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LAKE CONJOLA FLOOD STUDY

Final Report

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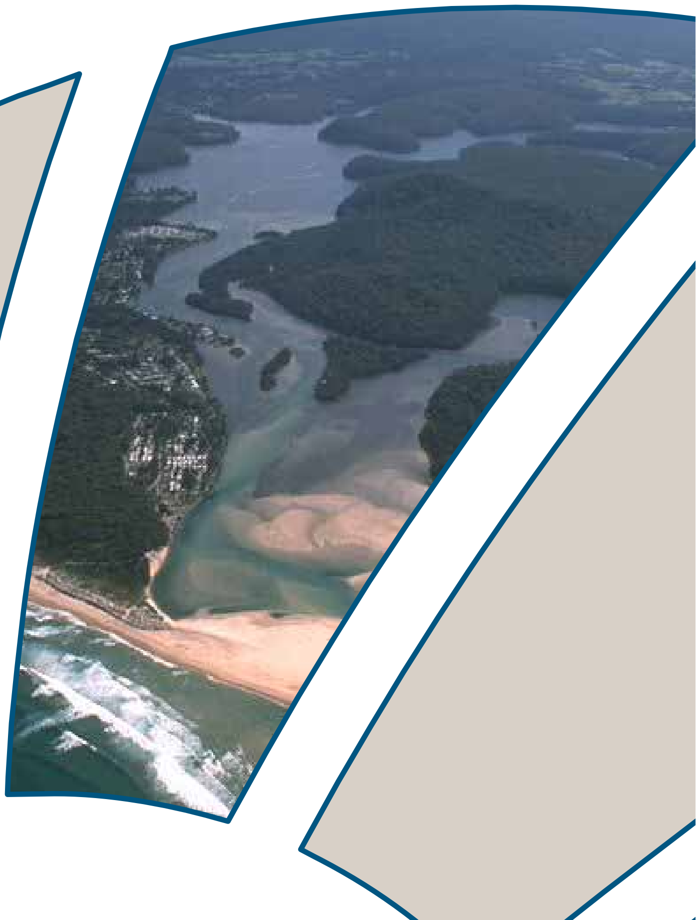

Lake Conjola Flood Study

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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 Lake Conjola 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.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

GLOSSARY

The following glossary has been adapted from the 2005 NSW Floodplain Development Manual (with references made to various section and appendices of the Manual within the definitions).

annual exceedance probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. Eg, if a peak flood discharge of 500 m3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m3/s or larger events occurring in any one year (see ARI).
attenuation	Weakening in force or intensity
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
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 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
bathymetry	Measurement of water depth
catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
development	Is defined in Part 4 of the EP&A Act: Infill development: refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. New development: refers to development of a completely different nature to that associated with the former land use. Eg, the urban subdivision of an area previously used for rural purposes. New developments involve re-zoning and typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. Redevelopment: refers to rebuilding in an area. Eg, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either re-zoning or major extensions to urban services.
discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic meters per second (m3/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).
Entrance channel	With specific relation to Conjola Lake, this refers to the section of waterway between the ocean and the deep lake basin, upstream of The Steps.

Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage (refer Section C6) before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defenses excluding tsunami.
Flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
Flood liable land	Is synonymous with flood prone land (ie) land susceptible to flooding by the PMF event. Note that the term flood liable land covers the whole floodplain, not just that part below the FPL (see flood planning area).
Floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
Floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
Flood planning area	The area of land below the FPL and thus subject to flood related development controls. The concept of flood planning area generally supersedes the “flood liable land” concept in the 1986 Manual.
Flood planning levels (FPL’s)	Are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the “standard flood event” in the 1986 manual.
Flood prone land	Land susceptible to flooding by the PMF event. Flood prone land is synonymous with flood liable land.
Flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. <u>existing flood risk</u>: the risk a community is exposed to as a result of its location on the floodplain. <u>Future flood risk</u>: the risk a community may be exposed to as a result of new development on the floodplain. <u>Continuing flood risk</u>: the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.

Flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
Floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.
Geomorpholgy	The study of the characteristics, configuration and evolution of rocks and land forms
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in Appendix L.
hydrodynamic	Study of fluid (eg water) in motion
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
Hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
isohyetal	A line on a map or chart connecting areas of equal rainfall
local drainage	Smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
morphology	See “geomorphology”
peak discharge	The maximum discharge occurring during a flood event.
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. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

probable maximum precipitation	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to water level (both measured with reference to a specified datum).
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
temporal	Relating to time
tidal delta (also known as the flood tide delta)	The section of an estuary that contains marine sand shoals. These shoals have been formed by the deposition of marine sand during the flood tide (incoming tide).
topography	Study of the surface features of an area

1 INTRODUCTION

1.1 Background

Conjola Lake is located on the New South Wales South Coast, about 200km south of Sydney and 50km south of Nowra. The lake is within the local government area of the City of Shoalhaven. Planning landuse of adjoining lands come under the statutory responsibility of Shoalhaven City Council. A locality map is presented in Figure 1-1 with more detailed catchment maps in Figure 1-2 and Figure 1-3.

Conjola Lake is an estuarine lake that is connected to the Tasman Sea via a shoaled entrance channel. The lake has a surface area of 4.3 km^2 which can be conveniently divided into three sections:

- The entrance channel, connecting the lake to the Tasman Sea. The channel is shallow for most of its length with significant sand shoaling at the downstream end. The morphology of the channel is subject to catchment inflows and ocean processes. As a consequence, the channel width and shape varies with time depending on antecedent and prevailing runoff and coastal conditions. This can lead to partial or even total entrance closure at times. The entrance channel is approximately 3km long, and contains small sandy vegetated islands (Conjola Island, Princess Island, Chinamans Island, plus a number of smaller unnamed islands). Chinamans Island is reported to host a number of residential cottages.
- The main body of the lake (Conjola Lake). The lake is a drowned river valley, connected to the sea via the entrance channel. As such, the lake is deep (up to 10m) with marine sand ingress terminating at "the Steps", about 3km upstream of the sea.
- A secondary lake (Berringer Lake) to the north. Berringer Lake is connected to the entrance channel via a relatively short and narrow passage.

Conjola Lake drains a catchment of approximately 145 km^2. It is fed by numerous creeks mostly draining runoff from western subcatchments. The major tributaries are Luncheon, Conjola and Gooloo Bunnair Creeks, which originate in the mountainous western section of the lake's catchment. Most of the catchment is forested and grazing (cattle and dairy) land, with five small urban villages located on the foreshores of the lake and the adjacent coast. The catchment slope is generally steep, resulting in a quick flow of runoff to the main body of the lake.

The village of Lake Conjola is located on the southern side of the entrance channel and is the largest residential area around the lake. It contains a number of caravan parks located on very low-lying land fringing the lake foreshores. The township of Manyana/Cunjurong Point is located on the northern side of the entrance channel and should not be affected by Conjola Lake floods. Conjola Park is located on the foreshores of one of the south-western embayments of Conjola Lake (Yooralla Bay). Fishermans Paradise is located on the western side of Conjola Creek at the north-western extremity of the lake. Other small urban areas comprise Killarney on the southern foreshore of Conjola Lake and Berringer Lake on the eastern foreshore of Berringer Lake.

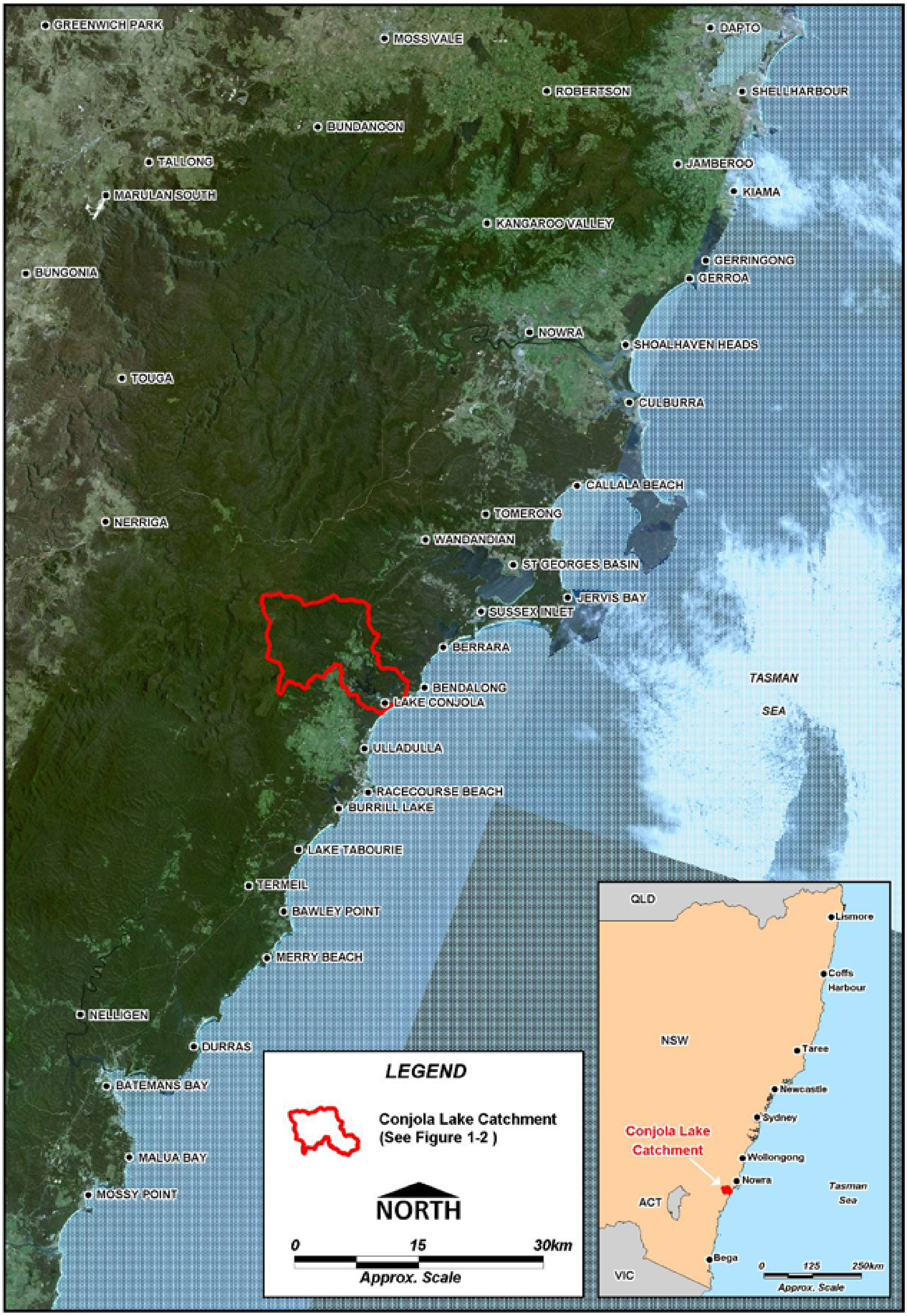

Figure 1-1 Conjola Lake Catchment Locality Map

Figure 1-2 Conjola Lake Catchment

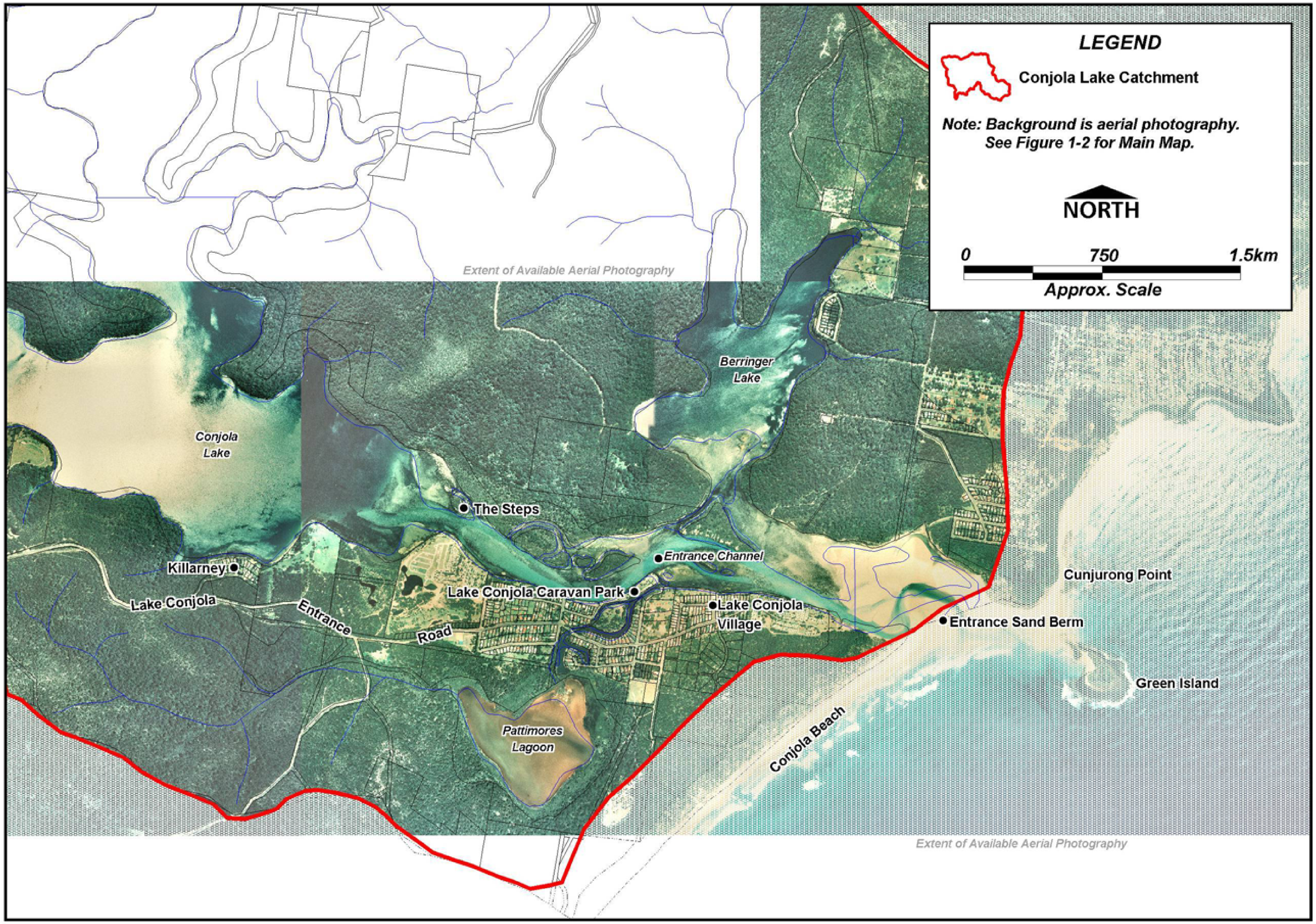

Figure 1-3 Conjola Lake Catchment (inset)

The Princes Highway, running north-south, divides the Conjola Lake catchment into two zones: the upper catchment to the west and the immediate lake catchment to the east. Road access to the southern shores of Conjola Lake is provided via the Lake Conjola Entrance Road, while access to the northern parts of the lake is via Bendalong Road.

1.2 The Need for Floodplain Risk Management at Lake Conjola

In 1999, Patterson Britton and Partners (PBP) carried out a study on the entrance processes of Conjola Lake. The main objectives of this study were to provide an understanding of the coastal and estuarine processes that impact on the shoaled condition of the entrance, and to provide a series of options for managing the entrance.

As part of the PBP (1999) study, a simple flood assessment was carried out to ascertain the current level of flood risk experienced by residents around Conjola Lake, and to determine the changes to this flood risk associated with proposed entrance management options. The PBP flood assessment indicated that within the township of Lake Conjola, the 1 in 100 year ARI flood level was in the order of 2.9m AHD, while the level in the more upstream and deeper sections of the lake (i.e. upstream of “the Steps”) could be up to 4.0m AHD. These levels are considerably higher than Council’s current flood planning level in the area, which has been set at RL 2.5m AHD for the entire Lake Conjola area. It is understood that the current planning level was established based on historical flood levels (2.4 m AHD at Lake Conjola village in 1971). The current flood planning level of 3.0m AHD is based on this flood level. Shown in Table 1-1 are the results of the flood assessment carried out by PBP (1999).

Table 1-1 Predicted Flood Levels in Lake Conjola (PBP, 1999)

Location	Flood Level (mAHD)				
	storm surge only[1]	major flooding scenario[2]	1 in 100 yr flood with no storm surge	minor flooding scenario[3]	1 in 5 yr flood with no storm surge
Ocean Tailwater	2.2	2.2	0.6	2.2	0.6
d/s Caravan Park	1.5	2.6	1.7	2.2	1.3
Lake Conjola post office	1.5	2.9	2.0	2.3	1.4
Conjola Lake	1.5	4.0	3.1	2.9	1.9

1 1 in 5 year storm surge of 1.6 m superimposed on a tide of mean spring range ie. peak ocean level of 2.2 m AHD.
2 1 in 100 year freshwater flow coinciding with 1 in 5 yr storm surge.
3 1 in 5 freshwater flow coinciding with 1 in 5 yr storm surge.

In response to the PBP flood assessment results, Council appointed WBM to carry out a formal flood study to accurately determine the current level of flood risk to the local community.

1.3 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 1-4.

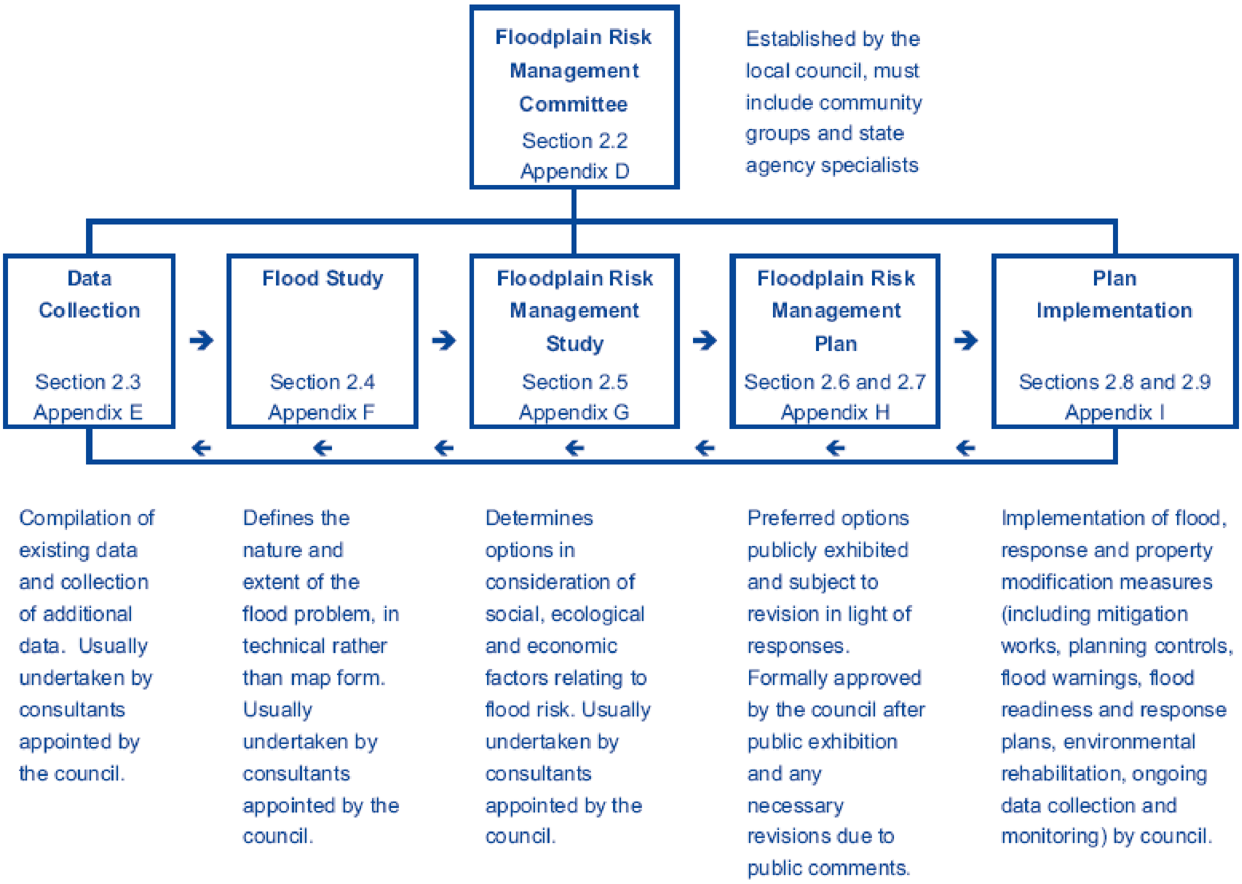

Figure 1-4 The NSW Floodplain Risk Management Process (Source: 2005 NSW Floodplain Development Manual)

1.4 Study Objectives

The primary objective of the Lake Conjola Flood Study is to examine and define the flood behaviour of Conjola Lake in the vicinity of the villages lying on its foreshores, which include Lake Conjola village, Conjola Park, Fisherman's Paradise, and Berringer Lake. It is proposed that the Flood Study will provide input for the subsequent floodplain management study and plan.

Specifically, this study developed computer models to predict the flood risks for areas surrounding Conjola Lake. These models have been used to determine hydraulic categories and the provisional flood hazard.

During the Floodplain Risk Management Study, which will be carried out subsequent to the present Flood Study, the models will be used to:

- Determine adequacy of existing flood protections;
- Assess various flood mitigation options;
- Review the appropriateness of Council's adopted flood planning levels;
- Establish the effects on flood behaviour by future urban development; and

- Test the impacts of specific development proposals on flooding.

1.5 About This Report

This report documents the Study's objectives, results and conclusions. It is divided into a main report that presents the 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's calibration process.

Section 6 presents the design flood conditions (rainfall, ocean, topography).

Section 7 presents the design flood results.

2 STUDY METHODOLOGY

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 computer models;
- Calibration and verification of models to available historical information; and
- Establish design flood conditions and behaviour by numerical modelling.

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

2.1 Compilation and Review of Available Information

Flood investigations carried out in the past have addressed various aspects of flooding within the Lake Conjola area. Relevant previous studies include:

- Lake Conjola, Stage 1: Estuary Processes Study Draft (August 1996), *Gutteridge Haskins & Davey Pty Ltd*, referred in this document as GHD (1996);
- Lake Conjola Entrance Study (Issue No. 2, May 1999), *Patterson Britton & Partners Pty Ltd*, including RAFTS-XP hydrology model and RMA hydraulics model, referred in this document as PBP (1999);
- Lake Conjola Entrance Management Plan (January 2003), *Manly Hydraulics Laboratory*, referred in this document as MHL (2003); and
- Lake Conjola Sewerage Investigations and corresponding EIS, *Shoalhaven Water*.

Appendices A and B present details from the previous studies that are considered relevant as background information for the flood study.

Information that was able to be extracted from the above data sources and used within the present study include:

- Surveys of lake cross-section profiles;
- Topographic data such as ground contours and spot heights for residential areas;
- Historical aerial photographs of the catchment and the entrance channel; and
- Measured levels and flows for specific historical events.

In addition, Geographical Information System (GIS) data such as roads, cadastre, waterways, etc. and aerial photography was obtained from Council or State Government.

All relevant information has been incorporated into the study, and is described where appropriate within other sections of this report.

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 also 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, and liaison with community members.

2.3 Identification of Historical Changes to Topography

The computer models developed as part of this study were calibrated to three historical floods (1971, 1975 and 1992) 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.

Comparison of different historical aerial photographs of the catchment shows that the vast majority of the catchment was in a forested condition at the time of these historical flood events, and that this condition largely remains today. Small increases in residential development over the past 30 years or so remains limited to areas around the lake foreshore and would have negligible impact on the broad scale flooding behaviour of the lake.

The only change to the lake that has significant potential to alter flood behaviour has been associated with the condition of the entrance channel. The channel is considered to be quite dynamic, responding to major catchment runoff and coastal storm events, particularly in respect to the height of the entrance berm. Aerial photograph interpretation has shown a high degree of variability in entrance channel morphology over the past 30 years (PBP 1999).

2.4 Collection of Historical Flood Information

Only limited historical flood information was found in the previous studies (GHD 1996, PBP 1999). To supplement this information a resident survey of the local communities was carried out, with responses from approximately 200 individuals, 58 of whom included information about historical floods in Conjola Lake. Analysis of the returned questionnaires resulted in further interviews (phone and site visit) with the residents, leading to the identification of additional information on their recollection of:

- The location of historical flood marks; and
- The historical flood behaviour (time, flow direction, velocities etc).

The historical flood information obtained from the interviews and subsequent survey of flood marks has been compiled directly into the historical flood database as described in Section 3.2.2.1 and reproduced in Appendix D. The returned questionnaires have been handed over to the Council for their confidential records.

2.5 Additional Survey Data

The existing topographic information (listed in Section 2.1) was complemented by additional topographic survey. The additional survey was carried out to establish:

- Structure levels and dimensions in the creeks flowing into Conjola Lake;
- Cross-sections of the creeks and floodplains upstream of the lake at Fisherman's Paradise; and
- Historic flood levels identified during the resident survey.

2.6 Computer Models

For the purpose of the Flood Study, four (4) different computer models were developed:

- Digital Elevation Model (DEM);
- Hydrologic Model;
- Hydraulic Model; and a
- Geomorphologic Model

2.6.1 Digital Elevation Model (DEM)

A Digital Elevation Model (DEM) was developed to provide a topographic base surface for the hydraulic model. The DEM is a simplified representation of the topography, which assigns an average ground elevation for each compartment of a square-cell rectilinear grid based on linear interpolations between the existing topographic survey points.

The DEM is explained and discussed in Section 4.4.

2.6.2 Hydrologic Model

A hydrologic model, discussed in Section 4.3, 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 hydraulic model. These hydrographs are used by the hydraulic model to simulate the passage of a flood through Conjola Lake.

2.6.3 Hydraulic Model

The hydraulic model, discussed in Section 4.5, developed for this study includes:

- A high resolution two-dimensional model of the shallow parts of the lake, the entrance channel and the urbanised floodplains; and
- A low resolution one-dimensional model representation of the main body of the lake and non-urban floodplains.

2.6.4 Geomorphologic Model

The geomorphologic model developed for the study (refer Section 4.6) has been combined with the hydraulic model to assess bed and bank scour conditions at the lake entrance. The geomorphologic model uses the hydraulic parameters to predict erosion and accretion, and then recalculates the bed levels for the hydraulic model at each time step.

2.7 Calibration and Verification of Models

The hydraulic model was 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. Default values were adopted for the hydrologic model as no flow data was available for calibration or verification.

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

- The availability, completeness and quality of rainfall, flood level and other hydrograph data;
- The amount of data collected during the historical flood information survey – events which have substantially more 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 six floods with sufficient data to potentially support a calibration process. Further analysis on the quality of the data and the importance of the floods reduced the number to three. The largest event, February 1971, along with the February 1992 event, were chosen for calibration of the models, while the March 1975 flood was used for model verification.

Further information regarding the calibration and verification events, particularly specific details of flood behaviour, is provided in Section 3.3.

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

2.8 Establish Design Flood Conditions

The hydrologic model (rainfall losses) as well as the calibrated and verified hydraulic model (topography) was modified as necessary to represent present day conditions. The models were then run to define present day design flood conditions. The entrance channel configuration was based on the preferred entrance configuration as shown in the Draft Lake Conjola Entrance Management Plan (MHL 2002).

Design floods are statistical-based events which have a particular probability of occurrence. For example, the 1% Annual Exceedance Probability (AEP) event, which may also be known as the 1 in 100 year ARI flood, is the best estimate of a flood that has a 1% (i.e. 1 in 100) chance of occurring in any one year (on average). In the case of Conjola Lake, no stream flow records allow an analysis of long-term historical records of floods in the lake. Design floods were therefore based on design rainfall estimates according to Australian Rainfall and Runoff (IE Aust 2001).

A series of sensitivity tests were carried out on the predicted modelling results. These tests were conducted to determine the relative importance of different hydrologic and hydraulic factors, such as spatial rainfall variability, flow resistance coefficients and boundary conditions. The tests assist in establishing the relative accuracy of the modelling results.

The design flood conditions are presented in Section 6.

2.9 Design Flood Mapping

Design flood mapping was undertaken using output from the hydraulic model. Maps were 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 hydraulic model results and are also mapped. These maps are described and presented in Section 7 and/or the attached CD-Rom.

In addition, longitudinal profiles of the maximum water levels along Conjola Lake, from Fisherman's Paradise to the entrance channel, are presented in Section 7.

3 HISTORICAL FLOOD INFORMATION

3.1 General Flood Behaviour at Conjola Lake

Flooding in Conjola 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 ocean, resulting in storage of flood waters within the lake. Increased water levels in the lake potentially inundate floodplain areas behind foreshores. The maximum height of flood levels in the lake is a function of the difference between inflow and outflow rates. Thus, if the entrance channel is heavily shoaled or even closed, the difference would be great and water levels would potentially be high. In these circumstances, maximum flood levels would be controlled by the condition of the sand berm, as this berm undergoes considerable scour and increases the size of the entrance when overtopped by flood waters.
- Severe ocean conditions comprising barometric storm surge and wave setup due to a large offshore wave climate. Elevated ocean water levels push marine water into the lake through the entrance channel, which can subsequently inundate low-lying properties around the lake. 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.

Through community consultation, another flooding mechanism was identified for the village of Lake Conjola. During an intense rainfall event, with high water levels in the lake, the local runoff coming from Pattimores Lagoon is too great for the Lake Conjola Entrance Road bridge opening. The structure generates a significant afflux, which increases upstream water levels resulting in overbank flooding between Pattimores Lagoon and the Lake.

3.2 Recorded Information

3.2.1 Rainfall

Only one operational rain gauge station is located within the Conjola Lake catchment. The station is at Bendalong, near the entrance channel, and has been recording daily rainfalls since May 1939.

Bendalong station was used in a hydrologic analysis to estimate the major storm events that affected the project area. However, as the station reflects rainfall at the downstream end of the catchment, it is not necessarily representative of the whole catchment.

Table 3-1 ranks the 50 most severe storm events recorded at Bendalong. The analysis was undertaken for both 24-hour and 48-hour storm durations. The highlighted storms correspond to the events used for the study calibration.

Table 3-1 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

Additional rainfall station records were obtained from within a 50km radius around Conjola Lake for the historical floods chosen for the project calibration. Data from a maximum of 15 additional rainfall stations was available per calibration event. Of particular interest are 6-minute pluviograph records collected at Nowra RAN Air station, which is approximately 35km north of Conjola 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 Conjola Lake

3.2.2.1 Historical Flood Levels

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

- 12 historical flood levels were recorded for the February 1971 flood;
- 6 historical flood levels were recorded for the March 1975 flood; and
- 6 historical flood levels were recorded for the February 1992 flood.

In comparison, no more than two flood level records are available for all other major historical floods in Conjola Lake.

The majority of the recorded flood levels were located in the village of Lake Conjola. Other flood levels were available in Fisherman's Paradise. These additional levels at different locations along the lake were used to estimate the water level gradient occurring in the lake.

The quality (or assessed reliability) of the flood marks varied significantly. For example, some flood marks were indicated as precise positions on a brick wall, while others were merely a height estimate over caravan park land (with no defined geographical location of this height). The recorded historical flood marks were weighted depending on their description and their usefulness was judged accordingly. Comments on the flood levels are presented with the associated calibration events in Appendix D.

3.2.2.2 Water Level Gauge

A water level gauge is located within the lake, adjacent to the Lake Conjola Caravan Park. The gauge has been operating since September 1992, which unfortunately is approximately six months after a calibration flood. The gauge records water levels every hour and provides an indication of the tidal influence within the lake.

Measured tidal range within Conjola Lake is shown in Figure 3-1 and Figure 3-2. The large tidal range during 1992/3 can be explained by the entrance channel being scoured during the February 1992 flood. The tidal range then reduced with time, as the entrance became increasingly shoaled due to marine sand deposition. Records show that the entrance channel became completely blocked in 1998, resulting in no tidal variations in water level. In December 1998 the entrance was artificially opened after which tidal conditions were restored to the lake.

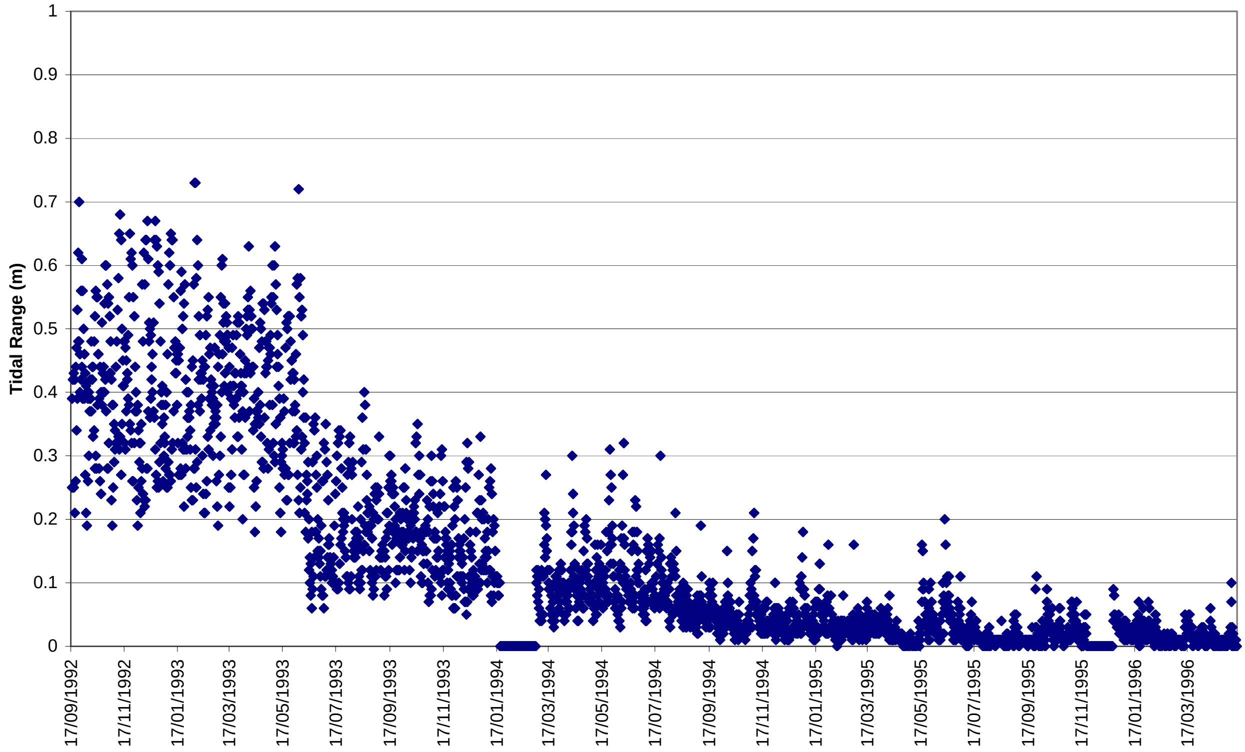

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Figure 3-1 Measured Tidal Range in Conjola Lake (1992-1996)

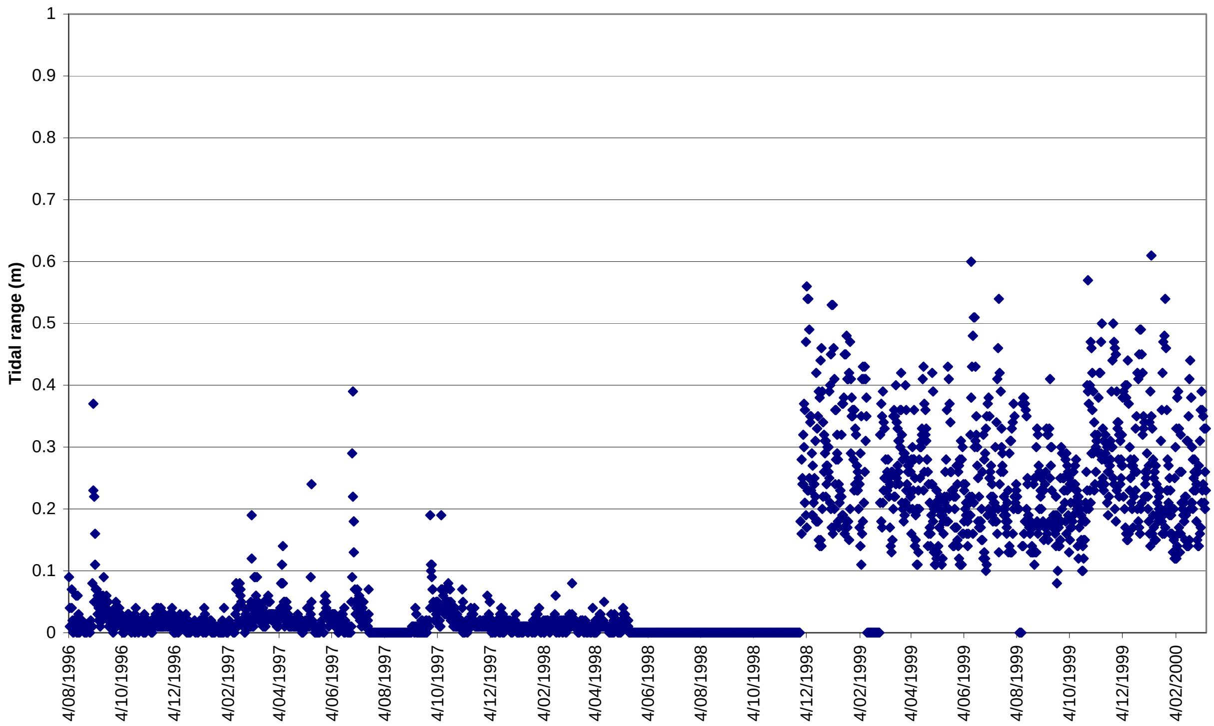

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Figure 3-2 Measured Tidal Range in Conjola Lake (1996-2000)

3.2.3 Discharge

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

3.2.4 Sea Levels

Conjola Lake discharges directly into the Tasman Sea. The sea levels can, under specific circumstances, influence the flood levels within the lake. Data was collected to assess the sea 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 sea levels.

3.2.4.1 Tide Levels

In this report the tide levels are referred to as being the sea levels resulting from astronomical tide plus barometric setup. There are no sea level recorders located at Conjola Lake that measure tide levels in the Tasman Sea. The nearest sea level recorder is located in Jervis Bay, which is approximately 20km north of Conjola Lake and has been recording hourly tide levels since July 1995.

Tide records in Sydney Harbour, which date back more than 100 years, were also collected.

A statistical analysis between Jervis Bay and Sydney tide records shows a good correlation between the two sets of data. As such, the Jervis Bay records were extrapolated, based on the Sydney Harbour data, to cover the periods containing flood calibration and verification events.

3.2.4.2 Wave Height

The Wave setup component can be a significant factor in elevating water levels above normal tide levels during storm conditions.

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 data is collected by Manly Hydraulic Laboratory (MHL). The analysis of offshore wave height is necessary in the determination of the wave setup component of sea levels.

Unfortunately, no wave data is available for the 1971 and 1975 flood events. However, offshore wave data was obtained for the period 09-Feb-1992 to 11-Feb-1992 from the Port Kembla and Batemans Bay waverider buoys. The data shows that a moderate storm event (maximum wave height of 4.5m) was recorded by both waverider buoys during this period.

The waverider buoy data is detailed further in Section 5.4.4.1.

3.3 Description of Historical Floods used for Calibration

3.3.1 The February 1971 Flood

3.3.1.1 Rainfall Temporal and Spatial Distribution

The rain gauges located around Conjola Lake catchment recorded rainfall almost every day in the first week of February 1971. The flood in Conjola Lake resulted from an increase in rainfall intensity. The area, already saturated from the preceding rainfall, received an average of 27mm, 42mm, 153mm and 119mm of rain in the 4 successive days leading up to 7 February 1971.

The isohyets of the 48-hour rainfall records to 9am on 7 February 1971 are presented in Figure 3-3. It can be seen from Figure 3-3 that the centre of the storm was located at the downstream end of the Conjola Lake catchment. It is possible that even greater rainfall intensity occurred over the ocean.

3.3.1.2 Flood Description

The 1971 flood event is the largest flood on record for Conjola Lake. The flood resulted from the occurrence of lengthy and intense rainfall over the catchment (700mm in one week, 365mm on day 6 of the event at Bendalong) and a blocked (closed) entrance channel.

The water levels in the lake rose due to the slow accumulation of catchment runoff behind the sand berm. The level of the sand berm blocking the entrance channel is not known, nor is the date and time when it started being overtopped. Local residents described the sea levels as being high, but it was not possible to assess the actual entrance channel conditions or sea levels.

There were reports that the flood level remained high for two to three days. Some people were living in Sydney and came down for the weekend only to find the access road still closed. A resident anecdotally narrates the inconvenience created by the lengthy flood duration: as a host to some urban guests at the time, he was forced to purchase a portable toilet to make up for the flooded sceptic tank.

As a result of the flood, the sand berm was overtopped and eroded resulting in re-opening at the entrance channel.

The 1971 flood is the principal calibration event as it represents the most severe flood event in living memory in Conjola Lake, and the amount of historical information is sufficient to provide indicative conclusions.

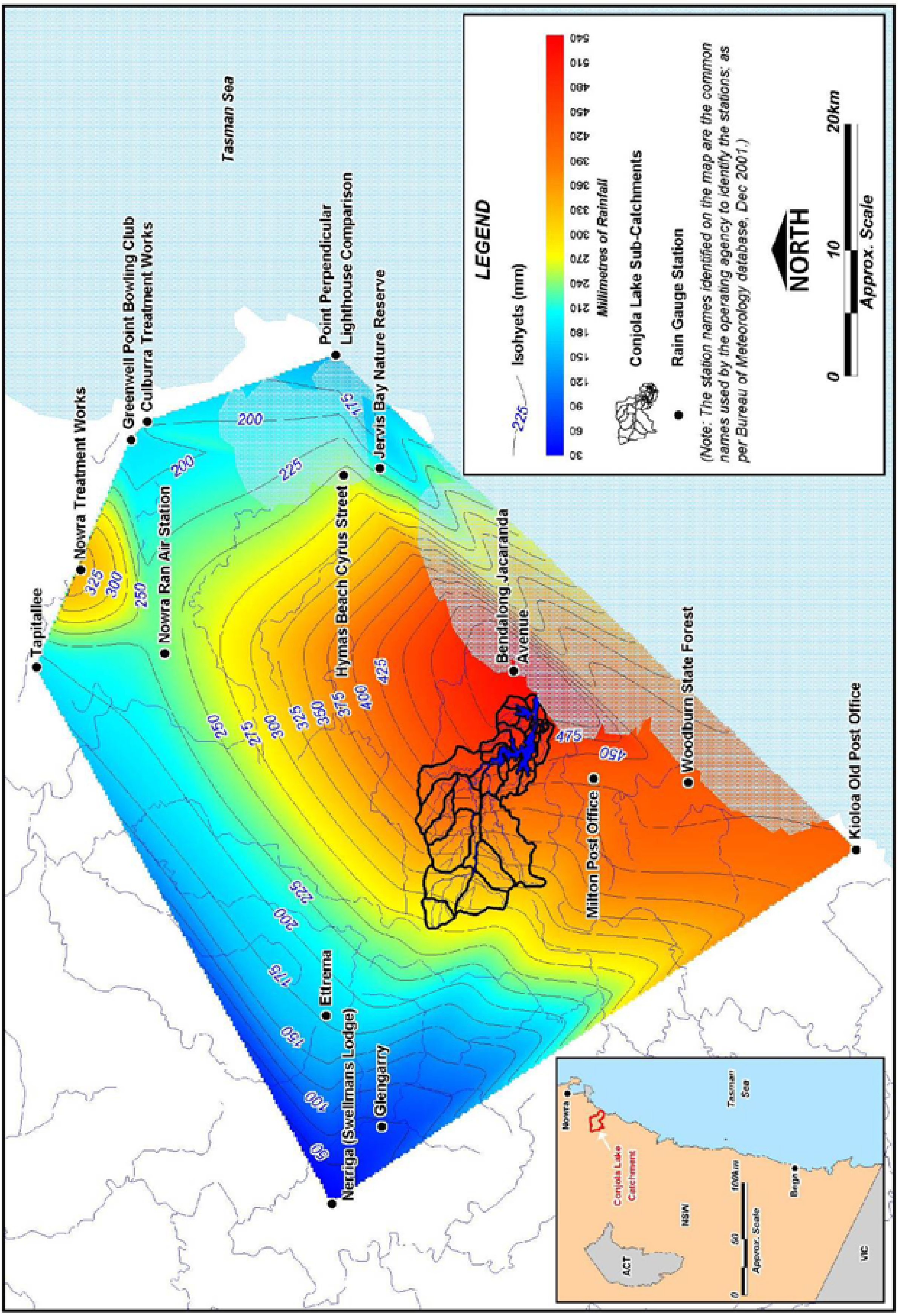

Figure 3-3 Isohyetal Map: 48 hours to 9am 7 February 1971

3.3.2 The March 1975 Flood

3.3.2.1 Rainfall Temporal and Spatial Distribution

The rain gauges located around the Conjola Lake catchment recorded virtually no rainfall before the 10 March 1975. The area received an average of 129mm and 159mm of rain in two successive days. Relatively intense rain continued for the following days, lengthening the tail of the flood hydrograph.

The isohyets of the 48-hour rainfall records to 9am on 11 March 1975 are presented in Figure 3-4. It can be seen from Figure 3-4 that the centre of the storm was mostly located on the ocean side of the catchment, similar to the February 1971 flood.

3.3.2.2 Flood Description

Four years after the 1971 event, another big flood was experienced by Lake Conjola residents, with 320mm of rain recorded in 2 days at Bendalong. This event can be considered as the second biggest flood event (in terms of rainfall) since 1971.

Local residents provided a similar description of the 1975 flood as the 1971 flood. Water levels rose following a combination of long and intense rainfall and a shoaled but open entrance.

In addition to flood damages created by elevated lake levels, it was reported that local runoff from Pattimores Lagoon created some overland flooding alongside the canal that links the lagoon to the lake.

The 1975 flood is considered important due to its high rainfall. Unfortunately the quality of the collected data is questionable and the event was only considered for verification.

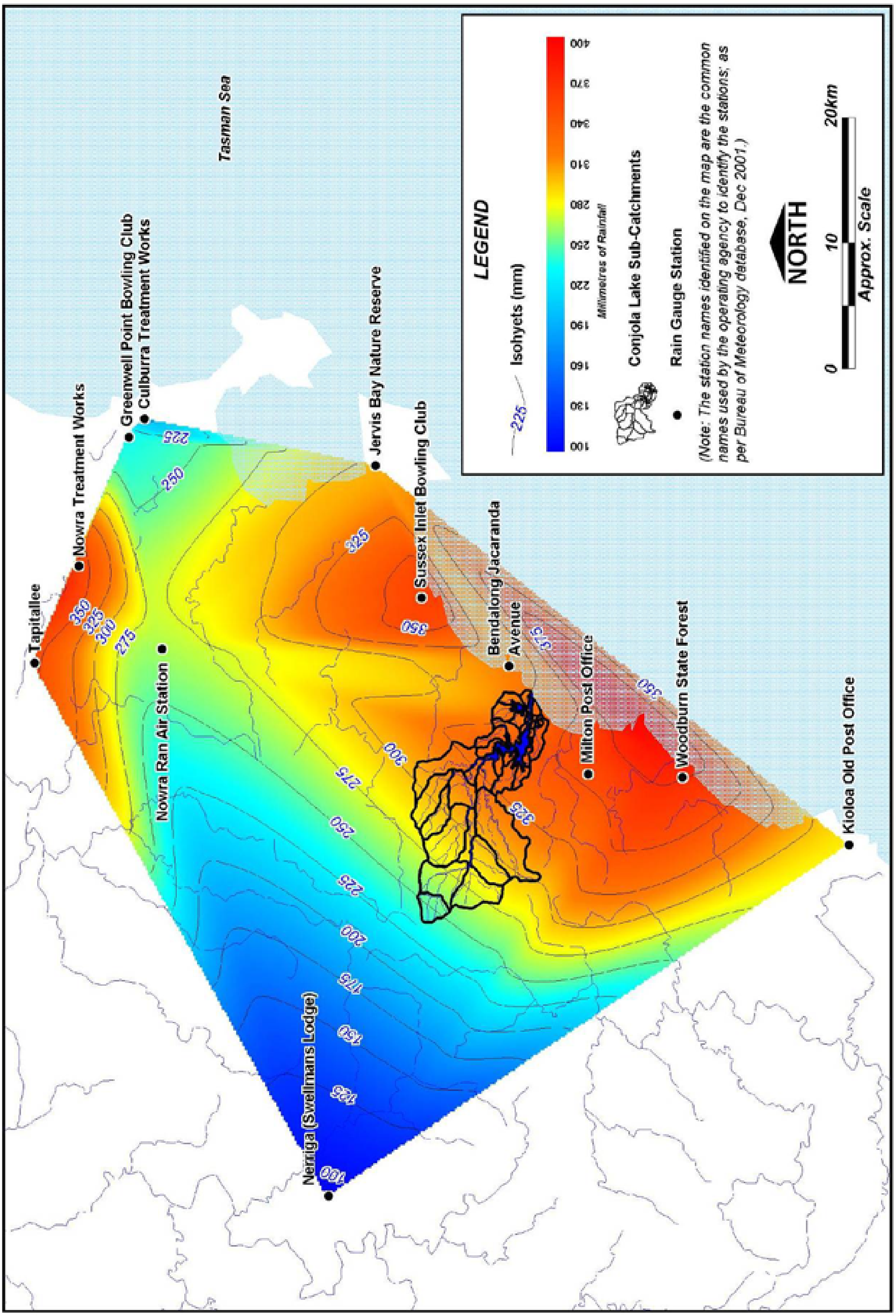

Figure 3-4 Isohyetal Map: 48 hours to 9am 11 March 1975 Isohyetal Map

3.3.3 The February 1992 Flood

3.3.3.1 Rainfall Temporal and Spatial Distribution

The rain gauges located around Conjola Lake catchment recorded some rainfall before the 10 February 1992, which resulted in saturating the catchment soils. During the event, the area received an average of 139mm and 66mm of rain in 2 successive days.

The isohyets of the 48-hour cumulative rainfall records of 10 and 11 February 1992 are presented in Figure 3-5. It can be seen from Figure 3-5 that the centre of the storm was located on the ocean front and probably affected the region south of Conjola Lake more than the immediate lake catchment.

3.3.3.2 Flood Description

The 1992 flood was not particularly significant in terms of total rainfall (being ranked as only 15th highest since 1971). However, the resident survey results provided a significant amount of information on the flood, with eight separate descriptions of the flood event.

Local residents indicated that flow was coming from Pattimores Lagoon. Some of the houses surveyed did not get flooded from the lake but rather by the surface runoff coming from the hill behind the houses. In particular, properties around Lake Conjola Entrance Road appear to have experienced this surface runoff flooding mechanism.

The 1992 flood event is considered as being the second most suitable calibration event after the 1971 flood. The amount of historical data and their quality also enables an assessment of the Pattimores Lagoon flooding mechanisms. The 1992 flood was used for calibration.

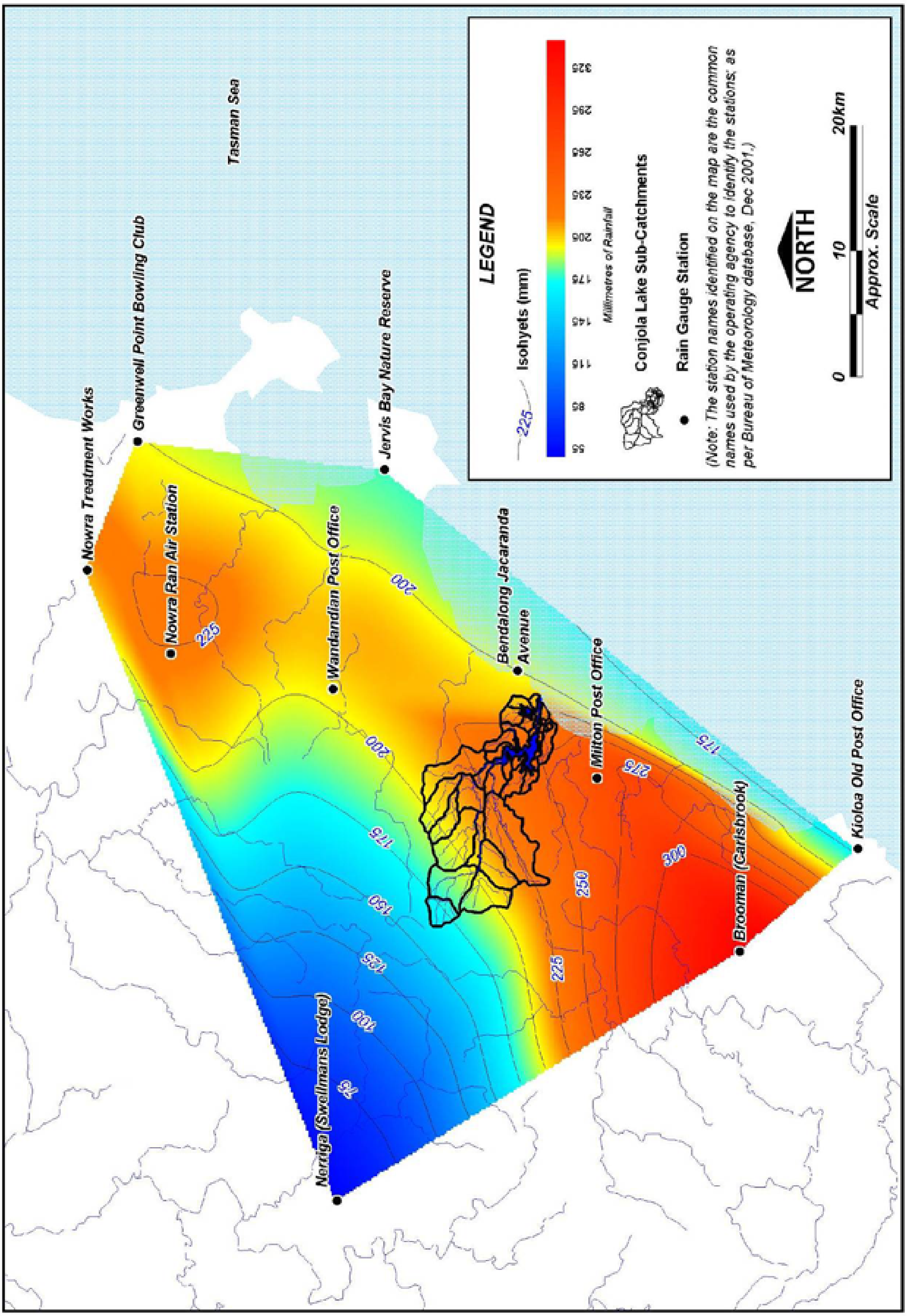

Figure 3-5 Isohyetal Map: 48 hours to 9am 11 February 1992 Isohyetal Map

4 COMPUTER MODELS

4.1 Introduction

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

- A square-cell rectilinear grid Digital Elevation Model of the whole catchment based on linear interpolation between topographic survey points;
- A hydrologic model, covering all the sub-catchments of the project area;
- A two-dimensional (2D) hydraulic model extending from “The Steps” to the ocean with additional 2D elements to represent the other residential areas of Conjola Park, Fisherman’s Paradise and Berringer Point, which are joined together with one-dimensional (1D) storage-based elements; and
- A geomorphologic model of the entrance channel.

The **Digital Elevation 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 hydraulic model.

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

The **geomorphologic model** updates the hydraulic model bed and bank geometry depending on calculated erosion and deposition rates at the entrance only.

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 ocean levels 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, lakes, floodplains, etc (see Section 4.1.1).

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

3 Setting up of hydrographic databases (rainfall, sea levels, flood levels) for historic events.

4 Calibration to one or more historic 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 historic 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.

4.1.1 Models Discretisation

Models discretisation is necessary to simplify the real-world into one that can be represented by discrete elements. The computer then solves equations at every discrete element to simulate the hydrologic, hydraulic 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 needs to take into account:

- location of available data (eg. creek/lake section surveys);
- location of recorded data (eg. recorded level sites);
- location of controlling features (eg. bridges, flow constrictions);
- required 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.

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 from the Reticulated Sewerage Survey Scheme EIS (Shoalhaven Water 1995);
- Hydrographic survey of the lake from the Department of Public Works and Services (1992-1993);
- Topographic survey collected for the study in June 2003, which includes:
  - Cross-sections of Conjola Creek upstream of Fisherman's Paradise;
  - Cross-sections of channel and structures along the canal linking Pattimores Lagoon with Conjola Lake;
- Historic flood descriptions collected through resident survey;
- Rainfall data for historic events from the Bureau of Meteorology;
- Tasman Sea tide levels;
- Tasman Sea levels from waverider buoys for the February 1992 flood; and
- Flood level data for historic events collected through resident survey.

4.3 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 dependent 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 lakes. 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; and
- 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.

4.3.1 Model Setup

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

The Conjola Lake catchment, with a drainage area of about 145 km^2, has been discretised into 34 subcatchments, which feed into Conjola Lake and the associated floodplains at 12 different locations. The average sub-catchment surface area is 4.25 km^2, with a standard deviation of 4.7 km^2. The biggest sub-catchment has an area of 21.9 km^2. The surface of the lake itself accounts for 6.1 km^2 of the total catchment area. Smaller subcatchments are more likely to receive uniform rainfall distributions.

The sub-catchment limits have been defined so that the descriptive parameters are generally uniform within the boundaries (which are slope, landuse, permeability, roughness). The topographic maps provided the data to calculate the subcatchments' slopes and to determine landuse characteristics. The slopes vary from 0.1% (adopted for the lake) to 18% (average 5.9%, standard deviation 6.9%). The vegetation cover is mostly forested for the entire catchment (apart from the urbanised areas). The hydrologic model sub-catchments are shown in Figure 4-1.

The rainfall intensity for each of the Conjola Lake subcatchments has been calculated for the 1971 flood. A spatial variability in rainfall intensity is observed within the subcatchments, ranging from +/-1% to +/-14% of the maximum rainfall intensity (average +/-5%, standard deviation +/-4%). The variability falls within the boundaries of the model accuracy. In the case of the 1% AEP design flood, the spatial variability in rainfall intensity has been estimated to a maximum of +/-9% over the whole Conjola Lake catchment. The spatial rainfall variability for design events is almost insignificant at the scale of the model.

The schematic of the model network is shown in Figure 4-1.

4.4 Digital Elevation Model (DEM)

A Digital Elevation Model (DEM) is a three-dimensional (3D) representation of the ground surface. The hydraulic model requires topographic information for each 2D cell. Given that ground levels are required for over 54,000 individual elements (grid cell corners and centre) within the hydraulic model (based on the grid cell size adopted for the design flood modelling), an automated process of assigning ground levels is required. Information is provided to the hydraulic model by interrogating a geo-referenced electronic surface of ground elevation (i.e. a DEM).

Available topographic information was processed by the DEM to create a Triangular Irregular Network (TIN), which is a series of interconnected triangles with vertices based on surveyed data points. The DEM is then derived from the TIN using linear interpolation of the planar surface that is defined by the three vertices. All the planar surfaces are merged into one overall surface, which is then discretised and reformatted in small square cells, each of constant elevation.

The Conjola Lake DEM is based on a 5m grid cell, and is composed of about 2.5 million cells, each assigned an independent ground level.

The Conjola Lake DEM was created from five different survey data sources:

- Bathymetric data from the PBP (1999) RMA model, based on the DPWS's 1993 bathymetric survey of the lake;
- Ground survey of residential areas undertaken in 1995 for the Reticulated Sewerage Survey Scheme EIS (Shoalhaven Water, 1995);
- Land & Information Centre (LIC) 2m contour data, as shown on orthophoto maps, to define bank levels in the forested national park areas;

- Land & Information Centre (LIC) 10m contour data, as shown on topographic maps, to define higher ground where data was missing; and
- Ground survey of creeks and structures undertaken in June 2003 as part of this Flood Study.

Of the above data sources, the 1995 ground survey represents the most comprehensive dataset. It was undertaken in each of the residential areas, providing a valuable support for the accuracy of the flood model outputs.

The PBP (1999) RMA model data was used as the basis for model bathymetry within the sub-tidal sections of the catchment. The RMA model data was based on a DEM generated from a cross-sectional survey of the lake by DPWS in 1993. Bathymetric levels were extracted and provided electronically at each RMA model node location. Although it would have been preferable to generate a new DEM directly from the cross-section points, checks were carried out to verify that the original cross-section geometries had not been altered in the process, thus ensuring the quality of the DEM generated from extracted levels from the RMA model.

The LIC data is the least accurate data source, with a tolerance of +/- 1-2 metres. This data source was only used to define the overbank part of the deep lake cross-sections (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%).

Figure 4-2 presents a 3D shaded view of the Conjola Lake area produced from the square-cell rectilinear grid DEM. Changes in lake bed level colours clearly show the difference in depths between the main lake to the west of “The Steps” and the entrance channel.

Figure 4-3 shows the extent of the two main sources of survey data used for the creation of the hydraulic model DEM: the RMA bathymetric data (yellow points) and the Sewerage Scheme EIS ground survey (black lines). The LIC information was used for the remaining sections of the study area.

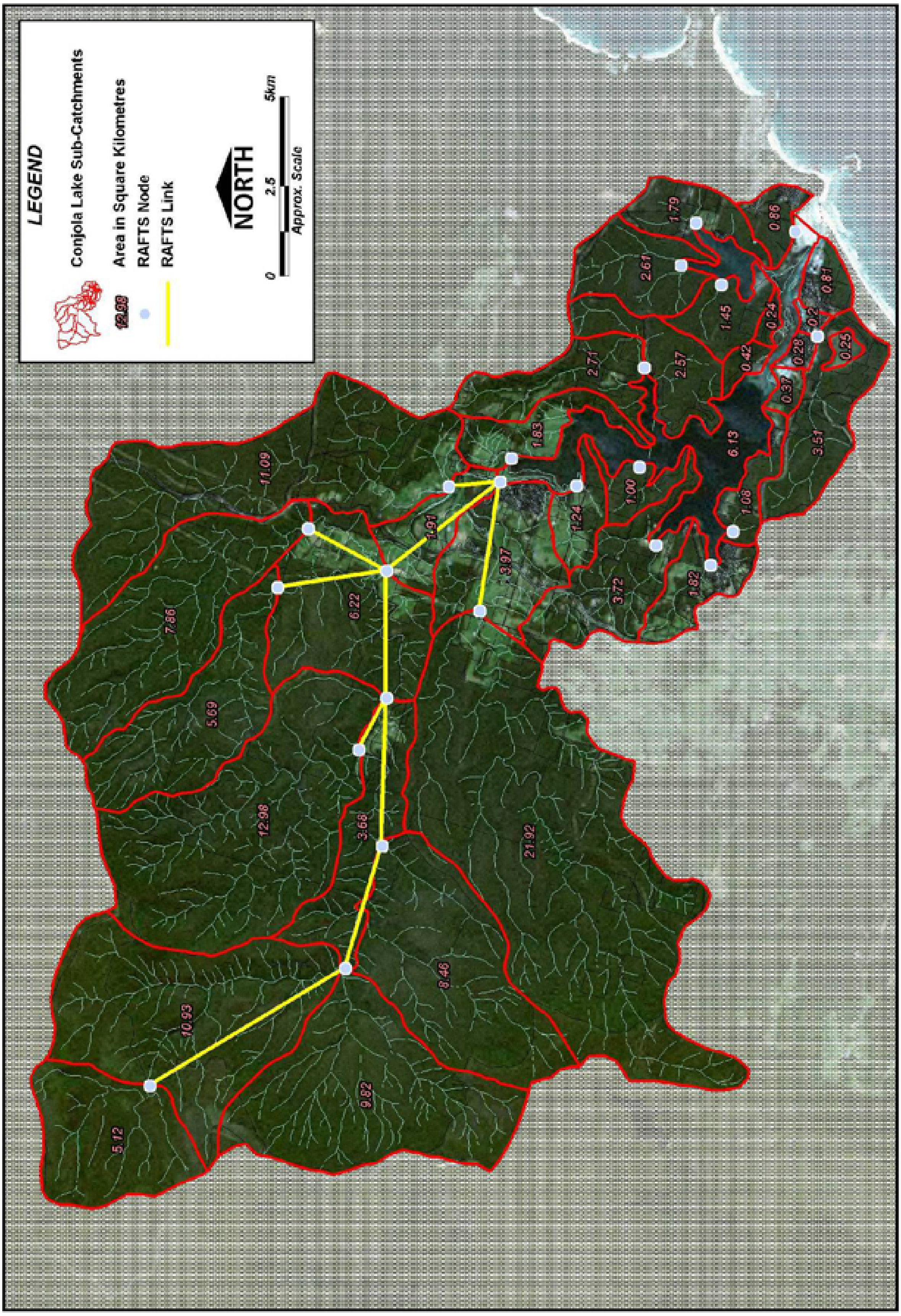

Figure 4-1 RAFTS-XP Conjola Lake Catchment Model Network

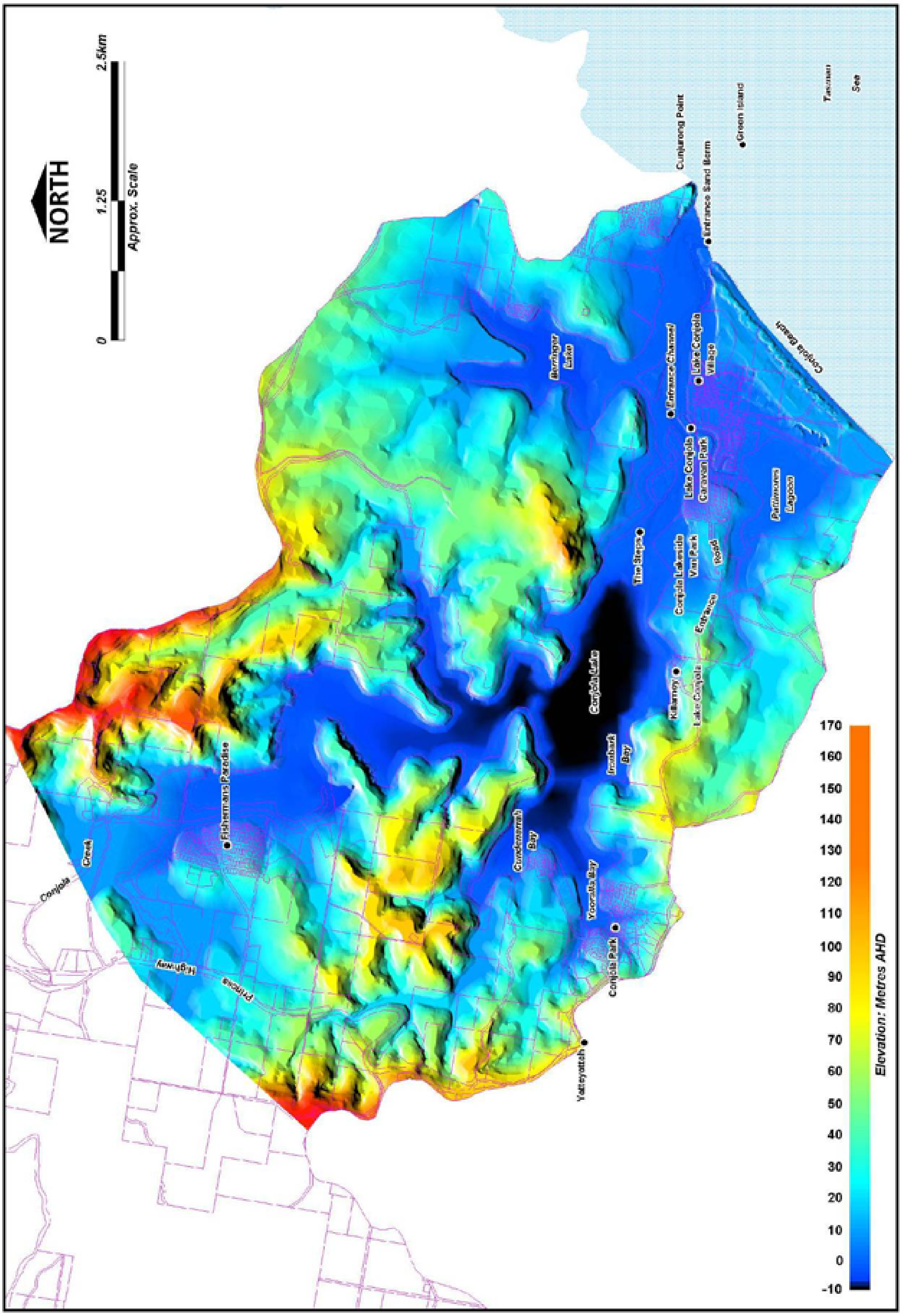

Figure 4-2 3D Shaded View of DEM Ground Levels of Conjola Lake Catchment

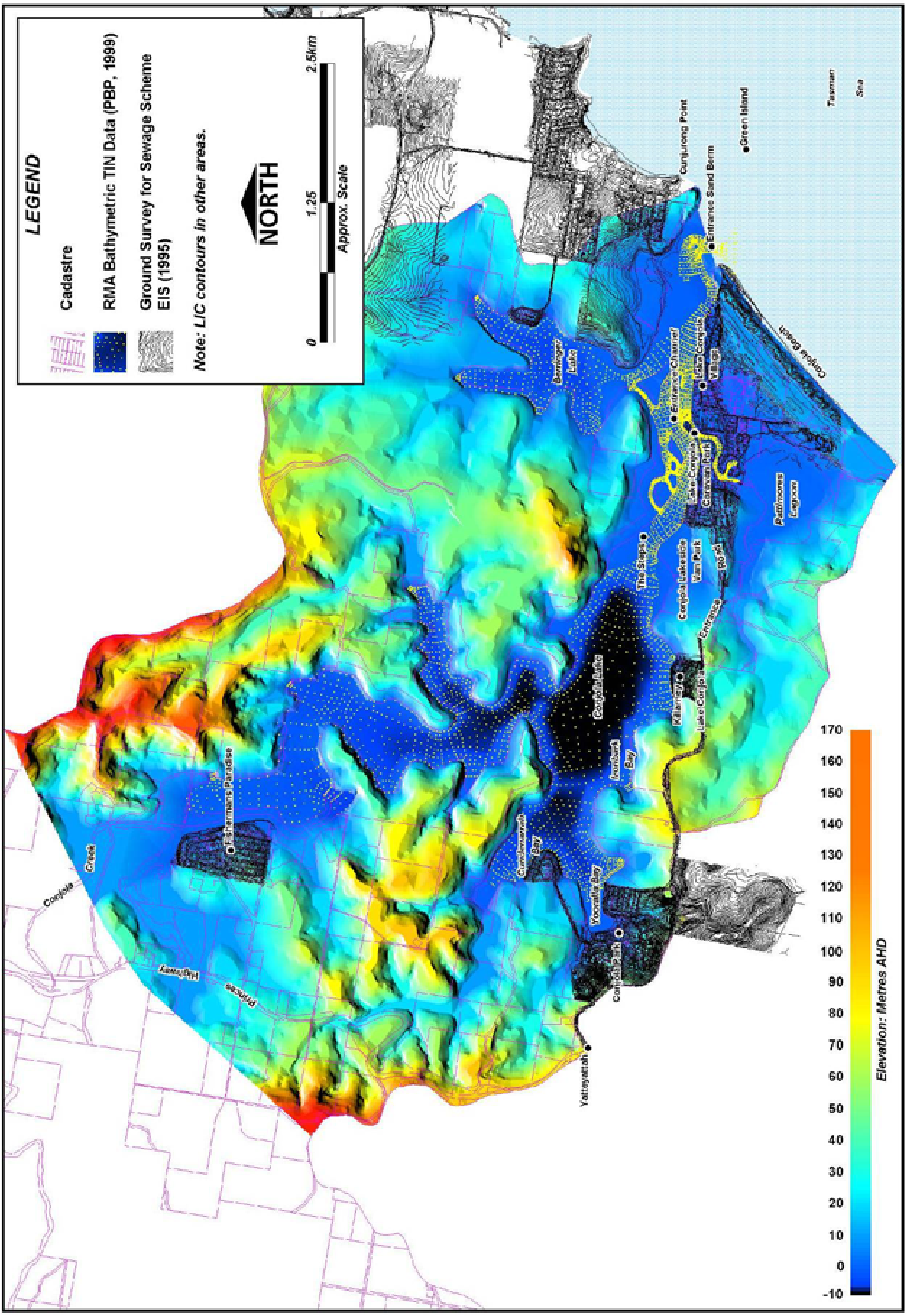

Figure 4-3 Extents of Different Data Sources that made up DEM

4.4.1 Historical Changes to the DEM

Structures like roads, railway lines, bridges, culverts, embankments, and levees, can change over time. As topographic and some structure details are incorporated directly into the DEM, separate DEMs are required to represent the specific years in which events occur (associated with historical calibration and verification events).

Information was gathered on changes to topographic and structure details over the past 40 years. Apart from the entrance channel, no significant changes to the lake and its floodplain were identified that would have an impact on flood behaviour.

Within the entrance channel, only the downstream end has changed over the years. This section of the entrance channel, which is also referred to as the tidal delta (or flood tide delta) in other reports, is subject to constant shoaling and erosion processes associated with coastal storm and catchment runoff/flood conditions. A catalogue of 19 aerial photographs were presented and discussed in PBP (1999), which describe the morphologic changes to the entrance over the past 60 years or so.

In terms of model calibration, separate DEMs were generated for the lake based on entrance conditions presented in relevant air photos, as follows:

- March 1967 for the February 1971 flood;
- June 1972 for the March 1975 flood; and
- November 1991 for the February 1992 flood.

The historical photographs were compared with the latest aerial photograph representing the most accurately surveyed bathymetry. Comparisons between shoal locations and apparent water depths, from fixed points like rock formations, led to the estimates of the historical entrance geometries. Figure 4-4 presents the historical aerial photographs and their associated DEMs for the three calibration and verification events. By comparison, the DEM in Figure 4-2 represents present day conditions.

For the modelling of the design floods, two more DEMs were constructed, with the entrance geometries reflecting the two “managed” options adopted by Council and as described in MHL (2003). The two design “managed” entrance are presented in Section 6.

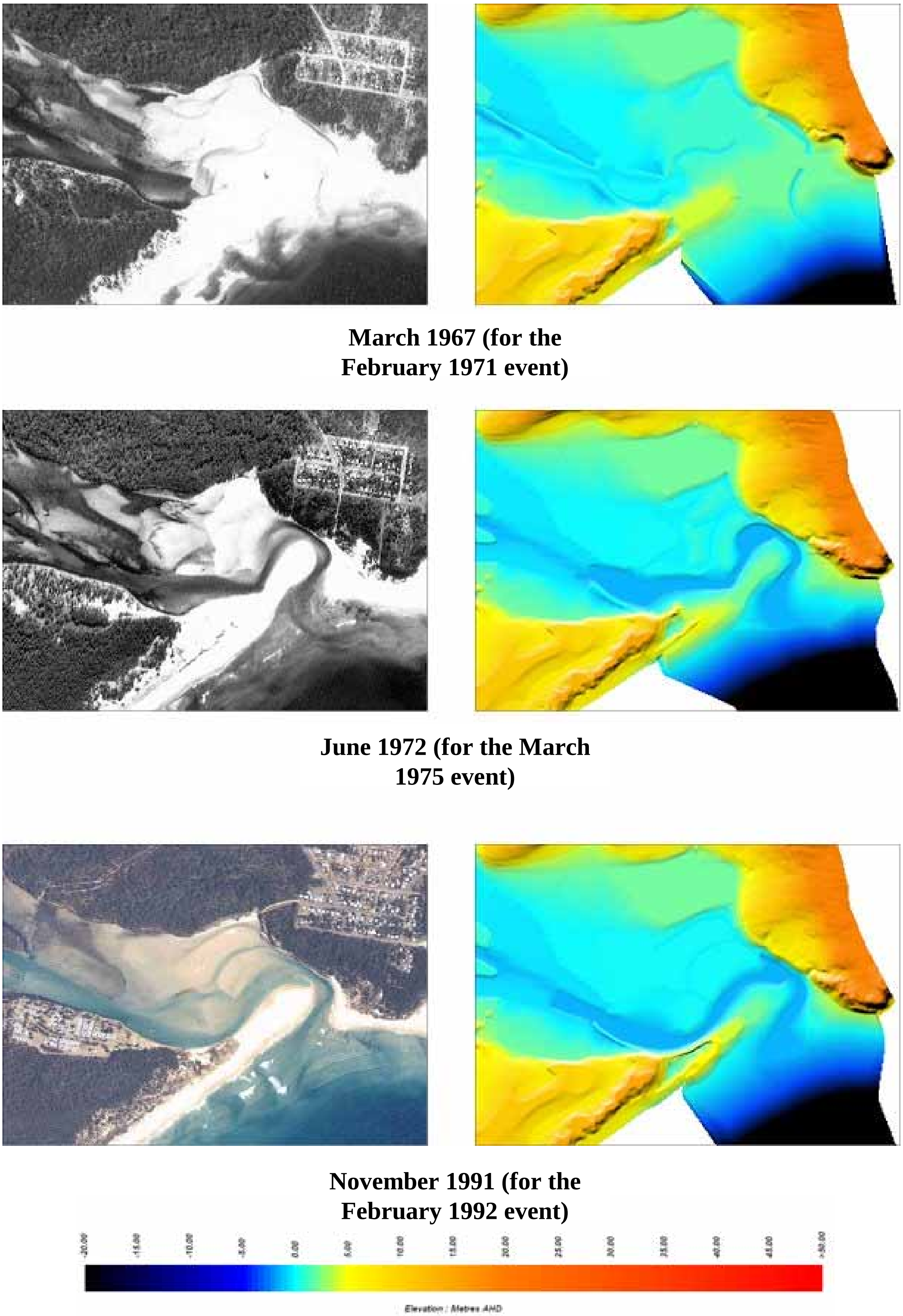

Figure 4-4 Aerial Photographs and adopted DEMs for Calibration Events

4.4.2 Accuracy of the DEM

The accuracy of a two-dimensional model is largely dependent on the vertical accuracy of the associated DEM, as it is the topography of the ground that largely controls flow behaviour during times of flood.

The ground survey carried out for the Sewerage Scheme EIS (Shoalhaven Water, 1995) is likely to have an accuracy of about 0.1 metres.

The lake bathymetric survey undertaken in 1993 is likely to have a high level of accuracy. However currents, floods and sedimentation may have modified the bathymetric geometry since that time. The Lake Conjola Estuary Processes Study (GHD, 1996) reported that the catchment does not produce much sediment, thus changes due to sedimentation would be minor. As a consequence, it is estimated that the bathymetric survey has an accuracy of about 0.1 to 0.4 metres depending on the location in the lake. This accuracy does not apply to the entrance channel however. 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 known to have an accuracy of about +/- 2 metres.

The accuracy of the DEM generated at the entrance channel for historical events is expected to be in the order of 0.5 to 1 metre as ground and bed levels were estimated from aerial photographs without field measurement of ground control points. Further, the time between the date of the photography and the calibration flood event is almost four years in the case of the 1971 flood. Given the dynamic nature of the entrance channel, it is highly likely that the entrance conditions for the 1971 event were somewhat different to those represented in the 1967 air photo. The influence of accuracy of the entrance channel DEM for historical events is discussed in Section 5.4.2.

For probabilistic design flood events, the adopted entrance channel geometry is based on the extreme entrance conditions permitted by Council's entrance management plan.

4.5 Hydraulic Model

4.5.1 Model Setup

The hydraulic model simulates the dynamic flooding behaviour in Conjola Lake, including the interactions between the main part of the lake, the entrance channel and the floodplains.

The modelling software, TUFLOW, was used to develop a 2D/1D hydraulic model of the study area. The model is a mixture of one-dimensional (1D) and two-dimensional (2D) domains with the 2D domains covering the key areas of existing and future management interest. 2D domains produce a significantly higher order of resolution in terms of hydraulic 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).

2D domains were established in the following areas:

- Fishermans Paradise;
- Cundenarrah Bay;

- Yooralla Bay;
- Ironbark Bay;
- Berringer Lake;
- Entrance channel; and
- Pattimores Lagoon.

The hydraulic model network and its relevant branches are provided in Figure 4-5.

As TUFLOW is a finite difference model, the Conjola Lake Flood Model has been constructed using elements with a regular grid of size 10m x 10m. This means that hydraulic parameters are calculated separately for every 10m square of the 5.4km^2 study area represented in 2D. Over 54,000 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 DEM (see Section 4.4).

The two-dimensional TUFLOW model domains are also dynamically linked to a one-dimensional model, representing the deep part of the lake that separates the specific areas being modelled in 2D. The hydraulic behaviour of the deeper sections of the lake is unidirectional and can be accurately represented by 1D elements. The extent of the 2D model domains and the locations of the one-dimensional elements are provided in Figure 4-5.

The selection of the computational timestep is critical to the success of the model. The simulation run time is directly proportional to the number of timesteps required to calculate model behaviour for the required time period. A too long timestep may over-simplify the rapidly changing response of the system and result in model instabilities, while a too short timestep may result in unnecessary computational demand. The optimum timestep can generally be determined by calculation of the Courant stability criterion (C_r). For a typical 2D scheme, the Courant criterion generally needs to be less than 10 and is typically around 5 for most real-world applications (Syme 1991). The computation timestep for the hydraulic model was therefore defined in accordance with this criterion, as given in the equation below:

$$C_r = \frac{\Delta t \sqrt{2gH}}{\Delta x} \qquad \text{2-D Square Grid} \quad (1)$$

$$\begin{aligned}
&where \\
&\Delta t = timestep, s \\
&\Delta x = length\ of\ model\ element, m \\
&g = acceleration\ due\ to\ gravity, ms^{-2} \\
&H = depth\ of\ water, m
\end{aligned}$$

The adopted computational timestep used for the TUFLOW model is two (2) seconds. This means that the hydraulics within each of the 54,000 model elements is recalculated for each two second time step throughout the flood event. For a 24 hour flood simulation, this equates to nearly 930 million calculations.

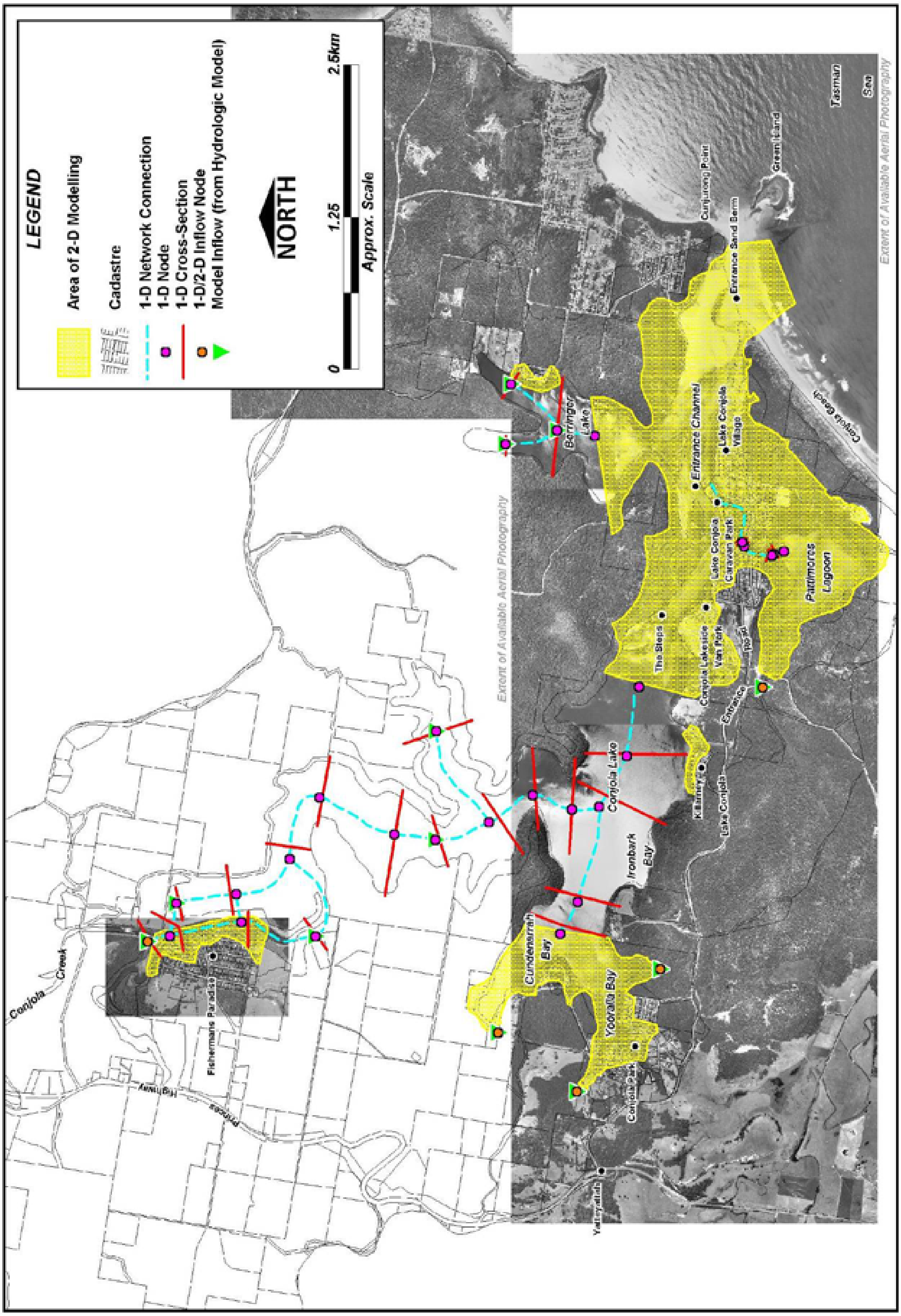

Figure 4-5 Conjola Lake Hydraulic Model Layout

BMT WBM

Information on the TUFLOW software is presented further in Appendix C, while additional details and examples of applications can be found at www.tuflow.com

4.5.2 Model Inputs

Inputs to a hydraulic model include:

- **Topography / bathymetry** of the waterway and the floodplain, based on the DEM and creek cross-sections. The DEM is discussed in Section 4.4;
- **Hydraulic roughness** of the channel bed and the floodplain land. The final set of hydraulic roughness values were determined during the calibration process; these did not require modification for the design as ground cover (hydraulic roughness) conditions have not changed from the calibration and verification events;
- **Hydraulic structures** along the canal linking Pattimores Lagoon to Conjola Lake;
- **Inflows:** the rainfall runoff calculated by the hydrologic model at nodes adjacent to the lake and over the lake body (see Section 4.3). The runoff hydrographs are introduced into the hydraulic model at 12 different locations around Conjola Lake. The location of the model inflows are presented in Figure 4-5; and
- **Boundary water levels:** depending on the entrance channel geometry, the sea levels can influence water levels in the lake. The downstream sea condition takes into account all dominant sea level parameters (astronomical tide, storm surge and wave setup).

A range of sensitivity tests and checks on the hydraulic model were carried out during the course of the model calibration. The checks carried out confirm the correct interpretation of the input data and sensitivity tests were carried out to develop an understanding for the most influential hydraulic parameters. Examples of tests and checks carried out are:

- Checks for any irregularities in the conveyance values;
- Influence of the downstream sand berm; and
- Mass balance checks.

4.5.3 Model Outputs

Model outputs are flood levels, flows, and velocities describing the flood behaviour over time for a given flood event. Based on these outputs, hydraulic categories and preliminary flood hazards (based on depth and velocity only), were determined.

Individual model outputs are provided for every 2D element within the model. This means that for the Conjola Lake model, results at over 54,000 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 presentation of the spatially-dependent results.

4.6 Geomorphologic Model

This Section describes the technical details of the geomorphologic model, and relies on the reader having a basic level of understanding of sediment transport processes.

4.6.1 Introduction

The Conjola Lake entrance is located at the northern end of Conjola Beach. Marine sand is transported laterally along the beach in a northerly direction, before being deposited inside the entrance channel. Marine sand can also be deposited within the entrance by onshore transport of incipient nearshore sand bars (particularly following entrance scour events).

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 to the entrance during a flood is therefore critical to the flood study.

In order to integrate sediment transport into the flood study process, a geomorphologic module was added to the hydrodynamic software, based on the Van Rijn (1990) theory of sediment transport.

The Van Rijn formulation is generally accepted as being the latest and most accurate method for estimating sand transport. It does, however, contain approximations related to the little understood complexity of sand transport. Although these approximations are unavoidable, the Van Rijn method is still appropriate to combine with the 2D (depth-averaged) TUFLOW hydraulic routines, as it determines the bed shear stress from the near-bed shear velocity, calculated using conventional theory.

4.6.2 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 wave orbital motion 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.

4.6.3 Wave/Current Relationships

Comprehensive formulae that incorporate the independent influences of currents, waves and sand properties have been developed and verified by a range of researchers. Methods such as Van Rijn (1990) are the most comprehensive as they incorporate the fundamental processes of:

- Bed shear stress due to the action of the currents at the bed, providing for both the shear velocity at the near-bed boundary layer for both waves and current, together with the influence of bed form (ripples and dunes) on the shear stress;

- A critical bed shear stress below which no sediment motion occurs;
- The near-bed sediment concentration (at the designated reference height), transported at the near-bed shear velocity as bed load; and
- The vertical distribution of both the current speed and the suspended sediment concentration, combining to provide the vertical distribution of the suspended load sediment transport in the water column.

The Van Rijn formulation is presented in Appendix C.

4.6.4 Slope Stability

The geomorphologic model applies the erosion and the deposition equations to wet cells only. In reality, with increasing erosion at the boundary between a sand channel and its sandy banks, the bank slope becomes increasingly steep until reaching the threshold of stability. The dry bank then collapses into the channel, with the quantity of collapsed sand added to the sand concentration in the water column (modifying the rate of vertical erosion in the surrounding area), and widening the channel cross-section.

This phenomenon is integrated into TUFLOW through a control on the slope between wet and dry cells within the geomorphologic model extent. Should the slope steepen above a stability threshold (manual input in the TUFLOW model file), TUFLOW reduces the dry cell ground elevation to maintain the stability slope. The volume of sand removed from the dry cell is added to the sand concentration in the wet cell.

This computation is performed until the erosion stops in the channel or the initial dry cell becomes wet (due to falling elevation). In the later case, the slope stability test is moved to the next wet/dry cell boundary.

4.6.5 Geomorphologic Modelling Extent

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

The geomorphologic model is laid over a section of the TUFLOW hydrodynamic mesh. The area selected to incorporate sediment transport calculations and the variable bed feature is centred on the sand berm. It covers an area of 0.21 km^2 for the calibration events and 1.8 km^2 for the design flood events. The geomorphologic modelling area ensures that it extends laterally into permanently dry land, downstream into deep water (20m depth) to allow for deposition of material eroded from the entrance berm, and upstream to the Lake Conjola Entrance Caravan Park, where the flows start to accelerate due to the entrance channel constriction (coinciding with where the upstream bed erosion does not impact on adjacent flood levels).

Due to computational demands of the geomorphologic module, the extents of the model coverage have been kept to a minimum. During the calibration events, the erosion of the sand berm was restricted to a limited area, however, for the design floods, the area of sediment transport was much larger (due to the assumed open entrance conditions). Consequently, the geomorphologic model extent was set wider for the design events in order to conservatively include all potential critical erosion areas throughout the entrance channel.

4.6.6 Geomorphologic Model Input

Inputs to the geomorphologic model include:

- D50: 0.25 mm;
- D90: 0.5 m;
- Fall Velocity: 0.015 m/s;
- Sand Grain Density: 2650 kg/m^3;
- Water Density: 1035 kg/m^3; and
- Wet/Dry cell boundary slope stability: 5m (horizontal): 1m (vertical).

5 MODEL CALIBRATION AND VERIFICATION TO HISTORIC FLOODS

As outlined in Section 5.1, a conventional calibration of both the hydrologic and hydraulic models for the Lake Conjola Flood Study was not possible due to a lack of critical historical data.

Nonetheless, a series of tests and validations have been carried out as part of a 'calibration process' to demonstrate the suitability of the model for future design simulations. The validation tests have focused on replicating historical flood information as discussed previously in Section 3.

5.1 Theoretical Basis for Conventional Calibration

The hydrologic and hydraulic models are mathematical tools based on a series of theoretical and empirical equations. The models respond to a multiplicity of 'input' combinations that represent a range of environmental conditions being simulated by the model. Conventional calibration is based on using known outputs (eg historical flood marks) to back-calculate the appropriate input parameters that generate such model results. For typical flood studies, most input parameters are known (green boxes in Figure 5-1), and there is only one unknown input parameter (red boxes in Figure 5-1). As such the model equations can be used to determine the one unknown (which is typically the catchment details (e.g. rainfall losses, lag time, storage coefficient) for the hydrologic model, and the bed roughness for the hydraulic model).

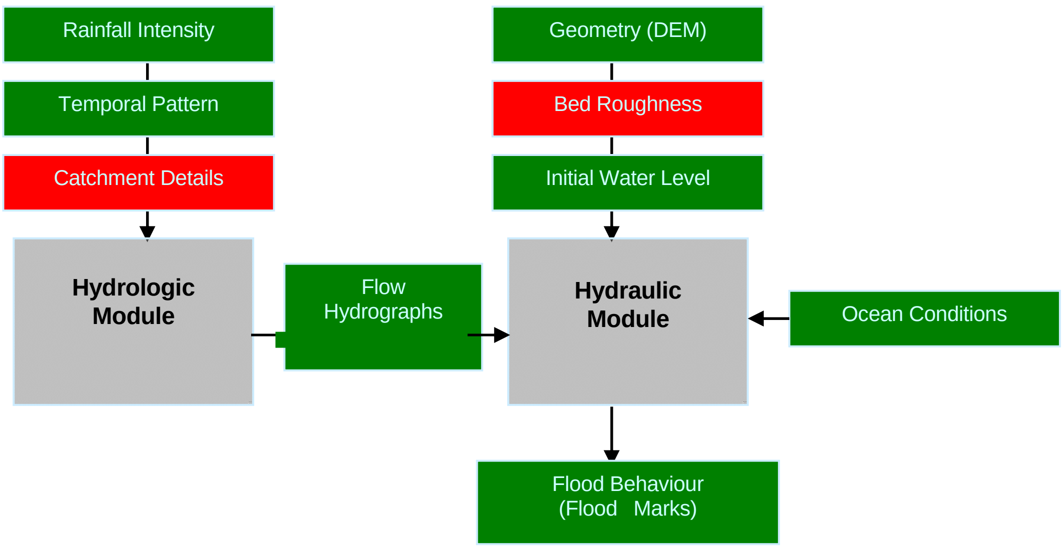

Figure 5-1 Conventional Calibration Process

Unfortunately, conventional calibration of the Conjola Lake models was not possible as more than one input was unknown, or lacked effective accuracy to be suitable for calibration (green and red hatched boxes in Figure 5-2). Additional unknown inputs for the models were:

- For the hydrologic model:
    1. Rainfall temporal pattern; and
    2. Gauged flow hydrographs.
- For the hydraulic model:

3 Entrance channel topography and bathymetry;

4 Initial lake water levels prior to the rainfall event;

5 Inflow hydrographs; and

6 Ocean conditions.

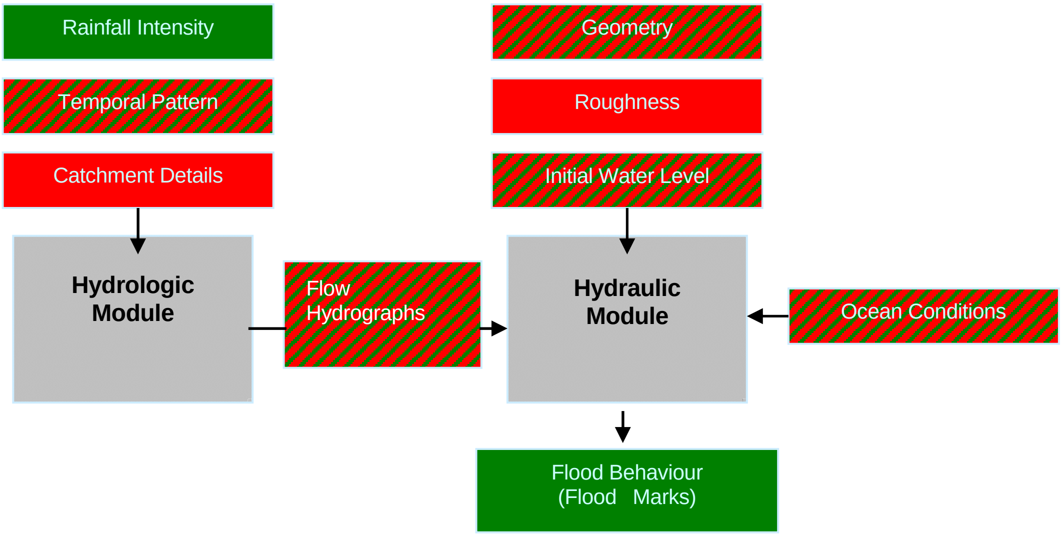

Figure 5-2 Conjola Lake Calibration Process

To fulfil the conventional calibration approach, the additional unknown parameters needed to be fixed, otherwise the models would be unsolvable (i.e. too many unknowns within the equations). In doing so, however, it is recognised that the calibration is based on a series of 'assumed' input parameters, which in essence negates the outcomes of the calibration. However, without any reasonable alternative calibration approach, considerable care was given in assigning model assumptions, ensuring that adopted values were practical and justifiable (see Section 5.2 and 5.4 for further details).

Reasonable assumptions were made for rainfall temporal pattern, entrance channel geometry, initial water level and sea levels based on historical records in and around the catchment and best engineering judgement. To support and justify these assumptions, the models were also tested for sensitivity to the assumed input values. Parameters that were relatively insensitive to the model results were regarded as having little influence on the model calibration process or the outcomes for future design simulations. On the other hand, parameters that were relatively sensitive to model results meant that the 'assumed' values were critical to the calibration process and results of future design simulations.

The sensitivity analysis provided valuable information that, when combined with the expertise of a flood engineer, can confirm whether the results produced by the model are realistic. In this case, the model can be considered to be validated to theoretical tests, rather than calibrated to actual historical flood events.

Sections 5.2 and 5.4 present the outcomes of sensitivity tests and assessments of various model inputs, leading to an overall validation of the Conjola Lake flood models.

5.2 Justification of Hydrologic Model Assumptions

The absence of stream gauging stations in the Conjola Lake catchment prevented a conventional calibration of the hydrologic model. To overcome this problem, default values are usually adopted for the hydrologic model and the calibration of the model is carried out in tandem with the hydraulic model. Modifications to the default parameters used in the hydrologic model are only considered if calibration of the hydraulic model cannot be achieved using hydraulic parameter values within a 'normal' range. In the case of Conjola Lake, the lack of information associated with other inputs to the hydraulic model prevented justification for modifying the default parameters used in the hydrologic model.

Default values from the RAFTS-XP Manual were applied in the hydrologic model for the following parameters:

- Catchment area: as measured;
- Catchment slope: as measured (highest catchment elevation minus the catchment outlet elevation, divided by the flowpath length between the two points);
- Catchment imperviousness ratio: as estimated from aerial photographs and maps;
- Catchment Roughness (ranging from 0.015 for urban impervious to 0.05 for rural pasture, to 0.1 for forested areas);
- Default storage B coefficient (B=1);
- Initial losses (0mm for wet antecedent conditions for historical events, 10mm for dry antecedent conditions for historical events and 15mm for design events); and
- Continuous losses (0.0 mm/hr for wet antecedent conditions for historical events, 1.0 mm/hr for dry antecedent conditions for historical events and 1.5 mm/hr for design conditions).

A comparison between the RAFTS-XP model prediction and the rational method for the 100 year ARI design peak flow was undertaken to confirm the validity of adopted parameters in the hydrologic model. The comparison showed that there was only 8% difference in 100 year ARI design peak flows between the hydrologic model and the estimates using the Rational Method. This result is considered to be within the acceptable range of differences between the two methods. Although not considered to be 'calibrated', the results provide confidence that the RAFTS-XP hydrologic model of the Conjola Lake catchment is reasonable and appropriate for use in simulating design flood flow conditions. Details of the rational method analysis are presented in Appendix D.

5.3 Predicted Historical Flood Hydrographs

Adopting the standard parameters presented in Section 5.2, the hydrologic model was used to predict historical flood hydrographs for the calibration events as defined previously in Section 3.3.

The following hydrologic model inputs were required when predicting historical flood hydrographs:

- <u>Historical rainfall intensities</u>. The isohyetal maps presented in Section 3.3 provide interpolated rainfall totals over periods of only 24 hours or 48 hours. However, the real duration of the calibration rainfall events is actually unknown. From statistical analysis of the recorded rainfalls (see Table 3-1) with the local design intensities, coupled with monthly record investigations, it was estimated that the main rainfall burst of the calibration events lasted approximately:

- ➢ 48 hours for the February 1971 flood;
- ➢ 36 hours for the March 1975 flood; and
- ➢ 36 hours for the February 1992 flood.

- Rainfall temporal pattern. Only the Nowra RAN Air Station hyetograph is located near the Conjola Lake catchment, approximately 35km north of the project area. The isohyetal maps (Section 3.3) show that Nowra RAN Air Station seems to have been less affected than surrounding rain gauges within the Conjola Lake catchment during the 1971 and 1975 storms. It highlights the fact that the Nowra RAN Air Station rainfall hydrograph might not be representative of conditions at Conjola Lake, and thus might be inadequate for use in the calibration process.

It was possible to test the influence / sensitivity of the adopted rainfall temporal patterns. In the case of the 1971 flood event, the Bendalong rain gauge recorded daily precipitation from 3 February until 11 February 1971, with the peak of the rainfall occurring on 6 and 7 February. The following temporal patterns were used in the model to distribute the measured rainfall totals:

- Nowra RAN Air Station recorded 6 minute rainfall pattern; and
- AR&R design regional temporal pattern for durations of 30hrs, 36hrs and 48hrs.

The rainfall hyetograph from the four methods are presented in Figure 5-3. In all cases, the pattern has been applied to the total of the 6 and 7 February records.

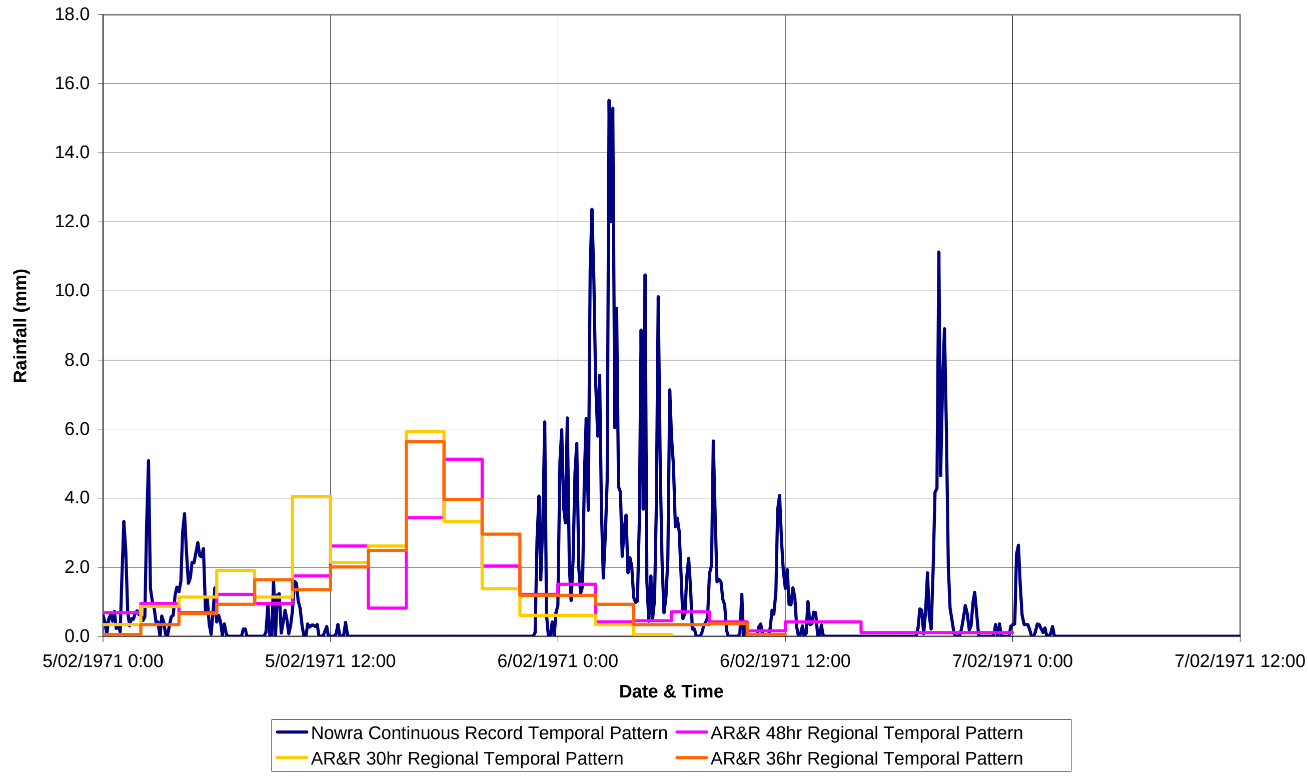

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Figure 5-3 1971 Calibration – Possible Rainfall Temporal Patterns

All temporal distribution methods considered contain inherent uncertainties as follows:

- Based on the isohyetal maps, it is expected that the weather pattern that caused the 1971 flood in Conjola Lake was missed by Nowra RAN Air Station. Therefore the temporal pattern is expected to be unrepresentative of rainfall over Conjola Lake; and
- AR&R design regional temporal patterns are probabilistic, and do not reflect realistic events.

When simulating each of the different rainfall temporal patterns in the hydrologic model, the results showed a wide range of possible peak flows at Conjola Creek outlet (see Figure 5-4). The 48 hour AR&R storm patterns leads to a peak flow of approximately 800 m^3/s, while the Nowra RAN Air pattern leads to a peak flow of about 1200 m^3/s.

However, the following comments and caveats are made on the results:

- The Nowra RAN Air Station did not record any rainfall between 05/02/71 13:00 and 05/02/71 23:00. The same station recorded 50mm of rain less than all the surrounding stations. It is possible that it rained at Conjola Lake despite being dry at Nowra. If so, the rainfall peak over the Conjola Lake catchment would be less than the one modelled, and consequently the peak runoff flow would be similarly less. It is therefore expected that adopting the Nowra RAN Air Station temporal pattern for the calibration event over-estimates the runoff flow; and
- The nature of the AR&R temporal patterns smooths the temporal variations. More intense rain bursts at the height of the flood are generally responsible for dramatically increasing the peak flows. The AR&R temporal pattern does not display such bursts of rainfall. Consequently, it can be expected that the AR&R temporal pattern under-estimates the peak runoff flow.

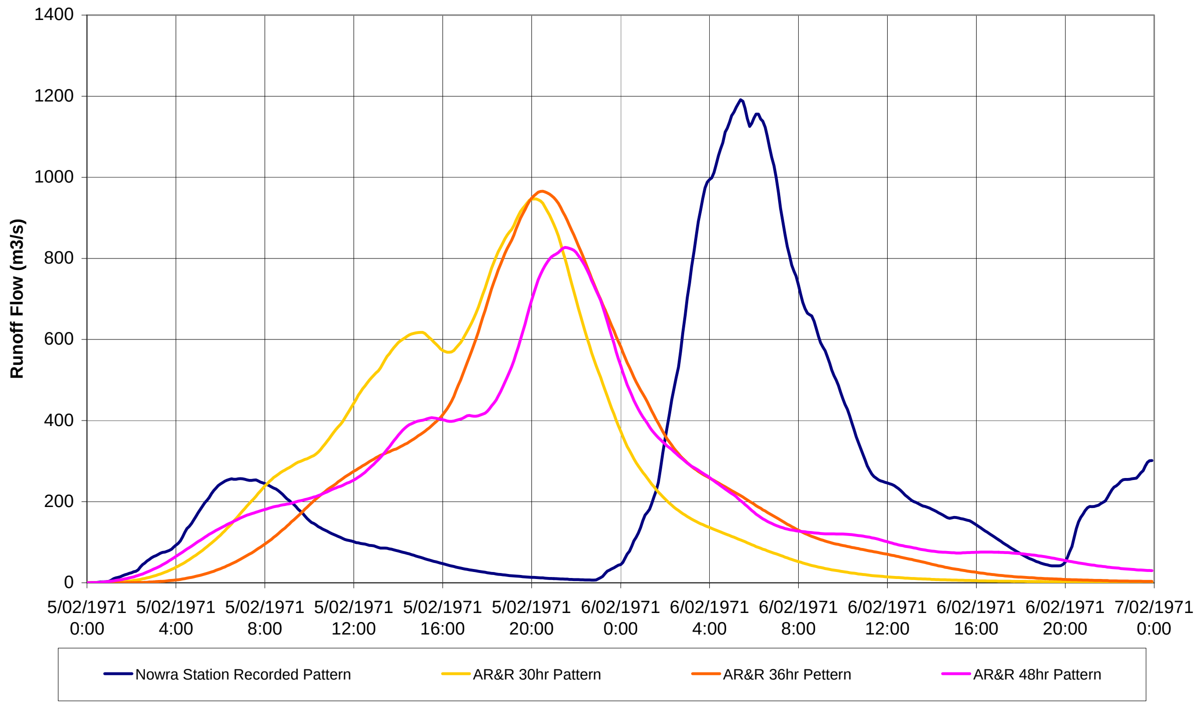

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Figure 5-4 1971 Calibration – Possible Flood Flows at Conjola Creek Outlet

There is no absolute basis that could justify the adoption of one temporal pattern over the other. As only one pattern and its derived flow hydrograph could be used in the hydraulic model calibration attempt, it was decided that the 36hr temporal pattern, which results in the most average inflow hydrograph at Conjola Creek outlet (with peak flow value of 1,000 m^3/s for the February 1971 flood), would be most appropriate for subsequent hydraulic model analyses. It is recognised that this adopted temporal pattern is based on an 'unrealistic' AR&R probabilistic approach, however, the degree of uncertainties relating to other historical data inputs means that the consequences of the assumed temporal pattern are relatively minor. It is also recognised that any alternative temporal pattern which would lead to inflow hydrographs with a peak flow value of +/– 20% would be equally acceptable.

A similar process was adopted for the March 1975 and February 1992 flood events:

- For the March 1975 flood, the AR&R temporal patterns predicted a peak flow of between 700 and 800 m^3/s. The Nowra RAN Air Station pluviograph was not operational during this event, so the chosen temporal pattern for the 1975 calibration was the 36hr AR&R design pattern, giving a peak flow of 800 m^3/s; and
- For the February 1992, peak flows were predicted to be between 485 m^3/s (both 30 hour AR&R and Nowra RAN Air Station patterns) and 550 m^3/s. The inflow hydrograph chosen for the 1992 calibration was the Nowra RAN Air Station patterns, however, the 20 hour AR&R pattern is equally applicable.

5.4 Justification of Hydraulic Model Assumptions

The absence of historical records at Conjola Lake prevents a conventional calibration of the hydraulic model to be undertaken. The extent of the missing flood data records for the calibration events is presented schematically in Figure 5-5.

As discussed in Section 5.1, a conventional hydraulic calibration exercise involves determining the energy loss elements in the model. In the case of Conjola Lake, the major component controlling the hydraulic profile is lake bed friction. The water level profile along the lake is essentially influenced by the bed roughness of the entrance channel and the deeper sections of the lake. The floodplain roughness in urbanised or forested areas has only a minor impact, as these areas convey a small proportion of the flood flows, at slower velocities.

A localised hydraulic control is the Lake Conjola Entrance Road bridge over the canal linking Pattimores Lagoon with the lake. The bridge is understood to surcharge during flood events, leading to overland flooding.

As much of the data required to perform a conventional calibration was missing (refer Figure 5-5), the energy loss elements could only be determined at the basis of assumed values for other model parameters. However, analysis of the hydraulic conditions can support the choice of some of the hydraulic parameters within the model, which are discussed in the following sections.

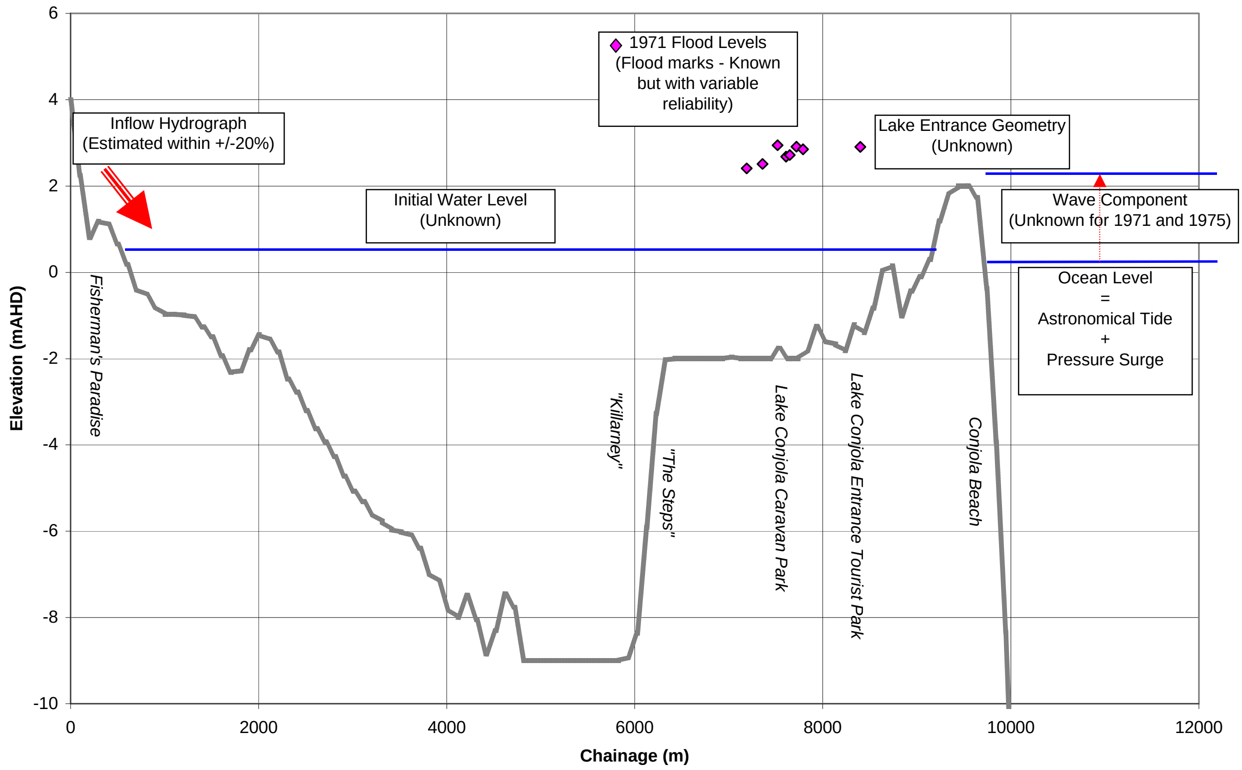

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Figure 5-5 Missing Historical Records for Hydraulic Calibration for Conjola Lake

5.4.1 Model Bed Roughness

Assumed hydraulic model bed roughness was derived from standard hydraulic literature and channel hydraulic conveyance calculations.

Initial hydraulic analysis showed that the entrance channel geometry was the critical parameter influencing flood levels in Conjola Lake. The lack of information on the sand berm at all stages of the flood prevented the determination of the lake's bed roughness through a complete computation of the model (which would include the sand berm).

It was possible, however, to analyse the friction losses in the entrance channel through the use of historical flood levels. The survey of 1971 flood marks provided two high quality flood levels, one kilometre apart, within the entrance channel (see Figure 5-6). The channel cross-sections (see Figure 5-7) at the two locations were extracted from the 1967 topography DEM (see Section 4.4). The hydraulic characteristics of the cross-sections were calculated and used in the Manning's equation:

$$Q = \frac{1}{n}\sqrt{S_f}\,R_h^{2/3}A$$

where Q Flow = 1,000 m^3/s. This discharge flow includes the inflows from Conjola Creek as well as the other subcatchments draining directly to the lake, as well as the flow attenuation of the lake storage volume;

A Cross-section area, = 1,800 – 1,700 m^2;

R_h Hydraulic radius, = 3.2 – 3.4 m;

S_f Water surface slope, calculated from historical flood levels, = 0.04 m/km;

n Hydraulic roughness, Manning's coefficient.

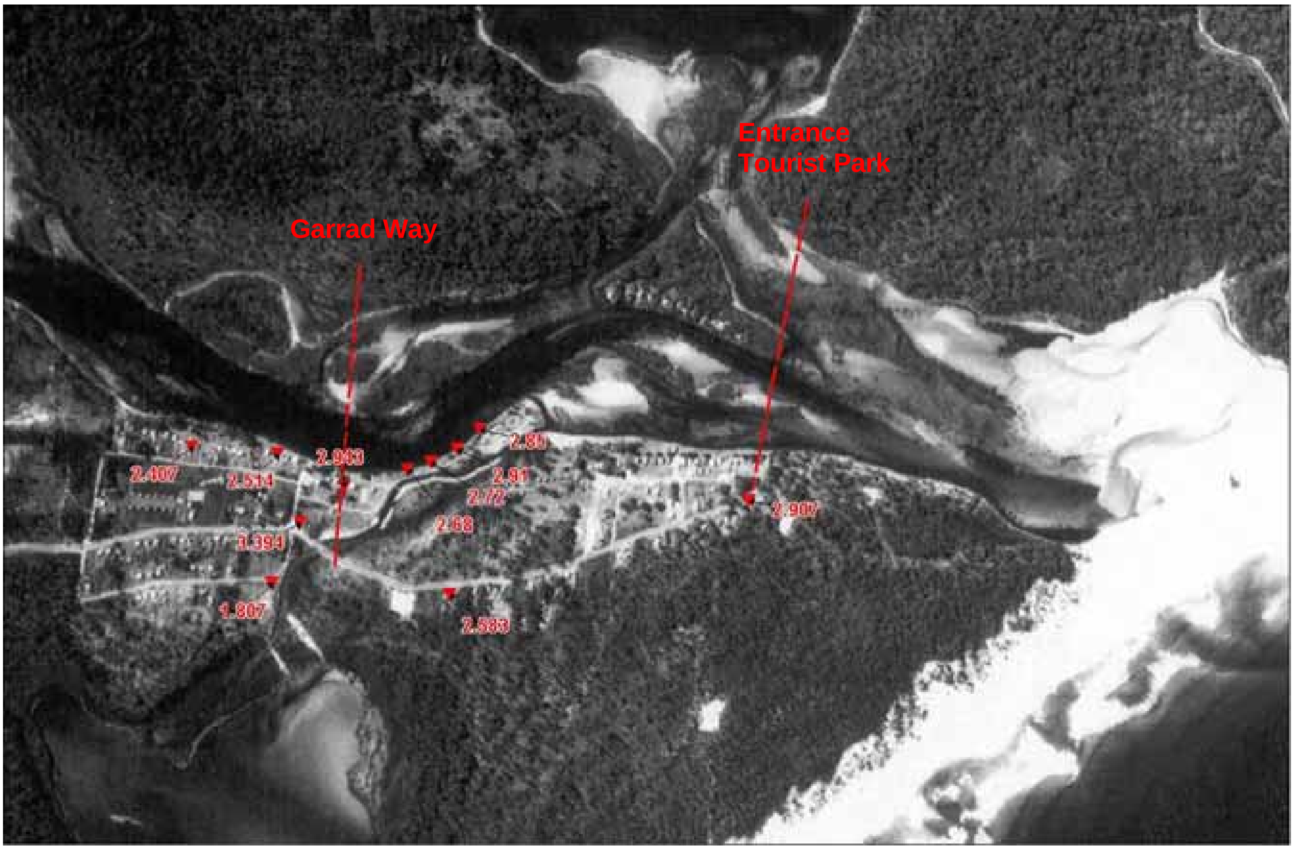

Figure 5-6 1971 Historical Flood Marks at Lake Conjola Village

The analysis indicates that sand bed roughness values of between 0.023 and 0.025 (Manning's n) are appropriate for the Conjola Lake entrance channel. The 1D nature of the estimation includes the losses due to turbulence created by meandering. The TUFLOW model used in this study calculates these losses directly through its two-dimensional discretisation.

Unfortunately, the location and precision of the historical flood levels for the 1975 and 1992 events do not allow the same analysis for bed roughness for these events.

The same hydraulic analysis of water surface gradient between Entrance Tourist Park and Garrad Way was undertaken using the 2D TUFLOW model. The model shows that a sand roughness value of 0.020 is appropriate for modelling the Conjola Lake bed in a 2D hydraulic model. This value is within the conventional range of this parameter, and is justifiably smaller than the 1D calculated value, as the 2-D model calculates losses due to turbulence by meandering separately.

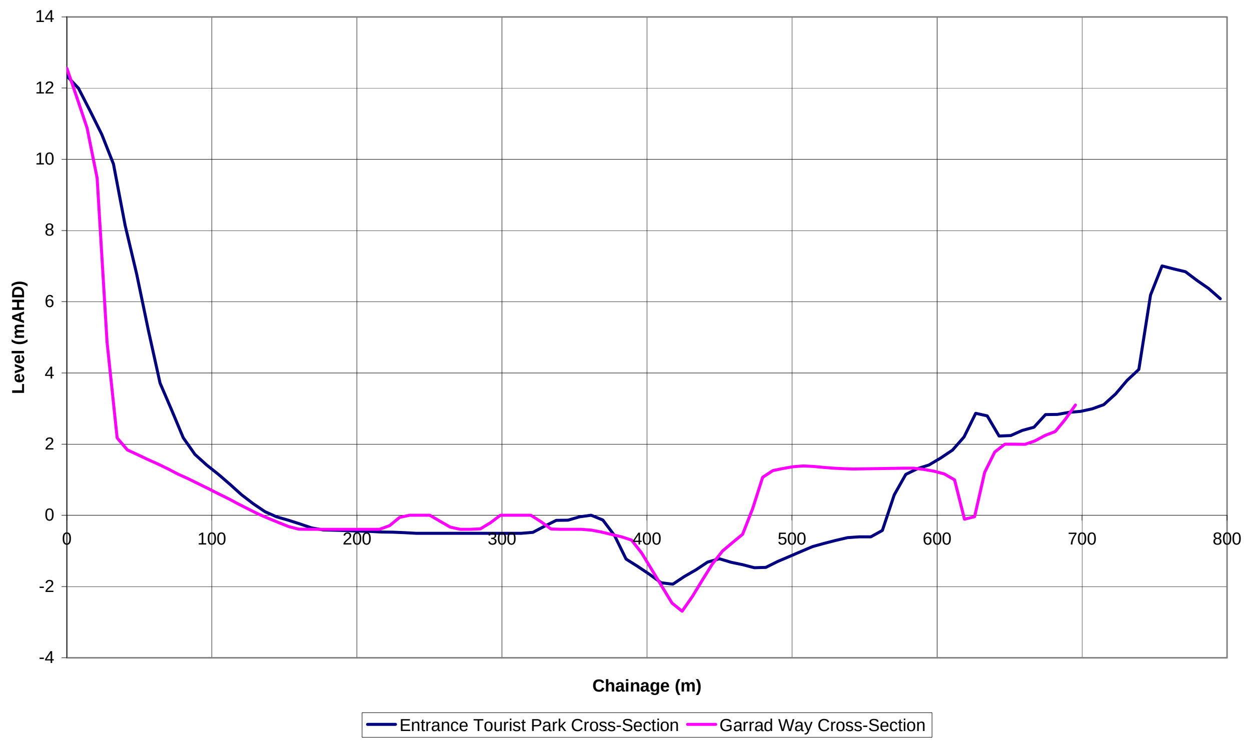

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Figure 5-7 Conjola Lake Cross-Sections at Entrance Tourist Park and Garrad Way

The definition of roughness for the other areas (villages, islands, national park) was determined in agreement with appropriate engineering standards (e.g. Ven Te Chow, Arcement and Schneider) and flood modelling experience:

- n = 0.15 for forested and vegetated areas (essentially all lake banks not part of an urbanised area);
- n = 0.15 for islands located in the tidal delta. The level of vegetation is comparable with the lake banks;
- n = 0.06 for urban areas (villages), which consist principally of dwellings, large grassed gardens, occasional trees (especially along the lake banks and open spaces; and
- n = 0.02 for lake, water bodies and sand. This is the value most applicable for flood conveyance. As such, it has a greater influence on model results than the other areas with alternative friction co-efficients.

The areas on the lake fringe and overbank floodplains provide minimal conveyance to the overall Conjola Lake flood flows. As such, the roughness values associated with these areas are relatively insensitive to the model results (i.e. computed water levels and flood extents). However, the adopted roughness directly influences the computed velocities and therefore affects the provisional hazard calculated in these areas.

5.4.2 Historical Entrance Channel Geometry

The adopted entrance channel geometry is critical in predicting flood levels in Conjola Lake. Unfortunately, historical entrance channel geometry data for the calibration events is too inaccurate to be used with confidence in the hydraulic model calibration process. To overcome this issue, a theoretical eroded entrance channel was determined and used in the hydraulic model to correspond to the time of the historical flood peak.

This Section describes the influence of the entrance channel, and provides justification for the use of the adopted eroded channel geometry.

5.4.2.1 Impact of Entrance Channel and Entrance Sand Berm

Preliminary hydraulic calculations demonstrated that the entrance channel geometry at the location of the constriction created by the entrance sand berm is the critical parameter determining flood levels in Conjola Lake.

The downstream end of the entrance channel is constricted by the formation of a sand spit (berm) across the entrance compartment, due to deposition by sand moving alongshore and onshore under the influence of coastal (wave) processes. The sand spit is referred to as the entrance sand berm. The entrance berm locally reduces the effective entrance channel width. Within the open entrance compartment, coastal processes try to close the entrance by depositing marine sands within the entrance channel. Ebb (outflowing) tides tend to scour out much of the sand deposited by incoming tides and wave processes. The entrance sand berm is variable with time, and can extend across the entire entrance channel width to completely close the entrance (although this is likely to be prevented in the future as part of the Lake Conjola Entrance Management Plan). When closed, the entrance berm is higher than the water levels within the lake. Elevated lake levels are then required to overtop the berm and initiate a breakout.

The geometry of the entrance sand berm defines the extent of flow constriction within the entrance channel. The presence of the constriction forces the upstream water levels to rise in order to provide sufficient potential energy to convey the flood flows through the channel constriction. The resulting energy head upstream of the entrance sand berm dominates flood levels upstream.

Flow through the entrance constriction is controlled by the conveyance of the cross-section. From the Manning's equation, cross-sectional conveyance is defined by:

$$\text{Conveyance} = R_h^{2/3} A$$

Where A Cross-section area;

R_h Hydraulic radius (= Wetted perimeter / cross-sectional area) = approximately equals the water depth where channels are wide relative to their depth;

The cross-sectional conveyance can be interpreted as being the geometric constraint of a cross-section in determining hydraulic discharges. For fixed roughness and energy slope conditions, the discharge through a cross-section will be proportional to its cross-sectional conveyance capacity.

At the height of the 1971 flood, approximately 1,000 m^3/s (refer to Section 5.3) flowed over an eroded but still constricted entrance. The cross-sectional conveyance of the entrance channel section at the entrance sand berm location was determined t be 200 $m^{8/3}$, while the upstream sections in the entrance channel (for instance at Lake Conjola Entrance Tourist Park) had an approximate cross-sectional conveyance of 2000 $m^{8/3}$. The significant difference between the two conveyances translates to a difference ratio of 100 between energy slopes. This example illustrates the importance of the entrance channel geometry. It also highlights the flatness of the water surface gradient in sections upstream of the entrance channel.

5.4.2.2 Erosion of Entrance Channel and Sand Berm

The downstream end of the entrance channel erodes intensively during a flood event. Given the importance of the entrance channel geometry on determining upstream flood levels within Conjola Lake (see Section 5.4.2.1), the prediction of the channel erosion is considered critical to the computation of lake flood levels.

For simulation of the historical flood events, given that the initial entrance channel geometry was not known with sufficient accuracy, a theoretical eroded geometry was introduced to the model to replicate adequate hydraulic conveyance through the entrance channel at the peak of the floods.

From a coastal processes perspective, the entrance channel extends upstream to “The Steps” (i.e. the upstream limit of the marine flood tide delta). During a flood event, the flow has a bed scouring potential that is dependent on near-bed velocities, depth and sand characteristics. In relative terms, due to the much smaller cross-sectional conveyance (and hence much higher near-bed velocities), there is significantly greater potential to scour the entrance channel at the downstream end.

At the scale of the study, the sediment transport routine, based on the Van Rijn equations, does not determine exact cross-sectional profiles. On the other hand, the model does allow the integration of scouring processes at the sand berm in terms of cross-sectional conveyance. The scouring rate is based on inter-related parameters: flood flows, initial water levels, downstream sea levels and, of greatest importance, the original sand berm geometry. However, the original sand berm geometry, which was determined in Section 4.4.1, is too indeterminate to allow the historical erosion process to be replicated for the calibration events. Combined with the broad assumptions on flood flows, initial water levels and ocean conditions, an accurate calculation of the entrance geometry is unattainable, especially at the time of the peak flow.

Preliminary hydraulic modelling has proven that the calculated eroded sand berm levels can be unrealistically too high or too low, depending on the original sand berm geometry, which can lead to inappropriate water levels in the lake.

Reproduction of historical flood behaviour and levels can only be achieved if an appropriate sand berm geometry is established at the peak of the flood. For the calibration events, an un-erodable bed was introduced in the model below the sand berm (approximately 1m below the initial entrance channel bed level). The un-erodable bed represents the maximum extent of the vertical erosion allowed in the geomorphologic model. Definition of the un-erodable bed was calculated iteratively by provision of an appropriate cross-sectional conveyance within the entrance at the peak of the historical floods to match calculated flood levels with the historical flood marks.

Definition of the geomorphological model parameters allowed the erosion of the entrance channel bed to reach the un-erodable bed level before the flood peak. This ensured that the correct conveyance occurred in the model at the time of the flood peak integrating vertical and lateral scouring, which then dictated the upstream peak water levels. Whilst the adoption of an inerodable bed introduces a new parameter to the model, it is considered that the exact shape of the cross-section is not considered to be significant, providing that the correct conveyance is simulated. The basis of this claim is discussed below and demonstrated in Figure 5-8 and Figure 5-9.

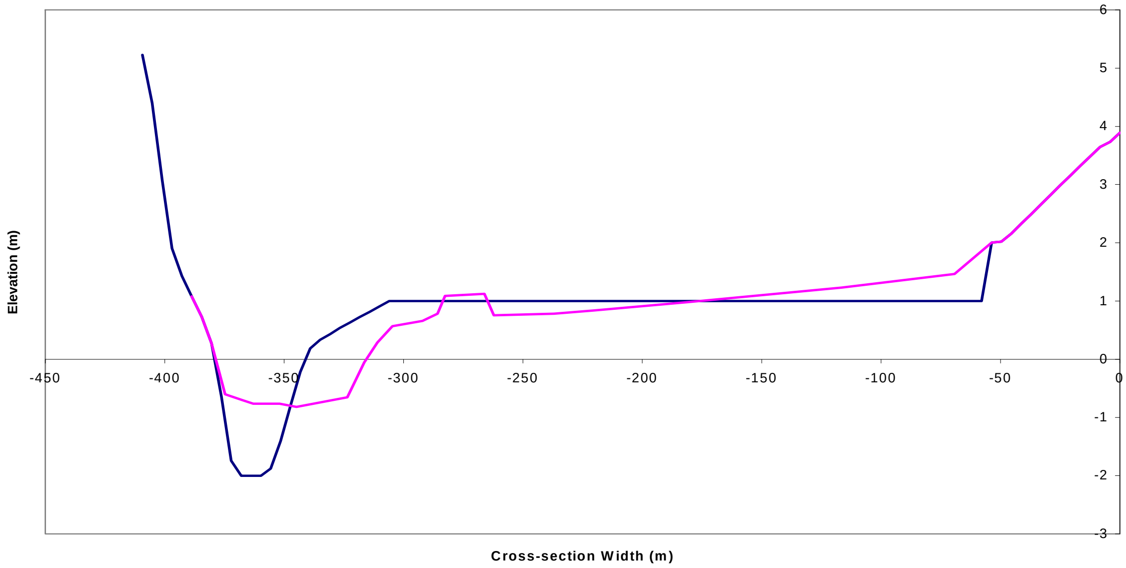

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Figure 5-8 Theoretical Conjola Lake Entrance Cross-Section Shapes

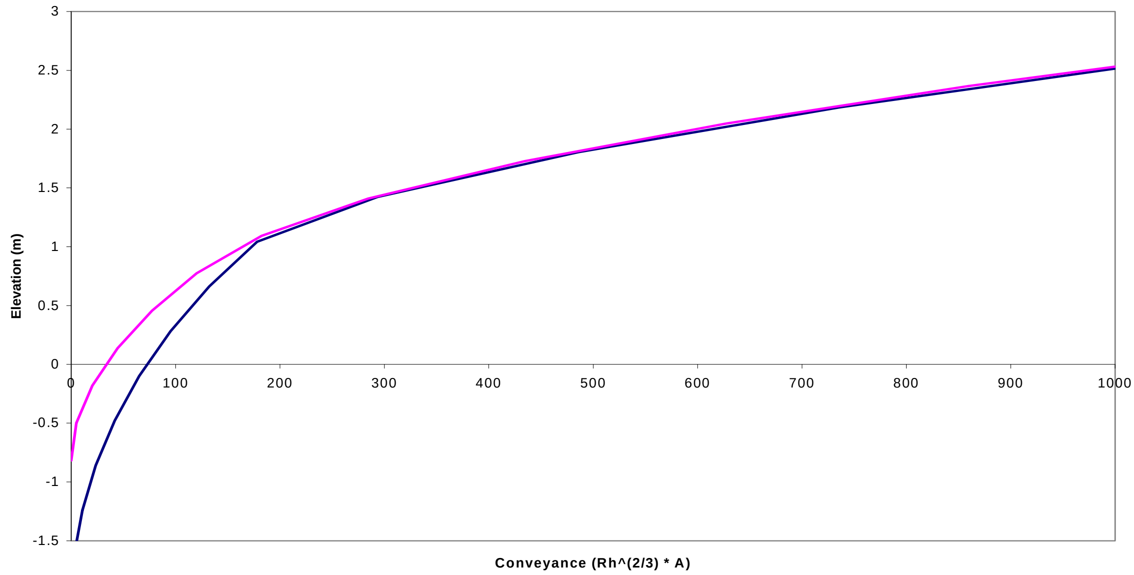

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Figure 5-9 Cross-sectional Conveyance Capacity associated with Figure 5-8

Figure 5-8 presents two arbitrary but possible entrance cross-sections at the height of the 1971 flood (refer Section 4.4.1). Figure 5-9 shows the respective cross-sectional conveyance for the two sections relative to water level elevation. As can be seen from Figure 5-9, both sections have almost identical conveyance for water levels above 1.0 m AHD, despite their shapes being quite different. Based on their equal conveyance, the two sections would result in the same upstream water levels within the lake. Consequently, provided that flow conveyance is adequately represented, definition of the inerodable bed geometry is somewhat arbitrary.

A similar process of estimating maximum bed erosion was also carried out for the 1975 and 1992 floods.

5.4.3 Initial Lake Water Levels

The initial lake water level for a shoaled (closed) entrance scenario influences the time taken for the sand berm to become overtopped and thus initiate entrance channel erosion, but *a fortiori*, it controls the extent of erosion at the time of the flood peak. For the historical events, as the geometry of the entrance channel defined theoretically (see Section 5.4.2), the initial lake water level was prescribed to ensure maximum vertical erosion at the time of the flood peak.

The Conjola Lake entrance was reportedly closed prior to the 1971 and 1975 floods, and shoaled before the 1992 flood. For a totally blocked entrance condition, catchment runoff will be stored in the lake until the entrance sand berm is overtopped / breached.

When the initial water level at the beginning of a rain event is unknown, the available storage volume within the lake is also unknown. Because this volume is critical to routing of the flood hydrograph through the lake, the initial water level has a major influence on the entire scouring dynamics at the entrance. Coupled with the lack of information regarding the sand berm topography, these unknowns compromise the ability of the model to accurately predict the flood rise and peak flood level.

As an indication of the flood severity and the lake's storage potential, the theoretical maximum rise in the lake water levels has been calculated for the historical flood events, assuming a totally blocked entrance, no losses and an initial water level in the lake of 1.0 mAHD:

- 1971 flood: a rise of more than 7m;
- 1975 flood: a rise of more than 5m; and
- 1992 flood: a rise of approximately 4m.

As the closed sand berm would typically not exceed a level of about RL 2-2.5m AHD, it is clear that each of these events would have resulted in breaching of the entrance berm and subsequent entrance scour, which would have had significant repercussions on lake flood levels.

For the historical events, an initial water level of 0.25m below the level of the lowest point in the shoaled entrance crest was chosen. These adopted initial water levels ensured that the required hydraulic conveyance at the flood peak was reached.

5.4.4 Downstream Sea Levels

5.4.4.1 Historical Sea Level Estimation

The offshore sea levels can have an impact on flood behaviour in Conjola Lake. It is possible that, under certain (sub-critical) conditions, backwater effects can affect the upstream water levels.

As there is no offshore sea level gauge, the historical levels were estimated from nearby data. Water levels recorded in Jervis Bay are the closest available to the project area. The proximity to Conjola Lake (20km) ensures that there is little difference between Jervis Bay tide levels and those offshore at Conjola Lake. The use of the Jervis Bay tide records is considered to be appropriate for the project.

The Jervis Bay station has, however, only been recording tide levels since July 1995, and hence there is no data available for the calibration events. Sydney Harbour has however been recording tide data for more than one hundred years, which includes the historical flood event dates. It was considered that tide level data from Sydney Harbour could be used to represent offshore sea level conditions at Conjola Lake, providing that the data is transposed to account for recorded differences between the Sydney Harbour and Jervis Bay gauges. A statistical analysis was undertaken for the period of overlapping Jervis Bay and Sydney Harbour station records (7 years of hourly records). The analysis results show:

- 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 Conjola Lake flood simulations, providing a 0.15m increase is applied to the Sydney data to account for the difference between the Sydney and Jervis Bay results.

The transposed Sydney Harbour tidal data represents the combination of astronomical tide with the barometric anomalies that accompany weather patterns. Wind setup is not included in the data, but is considered marginal, with very limited impacts on Conjola Lake floods.

Set-up of nearshore sea levels can occur due to wave processes. The wave setup is calculated as approximately 10 to 15% of the offshore breaking waves height (Blain, Bremmer and Williams, 1985). The breaking wave height H_b is derived from offshore wave measurements (H_0, offshore wave height, T, offshore wave period):

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

Offshore wave measurements for Jervis Bay are available for the 1992 flood event. The maximum calculated wave setup for the 10 February 1992 storm event was estimated to be 0.5m, corresponding to offshore wave heights of 4.5m with a period of 10s (ie. typical coastal storm conditions). This estimate is considered to be conservative due to the methodology used. The 1992 wave setup component and resulting sea levels at Conjola Beach are shown on Figure 5-10.

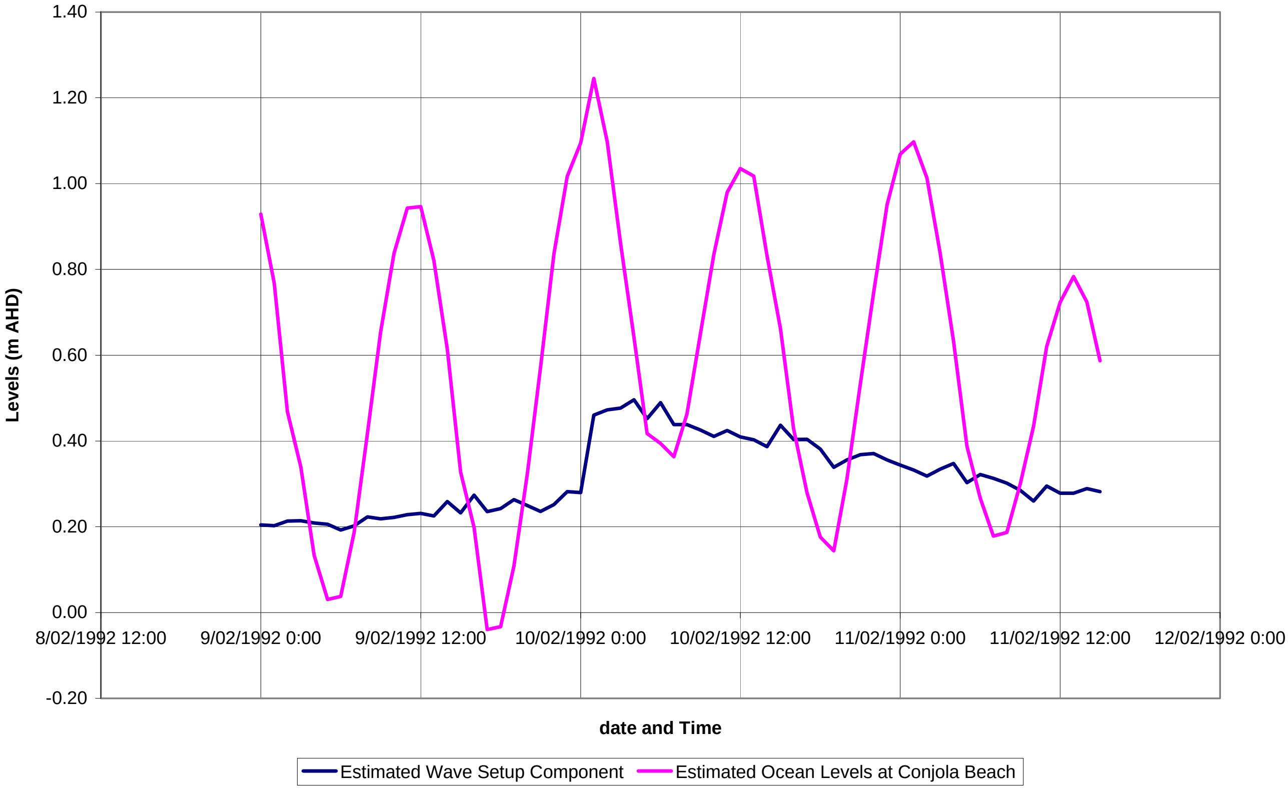

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Figure 5-10 February 1992 Ocean Conditions at Conjola Beach

The same 0.5m wave setup was used for the 1971 and 1975 flood events, based on the assumption of coinciding coastal storm conditions (which are considered likely given the severe weather fronts responsible for the catchment flooding). Sensitivity tests were carried out to assess the influence of this parameter on model results (refer Section 5.4.4.2).

5.4.4.2 Sensitivity Analysis

The influence of the downstream sea level conditions on the flood levels in Conjola Lake was assessed during the calibration process. Different wave setups, from 0m to 1.9m, were considered to complement the measured astronomical tide and storm surge. A wave setup of 1.9m was calculated as 15% of the height of the 1% AEP off-shore breaking wave height, which is considered to correspond to a 1% AEP wave setup condition. It is important to appreciate that combining a 1% AEP runoff flood with a 1% AEP wave setup would represent conditions that have a probability of occurrence of < 1% per annum, given the likelihood of both conditions coinciding.

The results of the sensitivity analysis showed that the upstream effects of ocean levels are limited during a significant catchment runoff event. For the 1971 event, the effects of the sea level do not exceed 0.22m height 1km upstream of the sand berm (location of the Lake Conjola Entrance Tourist

Park) even for 1% AEP wave setup conditions. Results of the sensitivity analysis are presented in Table 5-1.

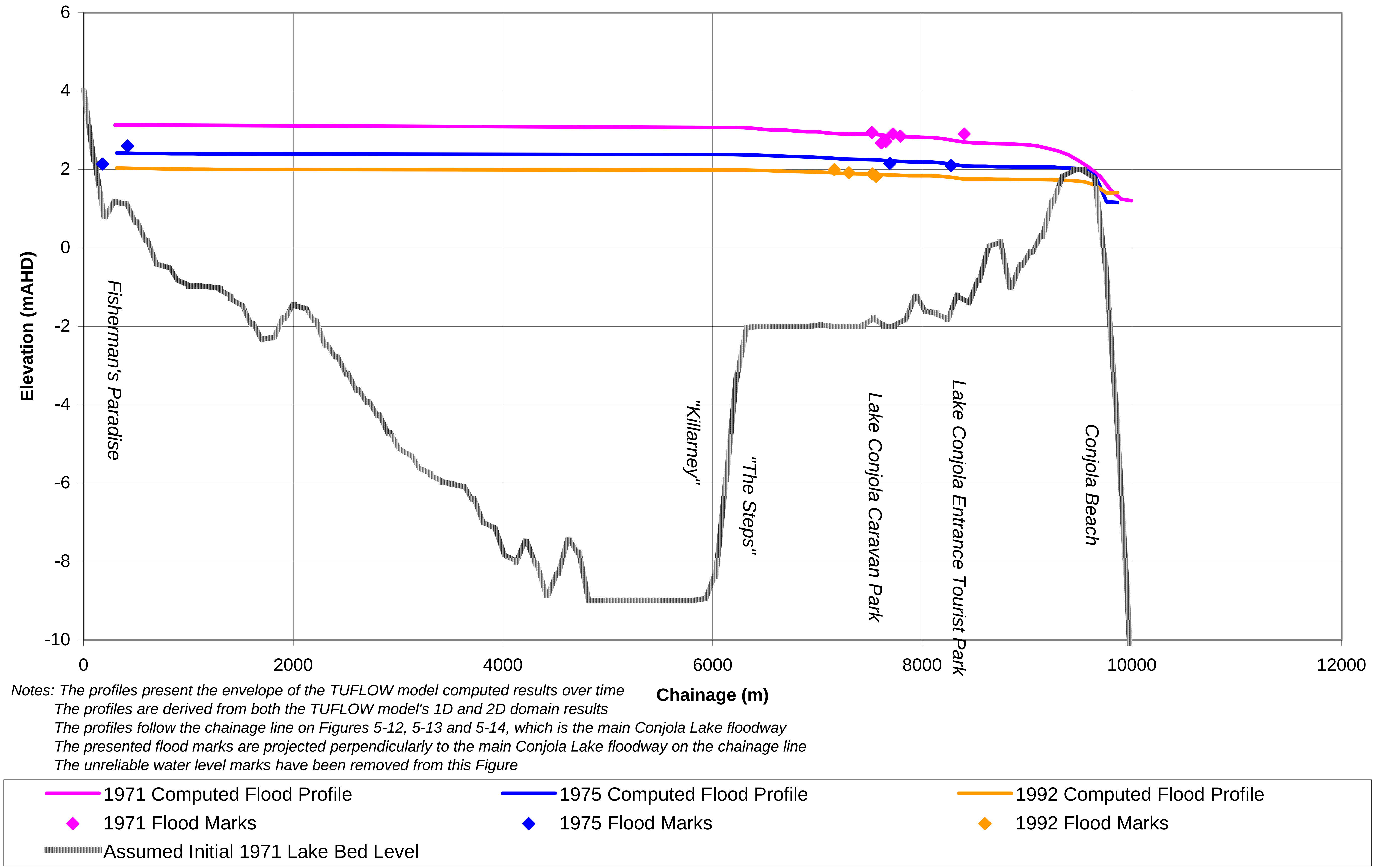
Table 5-1 Influence of Ocean Levels on 1971 Flood Conditions

	Peak Sea Level (mAHD)	Peak Level 1km Upstream of Sand berm (mAHD)	Wave Setup Impact 1km Upstream of Sand berm (m)	Flow Regime at Sand berm
Maximum Astronomical + Surge Tide (Wave setup = 0.0m)	0.92	2.86		Super-critical
Wave Setup = 1.0m	1.92	2.86	0	Critical
Wave Setup = 1.5m	2.42	2.91	0.05	Sub-critical
Wave Setup = 1.9 (15% of 1% AEP off-shore wave height	2.8	3.08	0.22	Sub-critical

5.4.5 Comparison with Historical Flood Marks

Uncertainties regarding model inputs for historical flood conditions compromise the ability of the model to accurately predict flooding within Conjola Lake. However, by using well-justified and realistic ‘assumed’ input values, the model has been able to reproduce historical flood conditions, as defined by records of maximum flooding. Figure 5-11 presents the longitudinal profiles through the lake while Figures 5-12 to 5-14 present flood level maps based on the hydraulic model results for the calibration and verification events. The quality of some of the historical flood levels is considered to be questionable. For example, one 1971 flood level was quoted to be “5 foot depth over the caravan park”. The caravan park ground level was surveyed at 4 different points and shows a 0.25m variability. The quality of such a flood level is weighted poorly. Comments regarding the calibration flood marks are provided in Appendix D.

The match between predicted and measured flood results is considered satisfactory, to an accuracy of about +/- 0.2m (which is considered a reasonable accuracy for the reliable historical flood levels).



Notes: The profiles present the envelope of the TUFLOW model computed results over time
The profiles are derived from both the TUFLOW model's 1D and 2D domain results
The profiles follow the chainage line on Figures 5-12, 5-13 and 5-14, which is the main Conjola Lake floodway
The presented flood marks are projected perpendicularly to the main Conjola Lake floodway on the chainage line
The unreliable water level marks have been removed from this Figure

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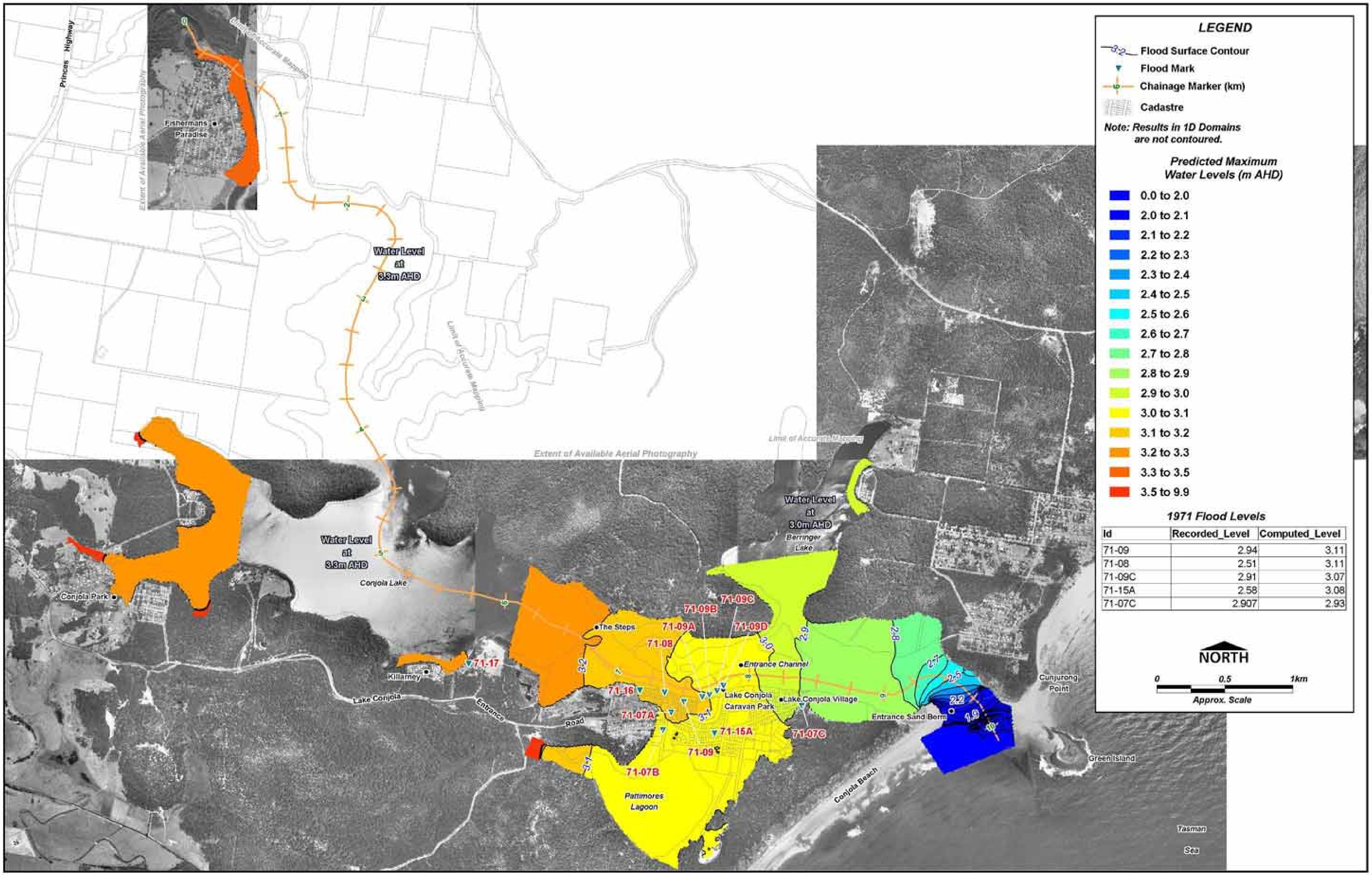
Figure 5-11 Computed Longitudinal Profiles for Calibration Events



1971 Flood Levels

Id	Recorded_Level	Computed_Level
71-09	2.94	3.11
71-08	2.51	3.11
71-09C	2.91	3.07
71-15A	2.58	3.08
71-07C	2.907	2.93

Figure 5-12 1971 Calibration Flood Level Map

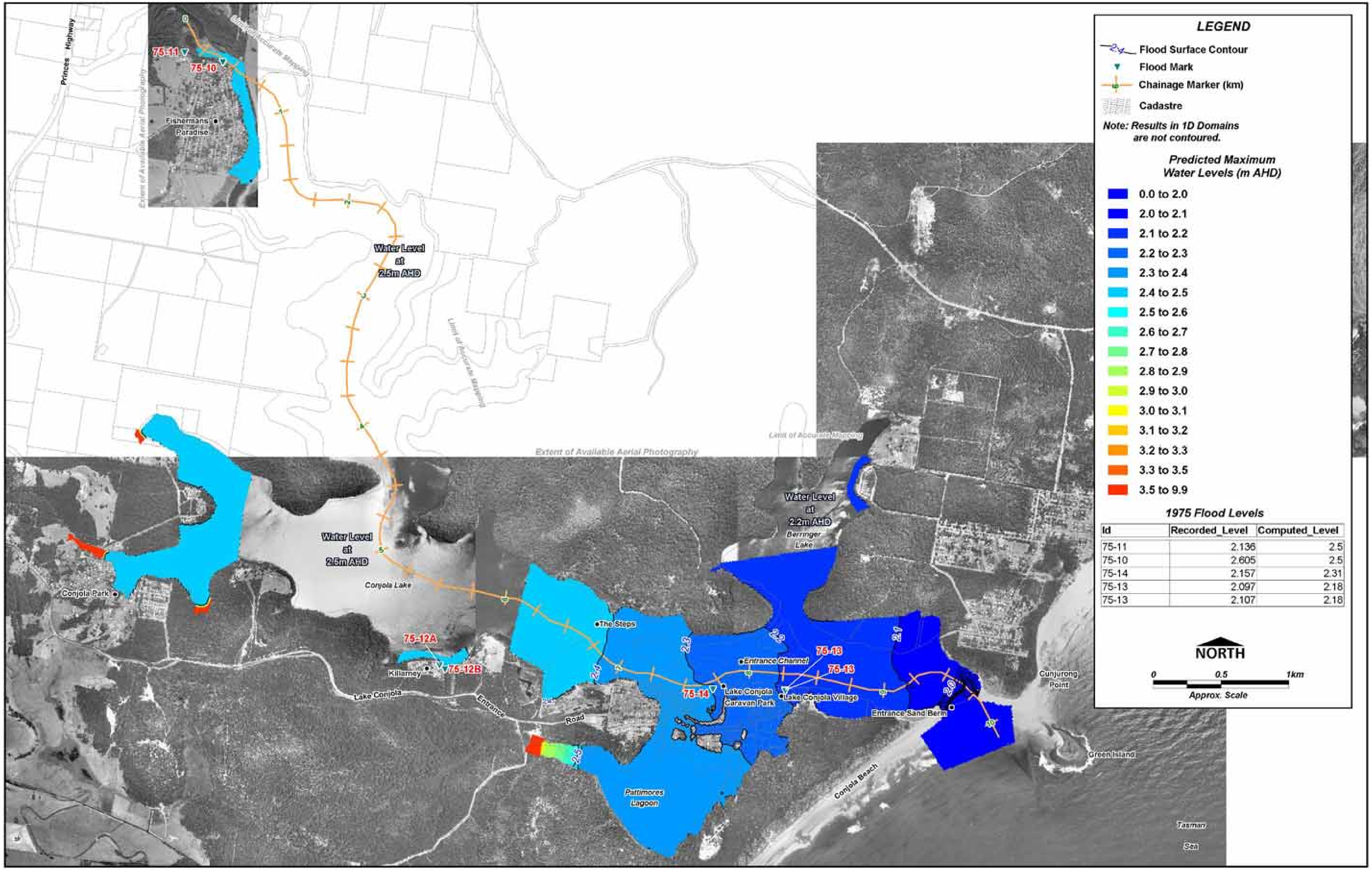


Figure 5-13 1975 Calibration Flood Level Map

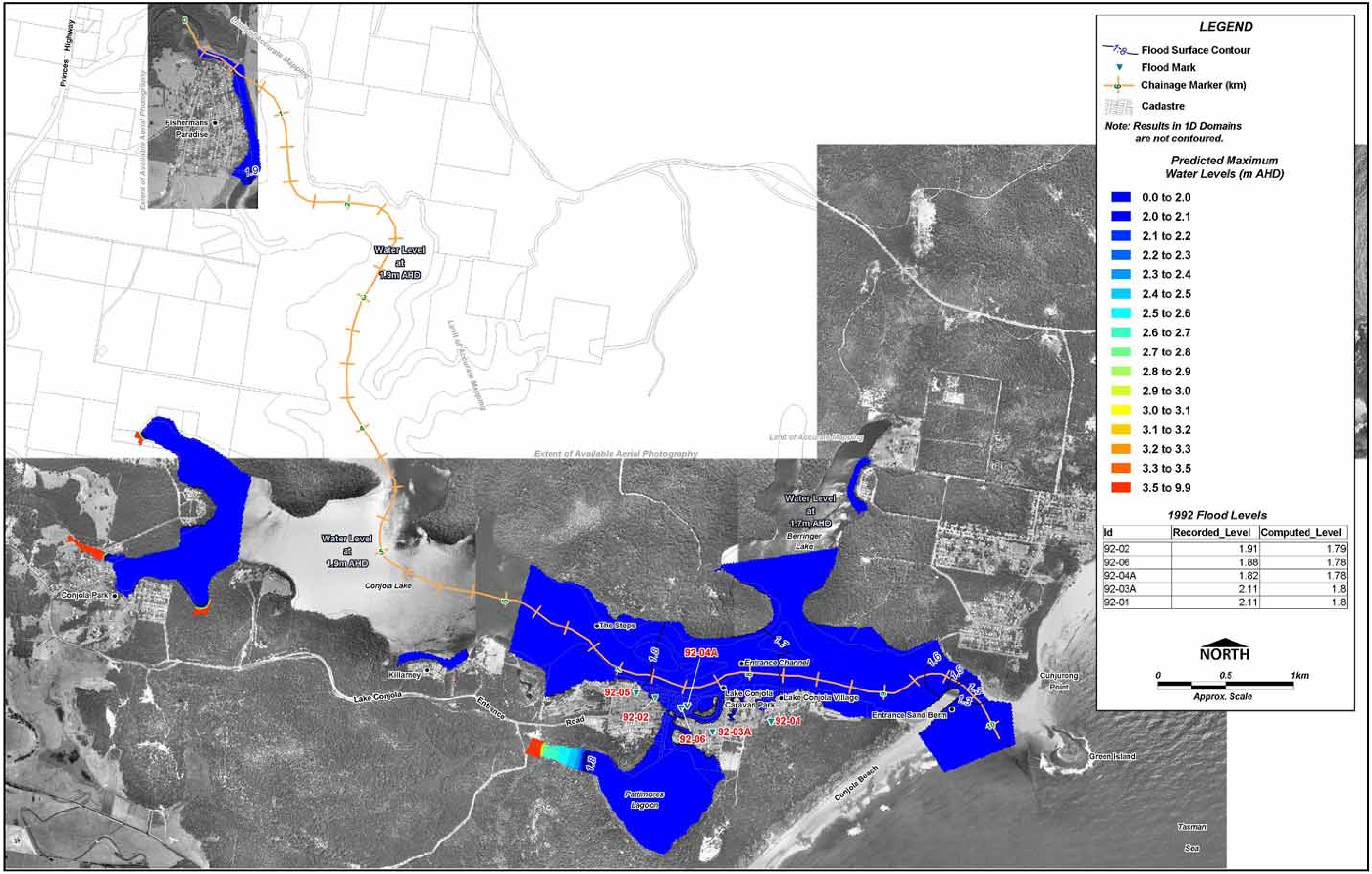


Figure 5-14 1992 Calibration Flood Level Map

5.5 Conclusion on Model Validation

A conventional calibration of the hydrologic and hydraulic flood models was not possible for the Lake Conjola Flood Study due to the complexities and unknown historical conditions associated with key model inputs, such as the sand berm and initial water levels in the lake at the start of the floods.

Sensitivity tests and analyses undertaken as part of the calibration process, however, provide validation of the appropriateness of the model, as follows:

- The lake's bed roughness is supported by hydraulic calculations and model results;
- The entrance channel geometry controls the lake's flood levels; and
- The ocean level has little impact over the lake's flood levels under catchment runoff conditions (February 1971).

In addition, assumed input parameters for the hydrologic model were validated by similarities in results with an independent desk-top hydrologic method (ie. Rational Method).

The Van Rijn sediment transport theory is the most accurate method to model the sand erosion in the Conjola Lake entrance channel and over the sand berm. Although it is not possible to calibrate the sediment transport aspect, the geomorphologic routine has been tested and calibrated in previous projects ensuring its suitability (eg. Murray Mouth Morphology Study, WBM 2003).

The hydrologic and hydraulic models have been tested as rigorously as possible given the limited available data. As a result of these tests, it is concluded that the hydrologic and hydraulic models are robust and are suitable for predicting design flood conditions associated with future floodplain risk management of Conjola Lake.

6 DESIGN FLOOD CONDITIONS

6.1 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.

This report uses the AEP terminology. Table 6-1 provides a description of the different design floods considered as part of this study.

Table 6-1Design Flood Terminology

AEP[1]	ARI[2]	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.
Extreme Flood / PMF[3]		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.

1 Annual Exceedance Probability (%)
2 Average Recurrence Interval (years)
3 Probable Maximum Flood

In determining the design floods for Conjola Lake it is necessary to take into account the critical storm duration of the catchment. This is defined by the most severe flooding conditions, which is taken as that causing the highest flood levels.

6.2 Source of Design Floods

Design floods are developed using design rainfall. Design rainfall parameters are sourced from *Australian Rainfall and Runoff* by the Institution of Engineers Australia (AR&R, 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 Conjola 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).

6.3 Critical Storm Duration

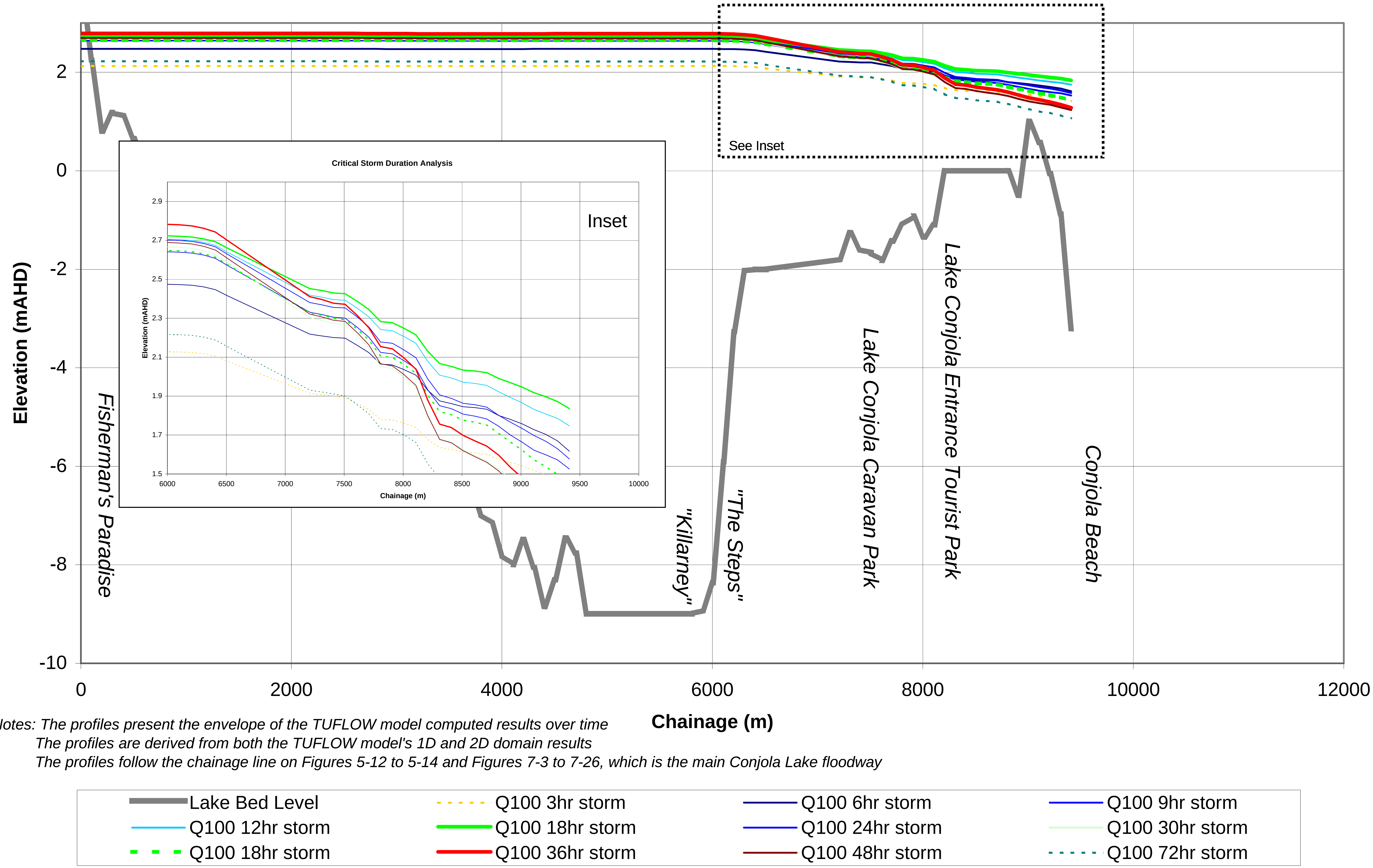
The critical storm duration is that duration of rainfall that will result in the highest peak flood levels at a particular location. The determination of the critical duration for Conjola Lake requires combining the design flood conditions associated with a range of storm durations. The design flood conditions include:

- Design rainfall depths (See Section 6.4);
- Design rainfall temporal pattern (See Section 6.5);
- Design rainfall spatial distribution (See Section 6.6);
- Entrance channel geometry (See Section 6.8); and
- Downstream ocean boundary levels (See Section 6.9).

For Conjola Lake, a range of storm durations from 3 hours to 72 hours was investigated for a storm ARI of 100 years.

Figure 6-1 shows longitudinal profiles of maximum water levels along the lake for the different storm durations. The models predict that the 36 hour storm generates higher peak water levels for the 100 year ARI design flood at all locations upstream of Lake Conjola Caravan Park compared to the other storm durations. For locations downstream of Lake Conjola Caravan Park, the 18 hour catchment flood was the most critical. However, the section of the lake downstream of the Caravan Park is also influenced strongly by ocean inundation. In fact, the 1 in 100 year ARI ocean level generates flood levels downstream of the Caravan Park that are higher than the 1 in 100 year ARI flood levels due to catchment runoff. Consequently, the 36 hour storm was the only catchment runoff event that was considered when determining design flood levels.

It should be noted that the 6 hour storm generated the largest flood discharge at the outlet of Conjola Creek to the lake (refer Table 6-2). Flood levels in the lake, however, relate more to the volume of runoff, and thus the longer storm duration with larger total volumetric runoff governs despite lower peak intensities.



Notes: The profiles present the envelope of the TUFLOW model computed results over time
The profiles are derived from both the TUFLOW model's 1D and 2D domain results
The profiles follow the chainage line on Figures 5-12 to 5-14 and Figures 7-3 to 7-26, which is the main Conjola Lake floodway

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Figure 6-1 Critical Storm Duration Results

It should also be highlighted that the critical storm duration analysis was undertaken with the entrance geomorphology model operational. The rate of the entrance channel erosion when combined with the AR&R (2001) design intensities and temporal patterns, explains the differences between the different longitudinal profiles shown in Figure 6-1.

Table 6-2Critical Storm Duration Analysis – Conjola 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 Conjola Creek Outlet (m^3/s)	1210	1410	1295	1310	1210	1210	1140	1250	1190	975

6.4 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).

6.4.1 5 year to 100 year ARI Events

The design rainfall depths for frequent to rare design flood events are calculated from the IFD coefficients of AR&R (2001). The IFD coefficients vary around the Conjola Lake catchment leading to small design rainfall intensity variability (+/- 5.5%). As an example, Table 6-3 shows the 100 year ARI 36hr storm design rainfall intensities for the four geographical extremities of the catchment.

Table 6-3Spatial Variability of 100 Year ARI Design Rainfall Depths Across Conjola Lake Catchment

Location in Conjola Lake Catchment	Rainfall Intensity (mm/hr)
East	12.0
West	13.8
North	14.4
South	13.4

Due to the relatively small variation in rainfall intensities across the catchment, an average rainfall intensity was applied to the entire catchment for all design events. Table 6-4 presents the design rainfall depths for the 36 hour critical duration storm for the Conjola Lake catchment.

Table 6-4 Adopted Design Rainfall Depths at Conjola Lake for 36 hour Storm

Recurrence Interval (ARI)	Total Rainfall Depth (mm)	Rainfall Intensity (mm/hr)
5 year	269	7.5
10 year	309	8.6
20 year	362	10.1
50 year	432	12.0
100 year	487	13.5

6.4.2 Probable Maximum Flood (PMF) Event

The Probable Maximum Precipitation (PMP) leads to 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 to 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 (BoM) undertake 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 Conjola 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 Conjola 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 = 145 km^2;

- 1 in 50 AEP 72 hour rainfall intensity, ${}^{50}I_{72}$ = 10.6 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 = 36 hours.

The regional prediction equation is provided for the 36 hour in AR&R (2001, Book VI Table 9):

$$PMP = 138.5 - 4.055\sqrt{AREA} + 450.5\sqrt{{}^{50}I_{72}}$$

The PMP for the Conjola Lake catchment based on AR&R (2001) preliminary estimation procedures is thus estimated to be 1555 mm, corresponding to a rainfall intensity of 43.2 mm/hr (average value applied to the entire catchment). This rainfall depth is more than three times the 100yr ARI rainfall depth.

6.5 Temporal Patterns

Temporal patterns define the way in which the rainfall depth for an event is distributed in time throughout the event. For example, the 100 year ARI event has a rainfall depth of 487mm and will be distributed non-uniformly throughout the 36 hours of 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 36 hour storm temporal pattern for Conjola Lake is presented in Figure 6-2.

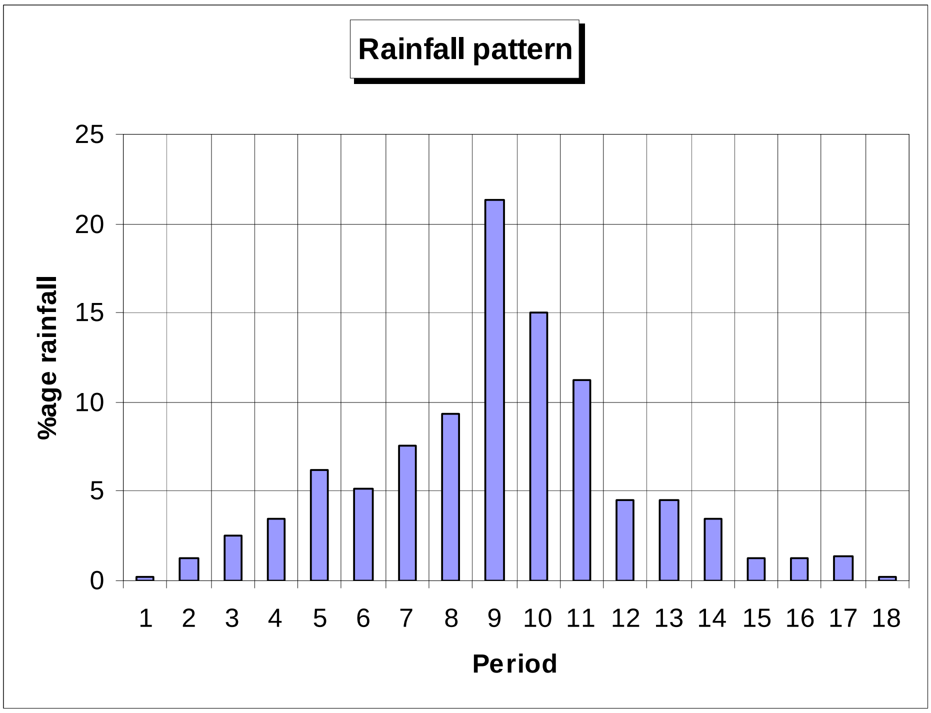

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Figure 6-2 100 Year ARI 36 hour Storm Temporal Pattern (Source: AR&R, 2001)

For the PMF event, AR&R (2001, Book VI Table 7) recommends the use of the smoothed GSAM temporal patterns. For the Conjola Lake catchment these were derived as described in BOM (HRS No 5, 1998). The resultant 36 hour PMP cumulative temporal pattern for Conjola Lake is presented in Figure 6-3.

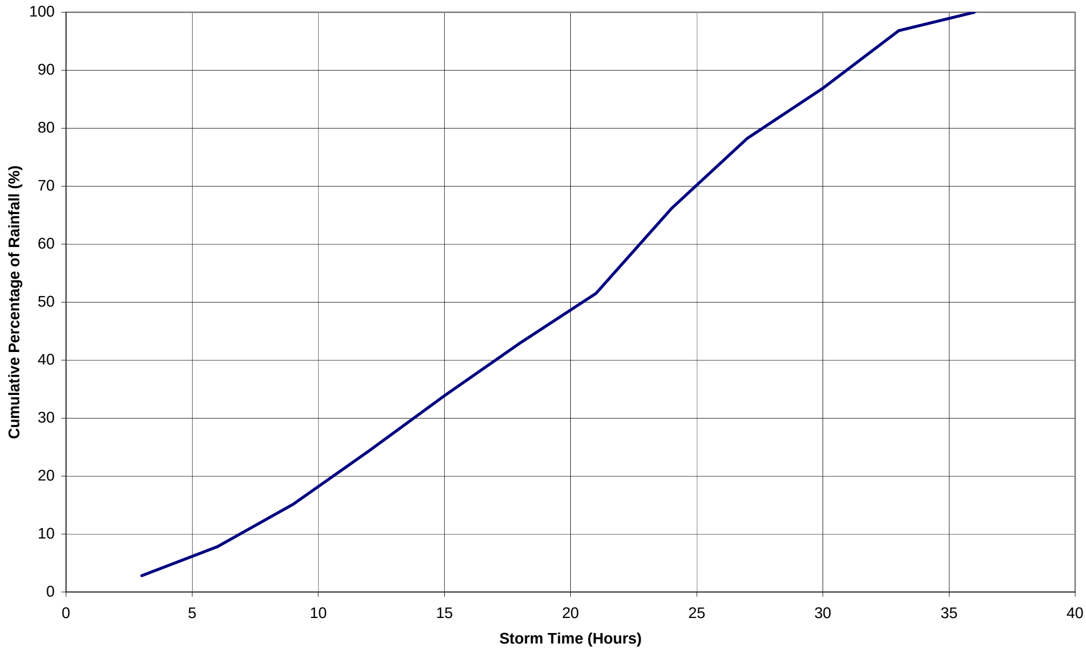

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Figure 6-3 36 hour PMP Cumulative Temporal Pattern (Source BoM, HRS No 5, 1998)

6.6 Spatial Distribution Reduction Factor

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 0.97 can be extrapolated for the 150 km^2 of the Conjola Lake catchment from Figure 1.6 in AR&R (2001), Book II. Even though no 36hr curve is printed on Figure 1.6 (AR&R, 2001), the trend between the different storm duration values for a 150 km^2 catchment is such that the 36hr storm value can be confidently extrapolated.

As a consequence, the design rainfall intensities presented in Table 6-4 were modified (multiplied by 0.97) and then applied uniformly across the Conjola Lake catchment in the hydrologic model.

6.7 Design Rainfall Losses

The design rainfall losses applied in the hydrologic model were modified from the calibration simulations to reflect the more conservative scenario of a saturated catchment at the time of the design event (that is, the design event follows a period of sustained rainfall). The adopted initial and continuing losses, as specified below, are lower that design loss rates suggested by AR&R (2001), which would result in higher catchment runoff, and therefore conservatively higher flood discharges and levels within the lake:

- Initial Losses: 15mm
- Continuous Losses: 1.5 mm/hr

6.8 Entrance Channel Geometry

The entrance channel is subject to geomorphologic change through the regular tidal movement of seawater and the episodic freshwater runoff from the catchment draining to the sea. 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.

The condition of the Conjola Lake entrance is primarily controlled and managed through Council's entrance management policy, which calls for entrance dredging when the sand berm constricts tidal flows beyond a defined condition.

DNR's Draft Floodplain Management Guideline No. 5 – Ocean Boundary Conditions – discusses the geometric aspects associated with the possible entrance channel types. Conjola Lake is naturally subject to episode closure but council have a policy of managing the entrance to prevent closure. The Conjola 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:

- The most restricted entrance channel geometric conditions for catchment flood; and
- The least restricted entrance channel geometric conditions for ocean flood.

For managed entrances, like Conjola Lake, these guidelines involve modelling the two extreme entrance geometries that are prescribed within the management plan:

- The most restricted entrance (ie the geometry that triggers management intervention under the entrance management policy); and
- The least restricted entrance (ie the geometry immediately following management intervention).

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).

6.8.1 Catchment Flooding

The trigger conditions for the entrance dredging decision support system are not based on the entrance channel geometry *per se*, but on the magnitude of tidal variations within the lake (as a surrogate for the degree of entrance shoaling). The amplitude of the M2 tidal constituent is taken as the gauge for entrance management actions. The M2 tidal constituent is the principal lunar component of the tide, and has a 12.5 hour period, which drives the normal semi-diurnal characteristic of tides on the east coast of Australia (see D.T. Pugh, 1987 for further information on tidal characteristics). The amplitude of the M2 tidal constituent is calculated by undertaking a formal tidal harmonics analysis of water levels recorded within Conjola Lake.

Trigger for action associated with the Conjola Lake Entrance Management Plan commence once the amplitude of the M2 tidal component reduced to below 0.15m.

For design scenario purposes, the DEM representing the existing situation (based on the latest survey information) was modified to include a shallow channel along the line defined in the Entrance Management Plan (MHL, 2002). Upon reaching the entrance management trigger condition, the channel was estimated to have a bed level of –0.5m AHD. This was confirmed by application of the model using a uniform neap tide cycle in the lake with no catchment inflow. Steady-state conditions in the model corresponded to a tidal amplitude of around 0.15m inside the lake, at the location of the existing water level gauge (refer Figure 6-4). Consequently, the adopted channel conditions were considered representative of the maximum degree of shoaling within the entrance (corresponding to the trigger conditions for entrance management). These conditions were adopted as the initial conditions for all design catchment flood simulations.

6.8.2 Ocean Flooding

The geometry of the entrance channel immediately after dredging is defined in the Entrance Management Plan (MHL, 2002).

The DEM representing the existing situation (based on the latest survey information) was modified to include a channel along the line defined in the Entrance Management Plan (MHL, 2002). The design channel bed level after dredging was fixed at –2.5 mAHD.

6.9 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 managed entrances, like Conjola Lake, this involves modelling two design flood scenarios:

- A flood induced by large catchment runoff and small ocean surge (low ocean tail water level). This design flood condition is to be applied coinciding with the most restricted entrance channel geometry permitted by the entrance management plan; and
- A flood induced by small catchment runoff and large ocean storm (large ocean tail water level). This design flood condition is to be applied coinciding with the least restricted entrance channel permitted by the entrance management plan (excluding a scoured entrance condition following previous flood events).

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).

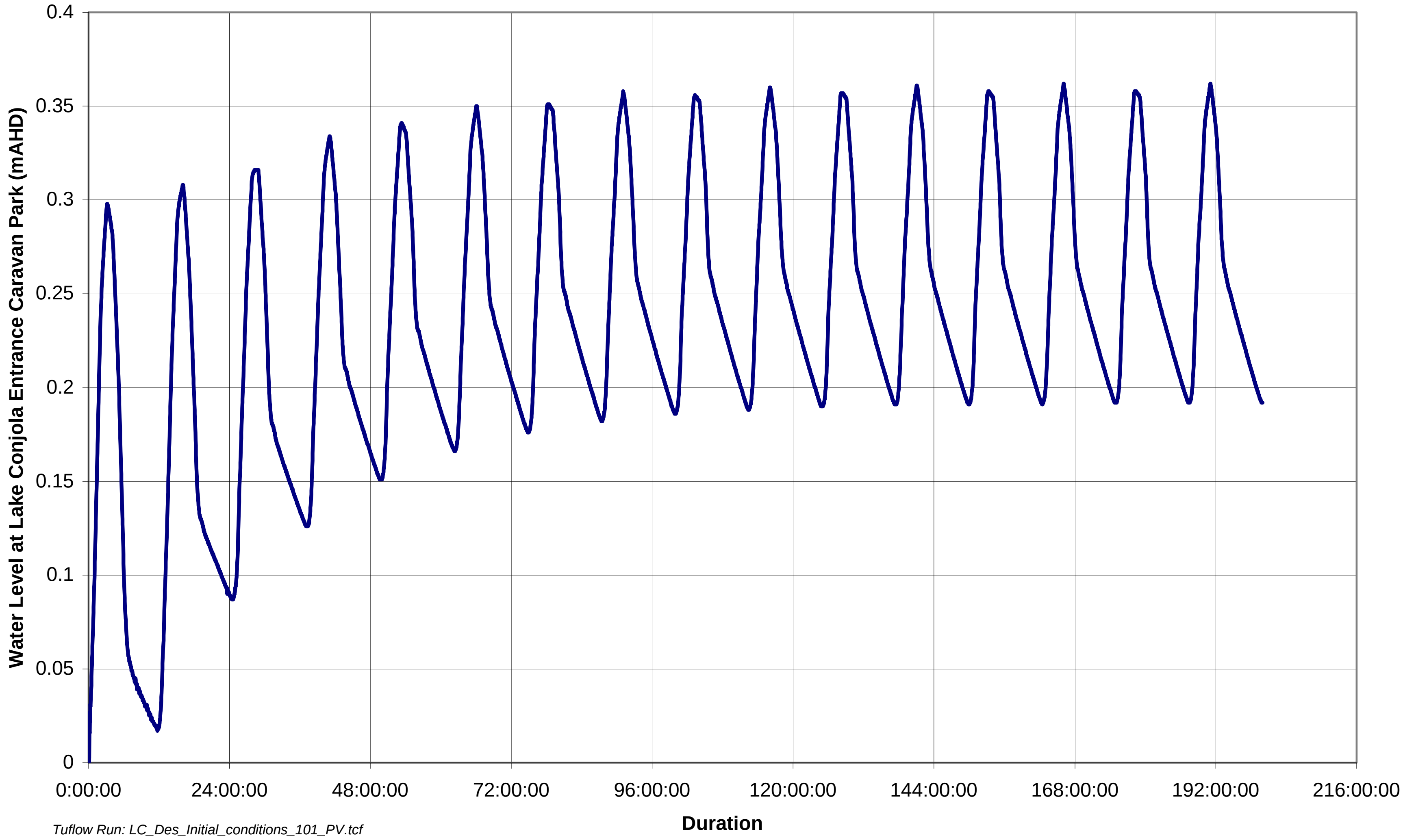

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Figure 6-4 Setup of Uniform Neap Tide Conditions in Conjola Lake

6.9.1 Catchment Flooding

Downstream boundary conditions for the simulated catchment flood conditions are defined in Table 6-5.

Table 6-5 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 (Standard Neap Tide)
10 Year ARI	10 Year	0.6 (Standard Neap Tide)
20 Year ARI	20 Year	0.6 (Standard Neap Tide)
50 Year ARI	50 Year	0.6 (Standard Neap Tide)
100 Year ARI	100 Year	0.6 (Standard Neap Tide)
PMF	PMP	2.6 (100 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 mAHD, which leads to peak tide water levels in Conjola Lake of approximately 0.35 mAHD.

In the specific case of the PMF event, the exceptional circumstances associated with the storm event are likely to raise the ocean level significantly. To represent the high tail water levels, a 100 year ARI ocean tidal cycle was used, peaking at 2.6 mAHD (high tide), and coinciding with the catchment flood peak.

6.9.2 Ocean Flooding

Downstream boundary conditions for ocean storm-based flooding are presented in Table 6-6:

Table 6-6 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)
PMF	PMP	2.60 (100 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 6-5. The first cycle is a neap tide cycle.

When comparing Table 6-5 and Table 6-6, 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).. Consequently, simulation of the 5 year ARI catchment flood event was not required.

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 were modelled.

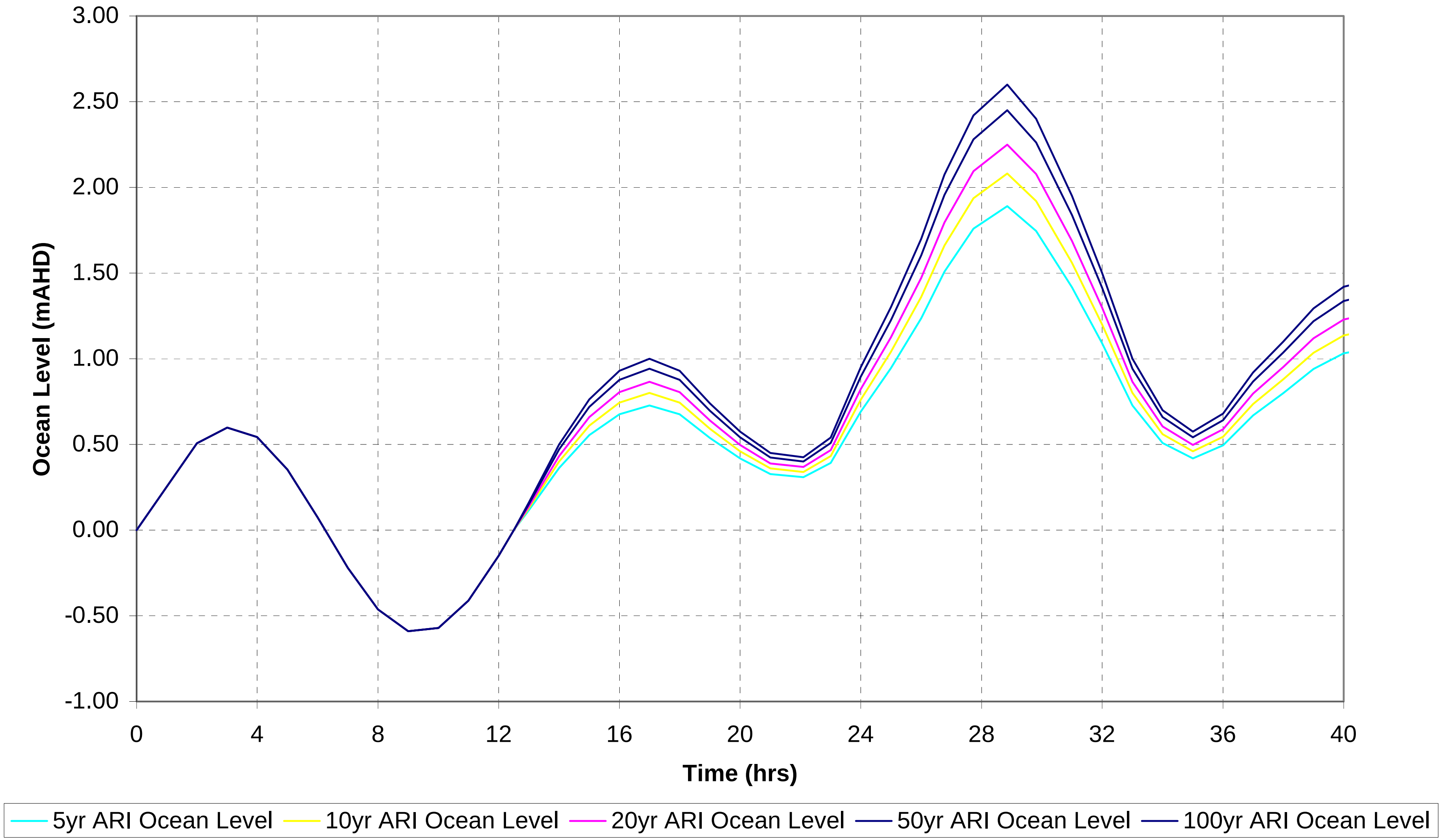

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Figure 6-5 Floodplain Management Guideline No.5 - Design Ocean Water Levels at Conjola Lake

7 DESIGN FLOOD PRESENTATION

7.1 Design Flood Description

The Conjola Lake flood behaviour differs depending on the source of flooding: catchment or ocean, as described below:

- Catchment flooding generates a large inflow in the lake associated with a low ocean tailwater level. The flow is accelerated at the downstream end of the entrance channel, and presents super-critical flow characteristics across the sand berm and on the beach front. The entrance channel is severely scoured, especially during the low tide cycles.

  A water level gradient is generated between “the steps” and the downstream sand berm. The steepness of the gradient increases with the flood severity. For the 100 year ARI flood, the maximum gradient is 0.5 m/km. A difference of around 1m in flood levels is noticeable between the upstream and downstream ends of Lake Conjola village.

  Upstream of “the steps”, the water level is almost horizontal up to Fisherman’s Paradise, where the narrower lake cross-section steepens the water level gradient.

- Ocean flooding is generated by a large offshore storm that lifts the ocean levels at the beach through the combined effects of pressure surge, wind and wave setups. The resulting ocean level on Conjola Beach can be 2m higher than a neap tide. Combined with a minor catchment flood, which still generates a great volume of runoff, the ocean flooding is capable of filling the entire lake.

  Although the entrance channel is too narrow to allow much of the ocean water to come inside the lake, the high tailwater level prevents the incoming catchment flood to flow out of the lake. The result is a flat water level gradient between “the steps” and the downstream sand berm. The steepness of the gradient is almost independent from the flood severity, as the same incoming flow is modelled (5 year ARI). The maximum gradient is around 0.05 m/km, approximately 10 times less than a catchment flood gradient. A difference of around 0.1m in flood levels is noticeable between the upstream and downstream ends of Lake Conjola village.

  Upstream of “the steps”, the water level is almost horizontal up to Fisherman’s Paradise, where the effect of the incoming catchment flood and the narrower lake cross-section steepens the water level gradient.

The modelling process has highlighted that under the defined design conditions the ocean flooding mechanism produces higher flood levels in most of Conjola Lake than equivalent design catchment flooding.

7.2 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 and velocities. 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 when considering the duration of the flood event and different possible flood mechanisms (i.e. they present an envelope of maximums over time and between design scenario alternatives). 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. Consequently, it is 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 7-1.

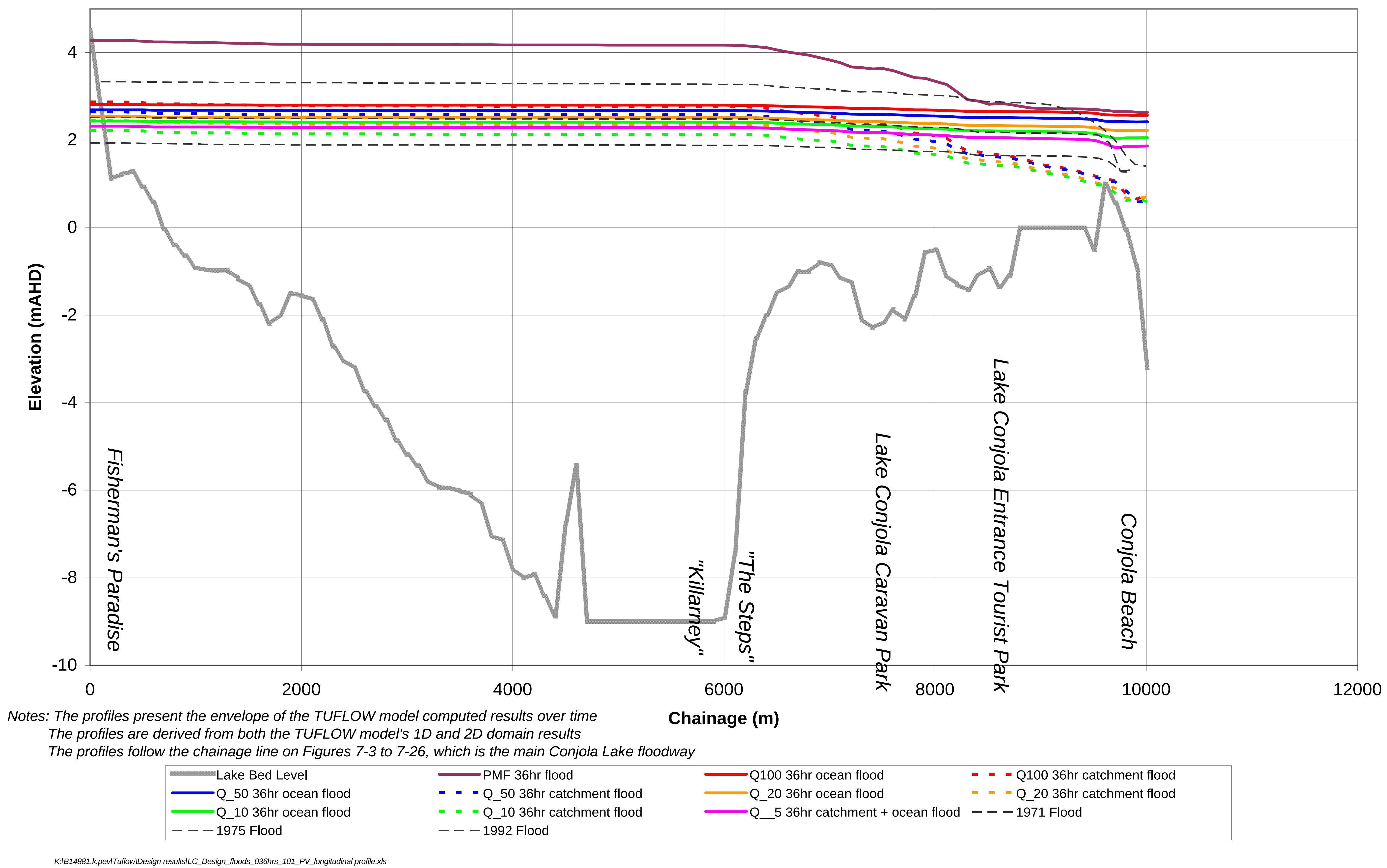

Figure 7-1 Conjola Lake Longitudinal Profile of Design Flood Water Levels

7.2.1 Design Peak Flood Levels, Depths and Velocities

Design peak flood levels, depths and velocities are provided in the Figures as listed in Table 7-1.

Table 7-1 Figure Numbers – Peak Flood Levels, Depths, Velocities and Provisional Hazard

Event ARI (years)	Peak Flood Levels	Peak Flood Depths	Peak Flood Velocities	Provisional Hazard
5 year	Figure 7-7and Figure 7-8	Figure 7-19	Figure 7-25 and Figure 7-26	Figure 7-37
10 year	Figure 7-9and Figure 7-10	Figure 7-20	Figure 7-27 and Figure 7-28	Figure 7-38
20 year	Figure 7-11 and Figure 7-12	Figure 7-21	Figure 7-29 and Figure 7-30	Figure 7-39
50 year	Figure 7-13 and Figure 7-14	Figure 7-22	Figure 7-31 and Figure 7-32	Figure 7-40
100 year	Figure 7-15 and Figure 7-16	Figure 7-23	Figure 7-33 and Figure 7-34	Figure 7-41
PMF	Figure 7-17and Figure 7-18	Figure 7-24	Figure 7-35 and Figure 7-36	Figure 7-42

7.2.2 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 L1 and L2 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 7-2 below.

The provisional flood hazard categorisation is presented in the result figures, as listed in Table 7-1.

7.2.3 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 to determine preliminary hydraulic categories that has been adopted for the Lake Conjola Flood Study has been used by WBM for other NSW flood studies, to the satisfaction of local and state government. The methodology uses both the unit flow (velocity 3 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 according to the NSW Floodplain Development Manual. 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 flow lateral repartition, the total flow can be divided in unit flows (flow per meter width) across the floodplain. Unit flows are a standard TUFLOW output. The integration of the peak unit flows along lines perpendicular to the main flow provides similar total flow values, which are close to the Conjola Creek peak inflow. A Floodway can be considered as following the continuous line that joins the maximum unit flows at each section perpendicular to the main flow. 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 Lake Conjola Flood Study, the threshold value was chosen to be the average unit flow value across the entire floodplain. Defined by unit flows higher than the average unit flow, the floodway is the area where, in most cases, the flow rate is overwhelmingly greater than the surrounding 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. In the case of the Conjola Lake 1 in 100 year ARI design flood, the floodway area includes 493 ha of floodplain area and 29.0 Mm^3 of water. The remainder of the floodplain covers an area of 696 ha and contains 22.3 Mm^3 of water (43% of total floodplain volume). The volume of water within the floodplain area (22.3 Mm^3) is such that, if the floodplain were blocked, 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 contains a volume of water of small significance for the flood behaviour. This volume of water was estimated as being an additional 0.1m depth on the top of the floodway areas, that is, 493,300 m^3. Through an iterative process of calculating the volume of water available in the floodplain (outside of floodways) at a range of different depths, it was determined that the flood fringe areas in the Conjola Lake floodplain are those areas outside of the floodways and with 1 in 100 year ARI flood extents with depths less than 1.0m.

The results of the preliminary hydraulic categorisation exercise are presented in Figure 7-43 and Figure 7-44. The floodway, storage and fringe areas were defined based on the 1 in 100 year ARI design flood. The extent of the flood fringe areas was increased to include the additional floodplain extent from the PMF.

The methodology ensures that the preliminary hydraulic categories are determined cumulatively across the entire Conjola Lake floodplain, and does not consider individual elements separately, which is a requirement from the NSW Floodplain Development Manual. It also ensures that the process can be reproduced easily as it is based on model results and automated post-treatments. Confirmation of hydraulic categories across the Conjola Lake floodplain will be undertaken as part of the subsequent Floodplain Risk Management Study.

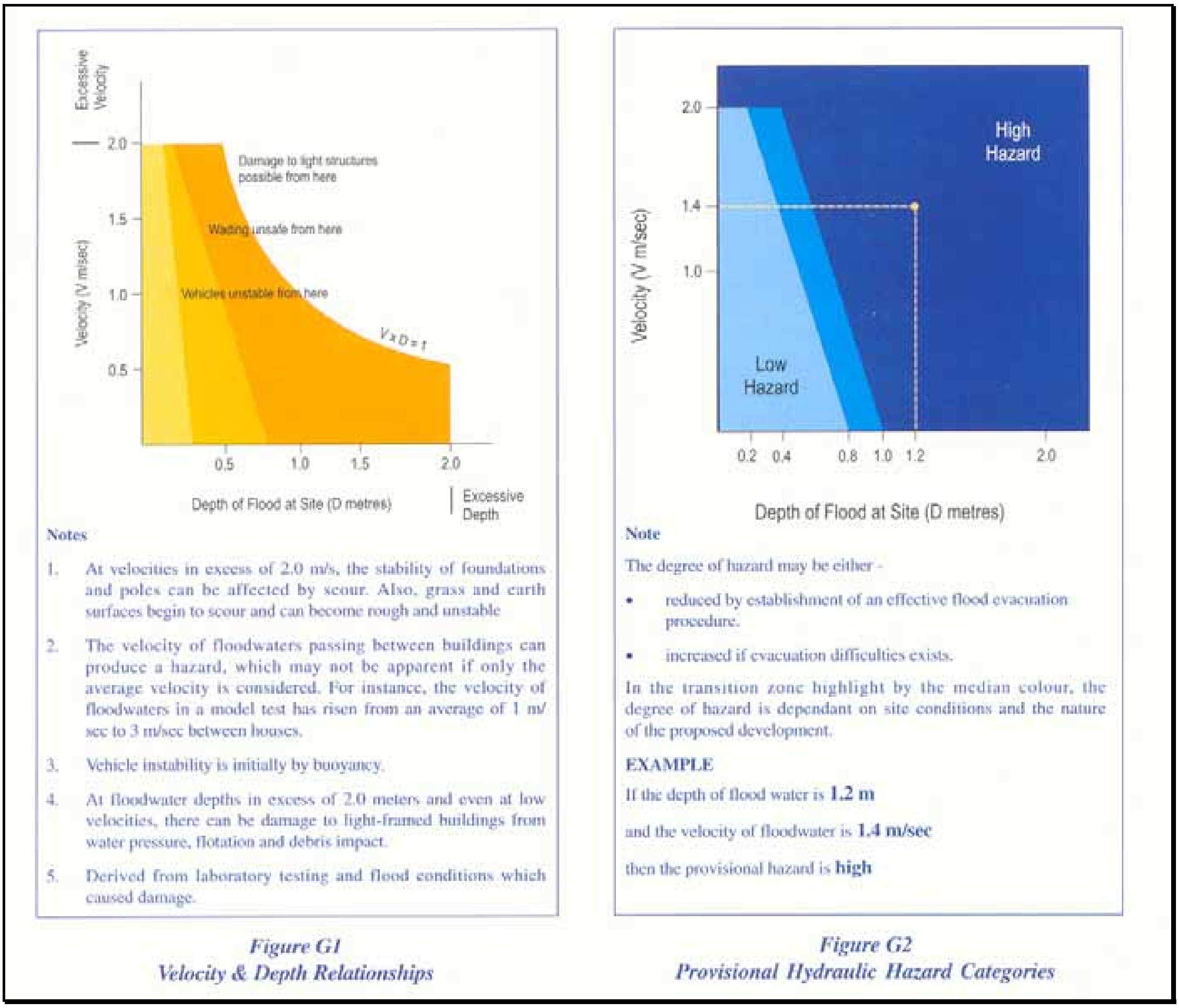

Figure 7-2 Hazard Determination (Source: NSW Floodplain Development Manual, 2005)

7.3 Flood Hydrographs

Water level hydrographs were extracted at two locations for each of the critical design floods:

- "The Steps", which water level is representative of all the Conjola Lake upstream parts; and
- Lake Conjola Entrance Caravan Park, immediately downstream of the Pattimores Creek outlet.

The hydrographs are presented in Figure 7-3 and Figure 7-4.

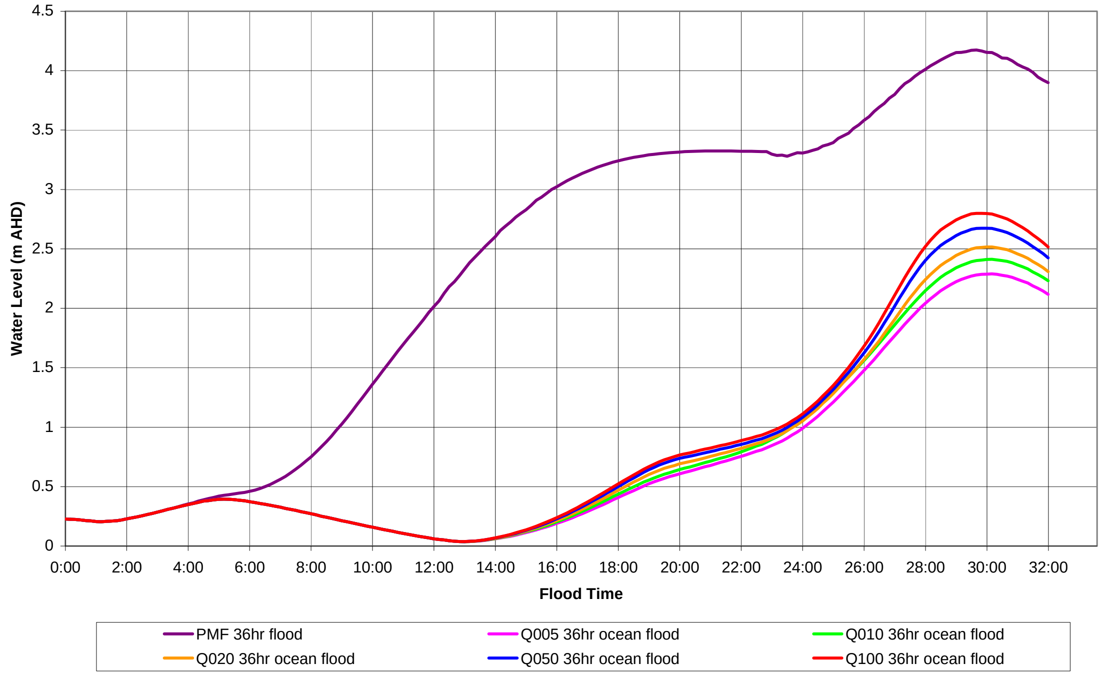

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Figure 7-3 Predicted Design Flood Water Level Hydrographs at "The Steps"

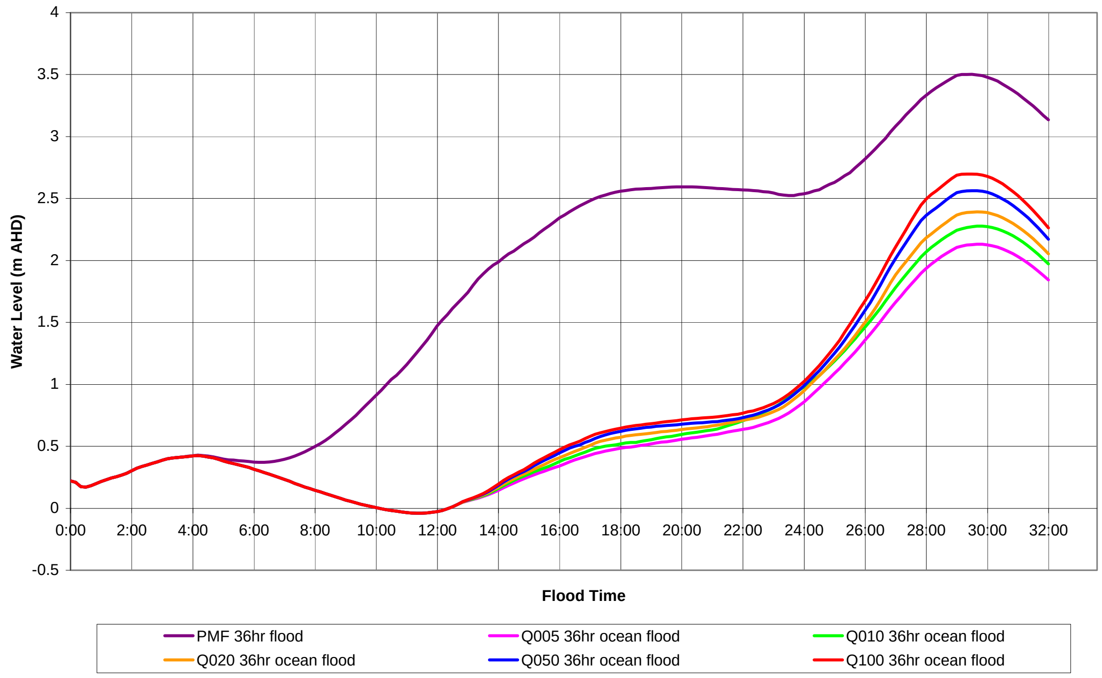

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Figure 7-4 Predicted Design Flood Water Level Hydrographs Downstream of Lake Conjola Caravan Park

7.4 Flooding of Important Evacuation Routes

The only access road to Lake Conjola is the Lake Conjola Entrance Road. Along the 700m of road between the intersection with Milhan St and the intersection with Carroll Av, Lake Conjola Entrance Road contains several low points, which would be affected significantly during design floods. The road profile is presented in Figure 7-5.

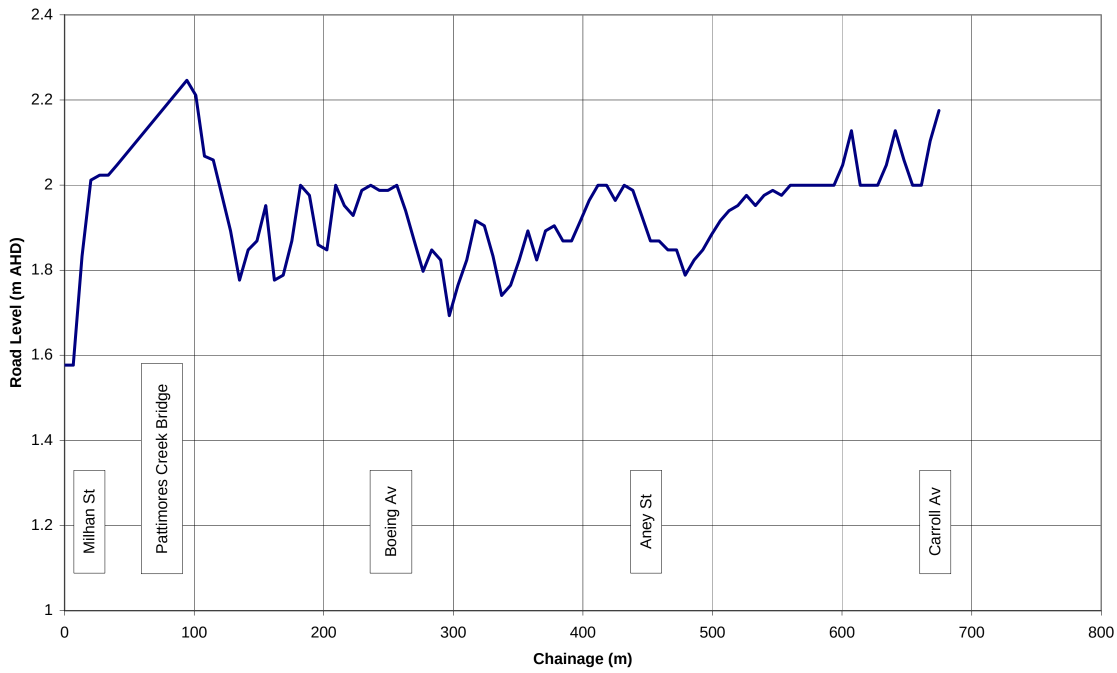

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Figure 7-5 Lake Conjola Entrance Road Longitudinal Profile

Water levels at the Pattimores Creek road bridge reach 2.7 m AHD at the peak of the 1 in 100 year ARI design flood. This implies that the road would be inundated with water depths as high as one metre in places. The design flood hydrographs at the bridge are presented in Figure 7-6. Comparison between Figure 7-5 and Figure 7-6 identifies the threshold of flooding for the different design floods, and the time when the road gets inundated for varying depths.

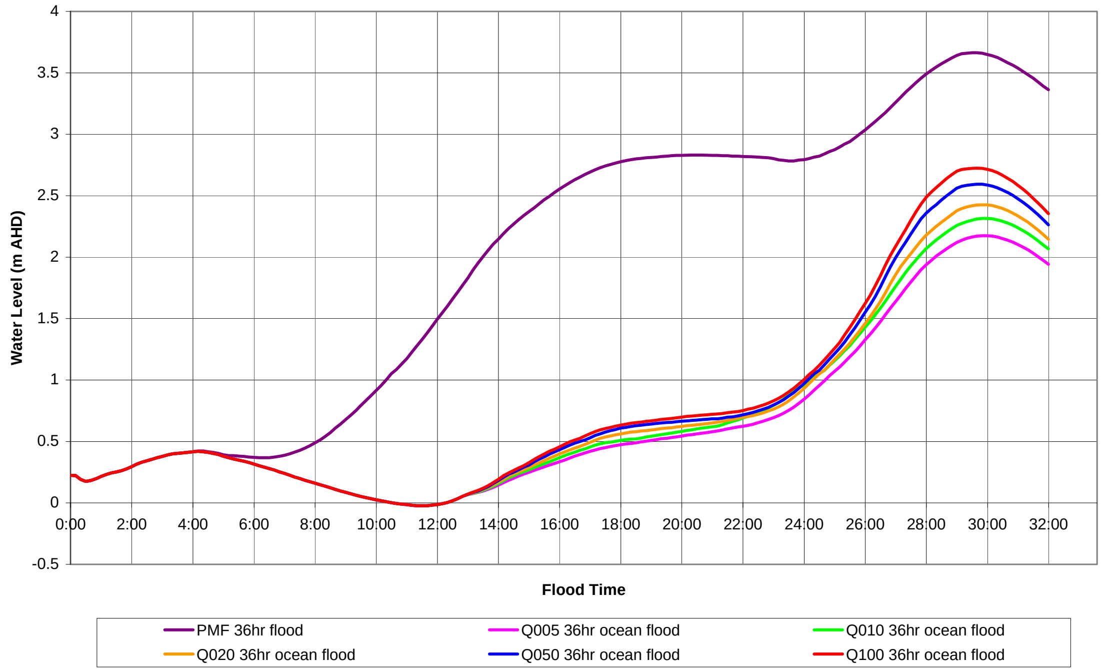

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Figure 7-6 Predicted Design Flood Water Level Hydrographs at Pattimores Creek Bridge

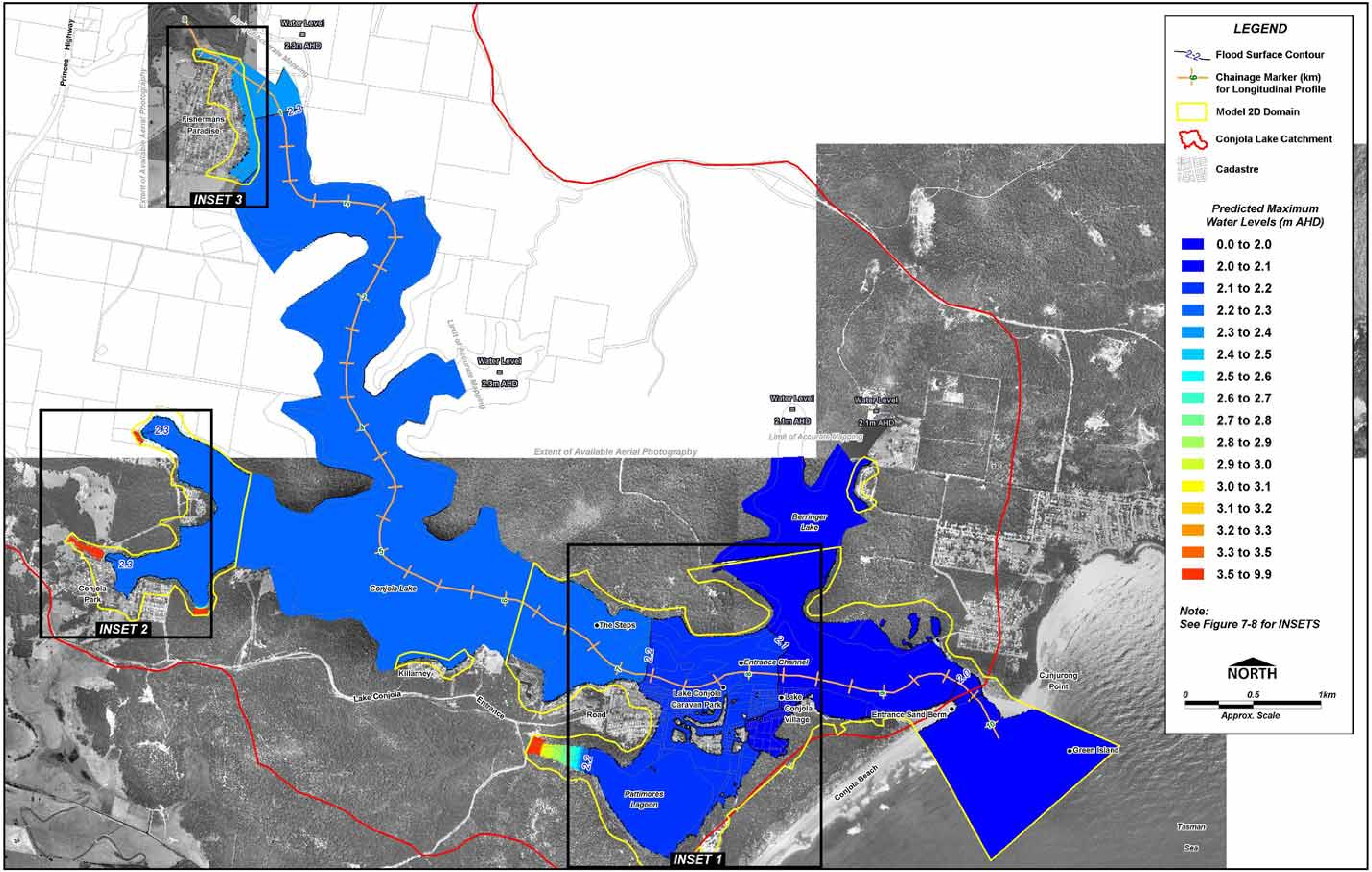

Figure 7-7 Maximum 5 Year ARI Design Flood Water Levels

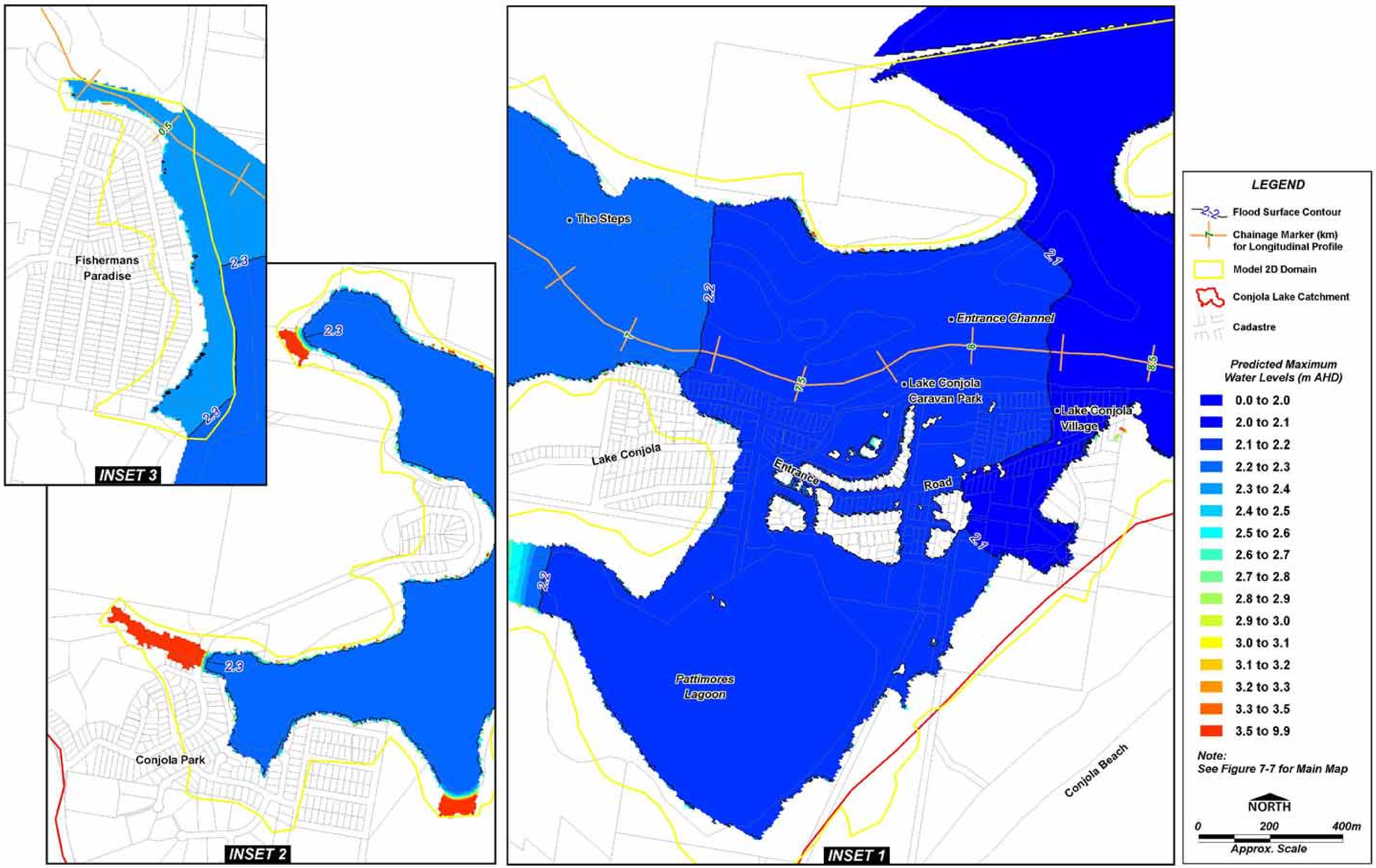

Figure 7-8 Maximum 5 Year ARI Design Flood Water Levels (insets)

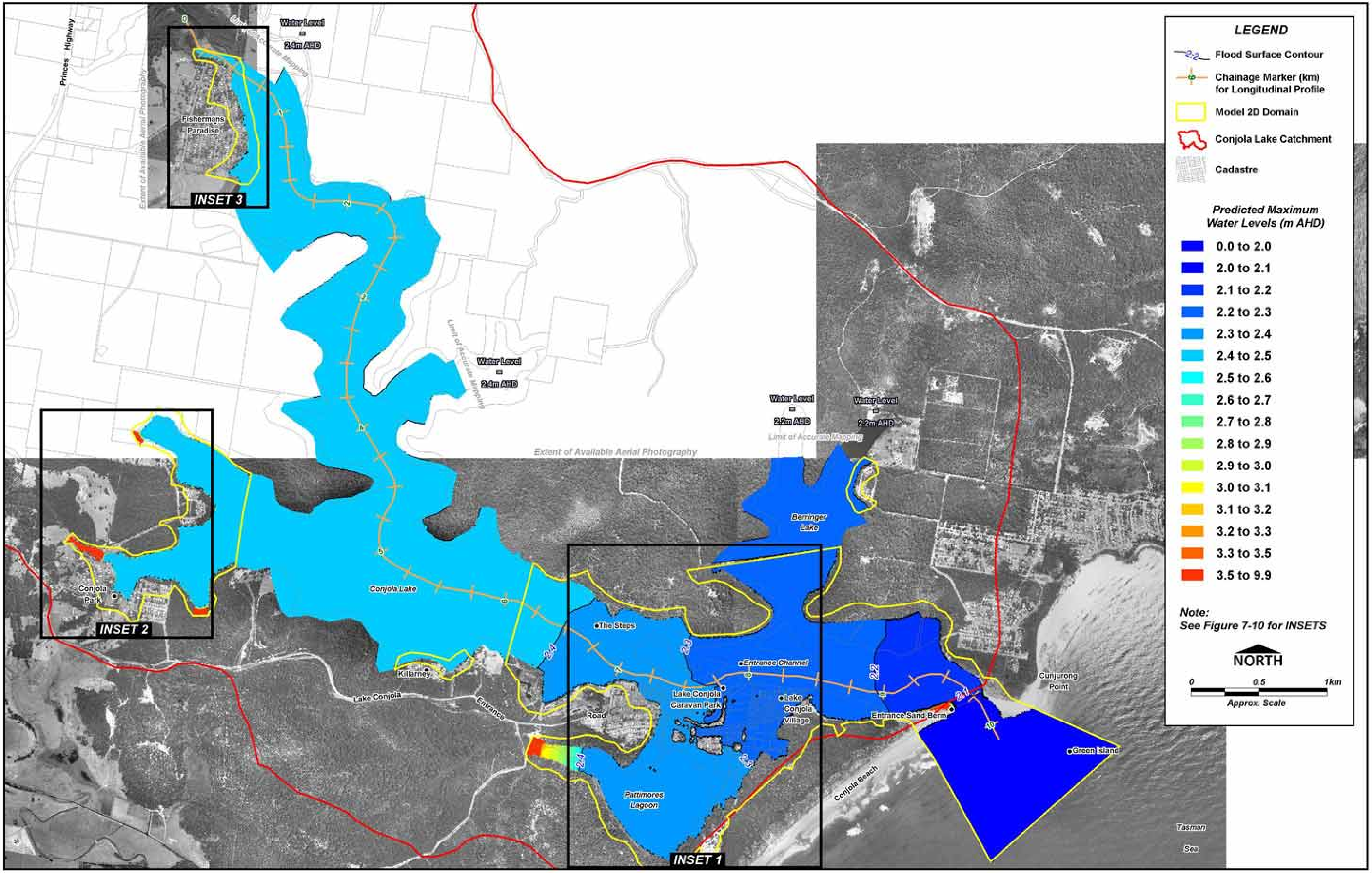

Figure 7-9 Maximum 10 Year ARI Design Flood Water Levels

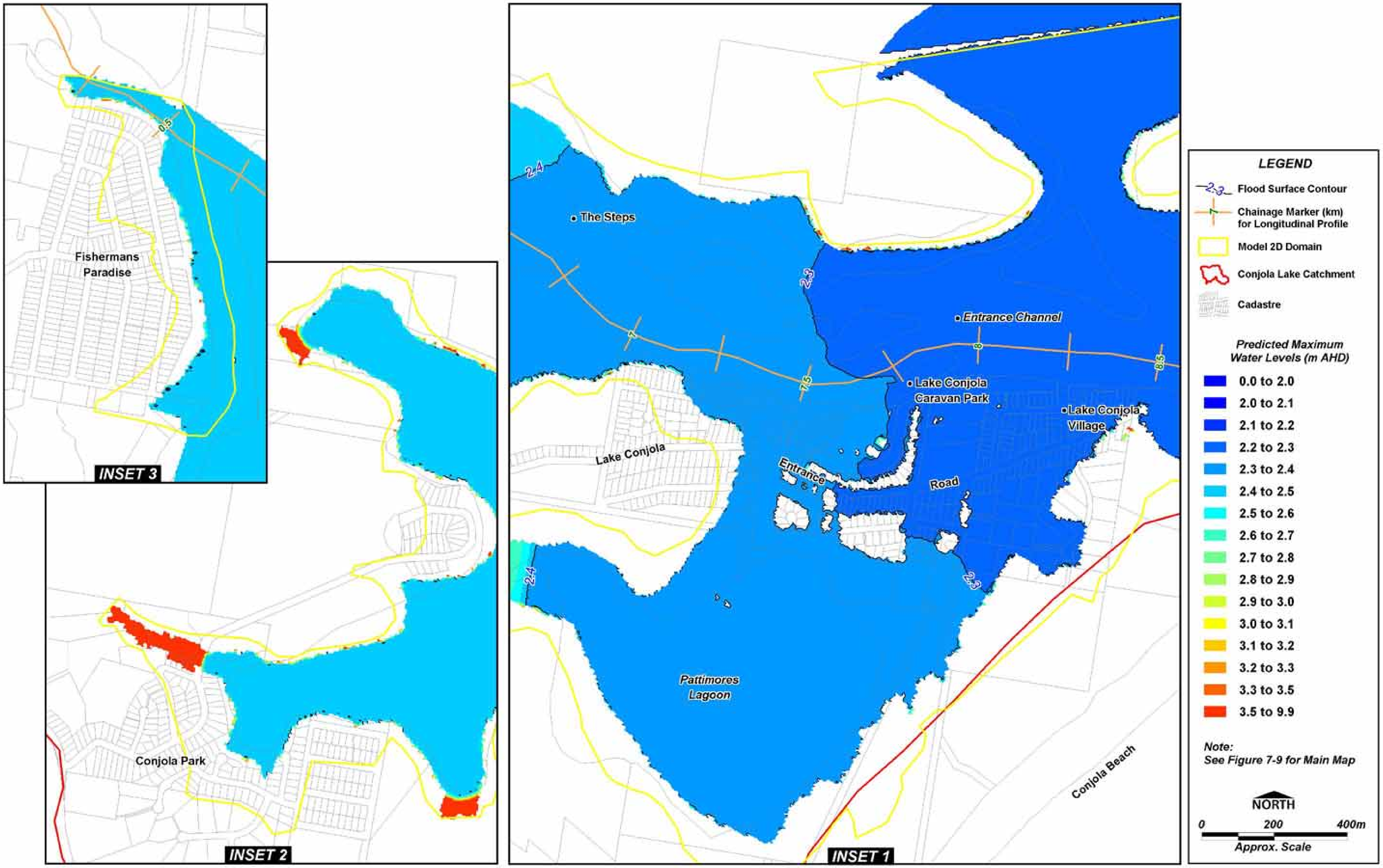

Figure 7-10 Maximum 10 Year ARI Design Flood Water Levels (insets)

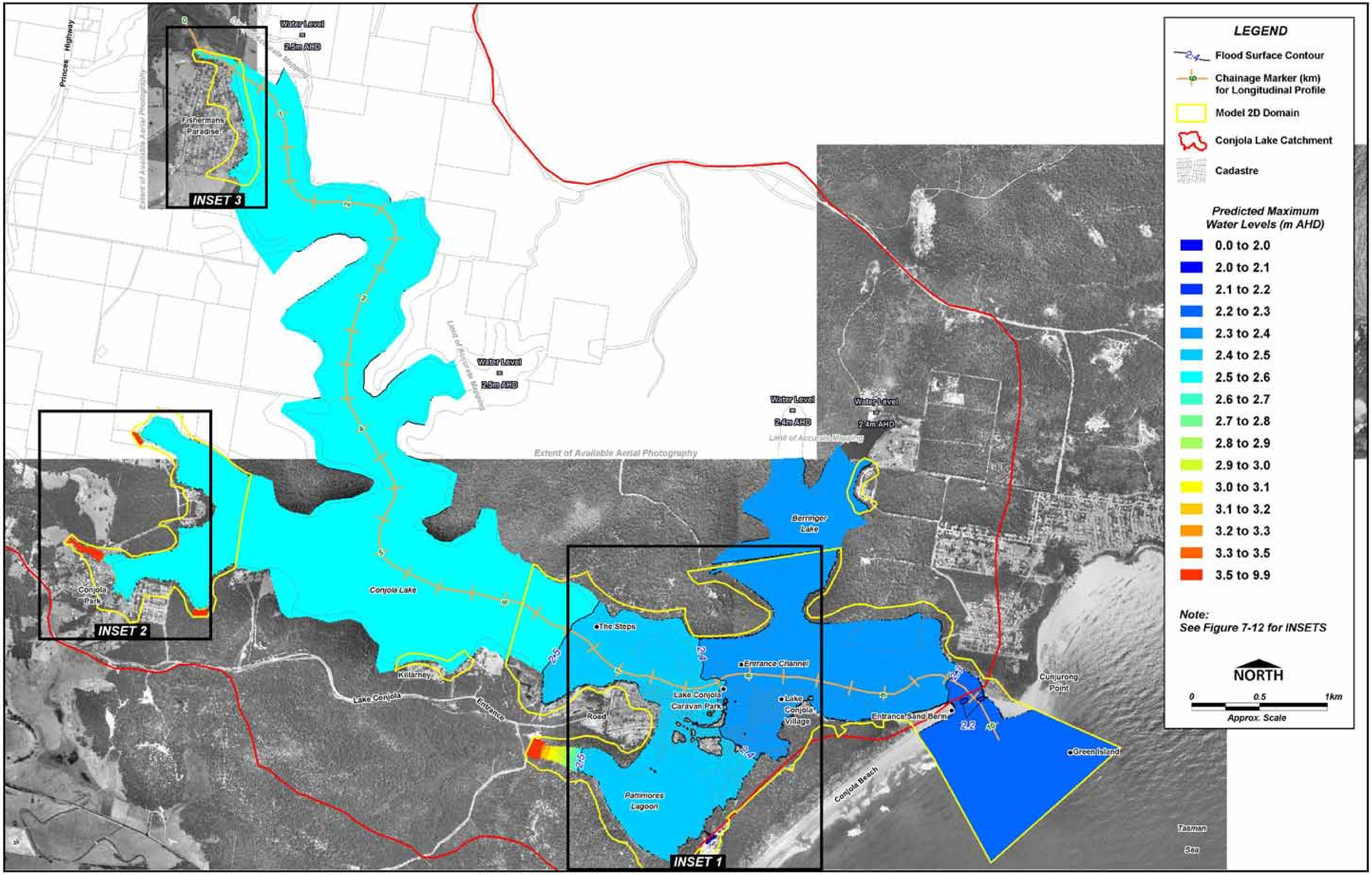

Figure 7-11 Maximum 20 Year ARI Design Flood Water Levels

BMT WBM

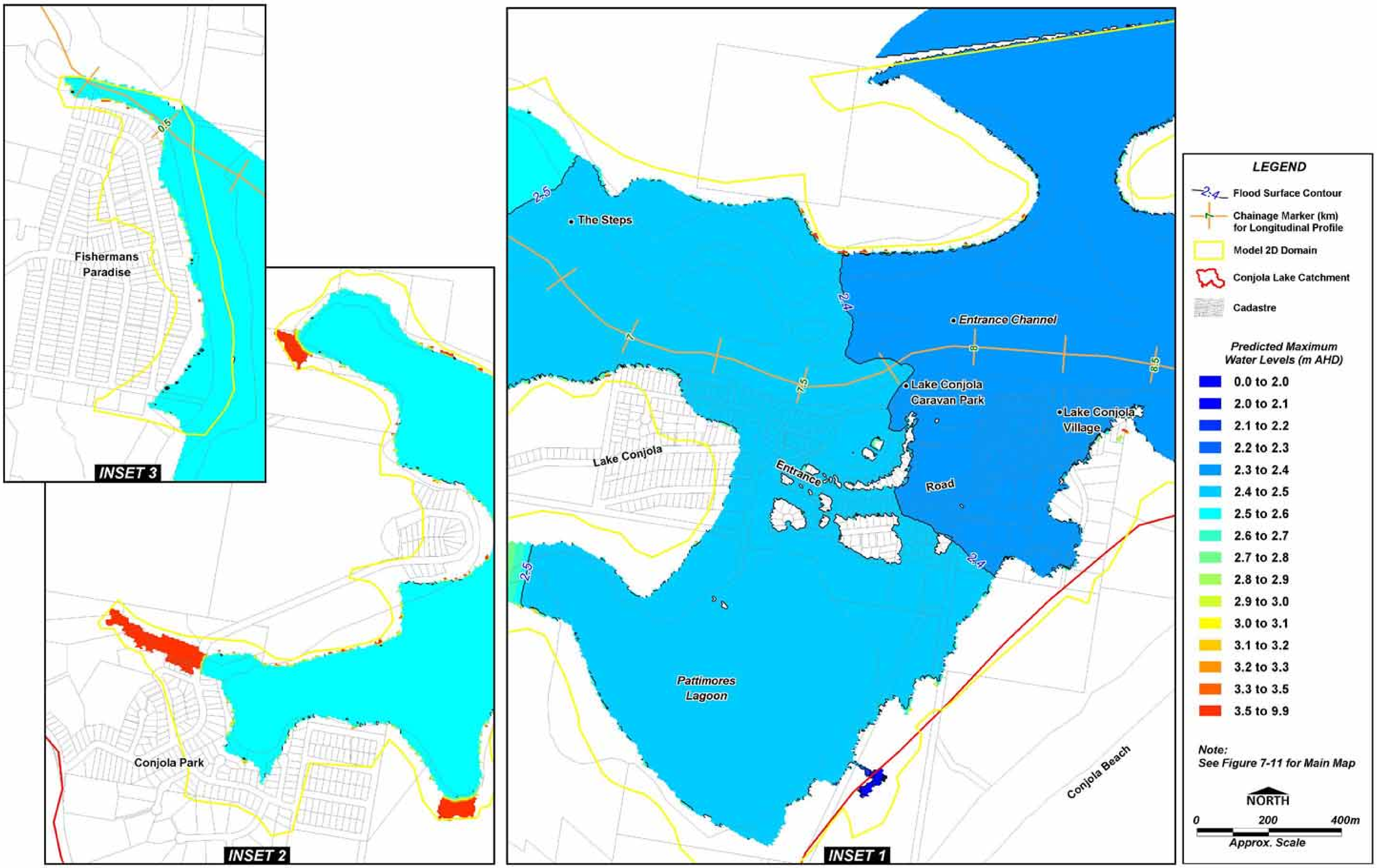

Figure 7-12 Maximum 20 Year ARI Design Flood Water Levels (insets)

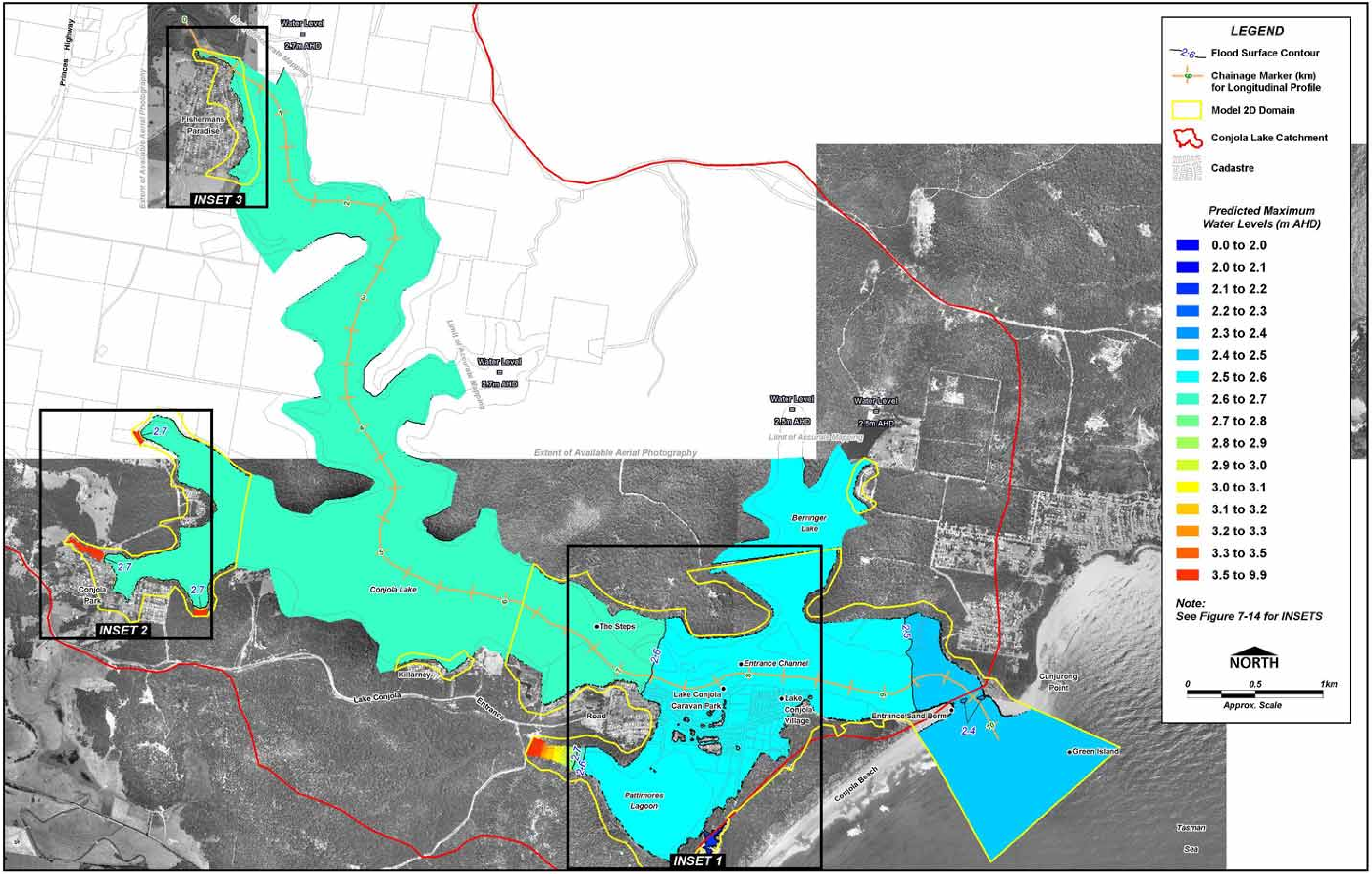

Figure 7-13 Maximum 50 Year ARI Design Flood Water Levels

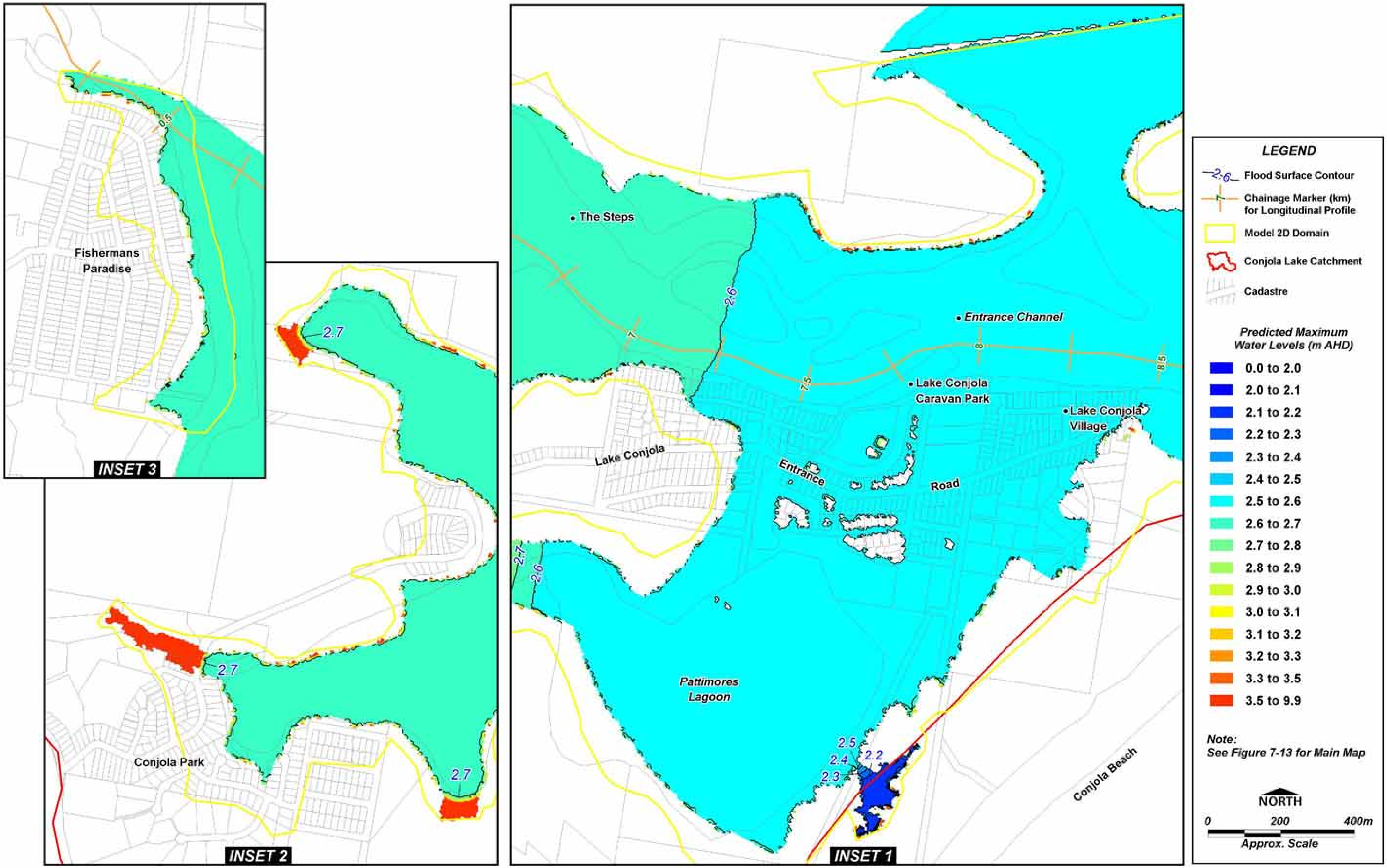

Figure 7-14 Maximum 50 Year ARI Design Flood Water Levels (insets)

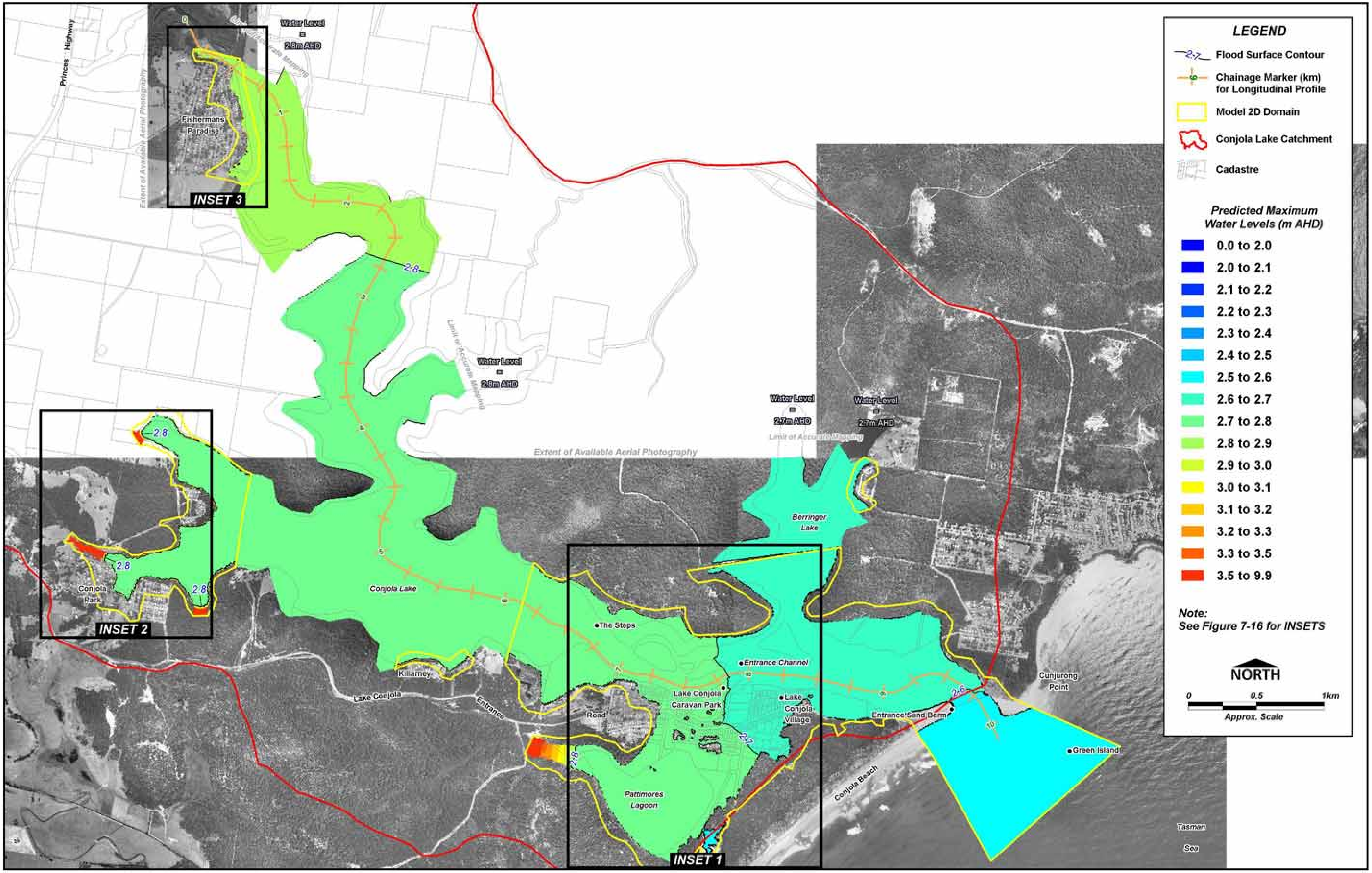

Figure 7-15 Maximum 100 Year ARI Design Flood Water Levels

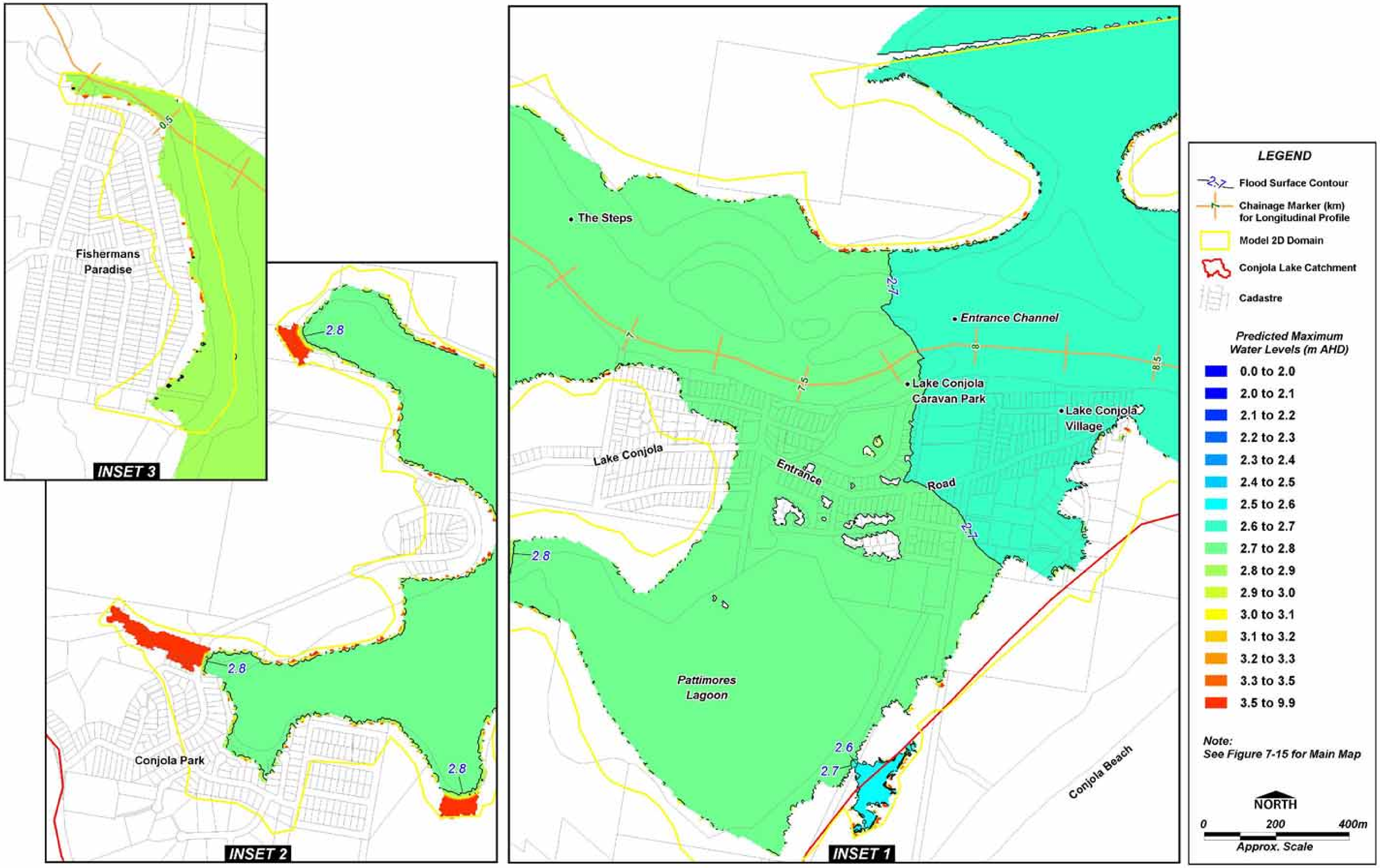

Figure 7-16 Maximum 100 Year ARI Design Flood Water Levels (insets)

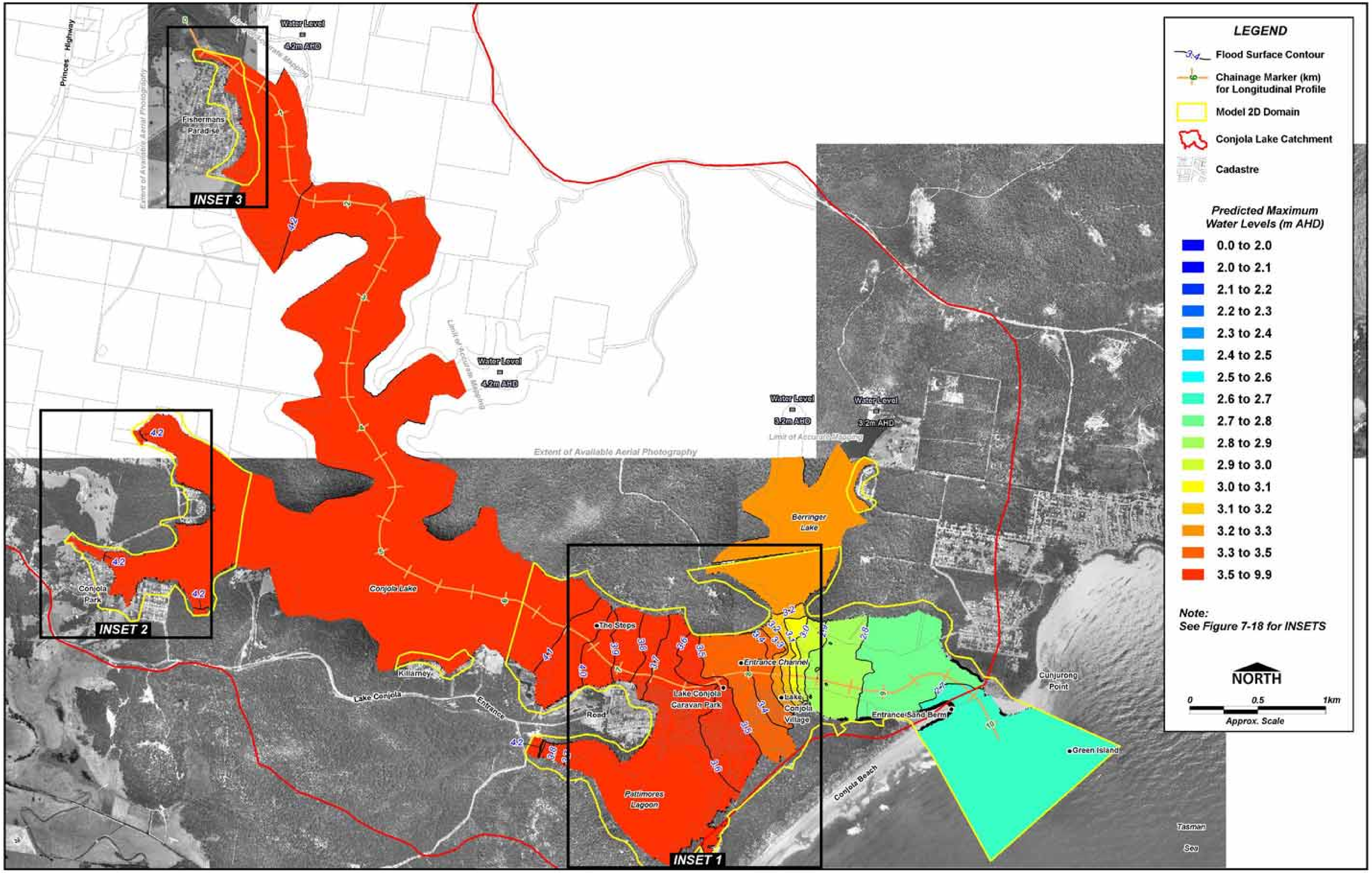

Figure 7-17 Maximum PMF Design Flood Water Levels

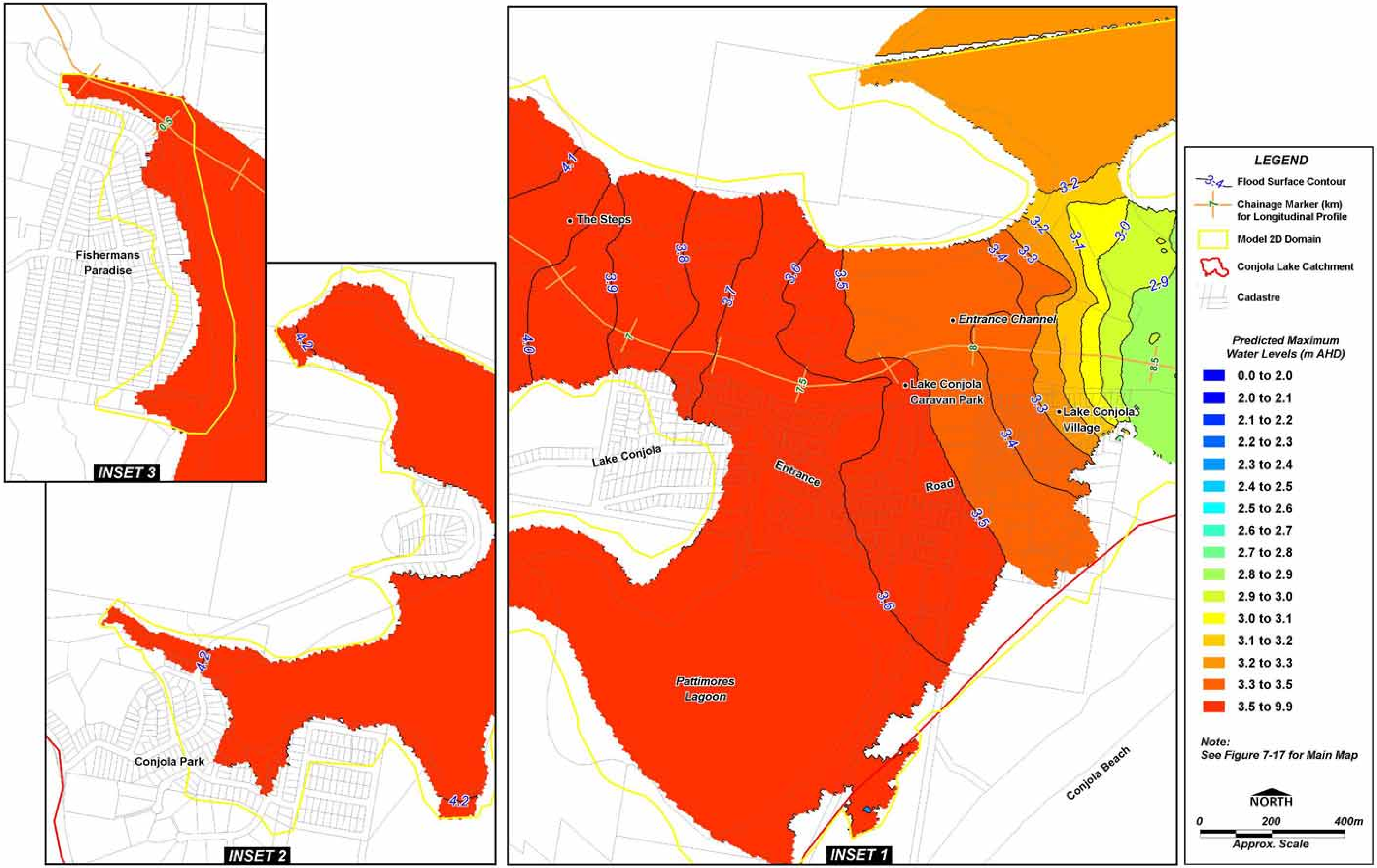

Figure 7-18 Maximum PMF Design Flood Water Levels (insets)

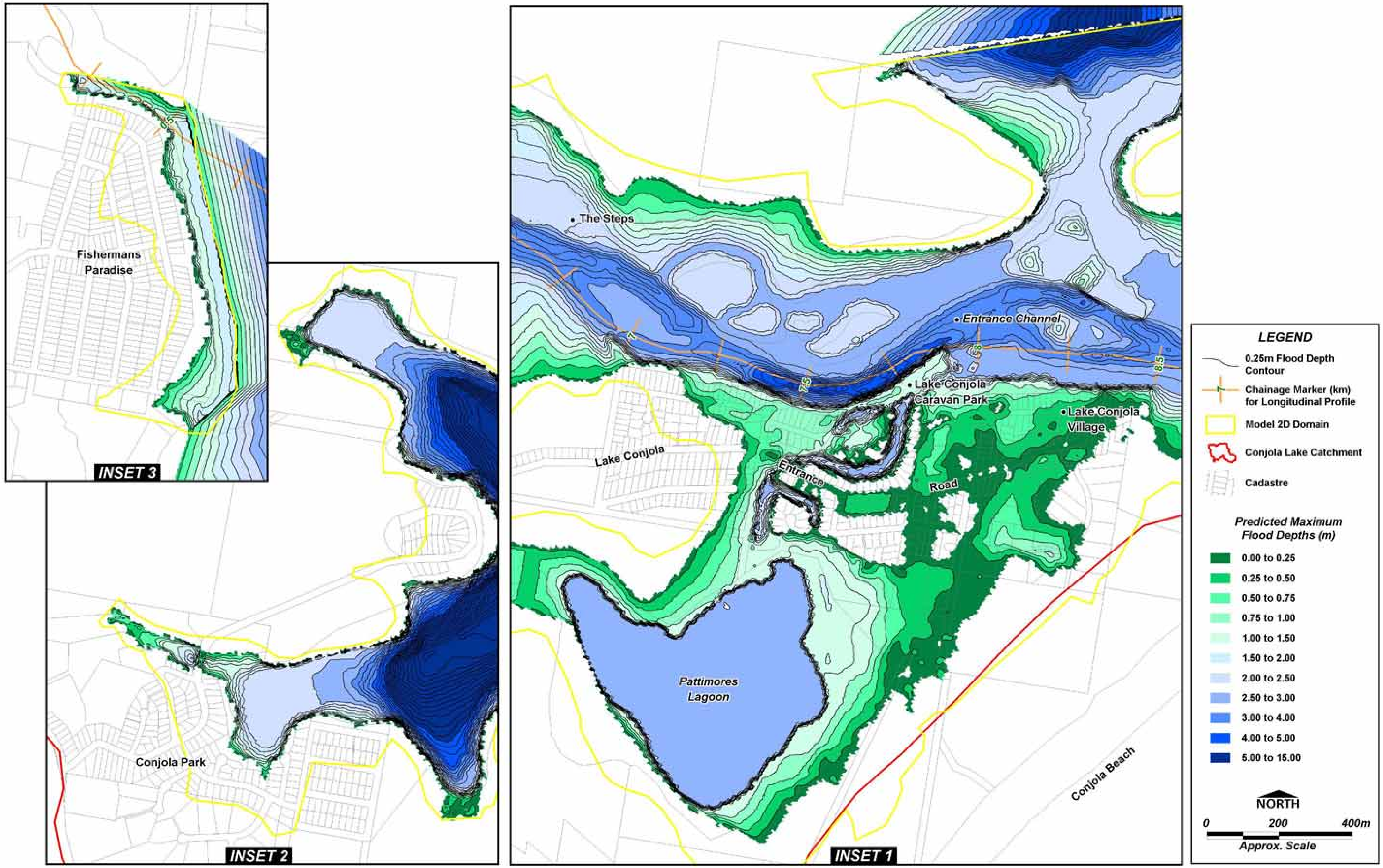

Figure 7-19 Maximum 5 Year ARI Design Flood Depths

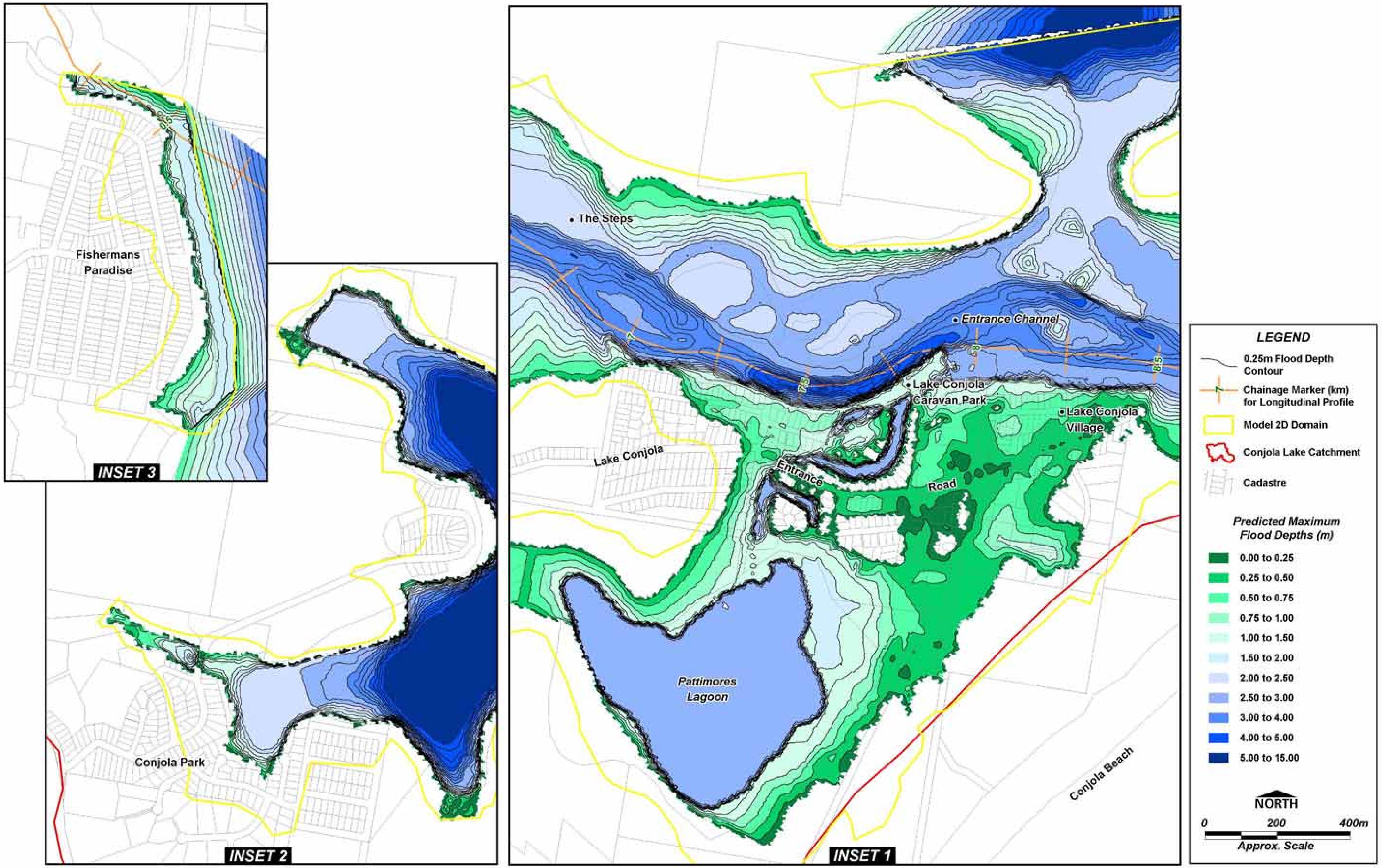

Figure 7-20 Maximum 10 Year ARI Design Flood Depths

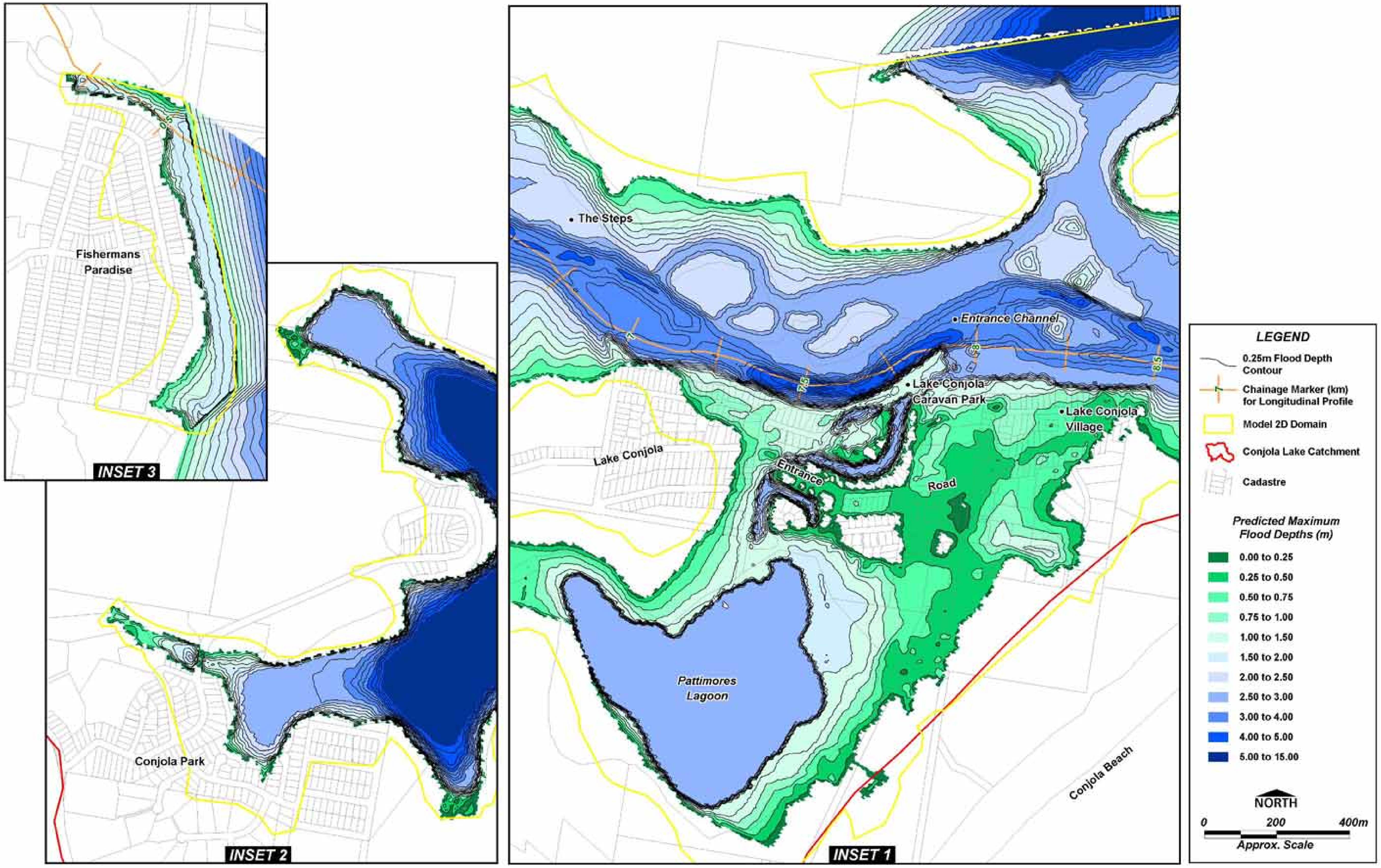

Figure 7-21 Maximum 20 Year ARI Design Flood Depths

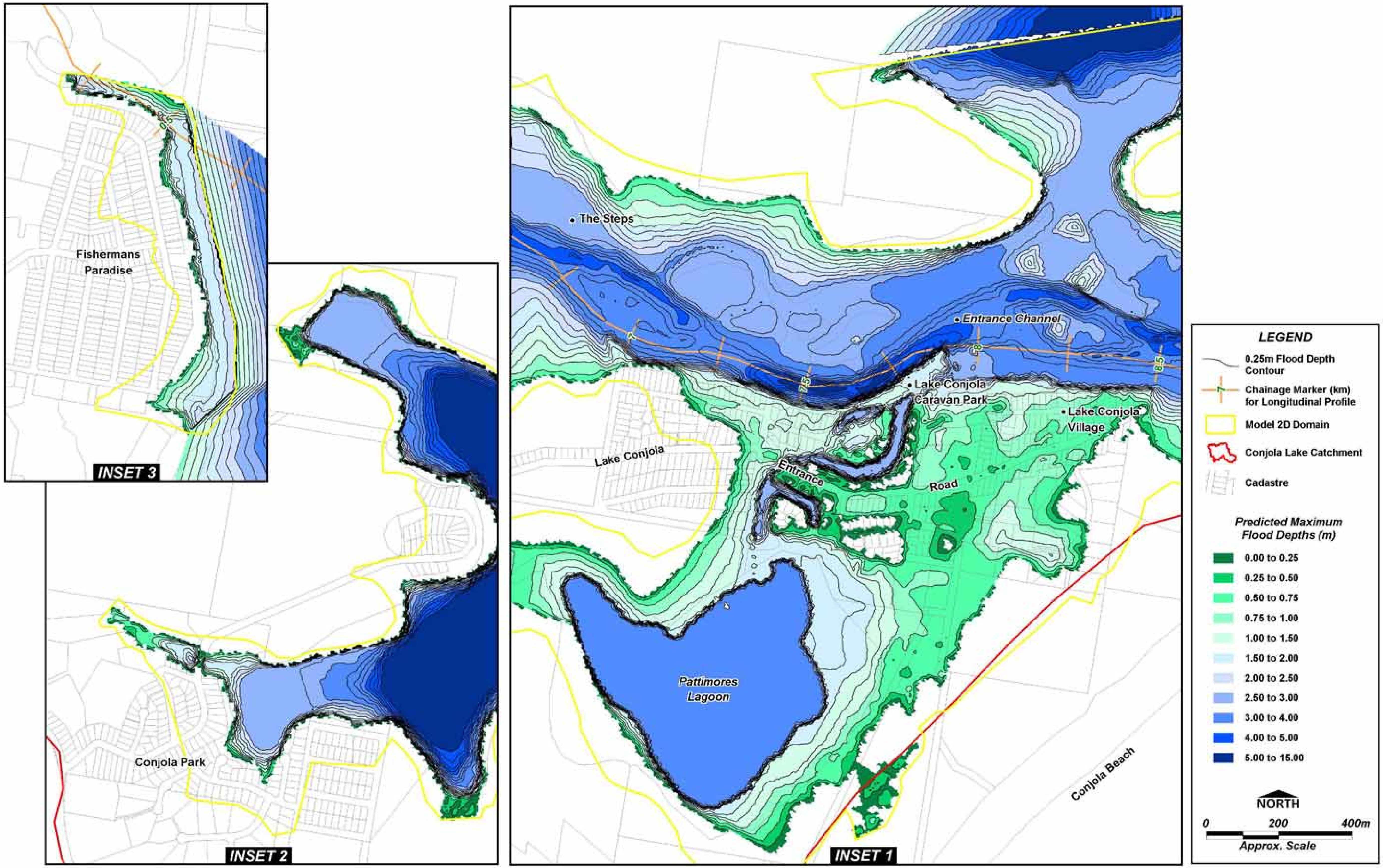

Figure 7-22 Maximum 50 Year ARI Design Flood Depths

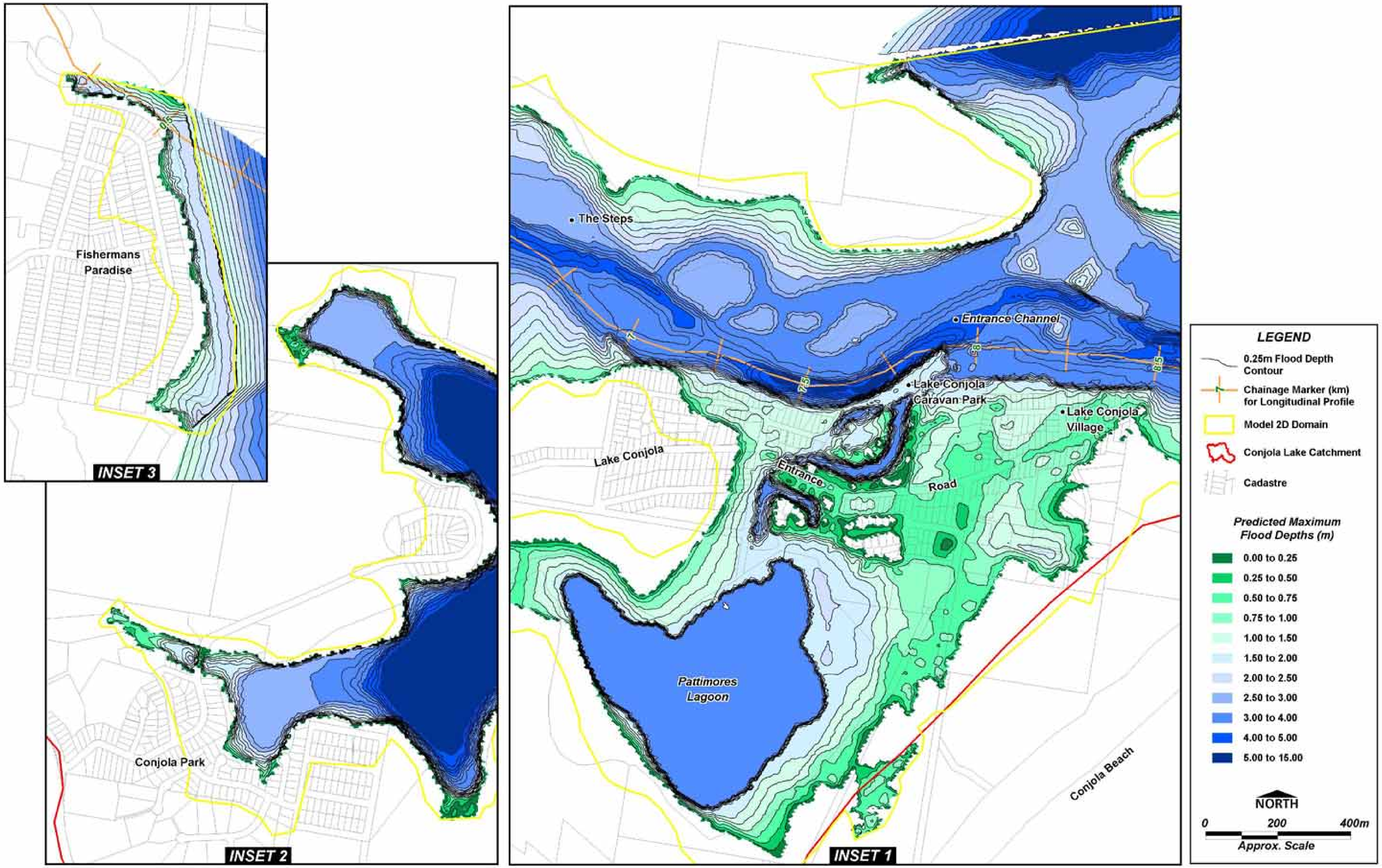

Figure 7-23 Maximum 100 Year ARI Design Flood Depths

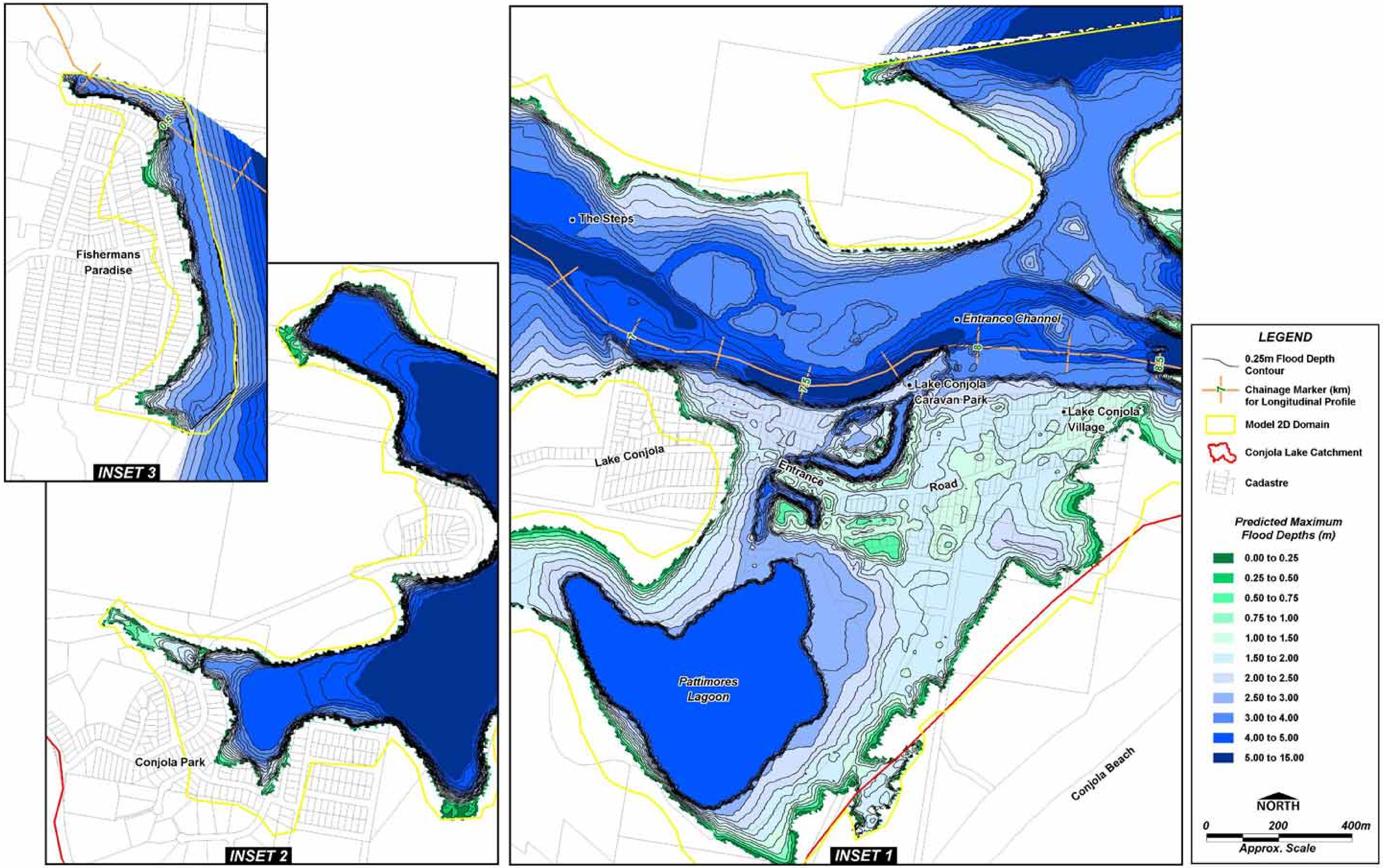

Figure 7-24 Maximum PMF Year ARI Design Flood Depths

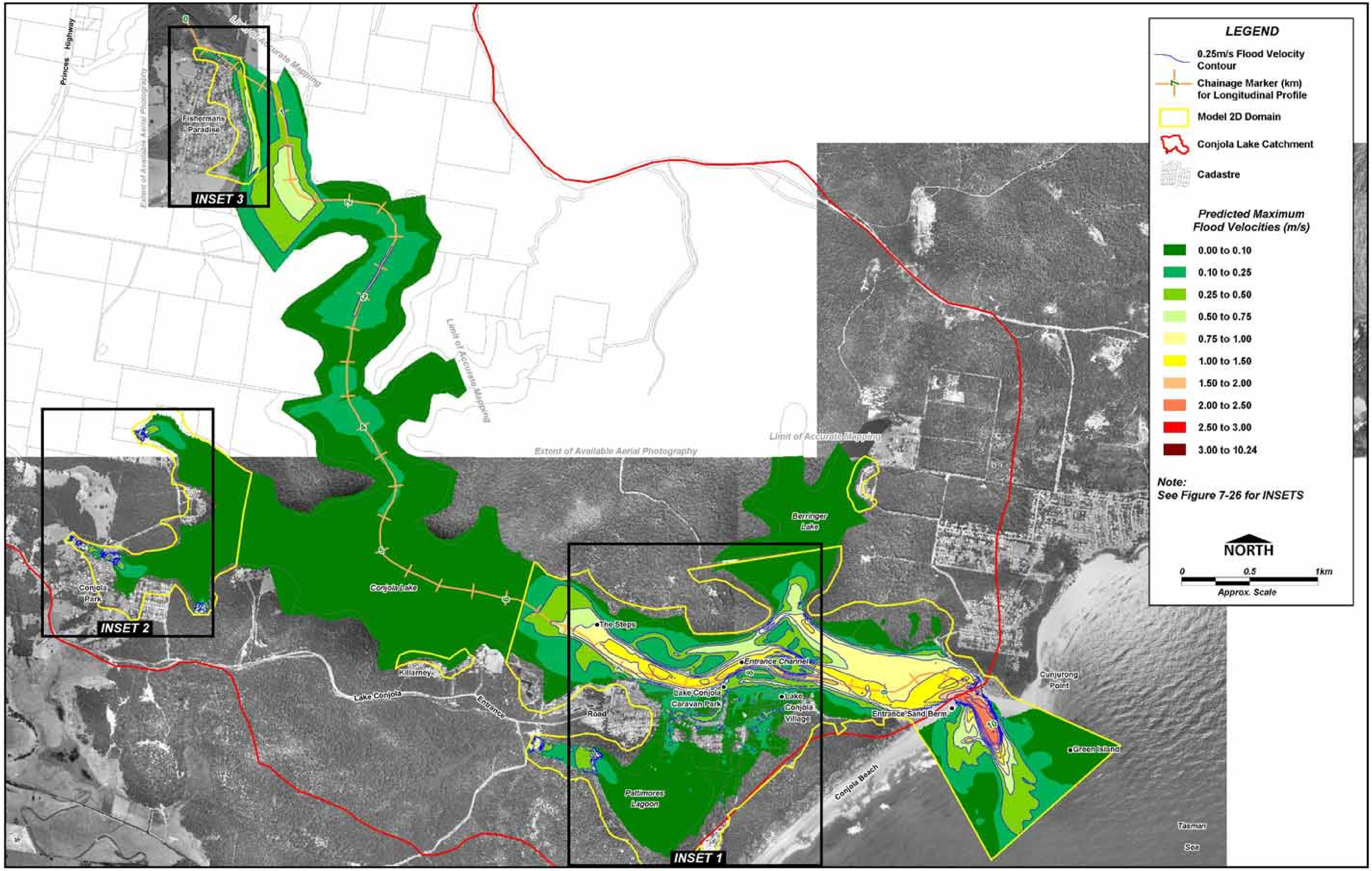

Figure 7-25 Maximum 5 Year ARI Design Flood Velocities

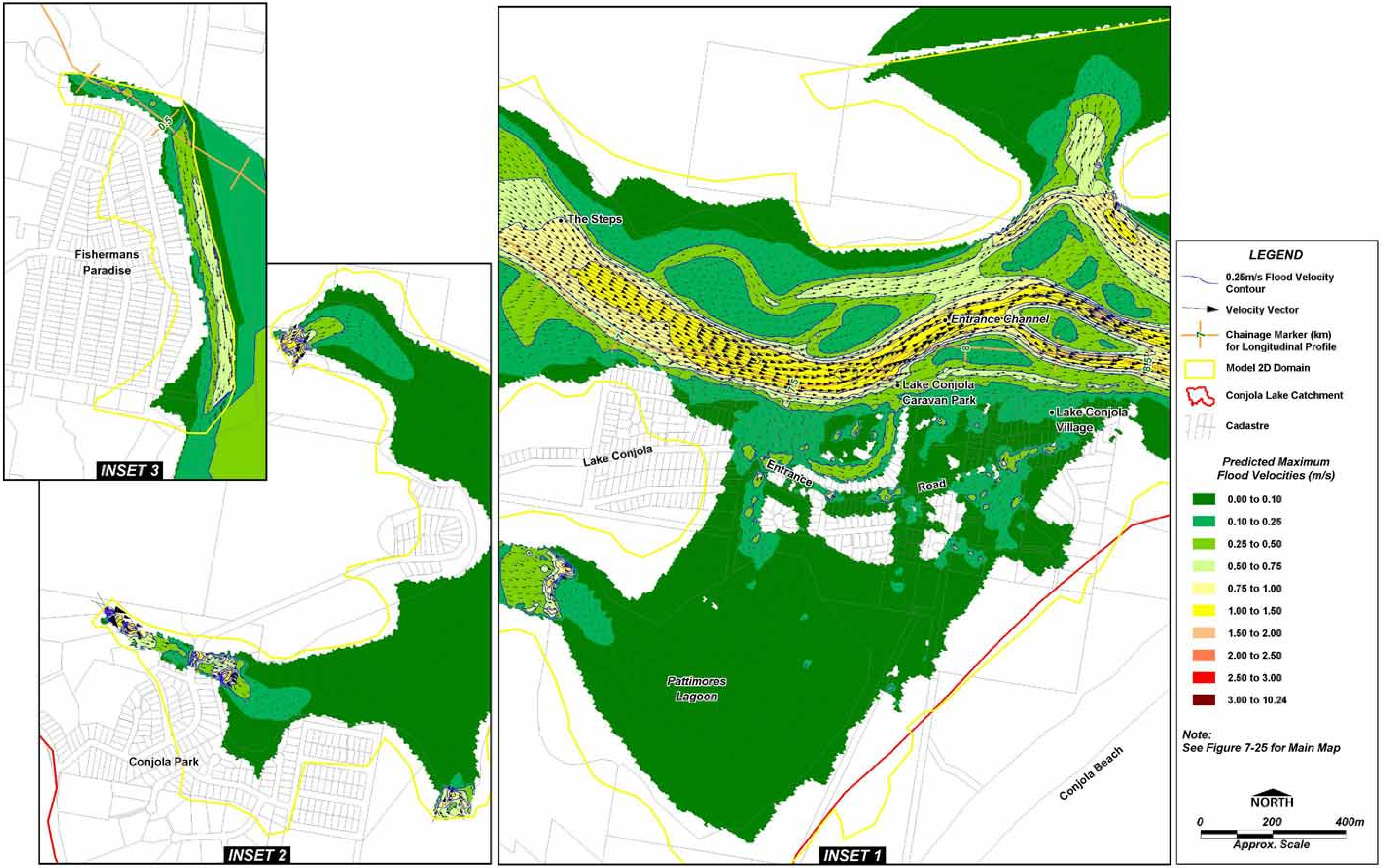

Figure 7-26 Maximum 5 Year ARI Design Flood Velocities (insets)

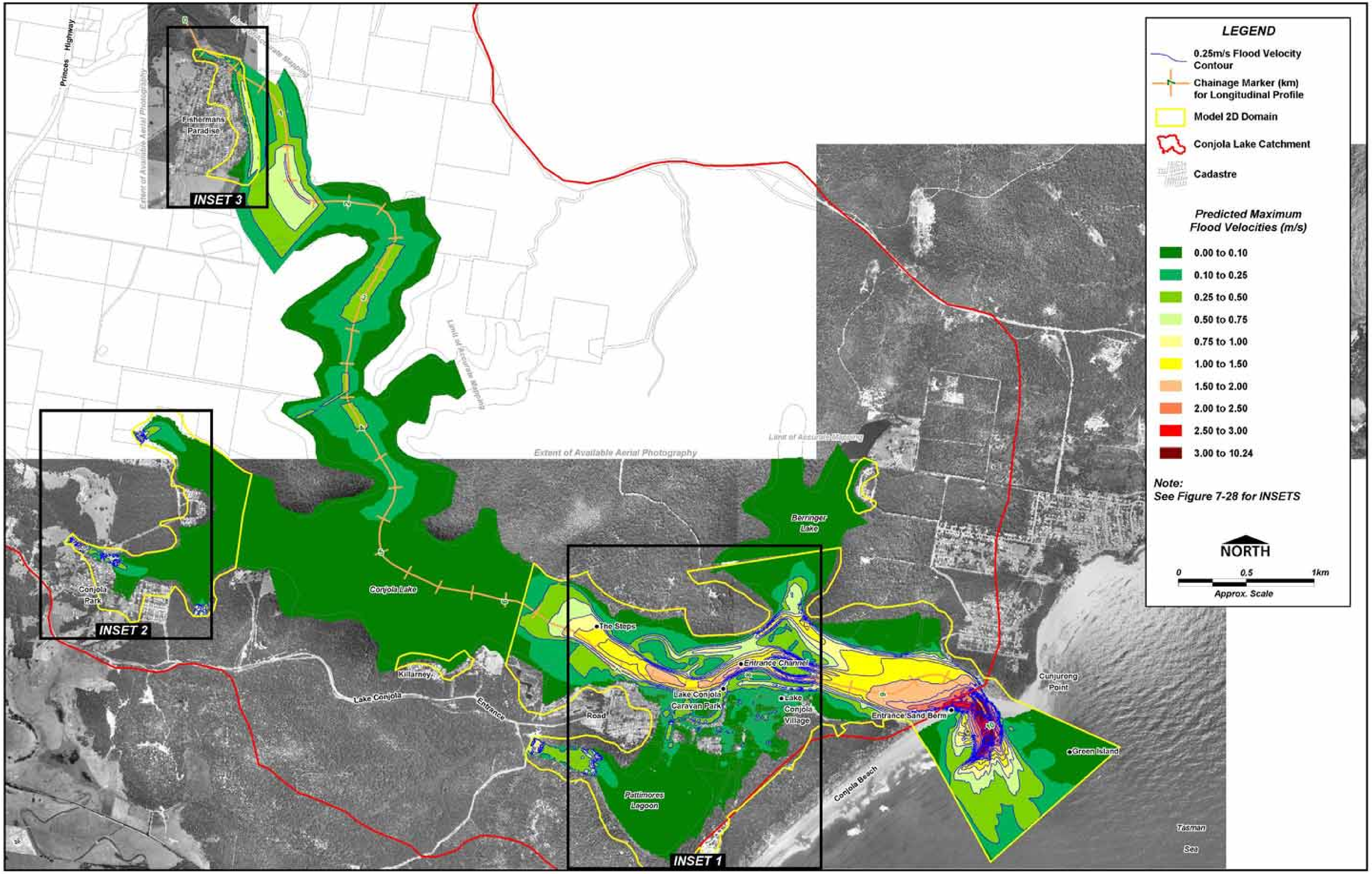

Figure 7-27 Maximum 10 Year ARI Design Flood Velocities

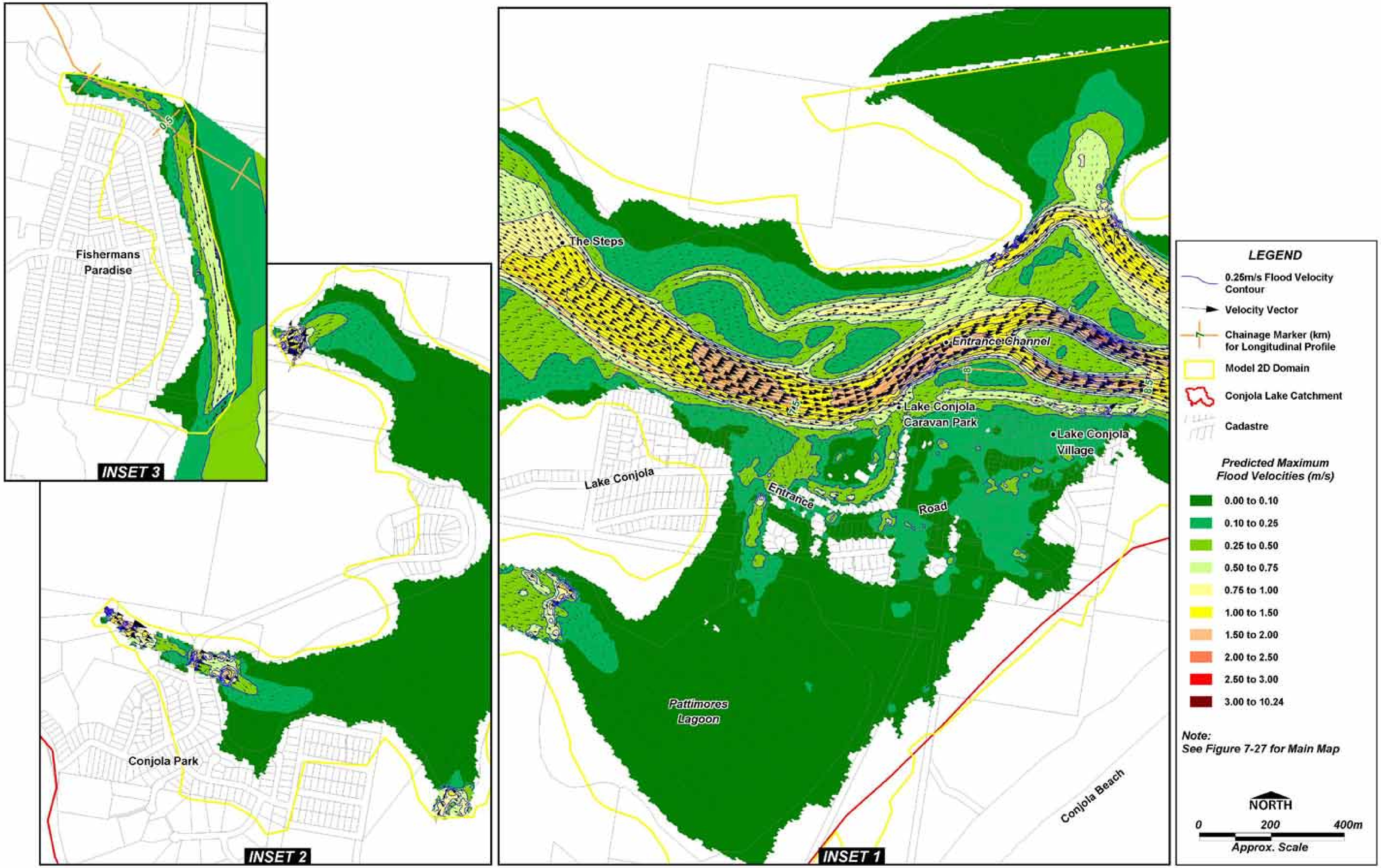

Figure 7-28 Maximum 10 Year ARI Design Flood Velocities (insets)

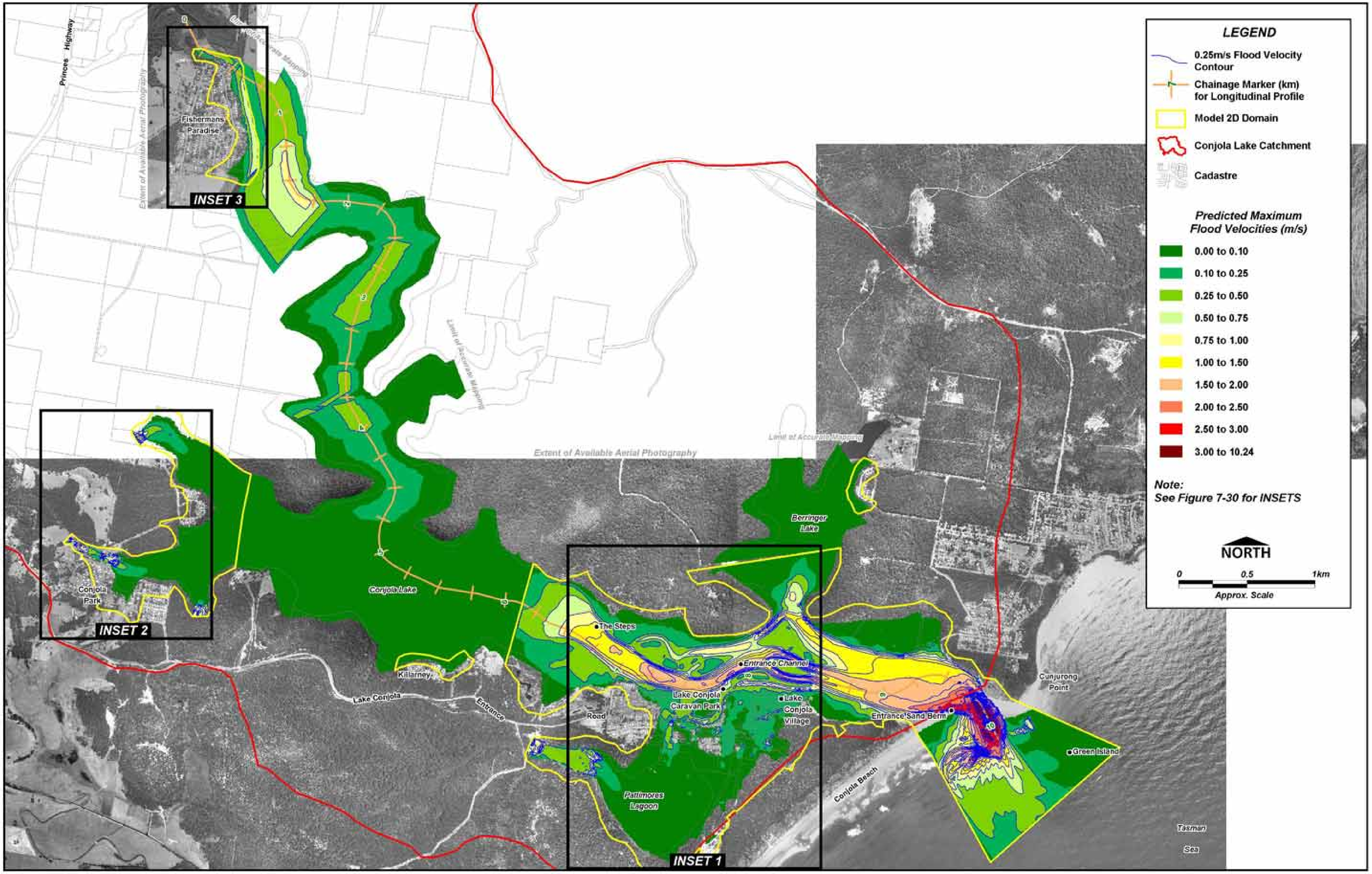

Figure 7-29 Maximum 20 Year ARI Design Flood Velocities

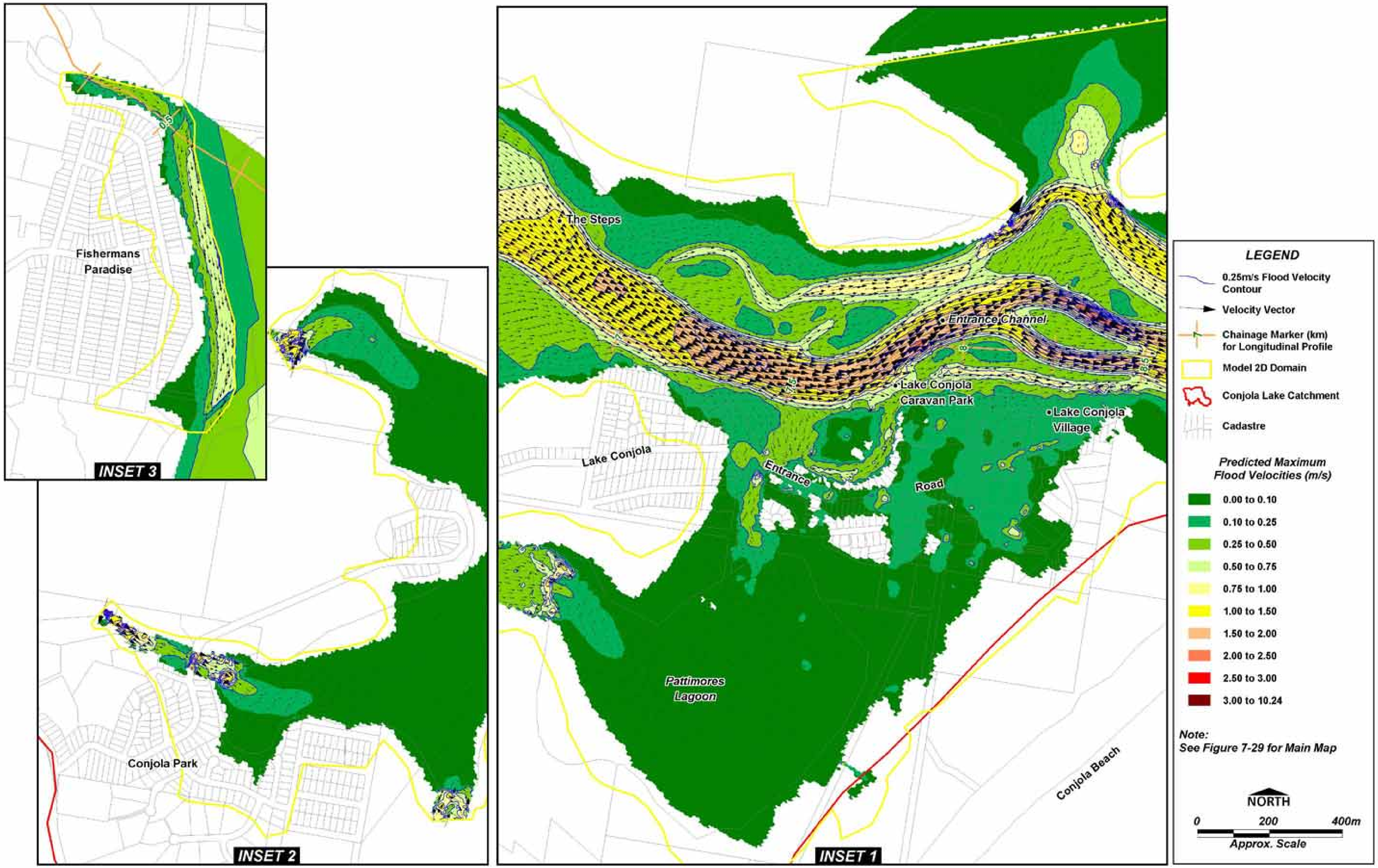

Figure 7-30 Maximum 20 Year ARI Design Flood Velocities (insets)

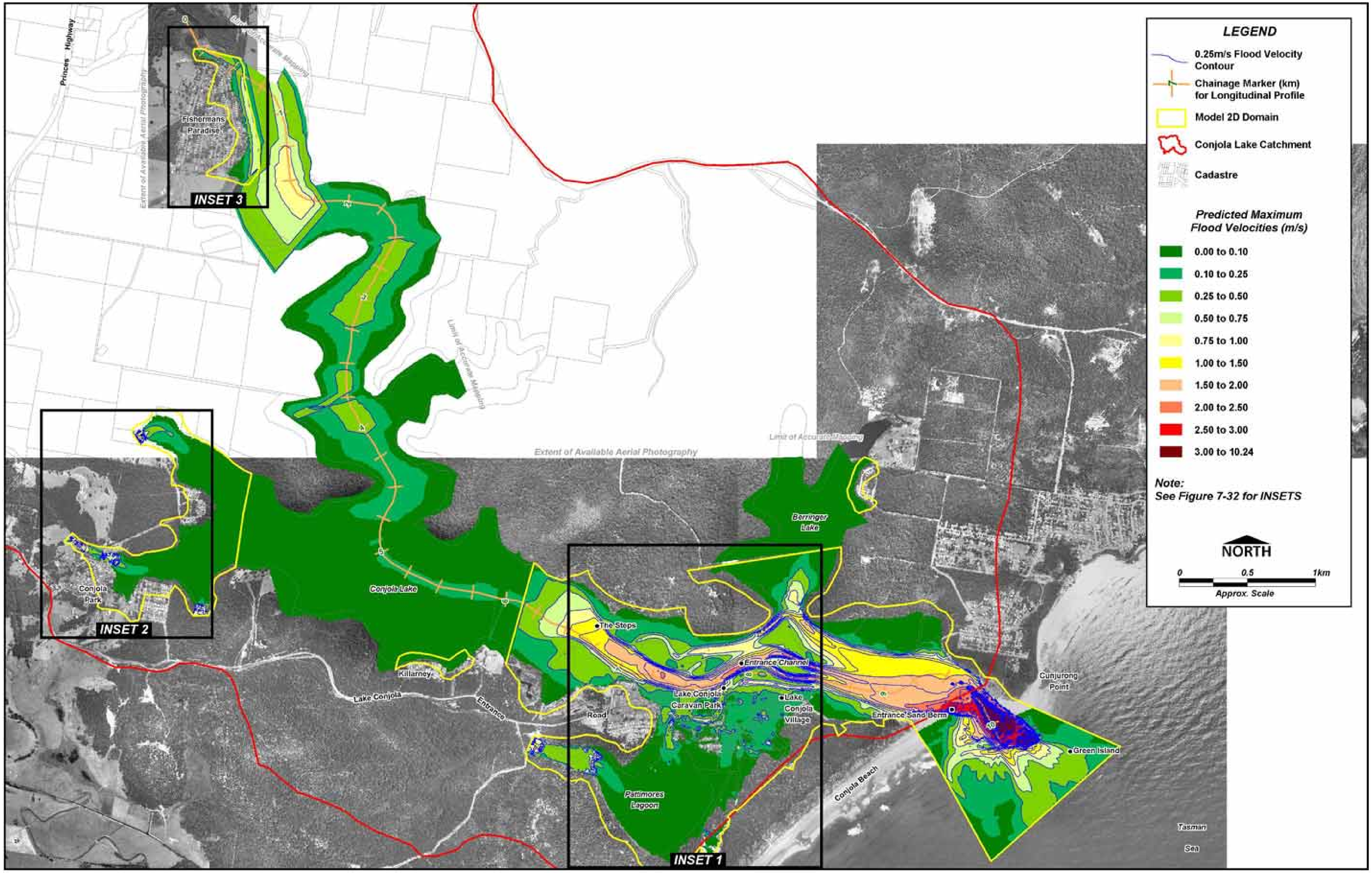

Figure 7-31 Maximum 50 Year ARI Design Flood Velocities

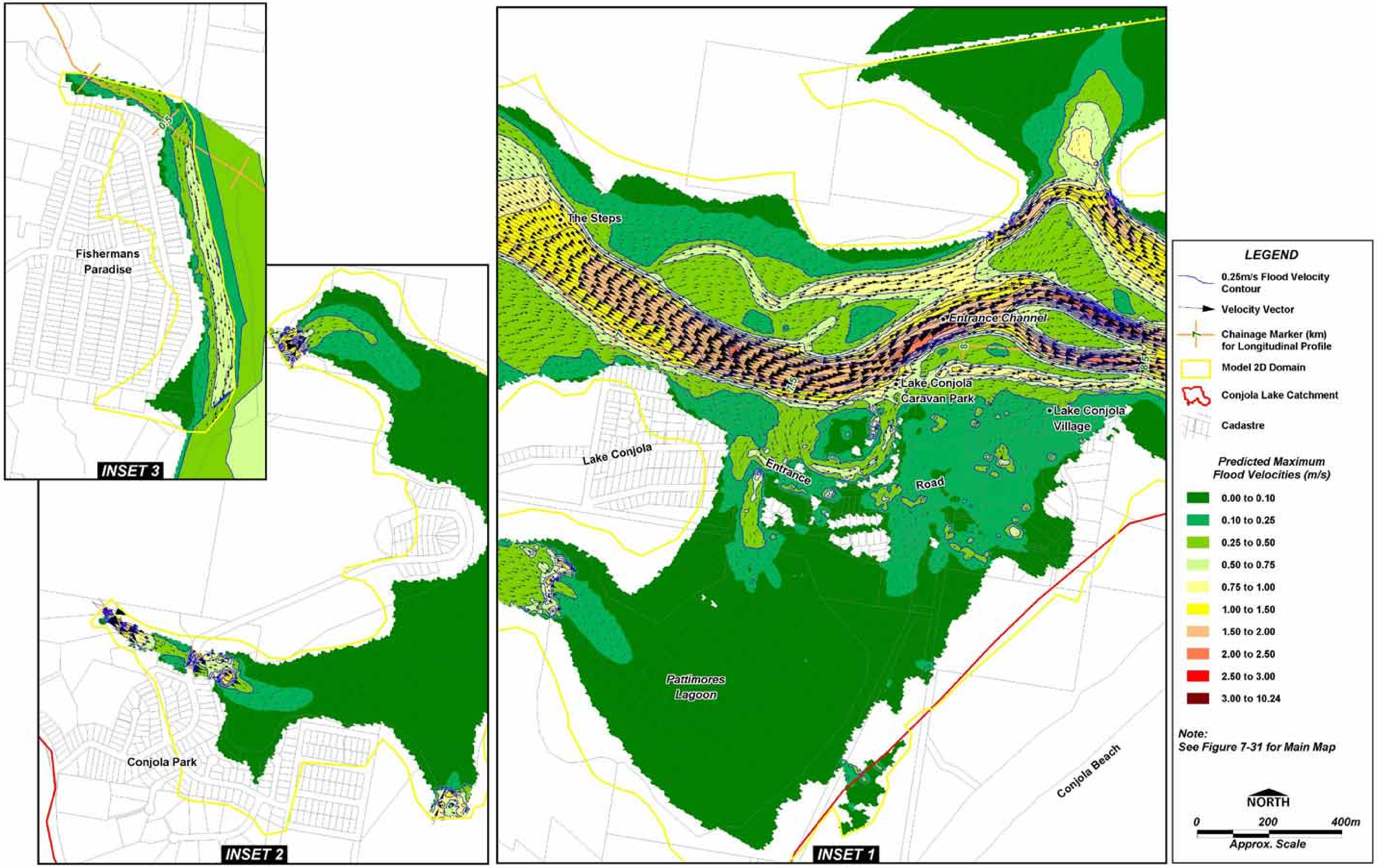

Figure 7-32 Maximum 50 Year ARI Design Flood Velocities (insets)

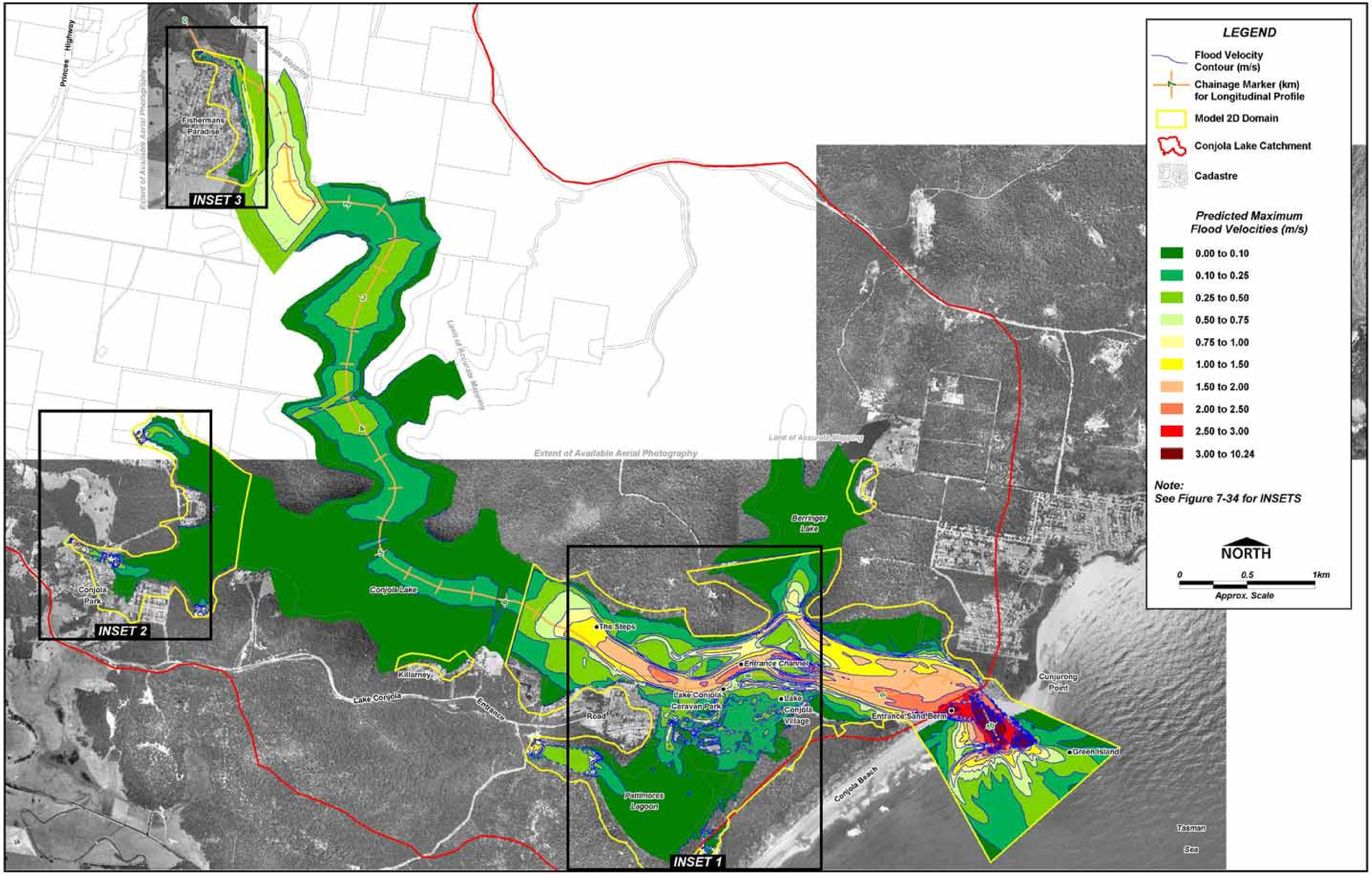

Figure 7-33 Maximum 100 Year ARI Design Flood Velocities

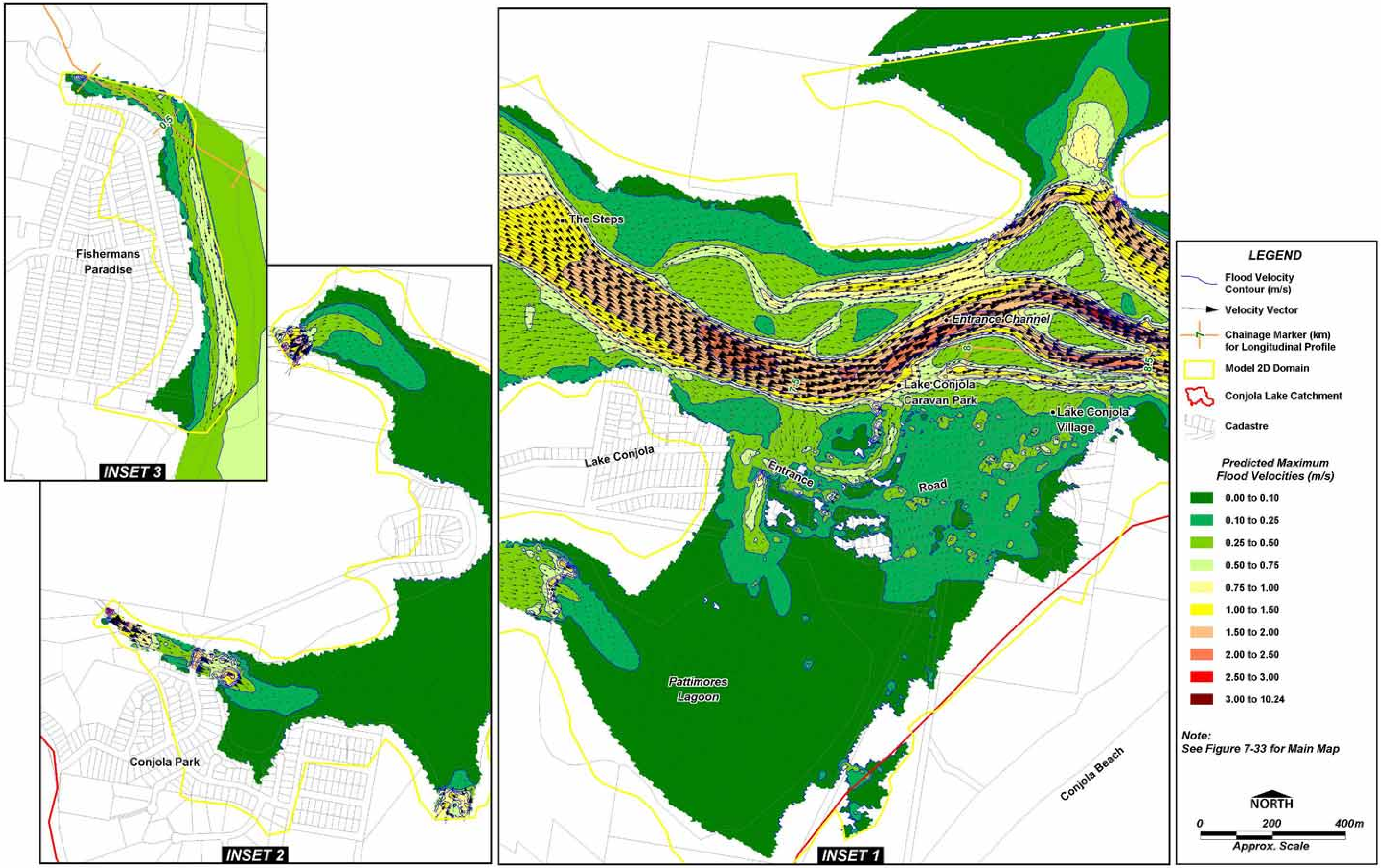

Figure 7-34 Maximum 100 Year ARI Design Flood Velocities (insets)

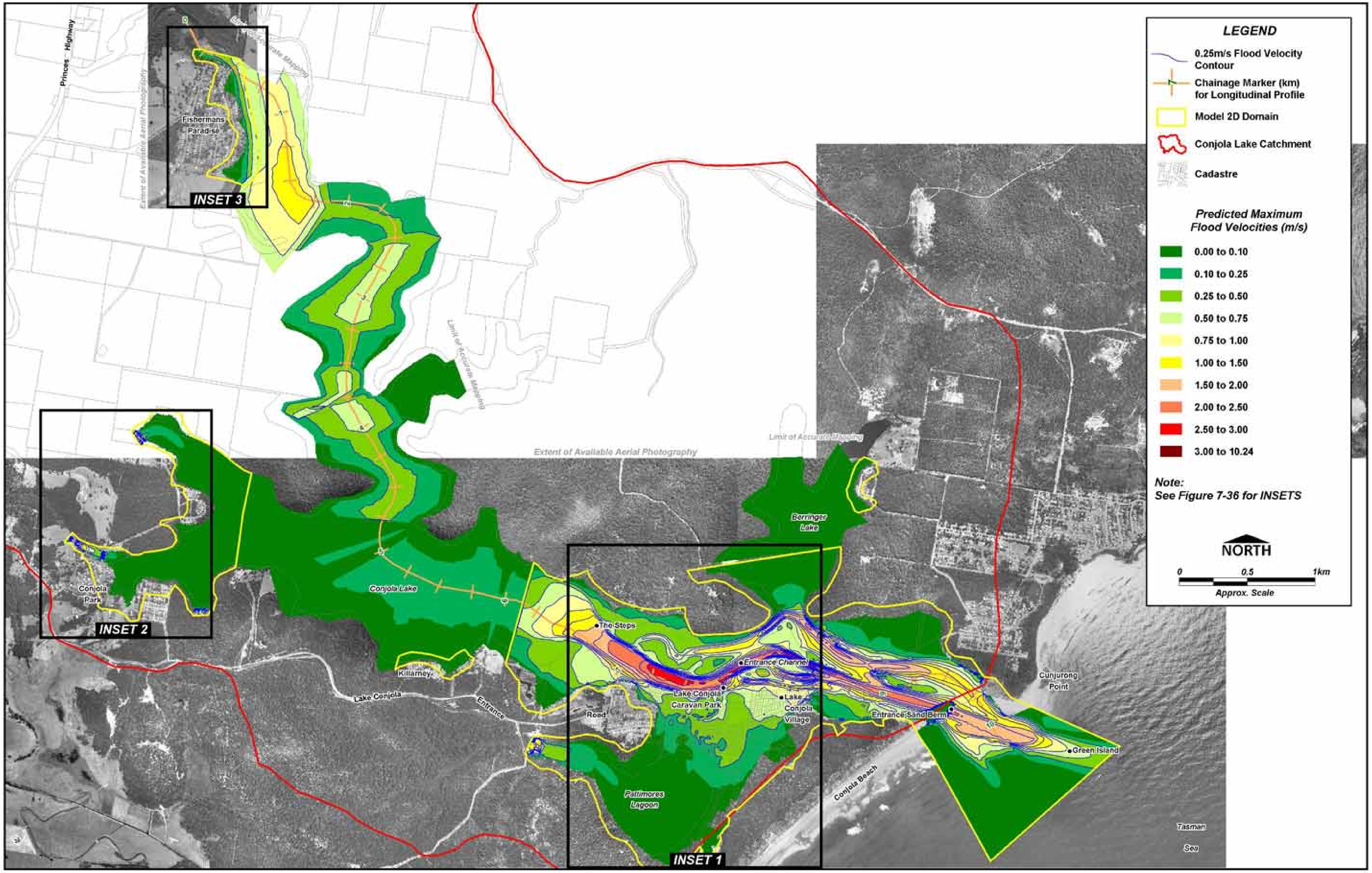

Figure 7-35 Maximum PMF Design Flood Velocities

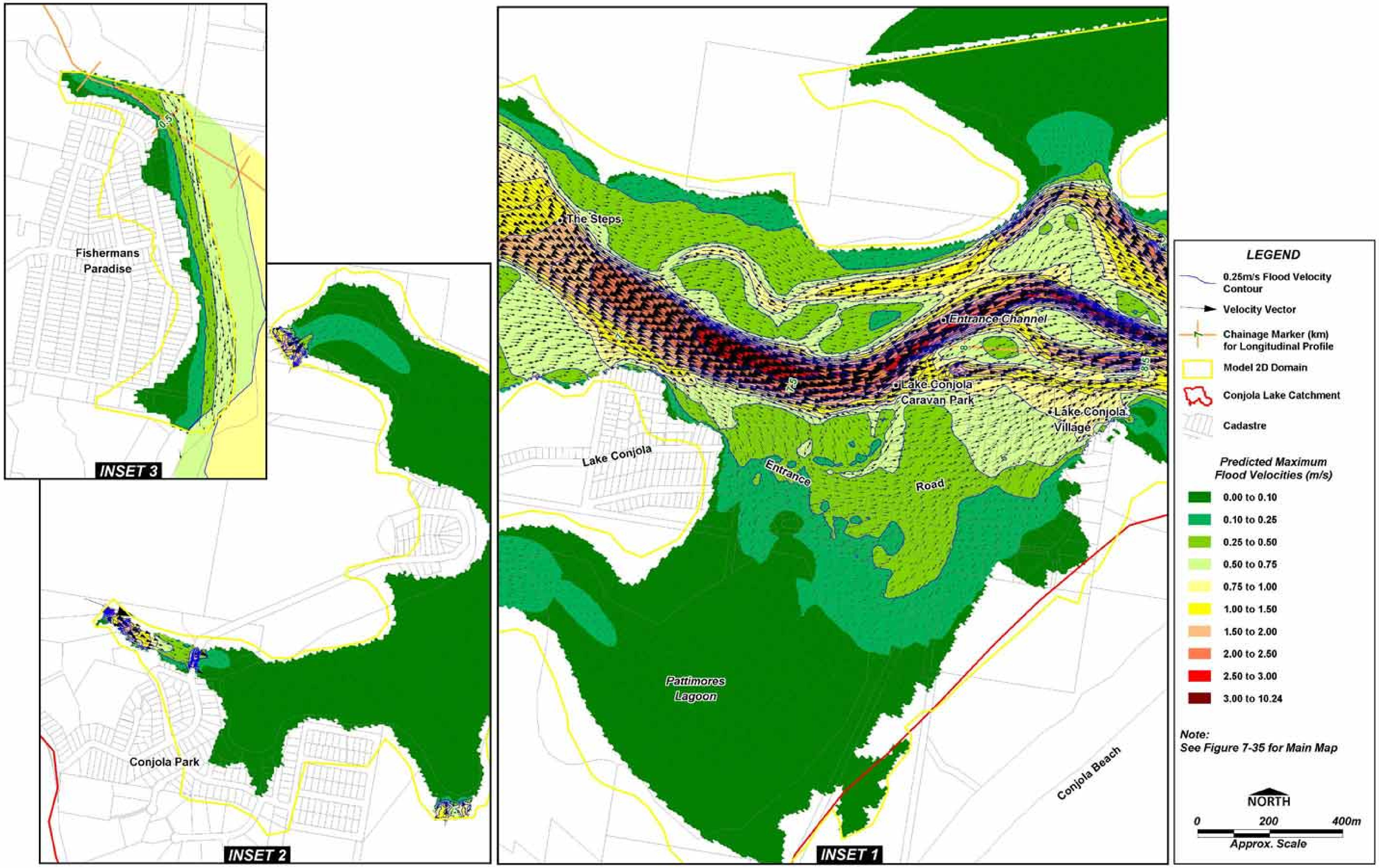

Figure 7-36 Maximum PMF Design Flood Velocities (insets)

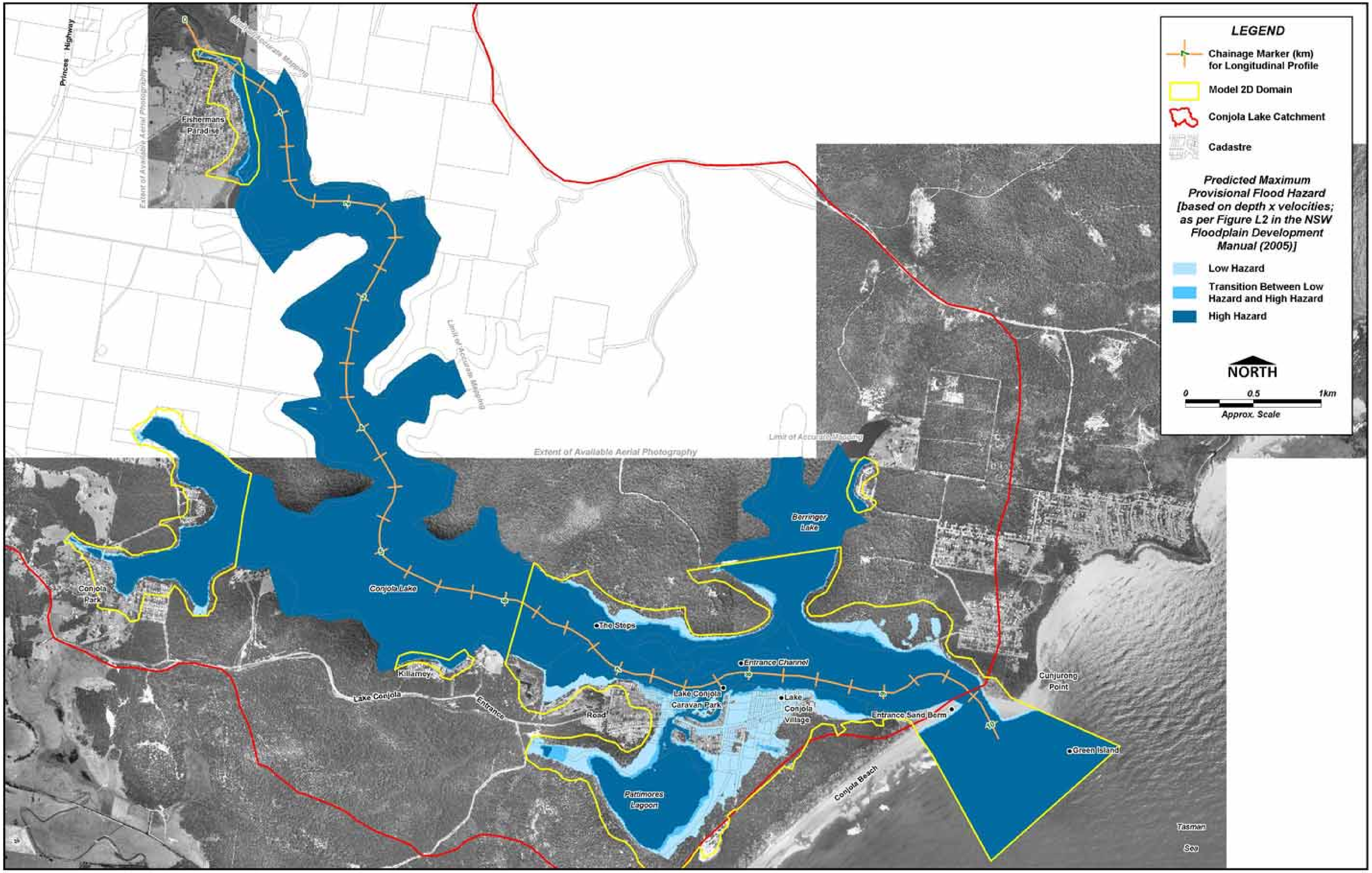

Figure 7-37 Maximum 5 Year ARI Design Flood Provisional Hazard

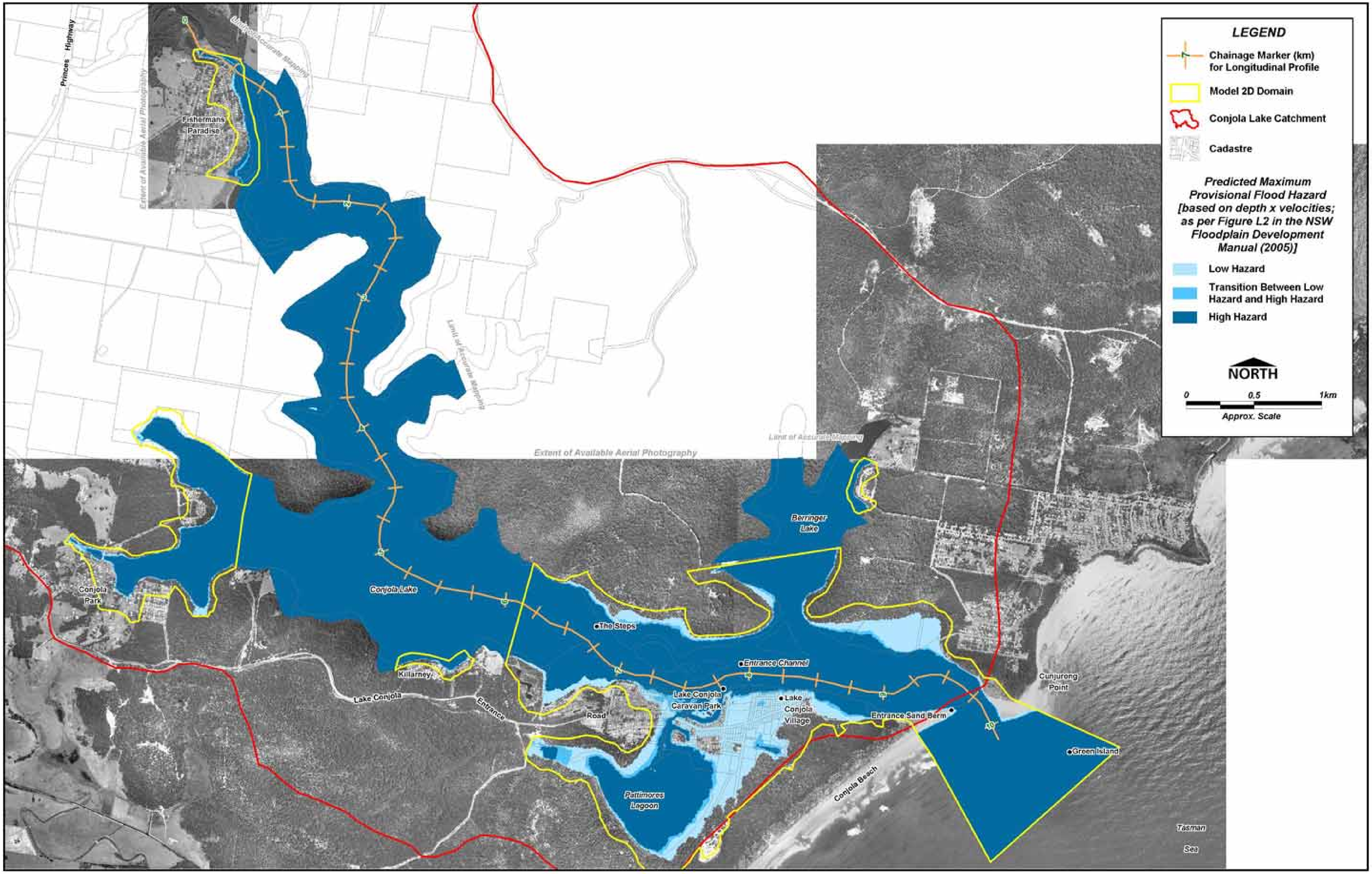

Figure 7-38 Maximum 10 Year ARI Design Flood Provisional Hazard

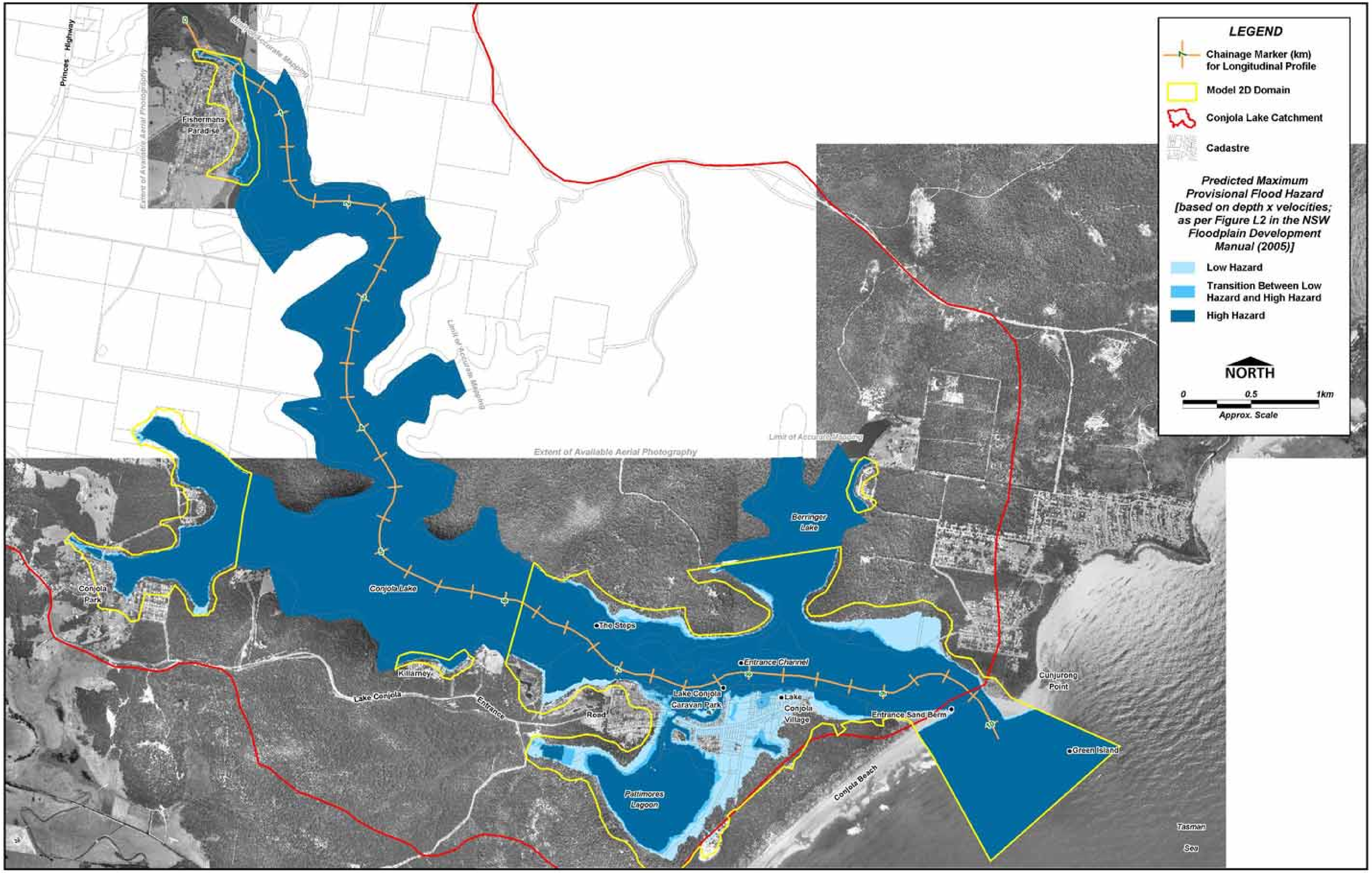

Figure 7-39 Maximum 20 Year ARI Design Flood Provisional Hazard

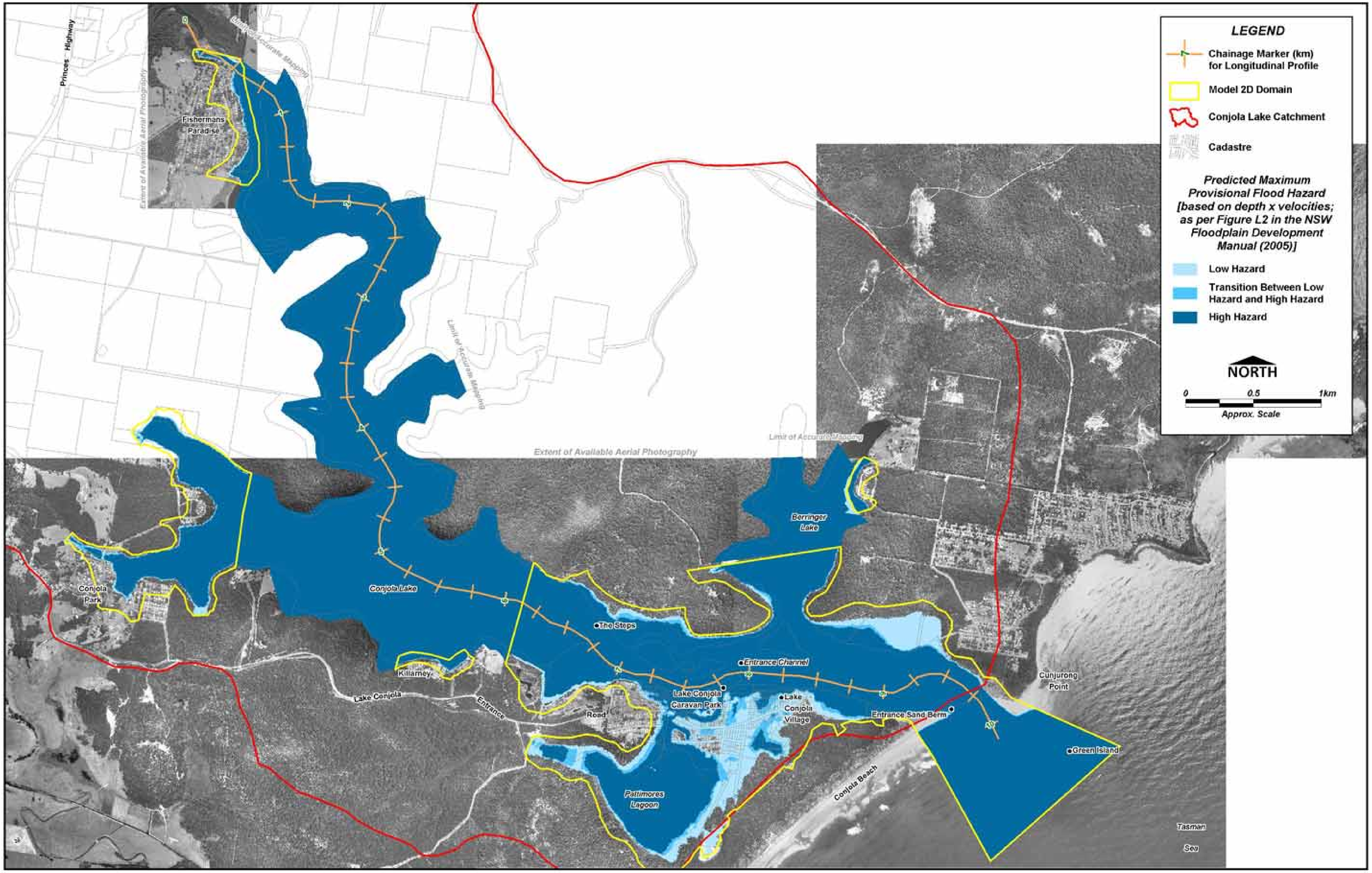

Figure 7-40 Maximum 50 Year ARI Design Flood Provisional Hazard

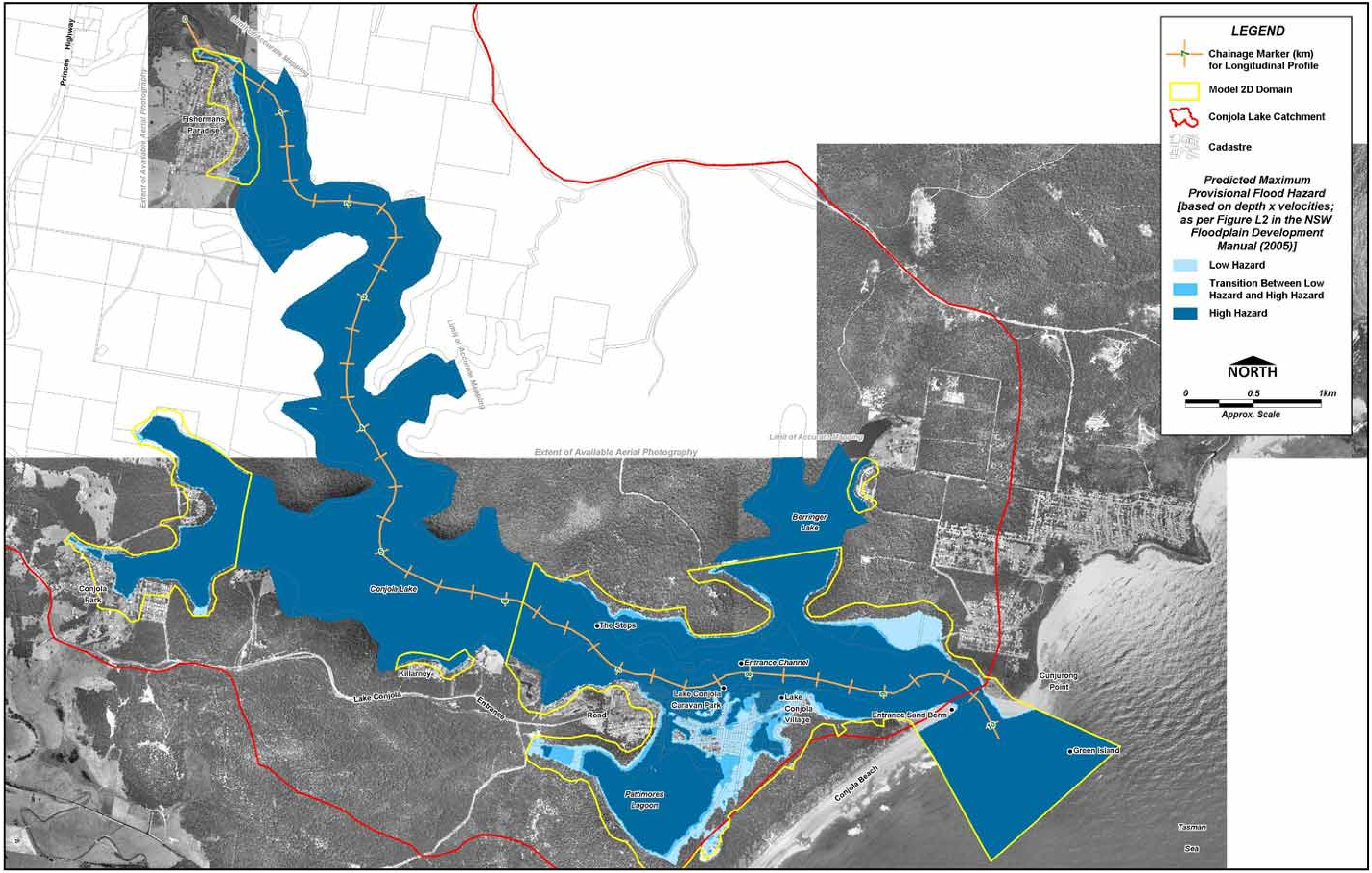

Figure 7-41 Maximum 100 Year ARI Design Flood Provisional Hazard

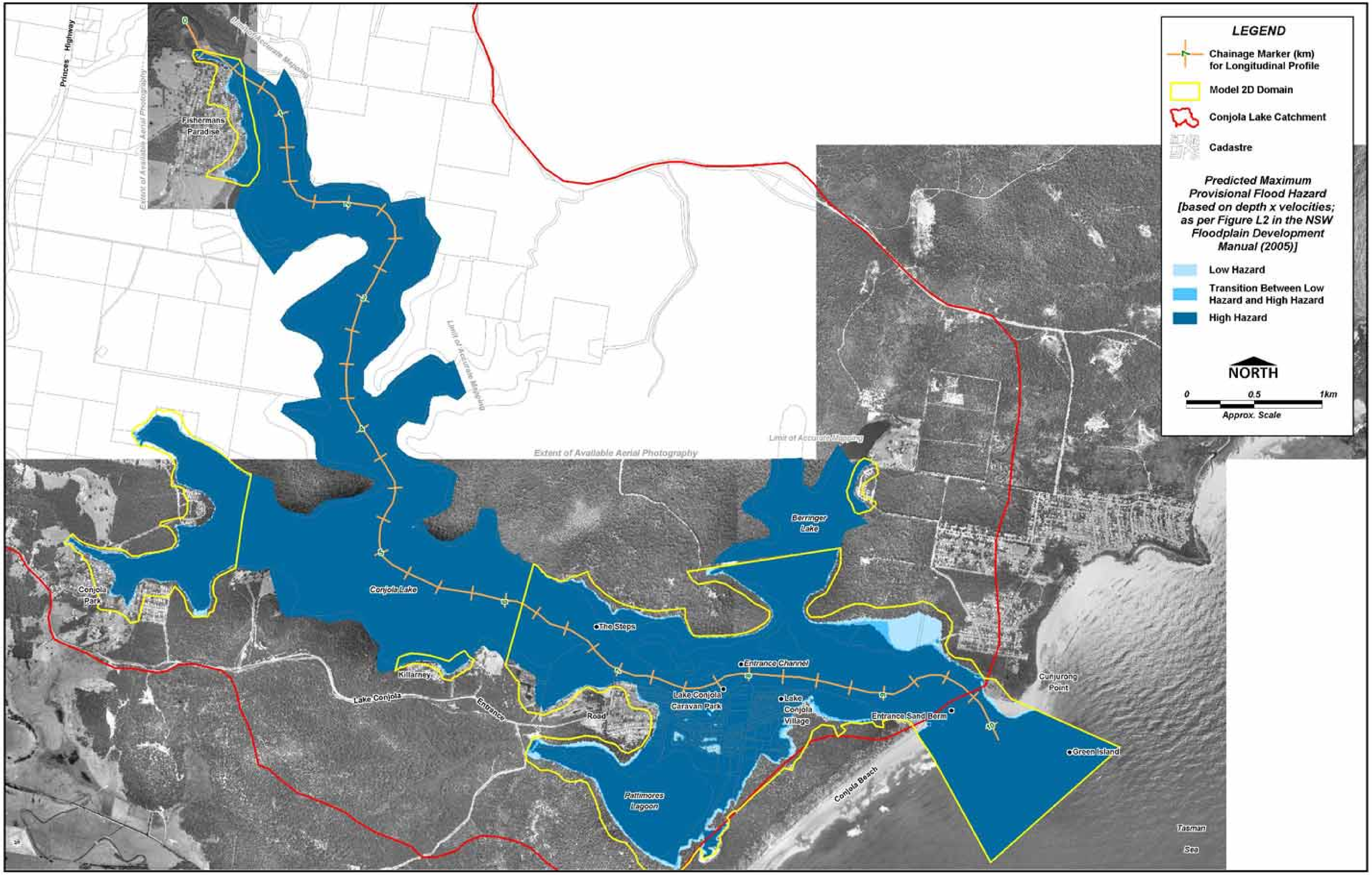

Figure 7-42 Maximum PMF Design Flood Provisional Hazard

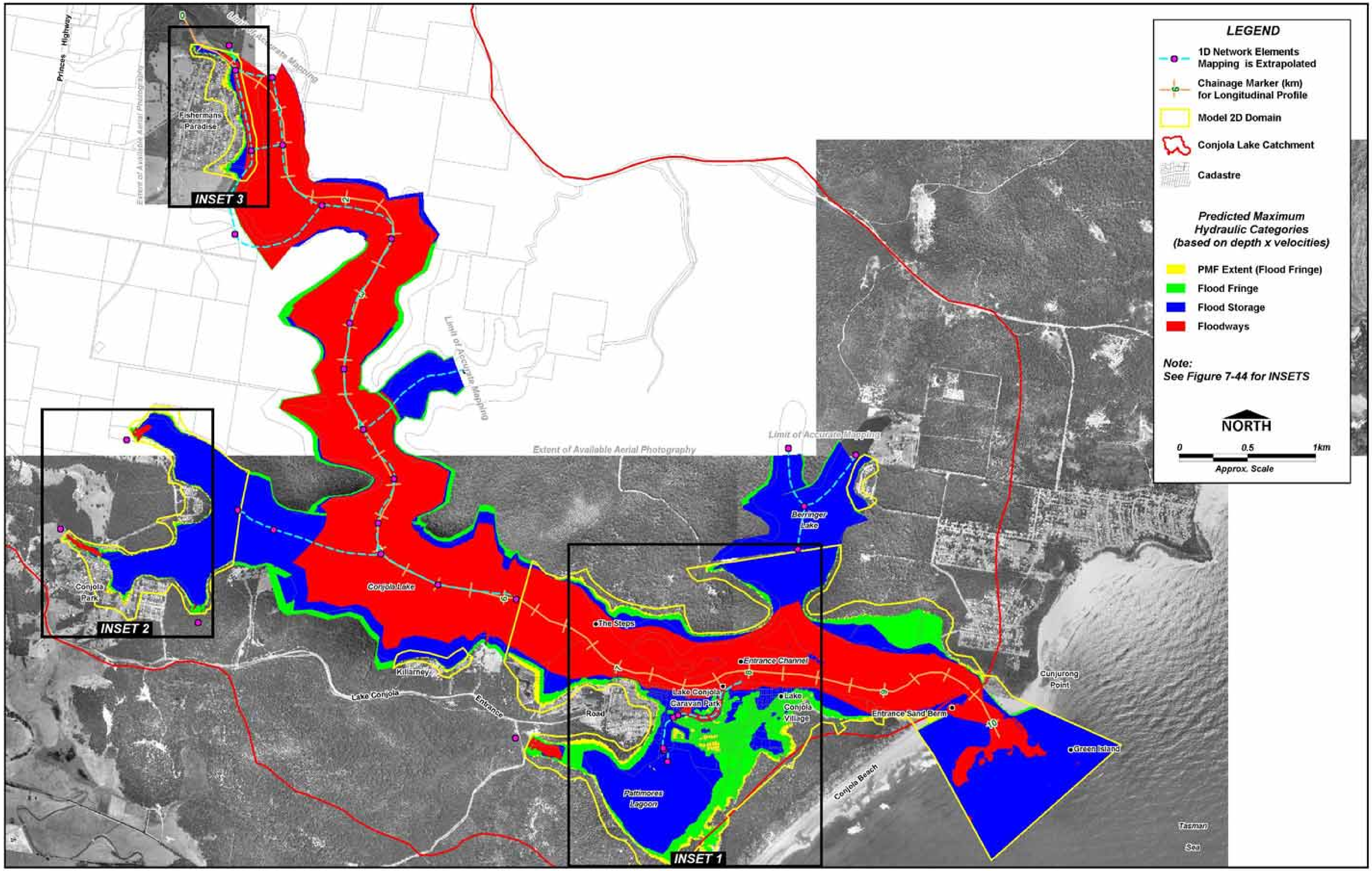

Figure 7-43 Conjola Lake Design Hydraulic Categories

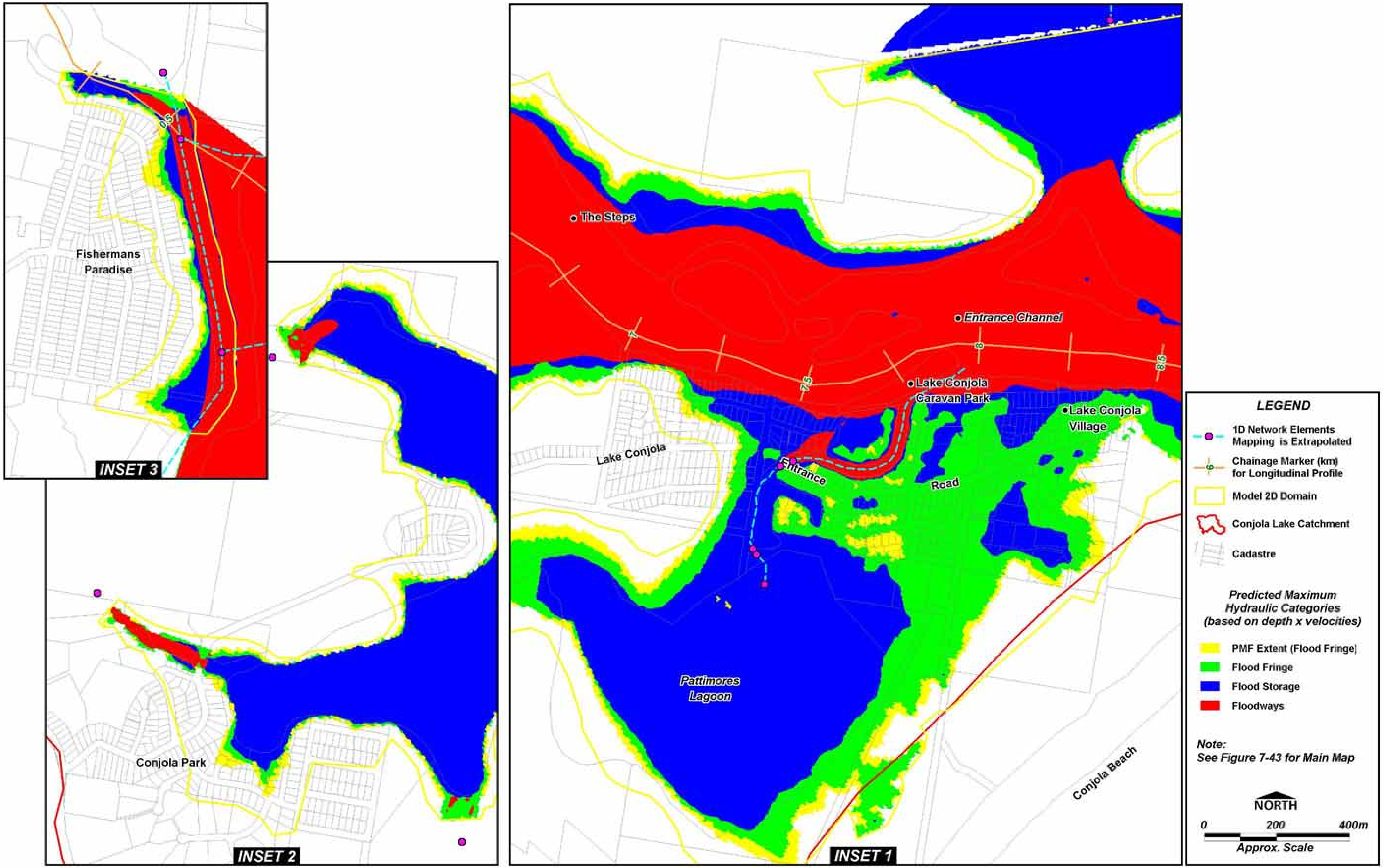

Figure 7-44 Conjola Lake Design Hydraulic Categories (insets)

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APPENDIX A: REVIEW OF SELECTED REFERENCES

A number of background reference reports were reviewed as part of this study, including:

- Lake Conjola, Stage 1: Estuary Processes Study Draft (August 1996), *Gutteridge Haskins & Davey Pty Ltd*;
- Lake Conjola Entrance Study (Issue No. 2, May 1999), *Patterson Britton & Partners Pty Ltd*, including RAFTS-XP hydrology model and RMA hydraulics model; and
- Lake Conjola Entrance Management Plan (January 2003), *Manly Hydraulics Laboratory*.

The above documents essentially focused on the estuary processes affecting Conjola Lake, their impact on the entrance morphology and the implications on the lake's parameters (environment, sedimentology, flooding, landuse, water quality, statutory considerations). Provided below is a short summary of the report outcomes that were found to be of interest for the development of a predictive numerical model of flood behaviour at Conjola Lake.

Lake Conjola – Estuary Processes Study Draft, 1996

The Lake Conjola Estuary Processes Study was initiated by Shoalhaven City Council, to provide a framework for decision making regarding the long-term health and amenity of the lake and adjoining environs.

Conjola Lake was chosen for a management study because of its popularity as a holiday destination, recent urban growth and pressure for expansion, diverse use of the waterways and lake catchment lands, and recent evidence of water pollution. These factors, combined with the existence of a reticulated water supply without a reticulated sewerage service, are suspected to have caused some degradation of the natural environment.

The principal tasks undertaken in the Lake Conjola Estuary Processes Study were to develop an understanding of the hydrodynamic behaviour of the lake and lagoon system (tidal, flooding, entrance and sediment processes and the influence of this behaviour on water quality and estuarine habitat, biota and ecosystems).

The following conclusions were taken directly from the study report. If they are not specifically related to flooding issues, they are still of interest in terms of background knowledge:

- "Lake Conjola is a barrier estuary at an immature stage of development. Bedrock within the catchment has a high resistance to weathering, resulting in relatively low sediment yields to the lake and a consequently low infill rate.
- The lake has a relatively small area of seagrass and there is some evidence that the extent of seagrass has declined over the period from 1985 to 1995, possibly due to increased sedimentation. The loss of seagrass as a habitat may have a negative impact on fish and other aquatic fauna within the lake.
- There are reports of a reduction in the abundance of popular angling species of fish, with a corresponding increase in abundance for less popular species. One hundred species of fish were captured in Lake Conjola during a sampling programme indicative of a diverse assemblage

of fish fauna. About 75% of all species were found in both seagrass and shallow sandy habitats and 35% over deeper more central areas.

- Sediment characteristics within the lake are strongly related to the origin of the sediments. In the tidal entrance channels to Lake Conjola, Berringer Lake and Pattimores Lagoon, well sorted and rounded sands of marine origin are present. The fluvial deltas at the outlets from Conjola Creek and other watercourses are composed of poorly sorted sands with a high organic content. Within the deeper areas of the lake, fine silts and clays occur.
- There are few existing erosion problems within the main lake. Most of the active erosion and navigation problems occur along the tidal channels which connect the main water bodies to the ocean. Flood waters coupled with natural channel migration are responsible for embankment and bed scour along the entrance channels, particularly around channel bends. Although there is sufficient evidence to indicate the presence of progressive natural channel migrations associated with tidal flows, these could not be quantified from historic aerial photography. The most detectable damage seems to occur during major floods.
- In general, the Lake Conjola entrance is characteristically stable compared to the entrances of other similar tidal inlets on the New South Wales coast. A low-lying rocky outcrop which runs through the surf zone on the northern side of the entrance is partly responsible for the general stability of the entrance, which has only been closed during two known periods, once in the late 1940s and again in the late 1960s and early 1970s. The long-term stability of Conjola entrance is dependent on the relative magnitude of tidal forcing, sediment loads and the frequency and magnitude of fresh water flood flows and/or artificial remedies such as periodic dredging.
- The main tidal channel into Lake Conjola has progressive shoals between certain channel bends which make navigation hazardous. Sporadic channel dredging works since the late 1960s have provided improved navigation as well as contributing to the prolonged permanency of the entrance.
- The small outlet creek from Pattimores Lagoon was upgraded in the 1970s, increasing the tidal flushing rate in the lagoon. This appears to have had a negative impact on the aquatic and terrestrial ecosystems within the lagoon. A weir was constructed across the outlet channel to limit tidal flushing, although this has partially collapsed.
- There are reports of conflicts between active and passive users of the lake, although these have been resolved to some extent by management actions undertaken by the Waterways Authority in consultation with local community.
- The overall water quality of Lake Conjola is good. There are isolated areas of the lake where relatively poor water quality has been recorded. This primarily relates to bacterial pollution, depressed oxygen levels and sedimentation in areas adjacent to urban areas with comparatively poor tidal flushing characteristics."

The full content of the report was analysed in more detail, and some information was extracted for the purpose of the flood study, as listed below:

- The sedimentology study presented a map of Lake Conjola with the different sediment characteristics and mean grain sizes (see Figure A 1). The sediments at the downstream end of the tidal delta (where the TUFLOW model has a variable bed feature) are of marine source, with a typical round shape of 0.2-0.3mm diameter. It was also noted that the participation of fluvial sediment in the shoaling of the entrance is minimal.

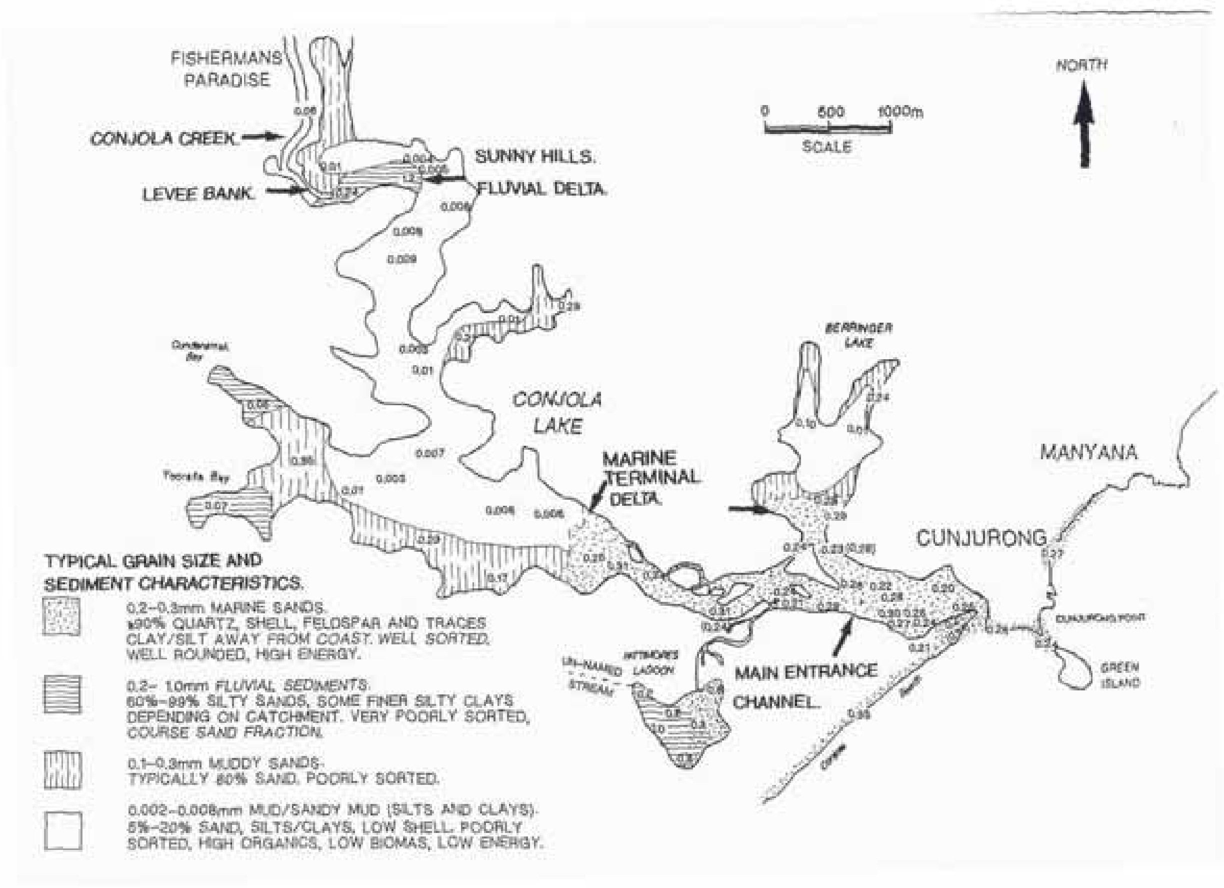

Figure A 1 Sedimentology of Lake Conjola and Adjoining Waterways (source Gutteridge Haskins & Davey)

- A hydrologic and hydraulic models were built for the study of the catchment hydrology and the lake hydrodynamics. The hydrologic modelling was actually undertaken by the Public Works Department and no data is available regarding its structure or calibration process. It was concluded that the critical storm duration in terms of peak flows was a 9 hour storm. Design hydrographs that were used in the hydraulic model were 9 hour storms. The design storm hydrographs were estimated to be approximate, but adequate to the purpose of the study, which was not a full hydrology analysis. Ultimately, it was stated that if the 9 hour storm produces the higher peak flows, it does not necessarily imply the higher flood levels, which should be the important parameter of a flood investigation.
- A hydraulic model was primarily built to observe the consequences of entrance closure over the lake flood levels. One moderate and one rare event were analysed, although the AEP of the events were only indicative. If the results demonstrate that the smaller the cross-sectional area of the entrance channel the smaller the tidal range within the lake, but the higher the flood levels within the lake, it was more a qualitative process rather than a quantitative process. The results were said to be indicative only. Nevertheless, the study used a precise tidal gauging recorded on the 30 October 1992 for its calibration. The results of the gauging is presented in Table A-1.

Table A-1 Lake Conjola Tidal Gauging 30 October 1992, (source Gutteridge Haskins & Davey)

Parameter	Lake Conjola	Berringer Lake
Ocean Tide Range, Δ_0 (m)	1.18	1.18
Mean Ocean Level (m AHD)	+ 0.01	+ 0.01
Mean Lake Level (m AHD)	+ 0.02 – Yooralla + 0.05 – Sunny Hills + 0.10 – Fishermans	+ 0.15
Lake Tidal Range, Δ_L (m)	0.26	0.10
LWL in lake (m AHD)	-0.02	0.09
Tidal Prism, P (m^3)	1.6×10^6	0.1×10^6
Maximum Discharge, Q[2]		
Flood (m^3/s)	120	13
Ebb (m^3/s)	66	5
Maximum Average Velocity[3]		
Flood (m/s)	0.62	Not Calculated
Ebb (m/s)	0.64	Not Calculated
Maximum Gauged Velocity [4]		
Flood (m/s)	0.93	0.23
Ebb (m/s)	0.92	0.32

[1] Average of ebb and flood tide values unless otherwise stated
[2] Highest total discharge measured during the gauging period
[3] Mean cross-sectional velocity at the time of maximum discharge
[4] Highest local velocity measured during the gauging period

Due to the difference in entrance channel geometry between the 1992 tidal gauging and the current (managed) entrance conditions, comparison between the gauged results and the flood study design results is not possible.

The report concluded that it is imperative to model the dynamic entrance response to flood events in order to obtain results valuable for a flood study.

Lake Conjola Entrance Study, 1999

The Lake Conjola Entrance Study was commissioned by Shoalhaven City Council with the following objectives:

- To develop an understanding of existing lake processes;
- To determine the effect of existing entrance conditions on flood behaviour and water quality (particularly faecal contamination);

- To determine the relative impact of different entrance channel manipulation on lake flood behaviour and water quality;
- To discuss other options for alleviating water quality and flooding concerns; and
- To undertake an economic assessment of the various management options in terms of their cost, reduction in flood damages and improvements in water quality.

With respect to entrance stability the study concluded:

- Entrance closures are caused by severe coastal storms;
- Periods of entrance stability corresponded with periods of little storm activity; and
- The key to improving entrance stability is preventing storm washover deposits.

Six basic options for the long-term management of the entrance to Conjola Lake were identified:

- Entrance breakwaters;
- Stub groyne and internal training wall;
- Stub groyne and internal groyne field;
- Stub groyne with partial spit stabilisation;
- Managed entrance; and
- Existing opening policy.

The Council's Estuary Management Task Force for Lake Conjola considered these options and selected the managed entrance as the preferred option. This option involves a proactive system of monitoring, an effective decision-making process and planned maintenance dredging at the appropriate time. An information management system will be established to monitor the condition of the entrance and run a predictive model that will signal when entrance closure is imminent. A work plan outlining the maintenance dredging procedures will be established in readiness to be implemented once the predictive model signals that closure is becoming imminent.

As part of the study, a flood assessment was carried out to ascertain the current level of flood risk experienced by residents around Lake Conjola, and to determine the changes to this flood risk associated with proposed entrance management options. The PBP (1999) flood assessment indicated that within the township of Lake Conjola, the 1 in 100 year flood level was in the order of RL 2.9m AHD, while the level in the more upstream and deeper sections of the lake (ie. upstream of "the Steps") could be up to RL 4.0m AHD. These levels are considerably higher than Council's current flood planning level in the area, which has been set at RL 2.5m AHD for the entire Lake Conjola area. It is understood that the current planning level was established based on historical flood levels (RL 2.4m at Lake Conjola village). The current flood planning level of RL 3.0m AHD is based on this flood event (which occurred in 1971). Shown below are the results of the flood assessment carried out by PBP (1999).

Location	Flood Level (mAHD)				
	storm surge only[1]	major flooding scenario[2]	1 in 100 yr flood with no storm surge	minor flooding scenario[3]	1 in 5 yr flood with no storm surge
Ocean Tailwater	2.2	2.2	0.6	2.2	0.6
d/s Caravan Park	1.5	2.6	1.7	2.2	1.3
Lake Conjola post office	1.5	2.9	2.0	2.3	1.4
Conjola Lake	1.5	4.0	3.1	2.9	1.9

1 1 in 5 year storm surge of 1.6 m superimposed on a tide of mean spring range ie. peak ocean level of 2.2 m AHD.
2 1 in 100 year freshwater flow coinciding with 1 in 5 yr storm surge.
3 1 in 5 freshwater flow coinciding with 1 in 5 yr storm surge.

With regards to the impact of the entrance management options on flooding in Conjola Lake, the PBP (1999) study concluded that only the managed entrance option would reduce flood levels and hence flood damages compared to the existing entrance condition.

In its flood damages analysis PBP (1999) commented that the damage potential could be reduced by a number of floodplain management options, but only house raising and emergency response planning were highlighted as being applicable to Lake Conjola. All flood mitigation options considered by PBP should be re-evaluated during future floodplain risk management study.

Lake Conjola Entrance Management Plan, 2001

The Entrance Management Plan describes the system for management of the entrance of Conjola Lake to ensure a permanently open entrance.

The management plan describes:

- The background to the project and the planning framework;
- The entrance physical processes;
- The entrance works to be initiated to prevent entrance closure;
- The ongoing monitoring of the entrance;
- The decision support system to warn of closure;
- The procedure for initiating works and the responsibilities for management of the entrance; and
- The commitment document to ensure ongoing commitment to the plan.

Apart from the description of the managed entrance option that was selected (which in broad terms will be listed in the following Appendix B) no information could be extracted from the report to be used in the Flood Study.

APPENDIX B: THE LAKE CONJOLA MANAGED ENTRANCE OPTION

The entrance of Conjola Lake tidal delta is categorised as a tidal delta of clean marine sand with pronounced sand lobes, which can be elevated up to 1m above mean sea level. The entrance shoals change constantly due to the dynamic interchange between floods, tidal flows, storm waves, littoral sand supply and wind-blown sand from Conjola Beach.

The entrance is prone to periodic closure – over the last 60 years there have been eight closures. The entrance can remain closed for years until opened by floods, usually assisted by mechanical excavation of a pilot channel. When the entrance is closed, the lake water level builds up with fresh water and there can be a marked decline in overall water quality. The last closure (1994-1998) caused considerable hardship and concern to the local communities.

As a consequence, Shoalhaven City Council commissioned the establishment of a entrance management plan, which will ensure a permanently open entrance with minimal interference with natural environmental processes.

A draft report of the entrance management plan was released in April 2002, by the Manly Hydraulic Laboratory. It summarises the previous researches and studies as well as the conclusions of the Lake Conjola Estuary Management Task Force. The MHL report decides on the preferred management option to be implemented.

The chosen option is a 'managed entrance', which means that when the entrance is close to closure, a channel will be dredged along an historic flood cut to re-establish oceanic exchange. The Entrance Processes Study (PBP 1999) specified that the dimensions of the channel should be 40m wide at the base, 2m deep with side slopes of approximately 1:3 (V:H). The layout is shown in Figure B 1. An optional substantial sand/wave trap (as shown in Figure B 2) could be dredged at the same time as the channel to increase the interval between dredging, but as this work lowers the benefit cost ratio of the whole option it was not recommended.

The managed entrance policy also requires the southern part of the entrance spit to be built up to a level of about 7.0 mAHD and stabilised so that it is not susceptible to storm washover. It is recommended that the northern part (approximately 200m) of the spit should not be raised above 1.0-1.5 mAHD so that the entrance will be free to scour to its full width during major floods. The area to be built up and stabilised and the sacrificial fuse plug area are shown in Figure B 2.

The managed entrance option includes a constant monitoring of the entrance to allow the identification of early signs of closure, the evaluation of the health of the estuary and also to provide additional data to optimise the dredging.

For the purpose of the Flood Study design events it was necessary to adopt a most likely geometry. It is impossible to forecast the exact evolution of the entrance because it depends on the occurrence of floods and severe ocean storms with waves with a southerly directional component. Analysis of the environmental records has shown prolonged periods with both frequent occurrences and absence of storms and floods. The entrance geometry selected for the design event analysis was chosen as being the 'managed entrance' one, with sensitivity analysis reflecting the degree of shoaling between dredging times.

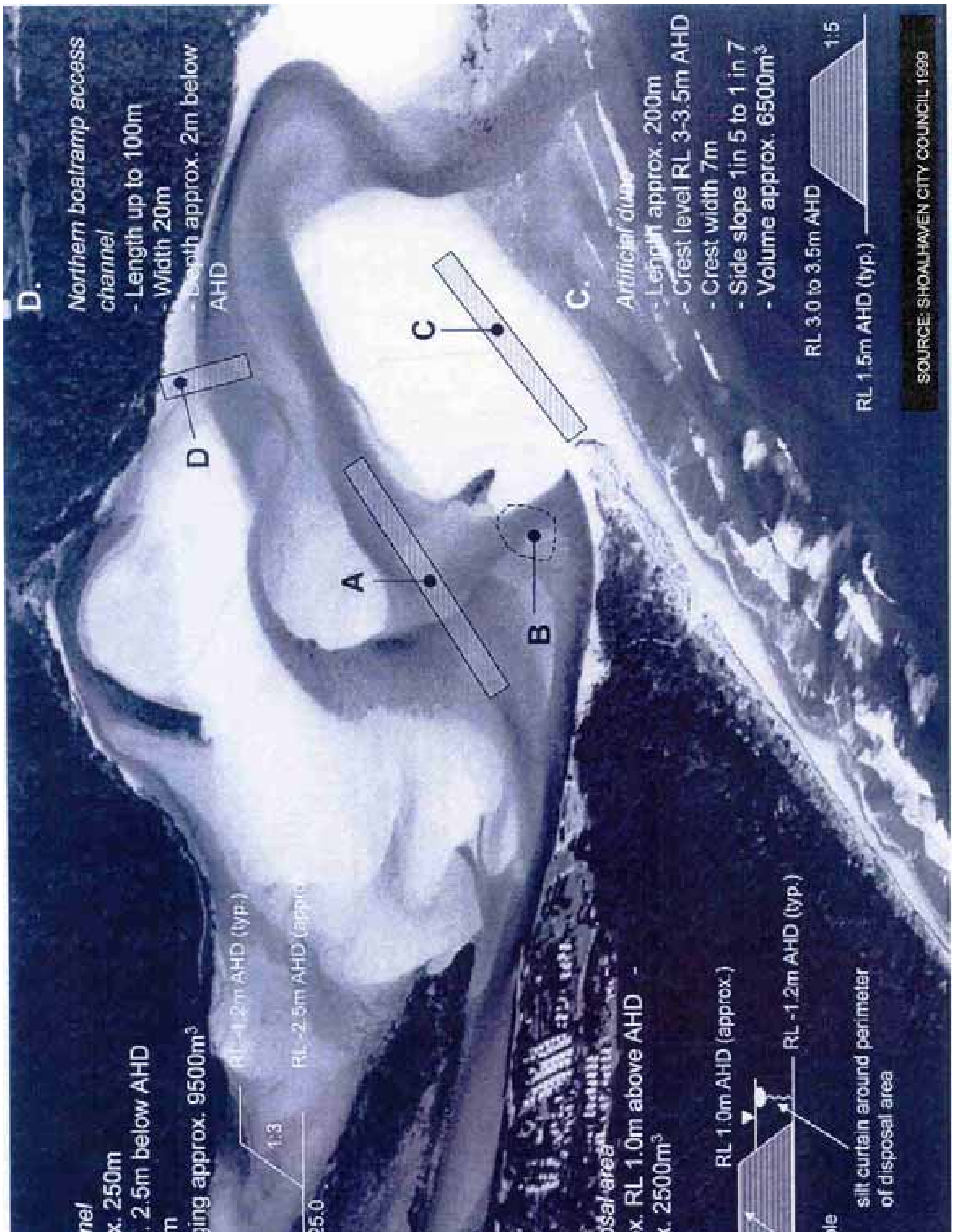

Figure B 1 Lake Conjola Managed Entrance Initial Works (1999)

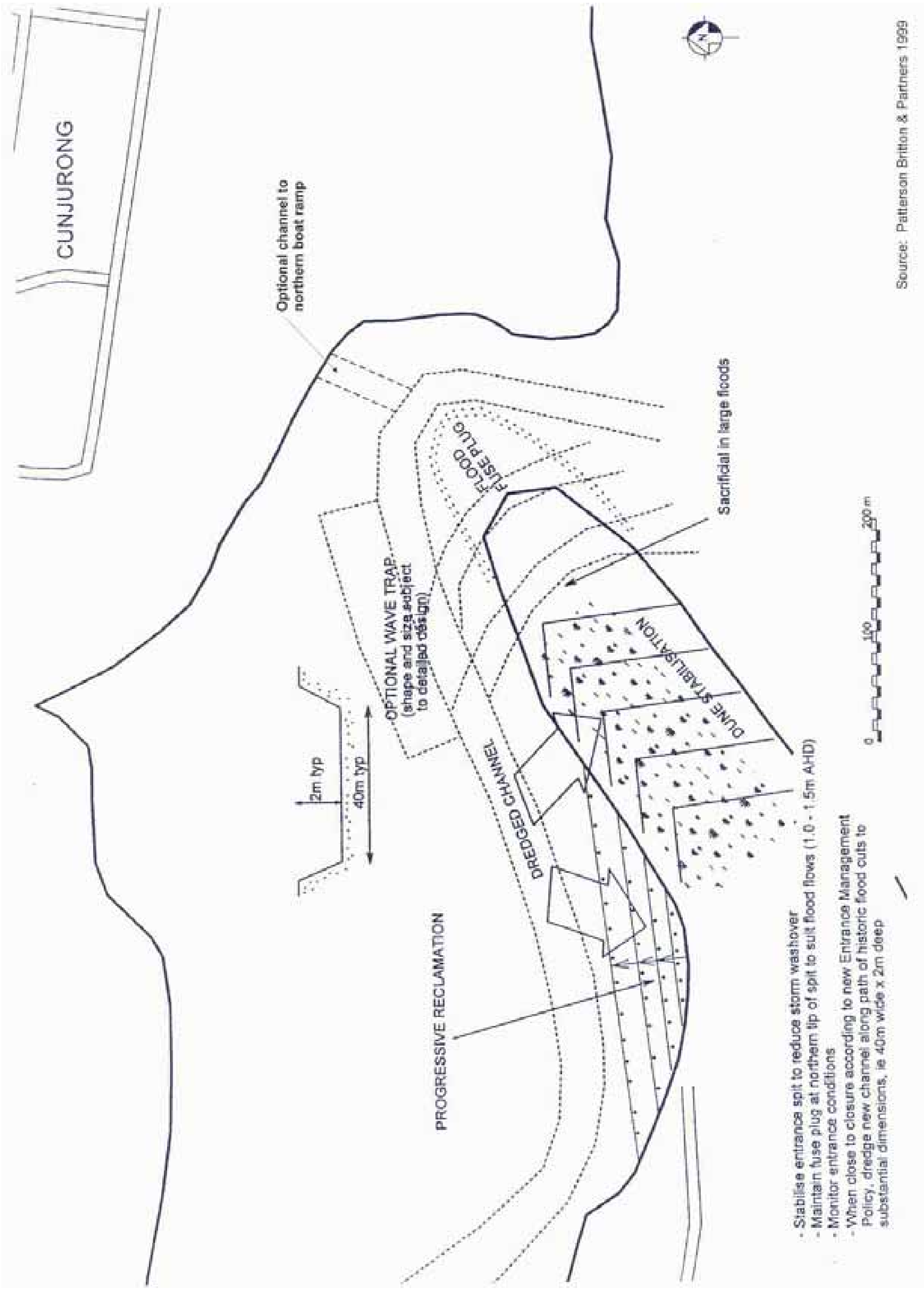

Figure B 2 Lake Conjola Managed Entrance Final Works

APPENDIX C: 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 1987 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.

Three levels of hydraulic routing are possible including simple Manning's based lagging in pipes and channels, the Muskingum-Cunge procedure to route hydrographs through channel or river reaches or the hydrographs may be transferred to the XP-SWMM/XP-UDD Hydro- Dynamic simulation model.

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 river channel);
- Decide on the location of the model boundaries. The boundaries are the outer points of the model where, for example, the river flows 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; and
- 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: $\hat{U}_{\delta}$

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

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

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 ($a=½\Delta_r$) 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 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 D: MODEL CALIBRATION AND SENSITIVITY

Rational Method

The Flood Study objectives are to determine the design flood conditions in Conjola Lake. As the hydrologic model's accuracy could not be directly verified against historical flow measurements, it was considered worthwhile instead to compare model results with a probabilistic method of flood prediction. The Rational Method is an accepted Australian standard when no detailed data is available. The Rational Method is a probabilistic or statistical method used in estimating design peak flood flows, as opposed to real floods. -It is used to estimate a peak flow of selected average recurrence interval from an average rainfall intensity of the same average recurrence interval.

The Rational Method is presented in Australian Rainfall & Runoff (AR&R), Book IV IEAust (1999&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 related to the catchment characteristics, must be specified as part of the procedure. The critical rainfall duration is t_c, is considered to be the travel time from the most remote point of the catchment to the outlet, or, in other words, the time taken from the start of the rainfall until all of the catchment is contributing simultaneously to the outlet flow. The Eastern NSW formula was used in the Flood Study to calculate the time of concentration.

The Rational Method analysis of the catchment upstream of Conjola Creek Outlet (located at Fishermans' Paradise), which accounts for 75% of the whole Conjola Lake catchment, predicts a 100 year ARI design peak flow of 1460 m^3/s for a time of concentration of 4.5 hours (calculated to be the critical time of concentration using the Eastern NSW formula).

The choice of Conjola Creek outlet for the Rational Method calculation is justified by the storage influence of Conjola Lake. The method cannot integrate this storage effect, nor does the Conjola Lake RAFTS-XP model. Conjola Creek outlet is the most representative point of comparison for the catchment. It also accounts for three quarters of the total Conjola Lake catchment, and is the principal inflow location for the hydraulic model.

The RAFTS-XP hydrologic model estimates a 100 year ARI design peak flow of 1410 m^3/s for a time of concentration of 6 hours (determined to be the most critical duration when using RAFTS-XP). There is only 3.5% difference in 100 year ARI design peak flows between the hydrologic model and the Rational Method. This result is considered to be within the acceptable range of differences between the two methods. It provides confidence that the RAFTS-XP hydrologic model of the Conjola Lake catchment is appropriate to establish historical design flood flow conditions.

The comparison was undertaken using design parameters in the RAFTS-XP model, as described in Section 6.7, (viz: Initial Loss = 15mm and Continuous Loss = 1.5 mm/hr, due to an assumed initial saturation of the catchment; and Storage Coefficient B = 1).

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, not to say contradiction, between the flood marks that cannot be explained hydraulically. It is believed that some of the flood marks were difficult to recollect accurately due to the long period of time since the flood events.

February 1971

ID	Address	Recorded Level (mAHD)	Computed maximum 1971 water level (mAHD)	Comments	Reliability
71-09	Garrad Way	2.94	3.11	Recorded at the level of the first board on a Garrad Street elevated house. The level suggests that the flood level only reached the house floor level. The quality of the flood mark has been rated high.	Very Good
71-08	47 Edwin Ave	2.51	3.11	Along Edwin. The flood mark is located on the sidewall of one house, about 13 bricks high.	Good
71-16	23 Edwin Av	2.41	3.12	In Edwin Street, the resident recollected 3 inches of water inside the house. Pictures taken by the surveyor show different levels within the house.	Low (unsuitable for calibration)
71-09A 71-09B 71-09C 71-09D	Lake Conjola Caravan Park	2.68 2.72 2.91 2.85	3.11 3.09 3.07 3.06	The resident estimated a 5 foot depth of water in the caravan park. Four ground levels in the CP were surveyed. There is a 0.25m variability over the ground levels. The precision of the flood mark is judged as poor.	Low (unsuitable for calibration)
71-15A	74 Entrance Rd	2.58	3.08	Located on Entrance Road. The resident estimated the depth to be 2'8". The floor and ground levels were surveyed. The floor level was used.	Good

ID	Address	Recorded Level (mAHD)	Computed maximum 1971 water level (mAHD)	Comments	Reliability
71-17	Killarney Caravan Park	4.79	3.27	Flood mark too high in Killarney. The resident remembered that the water was over the top of her caravan. However, the exact height and location of the van was not determined. Only an indicative level was surveyed.	Very Low (unsuitable for calibration)
71-07C	Carroll Av	2.91	2.93	On Carroll Avenue. The flood mark was located at half the height of the garage door. The quality of the flood mark has been rated high.	Very Good
71-07A	Milham St	3.39	3.10	On Milham Street. A window sill was indicated to the surveyor. (coordinates: 252935.307/1095459.468)	Low (unsuitable for calibration)
71-07B	Milham St	1.81	3.10	On Milham Street. A painted mark on a wooden post was surveyed. (coordinates: 252874.674/1095331.353)	Low (unsuitable for calibration)

March 1975

ID	Address	Recorded Level (mAHD)	Computed maximum 1975 water level (mAHD)	Comments	Reliability
75-11	1 Anglers Parade	2.14	2.5	In Fisherman's Paradise. The resident showed the flood mark to the surveyor. The location is almost identical to 75-10, which indicates a 0.5m higher flood mark. Floodmark recalled from memory.	Low (unsuitable for calibration)
75-10	12 Anglers Parade	2.61	2.5	In Fisherman's Paradise. The resident showed the flood mark to the surveyor. The location is almost identical to 75-11, which indicates a 0.5m lower flood mark. Floodmark recalled from memory.	Low (unsuitable for calibration)
75-14	Lake Conjola Caravan Park	2.16	2.31	Lake Conjola caravan park. The resident estimated a 1m depth of water in the caravan park. The caravan park ground level was surveyed at different locations. A variability in ground level of 0.25m was recorded. Floodmark recalled from memory.	Low (unsuitable for calibration)
75-13	45 Carroll Av	2.10 2.11	2.18 2.18	In Carroll Avenue. The resident remembered water levels reaching floor board level. The quality of the flood mark has been rated high.	Very Good
75-12A	8 Prior St	3.82	2.46	In Killarney. The resident estimated that the flood extended 3 to 4 feet from the front fence.	Low (unsuitable for calibration)
75-12B	8 Prior St	3.55	2.46	In Killarney. The resident remembered that a boat had been tied up to the fence at the end of the street. The fence was surveyed.	Low (unsuitable for calibration)

February 1992

ID	Address	Recorded Level (mAHD)	Computed maximum 1992 water level (mAHD)	Comments	Reliability
92-01	32 Entrance Rd	2.11	Dry (1.80)	Entrance Road: The model shows the area as dry. During the historical flood, the water came from Pattimores Lagoon. The small size of the catchment makes it more reactive to rainfall peaks. The rainfall hydrographs used for the modelling had average rainfall intensities over 2 hour periods. Such a period is too large to reproduce short rainfall bursts that would lead to high peak runoff flows. The effects of intense runoff in Pattimores Lagoon are therefore minimised in the model, which explains it does not reproduce the inland flood surge. To overcome the issue, a more detailed rainfall/runoff model would need to be undertaken using pluviograph records of the local catchment. However, such data is not available.	Good
92-03A	78 Entrance Rd	2.11	Dry (1.80)	Entrance Road: The model shows the area as dry. During the historical flood, the water came from Pattimores Lagoon. The small size of the catchment makes it more reactive to rainfall peaks. The rainfall hydrographs used for the modelling had average rainfall intensities over 2 hour periods. Such a period is too large to reproduce short rainfall bursts that would lead to high peak runoff flows. The effects of intense runoff in Pattimores Lagoon are therefore minimised in the model, which explains it does not reproduce the inland flood surge.	Good
92-04A	15 Garrad Way	1.82	1.78	Garrad Way: The resident estimated a 2 feet depth of water his garden.	Good

ID	Address	Recorded Level (mAHD)	Computed maximum 1992 water level (mAHD)	Comments	Reliability
92-06	8 Garrad Way	1.88	1.78	Garrad Way: The resident estimated a 2 feet depth of water on the front gate.	Good
92-02	40-42 Edwin Ave	1.91	1.79	Edwin Ave: The resident mentioned flood levels reaching 2 boards on the garage door. For the surveyor's photograph shows, it is estimated that the wrong door was surveyed. The difference was visually estimated to be around 600mm.	Average
92-05	Edwin Ave	1.99	Dry (1.80)	Edwin Ave: The resident estimated that the flood extent limit was about 250m from crossroad. The road is however steep and a 20m horizontal error would lead to a 0.6m vertical error in flood water levels. The precision of the flood mark is judged as poor.	Low (unsuitable for calibration)

APPENDIX E: SUMMARY OF KEY PARAMETERS, DATA AND ASSUMPTIONS

GENERAL	Conjola Lake Catchment	145 km^2		
	Lake Area	4.3 km^2		
CALIBRATION	Historical Events	6-7 Feb 1971	11 Mar 1975	11 Feb 1992
	Topography	1995 1m contours of urban areas 1993 hydrographic survey of lake 2003 cross-sections Historical aerial photographs		
	Hydrology			
	▪ Rainfall Depths	24 hour totals from 15 gauges	24 hour totals from 12 gauges	24 hour totals from 10 gauges
	▪ Temporal Patterns	AR&R 36 hour	AR&R 36 hour	Nowra RAN Air Station [68076]
	▪ Stream Gauge	None		
	▪ Initial Loss	0mm	10mm	0mm
	▪ Continuing Loss	0.0mm/h	1mm/h	0.0mm/h
	Hydraulics			
	▪ Initial Lake Level	No Records		
	▪ Ocean Levels	Sydney & Jervis Bay tide records		
	▪ Wave Set-up	No Records	No Records	Batemans Bay Wave Rider
	▪ Number Recorded Flood Levels	9	6	6
DESIGN	Design Events	5 year, 10 year, 20 year, 50 year, 100 year ARI events and the PMF		
	Critical Duration	36 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 • Continuous Rainfall Loss	 15mm 1.5 mm/hr
	Hydraulic Model - Roughness • Sand • Urban areas • National Parks • Island	 0.02 0.06 0.15 0.15
	Peak Flows at Conjola Lake ▪ 5 year ARI (36 hour) ▪ 10 year ARI (36 hour) ▪ 20 year ARI (36 hour) ▪ 50 year ARI (36 hour) ▪ 100 year ARI (36 hour) ▪ PMF (36 hour)	 700 m^3/s 820 m^3/s 970m^3/s 1100 m^3/s 1250 m^3/s 2140 m^3/s

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