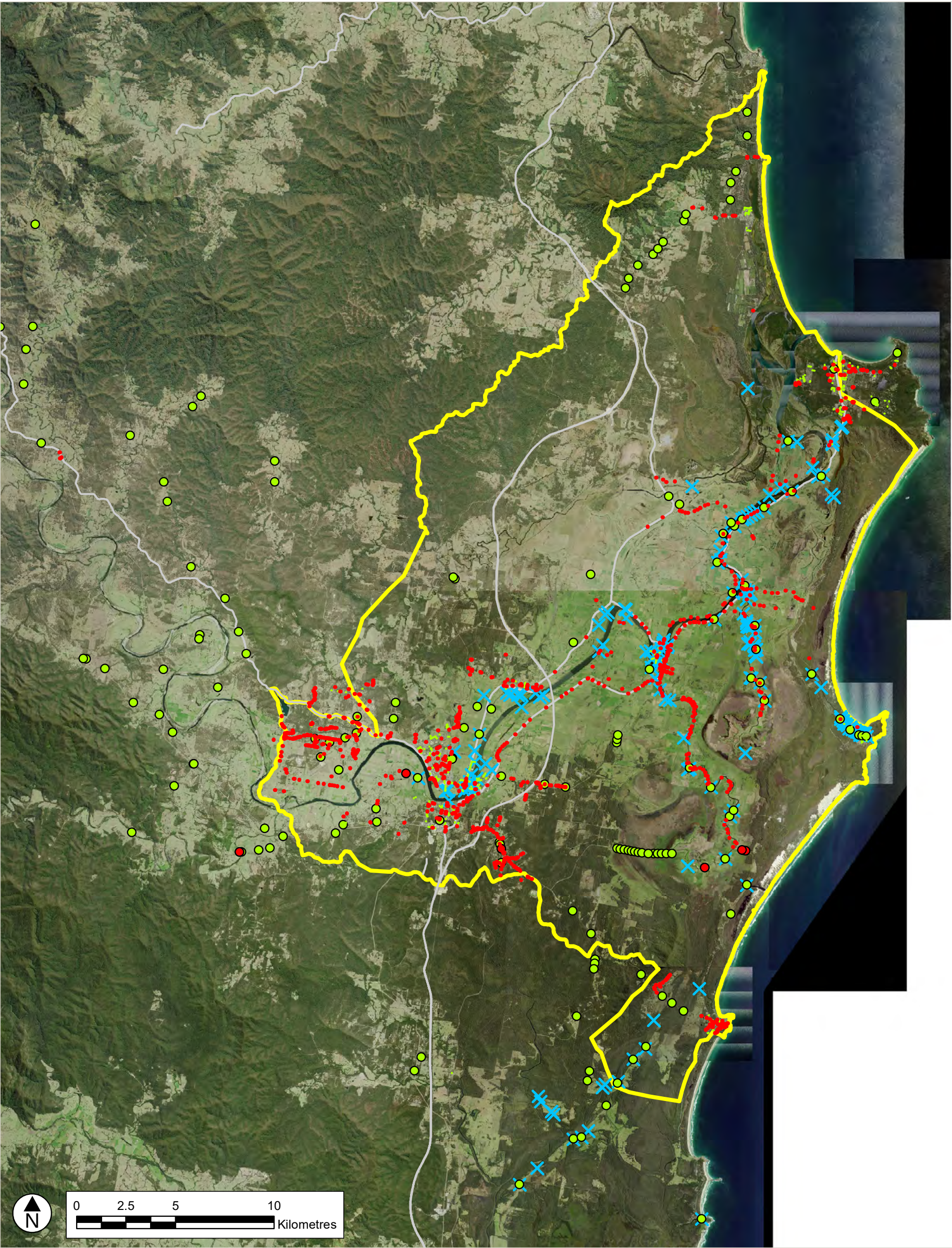
Hydrographic survey of selected locations in the main river was also commissioned to validate and supplement the 2003-2005 OEH bathymetric survey data. This recent survey was collected in the vicinity of the villages where changes in the river bed may impact on flooding estimates in the villages.

Channel cross section data of Collombatti Creek and Seven Oaks Drain collected by UNSW Water Research Laboratory (WRL, c. 2016-2018) for development of a MIKE-FLOOD model of the Clybucca Wetlands was obtained for the current TUFLOW hydraulic model.

3.12 Spatial Data Provided by Council

Spatial data provided by Council includes:

- Floodgates (Council and joint-owned). Floodgates are shown on Figure 3-6.
- Drains (Council and joint-owned).
- Culverts and stormwater pipes. The locations of culverts and pipes larger than 600mm are shown on Figure 3-6. Key features have been surveyed.
- Levees.
- Bank rock protection along the Macleay River banks
- Timber bridges crossing the floodplain drains. These are typically minor structures.
- House levels and floor levels for properties on the floodplain.
- Streams and rivers
- Flood planning level/area (currently adopted by Council)
- Properties marked as affected in the March 2001 flood event.



GISMAP file: IA176800_R01_Fig2-8_Culverts_01

Legend

- Study Area
- Floodgates
- Culverts
- Culverts No Data
- Pipes larger than 600mm
- Pipes No Data



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SHEET		1 of 1		A3	
		GDA 1994 MGA Zone 56			
TITLE		Culverts and Floodgates			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
LC	IA176800	Figure 3-6	1	1	
CHECK	DATE				
AH	18/04/2018				

4. Hydrologic Assessment

4.1 Approach

Flood flow estimates for this study were derived by the following methods:

- Macleay River inflows from the upstream catchment: River flows entering the study area (from upstream of Euroka) are based on the inflow hydrographs adopted in the Kempsey CBD modelling (WMAwater, 2015) or by gauged river flows from the stream gauge at Turners Flat, for historic events.
- Local catchment runoff into and onto the floodplain in the study area: A DRAINS/RAFTS hydrologic model has been developed to simulate historic calibration flood events and the design flood events, estimating catchment inflow hydrographs based on rainfall data for recorded or design flood events.

Aspects of these two hydrologic assessment components are discussed in this section.

4.2 Macleay River Inflows

4.2.1 Historic Flood Events

River inflows into the study area as adopted in the Kempsey CBD flood modelling (WMAwater, 2015) were compared to gauged flows at Turners Flat, approximately 25km further upstream. Refer to Table 4-1. The flows adopted in WMAwater (2015) are based on a calibrated WBNM hydrologic model of the catchment. Gauged flows were extracted from PINNEENA.

Table 4-1 Comparison of gauged flows and flood study adopted flows for historic flood events (m³/s) upstream of Kempsey

Event	Gauged (Turners Flat)	WMAwater 2015	Difference
March 2001	8,366	8,006	4.3%
February 2013	5,399	5,880	8.2%

There is a minor difference (-4.3%) in the peak flow between the gauged and the flood study adopted river flow for the March 2001 event. Comparison of the flow hydrographs in Figure 4-1 indicates a very similar shape and flood volume.

The difference in peak flow in the February 2013 event is more substantial (8.2%). Additionally, comparison of the flow hydrographs in Figure 4-2 indicates a significant difference in flood volume. This may be partly due to gauging issue, with a discontinuity shown on the gauged line on 23/02/2013. Additionally, a review of rainfall data indicates that a significant amount of rainfall fell on sub-catchments downstream of the Turners Flat gauge during the flood event, which may explain the higher modelled flows.

The adoption of river inflows for the calibration and verification of the current modelling is discussed in Section 6.

Figure 4-1 Comparison of gauged and flood study adopted flows upstream of Kempsey – March 2001 event

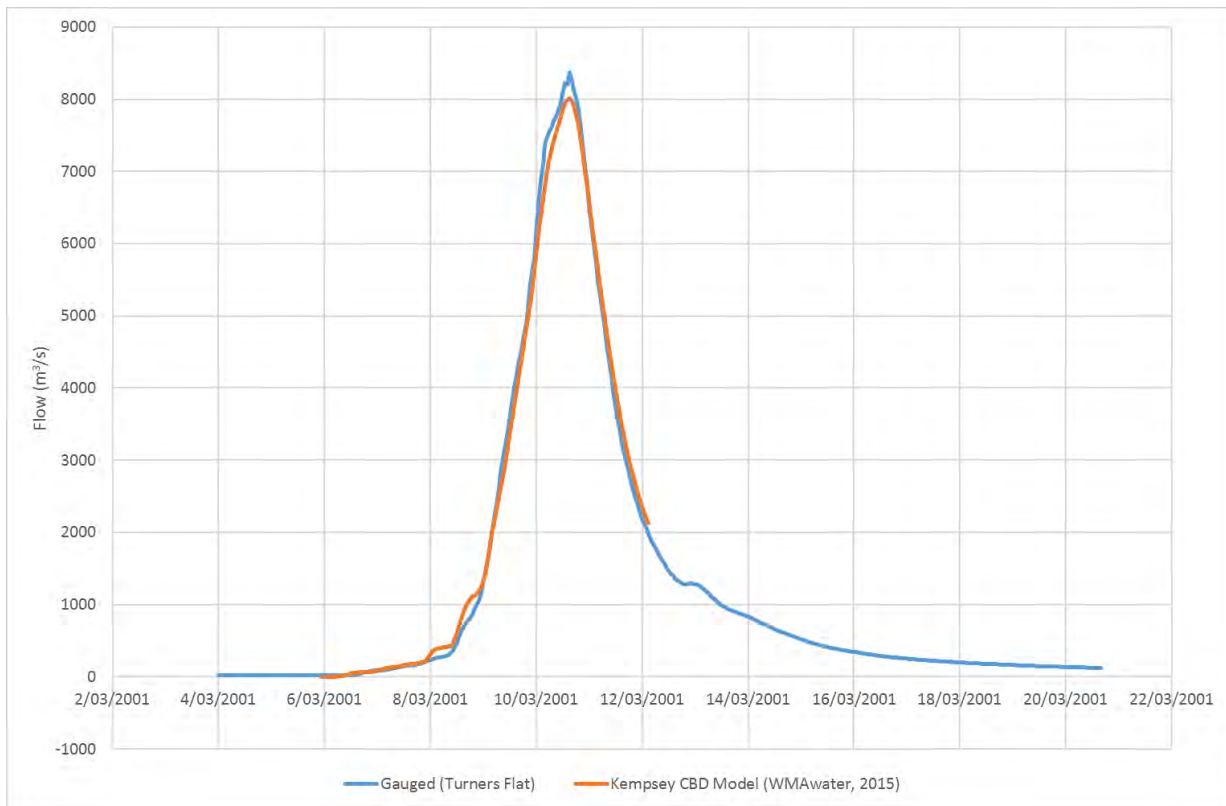
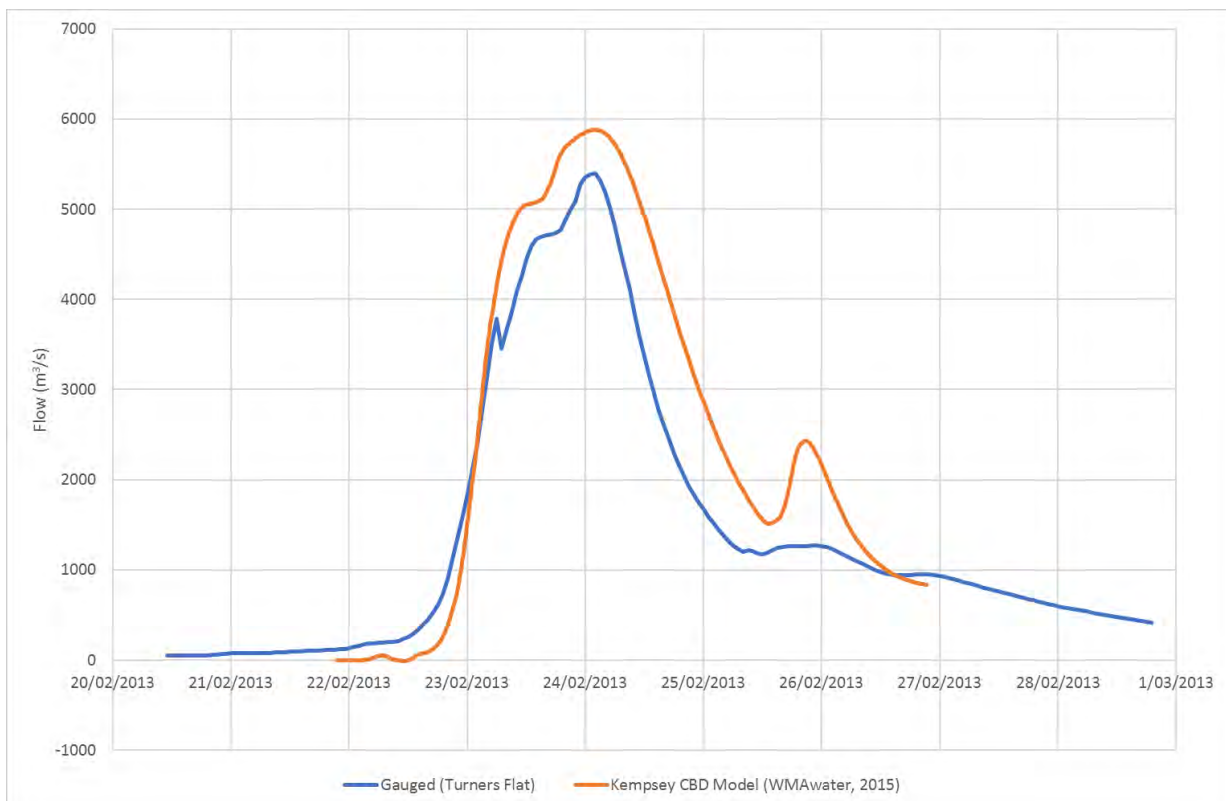


Figure 4-2 Comparison of gauged and flood study adopted flows upstream of Kempsey – February 2013 event



4.2.2 Previous Estimates for Design Flood Events

Design flood flows at Kempsey were estimated in previous flood studies. Flows were initially estimated based on at-site flood frequency analysis of flow data at Bellbrook gauging station, located approximately 45km upstream of Kempsey. This gauging station was open from 1937 to 1984. A WBNM hydrologic model of the Macleay River catchment was then developed and validated to this stream gauge to estimate the design flood flows for the full range of design flood events. The WBNM model was considered to be more reliable than the flood frequency analysis for flood events rarer than 4.5% AEP and hence was adopted as a preferred method. Refer to Macleay River Flood Study (Webb McKeown and Associates, 1989) for further details.

Further refinement to the design flow estimates were made with the updated flows adopted in the *Kempsey Flood Study Hydraulic Modelling Report* (Webb McKeown and Associates, 2009) and then the *Kempsey Hydraulic Model TUFLOW Update Report* (WMAwater, 2015). The latter are the current design flood estimates and have been adopted in this study, refer to Table 7-1.

While the WBNM modelling was undertaken in the previous studies based on ARR 1987 design rainfall data, it was not considered necessary to update the hydrologic modelling based on the current ARR 2016 design rainfalls. Any updated hydrologic modelling with ARR 2016 rainfall would be calibrated to the same flood frequency analysis design peak flows as the previous ARR 1987 – based modelling. Hence, the design flood flows would not be changed as a result.

4.2.3 Updated Validation of Design Flows

An attempt was made to validate the design flood estimates based on an updated flood frequency analysis during this study. However, a number of issues were identified:

- No additional data was available for the Bellbrook gauging station following the Webb McKeown and Associates 1989 Flood Study. It was reinstated some time after it was closed in 1984 as a floodwatch gauge, but there is uncertainty whether the new gauge was installed in the same location where the old gauge was located.
- A partial series analysis of available data was undertaken using the Turners Flat gauging station data, however, this did not include the two largest floods on record including the 1949 and 1950 events, thus skewing the analysis. As a result, the analysis gave a 1% AEP peak flow estimate less than half of the adopted design flow.
- Although there is a significant period of record for the Kempsey Traffic Bridge gauge with data extending back to 1863, there are concerns with the rating table and flood height series at this site due to non-homogeneity of recorded data due to ongoing development changing the hydraulic characteristics, and quality of the rating table due to no gauging being undertaken at higher flows (Webb McKeown and Associates, 1989).

Hence, an updated validation of the design flood estimates could not be undertaken.

4.2.4 Recommendations for Future Validation and Update of Design Flow Estimates

In light of the constraints discussed above preventing the validation of design flows at this time, it is recommended that the following actions be considered:

- Confirm that the reinstated and currently active Bellbrook gauging station is in the same location as the historic gauge. This would allow extension of the previous streamflow data series with more recently recorded data.
- Review and update of the rating curve for Bellbrook gauging station.
- Consultation with relevant agencies including WaterNSW to conduct the above two actions.
- Review the flood frequency analysis for Bellbrook gauging station and rerun WBNM catchment hydrologic model using ARR 2019 to update design flow estimates at downstream locations including Kempsey.

4.3 Hydrologic Modelling of Local Catchments

4.3.1 Model Selection

Hydrologic modelling of the local catchments within the study area is undertaken using the RAFTS hydrology module in the DRAINS modelling software. The DRAINS software is one of the few modelling packages that currently incorporate Australian Rainfall and Runoff 2016 (ARR 2016¹) design rainfalls and procedures.

4.3.2 Sub-Catchment Data

The catchment area downstream of Aldavilla is subdivided into over 900 sub-catchments which drain into the study area. Mapping of the sub-catchment boundaries is shown on Figure 4-3. Sub-catchments were generated based on SRTM topographic data of the catchment. Land use was identified from review of aerial photography.

The area modelled in the hydrologic model is predominantly rural with some areas of forest and watercourses and small areas of urban or village development. A PERN catchment roughness value of 0.05 was adopted as the typical roughness parameter value for the catchment.

An impervious fraction of 0% was adopted for the typically rural and vegetated floodplain areas. Watercourses were assumed to be effectively 100% impervious areas and the impervious fraction of sub-catchments falling on watercourse areas were increased accordingly. The impervious fractions for sub-catchments covering the developed areas of Kempsey were estimated based on the paved and building areas digitised for the previous Kempsey CBD TUFLOW model. Sub-catchments in these areas have impervious fractions of up to 77%.

4.3.3 Hydrologic Parameters

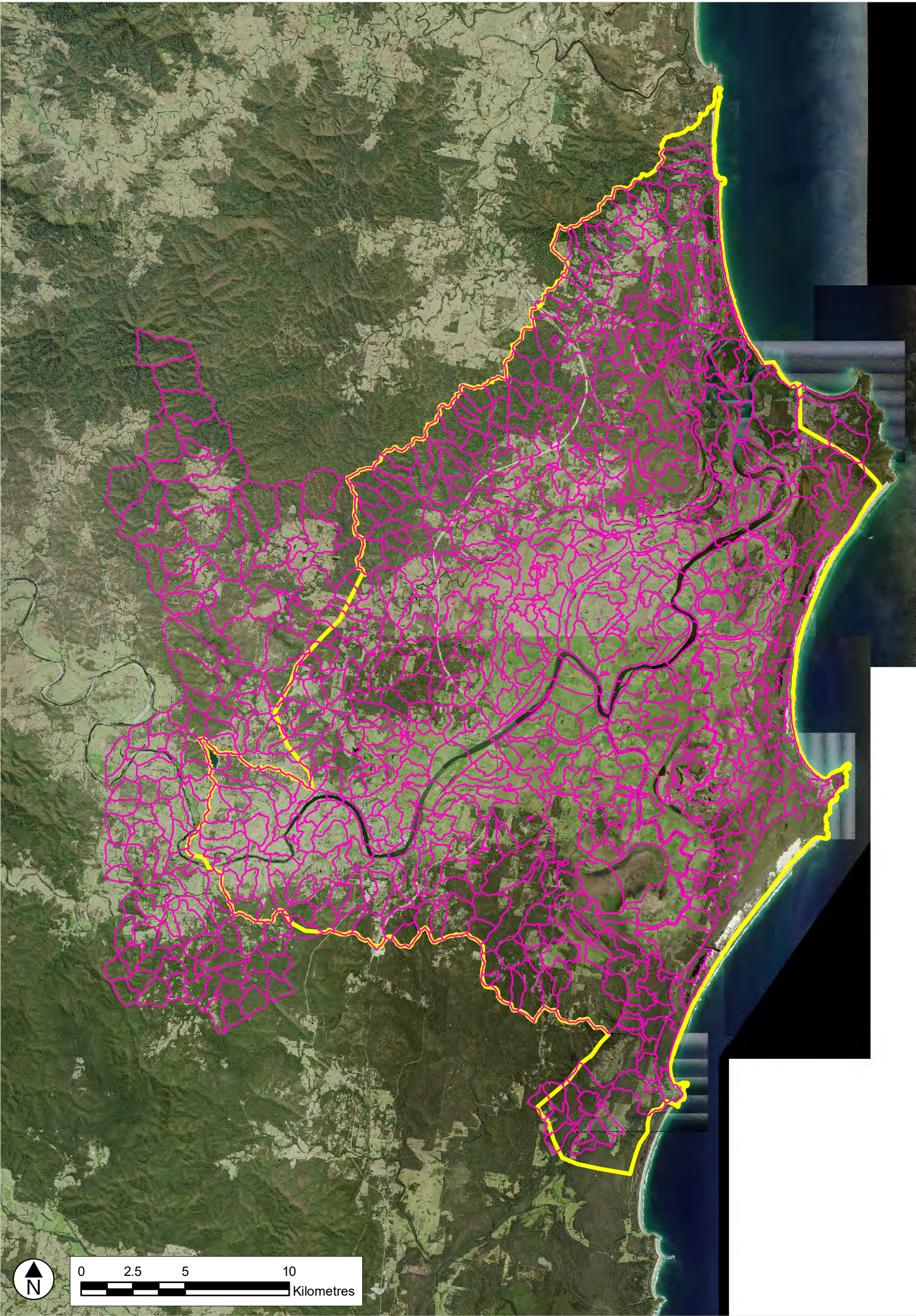
4.3.3.1 Rainfall Losses

Previous flood studies for Kempsey and the Lower Macleay Valley (Webb McKeown and Associates, 2009; WMAwater, 2015) derived the following rainfall losses for hydrologic modelling, which have been adopted in this current study. These loss parameter values were selected by the previous studies based on calibration of the hydrologic modelling to a number of historic flood events. While the adopted values do differ from the regionally-derived values recommended by ARR 2016, the adopted values are considered appropriate for this flood study given that they are catchment-specific and based on actual major flooding events in the Macleay catchment.

Table 4-2 Rainfall Losses

Flood Event	Rainfall Losses	Comment
Design flood events	IL 0mm, CL 2mm/hr	Based on model calibration to 1949, 1963, 1977 and 1980 historic flood events in Macleay River Flood Study (Webb McKeown and Associates, 1989). Lower and more conservative than ARR 2016 design rainfall losses.
March 2001	IL 70mm, CL 0.7mm/hr	Same as those in calibration of the WBNM hydrologic model in Kempsey Flood Study Hydraulic Modelling Report to 2001 event (Webb McKeown and Associates, 2009)
February 2013	IL 15mm, CL 0.7mm/hr	Initial loss selected from review of rainfall data prior to main burst on 22-23 February. Note that 50mm (inland areas) to 200mm (coastal area) of rain fell in the week prior to the 22 February.

¹ Australian Rainfall and Runoff 2016 design rainfalls and industry guidance has since been finalised in early 2019 during the advanced stages of this flood study, and is now referred to as Australian Rainfall and Runoff 2019 (ARR 2019).



GIS MAP file : IA176800_R04_Fig4-3_SubCatch_01

Legend

-  Study Area
-  Local Sub-Catchments



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SHEET		1 of 1		A3	
		GDA 1994 MGA Zone 56			
TITLE		Study Area Local Sub-Catchments			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
LC	IA176800	FIGURE 4-3	2	1	
CHECK	DATE				
AH	30/08/2019				

4.3.3.2 Storage Routing Factor

RAFTS includes the “Bx” storage routing factor which can be adjusted to change the runoff response of the catchment. With a default value of 1.0, the factor can be reduced to increase the runoff response, resulting in a more peaky flood. It is usually adjusted when there is sufficient data, such as flow gauging, to validate the adjustments. The default Bx value of 1.0 was retained in this study.

4.3.4 Flow Routing

Flows were routed between sub-catchment nodes in the DRAINS/RAFTS model using the kinematic wave method. Note that sub-catchment flow hydrographs from the DRAINS model were directly input and routed in the TUFLOW model for the large majority of the sub-catchments, with routed flows from the DRAINS model input at the boundary of the TUFLOW model in Collombatti Creek, Christmas Creek and an unnamed flow path in the south-west of the model domain.

4.4 Comparison of Design Rainfall Depths

A comparison between rainfall frequency derived using data recorded at rain gauges in and around the Lower Macleay study area and design rainfall depths from ARR 1987 and ARR 2016 was undertaken. In certain locations in NSW (Illawarra Escarpment, Coffs Harbour region) steep gradients in rainfall spatial patterns occur due to orographic and other topographic influences on rainfall storms, causing sharp differences in the rainfall depths experienced between nearby locations. The statistical analyses used to derive the ARR design rainfall estimates may sometimes reduce the definition of this steep rainfall gradient. Review of the rainfall data recorded at individual gauging stations may reveal these anomalies.

At-site rainfall frequency analysis was undertaken using daily rainfall data from BOM rain gauges across the Lower Macleay, capturing a mixture of inland versus coastal and lowland versus hilly areas. Details on the gauges selected for the analysis are summarised in Table 4-3. Pluviograph data from Kempsey Airport was considered for assessment for sub-daily durations, but was found to have a significant data missing throughout the period of record and was thus not considered further. In any case, storm durations exceeding one day are more significant as flood-producing events in the Lower Macleay floodplain than sub-daily events.

Table 4-3 Selected rain gauges for IFD comparison

Gauge Name	Number	Period of Record	Length of Record
Kempsey (Wide Street)	059017	1882 – 2015	133 years
South West Rocks (Smoky Cape)	059030	1939 – 2018	79 years
Eungai (Searles Road)	059109	1973 - 2004	30 years

The daily data was analysed to assess storm durations of 1 to 5 days. An annual series analysis was conducted where the annual maximum rainfall depths recorded over various storm durations (e.g. maximum 1-day depths during each year) were calculated and ranked. The Cunnane plotting position for a simple distribution free formula ($P_i = (i-0.4)/(n+0.2)$, where the largest data value is assigned a rank $i=1$ and n is the sample size of the data set) was used to assign a plotting position to each annual maximum rainfall. The at-site data for each rain gauge, in terms of storm AEP versus rainfall depth, was plotted and compared to both ARR 1987 and ARR 2016 design rainfalls for storm durations of 1 to 5 days on the plots in Appendix B.

The following key observations are made:

- ARR 1987 design rainfall is generally similar to ARR 2016 design rainfall at Eungai and Kempsey, but lower than ARR 2016 at Smoky Cape.

- The at-site design rainfall is lower than or similar to ARR 2016 at all stations and for all storm durations assessed. This is attributed to the ARR 2016 analyses pooling data from a number of nearby stations, which increases the likelihood that higher rainfall storms are captured, thus increasing the design rainfall estimates. A number of isolated intense storm events are recorded in the at-site data which exceed the ARR 2016, although if a theoretical probability distribution line were to be fitted to the at-site data the line would sit below the ARR 2016 line.
- The at-site design rainfall estimates are generally similar to, or less than, the ARR 1987 design rainfall, with the exception of Smoky Cape for 4 and 5 day durations.

In recognition of uncertainties involved in rainfall frequency analysis, ARR 2016 design rainfall estimates are adopted in this study as the ARR 2016 design rainfall estimates are based on a more extensive database, with more than 30 years of additional rainfall records and inclusion of data from an extra 2,300 rainfall stations across Australia than ARR 1987. By combining contemporary statistical analysis with this expanded rainfall database, the ARR 2016 provides more accurate design rainfall estimates for Australia hence was considered appropriate for use in this study.

5. Hydraulic Modelling

5.1 Model Selection

A TUFLOW combined one-dimensional (1D) and two-dimensional (2D) hydrodynamic model was developed for this study. TUFLOW is an industry-standard flood modelling platform, which was selected for this assessment as it has:

- Capability in representing complex flow patterns in the study area. Examples of such complex flows include dispersed flows on the floodplain, changes in flow direction in channel bends, levee overtopping and flood storage.
- Capability in representing one-dimensional drainage elements including finer-scale drains and channels, culverts, floodgates and flood control structures.
- Capability in modelling bridge and levee structures.
- Dynamic (time-varying) estimation of flood flows and interaction with tidal ocean boundaries
- Easy interfacing with GIS and capability to present the flood behaviour in easy-to-understand visual outputs.

Existing flood models developed for sections of the Macleay Valley for Council and for Pacific Highway design and flood assessments are also built in TUFLOW, hence the current modelling maintains consistency with these studies. The model was developed and run in TUFLOW 2018-03-AA-IDP-w64, in the TUFLOW “Classic” computation scheme.

5.2 Configuration of Hydraulic Model

5.2.1 Extent and Structure

The TUFLOW model is comprised of:

- A parent 2D domain of the study area surface consisting of 20m x 20m cells covering the overall floodplain, and reflecting the catchment topography, with varying roughness as dictated by land use. Main watercourses are in general modelled in 2D.
- A series of finer nested grids with 5m x 5m cells covering village areas on the floodplain. Refer to Section 5.2.2 for discussion.
- A 1D network of selected floodplain drains and channels which are narrow (generally 20m or less width) compared to the 2D grid. While the drains would not significantly influence peak flooding, they may assist in drainage of the floodplain following the peak for improved assessment of duration of inundation.
- Other 1D elements including road cross drainage culverts, floodgates and flood control structures.
- Bridge structures, modelled in 1D or 2D depending on their size.

The model covers an area of 766km² of the floodplain and adjacent catchment areas downstream of Euroka, extending to Grassy Head in the north and past Crescent Head in the south. Interaction with the ocean occurs along the entire coastal extent in the model with ocean outlets at the Macleay River mouth, Korogoro Creek, Killick Creek and Ryans Cut. Additional outflow locations to the ocean may occur if floodwaters overtop the coastal dunes. An inflow/outflow boundary, depending on relative water levels, is defined on Connection Creek south of Maria River Road crossing to represent the hydraulic connection with the Maria River and Hastings River. Refer to the following report sections for further details on these features. The model domain is shown on Figure 5-1.



Legend

- TUFLOW Model Extent
- 5m Nested Grid
- Tailwater Boundary
- Inflow Boundary
- 1D Key Channels Draining Floodplain
- 1D Culverts
- 1D Bridges
- 2D Bridges
- 2D Breakline Roads Rail
- 2D Breakline Levee
- 2D Breakline Natural Embankment
- 2D Breaklines for Drains
- 2D Breakline Training Wall and other River Feature

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SHEET		1 of 1		GDA 1994 MGA Zone 56	
TITLE		TUFLOW Model Configuration			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
LC	IA176800	FIGURE 5-1	1	1	
CHECK	DATE				
AH	4/09/2019				

5.2.2 Model Resolution and Nested Grids

The TUFLOW model was developed with a 20m parent grid to assess flooding behaviour in the broader floodplain. Nine (9) separate 5m nested model grid were developed to define flooding conditions in finer detail in the floodplain villages including South West Rocks, Frederickton, Smithtown, Gladstone, Kinchela, Hat Head, Crescent Head, Stuarts Point and Fishermans Reach. The 5m grids were nested within the parent 20m grid of the overall floodplain, allowing flood depths, velocities and flood hazard to be analysed in greater detail in these built-up areas, such as active flow areas in the road corridors between neighbourhood blocks.

The extents of the finer nested grids are shown on zoomed-in design flood maps in E.1.

5.2.3 Model Topography and Bathymetry

The floodplain surface was defined based on LiDAR data described in Section 3.5.3, with town levees defined as breaklines based on survey in the levee audit reports described in Section 3.1.13. Additional breaklines were cut from the LiDAR for raised roads and natural river embankments. Breaklines defined in the previous Kempsey CBD TUFLOW model were utilised for available areas.

Bathymetry of the river and watercourses was based on the hydrographic survey described in Section 3.8 with supplementary survey of the river as described in Section 3.11.

Note that Rowes Cut, for the calibration and design event runs, was modelled in its un-maintained, dune-filled state, based on the LiDAR levels.

5.2.4 Minor Channels and Drains

Most river channels and watercourses were represented in 2D including the main river, Belmore River and Kinchela Creek. Selected minor channels and drains were represented as 1D features to assist in future assessment of floodplain drainage. The channel cross sections were typically cut from the LiDAR. The cross sections of Seven Oaks Drain and other drains around Collombatti Creek were modelled based on channel survey obtained from University of NSW Water Research Laboratory (WRL). WRL collected this data for development of a MIKE-FLOOD model of the Clybucca Wetlands for assessment of drainage management options.

5.2.5 Hydraulic Structures

Key hydraulic structures were represented in the TUFLOW model, including culverts, floodgates and flood control structures. Details of these structures were obtained from Council GIS layers, work-as-executed plans (Pacific Highway upgrade and Kempsey Bypass) and survey. The details of those structures which were modelled in the Kempsey CBD TUFLOW model were input in this current modelling. The modelled structures are indicated on Figure 5-1.

Local drainage through the levees is represented at Hat Head for the 0.2EY and 5% AEP events, when the levees are not overtopped by mainstream flooding and flooding inside the levees occurs due to local runoff. Modelled flood levels inside the levees are sensitive to the provision of these drainage structures in these events. These structures were omitted for rarer events and at other villages where significant overtopping of the levees by mainstream flooding dictates peak flood levels and is not sensitive to provision of these local drainage structures.

Figure 5-2 Belmore River Flood Control Structure



Figure 5-3 Kinchela Creek Left Bank Flood Control Structure



5.2.6 Bridges

The details of main bridge structures in the Kempsey CBD TUFLOW model were reviewed, including Kempsey traffic bridge and rail bridge and the Pacific Highway bridge and floodway viaduct. The traffic and rail bridges were observed to be set well above the PMF, while the Pacific Highway bridge was set up to 2m too low (compared to road surface elevations at the bridge approaches) in the previous modelling. Hence, the bridge deck details were updated based on raw LiDAR data, which detects the deck levels, and other available information. The bridge details adopted in the previous modelling would not have affected the flood hydraulics in events up to and including the 1% AEP event, but would have influenced the PMF.

Deck levels of other bridges in the study area were typically estimated from raw LiDAR, including Smithtown bridge and Jerseyville bridge, among others. Bridge deck and opening dimensions on the upgraded Pacific Highway north of the Macleay River were determined from RMS design data. Deck thickness and railing height were assumed based on typical highway bridge details.

Bridge pier loss coefficient values were nominally set at 0.2, which is considered slightly conservative. This parameter dictates the head loss as floodwaters flow under the bridge. This value was validated for Smithtown bridge in the TUFLOW bridge loss calculator, which uses procedures outlined in Hydraulics of Bridge Waterways (U.S. Federal Highway Administration, 1978), resulting in a coefficient value of 0.12.

Bridges were typically modelled as 2d structures, particularly on the main watercourses. Some narrow waterway bridges on the Pacific Highway were modelled as 1D structures.

Figure 5-4 Plummers Lane Bridge, Jerseyville



5.2.7 Surface Hydraulic Roughness

All parts of the study area within the TUFLOW model were assigned hydraulic roughness (Manning's n) values in a "materials layer" according to the river form, floodplain land use and ground cover. The values in particular for the river channel and pastoral land were refined in the model calibration. The adopted Manning's n values are based on widely-used parameter values from literature (Chow, 1959) and guidance from ARR 2019, and are mapped on Figure 5-5 and summarised in Table 5-1.

Table 5-1 TUFLOW Model Hydraulic Roughness Values

Land Use Type/Material	Manning's n	Comment
River channel, watercourse, ocean bed	0.021 – 0.035	Higher Manning's n values on river bends.
Floodplain	0.03 – 0.035	Typically pastoral land.
Swamp	0.06	
Light vegetation	0.06	
Medium density vegetation	0.08	
Heavy vegetation and mangroves	0.12	
Paved (Kempsey CBD and villages model nested grids)	0.02	Roads, car parks etc.
Village neighbourhood blocks and yards (5m nested grid)	0.06	Mainly lawns on properties and in yards, accounts for fences, landscaping and other fine-scale features. Buildings represented separately and overriding this land use type.
Urban commercial and industrial (5m nested grid)	0.04	Mainly paved, accounts for fences, landscaping, blocks of buildings and other fine-scale features.
Built areas (20m parent grid)	0.1	Urban blocks in 20m grid, retained from WMAwater (2015).
Grass areas in villages	0.03	Open space
Buildings	1.0	Includes large commercial and industrial buildings e.g. Nestle in the 20m model grid, and smaller individual buildings and dwellings in the 5m nested grids.

5.3 Model Boundaries

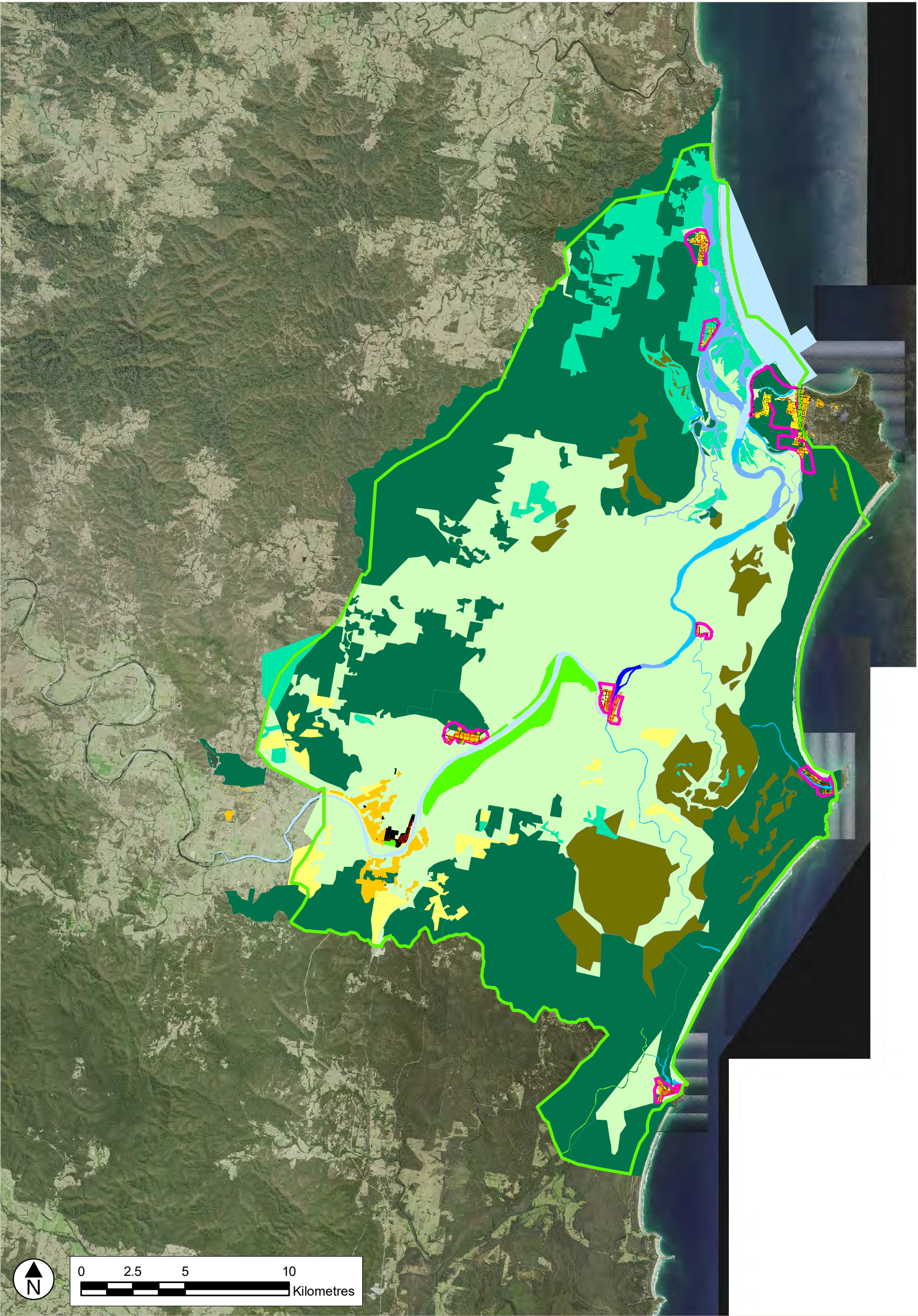
5.3.1 Inflow Hydrographs

The river flood flow hydrograph from the upstream catchment is input into the channel and floodplain at Euroka. Local inflow hydrographs from local sub-catchments are distributed within those sub-catchment boundaries into the TUFLOW model. Routed channel flows in Christmas Creek, Collombatti Creek and an unnamed flow path are input at the TUFLOW model boundary in those watercourses. The inflow boundaries are shown Figure 5-1.

Local catchment inflows are input inside the levee-protected areas at Frederickton and Hat Head to represent the ponding due to runoff generated inside the levee, particularly for the flood events where the levee is not overtopped (about 1% AEP for Frederickton and 5% AEP for Hat Head). The local inflows were not input at other villages such as Smithtown and Gladstone as they are affected by mainstream flooding in frequent events and the local inflows provide only a minor contribution to flooding.

5.3.2 Ocean Boundaries

The model was extended out into the ocean at the outlets at the Macleay River entrance and up to Grassy Head, Korogoro Creek and Killick Creek to Ryans Cut. A dynamic ocean water level boundary was applied along these boundaries. Refer to Figure 5-1 for model boundary locations.



GIS MAP file : IA176800_R03_Figs-5_Mannings_01

Legend

- TUFLOW Model Extent
- 5m nested grid
- Channel. n = 0.021
- Channel. n = 0.025
- Channel. n = 0.029
- Channel. n = 0.035
- Floodplain. n = 0.035
- Grass, short pasture. n = 0.03
- Dense vegetation and mangroves. n = 0.12
- Medium vegetation. n = 0.08
- Light vegetation. n = 0.06
- Swamp. n = 0.06
- Paved. n = 0.02
- Developed block (20m). n = 0.1
- Building n = 1.0
- Urban block (5m). n = 0.04 - 0.06

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SHEET		1 of 1		GDA 1994 MGA Zone 56	
TITLE		TUFLOW Land Use and Manning's n			
PROJECT		Lower Macleay Flood Study			
DRAWN	PROJECT #	MAP #	REV	VER	
LC	IA176800	FIGURE 5-5	1	1	
CHECK	DATE				
AH	5/07/2019				

5.4.1 Interface with Hastings River Catchment

A dynamic water level boundary was applied at the southern boundary interface with the Hastings River/Maria River on Connection Creek, to the south of Maria River Road. Water levels recorded at Green Valley on the Maria River (MHL Station 207406) were used at this boundary for the model calibration and verification to historic flood events, while time-varying design flood level data was obtained from the Hastings River Flood Study (Patterson Britton and Partners, 2006) for the design flood estimation. Refer to Figure 5-1 for this boundary location.

5.5 Consideration of River Entrance Dynamics

5.5.1 General

The Macleay River entrance at the time of the hydrographic survey collection (2003) had an entrance bar with elevation of up to a high point of -2.0 to -3.0m AHD. Sand bars and shoals had also formed just inside the river since the previous flood to high points of -1.0 to -2.5m AHD. This is typical geomorphic behaviour of many coastal river systems in NSW and other parts of Australia, whereby sediment moves around due to wind and tide and is deposited during non-flood periods. At the river entrance to the ocean, sediment transport is dominated by infilling of marine sediments due to tidal currents forming a flood tide delta (sand shoals just inside the entrance) and ebb tide delta (the entrance sand bar just seaward of the entrance) where tidal currents slow sufficiently for deposition (Roy, 1984). In the Macleay River the flood tide delta is limited due to the extensive training of the river channel and banks to a distance of 3km upstream of the entrance, although there is still shoal formation in this reach. Between the flood and ebb tide delta there are typically deep holes formed due to swift local tidal currents at the entrance. At the Macleay River entrance these holes are up to 12m deep.

During flood events large volumes of sediment are scoured and transported seaward with the shoals associated with the flood tide delta eroded by flood flows. The ebb tide may also shift seaward as a result of the flood flows.

5.5.2 Initial Modelling of Bed Scouring

Initial TUFLOW model runs during the model calibration deduced that the flood tide shoals are likely to be scoured significantly during a flood event as a result of high flood flow velocities. Based on iterative model runs, these sand bars were estimated to be scoured by 2 to 3.5m to bed levels of -4.0 to -6.0m AHD, noting that there are deep channel sections adjacent to these sand bars to bed levels of -8.0 to -12.0m AHD, which are assumed to remain. The assumed scoured bed levels were selected based on calibration to the South West Rocks water level gauge, located 2.6km upstream of the entrance.

Modelling of a scoured entrance condition is a typical practice in flood studies of coastal rivers, particularly those which form relatively shallow entrances. Examples include the *Review and Update of the Manning River Flood Study* (BMT WBM, 2016), *Ballina Flood Study Update* (BMT WBM, 2008) and *Nambucca Heads Flood Study* (SKM, 2011). In the *Ballina Flood Study Update* hydraulic modelling, the Richmond River bed at the entrance was lowered by 2m for calibration and design flood estimation.

The available hydrographic survey data indicates channel bed levels at Korogoro Creek and Killick Creek entrances of -0.7 to -0.9m AHD. The LiDAR data shows a beach sand berm at Ryans Cut entrance to an elevation of 2.5m AHD. These entrances were assumed to scour to a bed level of -1m AHD, which is the surveyed bed elevation of the creeks heading upstream from the outlets, to the full width of the watercourse at the entrance.

All scoured features were assumed to be static (i.e. down to their scoured levels) from the start and throughout the model simulation.

Note that sediment transport in the Macleay River is highly dynamic and different flood events are likely to have different impacts on sediment scouring and deposition and on river morphology as a whole. This is well demonstrated by the historic major flood in 1893, whereby the current river entrance was formed from scouring of the coastal dune system, 10km south of its former entrance site at Grassy Head.

Following the initial model setup and calibration, additional detailed geomorphic modelling was undertaken separately during the design event modelling to validate the assumptions on the degree of scour during design flood events and the calibration events. This is discussed in further detail in Section 7.1.7.

6. Model Calibration and Verification

6.1 Overview

The TUFLOW model was calibrated and verified to two recent historic flood events:

- March 2001 flood event: 1 in 13, or 8% AEP event
- February 2013 flood event: 1 in 7, or 14% AEP event.

These events were selected by Council for the model calibration process. Other flood events have occurred in recent times including in 2009 (1 in 9 AEP) and 2011 (1 in 5 AEP). However, the 2001 event represents the largest event in the last 20 years while the 2013 event is one of the most recent and significant events, on par with the 2009 and 2011 events. The selected events have the benefit of a relatively large compilation of flooding observations including flood aerial photography being readily available from Council. While there is calibration data available for the 2009 and 2011 events, there is probably limited value in calibrating the model to additional events of similar magnitude.

There have been a number of significant flood events prior to 2001 including the 1949 (1 in 90 AEP), 1950 (1 in 80 AEP) and 1963 (1 in 17 AEP) events which were not considered for calibration in this study due to uncertainties in input data, including rainfall, river bed form and floodplain features. There have been significant changes in the floodplain since these events, including construction of numerous levees and hydraulic structures associated with the Macleay Flood Mitigation Scheme, built between the 1950s and 1970s. Additionally the quantity and quality of flood observations from these early events is relatively low, compared to a higher number of flooding observations across the floodplain in addition to availability of continuous water level data at several locations throughout the study area for the selected calibration events.

The model calibration process generally consisted of adjusting the hydraulic roughness parameter value (Manning's n) in the river channel and in the floodplain within acceptable limits to capture the effects of the channel form (e.g. channel bed material, irregularity and sinuosity) and land use and land cover on the flow behaviour. The bed form of the river was observed from historic aerial photos to be mobile, and was reviewed and adjusted where necessary to obtain a good calibration fit. This was particularly the case near the river entrance where the entrance bar and shoals could be assumed to be scoured out during a flood event. Other parameter values were also reviewed prior to selection of the adopted conditions. These are discussed separately for the 2001 and 2013 flood events in Sections 6.4.1 and 6.5.1, respectively.

6.2 Calibration Data

The data used to calibrate and verify the TUFLOW model is summarised and appraised in Table 6-1. The source of the data is also stated. Data which was reviewed but subsequently excluded from the calibration is also described.

It is noted that while there is a significant amount of calibration data in the study area, these are generally concentrated on the main river. There is limited calibration data on floodplain areas. Future data collection programs should consider capture of future flood event data in these areas to enable ongoing refinement of the Lower Macleay flood modelling and calibration.

Table 6-1 Summary and appraisal of available calibration data

Source ID	Description	Location and Availability	Source	Comment
1	Continuous water level data	- Kempsey Traffic Bridge - Smithtown Bridge - South West Rocks Data available for both 2001 and 2013 events	MHL	Considered to be high quality and reliable data. Useful for calibrating the timing of flood peaks
2	Continuous water level data	- Jerseyville - Plummers Lane - Seven Oaks Bend - Collombatti Data available for the 2013 event	BOM/MHL	Data collected for Kempsey Flood Watch program. Moderate reliability. Some uncertainty about whether gauge has been connected to AHD (BOM and MHL could not confirm though Council believes so).
3	Peak water levels	Numerous. Range of flood events	BOM via Council Refer to Table 3-2.	Considered to be reliable data. Note the discrepancy between stated flood peak from this source (7.11m AHD) and source 1 (6.92m AHD) for Feb 2013 event.
4	Peak water levels	Numerous. Range of flood events	Council Refer to Table A-1 in Appendix A	Mixed reliability. Some events and locations had two water levels reported but no notation on which value was to AHD.

5	Peak water levels (spot levels)	Kempsey CBD to Old Pola Creek (2001 event) Kempsey CBD to Bellimbopinni and Austral Eden (2013 event)	Mapping and tables in WMAwater (2015)	Mixed reliability as denoted in the previous flood study reports
6	Peak water levels	Kempsey to Jerseyville 2001 event	Table 2 in Webb McKeown and Associates (2009)	Mixed reliability
7	Peak water levels	• Kempsey • Seven Oaks • Belmore River • Upper Kinchela Creek • Kinchela Creek Entrance • Killick Creek • Jerseyville • Clybucca Creek 2001 event	“Report on Macleay Flood of March, 2001. River Levels, Rainfall and Recovery”. Kempsey Shire Council, year unknown. Also in Appendix B of Webb McKeown and Associates, 2004b.	Similar data and reported flood levels to Item 4. Mixed reliability. Some levels not connected to AHD. Some data suspect with flood levels for upstream locations significantly lower than downstream locations (compared to this source and other recorded data e.g. MHL recorded water levels). Exact locations of some reported locations unknown.
8	Observed flood depths above ground and above floor	Smithtown 2001 event	Figure 6 in Webb McKeown and Associates, 2004b. Based on resident questionnaire	Above floor flooding depths were paired with surveyed floor levels supplied by Council but the inferred flood levels were 0.2 – 2.7m higher than the recorded flood levels at Smithtown Bridge and hence are considered unreliable. Data excluded from calibration. Above ground depths considered indicative due to uncertainty about exact location of observation on each property.

9	Flood aerial photos 2001 event	Mostly Kempsey CBD. Limited number in Lower Macleay	Webb McKeown and Associates, 2004b.	Limited number of photos in area of interest. Unable to determine exact location of some photos.
10	Flood aerial photos 2013 event	Numerous throughout Lower Macleay	Council	Good quality photos, most locations could be determined. Appears to be mixture of photos at/ near flood peak and after peak.

6.3 Tidal Calibration

Preliminary runs of the TUFLOW model were undertaken for a non-flood period during the model development phase to calibrate the tidal phasing along the river, prior to calibration to the historic flood events. Calibration points included the water level gauges at Kempsey Traffic Bridge, Smithtown and South West Rocks. The tidal calibration was considered satisfactory, with a good fit to the tidal amplitude and phase shown for the pre-flood peak periods on the flood level hydrographs for the 2001 and 2013 events, refer to Figures C-6 to C-8 in Appendix C for the 2001 event and Figures D-5 to D-7 in Appendix D for the 2013 event.

6.4 March 2001 Flood Event

6.4.1 Adopted parameter values and assumptions

6.4.1.1 River inflows

The gauged river flows from the Turners Flat water level gauge have been adopted for the model calibration. These flows have been compared to the flow data previously adopted in the Kempsey CBD modelling (WMAwater, 2015), refer to Section 4.2.1. The flows were lagged by 2.5 hours. This provided the best fit to the rising limb at the Kempsey Traffic Bridge gauge.

6.4.1.2 Rainfall spatial and temporal distribution

Rainfall and runoff on the local catchments draining directly to the Lower Macleay floodplain are input into the TUFLOW model. While the main driver of flooding in the study area is the river inflows from the catchment upstream of the study area and which are estimated from river gauge data, the local catchment inflows do determine the available floodplain storage and hence the potential peak flood flows in the river when there are overbank flood events. The local catchment inflows also likely to influence the peak flood levels in areas more removed from the main river, including the Collombatti Creek floodplain and secondary floodplain outlets at Hat Head and Crescent Head.

The spatial distribution of rainfall on the Lower Macleay floodplain is illustrated on Figure C-1 in Appendix C. The rainfall isohyets (contours of total rainfall depth) for the period 6 – 12 March 2001 were replicated from Figure B1 in Appendix B of Webb McKeown and Associates (2009). A sharp gradient in the rainfall depths is observed immediately west of Kempsey, with recorded rainfall depths in the vicinity of Kempsey ranging from 270 – 290mm. The sharp gradient extends to the catchment boundary to the north of Kempsey, with a rainfall depth of 392mm recorded at Eungai (Searles Road) just outside the catchment boundary. A comparison with gridded rainfall data downloaded from the BOM website for the event indicates that the data sets are comparable, although the BOM data tends to be more generalised and the isohyets (Webb McKeown and Associates, 2009) are in reasonably similar to the observed spot depths at rainfall stations. Note that flooding along the main river is likely to be insensitive to the exact isohyetal rainfall distribution on the local catchments, particularly for moderate flood events such as the 2001 event where flooding is highly influenced by the main river flows and overtopping of the riverbanks and levees.

The floodplain downstream of Kempsey experiences a lower gradient in the rainfall depths. Coastal rain gauges record rainfall depths of 184mm (Smoky Cape) and 191mm (Crescent Head), increasing to 236mm in the western part of the floodplain (Collombatti).

The modelled rainfall spatial and temporal variation is represented in Figure C-1 and C-2, respectively, in Appendix C. Sub-catchments are assigned total rainfall depths based on recorded depths at Kempsey, South West Rocks (Smoky Cape) and Eungai (Searles Road) according to the isohyetal map contours. The daily-read rainfall depths at Smoky Cape and Eungai are distributed based on 15 minute rainfall recorded Kempsey Airport automatic weather station. The Smoky Cape data was scaled up to a depth of 200mm to represent the average depth across the coastal floodplain.

6.4.1.3 Rainfall losses

Rainfall losses of 70mm initial loss and 0.7mm/hr continuing loss were selected based on the adopted parameter values in the catchment hydrologic modelling described in Webb McKeown and Associates (1989) and Webb McKeown and Associates (2009). No flow data is available for local catchments in the Lower Macleay to permit hydrologic calibration, hence these values were not adjusted.

6.4.1.4 Tailwater conditions

Ocean water levels recorded at the MHL Coffs Harbour tide gauge were adopted as the tailwater boundary in the vicinity of the Macleay River entrance (and north to Grassy Head), Korogoro Creek, Killick Creek and Ryans Cut entrances.

Recorded water levels at Green Valley on the Maria River were adopted as the water level boundary on Connection Creek.

Refer to the tailwater level hydrographs on Figure C-2 in Appendix C.

6.4.1.5 Initial water levels

Initial water levels in the waterways directly connected to the ocean, Macleay River and Hastings/Maria Rivers were selected based on ocean water levels and Maria River levels at the start of the model simulation. The model was run for a warm-up period of several days prior to the main flood event for the river water levels to adjust to the tidal ocean boundary.

An initial water level on the floodplain of 0.1m AHD was selected based on initial model runs. This initial water level translates to the floodplain being essentially empty, apart from any standing water in swamp areas which is detected by the LiDAR data. Note that there is no available data to verify the starting water levels on the floodplain prior to the start of the March 2001 flood event.

6.4.1.6 Pacific Highway Upgrade and other Roads

The embankment formations of the upgraded Pacific Highway are represented in the LiDAR terrain data. The model terrain was adjusted to remove the highway embankment for the March 2001 event simulation, as it was constructed in 2011 – 2013.

Discussions with landowners on the floodplain during the collection of topographic data indicated that landowners have made some modifications to road embankments in recent times. However, since no firm data is available on the timing and nature of these modifications the model terrain was not adjusted to account for these works.

6.4.1.7 Operation of Belmore and Kinchela Floodways

The flood control structures at Belmore River, Kinchela Creek Right Bank and Kinchela Creek left Bank are normally left shut and are manually opened by Council staff during minor flood events in the Macleay River. No information is available on the timing of opening of the flood control structures during the 2001 flood event. For the purposes of the calibration, the flood control structures are assumed to be fully open over the course of the flood event. Sensitivity testing indicated minimal impact to peak flood levels in the Macleay River with the flood control structures closed during the 2001 event simulation, noting that the flood mitigation scheme including the floodways was designed to service the 1 in 2.5 year event, while the 2001 event was a 1 in 13 AEP event.

6.4.1.8 Scouring of river entrance

Sand bars and shoals at the Macleay River entrance are assumed to be scoured by 2 to 3.5m to bed levels of -4.0 to -6.0m AHD as informed by iterative model calibration runs and as discussed in Section 5.5.2.

6.4.2 Comparison to Observed Flooding Conditions

Comparison of modelled peak flood levels to the observed calibration data is presented in Table 6-2 for locations within the Lower Macleay floodplain. Mapping of the peak flood depths and comparison of peak spot flood levels is presented in Appendix C. Flood level hydrographs for the observed and modelled flood levels at Kempsey, Smithtown and South West Rocks in addition to flood surface long section profiles are also presented in Appendix C. Water level recorder data was not available at Korogoro Creek or Killick Creek for the model calibration or verification. Refer to Table 6-1 for the sources of available calibration data.

Table 6-2 Comparison of observed and modelled peak flood levels and depths – March 2001 event

Location	Observed (m AHD)	Modelled (m AHD)	Difference (m)	Comment
Water Level Gauge Data				
Kempsey Traffic Bridge (Source 3)	7.44	7.44	0.00	Good calibration. Manual read peak level quoted.
Smithtown (Source 1/3)	4.34	4.35	+0.01	Good calibration. Minor (20mm) difference in reported peak between source 1 and 3.
South West Rocks (Source 1)	1.35	1.33	-0.02	Good calibration.
Peak Water Levels (Spot Levels) from Source 5				
Comments on reliability are quoted from Source 5 (WMAwater (2015))				
Locations within Lower Macleay floodplain only shown.				
J. Farley's Dairy Farm	6.26	6.63	0.37	Reliable. Poor calibration though similar to KCBD model.
Collin Paix property	6.59	6.75	0.16	Reliable. Fair calibration.
Other Peak Water Level Readings				
Belmore River Entrance (Source 4)	4.5	4.53	0.03	Good calibration
Kinchela Creek Entrance (Source 4)	3.44	3.46	0.02	Good calibration
Upper Kinchela	2.94	3.13	0.19	Unsure of exact location. Fair calibration if taken immediately upstream of Kinchela flood gate.
Seven Oaks (Source 4)	4.8	5.06	0.26	Poor calibration
Park Street, Smithtown (Nestle) (Source 6)	4.66	4.59	-0.07	Good calibration
Plummers Lane (Source 4)	2.75	2.71	-0.04	Good calibration
Jerseyville (Source 4)	2.10	2.24	0.14	Fair calibration
Rainbow Reach (Source 4)	2.00	2.02	0.02	Good calibration
Clybucca Creek (Source 7)	3.10	2.88	-0.22	Unsure of exact location. Fair calibration if taken immediately upstream of Clybucca flood gate.
Killick Creek (Source 7)	2.66 (?)	1.55	1.11	Unreliable. Unsure of gauge datum. Unsure of exact location.

* A similar rating of the calibration to that in WMAwater (2015) has been adopted. Good = $\leq \pm 0.1$ m compared to observed. Fair = $\pm 0.1 - 0.2$ m. Poor = $> \pm 0.2$ m.

Table 6-3 Comparison of observed and modelled above ground flood depths, Smithtown – March 2001 event (ref: Source 8)

Address	Observed (m)	Range on Lot – Modelled (m)	Comment
88 Belmore Street	1	0.4 - 0.9	Consistent with upper range
84 Belmore Street	1	0.4 – 1.1	Consistent with upper range
1 Belmore Street	0.5	0.7 - 1.4	Below lower range
37 Belmore Street	0.9	0.6 - 0.1	Consistent with observed
49 Belmore Street	0.6	0.4 - 0.8	Consistent with observed
53 Belmore Street	0.6	0.5 - 0.8	Consistent with observed
73 Belmore Street	1	1 - 1.3	Consistent with observed
75 Belmore Street	1.5	1 - 1.2	Consistent with observed
77 Belmore Street	0.8	1 - 1.4	Below lower range of modelled
2 – 4 Jeffery Street	0.4	0.3 - 0.6	Consistent with observed
7 Jeffery Street	1	0.9 - 1.2	Consistent with observed
14 Morton Avenue	0.5	0.2 - 0.5	Consistent with observed
7 Morton Avenue	1	0.4 - 0.5	Significantly above upper range
13 Morton Avenue	0.5	0.4 - 0.6	Consistent with observed
6 Park Street	0.1	0.2 - 0.4	Below lower range of modelled
70 Rawson Street	1	0.8 - 1.2	Consistent with observed
66 Rawson Street	0.8	0.8 - 0.9	Consistent with observed
64 Rawson Street	1.1	0.8 - 0.1	Slightly above upper range
54 Rawson Street	1	0.8 – 1.2	Consistent with observed
44 Rawson Street	1	0.7 – 1.1	Consistent with observed
38 Rawson Street	1.5	0.8 - 1.2	Significantly above upper range
4 Russell Avenue	0.45	0.3 - 0.6	Consistent with observed
7 Russell Avenue	0.2	0.3 – 1.3	Below lower range of modelled
9 Russell Avenue	1.5	0 – 1.3	Above upper range
2 Verge Street	0.3	0.3 - 0.5	Consistent with observed
10 Vincent Avenue	0.5	0.4 – 1.1	Consistent with observed
15 Main Street	0.5	0.7 - 0.8	Below lower range of modelled
11 Main Street	0.5	0.5 - 0.7	Consistent with observed
9 Main Street	0.5	0.5 - 0.7	Consistent with observed
Smithtown Tennis Courts	0.5	0 – 2.1	Uncertainty about exact location

6.4.3 Discussion on Model Calibration to March 2001 Flood Event

The flood stage hydrographs in Appendix C indicate that the model replicates the hydrograph shape, timing of the flood rising limb and peak at Kempsey Traffic Bridge. The model matches the peak flood level recorded at the traffic bridge gauge, as shown on Table 6-2, hence the model is satisfactorily calibrated in this area.

The modelled peak flood level at Smithtown gauge match the recorded data well, with a variance of +0.01m. There is generally a good match of the hydrograph shape, including timing of the rising limb and rate of rise. The peak of the hydrograph at Smithtown is slightly lower and flattened, which results in some difference in the exact timing of the peak and the duration at and near the peak, due to the complex engagement of overbank areas and floodplain. The modelled peak flood level occurs approximately 5 hours after the recorded peak, however, it is noted that both the modelled and recorded flood levels reach near-peak at approximately the same time. The model produces a similar rate of rise and reaches the minor flood level of 3.5m AHD approximately 2 hours earlier, and the moderate flood level of 4m AHD approximately 4 hours earlier. The model calibration is considered satisfactory at Smithtown.

Flooding within Smithtown village is dictated by the degree of overflows over the levees. Preliminary model runs were had assigned the adopted Manning's n value for an urban block ($n = 0.1$) which resulted in underestimation of overflows over the levee. A reduced value of $n = 0.03$, representing short grass and lawns, was adopted in the calibration for the levee crest and this improved the flooding estimates within the village. Although there is some uncertainty about the location of the observed flood depths on properties in the village, it is noted that the modelled flood depths are generally consistent with the reported depths.

In the lower reaches of the river between Smithtown and South West Rocks, the model produces similar flood levels to the reported flood peaks at Plummers Lane, Jerseyville and Rainbow Reach. The model also provides a good match to reported flood levels at Belmore River Entrance and Kinchela Creek Entrance, although there is some uncertainty about the reliability of the reported flood levels at these locations given that two values are reported at each location.

The flood level hydrograph at South West Rocks has some variation in the shape between the recorded and modelled data. The model produces a good match to the peak water level and timing of the peak and the overall time-varying tidal water levels. There is some variation in the high tide water levels before and after the flood peak of 0.1 – 0.2m, which is likely to be due to the highly dynamic changes in actual river bed levels over the course of the flood event, compared to the assumed static, scoured bed in the model. Overall, this variance is considered minor and the model calibration is satisfactory at the river entrance.

Flooding on the floodplain storage areas including the southern (Belmore/Kinchela) floodplain is driven mainly by floodwaters overtopping the Macleay River right bank particularly around the Old Pola Creek and Austral Eden areas, in addition to interflows from the Maria River via Connection Creek. Flooding observations available for the March 2001 event are limited to one reported peak water level at Upper Kinchela. The modelled flood level provides a reasonably good fit to the observed water level at this location.

Flooding in the northern (Collombatti/Clybucca) floodplain is influenced by inflows from Collombatti Creek catchment and overflows from the Macleay River around the Bellimbopinni and Seven Oaks areas. Again, flooding observations are limited to one reported flood level at Clybucca Creek. The modelled flood level also provides a reasonably good fit to the observed water level at this location.

Given an absence of water level gauging in the floodplain storage areas there is uncertainty in the initial floodplain water levels. Floodplain water levels cannot be inferred from the streamflow gauging data as the main river and floodplains are essentially unconnected hydraulically until river flooding overtops the riverbanks. Hydrologic and hydrodynamic modelling of rainfall and runoff in the preceding weeks prior to the main flood was considered but deemed impractical due to exceedingly long run times and complexity of hydrologic processes (infiltration and evapotranspiration) which would affect long term floodplain water storage.

However, since the crest levels of the levees and natural banks along the Macleay River (generally 3 – 4m AHD) are typically higher than the low points of the floodplain storage areas (generally less than 1m AHD), peak flood levels along the river are likely to be insensitive to the initial water levels under normal conditions.

Floodplain storages would probably need to be relatively full, such as following a minor flood event, for there to be an impact on peak flooding in the river. Clearly, peak flood levels in an around the floodplain areas would be sensitive to the water levels at the start of a flood event. This sensitivity will be tested following the design flood modelling.

In general, a good calibration was achieved for the March 2001 flood event for the Lower Macleay floodplain downstream of Kempsey, with variance between modelled and observed flood levels at the calibration points generally being less than +/- 0.2m. There is uncertainty about the starting water levels on the floodplain prior to the main flood, however, the modelling results are consistent with the observed flood levels at Upper Kinchela and Clybucca Creek.

6.5 February 2013 Flood Event

6.5.1 Adopted Parameter Values and Assumptions

6.5.1.1 River inflows

The flood inflow hydrograph used in the Kempsey CBD TUFLOW model verification (WMAwater, 2015) for the February 2013 event were also adopted for the verification process in this study. These flows were derived from WBNM hydrologic modelling of the event and provided the best fit to the rising limb at the Kempsey Traffic Bridge gauge. A flow hydrograph derived from the Turners Flat gauged flows was tested in the TUFLOW model, however, appeared to produce insufficient flood volume when compared to the recorded stage hydrographs at Kempsey Traffic Bridge and Smithtown. This may be attributed to the additional inflows from the Dungay Creek sub-catchment, joining the Macleay River at Aldavilla, downstream of Turners Flat.

6.5.1.2 Rainfall spatial and temporal distribution

Previous flood studies did not prepare a detailed rainfall isohyet map for the February 2013 event. Hence, a rainfall isohyet map was prepared based on gridded rainfall data downloaded from the BOM website (<http://www.bom.gov.au/jsp/awap/rain/archive.jsp?colour=colour&map=totals&period=daily&area=nat>) for the period 20 – 24 February 2013, with typical rainfall depths of 200 – 250mm on the study area. Isohyets derived from the cumulated grid were adjusted to incorporate the higher total rainfall depths exceeding 300mm recorded in a band from Crescent Head, stretching north-west through Kempsey. The isohyet map is shown on Figure D-1 in Appendix D, along with the assigned rainfall station temporal patterns for the February 2013 event.

6.5.1.3 Rainfall losses

Rainfall losses for the March 2013 event model verification were nominally selected at 15mm initial loss and 0.7mm/hr continuing loss, accounting for preceding rainfall of 50 to 200mm in the week prior to the main storm on 22 February 2013.

6.5.1.4 Initial water levels

Initial water levels in the main waterways were selected based on ocean water levels and Maria River levels at the start of the model simulation, similar to the March 2001 model calibration runs.

An initial water level on the floodplain of 0.1m AHD was assumed, similar to the calibration runs, given that there is no available data to verify the starting water levels on the floodplain prior to the start of the 2013 flood event.

6.5.1.5 Tailwater conditions

Refer to the plots of the tailwater levels on Figure D-2 in Appendix D for the time-varying water levels applied at the ocean and Maria River model boundaries.

6.5.1.6 Pacific Highway Upgrade

The Pacific Highway Upgrade, including road embankment, main river and floodplain bridge and other hydraulic structures, were represented for the model verification. The upgraded highway was present for the February 2013 event, noting that the road was opened to traffic later that year.

6.5.1.7 Operation of Flood Control Structures

The Belmore and Kinchela flood control structures were represented as opening and closing hydraulic structures based on the timing of opening indicated in Council records. Aerial photos provided by Council confirmed all three structures were opened during the 2013 flood event. The initial closure then opening of the flood control structures was implemented in the TUFLOW model using variable z-shape objects at the structure outlets.

6.5.1.8 Yarrahapinni Floodgates

The floodgates and barrage at Yarrahapinni were removed recently as a part of an environmental program to restore the health of the Yarrahapinni Wetlands. The Yarrahapinni Wetlands Plan of Management (OEH, 2013) and Google Earth aerial photos indicated that the floodgates and barrage were in place in April 2013, hence these were represented in the TUFLOW model for the February 2013 flood event.

6.5.1.9 Scouring of river entrance

Sand bars and shoals at the river and creek entrances were assumed to be scoured out to -4 to -6m AHD as per the 2001 event model calibration.

6.5.2 Comparison to Observed Flooding Conditions

Comparison of modelled peak flood levels the observed calibration data is presented in Table 6-4. Mapping of the peak flood depths and comparison of peak spot flood levels is presented in Appendix D. Flood stage hydrographs for the observed and modelled flood levels at Kempsey, Smithtown and South West Rocks in addition to flood surface long section profiles are also presented in Appendix D. Refer to Table 6-1 for the sources of available calibration data.

Table 6-4 Comparison of observed and modelled peak flood levels and depths – February 2013 event

Location	Observed (m AHD)	Modelled (m AHD)	Difference (m)	Comment
Water Level Gauge Data				
Kempsey Traffic Bridge (Source 1)	6.94	6.96	0.02	Reliable. Good calibration. Note, Source 4 quotes 7.14m AHD. Kempsey CBD TUFLOW model (WMAwater, 2015) calibrated to this level.
Smithtown (Source 1)	4.18	4.27	0.09	Reliable. Good calibration.
South West Rocks (Source 1)	1.18	1.10	-0.08	Reliable. Good calibration.
Borirgala Creek (Source 1)	1.25	1.48	0.23	Reliable. Poor calibration. Attributed to difference between recorded and modelled initial water levels in Yarrahapinni Wetland.
Peak Water Levels (Spot Levels) from Source 5				
Comments on reliability are quoted from Source 5 (WMAwater (2015))				
Locations within Lower Macleay floodplain only shown.				

Location	Observed (m AHD)	Modelled (m AHD)	Difference (m)	Comment
Macleay Valley Way, north Kempsey floodplain	6.32	6.38	0.06	Reliable 9/10. Good calibration.
Frederickton levee / Flood gauge	5.83	5.93	0.1	Reading taken 24/02/2013 at 10:23 - Approximate level on frame of the flood gauge. Good calibration.
Rowe Property upstream	5.43	5.43	0.00	Good calibration.
Rowe Property, downstream Ferry Lane	5.4	5.40	0.00	Good calibration.
Jeffery Property 1 - shed	4.78	4.93	0.15	Unreliable. Fair calibration.
Jeffery Property 2	5.39	5.25	-0.14	Fair calibration.
Jeffery Property 3	5.6	5.42	-0.18	Fair calibration.
Old Pacific Highway, Bellimbopinni	4.84	5.07	0.23	Reliable 9/10. Poor calibration.
Old Pacific Highway, Bellimbopinni	5.09	5.03	-0.06	Reliable 8/10 Good calibration.
Old Pacific Highway, Bellimbopinni	4.95	5.18	0.23	Reliable 9/10. Poor calibration.
Outer Road, Austral Eden	5.45	5.39	-0.06	Good calibration.
Hutcheson Street, Austral Eden	3.89	4.21	0.32	Reliable 8/10. Poor calibration.
Inner Road, Austral Eden	4.11	4.30	0.19	No note on reliability but seems reliable. Fair calibration.
Old Station Road	3.34	3.36	0.02	Good calibration.
Inner Road, Austral Eden	4.23	4.41	0.18	Fair calibration.
Inner Road, Austral Eden	4.35	4.39	0.04	Good calibration.
Rawson Street, Smithtown	3.73	4.09	0.36	Poor calibration.
Other Peak Water Level Readings				
Kempsey Traffic Bridge Telemetry (Source 4)	7.14	6.96	0.18	Unsure of reason for discrepancy with continuous water level recording data (Source 1). Fair calibration.
Belmore River Entrance (Source 4)	4.51	4.45	0.06	Unsure of reliability. Good calibration.
Kinchela Creek Entrance (Source 4)	1.62	3.43	1.81	Considered unreliable. Unsure of gauge datum. Inconsistent with upstream and downstream data
Seven Oaks (Source 2)	4.79	4.99	0.2	Fair calibration.
Plummers Lane (Source 2)	2.41	2.46	0.05	Good calibration.
Jerseyville (Source 2)	2.27	1.88	-0.39	Poor calibration.
Rainbow Reach (Source 4)	0.65	1.56	0.91	Unreliable. Source 2 indicates gauge failed or data error.

Location	Observed (m AHD)	Modelled (m AHD)	Difference (m)	Comment
Collombatti Creek (Source 2)	3.54	2.95	-0.59	Poor calibration. Possibly affected by backwater flooding from temporary works at Pacific Highway Upgrade site, approx. 2km downstream of gauge site.

* A similar rating of the calibration to that in WMAwater (2015) has been adopted. Good = ± 0.1 m compared to observed. Fair = $\pm 0.1 - 0.2$ m. Poor = $> \pm 0.2$ m.

6.5.3 Discussion on Model Verification to February 2013 Flood Event

The model verification results are generally consistent with the observed flood levels throughout the Lower Macleay, with the variance between observed and modelled flood levels being typically less than ± 0.2 m at the reliable flood marks.

The modelled flood level hydrographs at Kempsey and Smithtown show a markedly higher second flood peak on 26 March 2013 (approximately 0.5m higher at Smithtown) and an elongated main flood peak, particularly at Smithtown. This is likely to be due to the WBNM hydrologic modelling, used to derive the river inflows upstream of Kempsey for the verification event, adopting high recorded rainfall depths for this latter period of the storm uniformly across the Dungay Creek sub-catchment, which joins the Macleay River between Turners Flat and Kempsey. The BOM rainfall records, indicating actual rainfall patterns, suggest sharp rainfall gradients in this area with depths recorded at Wittitrit (upper Dungay Creek sub-catchment) at 9 AM on the 25th and 26th March 2013 of 40mm and 83mm respectively, compared to nearby rainfall stations including 7.8mm and 2.4mm at Kempsey Airport and 4mm and 61mm at Majors Creek. This difference between assumed and actual rainfall distribution for the late stage of the storm does not affect the quality of the model verification for peak flood levels and overall flood hydrograph shape.

Similar to the model calibration, the observed flood levels are generally on or close to the main river and there is limited verification data for floodplain areas. There is water level gauge data for Collombatti Creek upstream of the upgraded Pacific Highway, however, a satisfactory model verification was not achieved there. This may be due to the impact of temporary works during the construction of the highway upgrade, including access road embankments crossing Collombatti Creek, Seven Oaks Drain and other watercourses. The construction access road crossing the watercourses on the upstream side of the highway is visible on the aerial photography dated 2014, but was removed by the time of the 2016 LiDAR data capture. Blockage of the watercourses by the raised access road would result in elevated upstream water levels and backwater flooding, and is attributed to the recorded flood levels being 0.6m higher than the modelled flood levels at this location.

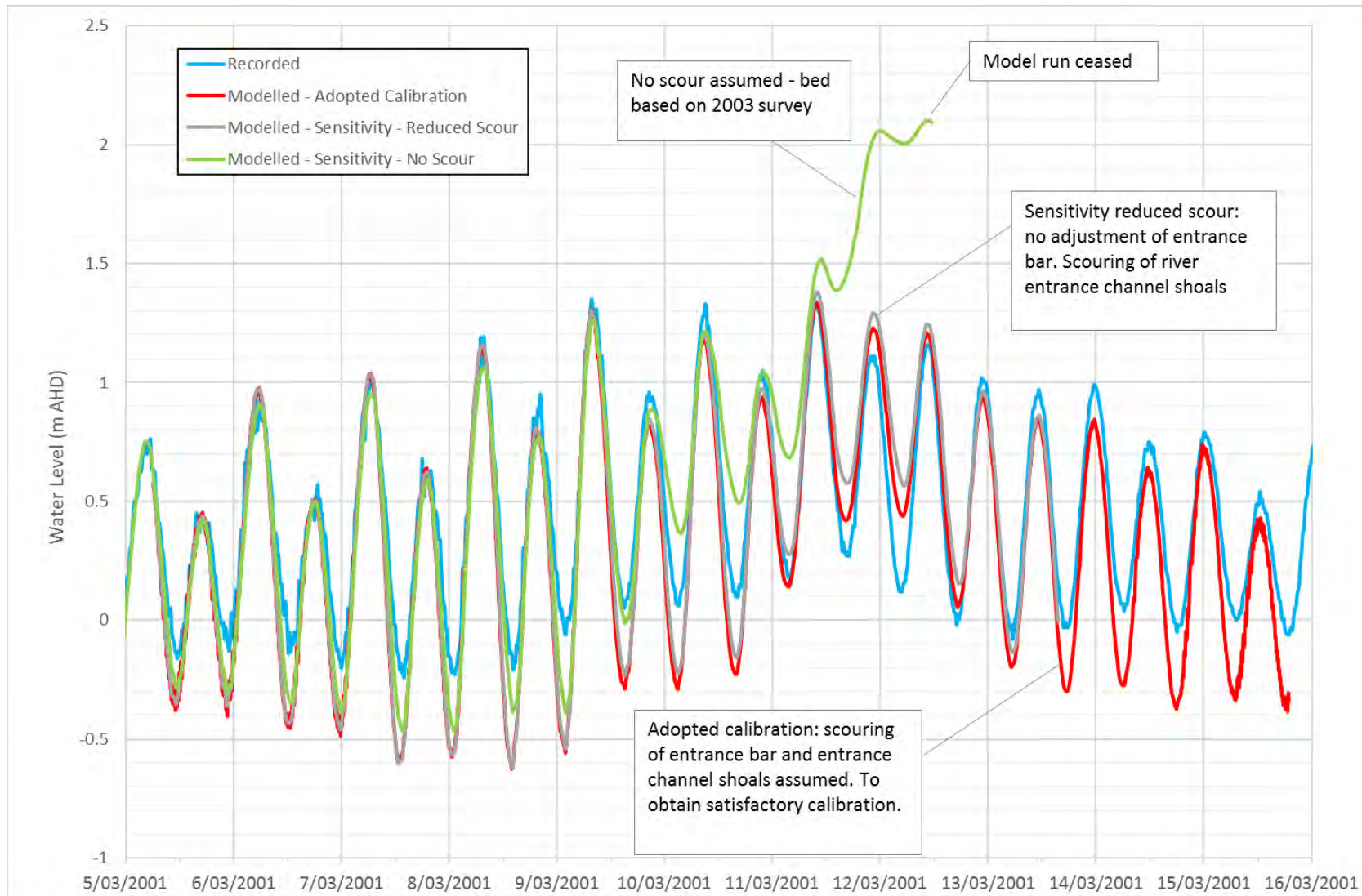
The model results and modelled flood behaviour are generally consistent with the aerial flood photos provided by Council. Photos of flood flow through Ryans Cut were used to inform the assumed modelled scouring of that entrance. The photos indicate low flow velocities in the upper Korogoro Creek, with the model simulating flow velocities of 0.6 – 1m/s which is consistent. High discharge and flow velocities are modelled at Killick Creek entrance, which is consistent with the aerial photos at that location. The modelling indicates approximately 0.5m higher flood levels and extents at Clybucca flood gates compared to the photos.

6.6 Sensitivity of Flood Behaviour to Scour Condition

The sensitivity of flooding to entrance shoaling and scouring conditions is shown on for the March 2001 calibration. The adopted calibration run includes the degree of scouring as discussed in Section 5.5.2. There is a large discrepancy from recorded water levels at South West Rocks gauge when no scouring is represented.

A run was also undertaken with the same assumed scour as the calibration in the entrance channel but with no scouring of the entrance bar. This run showed minor sensitivity (approximately 100mm) of flood levels to the entrance bar assumed as being unscoured.

Figure 6-1 Flood level sensitivity to modelled entrance shoaling and scouring – South West Rocks gauge – March 2001 calibration event



6.7 Comparison to Previous TUFLOW Modelling Calibration

The modelled water surface profiles from the previous Kempsey CBD TUFLOW model (WMAwater, 2015) are compared to the current modelling on Figure C-7 in Appendix C and Figure D-7 Appendix D for the 2001 and 2013 flood events, respectively. It is observed that the downstream boundary water level in the Kempsey CBD models are markedly lower than the modelled flood levels in the current TUFLOW model. The Kempsey CBD model downstream boundary conditions are the simulated flood levels from RUBICON model of the Macleay River for both events, rather than an observed water level.

It is noted from Table 2 in WMA (2009) that the RUBICON model simulated flood levels 0.17m lower at the Nestle Factory at Smithtown (downstream of the Kempsey CBD boundary) and downstream when compared to the observed flood levels in the 2001 event. In contrast, the Lower Macleay TUFLOW model simulated a flood level 0.07m lower than the observed at this location. The difference in the modelled flood levels partially explains the difference in flood levels at the Kempsey CBD model boundary. It is suspected that the Lower Macleay TUFLOW model simulates a higher head loss at Seven Oaks Bend, in the range of up to 0.26m for both the 2001 and 2013 events, compared to approximately 0.2m in the RUBICON model. These two variances in the model results explain the flood level differences between the current modelling and the Kempsey CBD modelling at the Kempsey CBD downstream boundary.

The RUBICON flood levels, which were used to derive the Kempsey CBD downstream water levels, are also plotted on Figure C-7 in Appendix C for the 2001 event. The plot shows that the modelled water levels are low compared to the Lower Macleay TUFLOW levels up to Seven Oaks Bend and the Kempsey CBD downstream boundary. This lower water level may have been required to achieve a satisfactory calibration at Kempsey Traffic Bridge. In comparison, the Lower Macleay TUFLOW water levels are higher at Seven Oaks as a result of achieving a good calibration at Smithtown gauge and Nestle. This was a higher priority given the key location of Smithtown and Gladstone within the floodplain and in the context of this flood study. While there is a poorer fit to observed levels around Seven Oaks and the Farley property, the Lower Macleay TUFLOW model also calibrates satisfactorily to Kempsey Traffic Bridge gauge.

The difference in flood levels between the two models reduces between Seven Oaks, through Frederickton to Kempsey. Flood levels at the Kempsey Traffic Bridge gauge are similar. Lower Manning's n values, though within reasonable limits, were adopted in the current model to achieve a good calibration at the gauge.

Flood levels upstream of the traffic bridge are higher in the Lower Macleay flood model. It is suspected that the coarser 20m grid in the current model does not replicate the actual river conveyance compared to the more detailed 10m Kempsey CBD model grid. There are two narrow constrictions in the channel upstream of the traffic bridge (down to 110m wide) at which the current model appears to estimate higher head losses. As previously stated, the current study area is concerned with the Lower Macleay floodplain downstream of Kempsey and the TUFLOW model has been calibrated satisfactorily downstream of the Kempsey Traffic Bridge. It is recommended that the Kempsey CBD TUFLOW model (WMAwater, 2015) be referred to for design flood levels in and around the CBD area.

6.8 Conclusions on Model Calibration and Verification

The Lower Macleay TUFLOW model was satisfactorily calibrated to observed water levels for the selected historic flood events. In particular, the model replicates peak flood levels, times to peak flood levels, rate of rise and overall hydrograph shape, which indicates the flood volume, at the key reliable stream gauges located between Kempsey and the river entrance. There are some generally minor departures from the observed flood levels at some locations, although it is reasonable to expect some variances as the flood modelling does not represent some physical processes such as wave action and localised hydraulic effects which can result in the differences between simulated and observed levels. Differences may also arise from uncertainty in precise conditions, such as influence of debris and changing channel bedform, during actual flood events.

Overall, the model is considered reliable for its intended purposes of defining moderate to major flooding events with the observed variances in the calibration within acceptable limits.

7. Estimation of Design Floods

7.1 Adopted Parameter Values and Assumptions

7.1.1 River Inflows

The design flood flows upstream of Kempsey as defined in WMAwater (2015) are adopted in this study. The peak flows are summarised in Table 7-1. Design flow hydrographs are presented in Appendix E. The previous Macleay River flood studies (Webb McKeown and Associates, 2009; WMAwater, 2015) describe the critical storm event which results in the design flows below as being a two-day duration storm. Note that the PMF flow estimate here is significantly higher than the 40,000m³/s adopted in the *Macleay River Flood Study* (Webb McKeown and Associates, 1989) due to updated guidance on the estimation of extreme rainfall and flood events.

Table 7-1 Macleay River Peak Flows for Design Flood Events (source: Table 6 in WMAwater, 2015)

Event	Macleay River Inflow (m ³ /s)
0.2EY	5,400
5% AEP	9,900
1% AEP	15,300
0.2% AEP	24,800
PMF	67,500

The flood inflow hydrographs were derived (Webb McKeown and Associates, 2009; WMAwater, 2015) from the WBNM hydrologic modelling for the Macleay River catchment, based on ARR 1987 design rainfall. The hydrologic modelling was calibrated to historical events and validated using results from flood frequency analysis (FFA) of stream gauge data at Turners Flat, which included the 1949 and 1950 flood events which were estimated 1 in 90 and 1 in 80 AEP events, respectively. Update of the hydrologic modelling based on ARR 2016 guidelines would not be expected to significantly change peak flows or flood volumes, as the ARR 1987 and ARR 2016 design rainfall depths for the two-day duration storm at Kempsey are similar (refer to Figure B-4 in Appendix B) and any recalibration of the model would be to FFA-derived stream flows, which would not have changed from the previous studies.

7.1.2 Local Catchments Hydrologic Modelling

7.1.2.1 Design Rainfall

The design rainfalls on the local catchments in the study area were spatially varied in the design storm hydrologic modelling. Design IFD data is input based on rainfall gauges at upper Collombatti Creek/Eungai, Kempsey and Smoky Cape.

7.1.2.2 Areal Reduction Factor

An areal reduction factor was applied to account for the likelihood that the peak rainfall depths would not occur uniformly across the study area. Guidelines in ARR 2016 were used to estimate areal reduction factors based on the catchment area of Collombatti Creek upstream of the railway line (100km²) and varied by AEP. The areal reduction factors are summarised in Table 7-2. This factor was applied to all local catchment inflows. Note that the factor is applied only to the local catchment hydrology and not to the Macleay River inflows.

Table 7-2 Adopted Areal Reduction Factors

AEP	Areal Reduction Factor for Local Catchment Hydrology
0.2EY	0.86
5%	0.84
1%	0.82
0.2%, PMF	N/A. 1% AEP is the largest event modelled for local catchments. Refer Table 7-7.

7.1.2.3 Storm Durations and Temporal Patterns

Critical storms for the local catchment flooding were selected to maximise the design inflows and flooding in the upper sections of Collombatti Creek and Christmas Creek in the TUFLOW model, in addition to the contribution of rainfall to flooding on the main floodplain. A 2 hour storm duration was selected for the creek flooding, and a 48 hour duration for the rainfall on the floodplain local catchments. The adopted storm temporal patterns were selected based on ARR 2016 guidelines which were incorporated into the DRAINS hydrologic model. The selected storms are summarised in Table 7-3.

While flooding on the floodplain is expected to be dominated by main river overflows rather than local rainfall, the total rainfall depths for the 48 hour storm are in the range of 450mm which is expected to have a substantial contribution to flooding.

Table 7-3 Selected ARR 2016 Representative Storms for Design Flood Estimation in Local Catchments

Event	Selected Storm Pattern	
	Local Creek Catchments	Floodplain
0.2EY	2 hour (Storm 8)	48 hour (Storm 8)
5% AEP	2 hour (Storm 2)	48 hour (Storm 6)
1% AEP	2 hour (Storm 8)	48 hour (Storm 2)
0.2% AEP	The selected 1% AEP storm is adopted for this design run. Refer to Section 7.2.4.	
PMF		

7.1.2.4 Rainfall Losses

Design event rainfall losses used in the previous flood studies for the Macleay River were adopted, namely:

- Initial loss: 0mm
- Continuing loss: 2mm/hr.

Refer to Section 4.3.3.1. These were selected in the previous studies based on hydrologic model calibration to a number of major historic flood events. These are applied to the events modelled for local catchments, up to the 1% AEP. Note that the local catchment 1% AEP event was modelled coinciding with the Macleay 0.2% AEP and PMF events.

7.1.2.5 Local Catchment Inflows inside Levee Protected Areas

Local catchment inflows were input into selected village areas, including Frederickton and Hat Head, which are protected by levees and which are not overtopped by mainstream flooding in frequent events. The critical Lower Macleay floodplain storm for each AEP event, as indicated on Table 7-3, were input into these areas. Local overland flooding was not a focus for this flood study and may require further assessment, particularly for Frederickton where overland flooding in the Frederickton Drain local catchment may affect properties. This approach was agreed with Council.

Other villages have levees which become overtopped in frequent events (less than the 0.2EY event). Flooding conditions in these villages are dominated by the mainstream flooding with minor influence from local runoff. Local inflows were omitted from these areas.

7.1.3 Floodplain Initial Water Levels

There is a likelihood that there would be some standing water present on the floodplain at the start of a flood event due to preceding rainfall. For the purposes of the design flood estimation it was assumed that there would be up to 0.5m of water on the floodplain at the start of the design flood. Local observations indicate that such ponding depths may be expected in the low points on the floodplain following a substantial, but not flood-causing, rain event.

7.1.4 Recently Constructed Roads and Infrastructure

Recently constructed roadworks in the design flood modelling include the Pacific Highway upgrade and Kempsey Bypass road, bridge and drainage structures. These works were completed around 2013 and include substantial fill embankments and other structures. Council also recently completed roadworks at Plummers Lane West and Plummers Lane at Summer Island Road, and South West Rocks Road. These works were included in the TUFLOW model.

7.1.5 Operation of Flood Control Structures

The Belmore River and Kinchela Creek flood control structures were assumed shut initially and then opened when flooding at Kempsey and the entrances to Belmore River and Kinchela Creek reach threshold levels. Council monitors the KTB gauge levels and considers opening the structures once water levels reach 5.1m AHD and are expected to continue to rise. Under current practice, structures are manually opened once the Belmore River and Kinchela Creek levees are just starting to overtop the levees (Webb McKeown & Associates, 2004a). This is about 3.8m AHD for the Belmore River flood control structure and 3m AHD for the Kinchela Creek structures, although resources and staff safety are considered in the decision and timing of opening the structures and the actual operation during a flood event may be different. The initial closed condition and then opening of the flood control structures was implemented in the TUFLOW model using variable z-shape objects at the structure outlets. The water level trigger point for the modelled opening of the structures is at the entrance to Belmore River and Kinchela Creek, respectively.

7.1.6 Floodgates

Floodgates were assumed to be properly functioning, with full seal of the gate flaps on the inlet structure and non-reversing flow. Once flood levels downstream of the floodgates drop below levels upstream, water flows through the structure in the downstream direction. Yarrahapinni floodgates are removed and the tidal barrages breached, as per the conditions in 2019.

7.1.7 Scouring of Waterway Entrances

7.1.7.1 Macleay River Entrance

Preliminary modelling of the design flood events assumed that the sandbars and shoals in the Macleay River entrance would be scoured by 2 – 3.5m to bed levels of -4 to -6m AHD, similar to the calibration event simulations. Additional detailed modelling of the river entrance geomorphology and sediment transport during flood events was undertaken to provide a better understanding of bed scouring in flood events and to confirm and validate the assumed degrees of entrance bed scouring. This is documented in Appendix I.

In summary, the geomorphic modelling confirmed that the amount of bed scouring assumed in the 2001 calibration run could also be assumed for the design 1% and 0.2% AEP floods, despite these floods being significantly rarer than the 2001 calibration flood event (a 1 in 13 AEP event). The duration of flood flows is an important factor in determining the degree of scouring. Flood flow conditions occur for approximately 75 hours in the 2001 event, compared to approximately 50 hours in the design 1% and 0.2% AEP events. The difference in flood flow conditions for the design events contributes to reduced degree of scouring than might be expected.

In the PMF the entrance geomorphic modelling predicts significant degree of scour greater than the 1% and 0.2% AEP events. Bed scour levels of down to -8.5m AHD were adopted.

The scoured depths were estimated based on the critical 2-day duration flood event flows in the Macleay. While longer duration events may result in extended duration of scouring, the flow magnitudes are expected to be less, resulting in less scouring. Further, peak flood levels in these non-critical duration flood events would be lower. Hence, the approach taken in this study ensures the peak flooding at the river entrance is not underestimated.

The scoured channel flow area and associated scour depths at the time of peak flood flows in the entrance geomorphic model were adopted in the TUFLOW model, rather than peak scour depths at the end of the flood event.

7.1.7.2 Creek Outlets

Assumed scouring of the outlets of Korogoro Creek, Killick Creek and Ryans Cut for the design events was the same as for the calibration events, where the entrances were assumed to scour to a bed level of -1.0m AHD, to the full available width at the entrance. The scoured bed level is assumed to be limited by the normal bed levels in the creeks just upstream of the entrance. A qualitative assessment is provided in the entrance morphology analysis in Appendix I.

The entrance of South West Rocks Creek was not assumed to scour out under flood conditions due to the entrance being trained. The entrance is regularly dredged to maintain a navigable entrance for trailer boats.

7.1.7.3 Breaching of Coastal Sand Dunes

During the PMF event floodwaters could breach the coastal sand dunes at numerous locations along the coastline from Grassy Head down to Crescent Head. This process is highly dynamic and there is no certainty on the precise locations, nature and extent of dune breaches and the impact on flooding conditions. As a result, it was agreed with Council and OEH to adopt a conservative approach for the design flood estimation and that it is considered reasonable to assume that all breached dunes are static and do not erode in the PMF simulation. A qualitative assessment is provided in the entrance morphology analysis in Appendix I.

The sensitivity of PMF levels to a dune breach was assessed in sensitivity modelling. There is one particularly low and vulnerable section of dunes just to the north of the Macleay River entrance and a breach at this location was tested. Refer to Section 9.2 for details.

7.2 Concurrent Flooding and Tailwater Conditions

7.2.1 Description of Different Flooding Mechanisms

A number of different sources contribute to flooding in the study area:

- Macleay River mainstream flooding. River flood flows generated from rainfall on the catchment upstream of Kempsey.
- Local catchment rainfall-runoff. Floodwaters from rainfall on the local creek catchments (Collombatti Creek, Christmas Creek and other minor watercourses) and onto the Macleay River floodplain during the main flooding storm event, though this is a minor contributor to flooding compared to the main river flood on the majority of the floodplain. Rainfall on these areas occurring before the main flood may also contribute to standing water on the floodplain, and is represented by the initial water levels in the model (refer to Section 7.1.3).
- Overflows from the Maria River/Hastings River catchment during a coinciding flood event. Floodwaters from a Hastings River flood event may overflow into the Macleay River via Maria River and Connection Creek.
- Elevated ocean levels. Storm surge and wave set up at the river and creek entrances may contribute to oceanic inundation of low-lying areas which are not protected by levee and floodgate systems. Elevated ocean levels may also prevent river floodwaters from discharging to the ocean as freely.

The design flood levels estimated by this flood study need to account for the contributions to peak flooding conditions from each of these sources. However, it is generally considered unlikely that the peak flooding conditions from each mechanism and for each event AEP would occur concurrently.

The envelope approach is used in this study, where for example, the 1% AEP Macleay River flood coincides with a 5% AEP ocean level, and then the 1% AEP ocean level is analysed coinciding with the 5% AEP catchment flood. The maximum envelope is taken from the peaks of the two scenarios to define the peak flood level in the study area. This approach is recommended in *Floodplain Risk Management Guide – Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways* (OEH, 2015).

7.2.2 Joint Flooding Analysis

7.2.2.1 Ocean Water Levels

Guidance is provided in *Floodplain Risk Management Guide – Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways* (OEH, 2015) on elevated ocean levels coinciding with river flooding. A recommended ocean level is provided, depending on the type of waterway entrance discharging on the ocean and on the coincident river flood event. The waterway type for the waterway entrances in the study area according to the guideline are summarised in Table 7-4. The adopted joint AEP and ocean water levels are discussed in Section 7.2.3.

Table 7-4 Waterway Entrance Classification

Waterway Type	Description	Waterway	Comments
Waterway entrance Type B	A Group 3 Wave Dominated Estuary with fully trained (both sides) entrance which are not maintained as navigable ports. These entrances result in little ocean tide attenuation but have some potential for wave setup.	Macleay River	
Waterway entrance Type C	Group 3 Wave Dominated Estuary with untrained or partially trained entrances which	Korogoro Creek Killick Creek	Type C ocean level applied along beach between Smoky Cape and

	are likely to have very shallow flow depths across the entrance or may fully close from time to time.		Hat Head, and from Hat Head to Crescent Head.
	Group 4 Intermittently Closed Estuaries or ICOLLS	Ryans Cut	Beach ocean levels as described above apply to these entrances.
Assumed Waterway entrance Type B	Group 3 Wave Dominated Estuary with partially trained entrance	South West Rocks Creek	Has Type C characteristics but is dredged from time to time to maintain small boat navigation of the entrance. Due to proximity to main Macleay River entrance and for more simplicity of modelling, assumed to have Type B ocean levels. Not expected to have major influence on river flood levels (whether assumed to be Type B or Type C).

7.2.2.2 Maria River Flooding

Information on coinciding historic floods in the Macleay River and Maria River was obtained from the *Hastings River Flood Study* (Patterson Britton and Partners, 2006), recorded water levels at Green Valley on Maria River and from Macleay River historic flood data as described earlier in this report. Details on the identified coinciding historic floods are provided in Table 7-5.

Table 7-5 Coincident Historic Floods in Macleay River and Maria River

Date	Macleay River (Kempsey Traffic Bridge)		Maria River (Green Valley)	
	Flood Level ¹ (m AHD)	AEP ¹	Flood Level (m AHD)	AEP ²
August 1949	8.52	1 in 90	2.57 ²	~1 in 5
June 1950	8.31	1 in 80	No Data	No Data
April 1963	7.68	1 in 15	3.2 ²	1 in 50
January 1968	6.31	1 in 2-5	2.65 ²	1 in 5-10?
March 1974	6.23	1 in 2-5	2.94 ²	1 in 20
March 1978	No Data	No Data	2.41 ²	~1 in 5
March 2001	7.44	1 in 13	1.89 ³	<1 in 5
February 2013	7.11	1 in 7	2.42 ³	~1 in 5

1. Kempsey flood levels from Table 3-2. Flood AEPs as stated in *Kempsey Shire Local Flood Plan* (NSW SES, 2017) or estimated from design flood levels in *Kempsey Hydraulic Model TUFLOW Update Final Report* (WMAwater, 2015).
2. Data sourced from Figure 4, Location of Historic Flood Marks, at Oaklands (Maria River) near Green Valley. *Hastings River Flood Study* (Patterson Britton and Partners, 2006).
3. Data sourced from MHL water level recorder @ Green Valley, Maria River.

The Hastings River Flood Study modelling assumes that there is a “glass wall” at the model boundary preventing any outflow from the Maria River into the Macleay floodplain. Hence, the flood levels derived from that study are considered conservative.

No correlation in AEP was observed between concurrent flood events in the Macleay River and the Maria River, particularly for the rarer events, given that these rivers are located in large regional catchments which may not be affected to similar degrees by the same flood-producing storm event. Further, the Hastings River in which the Maria River is located is a complex system with two main rivers (the Hastings River and the Wilson River) influencing flooding in the Maria River. Hence, the assumed concurrent Maria River flooding was selected to be consistent with the OEH (2015) ocean flooding guidance as described in Section 7.2.2.1. Maria River flooding is expected to be influential on peak flooding levels only in the far south-eastern portion of the study area. The design peak tailwater levels in the Maria River are summarised in Table 7-6. The adopted tailwater hydrographs were extracted from modelling results files from the *Hastings River Flood Study* (provided by Port Macquarie Hastings Council) and are presented in Appendix E.

Table 7-6 Design Flood Levels in Maria River (ref: *Hastings River Flood Study*, Patterson Britton and Partners (2006))

AEP	Peak Flood Level (m AHD)	AEP	Peak Flood Level (m AHD)
0.2EY*	~2.4	0.5%	4.1
5%	2.9	0.2%*	~4.5
1%	3.7	PMF	6.9

* Design flood level inferred for this AEP.

While there is significant length of recorded water level data in other parts of the Hastings River and Wilson River (e.g. Wauchope) available on PINNEENA, this data was not reviewed as the gauge locations are at significant distances from the Maria River area and hence the recorded flooding at these gauges may not be correlated to Maria River flooding.

7.2.3 Adopted Concurrent Flooding Conditions

The adopted concurrent flooding and tailwater conditions are summarised in Table 7-7. Ocean water levels are modelled as tidally-varying with storm surge and wave setup included, based on recommended water level versus time plots for north of Crowdy Head in OEH (2015). Catchment flooding near the river entrance is generally assumed to coincide with high tide/storm surge, except for the 1% AEP Macleay River flood (low ocean tide level scenario) where catchment flooding is assumed to coincide with low tide (Indian Spring Low Water). This scenario is simulated to maximise flood flow velocities in the lower reaches. Refer to the figures in Appendix E for the design ocean water level hydrographs.

Table 7-7 Adopted Design Joint Flood Scenarios

Design Flood Envelope AEP	Dominant Flood Mechanism	Flood Event AEP			
		Macleay River	Local Catchment	Maria River	Ocean
0.2EY (i.e. 1 in 5)	Macleay River	0.2EY	0.2EY	0.2EY	HHWS(SS) 1.25m AHD
	Maria River	N/A. Incorporated into Macleay River flood run.			
	Ocean	N/A. HHWS(SS) is highest recommended TWL.			
5%	Macleay River	5%	5%	5%	HHWS(SS) 1.25m AHD
	Maria River	N/A. Incorporated into Macleay River flood run.			
	Ocean	N/A. HHWS(SS) is highest recommended TWL.			
1%	Macleay River	1%	1%	5%	5% Type B. TWL 2.0m AHD

					Type C. TWL 2.45m AHD
	Macleay River (Low Ocean Tide Level)	1%	1%	5%	ISLW. TWL -0.84m AHD Same tidal pattern as HHWS(SS) except the low tide coincides with catchment flooding.
	Maria River	5%	5%	1%	5% Type B. TWL 2.0m AHD Type C. TWL 2.45m AHD
	Ocean	5%	5%	5%	1% Type B. TWL 2.1m AHD Type C. TWL 2.65m AHD
0.2%	Macleay River	0.2%	1%	1%	1% Type B. TWL 2.1m AHD Type C. TWL 2.65m AHD
	Maria River	1%	1%	0.2%	5% Type B. TWL 2.0m AHD Type C. TWL 2.45m AHD
	Ocean	N/A. 1% AEP is highest recommended TWL.			
PMF	Macleay River	PMF	1%	1%	1% Type B. TWL 2.1m AHD Type C. TWL 2.65m AHD
	Maria River	1%	1%	PMF	5% Type B. TWL 2.0m AHD Type C. TWL 2.45m AHD
	Ocean	N/A. 1% AEP is highest recommended TWL.			

Abbreviations: "TWL" = Tailwater Level. "HHWS(SS)" = High High Water Spring (Solstice Spring) tide. "ISLW" = Indian Spring Low Water.

7.2.4 Relative Timing of Concurrent Flooding

The relative timing of the ocean storm surge to catchment flooding has the potential to affect peak flood levels particularly in the lower portions of the floodplain. Current guidance in OEH (2015) recommends coinciding the peak of the storm surge with the peak catchment flooding at key points of interest.

The impact of storm surge timing is greatest at the entrance and becomes negligible at Summer Island, just downstream of Kinchela (Webb McKeown and Associates, 1989). Timing the storm surge with peak flooding at the entrance was initially considered, however, this timing is thought to be unlikely. Flood-producing storms on the NSW North Coast are generally associated with ex- tropical cyclones or East Coast Lows, which also have potential to produce elevated ocean levels due to storm surge and wind setup. The nature of these storms is such that peak rainfall and peak ocean levels occur at about the same time (Webb McKeown and Associates, 1989). Flooding at the Macleay River entrance tends to peak about 2 days after peak rainfall, as illustrated on Figure C-2 and Figure C-7 for the March 2001 flood. By the time flooding peaks at the entrance the storm system has passed on and the elevated ocean levels will have subsided somewhat.

Previous flood studies on the Macleay River (Webb McKeown & Associates, 1989) and other rivers such as the Manning River (BMT WBM, 2016) have adopted a timing of peak ocean levels which coincide with peak inflows into the lower valley. A similar approach was adopted in this study, coinciding the peak storm surge with peak catchment inflows at Kempsey.

Sensitivity testing was done to assess the impact of varying the timing of peak storm surge. Refer to Section 9.2 for further details.

7.3 Peer Review

A technical peer review was conducted on the TUFLOW model development and configuration. Minor issues with the model setup were identified and incorporated into the modelling. Overall, the modelling was found to provide an adequate representation of flooding behaviour and suitable for the estimation of design flooding conditions.

8. Design Flood Results and Mapping

8.1 Flood Mapping

Flood mapping is provided in E.1 for the design 0.2EY, 5%, 1% and 0.2% AEP and PMF events. Depth and water level contours, velocity and flood hazard are presented. Mapping is provided for the broader study area in addition to zoomed in views of urban areas and villages at South West Rocks, Jerseyville, Fishermans Reach, Stuarts Point, Frederickton, Smithtown, Gladstone, Kinchela, Crescent Head and Hat Head. The extent of the nested finer 5m model grids at these villages is shown. The flow directions are indicated on the zoomed in village maps of the depth and velocity.

For those flood AEPs where different flooding mechanisms were modelled in separate scenarios (refer to Section Table 7-7 for overview of the scenarios), a maximum mapping envelope was derived from combination of the separate scenario results.

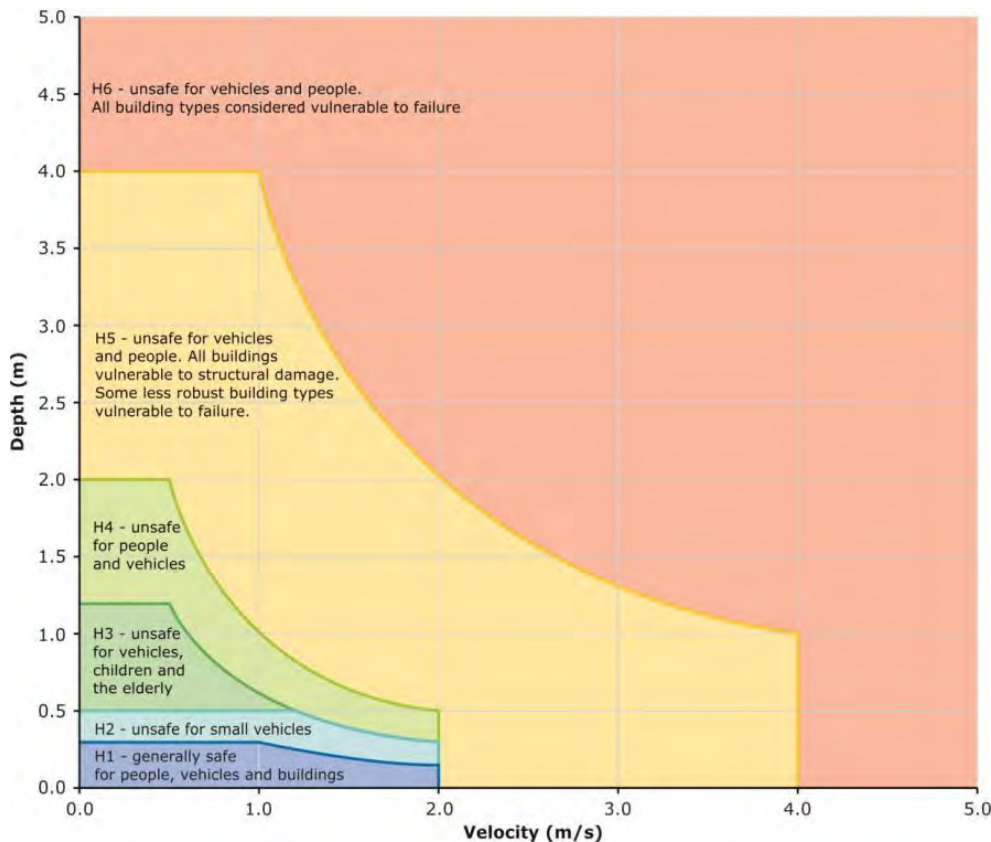
Note that the flood mapping shows the flood conditions over the ground surface beneath bridges crossing the floodplain, including the Pacific Highway bridge crossing the Macleay floodplain. This does not necessarily mean that the bridge is overtopped. Note also that the maps indicate the area around Kempsey CBD where the 2017 Kempsey CBD FRMS (WMAwater, 2017b) should be deferred to for the design flood conditions.

The flood hazard mapping was prepared based on the latest guidelines. Recent research has been undertaken into the hazard that flooding poses and the vulnerability of the public and assets when interacting with floodwaters. A combined flood hazard classification is presented in *Australian Disaster Resilience Handbook 7. Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia* (AIDR, 2017a) and *Guideline 7-3 Flood Hazard* (AIDR, 2017c) based on this research, and is illustrated in Figure 8-1. The flood hazard categories according to the AIDR definition are:

- H1 – Generally safe for people, vehicles and buildings;
- H2 – Unsafe for small vehicles;
- H3 - Unsafe for vehicles, children and the elderly;
- H4 - Unsafe for people and vehicles;
- H5 - Unsafe for people and vehicles. Buildings require special engineering design and construction; and
- H6 – Unsafe for people or vehicles. All buildings types considered vulnerable to failure.

The flood hazard mapping is provided in Appendix F and is denoted provisional. The provisional mapping is based on direct flood modelling outputs and has not been revised to reflect the “true” flood hazard to take into consideration evacuation, isolation and other emergency management aspects. It is anticipated that the true hazard will be defined in the subsequent floodplain risk management study of the Lower Macleay Valley.

Figure 8-1 General flood hazard vulnerability curves, Australian Institute for Disaster Resilience (AIDR) definition. Reproduced from Figure 6 in *Guideline 7-3: Flood Hazard* (AIDR, 2017b)

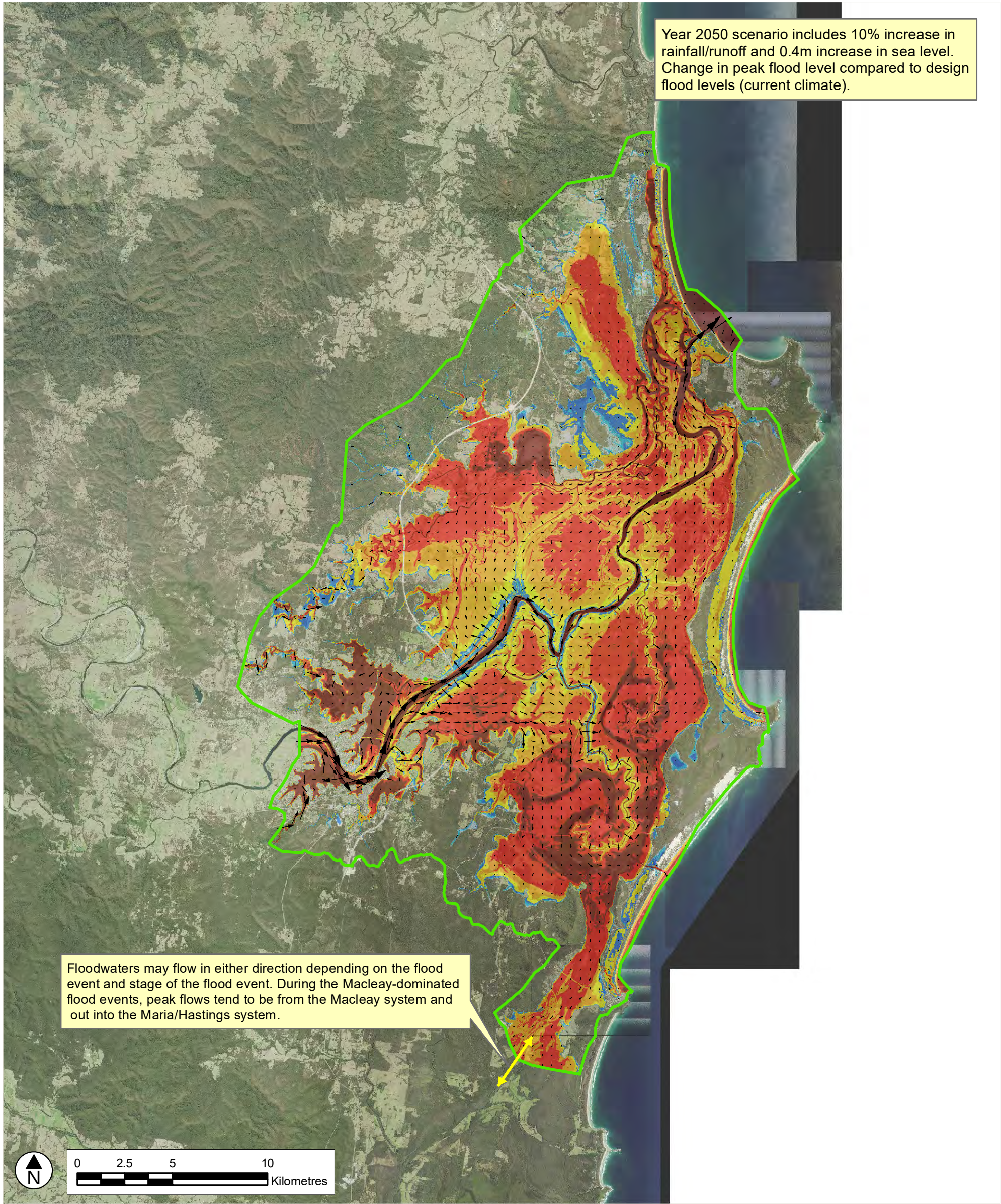


8.2 Flood Behaviour

Flow behaviour is a complex interaction between the Macleay River channel and the floodplains, in addition to coincident flooding in the Maria River. The flooding patterns and behaviour are summarised below based on the design flood modelling and generally in order of timing, although the magnitude of flows and the degree and timing of interaction between river and floodplain can differ from flood to flood.

Key flow patterns are indicated on Figure 8-2. Flood characteristics are summarised for the villages in the sub-sections in this report section.

- The main catchment flood flows occur as floodwaters enter the Lower Macleay floodplain from the Macleay River catchment. Christmas Creek and Collombatti Creek catchments also contribute a minor portion of floodwater. Flooding in the Lower Macleay generally peaks about 2 days after the start of the flood at Kempsey.
- Peak ocean levels (tide or tide/storm surge) are assumed to occur at the peak of flooding at Kempsey. Elevated ocean waters enter the system via the Macleay River mouth on each high tide, filling the estuary and low-lying floodplain areas with peak ocean inflows coinciding with the peak ocean level timing. Ocean water also enters the minor ocean creek outlets although intrusion into the main floodplain is halted by floodgates on these systems.
- Maria River flood levels rise and overtop low-lying Crescent Head Road (1.2m AHD) at early stages in the flood event, spilling into the southern floodplain via Connection Creek. Further discussion is provided in Section 8.2.10.



Legend

Depth (m)	Limit of Mapping	Flow Direction
0 - 0.2		→
0.2 - 0.5		
0.5 - 1		
1 - 2		
2 - 3		
3 - 4		
> 4		

JACOBS

LIMITATIONS: This mapping is based on data and assumptions identified in the Lower Macleay Flood Study prepared by Jacobs. Jacobs does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

A3			
SHEET	1 of 1	GDA 1994 MGA Zone 56	
TITLE	Flow Patterns in 1% AEP Event		
PROJECT	Lower Macleay Flood Study		
DRAWN	PROJECT #	MAP #	REV VER
LC	IA176800	FIGURE 8-2	1 1
CHECK	DATE		
AH	11/09/2019		

- Floodwaters flowing down the Macleay River encounter a major hydraulic control at Seven Oaks bend, with floodwaters building up due to this bend and Smithtown Bend in addition to the narrowed channel in several points through these sections. Floodwaters tend to break out of the river at Seven Oaks bend, prior to breaking out at other locations, onto the floodplain to the north through Bellimbopinni and into the northern (Collombatti Creek) floodplain. This is despite a constructed levee being present on the outside of the bend. Approximately 3,000m³/s overflows onto the northern floodplain in this area in the 1% AEP event. These overflows split to the east and west of Macleay Valley Way. To the west of the road floodwaters drain to the area around Clybucca Wetlands. To the east of the road floodwaters drain to a floodplain area north of Smithtown. Both these areas generally drain to the north-east via Clybucca Creek and Andersons Inlet to Macleay Arm, with some floodwater rejoining the main river around Rainbow Reach.
- As floodwaters rise at Belmore River and Kinchela Creek entrances these flows back up into these waterways. On-line floodgates in the upper reaches prevent flows from backing up via the natural watercourse and the lateral flood control structures at Belmore River left bank and Kinchela Creek left bank and right bank are assumed to open as the levee system is about to overtop (refer to Section 7.1.5) to allow floodwaters to spill into the southern floodplain. Total peak flow through the flood control structures is about 500m³/s in the 1% AEP.
- Significant amounts of flow break out between Austral Eden and Pola Creek onto the southern floodplain particularly in the 5% AEP event and rarer. Peak breakout flow rate is about 7,000m³/s in the 1% AEP. Depending on the magnitude of the flood event these overflows may occur before the Belmore River and Kinchela Creek flood control structures are opened. In any case, the magnitude of these overflows are significantly greater than the total capacity of the flood control structures.
- The floodwaters breaking out onto the southern floodplain between Austral Eden and Pola Creek generally flow southward to the west of Belmore River, around the head of Belmore River and then northward to the east of Belmore River and Kinchela Creek including through Swan Pool. The floodplain flows generally re-enter the main river in the vicinity of Jerseyville.
- Maria River inflows tend to occur earlier in the flood event. As the Macleay overflows into the southern floodplain peak later the water level differential changes and floodwaters then begin to flow into the Maria River system. Maria River inflows into the Lower Macleay peak at 500m³/s in the 1% AEP (refer to Section 8.2.10 and Figure 8-3) which is less than 10% of the peak overflows from the Macleay into the southern floodplain.
- As the southern floodplain fills, floodwaters begin to flow into Korogoro Creek, Killick Creek and Ryans Cut. These creeks and adjacent bank areas may be initially affected by coastal inundation but then become dominated by flooding from the Lower Macleay floodplain.
- Bank overflows from the river occur between Kempsey and Frederickton causing significant flooding on the Christmas Creek floodplain, with floodwaters backing up past the railway. Backwater flooding contributes significantly more to the flood volume in this floodplain area compared to Christmas Creek catchment flows. This flooding can also back up over Kempsey First Lane Levee and contribute to flooding of the CBD.
- In the northern portion of the study area floodwaters back up into Macleay Arm, affecting Fishermans Reach and Stuarts Point, in addition to up Yarrahapinni Wetlands and Borirgala Creek.
- The Macleay River entrance is the main system outlet and acts as a significant flow constriction. This is evident by the flat water surface profiles shown on the design flood profile plot on Figure F-16 in Appendix F. The large majority of floodwaters converge and flow out the trained entrance. Floodwaters overtop the southern training wall and flow out to the ocean via South West Rocks Creek, with minor overtopping in the 0.2EY event and more substantial overtopping and overflows in the 5% AEP and rarer.
- Floodwaters up to the 1% AEP event do not overtop, or only just overtop a low section of the coastal dunes just north of the main entrance. In the 0.2% AEP event overtopping locations are more common along the entire stretch between Grassy Head and Crescent Head. In the design PMF event overtopping is widespread. Note that since it is assumed in the modelling that there is no erosion of the dunes by floodwaters, the design PMF estimate is conservative. It is possible that floodwaters do not need to reach and overtop the dune crests to initiate breaching of the dunes and that infiltration and piping through the dunes could also result in eventual breach of the dunes, however it is not possible to quantify such an occurrence.

8.2.1 South West Rocks and Jerseyville

South West Rocks is affected by flooding from the Macleay River, South West Rocks Creek and Spencers Creek. Property yard flooding occurs along the river in the 5% AEP event and the Riverside Tavern, built on a filled pad, becomes surrounded by floodwaters but not submerged. Properties along South West Rocks Creek are not flood-affected in the 0.2EY event as flood levels are dictated mainly by ocean tide levels in up to the 5% AEP event.

Property flooding becomes more substantial in the 1% AEP event particularly along South West Rocks Creek when significant overflow occurs from the Macleay River into South West Rocks Creek, and on residential and industrial properties in southern South West Rocks from Spencers Creek, with depths of up to 1m. The ground around Riverside Tavern is submerged.

Significant flooding occurs in the 0.2% AEP and PMF as river overflows into South West Rocks Creek result in depths of over 2m in the PMF affecting the holiday park and floodwaters back up into the gully and marsh area south of Gordon Young Drive. Flood depths in southern South West Rocks are over 2m in the 0.2% AEP and over 4m in the PMF on properties.

In Jerseyville due to the low-lying nature of the properties the village is inundated in the 0.2EY event to depths of up to 1m, and 1 – 2m in the 5% AEP. Flood depths exceed 2m in the 1% AEP.

Flow velocities are relatively low (less than 0.5m/s) in all flooded properties in South West Rocks and Jerseyville in up to the 0.2% AEP event, with velocities up to 0.75m/s in Jerseyville in the PMF. There are localised areas of higher velocity. However, due to the high depths of flooding Jerseyville is affected by high (H3 – H4) flood hazard rating in the 5% AEP event and very high (H5) flood hazard rating in the 1% AEP.

8.2.2 Fishermans Reach

Flooding is contained in the watercourse in up to the 5% AEP event, and overflows the banks affecting property in the 1% AEP generally to 0.5m depth with areas up to 2m. Depths are generally 2m in the 0.2% AEP and up to 4m in the PMF. Flooding around Fishermans Reach is dictated by backwater flooding from the Macleay River backing up northward up Macleay Arm. Flow velocities are generally benign in the 1% AEP, around 0.25m/s with areas up to 0.5m/s. In the PMF velocities are typically 0.5m/s with localised areas at 1m/s or more.

Flood hazard is generally up to high rating (H3) in the 1% AEP and very high to extreme (H5 – H6) in the PMF.

8.2.3 Stuarts Point

Flooding begins to affect Stuarts Point village in the 5% AEP when the caravan park becomes inundated to over 0.5m depth, with up to 2m flooding depths in the 1% AEP. The majority of the village is flood-free up to the 0.2% AEP event and is affected by depths of 1 – 2m in the PMF. Flow velocities are generally up to 0.5m/s in the PMF in the village with areas in roadways experiencing up to 2m/s velocities. Flood hazard is generally up to high rating (H4) in the PMF.

Similar to Fishermans Reach, flooding around Stuarts Point is dictated by backwater flooding from the Macleay River backing up northward up Macleay Arm. There is also localised flooding in the low points between the dune features to the west of the village, which affects some properties there in the 5% AEP and rarer.

8.2.4 Frederickton

Frederickton is affected by flooding from the Macleay River as well as from two gullies in the east and north of the village. Both gullies are impacted by backwater flooding from Clybucca Wetlands and the northern floodplain. There is also flooding in Christmas Creek in the west of the village which is mainly caused by backwater from the Macleay River.

Properties in the south-east of the village experience yard flooding in the 0.2EY event from the river, and up to two properties are affected by flooding in the eastern gully. In the 5% AEP the river overtops its bank in the

vicinity of Frederickton Hotel and causes flooding of properties to depths of 0.5m. Backwater flooding in the eastern gully causes property flooding of 0.5 – 1m.

Most of the village is flood-free in up to the 0.2% AEP event. Some properties are affected by flood depths of 0.5 – 2m in the 1% AEP event and 1 – 3m in the 0.2% AEP event. In the PMF properties in the south-east and east of the village are affected by combined flooding from the Macleay River in addition to the eastern gully, with depths exceeding 4m on some properties.

Flow velocities on properties are generally up to 0.5m/s in up to the 0.2% AEP event and exceed 2m/s in some locations in the PMF. Flood hazard on affected properties is generally up to high (H3 – H4) in the 1% and 0.2% AEP with some very high (H5) hazard in deeper areas and flow paths, and very high to extreme (H5 – H6) in the PMF.

Some properties in Frederickton may also be affected by overland flooding in the gullies running through the village, however, this was not a focus of this study and the overland flooding is not shown on the flood mapping. Further assessment is recommended to define the overland flooding in this village.

8.2.5 Smithtown

Smithtown is affected in the 0.2EY event with the Macleay River overtopping the village levees and river banks at several locations, including on the western side north of Jeffery Street, at the southern end of the village and along the eastern side of the village. Flood depths are greatest in the middle of the village where there is a natural low point, where floodwaters generally flow northward on the floodplain. Depths are in the range of 1m in the 0.2EY event, up to 2m in the 1% AEP, 2 – 3m in the 0.2% AEP and over 5m in the PMF. The highest point of the Smithtown bridge western approach ramp is above the PMF, as is the bridge deck itself if it is not damaged during the flood event.

The highest flow velocities and hazard are due to the river breakout on the western side of the village, where velocities are 1.5m/s in the 0.2EY and 5% AEP events, and over 2m/s in the 1% and 0.2% AEP events. high flow velocities are also present as floodwaters flow northward over the Smithtown Road embankment. Flow velocities exceed 2.5m/s in the PMF.

Flood hazard is up to high (H3 – H4) in the 0.2EY event, mostly high in the 5% AEP event and very high (up to H5) in the 1% AEP event.

8.2.6 Gladstone

Gladstone is affected by bank overflows from the Macleay River. Depths of this bank overflow flooding are shallow in the 0.2EY event, less than 0.2m although there are areas with depths up to 0.5m. In the 5% AEP the bank overflows are also relatively shallow on the higher ground along the west side of the village, with greater depths of around 1m as backwater from the floodplain to the east begins to contribute to flooding. Flood depths are up to 2m in the 1% AEP, up to 3m in the 0.2% AEP and over 5m in the PMF. The highest point of the Smithtown bridge eastern approach ramp is above the 0.2% AEP flood level.

Flow velocities in the village are generally low (less than 0.25m/s) in up to the 5% AEP with velocities in flow paths and roads around 1 - 1.5m/s. In the 1% and 0.2% AEP the maximum flow path velocities are up to 2m/s. PMF velocities are 1 – 2m/s. Floodwaters tend to flow from the river and towards the east onto the floodplain in all events.

Flood hazard rating is up to high (H3) in the 5% AEP event, high to very high (H4 – H5) in the 1% and 0.2% AEP events and extreme (H6) in the PMF.

8.2.7 Kinchela

Kinchela flooding is caused by bank overflows from the Macleay River and Kinchela Creek. Depths of flooding are up to 0.5m in the 0.2EY event and 1m in the 5% AEP event. Flood flows pond and build up against South West Rocks Road and Hat Head Road before overflowing onto the wider floodplain to the east. In the 1% AEP

flood depths on properties are 0.5 – 2m, 2 – 3m in the 0.2% AEP event and over 6m in the PMF. The junction area of South West Rocks Road and Hat Head Road is above the 1% AEP flood, while the highest points of the bridge approaches over Kinchela Creek entrance are above the 0.2% AEP flood.

Flow velocities in the village are less than 0.5m/s in up to the 1% AEP. Higher flow velocities are present where floodwaters overtop and spill over the road embankments and river levee. Velocities exceed 2m/s in the 0.2% AEP and the PMF to the east of the village where flows break out of Kinchela Creek.

Flood hazard rating is generally low (H1 – H2) with some areas up to high (H3) in the 5% AEP event, high to very high (H3 – H5) in the 1% and 0.2% AEP events and extreme (H6) in the PMF.

8.2.8 Crescent Head

Crescent Head is affected by flooding from the Lower Macleay southern floodplain, on the western side of the village, in addition to flooding in Killick Creek which is influenced by flows from the southern floodplain into the creek combined with elevated ocean levels entering the creek entrance. In the 0.2EY event Killick Creek flooding is influenced mainly by high ocean tides with minor contribution of flows from the southern floodplain into the head of the creek. A few properties experience shallow yard flooding.

In the 5% AEP event floodplain inflows into Killick Creek increase and properties along the creek are flooded to depths of 0.5 – 1m. Flooding in the southern floodplain begins to encroach on the properties in the western side of town. Depths increase to 1 – 2m in the 1% AEP event along Killick Creek properties and up to 0.5m on western side properties. Floodwater begins to overflow from the southern floodplain to Killick Creek through properties.

In the 0.2% AEP event maximum flood depths on Killick Creek properties is over 2m and 1 – 2m on western side properties. The southern floodplain overflows at two locations. In the PMF depths on affected properties of 3 – 4m are common, with overflows from the southern floodplain to depths of over 2m through the town. The southern half of town is located on elevated ground above the PMF.

Active flows are contained mainly in Killick Creek with velocities over 2m/s in the 5% AEP event and rarer. Flow velocities in the southern floodplain are generally low, less than 0.5m/s, affecting the western side of town in up to the 0.2% AEP event. There are localised higher velocity flows in streets in the 0.2% AEP event. In the PMF there are numerous locations where velocities exceed 1.5m/s due to overflows from the southern floodplain.

Flood hazard is generally low – moderate (H1 – H2) on affected properties in the 1% AEP, with some low-lying properties along Killick Creek with high hazard (H4). The extent of low – moderate hazard increases through the centre of the village in the 0.2% AEP with maximum hazard on properties increasing to very high (H5). In the PMF there are extensive areas of very high – extreme (H5 – H6) hazard through the town. The sewage treatment plant to the north of the town appears to be mostly unaffected by the 0.2% AEP flood.

8.2.9 Hat Head

Hat Head is affected by flooding in Korogoro Creek, which is influenced by flows from the southern floodplain into the creek combined with elevated ocean levels at the creek entrance. Levees along the northern and southern sides of the creek provide protection to village areas. The levees are overtopped in events rarer than the 5% AEP event.

Local catchment runoff contributes to flooding of the levee-protected areas in the northern and southern parts of the village. These flows pond in the low-lying area behind the levee on both sides of the creek. This is the dominant source of flooding in the village in the 0.2EY and 5% AEP events, when the levees are not overtopped and mainstream floodwaters are contained in the creek. Some properties to the south of the creek are affected by flooding to depths of up to 0.8m in the 0.2EY event and up to 1m in the 5% AEP in the low points of properties. Depths of flooding in the drainage low points in the village area to the north of the creek are generally shallow, up to 0.2m in the 0.2EY event and up to 0.3m in the 5% AEP event.

In the 1% AEP event the village on both sides of the creek are inundated typically to depths 0.6 – 1m and localised areas over 1m. Depths in the village increase to over 2m in the 0.2% AEP. This is predominantly due to floodwaters overtopping the levee and causing flooding of the village areas in both of these events.

Flood depths in the PMF are 4 – 5m in the village. Flows into Korogoro Creek from the southern Lower Macleay floodplain are limited in up to the 0.2% AEP event due to the constriction of the hydraulic connection through the sand hills between the floodplain and the head of the creek, to the north of the village. There are significant overflows over the sand hills separating the floodplain and Korogoro Creek which entirely fills the creek floodplain between the sand hills and the coastal beach dunes. The narrow creek outlet abutting Korogoro Point headland is the only outlet for these floodwaters. Most of the village is inundated in the PMF, with some properties to the south of the creek located on high ground above the PMF.

Flow velocities on the floodplain and through properties are low in up to the 1% AEP, generally less than 0.5m/s in the streets. These velocities increase to 0.75m/s in the 0.2% AEP. In the PMF maximum flow velocities are generally above 1m/s with higher velocities near Hat Head Road crossing where flows break out of the creek and near the boat ramp where there are steeper ground slopes and flows re-enter the creek. There are also high velocities where floodwaters spill over the sand dunes onto the beach.

There are areas of low to high (H1 – H3) flood hazard in the southern part of the village in the 0.2EY event. The northern part of the village is mainly not flooded in this event with only some nuisance flooding in low points. In the 5% AEP event the flood hazard is generally high (H3) in flood-affected areas in the southern part of the village. The hazard rating increases to high (H3) in the northern part of the village in the 1% AEP event and is rated moderate to high (H2 – H3, some H4) in the southern side of the village. Flood hazard is very high (H5) in the 0.2% AEP and extreme (H6) in the PMF.

Note that further assessment of local catchment flooding inside the levees is required to confirm peak flooding, particularly in the 0.2EY and 5% AEP events.

8.2.10 Maria River Interflow

Floodwater interflows from the Maria River into the southern Macleay floodplain, and vice versa, occur in frequent, small magnitude flood events and up to large events due to the low-lying terrain of the floodplain of the Maria River and Connection Creek in addition to the low elevation of Crescent Head Road, which is the main flow control in the area. The direction of flow depends on the relative flood levels:

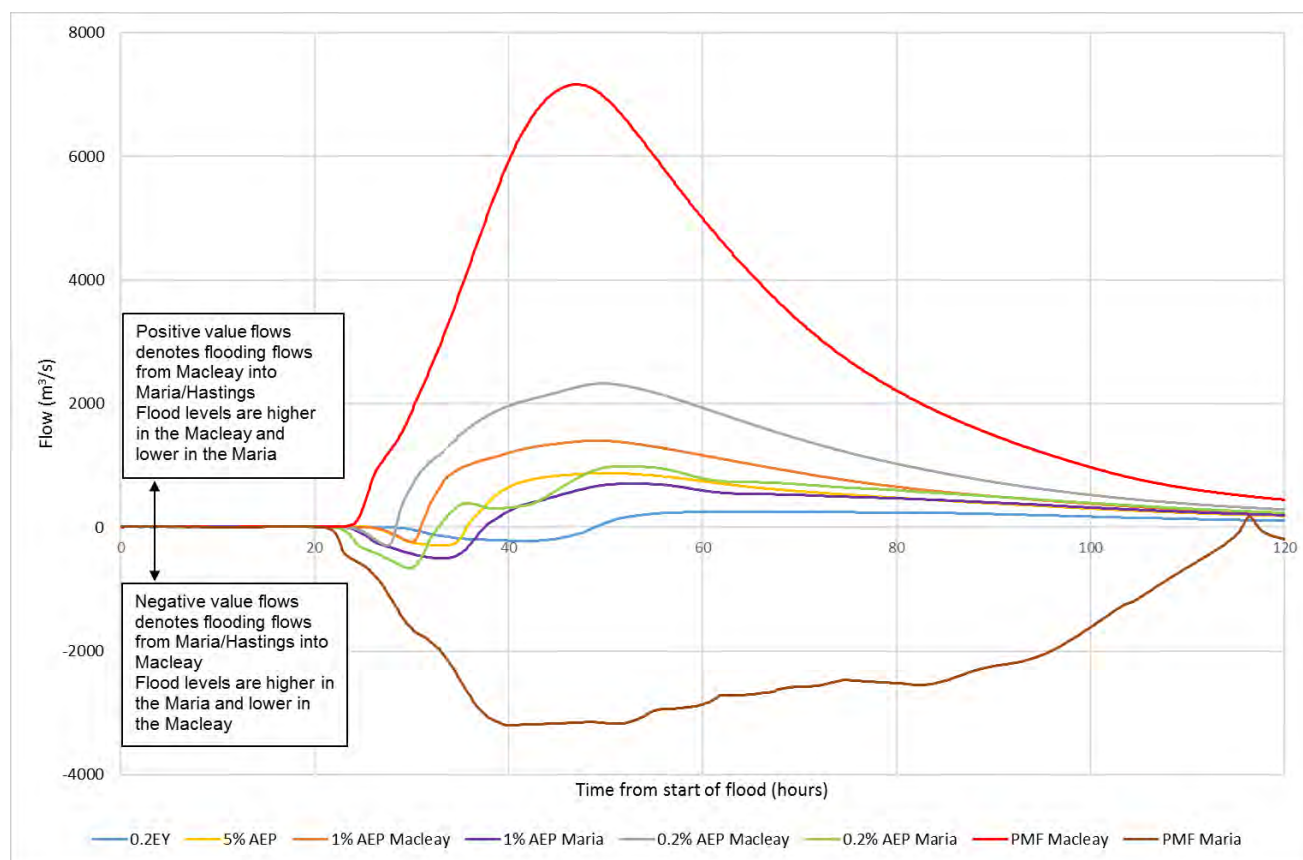
- Maria River flood levels higher than Macleay floodplain flood levels – results in flows from Maria River into Macleay floodplain
- Macleay floodplain flood levels higher than Maria River flood levels – results in flows from Macleay floodplain into Maria River.

The direction of flow can change during the course of a flood event as the water levels in the Maria River and the Macleay floodplain change relative to each other. For example, during the design events flooding the Maria River is expected to peak earlier than the Macleay floodplain and generally it is expected that there would be flow from the Maria to the Macleay. When the Macleay flooding peaks the Maria is expected to be receding and hence floodwaters would flow from the Macleay into the Maria. The flow rates of the interflow for the individual design events are shown in Figure 8-3, which show this reversal of flow.

Generally for the design events it is expected that the Maria River would contribute a relatively small flood inflow volume into the Macleay floodplain early in the event, then as the Macleay main river peaks and overflows into the floodplain, the flows then reverse and a larger outflow volume then flows from the Macleay floodplain into the Maria River.

During actual flood events there may not be flow reversal due to a relatively large flood occurring in one catchment and a smaller or no flooding occurring in the other. In such events flow velocities are expected to be relatively high due to the larger flood level difference and steeper flood gradient at the floodplain interface.

Figure 8-3 Design event flow rates of interflow between Maria River and Macleay floodplain



8.3 Design Flood Levels

Flood surface profiles along the Macleay River are also provided in Figure F-16 in Appendix F. Design flood levels are summarised at key locations throughout the study area in Table 8-1. Locations are shown on Figure F-17, and the flood level hydrographs at these locations are provided in Figures F-18 to F-31 in Appendix F.

Table 8-1 Design Flood Levels at Selected Locations (m AHD)

Event AEP	AEP				
	0.2EY	5%	1%	0.2%	PMF
Frederickton	5.84	6.26	6.56	7.01	9.32
Smithtown (Nestle)	4.50	4.72	5.03	5.68	8.79
Smithtown (Gauge)	4.25	4.43	4.64	5.61	8.71
Kinchela	3.42	3.57	4.38	5.56	8.68
Jerseyville	1.95	2.67	4.02	5.35	8.40
South West Rocks (Gauge)	1.19	1.93	3.47	4.94	7.76
Fishermans Reach	1.19	2.04	3.47	4.88	7.51

Stuarts Point	1.18	2.05	3.47	4.82	6.31
Collombatti D/S of Pacific Highway	2.81	3.46	4.39	5.61	8.73
Clybucca Floodgates U/S	2.40	3.16	4.28	5.51	8.57
Upper Belmore	2.67	3.79	4.49	5.57	8.61
Upper Kinchela	2.26	3.56	4.47	5.58	8.68
Hat Head (Creek)	1.27	1.71	2.62	4.08	6.81
Crescent Head (Creek)	1.32	2.16	2.61	3.25	4.87

8.4 Dominant Flooding Source

The dominant mechanism of flooding in the 1% AEP event is mapped across the study area in Figure F-32 in Appendix F. The mapping indicates the following:

- Flooding originating from the Macleay River catchment is the predominant critical flooding source across most of the Lower Macleay floodplain.
- Flooding from the Maria River is the critical mechanism for the floodplain area from just north of Crescent Head Road southward into Connection Creek. Maria River 1% AEP flood levels are 0.15m higher than the Macleay River 1% AEP flood levels at Crescent Head Road, up to 0.8m higher at the model boundary 5km south of Crescent Head Road.
- The 1% AEP ocean inundation event is dominant only in the far lower reaches of the Macleay River and the outlets of Korogoro Creek, Killick Creek, Ryans Cut and South West Rocks Creek.

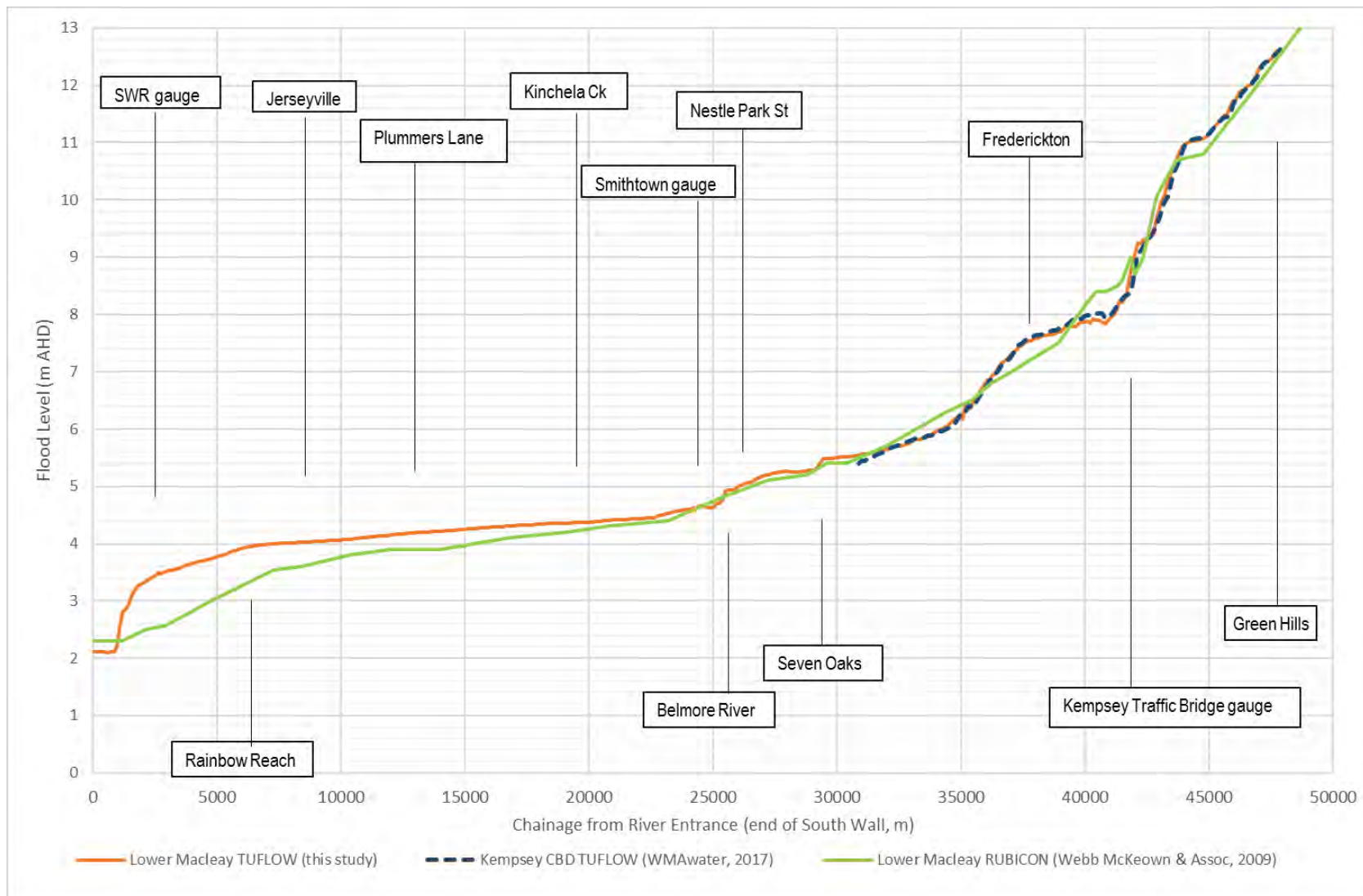
8.5 Annualised Risk of Flooding

The annualised risk of flooding is mapped in Figure F-33 in Appendix F, derived from overlaying the various AEP flood extents over each other. It represents the probability of locations across the study area from being flooded in any given year. The PMF is assigned an AEP of 1 in 10,000 (0.01%) based on the catchment area and guidance on Figure 8.3.2 from Book 8 in ARR 2016.

8.6 Validation of Results

Validation of model results is undertaken to check the differences or similarities in this study's flood level estimates to those in previous studies. The design 1% AEP flood level profile is plotted and compared to those from previous studies on Figure 8-4. The previous studies include RUBICON hydraulic modelling undertaken for the Lower Macleay, and reported in Webb McKeown & Associates (2004a) and the TUFLOW hydraulic modelling for the *Kempsey CBD Floodplain Risk Management Study* (WMAwater, 2015 and 2017b).

Figure 8-4 Comparison of TUFLOW Flood Profile with Previous Studies

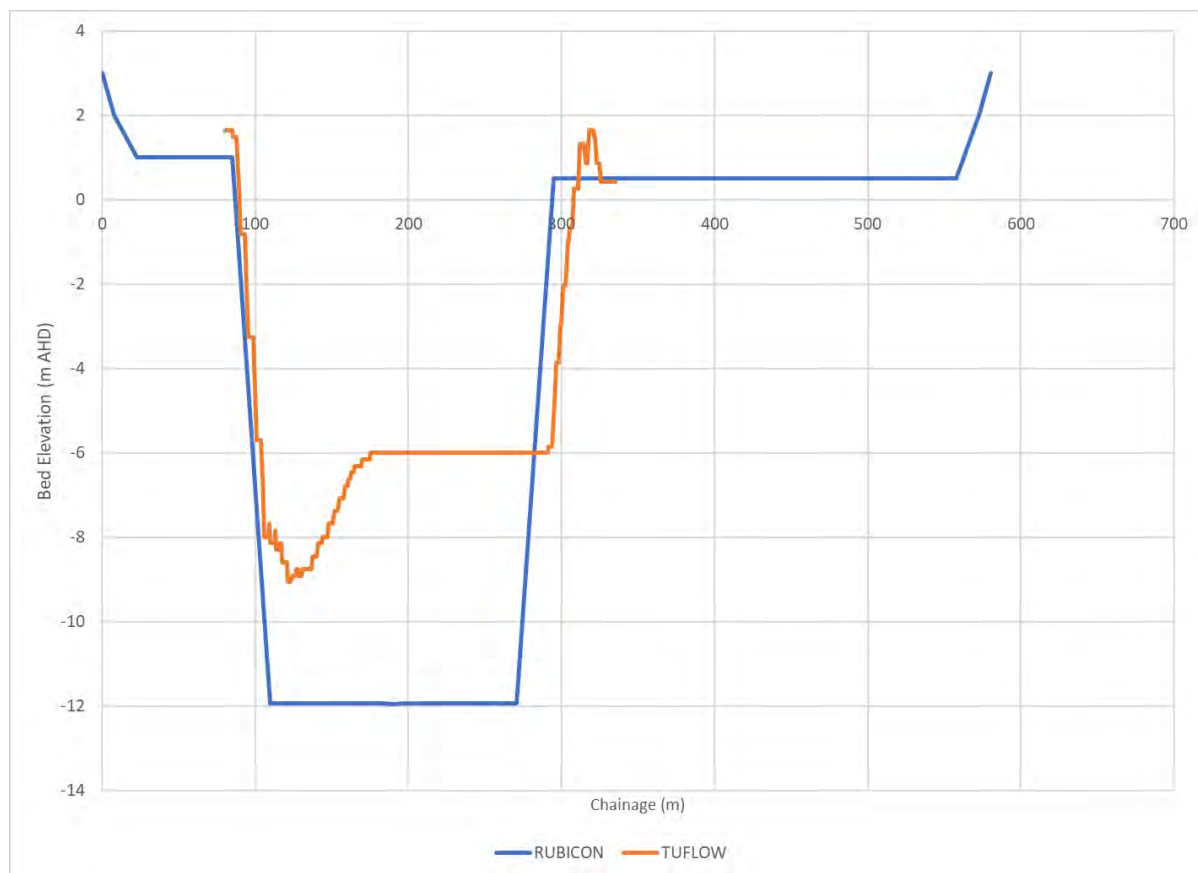


The design flood levels derived from this study and from the Kempsey CBD study are similar, with the variance in peak 1% AEP flood level typically +/- 0.1m. The similarity in flood levels is due to the same bathymetric and terrain data in addition to the same river inflow boundaries adopted in each study. Both studies also used TUFLOW modelling software. The minor differences are attributed to the finer 10m grid used in the Kempsey CBD modelling compared to the 20m parent grid in this study. Note that the Kempsey CBD modelling adopts design flood level hydrographs at its downstream boundary at Frederickton, derived from the RUBICON modelling. The peak flood level is 0.14m higher in the current study modelling in this location.

The modelled 1% AEP flood levels from the RUBICON model are also plotted. There is significant difference in the 1% AEP flood levels, with the current TUFLOW modelling producing peak flood levels about 1m higher than RUBICON in the vicinity of the Macleay River gauge at South West Rocks. The difference in modelled flood level reduces to 0.3m at Jerseyville, and diminishes with distance upstream until Smithtown and Belmore River. Data from the RUBICON model was reviewed and it was observed that the channel flows were similar in both models, hence this was ruled out as a likely cause for the differences.

Review of the model bathymetry at the Macleay River entrance indicated that the adopted channel cross-section in the RUBICON is significantly deeper than that in the current TUFLOW model, as shown on Figure 8-5. The RUBICON model channel has 60% greater flow area, which is likely to be the main cause for the lower flood levels from that modelling.

Figure 8-5 Comparison of Channel Profile near Macleay River Entrance – TUFLOW and RUBICON

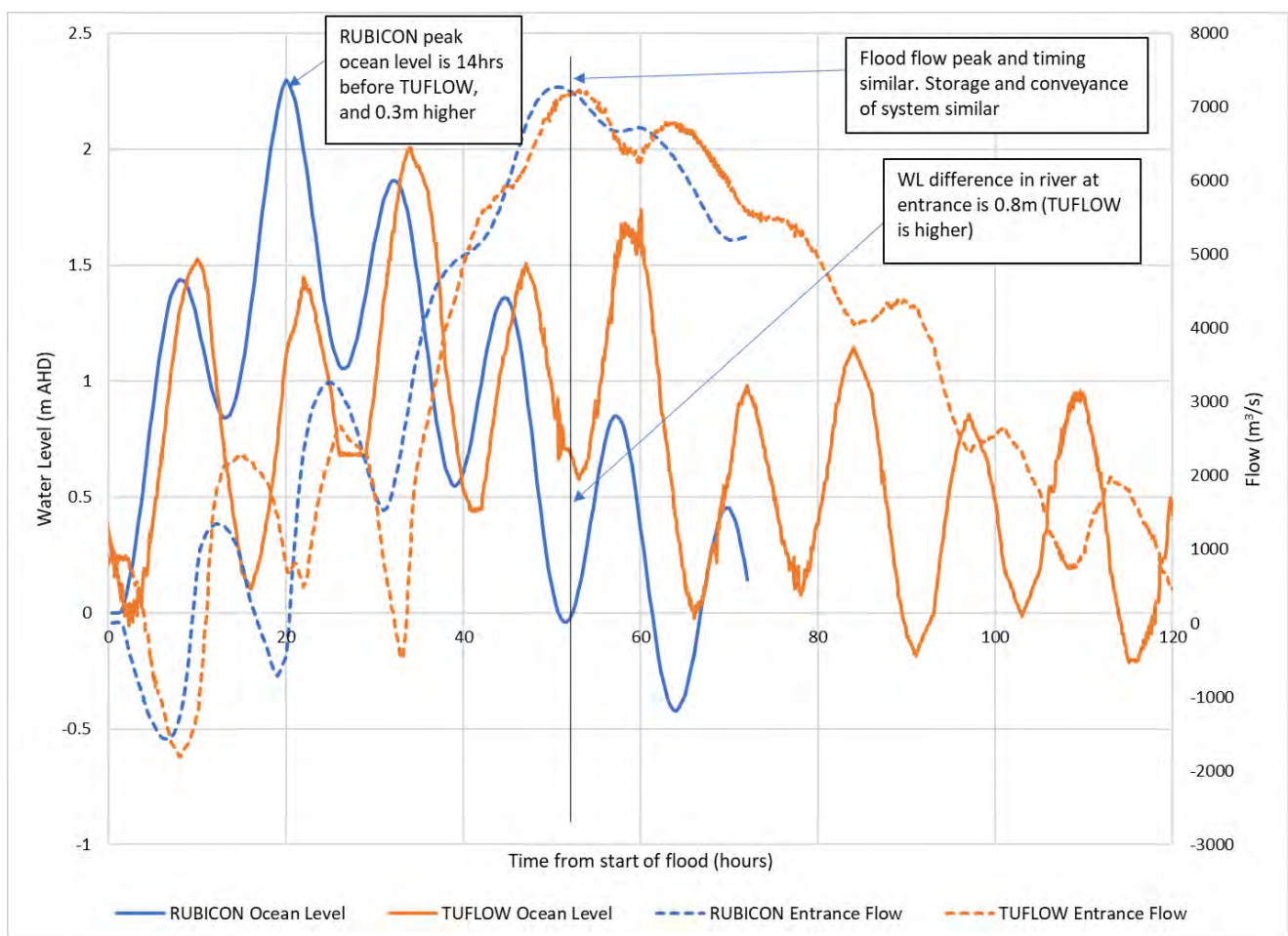


The current TUFLOW model bathymetry is based on hydrographic survey in addition to estimated depths of bed scour from detailed sediment transport modelling (refer to Section 7.1.7). The bed depth of the RUBICON model extends from inside the entrance into the ocean past the entrance bar. The previous model's developers, WMAwater (previously Webb McKeown and Associates) were consulted on the basis for this model assumption

and it may have been set at that depth to aid model numerical stability, however, the exact rationale was uncertain. It is noted that the maximum surveyed bed level is -11 to -12m AHD in the deep holes near the entrance, but it is unlikely that the bed would be uniformly scoured to the maximum hole depth during a flood event.

The adopted ocean tidal boundaries in each model were also reviewed, refer to Figure 8-6. Although the RUBICON ocean water levels are 0.3m higher at the peak, it is assumed to peak earlier than the TUFLOW model. By the time the flood discharge peaks at the entrance the RUBICON storm surge has receded and TUFLOW ocean levels are generally higher, by up to 0.8m, and the elevated ocean levels stay up for longer which restricts the flood discharge through the entrance. The difference in tidal boundaries is also likely to contribute to the difference in flood level estimates.

Figure 8-6 Comparison of TUFLOW and RUBICON ocean boundary levels



Referring to Figure 8-4, the modelled flood levels upstream of Belmore River to Seven Oaks are generally similar (± 0.2 m). There are differences of up to ± 0.6 m from Seven Oaks to Kempsey, due to the modelling approach in RUBICON including a coarser estimation of how the flood levels vary through the river bend at Seven Oaks in addition to how bank overflows and interaction with floodplain storages are modelled. There are similar variances in the modelled flood levels upstream of Kempsey.

Overall, the current TUFLOW modelling is considered reliable in the lower portion of the study area, downstream of Smithtown. Peak flood levels from the TUFLOW model are higher than previous estimates using the RUBICON model, and are influenced by the assumed river bed condition at the entrance. The adopted channel bed form in the TUFLOW model has been validated with detailed sediment transport analysis. Flood levels in the upper portion of the study area are similar and comparable to the design flood levels from the Kempsey CBD flood modelling which utilises a finer model grid resolution.

9. Sensitivity Testing

9.1 Overview

Sensitivity testing was undertaken in the TUFLOW model to assess the changes in flooding behaviour due to the following:

- Variation in model parameter values
- Removal of the works implemented as a part of the Lower Macleay flood mitigation scheme (refer to description in Section 2.6)
- Climate change impacts.

The testing and outcomes are discussed in the following sections.

9.2 Sensitivity to Model Parameter Values

A number of scenarios were assessed for the 1% AEP event to test the sensitivity of the model results to changes in the adopted parameter values. The sensitivity scenarios were assessed for the 1% AEP Macleay catchment flood, which is the dominant flood mechanism in the majority of the study area. The scenarios are described below:

- Increased hydraulic roughness: increase Manning's n values in channel and floodplain by 20%
- Decreased hydraulic roughness: decrease Manning's n values in channel and floodplain by 20%
- Low ocean water levels: neap tide cycle with no storm surge. Indian Spring Low Water coinciding with flood peak at river entrance. This scenario was assessed as a part of the design event runs, see Table 7-7
- Coinciding high ocean levels: the elevated ocean levels adopted for the 1% AEP Macleay flood event were timed to coincide with flood peak at the river entrance. Design runs assume peak ocean is at same time as peak flood at Kempsey. The sensitivity run lags the ocean boundary condition by 22 hours
- Breach of sand dunes in flood event: assumes the sand dune is breached at low point just north of the river entrance to a width of 300m. Assessed for PMF only, the 1% and 0.2% AEP events do not breach the dune low point. Breached modelled as time-varying and initiates when upstream flood level reaches the dune crest level.

Note that given the magnitude of the PMF event it is possible that breaching of the coastal dunes and erosion of multiple floodplain outlets to the ocean would occur, however there is low certainty on the locations and scale of these openings. The design flood estimate assumed no additional openings.

The change in flood levels are mapped on Figures G-1 to G-5 in Appendix G and the changes in flooding summarised in Table 9-1.

Table 9-1 Sensitivity Analysis Description and Results

Scenario Description	Change in Flood Level
Increase Manning's n by 20%	• Typical increase in 1% AEP flood level on floodplain 0.15 – 0.2m • Increases on south bank upstream of new Pacific Highway 0.3 – 0.5m • Upstream of Kempsey > +0.5m • North/downstream of Jerseyville increases of 0.05 – 0.1m.
Decrease Manning's n by 20%	• Typical decrease in 1% AEP flood level on floodplain 0.15 – 0.3m • decreases on south bank upstream of new Pacific Highway 0.3 – 0.5m • Upstream of Kempsey decreases of more than 0.5m • North/downstream of Jerseyville decreases of 0.05 – 0.1m. • Minor increase at entrance and in South West Rocks Creek of 0.04m due to higher flow conveyance upstream, conveying flows more efficiently to downstream sections.

Low ocean tide levels	• Reductions in flood levels of 0.9m in the channel at the entrance and at other minor ocean outlets as a direct result of the lower tide levels. • Reductions of 0.1 – 0.3m from entrance to Kinchela. • Reductions of less than 0.1m from Kinchela to Smithtown and on most of northern and southern floodplain. • No change upstream of Smithtown.
High ocean levels coinciding with flood peak at entrance	• Increases in flood levels of up to 0.4m in the channel at the entrance and at other minor ocean outlets as a direct result of the higher coinciding tide levels. increases in South West Rocks Creek of up to 0.3m. • Increase of up to 0.1m through Macleay Arm, through Fishermans Reach and Stuarts Point. • Reductions in peak flood levels in the river between Rainbow Reach and in the northern and southern floodplains of 0.05m. In the design flood the peak tide and storm surge occurs when the flood peaks at Kempsey. The timing of peak tide and storm surge is lagged by 22 hours in the sensitivity scenario. The earlier timing in the design flood estimation allows more inflow of ocean water with the tide and storm surge, resulting in higher water levels in the estuary at the time the flood wave reaches the middle part of the study area. This tidal inflow and resulting estuary water levels are reduced in the sensitivity scenario, hence the peak flood levels are lower. • No change in peak flood level upstream of Smithtown. • Increases in peak flood levels in Korogoro Creek and Killick Creek of 0.1 – 0.2m. • No change estimated in Connection Creek, as the effects of tide timing on Maria River flooding was not assessed.
Breach of sand dunes	• Localised reduction in PMF level of 0.5m around the breach location, including Fishermans Reach. • The reductions in flood levels drop with distance from the breach site. Decreases in PMF level of 0.3m are experienced at South West Rocks and Stuarts Point. • Typical reductions in the PMF level of 0.05 – 0.3m are present across most of the Lower Macleay floodplain. • No change upstream of Frederickton. • Further decreases in flood level may be expected with additional dune breaches during an actual flood event.

9.3 Removal of Flood Mitigation Scheme

The sensitivity of flood behaviour to the scenario where the Lower Macleay flood mitigation scheme was not implemented was assessed. Modelling of the scenario and results are discussed below.

9.3.1 Baseline Condition for Assessment

Firstly, a baseline flooding condition was established for this sensitivity assessment. Major floodgates at Euroka Creek, Christmas Creek, Belmore River (online floodgates), Kinchela Creek (online floodgates) and Clybucca Creek floodgates were represented, under Council advice that these structures would be locked shut by field staff at the first flood warning during a flood event. This differs from the design flood estimation whereby the floodgates are assumed functioning, preventing backflow through the structures but allowing outflow as the river flood levels drop below the floodplain flood levels.

The Belmore River and Kinchela Creek flood control structures and floodways were assumed to operate as per the design flood estimation (refer to Section 7.1.5).

Although the Rowe's Cut ocean outlet off Korogoro Creek has been filled in by the beach sand dunes under natural processes, the outlet cutting was represented in the baseline case as a part of modelling of the full mitigation scheme. Exact details of Rowe's Cut spillway are not available so it was assumed that the spillway would start at the base of the dune system next to Korogoro Creek (1.5m AHD) and end at the ocean waterline, with a channel width of 50m which is slightly wider than Korogoro Creek. Similar to the other minor ocean outlets Rowe's Cut was represented for the whole model simulation as fully-scoured due to flood flows.

Modelling of the baseline condition was conducted for the 0.2EY and 1% AEP (Macleay flood) and PMF (Macleay flood) events.

9.3.2 Flood Mitigation Works Removed

Modifications to the design flood model setup to represent the non-implementation of the flood mitigation scheme are summarised below:

- Constructed levees, as identified by Council's levee audit reports and spatial layers, were levelled off to the natural ground levels adjacent to the levees. This retains any natural banks along the floodplain.
- Major floodgates and headworks at Euroka Creek, Christmas Creek, Belmore River, Kinchela Creek and Clybucca Creek floodgates were omitted. The natural watercourse bed levels were represented across the headworks locations.
- Belmore River and Kinchela Creek flood control structures were omitted. Floodways at these locations were filled in.
- Constructed channels connecting Clybucca Creek and Andersons Inlet were filled in.
- The channels and cuttings connecting the Lower Macleay southern floodplain and Korogoro Creek, Killick Creek, Ryans Cut and Connection Creek/Maria River were filled. Cuttings through dunes were filled to the top of dune level.
- Rowe's Cut was retained at its current (filled in) condition.
- Floodplain agricultural drainage channels and the floodgates at their outlets were retained as existing. These features would have been constructed irrespective of the flood mitigation scheme.

Modelling of the sensitivity condition was conducted for the 0.2EY and 1% AEP (Macleay flood) and PMF (Macleay flood) events.

9.3.3 Results

The difference in flood levels between the removed scheme scenario and baseline scenario are shown on Figures G-6 to G-8 in Appendix G. The effects are described below for each flood event.

Changes to the duration of inundation on the floodplain could not be ascertained from the flood modelling. Major drains were represented in both the baseline and sensitivity scenarios as it was reasoned that these drains would be installed regardless of the flood mitigation scheme in order to improve agricultural lands. Further analysis could be undertaken subsequent to this study to estimate the changes in inundation times on the floodplain. This would require refinement of the modelling of major and minor agricultural drains and extending the model simulation time to the order of several weeks. The flooding assessment limited the simulated time to five days, which is sufficient to pass the flood wave through to the river outlet.

9.3.3.1 0.2 EY Event

Removal of the mitigation scheme in the 0.2EY event results in increases in flood levels on the northern floodplain of 0.1 – 0.35m, while there are reductions in flood levels of up to 0.1 – 0.4m on the southern floodplain to the west of Belmore River, and 0.03 – 0.1m to the east of Belmore River. This is due to the removal of the levee on the northern bank at Seven Oaks bend, which is the first location for floodwaters to break out of the river during a flood event. The levee is up to 1m high in some locations here. This reduces the flood levels in the river upstream of Seven Oaks, which in turn reduces the flood volume spilling into the southern floodplain between Austral Eden and Pola Creek.

The increased flows onto the northern floodplain work their way downstream via a more efficient route compared to flows in the southern floodplain, and result in increased flood levels in the lower reaches downstream of Jerseyville. Backwater flooding is increased up Macleay Arm by 0.15m. The increase in flooding around Jerseyville is around 0.3m, also affecting southern parts of South West Rocks. Increases in flood levels at the Macleay entrance are around 0.03m.

There are increases in flooding of up to 0.6m immediately upstream of the removed online floodgate on Belmore River, diminishing with distance upstream. There is no corresponding increase upstream of Kinchela floodgates. River flood levels are reduced by 0.25m at Smithtown and Kinchela. The reduced flooding on the southern floodplain results in less flows conveyed into Korogoro Creek, Killick Creek and Ryans Cut. Flood levels are reduced by 0.1 – 0.4m in Korogoro Creek at Hat Head, and by up to 1m behind the southern levee in Hat Head as floodwaters can drain through the floodgates through the levee more freely. There are minor reductions in flood levels in Crescent Head of up to 0.03m.

Overall, the flood levels are increased in Jerseyville, South West Rocks, Fishermans Reach and Stuarts Point. Flood levels are reduced in Smithtown, Gladstone, Frederickton, Kinchela, Hat Head and Crescent Head. The impacts of the removal of the flood mitigation scheme features is more pronounced for the 0.2EY event compared to the 1% AEP and PMF events.

9.3.3.2 1% AEP Event

Removal of the mitigation scheme in the 1% AEP event again results in increases in flood levels on the northern floodplain of 0.05 – 0.1m, with localised increases around Seven Oaks of 0.3m. There are also reductions in flood levels in the southern floodplain west of Belmore River of 0.03 – 0.2m and 0.3m in the river at Seven Oaks. The mechanism for these changes is similar to that described for the 0.2EY event.

Broadly, there appears to be a net increase in flooding with increases also on the southern floodplain around Kinchela Creek, which is in contrast to the reductions in this area in the 0.2EY event. This is attributed to the floodplain connection to Korogoro Creek, Killick Creek and Ryans Cut being filled in and hence reducing the outflow capacity from the floodplain.

As a result of the floodplain connections to the creeks being filled in, there are reductions in flood levels in parts of the creeks of up to 0.4m. The part of Crescent Head along Killick Creek receives reductions in flood levels of only 0.1m due to the higher influence of tide and ocean levels here. Note the western side of Crescent Head adjoins the southern floodplain and experiences a 0.1m increase.

Although Korogoro Creek through Hat Head experiences reductions in flood levels of 0.1 – 0.4m, the main part of the village itself would see flood levels increase by 0.08m as a result of removal of the levee.

Overall, the flood levels are increased in Jerseyville, South West Rocks, Kinchela, Fishermans Reach, Stuarts Point and the northern part of Hat Head. Flood levels are reduced in Smithtown, Gladstone, Frederickton, the southern part of Hat Head and Crescent Head.

9.3.3.3 PMF Event

Removal of the mitigation scheme PMF event again results in broad areas of flood level increases of 0.03 – 0.05m across most of the Lower Macleay. There are increases of typically 0.07m in Korogoro Creek including in Hat Head, with localised increases exceeding 0.4m at Rows Cut, as a result of removal of that outlet from baseline conditions. The nature of impacts in the PMF differ from the 0.2EY and 1% AEP, as the PMF is able to overtop the sand hills between the Lower Macleay floodplain and Korogoro Creek, while the cutting from the floodplain to Korogoro Creek is filled in the sensitivity scenario.

There are no changes in flood level in Killick Creek at Crescent Head, while there are reductions of 0.02m in the area connected to Ryans Cut. There are minor decreases in Christmas Creek of 0.02m due to removal of the levee and floodgates in that area.

9.4 Climate Change Assessment

The *Floodplain Development Manual* (NSW State Government, 2005) requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of climate change on flood behaviour. A sensitivity assessment was undertaken to estimate the potential impact of climate change on flooding in the Lower Macleay. Key elements of future climate change including sea level rise, increased storm rainfall intensity and resulting increased flood runoff, and their combined effects on flooding.

Previous guidance from the NSW Government recommended considering a 0.4m rise in sea level from current (1990) levels for the year 2050, and 0.9m rise for the year 2100 in assessing the impact of climate change on flooding. These projections are based on research by the Intergovernmental Panel on Climate Change (IPCC) and were refined for the Australian region.

Australian Rainfall and Runoff 2016 provides interim climate change factors for a range of climate change impact increase scenarios up to the year 2090. For the upper range RCP 8.5 scenario, in the year 2050 and the year 2090 an increase in rainfall depth and intensity of 9.0% and 19.7%, respectively, is predicted. Representative Concentration Pathways (RCP) 8.5 refers to the upper range projection of greenhouse gas concentrations in the atmosphere as adopted by the IPCC in 2014 for the assessment of climate change impacts.

The previous *Kempsey Hydraulic Model TUFLOW Update Report* (WMAwater, 2015) assessed climate change sensitivity including estimation of design flood hydrographs in the Macleay River at Kempsey assuming a 10%, 20% and 30% increase in 1% AEP storm rainfall over the upper Macleay Catchment. For the purposes of this study these updated flows were adopted for the climate change assessment and combined with the sea level rise projections for approximate flooding conditions in future years, including:

- Year 2050: Sea level rise of +0.4m and a 10% increase in rainfall/runoff
- Year 2100: Sea level rise of +0.9m and a 20% increase in rainfall /runoff.

The 1% AEP event was assessed including the separate flooding scenarios dominated by Macleay River, Maria River and ocean inundation dominated flooding, as per the design events. Macleay River inflows were adopted as per WMAwater (2015) climate change flows. Local runoff on the Lower Macleay sub-catchments was factored up by 10% and 20% for the year 2050 and 2100 scenarios, respectively. Adjusted Maria River flood level boundaries were inferred based on design flood levels and climate change assessment (sea level rise and increased rainfall intensity) undertaken for the Hastings River (Patterson Britton, 2006; Advisian, 2018). The results from the climate change flooding assessment for the Lower Macleay are mapped on Figures H-1 and H-2 in Appendix H, with the climate change flood profiles along the Macleay River plotted on Figure H-3 and compared to the design 1% AEP flood.

Flooding conditions in the 2050 scenario include increase of 0.5 – 0.8m upstream of Kempsey, while the 2100 scenario sees increases of over 1m. At Frederickton the increases are 0.2m in 2050 and 0.5m in 2100. These increases are due mainly to the increase in flood flows in the river.

Between Frederickton and Seven Oaks the change in flood levels drop as a result of the river channel hydraulics. Increases at Seven Oaks are 0.1m in 2050 and 0.2m in 2100. The constrained flow capacity of the river in this section results in the limited increase in flood levels. Consequently, the increased river flood flows spill out onto the floodplains, with the majority of the Lower Macleay floodplain experiencing flood level increases of 0.3 – 0.5m in 2050 and 0.7 – 1m in 2100.

In the lower reaches of the Macleay and up Macleay Arm and Yarrahapinni Wetlands flood levels increase by up to 0.5m in 2050 and up to 1.2m as a result of the increased flood flows in combination with sea level rise. The long section flood profiles indicate the effect of the increased outflows and ocean levels decreases with distance from the outlet to Seven Oaks.

At Hat Head and Crescent Head the flood levels are 0.4m higher in the 2050 scenario, while they are about 0.9m higher in Crescent Head and over 1m higher at Hat Head in the 2100 scenario.

9.5 Summary and Implications of the Sensitivity Assessment Findings

The effects of the sensitivity modelling on flooding conditions is discussed in more detail in the previous sections. Consideration should be made of these findings in the floodplain risk management for the Lower Macleay. In summary:

- Flood levels in the 1% AEP event vary typically by +/- 0.2m and up to +/- 0.5m from design flood levels by varying the hydraulic roughness of the river and floodplain by +/- 20%. This indicates the modelled flood

levels are fairly sensitive to global changes in the roughness values. While localised changes to land use on the floodplain are unlikely to result in large impacts to flood levels, widespread changes to land use, for example change from a pasture land cover to cropping land use, may have marked effects on the peak flood levels. Future changes in floodplain agriculture and vegetation cover may warrant revision of the design flood estimates.

- Flood levels across the floodplain reduce by -0.1m to -0.3m due to low tide conditions compared to the design ocean conditions. Separately, a coinciding high tide condition results in changes ranging from minor reductions up to increases in flood levels of up to +0.2m which affect developed areas near the river entrance. There are higher variances localised to the river outlets, which do not impact on existing developed areas. Minimal or no change upstream of Smithtown for both sensitivity conditions. These changes in flood levels do not significantly change the flood affectation of development, noting that the coinciding high tide condition is an unlikely occurrence. Hence it is not considered incorporate the coinciding high tide condition into the design flood estimate.
- Flood levels reduce by -0.3m in a fairly localised area due to a coastal dune breach in the PMF which forms an additional outlet for floodwaters. The PMF levels are relatively insensitive to this condition and it is considered prudent to assume non-breach of the coastal dunes in the PMF estimation.
- With no Lower Macleay flood mitigation scheme, in the 0.2EY event flood levels are increased by up to 0.4m on the northern floodplain and reduced by up to 0.4m in the southern floodplain. In the 1% AEP event there are up to 0.1m increases and up to 0.2m decreases on the northern and southern floodplains, respectively, with localised higher changes. In the PMF there are broadly 0.05m increases with localised increases up to 0.4m. These are the likely upper bound changes to flood conditions from any proposed future modifications to the form and operation of the flood mitigation scheme.
- In the climate change assessment, the 1% AEP flood levels broadly increase on the Lower Macleay floodplain by 0.3 – 0.5m in the 2050 scenario, and 0.7 – 1m in the year 2100 scenario. Increases of up to 0.8m in the year 2050 and up to 1.2m in the year 2100 scenarios occur at the main river outlet and minor outlets, in addition to upstream of Kempsey. These impacts are due to the combination of increased rainfall and flood flows in addition to sea level rise. Given the sensitivity of flooding conditions in the climate change scenarios, it would be prudent to consider incorporation of climate change impacts into planning controls for development of the Lower Macleay floodplain.

10. Limitations of this Study

The purpose of this flood study was to develop appropriate flood modelling for estimating flooding conditions and behaviour for design flood events in the Lower Macleay floodplain, particularly for larger flood events such as the 5% and 1% AEP events and rarer. Additional assessment is required outside the scope of this study to achieve a number of other floodplain management objectives of Council. Further, the design flood information from previous more detailed studies may supersede this study in specific areas of the floodplain. These are discussed in this section.

10.1 Design Flood Levels around Kempsey CBD

The focus of this study was to define the design flooding conditions in the broader Lower Macleay floodplain as well as in the villages and urban areas in the floodplain. While the modelling undertaken in this study extends upstream of and covers the Kempsey CBD area, this area has previously been analysed in greater detail in the *Kempsey CBD Floodplain Risk Management Study* (WMAwater, 2017b) using a 10m model resolution compared to the 20m resolution in this current study. Floodplain management actions have already been established and adopted based on the floodplain risk management study. It is considered appropriate to defer to the 2017 study for design flood levels and conditions in and around the Kempsey CBD, downstream to Frederickton. The area where the 2017 study should be referred to for design flood levels around Kempsey CBD is indicated on the flood mapping in Appendix F.

10.2 Design Flood Conditions outside Study Area

This study focussed on the assessment of flooding in the Lower Macleay valley. While parts of the Maria River and Connection Creek floodplain were represented in the TUFLOW model, inclusion of the broader Maria River floodplain and catchment, which is within the Hastings River catchment, was outside the scope of the study. Parts of the Maria River floodplain are in Kempsey LGA.

To assist Council with defining design flood levels in the Maria River floodplain, interpretation of the results from this study and from the *Hastings River Flood Study* (Patterson Britton and Partners, 2006) was undertaken. Refer to Appendix J for details.

10.3 Model Resolution and Minor Flows

The broad floodplain modelling was developed with a 20m grid which is an appropriate model resolution for analysis of major floods on a large floodplain such as the Lower Macleay. A number of minor watercourses of interest were incorporated in the 20m grid, including Belmore River and Kinchela Creek. These watercourses are relatively narrow and the conveyance of the channels may be underrated in the model particularly for low flow conditions during minor flood events more frequent than the 0.2EY event.

It is anticipated that the Lower Macleay TUFLOW model will be used in subsequent assessments to help refine operation of the Belmore River and Kinchela Creek flood control structures for management of frequent floods such as the 1 in 2 AEP event. This will require the watercourse channels to be represented in higher detail for more precise channel conveyance modelling, which could be achieved by modelling the channels as 1D features using cross-sectional data of the channels. Note that this approach may be suitable only for low-flow, frequent flood events. Significant effort was spent trying to configure the current model with 1D channels for the 2001 flood event model calibration but it was not possible to develop a model with satisfactory performance within the constraints of this project.

It is recommended that any refinement of the model and further detailed investigation of the performance of the key flood control structures be undertaken in the subsequent floodplain risk management study for the Lower Macleay Valley.

10.4 Probable Maximum Flood in Local Catchments

A primary focus on flooding on the broader Lower Macleay floodplain means that less focus was placed in the assessment of flooding in small local catchments for events rarer than the 1% AEP. These include minor watercourses crossing the Pacific Highway and North Coast Railway.

During the 0.2% AEP and PMF events, flooding on the main floodplain areas as well as the lower portions of these local catchments is dominated by flooding from the main river. Ascending upstream in these local catchments the critical flooding mechanism shifts to local catchment flood flows at some point. The PMF in the local catchment watercourses was not assessed in detail, as the maximum local catchment flood event that coincides with river flooding is the 1% AEP event. Hence, the flooding and potentially cutting of the highway and railway at these minor watercourses by extreme flood events was not assessed.

It is recommended that this be investigated in the subsequent floodplain risk management study in order to quantify possible disruption of the highway as a flood evacuation and access route and the railway as a transport link.

10.5 Local Catchment Flooding in Levee Protected Areas

Local catchment flooding in areas protected by levees was not a primary focus of this study. Local inflows from the Lower Macleay floodplain critical storm events (48 hour duration) were input for more sensitive locations including Frederickton and Hat Head. Assessment of additional, shorter storm durations should be undertaken to confirm peak local overland flooding conditions in these areas. It was agreed with Council that this should be deferred to a future local drainage study for Frederickton, for that location. A similar study is less critical for Hat Head, which is significantly affected by mainstream 1% AEP flooding which would dictate the flood planning levels for that village.

10.6 Static River Morphology during Flood Simulation

The Macleay River is a highly dynamic river system which may undergo geomorphic change including shift in bed and channel shape and alignment as a result of, and also during, flooding events. Sand shoals are likely to be eroded and deposited elsewhere, river meanders can become more sinuous or alternatively become cut-off by the river short-circuiting the meander, and new river outlets can be formed as demonstrated by the 1893 flood event.

While the flood modelling in this study does approximate the likely scouring of the waterway entrances during flood events for estimating flood behaviour, it assumes a static river and floodplain form in the rest of the river system during the simulation of flooding. This is an acceptable practice in flood studies given the uncertainty of how the river morphology may change during a flood. It should be noted that morphological changes may differ between different flood events.

10.7 Macleay River Design Flood Flows

It is recognised that the design flow estimates for the Macleay River upstream of Kempsey are based on design storm rainfall depths and hydrologic procedures from Australian Rainfall and Runoff 1987 (ARR 1987). The ARR 2019 revisions of this design data and procedures were finalised by BOM and Engineers Australia in June 2019, during the latter stages of this study and hence there was no opportunity to assess the Macleay River flows using ARR 2019.

ARR 2019 incorporates approximately 30 years of additional recorded rainfall data, in addition to more contemporary statistical techniques, in updating the design rainfall estimates in locations across Australia. In some locations the design rainfalls have increased while it has decreased in other locations. The charts in Figures B-3 to B-17 in Appendix B indicate how the design rainfall depths have changed from ARR 1987 to ARR 2019 (previously referred to as ARR 2016 in its draft form) across the Lower Macleay. ARR 2019 also updated the design rainfall temporal patterns and provided regional guidance for rainfall losses for estimating design flood flows.

The implications of this are that Macleay River flood flows may change, and hence design flood levels change, when the design flow estimates are updated to ARR 2019 in future studies. Note that while the Macleay River flows adopted in this study were derived based on ARR 1987, the local catchment flows onto the Lower Macleay floodplain were estimated using ARR 2019.

11. Conclusions and Recommendations

11.1 Conclusions

This flood study provides an expansive floodplain-wide assessment of flooding conditions according to the latest floodplain management guidance and policy. Numerous flooding studies and assessments have been conducted for the Lower Macleay River in the past, with the recent studies including the *Kempsey CBD Floodplain Risk Management Study* in 2017. The recent studies are fragmented in their study extents. The purpose of this study was to undertake investigation to assess mainstream flood behaviour in the broader Lower Macleay floodplain including villages and urban areas including Smithtown, Gladstone, Kinchela, Jerseyville, Fishermans Reach, Stuarts Point, South West Rocks, Hat Head and Crescent Head.

This flood study has undertaken hydraulic modelling in TUFLOW to estimate design flood behaviour in the Lower Macleay floodplain, downstream of Kempsey to the Macleay River entrance. The modelling accounts for the main flood flows from the Macleay River catchment upstream of Kempsey in addition to minor tributary inflows including Christmas Creek and Collombatti Creek and local rainfall-runoff on the floodplain. Hydrologic analysis was conducted to validate the river design flood flows at Kempsey and the design rainfall depths for the study area. Apart from the main river entrance the modelling also represents minor coast creek outlets which also drain floodwaters from the floodplain, in addition to possible interflows between the Macleay River floodplain and the adjacent Hastings River floodplain during flood events.

The TUFLOW model is based on a 20m parent grid of the broader floodplain, with finer 5m grids nested into the parent grid and covering each of the villages in the floodplain. Inflows in the Macleay River into the model for design events are adopted from previous and recent flood study undertaken for Kempsey CBD. Additional inflows in tributary catchments into the Lower Macleay and on local catchments within the floodplain were estimated with DRAINS/RAFTS hydrologic modelling. Inflows from the Maria River were represented using recorded water levels in the Maria River. The model was calibrated to the March 2001 flood event and verified against the February 2013 flood event.

Modelling assessment was undertaken of sediment transport processes and bed scouring at the Macleay River entrance during flood events. This assessment validated the degree of bed scouring inferred during the model calibration process, and confirmed bed scouring levels for the design flood events. Assessment of entrance conditions at the minor creek ocean outlets was also conducted for input into the TUFLOW modelling.

Design flood conditions for the 0.2EY, 5%, 1% and 0.2% AEP and PMF were estimated with consideration of the different flood mechanisms affecting the Lower Macleay, including Macleay River catchment flooding, Maria River/Hastings River catchment flooding and oceanic inundation. Flood depths, water levels, flow velocities and flood hazard were mapped which describe flooding conditions. Zoomed in mapping was prepared for each of the villages on the floodplain. Additional outputs including flood profiles, flood level hydrographs, mapping indicating the dominant flood mechanisms and mapping of the annualised risk of flooding across the floodplain were prepared.

In summary, the flood affectation of the villages are described as:

- Relatively low to moderate affectation – South West Rocks, Fishermans Reach, Stuarts Point, Frederickton and Crescent Head. A high portion of these towns and villages is elevated and flooding is generally limited to the edges of these settlements.
- High affectation – Hat Head (moderate to high) Jerseyville, Smithtown, Gladstone and Kinchela. These villages are mostly or fully inundated in 1% AEP mostly to high depths of flooding and are flood-affected in frequent events. There are significant flows through the villages.

Sensitivity assessment of modelling assumptions and parameters, removal of the flood mitigation scheme works from the study area and climate change impacts on flooding were also undertaken. Mapping and discussion of these impacts on flooding are presented. The analyses find that

- Flood levels in the 1% AEP event vary typically by +/- 0.2m and up to +/- 0.5m from varying the hydraulic roughness of the river and floodplain by 20%

- Flood levels across the floodplain vary typically by up to +/- 0.3m due to variation in ocean tide conditions in the 1% AEP and due to coastal dune breach to form an additional outlet in the PMF. There are higher variances localised to the river outlets. Minimal or no change upstream of Smithtown.
- With no Lower Macleay flood mitigation scheme, in the 0.2EY event flood levels are increased by up to 0.4m on the northern floodplain and reduced by up to 0.4m in the southern floodplain. In the 1% AEP event there are up to 0.1m increases and up to 0.2m decreases on the northern and southern floodplains, respectively, with localised higher changes. In the PMF there are broadly 0.05m increases with localised increases up to 0.4m.
- In the climate change assessment the 1% AEP flood levels broadly increase on the Lower Macleay floodplain by 0.3 – 0.5m in the 2050 scenario, and 0.7 – 1m in the year 2100 scenario. Increases of up to 0.8m in the year 2050 and up to 1.2m in the year 2100 scenarios occur at the main river outlet and minor outlets, in addition to upstream of Kempsey. These impacts are due to the combination of increased rainfall and flood flows in addition to sea level rise.

The overall objective of this flood study has been achieved, in developing appropriate flood modelling for estimating flooding conditions and behaviour for design flood events in the Lower Macleay floodplain, particularly for larger flood events such as the 5% and 1% AEP events and rarer.

Additional assessment is required outside the scope of this study to achieve a number of other floodplain management objectives of Council, including refined modelling of minor channel and watercourses for assessing drainage and flood control structure operation and estimation of PMF conditions in local watercourses crossing the Pacific Highway and North Coast Railway. Further, the design flood information from previous more detailed studies supersede this study in specific areas of the floodplain, specifically around Kempsey CBD.

Parts of Kempsey LGA fall in the Maria River floodplain outside of the Lower Macleay flood study and were excluded from the modelling in this study. Design flood levels for the Maria River floodplain were inferred from available information and estimated flood extents mapped for the 5% and 1% AEP and PMF events. These estimates should be refined with future more detailed assessment.

11.2 Recommendations

A number of recommendations are made for further assessment in subsequent studies and data collection to help enhance the knowledge on flood behaviour in the Lower Macleay and surrounds:

- 1) Further detailed modelling is recommended to better define design flood conditions in the Maria River floodplain area, between the Lower Macleay model southern boundary and the Kempsey local government area boundary with Port Macquarie Hastings local government area, in the vicinity of Point Plomer. While an estimate of the design flood levels and extents has been prepared based on available data (refer Appendix J) the future modelling assessment should provide more detailed information on flood behaviour, flood hazard etc. It is recommended that this further modelling be undertaken in the subsequent Lower Macleay Floodplain Risk Management Study (FRMS) which could include extension of the hydrologic and hydraulic modelling to cover the Maria River. As a part of the future modelling updates, flooding scenarios such as a rare flood event in the Maria River (e.g. 1% AEP) coinciding with a frequent flood event in the Macleay River (e.g. 0.2EY), and vice versa, should be analysed to maximise the flood gradient in this area to determine the sensitivity of flow velocities and flood hazard.
- 2) Hydrologic modelling enhancement and updates is recommended including update with ARR 2019 to align with the latest design rainfall and hydrologic guidance. This may result in revision of design flood flows in the Macleay River. It is recommended that this is undertaken in the subsequent Lower Macleay FRMS.
- 3) Further refinement of the TUFLOW hydraulic model developed in this study is recommended in the subsequent Lower Macleay FRMS, including:
 - a) Refinement of representation of smaller-scale watercourses in the study area, including Belmore River and Kinchela Creek, and including representation of these as 1D channel features. This would permit more reliable representation of channel conveyance for the purpose of flood mitigation options assessment in minor flood events smaller than the 0.2EY event as assessed in this study.

- b) Assessment of very rare to extreme (0.2% AEP, PMF) flood events in local catchments. In particular this will assist with assessment of probable maximum flooding in minor watercourses and flood immunity of roads (Pacific Highway, etc.) for emergency management purposes.
 - c) Additional general model sensitivity testing should be considered in the subsequent FRMS including sensitivity of the hydrologic modelling to ARR 2019 parameters (design rainfalls, rainfall losses, areal reduction factors, hydrologic modelling procedures) and catchment routing parameter (Bx).
- 4) Further assessment of local catchment flooding is recommended in particular for Frederickton, where shorter duration storm events should be assessed to determine peak overland flooding conditions in Frederickton Drain. It is recommended that this be undertaken in a future local drainage study, as proposed by Council.
 - 5) Development of a Strategic Water Level Gauging Network Plan that provides future consideration for locations to record data in future flood events to improve model calibration in the future. Locations for monitoring include those areas on the floodplain, such as Clybucca Wetlands, Upper Belmore and Upper Kinchela Floodplains, and on tributary watercourses such as at Belmore and Kinchela flood control structures, which are not currently monitored. Consideration should also be made for water level monitoring of ocean levels locally so there is less reliance on tidal monitoring at Coffs Harbour. While an oceanway (i.e. at the Macleay entrance) site may not be suitable for ocean level monitoring, suitable sites in Trial Bay, to which the Macleay discharges, may be available.
 - 6) Due to the influence that sedimentation and siltation has on flood conditions and the performance of the current mitigation scheme, updated detailed bathymetry survey is recommended for all major waterways in the Lower Macleay upstream to Greenhill. Additionally, since the oceanway acts as a hydraulic control and upstream water levels in the estuary are sensitive to actual scour depths, it is recommended that there is annual survey of the entrance bed and other locations identified as hydraulic controls or particularly dynamic bedforms. Survey should be undertaken in dry conditions and immediately after a significant flood event.
 - 7) Consideration should be made for future update of the flood model and design flood estimates following substantial changes to floodplain conditions, including development, land use and river morphology.
 - 8) Consideration should be made in the subsequent FRMS for incorporation of climate change impacts to flooding into Council's planning and development controls, given the projected significant changes to flooding conditions as a result of climate change.
 - 9) Council and the flood risk management committee will be consulted on this draft flood study report. It is recommended that the final draft report subsequently be placed on public exhibition to consult the community and stakeholders on the findings of this study.

12. Acknowledgements

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- Port Macquarie Hastings Council officers;
- Roads and Maritime Services and their consultants WMAwater;
- UNSW Water Research Laboratory;
- Manly Hydraulics Laboratory;
- NSW State Emergency Service;
- Bureau of Meteorology; and
- NSW Department of Planning, Industry and Environment.

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WMAwater (2017b) Kempsey CBD Floodplain Risk Management Study.

14. Glossary

Annual Exceedance Probability (AEP)

The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. In this study AEP has been used consistently to define the probability of occurrence of flooding. It is to be noted that design rainfalls used in the estimation of design floods up to and including 100 year ARI (ie. 1% AEP) events was derived from 1987 Australian Rainfall and Runoff. Hence the flowing relationship between AEP and ARI applies to this study.

20% AEP = 5 year ARI; 10% AEP = 10 year ARI; 5% AEP = 20 year ARI; 2% AEP = 50 year ARI; 1% AEP = 100 year ARI.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.50
	1	63.2	1.58	1.00
Frequent	0.69	50.00	2	1.44
	0.5	39.35	2.54	2.00
	0.22	20.00	5	4.48
	0.2	18.13	5.52	5.00
	0.11	10.00	10.00	9.49
Infrequent	0.05	5.00	20	20.0
	0.02	2.00	50	50.0
	0.01	1.00	100	100
	0.005	0.50	200	200
Rare	0.002	0.20	500	500
	0.001	0.10	1000	1000
	0.0005	0.05	2000	2000
	0.0002	0.02	5000	5000
Extremely Rare				
Extreme			PMP	

Australian Height Datum (AHD)

A common national surface level datum approximately corresponding to mean sea level.

Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long-term average number of years between the occurrences of a flood as big as or larger than the selected event. For example, floods with a discharge as great as or greater than the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
Catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
Development	Is defined in Part 4 of the EP&A Act <u>In fill development</u>: refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. New development: refers to development of a completely different nature to that associated with the former land use. Eg. The urban subdivision of an area previously used for rural purposes. New developments involve re-zoning and typically require major extensions of exiting urban services, such as roads, water supply, sewerage and electric power. Redevelopment: refers to rebuilding in an area. Eg. As urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either re-zoning or major extensions to urban services.
Effective Warning Time	The time available after receiving advise of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
Flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
Flood liable land	Is synonymous with flood prone land (i.e.) land susceptibility to flooding by the PMF event. Note that the term flooding liable land covers the whole floodplain, not just that part below the FPL (see flood planning area)
Floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is flood prone land.

Floodplain risk management options	The measures that might be feasible for the management of particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
Floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually include both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defines objectives.
Flood Emergency Sub Plan	May also be known as (local) flood plan. A sub-plan of a disaster plan that deals specifically with flooding. They can exist at state, zone and local levels. Local flood plans are prepared under the leadership of the SES.
Flood planning levels (FPLs)	Are the combination of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the "designated flood" or the "flood standard" used in earlier studies.
Flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings and structures subject to flooding, to reduce or eliminate flood damages.
Flood readiness	Readiness is an ability to react within the effective warning time.
Flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. <u>Existing flood risk:</u> the risk a community is exposed to as a result of its location on the floodplain. <u>Future flood risk:</u> the risk a community may be exposed to as a result of new development on the floodplain. <u>Continuing flood risk:</u> the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
Flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas
Floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.

Freeboard	Provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
Hazard	A source of potential harm or situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community.
Local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
m AHD	Metres Australian Height Datum (AHD)
m/s	Metres per second. Unit used to describe the velocity of floodwaters.
m ³ /s	Cubic metres per second or "cumecs". A unit of measurement of creek or river flows or discharges. It is the rate of flow of water measured in terms of volume per unit time.
Mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
Modification measures	Measures that modify either the flood, the property or the response to flooding.
Overland flow path	The path that floodwaters can follow as they are conveyed towards the main flow channel or if they leave the confines of the main flow channel. Overland flow paths can occur through private property or along roads.
Probable Maximum Flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall which actually ends up as a streamflow, also known as rainfall excess.
Stage	Equivalent to water level (both measured with reference to a specified datum)

TUFLOW

TUFLOW is a computer program which is used to simulate free-surface flow for flood and tidal wave propagation. It provides coupled 1D and 2D hydraulic solutions using a powerful and robust computation. The engine has seamless interfacing with GIS and is widely used across Australia.