A part of BMT in Energy and Environment

Tabourie Lake Flood Study

Final Report

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Shoalhaven City Council

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Title :	Tabourie Lake Flood Study – Final Report
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EXECUTIVE SUMMARY

Introduction

The Tabourie Lake Flood Study has been prepared for Shoalhaven City Council (Council) to define the existing flood behaviour in the Tabourie Lake catchment and establish the basis for subsequent floodplain management activities.

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

- Compilation and review of existing information pertinent to the study and acquisition of additional data including survey as required;
- A community consultation and participation program to identify local flooding concerns, collect information on historical flood behaviour and engage the community in the on-going floodplain management process;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design events including the 20% AEP, 5% AEP, 2% AEP, 1% AEP and extreme flood event; and
- Presentation of study methodology, results and findings in a comprehensive report incorporating detailed flood mapping.

Catchment Description

The Tabourie Lake catchment encompasses an area of approximately 47km^2 located on the New South Wales South Coast. The main ‘Broadwater’ of the Lake is located in the east of the catchment, from which Tabourie Creek flows in a southerly direction to the entrance, located in the south-east of the catchment.

Tabourie Lake Broadwater is fed primarily by Lucy Kings Creek and Munno Creek and forms the flow source of Tabourie Creek. The major tributary of Branderee Creek merges with Tabourie Creek upstream of the Princes Highway Bridge, almost doubling the catchment area from 21km^2 to 40km^2. Lemon Tree Creek completes the major catchment contributions, joining Tabourie Creek toward the entrance.

Land use within the catchment primarily consists of forested areas (90%). Other land uses include pastureland (5%), open water (3%) and urban development (2%). The floodplain area of Tabourie Creek is largely developed, occupied by Tabourie Lake village, whereas that of Branderee Creek principally remains undeveloped and largely occupied by rural farming.

The catchment is traversed by one principal local transport route, the Princes Highway. The highway is embanked for a length of around 550m across the floodplain. This section will be overtopped from Tabourie Creek and the Branderee Creek floodplain during high order flood events.

Historical Flooding

The majority of significant flooding in Tabourie Lake has coincided with a build up of the entrance berm height and/or rainfall events occurring with high antecedent water levels in the lake. Major historical flood events to have occurred in Tabourie Lake include 1971, 1975, 1988 and 1991.

There is limited historical flood level data available for Tabourie Lake. Water levels in the Lake have been continuously recorded since September 1992 at the MHL water level gauge situated within Tabourie Creek. However, there have been no major flooding events during this period, with the highest lake level recorded at 1.53m AHD in September 1996. There is no comprehensive record of water levels in Tabourie Lake prior to operation of the continuous water level gauge in 1992.

The highest known lake flood level is reportedly 2.5m AHD in February 1971. Flood levels for the 1975 and 1988 events have been obtained from a survey of historic flood marks identified through the community consultation undertaken for the flood study.

Community Consultation

Community consultation undertaken during the study has aimed to collect information on historical flooding and previous flood experience, and inform the community about the development of the flood study and its likely outcome as a precursor to floodplain management activities to follow. The key elements of the consultation process have included distribution of a questionnaire relating to historical flooding, and a community information session held at the Tabourie Lake Rural Fire Service Depot in Beach Street on Tuesday 19th February, 2008.

A total of 17 individuals added their names to the attendance register, although it is estimated that around 10 more individuals also attended without signing the register. The level of attendance was striking for a community of this size.

Model Development (and additional survey)

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

The floodplain topography is defined using a digital elevation model (DEM) derived from topographic, hydrographic and photogrammetric data provided by Council. To supplement the available data, additional cross section survey of the Branderee Creek and Lemon Tree Creek channels and significant hydraulic structures was acquired during the course of the study.

With consideration to the available survey information and local topographical and hydraulic controls, a hydraulic model was developed extending from the Tabourie Creek entrance at the downstream limit, upstream along the major tributary routes. The floodplain area modelled within the 2D domain comprises a total area of some 7km^2 which represents the lower 15% of the entire Tabourie Lake catchment.

Model Calibration and Validation

The selection of suitable historical events for calibration of the computer models is largely dependent on available historical flood information. Within the Tabourie Lake catchment however, there is a distinct lack of historical flood data.

The calibration process for the Tabourie Lake catchment is somewhat complicated by the significant influence of the entrance condition. This influence emerges for events occurring both during closed and opened periods. Further to the uncertainties in regard to the entrance condition, the catchment is ungauged with respect to flow rates. Accordingly, there is no recorded flow data suitable to calibrate the hydrological model for historical events.

An alternative approach to calibration of the catchment hydrology is considered by deriving flow rates from recorded water levels in Tabourie Creek. In considering the best approach to model calibration and validation using the available data, the following analyses have been undertaken:

- Volumetric analysis of catchment inputs to Tabourie Lake for significant rainfall events during closed entrance periods between 1992 and 2008;
- Calibration of hydrological model to derived volumetric catchment inputs for "closed-entrance" events; and
- Simulation of major historical events (March 1975 and April 1988) with sensitivity testing of assumed conditions such as entrance berm configuration and initial lake water levels.

In absence of detailed data for other historical events, conventional model validation cannot be undertaken. However, the March 1975 and April 1988 events have been simulated for comparison. Given the lack of available data it is difficult to confirm whether the model simulations are an accurate representation of the actual flood behaviour for these events. Nevertheless, the calibration simulation and sensitivity tests have enabled an understanding of the catchment flood behaviour to be realised, in particular the key mechanisms driving peak flood conditions in the Lake.

Design Event Modelling and Output

The developed models have been applied to derive design flood conditions within the Tabourie Lake catchment. Design rainfall depth is based on the generation of intensity-frequency-duration (IFD) design rainfall curves utilising the procedures outlined in AR&R (2001). A range of storm durations using standard AR&R (2001) temporal patterns, were modelled in order to identify the critical storm duration for design event flooding in the catchment.

A suite of design event scenarios was defined that is most suitable for future floodplain management planning in Tabourie Lake. Consideration was given to flood events driven by both catchment and ocean processes. The potential impact of climate change on flood behaviour within Tabourie Lake has also been considered. The catchment derived events were found to be the critical events in terms of determining maximum flood levels.

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

Provisional flood hazard categorisation in accordance with Figure L2 of the NSW Floodplain Development Manual (2005) has been mapped for the 20% AEP, 1% AEP and the PMF events, in addition to the hydraulic categories (floodway, flood fringe and flood storage) for flood affected area for the 20% AEP, 1% AEP and the PMF events also.

(mapping attached to Executive Summary for Committee reference) – The simulated flood inundation extents in the Tabourie Lake catchment for the 5% AEP, 1% AEP and PMF events are shown in Figure 8-3. The entrance berm forms a significant control which effectively drives flood levels in Tabourie Lake village. Locations within the Tabourie Lake floodplain identified as being most at risk include properties along the Princes Highway, the area around the Tabourie and Lemon Tree Creeks confluence and the Tabourie Lake Holiday Park.

The flood mapping series for the 1% AEP design event is attached for reference. This includes detailed floodplain mapping of peak design flood level and depth (refer Figure A-7), peak flood velocity (refer Figure A-8), hydraulic category (refer Figure A-13) and provisional hazard category (refer Figure A-14).

The impact of future climate change will likely see an increase in the height of the entrance berm. This will result in a similar increase in flood levels within Tabourie Lake village.

Sensitivity Testing

A series of sensitivity tests have been undertaken on the modelled flood behaviour of the Tabourie Lake catchment. The tests provide a basis for determining the relative accuracy of modelling results, and an initial focus for future floodplain management planning. The tests undertaken include:

- Initial lake level – given the volumes of catchment runoff generated in flood events relative to the available storage of the lake system, the peak flood level was found to be relatively insensitive to initial lake water levels;
- Berm height – the berm height has been demonstrated to be the principal control on design flood water levels in Tabourie Lake. Peak flood level sensitivity to other model parameter or assumptions are generally minor in comparison to the relative impact of berm height;
- Entrance scour – for the catchment derived flood events, both a fixed berm and mobile berm condition were simulated, with the fixed berm representing a conservative condition. The initiation and propagation of a natural breakout is a very complex process. The simulated entrance breakout provides a representative effect of the development of a scour channel, showing a moderate reduction in peak flood levels compared to the fixed berm;
- Bed Roughness model parameter - current related bed roughness is a coefficient in the morphological model used in the sand transport equations associated with scour of the entrance. The parameter, which impacts on the degree of scour, was found to only have a minor impact on design peak flood level condition; and
- Structure blockages – structure blockage due to flood debris can result in significant increases to flood levels and redistributions of flood flows. Blockage scenarios of 25% blockage from the bottom up have been simulated for major structures on the main channel alignments.

The sensitivity tests generally provide only modest changes to simulated design flood conditions. Significantly, increases in simulated inundation extents considering sensitivity tests do not result in any significant increase in flood affectation for existing property. However, for future development in the catchment, increases in design flood conditions over and above the adopted design flood standards should consider the potential impacts of these scenarios.

Conclusions

The berm height is the key factor in the design flood level estimation for catchment derived flood events in Tabourie Lake. The development of the entrance berm is a dynamic process driven by both catchment hydrology and prevailing tidal and wave conditions. While typical berm elevations are around 2.0 m AHD, Shoalhaven City Council's present management strategy is to excavate a channel through the barrier when water levels reach a level of 1.17, primarily to relieve flooding of low lying properties fringing the Lake.

The results of the flood modelling undertaken clearly demonstrate the major influence of the entrance berm condition on peak flood levels for catchment derived flooding events. At present, manual opening of the entrance is undertaken to control water levels in Tabourie Lake. However, from a design flood perspective, where warning times may be limited to a few hours, a manual breakout may not be possible. As such design flood level estimation has been based on the assumption of the berm being intact at the onset of a major design rainfall event.

The storage volume in the Lake system is relatively minor in comparison to the flood volumes generated from catchment runoff in major events. Accordingly, the peak design flood levels are relatively insensitive to the initial lake condition. This is significant from a floodplain management perspective in that Lake levels would be expected to rise relatively quickly, thereby limiting opportunity to undertake a manual breakout of the entrance.

Ocean derived flood events, such as the major event that occurred in 1974 in Tabourie Lake, pose significant risk to Tabourie Lake village. Extreme ocean conditions are likely to scour the entrance or indeed overtop the entrance berm providing for unrestricted penetration of elevated ocean levels into the estuary. The design ocean flood conditions pose a similar risk in terms of peak flood inundation to Tabourie Lake village as the catchment derived floods.

Potential climate change scenarios have a major influence on design flood conditions at Tabourie Lake. The potential for sea level rise, which in turn provides for increases in berm heights, and increase in design rainfall intensities have been assessed in the flood study for the 2050 and 2100 planning horizons. The future climate change scenarios provide for substantial increases in design peak flood levels above existing conditions. An interesting observation of the design flood results are the relatively similar peak flood level estimations in major events for both the catchment derived flooding (fixed berm conditions) and the ocean derived flood levels. Therefore design flood planning levels in Tabourie Lake are expected to be of similar magnitude irrespective of entrance management and long term berm geometries, given the similar flood risk associated with ocean derived flooding.

The opportunity for ongoing entrance management in line with existing policy may be limited if sea level rise projections manifest. The current management level of 1.17m AHD is approximately equal to the projected 2100 mean water level in Tabourie (i.e. current mean sea level of 0.25m AHD +0.91m AHD sea level rise).

The flood study will form the basis for the subsequent floodplain risk management activities, being the next stage of the floodplain management process. The flood planning process will undoubtedly have strong links with entrance and estuary management.

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GLOSSARY

annual exceedance probability (AEP)	The chance of a flood of a given size (or larger) occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (i.e. a 1 in 20 chance) of a peak discharge of 500 m^3/s (or larger) occurring in any one year. (see also average recurrence interval)
Australian Height Datum (AHD)	National survey datum corresponding approximately to mean sea level.
Astronomical Tide	Astronomical Tide is the cyclic rising and falling of the Earth's oceans water levels resulting from gravitational forces of the Moon and the Sun acting on the Earth.
attenuation	Weakening in force or intensity
average recurrence interval (ARI)	The long-term average number of years between the occurrence 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 20yr ARI design flood will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event. (see also annual exceedance probability)
catchment	The catchment at a particular point is the area of land that drains to that point.
design flood	A hypothetical flood representing a specific likelihood of occurrence (for example the 100yr ARI or 1% AEP flood).
development	Existing or proposed works that may or may not impact upon flooding. Typical works are filling of land, and the construction of roads, floodways and buildings.
discharge	The rate of flow of water measured in tems of vollume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
flood	Relatively high river or creek flows, which overtop the natural or artificial banks, and inundate floodplains and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
flood behaviour	The pattern / characteristics / nature of a flood.
flood fringe	Land that may be affected by flooding but is not designated as floodway or flood storage.
flood hazard	The potential risk to life and limb and potential damage to property resulting from flooding. The degree of flood hazard varies with circumstances across the full range of floods.
flood level	The height or elevation of floodwaters relative to a datum (typically the Australian Height Datum). Also referred to as "stage".

flood liable land	see flood prone land
floodplain	Land adjacent to a river or creek that is periodically inundated due to floods. The floodplain includes all land that is susceptible to inundation by the probable maximum flood (PMF) event.
floodplain management	The co-ordinated management of activities that occur on the floodplain.
floodplain risk management plan	A document outlining a range of actions aimed at improving floodplain management. The plan is the principal means of managing the risks associated with the use of the floodplain. A floodplain risk management plan needs to be developed in accordance with the principles and guidelines contained in the NSW Floodplain Management Manual. The plan usually contains both written and diagrammatic information describing how particular areas of the floodplain are to be used and managed to achieve defined objectives.
Flood planning levels (FPL)	Flood planning levels selected for planning purposes are derived from a combination of the adopted flood level plus freeboard, as determined in floodplain management studies and incorporated in floodplain risk management plans. Selection should be based on an understanding of the full range of flood behaviour and the associated flood risk. It should also take into account the social, economic and ecological consequences associated with floods of different severities. Different FPLs may be appropriate for different categories of landuse and for different flood plans. The concept of FPLs supersedes the “standard flood event”. As FPLs do not necessarily extend to the limits of flood prone land, floodplain risk management plans may apply to flood prone land beyond that defined by the FPLs.
flood prone land	Land susceptible to inundation by the probable maximum flood (PMF) event. Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Risk Management Plans should encompass all flood prone land (i.e. the entire floodplain).
flood source	The source of the floodwaters. In this study, Burrill Lake is the primary source of floodwaters.
flood storage	Floodplain area that is important for the temporary storage of floodwaters during a flood.
floodway	A flow path (sometimes artificial) that carries significant volumes of floodwaters during a flood.
freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determing the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
geomorphology	The study of the origin, characteristics and development of land forms.
gauging (tidal and flood)	Measurement of flows and water levels during tides or flood events.

historical flood	A flood that has actually occurred.
hydraulic	The term given to the study of water flow in rivers, estuaries and coastal systems.
hydrodynamic	Pertaining to the movement of water
hydrograph	A graph showing how a river or creek's discharge changes with time.
hydrographic survey	Survey of the bed levels of a waterway.
hydrologic	Pertaining to rainfall-runoff processes in catchments
hydrology	The term given to the study of the rainfall-runoff process in catchments.
isohyet	Equal rainfall contour
morphological	Pertaining to geomorphology
peak flood level, flow or velocity	The maximum flood level, flow or velocity that occurs during a flood event.
pluviometer	A rainfall gauge capable of continously measuring rainfall intensity
probable maximum flood (PMF)	An extreme flood deemed to be the maximum flood likely to occur.
probability	A statistical measure of the likely frequency or occurrence of flooding.
riparian	The interface between land and waterway. Literally means "along the river margins"
runoff	The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek.
stage	See flood level.
stage hydrograph	A graph of water level over time.
sub-critical	Refers to flow in a channel that is relatively slow and deep
topography	The shape of the surface features of land
velocity	The speed at which the floodwaters are moving. A flood velocity predicted by a 2D computer flood model is quoted as the depth averaged velocity, i.e. the average velocity throughout the depth of the water column. A flood velocity predicted by a 1D or quasi-2D computer flood model is quoted as the depth and width averaged velocity, i.e. the average velocity across the whole river or creek section.
water level	See flood level.

1 INTRODUCTION

The Tabourie Lake Flood Study has been prepared for Shoalhaven City Council (Council) to define the existing flood behaviour in the Tabourie Lake catchment and establish the basis for subsequent floodplain management activities.

This project has been conducted under the State Assisted Floodplain Management Program and received State financial support.

1.1 Study Location

The Tabourie Lake catchment encompasses an area of approximately 47km^2 located on the New South Wales South Coast as shown in Figure 1-1. The main 'Broadwater' of the Lake is located in the east of the catchment, from which Tabourie Creek flows in a southerly direction to the entrance, located in the south-east of the catchment.

Tabourie Lake Broadwater is fed primarily by Lucy Kings Creek and Munno Creek, forming the principal flow source of Tabourie Creek. The major tributary of Branderee Creek merges with Tabourie Creek upstream of the Princes Highway Bridge, almost doubling the catchment area from 21km^2 to 40km^2. Lemon Tree Creek (or Saltwater Creek) completes the major catchment contributions, joining Tabourie Creek toward the entrance.

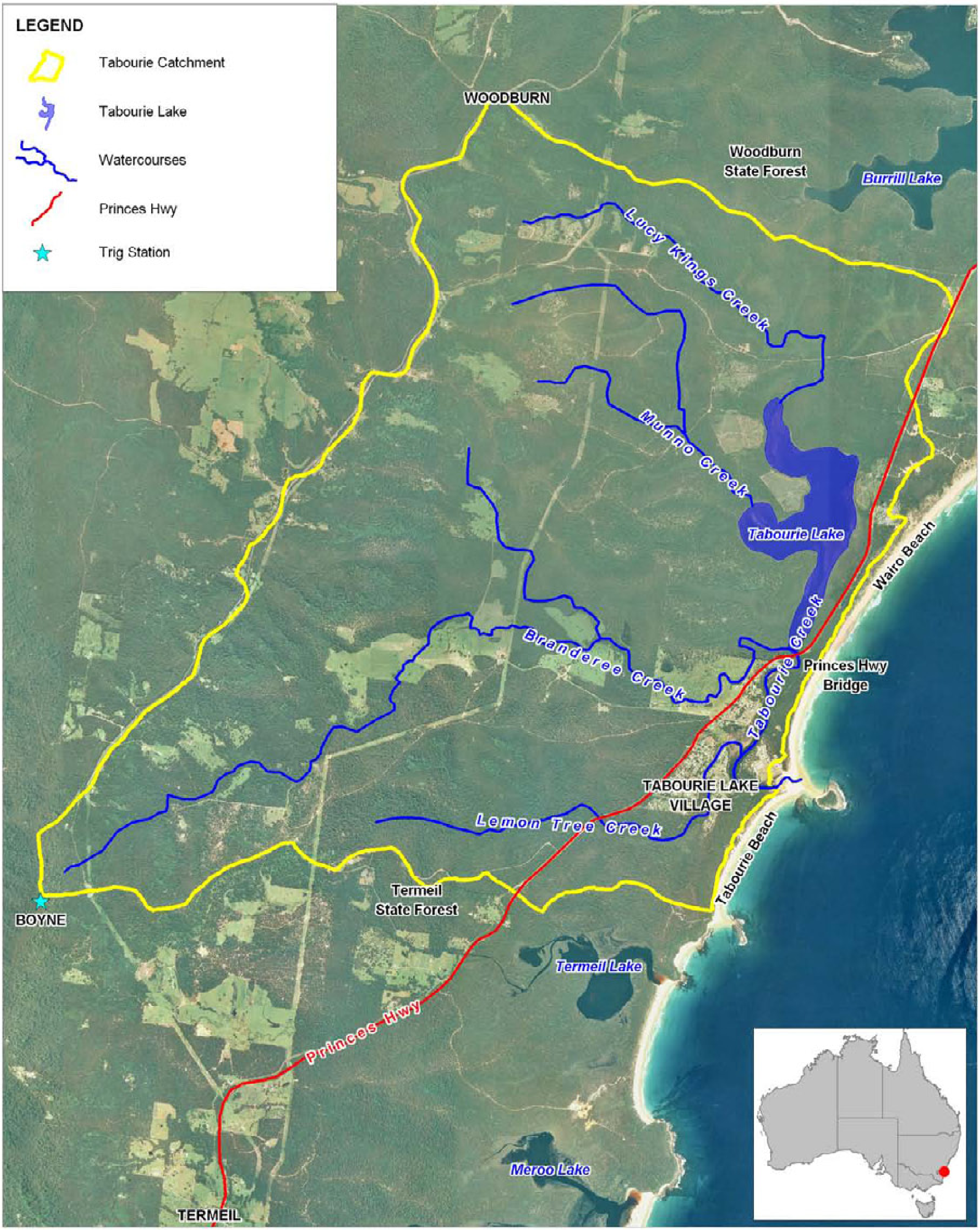
The village of Tabourie Lake is the only existing community within the catchment. It is situated between the Princes Highway and Tabourie Creek, towards the creek entrance in the south-east of the catchment. The township also straddles Lemon Tree Creek, with the Centre Street bridge providing the sole access route to the eastern section of the village. A caravan park also exists between Tabourie Creek and the ocean, directly opposite Tabourie Lake village, on the northern side of the channel. It is accessed via a roadway leading south from the Princes Highway.

Apart from the existing development areas discussed above, land use in the catchment is predominantly forested, with some areas of rural pasture.

1.2 Study Background

There has been no previous detailed investigation of the flood behaviour of the Tabourie Lake catchment. Nevertheless, there has been a history of flooding resulting in inundation, particularly to properties east of Centre St and those located on the Princes Highway, to the west of the bridge.

The majority of significant flooding in Tabourie Lake has coincided with a build up of the entrance berm height and/or rainfall events occurring with high antecedent water levels in the lake. Major historical flood events to have occurred in Tabourie Lake include 1971, 1975 and 1988. An ocean driven event in 1974 also resulted in inundation around Tabourie Lake.



Title:
Study Locality

Figure: 1-1

Rev: A

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

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Previous studies within the catchment have recognised the impacts of flooding on Tabourie Lake village as a key issue, as the developed area of the catchment is generally at a low elevation. The lake storage volume has been previously estimated to be less than 15% of the annual average catchment runoff, indicating a high frequency of elevated lake levels.

1.3 The Need for Floodplain Management at Tabourie Lake

It has been previously reported (Sinclair Knight Merz, 1995) that a number of properties downstream of the Princes Highway are very low lying and include:

- An area just downstream of the confluence of Tabourie and Brandaree Creeks;
- A number of low lying properties on Oak Avenue fronting Lemon Tree Creek .

It is understood that these properties are exposed due to an overland flow path resulting from Branderee Creek effecting a short cut over the Princes Highway during flooding. Most of the floor levels in this area have been raised to a minimum floor level of 2.8 m AHD, as required by Council's Policy.

Most of Tabourie Lake village is built on land which is typically around 2 m AHD. Flood mitigation is presently achieved through artificial opening of the lake entrance, which is understood to occur, on average, once every eight months. However, it has been recognised that effecting such an opening during a 1% AEP design event would be difficult (Sinclair Knight Merz, 1995). Floods can result from the combined effects of catchment runoff and ocean storms.

While the village of Tabourie Lake has a resident population of somewhat more than 600 people, the population swells by a factor of five during peak holiday periods. The Lake boasts three caravan parks, three cabin cottages and holiday unit complexes and a motel (Shoalhaven City Council, 2001).

The Tabourie Lake Estuary Management Plan (Shoalhaven City Council, 1997) noted that the flooding of low lying properties was a key issue for Tabourie Lake. The investigation of flooding and the development of a refined floodplain policy for new development were also identified as "Priority 1" strategies for implementation by the Estuary Management Plan.

The SES floodplan for the Shoalhaven LGA notes that there are around 100 flood prone houses and a caravan park in Tabourie Lake (State Emergency Service, 2004). In addition, the SES notes that little is known about flooding in this area due to the shortage of flood studies. The floodplan also notes that the Caravan Park has 20 permanent vans, of which none are easily removed along with 12 tent sites. The Caravan Park site is set to be evacuated if the water level at the gauge in Tabourie Lake reaches 1.4 m AHD (Tabourie Lake Caravan Park Emergency Management Plan). The evacuation location is the Tabourie Lake Motor Inn on the Princes Highway.

Current practice in floodplain management generally requires consideration of the impact of potential climate change scenarios on design flood conditions. For the Tabourie Lake catchment this includes both increases in design rainfall intensities and sea level rise scenarios impacting on ocean boundary conditions. Accordingly, these potential changes will translate into increased design flood inundation in the Tabourie Lake catchment, such that future planning and floodplain management in the catchment will need to take due consideration of this increased flood risk.

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

Accordingly, Council desires to approach local floodplain management in a considered and systematic manner. This study comprises the initial stages of that systematic approach, as outlined in the Floodplain Development Manual (NSW Government, 2005). The approach will allow for more informed planning decisions within the floodplain of Tabourie Lake.

1.4 The Floodplain Management Process

The State Government's Flood Prone Land Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the Government's Floodplain Development Manual (2005).

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

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

Stages of Floodplain Management

	Stage	Description
1	Flood Study	Determines the nature and extent of the flood problem.
2	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed developments.
3	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain.
4	Implementation of the Floodplain Risk Management Plan	Construction of flood mitigation works to protect existing development. Use of environmental plans to ensure new development is compatible with the flood hazard.

This study represents Stage 1 of the above process and aims to provide an understanding of flood behaviour within the Tabourie Lake catchment.

1.5 Study Objectives

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

- Compilation and review of existing information pertinent to the study and acquisition of additional data including survey as required;

- Undertake a community consultation and participation program to identify local flooding concerns, collect information on historical flood behaviour and engage the community in the on-going floodplain management process;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design event including the 20% AEP, 5% AEP, 2% AEP, 1% AEP and extreme flood event; and
- Presentation of study methodology, results and findings in a comprehensive report incorporating appropriate flood mapping.

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

1.6 About This Report

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

Section 1 introduces the study.

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

Section 3 outlines the community consultation program undertaken.

Section 4 provides information on the additional survey collected for this study.

Section 5 details the development of the computer models.

Section 6 details the model calibration and validation process including sensitivity tests.

Section 7 presents the adopted design flood inputs and boundaryconditions .

Section 8 presents a design flood simulation results and associated flood mapping.

2 STUDY APPROACH

2.1 The Study Area

2.1.1 Catchment Description

Tabourie Lake is situated on the south coast of NSW approximately 10km south-west of the township of Ulladulla. The Tabourie Lake catchment occupies a total catchment area of around 47km^2, extending from the ridge along which Woodburn Road runs from Woodburn to Boyne, and flowing generally east to the coast and then south to the entrance via Tabourie Creek.

The topography of the catchment is shown in Figure 2-1. From a high elevation of around 350m AHD at the top of the catchment, the topography grades steeply from the upper slopes to the floodplain areas west of Tabourie Lake. The lower reaches of the major watercourses in the catchment are characterised by low-lying swampy depressions. These swamp areas provide storage for lower order rainfall events and under normal conditions water entering Tabourie Lake and Creek is primarily from direct rainfall.

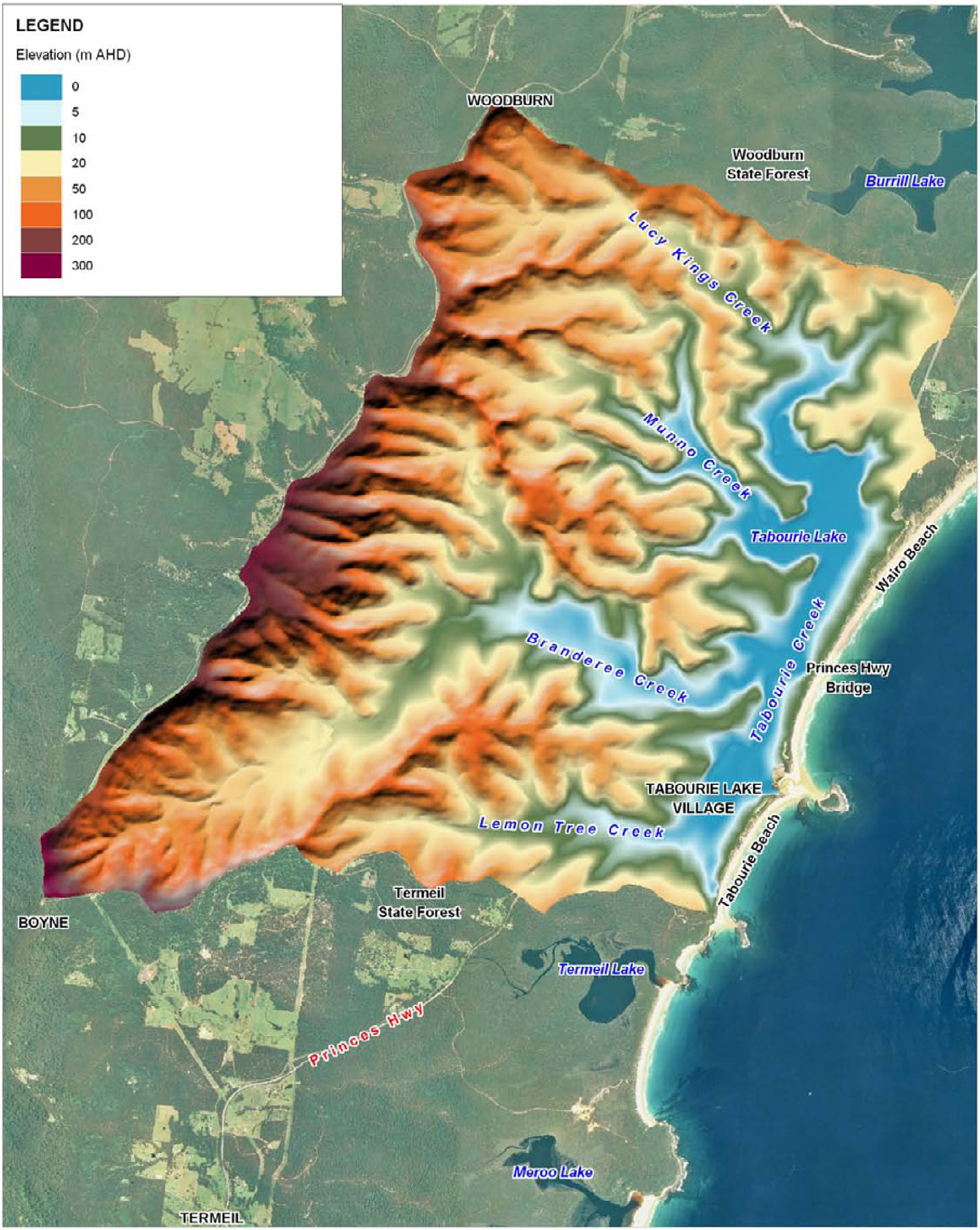
Tabourie Lake Broadwater is fed primarily by Lucy Kings Creek and Munno Creek and forms the flow source of Tabourie Creek. The major tributary of Branderee Creek merges with Tabourie Creek upstream of the Princes Highway Bridge, almost doubling the catchment area from 21km^2 to 40km^2. Lemon Tree Creek completes the major catchment contributions, joining Tabourie Creek toward the entrance.

Land use within the catchment primarily consists of forested areas (90%). Other land uses include pastureland (5%), open water (3%) and urban development (2%). The floodplain area of Tabourie Creek is largely developed, occupied by Tabourie Lake village, whereas that of Branderee Creek principally remains undeveloped and largely occupied by rural farming.

The village of Tabourie Lake is the only existing community within the catchment. It is situated between the Princes Highway and Tabourie Creek, towards the creek entrance in the south-east of the catchment. The township also straddles Lemon Tree Creek, with the Centre Street bridge providing the sole access route to the eastern section of the village. Some development between Centre Street and Tabourie Creek and on the Princes Highway to the west of the bridge, represents the existing development with the highest flood risk exposure.

The Lake Tabourie Tourist Park, consisting of a range of permanent cabins, caravan and camping sites is situated between Tabourie Creek and the ocean, directly opposite Tabourie Lake village on the northern shore of Tabourie Creek. It is accessed via a roadway leading south from the Princes Highway.

The catchment is traversed by one principal local transport route, the Princes Highway. The highway is embanked for a length of around 550m across the floodplain. This section will be overtopped from Tabourie Creek and the Branderee Creek floodplain during high order flood events.



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Topography of the Tabourie Lake Catchment

Figure: 2-1

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

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2.1.2 History of Flooding

The majority of significant flooding in Tabourie Lake has coincided with a build up of the entrance berm height and/or rainfall events occurring with high antecedent water levels in the lake. Major historical flood events to have occurred in Tabourie Lake include 1971, 1975, 1988 and 1991.

There is very little existing flood data for the Tabourie Lake catchment. To get some perspective on historical rainfall events across the Tabourie Lake catchment, the Milton daily rainfall data has been analysed. The Milton rainfall gauge is an Australian Bureau of Meteorology (BoM) operated site, located around 13km north of Tabourie Lake. It has recorded rainfall totals at a daily interval since 1876. Although located outside of the Tabourie Lake catchment, the close proximity of the Milton gauge should provide a reasonable indication of historic rainfall within the catchment.

Table 2-1 presents the highest 1-day, 2-day and 3-day rainfall totals from the Milton data set and their respective year of occurrence. Historical floods in the Tabourie Lake catchment are often associated with large rainfall events but are also heavily influenced by the height of the entrance berm and/or the antecedent water level in the lake. For instance the 1988 event, which resulted in significant flooding of Tabourie Lake village, was only of the order of a 50% AEP rainfall event. The 1977 event, which features prominently in the list of highest 2-day and 3-day rainfall events apparently did not result in a significant flood event.

Table 2-1 Major Rainfall Event Totals

	1-day Total		2-day Total		3-day Total	
Rank	Year	Rainfall (mm)	Year	Rainfall (mm)	Year	Rainfall (mm)
1	1959	311	1927	435	1927	458
2	1945	291	1977	403	1916	457
3	1927	289	1919	392	1919	452
4	**1991**	**268**	**1991**	**376**	1927	435
5	1911	264	1916	375	**1971**	**435**
6	1915	249	**1975**	**354**	1977	423
7	1943	248	1959	352	1977	412
8	1919	245	1951	348	**1975**	**405**
9	1961	239	1911	329	1951	403
10	1916	231	1911	317	1919	392

Significantly there are no major recent events (since 1991) within Table 2-1. Some high rainfall events have occurred within the Tabourie Lake catchment within this time, however, in relation to long term historical records, recent rainfall events have in general been less severe.

It is important to recognise that local rainfall variations between the catchment and the Milton gauge may occur, and indeed shorter (<24 hours) more intense rainfall resulting in significant flooding will not be evident in the daily total time series.

Nevertheless, the data is indicative of broad scale weather systems and does provide some resemblance to known flooding patterns in the catchment. Some of the largest 2-day/3-day rainfall events within the Tabourie Lake catchment identified in the Milton data analysis correspond to known major events within Tabourie Lake village including the 1971, 1975, and 1991 floods.

2.1.3 Previous Investigations

There has been no previous detailed investigation of the flooding characteristics of the Tabourie Lake catchment. However, the Tabourie Lake Estuary Management Study (SKM, 1994), the Tabourie Lake Estuary Management Plan (SCC, 1997) and the Tabourie Lake Entrance Management Policy (SCC, 2005) all recognise flooding in Tabourie Lake village as an issue within the catchment.

Further details of these previous investigations and their relevance in the context of the current flood study are presented in Section 2.2.1.

2.2 Compilation and Review of Available Data

2.2.1 Previous Studies

2.2.1.1 Tabourie Lake Estuary Management Plan (SCC, 1997)

The Tabourie Lake Estuary Management Plan makes reference to a hydrologic study that had been carried out by the NSW PWD. This assessment determined the critical duration for the 1% AEP event to be three hours. Peak discharges for this event at the Princes Highway Bridge and the Tabourie Creek entrance were estimated at 513m^3/s and 603m^3/s respectively.

An assessment of the existing flood behaviour at Tabourie Creek was also made using simple backwater calculations. The limited survey cross sections presented in the Tabourie Lake Tidal Gauging Report (MHL, 1992) were used in these analyses. Information on tide levels associated with average recurrence intervals was not available. A water level of 0.6m AHD was adopted as the downstream tailwater level just inside the ocean entrance. This is considered to be a maximum water level at this location during springtides (during a tidal gauging exercise over a springtide, the highest level recorded at this point was 0.53m AHD) (MHL, 1992).

The results of the backwater analyses indicated that the main channel discharge capacity of Tabourie Creek downstream of the Princes Highway Bridge and extending to the caravan park near the entrance is only of the order of 100m^3/s. Based on the estimated flood discharges it was concluded that large areas would be subjected to flooding during the 1% AEP event.

It is stated that Council's interim flood policy has adopted a 1% AEP flood level of 2.5m AHD for the entire village area. Most houses in Tabourie Lake village are raised to a minimum floor level of 2.8m AHD (2.5m flood level plus 0.3m freeboard), as required by Council's interim flood policy. (Note: the freeboard level has subsequently been raised to 0.5m in 2006).

Discussions with Council and local residents had indicated that the two most flood prone areas include around ten houses immediately downstream of the Princes Highway Bridge and also low-lying properties in Oak Avenue. It is also noted that during the flood events water from the Branderee Creek floodplain can overtop the Princes Highway to the south.

Flood mitigation is provided through the control of downstream tailwater levels. This involves the mechanical opening of the lake entrance once the water level at the Princes Highway Bridge reaches 1.25m AHD (note: management level now superseded). It was noted that the mechanical opening of the entrance would likely be ineffective during a 1% AEP flood event due to the limited capacity of the creek channel.

2.2.1.2 Tabourie Lake Entrance Management Policy (SCC, 2005)

The Tabourie Lake Entrance Management Policy recognises the impacts of flooding on Tabourie Lake village as a key issue, as the developed area of the catchment is generally at a low elevation. It estimates the lake storage volume to be less than 15% of the annual average catchment runoff, indicating a high frequency of elevated lake levels.

It is suggested that a conventional half to one day duration rainfall event of about 100mm could raise a closed lake by up to 0.5m. If the lake level were at its average level of 0.48m AHD (when the entrance was closed), it would require about 150mm of rainfall for the lake to attain its current opening level.

A summary of available terrain elevation and surveyed floor levels is presented, from which key flood levels are extracted. Reference is also made to the February 1971 event which produced the highest recorded lake level, reported to be around 2.5m AHD. The highest likely level that the lake may reach during a flood event is said to be between 3m AHD and 3.5m AHD. These key flood levels are summarised in Table 2-2. Outdoor flooding to properties near the Princes Highway Bridge may occur at a level as low as 1.0m AHD. The lowest floor level within the village is 1.84m AHD and is located on Oak Avenue.

Table 2-2 Key Tabourie Lake Flood Levels (m AHD)

Description	Flood Level
Level at which flooding to low-lying properties will occur	1.2
Level at which flooding to garages and the caravan park will occur	1.6
Level at which flooding above floor level will occur	2.0
Highest recorded Tabourie Lake flood level	2.5
Maximum likely Tabourie Lake flood level	3.5

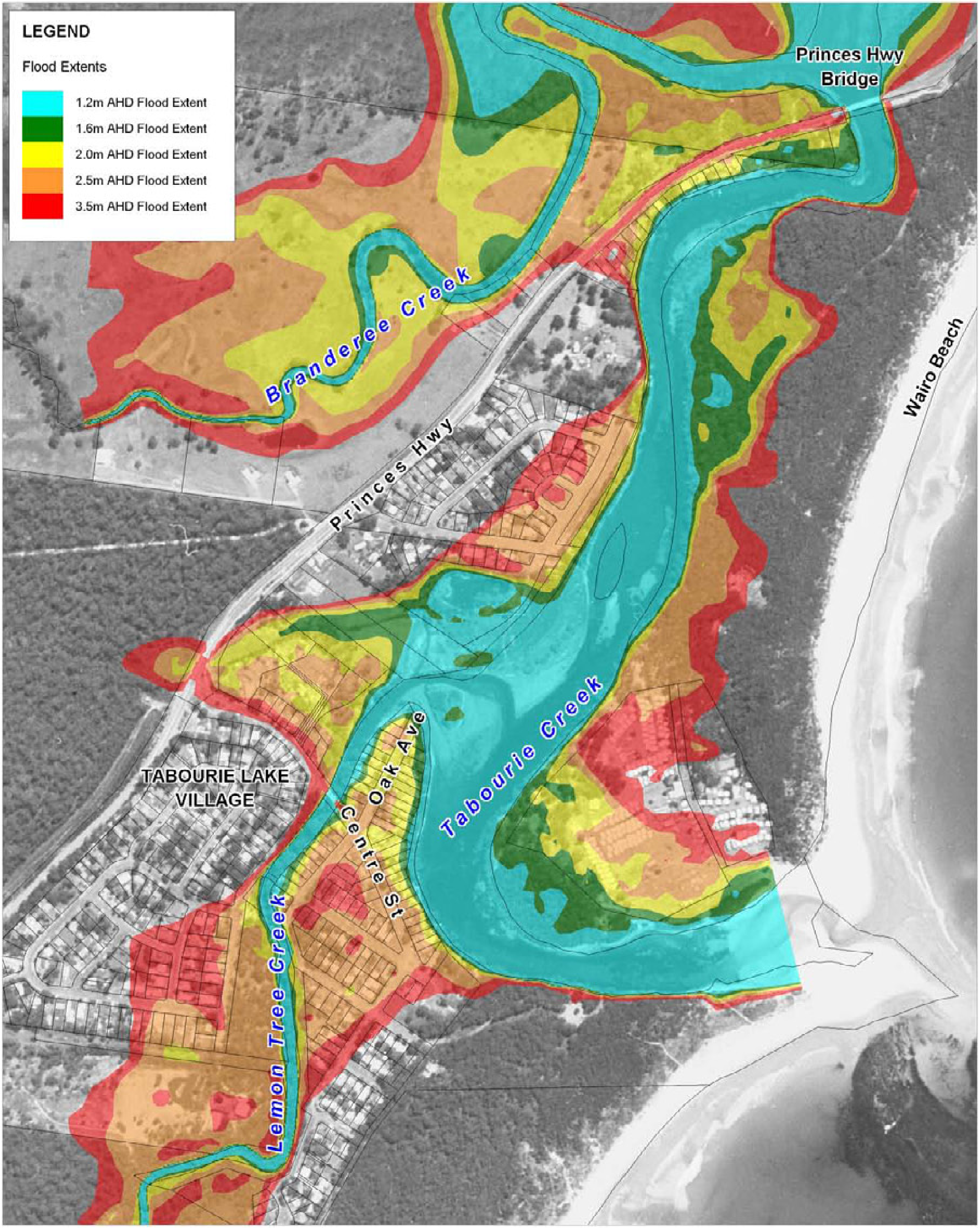
The flood extents associated with the levels presented in Table 2-2 have been mapped using the Tabourie Lake Digital Elevation Model (DEM) and are shown in Figure 2-2 to indicate the areas that would be affected by these flood levels.

Council has maintained a policy of artificially opening the lake entrance when the water level at the Princes Highway Bridge rises to 1.17m AHD. It is understood that this relatively low trigger level is adopted given the rapid water level rise during a rainfall event and subsequent flood risk to existing low-lying urban development (particularly the risk of inundation of fringing septic and on-site sewage treatment systems). A summary of entrance openings since 1987 is presented in Table 2-3.

Table 2-3 Opening History of Tabourie Lake

Date of Opening	Type of Opening	Opening Level (m AHD)*	Closure Date	Period Open (weeks)	Period Closed (weeks)
17/08/87	Council	(1.19)	Prior to 12/11/87	<12	-
29/07/88	Council	(1.33)	Prior to 14/12/88	<20	>38
29/07/89	Council	(1.15)	Prior to 06/12/89	<18	>32
15/12/89	Council	Unknown	Unknown	Unknown	>1
04/02/90	Council	(1.16)	Prior to 05/04/90	<9	-
05/04/90	Council	(1.27)	Prior to 25/12/90	<38	-
09/06/91	***Natural***	Unknown	22/10/92	70	>23
05/12/92	Council	1.03	19/03/93	15	6
13/04/94	Council	1.2	09/08/94	17	56
16/06/95	Council	1.09	18/06/95	0	45
07/12/95 ??	Council	(1.27)	06/02/96	8	25
01/09/96	Council	1.53	21/11/96	12	29
06/03/97	Council	1.12	10/05/97	9	14
27/06/97	Council	1.17	01/07/97	0.5	7
07/07/97	***Natural***	0.83	Prior to 03/09/97	<8	1
09/10/97	Council	1.16	01/05/98	29	>5
16/06/98	Council	1.09	07/02/99	32.5	6.5
14/07/99	Council	(1.21)	14/07/99	0	22
15/07/99	Council	(1.32)	15/07/99	0	-
16/07/99	Council	(1.35)	24/10/99	14	-
25/10/99	Council	1.16	11/12/99	7	-
14/11/00	Council	1.08	12/03/01	17	48
30/08/01	Council	Unknown	31/01/03	70	15
16/05/03	Council	1.21	16/08/03	13	61.5
22/10/04	Council	1.21	29/12/04	10	27
09/07/05	Council	1.29	16/08/05	6	-

** Unbracketed water levels are from logger data from MHL, levels in brackets are estimated by Council. Note that due to an unexplained discrepancy, Council estimates possibly over-estaimate the lake level by about 0.1m. Source: Tabourie Lake Entrance Management Policy (SCC, 2005)*



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Inundation at Key Flood Levels within Tabourie Lake Village

Figure: 2-2

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2.2.2 Water Level Data

The Manly Hydraulics Laboratory (MHL) operates a continuous water level recorder on Tabourie Lake in operation since September 1992. The recorder is located in the Tabourie Creek channel approximately 1km upstream of the ocean entrance and is representative of the general level in the broader waterway.

A time series of the peak daily water level over the total period of record is shown in Figure 2-3. Evident in the time series are periods of both an open and closed entrance condition. Periods with an open entrance regime show a tidal signal indicating daily high tide levels in the Lake. The highest levels generally correspond to closed entrance periods. The maximum water levels in the Lake are generally limited by manual opening of the entrance to relieve the high water level conditions and provide some flood protection to low-lying property. The manual entrance openings corresponding to the peak lake water levels are shown in Figure 2-3, with Council's current entrance management trigger level of 1.17m AHD shown for reference.

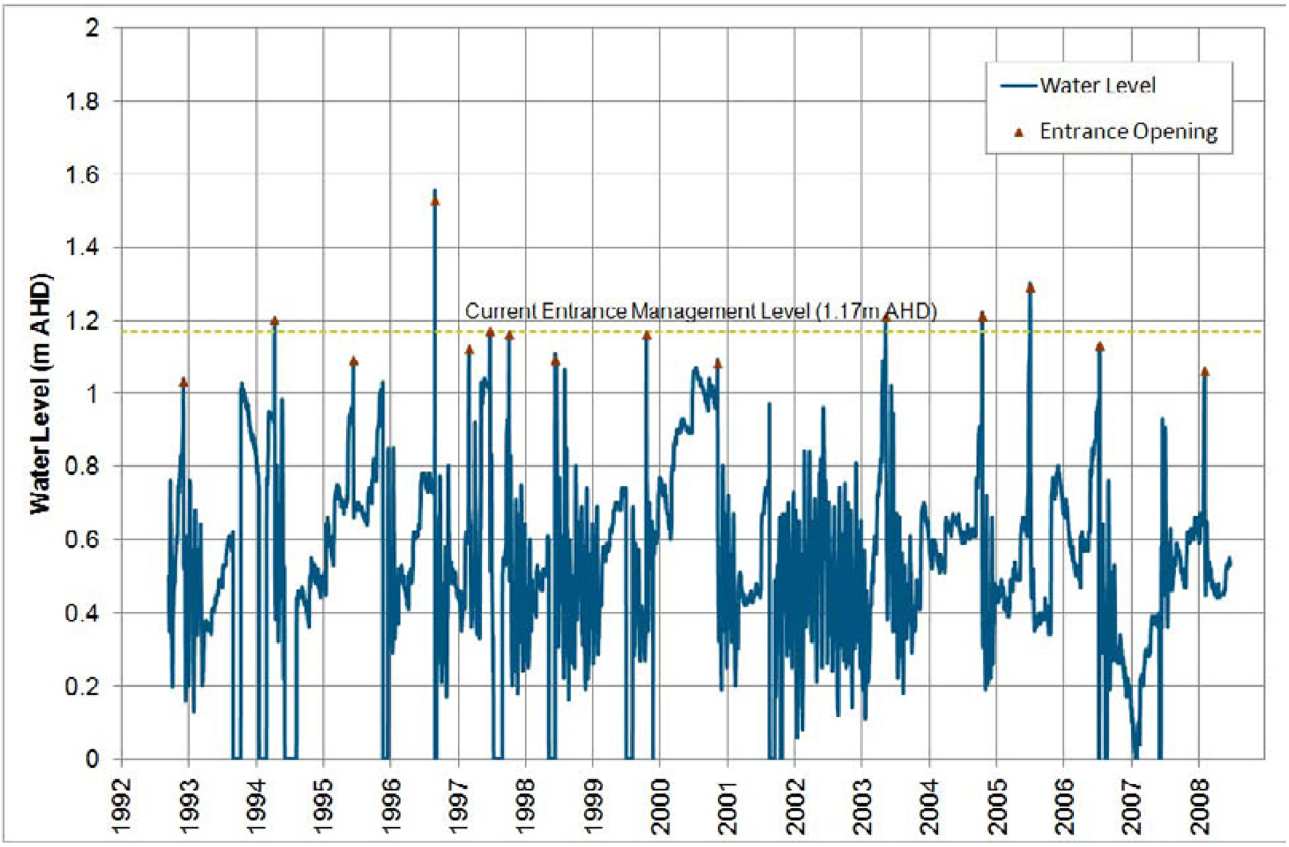


Figure 2-3 Recorded Peak Daily Water Level Time Series at Tabourie Lake

The highest recorded lake level during the period of operation of the water level gauge is 1.53m AHD in September 1996 that included a manual breakout of the entrance. At this water level there is no significant overbank flooding as shown in Figure 2-2. As evidenced in the water level time series, there have been no major flood events in the catchment during the period of operation of the water level recorder. Further discussion on historical flood levels is provided in Section 2.2.3

An analysis of tidal planes at the water level recorder was undertaken by Manly Hydraulics Laboratory and the results were published within a Foreshore Erosion Stabilisation Study (WBM Oceanics Australia, 2003). The analysis covered a period of four months from May to July 2002.

The calculated tidal planes, which may be considered as somewhat representative for the times that the Lake is open to the Ocean are reproduced in Table 2-4.

Table 2-4 Tabourie Lake Tidal Plane Analysis

Tidal Plane	Level (m AHD)
High High Water	0.57
Mean High Water Springs	0.42
Mean High Water	0.39
Mean High Water Neaps	0.35
Mean Sea Level	0.25
Mean Low Water Neaps	0.14
Mean Low Water	0.12
Mean Low Water Springs	0.07
Indian Spring Low Water	-0.04

2.2.3 Historical Flood Levels

There is limited historical flood level data available for Tabourie Lake. Water levels in the Lake have been continuously recorded since September 1992 at the MHL water level gauge situated within Tabourie Creek. However, as previously discussed there have been no major flooding events during this period, with the highest lake level recorded at 1.53m AHD in September 1996.

There is no comprehensive record of water levels in Tabourie Lake prior to operation of the continuous water level gauge in 1992. Peak flood water levels within Tabourie Lake village have been identified for a number of significant flood events as summarised in Table 2-5.

Table 2-5 Historical Peak Flood Levels (m AHD)

Date	Flood Level
September 1996	1.53
April 1988	2.0
March 1975	2.4
February 1971	2.5

The highest known lake flood level is reportedly 2.5m AHD in February 1971 (SCC, 2005). Flood levels for the 1975 and 1988 events have been obtained from a survey of historic flood marks identified through the community consultation undertaken for the flood study.

2.2.4 Rainfall Data

There is an extensive network of rainfall gauges across the wider Shoalhaven area operated by the Bureau of Meteorology (BoM) and the Sydney Catchment Authority (SCA). The full list of rainfall stations, including closed stations, within approximately a 50km radius of the Tabourie Lake catchment is shown in Table 2-6 with their respective period of record. The distribution of these gauges is shown in Figure 2-4.

There are no official gauges located within the Tabourie Lake catchment. The nearest daily read gauge is located just outside the catchment at Woodburn State Forest. However, this gauge ceased recording in 1980. The nearest daily read gauge with an extensive period of record is the Milton Post Office gauge, located approximately 13km to the north. Other local daily read gauges that have a reasonable period of record include Brooman -Carisbrook (1979 to 2008) and Kioloa Old Post Office (1957 to present).

The distribution of continuous rainfall gauges in the vicinity of the Tabourie Lake catchment is much sparser. There are only two continuous rainfall gauges located in close proximity to the Tabourie Lake catchment at Ulladulla and Kioloa. These gauges have a relatively short period of record, with 16 and 8 years respectively. The Kioloa gauge ceased recording in 1988. The Ulladulla gauge is currently operational and began recording in 1994. The stations operated by the Sydney Catchment Authority are located over 30km from the centre of the Tabourie Lake catchment and are west of the range. The stations are therefore considered to be largely unrepresentative of rainfall within the study area.

Further discussion on recorded rainfall data for historical events is presented with the calibration and validation of the models developed for the study in Section 6.

2.2.5 Council Data

Digitally available information such as aerial photography, cadastral boundaries, topography, watercourses, drainage networks, land zoning, vegetation communities and soil landscapes were provided by Council in the form of GIS datasets.

A variety of relevant planning documents, where available, were also reviewed and considered as part of the study. These documents include Council's LEP, Council's Flood Policy, Development Control Plans, and SES Flood Plan.

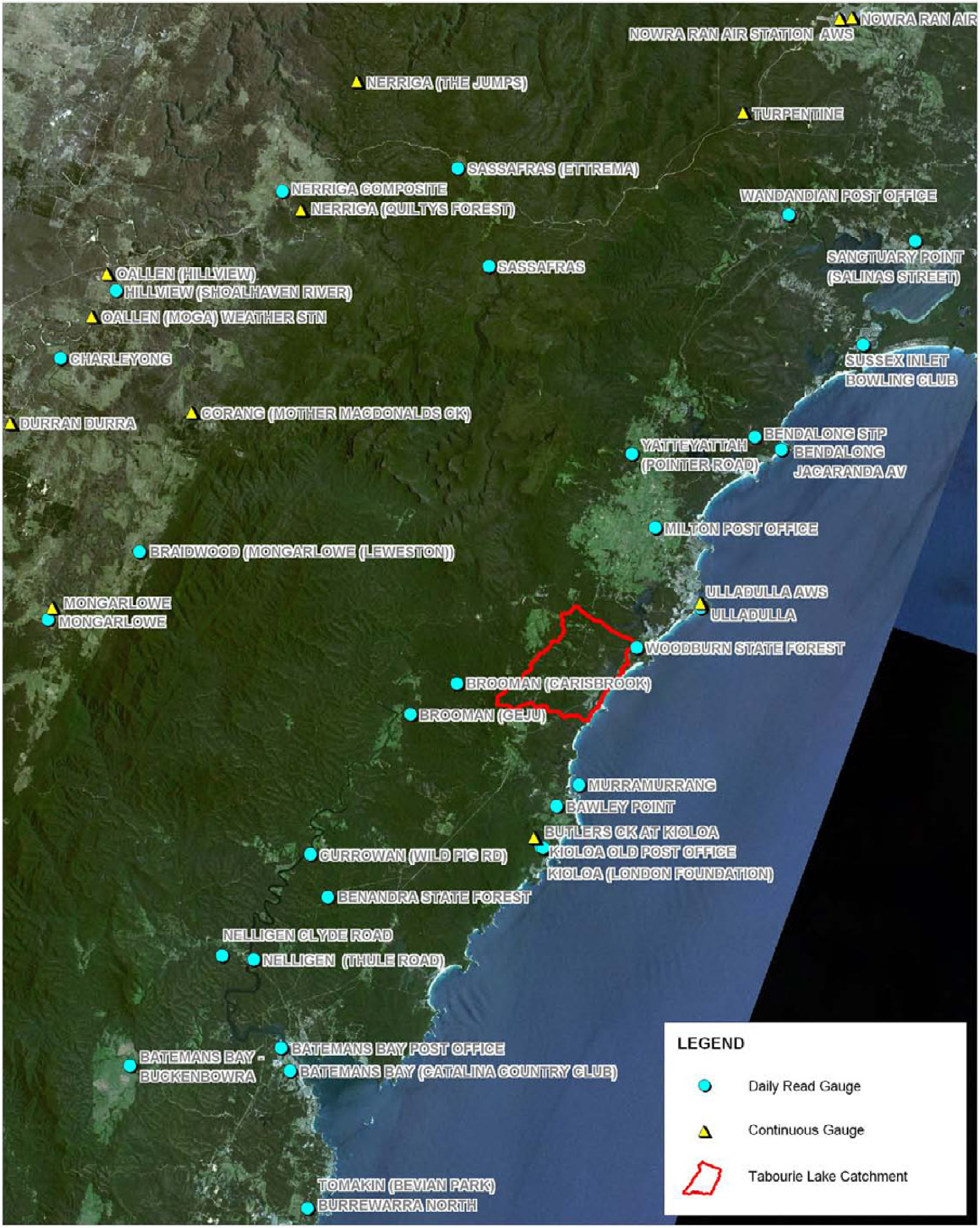
2.3 Site Inspections

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

- Presence of local structural hydraulic controls including the Princes Highway bridge and road embankment, Centre Street crossing of Lemon Tree Creek;
- General nature of Tabourie Creek and tributary channels and associated floodplain noting river plan form, vegetation type and coverage and the presence of significant flow paths;

Table 2-6 Summary of Rainfall Gauges in the Tabourie Lake Locality

Station No.	Name	Operator	Type	Start Year	End Year
216008	Butlers Ck At Kioloa	DIP	Pluvio	1981	1988
568086	Turpentine	SCA	Pluvio	1974	current
568134	Nerriga (The Jumps)	SCA	Pluvio	1977	current
569001	Corang (Mother Macdonalds Ck)	SCA	Pluvio	1972	current
569002	Durran Durra	SCA	Pluvio	1972	current
569003	Mongarlowe	SCA	Pluvio	1972	current
569004	Nerriga (Quiltys Forest)	SCA	Pluvio	1973	current
570347	Oallen (Hillview)	SCA	Pluvio	1973	current
570351	Oallen (Moga) Weather Stn	SCA	Pluvio	1994	current
68072	Nowra Ran Air Station Aws	BoM	Pluvio	2000	current
68076	Nowra Ran Air Station	BoM	Pluvio	1942	2000
69138	Ulladulla Aws	BoM	Pluvio	1994	current
68088	Sanctuary Point (Salinas Street)	BoM	Daily	2000	current
68203	Sassafras (Ettrema)	BoM	Daily	1954	1972
68204	Sussex Inlet Bowling Club	BoM	Daily	1952	current
68222	Wandandian Post Office	BoM	Daily	1985	2003
68229	Bendalong Stp	BoM	Daily	1939	current
69001	Batemans Bay Post Office	BoM	Daily	1895	1996
69004	Benandra State Forest	BoM	Daily	1936	1959
69008	Yatteyattah (Pointer Road)	BoM	Daily	1998	current
69016	Milton Post Office	BoM	Daily	1876	current
69020	Murramurrang	BoM	Daily	1946	1952
69023	Nelligen (Thule Road)	BoM	Daily	1898	current
69025	Bendalong Jacaranda Av	BoM	Daily	1971	1989
69031	Ulladulla	BoM	Daily	1937	1974
69040	Kioloa Old Post Office	BoM	Daily	1957	current
69041	Charleyong	BoM	Daily	1951	current
69046	Mongarlowe	BoM	Daily	1960	1966
69049	Nerriga Composite	BoM	Daily	1898	current
69052	Batemans Bay - Buckenbowra	BoM	Daily	1943	current
69053	Burrewarra North	BoM	Daily	1962	1970
69081	Sassafras	BoM	Daily	1901	1918
69092	Nelligen Clyde Road	BoM	Daily	1967	1971
69098	Tomakin (Bevian Park)	BoM	Daily	1968	1973
69106	Woodburn State Forest	BoM	Daily	1926	1980
69113	Brooman (Geju)	BoM	Daily	1974	1974
69121	Brooman (Carisbrook)	BoM	Daily	1979	2008
69124	Bawley Point	BoM	Daily	1913	1920
69126	Kioloa (London Foundation)	BoM	Daily	1980	1986
69134	Batemans Bay (Catalina Country Club)	BoM	Daily	1985	current
69141	Currowan (Wild Pig Rd)	BoM	Daily	1993	2005
69150	Braidwood (Mongarlowe (Leweston)	BoM	Daily	2004	current
70335	Hillview (Shoalhaven River)	BoM	Daily	2000	current



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Rain Gauges in the Vicinity of the Tabourie Lake Catchment

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Approx. Scale

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- Entrance conditions and connectivity between Tabourie Lake and the ocean, and configuration of beachfront topography incorporating Tabourie Beach, Wairo Beach and Crampton Island.
- Location of existing development and infrastructure on the floodplain, particularly in Tabourie Lake village.

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

2.4 Survey

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

Hydrographic survey data of Tabourie Lake and Tabourie Creek from 1993;

Photogrammetric survey data of Tabourie Lake village from 2005;

Topographic survey data of Tabourie Lake Tourist Park from 2007;

Topographic survey data of the closed lake entrance from 2008; and

10m interval contours digitised from the Geoscience Australia topographic map sheets.

Further information on the extent of these datasets is presented in Section 5.3.2 which details the development of the model DEM.

The review of available topographic data identified the requirement for additional survey to be undertaken to provide the necessary coverage and detail required to build the hydraulic model. The acquisition of the additional survey is discussed in further detail in Section 4.

2.5 Community Consultation

The key elements of the consultation process in undertaking the flood study to date have been:

- Issue of a questionnaire to obtain historical flood data and community perspective on flooding issues; and
- Community information session to gain feedback on the flood questionnaire and provide information to the public on the direction of the flood study.

A questionnaire was delivered to households in the vicinity of Tabourie Lake from which seventy-one (71) individual responses were received. The responses provided an invaluable resource for identifying problem areas within the study area and also provided an indication of locations where historical flood marks were available for survey. Individuals who said they could provide additional information (13) were contacted by telephone to undertake an interview.

A community drop-in Session was held at the Tabourie Lake Rural Fire Service Depot in Beach Street on Tuesday 19th February, 2008. A total of 17 individuals added their names to the attendance register, although it is estimated that around 10 more individuals also attended without signing the register. The level of attendance was striking for a community of this size.

While the issues discussed were wide ranging, dealing with broader environmental issues associated with Tabourie Lake, a significant amount of information relating to the flooding history of the area was obtained.

2.6 Development of Computer Models

2.6.1 Hydrological Model

For the purpose of the Flood Study, a hydrologic model (discussed in Section 5.1) was developed to simulate the rate of storm runoff from the catchment. The model predicts the amount of runoff from rainfall and the attenuation of the flood wave as it travels down the catchment. This process is dependent on:

Catchment area, slope and vegetation;

Variation in distribution, intensity and amount of rainfall; and

Antecedent conditions of the catchment.

The output from the hydrologic model is a series of flow hydrographs at selected locations such as at the boundaries of the hydrodynamic model. These hydrographs are used by a hydrodynamic model to simulate the passage of a flood through the Tabourie Lake catchment to the downstream study limits at the lake entrance into the Tasman Sea.

2.6.2 Hydraulic Model

The hydraulic model (discussed in Section 5.3) developed for this study includes:

- two-dimensional (2D) representation of the Tabourie Lake floodplain covering an area of approximately 7 km^2 of the catchment (approximately 15% of total catchment area), which includes all of the floodplain in the developed areas of Tabourie Lake village.; and
- morphological representation of the channel entrance and surrounds, utilising the van Rijn erosion method.

The hydraulic model is applied to determine flood levels, velocities and depths across the study area for historical and design events.

2.7 Calibration and Sensitivity Testing of Models

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

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

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

Detailed analysis of significant rainfall events (between 1992 and 2008) during periods of a closed entrance were used to validate the simulation of hydrological processes in the Tabourie Lake. The major historical flood events of March 1975 and April 1988 were identified as suitable events for calibration/validation of the developed models. Assessment of the model performance also incorporated a range of sensitivity tests of key variables/model assumptions.

2.8 Establishing Design Flood Conditions

Design floods are statistical-based events which have a particular probability of occurrence. For example, the 1% Annual Exceedance Probability (AEP) event is the best estimate of a flood with a peak discharge that has a 1% (i.e. 1 in 100) chance of occurring in any one year. For the Tabourie Lake catchment, design floods were based on design rainfall estimates according to Australian Rainfall and Runoff (IEAust, 2001).

The design flood conditions form the basis for floodplain management in the catchment and in particular design planning levels for future development controls. The predicted design flood conditions are presented in Section 7.

2.9 Mapping of Flood Behaviour

Design flood mapping is undertaken using output from the hydrodynamic model. Maps are produced showing water level, water depth and velocity for each of the design events. The maps present the peak value of each parameter. Provisional flood hazard categories and hydraulic categories are derived from the hydrodynamic model results and are also mapped. The mapping outputs are described in Section 8 and presented in Appendix A.

3 COMMUNITY CONSULTATION

3.1 The Community Consultation Process

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

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

- Distribution of a questionnaire to landowners, residents and businesses within the study area;
- An information session for the community to present information on the progress and objectives of the flood study and obtain feedback on historical events in the catchment and other flooding issues; and
- Public exhibition of the draft Flood Study (to be undertaken).

These elements are discussed in detail below.

3.2 Community Questionnaire

In February 2008, a short questionnaire was sent to landowners, residents and businesses located within the study area. The questionnaire was sent to over 400 property holders (Council to confirm), with Council receiving 104 responses.

The questionnaire asked residents to provide as much information as possible in regard to historical flood events within the catchment. A detailed summary of responses to the questionnaire is included in Appendix D. Provided hereunder is a summary of the key information provided in the responses.

Historical Flooding

Respondents were asked to acknowledge dates of previous flood events within the catchment from personal experience. A wide range of responses were received, with no individual year noted by more than four respondents. A summary of the given responses is presented in Figure 3-1. The largest concentration of responses relates to flooding in the early 1970s, with the 1974 event being recalled most prominently.

The 1974 event was a significant ocean flooding event in which the dune system at the entrance was demolished and redeposited in the entrance channel. There were also a number of significant fluvial events in this period, including the 1971 and 1975 events, the two largest recent events to have occurred in the catchment. The number of responses (11) received for the period 1970-75 may refer to any of these three events as the close proximity of these events to each and the progression of time since these events would make precise recollection difficult.

Other events that stand out in Figure 3-1 are the 1988 and 1991 events, both of which were known to be significant events within the catchment. The significance of the 2007 event may be overstated by those that have only been resident in the catchment for a short length of time. The highest water level recorded in Tabourie Lake during 2007 was 0.93m AHD, as shown in Figure 2-3. This is not a high enough level to result in lake flooding, however the responses may be related to surface runoff issues.

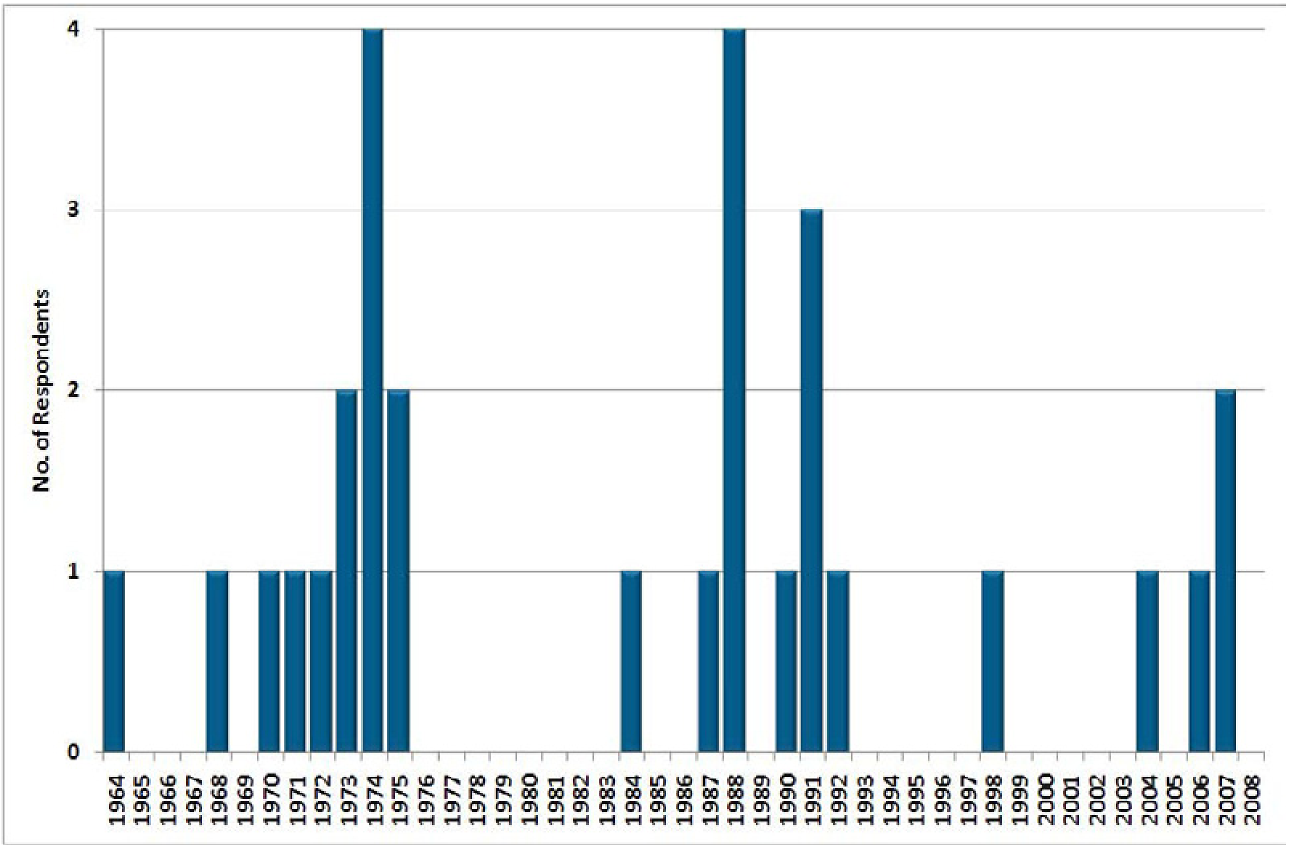
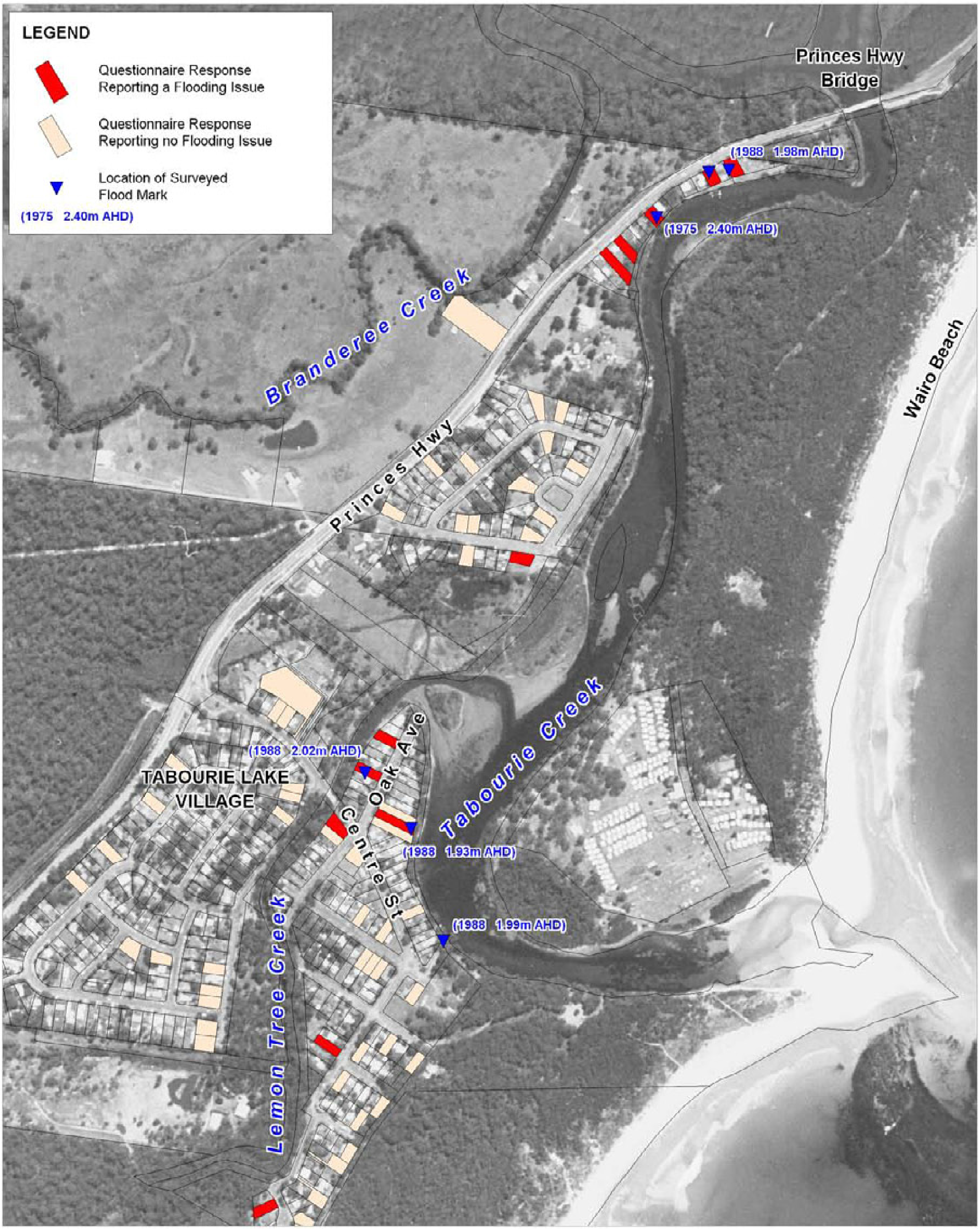


Figure 3-1 Largest Storm Experienced as Indicated by Residents

A key objective of the flood questionnaires was to obtain peak flood level reference points for model calibration purposes. There were seven responses which included reference to recorded flood marks. These flood marks were later surveyed to obtain peak flood levels where possible. Most of these flood marks related to the 1988 event and also included a recorded level for the 1975 event. The location of respondents that reported flooding issues on their property and the locations of the surveyed flood marks are shown in Figure 3-2.



Title:
Distribution of Responses to the Questionnaire

Figure: 3-2

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3.3 Community "Drop-in" Session

A community drop-in Session was held at the Tabourie Lake Rural Fire Service Depot in Beach Street on Tuesday 19th February, 2008. A total of 17 individuals added their names to the attendance register, although it is estimated that around 10 more individuals also attended without signing the register. The level of attendance provided good representation for a community of this size.

While the issues discussed were wide ranging, dealing with broader environmental issues associated with Tabourie Lake, a significant amount of information relating to the flooding history of the area was obtained. A summary of the comments offered during the session are included in Appendix B.

Some of the key information arising from the community information session includes:

- The locations of properties perceived to be most vulnerable to flood risks were property on the immediate downstream side of the Princes Highway Bridge, lower parts of Portland Way, property on Oak Avenue, properties to the north of River Road and the Caravan Park;
- There have been a number of changes to the natural topography within the village area including numerous fill sites for house construction representing present-day conditions;
- Prior to the raising of the Princes Highway in the early 1980's, the road was cut in numerous flood events; and
- Recognition of the 1974 ocean event as the largest storm experienced that resulted in significant change to the morphology of the entrance.

3.4 Individual Interview – Mr. Mike James

Shoalhaven City Council suggested contact be made with Mr Mike James of Shoalhaven Heads, who agreed to an on-site interview regarding the study. Mr. James lived in Tabourie Lake between 1956 and 1974 and maintained a strong involvement with the township until 2002.

A summary of the key points discussed with Mr. James is given in Appendix B. The discussions largely centred around the observations of the 1974 ocean driven events that resulted in significant inundation of Tabourie Lake and significant change to the dunes, entrance and sand distribution within the Tabourie Creek.

3.5 Public Exhibition

The Draft Flood Study was placed on exhibition for the period 15th September 2010 to 26th October 2010. A community information session was held at the Tabourie Lake Rural Fire Service Depot during the exhibition period. Only one formal comment was received in relation to flood planning levels which will be addressed in the next stage of the floodplain management process.

4 ADDITIONAL SURVEY

The following sections outline the additional survey data collected to supplement the existing data and enable the establishment of a suitable two-dimensional model representation of the Tabourie Lake tributary channels and floodplain.

A number of areas were identified as having insufficient detail in the available datasets to provide a suitable representation of topography within the model. The areas requiring additional data were principally:

- Channel and floodplain data for the lower reaches of Branderee Creek;
- Channel and structure data for the lower reaches of Lemon Tree Creek;
- Floodplain areas to the east and south of Tabourie Creek;
- Structure survey and road crest details for the Princes Highway.

4.1 Channel Cross Sections

Other than the 1993 hydrographic survey of Tabourie Lake and Tabourie Creek, there was no detailed survey of watercourses available. For Branderee and Lemon Tree Creeks cross-section surveys were required to accurately define the shape of the watercourses.

Figure 4-1 shows the location of cross sections that were surveyed by Allen, Price and Associates to provide additional waterway information for significant tributaries of Tabourie Creek.

The sections extend from the confluence of Lemon Tree Creek and Tabourie Creek to approximately 1.5km upstream on Lemon Tree Creek, and from the Princes Highway Bridge to approximately 2.3km upstream on Branderee Creek. These limits correspond to the extent of the modelled channel network. The distribution of cross sections shown represents an average cross section spacing of 100m along the main tributary alignments in the catchment. The cross section locations also coincide with the location of major hydraulic structures as discussed in Section 4.2. This distribution and average spacing of cross sections was defined to provide an appropriate level of detail to develop the hydraulic model.

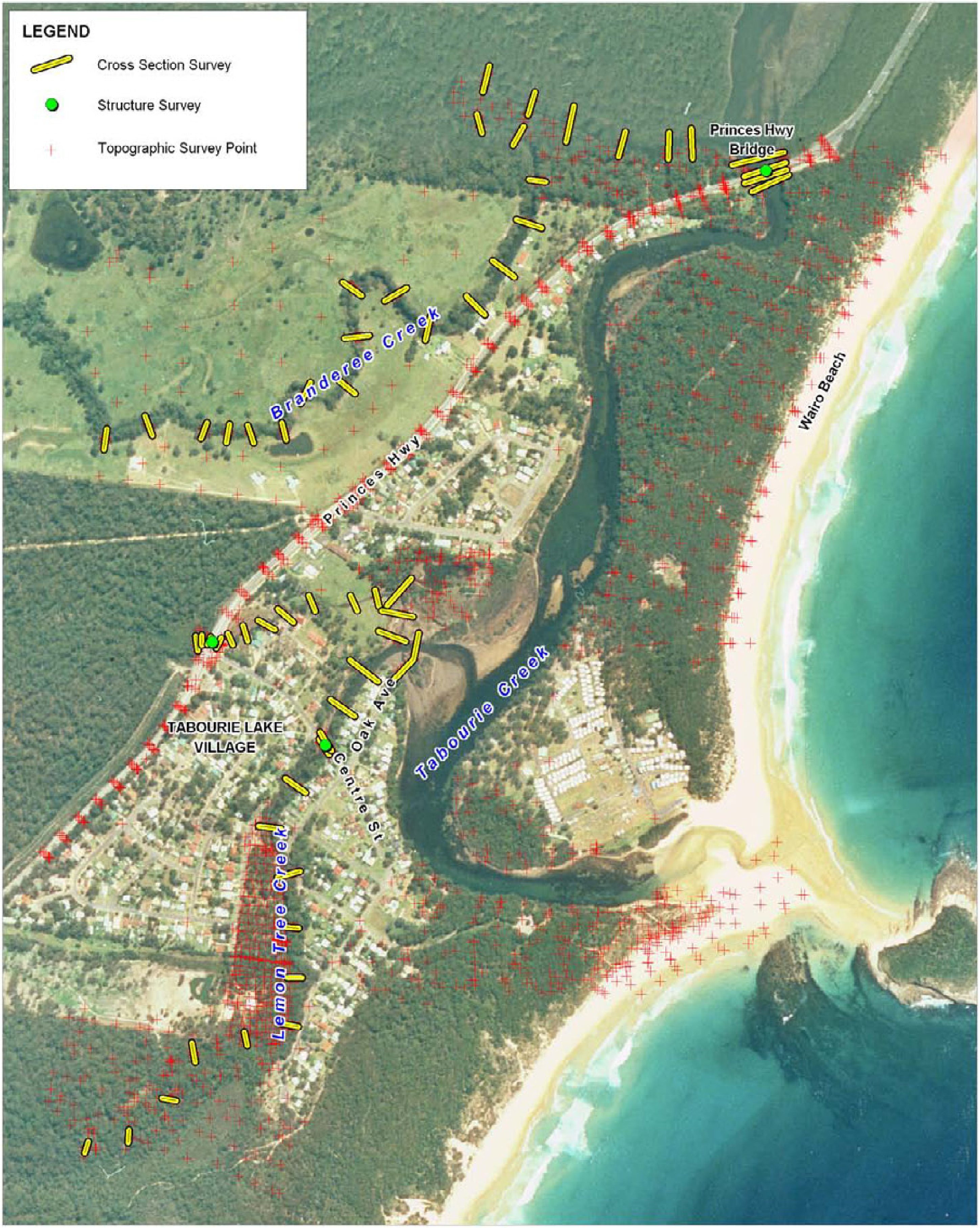
4.2 Structures

There are three hydraulic structures within the study area, for which no existing survey detail was available. Accordingly, structure surveys were incorporated into the additional ground survey to provide the details required to build the hydraulic model such as configuration, dimensions, waterway areas and invert levels.

These structures include bridges over Tabourie Creek and Lemon Tree Creek and a culvert on an unnamed tributary. Further structure details and their respective model configuration are presented in Section 5.

4.3 Floodplain Topography

In addition to the surveyed channel cross section and structure details, elevations were also obtained for areas of floodplain where no detailed information was available. This survey incorporated a series of ground level "spot heights". These survey points are typically spaced at around 25m, the locations of which are shown in Figure 4-1.



Title:
Location of Cross Section and Structure Survey

Figure: 4-1

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5 MODEL DEVELOPMENT

Computer models are the most accurate, cost-effective and efficient tools to assess a catchment's flood behaviour. For this study, two types of models were used:

- A hydrologic model of the entire Tabourie Lake catchment; and
- A hydraulic model including Tabourie Lake, Tabourie Creek and the lower reaches of Branderee and Lemon Tree Creeks.

The **hydrologic model** simulates the catchment rainfall-runoff processes, producing the river/creek flows which are used in the hydraulic model.

The **hydraulic model** simulates the flow behaviour of the channel and floodplains, producing flood levels, flow discharges and flow velocities.

Both of these models were calibrated interactively.

Information on the topography and characteristics of the catchments, watercourses and floodplains are built into the models. Recorded historical flood data, including rainfall, flood levels and river flows, are used to simulate and validate (calibrate and verify) the models. The models produce as output, flood levels, flows (discharges) and flow velocities.

Development of a hydraulic model follows a relatively standard procedure:

1. Discretisation of the catchment, watercourses, floodplain, etc.
2. Incorporation of physical characteristics (river cross-sections, floodplain levels, structures etc).
3. Establishment of hydrographic databases (rainfall, river flows, flood levels) for historic events.
4. Calibration to one or more historic floods (calibration is the adjustment of parameters within acceptable limits to reach agreement between modelled and measured values).
5. Verification to one or more other historic floods (verification is a check on the model's performance without further adjustment of parameters).
6. Sensitivity analysis of parameters to measure dependence of the results upon model assumptions.

Once model development is complete it may then be used for:

- establishing design flood conditions;
- determining levels for planning control; and
- modelling development or management options to assess the hydraulic impacts.

5.1 Hydrological Model

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

- the catchment slope, area, vegetation and other characteristics;

- variations in the distribution, intensity and amount of rainfall; and
- the antecedent conditions (dryness/wetness) of the catchment.

These factors are represented in the model by:

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

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

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

5.1.1 Catchment Delineation

The Tabourie Lake catchment drains an area of approximately 47km^2 to the Tabourie Creek entrance at the Tasman Sea. For the hydrological model this area has been delineated into 27 sub-catchments as shown in Figure 5-1. The sub-catchment delineation provides for generation of flow hydrographs at key confluences or inflow points to the hydraulic model.

Table 5-1 summarises the key catchment parameters adopted in the RAFTS-XP model, including catchment area, vectored slope and PERN (roughness) value estimated from the available topographic information and aerial photography. The adopted PERN values considered the proportion of forested catchment to cleared/pasture area. As indicated in the table and evident from aerial photography, the greater proportion of the Tabourie Lake catchment is largely forested.

The PERN values provided in Table 5-1 represent the largely undeveloped catchment area of Tabourie Lake. For sub-catchments C10, C23 and C24, that contain regions of urban development and/or lake and channel surface area, proportionate adjustments to PERN values have been made to reflect the increased responsiveness of these land use types.

Table 5-1 RAFTS-XP Sub-catchment Properties

Catchment Label	Area (ha)	Slope (%)	PERN	Catchment Label	Area (ha)	Slope (%)	PERN
C1	205.8	2.4	0.12	C14	71.9	1.5	0.15
C2	176.1	1.4	0.15	C15	133.6	2.3	0.1
C3	101.2	2.9	0.15	C16	138.8	2	0.15
C4	243.2	1	0.15	C17	216.6	1.7	0.1
C5	101.8	3.6	0.12	C18	100.4	0.5	0.1
C6	67.4	4.1	0.15	C19	115.6	7.5	0.15
C7	161.3	1	0.15	C20	114.8	0.4	0.12
C8	275.5	1.1	0.15	C21	310.9	1.6	0.12
C9	162.3	1.5	0.15	C22	39	1.9	0.15
C10	227.5	0.6	0.15	C23	121.5	0.4	0.15
C11	159.7	1.7	0.15	C24	51.2	0.3	0.15
C12	414.3	2.2	0.15	C25	194.5	6.2	0.15
C13	134.4	4.1	0.15	C26	194.3	2.1	0.15
C27	292.1	0.9	0.15	C27	292.1	0.9	0.15

5.2 Rainfall Data

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

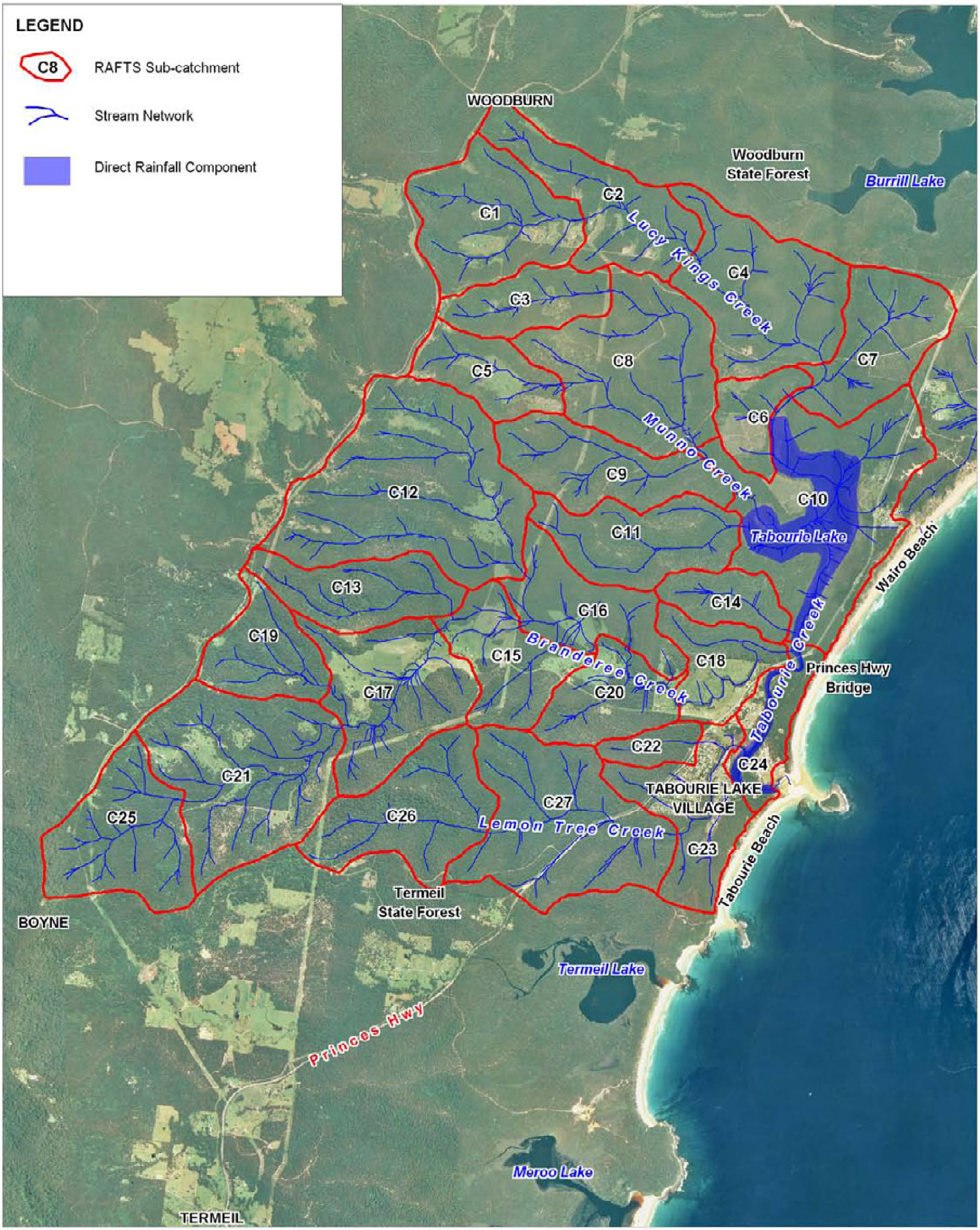
- Rainfall depth – the depth of rainfall occurring across a catchment surface over a defined period (e.g. 270mm in 36hours or average intensity 7.5mm/hr); and
- Temporal pattern – describes the distribution of rainfall depth at a certain time interval over the duration of the rainfall event.

Both of these properties may vary spatially across the catchment.

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

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

The rainfall inputs for the historical calibration/validation events are discussed in further detail in Section 6 and design events discussed in Section 7.



Title:

RAFTS Model Sub-catchment Layout

Figure: 5-1

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5.3 Hydraulic Model

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

5.3.1 Topography

The ability of the model to provide an accurate representation of the flow distribution on the floodplain ultimately depends upon the quality of the underlying topographic model. For the Tabourie Lake catchment, a 5m by 5m gridded DEM was derived from topographic, hydrographic and photogrammetric survey provided by Council.

Interpolation of DEMs from multiple data sources, particularly when incorporating survey data in clustered sampling locations (such as cross-section data), can generate irregularities within the output data. Care has been taken to ensure that the DEM provides as good a representation as possible of channel and floodplain topography from the available data sources.

The topography of Tabourie Lake, Tabourie Creek and the wider floodplain has been interpolated form the available data using appropriate techniques. Additional breaklines were incorporated where necessary, in order to maintain a representative terrain profile. Branderee Creek and Lemon Tree Creek had separate DEMs constructed from surveyed cross-section data. Linear features such as the Princes Highway have been written directly to the model topography from topographic survey levels.

Figure 5-2 shows the location of survey data sources that have been used for the construction of the DEMs. A TUFLOW 2D domain model resolution of 4m was adopted for the study area. It should be noted that TUFLOW samples elevation points at the cell centres, mid-sides and corners, so a 4m cell size results in elevations being sampled every 2m. This resolution was selected to give necessary detail required for accurate representation of floodplain topography and its influence on out-of-bank flows. The ground surface elevations for the TUFLOW model grid points are sampled directly from the DEMs discussed above.

5.3.2 Extents and Layout

With consideration to the available survey information and local topographical and hydraulic controls, a 2D model was developed extending from the Tabourie Creek entrance at the downstream limit, upstream along the major tributary routes. The model incorporates the whole of Tabourie Lake and Tabourie Creek and the lower reaches of both Branderee and Lemon Tree Creeks. The model layout is presented in Figure 5-3.

The floodplain area modelled within the 2D domain comprises a total area of some 7km^2 which represents the lower 15% of the entire Tabourie Lake catchment. A DEM was derived for the study area from the topographic, hydrographic and photogrammetric data provided by Council, as discussed in Section 5.3.1.

5.3.3 Hydraulic Roughness

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

The hydraulic roughness is one of the principal calibration parameters within the hydraulic model and has a major influence on flow routing and flood levels. The roughness values adopted from the calibration process is discussed in Section 6.

5.3.4 Structures

There are three bridge and culvert crossings over the main channel alignments within the model extents as detailed in Table 5-2 (refer to Figure 5-3 for locations). These structures vary in terms of construction type and configuration, with varying degrees of influence on local hydraulic behaviour. Incorporation of these major hydraulic structures in the models provides for simulation of the hydraulic losses associated with these structures and their influence on peak water levels within the study area.

Table 5-2 Major Hydraulic Structures within Model Area

ID	Location	Structure
S1	Princes Hwy Bridge (Tabourie Creek)	Concrete bridge (approx. 59m span)
S2	Centre St Bridge (Lemon Tree Creek)	Concrete bridge (approx. 21m span)
S3	Princes Hwy Culvert (Un-named Tributary)	Concrete culvert (2.7m x 0.9m box)

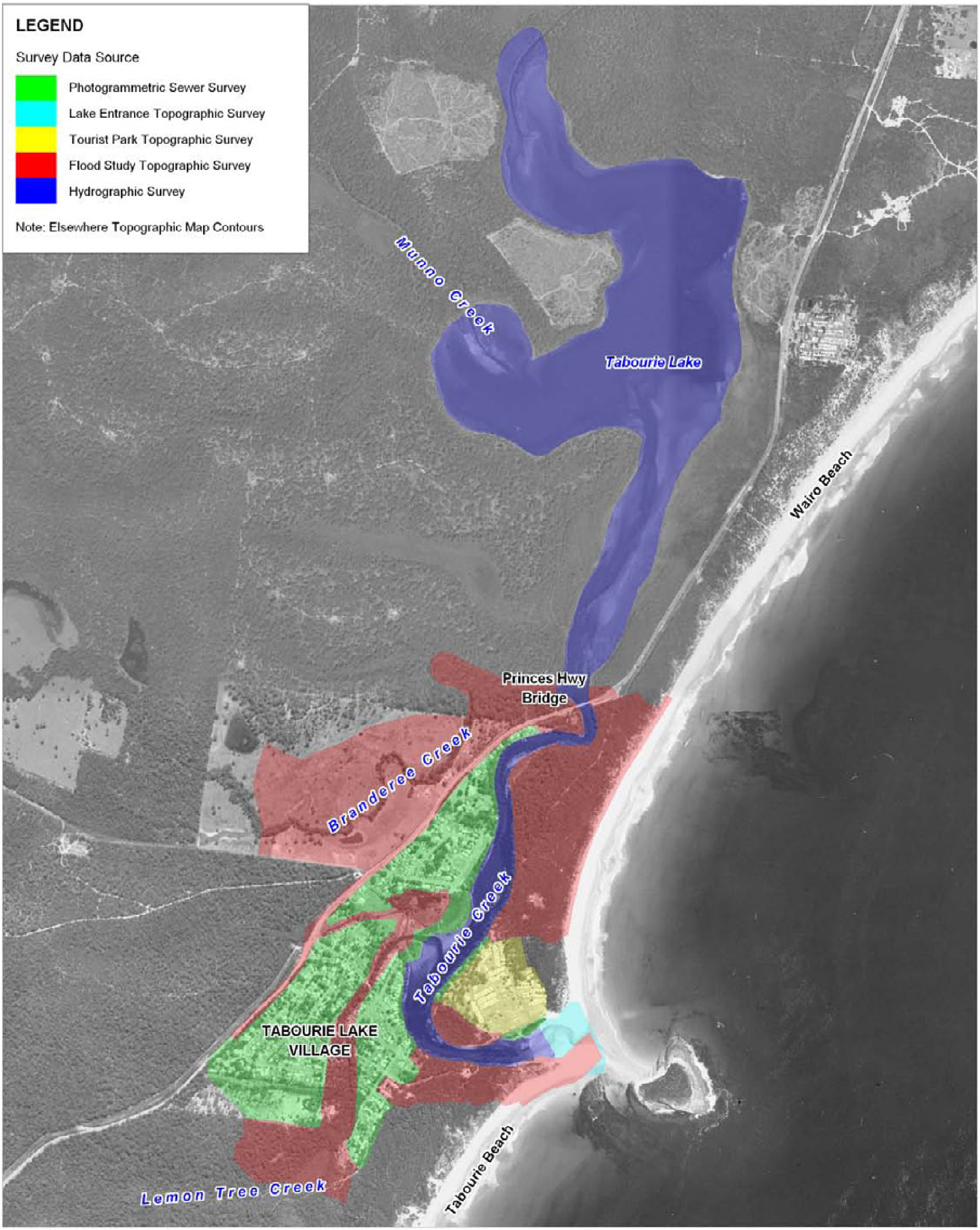
5.3.5 Boundary Conditions

The model boundary conditions are derived as follows:

- Inflows - the rainfall runoff calculated by the hydrologic model at major sub-catchment inflow points and along the modelled watercourse alignments of the Tabourie Creek channel and significant tributaries. This included representation of direct rainfall input to Tabourie Lake. (refer Figure 5-3 for inflow locations); and

- Downstream Water Level– the downstream model limit corresponds to the water level of the Tasman Sea. A water level time series has been applied at this location.

The adopted water levels for the downstream boundary condition (i.e. water levels within the Tasman Sea) for the calibration and design events are discussed in Section 6 and Section 7 respectively.



Title:
Survey Data Sources used for DEM Construction

Figure: 5-2

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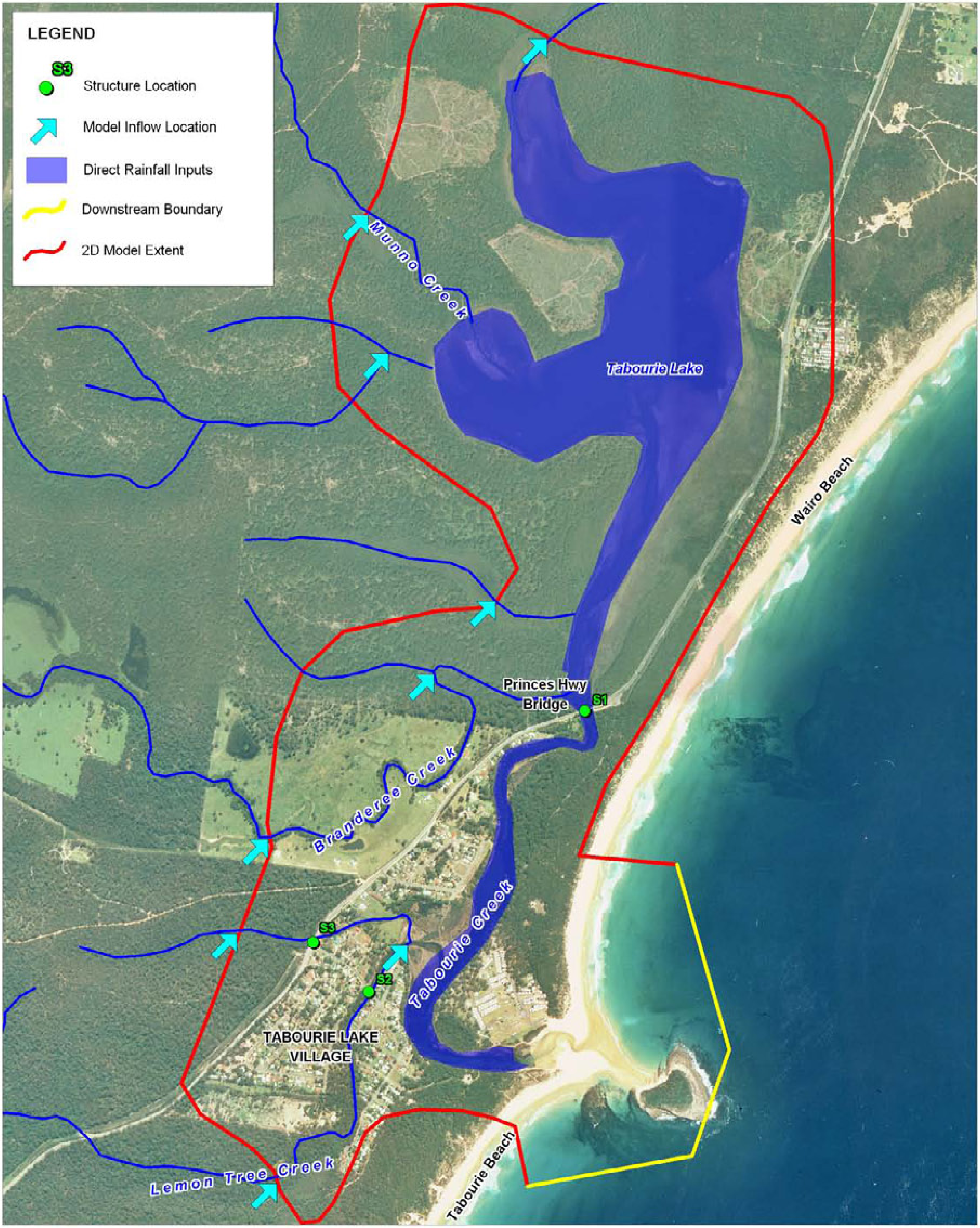

Title:
Linked 1D/2D Model Layout

Figure: 5-3

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6 MODEL CALIBRATION AND VALIDATION

6.1 Selection of Calibration Events

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

Within the Tabourie Creek catchment however, there is a distinct lack of historical flood data. As previously discussed, much of the existing flood information relates to the MHL water level recorder, which dates back to 1992. Since this time there have been no major flood events within Tabourie Lake village given both the lack of major rainfall events and the management of the entrance condition for low-order flood control. The highest recorded lake level during this period was 1.53m AHD in September 1996.

The three largest historical flood events in terms of recorded peak water levels have been February 1971 (2.5m AHD), March 1975 (2.4m AHD) and August 1988 (2.0m AHD). However, the data available for these events are limited to a small number of peak flood levels around the Lake foreshore.

The calibration process for the Tabourie Lake catchment is somewhat complicated by the significant influence of the entrance condition. This influence emerges for events occurring both during closed and opened periods. Under closed entrance conditions, peak water levels are largely driven by maximum berm heights and the occurrence of natural breakouts during the flood event. For natural breakouts, the initial height of the breakout, the timing during an event, and the subsequent development of the breakout channel are variables for which calibration data is not available. Similarly for open periods, the hydraulic efficiency of the open entrance is governed by the topographical configuration of the entrance at the time of the event and the interaction with the prevailing tidal conditions. Again profile data for the entrance at the commencement of a flood event is unlikely to be available.

Further to the uncertainties in regard to the entrance condition, the catchment is ungauged with respect to flow rates. Accordingly, there is no recorded flow data suitable to calibrate the hydrological model for historical events.

An alternative approach to calibration of the catchment hydrology is considered by deriving flow rates from recorded water levels in Tabourie Creek. For rainfall events when the lake entrance is closed, the recorded water level time series can be converted to a lake volume time series using a stage-storage relationship. The incremental volumetric increase is representative of the hydrograph of combined catchment inputs. These derived catchment inputs may then be used for calibrating the catchment runoff generated from the hydrological model.

In considering the best approach to model calibration and validation using the available data, the following analyses have been undertaken:

- Volumetric analysis of catchment inputs to Tabourie Lake for significant rainfall events during closed entrance periods between 1992 and 2008;
- Calibration of hydrological model to derived volumetric catchment inputs for “closed-entrance” events; and
- Simulation of major historical events (March 1975 and August 1988) with sensitivity testing of assumed conditions such as entrance berm configuration and initial lake water levels.

6.2 Volumetric Analysis of Tabourie Lake Inflows

For rainfall events when the lake entrance was closed, the recorded water level increase in Tabourie Lake can be converted to a volumetric increase using a stage-storage relationship derived from the DEM. Rainfall events when the lake entrance is open are not suitable as there is no record available for volumes lost as flow through the entrance.

6.2.1 Available Lake Storage

A stage-storage relationship for the Tabourie Lake system (including Tabourie channel) has been derived from the DEM of the study area (refer Section 5.3.1). Figure 6-1 shows the stage-storage relationship from a minimum Tabourie Lake bed level of approximately -0.5m AHD (note: Tabourie Creek invert levels to about -2m AHD) up to a level of 3m AHD. At a level of 3m AHD, there is significant overbank inundation (refer Figure 2-2) incorporated in the storage relationship.

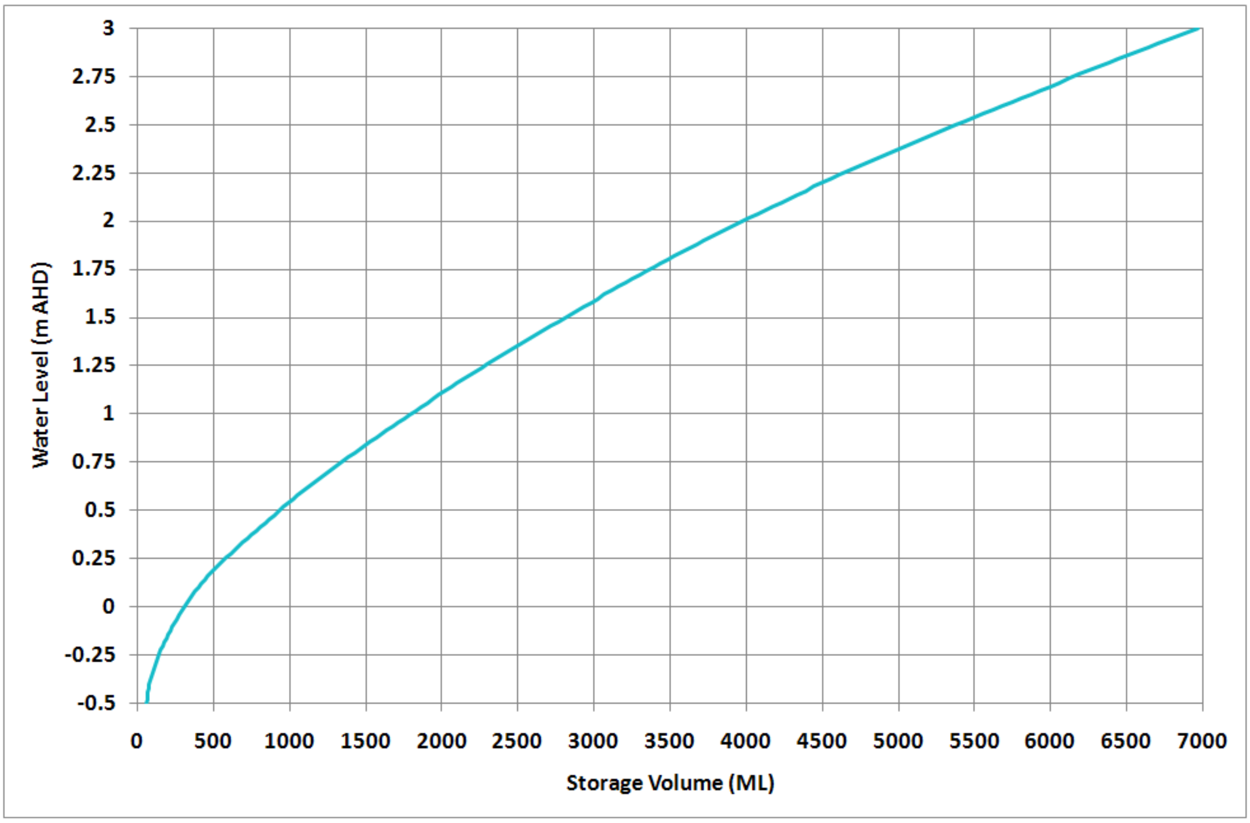


Figure 6-1 Stage-Storage Relationship for Tabourie Lake and Creek

6.2.2 Rainfall Data

The most representative rainfall record for the Tabourie Lake catchment is the private daily read record of local resident Mr John Collins. Records for this gauge date back to 1995, a similar period of record to that of the MHL water level gauge. This rainfall data was analysed to identify the largest events since 1995. Table 6-1 presents the 1-day, 2-day and 3-day rainfall totals from the record.

The August 1998 event represents one of the largest recent rainfall events in the Tabourie Lake catchment, with a 2-day rainfall total of 235mm. However, reference to the MHL water level recorder shows that this event occurred during a period when the lake entrance was open and is therefore unsuitable for volumetric analysis. The lake entrance was also open during the October 1999, February 2002 and April 2002 events. The water level gauge did not record data during the July 1999 event. Of the 16 largest events (1-day to 3-day totals) to have occurred since 1995, this leaves 11 available for volumetric analysis.

Table 6-1 Recent Rainfall Event Totals

	1-day Total		2-day Total		3-day Total	
Rank	**Date**	**Rainfall (mm)**	**Date**	**Rainfall (mm)**	**Date**	**Rainfall (mm)**
1	Mar-97	144	Aug-98	235	Aug-98	255
2	Aug-98	129	Mar-97	155	Mar-97	165
3	Oct-05	111	Jul-99	139	Feb-02	160
4	Oct-99	110	Oct-99	138	Jun-97	147
5	Jun-97	100	Sep-96	137	Jul-99	146
6	Apr-02	93	Jun-97	126	Oct-99	146
7	Oct-04	90	Oct-05	117	Sep-96	139
8	Sep-96	89	Jun-06	110	Jun-06	134
9	Jun-05	83	May-03	108	May-03	133
10	Feb-02	80	Oct-04	105	Oct-05	122
11	Apr-03	79	Jul-01	104	Jul-01	118
12	Jun-06	78	Feb-97	96	Feb-97	117
13	Jul-99	76	Feb-02	95	Oct-04	109
14	May-03	73	Apr-02	94	Jun-05	97
15	Feb-97	72	Jun-05	93	Apr-02	94
16	Jul-01	70	Apr-03	83	Apr-03	88

The design 1% AEP rainfall totals from IFD analysis for the 1-day, 2-day and 3-day durations are 106mm, 137mm and 153mm respectively. With the exception of March 1997 and August 1998, the rainfall events presented in Table 6-1 are all less than a 1% AEP event for the durations given. It should be noted however, that more intense rainfall bursts within the daily recorded period may have occurred, that may indeed represent a much more significant event. This cannot be confirmed however in the absence of a continuous rainfall record.

6.2.3 Event Analysis

For each of the 11 events selected for further analysis, a cumulative volumetric increase in lake storage was derived from the recorded lake levels using the stage-storage relationship of the lake system. The times and lake volumes were standardised to zero at the onset of the events to enable direct comparison, with the resulting time-volume plots presented in Figure 6-2.

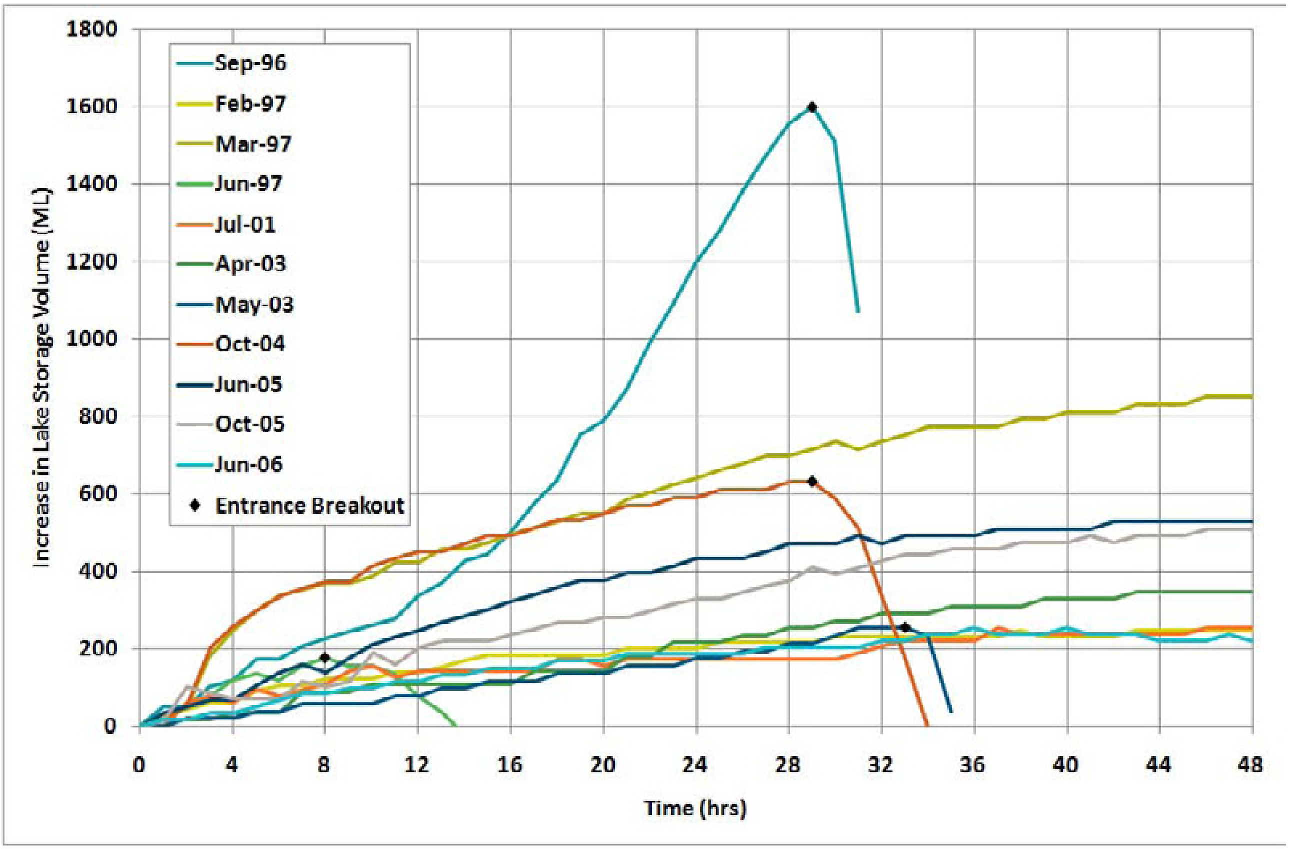


Figure 6-2 Volumetric Analysis of Recent Rainfall Events

With the exception of the September 1996 event, all of the other events presented in Figure 6-2 show a similar pattern of volumetric increase. There is generally a relatively rapid increase in lake volume over the first eight hours of the event, followed by around 24 hours of a more steady, constant rate of increase. This rate of increase then reduces at around 32 hours after the start of the events. It can also be seen that the September 1996, June 1997, May 2003 and October 2004 events have involved a breakout of the lake entrance, with recorded water levels rapidly decreasing following the breakout as the lake is drained.

The September 1996 event is the only event to have shown a significant difference in response. It shows a similar pattern of volume increase to the other events until around 11 hours after the onset of the event, after which there is a significant increase in the rate of rise in the lake volume attributable to catchment runoff. Further detailed discussion on the September 1996 event is given in Section 6.2.4.

The general pattern of lake filling for the events analysed is typical for long duration volume driven events in Tabourie Lake. As noted in Section 6.2.2, the events analysed generally represent less than 1% AEP daily rainfalls. Whilst peak catchment flows are relatively insignificant for these events, the runoff volumes generated are significant in relation to the available storage, resulting in potential flooding of the Tabourie Lake under closed entrance conditions.

The October 2005 event has been analysed in detail to further highlight the general characteristics of the observed catchment response for these typical long duration events. Rainfall data available for this event from the Ulladulla AWS continuous gauge provides a representative temporal pattern. The October 2005 event also showed a good match between daily rainfall values recorded at Tabourie Lake, Ulladulla AWS and Milton gauges, being 111mm, 102mm and 108mm respectively.

The volumetric analysis of the October 2005 event is presented in Figure 6-3. Included in the figure is the cumulative increase in lake volume during and subsequent to the rainfall event as derived from the MHL water level recorder data. Also shown for reference is the adopted rainfall hyetograph, shown as hourly rainfall totals throughout the event. A third series is presented which shows the cumulative increase in lake volume attributable to direct rainfall on the lake surface only.

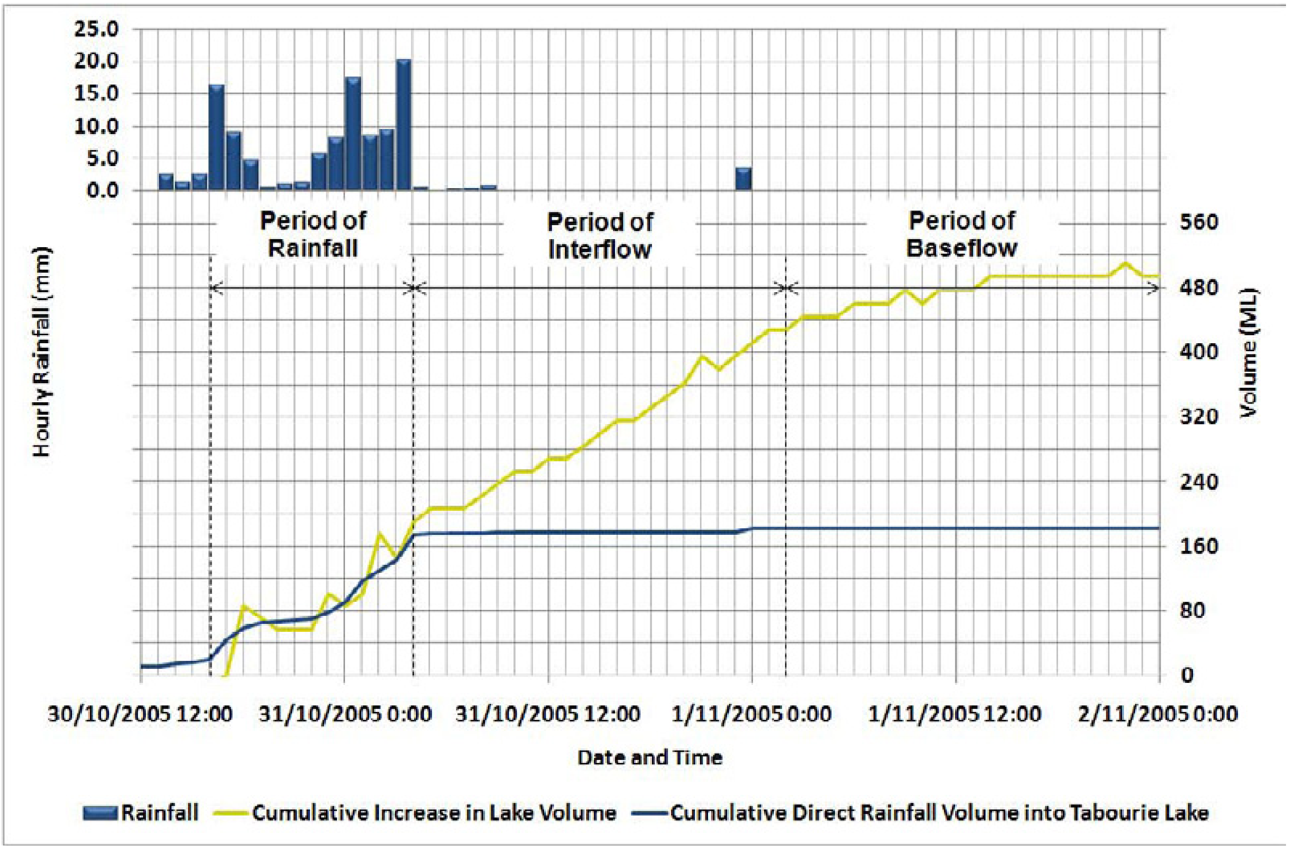


Figure 6-3 Volumetric Analysis of the October 2005 Event

The lake response in terms of the volumetric change over the event period is presented in Figure 6-3 as three distinct periods:

- Period of direct rainfall on Lake surface;
- Period of catchment controlled interflow; and
- Period of baseflow.

From Figure 6-3 it can be seen that during the period of rainfall there is a good match between the volume of direct rainfall to the lake storage (based on the recorded rainfall depths and surface area of the lake) and the observed increase in lake volume (based on the recorded water levels and the DEM stage-storage relationship). In all analysed events, the direct rainfall period provides all of the initial lake response.

The following period of around 24 hours after the rainfall event shows lake volume increasing at a fairly constant rate at around 11ML/h for October 2005. The additional volume of water entering the lake during this period is around 240ML. When compared to the total volume of rain falling on the catchment during the event, which is around 5500ML, this shows that around only 4% of the catchment rainfall reaches the lake within the day following the event.

The relatively small contribution of volume and the day-long period over which this volume is discharged to the lake are indicative of an interflow component. This effectively represents a slow release of water from the catchment to the Lake and is expected to be largely driven by available storage in the main contributing catchments.

Following this period of interflow there is a more gradual volume contribution, which can be considered a more general baseflow component. This can continue for a number of days following the main rainfall period.

For this October 2005 event, and the others in Figure 6-2 (with the exception of September 1996) that show a similar response, it would appear that there is an absence of any significant surface runoff contribution to the Lake via the main tributary channels. The observed response of the lake volume to rainfall events suggests that there is a significant storage component within the catchment, attenuating catchment runoff and regulating inflows to Tabourie Lake.

The topographic maps of the catchment show a large number of creek lines with limited connectivity throughout the catchments. These are typically located within the lower reaches of the main tributaries to Tabourie Lake and Tabourie Creek, where the topography of the floodplain is low-lying and flat. Inspection of the aerial photography also shows a number of areas of standing water and swamp in these locations.

These swamp areas are considered to provide enough storage to attenuate catchment flows for most rainfall events, as shown in Figure 6-2. Once the storage capacity of these swamps is exceeded, the excess runoff then rapidly increases the volume of water within the lake, as can be seen from the response of the September 1996 event. Antecedent conditions and the temporal pattern of rainfall may have a significant influence on the behaviour of this catchment storage.

The September 1996 is the only event to have shown any significant catchment surface runoff since 1995 during closed entrance periods. This event has been analysed further as discussed in 6.2.4 considering the potential capacity of the catchment storage and appropriate hydrological model parameters to simulate the catchment response.

6.2.4 September 1996 Event

The September 1996 event is associated with a significant east coast low that resulted in heavy rainfall and gale force winds over much of the NSW coast from the 31st August. The available continuous rainfall data from the Ulladulla AWS and Nowra gauges, along with the observed increase in lake volume (based on the recorded water levels and the DEM stage-storage relationship) are presented in Figure 6-4. During this event, the Ulladulla gauge ceased recording at around 08:00 on 31st August and the Nowra gauge ceased recording around three hours later. Accordingly, there is no local record of rainfall temporal pattern for the duration of the event.

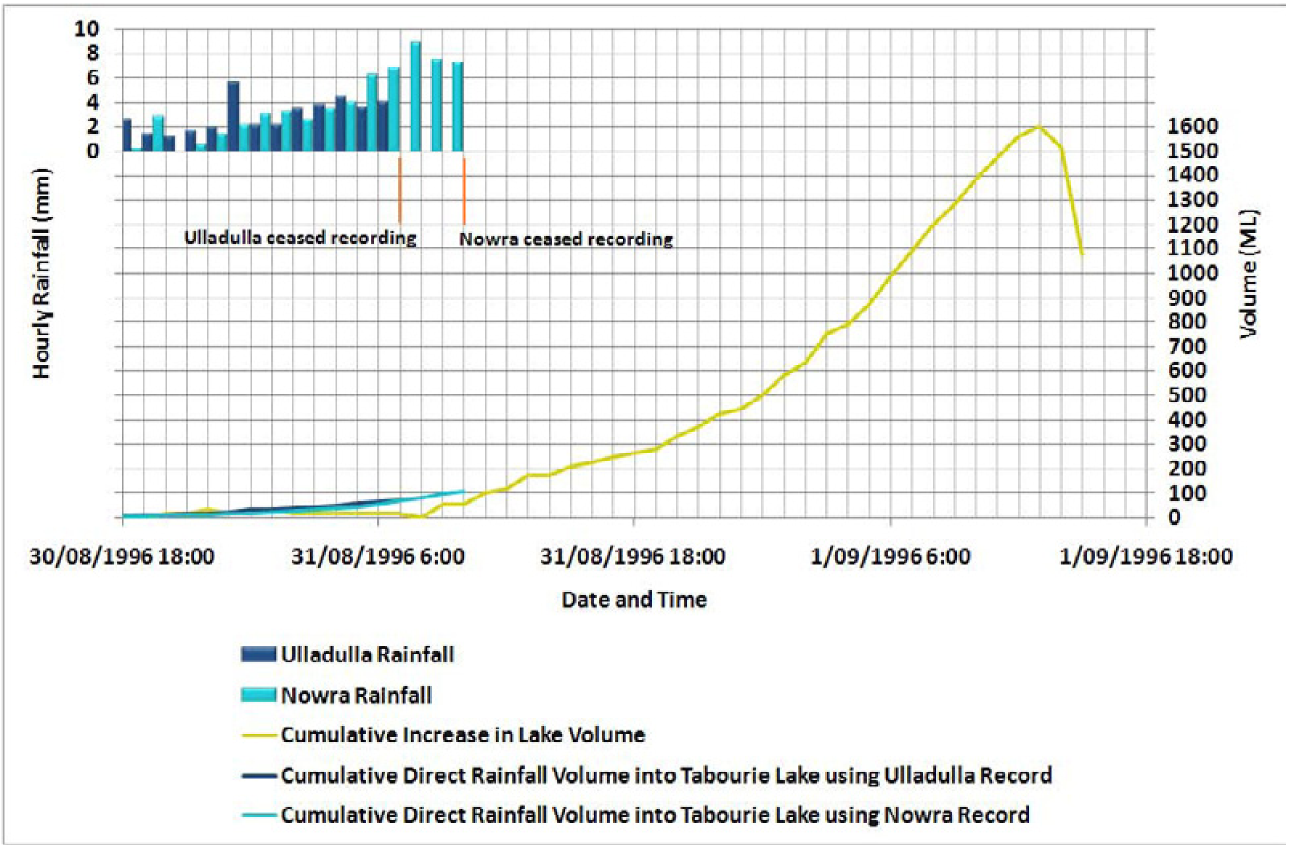


Figure 6-4 Volumetric Analysis of the September 1996 Event

The cumulative direct rainfall volumes into Tabourie Lake that have been derived from the available period of recorded rainfall data are also presented in Figure 6-4. There is a significant difference in the timing of the actual lake response compared with predicted lake response from the Ulladulla and Nowra rainfall. The observed increase in lake volume shows that the main period of rainfall falling directly on to the lake storage began later than that recorded at the gauges and was also more intense. This suggests that the temporal pattern of the rainfall recorded at both Ulladulla and Nowra differs from that within the Tabourie Lake catchment.

The daily read totals for the September 1996 event at the nearby gauges of Milton, Kioloa Old Post Office and Brooman (Carisbrook) are presented in Table 6-2. All gauges show a similar pattern in initial rainfall of 40-70mm recorded to 9:00am on 31st August 2006, followed by more substantial totals in the following 24-hours. The second day total recorded on 1st September 1996 at Tabourie Lake is considerably lower than the other gauge totals.

To enable further analysis of the event, even for comparative purposes in lieu of a detailed calibration, a representative temporal pattern needs to be derived for Tabourie Lake. The analysis of other volume driven rainfall events has demonstrated the relationship between direct rainfall and lake volume increase as derived for the available continuous water level record.

Table 6-2 Daily Rainfall Totals for the September 1996 Event

Rain Gauge	Rainfall (mm)	
	31/08/96	01/09/96
Tabourie Lake	48	89
Milton	55	137
Kioloa Old Post Office	41	122
Brooman (Carisbrook)	76	165

For the September 1996 event, the water level records show an initial lake level increase over a period of approximately 5 hours (from 7:00pm 30/08/1996) corresponding to a total rainfall of approximately 30mm. Following this initial period the lake level remains static prior to a second period of Lake rise from approximately 8:00am on 31/08/1996. The Lake response to this second period of direct rainfall is estimated to occur over approximately 7 hours.

Accordingly, the rainfall event spans the two daily recording periods to 9am on the 31st August and 1st September. The event rainfall total recorded at Tabourie Lake was 137mm over the two recording days. Following the initial estimated 30mm burst over 5 hours, the remaining 107mm of the event total is assumed to have occurred in the second 7-hour rainfall period. In the absence of recorded temporal data, the event hyetograph is assumed to comprise a 5-hour period of average intensity 6mm/hr followed by a second 7-hour period of 15mm/hr. This assumed hyetograph has been adopted for the event in the hydrological modelling discussed hereunder.

The adopted hyetograph and the recorded change in Lake volume for the September 1996 event is shown in Figure 6-4. The RAFTS hydrological model developed for the Tabourie Lake catchment (refer to Section 5.1) was used to simulate the catchment inflows to the Lake and corresponding storage changes. The general hydrological parameters adopted in the simulation were:

- Catchment properties as summarised in Table 5-1;
- Initial loss of 15mm and continuing loss 2.5mm/hr; and
- Bx storage routing factor = 1.0 (default).

The change in lake volume from the simulated catchment inflows is shown in Figure 6-5. Two modelled scenarios are presented with and without a provision for catchment storage. As discussed previously, catchment storage is considered to have a significant influence on the catchment response to rainfall and the volume and rate of inflows to Tabourie Lake. This influence is further illustrated by the simulated catchment response shown in Figure 6-5.

The model simulation undertaken with no catchment storage shows a relatively rapid lake response to rainfall. However, the simulated rise bears little resemblance to the actual recorded water level time series for the event. Simulated catchment inflows continue to elevate lake levels after the initial rainfall burst providing direct volume to the Lake. Additionally, after the second rainfall period there is minimal lag between surface runoff generated in the catchment and the corresponding increase in lake water levels, again in contrast to observed conditions.

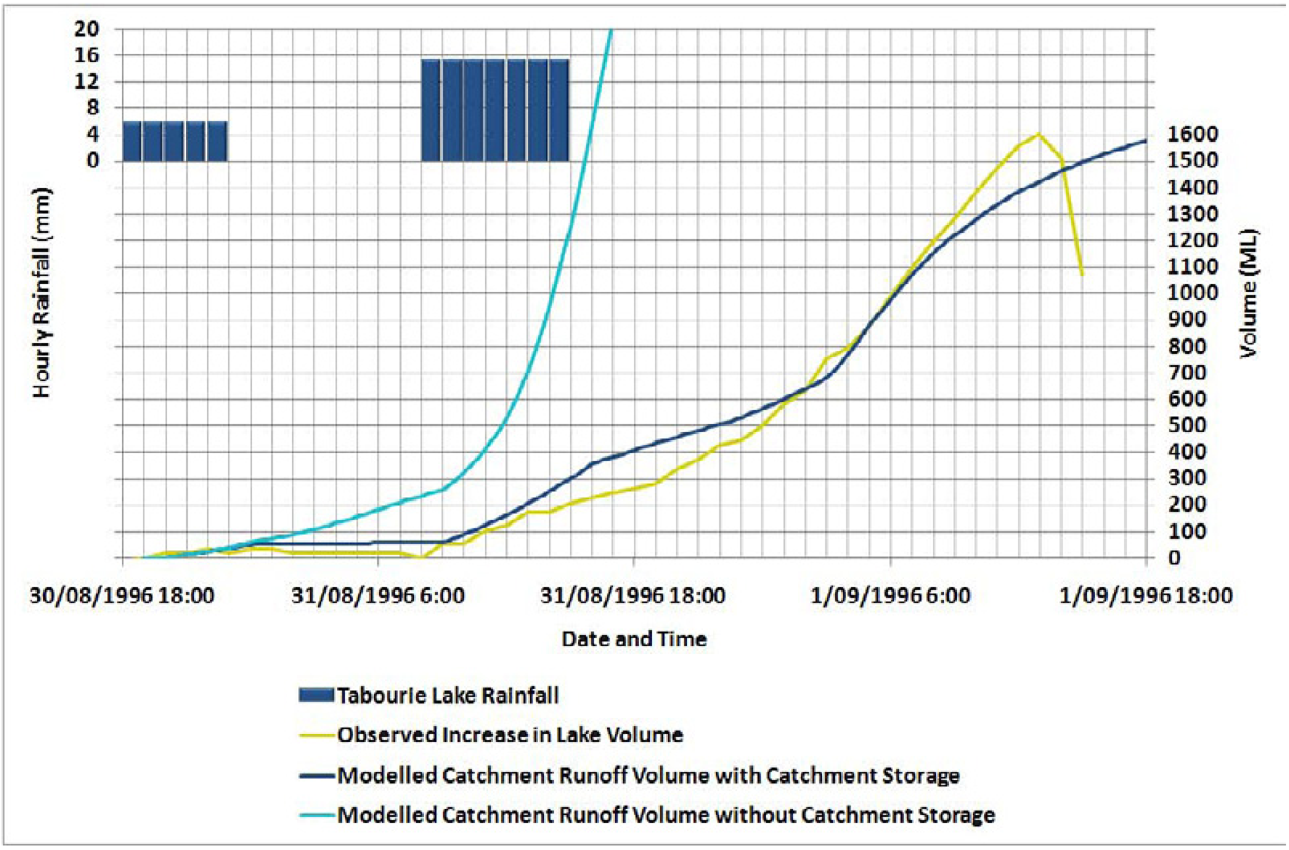


Figure 6-5 Comparison of Simulation and Observed Lake Response September 1996

Catchment storage is expected to have significant influence on the rainfall runoff rates and volumes to Tabourie Lake. For the 11 events for which a volumetric analysis was presented in section 6.2.3, only the September 1996 event showed a direct surface run-off response. Considering the October 2005 event discussed previously, no surface runoff response was observed for a daily rainfall total of 111mm, one of the highest daily rainfall totals recorded since 1995. There are numerous other events in the order of a 100mm daily rainfall (refer Table 6-1) which exhibited a similar lack in surface runoff response. It would appear therefore that the catchment has potential storage capacity to absorb up to the order of 110mm of daily rainfall, which limits direct surface runoff, and provides for a more controlled release or interflow to Tabourie Lake over an extended duration beyond the main rainfall period.

The sub-catchment properties within the RAFTS model were modified to incorporate a storage component with sufficient volume to contain up to 110mm of rainfall. This storage was applied at the lower parts of the catchment prior to discharge to the Lake. Stage versus volume relationships were derived for sub-catchments utilising available contour information, with spill levels set to provide for surface runoff for rainfall totals in excess of 110mm.

The results of the simulation with catchment storage included in the hydrological model show a significant improvement in comparing recorded Lake volume increases. The timing and rate of response are a reasonable representation of the actual event considering the uncertainties surrounding the hyetograph and catchment storage simplifications.

Given the lack of available calibration data, the assumptions required and inherent uncertainty in the model process, further refinement of the hydrological model is not warranted. The model simulations are indicative of relative catchment response and show the influence of storage in the catchment on low order rainfall events.

For design flood simulations however, further consideration is given to antecedent conditions and lead-in rainfall to the major design events. In the case of Tabourie Lake, the volumetric analysis generally shows continuous flow period of a few days following significant daily rainfall. It may therefore be appropriate to adopt conservative conditions for the major design event simulation in assuming a high initial catchment wetness and low catchment storage capacity at the start of the event.

6.3 Major Historical Flooding Events

The historical events analysed in the preceding section were limited to relatively minor magnitude events (<1% AEP) to investigate the hydrological characteristics of the catchment during closed entrance periods.

The largest recorded flood events in the catchment as discussed in Section 2.2.3 include February 1971, March 1975 and April 1988. The most reliable data in the form of surveyed flood levels exists for the March 1975 and April 1988 events. The data is however limited. Despite the lack of comprehensive data for these events, model calibration was attempted using the best available information.

Rainfall data for the March 1975 and April 1988 events is limited to daily read totals available at nearby gauges including Milton, Kioloa Old Post Office and Brooman - Carisbrook (refer to Figure 2-4 for gauge locations). Note that the continuous rainfall gauges at Ulladulla and Nowra commenced in 1992 and 1989 respectively. The March 1975 and April 1988 events were both long duration events occurring over a number of days. The recorded rainfall totals for the 1975 and 1988 events are summarised in Table 6-3 and Table 6-4 respectively.

The Milton gauge provides a complete record of recorded daily totals for each of the events in consideration and has been adopted as representative for the Tabourie Lake catchment. To gain an appreciation of the relative magnitude of the events, the rainfall totals for the 1-day, 2-day and 3-day durations is compared with the design IFD data for the Tabourie Lake catchment as shown in Figure 6-6.

Table 6-3 Daily Rainfall Totals for the March 1975 Event

Rain Gauge	Rainfall (mm)			
	10/03/75	11/03/75	12/03/75	13/03/75
Milton	125	229	51	45
Kioloa Old Post Office	75	210	62	61
Brooman (Carisbrook)	Station commenced 1979			

Table 6-4 Daily Rainfall Totals for the February 1988 Event

Rain Gauge	Rainfall (mm)		
	29/04/88	30/04/88	01/05/88
Milton	77	106	49
Kioloa Old Post Office	130 (2-day total)		Not recorded
Brooman (Carisbrook)	75	178	Not recorded

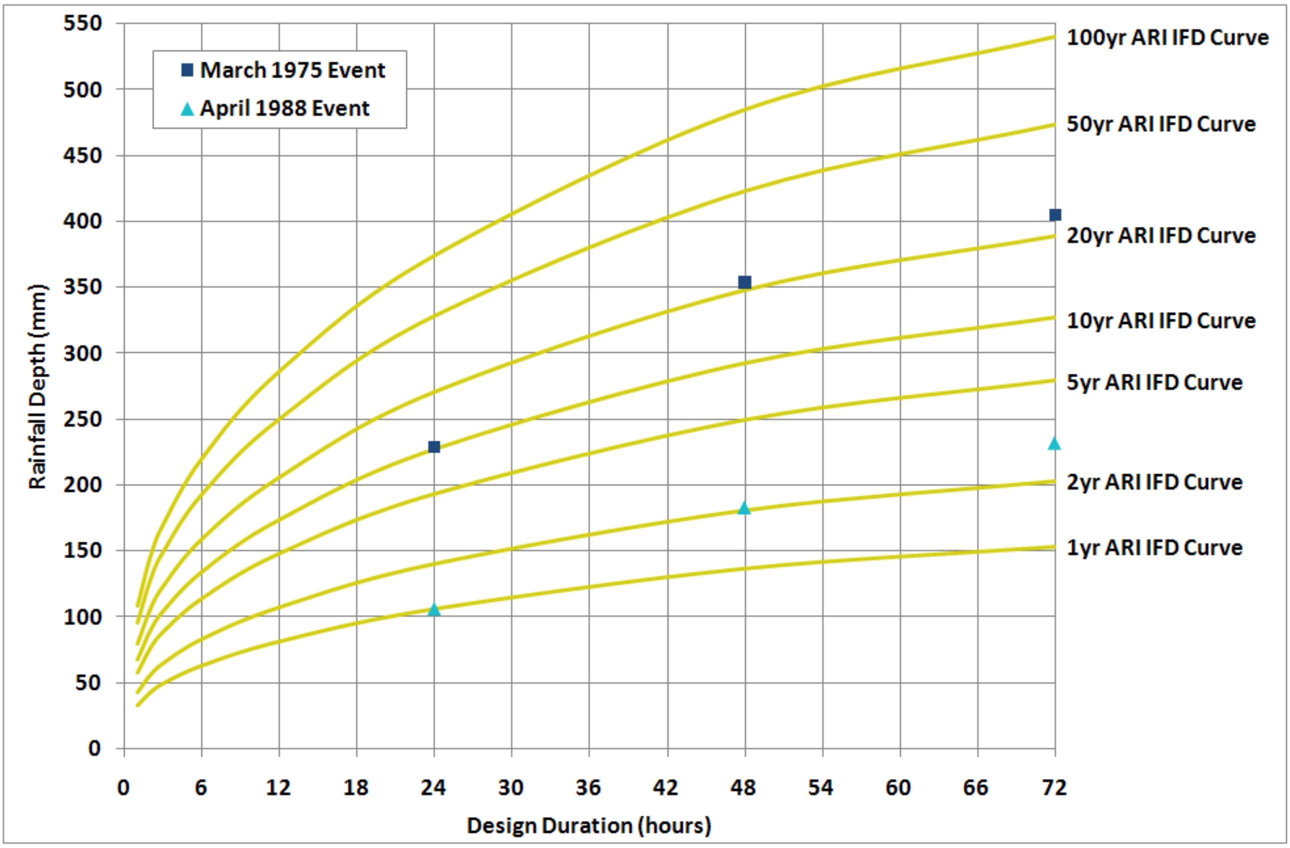


Figure 6-6 Comparison of 1988 and 1975 Event Rainfall with IFD Relationships

The recorded depth vs. duration profile for the March 1975 event shows it tracking above the design 5% AEP. The recorded depth for the 72 hour period of 405mm represents approximately 5% higher rainfall than the 5% AEP design rainfall depth. The April 1988 is a significantly lower magnitude event (<20% AEP) with the recorded depth for the 72 hour period of 232mm representing approximately 15% higher rainfall than the 50% AEP design rainfall depth.

6.3.1 March 1975 Event Simulation

The hydrological and hydraulic models developed for the study (refer Section 5) have been applied to simulate the March 1975 flood event in Tabourie Lake. There is significant uncertainty surrounding some of the key model inputs/parameters given the limited data available for the event. The most significant of these are considered to be:

- Event rainfall pattern;
- Entrance conditions; and
- Initial Lake water level.

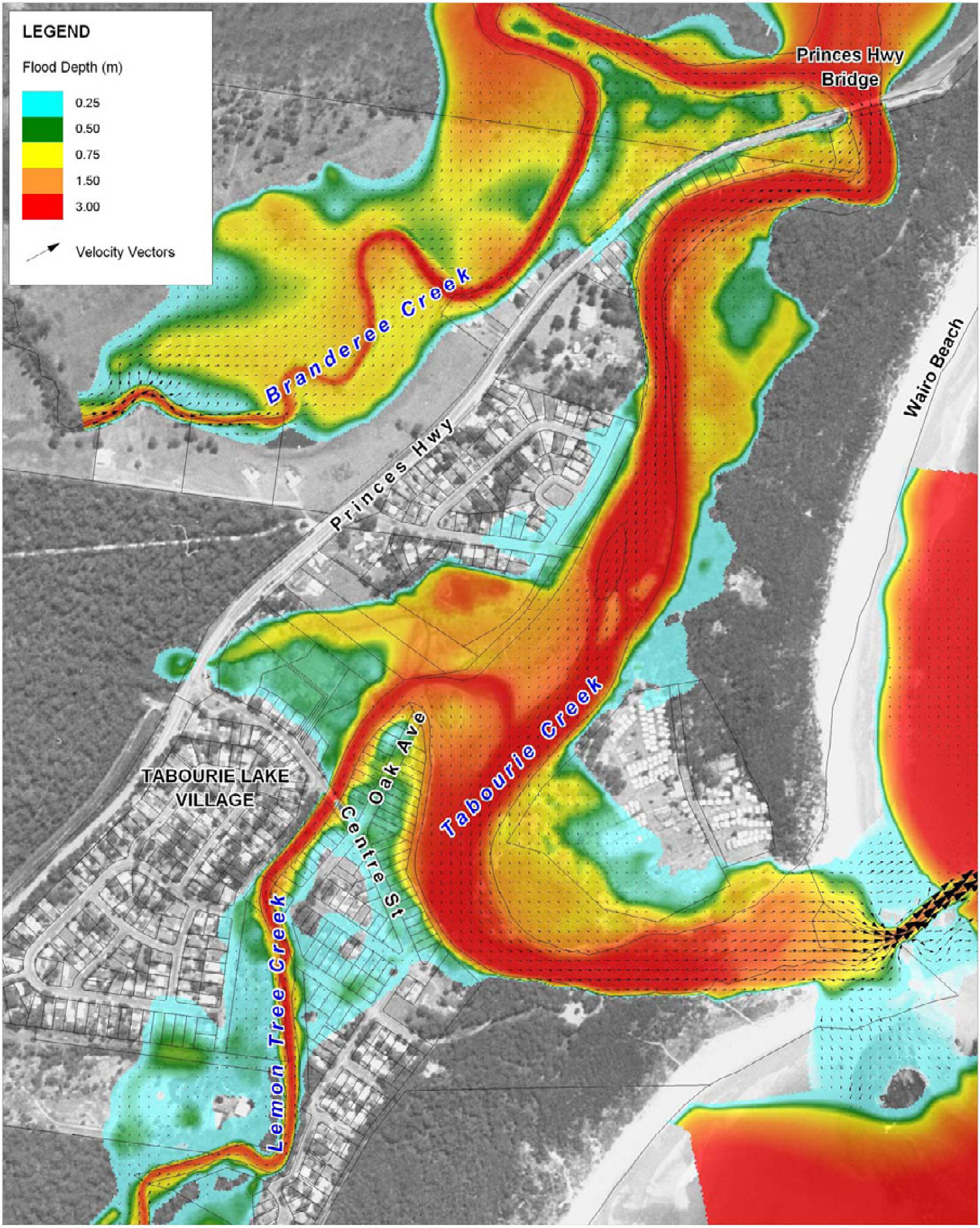
Despite no active rainfall gauge within the Tabourie Lake catchment for the March 1975 event, the recorded daily totals at Milton are expected to be indicative of the catchment rainfall. In the absence of a recorded temporal pattern, the design temporal pattern for a 72-hour event as presented in AR&R (2001) has been adopted for the event. Whilst assumed temporal patterns may have an influence on peak flows, the March 1975 event is largely a long duration volume driven event for the Tabourie Lake catchment, such that discrepancies in assumed and actual rainfall temporal pattern may be less significant.

The greatest influence on flood behaviour and perhaps the most significant uncertainty in terms of the model configuration is the entrance berm conditions at the time of the event. For both the March 1975 and August 1988 events the entrance was closed, however, natural breakouts are understood to have occurred during the events. Given the dynamic nature of the berm building process, it is impossible to predict berm heights for calibration event periods from limited available data such as historical aerial photos or survey.

Simulations for two different berm height configurations have been undertaken to provide some indication on the sensitivity of the model results.

Given the relatively small available storage volume in the Lake compared with the catchment size and expected runoff volumes during flood events, initial lake levels are not expected to have a major influence on peak flood water levels. However, from a design event perspective, the initial lake condition may have a significant influence on available warning times, particularly for the short duration events in which rapid lake water rise is expected. Nevertheless, for the calibration events considered where no data on initial lake levels is available, the sensitivity of the simulated flood behaviour with varying initial lake water level is demonstrated.

A sample output from the model simulation of the March 1975 event is shown in Figure 6-7. The output shows flood depths and velocity patterns at the peak of the event (peak Lake flood level). The output is shown at the initiation of the berm breach utilising the Tuflow morphological module. Evident is the formation of the initial breakout channel with high flow velocity extending through the channel to beyond the beachfront. Elsewhere, flow velocities are more sedate with higher velocities confined to the in-bank regions of Tabourie Creek. Minor inundation of low-lying overbank areas within Tabourie Lake village is shown including inundation of the Princes Highway.



Title:
Sample Tuflow Model Output - 1975 Event with Berm Breach

Figure: 6-7

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Approx. Scale

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www.wbmpl.com.au

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The simulated peak flood level for the March 1975 event in Tabourie Lake village for the range of entrance configurations considered is shown in Table 6-5. The fixed berm condition provides for no scour of the bed or berm, whilst the mobile berm implies use of the morphological module to simulate the dynamic scour processes at the entrance. The recorded peak flood level for the event is 2.4m AHD.

Table 6-5 Entrance Berm Sensitivity March 1975 Event

Berm Condition	Peak Flood Level
Fixed Berm (1.8m AHD saddle / 2.2m AHD crest)	2.7
Mobile Berm (1.8m AHD saddle / 2.2m AHD crest)	2.4
Mobile Berm (1.5m AHD saddle / 1.7m AHD crest)	2.1

For the first two scenarios presented in Table 6-5 with similar starting berm heights, it can be seen that the initiation and propagation of a natural breakout channel provides for some peak flood water level reduction in Tabourie Lake. Of the modelled scenarios, the initial entrance berm crest of 2.2m AHD and propagation of a natural breakout provides for similar peak water levels as recorded in Tabourie Lake. Given the influence of the berm condition on peak flood level, it is likely that the actual berm crest level at the time of the event was indeed of the order of 2.2m AHD.

Berm height analysis undertaken by Wainwright and Baldock (2009), as a research component of the Tabourie Lake Flood Study, indicates that a berm height in excess of 1.7m AHD can be anticipated for 90% of the time during closed entrance periods. The berm height set at this level represents a lower limit for flood analysis of closed entrance periods. Conversely, the 2.2m AHD berm crest height is expected to be exceeded less than 1% of the time during closed entrance periods, and accordingly may represent an upper limit for flood analysis. For extended periods of closure however, this berm crest level is likely to be achieved.

Both the simulated flood conditions and berm height probability assessment suggest the adopted 2.2m crest height to be appropriate for the March 1975 event. This provides for agreement between the simulated and observed peak flood condition for the 1975 event. However the sensitivity of the simulated peak flood water level to the initial berm condition is significant.

Further sensitivity testing on the simulated March 1975 event has been undertaken for the initial lake water level condition. Analysis of the MHL water level record for Tabourie Lake indicates a long-term average water level of the order of 0.7m AHD. This level has been adopted for the base scenario in simulating the March 1975 event. Additional simulations with significantly lower (0.2m AHD) and significantly higher (1.2m AHD) initial lake levels have been undertaken.

There is no change in simulated peak flood level for the 1975 event between the different initial lake water level scenarios. Each scenario provided for the simulated peak water level of 2.4m AHD as per the second simulation presented in Table 6-5 (adopted as the base run). The available Lake storage is small in comparison to the catchment generated flood water volume. Accordingly, the lake fills relatively quickly in terms of the catchment inflow hydrograph until the entrance berm is breached or overtopped, thereby providing the similar peak flood water level. The only influence of the initial lake level is the relative timing of the storage filling and occurrence of the breach.

The result of the sensitivity analysis on initial lake water level is shown in Figure 6-8. The combined inflow hydrograph to Tabourie Lake from all contributing catchments is shown in conjunction with the cumulative catchment runoff volume. The relative timing of the breaching of the entrance berm is shown for the three initial lake water level scenarios. It is evident that the initial Lake water level, and hence available Lake storage at the start of the rainfall event is relatively insignificant for a flood event of this magnitude. There is a minor shift in the timing of the breach which may be more significant for short duration events where effective warning times are minimal.

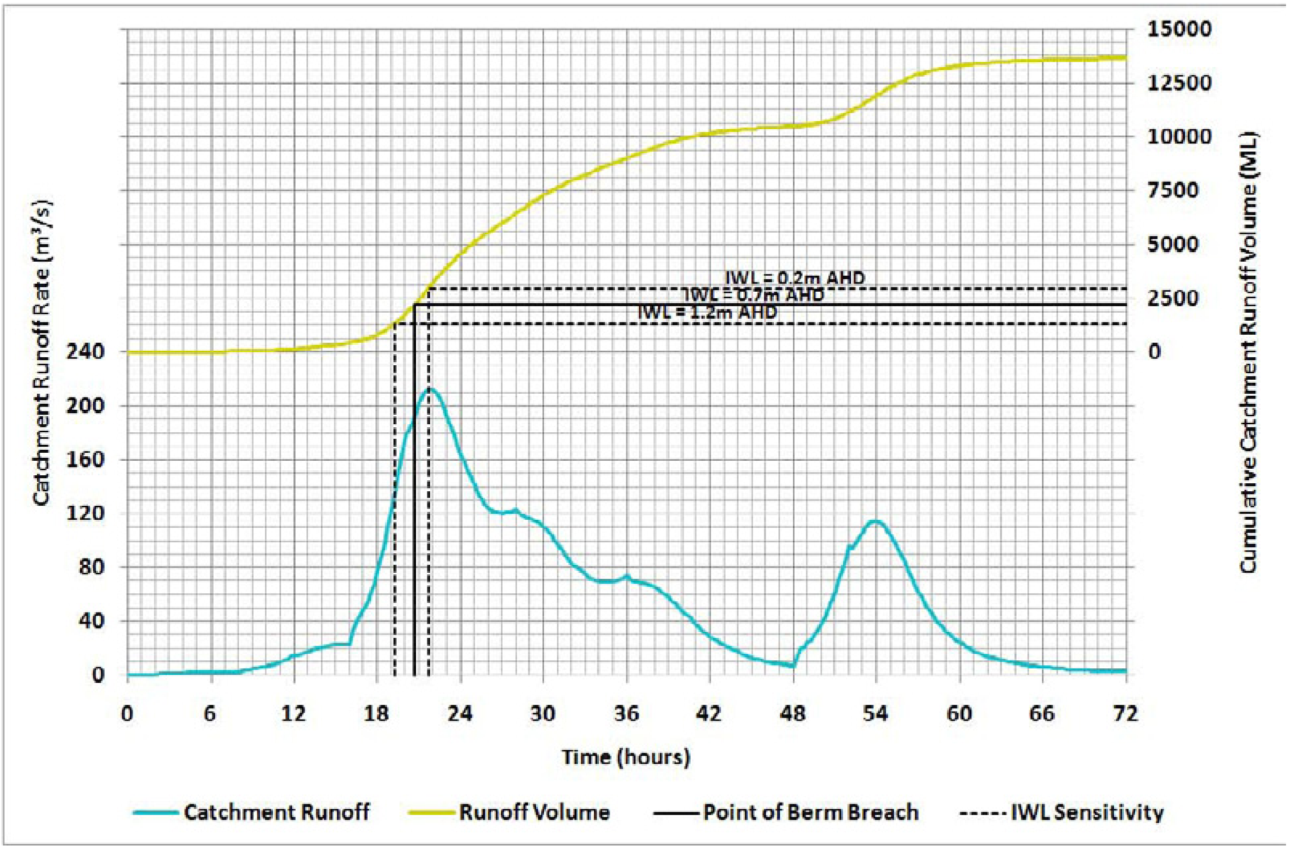


Figure 6-8 Sensitivity of the March 1975 Event Simulation to Initial Lake Water Level

The simulated peak inflow to Tabourie Lake for the event is approximately 210 m^3/s. The event was estimated to be of the order of a 5% AEP return period when considering the design rainfall data.

Given the lack of available data it is difficult to confirm the model simulations are an accurate representation of the actual flood behaviour for the March 1975 event. Nevertheless, the calibration simulation and sensitivity tests have enabled an understanding of the catchment flood behaviour to be realised, in particular the key mechanisms driving peak flood conditions in the Lake.

6.4.1 April 1988 Event

A second historical flood event has been simulated using the developed models for further model validation and analysis.

As for the March 1975 event, despite no active rainfall gauge within the Tabourie Lake catchment for the April 1988 event, the recorded daily totals at Milton are expected to be indicative of the catchment rainfall. In the absence of a recorded temporal pattern, the design temporal pattern for a 72-hour event as presented in AR&R (2001) has been adopted for the event.

The simulated peak flood level for the April 1998 event in Tabourie Lake village for the range of entrance configurations considered (as per the March 1975 event simulations) is shown in Table 6-5. The fixed berm condition provides for no scour of the bed or berm, whilst the mobile berm implies use of the morphological module to simulate the dynamic scour processes at the entrance. The recorded peak flood level in Tabourie Lake village for the event is 2.0m AHD.

Table 6-6 Entrance Berm Sensitivity April 1988 Event

Berm Condition	Peak Flood Level
Fixed Berm Height (1.8m AHD saddle / 2.2m AHD crest)	2.3
Mobile Berm Height (1.8m AHD saddle / 2.2m AHD crest)	2.2
Mobile Berm Height (1.5m AHD saddle / 1.7m AHD crest)	1.9

As previously indicated, the peak flood level is sensitive to the adopted entrance berm condition. On assuming other model variables and system representation is suitable, the results suggest that a berm crest level between 1.7m AHD and 2.2m AHD is applicable for the start of the August 1998 event. As per the analysis of Wainwright and Baldock (2009), these levels are within the expected range of berm crest levels during extended entrance closure periods.

Similar tests on initial lake water level sensitivity were undertaken for the April 1988 event. The results of the initial lake water level sensitivity for the April 1988 event are shown in Figure 6-9.

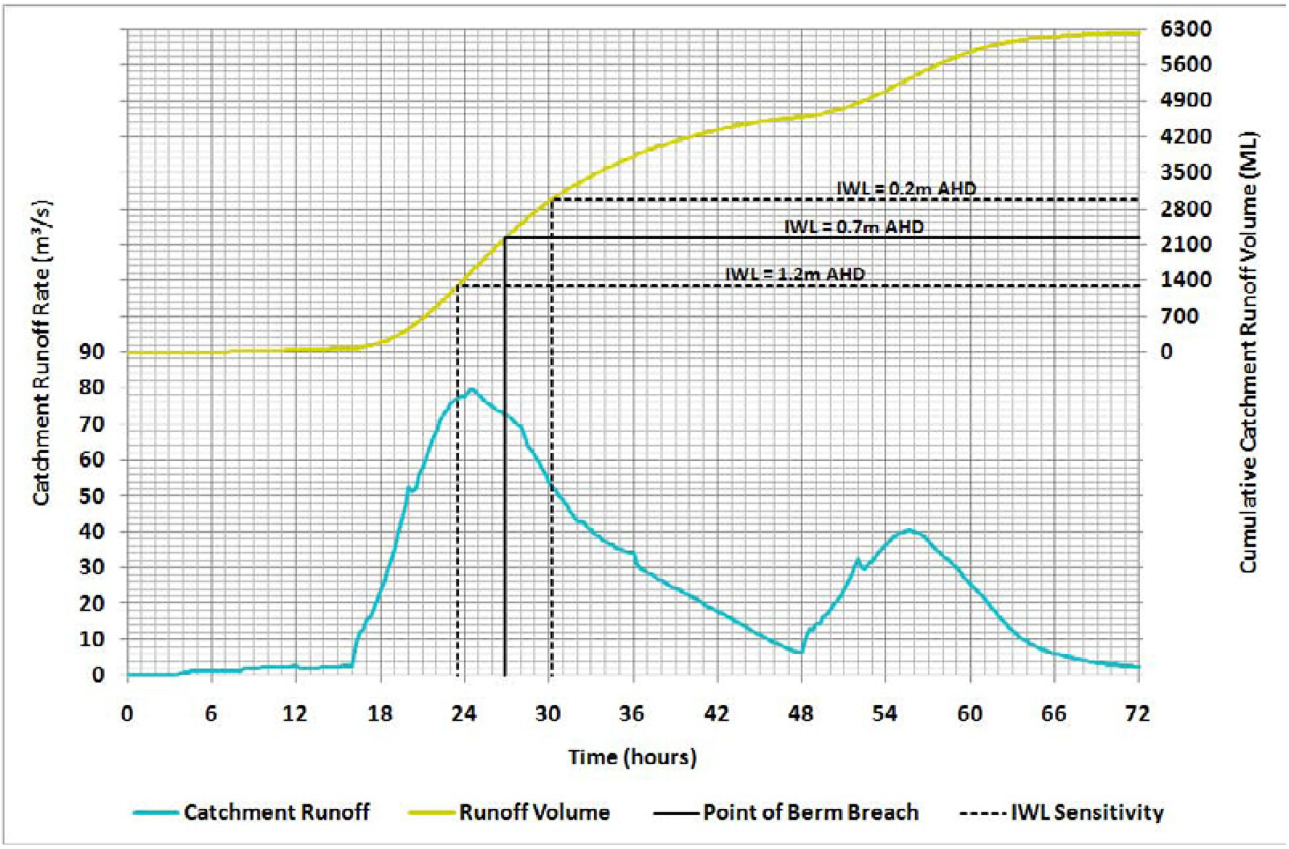


Figure 6-9 Sensitivity of the April 1988 Event Simulation to Initial Lake Water Level

As indicated previously, the available lake storage is relatively small in comparison to generated runoff volumes from the catchment. The April 1988 event is estimated as having an equivalent annual exceedance probability of less than 20%. However, even for such a relatively minor flood event magnitude, the initial water level condition has minimal influence on the peak flood conditions given the generated runoff volumes. The simulated peak flood levels are largely controlled by the berm height. Once the storage is filled and the berm breached, peak flood water levels are effectively capped near the breach level. Accordingly, the simulated peak flood level in Tabourie Lake village was the same for all three initial water level runs.

7 DESIGN FLOOD CONDITIONS

Design floods are hypothetical floods used for planning and floodplain management investigations. They are based on having a probability of occurrence specified as Annual Exceedance Probability (AEP) expressed as a percentage.

Refer to Table 7-1 for a definition of AEP.

Table 7-1 Design Flood Terminology

AEP	Comments
0.5%	A hypothetical flood or combination of floods which represent the worst case scenario with a 0.5% chance of occurring in any given year.
1%	As for the 0.5% AEP flood but with a 1% probability.
2%	As for the 0.5% AEP flood but with a 2% probability.
5%	As for the 0.5% AEP flood but with a 5% probability.
10%	As for the 0.5% AEP flood but with a 10% probability.
Extreme Flood / PMF[1]	A hypothetical flood or combination of floods which represent an extreme scenario.

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

In accordance with Council's brief, the design events to be simulated include the 20% AEP, 5% AEP, 2% AEP, 1% AEP and PMF event. The 1% AEP flood is generally used as a reference flood for development planning and control.

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

- Design rainfall parameters (rainfall depth, temporal pattern and spatial distribution). These inputs drive the hydrological model from which design flow hydrographs will be extracted as inputs to the hydraulic model;
- Design entrance channel geometry. As discussed, the entrance condition is probably the most significant feature in terms of flood water level controls in Tabourie Lake. As outlined in the Department of Environment, Climate Change and Water's (DECCW's) Draft Flood Risk Management Guide: Incorporating sea level rise benchmarks in flood risk assessments (2009), both closed and open entrance scenarios are to be modelled;
- Design downstream ocean boundary levels. A fully scoured entrance condition will provide for the critical case for ocean flooding, whilst for closed condition and intermediate scouring, coincident fluvial and tidal conditions may dictate flooding;
- The impact of future climate change on berm heights, ocean levels and catchment inflows.

7.1 Simulated Design Events

In consultation with Council a suite of design event scenarios was defined that is most suitable for future floodplain management planning in Tabourie Lake. Consideration was given to flood events driven by both catchment and ocean processes. The potential impact of climate change on flood behaviour within Tabourie Lake has also been considered.

7.1.1 Catchment Derived Flood Events

A range of design events was defined to model the behaviour of catchment derived flooding within Tabourie Lake including the 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF events. An overview of adopted model conditions for these design events is presented in Table 7-2. The adopted storm durations are discussed in Section 7.2.4.

Table 7-2 Design Model Runs for Catchment Derived Flood Events

Design Flood	Rainfall	Berm Geometry	Ocean Boundary Peak Water Level (m AHD)	Initial Water Level
20% AEP	20% AEP 9h duration	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
5% AEP	5% AEP 9h duration	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
2% AEP	2% AEP 9h duration	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
1% AEP	1% AEP 9h duration	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
PMF	PMP 6h duration	Closed (2.0m AHD Berm Saddle)	2.60 (0.5% AEP)	0.60 (Average Closed WL)

7.1.2 Ocean Derived Flood Events

A range of design events was defined to model the behaviour of ocean derived flooding within Tabourie Lake including the 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP and 0.5% AEP events. An overview of adopted model conditions for these design events is presented in Table 7-3.

Table 7-3 Design Model Runs for Ocean Derived Flood Events

Design Flood	Rainfall	Berm Geometry	Ocean Boundary Peak Water Level (m AHD)	Initial Water Level
20% AEP	20% AEP 9h duration	Open (fully scoured entrance)	1.89 (20% AEP)	0.25 (Average Open WL)
5% AEP	20% AEP 9h duration	Open (fully scoured entrance)	2.25 (5% AEP)	0.25 (Average Open WL)
2% AEP	20% AEP 9h duration	Open (fully scoured entrance)	2.45 (2% AEP)	0.25 (Average Open WL)
1% AEP	20% AEP 9h duration	Open (fully scoured entrance)	2.510 (1% AEP)	0.25 (Average Open WL)

7.1.3 Climate Change

A range of design events was defined to model the potential impacts of future climatic change within the Tabourie Lake catchment. There are three outcomes of current climate change predictions which may have a significant impact of flood behaviour within Tabourie Lake:

- Future sea-level rise;
- Elevated berm heights, themselves a function of sea-level rise;
- Increased extreme rainfall intensities.

These three factors were considered separately and in combination with each other for two future horizons, 2050 and 2100. The outcomes of these climate change considerations will help understand the potential changes in future flood behaviour and how to best plan for future development within the catchment. The design events for which climate change impacts were considered were therefore focussed on the main planning events: the 5% AEP, 1% AEP and PMF events. An overview of adopted model conditions for these climate change events is presented in Table 7-4.

7.2 Design Rainfall

Design rainfall parameters are derived from standard procedures defined in AR&R (2001) which are based on statistical analysis of recorded rainfall data across Australia. The derivation of location specific design rainfall parameters (e.g. rainfall depth and temporal pattern) for the Tabourie Lake catchment is presented below.

7.2.1 Rainfall Depths

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

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

A range of storm durations were modelled in order to identify the critical storm duration for design event flooding in the catchment. Design durations considered included the 1-hour, 3-hour, 6-hour, 9-hour, 12-hour, 24-hour, 48-hour and 72-hour durations.

Table 7-4 Design Model Runs for Climate Change Considerations

Design Flood	Rainfall	Berm Geometry	Ocean Boundary Peak Water Level (m AHD)	Initial Water Level
Catchment Derived Flood Events – 2050 Planning Horizon				
5% AEP	5% AEP 9h duration	Closed (2.5m AHD Berm Saddle)	1.00 (Regular Neap Tide +0.40m to 2050)	1.00 (Average Closed WL +0.40m to 2050)
1% AEP	1% AEP 9h duration	Closed (2.5m AHD Berm Saddle)	1.00 (Regular Neap Tide +0.40m to 2050)	1.00 (Average Closed WL +0.40m to 2050)
PMF	PMP 6h duration	Closed (2.5m AHD Berm Saddle)	1.00 (Regular Neap Tide +0.40m to 2050)	1.00 (Average Closed WL +0.40m to 2050)
Catchment Derived Flood Events – 2100 Planning Horizon				
5% AEP	5% AEP 9h duration	Closed (3.0m AHD Berm Saddle)	1.50 (Regular Neap Tide +0.90m to 2100)	1.50 (Average Closed WL +0.90m to 2100)
1% AEP	1% AEP 9h duration	Closed (3.0m AHD Berm Saddle)	1.50 (Regular Neap Tide +0.90m to 2100)	1.50 (Average Closed WL +0.90m to 2100)
PMF	PMP 6h duration	Closed (3.0m AHD Berm Saddle)	1.50 (Regular Neap Tide +0.90m to 2100)	1.50 (Average Closed WL +0.90m to 2100)
Ocean Derived Flood Events – 2050 Planning Horizon				
5% AEP	20% AEP 9h duration	Open (fully scoured entrance)	2.65 (5% AEP +0.4m to 2050)	0.65 (Average Open WL +0.40m to 2050)
1% AEP	20% AEP 9h duration	Open (fully scoured entrance)	3.00 (1% AEP +0.4m to 2050)	0.65 (Average Open WL +0.40m to 2050)
PMF	20% AEP 9h duration	Open (fully scoured entrance)	3.30 (0.5% AEP +0.4m to 2050)	0.65 (Average Open WL +0.40m to 2050)
Ocean Derived Flood Events – 2100 Planning Horizon				
5% AEP	20% AEP 9h duration	Open (fully scoured entrance)	3.15 (5% AEP +0.9m to 2100)	1.15 (Average Open WL +0.90m to 2100)
1% AEP	20% AEP 9h duration	Open (fully scoured entrance)	3.50 (1% AEP +0.9m to 2100)	1.15 (Average Open WL +0.90m to 2100)
PMF	20% AEP 9h duration	Open (fully scoured entrance)	3.80 (0.5% AEP +0.9m to 2100)	1.15 (Average Open WL +0.90m to 2100)
Catchment Derived Flood Events +10% Rainfall – Current Planning Horizon				
5% AEP	5% AEP 9h +10% intensity	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
1% AEP	1% AEP 9h +10% intensity	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
PMF	PMP 6h +10% intensity	Closed (2.0m AHD Berm Saddle)	0.60 (Regular Neap Tide)	0.60 (Average Closed WL)
Catchment Derived Flood Events +10% Rainfall – 2100 Planning Horizon				
5% AEP	5% AEP 9h +10% intensity	Closed (3.0m AHD Berm Saddle)	1.5 (Regular Neap Tide +0.9m to 2100)	1.50 (Average Closed WL +0.90m to 2100)
1% AEP	1% AEP 9h +10% intensity	Closed (3.0m AHD Berm Saddle)	1.5 (Regular Neap Tide +0.9m to 2100)	1.50 (Average Closed WL +0.90m to 2100)
PMF	PMP 6h +10% intensity	Closed (3.0m AHD Berm Saddle)	1.5 (Regular Neap Tide +0.9m to 2100)	1.50 (Average Closed WL +0.90m to 2100)

Table 7-5 shows the average design rainfall intensities based on AR&R adopted for the modelled events. The full IFD table with durations from 5-minutes to 72-hours derived for the Tabourie Lake catchment is included in Appendix C.

Table 7-5 Average Design Rainfall Intensities (mm/hr)

Duration (hours)	Design Event Frequency					
	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP
1	58	67	79	96	109	124
3	29.8	34.9	41.3	50	57	63
6	19.1	22.4	26.6	32.3	36.8	41.5
9	14.9	17.4	20.7	25.1	28.6	32.3
12	12.4	14.5	17.2	20.9	23.9	27
24	8.08	9.48	11.3	13.7	15.6	17.7
48	5.21	6.10	7.26	8.82	10.1	11.3
72	3.89	4.55	5.41	6.58	7.50	8.50

7.2.2 Temporal Patterns

The IFD data presented in Table 7-5 provides for the average intensity (or total depth) that occurs over a given storm duration. Temporal patterns are required to define what percentage of the total rainfall depth occurs over a given time interval throughout the storm duration. The temporal patterns adopted in the current study are based on the standard patterns presented in AR&R (2001).

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

7.2.3 Rainfall Losses

The hydrologic model parameters adopted for the design floods were similar to those used in the hydrologic model calibration and verification. For the initial and continuing rainfall losses, values of 15mm and 2.5mm/h were used. These are consistent with the recommended ranges for design event losses in AR&R (2001).

7.2.4 Critical Duration

A series of model runs were carried out to identify the critical storm duration for the catchment. Outputs from the RAFTS hydrological model indicate that the maximum peak inflows are derived when using a storm duration of nine hours.

A series of hydraulic model runs were also undertaken to identify any impacts of the entrance conditions on the critical storm duration for obtaining peak water levels within Tabourie Creek. With a closed entrance condition the critical storm duration driving peak water levels was found to be the 12 hour duration.

The critical storm duration required to produce the maximum peak water levels within Tabourie Creek is driven by the available storage behind the entrance berm prior to the commencing of a breach. The available storage will be influenced by a number of factors including flood event magnitude, berm height and initial water levels. The required design model runs will involve a range of values adopted for each of these factors. Therefore, it is appropriate to use the storm duration which produces the highest model inflows, i.e. the nine hour storm duration. This will provide consistency throughout the range of design model runs. It will also provide maximum peak water levels for reaches of Branderee Creek and Lemon Tree Creek which are upstream from the influence of the lake storage.

The difference in peak water levels modelled in Tabourie Creek, under a closed entrance condition, for the nine hour and 12 hour storm durations were largely within 20mm of each other. Therefore, adopting the nine hour storm duration will not result in a significant underestimation of potential peak water levels within Tabourie Creek. The uncertainties associated with adopted values for design berm heights and initial water levels will exert a greater influence on modelled peak water levels than this margin of difference.

The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology. The critical storm using this method was found to be the six hour duration.

7.2.5 Climate Change Impact on Design Rainfall

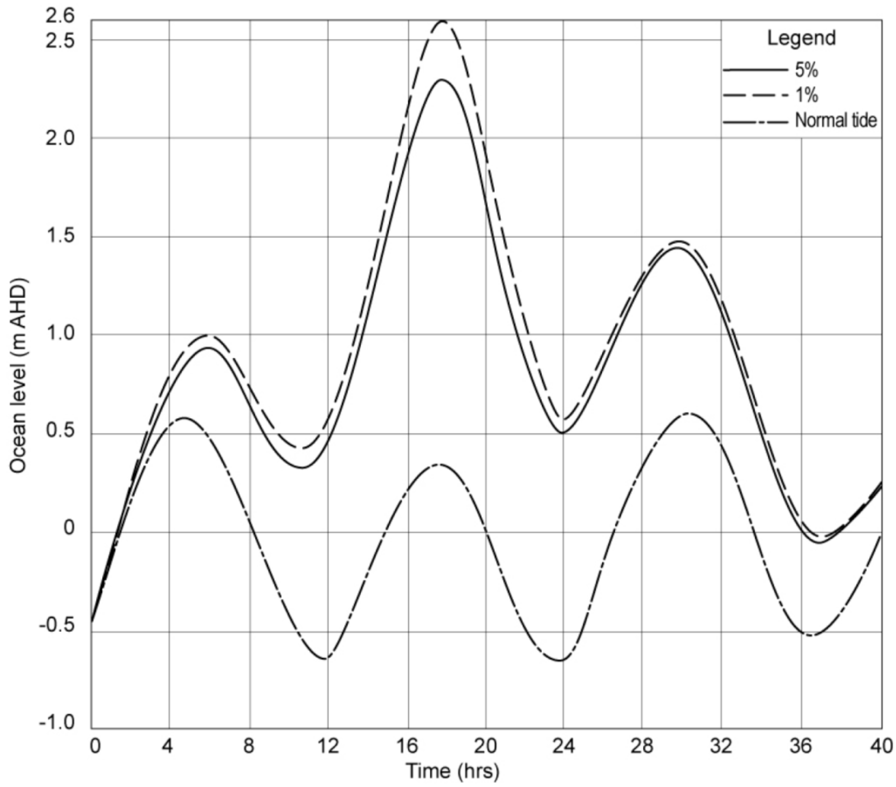
Current guidelines predict that a likely outcome of future climatic change will be an increase in extreme rainfall intensities. Climate Change in New South Wales (CSIRO, 2004) provides projected increases in annual extreme rainfall intensities for south-east NSW of 7% and 5%, for the years 2030 and 2070 respectively. The summer extreme rainfall intensities are projected to increase by 12% and 10% for the years 2030 and 2070 respectively. These figures are based on a 2.5% AEP 24h duration rainfall event. Based on these guidelines a design rainfall intensity increase of 10% was selected as being appropriate for assessing the potential impact of climate change on design rainfall in the Tabourie Lake catchment.

7.3 Design Ocean Boundary

Design ocean boundaries for use in flood risk assessments are recommended by Appendix A of the Draft Flood Risk Management Guide (DECCW, 2009). This appendix was formerly Guideline 5 of Ocean Boundary Conditions for Hydraulic Flood Modelling. The design ocean boundaries from Figure 3 of this document are presented in Figure 7-1. The recommended normal ocean boundary has been adopted for the catchment derived flood events. However, for the derivation of ocean derived flood event boundaries a more detailed analysis was undertaken. This analysis is discussed in Section 7.3.2.

7.3.1 Catchment Derived Flood Events

The adopted tidal boundary for catchment derived flood events was based on the normal tide recommendation and is shown in Figure 7-2. The timing of the 0.6m AHD peak water level was adjusted to coincide with the peak catchment inflow, which occurs at between T=10 and T=11 hours.



Source: Figure3, Appendix A, Draft Flood Risk Management Guide (DECCW, 2009)

Figure 7-1 DECCW Recommended Design Ocean Boundaries

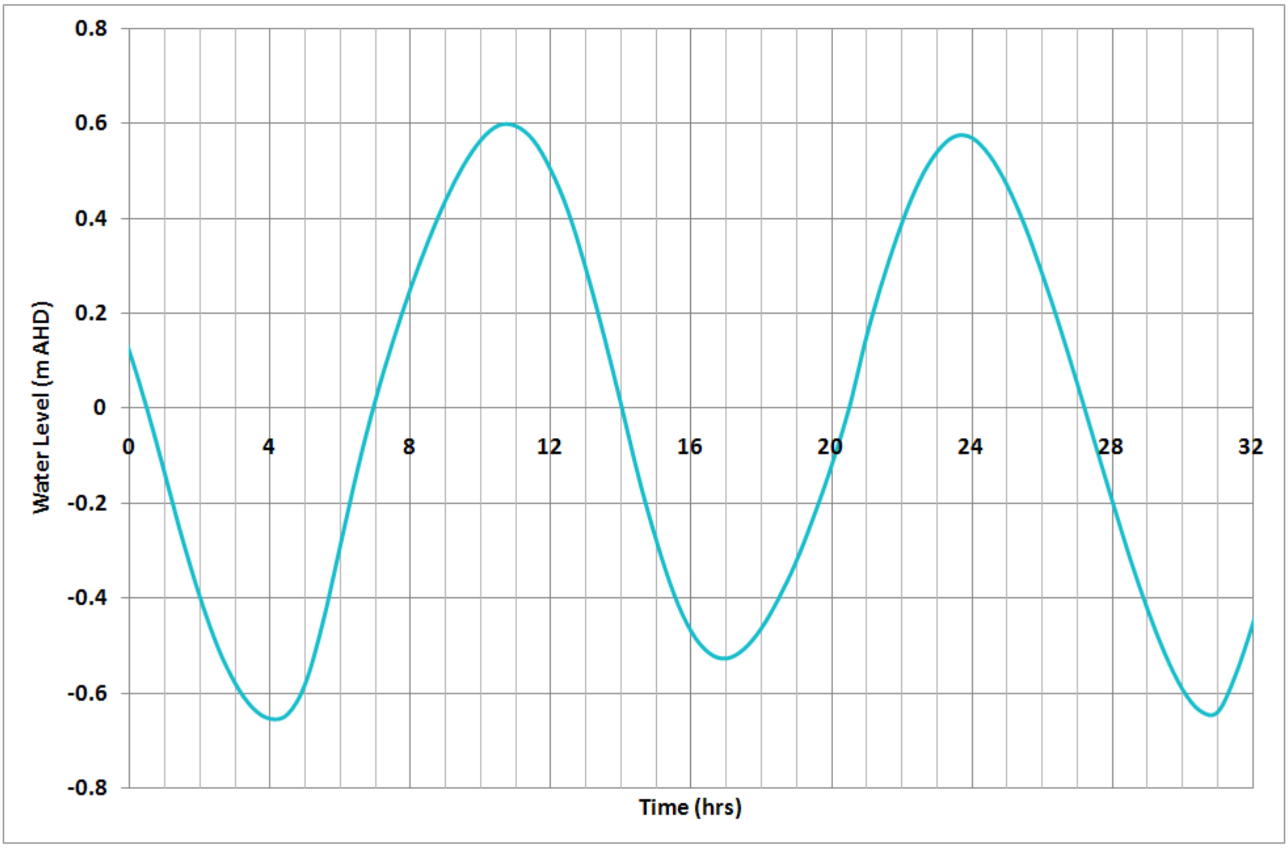


Figure 7-2 Design Ocean Boundary – Regular Neap Tide

7.3.2 Ocean Derived Flood Events

Elevated water levels typically comprise a combination of:

- Barometric pressure set up of the ocean surface due to the low atmospheric pressure of the storm;
- Wind set up due to strong winds during the storm “piling” water upon the coastline;
- Astronomical tide, particularly the HHWSS; and
- Wave set up.

The design still water levels used for this study were derived through analysis of data at Fort Denison, Sydney, as recommended in Draft Coastal Risk Management Guide (DECCW, 2009). These design still water levels inherently incorporate allowance for tides, meteorological influences and other water level anomalies, but exclude wave setup influences. These design still water levels are presented in Table 7-6.

Wave set up is defined as the elevation of the water surface due to the release of energy by breaking waves. Wave set up is directly related to wave height, and so will be greater during storm conditions when wave heights are also larger. Wave set up occurs within the nearshore zone, and so is often calculated separately, then added to elevated still water levels. Storm surge calculations typically comprise the component of the elevated ocean level due to barometric pressure set up and wind set up, through a complex assessment of storm synoptic types and measured data.

Table 7-6 Derivation of Design Elevated Water Levels for Ocean Boundaries

Design Flood	Still Water Level (m AHD)	6hr Duration Significant Wave Height (m)	Wave Set Up (m)	Still Water Level + Wave Set Up (m AHD)
20% AEP	1.32	5.4	0.81	2.13
5% AEP	1.38	6.2	0.93	2.31
2% AEP	1.42	6.7	1.01	2.43
1% AEP	1.44	7.1	1.07	2.51
0.5% AEP	1.46	7.6	1.14	2.60

Wave set up is typically taken to be approximately 15% of the offshore significant wave height at the shoreline (WBM, 2003; WP Geomarine, 1998). MHL has provided the most recent return interval analysis of storm wave heights using the 21 years of data at Batemans Bay (May 1986 to December 2007, refer Figure 7-3). The 6 hour storm duration wave heights have been utilised in the assessment of wave set up, as these will be sustained over a period of time similar to that of a high tide cycle. The 6 hour storm duration wave heights and subsequent wave set up values are given in Table 7-6.

The temporal pattern of the design boundaries for ocean derived flood events was based on the recommended ocean design events for 5% AEP, as shown in Figure 7-1. The timing of the peak water level was adjusted to coincide with the peak catchment inflow, which occurs at around T=11 hours. The water levels were then scaled accordingly to match those from Table 7-6. The design ocean boundaries used in this study are presented in Figure 7-4.

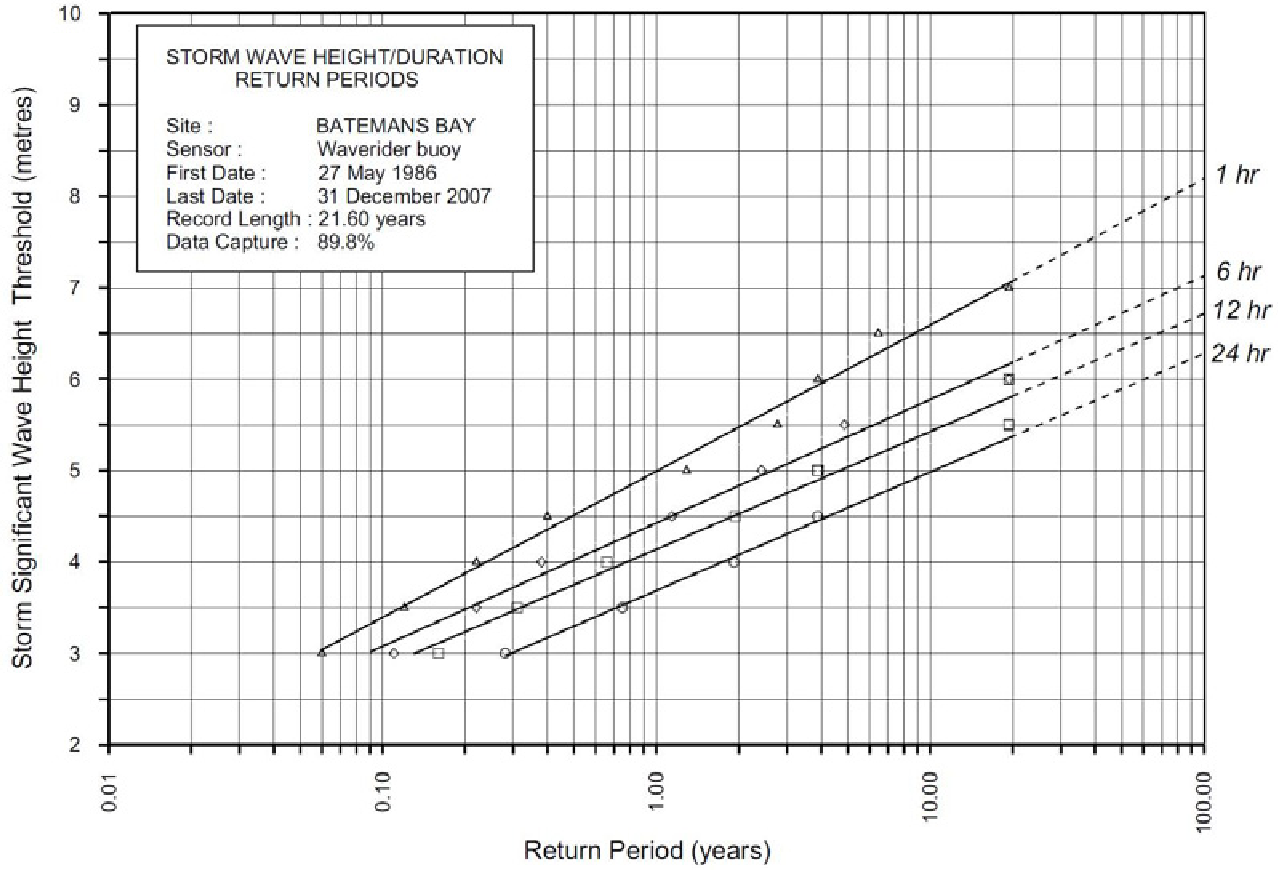


Figure 7-3 Storm Wave Return Periods for Batemans Bay

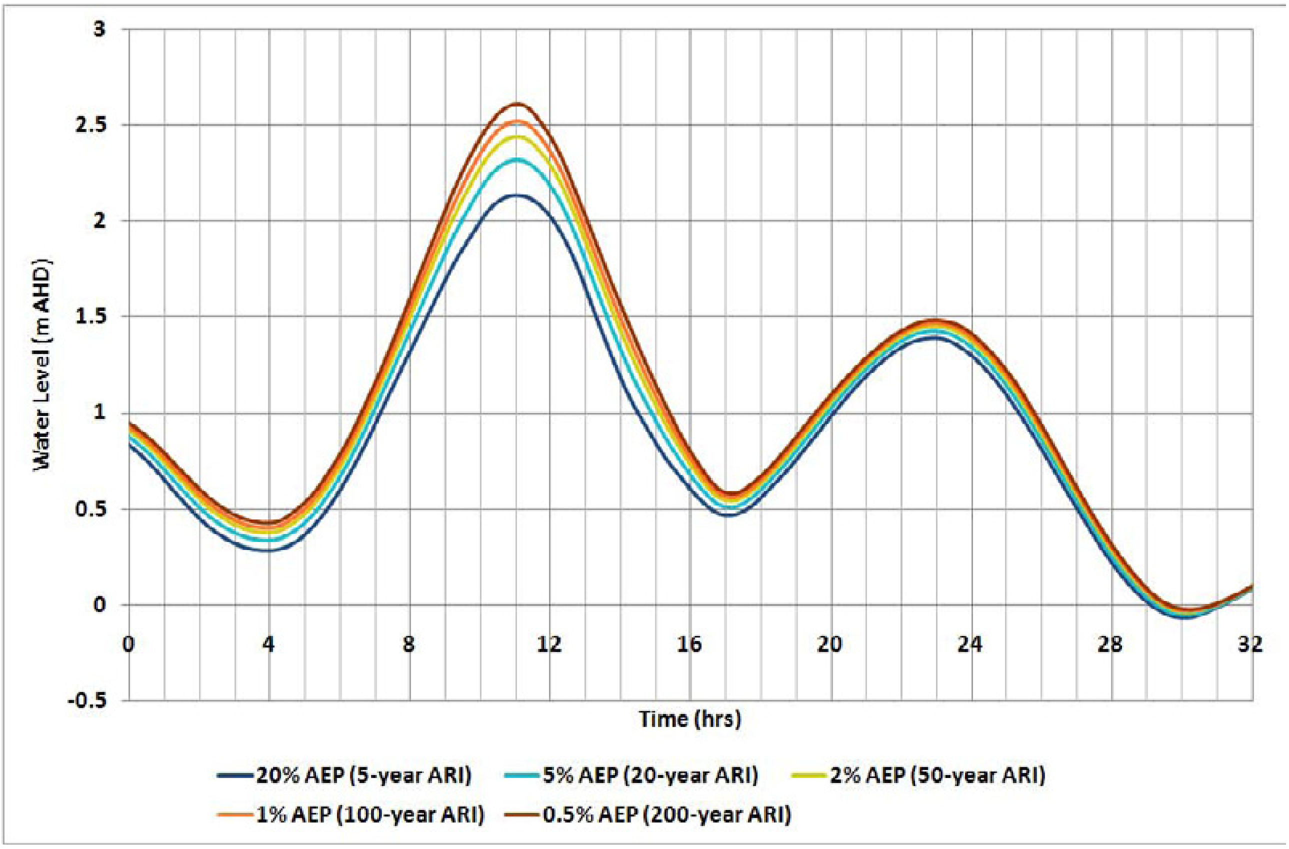


Figure 7-4 Design Ocean Boundaries for Ocean Flood Events

7.3.3 Climate Change

Current guidelines predict that a likely outcome of future climatic change will be an increase in mean sea level. NSW Sea Level Rise Policy Statement (DECCW, 2009) provides projected increases in mean sea level for NSW of 0.4m and 0.9m, for the years 2050 and 2100 respectively. Based on these guidelines the design ocean boundaries have been raised by 0.4m and 0.9m to assess the potential impact of climate change on flood behaviour in the Tabourie Lake catchment.

Climate change may also result in an increase in the frequency and intensity of storms, further exacerbating the effects of sea level rise on coastal flood behaviour. The data provided in Projected Changes in Climatological Forcing for Coastal Erosion in NSW (CSIRO, 2007) indicates that a conservative approach would be to adopt around a 10% increase in significant wave heights for the 50 year planning horizon and around a 30% increase for the 100 year planning horizon. An increase in significant wave heights for ocean events would result in an increased wave set up. Using the significant wave heights and wave set up values from Table 7-6, appropriate potential future increases in wave set up have been derived for use in this study and are presented in Table 7-7.

Table 7-7 Future Climate Change Increase in Wave Set Up Values

Design Flood	6hr Duration Significant Wave Height (m)	Potential Future Increase in Significant Wave Height (m)		Potential Future Increase in Wave Set Up (m)	
		2050	2100	2050	2100
20% AEP	5.4	0.54	1.62	0.08	0.24
5% AEP	6.2	0.62	1.86	0.09	0.28
2% AEP	6.7	0.67	2.01	0.10	0.30
1% AEP	7.1	0.71	2.13	0.11	0.32
0.5% AEP	7.6	0.76	2.28	0.11	0.34

7.4 Design Berm Geometry

The design berm geometry is the most significant factor that will influence catchment derived flooding behaviour within Tabourie Lake. Recent research has been undertaken (Wainwright and Baldock, 2010) to understand the long-term trends in berm erosion and deposition and the likely probabilistic berm conditions at Tabourie Lake. Key findings from this research, included in Appendix D, have been applied in defining the design berm conditions for future planning horizons.

In defining the entrance condition for the design flood analysis, consideration is given to the geometry of the berm for open and closed conditions, for existing and future scenarios considering potential sea level rise influences.

7.4.1 Catchment Derived Flood Events

The berm saddle height adopted for the catchment derived flooding design events is 2.0m AHD. This is similar to the mean annual maximum berm height, derived from a probabilistic assessment of berm heights from Wainwright and Baldock (2010). The results of this analysis are presented in Table 7-8.

Table 7-8 Frequency Analysis of Annual Maximum Berm Heights

Annual Exceedance Probability	Annual Maximum Berm Saddle Height (m AHD)
50%	1.98
20%	2.03
10%	2.06
5%	2.08
1%	2.12

The values in Table 7-8 refer to the height of the berm saddle. The crest height adopted for the majority of the berm is 200mm higher than the saddle, as is typical of the Tabourie Lake entrance conditions. This gives a berm crest height of 2.2m AHD, which is similar to that used in the calibration modelling, discussed in Section 6.3. The results of this calibration suggest that the berm heights during the events of March 1975 and April 1988 were somewhere in the order of this value, so it would appear to be a reasonable to adopt for Tabourie Lake design purposes.

A design berm topography was constructed for the hydrodynamic model using these berm crest and saddle heights. Adopted slope gradients on the lake side and ocean side of the berm were 1 in 50 and 1 in 15 respectively. These values were obtained from topographic survey data of the Tabourie Lake entrance berm and considered appropriate to use for design purposes. Figure 7-5 shows cross-sectional and long-sectional profiles of the adopted design berm geometry.

7.4.2 Ocean Derived Flood Events

The berm geometry for ocean derived flood events was obtained from the scoured berm geometry output from the catchment derived flood events. Using this data provides for a largely unrestricted entrance condition, as recommended for use in ocean derived flood events by Appendix A of the Draft Flood Risk Management Guide (DECCW, 2009).

The scoured or open entrance condition is depicted by the open berm profile in Figure 7-5. Of note is the elevated bed profile extending a few hundred metres upstream of the berm that represents the existing shoaling in the entrance channel.

7.4.3 Climate Change

There are no government guidelines concerning the impact of future climatic change of entrance berm geometries. A change in entrance berm processes is likely to result from the predicted sea level rise and changes to coastal storm intensity. From this change, a net upward shift in typical berm heights at the entrance may be expected commensurate with sea level rise estimates.

The research of Wainwright and Baldock (2010) indicates projected future increase in berm height at Tabourie Lake is slightly greater than the corresponding rise in mean sea level. For the purposes of this study a berm height increase of 0.5m and 1.0m has been adopted for the 2050 and 2100 horizons respectively. This gives a berm saddle height of 2.5m AHD for the 2050 planning horizon and 3.0m AHD for the 2100 planning horizon.

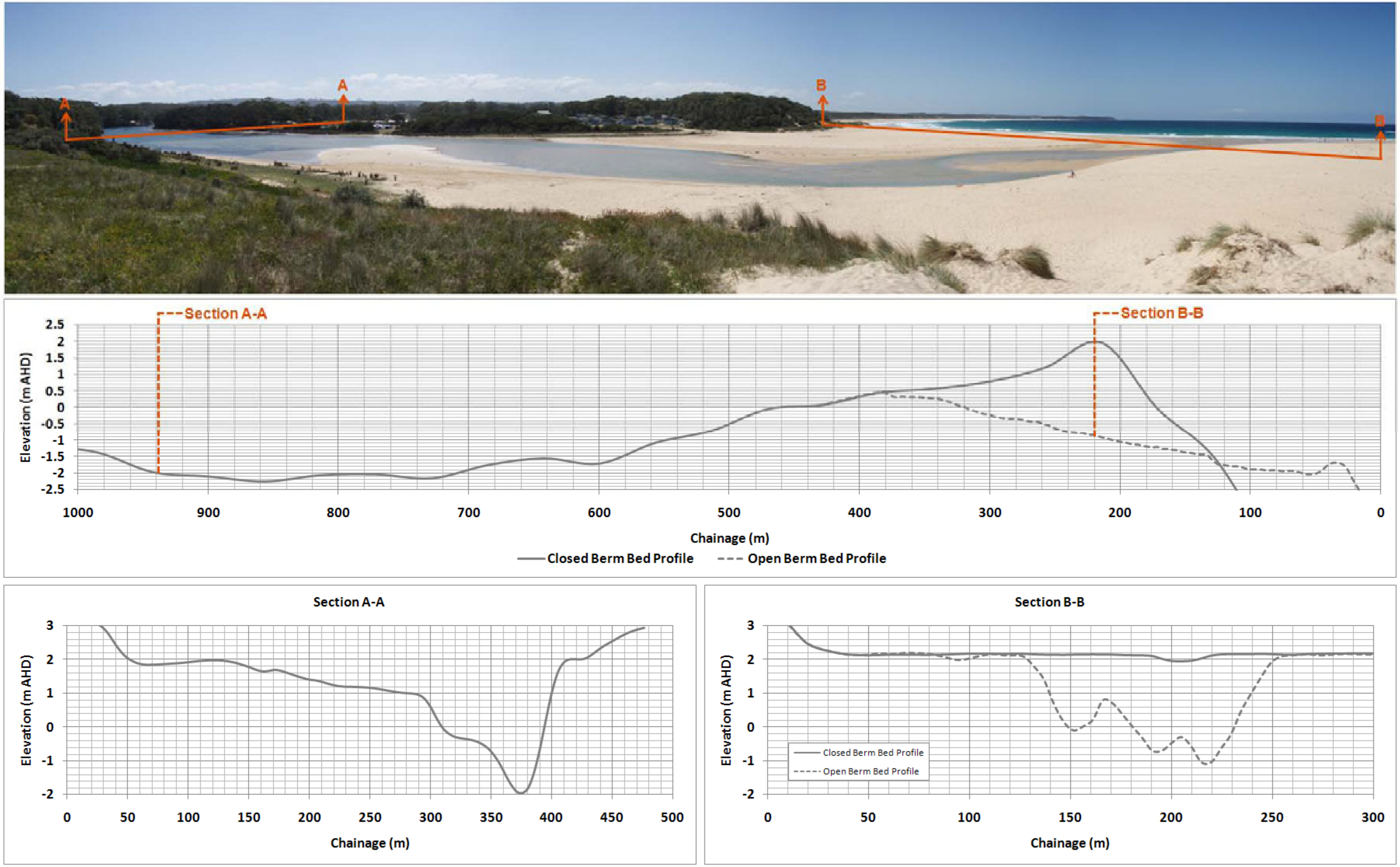


Figure 7-5 Design Berm Geometry

7.5 Initial Water Levels

Initial water levels in Tabourie Lake for design flood events have been derived through analysis of available water level records from the MHL operated gauge located in Tabourie Creek. The analysis considered separate periods of open and closed entrance conditions as detailed below.

7.5.1 Catchment Derived Flood Events

To determine a suitable initial water level in Tabourie Lake for catchment derived flood events, an analysis of water levels during periods in which the lake entrance was closed was undertaken. The results of this analysis gave an average water level of 0.60m AHD.

It is noted from the sensitivity analysis of initial water level conditions undertaken for the March 1975 and April 1988 calibration events, that the peak flood level was insensitive to the initial lake condition. This is largely due to the Lake storage being relatively small in comparison to the catchment generated flood water volume.

7.5.2 Ocean Derived Flood Events

To determine a suitable initial water level in Tabourie Lake for ocean derived flood events, an analysis of water levels during periods in which the lake entrance was open was undertaken. The results of this analysis gave an average water level of 0.25m AHD which conforms to the mean level from the Tabourie Lake tidal plane analysis (refer to Table 2-4).

7.5.3 Climate Change

The changes in water levels within Tabourie Lake under future climate change predictions will be influenced by a number of factors including:

- The rise of mean sea levels;
- The rise in entrance berm heights;
- The change in catchment runoff regimes.

In the absence of any specific guidelines, the projected increases in mean sea level have also been applied to the mean water levels in Tabourie Lake. The water level in the lake will be largely a function of the ocean level. It is considered appropriate to apply the same increases in mean sea level to the initial conditions in the lake. The adopted initial water levels in Tabourie Lake under a closed entrance condition are therefore 1.0m AHD and 1.5m AHD for the 2050 and 2100 horizons respectively. Under an open entrance condition the initial water levels will be 0.65m AHD and 1.15m AHD for the 2050 and 2100 horizons respectively.

8 DESIGN FLOOD RESULTS

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

The impact of future climate change on flooding in Tabourie Lake was also considered for both catchment derived and ocean derived flood events, focussing on the 5% AEP, 1% AEP and PMF flood events.

8.1 Peak Flood Conditions

8.1.1 Catchment Derived Flood Events

The design flood results are presented in a flood mapping series in Appendix A. For the simulated design events including the 20% AEP, 5% AEP, 2% AEP, 1% AEP and PMF events, a map of peak flood level, depth and velocity is presented covering the modelled area.

Predicted flood levels at selected locations are shown in Table 8-1 for the full range of design event magnitudes considered. For each event, results for the following entrance conditions are presented:

- Open entrance condition;
- Closed entrance with a berm saddle elevation of 2.0m AHD and incorporating an entrance breakout scenario;
- Closed entrance with a fixed berm saddle elevation of 2.0m AHD and no simulation of an entrance breakout.

The entrance condition is highly variable and has a significant impact on modelled peak water levels, particularly for catchment derived flood events. Therefore, these entrance conditions were modelled to provide a 'worst case' (i.e. fixed berm condition), 'best case' (i.e. open entrance condition) and 'intermediate' (i.e. entrance breakout) comparison.

Longitudinal profiles showing predicted flood levels along Tabourie Creek for the berm breakout entrance condition are shown in Figure 8-1. Longitudinal profiles showing predicted flood levels along Tabourie Creek are also shown in Figure 8-2 for the 1% AEP event, for the full range of entrance conditions. The 1% AEP ocean derived flood event profile is also included on this figure for the purposes of comparison.

From Figure 8-1 it can be seen that the dominant control on flood levels within Tabourie Creek is the entrance berm. During a flood event with no human intervention, to initiate a breakout of the entrance berm, the water level must rise to a level above the crest of the berm. This can only happen if there is a sufficient catchment runoff volume. The excess flood volume then exits to the sea via overtopping of the entrance berm. An overtopping depth of over 0.3m is then required before erosion of the berm commences. With a berm width of a few hundred metres this can require a reasonably large flow. The model simulations indicate that a 20% AEP event is not sufficient to initiate a significant entrance breakout, as can be seen from the minimal hydraulic gradient. For events of a larger magnitude, when sufficient overtopping depths and flow rates are attained, an entrance breakout commences and the flood waters can subside. The large width of overtopping results in a relatively small difference to flood levels upstream of the berm, indicative of the relatively efficient flow regime associated with extensive berm overtopping. There is less than 0.1m difference between peak flood levels at the entrance for the 5% AEP and 1% AEP events.

Table 8-1 Estimated Peak Flood Levels for Catchment Derived Design Events

Event Conditions	Design Event Frequency				
	20% AEP	5% AEP	2% AEP	1% AEP	PMF
Entrance (Tabourie Creek)					
Open Berm	0.86	1.07	1.20	1.30	2.82
Berm Breakout	2.34	2.44	2.50	2.53	2.90
Fixed Berm	2.35	2.55	2.66	2.75	3.52
MHL Recorder (Tabourie Creek)					
Open Berm	1.07	1.38	1.59	1.77	3.86
Berm Breakout	2.35	2.48	2.58	2.63	3.97
Fixed Berm	2.35	2.58	2.73	2.85	4.22
D/S Princes Hwy (Tabourie Creek)					
Open Berm	1.45	1.91	2.21	2.42	4.66
Berm Breakout	2.36	2.60	2.78	2.91	4.75
Fixed Berm	2.36	2.66	2.88	3.04	4.86
U/S Princes Hwy (Tabourie Creek)					
Open Berm	1.51	2.01	2.34	2.57	4.69
Berm Breakout	2.36	2.65	2.86	3.01	4.78
Fixed Berm	2.36	2.70	2.93	3.09	4.89
Tabourie Lake					
Open Berm	1.52	2.02	2.35	2.59	4.71
Berm Breakout	2.36	2.66	2.87	3.02	4.80
Fixed Berm	2.36	2.70	2.94	3.10	4.91
Centre St (Lemon Tree Creek)					
Open Berm	1.18	1.53	1.77	1.95	3.88
Berm Breakout	2.35	2.50	2.60	2.66	3.99
Fixed Berm	2.35	2.59	2.74	2.86	4.23
South St (Lemon Tree Creek)					
Open Berm	1.74	2.20	2.44	2.58	3.90
Berm Breakout	2.35	2.55	2.69	2.78	4.01
Fixed Berm	2.36	2.61	2.77	2.89	4.24

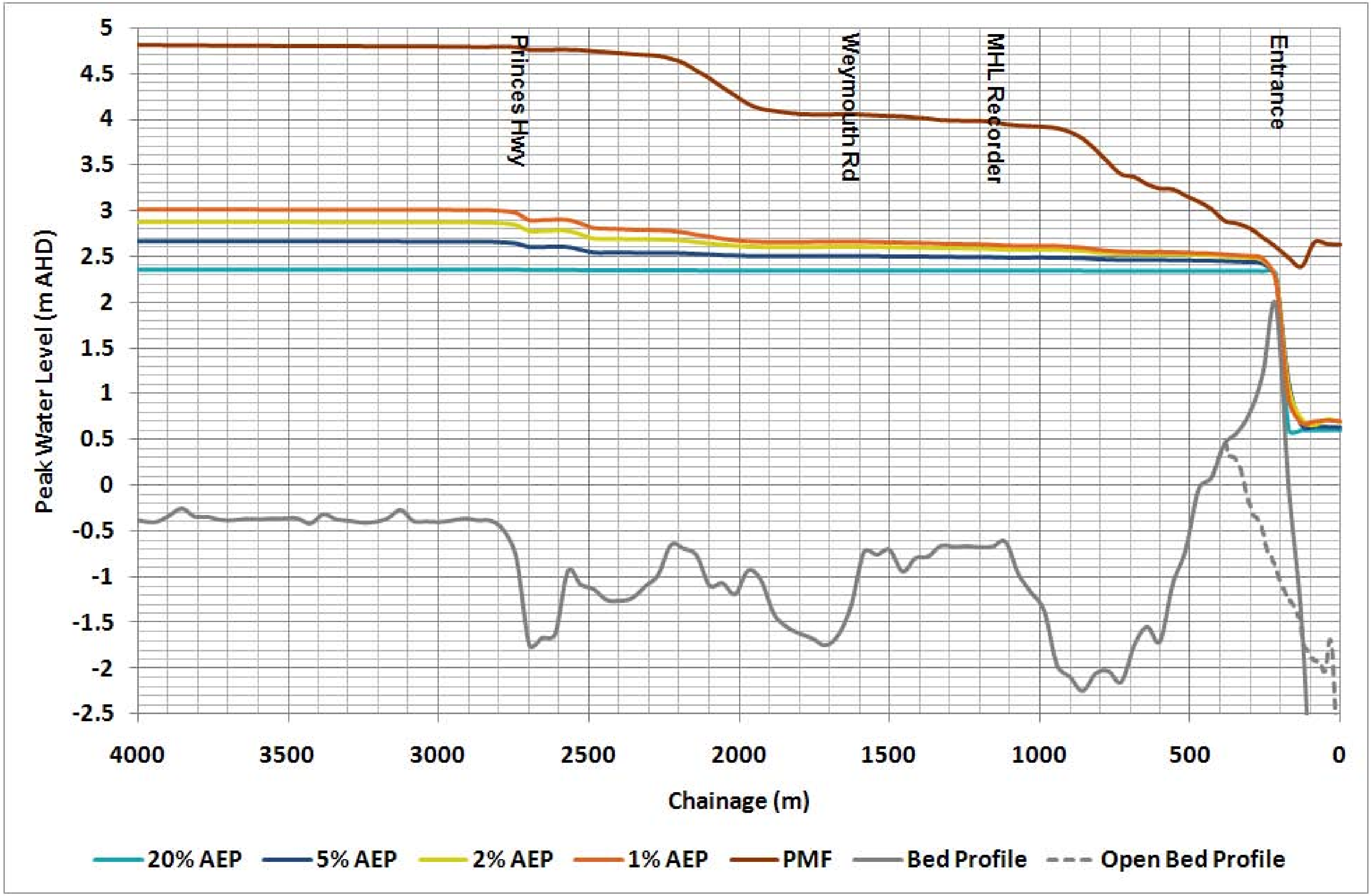


Figure 8-1 Design Flood Level Profiles for Catchment Derived Events

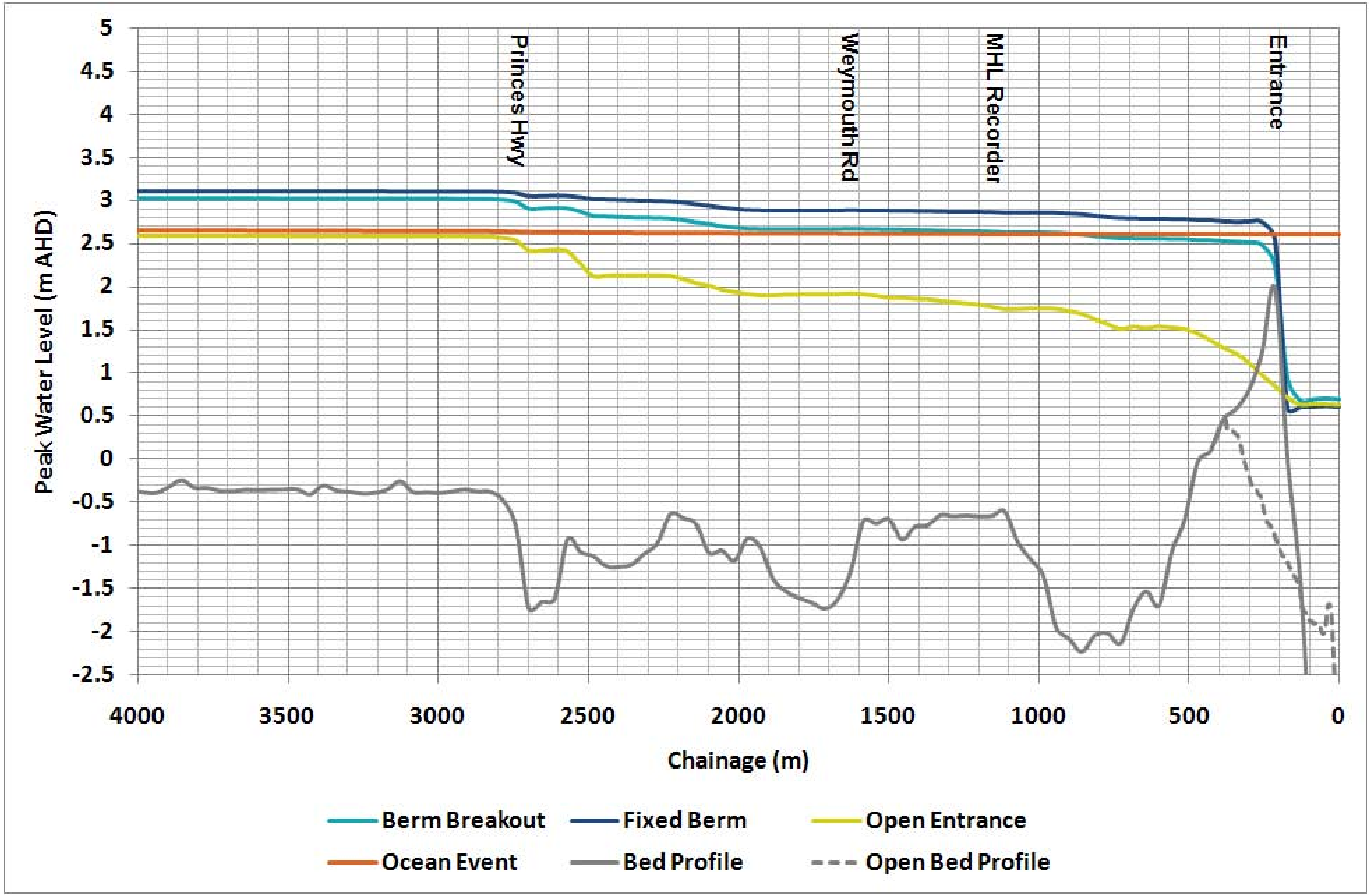


Figure 8-2 Design Peak Flood Level Profiles for the 1% AEP Event

There are a few other hydraulic controls evident from the changes in water level gradients on the longitudinal profiles shown in Figure 8-1. Both the Princes Highway bridge and a bottleneck in the floodplain 200m downstream generate a combined head drop of around 200mm for most events up the 1% AEP. The level of hydraulic control of the Princes Highway bridge is far less significant for the PMF event, when there is significant overtopping of the Princes Highway. For the PMF event the major hydraulic controls shift from the Princes Highway bridge to other natural floodplain bottlenecks in the vicinity of Portland Way and further downstream of the MHL recorder.

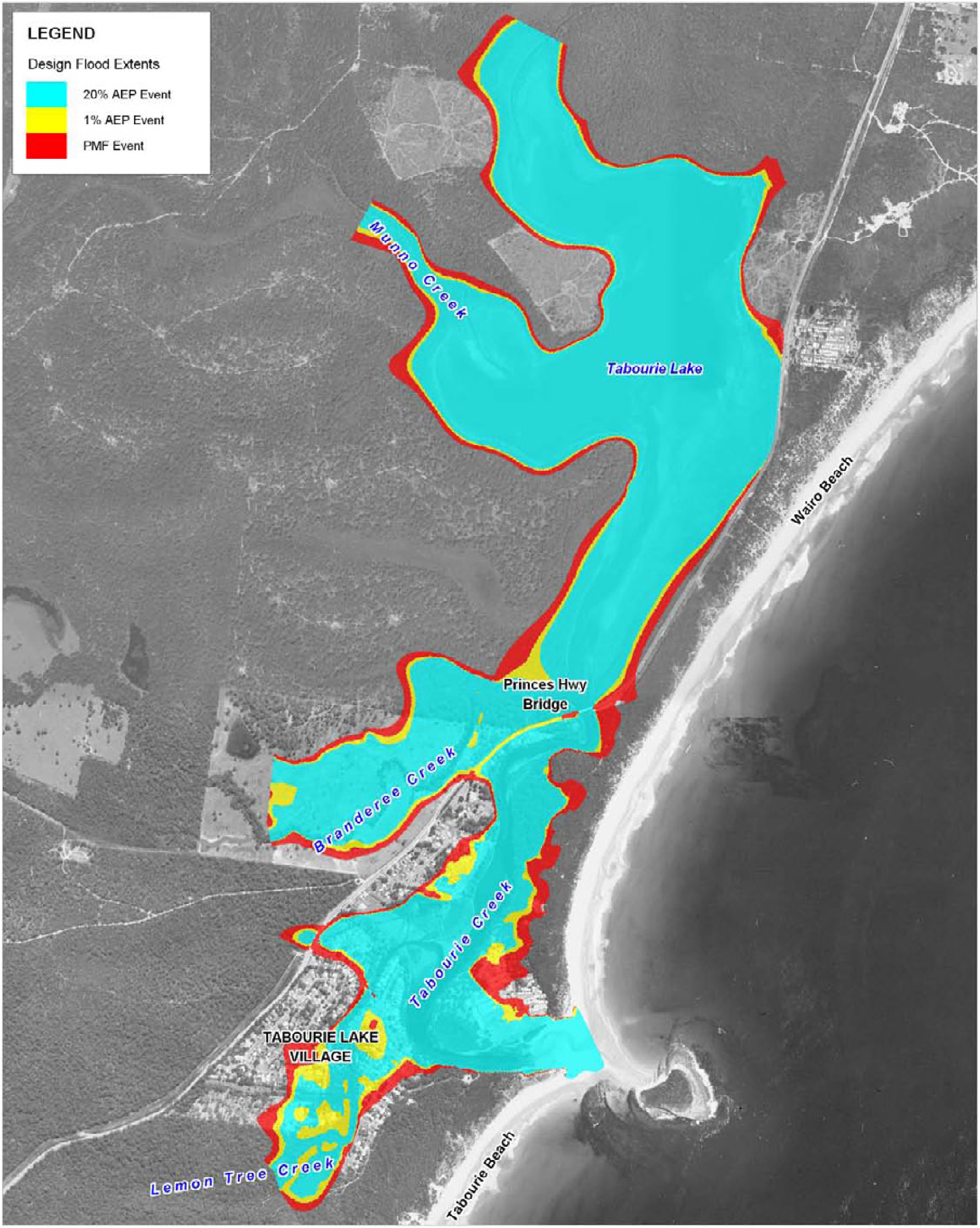
Figure 8-2 shows the significant influence of the entrance conditions on peak flood levels for the 1% AEP event. The model simulations presented include a fixed berm, entrance breakout and open entrance condition. The simulated flood levels in Tabourie Creek are the greatest for the fixed entrance berm (i.e. no entrance breakout or scour), and generally are of the order of 200mm higher than the flood levels simulated allowing the entrance to breakout. This relatively small difference between the fixed berm and entrance breakout conditions is due to the fact that significant entrance scour does not occur until overtopping of the berm occurs.

When modelled with the open entrance design conditions the 1% AEP flood levels are greatly reduced, by around 0.5 to 1.0m. The other hydraulic controls previously discussed in the channel further upstream of the entrance become more significant with the increased hydraulic gradient following removal of the berm control on water levels. The actual entrance topography during any given flood event, under an open entrance condition, would be highly variable given changing shoaling patterns in the channel. This variability would have a significant impact on resultant flood profiles through Tabourie Creek.

The 1% AEP ocean event is also presented in Figure 8-2 for comparative purposes. The fixed berm scenario dominates the peak flood level condition for the 1% AEP event, however the ocean condition is within approximately 0.3m below this level towards the entrance. The difference between the catchment and ocean events increases in the upstream direction with the development of the flood gradient for the incoming catchment flows.

The Tabourie Lake Tourist Park is located within a meander of Tabourie Creek, just upstream of the entrance. Of particular note is the simulated velocity distribution for the PMF flood conditions. Under these extreme flood conditions, a major floodway is initiated which runs through the Tourist Park, effectively short-cutting the main Tabourie Creek channel alignment. Under these PMF conditions peak velocities exceed 1 m/s.

Figure 8-3 shows the modelled flood inundation extents for catchment derived events. The 20% AEP, 1% AEP and PMF inundation extents are shown. The results that are presented are all modelled with an entrance berm breakout condition.



Title:
Catchment Derived Design Flood Inundation Extents

Figure: 8-3

Rev: A

N

0 375 750m
Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_011_091217 Inundation Extents.WOR

8.1.2 Ocean Derived Flood Events

The design flood results are presented in a flood mapping series in Appendix A. For the simulated design events including the 20% AEP, 5% AEP, 2% AEP and 1% AEP, a map of peak flood level, depth and velocity is presented covering the modelled area.

Predicted flood levels at selected locations are shown in Table 8-2 for the full range of design event magnitudes considered. All events were modelled with an open entrance condition. For lower order events that occur during a closed entrance condition the berm may offer some flood protection. However, for larger events the berm would be overtopped or even destroyed, as occurred in 1974.

Longitudinal profiles showing predicted flood levels along Tabourie Creek for the ocean derived design events are shown in Figure 8-4. It can be seen that the flood levels throughout Tabourie Creek are similar to the peak level of the design ocean boundary. There is a small hydraulic gradient associated with the coincident 20% AEP catchment event, which has been adopted for the ocean derived design events.

Table 8-2 Estimated Peak Flood Levels for Ocean Derived Design Events

Event Conditions	Design Event Frequency			
	20% AEP	5% AEP	2% AEP	1% AEP
Entrance	2.13	2.31	2.43	2.51
MHL Recorder	2.15	2.33	2.44	2.52
D/S Princes Hwy	2.20	2.37	2.47	2.55
U/S Princes Hwy	2.22	2.38	2.49	2.56
Tabourie Lake	2.22	2.39	2.50	2.57
Centre St	2.16	2.33	2.45	2.52
South St	2.20	2.36	2.46	2.54

The ocean events propagate through the entrance and into Tabourie Creek, attaining a similar flood level within Tabourie Lake to that in the ocean. Typical flood depths within Tabourie Lake village are in the order of 0.25 to 0.5m for the 1% AEP event. Flood velocities are minimal.

The flood levels for the equivalent catchment derived event (closed entrance condition) are higher than those modelled for the ocean derived events, particularly for the lower order events and upstream of the Lemon Tree Creek confluence.

The adopted design berm height (assuming no ongoing entrance management) was a saddle of 2.0m AHD and berm crest level of 2.2m AHD. As can be seen in Table 8-2, these berm levels are exceeded by the design ocean event water levels. Accordingly, the existing design ocean flooding conditions would result in significant inundation of parts of Tabourie Lake, irrespective of the berm condition and entrance management.

Figure 8-5 shows the modelled flood inundation extents for ocean derived events. The 20% AEP, 5% AEP and 1% AEP inundation extents are shown, modelled with an open entrance condition.

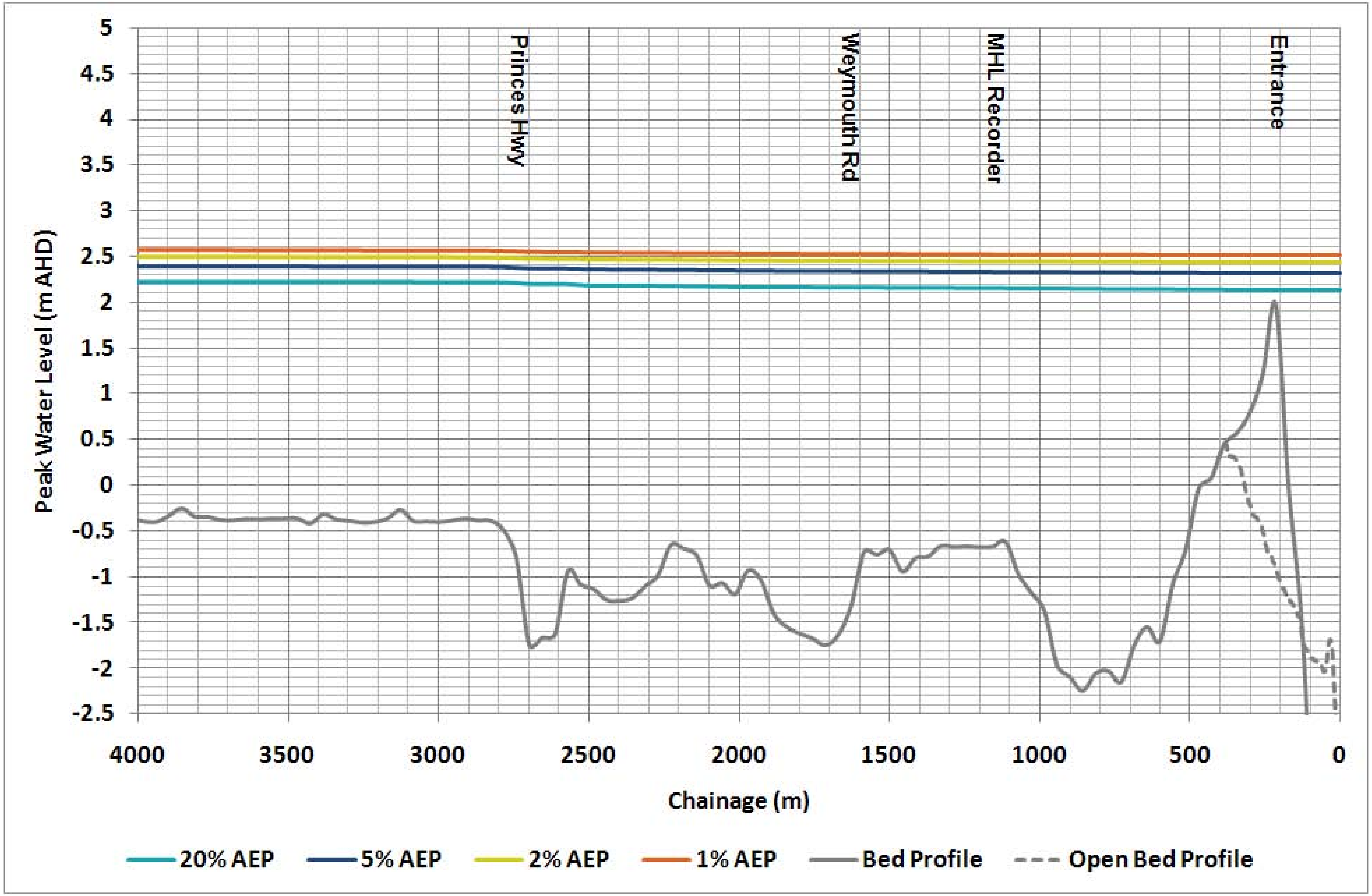
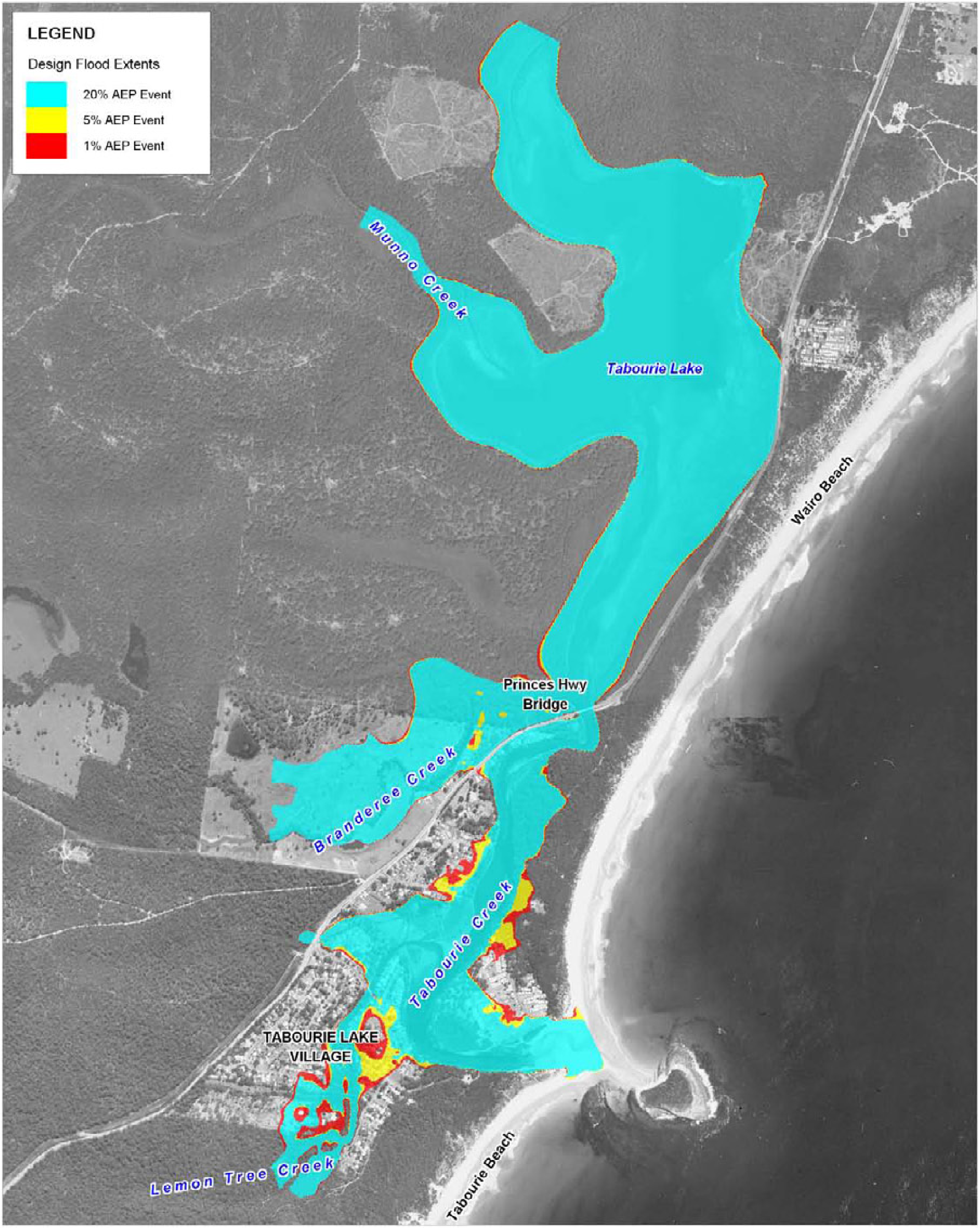


Figure 8-4 Design Flood Level Profiles for Ocean Derived Events



Title:
Ocean Derived Design Flood Inundation Extents

Figure: 8-5

Rev: A

N

0 375 750m

Approx. Scale

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8.2 Flood Hydrographs

Predicted peak flood flows at selected locations are shown in Table 8-3 for the full range of catchment derived design event magnitudes considered. For each event results for the following entrance conditions are presented:

- Open entrance condition;
- Closed entrance with a berm saddle elevation of 2.0m AHD and incorporating an entrance breakout scenario;
- Closed entrance with a fixed berm saddle elevation of 2.0m AHD and no simulation of an entrance breakout.

Table 8-3 Peak Flows for Catchment Derived Design Events

Event Conditions	Design Event Frequency				
	20% AEP	5% AEP	2% AEP	1% AEP	PMF
Centre St Bridge					
Open Berm	20	35	46	55	191
Berm Breakout	19	32	41	49	183
Fixed Berm	19	32	41	48	175
Pacific Hwy Bridge					
Open Berm	71	122	163	200	869
Berm Breakout	28	143	196	247	896
Fixed Berm	28	104	164	216	876
Entrance					
Open Berm	84	144	195	240	1006
Berm Breakout	34	192	257	309	1045
Fixed Berm	34	121	190	248	1015

Flood hydrographs for selected design events are presented in Figure 8-6. The hydrographs represent flow rates through the entrance under an open condition. The design events are a 9-hour storm duration, which was determined as being the critical storm duration for the catchment.

Figure 8-7 shows the relative timing of the flood hydrographs on Tabourie Creek (Princes Highway bridge) and Lemon Tree Creek (Centre Street bridge). The hydrographs are for the 1% AEP event under an open entrance condition. The peak flow on Lemon Tree Creek occurs some three hours before that of Tabourie Creek given the smaller catchment size. The relative contribution of the Lemon Tree catchment to the total design flow is significantly lower than the upper catchments of Tabourie Creek and Brandaree Creek.

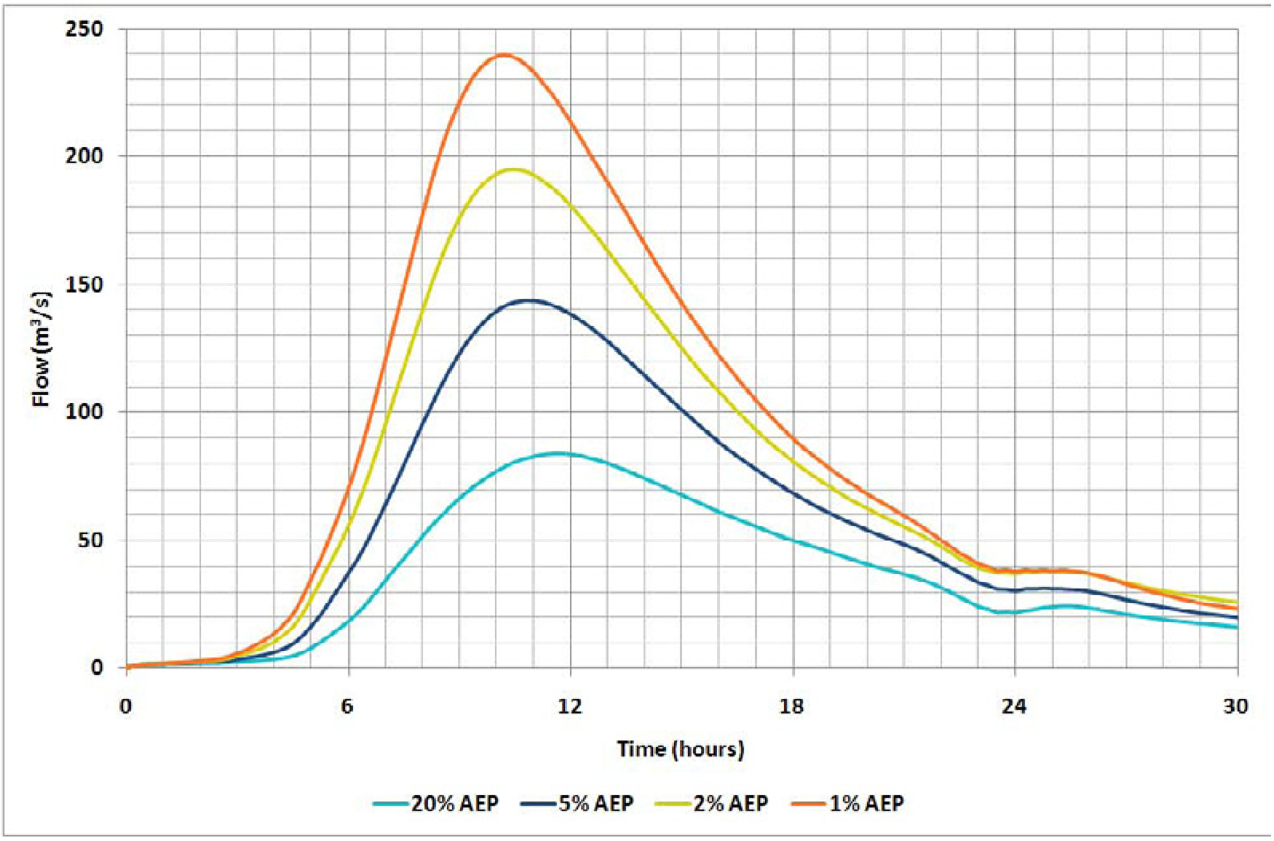


Figure 8-6 Catchment Derived Design Flood Hydrographs for Tabourie Lake Entrance

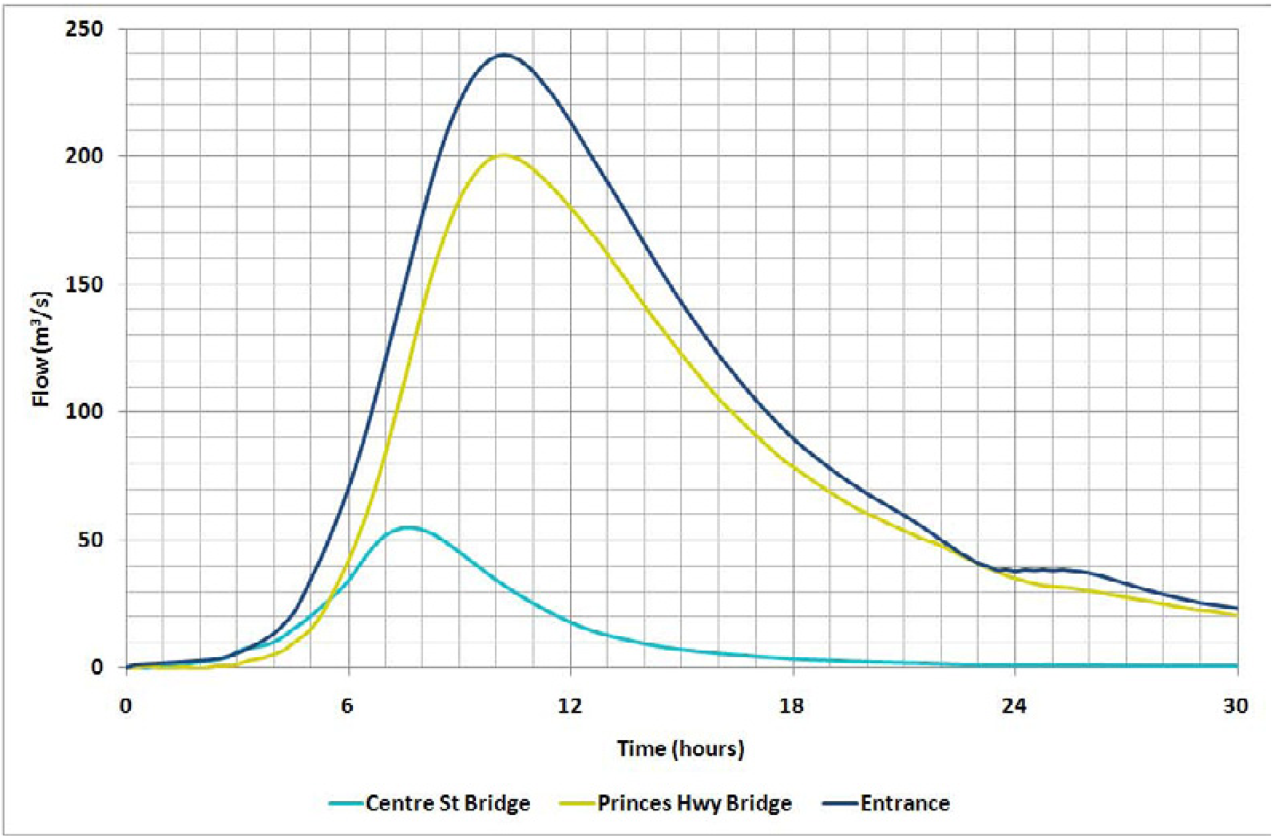


Figure 8-7 Timing of Catchment Derived Design 1% AEP Hydrographs for Tabourie Lake

The entrance condition has a significant impact on the simulated flood hydrographs, shown in Figure 8-8 for the 1% AEP event. For the closed berm conditions (fixed berm and breakout scenarios) the rising limb of the flood hydrograph is effectively detained behind the entrance berm until the upstream storage capacity is reached. As overtopping of the berm occurs the discharge to the ocean increases rapidly and results in higher peak discharges at the ocean boundary in comparison to the open entrance condition. The peak discharge is greater for the entrance breakout scenario, given the larger conveyance capacity developed through scouring of the berm profile.

The flood volume represented in the hydrograph shown in Figure 8-8 for the fixed berm condition is considerably lower than the other two scenarios. This is a function of the storage retained upstream of the berm.

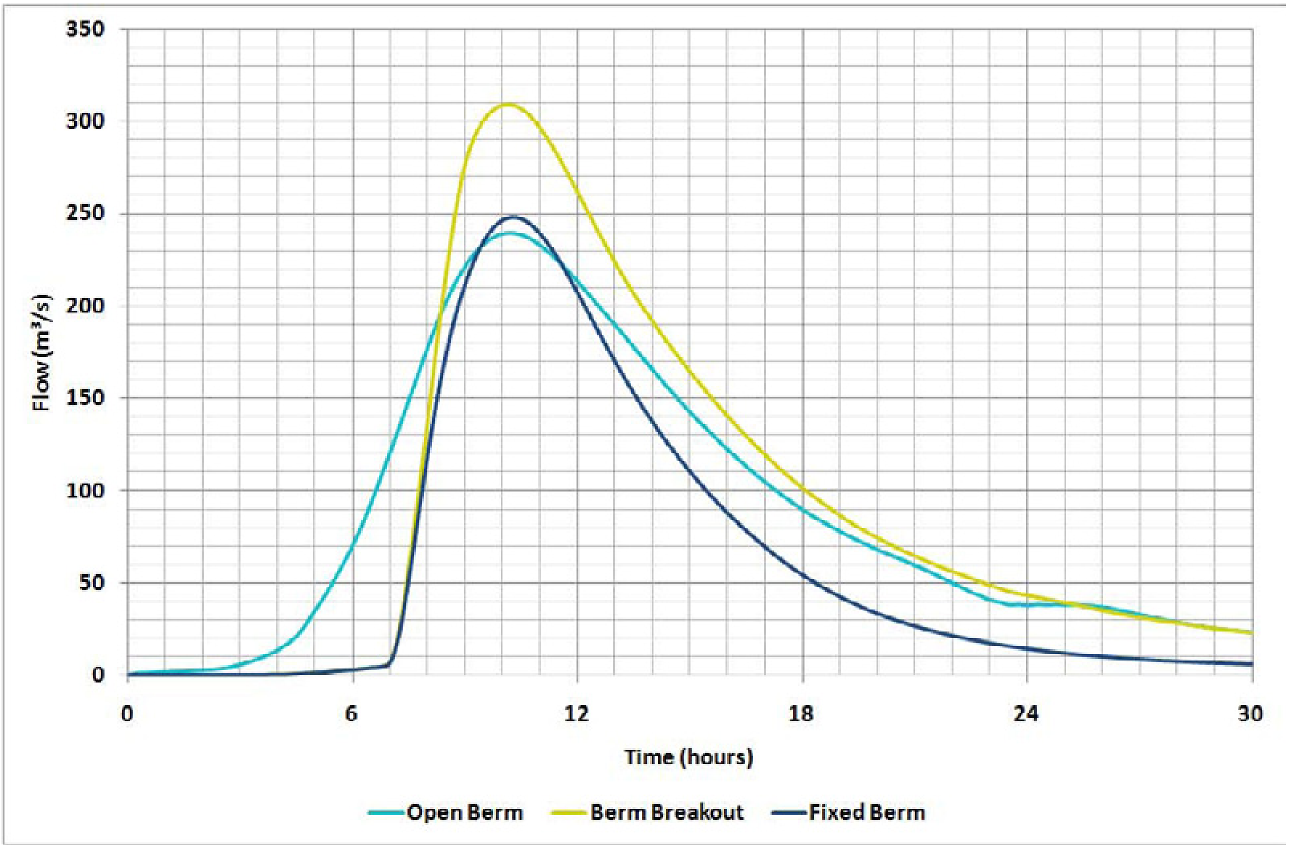


Figure 8-8 Impact of Entrance Condition on Catchment Derived 1% AEP Flow Hydrograph

8.3 Climate Change

8.3.1 Catchment Derived Flood Events

The most significant impact of future climate change on catchment derived flood events within Tabourie Lake will be an increased berm height predominantly in association with sea-level rise. The forecast increase in berm height of around 0.5m by 2050 and 1.0m by 2100 is similar to the increase in modelled water levels for these climate change events. Table 8-4 shows modelled peak flood levels at key locations for the range of catchment derived climate change events. By comparing the baseline catchment derived flood levels at the MHL recorder of 2.48m AHD and 2.63m AHD, for the 5% AEP and 1% AEP events respectively, it can be seen that the flood levels for the 2050 and 2100 planning horizons increase by approximately 0.5m and 1.0m accordingly. The additional 10% rainfall input also considered for the 2100 planning horizon results in only a minimal increase in modelled flood levels.

Table 8-4 Estimated Peak Flood Levels for Catchment Derived Design Events Incorporating Future Climate Change Predictions

Location	Design Event Frequency and Climate Change Condition					
	5% AEP 2050	5% AEP 2100	5% AEP 2100 +10% rainfall	1% AEP 2050	1% AEP 2100	1% AEP 2100 +10% rainfall
Entrance	2.92	3.40	3.43	3.03	3.53	3.55
MHL Recorder	2.95	3.41	3.45	3.10	3.58	3.61
D/S Princes Hwy	2.99	3.42	3.48	3.26	3.67	3.73
U/S Princes Hwy	3.00	3.43	3.48	3.29	3.69	3.75
Tabourie Lake	3.01	3.43	3.49	3.30	3.69	3.76
Centre St	2.95	3.41	3.45	3.11	3.58	3.61
South St	2.95	3.41	3.45	3.13	3.58	3.62

Longitudinal profiles showing predicted flood levels along Tabourie Creek for the catchment derived climate change events are shown in Figure 8-9. It can be seen that the increased tailwater conditions result in a fairly consistent increase in flood levels throughout Tabourie Creek and Tabourie Lake. When compared to the baseline catchment derived flood profiles in Figure 8-1, it is evident that as the tailwater level increases the impact of the hydraulic controls within Tabourie Creek (excluding the entrance berm itself) diminishes.

8.3.2 Ocean Derived Flood Events

The most significant impact of future climate change on ocean derived flood events within Tabourie Lake will be an increased sea-level and to a lesser extent an increase in significant wave heights for ocean storms. The combined increase in peak flood levels (from both sea-level rise and wave set up) for design ocean storms is around 0.5m by 2050 and 1.2m by 2100.

Figure 8-10shows modelled peak flood levels at key locations for the range of ocean derived climate change events. By comparing the baseline ocean derived flood levels at the entrance of 2.31m AHD and 2.51m AHD, for the 5% AEP and 1% AEP events respectively, it can be seen that the flood levels for the 2050 and 2100 planning horizons also increase by 0.5m and 1.2m accordingly.

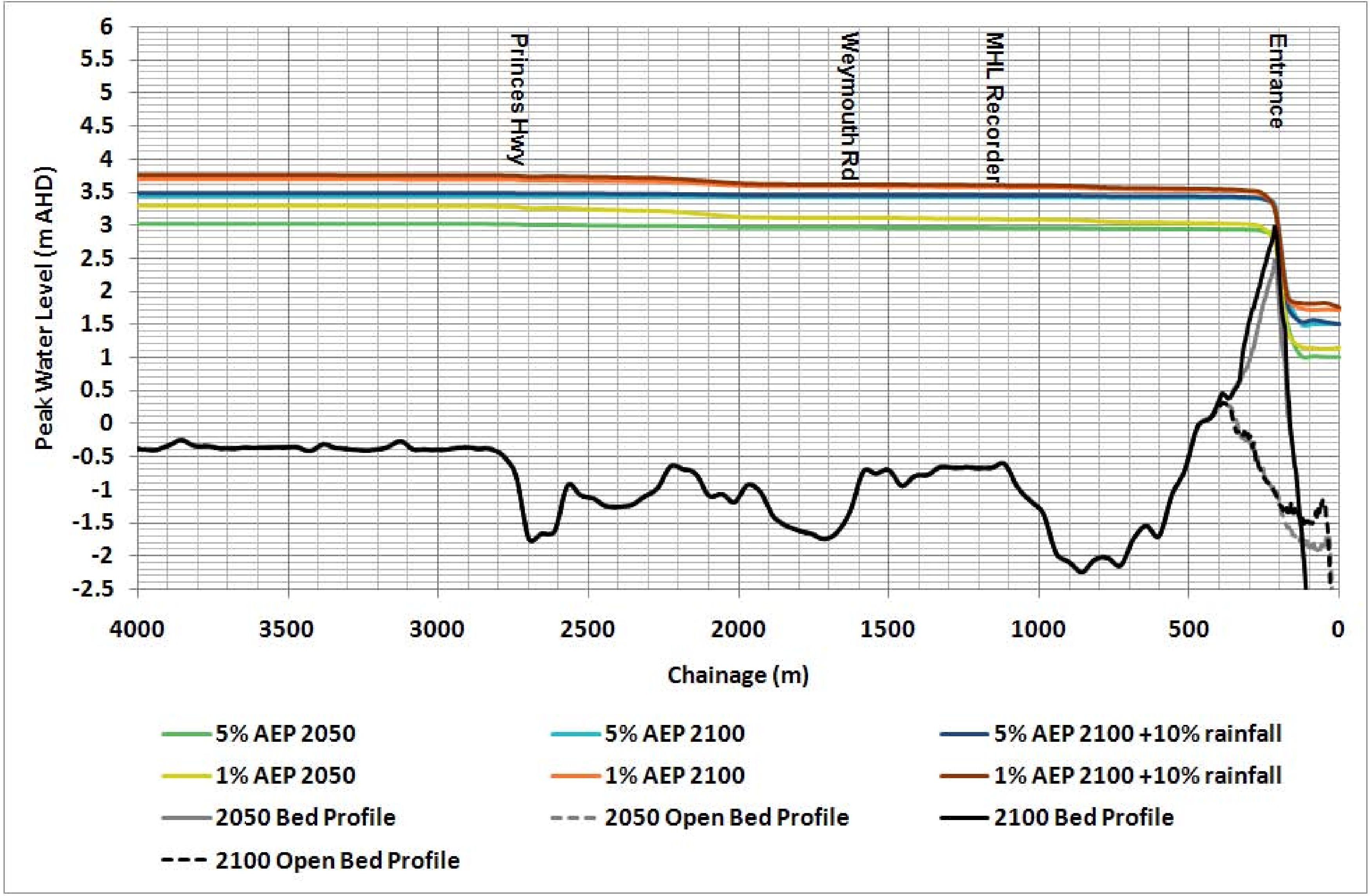


Figure 8-9 Design Flood Level profiles for Catchment Derived Events Incorporating Future Climate Change Predictions

Table 8-5 Estimated Peak Flood Levels for Ocean Derived Design Events Incorporating Future Climate Change Predictions

Location	Design Event Frequency and Climate Change Condition			
	5% AEP 2050	5% AEP 2100	1% AEP 2050	1% AEP 2100
Entrance	2.80	3.50	3.02	3.73
MHL Recorder	2.82	3.51	3.03	3.75
D/S Princes Hwy	2.85	3.54	3.06	3.78
U/S Princes Hwy	2.86	3.54	3.07	3.78
Tabourie Lake	2.87	3.55	3.08	3.79
Centre St	2.82	3.51	3.03	3.75
South St	2.82	3.51	3.04	3.75

Longitudinal profiles showing predicted flood levels along Tabourie Creek for the ocean derived climate change events are shown in Figure 8-10.

It can be seen that the flood levels throughout Tabourie Creek are similar to the peak level of the design ocean boundary. There is a small hydraulic gradient associated with the coincident 20% AEP catchment event which has been adopted for the ocean derived design events.

8.3.3 PMF Events

As for the catchment derived events, the most significant impact of future climate change on catchment derived flood events within Tabourie Lake will be an increased berm height. The rate of rise in the berm elevation is similar to that of sea-level rise. Climate change increases have also been applied to the 0.5% design ocean boundary utilised in the PMF event. The combined increase in peak ocean levels (from both sea-level rise and wave set up) for design ocean storms is around 0.5m by 2050 and 1.2m by 2100.

Table 8-6 summarises modelled peak flood levels at key locations for the PMF climate change events. By comparing the baseline PMF flood level at the entrance of 2.90m AHD it can be seen that the flood levels for the 2050 and 2100 planning horizons increase by 0.46m and 1.15m respectively. This is similar to the increased ocean boundary. Elsewhere, the flood levels increase by 0.14m to 0.21m for the 2050 planning horizon and by 0.38m to 0.62m for the 2100 planning horizon. Although not equivalent to the increase in berm height, this increase can be approximated as being around half the increase in berm height. The additional 10% rainfall input also considered for the 2100 planning horizon results in a reasonable increase in modelled flood levels, typically between 0.1m to 0.2m.

Table 8-6 Design PMF Flood Levels Incorporating Climate Change

Location	Design Event Frequency and Climate Change Condition		
	PMF 2050	PMF 2100	PMF 2100 +10% rainfall
Entrance	3.36	4.05	4.07
MHL Recorder	4.18	4.59	4.70
D/S Princes Hwy	4.89	5.14	5.32
U/S Princes Hwy	4.92	5.16	5.34
Tabourie Lake	4.94	5.18	5.36
Centre St	4.19	4.60	4.71
South St	4.20	4.60	4.71

8.4 Hydraulic Categorisation

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

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

Floodway - Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.

Flood Storage - Areas that are important in the temporary storage of the floodwater during the passage of the flood. If the area is substantially removed by levees or fill it will result in elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.

Flood Fringe - Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant affect on the flood pattern or flood levels.

A number of approaches were considered when attempting to define flood impact categories across the Tabourie Lake catchment. Approaches to define hydraulic categories that were considered for this assessment included partitioning the floodplain based on:

- Peak flood velocity;
- Peak flood depth;
- Peak velocity * depth (sometimes referred to as unit discharge);
- Cumulative volume conveyed during the flood event; and
- Combinations of the above.

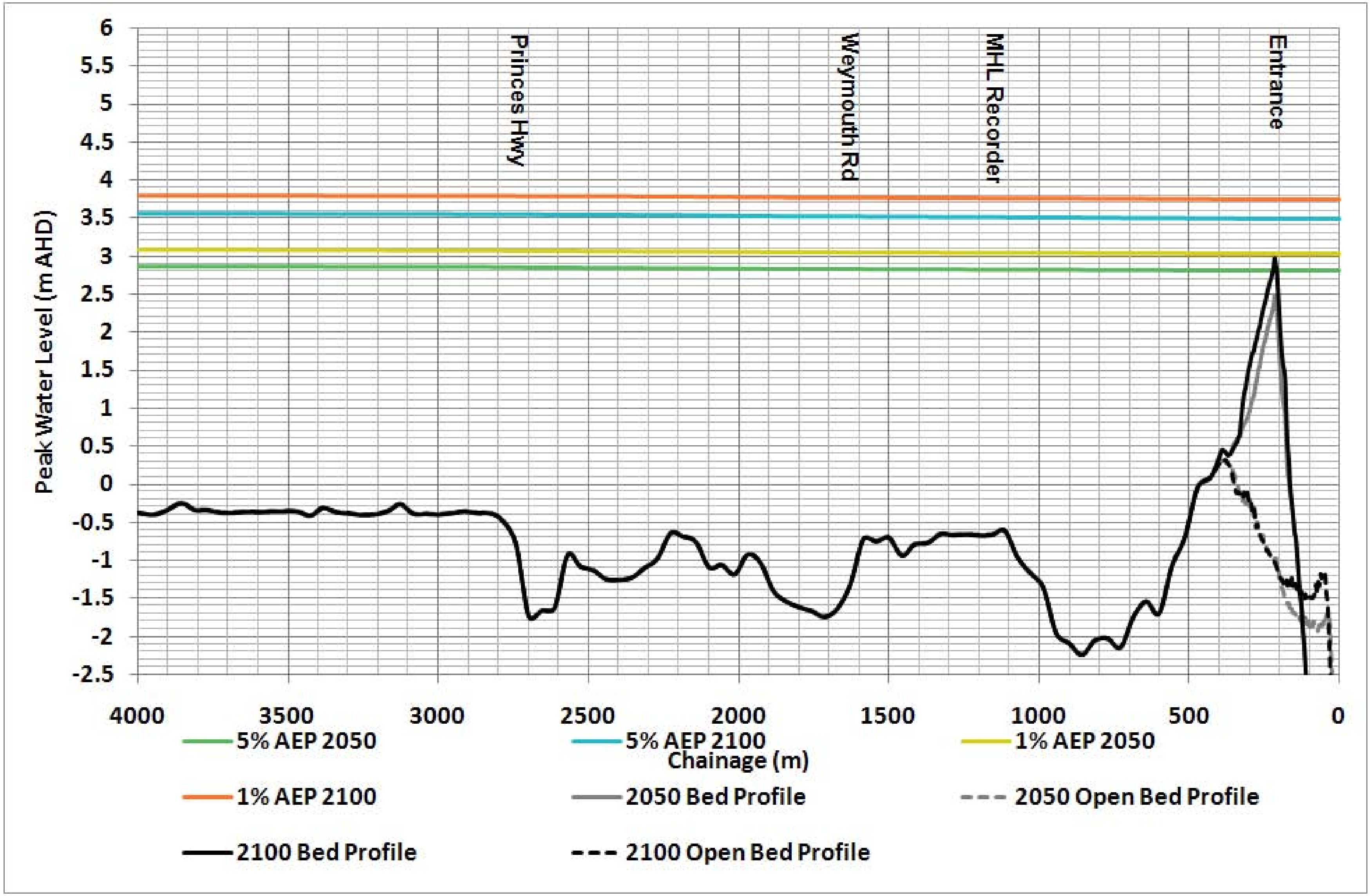


Figure 8-10 Design Flood Level Profiles for Ocean Derived Events Incorporating Future Climate Change Predictions

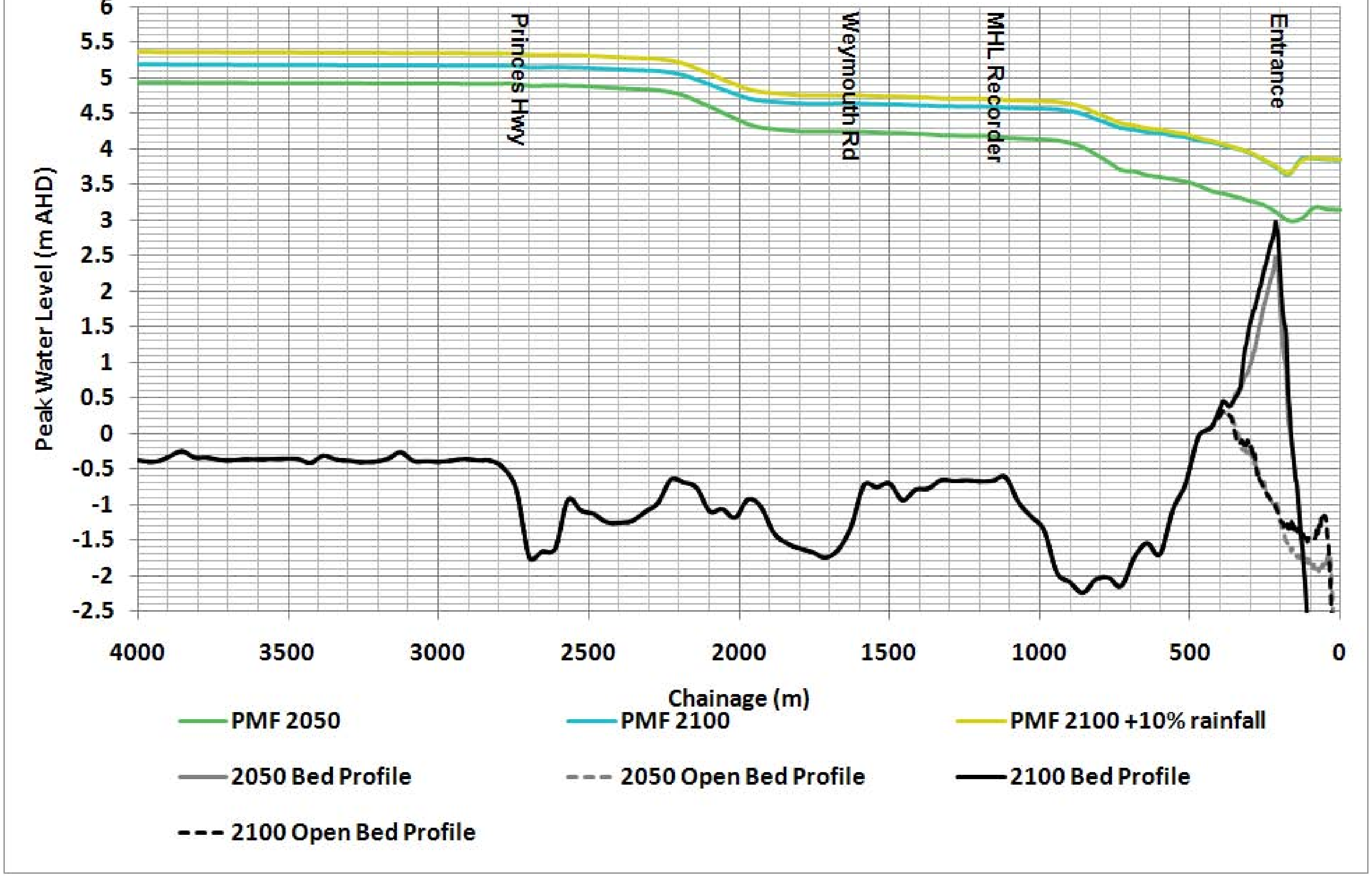


Figure 8-11 Design Flood Level Profiles for PMF Events Incorporating Future Climate Change Predictions

The definition of flood impact categories that was considered to best fit the application within the Tabourie Lake catchment, was based on a combination of velocity*depth and depth parameters. The adopted hydraulic categorisation is defined in Table 8-7.

Preliminary hydraulic category mapping for the 5% AEP, 1% AEP and PMF design events is included in Appendix A. It is also noted that mapping associated with the flood hydraulic categories may be amended in the future, at a local or property scale, subject to appropriate analysis that demonstrates no additional impacts (e.g. if it is to change from floodway to flood storage).

Table 8-7 Hydraulic Categories

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

8.5 Provisional Hazard

The NSW Government's Floodplain Development Manual (2005) defines flood hazard categories as follows:

High hazard – possible danger to personal safety; evacuation by trucks is difficult; able-bodied adults would have difficulty in wading to safety; potential for significant structural damage to buildings; and

Low hazard – should it be necessary, trucks could evacuate people and their possessions; able-bodied adults would have little difficulty in wading to safety.

The key factors influencing flood hazard or risk are:

- Size of the Flood
- Rate of Rise - Effective Warning Time
- Community Awareness
- Flood Depth and Velocity
- Duration of Inundation
- Obstructions to Flow
- Access and Evacuation

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

Figures L1 and L2 in the Floodplain Development Manual (NSW Government, 2005) are used to determine provisional hazard categorisations within flood liable land. These figures are reproduced in Figure 8-12.

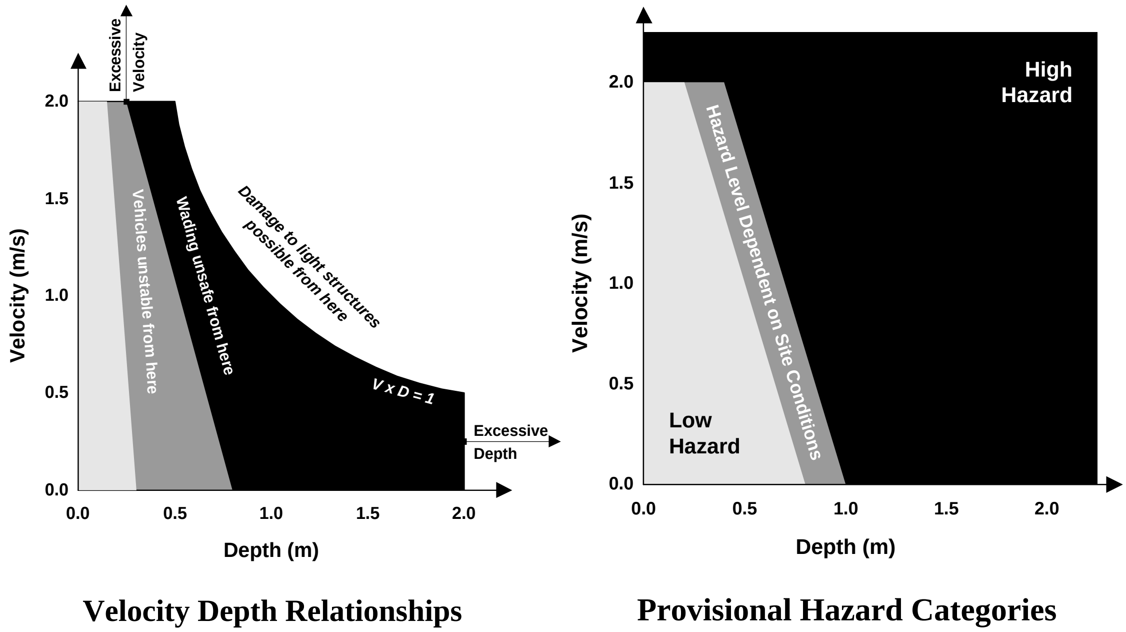


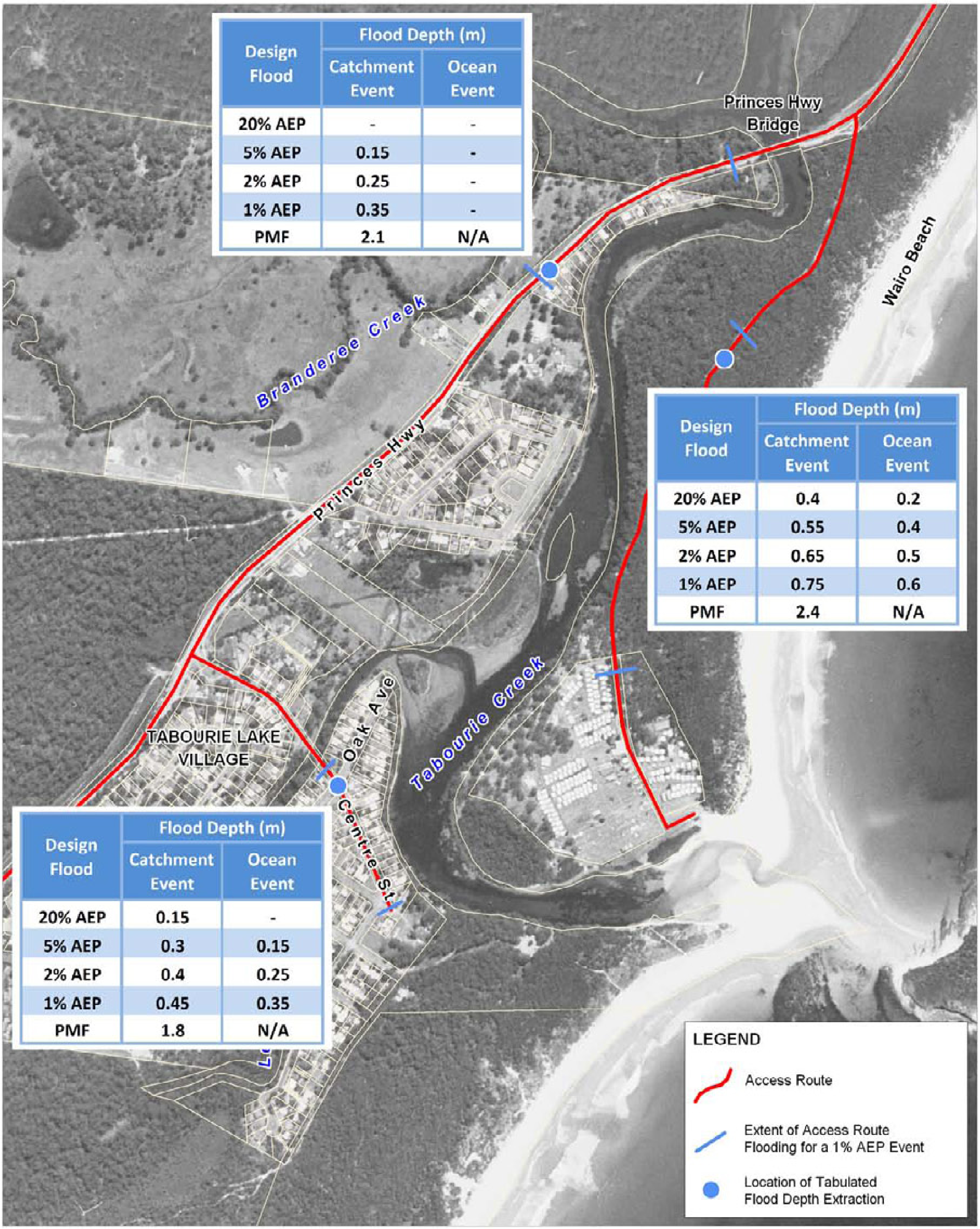
Figure 8-12 Provisional Flood Hazard Categorisation

The provisional hydraulic hazard is included in the mapping series provided in Appendix A for the 5% AEP, 1% AEP and PMF events.

8.6 Flooding of Access Routes

The potential flooding of access routes is a key concern when planning for and managing emergency response to a flood event. Tabourie Lake village is located beside several natural physical barriers, including Tabourie Creek, Branderee Creek, Lemon Tree Creek and the ocean. A few locations within the village are served by a single access route. This can leave certain areas susceptible to risk of isolation during a flood event. Understanding these risks enables better planning and management for emergency response.

As was shown on Figure 8-3, flooding poses a risk to access routes such as the Princes Highway, Centre Street and the Caravan Park access road. Figure 8–13 highlights these access routes and provides information on the extent and depth of flooding along these routes. The locations selected for the presentation of flood depths on the Princes Highway and the Caravan Park access road represent both the point of initial overtopping and maximum flood depth. The location on Centre Street, immediately to the east of the bridge has been selected as flooding in this location may prevent access across Lemon Tree Creek.



Design Flood	Flood Depth (m)	
	Catchment Event	Ocean Event
20% AEP	-	-
5% AEP	0.15	-
2% AEP	0.25	-
1% AEP	0.35	-
PMF	2.1	N/A

Design Flood	Flood Depth (m)	
	Catchment Event	Ocean Event
20% AEP	0.4	0.2
5% AEP	0.55	0.4
2% AEP	0.65	0.5
1% AEP	0.75	0.6
PMF	2.4	N/A

Design Flood	Flood Depth (m)	
	Catchment Event	Ocean Event
20% AEP	0.15	-
5% AEP	0.3	0.15
2% AEP	0.4	0.25
1% AEP	0.45	0.35
PMF	1.8	N/A

Title:

Location and Depth of Flooding to Access Routes

Figure: 8-13

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8.7 Sensitivity Tests

A number of sensitivity tests have been undertaken on the modelled flood behaviour in Tabourie Lake. As previously discussed in the simulation of historical events for calibration and in simulation of design flood conditions, the following model sensitivities have been considered:

- Initial lake level – given the volumes of catchment runoff generated in flood events relative to the available storage of the lake system, the peak flood level was found to be relatively insensitive to initial lake water levels;

- Berm height – the berm height has been demonstrated to be the principal control on design flood water levels in Tabourie Lake. Peak flood level sensitivity to other model parameter or assumptions are generally minor in comparison to the relative impact of berm height; and

- Entrance scour – for the catchment derived flood events, both a fixed berm and mobile berm condition were simulated, with the fixed berm representing a conservative condition. The initiation and propagation of a natural breakout is a very complex process. The simulated entrance breakout provides a representative effect of the development of a scour channel, showing a moderate reduction in peak flood levels compared to the fixed berm.

Further sensitivity tests are described in the following sections. The sensitivity of flood behaviour to the blockage of major hydraulic structures is also considered as discussed in Section 8.7.1. Additional sensitivity testing of the berm morphology has been undertaken considering the current related roughness parameter as described in Section 8.7.2.

8.7.1 Structure Blockage

There are two structures within the Tabourie Lake catchment that may exacerbate flood risk should a blockage occur. These structures are the Princes Highway bridge across Tabourie Creek and the Centre Street bridge over Lemon Tree Creek. The Princes Highway bridge is supported by piers whilst the Centre Street bridge is a clear span type. The diagonal opening length for both structures exceeds 6m and so a 25% blockage from the bed up has been assumed for sensitivity testing purposes.

Another minor culvert is located on a minor tributary crossing the Princes Highway (refer to Section 5.3.4. The culvert is located upstream of Tabourie Lake village. A blockage of this culvert would provide for storage behind the Princes Highway, thereby attenuating peak flows. Given that culvert blockage would reduce design peak flows through Tabourie Lake village, no blockage was assumed for this structure.

The modelled peak flood levels at key locations are provided in Table 8–8. This compares flood levels for the 25% blockage scenario to the baseline conditions. It can be seen that the blockage to the Princes Highway bridge has no discernable impact on modelled peak flood levels. This is due to the relatively small hydraulic gradient through the bridge given the backwater influence of the design berm condition. The blockage of the Lemon Tree Creek structure has a very localised effect, with a small increase (less than 100mm) immediately upstream of the bridge.

Table 8-8 Estimated Peak Flood Levels for Blockage Sensitivity Tests on the 1% AEP Event

Location	Sensitivity Analysis Condition for the 1% AEP Event	
	Baseline	25% Blockage
Entrance	1.30	1.30
MHL Recorder	1.77	1.77
D/S Princes Hwy	2.42	2.42
U/S Princes Hwy	2.57	2.57
Tabourie Lake	2.59	2.59
Centre St	1.95	2.02
South St	2.58	2.60

8.7.2 Current Related Roughness

The current related roughness is a coefficient in the morphological model used in the sand transport equations associated with scour of the entrance. A higher roughness results in greater bed shear stress, sediment transport and resulting scour. The current related roughness value adopted for the Tabourie Lake model runs is 0.10. Sensitivity runs were carried out with current related roughness values of 0.05 and 0.20. Table 8-9 shows the modelled peak flood levels at key locations for the baseline 1% AEP event and the same event with both increased and decreased current related roughness. It can be seen that the adopted current related roughness values have minimal impact on the modelled peak flood levels. There is a marginal decrease in flood levels when the current related roughness is increased.

Table 8-9 Estimated Peak Flood Levels for Current Related Roughness Sensitivity Tests on the 1% AEP Event

Location	Sensitivity Analysis Condition for the 1% AEP Event		
	Baseline	Increased CRR	Decreased CRR
Entrance	2.53	2.52	2.53
MHL Recorder	2.63	2.62	2.63
D/S Princes Hwy	2.91	2.90	2.91
U/S Princes Hwy	3.01	3.01	3.01
Tabourie Lake	3.02	3.02	3.02
Centre St	2.66	2.65	2.66
South St	2.78	2.78	2.78

9 CONCLUSIONS

The objective of the study was to undertake a detailed flood study of the Tabourie Lake catchment and establish models as necessary for flood level prediction. Central to this was the development of a two-dimensional hydraulic model of the floodplain incorporating Tabourie Lake, Tabourie Creek and the Tabourie Lake village area.

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

- Collation of database of historical flood information for the Tabourie Lake catchment;
- Acquisition of topographical data for the catchment including cross section and hydraulic structure survey;
- Consultation with the community to acquire historical flood information and liaison in regard to flooding concerns/perceptions and future floodplain management activities;
- Development of a hydrological model (using RAFTS-XP software) and hydraulic model (using TUFLOW software) to simulate flood behaviour in the catchment;
- Calibration of the developed models using the March 1975 and April 1988 flood events;
- Prediction of design flood conditions in the catchment using the developed models; and
- Production of design flood mapping series.

The berm height is the key factor in the design flood level estimation for catchment derived flood events in Tabourie Lake. The development of the entrance berm is a dynamic process driven by both catchment hydrology and prevailing tidal and wave conditions. While typical berm elevations are around 2.0 m AHD, Shoalhaven City Council's present management strategy is to excavate a channel through the barrier when water levels reach a level of 1.17, primarily to relieve flooding of low lying properties fringing the lake.

The results of the flood modelling undertaken clearly demonstrate the major influence of the entrance berm condition on peak flood levels for catchment derived flooding events. At present, manual opening of the entrance is undertaken to control water levels in Tabourie Lake. However, from a design flood perspective, where warning times may be limited to a few hours, a manual breakout may not be possible. As such design flood level estimation has been based on the assumption of the berm being intact at the onset of a major design rainfall event.

The storage volume in the Lake system is relatively minor in comparison to the flood volumes generated from catchment runoff in major events. Accordingly, the peak design flood levels are relatively insensitive to the initial lake condition. This is significant from a floodplain management perspective in that lake levels would be expected to rise relatively quickly, thereby limiting opportunity to undertake a manual breakout of the entrance.

Ocean derived flood events, such as the major event that occurred in 1974 in Tabourie Lake, pose significant risk to Tabourie Lake village. Extreme ocean conditions are likely to scour the entrance or indeed overtop the entrance berm providing for unrestricted penetration of elevated ocean levels into the estuary. The design ocean flood conditions poses similar risk in terms of peak flood inundation to Tabourie Lake village as the catchment derived floods.

Potential climate change scenarios have a major influence on design flood conditions at Tabourie Lake. The potential for sea level rise, which in turn provides for increases in berm heights, and increase in design rainfall intensities have been assessed in the flood study for the 2050 and 2100 planning horizons. The future climate change scenarios provide for substantial increases in design peak flood levels above existing conditions. An interesting observation of the design flood results are the relatively similar peak flood level estimations in major events for both the catchment derived flooding (fixed berm conditions) and the ocean derived flood levels. Therefore design flood planning levels in Tabourie Lake are expected to be of similar magnitude irrespective of entrance management and long term berm geometries, given the similar flood risk associated with ocean derived flooding.

The opportunity for ongoing entrance management in line with existing policy may be limited if sea level rise projections manifest. The current management level of 1.17m AHD is approximately equal to the projected 2100 mean water level in Tabourie (i.e. current mean sea level of 0.25m AHD +0.91m AHD sea level rise).

The flood study will form the basis for the subsequent floodplain risk management activities, being the next stage of the floodplain management process. The flood planning process will undoubtedly have strong links with entrance and estuary management.

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APPENDIX A: DESIGN FLOOD MAPPING

Existing Conditions

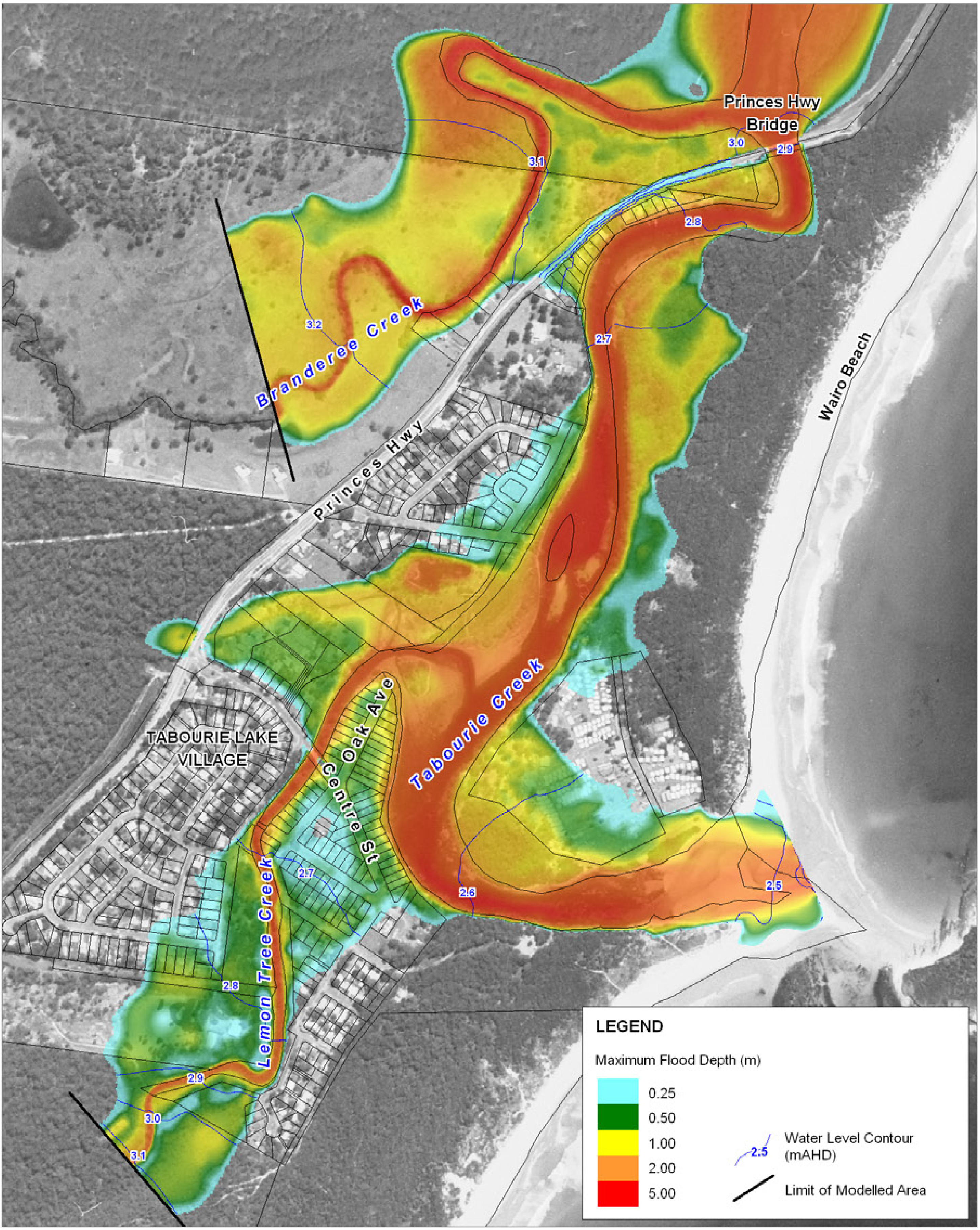
A-1 Catchment Derived Flood Event - 1% AEP Maximum Flood Depths and Water Levels
A-2 Catchment Derived Flood Event - 1% AEP Maximum Flood Velocities
A-3 Catchment Derived Flood Event - 1% AEP Flood Hazards and Hydraulic Categories
A-4 Catchment Derived Flood Event - 5% AEP Maximum Flood Depths and Water Levels
A-5 Catchment Derived Flood Event - 5% AEP Maximum Flood Velocities
A-6 Catchment Derived Flood Event - 5% AEP Flood Hazards and Hydraulic Categories
A-7 Catchment Derived Flood Event - 20% AEP Maximum Flood Depths and Water Levels
A-8 Catchment Derived Flood Event - 20% AEP Maximum Flood Velocities
A-9 Catchment Derived Flood Event - 20% AEP Flood Hazards and Hydraulic Categories
A-10 Catchment Derived Flood Event - PMF Maximum Flood Depths and Water Levels
A-11 Catchment Derived Flood Event - PMF Maximum Flood Velocities
A-12 Catchment Derived Flood Event - PMF Flood Hazards and Hydraulic Categories
A-13 Ocean Derived Flood Event - 1% AEP Maximum Flood Depths and Water Levels
A-14 Ocean Derived Flood Event - 1% AEP Maximum Flood Velocities
A-15 Ocean Derived Flood Event - 1% AEP Flood Hazards and Hydraulic Categories
A-16 Ocean Derived Flood Event - 5% AEP Maximum Flood Depths and Water Levels
A-17 Ocean Derived Flood Event - 5% AEP Maximum Flood Velocities
A-18 Ocean Derived Flood Event - 5% AEP Flood Hazards and Hydraulic Categories
A-19 Ocean Derived Flood Event - 20% AEP Maximum Flood Depths and Water Levels
A-20 Ocean Derived Flood Event - 20% AEP Maximum Flood Velocities
A-21 Ocean Derived Flood Event - 20% AEP Flood Hazards and Hydraulic Categories

2050 Conditions

A-22 Catchment Derived Flood Event - 1% AEP Maximum Flood Depths and Water Levels
A-23 Catchment Derived Flood Event - 1% AEP Maximum Flood Velocities
A-24 Catchment Derived Flood Event - 1% AEP Flood Hazards and Hydraulic Categories
A-25 Catchment Derived Flood Event - 5% AEP Maximum Flood Depths and Water Levels
A-26 Catchment Derived Flood Event - 5% AEP Maximum Flood Velocities
A-27 Catchment Derived Flood Event - 5% AEP Flood Hazards and Hydraulic Categories
A-28 Catchment Derived Flood Event - PMF Maximum Flood Depths and Water Levels
A-29 Catchment Derived Flood Event - PMF Maximum Flood Velocities
A-30 Catchment Derived Flood Event - PMF Flood Hazards and Hydraulic Categories
A-31 Ocean Derived Flood Event - 1% AEP Maximum Flood Depths and Water Levels
A-32 Ocean Derived Flood Event - 1% AEP Maximum Flood Velocities
A-33 Ocean Derived Flood Event - 1% AEP Flood Hazards and Hydraulic Categories
A-34 Ocean Derived Flood Event - 5% AEP Maximum Flood Depths and Water Levels
A-35 Ocean Derived Flood Event - 5% AEP Maximum Flood Velocities
A-36 Ocean Derived Flood Event - 5% AEP Flood Hazards and Hydraulic Categories

2100 Conditions

A-37 Catchment Derived Flood Event - 1% AEP Maximum Flood Depths and Water Levels



Title:

Catchment Derived Flood Event (Existing Conditions)
1% AEP Maximum Flood Depths and Water Levels

Figure: A-1

Rev: A

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N

0 200 400m

Approx. Scale

BMT WBM

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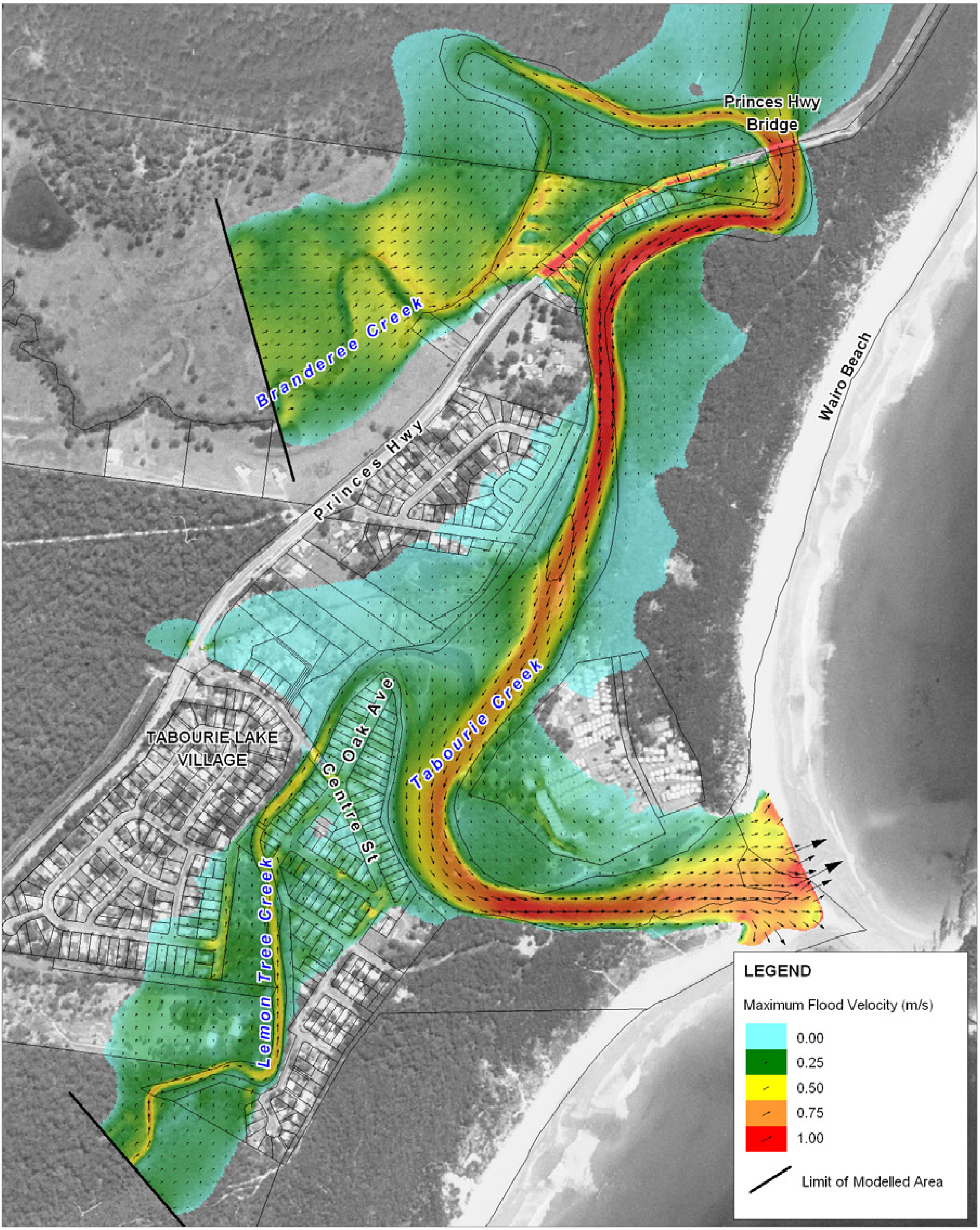
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_101_101115 EX CA 1% Depths.WOR



Title:

Catchment Derived Flood Event (Existing Conditions) 1% AEP Maximum Flood Velocities

Figure: A-2

Rev: A

N

0 200 400m

Approx. Scale

BMT WBM

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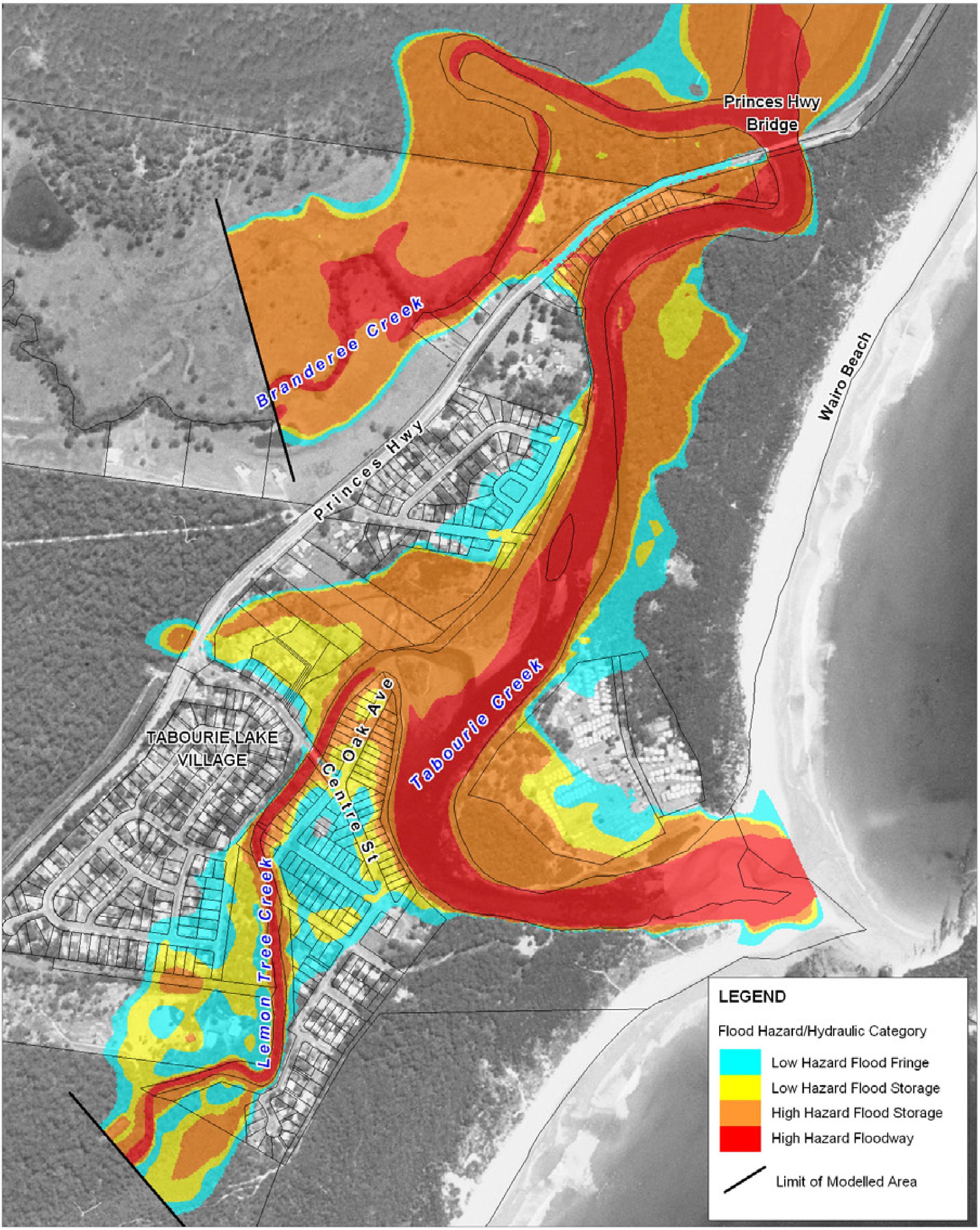
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_102_101115 EX CA 1% Velocities.WOR



Title:

Catchment Derived Flood Event (Existing Conditions) 1% AEP Flood Hazards and Hydraulic Categories

Figure: A-3

Rev: A

N

0 200 400m

Approx. Scale

BMT WBM

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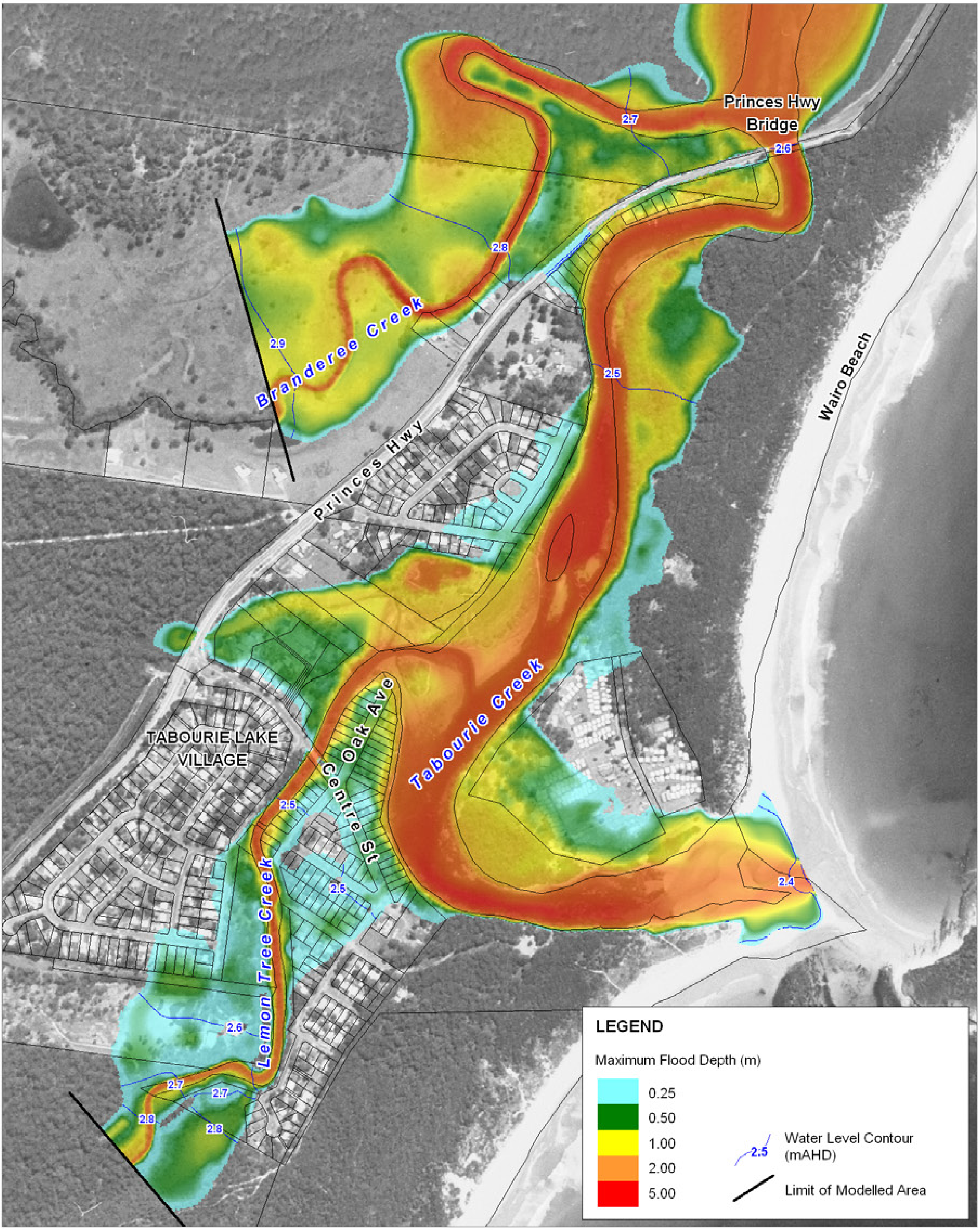
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_103_101115 EX CA 1% Hazards.WOR



Title:

**Catchment Derived Flood Event (Existing Conditions)
5% AEP Maximum Flood Depths and Water Levels**

Figure: A-4

Rev: A

N

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Approx. Scale

BMT WBM

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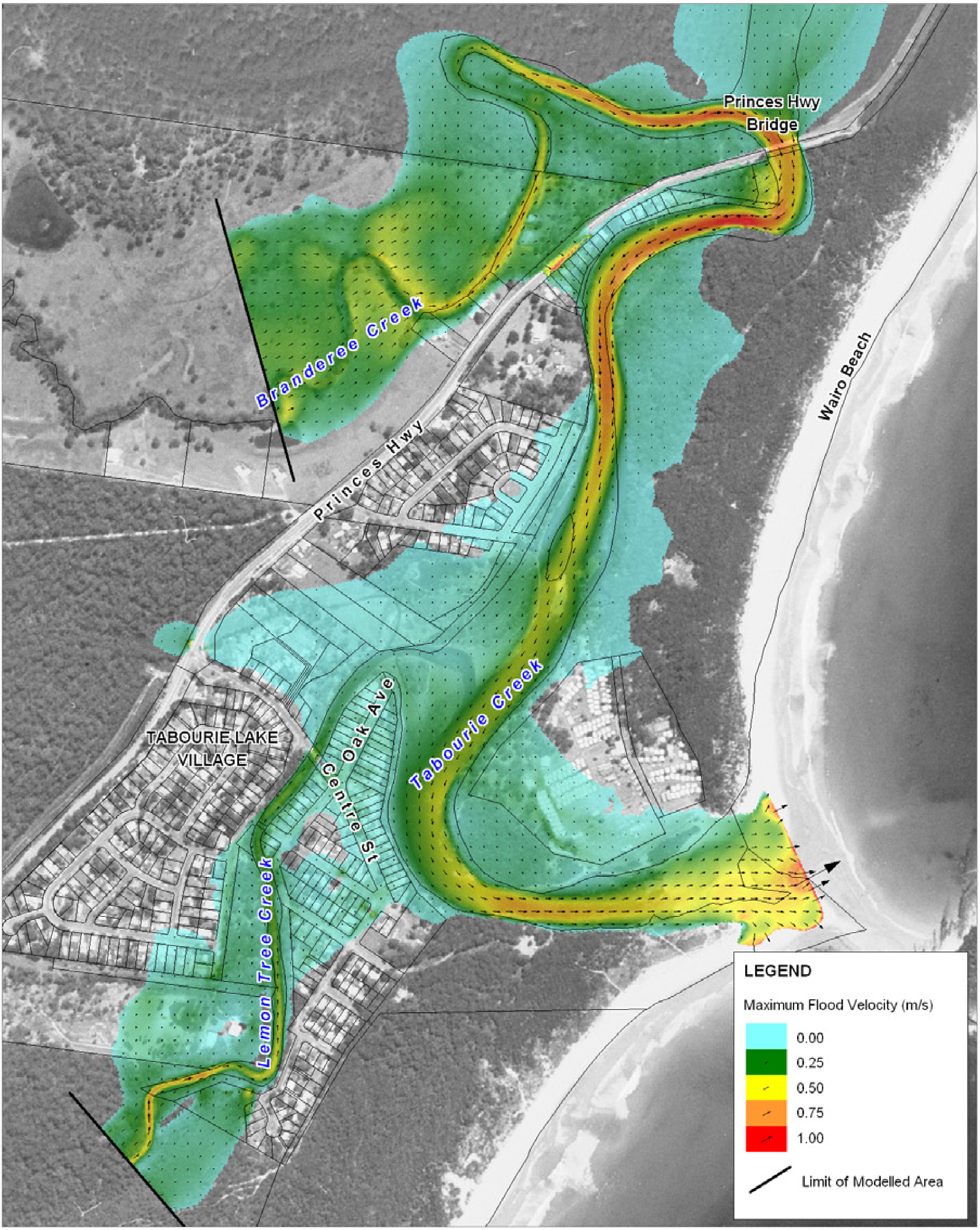
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_104_101118 EX CA 5% Depths.WOR



Title:

Catchment Derived Flood Event (Existing Conditions)
5% AEP Maximum Flood Velocities

Figure: **A-5**

Rev: **A**

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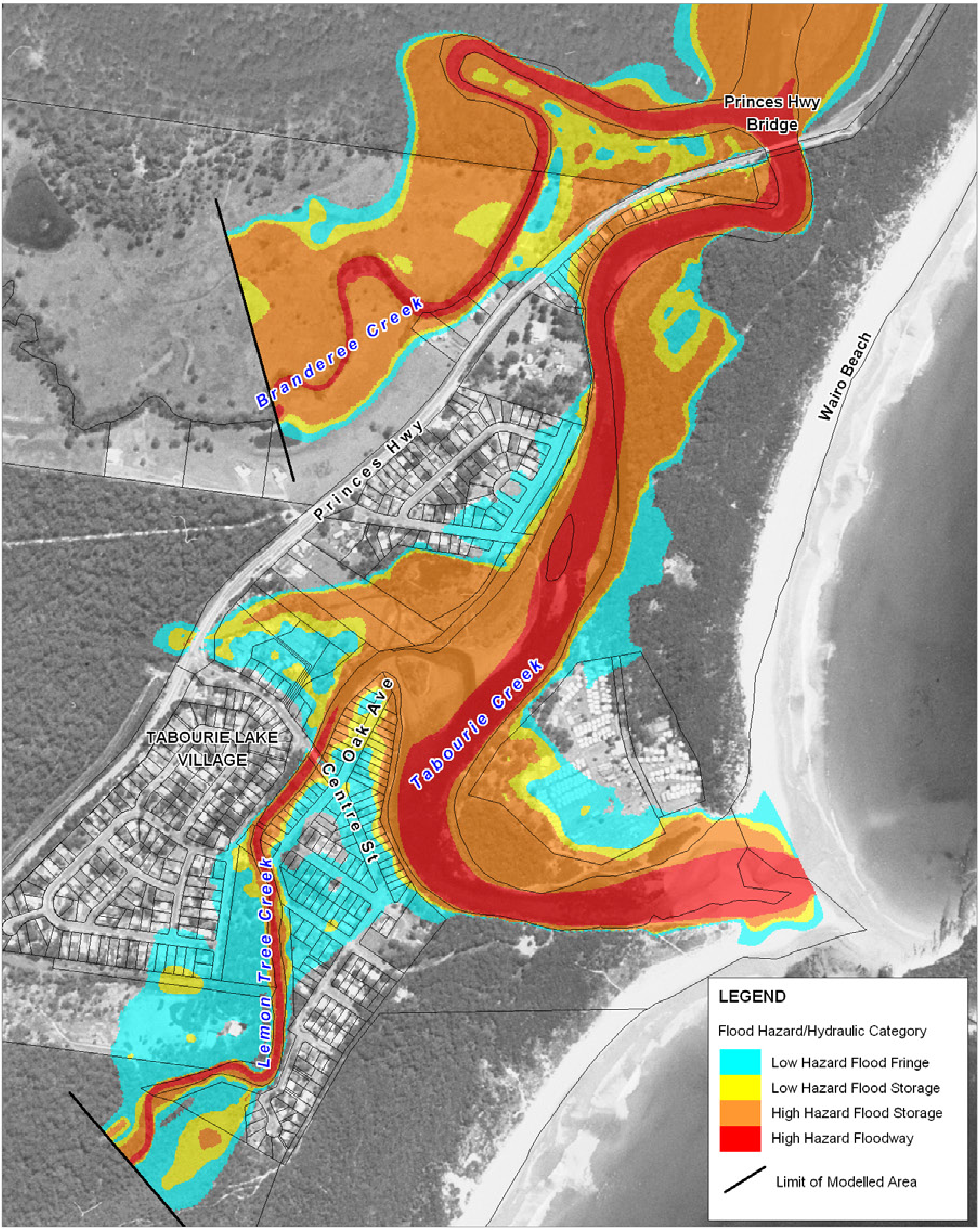
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_105_101118 EX CA 5% Velocities.WOR



Title:

Catchment Derived Flood Event (Existing Conditions)
5% AEP Flood Hazards and Hydraulic Categories

Figure: A-6

Rev: A

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N

0 200 400m
Approx. Scale

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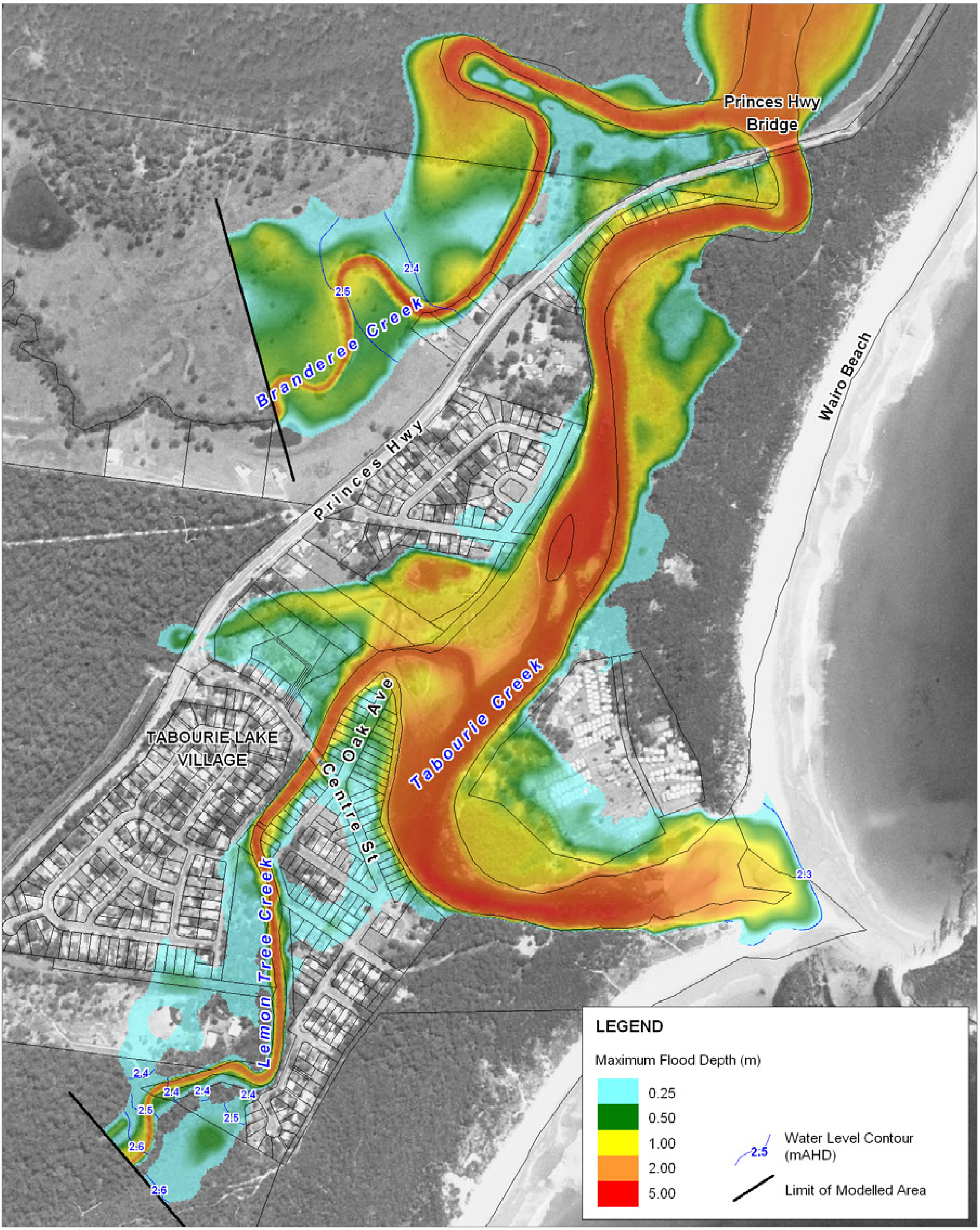

Title:

Catchment Derived Flood Event (Existing Conditions)
20% AEP Maximum Flood Depths and Water Levels

Figure: A-7

Rev: A

N

0 200 400m
Approx. Scale

BMT WBM
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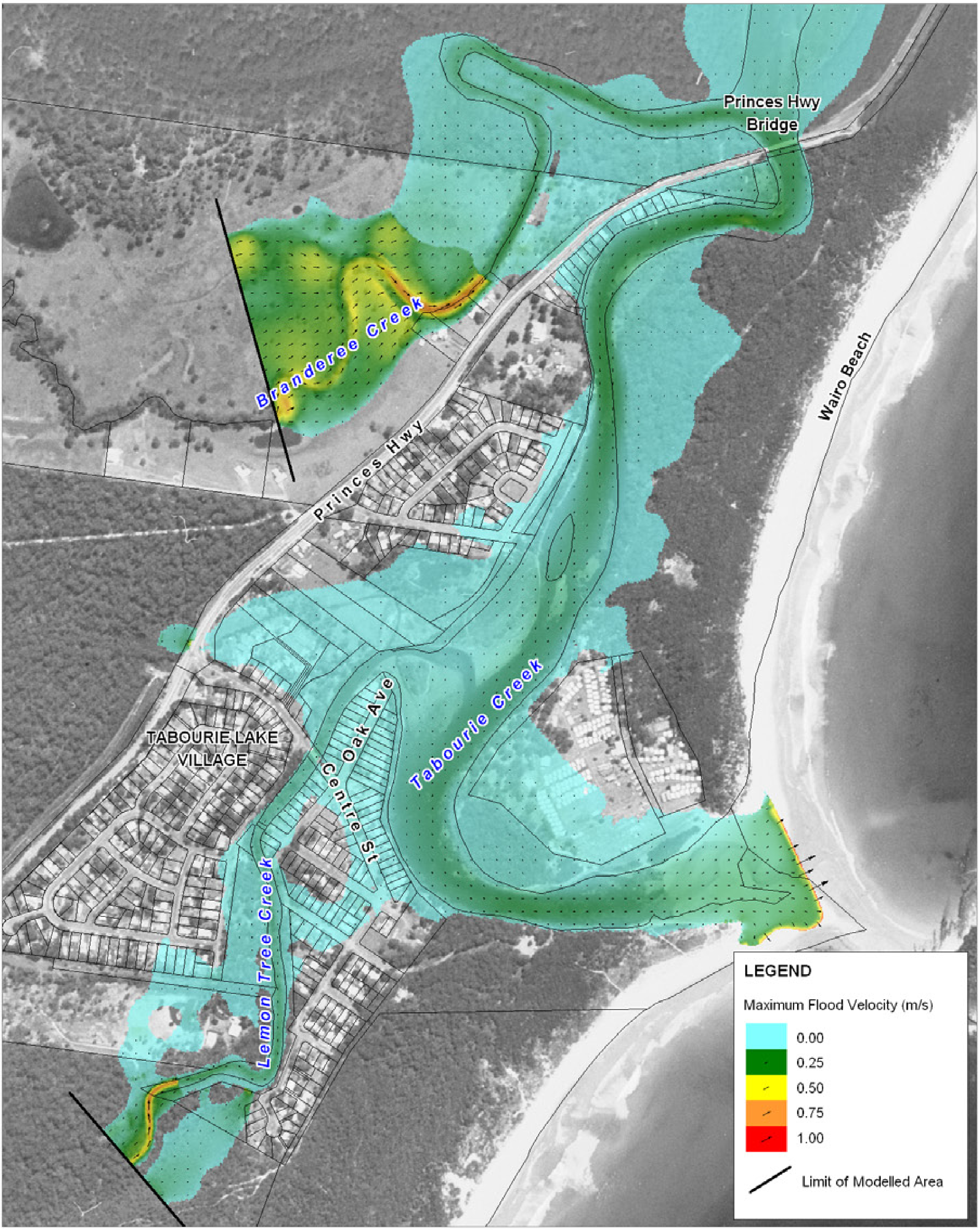
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_107_101118 EX CA 20% Depths.WOR



Title:

Catchment Derived Flood Event (Existing Conditions) 20% AEP Maximum Flood Velocities

Figure: A-8

Rev: A

N

0 200 400m

Approx. Scale

BMT WBM

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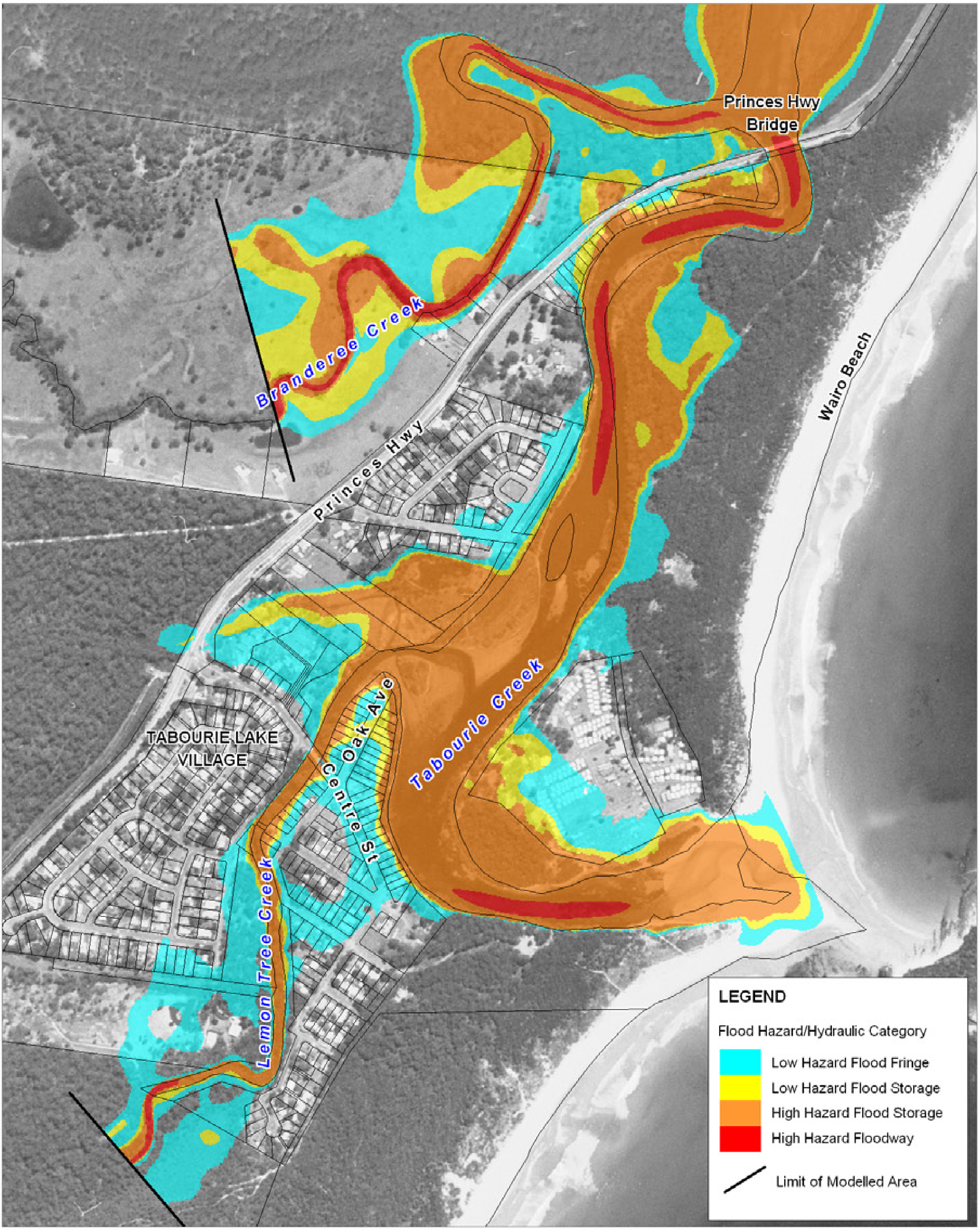

Title:

Catchment Derived Flood Event (Existing Conditions)
20% AEP Flood Hazards and Hydraulic Categories

Figure: **A-9**

Rev: **A**

N

0 200 400m
Approx. Scale

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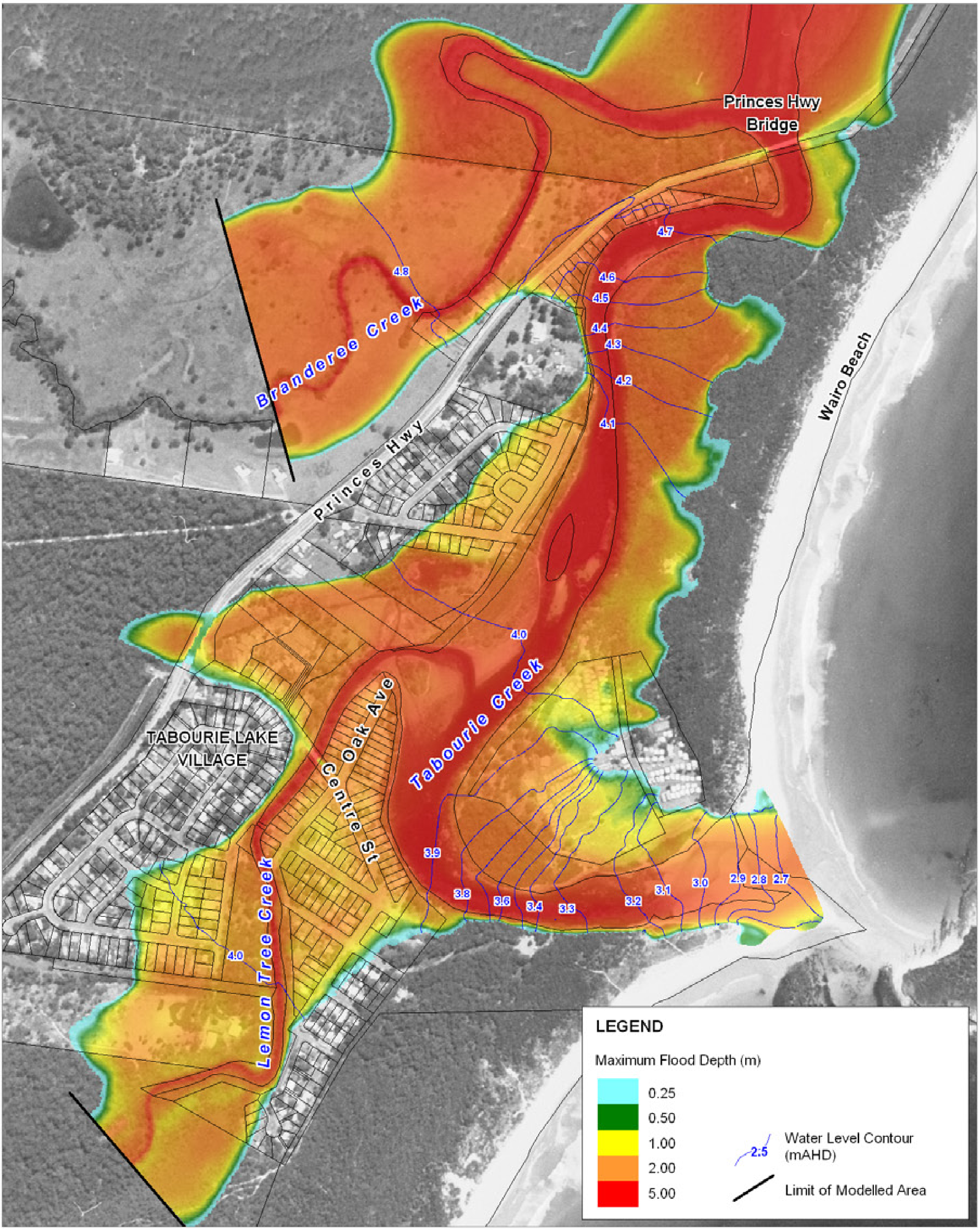
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_109_101118 EX CA 20% Hazards.WOR



Title:

PMF Flood Event (Existing Conditions)
Maximum Flood Depths and Water Levels

Figure: A-10

Rev: A

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N

0 200 400m

Approx. Scale

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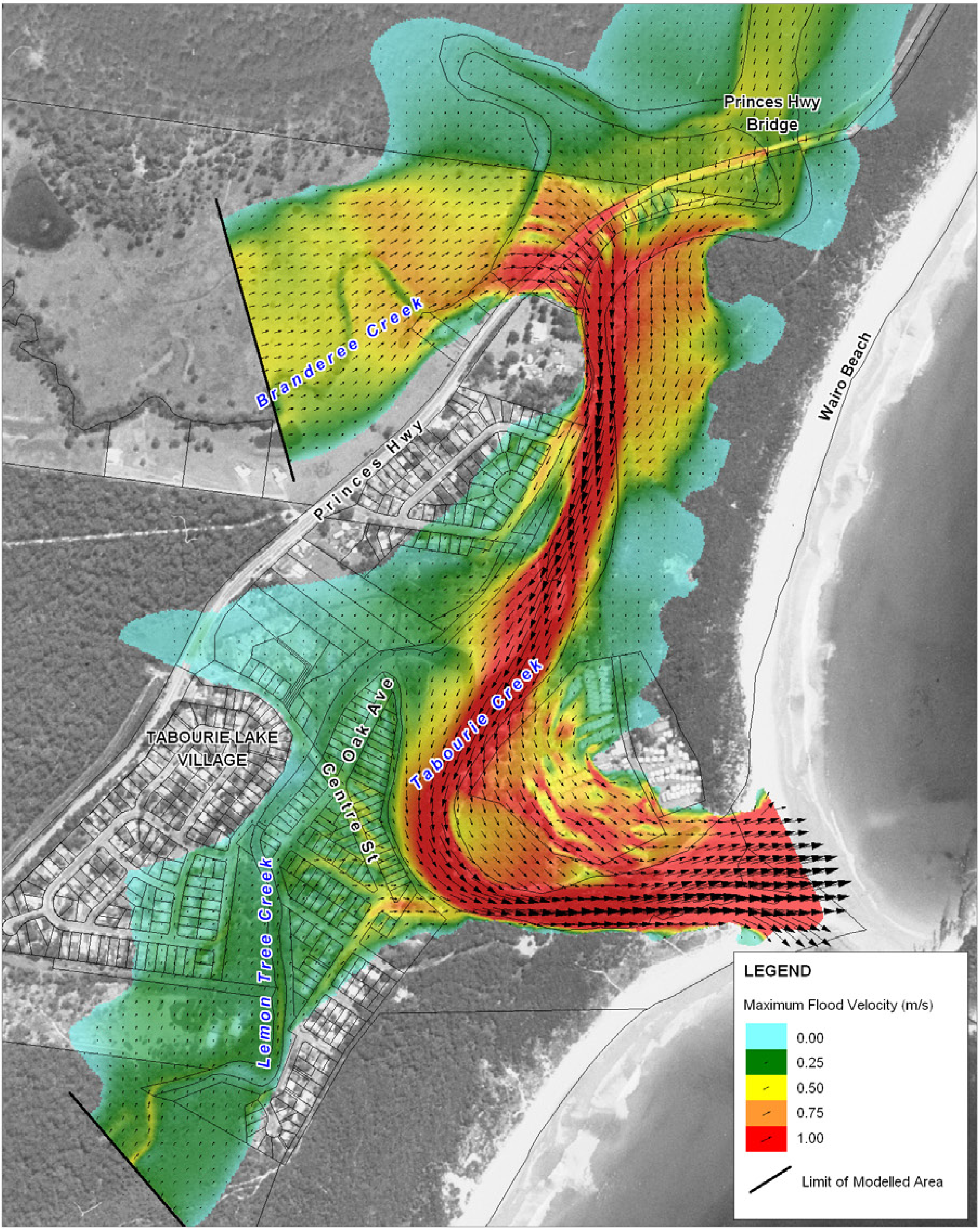
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_110_101118 EX PMF Depths.WOR



Title:

PMF Flood Event (Existing Conditions)
Maximum Flood Velocities

Figure: **A-11**

Rev: **A**

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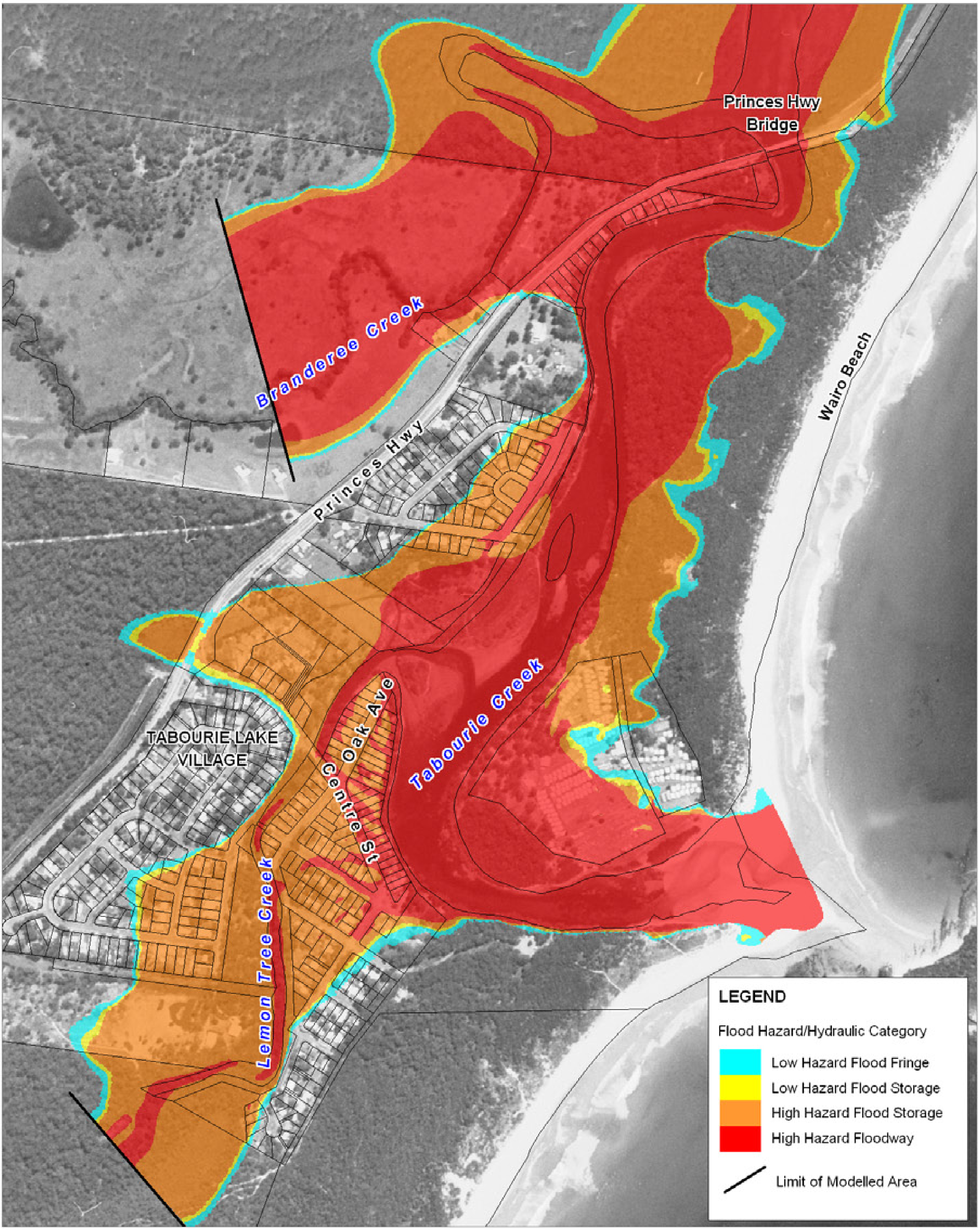
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_111_101118 EX PMF Velocities.WOR



Title:

PMF Flood Event (Existing Conditions) Flood Hazards and Hydraulic Categories

Figure: A-12

Rev: A

N

0 200 400m

Approx. Scale

BMT WBM

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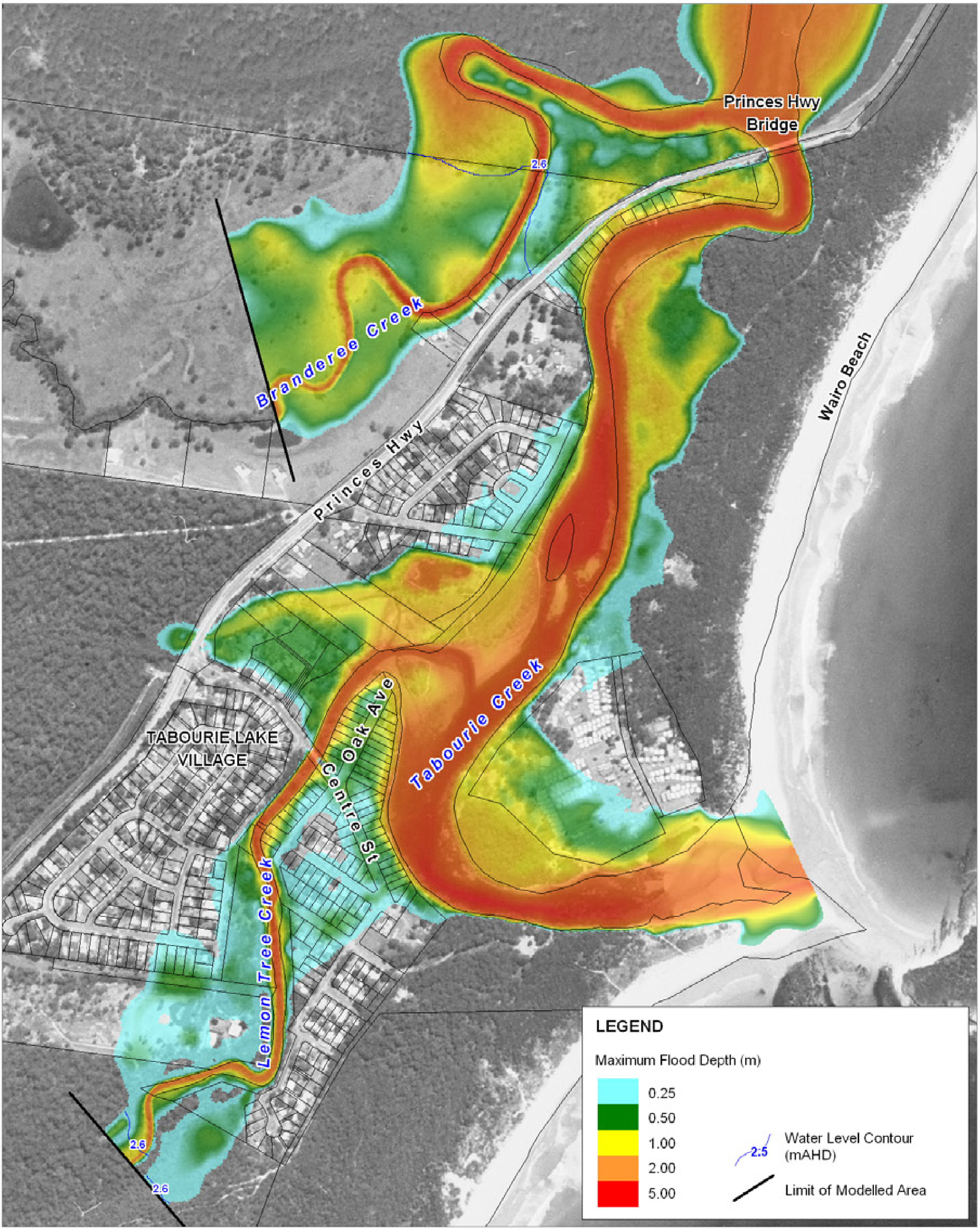
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_112_101118 EX PMF Hazards.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
1% AEP Maximum Flood Depths and Water Levels

Figure: **A-13**

Rev: **A**

N

0 200 400m
Approx. Scale

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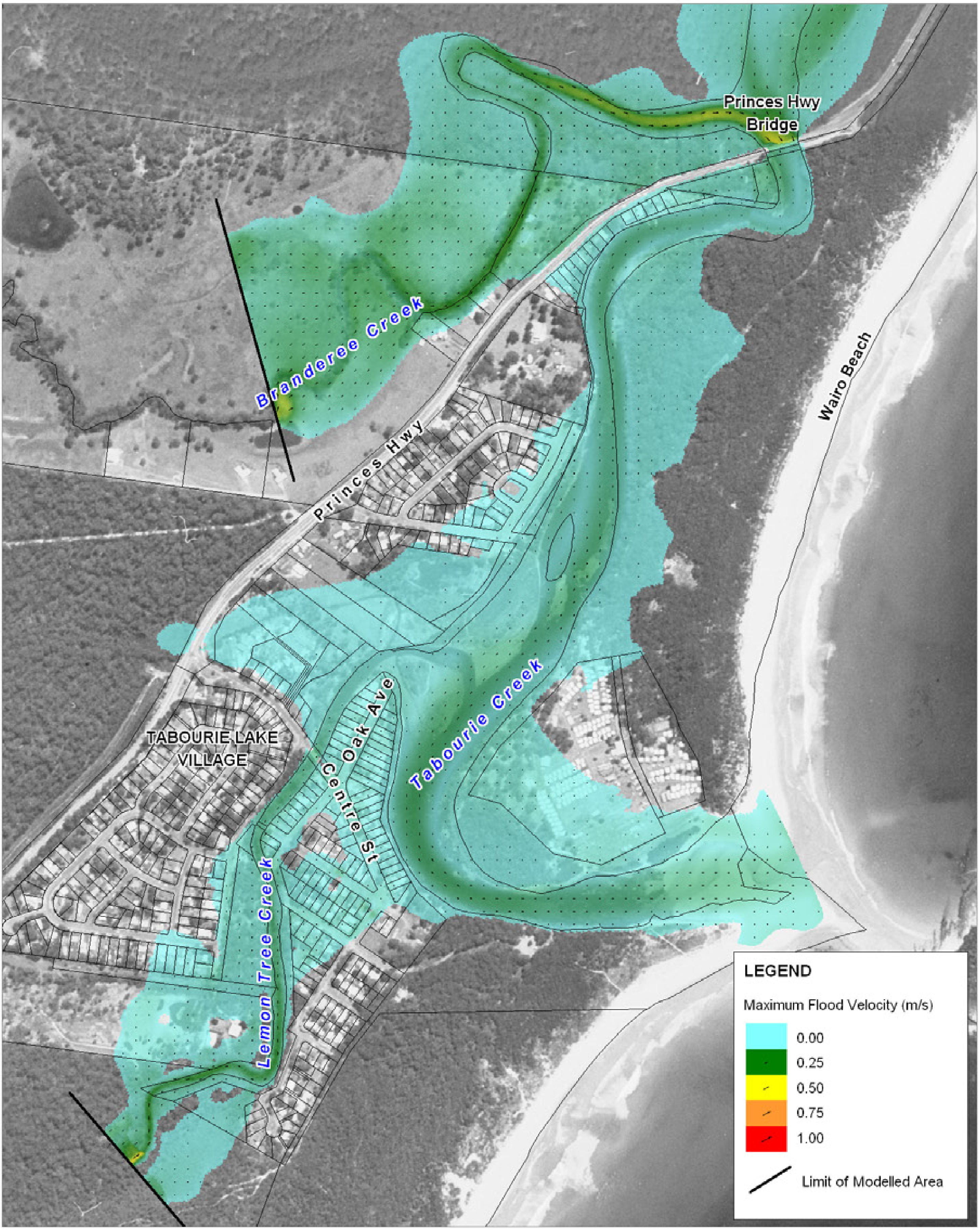
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_113_101118 EX OC 1% Depths.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
1% AEP Maximum Flood Velocities

Figure: A-14

Rev: A

N

0 200 400m

Approx. Scale

BMT WBM

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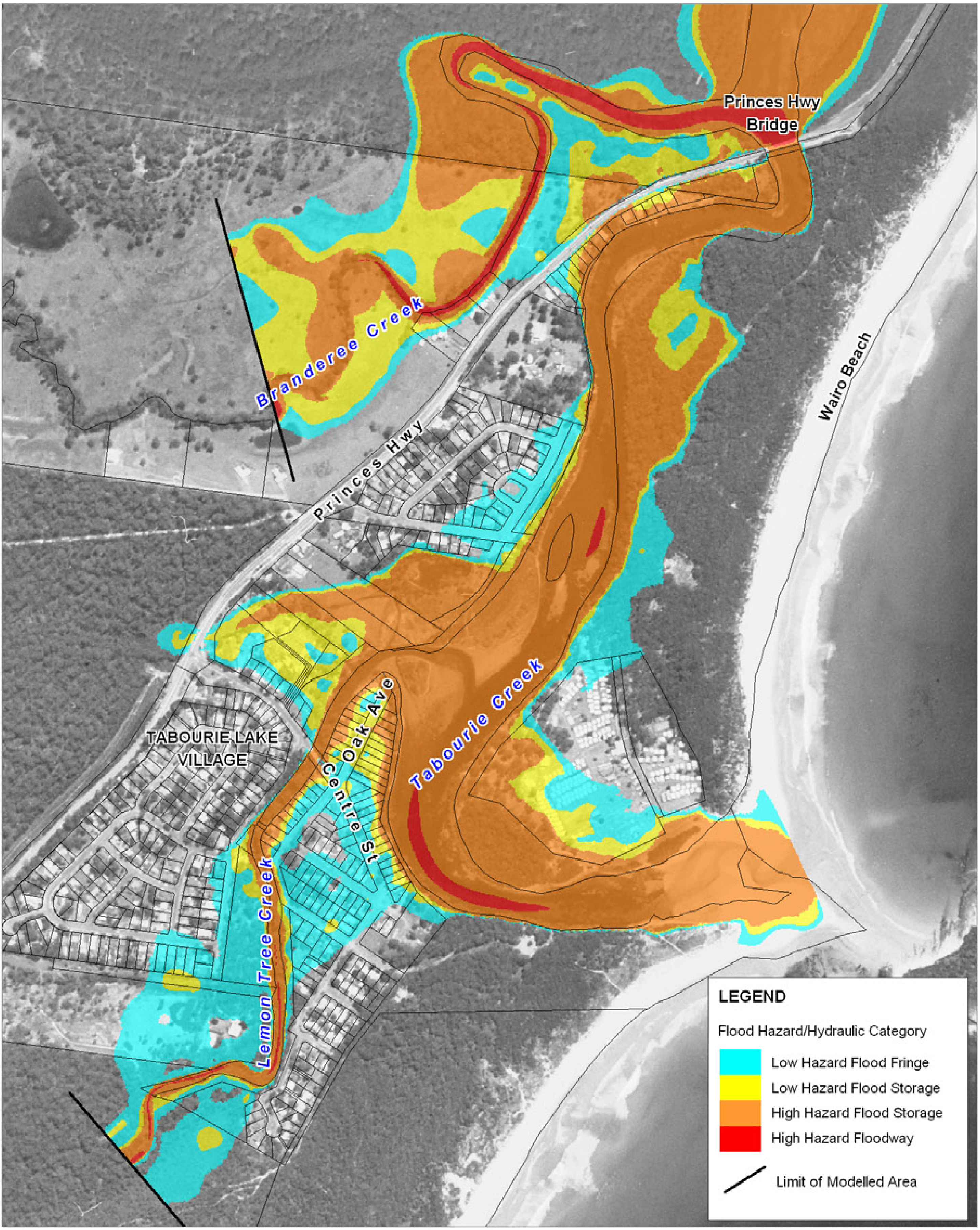
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_114_101118 EX OC 1% Velocities.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
1% AEP Flood Hazards and Hydraulic Categories

Figure: **A-15**

Rev: **A**

N

0 200 400m
Approx. Scale

BMT WBM
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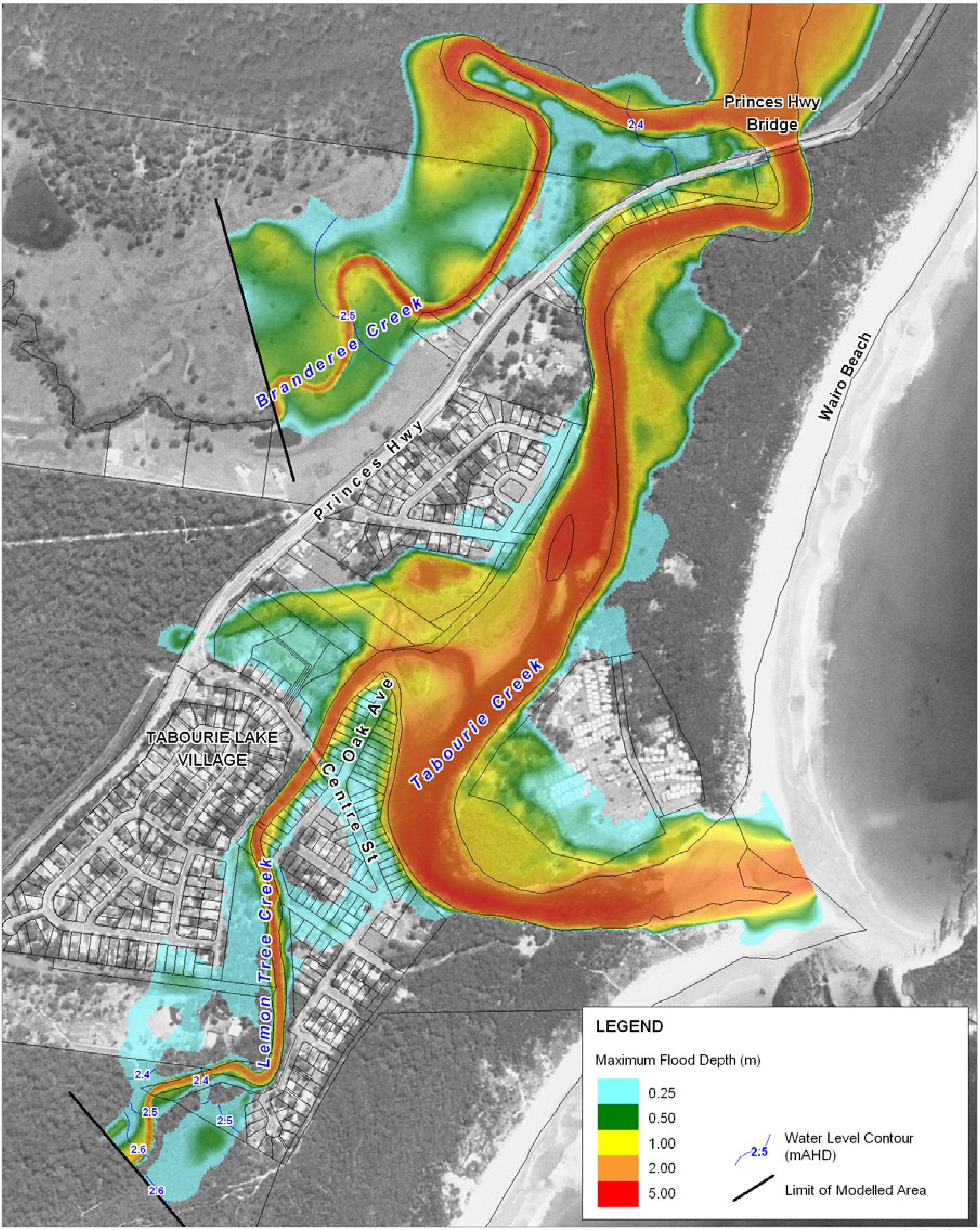
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_115_101118 EX OC 1% Hazards.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
5% AEP Maximum Flood Depths and Water Levels

Figure: **A-16**

Rev: **A**

N

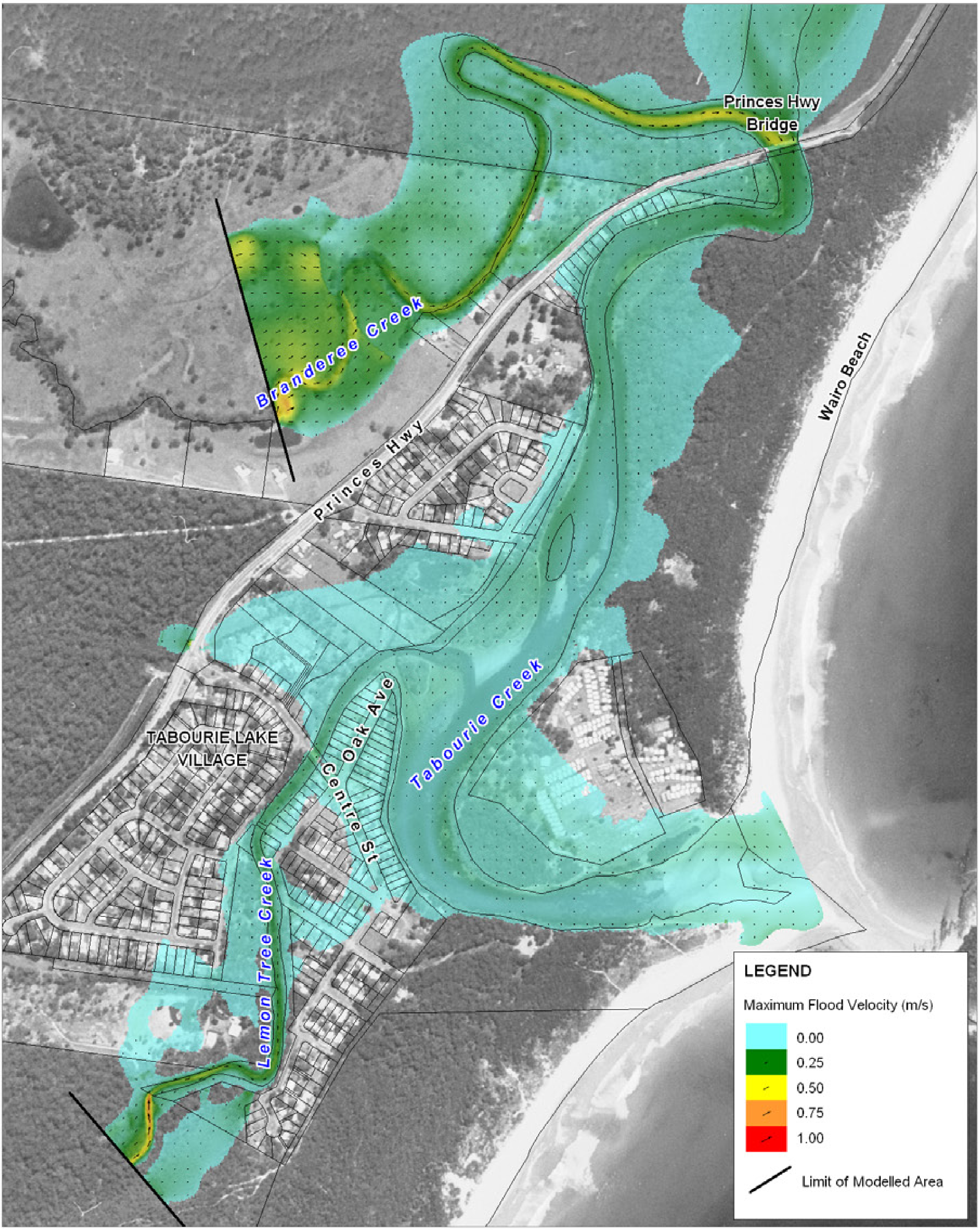
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_116_101118 EX OC 5% Depths.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
5% AEP Maximum Flood Velocities

Figure: A-17

Rev: A

N

0 200 400m
Approx. Scale

BMT WBM
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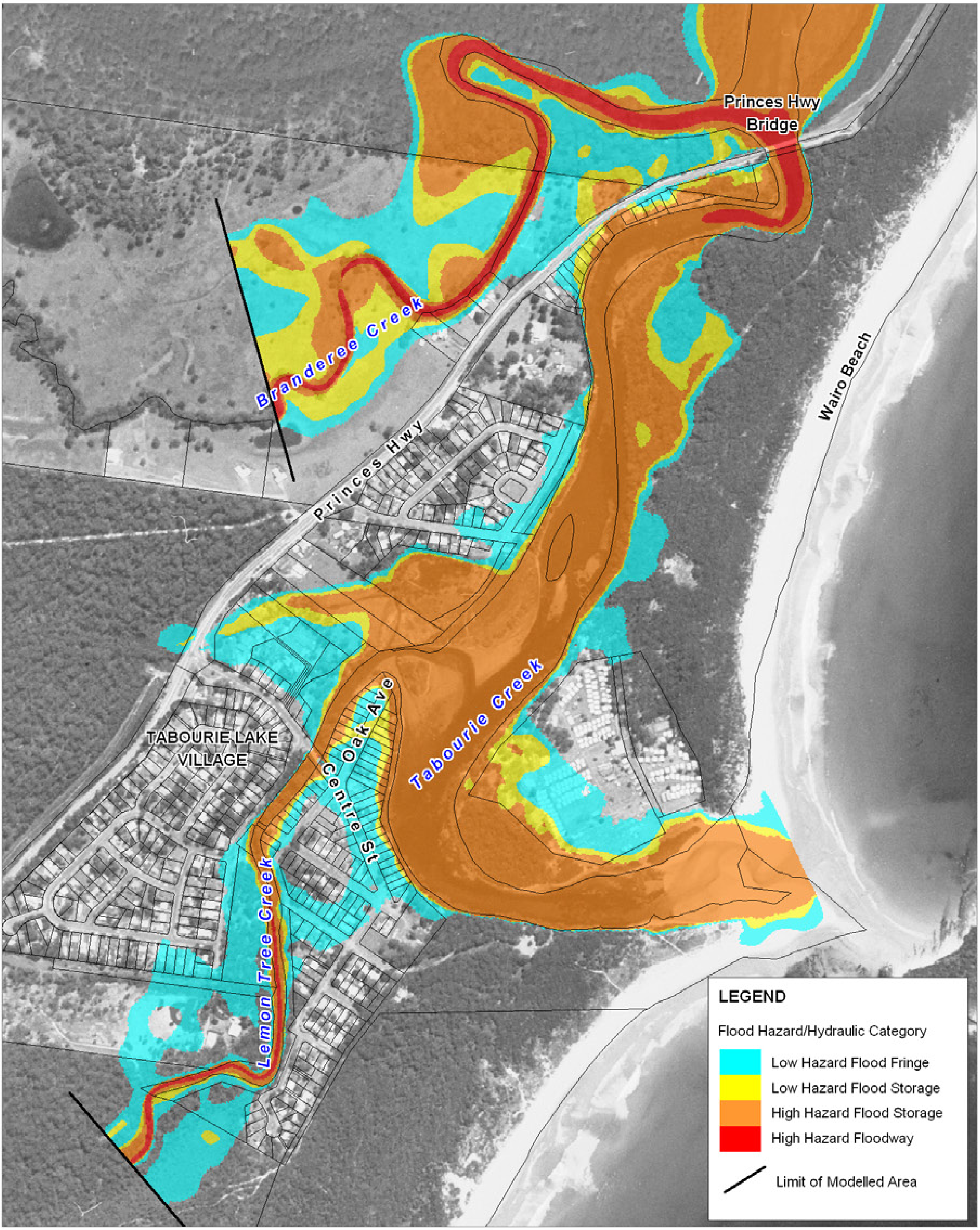
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_117_101118 EX OC 5% Velocities.WOR



Title:

Ocean Derived Flood Event (Existing Conditions) 5% AEP Flood Hazards and Hydraulic Categories

Figure: A-18

Rev: A

N

0 200 400m

Approx. Scale

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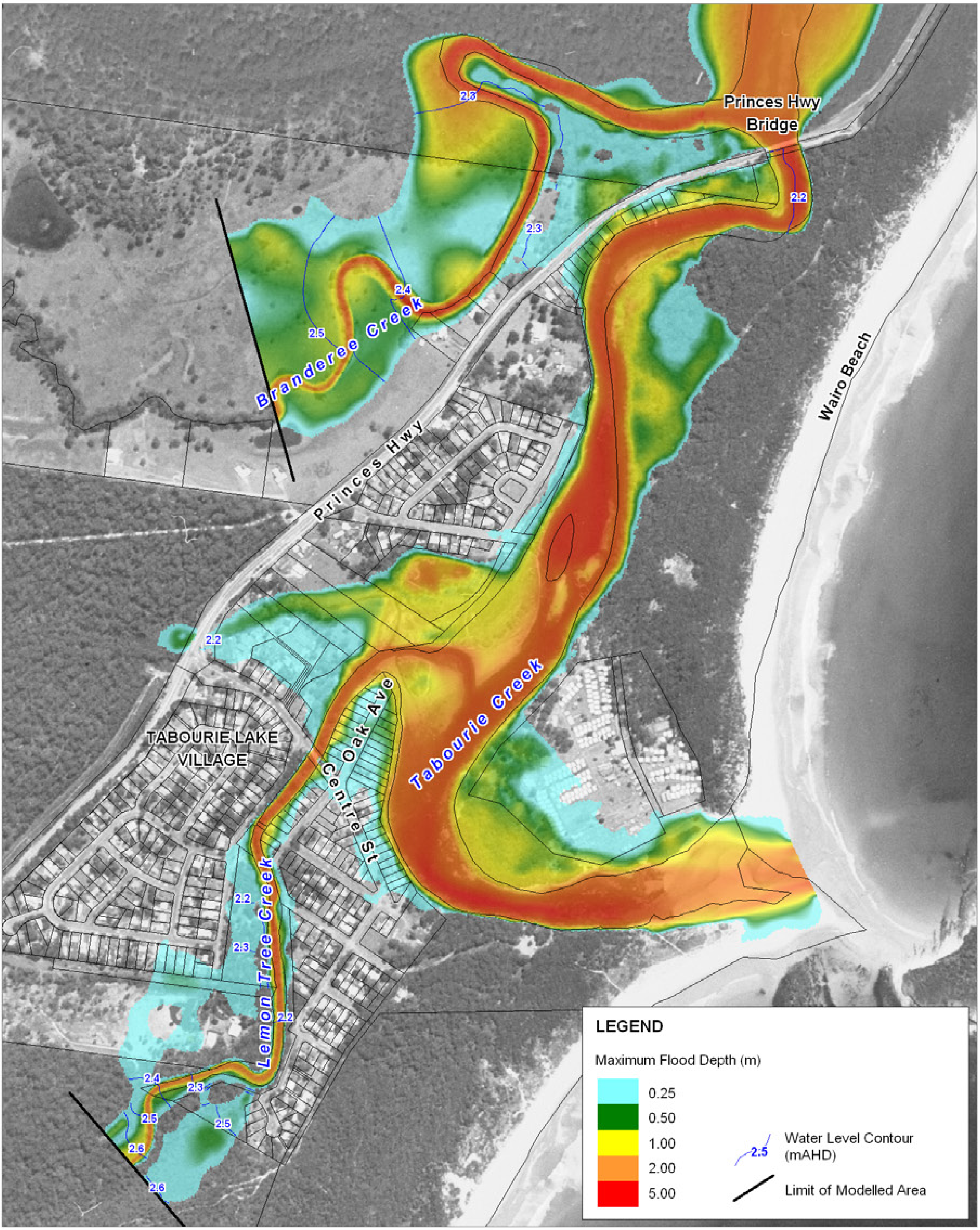
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_118_101118 EX OC 5% Hazards.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
20% AEP Maximum Flood Depths and Water Levels

Figure: **A-19**

Rev: **A**

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N

0 200 400m
Approx. Scale

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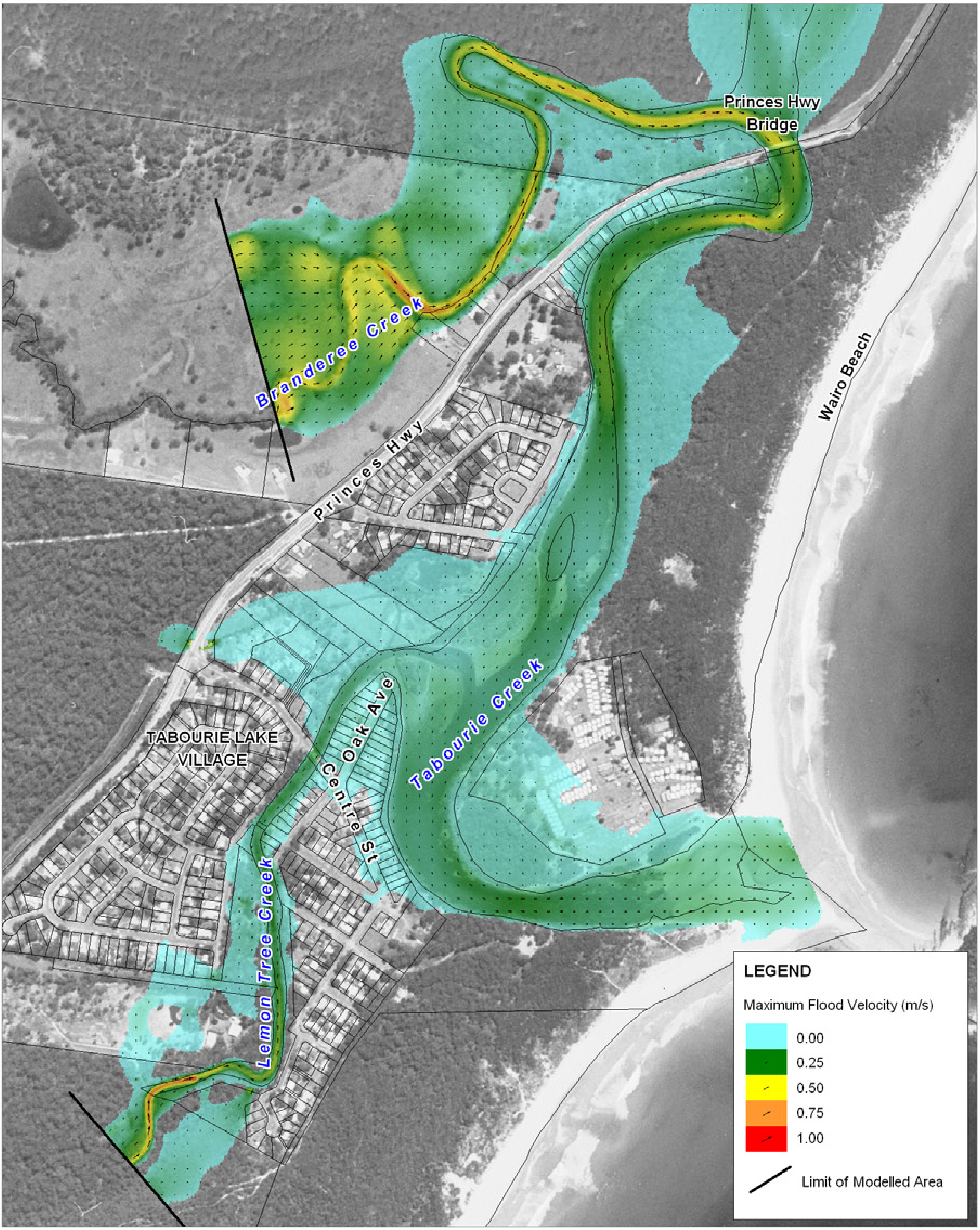
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_119_101118 EX OC 20% Depths.WOR



Title:

Ocean Derived Flood Event (Existing Conditions)
20% AEP Maximum Flood Velocities

Figure: **A-20**

Rev: **A**

N

0 200 400m
Approx. Scale

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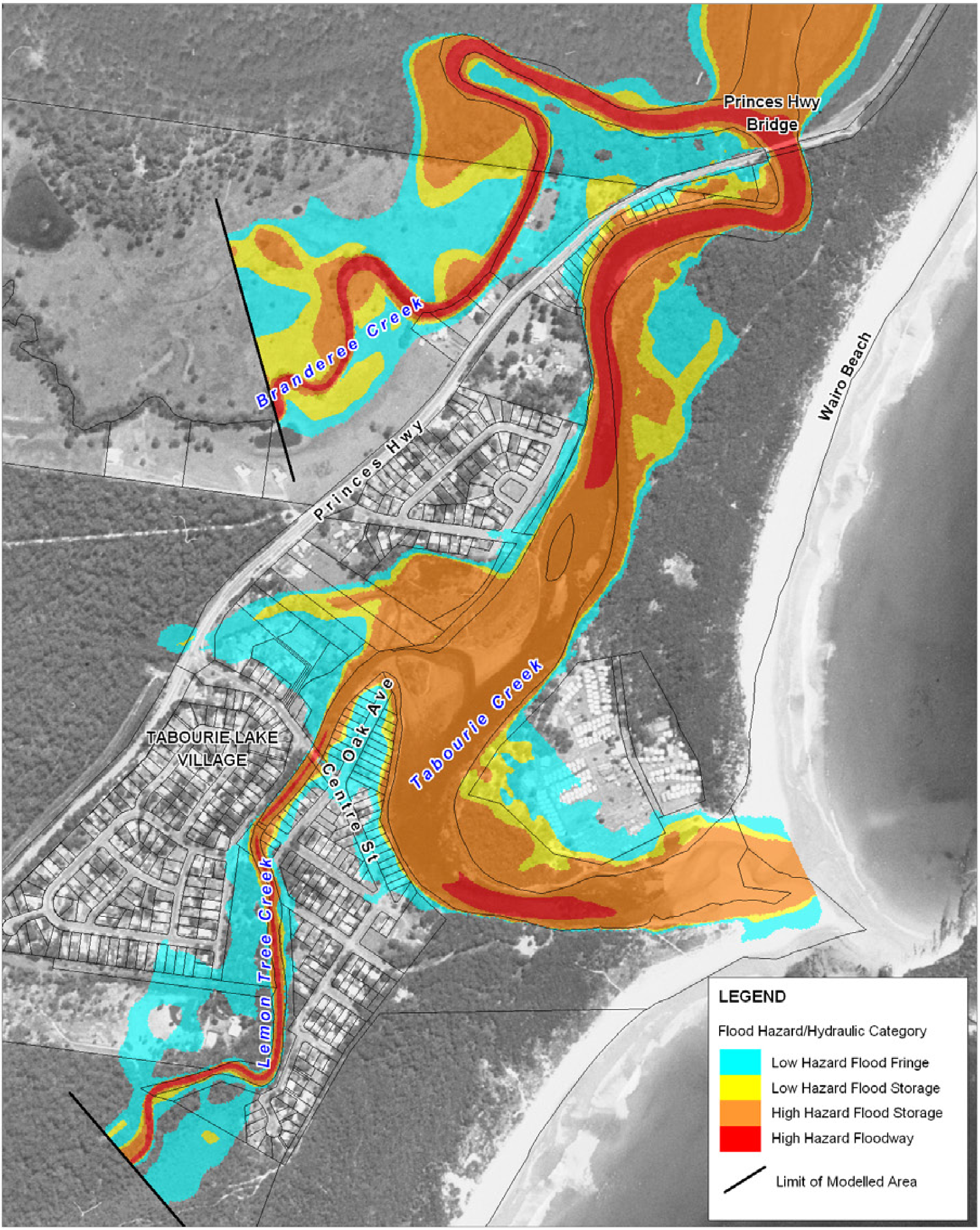

Title:

Ocean Derived Flood Event (Existing Conditions)
20% AEP Flood Hazards and Hydraulic Categories

Figure: A-21

Rev: A

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N

0 200 400m
Approx. Scale

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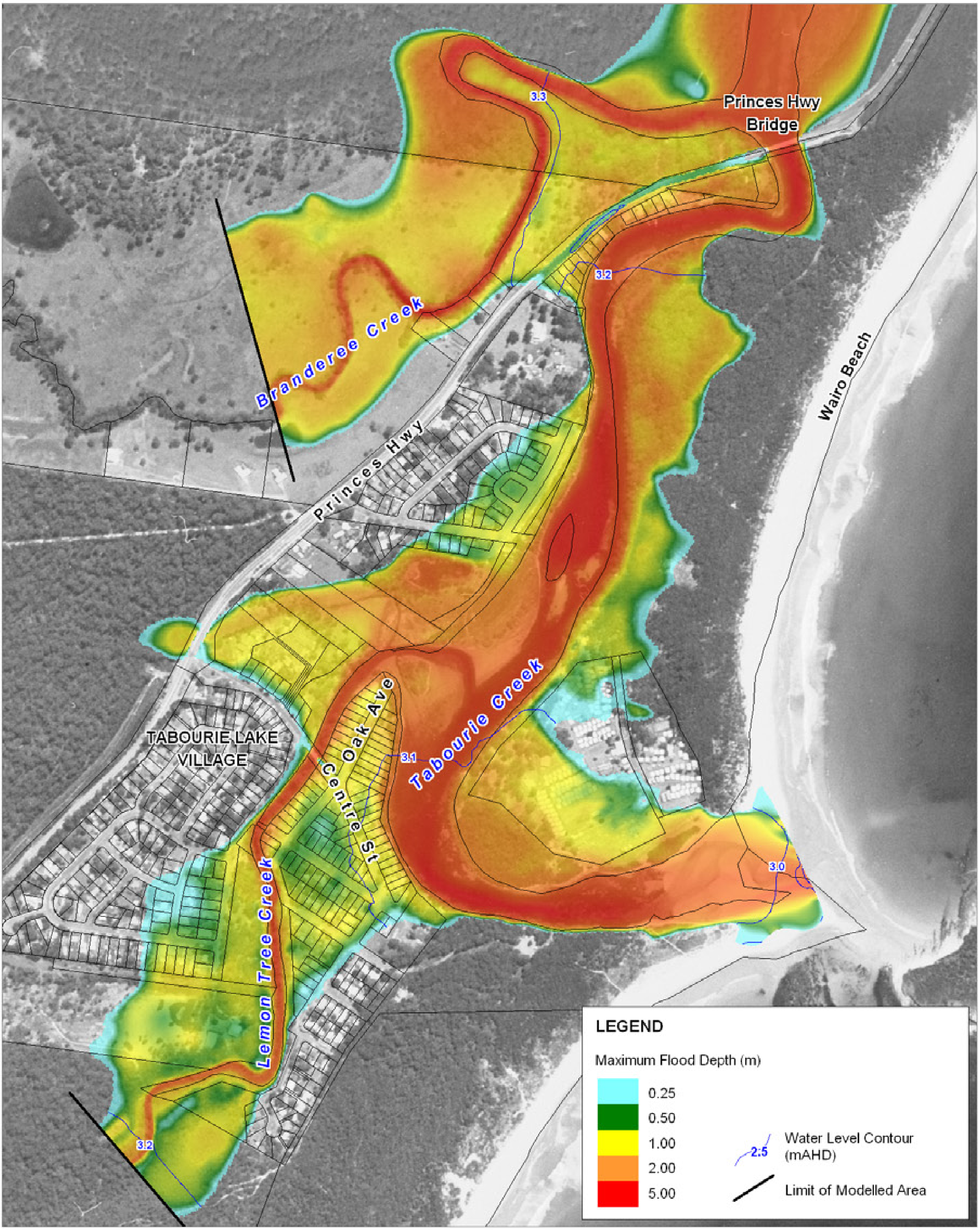

Title:
Catchment Derived Flood Event (2050 Conditions)
1% AEP Maximum Flood Depths and Water Levels

Figure: **A-22**

Rev: **A**

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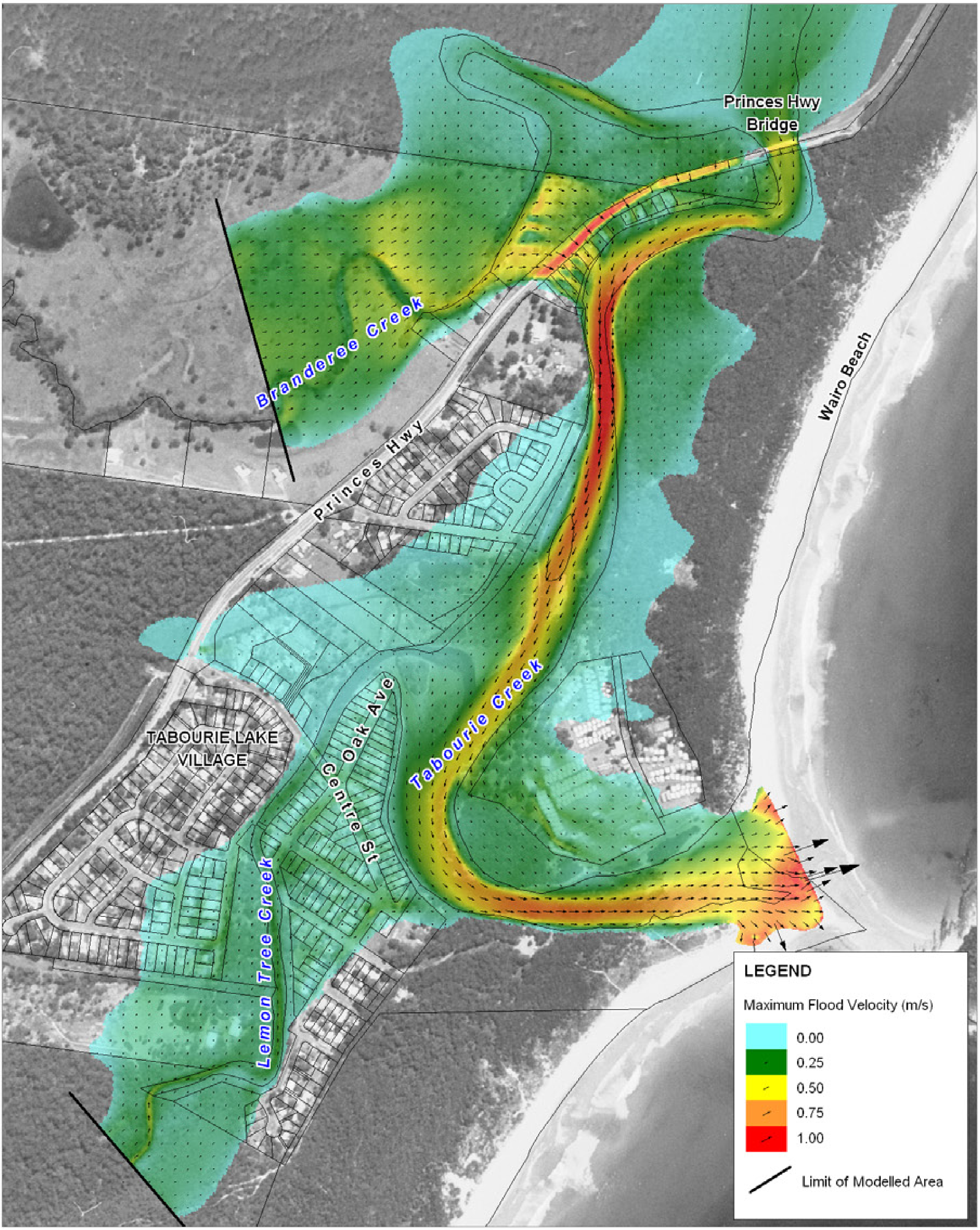
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_122_101119 2050 CA 1% Depths.WOR



Title:

Catchment Derived Flood Event (2050 Conditions)
1% AEP Maximum Flood Velocities

Figure: **A-23**

Rev: **A**

N

0 200 400m

Approx. Scale

BMT WBM
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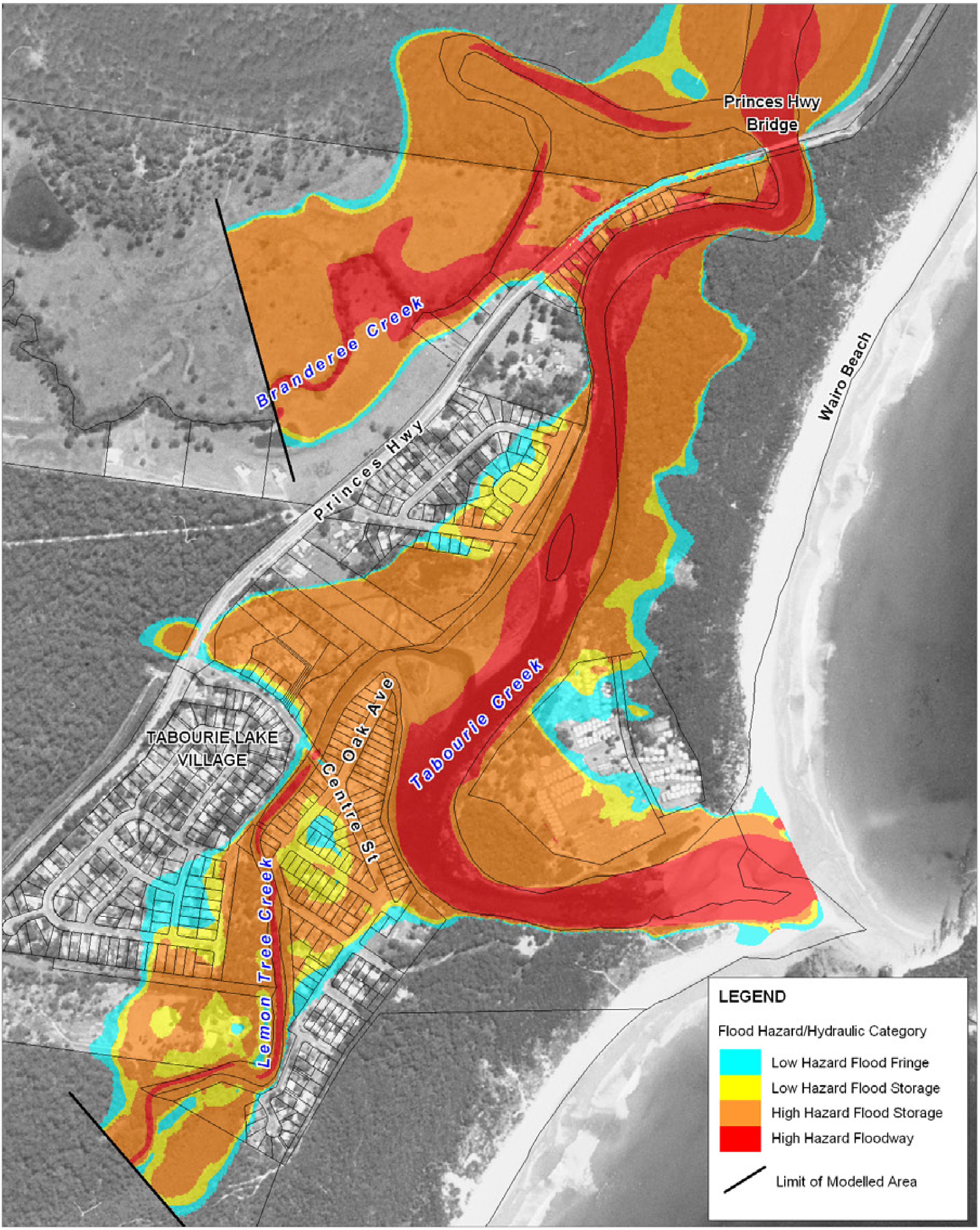
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_123_101119 2050 CA 1% Velocities.WOR



Title:

Catchment Derived Flood Event (2050 Conditions)
1% AEP Flood Hazards and Hydraulic Categories

Figure: A-24

Rev: A

N

0 200 400m

Approx. Scale

BMT WBM

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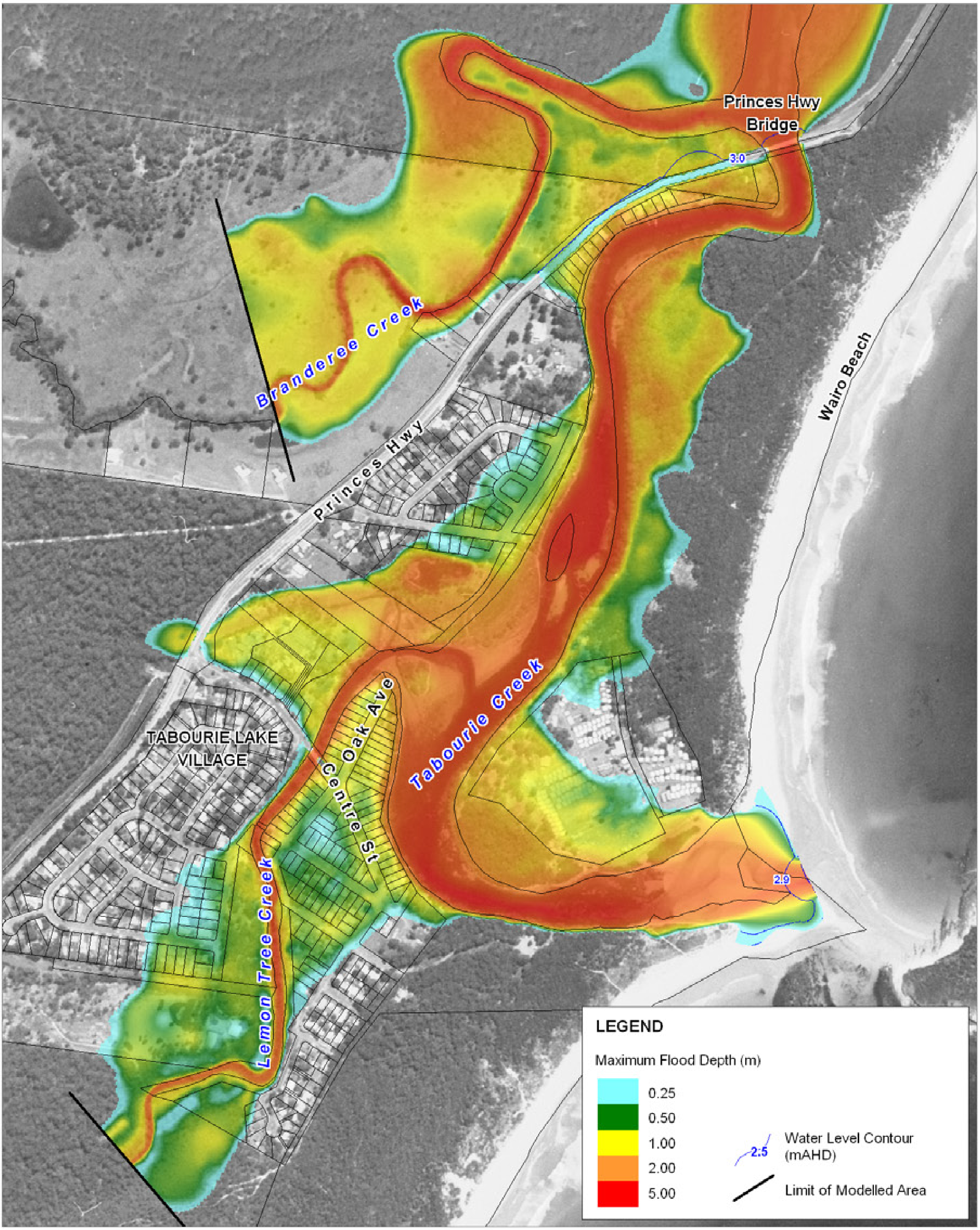
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_124_101119 2050 CA 1% Hazards.WOR



Title:

Catchment Derived Flood Event (2050 Conditions)
5% AEP Maximum Flood Depths and Water Levels

Figure: **A-25**

Rev: **A**

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N

0 200 400m
Approx. Scale

BMT WBM
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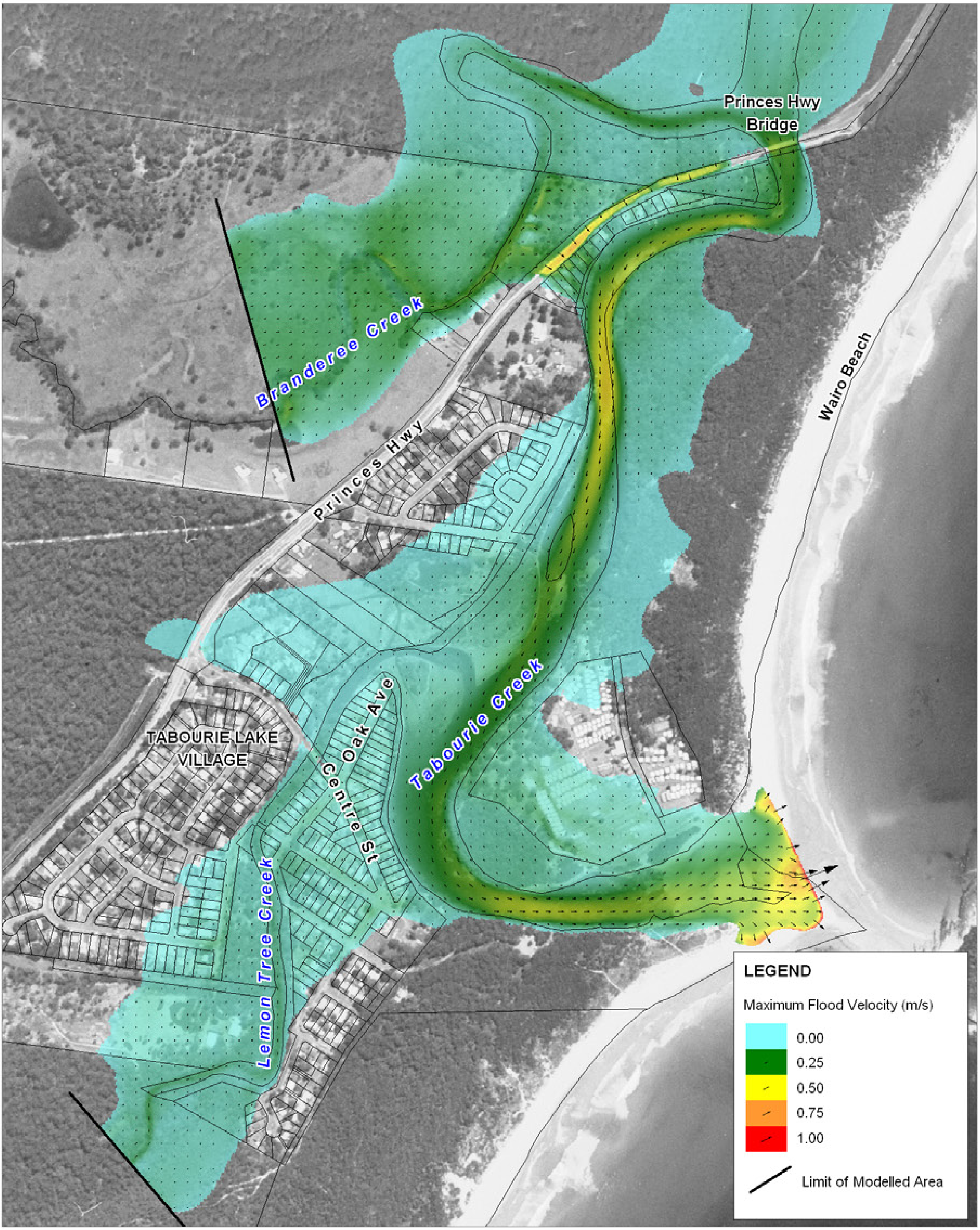
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_125_101119 2050 CA 5% Depths.WOR



Title:

Catchment Derived Flood Event (2050 Conditions)
5% AEP Maximum Flood Velocities

Figure: A-26

Rev: A

N

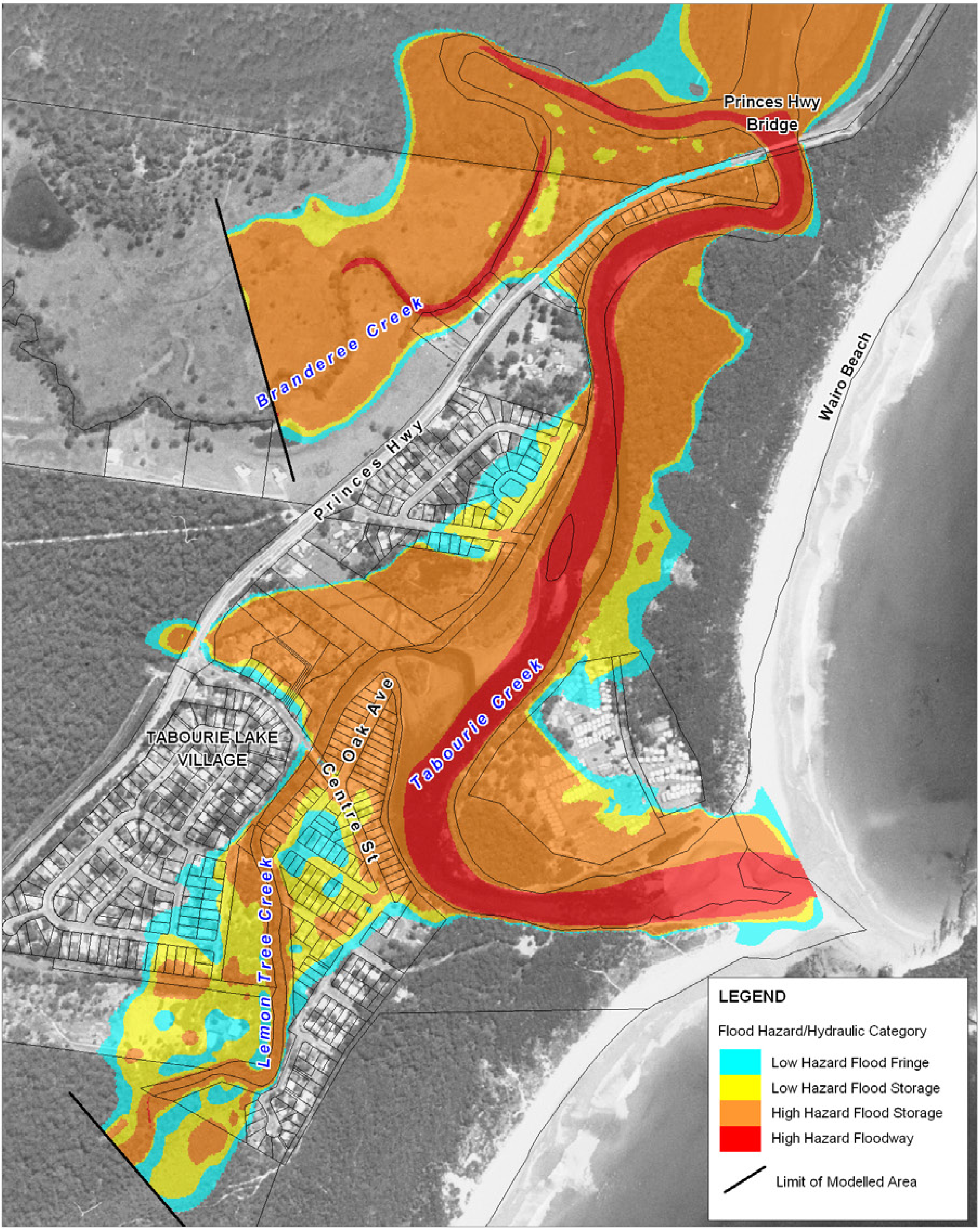
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_126_101119 2050 CA 5% Velocities.WOR



Title:

Catchment Derived Flood Event (2050 Conditions)
5% AEP Flood Hazards and Hydraulic Categories

Figure: A-27

Rev: A

N

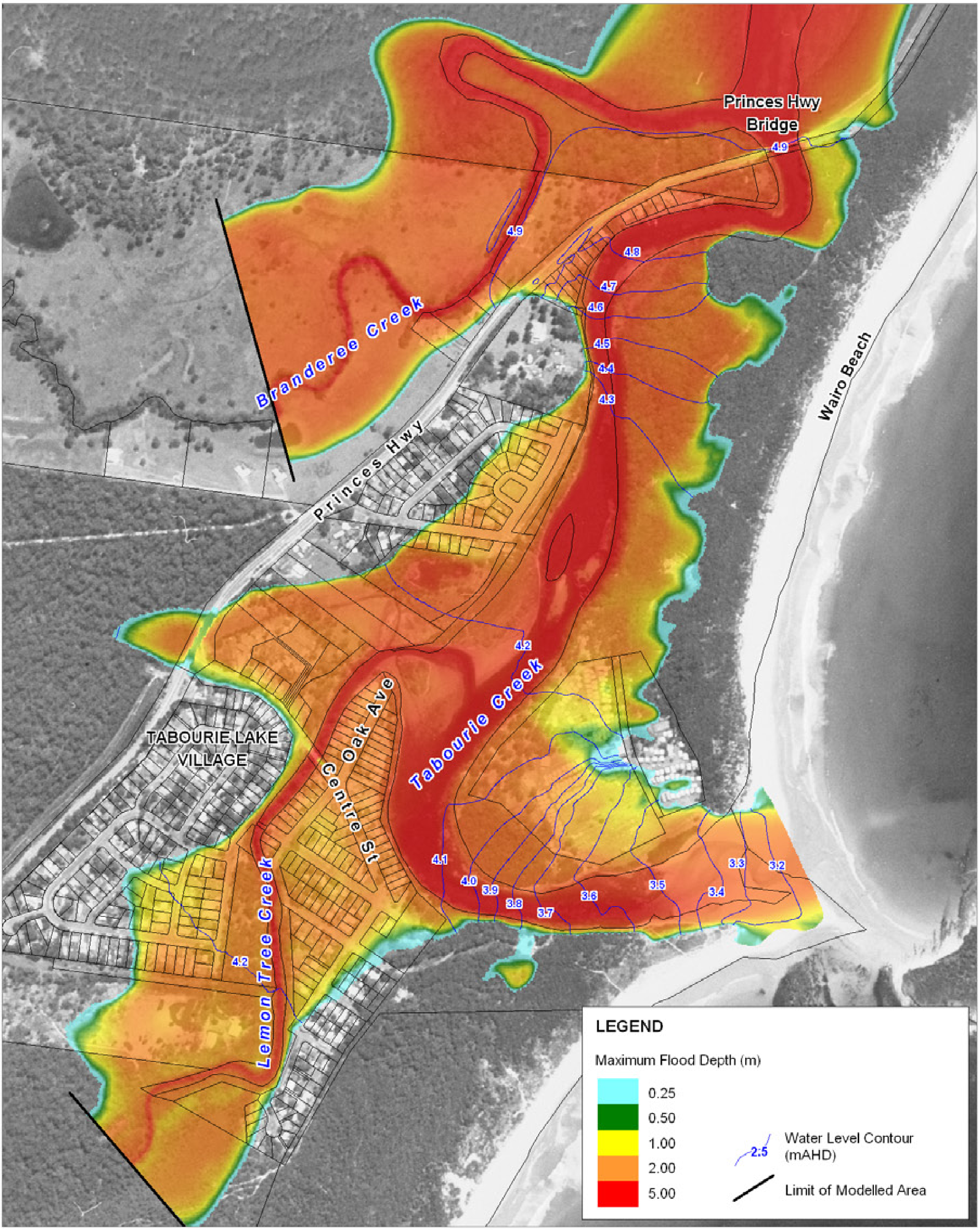
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_127_101119 2050 CA 5% Hazards.WOR



Title:

PMF Flood Event (2050 Conditions)
Maximum Flood Depths and Water Levels

Figure: A-28

Rev: A

N

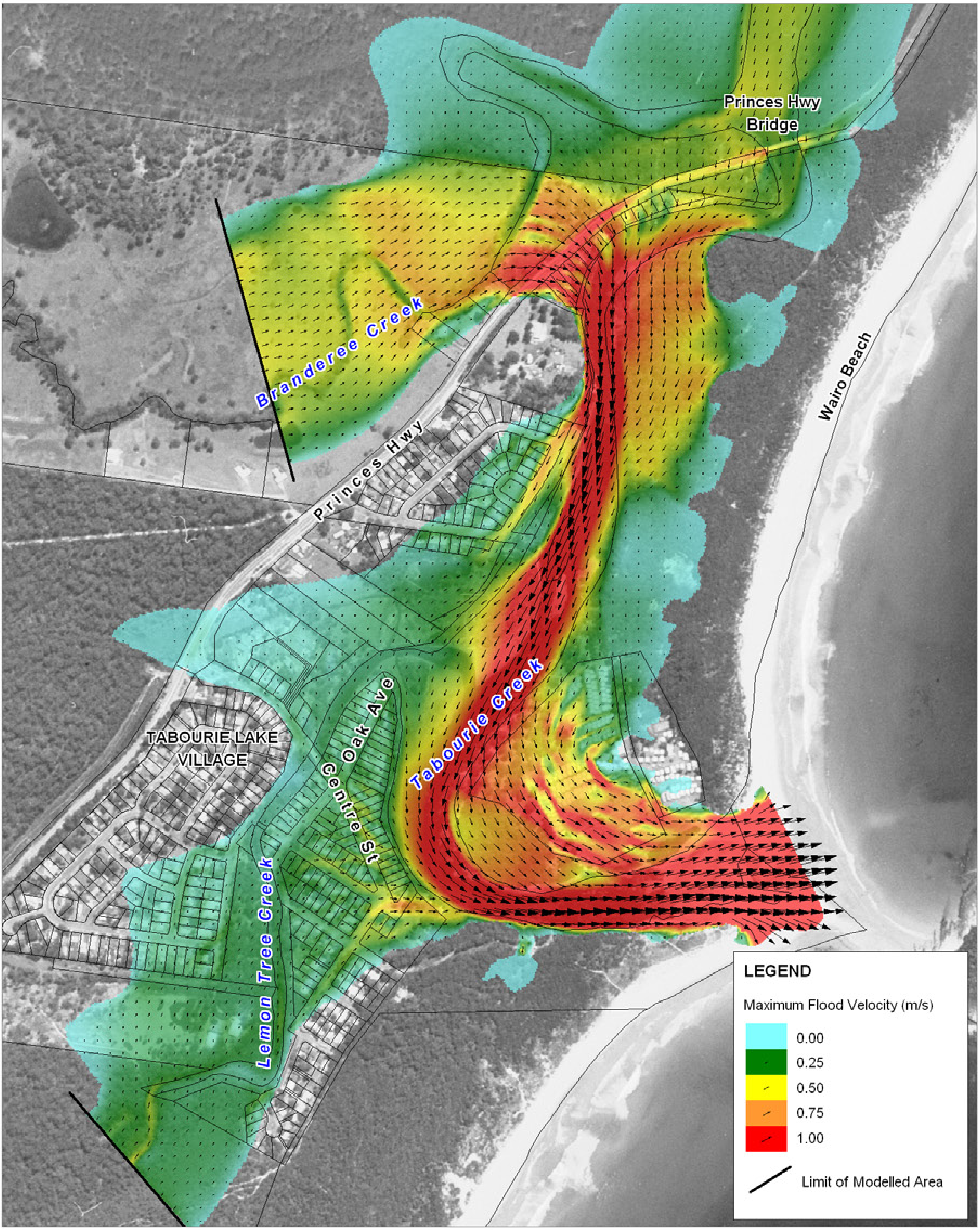
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_128_101119 2050 PMF Depths.WOR



Title:
PMF Flood Event (2050 Conditions)
Maximum Flood Velocities

Figure: **A-29**

Rev: **A**

N

0 200 400m
Approx. Scale

BMT WBM
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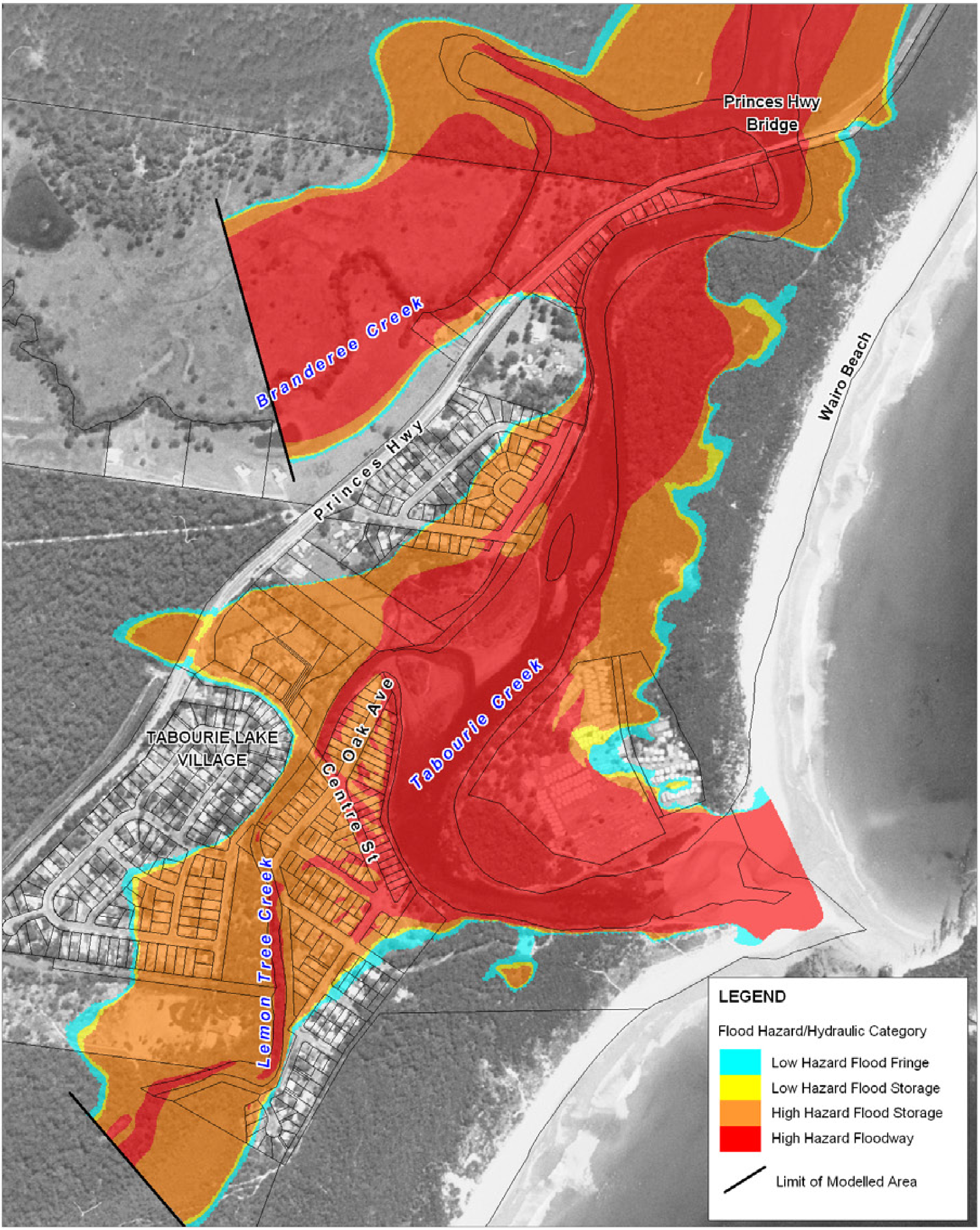
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_129_101119 2050 PMF Velocities.WOR



Title:

PMF Flood Event (2050 Conditions)
Flood Hazards and Hydraulic Categories

Figure: **A-30**

Rev: **A**

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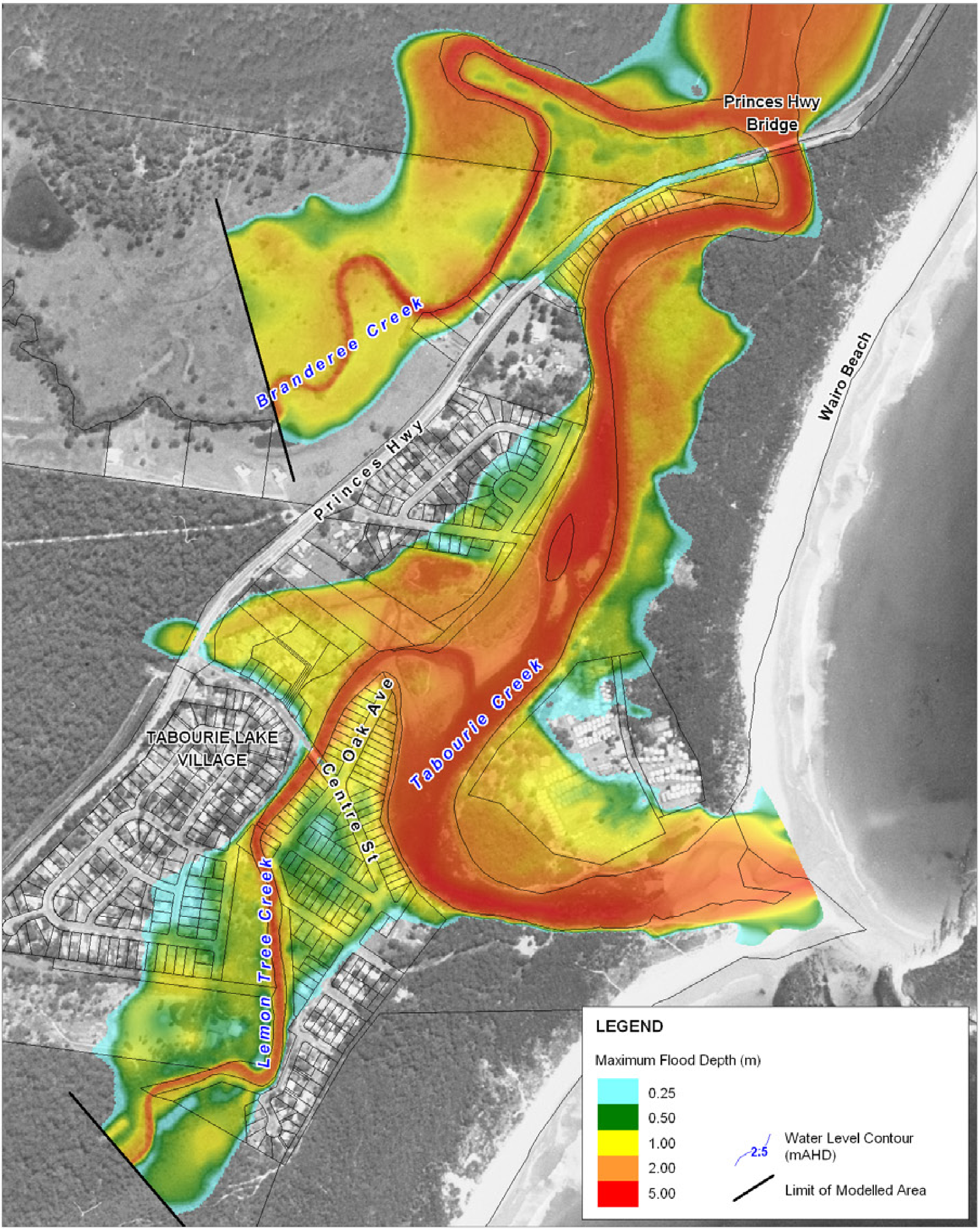

Title:
Ocean Derived Flood Event (2050 Conditions)
1% AEP Maximum Flood Depths and Water Levels

Figure: **A-31**

Rev: **A**

N

0 200 400m
Approx. Scale

BMT WBM
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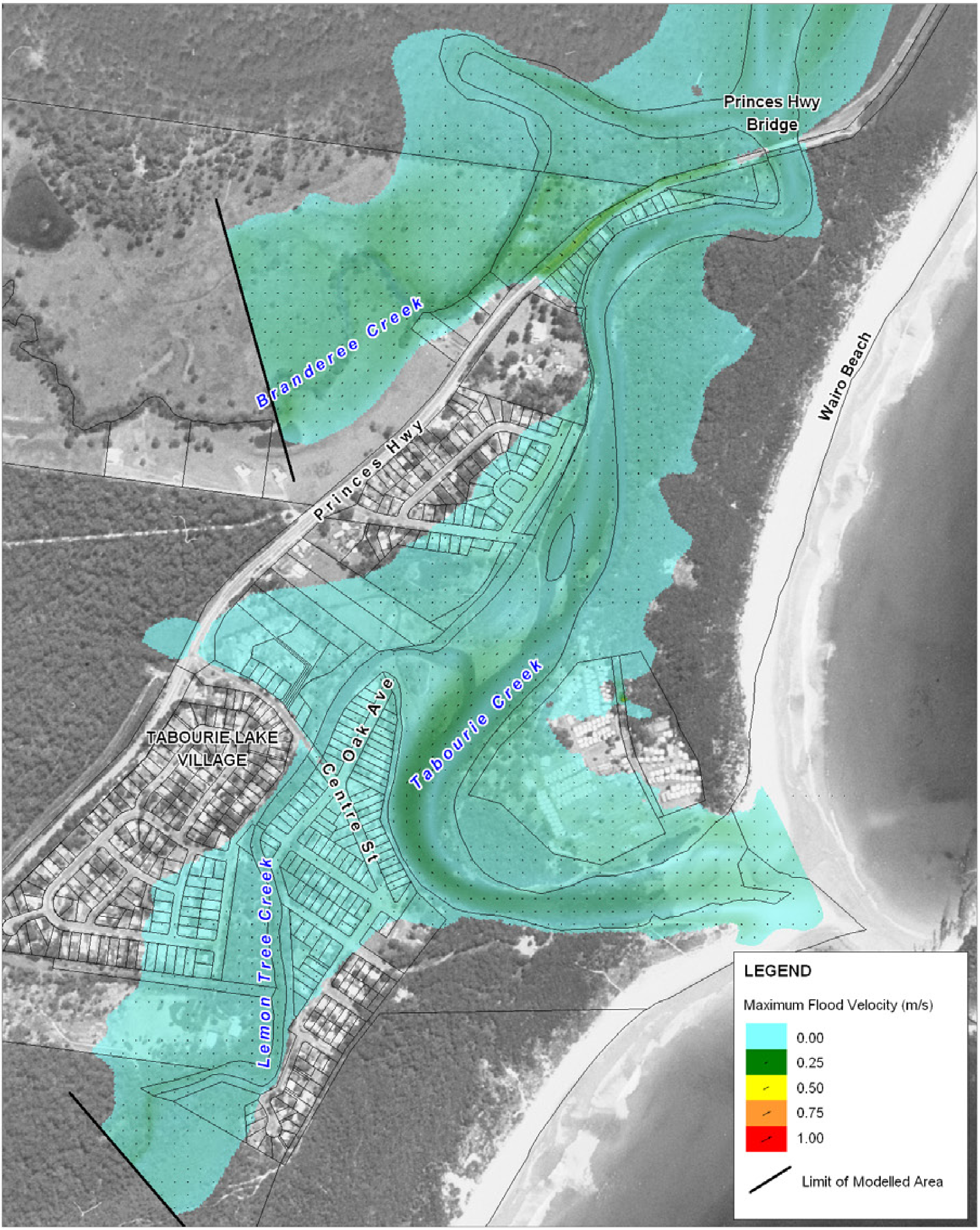
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_131_101119 2050 OC 1% Depths.WOR



Title:

Ocean Derived Flood Event (2050 Conditions) 1% AEP Maximum Flood Velocities

Figure: A-32

Rev: A

N

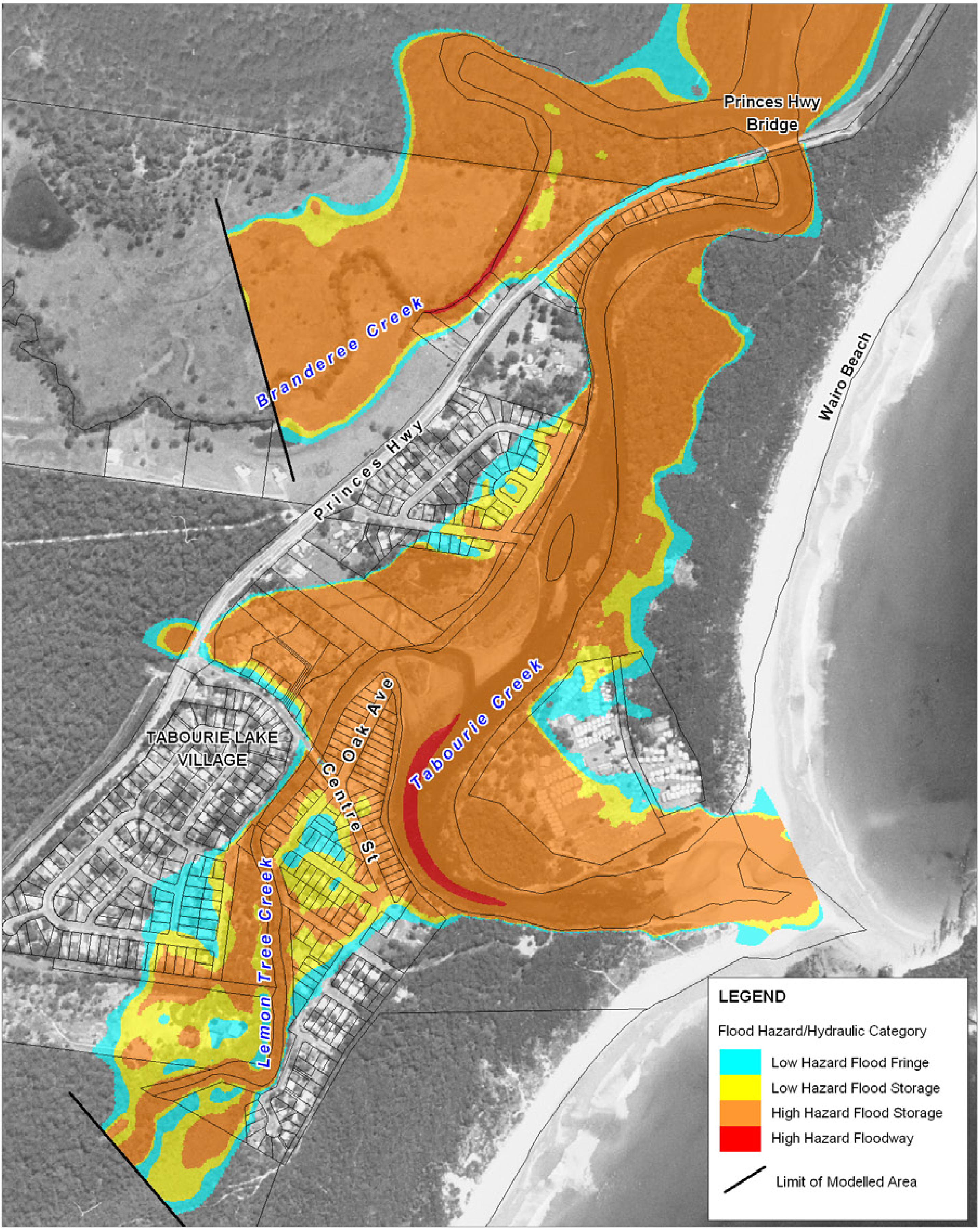
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_132_101119 2050 OC 1% Velocities.WOR



Title:

Ocean Derived Flood Event (2050 Conditions) 1% AEP Flood Hazards and Hydraulic Categories

Figure: A-33

Rev: A

N

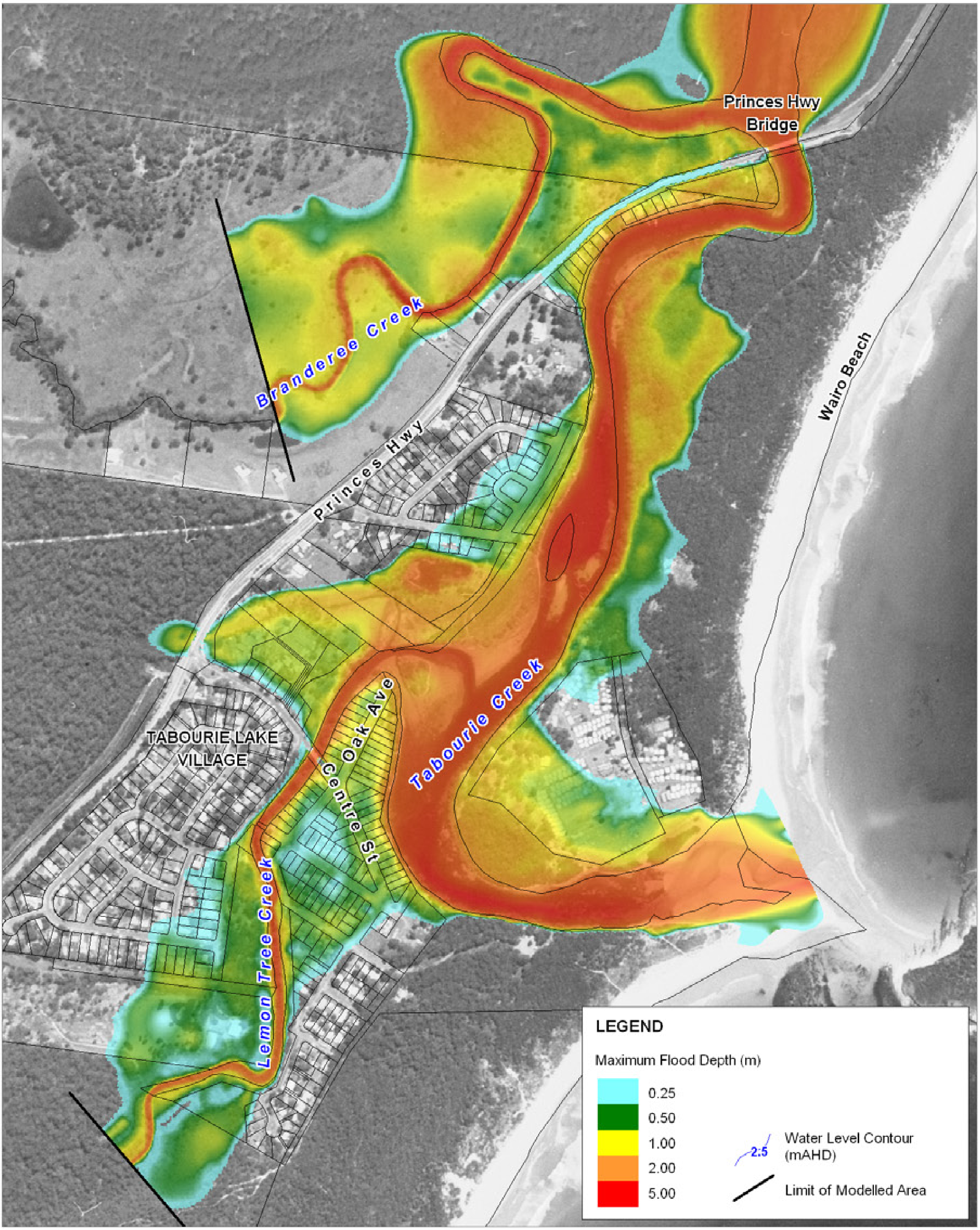
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_133_101119 2050 OC 1% Hazards.WOR



Title:

Ocean Derived Flood Event (2050 Conditions)
5% AEP Maximum Flood Depths and Water Levels

Figure: A-34

Rev: A

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

N

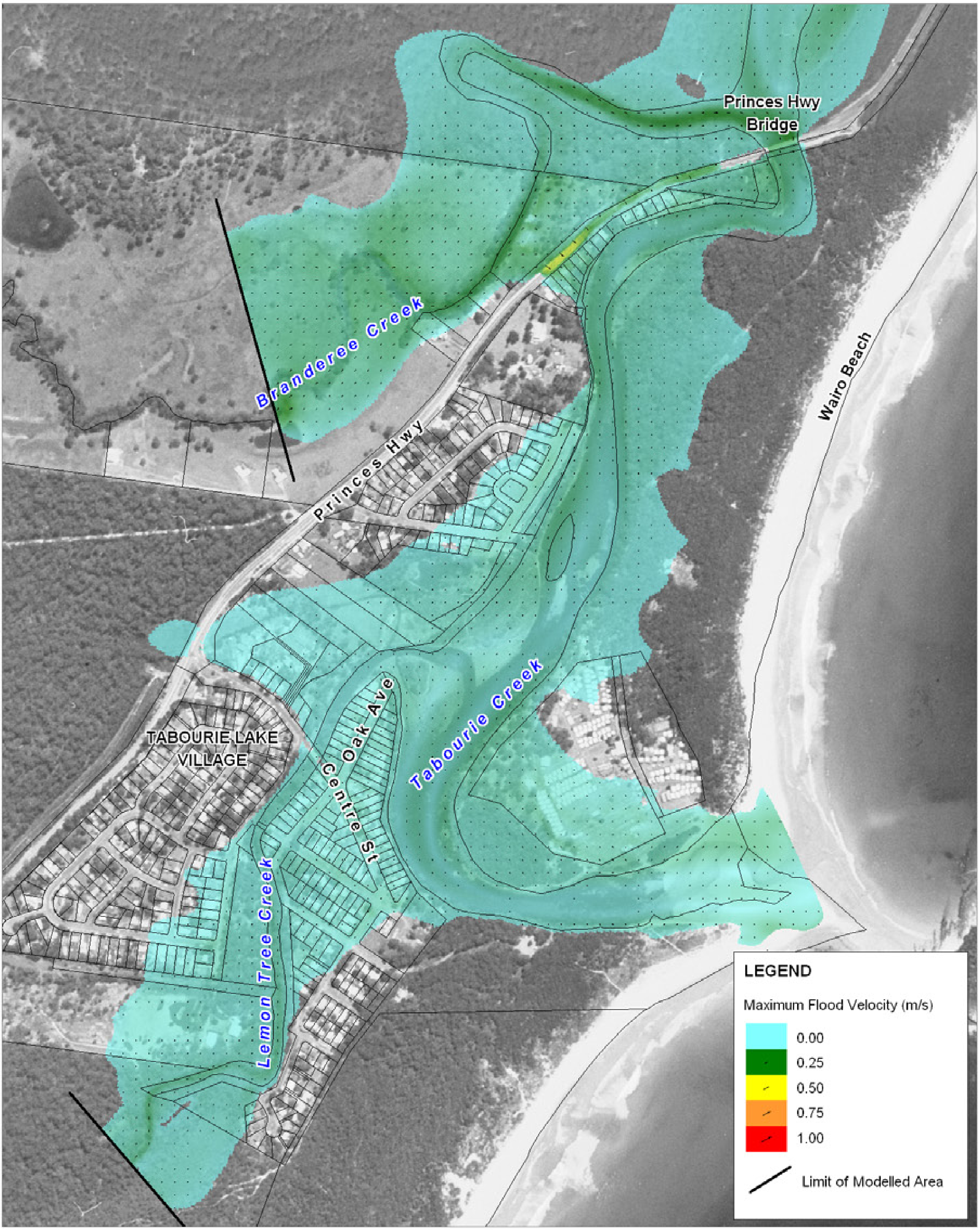
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_134_101119 2050 OC 5% Depths.WOR



Title:

Ocean Derived Flood Event (2050 Conditions)
5% AEP Maximum Flood Velocities

Figure: A-35

Rev: A

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

N

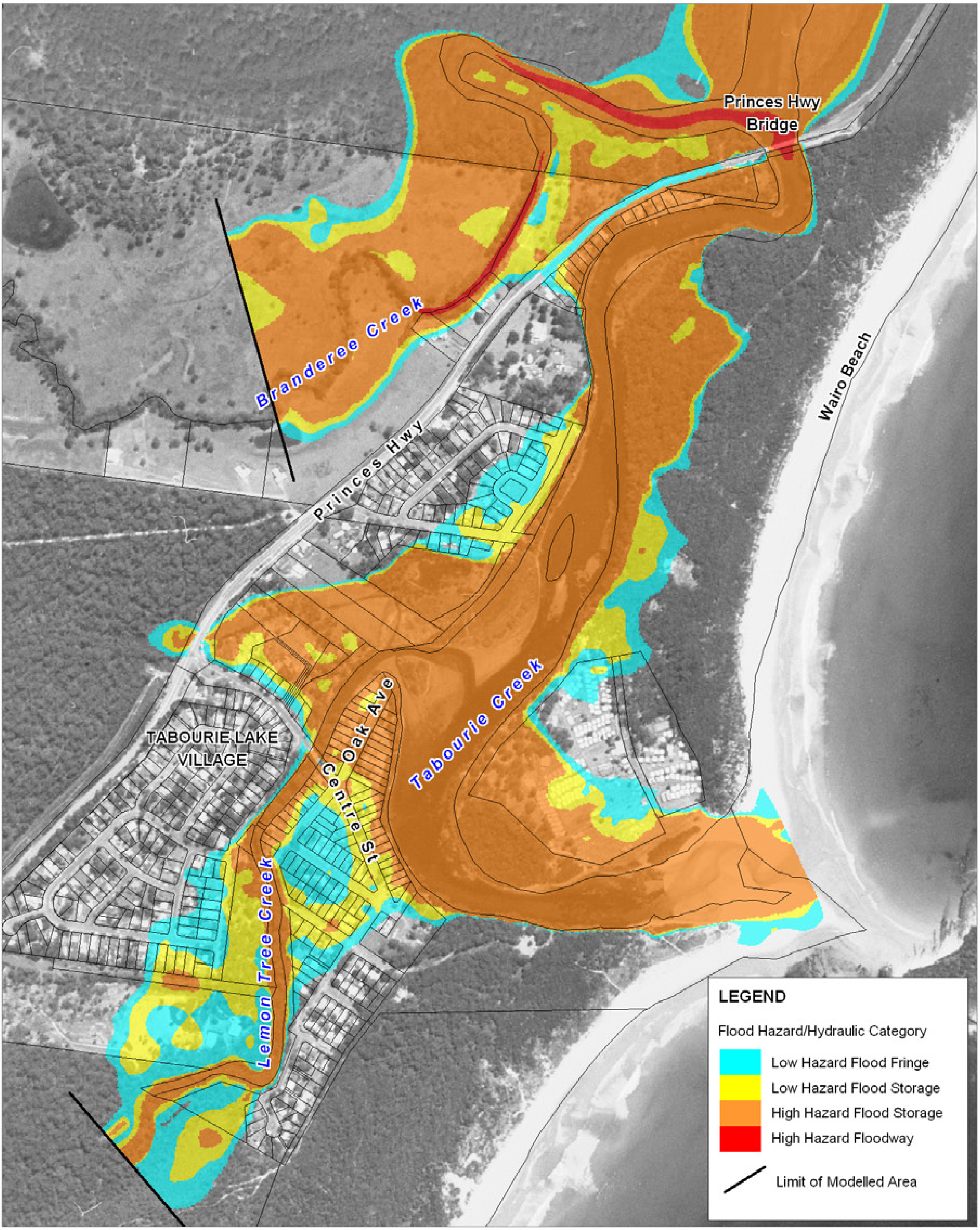
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_135_101119 2050 OC 5% Velocities.WOR



Title:

Ocean Derived Flood Event (2050 Conditions)
5% AEP Flood Hazards and Hydraulic Categories

Figure: A-36

Rev: A

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

N

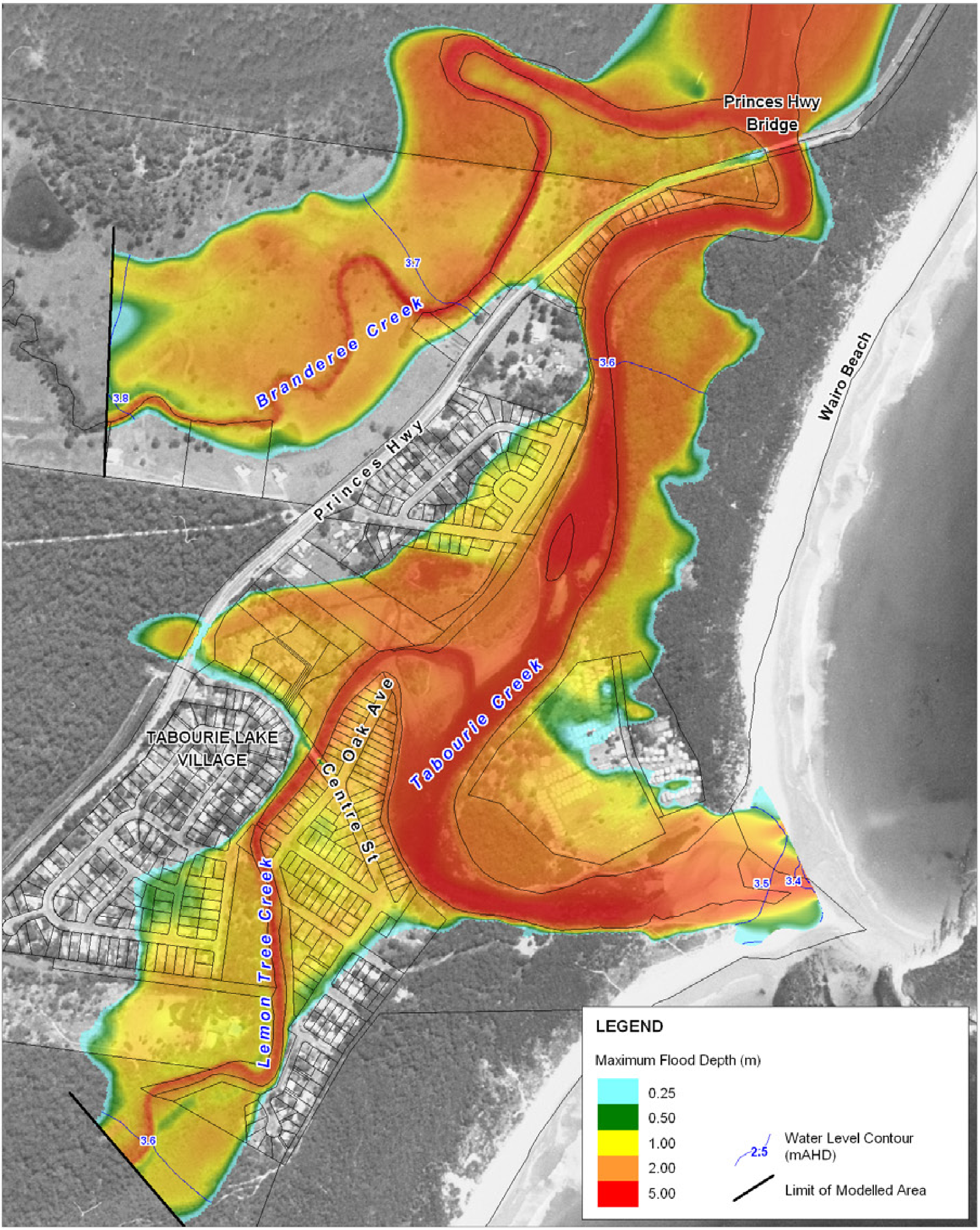
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_136_101119 2050 OC 5% Hazards.WOR



Title:

Catchment Derived Flood Event (2100 Conditions)
1% AEP Maximum Flood Depths and Water Levels

Figure: A-37

Rev: A

N

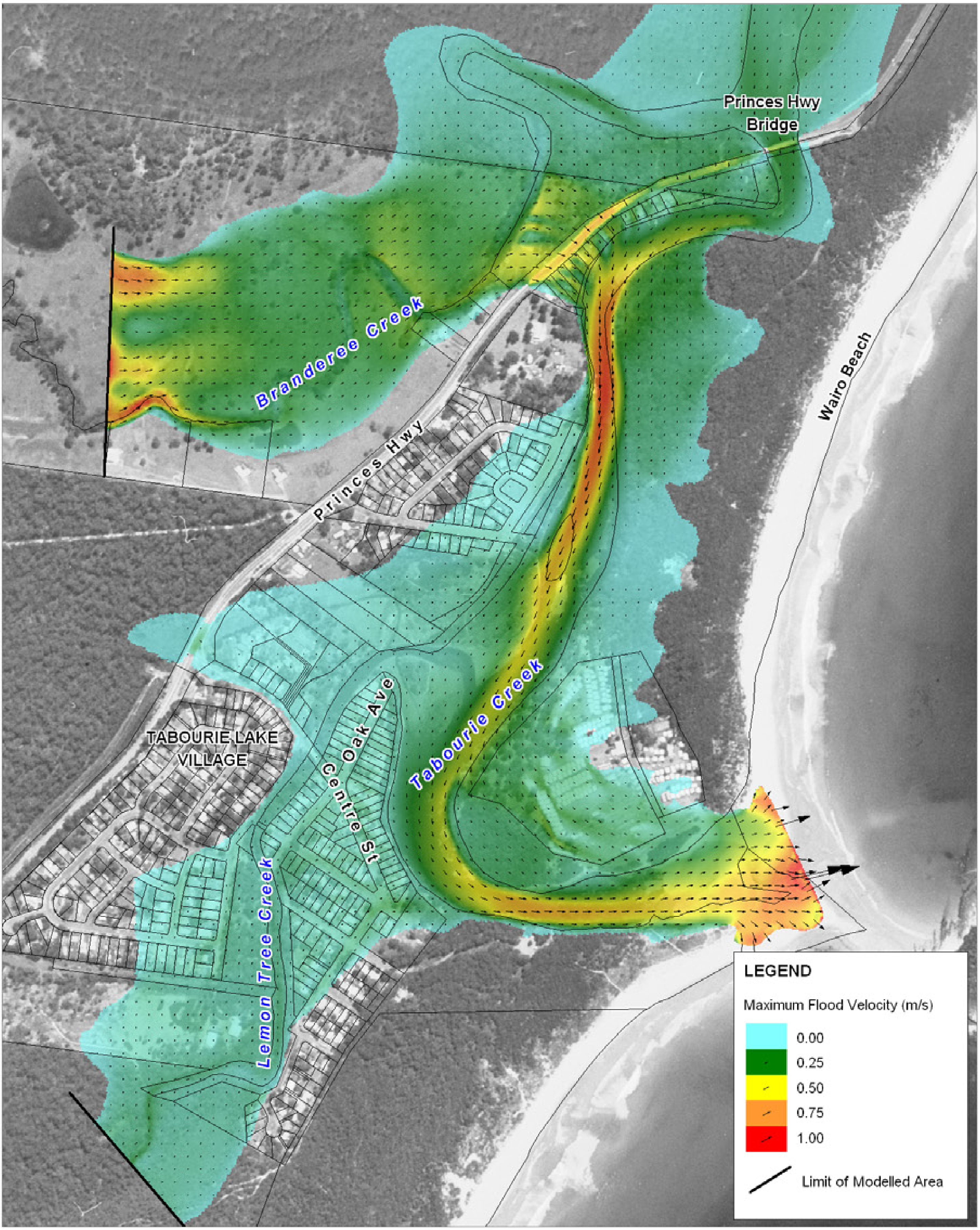
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_137_101119 2100 CA 1% Depths.WOR



Title:

Catchment Derived Flood Event (2100 Conditions)
1% AEP Maximum Flood Velocities

Figure: **A-38**

Rev: **A**

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

N

0 200 400m
Approx. Scale

BMT WBM
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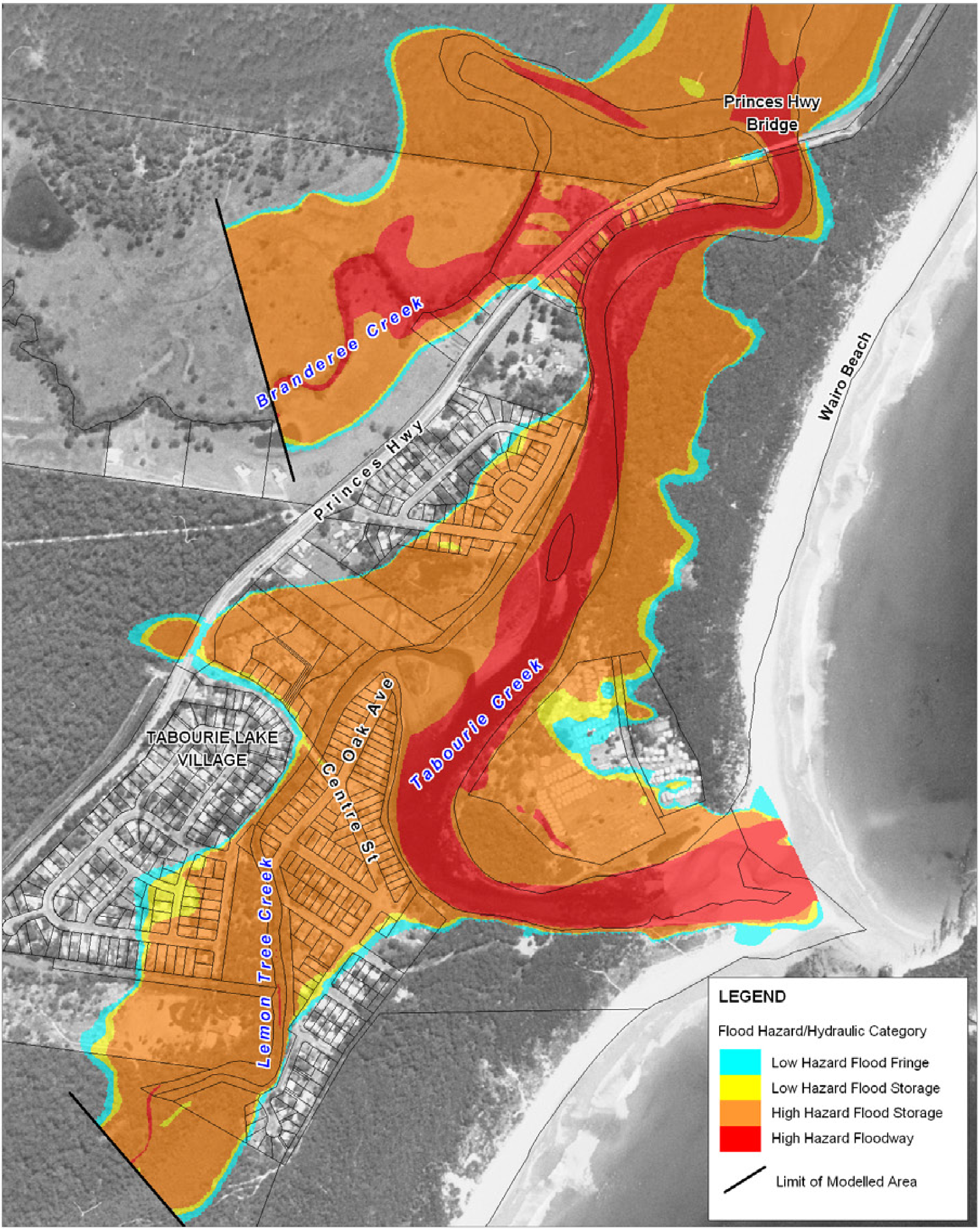
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_138_101119 2100 CA 1% Velocities.WOR



Title:

Catchment Derived Flood Event (2100 Conditions)
1% AEP Flood Hazards and Hydraulic Categories

Figure: A-39

Rev: A

N

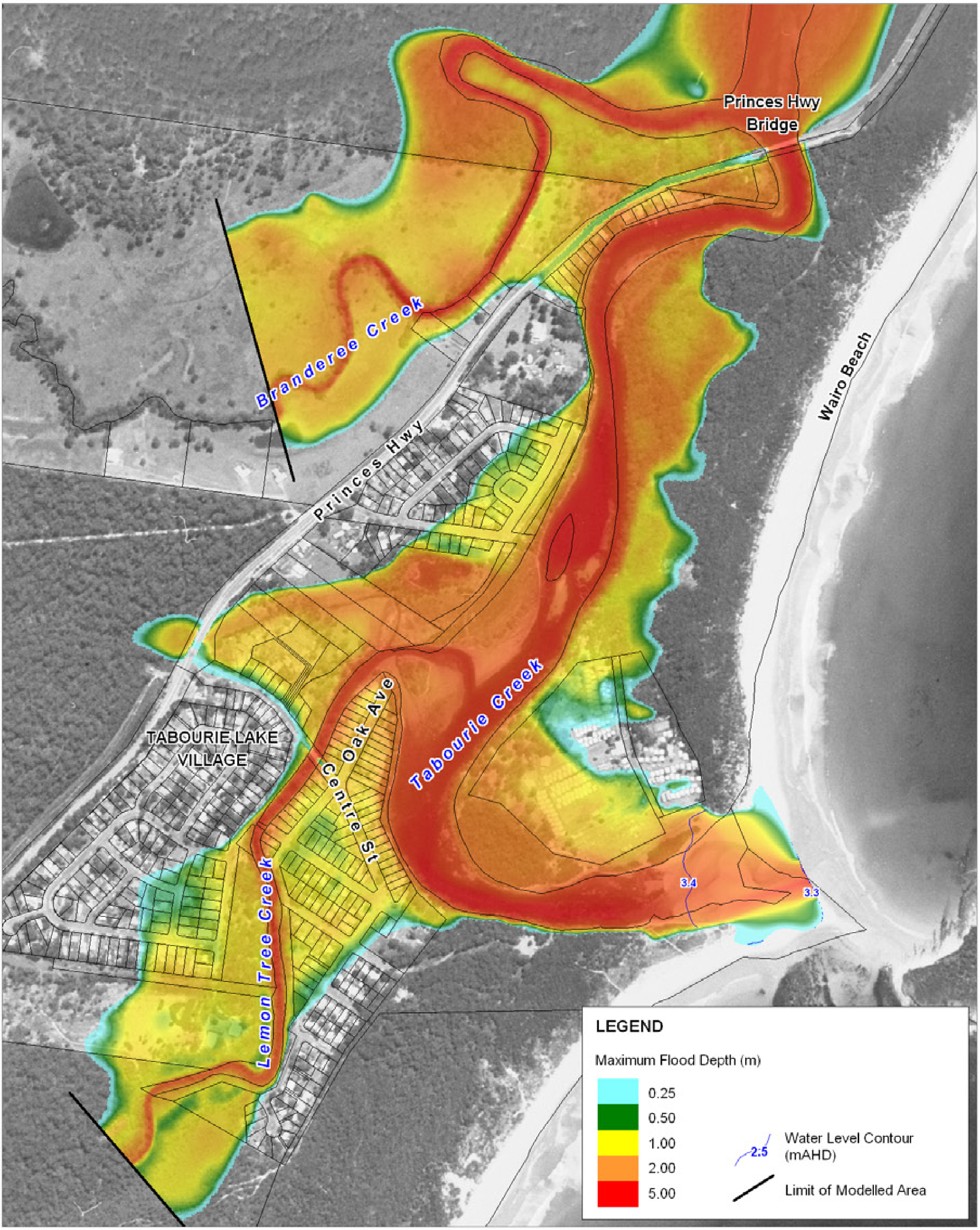
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_139_101119 2100 CA 1% Hazards.WOR



Title:

Catchment Derived Flood Event (2100 Conditions)
5% AEP Maximum Flood Depths and Water Levels

Figure: A-40

Rev: A

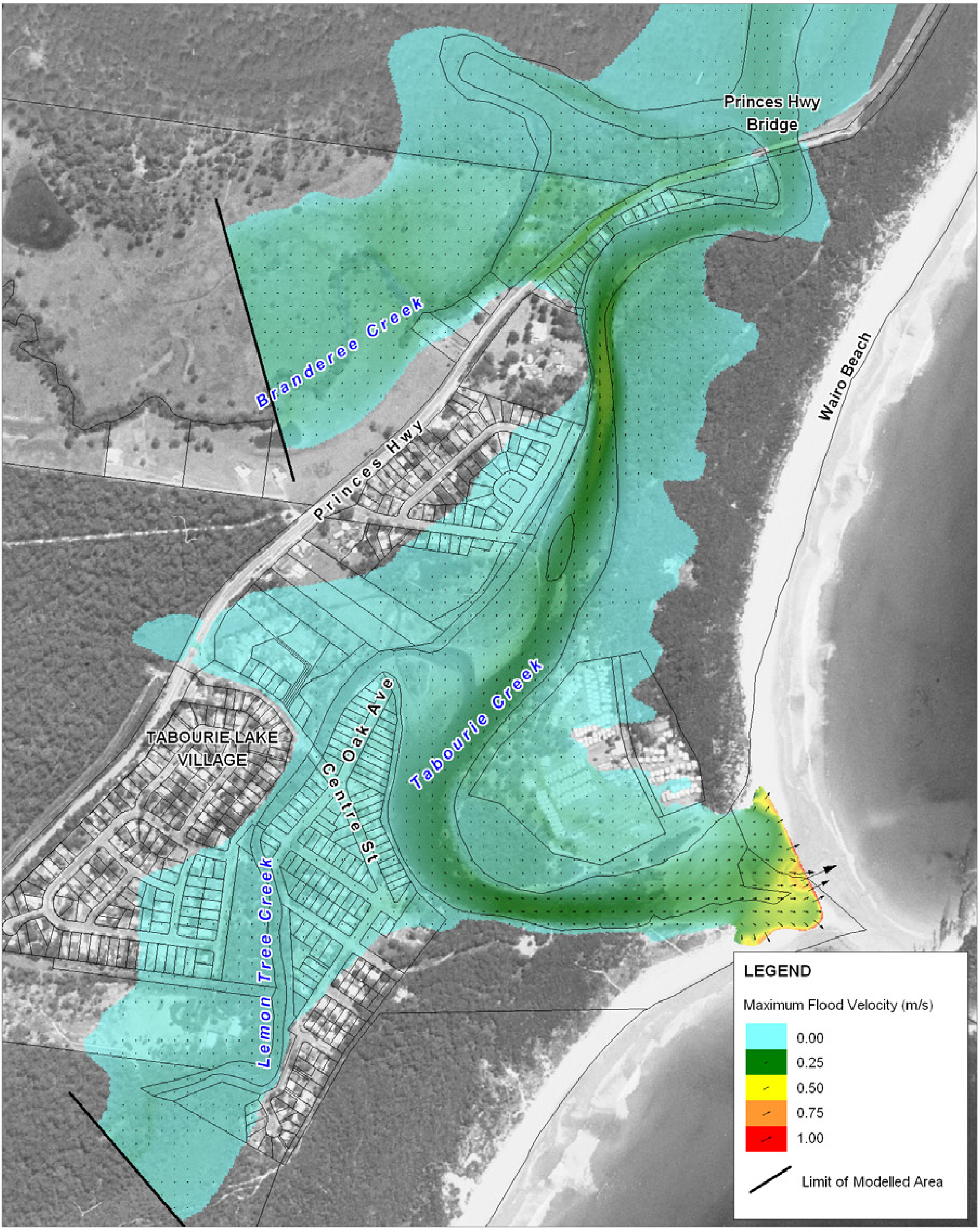
N

0 200 400m

Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_140_101119 2100 CA 5% Depths.WOR



Title:

Catchment Derived Flood Event (2100 Conditions) 5% AEP Maximum Flood Velocities

Figure: A-41

Rev: A

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

N

0 200 400m

Approx. Scale

BMT WBM

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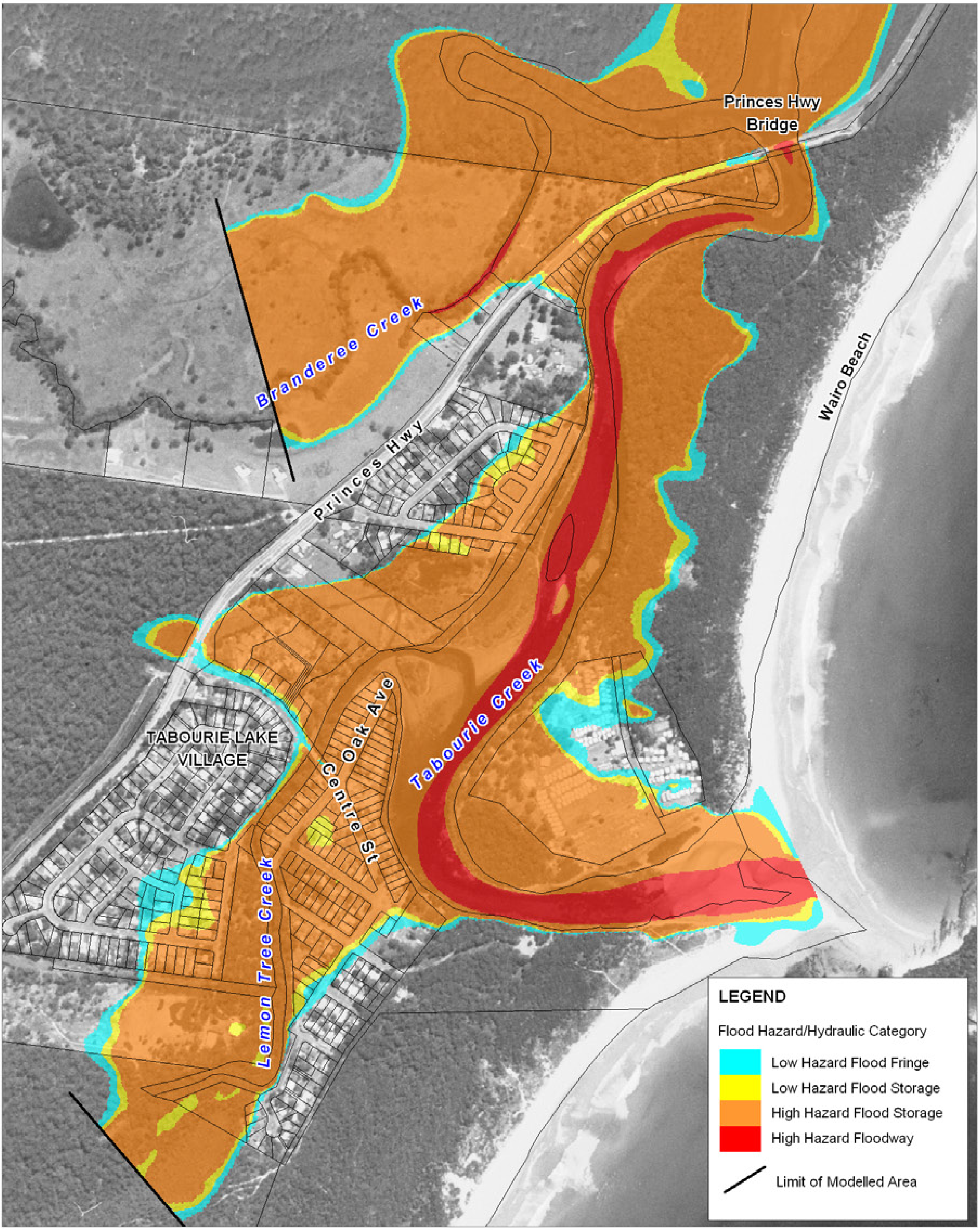
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_141_101119 2100 CA 5% Velocities.WOR



Title:

Catchment Derived Flood Event (2100 Conditions)
5% AEP Flood Hazards and Hydraulic Categories

Figure: A-42

Rev: A

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

N

0 200 400m

Approx. Scale

BMT WBM

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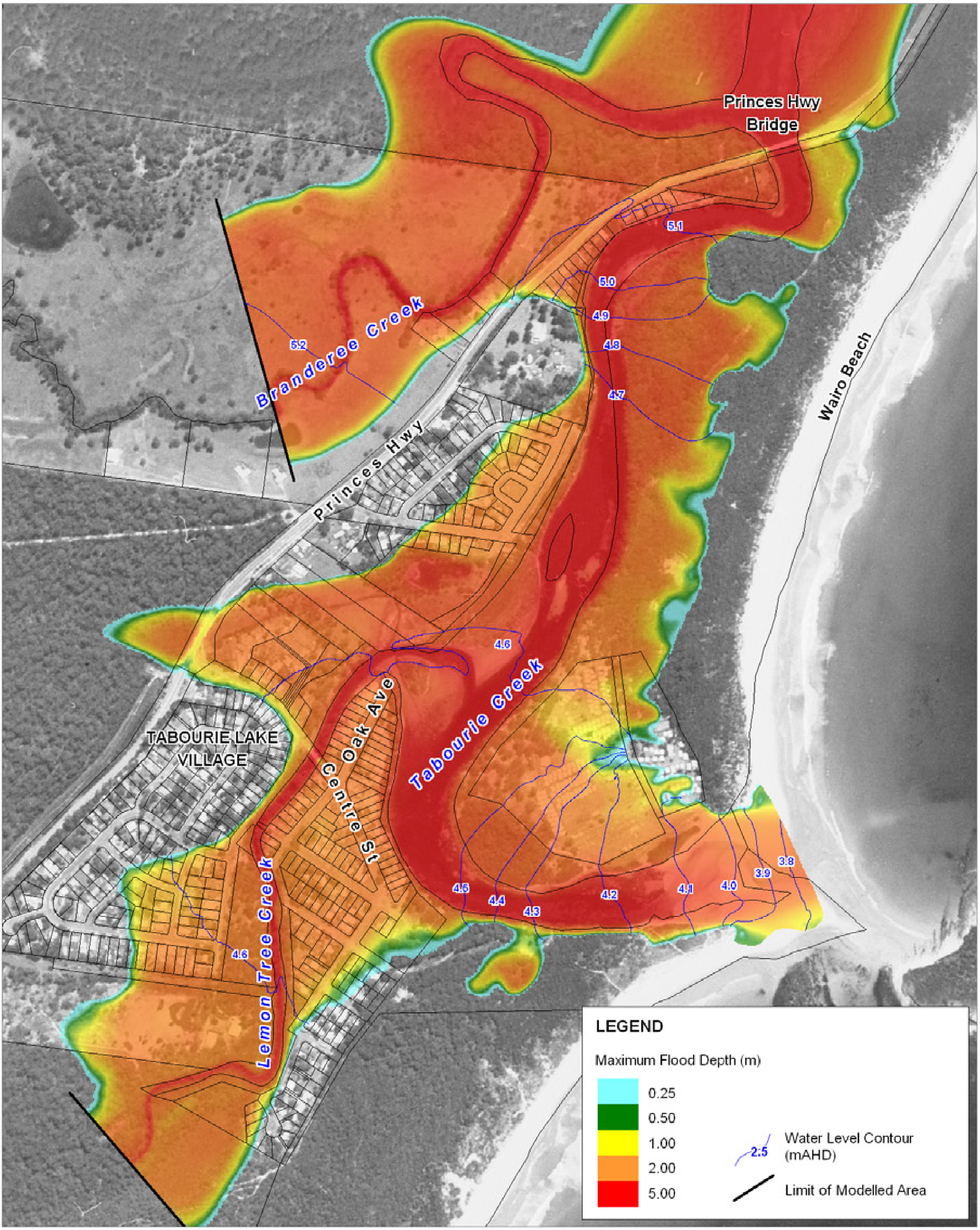
Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_142_101119 2100 CA 5% Hazards.WOR



Title:

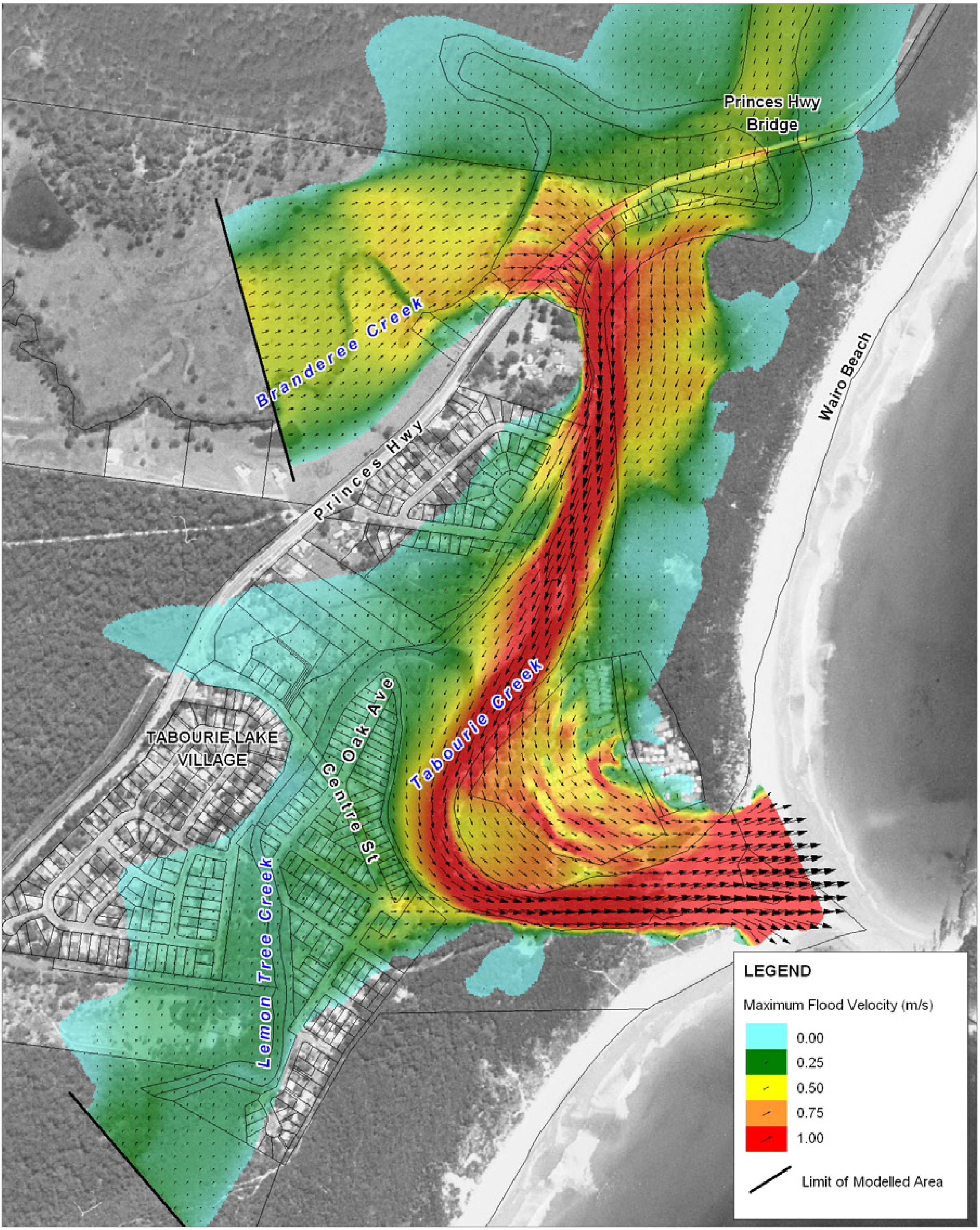
PMF Flood Event (2100 Conditions)
Maximum Flood Depths and Water Levels

Figure: A-43

Rev: A

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_143_101119 2100 PMF Depths.WOR



Title:

PMF Flood Event (2100 Conditions)
Maximum Flood Velocities

Figure: A-44

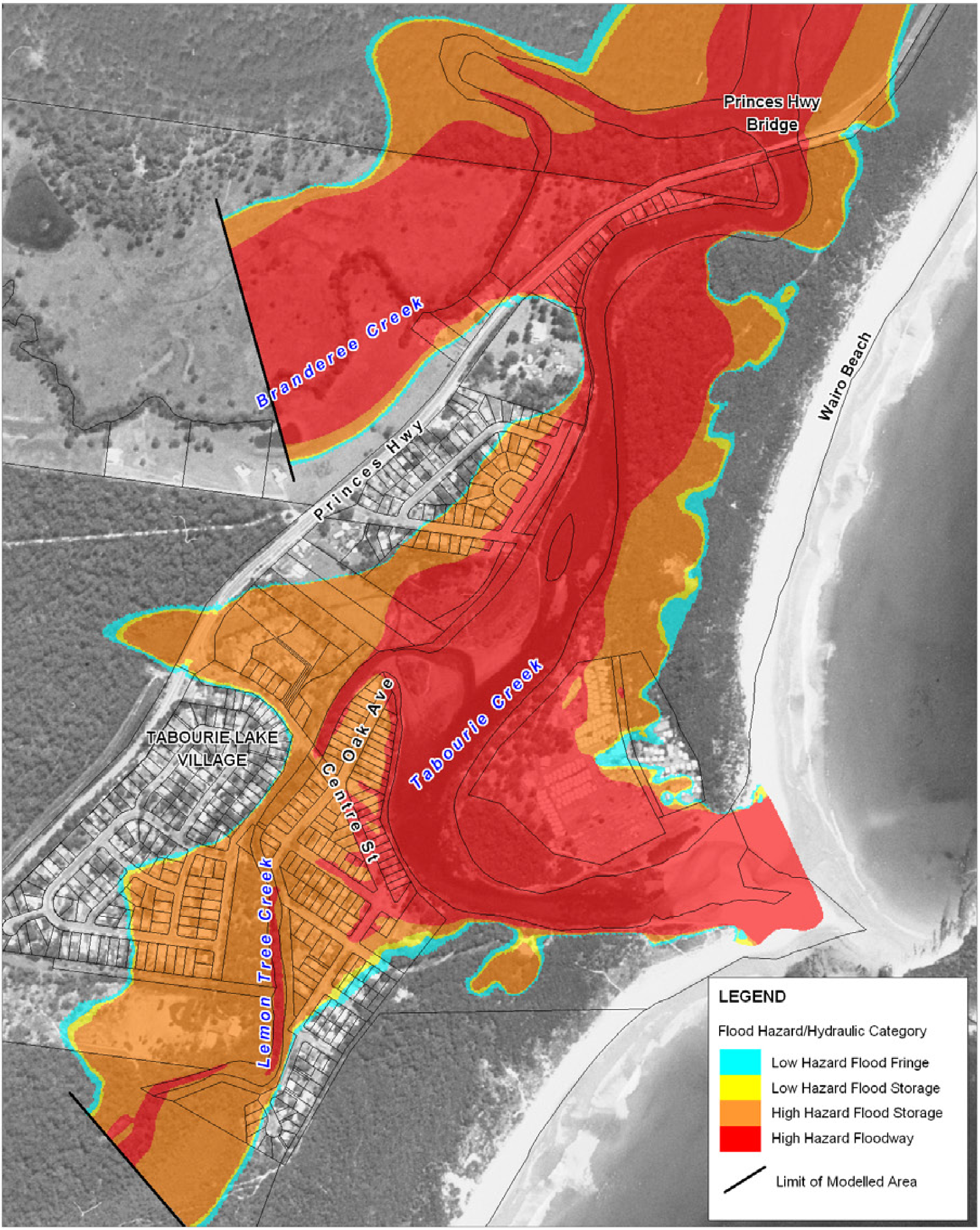
Rev: A

N

0 200 400m
Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_144_101119 2100 PMF Velocities.WOR



Title:

PMF Flood Event (2100 Conditions)
Flood Hazards and Hydraulic Categories

Figure: **A-45**

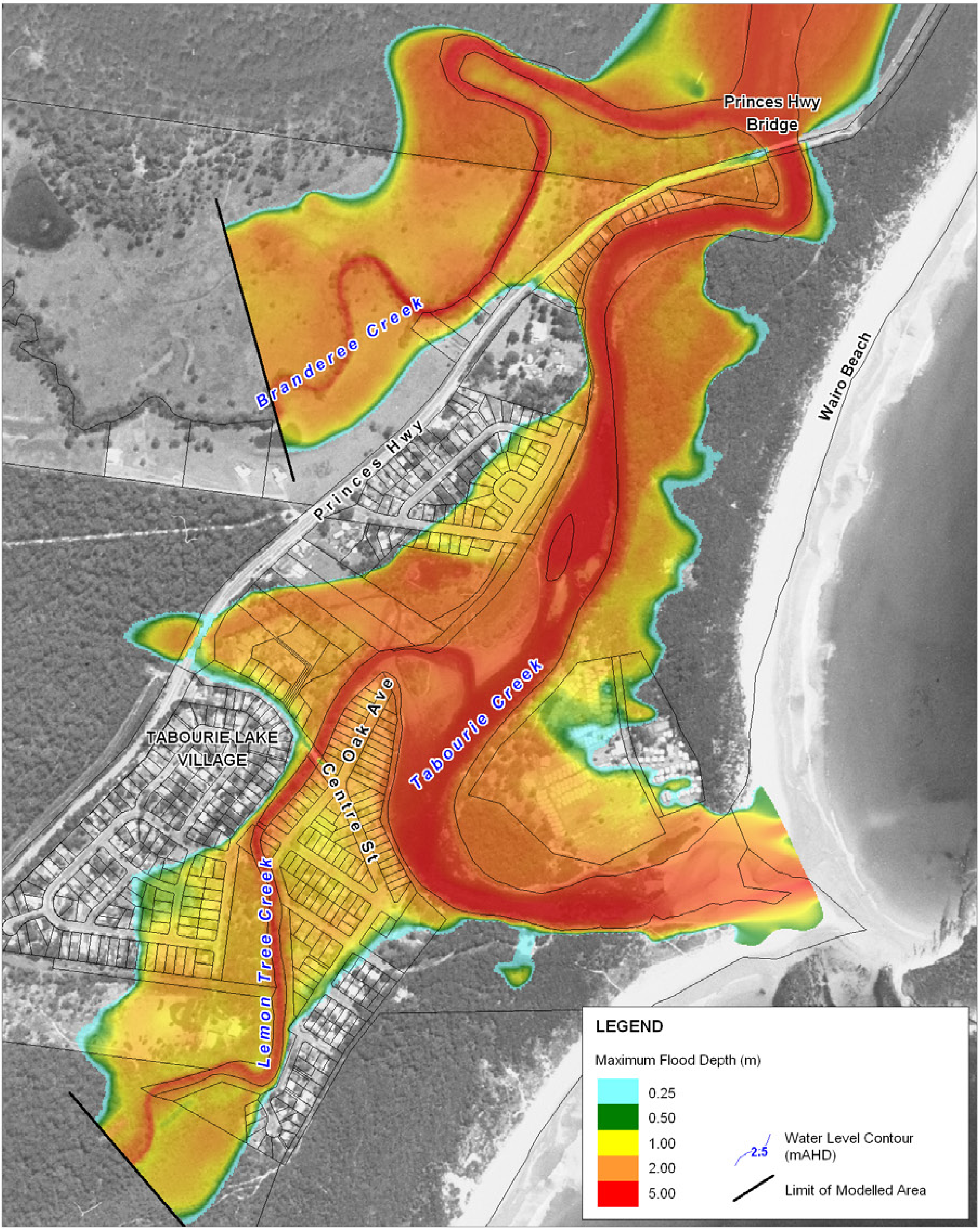
Rev: **A**

N

0 200 400m
Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_145_101119 2100 PMF Hazards.WOR



Title:

Ocean Derived Flood Event (2100 Conditions)
1% AEP Maximum Flood Depths and Water Levels

Figure: A-46

Rev: A

N

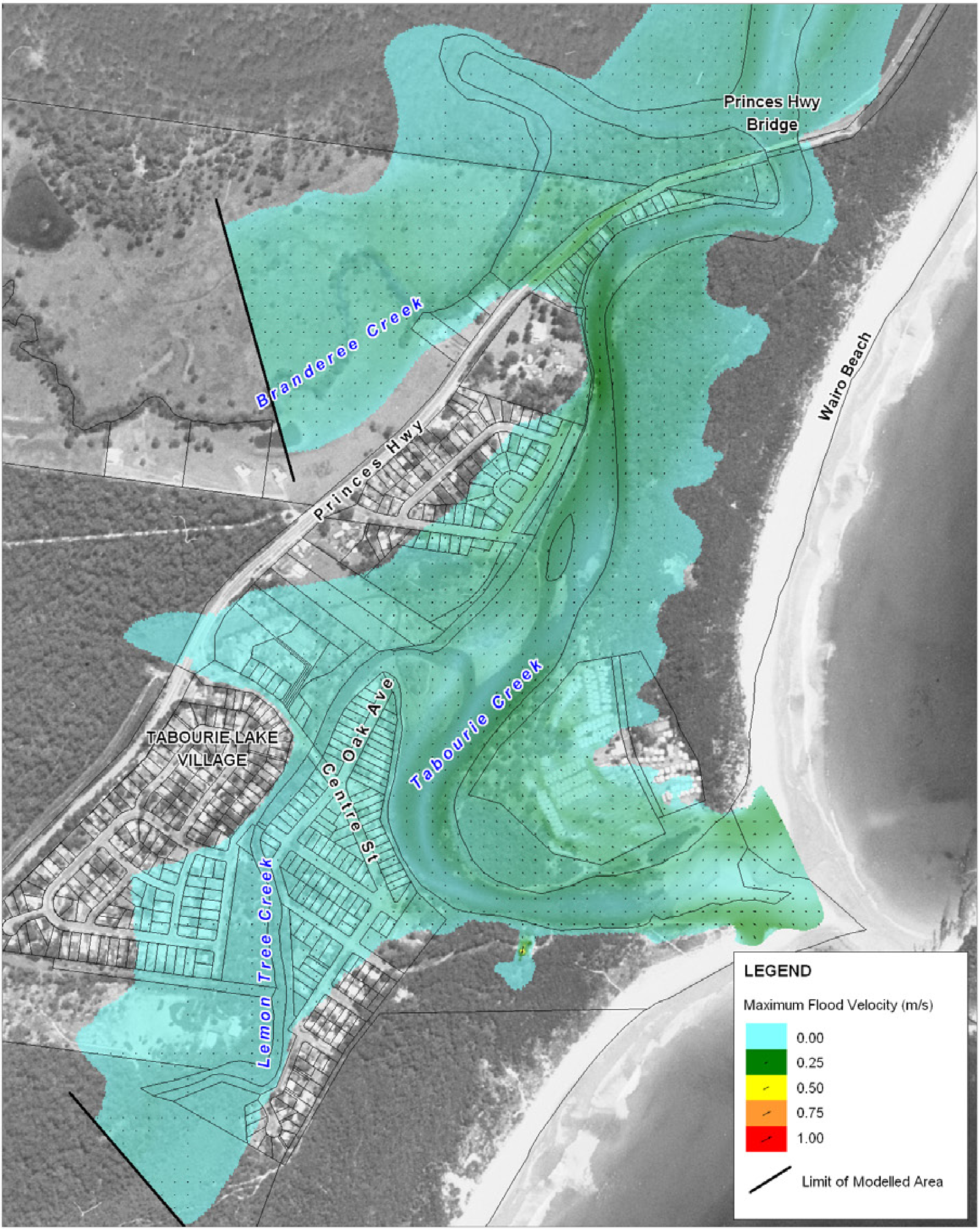
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_146_101119 2100 OC 1% Depths.WOR



Title:

Ocean Derived Flood Event (2100 Conditions)
1% AEP Maximum Flood Velocities

Figure: **A-47**

Rev: **A**

N

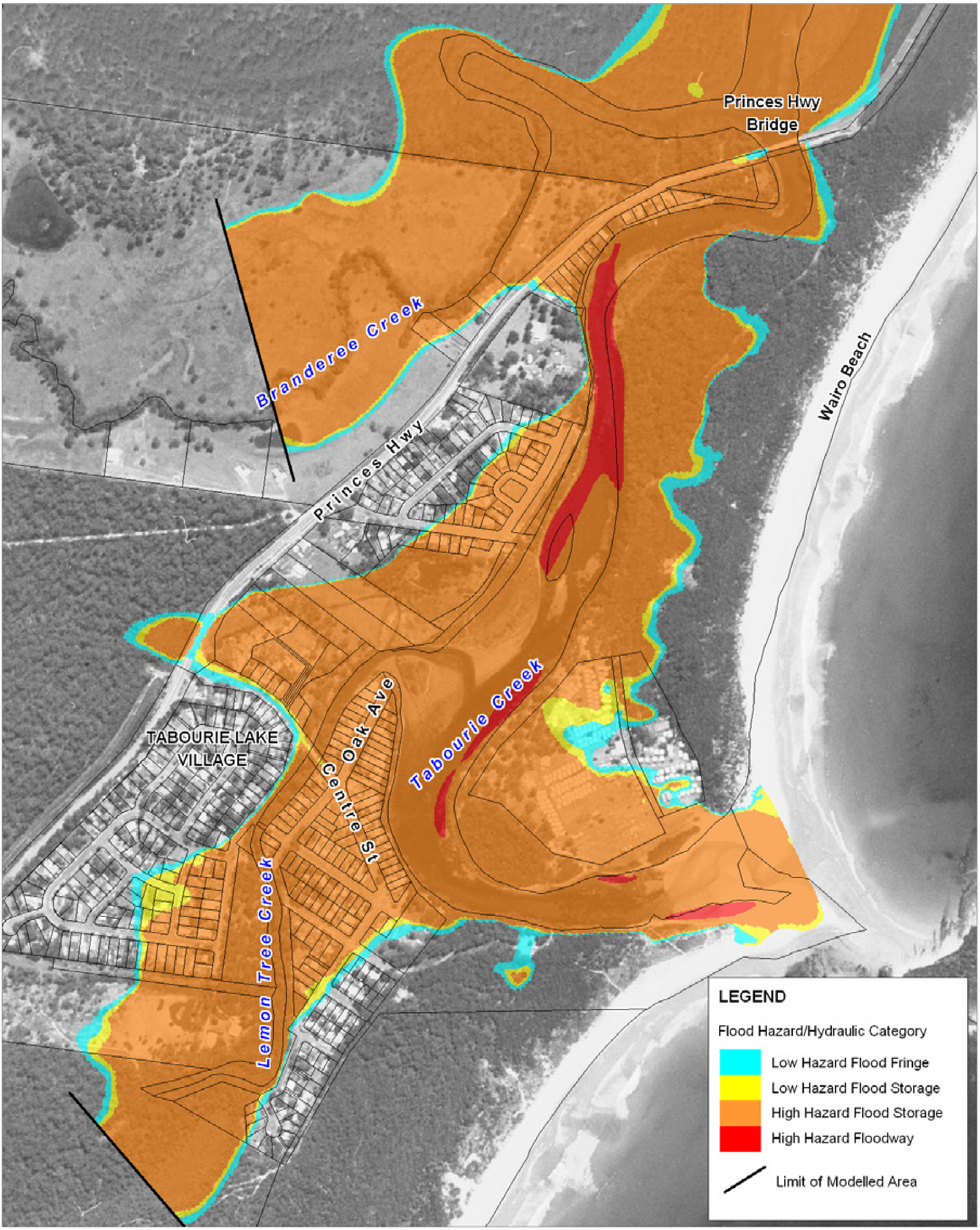
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_147_101119 2100 OC 1% Velocities.WOR



Title:

Ocean Derived Flood Event (2100 Conditions)
1% AEP Flood Hazards and Hydraulic Categories

Figure: A-48

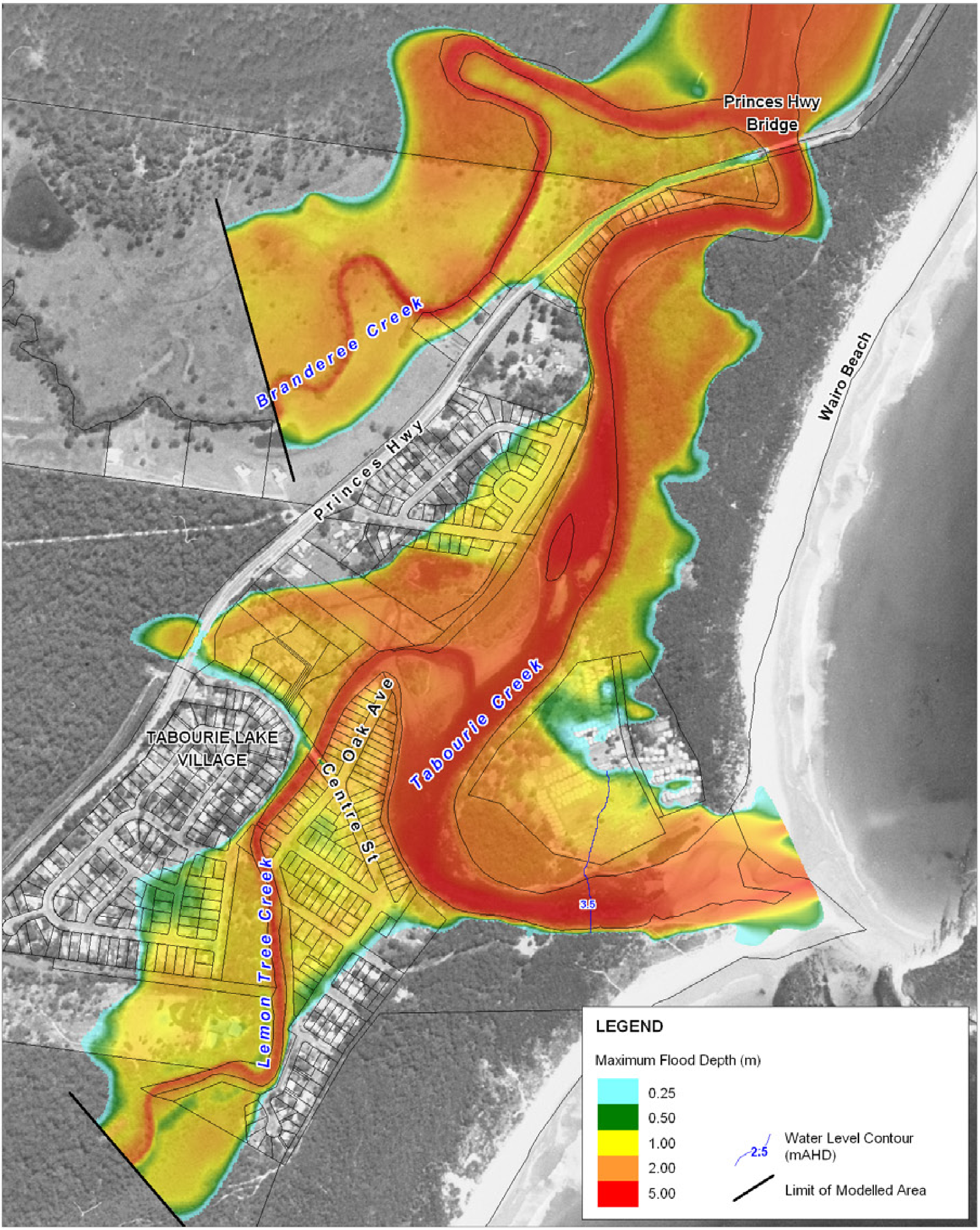
Rev: A

N

0 200 400m
Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_148_101119 2100 OC 1% Hazards.WOR



Title:

Ocean Derived Flood Event (2100 Conditions)
5% AEP Maximum Flood Depths and Water Levels

Figure: A-49

Rev: A

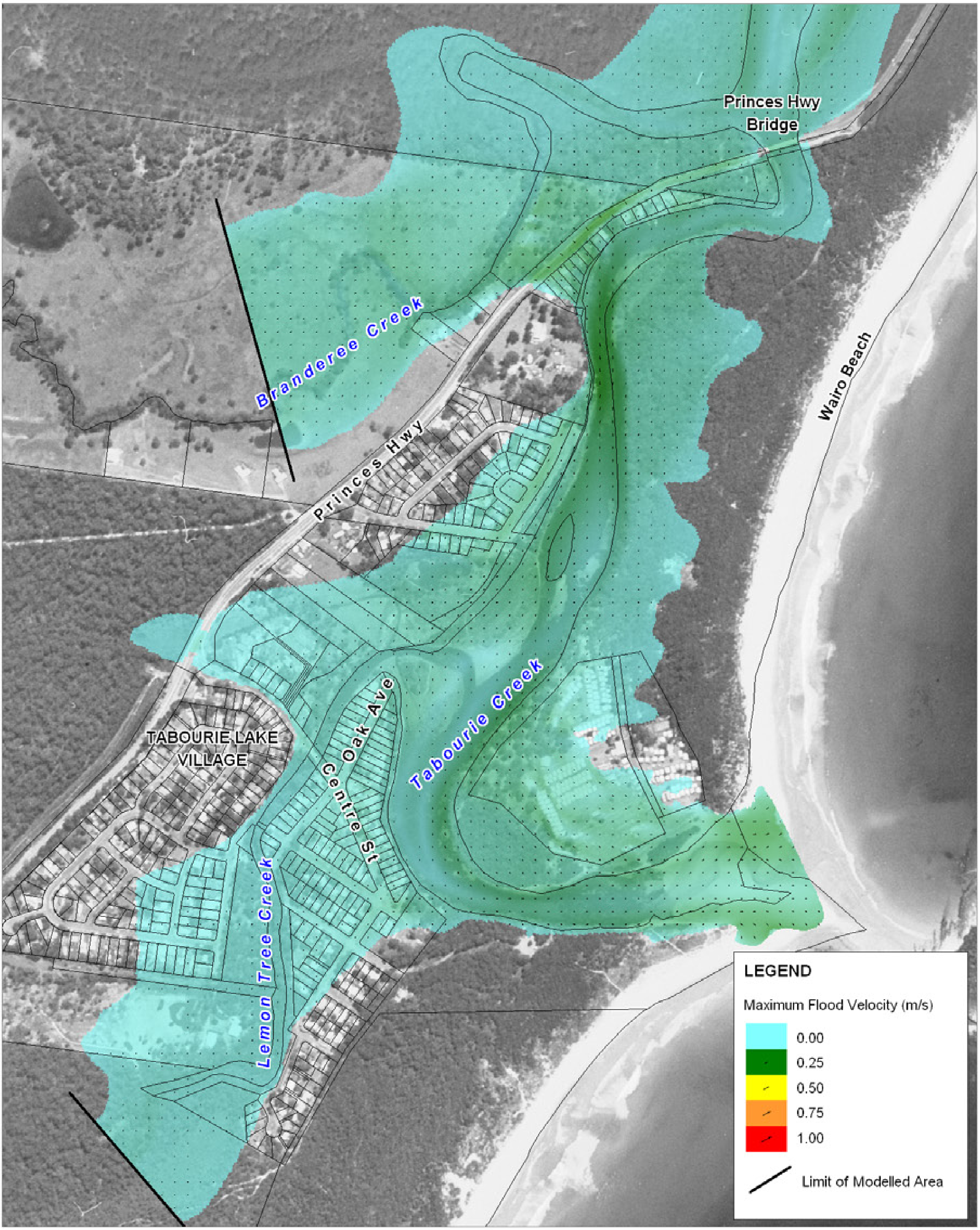
N

0 200 400m

Approx. Scale

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_149_101119 2100 OC 5% Depths.WOR



Title:

Ocean Derived Flood Event (2100 Conditions)
5% AEP Maximum Flood Velocities

Figure: A-50

Rev: A

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

N

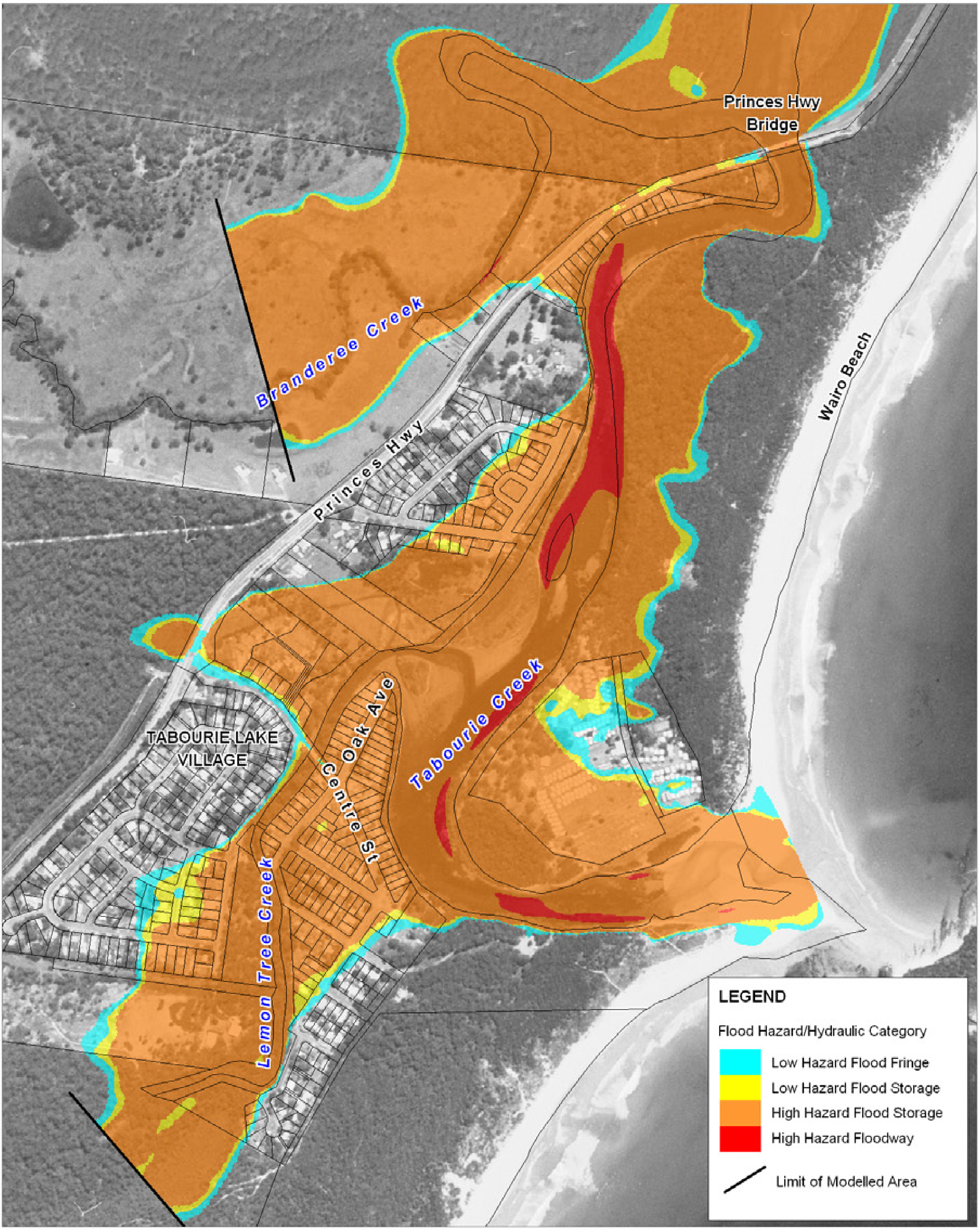
0 200 400m

Approx. Scale

BMT WBM

www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_150_101119 2100 OC 5% Velocities.WOR



Title:
Ocean Derived Flood Event (2100 Conditions)
5% AEP Flood Hazards and Hydraulic Categories

Figure: A-51

Rev: A

BMT WBM
www.wbmpl.com.au

Filepath : K:\N1268_Tabourie_Lake_Flood_Study\MapInfo\Workspaces\DRG_151_101119 2100 OC 5% Hazards.WOR

APPENDIX B: COMMUNITY CONSULTATION RESPONSES

B1 – Summary of Questionnaire Responses

B2 – Summary of Community "Drop-in" Session Comments

Below is summary of comments made by residents during the community "drop-in" session held at the Tabourie Lake Rural Fire Service Depot in Beach Street on Tuesday 19th February 2008.

The following historical changes to the Tabourie Lake township and surrounding areas were noted:

- During the severe 1974 coastal storms, a mobile dune system to the south of the entrance (at the northern end of Tabourie Beach) was washed into the entrance compartment of the Lake. Prior to this time, the Lake entrance in this area was relatively deep. Subsequent attempts to reform the dune and revegetation of the dune by the Soil Conservation Service restored and fixed the Dune in its present location. However not all of the sand thrown into the entrance waterway by the 1974 storms was subsequently extracted and the entrance has remained shallow since that time;

- An anabranch of Lemon Tree Creek, which existed between the present channel and River Road, was filled in at sometime in the past;

- Dredging of material was undertaken to raise ground levels in the vicinity of Portland Way, Weymouth Road and Lullworth Crescent prior to construction of residential homes within this area. Material was removed from the floodplain to the south of Weymouth Road (present location of small wetland) and from the main channel of Tabourie Creek, to the East of Portland Road. The area to the south of Weymouth Road is considered to be at the original level.

- During construction of the existing bridge over Lemon Tree Creek at River Road, a sand barrier was constructed across the creek, and it is claimed that this was not fully removed following completion of the construction. It is argued by some that this has resulted in significant shallowing of the Creek and the deposition of sediment at the mouth of Lemon Tree Creek, where it flows into Tabourie Creek. However, a number of individuals within the community also indicate that the mouth of Lemon Tree Creek has been shallow for over 50 years.

- Prior to the construction of properties to the north of River Road and Centre Street, ground filling was undertaken to the North of River Road (West of Lemon Tree Creek) and at the Northern end of Oak Avenue.

- The Princes Highway, immediately to the south of the Highway Bridge was previously inundated until the road was raised in the 1980's

The following dates were identified by the Community as when significant floods occurred:

- 1950's recollections of water "coming up high" in caravan park;

- 1960's, Princes Highway just south of Bridge cut by flood waters;

- 1960's flooding witnessed up Oak Avenue;

- Late 1960's – a really big flood;

- 1970's, Princes Highway just south of Bridge cut by floodwaters;

- 1971, Oak Avenue Flooded

- 1974 – by far the worst storm with wave coming through the lake entrance;
- Late 1980's, houses on the Highway were flooded;
- 1991 – Houses at end of Oak Avenue were flooded from the Ocean side, although the Lake was closed at the time;
- 1980's Caravan Park flooded;
- 1992
- 1997 - Flooding to the north of River Road witnessed
- years ago – A flood combined with the tide inundated Oak Avenue and River Road to a depth of 1ft.
- 1998 – Ponding was witnessed at the end of Centre Street, the weekend before the major floods in Wollongong in 1998.
- 2004/2005 - Flooding to the north of River Road witnessed

It is clear that people's recollections of the timing of floods are many and varied with corroborating evidence rare. However, the locations and mechanisms are recalled reasonably well.

The following areas and mechanisms for flooding were identified:

- The most at risk properties comprise eight houses on the eastern side of the Princes Highway, immediately south of the Highway Bridge. These properties can inundate regularly, without significant rainfall, as a result of the Lake Level rising. In some cases, local residents have attempted to alleviate the problem by constructing their own drainage channels.
- Additional properties that are at risk include those along the lower parts of Portland Way.
- The Caravan Park (north of the entrance) and its entrance road is also considered to be at risk from inundation by the SES.
- Properties to the north of River Road are considered by the community to be prone to flooding from the local creek which drains below the Princes Highway.
- Flow has been witnessed through the drainage easements taking water from Dermal and Beach Streets to Lemon Tree Creek;
- After flooding, the water drains rapidly through the sandy soils as the water recedes.

B3 – Summary of Individual Interview with Mr. Mike James

Shoalhaven City Council suggested contact be made with Mr Mike James of Shoalhaven Heads, who agreed to an on-site interview regarding the study. Mr. James lived in Tabourie Lake between 1956 and 1974 and maintained a strong involvement with the township until 2002. Properties owned by Mr. James' family included lots at the end of Centre St. (Near Oak Avenue) including the Castaway Holiday Flats and cottages which previously existed on top of the southern Dune system between 1956 and 1967.

Mr. James recalls seasonal algae growth within the Lake, and considers that it has occurred as long as he can recall. The blooms occur in late winter, but die back in Summer.

Mr. James recalls that the 1974 storms (25^{th} – 26^{th} May, followed by 11^{th} – 13^{th} June) were responsible for the most of the deposition of sand in from the (previously mobile) southern dune system into the Lake's entrance compartment. Mr. James recalls that the main impact was felt due to a combination of both storms, with the first storm demolishing the dune system, and the second pushing sand back into the entrance, causing the entrance deposits which remain to the present day.

These storms were strong enough to drive ocean swell into the Lake to impact upon the foreshores adjacent to residential areas in Tabourie Lake.

In concert with the impacts from ocean swell, the May storm was also accompanied by major catchment rainfall. The main Broadwater' of the Lake rose to such an extent that it overtopped the Princes Highway near Wairo. In addition, Mr. James recalls water flowing around 1 m deep across the Princes Highway, immediately to the north of River Rd, where it intersects the Princes Highway. Mr. James also recalls Lemon Tree Creek breaking its banks, and causing water to flow eastwards from Oak Avenue and Centre Street towards Tabourie Creek. The previous bridge across Lemon Tree Creek (i.e. at Bridge Rd.) was washed out by this flood.

Following the May 1974 Storm, Mr. James recalls that the entrance was as wide as he has ever seen it. The storm had scoured all sand from the entrance berm, revealing bedrock between Crampton Island and the location of the breach present in February 2008 (i.e. near the southern dune system). Northwards of this location, Mr. James recalls that the completely scoured bed comprised a mixture of clay and siltstone.

Mr. James swam in the entrance following the May 1974 storm and recalls that *8-10 feet of water existed over the hard stratum across the entrance at low tide*.

Mr. James' long association with the Town has also enabled him to gain an appreciation of the variation in weather patterns as they affect entrance conditions at Tabourie Lake. Mr. James notes that the strong westerlies, which he considers to be particularly prevalent at this location during the La Nina cycle, are capable of flattening and pushing the dune system further out to see. Mr. James believes that this results in a lower dune height than would be experienced on the open coast.

Mr. James noted that a large 'swamp' exists to the south of the town in Lemon Tree Creek.

APPENDIX C: DESIGN RAINFALL DATA

Intensity-Frequency-Duration Table

Location: 35.425S 150.400E NEAR.. Tabourie Lake Issued: 24/6/2009

Rainfall intensity in mm/h for various durations and Average Recurrence Interval

Average Recurrence Interval

Duration	1 YEAR	2 YEARS	5 YEARS	10 YEARS	20 YEARS	50 YEARS	100 YEARS
5Mins	105	134	172	194	223	261	290
6Mins	97.9	126	161	182	209	246	273
10Mins	80.3	104	134	152	176	208	232
20Mins	58.9	76.7	101	116	136	161	181
30Mins	48.0	62.8	83.9	96.8	113	136	153
1Hr	32.6	42.9	58.2	67.6	79.8	96.1	109
2Hrs	21.3	28.1	38.5	44.9	53.2	64.4	73.2
3Hrs	16.5	21.7	29.8	34.9	41.3	50.1	57.0
6Hrs	10.5	13.9	19.1	22.4	26.6	32.3	36.8
12Hrs	6.78	8.96	12.4	14.5	17.2	20.9	23.9
24Hrs	4.42	5.85	8.08	9.48	11.3	13.7	15.6
48Hrs	2.85	3.77	5.21	6.10	7.26	8.82	10.1
72Hrs	2.13	2.82	3.89	4.55	5.41	6.58	7.50

(Raw data: 43.34, 8.98, 2.83, 96.86, 20.78, 6.52, skew=0.04, F2=4.26, F50=15.76)

APPENDIX D: RESEARCH PAPER

50th Annual Floodplain Management Authorities Conference
2060, What Will Your Flood Legacy Be?
Gosford, Central Coast, 23 – 26 February 2010

A FRAMEWORK FOR PROBABILISTIC BERM HEIGHT DETERMINATION – APPLICATION TO ICOLL FLOOD STUDIES

David Wainwright[1] **and Tom E. Baldock**[2]

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email: djwainwright@wbmpl.com.au
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St. Lucia, Queensland, 4072.

ABSTRACT

Previous work has highlighted the difficulties associated with flood studies of the areas surrounding intermittently open and closed coastal lakes (ICOLLs). In particular, the entrance berm height adopted for modelling affects the results of those flood studies. It is common for the entrance berm height to be determined based on the collective judgement of people involved in the study. This judgement is normally based on available historical aerial photography and survey data, and often a 'worst-case' or highest level from that data is adopted when considering catchment based flooding which fills the closed lake before overtopping the berm. The data available for making the decision are normally limited and there remains uncertainty regarding the appropriateness of the berm height adopted. This paper presents an approach to dealing with this uncertainty. The approach includes representation of catchment runoff volumes, a stage-volume relationship for the water body, offshore wave processes, and processes at the beach. The approach allows multiple long term computational simulations of berm height and water levels to be undertaken economically. A Monte-Carlo approach is used to synthesise time series for wave height, direction and rainfall. Varying sea level, wave climate and rainfall can be represented. The approach is presently being applied to inform a flood study at Tabourie Lake, and predicts realistic berm heights and exceedance probabilities.

INTRODUCTION

There are around 70 ICOLLs on the New South Wales Coast. They vary between being hydraulically open to the ocean, or separated from it by a continuous sand barrier. These lagoons commonly exist adjacent to urban development, and around 70% of them are closed most of the time. It is common practice for the barrier to be artificially breached to relieve flooding of low-lying development. At many sites, this practice will become ineffective in mitigating against flooding within a couple of generations, if sea level rise manifests in accordance with present predictions.

Determining flood planning levels on the basis of such management practices continuing may no longer be appropriate. Flood risk planning should consider the implications for artificial entrance management to be, at least, reduced in the future (Haines et. al, 2008).

Previous flood studies (e.g. BMT-WBM 2007) have highlighted the importance of assumptions made regarding the height of the barrier at the onset of a flood. The present paper outlines a framework for determining the probability of certain barrier heights, both now and in the future. This framework is presently being applied within the Tabourie Lake Flood Study.

TABOURIE LAKE

Tabourie Lake is around 240 km south of Sydney, Australia and around 8 km south west of the town of Ulladulla, within the City of Shoalhaven. The broader expanse of the Lake is connected to the Tasman Sea at its southern end by Tabourie Creek (Figure 1).

The overall waterway area of Tabourie Lake is around 1.4 km^2, which is small compared to its catchment area (43 km^2). Depths are typically small (<1 m) and at mean high water level, the volume of the Lake is around 350 ML. In recent decades, the entrance has been managed by opening with earthmoving machinery and, under this regime, is open to the ocean around 47% of the time.

Figure 1 Lake Tabourie

While typical barrier elevations are around 2.0 m AHD, Shoalhaven City Council's present management strategy is to excavate a channel through the barrier when water levels reach a level of 1.17, primarily to relieve flooding of low lying property fringing the waterway.

REPRESENTATION OF THE SYSTEM

The processes affecting breaching, the lake water levels and the berm height have been simplified into four discrete components: a) Catchment; b) Waterway; c) Barrier; and d) Ocean. These different components have been arranged and processed using a time stepping model which takes into account rainfall, the movement and storage of water, and the actions of waves and tides in berm building on an hourly basis. The main inputs to the model are a) Rainfall; b) Bathymetry; c) Waves; and d) Tides.

Synthetic time series of waves and rainfall are calculated from statistical analyses, and those time series can be modified to incorporate the effects of climate change over time. The tide level is also adjusted using a randomly generated value to account for meteorological effects.

Catchment

The purpose of the catchment component is to simulate the volume of water entering the lagoon on a daily basis, using rainfall statistics and assumptions relating to rainfall losses. Using standard IFD values, and statistics derived from the daily rainfall record from Milton, a time series of daily rainfall values including both “storm” and more “ambient” conditions is generated. The occurrence, or otherwise, of a storm is determined stochastically, as is the amount of rainfall occurring on any given day.

The time series of rainfalls was subsequently used to generate daily flow volumes from the catchment to the Lake. Appropriate runoff coefficients were determined based on a water balance analysis of the Lake, utilising measured water level, rainfall, evaporation and a digital elevation model of the Lake and surrounds. A sensitivity analysis indicated that seepage through the entrance barrier when the Lake was closed was not important.

Waterway

Evaporation data from 1997 (determined to be an “average” year) was applied directly to the Lake surface on a daily basis throughout the simulations. The digital elevation model governs how the simulated Lake water level responds to catchment inflows and evaporation.

Ocean

The ocean component of the model represents the effect that waves and tides have on berm building processes. Tidal constituents for nearby Jervis Bay were acquired from the *Australian Tide Tables* (Australian Hydrographic Service, 2005). These constituents were then used to generate a typical year long ocean tide time series, utilizing the methods detailed in Foreman, (1996). The astronomic tide was further adjusted for barometric effects, coastal trapped waves and wind stresses using “Residual Error” values published in MHL (2003). The adjusted tidal level was used as the baseline above which the calculated wave run-up (as described below under “Barrier”) was considered to reach. Wave set-up is integrally calculated as part of the run-up determination.

In order to predict a time series of run-up levels, it is also necessary to have a time series of offshore wave conditions. Implicit in the run-up formula used is the assumption that waves approach the shoreline from a shore normal direction. At Lake Tabourie, where wave climate is dominated by swells from the South - South Easterly sector, the wave shading effect of Crampton Island has to be taken into consideration. The spectral wave model SWAN was used to investigate this effect and appropriate refraction coefficients derived.

Wave statistics were determined from the existing Batemans Bay WaveRider™ buoy data record, and these were used to synthesise a time series of offshore wave heights.

Barrier

The barrier was modelled as a single elevation value, representing the crest of the berm. This is a simplified representation of the barrier which can vary dramatically in shore normal location, width and orientation as a result of coastal processes. If the water level in the Lake was simulated to exceed the berm crest elevation, the Lake was assumed to have breached.

Aside from breaching, the model can increase the barrier elevation as a result of coastal processes. The potential for barrier building for a given offshore wave condition was determined using a wave steepness criterion outlined in Weir et al (2004)

A field data collection exercise involving the measurement of swash run-up heights at Tabourie Beach was undertaken on February 21st, 2008. The measured heights of run-up were fitted to a Rayleigh distribution, following the methods of Nielsen & Hanslow (1991) and accounting for the refraction coefficient described above. The derived coefficients for the fitted probability distribution were consistent with the findings of Neilsen and Hanslow. Weir (2007) finds that significant sediment deposition does not occur landward of 2% run-up elevation. The 2% exceedance level was adopted for determining the height to which the barrier might be built.

RESULTS

Daily time series of values have been analysed for three sea level rise simulations:

1. Present tidal levels run over a 500 year period;

2. Projected 2050 time frame tidal levels (+ 0.40), also run over a 500 year period; and
3. 100 year sea level rise (0.90 by 2100), calculating 50% berm levels on a decadal basis.

The results are presented in Table 1 and Table 2.

Table 1 Preliminary Estimate of Barrier Elevations Exceeded for specified Proportions of time (m AHD)

% Time Exceeded	Present Day	2050
80	1.80	2.22
50	1.91	2.33
20	1.99	2.41
10	2.03	2.45
5	2.06	2.48
1	2.10	2.52

Table 1 illustrates that, under sea level rise alone, the increase in berm level that would be exceeded for a certain percentage of the time is slightly more than the increase in mean sea level (0.42 m c.f. 0.40 m). This is due to variation in the stage volume relationship of the lagoon at higher water levels.

Table 2 Preliminary Assessment of 50% Berm Height Exceedance Level, with Rising Sea Level (Decadal Basis)

Decade	Elevation (m AHD)
2010's	2.00
2020's	2.09
2030's	2.18
2040's	2.30
2050's	2.41
2060's	2.49
2070's	2.56
2080's	2.70
2090's	2.78

Table 2 shows that the median berm height that can be expected in any given decade could be expected to rise, on average, at around 0.1 m / decade, providing that sea level has risen by 0.90 m at 2100. Variations from the trend are related to the probabilistic simulation of extended dry or wet periods during any particular decade.

SHORTCOMINGS AND FUTURE IMPROVEMENTS

The model is presently being developed further to improve its rigour. Some improvements include: a) the process of berm erosion and other cross shore processes; b) the joint probability of wind set up (contributing to tidal residual) and waves; c) seasonality which is presently neglected; d) variation in flood rainfall intensities and wave direction with a changing climate; e) an improved representation of time for closure following breaching.

TAKE HOME MESSAGES

The developed model has been shown to replicate typical berm levels at the site, given present day conditions. It has been used to probabilistically assess berm level heights under sea level rise scenarios, resulting in predictions that appear sensible, although work continues.

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