TARCUTTA, LADYSMITH AND URANQUINTY FLOOD STUDIES

DEVELOPMENT AND TESTING OF FLOOD MODELS

VOLUME 1 - REPORT

FINAL REPORT

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FOREWORD

The State Government's Flood Policy is directed at providing solutions to existing flooding problems in developed areas and to ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

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

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

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

The Tarcutta, Ladysmith and Uranquinty Flood Studies are jointly funded by Wagga Wagga City Council and the NSW/Commonwealth Governments, via the Office of Environment and Heritage, Department of Premier and Cabinet. The Flood Studies constitute the first stage of the Floodplain Risk Management process for the villages and have been prepared for Wagga Wagga City Council to define flood behaviour under current conditions.

The Flood Studies have been prepared under the guidance of the Floodplain Management Committee comprising representatives from Wagga Wagga City Council, the Office of Environment and Heritage, Department of Premier and Cabinet and the Consultant, NSW State Emergency Service and Community Representatives from the three villages.

ACKNOWLEDGEMENT

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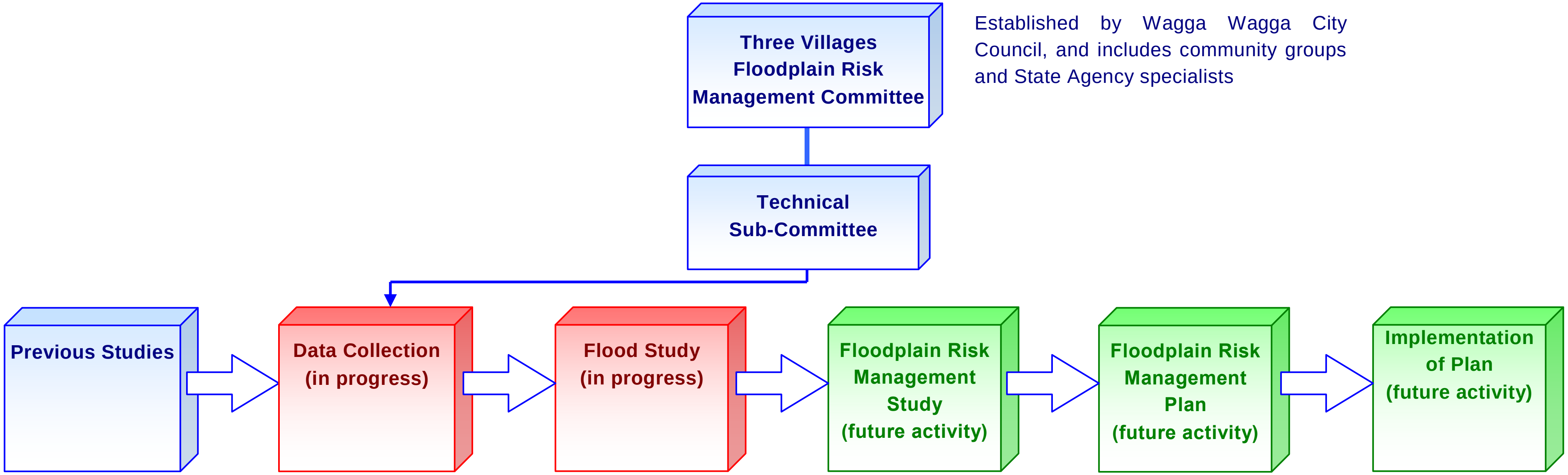
FLOODPLAIN RISK MANAGEMENT PROCESS



The NSW State Emergency Service commissioned studies to capture flood intelligence following the October 2010 and March 2012 floods. Several studies were also undertaken during the planning and detail design of the Tarcutta Bypass (Hume Highway Upgrade).

Involves the compilation of existing data and the collection of additional data.

Involves detailed hydrologic and hydraulic modelling of the Tarcutta Creek, Kyeamba Creek and Sandy Creek catchments.

The Floodplain Risk Management Study will determine options which will seek to reduce the impact of flooding on the community in consideration of social, ecological and economic factors.

Preferred floodplain management options will be publicly exhibited and the responses from the community incorporated in the Plan. The Plan will then be formally approved by Council following the public exhibition period.

Implementation of the Plan will allow Council to reduce the impact of flooding on the community through flood, property, and response modification measures. The measures may include structural works, planning controls, flood warnings, flood readiness and response plans, ongoing data collection and monitoring.

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NOTE ON FLOOD FREQUENCY

The frequency of floods is generally referred to in terms of their Annual Exceedance Probability (**AEP**) or Average Recurrence Interval (**ARI**). For example, for a flood magnitude having 5% AEP, there is a 5% probability that there will be floods of greater magnitude each year. As another example, for a flood having a 5 year ARI, there will be floods of equal or greater magnitude once in 5 years on average. The approximate correspondence between these two systems is:

ANNUAL EXCEEDANCE PROBABILITY (AEP) %	AVERAGE RECURRENCE INTERVAL (ARI) YEARS
0.2	500
0.5	200
1	100
2	50
5	20
10	10
20	5

The report also refers to the Probable Maximum Flood (**PMF**). This flood occurs as a result of the Probable Maximum Precipitation (**PMP**). The PMP is the result of the optimum combination of the available moisture in the atmosphere and the efficiency of the storm mechanism as regards rainfall production. The PMP is used to estimate PMF discharges using a model which simulates the conversion of rainfall to runoff. The PMF is defined as the limiting value of floods that could reasonably be expected to occur. It is an extremely rare flood, generally considered to have a return period greater than 1 in 10^5 years.

ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff
BOM	Bureau of Meteorology
DEM	Digital Elevation Model
DPIOW	Department of Primary Industries - Office of Water
FCV	Flow Constriction Value
FDM	Floodplain Development Manual, 2005
GEV	General Extreme Value
LHS	Left Hand Side
LP3	log-Pearson Type 3
OEH	Office of Environment and Heritage, Department of Premier and Cabinet (formerly Department of Environment, Climate Change and Water [DECCW])
RFA	Request for Assistance
RIA	Rapid Impact Assessment
RHS	Right Hand Side
RMS	Roads and Maritime Services (formerly Roads and Traffic Authority)
NSWSES	New South Wales State Emergency Service
WWCC	Wagga Wagga City Council

Chapter 6 of the report contains definitions of flood-related terms used in the study.

S1 SUMMARY

S1.1 General

This report, *Tarcutta, Ladysmith and Uranquinty Flood Studies – Development and Testing of Flood Models* is the second of the three reports dealing with the flood studies project and presents the results of the development and testing of the hydrologic and hydraulic models set up to assess historic flooding patterns at the three villages. The objective of the *Tarcutta, Ladysmith and Uranquinty Flood Studies* is to define flood behaviour at the three villages *under present day conditions* for floods ranging between 5 and 500 year ARI, as well as for the PMF.

This report builds on the results of *Tarcutta, Ladysmith and Uranquinty Flood Studies – Data Collection Report* (L&A, 2012), which reviewed previous flood studies undertaken at the villages, as well as rainfall-runoff data available for testing the flood models. Recommendations contained in that report lead to a floor level survey being undertaken in properties which experienced above floor inundation during the October 2010 flood. An inbank cross sectional survey was also recommended at locations where both scour and deposition of bed material was observed to have occurred during the October 2010 flood, as was survey of critical hydraulic structures. Another key recommendation of L&A, 2012 was the development of an "independent" TUFLOW model of the Tarcutta Creek floodplain in lieu of the study adopting the existing TUFLOW model which was developed as part of the investigation and design of the Tarcutta Bypass (Hume Highway Upgrade).

This report deals with the flood models which have been developed for the three villages in a single report, as the area is essentially located in a climatically homogeneous zone, with hydrologic model parameters likely to be transferrable between the catchments after scaling to account for the various catchment areas.

For the purposes of the *Tarcutta, Ladysmith and Uranquinty Flood Studies*, hydrologic models of the study catchments were used to generate flood flows and hydraulic models of the channels and floodplains at each village were used to convert flows into flood levels, flow patterns and velocities. The hydrologic models were based on the RAFTS and DRAINS rainfall-runoff software, while the hydraulic models were based on the TUFLOW two-dimensional modelling system.

Stream flow and telemetered rainfall data were available on both the Tarcutta Creek catchment for defining flooding at Tarcutta and on the Kyeamba Creek system in the case of Ladysmith. Flood data were analysed for several recent historic storms occurring in the last few years to tune the hydrologic models for these catchments. For design flood estimation (which will be discussed in the future, third report of the series *Tarcutta, Ladysmith and Uranquinty Flood Studies – Design Flood Modelling*), the RAFTS hydrologic model parameters for the two gauged catchments will be used as a guide to assigning design parameters for the ungauged Sandy Creek catchment at Uranquinty. Parameters found to apply for the testing of the TUFLOW hydraulic models will guide the selection of parameters for design flood modelling.

S1.2 Scope of Investigation

S1.2.1 Tarcutta

L&A, 2012 identified that the Department of Primary Industries – Office of Water's (**DPIOW's**) rating curve for its Old Borambola gauging station, which is located approximately 21 km downstream of Tarcutta on Tarcutta Creek, underestimates the peak discharge for out-of-bank floods. The reason for the underestimation of peak flows is attributed to the high flow portion of the rating curve having been estimated by "eye" rather than by accepted calculations.

The high flow portion of the rating curve at Old Borambola was therefore re-assessed as part of the present investigation using hydraulic modelling of the floodplain based on the UNET software. In order to assess the potential attenuation of flows the modelling extended from Tarcutta to the gauging station and the model was run in unsteady mode. This approach yielded a more accurate estimate of attenuation of flow than the alternative procedure of using the channel routing feature contained in the RAFTS software. RAFTS was used to estimate discharge hydrographs at the boundaries of the TUFLOW hydraulic model of the floodplain.

A flood frequency analysis was undertaken using the 74 year period of record which is available for the Old Borambola stream gauge. The analysis was undertaken using both the recorded inbank and adjusted out-of-banks flows. The findings of the flood frequency analysis were used to assign an indicative ARI to the historic floods at Tarcutta.

The TUFLOW model of the Tarcutta Creek floodplain was prepared using LiDAR survey data provided by Wagga Wagga City Council (**WWCC**). Conditions on the floodplain at the time of the October 2010 and March 2012 floods were also taken into account, namely in relation to the Tarcutta Bypass (Hume Highway Upgrade), which was under construction at the time of the earlier flood event; as well as observed changes in riparian vegetation between the two floods. Survey of the road corridor undertaken in late 2010 and the road design model were respectively used to adjust the geometry of the TUFLOW model so as to represent contemporaneous floodplain conditions at the time of the October 2010 and March 2012 floods. Field survey of the channel and floodplain of Tarcutta Creek downstream of Sydney Street was also commissioned as part of the present investigation to incorporate changes in channel dimensions and floodplain levels which occurred during the October 2010 flood.

A base-flow separation analysis was undertaken for the March 2010, October 2010, December 2010 and March 2012 storm events, after which the RAFTS model, in combination with the aforementioned UNET modelling approach, was successfully tuned to reproduce recorded flow at the Old Borambola stream gauge (GS 410047) as derived from the upgraded rating curve. Results of the RAFTS model testing process are reported in **Section 3.4.1**. The TUFLOW flood model of Tarcutta Creek was then successfully tuned to reproduce recorded level data and observed flood behaviour for the October 2010 and March 2012 floods. Results of the TUFLOW model testing process are reported in **Section 4.3** of this report.

S1.2.2 Ladysmith

The high flow portion of the rating curve for the gauging station on Kyeamba Creek at Ladysmith (GS 410048) had also been estimated by "eye" and was also refined using hydraulic modelling. In this case, the TUFLOW one and two-dimensional modelling of the floodplain in the vicinity of the gauging station was used. The adjusted rating curve incorporates the effects of major scour which is believed to have occurred at one of the openings in the disused railway line during the October 2010 flood.

A flood frequency analysis could not be undertaken for the Ladysmith stream gauge similar to that which was undertaken for the Old Borambola stream gauge as there are too many gaps in the available record.

A base-flow separation analysis was undertaken and the RAFTS model tuned in combination with the TUFLOW flood model to reproduce recorded level data along Kyeamba Creek for the October 2010 and March 2012 storm events.

Results of the RAFTS and TUFLOW model testing process are reported in **Sections 3.4.2** and **4.4** of this report, respectively.

S1.2.3 Uranquinty

There are no stream gauges located in the Sandy Creek catchment. The RAFTS model parameters which were found to apply for the two gauged catchments were adopted when deriving discharge hydrographs for the historic floods at Uranquinty. A frequency analysis of recorded peak flows, which would have allowed an indicative ARI to be assigned to the historic floods, could not be undertaken given the absence of a stream gauge.

The TUFLOW hydraulic model of the Sandy Creek floodplain was then successfully tuned to reproduce flood levels for the October 2010 and March 2012 floods.

Results of the RAFTS and TUFLOW model testing process are reported in **Sections 3.4.3** and **4.5** of this report, respectively.

S1.3 Key Findings

S1.3.1 Tarcutta

The key findings of the model development and testing phase of the study as they relate to Tarcutta were as follows:

- DPIOW's current rating curve for the Old Borambola stream gauge (GS 410047) was adjusted by fitting a second order polynomial equation (refer ***Equation 2.1*** in **Section 2.5.1**) to the stage versus discharge relationship generated by the UNET software for gauge heights above 4.05 m. The current and adjusted rating curves are shown on the LHS of **Figure 2.11**.
- A flood frequency analysis was undertaken using both recorded and adjusted annual peak discharge data for the Old Borambola stream gauge (refer **Annexure C** for details) **Figures 2.12** and **2.13** show the results of the flood frequency analysis. Based on the findings of that analysis, the major flood that occurred in October 2010 had an equivalent ARI of between 50 and 70 years, whilst the December 2010 flood had an equivalent ARI of between 22 and 30 years. The March 2012 flood had an equivalent ARI of between 18 and 22 years (refer **Section 2.6.1** for details).
- Reasonable correspondence between recorded and modelled stage and discharge hydrographs was achieved at the Old Borambola stream gauge for all four historic floods after the discharge hydrographs generated by the RAFTS model were routed from Tarcutta to the gauge site using the UNET software.
- There is a minor attenuation of the flood wave as it travels the 21 km from Tarcutta to the Old Borambola stream gauge.
- The response time of the catchment following the onset of heavy rainfall did not vary greatly for the three floods which occurred in 2010, when it took between 18-20 hours for flows in Tarcutta Creek to peak at Tarcutta following the onset of heavy rain. The same was not the case for the March 2012 flood, where flows did not peak at Tarcutta until 24 hours after the onset of heavy rain. The reason for this is attributed to a band of heavy rain, which by inspection of the Adelong (Etham Park) and Batlow rain gauges moved in generally a southerly direction (i.e. in an upstream direction) on the rain day of 4 March 2012 (i.e. the 24 hours prior to 09:00 hours on 4 March), with the heavier falls in the upper reaches of the catchment occurring further into the storm event.

- The peak height recorded at the Tarcutta Depth gauge in the March 2012 flood was 10 mm higher than was reached in the December 2010 flood, even though the peak flow in Tarcutta Creek for the December 2010 flood was about 100 m^3/s higher than in the March 2012 flood. The reason for the difference in peak flood heights is attributed to an increase in the density of vegetation on the floodplain downstream of the Sydney Street bridges over the intervening 2 year period which had the effect of slowing the velocity of floodwater and thereby increasing peak flood levels.
- The TUFLOW models developed as part of the present investigation were able to broadly reproduce observed flood behaviour for both the October 2010 and March 2012 floods. The TUFLOW model was found to underestimate the peak flood level at the Tarcutta Depth Gauge by about 200 mm for the October 2010 flood.[1] The reason for this is attributed to the major scour that occurred in the main channel of Tarcutta Creek during the flood event and the fact that the hydraulic model incorporates the cross sectional area of the creek after the occurrence of the scour. The results of the TUFLOW modelling are shown on **Figures 4.5** to **4.7**.

S1.3.2 Ladysmith

The key findings of the model development and testing phase of the study as they relate to Ladysmith were as follows:

- DPIOW's current rating curve for the Ladysmith stream gauge (GS 410048) was adjusted by fitting a series of second, third and fourth order polynomial equations (refer ***Equation 2.2*** to ***2.8*** in **Section 2.5.2**) to the stage versus discharge relationship generated by the TUFLOW software. **Figure 2.11** (RHS) shows both the current and adjusted rating curves which represent both pre- and post-railway scour conditions. .
- There are too many gaps in the stream flow record for a flood frequency analysis to be undertaken for Kyeamba Creek at Ladysmith. As a result, it is not possible at this stage of the investigation to assign an approximate ARI to the four most recent historic floods, as the findings of the future design flood modelling will need to be used for this purpose.
- Tuning of the Kyeamba Creek flood models was limited to the October 2010 and March 2012 floods as by inspection of the flows recorded by the Ladysmith gauge (refer **Figure 3.4**), there appears to be significant attenuation and prolongation of the flood wave occurring upstream of the village during minor flood events (a feature which cannot be reproduced by the RAFTS model which uses a simple time lag approach to routing the flood hydrograph down the valley).
- The time between the recorded peaks at the Book Book and Ladysmith stream gauges computed by the RAFTS model was 3 and 4 hours for the October 2010 and March 2012 floods, respectively. These compare closely with the recorded data of 5 hours (October 2010 flood) and 2 hours 45 minutes (March 2012 flood). The minor difference in the recorded and modelled times likely lies in the temporal variability of the rainfall across parts of the catchment which was not captured by BOM's network of rain gauges (and hence not incorporated in the RAFTS model).
- The TUFLOW models developed as part of the present investigation were able to reproduce observed flood behaviour for both the October 2010 and March 2012 floods. TUFLOW model results for the two historic floods are shown on **Figures 4.8** to **4.10**.

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[1] As the waterway area in Tarcutta Creek at the time of the flood peak cannot be determined with any confidence, the inbank survey commissioned as part of the present investigation (i.e. post the October 2010 flood) has been used in the development of the hydraulic model.

- Whilst existing development in the village was not impacted by main stream flooding, the modelling indicates that properties located to the south of Tywong Street may have been impacted by relatively shallow overland flow which crossed Tumbarumba Road to their east. Furthermore, depths of overland flow in the village are likely to have been greater in the March 2012 flood than was experienced in the October 2010 flood.

S1.3.3 Uranquinty

The key findings of the model development and testing phase of the study as they relate to Uranquinty were as follows:

- No stream flow data are available for the Sandy Creek catchment. Testing of the Uranquinty flood models was therefore based on the outcomes of the model testing process for both the gauged catchments, in combination with a comparison between modelled and observed flood behaviour for the October 2010 and March 2012 floods.
- In the absence of a flood frequency analysis it is not possible to assign an approximate ARI to the four most recent historic floods. The findings of the future design flood modelling will be used for this purpose.
- It was found that in order to reproduce the time when Deane Street was surcharged by floodwater during both the October 2010 and March 2012 floods, an average flow velocity of 1.0 m/s needed to be applied to the derivation of the lag times in the RAFTS model links.[2] This reduced flow velocity is attributed to the flatter nature of the Sandy Creek catchment when compared to both the Tarcutta Creek and Kyeamba Creek catchments.
- The TUFLOW model developed as part of the present investigation was able to reproduce observed flood behaviour for both the October 2010 and March 2012 floods, especially the depth to which water ponded behind the existing flood protection levee. TUFLOW model results for the two historic floods are shown on **Figures 4.11** to **4.13**.
- Whilst peak flood levels on the southern (upstream) side of Deane Street were only slightly lower than occurred in the October 2010 flood (RL 201.65 m AHD in October 2010 versus RL 201.59 m AHD in March 2012), there was a significant reduction in the peak flow which surcharged Deane Street west of Connorton Street. Whereas a peak discharge of about 20 m^3/s is estimated to have surcharged Deane Street in the October 2010 flood, only about 6 m^3/s is estimated to have surcharged the roadway in March 2012. This result highlights the major impact minor differences in peak flood levels can have on flooding conditions in existing development which lies behind the levee bank.

S1.4 The Next Step

The next step in the flood studies will involve the derivation of design flood hydrographs through the use of the calibrated RAFTS models and their application to the calibrated TUFLOW models, to show flooding behaviour in the three villages for floods with ARI's of between 5 and 500 years, together with the PMF. The findings of that investigation will be incorporated in the report *Tarcutta, Ladysmith and Uranquinty Flood Studies – Design Flood Modelling*. That future report will also include the findings of several sensitivity studies, as well as figures showing the division of the floodplain into provisional flood hazard and hydraulic categories for the 100 year ARI event. Figures showing the extent of the Interim Flood Planning Area, which will be set equal to the 100 year ARI peak flood level plus 500 mm freeboard, will also be presented.

[2] An average flow velocity of 1.5-1.8 m/s was found to give good correspondence with the recorded stream flow data on the two gauged catchments.

1 INTRODUCTION

1.1 Approach to Flood Modelling

Flood behaviour was defined using computer based hydrologic models of the catchments and hydraulic models of the creeks and their respective floodplains. The hydrologic model was a rainfall-runoff routing model based on the RAFTS software which converted historic storm rainfalls to discharge hydrographs from the rural parts of the study area. Flows derived from the sub-catchments of the urban areas of the villages, which are drained by sections of open channels and pipes were defined using the DRAINS software.

A dynamic hydraulic modelling approach was adopted for the analysis to account for the time varying effects of flow in the creeks, the routing effects of the floodplain storage and the two-dimensional effects of flow over the floodplain and in the urban parts of the study areas. A depth-averaged, one and two-dimensional free surface flow modelling approach was chosen as it allows for the interaction of flow between the channels and the floodplains, through culverts and over control structures such as road embankments. The TUFLOW hydraulic modelling software was adopted for this purpose.

Historic storm rainfalls were applied to the hydrologic models to generate discharge hydrographs within the study area. These hydrographs constituted the upstream boundaries and lateral flows applied to the hydraulic models.

The models were tested and their parameters tuned using rainfall and flood data which were collected for the historic storms of March 2010, October 2010, December 2010 and March 2012. These storms had been identified in L&A, 2012 as suitable for this purpose due to the availability of 3 hourly rainfall depths recorded by BoM's network of flood warning rain gauges for all four storms, in combination with a large amount of flood intelligence which has been gathered by both NSW State Emergency Service (**NSWSES**) and WWCC, namely for the October 2010 and March 2012 events.

The calibrated models will be used as the basis for defining flood behaviour in the three villages for floods of between 5 and 500 year ARI, together with the PMF.

1.2 Study Tasks

The study had three components:

- ***Additional data collection and analysis, as recommended in the Data Collection Report (L&A, 2012)***. This work involved preparation of a brief for in-bank cross sectional survey along several of the streams, surveying of hydraulic control structures and levelling floor levels. (Casey Surveying and Design undertook the survey, with the data provided in both spreadsheet and CADD format). Hydraulic analysis was also undertaken to refine the high flow rating curves at gauging stations on Tarcutta Creek downstream of Tarcutta and on Kyeamba Creek at Ladysmith to provide better data for tuning the flood models for the flood investigations at those villages.
- ***The hydrologic component*** which included preparation of the hydrologic models of the study catchments, tuning of the models to gauged discharge hydrographs and selection of model parameters for design flood estimation.

- ***The hydraulic component*** which comprised the preparation and testing of hydraulic models of the main streams and floodplain areas and the application of discharge hydrographs to the models to define extents and depths of inundation, water surface profiles, flows and velocities for the design floods.

1.3 Overview of Report

Chapter 2 contains details of the additional data that were collected and analysed as part of the present investigation. This included an inbank survey, as well as the levelling of the floors of several properties which were identified in Bewsher, 2011 as having been inundated by floodwater during the October 2010 flood. Details of adjustments which have been made to the high flow portion of both the Old Borambola and Ladysmith stream gauges to account for deficiencies in their current rating curves are given. The results of a flood frequency analysis which was undertaken using a combination of recorded and adjusted annual peak flows at the Old Borambola stream gauge are also presented.

Chapter 3 is a brief outline of the procedures which were used to generate discharge hydrographs for input to the hydraulic model. This step involved the application of historic storm rainfall depths over the study catchments, and the conversion of the rainfall hyetographs to discharge hydrographs using the RAFTS-DRAINS modelling software. A base flow separation analysis was also undertaken for flows recorded at both the Old Borambola and Ladysmith gauges as part of the model calibration process.

Chapter 4 is a brief outline of the TUFLOW modelling procedure, which was used to route the historic discharge hydrographs determined by RAFTS–DRAINS through the channels and floodplains and define the flood behaviour in the study areas. Also presented in this chapter of the report are the results of the model testing process, including a comparison between flood behaviour that was observed during the October 2010 and March 2010 floods at the three villages with that derived by the flood models.

Chapter 5 contains a list of references, while **Chapter 6** contains a list of flood-related terminology that is relevant to the scope of the study.

2 DATA COLLECTION AND ANALYSIS

2.1 Rainfall and Stream Flow Data

The rainfall and stream flow data that were collected and analysed as part of L&A, 2012 was relied upon for the model development and testing phase of the study. **Figure 2.1** shows the cumulative rainfall which was recorded at the various rain gauges during the historic storms that occurred in March 2010, October 2010, December 2010 and March 2012, whilst **Figures 2.2** to **2.5** show isohyetal rainfall depths for each. Figures showing a comparison between historic rainfalls and design intensity-frequency-duration curves for each gauge site are contained in **Annexure A**.

Stream flow data recorded at DPIOW's Tarcutta Creek at Old Borambola (GS 410047) and Kyeamba Creek at Ladysmith (GS 410048) stream gauges are respectively shown on **Figures 3.2** and **3.4** in **Chapter 3**. **Table 2.1** summarises peak stage data which are contained in Bewsher, 2011 and Yeo, 2013 for stream gauges located in the Tarcutta Creek and Kyeamba Creek catchments.

TABLE 2.1
HISTORIC PEAK STAGE DATA(1)
VALUES IN m

Catchment	Stream Gauge and Station Number	Historic Storm Event			
		March 2010	October 2010	December 2010	March 2012
Tarcutta Creek	Westbrook (GS 410155)	0.68	2.79	3.38	2.38
	Belmore Bridge (GS 410155)	1.23	4.14	4.78	3.78
	Tarcutta Depth Gauge(2)	-	4.49 [-1 to -2]	3.85 [9]	3.86 [-0.5]
	Old Borambola (GS 410047)(3)	4.22	5.42 [8-9]	5.03(4) [8]	4.86 [6-7]
Kyeamba Creek	Book Book (GS 410156)	-	2.75	2.78	3.35
	Ladysmith (GS 410048)(5)	-	6.67 [5:00]	5.17 [4:45]	6.13 [2:45]

1. Source: Bewsher, 2011 and Yeo, 2013
2. Values in [ ] are the difference in the time of the recorded flood peak at the Belmore Bridge and Tarcutta stream gauges.
3. Values in [ ] are the difference in the time of the recorded flood peak at the Tarcutta and Old Borambola stream gauges.
4. DPIOW's database of levels gives peak recorded stage as 4.95 m. The peak stage contained in DPIOW's database has been used for model testing purposes.
5. Values in [ ] are the difference in the time of the recorded flood peak in hours and minuted (hh:mm) at the Book Book and Ladysmith stream gauges.

2.2 Drainage Layouts

Data collected and compiled as part of L&A, 2012, as well as additional information which was obtained from various sources during the present phase of the study, were used to compile the following figures which show key elements of the drainage system at each village:

- **Figure 2.6**, which shows the key elements of the drainage system in the village of Tarcutta. These include the Hambledon, Tarcutta and Old Tarcutta Inn levees, as well as the twin bridges on both Sydney Street and the Hume Highway. The Hume Highway bridge was under construction at the time of the floods that occurred in 2010.
- **Figure 2.7**, which shows the key elements of the drainage system at Ladysmith, with the main feature that influences flood behaviour on Kyeamba Creek being the disused railway embankment and its twin openings.
- **Figure 2.8**, which shows the key elements of the drainage system at Uranquinty, with the main feature being the network of levees which protect existing development from both main stream flooding and local overland flow.

The hydraulic models developed as part of the present investigation incorporated all of the key features identified in the above mentioned figures. Further details are provided in **Chapter 4**.

2.3 Field Survey Data

Following the completion of the L&A, 2012, a Survey Brief was prepared which included the following:

- Requirements for the survey of several major hydraulic structures, including the Sydney Street bridges on Tarcutta Creek, the disused railway bridges on Kyeamba Creek and the highway and railway culverts at Uranquinty.
- The location of historic flood marks which are identified in Bewsher, 2011 for all three villages. Only the floor level and surrounding ground level at each property was surveyed, with the peak flood level computed by adding to the surveyed floor level, the depth of above floor inundation identified in Bewsher, 2011 (in relation the October 2010 flood) and in Yeo, 2013 (in relation the March 2012 flood).
- The locations where inbank survey was required along Tarcutta Creek downstream of the Sydney Street road bridges. The need for this survey was identified during discussions with local residents who believe that both the scour and deposition of bed material during the October 2010 flood altered the conveyance capacity of Tarcutta Creek.
- Line survey along the top of the disused railway embankment which crosses Kyeamba Creek a short distance downstream of Ladysmith.

Figures A1 to **A3** in **Appendix A** show the extent of field survey undertaken in the three villages.

The Office of Environment and Heritage (**OEH**) provided a survey model which was prepared by the NSW Roads and Maritime Services (**RMS**) along the Hume Highway upgrade project at Tarcutta (also referred to as the Tarcutta Bypass) following the October 2010 flood event. The survey model was used to define ground levels within the road corridor at the time of the flood event. In addition to the ground model, OEH also provided details of several flood marks which were surveyed by RMS in Tarcutta immediately following the October 2010 flood.

During the course of the present investigation WWCC engaged a surveyor to level the three depth gauge markers that are located on the western (downstream) side of Sydney Street immediately north of Bridge No. 1 (refer **Figure 2.6** for location). A photograph showing the elevation of the depth markers, which are collectively denoted herein as the Tarcutta Depth Gauge, is contained in **Annexure B**.

2.4 Historic Flood Photographs

In response to the Community Flyer which was disseminated by WWCC at the commencement of the study, a number of residents provided photographs showing historic flooding in both Tarcutta and Uranquinty. **Appendices B** and **C** respectively contain a series of plates showing flood behaviour that was observed in Tarcutta and Uranquinty during both the October 2010 and March 2012 floods. No photographs were provided by the local community showing the flooding that was experienced (as limited as it was) in Ladysmith.

2.5 Rating Curve Adjustments

2.5.1 Tarcutta Creek at Old Borambola

As identified in L&A, 2012, DPIOW's rating curve for the Old Borambola stream gauge underestimates the flow in Tarcutta Creek for out-of-bank flood events. The layout of the cross sectional based UNET model which was developed as part of L&A, 2012 (denoted herein as the "Tarcutta Creek UNET Model") is shown on **Figure 2.9**.

It was found that a Manning's n value of 0.06, when applied to the inbank area of Tarcutta Creek, gave good correspondence with actual stream gauging data, whilst a value of 0.07 was required on the overbank area of the creek in order to reproduce the historic flood peaks for the March 2010, October 2010, December 2010 and March 2012 events (refer **Table 2.2** and **Columns M** and **N** in **Table 3.1** in **Section 3** for peak recorded and modelled historic gauge heights). **Figure 2.10** shows water surface profiles along the modelled reach of Tarcutta Creek for the four historic flood events.

A second order polynomial (refer **Equation 2.1**) was found to provide a good fit to the stage-discharge relationship generated by the UNET model at the location of the stream gauge for flows corresponding to a gauge height above 4.05 m. Note that the numbers in the various terms of **Equation 2.1** are quoted to a larger number of significant figures than would normally be the case in order to derive a stable rating curve. (A similar situation occurs for **Equations 2.2** to **2.8** following.)

Figure 2.11 (LHS) shows a comparison between DPIOW's rating curve and that derived as part of the present investigation for the Old Borambola stream gauge, whilst **Table 2.2** over gives a comparison of peak flows that have been derived for the four historic floods using both the current and adjusted rating curves.

$$Q = 4606004.46 - (47668.6945 \times GH) + (123.3311854 \times GH^2) \qquad \textbf{\textit{Equation 2.1}}$$

Where: Q = Total flow in Tarcutta Creek in m^3/s.
GH = Gauge height in m AHD.

TABLE 2.2
PEAK STAGE AND FLOWS FOR HISTORIC FLOOD EVENTS

Stream Gauge	Historic Flood Event	Recorded Peak Height (m / m AHD)	Peak Discharge (m^3/s)	
			DPIOW Rating Curve	Adjusted Rating Curve
Tarcutta Creek at Old Borambola Stream Gauge[(1)]	March 2010	4.22 / 194.92	202	252
	October 2010	5.42 / 196.12	447	916
	December 2010	4.95 / 195.65	336	608
	March 2012	4.86 / 195.56	317	557
Kyeamba Creek at Ladysmith Stream Gauge[(2)]	March 2010	5.85 / 201.07	243	273
	October 2010	6.67 / 201.89	393	465
	December 2010	5.17 / 200.39	132	140
	March 2012	6.13 / 201.35	288	385

1. Gauge zero on Tarcutta Creek at Old Borambola Stream Gauge = 190.699 m AHD.
2. Gauge zero on Kyeamba Creek at Ladysmith Stream Gauge = 195.224 m AHD.

2.5.2 Kyeamba Creek at Ladysmith

Attempts to calibrate the flood models for Ladysmith (refer **Sections 3** and **4** for details) lead to the conclusion that DPIOW's rating curve for its Ladysmith gauge underestimates peak flows for major flood events. The calibration of the flood models is further complicated by the major scour that occurred during the October 2010 flood on the left bank of Kyeamba Creek at the location of the disused railway bridge (refer Bewsher, 2011 for background information and also adjacent plate).

*Photograph taken looking upstream at Railway Bridge No. 2 (refer **Figure 2.7** for location). Major scour of the western bridge abutment can be seen on RHS of photograph.*

It was therefore necessary to generate two rating curves for Ladysmith stream gauge, one that describes the stage-discharge relationship which applied under pre-railway scour conditions and the other that represents current conditions on the floodplain (i.e. post-railway scour conditions). **Figure 2.11** (RHS) shows a comparison between DPIOW's rating curve and those derived as part of the present investigation. The following second, third and fourth order polynomials were found to provide a good fit to the stage-discharge relationship generated by the TUFLOW model at the location of the stream gauge under pre- and post-railway scour conditions:

*For gauge heights up to 4.0 m under <u>both pre- and post-railway scour conditions</u> (**Equation 2.2**):*

$$Q = 97.32944625 - (231.0989841 \times GH) + (177.9686391 \times GH^2) - (52.95455189 \times GH^3) + (5.49690657 \times GH^4)$$

*For gauge heights between 4.0 m and 4.97 m under <u>pre-railway scour conditions</u> (**Equation 2.3**):*

$$Q = 300.6314703 – (159.2250619 \times GH) + (23.49167013 \times GH^2)$$

*For gauge heights between 4.97 m and 6.0 m under <u>pre-railway scour conditions</u> (**Equation 2.4**):*

$$Q = -9612.055196 + (5108.446316 \times GH) – (921.4552547 \times GH^2) + (57.61966894 \times GH^3)$$

*For gauge heights above 6.0 m under <u>pre-railway scour conditions</u> (**Equation 2.5**):*

$$Q = -85499.37891 + (40733.49393 \times GH) – (6470.370852 \times GH^2) + (344.1792519 \times GH^3)$$

*For gauge heights between 4.0 m and 4.97 m under <u>post-railway scour conditions</u> (**Equation 2.6**):*

$$Q = 291.1262957 – (156.3245486\ GH) + (23.37158867 \times GH^2)$$

*For gauge heights between 4.97 m and 6.0 m under <u>post-railway scour conditions</u> (**Equation 2.7**):*

$$Q = -7501.344378 + (3876.588672 \times GH) – (687.3494966 \times GH^2) + (43.21220704 \times GH^3)$$

*For gauge heights above 6.0 m under <u>post-railway scour conditions</u> (**Equation 2.8**):*

$$Q = -66355.23127 + (31514.00143 \times GH) – (4998.634313 \times GH^2) + (266.5248449 \times GH^3)$$

Where: Q = Total flow in Kyeamba Creek in m^3/s.
GH = Gauge height in m AHD.

It is noted that the adjusted rating curves do not match DPIOW's rating curve below the level of the creek bank (i.e. below a level of about 5 m on the stream gauge). A review of the available stream gauging data upon which DPIOW has based its rating curve shows that the stage-discharge relationship derived by the TUFLOW model lies within the broad range of recorded data.[3] Based on this finding, the rating curves derived by the TUFLOW model are considered suitable for use in estimating the peak flow in the creek over the full range of gauge heights.

Table 2.2 above gives peak flows which have been derived for the four historic floods using both the current and adjusted rating curves.

2.6 Flood Frequency Analysis

2.6.1 Tarcutta Creek at Old Borambola

A log-Pearson Type 3 (**LP3**) distribution using L moments was fitted to the annual series of flood peaks for the 74 year period of record commencing in 1939 and ending in 2012. Peak discharges in Tarcutta Creek above bankfull flow conditions were increased in accordance with the adjusted rating curve described in **Section 2.5.1**. Peak height and discharge data are provided in **Tables C1** and **C2** in **Annexure C**. The resulting frequency curves, along with 5% and 95% confidence limits are shown on **Figure 2.12** (LHS).

[3] Initial runs of the flood models for the design flood events shows that water levels at the gauge site are impacted by the relative timing of flow in Kyeamba Creek and Wrights Gully, which would explain the large scatter in the recorded gaugings below 5 m on the Ladysmith gauge.

As the recorded flood peaks are only a small sample of peaks actually occurring over a longer duration, an expected probability adjustment was also made using the procedure set out in Australian Rainfall and Runoff (ARR, 1998). ARR, 1998 recommends implementing the expected probability adjustment to remove bias from the estimate.

Values at the low end of the observed range of flood peaks can distort the fitted probability distribution and affect the estimates of large floods. Deletion of these low values may improve the fitting of the remaining data. **Figure 2.12** (RHS) shows the results of omitting the thirty annual flows less than 50 m^3/s from the analysis and applying the expected probability adjustment to the remaining data.

Frequency analysis was also carried out fitting the annual peaks to the General Extreme Value (**GEV**) distribution using LH moments. **Figure 2.13** shows the results for both the full period of record (LHS) and after the thirty annual flows less than 50 m^3/s were omitted from the data set (RHS).

Table 2.3 gives the indicative ARI of the four historic floods which have been used to calibrate the flood models, whilst **Table 2.4** over shows that both methods give similar estimates of peak flows over the full range of recurrence interval design flood events.

TABLE 2.3
APPROXIMATE ARI OF HISTORIC FLOODS
TARCUTTA CREEK AT OLD BORAMBOLA STREAM GAUGE

Historic Flood	Recorded Peak Flow(1) (m^3/s)	Approximate ARI (years)	
		LP3 with Low Flows Omitted(2)	GEV with Low Flows Omitted(3)
March 2010	252	6	5
October 2010	916	50	70
December 2010	608	22	30
March 2012	557	18	22

1. Peak flows based on adjusted rating curve. Refer **Section 2.5.1** for details.
2. Refer **Figure 2.13** (RHS) for fitted distribution.
3. Refer **Figure 2.14** (RHS) for fitted distribution.

2.6.2 Kyeamba Creek at Ladysmith

Whilst stream flow data for the Ladysmith gauge dates back to 1938, Bewsher, 2011 identified a large gap in the record between the years 1987 to 2001. Inspection of the available stream flow record also shows that there are missing data in most years, which raises doubts as to whether the maximum water level in each year was captured by the recorder. Given the large amount of missing data, coupled with the uncertainties regarding the capture of annual maximums, a flood frequency analysis was not undertaken for the Ladysmith gauge.

TABLE 2.4
DESIGN PEAK FLOWS
TARCUTTA CREEK AT OLD BORAMBOLA STREAM GAUGE
VALUES IN m^3/s

ARI (years)	LP3 Distribution (L Moments)		GEV Distribution (LH Moments)	
	Full Period of Record	Low Flows Omitted	Full Period of Record	Low Flows Omitted
5	230	170	220	240
10	410	380	340	370
20	640	580	510	520
50	1040	910	810	790
100	1430	1030	1140	1050
200	1900	1800	1570	1390
500	2660	2700	2380	1970

3 HYDROLOGIC MODEL DEVELOPMENT AND TESTING

3.1 Hydrologic Modelling Approach

3.1.1 General

There are several land-uses present within the study catchments: the relatively steep areas which lie in the headwaters of the catchment; the cleared pastoral land which surrounds the villages; and the urbanised areas. In order to represent the rainfall-runoff process from these land-uses, the RAFTS modelling approach was used for the undeveloped areas, while DRAINS was used for the urbanised areas. Both the RAFTS and DRAINS modelling software enable their catchment lag parameters to be varied throughout the sub-catchments.

3.1.2 Brief Overview of RAFTS Modelling Approach

The RAFTS software converts storm rainfall to discharge hydrographs using a procedure known as rainfall-runoff routing and envisages the catchment to be comprised of a series of concentrated storages which represent sub-catchments defined on watershed lines, plus concentrated special storages which could simulate flood storage areas.

Each sub-catchment model is represented by a series of ten non-linear concentrated cascading storages. Within RAFTS each of the sub-areas in a sub-catchment is treated as a concentrated storage with a storage-discharge relation:

$$S = k(Q).Q \qquad \textbf{\textit{Equation 3.1}}$$

where $$k(Q) = B.Q^n \qquad \textbf{\textit{Equation 3.2}}$$

The parameters n and B represent the catchment non-linearity and sub-catchment storage delay coefficient respectively.

The storage delay coefficient B is either directly input for each sub-catchment or estimated from ***Equation 3.3*** which was derived from an analysis of the response of several gauged catchments undertaken by the developers of the RAFTS software.

$$Bav = 0.285\ A^{0.52}\ (1+U)^{-1.97} Sc^{-0.50} \qquad \textbf{\textit{Equation 3.3}}$$

Where:

Bav	=	mean value of coefficient B for each sub-catchment;
A	=	sub-catchment area (km^2);
U	=	fraction of catchment that is urbanised (where U = 1.0, the catchment is fully urbanised and when U = 0.0, the catchment is completely rural); and
Sc	=	main drainage slope of the sub-catchment (%).

Reducing the value of B during model testing increases both the rate of rise and the peak of the modelled discharge hydrograph; and conversely.

An additional empirical parameter (PERN) was added to the RAFTS code by the software developers to account for the effect on flows of the roughness of the surface of the sub-catchment. The parameter PERN is a function of the Mannings 'n' roughness of the sub-catchment. The storage delay coefficient B is then modified in accordance with the following table:

Mannings 'n' value	PERN Multiplication Factor
0.010	0.4
0.015	0.5
0.025	1.0
0.1	3.0

Adoption of a value of n equal to 0.025, which practitioners typically assign to urbanise catchments, will give a factor of 1.0 for PERN and therefore no adjustment to B. A higher value of PERN may be considered representative of more densely vegetated areas and will result in a larger value of the multiplier PERN and hence a larger value for B. For example, a value of 0.1 for n will result in a multiplier of 3.0 being applied (via the parameter PERN) to the computed value of B, with a corresponding reduction in the peak discharge generated by RAFTS.

The steps which were taken in order to calibrate the RAFTS model and the parameters which were found to provide good correspondence with recorded stream flow data are set out in **Section 3.3**.

3.1.3 Brief Overview of DRAINS Modelling Approach

The DRAINS software has been developed primarily for use in modelling the passage of a flood wave through urban catchments. The hydrologic model in DRAINS uses time-area calculations and Horton infiltration procedures to calculate sub-area discharge hydrographs that are assumed to enter the drainage system, subject to constraints imposed by its entrance and conveyance capacity. DRAINS is able to calculate hydraulic grade lines throughout a drainage network, enabling users to analyse the magnitude of overflows and stored water for established drainage systems.[4]

The time-area method utilised in DRAINS is a form of catchment routing model in which a hyetograph of rainfall is combined with a time-area diagram to produce a flow hydrograph. The procedure effectively divides a catchment into a number of equal sub-areas, and superimposes the individual flows from these sub-areas, allowing for time lags depending on their distance from the outlet. The time-area diagram is considered to be a triangular shape, with the increase in area per time step being constant.

DRAINS uses the depression storage (or initial loss) model for rainfall applied to impervious surfaces and the Horton infiltration model for rainfall applied to pervious surfaces. Horton's equation is the most common relationship for describing infiltration capacity in a soil. It describes the decrease in capacity as water is progressively absorbed by the soil, and has the form:

$$f = f_c + (f_0 - f_c).e^{-kt} \qquad \textbf{\textit{Equation 3.4}}$$

where: f is the infiltration capacity (mm/h) at time t;

f_0 and f_c are the initial and final constant rates of infiltration (mm/h);

k is a shape factor (fixed at a value of 2 /h in ILSAX); and

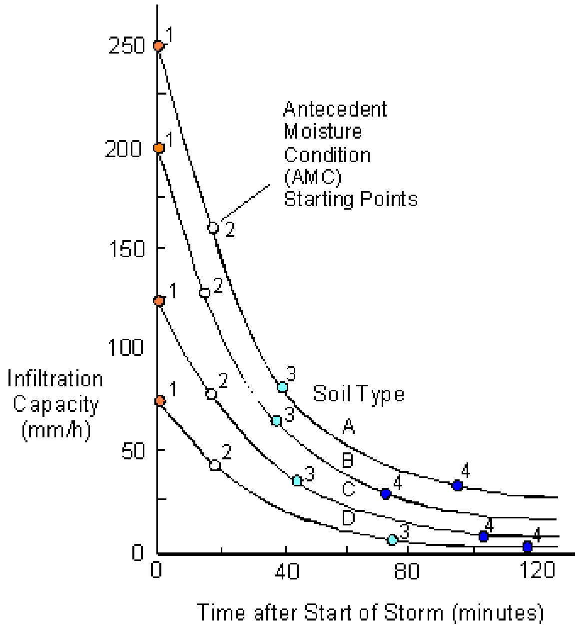
t is the time from the start of rainfall (h).

[4] This capability within DRAINS was not utilised as part of this present investigation, as the TUFLOW software was used for this purpose.

The soil type specified in DRAINS determines values for f_0 and f_c. There are four soil types involving different infiltration characteristics:

- Type 1 (or A) low runoff potential, high infiltration rates (sand and gravels),
- Type 2 (or B) moderate infiltration rates and moderately well-drained,
- Type 3 (or C) slow infiltration rates (may have layers that impede downward movement of water),
- Type 4 (or D) soils with high runoff potential, very slow infiltration rates (consisting of clays with a permanent high water table and a high swelling potential).

Users can specify a number between 1 and 4. DRAINS will interpolate between the standard infiltration factors applying to values of 1, 2, 3 or 4. The infiltration curves for these standard soil types are presented in the adjacent illustration.



Antecedent rainfall is the rainfall that occurs prior to the start of a storm event. It increases soil moisture levels and affects rates of infiltration into the soil.

The Antecedent Moisture Condition (AMC) is a parameter used in the loss calculations to specify the wetness or dryness of a catchment at the start of a storm. It is used to set the starting levels for infiltration relationships, and can have a significant effect on the flow rates generated by DRAINS.

An AMC number corresponds to a starting point on an infiltration curve, as shown in the above illustration. The curve defines the rate at which rainwater can penetrate into the soil. During a storm event, this will decrease, due to the soil becoming wetter, soil swelling and other effects. In research on DRAINS and related models, it has proved to be reasonably accurate to relate the AMC value of 1 to 4 to the rainfall in the previous 5 days.

As it is less important to estimate these losses correctly compared with the RAFTS component of the model (as the urban catchment is a small proportion of the overall catchment), the following DRAINS model parameters were adopted for testing the response of DRAINS to historic rainfalls. These values were also adopted for the design flood modelling (L&A, 2014):

- Soil Type = 3.0
- AMC = 3.0
- Paved area depression storage = 2 mm
- Grassed area depression storage = 10 mm
- Paved flow path roughness = 0.02
- Grassed flow path roughness = 0.07

3.2 Hydrologic Model Setup

3.2.1 General

Careful consideration was given to the definition of the sub-catchments which comprise the hydrologic model to ensure peak flows at various flow control structures were properly assessed. In addition to using the LiDAR survey data, the layout of the local stormwater drainage system in the urbanised parts of the villages was also taken into consideration when deriving the boundaries of the various sub-catchments. Percentages of impervious area were assessed using the aerial photography and cadastre boundary data.

In the upper reaches of the catchments, it was necessary to route the flow generated by several of the RAFTS sub-catchments to the upstream boundary of the hydraulic model. The outlets of these sub-catchments were linked and the lag times between each assumed to be equal to the distance along the main drainage path divided by an assumed flow velocity which was determined as part of the model calibration process.

Sub-catchment slopes used for input to the RAFTS component of the hydrologic model were derived using the vectored average slope approach, whilst the average sub-catchment slope computed by the Vertical Mapper software was used for input to the DRAINS component of the hydrologic model. The LiDAR survey data was used as the basis for computing the slope for both methods.

3.2.2 Tarcutta Creek

Figure 3.1 (2 Sheets) shows the layout of the RAFTS model which was developed for the Tarcutta Creek catchment. The following three principal sub-catchments make up the 1,341 m^2 catchment at the village:

- Tarcutta Creek (also known as Oberne Creek) which rises to the south near Tumbarumba and contributes runoff from 575 km^2 of catchment area;
- Umbango Creek (588 km^2), which joins Tarcutta Creek about 30 km upstream of the village; and
- Keajura Creek (178 km^2), which joins Tarcutta Creek just a short distance upstream of the village.

Because of its proximity to Tarcutta, the Old Borambola stream gauge has been used as the primary gauge for tuning the RAFTS model of Tarcutta Creek. As discussed in **Section 2.5.1**, it was necessary to update the high flow portion of DPIOW's rating curve for the gauge as it was found to underestimate the peak discharge in the creek for out-of-bank floods. The adjusted rating curve is shown on **Figure 2.11** (LHS). The recorded and adjusted discharge hydrographs for the March 2010, October 2010, December 2010 and March 2012 floods are shown on **Figure 3.2**.

3.2.3 Kyeamba Creek

Figure 3.3 (2 Sheets) shows the layout of the RAFTS model of Kyeamba Creek which drains a catchment of 530 km^2 at the Ladysmith stream gauging station. The catchment is elongated, with significant tributaries – O'Briens Creek (221 km^2) and Tywong Creek (32 km^2) joining Kyeamba Creek just upstream of Ladysmith.

As discussed in **Section 2.5.2**, it is believed that DPIOW's rating curve for the gauge underestimates peak flows in the creek for out-of-bank-floods. The rating curve also does not take account of the increase in conveyance which has resulted from the major scour that occurred to Railway Bridge No. 2 during the October 2010 flood. The adjusted rating curves for pre- and post-scour conditions are shown on **Figure 2.11** (RHS). The recorded and adjusted discharge hydrographs for the March 2010, October 2010, December 2010 and March 2012 floods are shown on **Figure 3.4**.

3.2.4 Sandy Creek

Figure 3.5 (2 Sheets) shows the layout of the RAFTS model for Sandy Creek which drains an area of 128 km^2 at Uranquinty. The creek flows about 27 km in a generally NNW direction to the village, and continues to the Murrumbidgee River. The RAFTS model includes the flash flooding local catchment to the east of the village. Overland flows from that catchment overtopped the Connorton Street Levee in the October 2010 and March 2012 floods.

3.3 Steps in Hydrologic Model Testing Process

The following steps were undertaken in the testing of the hydrologic models for the three study catchments:

- **Step 1** – A series of isohyetal maps were prepared for the March 2010, October 2010, December 2010 and March 2012 storm events, from which a set of Digital Elevation Models (**DEM's**) were prepared. **Figure 2.1** shows the cumulative depth of rain recorded by the various rain gauges, whilst **Figures 2.2** to **2.5** show the isohyetal rainfall depths for a select number of rain days for each historic storm event.
- **Step 2** – Pluviographic traces recorded during each storm event were applied to the various sub-catchments in the hydrologic models using the Theisson polygon approach. **Figures 2.2** to **2.5** show which rain gauges were used to describe the temporal variability of rain across the study catchments for the four historic storm events.
- **Step 3** - The average depth of rain for each sub-catchment was derived by interrogation of the DEM's which were generated as part of **Step 1**.
- **Step 4** – The total depth of rain associated with the individual pluviographic traces assigned to each sub-catchment was adjusted using a constant factor so that it matched the average depth of rainfall derived as part of **Step 3**.
- **Step 5** – A Manning's n value of 0.04 was applied to the RAFTS sub-catchments to describe the largely rural nature of the three study catchments.
- **Step 6** – Discharge hydrographs recorded at the Old Borambola gauge on Tarcutta Creek for the four historic storm events were adjusted using the results of hydraulic modelling undertaken using the UNET software (refer **Section 2.5.1** for further details).
- **Step 7** – A base flow separation analysis was undertaken using the adjusted hydrographs derived as part of **Step 6** and the volume of rainfall excess computed.
- **Step 8** – Adopting a constant continuing loss value of 2.5 mm/hr, the initial loss value for each storm event was adjusted until the volume of rainfall excess corresponded with the values derived as part of **Step 7**.[5]

[5] It was found that a continuing loss value of 1.7 mm/hr was needed to improve the fit to the recorded data in the case of the March 2012 flood.

- **Step 9** – The lag times in the RAFTS model links,[6] as well as the Bx factor were adjusted until the routed hydrographs[7] corresponded with the adjusted hydrographs derived as part of **Step 6**. **Figures 3.2** gives a comparison between recorded and modelled discharge hydrographs at the Old Borambola gauge.

- **Step 10** – A similar analysis to that described under **Steps 7** to **9** was undertaken for the Kyeamba Creek catchment, although it was necessary to tune the hydrologic model through an iterative approach, whereby model parameters were adjusted until the computed flood hydrographs when applied to the TUFLOW hydraulic model gave reasonable correspondence with available flood data.[8] **Figures 3.4** gives a comparison between recorded and modelled discharge hydrographs at the Ladysmith stream gauge.

- **Step 11** – Using the results of the model calibration process described above and taking account of the flatter nature of the Sandy Creek catchment,[9] hydrologic model parameters were adjusted in combination with running the TUFLOW hydraulic model until reasonable correspondence was achieved with available flood data. **Figures 3.6** shows discharge hydrographs which were generated by the RAFTS model at the Olympic Highway road culverts for the floods that occurred in October 2010 and March 2012.

Table 3.1 summarises the hydrologic model parameters which were found to provide a reasonable fit to historic flood data. A comparison between recorded and modelled peak heights and flows at both the Old Borambola and Ladysmith stream gauges is also provided in **Table 3.1**.

3.4 Discussion on Hydrologic Model Testing Process

3.4.1 Tarcutta Creek

Good agreement was achieved at the Old Borambola stream gauge between recorded and modelled stage and discharge for all four flood events after the discharge hydrographs generated by the RAFTS model were routed from Tarcutta to the gauge site using the Tarcutta Creek UNET model (refer **Columns M** and **N** in **Table 3.1**).

It was found that there is a minor attenuation of the flood wave as it travels the 21 km from Tarcutta to the Old Borambola stream gauge, as demonstrated by the difference in peak flows given in **Columns K** and **L** in **Table 3.1**.[10]

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[6] Individual lag times assigned to each link were computed by dividing the length of the link by an assumed average flow velocity.

[7] The discharge hydrographs generated by the RAFTS model were routed from the village of Tarcutta to the Old Borambola stream gauge using the UNET software. **Section 2.5.1** provides details of the UNET modelling which was undertaken as part of the present investigation.

[8] The availability of historic flood data combined with the significant amount of attenuation which was observed at the Ladysmith stream gauge for the smaller floods of March 2010 and December 2010 (refer **Figure 3.4**), meant that the Kyeamba Creek flood model could only be tuned to the larger floods of October 2010 and March 2012.

[9] It was found that the lag time in the RAFTS model links had to be increased when compared to the Kyeamba Creek and Tarcutta Creek models in order to achieve reasonable correspondence with historic flood data. The increase in the travel time of the flood wave was attributed to the flatter nature of the catchment and hence a slower average flow velocity in the streams which drain the Sandy Creek catchment.

[10] The peak flows presented in Column L for Tarcutta Creek take account of the attenuating effects of the natural floodplain storage, whereas those in Column K do not.

TABLE 3.1
ADOPTED HYDROLOGIC MODEL PARAMETERS AND PEAK FLOW/LEVEL COMPARISON AT GAUGE SITES
HISTORIC STORM EVENTS

Catchment/ Location	Historic Storm Event	Volume of Surface Runoff (m^3)		Initial Loss (mm)	Continuing Loss Rate (mm/hr)	Bx Factor	Assumed Flow Velocity in Links (m/s)	Peak Flow (m^3/s)		Modelled Peak Flow (m^3/s)		Peak Stage at Stream Gauge(2) (m AHD / m)	
		Recorded Hydrograph	Modelled Hydrograph					DPIOW's Rating Curve	Adjusted Rating Curve	RAFTS	UNET/ TUFLOW(2)	Recorded(3)	Modelled(4)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]
Tarcutta Creek at Old Borambola Stream Gauge	March 2010	2.06×10^7	2.13×10^7	64	2.5	1.0	1.5	202	**252**	269	262	194.92 / 4.22	194.95 / 4.25
	October 2010	7.97×10^7	7.91×10^7	33	2.5	1.0	1.8	447	**916**	992	933	196.12 / 5.42	196.13 / 5.43
	December 2010	4.11×10^7	3.91×10^7	37	2.5	1.0	1.5	336	**608**	667	607	195.65 / 4.95	195.66 / 4.96
	March 2012(1)	5.70×10^7	4.84×10^7	22	1.7	0.9	1.5	317	**557**	568	557	195.56 / 4.86	195.57 / 4.87
Kyeamba Creek at Ladysmith Stream Gauge	October 2010(1)	1.40×10^7	1.43×10^7	43	2.5	0.9	1.5	393	**458**	456	474(5)	201.90 / 6.67	201.88 / 6.66
	March 2012(1)	1.15×10^7	2.17×10^7	44	0.9	0.9	1.5	288	**381**	378	384(6)	201.35 / 6.13	201.33 / 6.11
Sandy Creek at Uranquinty	October 2010	-	-	40	2.5	0.9	1.0	-	-	172	153	-	-
	March 2012	-	-	50	0.9	0.9	1.0	-	-	123	121	-	-

Notes

1. Base flow separation analysis undertaken on second flood peak only.
2. Recorded gauge heights sourced from provisional river data which is available DPIOW's web site.
3. Gauge zero on Tarcutta Creek at Old Borambola Stream Gauge = 190.699 m AHD. Gauge zero on Kyeamba Creek at Ladysmith Stream Gauge = 195.224 m AHD.
4. UNET software used at Old Borambola stream gauge, whilst TUFLOW software used at Ladysmith stream gauge.
5. TUFLOW model incorporates base flow of 25 m^3/s.
6. TUFLOW model incorporates base flow of 10 m^3/s.

Table 3.2 gives peak flows which were generated by the RAFTS model at key locations within the Tarcutta Creek catchment, as well as the time that it took for flows to peak following the onset of heavy rain. By inspection of the values given in the table, major flows emanated from the Umbango Creek catchment for all but the December 2010 flood, when intense rain over the upper reaches of the Tarcutta Creek catchment resulted in significantly higher flows in this stream than were recorded during the October 2010 flood.[11]

TABLE 3.2
SUMMARY OF RAFTS PEAK FLOWS FOR HISTORIC FLOOD EVENTS
TARCUTTA CREEK CATCHMENT
VALUES in m^3/s

Location	Historic Flood Event(1)			
	March 2010(2)	October 2010(3)	December 2010(4)	March 2012(5)
Tarcutta Creek at Westbrook stream gauge (GS 410058)	0 [-]	242 [12]	309 [13]	162 [22]
Tarcutta Creek at Belmore Bridge stream gauge (GS410155)	0 [-]	271 [13]	406 [13]	184 [23]
Tarcutta Creek immediately upstream of confluence with Umbango Creek	5 [18]	340 [16]	439 [18]	208 [27]
Umbango Creek immediately upstream of confluence with Tarcutta Creek	223 [18]	527 [18]	160 [16]	351 [21]
Tarcutta Creek immediately downstream of confluence with Umbango Creek	223 [18]	857 [18]	589 [18]	488 [24]
Tarcutta Creek immediately upstream of confluence with Keajura Creek	236 [18]	872 [18]	598 [19]	493 [25]
Keajura Creek immediately upstream of confluence with Tarcutta Creek	34 [6]	74 [15]	82 [12]	69 [17]
Tarcutta Creek immediately downstream of confluence with Keajura Creek	253 [18]	931 [18]	638 [19]	536 [24]
Tarcutta Depth Gauge	253 [18]	936 [18]	531 [20]	538 [24]
Tarcutta Creek at Old Borambola stream gauge (GS 410047)	269 [24]	992 [23]	667 [25]	568 [30]

1. Values in [ ] refer to time to peak in hours after the onset of heavy rain.
2. Time zero equal to 09:00 hours on 7 March 2010.
3. Time zero equal to 03:00 hours on 15 October 2010.
4. Time zero equal to 18:00 hours on 8 December 2010.
5. Time zero equal to 12:00 hours on 3 March 2012.

Whilst BOM's Belmore Bridge and Westbrook flood warning stream gauges are not rated, the relativity of peaks flows generated by the RAFTS model for the four historic floods is consistent with the peak stages given in **Table 2.1**.

[11] This finding is consistent with that of Yeo, 2013, which identified that water levels at both the Westbrook and Belmore Bridge stream gauges exceeded previously recorded heights during the December 2010 flood.

In regard to the March 2010 flood, almost all the flow in Tarcutta Creek appears to have emanated from the Umbango Creek catchment, with initial losses resulting in very little surface runoff being generated in the upper reaches of the Tarcutta Creek catchment. The heavy rain that was recorded at the Book Book rain gauge (which is considered to be indicative of the rain which fell on the western portion of the catchment near Tarcutta) also preceded that which was recorded at the Carabost rain gauge, which had the effect of desynchronising flows in Tarcutta Creek and Keajura Creek. The spatial and temporal variability of the storm event in combination with relatively high initial losses in the catchment resulted in only a minor flood being experienced at Tarcutta in March 2010.

It is noted that the peak flow at Tarcutta for the December 2010 flood was about 100 m^3/s higher than for the March 2012 flood, even though the peak height recorded on the Tarcutta Depth Gauge was 10 mm higher for the more recent flood.[12] The difference in peak flood heights is attributed to an increase in the density of vegetation on the floodplain downstream of the Sydney Street bridges over the intervening 2 year period had the effect of slowing the velocity of floodwater and thereby increasing peak flood levels.

Adoption of the Theisson Polygon approach to the assigning of temporal patterns to sub-catchments in the RAFTS model generally reproduced the recorded timing of the flood peaks, with the exception of the March 2012 flood. It was found that it was necessary to apply the rainfall which was recorded at BOM's Batlow rain gauge (GS 72004) to sub-catchments T1 and T2 in the RAFTS model in order to reproduce the timing of the observed flood peaks at BOM's Westbrook and Book Book stream gauges. It was also found that a 22 mm initial loss in combination with a continuing loss rate of 1.7 mm/hr was required to more closely match the observed timing of the flood peaks at the Tarcutta Depth Gauge and the Old Borambola stream gauge.[13]

The response time of the catchment following the onset of heavy rainfall did not vary greatly for the three floods which occurred in 2010, when it took between 18-20 hours for flows in Tarcutta Creek at Tarcutta to peak following the onset of heavy rain. The same was not the case for the March 2012 flood, where flows did not peak at Tarcutta until 24 hours after the onset of heavy rain. The reason for this is attributed to a band of heavy rain, which by inspection of the Adelong (Etham Park) and Batlow rain gauges moved in generally a southerly direction (i.e. in an upstream direction) on the rain day of 4 March 2013, with the heavier falls in the upper reaches of the catchment occurring later in the storm event.

3.4.2 Kyeamba Creek

As previously mentioned, the flood models for Kyeamba Creek were tuned through an iterative process whereby the RAFTS and TUFLOW model parameters were adjusted until a good fit was achieved with the available flood data. This process also included making adjustments to the rating curve for the Ladysmith stream gauge based on the results of the flood modelling, further details on which are contained in **Section 2.5.2**.

[12] Yeo, 2013 gives the peak height recorded at the Tarcutta Depth Gauge as 3.85 m and 3.86 m for the December 2010 and March 2012 floods, respectively.

[13] Note that only the second burst of rainfall which commenced at around 12:00 hrs on 3 March 2012 was modelled in RAFTS.

Tuning of the Kyeamba Creek flood models was limited to the October 2010 and March 2012 floods as by inspection of the flows recorded by the Ladysmith gauge (refer **Figure 3.4**), there appears to be significant attenuation and prolongation of the flood wave occurring upstream of the village during minor flood events (a feature which cannot be reproduced by the RAFTS model which uses a simple time lag approach to routing the flood hydrograph down the valley). **Table 3.3** gives peak flows which were generated by the calibrated RAFTS model at key locations within the Kyeamba Creek catchment for the October 2010 and March 2012 floods, along with the times that it took for flows to peak following the onset of heavy rain.

TABLE 3.3
SUMMARY OF RAFTS PEAK FLOWS FOR HISTORIC FLOOD EVENTS
KYEAMBA CREEK CATCHMENT
VALUES in m^3/s

Location	Historic Flood Event(1)	
	October 2010(2)	March 2012(3)
Kyeamba Creek at Book Book stream gauge (GS 410156)	133 [14]	106 [17]
Kyeamba Creek immediately upstream of confluence with O'Briens Creek	183 [18]	178 [21]
O'Briens Creek immediately upstream of confluence with Kyeamba Creek	358 [15]	208 [19]
Kyeamba Creek immediately downstream of confluence with O'Briens Creek	470 [16]	380 [20]
Kyeamba Creek immediately upstream of confluence with Tywong Creek	471 [17]	383 [21]
Tywong Creek immediately upstream of confluence with Kyeamba Creek	44 [13]	32 [13]
Kyeamba Creek immediately downstream of confluence with Tywong Creek	486 [17]	398 [21]
Kyeamba Creek at Ladysmith stream gauge (GS 410048)	486 [17]	403 [21]

1. Values in [ ] refer to time to peak in hours after the onset of heavy rain.
2. Time zero equal to 03:00 hours on 15 October 2010.
3. Time zero equal to 12:00 hours on 3 March 2012.

Similar to the finding for the Tarcutta Creek catchment, a reduced value of continuing loss was required to more closely match the observed timing of the flood peak at the Ladysmith stream gauge for the March 2012 flood (in this case a value of 0.9 mm/hr as opposed to 1.7 mm/hr for the Tarcutta Creek catchment).

By inspection of the values given in **Table 3.3**, peak flows generated by the O'Briens Creek catchment exceeded those on the main arm of Kyeamba Creek during both the October 2010 and March 2012 floods. Flows on O'Briens Creek also peaked a few hours prior to the arrival of the flood peak on the main arm of Kyeamba Creek during both flood events.

The time between the recorded peaks at the Book Book and Ladysmith stream gauges computed by the RAFTS model was 3 and 4 hours for the October 2010 and March 2012 floods, respectively. These compare closely with the difference in the record data of 5 hours (October 2010 flood) and 2 hours 45 minutes (March 2012 flood) (refer **Table 2.1**). The minor difference in the recorded and modelled times is due to the temporal variability of the rainfall across parts of the catchment which was not captured by BOM's network of rain gauges (and hence not incorporated in the RAFTS model).

3.4.3 Sandy Creek

Similar to the Kyeamba Creek flood models, those for Sandy Creek were tuned through an iterative process whereby the RAFTS and TUFLOW model parameters were adjusted until a good fit was achieved with the available flood data. As there is no stream gauges located on Sandy Creek, tuning of the flood models was based on the outcomes of the model tuning process for the Tarcutta Creek and Ladysmith catchments, as well as comparison of modelled and recorded flood behaviour in Uranquinty.

It was found that in order to reproduce the time when Deane Street was surcharged by floodwater during both the October 2010 and March 2012 floods, an average flow velocity of 1.0 m/s needed to be applied to the derivation of the lag times in the RAFTS model links. This reduced flow velocity is attributed to the flatter nature of the Sandy Creek catchment when compared to both the Tarcutta Creek and Kyeamba Creek catchments.

Table 3.4 gives the peak flows, and also times to peak, at key locations within the Sandy Creek catchment for both the October 2010 and March 2012 floods. It is noted that peak flows in Little Sandy Creek were significantly larger than those on the main arm of Sandy Creek during both flood events, even though the response time of the two creeks at their confluence was the same.

TABLE 3.4
SUMMARY OF RAFTS PEAK FLOWS FOR HISTORIC FLOOD EVENTS
SANDY CREEK CATCHMENT
VALUES in m^3/s

Location	Historic Flood Event(1)	
	October 2010(2)	March 2012(3)
Sandy Creek immediately upstream of confluence with Little Sandy Creek	28 [12]	11 [12]
Little Sandy Creek immediately upstream of confluence with Sandy Creek	65 [12]	32 [12]
Sandy Creek immediately downstream of confluence with Little Sandy Creek	93 [12]	43 [12]
Sandy Creek immediately upstream of confluence with	141 [15]	90 [14]
Coloboralli Creek immediately upstream of confluence with Sandy Creek	35 [13]	23 [13]
Sandy Creek immediately downstream of confluence with Coloboralli Creek	165 [15]	112 [14]
Sandy Creek at inflow boundary of TUFLOW model(4)	172 [15]	123 [15]

1. Values in [ ] refer to time to peak in hours after the onset of heavy rain.
2. Time zero equal to 03:00 hours on 15 October 2010.
3. Time zero equal to 12:00 hours on 3 March 2012.
4. Refer inflow boundary Ura_SC1 on **Figure 4.4** for location.

3.5 Hydrologic Model Parameters for Design Flood Estimation

Table 3.5 sets out the RAFTS model parameters which are recommended for use in the derivation of design discharge hydrographs for input to the TUFLOW hydraulic models for each village. The values of initial and continuing loss which are recommended for design flood estimation are based on the recommendations of Walsh et al, 1991.

The DRAINS model parameters recommended for use in design flood estimation are set out in **Section 3.1.3**.

TABLE 3.5
RECOMMENDED RAFTS MODEL PARAMETERS FOR USE IN DESIGN FLOOD ESTIMATION

Catchment	RAFTS Model Parameter								
	Initial Loss (mm)					Continuing Loss (mm/hr)		Bx Factor	Assumed Flow Velocity in Links (m/s)
	5 and 10 year ARI	20 year ARI	50 year ARI	100, 200 and 500 year ARI	PMF	Up to 500 year ARI	PMF		
Tarcutta Creek	25	20	15	15	0	2.5	0	1.0	1.5
Kyeamba Creek	25	20	15	15	0	2.5	0	0.9	1.5
Sandy Creek	25	20	15	15	0	2.5	0	0.9	1.0

4 HYDRAULIC MODEL DEVELOPMENT AND TESTING

4.1 TUFLOW Modelling Approach

TUFLOW is a true two-dimensional hydraulic model which does not rely on a prior knowledge of the pattern of flood flows in order to set up the various fluvial and weir type linkages which describe the passage of a flood wave through the system. The basic equations of TUFLOW involve all of the terms of the St Venant equations of unsteady flow. Consequently the model is "fully dynamic" and once tuned will provide an accurate representation of existing flood behaviour in terms of depth, velocity and distribution of flow.

TUFLOW solves the equations of flow at each point of a rectangular grid system which represent overland flow on the floodplain and along streets. The choice of grid point spacing depends on the need to accurately represent features on the floodplain which influence hydraulic behaviour and flow patterns (e.g. buildings, streets, changes in channel and floodplain dimensions, hydraulic structures which influence flow patterns, etc.).

Pipe drainage and channel systems can be modelled as one-dimensional elements embedded in the larger two-dimensional domain which typically represents the wider floodplain. Flows are able to move between the one and two-dimensional elements of the model depending on the capacity characteristics of the drainage system being modelled.

The TUFLOW models as part of the present investigation allow for the assessment of potential flood management measures, such as detention storage, increased channel and floodway dimensions, augmentation of culverts and bridge crossing dimensions, diversion banks and levee systems.

4.2 TUFLOW Model Development

4.2.1 Model Structure

Figures 4.1 to **4.4** show the layout of the various components which comprise the TUFLOW models at the three villages. A 5 m grid spacing was found to provide the appropriate balance between the need to define features on the floodplain versus model run times. Grid data were based on the LiDAR survey of the floodplain, with ridge and gully lines added to the model where the grid spacing was considered too coarse to accurately represent important topographic features, such as the flood protection levees at Tarcutta and Uranquinty and the disused railway and its embankment at Ladysmith. Cross sections surveyed by the ground survey were used to define the in-bank waterway characteristics at hydraulic structures located in the channel system.

Consideration was given to selection of the appropriate method of modelling urban development located in the two-dimensional domain. Options available were to model buildings and structures as either permeable or impermeable to the passage of flow, or even to excise them from the floodplain altogether. Each approach has its advantages and disadvantages in providing accurate solutions to the problem of modelling the passage of shallow overland flow, which are discussed in detail in the documentation for the TUFLOW software.

After consideration, the footprints of a large number of individual buildings located in the two-dimensional model domain were digitised and assigned an artificially high hydraulic roughness value which accounted for their blocking effect on flow while maintaining storage in the model.

Individual allotments where development is present were also digitised and assigned an artificially high hydraulic roughness value (although not as high as for individual buildings) to account for the reduction in conveyance capacity which will result from fences and other obstructions within these properties.

Field survey was used to obtain details of pipes and box culverts which were incorporated into the TUFLOW models. Uni-directional pipes were incorporated in the model to represent those conduits which have flood gates fitted to their outlets (1 off at Tarcutta and 5 off at Uranquinty).

The following features which were unique to the floodplain at the time of the October 2010 and March 2012 floods were incorporated in the structure of the individual TUFLOW models used to simulate those two events:

- **Tarcutta TUFLOW Model** – As the Tarcutta Bypass (Hume Highway Upgrade) was under construction at the time of the October 2010 flood, the corridor survey undertaken by RMS immediately following the event was used to update the grid levels in the hydraulic model. A design model of the temporary access roads which crossed Tarcutta Creek either side of the highway crossing was also incorporated in the model. **Figure 4.1** shows the plan extent of the road works which were incorporated in the hydraulic model.

  Modelling of the March 2012 flood was undertaken by removing the corridor survey and temporary access track design model and updating the grid levels based on RMS' road design model of the highway upgrade. Flow Constriction Values (**FCV's**) were also applied to the cells which lie directly below Hume Highway Bridge No. 1 (FCV = 0.08) and Hume Highway Bridge No. 2 (FCV = 0.04) to account for the increased losses associated with flow around the bridge piers. **Figure 4.2** shows the plan extent of the completed road works which were incorporated in the hydraulic model. The model shown in **Figure 4.2** will be adopted for design flood modelling.

  The inbank survey undertaken by Casey Surveying and Design Pty Ltd in 2013 as part of the present investigation was used to represent the conveyance capacity in the creek system for both the October 2010 and March 2012 floods.

  No information was available on the size of the culverts that were installed approximately 900 m north of the Hume Highway Bridge No. 1 along the route of the Tarcutta Bypass (Hume Highway Upgrade). It was assumed that these structures had the same configuration as the 6 off 900 mm diameter pipes which were installed further upstream beneath Sydney Street as part of the highway upgrade.

- **Ladysmith TUFLOW Model** – As mentioned in **Section 2.5.2**, major scour occurred during the October 2010 flood on the left (western) abutment of Railway Bridge No. 2. Initial runs of the hydraulic model showed that the recorded gauge height of 6.67 m could not be reproduced had the scour occurred prior to the arrival of the flood peak. As a result, the opening in the railway embankment was assumed not to have scoured when modelling the October 2010 flood. The scoured opening was incorporated in the March 2012 flood model and will be retained for design flood modelling.

- **Uranquinty TUFLOW Model** – Bewsher, 2011 notes that efforts were made late on the morning of 4 March 2012 to "top up" the height of the Town Levee (South) and that it was not clear how much additional inundation this prevented given Deane Street had been overtopped earlier in the day (**Plates 45** to **47** in **Appendix C** show the temporary sandbagging which was installed along Deane Street late on the morning of 4 March 2012). Based on this finding, flood behaviour in the March 2012 event was assessed assuming the temporary levee upgrade works were not in place at the time of the peak, which modelling showed likely occurred prior to daybreak at around 03:30 hrs on 4 March 2012 (refer **Section 4.5.2** for further details).

4.2.2 Model Boundary Conditions

The locations where inflow hydrographs were input to the upstream limits of the two-dimensional model domain are shown on the model layouts. Internal to the models, discharge hydrographs were input as follows:

- In the urbanised parts of the study area, inflow hydrographs were input directly to the upstream reach of individual one-dimensional elements in the TUFLOW models. These typically coincided with the location of major drainage structures. The locations where flow was input to the TUFLOW models generally corresponded with the downstream limit of the sub-areas in the hydrologic model.
- In parts of the study area, inflow hydrographs were input to the TUFLOW models over individual regions called "Rain Boundaries". The areal extent of Rain Boundaries generally corresponded with the sub-areas in the hydrologic model.

  The Rain Boundaries act to "inject" flow into the one and two-dimensional domains of the TUFLOW model, firstly at a point which has the lowest elevation, and then progressively over the extent of the Rain Boundary as the grid in the two-dimensional model domain becomes wet as a result of overland flow.

  The approach of having the model inject flow progressively along the flow paths as cells become wet and as overland flows are initiated is more realistic than the traditional approach where inflow hydrographs (determined by hydrologic modelling) are applied at fixed locations along the model drainage lines. Because in the real drainage system, the inflows are dispersed rather than "lumped", the latter approach tends to either underestimate or overestimate the magnitude of the peak flow rate along the extent of the drainage path.

The boundaries of the TUFLOW model were taken a sufficient distance downstream so that uncertainties in the stage versus discharge relationship for the relevant creek did not influence results in the villages.

4.2.3 Model Roughness

The main physical parameter for TUFLOW is the hydraulic roughness. Hydraulic roughness is required for each of the various types of surfaces comprising the overland flow paths, as well as for the cross sections representing the geometric characteristics of the channels. In addition to the energy lost by bed friction, obstructions to flow also dissipate energy by forcing water to change direction and velocity and by forming eddies. Hydraulic modelling traditionally represents all of these effects via the surface roughness parameter known as "Mannings n". Flow in the piped system also requires an estimate of hydraulic roughness.

Assessment of Mannings n values for sections of channel was relatively straightforward, as cross sections taken normal to the direction of flow have traditionally been used when modelling one-dimensional waterways. Channel roughness was estimated from site inspection, past experience and values contained in the engineering literature.

Table 4.1 presents the “best estimate” of hydraulic roughness values adopted for model testing purposes. These values were subsequently found to give reasonable correspondence with observed flood behaviour. The adoption of a value of 0.02 for the surfaces of roads, along with an adequate description of their widths and centreline and kerb elevations, allowed a reasonably accurate assessment of their conveyance capacity to be made. Similarly the high value of roughness adopted for buildings recognised that they completely blocked the flow but were capable of storing water when flooded.

Modelled buildings with their high values of hydraulic roughness, block the passage of flow, although the model recognises that they store floodwaters when inundated and therefore correctly accounts for flood storage. The flow is conveyed along the roads and across the open parts of the allotments.

TABLE 4.1
“BEST ESTIMATE” OF HYDRAULIC ROUGHNESS VALUES
ADOPTED FOR TUFLOW MODEL TESTING

Surface Treatment	Mannings n Value
Paved road and railway (all villages)	0.02
Dirt road (all villages)	0.03
Unmaintained grass and floodplain (all villages)	0.05
Lightly vegetated areas (all villages)	0.07
Fenced properties (all villages)	0.10
Buildings (all villages)	10
Creek bed (all creeks)	0.04
Riparian vegetation between Sydney Street and the Hume Highway on Tarcutta Creek (March 2012 flood)	0.2
Riparian vegetation between Sydney Street and the Hume Highway on Tarcutta Creek (October 2010 flood)	0.08
Riparian vegetation along banks of Kyeamba Creek, Ladysmith (March 2012 and October 2010 floods)	0.2

4.3 Testing of Tarcutta TUFLOW Model

4.3.1 October 2010 Flood

Figure 4.5 shows the water surface profile along the modelled reach of Tarcutta Creek, and **Figure 4.6** (2 Sheets) indicative depths of inundation, water surface contours and flow direction arrows (Sheet 2 of 2 only) for the October 2010 flood. **Appendix B** contains several plates that show the flooding which was experienced in parts of Tarcutta on 16 October 2010. Unless otherwise noted, the locations of flood marks referred to in the following discussion are shown on **Figure 4.6** (Sheet 2 of 2).

Flooding patterns generated by the Tarcutta TUFLOW Model are in close agreement with those shown on Figure 13.2 in Bewsher, 2011. Close comparison with recorded flood marks was also achieved in the village (refer Flood Marks FMT4, FMT5, FMT6, FMT7 and FMT10), but only after the waterway area of pipes located on the right overbank of Tarcutta Creek beneath the temporary access road were blocked and the far right flood runner assumed to be partially filled with bed material at the peak of the flood (refer **Figure 4.6** (Sheet 2 of 2) for location of temporary pipes).[14]

Whilst the model underestimated flood levels when compared to those recorded in two properties located on Westbrook Street directly behind the Tarcutta Levee (refer Flood Marks FMT8 and FMT9), the difference in levels is attributed to local velocity head effects which may have arisen as a result of the floodwater which overtopped the levee, since this area was inundated principally as a result of backwater flooding which extended into this area from the direction of Sydney Street. It is noted that the TUFLOW model was able to reproduce the minor overtopping which occurred along a short length of the Town Levee.

The peak flood level computed by the model at the location of the Tarcutta Depth Gauge was 200 mm below the recorded height of 4.49 m (or RL 229.17 m AHD). The reason for the model underestimating the peak flood level at this location could be due to it incorporating the final bed profile and waterway area of the creek following the major scour that occurred in the main channel over the duration of the flood event, rather than representing the actual waterway which was present at the time of the flood peak.[15]

The model overestimated the levels that were recorded upstream of the Sydney Street bridges (refer Flood Marks FMT11 and FMT12) by 200 mm. A review of the source data shows that a low level of confidence was assigned to these two flood marks and it is noted that they are located on land which lies about 1 m above the nominated flood level.

Whilst the hydraulic model closely matched the flood level which was recorded on the left overbank of Tarcutta Creek along the Hume Highway corridor (refer Flood Mark FMT3), it underestimated the flood level on the right overbank of the creek immediately upstream of one of the temporary access roads by about 400 mm (refer Flood Mark FMT4). The lower peak flood level computed by the hydraulic model at this location is considered to be a function of the high velocity flow in the adjacent channel which was assumed not to have been obstructed by the temporary access road. By inspection of the water surface contours shown on **Figure 4.6** (Sheet 2 of 2), had the temporary access road impeded the flow and caused a reduction in flow velocities in the channel (e.g. as a result of a partial or total blockage of the waterway crossing), then the peak flood level generated by the model would have increased to about RL 227.2 m AHD, which compares closely with the recorded level. The flood mark is also located in an area in which eddies in the flow would have formed, which may have also contributed to the higher flood level than the hydraulic model was able to predict.

Close correspondence was achieved with a flood mark which was surveyed downstream of the Hume Highway corridor on the right (eastern) bank of Tarcutta Creek (refer Flood Mark FMT1 on **Figure 4.6** (Sheet 1 of 2)).

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[14] The owner of the Old Tarcutta Inn advised that a pipe (or pipes) located beneath the temporary construction access track were observed to have been blocked with bed material following the October 2010 flood and that it had also deposited in the upstream reach of channel during the event.

[15] As the waterway area in Tarcutta Creek at the time of the flood peak cannot be determined with any confidence, the inbank survey commissioned as part of the present investigation (which actually reflects post – October 2010 flood conditions) has been used in the development of the hydraulic model.

By comparison with the values given in **Table 4.2** over, the flood models were able to closely reproduce the timeline of events which were observed both on the rising limb and at the peak of the flood.

4.3.2 March 2012 Flood

Figure 4.5 shows the water surface profile along the modelled reach of Tarcutta Creek, and **Figure 4.7** (2 Sheets) indicative depths of inundation, water surface contours and flow direction arrows (Sheet 2 of 2 only) for the March 2012 flood. **Appendix B** contains several plates that show the flooding which was experienced in parts of Tarcutta in early March 2012. The locations of flood marks referred to in the following discussion are shown on **Figure 4.7** (Sheet 2 of 2).

TABLE 4.2
COMPARISON OF MODELLED VERSUS OBSERVED TIMELINE OF EVENTS AT TARCUTTA

Location and Feature	Time and Date of Occurrence			
	October 2010 Flood		March 2012 Flood	
	Modelled	Observed[1]	Modelled	Observed[2]
Backwater flooding across Sydney Road adjacent to the Tarcutta Hotel	18:00 hrs 15/10/10	19:00 hrs 15/10/10	-	-
Overtopping of the Tarcutta Levee adjacent to Centenary Avenue	21:30 hrs 15/10/10	21:14 hrs 15/10/10	-	-
Peak height reached on Tarcutta Depth Gauge	22:20 hrs 15/10/10	19:41 hrs[3] 15/10/10	12:55 hrs 4/3/12	13:00 hrs 4/3/12

1. Source: Bewsher, 2011
2. Source: Yeo, 2013
3. Time of peak is based on RFS log of events at Tarcutta. Timing is clearly not correct as the same log file states that floodwater was observed to be overtopping the Town Levee at 21.14 hrs. Residents state that floodwater commenced to overtop the Town Levee at about 20:00 hrs.

Close correspondence with flood levels recorded at the Tarcutta Hotel (refer Flood Mark FMT13) and the Tarcutta Depth Gauge was achieved for the March 2012 flood, after the Mannings n hydraulic roughness value on the floodplain of Tarcutta Creek between Sydney Street and the Hume Highway was increased from 0.08 to 0.2. This finding is supported by available aerial photography which shows that there was a reduced level of ground cover on the overbank area of Tarcutta Creek at the time of the October 2010 flood when compared to conditions that were observed during a site inspection which was undertaken immediately following the March 2012 flood.

Whilst the sag in Sydney Street adjacent to the Tarcutta Hotel was not inundated by floodwater, residential properties immediately to the east along Centenary Avenue were inundated by backwater flooding which extended up the 2 off 600 mm diameter pipes which cross the road corridor immediately north of the hotel (refer **Figure 4.7** (Sheet 2 of 2) for location).[16]

[16] Whilst a flood gate was observed on the outlet of the 750 mm diameter pipe which runs beneath the Town Levee on the eastern (upstream) side of Sydney Street (refer **Figure 4.7** (Sheet 2 of 2) for location), no flood gates were observed on the outlet of the two 600 mm diameter pipes.

It is noted that whilst the earth levee surrounding the Old Tarcutta Inn was overtopped, the reinforced block retaining wall which was built to protect the existing building post the October 2010 flood was not overtopped by floodwater. The Hambledon Levee was also not overtopped by the March 2012 flood, with minor ponding shown to likely have occurred immediately behind the levee due to local catchment runoff.

By comparison with the values given in **Table 4.2**, the flood models were able to closely reproduce the time that flood levels peaked at the Tarcutta Depth Gauge.

4.4 Testing of Ladysmith TUFLOW Model

4.4.1 October 2010 Flood

Figure 4.8 shows the water surface profile along the modelled reach of Kyeamba Creek, and **Figure 4.9** indicative depths of inundation and water surface contours for the October 2010 flood.

Patterns of overland flow and extents of inundation generated by the Ladysmith TUFLOW Model are in reasonable agreement with those shown on Figure 14.7 in Bewsher, 2011. Close correspondence was achieved with the three available flood marks (i.e. FML1, FML2 and FML3), as well as the recorded water level at the Ladysmith stream gauge. It is noted that flood mark FML3 relates to the elevation that the water level reached in No. 9080 Tumbarumba Road, where the existing residence was inundated to a depth of about 500 mm.

The peak height on the Ladysmith stream gauge of 6.67 m was recorded at 20:15 hrs on 15 October, which compares closely to the modelled peak of 21:00 hrs. No other time based flood data is available for Ladysmith to allow further comparison between modelled and observed flood behaviour.

Whilst existing development in the village was not impacted by main stream flooding, the modelling indicates that properties located to the south of Tywong Street may have been impacted by relatively shallow overland flow which crossed Tumbarumba Road to their east. Depths of overland flow were generally found to be less than 50 mm in these properties, although pockets of deeper flowing water may have been present immediately west (downslope) of Tumbarumba Road.[17]

4.4.2 March 2012 Flood

Figure 4.8 shows the water surface profile along the modelled reach of Kyeamba Creek, and **Figure 4.10** indicative depths of inundation and water surface contours for the March 2012 flood.

Whilst no flood marks were available for the March 2012 flood, the model was able to reproduce the peak height recorded by the Ladysmith stream gauge. Table 20.1 in Yeo, 2013 also contains a note which states that NSWSES flood intelligence identified that above floor inundation was experienced in the residence at No. 9080 Tumbarumba Road on 4 March 2012 (refer **Figure 4.10** for location of property). Whilst the depth of above floor inundation is not given, it is noted that the computed peak flood level of RL 200.81 m AHD is 60 mm above the surveyed floor level of the residence.

[17] No information on historic flooding was provided by the owners of these properties in response to the Community Information Flyer that was disseminated at the commencement of the present investigation.

The peak height on the Ladysmith stream gauge of 6.13 m was recorded at 09:15 hrs on 4 March, which compares closely to the modelled peak of 10:00 hrs. Similar to the October 2010 flood, no other time based data is available for Ladysmith to allow further comparison between modelled and observed flood behaviour.

Whilst the peak flow on the main arm of Kyeamba Creek was less than that experienced in October 2010, localised heavy rainfall likely resulted in greater depths of overland flow in the urbanised parts of the village.

4.5 Testing of Uranquinty TUFLOW Model

4.5.1 October 2010 Flood

Figure 4.11 (2 Sheets) shows the water surface profile along the main arm of Sandy Creek, the Town Levee (North and South) and the Connorton Street Levee, while **Figure 4.12** (2 Sheets) shows indicative depths of inundation, water surface contours and flow direction arrows (Sheet 2 of 2 only) for the October 2010 flood. **Appendix C** contains several plates which show the flooding that was experienced in parts of Uranquinty on 15 October 2010. The locations of flood marks referred to in the following discussion are shown on **Figure 4.12** (Sheet 2 of 2).

Patterns of overland flow and extents of inundation generated by the Uranquinty TUFLOW Model are in close agreement with those shown on Figure 15.3 in Bewsher, 2011.

A large percentage of the floodwater generated by the upstream catchment surcharged the right (eastern) bank of Sandy Creek a short distance upstream of the village, where it flowed toward Deane Street at depths of up to about 700 mm. **Table 4.3** gives the distribution of flow across the Sandy Creek floodplain upstream of the village at the peak of the October 2010 flood.

TABLE 4.3
DISTRIBUTION OF FLOW ACROSS SANDY CREEK FLOODPLAIN
HISTORIC FLOOD EVENTS

Historic Flood Event	Peak Discharge (m^3/s)		
	Left (Western) Overbank	Channel	Right (Eastern) Overbank
October 2010	31	12	97
March 2012	20	12	75

Floodwater was initially conveyed along the toe of the Town Levee (South) toward the Olympic Highway road culverts in the engineered sections of channel. However, increases in flow on the right overbank of Sandy Creek resulted in water levels exceeding the height of Deane Street early on the afternoon of 15 October. Bewsher, 2011 notes that floodwater was observed to overtop Deane Street commencing at about 15:30 hours on 15 October, whereas the flood models show Deane Street commencing to be overtopped 40 minutes earlier at about 14:50 hours. The flood models also show the flood peaking earlier at about 18:30 hours, whereas Bewsher, 2011 notes that residents indicated that the flood peaked at around 19:00 hours on 15 October. The minor differences in the observed and modelled times is attributed to limitations in the available data coupled with the adopted modelling approach (e.g. use of Wagga Wagga AWS data to describe the temporal variability of rainfall across the whole of the Sandy Creek catchment and the adoption of the simple lag approach to defining the travel time of the flood wave down the valley).

The flood models were able to reproduce the locations where Deane Street was observed to have been overtopped and where floodwater discharged through existing residential development (e.g. Nos. 6 and 10 Connorton Street and Nos. 1 and 3 Deane Street).

The TUFLOW models show that the 97 m^3/s which surcharged the right bank of Sandy Creek and flowed toward Deane Street, about one fifth, or 20 m^3/s surcharged the road where it impacted existing development located behind the Town Levee (South).

The flood models were also able to reproduce the height to which water reached in existing development located behind the Town Levee (South), with differences between modelled and observed flood heights shown to be less than 0.1 m. Flood Mark FMU7, which lies on the eastern (upstream) side of Deane Street appears to be too low, given that the recorded level lies below the elevation of the crown in the road, which photos show was overtopped by the floodwater.

The observed overtopping of Connorton Street Levee was also reproduced by the flood models. The timing of the overtopping event as predicted by the flood models is similarly 30 minutes earlier that was observed by residents.

Properties located to the north of Connorton Street (e.g. along O'Conner Street and Spaul Street) are shown to have been affected by relatively shallow overland flow. Overland flow is also shown to have inundated Morgan Street at its intersection with Yarragundry Street to depths exceeding 200-300 mm, a feature which was noted in Bewsher, 2011.

Minor overtopping is shown to have occurred along the Sydney-Melbourne Railway Line opposite Ryan Street, with floodwater impacting on existing development located at the western end of Pearson Street and Best Street. Depths of overland flow in this area are relatively shallow, typically in the range 0-200 mm. This finding is not necessarily consistent with those of Bewsher, 2011 which noted that water levels reportedly reached to within 1-2 inches of the railway tracks, inferring that overtopping of the Sydney-Melbourne Railway Line did not occur during the flood event.

Whilst not reported in Bewsher, 2011, the flood models indicate that floodwater ponded behind the Town Levee (North) at the inlet of the two pipes which have been fitted with floodgates as they discharge through the levee to Sandy Creek. The ponding was a result of elevated water levels in Sandy Creek which caused the flood gates to close.

4.5.2 March 2012 Flood

Figure 4.11 (2 Sheets) shows the water surface profile along the main arm of Sandy Creek, the Town Levee (North and South) and the Connorton Street Levee, while **Figure 4.13** (2 Sheets) shows indicative depths of inundation, water surface contours and flow direction arrows (Sheet 2 of 2 only) for the March 2012 flood. **Appendix C** contains several plates which show the flooding that was experienced in parts of Uranquinty on 4 March 2012. The locations of flood marks referred to in the following discussion are shown on **Figure 4.13**. Reference to historic flooding in the following discussion is taken from Yeo, 2013.

Similar to the October 2010 flood, the majority of flow generated by the upstream catchment surcharged the right (eastern) bank of Sandy Creek and flowed toward Deane Street, although at a slightly shallower depth. **Table 4.3** gives the distribution of flow across the Sandy Creek floodplain upstream of the village at the peak of the March 2012 flood.

Whilst RFS logs indicate that floodwater was overtopping Deane Street at about 07:00 hours on 4 March, NSWSES Requests for Assistance (**RFA's**) show that No. 10 Morgan Street was evacuated due to rising floodwater at around 00:00 hours on the same day. This timing is consistent with the results of flood modelling, which showed overtopping of Deane Street commenced at around 23:30 hours on 3 March, 2012.

Whilst efforts were made on the morning of 4 March to 'top up' the height of the Town Levee (South), this likely did not affect the elevation which water reached behind the Town Levee (South) given the flood models show the peak likely occurred before day break at around 03:30 hours.[18]

Whilst peak flood levels on the southern (upstream) side of Deane Street were only slightly lower than occurred in the October 2010 flood (RL 201.65 m AHD in October 2010 versus RL 201.59 m AHD in March 2012), there was a significant reduction in the peak flow which surcharged Deane Street west of Connorton Street. Whereas a peak discharge of about 20 m^3/s is estimated to have surcharged Deane Street in the October 2010 flood, only about 6 m^3/s is estimated to have surcharged the roadway in March 2012. This result highlights the major impact minor differences in peak flood levels can have on flooding conditions in existing development which lies behind the levee bank.

Whilst the flood models closely matched observed flood behaviour in the vicinity of Morgan Street, flood mark FMU10 appears too low when compared to the other two recorded levels, given that all three are located in the level pool which formed behind the Town Levee (South).

Oddly, ponding was observed to extend into several properties which are located behind the Town Levee (North) in King Street and Barker Street, a feature which was not reported following the larger October 2010 flood.[19]

The flood models indicate that minor overtopping of the Connorton Street Levee occurred at around 15:40 hours on 3 March 2012, although there are no records of this having occurred. It is possible that the height of the levee was raised following the October 2010 flood, a feature which is not reflected in the LiDAR survey data used to develop the hydraulic model.

4.6 Model Structure and Parameters for Design Flood Modelling

Based on the findings of the model testing process, the hydraulic models that have been developed to represent conditions which were present on the floodplain at the time of the March 2012 flood (i.e. those models that include the Tarcutta Bypass (Hume Highway Upgrade) in the case of the Tarcutta TUFLOW Model and the scoured bridge opening in the case of the Ladysmith TUFLOW Model), should be used as the basis for defining flooding behaviour at the three villages for design events up to 500 year ARI, as well as for the PMF.

[18] One RFA indicates that sandbags were requested at around 04:00 hours on 4 March 2012 due to rising floodwater in No. 14 Morgan Street.

[19] Four out of the five properties which were identified as being subject to flooding in this area arose as a result of a Rapid Impact Assessment (**RIA**) which was undertaken immediately following the flood event (i.e. not as a result of complaints by residents), which might explain why similar observations were not made following the October 2010 flood.

5 REFERENCES

Bewsher (Bewsher Consulting), 2011. *"Flood Intelligence, Collection and Review for Towns and Villages in the Murray and Murrumbidgee Regions following the October 2010 Flood".* Final Draft Report.

IEAUST (Institution of Engineers Australia), 1998 *"Australian Rainfall and Runoff – A Guide to Flood Estimation"*

L&A (Lyall & Associates), 2012 *"Tarcutta, Ladysmith and Uranquinty Flood Studies – Data Collection Report"*

Walsh et al (Walsh, M.A, Pilgrim, D.H, Cordery, I) (1991). *"Initial Losses for Design Flood Estimation in New South Wales"* Intn'l Hydrology & Water Resources Symposium, Perth.

Yeo (Stephen Yeo), 2013. "*Flood Intelligence, Collection and Review for Towns and Villages in the Murray and Murrumbidgee Regions following the March 2012 Flood".* Final Draft Report.

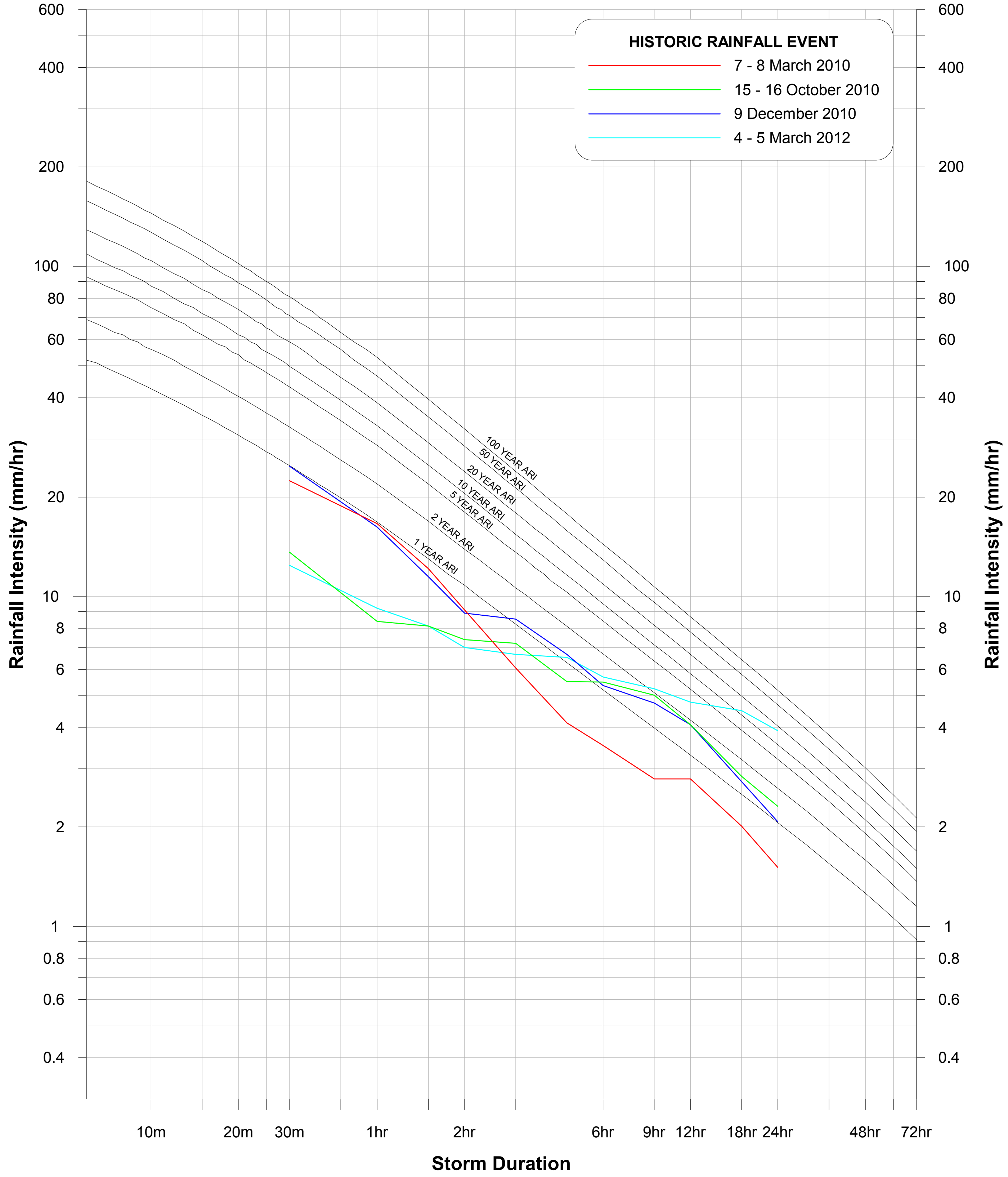
6 FLOOD-RELATED TERMINOLOGY

Note: For an expanded list of flood-related terminology, refer to glossary contained within the Floodplain Development Manual, NSW Government, 2005).

TERM	DEFINITION
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger events occurring in any one year (see average recurrence interval).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Recurrence Interval (ARI)	The average period in years between the occurrence of a flood of a particular magnitude or greater. In a long period of say 1,000 years, a flood equivalent to or greater than a 100 year ARI event would occur 10 times. The 100 year ARI flood has a 1% chance (i.e. a one-in-100 chance) of occurrence in any one year (see annual exceedance probability).
Catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
Discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving (e.g. metres per second [m/s]).
Flood prone land	Land susceptible to flooding by the Probable Maximum Flood. Note that the flood prone land is synonymous with flood liable land.
Flood storage area	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
Floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event (i.e. flood prone land).
Mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
Overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
Peak discharge	The maximum discharge occurring during a flood event.

TERM	DEFINITION
Peak flood level	The maximum water level occurring during a flood event.
Probable Maximum Flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land (i.e. the floodplain). The extent, nature and potential consequences of flooding associated with events up to and including the PMF should be addressed in a floodplain risk management study.
Probability	A statistical measure of the expected chance of flooding (see annual exceedance probability).
Runoff	The amount of rainfall which actually ends up as stream flow, also known as rainfall excess.
Stage	Equivalent to water level (both measured with reference to a specified datum).

ANNEXURE A
INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS



Note:
Dates given in the legend
relate to rain days,
not calendar days.

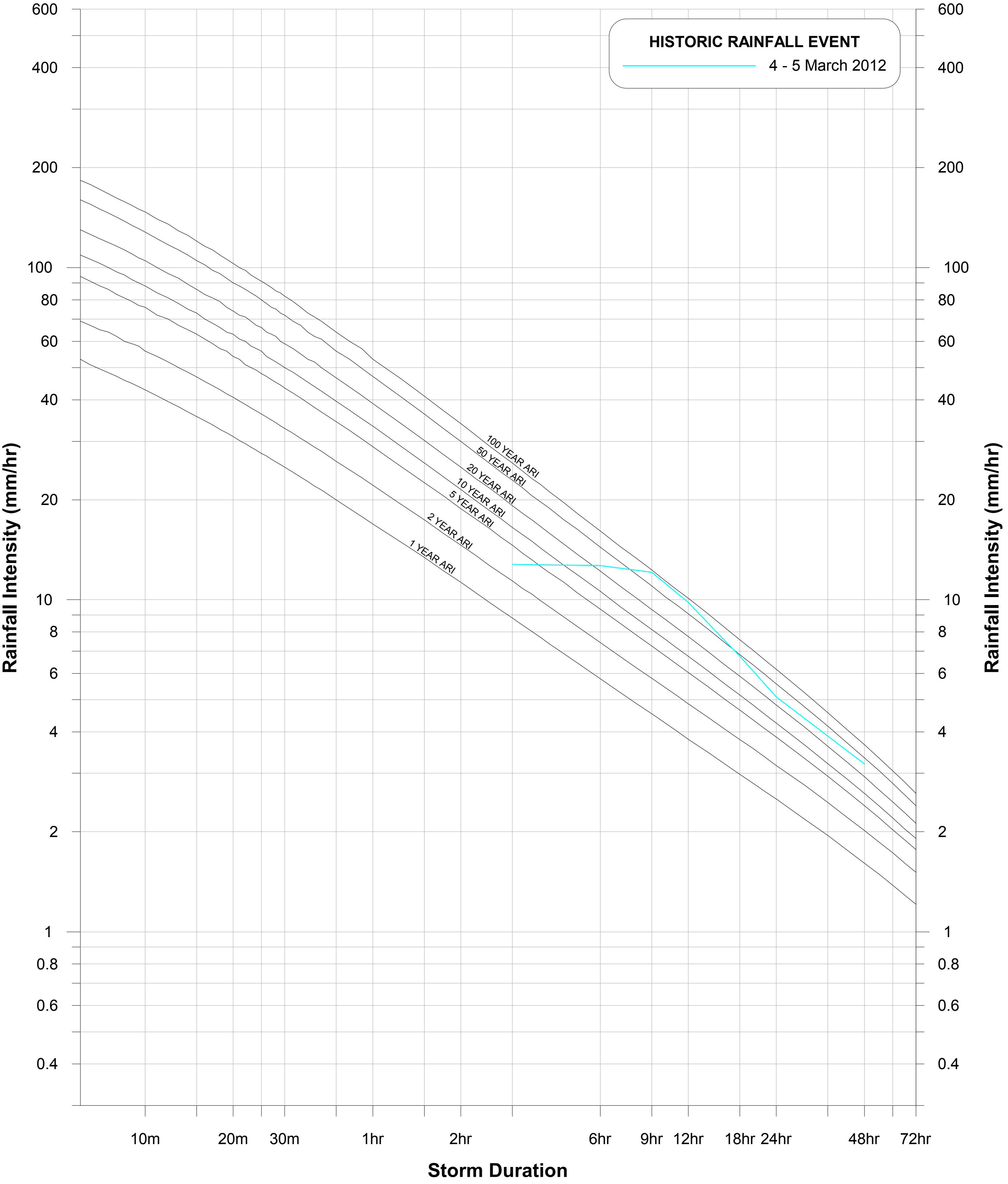
TARCUTTA, LADYSMITH AND URANQUINTY FLOOD STUDIES
DEVELOPMENT AND TESTING OF FLOOD MODELS

Figure A1

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
ADELONG (ETHAM PARK) (GS 72159)

Lyall & Associates

Document Set ID: 3475086
Version: 1, Version Date: 09/09/2015



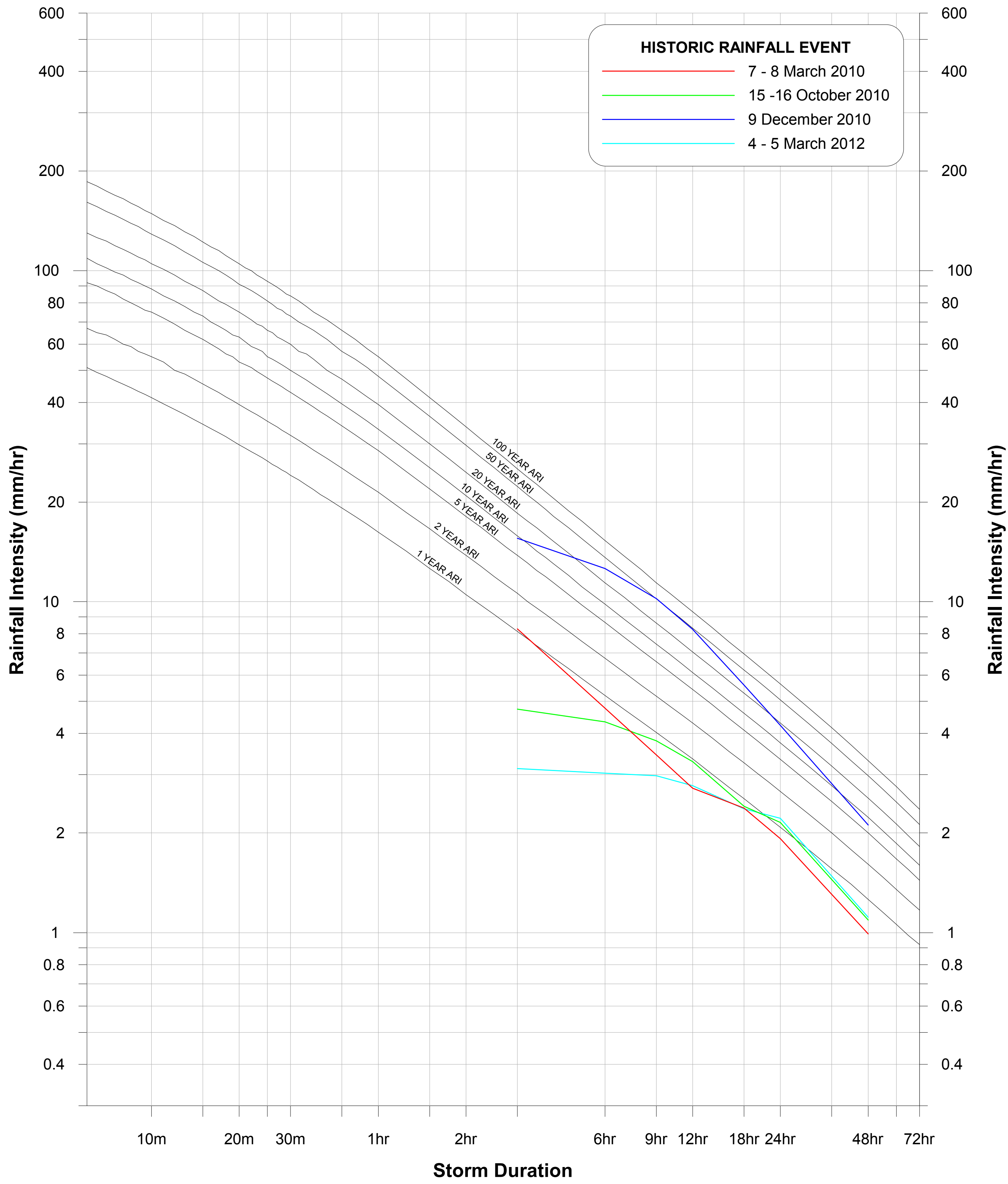
Note:
Dates given in the legend
relate to rain days,
not calendar days.

TARCUTTA, LADYSMITH AND URANQUINTY FLOOD STUDIES
DEVELOPMENT AND TESTING OF FLOOD MODELS

Figure A2

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
BATLOW (GS 72004)

Lyall & Associates

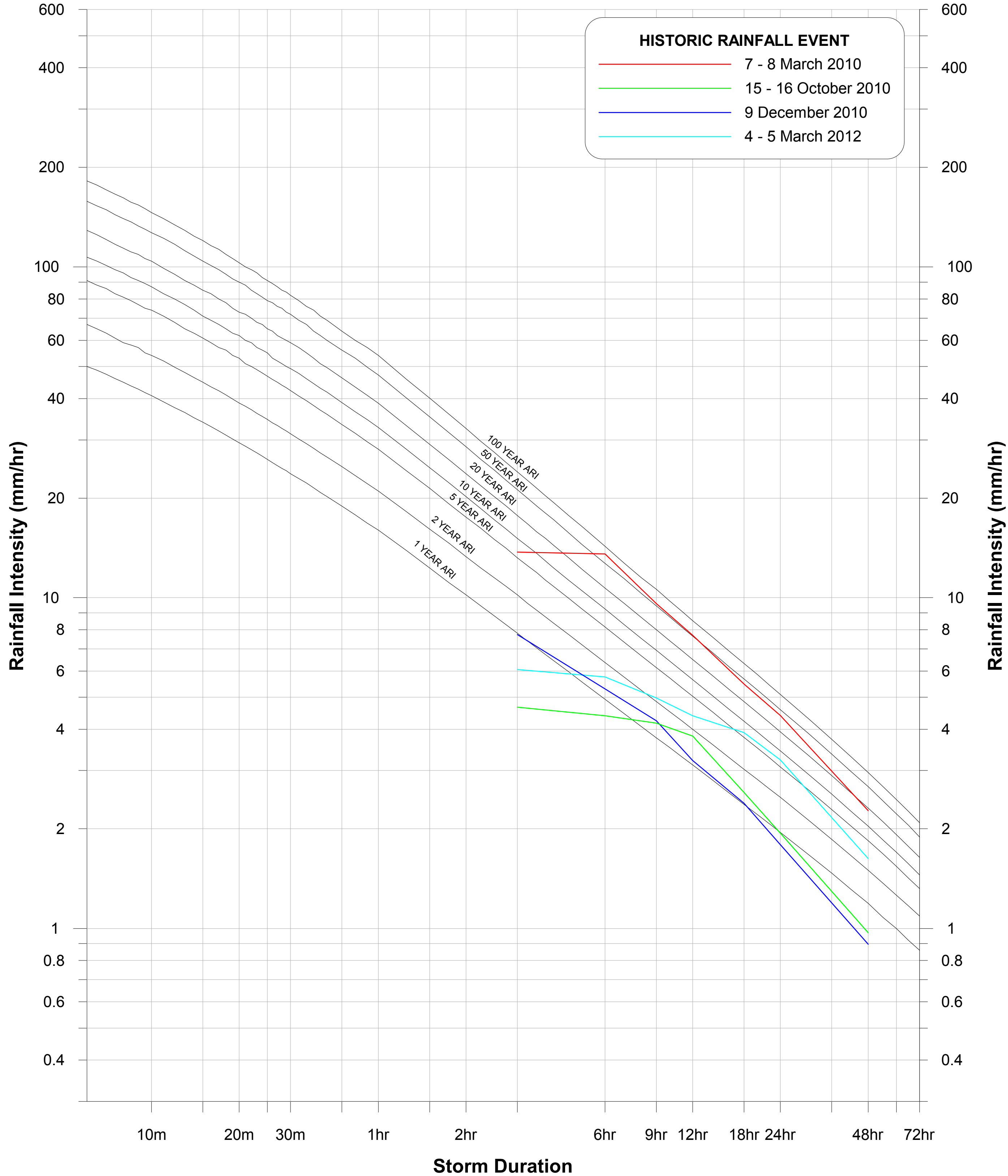


Note:
Dates given in the legend
relate to rain days,
not calendar days.

TARCUTTA, LADYSMITH AND URANQUINTY
DEVELOPMENT AND TESTING OF FLOOD MODELS

Figure A3

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
BELMORE BRIDGE (GS 572010)



Note:
Dates given in the legend relate to rain days, not calendar days.

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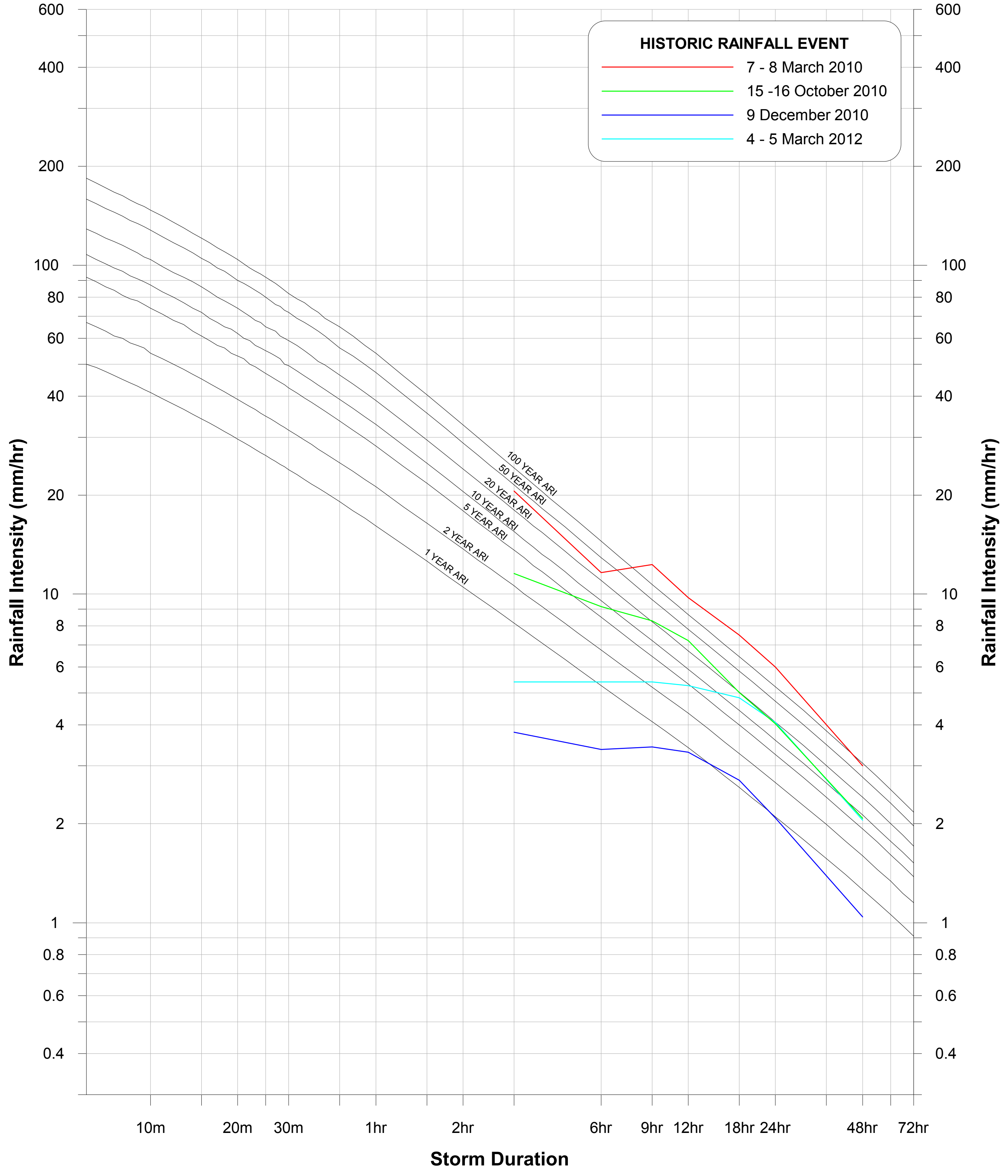
TARCUTTA, LADYSMITH AND URANQUINTY
DEVELOPING AND TESTING OF FLOOD MODELS

Figure A4

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
BOOK BOOK (GS 572008)



Note:
Dates given in the legend relate to rain days, not calendar days.

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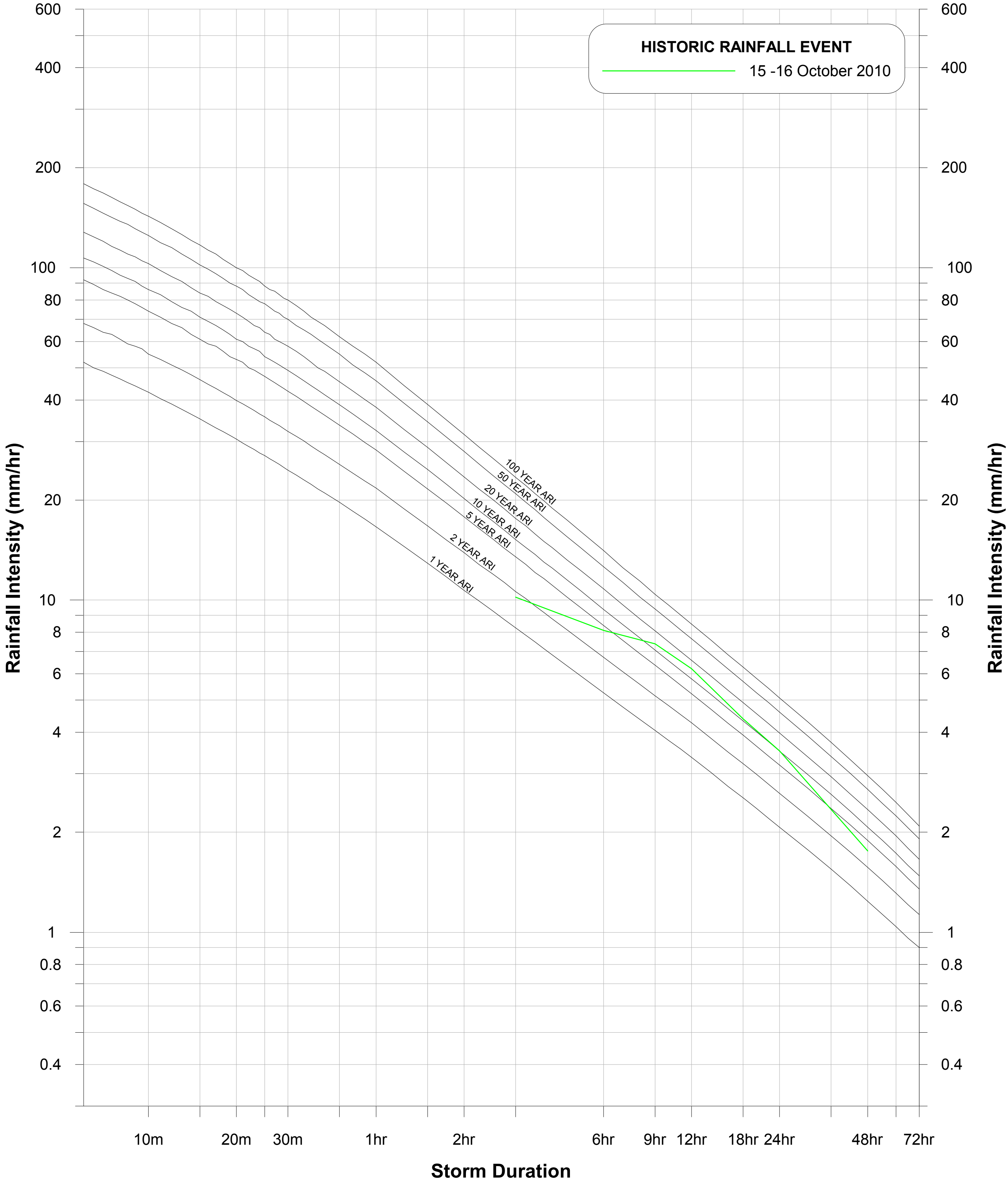
TARCUTTA, LADYSMITH AND URANQUINTY DEVELOPMENT AND TESTING OF FLOOD MODELS

Figure A5

INTENSITY-FREQUENCY-DURATION CURVES AND HISTORIC STORM RAINFALLS CARABOST (GS 72012)



Note:
Dates given in the legend
relate to rain days,
not calendar days.

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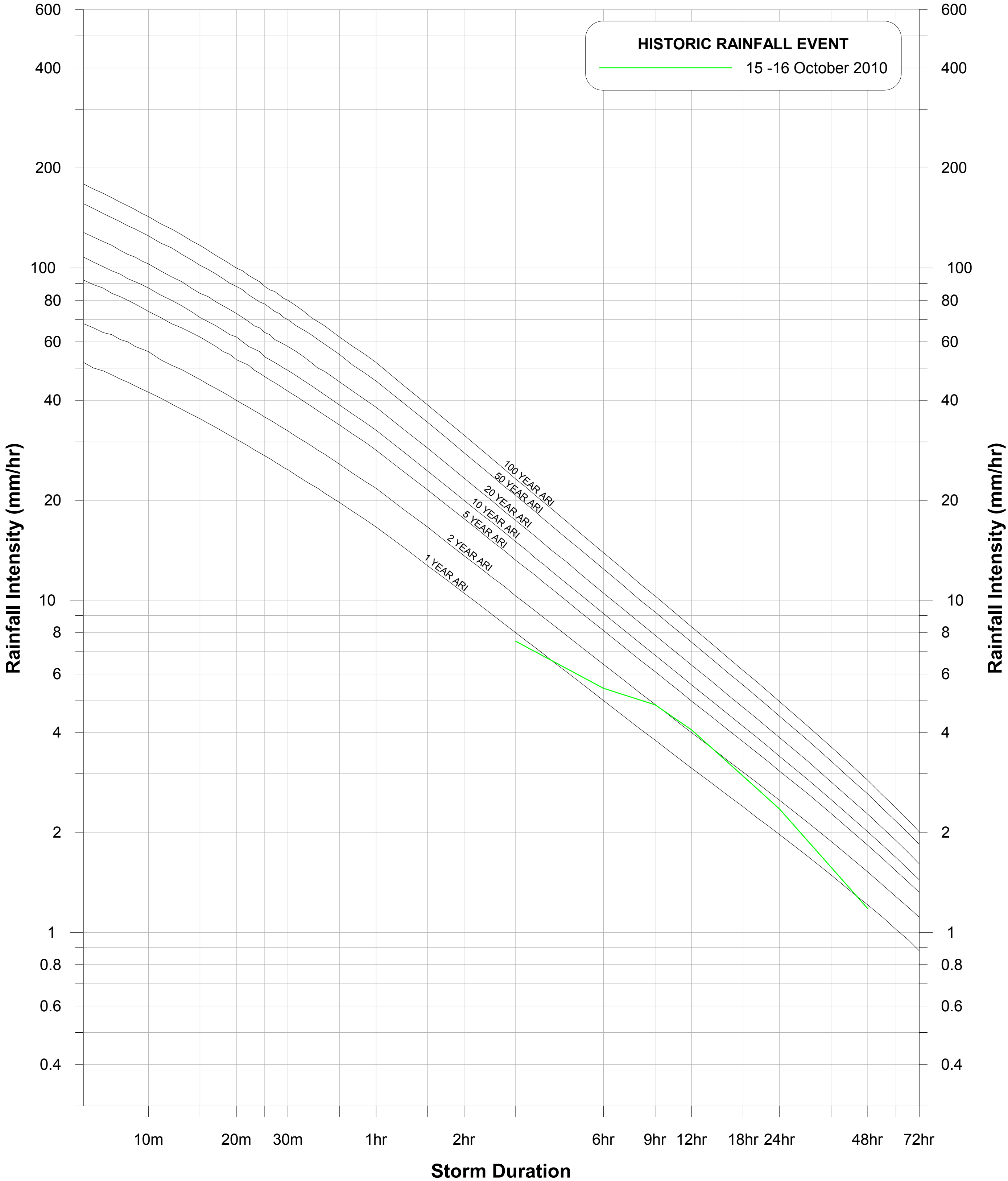
TARCUTTA, LADYSMITH AND URANQUINTY
DEVELOPMENT AND TESTING OF FLOOD MODELS

Figure A6

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
HUMULA

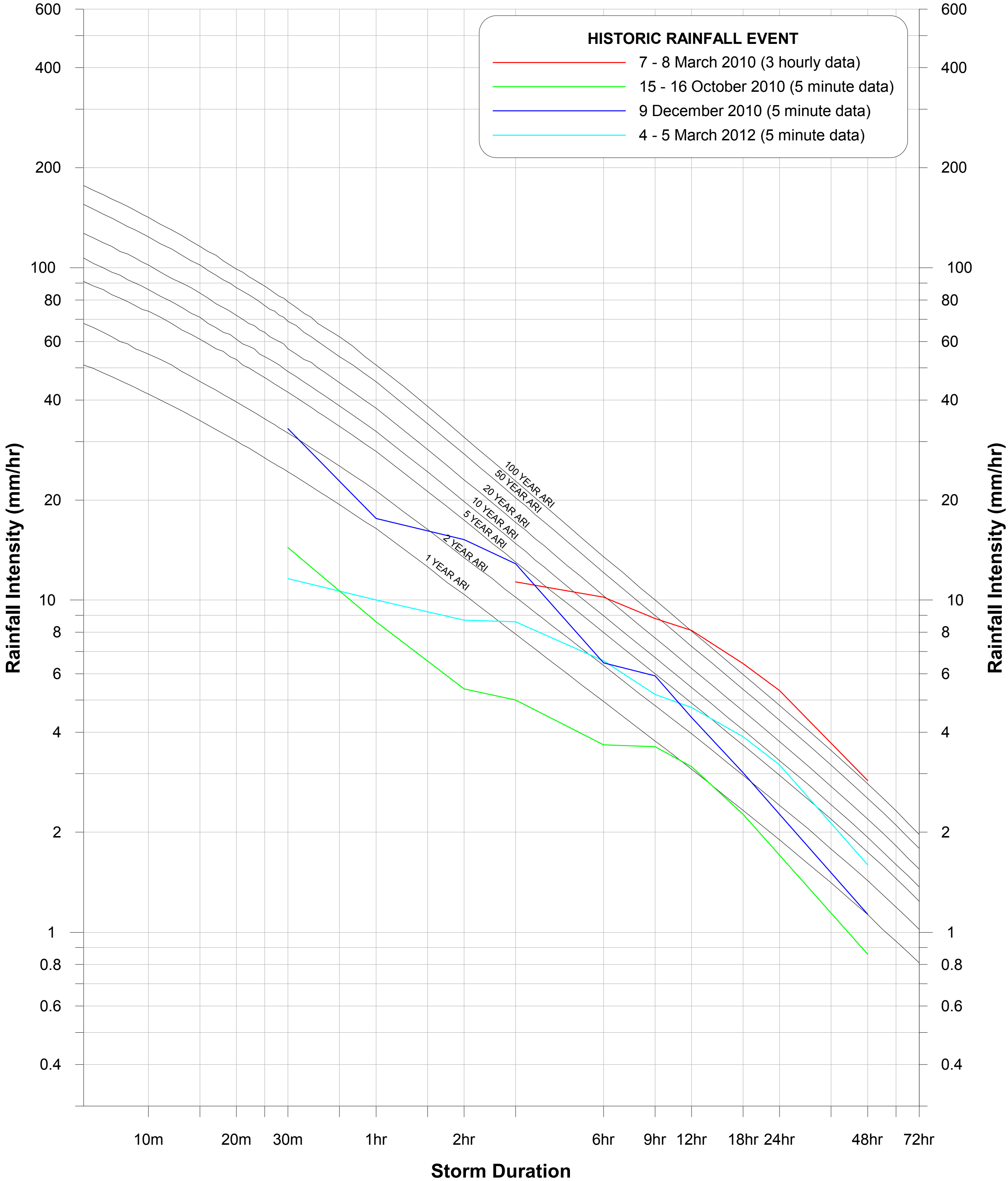


Note:
Dates given in the legend
relate to rain days,
not calendar days.

TARCUTTA, LADYSMITH AND URANQUINTY
DEVELOPMENT AND TESTING OF FLOOD MODELS

Figure A7

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
TARCUTTA



Note:
Dates given in the legend
relate to rain days,
not calendar days.

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TARCUTTA, LADYSMITH AND URANQUINTY
DEVELOPMENT AND TESTING OF FLOOD MODELS

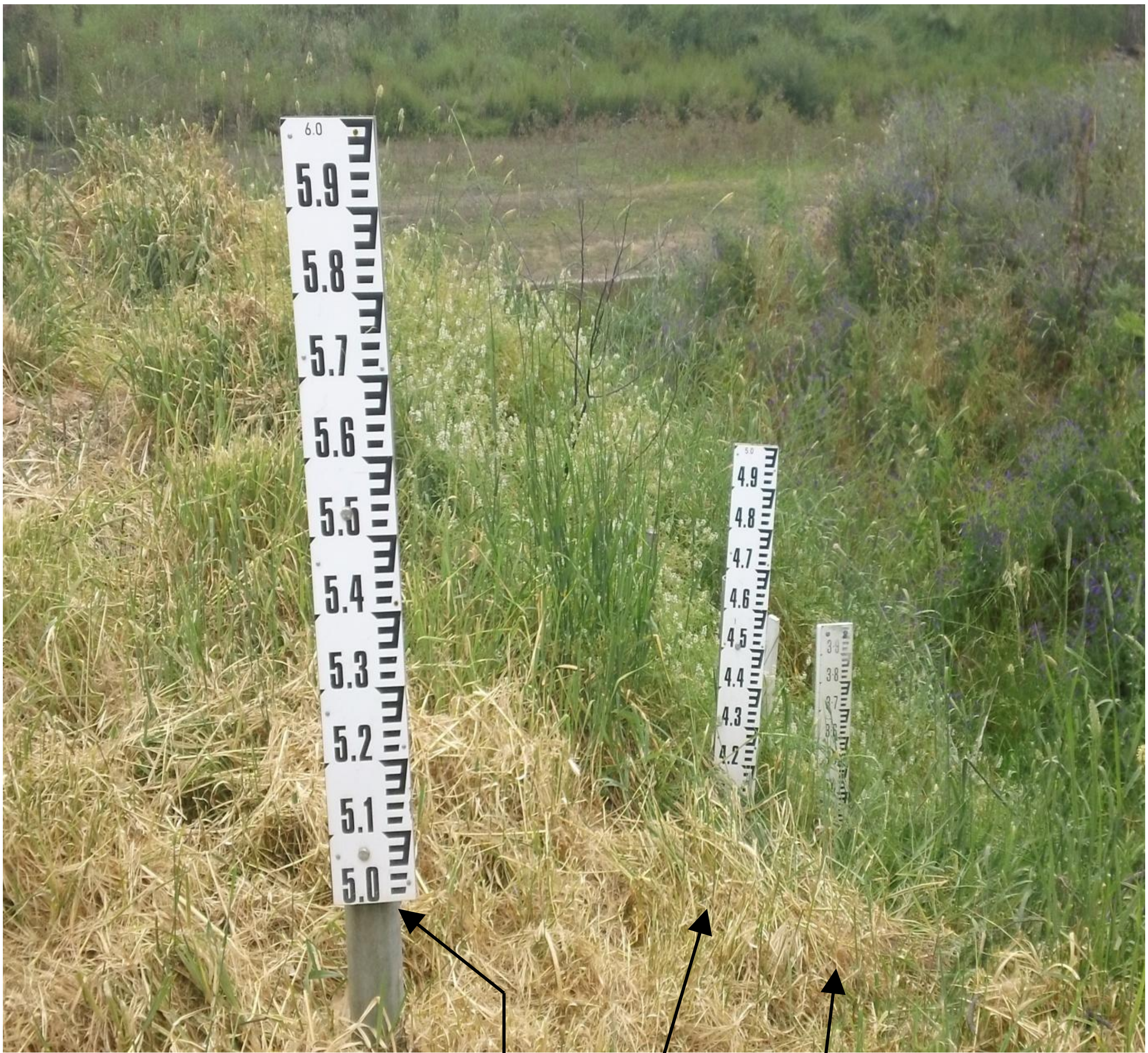
Figure A8

INTENSITY-FREQUENCY-DURATION CURVES
AND HISTORIC STORM RAINFALLS
WAGGA WAGGA AWS (GS 72150)

ANNEXURE B
TARCUTTA DEPTH GAUGE SURVEY

TARCUTTA CREEK

FLOOD GAUGE AT TARCUTTA BRIDGE



5.0m Gauge Height 229.63m AHD

4.0m Gauge Height 228.68m AHD

3.0m Gauge Height 227.68m AHD

ANNEXURE C
OLD BORAMBOLA STREAM GAUGE DATA

TABLE C1
RECORDED PEAK HEIGHT AND DISCHARGE DATA IN DATE ORDER
OLD BORAMBOLA STREAM GAUGE(1)

Year	Peak Height (m)	Peak Discharge (m^3/s)		Year	Peak Height (m)	Peak Discharge (m^3/s)	
		Recorded	Adjusted(2)			Recorded	Adjusted(2)
1939	4.572	271.0	403.1	1976	4.123	170.3	204.6
1940	0.71	4.6	-	1977	3.413	92.5	-
1941	1.829	34.4	-	1978	4.68	269.3	458.2
1942	3.048	104.9	-	1979	2.33	41.8	-
1943	1.625	26.3	-	1980	2.999	73.1	-
1944	0.585	3.0	-	1981	4.638	260.3	436.4
1945	2.591	73.7	-	1982	0.911	5.1	-
1946	2.082	44.4	-	1983	5.203	400.9	766.0
1947	2.438	64.6	-	1984	4.015	155.1	-
1948	1.548	21.8	-	1985	2.54	52.0	-
1949	3.81	138.9	-	1986	4.262	191.8	260.8
1950	3.962	148.0	-	1987	2.916	69.0	-
1951	3.709	118.7	-	1988	3.915	141.8	-
1952	4.014	155.0	-	1989	3.557	110.0	-
1953	2.158	38.7	-	1990	4.252	190.1	256.6
1954	2.071	33.9	-	1991	2.184	36.9	-
1955	4.139	172.7	210.9	1992	4.884	315.5	570.2
1956	4.437	221.9	338.2	1993	4.649	262.6	442.1
1957	1.175	11.0	-	1994	1.293	11.1	-
1958	3.095	75.6	-	1995	4.356	207.6	301.4
1959	1.321	14.5	-	1996	3.189	84.3	-
1960	3.136	77.7	-	1997	1.507	18.0	-
1961	1.956	31.2	-	1998	3.741	122.2	-
1962	1.802	26.4	-	1999	2.729	62.9	-
1963	1.458	16.7	-	2000	3.432	97.4	-
1964	3.596	108.6	-	2001	1.311	14.5	-
1965	1.187	9.0	-	2002	1.15	10.7	-
1966	4.186	179.6	229.5	2003	2.16	41.2	-
1967	0.947	4.2	-	2004	1.971	30.3	-
1968	3.162	79.1	-	2005	4.204	182.4	236.8
1969	3.697	116.1	-	2006	0.59	1.7	-
1970	4.952	331.9	609.9	2007	1.364	12.9	-
1971	2.366	42.9	-	2008	1.195	9.0	-
1972	2.176	34.6	-	2009	2.001	31.3	-
1973	3.802	127.6	-	2010	5.424	447.2	916.3
1974	5.22	405.9	777.1	2011	3.311	101.0	-
1975	4.249	189.6	255.3	2012	4.861	317.0	557.1

1. Gauge zero = RL 190.699 m AHD
2. Peak discharge adjusted in accordance with **Equation 2.1** in **Section2.5.1.**

TABLE C2
RECORDED PEAK HEIGHT AND DISCHARGE DATA IN ORDER OF MAGNITUDE
OLD BORAMBOLA STREAM GAUGE (1)

Year	Peak Height (m)	Peak Discharge (m^3/s)		Year	Peak Height (m)	Peak Discharge (m^3/s)	
		Recorded	Adjusted(2)			Recorded	Adjusted(2)
2010	5.424	447.2	916.3	1942	3.048	104.9	-
1974	5.22	405.9	777.1	1980	2.999	73.1	-
1983	5.203	400.9	766	1987	2.916	69	-
1970	4.952	331.9	609.9	1999	2.729	62.9	-
1992	4.884	315.5	570.2	1945	2.591	73.7	-
2012	4.861	317	557.1	1985	2.54	52	-
1978	4.68	269.3	458.2	1947	2.438	64.6	-
1993	4.649	262.6	442.1	1971	2.366	42.9	-
1981	4.638	260.3	436.4	1979	2.33	41.8	-
1939	4.572	271	403.1	1991	2.184	36.9	-
1956	4.437	221.9	338.2	1972	2.176	34.6	-
1995	4.356	207.6	301.4	2003	2.16	41.2	-
1986	4.262	191.8	260.8	1953	2.158	38.7	-
1990	4.252	190.1	256.6	1946	2.082	44.4	-
1975	4.249	189.6	255.3	1954	2.071	33.9	-
2005	4.204	182.4	236.8	2009	2.001	31.3	-
1966	4.186	179.6	229.5	2004	1.971	30.3	-
1955	4.139	172.7	210.9	1961	1.956	31.2	-
1976	4.123	170.3	204.6	1941	1.829	34.4	-
1984	4.015	155.1	-	1962	1.802	26.4	-
1952	4.014	155	-	1943	1.625	26.3	-
1950	3.962	148	-	1948	1.548	21.8	-
1988	3.915	141.8	-	1997	1.507	18	-
1949	3.81	138.9	-	1963	1.458	16.7	-
1973	3.802	127.6	-	2007	1.364	12.9	-
1998	3.741	122.2	-	1959	1.321	14.5	-
1951	3.709	118.7	-	2001	1.311	14.5	-
1969	3.697	116.1	-	1994	1.293	11.1	-
1964	3.596	108.6	-	2008	1.195	9	-
1989	3.557	110	-	1965	1.187	9	-
2000	3.432	97.4	-	1957	1.175	11	-
1977	3.413	92.5	-	2002	1.15	10.7	-
2011	3.311	101	-	1967	0.947	4.2	-
1996	3.189	84.3	-	1982	0.911	5.1	-
1968	3.162	79.1	-	1940	0.71	4.6	-
1960	3.136	77.7	-	2006	0.59	1.7	-
1958	3.095	75.6	-	1944	0.585	3	-

1. Gauge zero = RL 190.699 m AHD
2. Peak discharge adjusted in accordance with **Equation 2.1** in **Section2.5.1.**