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**GREATER TAREE CITY COUNCIL**

# Lansdowne Flood Study

## Review, Upgrade and Extension

301015-02267

29 May 2015

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# FOREWORD

The Lansdowne 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 and around Lansdowne. This, in conjunction with an improved Hydrologic Model of catchment, will provide refined flood data for Lansdowne and therefore establish a sound base for the development of a Lansdowne 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 "Lansdowne Floodplain Risk Management Study" and the "Lansdowne Risk Management Plan" (both produced 2011 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 301015-02267 - LANSDOWNE FLOOD STUDY**

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|---|---|---|---|---|---|---|---|
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## Appendices

# 1. INTRODUCTION

## 1.1 Overview

Lansdowne is located approximately 15 kilometres upstream along the Lansdowne River from the confluence of the Manning River. The Manning valley was first settled in the early nineteenth century and due to the importance of streams as transport routes for primary produce, numerous settlements were located adjacent to rivers below the tidal limit. In this way, the village of Lansdowne is located close to the tidal limit of the Lansdowne River, which is approximately two kilometres downstream from the railway bridge at Lansdowne. At the 2006 census, Lansdowne had a population of 433 people.

Lansdowne village is located in the interfluve between the Lansdowne River and Cross Creek. The numerous gullies in and around the town indicate the alluvial origin of the land with the majority of the developed town elevated between 5m AHD and 15m AHD.

The greater Manning valley catchment drains an area of approximately 8000 $km^2$ with the Lansdowne sub-catchment comprising approximately 215 $km^2$. The Manning catchment extends over 175 km inland from the coast with the upper catchment regions being generally mountainous, undeveloped and elevated up to above 1200m AHD. The Manning River catchment is surrounded by the Hasting and Peel Catchments to the north and the Hunter and Karuah Catchments to the south.

The Lansdowne River catchment consists of a heavily forested and mountainous upper catchment which transitions to a mostly flat lower catchment comprised of grazing paddocks for the beef and dairy industries and bushland.

In the Manning Valley's 180 years of European settlement, many floods of varying severity and impact have been recorded. However in this time, none have conclusively been estimated to have exceeded an Annual Exceedance Probability (AEP) of 1% (that is, an Average Recurrence Interval (ARI) of 1 in 100 years). For the Manning Valley as a whole, the largest floods were approximately equal to a 1% AEP and occurred in July 1866 and February 1929. More recent floods of moderate magnitude have occurred such as in 1978 and 1990. The 1978 flood in particular was one of the largest floods of recent history in the Manning Valley (estimated to be between a 2% and 1% event) which required the evacuation of residents and led to substantial property damage through large parts of the catchment.

The purpose of this Flood Study is to develop a sophisticated, calibrated 2D hydraulic model that will accurately simulate flooding in the region of Lansdowne; the results of which can be used to measure, manage and mitigate flood risks in the study area.

The Flood Study requires a hydrological analysis of rainfall over the Lansdowne catchment in order to produce the necessary hydrograph inputs for the Lansdowne River, Koolah, Rock, Cross and Newbys Creeks (as well as several other smaller unnamed tributaries). A hydrological assessment associated with the Manning River as a whole was undertaken as part of the *"Manning River Flood Study" (NSW Public Works Department; 1991)*. Portions of this study relating to the Lansdowne Area and lower Manning Catchment 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 for the completion of the Lansdowne Floodplain Risk Management Process consists of the extents of Lansdowne Village and its immediate surroundings (Figure 1, Figure 2).

The hydrological component of the study requires the entire Lansdowne sub-Catchment which stretches north approximately 30 kilometres from its confluence with the Manning River and is approximately 10 kilometres wide (east to west).

The hydraulic component of the study requires an area that extends from the confluence of the Manning River upstream along the Lansdowne River to approximately 5 kilometres north-west of Lansdowne village. This includes all land within this bracket of the catchment that has an elevation of less than 20m AHD (based on ensuring that the wet extents of all modelled floods were not restricted). Further details are given in subsequent sections.

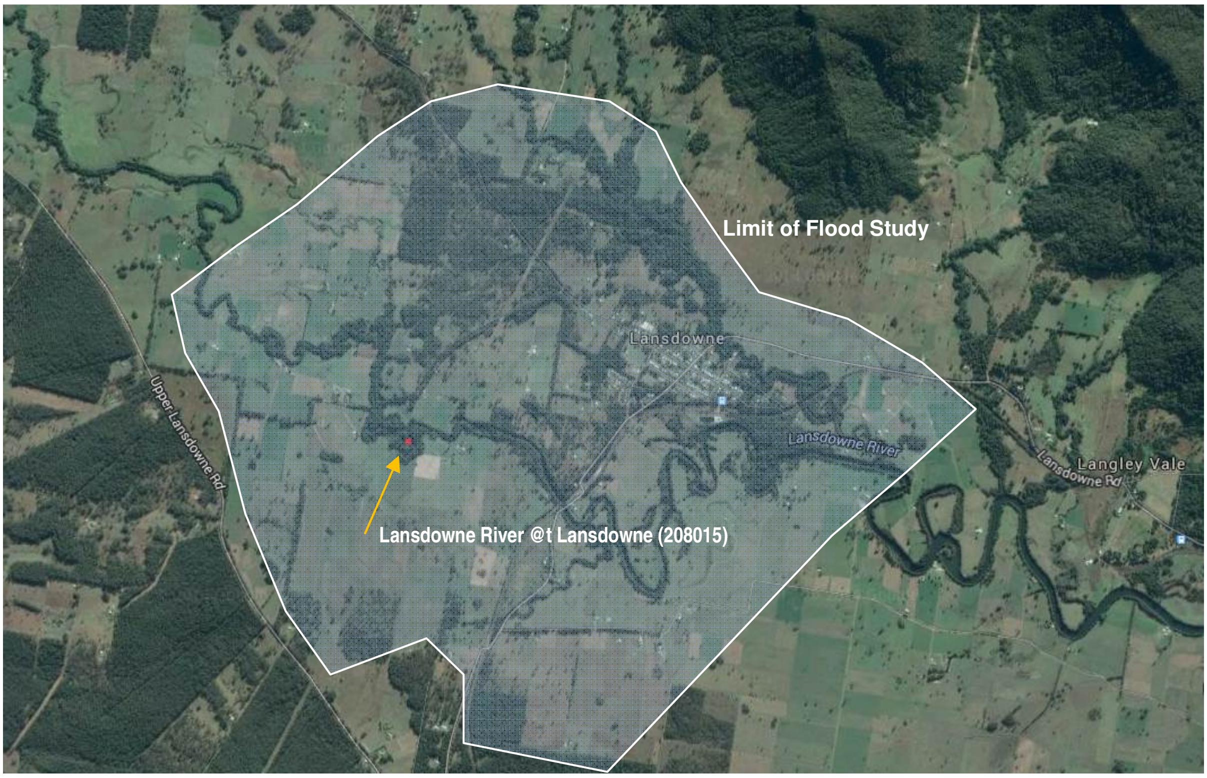


**Figure 1: Study area, including location of Lansdowne River Gauge @ Lansdowne**


**Figure 2: Closeup of Lansdowne, showing Street Names**

## 1.3 Flood History

The "*Manning River Flood History 1831-1979*" (Public Works Department New South Wales) summaries recorded flood data for the Manning Valley. The majority of data available relates to Wingham and Taree and "significant" floods are defined as those reaching a peak height of 3.3m AHD at Taree and/or 10.6m AHD at Wingham. Extremely limited data for the Lansdowne River exists prior to 1969, when a gauge was installed at approximately the tidal limit (several kilometres upstream of Lansdowne village). The earliest flood record for the Lansdowne River is for the flood of 1929, which is generally regarded as one of the largest floods in the Manning Valley since European settlement.

All available data for the region was extracted with flood levels prior to 1969 taken from the "*Manning River Flood History 1831-1979*" and flood levels after 1969 extracted direct from archived gauged data held by the NSW Office of Water (*# 208015*). This data revealed that "significant" flood levels at Lansdowne were not necessarily linked to those at Wingham or Taree. This is supported most prominently through the 1978 Manning Valley flood event which occurred from the 18th of March. Intense widespread rainfall in the upper catchment led to some of the highest flood levels on record in much of the Manning River floodplain however upstream along the Lansdowne River levels were relatively minor. From the 23rd of March, 1978, a more localised storm cell affected the Lansdowne sub-Catchment leading to peak levels in the Lansdowne River that were at least 0.6 metres higher than had occurred in the week prior and this event had a greater impact on Lansdowne Village than the well-known 1978 Manning Valley Flood Event. The latter storm did not adversely affect the greater Manning Catchment.

Another albeit less extreme example of this occurred in March1995, when the Lansdowne gauge recorded its highest level since it was installed. Conversely, this flood was a "minor" flood for Wingham, Taree and the majority of the greater Manning Catchment.

These weather events imply that flooding in Lansdowne is sensitive to variations in local rainfall compared with the Manning Catchment as a whole.

Prior to 1969, flood levels for "significant floods" had been recorded at Coopernook Road Bridge, Coopernook village, Langley Vale and Moto which provides some measure of the magnitude of flood levels likely to have been experienced in the area of Lansdowne village. These flood levels, where available, were converted to approximate levels at Lansdowne by estimating a typical water surface slope between locations.

A flood level of 7.0m AHD was selected as a "significant" at Lansdowne gauge (located several kilometres upstream of Lansdowne village), because this would cause some floodplain inundation and affect properties in low lying regions based on the available flood history and elevation data (as mentioned, the majority of Lansdowne village occupied by residential development is elevated between 5 and 15m AHD).

Figure 3 shows the date and level of these "significant" flood events (NB: prior to 1969 floods were recorded based on their impact in the Manning Catchment and therefore flood events that predominantly involving the Lansdowne Catchment alone are likely to have been omitted).

From this information, at least 25 floods exceeding 7.0m AHD have occurred in the region of Lansdowne since 1929.

The four largest floods recorded at the Lansdowne gauge occurred in 1929, 1995, 1930 and 1974 with a peak level[1] of 10.3, 9.8, 9.7 and 9.4m AHD respectively.

Large floods are most likely to occur as a result of the summer to autumn cyclonic weather pattern encountering the region illustrated by the fact that nearly one third of floods exceeding 7.0m AHD were recorded in the month of March.

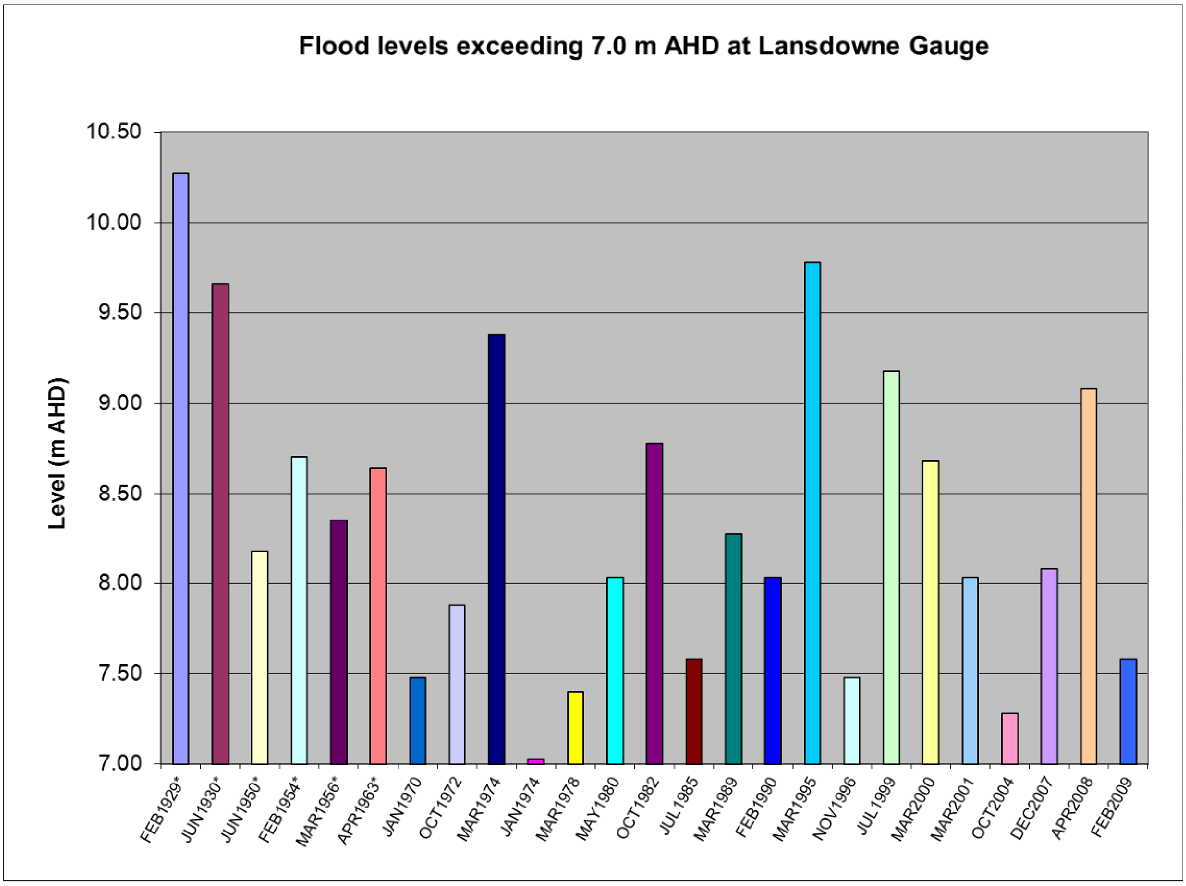


**Figure 3: Floods exceeding 7.0m AHD at the location of the Lansdowne gauge (several kilometres upstream of Lansdowne village).**

***Prior to 1969, levels were estimated based on records at other sub-catchment locations using average water surface slopes based on available data.**

[1] **The peak levels are reported at the position of gauge 208015 which is located several kilometres upstream of Lansdowne village. The peak levels prior to the inception of the gauge in 1969 were estimated from other locations on the Lansdowne River using average relationships between the documented points and the position of the gauge. Therefore levels prior to 1969 are only approximate.**

## 1.4 Floodplain Development Manual Framework

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, economic 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.

# 2. FLOOD STUDY SUMMARY

The broad aim of the Lansdowne Flood Study (referred to as "the flood study" from herein) 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 Flood (PMF). This constitutes the major technical foundation on which the Lansdowne Floodplain Risk Management Study and Plan are based.

The flood study was undertaken in two parts; a hydrologic and a hydraulic analysis. The hydrological analysis involved a comprehensive upgrade of the *"Manning River Flood Study" (NSW Public Works Department; 1991)* (herein referred to as "the previous flood study"). The current flood study considered hydrological aspects that were omitted from the previous study, such as the localised variation in flows along tributaries of the Lansdowne River and the flow in the Lansdowne River upstream of the village of Lansdowne.

This would be used to provide the necessary inputs required to perform the hydraulic analysis which focuses on the village of Lansdowne and its immediate surroundings. Levels for the lower reaches of the Lansdowne and Manning Rivers were critically reviewed and utilised as a boundary condition from the previous flood study along with an analysis of historic relationships.

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 data. This section summarises this process.

The following documents contain local and regional information with relevance to the study area:

- *"Report on Manning River Flood Mitigation" (NSW Public Works Department; 1965)*
- *"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)*
- "*Floodplain Development Manual" (New South Wales Government; 2005)*

The *"Report on Manning River Flood Mitigation" (NSW Public Works Department; 1965)* represented the first general study of flooding in the Manning Valley. In this report part of the Lansdowne sub-catchment was referenced through an area spanning from Langley Vale to Coopernook with several recommendations involving flood gates, drainage and levee banks proposed.

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 fewer 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 modelling 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. For the region of Lansdowne and Coopernook, 66 properties were identified as being affected by the 1% AEP flood event along with the mention of some general mitigation measures.

The ESTRY 1D hydraulic model was updated in the *"Taree to Coopernook Pacific Highway Upgrade Flood / Drainage Management Plan"* (WBM*, 2001)* with minimal changes to the existing modelled flood behaviour.

Due to the broad nature of the Flood and Floodplain Study there existed a need to provide localised flood data on Lansdowne such that a more complete Floodplain Risk Management Study and Plan could be produced.

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

- *Air photos of the Lansdowne 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, 2010)*
- *Manning River Hydrosurvey (extending to the tidal limit of the Lansdowne River) (1998/1999 Department of Environment and Climate Change (DECC))*
- *Geomorphic assessment of the river channel and it's prominent features (WP, 2010)*
- *A Digital Terrain Model (DTM) of the above water level topography derived from Aerial Laser Survey data undertaken by Greater Taree City Council (WP, 2010)*

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). 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 Lansdowne River channel was developed based on the hydrosurvey undertaken by the DECC in 1998/1999. This data was combined with adjacent ALS data, 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 a survey was conducted by WorleyParsons analysing these structures in the Lansdowne region.

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: New South Wales Surface Water Data Archive" and online data from the NSW Office of Water (NSW Government, 2010)*
- *"Manning River Flood Study" Volume 1 and Volume 2 (NSW Public Works Department, 1991)*
- *Bureau of Meteorology Historic Data Archives (Commonwealth of Australia 2010, Bureau of Meteorology)*
- *The Lansdowne 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. This process gave WorleyParsons access to several historic water levels known by residents in the vicinity of Lansdowne that assisted in calibrating 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.

The Lansdowne sub-catchment, being located in the lower reaches of the overall Manning Catchment, experiences two distinctly different responses from local waterways. Flows in the Manning River would tend to be gradual, broad and sustained lasting from hours to weeks. This type of flow would cause a backwater effect within the Lansdowne River, leading to the slow "filling" of low lying areas in the Lansdowne floodplain. This is the type of flooding that would have been modelled in the previous flood study.

The second type of response would be a direct result of intense rainfall over the Lansdowne sub-Catchment. The upper reaches of the Lansdowne sub-Catchment are mountainous with high energy gradients. Local rainfall would therefore elicit a sharp response in upper Lansdowne stream levels similar to those experienced in the west of the Manning Catchment. This response would broaden as flows continued downstream towards the village of Lansdowne with the resultant response of the waterway becoming a superposition of the backwater from the Manning River and the Lansdowne sub-Catchment flows.

Whilst a rapid rise in levels of the Lansdowne River and its tributaries would 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 Lansdowne sub-Catchment would never be expected to coincide with peak levels in the Manning Catchment at the Lansdowne River. This is not only because of the high energy gradients of the Lansdowne sub-Catchment, but also 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.

In summary, the most likely flood mechanism in the Lansdowne sub-Catchment would occur first with a rapidly rising, sharp hydrograph elicited by intense local rainfall that would be supplemented with a smaller backwater affect from the Manning River. Flooding would be characterised by rapidly rising and falling levels with a water surface gradient from upper to lower reaches of the local waterways. At this point in time, levels in the downstream reaches of the Manning River would be slowly rising.

As levels in the Manning River continued to rise and levels in the Lansdowne sub-Catchment continued to fall, a point of inflection would exist when flow would begin to "back fill" from the Manning River. Flooding would be characterised by slow to still flows with little or no surface gradient. Therefore, depending on the severity of rainfall and the progression of the storm system, levels in the streams of the Lansdowne sub-Catchment would rapidly rise, slow or decrease before rising again slowly due to back-water from the Manning River.

Of course the exact flooding response would depend on the relative intensity of rainfall over the Lansdowne Catchment compared with the Manning Catchment as a whole.

# 5. HYDROLOGY

The hydrological assessment is a major component of the flood study and it involves an analysis of the relationship between rainfall and stream flows in the Lansdowne sub-Catchment. The primary outcome from the hydrological component of the flood study is to provide stream hydrograph inputs for the hydraulic model.

This process involves the development and calibration of a hydrological model that 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 hydrological model was developed based on the Watershed Boundary Network Model software (WBNM) version 1.04 (Jan 2007).

The flow in the Lansdowne River was a minor part of the previous flood study, and was undertaken using the RORB hydrology model treating the Lansdowne sub-Catchment as a single entity. Whilst the current model analyses the Lansdowne sub-Catchment and tributaries in greater detail, results from the previous study were used as a first step calibration test method. This was further calibrated and verified using historic data from several documented flood events where both rainfall and river flow data exists in the Lansdowne sub-Catchment.

The calibrated hydrologic model was then used to generate hydrograph inputs for the 5%, 2%, 1% and 0.5% AEP design events as well as the Probable Maximum Flow (PMF) for the Lansdowne River, Koolah, Rock, Cross and Newbys Creeks. In addition, inputs for several smaller unnamed tributaries were also generated during the hydraulic component of this study.

## 5.1 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 perform the hydrologic simulation of the study area. This software package uses a new graphical interface through Microsoft Excel® and Visual Basic (VBA) and it uses the most recent data from Australian Rainfall and Runoff (ARR; 1998 / Bulletin 53; 2003).

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 Lansdowne sub-Catchment, air photos, 25k topographic maps and Council's ALS (converted into a DTM) were analysed using the waterRIDE software package.

The catchment was divided along topographical boundaries into 41 sub-catchments with a total catchment area of approximately 200 km$^2$ (the model did not include catchment areas downstream of Coopernook as these were not required in terms of contributing to the hydrograph outputs). 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 Lansdowne River or major tributary channels) 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 and were therefore omitted.

Once these parameters had been given initial values and then calibrated (details are given in subsequent sections), design rainfall could then be simulated, which varied based on the number of rainfall gauges input into the model (9 gauges were created in and around the Lansdowne sub-Catchment to input rain into the model, based on data from AR&R).

Design storms were derived from AR&R and storms with the required AEP were simulated with durations that varied from 5 to 4320 minutes. In this way, the storm duration that produces the most extreme response of the Lansdowne sub-Catchment (the highest peak discharge) was identified.

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

The hydraulic model required inputs for the Lansdowne River and several of its tributaries at specific locations. These streams and their corresponding sub-Catchment location (referenced in the hydrologic model) are listed below:

- Lansdowne River (S14 OUT)
- Cross Creek (S33 IN)
- Koolah Creek (S39 OUT)
- Rock Creek (S40 OUT)
- Haywards Gully (S30 OUT)
- Unnamed Gully near Brimbin Hill (S17 OUT)

Figure 4 shows a visualisation of the hydrologic model overlaid on the DTM used to subdivide the catchment, and with primary stream channels highlighted.

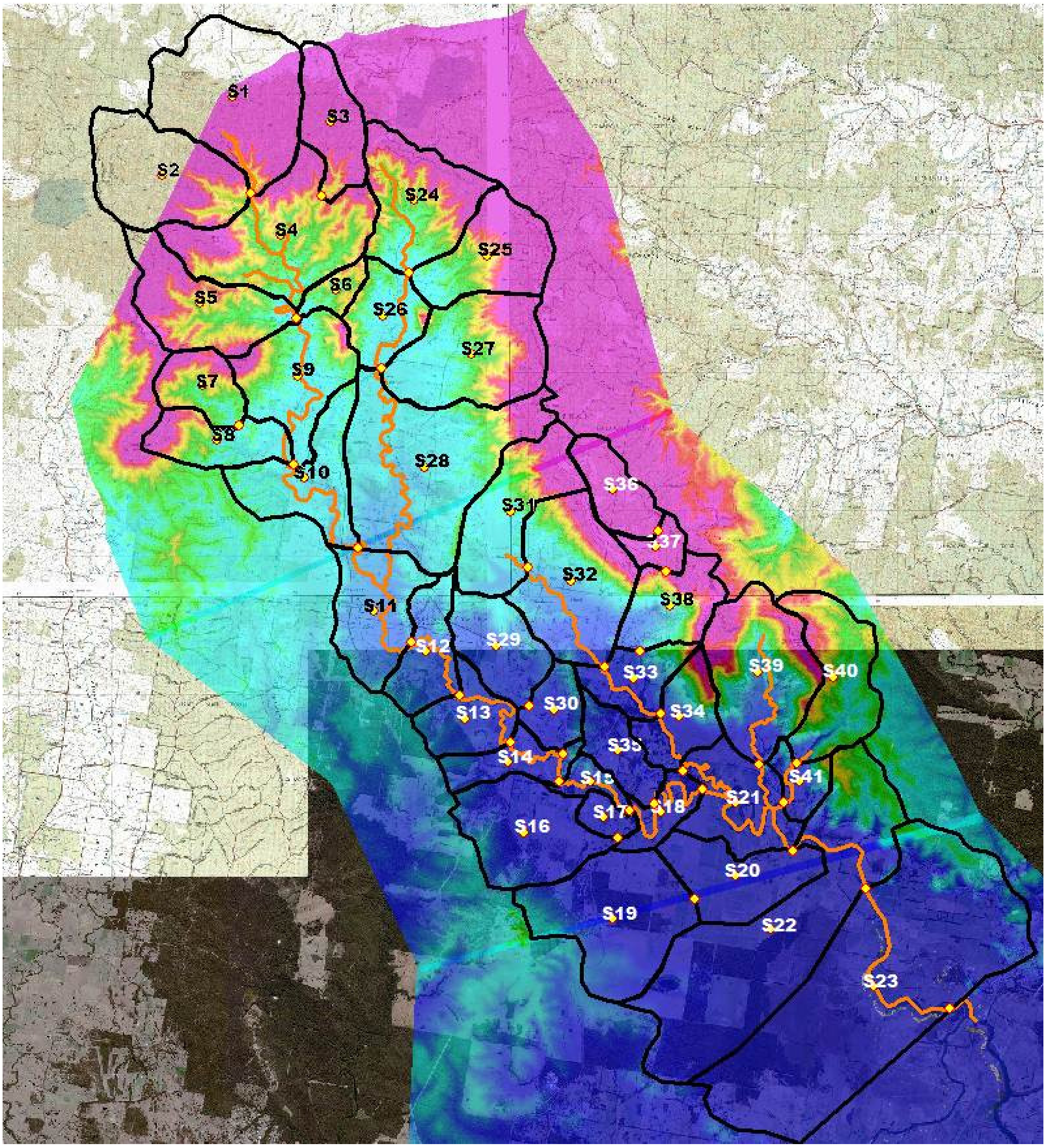


**Figure 4: Visualisation of the hydrologic model of the Lansdowne Catchment. The model's sub-catchments are shown as black outlines with their centroids, inflows and outflows marked in yellow. The primary stream channels are shown in orange. These are all overlaid on air photos, topographic maps and the ALS DTM (with terrain coloured from 0 to 200m AHD).**

## 5.2 Model Calibration and Verification

Calibration of the hydrologic model is associated with accurately simulating historic flood events. This involves inputting recorded rainfall data (in the form of hyetographs) and comparing the calculated hydrograph with a recorded hydrograph. Only a relatively small amount of historic data was available for the Lansdowne sub-Catchment which included level data from the Lansdowne River gauge since 1969 and generally one or two temporal rainfall gauges at varying times in the same period.

Calibration of the hydrologic model to historic flood evens was undertaken using three result parameters, being:

- Peak flow rate
- Flow volume
- Timing and “shape” of the hydrograph produced

This requires extensive rainfall data that accurately discretises the temporal and spatial distribution of rainfall in the catchment and river flow data showing the hydrograph produced at one or more locations within the catchment. Furthermore, this data must overlap and comprehensively cover the period of time in which the storm and flood occurred.

The Lansdowne River level gauge was analysed and a list of flood events was compiled. During some periods data was corrupt or the gauge had failed.

Next, a list of rainfall gauges operated by the BoM in the vicinity of the Lansdowne Catchment was compiled. It was found that only four rainfall gauges provided time-varying data (and this was not necessarily overlapping or continuous as some gauges had ceased operation or had only started operating recently).

Comparing the flood events recorded at the Lansdowne gauge with available temporal rainfall data, a list of historic calibration events was compiled. Whilst it is important to calibrate a hydrologic model to a variety of historic flood events with varying hydrograph peak flows, it was found that only several flood events had both river level data and at least one recorded hyetograph recorded near the Lansdowne sub-Catchment. These events, in chronological order, were:

- March 1974
- March 1978
- October 1982
- March 1995
- September 1999
- April 2008

Whilst data from these events contained some rainfall and river level data, this was not necessarily sufficient to calibrate and/or verify the hydrologic model. Rainfall data was most often limited to Comboyne South (*# 060080*) which represented rainfall a small distance to the north of the Lansdowne Catchment and / or Taree *(# 060030*) which represented rainfall a significant distance to the south-west of the Lansdowne Catchment. Of these events, the 1999 and 1995 flood events were selected to be used for calibration and verification of the model as these events were considered to have the best available data for both hydrologic and hydraulic purposes. Due to the historic importance of the1978 Manning Valley flood, this event was also modelled however available data was insufficient to be definitely used for calibration purposes.

### 5.2.1 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 show 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 $km^2$).

Furthermore, results for a larger data set of 54 catchments and 584 historic storms (Boyd and Bodhinayake, Australian Journal of Water Resources, 2006), show that the average value of 'C' for catchments in NSW was 1.74. Boyd et al recommend a value of 1.6 for ungauged catchments however their research showed that the value of 'C' could vary significantly for some catchments, varying between 0.89 and 2.79. This information was considered in latter stages of the calibration process because the Lansdowne Catchment *is* gauged and the true value of 'C' could therefore be determined by analysing corresponding rainfall and river level data. The recommended value of 'C', 1.6, was set as the initial value prior to historic calibration.

It should be noted that the 'C' value used in the previous flood study was 1.1 used across the whole Manning Catchment. This may not be the most accurate value for the Lansdowne Sub-Catchment because calibration was undertaken with the Manning River as a priority.

WBNM documentation and associated research recommends that the nonlinearity parameter be -0.23 (or m=0.77 which is defined as 1 minus the nonlinearity parameter). This is primarily based on studies of floods in natural catchments by Askew (1968, 1970) which found that the nonlinearity parameter value did not vary significantly 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 values can not only vary based on a catchment's properties but also within each catchment for a particular storm event (through the antecedent soil moisture conditions). In this way, typically a set of "average" rainfall losses are used for the final calibrated hydrologic model. Information on rainfall losses is provided in AR&R87 (Volume 1, Book II), with mean and median loss rate values provided for a number of catchments around Australia based on historic data. No specific data is available for the Manning or Lansdowne River Catchments so initial values 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 within a reasonable range of values obtained for other eastern NSW catchments.

### 5.2.2 Calibration and Verification using the 1995, 1999 and 1978 Historic Flood Events

Rainfall data for the 1995 and 1999 events was available at both the Comboyne South and Taree gauges in 6 minute pluviographs whilst the 1978 event was only available in hourly increments. A portion of the 1995 pluviograph recorded at the Taree gauge was not available and this was estimated based on the relationship between data recorded at the Comboyne gauge during other portions of the storm.

These pluviographs are shown in Figure 5 through Figure 7.

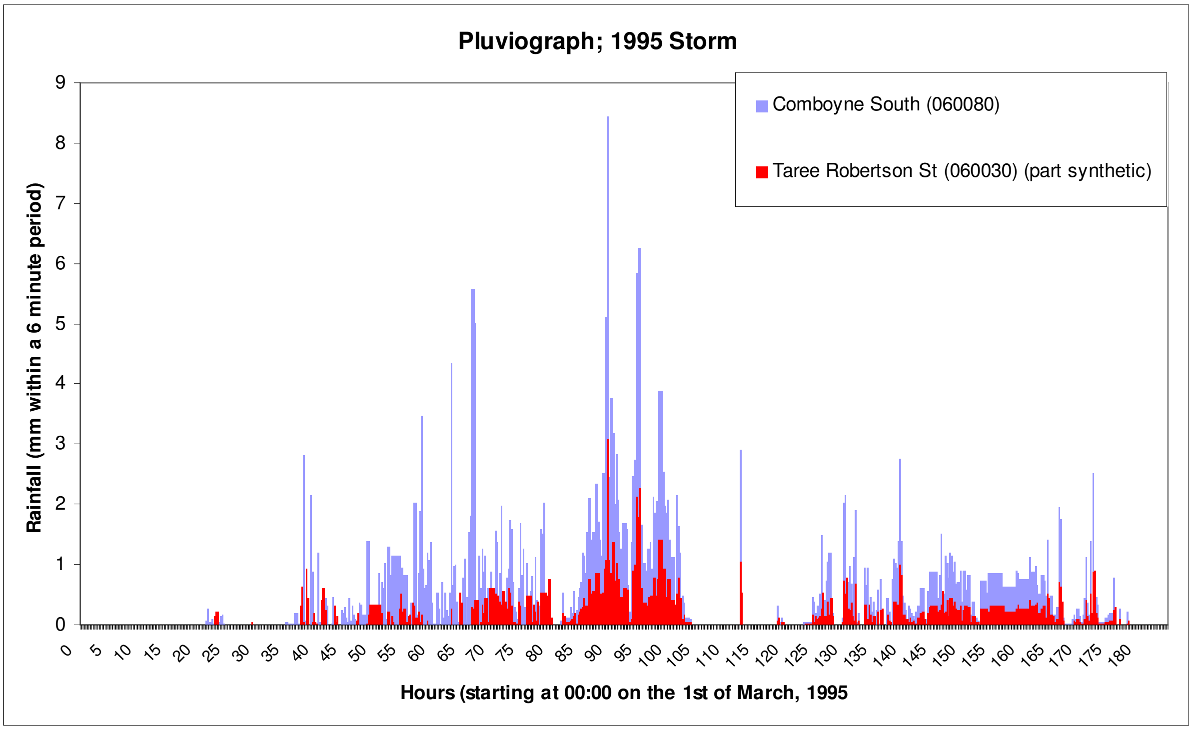


**Figure 5: Pluviograph for the 1995 event (Whilst the primary storm burst was adequately captured, portions of the data for the Taree gauge was unavailable. This data was extrapolated/estimated using the Comboyne records from approximately 110 hours onwards).**

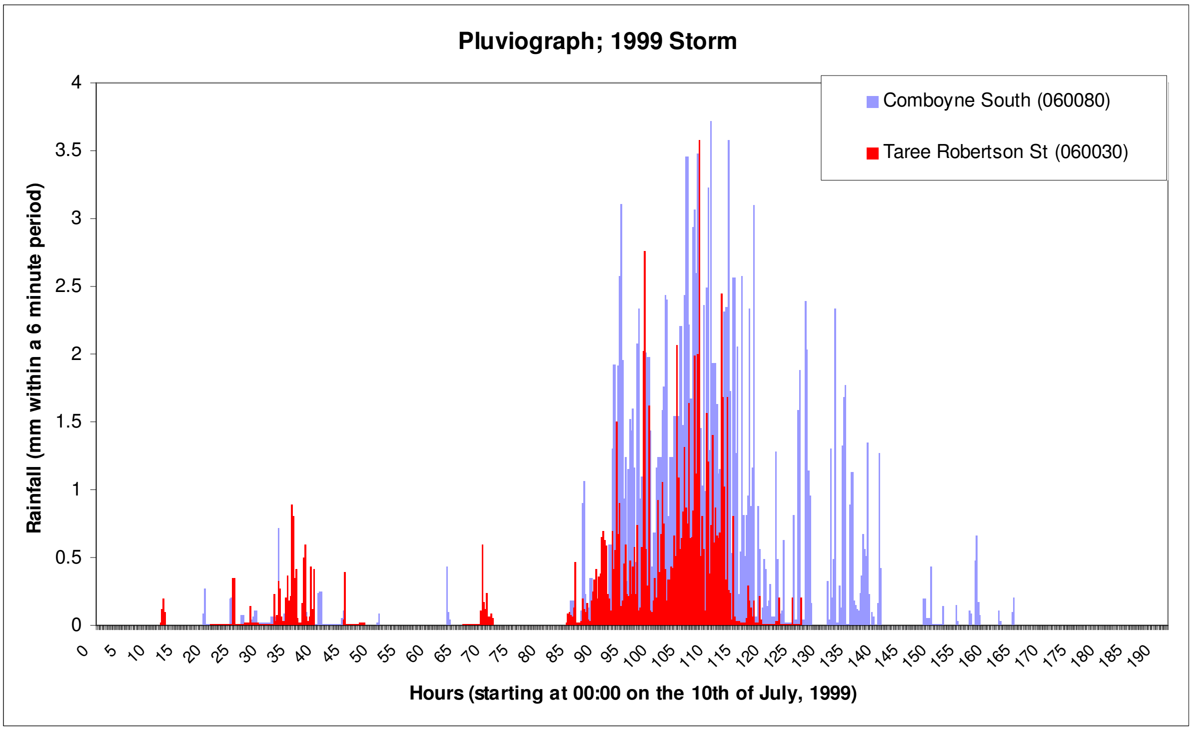


**Figure 6: Pluviograph for the 1999 event**

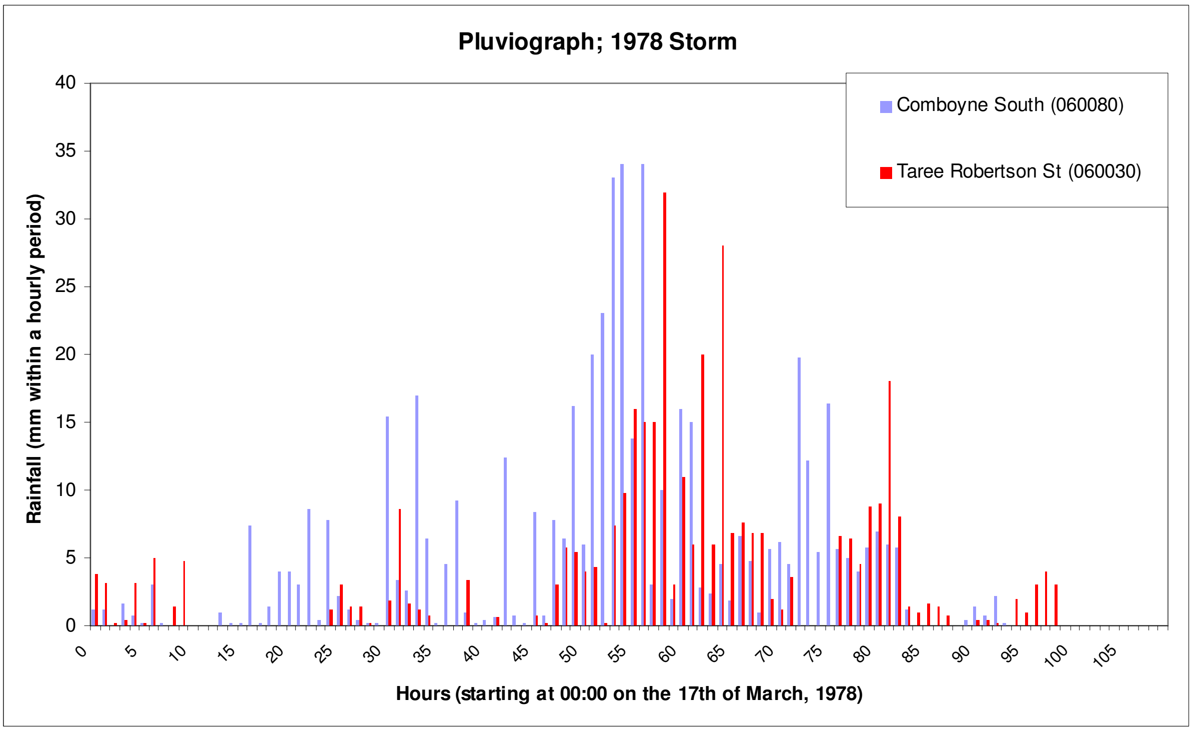


**Figure 7: Pluviograph for the 1978 event (data was only available in hourly increments)**

Rainfall data was input in the iWBNM model using the initial calibration parameters. Output hydrographs at S14 were processed using Excel© and compared to recorded hydrographs from the Lansdowne River gauge.

For all flood events, using initial calibration parameters, it was found that the recorded hydrograph significantly lagged the simulated hydrograph. Some lag could be expected a result of the distance between the Lansdowne Catchment and the Taree rainfall pluviograph and the typical east to west progression of storms in eastern NSW (that is, rainfall would encounter the Lansdowne Catchment sooner than Taree would, so by using a rainfall gauge located in Taree, the recorded hydrograph would be expected to lag the simulated hydrograph at Lansdowne). However this could not account for the entire lag in time seen.

From this it was clear that the lag parameter 'C' was not sufficient to accurately model the behaviour of the catchment and was further investigated. The parameter was varied and simulated results compared such that a single 'C' value provided a best-fit to the shape and timing of the hydrograph and peak flow value for the 1995, 1999 (and 1978 events). At the same time, rainfall losses were adjusted to calibrate to the peak flow rate and volume of each hydrograph.

In this way, a set of "average" parameters were calibrated that provided a reasonably good fit to all historic events (considering the limited data available).

It should be noted that the hydrograph available for the 1978 event appears to have been recorded using daily measurements and therefore this event provided the most limited results in terms of calibration.

Figure 8 to Figure 10 show the recorded and simulated hydrographs generated using the calibrated hydrologic model for the 1995, 1999 and 1978 historic flood events.

A summary of the differences between the recorded and simulated hydrographs is shown in Table 1.

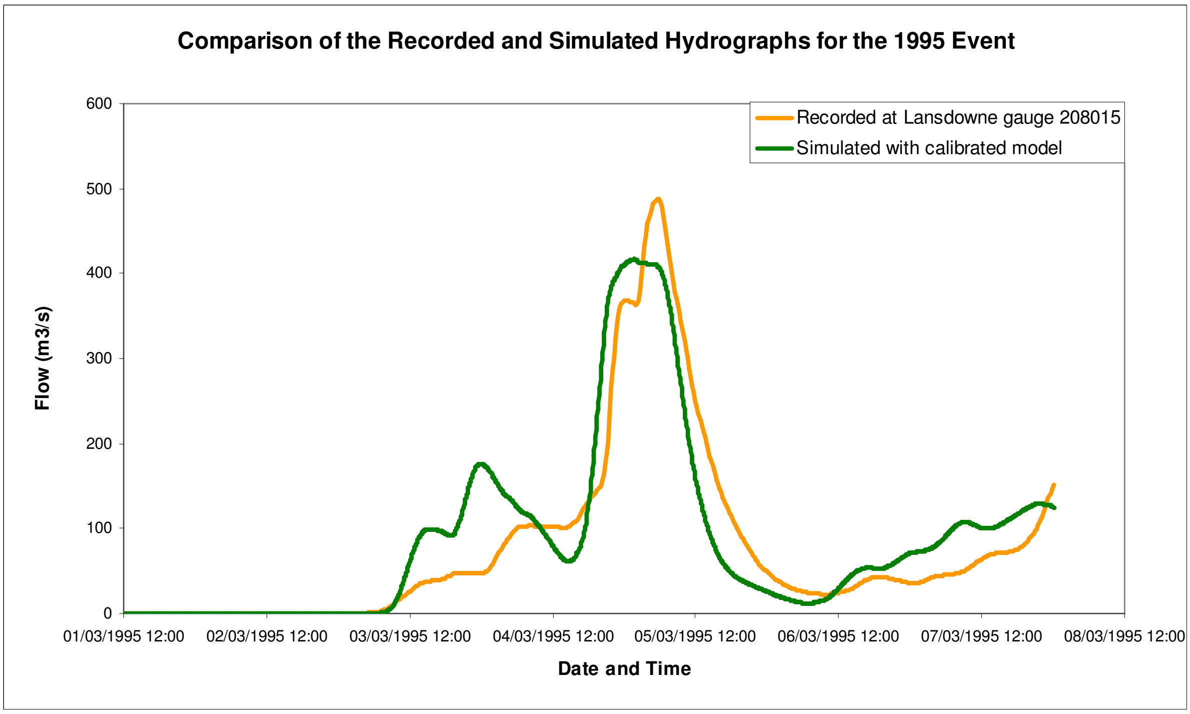


**Figure 8: Comparison of the recorded 1995 hydrograph at Lansdowne and the simulated hydrograph produced by the calibrated hydrologic model.**

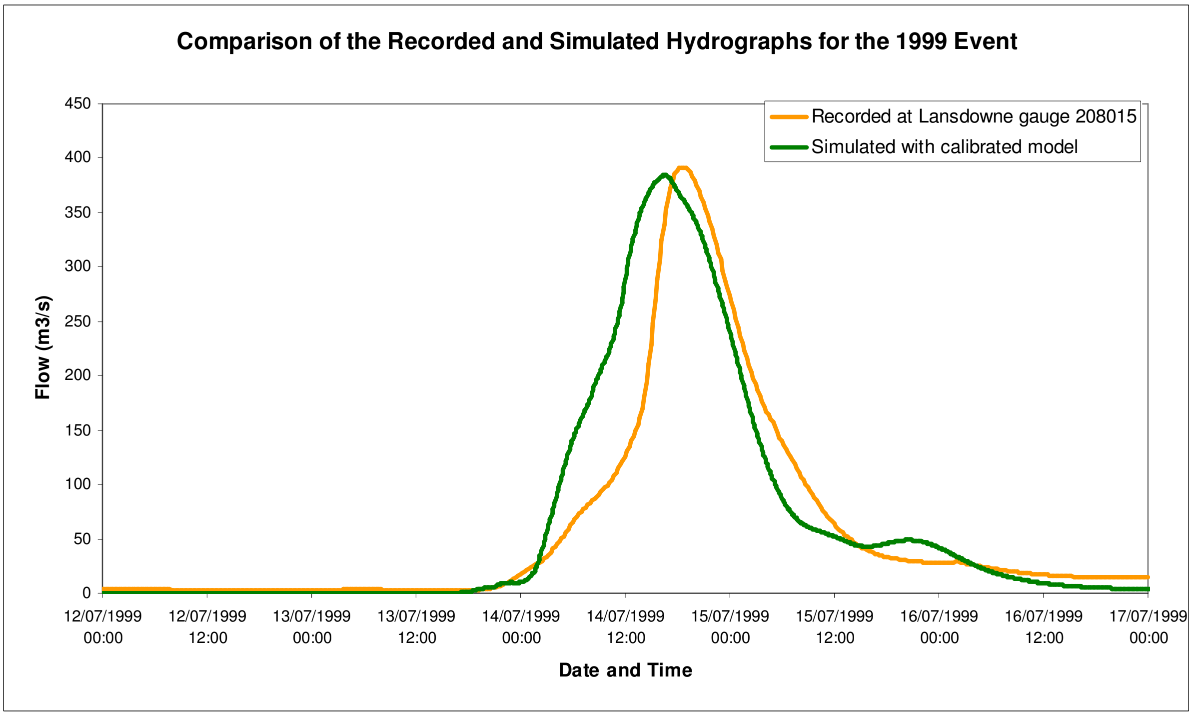


**Figure 9: Comparison of the recorded 1999 hydrograph at Lansdowne and the simulated hydrograph produced by the calibrated hydrologic model.**

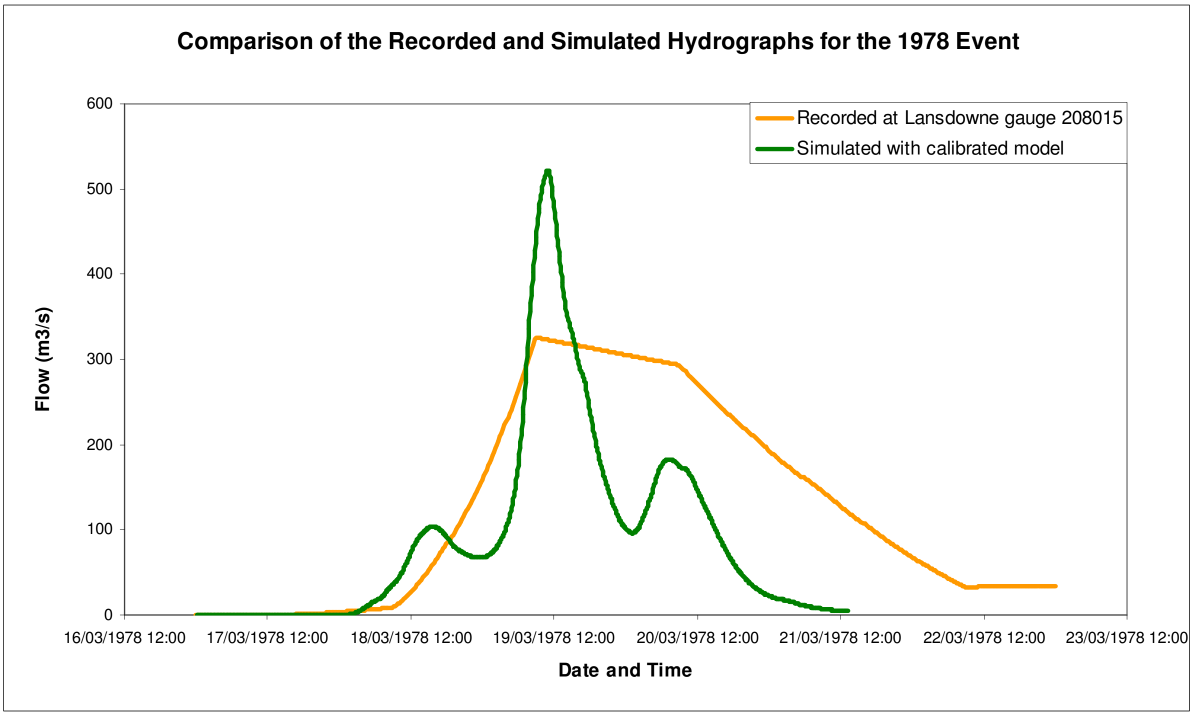


**Figure 10: Comparison of the recorded 1978 hydrograph at Lansdowne and the simulated hydrograph produced by the calibrated hydrologic model.**

**Table 1: Summary of the differences between the calibrated and simulated hydrograph data**

| Historic Event | % difference between Simulated and Recorded Peak Flow | % difference between Simulated and Recorded Volume | Time difference between Simulated and Recorded Peak Flow |
|---|---|---|---|
| 1995 | 14.5% | 0.2% | 4 hours |
| 1999 | 1.8% | 9.1% | 2 hours |
| 1978 | 60.3% | 45.0% | 1 hour |

Table 2 summarises the calibrated hydrologic model parameters.

**Table 2: Summary of Calibrated Model Parameters**

| *Parameter* | *Value* |
|---|---|
| *Lag Parameter, C* | *2.10* |
| *Nonlinearity Parameter, m* | *0.77* |
| *Initial Rainfall Loss* | *35 mm* |
| *Continuing Rainfall Loss* | *1.5 mm/hour* |

It should be noted that the final lag parameter 'C' value selected is much greater than the typical value of 1.6 to 1.74 however it is still within the range of expected values for NSW. Boyd et al, a co-developer of WBNM, studied the variation of the lag parameter extensively through south and eastern NSW (WBNM runoff routing parameters for south and eastern Australia; Boyd, Bodhinayake; Australian Journal of Water Resources, Vol 10 No 1; 2005) and it was found that this parameter is similar to that obtained for some other catchments in the region.

## 5.3 Design Storm Simulations

The calibrated and verified hydrologic model was used to simulate the PMF, 0.5% 1%, 2% and 5% AEP Design Storms on the Lansdowne sub-Catchment. The model was setup to simulate these storms with varying durations from 5 to 4320 minutes and hydrographs for the Lansdowne River and its tributaries were exported for the most critical durations.

### 5.3.1 Results

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

**Table 3: Critical Storm Durations for the Lansdowne sub-Catchment**

| *Design Storm* | *Critical Duration (mins)* |
|---|---|
| *5%* | *2160* |
| *2%* | *2160* |
| *1%* | *2160* |
| *0.50%* | *2160* |
| *PMF* | *270* |

Hydrographs for these critical durations are shown in Figure 11 through Figure 15.

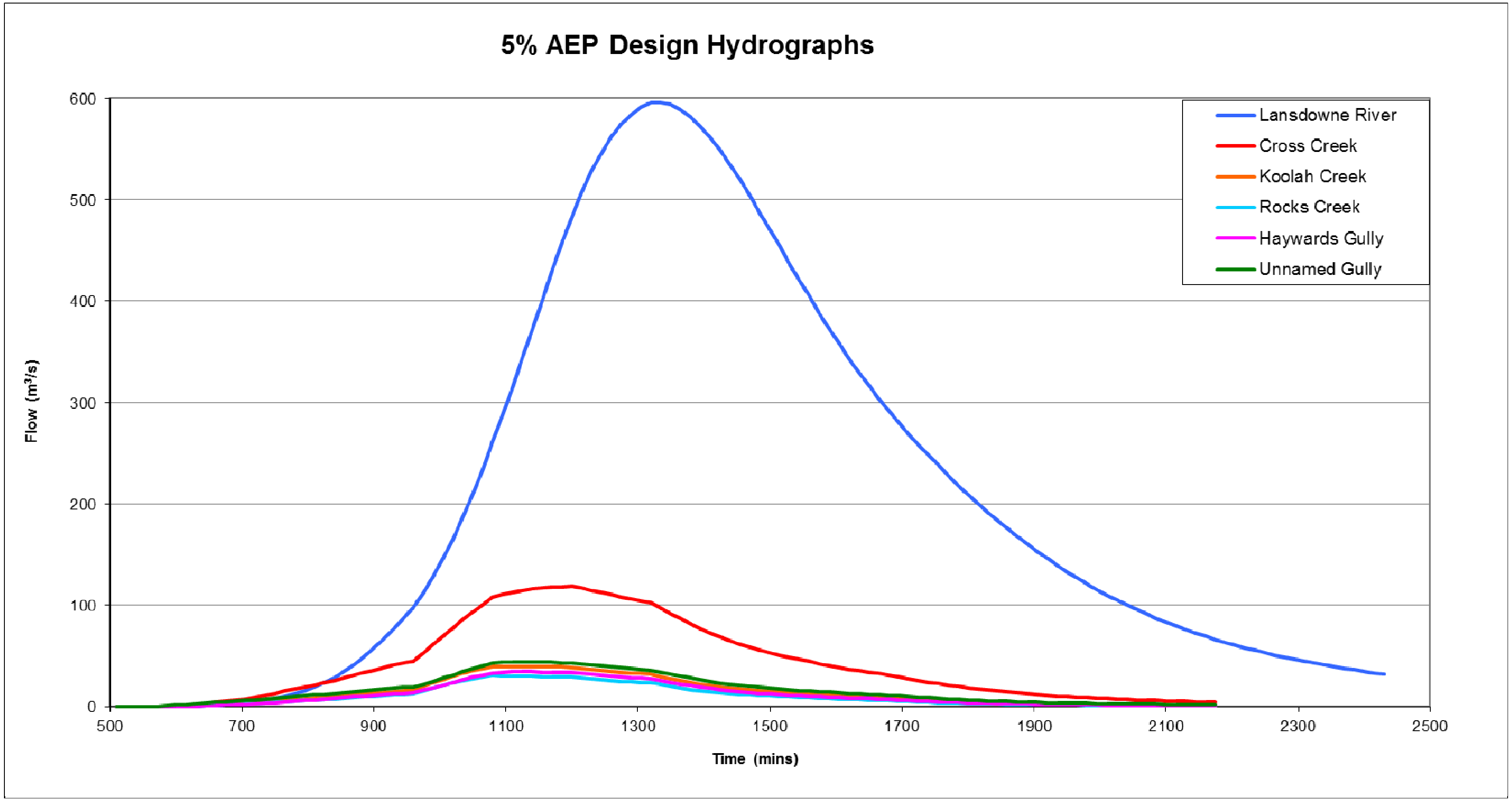


**Figure 11:5% AEP design hydrographs generated from the calibrated hydrologic model at the stream input locations for the hydraulic model**

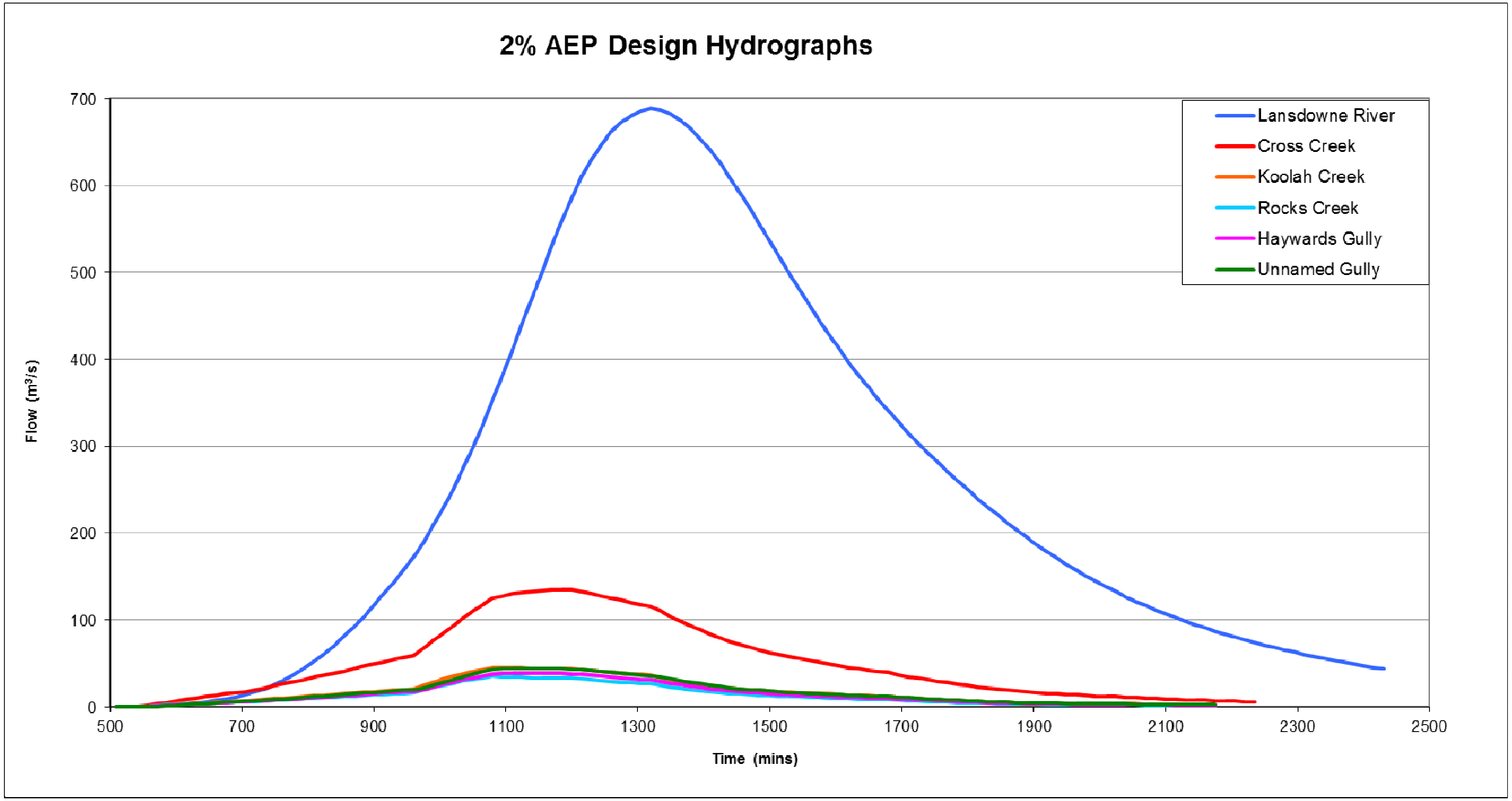


**Figure 12: 2% AEP design hydrographs generated from the calibrated hydrologic model at the stream input locations for the hydraulic model**

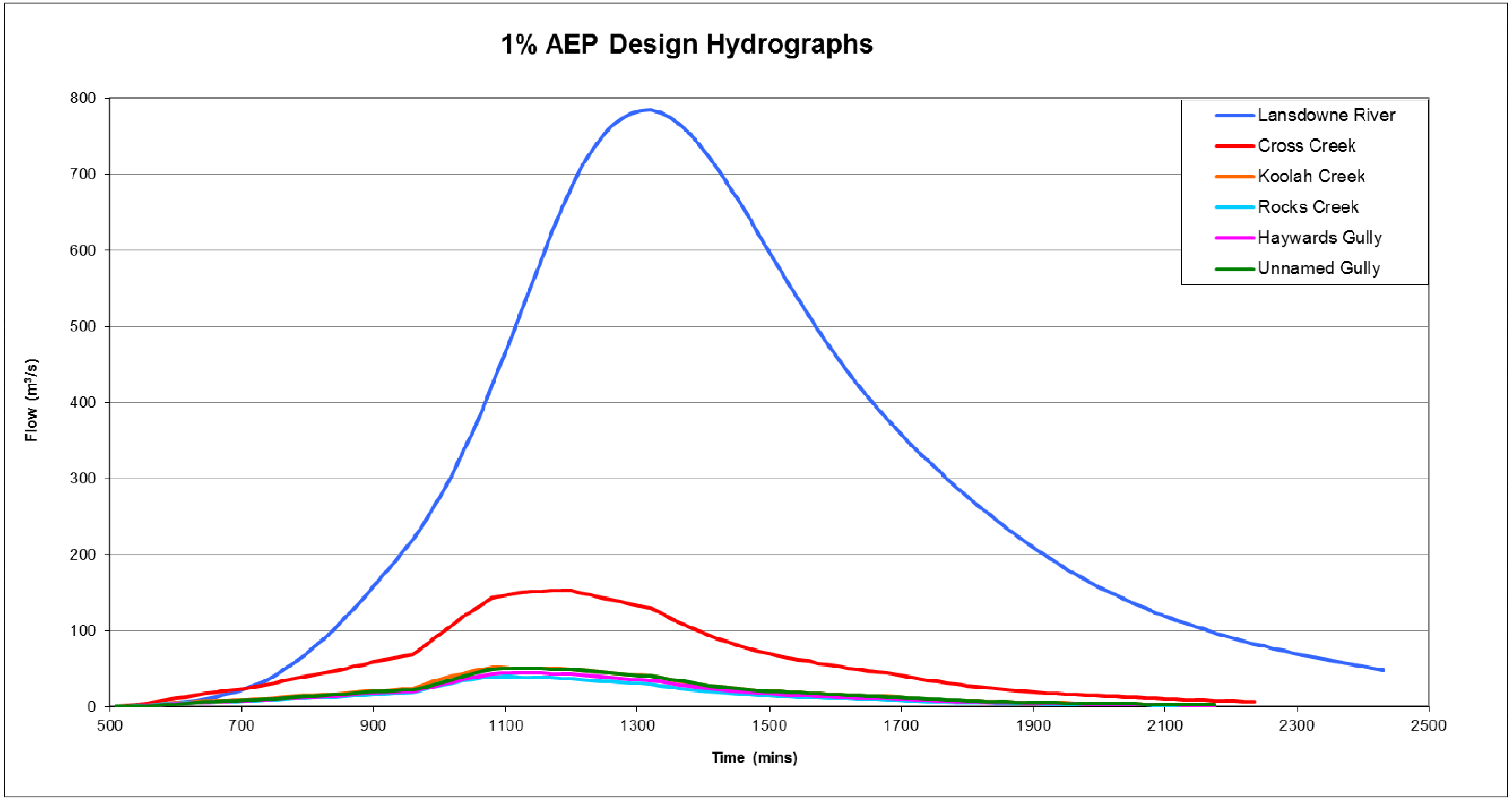


**Figure 13: 1% AEP design hydrographs generated from the calibrated hydrologic model at the stream input locations for the hydraulic model**

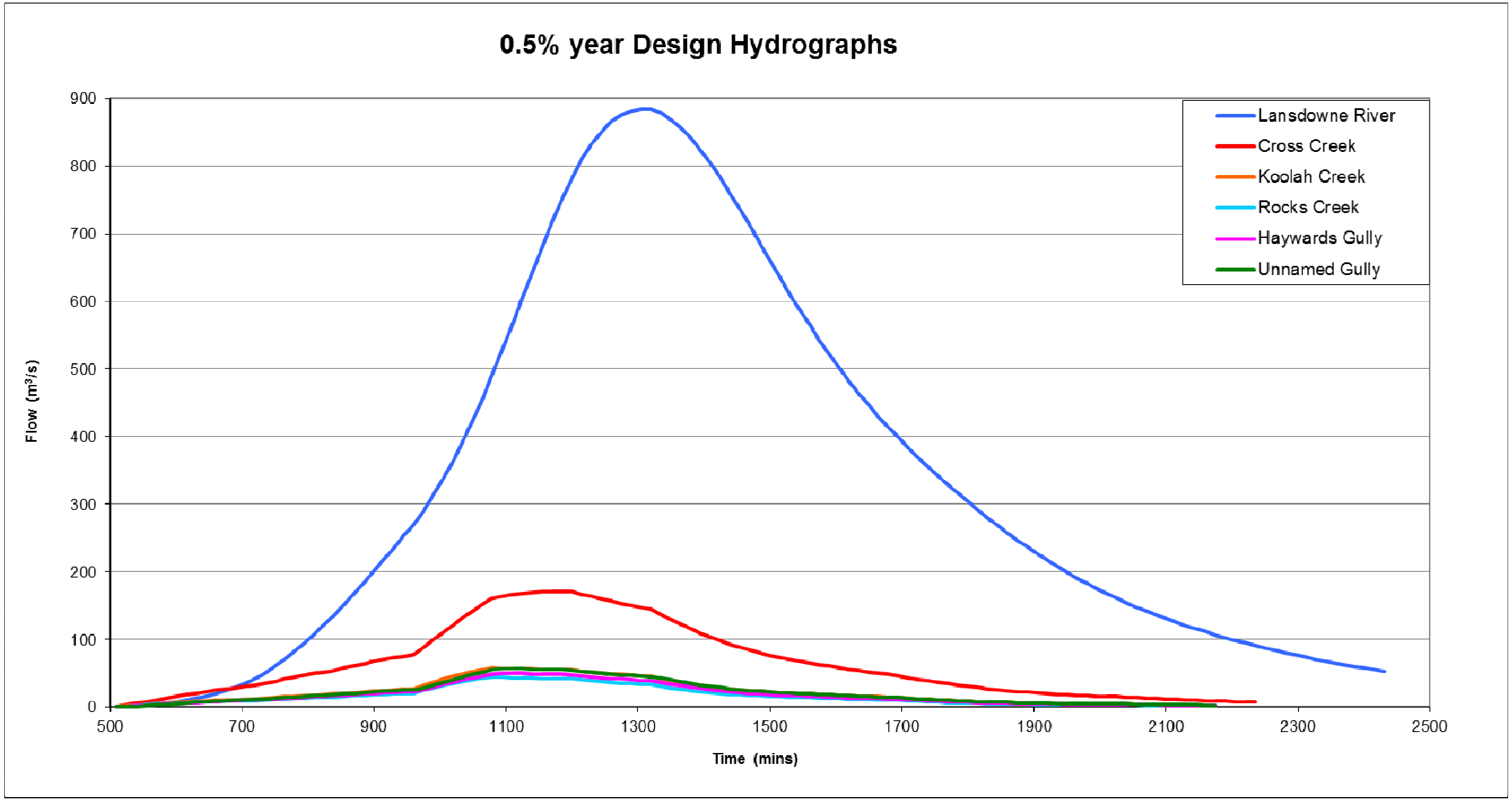


**Figure 14: 0.5% AEP design hydrographs generated from the calibrated hydrologic model at the stream input locations for the hydraulic model**

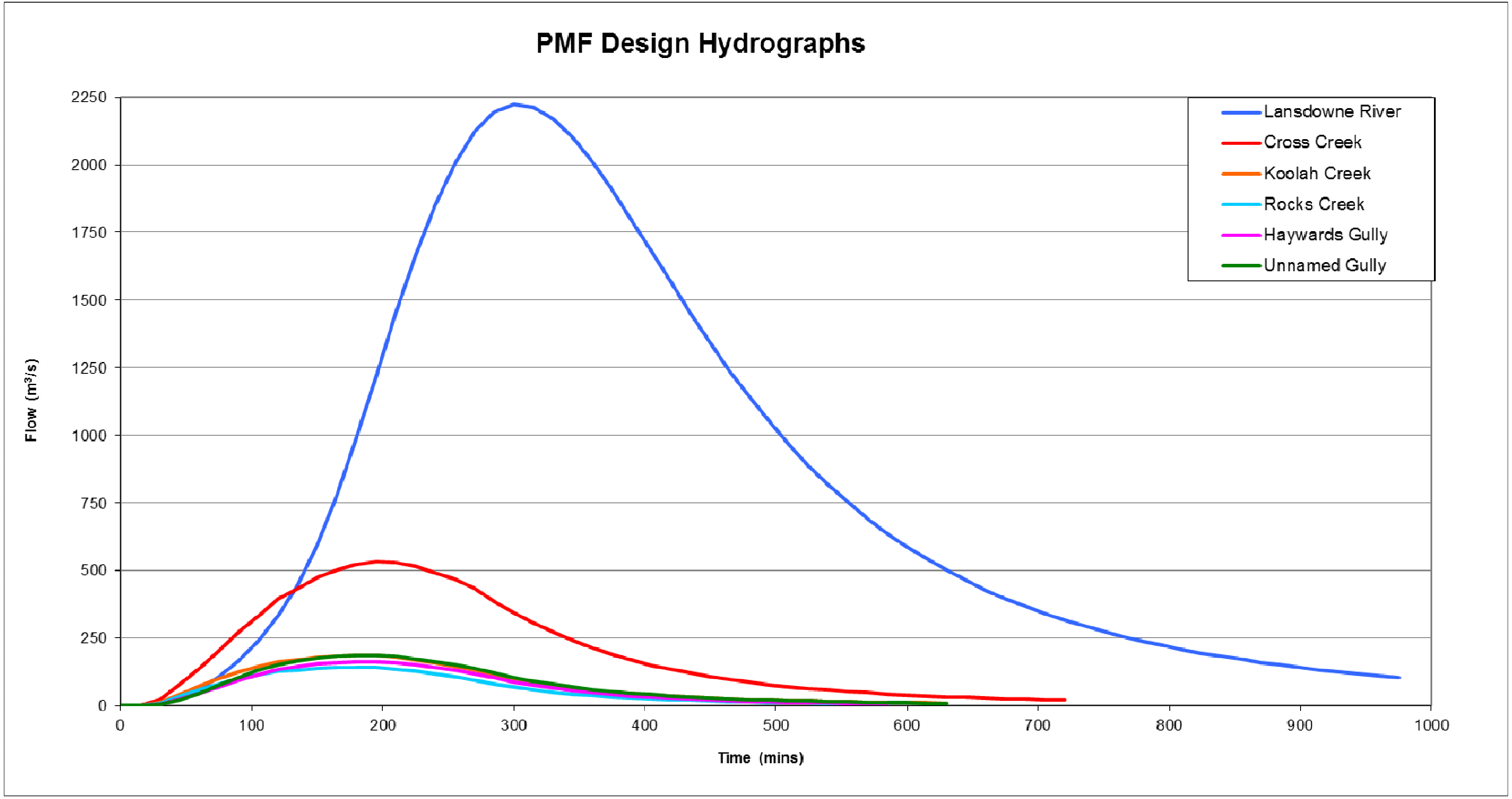


**Figure 15: PMF design hydrographs generated from the calibrated hydrologic model at the stream input locations for the hydraulic model**

# 6. HYDRAULICS

The hydraulic component requires the results of the hydrologic model in combination with the topography, bathymetry and material properties to produce what is the primary outcome of the flood study; a qualitative and quantitative description of the flow in the study area. These results are the basis of the subsequent floodplain risk management process.

The hydraulic analysis was performed using a hydraulic model to simulate the behaviour of flooding using the finite element program RMA-2. The study area is discretised into ‘elements’ with an aim of minimising analysis run-time (through the number of elements) whilst still correctly modelling the 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 which were derived in the hydrologic component of the flood study. Similarly, the outflow of the model is regulated based on a defined downstream relationship (detailed in subsequent sections). At all other boundaries, symmetry is maintained, however the aim is to maintain “dry” boundaries so that the wetted extents are a true representation of reality.

## 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. The previous model was focused on the Manning Valley and the Manning River as its primary flow path. The Lansdowne River, compared to other Manning River tributaries is relatively small and therefore was not a significant portion of the previous study.

The previous study extended upstream along the Lansdowne River to the village of Lansdowne. A total of four nodes were used to represent the reach of the Lansdowne River upstream of the Pacific Highway and in all design flood simulations, the water surface profile was essentially flat (varying by no more than 2 cm). The study estimated that a 1% AEP flood level in Lansdowne would have a level of 3.00m AHD. This is viewed as being inconsistent with historic data recorded at the Lansdowne River gauge. Since 1969, levels have exceeded 3.0m AHD almost every year. This highlights the need for a detailed flood study of the Lansdowne sub-catchment which includes all major tributaries and a focus on flows derived from the sub-catchment.

Despite the limitations of the previous study in the region of Lansdowne, the results for the Manning River are considered to be fair considering the number of elements used in the total model and the results compared to historic records.

In lieu of further data, levels at the confluence of the Lansdowne and Manning Rivers were considered useful as a downstream boundary condition to control outflow on the upgraded Lansdowne Flood Study.

## 6.2 Methodology

Although the flood study centres on the village of Lansdowne and its immediate surroundings (refer Figure 1), a hydraulic model typically requires a much larger area of study. This ensures that the boundary conditions do not profoundly affect the results within the study area, ensuring that they are more reliable and accurate.

The modelled area extended from approximately 5.5 km north-west of Lansdowne village, downstream along the Lansdowne River to the confluence of the Manning River. The modelled area extended across both floodplains of the Lansdowne River to an elevation of generally up to 20m AHD to ensure that the wetted extents were not truncated. The model incorporated a significant portion of Cross, Koolah and Rock Creek as well as Haywards Gully and an Unnamed Gully near Brimbin Hill.

The resulting study area was discretised using approximately 14,800 elements comprising 34,600 nodes (including mid-side nodes, refer Figure 16). 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 air photos, Council's ALS data, hydrosurvey data from DECC and site surveys undertaken by WorleyParsons.

Outside the tidal reach of the Lansdowne River (and the majority of its tributaries) where hydrosurvey data was not available, estimates were used based on site observations and air photos.

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.

The downstream boundary condition was modelled as a dual stage hydrograph (a dual elevation boundary condition was used due to the size of the floodplain at the confluence of the Lansdowne and Manning Rivers). During the calibration of the model, this boundary condition was based on historic data whilst the design simulations utilised a combination of results from the previous flood study and historic data (described in more detail in subsequent sections).

Once the model was discretised and general parameters set, it was calibrated and verified using recorded historic flood events.

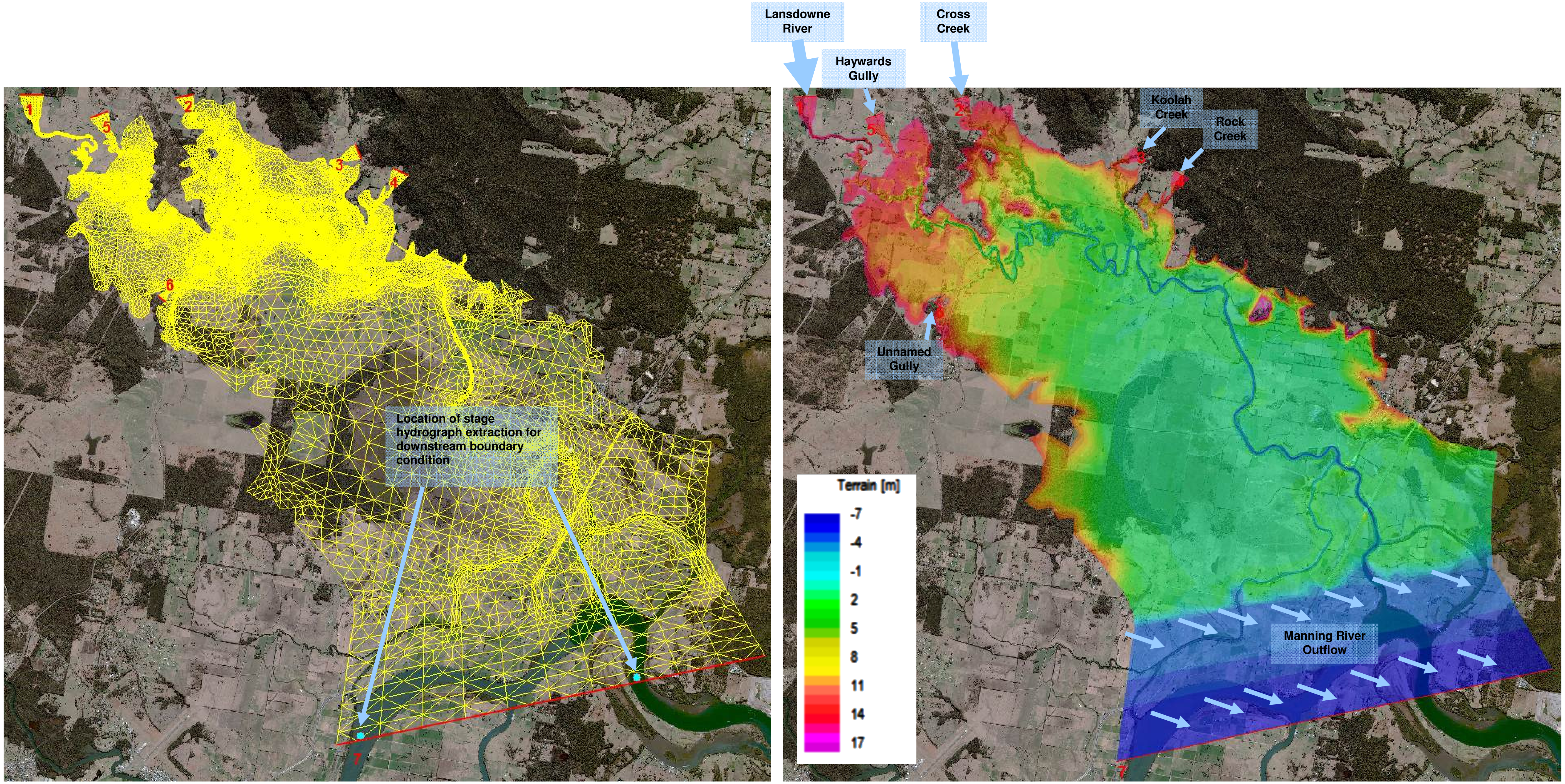


**Figure 16: Hydraulic Model showing the element mesh network (left); showing the location of boundary inflows and outflow with the topography (inm AHD) indicated by element shading (right)**

## 6.3 Model Calibration and Verification

There are three primary calibration variables in hydraulic modelling, being the:

1. Manning roughness ("n") of each element which is based on its surface characteristics / material properties and its interaction with fluid flow,

2. boundary conditions such that the correct inflow and outflow are entering and leaving the model domain (or that the correct stage is associated with outflow), and

3. mesh element network such that it adequately discretises flow paths, topography and does not introduce modelling errors through element size variations or aspect ratios.

Whilst the first variable above is typically the primary calibration parameter used, all three were used to ensure the model was accurately calibrated.

This process requires both upstream and downstream boundary conditions to control inflow and outflow from the model based on historic flood events whilst comparing known flood levels at locations within the model. This usually involves recorded inflow hydrograph(s) and an outflow stage hydrograph with several stage hydrographs (or peak readings) through the area of the study. The calibration parameters can then be adjusted using an iterative process to ensure adherence to a series of historic flood events.

After an extensive review of the available data (refer Sections 1.3, 2, 3 and 4) it was found that only a relatively small pool of historic data exists in the Lansdowne sub-Catchment. This typically consisted of only a single peak water level or stage hydrograph recorded at the Lansdowne gauge. For some historic events, one or two other peak levels were sometimes available within the sub-Catchment although the exact location and time that these were recorded was often unclear. For the vast majority of historic flood events, no data existed in close vicinity to Lansdowne village on or close to the Lansdowne River other than what was recorded at the gauge. For some flood events, such as the 1978 Manning Event, peak flood levels had been recorded at several locations within proximity to Lansdowne village. However incomplete or inaccurate records in both the inflow and outflow hydrographs meant that this data was of little use. Furthermore, there were additional ambiguities arising from the fact that a significant flood occurred in the Lansdowne sub-Catchment the week after the Manning Valley flood in 1978. These factors meant that calibration using the 1978 flood event(s) was not of any value, however due to the historic importance of the1978 Manning Valley flood, this event was modelled for reference.

Based on the analysis of historic data, the only reliable record during a flood event was recorded at the Lansdowne gauge, which was also the source of the model's inflow hydrograph on the Lansdowne River. This inherent problem led to the following solution:

a) The hydraulic model was extended upstream along the Lansdowne River (to its current location upstream of Haywards Gully),

b) The calibrated hydrologic model was used to generate historic hydrograph inputs at this location and was compared with simulated hydrographs at the gauge to obtain a "reduction ratio".

c) This ratio, which describes how the simulated hydrograph reduced (by moving the inflow location upstream), was applied to the historic recorded hydrograph at Lansdowne gauge and these hydrographs were then input into the model at the new upstream boundary location of the Lansdowne River.

d) Inflow into the model at all other inflow locations was derived from the calibrated hydrologic model. This included the inflow at Haywards Gully, which contributes flow to the Lansdowne River upstream of the gully (the volume was approximately equal to the loss associated with moving the inflow hydrograph upstream).

e) The peak level recorded at the gauge was then used as a calibration point for the model.

Whilst this was far from ideal, it was required in order to at least partially calibrate the model whilst further historic data was sourced. For the 1978 flood event, the hydrologic model was used to generate the inflow hydrograph on the Lansdowne River due to the limitations in the recorded data.

At least two or more gauges recorded water levels on the Manning River in relatively close proximity to the confluence of the Lansdowne River for the majority of historic flood events. This allowed the development and calibration of the model's downstream boundary condition.

Based on the aforementioned data availability, the initial calibration of the hydraulic model involved the:

- allocation of a downstream dual elevation boundary condition based on two recorded stage hydrographs on the Manning River for a series of historic flood events (Figure 16)
- allocation of inflow hydrographs in the hydraulic model based on the fully calibrated hydrologic model and recorded data for a series of historic flood events
- comparison of peak recorded levels at Lansdowne gauge with historic simulated flood levels.

Due to the heavy reliance of this process on the calibrated hydrologic model, it was important that the same historic events were used for calibration and verification of the hydraulic model (and these events also typically had the best records available). Therefore the 1999 and 1995 flood events were selected to be used for calibration and verification of the model.

After an initial calibration of the hydraulic model to the 1999 and 1995 historic flood events based on available data, further information was sought from the community.

The Community Consultation Program was accelerated and a survey was issued to residents of Lansdowne requesting historic flood recollections and records. Some general data was obtained from this process and a Community Workshop was undertaken on the 9$^{th}$ of December, 2010.

This provided residents of Lansdowne with an opportunity to critique and contribute to the model's simulation of the historic 1999, 1995 (and 1978) flood events (at the same time, the 100 year and PMF flood events were also simulated based on initial calibration parameters for the Community to comment on; more on this is discussed in subsequent sections).

The Community Workshop yielded important additional information for the final calibration of the hydraulic model with respect to the 1999 and 1995 historic flood events. One particularly important peak water level was obtained and confirmed by several residents at a location south of the railway line adjacent to the Lansdowne River (on the property at 1344 Cundle Road, Lansdowne). At this property, two locations were surveyed where depths had been measured during the 1999 and 1995 flood events. This was undertaken by WorleyParsons during the Community Workshop with the assistance of local residents and converted to a peak water surface level for each event using Council's ALS data.

This additional water level for both the 1995 and 1999 flood events allowed final calibration and verification of the hydraulic model.

### 6.3.1 Calibration and Verification Summary using the 1995, 1999 (and 1978) Historic Flood Events

Inflow data from the calibrated hydrologic model was used to generate hydrographs at the upstream boundary locations of the model. For the Lansdowne River, hydrographs were based on those recorded at the Lansdowne Gauge (with the exception of the 1978 event) whilst all others inflow tributaries were extracted directly from the calibrated hydrologic model. Initially only inflow hydrographs for the Lansdowne River, Cross, Koolah and Rocks Creek were utilised but during the calibration process the need to account for further inflow into the model was required. This process led to the additional inflow hydrographs of Haywards Gully and the Unnamed Gully near Brimbin Hill.

The final input hydrographs used in the hydraulic model are shown in Figure 17 through Figure 19 for the 1999, 1995 and 1978 historic events.

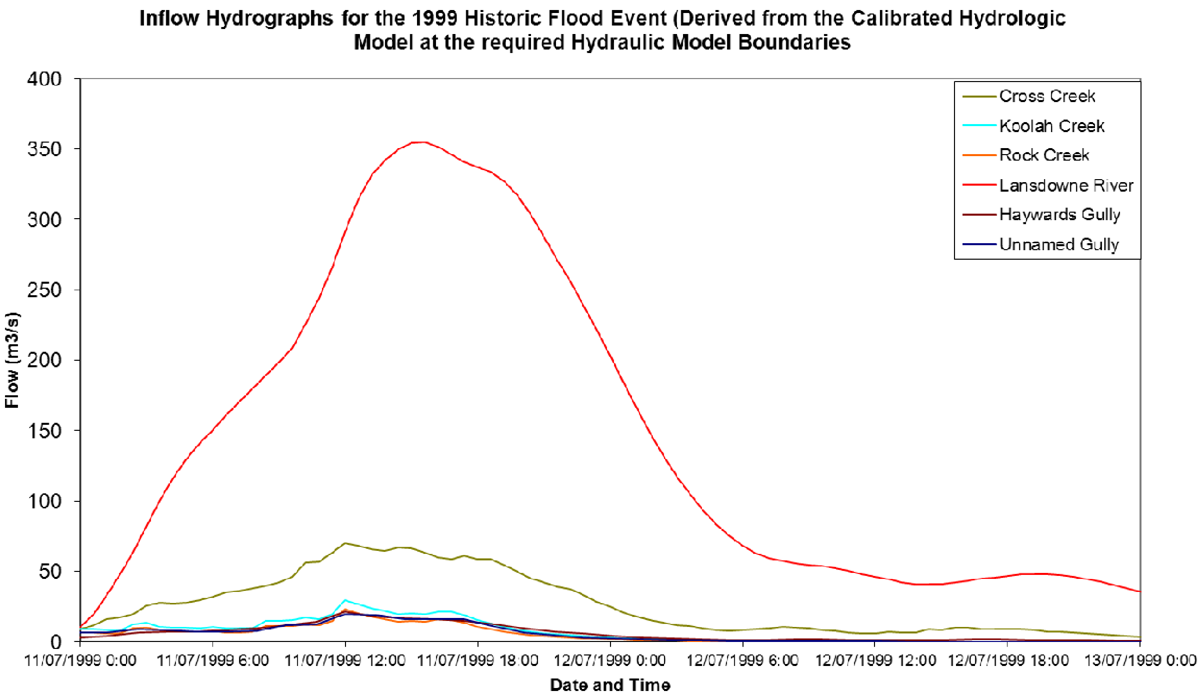


**Figure 17: Inflow hydrographs used for the 1999 historic flood event.**

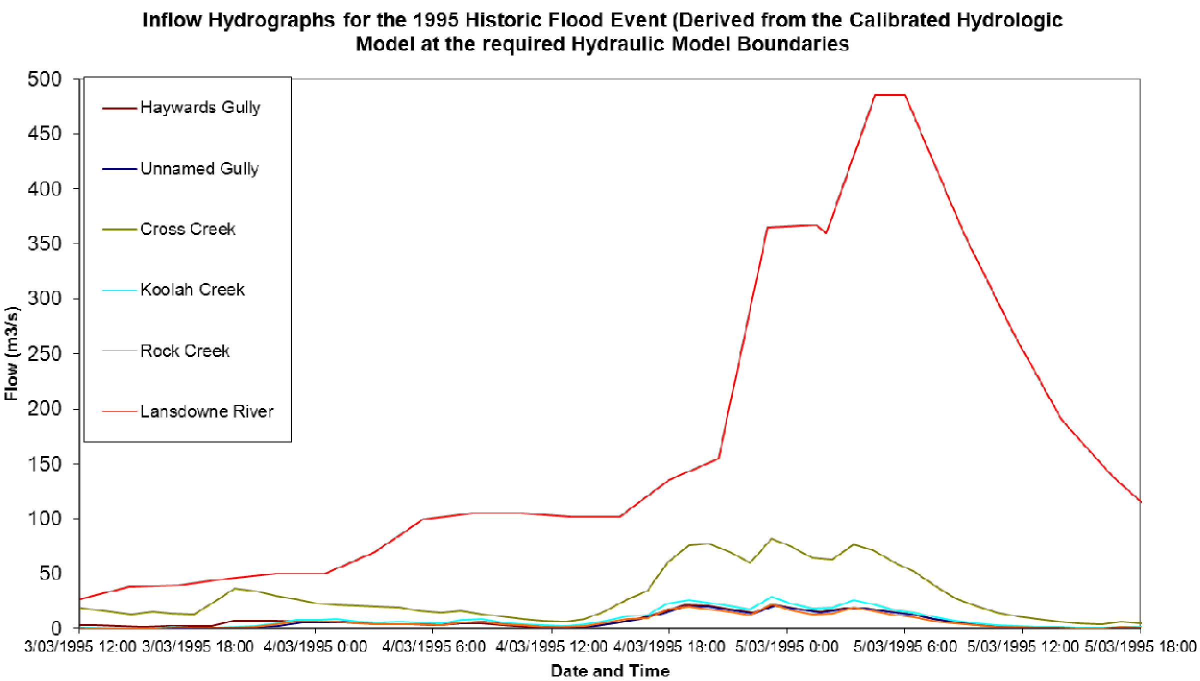


**Figure 18: Inflow hydrographs used for the 1995 historic flood event.**

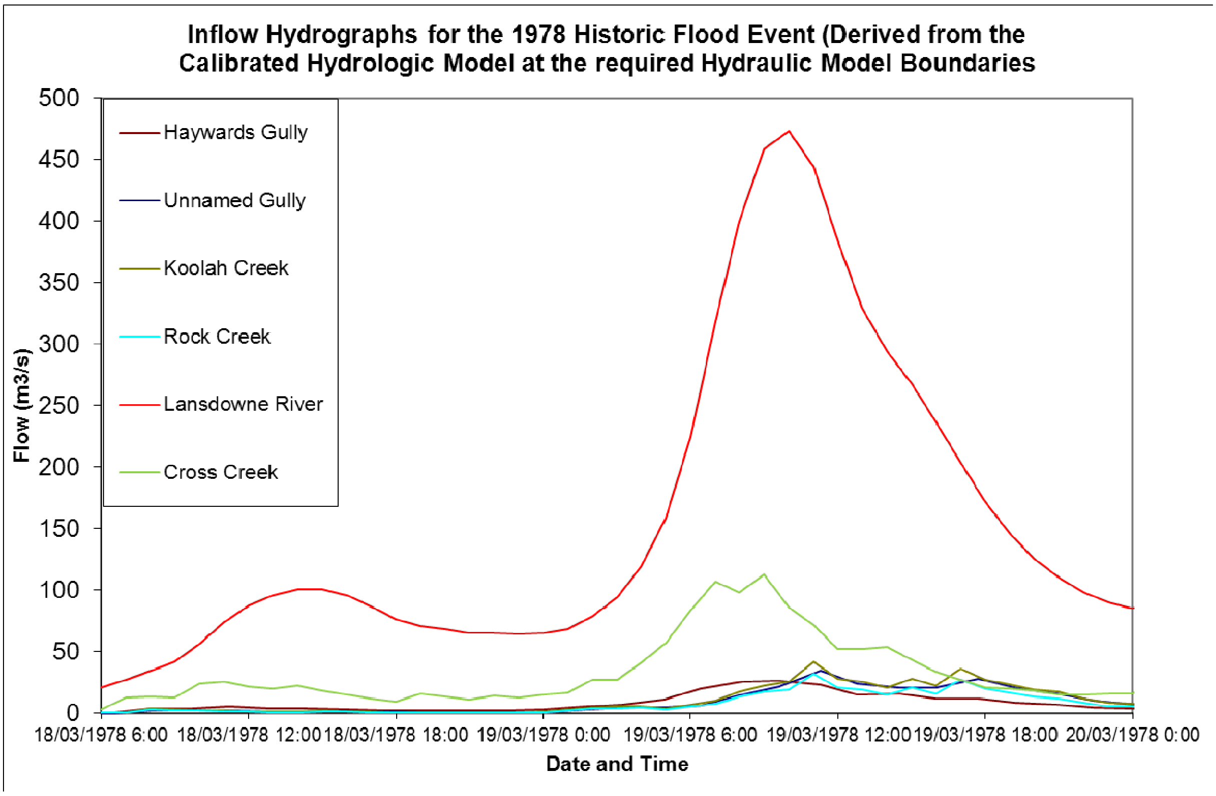


**Figure 19: Inflow hydrographs used for the 1978 historic flood event.**

The outflow dual elevation boundary condition was based on historic data recorded during the flood events at the Taree and Croki gauges (operated by Manly Hydraulic Laboratories). Data from these gauges was obtained and used to calculate an average water surface slope across the model's downstream boundary condition. This was done through the full stage hydrograph at both locations with levels at each point in time interpolated (and extrapolated) to define levels across the boundary.

It should be noted that during the 1978 Manning Valley flood event, the Croki gauge failed, with only a portion of the rising limb recorded. The downstream boundary condition was therefore only based on data recorded at Taree and estimates based on other recorded events. This is likely to a source of significant error in modelling the 1978 event as the downstream boundary was seen to control a large portion of the Lansdowne sub-Catchment through backwater flows.

The stage dual elevation hydrographs for the 1999, 1995 and 1978 events are shown in Figure 20, Figure 21 and Figure 22.

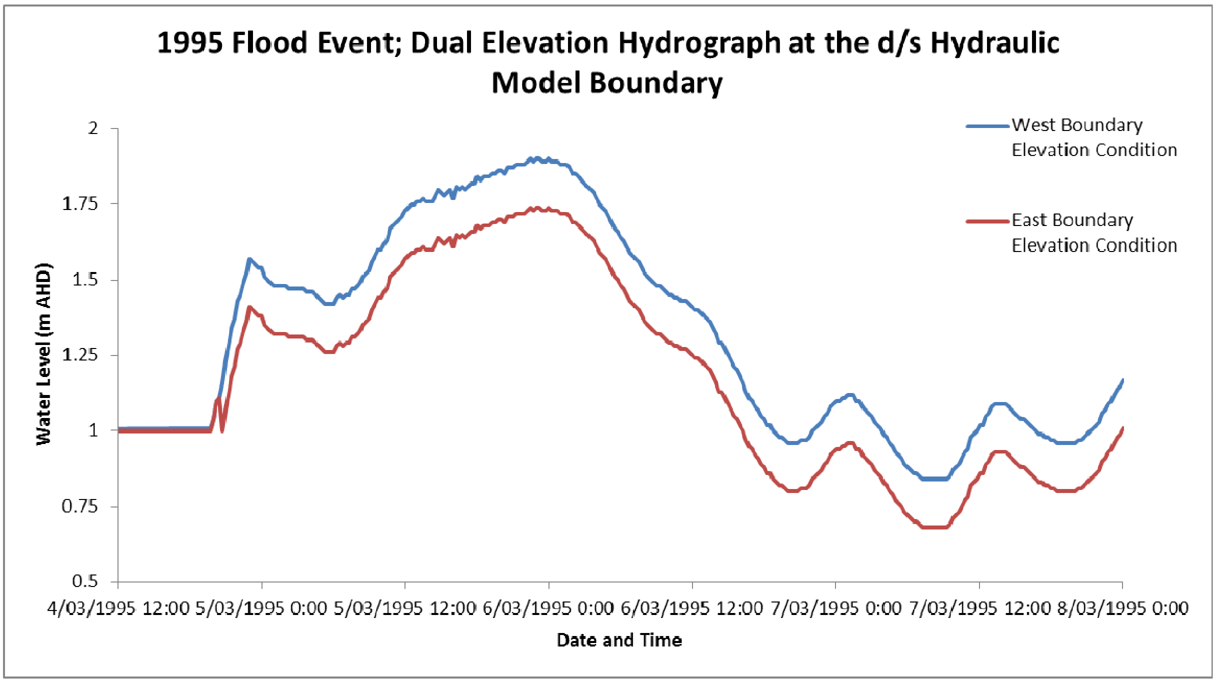


**Figure 20: Dual stage hydrographs defined on the western and eastern boundary of the model (along the Manning River) used for the 1995 historic flood event.**

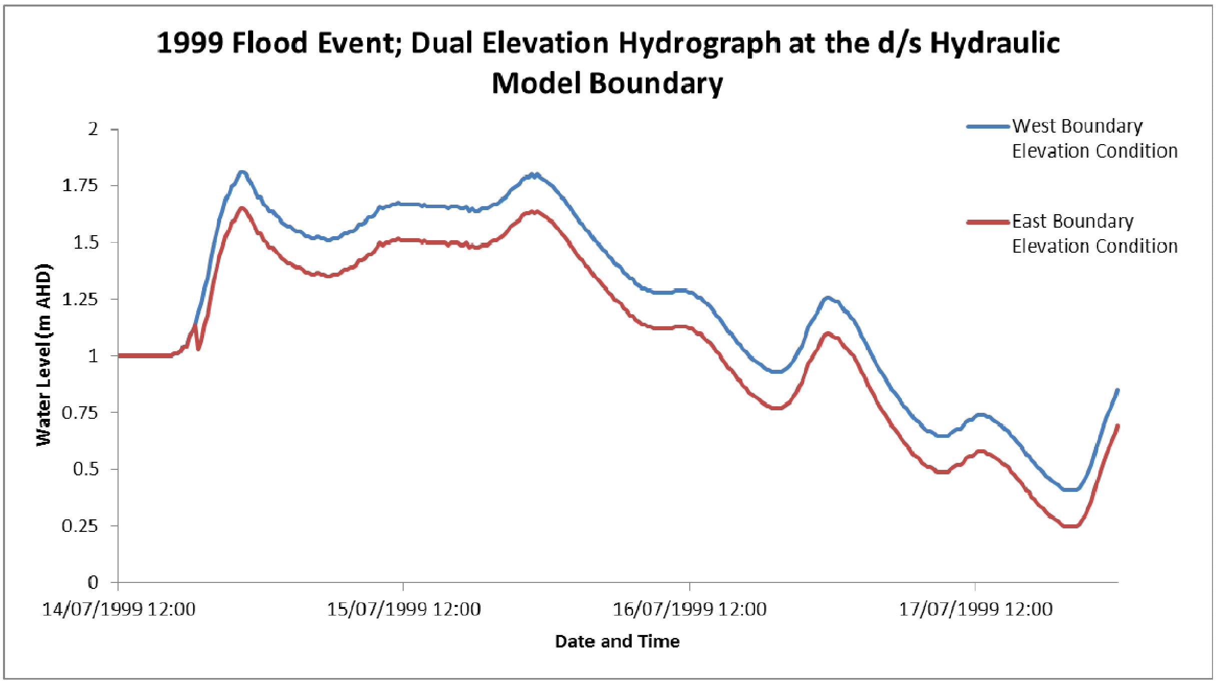


**Figure 21: Dual stage hydrographs defined on the western and eastern boundary of the model (along the Manning River) used for the 1999 historic flood event.**

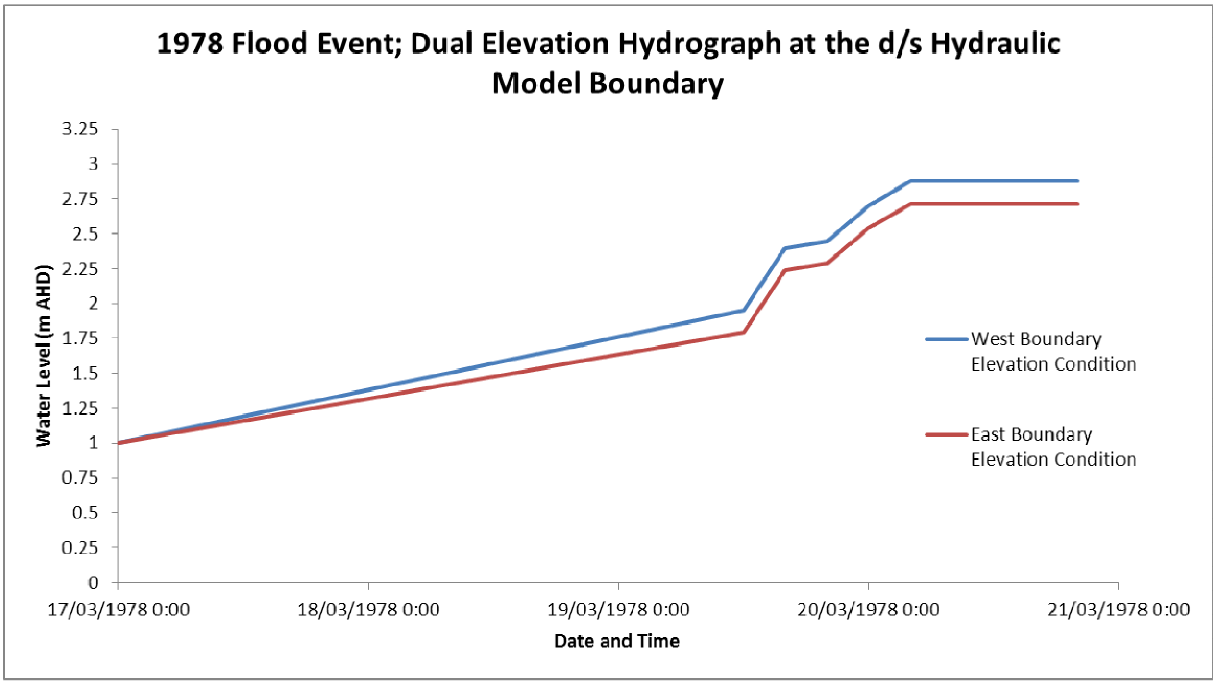


**Figure 22: Dual stage hydrographs defined on the western and eastern boundary of the model (along the Manning River) used for the 1978 historic flood event.**

Calibration and verification was undertaken by varying the calibration parameters listed in the previous section using the inflow and outflow conditions defined for the 1999 and 1995 events. Recorded and simulated peak levels within the study area were compared and the model incrementally modified as needed until levels matched (within several centimetres).

The location and details of these records are summarised in Table 4.

The peak levels used to calibrate the hydraulic model are summarised in Table 5, showing the recorded values, modelled values and the differences for the 1999 and 1995 flood events as recorded at the Lansdowne Gauge (those for the 1978 event are shown in Table 6).

**Table 4: Calibration and Verification Data Summary**

| Location Code | Location Description | Data Description | Approx. Coordinates (MGA 56) | Applicable Flood Event |
|---|---|---|---|---|
| R1 | Lansdowne River Gauge | Recorded at gauge | 453950, 6483030 | 1999, 1995, 1978 |
| R2 | 1344 Cundle Road, Lansdowne | Measured based on recollections of residents | 455570, 6483080 | 1999, 1995 |
| R3 | Supplied by NSW PWD | Recollections of resident (unclear of date/time) | 457190, 6482380 | 1978 |
| R4 | Supplied by Council | Based on debris (unclear of details (unclear of date/time) | "Croki Street" | 1978 |

**Table 5: Calibration and Verification Results Summary**

| Historic Flood Event | Location | Peak level recorded (m AHD) | Peak level modelled (m AHD) | Difference in recorded and modelled levels (m) |
|---|---|---|---|---|
| 1995 | R1 | 9.78 | 9.75 | -0.03 |
| | R2 | 5.30 | 5.25 | -0.05 |
| 1999 | R1 | 9.18 | 9.16 | -0.02 |
| | R2 | 4.80 | 4.79 | -0.01 |

**Table 6: Comparison of the 1978 Manning Valley Flood Event**

| Location | Peak level recorded (m AHD) | Peak level modelled (m AHD) | Difference in recorded and modelled levels (m) |
|---|---|---|---|
| R1 | 7.4 | 8.9 | +1.5 |
| R3 | 2.7 | 3.4 | +0.7 |
| R4 | 6.9 | 6.0 | -0.9 |

Figure 23 through Figure 25 show the peak water surface elevation in the study area for the 1995, 1999 and 1978 historic events. The peak water surface profile along the approximate thalweg of the Lansdowne River from the confluence of Haywards Gully to the old Coopernook Bridge is shown in Figure 26 for all three flood events, with corresponding chainages shown on Figure 27.

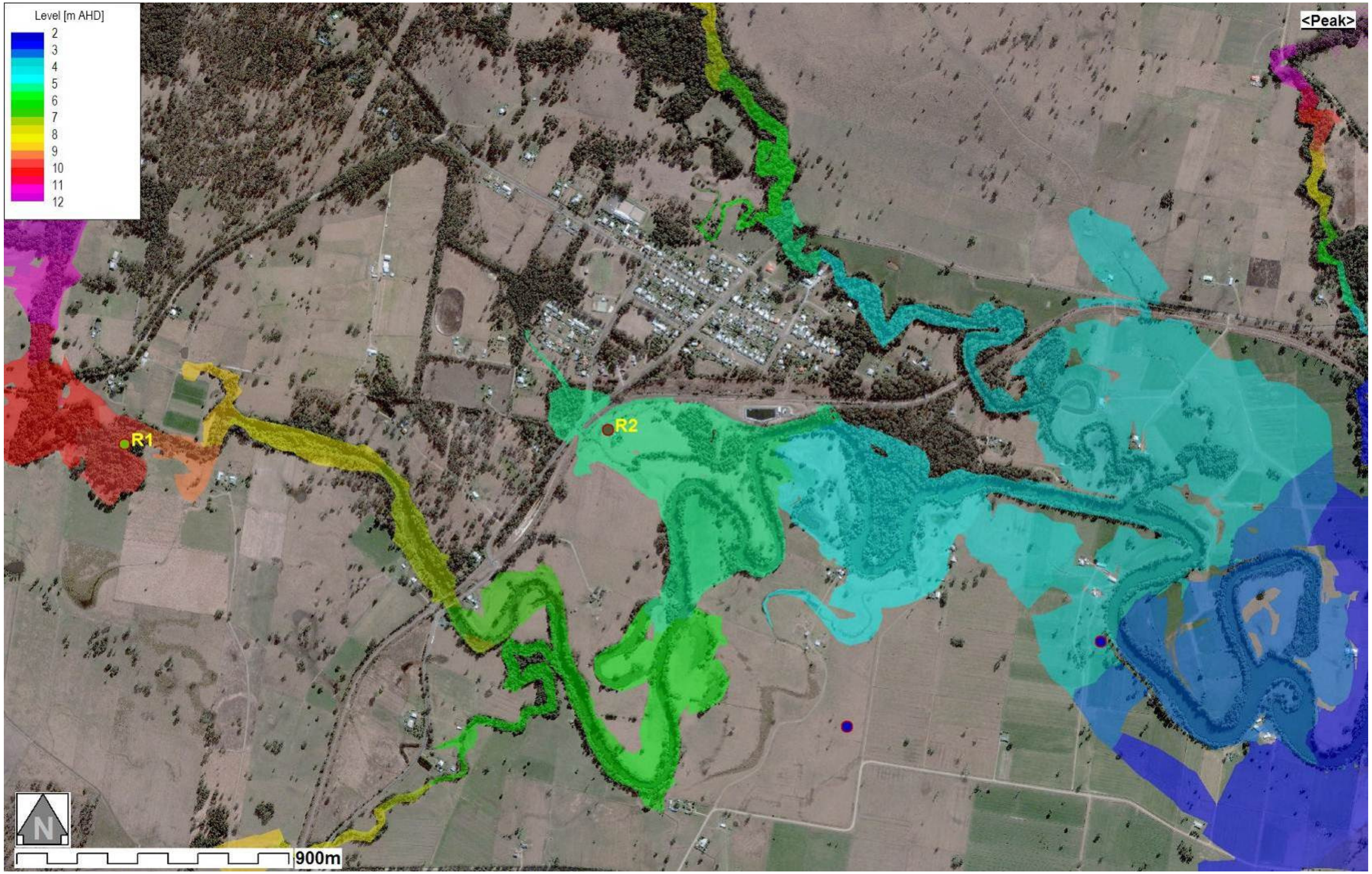


**Figure 23: Peak water level plot for the calibrated 1995 flood event. This represents the best fit to all available data.**

**The location of recorded water levels used in the calibration process are shown as R1 and R2**

**(refer Table 4: Calibration and Verification Data Summary for values and descriptions).**

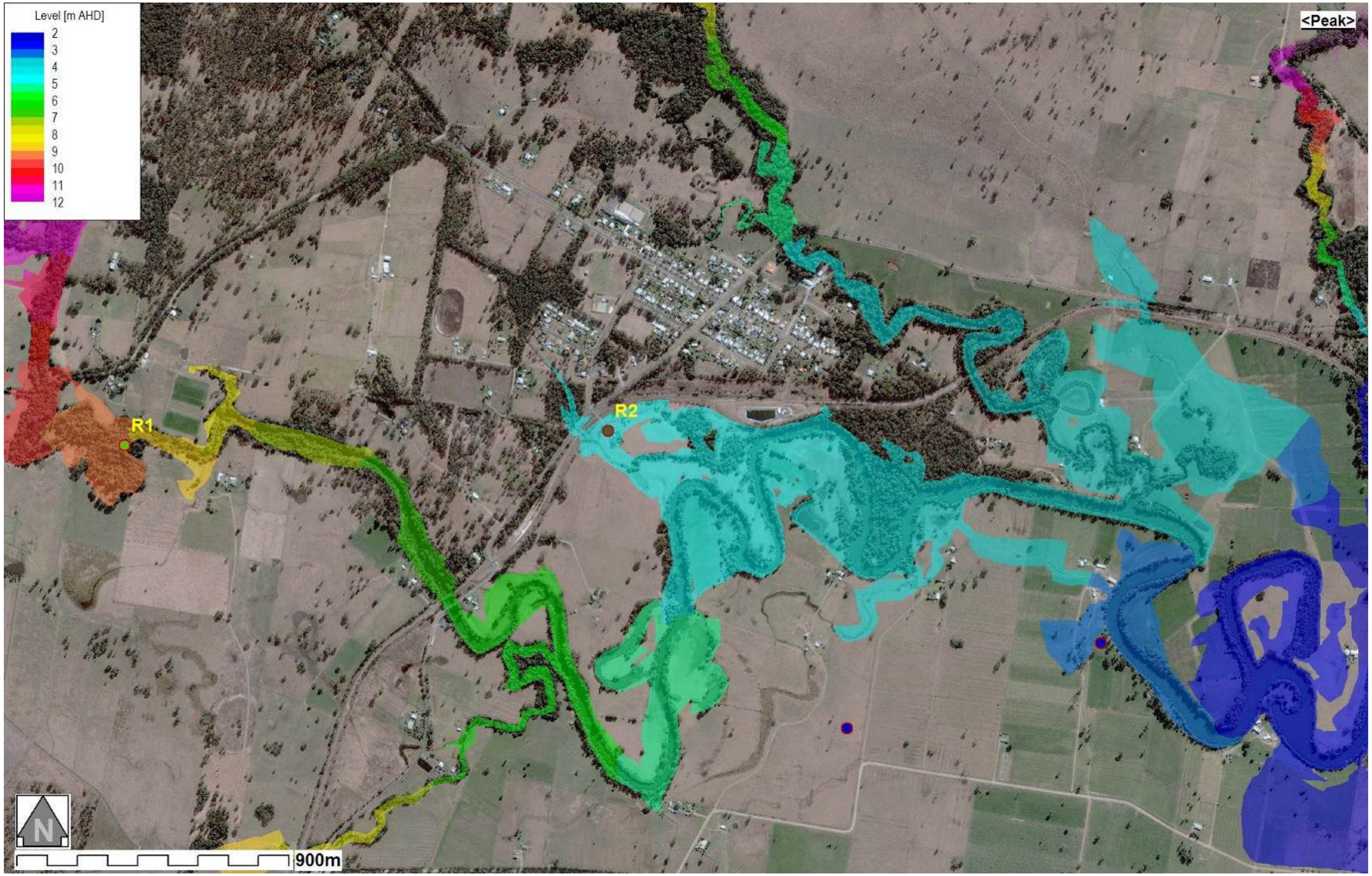


**Figure 24: Peak water level plot for the calibrated 1999 flood event. This represents the best fit to all available data.**

**The location of recorded water levels used in the calibration process are shown as R1 and R2**

**(refer Table 4: Calibration and Verification Data Summary for values and descriptions).**

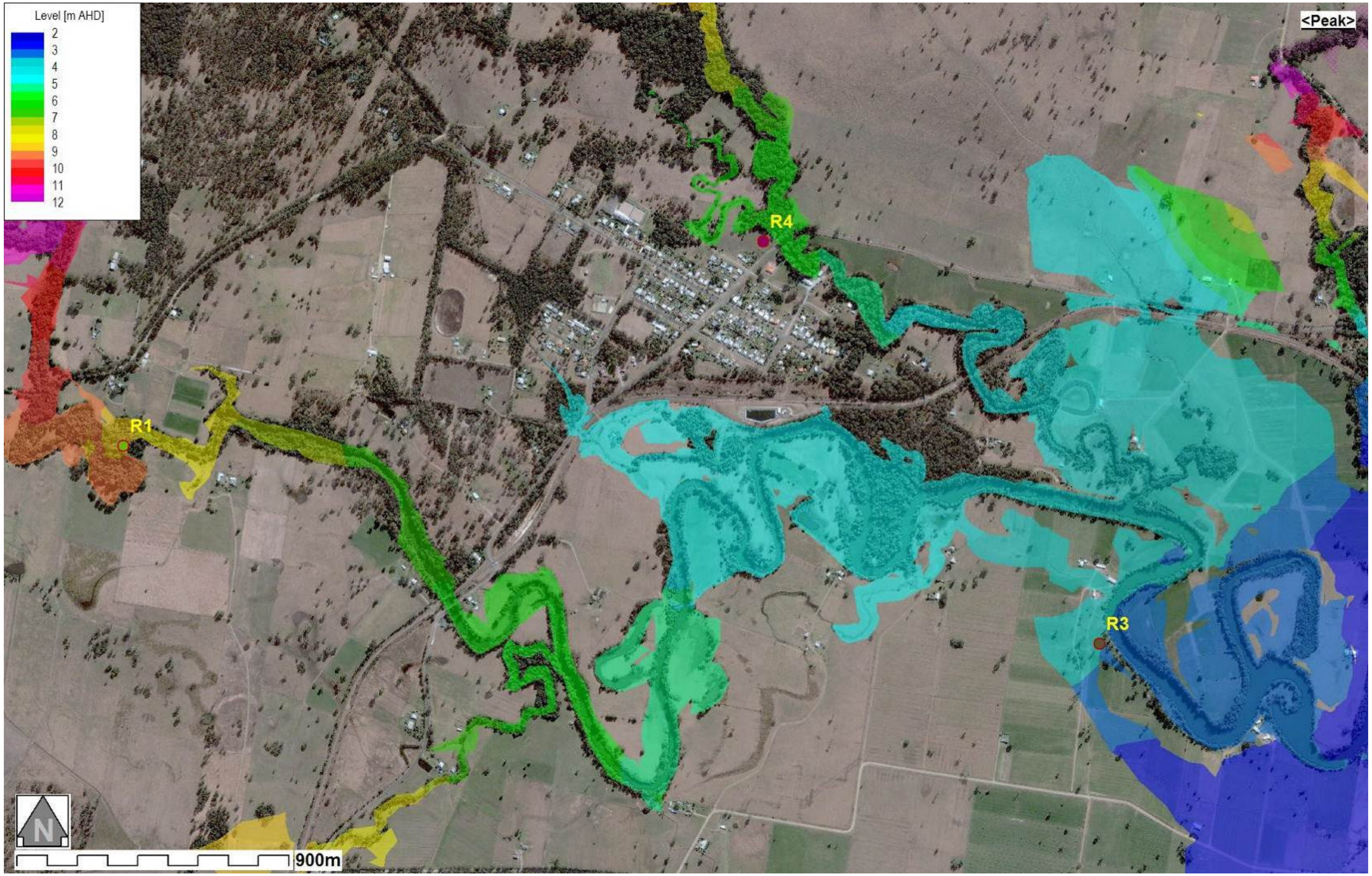


**Figure 25: Peak water level plot for the calibrated 1978 flood event.**

**This historic flood event was not used to calibrate the model as available data was insufficient. Nevertheless, the location of recorded water levels are shown as R1, R3 & R4**

**(refer Table 4: Calibration and Verification Data Summary for values and descriptions).**

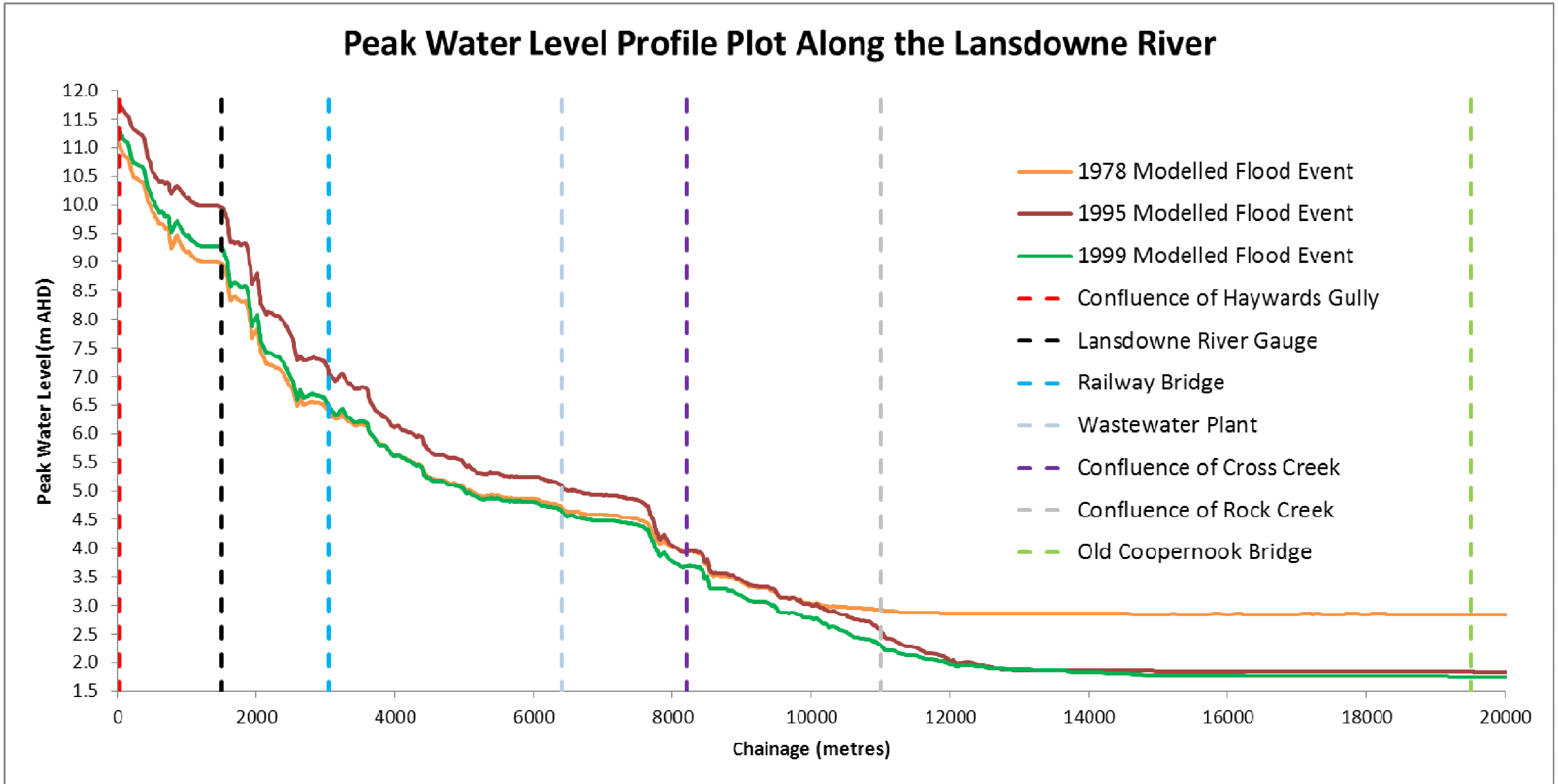


**Figure 26: Peak water profile for Lansdowne River**

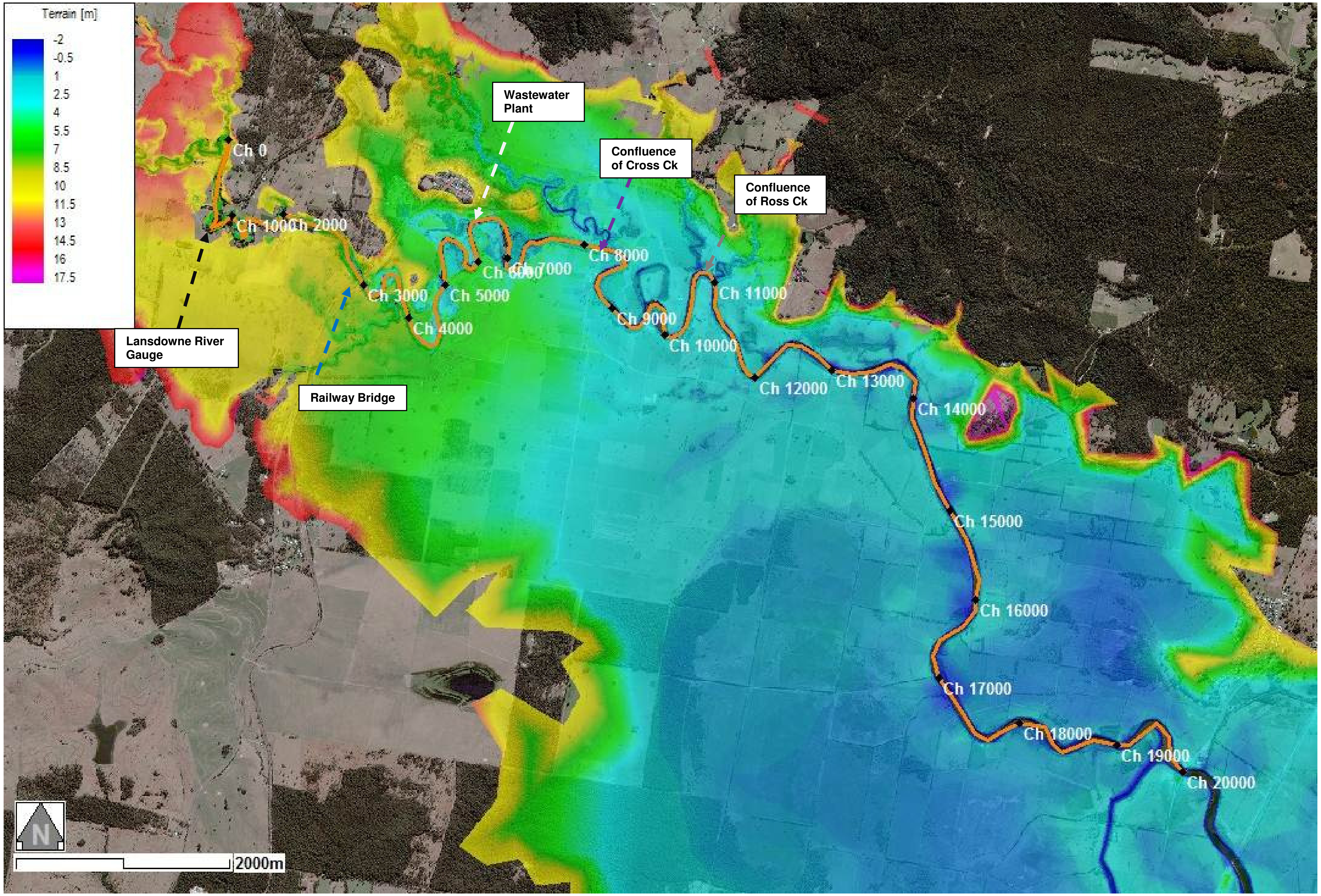


**Figure 27: WL Profile Chainages for the Lansdowne River**

Calibration of the model is considered to be good, sufficiently matching levels recorded in the 1999 and 1995 weather events which utilise the best available data in the study area. As mentioned, the 1978 Manning Event was not utilised for calibration because there was:

1. Uncertainty in the location and timing of the recorded data (there were two large flood events within one week of each other in 1978).
2. Insufficient detail in the inflow hydrograph recorded for the Lansdowne River (daily peak levels appear to have been used).
3. Only one stage hydrograph on the Manning River (the Croki gauge failed on the rising limb).

Considering the limitations of the available data, the 1978 flood event was only modelled for reference.

Final calibration of the model required modification to the Manning roughness as well as additional detail to the model network in some areas (refinements made based on ALS, air photos and bathymetry data available). An overview of the final network is shown in Figure 16, with finer detail shown in Figure 29 as a close-up view of the calibrated model in the study area with the topography coloured.

The calibrated and verified hydraulic model had a Manning roughness coefficient that varied between 0.030 and 0.110. The distribution of model roughness is shown in Figure 28 where the colour representation is:

- Light blue; 0.030,
- Green; 0.035
- Red; 0.042
- Pink; 0.052
- Dark blue; 0.062
- Brown; 0.090
- Purple; 0.110

Where the majority of the model was comprised of the first four coefficients listed. Areas of dense vegetation and the effects of structures such as bridges (that are hydraulic controls) were modelled with the latter three coefficients.

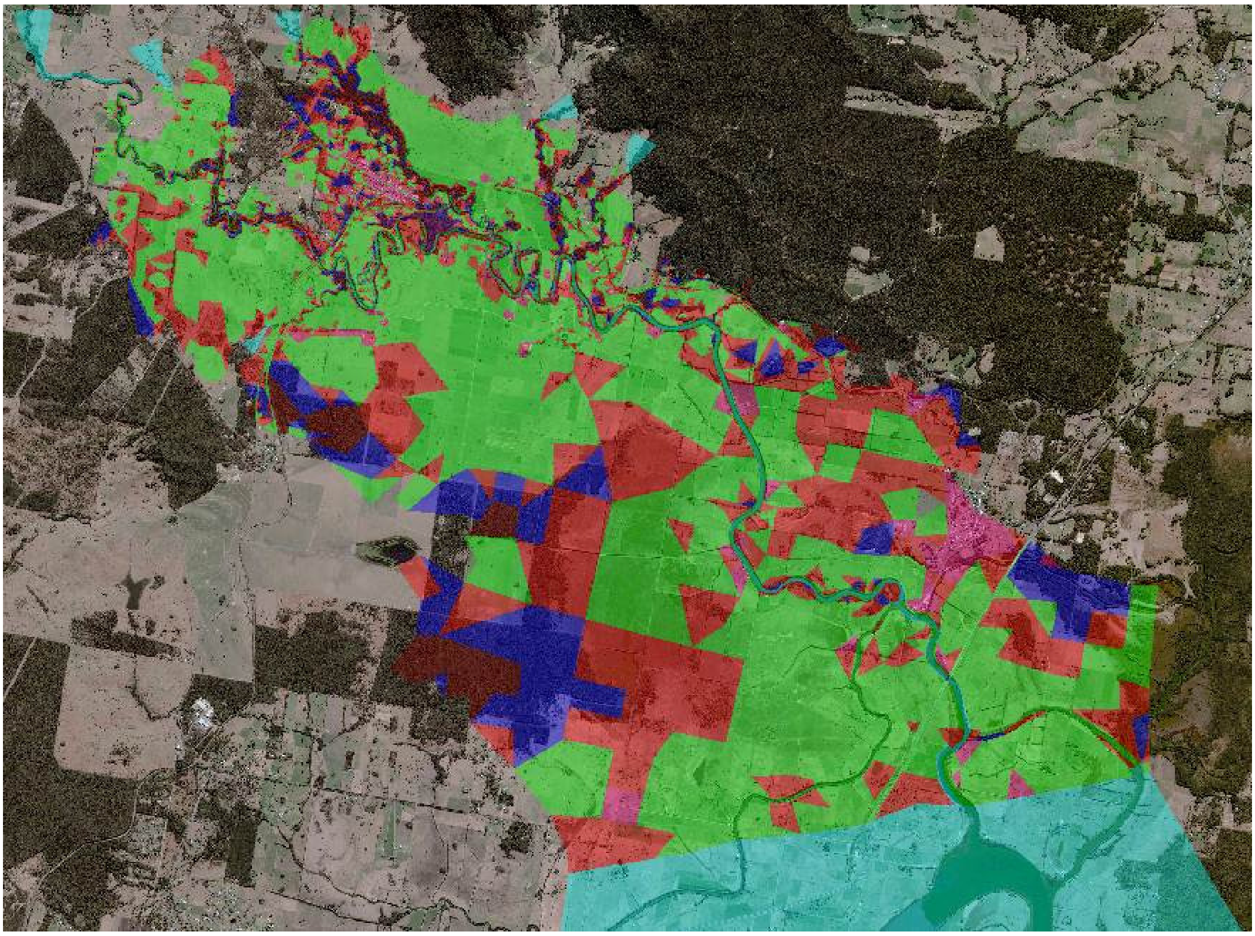

**Figure 28: Overview of the hydraulic model and its calibrated roughness, where different coloured elements represent areas that have different Manning roughness coefficients.**

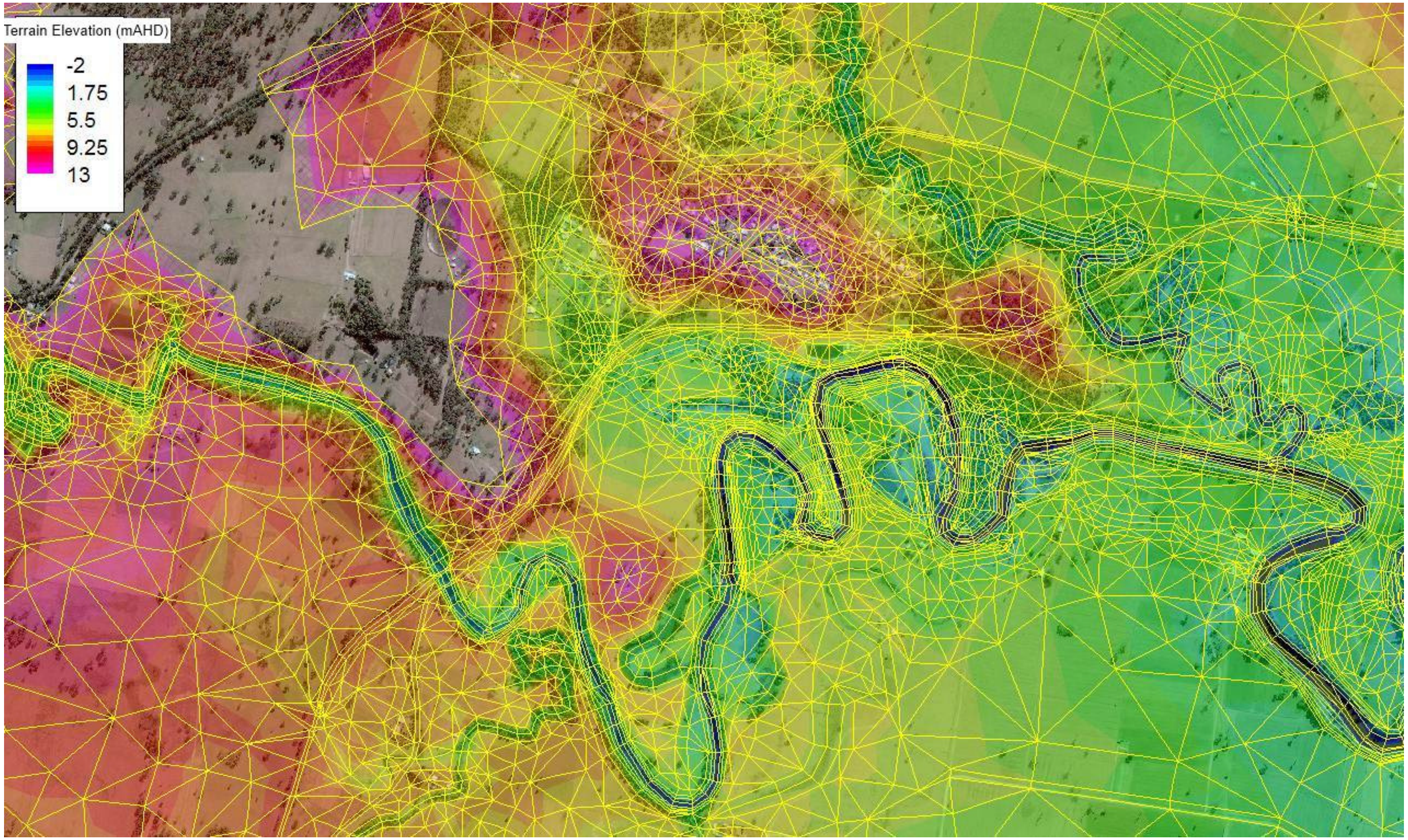


**Figure 29: Close view of the final calibrated hydraulic model in the study area showing the network with elevations coloured.**

## 6.4 Design Flood Simulations

The calibrated hydraulic model was used to simulate the 5%, 2%, 1% and 0.5% AEP floods as well as the PMF.

At the downstream end of the model, a dual stage hydrograph relationship was used in the same way as for the calibration process. This was defined based on the results from the previous flood study (ESTRY results) and historic data in the following way:

- The results of the previous flood study showed that the peak flows take approximately 12 to 13 hours to travel from Killawarra along the Manning River to the confluence of the Lansdowne River.
- Historic water level data recorded at the Killawarra and Lansdowne gauges (# 208004 and #208015 respectively) show that peak levels occur at these gauges anywhere from 5 to 20 hours apart. This means that peak levels in the Lansdowne River at Lansdowne have the potential to peak within 5 hours of peak levels occurring at Killawarra on the Manning River. Or, using the data in the previous point, within approximately 17 hours of peak levels arriving at the confluence of the Manning and Lansdowne Rivers.
- Based on initial hydraulic model simulations, the difference in time between the Lansdowne River upstream boundary location and the location of the Lansdowne River gauge is approximately 1 hour.

Using the aforementioned data, the downstream boundary condition was modelled using data from the previous flood study. The hydrograph at the confluence of the Lansdowne and Manning Rivers was extracted for each recurrence interval and the peak was offset in time to ensure that the peak inflow hydrograph for the Lansdowne River occurred approximately 18 hours prior.

For example, the hydrograph for the 1% AEP Manning River flood event was extracted from the previous model (ESTRY) at two locations corresponding to the downstream boundary of the hydraulic model (western and eastern boundaries). The peak level at these gauges was then shifted in time so that it occurred approximately 18 hours after the peak inflow from the Lansdowne River at the upstream boundary condition. This method meant that only part of the Manning stage hydrographs was needed as peak levels in the Lansdowne River occurred when Manning levels are on the rising limb.

This is considered to be a conservative approach to modelling the hydraulic design floods in the Lansdowne sub-Catchment because each design flood was modelled with a flood of the same recurrence interval occurring in the Manning River.

The stage hydrographs used to define the downstream boundary condition of the model for the design flood simulations are shown in Figure 30 and Figure 31.

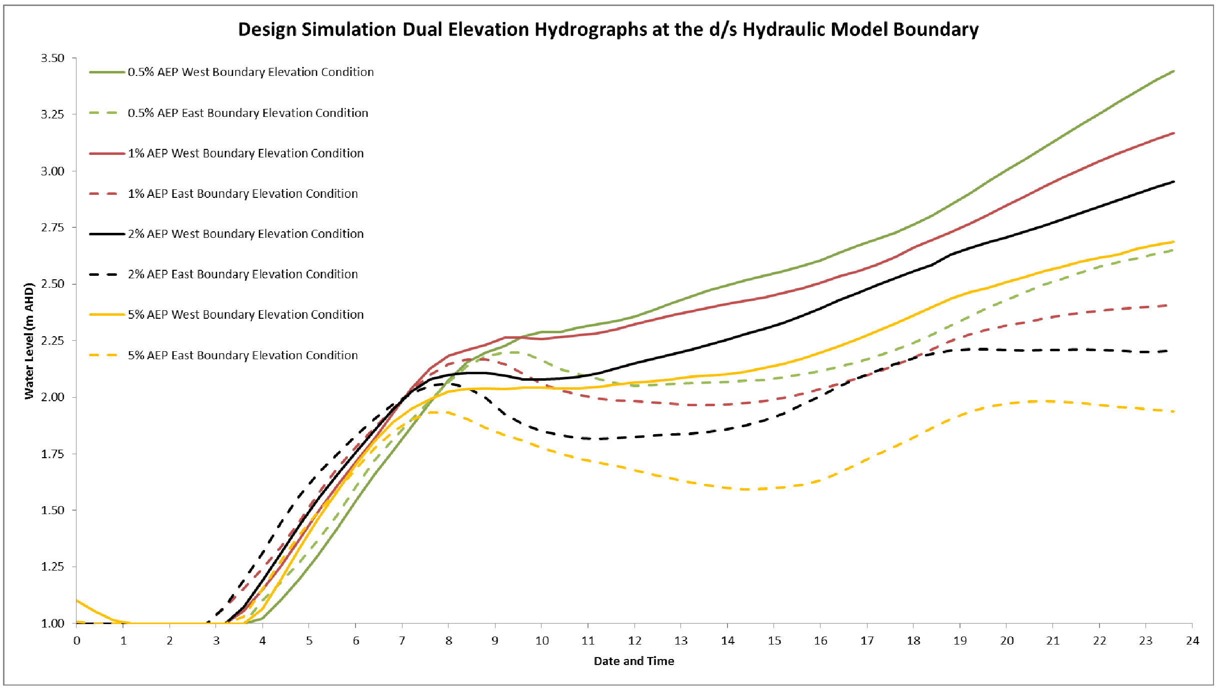


**Figure 30: Dual stage hydrographs defined on the western and eastern boundary of the model (along the Manning River) used for the design flood events (except for the PMF).**

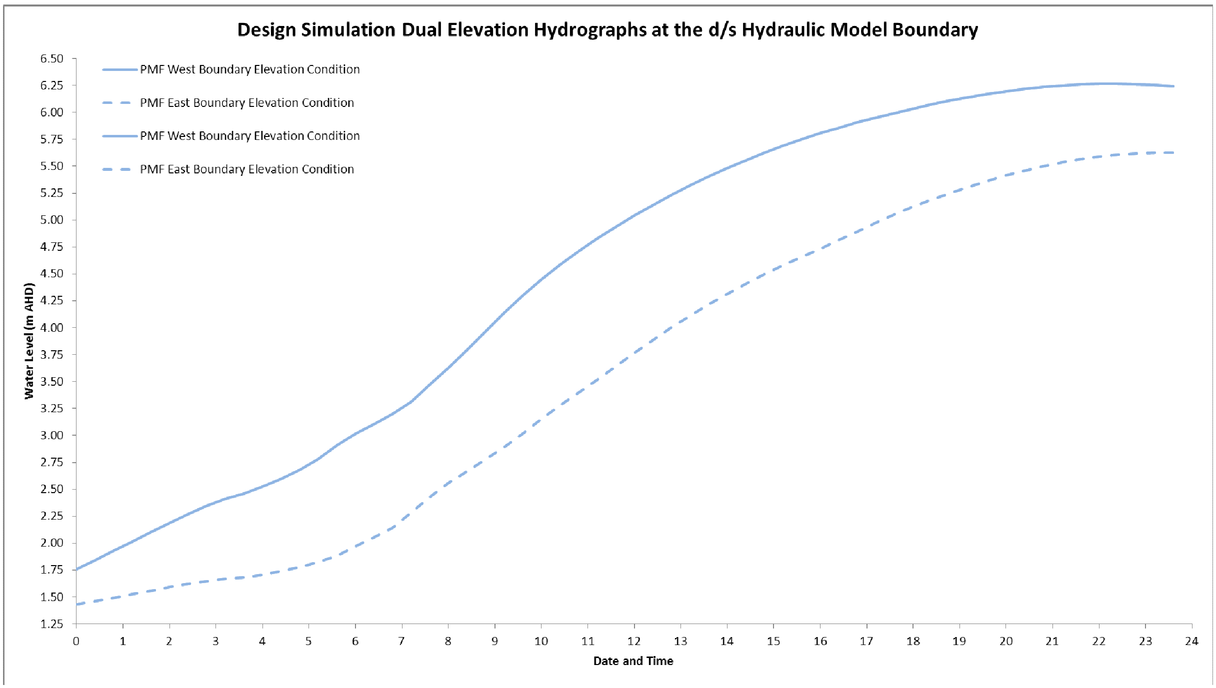


**Figure 31: Dual stage hydrographs defined on the western and eastern boundary of the model (along the Manning River) used for the PMF design flood event.**

## 6.5 Sensitivity Analysis

As part of the verification process, a sensitivity analysis was performed on the hydraulic models. This involved testing the effects of the downstream boundary condition water level on the hydraulic model within the extent of the flood study area.

The 5% AEP flood model was selected for the sensitivity analysis. In addition to the downstream boundary condition as calculated in Section 6.4, a new boundary condition was also selected. Below is a comparison of the 2 boundary condition water levels:

**Table 7: Peak Water Levels used for Sensitivity Analysis**

| Peak Water Levels (Original Boundary Condition) | | Peak Water Levels (Sensitivity Boundary Condition) | |
|---|---|---|---|
| West Elevation | 2.69 m AHD | West Elevation | 2.3 m AHD |
| East Elevation | 1.98m AHD | East Elevation | 1.6 m AHD |

As can be seen in Table 8, modifying the downstream boundary conditions has minimal impact on the water profile within the study area.

**Table 8: Comparision of water profiles between original and sensitivity downstream boundary conditions**

| Chainage (m) | 5% AEP Profile (Original Boundary Conditions) | 5% AEP Profile (Sensitivity Conditions) | Difference (m) |
|---|---|---|---|
| **1000** | 10.73 m AHD | 10.73 m AHD | 0 |
| **1500** | 10.6 m AHD | 10.6 m AHD | 0 |
| **2000** | 9.39 m AHD | 9.39 m AHD | 0 |
| **2500** | 8.29 m AHD | 8.29 m AHD | 0 |
| **3000** | 7.77 m AHD | 7.77 m AHD | 0 |
| **3500** | 7.32 m AHD | 7.32 m AHD | 0 |
| **4000** | 6.51 m AHD | 6.51 m AHD | 0 |
| **4500** | 6.09 m AHD | 6.09 m AHD | 0 |
| **5000** | 5.82 m AHD | 5.81 m AHD | 0.01 |
| **5500** | 5.64 m AHD | 5.63 m AHD | 0.01 |
| **6000** | 5.59 m AHD | 5.59 m AHD | 0 |
| **6500** | 5.34 m AHD | 5.34 m AHD | 0 |
| **7000** | 5.24 m AHD | 5.24 m AHD | 0 |
| **7500** | 5.14 m AHD | 5.14 m AHD | 0 |
| **8000** | 4.24 m AHD | 4.24 m AHD | 0 |
| **8500** | 4.02 m AHD | 4.02 m AHD | 0 |
| **9000** | 3.66 m AHD | 3.66 m AHD | 0 |

**Culvert and Bridge Blockages:**

There are a number of stream crossings within the study area. The most prominent are the Lansdowne Road and Railway bridges across the Lansdowne River southwest of the town and across Cross Creek east of the town. In addition there is a culvert crossing of Cundle Road and the adjacent railway in town, a low level bridge crossing at Warrens Lane north of the town, and a road and rail crossing at Koolah Creek east of the town.

The methodology in Australian Rainfall & Runoff's Stage 2 Report on blockage of hydraulic structures was applied to assess potential blockages at these crossings. In the absence of historical records on blockages, Scheme A, Assessment Procedure for an AEP Neutral Blockage Level was used to assess the most likely blockage levels, Table 9.

**Table 9 - Debris Blockage Assessment**

| Criteria | Assessment | Comment |
|---|---|---|
| Debris Availability | Low | Rural lands, grazed paddocks, streams with moderate to flat slopes and stable banks |
| Debris Mobility | Medium | Moderate rainfall intensities and moderately sloped catchments |
| Debris Transportability | Low to Medium | Flat bed slopes (< 1%) and stream size varying from narrow to moderate in comparison to debris load dimensions |
| At Site Base Debris Potential | Low | Based on the combination of the above |
| At Site Debris Potential (adjustment for AEP) | Low | Up to 0.5% AEP |
| Most Likely Blockage Levels, Lansdowne River crossings | 0% | Length of longest 10% of debris is less than the width of the crossings |
| Most Likely Blockage Levels, tributary crossings | 25% | Debris length may be greater than the width of the crossing |
| Risk Based Blockage Assessment | 0% | Given the rural locations of all these sites and the limited assets at risk the consequences of severe blockage are very low |

There is a further mitigating factor that is relevant to the tributary crossings. During a major storm event, flooding from these tributaries is significantly impacted by backwater from the Lansdowne River. As a result, any blockage on these tributaries that may have occurred during the early stages of flooding will be drowned by the backwater and not have a significant impact on flood behaviour.

**Greenhouse induced sea level rise:**

The Lansdowne River model terminates at the Manning River and sea level rises only affect the Lansdowne River indirectly through rises in the Manning River. The GTCC Coastal Zone Management Plan is generally consistent with the State Government estimate of sea level rise of 0.4m by 2050 and 0.91m by 2100. The corresponding rise upstream in the Manning River at the mouth of the Lansdowne River will depend on the flow and the headloss through the entrance. It is unlikely that all of the rise will be manifest upstream and it is outside the scope of this study to model this effect.

It would, however be instructive to consider some amount of rise and then using the results of the modelling, interpolate a equivalent rise at Lansdowne. A rise of 0.5m in the tailwater level for the 1% AEP design flood is approximately equivalent to a 0.33% AEP event by interpolation between the 0.5% AEP and PMF surfaces. Likewise, by an equivalent interpolation upstream, this increase reduces to 0.1m at Lansdowne and would be even less considering the increased flow in the Lansdowne River for the 0.33% AEP flood compared to the 1% AEP.

Another approach would be to infer the impacts by using the sensitivity assessment of tailwater levels. The tailwater sensitivity (Table 8) indicates that levels 2km upstream of the confluence of Ross Ck and Lansdowne River (Figure 27) – or upstream of chainage 9km in the model – are not affected by tailwater levels which are ± 0.4m from the adopted tailwater levels for the study.

Thus the influence of any coastal sea level rise is initially diminished through the Harrington entrance, then further diminished through the backwater affected reaches along the lower Lansdowne River. The freeboard allowance of 0.5m included in the flood planning level is more than sufficient to accommodate any likely effect from sea level rise.

## 6.6 Results

Figure 32 shows the peak water surface profile along the approximate thalweg of the Lansdowne River for the design flood simulations with key chainage locations noted. Corresponding chainages are shown on Figure 27.

Figure 33 to Figure 52 show the design simulation results for the calibrated hydraulic model within the study area, 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)
- Hydraulic Hazards (only for the 1% AEP and PMF design flood simulations)

Hydrographic flood information for the Lansdowne River are provided in table form in Appendix A.

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

For greater detail, Figure 2 can be used to reference the location of the streets in Lansdowne

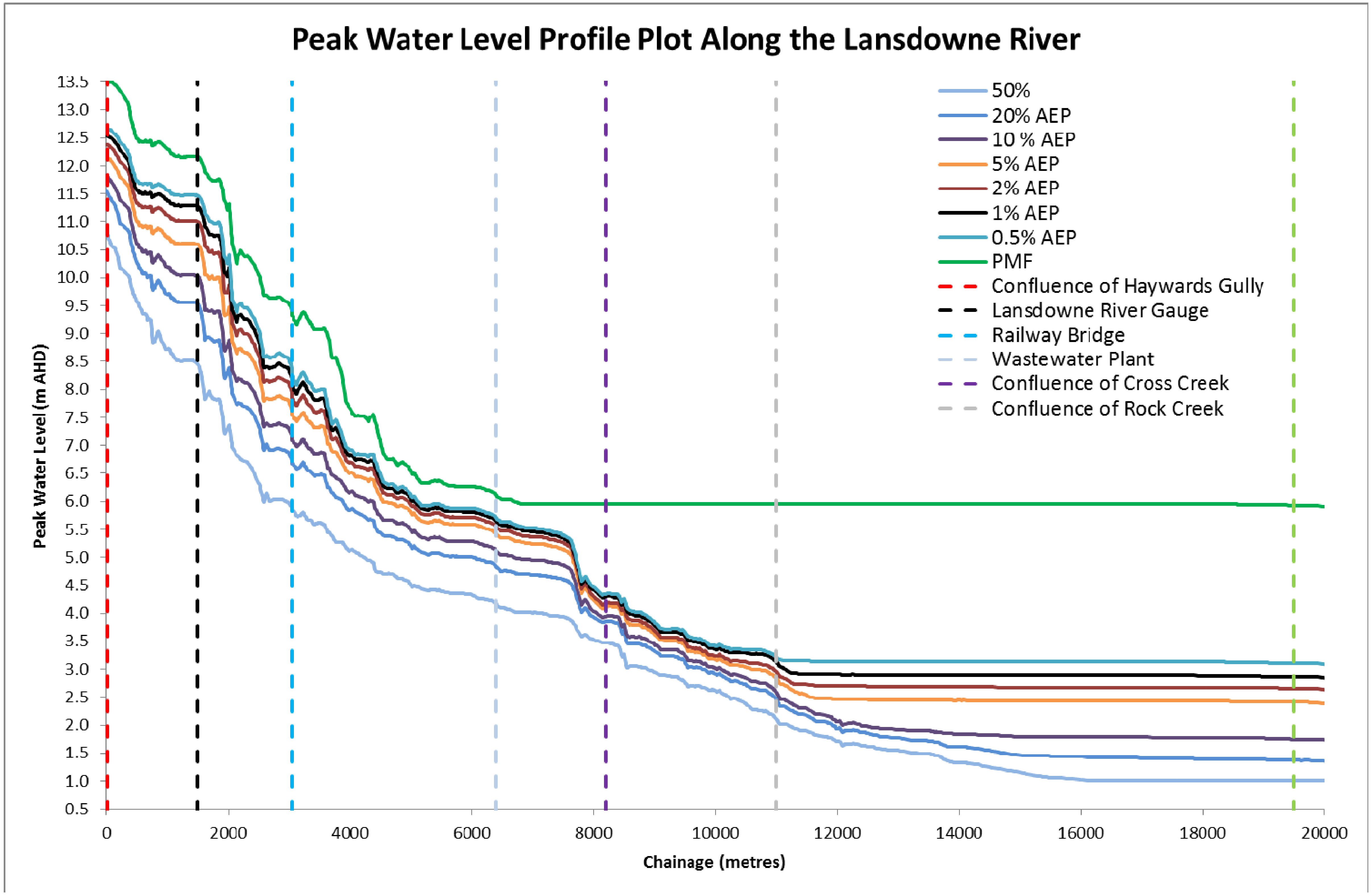


**Figure 32: Peak water surface level along the approximate thalweg of the Lansdowne River for the design flood hydraulic simulations**

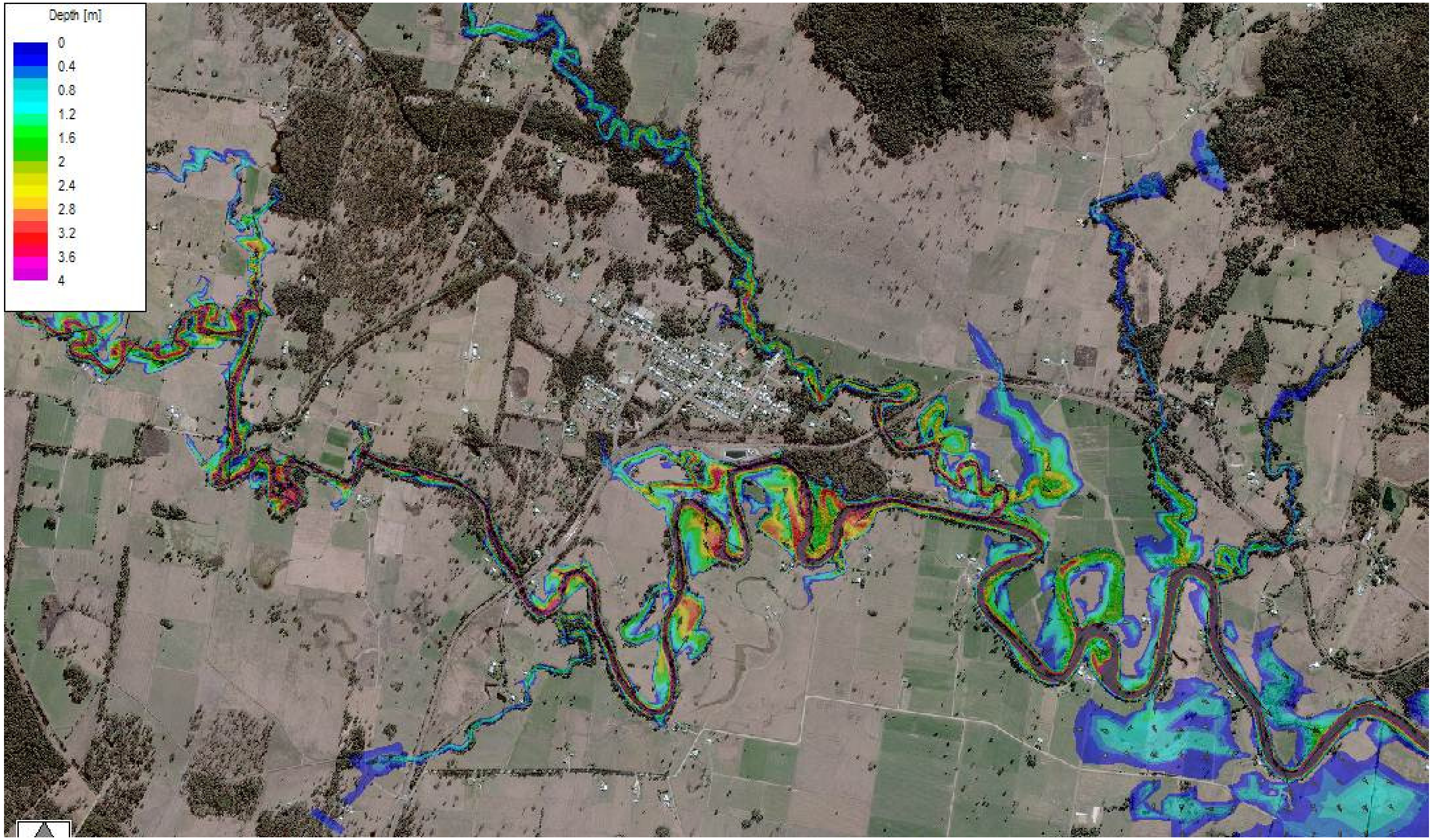


**Figure 33: 50% AEP Design Flood; Depth Coloured with Velocity Vector**

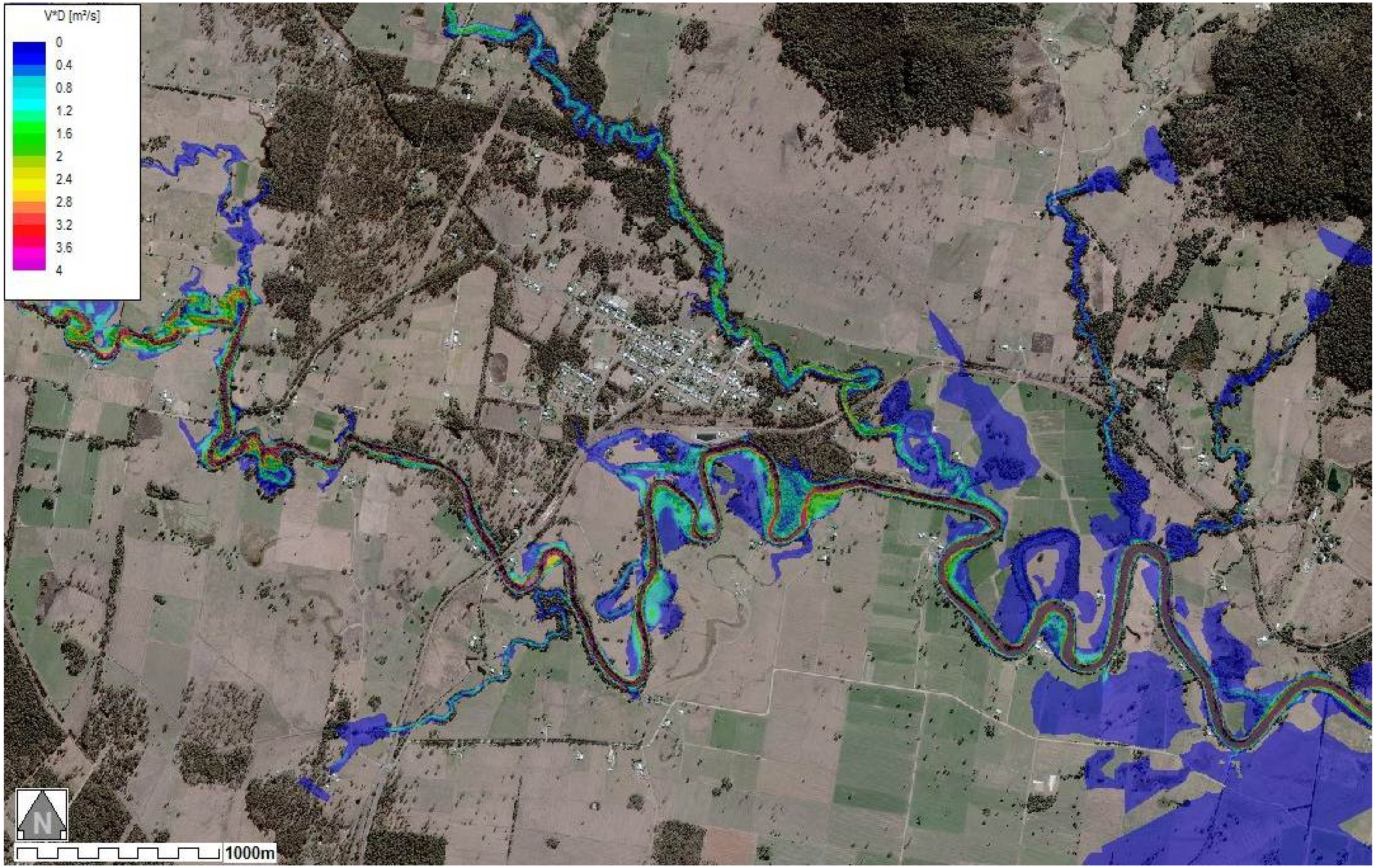


**Figure 34: 50% AEP Design Flood; Velocity times Depth Coloured**

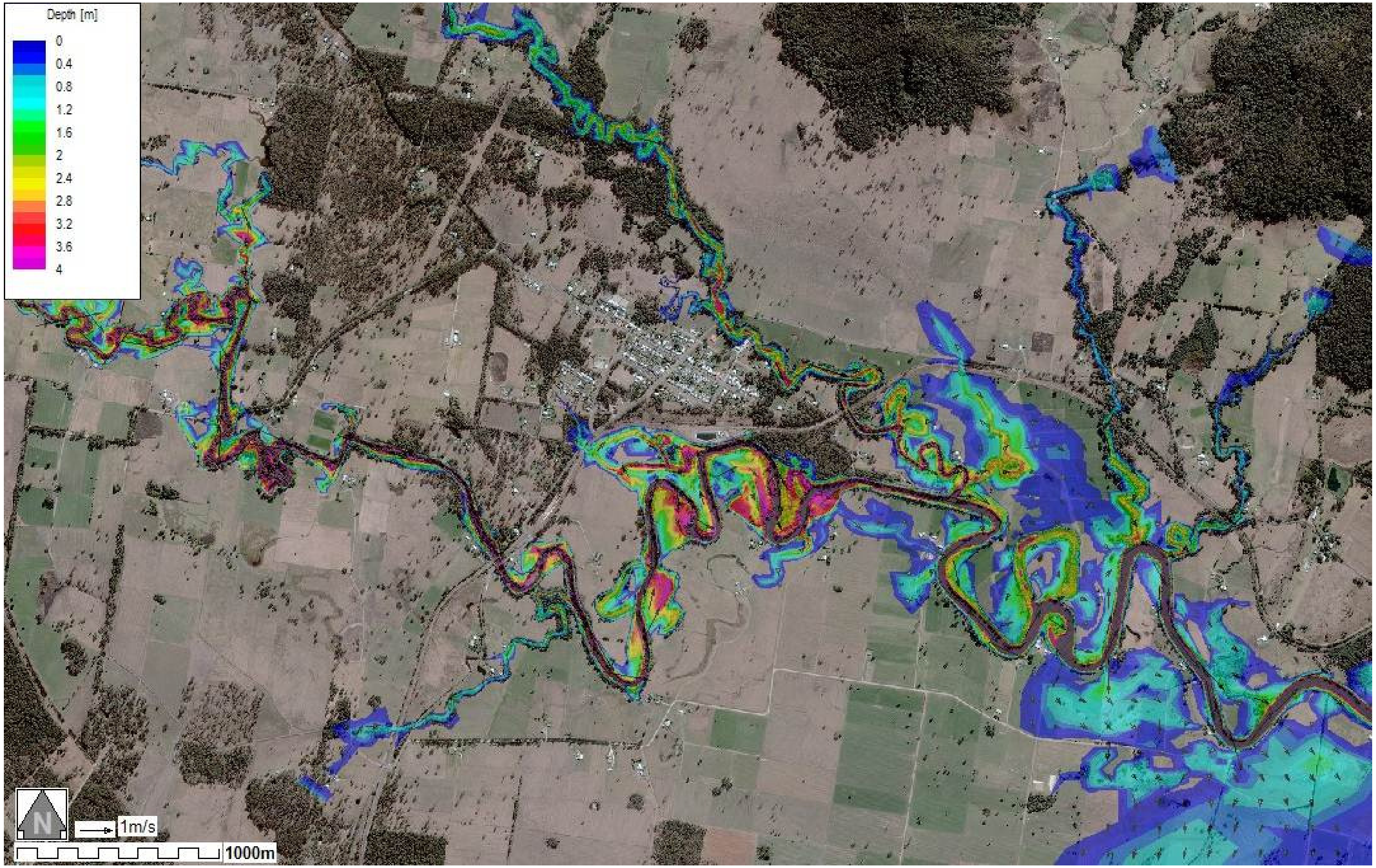


**Figure 35: 20% AEP Design Flood; Depth Coloured with Velocity Vector**

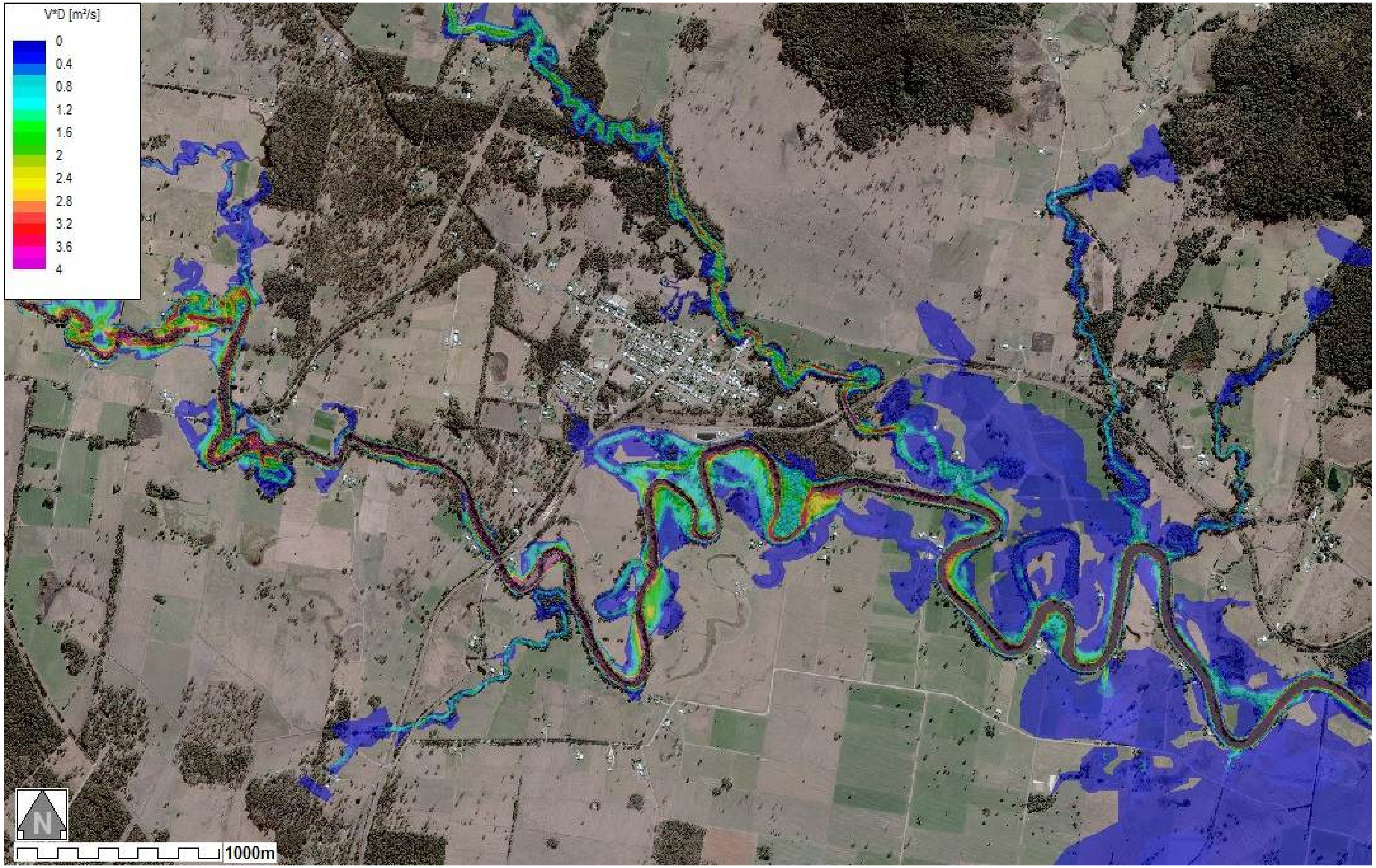


**Figure 36: 20% AEP Design Flood; Velocity times Depth Coloured**

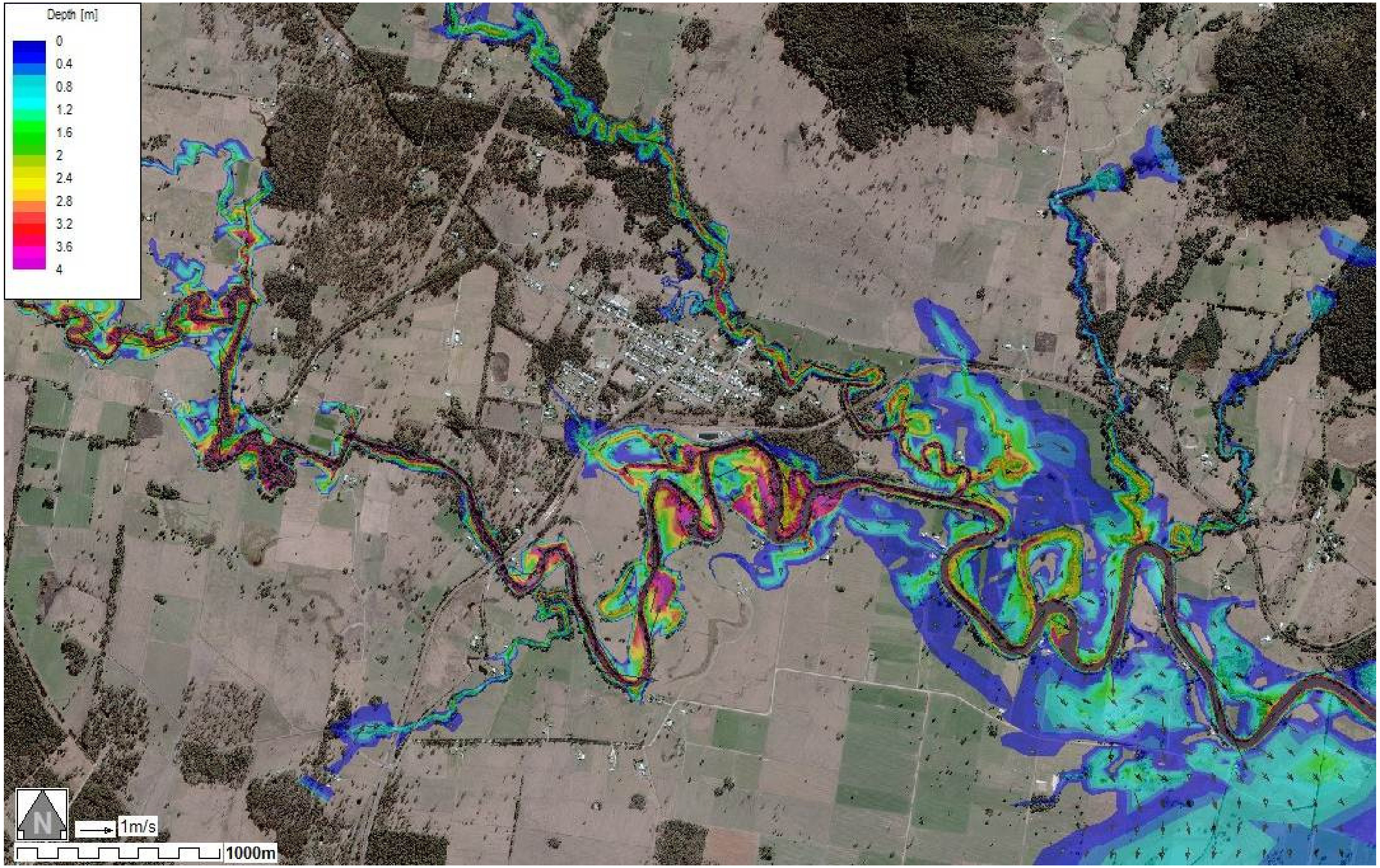


**Figure 37: 10% AEP Design Flood; Depth Coloured with Velocity Vector**

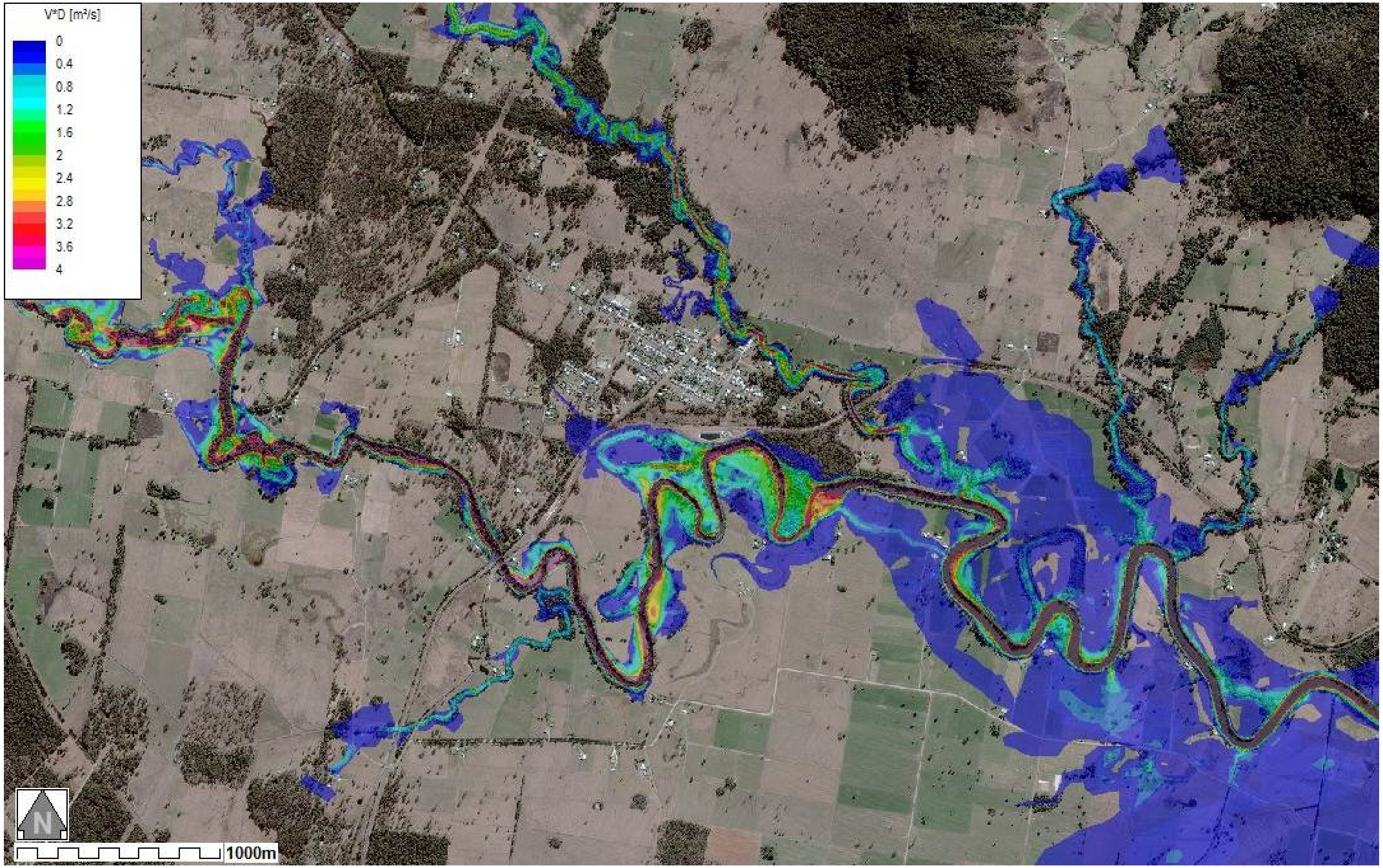


**Figure 38: 10% AEP Design Flood; Velocity times Depth Coloured**

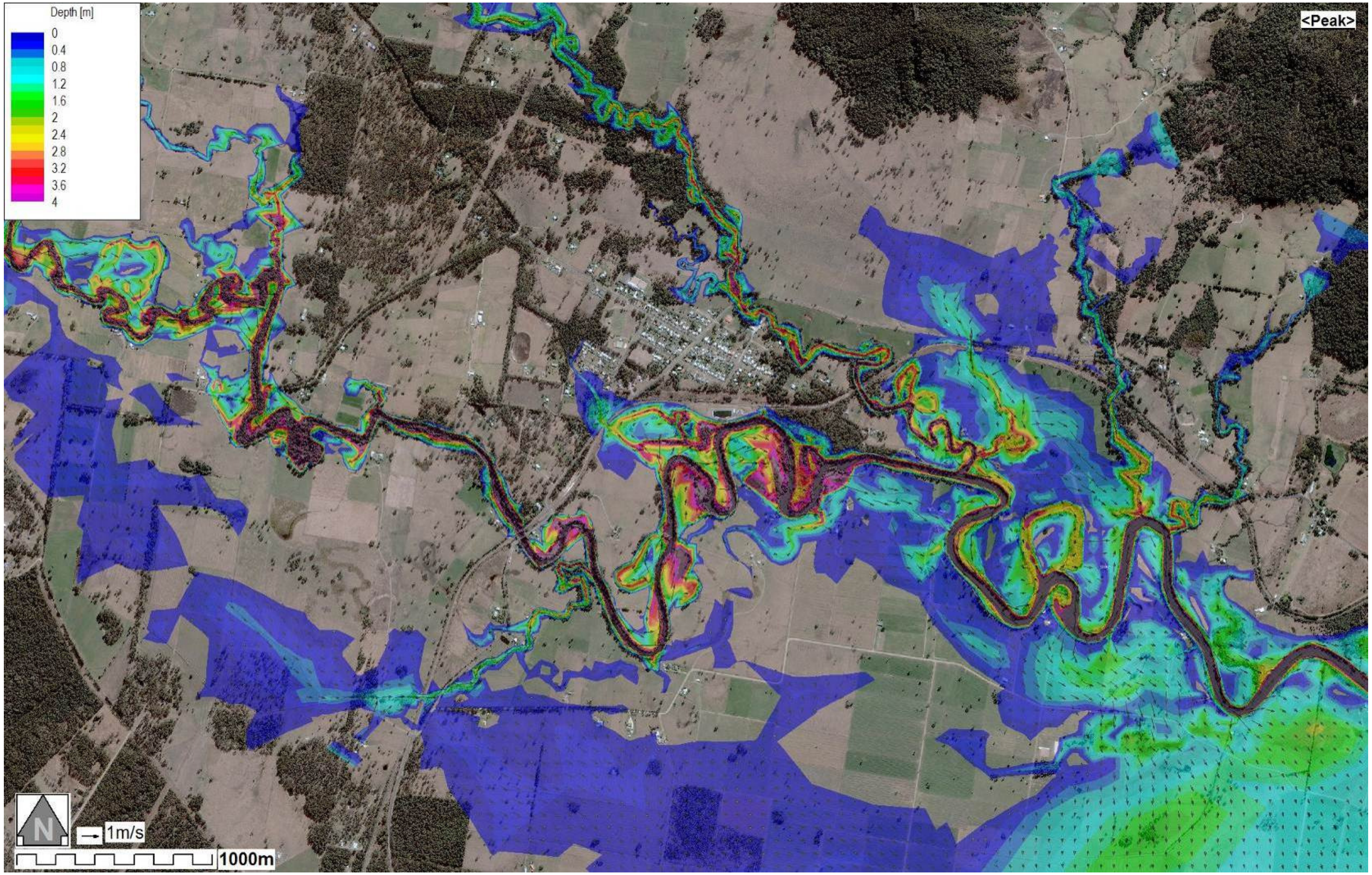


**Figure 39: 5% AEP Design Flood; Depth Coloured with Velocity Vector**

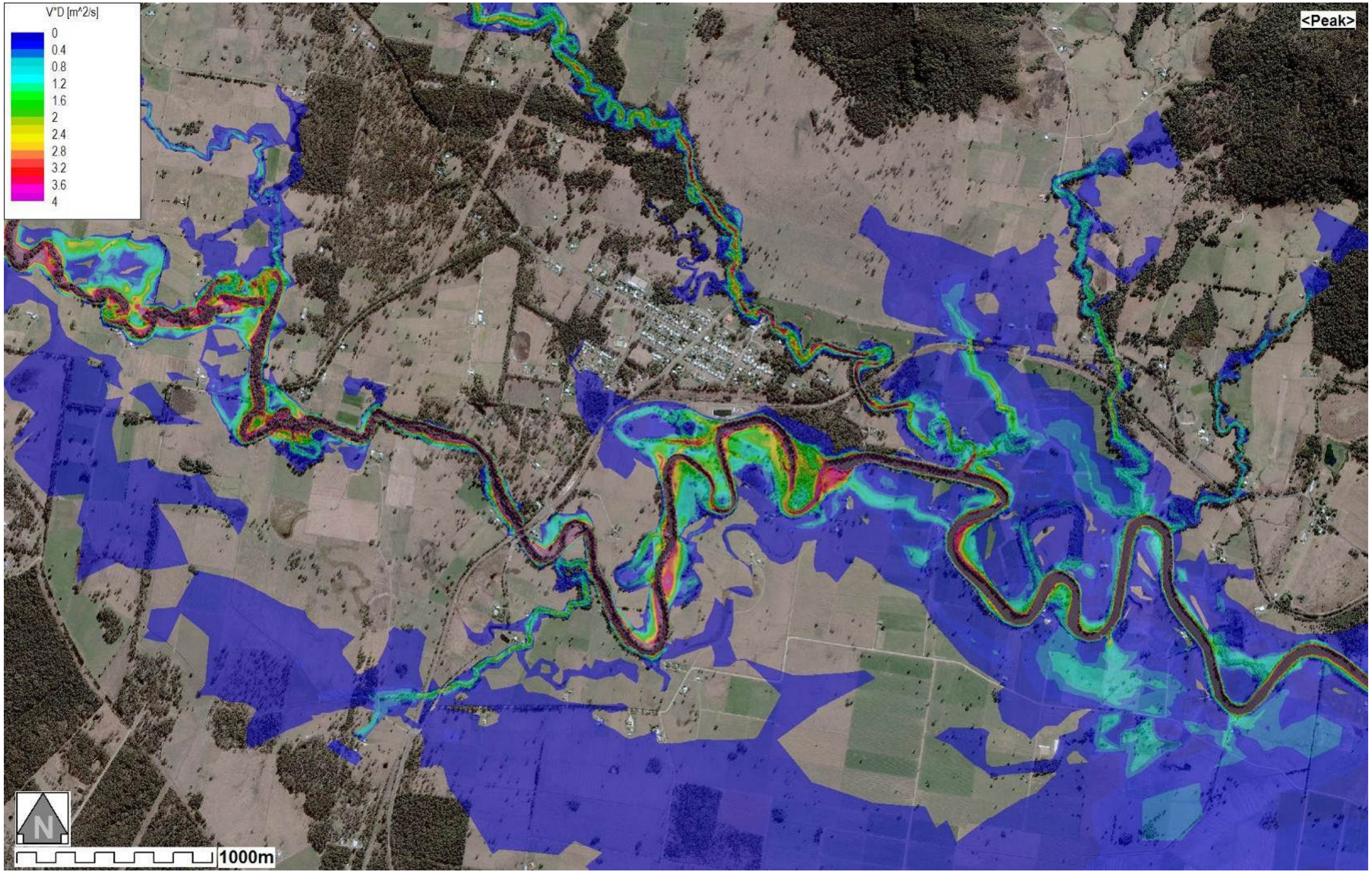


**Figure 40: 5% AEP Design Flood; Velocity times Depth Coloured**

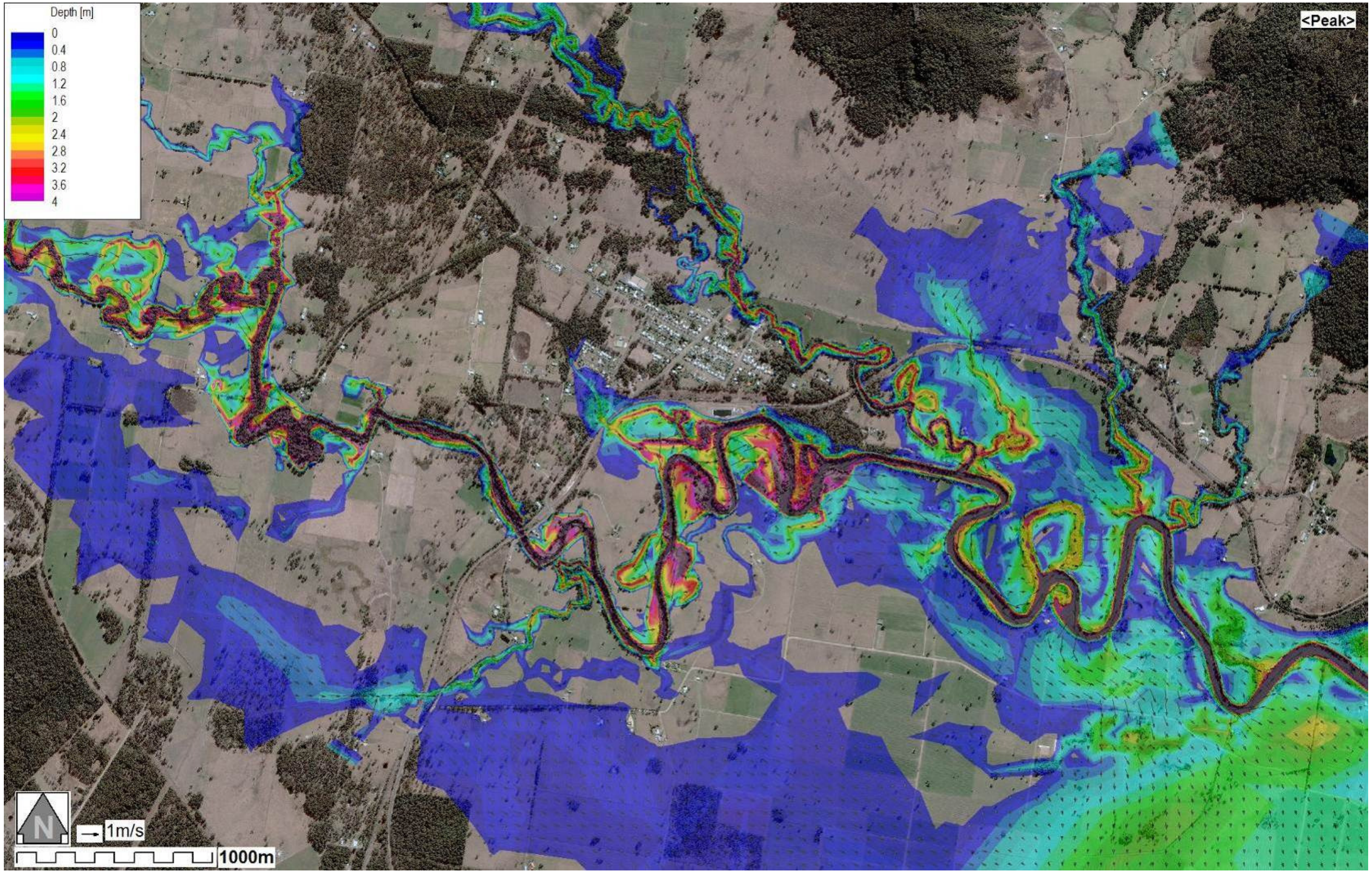


**Figure 41: 2% AEP Design Flood; Depth Coloured with Velocity Vectors**

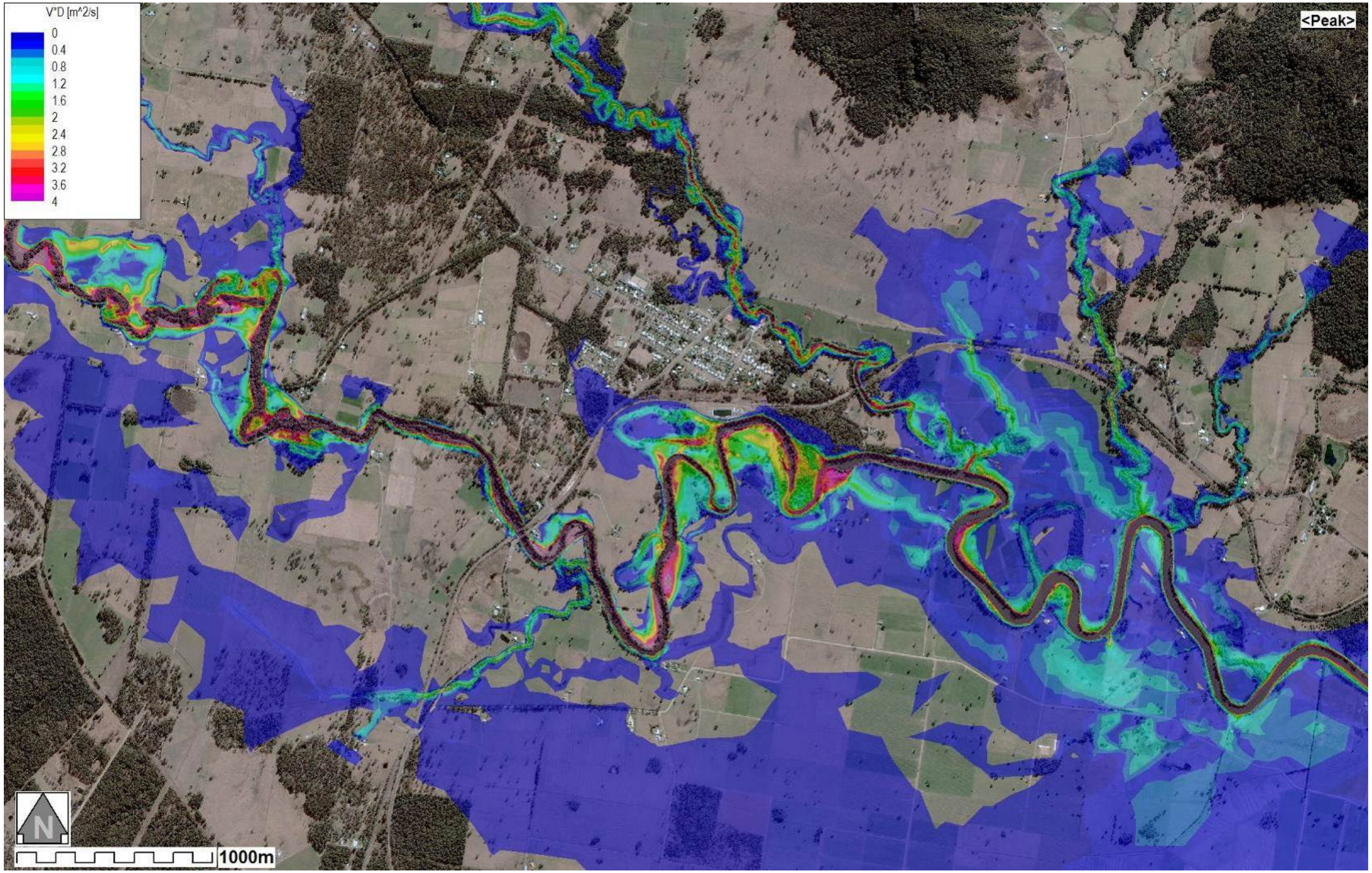


**Figure 42: 2% AEP Design Flood; Velocity times Depth Coloured**

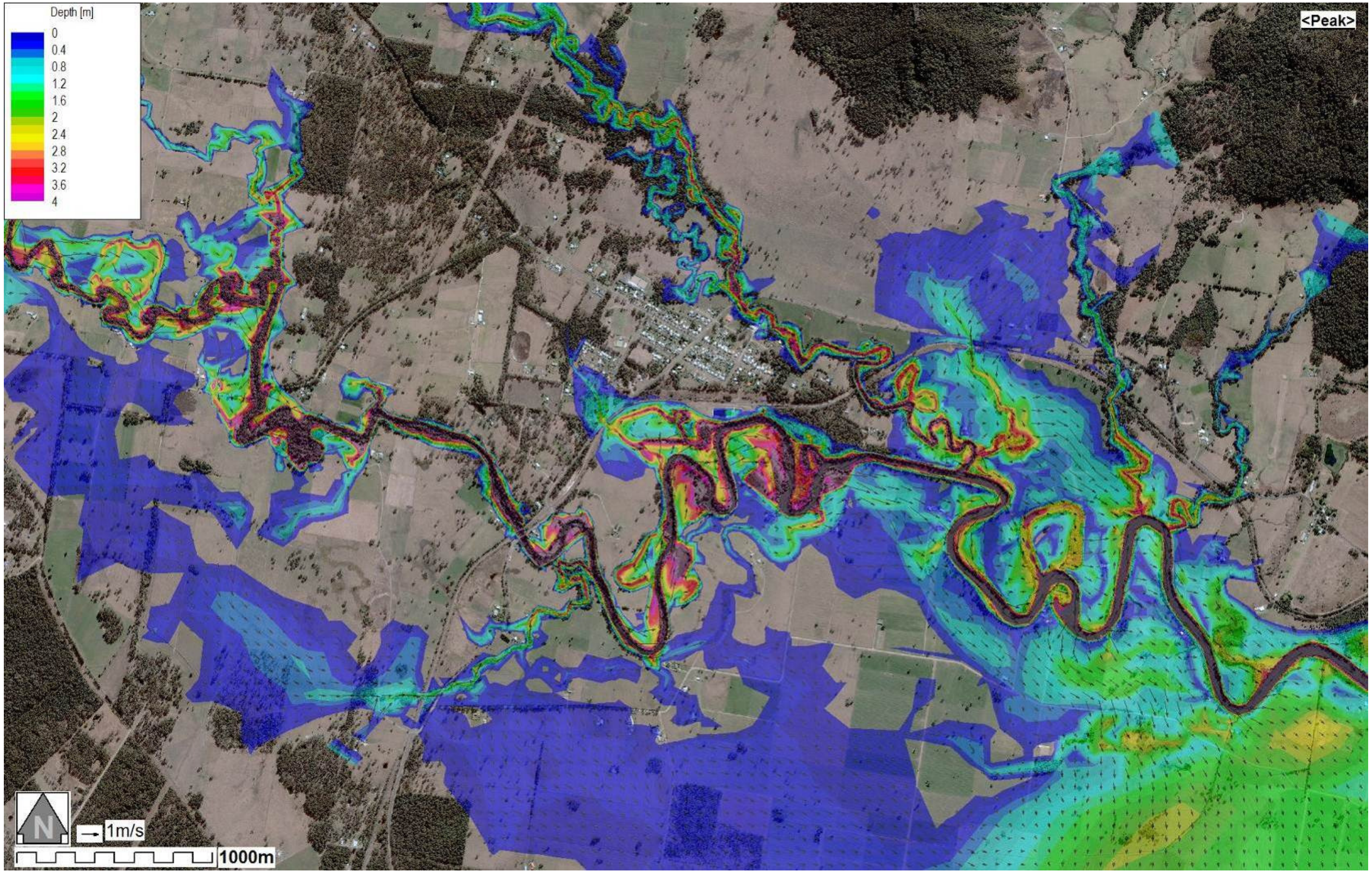


**Figure 43: 1% AEP Design Flood; Depth Coloured with Velocity Vectors**

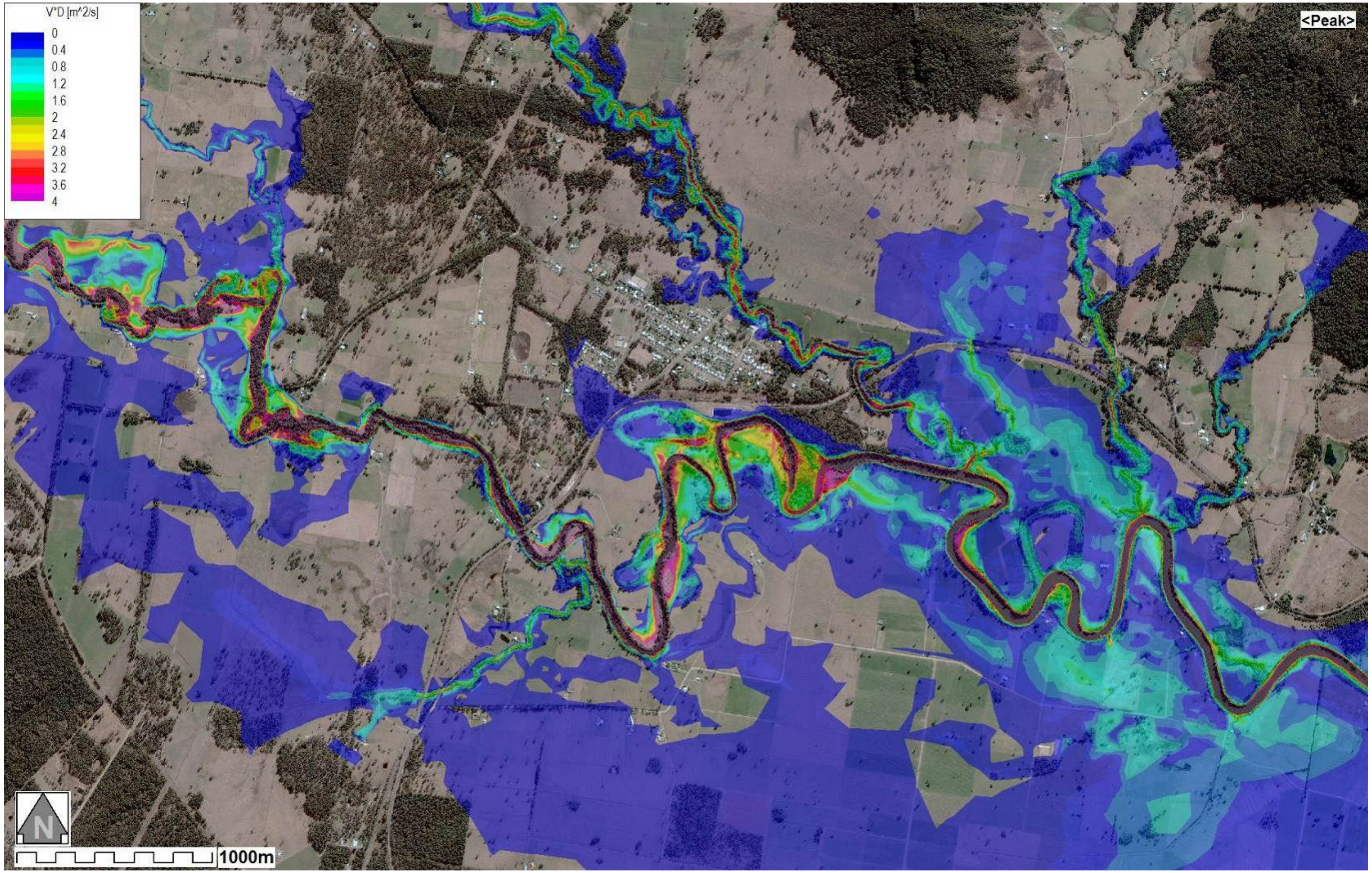


**Figure 44: 1% AEP Design Flood; Velocity times Depth Coloured**

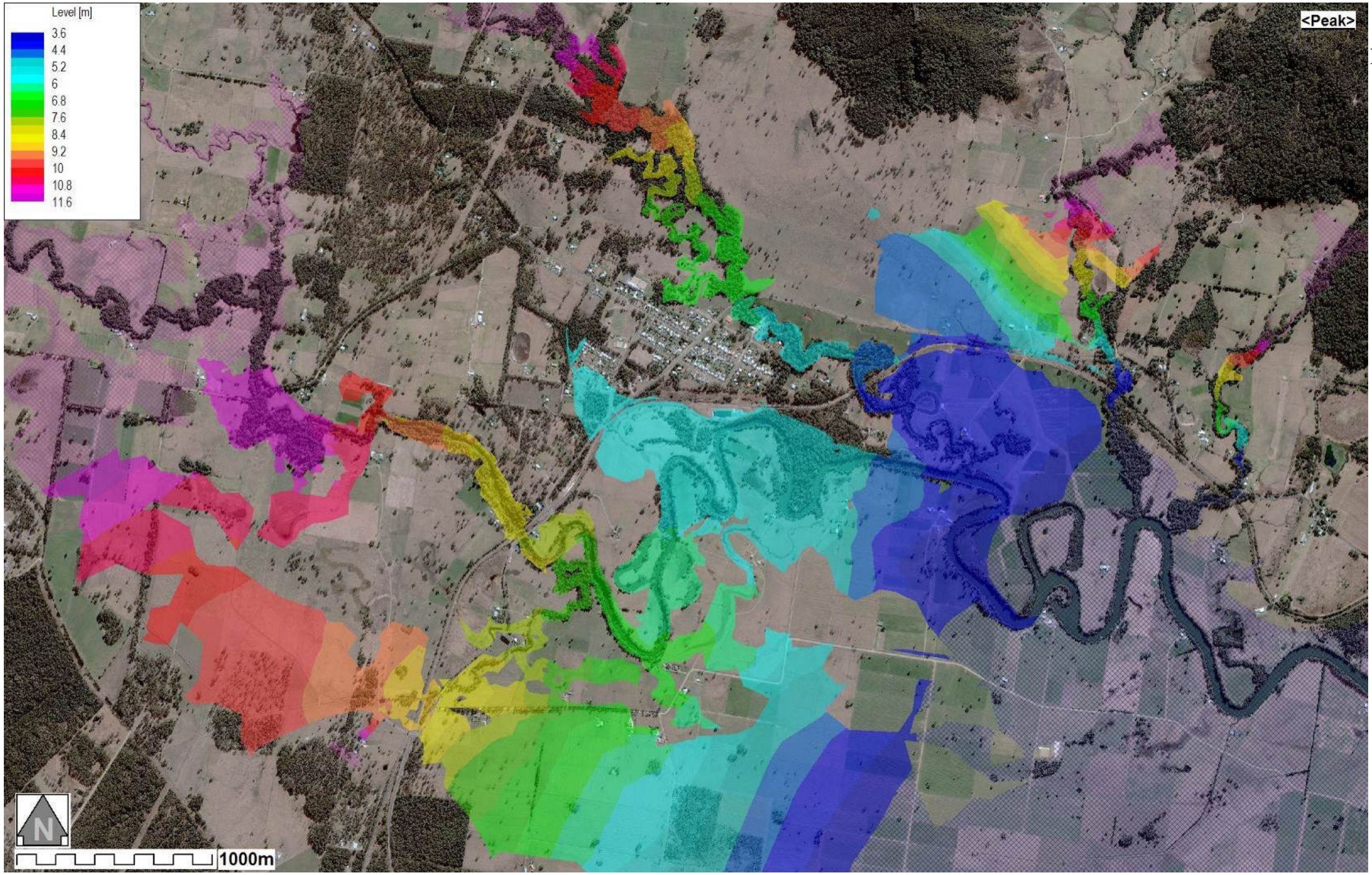


**Figure 45: 1% AEP Design Flood; Water Level Coloured**

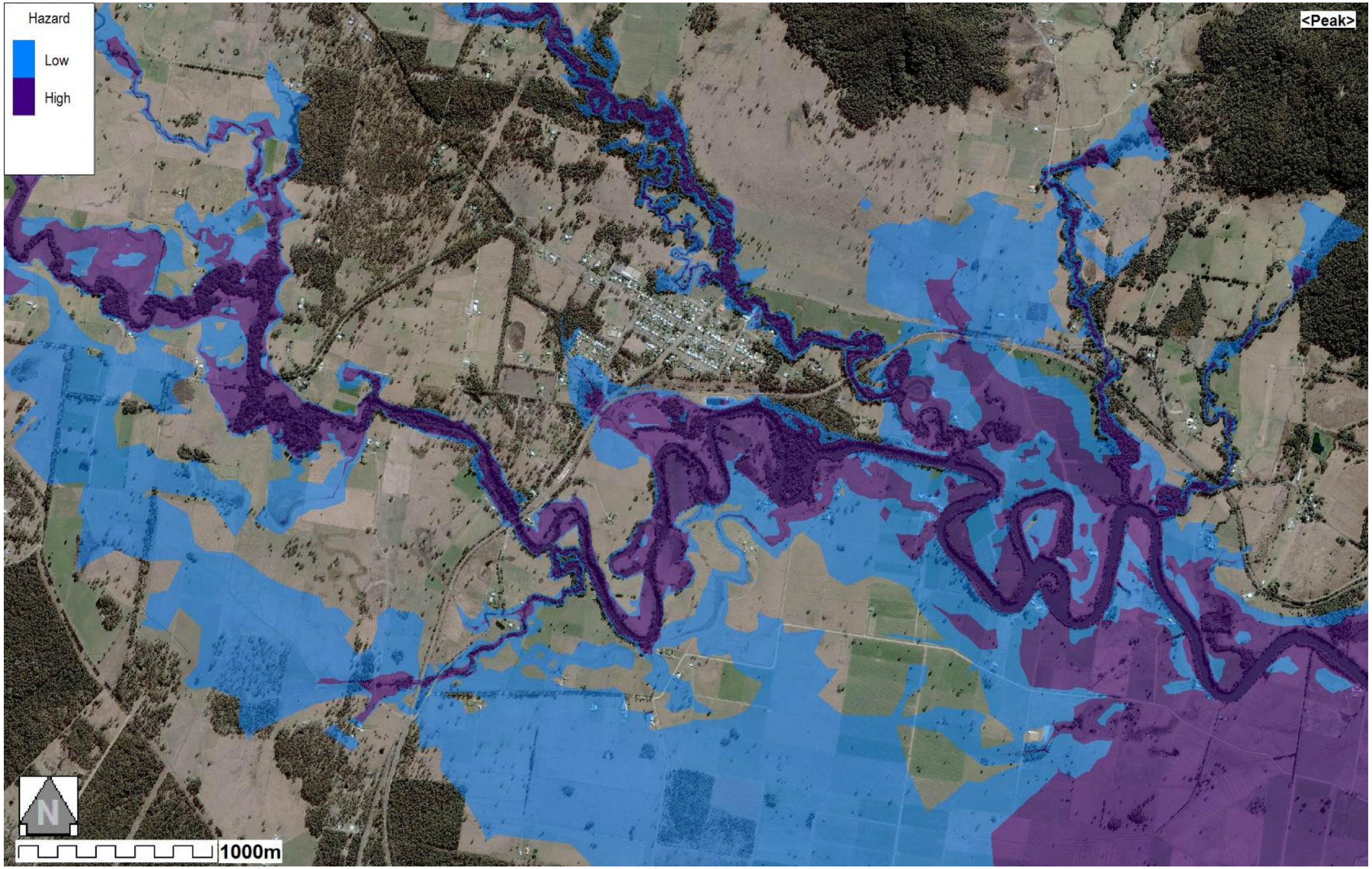


**Figure 46: 1% AEP Design Flood; Preliminary Hydraulic Hazard Coloured**

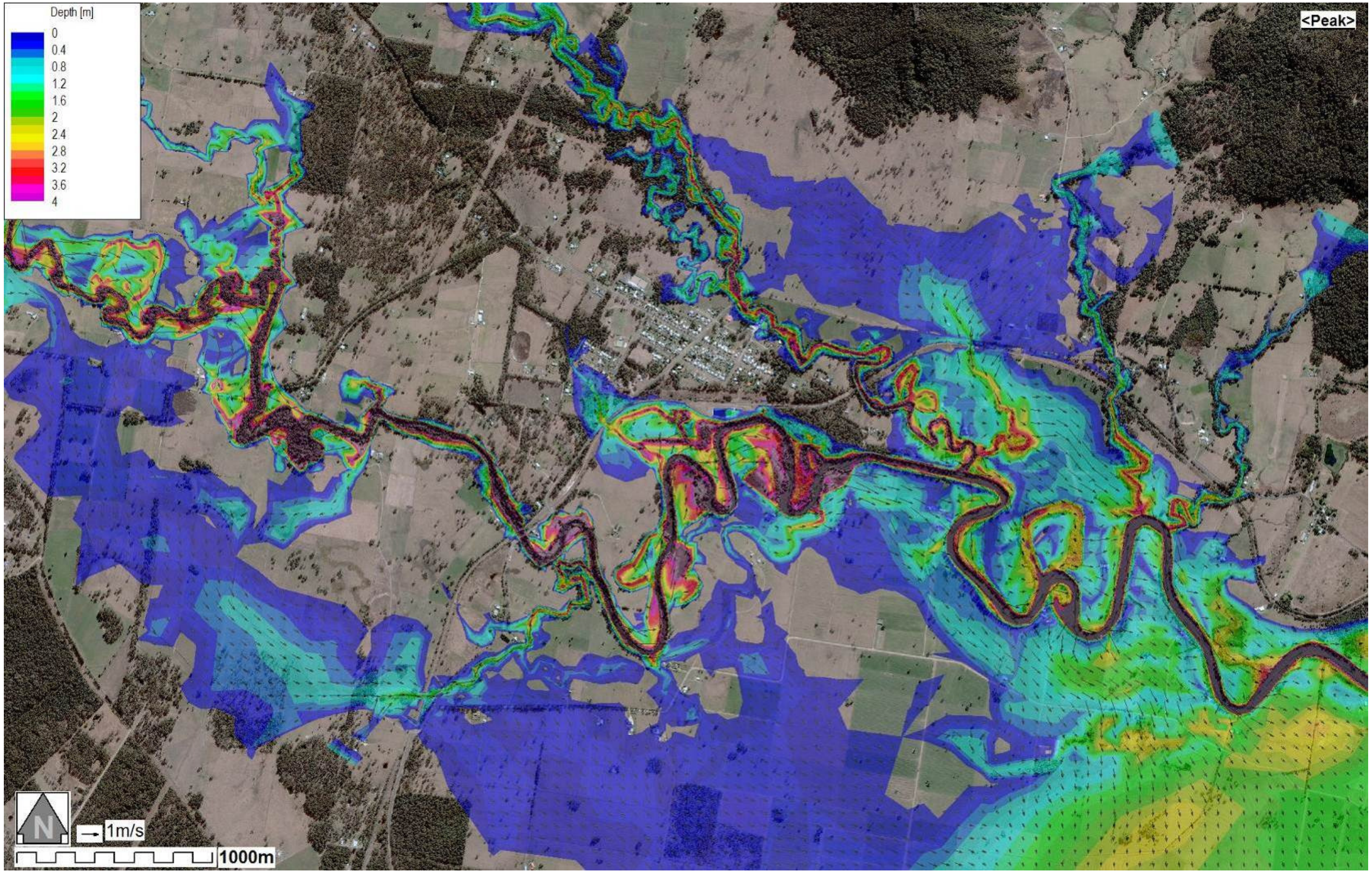


**Figure 47: 0.5% AEP Design Flood; Depth Coloured with Velocity Vectors**

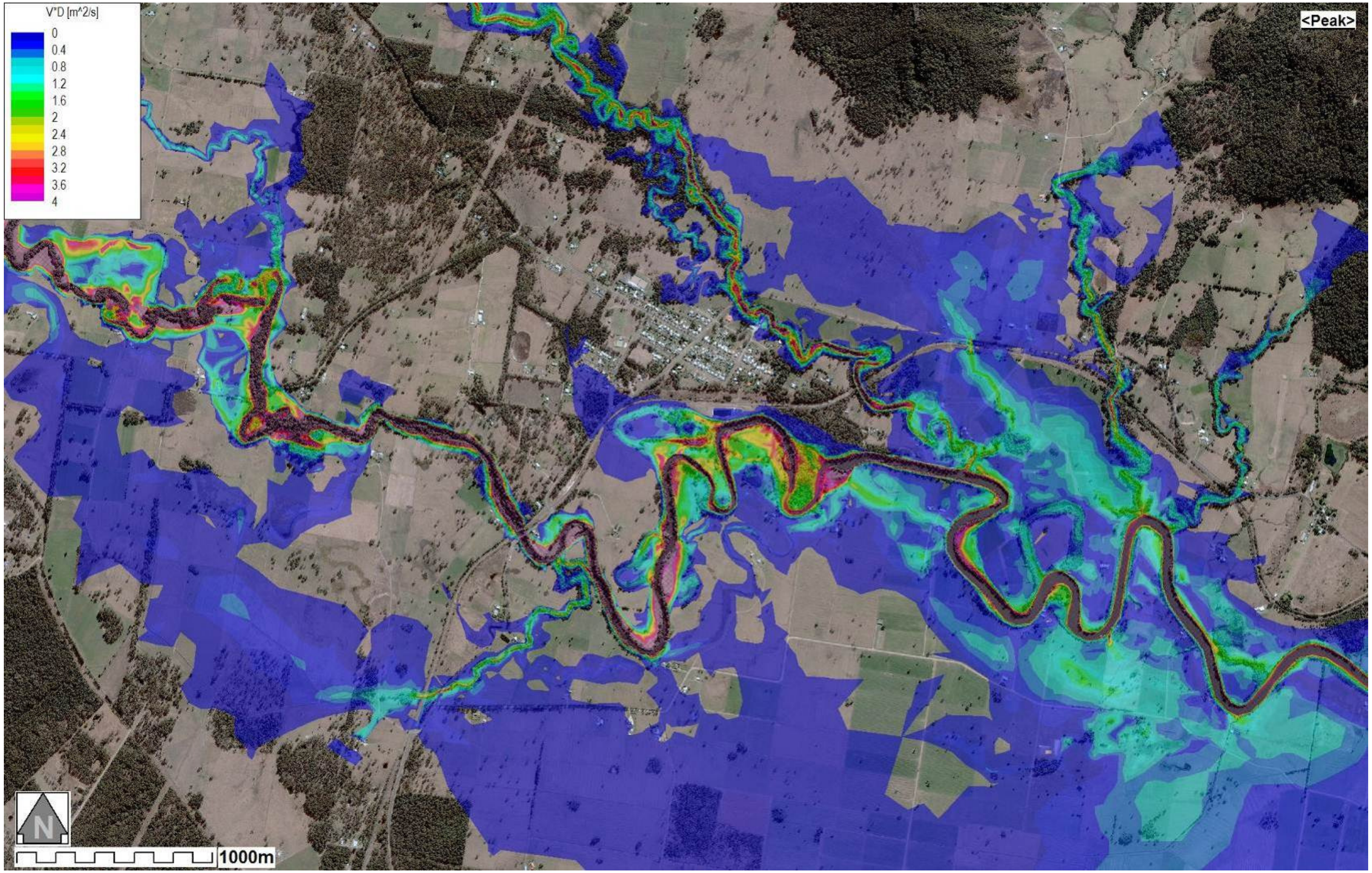


**Figure 48: 0.5% AEP Design Flood; Velocity times Depth Coloured**

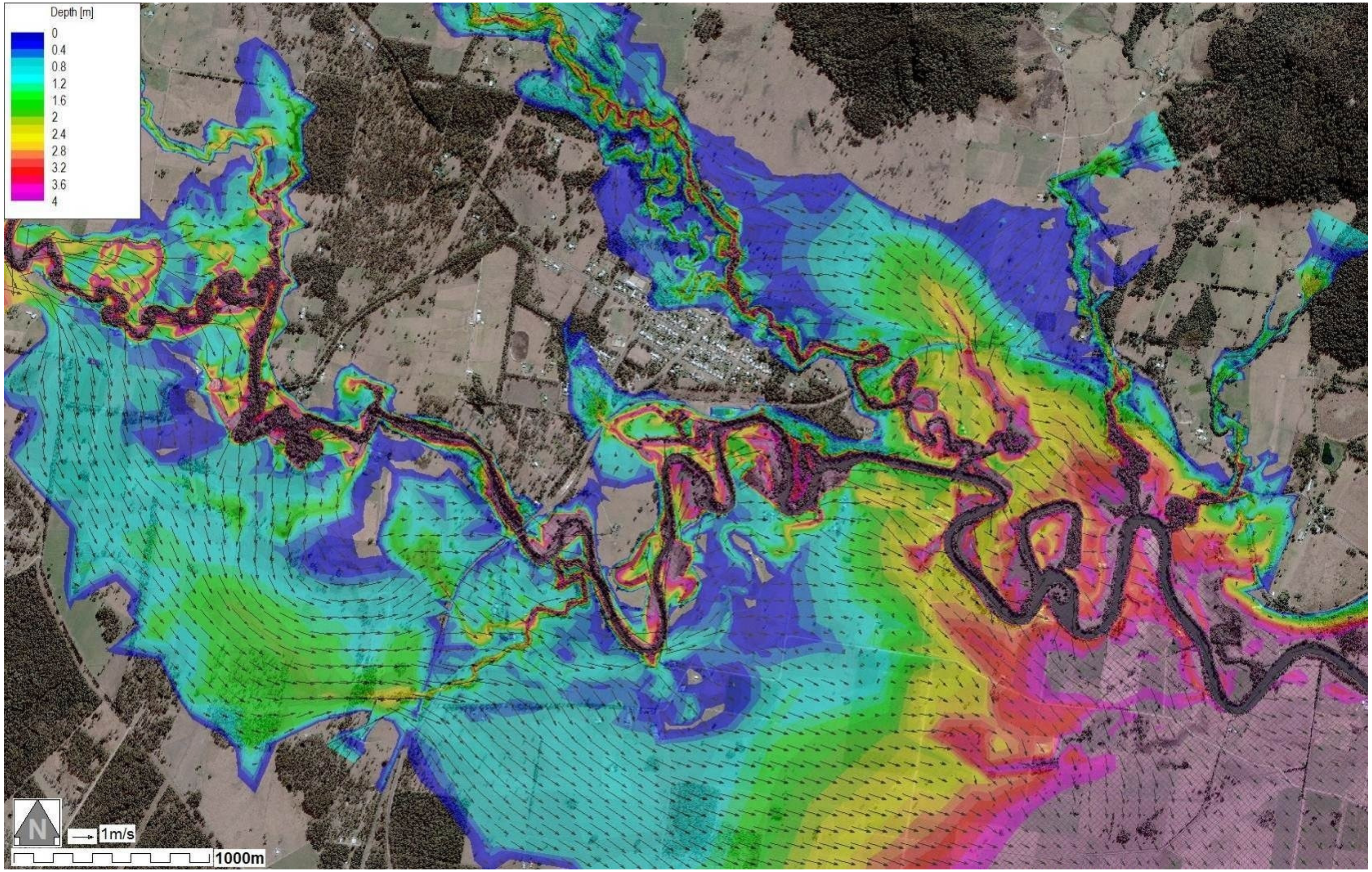


**Figure 49: PMF Design Flood; Depth Coloured with Velocity Vectors**

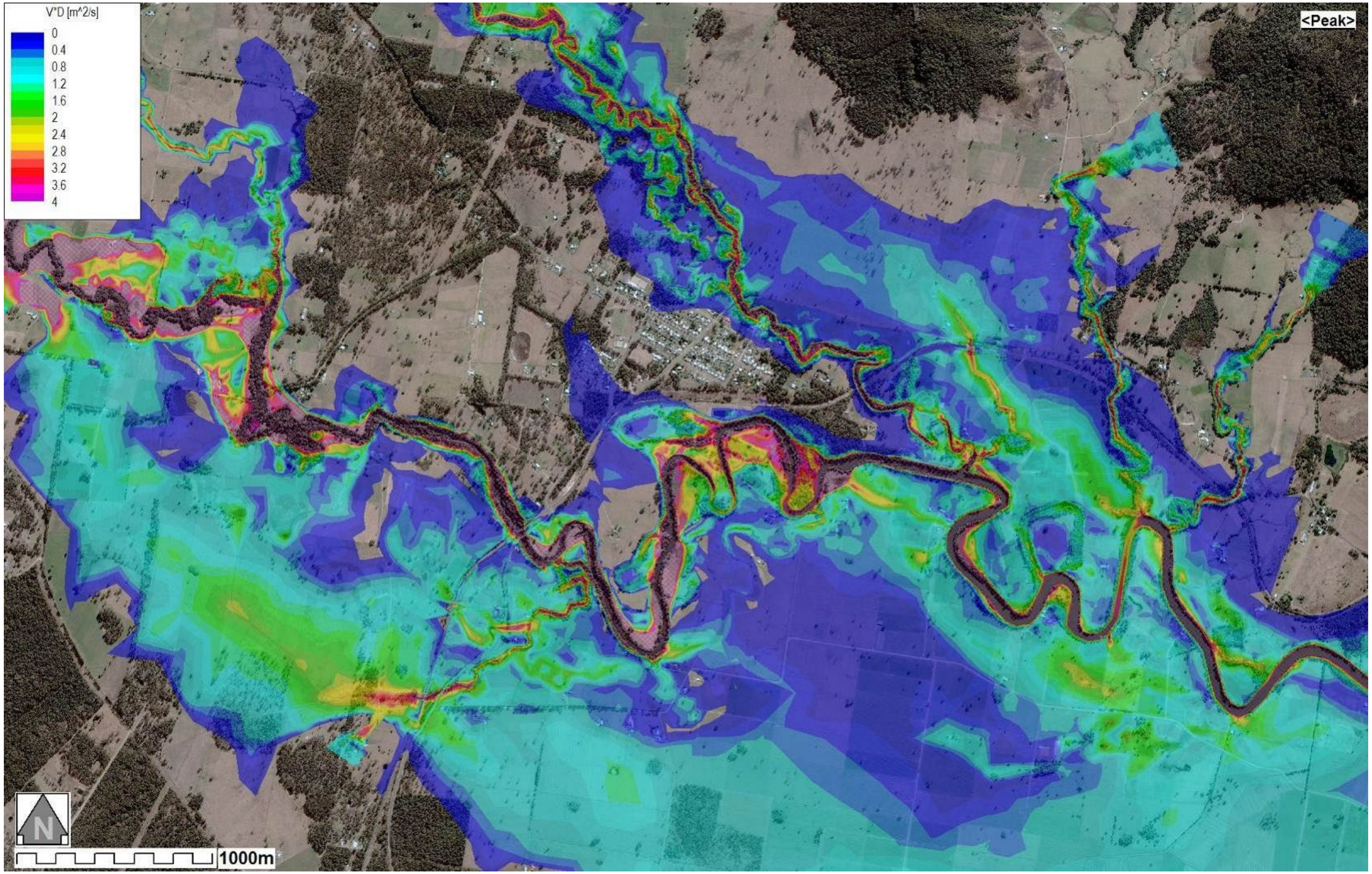


**Figure 50: PMF Design Flood; Velocity times Depth Coloured**

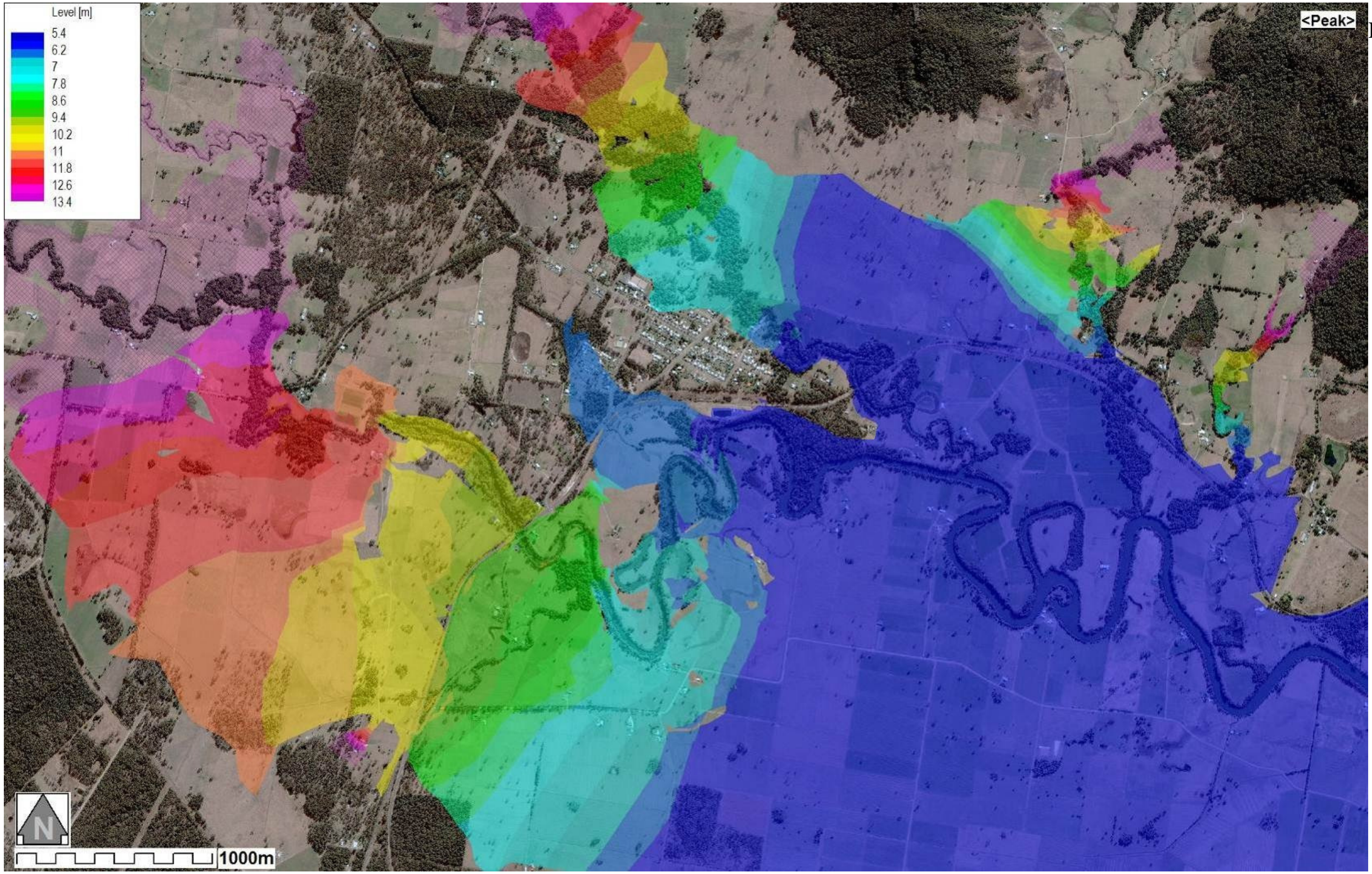


**Figure 51: PMF Design Flood; Water Level Coloured**

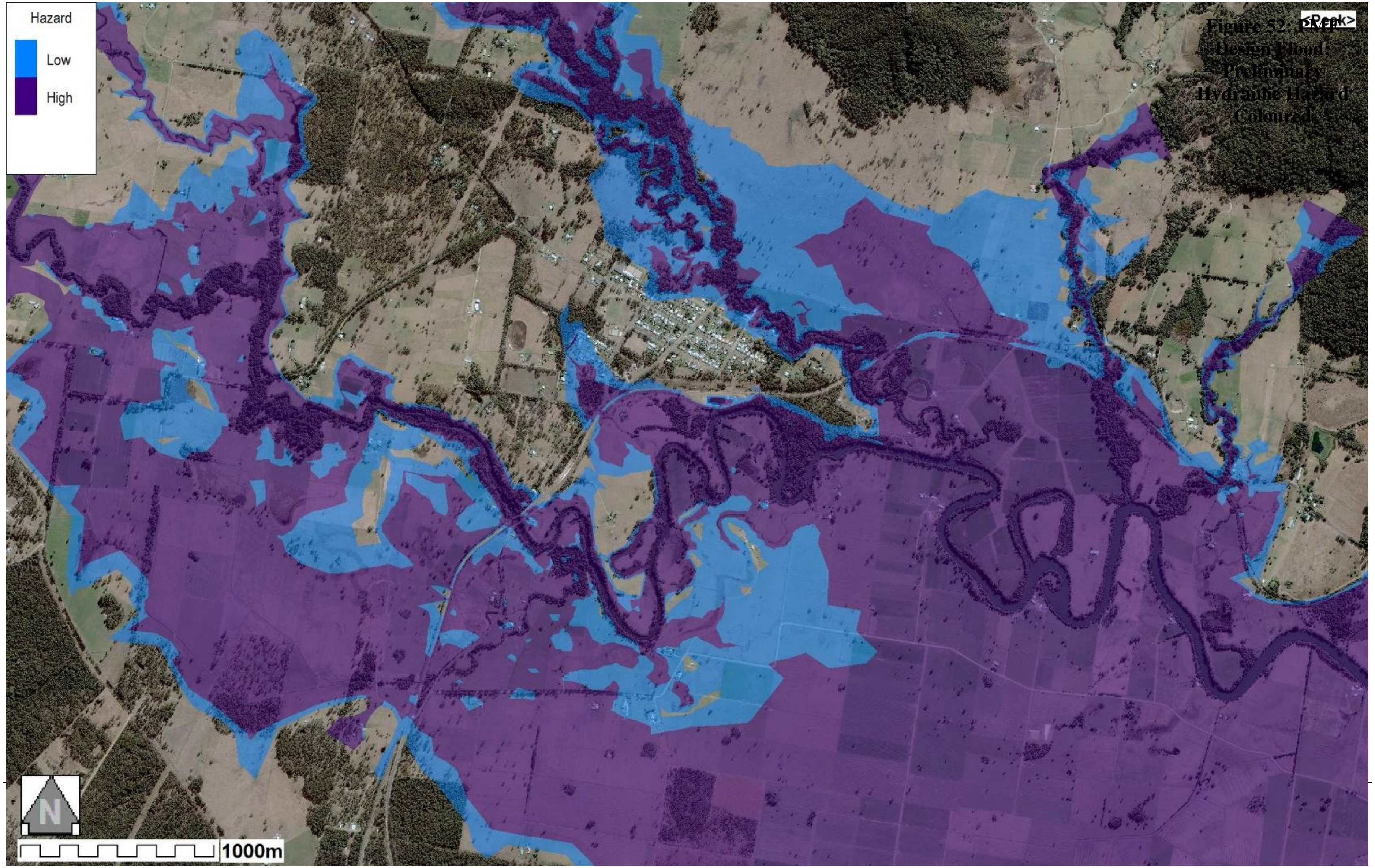


Figure 52: PMF Design Flood; Preliminary Hydraulic Hazard Coloured

# 7. REFERENCES

1. The Institute of Engineers, Australia; "Australian Rainfall and Runoff", Volume One and Volume Two, 1998
2. The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method; Bureau of Meteorology, Commonwealth of Australia, June 2003
3. Department of Infrastructure, Planning and Natural Resources, NSW; "Floodplain Development Manual"; April 1995
4. Public Works Department, NSW, WBM Oceanics; "Manning River Flood Study", Volume One and Volume Two; November 1991
5. Public Works Department, NSW; "Manning River Flood History 1831-1979"; October 1981
6. Greater Taree City Council, Willing & Partners, ERM Mitchell & McCotter, WBM Oceanics; "Draft Manning River Floodplain Management Study", Volume 1 and Volume 2; May 1996
7. Public Works Department, NSW; "Report on the March 1978 Flood – Manning River"; 1978
8. Public Works Department, NSW; "Manning River Flood Stage Hydrographs", January 1981
9. Manning River Times; Various newspaper editions
10. Department of Infrastructure, Planning & Natural Resources, NSW; "PINNEENA Version 8, New South Wales Surface Water Data Archive", 2004
11. Bureau of Meteorology, Commonwealth of Australia; Various data
12. Manly Hydraulics Laboratory, Public Works Department, NSW; Various data
13. NSW Office of Water; Various data
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. Laurie, Montgomerie & Pettit; "New South Wales Coastal Rivers Flood Plain Management Studies Summary Report Manning Valley; October 1980
17. WBNM runoff routing parameters for south and eastern Australia; Boyd, Bodhinayake; Australian Journal of Water Resources Vol 10 No 1; IEAUST 2006
18. Australian Rainfall & Runoff "Project 11: Blockage of Hydraulic Structures, Stage 2 Report", February 2013

# Appendix A – Tabulated Hydrograph Data

**TableA1: Flood levels (in m AHD) for selected chainages and locations along Lansdowne River**

| Chain. | Notable Location | 50% AEP | 20% AEP | 10% AEP | 5% AEP | 2% AEP | 1% AEP | 0.5% AEP | PMF AEP |
|---|---|---|---|---|---|---|---|---|---|
| **0 m** | | 10.82 | 11.54 | 11.83 | 12.15 | 12.38 | 12.54 | 12.67 | 13.53 |
| **20 m** | Confluence of Haywards Gully | 10.77 | 11.51 | 11.80 | 12.14 | 12.37 | 12.54 | 12.67 | 13.52 |
| **1000 m** | | 8.73 | 9.72 | 10.19 | 10.73 | 11.12 | 11.39 | 11.57 | 12.31 |
| **1500 m** | Lansdowne River Gauge | 8.45 | 9.52 | 10.02 | 10.60 | 11.01 | 11.28 | 11.47 | 12.17 |
| **2000 m** | | 7.33 | 8.35 | 8.84 | 9.39 | 9.78 | 10.07 | 10.29 | 11.23 |
| **3000 m** | | 5.95 | 6.83 | 7.27 | 7.77 | 8.09 | 8.34 | 8.52 | 9.53 |
| **3050 m** | Railway Bridge | 5.86 | 6.69 | 7.09 | 7.54 | 7.85 | 8.08 | 8.26 | 9.32 |
| **4000 m** | | 5.14 | 5.87 | 6.18 | 6.51 | 6.70 | 6.83 | 6.92 | 7.66 |
| **5000 m** | | 4.47 | 5.16 | 5.45 | 5.82 | 5.95 | 6.06 | 6.13 | 6.56 |
| **6000 m** | | 4.31 | 5.00 | 5.28 | 5.59 | 5.72 | 5.81 | 5.87 | 6.26 |
| **6400 m** | Wastewater Plant | 4.11 | 4.78 | 5.06 | 5.42 | 5.55 | 5.65 | 5.71 | 6.11 |
| **7000 m** | | 4.01 | 4.69 | 4.95 | 5.24 | 5.37 | 5.46 | 5.52 | 5.96 |
| **8000 m** | | 3.47 | 3.84 | 3.92 | 4.24 | 4.31 | 4.42 | 4.47 | 5.96 |
| **8200 m** | Confluence of Cross Creek | 3.46 | 3.85 | 3.95 | 4.11 | 4.17 | 4.29 | 4.34 | 5.96 |
| **9000 m** | | 2.89 | 3.24 | 3.35 | 3.66 | 3.73 | 3.82 | 3.88 | 5.96 |
| **10000 m** | | 2.62 | 2.92 | 3.03 | 3.17 | 3.24 | 3.36 | 3.42 | 5.96 |
| **11000 m** | Confluence of Rock Creek | 2.03 | 2.36 | 2.49 | 2.86 | 2.98 | 3.16 | 3.23 | 5.96 |
| **12000 m** | | 1.69 | 1.92 | 2.05 | 2.47 | 2.70 | 2.91 | 3.14 | 5.96 |
| **13000 m** | | 1.50 | 1.73 | 1.90 | 2.45 | 2.69 | 2.90 | 3.14 | 5.96 |
| **14000 m** | | 1.33 | 1.61 | 1.84 | 2.45 | 2.69 | 2.90 | 3.13 | 5.96 |
| **15000 m** | | 1.08 | 1.45 | 1.80 | 2.44 | 2.68 | 2.89 | 3.13 | 5.96 |

| Chain. | Notable Location | 50% AEP | 20% AEP | 10% AEP | 5% AEP | 2% AEP | 1% AEP | 0.5% AEP | PMF AEP |
|---|---|---|---|---|---|---|---|---|---|
| **16000 m** | | 1.00 | 1.43 | 1.79 | 2.44 | 2.68 | 2.89 | 3.13 | 5.95 |
| **17000 m** | | 1.00 | 1.41 | 1.78 | 2.43 | 2.68 | 2.89 | 3.13 | 5.95 |
| **18000 m** | | 1.00 | 1.40 | 1.78 | 2.43 | 2.68 | 2.88 | 3.13 | 5.95 |
| **19000 m** | | 1.00 | 1.38 | 1.76 | 2.42 | 2.67 | 2.87 | 3.12 | 5.94 |
| **19500 m** | Old Coopernook Bridge | 1.00 | 1.37 | 1.75 | 2.41 | 2.66 | 2.87 | 3.11 | 5.93 |
| **20000 m** | | 0.00 | 0.00 | 0.00 | 2.39 | 2.64 | 2.85 | 3.09 | 5.91 |