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GREATER TAREE CITY COUNCIL

Wingham Flood Study

Review and Upgrade

301015-01997

12-May-2011

Advanced Analysis
Level 12, 141 Walker Street
North Sydney NSW 2060 Australia
Tel: +61 2 8456 6934
Fax: +61 2 8456 6966
Web: http://www.worleyparsons.com
WorleyParsons Services Pty Ltd
ABN 61 001 279 812

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FOREWORD

The Wingham Flood Study involves the comprehensive review and upgrade of the broad based *"Manning River Flood Study" (NSW Public Works Department; 1991)*. The primary objective is to create a sophisticated 2D hydraulic model which will accurately simulate flooding in Wingham. This, in conjunction with an improved Hydrologic Model of the Cedar Party Catchment, will provide refined flood data on Wingham and therefore establish a sound base for the development of a Wingham Floodplain Risk Management Study and Plan according to the "*Floodplain Development Manual" (New South Wales Government; 2005)*.

This document should be read in conjunction with the "Wingham Floodplain Risk Management Study" and the "Wingham Risk Management Plan" (both produced 2010 by WorleyParsons) where the collective objective is to reduce the impact of flooding and to reduce private and public losses resulting from floods. At the same time, the unnecessary sterilisation of flood prone land is avoided by recognising the benefits arising from its use, occupation and development.

Disclaimer

PROJECT 301015-01997 - WINGHAM FLOOD STUDY

REV	DESCRIPTION	ORIG	REVIEW	WORLEY-PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
A	Internal Review	J Merhebi	D McConnell	N/A	15-Jan-2010	N/A	
B	GTCC Draft Review	J Merhebi	D McConnell		01-Feb-2010		
C	Revised Draft	J Merhebi	D McConnell		20-April-2010		
004	Final	J Merhebi	D McConnell		12-May-2011		

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1. INTRODUCTION

1.1 Overview

Wingham is located approximately 45 kilometres upstream along the Manning River at the confluence of Cedar Party Creek. Due to the importance of the Manning River as a transport route, Wingham was established at the furthest point supply boats could reach up the river and therefore became the regions major port.

A large portion of Wingham is elevated high above the floodplain; however some portions, including Wingham peninsula, consist of undulating river terrace at a general elevation of less than 12 m AHD.

In its 178 years of European settlement, many floods of varying severity and impact have been recorded in Wingham. However in this time, none have had an Annual Exceedance Probability (AEP) greater than 1%, with the largest being approximately equal to a 1% AEP and occurring in July 1866. More recent floods of moderate magnitude have occurred in 1978 and 1990. The 1978 flood in particular was one of the largest floods on record (estimated to be less than a 1% but greater than a 2% event) which required the evacuation of residents and led to substantial property damage.

These events, and other significant floods in Wingham, have led to large property losses, injury and in some instances, loss of life. The most important issue, reported by those who experienced these flood events, was not so much the peak level of the flood but was the rapid rise in levels which left little time to respond. In portions of Wingham Peninsula, this risk is exacerbated further by the undulating topography which can become rapidly isolated.

The purpose of this Flood Study is to develop a sophisticated, calibrated 2D hydraulic model that will accurately simulate flooding in the region of Wingham. This requires a hydrological analysis of rainfall over the Cedar Party Creek catchment in order to produce the necessary hydrograph inputs for Stony and Cedar Party Creek. A hydrological assessment associated with the Manning River and Dingo Creek was undertaken as part of the *"Manning River Flood Study" (NSW Public Works Department; 1991)*. This will be reviewed and utilised as part of the current Flood Study. A complete depiction of flood behaviour, and flood hazards will be produced for the range of design floods and hydraulic scenarios analysed. Further details on the analyses undertaken are given in subsequent sections.

1.2 Study Area

The study area comprises of a substantial portion of the Manning catchment concentrating on Wingham and its surrounding communities. The hydrological study area consists of the entire Cedar Party Catchment which stretches north 22 kilometres and several kilometers east and west of Wingham. The hydraulic study area extends upstream along the Manning River (to the west) to the Killawarra Bridge and to the south-east downstream to Mondrook Creek. From Wingham, the hydraulic study area extends several kilometers north. Further details are given in subsequent sections.

The Manning catchment drains an area of approximately 8200 km^2 and extends over 175 km inland from the coast. Significant upper tributaries of the Manning River include the Barnard, the Nowendoc and the Barrington River. In this region the Manning Catchment is elevated up to above 1200 m and is generally mountainous and undeveloped.

Dingo Creek, approximately 11 kilometers upstream from Wingham, drains a sub-catchment of approximately 560 km^2. Tidal influence on the Manning River extends to Abbotts Falls, approximately 5km upstream of Wingham. The Cedar Party Creek sub-catchment drains an area of approximately 143 km^2 and extends approximately 22 km north of Wingham. Primary tributaries of Cedar Party Creek are Stony (Gorman) and Killabakh Creeks which branch from the main channel approximately 4 and 12 km upstream of the Cedar Party Creek / Manning River confluence where Wingham is located.

The Manning River catchment is surrounded by the Hasting and Peel Catchments to the north and the Hunter and Karuah Catchments to the south.

Wingham Peninsula is located to the east of the town centre, in the interfluve between the Manning River and Cedar Party Creek. The numerous gullies on the peninsula, which generally run parallel to the Manning River, indicate the terrace's alluvial origin. The majority of Wingham Peninsula is zoned Rural Residential and as it is elevated between 5 and 20 mAHD, it is some of the lowest lying land in the Wingham region.

1.3 Flood History

The SES *FloodSafe* guide to Wingham indicates that a peak flood level less than 4.90 mAHD can be classified as 'minor', up to 8.90 mAHD as 'moderate' and greater than 11.90 mAHD as 'major'. It must be emphasised that this flood classification system is based on the extent of human impact and not on recurrence interval. Therefore the level classified as 'major' is considerably below the level at which a hydrologist would so classify a flood.

According to the "*Manning River Flood History 1931-1979*" (Public Works Department New South Wales), floods reaching a height of at least 10.6 mAHD at Wingham bridge can be considered "significant". Using this same level as a guide, which corresponds approximately to the level of a 20% AEP flood, at least 29 significant floods have been recorded in Wingham since 1831 (when European records begin). The irregularity at which significant floods can occur is highlighted by the fact that some significant floods are very closely spaced, even occurring within the same year (1870, 1956); while at other times there are long periods without significant flooding (1831 to 1857, 1930 to 1950, and 1990 to present). The three largest floods recorded in Wingham occurred in 1866, 1929 and 1978 and reached a peak level of 15.5 mAHD, 14.9 mAHD and 14.9 mAHD respectfully. Figure 1 shows the significant floods that were recorded in Wingham with a time scale that also shows that there were periods where no significant flooding occurred. Figure 2 shows only the years where significant floods were recorded in Wingham since 1831.

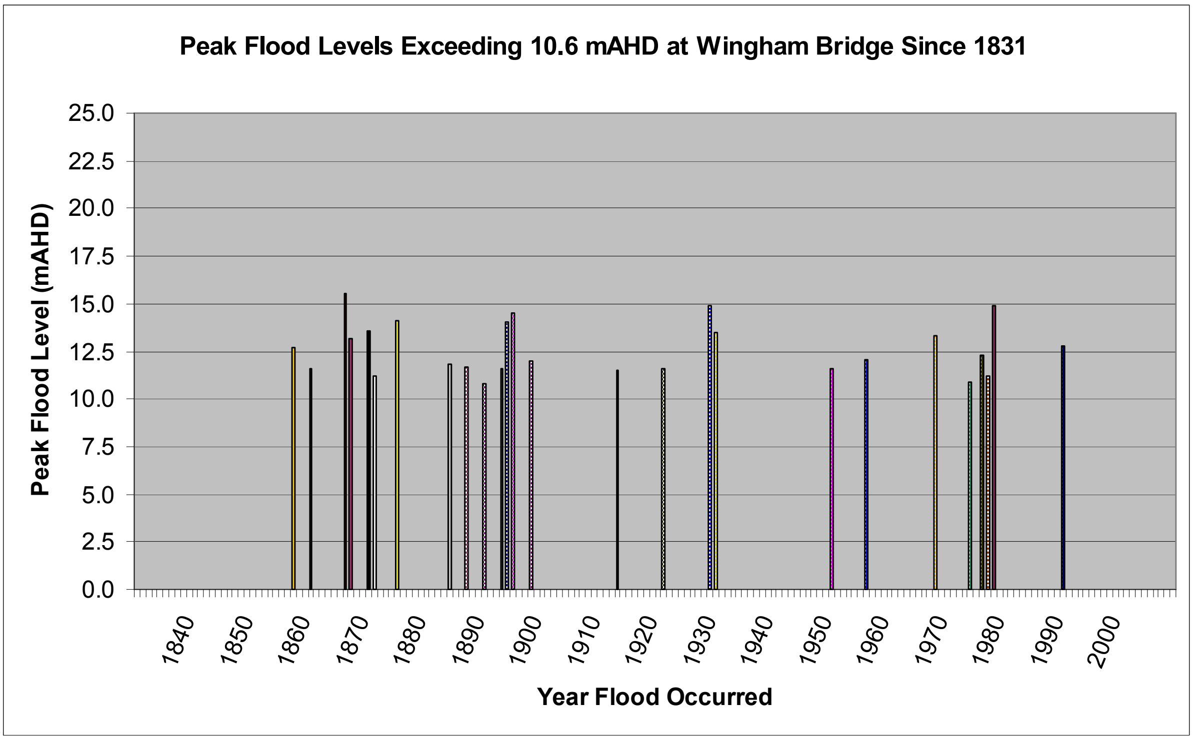


Figure 1: Floods recorded at Wingham Bridge exceeding 10.6 mAHD since 1831

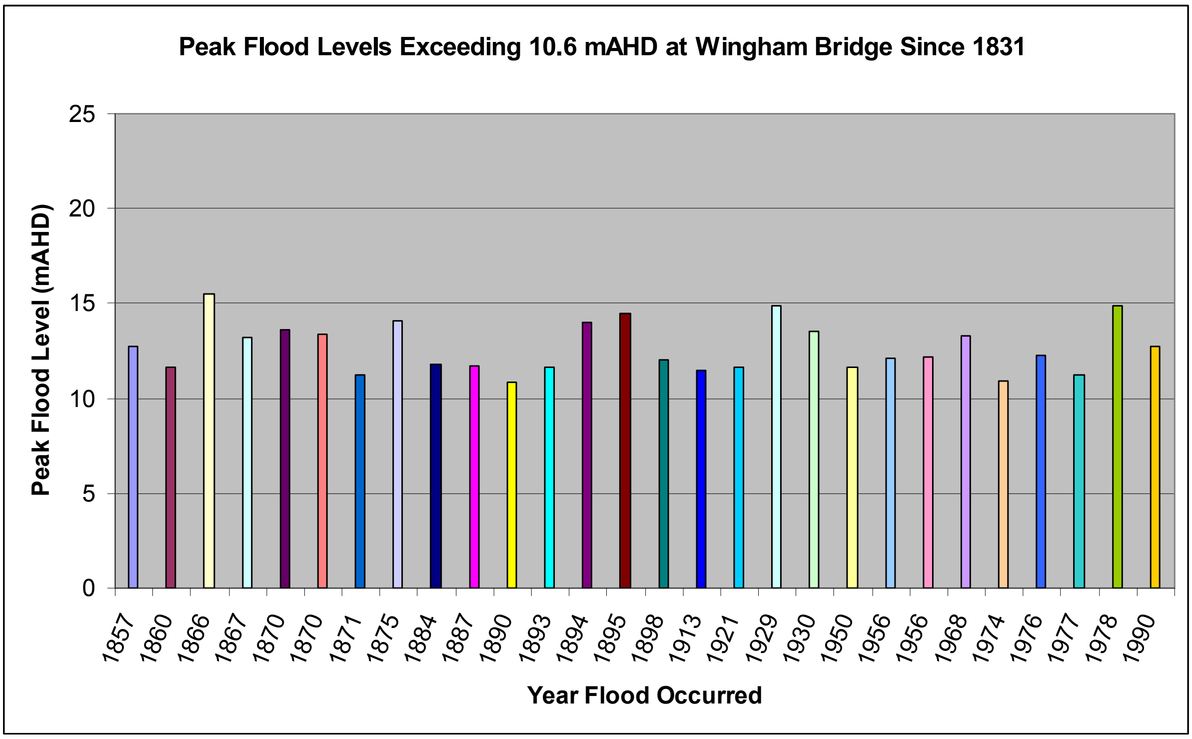


Figure 2: Years where floods recorded at Wingham Bridge exceeded 10.6 mAHD

The 1978 flood event, of which a significant amount of information exists, had a rate of rise as high as 1.5 m per hour at Wingham and caught many by surprise who did not expect flood levels to rise to their ultimate peak. The difference in time between when the major flood warning from the Bureau of Meteorology / SES was given and the time at which the major flood level was exceeded in Wingham was in the order of 4 hours. This highlights the potential danger that exists in Wingham, when a delay in a decision to evacuate, or a misjudgement on the peak level of a rising flood, can rapidly lead to severe risks to life and property. More recently, Wingham has experienced ‘moderate’ flooding in March 1995 when 10.35 mAHD was recorded at Wingham Bridge which was only marginally below the 10.6 mAHD ‘significant’ level.

Since European settlement, Wingham has not experienced an extreme flood event. These have the potential to reach a peak level of 22.3 mAHD.

1.4 Previous Studies and Policy

In 1986 the NSW Government released the first Floodplain Management Manual to assist in the management of flood liable land. This has been twice since revised in 2001 and 2005. The current NSW Floodplain Development Manual (FPDM) aims to optimally maintain the safe use of the floodplain whilst reducing the impacts of flooding, both publicly and privately. The most recent revision sought to ensure consistent interpretations of important strategic variables such as the flood planning level (FPL) and its interaction with rare events up to the PMF.

The FPDM provides a framework for the implementation of a policy based on the following steps:

1. Data Collection; which involves the review and compilation of all relevant data to be used
2. Flood Study; providing technical and quantitative information on flooding in the study area
3. Floodplain Risk Management Study; determining options in consideration of social, economical and ecological factors relating to flood risk
4. Floodplain Risk Management Plan; a selection of options from the study based on community and council endorsement, that will reduce flood risk
5. Plan Implementation; where flood, response and property modification measures are implemented and data collection and monitoring are continued.

After the initial release of the 1986 Manual, the Greater Taree Council implemented an *“Interim Flood Management Policy”* (1987) which specified a FPL equal to the 1% AEP, with less restrictions on commercial and industrial developments.

Following this, the *“Manning River Flood Study” (NSW Public Works Department; 1991)* was produced which constituted step 2 of the FPDM policy. In this way, design flood extents, levels, flows and velocities were estimated in a broad sense across the Manning Catchment, from Wingham to Harrington and the Farquar Inlet. This was completed using a RORB hydrological and an ESTRY 1-D hydraulic model. Limits in computing power and the inherent limitations of one dimensional modeling meant that these results were limited to a general overview of the study area.

The *"Manning River Floodplain Management Study" (Greater Taree City Council; 1996)* was undertaken and constituted step 3 in the FPDM process where the Manning Catchment was divided into 13 areas. Flood Hazards, damages and management options were derived in a broad sense for each area. From this study, a number of townships were identified that required a separate Floodplain Management Plan.

In 2000, the *"Wingham Peninsula Floodplain Management Study & Plan" (Patterson Britton & Partners; 2000)* was subsequently produced which constituted steps 3 and 4 of the FPDM policy with a localised focus on Wingham. From this, a more detailed examination of flood hazards and management options was carried out with the development of a strategy for floodplain management in Wingham. This relied on the results of the ESTRY hydraulic model of the Manning River.

Due to the broad nature of the Flood Study and the resulting data derived from the ESTRY hydraulic model, there existed a need to provide localised flood data on Wingham such that a more complete Floodplain Risk Management Study and Plan for Wingham could be produced.

Therefore the current objectives involve a review and upgrade of the FPDM process with a focus on Wingham and its surroundings.

2. FLOOD STUDY SUMMARY

The broad aim of the Wingham Flood Study (referred to as “the flood study” from therein) is to provide comprehensive technical information on flood behaviour in terms of levels, velocities and extents for floods up to and including the Probable Maximum Flow (PMF). This constitutes the major technical foundation on which the Wingham Floodplain Risk Management Study and Plan are based. In this way, the flood study will provide hydraulic categories, hazards and damage assessments within Wingham.

The flood study was undertaken in two parts; a hydrologic and a hydraulic analysis. The hydrological analysis involved a review of work undertaken in the *“Manning River Flood Study” (NSW Public Works Department; 1991)* (therein referred to as “the previous flood study”) with a focus on a revised hydrological examination of the Cedar Party Creek Catchment (which includes Stony / Gorman Creek). This would be used to provide the necessary inputs required to perform the hydraulic analysis. The hydraulic analysis focused on the town of Wingham and its immediate surroundings. Due to this, it was necessary to utilise boundary conditions from outside of the study area from the previous flood study. In this way, the hydraulic component of the flood study involved a critical review of the previous flood study to ensure that the data was correctly utilised and any more recent and relevant data was incorporated. Core information utilised from the previous flood study included design hydrographs for the Manning River and Dingo Creek as well as the downstream stage-discharge relationship of the Manning River.

More details on the methodology of the hydrologic and hydraulic models, calibration and results of these analyses are given in subsequent sections.

3. DATA COLLECTION

The first step in the flood study process, in accordance with the FPDM, is a collection and review of available and relevant data. This section summarises the data utilised in the analyses.

The following list comprises local and region studies / policies that have relevance to the study area and region:

- *"Interim Flood Management Policy" (Greater Taree City Council; 1987)*
- *"Manning River Flood Study" (NSW Public Works Department; 1991)*
- *"Manning River Floodplain Management Study" (Greater Taree City Council; 1996)*
- *"Wingham Peninsula Floodplain Management Study & Plan" (Patterson Britton & Partners; 2000)*
- "*Floodplain Development Manual" (New South Wales Government; 2005)*

Other technical data used in the hydrological and hydraulic analyses included:

- *Air photos of the Wingham region and its surroundings, (GTCC)*
- *Greater Taree Council's Aerial Laser Survey (ALS) floodplain topography data (GTCC)*
- *Site inspection and survey of built environment flow controls (WP, 2009)*
- *Riverbed survey data of the Manning River and Cedar Party Creek (WP, 2009)*
- *Geomorphic assessment of the river channel and it's prominent features (WP, 2009)*
- *A Digital Terrain Model (DTM) of the above water level topography derived from Aerial Laser Survey data undertaken by Greater Taree City Council*
- *A DTM of the below water level topography for the Manning River and Cedar Party Creek derived from WorleyParsons Hydrosurvey data*

The ALS ground data essentially provides a 3D representation of the topography. This was clipped, filtered and triangulated using waterRIDE tools to create a triangulated irregular network (TIN) digital terrain model (DTM). The clipping and filtering process firstly excludes points outside the floodplain and secondly reduces the density of points in flat terrain areas. A limitation of the ALS data is the fact that it maps the surface of the landscape, which is a problem for covered areas. The most important of these covered areas is the riverbed, but areas below bridges, culverts and urban areas also pose similar problems.

A DTM of the Manning River and Cedar Party Creek channel was developed from a Hydrosurvey undertaken by WorleyParsons in November 2009. Spot survey data points were measured at over 800 underwater locations. This data was combined with tidal data and known elevations, which was then interpolated and interpreted to form a digital terrain matrix of the underwater channel topography.

Structures such as bridges, piers, underpasses and culverts can all significantly influence flow levels and are not accurately represented within the ALS data. As a result, a survey was conducted by WorleyParsons analysing these structures in the Wingham region. Approximately 12 different structures were identified as important to flows and these were photographed and dimensioned. Details are included in the Appendix.

Historic flooding information, used for calibration and validation of the analyses, was available from the following sources:

- *"Manning River Flood History 1831-1979" (NSW Public Works Department; 1981)*
- *"Manning River – Flood Mitigation; Report on the March 1978 Flood – Manning River" (NSW Public Works Department; 1979)*
- *"Manning River Times" (Various Editions)*
- *"PINNEENA Version 8: New South Wales Surface Water Data Archive" (Department of Infrastructure, Planning & Natural Resources, NSW Government, 2004)*
- *"Manning River Flood Study" Volume 1 and Volume 2 (NSW Public Works Department, 1991)*
- *Bureau of Meteorology Historic Data Archives (Commonwealth of Australia 2009, Bureau of Meteorology)*
- *SES Archive Data (State of New South Wales through NSW State Emergency Service)*
- *The Wingham Community; information was collected through the use of a survey and community workshop. The survey was used to gauge general flood issues and information from the community whilst the workshop enabled residents to directly input their local knowledge into the calibration of the hydraulic model.*

4. HISTORICAL FLOOD BEHAVIOUR AND FLOODING MECHANISMS

The Manning catchment is a medium-sized coastal catchment with a high energy upper catchment region to the west. Rainfall over the Manning Catchment will tend to more rapidly affect these high energy regions, with orographic effects producing more intense rainfall in these regions leading to the generation of rapid, narrow albeit high-peaked hydrographs in waterways in this region. In the lower energy eastern portions of the catchment, levels would respond more slowly, achieving flatter, lower peak hydrographs that can be sustained for longer periods.

Wingham is in the transition zone between and as a result can experience a rapid and sustained rise in water levels.

Intense rainfall can occur throughout the entire catchment, although typically it focuses on the north-eastern Comboyne region of the catchment. Statistically, significant floods are most likely to occur in summer and autumn, being a result of ex-cyclonic storms and east coast lows. Figure 3 shows the monthly trend for significant floods plotted from historic data recorded since 1831 (where available).

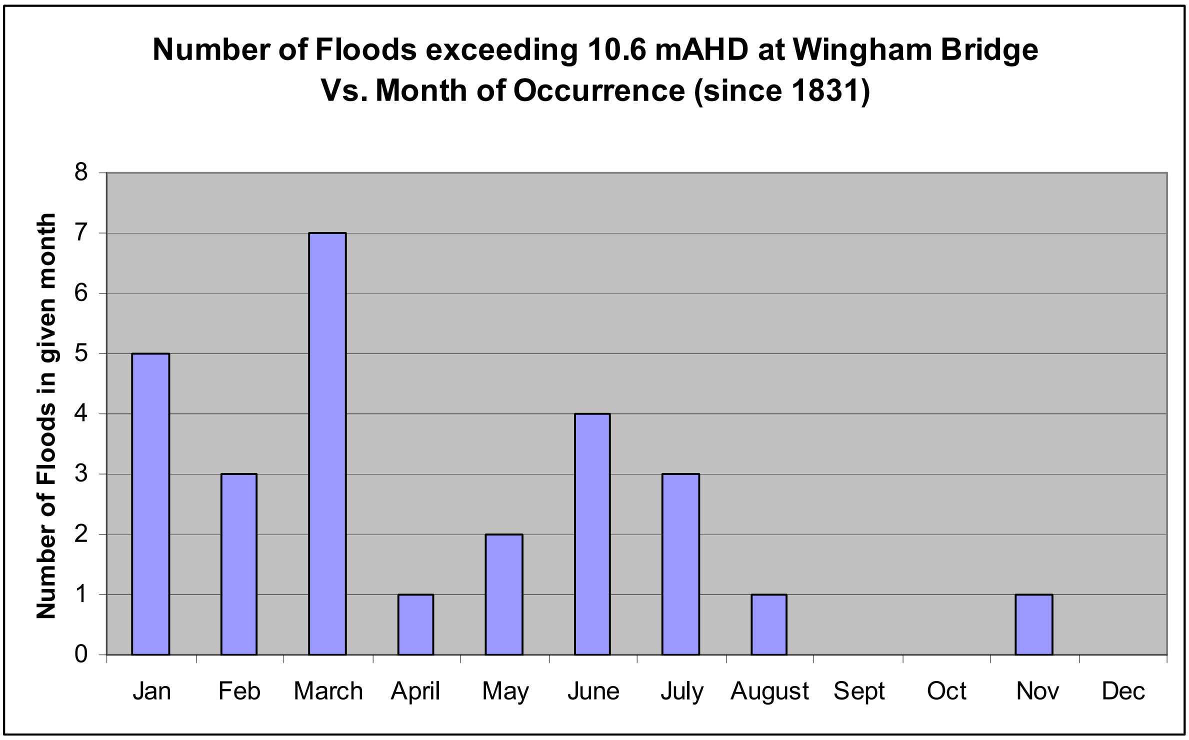


Figure 3: Historic occurrences of significant floods by month (from records dating to 1831)

Major flooding in Wingham does not primarily result from the Cedar Party Catchment. This is because the catchment's small size and steep upper gradients mean that the response to rainfall is always rapid. Peak water levels can be expected to occur within hours of the peak rainfall across the catchment, often whilst rainfall is still occurring. Furthermore, rainfall is likely to be distributed over the entire catchment, with perhaps more intense rainfall occurring to the north where orographic effects are more influential. Therefore, whilst a rapid rise in levels of Cedar Party (and Stony) Creek can be expected, this would similarly mean that levels would also rapidly fall. In contrast, the response of the Manning River, which is fed by a medium-sized catchment, would be much slower (although relatively rapid in contrast to the full spectrum of catchment sizes in NSW). Therefore, peak levels in the Cedar Party Catchment would never be expected to coincide with Peak Levels in the Manning Catchment, in the vicinity of Wingham. The difference in timings would be further exacerbated by the typical westerly progression of rainfall in this region, where moist warm air from the Pacific Ocean is directed into the upper atmosphere over the coast and inland.

More specifically, peak levels in Cedar Party Creek would typically occur on the rising limb of levels in the Manning River in the vicinity of Wingham. Flow through Cedar Party Creek would then rapidly begin to decrease, leading to a slowing in the rate of rise or even a decrease in levels. There would, however, be a point of inflection when rising levels in the Manning River would lead to a back-water flow in Cedar Party Creek. Therefore, depending on the severity of rainfall and the progression of the storm system, levels in Cedar Party Creek would rapidly rise, slow or decrease before rising rapidly again due to back-water from the Manning River. Depending on the severity and progression of the storm system, flooding in Wingham would follow a similar pattern.

In this way, Cedar Party Creek produces two distinct types of flooding events. One, resulting from rainfall over the Cedar Party Catchment would lead to a rapid rise, fall and sharp peak in water levels characterised by high velocity flows. The other, resulting from the Manning River back-water, results in a more prolonged hydrograph and lower velocity flows. As the result of one particular storm event, one or both of these flood events may occur.

The lowest portions of Wingham Peninsula, adjacent to the confluence of the two waterways, readily inundates for low level floods. Several low gullies on the peninsula, which run parallel with the Manning River, convey the flow across this portion of the floodplain. Apple Tree, East Combined and their side streets become increasingly inundated on the Peninsula with rising levels, whilst portions of Primrose and Mortimer Streets adjacent to Cedar Party Creek in central Wingham also begin to inundate. Flett and Queen Streets similarly inundate, due to the presence of relatively large gullies conveying flow from Cedar Party Creek.

4.1 Summary of the 1978 Flood Event

Due to its relatively recent occurrence and its impact on communities in the Manning Catchment, the 1978 flood is one of the most important historical flood events. Relatively large amounts of information exist on this flood event and as a result, a sequence of events can be recreated to assist with the calibration of this flood study. The following description summaries the information that was available on the 1978 flood event, providing an overview of the causes and results of the inundation that occurred.

Early on Sunday March 19th, 1978, a complex rain depression centred over inland NSW drove heavy rainfall over the Manning Catchment from east to west. The formation of another low off the east coast of Newcastle exacerbated conditions, with heavy rainfall continuing throughout that afternoon. River levels in the catchment responded swiftly and at 4pm, the Bureau of Meteorology (BoM) warned that "Major" flooding would occur within hours and predicted a peak level of 4 mAHD at Taree. At 7pm, the BoM revised this predicted flood level to 5 mAHD and warned that higher peak levels could be predicted and landowners should therefore be warned that there could be a fast rise in the Manning River.

At 1:30 am on Monday the 20th of March, 1978, Manning River levels were recorded at 3.40 mAHD at Wingham and swiftly rising. Soon after this, levels overtopped Wingham's Bight Bridge and portions of Wingham Peninsula. A peak level of 14.9 mAHD was achieved in Wingham sometime around 9 am that day. This equates to an eleven and a half metre rise in Manning levels at Wingham in approximately 8 hours. In other words, the rate of rise of the Manning River at Wingham was in the order of 1.5 metres per hour. This rate of rise could prove to be highly hazardous in regions where topographical variations lead to rapidly isolated areas and provides an insight into the risk to property and life that can exist. Downstream, a peak level of 5.45 mAHD was reached in Taree at 12 pm which was in excess of all predictions by the BoM.

In Wingham, houses were inundated and residents were evacuated from the north side of the railway line in Queen, Flett, Mortimer, Combined, Apple tree, Prince and Primrose streets. In total, over 30 houses were abandoned, mainly located in Primrose, Combined and Isabella Streets. Eight businesses were also inundated in the vicinity of Isabella and Primrose Streets. The Wingham-Taree train line was also cut by flood waters.

Older Wingham residents reported that the extent of the 1978 flood was much the same as the 1929 event.

The 1978 event is used both in the calibration of the hydrological and the hydraulic model and is discussed further in subsequent sections.

5. HYDROLOGY

The hydrological assessment is a major component of the flood study, which involves an analysis of the relationship between rainfall and flows in the catchment. The primary outcome from the hydrological component of the flood study is to provide inputs for the hydraulic model. To do this, a hydrological model is used, which simulates the accumulation of rainfall over the catchment, its movement along defined flow paths and the subsequent time-varying hydrograph produced at a downstream location.

The hydrology component of this flood study also involves a review of the previous flood studies' hydrology, which was undertaken using the RORB hydrology model. Design input hydrographs were generated for the Manning River and its tributaries for the 5%, 2%, 1% and 0.5% AEP and PMF events.

After being reviewed, hydrographs for the Manning River at Killawarra and Dingo Creek will be used as part of this flood study. A major component of work involved in the hydrology component of this flood study involves the analysis of the Cedar Party Catchment and the generation of updated design inflow hydrographs for Cedar Party and Stony Creeks.

The hydrological analysis of the Cedar Party Catchment was undertaken using the Watershed Boundary Network Model software (WBNM) version 1.04 (Jan 2007).

5.1 Review of previous Flood Study's Hydrology

The hydrology associated with the previous flood study was undertaken using the RORB runoff routing program. The concepts utilised in this model are similar to those of WBNM. The model consisted of 33 sub-catchment areas where calibration was primarily based on a parameter "k_c" which is a measure of the catchment's ability to store and delay flow. Secondary calibration parameters involved initial and continuing rainfall losses (as rainfall permeates the soil).

Calibration was undertaken using historic events, where rainfall and flow data was available. Tributary flows, such as that associated with the Cedar Party Catchment, are not gauged and therefore this method for calibration was and still is unavailable. Furthermore, calibration was also undertaken against recorded discharge-frequency data relating flows at different locations in the catchment. In addition to historic data calibration, an early version of WBNM was used to assist in the verification of the RORB hydrologic model.

In some cases, the rainfall hyetographs were synthesised in order to better reproduce recorded flows. Those associated with Dingo Creek were still often significantly different from those that were recorded. It is unclear why this occurred but as Dingo Creek is not the focus of this flood study, further analysis of the RORB model in this region was not undertaken. Manning River hydrographs at Killawarra matched better those that were recorded in some cases. Results for the 1976, 1978* and 1990 events were satisfactory when parts of the rainfall hyetograph were synthetised, whilst those simulated for the 1968 and 1977 were significantly different. One possible cause for this was the lack of available and reliable rainfall data.

The RORB model was verified against a parallel model generated using an early version WBNM focusing on the 1978 event. The adopted storage/delay coefficient (C) and nonlinearity exponent (b) were set at 1.1 and 0.23. These values are considered to be outside of the normal range and recommendations provided in WBNM documentation.

The calibration and verification of the model's variables can be considered as limited but satisfactory, considering the lack of available data and inconsistencies in data that was available. Design flood hydrographs produced by the RORB model for the Manning River and Dingo Creek were therefore adopted and are shown in Section 6.4.

5.2 Methodology

WBNM is an integrated hydrograph software package for hydrological studies on natural and urban catchments. The most recent version available, known as iWBNM, was used to model the Cedar Party Catchment. This software package uses a new graphical interface through Microsoft Excel® and Visual Basic (VBA). The WBNM software package is an event based hydrologic model and calculates flood hydrographs from storm rainfall hyetographs, using design storms from Australian Rainfall and Runoff (ARR).

The iWBNM model requires the sub-division of a catchment, such that runoff from each sub-catchment is routed to the next along a defined flow path. In order to divide the Cedar Party Catchment, air photos and the Council's ALS DTM were analysed in waterRIDE Flood Manager. The catchment was divided along visible topographical boundaries into 49 sub-catchments with a total catchment area of just over 143 km^2. The coordinates of the centroid of each sub-catchment, its area and the coordinates of its outflow point were calculated and input into the iWBNM model.

Sub-catchments containing the primary flow paths (the primary creek channel) were identified and lag and loss parameters were set. The primary variable allowing calibration, assuming the catchment has been correctly sub-divided, involves the lag parameter "C" and nonlinearity component "m". Other variables include the initial loss and continuing loss of rainfall to the soil. These two latter calibration variables can be associated with both pervious and impervious surfaces; however impervious surfaces were considered negligible in the Cedar Party Catchment and were therefore omitted.

Once these parameters have been set and the model calibrated, rainfall can then be simulated across the catchment, which can be variable based on a number of rainfall gauge locations or constant across the catchment. Design storms are derived from AR&R and storms with an AEP from 1% to the PMF can be simulated with durations that vary from 5 to 4320 minutes. In this way, the storm duration that produces the most extreme response of the Cedar Party Catchment for a given rainfall AEP can be examined and its hydrograph produced.

Hydrographs can be extracted at any sub-catchment within the model, and thus provide flow input at the approximate location required for the hydraulic model.

The following figure shows a visualisation of the hydrologic model where the output hydrographs from sub-catchments "3" and "5" were designated for the input of Stony and Cedar Party Creeks in the hydraulic model.

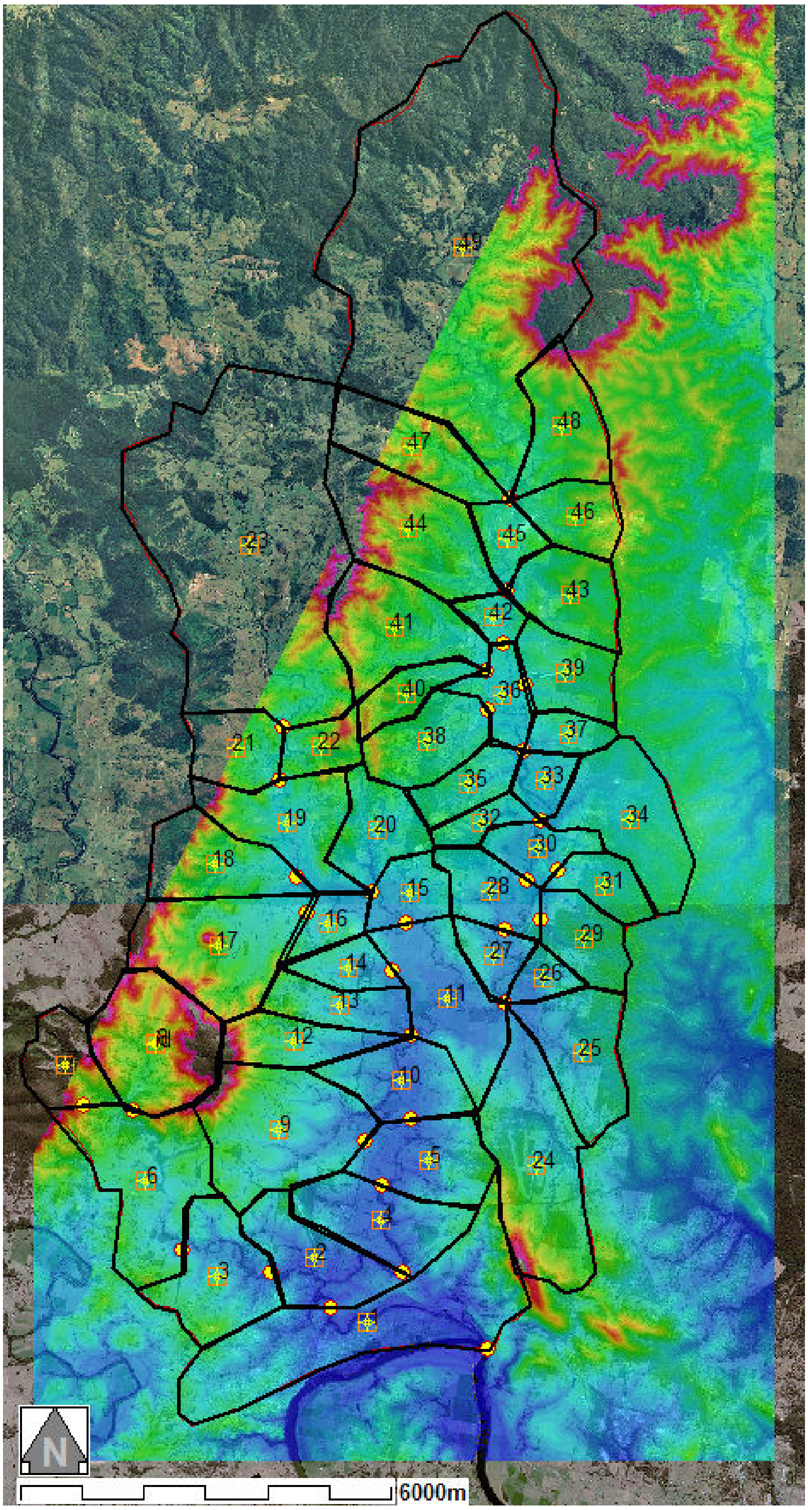


Figure 4: Visualisation of the Hydrologic Model of the Cedar Party Catchment. The model's sub-catchments are shown as black outlines with their centroids, inflows and outflows marked in yellow. Overlaid are the air photos and ALS DTM used to sub-divide the catchment.

5.3 Model Calibration and Verification

Due to the lack of any historic data for the Cedar Party Catchment that contains both rainfall *and* flow data, calibration of the hydrologic model was limited to literature, WBNM documentation, hydrology hand-calculations and the previous RORB model.

5.3.1 Initial Calibration Parameters

An analysis of the Lag Parameter, C, based on 129 historic storms on 10 catchments in eastern NSW was undertaken as part of the development of WBNM and provided in the documentation. The results showed that there is no trend for the Lag Parameter to either increase or decrease with increasing flood size, indicating that the built in nonlinearity is correct. Furthermore, the average value of C was 1.70. These results were confirmed over a wide range of flood and catchment sizes (flood peaks from 0.3 to 1400 m^3/s and catchments from 0.04 to 9000 km^2). They have also been confirmed by the studies of Webb and O'Loughlin (1981) and Sobinoff et al (1983) and for design storms by Boyd and Cordery (1989). Furthermore, results for a larger data set of 54 catchments and 584 historic storms (Boyd and Bodhinayake, Australian Journal of Water Resources, 2006), shows that the average value of C for catchments in NSW was 1.74. WBNM documentation does not recommend using a value of less than 1.60. With this strong experimental data, the lag parameter, C, was set to 1.74 for the hydrologic model of the Cedar Party Catchment. It should be noted that this differs from the value used in the previous flood study, which used a lag parameter value of 1.1, well outside of recommendations from this more recent research.

The nonlinearity parameter is recommended automatically by WBNM to be -0.23 (or m=0.77 which is defined as 1 minus the nonlinearity parameter). This is based on studies of floods in natural catchments by Askew (1968, 1970). Askew found that the nonlinearity parameter value did not vary for different catchments and adopted an average of –0.23.

The two remaining calibration parameters involve the initial and continuing loss of rainfall due to surface permeability. These were set at the average used for the RORB model in the previous flood study, being 20 mm of initial loss and 2.5 mm per hour loss thereafter. These values are in the range that is generally supported by the majority of literature and historic data.

The Hydrologic Model's parameters are not likely to be significantly varied during calibration because the majority of its parameters set to values that are based on solid scientific and experimental data for this region. Nevertheless, calibration was undertaken to fine-tune these parameters and verify the model's results.

5.3.2 Flow Volume Calibration

The first method used to check the validity of the hydrologic model involved a flow volume summation. This would indicate if there were any broad errors in the model or its parameters. A 9 hour, 100 year ARI storm was simulated over the catchment. The total volume output of the hydrologic model should therefore be approximately 23 158 270 m^3 after losses are subtracted (with a catchment size of 143 km^2). Numerical integration of the resulting hydrograph produced by the

WBNM model yielded a volume of 21 121 716 m^3 which is within 8.8% of what would have been expected.

Considering that there exist errors in numerical integration, this value is deemed to be accurate. No changes were deemed necessary to the model's calibration parameters.

5.3.3 Peak Flow Calibration using the Rational Method

The next step in checking the validity of the hydrologic model involved the Rational Method. The Rational Method, detailed in AR&R, is a commonly used method that allows the peak discharge of the catchment to be probabilistically estimated. This method was used to check the peak discharge of the hydrologic model for the 50 year and 100 year ARI design storms over the Cedar Party Catchment.

The formula of the Rational Method, according to AR&R Book IV Volume 1, is:

$$\mathbf{Q_y = 0.278\ C_y\ I_{tc,\ y}\ A} \qquad \mathbf{(1.1)}$$

Where

$\mathbf{Q_y}$ is the peak flow rate for ARI of "y" years (m^3/s)

$\mathbf{C_y}$ is the runoff coefficient for ARI of "y" years (dimensionless)

$\mathbf{I_{tc,\ y}}$ is the average rainfall intensity for a design duration of $\mathbf{t_c}$ hours and ARI of "y" years (mm/hour)

A is the catchment area (km^2).

>>The Cedar Party Catchment area is approximately 143 km^2

The first step in implementing the Rational Method is to obtain the Time of Concentration, $\mathbf{t_c}$, which, using the probabilistic interpretation of the Rational Method is given by:

$$\mathbf{t_c = 0.76\ A^{0.38}} \qquad \mathbf{(1.2)}$$

Formula 1.2 is valid for use in eastern NSW and is a revision by G.E. Mittelstadt and D.H. Pilgrim of the procedure developed by Pilgrim and McDermott (1982). It is based on data from 308 gauged catchments (Mittelstadt et al., 1987), using updated flood and rainfall data and is applicable to catchments up to 250 km^2 in area.

>>For the Cedar Party Catchment, t_c is approximately 5.00 hours, using formula 1.2

>>Then $I_{5,\ 50}$ and $I_{5,\ 100}$ are 28.1 mm/hour and 31.3 mm/hour respectively, using the Rainfall Intensity-Frequency distributions for the Cedar Party Catchment contained in Book II, AR&R Volume 1

$\mathbf{C_y}$ is given by: estimated based on the 10 year ARI runoff coefficient, multiplied by a Frequency Factor;

$$\mathbf{C_y = C_{10}\ FF_y} \qquad \mathbf{(1.3)}$$

Where

C_{10} is the runoff coefficient for a 10 year ARI event, which can be read from the maps produced in AR&R.

>>For the Cedar Party Catchment, C_{10} is approximately equal to 0.60, (Figure 5.1, AR&R Volume 2)

FF_y is the Frequency Factor for ARI of "y" years and is contained in AR&R. The Cedar Party Catchment lies in ZONE A and is elevated less than 500 mAHD so therefore separate formulae are used to calculate the value of FF_{50} and FF_{100} which are based on studies of eastern NSW catchments.

>>Then FF_{50} and FF_{100} for the Cedar Party Catchment are 1.170 and 1.277 respectively (Table 1.1, Book IV of AR&R Volume 1)

>>Therefore C_{50} and C_{100} are, 0.700 and 0.766 respectively, using formula 1.3

>>Using formula 1.1 of the Rational Method

>>Q_{50} = (0.278) (0.700) (28.1) (143) ≈ 782 m^3/s

>>Q_{100} = (0.278) (0.766) (31.3) (143) ≈ 953 m^3/s

It must be remembered that the Rational Method does not take into account rainfall losses and therefore will always over-estimate the peak discharge. The peak discharges of the simulated 50 year and 100 year ARI events, as output by the hydrologic model, are 776 m^3/s and 903 m^3/s respectively. These are within 1 – 5% of the values estimated by the Rational Method, showing that the hydrologic model is accurately calibrated and does not require any parameter changes thus far.

5.3.4 Verification using the 1978 Historic Flood Event

The hydrologic model was then verified using the 1978 storm event. Rainfall pluviographs were available from the BoM that show the recorded rainfall in hourly increments at three locations in the Manning Catchment that are in close proximity to the Cedar Party Sub-Catchment. The location of these gauges were Taree, South Comboyne and Number One.

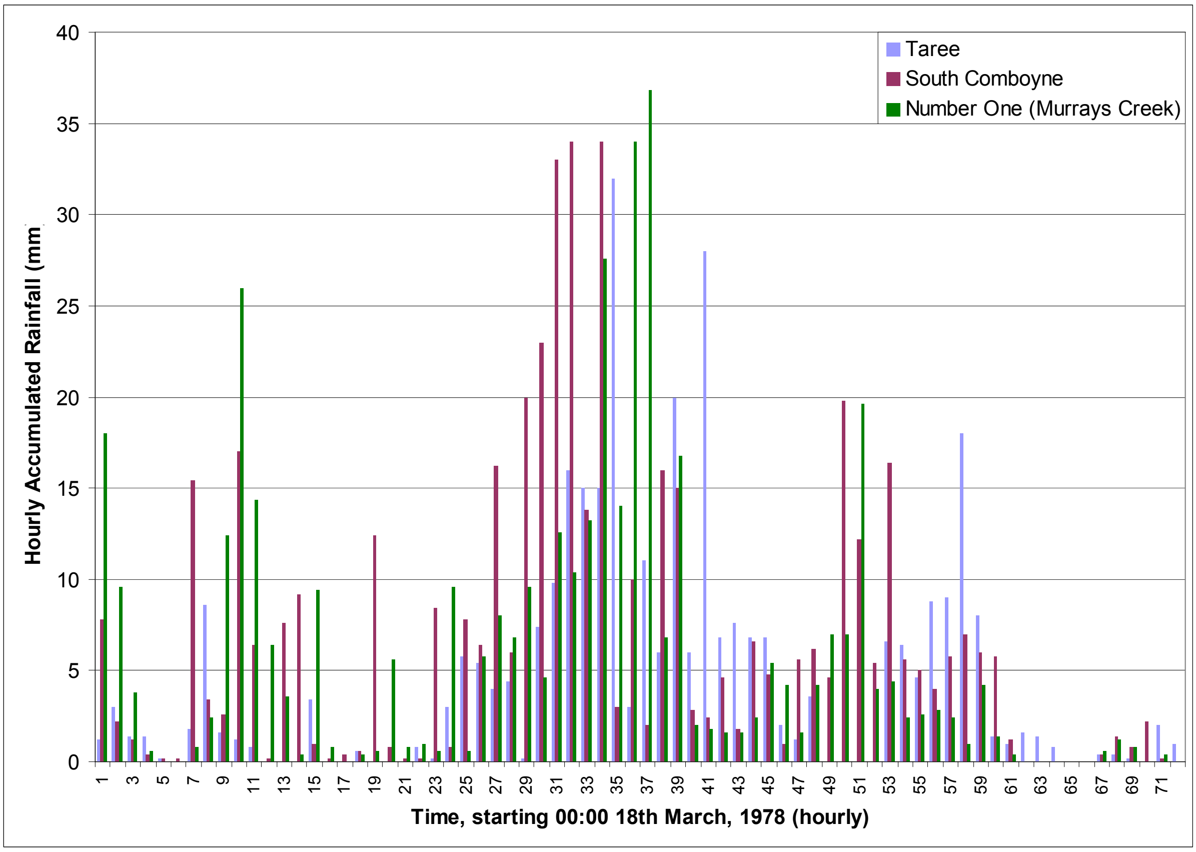
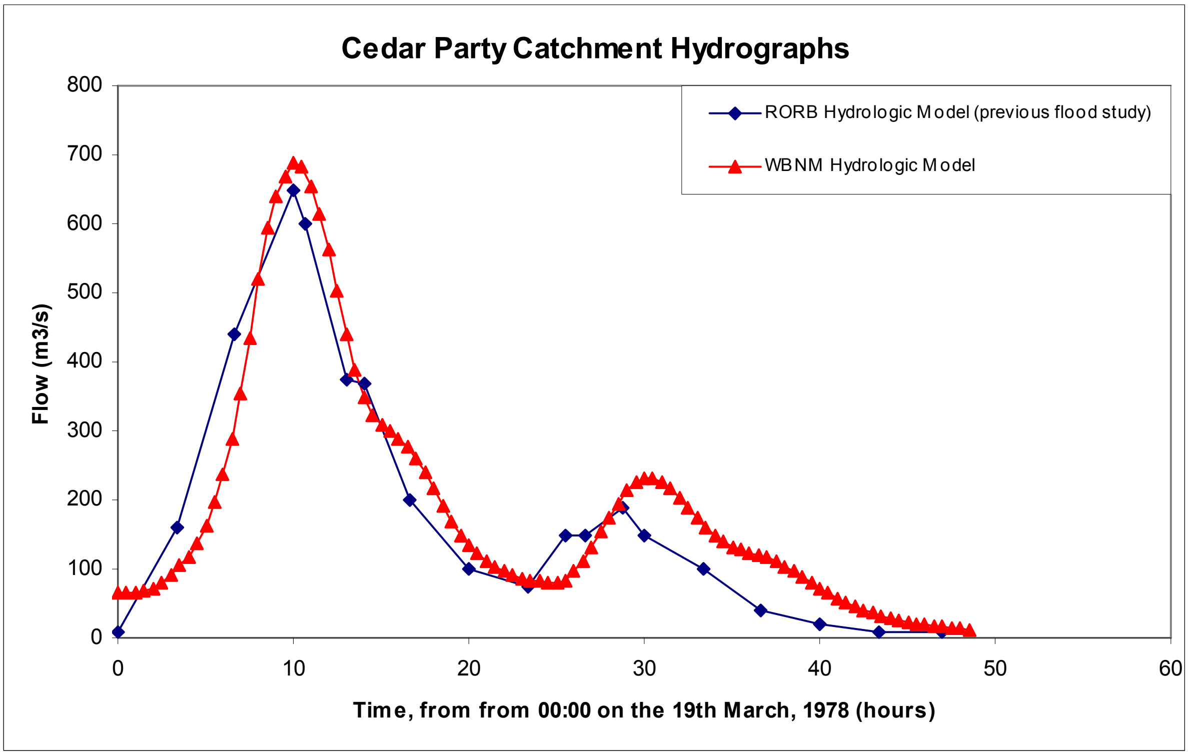


Figure 5: Pluviographs recorded by the BoM at three stations near the Cedar Party Catchment leading to the 1978 Flood

Rainfall data was input into the WBNM model and runoff was simulated according to the calibrated hydrologic model. Whilst no recorded hydrograph exists for the Cedar Party Catchment during the 1978 event, the outflow hydrograph of the WBNM model closely matched that obtained using the RORB hydrologic model of the previous flood study.



Apart from being composed of a much finer increment of time, the WBNM simulated hydrograph has a slightly greater peak. This is considered to be an extremely good comparison.

The WBNM Cedar Party Catchment Hydrologic Model is therefore considered to be sufficiently calibrated and verified.

5.3.5 Summary of Verified Calibration Parameters

Calibration of the Hydrologic Model did not require any changes to the initial model parameters because these parameters were selected based on thorough experimental data, as detailed in previous sections.

The following table summarises the calibrated Hydrologic Model parameters:

Table 1: Summary of Calibrated Model Parameters

Parameter	*Value*
Lag Parameter, C	*1.74*
Nonlinearity Parameter, m	*0.77*
Initial Rainfall Loss	*20 mm*
Continuing Rainfall Loss	*2.5 mm/hour*

5.4 Design Storm Simulations

The hydrologic model was used to simulate the PMF, 0.2% 0.5% 1%, 2%, 5%, 10%, 20% and 50% AEP Design Storms on the Cedar Party Catchment. The model was setup to simulate these storms within varying durations from 5 to 4320 minutes. Hydrographs for Cedar Party and Stony Creeks were then output for the most critical duration for each event where Cedar Party and Stony Creeks were treated as separate sub-catchments. That is, their critical duration was evaluated for each creek separately.

5.5 Results

The following table summarises the critical storm duration for each design storm event:

Table 2: Critical Storm Durations for the Cedar Party Catchment

	Critical Duration (mins)	
Design Storm	***Cedar Party Creek***	***Stony Creek***
50%	*2160*	*2160*
20%	*2160*	*2880*
10%	*2160*	*2880*
5%	*2160*	*540*
2%	*540*	*540*
1%	*540*	*540*
0.50%	*540*	*540*
0.20%	*540*	*540*
PMF	*180*	*180*

Hydrographs for these critical durations, output for Stony and Cedar Party Creeks is shown Figure 6 and Figure 7.

Tabulated data for these hydrographs are provided in Appendix A. The required design hydrographs produced in this section for Cedar Party and Stony Creeks will be used as inputs for the hydraulic simulations detailed in subsequent sections.

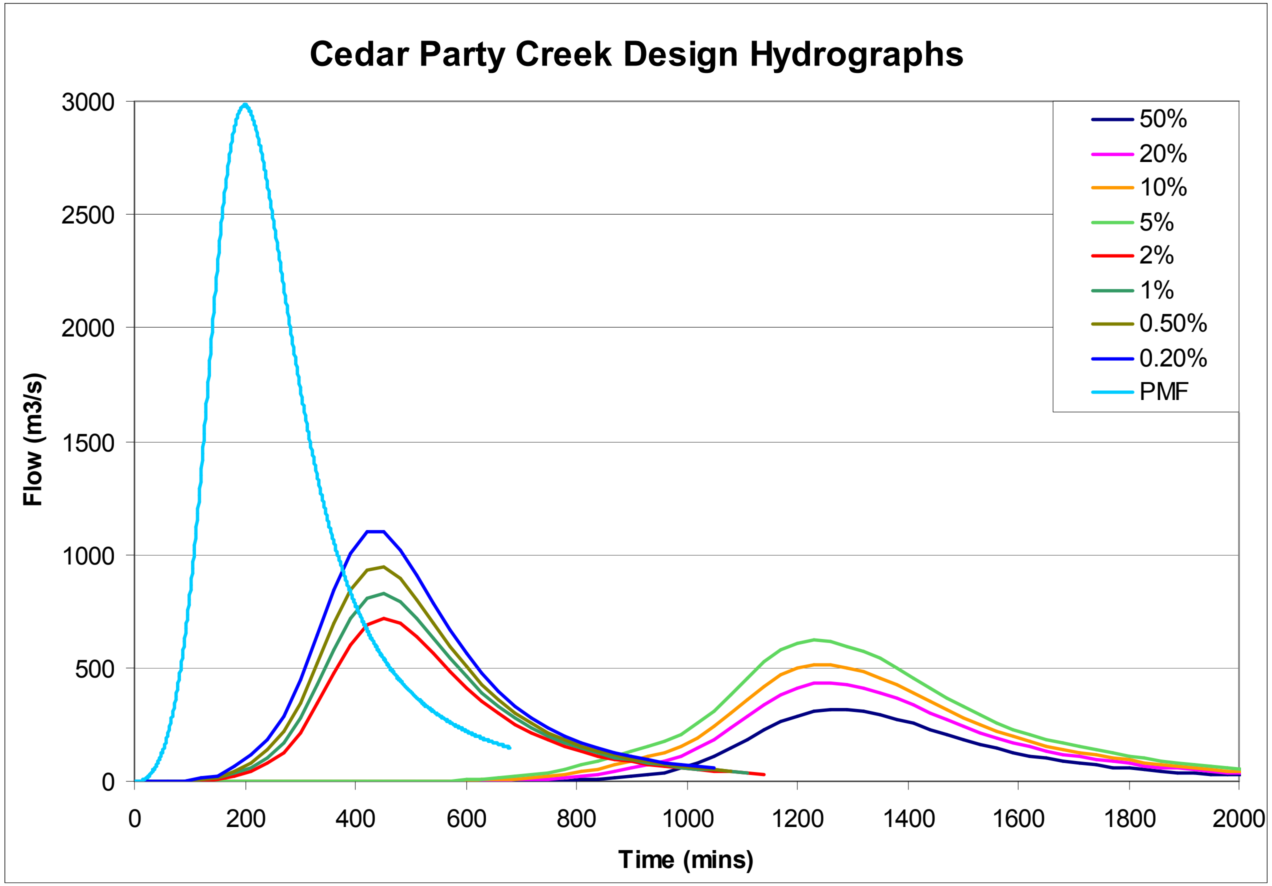


Figure 6: Design Hydrograph outputs for Cedar Party Creek

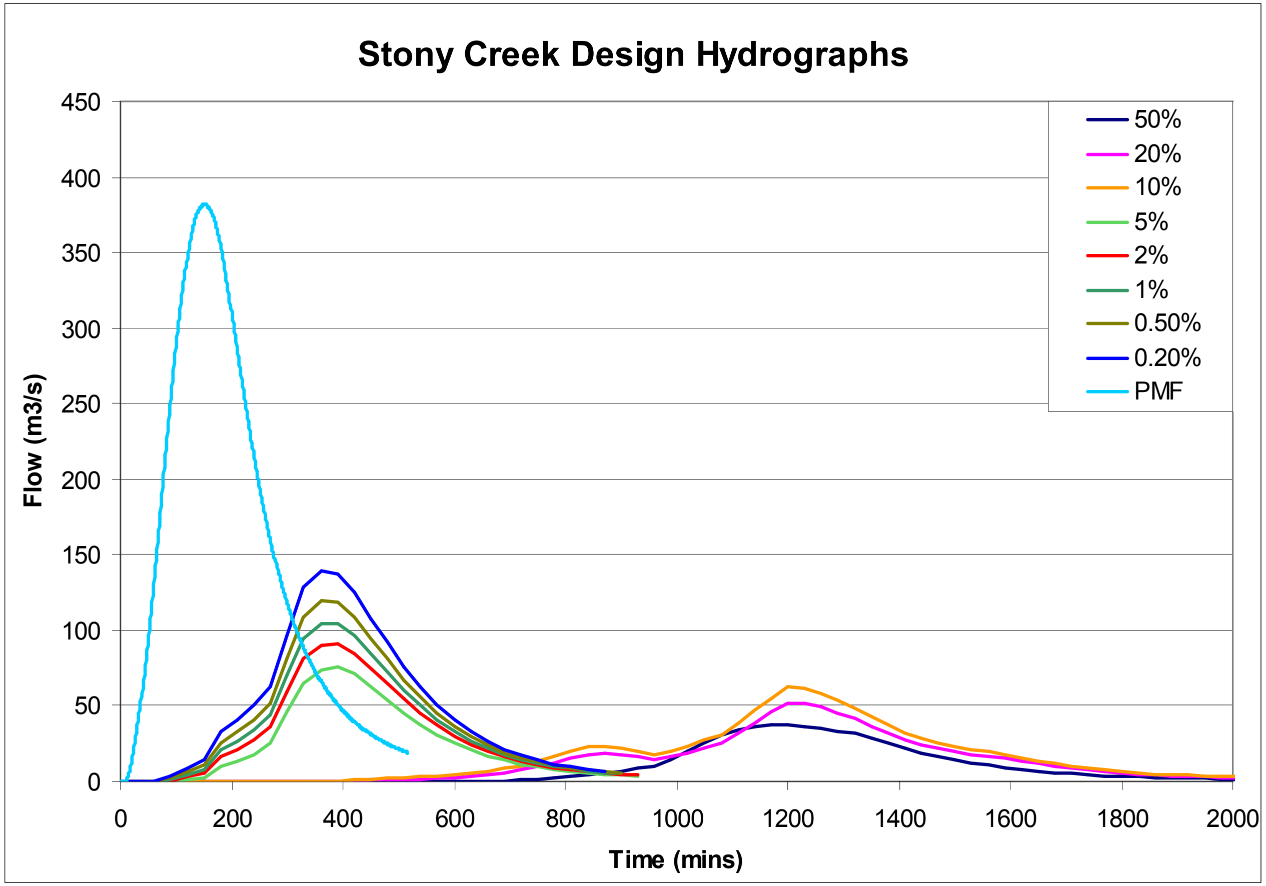


Figure 7: Design Hydrograph outputs for Stony Creek

6. HYDRAULICS

The primary outcomes of the flood study derive from the hydraulic analysis of the study area. The hydraulic analysis is performed using a hydraulic model to simulate the behaviour of flooding where the study area is discretised into 'elements' for use with the finite element program RMA-2. Discretisation is undertaken with a view of minimising analysis run-time whilst maximising adherence to the study area's "flow" properties. In other words, the number of elements is minimised based on the limit to which these elements can correctly generalise relevant properties associated with topology, resistance to flow (manning roughness) and velocity gradients. Each element takes a triangular or rectangular shape and is allocated roughness properties, with nodes located at each element vertex that are allocated topographical data.

The model utilises a series of input hydrographs describing the inflow into the study area with time. Similarly, the outflow of the model is regulated based on a defined downstream relationship. At all other boundaries, symmetry is maintained, however the ideal solution involves a model that does not have any other boundaries other than the defined inflow or outflow boundaries (that is, all other boundaries are "dry").

6.1 Review of previous Flood Study's Hydraulic Analyses

The previous flood study utilised an ESTRY 1D hydraulic model which is inherently less complex and therefore able to provide less refined results than the RMA-2 2D model. However on a broad basis, results from this model should be reliable if the proper inputs were properties input and adequate calibration was undertaken. It is unclear what roughness parameters were utilised in the ESTRY model as these are not explicitly provided in the relevant published reports.

Due to both the size of the study area and the nature of 1D hydraulic model, only limited topographical data was contained in the model. From Killawarra to just downstream of Wingham, the ESTRY model contained 7 nodes where topographical information could be stored. Furthermore, the nature of the 1D modelling processes means that between these cross-sections, the model "sees" or solves the equations of fluid flow based on a straight line channel that changes linearly between cross-sections. In contrast, the RMA-2 model contains a total of 10 526 nodes in the study area.

Two calibration techniques were used for the ESTRY model. One involved calibration to known tidal data and the other to historic flood events. The former sought to establish properties for the river channels, whilst the later was used to calibrate more elevated portions of the model. The model was calibrated sufficiently in both circumstances although problems were encountered with respect to historic calibration. The primary historic flood event of interest, from which the most data important data was collected, was the 1978 event. As mentioned, the Killawarra gauge station failed on the rising limb of the hydrograph and therefore complicated calibration of the model due to uncertainty in the primary model input. The "recorded" hydrograph therefore is a combination of early recorded data followed by estimates of the likely peak and shape of the hydrograph by the Department of Public Works NSW. In order to calibrate the model, the 1978 Killawarra hydrograph was re-estimated. The full extent as to how this re-estimate was undertaken is unclear, with even the early portions of the hydrograph prior to failure being re-estimated. Tributary inflows, notably in the case of Dingo Creek, were also re-estimated even though there is no information available that could be used to validly question the hydrograph recorded at the Dingo Creek gauging station (other than results obtained from the hydrological model). Similar re-estimates of the input hydrographs were undertaken for the 1976, 1977 and 1990 events, often without noted justification, as the Killawarra and Dingo Creek gauging stations had not failed during the respective events. Calibration, or more so validation of the model (the model was first calibration to tidal flow data) was undertaken with some high level and questionable modifications to model inputs.

Design inflow hydrographs, as produced by the hydrological model were input into the ESTRY model at the relevant tributary inputs, with a tidal boundary condition at the downstream end. The results appeared to be consistent between design flood analyses. All results were extracted several kilometres downstream of the Cedar Party and Manning River confluence and were subsequently used to produce a stage-discharge relationship to be utilised to derive the downstream boundary conditions of the RMA-2 model.

6.2 Methodology

This section provides further details on the methodology used to generate the RMA-2 model.

Two separate hydraulic analysis models were utilised. The second model involved the Cedar Party Catchment sub-set of the first model. In other words, the first model involved the discretisation of the whole study area focusing on the effects of the Manning River (however still including the Cedar Party Creek flows), whilst the second was reduced in spatial size in order to focus purely on Wingham and the effects of Cedar Party Creek.

More details on the differences between these models will be discussed in subsequent sections.

6.2.1 Manning Catchment Hydraulic Model

The Manning Catchment Hydraulic Model was the primary hydraulic model utilised in the flood study due to the resulting levels from high flows in the Manning River.

The study area extended from just upstream of Killawarra Bridge, downstream along the Manning River to Mondrook. The model incorporated a significant portion of Dingo Creek with its upstream boundary approximately located at the Wingham-Killawarra Railway Line. Along the main channel of Cedar Party Creek, the model extends just over 5.5 km upstream of the Wynter Street Bridge and includes almost 4 km of Stony Creek. Between these boundary conditions, all topography below 40 mAHD was included in the model.

The resulting study area was discretised using 15350 elements comprising of 36400 nodes (including mid-side nodes). Discretisation was based on minimising the model run time whilst capturing the maximum topographical and terrain roughness characteristics. Areas where large topographical gradients existed were discretised with many smaller elements whilst largely flat areas were approximated with limited elements. Similarly, terrain that was largely covered in the same vegetation or urban surface was grouped where appropriate. Flow paths were given a greater spatial resolution across their flow cross-section and in areas where flow changed direction, elements with aspect ratios close to 1 were utilised. This was undertaken with the use of both air photos and the Council's ALS survey data. This survey data was then used to allocate the majority of dry nodal elevations. WorleyParsons hydrosurvey data was used to allocate elevations to nodes below the water surface on the Manning River and Cedar Party Creek systems. In shallow parts of the upper Cedar Party Creek Catchment, where hydrosurvey recordings were not possible, estimates were used based on site observations. The below-water profile of Dingo Creek, which lies outside the primary scope of the Flood Study, was also estimated.

Site survey data was used to construct bridges, culverts and other covered flow paths within the model.

Roughness parameters were estimated (prior to calibration) based on vegetative covering that was observed during site visits and with the use of air photos. Typical Manning roughness values ranged between 0.025 and 0.080. The backup effect associated with the reduction in flow area at bridges due to piers was modelled by increasing roughness to 0.125 over the span of the bridge.

At the downstream end of the model, a stage-discharge relationship was used to define tailwater conditions. This was calculated (prior to calibration) based on ESTRY data at this location in the following way:

- The ESTRY design flood discharges were plotted against stage. Each design flood had a hysteresis effect (variation based on rising and falling limb of the hydrograph).
- An approximate line of best fit was drawn through these curves.
- This curve was tabulated and used as a boundary condition of the model. Between each tabulated value, data was linearly interpolated.

Design hydrograph inflows for the Manning River at Killawarra Bridge and Dingo Creek were taken from the previous flood study's hydrology model. Inflows for Cedar Party and Stony Creek's were tabulated based on the hydrology undertaken in Section 5.

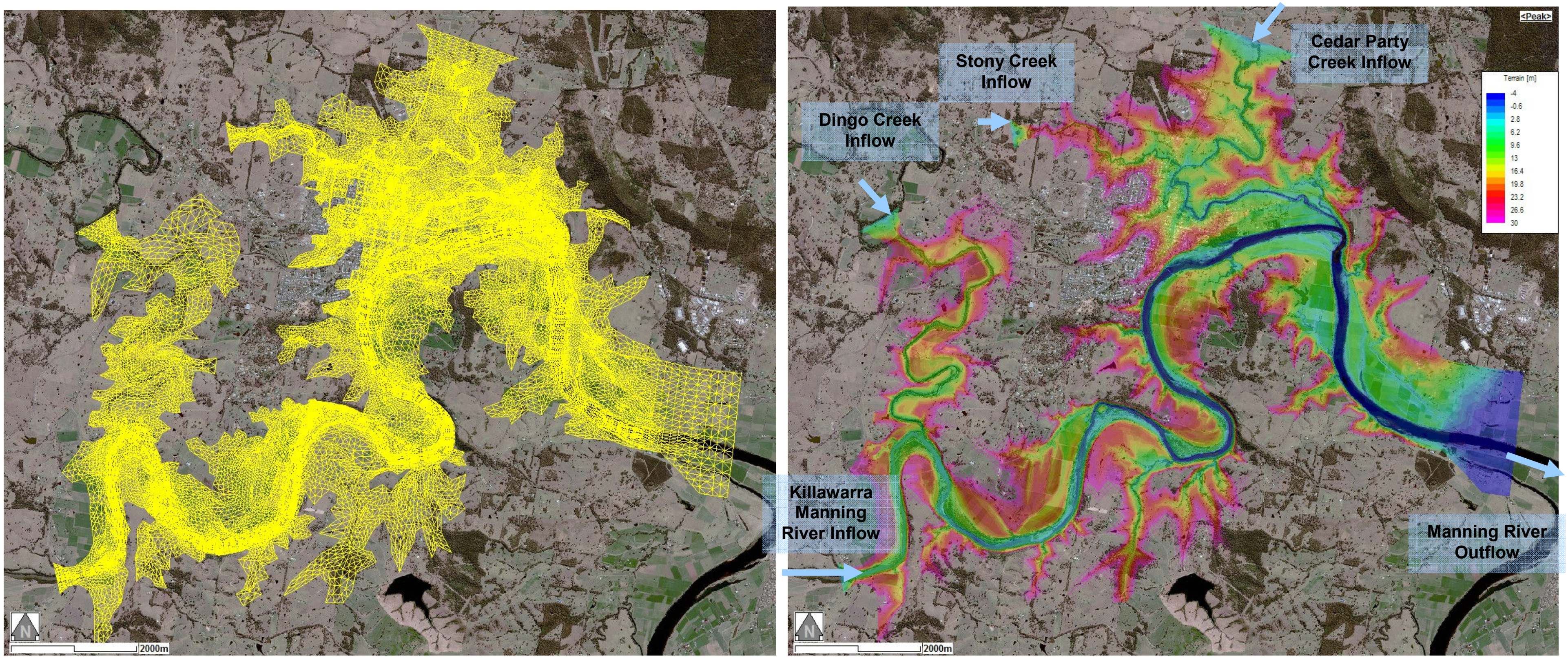


Figure 8: Manning Catchment Hydraulic Model showing the element mesh network of the discretised study area (left); showing the location of boundary inflow and outflows and highlighting the captured topographical information within the network indicated by element shading (right)

6.2.2 Cedar Party Catchment Hydraulic Model

The Cedar Party Catchment Hydraulic model is a subset of the Manning Catchment Hydraulic model.

The model was generated to analyse the specific effects of Cedar Party Creek on flooding in greater Wingham. Therefore the only important portions of the greater Manning Catchment Model were the portions north of the Cedar Party / Manning confluence. The model was therefore constructed by reducing the size of the greater Manning Catchment hydraulic model so that it only included Wingham and areas within the influence of Cedar Party Creek.

Due to the reduction in the model, the existing outflow boundary condition on the Manning River was no longer relevant. Therefore for this model, a new outflow boundary condition was required and was created several hundred metres downstream of the confluence of Cedar Party Creek. In order to establish controls for this boundary condition, the relationship between flow in the Manning River and Cedar Party Creek needed to be analysed further. This was undertaken after successful calibration and verification of the greater Manning Hydraulic model and after the design flood simulations had been completed.

The size of the Cedar Party Creek catchment in comparison to the overall size of the Manning Catchment, in combination with the nominal progression of rainfall in this region (from east to west as driven by an East Coast Low or tropical storm) would mean that the AEP of the flow (or level) in Cedar Party Creek would most likely exceed that experienced in the Manning River for a given rainfall event and for a point in time during an event. This is because the

a) rainfall is likely to cover a greater proportion of the Cedar Party Catchment than a portion of the Manning Catchment for a given storm, giving rise to an overall difference in the peak flow and level AEP of Cedar Party Creek relative to the Manning River

b) Cedar Party catchment will inherently respond to rainfall much quicker than the Manning Catchment, leading to a sharp rise and fall in its stage and flow hydrograph. At the point in time when peak flow is achieved in Cedar Party Creek, the flow in the Manning River is likely to be on the rising limb of its hydrograph and therefore below its peak flow level, even if rainfall happens to produces the same AEP hydrograph for both waterways.

Therefore given the response of Cedar Party Creek relative to the Manning River, (which was analysed after the design flood simulations had been completed for the complete Manning Hydraulic Model – Section 6.4), the hydrograph of Cedar Party Creek occurs whilst levels in the Manning River at the confluence remain almost constant. Therefore a constant elevation boundary condition at this location for the Cedar Party Hydraulic Model would be sufficiently accurate.

There were generally two ways that the elevation for this boundary condition could be determined. In reality, the exact level would vary depending on the storm system that encountered the region.

One approach, using point a) above, would be to assume that the level at the confluence of Cedar Party Creek would be determined by the peak level of a Manning River design flood that had an AEP less than that for a given design flood in Cedar Party Creek. For example, for a 1% AEP Cedar Party Creek design flood, the level at the Cedar Party confluence would be equivalent to the peak level of a

5% AEP Manning River design flood. The primary problem with this method is that the selection of a Manning River design flood AEP is complicated and not necessarily conservative.

One approach, that would be conservative, would be to assume that the rainfall intensity across the region generated the same flow AEP for the Manning River as with Cedar Party Creek. This would be conservative because, as mentioned, a lower AEP flow for the Manning River compared with Cedar Party Creek would be more likely to occur for a given storm due to catchment size. The level at the Cedar Party Creek confluence could then be designated by an average lag time between the peak of the hydrographs of Cedar Party Creek and the Manning River. Using historical flood events summarised in the previous flood study, the average lag time between the peak flow in Cedar Party Creek and the Manning River was 12.3 hours (that is, Cedar Party Creek peaked, on average, 12.3 hours prior to the Manning River in the vicinity of Wingham). Then for each design flood simulation using the greater Manning hydraulic model, the level at the Cedar Party confluence 12.3 hours prior to the peak would determine the level to be used for Cedar Party hydraulic model boundary condition. For example, the peak level at the confluence of Cedar Party Creek occurs at approximately 20.6 hours for the 1% AEP design flood simulation using the Manning Catchment Hydraulic Model (see results Section 6.4). The level at this location approximately 12.3 hours prior is in the order of 8.0 m AHD. Therefore, for the 1% AEP design flood on Cedar Party Creek, the downstream boundary condition would be set to a constant 8.0 mAHD. In this way, the downstream boundary condition is constant for each Cedar Party design flood but changes from one to the next.

This method was used to generate the boundary conditions at the downstream end of the Cedar Party Catchment Hydraulic Model.

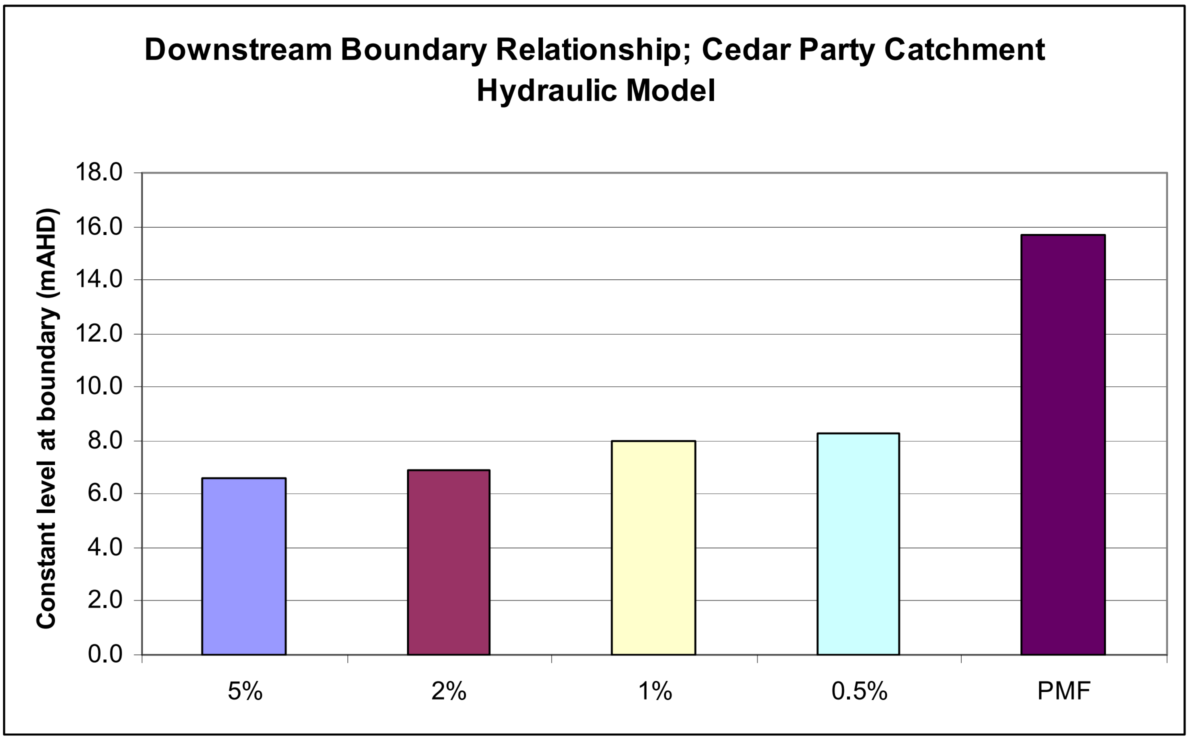


Figure 9: Cedar Party Catchment Hydraulic Model Boundary constant water level for each design flood

The Cedar Party Catchment Hydraulic Model consists of 6050 elements comprising of 14550 nodes (including mid-side nodes). The mesh within the study area, with the exception of that in the immediate vicinity of the new downstream boundary condition, was identical to that of the greater Manning Catchment Model, having the same topographical and roughness characteristics of the calibrated model.

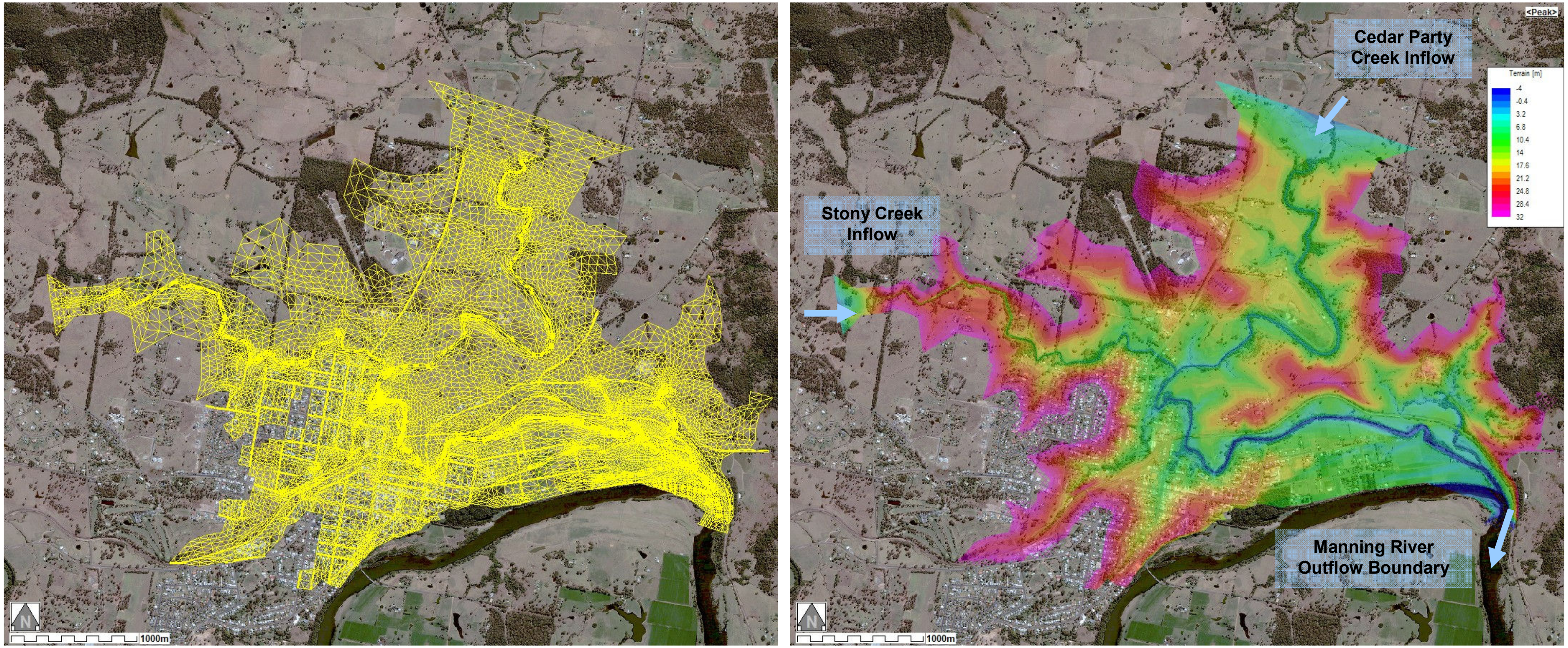


Figure 10: Cedar Party Catchment Hydraulic Model showing the element mesh network of the discretised study area (left); showing the location of boundary inflow and outflows and highlighting the captured topographical information within the network indicated by element shading (right)

6.3 Model Calibration and Verification

Model calibration essentially involved only the larger Manning Catchment Hydraulic Model because the Cedar Party Catchment hydraulic model is a subset of this and therefore would also be inherently calibrated (and was generated after the Manning Catchment Hydraulic Model had been calibrated and verified).

There are two primary calibration variables in hydraulic modelling; assuming that the element mesh has been completed correctly (in terms of flow paths, topography, terrain and element aspect ratios). These are roughness and the outflow boundary condition.

The outflow boundary condition, in the case of the Manning Catchment Hydraulic Model, is a stage-discharge relationship at Mondrook based on results from the ESTRY model. Due to the slight variation in this relationship between design floods, there is a degree of uncertainty which allows a small variation in this relationship to be made for calibration purposes. In other words, the relationship between the flood level and the flow rate at the downstream end of the model is bounded by results from the ESTRY model (which was itself calibrated to tidal data). Within these bounds, the relationship can be varied to ensure that the model accurately reproduces real flow conditions at this point.

Roughness is the degree to which flow is inhibited from flowing over a material, similar to the concept of friction between two solid bodies. RMA-2 utilises the Manning roughness coefficient which comes from the empirical Manning equation that broadly states that the flow rate in an open channel is inversely proportional to the channel’s roughness and proportional to its flow area and slope.

Variations in the model’s roughness coefficients therefore have the ability to “shift” the water surface up or down, whilst variations in the model’s downstream boundary relationship can “rotate” the water surface about the downstream boundary point. In this way, several known water surface (elevation) readings, in combination with known inflow rates, can be used to vary these calibration variables to ensure that the RMA-2 model accurately reproduces the behaviour of the real system.

The overall calibration and validation process then essentially involves simulating several real, recorded historic flood events. The broad process involves using one historic event to fine tune the calibration variables then using another two historic events to ensure that the fine-tuned model also simulates these accurately. Further minor changes can be made and the process reiterated.

Ideally, for each historic event, recorded flow hydrographs are input into the model and the resulting peak levels recorded are compared with levels produced within the model at the same geographical location. Roughness and / or the downstream stage-discharge relationship are modified in order to reduce discrepancies (depending on the magnitude and variation in the discrepancies in the levels) and then the process is iterated.

This is often complicated by the limited and unreliability of historic records. In the past, prior to the relatively recent development of finite-element flood modelling, the accuracy of data collection during a flood was naturally not seen as having as paramount an importance as is now required. This limitation can, in combination with many other factors, complicate the calibration process. Sources for discrepancies in historic data can be generally linked gauges (incorrect readings, damaged or sloping gauges, the location of gauges near hydraulic controls such as bridges or river bends), timing (levels, photos, flood extents are not necessarily read or recorded at the peak of the flood and/or recorded with time) and changes to the environment (urban development, clearing vegetation, manmade or natural changes to the channel). Calibration can be further complicated by conflicting data or records and estimates that are not labelled or that do not provide assumptions. Therefore in practise, discrepancies between historic events as simulated in the model can be difficult to reconcile.

6.3.1 Initial Calibration Process

Once the model geometry had been completed, a trial hydrograph based on the 1% AEP was run several times in order to test the overall behaviour and performance of the model to ensure there were no large errors in the mesh. This was also used as a first step in generating the time-step control file for the model. The time-step control file provides the model with an estimate time-step increment used to solve the fluid flow equations. Too large a time-step during which large changes in flow rate, velocity and / or depth are occurring can prevent the convergence of the time steps. This affects the stability and overall runtime of the model, but not the end results. Whilst the optimum time-step file required will be different for each set of input hydrographs, one file is usually a good basis for developing others due to the dependence of fluid flow on the model's geometry.

The next step involved the selection of several historic events that could be used for model calibration. Input hydrographs were required for the Manning River, Dingo Creek and Cedar Party / Stony Creeks. The Manning River gauging station at Killawarra has been in operation since 1949 and therefore could provide hydrographs for input into the model. Unfortunately, in many high flow flood events, the gauging station had failed. This meant that the most important portions of the hydrographs were often not directly recorded and instead had been estimated. In addition to the primary hydrograph inputs of the Manning River at Killawarra, additional inputs were required at the Dingo, Cedar Party and Stony Creeks. Whilst Dingo Creek was not part of the flood study, it was important to include these inflows into the model as these would essentially contribute to the flow in the Manning River upstream of Wingham. A gauging station exists on Dingo Creek at Munyaree Flat where data was available for some period of time. No gauging station exists or has ever existed on Cedar Party or Stony Creeks, and therefore these historic inflows would need to be estimated based on rainfall data, if available and other similar historic events.

It was necessary to choose historic flood events based on the availability of recorded information and in this way, these events tended to be more recent. Additionally, a variety of flood intensities (or reoccurrence intervals) is desirable as each would help to calibrate an additional portion of the floodplain, whilst combining to thoroughly calibrate the primary river and creek channels and the downstream stage-discharge relationship. However, these two criteria for selecting flood events are not necessarily congruent.

The three most significant flood events in the Manning Catchment occurred in 1866, 1929 and 1978, whilst the 1978, 1990 and 1995 floods have the most data and information available. Other floods where information existed were for those that occurred in 1968, 1976 and 1977. After thoroughly examining the available data, it was decided that the 1978, 1990 and 1995 flood events would provide a good opportunity to calibrate the model as these floods varied significantly in severity *and* all three had sufficient amounts of data available.

Furthermore, more recent flood events are also useful from the point of view that the river channel and floodplain are more likely to be unchanged, whereas more distant flood events may mean more significant differences in geomorphology and / or urban development.

The following sections detail the calibration and validation of the model for each of the selected historic events.

6.3.2 1978 Flood Event

The 1978 flood is in the top three largest flood events to have been recorded in Wingham in the 178 years of European settlement with a peak level of approximately 14.87 mAHD being recorded at Wingham Bridge. Furthermore, experience with more significant flooding is limited in living memory to the 1978 flood. Consequently, a large amount of data exists on the 1978 event that is useful in terms of this flood study and is summarised in Section 4.1.

Tributary Hydrograph Inputs

Hydrographs for Cedar Party and Stony Creeks were developed using the hydrology model and recorded rainfall. These were validated against those produced independently in the previous flood study.

Hydrographs for Dingo Creek were available in two forms. The first was a recorded hydrograph from the Munyaree Flat gauging station (208019), approximately 10km upstream of the Dingo Creek limit in the model. The second was the ESTRY hydrograph used in the previous flood study. It is unclear how this hydrograph was estimated as its peak flow differs from that of the recorded hydrograph by 230%.

Without any obvious reasoning behind the need for such a distinctly different hydrograph, the recorded hydrograph was adopted. Two modifications were made to this hydrograph because it was recorded 10 km upstream of the Dingo Creek boundary on the model. One modification involved a delay in time based on a nominal flow velocity between Munyaree Flat and the Dingo Creek model boundary. Trial runs indicated that an approximate average flow velocity of 2.5 m/s, would shift the hydrograph forward in time by approximately 1.11 hours. In other words, it would take a further 1.11 hours for the flow at Munyaree Flat to reach the geographical location of the upstream Dingo Creek boundary condition in the model. The second modification to the hydrograph was associated with the catchment size between Munyaree Flat and the Dingo Creek model boundary. It is estimated, from broad topographical features, that this portion of the catchment is equal to approximately 12% of the total catchment size upstream of Munyaree Flat. Therefore, to account for the increased flow, the hydrograph recorded at Munyaree Flat was increased by 12%. In other words, between Munyaree Flat and the upstream Dingo Creek boundary condition of the model, an estimated increase of 12% in flows was likely to have occurred due to this portion of the catchment.

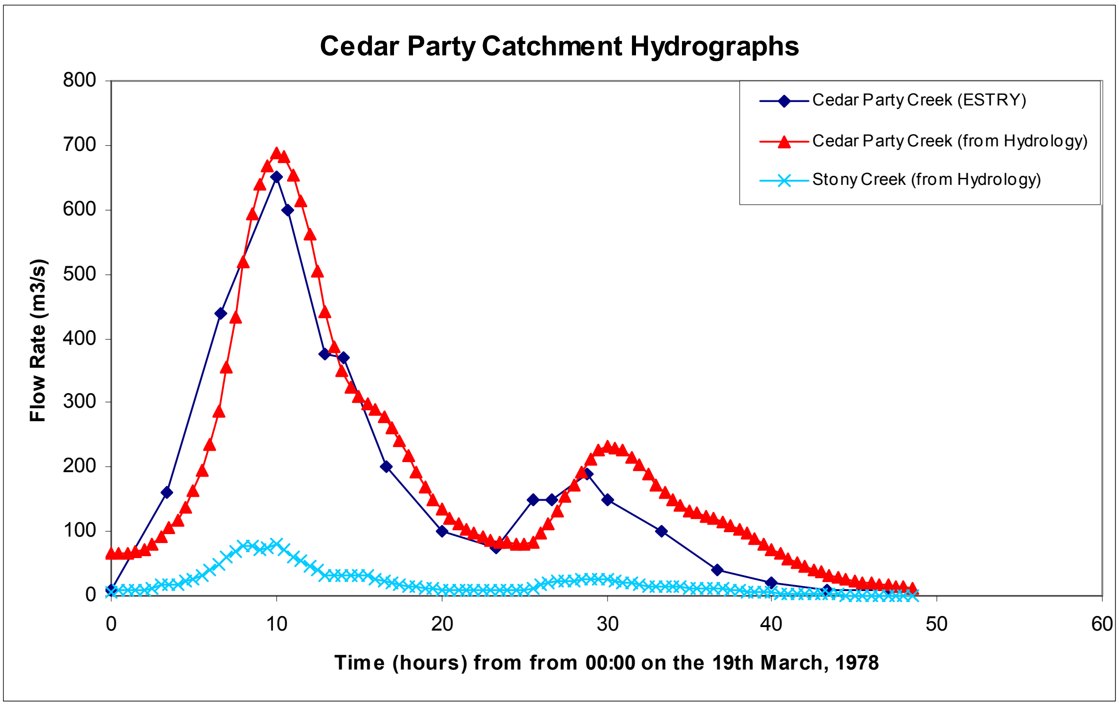


Figure 11: Cedar Party (& Stony) Creek hydrographs. The ones marked as "hydrology" were used in the calibration simulations.

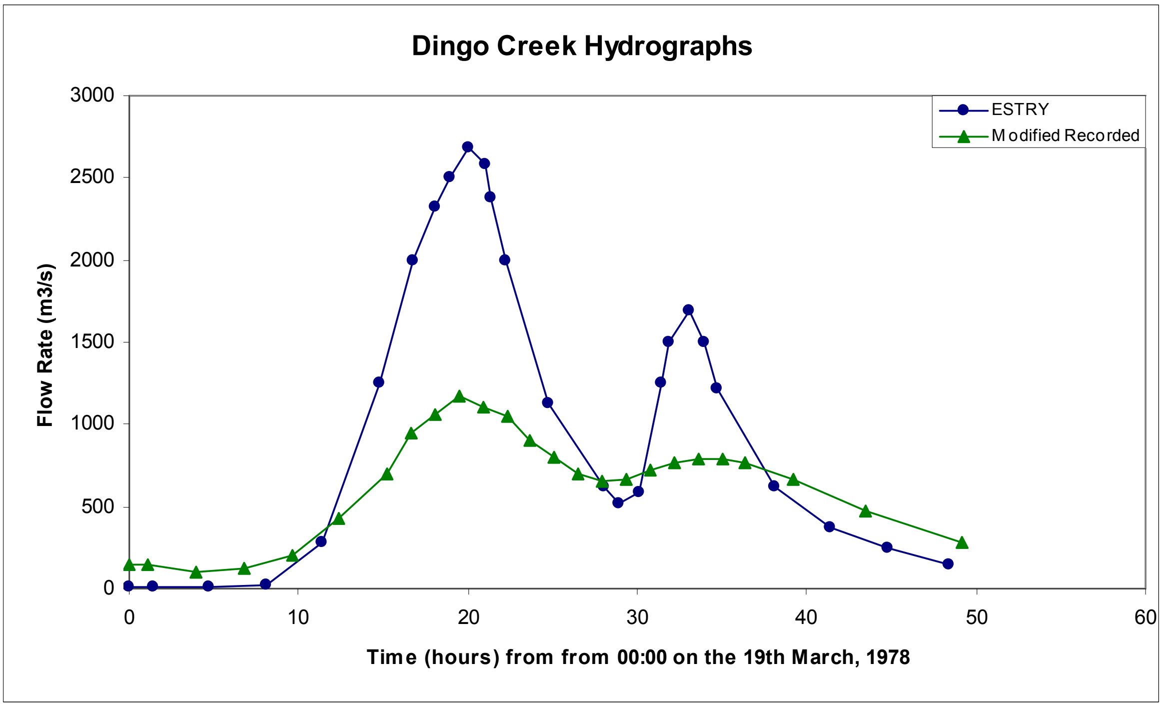


Figure 12: Dingo Creek hydrographs showing the modified recorded hydrograph used in the calibration simulations

MANNING RIVER HYDROGRAPH INPUT

Hydrographs for the primary model input on the Manning River at Killawarra were also available in two forms.

The first is the Recorded/Estimated hydrograph; where the Killawarra gauging station (208400) had recorded the level and flow through the Manning River with time up until approximately 11:00pm on the 19th of March, prior to the peak. At this point the gauge had failed and as a result, the hydrograph from this point in time was estimated based on debris. This hydrograph consists of a single peak of approximately 10 500 m^3/s occurring 1:00am on the 20th of March, 1978. The accuracy of this hydrograph is highly questionable as it does not match in shape to those stations upstream or downstream of Killawarra.

The second is the ESTRY hydrograph. This is a re-estimate based on upstream gauge information and the RORB hydrology model using synthetic pluviograph data. This hydrograph consists of two peaks with the first of 9700 m^3/s occurring at approximately 1:30am on the 20th March and the second of 10 300 m^3/s occurring approximately 8 hours later. This hydrograph is much broader than the recorded hydrograph and therefore consists of a greater volume of water than the recorded hydrograph. It follows more closely the pattern of flow in gauges upstream of Killawarra and of that recorded in Wingham. For these reasons it is regarded as having a better reliability than the recorded hydrograph. However a primary problem with the ESTRY hydrograph is the fact that it does not agree with the portion of the recorded hydrograph prior to the gauge failure. Furthermore, without detailed explanation of the steps performed to generate the hydrograph, it could not be wholly adopted without a separate analysis.

The previous flood study provided the recorded hydrographs at two upstream stations in the Manning Catchment. These are the Nowendoc River at Rocks Crossing and the Gloucester River at Doon Ayre. These recorded hydrographs were plotted and shifted forwards in time to coincide with the time when their flows would have reached Killawarra. The delays used were 3.6 and 3.0 hours respectively, assuming an approximate average flow velocity of 2.5 m/s.

From a broad topographical assessment of the Manning catchment, it is estimated that the sub-catchments upstream of the Nowendoc River at Rocks Crossing and the Gloucester River at Doon Ayre are equal to approximately half the total catchment upstream of Killawarra. The other half lies in the region between these sub-catchments; and is drained by the upper Manning and Barnard Rivers and their tributaries. This sub-catchment is ungauged and therefore is the only “unknown” portion of the total catchment contributing to flows at Killawarra. In other words, the superposition of flow from these three broad sub-catchments would produce the actual Killawarra hydrograph.

Therefore, in order to produce an accurate estimate of the Killawarra hydrograph, an estimate of the hydrograph in the Manning-Barnard Rivers was required. Part of this was available by subtracting the time-shifted hydrographs of the Rocks Crossing and the Doon Ayre gauges from the recorded portion of the Killawarra gauge (prior to failure). This showed a steep rise in the Manning-Barnard hydrograph. An analysis of the rainfall distribution and the resulting response of the Rocks Crossing and the Doon Ayre hydrographs showed that the Manning-Barnard hydrograph would have consisted of two primary peaks in flow separated by between 10 and 14 hours. The first peak was larger in both instances, indicating that the initial rainfall was more intense. Both the Rocks Crossing and the Doon Ayre sub-catchments are of a similar size; however the Rocks Crossing hydrograph is almost twice that of Doon Ayre. This, in combination with the recorded accumulated rainfall, shows that the storm was more intense in the north than in the south. Furthermore, the hydrographs show that the storm driving the rainfall must have progressed from the south to the north as Doon Ayre responded prior to Rocks Crossing. The location of the Manning-Barnard sub-catchment is north of the Doon Ayre catchment and therefore would be expected to have received more intense rainfall, slightly later than the Doon Ayre sub-catchment. Using this information, in combination with the estimate of the size of the Manning-Barnard sub-catchment, a hydrograph was estimated.

The flow in the Manning River at Killawarra was then estimated by summing the three sub-catchment hydrographs (one estimated and two recorded). The estimated hydrograph was iteratively calibrated using data collected from the Wingham Community in combination with the Hydraulic Model. The result was a broader hydrograph with two peaks. The peak flow rate is estimated to be 10,123 m^3/s occurring at 7:18am on the 20th of March, 1978. This is approximately 2% to 4% different to the peak values produced by the previous study and that of the NSW Public Works Department respectively (Figure 14).

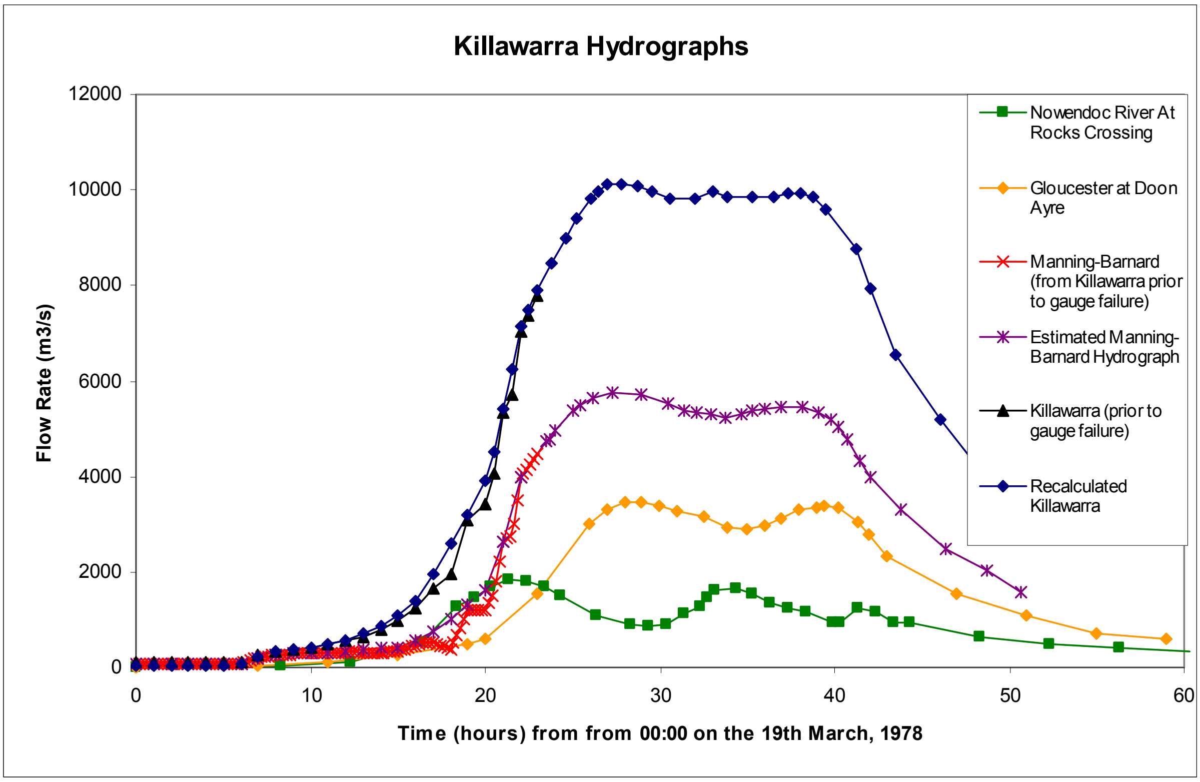


Figure 13: Manning River hydrographs, showing the how the Killawarra hydrograph was calculated from two upstream recorded hydrographs, the recorded rise in levels at Killawarra prior to failure and an estimate of the hydrograph from the remaining ungauged portion of the Manning Catchment upstream of Killawarra. The recalculated Killawarra hydrograph was used for calibration simulations.

This hydrograph represents the best estimate of the true Killawarra hydrograph, as it was constructed with two recorded upstream hydrographs and an assessment of the upstream catchment's response to the rainfall distribution that occurred. This hydrograph is therein referred to as the "recalculated Killawarra hydrograph".

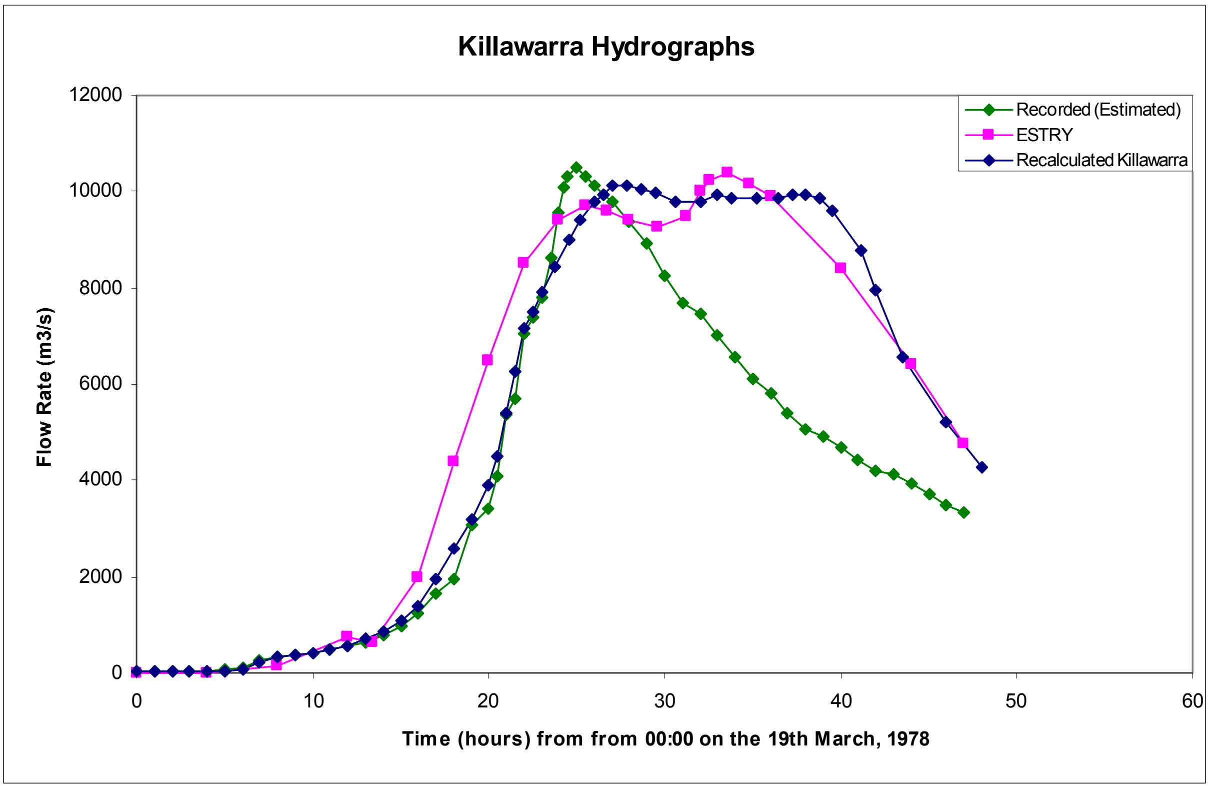


Figure 14: Comparison of the Manning River hydrographs at Killawarra. Shown is the recorded/estimated hydrograph available from the NSW Department of Public Works, the ESTRY estimated hydrograph used in the previous flood study and the Recalculated Killawarra hydrograph used in this Flood Study.

CALIBRATION OBJECTIVES AND DATA

The initial roughness of the model was estimated based on site observations and an air photo assessment in combination with published data found in literature. Roughness initially varied from 0.025 to 0.080 (Manning Coefficient). Elements in clear, open water (river and creek channels away from the banks) were allocated a roughness 0.025 and smooth surfaces (such as some banks areas, grassland, urban surfaces) were allocated a roughness of 0.030. These constitute the two most important roughness coefficients in terms of model calibration as they cover the majority of the major flow paths. The roughness of the riverbed (in open water) is generally adjustable between 0.020 up to 0.040, although from experience with previous hydraulic models, should be in the range of 0.025 to 0.035. The starting point of 0.025 was considered a good lower bound.

Using the input hydrographs, the objective of the 1978 calibration simulations was to vary the roughness and / or the downstream stage-discharge relationship (within the limits of the ESTRY model) in order to obtain as close a match to the data recorded in Wingham during the 1978 event as possible. This data comprised of:

- A stage hydrograph recorded at the Wingham Bridge gauge, as recorded by the NSW department of Public Works with a peak level of 14.87 mAHD recorded between 9 am and 12 pm on 20th March, 1978;

  This is considered to be a good source of data on the 1978 flood event because the data was recorded *during* the 1978 flood and not at a later stage. However, because the data was recorded by hand, it should still be treated with caution, especially because the gauge that existed in 1978 has since been replaced and it is unclear if what condition the gauge was in at the time. The data consists of level readings in increments of between 30 minutes and 3 hours. At the peak of the flood, a 3 hour increment of time was used between gauge readings meaning that the real peak level of the flood may have been slightly different to that recorded.

  A significant problem encountered with this recorded data was associated with the location of the gauge. The general location of the gauge reading was near the Bight Bridge, on the bend of the Manning River however it became apparent through the initial Hydraulic model simulations that the exact location was very important because levels differed significantly in this region of the model (due to the effects of the bend, bridge and composition of the riverbed).

  Initially the gauge was assumed to be in the same location as the current Wingham gauge, operated by Manly Hydraulics Laboratory (MHL). However records from the Bureau of Meteorology (BoM) indicated that there were two gauge locations in this area in the past. Furthermore, the coordinates of the current gauge (as referenced by MHL and the BoM to be on the second most southern pier of the Wingham Bridge) do not match with those recorded by the NSWPWD to which the 1978 records were kept which locates the gauge on the northern bank of the Manning River adjacent to Wingham Tinonee Road which leads to Wingham Bridge. The difference between the two locations is approximately 145 metres. Reference to a change in the gauge location was found (Reference 4) indicating that after the 1978 event, the existing gauge was moved to where it currently exists.

- A map of the approximate flood extents in Wingham; produced by the Wingham Municipal Council;

  This is considered to be a fair source of data as the flood extents however it is unclear when the map was produced or what information was utilised to produce it. It therefore can not be relied upon heavily, but nevertheless gives a good *overall* indication of the extents of the 1978 flood.

- A collection of ten spot level readings located in Wingham, summarised by the NSW department of Public Works from *"field investigations during 1979-1980 (survey, personal interviews etc.)"* and the Wingham Chronicle (newspaper).

This data is considered to be the least reliable for a number of reasons. Firstly, surveys and personal interviews were undertaken one to two years after the event. Whilst the errors may only amount to 0.1 or 0.2 metres, this is significant in terms of model calibration. This is highlighted by the fact that two readings, within 85 metres of each other near Guilding Street on Wingham Peninsula, differ by 0.2 metres. Furthermore, levels reported within local media, may not necessarily use the correct datum origin and are generally not obtained for scientific purposes and can not be assumed to have the associated level of accuracy.

- The Wingham community survey and workshop;

  This data was collected by WorleyParsons and Council directly during the process of the Flood Study and consisted of community information (opinions, flood extents, levels of inundation, flow velocities, photos). Whilst this information was highly detailed, not all information was accurate due to the lapse in time and inconsistencies or unknowns when a measurement or photo was taken.

  Data was considered reliable if it was consistent with other data sources or information provided independently by more than one member of the community. Data that was readily measurable was also considered to be satisfactory.

  The information obtained was very good in a general sense, providing the most detailed input in terms of flood extents, than was available from any other source.

  Additionally, two levels obtained from the Community Workshop on East Combined Street suggest that the 1978 peak water level was between 12.8 and 12.9 mAHD on the peninsula. At one location, the resident produced a water level marking which remained from the event itself, whilst at another location the former resident gave a depth that he had noted at the time from the inside of his property. These depths, when added to the ALS data, produced levels that agreed within 0.1 metres.

The most significant issue concerning all data obtained was that whilst it agreed in a general sense, a more detailed look nearly always produced conflicting information. This inevitability associated with conflicting historic flood data arises from the fact that the collection of flood information is not paramount when a flood is occurring. As a result, the “best” calibration of the model often involves a compromise that agrees most with *all* reliable sources of data.

CALIBRATION PROCESS

All three Killawarra hydrographs were run through the hydraulic model for comparative purposes (the recorded/estimate, ESTRY and recalculated Killawarra hydrographs). These initial runs produced peak levels at Wingham Bridge that were approximately 2.0, 1.5 and 1.0 metres less than the peak recorded level respectively.

Several more model runs were produced for each hydrograph whilst the river channel and bank roughness were modified from 0.025 and 0.030 to 0.035 and 0.037 respectively. These values were viewed as being the upper realistic limit for the manning roughness of these areas. Levels remained significantly lower than those recorded at Wingham Bridge, and furthermore, produced a different stage hydrograph than that which was recorded. The recalculated hydrograph was however, thus far the best at reproducing recorded results.

At this stage it became clear that the stage-discharge relationship at the downstream boundary condition would need to be fine-tuned. Without an additional recorded peak level along the Manning River within the study area, it was necessary to perform simulations on the other selected historic events (had additional recorded levels been available on the Manning River in the study area, these could have been used to rapidly adjust the boundary condition as it would cause the water surface to “rotate” about the boundary). These analyses showed that the downstream end of the model was pushing the water surface up too much for low flows and down too much for higher flows. This was adjusted and the changes made were well within the results obtained from the ESTRY model.

Further simulations were performed for the 1978 event using all three hydrographs whilst varying the base model roughness from 0.025 to 0.035. At this stage, in combination with the simulation of the other historic events, it became clear that the recalculated Killawarra hydrograph was far superior to the recorded/estimate and ESTRY Killawarra hydrographs. These two latter hydrographs consistently produced levels that were below those recorded, and were significantly different from the recorded stage hydrograph and did not match the recorded flood extent map. This was the case for the most extreme calibration parameters which produced opposing affects on the results of the 1990 and 1995 historic events. In this way, the recalculated hydrograph was legitimised as the sole hydrograph to be used for calibration of the 1978 event.

As mentioned, a primary problem with all data sources was that whilst in a general sense they were in agreement, the high level details were inconsistent. Therefore it was found that, for example, the model could be calibrated to match the stage hydrograph recorded at Bight Bridge, however this would lead to high inconsistencies with the extent produced on Wingham Peninsula obtained from the community. Therefore the calibration of the model to the 1978 event was done iteratively in combination with both the 1990 and 1995 events such that one set of calibration parameters provided the best fit to all historic events (as well as a best fit to the various data sources for the 1978 event alone).

From this, a river channel base roughness of 0.028 and a river bank roughness of 0.033, in combination with the modified stage-discharge downstream relationship, was found to produce the best match (all other roughness coefficients were unchanged from the initial assessments). The stage-discharge relationship is shown in Section 6.3.5.

Using these calibration parameters, the maximum peak level recorded in the vicinity of Bight Bridge produced in the simulation was 14.21 mAHD, which is within 0.66 m (5%) of the recorded peak level of 14.87 mAHD. Furthermore, the stage hydrograph produced at the Wingham Bridge was found to approximately match that which was recorded (Figure 16).

On Wingham Peninsula, these parameters produced a level of approximately 12.85 mAHD which is in close agreement with the 12.8 and 12.9 mAHD measurements obtained from the Community Workshop.

The flood extents produced in the simulation also match well with those obtained from the community in relation to both Appletree Street and East Combined Street.

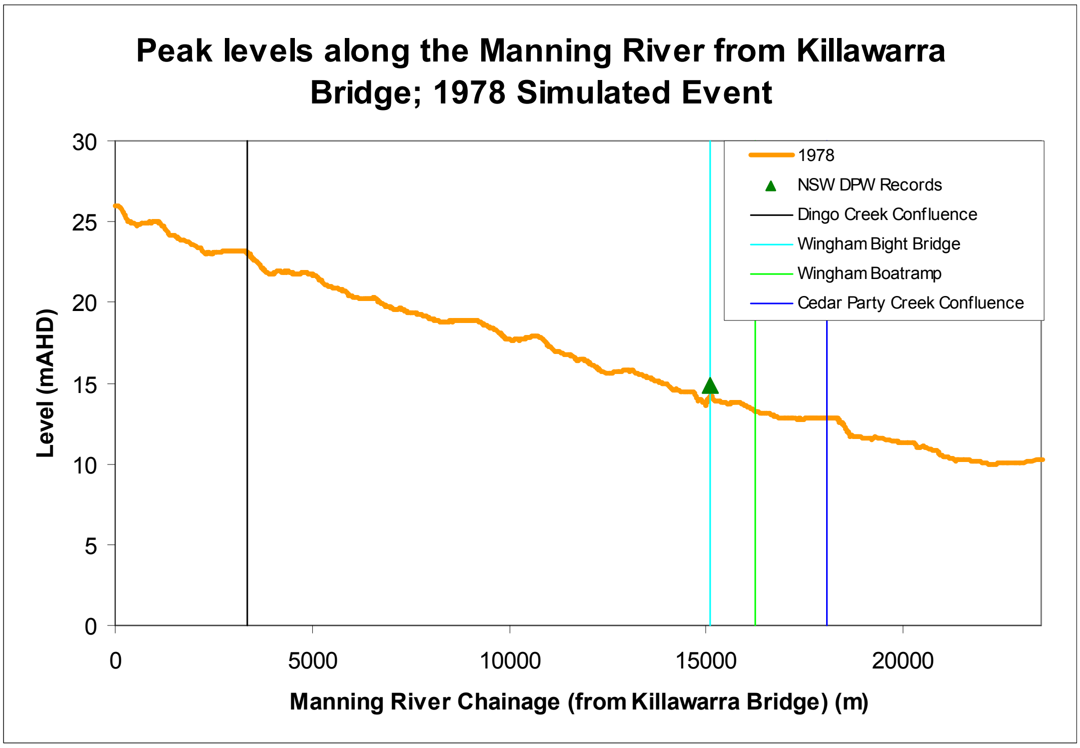


Figure 15: Calibrated Water Surface Profile along the Manning River for the 1978 flood event

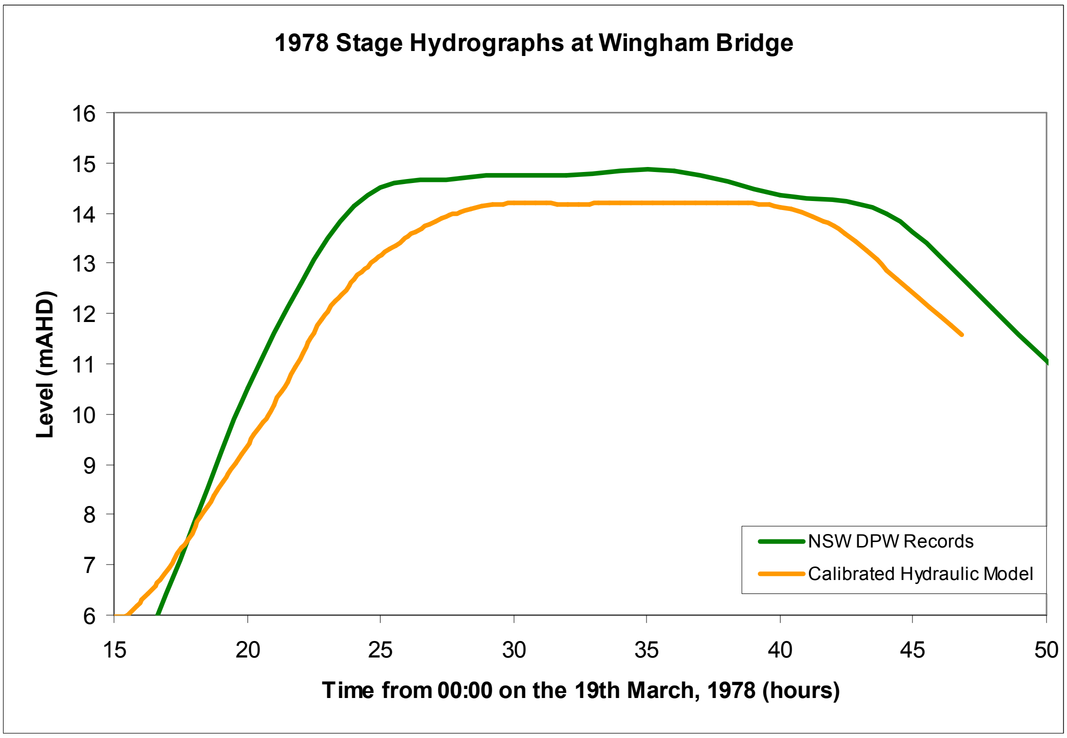


Figure 16: Comparison of the stage hydrographs at Wingham Bridge for the 1978 flood event

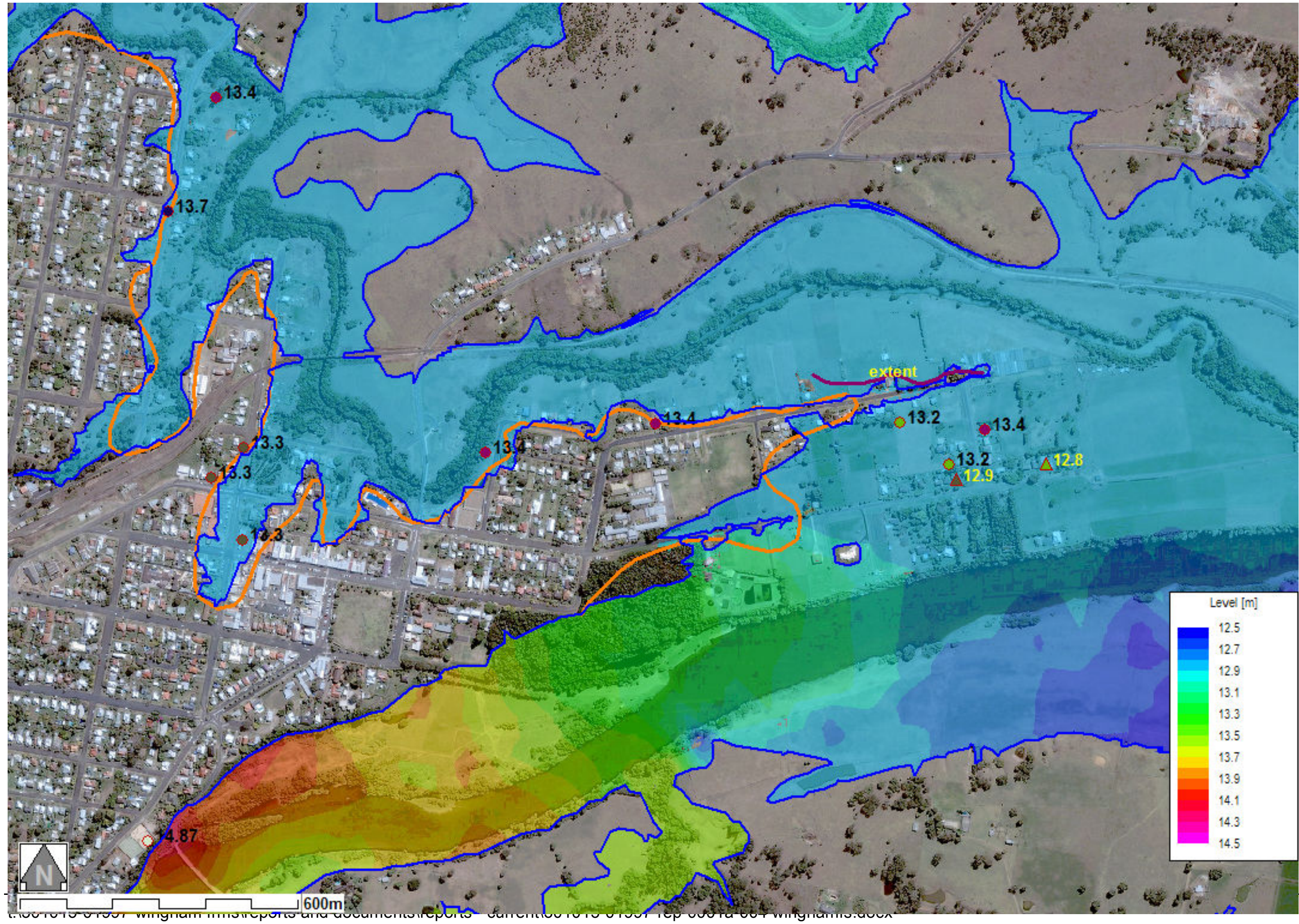


Figure 17: Peak water level plot for the calibrated 1978 flood event. The peak water level through the majority of Wingham is approximately 12.85 mAHD. This represents the best fit to all supplied data. The extents of the modelled 1978 flood have been highlighted in dark blue.

Shown in orange is the approximate flood extents recorded according to the Wingham Municipal Council.

Shown in maroon is a portion of the approximate flood extents on West Appletree Street recorded by local residents.

The location (and magnitude) of spot water level readings obtained from the NSWPWD are shown as circles with black writing.

The location (and magnitude) of spot water level readings obtained from the community workshop are shown as triangles with yellow writing.

Furthermore it must be remembered that this represents how the 1978 flood would affect the area with the *current* topography, landscape and riverbed features.

6.3.3 1990 Flood Event

The 1990 flood is the most recent flood to have occurred in Wingham that was categorised as "significant" according to the SES and BoM scale of flood events. Rainfall occurred throughout the Manning Catchment on the 2nd, 3rd and 4th of February, 1990 with levels peaking in the Manning at Wingham on the late on the 4th. The peak level at Wingham Bridge was approximately 12.73 mAHD.

Tributary Hydrograph Inputs

Data for the 1990 event was not available at the Dingo Creek gauge.

Hydrographs for Dingo, Cedar Party and Stony Creeks were estimated based on an assessment of the catchment rainfall that occurred in combination with other known events, such as the 1978 and 1976 events. Furthermore, the contribution of flows from these tributaries was unlikely to significantly influence peak flood levels and therefore was not viewed as paramount to the calibration analyses. The reasons behind this assessment were that flows in these tributaries would:

- peak substantially before the Manning River in the vicinity of Wingham
- be expected to be significantly less than those that occurred during the 1978 event because the rainfall accumulation in these portions of the catchment was less than half and occurred over a longer period of time.

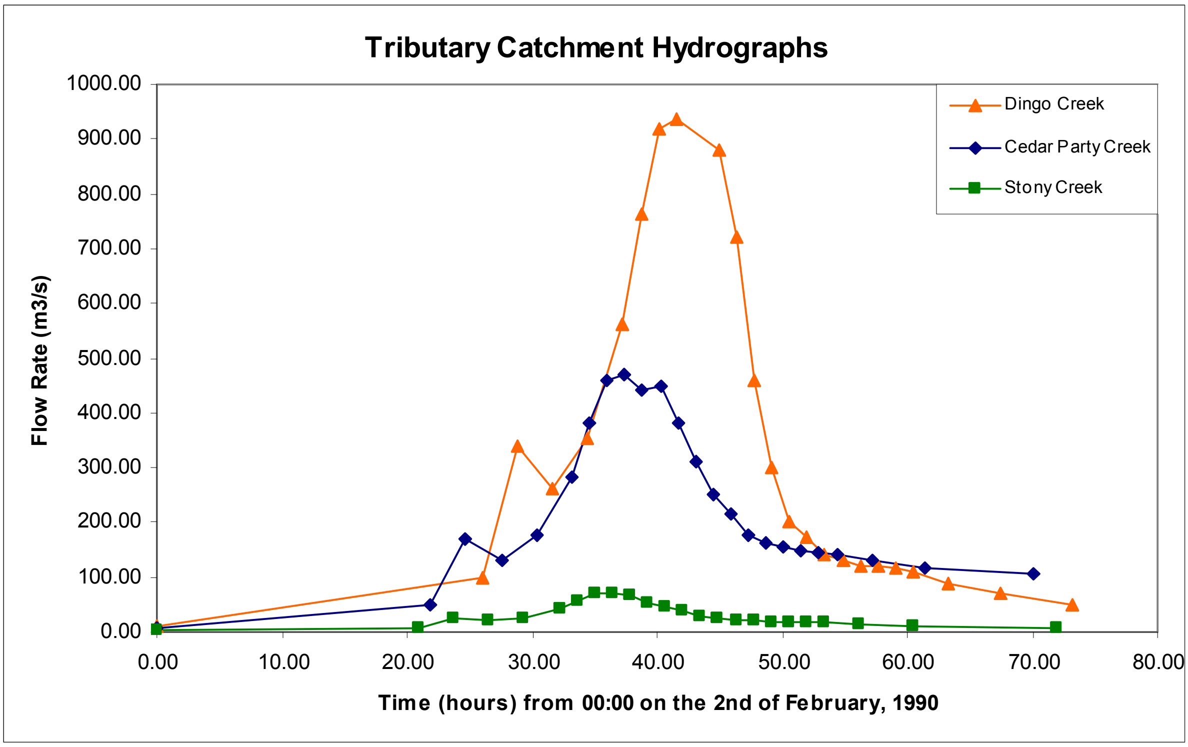


Figure 18: Estimated 1990 tributary inflow hydrographs used in the calibration analyses

MANNING RIVER HYDROGRAPH INPUT

The complete Manning River hydrograph was available from the Killawarra gauging station 208400.

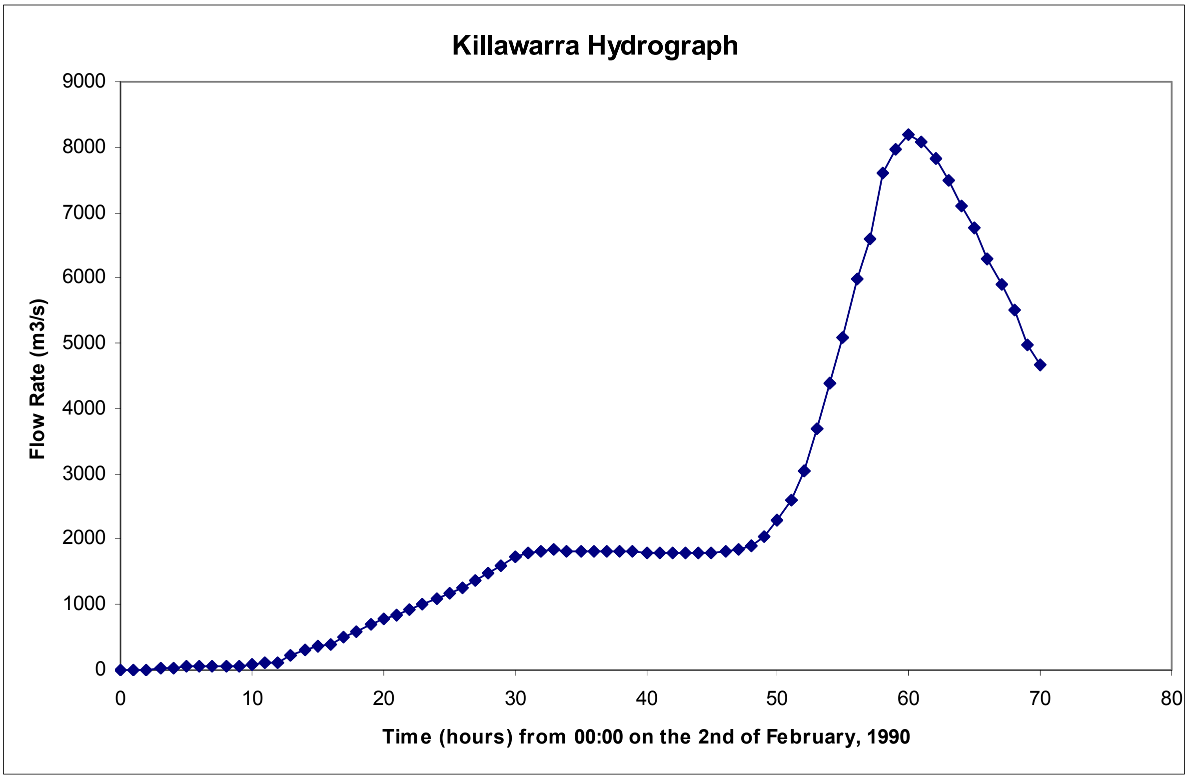


Figure 19: Recorded 1990 Killawarra inflow hydrographs used in the calibration analyses

CALIBRATION OBJECTIVES AND DATA

Using the input hydrographs, the objective of the 1990 calibration simulations was to verify the roughness and the downstream stage-discharge relationship that had been obtained with the calibration of the 1978 event (calibration was done iteratively between the 1978, 1990 and 1995 events such that one set of calibration parameters provided the best fit to all historic events).

For the 1990 event, the calibration data comprised of:

- A peak water level of 12.73 mAHD recorded at the Wingham Bridge gauge by the NSW DPW;

  It is understood that the gauge was still manually read at this time, and the exact location of this reading at the Bight Bridge was not clear. In the location of the current gauge (as per the MHL coordinates), water levels are generally lower than those where the older gauge, used during the 1978 event, was located.

- The Wingham community survey and workshop;

Similarly to the 1978 event, information collected from the Wingham community through a survey and workshop was very good in a general sense, providing the most detailed input in terms of flood extents, than was available from any other source.

CALIBRATION PROCESS

Initial simulation runs were performed with the 1990 event in order to fine-tune the tailwater stage-discharge relationship. Base roughness of the model was also varied from 0.025 to 0.035 in order to check the outcomes of changing the tailwater condition.

After initial calibration of the model to the 1978 event was complete, the same parameters were used for the 1990 event in order to verify their accuracy and this process was iterated.

The calibrated river channel roughness was set to 0.028 and the bank roughness to 0.033 (all other roughness coefficients were unchanged from the initial assessments). With the calibrated tailwater relationship, these parameters produced a peak water level of 12.63 mAHD at Wingham Bridge, which is within 1% of the recorded peak of 12.73 mAHD and well within the limits of accuracy expected. Furthermore, the flood extents also matched key areas where members of the community had provided information.

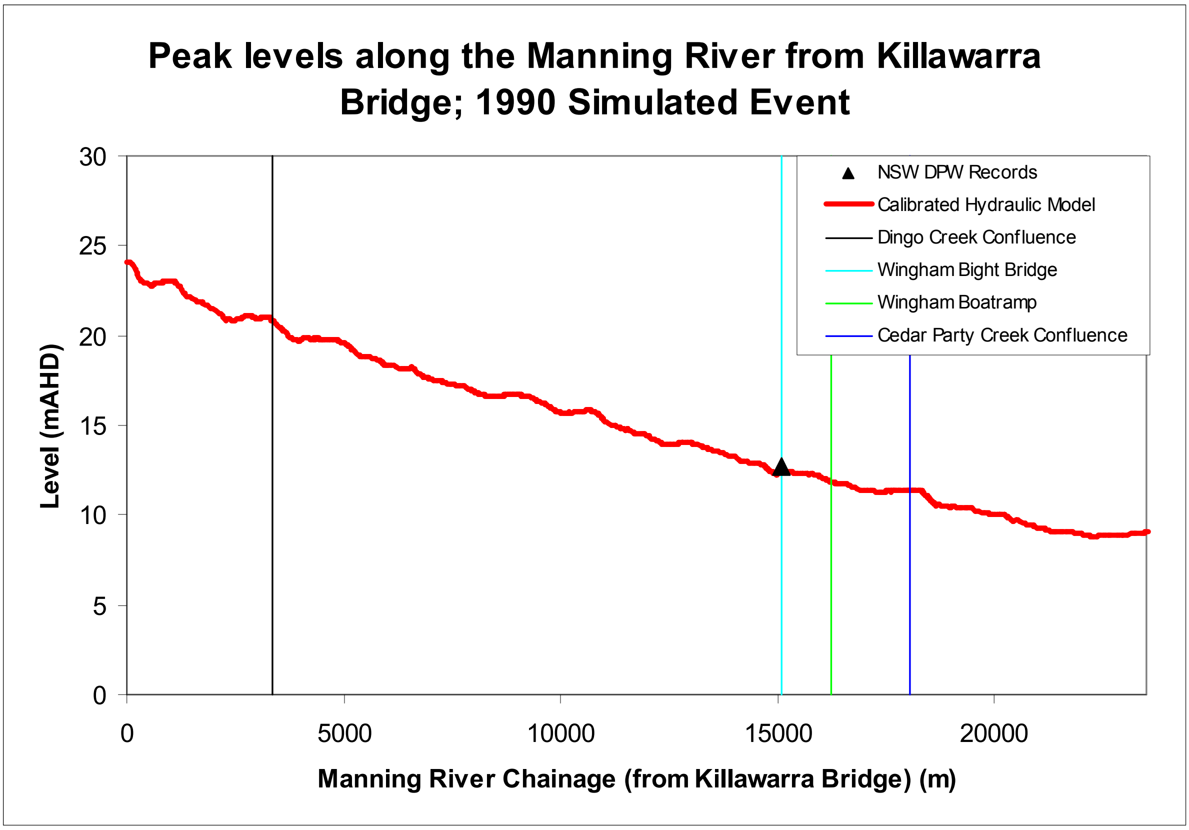


Figure 20: Calibrated Water Surface Profile along the Manning River for the 1990 flood event

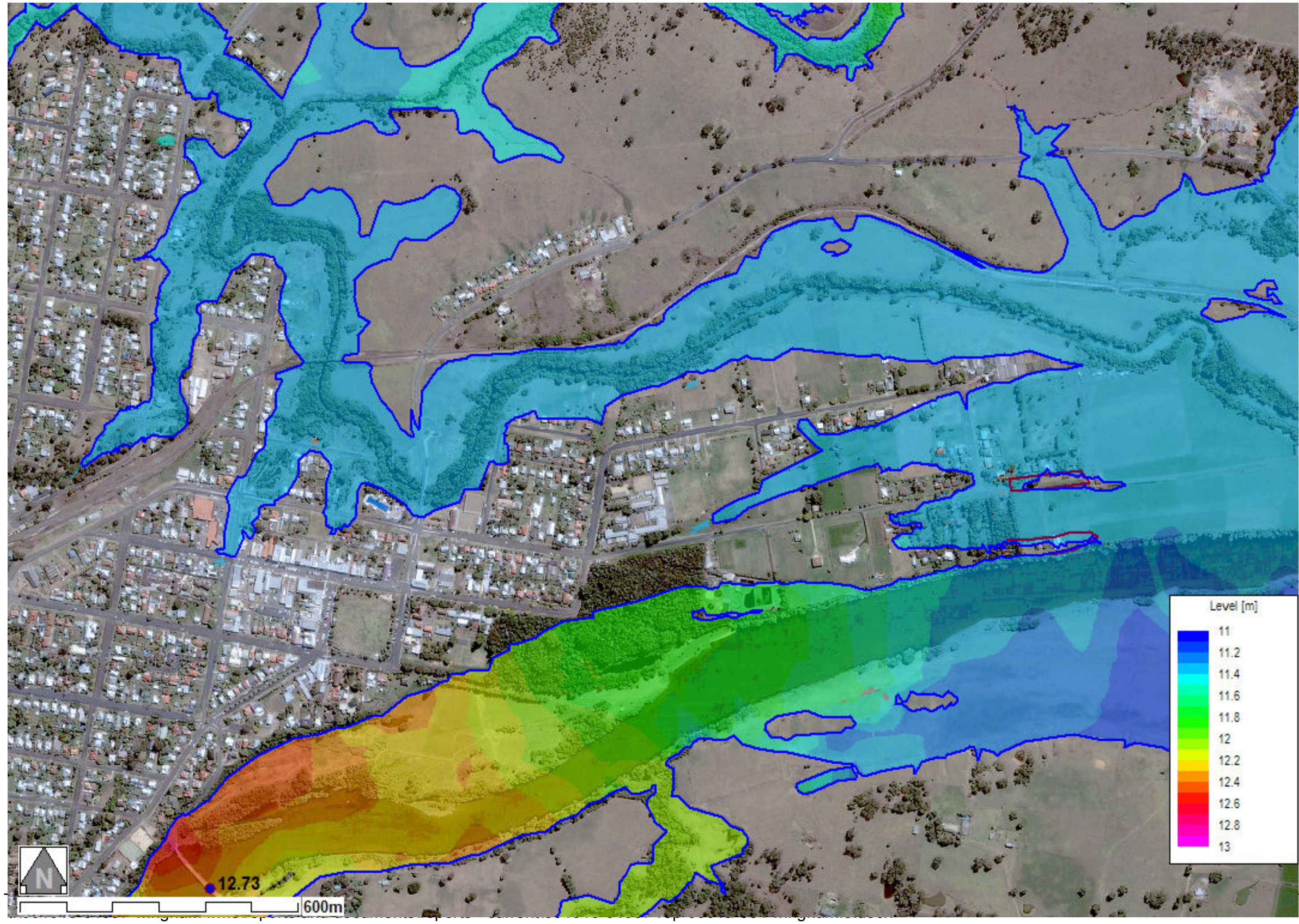


Figure 21: Peak water level plot for the calibrated 1990 flood event. The peak water level through the majority of Wingham is approximately 11.38 mAHD. This represents the best fit to all supplied data. The extents of the modelled 1990 flood have been highlighted in dark blue.

Shown in maroon is a portion of the approximate flood extents near East Combined Street provided by local residents which shows an excellent match to that modelled.

The location (and magnitude) of spot water level readings obtained from the NSWPWD are shown as circles with black writing.

Furthermore it must be remembered that this represents how the 1990 flood would affect the area with the *current* topography, landscape and riverbed features.

6.3.4 1995

The 1995 flood was categorised as "moderate" in Wingham according to the SES and BoM scale of flood events. Rainfall occurred throughout the Manning Catchment on the 4th and 5th of March, 1995 with levels peaking in the Manning at Wingham on the 5th. The peak level at Wingham Bridge was approximately 10.30 mAHD.

Tributary Hydrograph Inputs

Data for the 1995 event was not available at the Dingo Creek gauge.

- Hydrographs for Dingo, Cedar Party and Stony Creeks were estimated based on an assessment of the catchment rainfall that occurred in combination with other known events, such as the 1978 and 1976 events. Furthermore, the contribution of flows from these tributaries was unlikely to significantly influence flood levels due to similar reasons given in section 6.3.3 for the 1990 event.

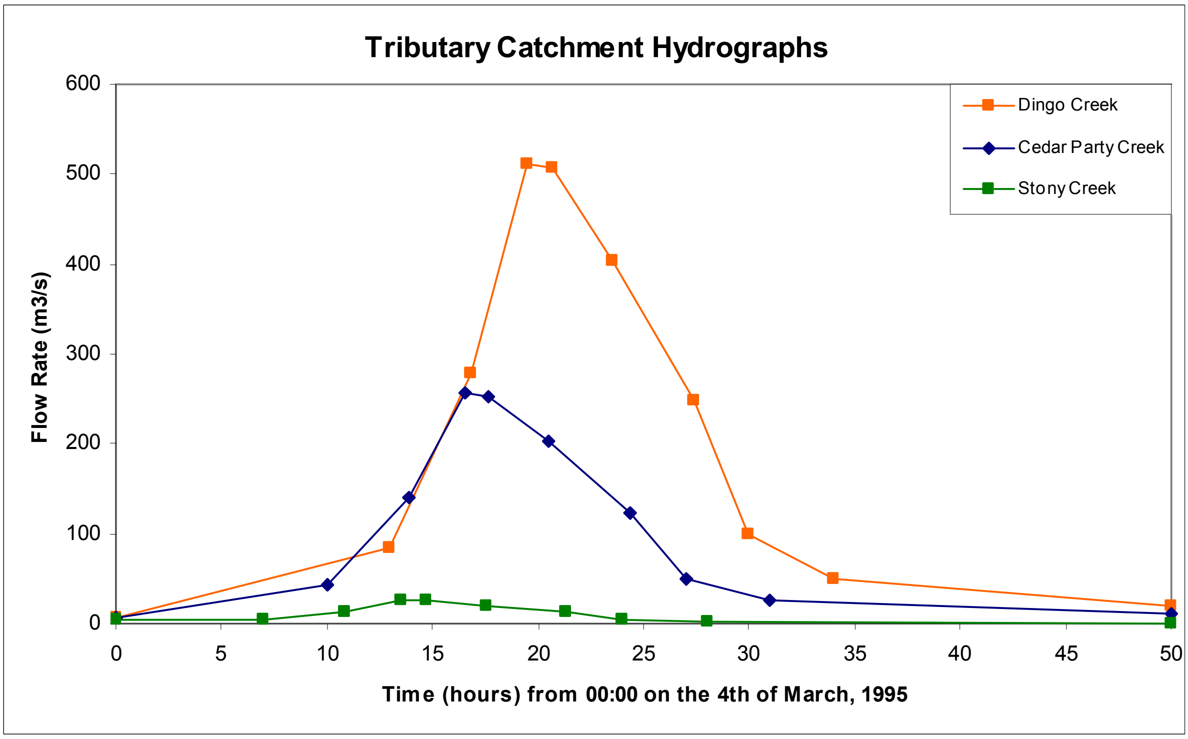


Figure 22: Estimated 1990 tributary inflow hydrographs used in the calibration analyses

Manning River Hydrograph Input

The complete Manning River hydrograph was available from the Killawarra gauging station 208400.

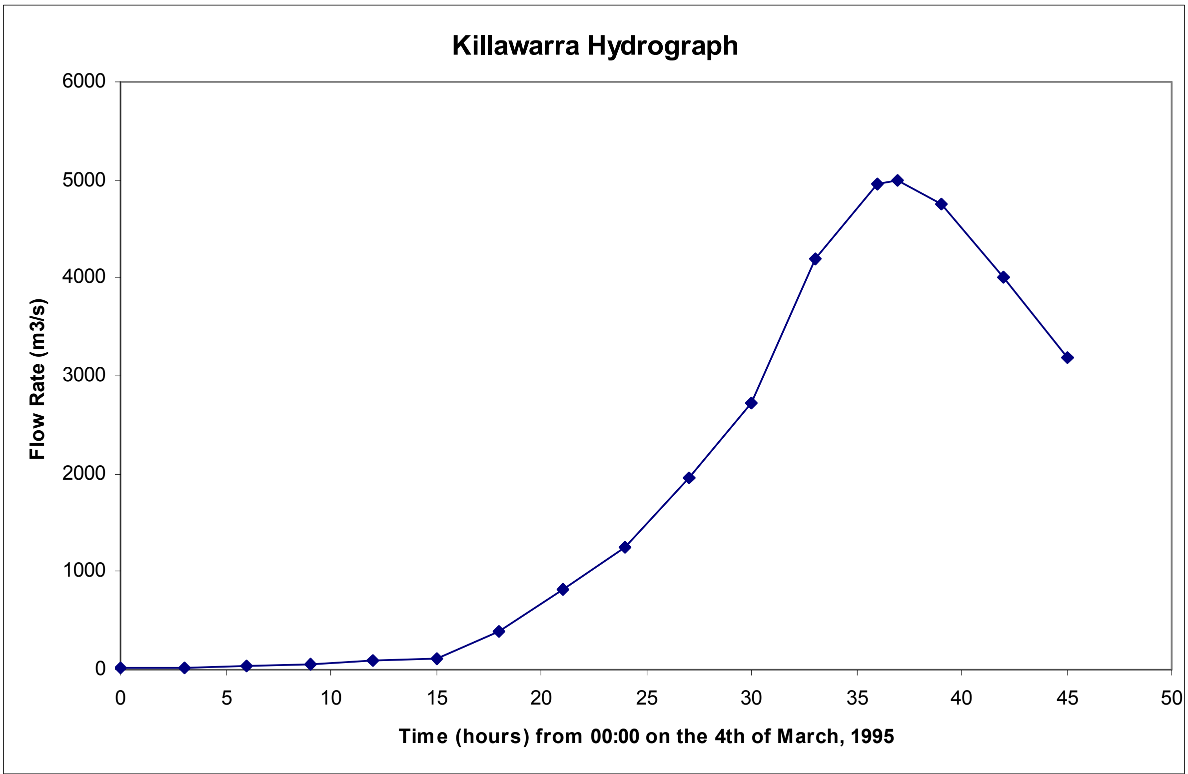


Figure 23: Recorded 1995 Killawarra inflow hydrographs used in the calibration analyses

Calibration Objectives and Data

The objective of the 1995 historic simulation was to verify the roughness coefficients and tailwater stage-discharge relationship that were calibrated using the 1978 (and 1990) historic flood events. The 1995 event represented a lower flow and subsequently lower peak levels than both the 1978 and 1990 events and therefore provided a good opportunity to verify the in-bank channel roughness almost exclusively.

For the 1995 event, the only available calibration data consisted of a single peak water level of 10.3 mAHD recorded at the Wingham Bridge gauge by the NSW DPW. The location of this reading was also assumed to be at the location of the current gauge.

Calibration Process

The base river channel roughness was set to 0.028 and the bank roughness to 0.033 as calibrated using the 1978 and 1990 events. All other roughness coefficients were unchanged. With the calibrated tailwater relationship, these parameters produced a peak water level of approximately 10.28 mAHD at the Bight Bridge, which is within 1% of the recorded peak of 10.30 mAHD and well within the limits of accuracy expected from both the hydraulic model and the recorded data. This was deemed to satisfactorily verify that the hydraulic model was fully calibrated.

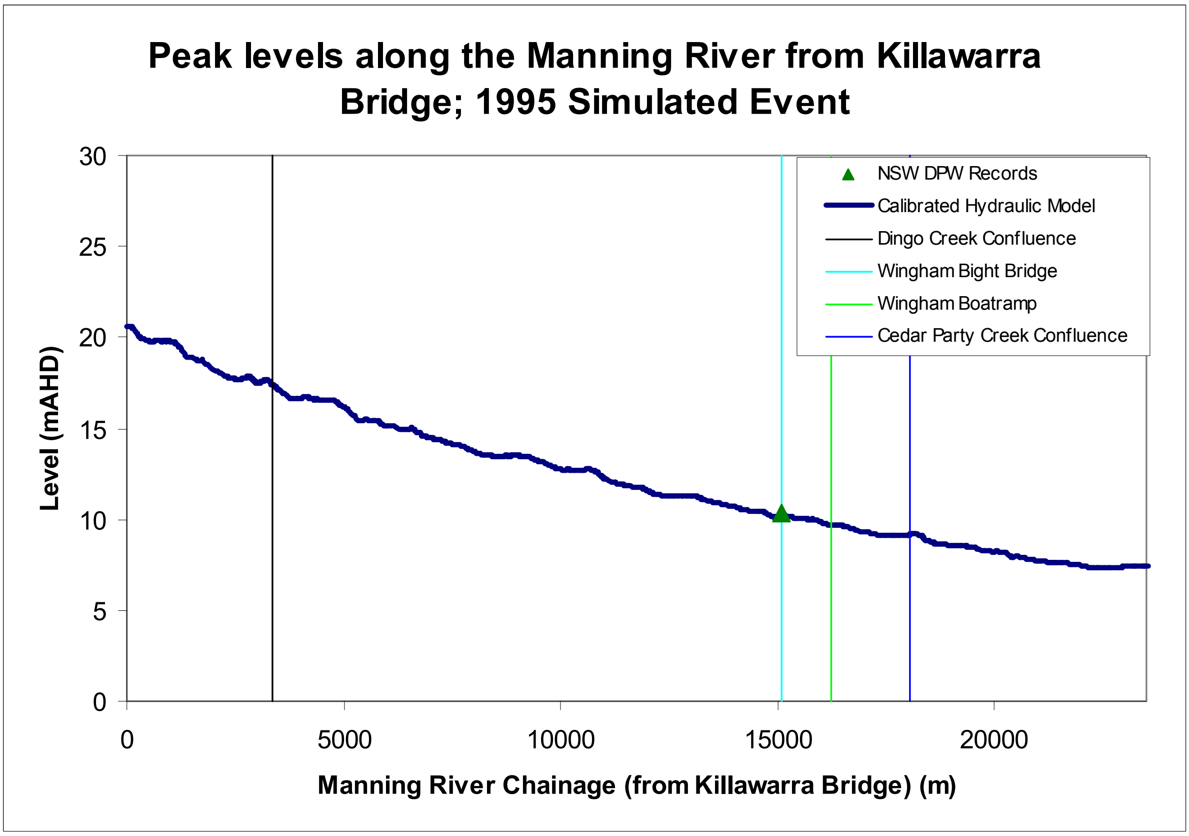


Figure 24: Calibrated Water Surface Profile along the Manning River for the 1995 flood event

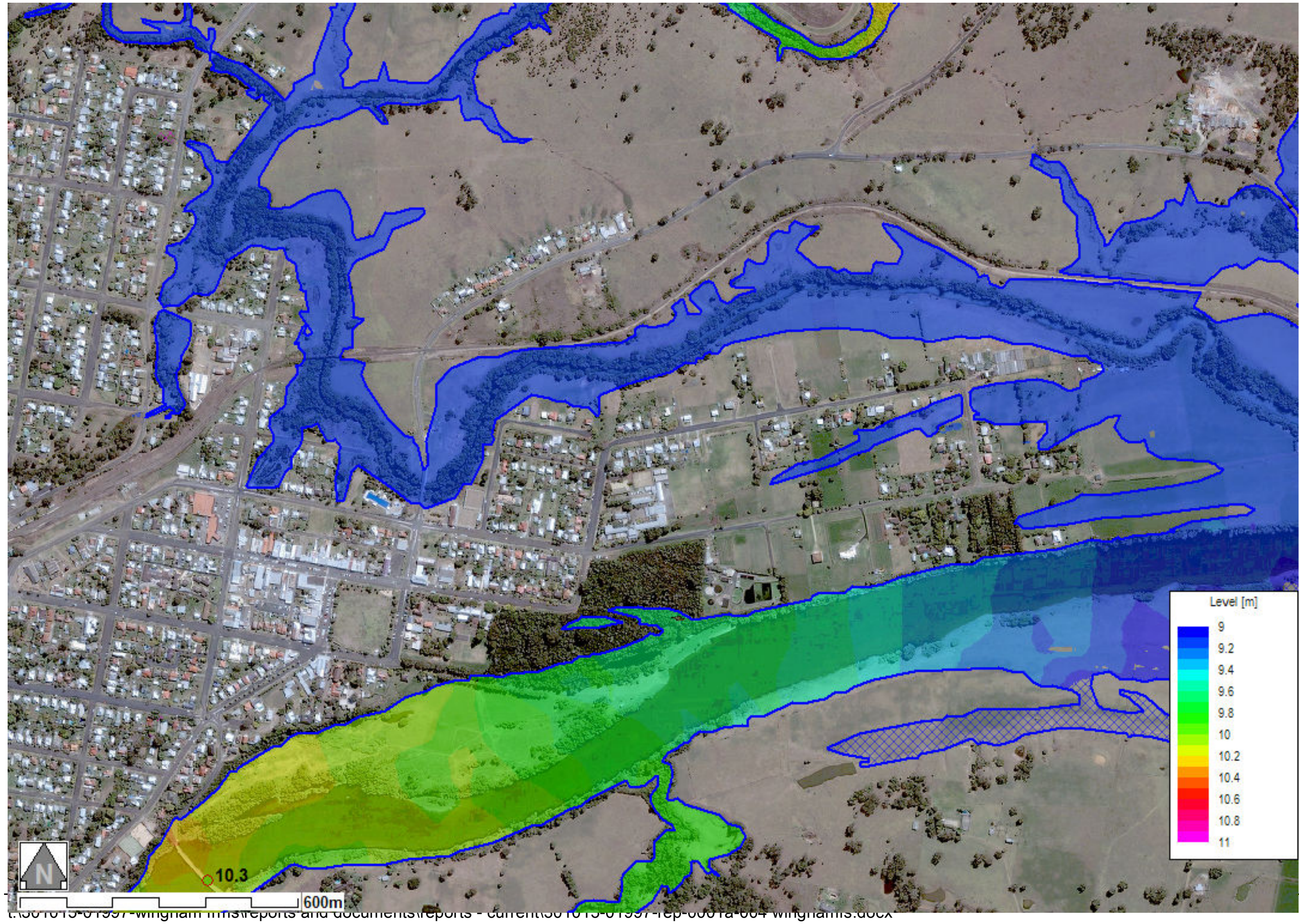


Figure 25: Peak water level plot for the calibrated 1995 flood event. The peak water level through the majority of Wingham is approximately 9.17 mAHD. This represents the best fit to all supplied data. The extents of the modelled 1995 flood have been highlighted in dark blue.

The location (and magnitude) of spot water level readings obtained from the NSWPWD are shown as circles with black writing.

Furthermore it must be remembered that this represents how the 1995 flood would affect the area with the *current* topography, landscape and riverbed features.

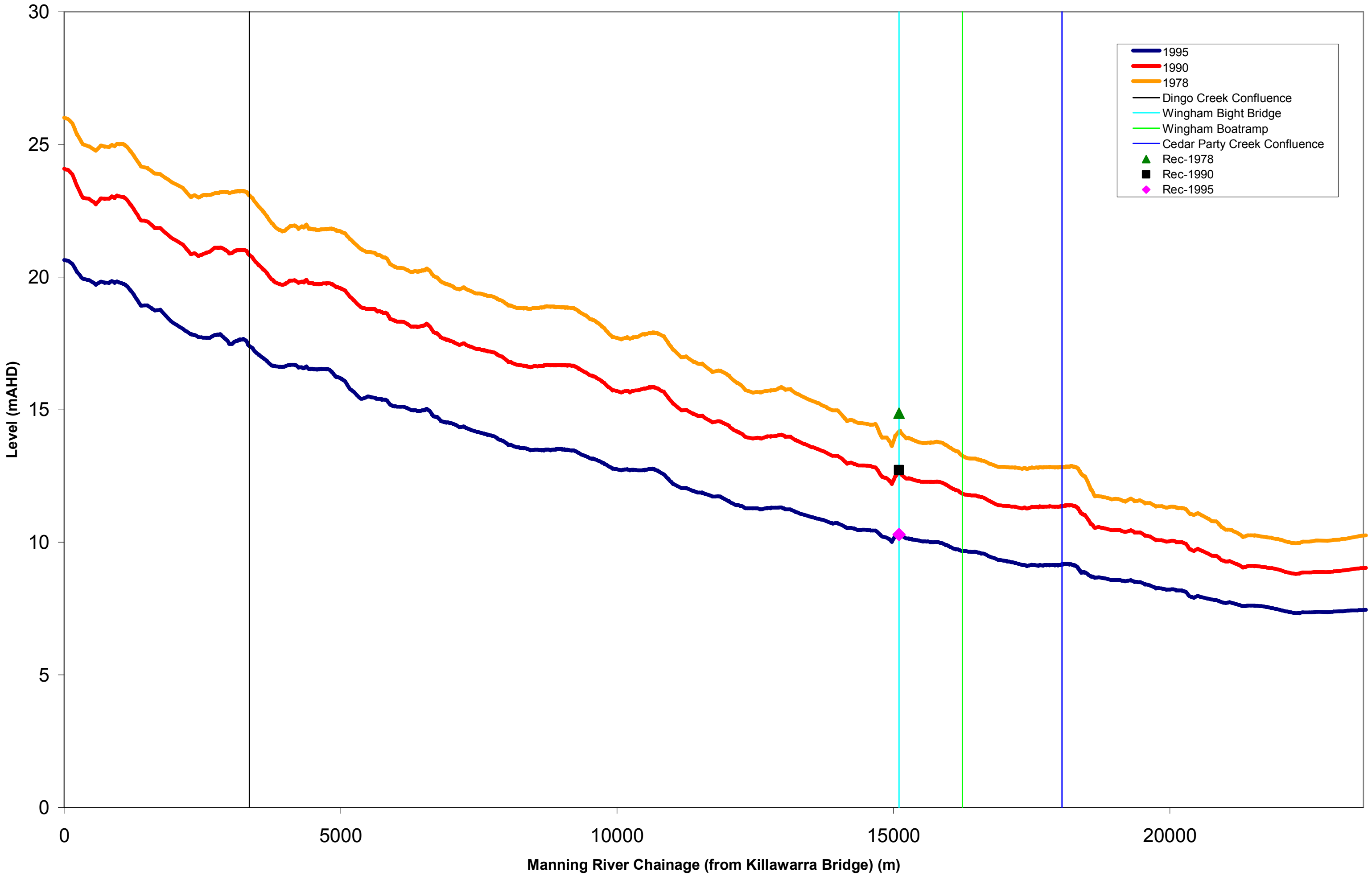


Figure 26: Water profile plot along the Manning River comparing the calibrated model for all three historic events

6.3.5 Summary of Verified Calibration Parameters

The calibrated and verified hydraulic model had a roughness coefficient that varied between 0.028 and 0.080. The distribution of model roughness is shown in the following figure. Azure blue represents the lower bound of roughness (0.028) and light green represents the next most common roughness of 0.033, used primarily for the smooth river bank and grassland areas of the floodplain (the boundary conditions are shown as blue “ramps”, which aids in the modelling purposes).

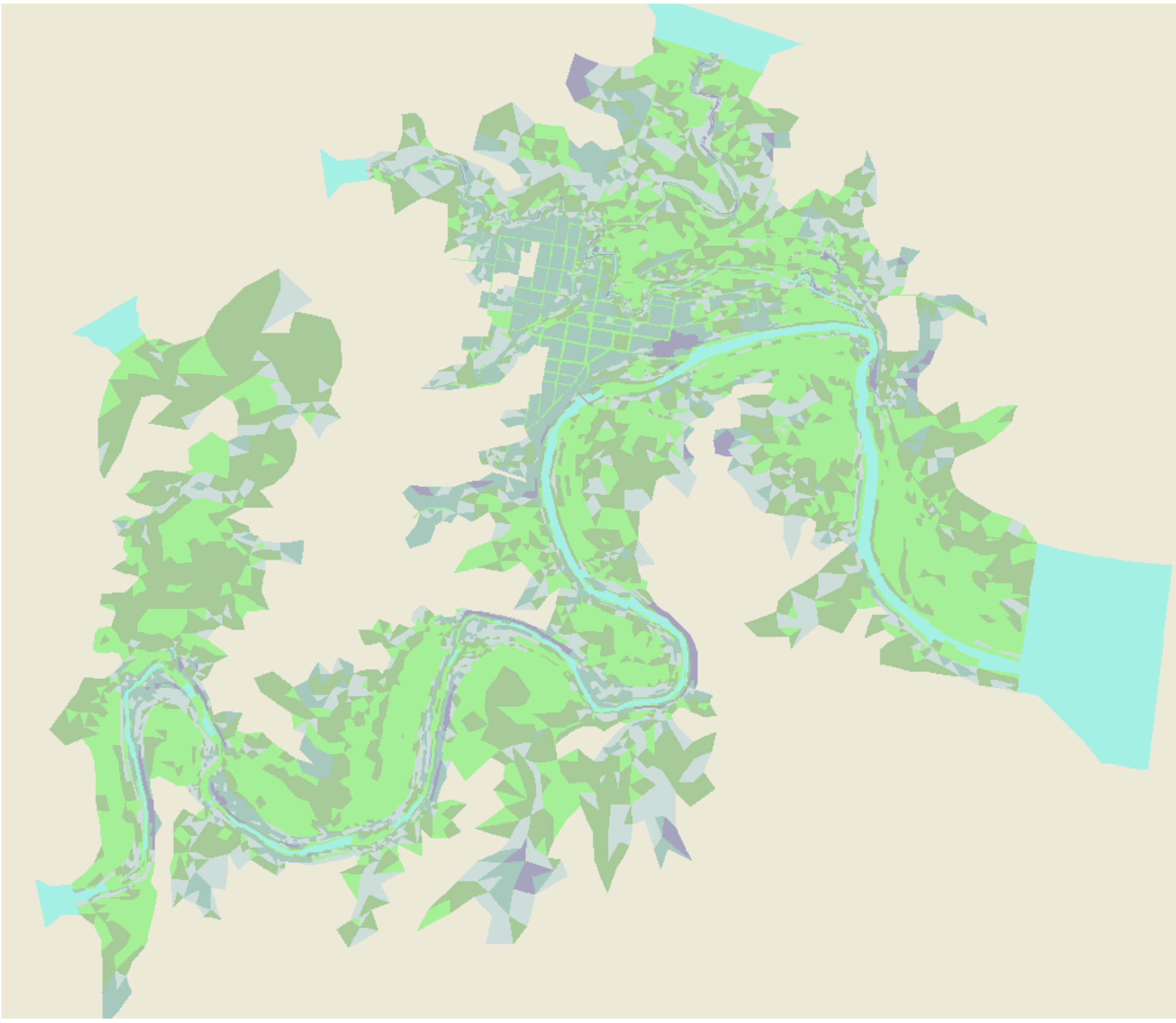


Figure 27: Overview of the hydraulic model and its calibrated roughness, where different coloured elements represent areas that have different manning roughness coefficients.

The following plot shows the tailwater stage-discharge relationship at the outflow boundary of the hydraulic model (geographically near Mondrook). Overlaid on this figure are the ESTRY design flood results at this location showing that the relationship is within the limits of the ESTRY model.

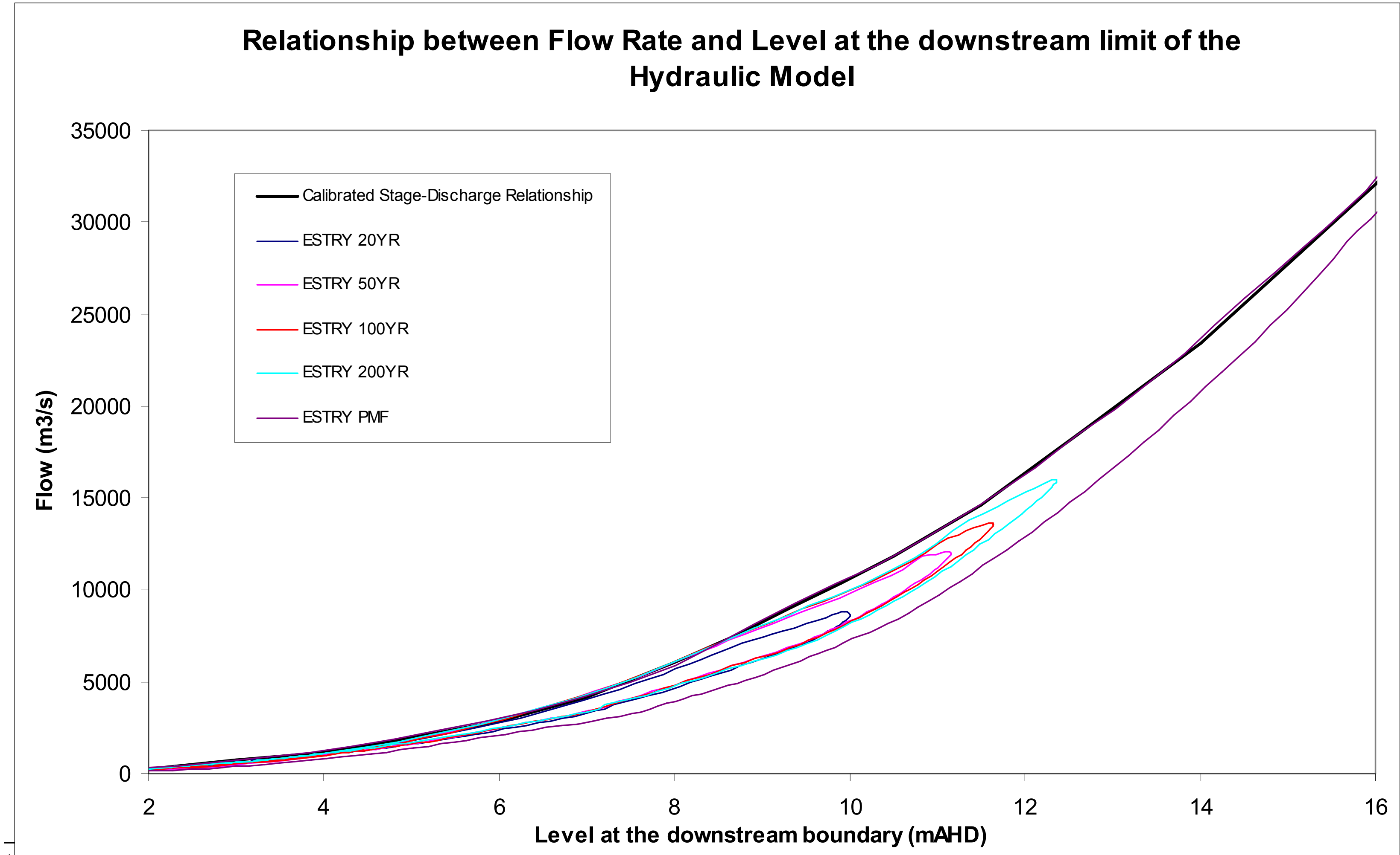


Figure 28: Calibrated downstream stage-discharge relationship which governs the outflow in the hydraulic model. This boundary condition is derived primarily from the ESTRY model design flood results of the previous flood study and was calibrated for use in the current flood study with the use of historic flood data.

6.4 Design Flood Simulations

The Manning Catchment Hydraulic Model design runs consisted of input hydrographs for the Manning River, Dingo, Cedar Party and Stony Creeks for the 5%, 2%, 1% and 0.5% AEP floods as well as the PMF. The Manning River and Dingo Creek design hydrographs were produced as part of the previous flood study whilst those for Stony and Cedar Party Creeks were determined as part of the hydrological model in this flood study (Section 5). The calibrated downstream-stage discharge relationship was used at the outflow boundary condition of the model. This model analysed the overall effects of flooding in the study area, resulting primarily from the Manning River.

The Cedar Party Catchment Hydraulic Model design runs consisted of input hydrographs for Cedar Party and Stony Creeks for the 5%, 2%, 1% and 0.5% AEP floods as well as the PMF. The constant elevation boundary condition, developed with the results from the Manning Catchment Hydraulic Model design simulation, was used at the outflow of the model. This model analysed the localised effects of flooding in Wingham, resulting from flows in the Cedar Party catchment “backing up” from elevated levels in the Manning River.

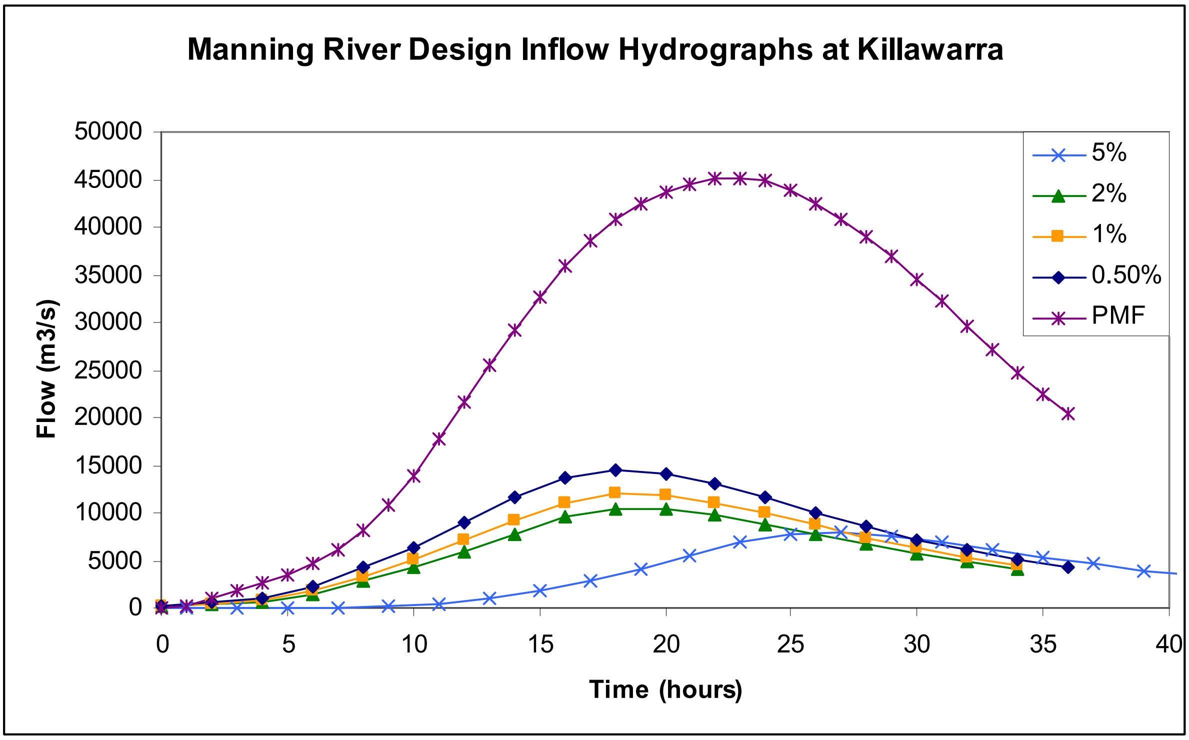


Figure 29: Design flood hydrographs for the Manning River at Killawarra Bridge.

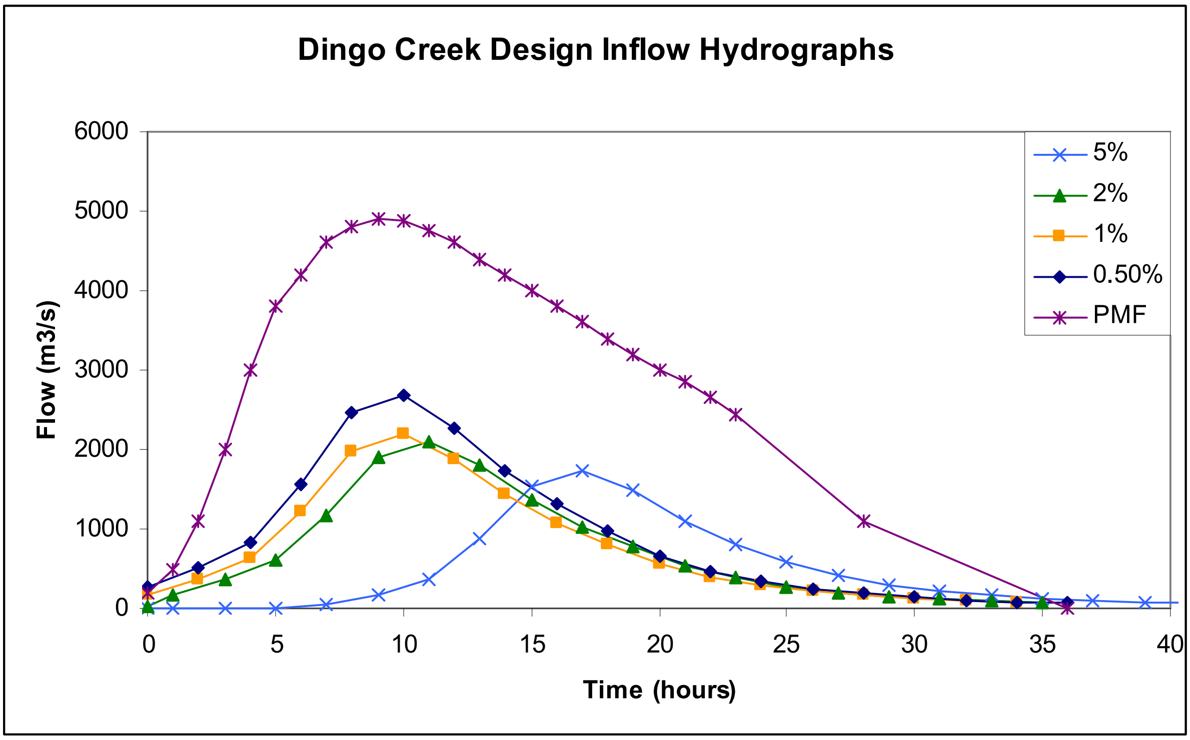


Figure 30: Design flood hydrographs for Dingo Creek

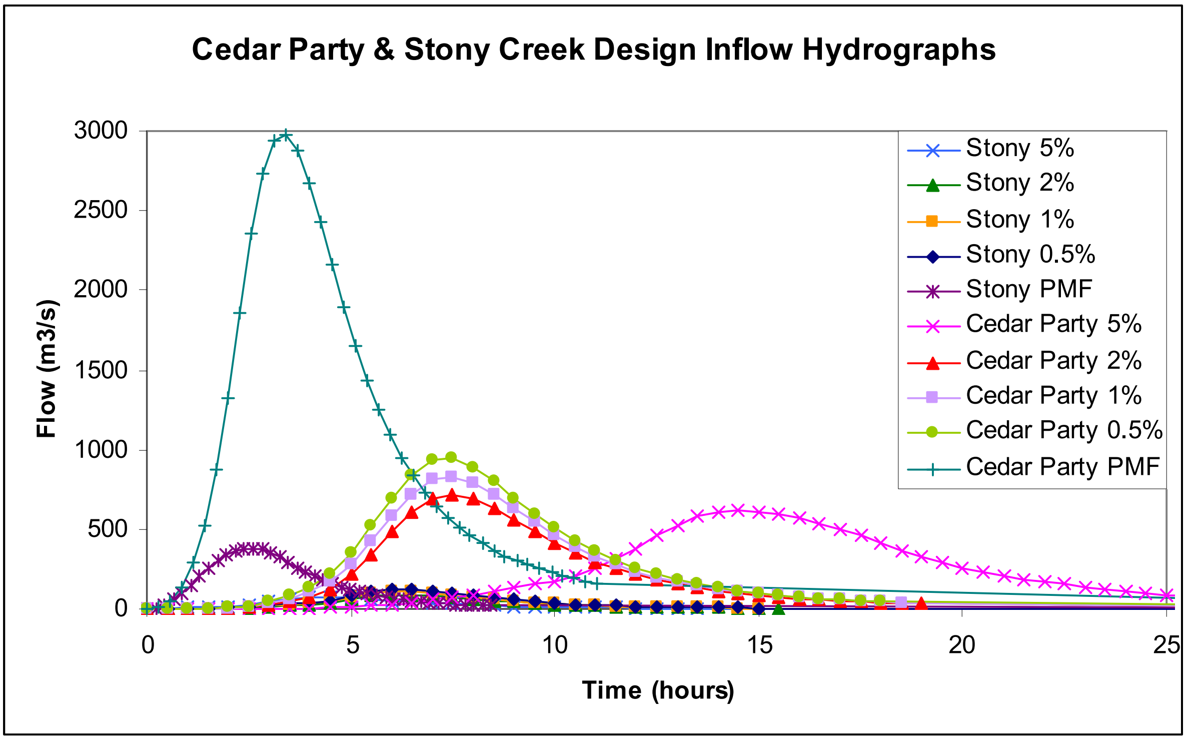


Figure 31: Design flood hydrographs for Cedar Party and Stony Creeks

6.5 Results

6.5.1 Manning Catchment Hydraulic Model

Figure 32 shows the peak water surface profile along the Manning River for the Manning Catchment Hydraulic Model simulations.

Figure 33 to Figure 46 show the results from the Manning Catchment Hydraulic Model centred on Wingham, showing the peak:

- Depth with velocity vectors (for all design flood simulations)
- Velocity times depth (for all design flood simulations)
- Water level (only for the 1% AEP and PMF design flood simulations); peak levels are essentially constant in Wingham for each design flood
- Hydraulic Hazards (only for the 1% AEP and PMF design flood simulations)

Further figures, showing the peak water level with contours for all design flood simulations for the whole model network, are provided in Appendix B.

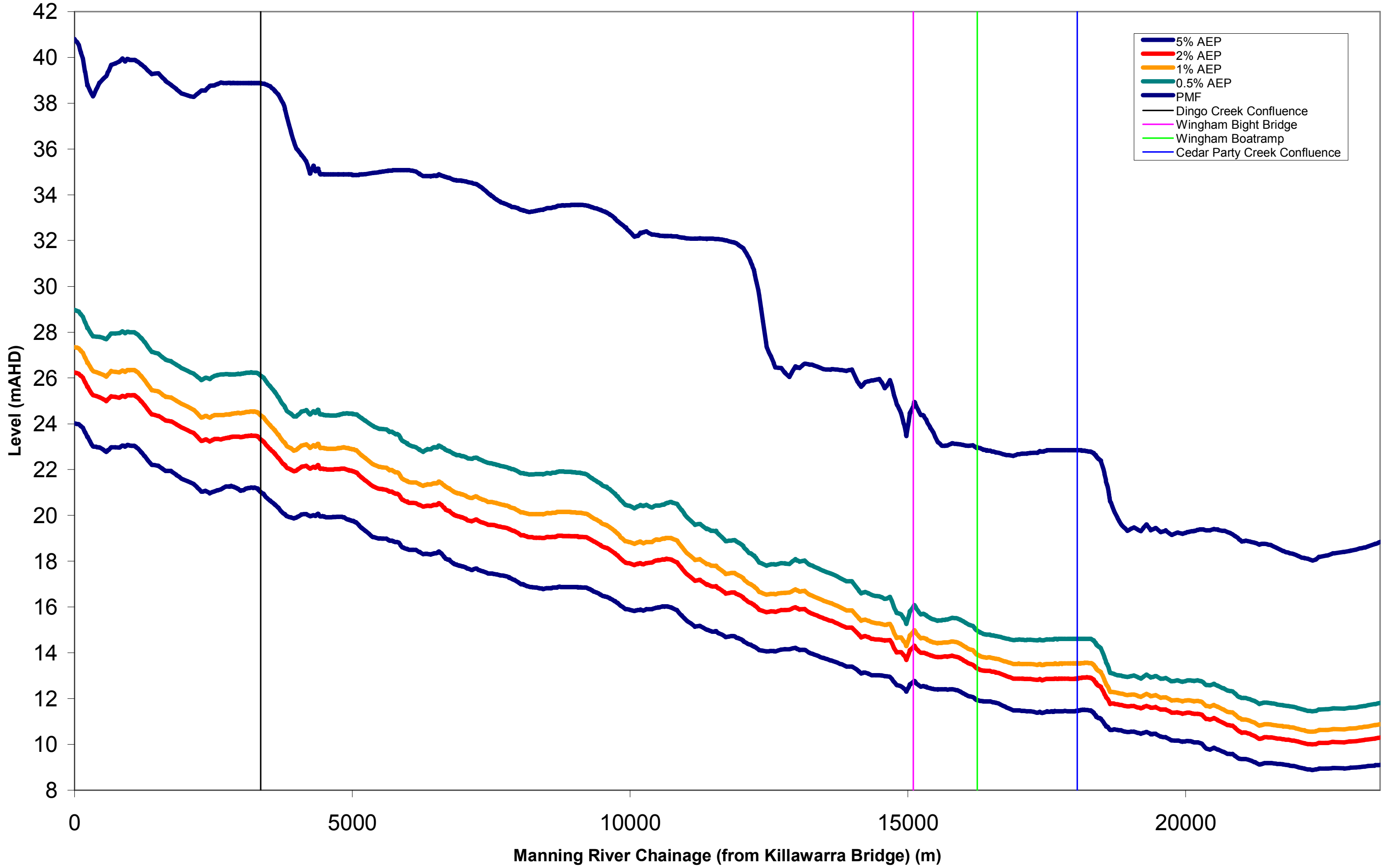


Figure 32: Manning River Peak Water Surface Profile (5%, 2%, 1%, 0.5% and PMF)

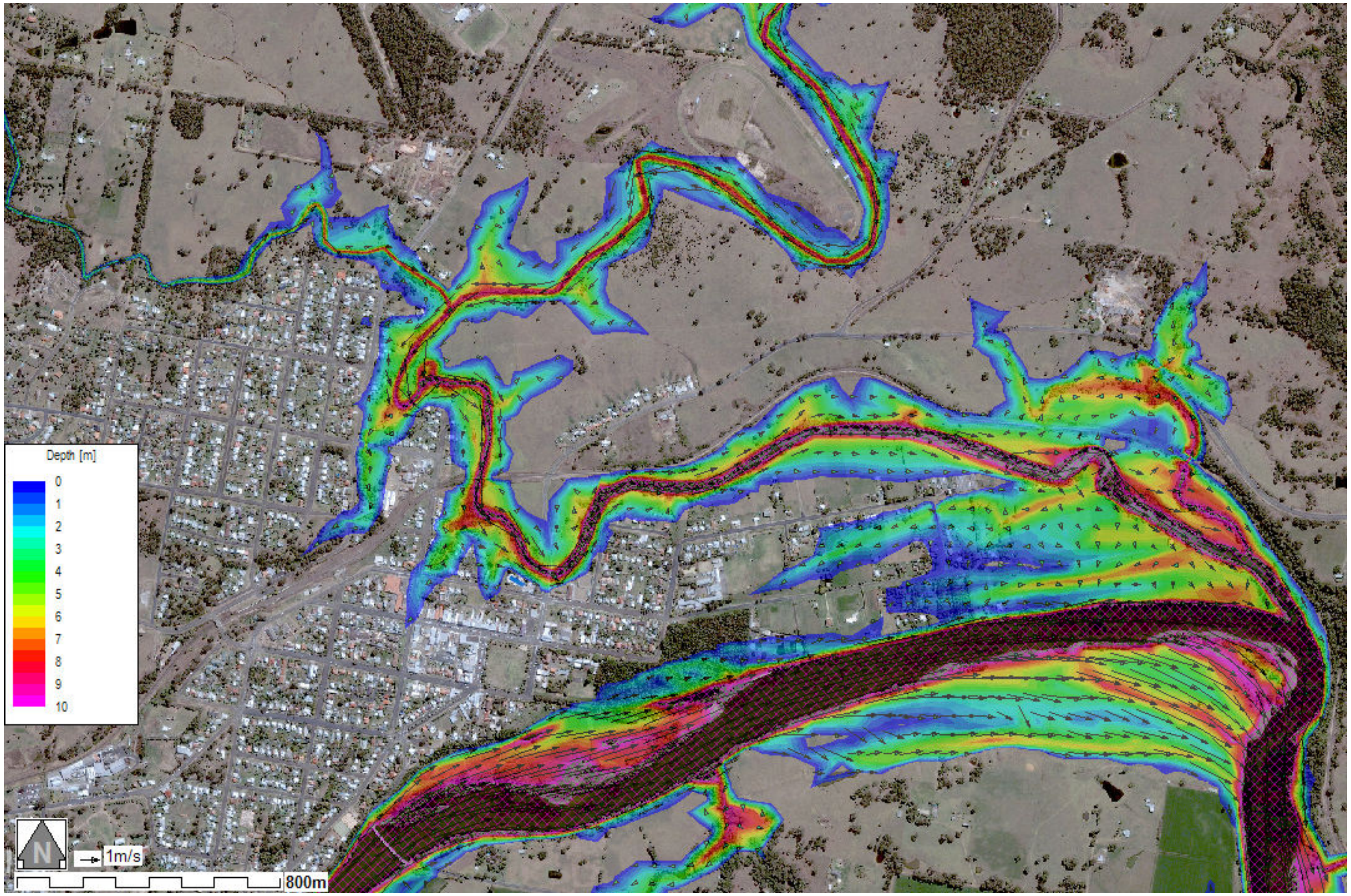


Figure 33: 5% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Manning Catchment Model)

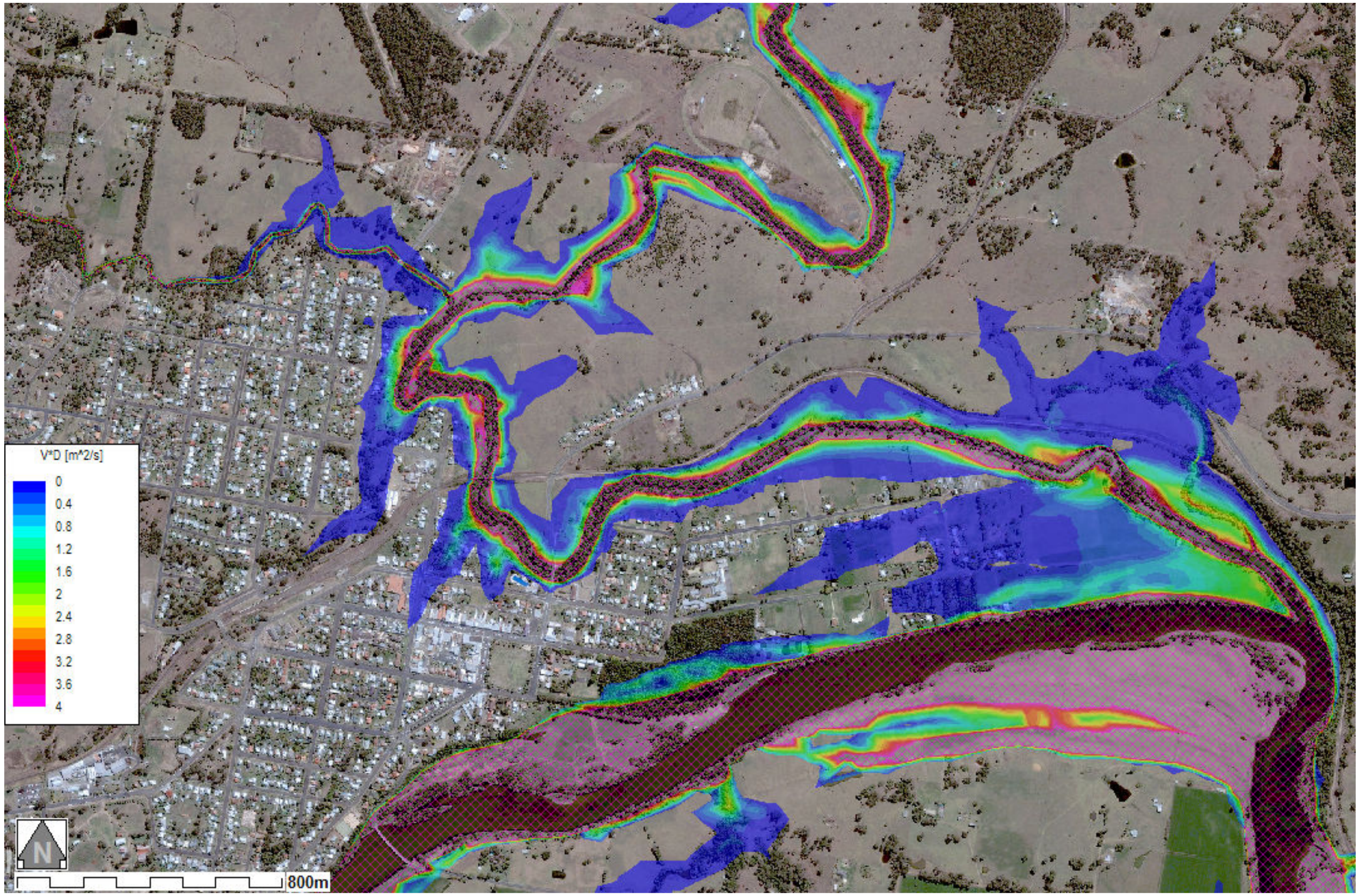


Figure 34: 5% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Manning Catchment Model)

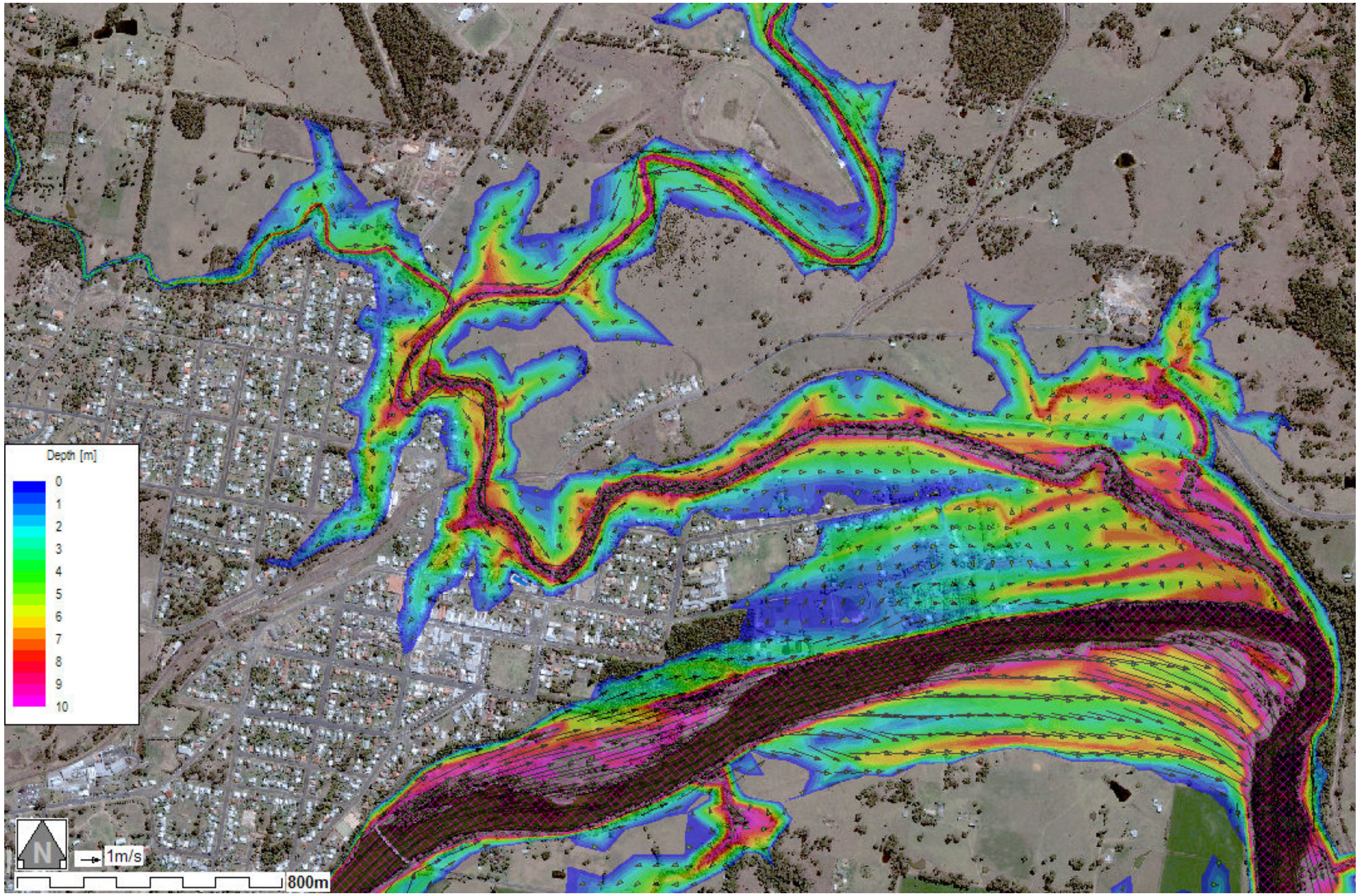


Figure 35: 2% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Manning Catchment Model)

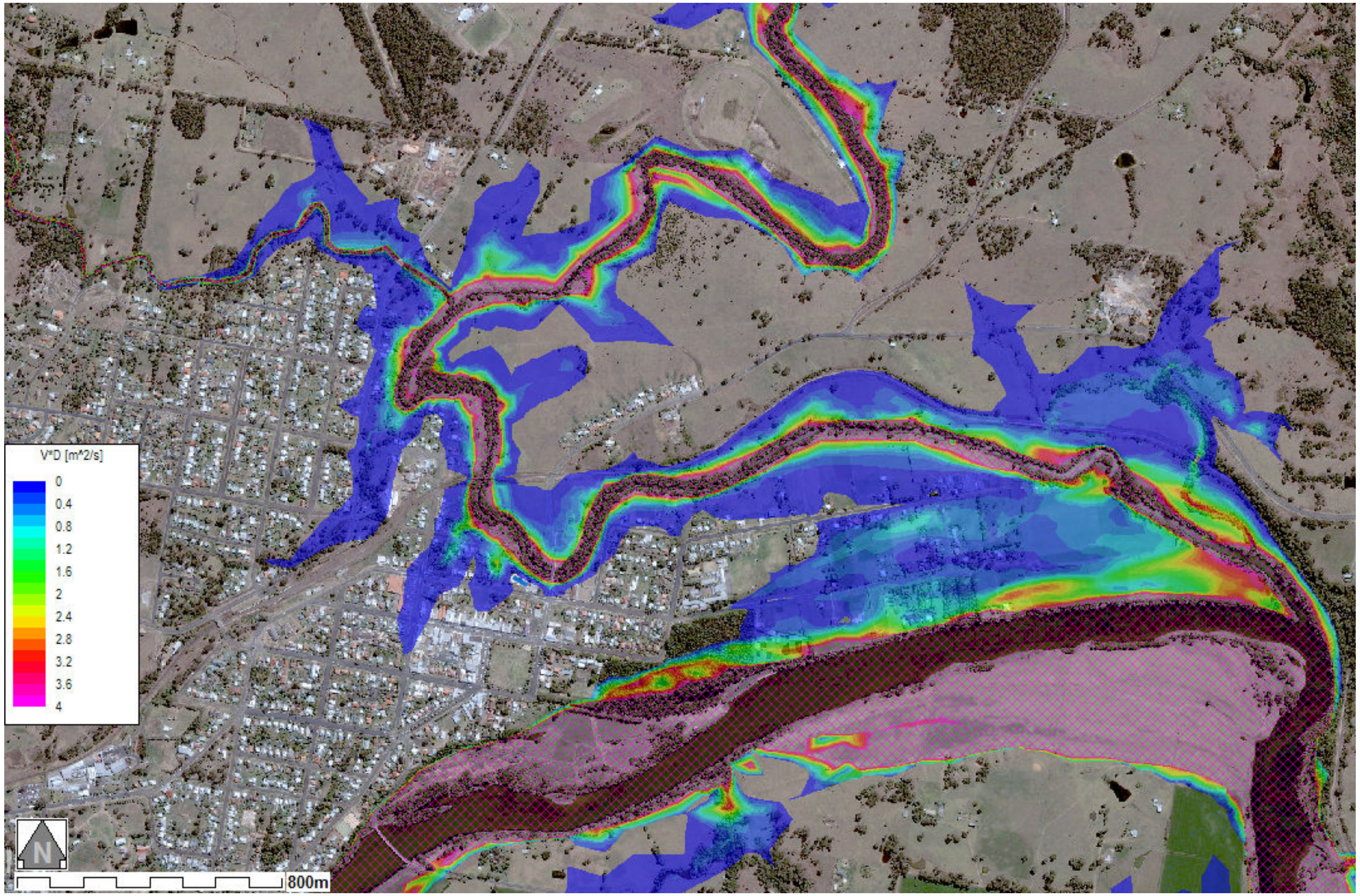


Figure 36: 2% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Manning Catchment Model)

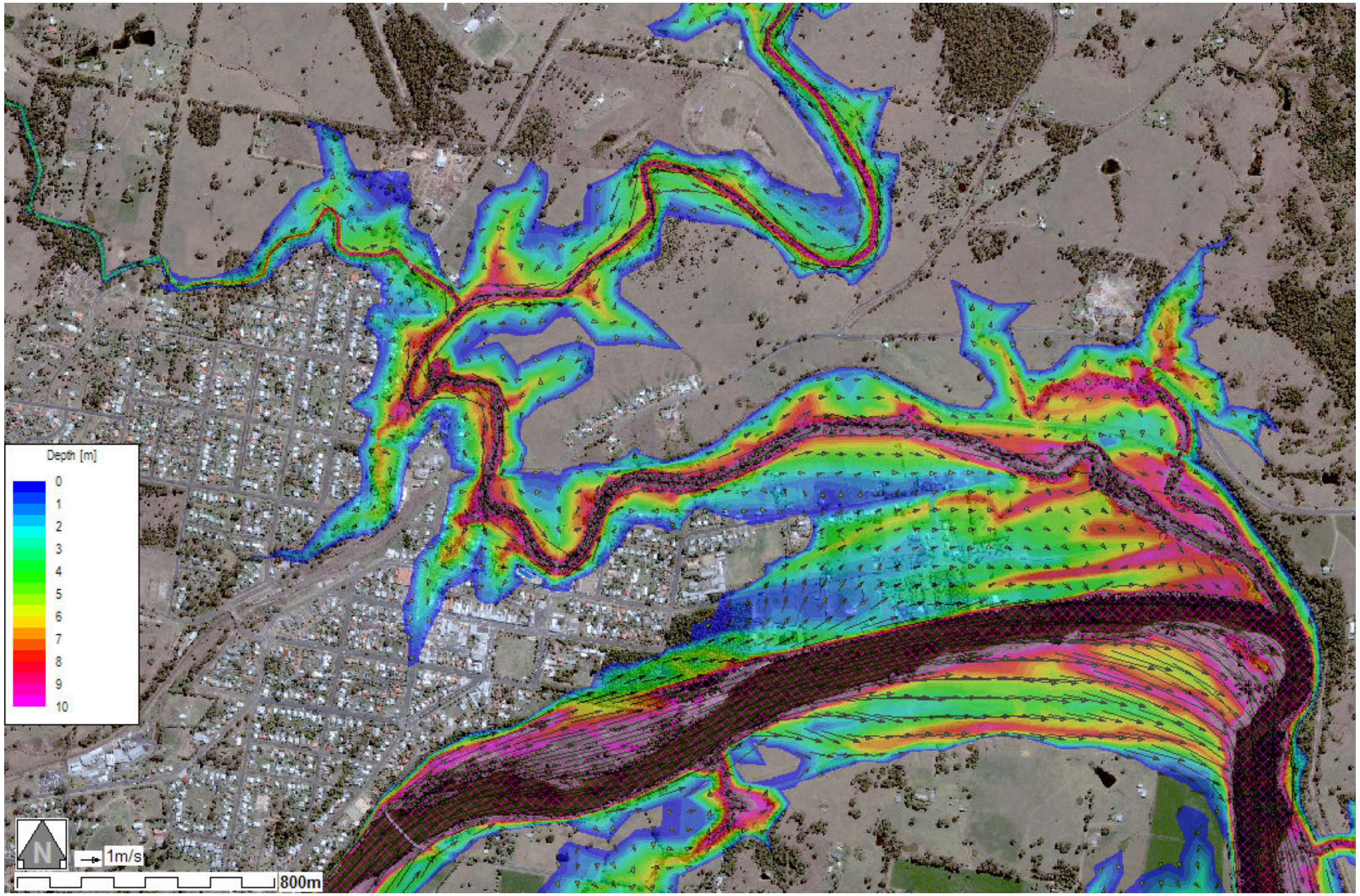


Figure 37: 1% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Manning Catchment Model)

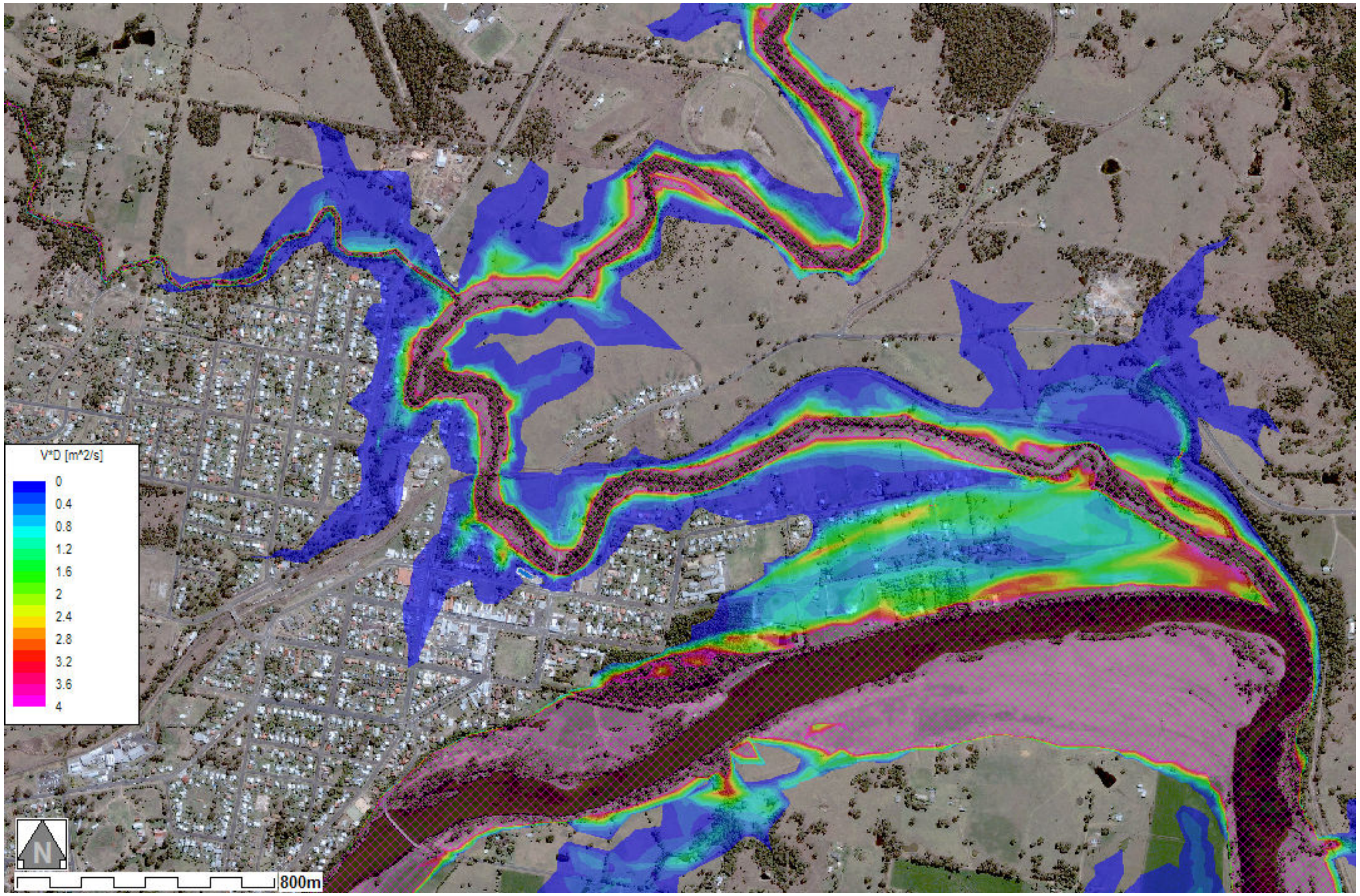


Figure 38: 1% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Manning Catchment Model)

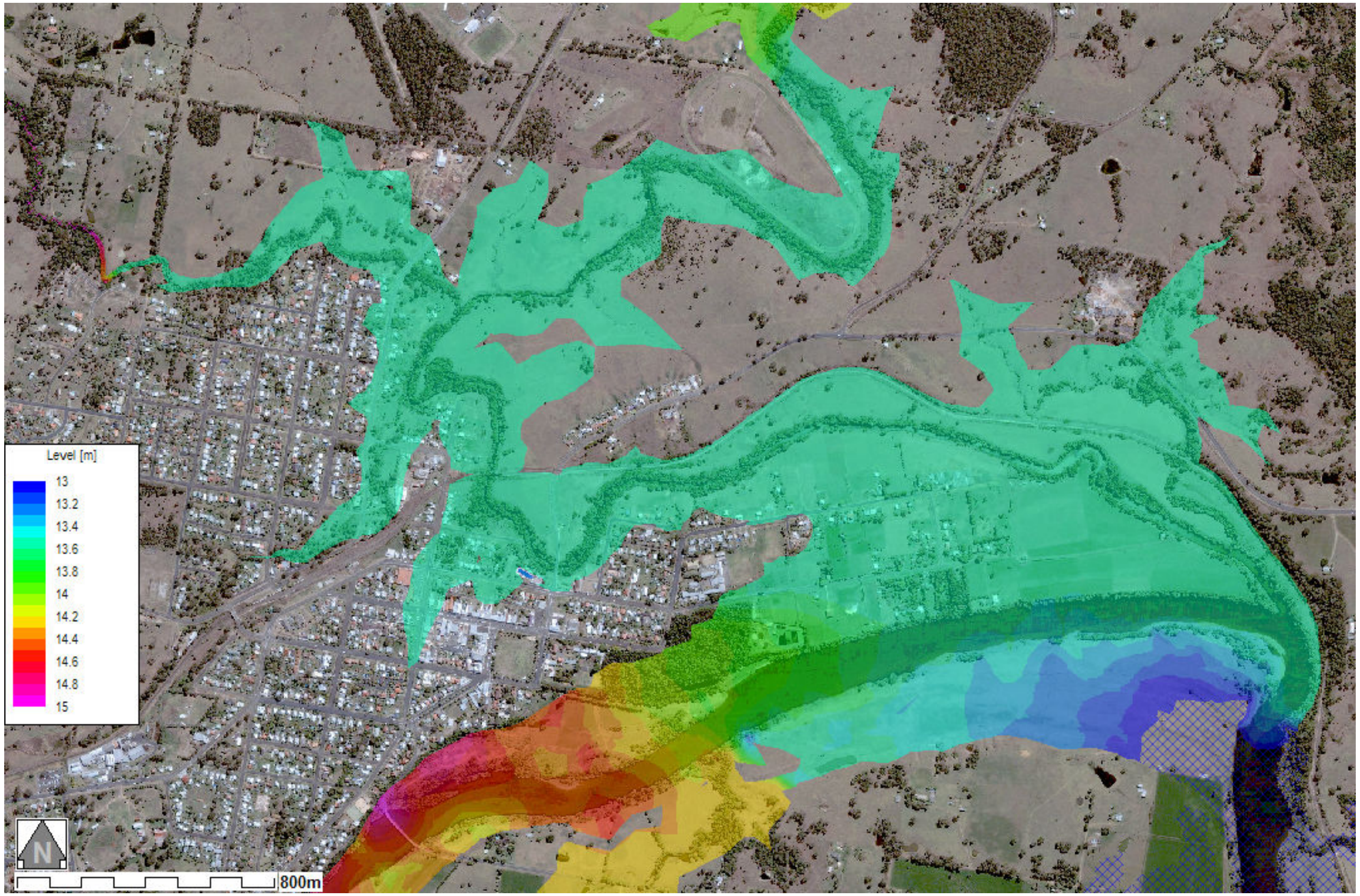


Figure 39: 1% AEP Design Flood; Wingham Overview – Water Level Coloured (Manning Catchment Model)

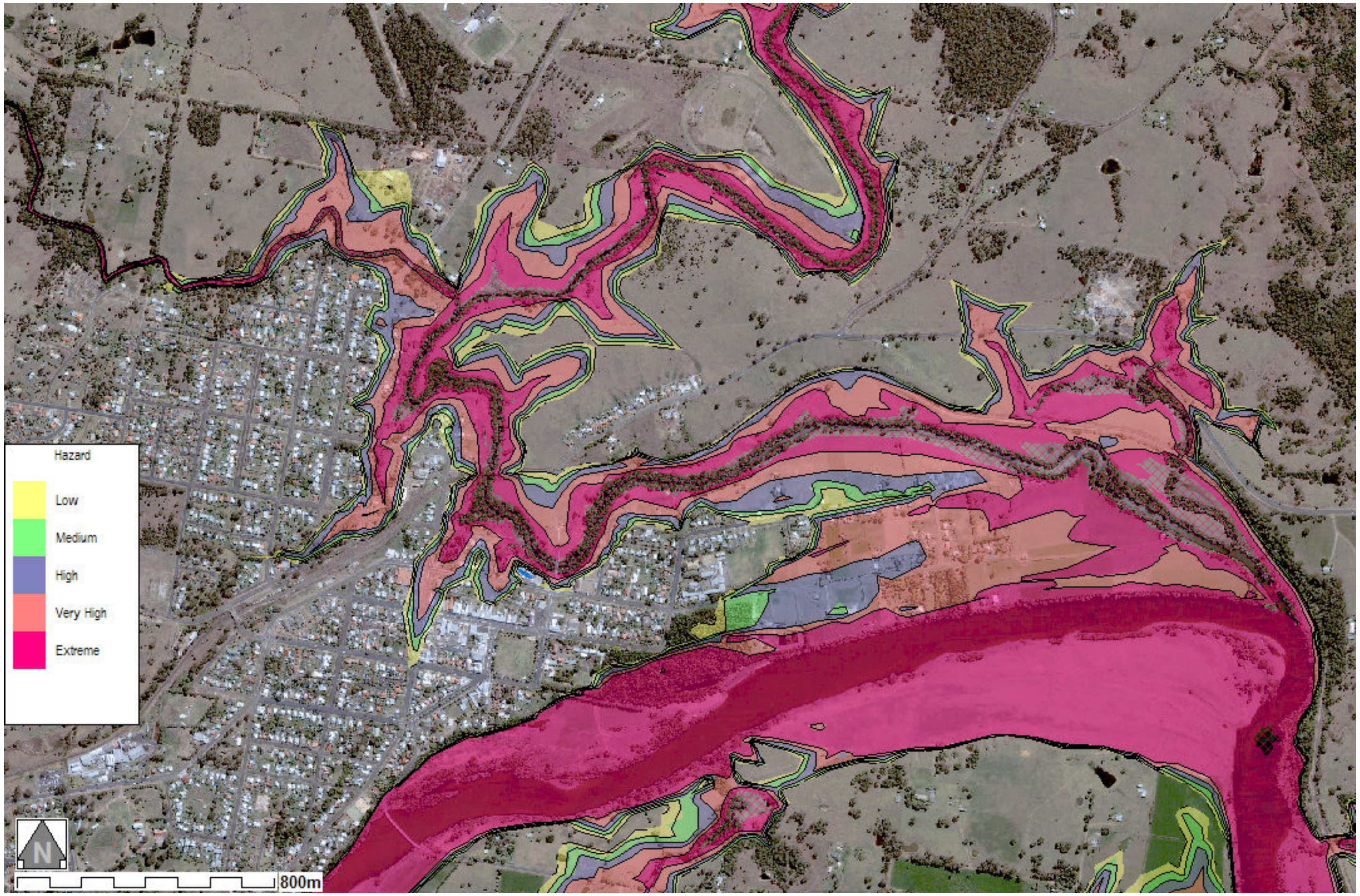


Figure 40: 1% AEP Design Flood; Wingham Overview – Hydraulic Hazard Coloured (Manning Catchment Model)

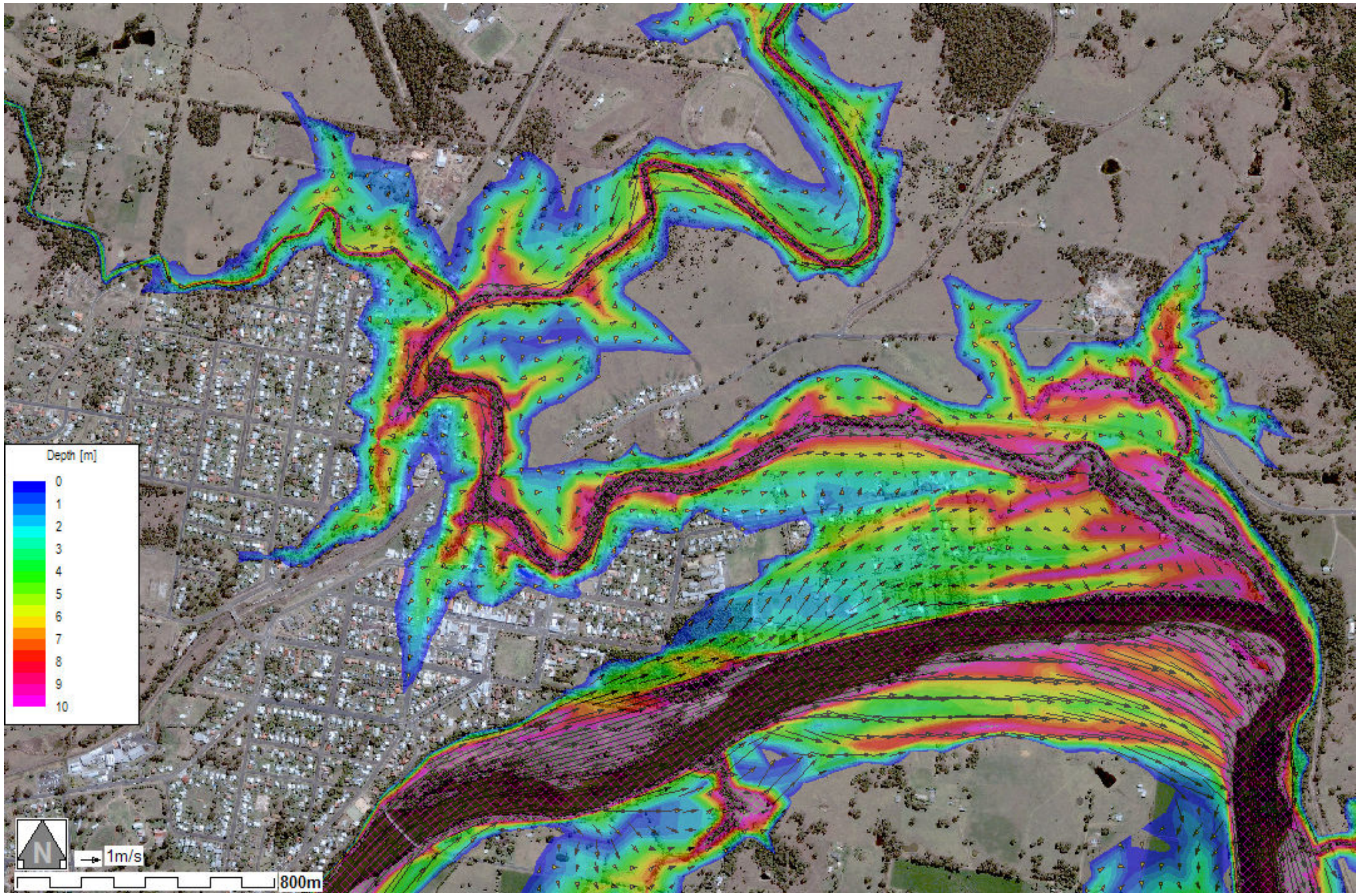


Figure 41: 0.5% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Manning Catchment Model)

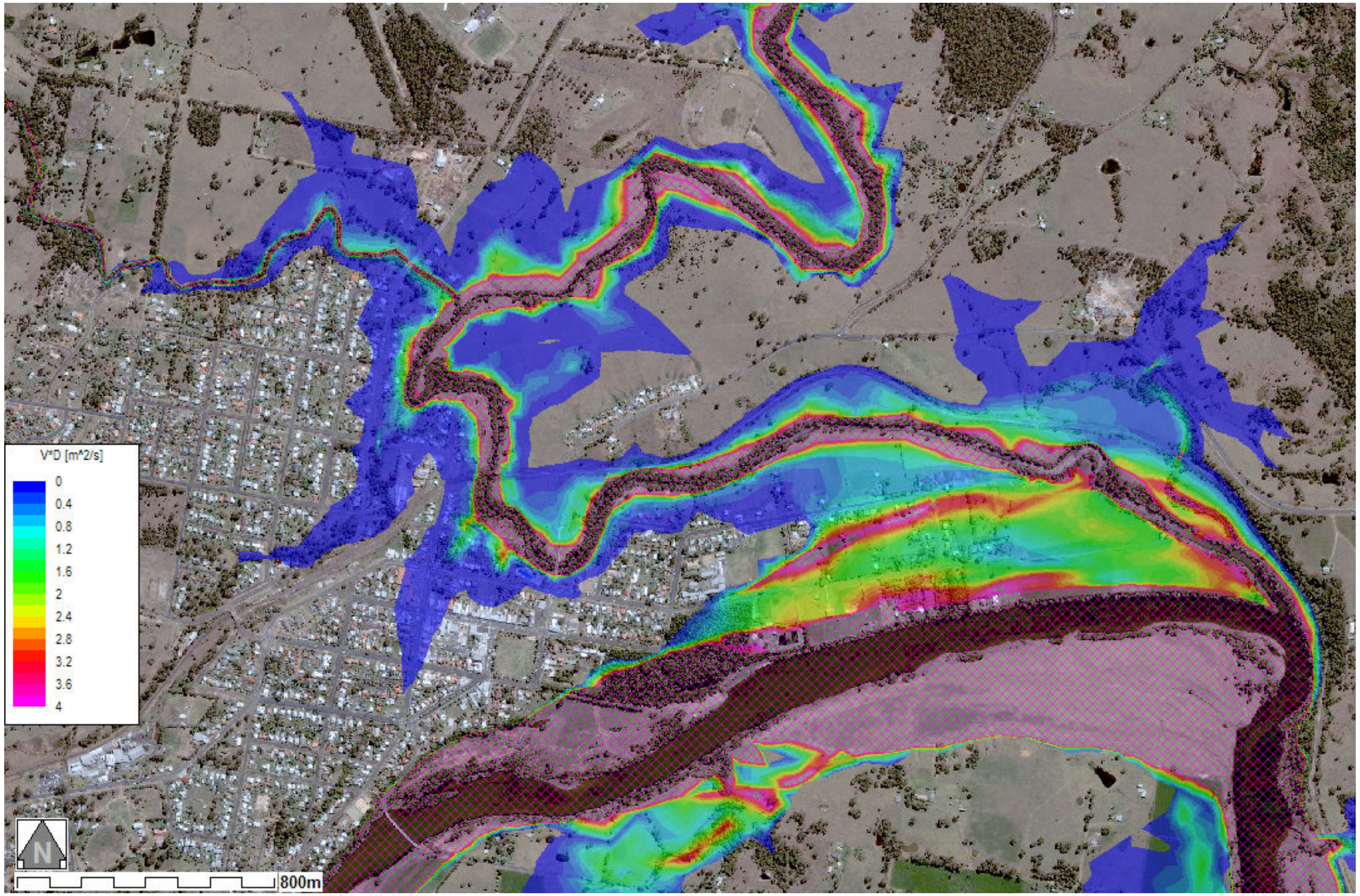


Figure 42: 0.5% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Manning Catchment Model)

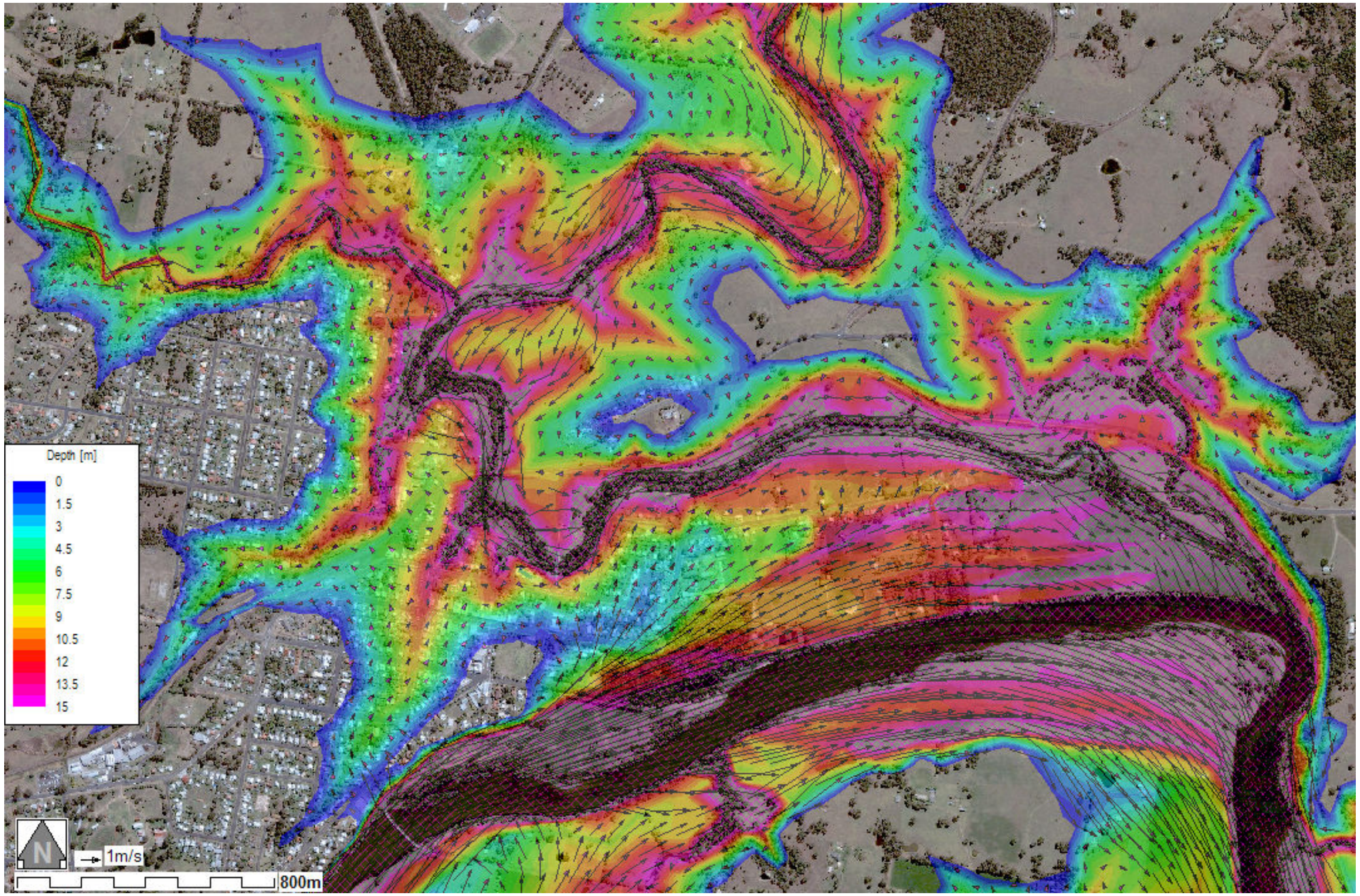


Figure 43: PMF Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Manning Catchment Model)

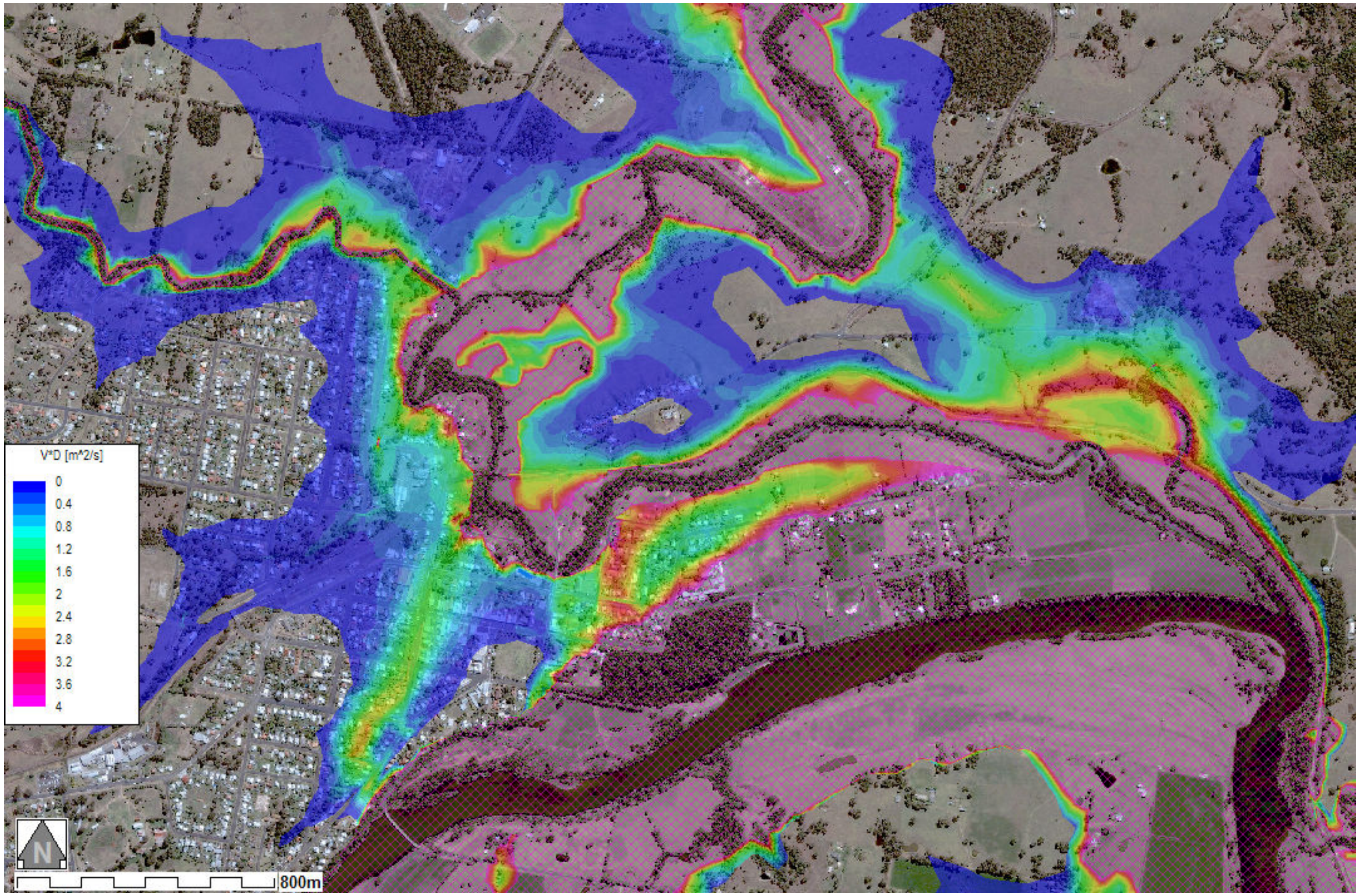


Figure 44: PMF Design Flood; Wingham Overview – Velocity times Depth Coloured (Manning Catchment Model)

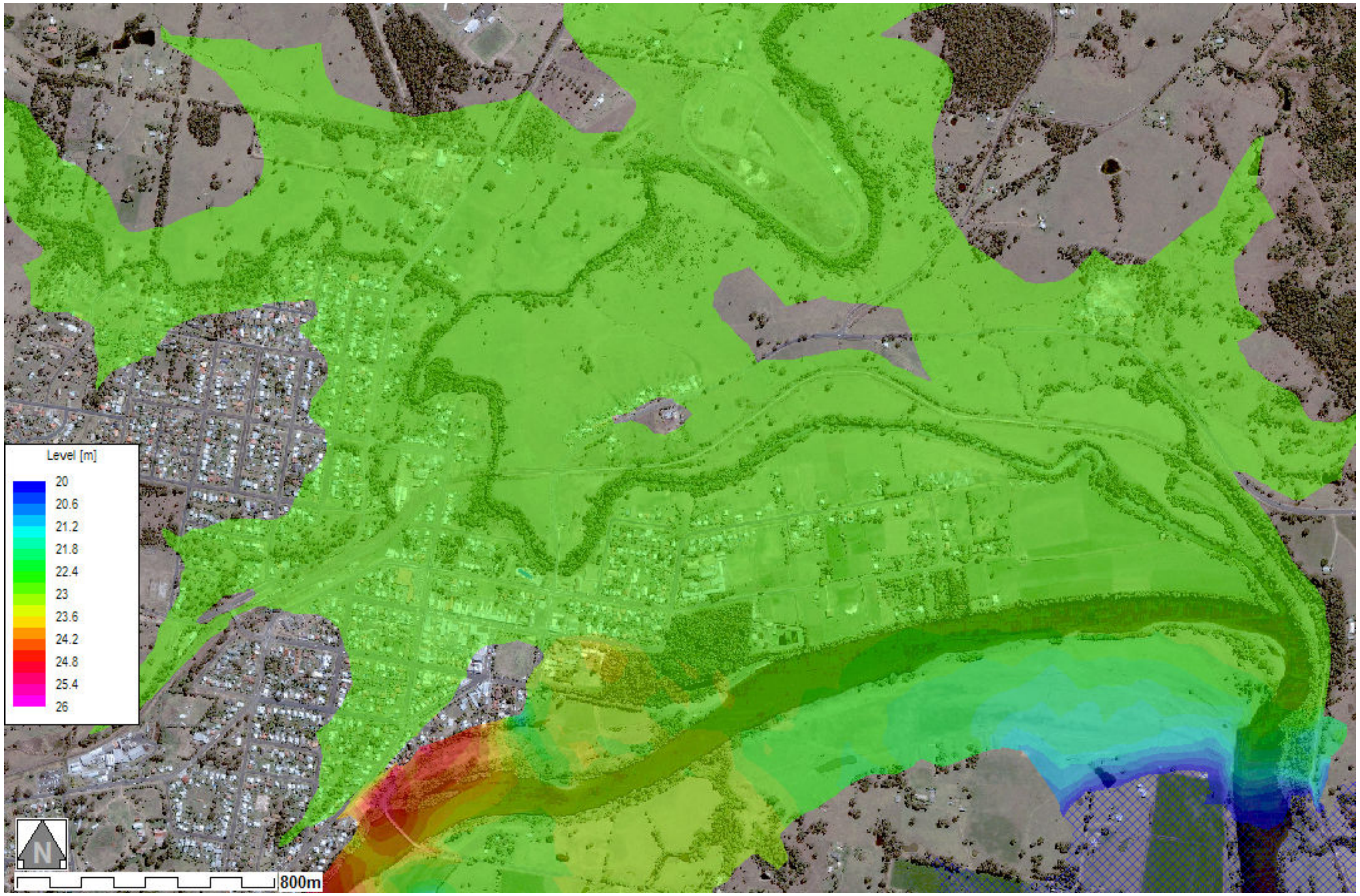


Figure 45: PMF Design Flood; Wingham Overview – Water Level Coloured (Manning Catchment Model)

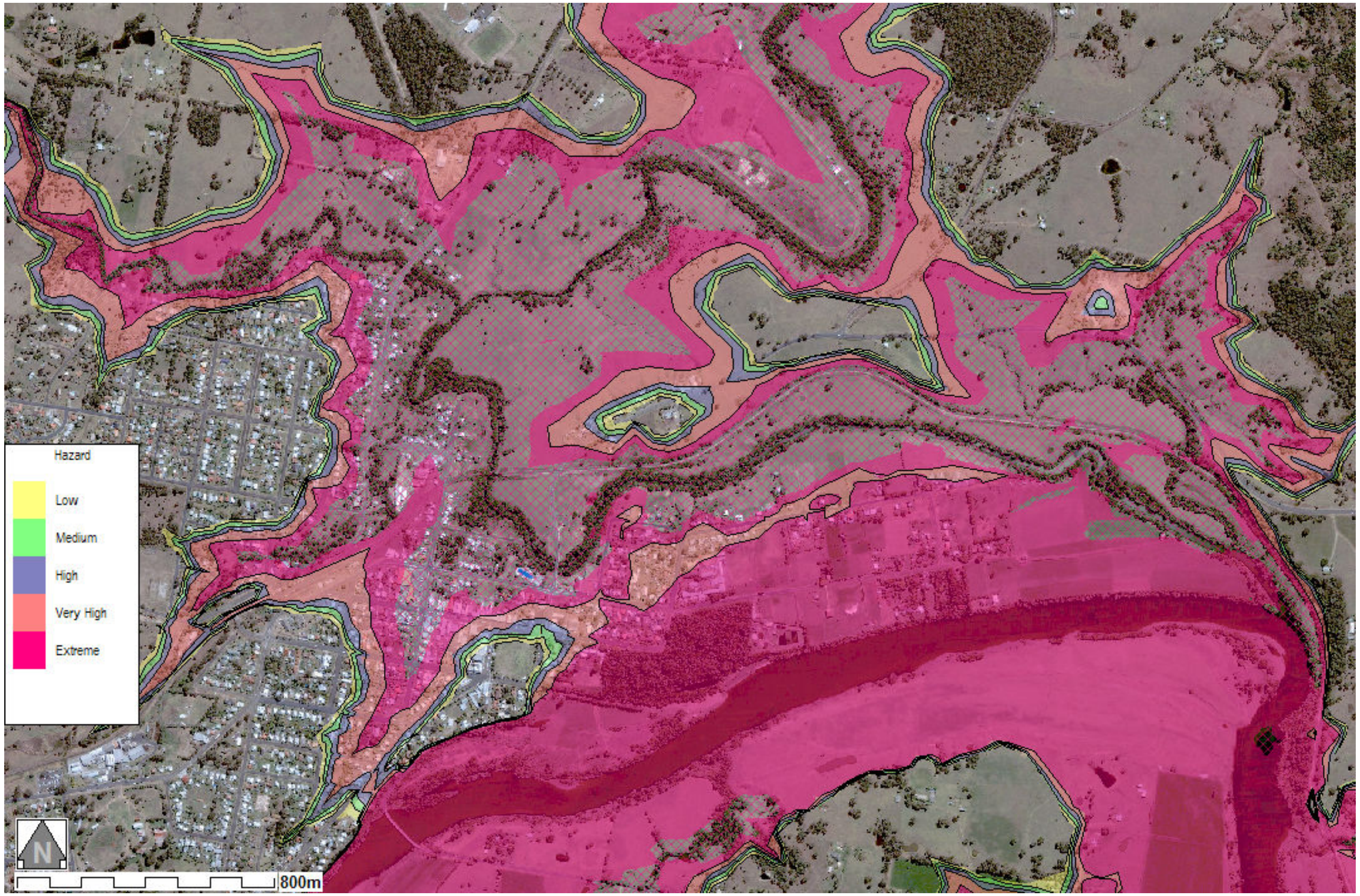


Figure 46: PMF Design Flood; Wingham Overview - Hydraulic Hazard Coloured (Manning Catchment Model)

6.5.2 Cedar Party Catchment Hydraulic Model

Figure 47 shows the peak water surface profile from the confluence of Cedar Party Creek and the Manning River, along Cedar Party Creek and along Stony Creek after the Stony-Cedar Party Confluence.

Figure 48 to Figure 61 show the results from the Cedar Party Catchment Hydraulic Model centred on Wingham, showing the peak

- Depth with velocity vectors (for all design flood simulations)
- Velocity times depth (for all design flood simulations)
- Water level (only for the 1% AEP and PMF design flood simulations); peak levels are essentially constant in Wingham for each design flood
- Hydraulic Hazards (only for the 1% AEP and PMF design flood simulations)

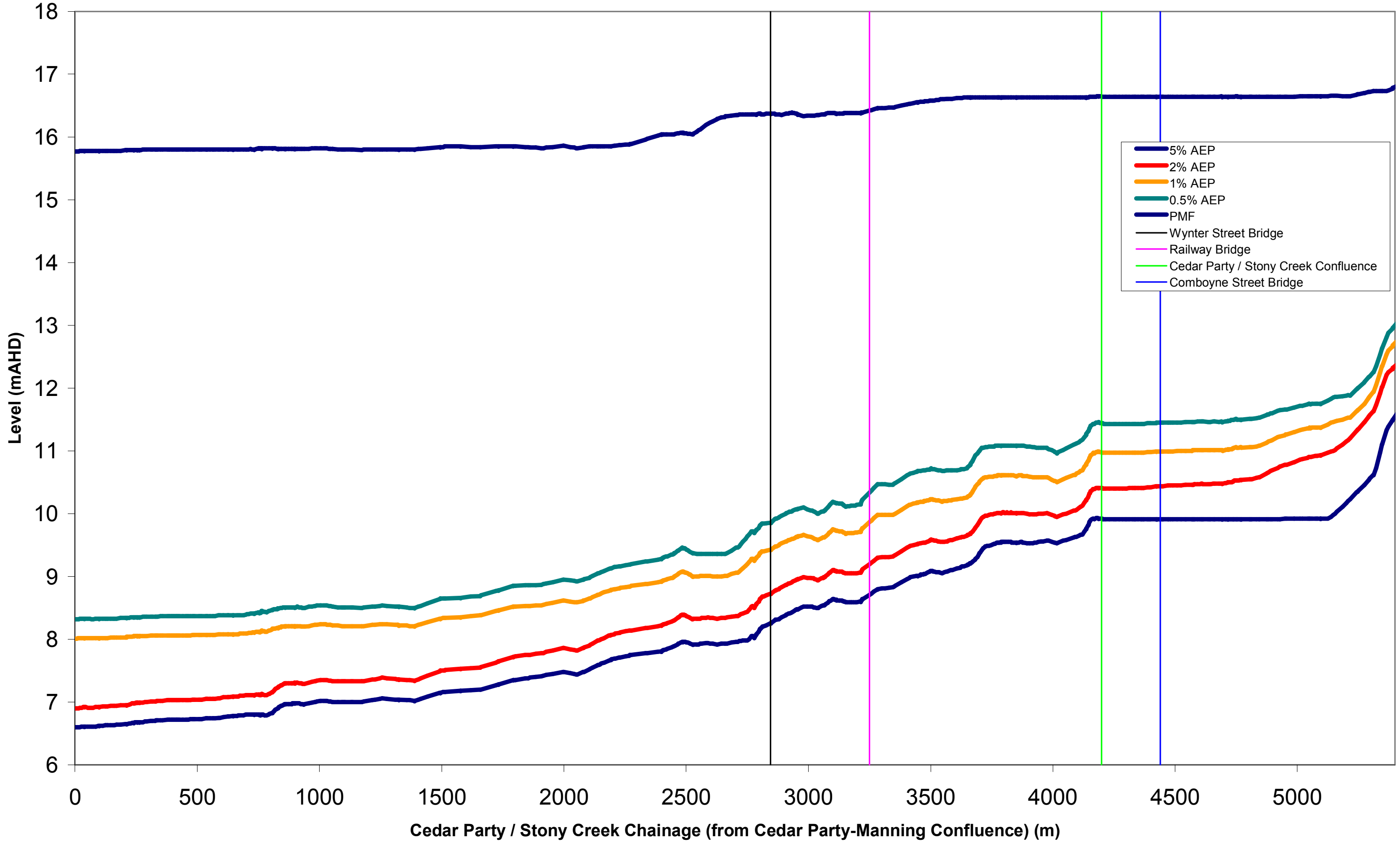


Figure 47: Cedar Party / Stony Creek Peak Water Surface Profile (5%, 2%, 1%, 0.5% and PMF)

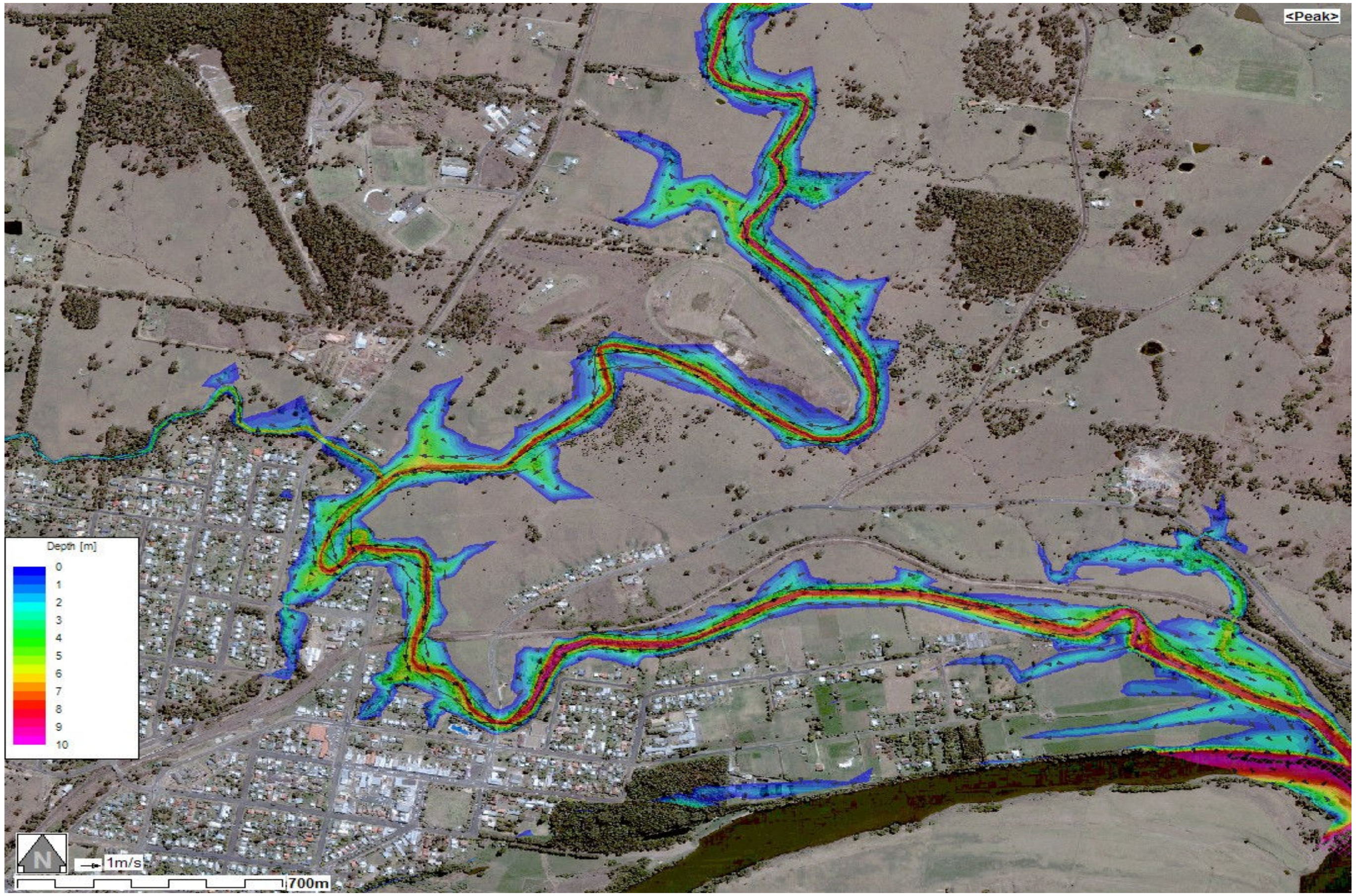


Figure 48: 5% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Cedar Party Catchment Model)

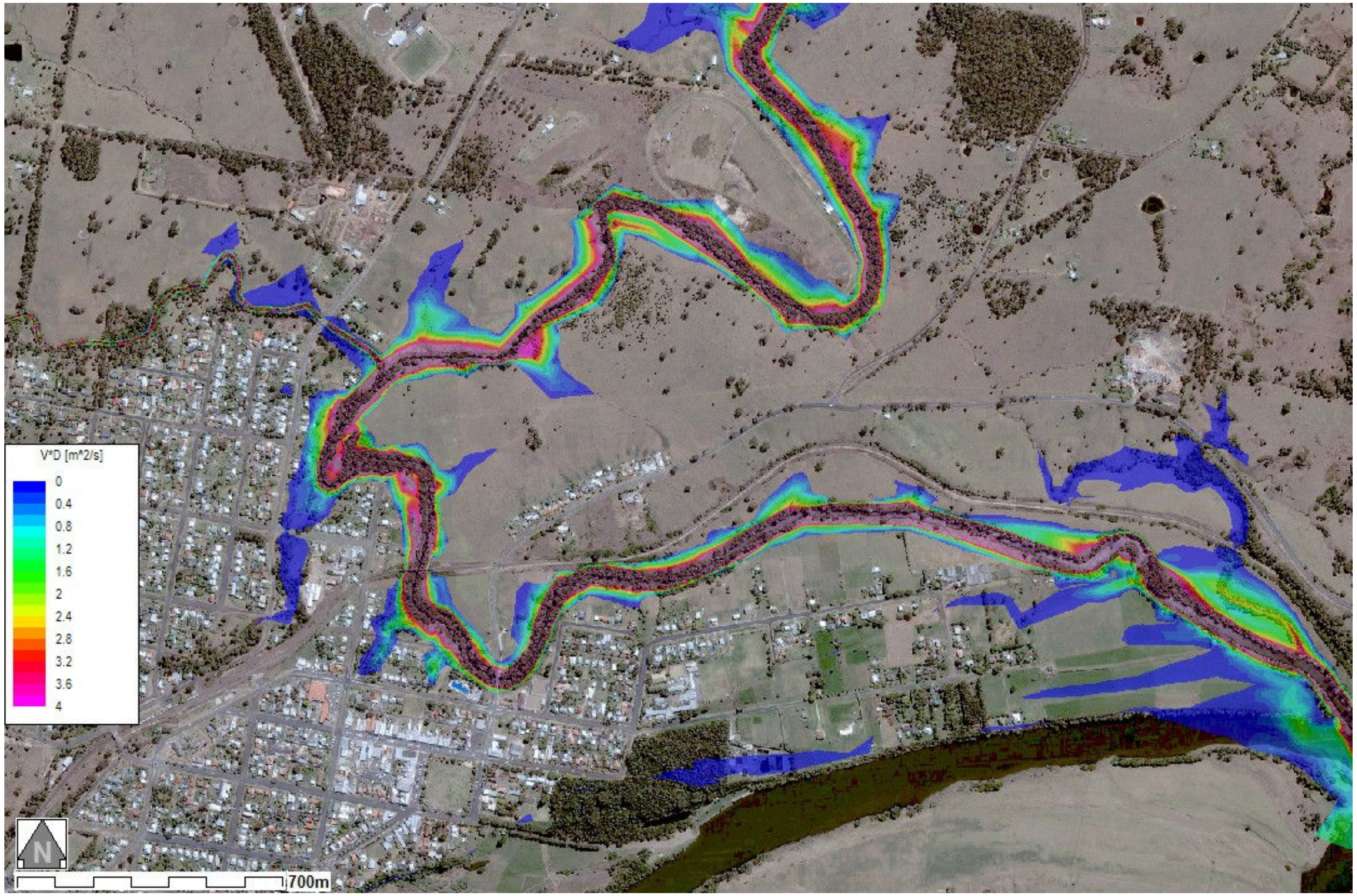


Figure 49: 5% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Cedar Party Catchment Model)

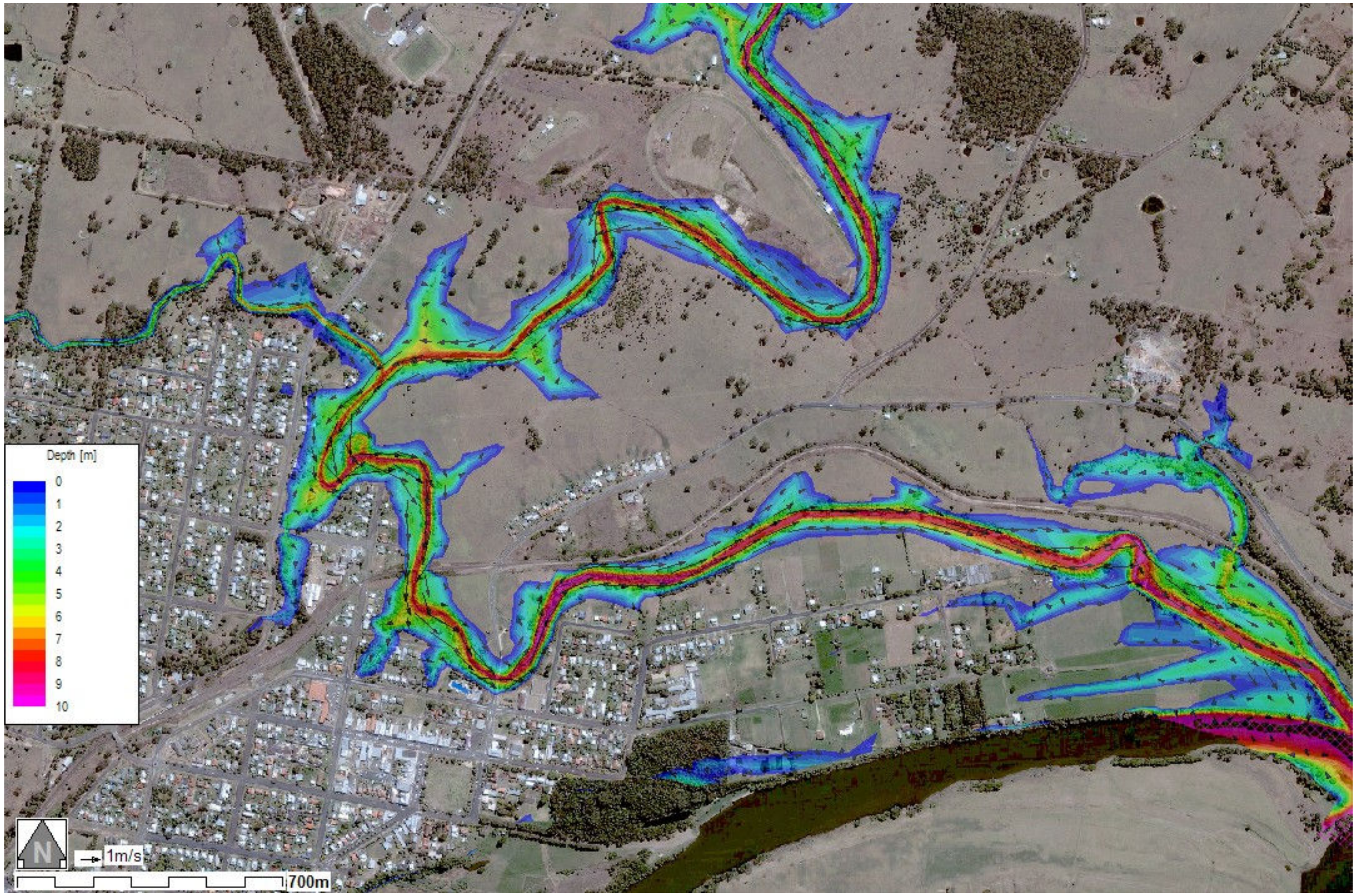


Figure 50: 2% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Cedar Party Catchment Model)

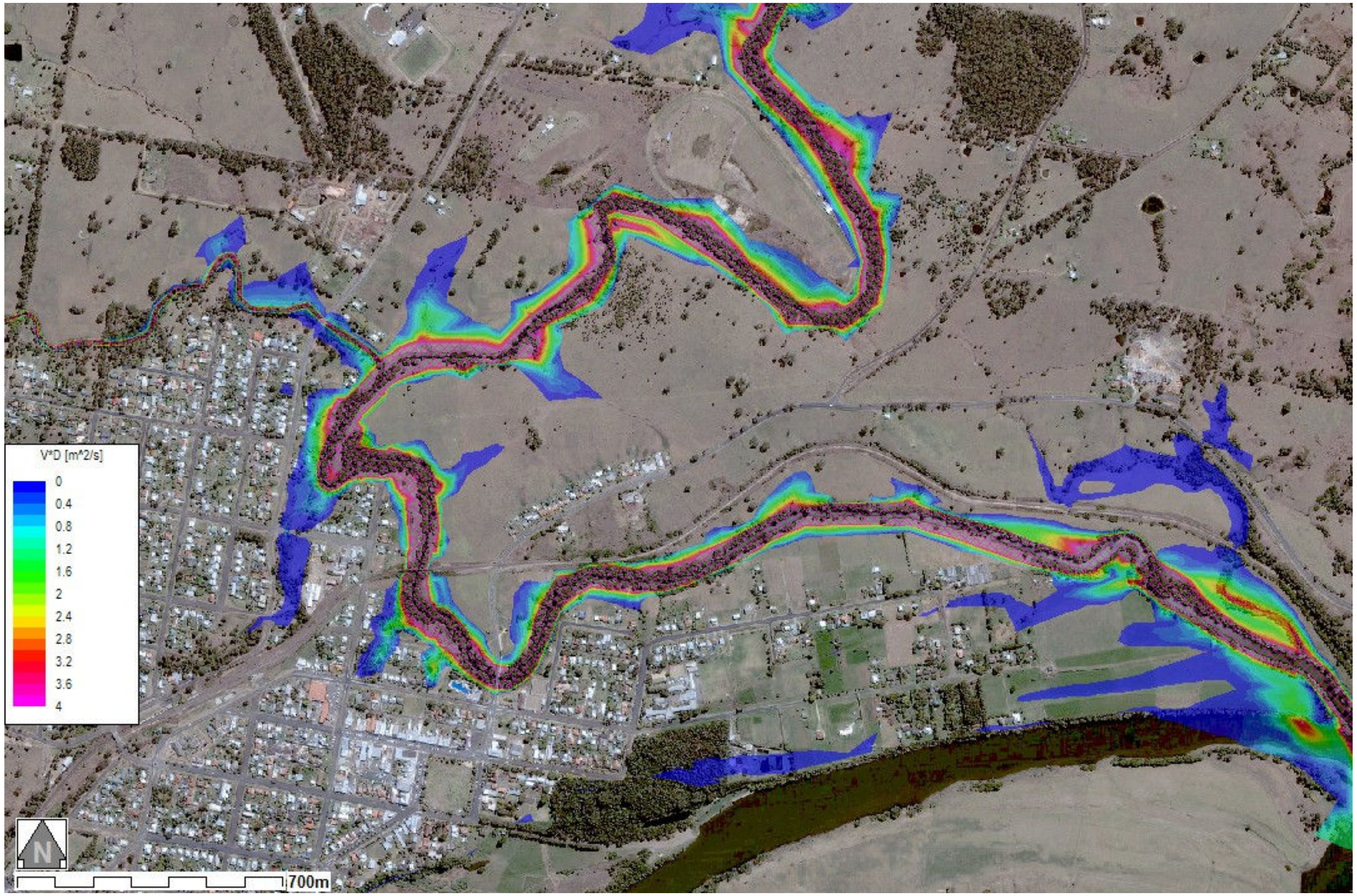


Figure 51: 2% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Cedar Party Catchment Model)

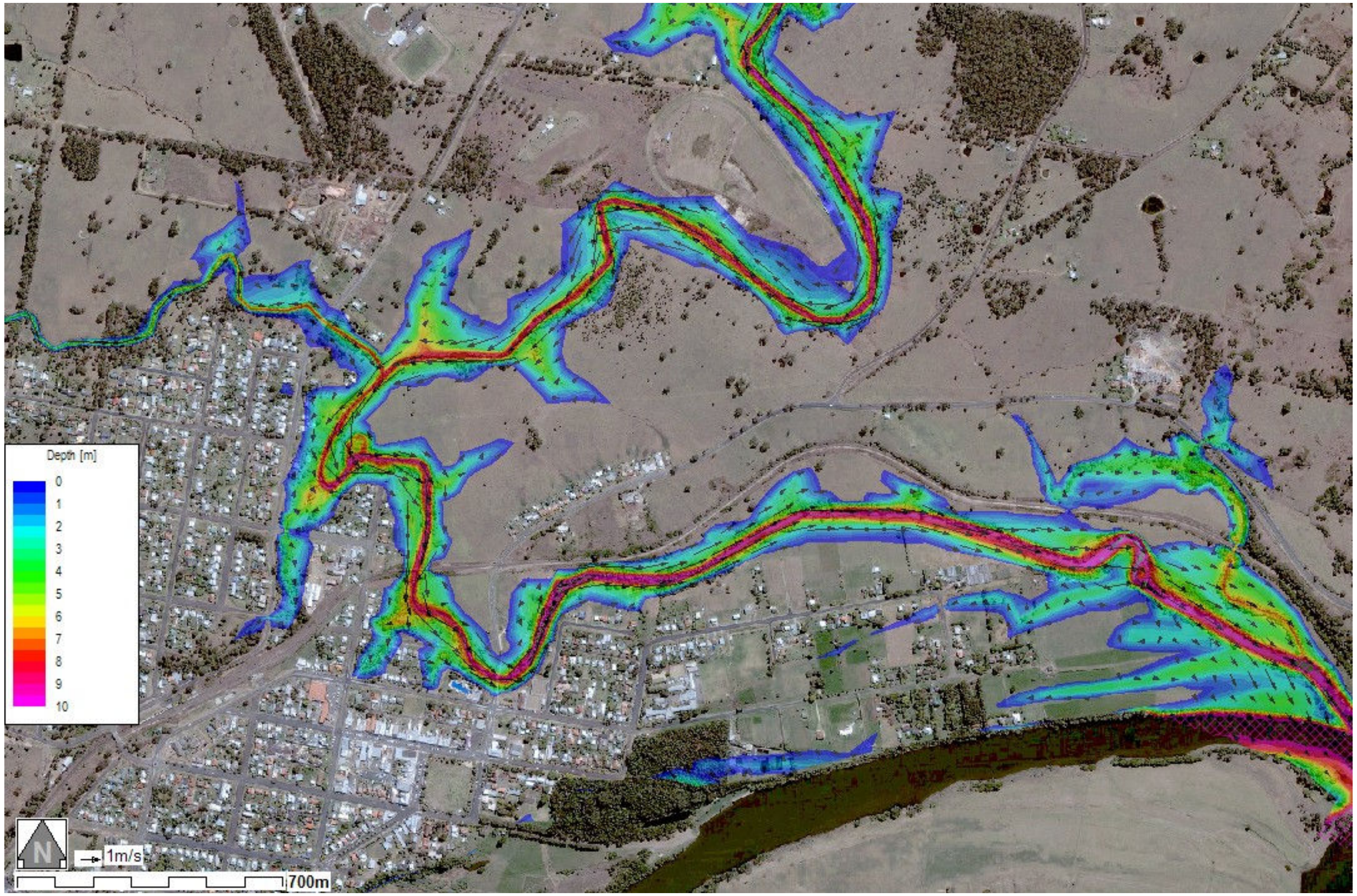


Figure 52: 1% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Cedar Party Catchment Model)

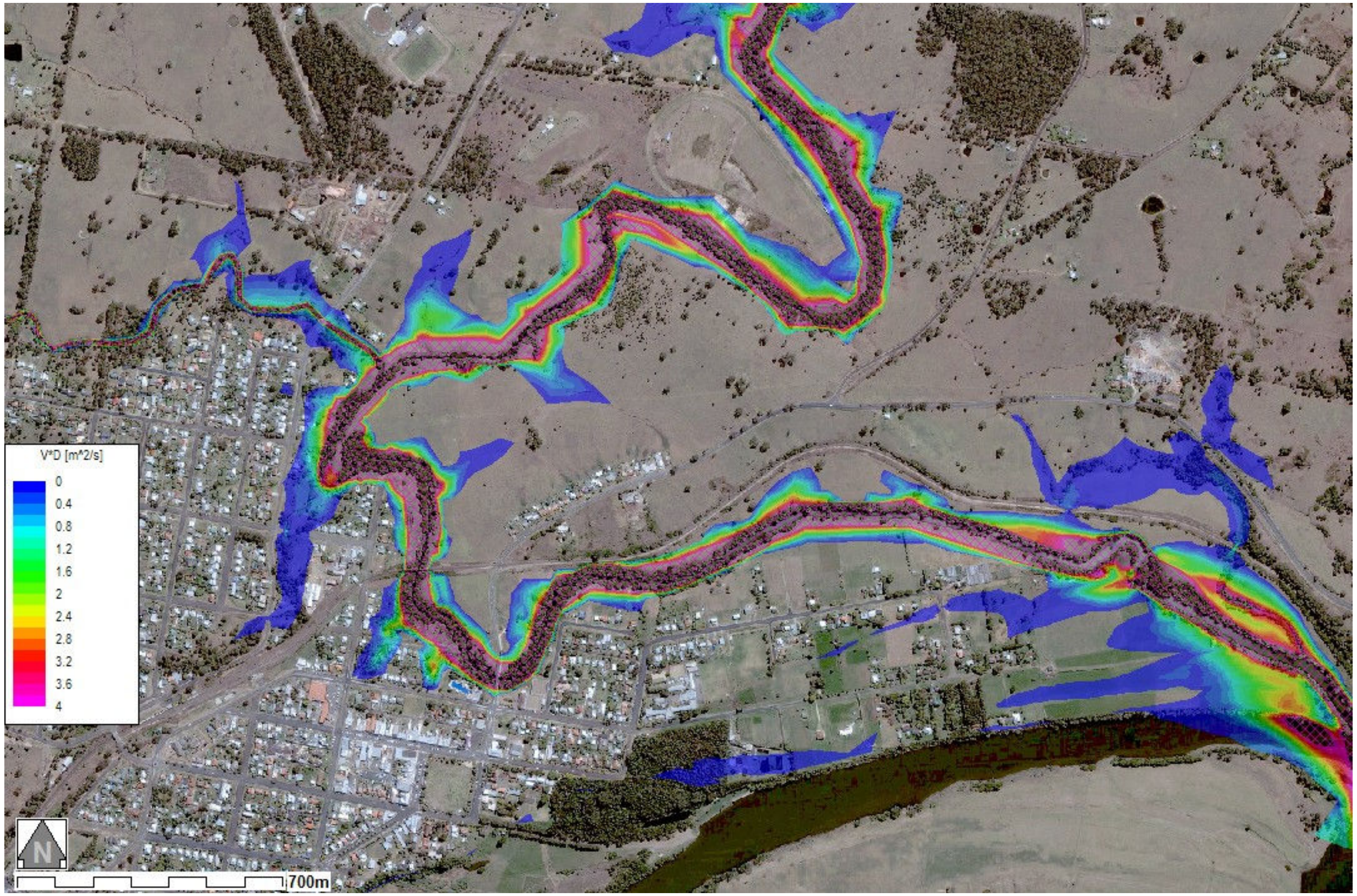


Figure 53: 1% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Cedar Party Catchment Model)

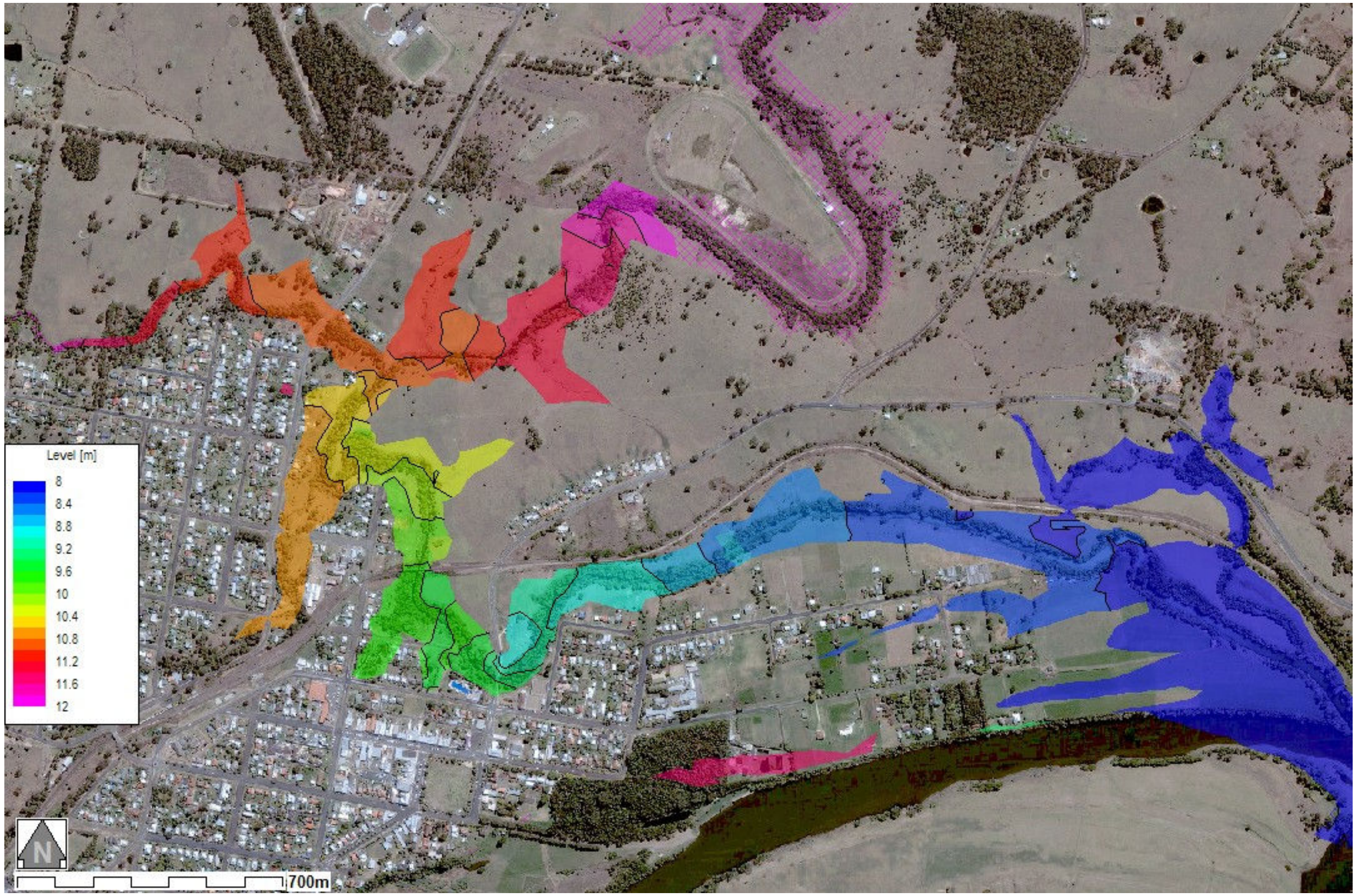


Figure 54: 1% AEP Design Flood; Wingham Overview – Water Level Coloured (Cedar Party Catchment Model)

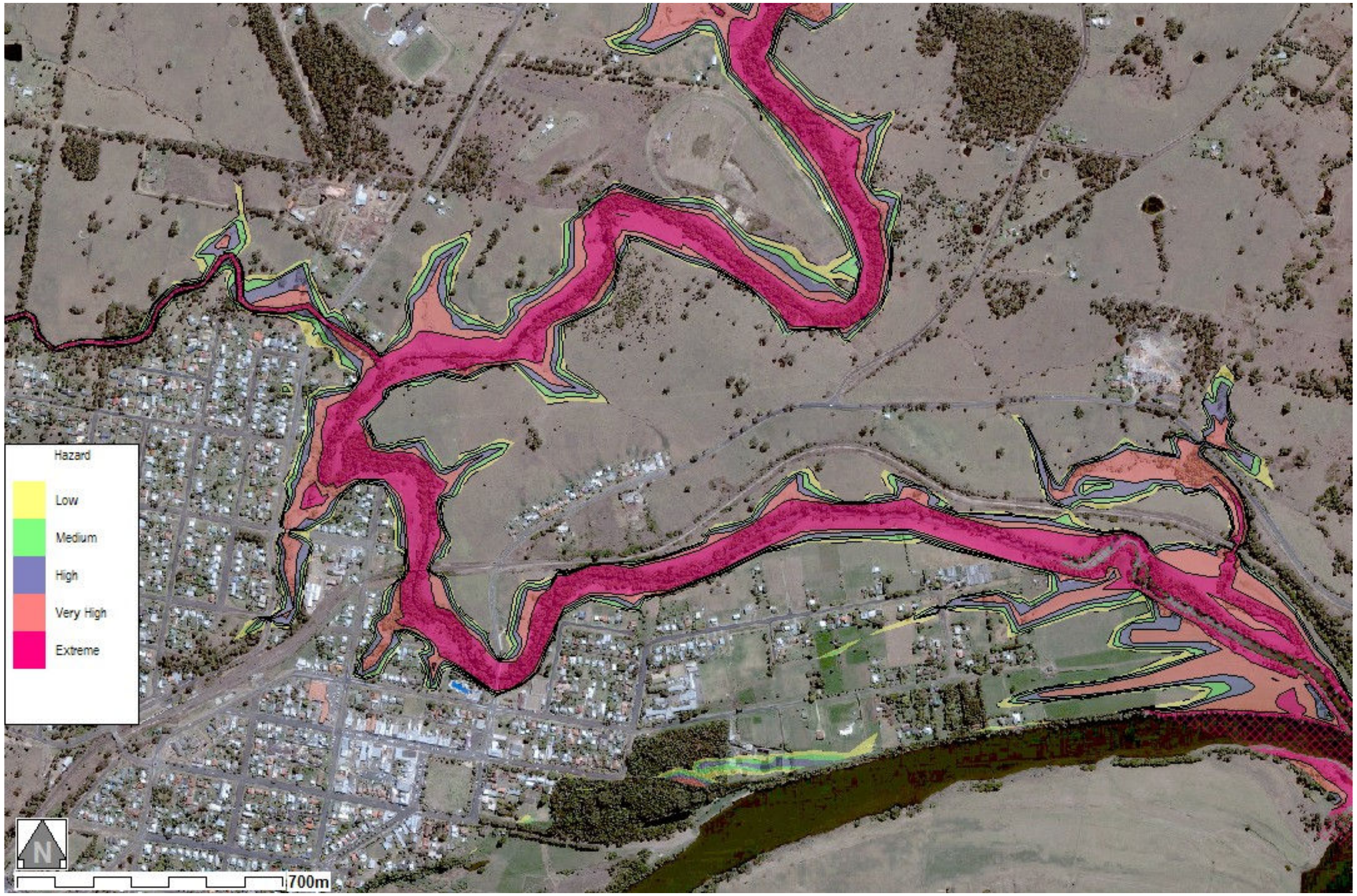


Figure 55: 1% AEP Design Flood; Wingham Overview - Hazard Coloured (Cedar Party Catchment Model)

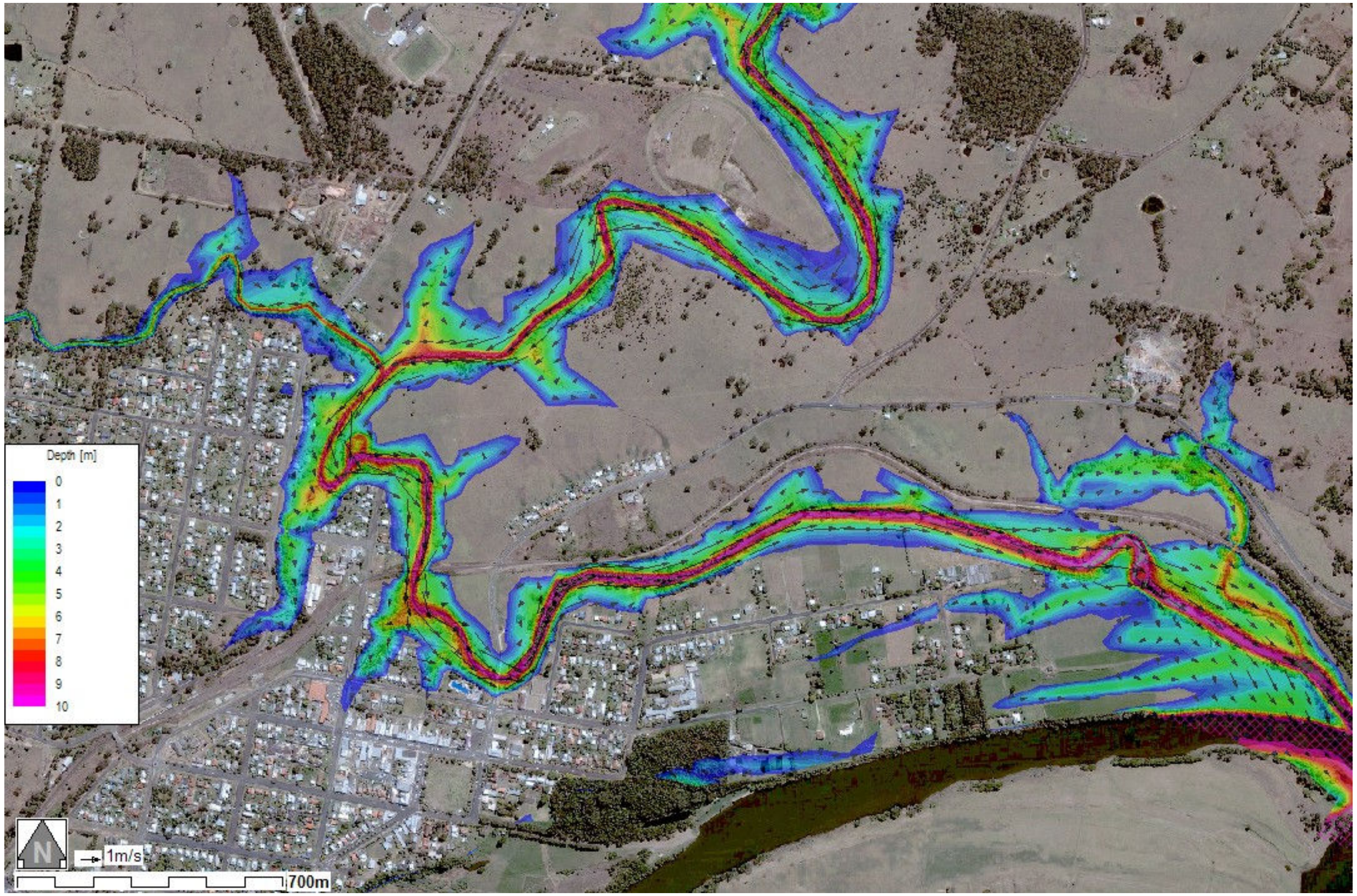


Figure 56: 0.5% AEP Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Cedar Party Catchment Model)

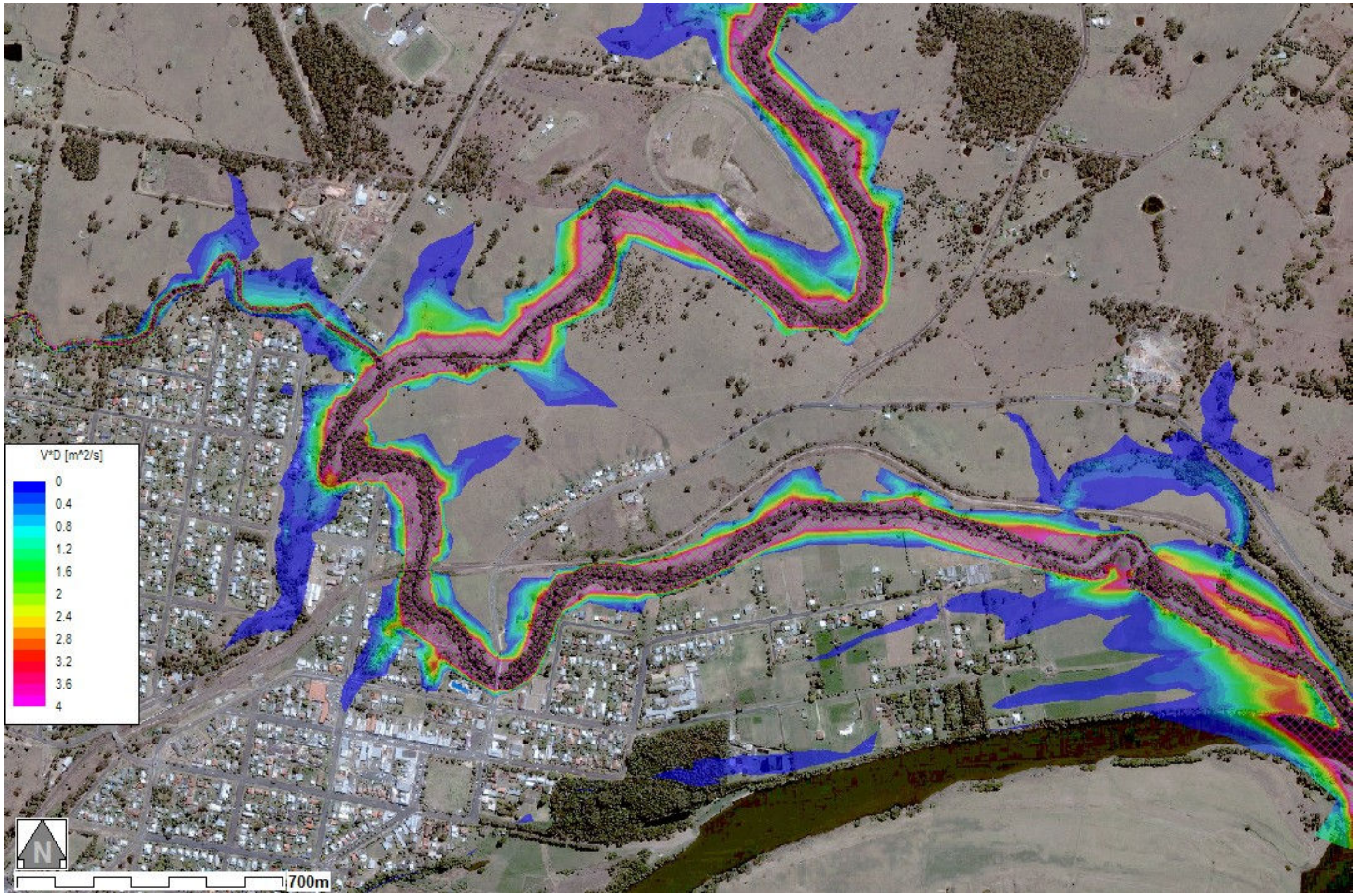


Figure 57: 0.5% AEP Design Flood; Wingham Overview – Velocity times Depth Coloured (Cedar Party Catchment Model)

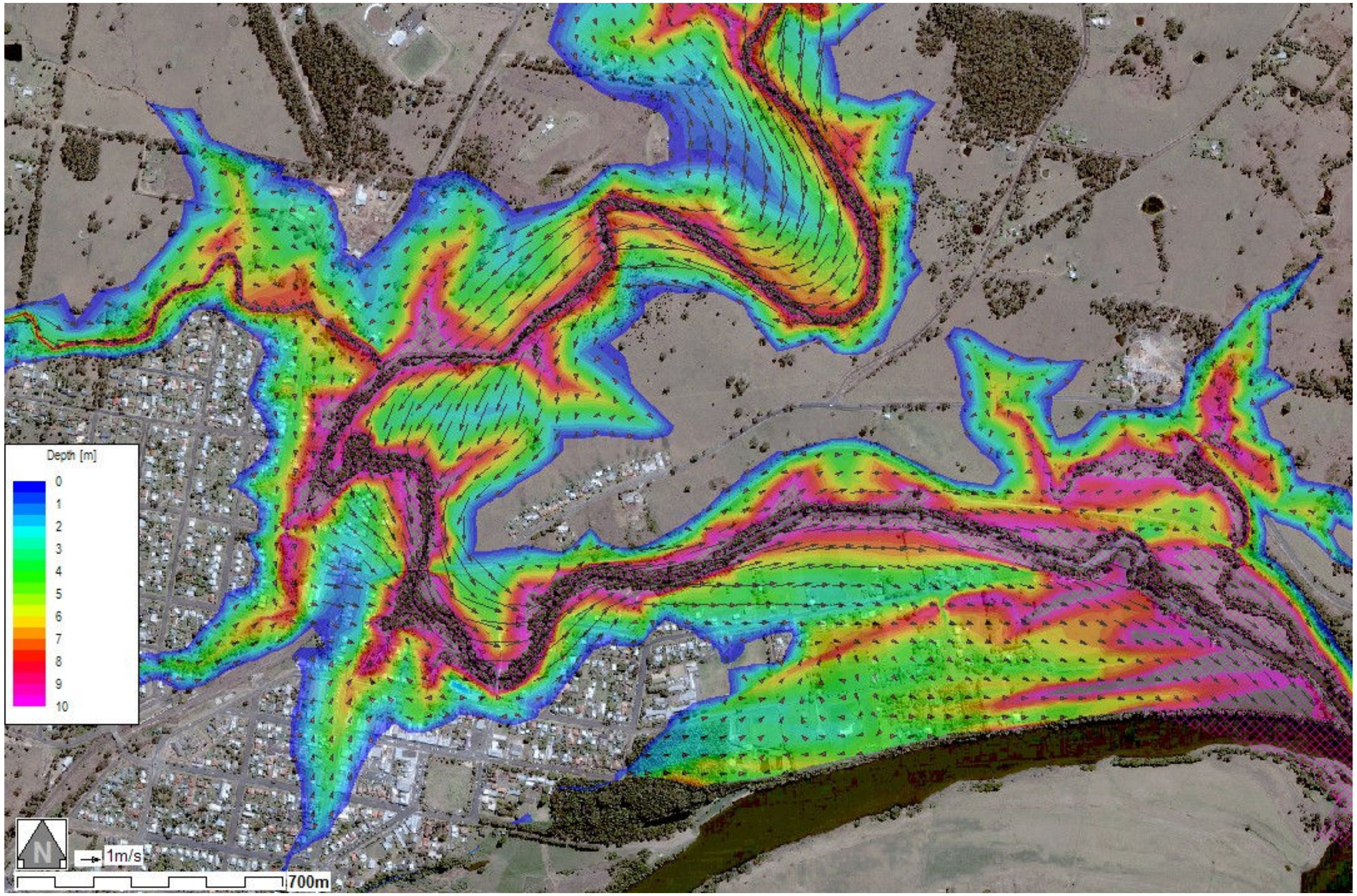


Figure 58: PMF Design Flood; Wingham Overview - Depth Coloured with Velocity Vectors (Cedar Party Catchment Model)

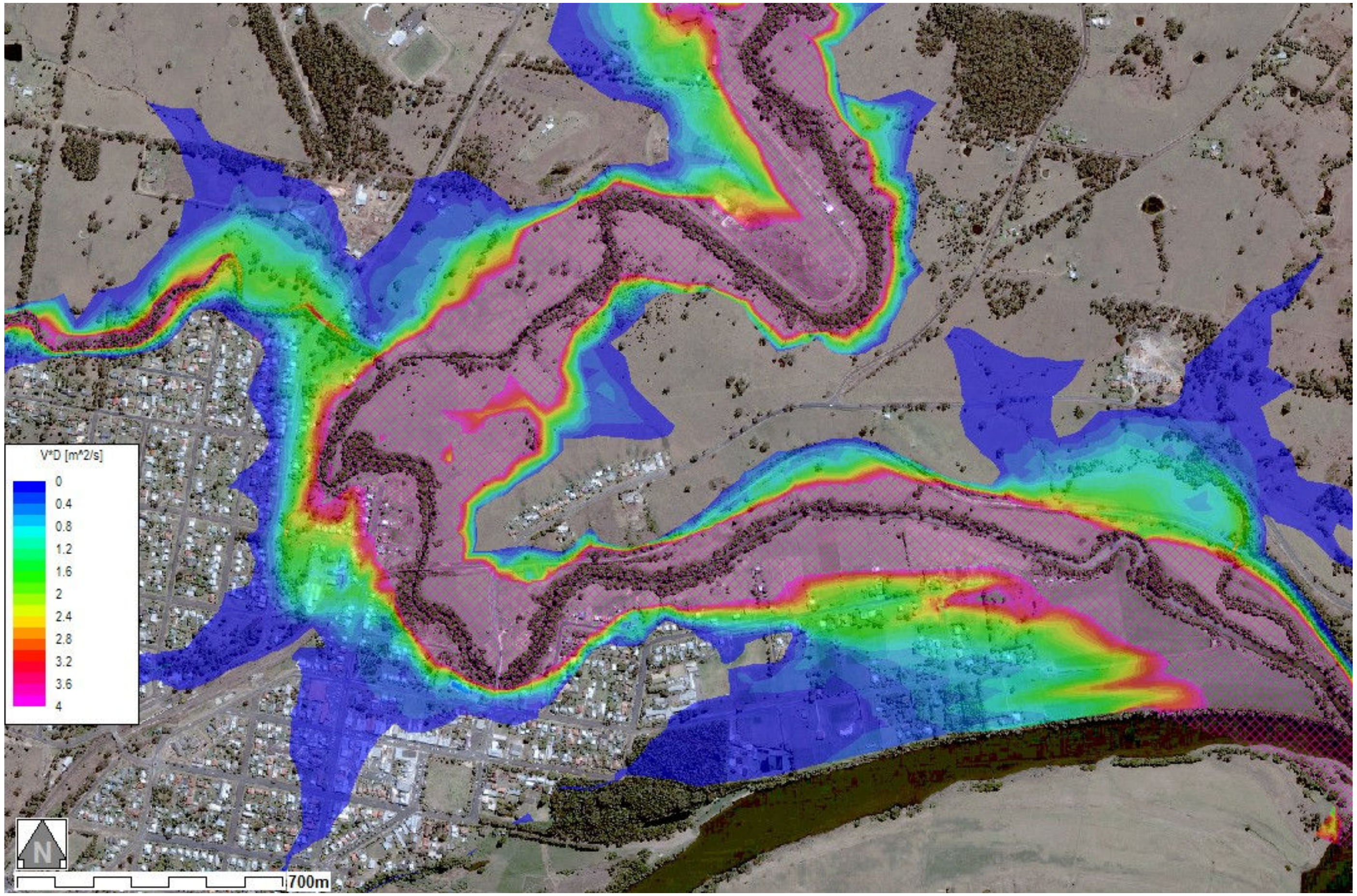


Figure 59: PMF Design Flood; Wingham Overview – Velocity times Depth Coloured (Cedar Party Catchment Model)

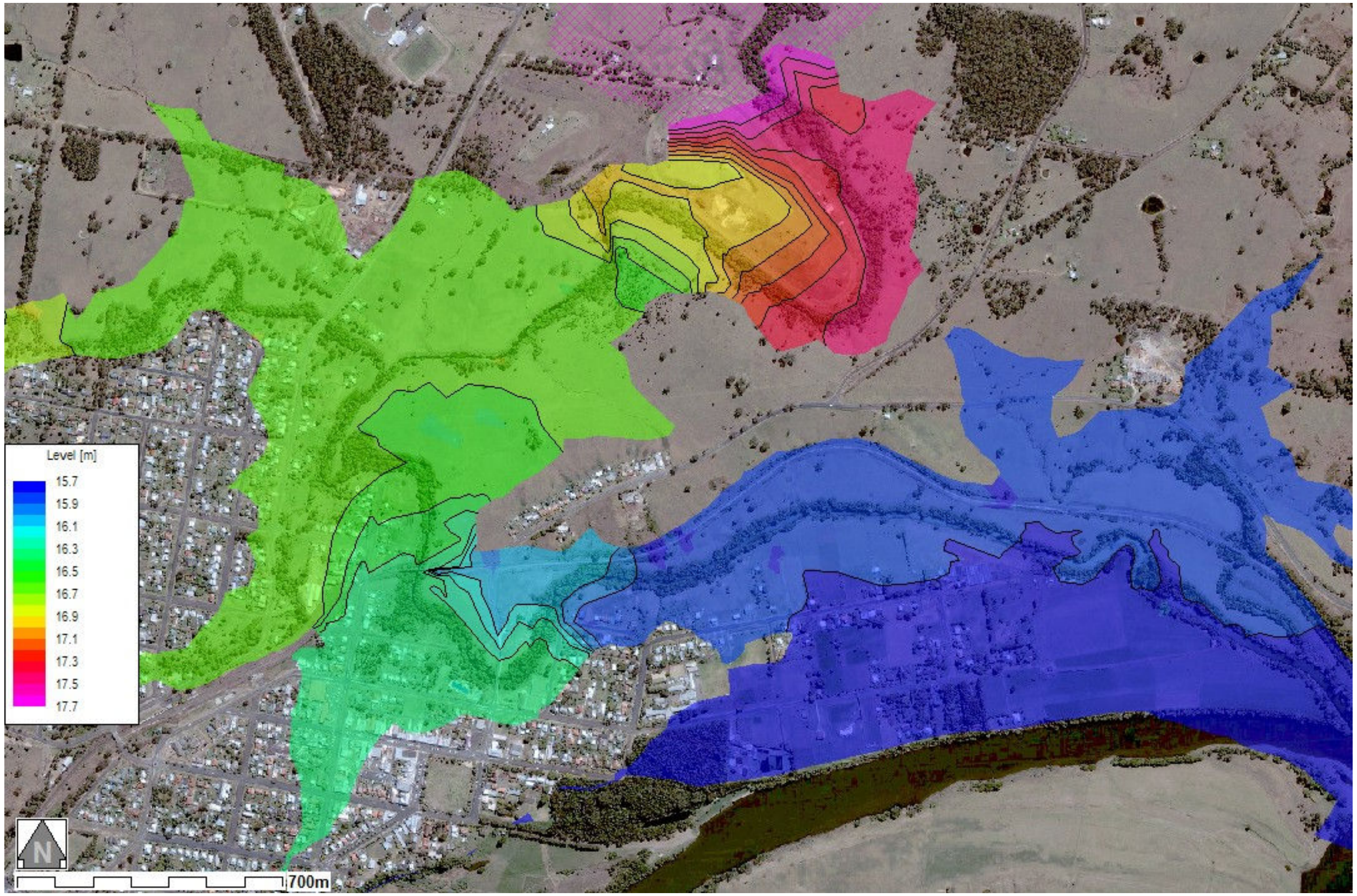


Figure 60: PMF Design Flood; Wingham Overview – Water Level Coloured (Cedar Party Catchment Model)

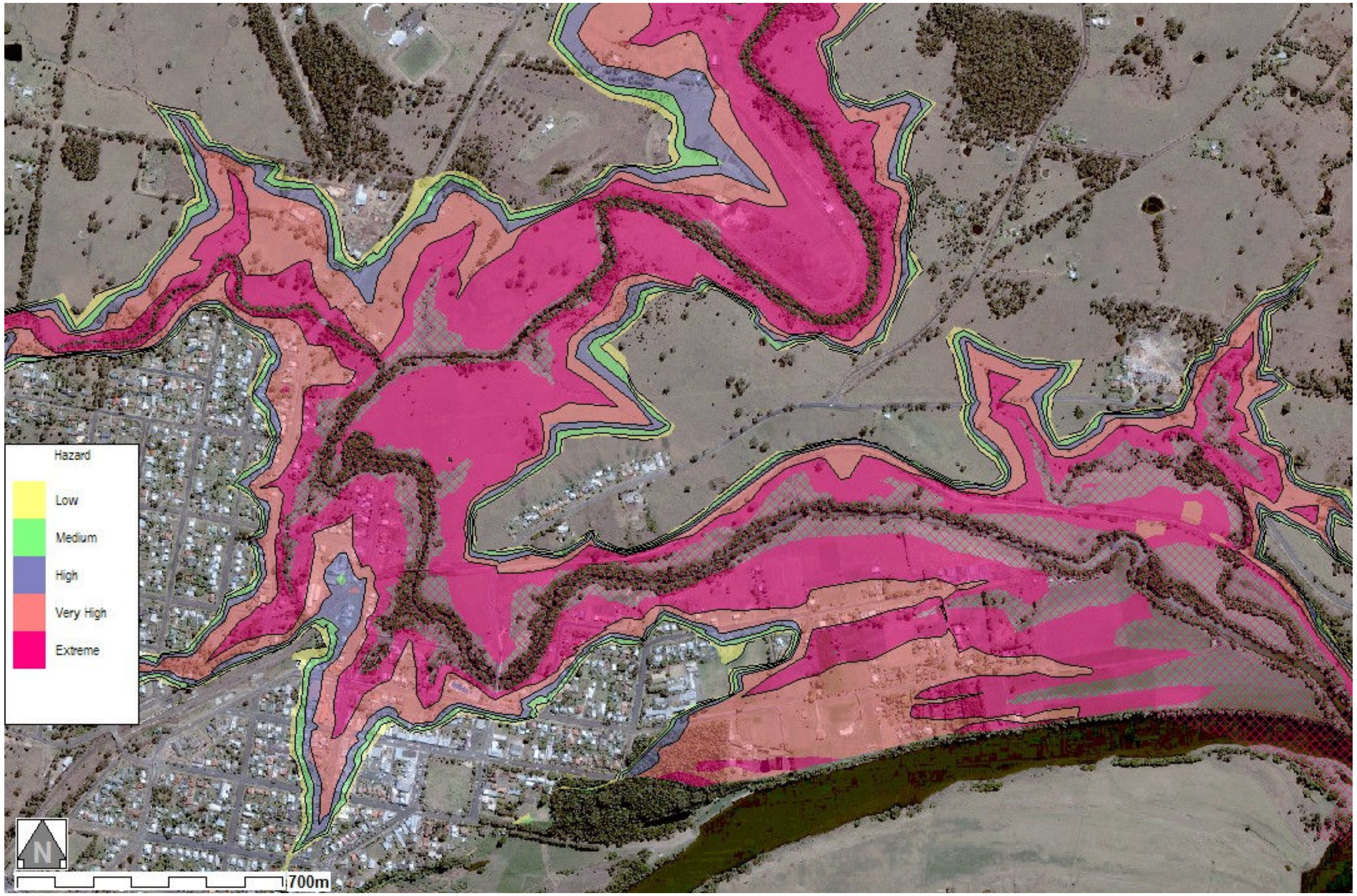


Figure 61: PMF Design Flood; Wingham Overview – Hazard Coloured (Cedar Party Catchment Model)

7. REFERENCES

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14. Ted Rigby, Michael Boyd and Rudy VanDrie; "Watershed Bounded Network Model" WBNM2003 version 1.04 and Documentation Manuals; January 2007
15. Ian P. King, Resource Modelling Associates Sydney, Australia; "RMA2 – A Two Dimensional Finite Element Model For Flow In Estuaries and Streams" Version 8.0 and Documentation; March 2007.
16. Patterson Britton & Partners; "Wingham Peninsula Floodplain Management Plan Draft"; February 2000
17. Laurie, Montgomerie & Pettit; "New South Wales Coastal Rivers Flood Plain Management Studies Summary Report Manning Valley; October 1980

Appendix A – Tabulated Hydrograph Data

The following table contains the design hydrographs for Cedar Party and Stony (Gorman) Creek output from the hydrologic study for use in the hydraulic study.

Table 3: Cedar Party Creek Design Inflow Hydrographs (50%, 20%, 10%, 5% AEPs)

	1 in 2 yr ARI (50% AEP)	1 in 5 yr ARI (20% AEP)	1 in 10 yr ARI (10% AEP)	1 in 20 yr ARI (5% AEP)
Time (mins)	Flow (m^3)	Flow (m^3)	Flow (m^3)	Flow (m^3)
0	0	0	0	0
30	0	0	0	0
60	0	0	0	0
90	0	0	0	0
120	0	0	0	0
150	0	0	0	0
180	0	0	0	0
210	0	0	0	0
240	0	0	0	0
270	0	0	0	0
300	0	0	0	0
330	0	0	0	0
360	0	0	0	0
390	0	0	0	0
420	0	0	0	0
450	0	0	0	0
480	0	0	0	0
510	0	0	0	0
540	0	0	0	1
570	0	0	1	2
600	0	1	2	5
630	0	1	4	8
660	0	2	6	13
690	1	4	9	20
720	1	6	14	29
750	2	9	20	40
780	3	14	29	55
810	6	21	41	72
840	9	30	54	90
870	13	41	70	110
900	20	55	88	132
930	28	71	108	154
960	39	89	128	176
990	55	113	156	209
1020	78	146	193	253

1050	108	186	241	311
1080	145	236	299	382
1110	185	289	361	457
1140	224	340	419	525
1170	260	383	467	578
1200	289	414	499	610
1230	307	429	512	620
1260	314	431	510	613
1290	312	424	498	595
1320	306	410	481	572
1350	292	390	455	540
1380	274	364	424	501
1410	253	335	389	458
1440	231	303	352	413
1470	207	272	315	369
1500	185	242	279	327
1530	164	214	248	290
1560	145	190	220	258
1590	128	169	196	231
1620	113	150	175	207
1650	100	134	157	187
1680	89	120	142	169
1710	79	108	128	154
1740	70	97	115	139
1770	62	87	104	125
1800	55	78	93	112
1830	49	70	83	101
1860	44	62	74	90
1890	39	56	66	80
1920	35	50	59	71
1950	32	45	53	63
1980	29	40	47	56
2010	26	36	42	50
2040	23	32	38	44
2070	21	29	34	40
2100	19	26	30	35
2130	17	24	27	32
2160	16	21	25	29
2190	15	19	22	26

Table 4: Cedar Party Creek Design Inflow Hydrographs (2%, 1%, 0.5%, 0.2% AEPs)

	1 in 50 yr ARI (2% AEP)	1 in 100 yr ARI (1% AEP)	1 in 200 yr ARI (0.5% AEP)	1 in 500 yr ARI (0.2% AEP)
Time (mins)	Flow (m^3)	Flow (m^3)	Flow (m^3)	Flow (m^3)
0	0	0	0	0
30	0	0	0	0
60	0	0	0	0
90	0	1	2	4
120	3	5	8	12
150	8	11	17	25
180	24	33	46	63
210	45	61	83	114
240	77	104	138	187
270	126	166	217	287
300	216	277	348	444
330	343	426	520	644
360	480	583	694	842
390	604	720	843	1008
420	690	809	933	1100
450	718	830	945	1099
480	695	793	892	1023
510	636	718	801	909
540	562	630	698	787
570	486	542	597	670
600	414	460	506	566
630	351	389	426	475
660	297	328	358	398
690	251	276	301	333
720	213	233	253	279
750	180	197	214	235
780	154	168	181	198
810	132	143	154	168
840	113	123	132	144
870	98	106	114	124
900	85	92	99	107
930	75	80	86	93
960	66	70	75	81
990	58	62	66	71
1020	51	55	58	63
1050	46	49	52	55
1080	41	43	46	
1110	36	39		
1140	33			

Table 5: Cedar Party Creek Design Inflow Hydrographs (PMF)

PMF (1 in 10 000 year)			
Time (mins)	Flow (m^3)	Time (mins) Continued	Flow (m^3) Continued
0	0	310	1602
10	0	320	1475
20	11	330	1358
30	37	340	1251
40	73	350	1153
50	128	360	1063
60	206	370	982
70	310	380	907
80	447	390	840
90	615	400	779
100	823	410	723
110	1069	420	672
120	1352	430	625
130	1662	440	583
140	1979	450	544
150	2278	460	508
155	2413	470	476
160	2531	480	446
165	2638	490	418
170	2733	500	393
175	2813	510	369
180	2878	520	348
185	2926	530	328
190	2959	540	309
195	2978	550	292
200	2982	560	276
205	2974	570	261
210	2954	580	247
215	2923	590	234
220	2882	600	221
225	2834	610	210
230	2777	620	200
235	2715	630	190
240	2648	640	180
245	2577	650	171
250	2503	660	163
255	2426	670	156
260	2348		
270	2190		
280	2034		
290	1882		
300	1738		

Table 6: Stony (Gorman) Creek Design Inflow Hydrographs (50%, 20%, 10%, 5% AEPs)

1 in 2 yr ARI (50% AEP)		1 in 5 yr ARI (20% AEP)		1 in 10 yr ARI (10% AEP)		1 in 20 yr ARI (5% AEP)	
Time (mins)	Flow (m^3)	Time (mins)	Flow (m^3)	Time (mins)	Flow (m^3)	Time (mins)	Flow (m^3)
0	0	0	0	0	0	0	0
720	1	450	1	420	1	120	1
750	1	510	1	450	1	150	2
780	2	540	2	480	2	180	10
810	3	600	2	510	3	210	13
840	4	630	3	540	3	240	18
870	5	660	4	570	4	270	25
900	7	690	6	600	4	300	46
930	8	720	7	630	6	330	65
960	10	750	10	660	7	360	74
990	14	780	12	690	9	390	76
1020	20	810	15	720	10	420	71
1050	25	840	18	750	13	450	63
1080	31	870	18	780	17	480	54
1110	34	900	17	810	20	510	45
1140	36	930	16	840	23	540	38
1170	37	960	15	870	23	570	31
1200	38	990	16	900	22	600	25
1230	37	1020	19	930	20	630	21
1260	35	1050	22	960	18	660	17
1290	33	1080	26	990	20	690	14
1320	32	1110	32	1020	23	720	11
1350	29	1140	39	1050	27	750	10
1380	25	1170	46	1080	31	780	8
1410	22	1200	52	1110	39	810	7
1440	19	1230	52	1140	47	840	6
1470	16	1260	49	1170	55	870	5
1500	14	1290	45	1200	62	900	4
1530	12	1320	41	1230	61	930	4
1560	10	1350	37	1260	58		
1590	9	1380	32	1290	53		
1620	8	1410	28	1320	48		
1650	7	1440	24	1350	43		
1680	6	1470	22	1380	37		
1710	5	1500	20	1410	32		
1740	4	1530	18	1440	28		
1770	4	1560	17	1470	25		
1800	3	1590	15	1500	23		
1830	3	1620	13	1530	21		
1860	2	1650	12	1560	20		
1950	2	1680	10	1590	18		
1980	1	1710	9	1620	16		

2190	1	1740	7	1650	14		
		1770	6	1680	12		
		1800	5	1710	10		
		1830	5	1740	9		
		1860	4	1770	7		
		1890	4	1800	6		
		1920	3	1830	5		
		1980	3	1860	5		
		2010	2	1890	4		
		2100	2	1950	4		
		2130	1	1980	3		
		2430	1	2040	3		
				2070	2		
				2160	2		
				2190	1		
				2490	1		

Table 7: Stony (Gorman) Creek Design Inflow Hydrographs (2%, 1%, 0.5%, 0.2% AEPs)

	1 in 50 yr ARI (2% AEP)	1 in 100 yr ARI (1% AEP)	1 in 200 yr ARI (0.5% AEP)	1 in 500 yr ARI (0.2% AEP)
Time (mins)	Flow (m^3)	Flow (m^3)	Flow (m^3)	Flow (m^3)
0	0	0	0	0
90	1	1	2	3
120	3	4	6	9
150	5	8	10	14
180	16	20	26	33
210	21	26	33	41
240	27	34	41	50
270	36	43	52	62
300	59	70	82	97
330	81	94	109	128
360	90	104	119	139
390	91	104	118	137
420	85	97	109	125
450	75	85	95	108
480	65	73	81	92
510	54	61	67	76
540	45	50	56	63
570	37	41	45	51
600	30	33	36	41
630	24	27	29	32
660	20	22	24	26
690	16	18	19	21
720	13	14	16	17
750	11	12	13	14
780	9	10	11	11
810	8	8	9	10
840	6	7	7	8
870	5	6	6	7
900	5	5	5	
930	4			

Table 8: Stony (Gorman) Creek Design Inflow Hydrographs (PMF)

PMF (1 in 10 000 year)			
Time (mins)	Flow (m^3)	Time (mins) Continued	Flow (m^3) Continued
0	0	430	37
10	0	440	34
20	12	450	31
30	35	460	29
40	61	470	27
50	93	480	25
60	131	490	23
70	169	500	21
80	212	510	20
90	249		
100	287		
110	319		
120	346		
130	366		
140	378		
150	382		
160	379		
170	371		
180	356		
190	334		
200	310		
210	286		
220	261		
230	238		
240	216		
250	195		
260	177		
270	159		
280	144		
290	130		
300	118		
310	107		
320	97		
330	88		
340	80		
350	73		
360	67		
370	61		
380	56		
390	51		
400	47		

410	43
420	40