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Burrill Lake Catchment Flood Study

R.N0848.002.02.doc

July 2007

Burrill Lake Catchment Flood Study

Prepared For:	Shoalhaven City Council
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Title :	Burrill Lake Catchment Flood Study
Author :	David Wainwright
Synopsis :	Report for the Burrill Lake Catchment Flood Study covering the development and calibration of computer models, establishment of design flood behaviour and flood mapping.

REVISION/CHECKING HISTORY

REVISION NUMBER	DATE OF ISSUE	CHECKED BY		ISSUED BY	
0 (calibration report)	21/2/05	PEH / WJS		DJW	
1 (draft report)	17/10/06	PEH		DJW	
2 (final report)	20/07/07	PEH		DJW	

DISTRIBUTION

DESTINATION	REVISION			
	0	1	2	3
SCC	3	3	20	
WBM File	1	1	1	
WBM Library			1	

FOREWORD

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 the first of the four stages for the Burrill Lake area. It has been prepared for Shoalhaven City Council to describe and define the existing flood behaviour and establish the basis for floodplain management activities in the future.

The NSW State Government has provided financial assistance towards the cost of this study under its Floodplain Management Program.

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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)
barometric setup	The increase in mean sea level caused by a drop in barometric pressure.
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.
fluvial delta	Terrestrial sediment deposited at the mouth of a creek or river under the influence of non-tidal flows
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
marine dropover	The area within a tidal channel, where shallow bathymetry related to the movement of marine sand transitions into deeper bathymetry. Common at the upstream end of a tidal inlet where it meets the deeper water of a coastal lake.
morphological	Pertaining to geomorphology
Orographic	Relating to the influence of local topography, especially mountains
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.
untrained entrance	A coastal entrance that does not have features, such as breakwaters, which train (constrain the location of) tidal flows
water level	See flood level.

1 INTRODUCTION

1.1 Study Location

Burrill Lake is located on the New South Wales south coast, about 180 km south of Sydney (see Figure 1-1). The Lake entrance is around 5 kilometres south-south-west of Ulladulla. The Lake is located within the Local Government Area (LGA) of the City of Shoalhaven. An oblique view of the Burrill Lake catchment is presented in Figure 1-2, showing key locations and areas of interest.

1.2 Waterway Characteristics

Burrill Lake (refer Figure 1-3) has a surface area of $4km^2$ (WBM, 2001) and is connected to the Tasman Sea by Burrill Inlet, a 3km long, shallow and sinuous channel with a typical depth of around 1m at low tide. The Inlet is crossed by a causeway and bridge. The level of the causeway, which was originally built in the 1880’s, was raised in the 1960's to reduce the frequency of inundation of the highway (SCC, 2003). The span of the bridge over the Inlet is approximately 45 metres. Upstream of the bridge, gross channel positions and the extent of the intertidal shoal have effectively remained unchanged.

The entrance to Burrill Lake has intermittently closed in the past, however, it is generally open albeit heavily shoaled. The entrance closed most recently in early 2005 (pers comm. D.Heubusch SCC, 2005), but prior to this remained continuously open from 1987 (Shoalhaven City Council, 2002) since at least the 1940's (PWD, 1992). The entrance shoals are the most active along the Inlet.

The main body of the Lake comprises two basins, one to the north and one to the south of the marine dropover, where Burrill Inlet meets the Lake. The northern basin extends for around two kilometres northwards from the dropover, with a typical width of around one kilometre and depths of up to 9 metres (below AHD). The Lake’s main tributary, Stony Creek, flows into the northern end of the northern basin. The southern basin extends for around two kilometres in a south-westerly direction from the dropover, with a typical width of around 500 metres and depths of up to 9 metres (below AHD). A small, unnamed tributary flows into the bottom end of the southern basin.

1.3 Catchment Characteristics

The Burrill Lake catchment covers an area of 78 km^2 and is largely covered by agricultural grazing lands to the north and eucalypt forest in the south and west. The main land uses in the Burrill Lake catchment are rural (56%) and forest (37%). The majority of the Stony Creek catchment, particularly towards the northern part of the catchment, has been cleared for agriculture. Most of the foreshore of Burrill Lake remains naturally vegetated.

In total there are six urban areas in the catchment: Dolphin Point, Burrill Lake, Bungalow Park, Kings Point, Milton and west Ulladulla.

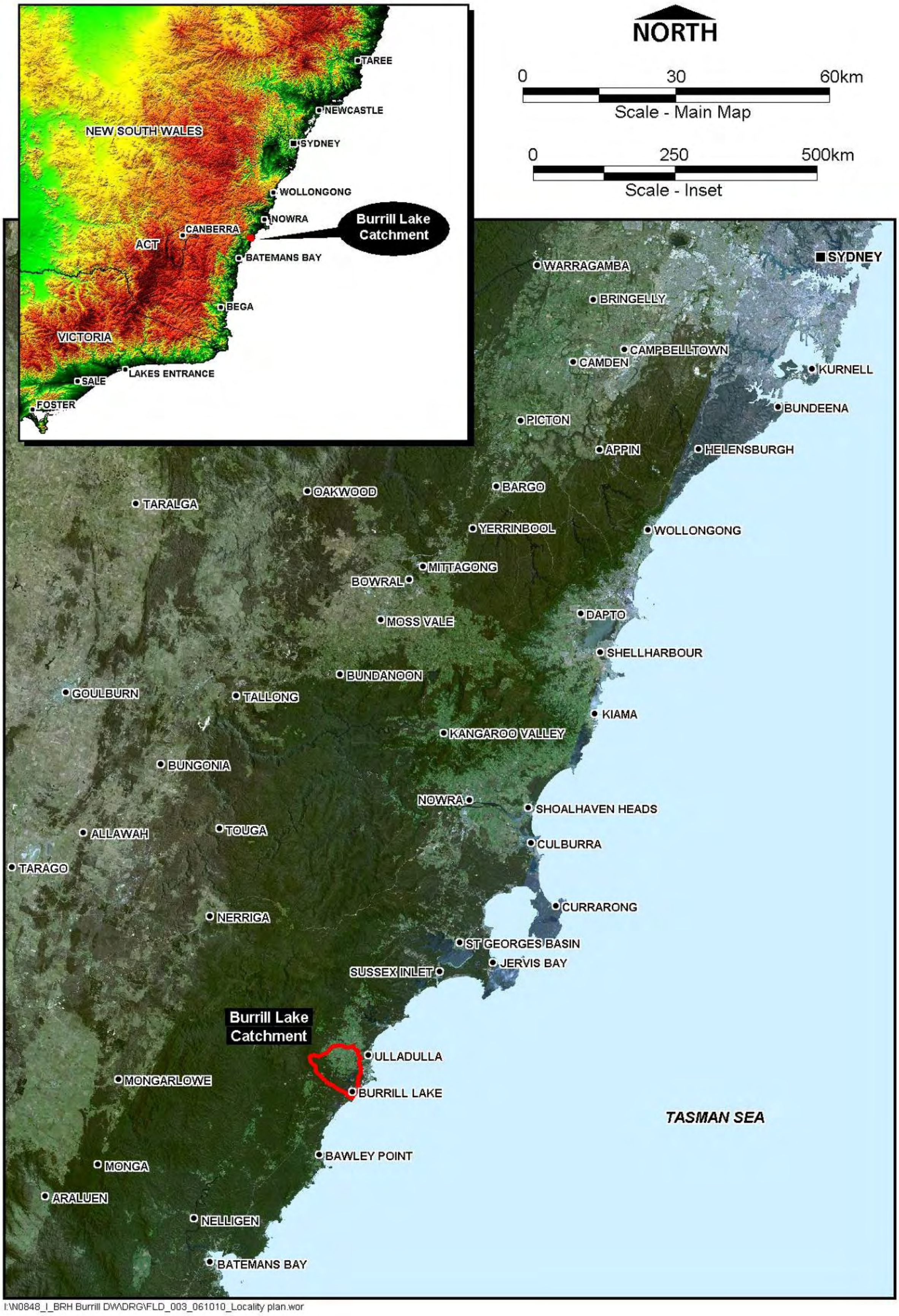


Figure 1-1 Burrill Lake Catchment Locality Map

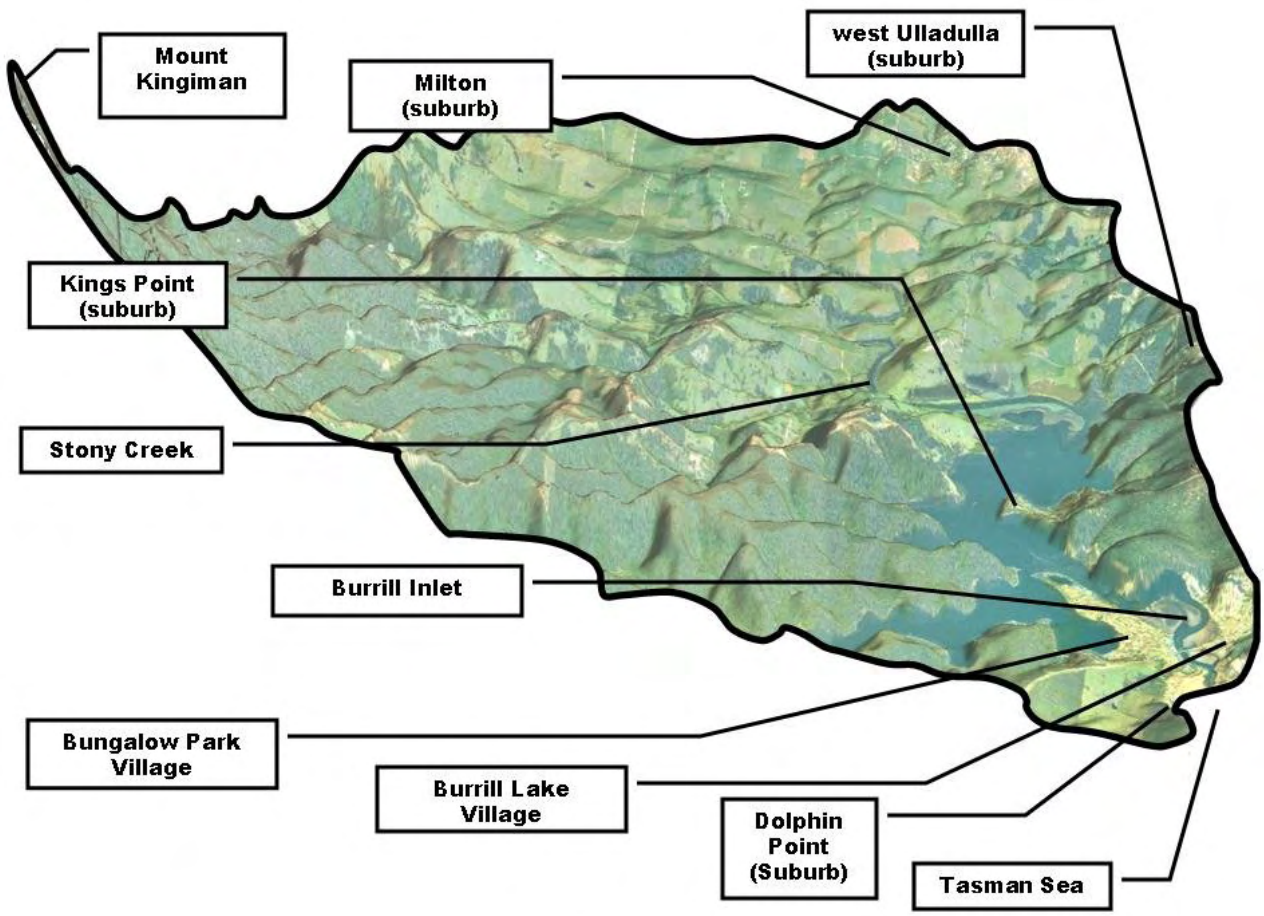


Figure 1-2 Burrill Lake Catchment
(Oblique 4 times Vertical Exaggeration)

The small village of Burrill Lake lies on the northern side of Burrill Inlet. The Dolphin Point suburb occupies high ground on the headland to the south of Burrill Inlet. The suburb of Bungalow Park occupies the southern bank of Burrill Inlet. A small portion of west Ulladulla, an area set aside for alternative rural development, is located along the eastern fringe of the catchment, to the north of the Lake. The more recently developed suburb of Kings Point is located on a peninsula that extends westwards into the northern basin of the Lake.

South of the Inlet, the land is generally low lying and flat, rising to Dolphin Point. North of the Inlet, and west of Burrill Lake village, the terrain is steep and densely timbered.

The forest and riparian zone has been cleared and/or thinned to allow for residential development at Kings Point.

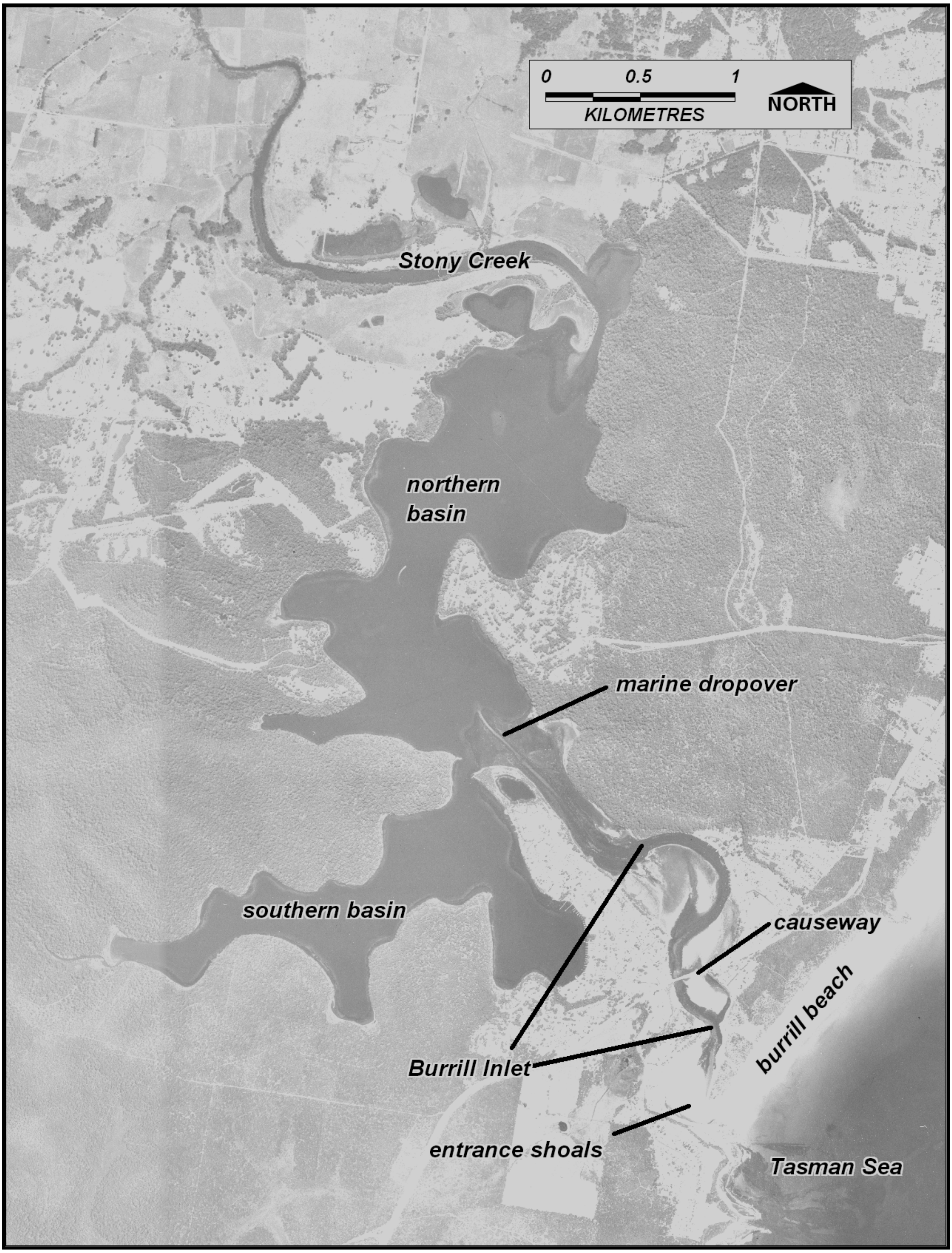


Figure 1-3 Features of the Burrill Lake Waterway

Figure 1-4 Features in the Vicinity of Burrill Inlet

To the west and south-west of the main body of the Lake, forestry has been carried out over extended periods, including selective logging and wholesale clearing and replanting. Areas not utilised for forestry comprise remnant closed eucalypt forest dominated by Bangalay, Spotted Gum and River Peppermint (WBM, 2003). A dense, shrubby understorey is present through undisturbed areas to the south west of the Lake.

Creek systems draining the northern sections of the catchment typically flow through areas that have been cleared for agriculture with little riparian vegetation (Shoalhaven City Council, 2002). Only small areas of the floodplain previously formed coastal wetlands. Those wetlands fronting Burrill Inlet have been filled for urban development.

1.4 The Need for Floodplain Management at Burrill Lake

The villages of Burrill Lake and Bungalow Park are low-lying and prone to flooding. Prior to this study, the community's understanding of flooding processes within the areas surrounding Burrill Inlet has been dependant upon local knowledge derived from experience of past floods.

Local knowledge of flooding is scarce. The most severe flood event recalled by surveyed residents occurred in February 1971. More commonly recalled events occurred in the early 1990's, but available information indicates that these floods were not as significant as the flood of 1971.

The NSW Public Works Department (1992) indicates that water levels in the Lake have been observed since about 1950 and that flooding has previously reached 2.0 m AHD. This level has subsequently been considered as indicative of the 1 in 100 yr event and as a basis for the flood planning level, in the absence of better information.

In addition, it is reported (NSW Public Works Department, 1992) that in February 1971, a peak flood level of around 2.2 m above low water (around 0.4 m above the soffit of the causeway bridge) was reached. Based on interpretation of levels surveyed during the course of the current study, and design drawings provided as an appendix to the 1992 study, it is estimated that the soffit level is around 1.65 m AHD. Accordingly, the 1971 peak flood level at the bridge is estimated to have been between 2.0 and 2.1 m AHD.

The scarcity of local knowledge of historical flood behaviour is mirrored by a lack of recorded data on rainfall, flows and water levels within the waterway and catchment. It has therefore been difficult to assess historical flood behaviour throughout the land surrounding Burrill Inlet.

Flooding has clearly occurred within the study area in the past. Nevertheless, the lack of knowledge about flooding around Burrill Lake, has made floodplain management decisions difficult. Consequently, SCC desires to approach local floodplain management in a more considered and systematic manner. This study comprises the initial stages of that systematic approach, as outlined in the Floodplain Management Manual (NSW Government, 2005). The approach will allow for more informed planning decisions within the floodplain of Burrill Lake.

1.5 General Floodplain Risk Management Approach

The Floodplain Risk Management Approach is described comprehensively in Sections 1, 2 and 3 of the 2005 NSW Floodplain Development Manual. The Floodplain Risk Management Process, as defined by the Manual, is summarised in Figure 1-5.

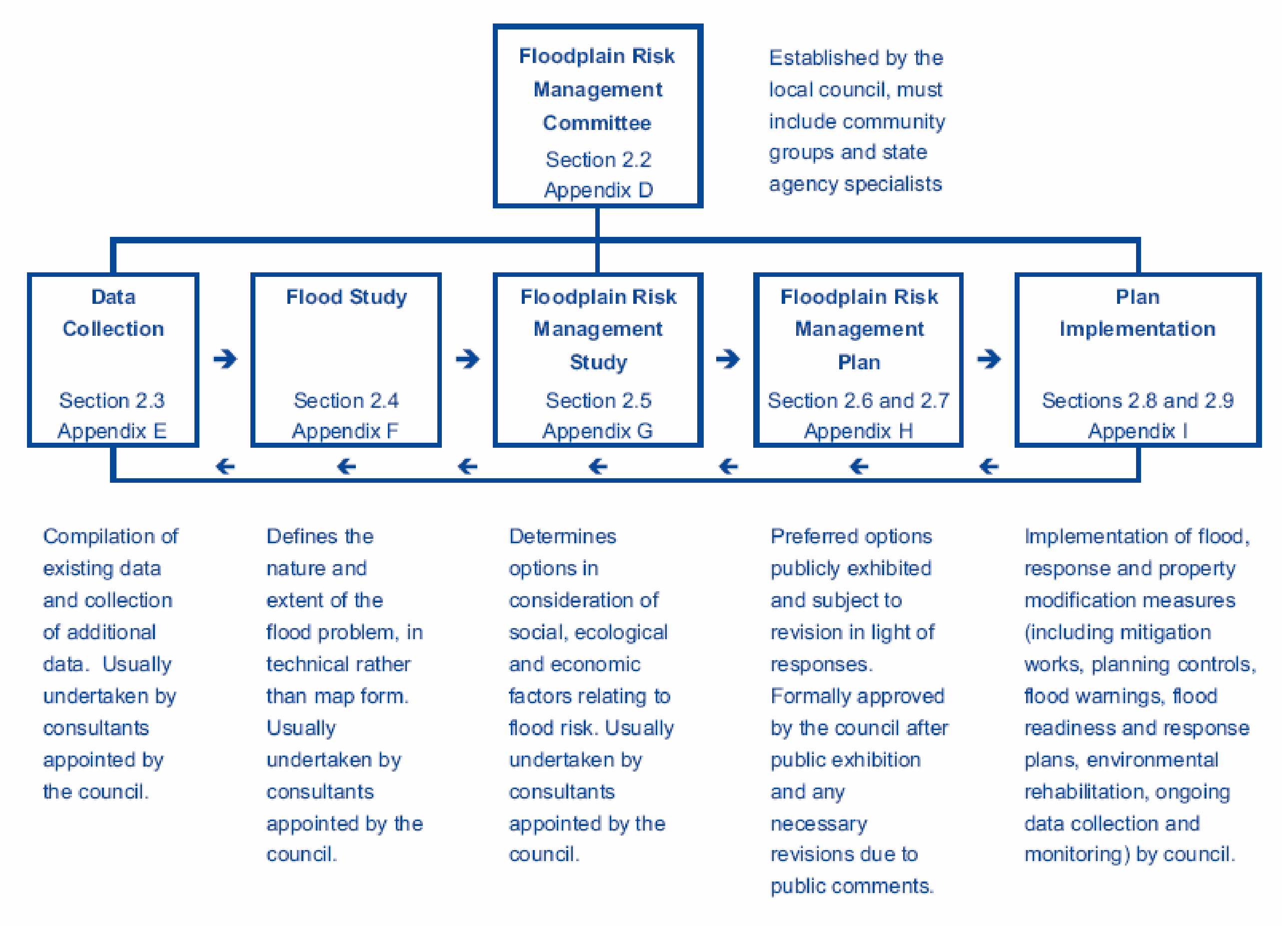


Figure 1-5 The NSW Floodplain Risk Management Process (Source: 2005 NSW Floodplain Development Manual)

1.6 Study Objectives

The primary objective of the Burrill Lake Catchment Flood Study is to examine and define the flood behaviour of the Lake in the vicinity of the villages surrounding Burrill Inlet. The Flood Study findings will provide input to the subsequent Floodplain Risk Management Study.

This study has involved the development of computer models to simulate the behaviour of floods in the areas surrounding Burrill Inlet. These models will also be used in to help in the preparation of the Floodplain Risk Management Study.

1.7 About This Report

This report documents the Flood Study's objectives, results and conclusions. It is divided into a main report that presents the Flood Study in a relatively non-technical manner, and several appendices containing additional data and further information.

Section 1 introduces the study.

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

Section 3 outlines the available historical flood information collected and collated for this study.

Section 4 details the development of the computer models.

Section 5 details the model calibration and validation process.

Section 6 describes additional sensitivity and model assessment tests.

Section 7 presents the design flood conditions.

2 STUDY APPROACH

The general approach and methodology employed to achieve the study objectives involved:

- compilation and review of available information;
- site inspections;
- identification of historical changes to topography;
- collection of historical flood information;
- collection of additional topographic survey data;
- setup of hydrology and hydrodynamic models;
- calibration and validation of models; and
- establish design flood conditions.

The above tasks are described generally in the following Sections, while results of the application of this methodology are discussed in subsequent Sections and Appendices, as appropriate.

2.1 Compilation and Review of Available Information

Available data and background information was collated and reviewed.

Two major hydrodynamic investigations have been undertaken in the past 15 years:

- Public Works Department NSW (1992) *Burrill Inlet Waterway Improvements : Feasibility Study*; and
- WBM Oceanics Australia (2001) *Burrill Inlet Causeway Options Study*

Although these sources tend to focus on the entrance morphological processes, there is some information provided on flooding processes.

Other key data sources that were available at the commencement of the study included the following:

- An hydrographic survey of the Lake and Burrill Inlet undertaken by the Department of Land and Water Conservation in 2001;
- A tidal gauging undertaken by the Department of Public Works and Services' Manly Hydraulics Laboratory in 2001;
- Historical air photography of the Burrill Inet.

It was found from a review of the available background information that there was generally a lack of detailed rainfall data for the catchment. Indeed, the closest pluviometer rainfall gauging stations is around 50 km to the north of the catchment. The only appropriate daily rainfall station with a suitable long record is located in Milton, adjacent to the northern fringe of the catchment.

No ground based floodplain survey information was available as part of the background review, to be incorporated into the flood model. Similarly, there was no survey information available for historical flood levels within the catchment although some anecdotal information was available. Furthermore, there was no flood flow gauging information available within the catchment.

A permanent automatic water level recording station was installed adjacent and upstream of the Princes Highway Bridge (Causeway) in November, 1991. One significant flood (February, 1992) was identified as having occurred since the installation of that recorder.

Where appropriate, the relevant data has been incorporated into the study. Nevertheless, it should be appreciated that there is a general paucity of flood data available for Burrill Lake and its catchment.

Given the data limitations, a broad and generic approach has been applied in the calibration and validation of both the hydrologic and hydrodynamic models.

Additional data, including an extensive survey of the floodplain, a survey of local residents (via questionnaire from which anecdotal information was acquired), and additional historical flood levels reported by the community, were collected during the course of this study.

2.2 Site Inspections

An initial site inspection was carried out to allow study personnel to become familiar with the area and to determine additional data requirements. Additional site inspections were conducted, on an as-required basis, during the course of the study to investigate specific details and confirm computer modelling assumptions.

Site inspections focused on determining structure sizes, current vegetation cover, general ground-truthing of topographic features, current geomorphology of the entrance channel (including the entrance shoals) and liaison with community members.

2.3 Identification of Historical Changes to Topography and Development Patterns

The computer models developed as part of this study were calibrated and verified to historical floods (February 1971 and February 1992, respectively) to check their performance against known flood behaviour. This task required the identification of significant changes to catchment, lake and floodplain topography over the past 30 years or so.

The changes that would have affected flood behaviour within Burrill Lake include ongoing variations in the entrance shoal configuration and gradual development within the catchment.

The entrance shoal configuration has varied notably over the past 30 years, as would be expected from the untrained entrance of an intermittently open and closed lake. This issue is discussed in significant detail in WBM Oceanics Australia (2001), and information from that reference has been used in the derivation of entrance conditions for the calibration events, and the design flood simulations.

Changes in land use in the catchment and around Burrill Inlet have been determined through examination of aerial photographs, as well as information contained within the available references.

Comparison of aerial photography from two dates (1959 & 2001) covering the majority of the catchment indicates that, in the past 40 years, the change in land use in the overall catchment to the south, west and north of the Lake has not been of a scale to significantly alter the hydrological behaviour of the catchment. The area to the west and south of the Lake has comprised a timbered catchment for this whole period, while the area to the north, and north-west of the Lake has comprised cleared land used for agricultural purposes. While there are some areas where clearing has taken place between these two dates (e.g. west Ulladulla), these changes are not substantial enough to have a significant impact on the overall hydrologic processes within the catchment.

The changes to the nature of land use in the vicinity of Burrill Lake will have affected flooding processes. Aerial photographs from a series of dates (1967, 1971-72, 1977, 1981, 1986, 1993, 1996 and 1999) were inspected to identify the areas of significant change. The areas where change has occurred are shown on Figure 2-1. Descriptions of the changes that have occurred are provided below.

Figure 2-1 Reference Areas for Historical Changes to Land Use around Burrill Inlet

- Area A (Kings Point): The residential area of Kings Point did not exist prior to 1967, although unsealed roads were present in the same location as the sealed roads of the current residential subdivision. Residential homes were built in the 1970's and 1980's, although the pattern of development was sporadic. Significant clearing and infill development at Kings Point has occurred during the last decade.
- Area B (Bungalow Park North): The residential area of Bungalow Park, to the north of Moore St (also referred to as the Max Auld Subdivision), remained undeveloped until at least 1977, although it appears that the land had been cleared before 1967, possibly for use as pasture. Between 1977 and 1981, the area appears to have been developed for residential lots including creation of the small lake located at the northern tip of the Peninsula. It is possible that material excavated to create the small lake was used as fill on residential lots in Area B. The subdivision was laid out and construction commenced between 1981 and 1986. By 1993, most of the residential lots had been developed.
- Area C (Burrill Lake North): The residential area of Burrill Lake North, comprising Canberra Crescent, Braidwood Avenue and Princess Avenue, was undeveloped in 1967, although some unsealed roads were present prior to this time. By 1972, the existing road layout was present although the residential lots remained predominantly undeveloped. Over the following decade, the available residential lots were developed.
- Area D (Burrill Lake South): By 1967, the residential area of Burrill Lake South, comprising the low lying land to the north of Burrill Inlet and west of the Princes Highway, was already cleared and developed to the same extent as it is at present.
- Area E (Bungalow Park South): By 1967, the residential area of Bungalow Park South, comprising the area to the south of Burrill Inlet, and located between Princes Highway and Moore St, was cleared and developed to the same extent as it is now.
- Area F (Bungalow Park West): In 1967, the residential area of Bungalow Park West, comprising Wallaroy Drive, Wyoming Avenue and Woodlawn Avenue, and the Bungalow Tourist Park had not yet been constructed although some unsealed roads were present. Over the following decade, it appears that the majority of residential lots were built upon, resulting in the current developed situation. The Bungalow Tourist Park was established between 1972 and 1977.
- Area G (Burrill Lake Caravan Park): The Burrill Lake Caravan Park, now known as Holiday Haven, did not exist in 1967. The area it now occupies comprised a partly cleared patch of coastal scrub. Between 1967 and 1977, the area was substantially cleared, although there was no evidence of a permanent facility on this site in 1977. The Caravan Park was established between 1977 and 1981.
- Area H (Lions Park): Lions Park was fairly densely timbered in 1967. Between 1967 and 1972, the vegetation appears to have been cleared towards the south west of the park and thinned in other areas. Public Works Department NSW (1992) indicates that dredging was undertaken between 1972 and 1975 from the channel adjacent to Lions Park. The material appears to have been spread to depths of between 1.5 and 3.0 m AHD over a combined area of around a hectare at Lions Park, resulting in the placement of around 25,000 to 30,000 m^3 of sand. This is likely to have occurred in the areas that appear as being cleared in the 1972 photograph. Since this time it is apparent that minimal change to the overall shape or level of Lions Park has occurred. The ground surface has been stabilised through the establishment of grass throughout the reserve.

- Area I (Dolphin Point Tourist Park): The caravan park in this area had not yet been developed in 1967, although the land had been cleared at this time. The park was established some time between 1972 and 1977. The park has maintained its configuration since that time, although it is more densely populated now, with larger vans.

2.4 Collection of Historical Flood Information

As discussed above, limited historical flood information had been identified in previous studies. To supplement this information, a resident survey was undertaken. A questionnaire was delivered to all households in the vicinity of Burrill Inlet, 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. As a result of these interviews a total of 12 new historical flood marks were recorded. Interestingly, information on a number of the marks obtained by the surveyors was gained from passers by as they completed their work, and not necessarily through the resident questionnaire. The locations of the historical marks are shown on Figure 2-2. The information provided by all respondents was entered into a database for analysis with the following conclusions:

- The dates of recalled flood events vary dramatically, however, the most commonly recalled events occurred during the following periods:
    - 1969;
    - 1970 – 1971;
    - 1974;
    - 1989 – 1992;
    - 2002 (Possibly April); and
    - 2003 (Possibly May).
- Of those recorded, the historical events that appear to be most vividly recalled by the greatest number of respondents occurred during 1991-1992 and 1971.
- Factors identified by respondents as contributing to flooding include physical processes (rain, tides, waves, wind, storms), construction within the catchment (particularly the causeway), closure of the Lake entrance, local stormwater issues such as the absence of kerb and gutter and a lack of maintenance of drainage infrastructure, and catchment modifications such as the filling of 'swamps' and land clearing.
- The community considers that, historically, flooding is more of an inconvenience than a danger. Evacuation has rarely been required.

Overall, the historical flood marks collected are not considered to be not particularly reliable. There are numerous inconsistencies in the flood marks attributed to particular flooding dates. In some instances, flood marks were provided for times when no flood is known to have occurred, highlighting a general uncertainty in the community regarding past flood events. Nevertheless, an attempt has been made to interpret the dates and timing of various flood marks provided by the community members. In many instances, this has relied heavily on considering supplementation information (such as the well documented overtopping of the causeway during the 1971 flood) and marks that represent a more tangible attempt at recording floods (e.g. a spike nailed into a power pole is considered more reliable than a statement saying “about two metres from the edge of my driveway”).

A review of the various flood marks is presented in Appendix A, while the levels and dates adopted for the various flood marks are presented in Figure 2-2.

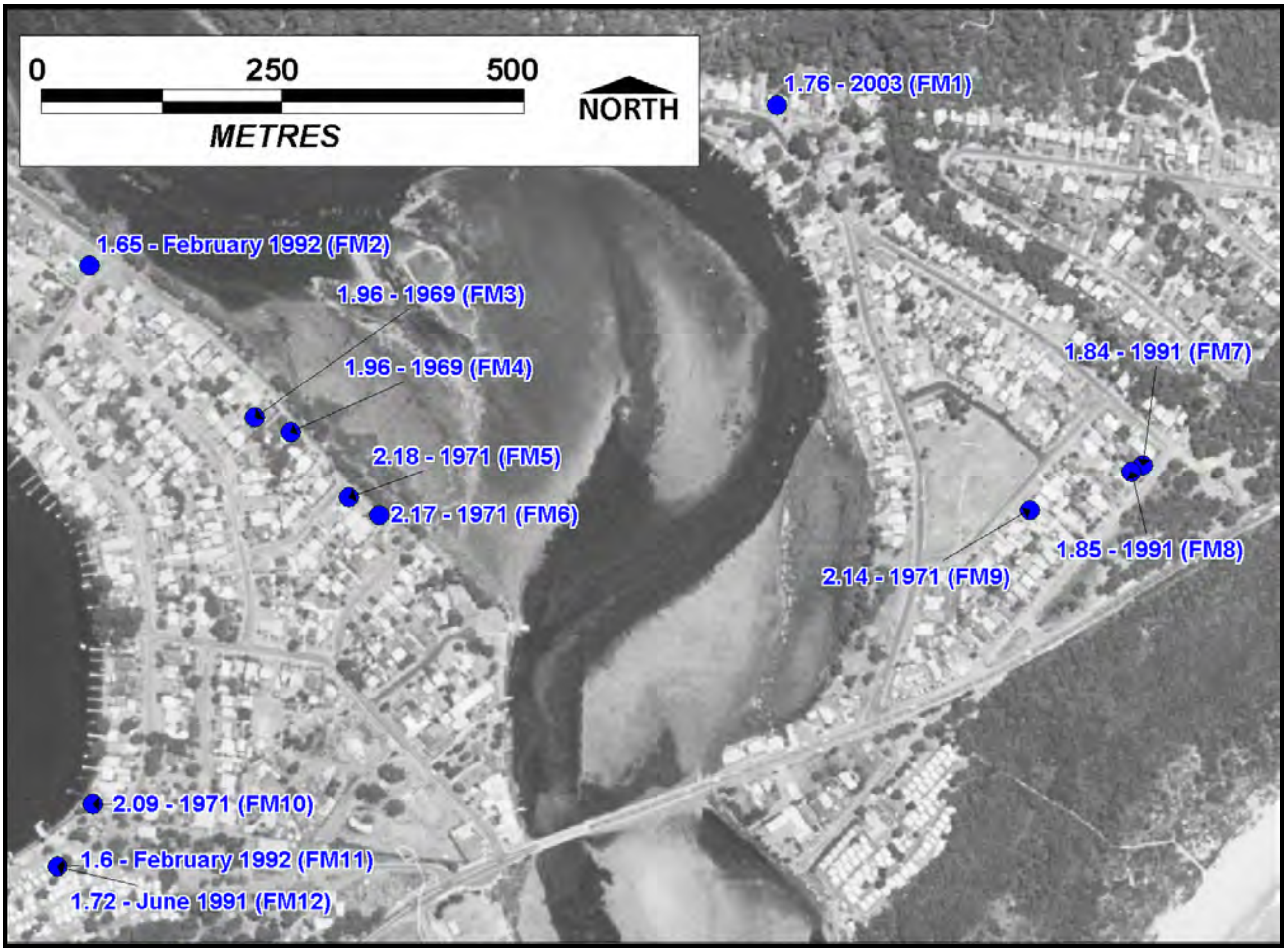


Figure 2-2 Surveyed Historical Flood Information – Burrill Inlet

2.5 Additional Survey Data

As discussed in Section 2.1, an extensive survey of the floodplain areas surrounding Burrill Inlet was undertaken during the course of this study to enable reliable computer flood models to be constructed.

The extent of the additional survey undertaken for this study is shown on Figure 2-3.

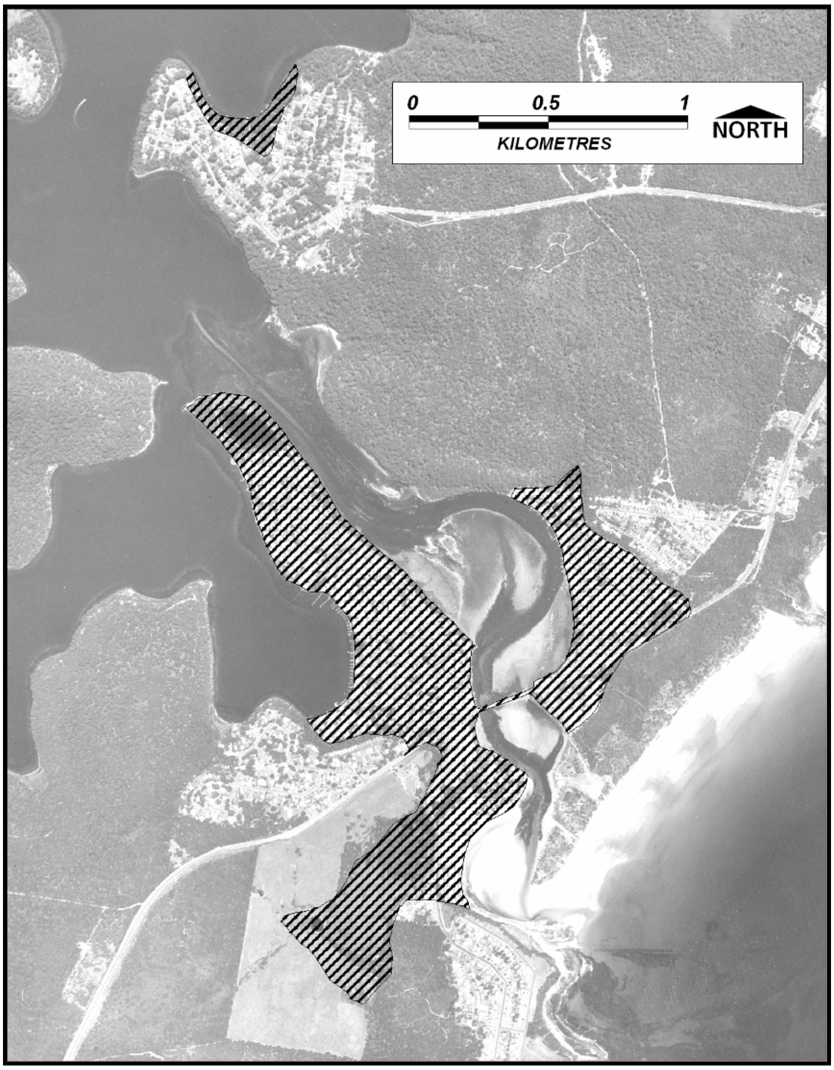


Figure 2-3 Extent of Additional Floodplain Survey Undertaken During Study

Survey within these areas comprised the following elements:

- Ground levels generally at a maximum spacing of 15m with additional detail where required;
- Foreshore embankment and structure crests and bed levels adjacent to the foreshores;
- Top of kerb levels at a maximum spacing of 10 m, with additional top of kerb levels taken at each street intersection;
- All significant changes in ground slope and/or level including high points and depressions on residential lots and roads;
- Ground levels along property boundaries where adjacent to public land;
- Ground levels along the Princes Highway causeway, including the bridge deck, centreline and embankment crests at a longitudinal spacing of no more than 10 m; and
- Bed levels and embankment crest and toe levels of creeks entering the Inlet. These included Coopers Creek, which runs into the Inlet from Burrill Lake village around 600 m to the north of the causeway and the unnamed tidal creek which flows through Lions Park.

The ground level survey developed as part of this study was augmented with additional information from the following sources to create a digital terrain model (DTM) representative of current conditions:

- data available on contour plans provided by Council, from a current aged care facility development site located at the intersection of Balmoral Drive and Princes Highway in Bungalow Park;
- Land Information Centre contours provided by Council;
- 2001 Hydrosurvey data for the whole Lake, Inlet and entrance barrier dune and berm provided by DNR;
- Bathymetric contours in the offshore zone extracted from the Tabourie 1:25,000 topographic map produced by the Central Mapping Authority of NSW; and
- Additional, less rigorous ground survey undertaken during the study within low lying areas near Wallaroy Drive (Bungalow Park) and Kings Point.

2.6 Computer Models

For the purpose of the Flood Study a number of computer models were developed, namely:

- A Digital Terrain Model (DTM);
- An Hydrologic Model;
- An Hydrodynamic Model; and
- A Geomorphic Model.

2.6.1 Digital Terrain Model

A digital terrain model (DTM), discussed further in Section 4.3, was developed to provide a topographic base surface for the hydrodynamic model. The DTM interpolates linearly between the existing topographic survey points.

The DTM was altered, as appropriate, for the calibration and design simulations undertaken as part of this study to represent the actual land surface conditions at the times of those floods.

2.6.2 Hydrologic Model

For the purpose of the Flood Study, a hydrologic model (discussed in Section 4.5) 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 slope, area 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 Burrill Lake and its Inlet.

2.6.3 Hydrodynamic Model

The hydrodynamic model (discussed in Section 4.6) developed for this study includes:

- a high two-dimensional (2D) resolution representation of Burrill Inlet and its floodplain, between Kings Point and the Tasman Sea. This area includes the low-lying residential areas of Bungalow Park, Burrill Lake and Dolphin Point. The 2D part of the model simulates hydrodynamics by performing calculations on a regular 10 m × 10 m grid; and
- a less detailed one-dimensional (1D) representation of the main Lake body, including all areas upstream of the upstream end of Burrill Inlet (adjacent to Kings Point). The 1D part of the model simulates hydrodynamics by performing calculations based on level data representing cross-sections within the waterway.

More information on the 1D and 2D models, and the linkages between them is provided in Appendix B, as well as Syme et al. (2004), Rogencamp & Syme (2003) and Rogencamp & Benham (2003).

The ground levels used in the hydrodynamic model were obtained from the Digital Terrain Model.

2.6.4 Geomorphic Model

The geomorphic model developed for the study (as discussed in 4.7), is fully integrated with the hydrodynamic model. The geomorphic model recalculates the hydrodynamic model bed levels at each time step in order to represent the erosion and deposition of sand within the entrance shoals.

2.7 Calibration and Sensitivity Testing of Models

The hydrologic and hydrodynamic models were calibrated and verified to historical flood events to establish the values of key model parameters and confirm that the models were capable of accurately predicting real flood events.

Historical events used for calibration or validation were selected using the following criteria:

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

An initial review of the available historical information highlighted three floods with sufficient data to potentially support a calibration process. These floods were from February 1971, June 1991 and February, 1992.

Further information regarding the calibration and validation events, including specific details of flood behaviour, is provided in 3.3.

The calibration and validation of the models is presented in Section 5.

A series of sensitivity tests were also carried out to evaluate the model. These tests were conducted to examine the performance and determine the relative importance of different hydrological and hydrodynamic factors. The sensitivity testing of the models is detailed in Section 6.

2.8 Establish Design Flood Conditions

The calibrated hydrologic and hydrodynamic models were modified as necessary to represent present day conditions, including topographical and landuse changes. The models were then used to define present day 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, which is sometimes referred to as the 1 in 100 year ARI flood, 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. In the case of Burrill Lake, the absence of stream flow records prevents an analysis of long-term historical records of floods at Burrill Lake. Design floods were therefore based on design rainfall estimates according to Australian Rainfall and Runoff (IEAust, 2001).

The design flood conditions are presented in Section 7.

2.9 Mapping of Design Flood Behaviour

Design flood mapping is undertaken using output from the hydrodynamic model. Maps are produced showing water level, water depth and velocity vectors 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. These maps are described and presented in Section 8.

In addition, longitudinal profiles of the maximum water levels along Burrill Lake and Burrill Inlet, from the mouth of Stony Creek to the Tasman Sea, are presented in Section 8.

3 HISTORICAL FLOOD INFORMATION

3.1 General Flood Behaviour at Burrill Lake

Flooding in Burrill Lake can be caused by one, or both, of the following mechanisms:

- Intense rainfall within the catchment, resulting in large volumes of surface runoff discharging into the Lake. The rate of inflow into the Lake can exceed the discharge to the Tasman Sea, resulting in storage of flood waters within the Lake. Increased water levels in the Lake potentially inundate floodplain areas around the foreshores.

  The maximum height of flood levels in the Lake is a function of the difference between inflow and outflow rates. Thus, if the Lake entrance is heavily shoaled or even closed, the inflow may initially exceed the outflow rates significantly. In this case, the rate of water level rise within the Lake can be relatively high. Depending on its initial level, the sand berm at the entrance to the Lake may be overtopped early during the flood, effectively scouring the entrance before the flood peak arrives.

- Severe conditions within the Tasman Sea, comprising barometric storm surge and wave setup due to a large offshore wave climate. Elevated Tasman Sea water levels push marine water into the Lake through the entrance, which can subsequently inundate low-lying properties in the vicinity of Burrill Inlet. Once again, the extent of inundation would be dependent on the entrance conditions, but in this case, a shoaled entrance would attenuate the flood ingress and minimise overbank impacts.

A variety of other mechanisms, which exacerbate flooding, were identified during the community consultation undertaken as part of this study. These include:

- Blocking of structures along main drainage channels (for example, the drainage swale that runs between Princes Highway and Princess Avenue); and
- Flooding associated with the limited hydraulic capacity of Coopers Creek and heavy rainfall within the small but responsive catchment that drains to this creek.

While these mechanisms can be the cause of localised, nuisance flooding, the most severe flood events result from intense rainfall over Burrill Lake's catchment and/or severe ocean conditions, as described above.

3.2 Recorded Information

3.2.1 Rainfall

Only one operational rain gauge station is located with the Burrill Lake catchment. The station is at Milton, along the northern edge of the catchment at an elevation of 76 m above sea level. Daily rainfalls have been recorded at this location since 1876.

Milton station was used in the hydrological analysis to estimate the major storm events that affected the catchment. However, as the station reflects rainfall at a reasonably elevated level, it is not necessarily representative of the whole catchment. To provide a more complete picture, the record from Bendalong, located on the coast at the downstream end of Lake Conjola, some 16 km to the north of Burrill Lake was also analysed. The Bendalong station has been operational since 1939.

Table 3-1 ranks the 50 highest daily rainfall events recorded at Milton. The analysis was undertaken for both 1 and 2 day periods. The results of the same analysis for the Bendalong station are shown in Table 3-2.

The records from other stations, which are close to the catchment (Brooman, Yattehyatteh and Ulladulla), were not considered suitable for the initial analysis due to their shorter length of record. The days of interest to this study (for which calibration data is available) are highlighted in Table 3-1 and Table 3-2.

From the data, it appears that a daily reading was missed at the Milton station on February 7, 1971. Data from the Bendalong gauge suggests that considerable rainfall occurred on this day, contributing to the February 1971 flood event.

Selected additional station records were obtained from within a 50km radius around Burrill Lake for the historical floods chosen for the model calibration. Of particular interest is 6-minute pluviograph records collected at Nowra RAN Air Station (managed by the Bureau of Meteorology with data available for the 1971 event), which is approximately 50 km north of Burrill Lake, and at Turpentine (managed by the Sydney Catchment Authority with data available from 1975 through to the present), which is approximately 40 km north of Burrill Lake. The collected rainfall data is presented in this report in the form of isohyetal maps, which are described further in Section 3.3.

3.2.2 Water Levels in Burrill Lake

3.2.2.1 Historical Flood Levels

A geographical database of historical flood levels based on information gathered from local residents was compiled. The number of recorded historical flood marks was the principal reason for the choice of the calibration and validation events:

- 4 historical flood levels were recorded for the February 1971 flood;
- 3 historical flood levels were recorded for the June 1991 flood;
- 2 historical flood levels were recorded for the February 1992 flood.

Following further consideration and preliminary modelling of the June 1991 event, it was considered that two of the historical flood marks for this event, namely marks FM8 and FM7 (refer Figure 2-2) reflected the effects of a local drainage issue, which was not replicated by the model. Respondents to the community questionnaire have indicated that this area is affected by local runoff from the areas to the north west (Canberra Crescent and Braidwood Avenue) and blockages in the local stormwater drainage system. Given this fact, known flood level data for the June 1991 event was limited to one mark on the periphery of the main Lake body (FM12). This event was subsequently not considered suitable for calibration or validation of the model. Based on available data and flood magnitudes, the February 1971 event was adopted for model calibration purposes, while the February 1992 event was adopted for model validation.

Table 3-1 Most Severe Daily Rainfall Events Recorded at Milton Station

Rank	Date	Rainfall over 24hrs (mm)	Rank	Date	Rainfall over 48hrs (mm)
1	30 Oct, 1959	311.2	1	19 Apr, 1927	435.4
2	18 Apr, 1927	289.1	2	24 Feb, 1977	403
3	12 Jun, 1991	268	3	27 Feb, 1919	392
4	13 Jan, 1911	264.4	4	12 Jun, 1991	376
5	10 Jun, 1915	249.4	5	06 Oct, 1916	375.4
6	26 Feb, 1919	245.4	6	30 Oct, 1959	351.8
7	05 Oct, 1916	231.1	7	26 Sep, 1951	348
8	11 Mar, 1975	229.2	8	13 Jan, 1911	328.7
9	06 Feb, 1971	224	9	14 Jan, 1911	317
10	25 Sep, 1951	223.5	10	05 Oct, 1916	312.4
11	24 Feb, 1977	216	11	03 Nov, 1959	311.2
12	08 Feb, 1971	210.8	12	26 Feb, 1919	305.1
13	19 Jan, 1950	193	13	11 May, 1925	294.6
14	27 Feb, 1909	191	14	18 Apr, 1927	289.1
15	16 Mar, 1936	190.5	15	12 Mar, 1975	280.4
16	19 Oct, 1965	188.2	16	13 Jun, 1991	278.2
17	23 Feb, 1977	187	17	24 Jul, 1918	273.8
18	11 May, 1925	185.4	18	06 Feb, 1971	267.4
19	12 Mar, 1890	185.4	19	11 Feb, 1992	266.8
20	16 Apr, 1969	183.4	20	12 Mar, 1890	261.6
21	13 Dec, 1910	179.3	21	10 Jun, 1915	260.8
22	25 May, 1950	169.9	22	16 Mar, 1936	259.1
23	02 Jun, 1930	167.6	23	11 Jun, 1915	253
24	16 May, 1883	165.1	24	06 Apr, 1882	248.5
25	04 Apr, 1950	162.6	25	19 Oct, 1881	246.2
26	26 Jun, 1928	162.6	26	25 Feb, 1977	236
27	24 Jul, 1918	160.8	27	09 Jun, 1991	230
28	16 Apr, 1907	160	28	11 Jul, 1904	229.8
29	05 May, 1953	158.8	29	11 Mar, 1975	229.2
30	10 Feb, 1992	157.2	30	27 May, 1925	227.3
31	06 Feb, 1878	155.4	31	19 Jan, 1950	227.3
32	01 Dec, 1961	149.9	32	07 Feb, 1971	224
33	25 Mar, 1890	147.8	33	25 Sep, 1951	223.5
34	09 Jun, 1991	147	34	16 Apr, 1969	216.9
35	27 Feb, 1919	146.6	35	17 Dec, 1888	216.2
36	19 Apr, 1927	146.3	36	13 Dec, 1910	213.6
37	06 Oct, 1916	144.3	37	09 Feb, 1971	212.6
38	16 Dec, 1888	141.5	38	16 Feb, 1929	210.9
39	19 Oct, 1881	140	39	08 Feb, 1971	210.8
40	12 Mar, 1974	139.8	40	07 Feb, 1878	209.2
41	01 Sep, 1996	137	41	20 Jan, 1950	209
42	06 Apr, 1882	136.7	42	12 Mar, 1958	208.3
43	12 Aug, 1929	135.6	43	16 May, 1883	206.2
44	19 Jun, 1983	135	44	11 Apr, 1974	204.6
45	10 Apr, 1974	134.6	45	05 Jun, 1899	204.4
46	04 May, 1917	133.4	46	20 Oct, 1965	197.9
47	15 Mar, 1989	133	47	23 Feb, 1977	196
48	09 Dec, 1970	129.5	48	27 Feb, 1909	194
49	25 Mar, 1926	129.5	49	01 Jul, 1976	193.8
50	20 Jun, 1984	128	50	17 Apr, 1907	193

Table 3-2 Most Severe Daily Rainfall Events Recorded at Bendalong Station

Rank	Date	Rainfall over 24hrs (mm)	Rank	Date	Rainfall over 48hrs (mm)
1	6-Feb-71	365.8	1	7-Feb-71	529.6
2	19-Jan-50	254	2	30-Oct-59	422.4
3	12-Jun-64	254	3	6-Feb-71	403.1
4	12-Jun-91	241.8	4	12-Jun-91	346
5	19-May-63	241.3	5	11-Mar-75	318.6
6	30-Oct-59	240.8	6	19-Jan-50	288.3
7	21-Apr-64	228.6	7	29-Apr-63	270
8	8-Jun-64	228.6	8	20-Jan-50	254
9	20-Nov-61	226.1	9	13-Jun-64	254
10	29-Apr-63	207.5	10	3-Mar-97	250.6
11	3-Mar-97	203.6	11	31-Oct-59	249.7
12	29-Oct-59	181.6	12	26-Mar-61	245.2
13	16-Apr-69	170.2	13	13-Jun-91	242.4
14	10-Mar-75	166.2	14	20-May-63	241.3
15	24-Mar-52	165.9	15	24-Feb-77	239
16	24-Jan-55	164.6	16	26-Sep-51	233.7
17	7-Feb-71	163.8	17	22-Oct-59	219.7
18	17-Sep-62	160.5	18	9-Apr-45	215.9
19	15-Jun-52	158.8	19	16-Jun-52	214.7
20	25-Mar-61	158.8	20	16-Apr-69	207.8
21	23-Feb-77	155.6	21	4-Mar-97	207.6
22	4-Aug-90	154.6	22	30-Apr-63	207.5
23	11-Mar-75	152.4	23	19-Aug-98	207.2
24	30-Jun-58	149.9	24	19-Feb-84	204.8
25	15-Oct-76	146.3	25	11-Feb-92	200.2
26	9-Apr-45	144.8	26	5-May-53	198.9
27	22-Nov-61	143.5	27	12-Mar-75	198.6
28	8-Dec-63	142.2	28	9-Jun-91	195.6
29	4-Mar-79	138	29	24-Jan-55	190
30	22-Feb-54	136.9	30	1-Jul-76	185.5
31	15-Mar-89	136.2	31	4-Aug-90	182
32	21-Oct-59	134.6	32	15-Apr-52	181.6
33	9-Jun-91	130.2	33	29-Oct-59	181.6
34	19-Aug-98	129	34	10-Mar-75	174.1
35	26-Sep-51	128.3	35	10-Dec-70	173.7
36	9-Dec-70	128	36	4-May-48	171.4
37	19-Feb-84	122.4	37	17-Apr-69	170.2
38	12-Mar-74	119	38	15-Oct-76	168.9
39	16-Jul-69	118.9	39	8-Feb-71	168.4
40	15-May-77	117.4	40	11-Jul-57	166.7
41	3-May-48	116.8	41	25-Jan-55	166.4
42	14-May-62	115.6	42	24-Mar-52	165.9
43	10-Feb-92	114.6	43	25-Mar-52	165.9
44	1-Aug-90	112	44	4-Feb-90	164.6
45	20-Feb-74	110	45	27-Mar-76	164
46	7-May-84	110	46	23-Feb-77	160.9
47	16-Jun-78	107	47	18-Sep-62	160.5
48	10-Jul-57	105.7	48	15-Jun-52	158.8
49	25-Sep-51	105.4	49	25-Mar-61	158.8
50	11-Jun-91	104.2	50	22-Feb-54	158.5

Three further flood marks were collected for other flood events, including one for an event in 2003, which was not supported by continuous water level monitoring carried out since 1991 (refer Section 3.2.2.2). The other two flood marks were obtained for events which 'probably occurred around the late 1960's and early 1970's'. As these recorded flood levels were considerably lower than flood marks recorded for the 1971 event, it is unlikely that they correspond to the 1971 event, or were observed at a time that did not coincide with the flood peak. It is possible that these points correspond to a smaller flood event in April 1969, although initial analyses undertaken of that event indicated that the rainfall was not intense enough to have resulted in such high water levels, unless the entrance had been very shoaled at that time. This seems unlikely given that the entrance was reportedly opened mechanically during 1968 (Shoalhaven City Council, 2002).

All recorded flood levels were located in the residential areas adjacent to Burrill Inlet and upstream of the causeway.

The quality of the flood marks varied significantly. For example, some flood marks were indicated as precise positions on a brick wall with an exact date, while others were merely an approximate location on the ground with no clear date provided. The recorded historical flood marks were weighted depending on their description, and their usefulness was judged accordingly. Comments on the recorded flood marks are presented in Appendix A.

3.2.2.2 Water Level Gauge

An automatic water level gauge is located upstream of the causeway bridge. The gauge has been operating since November 1991. The gauge records water levels every 15 minutes, which captures the tidal influence within the Lake.

Measured tidal range within Burrill Lake is shown in Figure 3-1 and Figure 3-2. The tidal ranges charted on these figures represent a 14 day moving average of tidal ranges, to account for the fortnightly variation of the spring-neap tidal cycle. From these charts, it can be seen that, during late 1991 through to mid 1993, tidal ranges were relatively high, due to the scouring effect of the 1991 and 1992 floods on the entrance shoals. During mid 1993, however, the entrance became significantly constrained, probably as a result of a major coastal storm (WBM Oceanics Australia, 2001). Between the end of 1993 and May 1997, the entrance was mostly unconstrained allowing effective tidal exchange between the Lake and the Tasman Sea, until another significant coastal event caused shoaling at the entrance. This was most likely a result of the 'Mother's Day' storm of 1997, which is the most significant coastal storm of the past decade.

A second coastal storm in February 1998 also caused significant entrance shoaling, however, tidal flows were sufficient to restore normal tidal range in the Lake (0.3 to 0.5 m) within the following month or two. From early 2002 onwards, the tidal range in Burrill Lake has been small, reflecting the period of drought experienced on the south coast of NSW. Similar hydrodynamic behaviour has also been observed at other south coast lakes, such as the Tuross estuary and Wonboyn Lake.

The degree to which the entrance is constrained or 'choked' with sand clearly has a significant impact on tidal hydrodynamics within the Lake.

3.2.3 Discharge

There are no stream gauges in the Burrill Lake catchment and hence measured flow information is not available.

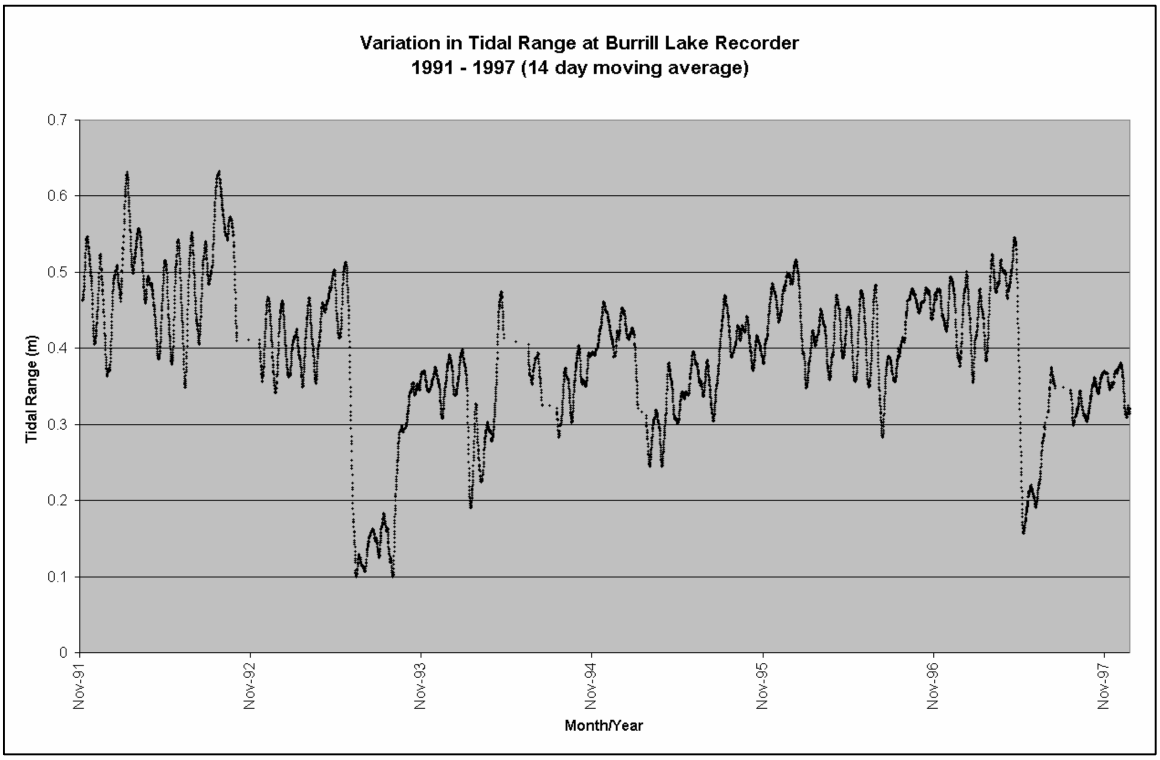


Figure 3-1 Tidal Range at Burrill Lake Causeway (1991-1997)

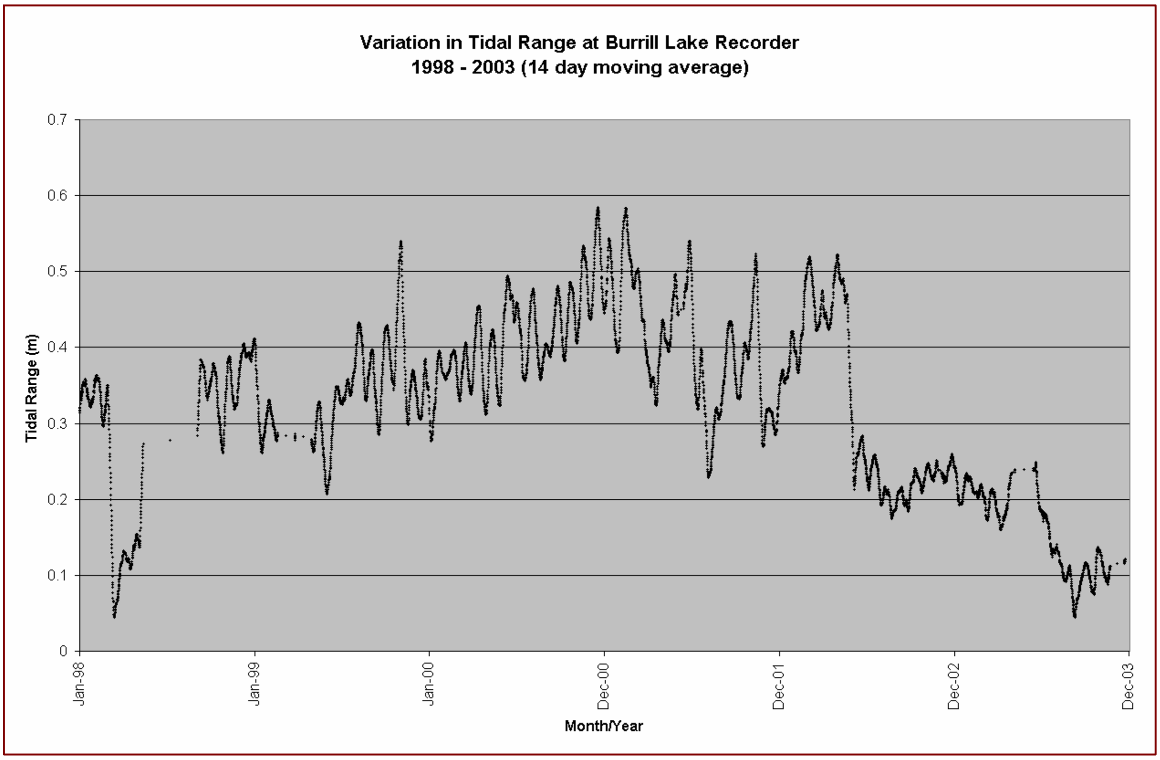


Figure 3-2 Tidal Range in Burrill Lake (1998-2003)

3.2.4 Tasman Sea Levels

3.2.4.1 Introduction

Burrill Lake discharges directly in the Tasman Sea. The Tasman Sea levels can, under specific circumstances, influence the flood levels within the Lake. Data was collected to assess the Tasman Sea water level conditions at the time of the calibration floods.

Coastal water levels are influenced by a variety of interacting factors that raise the water levels above the normal tide levels. Low atmospheric pressure, storms, strong onshore winds and large offshore waves, can all contribute to elevated water levels both within the Tasman Sea and inside Burrill Inlet.

3.2.4.2 Tide Levels

In this report the tide levels are referred to as being water levels in the Tasman Sea resulting from astronomical tide plus barometric setup. There are no tide level recorders located in the Tasman Sea adjacent to Burrill Lake. The nearest tide recorder that is representative of water levels in the Tasman Sea is located in Jervis Bay, which is approximately 35 km north of Burrill Lake, and has been recording hourly tide levels since July 1995.

Tide records in Sydney Harbour, which reflect water levels at Sydney and date back more than 100 years, have also been collected.

A statistical analysis between Jervis Bay and Sydney tide records (from 1995 onwards) was undertaken, and shows that:

- There is a 98.9% correlation between the two data sets;
- There is no time offset on tidal peaks between the two data sets;
- Jervis Bay records are 0.15m higher on average than Sydney Harbour; and
- There is a 0.06m standard deviation on the data set differences.

The high level of correlation between the two data sets supports the use of the historical Sydney Harbour records for the Burrill Lake calibration and validation events (given that these events pre-date the Jervis Bay data record). A 0.15m increase has been applied to the Sydney data to account for the geographical position.

The translated Sydney data represents the combination of astronomical tide with the barometric anomalies that accompany weather patterns.

Wind setup of coastal waters is considered marginal and with very limited impacts on Burrill Lake flood processes. It is therefore not accounted for in the historical ocean levels.

3.2.4.3 Wave Height

The Department of Natural Resources (DNR) owns a number of wave rider buoys along the coast of New South Wales, which record offshore wave heights. The analysis of offshore wave height is necessary in the determination of the wave setup component of ocean levels.

Unfortunately, no offshore wave data is available for the 1971 flood event, however, offshore wave data were obtained for February, 1992 from the Batemans Bay waverider buoy. Furthermore, initial analysis was undertaken for a flood event in June 1991 (eventually discarded due to a lack of reliable flood marks, refer Section 3.2.2.1). The data shows that a moderate coastal storm was recorded by the waverider buoy during these periods.

The wave setup is regarded as the largest contributor to lifting still water levels above normal tide levels during coastal storms. The wave setup is calculated as being 10 to 15% of the breaking wave height. The breaking wave height H_b is derived from offshore wave rider measurements (H_0, offshore wave height, T, offshore wave period) as follows:

$$H_b = \left(0.7\frac{H_0 T}{\sqrt{g}}\right)^{0.67}$$

The maximum calculated wave setup for the February 1992 storm event was estimated to be 0.74 m, corresponding to offshore wave heights of 4.9 m with a period of 11.1 seconds (ie. typical coastal storm conditions). Similarly, the maximum wave setup for the June 1991 event was estimated to be 0.62 m corresponding to an offshore wave height of 4.1 m with a period of 10.2 seconds. This estimate is considered to be conservative due to the methodology used. A representative wave setup during both storms was approximately 0.5 m.

Accordingly, in the absence of more reliable data, a constant 0.5 m wave setup was used for the 1971 calibration event.

The way in which wave setup was incorporated during the calibration and validation events is presented in Sections 3.3.2.3 and 0 for 1971 and 1992, respectively.

3.3 Description of Historical Floods used for Calibration

3.3.1 Available Data

Daily rainfall data for the calibration events was obtained from the Bureau of Meteorology for the stations shown in Figure 3-3. The data from these stations, in conjunction with pluviometer recordings at Nowra and Turpentine, have been used to derive the required rainfall patterns for input to the hydrologic model.

The Sassafras and Nerriga pluviometer stations were not considered appropriate for this study as their locations are well inland and elevated on the Great Dividing Range. Orographic rainfall effects make the records from these stations markedly different from those located closer to the coast.

It should be noted that the Nowra RAN Air Station and Turpentine pluviometer stations are a substantial distance away from the Burrill Lake catchment. Accordingly, the applicability of data from these stations, including rainfall volumes and temporal patterns, to the Burrill Lake catchment can be questioned. Nevertheless, as the records at these stations represent the best available data on actual temporal rainfall patterns in areas close to the Burrill Lake catchment for the calibration and validation events considered during this study, they have been adopted for this study.

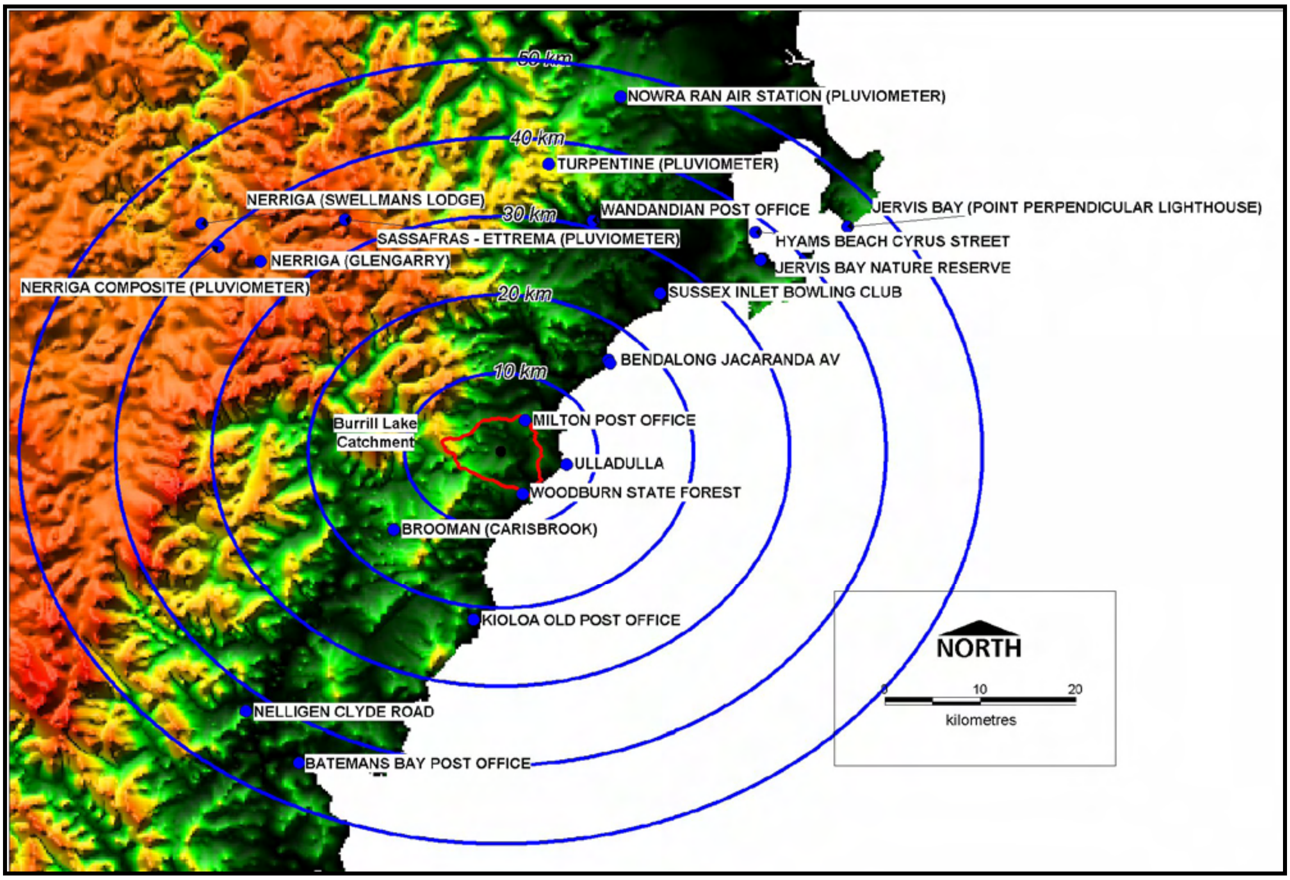


Figure 3-3 Rainfall Stations Examined for Calibration Events

The dates for which rainfall records are available at the above stations are shown in Table 3-3.

3.3.2 The February 1971 Flood

3.3.2.1 Rainfall Spatial Distribution

The rain gauges located around the Burrill Lake catchment recorded rainfall on almost every day during January 1971. The rainfall intensity towards the end of January steadily increased and decreased over a one week period before an intense rainfall event between the 4th and the 11th February. The peak rainfall during this event was on 5 - 6 February.

It is noted that daily rain gauge records at both Ulladulla and Milton were not obtained on February 7. In comparing records for the Woodburn State Forest gauge, which is immediately to the south of the catchment, it is likely that the rainfall on February 7 was reported as part of the February 8 record (see Figure 3-4).

Table 3-3 Availability of Rainfall Data

Station	Dates	Elevation Above Sea Level
-	-	-
Pluviograph stations within 50km of Burrill Lake		
Sassafras (Ettrema)	(Jan 1963 to Aug 1983)	760m
Nerriga Composite	(Feb 1971 to Jun 2004)	630m
Nowra Ran Air Station	(Aug 1964 to Aug 1986)	109m
Turpentine (Sca Site)	(1975 to Present)	220m
Daily rainfall stations within 50km of Burrill Lake		
Woodburn State Forest	(Feb 1926 to Sep 1980)	15m
Ulladulla	(Jan 1937 to Jun 1974)	9.1m
Ulladulla Aws	(Jun 1994 to Oct 2004)	35.7m
Milton Post Office	(Sep 1876 to Oct 2004)	76m
Yatteyattah (Pointer Road)	(Sep 1876 to Oct 2004)	78m
Brooman (Carisbrook)	(Aug 1979 to Aug 2004)	120m
Bendalong Jacaranda Av	(Jan 1971 to Apr 1989)	32m
Bendalong (Jacaranda Av)	(May 1939 to Sep 2004)	32m
Kioloa Old Post Office	(Jun 1957 to Aug 2004)	30m
Kioloa (London Foundation)	(Jun 1980 to Oct 1986)	15m
Brooman (Geju)	(Mar 1974 to Aug 1974)	61m
Sussex Inlet Bowling Club	(Jan 1952 to Sep 2004)	6m
Currowan (Wild Pig Rd)	(Feb 1993 to Sep 2004)	35m
Wandandian Post Office	(Apr 1985 to Jun 2003)	10m
Sanctuary Point (Salinas Street)	(Apr 2000 to Sep 2004)	9m
Jervis Bay Nature Reserve	(Mar 1958 to May 1993)	40m
Hymas Beach Cyrus Street	(Jan 1960 to Nov 1974)	15m
Sassafras (Ettrema)	(Jan 1954 to Nov 1972)	760m
Nerriga (Glengarry)	(Aug 1969 to Oct 1973)	564m
Nelligen (Thule Road)	(Oct 1898 to Sep 2004)	5m
Nelligen Clyde Road	(Jan 1967 to Oct 1971)	18m
Batemans Bay Post Office	(Nov 1895 to Feb 1996)	3m
Nerriga Composite	(Apr 1898 to Oct 2004)	630m
Batemans Bay (Catalina Country Club)	(Jan 1985 to Oct 2004)	11m
Jervis Bay (Point Perpendicular Lighthouse)	(Jul 1899 to Jun 2004)	85m
Jervis Bay (Pt Perpendicular Aws)	(May 2001 to Oct 2004)	85m
Nerriga (Swellmans Lodge)	(Aug 1969 to Jan 2004)	573m
Nowra Ran Air Station Aws	(Dec 2000 to Oct 2004)	109m
Nowra Ran Air Station	(Sep 1942 to Nov 2000)	109m
Hillview (Shoalhaven River)	(Dec 2000 to Oct 2004)	550m
Callala Bay (Donovan Close)	(May 2003 to Sep 2004)	10m

Table 3-4 Daily Rainfall (mm) Near Burrill Lake, February 1971.

Date of Reading	Milton	Ulladulla	Woodburn State Forest	Nowra RAN Station
6 February	224.0	228.6	146.1	190.2
7 February	0.0	0.0	288.3	55.0
8 February	210.8	176.5	3.8	1.5

The isohyet contours of the 48-hour rainfall records to 9am on 7 February 1971 are presented in Figure 3-6. In deriving this figure, rainfall measured to 9am on 8 February at Milton and Ulladulla was also used, as it was assumed that most of the recorded rainfall would have occurred prior to 9 am on 7 February.

Overall, it is considered that the total rainfall for both Woodburn State Forest and Milton are most representative of the rainfall within the catchment (i.e. between 430 and 440 mm over the two days). There may have been around 5% less rainfall closer to the coast, as shown at the Ulladulla gauge, but this difference is not considered significant.

3.3.2.2 Temporal Rainfall Pattern

The temporal rainfall pattern from Nowra RAN Station for the period of the February 1971 storm is provided as Figure 3-4.

3.3.2.3 Ocean Water Levels (Including Wave Effects)

The nature of available ocean water level data and the way it has been used to derive ocean water levels at the study site is discussed in Section 3.2.4.

The estimated ocean water level variation present during the February 1971 event and its components are shown on Figure 3-5.

3.3.2.4 Flood Description

The 1971 flood event was the largest flood on record for Burrill Lake. The flood resulted from the occurrence of lengthy and intense rainfall over the catchment (700mm in one week, including over 430 mm on days 5 and 6 of the event at Milton). During the event, it is noted that the Lake entrance was highly constrained (WBM, 2001), which was supported by several resident questionnaire responses.

Reports indicate that the flood was characterised by slow rising waters that eventually overtopped the bridge and causeway. The main causes for the flooding have been reported as heavy rain, high tides, big seas and a closed lake entrance.

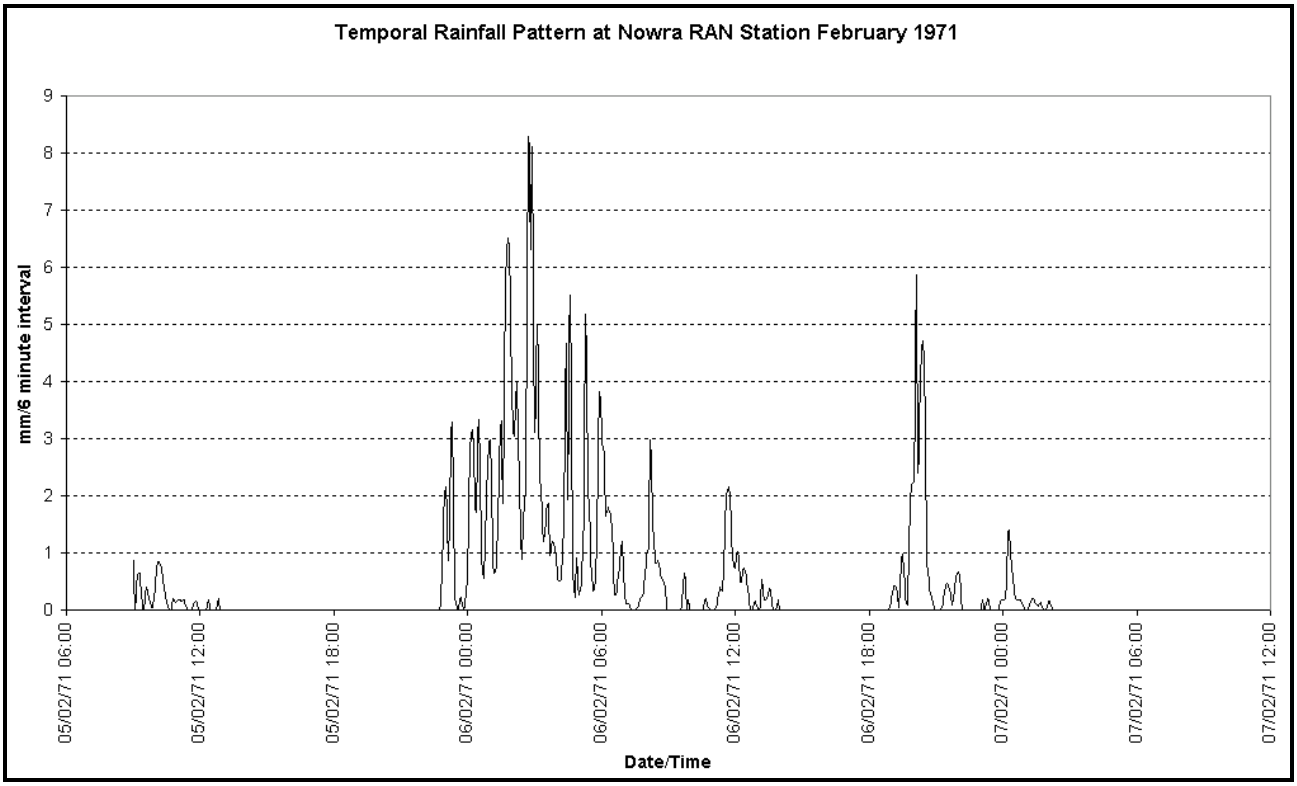


Figure 3-4 Temporal Rainfall Pattern for February 1971 Event

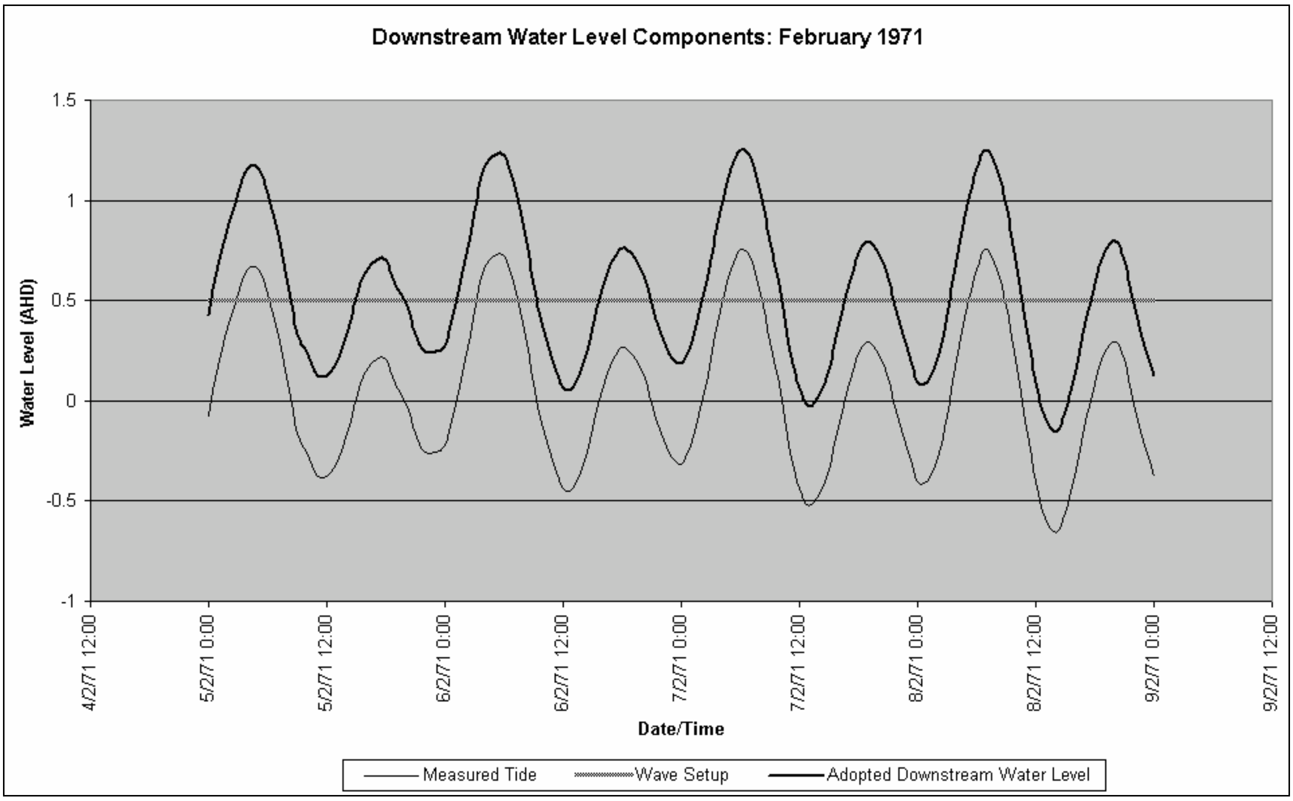


Figure 3-5 Estimated Ocean Water Levels for February 1971 Flood

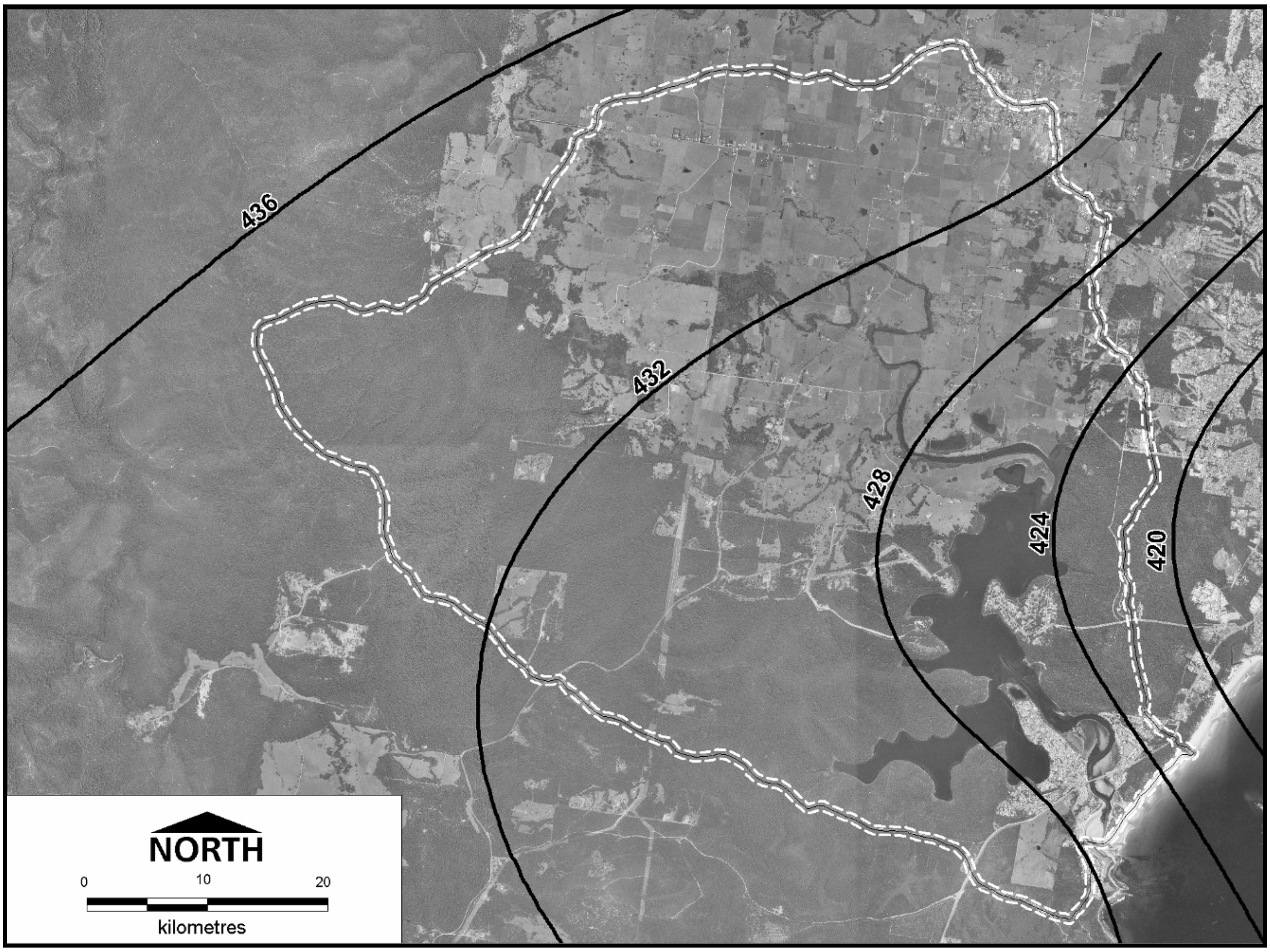


Figure 3-6 Isohyetal Map: 48 hours to 9am 7 February 1971 (values in mm)

There is evidence (both eyewitness and via inspection of aerial photography), that a second entrance was scoured across the beach berm during the 1971 event at a location around 500 m to the north of the main entrance. Photographs of the flood passing over the top of the causeway indicate that the causeway presented a major constriction to flood flows from Burrill Lake, with flood waters backing up behind this structure.

3.3.3 The February 1992 Flood

3.3.3.1 Rainfall Spatial Distribution

The rain gauges located around the Burrill Lake catchment recorded a small rainfall event (<40 mm) on February 6. This was followed by a few days of relatively dry weather until the 24 hours to 9:00 am on February 10 when rain began to fall heavily. Rain continued over a two day period. Rainfall was most intense at Brooman (15 km south of the catchment) and Milton (along the northern edge of the catchment), where 344 and 267 mm of rainfall was recorded over this period respectively. The next closest stations (Bendalong and Kioloa) recorded rainfalls of less than 200 mm over this period, indicating that the severest rainfall was actually experienced in the vicinity of the Burrill Lake catchment.

The closest available pluviometer record, from Turpentine, recorded a total of 263 mm over the two days. Unlike the stations at Milton and Brooman, however, a significantly larger proportion of the rainfall fell during the 24 hours leading to 9:00 am on 10 February. Closer examination of the rainfall record from Turpentine shows that between 6:00 am and 12:00 pm on February 10, close to 100 mm of rain fell, with this period from the record containing the most intense rainfall during the two day period. Considering that the rainfall reported for 10 February in Table 3-5 includes rain falling between 9:00 on February 9 through to 9:00 on February 10, it is clear that the daily totals calculated at Turpentine are still consistent with those at the daily stations. The rainfall data is consistent with a rainfall peak that occurred during mid to late morning on February 10, with the peak rainfall burst tracking southwards and intensifying with distance south.

The daily total rainfalls for Milton, Brooman and Turpentine are shown in Table 3-5.

Table 3-5 Daily Rainfall (mm) near Burrill Lake, February 1992.

Date of Reading	Milton	Brooman	Turpentine
10 February	157.2	156	217.5
11 February	109.6	188	45

The isohyet contours of the 48-hour rainfall records to 9am 11 February 1992 are presented in Figure 3-10.

3.3.3.2 Temporal Rainfall Pattern

The temporal rainfall pattern from Turpentine Station for the period of the February 1992 storm is provided as Figure 3-7.

3.3.3.3 Ocean Water Levels (Including Wave Effects)

The nature of available ocean water level data and the way it has been used to derive ocean water levels at the study site is discussed in Section 3.2.4.

The estimated ocean water level variation present during the February 1992 event and its various components are shown on Figure 3-8. Furthermore, the water level recorded inside the Lake, at the automatic recorder located upstream of the causeway, is provided for comparison. A chart showing the water levels upstream of the causeway for a longer period, including a number of days either side of the flood event, is provided as Figure 3-9.

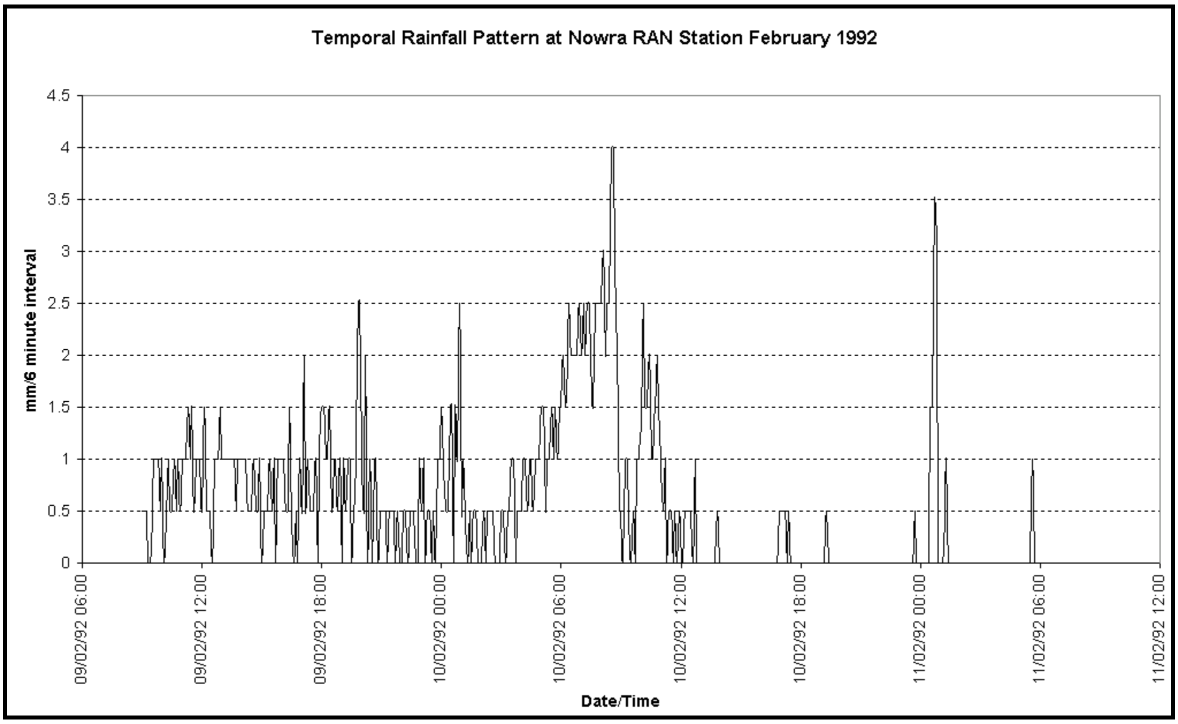


Figure 3-7 Temporal Rainfall Pattern for February 1992 Event

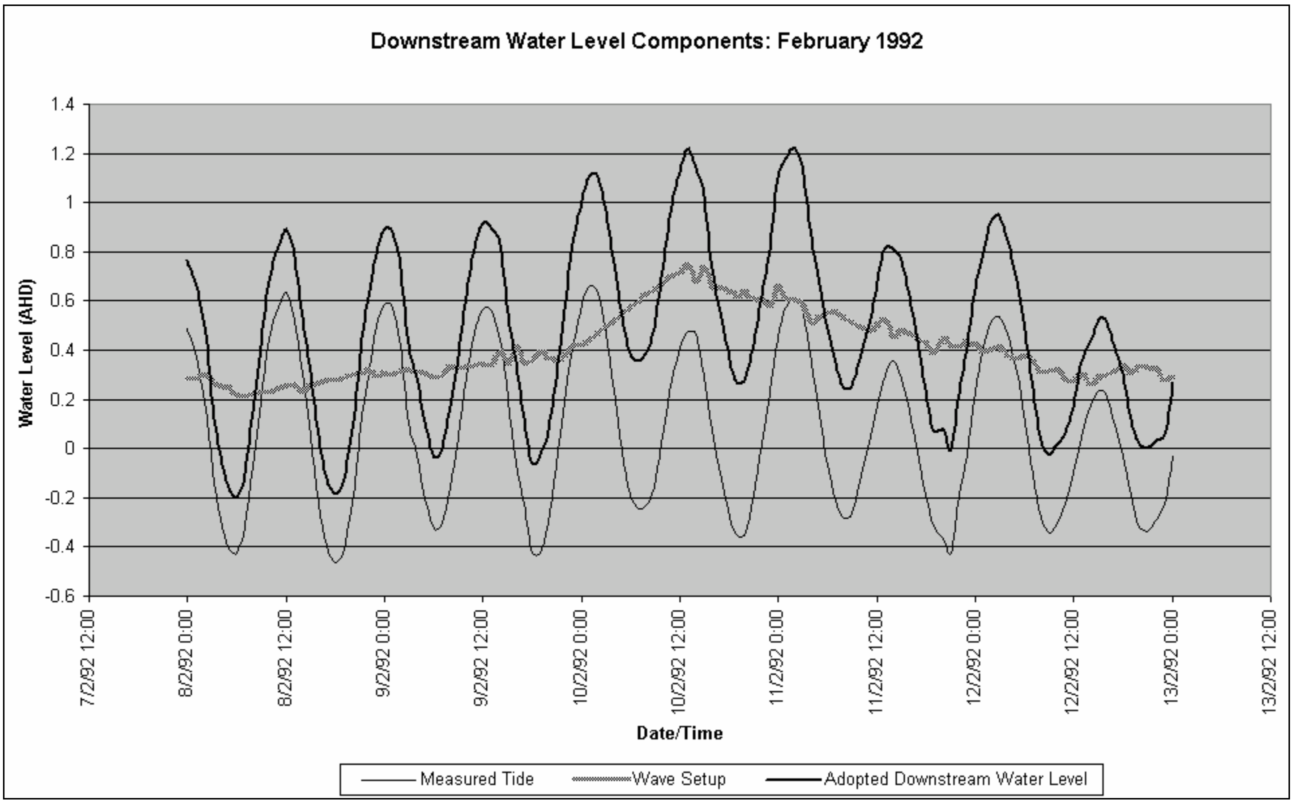


Figure 3-8 Estimated Ocean Water Levels for February 1992 Flood

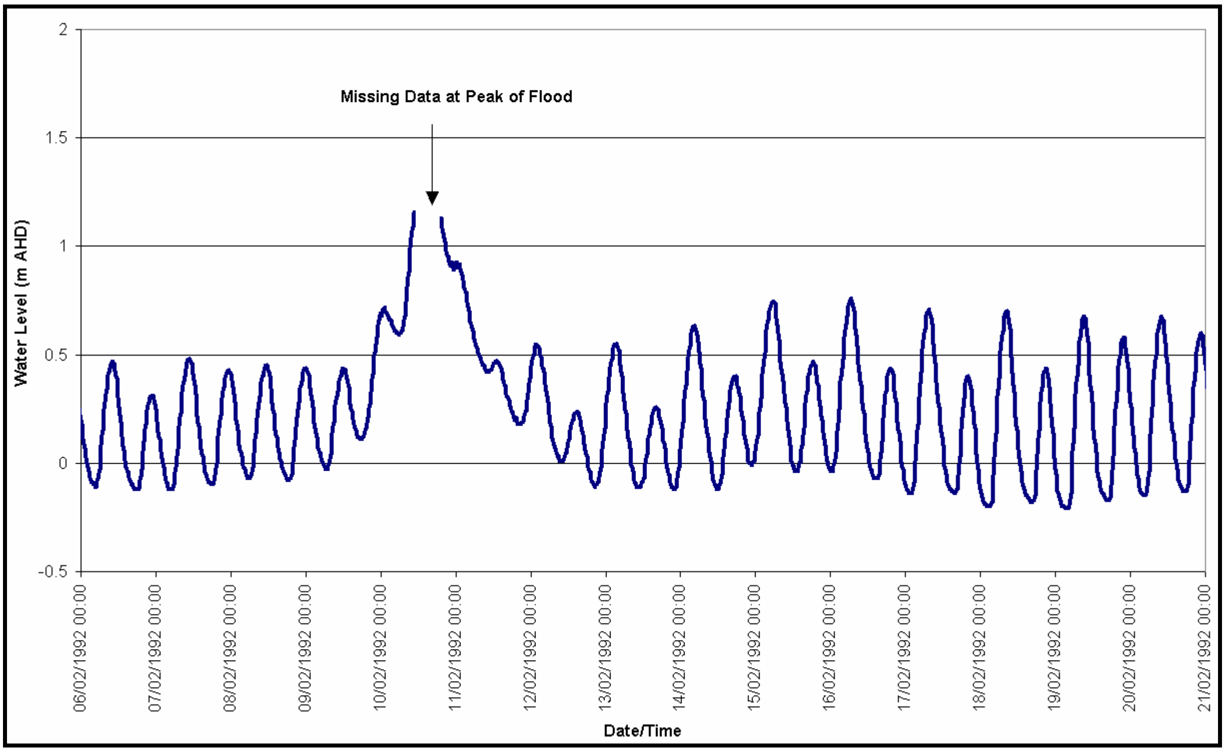


Figure 3-9 Burrill Lake Water Level Records for February 1992

3.3.3.4 Flood Description

As for other events, information relating to flood behaviour for the 1992 flood is scarce. Certainly, on the strength of available anecdotal information, it would seem that the effects of the flood were less severe than the June 1991 event. A number of respondents to the flood questionnaire indicated that their yards were flooded.

The flood was unique in that the water level recorder that exists upstream of the causeway at Burrill Lake was operational during this flood. Unfortunately, this recorder failed to record the peak of the flood (see Figure 3-9). Although the peak is missed, it is obvious that a flood occurred. With reference back to the historical marks shown on Figure 2-2, it is considered likely that the peak of the flood was in the vicinity of 1.55 to 1.70 m AHD and occurred at approximately 3:00 pm on February 10. A review of the predicted astronomical tides for February 10 indicates that a peak water level of 0.2 m AHD occurring at 1:15 pm. The tide levels and catchment floods would have acted together in causing this flood, however, from Figure 3-8, it is evident that the contribution of wave setup was particularly important for this event.

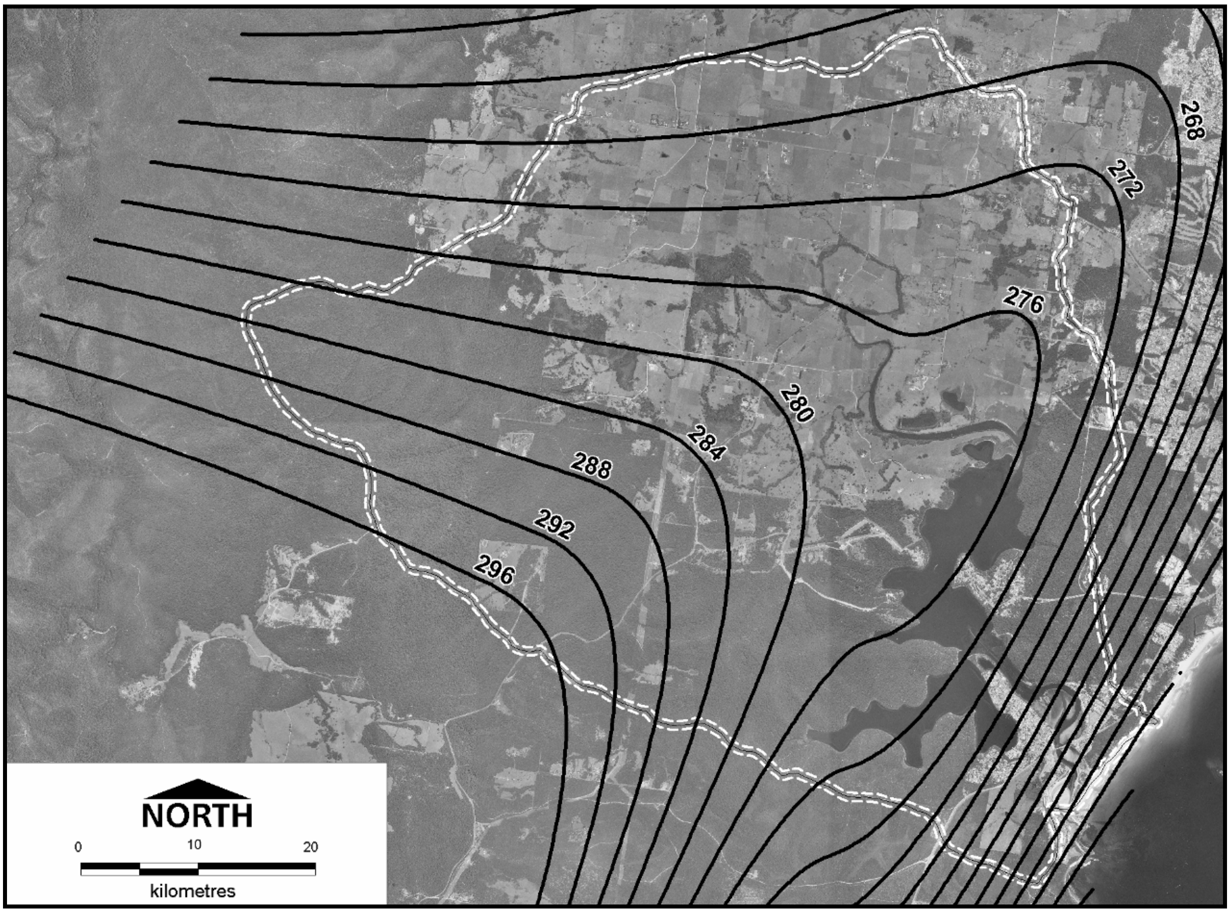


Figure 3-10 Isohyetal Map: 48 hours to 9am 11 February 1992 (values in mm)

3.4 Effect of the Rock Shelf 'Notch'

The importance of a 'notch' (in reality a series of crevasses) in the rock shelf at the entrance to Burrill Lake in suppressing peak water levels has been raised by the community.

The effect of the crevasses would be minimal during large flood events, such as the adopted calibration and validation events, as the elevated flood waters tend to scour a much larger channel across entrance shoals. The main effect of the crevasses is the continual draining during times of less significant catchment inflows. The overall effect would be to keep the initial water levels at the start of the flood lower by allowing water to continually discharge from the Lake at a pre-determined maximum level. This effect is highlighted in the water level records that are available within the Lake and has therefore been incorporated into the study, as appropriate.

The modelling undertaken for the study does not specifically include discharge through the crevasses. Incorporation of such detail in the model is not considered to be justified, and would introduce additional complexity and extended run time.

4 COMPUTER MODELS

4.1 Introduction

Computer models can be accurate, cost-effective and efficient tools to model the flood behaviour of a catchment. For this study, four types of models were used:

- A coarse digital terrain model (DTM) of the whole catchment, and a more detailed DTM of Burrill Inlet and its surrounds. Both DTM's comprised square celled rectilinear grids based on linear interpolation between points of known height;
- An hydrologic model, covering all the sub-catchments draining to Burrill Lake;
- A two-dimensional (2D) hydrodynamic model extending from the main body of the Lake to the ocean, with one-dimensional (1D) storage-based elements representing the northern and southern basins of the Lake; and
- A geomorphic model of the entrance shoals.

The **digital terrain models** interpolate the lake, floodplain and catchment bed and ground levels between the existing topographic survey points.

The **hydrologic model** simulates the catchment rainfall-runoff processes, producing the catchment inflows that are used in the hydrodynamic model.

The **hydrodynamic model** simulates the flow behaviour of the waterway and associated floodplains, producing flood levels, discharges and velocities.

The **geomorphic model** updates the hydrodynamic model bed geometry depending on sand erosion and deposition.

Information on the topography and characteristics of the catchment, creeks, lake, entrance channel and floodplains were built into the models. For each historic flood, data on rainfall, flood levels and water levels in the Tasman Sea were also used to simulate and validate (calibrate and verify) the models. The models produce as output, flood levels, flows (discharges), velocities (current speed and direction), hydraulic categories and provisional hazards.

Development of a computer model for a flood study follows a relatively standard procedure as shown below:

1 Discretisation of the catchment, creeks, waterway, floodplain, etc (see Section 4.4).

2 Incorporation of physical characteristics (catchment areas, cross-sections, etc).

3 Setting up of hydrographic databases (rainfall, water levels in the Tasman Sea, flood levels) for historical events.

4 Calibration to one or more historical floods (calibration is the adjustment of model parameters within acceptable limits to reach agreement between modelled and measured values).

5 Verification to one or more other historical floods (verification is a check on the model's performance without adjustment of parameters).

6 Sensitivity analysis of parameters to measure the dependence of the results to the model assumptions.

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

- establishing design flood conditions;
- providing flood information for development control plans; and
- modelling "what-if" management options to assess hydraulic impacts and benefit-cost ratios.

Only the first dot point above has been carried out as part of this Flood Study. The other two dot points are the subject of a subsequent Floodplain Risk Management Study, which will be prepared by Council at some time in the future.

4.2 Data Sources

A variety of data was collated and used to develop the different model databases or used to develop model parameters. The main sources of data were:

- Topographic maps (1:25,000);
- Orthophoto maps (1:4,000);
- Historical aerial photographs;
- Ground surface survey (collected as part of this study);
- Hydrographic survey of the Lake from the Department of Land and Water Conservation (2001);
- Historical flood descriptions collected through resident survey;
- Rainfall data for historical events from the Bureau of Meteorology;
- Tide levels at Jervis Bay and Sydney, and water levels at the recorder located upstream of the Causeway;
- Offshore wave conditions from waverider buoys for the February 1992 flood; and
- Flood level data for historic events collected through resident survey.

4.3 Digital Terrain Model (DTM)

A Digital Terrain Model (DTM) is a three-dimensional (3D) representation of the ground surface. A DTM is used to define the ground surface levels of the hydrodynamic model. Given that ground levels are required for nearly 39,000 individual elements within the model, a DTM represents the most effective way for these levels to be determined automatically.

A DTM of the Burrill Lake model was created from the following data sources:

- Data from a hydrosurvey undertaken by the Department of Land and Water Conservation in March 2001;
- Ground survey data collected during the course of this project as described in Section 2.5;
- Additional ground survey data collected in the vicinity of Wallaroy Crescent (Bungalow Park) and low-lying areas of Kings Point to assist with accurate mapping of predicted flood extents;

- Level information provided by Council for a current development on Balmoral Drive, Bungalow Park;
- Land and Information Centre (LIC) 2m contour data to define levels around the fringes of the Lake where other information was not available;
- Land and Information Centre (LIC) 10m contour data, as shown on topographic maps, to define higher ground where data was not available; and
- Nearshore bathymetric contours were taken from digital version of topographic maps.

The hydrosurvey and ground survey data comprise the most significant data sets for the study.

The data sets are all of good accuracy (with an expected tolerance of +/- 0.1 m), with the exception of the LIC data which has a tolerance of +/- 1-2 metres. This data source was only used to define the overbank part of the deep Lake cross-sections (primarily modelled in 1D). Due to the steepness of the valley sides, the LIC data inaccuracy would have negligible impact on the results of the model. Simple hydraulic analysis shows that an accuracy of +/- 2m on the section where this data was used has minimal impact on flood discharges (ie. +/- 0.1%).

The DTM was constructed as a Triangular Irregular Network (TIN), which is simply a mass of interconnected triangles. For each triangle, the ground level is defined at each of the three vertices, thereby defining a plane surface over the area of the triangle. Where the datasets indicate that there are significant variations in the ground surface slope and elevation, the density of triangles was increased.

The resulting DTM used in the hydrodynamic model is composed of nearly 3.2 million ground level points interpolated linearly between the TIN vertices.

Figure 4-1 presents the Burrill Lake DTM. Changes in Lake bed level colours clearly show the difference in depths between the area in the vicinity of Burrill Inlet and the deeper basins within the main body of the Lake.

Figure 4-2 shows the extent of the two main sources of survey data used for the creation of the hydrodynamic model DTM.

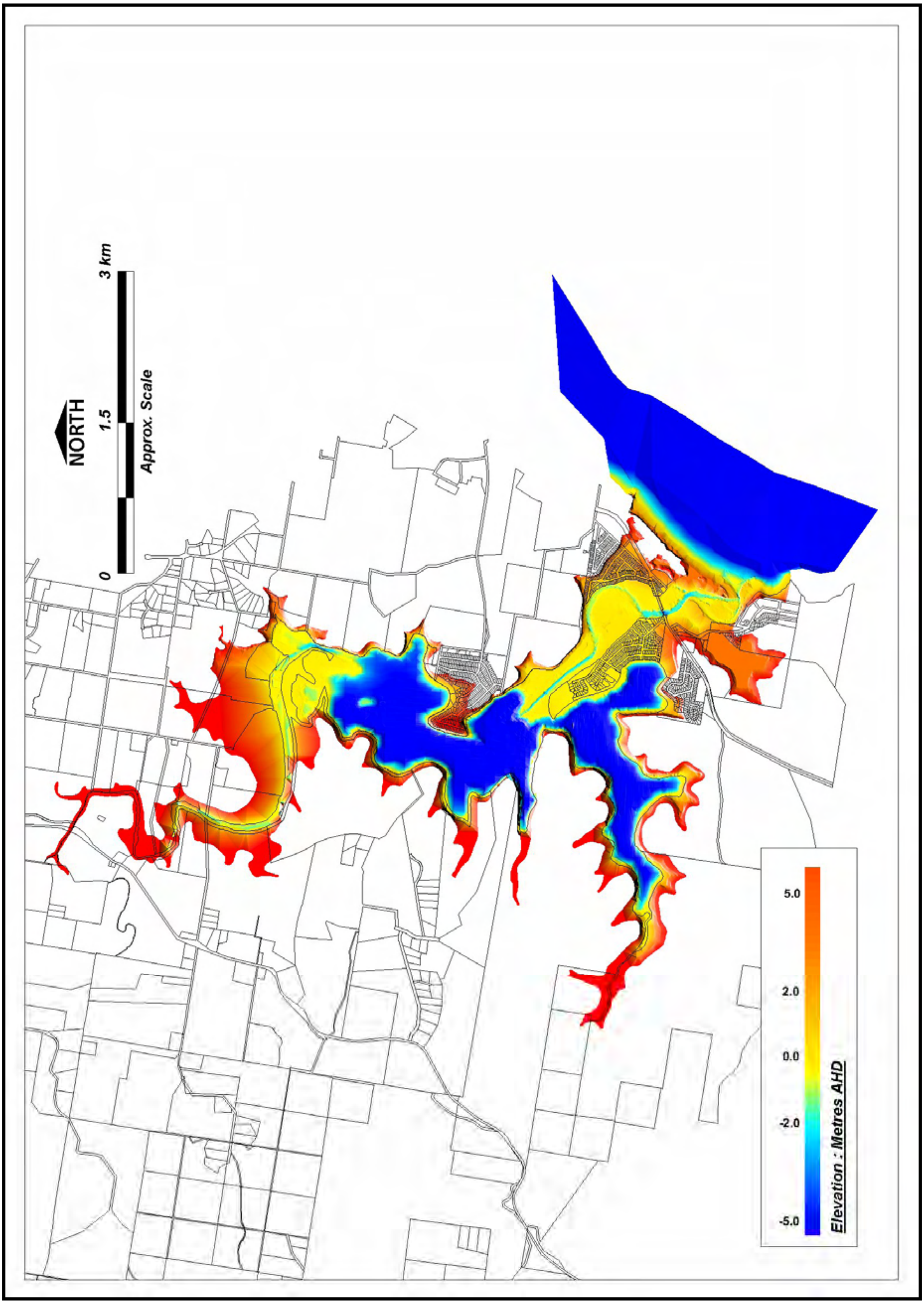


Figure 4-1 DTM Levels in the Vicinity of the Hydrodynamic Model

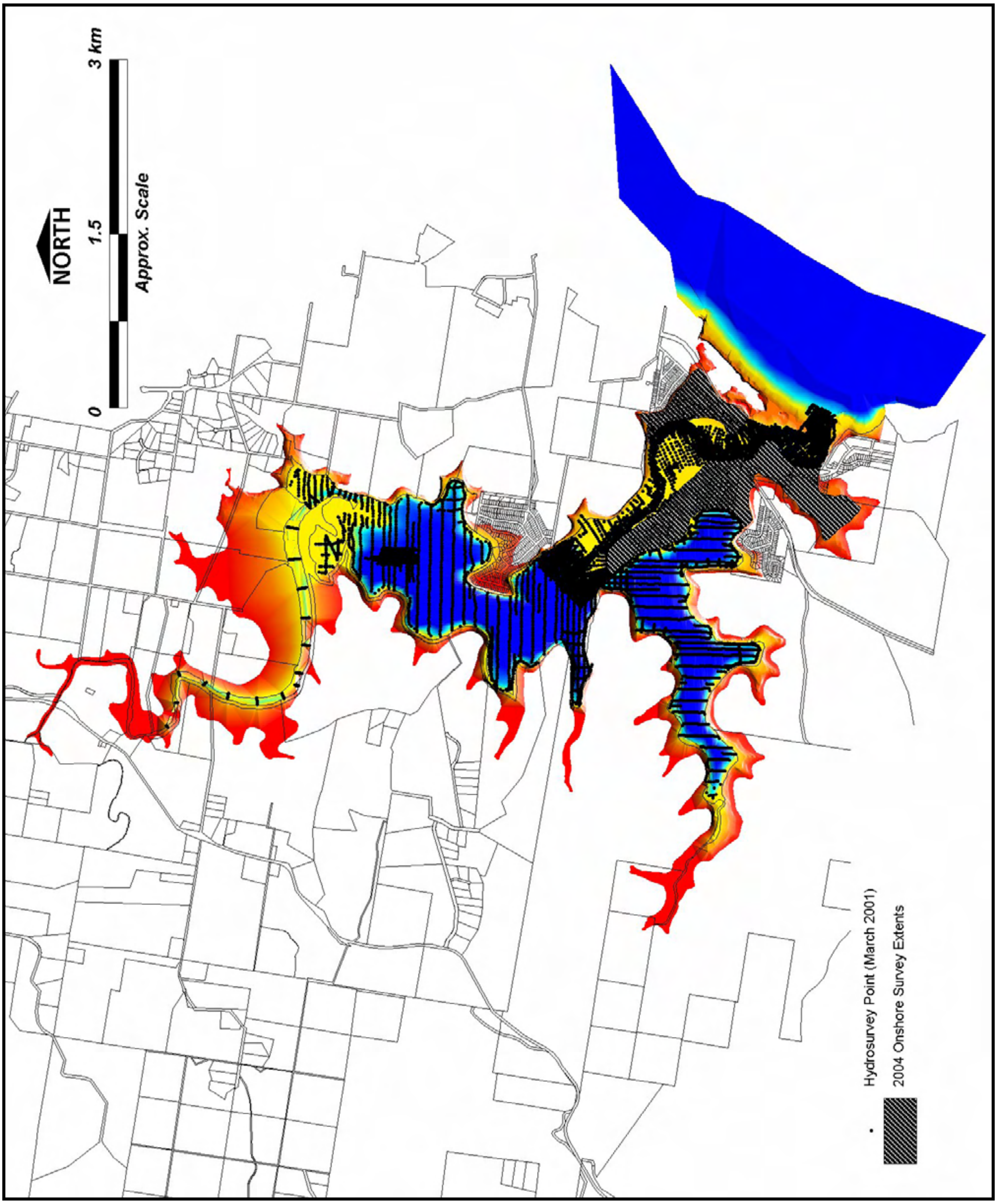


Figure 4-2 Extents of Major DTM Data Sources in the Vicinity of the Hydrodynamic Model

4.3.1 Historical Changes to the DTM

Structures like roads, bridges, culverts and embankments can change over time. Natural features such as the entrance shoals at Burrill Lake can also change. As topographic details and some structure details are incorporated directly into the DTM, it needs to be altered to represent the specific years in which events occur (associated with historical calibration and verification events).

Changes to the topography of the entrance shoals and land surrounding Burrill Inlet are discussed in Section 2.3. Two DTM's were developed, one for each calibration event. Issues considered when developing the DTM's were the following:

- The Max Auld subdivision, to the north of the peninsula upon which Bungalow Park is located, was created between 1977 and 1981. This involved the dredging of material from the northern tip of the peninsula to create the lagoon and using the extracted material to fill the residential lots above the design flood level. The Max Auld Subdivision and Lagoon are present in the 1991 DTM but not the 1971 DTM. Presentation of this area of the DTM for the two events is provided in Figure 4-3;
- Between 1972 and 1975, dredging of Burrill Inlet was undertaken to the south of Lions Park and the material extracted was used to fill low lying areas of Lions Park. The changes related to these works are present in the 1991 DTM but not the 1971 DTM. Presentation of this area of the DTM for the two events is provided in Figure 4-4 and Figure 4-5;
- The entrance compartment is in a constant state of change. WBM (2001) notes that significant changes since 1971 have been limited to the area downstream of the causeway. Appropriate changes to the DTM in this area have been derived based on aerial photography from relevant years as follows:
  - July 1967 aerial photograph for the February 1971 flood (refer Figure 4-4); and
  - April 1986 aerial photograph for the June 1991 and February 1992 floods (refer Figure 4-5).

The historical photographs were compared with the latest aerial photograph representing the most accurately surveyed entrance levels (i.e. 2001). Comparisons between shoal locations and apparent water depths, from fixed points like rock formations, assisted with the estimation of the historical entrance geometries.

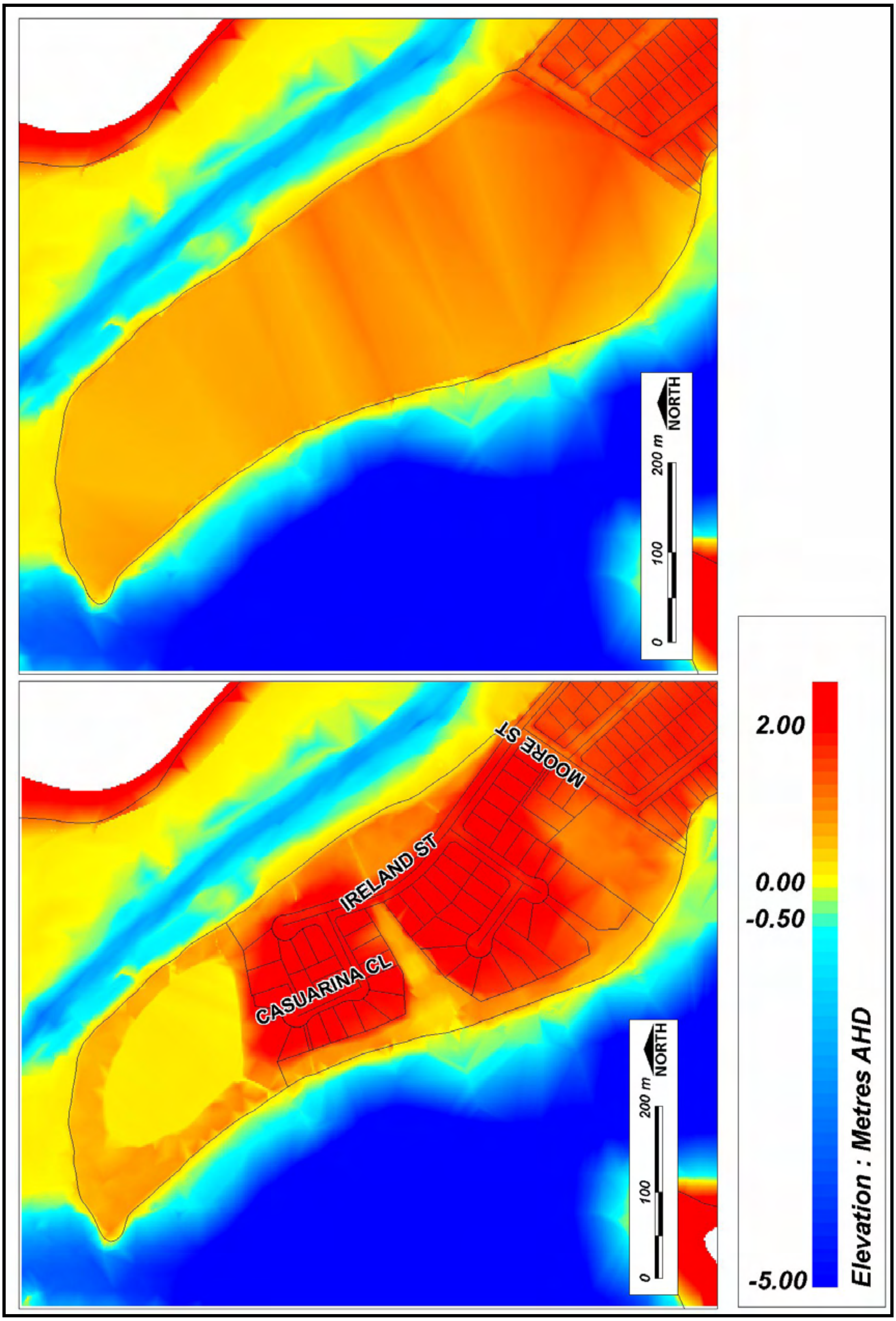


Figure 4-3 DTM's for 1971 (left/bottom) and 1991 (right/top) in the vicinity of the Max Auld Subdivision

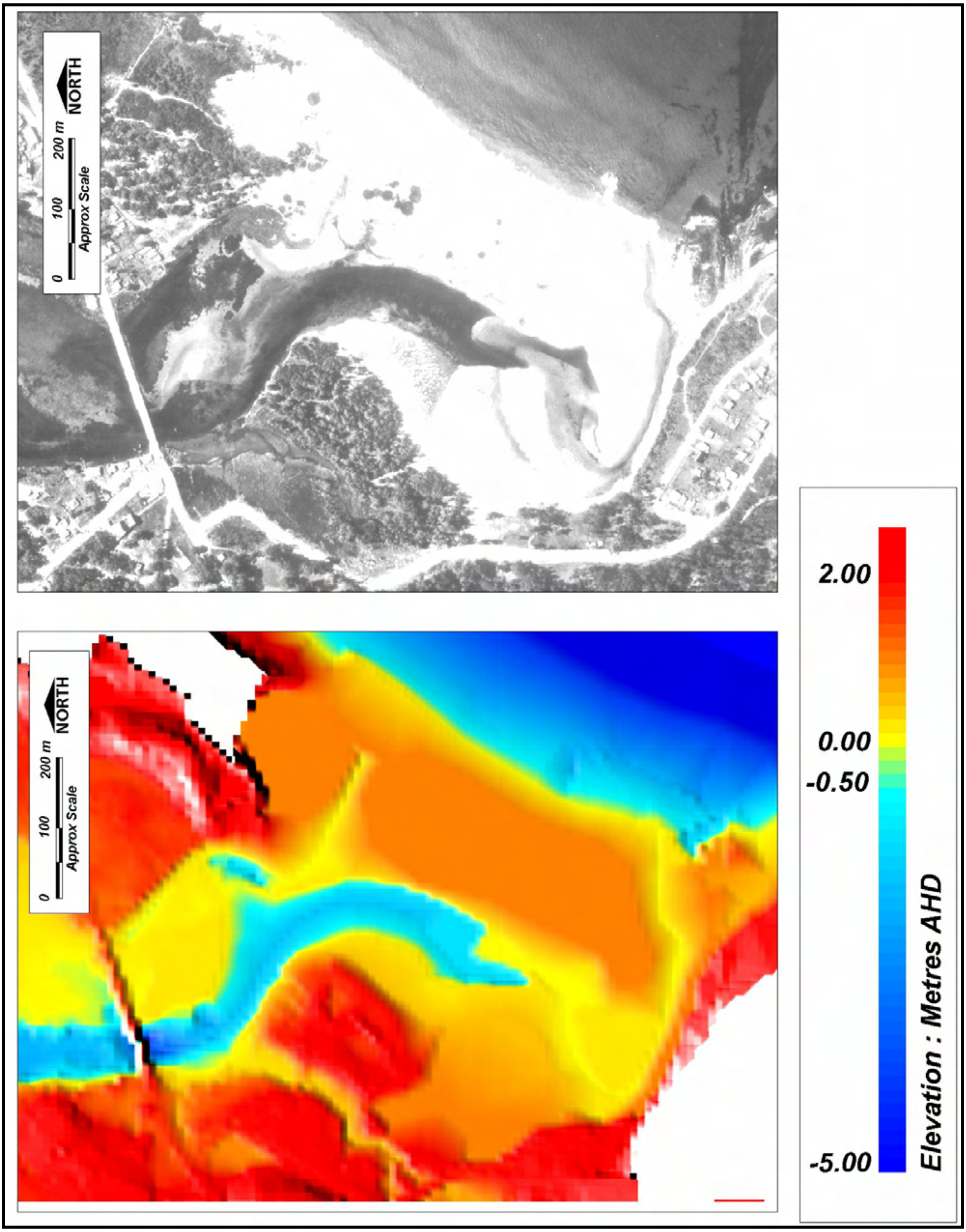


Figure 4-4 Entrance Compartment Aerial Photograph (July 1967) and DTM for 1971 Flood (Aerial Photo Tide Level ~ 0.2 m AHD)

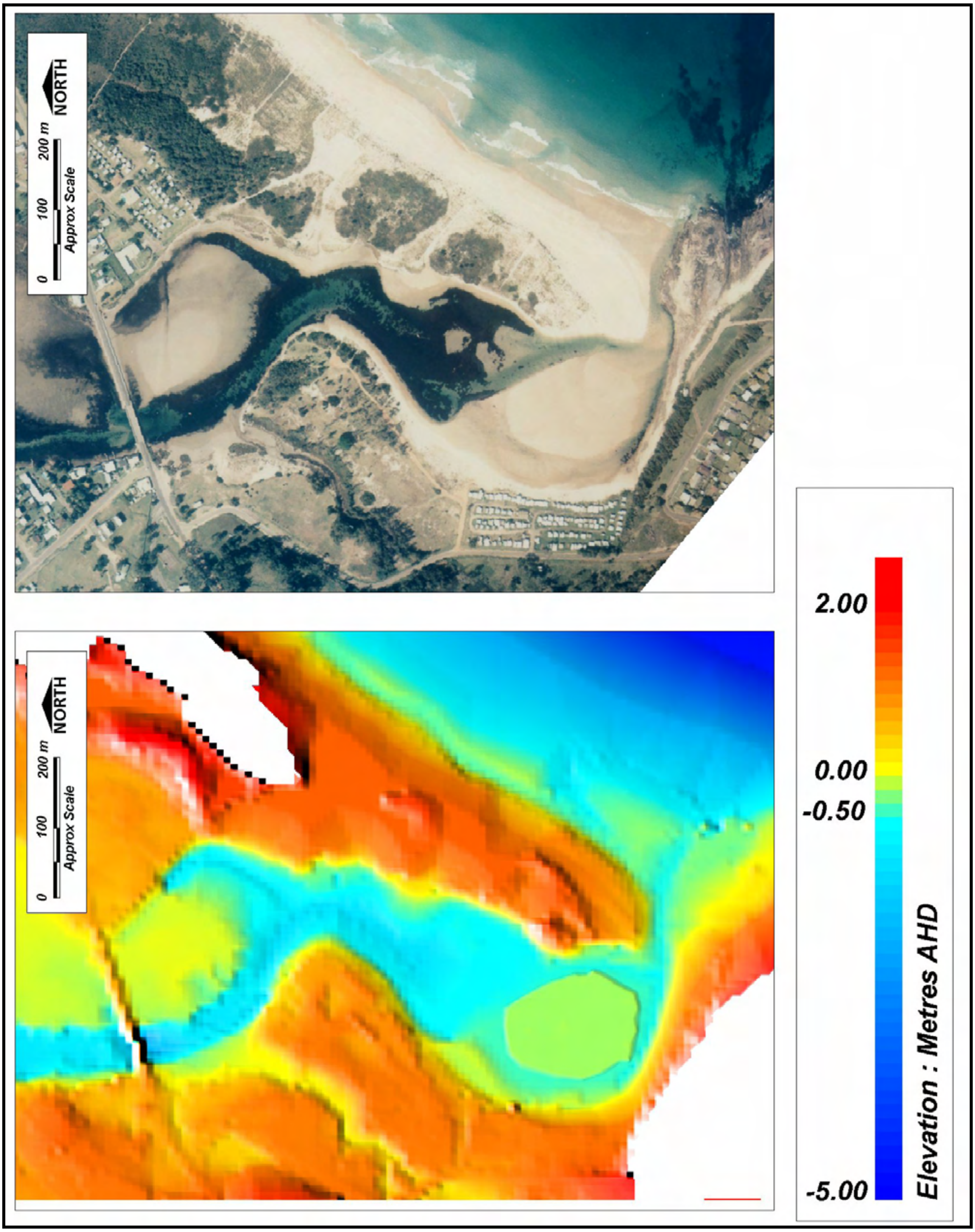


Figure 4-5 Entrance Compartment Aerial Photograph (April 1986) and DTM for 1991 & 1992 Floods (Aerial Photo Tide Level ~ -0.4 m AHD)

4.3.2 Accuracy of the DTM

The accuracy of a hydrodynamic hydraulic model is dependent on the vertical accuracy of the associated DTM, as the topography of the ground controls flow behaviour during times of flood. The ground levels input to the hydrodynamic model are extracted from the DTM.

The ground survey carried out for this project is likely to have an accuracy of about 0.1 metres.

The Lake bathymetric survey undertaken in 2001 is likely to have a high level of accuracy. However, currents, floods and sedimentation may have modified the entrance levels since that time. A conceptual model of sediment transport provided in the Burrill Inlet Causeway Options Study (WBM, 2001) indicates that there has been minimal change of shoals upstream of the entrance during the last 100 years. Jones et al. (2003), however, has reported significant progradation of the Stony Creek fluvial delta which is attributed to broad scale vegetation clearing and catchment sediment runoff. Fortunately, these changes would have no impact on flood behaviour in the Lake. Changes in the active entrance shoal areas (i.e. downstream of the causeway and to the south of Lions Park) have been taken into account as described in Section 4.3.1.

Given the low energy gradient across most of the Lake, inaccuracies in Lake bed levels are unlikely to have significant impacts on flood levels. The same remark applies to the LIC information, (which is only used in the higher parts of the DTM and is thus not inundated by even the most severe flood), which is known to have an accuracy of about +/- 2 metres.

The accuracy of the DTM in the vicinity of the entrance shoals is expected to be in the order of 0.5 to 1 metre as ground and bed levels were estimated from historical aerial photographs (i.e. measured ground control points were not available). Furthermore, the time between the date of photography and the calibration flood events are almost four and five years for the 1971 and 1991 floods respectively. Given the dynamic nature of the entrance channel, it is highly likely that the entrance conditions immediately prior to the flood were different to those represented in the air photos. Nonetheless, this still represents the best possible estimate of entrance conditions at the time of the calibration events. In addition, the geomorphologic model modifies the entrance configuration during the simulation to suit the discharge. This reduces the potential impacts of inaccuracies in the starting configuration of the entrance shoals.

4.4 Model Discretisation

Model discretisation is necessary to simplify the real-world into one that can be represented by discrete elements. The computer then solves equations within every discrete element to simulate the hydrologic, hydrodynamic and geomorphologic processes.

Within the model, the smaller the elements become, the closer the model approaches the real-world situation. However, as the number of elements increases, the computational resources required to run the model becomes more demanding, while the model also becomes more difficult to set up and manipulate. Also, there is a point where increasing the number of elements in a model may not provide any significant improvement in model predictions and accuracy. Therefore a suitable balance needs to be found between the number of elements used to represent the study area and the practicalities of using the model for future management purposes.

In constructing the model, the number, size and location of elements has taken into account the following:

- location of available data (eg. creek/lake section surveys);
- location of recorded data (eg. gauging sites);
- location of controlling features (eg. dams, embankments, bridges, weirs, flow constrictions);
- desired accuracy to meet the study's objectives;
- limitations of the computer software (ie. the number of elements the software can handle, and more importantly, to keep within the constraints of the mathematical solution); and
- limitations of the computer hardware.

The model discretisation process has resulted in hydrologic and hydrodynamic (incorporating both 1D and 2D elements) models as discussed further in Sections 4.5 and 4.6.

4.5 Hydrologic (Catchment Runoff) 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 subcatchments inter-connected by channel reaches representing the creeks and rivers. The subcatchments 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.

4.5.1 Model Setup

The RAFTS-XP software was used to develop the hydrologic model. Details of the software are presented in Appendix B.

The Burrill Lake catchment, with a drainage area of about 78 km^2, has been discretised into 26 subcatchments, which feed into Burrill Lake and the associated floodplains at 11 different locations.

The sub-catchment limits have been defined so that the descriptive parameters are generally uniform within the boundaries (which are slope, landuse, permeability, roughness). Digital topographic contours provided the necessary data to calculate the subcatchments' slopes and to determine landuse characteristics. The slopes vary from less than 1%, in the vicinity of the Lake, to 37.2% near Mount Kingiman and along the ridge lines draining to the western side of the Lake. Slopes within the catchment average 6.1%, with a standard deviation 5.3%. The vegetation cover is mostly forested for the south western parts of the catchment with cleared rural area comprising the north-western parts of the catchment. Urbanised areas are concentrated around Burrill Inlet, primarily within the boundaries of the hydrodynamic model.

The resulting average sub-catchment surface area is 2.35 km^2, with a standard deviation of 1.3 km^2. The biggest sub-catchment has an area of 4.41 km^2. The surface of the Lake itself accounts for 4.0 km^2 of the total catchment area. Smaller subcatchments are more likely to receive uniform rainfall distributions.

The variability of rainfall within the subcatchments has been assessed. For each of the subcatchments the total rainfall for the 48 hours to 9am on February 7, 1971 was assessed. The spatial minimum, maximum and average values were determined using GIS analysis. Furthermore, the maximum deviation of local rainfall values from the mean of the entire subcatchment was determined. For all subcatchments, the maximum variability did not exceed 3.0%. The average variability for all subcatchments was around 1.6% with a standard deviation of 0.72%.

Similarly the variability of rainfall was assessed for the 1992 event. For this event, the maximum variability for a given subcatchment was 10%. The average variability for all subcatchments was around 4.2% with a standard deviation of 2.3%.

Considering the spatial variation of rainfall across the entire catchment during both events, it is clear that rainfall could be considered as uniformly distributed for the purposes of calibration.

A schematic of the model network is provided on Figure 4-6.

4.6 Hydrodynamic Model

4.6.1 Model Setup

The hydrodynamic model simulates the dynamic flooding behaviour in Burrill Lake, including the interactions between the two basins of the Lake, Burrill Inlet and the floodplains.

The modelling software, TUFLOW, was used to develop a 2D/1D hydrodynamic model of the study area. The model is a mixture of one-dimensional (1D) and two-dimensional (2D) domains with the 2D domain covering the key areas of existing and future management interest. Two-dimensional domains produce a significantly higher order of resolution in terms of hydrodynamic computations. 1D domains are suited to modelling areas away from the areas of interest, where flow is essentially 1D, or where the 2D resolution does not adequately depict the shape of a key flowpath (eg. when the width of a creek is only covered by one or two cells).

The hydrodynamic model network and its relevant branches is provided in Figure 4-7.

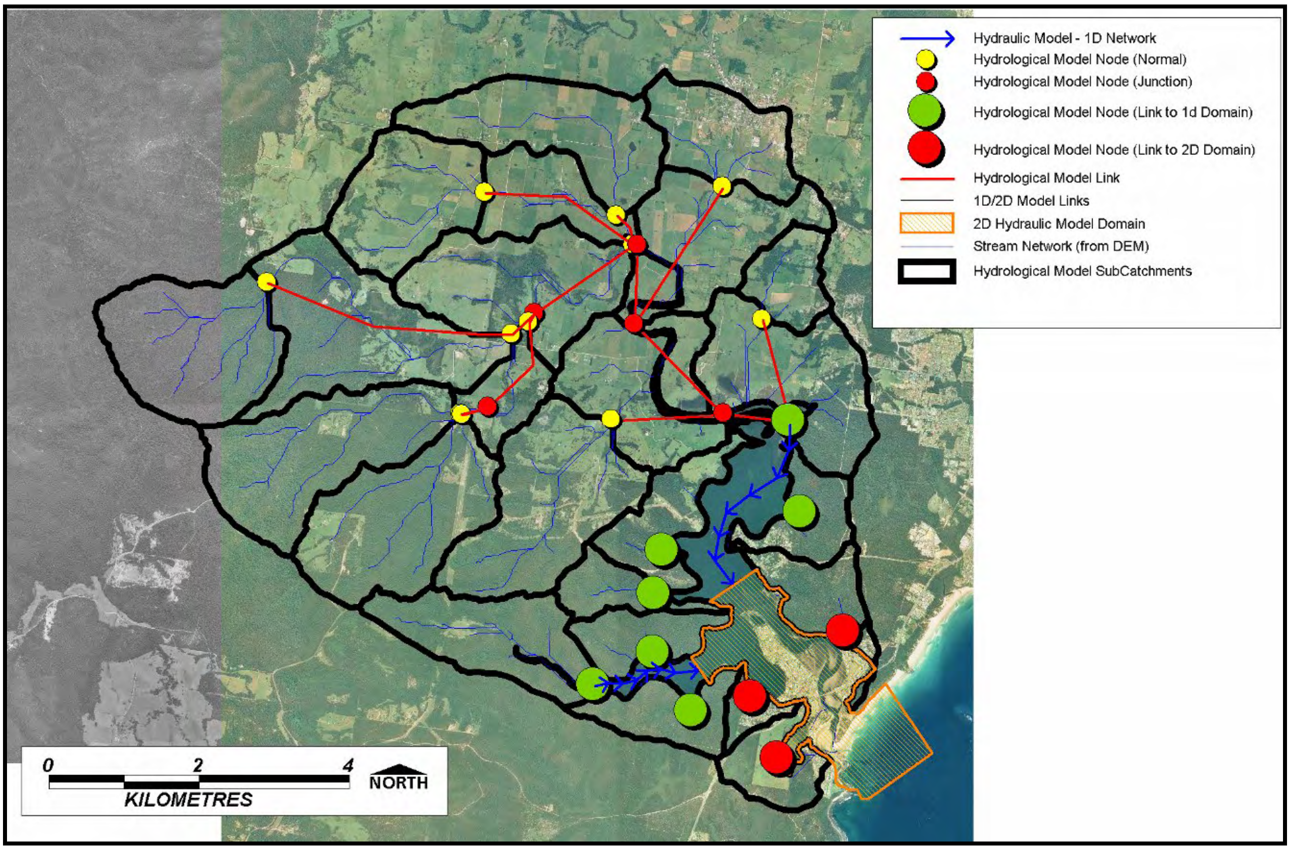


Figure 4-6 RAFTS-XP Burrill Lake Catchment Model Network (Including Links to Hydrodynamic Model)

As TUFLOW is a finite difference model, the Burrill Lake Flood Model has been constructed using elements with a regular grid of size 10m x 10m. Depths at each of the model grid cells were obtained from the DTM. This means that hydraulic parameters are calculated separately for every 10m square of the 3.88km^2 study area represented in 2D. Around 38,000 individual elements make up the flood model, each with individual levels, roughness, boundary conditions, flow constrictions and flow structure details where appropriate. The ground level at each corner, centre and mid-side location of these elements is obtained automatically by interrogating the DTM (see Section 4.3).

The two-dimensional TUFLOW model domain is dynamically linked to one-dimensional models, representing the deeper parts of the Lake upstream of the areas of interest. The hydrodynamic behaviour of the deeper sections of the Lake is unidirectional and can be accurately represented by 1D elements. The computational timestep used for the TUFLOW model is five (5) seconds. This means that the hydrodynamics within around 38,000 model elements is recalculated for each five second time step throughout the flood event. For a 24 hour flood simulation, this equates to around 260 million calculations.

Figure 4-8 presents the different development stages of the model.

Information on the TUFLOW software is presented further in Appendix B.

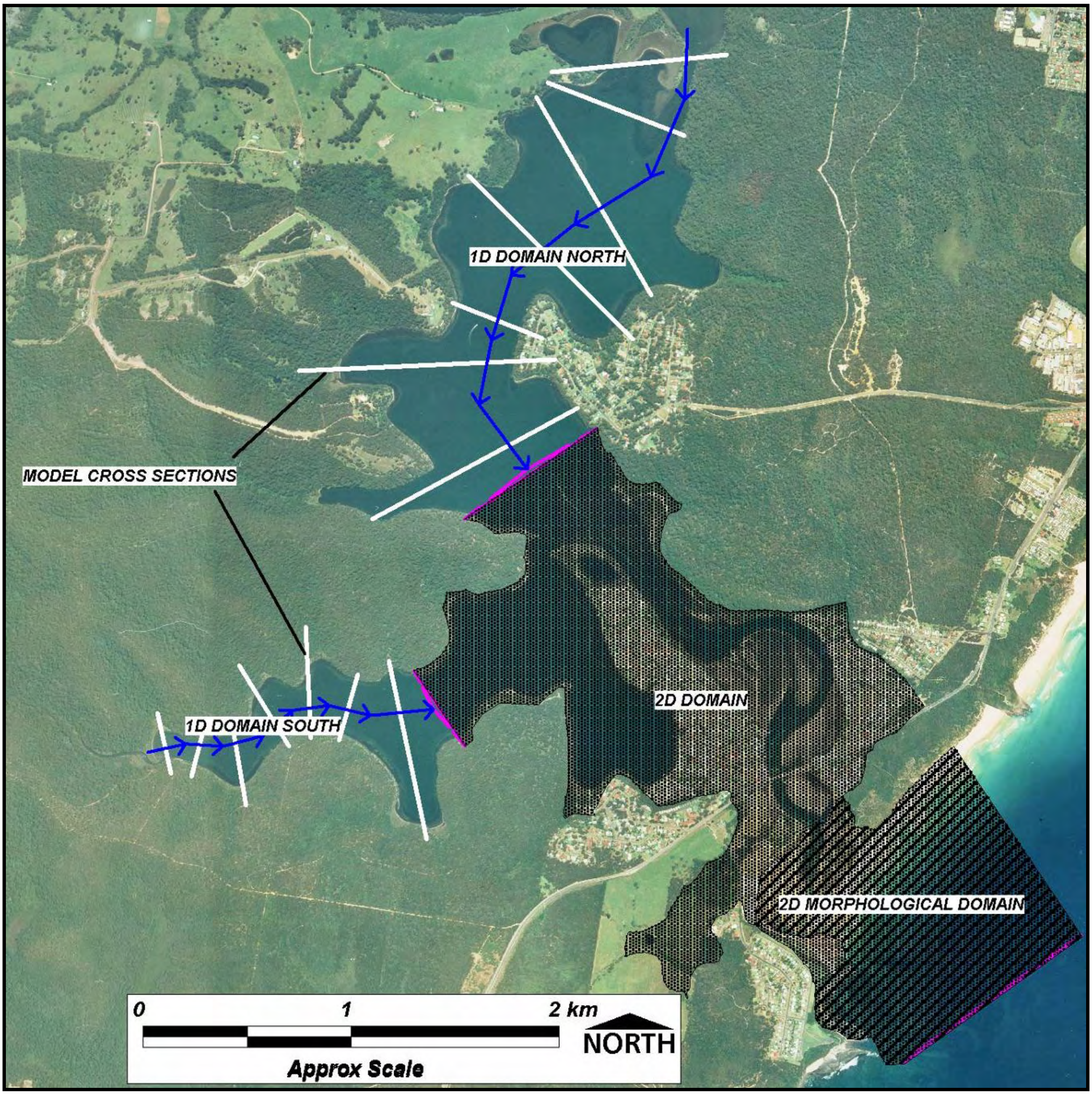


Figure 4-7 Burrill Lake Hydrodynamic Model Layout

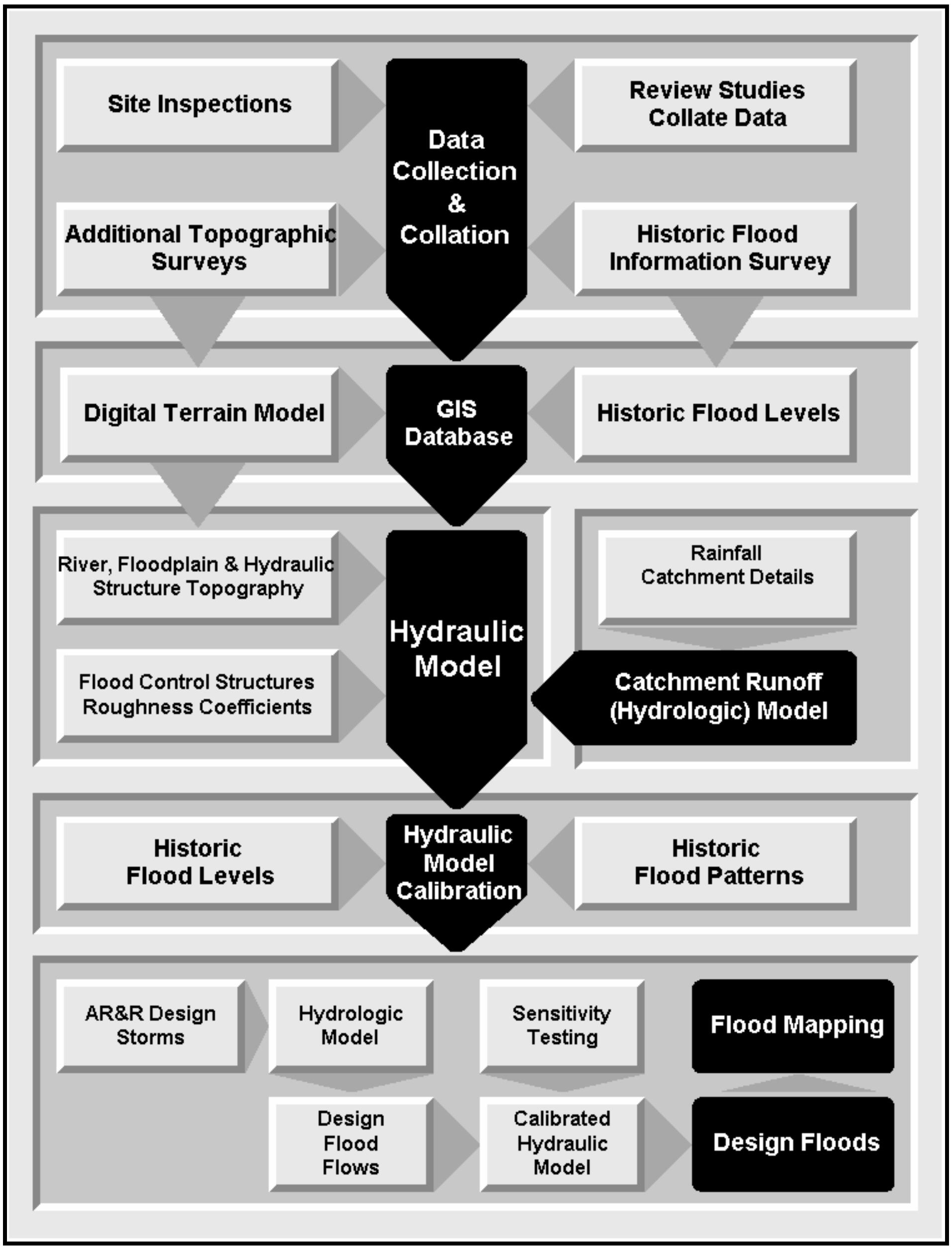


Figure 4-8 Flood Study Model Development Process

4.6.2 Model Inputs

Inputs to the Burrill Lake hydrodynamic model include:

- **Topography** of the lake and the floodplain based on the DTM and creek cross-sections. The DTM is discussed in Section 4.3;
- **Hydraulic roughness** of the lake and inlet bed and the floodplain land. The final set of hydraulic roughness values is determined during the calibration process;
- **Hydraulic structures,** particularly the causeway and bridge structure across Burrill Inlet and structures along Coopers Creek and under Dolphin Point Road;

- **Inflows:** the rainfall runoff calculated by the hydrologic model at nodes adjacent to the lake and over the lake body (see Section 4.5); and
- **Boundary water levels:** depending on the entrance shoal geometry, water level in the Tasman Sea can influence water levels in the lake. The downstream water level is varied over time to take into account all significant water level components (astronomical tide, storm surge and wave setup).

A range of sensitivity checks have been undertaken to evaluate the model. These tests are described further in Section 6.

4.6.3 Model Outputs

Model outputs are flood levels, discharges, and velocities describing the flood behaviour over time for a given flood event. Based on these outputs, hydraulic categories and provisional hazards associated with flood flows were also determined.

Individual model outputs are provided for every 2D element within the model. This means that for the Burrill Lake model, results at around 38,000 different locations (every 10m x 10m grid cell) are provided at every timestep. Model outputs are also available at 1D elements. Given this vast amount of output data, a Geographic Information System (GIS) was used to assist in interpretation and presentation of the spatially-dependent results.

4.7 Geomorphologic Model

This section describes the technical details of the geomorphologic model. Some basic background knowledge in sediment transport is recommended to fully understand Section 4.7 (e.g. Van Rijn, 1993).

4.7.1 Introduction

Burrill Inlet, which connects the Lake to the Tasman Sea is located at the southern end of Burrill Beach. Marine sand can be transported laterally along the beach and deposited within the entrance shoals during storm events. The direction of sand movement is related to the direction that waves approach the beach. For example, if waves from the north east dominate, the movement of sand can close Burrill Inlet to the Tasman Sea.

The entrance channel is influenced by two primary hydrodynamic processes. These are the regular tidal movement of seawater and the episodic freshwater runoff from the catchment draining to the sea. The hydrodynamics constantly rework marine sands around the entrance, particularly during times of flood.

The ability to model morphological changes in the entrance during a flood is therefore critical to the flood study, as it incorporates changes to the effectiveness of the entrance in conveying water out of the Lake during the flood event. The changing entrance shape affects peak water levels in the Lake during a flood.

The geomorphologic module within TUFLOW was used for this study. The module is based on the theory and methods described in Van Rijn (1990).

The Van Rijn formulation of sand transport is generally accepted as being currently the most feasible and accurate method for estimating sand transport. However, it must be noted that sand transport is a complex interaction of process that is still not fully understood. In order to account for these uncertainties, it is necessary to make approximations related to a number of the process interactions. Although these approximations are unavoidable, the Van Rijn method is still appropriate to combine with the 2D (depth-averaged) TUFLOW hydrodynamic routines to achieve realistic time-varying entrance shoal and beach berm levels and the accompanying simulated flood discharges.

4.7.2 Sand Transport Formulation

Quantification of sand transport rates is achieved by the use of two unifying and fundamental concepts:

(i) The combined action of currents and waves mobilises the bottom sands and sets them into motion, and

(ii) The bottom sediment, once mobilised, is moved in the direction of the prevailing net current. The net current can be the result of factors such as river flow, tides, wind, wave radiation stresses or asymmetry in the oscillatory wave motion, or a combination of these.

4.7.3 Geomorphologic Modelling Extent

The combined TUFLOW / Van Rijn equations can be implemented over the entire 2D model domain. However, the long modelling time imposes a practical limit for the sediment transport modelling area. While some sediment transport can be expected throughout Burrill Lake during flood events, preliminary calculations show that most transport occurs at the downstream end in the immediate vicinity of the sand berm.

The geomorphologic model is laid over part of the 2D domain of the TUFLOW model. For the Burrill Lake model, the geomorphologic equations were applied to a zone extending from the Tasman Sea, across the entrance berm and upstream to an area immediately adjacent to Lions Park, around 200 m downstream of the causeway. This corresponds to an area that was identified as having changed significantly over the last 100 years and is therefore the area of most interest during flood events.

Anecdotal evidence indicates that shallow bedrock exists in the entrance area, although there is no information available regarding the actual depths. Based on levels of the adjacent platform and experience elsewhere, a limit on the depth to which the entrance area could erode was set at –2.0 m AHD.

4.7.4 Geomorphologic Model Input

Inputs to the geomorphologic model include:

- D50 (median grain size of a representative sand sample): 0.30 mm;
- D90: 0.40 m (grain size which is exceeded by 10% of a representative sand sample);
- Fall Velocity (settling velocity of sand grains through water within a representative sand sample: 0.040 m/s);

- Sand Grain Density: 2650 kg/m^3; and
- Water Density: 1035 kg/m^3.

These values are consistent with previous modelling undertaken as described in WBM (2001), which based the sediment characteristics on grain size distributions provided in PWD (1992).

5 MODEL CALIBRATION AND VALIDATION TO HISTORICAL FLOODS

5.1 Introduction

From the information provided in preceding chapters it is clear that the data available to calibrate and verify the hydrologic and hydraulic models developed during this study is limited. It should be noted that it is common to not have all the information desired to calibrate flood models and that some judgement is normally required where data are not available. The limitations of the data are reiterated in Section 5.2 as a precursor to the discussion of the model calibration and validation process.

5.2 Limitations of the Data Set

5.2.1 Temporal Rainfall Patterns

Temporal rainfall patterns are available for both the calibration and validation events, but the stations at which they were recorded are a significant distance from the from the Burrill Lake catchment (ie 38 km for Turpentine – available for February 1992; and 48 km for Nowra RAN Station - available for February 1971). It could be questioned whether the temporal pattern at these locations are reflective of the temporal patterns that were present within the Burrill Lake catchment for the events being considered.

Nevertheless, in the absence of better data, the available temporal rainfall patterns have been adopted. We consider that, as the temporal patterns represent real rainfall that occurred within the vicinity of the catchment during the events of interest, this is the most appropriate data to adopt in the models. The total daily rainfall that fell at the pluviometer stations during the calibration and validation events was compared to the daily totals that were recorded within the Burrill Lake catchment and the temporal pattern was linearly scaled to account for any difference in total rainfall.

5.2.2 Absence of Flow Hydrographs

No flow gauges are present within the Burrill Lake catchment. For a conventional calibration, the historical record from a flow gauge would be used to calibrate the hydrologic model. In the absence of historical flow records, it has been necessary to undertake calibration and validation of the hydrologic and hydrodynamic models in conjunction with each other. It has been found that adoption of standard parameters within the hydrologic model has resulted in sensible output from the model. The standard parameters adopted were as follows:

- Initial Losses: 15 mm;
- Continuing Losses: 2.5 mm/hr; and
- Storage Coefficient (β): 1.

The initial and continuing loss values represent the defaults in the RAFTS-XP model, and standard values as recommended by Australian Rainfall and Runoff (2001).

Sensitivity testing on the performance of the hydrologic model, which compares design flow quantities predicted by the hydrologic model to the empirically based ‘rational method’, are presented in Section 6.2.2.

5.2.3 Initial Water Level Conditions

The initial water level condition at the onset of a flood can impact on the flood behaviour of Burrill Lake. When the initial water level at the beginning of the rain event is not known for a model simulation, the available storage volume prior to entrance berm overtopping is also not known. For example, if the water level is high initially, and the entrance to the Lake is closed, the initial rate of rise of flood waters can be relatively fast. This can result in a relatively early overtopping of the entrance berm, resulting in rapid scour and a fairly open entrance by the time the flood peak arrives. In this situation, the entrance is more capable of conveying flood flows at the flood peak, which would attenuate the maximum flood level. The impact of the initial water level is thus related to both the entrance condition and the rates of rainfall within the catchment. When combined, the result can be counter intuitive to the expected result when considering the factors individually.

The initial water level upstream of the causeway is available for the 1992 flood event as shown on Figure 3-9. Within the main body of the Lake (which represents the main storage area of the system) tidal gauging data from NSW Department of Public Works and Services (2001) indicates that the mean Lake level is around 5 to 10 cm higher than the mean level at the causeway. Furthermore, the tidal range within the Lake is indicated to be around half that at the causeway. Considering this information, along with the tidal graph shown in Figure 3-9, the initial water level within the Lake has been estimated to be 0.35 m AHD for the February 1992 event.

No water level data is available inside the Lake for the 1971 event. Based on the historical evidence provided in Section 3, it is considered that the Lake entrance was either closed or significantly shoaled. Therefore, it may be considered that the water level in the Lake may be somewhat elevated above normal levels when there is no tidal connection. Inspection of tidal levels during 2003, a period when the entrance was known to be heavily shoaled, indicates that water levels measured at the causeway were typically between 0.2 m AHD and 0.4 m AHD, with occasional peaks up to 0.6 m AHD and occasional lows down to 0.0 m AHD. On balance, it was considered appropriate to again adopt an initial water level of 0.35 m although we recognise that, dependant on the degree to which the entrance was shoaled (there is some uncertainty regarding this), the water level may have been either lower or higher.

As an indication of the flood severity and the Lake’s storage potential, the theoretical maximum rise in the Lake water level has been calculated for the calibration and validation events, assuming a totally blocked entrance, no losses and a constant Lake surface area of 4.0 km^2:

- 1971 flood: a rise of 8.55 m; and
- 1992 flood: a rise of 5.54 m.

As the closed sand berm would typically not exceed a level of about RL 2.0 m AHD these events would have resulted in significant breaching of the entrance berm (if the entrance was closed) and subsequent entrance scour, which would have had significant repercussions on Lake flood levels. When compared to a fully open entrance condition, it is clear that flood levels in the Lake could be easily affected by a closed or heavily shoaled entrance condition.

5.2.4 Tasman Sea Water Level and Wave Conditions

The Tasman Sea water levels adopted for the two events comprise the following components (see also Section 3.2.4):

- Tide levels from recorders that are representative of the ocean water levels from Jervis Bay (February 1992) and Fort Denison (February 1971);
- An allowance for the effects of wave set-up.

We note that the tidal records were not immediately offshore of the Burrill Lake entrance (particularly for the 1971 event), however, we consider that they are still reasonably representative of ocean conditions at the site.

The relationship used to derive the wave set-up from offshore wave conditions (refer Section 3.2.4) is an approximation that is considered appropriate for application to the boundary of a flood model.

For the February 1992 event, wave conditions have been taken from the Batemans Bay record (around 45 km to the south), which is considered relatively close to Burrill Lake, and suitable for adoption of sea conditions. However, no record of offshore wave conditions is available for the February 1971 event. As described in Section 3.2.4, an allowance of 0.5 m setup has been applied to the tidal record of February 1971, as this appears to be a reasonable assumption for the coastal conditions that would occur during a significant flood.

5.2.5 Condition of Entrance Berm

Preliminary hydraulic calculations and consideration of anecdotal evidence from the community have demonstrated that the constriction created by a shoaled Lake entrance is a critical parameter determining flood levels in Burrill Lake.

The entrance shoal geometry acts as a flow obstruction. The presence of the obstruction forces the upstream water levels to rise in order to provide sufficient potential energy to convey the flood flows across the entrance shoals, and subsequently scour the shoals. The resulting energy head upstream of the entrance shoals acts as a control of upstream water levels.

During the height of a flood, the flow has a bed scouring capacity that is dependent on near-bed velocities, water depths, and sand characteristics. In relative terms, the potential to scour the entrance channel is much greater at the downstream end compared to more upstream sections of the channel due to the much smaller cross-sectional conveyance capacity (and hence much higher near-bed velocities).

At the scale of the study, the sediment transport routine, based on the Van Rijn equations, does not determine exact cross-sectional profiles. However, the model does allow the integration of scouring processes at the sand berm in terms of cross-sectional conveyance capacity. The scouring rate is based on inter-related parameters: flood flows, initial water levels, downstream ocean levels and, of greatest importance, the original sand berm geometry.

Unfortunately the original sand berm geometry in the model is not known and the historical erosion process cannot be replicated for the calibration and validation events. Preliminary hydrodynamic modelling has proven that the calculated eroded sand berm levels can be unrealistically high or low depending on the original sand berm geometry. This can lead to inappropriate water levels in the Lake.

No survey of ground levels in the entrance area is available for either of the flood events. Ideally, survey would be available both before and after the flood event to enable determination of the extent to which the flood had scoured the entrance area. In turn, this would enable calibration of the geomorphic component of the model. This type of data is extremely rare, and as such it has been necessary to set geomorphic model parameter values based on literature (e.g. Van Rijn, 1990) and previous modelling experience (e.g. WBM 2001, 2003, 2005).

Of most relevance for the calibration and validation simulations is the current related roughness, which has been given a value of 0.1. For the critical slope (which enables collapse of the edges of the entrance channel as it scours – refer Appendix B) a value of 1 in 5 has been adopted as being a fair compromise between the angle of repose of sand (around 1 in 2) and the type of underwater slope that is likely to be stable under the highly turbulent and non-uniform flow conditions present during flood conditions (1 in 7 - 1 in 10).

The initial ground levels that were adopted in the vicinity of the entrance shoals have been derived based on aerial photograph interpretation in GIS using the most recently available aerial photographs prior to both events (refer Section 4.3). For the February 1971 event, entrance levels were based on an aerial photograph from July 1967, while for the February 1992 event, entrance levels were based on an aerial photograph from April 1986. Clearly significant change in the entrance bed levels may have occurred between the dates of the aerial photos and the date of the corresponding flood events. Nevertheless, this is the most appropriate data available for these particular events and it has therefore been adopted.

5.2.6 Lack of Flood Marks

Historical flood information is scarce for Burrill Lake as discussed in Section 3.2.2.1. In particular, all historical flood marks identified during the study are located in the residential areas upstream of the causeway. This means that variation of peak water levels across the main areas of interest (including the areas downstream of the causeway) is not represented by the available flood marks.

In total, four marks were available for the February 1971 flood event and two were available for the February 1992 flood event. This is considered less than ideal. However, the peak flood levels represented by the available marks show a variation of less than 0.1 m for both events. This provides confidence that the marks comprise a reasonable representation of the peak flood levels for the two events. Accordingly, calibration and validation of the models to these limited flood marks is considered appropriate.

5.3 Role of Additional Model Evaluation

The limitations in the data set are significant. While the calibration achieved against this data set is considered good (refer Section 5.5), additional sensitivity testing has been undertaken to examine the behaviour of the model if input parameters are varied. The testing was undertaken in consultation with SCC and DNR including determination of the parameters to be varied and the amount by which they were varied.

The sensitivity testing was also used to further inform the parameters that would be appropriate for adoption in the final design runs. Again, the decision on these parameters was undertaken in consultation with SCC and DNR. The model evaluation and sensitivity testing process is described in Section 6.

5.4 Behaviour of the Causeway and Lower Inlet

A one-dimensional model of the causeway area was established using HEC-RAS to investigate the behaviour of the causeway bridge during flood conditions, and to provide verification of the afflux predicted by TUFLOW across the bridge. HEC-RAS is widely acknowledged as being a suitable tool for this type of verification. Using conditions predicted by TUFLOW at different stages during the calibration flood, HEC-RAS was used to provide 'steady-state' predictions of backwater values at the site. Due to the different nature of the two computational methods, it is difficult to draw a direct comparison between the computed results, as the afflux at the bridge is dependant on a number of factors including the condition of the entrance, the state of the tide, and the instantaneous water levels and flow rates. However, the investigation showed that TUFLOW and HEC-RAS both exhibited similar behaviours as follows:

- Both models indicated a strong effect of tail water levels on afflux across the bridge, with the bridge being 'drowned' at high tailwater levels resulting in small head losses across the structure (of around 0.05 m).
- Lower tailwater levels, combined with higher upstream flood levels, result in bridge afflux typically between the range of 0.05 and 0.20 m. This is within the range that was expected for typical flood conditions (i.e. where the downstream water level does not exhibit a significant control over the hydrodynamics).
- Water levels upstream of the causeway are mainly influenced by the interaction of large catchment runoff events and backwater from the causeway. In comparison, water levels downstream of the causeway are primarily influenced by the tidal levels at the site and the degree to which the entrance is shoaled. For lower ocean levels, the immediate entrance shoals cause most of the friction losses along Burrill Inlet.

Following this investigation of hydrodynamic behaviour, it was concluded that the TUFLOW model predicted losses through the bridge correctly.

5.5 Model Calibration and Validation

5.5.1 Introduction

As described in Section 5.2.1, the hydrological model cannot be calibrated in isolation due to the absence of gauged flow data. Standard parameters have therefore been adopted in the hydrologic model, with no attempt to vary those parameters to modify the hydrologic model output.

Results from the hydrologic model provide runoff flow rates for the calibration and validation events. The runoff hydrographs are input into the hydrodynamic model at 10 different locations around Burrill Lake.

A conventional hydrodynamic calibration exercise involves modifying the hydraulic energy loss in the model until agreement is achieved between predicted and measured flood levels. In the case of Burrill Lake, the majority of energy loss occurs within the entrance channel (i.e. Burrill Inlet). The amount of energy loss within the channel controls flood levels within the upstream basins of the Lake. Factors that influence energy loss within the channel include the flow conditions (and associated velocity), the degree of entrance closure, the causeway, and bed friction.

The water level profile along the channel is a function of the energy loss, and is largely controlled by the bed roughness. The roughness of overbank floodplains in urbanised or forested areas has only a negligible impact on water levels, as these areas convey only a minor proportion of the flood flows, and at considerably slower velocities than in the entrance channel. In other words, overbank floodplains essentially act as flood storage areas rather than flow conveyance areas.

Unfortunately, some of the data required to perform a conventional calibration is not available, requiring best estimates and default values to be used for some input parameters, as discussed in Section 5.2. A summary of the missing information is shown graphically in Figure 5-1.

5.5.2 Model Roughness

Initial hydraulic analysis shows that the entrance channel behaviour, comprising the effects of entrance closure and the impact of the causeway, is the critical parameter influencing flood levels in Burrill Lake.

Ideally, a spatial spread of historical flood level data along the entrance channel describing the actual water slope would be used to calculate the required model bed roughness (expressed in terms of a Manning's 'n' value) using standard hydraulic backwater equations (eg Manning's equation). Unfortunately, the lack of measured points for calibration and the fact that these points tend to be clustered within a limited area, limits the direct calculation of channel roughness in Burrill Inlet.

For a similar flood study carried out for Lake Conjola (WBM, 2005), which is approximately30 km to the north of Burrill Lake, channel bed roughness (Manning's 'n') values of between 0.023 and 0.025 were determined based on historical flood event. Given the similarities between Lake Conjola and Burrill Lake, the same channel roughness values were adopted as an initial estimate (a Manning's 'n' value of 0.020 was actually adopted for the 2D model, as the model directly integrates losses due to flow meandering, contraction and expansion, which is not accounted for specifically in the Manning's equation).

Roughness values for every element of the hydrodynamic model were given an initial estimate, based literature (Chow, 1959; French, 1985) and past experience using the same computational package (TUFLOW). Care has been taken in interpreting literature values, as some published roughness values relate to one-dimensional hydraulic analysis (and as such, inherently account for factors that are determined directly by a two-dimensional approach).

An initial hydrodynamic model simulation of the 1971 event was run using the estimated roughness values, and the predicted peak water levels compared to the measured levels at the corresponding flood marks. Adjustment of the roughness parameters was subsequently undertaken to improve the fit between the predicted and measured peak flood levels.

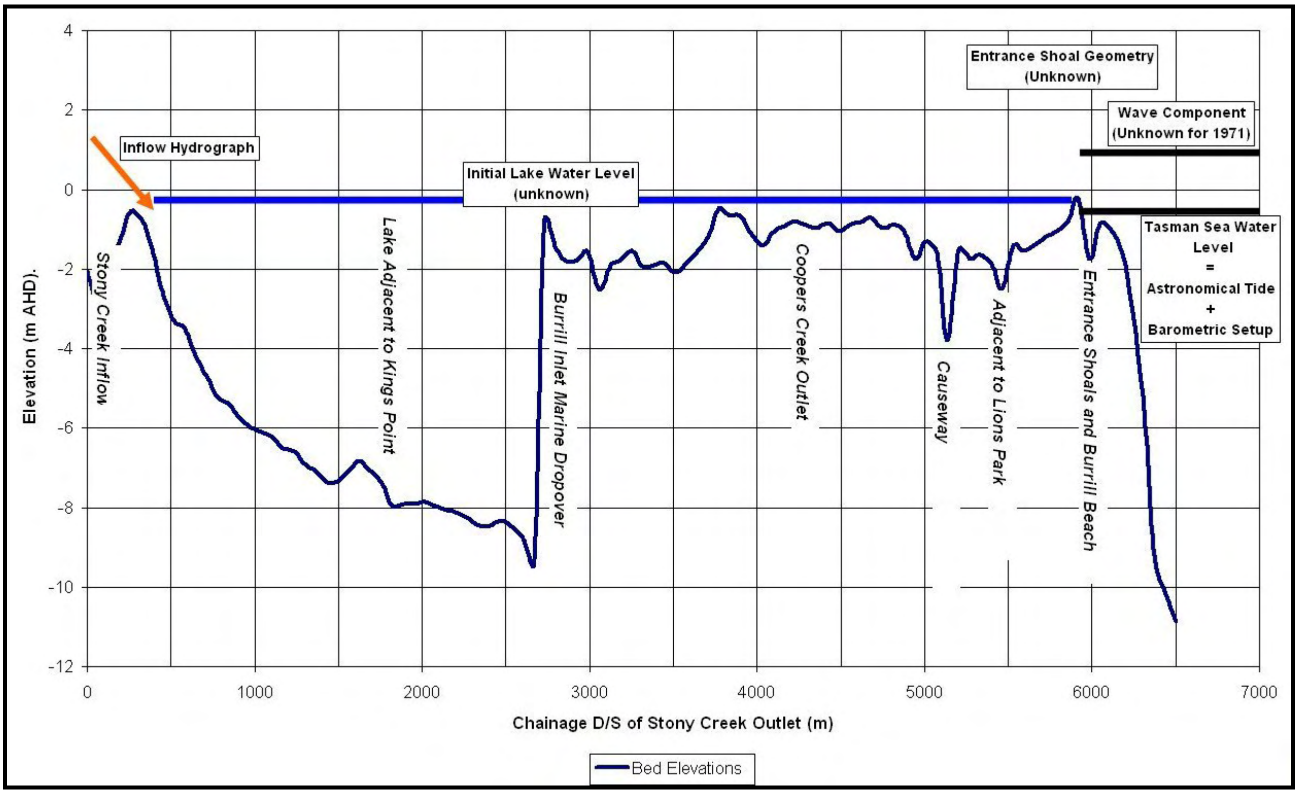


Figure 5-1 Longitudinal Profile Showing Aspects where Calibration Data is Missing or Incomplete

The roughness (equivalent Manning's 'n') values used in the model that gave the best fit between predicted and measured water levels for both the 1971 calibration event and the 1992 validation event were:

- n = 0.022 deeper water and entrance channel
- n = 0.020 for roads within the floodplain;
- n = 0.030 crest of the causeway;
- n = 0.030 shoals and near shore shallows;
- n = 0.200 residential areas;
- n = 0.200 caravan parks;
- n = 0.060 parkland;
- n = 0.080 wetlands;
- n = 0.030 mobile entrance area; and
- n = 0.025 rock in the vicinity of the entrance.

The results of calibration and validation are presented in Section 5.5.3.

Interestingly, the calibration exercise found that the peak flood levels predicted by the model are relatively insensitive to the roughness values adopted for residential areas and caravan parks. The adopted values for these areas are likely, however, to have a more localised impact, particularly in respect to flood velocities. For example, if the residential lots in the model had a higher roughness value, then flows would tend to concentrate more within the streets. This would clearly impact on the flood velocities within the streets, and thus the associated flood hazard (which may have implications for evacuation and more general floodplain risk management – to be investigated as part of the subsequent Floodplain Risk Management Study).

5.5.3 Presentation of Calibration and Validation Results

The calibration and validation process has demonstrated that the hydrodynamic model of Burrill Lake can reproduce historical floods, whilst using a combination of realistic / sensible model assumptions. Unfortunately, given the level of uncertainty relating to the historical input data, the results of the model calibration and validation still need to be considered with caution.

Figure 5-2 presents the results of the model calibration and validation as a longitudinal profile of peak water levels through the Lake and entrance channel. Figure 5-3 and Figure 5-4 present maps of predicted inundation within Burrill Inlet and associated historical flood mark data for the 1971 calibration event and the 1992 validation event, respectively. As discussed previously, the level of reliability of the historical flood marks varies. Comments regarding the historical flood marks used for calibration and validation are provided in Appendix A.

The calibration and validation results are considered satisfactory, as the model has been shown to predict historic levels to mostly within +/- 0.1m using measured input parameters where available and realistic estimates of other parameters, as discussed previously.

5.6 Calibration and Validation Conclusion

A conventional calibration and validation of the flood models was not possible for the Burrill Lake Flood Study due to the combination of unknown historical conditions, as discussed in Section 5.2. To overcome this limitation, a variety of alternative approaches were undertaken in order to test the reliability of the model against measured flood data, including:

- Testing of hydraulic behaviour using other hydraulic analysis methods (e.g. used in examining the afflux across the causeway bridge);
- Use of literature values and previous experience; and
- Use of standard or default parameters within model software (e.g. used in the hydrologic model).

As presented in Section 5.5.3, this approach has enabled a satisfactory fit between predicted and measured historical flood data, and thus provides confidence in the model for application of design flood events.

The methods used during this study, including the software packages Rafts-XP, TUFLOW and the Van Rijn sediment transport methodology have been applied previously to numerous other flood studies within Australia and overseas, with acceptable results.

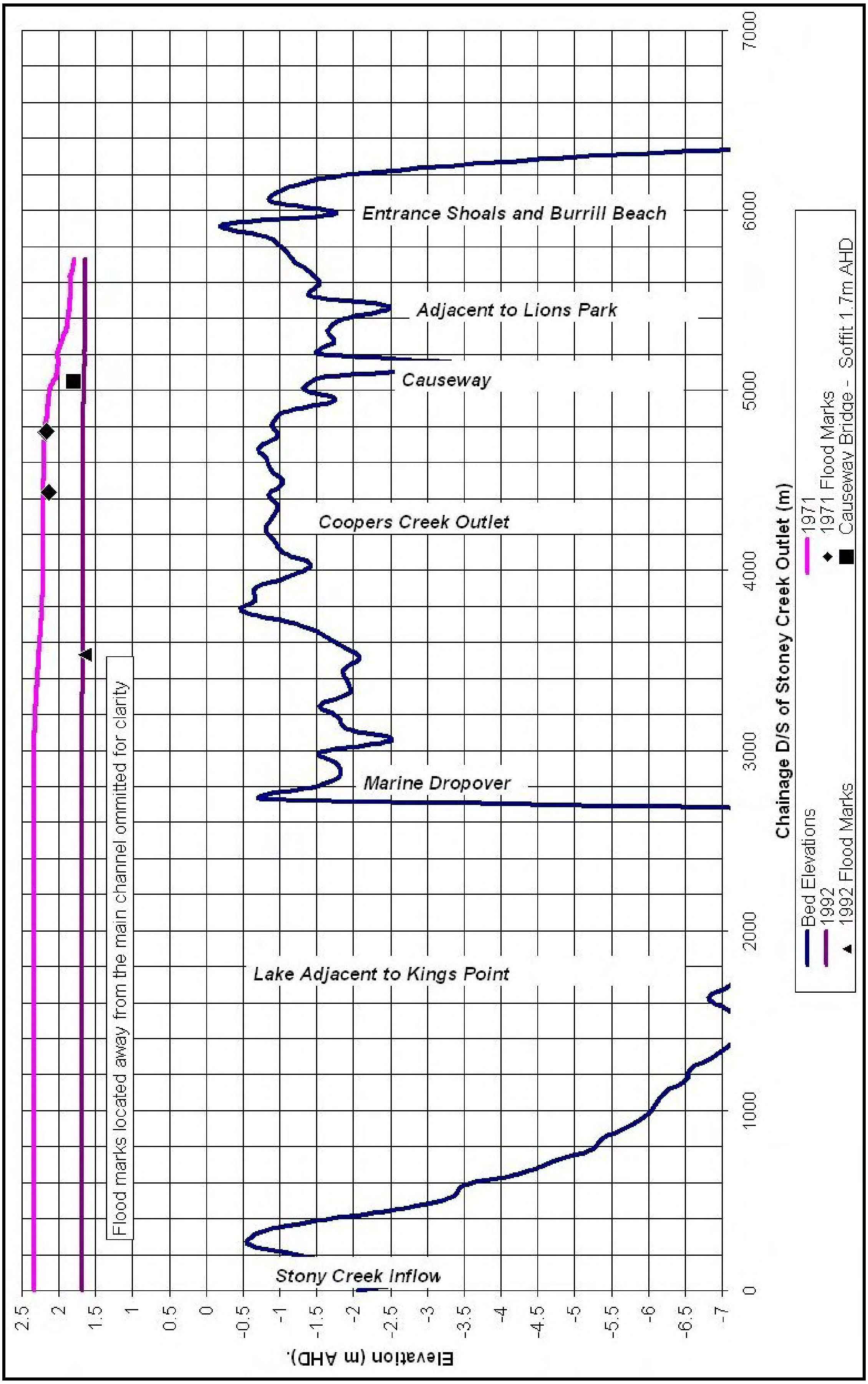


Figure 5-2 Computed Longitudinal Profiles for Calibration and Validation Events

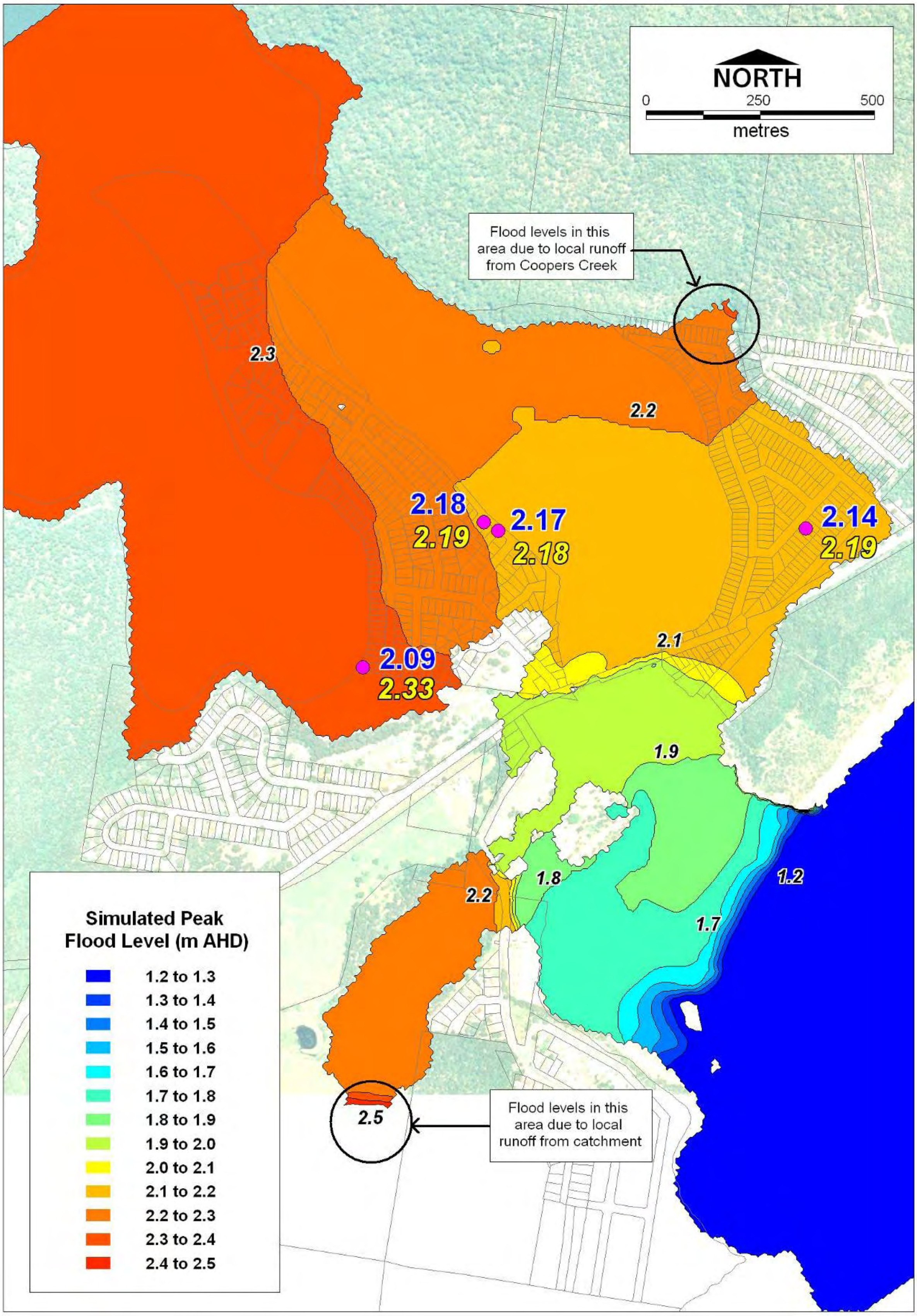


Figure 5-3 1971 Calibration Flood Level Map

(Simulated Values in Yellow, Historical Levels in Blue)

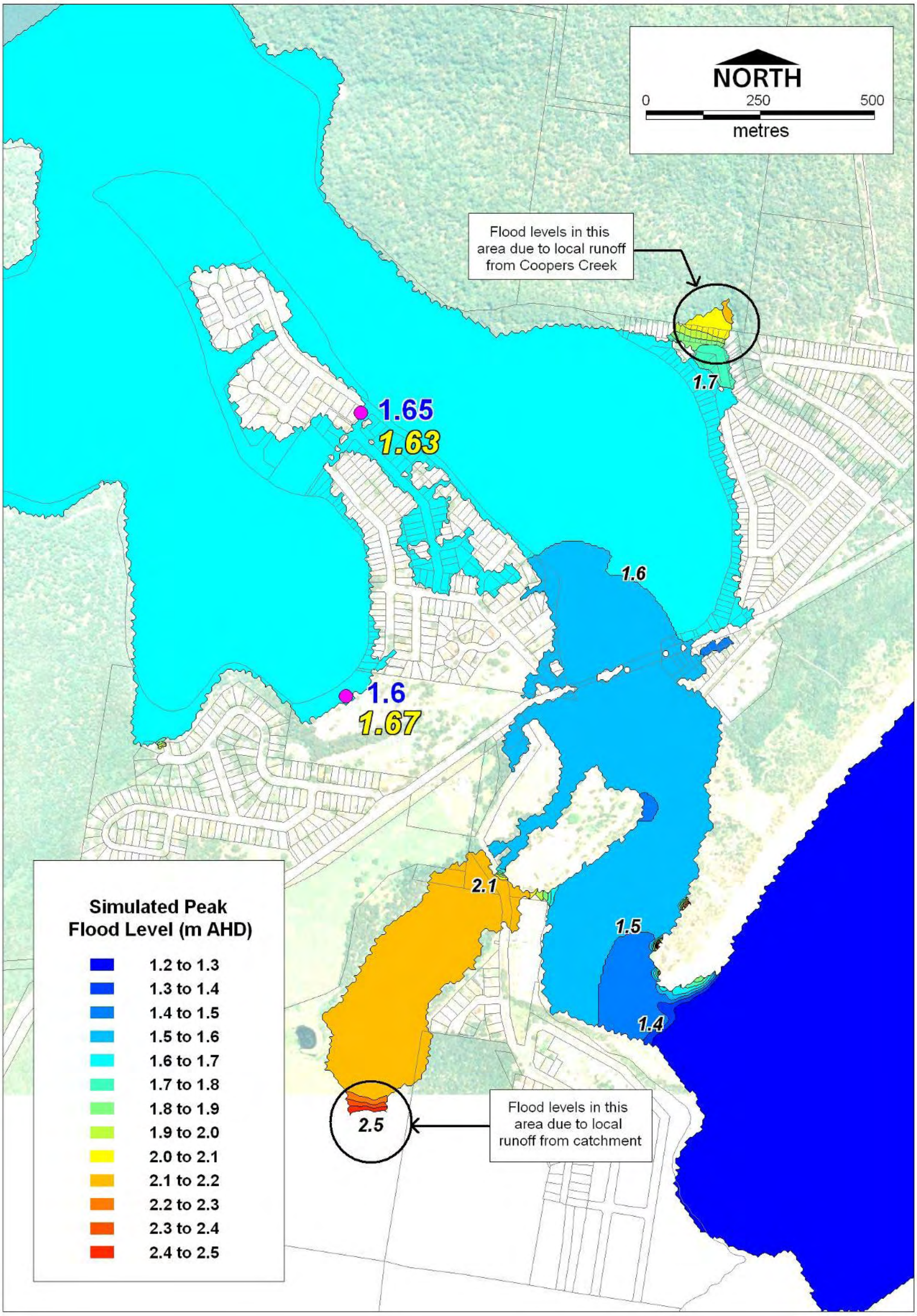


Figure 5-4 1992 Validation Flood Level Map

(Simulated Values in Yellow, Historical Levels in Blue)

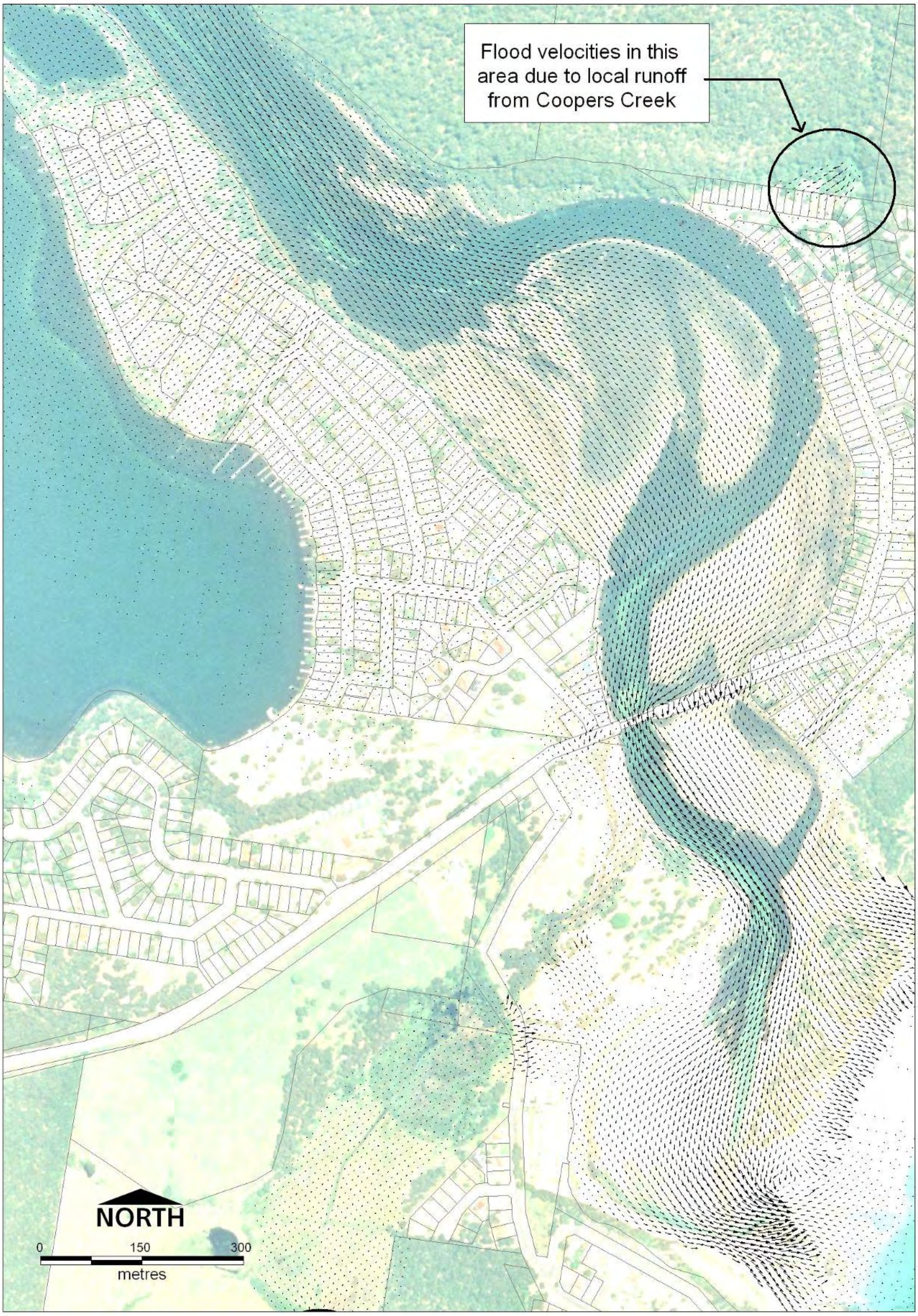


Figure 5-5 1971 Validation Flood Velocity Map

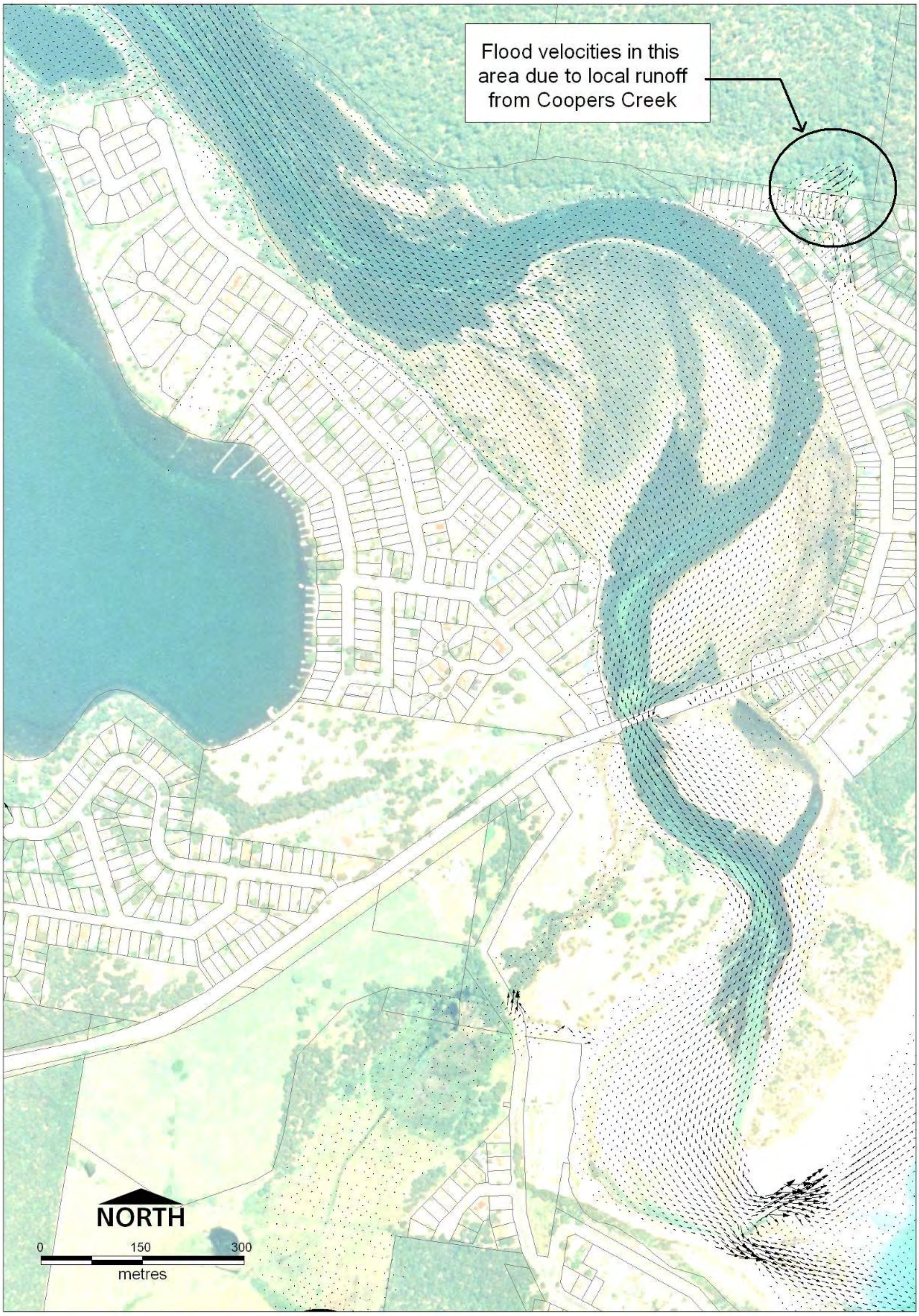


Figure 5-6 1992 Validation Flood Velocity Map

6 MODEL TESTING AND SENSITIVITY

6.1 Introduction

Following calibration and validation of the hydrologic and hydrodynamic models, additional testing was undertaken to examine the sensitivity of the models. The following process was adopted:

1. The results from the hydrologic model were compared with results derived using the Rational Method (refer Section 6.2);

2. Sensitivity testing of the hydrodynamic model was undertaken by adjusting various model parameters (refer Section 6.3);

3. Based on steps 1 & 2, the final model parameters to be used for model design runs were determined (refer Section 6.4).

6.2 Hydrologic Model Testing

6.2.1 Rational Method

As the accuracy of the hydrologic model could not be directly verified against historical flow measurements, it was necessary to compare model results with a probabilistic method of flood prediction.

The Rational Method is an accepted Australian standard of peak flood discharge determination when no detailed data is available. The Rational Method is a statistical method used in estimating design peak flood flows. It is used to estimate the peak flow of a selected Average Recurrence Interval (ARI) from an average rainfall intensity of the same ARI.

The Rational Method is presented in Australian Rainfall & Runoff (AR&R), Book IV (Institution of Engineers Australia, 2001). The basic formula is:

$$Q_Y = 0.278 C_Y I_{t_c Y} A$$

where Q_Y = peak flow rate (m^3/s) of average recurrence interval (ARI) of *Y* years

C_Y = runoff coefficient (dimensionless) for ARI of *Y* years

A = area of catchment (km^2)

$I_{t_c Y}$ = average rainfall intensity (mm/hr) for design duration of t_c hours and ARI of *Y* years.

The equation shows that the value of Q_y is dependent on the duration of rainfall. Therefore, a (critical) design rainfall duration must be specified as part of the procedure. The critical rainfall duration is t_c, and is considered to be the travel time from the most remote point of the catchment to the outlet. In other words, the critical duration is the time taken from the start of the rainfall event until all of the catchment is contributing simultaneously to the outlet flow. The critical time of concentration for Eastern New South Wales is calculated with the following formula:

$$t_c = 0.76A^{0.38}$$

where t_c = Time of concentration (hours)

A = area of catchment (km^2)

The rainfall intensities corresponding to the calculated time of concentration (for a given ARI) are determined using methods presented in Book II, Section I of Australian Rainfall and Runoff (Institution of Engineers Australia, 2001). Similarly, the runoff coefficient, C_Y, was determined from Book IV of the same publication, based on the specified ARI.

6.2.2 Comparison of Results to Rational Method

Using the methods detailed above, peak discharges from the mouth of Stony Creek were calculated. As the Rational Method cannot incorporate the storage effects of Burrill Lake, model testing was limited to the Stony Creek outlet only. The outlet of Stony Creek is the principal inflow location for the hydrodynamic model, and represents nearly three quarters of the entire Burrill Lake catchment.

The Stony Creek catchment area was determined to be 46.5 km^2, while the critical time of concentration was calculated to be approximately 196 minutes (3.3hrs).

Simulated peak discharges from the Stony Creek outlet, resulting from design rainfalls calculated within RAFTS-XP, were also determined. For this analysis, storm durations of 60 min, 90 min, 120 min, 3 hr, 6 hr, 9 hr, 12 hr, 18 hr, 24 hr, 30 hr, 36 hr, 48 hr and 72 hr were tested for ARI's of 5, 10, 20, 50 and 100 yrs. A comparison between the maximum peak discharges for each ARI (i.e. for the critical storm duration) and the corresponding value calculated from the Rational Method are presented in Table 6-1.

Table 6-1 Comparison of Stony Creek Peak Flows using RAFTS-XP and Rational Method

ARI	RAFTS-XP (m^3/s)			Rational Method		
	Critical Duration (hours)	Intensity (mm/hr)	Q (m^3/s)	I (mm/hr)	C	Q (m^3/s)
100 yr	9	27.1	435	50	1.15	746
50 yr	9	24.0	380	44.8	1.07	619
20 yr	18	13.3	316	37.6	0.99	481
10 yr	18	11.4	264	32.2	0.90	374
5 yr	18	9.81	221	28	0.81	293

It can be seen from Table 6-1 that the rainfall intensities used by the Rational Method are much higher than those used in the critical events determined by the simulations undertaken in RAFTS-XP. This is due to the longer critical storm durations determined in the RAFTS-XP model.

The critical duration for the three more frequent events (5, 10 and 20 yr ARI) is 18 hours compared to 9 hours for the less frequent events (50 and 100 yr ARI). This is because AR&R specifies a different temporal pattern for events of less than 30 yr ARI than those with greater than 30 yr ARI. It should be noted that the critical durations for flow at the mouth of Stony Creek are different than the critical durations for water levels within the Lake and Burrill Inlet. The Lake has a significant storage and flow attenuation effect, resulting in much longer critical durations when considering resulting Lake water levels.

The peak discharge values calculated by the hydrologic model are between 60 and 75% of the discharge values calculated by the Rational Method. Such discrepancy is no uncommon when comparing results of the Rational Method with results from more rigorous modelling. The Rational Method is probabilistic in nature, based on data from 308 gauged catchments. Also, there is no allowance for catchment slope in the calculation of the critical time of concentration.

Although there were differences in results between the two methods, these differences were considered to be accountable, and the RAFTS-XP model is considered appropriate for the purpose of design flood simulation.

6.3 Hydrodynamic Model

6.3.1 Introduction

Sensitivity testing of the hydrodynamic model was undertaken using the following methodology:

1. Derivation of a 'base case' scenario, representing the 100 yr ARI catchment flood. This involved determining the critical storm duration for the 100 yr event, and then coinciding the peak of that storm with the peak of a design ocean tide. This process is detailed in Section 6.3.2.

2. Undertaking twelve different simulations that varied from the base scenario by changing selected model variables to test the impact of those variables on model results in isolation. The resulting changes to peak flood levels and velocities at various locations within the hydrodynamic model were assessed both spatially (refer Appendix C for figures) and at a number of specific locations as listed in Table 6-2 and shown on Figure 6-1. The nature of the twelve sensitivity tests undertaken and the resulting variation of velocities and peak flood levels are described in Sections 6.3.3 through 6.3.14.

6.3.2 Derivation of Base Case for Comparison

The characteristics of the model used for sensitivity testing was essentially the same as the model used for calibration and validation, with the following exceptions:

- Ground levels were upgraded to represent typical present day conditions. In particular, this involved modification of the 1992 model bathymetry to include more recent survey data from

March 2001. During March 2001, the entrance was open, but constrained. This has been the normal condition of the entrance in recent years.

- An ocean tidal boundary was derived in accordance with the requirements of the Draft "Floodplain Management Guideline No. 5 – Ocean Boundary Conditions", issued by DNR. Essentially, this comprises a semi-diurnal tide with a peak water level of 0.6 m AHD.
- Design 100yr ARI rainfall was applied to the hydrologic model. Average rainfall intensities were determined using standard Intensity-Frequency-Duration (IFD) analyses as specified in Institution of Engineers Australia (2001). These were combined with design temporal patterns of rainfall within the hydrologic model to determine flows from the catchment. Derivation of the design rainfalls is described in more detail in Section 7.3.

The initial step in deriving the base case involved determining the critical storm duration. A series of model scenarios, representing 100 yr design rainfall events for 3 hr, 6 hr, 9 hr, 12 hr, 18 hr, 24 hr, 30 hr, 36 hr, 48 hr and 72 hr were undertaken and a profile of peak water levels extending from the mouth of Stony Creek through the northern basin of the Lake and down Burrill Inlet to the Entrance was plotted for each scenario. The peak discharges at the mouth of Stony Creek (extracted from the Rafts-XP model) are shown in Table 6-2. The resulting water profiles are shown on Figure 6-2, and show that the 36 hr storm is the critical duration storm for a 100 yr ARI design event flood levels (compared to 18 hours, being the critical storm duration for discharge from Stony Creek, refer Table 6-2).

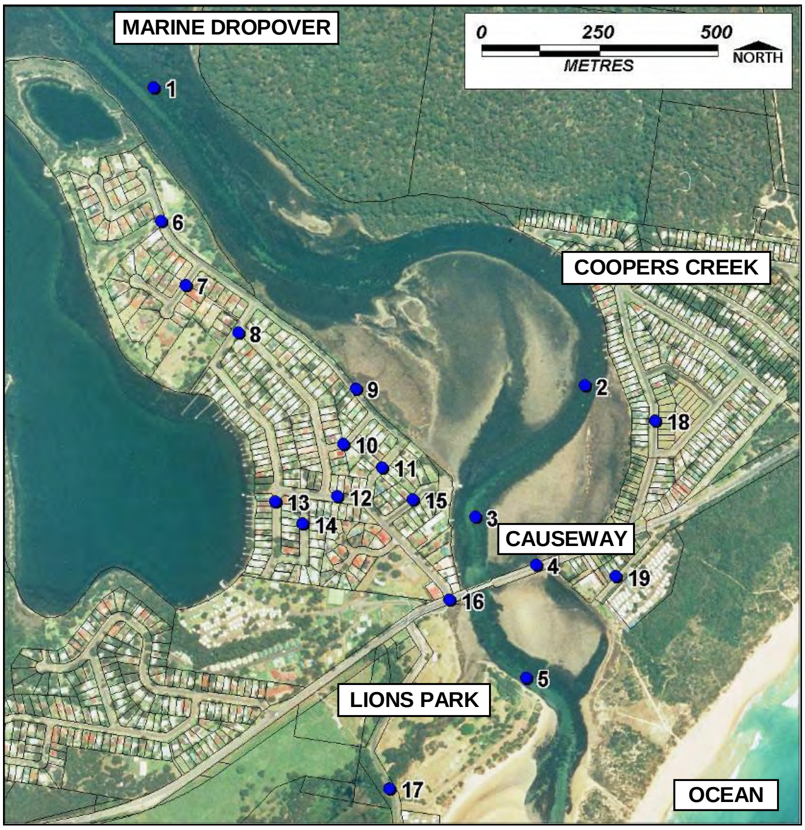


Figure 6-1 Points for Tabulated Comparison of Velocities and Water Levels

Table 6-2 Critical Storm Duration Analysis – Stony Creek Outlet Peak Discharge

Storm Duration	3 hrs	6 hrs	9 hrs	12 hrs	18 hrs	24 hrs	30 hrs	36 hrs	48 hrs	72 hrs
100 year ARI Peak Flow at Stony Creek Outlet (m^3/s)	340	400	435	405	425	390	350	395	375	330

Table 6-3 Description of Points for Tabulated Comparisons

Point Number	Description
1	Burrill Inlet North
2	Burrill Inlet East
3	Causeway Approach
4	Causeway
5	Causeway Exit
6	Ireland St.
7	Honeysuckle Cl.
8	Moore St.
9	Rackham Cr.
10	Ronald Ave
11	Rackham Cr. South
12	Balmoral Rd.
13	Lake View Dr.
14	George St.
15	Maria Ave.
16	Princes Highway
17	Dolphin Point Rd.
18	McDonald Ave.
19	Princess Ave. South

It should also be highlighted that the critical storm duration analysis was undertaken with the geomorphologic model operational. Differences in longitudinal profiles shown in Figure 6-2 relate to the rate of entrance scour, rainfall intensity, rainfall temporal patterns, and the storage effect of the Lake.

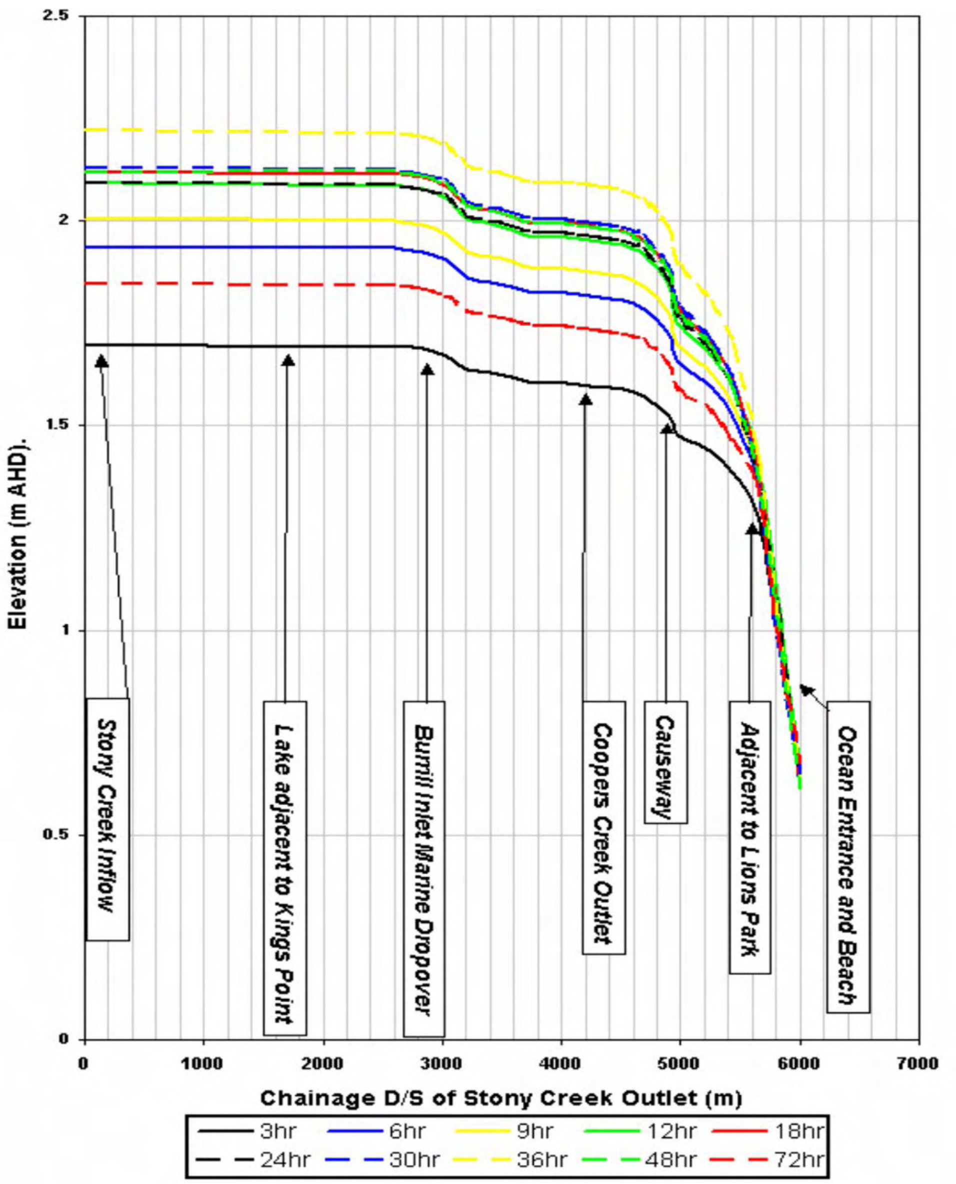


Figure 6-2 Results of Critical Duration Analysis

(For Locations see Figure 6-1)

A plot of peak water levels in the Northern Basin of the Lake versus durations is provided in Figure 6-3. For the critical duration simulations, a static ocean water level of 0.6 m AHD, representing the peak of the required design ocean boundary condition was used. In this way, the effects of a variable tide were eliminated and the critical duration storm from catchment processes only was determined. For the sensitivity analysis, however, it was necessary to apply a variable water level boundary to represent the design ocean boundary condition. In accordance with DNR's draft guideline No. 5, the peak of the ocean tide was required to coincide with the peak of the flood to cause the maximum flood levels. Accordingly, the simulations were rerun with this variable ocean boundary, ensuring that the peak considered a combination of flooding and ocean tides.

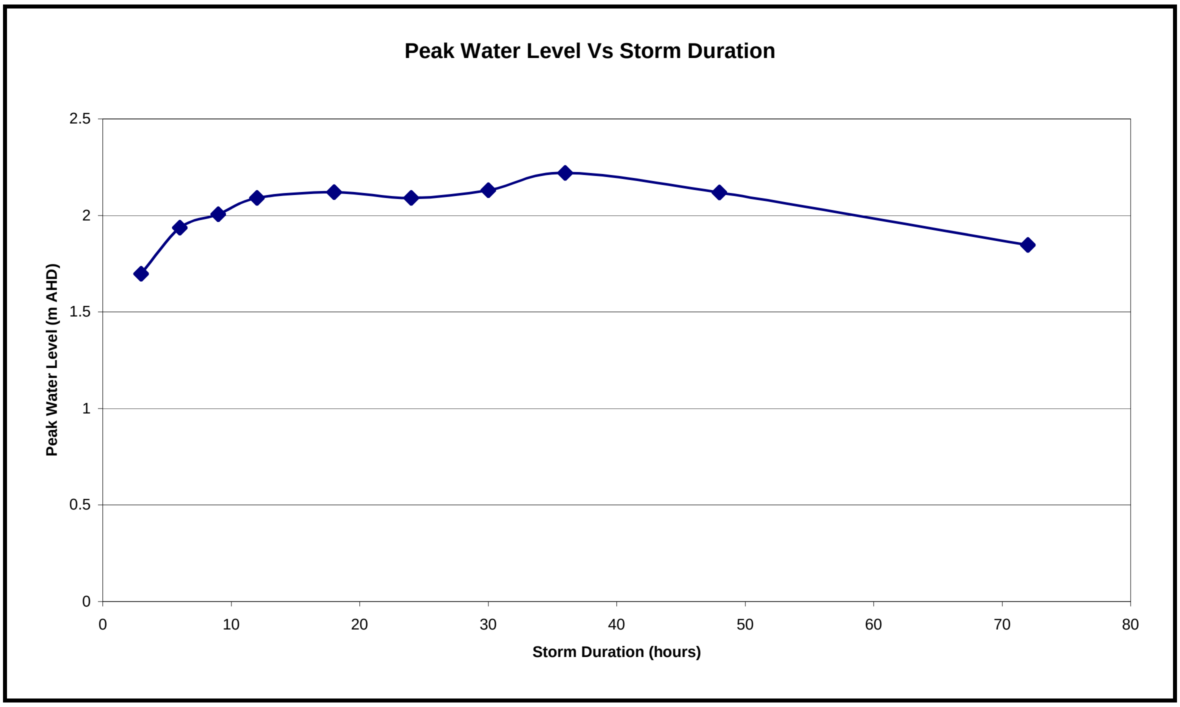


Figure 6-3 Peak Lake Water Levels for Different Storm Durations

Inspection of the results from the 36hr duration storm model simulation indicated that the peak due to catchment flooding occurred 23 hours into the simulated flood. Given the tidal lag between the ocean and the Lake, it was estimated that the peak tidal levels in the Lake would occur approximately 21 to 23 hours from the start of the simulated flood. Five additional simulations were carried out, with ocean peak water levels at 21, 21.5, 22, 22.5 and 23.0 hours (refer Figure 6-4), in order to determine the most critical coincident timing of the ocean tidal boundary conditions and the catchment flooding conditions. From this analysis, it was evident that the 21.5 hr Ocean Peak timing provides the highest peak water levels in the Lake when combined with the 36 hr storm duration (critical duration) 100 yr ARI design flood. Accordingly, this combination of catchment and ocean conditions was adopted as the base case simulation.

Figures showing the peak water levels and peak velocities during the base case flood are provided in Appendix C as Figure C.3 and Figure C.4 respectively. Velocity vectors are provided as Figure C.5 and Figure C.6. A general description of the peak flow directions for the base case is as follows:

- Flows across the Bungalow Park residential area are generally from west to east, with flows tending to concentrate along streets that are typically aligned in an east-west direction;
- Where Rackham Crescent lies adjacent to Burrill Inlet, the peak velocity is aligned in the direction of the inlet (with flows from the Lake to the Ocean);
- Flows cross Ireland Street from west to east to the south of Casuarina Close;
- Flows cross Dolphin Point Road from west to east near Lions Park;
- Flows also cross Dolphin Point Road and flow between Lions Park and Dolphin Point Caravan Park (to the south and east of Point 17 in Figure 6-1);

- Flows are concentrated along Princess Avenue South (to the south and east of the causeway) with flow from east to west;
- Flows in the Burrill Lake residential area are generally less than 0.1 m/s except adjacent to Burrill Inlet where they are slightly higher. Flows within this residential area are in the direction of the main inlet flow.

The simulated peak water levels and velocity magnitudes at the selected locations are provided in Table C.1 and Table C.2, respectively. Where appropriate, the identifying number for sites being described in the following sections is provided in brackets to assist when referring to the figures in Appendix C.

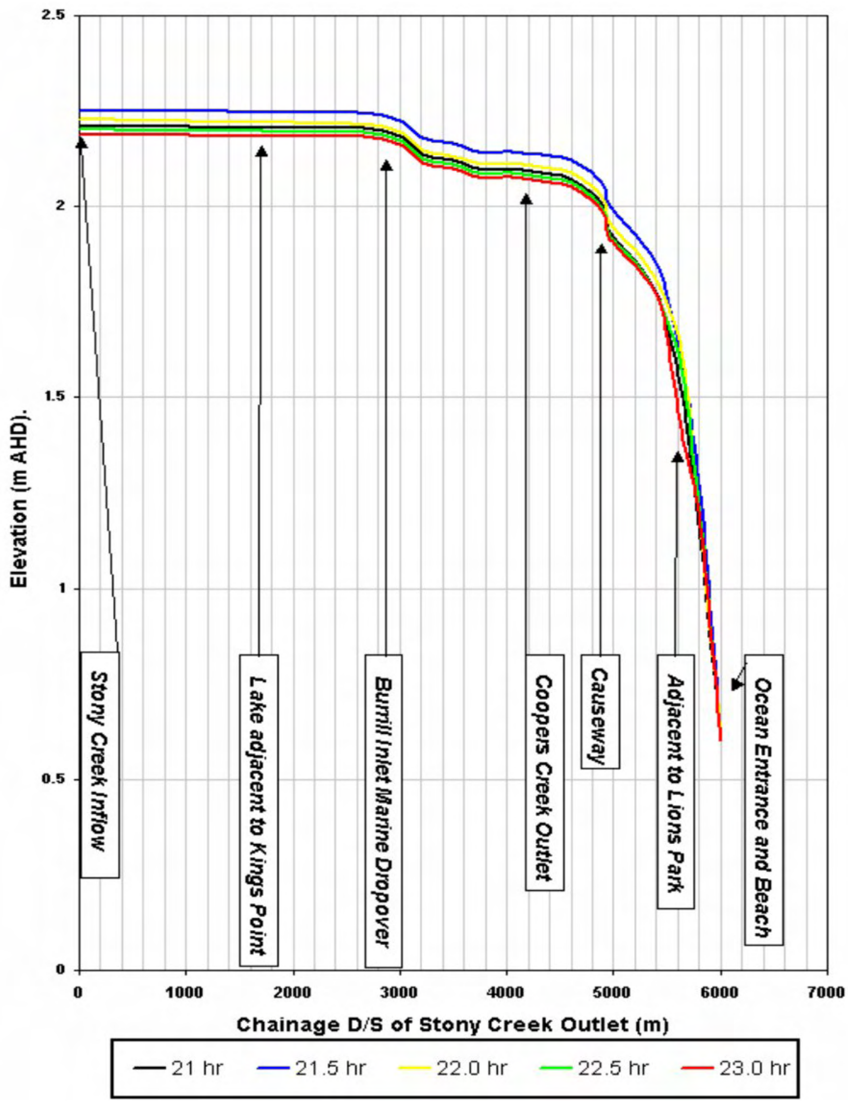


Figure 6-4 Results of Critical Peak Timing Analysis
(For Locations see Figure 6-1)

6.3.3 Sensitivity Test 1: Reduced Infiltration Losses

6.3.3.1 Description

This simulation represents the effects of a saturated catchment prior to the onset of the design flood rainfall. The variables altered in the hydrologic model included the continuing loss (set to 0 mm/hr compared to 2.5 mm/hr for the base case) and the initial loss (set to 10 mm compared to 15 mm in the base case).

6.3.3.2 Assessment of Results

The lowered infiltration values resulted in more flows from the catchment and consequent higher water levels in the Lake. Increases of up to 0.10 m were calculated upstream of the causeway and up to 0.06 m downstream of the causeway.

Velocity increases were also predicted, but were typically quite small (i.e. < 0.03 m/s). Large velocity increases were predicted across the crest of the Causeway (4). Velocities also increased along the streets within Bungalow Park, and in the main channel downstream of the Causeway Bridge (5). There were some reductions in flows predicted at the fringes of the main channel downstream of the Causeway Bridge, seemingly caused by enhanced scour in the entrance compartment which had the effect of concentrating flows along the main channel.

6.3.4 Sensitivity Test 2: Increased Infiltration Losses

6.3.4.1 Description

This simulation represents the effects of a very dry catchment prior to the onset of the design rainfall. The variables altered in the hydrologic model included the continuing loss (set to 5 mm/hr compared to 2.5 mm/hr for the base case) and the initial loss (set to 30 mm compared to 15 mm in the base case).

6.3.4.2 Assessment of Results

The increased infiltration values resulted in less flow from the catchment and consequent lower water levels in the Lake. Decreases in water level of between 0.10 and 0.17 m were predicted throughout the model. No comparative values are available for the Honeysuckle Close location (7) as it did not become inundated by this simulation.

Velocity decreases were also predicted, but were typically quite small (i.e. < 0.05 m/s). Larger velocity decreases were predicted at Ireland St (6: -0.53 m/s), across the Causeway (4: -0.23 m/s) and across the Princes Highway (16: -0.19 m/s). All of these values represent a reduction in flow velocities due to a decrease in hydraulic head driving the flow. Velocities also decreased along the streets within Bungalow Park, and in the main channel downstream of the Causeway Bridge (5). There were some predicted increases in flows at the fringes of the main channel downstream of the Causeway Bridge, possibly caused by a reduction in scour in the entrance compartment, which had the effect of spreading flows more evenly along the main channel. In addition, very small increases in peak velocity were predicted in the main channel adjacent to McDonald Avenue (2), as the lowered water levels would have encouraged more flow to pass through this area, instead of short-circuiting across the shoals adjacent to Rackham Crescent.

6.3.5 Sensitivity Test 3: Reduced Causeway Loss Coefficient

6.3.5.1 Description

This simulation tested the effects of the loss coefficient for the bridge at the causeway, which is one of the main controls on flood levels in Burrill Inlet. The loss coefficient affects the energy loss predicted through the bridge. For this test, the loss coefficient was set to 0.1 (compared to 0.2 in the base case), and represents the lower bound of reasonable values based on past experience.

6.3.5.2 Assessment of Results

Very minor reductions in water levels (less than 0.02 m) and velocities (typically less than 0.03 m/s) were predicted. There was a slight increase in velocities downstream of the Causeway (5 - due to more flow passing beneath the bridge) and a significant decrease at Honeysuckle Cl. (7: -0.13 m/s) due to the reduction in inundation at this location.

6.3.6 Sensitivity Test 4: Increased Causeway Loss Coefficient

6.3.6.1 Description

This test was similar to Test 3 except that the form loss coefficient was increased to 0.5 (compared to 0.2 in the base case), and represents the upper bound of reasonable values based on past experience.

6.3.6.2 Assessment of Results

Very minor increases in water level were predicted upstream of the Causeway (<0.02 m) and very minor decreases downstream of the Causeway (<1 cm). Significant velocity increases (0.1 m/s) were predicted across the Causeway (4) due to the increased water levels upstream of the Causeway. Furthermore, an increase in velocity (up to 0.3 m/s) was predicted near the Ireland St location (6) although this was not reflected at the location represented by Point 6 in Figure 6-1.

6.3.7 Sensitivity Test 5: Increased Roughness in Sediment Transport Model

6.3.7.1 Description

This simulation tested the effects of the bed roughness coefficient used within the sand transport equations associated with scour of the entrance. A higher bed roughness results in greater bed shear stress, sediment transport and resulting scour. For this test, the roughness coefficient was set to 0.15 (compared to 0.1 in the base case), which is considered the upper bound of reasonable values based on past experience and a review of Van Rijn (1993).

6.3.7.2 Assessment of Results

The simulation resulted in small reductions in water level throughout the model (<0.03 m). Very small velocity increases were also measured (<0.03 m/s) with slightly greater increases downstream of the causeway. A significant velocity reduction was measured at Honeysuckle Cl (7: -0.1 m/s) due to reduced inundation in this area. The higher roughness increases the rate of sediment transport, which accelerated the scour of the entrance, and thus resulted in lower water levels.

6.3.8 Sensitivity Test 6: Reduced Roughness in Sediment Transport Model

6.3.8.1 Description

This test was similar to Test 5 except that the bed roughness coefficient was reduced to 0.05 (compared to 0.1 in the base case), which is considered the lower bound of reasonable values based on past experience and a review of Van Rijn (1993).

6.3.8.2 Assessment of Results

Very small increases in water level (<0.02 cm) were predicted throughout the model. The reduction in scour has also impacted on the velocity distribution within the area of the model where scour has been simulated. Furthermore, an increase in velocity (up to 0.3 m/s) was predicted near the Ireland St (6) location although this was not reflected at the location represented by Point 6 in Figure 6-1.

6.3.9 Sensitivity Test 7: Reduced Roughness in Residential Areas

6.3.9.1 Description

This simulation tests the effects of the Manning's roughness coefficient within the residential and caravan park areas surrounding Burrill Inlet. Manning's roughness coefficient affects the resistance to flow across these areas. For this test, the Manning's roughness coefficient was set to 0.1 (compared to 0.2 in the base case). The roughness along roadways was maintained at 0.02 for this simulation.

6.3.9.2 Assessment of Results

Very small decreases in water level (<0.02 m) were predicted throughout the model. Moderate increases in velocity were predicted in Lake View Dr. (13), Moore St. (8), George St (14), and the Princes Highway (16). This is commensurate with a reduced roughness encouraging more flow across the residential areas. There was a decrease in velocity across the Causeway (4) due to the slightly reduced water level. Furthermore, an increase in velocity (up to 0.2 m/s) was predicted at the eastern end of the causeway, presumably resulting from more flow being encouraged through the Burrill Lake residential area to the north.

6.3.10 Sensitivity Test 8: Increased Roughness in Residential Areas

6.3.10.1 Description

This test was similar to Test 7 except that the Manning's roughness coefficient was increased to 0.3 (compared to 0.2 in the base case). The roughness along roadways was maintained at 0.02 for this simulation.

6.3.10.2 Assessment of Results

There was an overall reduction in peak water levels, although this was small (<0.02 m). Increases in current velocity were predicted throughout the main Inlet channel, (up to 0.04 m/s) with a corresponding decrease in current velocities throughout residential areas (typically < 0.05 m). This is commensurate with an overall increase in the resistance of the residential areas to flows.

It may seem counter-intuitive that both Test 8 and Test 7 caused small overall reductions in peak water levels throughout the model. This effect is due to the change in roughness in both cases offsetting the timing of the peak of the catchment flood. Accordingly, the peak of the catchment flood no longer coincides as well with the peak of the ocean tide. This highlights the fact that the small change in the relative timing of peak water levels (which causes a lowering of the water level) overrides the effect of increased roughness (which, *ceteris paribus* be expected to raise water levels in the Lake). It is likely that this trend would be reversed if higher roughness values were adopted (refer Test 12).

6.3.11 Sensitivity Test 9: 'Spring' Tide Ocean Boundary

6.3.11.1 Description

This simulation tested the effects of varying the peak of the downstream tidal boundary of the model. In this simulation, the tidal variation from DNR's Guideline No. 5 (which peaks at 0.6 m AHD) was replaced with a typical spring tide variation, which peaked at 0.7 m AHD.

6.3.11.2 Assessment of Results

For this test, the peak water levels decreased slightly throughout the model (< 0.02 m). The reason for this would seem to be related to fact that the peak water levels are not only influenced by the peak level reached at the ocean, but also by the preceding low tide level. As the preceding low tide level was lower than for the base case (given the typical spring tide variation, with higher highs and lower lows compared to the base case), the Lake had drained more prior to inundation by the inflowing (flood) tide. Regardless, the spring tide simulation was considered to have minimal impact on water levels.

Similarly, velocity variations were not affected significantly by the change to the ocean boundary. Of some note is a slight increase in velocities downstream of the causeway (5: -0.04 m/s) due to the flood wave receding in conjunction with a more rapidly falling tide.

6.3.12 Sensitivity Test 10: 'King' Tide Ocean Boundary

6.3.12.1 Description

This simulation also tested the effects of varying the peak of the downstream tidal boundary on the model. In this simulation, the tidal variation from DNR's Guideline No. 5 (which peaks at 0.6 m AHD) was replaced with a typical king tide variation, which peaked at 1.0 m AHD.

6.3.12.2 Assessment of Results

Increases in water level of up to 0.05 m were predicted within the main areas of interest. There was, however, a reduction in velocities, due to the incoming tide meeting the outgoing flood, resulting in the flood waters being held up in the Lake (and thus the increased water levels). Significant localised increases in velocity (up to 0.4 m/s) were predicted in a couple of locations within Bungalow Park that were not highlighted by the points shown in Figure 6-1 (Ireland St. and near the western end of Balmoral Rd.).

6.3.13 Sensitivity Test 11: 'Closed' Entrance Conditions

6.3.13.1 Description

This simulation tested the effects of a 'closed' entrance within the model. The base case used entrance bathymetry derived from the 2001 hydrosurvey undertaken by DLWC. In early 2005, the entrance to Burrill Lake closed. Council undertook a ground survey of the closed entrance. A digital elevation model of the closed entrance conditions was derived and input to the model for this simulation.

6.3.13.2 Assessment of Results

This test had significant impacts on flood levels in the Lake, with increases in water level between 0.25 to 0.30 m. One notable exception is the level at Dolphin Pt. Road, which is governed more by the water level of the Lagoon immediately to the east rising and overtopping the road.

Velocities tended to show a decrease, particularly downstream of the Causeway (5: -0.13 m/s), due to the impact of the closed entrance tending to hold back flood waters for a longer period when compared to the base case. However, within residential areas (e.g. Balmoral Rd (12), Maria Ave (15)) the tendency was for slightly increased velocities, presumably due to the more sudden drainage of these areas following breakout of the entrance. There were some notable areas where velocities increased by up to 0.2 m/s (Ireland St, Honeysuckle Cl, Casuarina Cl, Maria Ave and at the western end of Balmoral Rd). Peak velocities across Dolphin Pt Road (17) reduced by 0.59 m/s, due to downstream submergence of the road which acts like a free falling weir during the base scenario.

6.3.14 Sensitivity Test 12: Additional Increased Roughness in Residential Areas

6.3.14.1 Description

This test was similar to Test 8 except that the Manning's roughness on residential lots was increased to 0.6 (compared to 0.2 in the base case, and 0.3 in Test 8).

6.3.14.2 Assessment of Results

As for Test 8, there was an overall reduction in peak water levels with the reductions most pronounced in the vicinity of the causeway and areas downstream of the causeway. There was a very small decrease (< 1 mm) in the peak water level in the more upstream areas of the Lake, however, the flood levels took slightly longer to recede than in the base case. This may be partly attributed to the greater resistance introduced onto the residential lots, but may also be affected by the variation in timing of the catchment flooding and tidal peaks.

The change in peak water level is generally very small (< 0.03 m) with the exception of locations 16 (-0.04 m) and 19 (-0.05 m), on the Princes Highway, at either end of the causeway. The peak water levels at these locations were affected by increased roughness in the adjacent urban areas, which tends to discourage flow over the road embankment and encourage more flow through the main channel of the Inlet.

The flow velocities show a small reduction across the residential lots (a reduction in velocity of up to 0.08 m/s – Location 13) and small increases within the main channel of the Inlet (typically less than 0.05 m/s). Furthermore, there is a tendency for higher flood velocities along overland flow paths through the residential areas of Bungalow Park. This is mostly felt along Ireland St and Balmoral Rd (changes of up to 0.04 m/s, refer to Figure C-30 and Table C-2). Furthermore, there are velocity increases of up to 0.04 m/s along Macdonald Parade and Commonwealth Avenue in the residential areas to the east of the Inlet. At these two locations, however, the base case velocities are very small. The most marked change in velocity occurs at locations 16 and 19 at either end of the causeway over Burrill Inlet. Similarly to the effects on peak water levels at these locations, the added friction in residential areas retards flow with a reduction of up to 0.2 m/s in the vicinity of location 19 and up to 0.3 m/s in the vicinity of location 16 (refer figure C-30).

6.4 Determination of Design Model Parameters

Following the review of the model evaluation by staff from DNR, SCC and WBM, it was agreed to adopt the model parameters as used for the base case scenario, with the following modifications:

1. Lower infiltration losses to be used for all events up to the PMF. For design scenarios, initial loss is to be set at 10 mm, and continuing loss to 0 mm/hr, as per Sensitivity Test 1. In effect, this assumes a much wetter catchment than used during the calibration events, resulting in conservatively more catchment runoff flow and higher peak water levels.

2. Residential area roughness to be increased to 0.6, as per Sensitivity Test 12.

3. The closed entrance condition from early 2005, as adopted for Sensitivity Test 11, to be adopted for all catchment-derived flooding simulations.

4. The initial water level for the catchment-derived flooding simulations to be set at the maximum measured water level in early 2005 (which was 1.12 m AHD).

5. A scoured entrance condition to be adopted for the oceanic-derived flooding simulations. The scoured entrance condition is to be adopted as the resulting entrance bathymetry following a design 1 in 5 year ARI event.

6. Initial Lake water levels for the oceanic-derived flooding simulations to be established by simulating a continuous neap tide (of 0.6 m tidal range) ocean boundary until a consistent set-up in Lake levels is achieved. The initial Lake level is then adopted as the average Lake level (including the set-up component).

7 DESIGN FLOOD CONDITIONS

7.1 Introduction

Design floods are hypothetical floods used for planning purposes and floodplain risk management investigations. They are based on having a probability of exceedance specified either as:

- Annual Exceedance Probability (AEP) expressed as a percentage; or
- an Average Recurrence Interval (ARI) expressed in years.

Table 7-1 provides a description of the different design floods considered as part of this study.

Table 7-1 Design Flood Terminology

AEP	ARI	Comments
20%	5 years	A hypothetical flood or combination of floods which represent the worst case scenario likely to have a 20% chance of occurring in any one year, or in other words, is likely to occur or be exceeded once every 5 years on average.
10%	10 years	A hypothetical flood or combination of floods which represent the worst case scenario likely to have a 10% chance of occurring in any one year, or in other words, is likely to occur or be exceeded once every 10 years on average.
5%	20 years	A hypothetical flood or combination of floods which represent the worst case scenario likely to have a 5% chance of occurring in any one year, or in other words, is likely to occur be exceeded once every 20 years on average.
2%	50 years	A hypothetical flood or combination of floods which represent the worst case scenario likely to have a 2% chance of occurring in any one year, or in other words, is likely to occur or be exceeded once every 50 years on average.
1%	100 years	A hypothetical flood or combination of floods which represent the worst case scenario likely to have a 1% chance of occurring in any one year, or in other words, is likely to occur once or be exceeded every 100 years on average.
0.5%	200 years	A hypothetical flood or combination of floods which represent the worst case scenario likely to have a 0.5 % chance of occurring in any one year, or in other words, is likely to occur once or be exceeded every 200 years on average.
Extreme Flood / Probable Maximum Flood (PMF)		The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain.

In determining the design floods for Burrill Lake it is necessary to take into account the critical storm duration of the catchment. This is defined by storm duration that generates the highest flood levels and thus the most severe flooding conditions.

7.2 Source of Design Floods

The design flood conditions are derived from application of:

- Design rainfall parameters (rainfall depth, temporal pattern and spatial distribution-See Section 7.3);
- Design entrance channel geometry (See Section 7.5); and
- Design downstream ocean boundary levels (See Section 7.6).

7.3 Design Rainfall

Design rainfall parameters are sourced from the publication *Australian Rainfall and Runoff* by the Institution of Engineers Australia (AR&R, 1997/2001). AR&R (2001) contains statistical rainfall parameters covering all of Australia. These parameters are extracted for the location of interest (in this case for the Burrill Lake catchment) and used to produce design rainfall intensities (I) for varying frequencies (F) and durations (D) – (“IFD”).

About 7,500 daily rainfall stations with over 30 years of records and about 220 pluviometers with more than 12 years of record across all of Australia were used to derive the IFD parameters in AR&R (2001).

7.3.1 Design Rainfall Depths

The design rainfall depth represents the total rainfall over the catchment or a sub-catchment during the duration of the design storm. The design storm generally represents only the main burst(s) of rainfall and does not necessarily include the beginning and/or the end of the storm. The rainfall depth can be expressed as a total depth (mm) over the duration of the design storm, or as an average intensity (mm/hr).

7.3.1.1 5 year to 200 year ARI Events

As discussed in Section 6.3.2, the 36 hour duration storm was identified as being the critical storm for the base case scenario. However, with modification to some of the model parameters for design conditions (as discussed in Section 6.4), the critical duration was actually reduced to 18 hours. This is primarily due to the increase in initial water level, representing a more full Lake. That is, shorter and more intense rainfall events would have more impact in raising water levels before the entrance scoured sufficiently to release the flood waters.

The design rainfall depths for design flood events are calculated from the IFD coefficients of AR&R (2001). The IFD coefficients vary around the Burrill Lake catchment leading to small design rainfall intensity variability. As an example, Table 7-2 shows the 100 year ARI 18hr storm design rainfall intensities for the four geographical extremities of the catchment.

Table 7-2 Spatial Variability of 100 Year ARI Design Rainfall Intensities across Burrill Lake Catchment

Location in Burrill Lake Catchment	Rainfall Intensity (mm/hr)
East	18.1
West	20.3
North	20.0
South	18.1

Due to the relatively small variation in rainfall intensities across the catchment, the average rainfall intensity was applied to the entire catchment for all design events. Table 7-3 presents the design rainfall depths for the 18 hour critical duration storm for the Burrill Lake catchment. It should be noted that the rainfall depths shown in Table 7-3 have been reduced by the spatial distribution factor described in Section 7.3.3.

Table 7-3 Adopted Design Rainfall Depths at Burrill Lake for 18 hour Storm

Recurrence Interval (ARI)	Total Rainfall Depth (mm)	Rainfall Intensity (mm/hr)
5 year	187	10.4
10 year	220	12.2
20 year	259	14.4
50 year	315	17.5
100 year	335	18.6
200 year*	369	19.9

*The 200 year ARI design rainfall intensity was derived using the methods recommended for 'rare' events in Book 6 of AR&R (Section 3.6.2), as opposed to the other ARI design intensities which were derived from the procedures presented in Book 2 of AR&R.

7.3.1.2 Probable Maximum Flood (PMF) Event

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 (100 year ARI and less) to extrapolate the PMP.

The PMP can be estimated in a number of ways, with the method being chosen based upon catchment location, size and critical duration of the catchment, and the purpose for which the PMP is needed. Three methods are available:

- Generalised methods (Generalised Short Duration Method (GSDM), Generalised Southeast Australia Method (GSAM), Generalised Tropical Storm Method (GTSM) derived by the Bureau of Meteorology);
- The Bureau of Meteorology undertakes a detailed analysis; and
- Regional prediction equations in AR&R (2001).

The use of a PMF in a Flood Study is primarily to define the full extent of the floodplain and also to provide indicative levels for planning and evacuation purposes, and as such, exhaustive local analysis is not required. Therefore, the PMP for Burrill Lake was determined by the regional prediction preliminary estimate equations contained in AR&R (2001) Book 6.

For the purposes of estimating the PMP, the Burrill Lake catchment lies within the "Generalised Southeast Australia Method" (GSAM) zone. Regional prediction equations applicable to this zone are used for preliminary estimates of the PMP. These equations require the following variables:

- Catchment Area, A = 65 km^2;
- 1 in 50 AEP 72 hour rainfall intensity, ${}^{50}I_{72}$ = 7.75 mm/hr (Due to the high degree of approximation associated with PMP predictions, a spatially averaged value over the entire catchment was used); and
- Duration = 18 hours.

The regional prediction equation is provided for the 36, 24 and 12 hour durations in AR&R (2001, Book VI Table 9). Results from these equations were interpolated to determine the appropriate value for a storm of 18 hour duration.

The PMP for the Burrill Lake catchment based on AR&R (2001) preliminary estimation procedures is thus estimated to be 1062 mm, corresponding to an average rainfall intensity of 59.2 mm/hr (average value applied to the entire catchment). This rainfall depth is slightly more than three times the 100yr ARI rainfall depth.

7.3.2 Design Rainfall Temporal Patterns

Temporal patterns define the way in which the rainfall depth for an event is distributed during the event. For example, the 100 year ARI event has a rainfall depth of 335 mm and will be distributed non-uniformly throughout the 18 hour period defining that event.

For events up to and including the 100 year ARI event, Zone 1 temporal patterns were used as recommended in AR&R (2001, Book II Section 2). The 100 year ARI 18 hour storm temporal pattern for Burrill Lake is presented in Figure 7-1.

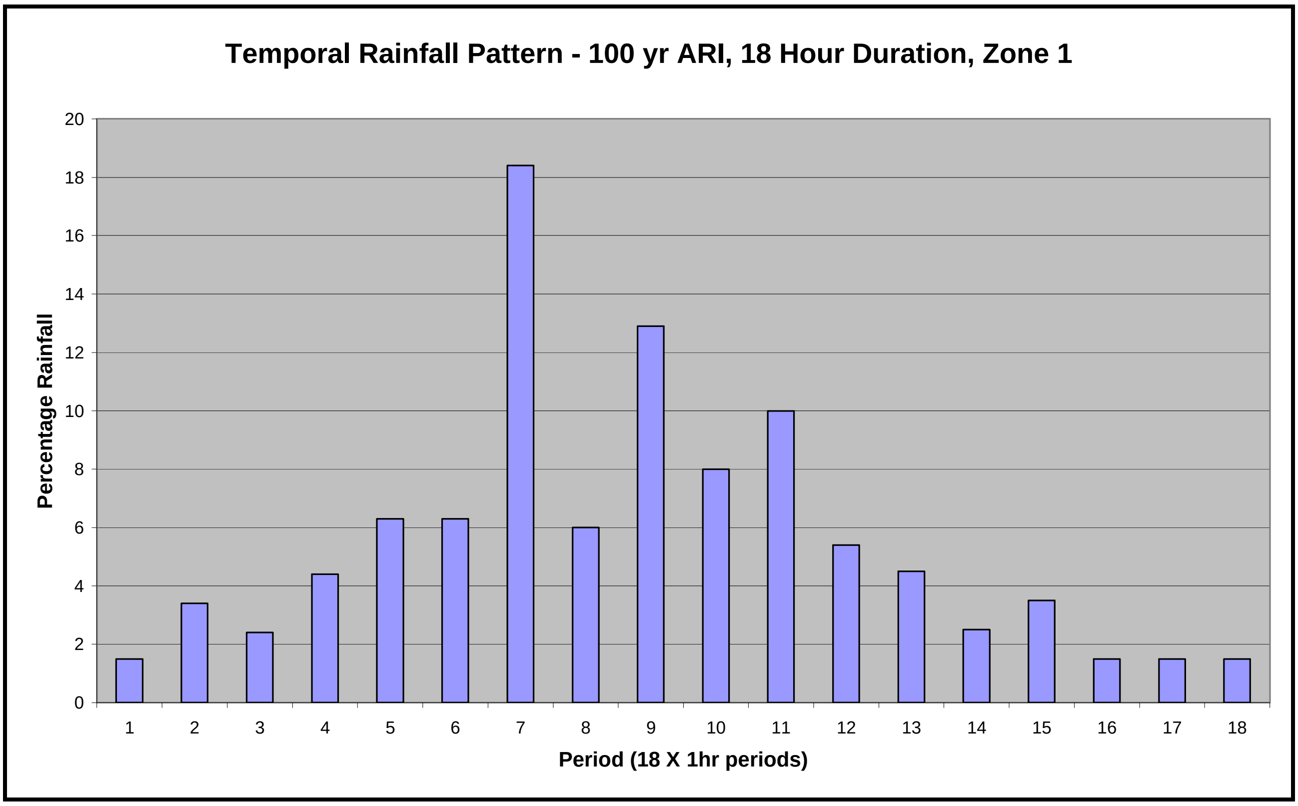


Figure 7-1 100 Year ARI 18 hour Storm Temporal Pattern (Source: AR&R, 2001)

For a 200 yr ARI event and 18 hr duration storm, AR&R (2001, Book VI Table 7) recommends weighting the unsmoothed 24 hr Generalised South-East Australian Method (GSAM) temporal pattern and the longest duration Generalised Short Duration Method (GSDM) temporal patterns. However, both these patterns were found to predict lower peak flow rates than those predicted for the 100 yr event at both the outlet to the Lake, and the outlet to Stony Creek when used in the hydrologic model. Consequently, the 100 yr temporal pattern was used for the 200 yr event.

For the PMF event and 18 hr duration storm, AR&R (2001, Book VI Table 7) recommends weighting results from the smoothed 24 hr Generalised South-East Australian Method (GSAM) temporal pattern and the longest duration Generalised Short Duration Method (GSDM) temporal patterns (in this case 6 hours). The cumulative rainfall patterns were redistributed into 18 equal time increments and both patterns input to the hydrologic model. AR&R recommends that the peak flow from both methods be used and weighted according to the storm duration to acquire the design peak flow. The most representative temporal pattern (in this case the 24 hr GSAM pattern) is factored to achieve that design peak flow. In the case of Burrill Lake, the difference in flow rates from both patterns was minimal (<1%). The redistributed smoothed GSAM pattern was adopted because, when used in the hydrologic model, it predicted marginally higher peak flow rates at the outlet from the Lake and the outlet from Stony Creek. This pattern is provided as Figure 7-2.

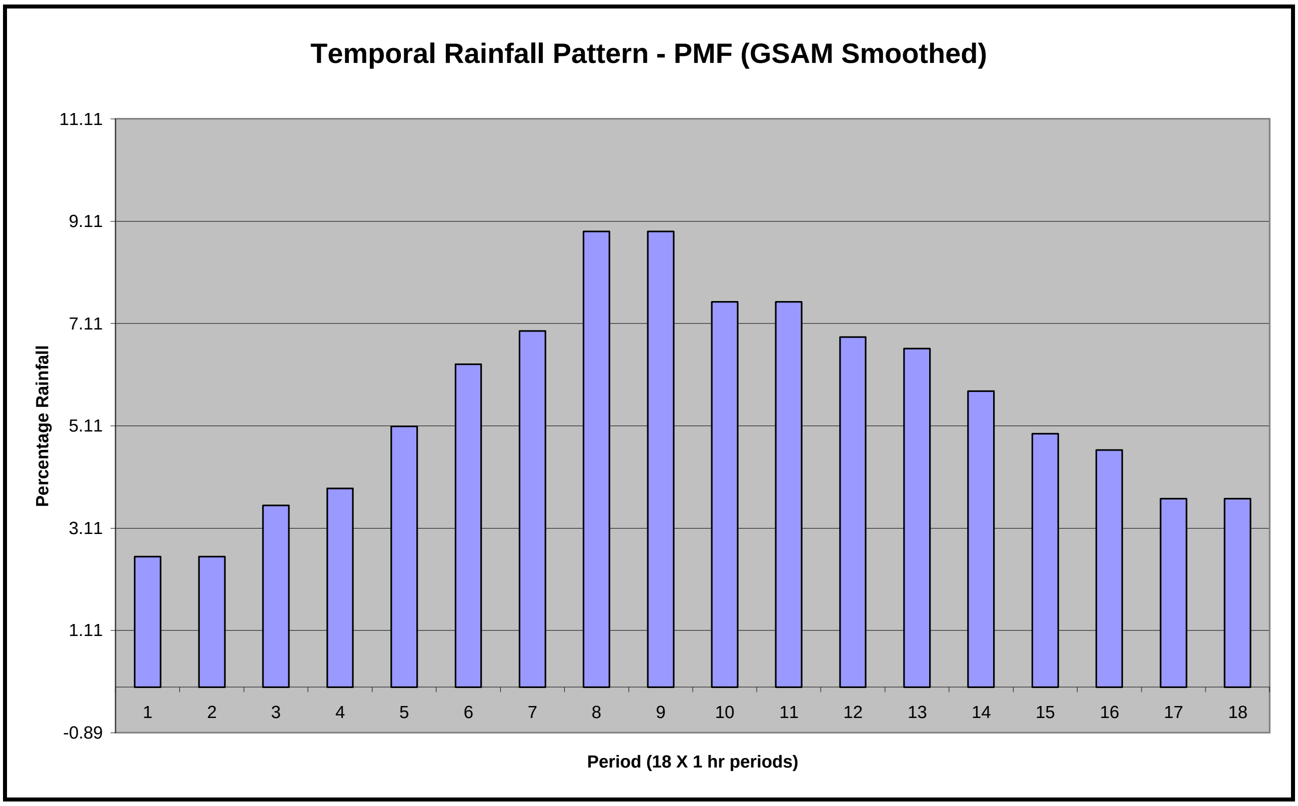


Figure 7-2 Derived 18 hour PMF Temporal Pattern

7.3.3 Spatial Distribution

Spatial distribution of the rainfall event defines the way in which the rainfall is spread over the catchment.

The spatial rainfall distribution is commented as follows in AR&R (2001): "*The rainfall IFD values derived in the* [hydrologic analysis] *are applicable strictly to a point, but they may be taken to represent IFD values over small areas (e.g. 4 km^2). For larger areas it is not realistic to assume that the same intensity can be maintained over the entire area, thus some reduction has to be made*".) An areal reduction factor of 0.975 can be extrapolated for the 65 km^2 of the Burrill Lake catchment from *Figure 1.6* in AR&R (2001), Book II. Even though no 36hr curve is provided on *Figure 1.6* (AR&R, 2001), the trend between the different storm duration values for a 65 km^2 catchment is such that the 36hr storm value can be confidently extrapolated.

As a consequence, the design rainfall intensities determined from AR&R were modified (multiplied by 0.975) and then applied uniformly across the Burrill Lake catchment in the hydrologic model.

AR&R also provides for the spatial variation of rainfall patterns over the catchment for rare to extreme floods. In order to undertake such analyses, assistance is required from the BoM in deriving topographic influence factors across the catchment. This analysis is beyond the scope of the present study and would result in a level of accuracy for the lower frequency floods (200 year & PMF) that is considered unnecessary. Given that there is limited spatial variation in both the historic and design events, even distribution of rainfall across the Burrill Lake catchment was adopted for the 1 in 200 yr ARI and PMF events.

7.4 Design Rainfall Losses

The design rainfall losses applied in the hydrologic model were agreed in consultation with SCC and DNR as discussed in Section 6.4. The loss values adopted were:

- Initial Losses: 10 mm
- Continuous Losses: 0.0 mm/hr

7.5 Entrance Channel Geometry

The entrance channel is subject to geomorphologic change through the regular tidal movement and the episodic flooding from the catchment. In order to determine the design conditions for the Flood Study, it was necessary to choose an adequate initial bed level in the entrance channel prior to the erosion induced by the flood.

Council does not have an officially adopted entrance management policy although one is currently in preparation. An interim guideline for entrance management is provided in the Burrill Lake Catchment and Estuary Management Plan (Shoalhaven City Council, 2002). The guideline states that:

> *"As an interim measure, it is proposed that Council would seek appropriate approvals and carry out negotiations with relevant state agencies when the lake water level reaches 1.25 m AHD "*

DNR's Draft Floodplain Management Guideline No. 5 *Ocean Boundary Conditions* discusses the geometric aspects associated with the possible entrance channel types. Burrill Lake is naturally subject to occasional closure but there is no official policy for keeping it open. Council has a policy of managing the entrance to prevent closure. The Burrill Lake catchment falls into the "catchments that drain direct to the ocean via shoaled entrances" category. The guidelines recommend the adoption of the following geometries:

- A restricted entrance channel geometric conditions for catchment-derived floods; and
- An open (scoured) entrance channel geometric conditions for ocean-derived floods.

In determining design flood conditions, consideration is given to both entrance conditions (e.g., an envelope of the highest computed water levels between the two design flood scenarios is adopted to represent the maximum design flood conditions).

The most restricted condition was based on a survey undertaken when the entrance was closed in early 2005. The least restricted entrance condition was determined in consultation with DNR & SCC and comprised the bathymetry resulting from a 5 yr ARI catchment flood. In other words, a 5yr catchment flood was input to the model and the geomorphic model used to predict a final bathymetry. This bathymetry was then used as the starting bathymetry for the ocean inundation simulations.

7.6 Initial Water Level

The initial water levels adopted for the design simulations were as follows:

- For catchment flooding: An initial level of 1.12 m AHD, representing the highest water level attained during the closed entrance event of early 2005.
- For ocean flooding: An initial water level derived by running a repeating 0.6 m neap tide through into the model to 'set-up' the water levels inside the Lake. It was determined that the Lake had reached a 'dynamic equilibrium' after 10 tidal cycles (approx 125 hours) and the resulting water levels were used to 'hot-start' the ocean inundation simulations.

7.7 Ocean Downstream Boundary Condition

Similarly to the design entrance channel geometry, DNR's Floodplain Management Guideline No. 5 – *Ocean Boundary Conditions* recommends the adoption of specific tidal cycles in conjunction with catchment and ocean flood combinations.

For entrances like Burrill Lake, this involves modelling two design flood scenarios:

- A flood induced by large catchment runoff coinciding with neap tide ocean conditions. This design flood condition is to be applied to the restricted entrance channel geometry; and
- A flood induced by small catchment runoff coinciding with a large ocean storm (large ocean tail water level). This design flood condition is to be applied to the open entrance channel geometry.

In determining design flood conditions, consideration is given to both flooding mechanisms (i.e., envelopes of the 'worse case' conditions for each flood parameter are developed).

7.7.1 Catchment-derived Flooding

Downstream boundary conditions for the simulated catchment flood conditions are defined in Table 7-4.

Table 7-4 Downstream Boundary Conditions for Catchment Flooding

Design Flood	Design Rainfall Average Return Interval	Design Ocean Peak Water Level (m AHD)
5 Year ARI	5 Year	0.6 (Regular Neap Tide)
10 Year ARI	10 Year	0.6 (Regular Neap Tide)
20 Year ARI	20 Year	0.6 (Regular Neap Tide)
50 Year ARI	50 Year	0.6 (Regular Neap Tide)
100 Year ARI	100 Year	0.6 (Regular Neap Tide)
200 Year ARI	200 Year	0.6 (Regular Neap Tide)
PMF	PMP	2.9 (200 Year ARI)

Design simulations were setup to ensure that the peak water levels in the downstream part of the Lake coincide with the peak of the ocean tide cycle. The neap tide cycle in the ocean peaks at 0.6 m AHD, which produces to peak tidal water levels in Burrill Lake of approximately 0.35 m AHD.

In the specific case of the PMF event, the exceptional circumstances associated with this storm event justifies adopting a much higher ocean boundary condition. To represent the high tail water levels, a 200 year ARI ocean tidal cycle was used, peaking at 2.9 m AHD (high tide), coinciding with the catchment flood peak.

7.7.2 Ocean-derived Flooding

Downstream boundary conditions for ocean storm-based flooding are presented in Table 7-5.

Table 7-5 Downstream Boundary Conditions for Ocean Flooding

Design Flood	Design Rainfall Average Return Interval	Design Ocean Peak Water Level (m AHD)
5 Year ARI	5 Year	1.89 (5 Year ARI)
10 Year ARI	5 Year	2.08 (10 Year ARI)
20 Year ARI	5 Year	2.25 (20 Year ARI)
50 Year ARI	5 Year	2.45 (50 Year ARI)
100 Year ARI	5 Year	2.60 (100 Year ARI)
200 Year ARI	5 Year	2.90 (200 Year ARI)
PMF	PMP	2.90 (200 Year ARI)

Design conditions were established to ensure that the peak flood levels in the downstream part of the Lake coincided with the peak of the ocean tide cycle (including set-up).

The design elevated ocean water levels due to the ocean storm are presented in Figure 7-3.

When comparing Table 7-4 and Table 7-5, it is clear that the 5 year ARI ocean flooding condition results in worse conditions than the 5 year ARI catchment flood (as they both have the same catchment inflows but higher downstream boundary conditions for the ocean flooding scenario).

It should also be noted that input conditions for the PMF ocean flood are the same as the conditions for the PMF catchment flood, and therefore only one set of PMF conditions was modelled.

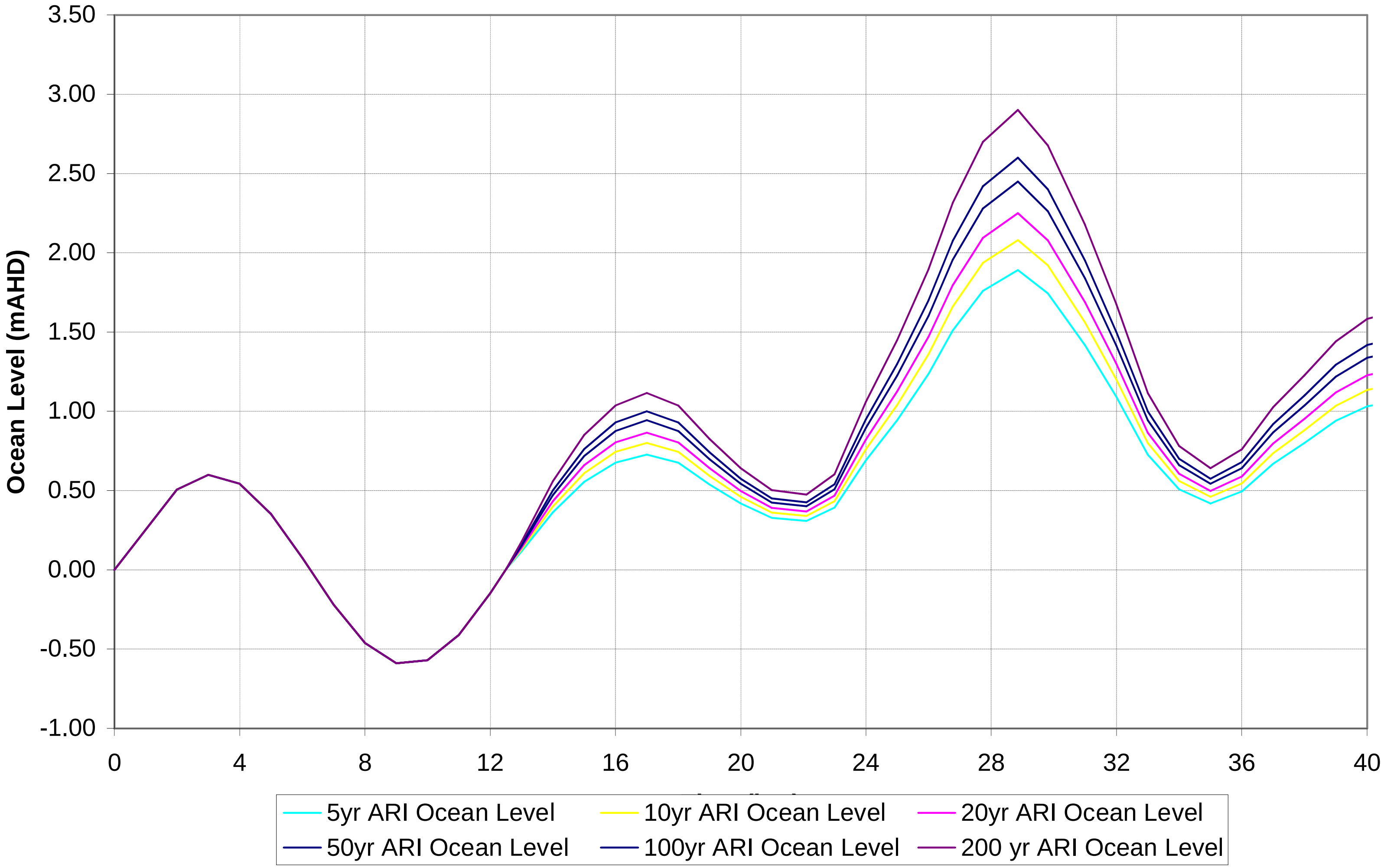


Figure 7-3 Floodplain Management Guideline No.5 - Design Ocean Water Levels at Burrill Lake

8 DESIGN FLOOD PRESENTATION

8.1 Design Flood Description

The behaviour of Burrill Lake design floods depend on the source of flooding: catchment or ocean.

Catchment-derived design floods are characterised by a low ocean tailwater level and a relatively steep water surface gradient through Burrill Inlet. The high discharge and low ocean tailwater conditions generates super-critical flow characteristics across the entrance sand berm (the entrance berm is assumed to be closed for catchment-derived design floods). The entrance berm and lower end of the entrance channel is subsequently scoured by the high flood velocities (especially during low tide stages of the tidal cycle). The water surface gradient through Burrill Lake is steepest at the causeway and over the entrance channel shoals, where frictional resistance is highest. The northern and southern basins of the Lake remain essentially flat (less than 3 cm difference), given the deep bathymetry, low velocities and thus negligible frictional resistance of the bed.

Ocean-derived design floods are characterised by a high ocean tailwater level and a flatter water surface gradient through Burrill Inlet. The high ocean tailwater level results from assumed combined effects of pressure surge, wind and wave setups. The resulting ocean level on Burrill Beach can be more than 2 metres higher than a neap high tide. The high ocean tailwater level prevents any substantial catchment runoff from discharging to the ocean, meaning that the catchment runoff is stored within the Lake, resulting in elevated water levels. Discharge from the Lake only occurs once sufficient head is generated across Burrill Inlet to overcome the frictional resistance of the shallows shoals and causeway structure. This happens when catchment runoff causes Lake levels to rise sufficiently above the ocean tailwater levels, or the ocean tailwater levels fall sufficiently below the level of the Lake (due to low tide in the ocean).

In considering both sources of flooding, it has been found that the ocean-derived design floods produce higher flood levels in most of Burrill Lake and Burrill Inlet than catchment-derived design floods of the equivalent probability of occurrence.

8.2 Design Flood Maps

The behaviour of each design flood event is best described using maps that show the spatial variation of key flood characteristics, such as peak flood levels, depths, velocities and hazards. Design flood maps were produced for each design flood event for several hydraulic parameters. The design flood maps show the ‘worst case’ conditions at every location within the model over the course of the full flood event duration and considering both possible flood mechanisms (i.e. they present an envelope of maximums over time and over design flood mechanisms). For example, the 100 yr ARI design flood map for water levels shows the maximum water levels at each model element when considering both flooding from the catchment and flooding from the ocean. It is therefore important to recognise that the design flood maps are not representative of a particular instant in time.

A longitudinal profile of the maximum design water levels, for all flood scenarios considered, is presented in **Figure 8-1**.

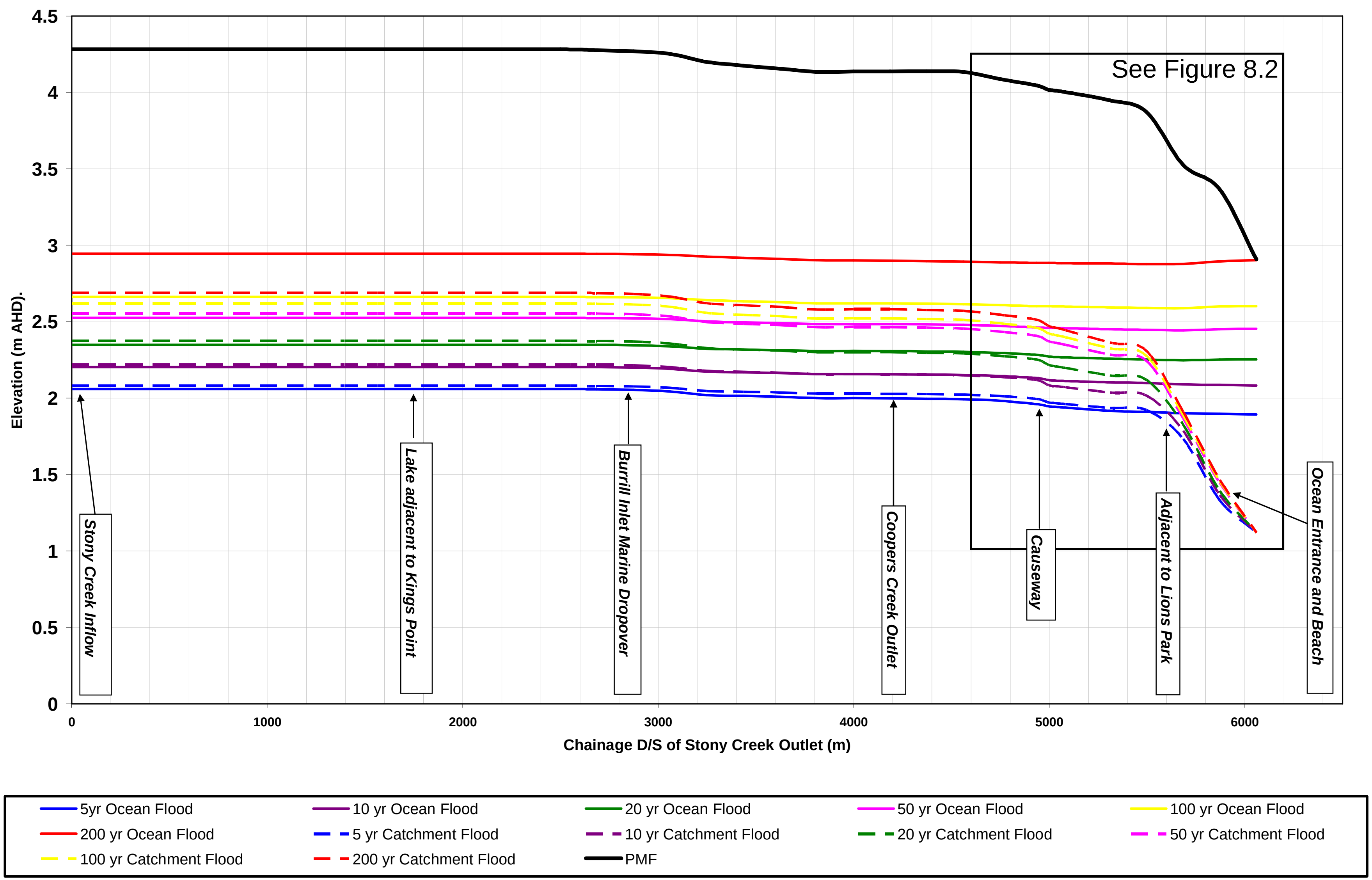


Figure 8-1 Burrill Lake Longitudinal Profile of Design Flood Water Levels

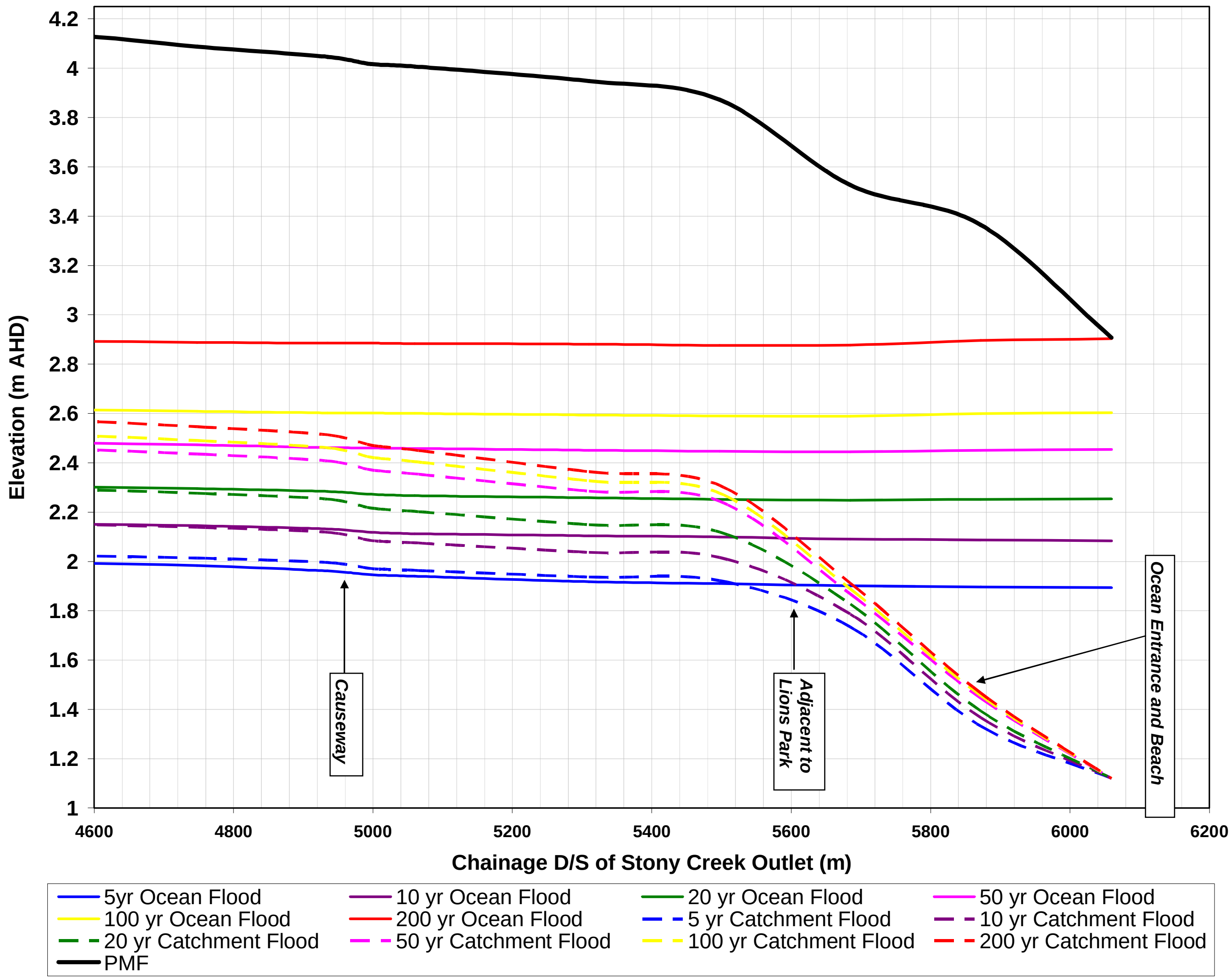


Figure 8-2 Design Flood Profiles – Detail of Lower Burrill Inlet

8.2.1 Design Peak Flood Levels, Depths and Velocities

Design peak flood levels, depths and velocities are provided in the Figures as listed in Table 8-1.

Table 8-1 Figure Numbers – Peak Flood Levels, Depths, Velocities and Provisional Hazard

Event ARI (years)	Peak Flood Levels	Peak Flood Depths	Peak Flood Velocities	Provisional Hazard	Hydraulic Categories
5 year	**Figure 8-8**	**Figure 8-15**	**Figure 8-22**	**Figure 8-29**	**N/A**
10 year	**Figure 8-9**	**Figure 8-16**	**Figure 8-23**	**Figure 8-30**	**N/A**
20 year	**Figure 8-10**	**Figure 8-17**	**Figure 8-24**	**Figure 8-31**	**N/A**
50 year	**Figure 8-11**	**Figure 8-18**	**Figure 8-25**	**Figure 8-32**	**Figure 8-36**
100 year	**Figure 8-12**	**Figure 8-19**	**Figure 8-26**	**Figure 8-33**	**Figure 8-37**
200 year	**Figure 8-13**	**Figure 8-20**	**Figure 8-27**	**Figure 8-34**	**Figure 8-38**
PMF	**Figure 8-14**	**Figure 8-21**	**Figure 8-28**	**Figure 8-35**	**Figure 8-39**

8.2.2 Provisional Hazards

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.

Figures G1 and G2 in the Floodplain Development Manual (NSW Government, 2005) are used to determine provisional hydraulic and hazard categorisations within floodprone land. These figures are reproduced in Figure 8-3.

The provisional flood hazard categorisation is presented in the result figures, as listed in Table 8-1.

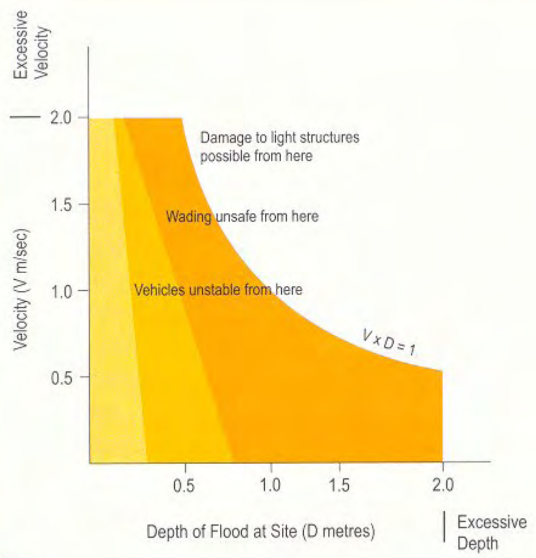
8.2.3 Hydraulic Categories

Three hydraulic categories are defined in the NSW Floodplain Development Manual (2005): floodways, flood storage and flood fringe. The definition of the three hydraulic categories is based on qualitative assessments rather than quantitative thresholds. As such, the determination of floodways, flood storages and flood fringes is open to some subjectivity. The methodology used to determine preliminary hydraulic categories for the Burrill Lake Flood Study has been adopted by WBM for a number of similar flood studies, to the satisfaction of local and state government. The methodology uses both the unit flow (velocity x depth) and depth results as described below:

- The floodways are determined first. Floodways are the areas where a significant volume of water flows during floods. A significant volume of water conveyed downstream can be achieved through great depth and slow velocity, shallow depth and high velocity and/or great depth and high velocity. When considering the floodplain from its upstream main inflow boundary to its downstream boundary, the floodways are the paths that the majority of the flow follows. For the purpose of studying the lateral extent of floodways, the total flow can be divided in unit flows (flow per meter width) across the floodplain. The width of the floodway is defined by the areas where the unit flows are greater than a threshold value. In the case of the Burrill Lake Flood Study, the threshold value was chosen to be the average unit flow value across the entire floodplain. Thus defined, the floodway is the area where the flow rate is typically significantly greater than adjacent areas. In the majority of cases, the defined floodway accounts for more than 50% of the total flow.
- Once the floodways have been determined, the remainder of the floodplain is a combination of flood storage and flood fringe areas. Flood storage comprises areas where, if blockage to flows into those areas occurs, significant impacts of flood flows and levels within the floodways would result. As a consequence, the floodplain areas outside of the floodways are essentially characterised as flood storage. The flood fringe areas are those areas within the flood storage that are of small significance for flood behaviour. The volume of total flood that this may represent was estimated to correspond to a depth of 0.1 m over the entire area of floodway.

The results of the preliminary hydraulic categorisation exercise for the 1 in 50 yr, 1 in 100 yr and 1 in 200 yr ARI floods and the PMF are presented in Figure 8-36 through Figure 8-39.

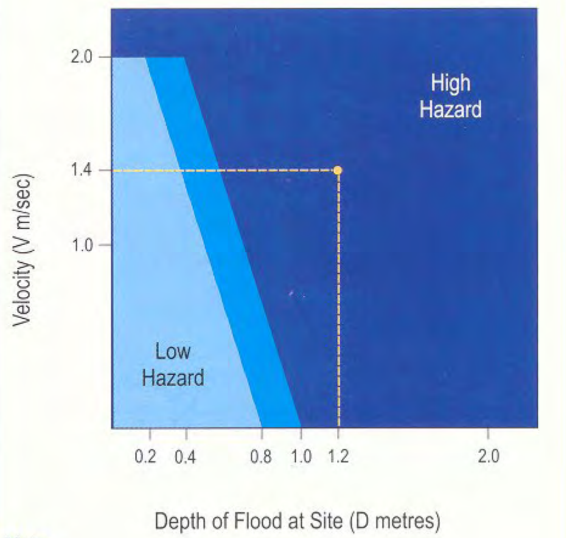
The methodology can be reproduced easily as it is based on model results and automated post processing. Confirmation of hydraulic categories across the Burrill Lake floodplain will be undertaken as part of the subsequent Floodplain Risk Management Study.



Notes

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

Figure G1
Velocity & Depth Relationships



Note

The degree of hazard may be either -

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

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

EXAMPLE

If the depth of flood water is **1.2 m**

and the velocity of floodwater is **1.4 m/sec**

then the provisional hazard is **high**

Figure G2
Provisional Hydraulic Hazard Categories

Figure 8-3 Hazard Determination (Source: NSW Floodplain Development Manual, 2005)

8.3 Flood Hydrographs

Water level hydrographs from both oceanic and catchment flood simulations were extracted at two locations:

- Within the northern basin, where the water level is representative of the deeper basins of Burrill Lake; and
- Upstream of the Causeway, where the water level represents the severity of flooding in the developed areas that are most susceptible.

The hydrographs are presented in Figure 8-4 and Figure 8-5 for catchment-derived flooding conditions, and Figure 8-6 and Figure 8-7 for ocean-derived flooding conditions.

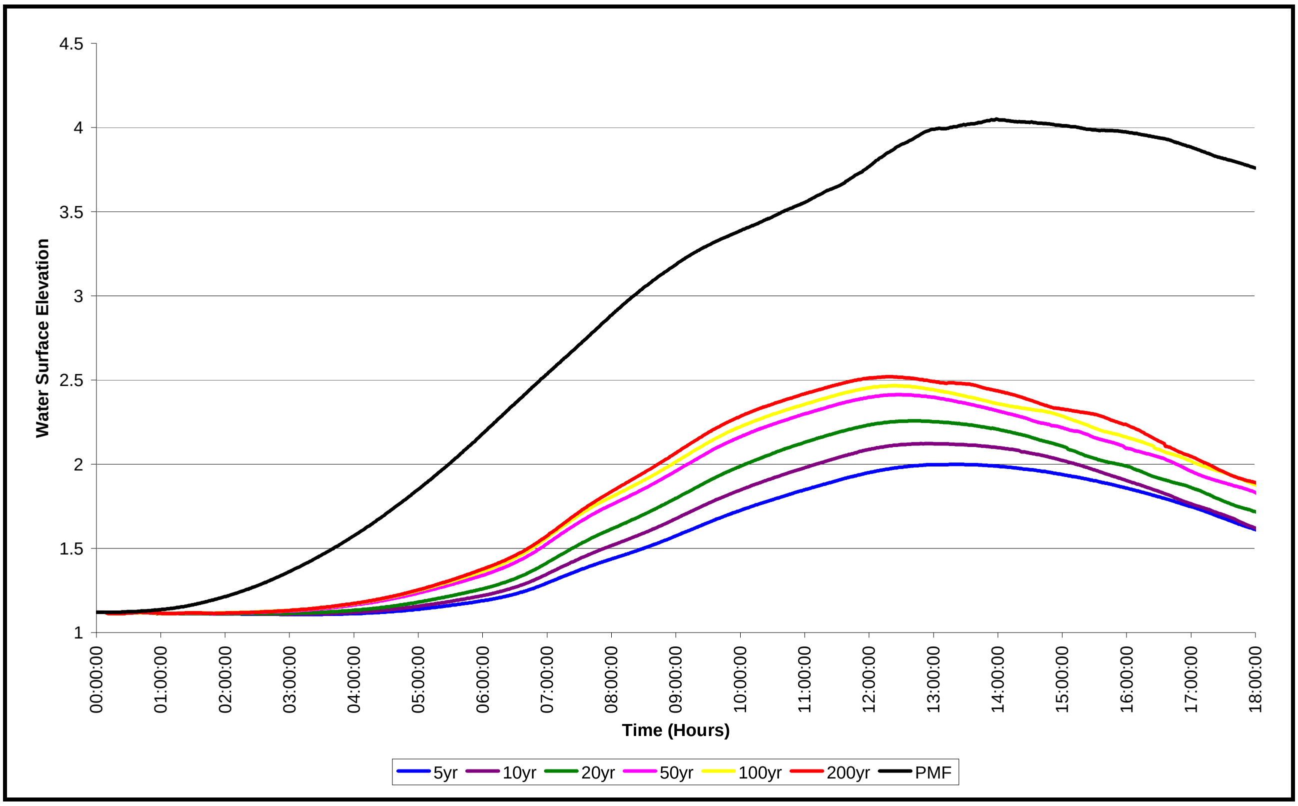


Figure 8-4 Predicted Catchment Design Flood Water Level Hydrographs at the Causeway

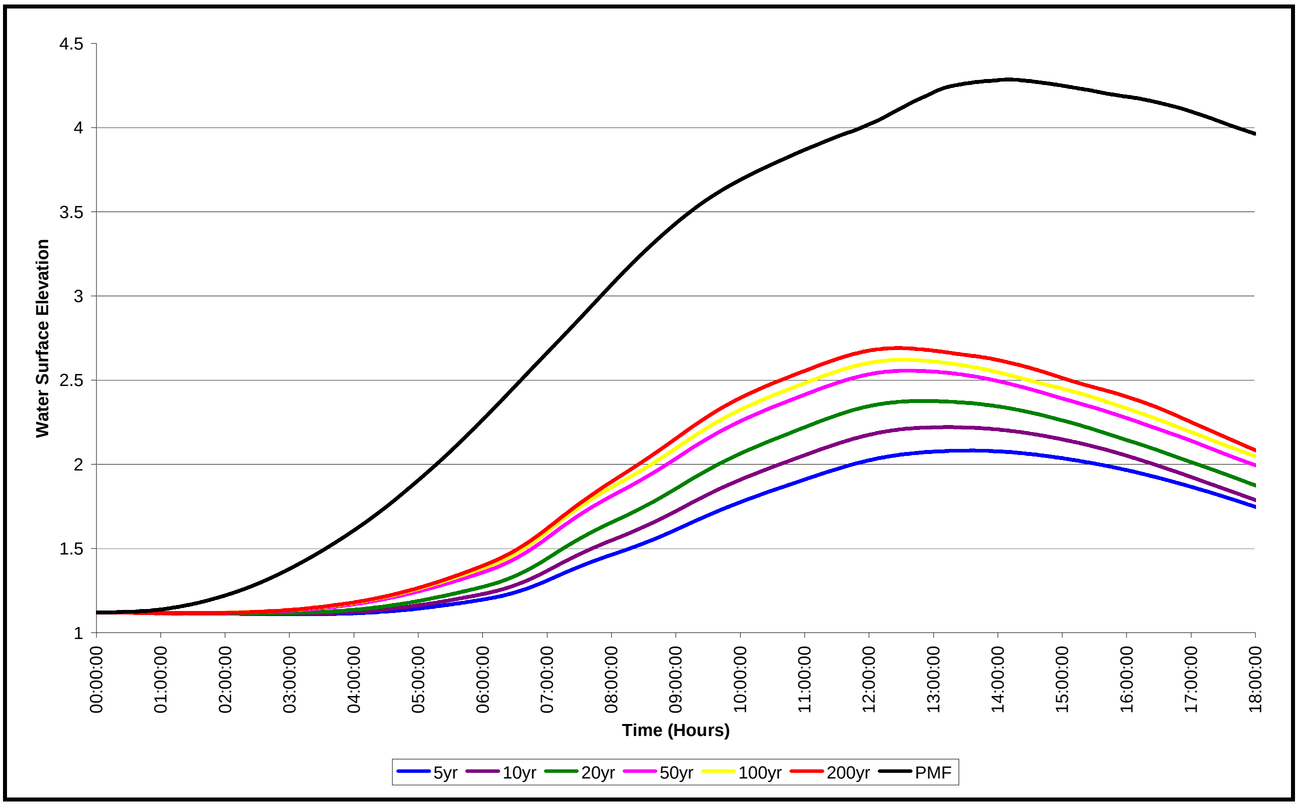


Figure 8-5 Predicted Catchment Design Flood Water Level Hydrographs in the Northern Basin

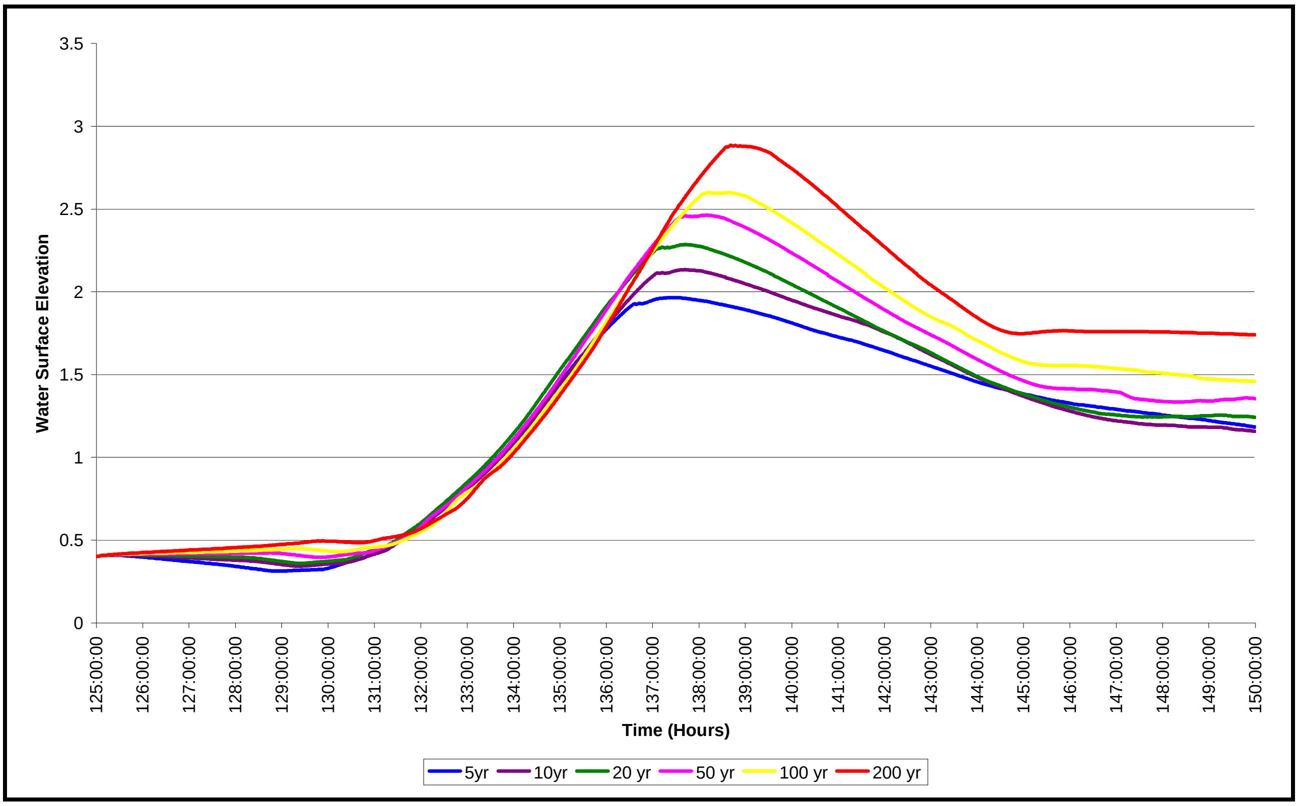


Figure 8-6 Predicted Oceanic Design Flood Water Level Hydrographs at the Causeway

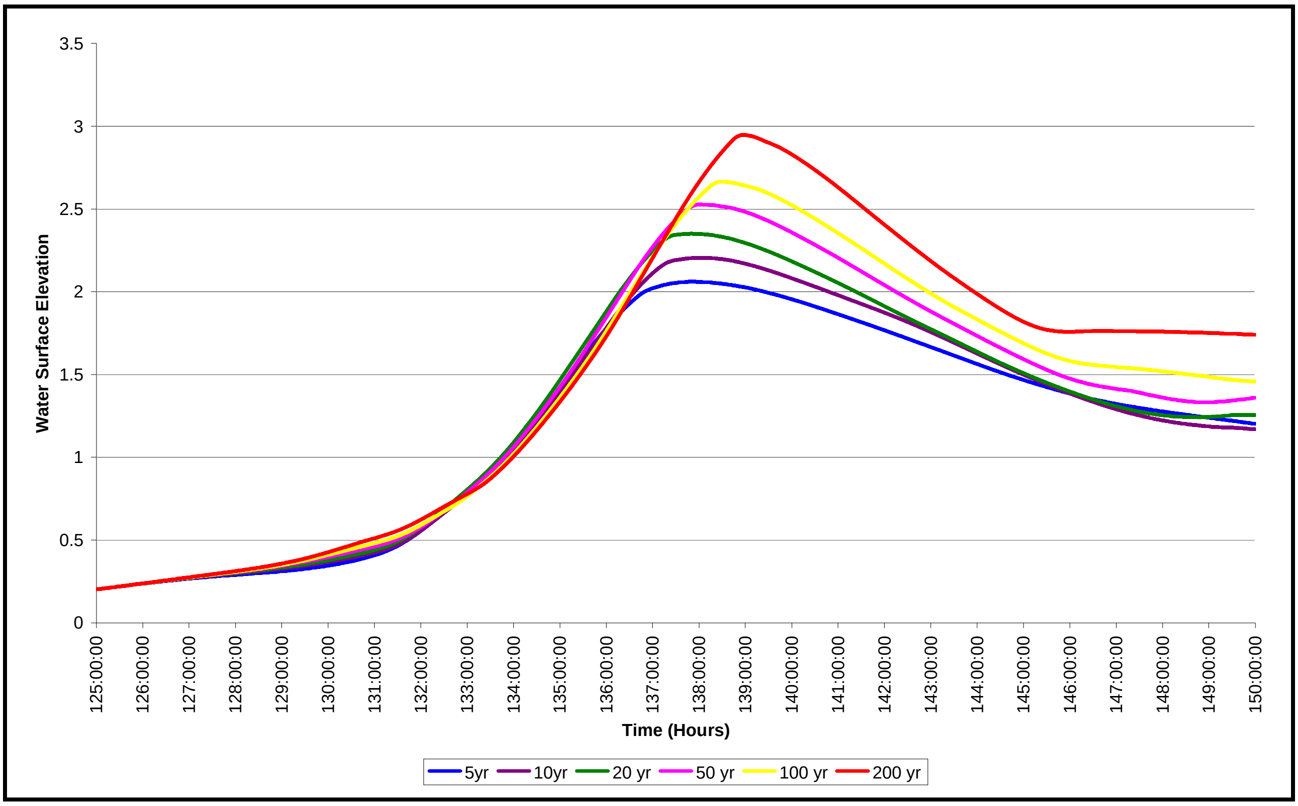


Figure 8-7 Predicted Oceanic Design Flood Water Level Hydrographs in the Northern Basin

8.4 Flooding Behaviour

8.4.1 Introduction

Descriptions of the flooding behaviour is key sections of the floodplain are provided below. These descriptions have been provided separately for catchment-derived and ocean-derived flooding conditions. The descriptions are based on the 1 in 100 year ARI design events, and represent a significant level of overbank inundation. Reference can be made to Figure 8-12, which displays the peak 1 in 100 year water levels around the lake.

8.4.2 Catchment-derived flooding

8.4.2.1 Bungalow Park

The first areas to become inundated by rising flood waters are the floodway between Honeysuckle Close and Casuarina Close, and parkland to the north of Moore Street, opposite the northern end of Lakeview Drive.

As flood levels rise, the western bank of Burrill Inlet is overtopped and water inundates low lying properties on Balmoral Road (near the Causeway) and the western end of Maria Avenue.

At around six hours of the simulated design event, the area between Thistleton and Rackham Crescent starts to become significantly inundated, and water flows westwards along Moore Street and southwards along Thisleton Drive. At this time properties along the western side of Lake View Drive also become inundated by rising water in the Southern Basin. After seven hours the areas to the north of Moore Street becomes isolated. Also, the low lying parts of Balmoral Road, near the intersection with the Princes Highway becomes inundated by more than 0.5 m of water.

Flooding continues from Thisleton Drive to the north of Balmoral Road, inundating most of the Bungalow Park Residential Area to the north of Maria Avenue after about eight hours of the simulation. Some parts of Lake View Drive remain elevated above the rising waters but are isolated (8 hours into simulation). Properties adjacent to the Lake at the end of Wallaroy Crescent also starting to experience flooding.

From eight to ten hours into the simulation, water levels continue to rise, and progressively inundate higher land between Balmoral Road and the Princes Highway (George St. Lakewood Grove). The high properties on Lake View Drive become completely inundated and substantial parts of the Bungalow Tourist Park are also inundated.

The flood eventually reaches its maximum extent after around 13 hours, at which stage almost all properties located on the Bungalow Park peninsula are inundated, with the exception of a few properties at the end of Ireland Street, and some higher properties along Balmoral Road, near the Princes Highway.

At the peak of the flood, parts of Thistleton Drive is inundated by nearly 1.5 m of water and flow passes between the southern basin and Burrill Inlet, across the residential area, primarily along Balmoral Street, Maria Avenue and Rackham Crescent. Flow also occurs along Moore St and across Ireland Street, which behaves like a weir in the vicinity of the floodway.

Properties within the Wallaroy Drive / Wyoming Avenue area remain relatively dry, with the exception of a few properties near the end of Wallaroy Drive.

The flood waters subside over the ensuing 10 hours of the design flood simulation.

8.4.2.2 Burrill Lake (village)

Around six hours into the simulation, floodwaters begin to inundate properties adjacent to the Inlet in McDonald Parade. The effect is most pronounced in the vicinity of Coopers Creek, which also contributes to flooding in this area. The severity of this flooding increases, with the intersection of McDonald Parade and Commonwealth Avenue becoming inundated after 7 hours. At this stage many of the properties to the north of this intersection, near Coopers Creek are inundated, as are most of the properties between McDonald Parade and Burrill Inlet. In addition, the Causeway becomes overtopped near its eastern end.

Between seven and eight hours into the simulation, substantial inundation occurs. This includes:

- Inundation of McDonald Parade near the intersection with the Princes Highway and inundation of the Princes Highway such that it becomes impassable (eastern end of Causeway);
- Inundation of properties between the Princes Highway and Princess Ave South;
- Inundation of properties between McDonald Parade, Commonwealth Avenue and Queanbeyan Avenue.

After around 8 hours, the intersection between Princess Avenue and the Princes Highway is inundated. This progressively worsens over the next two hours at which stage, all of the properties in this area become inundated, with a typical flood depth of between 0.5 – 1.0 m.

As for Bungalow Park, the flood peaks at around 13 hours into the design simulation.

8.4.2.3 Dolphin Point

Significant flooding does not occur until six hours into the simulation. At this time, Dolphin Point Road becomes inundated to the north of the Dolphin Point Tourist Park and overland flow is established along the northern boundary of the Dolphin Point Tourist Park and the entrance shoals. Between six and seven hours, Dolphin Point Road is further overtopped in the vicinity of the tidal creek that flows along the north-western edge of Lions Park and becomes impassable. At around 7.5 hours, flow from Bungalow Park overtops the Princes Highway and begins to substantially inundate parkland to the south of the Highway and east of Dolphin Point Road.

Leading up to the flood peak at 13 hours, the depth and extent of flooding increase, within the eastern parts of Lions Park, public land on either side of Dolphin Point Road near the Princes Highway and the northern parts of the Dolphin Point Tourist Park. Floodwaters drain from this area somewhat faster than the areas north of the Causeway, and most areas are dry within 8 hours following the flood peak.

8.4.3 Ocean-derived flooding

8.4.3.1 Bungalow Park

The first areas to become inundated by rising flood waters are the floodway between Honeysuckle Close and Casuarina Close, and parkland to the north of on Moore Street, opposite the northern end of Lakeview Drive. This occurs around ten hours into the simulation

As flood levels rise, the western bank of Burrill Inlet is overtopped and water inundates low lying properties on Balmoral Road (near the Causeway) and the western end of Maria Avenue.

By 11 hours into the simulation, the area between Thistleton and Rackham Crescent starts to become significantly inundated, and water flows westwards along Moore Street and southwards along Thistleton Drive. The pattern of flooding is similar to the catchment flood pattern, except that properties between Thistleton and Rackham Crescent are inundated from the east.

Over the following hour, the areas of inundation along the southern end of Lake View Drive, and properties along Thistleton Drive extend and join, effectively isolating the high points on Lake View Drive (south of Moore St) and eventually inundating these properties as well. Around this time, Ireland St. also overtops between Honeysuckle Close and Casuarina Close. A northward flow path also develops across the Princes Highway and along Balmoral Road.

Around 14 hours into the simulation, almost the entire area of Bungalow Park is inundated, with much of the area under more than 1.0 m of water. Furthermore, some of the properties at the end of Wallaroy Drive are partially inundated by backwater from the Southern Basin.

As the flood waters recede, significant flow paths develop across the peninsula, including flows along Lake View Drive, Balmoral Road, Maria Avenue, Rackham Crescent, Maria Avenue and across Ireland Street at the flood way, and after 24-25 hours, most of Bungalow Park is dry.

8.4.3.2 Burrill Lake (village)

As the ocean levels rise, the first area to become inundated (9 hours into simulation) is between Princess Avenue South and the Princes Highway. Between 9 and 10 hours, the Causeway and Princes Highway at its eastern end are overtopped and properties fronting the Inlet along McDonald Parade and the Burrill Lake Caravan Park start to be inundated.

After 10.5 hours, the intersection of MacDonald Parade and the Princes Highway becomes inundated and impassable.

By 11 hours into the simulation, the properties to the west of McDonald Parade and north of the intersection of McDonald Parade and Commonwealth Avenue are inundated and the Burrill Lake Caravan Park is substantially under water. By 12 hours into the simulation, water from Burrill Inlet has overtopped MacDonald Parade and has engulfed the remainder of the low lying streets and properties to the east (Queanbeyan, Princess, Federal and Commonwealth Avenues). A significant northward flow path has developed from Princess Avenue South towards Princess Avenue (i.e. across the highway).

The full flood extent in this area is reached after around 14.5 hours, at which time most of the Burrill Lake village is inundated by more than 1.0 m of water.

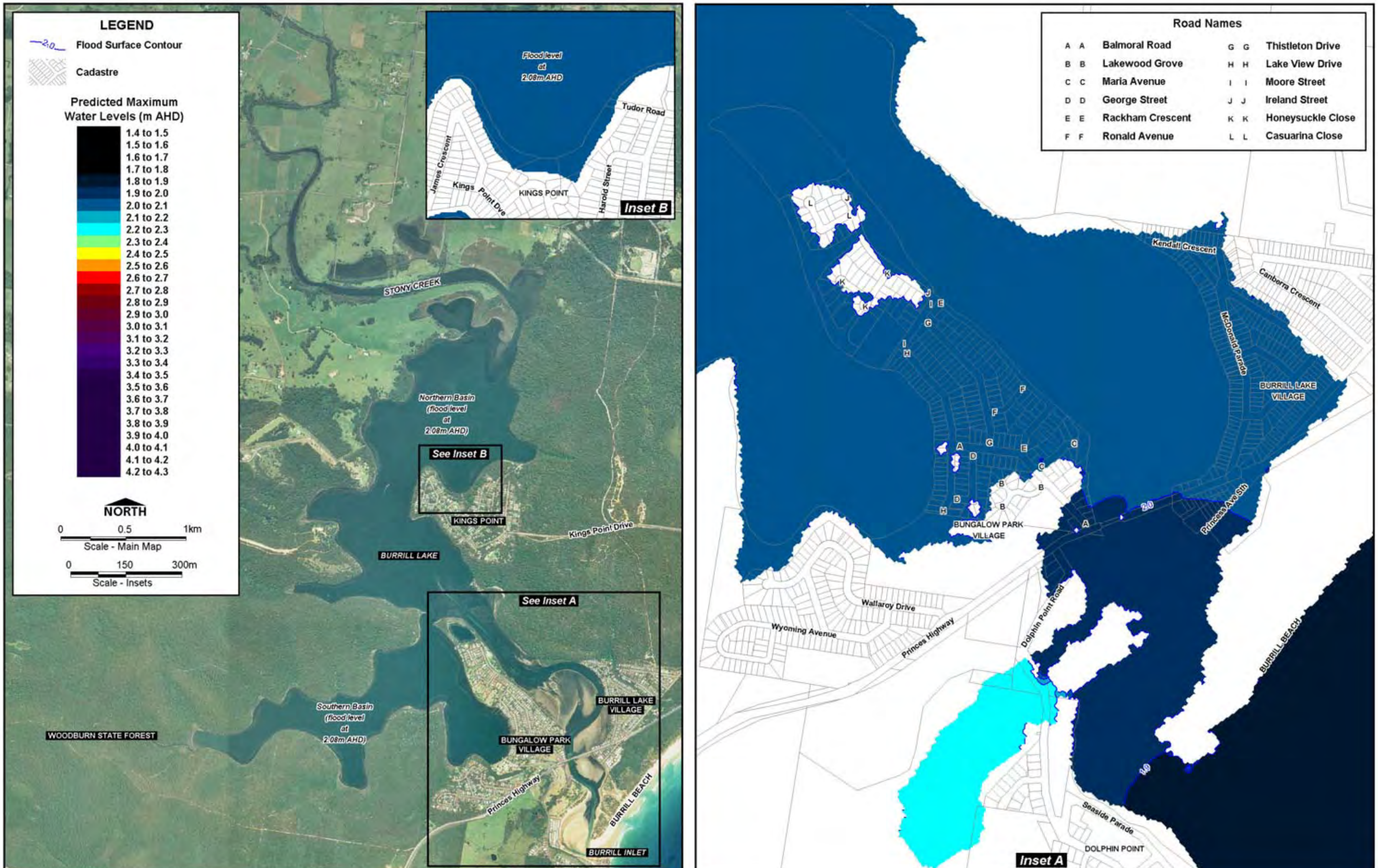
8.4.3.3 Dolphin Point

Around nine hours into the simulation, Dolphin Point Road becomes inundated at the northern end of the Dolphin Point Tourist Park through a combination of elevated downstream water levels and local catchment runoff. Furthermore, flow begins to overtop Dolphin Point at around the same time, near its intersection with the Princes Highway.

Over the following three hours the following occur:

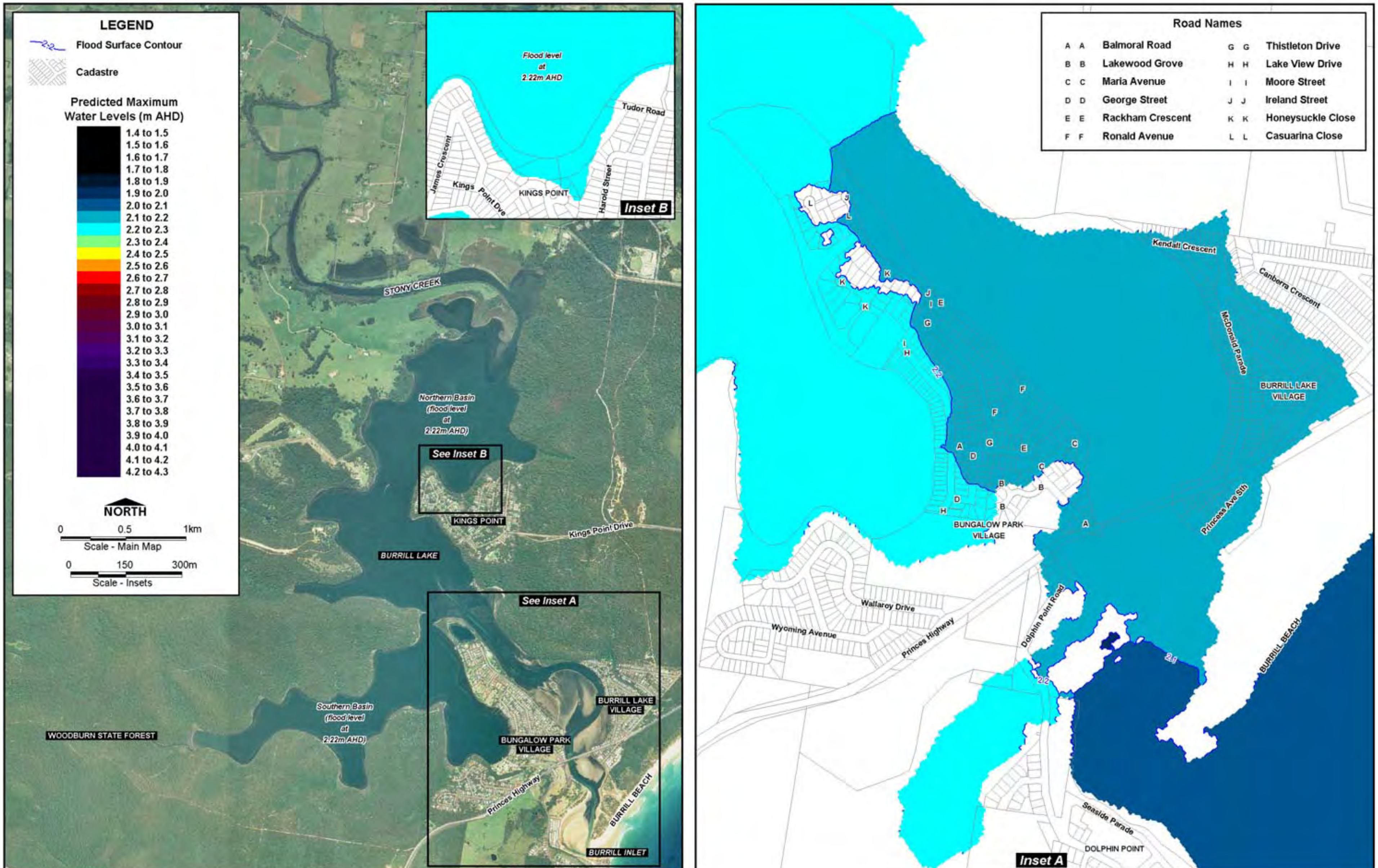
- Substantial inundation of the eastern end of Lions Park;
- Overtopping of Dolphin Point Road at the creek crossing;
- Establishment of overland flow to the north of Dolphin Point Tourist Park;
- Inundation of areas between Lions Park and the Princes Highway; and
- Overtopping of the Princes Highway at Bungalow Park and the eastern end of the causeway.

Within 12-14 hours of the beginning of the simulation, the flood has reached its peak, at this stage, almost all of Lions Park is inundated, in places by a depth of over 1 m. Flow occurs across Lions Park and a broad, overland flow path exists at the northern end of Dolphin Point Tourist Park.



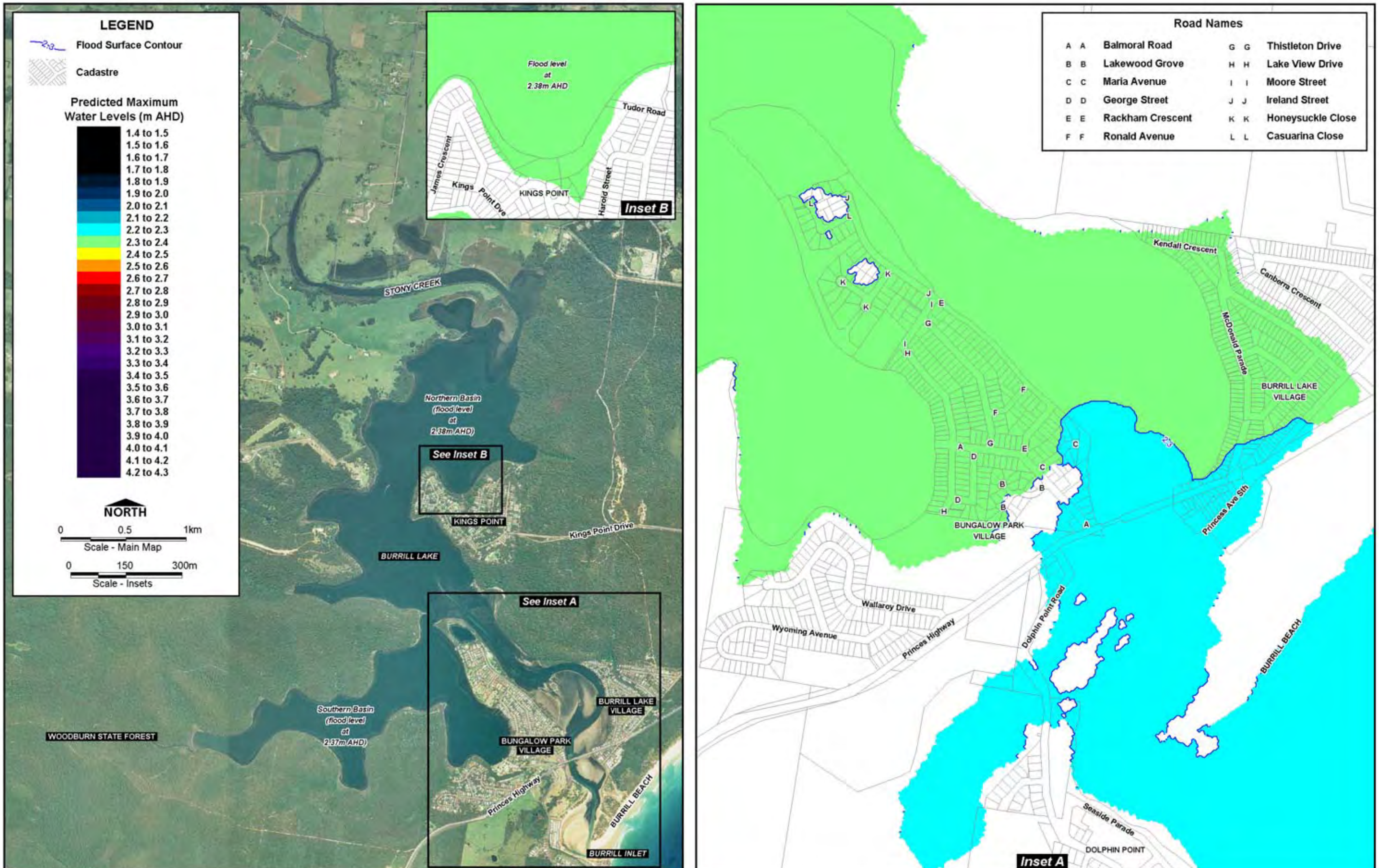
Maximum 5 Year ARI Design Flood Water Levels

Figure 8-8



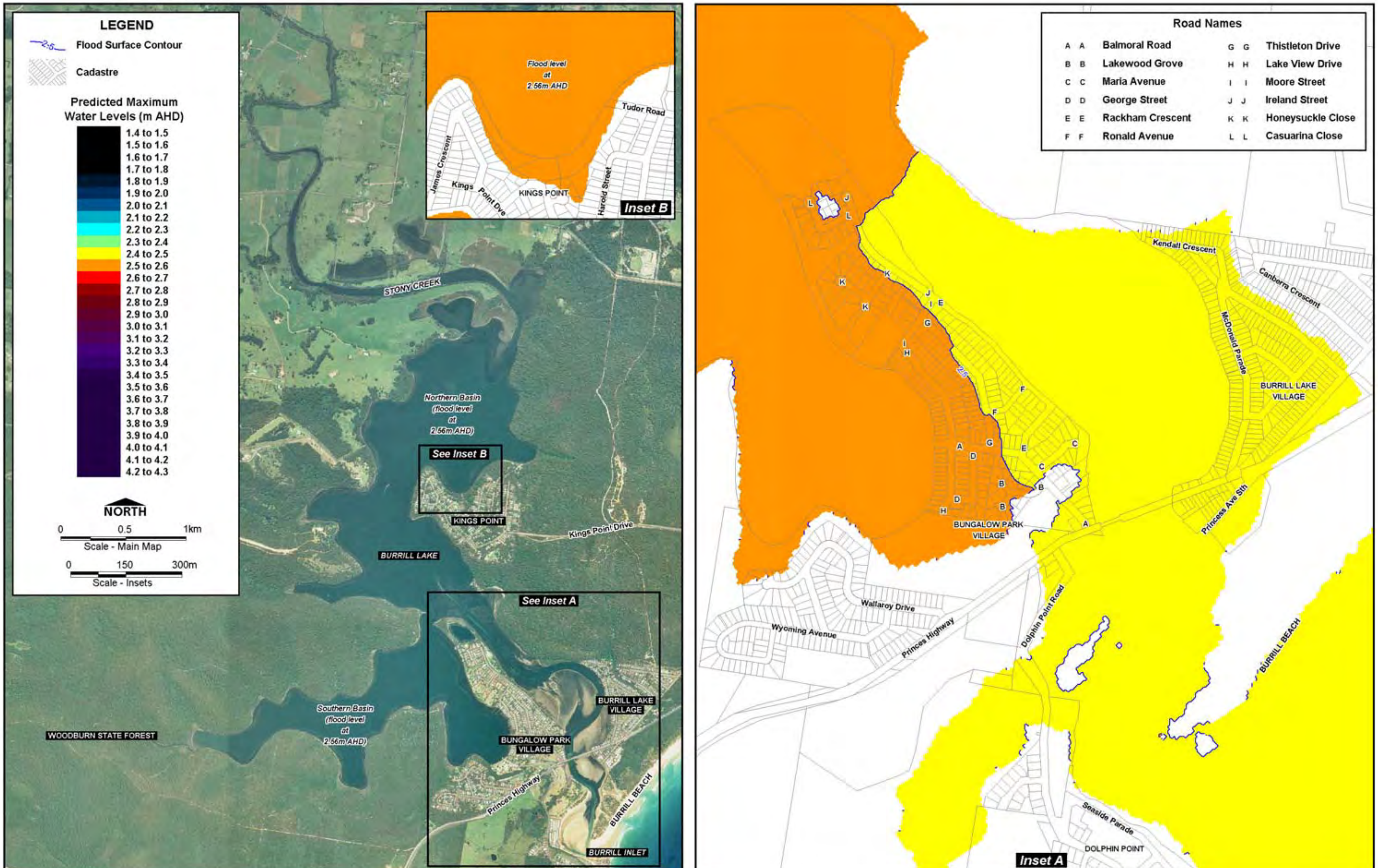
Maximum 10 Year ARI Design Flood Water Levels

Figure 8-9



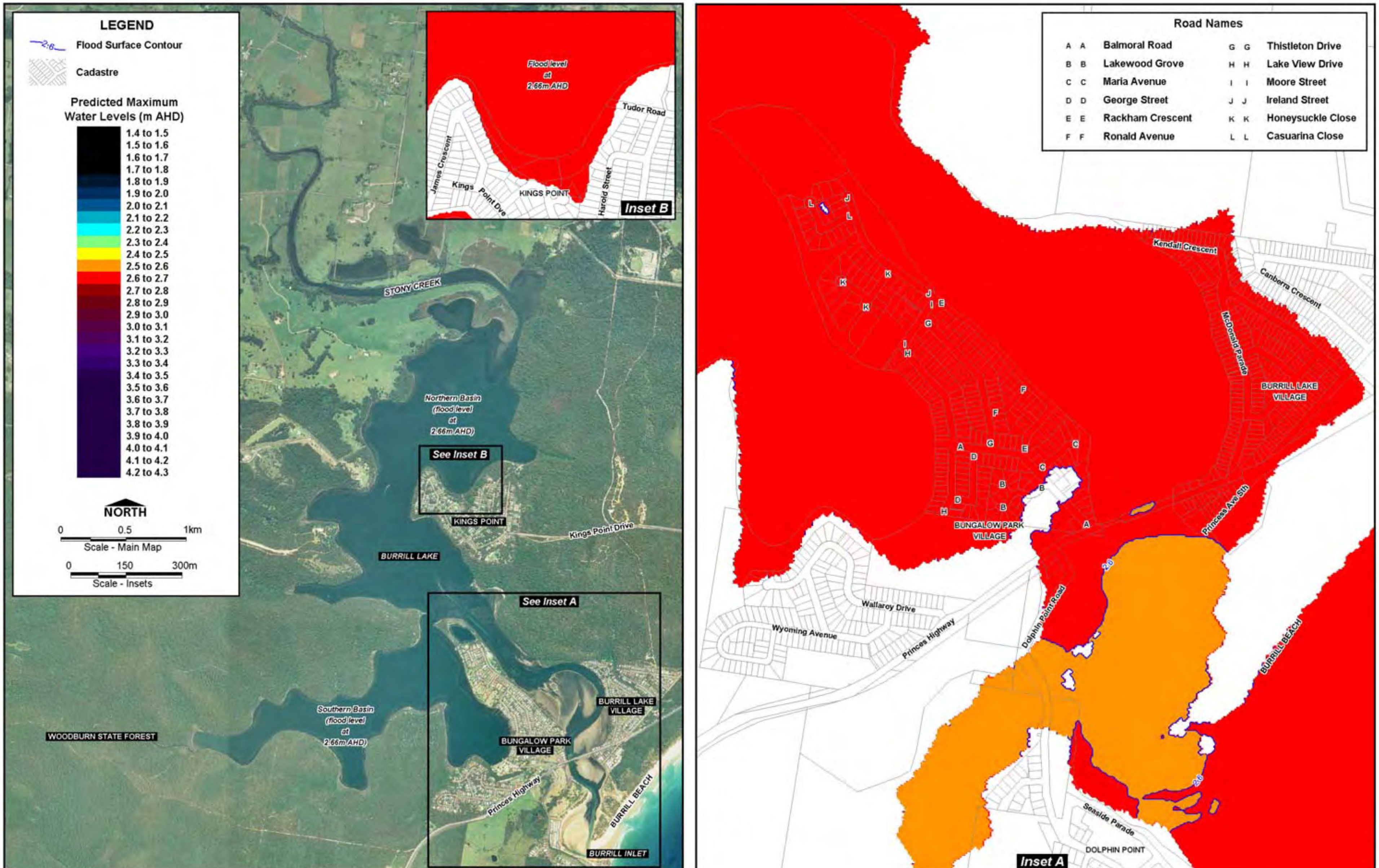
Maximum 20 Year ARI Design Flood Water Levels

Figure 8-10



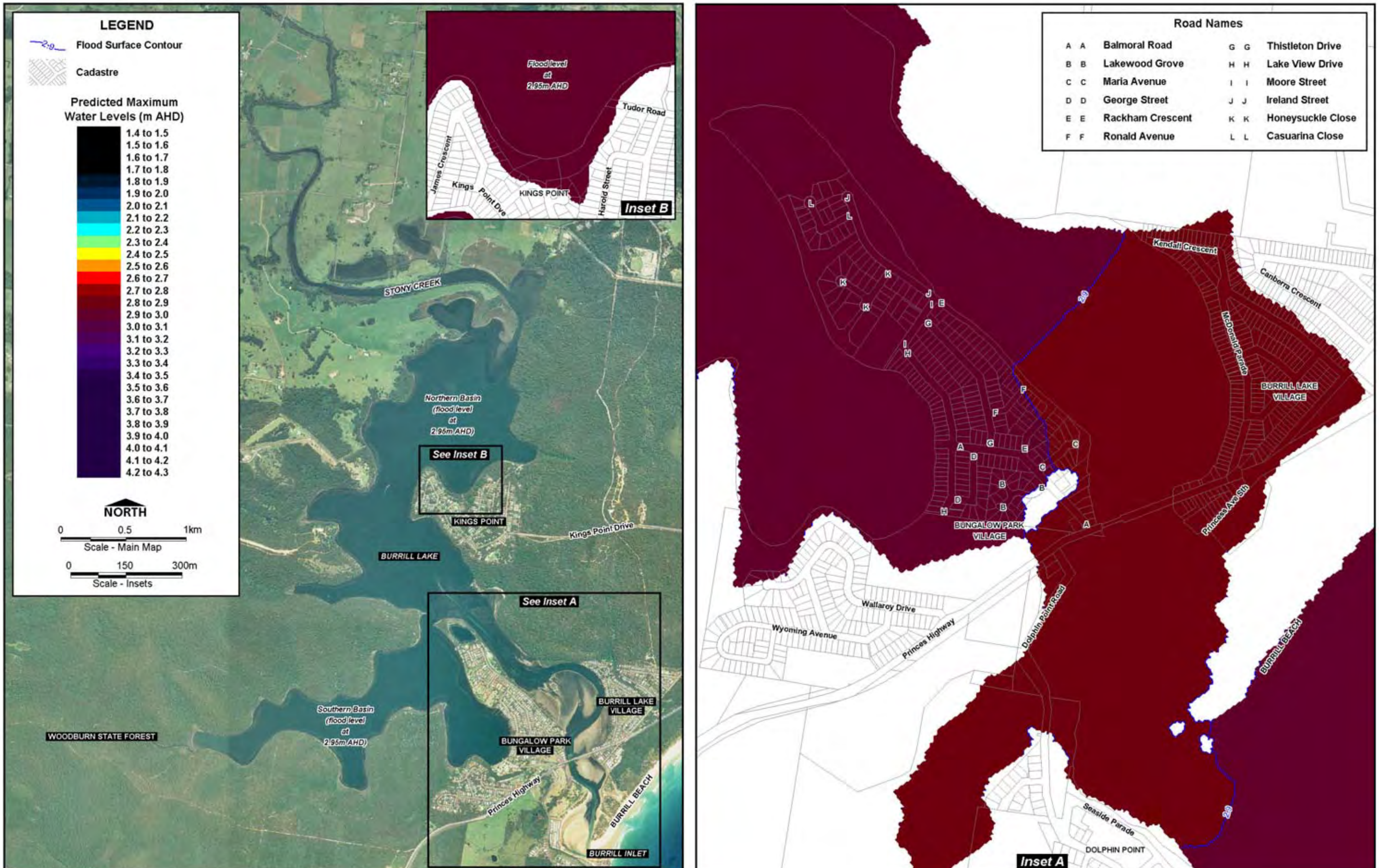
Maximum 50 Year ARI Design Flood Water Levels

Figure 8-11



Maximum 100 Year ARI Design Flood Water Levels

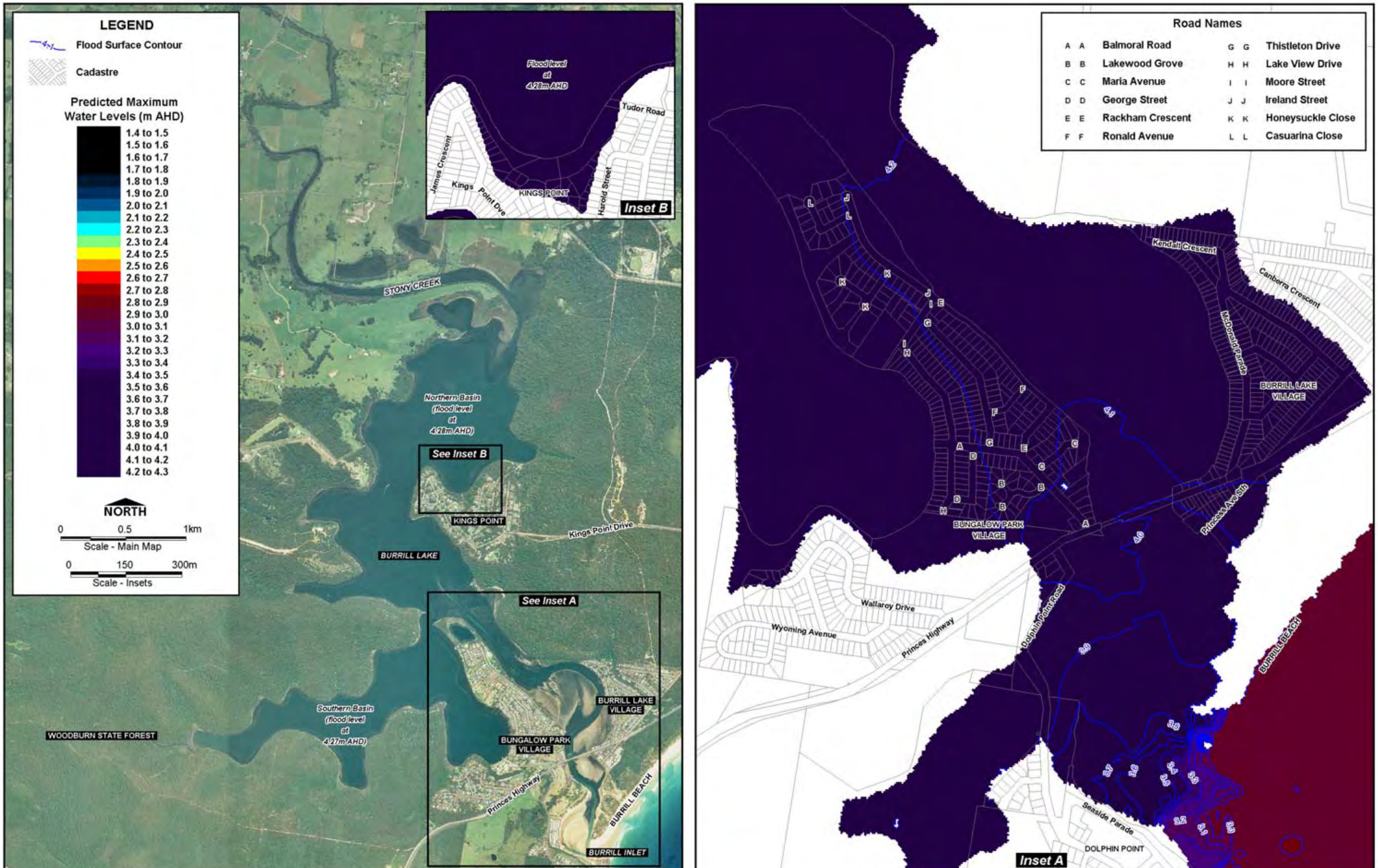
Figure 8-12



I:\N0848_I_BRH Burrill DW\DRG\FLD_029_061017_BL 200year flood level.wor

Maximum 200 Year ARI Design Flood Water Levels

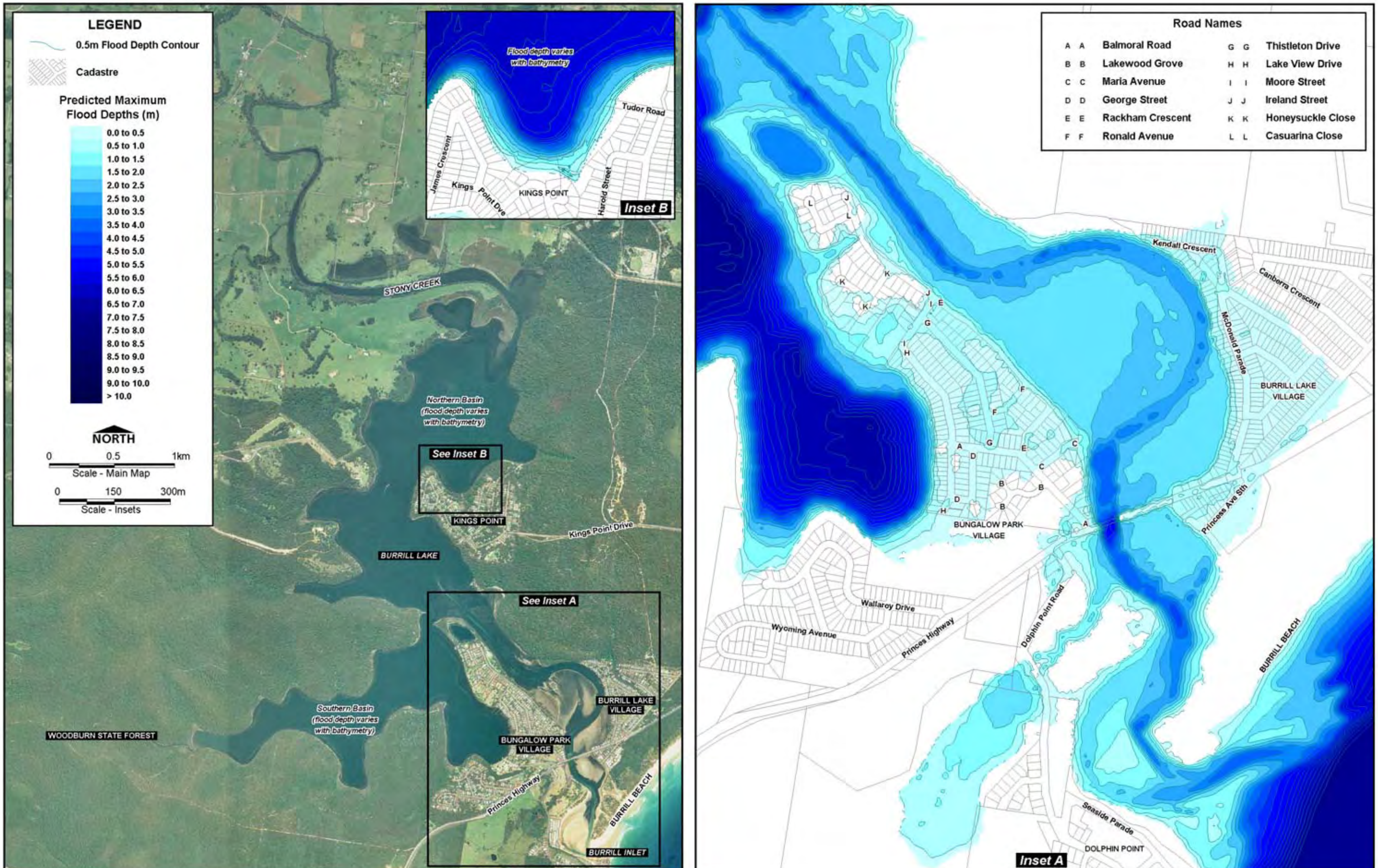
Figure 8-13



I:\N0848_I_BRH Burrill DW\DRG\FLD_030_061018_BL PMF flood level.wor

Maximum PMF Design Flood Water Levels

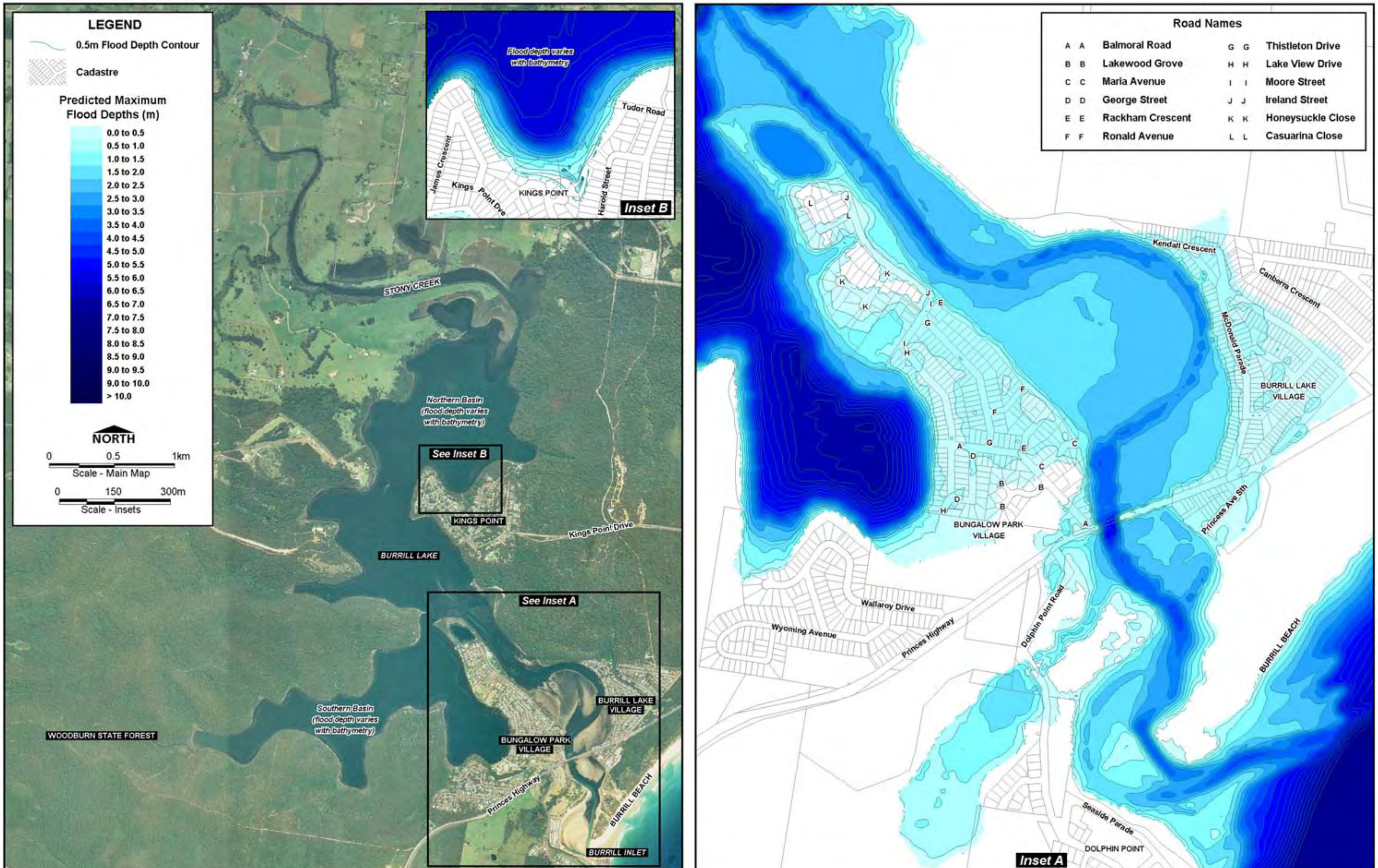
Figure 8-14



I:\N0848_I_BRH Burril DW\DRG\FLD_031_061018_BL 5year flood depth.wor

Maximum 5 Year ARI Design Flood Depths

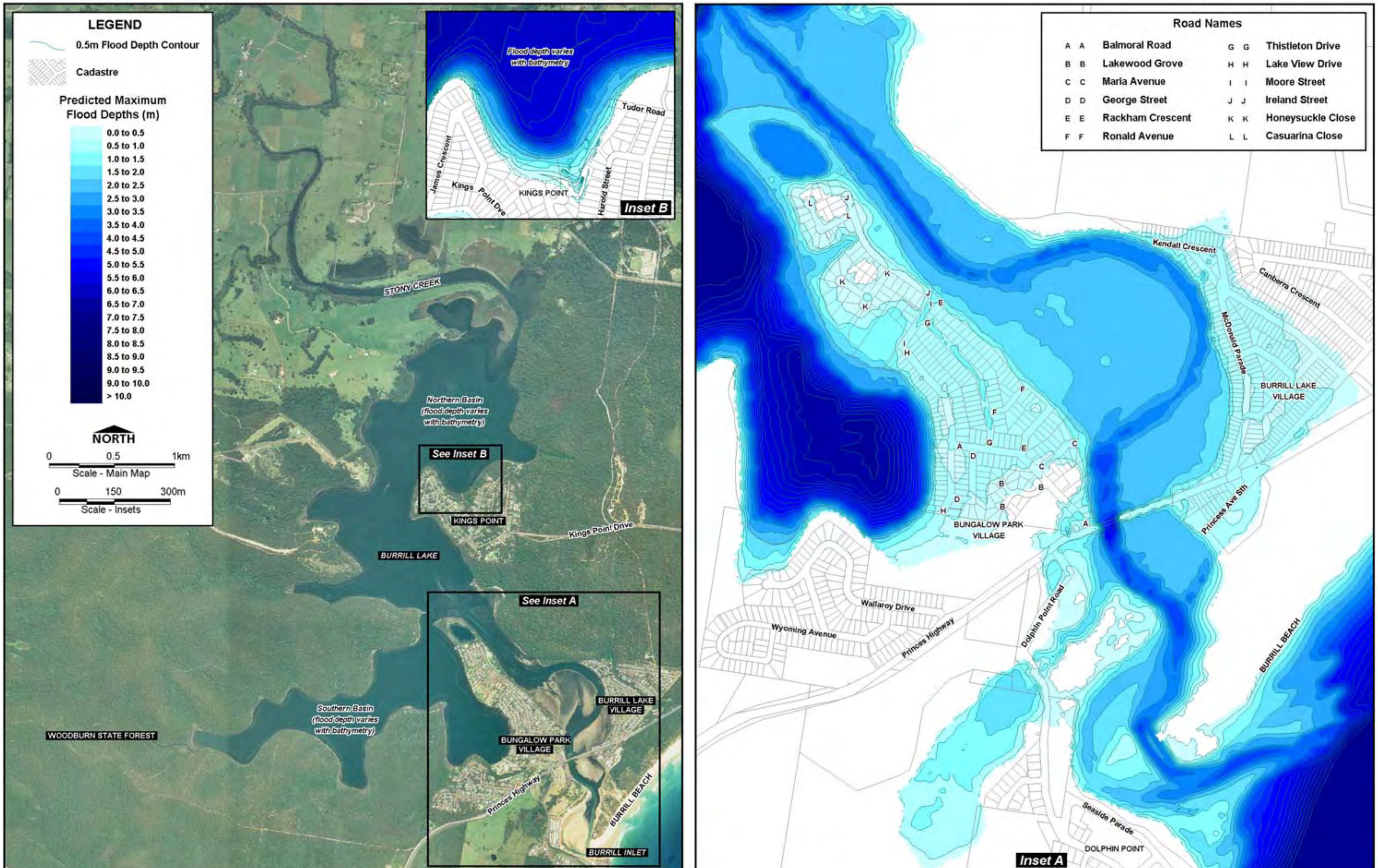
Figure 8-15



I:\W0848_I_BRH Burrill DW\DRG\FLD_032_061018_BL 10year flood depth.wor

Maximum 10 Year ARI Design Flood Depths

Figure 8-16

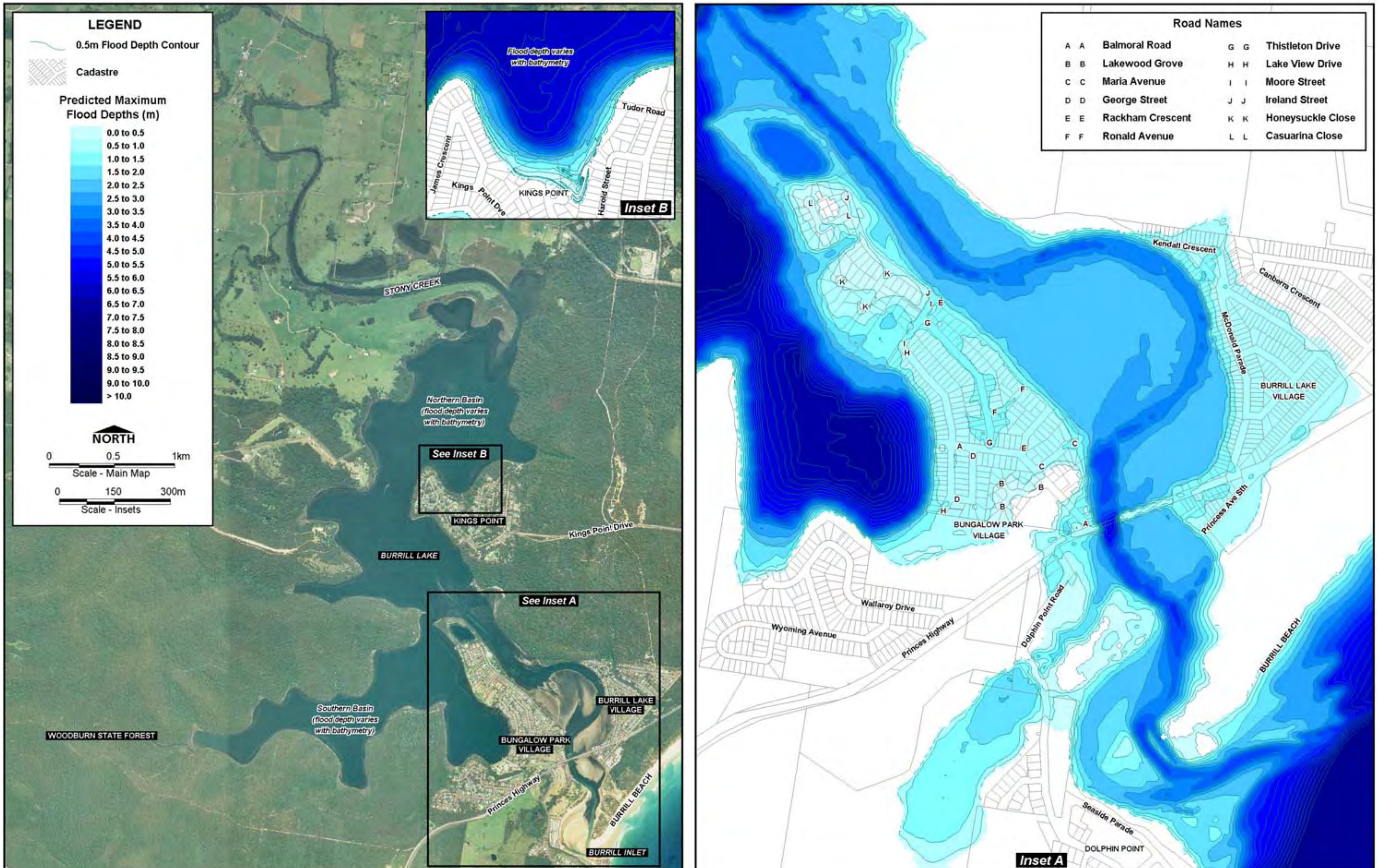


I:\N0848_I_BRH Burrill DW\DRG\FLD_033_061018_BL 20year flood depth.wor

Maximum 20 Year ARI Design Flood Depths

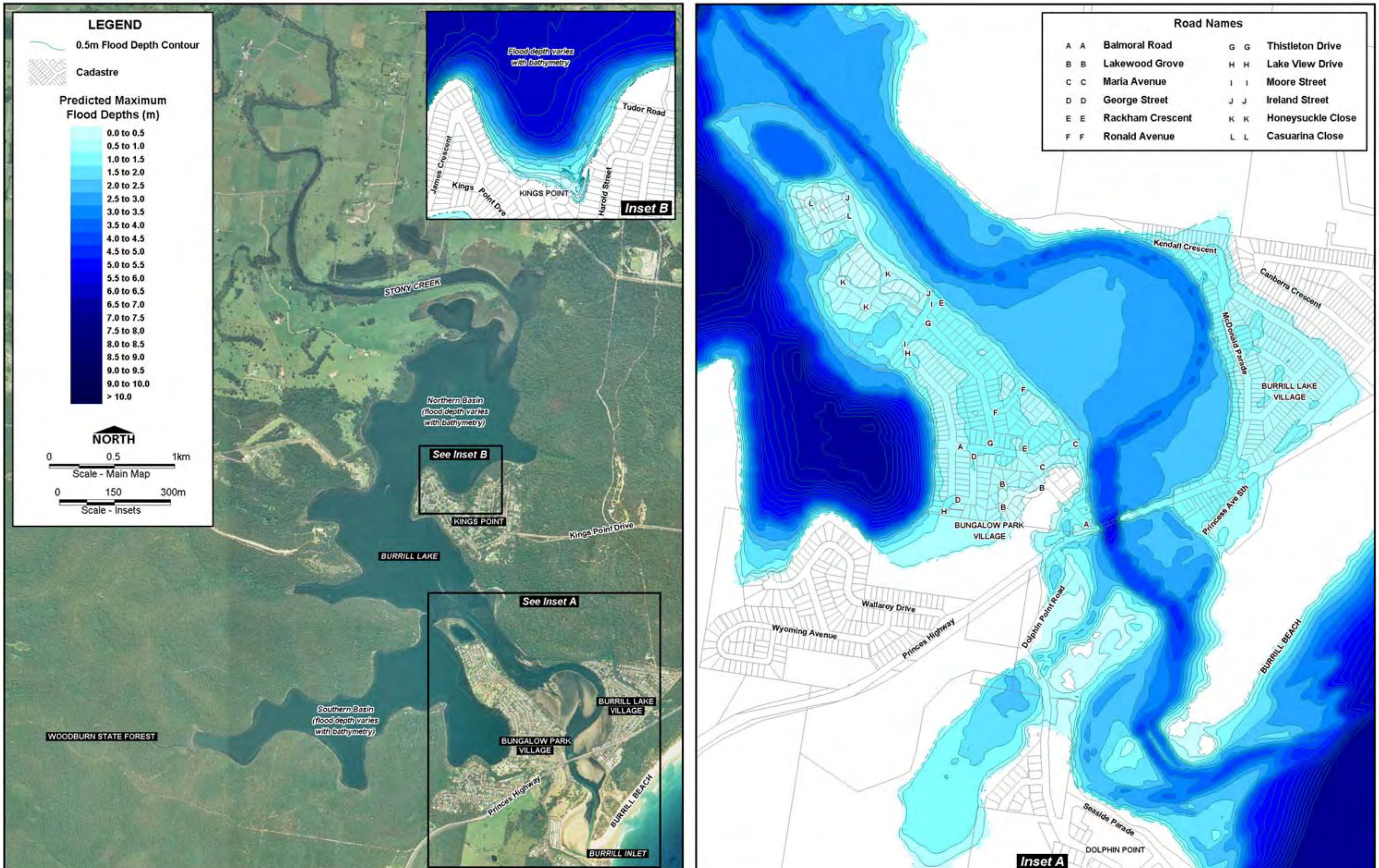
Figure 8-17

BMT WBM



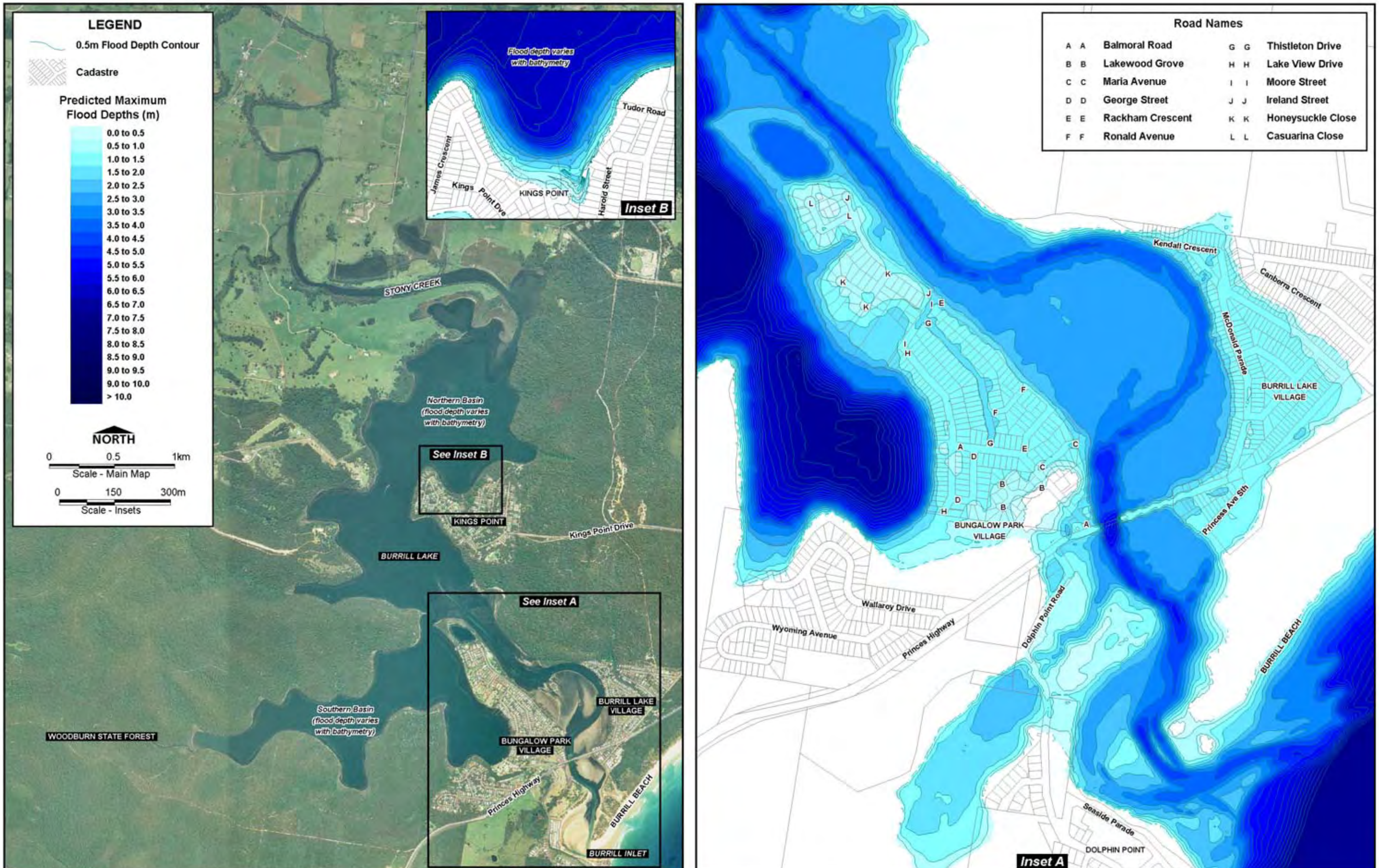
Maximum 50 Year ARI Design Flood Depths

Figure 8-18



Maximum 100 Year ARI Design Flood Depths

Figure 8-19

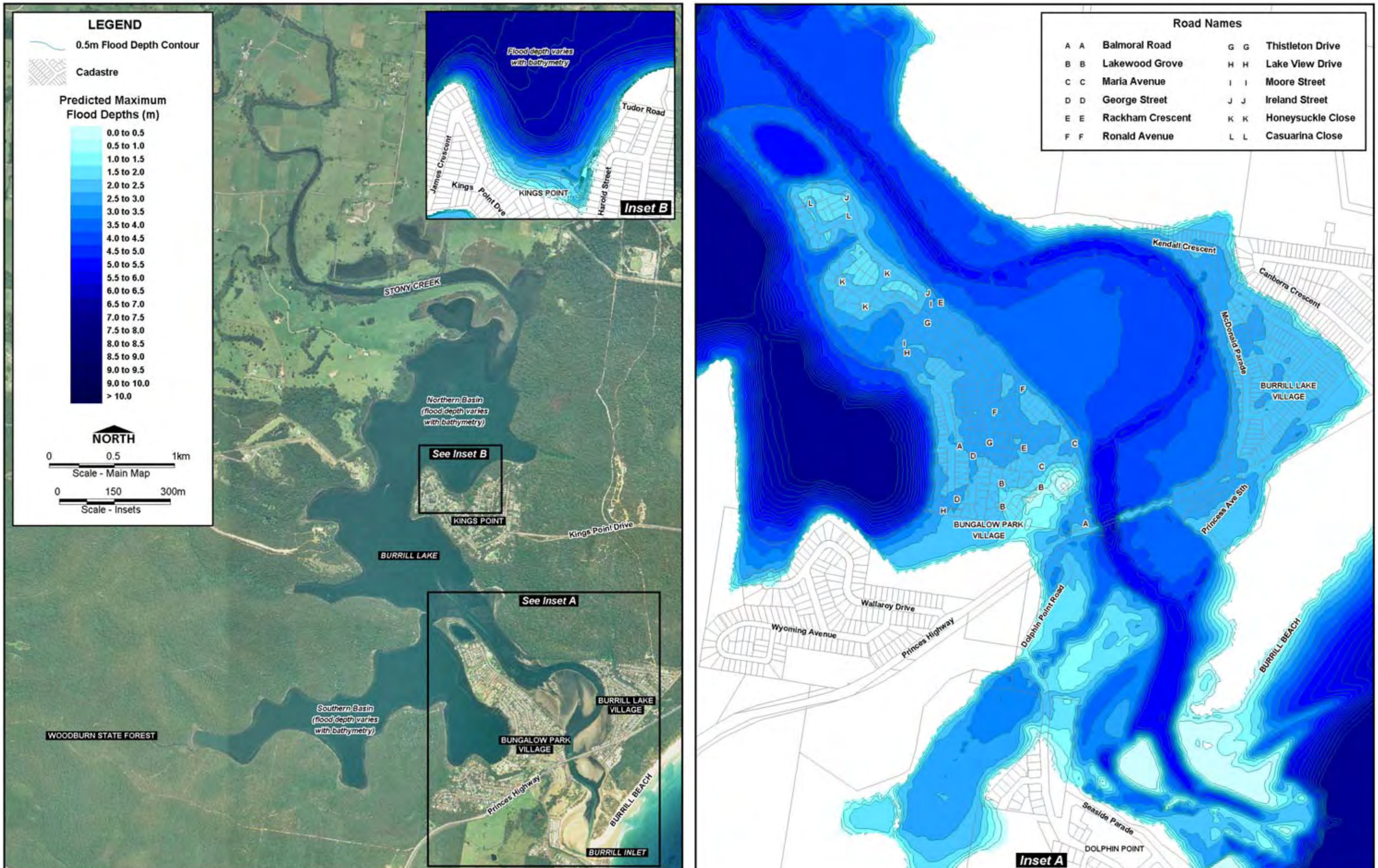


I:\N0848_I_BRH Burril DW\DRG\FLD_036_061018_BL 200year flood depth.wor

Maximum 200 Year ARI Design Flood Depths

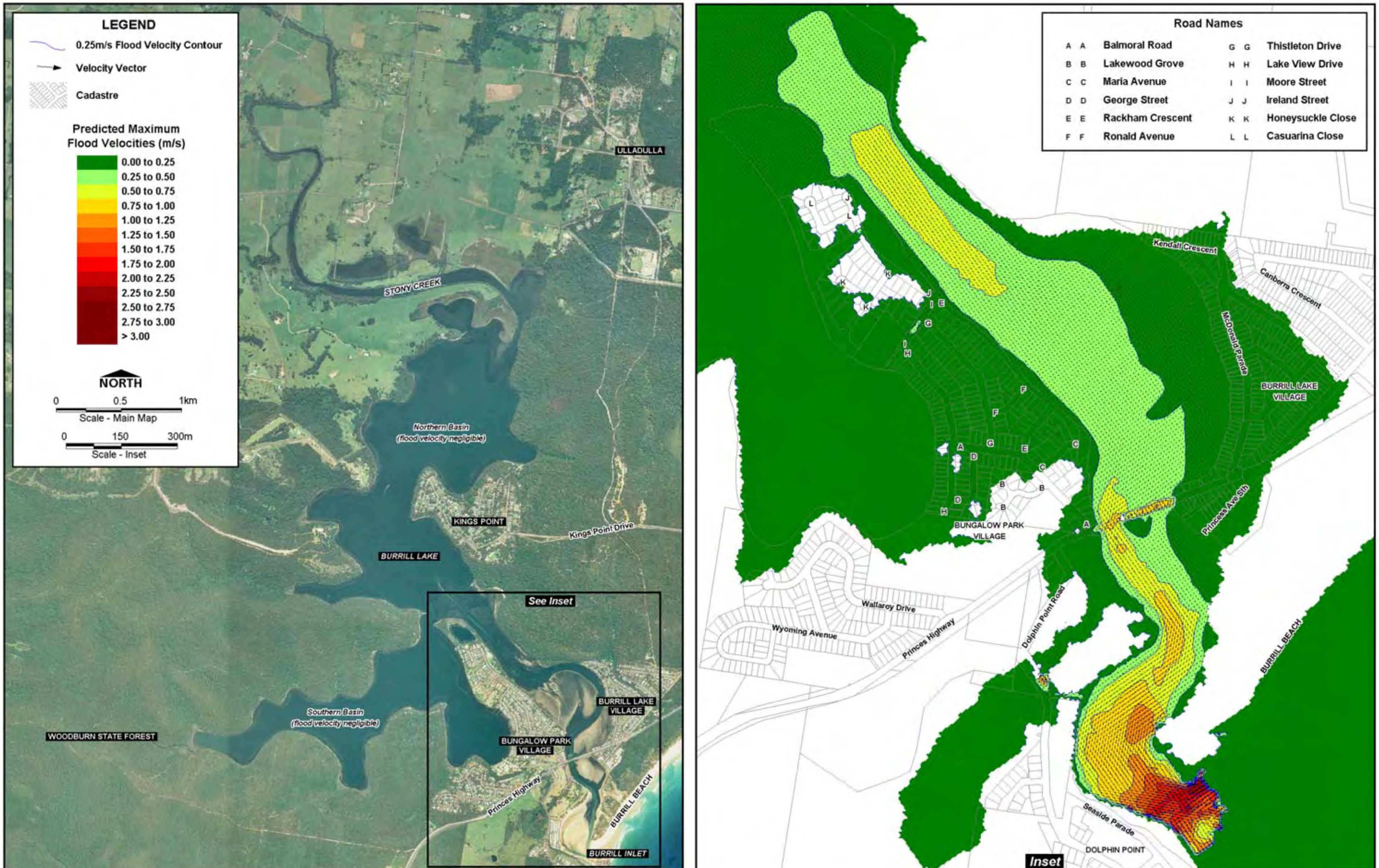
Figure 8-20

BMT WBM



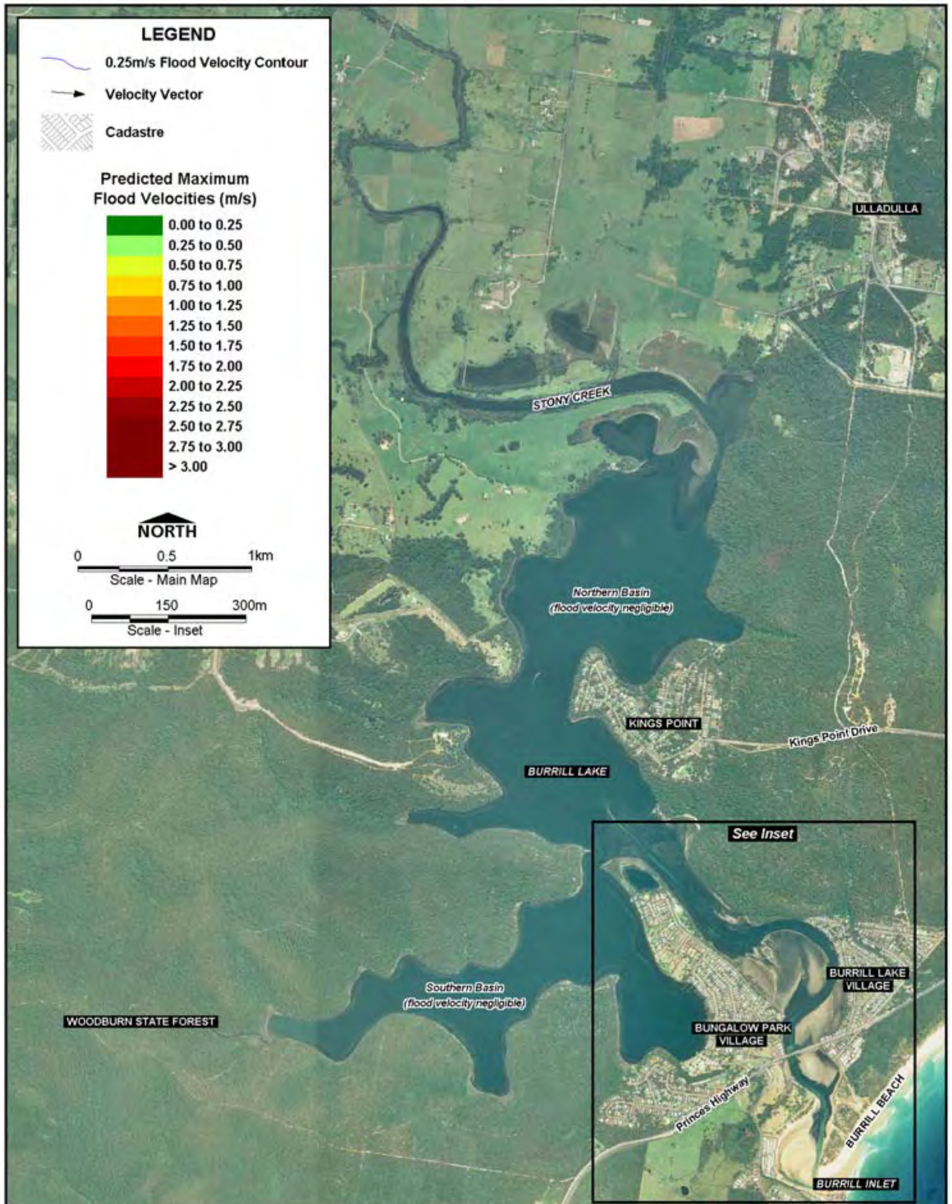
Maximum PMF Design Flood Depths

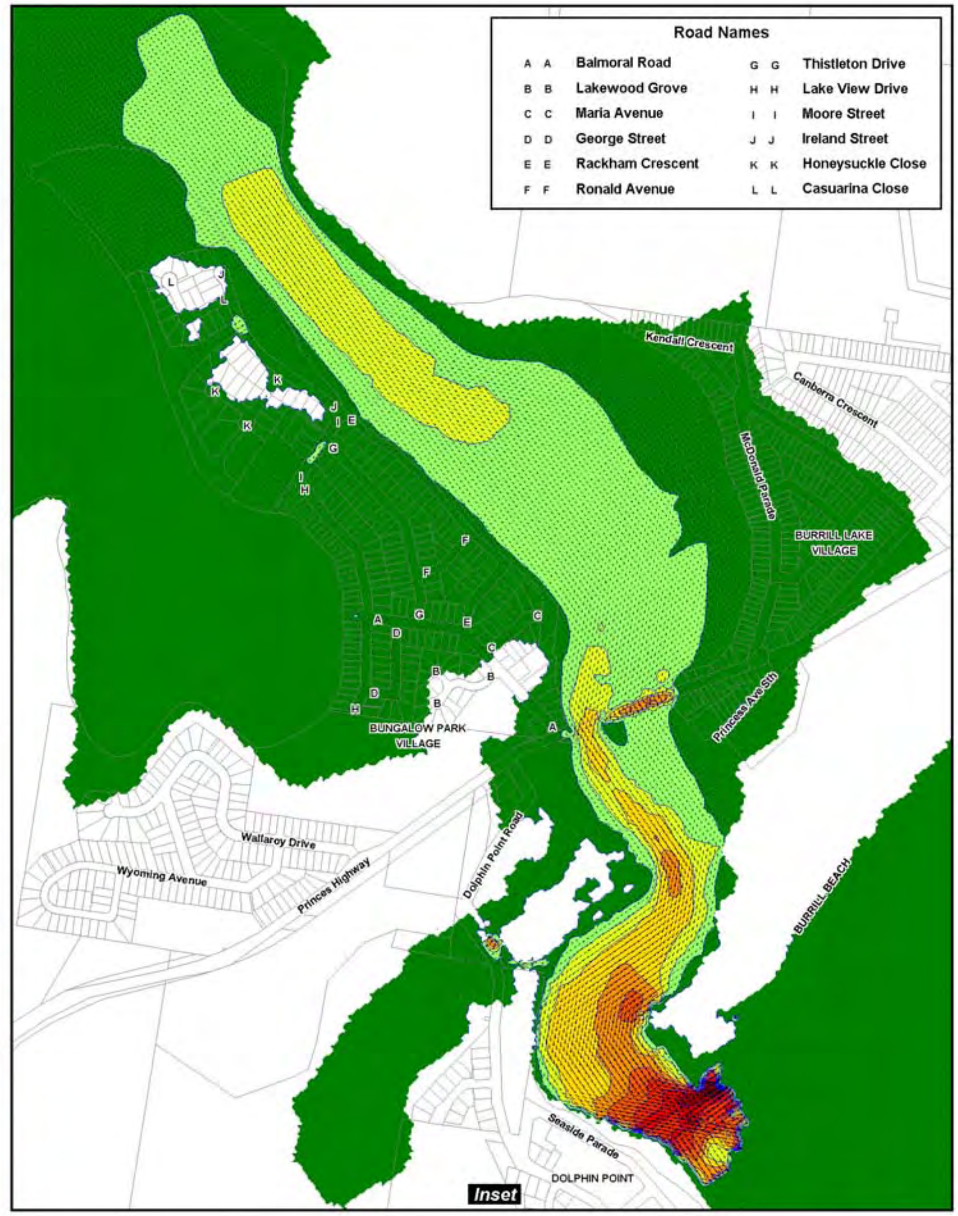
Figure 8-21



Maximum 5 Year ARI Design Flood Velocities

Figure 8-22

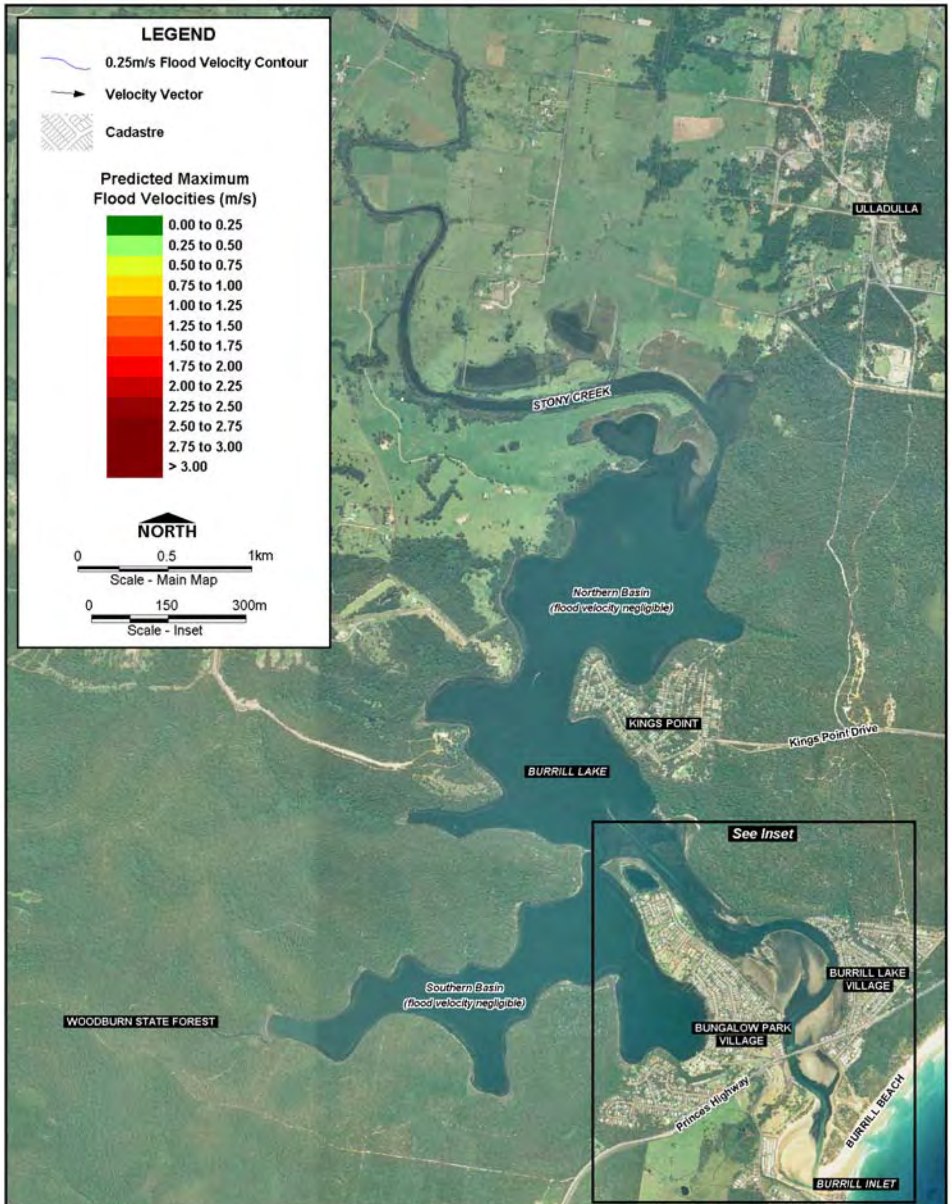


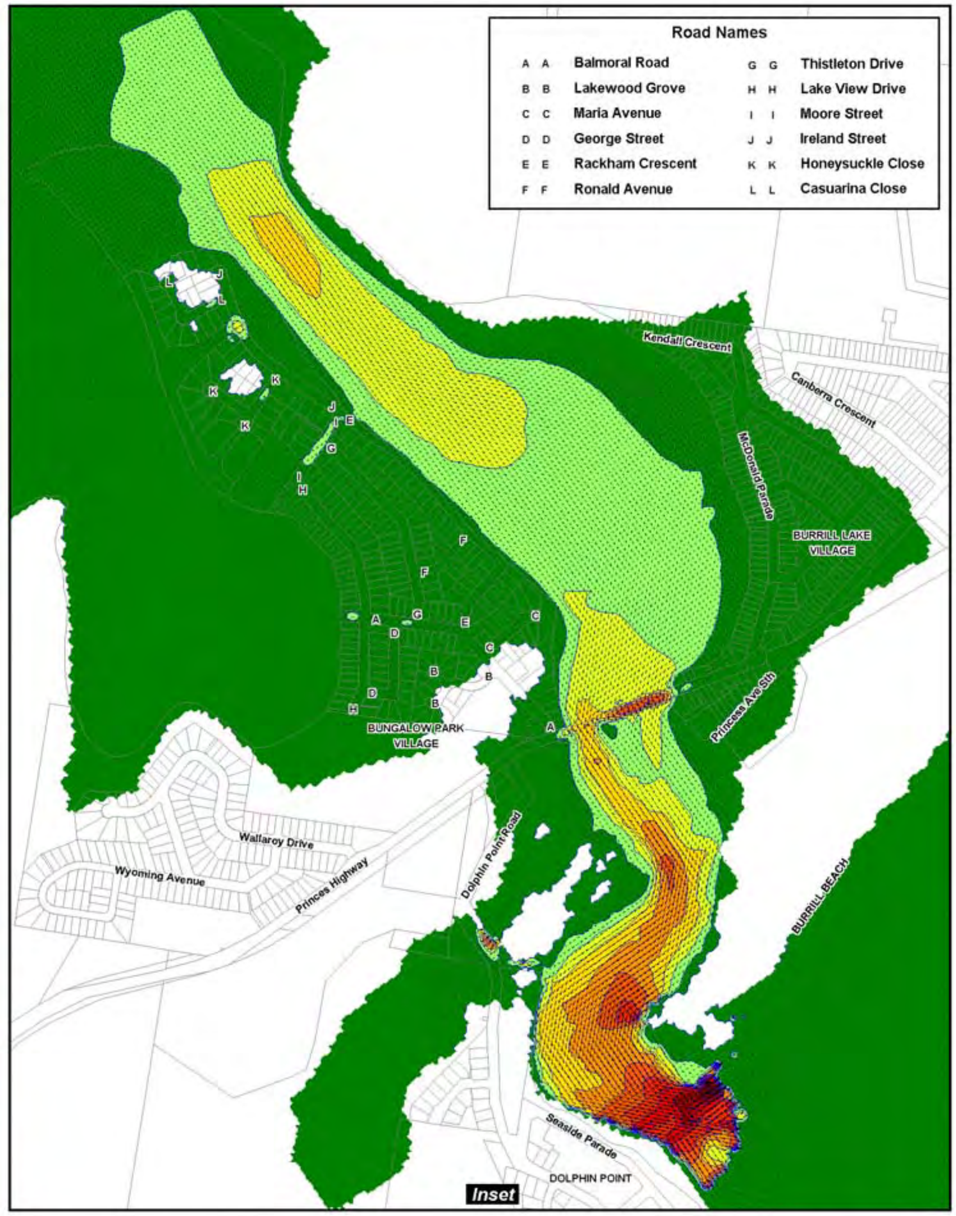


I:\N0848_I_BRH Burril DW\DRG\FLD_039_061018_BL 10year flood velocity.wor

Maximum 10 Year ARI Design Flood Velocities

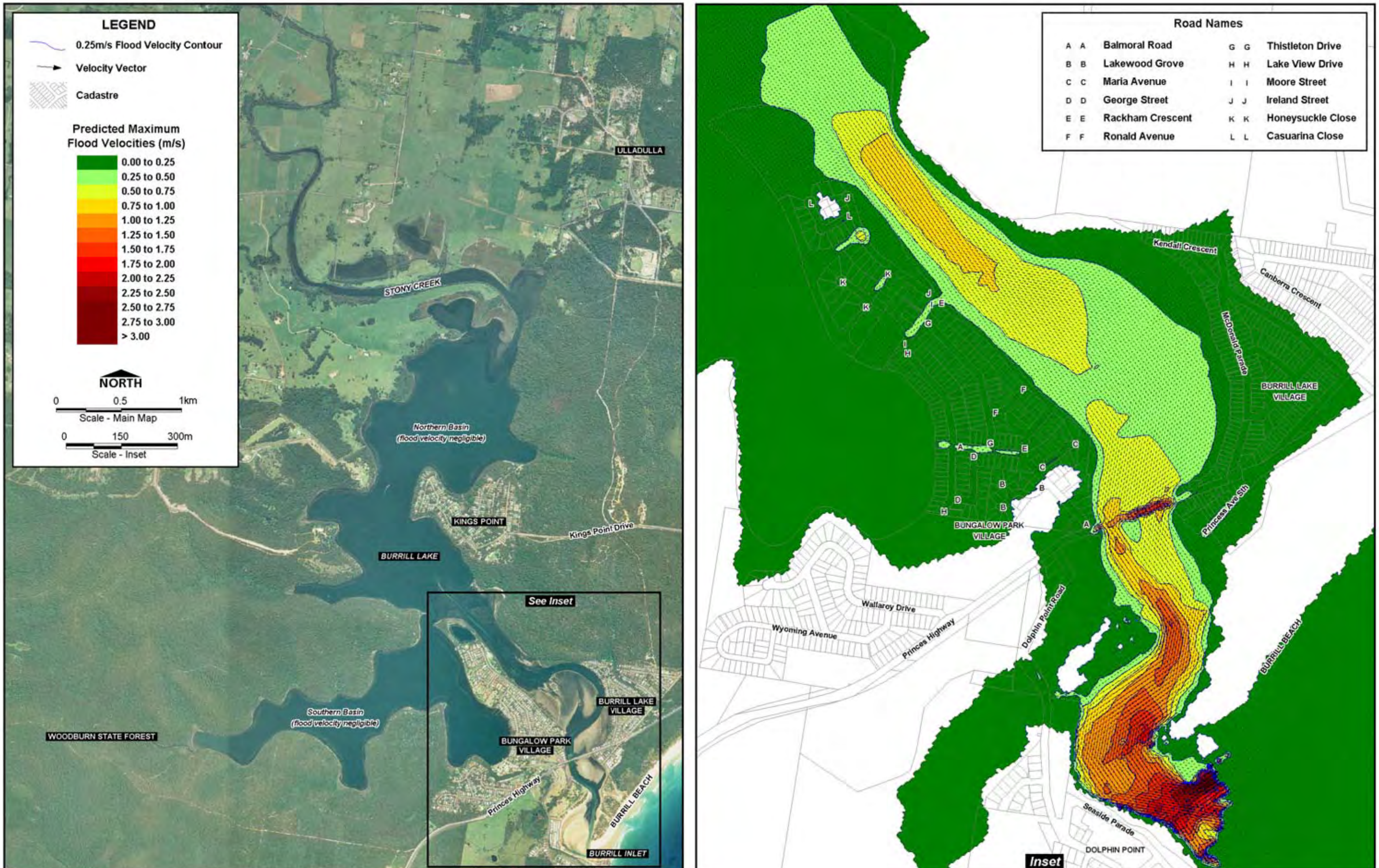
Figure 8-23





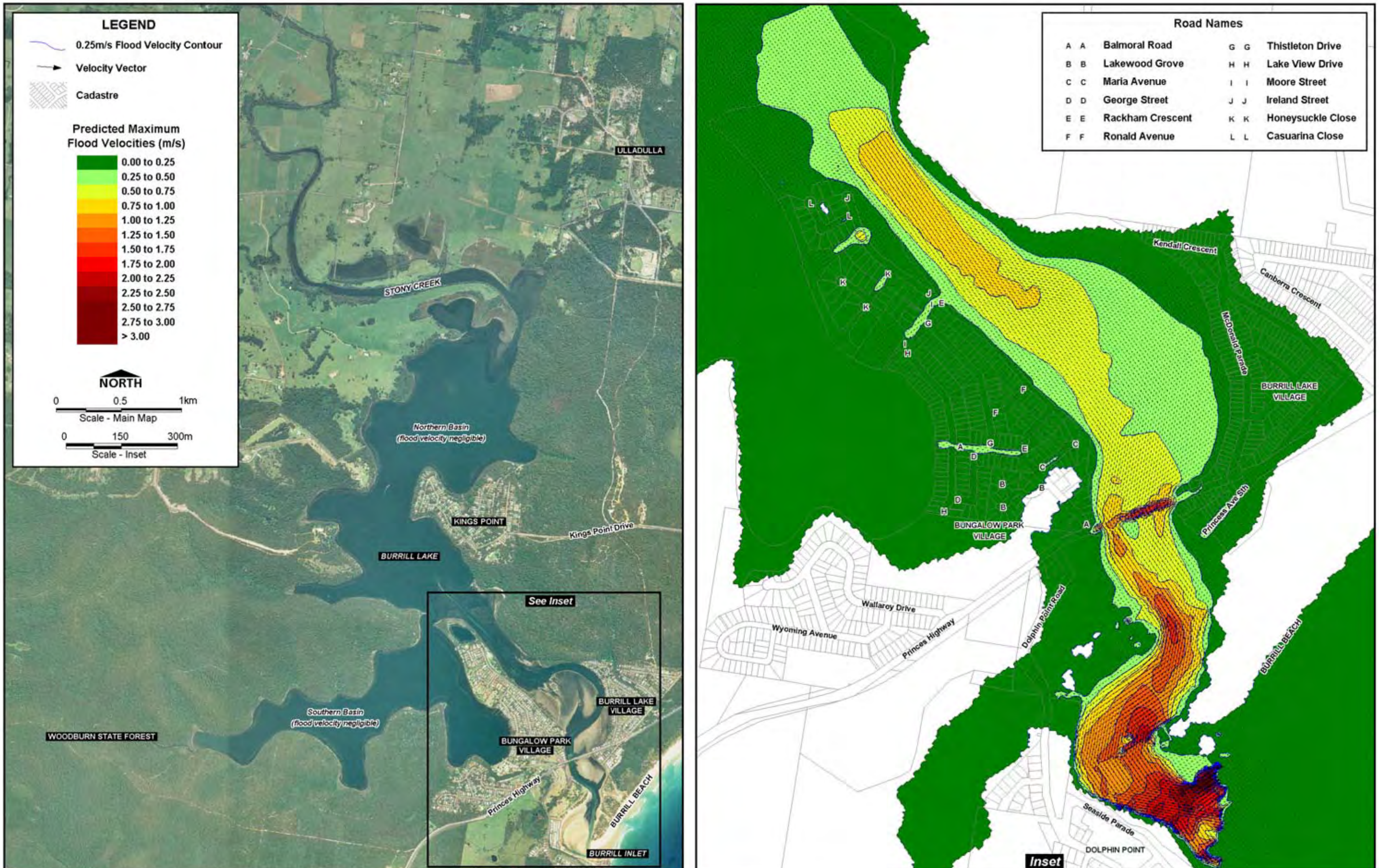
Maximum 20 Year ARI Design Flood Velocities

Figure 8-24



Maximum 50 Year ARI Design Flood Velocities

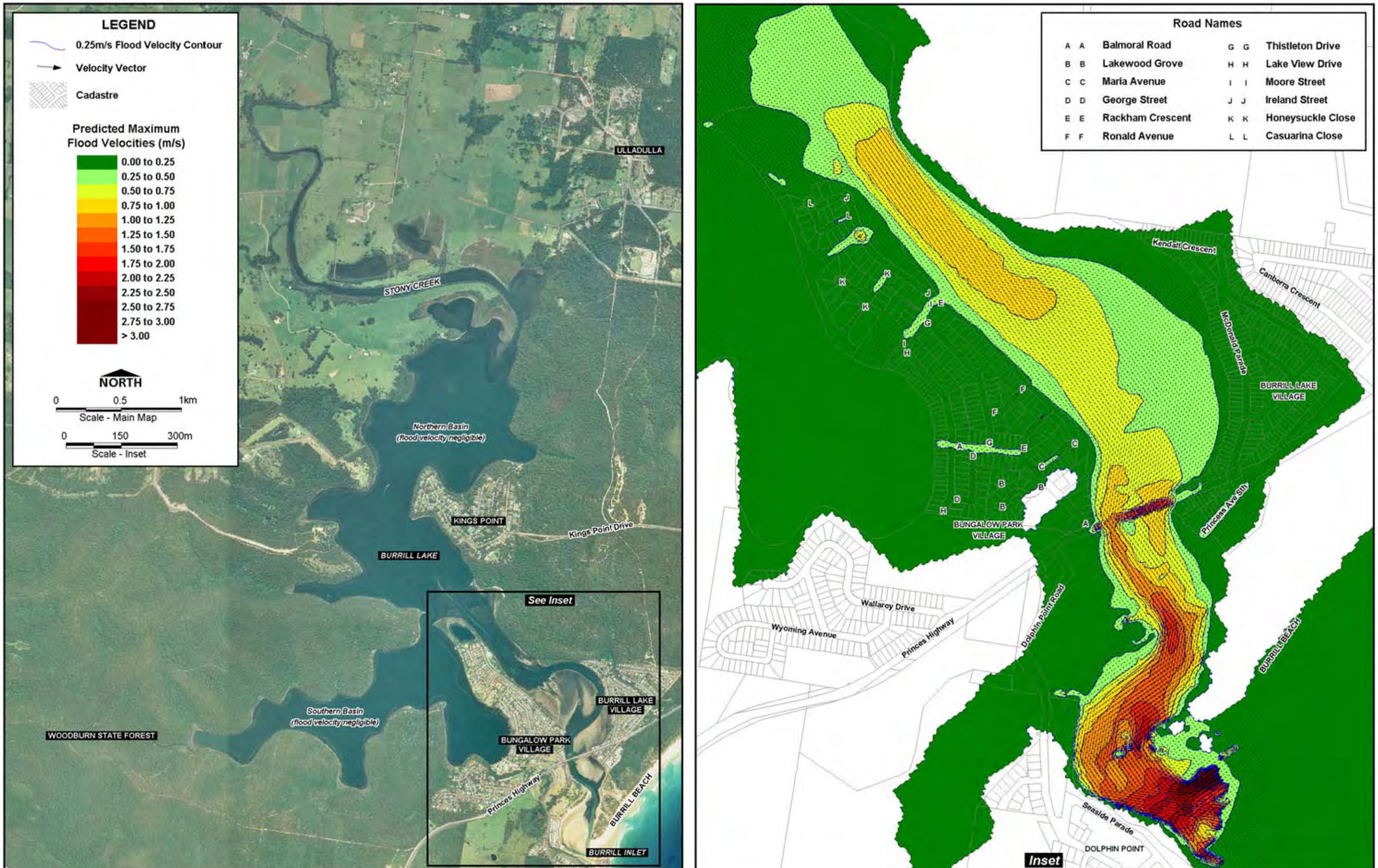
Figure 8-25



I:\N0848_I_BRH Burrill DW\DRG\FLD_042_061018_BL 100year flood velocity.wor

Maximum 100 Year ARI Design Flood Velocities

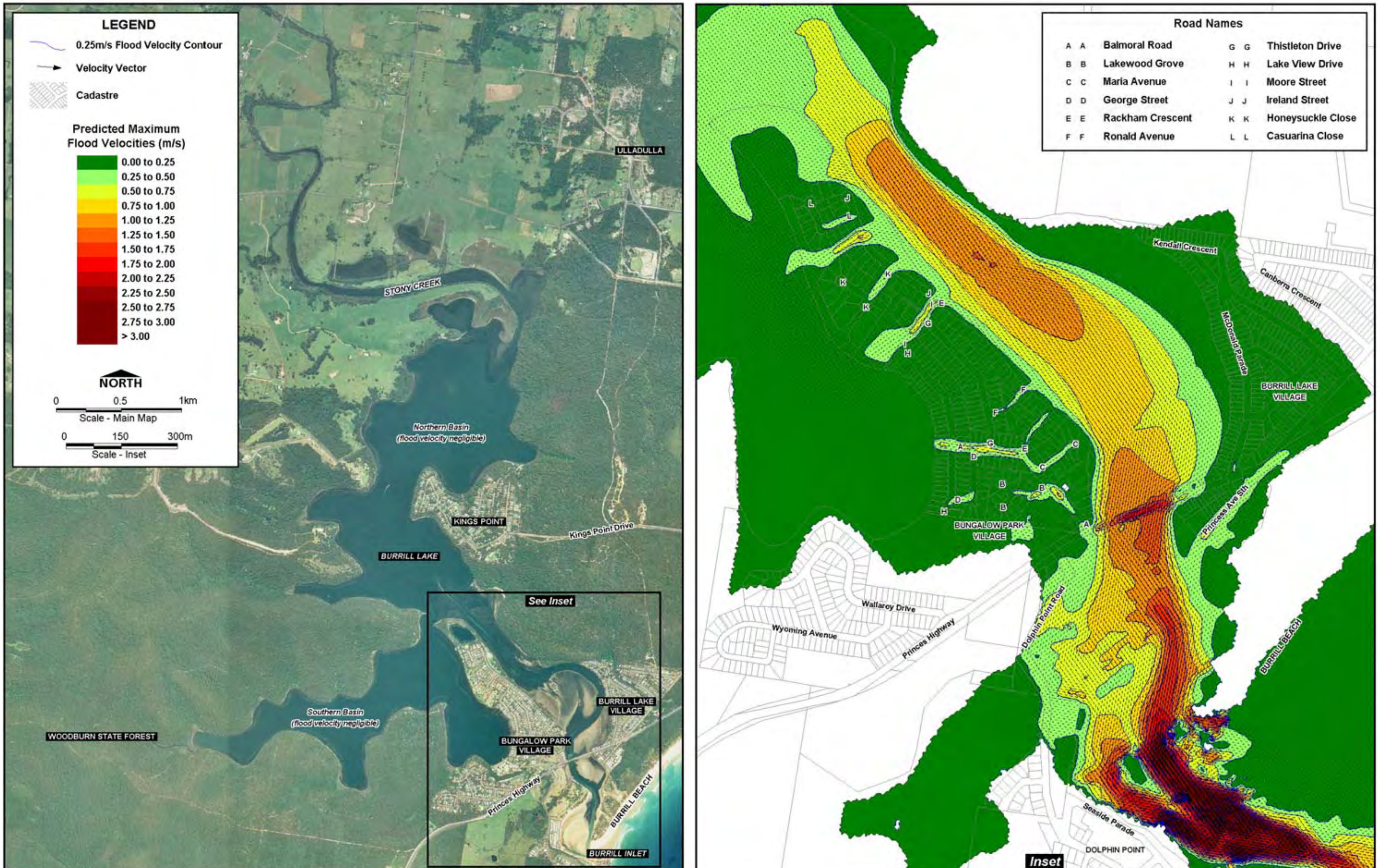
Figure 8-26



I:\N0848_I_BRH Burril DW\DRG\FLD_043_061018_BL 200year flood velocity.wor

Maximum 200 Year ARI Design Flood Velocities

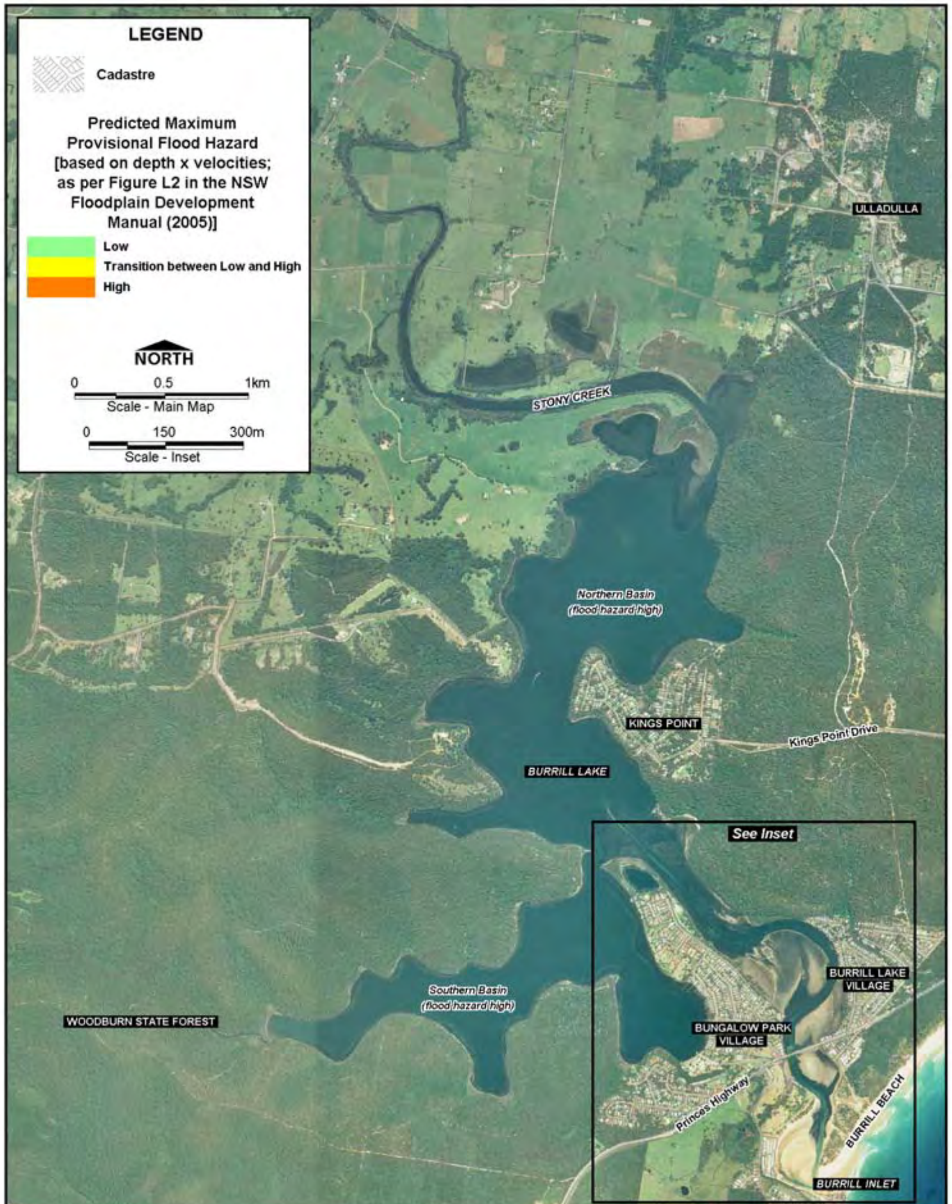
Figure 8-27

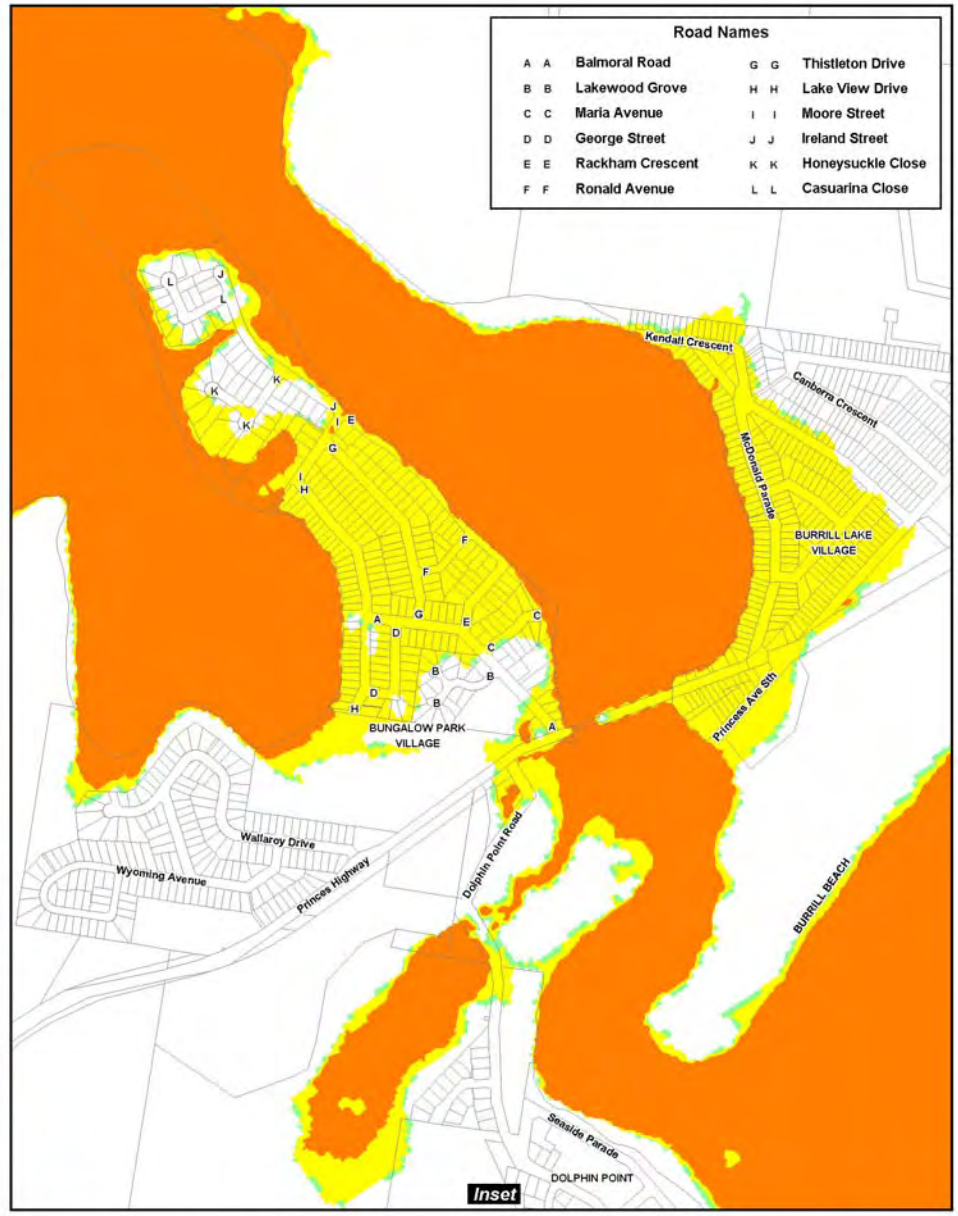


I:\N0848_I_BRH Burrill DW\DRG\FLD_044_061018_BL PMF flood velocity.wor

Maximum PMF Design Flood Velocities

Figure 8-28

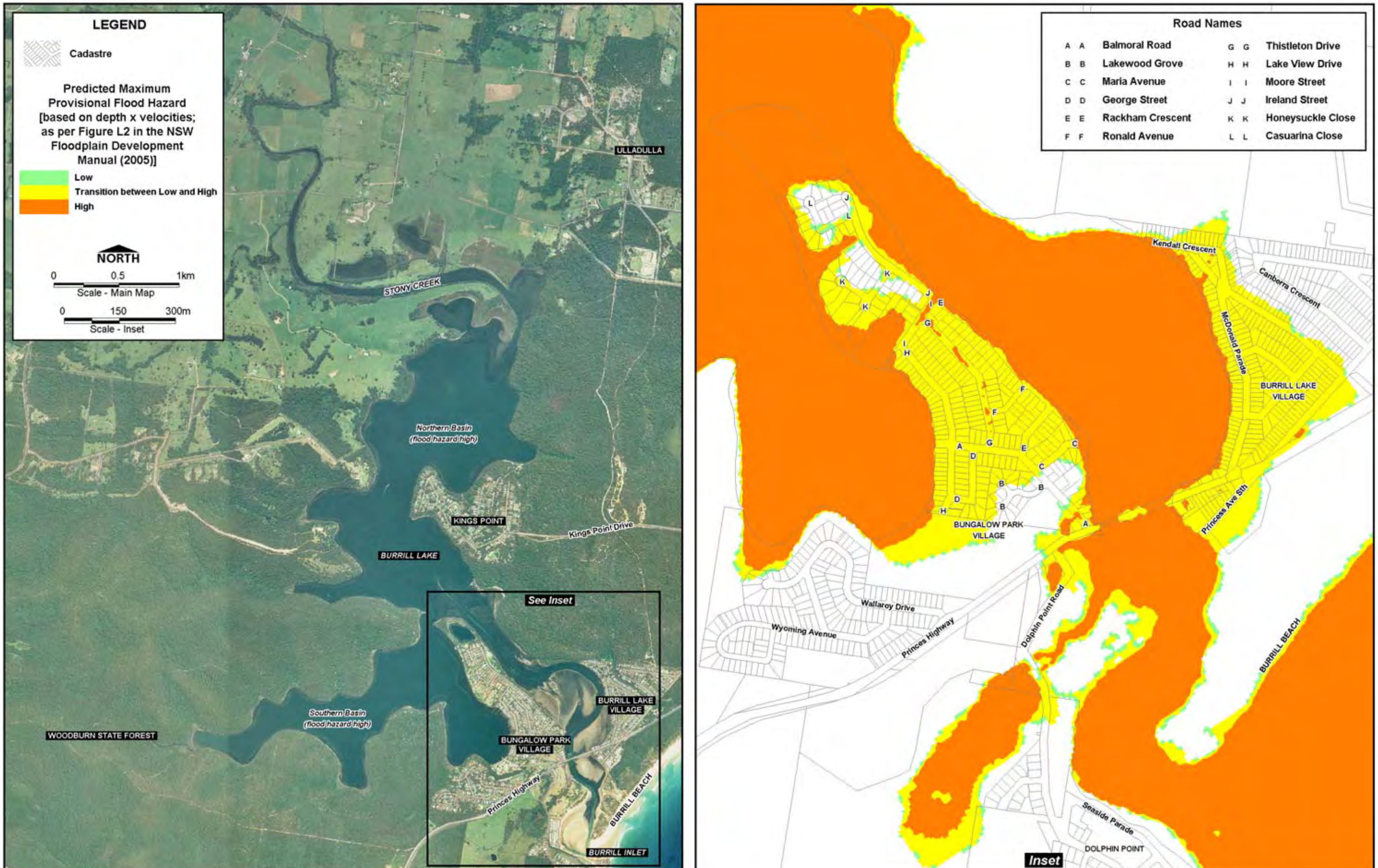




I:\N0848_I_BRH Burril DW\DRG\FLD_045_061018_BL 5year flood hazard.wor

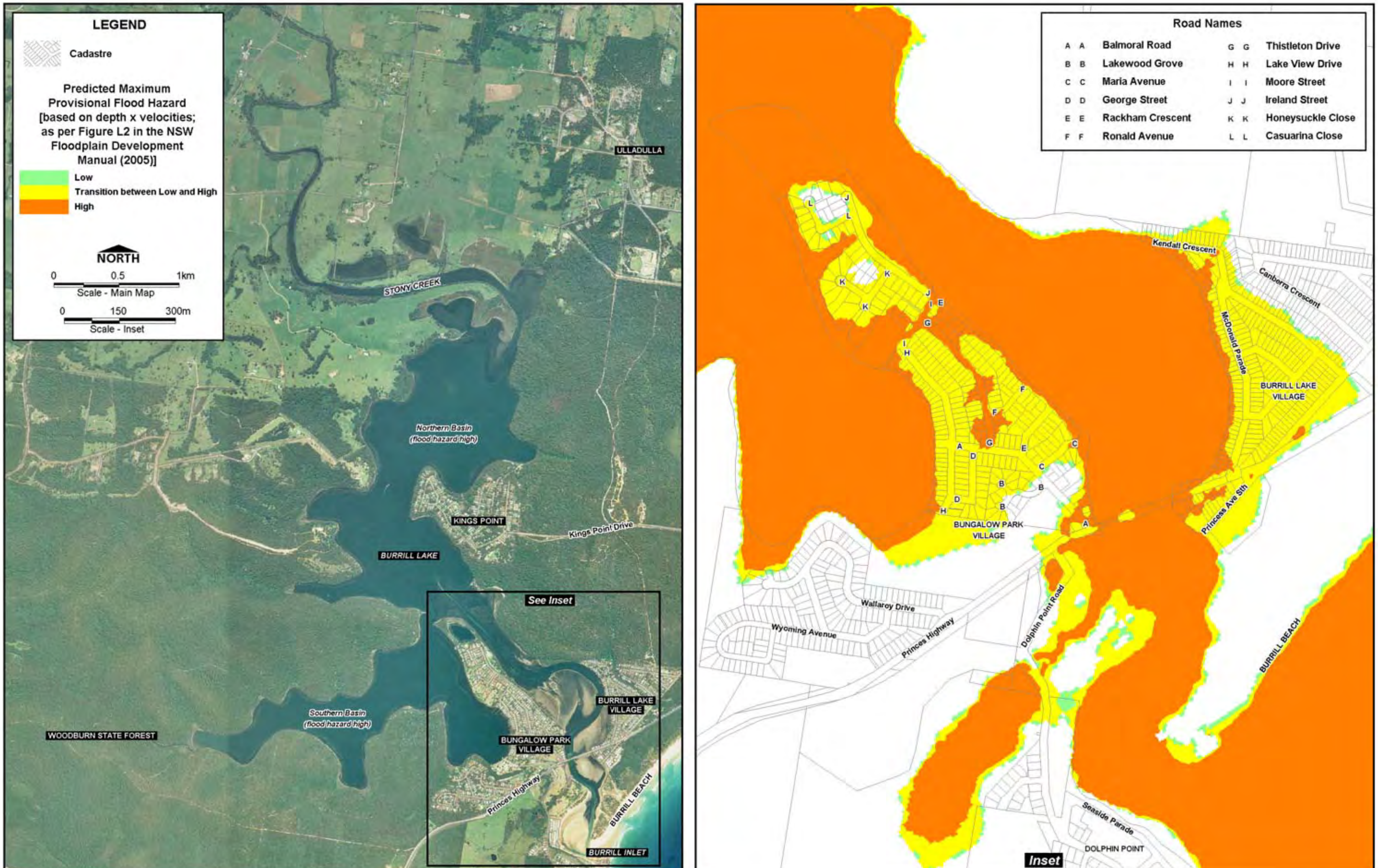
Maximum 5 Year ARI Design Provisional Flood Hazard

Figure 8-29



Maximum 10 Year ARI Design Provisional Flood Hazard

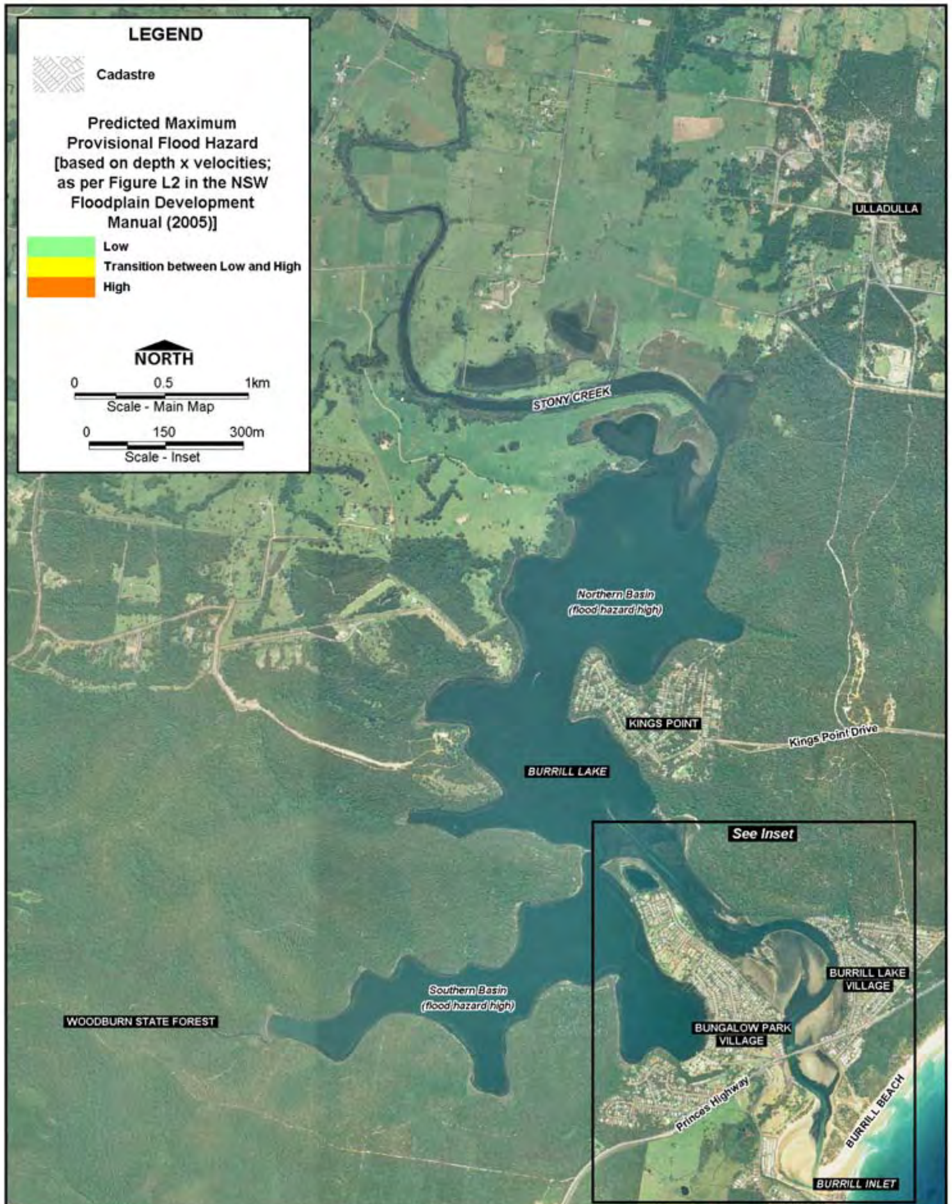
Figure 8-30

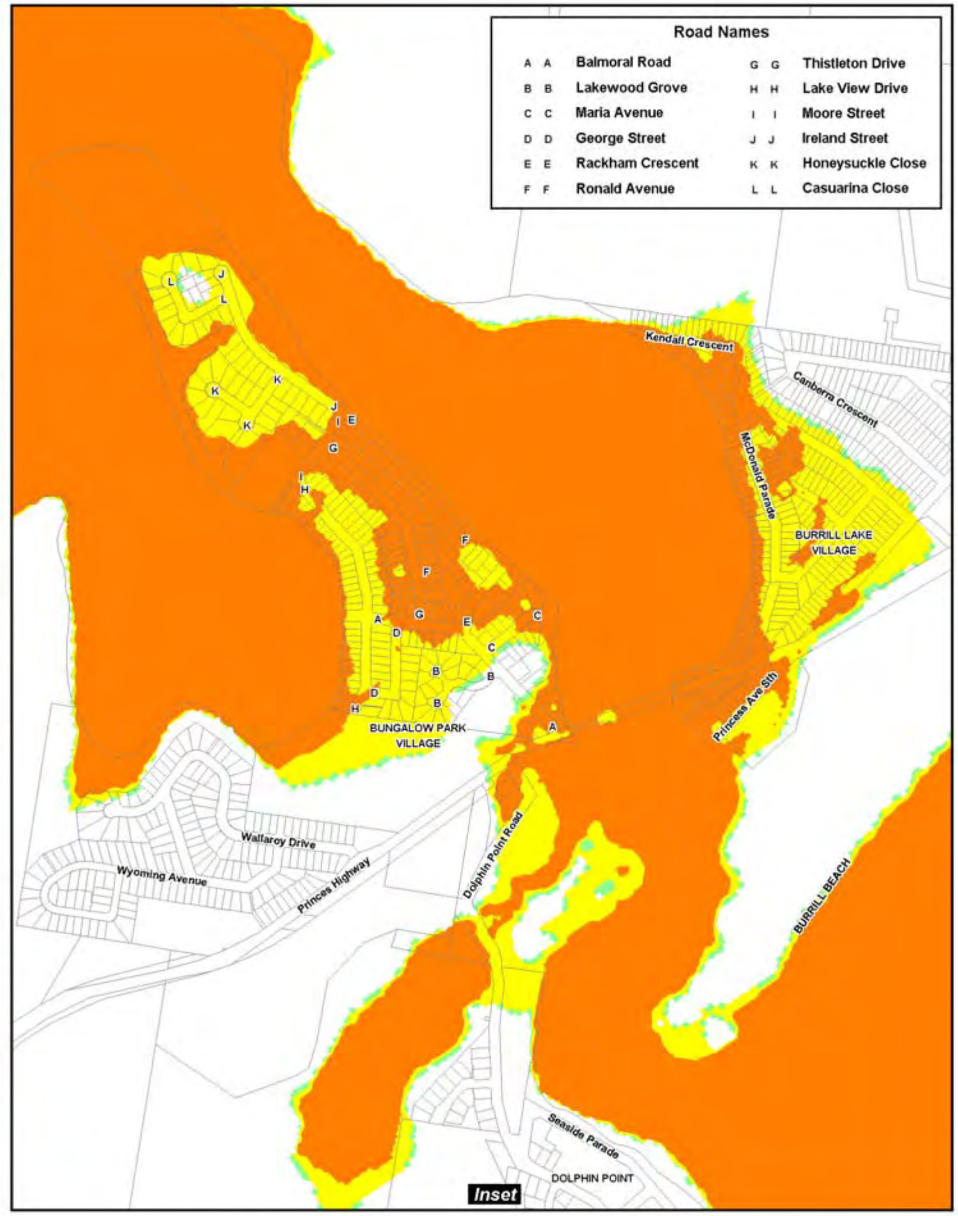


I:\W0848_I_BRH Burrill DW\DRG\FLD_047_061018_BL 20year flood hazard.wor

Maximum 20 Year ARI Design Provisional Flood Hazard

Figure 8-31

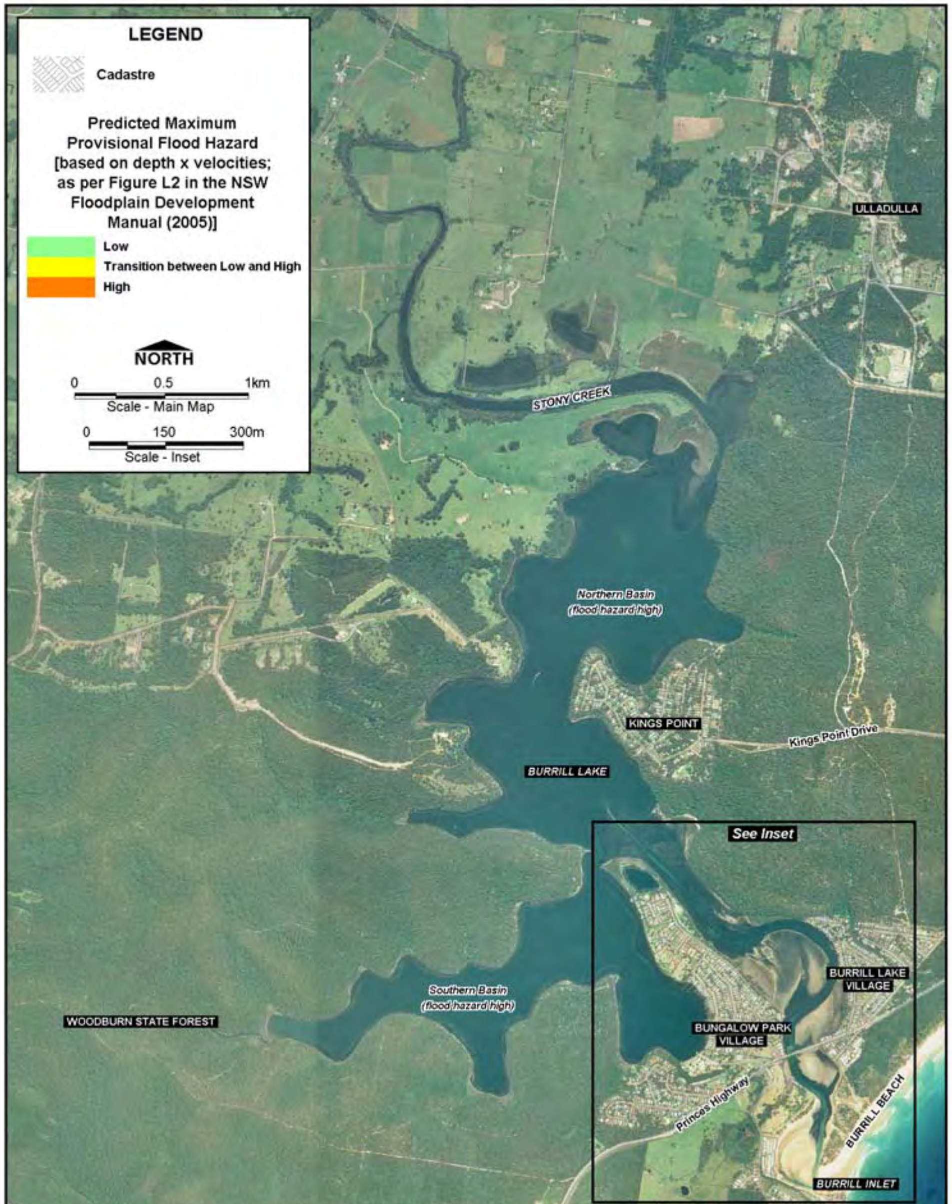


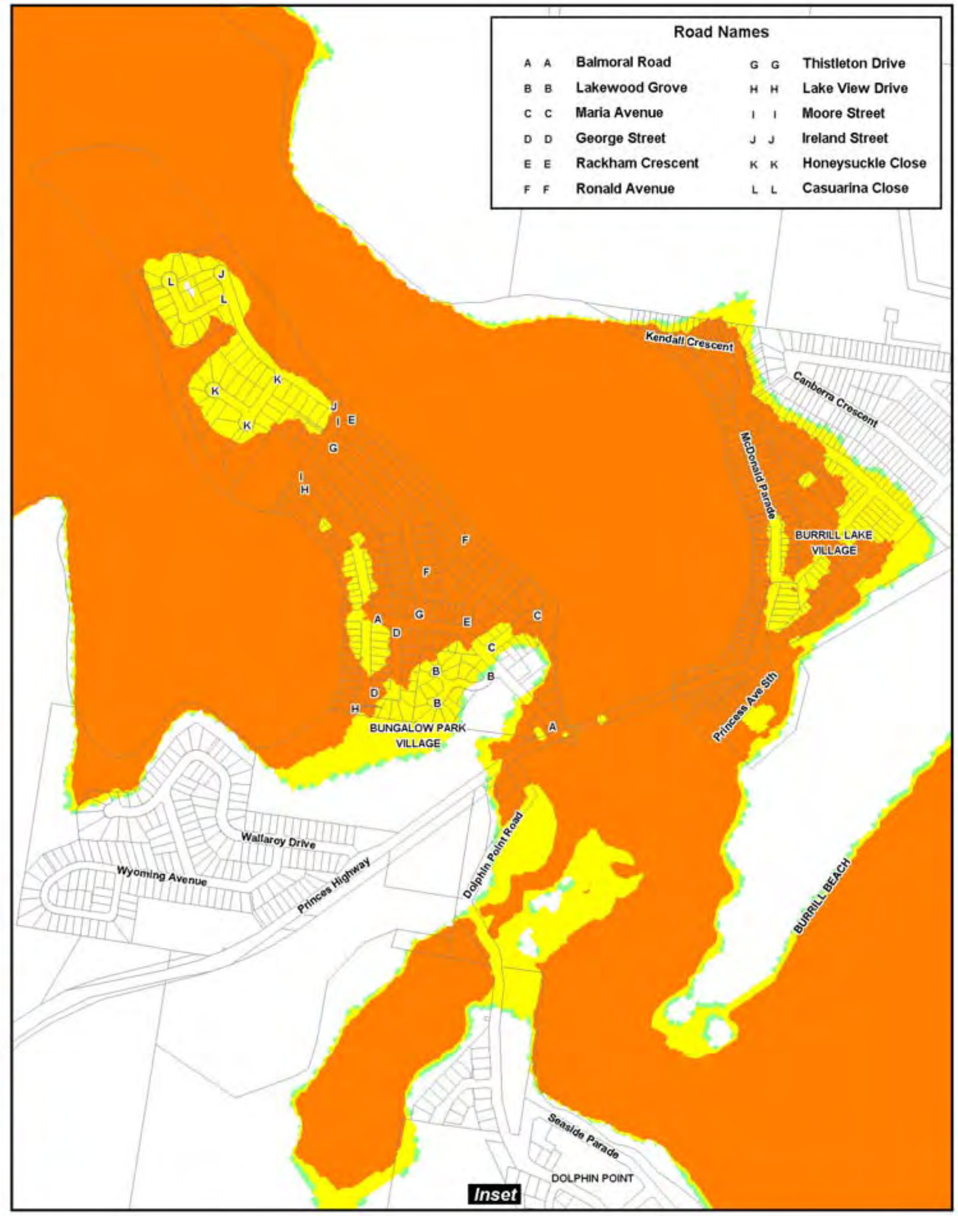


I:\N0848_I_BRH Burrill DW\DRG\FLD_048_061018_BL 50year flood hazard.wor

Maximum 50 Year ARI Design Provisional Flood Hazard

Figure 8-32

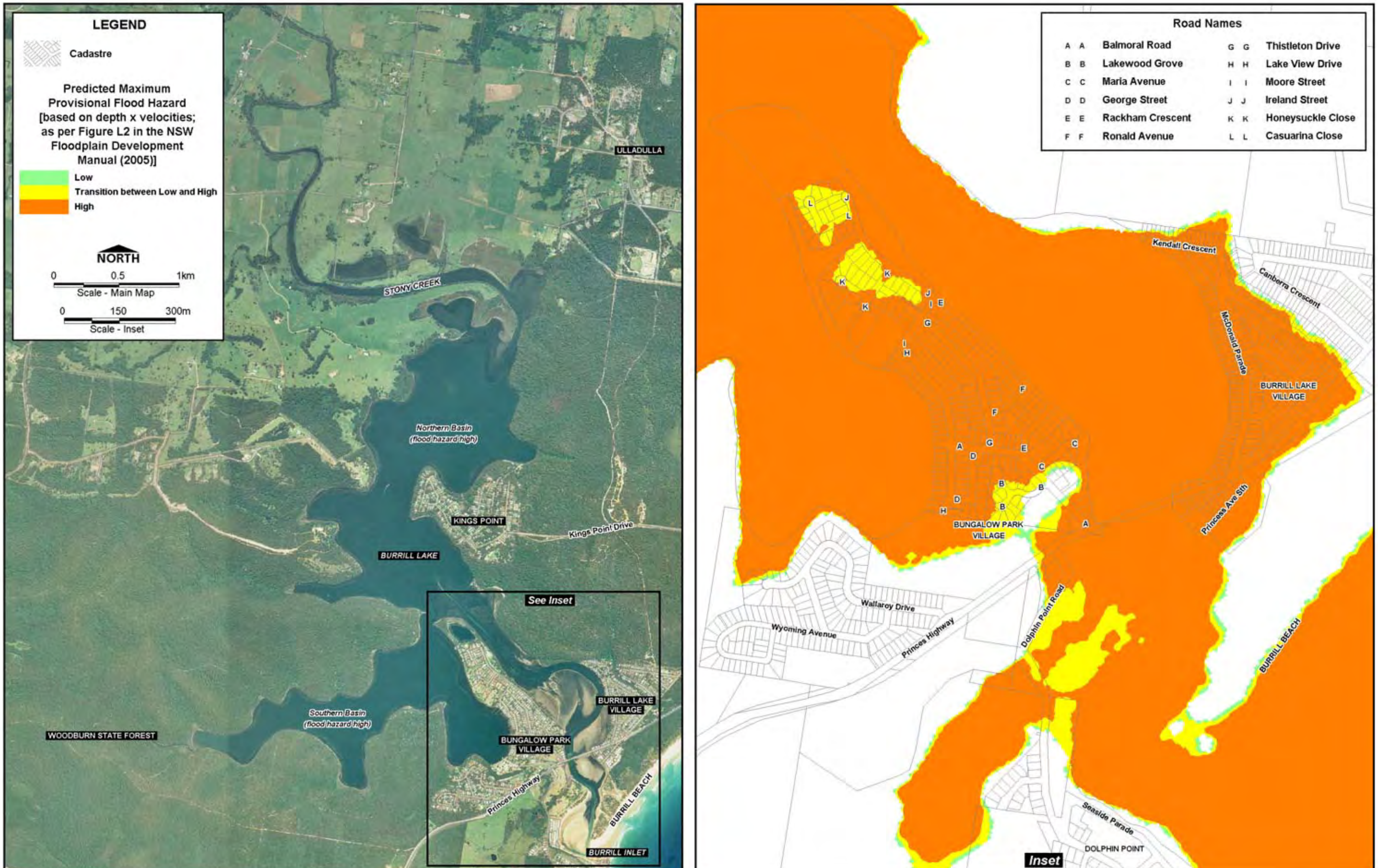




I:\N0848_I_BRH Burrill DW\DRG\FLD_049_061018_BL 100year flood hazard.wor

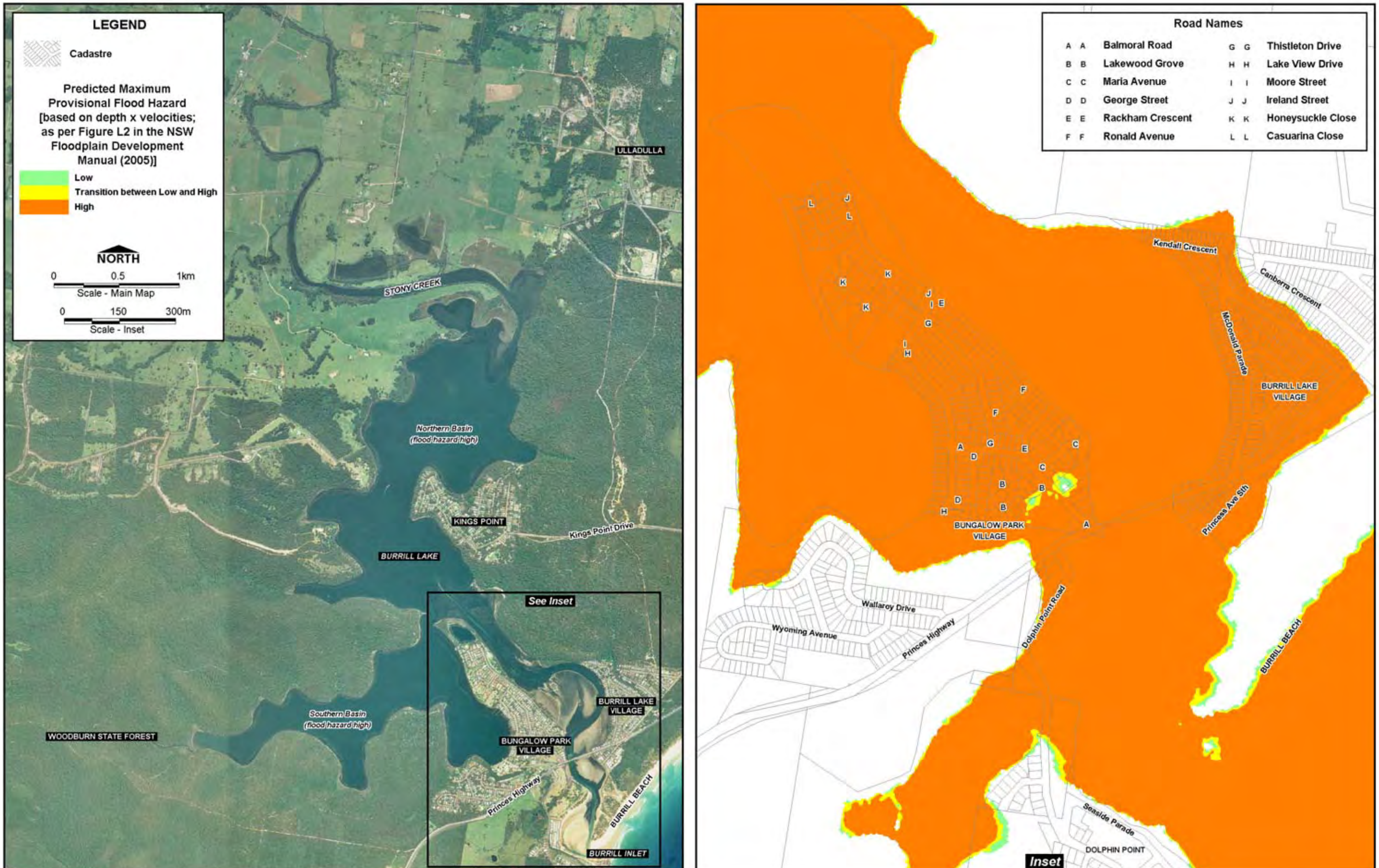
Maximum 100 Year ARI Design Provisional Flood Hazard

Figure 8-33



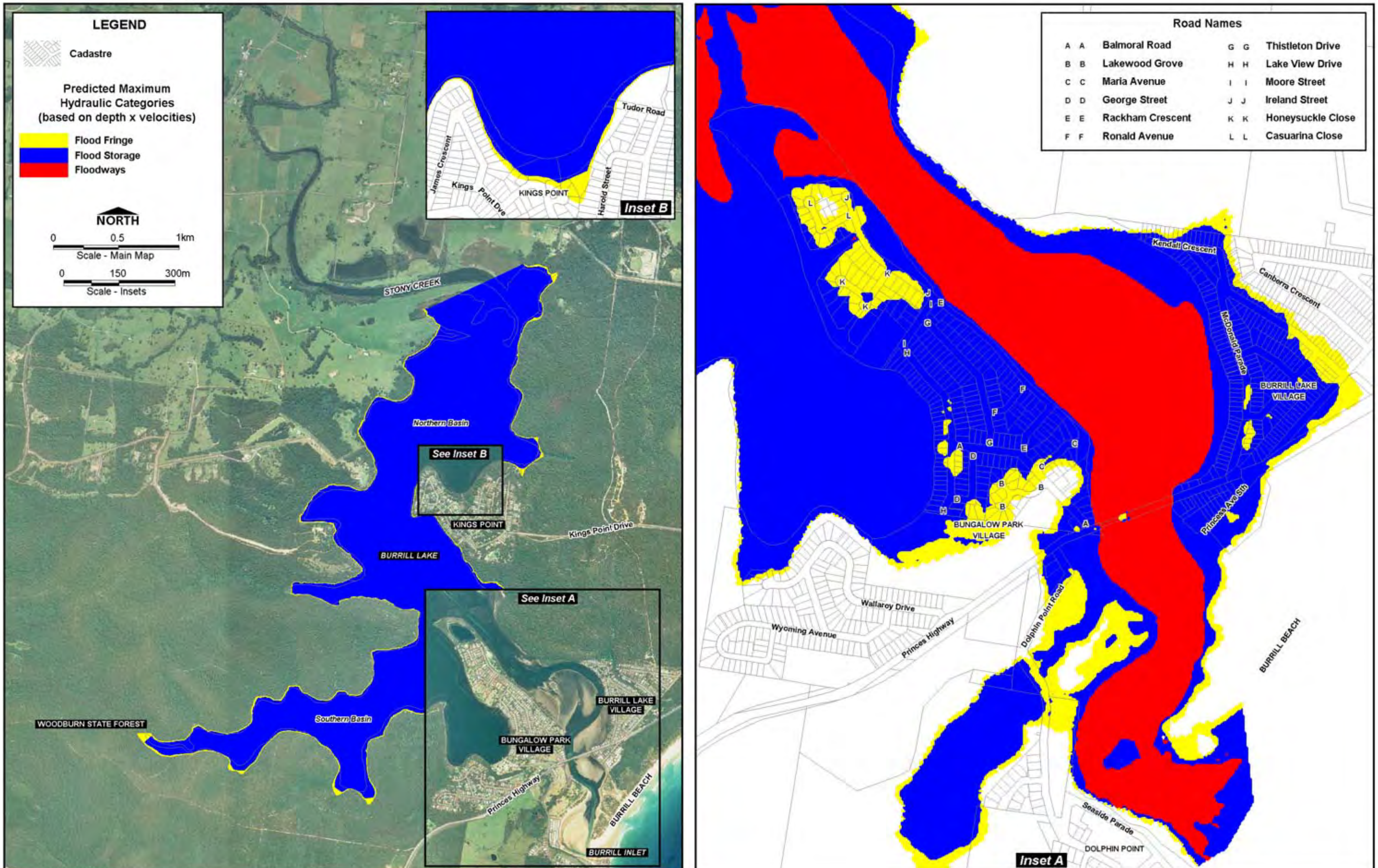
Maximum 200 Year ARI Design Provisional Flood Hazard

Figure 8-34



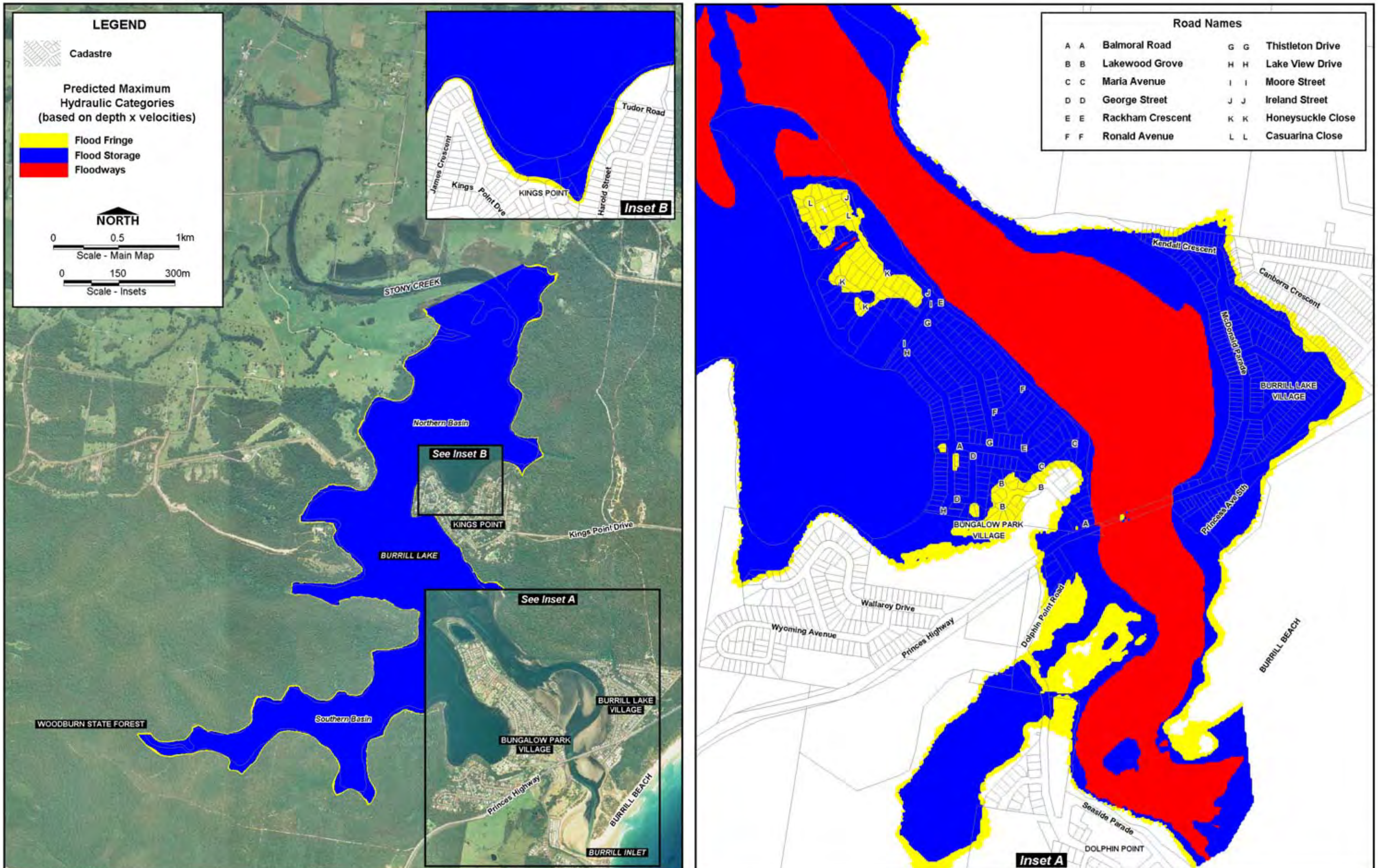
Maximum PMF Design Provisional Flood Hazard

Figure 8-35



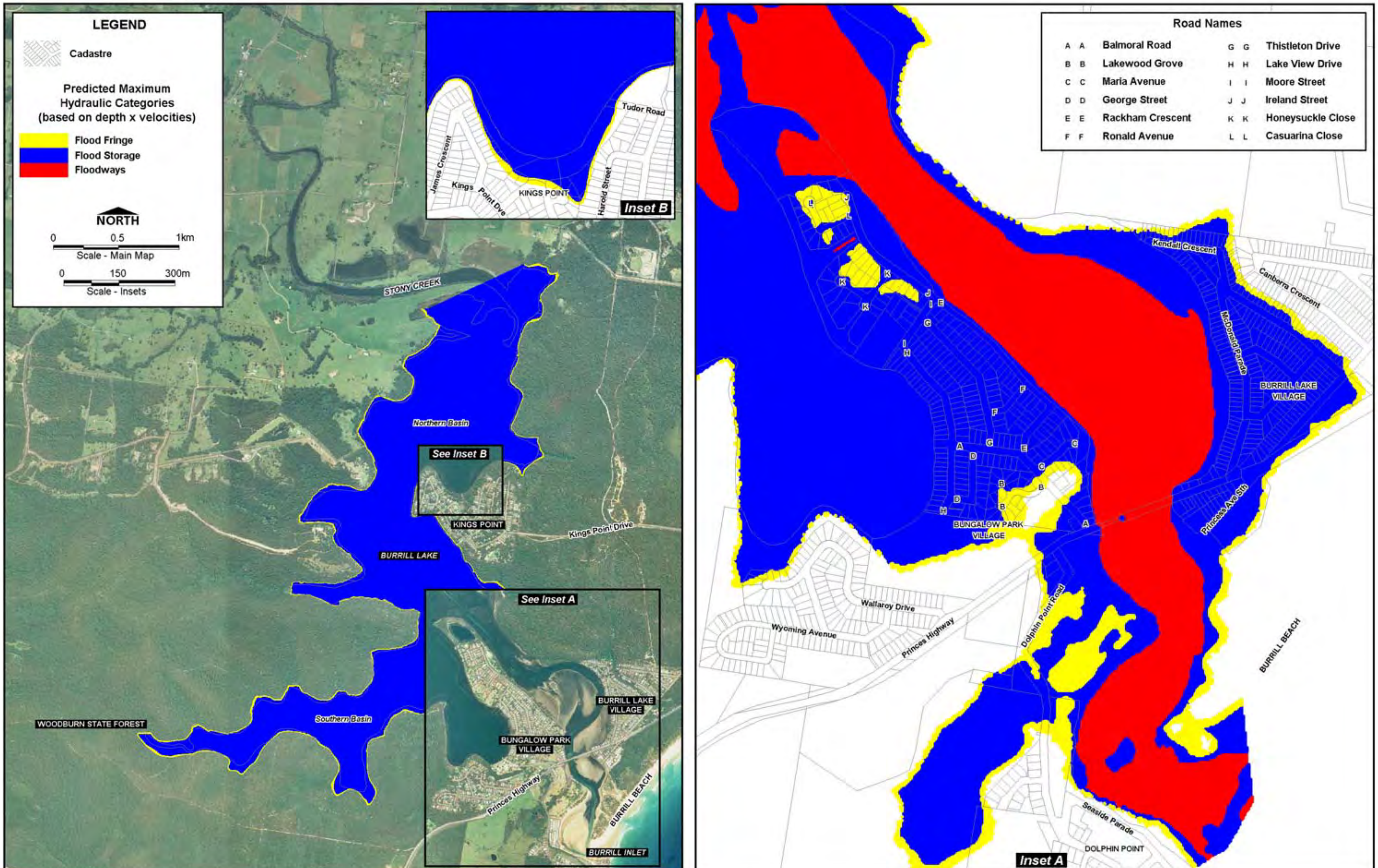
Maximum 50 Year ARI Preliminary Design Hydraulic Categories

Figure 8-36



Maximum 100 Year ARI Preliminary Design Hydraulic Categories

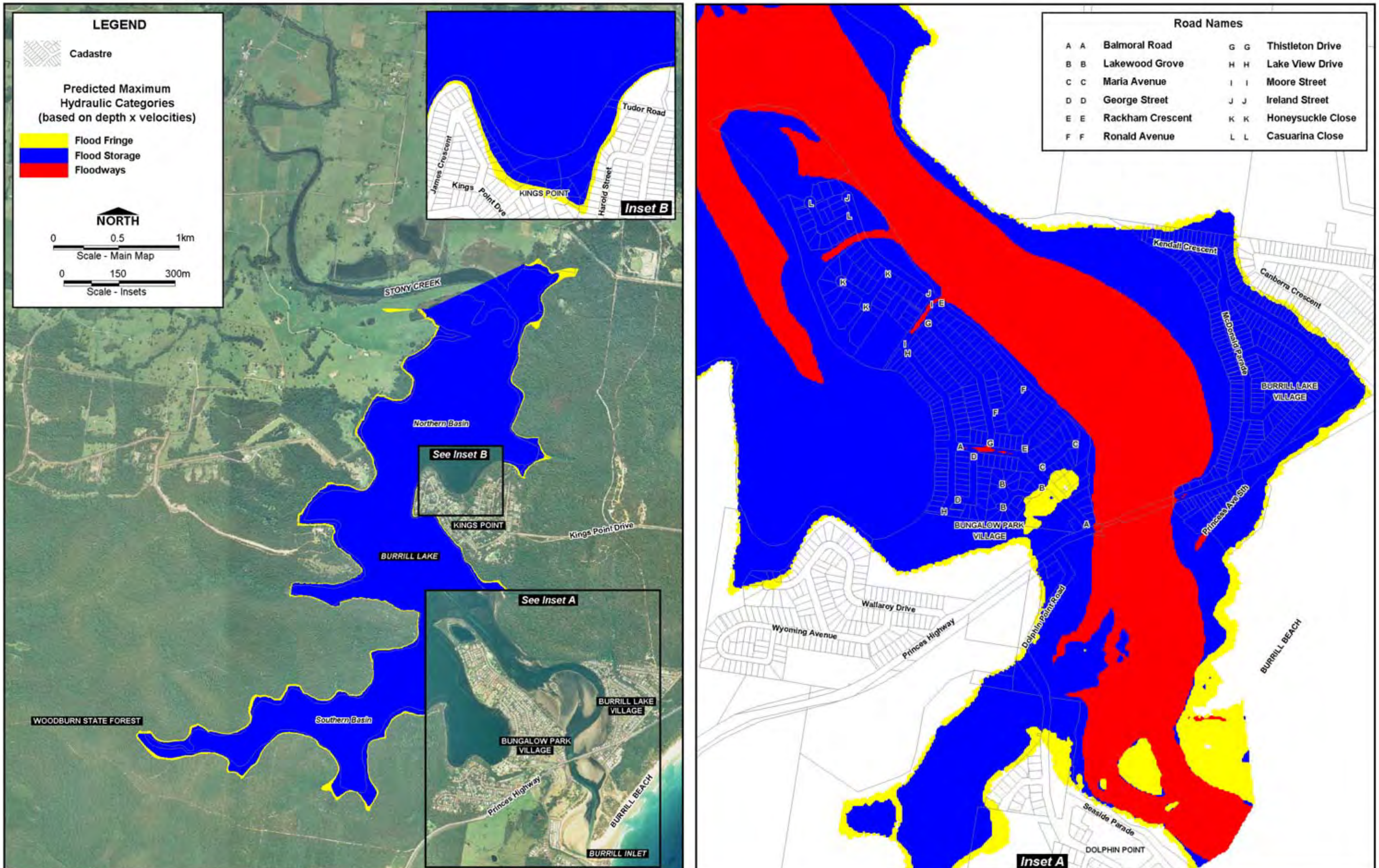
Figure 8-37



I:\N0848_I_BRH Burrill DW\DRG\FLD_054_061018_BL 200year hydraulic.wor

Maximum 200 Year ARI Preliminary Design Hydraulic Categories

Figure 8-38



I:\N0848_I_BRH Burrill DW\DRG\FLD_055_061018_BL PMF hydraulic.wor

Maximum PMF Preliminary Design Hydraulic Categories

Figure 8-39

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APPENDIX A: DATA CALIBRATION POINTS

All calibration points have been provided by local residents. The residents were asked to direct the surveyor to the flood marks they remembered. The analysis shows a lot of variability, and even contradiction, between the flood marks that cannot be explained by expected flood hydrodynamics. It is believed that some of the flood marks were difficult for the public to recollect accurately due to the long period of time since the flood events. The location of various flood marks is shown in Figure A-1.

Figure A-1 also indicates the year associated with each flood mark. In some cases, this differs from the recollection of the resident that provided details of the mark, as discussed in Table A-1.

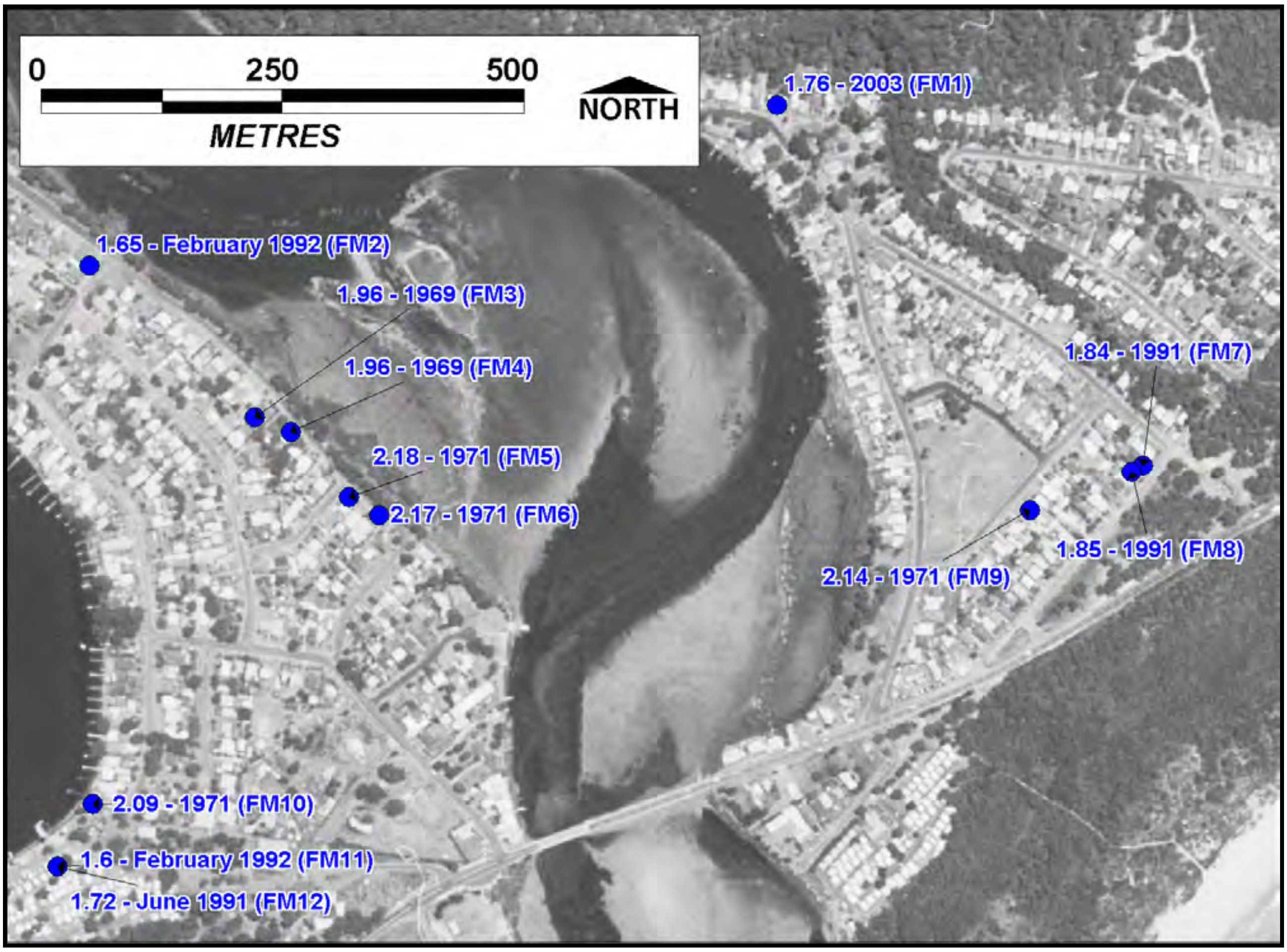

Figure A-1: Historical Flood Marks

Table A-1 highlights apparent confusion regarding the dates of some flood events. For example, it is understood that a major flood event occurred in February 1971, where a level of at least 2.1 m AHD was reached at the causeway (PWD, 1992; WBM, 2001). A number of undated flood marks were located with levels within 0.1 m of this value. Furthermore, the highest flood mark recorded (FM5) was reported to be from 1969 and it is considered that this may be an error. Furthermore, there are levels of 1.96 (FM3 & FM4) which are reported to be from 1971. It seems likely that these may actually be from a smaller event around the same time (probably April, 1969) or recorded before or after the peak of the flood event. There are further discrepancies with some people reporting levels from February 1991, when it seems most likely that they were from February 1992.

Overall, it seems that the most likely peak flood levels for the largest recent flood events are:

- between 2.0 and 2.2 m AHD for the February 1971 flood;
- between 1.7 and 1.9 m AHD for the June 1991 flood; and
- between 1.6 and 1.65 m AHD for the February 1992 flood.

Table A-1 – Consideration of Historical Flood Marks

ID	Month/Year	Surveyed Level (m AHD)	Computed maximum water level (m AHD)	Comments	Reliability
FM1	??/03	1.76		Respondent said around March-April, 2003. Six inches above garage floor. Floor Level = 1.611, Flood Level = 1.763. Probably local flooding	Moderate
FM2	02/92	1.65		Respondent said water came 1m from fence in Feb 1992.	Moderate
FM3	04/69	1.96		Flood mark inside toilet at rear of 23 Rackham Crescent. Natural Surface Level of 1.684. Floor of toilet at 1.694. Mark is noted 1971. However, compared to other marks seems too low. Is likely to be 1969 as is the same level as FM4.	Moderate
FM4	04/69	1.96		27 Rackham Crescent. Respondent said 12" above floor level in 1971. RL of front door was 1.657. Results in flood level of approx 1.96. Natural Surface level is 1.617. However is somewhat lower than other 1971 levels, probably a different flood (1969?)	Moderate
FM5	02/71	2.18		Respondent said 8-9 " above bottom step. Level of bottom step = 1.976 Therefore approx 2.18 m AHD Natural Surface 1.846. Respondent said 1969, but probably 1971 based on other information, as presented in PWD, 1991 and WBM 2000.	Moderate

ID	Month/Year	Surveyed Level (m AHD)	Computed maximum water level (m AHD)	Comments	Reliability
FM6	02/71	2.17		Spike in Power Pole, Probably from 1971 flood. Natural Surface 1.816	Good
FM7	06/91	1.84		Flood level was 1/4 inch down from weephole brick work. Respondent said 1992, but based on levels could also be June 91.	Good
FM8	06/91	1.85		Respondent said 6 to 8 inches above base of roller door. Floor of Garage = 1.689 + 7 inches = 1.867 Top of bottom step = 1.848. Probably June, 1991 (vis FM12)	Moderate
FM9	02/71	2.14		Flood Mark on Concrete Pier RL 2.137 - Probably 1971, Flood Mark on Concrete Pier. Respondent indicated that it might have been 1990s but probably 1971 as there are no events of this magnitude after 1971.	Good
FM10	02/71	2.09		Historical Flood Mark at 67 Lakeview Drive RL 2.085. Probably 1971	Good
FM11	02/92	1.60		Historical Flood Mark Brass Plaque on Caravan Feb 1991. Natural Surface 1.326. However, there was no flood in Feb 1991. Most likely that the year is 1992.	Moderate
FM12	06/91	1.72		Historical Flood Mark on Brass Plaque on Caravan June 1991- Natural Surface 1.326	Excellent

APPENDIX B: MODELLING SOFTWARE

Rafts-XP

RAFTS-XP is a non-linear runoff routing model used extensively throughout Australasia and South East Asia. RAFTS-XP has been shown to work well on catchments ranging in size from a few square metres to 1000's of square kilometres of both urban and rural nature. RAFTS-XP can model up to 2000 different nodes and each node can have any size sub-catchment attached as well as a storage basin.

RAFTS-XP uses the Laurensen non-linear runoff routing procedure to develop a stormwater runoff hydrograph from either an actual event (recorded rainfall time series) or a design storm utilising Intensity-Frequency-Duration data together with dimensionless storm temporal patterns as well as standard AR&R 2001 data. Three loss models may be employed to generate excess rainfall. They are (1) Initial/Continuing, (2) Initial/Proportional and (3) the ARBM water balance model. A reservoir (pond) routing model allows routing of inflow hydrographs through a user-defined storage using the level pool routing procedure and allows modelling of hydraulically interconnected basins and on-site detention facilities.

Tuflow

TUFLOW solves the full 2D shallow water equations based on the scheme developed by Stelling (1984). The solution is based around the well-known ADI (alternating direction implicit) finite difference method. A square grid is used to define the discretisation of the computational domain.

Improvements to the Stelling 1984 scheme, including a robust wetting and drying algorithm and greater stability at oblique boundaries, and the ability to dynamically link a quasi-2D model were developed by Syme (1991). Further improvements including the insertion of 1D elements or quasi-2D models inside a 2D model and the modelling of constrictions on flow such as bridges and large culverts, and automatic switching into and out of upstream controlled weir flow have been developed subsequently (WBM, 2000).

TUFLOW models have been successfully checked against rigorous test cases (Syme 1991, Syme et al 1998, Syme 2001), and calibrated and applied to a large range of real-world tidal and flooding applications. TUFLOW is a leading fully 2D hydrodynamic modelling system and has the ability to be dynamically linked to 1D models and have 1D models dynamically nested inside or through the fully 2D domain.

Hydraulic structure flows through large culverts and bridges are modelled in 2D and include the effects of bridge decks and submerged culvert flow. Flow over roads, levees, bunds, etc is modelled using the broad-crested weir formula when the flow is upstream controlled. For smaller hydraulic structures such as pipes or for weir flow over a bridge, ESTRY 1D models can be inserted at any points inside the 2D model area.

The procedure for the development of the 2D/1D flood model is:

- Compile all of the ground survey data for the area (photogrammetry and contours for the 2D flood plain, and cross-sections for the 1D sections).

- Decide on the location of the model boundaries. The boundaries are the outer points of the model where, for example, the inflows from the catchment are defined. It can also be the location of the interaction between 2D and 1D.
- Design the 1D branch network and its connections with the 2D, and define the location of structures.
- Develop a grid database for the 2D domain(s), and a cross-section database for the 1D domain(s), including topographic information, roughness, percentage of blockage, etc.
- Incorporate the details of each hydraulic structure (bridges, embankments, viaducts and culverts).

Van Rijn Formulation

Van Rijn (1990) addressed the issue that reliable models to predict the time-averaged concentration profile for a rippled bed or a plane sheet flow bed were lacking. He proposed a new method based on the convection-diffusion equation and separate current-related and wave-related mixing coefficients. This involved introduction of separate current-related and wave-related bed roughness values. The method was developed to apply for non-breaking or breaking waves over rippled or plane seabeds.

His relationship has the following form:

q = Bed Load Transport + Suspended Load Transport

= $q_b + q_s$

where:

bed load transport $\quad q_{b,c} = 0.25\, u'_{*,c}\, d_{50} \dfrac{T^{1.5}}{D_*^{0.3}}$

$$u'_{*,c} = \left[\bar{\tau}'_c / \rho\right]^{0.5}$$

$$T = \left(\bar{\tau}'_{cw} - \bar{\tau}'_{cr}\right) / \bar{\tau}_{cr}$$

$$D_* = d_{50}\left[(s-1)g / v^2\right]^{1/3}$$

bed-shear stress by current: $\quad \bar{\tau}'_c = \dfrac{1}{8}\rho\, {}^{\alpha}cw\, {}^{\mu}c\, f_a\left(\bar{V}_R\right)^2$

bed-shear stress by waves: $\quad \bar{\tau}'_w = \dfrac{1}{4}\rho\, {}^{\mu}w\, {}^{f}w\left(\hat{U}_\delta\right)^2$

bed-shear stress by current/waves: $\bar{\tau}'_{cw} = \bar{\tau}'_c + \bar{\tau}'_w$

wave orbital velocity: $\quad \hat{U}_\delta$

uniform current velocity: $\bar{V}_R$

efficiency factor current: $\mu_c = f_c^{'} / f_c$

efficiency factor waves: $\mu_w = 0.6 / D_*$

wave-current interaction coefficient:

$$^{\alpha}cw = \frac{1n^2\left(90\delta_w / k_a\right)}{1n^2\left(90\delta_w / k_{s,c}\right)}$$

$f_{c'}$ =current related friction factor from k_{sc}

f_c = grain size related friction factor

f_w = wave related friction factor from k_{sw}

f_a = friction factor derived from k_a

k_a =apparent bed roughness

= $k_{sc} \exp\left[\gamma\hat{U} / \bar{V}_r\right]$

bed concentration: $\boldsymbol{c_a = 0.015 \frac{d_{50}}{a} \frac{T^{1.5}}{D_*^{0.3}}}$

suspended load transport (numerical integration):

$$q_{s,c} = \int_a^h u\, c\, dz$$

This integral may be approximated by the alternate formulation as follows:

suspended load transport: $\boldsymbol{q_{s,c} = \left(F_c + F_w\right)\bar{V}_R h c_a}$

current-related correction factor: $F_c = \frac{\left[a / h\right]^{ZC} - \left[a / h\right]^{1.2}}{\left[1.2 - ZC\right]\left[1 - \left(a / h\right)\right]^{ZC}}$

wave-related correction factor: $F_w = \frac{\left[a / h\right]^{ZW} - \left[a / h\right]^{1.2}}{\left[1.2 - ZW\right]\left[1 - \left(a / h\right)\right]^{ZW}}$

current-related suspension number: $ZC = \frac{w_s}{\beta \kappa u_{*,c}}$

wave-related suspension number: $ZW = \alpha[\frac{^w s}{\bar{v}_R}]0.9[\frac{^{\bar{v}} R^{T'} p}{H_s}]1.05$

$\alpha = 7$ for $h \geq 100\delta_s$

$\alpha = 0.7(h/\delta_s)^{0.5}$ for $h < 100\delta_s$

The reader is referred to the Van Rijn (1990) reference for a description of all of the parameters involved in this formulation. For present purposes, it is sufficient to emphasise that the results of the method are strongly influenced by the bed roughness, reference level and near bed mixing layer thickness values. In particular, they depend intimately on how those parameter values are used in combination.

Van Rijn offers the following advice on selection of these values.

- Bed roughness $k_{s,c}$, $k_{s,w}$: A reasonable estimate for currents and non-breaking waves is $k_{s,c} = k_{s,w} \approx 3\Delta_r$, with values in the range of 0.03 to 0.1m. In case of breaking waves with sheet flow conditions the bed roughness will be of the order of the wave boundary layer thickness giving $k_{s,w} \approx \delta_w$ with values in the range of 0.01 to 0.02m.

- Reference level a: The reference level is proposed to be equal to half the rippled height ($a=\frac{1}{2}\Delta_r$) in the case of non-breaking waves and equal to the wave boundary layer thickness ($a=\delta_w$) in the case of sheetflow conditions.

- Near-bed mixing layer thickness δ_s: This parameter can be obtained from a relationship given in the reference, giving $\delta_s \approx 3\Delta_r$ in the ripple regime and $\delta_s = 3\delta_w$ in the sheet flow regime. Both expressions yield values in the range of 0.03 to 0.1m. In the case of breaking waves the δ_s value may be somewhat larger ($\delta_s \approx 0.2$m) due to the breaking effect. More field data from the surf zone are necessary to better define the δ_s parameter for breaking wave conditions.

The Van Rijn formulation was adopted for this project, given that it is the more recent generally accepted method that draws upon and attempts to improve the other available methods.

The Van Rijn method is combined with a bank collapse algorithm that enables the sides of oversteepened channel slopes to collapse. This allows for the gradual widening, for example, of an entrance breach channel with time.

The purpose of the routines currently available in TUFLOW are to simulate the likely results of entrance scour on flood water levels in the Lake. The post storm beach profile is considered secondary if the resulting flood levels can be adequately calibrated through the duration of a known flood. Where suitable data is unavailable for the temporal erosion of sand berms, the use of an appropriate method, such as Van Rijn's method, with the best available knowledge on local sand characteristics, and judgement based on experience from other sites is going to provide the most reliable results. It is accepted that the theory behind sediment transport processes is not complete and that some aspects of the process are not simulated. However, given the current state of widely available desktop computational capacity and the limitations of the available methods, some error is currently unavoidable and generally expected.

APPENDIX C: MODEL SENSITIVITY TESTING

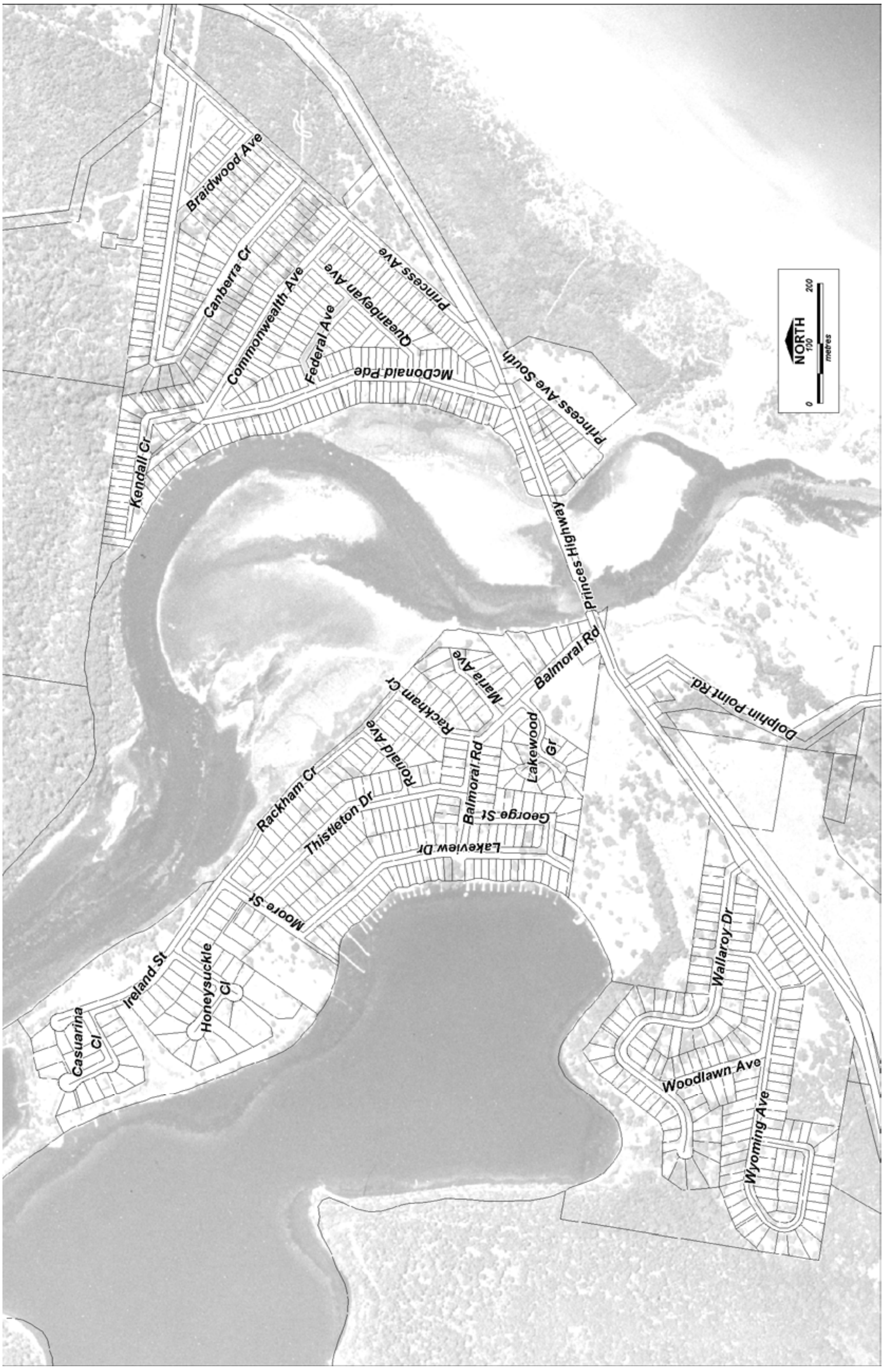

Figure C-1 Road Locality Plan Near Burrill Inlet

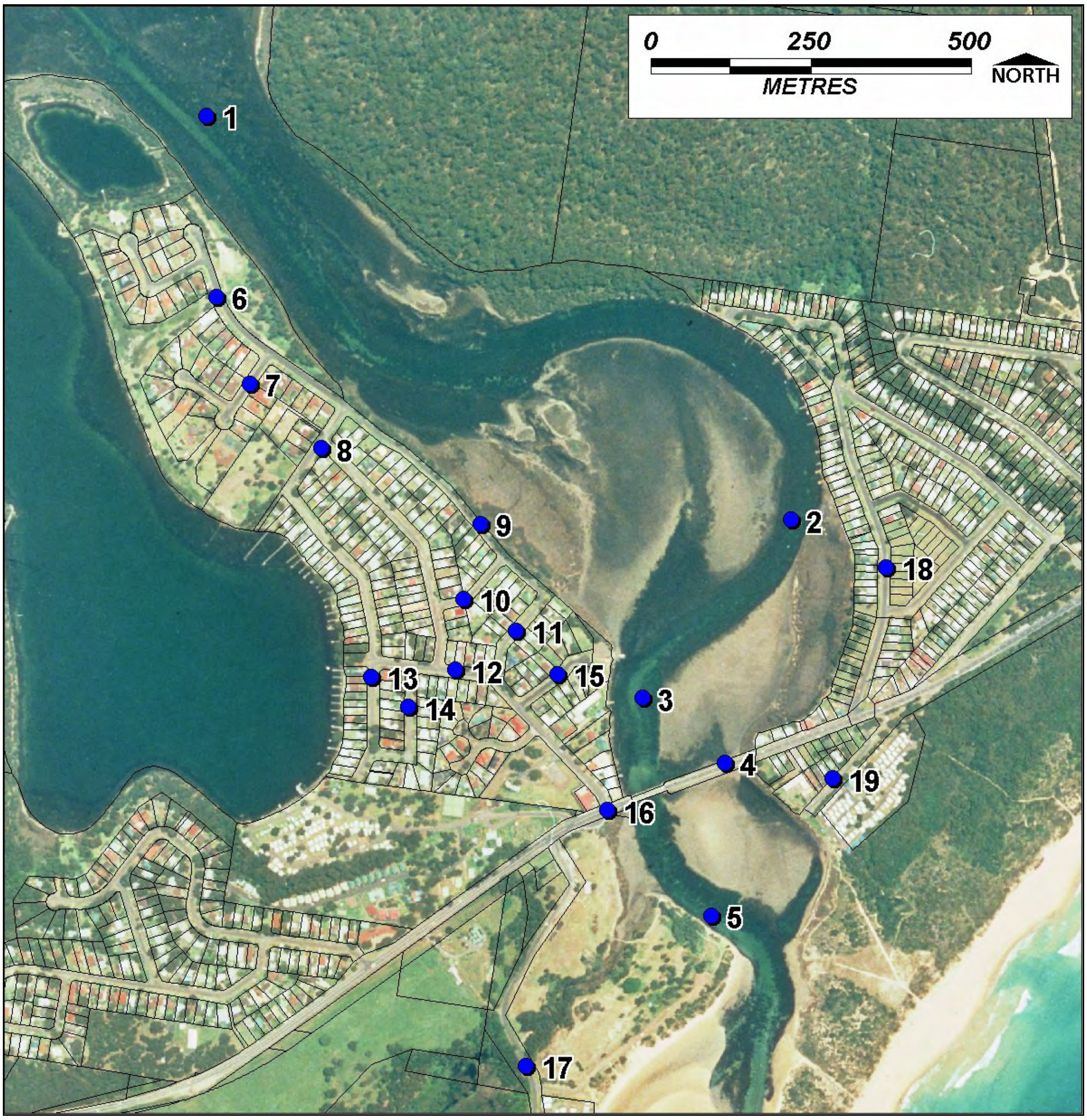

Figure C-2 Points for Tabulated Comparison of Velocities and Water Levels

Table C-1 Sensitivity Testing: Tabulated Peak Water Levels

		Base Case Peak Water Level (m AHD)	Changes in Peak Water Level for Sensitivity Simulations (m)											
Location	**Name**		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
1	Burrill Inlet North	2.22	0.09	-0.16	-0.02	0.01	-0.02	0.00	-0.01	-0.01	-0.01	0.03	0.25	0.00
2	Burrill Inlet East	2.13	0.09	-0.16	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.26	-0.01
3	Causeway Approach	2.09	0.09	-0.16	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.26	-0.02
4	Causeway	1.99	0.06	-0.13	-0.02	-0.01	-0.02	0.01	0.00	-0.01	-0.02	0.04	0.27	-0.03
5	Causeway Exit	1.95	0.06	-0.16	-0.02	-0.01	-0.03	0.02	-0.01	-0.01	-0.02	0.05	0.30	-0.03
6	Ireland St.	2.18	0.08	-0.13	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.01	0.03	0.24	-0.01
7	Honeysuckle Cl.	2.21	0.09	-	-0.02	0.02	-0.01	0.00	-0.01	-0.01	-0.01	0.03	0.24	0.00
8	Moore St.	2.19	0.09	-0.16	-0.02	0.01	-0.02	0.00	-0.01	-0.01	-0.02	0.03	0.25	-0.01
9	Rackham Cr.	2.14	0.09	-0.16	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.26	-0.01
10	Ronald Ave.	2.16	0.09	-0.16	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.25	-0.01
11	Rackham Cr. South	2.14	0.09	-0.16	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.26	-0.01
12	Balmoral Rd.	2.17	0.10	-0.16	-0.02	0.01	-0.02	0.01	-0.02	-0.01	-0.01	0.04	0.26	0.00
13	Lake View Dr.	2.21	0.10	-0.17	-0.02	0.01	-0.02	0.00	-0.01	-0.01	-0.01	0.03	0.25	0.00
14	George St.	2.20	0.10	-0.17	-0.02	0.01	-0.02	0.00	-0.01	-0.01	-0.02	0.03	0.25	0.00
15	Maria Ave.	2.11	0.09	-0.16	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.26	-0.01
16	Princes Highway	2.01	0.07	-0.17	-0.02	-0.01	-0.03	0.02	-0.01	-0.02	-0.02	0.04	0.29	-0.04
17	Dolphin Pt Rd.	2.27	0.01	-0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.00
18	McDonald Ave.	2.13	0.09	-	-0.02	0.01	-0.02	0.01	-0.01	-0.01	-0.02	0.04	0.26	-0.01
19	Princess Ave South	2.05	0.08	-0.15	-0.02	0.00	-0.02	0.01	-0.01	-0.02	-0.02	0.04	0.27	-0.05

Table C-2 Sensitivity Testing: Tabulated Peak Velocities

		Base Case Velocity Magnitude (m/s)	Changes in Velocity for Sensitivity Simulations (m/s)											
Location	**Name**		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
1	Burrill Inlet North	0.64	0.01	0.01	0.01	0.00	0.01	-0.02	-0.01	0.01	0.01	-0.03	-0.06	0.04
2	Burrill Inlet East	0.32	0.01	0.00	0.00	0.00	0.01	-0.01	0.02	0.00	0.00	-0.01	-0.02	0.01
3	Causeway Approach	0.68	0.03	0.00	0.01	0.00	0.02	-0.02	0.01	0.01	0.01	-0.04	-0.05	0.03
4	Causeway	1.57	0.22	-0.23	-0.01	0.10	0.01	-0.01	-0.08	0.04	-0.01	-0.06	-0.01	0.03
5	Causeway Exit	1.20	0.11	-0.01	0.03	0.02	0.03	-0.05	-0.05	0.04	0.04	-0.13	-0.13	0.03
6	Ireland St.	0.69	0.03	-0.53	-0.01	0.00	-0.01	-0.02	-0.02	0.00	-0.01	-0.04	-0.02	0.04
7	Honeysuckle Cl.	0.43	-0.02	-	-0.13	0.01	-0.10	-0.01	-0.03	-0.01	-0.03	0.07	-0.05	0.02
8	Moore St.	0.42	0.02	-0.01	0.00	0.00	0.01	-0.02	0.07	-0.03	0.00	-0.02	-0.02	-0.06
9	Rackham Cr.	0.19	0.00	0.01	0.00	0.00	0.00	-0.01	0.00	0.00	0.00	-0.01	-0.02	0.02
10	Ronald Ave.	0.16	0.03	-0.05	0.00	0.00	0.00	-0.01	0.04	-0.02	0.00	0.00	0.02	-0.02
11	Rackham Cr. South	0.20	0.03	-0.04	0.00	0.00	0.00	-0.01	0.05	-0.02	0.00	0.00	0.03	-0.03
12	Balmoral Rd.	0.24	0.04	-0.09	-0.01	0.00	0.00	-0.01	0.00	0.00	0.00	0.00	0.03	0.02
13	Lake View Dr.	0.26	-0.02	-0.01	0.01	-0.01	0.01	-0.01	0.07	-0.04	0.00	-0.02	-0.06	-0.08
14	George St.	0.26	0.00	-0.01	0.00	0.00	0.00	-0.01	0.06	-0.03	0.00	-0.02	-0.03	-0.05
15	Maria Ave.	0.17	0.05	-0.10	-0.01	0.01	0.00	0.00	0.04	-0.02	0.00	0.00	0.06	-0.03
16	Princes Highway	0.58	0.03	-0.19	-0.02	0.09	0.01	-0.10	0.07	-0.06	0.02	-0.06	-0.11	-0.26
17	Dolphin Pt Rd.	1.25	0.02	-0.05	0.00	0.00	0.00	0.00	-0.01	0.00	0.00	0.00	-0.59	0.00
18	McDonald Ave.	0.05	0.01	-	0.00	0.00	0.00	0.00	0.01	-0.01	0.00	0.00	0.01	-0.01
19	Princess Ave South	0.43	0.06	-0.05	-0.01	0.03	0.00	-0.01	0.05	-0.01	0.00	0.00	-0.01	-0.02

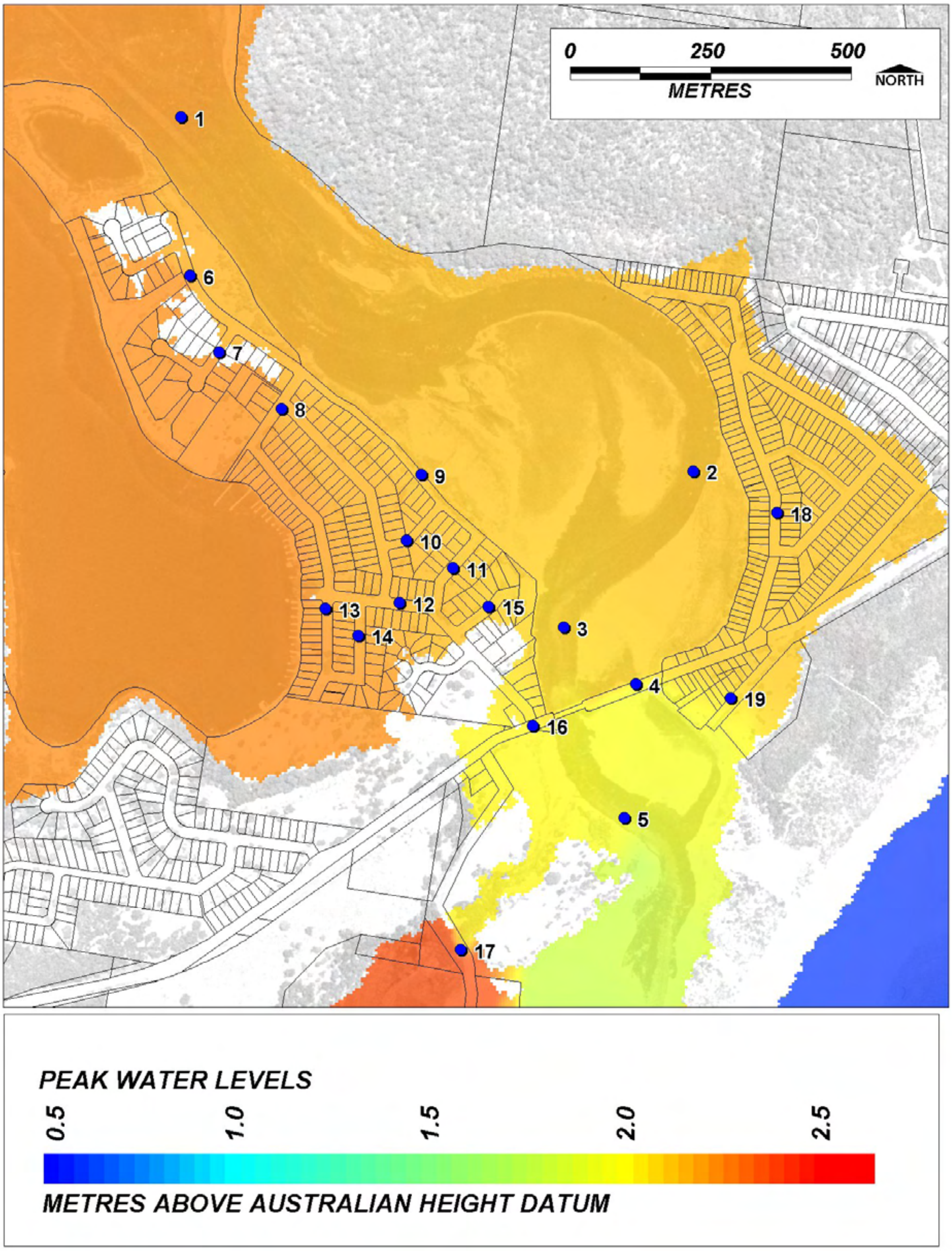

Figure C-3 Base Case Peak Water Levels

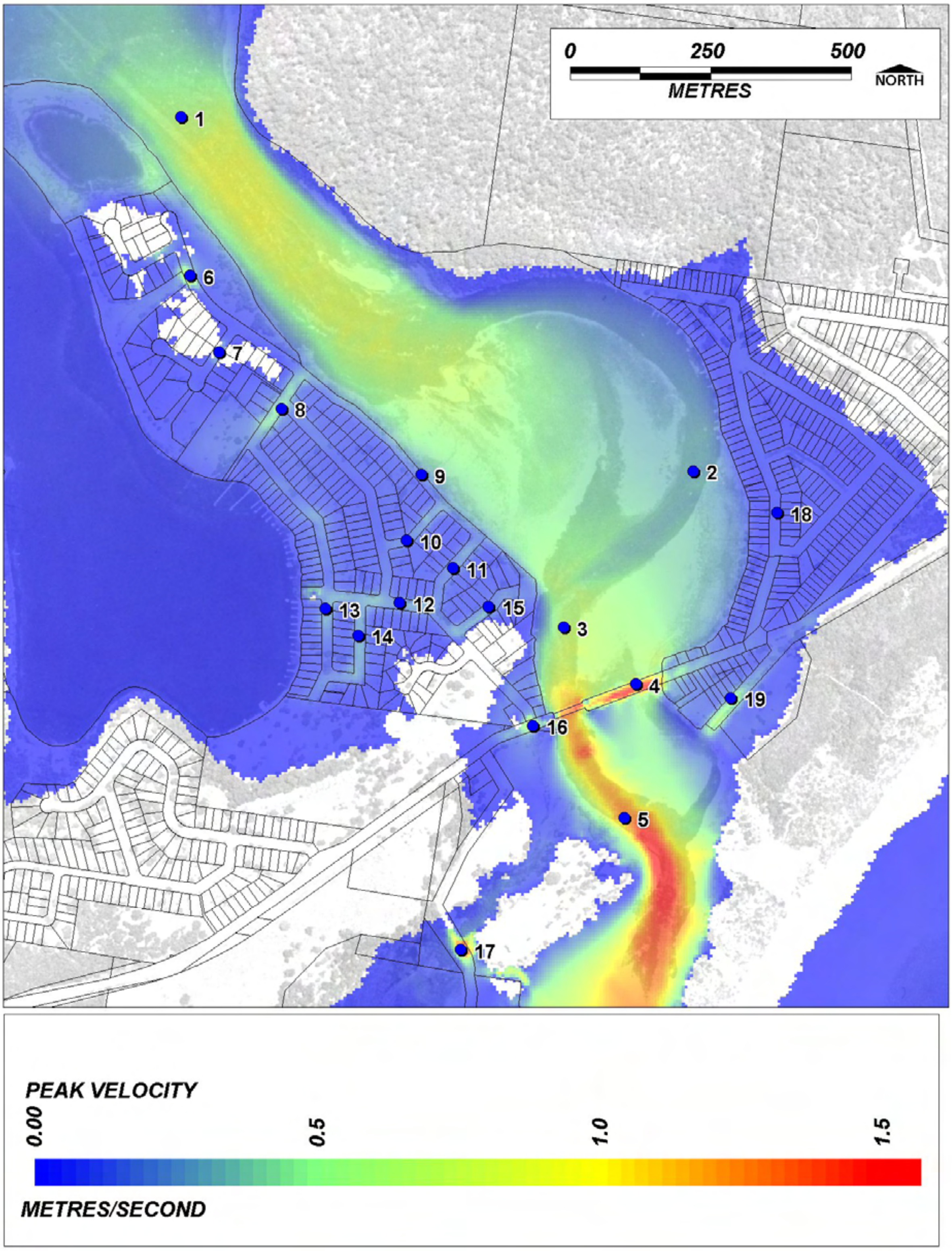

Figure C-4 Base Case Peak Speeds

Figure C-5 Base Case Peak Velocity Vectors (North)

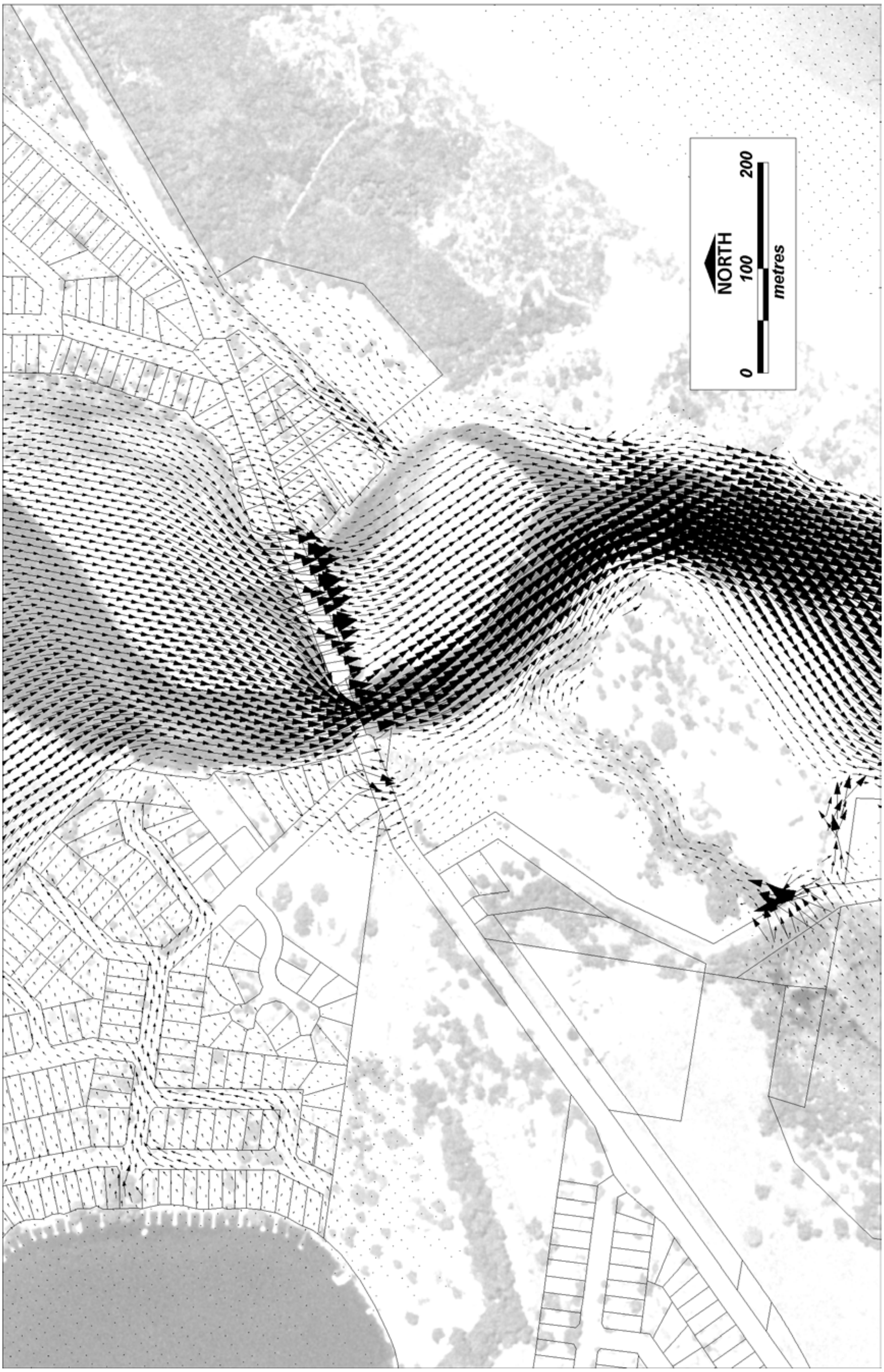

Figure C-6 Base Case Peak Velocity Vectors (South)

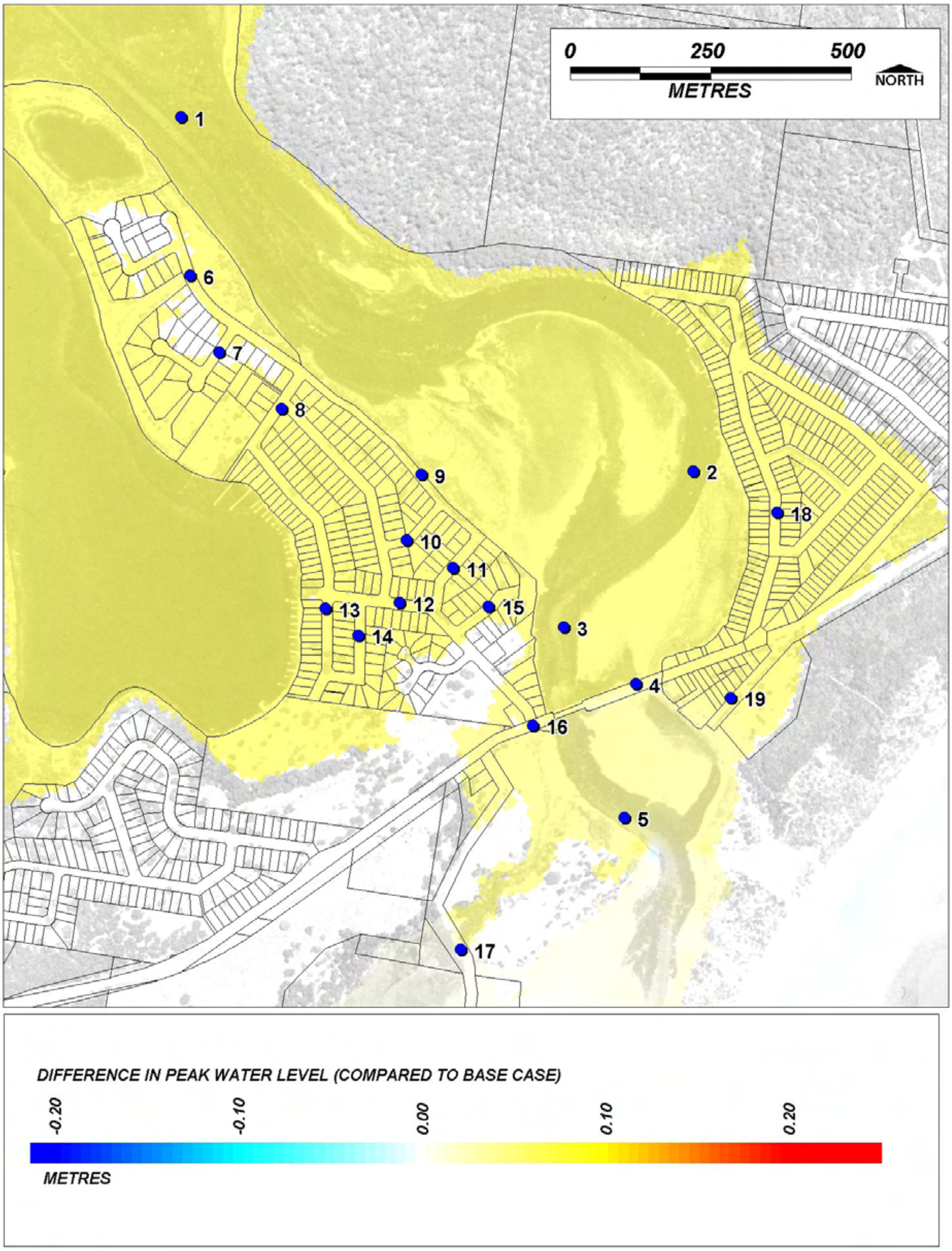

Figure C-7 Sensitivity Test 1 Peak Water Levels (Change from Base Case)

(Lowered Infiltration Losses)

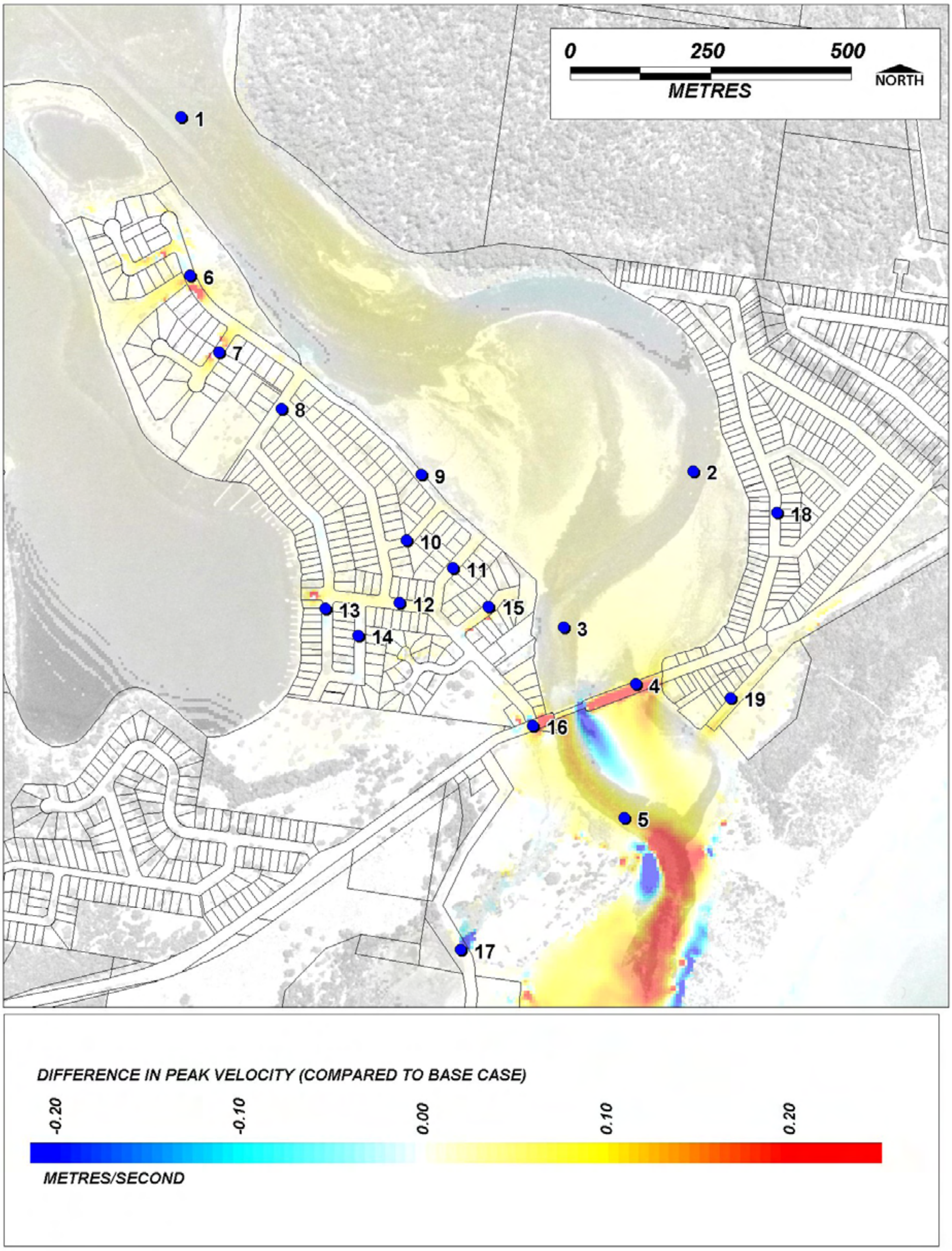

Figure C-8 Sensitivity Test 1 Peak Velocities (Change from Base Case)

(Lowered Infiltration Losses)

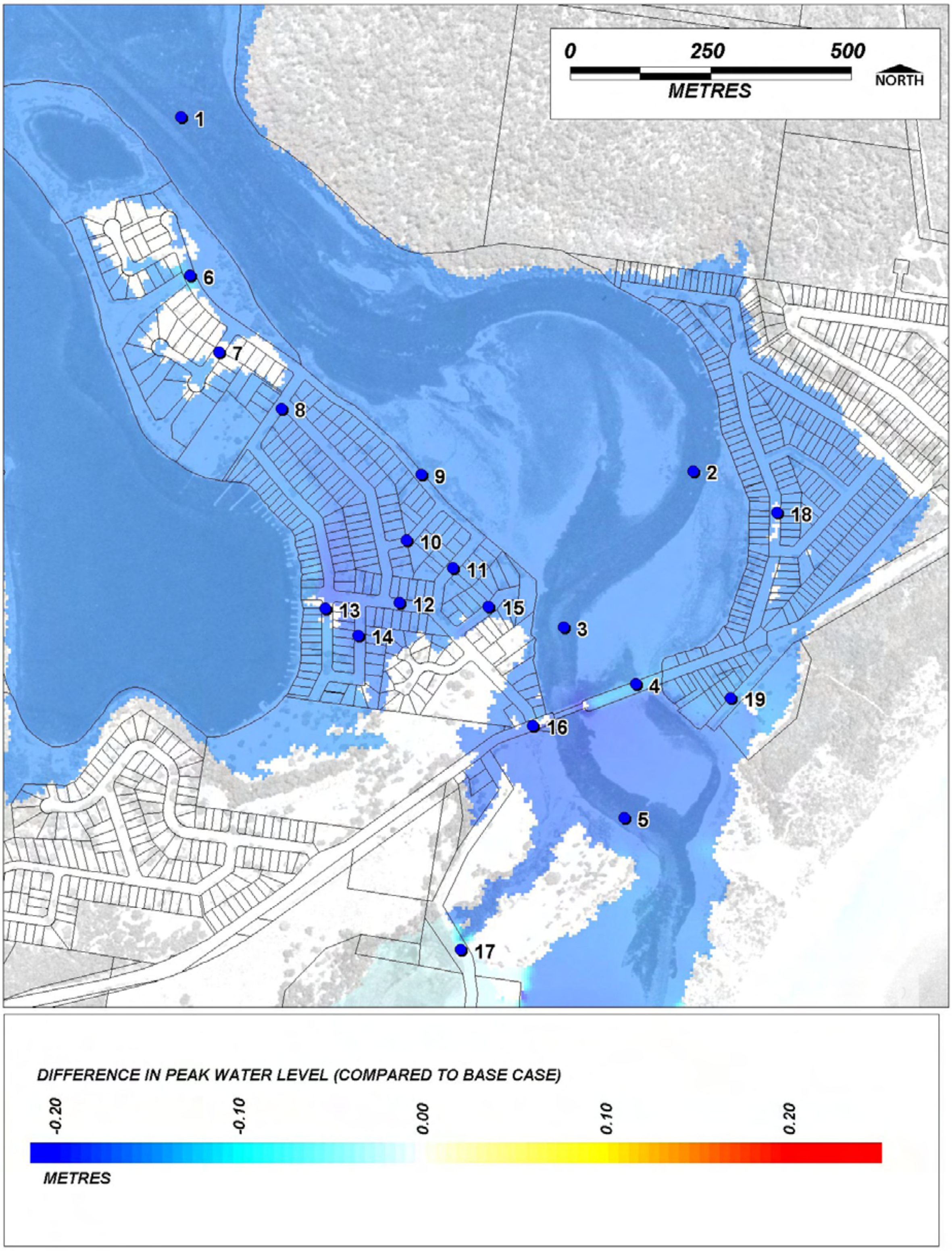

Figure C-9 Sensitivity Test 2 Peak Water Levels (Change from Base Case)
(Raised Infiltration Losses)

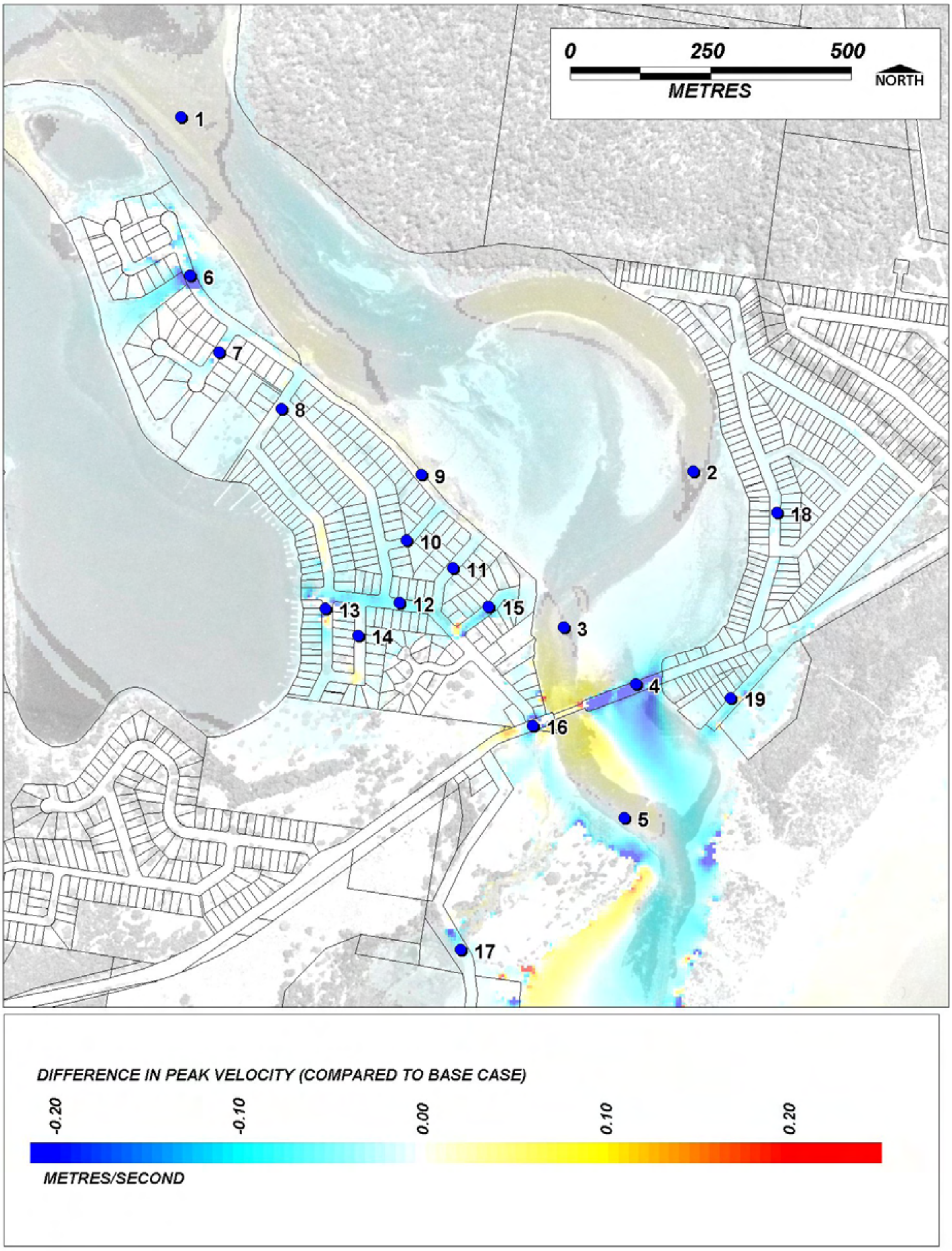

Figure C-10 Sensitivity Test 2 Peak Velocities (Change from Base Case)
(Raised Infiltration Losses)

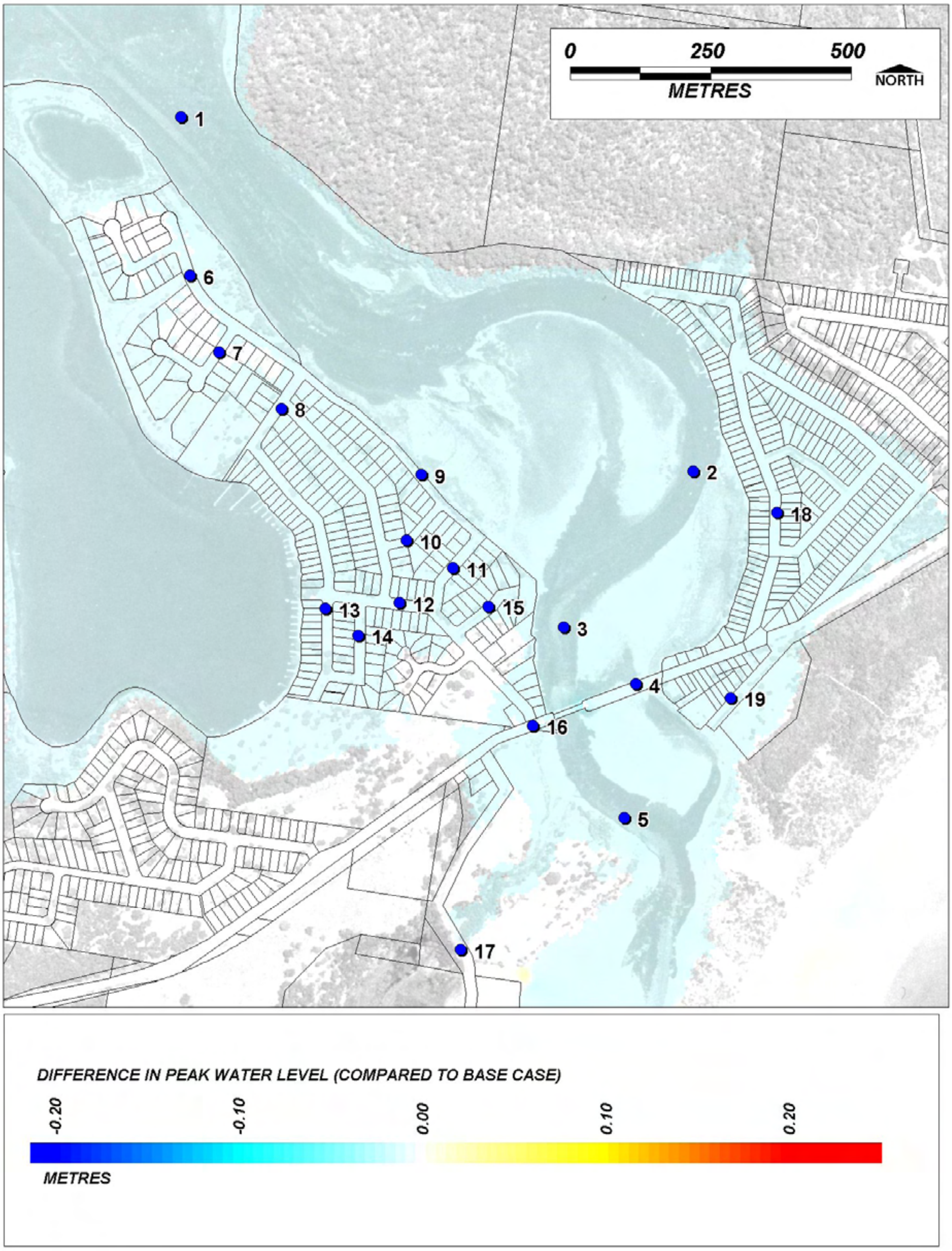

Figure C-11 Sensitivity Test 3 Peak Water Levels (Change from Base Case)

(Lowered Causeway Loss Coefficient)

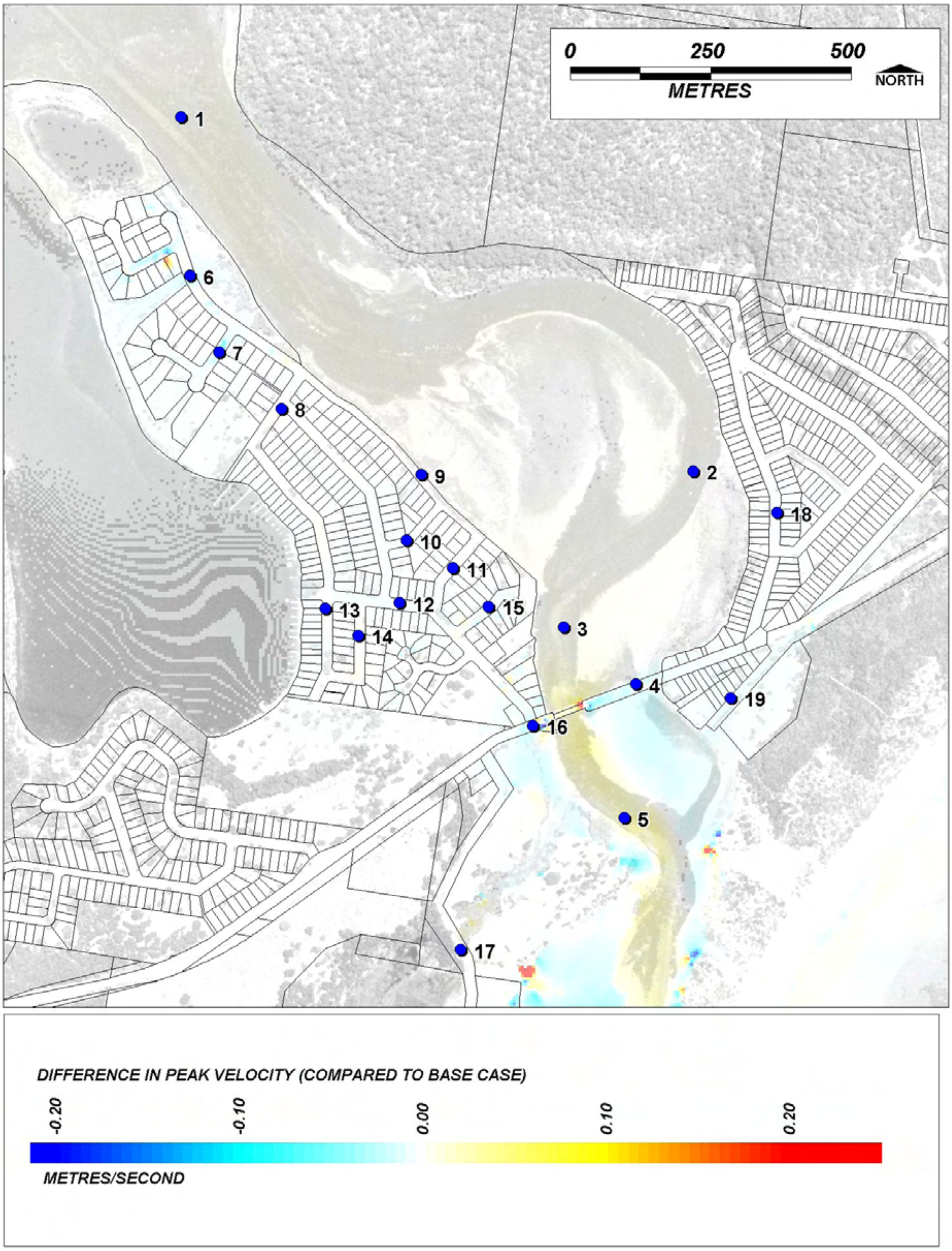

Figure C-12 Sensitivity Test 3 Peak Velocities (Change from Base Case)
(Lowered Causeway Loss Coefficient)

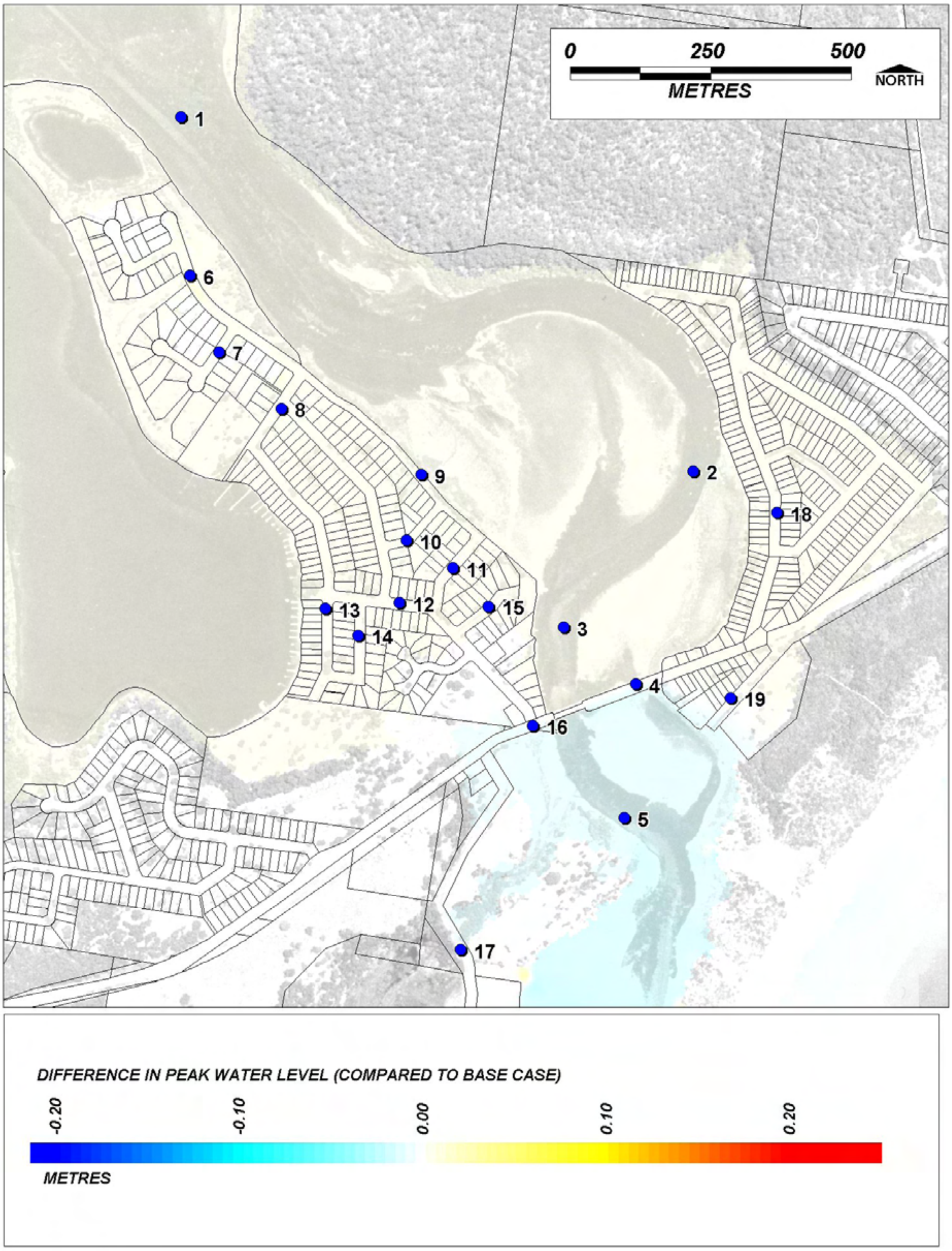

Figure C-13 Sensitivity Test 4 Peak Water Levels (Change from Base Case)

(Raised Causeway Loss Coefficient)

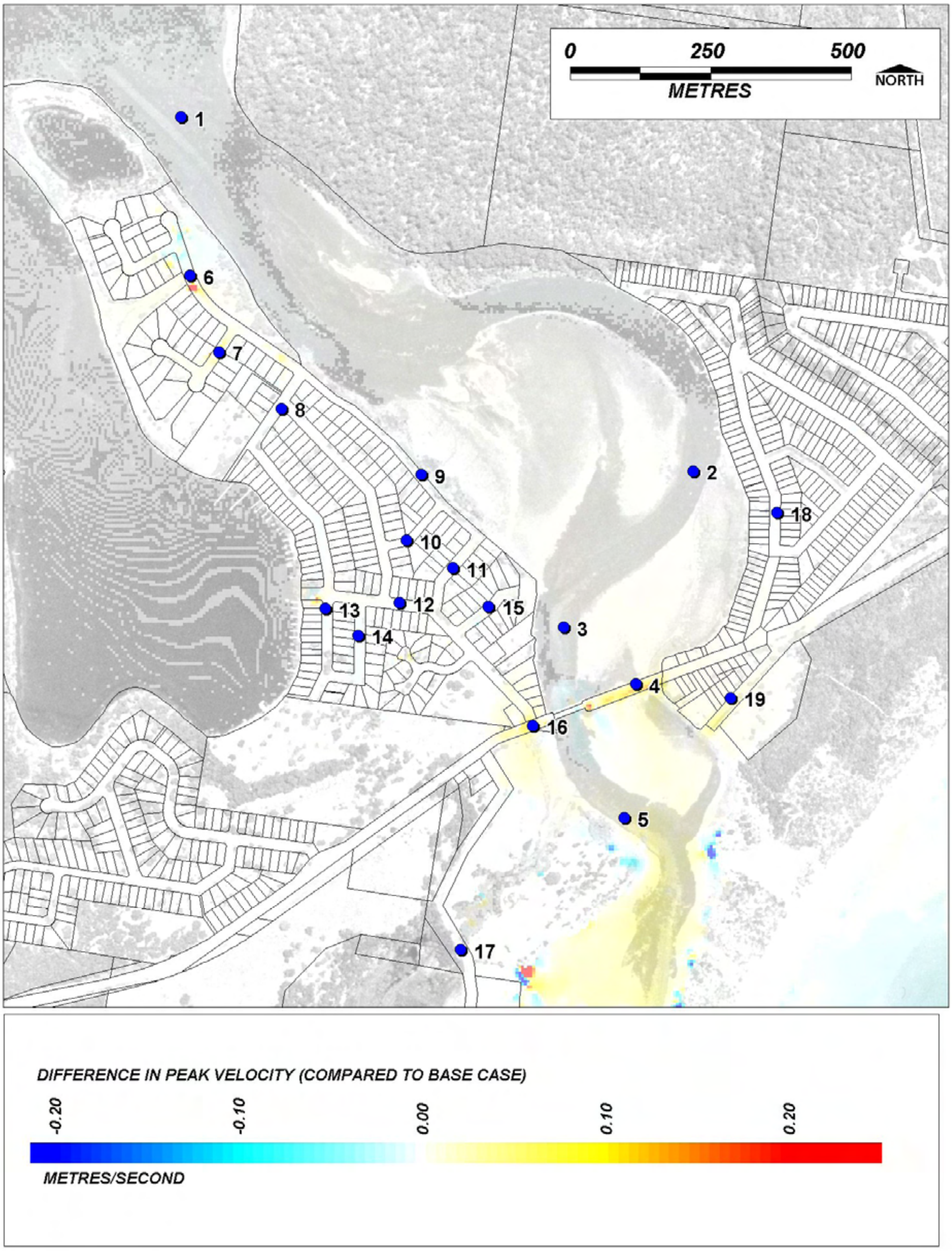

Figure C-14 Sensitivity Test 4 Peak Velocities (Change from Base Case)

(Raised Causeway Loss Coefficient)

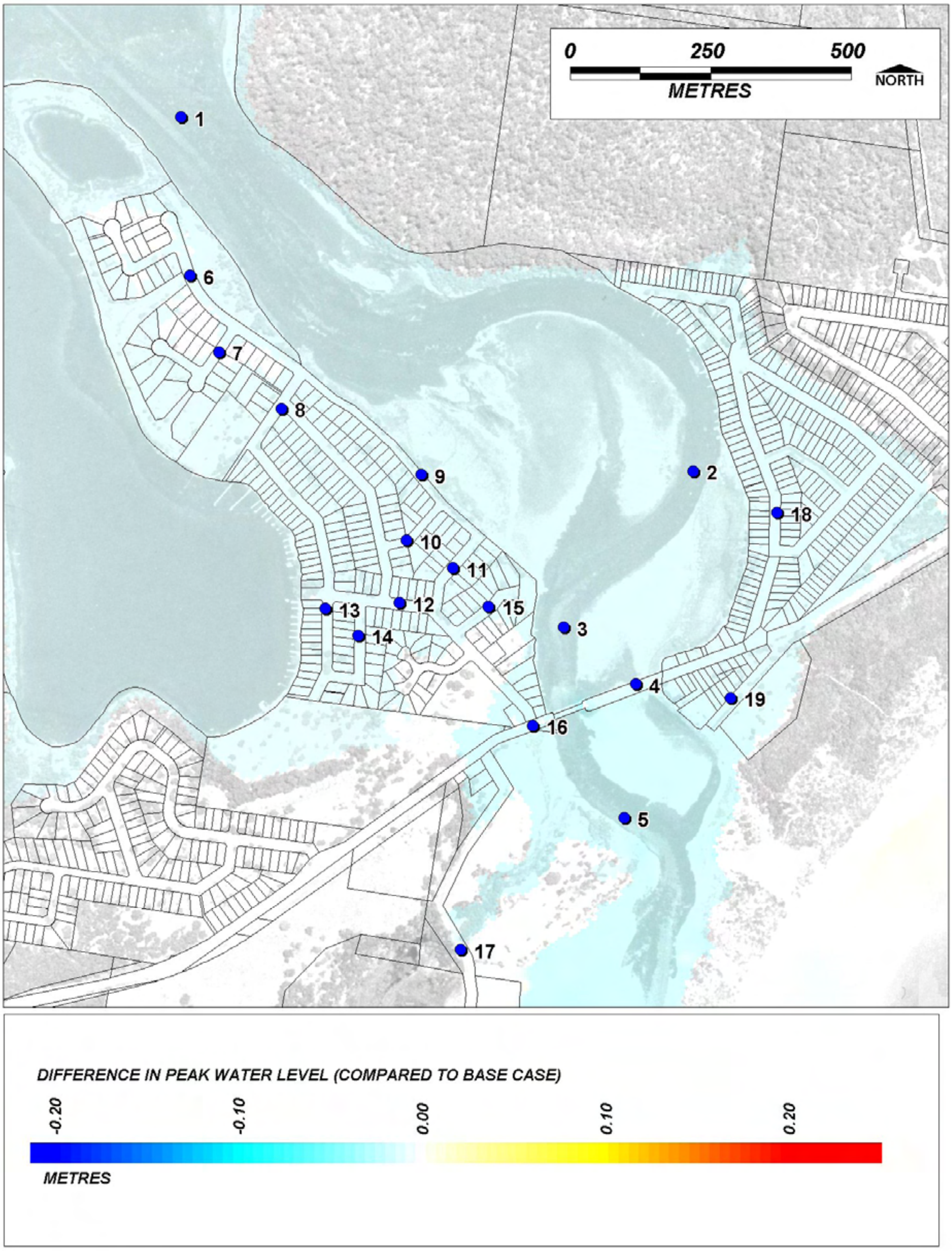

Figure C-15 Sensitivity Test 5 Peak Water Levels (Change from Base Case)

(Raised Sediment Transport Roughness)

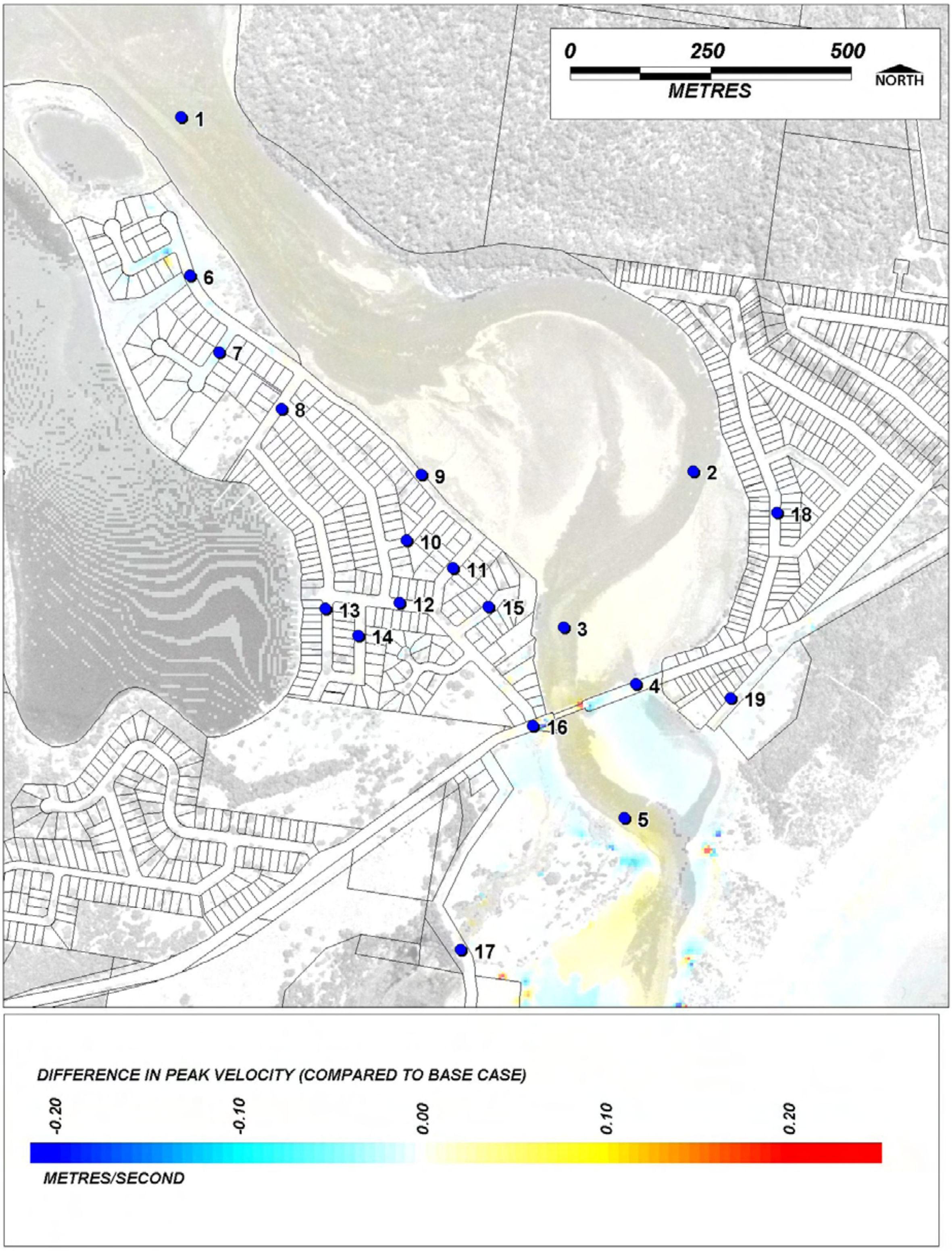

Figure C-16 Sensitivity Test 5 Peak Velocities (Change from Base Case)

(Raised Sediment Transport Roughness)

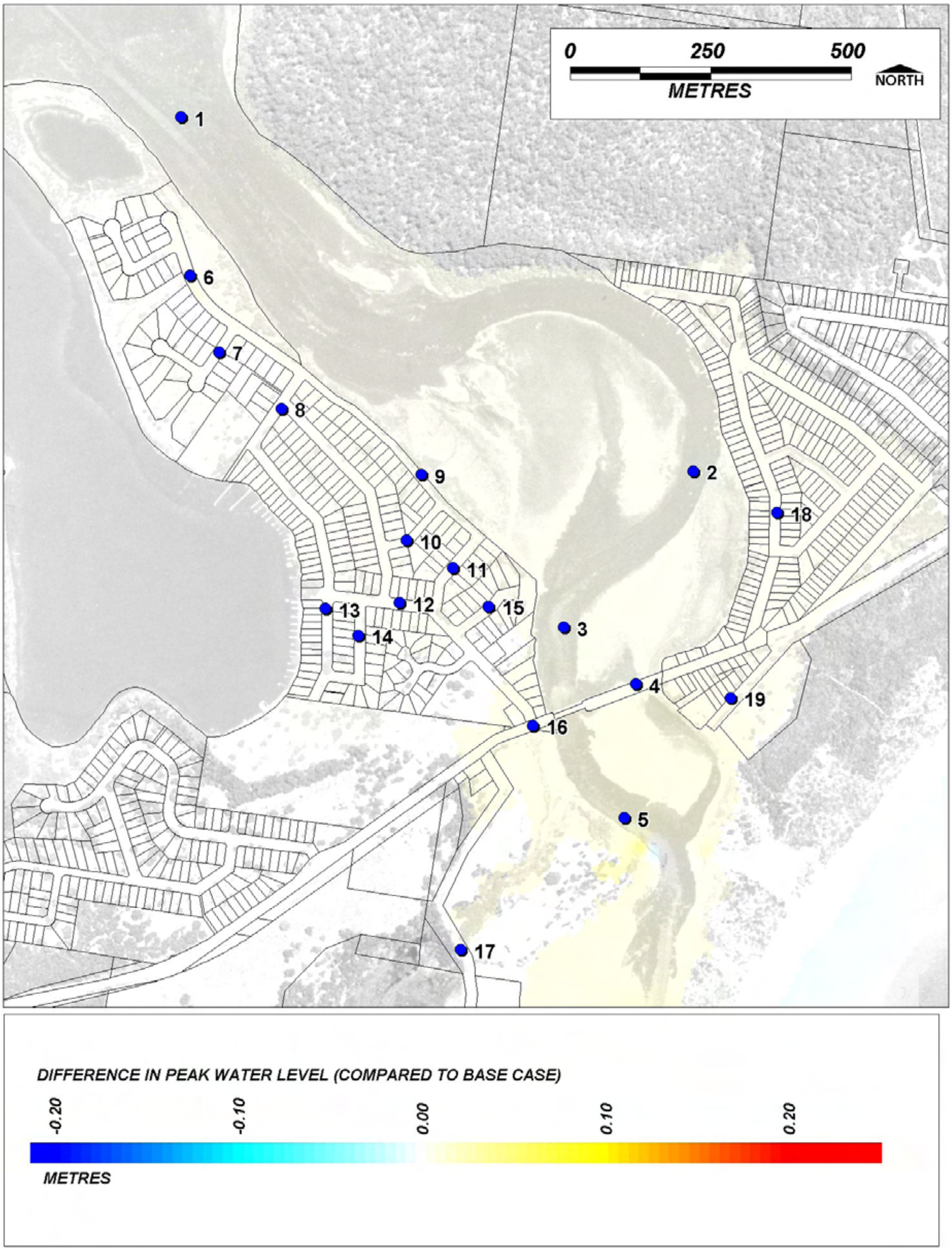

Figure C-17 Sensitivity Test 6 Peak Water Levels (Change from Base Case)

(Lowered Sediment Transport Roughness)

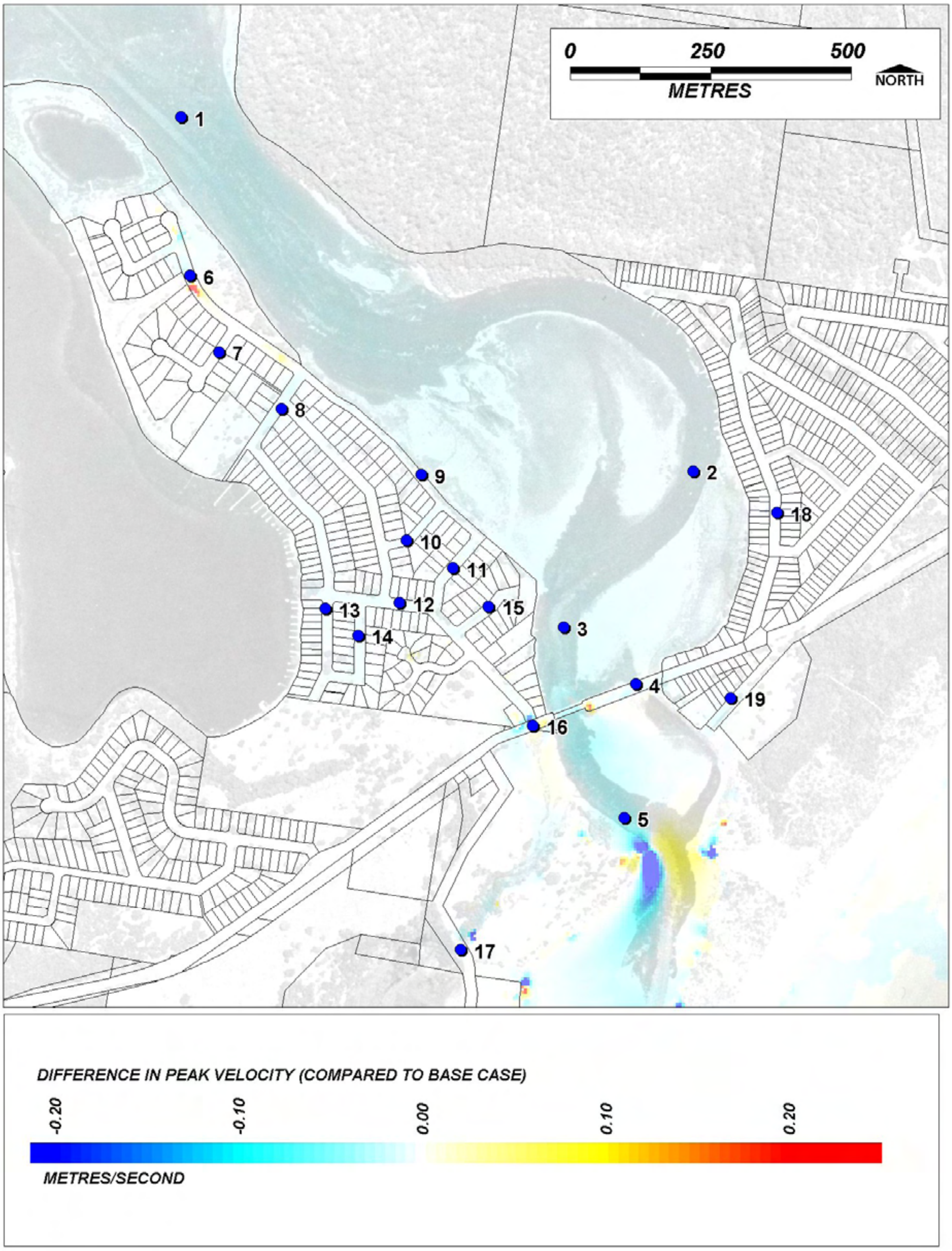

Figure C-18 Sensitivity Test 6 Peak Velocities (Change from Base Case)
(Lowered Sediment Transport Roughness)

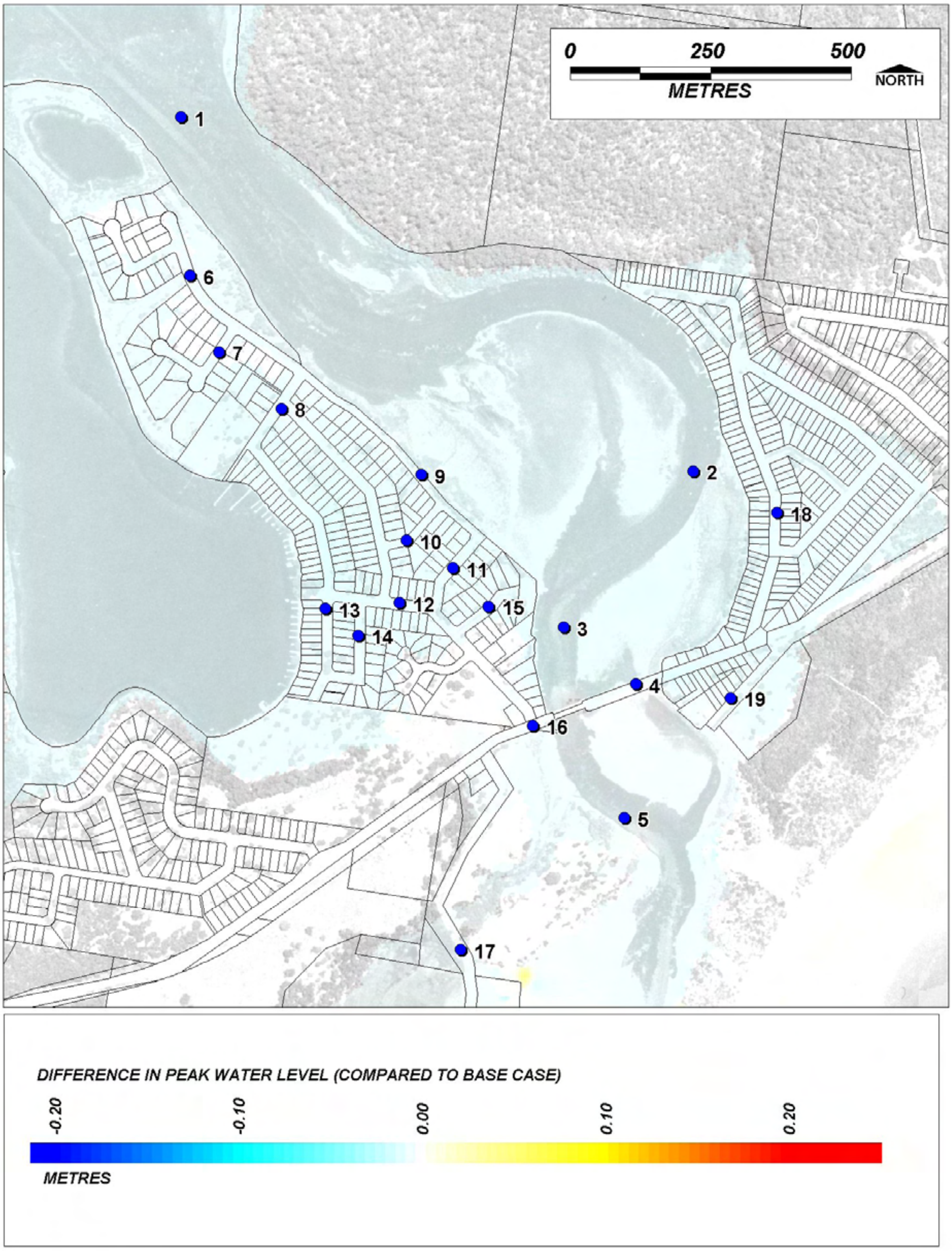

Figure C-19 Sensitivity Test 7 Peak Water Levels (Change from Base Case)

(Lowered Roughness on Residential Lots)

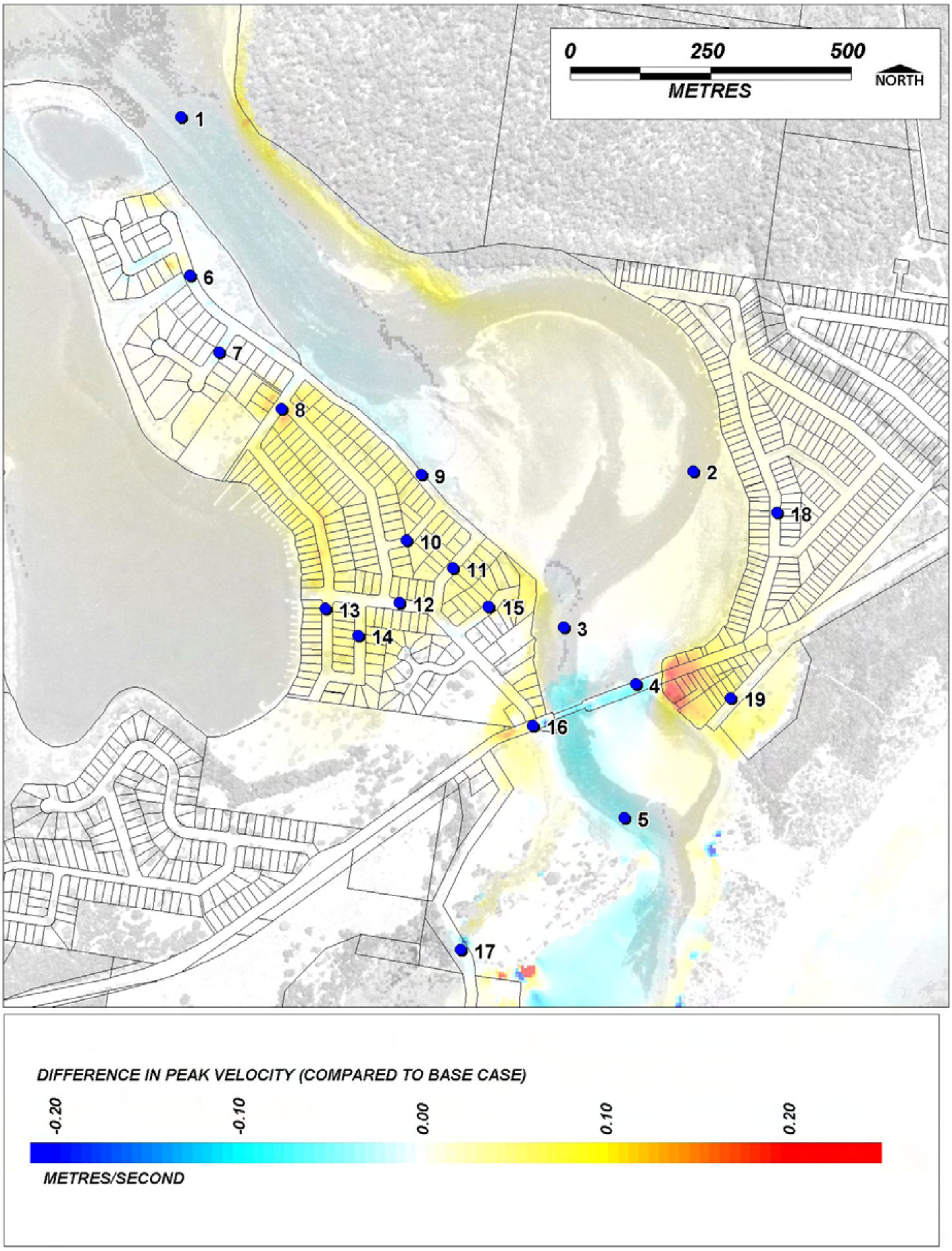

Figure C-20 Sensitivity Test 7 Peak Velocities (Change from Base Case)
(Lowered Roughness on Residential Lots)

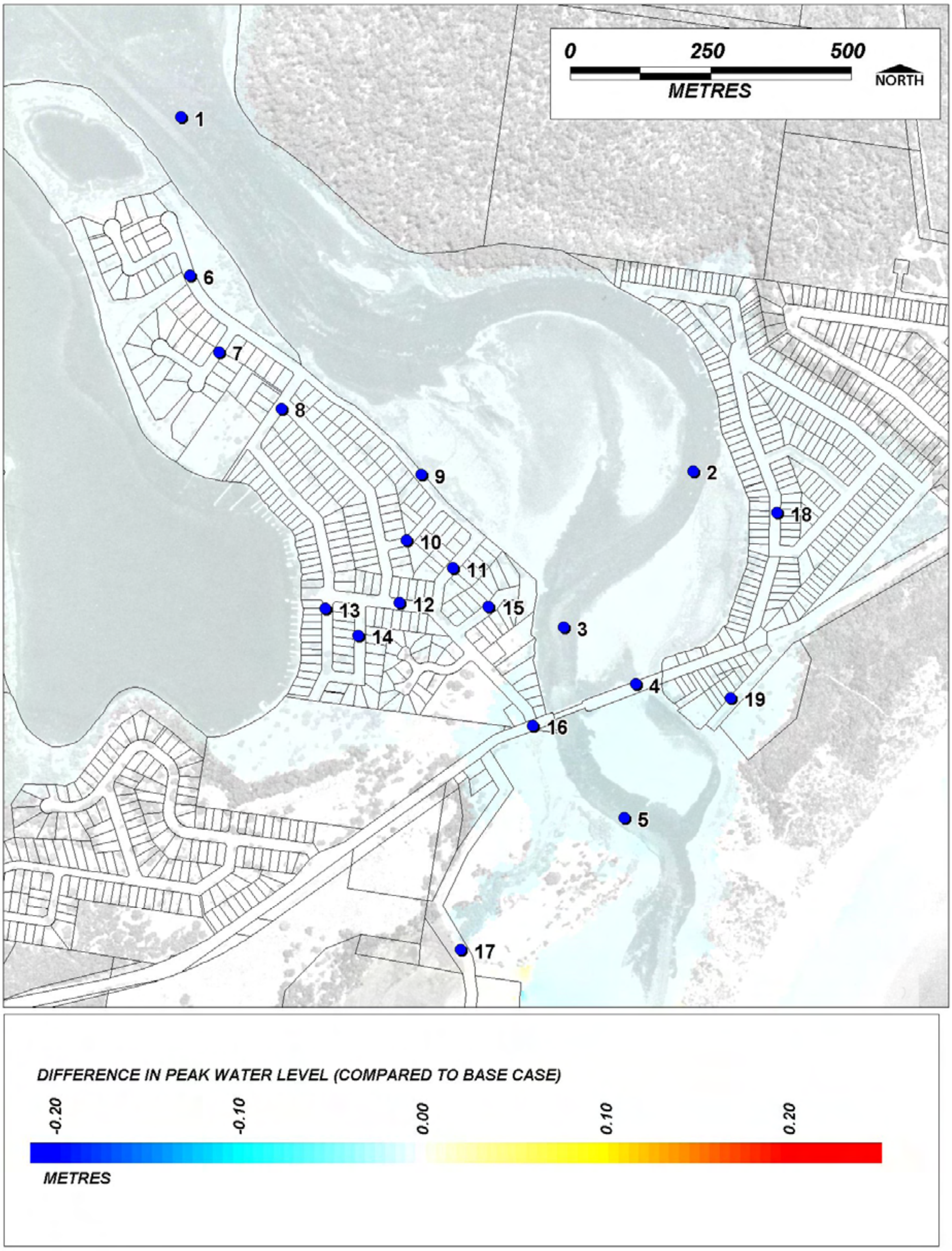

Figure C-21 Sensitivity Test 8 Peak Water Levels (Change from Base Case)
(Raised Roughness on Residential Lots)

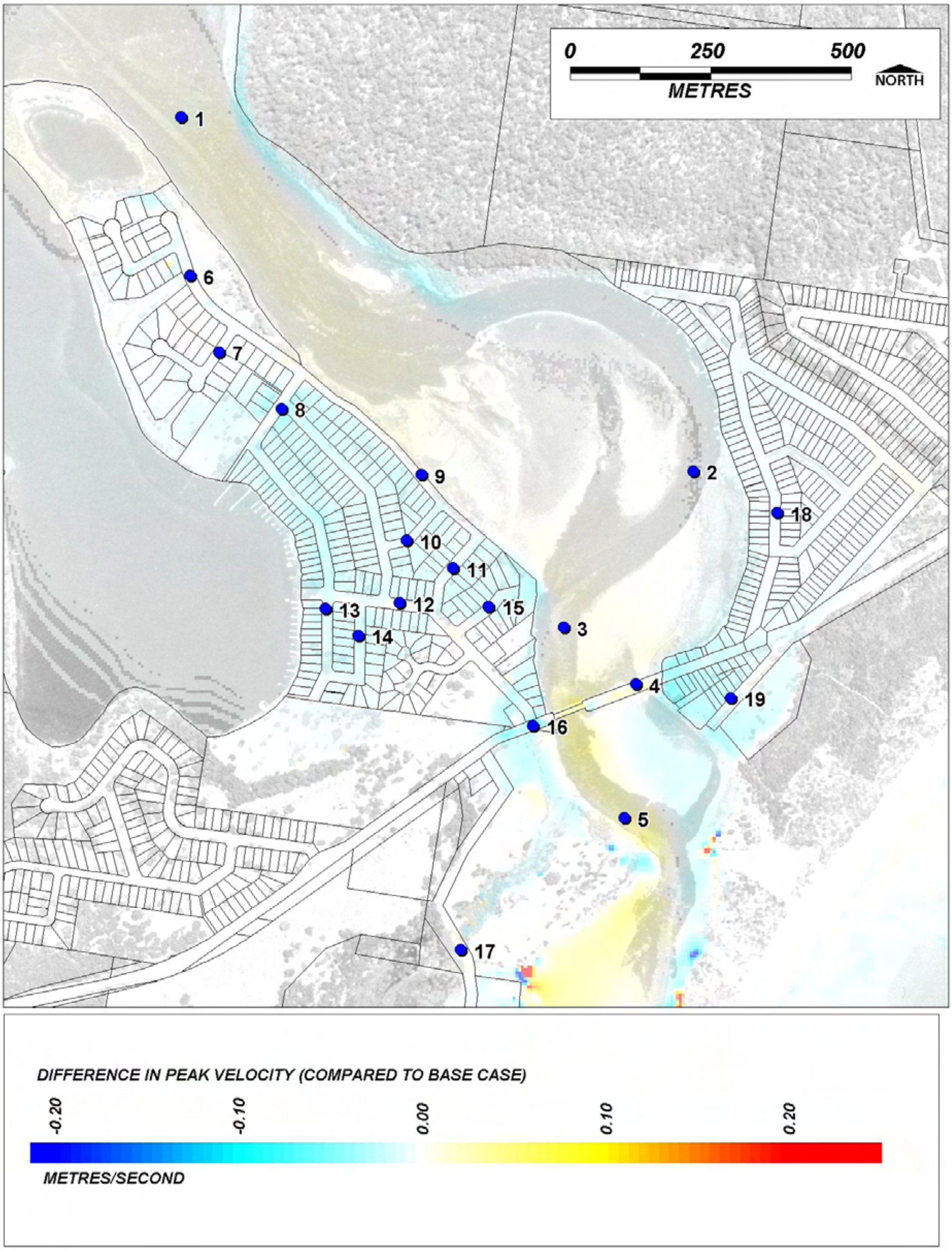

Figure C-22 Sensitivity Test 8 Peak Velocities (Change from Base Case)
(Raised Roughness on Residential Lots)

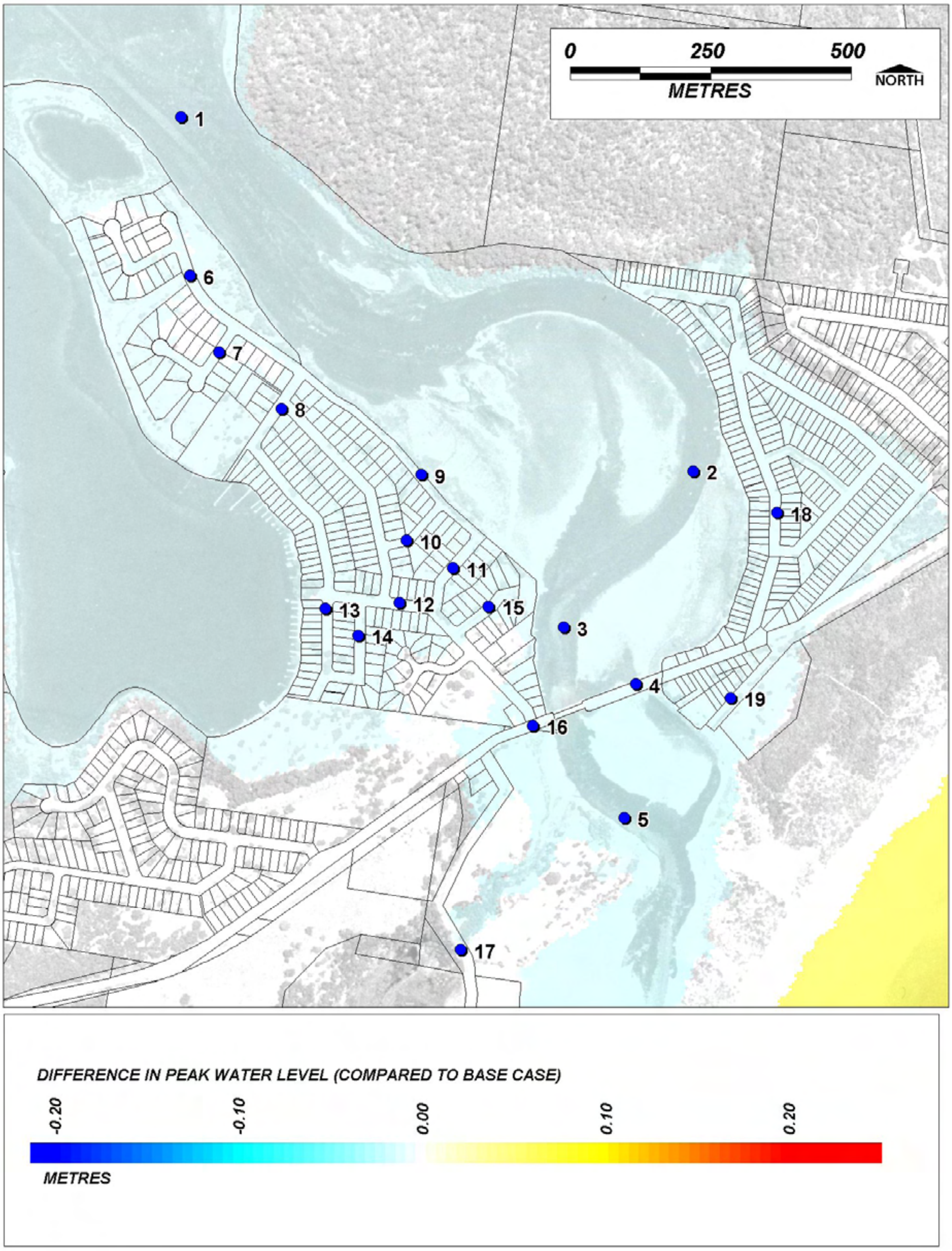

Figure C-23 Sensitivity Test 9 Peak Water Levels (Change from Base Case)

(Spring Tide Ocean Boundary)

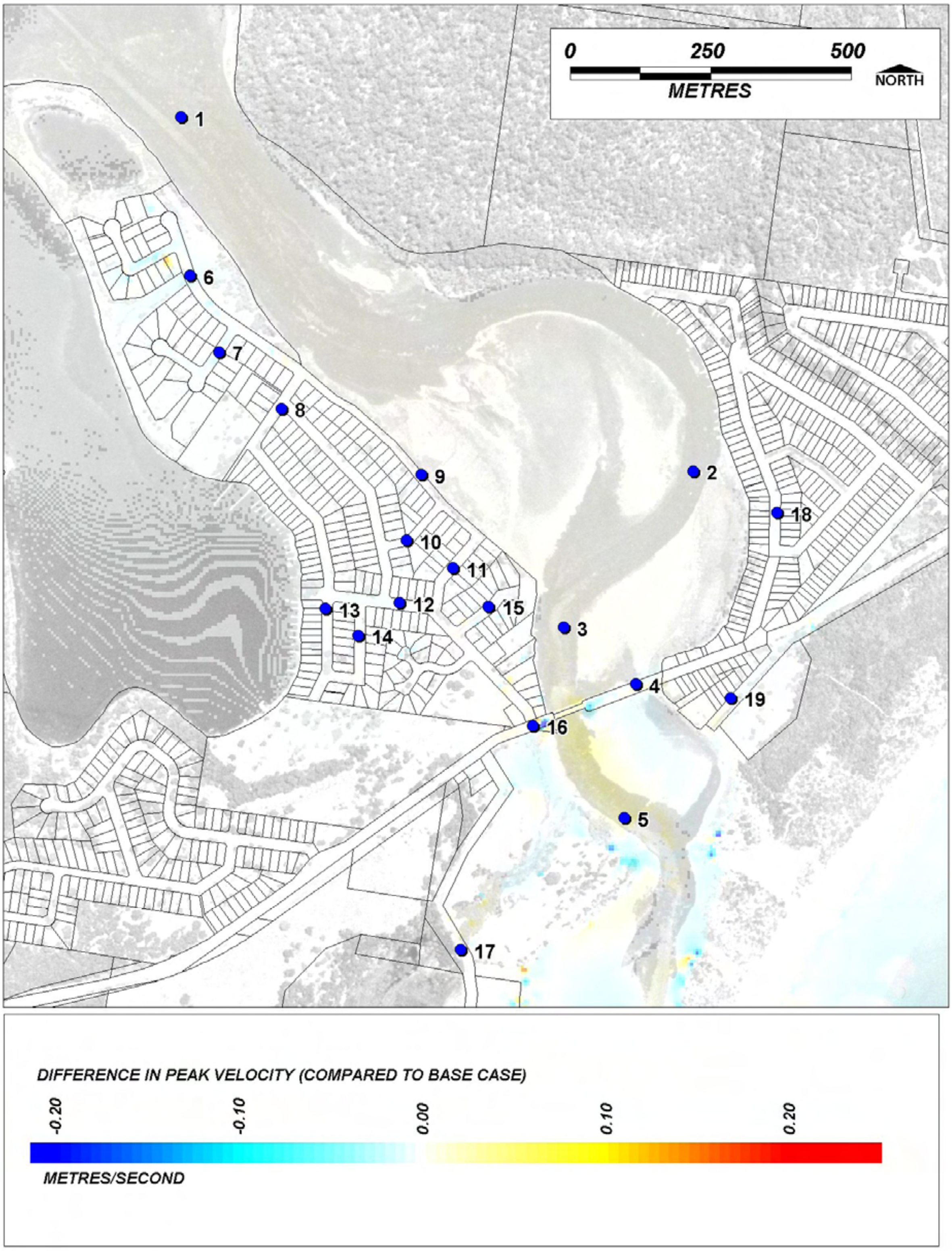

Figure C-24 Sensitivity Test 9 Peak Velocities (Change from Base Case)

(Spring Tide Ocean Boundary)

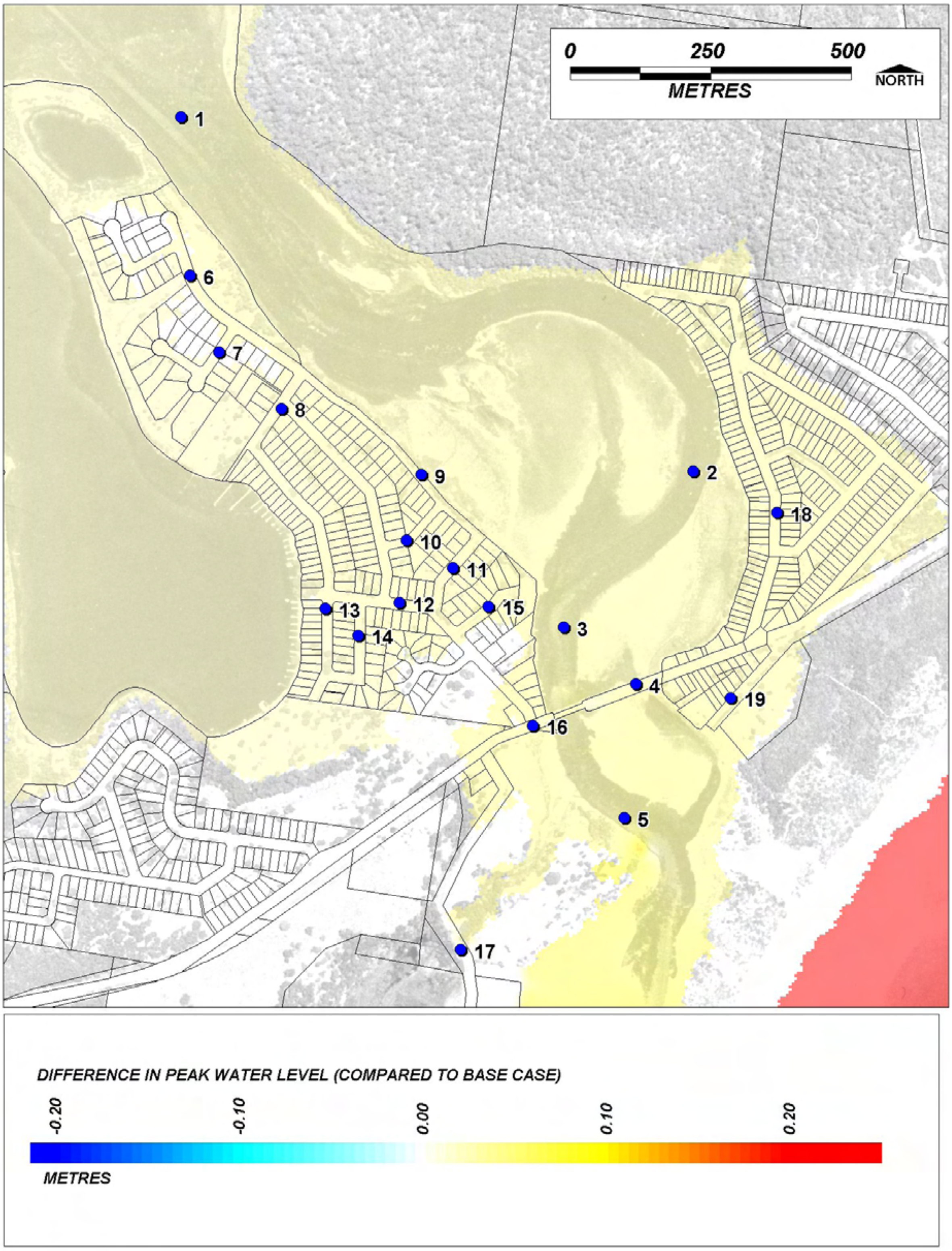

Figure C-25 Sensitivity Test 10 Peak Water Levels (Change from Base Case)
(King Tide Ocean Boundary)

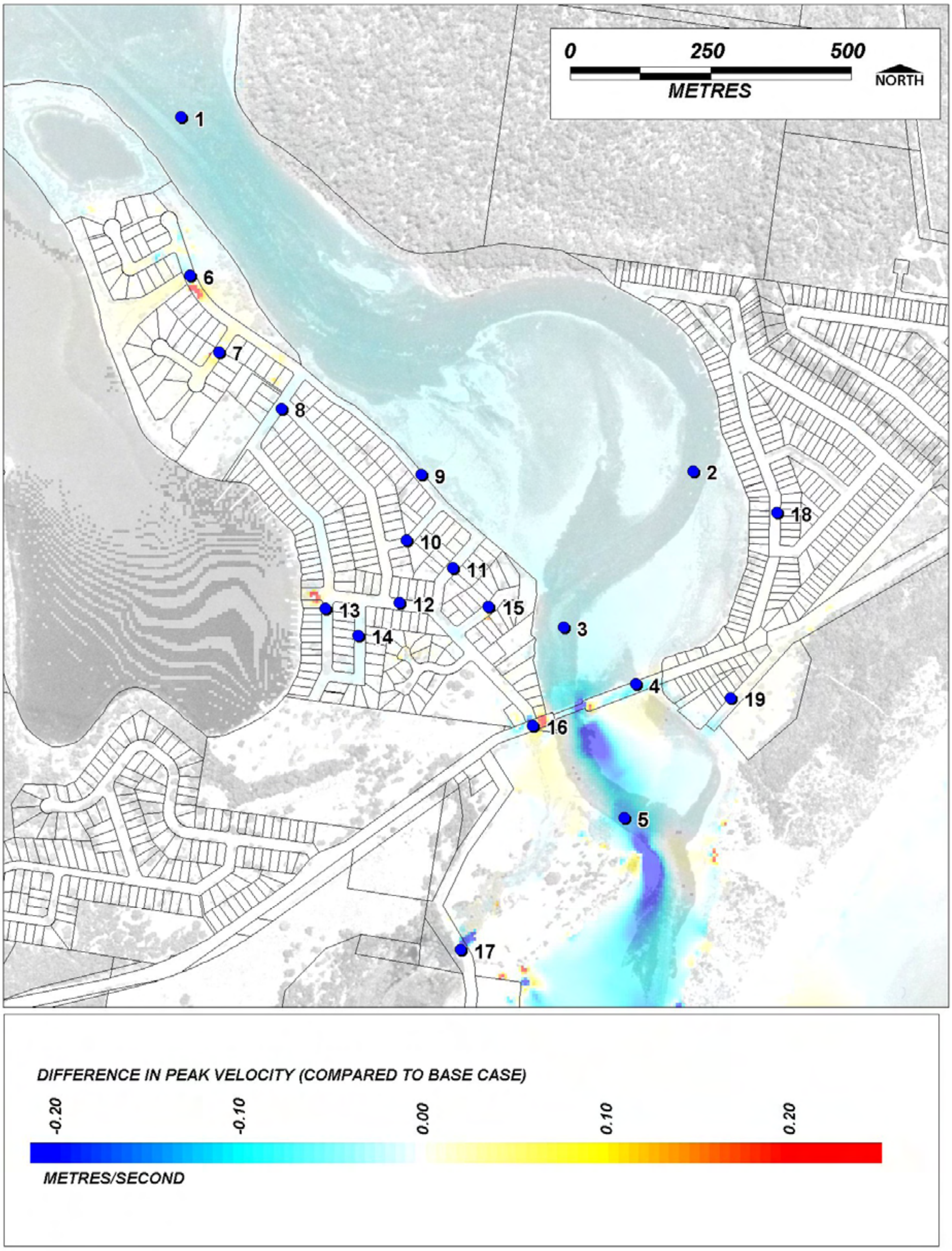

Figure C-26 Sensitivity Test 10 Peak Velocities (Change from Base Case)
(King Tide Ocean Boundary)

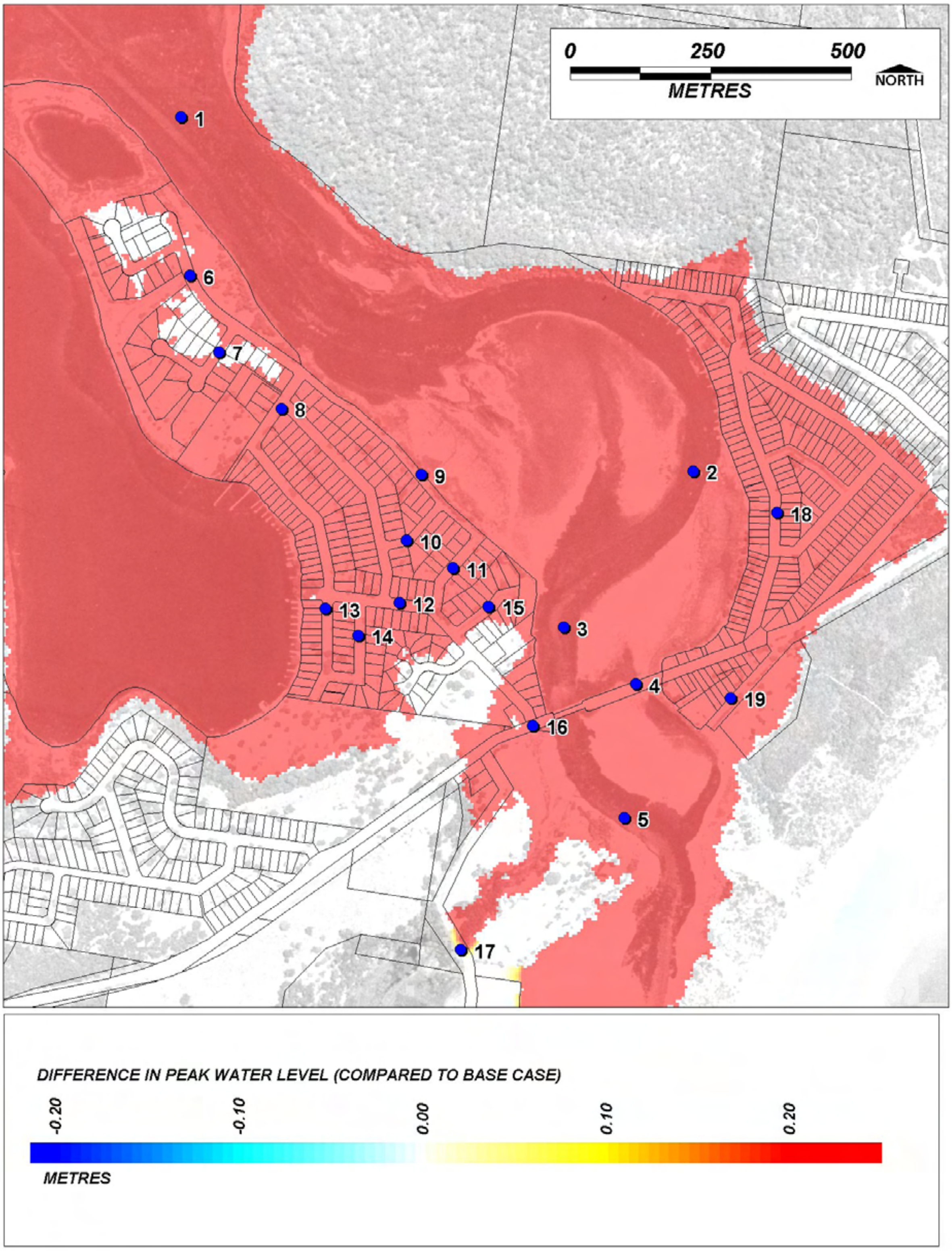

Figure C-27 Sensitivity Test 11 Peak Water Levels (Change from Base Case)
(Closed Entrance)

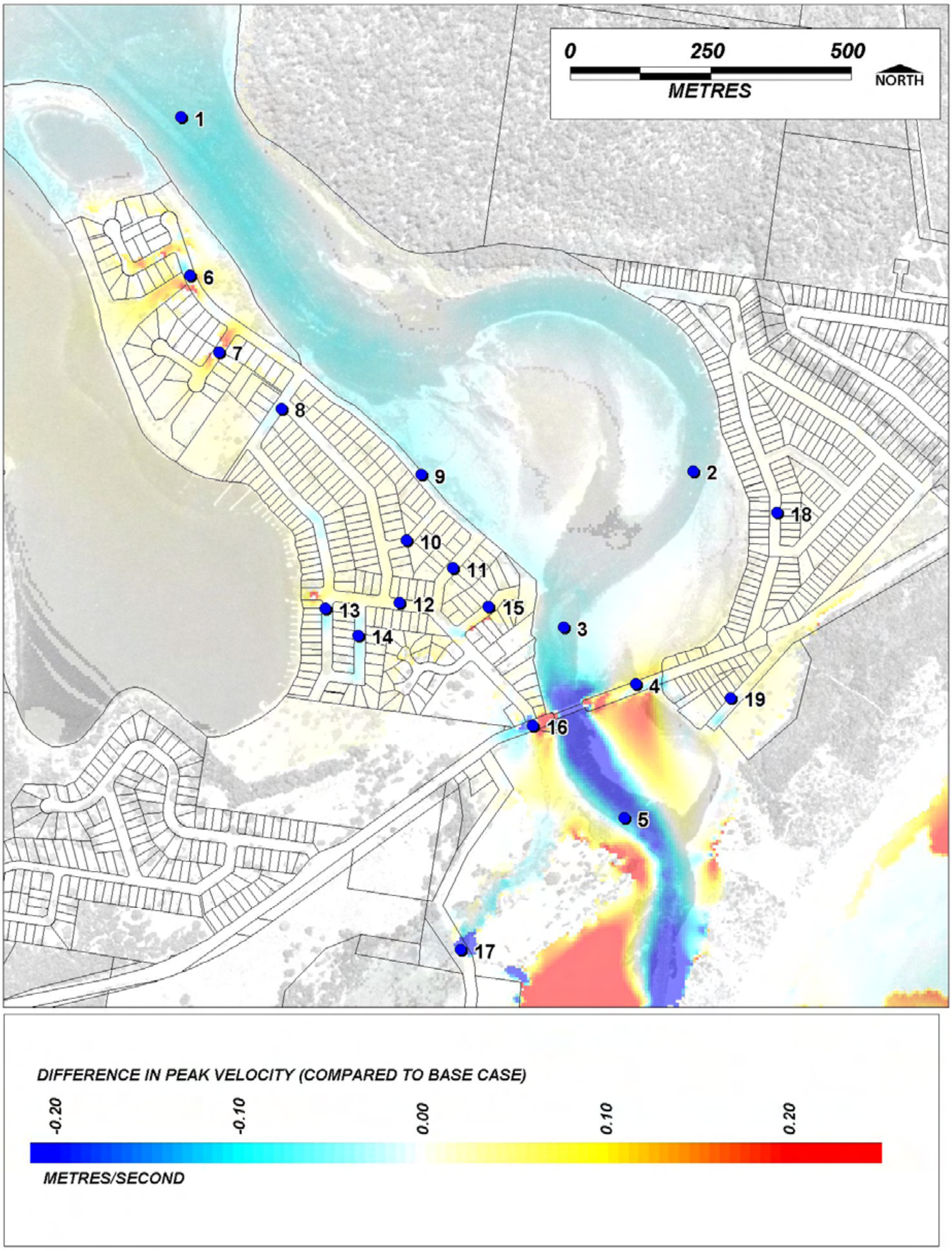

Figure C-28 Sensitivity Test 11 Peak Velocities (Change from Base Case)

(Closed Entrance)

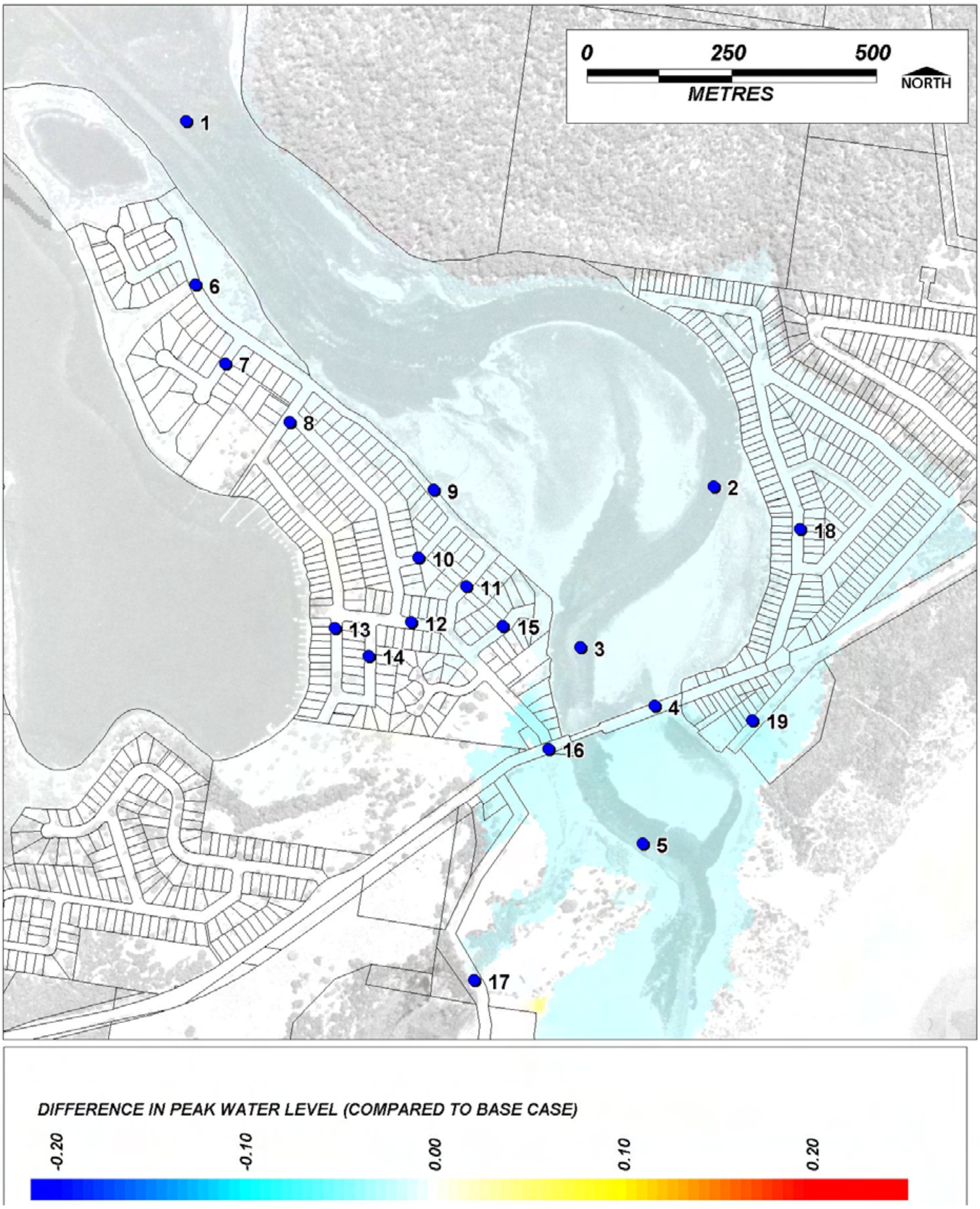

Figure C-29 Sensitivity Test 12 Peak Water Levels (Change from Base Case)

(High Roughness in Residential Areas)

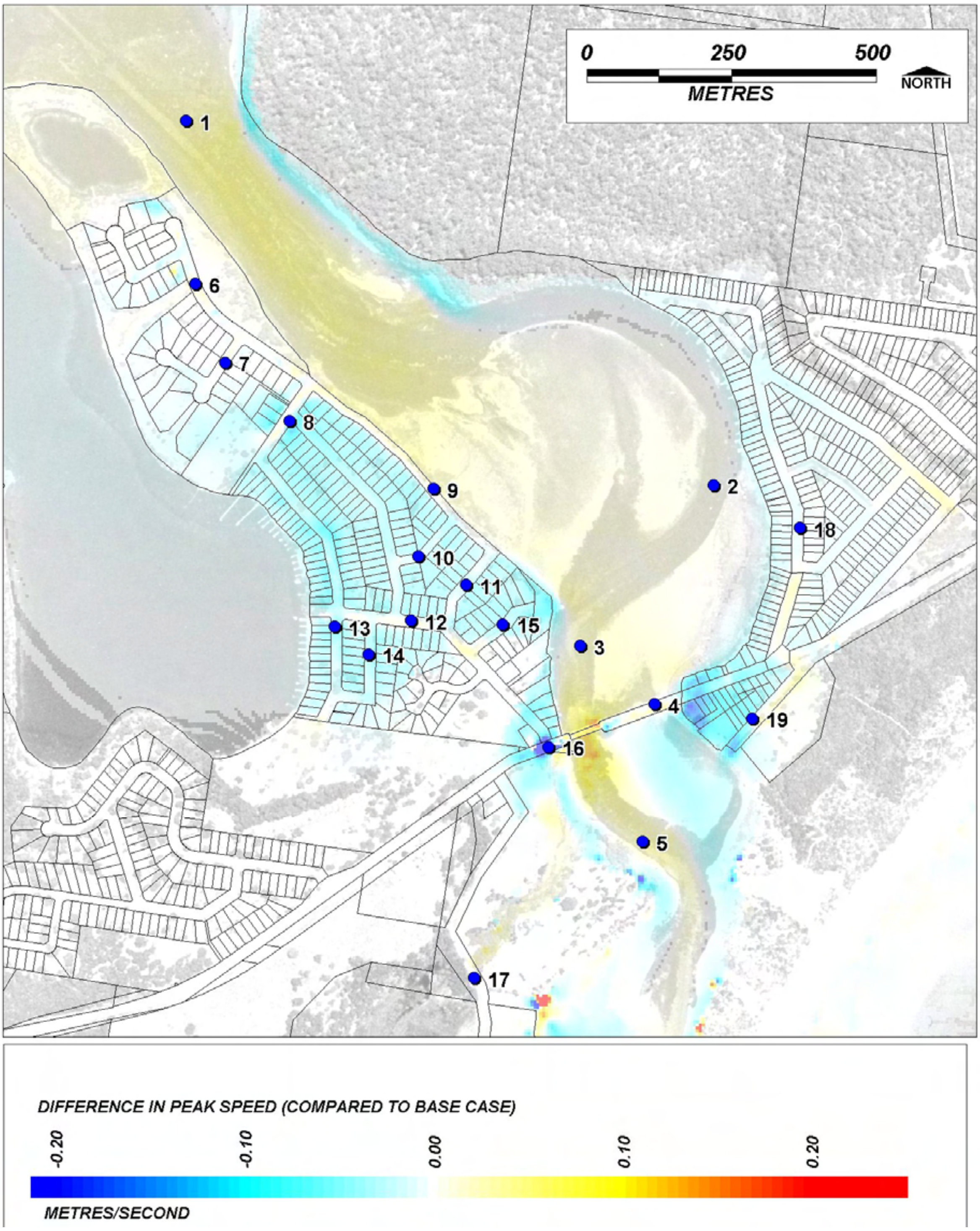

Figure C-30 Sensitivity Test 12 Peak Current Speed (Change from Base Case)

(High Roughness in Residential Areas)

APPENDIX D: SUMMARY OF KEY PARAMETERS, DATA AND ASSUMPTIONS

GENERAL	Catchment Area	78 km^2	
	Lake Area	4 km^2	
CALIBRATION	Historical Events	1971	1992
	Topography	2001 Hydrosurvey & Cross Sections 2004 Ground Survey of Floodplain Historical Aerial Photography	
	Hydrology		
	▪ Rainfall Depths	72 hour totals from 4 gauges	72 hour totals from 3 gauges
	▪ Temporal Patterns	Nowra RAN Air Station	Turpentine
	▪ Stream Gauge	None	None
	▪ Initial Loss	15 mm/hr	15 mm/hr
	▪ Continuing Loss	2.5 mm/hr	2.5 mm/hr
	Hydraulics		
	▪ Initial Lake Level	No Records, 0.35 adopted	0.35
	▪ Ocean Levels	Sydney and Jervis Bay tide records	Sydney and Jervis Bay tide records
	▪ Wave Set-up	No Records 0.5 m adopted	Batemans Bay Wave Rider
	▪ Number Recorded Flood Levels	4	2
DESIGN	Design Events	5, 10, 20, 50, 100, 200 year and PMF	
	Critical Duration	18 hours	
	Source of IFD	AR&R (2001) for all events except the PMF (BoM, HRS No 5)	
	Source of Temporal Patterns	AR&R (2001) for all events except the PMF (BoM, HRS No 5)	
	Source of Spatial Distribution	AR&R (2001) for all events except the PMF (BoM, HRS No 5)	
	Hydrologic Model		
	• Initial Rainfall Loss	10 mm	
	• Continuous Rainfall Loss	0 mm	

	Hydraulic Model Roughnesses	
	• Sand	0.020
	• Deep water	0.022
	• Causeway	0.030
	• Shoals	0.030
	• Residential	0.6
	• Caravan Parks	0.6
	• Wetlands	0.08
	• Entrance Shoals	0.030
	• Entrance Rock shelf	0.025