TARCUTTA, LADYSMITH AND URANQUINTY FLOOD STUDIES

DESIGN FLOOD MODELLING

VOLUME 1 - REPORT

DRAFT REPORT FOR PUBLIC EXHIBITION

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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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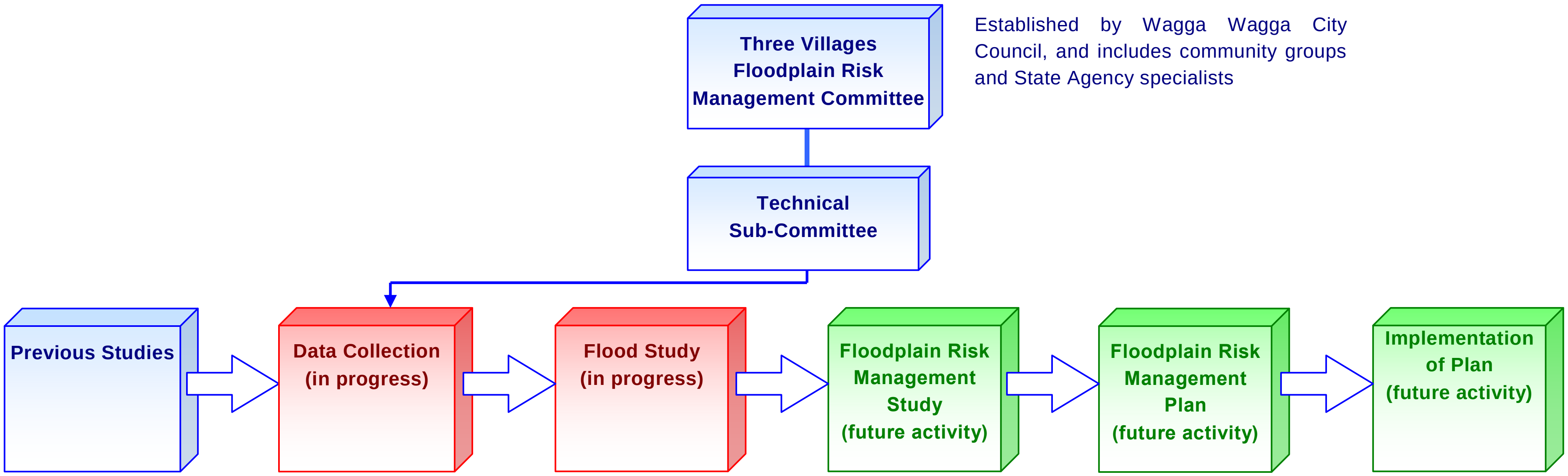
FLOODPLAIN RISK MANAGEMENT PROCESS



Previous Studies: 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).

Data Collection: Involves the compilation of existing data and the collection of additional data.

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

Floodplain Risk Management Study: 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.

Floodplain Risk Management Plan: 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 Plan: 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 per cent 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
FRSM&P	Floodplain Risk Management Study and Plan
GEV	General Extreme Value
IFF	Imminent Failure Flood
LiDAR	Light Detection and Ranging (aerial survey)
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])
MOF	Major Overland Flow
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 7 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 – Design Flood Modelling* is the final of the three reports dealing with the flood studies project and presents the results of the modelling undertaken for design flood analysis. The objective of the studies is to define flood behaviour at the three villages *under present day conditions* for design floods ranging between 5 and 500 year ARI, as well as for the PMF. For the purposes of defining flood behaviour, 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 these flows into flood levels, flow patterns and velocities. The hydrologic models were based on the RAFTS – DRAINS rainfall-runoff software, while the hydraulic models were based on the TUFLOW two-dimensional modelling system.

The report builds on the results of two companion studies:

- *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 in that report led to the collection of additional survey data including floor levels of properties which experienced above floor inundation during the major flood of October 2010. An in-bank cross sectional survey was also undertaken at locations where both scour and deposition of bed material was observed to have occurred in October 2010, as well as survey of critical hydraulic structures. Another key outcome was the decision to prepare a separate TUFLOW model of the Tarcutta Creek floodplain in lieu of the adoption of the TUFLOW model which had been developed for the investigation and design of the Hume Highway Upgrade.
- *Tarcutta, Ladysmith and Uranquinty Flood Studies – Development and Testing of Flood Models* (L&A, 2014). Stream flow and rainfall data were available for testing the respective hydrologic models of the Tarcutta Creek catchment at Tarcutta and for Kyeamba Creek at Ladysmith. The Sandy Creek catchment at Uranquinty is ungauged. Historic flood level data at the three villages assisted with the testing of the hydraulic models. The report described the preparation and calibration of the flood models developed for the three villages.

For design flood estimation, the RAFTS - DRAINS hydrologic model parameters for the two gauged catchments were used as a guide to assigning design parameters for the ungauged Sandy Creek catchment at Uranquinty. Similarly, parameters found to apply for the testing of the TUFLOW hydraulic models guided the selection of parameters for design flood analysis.

S1.2 Presentation of Results

The results are presented as water surface profiles along the main arms of the creek systems, as well as diagrams showing depths and indicative extents of inundation and flood contours. Stage and discharge hydrographs at key locations within each village are also presented.

The report includes diagrams showing the sensitivity of results to variations in hydraulic roughness of the channels and floodplains, as well as potential increases in flood levels as a result of future climate change. Figures are also presented showing the division of the floodplain into provisional flood hazard and hydraulic categories for the 100 year ARI event. The extent of the *Interim Flood Planning Area (**FPA**)* for Main Stream flooding, set equal to the area inundated by the 100 year ARI peak flood level plus 500 mm freeboard, is also presented at each village. In accordance with current practice, definition of the *FPA* in areas subject to the shallow and slower moving *Major Overland Flow* (***MOF***) through the village areas is left to the future *Floodplain Risk Management Study and Plan* (***FRMS&P***).

Chapter 2 of the report sets out the technical basis of the design flood assessment, which is common to each village, while results at Tarcutta, Ladysmith and Uranquinty respectively are discussed in **Chapters 3** to **5**.

S1.3 Key Findings

S1.3.1 General for the Three Villages

The flood models developed in the *Flood Studies* are suitable for use in the *FRMS&P* to examine strategies for the mitigation of flooding problems at the three villages. The following activities will be undertaken:

- Matching the extents and depths of inundation determined in the flood studies with the footprints and floor levels of existing residential development to estimate damages resulting from a range of flood events.
- Detailed hydraulic analysis of the upgrading of the various levees at Tarcutta and Uranquinty.
- Hydraulic and economic analysis of potential works at the strategic level of detail. This will enable a priority list of flood management measures to be prepared.

S1.3.2 Tarcutta

The key findings of the design flood modelling as they relate to Tarcutta were as follows:

- The design flood modelling results at Tarcutta represent an envelope of flows and levels occurring within the modelled area both with and without partial levee failure due to scour when the **Imminent Failure Flood** (**IFF**) level is exceeded. (**Section 2.7.4** explains the IFF concept and the assumed failure mechanism.) At some locations in the floodplain e.g. in protected areas, flows and levels will be increased by levee failure and in others they will be reduced. The IFF's for the levees at Tarcutta range between 20 year ARI for the Hambledon Levee and 20 – 25 years ARI for the Old Tarcutta Inn Levee (inner block wall) and the Tarcutta Levee.
- At the 100 year ARI, floodwaters on Tarcutta Creek extend over a width of 800 - 900 m along the extent of the study reach (**Figure 3.7**). The urban area of the village is also subject to MOF inundation from the local sub-catchments which drain westwards to Tarcutta Creek. Significant overland flows commence at the 10 year ARI level of flooding due to surcharges of the trunk drainage system.
- The time of rise of Tarcutta Creek under design flood conditions is around 15 hours. (**Figure 3.2**). The response time of the MOF paths through the urban area is shorter, with a time of rise generally limited to less than one hour.
- A preliminary assessment of the increase in 100 year ARI flood levels in protected areas due to partial failure of the levees is shown in **Figure 3.14**. Peak 100 year ARI water levels within the protected parts of the village could be increased by values in excess of 500 mm compared with the "no-failure" case (The levees will be overtopped with or without failure). A detailed hydraulic analysis involving a more rigorous assessment of the implications of levee failure is warranted during the preparation of the *FRMS&P*.
- Mitigation of existing flooding problems will require the upgrading of the levees. This will be one of the important management measures to be considered in the *FRMS&P*. Selection of appropriate crest levels will require consideration of potential rises in design flood levels due to increased hydraulic roughness in the floodplain, partial blockage of downstream bridge waterways and future climate change.

- The floodway and flood hazard extents shown in **Figure 3.11** are based on depth and velocity of flow considerations and should be regarded as provisional. A final determination of hazard should be made in the *FRMS&P* in areas subject to both Main Stream and MOF inundation, based on additional considerations summarised in **Section 2.6.2**.

S1.3.3 Ladysmith

The key findings of the design flood modelling as they relate to Ladysmith were as follows:

- At the 100 year ARI, floodwaters on Kyeamba Creek extend over a width of 500 – 700 m along the extent of the study reach (**Figure 4.7**). Although the urban part of the village is not affected by main stream flooding, even at the 100 year ARI, it is affected by MOF from the local sub-catchments which drain westwards to Kyeamba Creek. Significant overland flows commence at the 10 year ARI level of flooding due to surcharges of the trunk drainage system.
- The time of rise of Kyeamba Creek under design flood conditions is around 9 to 10 hours. (**Figure 4.2**). The response time of the MOF paths is shorter with a time of rise generally limited to less than two hours.
- The floodway and flood hazard extents shown in **Figure 4.11** are based on depth and velocity of flow considerations and should be regarded as provisional. A final determination of hazard should be made in the *FRMS&P* in areas subject to both Main Stream and MOF inundation, based on additional considerations summarised in **Section 2.6.2**.

S1.3.4 Uranquinty

The key findings of the design flood modelling as they relate to Uranquinty were as follows:

- At the 100 year ARI, floodwaters on Sandy Creek extend over a width of 500 - 1200 m along the extent of the study reach (**Figure 5.7**). The time of rise of Sandy Creek under design flood conditions is around 7 to 8 hours. (**Figure 5.2**).
- The design flood modelling results at Uranquinty represent an envelope of flows and levels occurring within the modelled area both with and without partial levee failure due to scour when the **Imminent Failure Flood** (**IFF**) level is exceeded. The IFF for the Connorton Street and Town Levee (South) systems is 5 years ARI.
- A preliminary assessment of the increase in 100 year ARI flood levels in protected areas due to partial failure of the levees is shown in **Figure 5.14**. Peak 100 year ARI water levels within the protected parts of the village could be increased by values in excess of 500 mm compared with the "no-failure" case with a corresponding increase in the area inundated by floodwater. (The levees will be overtopped with or without failure). A detailed hydraulic analysis involving a more rigorous assessment of the implications of levee failure is warranted during the preparation of the *FRMS&P*.
- Mitigation of existing flooding problems will require the upgrading of the levees. This will be one of the important management measures to be considered in the *FRMS&P*. Selection of appropriate crest levels will require consideration of potential rises in design flood levels due to increased hydraulic roughness in the floodplain, partial blockage of downstream bridge waterways and future climate change.
- The floodway and flood hazard extents shown in **Figure 5.11** are based on depth and velocity of flow considerations and should be regarded as provisional. A final determination of hazard should be made in the *FRMS&P* in areas subject to both Main Stream and MOF inundation, based on additional considerations summarised in **Section 2.6.2**.

1 INTRODUCTION

1.1 Study Catchments

This design flood modelling report deals with flooding at the villages of Tarcutta, Ladysmith and Uranquinty, located within the Local Government Area of Wagga Wagga City Council (**WWCC**). **Figure 1.1** is a plan showing the catchments upstream of the three villages.

1.1.1 Tarcutta Creek

The following three principal sub-catchments make up the 1,341 m^2 catchment contributing to flows at the village of Tarcutta:

- 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 a short distance upstream of the village.

Because of its proximity to Tarcutta, the Old Borambola stream gauge was used as the primary gauge for tuning the RAFTS model of the Tarcutta Creek catchment. As discussed in L&A, 2014, it was necessary to update the high flow portion of Department of Primary Industry - Office of Water's (DPIOW's) rating curve for the gauge as it underestimated the peak discharge in the creek for out-of-bank floods.

1.1.2 Kyeamba Creek

Kyeamba Creek 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.

It was necessary in L&A, 2014 to update the rating curve for the Ladysmith stream gauge which underestimated peak flows in the creek for out-of-bank-floods. The rating curve also did not take account of the increase in conveyance which resulted from the major scour that occurred at Railway Bridge No. 2 during the October 2010 flood.

1.1.3 Sandy Creek

Sandy Creek 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. There are no stream gauges on the Sandy Creek catchment.

1.2 The Drainage System

Data collected and compiled during the L&A, 2012 study, as well as additional information which was obtained during the preparation of the L&A, 2014 study, were used to compile the following figures which show key elements of the drainage system at each village:

- **Figure 1.2**, showing the drainage system in the village of Tarcutta. The system includes the Hambledon, Tarcutta and Old Tarcutta Inn levees, as well as the twin bridges on both Sydney Street and the Hume Highway, the latter of which were under construction at the time of the floods that occurred in March, October and December 2010.

- **Figure 1.3**, showing the drainage system at Ladysmith. The main feature that influences flood behaviour on Kyeamba Creek is the disused railway embankment and its twin openings.
- **Figure 1.4**, showing the drainage system at Uranquinty. The main features influencing flooding are the two crossings of Sandy Creek (Railway and Olympic Highway) and the network of levees which protect existing development from both main stream flooding and local overland flow.

1.3 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.

In L&A, 2014 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 Bureau of Meteorology's (**BoM's**) network of flood warning rain gauges for all four storms, in combination with a large amount of flood intelligence which had been gathered by both NSW State Emergency Service (**NSWSES**) and WWCC, particularly for the October 2010 and March 2012 events.

In this present report, the calibrated models were used as the basis for defining flood behaviour in the three villages for design floods between 5 and 500 year ARI, together with the PMF.

1.4 Overview of Report

Chapter 2 outlines the procedures used in the design flood estimation at each village, including the approach used for the generation of design discharge hydrographs using the RAFTS-DRAINS modelling software and their application to the TUFLOW models to determine flooding patterns. The approach used to test the sensitivity of model results to variations in hydraulic roughness, partial blockage of waterway structures, failure of key levees and future climate change is also outlined.

Chapters 3 to **5** present the results for each village in turn. For each village the results are presented as diagrams showing: water surface profiles along the main arms of the creek system; indicative extents and depths of inundation for each design flood event; the flood hazard and hydraulic categorisation of the floodplain and the results of the sensitivity studies, shown as "afflux" diagrams (afflux is defined as the variation occurring in peak level over the modelled extent, compared with the design flood result).

Chapter 6 contains a list of references, while **Chapter 7** contains a list of flood-related terminology that is relevant to the study.

2 DESIGN FLOOD MODELLING

2.1 Hydrologic Model Setup

2.1.1 General

The layouts of the catchment model are shown in **Figures 3.1** to **3.3** of L&A, 2014, which also contains a review of the methodology incorporated in the RAFTS-DRAINS approach to catchment modelling. 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 (refer **Table 2.2** for values adopted for design flood modelling).

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.

2.2 Design Storm Generation

2.2.1 Rainfall Intensity

The procedures used to obtain temporally and spatially accurate and consistent intensity-frequency-duration (IFD) design rainfall curves for the catchments are presented in Chapter 2 of ARR, 1998. Design storms were derived for storm durations up to 24 hours. The procedure adopted was to generate IFD data for each catchment by using the relevant charts in Volume 2 of ARR, 1998. These charts included design rainfall isopleths, regional skewness and geographical factors.

2.2.2 Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR, 1998 are applicable strictly to a point. In the case of a catchment of over tens of square kilometres area, it is not realistic to assume that the same rainfall intensity can be maintained. An area reduction factor (**ARF**) is typically applied to obtain an intensity that is applicable over the entire catchment. Chapter 2 of ARR, 1998 shows curves relating the ARF to catchment area for various storm durations.

The ARF for a particular catchment area and given design rainfall burst duration and AEP, represents the ratio between the areal design rainfall and the representative point duration rainfall for the catchment. ARR, 1998 recommended ARF's based on studies in the United States. For the three study catchments use of the ARR, 1998 data would yield ARF values around 0.9.

More recently, *Jordan et al, 2011* describe the derivation of ARF equations for NSW and ACT. Data from the record at over 6000 sites across those two areas was used to derive ARF factors for durations between 1 and 5 days and AEP between 1 in 2 and 1 in 100. For durations less than 1 day, short duration equations based on studies undertaken in Victoria were recommended.

The study catchments range in area between 123 km^2 and 1660 km^2. Using the *Jordan et al, 2011* relationships, ARF values ranged between 0.64 and 0.92 for storm durations between 2 and 24 hours. **Table 2.1** summarises the *Jordan et al, 2011* ARF's, which were adopted for deriving design flood flows at specific locations within each catchment.

As a sensitivity study, application of ARR, 1998 ARF's to the catchment models yielded peak flows which were higher than those achieved with the *Jordan et al, 2011* values. Typically, the 200 year ARI peak discharge derived using *Jordan et al, 2011* would approximate the100 year ARI peak derived from ARR, 1998.

TABLE 2.1
AREAL REDUCTION FACTORS ADOPTED FOR
DESIGN FLOOD ESTIMATION

Catchment	Location	Catchment Area (km^2)	Critical Storm Duration (hours)	ARF
Tarcutta Creek	Tarcutta (Sydney Street)	1340	18	0.82
	Old Borambola Stream Gauge (GS 410047)	1660	18	0.81
Kyeamba Creek	Ladysmith (Tywong Street)[(1)]	542	6 9	0.79 0.81
Sandy Creek	Uranquinty (Olympic Highway)[(2)]	123	6 9	0.86 0.88

1. The 9 hour duration storm is critical for the 5 and 10 year ARI floods, while the 6 hour duration storm is critical for floods with ARI's between 20 and 500 years.
2. The 9 hour duration storm is critical for a 5 year ARI flood, while the 6 hour duration storm is for floods with ARI's between 10 and 500 years.

2.2.3 Temporal Patterns

Temporal patterns for various zones in Australia are presented in ARR, 1998. These patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. The patterns may be used for ARI's up to 500 years where the design rainfall data is extrapolated to this ARI. The study catchments lie in Zone 2 as defined by ARR, 1998.

The derivation of temporal patterns for design storms is discussed in Chapter 3 of ARR, 1998 and separate patterns are presented in Volume 2 for ARI less than 30 years and ARI greater than 30 years. The second pattern is intended for use for rainfalls with ARIs up to 100 years, and to 500 years in those cases where the design rainfall data in Chapter 2 of ARR, 1998 are extrapolated to this ARI.

2.2.4 Estimation of Probable Maximum Precipitation

Estimates of probable maximum precipitation were made using the Generalised Short Duration Method (**GSDM**) as described in the BoM's update of Bulletin 53 (BoM, 2003). This method is appropriate for estimating extreme rainfall depths for catchments up to about 1000 km^2 in area and storm durations up to 6 hours.

The steps involved in assessing PMP for the catchment are briefly as follows:

- Calculate PMP for a given duration and catchment area using depth-duration-area envelope curves derived from the highest recorded US and Australian rainfalls.
- Adjust the PMP estimate according to the percentages of the catchment which are meteorologically rough and smooth, and also according to elevation adjustment and moisture adjustment factors.
- Assess the design spatial distribution of rainfall using the distribution for convective storms based on US and world data, but modified in the light of Australian experience. This procedure involves dividing the catchment into sub-areas (based on ellipses – refer **Figure 2.1**) within which average rainfall depths are computed.
- Derive storm hyetographs using the temporal distribution contained in BoM, 2003, which is based on pluviographic traces recorded in major Australian storms.

2.3 Design Hydrographs

The RAFTS and DRAINS components of the catchment models were run with the following set of parameters to obtain design flows for input to the hydraulic models.

<u>DRAINS Model Parameters Adopted for Design Flood Estimation</u>

- Soil Type = 3
- AMC = 3
- Paved area depression storage = 2.0 mm
- Grassed area depression storage = 10.0 mm
- Paved flow path roughness = 0.02
- Grassed flow path roughness = 0.07

<u>RAFTS Model Parameters Adopted for Design Flood Estimation</u>

The RAFTS model parameters set out in **Table 2.2** over page are the same as those recommended in L&A, 2014 with the exception of the initial loss values, where a small reduction was made to better align design peak flows with the flood frequency relationship which was derived for the Old Borambola stream gauge as part of L&A, 2014.

2.4 Hydraulic Modelling

2.4.1 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.).

TABLE 2.2
RAFTS MODEL PARAMETERS ADOPTED FOR DESIGN FLOOD ESTIMATION

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

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 set up for design flood modelling 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.

2.4.2 TUFLOW Model Structure

Figures 4.2 to **4.4** of L&A, 2014 show the layouts of the various components which comprise the design 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 and was adopted for design flood modelling. 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 at Tarcutta and 5 at Uranquinty).

Other important features of the model for each village are discussed in **Chapters 3** to **5**.

The following features were incorporated in the structure of the individual TUFLOW models:

- **Tarcutta TUFLOW Model** – Modelling of the design floods included grid levels which are based on the road design model of the highway upgrade supplied by the Roads and Maritime Service (**RMS**). This model was used in L&A, 2014 for the purposes of modelling the March 2012 flood. Flow Constriction Values (**FCV's**) were 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 losses associated with flow around the bridge piers. **Figure 4.2** of L&A, 2014 shows the plan extent of the completed road works which were incorporated in the hydraulic model.
- The in-bank survey undertaken by Casey Surveying and Design Pty Ltd for L&A, 2014 was used to represent the conveyance capacity in the creek system under design flood conditions.
- **Ladysmith TUFLOW Model** – As mentioned in **Section 2.5.2** of L&A, 2014, major scour occurred during the October 2010 flood on the left (western) abutment of Railway Bridge No. 2. The scoured opening, which was necessary to replicate flood levels recorded in the March 2012 flood, has been retained in the design flood model.
- **Uranquinty TUFLOW Model** – Bewsher, 2011 noted that efforts were made late on the morning of 4 March 2012 to raise the height of the Town Levee (South). Based on this finding, the March 2012 flood was modelled in L&A, 2014 assuming the temporary levee upgrade works were not in place at the time of the peak, which the modelling showed probably occurred prior to daybreak at around 03:30 hours on 4 March 2012. In view of uncertainties associated with the successful location of temporary protection measures during a flood emergency, the design flood modelling was undertaken assuming no temporary upgrade works would be in place.

2.4.3 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 (refer **Figures 4.2** to **4.4** of L&A, 2014). 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.

2.4.4 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 2.3 over page presents the hydraulic roughness values adopted for design flood modelling. These values were found in L&A, 2014 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.

2.5 Presentation and Interpretation of Results

2.5.1 Extents of Inundation

The relevant diagrams in **Chapters 3** to **5** show water surface profiles and indicative extents of inundation along the main arms of the creeks, as well as the overland flow paths in the urban area and the depths of inundation. Flood levels and extents of inundation are an "envelope" of maximum values applying for the critical storm duration.

TABLE 2.3
HYDRAULIC ROUGHNESS VALUES
ADOPTED FOR DESIGN FLOOD MODELLING

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 and design flood modelling)	0.2
Riparian vegetation between Sydney Street and the Hume Highway on Tarcutta Creek (October 2010 flood, model testing only)	0.08
Riparian vegetation along banks of Kyeamba Creek, Ladysmith (March 2012 and October 2010 floods and design flood modelling)	0.2

In order to create realistic results which remove most of the anomalies caused by inaccuracies in the LiDAR (which has a design accuracy such that 68 per cent of the points have an accuracy in level of +/- 150 mm), a filter was applied to remove depths of inundation over the natural surface less than 50 mm. This has the effect of removing the very shallow depths which are more prone to be artifacts of the model, while enabling the reader to identify the various overland flow paths.

As far as flooding in the main arms of the creeks is concerned, the filtering process does not have a significant effect on representation of the areal extent of Main Stream flooding because of the incised nature of the channels. It is also to be noted that while the flood level and velocity data derived from the analyses are consistent throughout the model, the flood extent diagrams should not be used to give a precise determination of depth of flood affectation in individual allotments bordering either the main arms of the creeks or the overland flow paths.

2.5.2 Accuracy of Hydraulic Modelling

The accuracy of results depends on the precision of the numerical finite difference procedure used to solve the partial differential equations of flow, which is also influenced by the time step used for routing the floodwave through the system and the grid spacing adopted for describing the natural surface levels in the floodplain. Open channels are described by cross-sections normal to the direction of flow, so their spacing also has a bearing on the accuracy of the results.

The results are also heavily dependent on the size of the two-dimensional grid, as well as the accuracy of the ALS data, which as noted above has a design accuracy in elevation whereby 68 per cent of points are within +/- 150 mm of the true level.

Given the uncertainties in the LiDAR data and the definition of features affecting the passage of flow, maintenance of a depth of flow of at least 100 mm is required for the definition of a "continuous" flow path in the areas subject to shallow overland flow approaching the main arm of the creek. Lesser modelled depths of inundation may be influenced by the above factors and therefore may be spurious, especially where that inundation occurs at isolated locations and is not part of a continuous flow path. In areas where the depth of inundation is greater than the 100 mm threshold and the flow path is continuous, the likely accuracy of the hydraulic modelling in deriving peak flood levels is considered to be around 100 mm.

The above limitations will need to be taken into account when using the flood study results to support the *draft Flood Policy*, (to be prepared during the future *FRMS&P*)*,* which will set out flood related controls over development proposals in the three villages. Policies typically specify that proposals should be assessed with the benefit of a site survey to be supplied by applicants, in order to allow any inconsistencies in results to be identified and given consideration. This comment is especially appropriate in the urban areas subject to shallow overland flow, where the errors in the LiDAR or obstructions to flow would have a proportionally greater influence on the computed water surface levels than in the deeper flooded main stream areas.

Flood policies usually specify that minimum floor levels for residential and commercial developments should be based on the 100 year ARI flood level plus appropriate freeboard (this planning level is defined as the *"Flood Planning Level"* [***FPL***]), to cater for uncertainties such as wave action, effects of flood debris conveyed in the overland flow stream and precision of modelling.

In accordance with OEH recommendations (DECCW, 2007), sensitivity studies have also been carried out (refer **Section 2.8**) to assess the impacts of future climate change. Increases in flood levels due to future increases in rainfall intensities may influence the selection of *FPL's*. The discussion on sensitivity studies provides guidance on freeboard under present day climatic conditions. Selection of interim *FPL's*, pending completion of the *FRMS&P*, is discussed in **Section 2.9**.

2.6 Flood Hazard Zones and Floodway Areas

2.6.1 General

According to Appendix L of the *Floodplain Development Manual, (FDM, 2005)*, in order to achieve effective and responsible floodplain risk management, it is necessary to divide the floodplain into areas that reflect:

1. The impact of flooding on existing and future development and people. To examine this impact it is necessary to divide the floodplain into *"flood hazard"* categories, which are provisionally assessed on the basis of the velocity and depth of flow. This task was undertaken in the Design Flood Modelling where the floodplain was divided into *Low Hazard* and *High Hazard zones.* A final determination of hazard will be undertaken during the preparation of the *FRMS&*P and involve will involve consideration of a number of additional factors which are site specific to the urban area of each village. **Section 2.6.2** below provides details of the procedure adopted.

2. The impact of future development activity on flood behaviour. Development in active flow paths (i.e. *"Floodways"*) has the potential to adversely re-direct flows towards adjacent properties. Examination of this impact requires the division of flood prone land into various *"hydraulic categories"* to assess those parts which are effective for the conveyance of flow, where development may affect local flooding patterns. Hydraulic categorisation of the floodplains of the main arms of the creeks and the overland flow paths was also undertaken. **Section 2.6.3** below summarises the procedure adopted.

The flood hazard and hydraulic categorisation led to the sub-division of the floodplain of each village into zones of varying flood risk, as shown in diagrams contained in **Chapters 3** to **5**.

2.6.2 Provisional Flood Hazard

As mentioned above, flood prone areas may be *provisionally* categorised into *Low Hazard* and *High Hazard* areas depending on the depth of inundation and flow velocity. A flood depth of 1 m in the absence of significant flow velocity represents the boundary between *Low* and *High Hazard* conditions. Similarly, a flow velocity of 2.0 m/s but with a small flood depth around 200 mm also represents the boundary between these two conditions. Interpolation may be used to assess the hazard for intermediate values of depth and velocity.

Flood hazards categorised on the basis of depth and velocity only are *provisional*. They do not reflect the effects of other factors that influence hazard. These other factors include:

- Size of flood – major floods though rare can cause extensive damage and disruption.
- Effective warning time – flood hazard and flood damage can be reduced by sandbagging entrances, raising contents above floor level and also by evacuation if adequate warning time is available.
- Flood awareness of the population – flood awareness greatly influences the time taken by flood affected residents to respond effectively to flood warnings. The preparation and promotion by Council of *Flood Studies* and *FRMS&P's* increases flood awareness, as does the formulation and implementation of response plans by NSWSES (Local Flood Plans) for the evacuation of people and possessions.
- Rate of rise of floodwaters – situations where floodwaters rise rapidly are potentially more dangerous and cause more damage than situations in which flood levels increase slowly.
- Duration of flooding – the duration of flooding (or length of time a community is cut off) can have a significant impact on costs associated with flooding. This duration is shorter in smaller, steeper catchments.
- Evacuation problems and access routes – the availability of effective access routes from flood prone areas directly influences flood hazard and potential damage reduction measures.

Provisional hazard categories may be reduced or increased after consideration of the above factors in arriving at a final determination in the *FRMS&P*. However, the Flood Hazard assessment presented in **Chapters 3** to **5** is based on considerations of depth and velocity of flow and is *provisional* only.

2.6.3 Floodways

According to the *FDM, 2005*, the floodplain may be subdivided into the following zones:

- *Floodways*;
- *Flood storage*; and
- *Flood fringe*.

Floodways are those areas where a significant volume of water flows during floods and are often aligned with obvious natural channels. They are areas that, even if partially blocked, would cause a significant increase in flood level and/or a significant re-distribution of flow, which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.

Flood Storage areas are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.

Flood Fringe is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

Floodplain Risk Management Guideline No. 2 Floodway Definition, offers guidance in relation to two alternative procedures for identifying floodways. They are:

- **Approach A**. Using a *qualitative approach* which is based on the judgement of an experienced hydraulic engineer. In assessing whether or not the area under consideration was a floodway, the qualitative approach would need to consider; whether obstruction would divert water to other existing flow paths; or would have a significant impact on upstream flood levels during major flood events; or would adversely re-direct flows towards existing development.
- **Approach B**. Using the hydraulic model, in this case TUFLOW, to define the floodway based on *quantitative experiments* where flows are restricted or the conveyance capacity of the flow path reduced, until there was a significant effect on upstream flood levels and/or a diversion of flows to existing or new flow paths.

One quantitative experimental procedure commonly used is to progressively encroach across either floodplain towards the channel until the designated flood level has increased by a significant amount (for example 0.1 m) above the existing (un-encroached) flood levels. This indicates the limits of the hydraulic floodway since any further encroachment will intrude into that part of the floodplain necessary for the free flow of flood waters – that is, into the floodway.

The *quantitative assessment* associated with **Approach B** is technically difficult to implement. Restricting the flow to achieve the 0.1 m increase in flood levels can result in contradictory results, especially in unsteady flow modelling, with the restriction actually causing reductions in computed levels in some areas due to changes in the distribution of flows along the main drainage line.

Accordingly, the *qualitative approach* associated with **Approach A** was adopted, together with consideration of the findings of *Howells et al, 2004* who defined the *floodway* based on velocity of flow and depth. Howells et al suggested the following criteria for defining those areas which operate as a floodway in a 100 year ARI event:

- Velocity x Depth greater than 0.25 m^2/s **and** Velocity greater than 0.25 m/s; or
- Velocity greater than 1 m/s.

Flood Storage areas were identified as those areas which do not operate as *Floodways* in a 100 year ARI event but where the depth of inundation exceeded 1 m.

The TUFLOW results also showed the direction and magnitude of flow velocity within the modelled area as scaled arrows. These data were also helpful in identifying *Floodways*, as they showed the extents of significant flow. Hydraulic categorisation based on the *Howells et al, 2004* approach has been adopted in this present investigation.

2.6.4 Combined Flood Hazard and Hydraulic Categorisation Diagrams

The combined provisional flood hazard and hydraulic categorisation of the floodplain for the 100 year ARI flood at each village is shown in diagrams presented in **Chapters 3** to **5**. The *Floodways* of the various creeks and major tributaries are continuous and generally encompass the extent of the channels, which are incised with a limited floodplain. It is also to be noted in the context of defining the *Floodway* that floods greater than 100 year ARI or future climate change would not result in the development of new flow paths. Therefore it is considered appropriate to base the hydraulic categorisation of the floodplain on the flooding patterns arising from the 100 year ARI event.

2.7 Sensitivity Studies

2.7.1 General

The sensitivity of the hydraulic model was tested to variations in model parameters such as hydraulic roughness and potential blockage of hydraulic structures. The main purpose of these studies was to obtain guidance on the appropriate freeboard to be adopted when setting floor levels of development in flood prone areas in the *FRMS&P.* The results, which also include an analysis of the impacts of levee failure, are summarised in the following discussion.

2.7.2 Sensitivity to Hydraulic Roughness

Figures in **Chapters 3** to **5** show the “afflux” (i.e. increase in flood levels) for the 100 year ARI critical duration storm resulting from a 20 per cent increase in roughness, compared with the design values of roughness shown in **Table 2.3**. The afflux is given in colour coded increments in metres and is shown along the creeks and stormwater drains, as well as in areas throughout the study area subject to overland flow. The figures also identify areas where land is rendered flood free, or where additional areas of land are flooded. Discussion on the results at each village is given in the relevant sections of **Chapters 3** to **5**.

2.7.3 Sensitivity to Blockage of Hydraulic Structures

The mechanism and geometrical characteristics of blockages in hydraulic structures and piped drainage systems are difficult to quantify due to lack of recorded data and would no doubt be different for each system and also vary with flood events. Realistic scenarios would be limited to waterway openings becoming partially blocked during a flood event (no quantitative data are available on instances of blockage of the drainage systems which may have occurred during the recent historic flood events).

EA, 2013 includes guidance on modes of blockage which are likely to be experienced for different hydraulic structures. Bridge structures with clear opening heights up to 3 m are considered to be susceptible to blockage in streams where large floating debris is conveyed by floodwater, due to debris becoming lodged in the clear opening of the bridge. For bridges of all heights, EA, 2013 considers that debris is likely to also wrap around the bridge piers.

The impact on flood behaviour of an accumulation of debris was assessed assuming the following three modes of blockage:

- **Blockage Mode 1**: Assumes a 1 m thick raft of debris lodges beneath the underside of the bridge deck.
- **Blockage Mode 2**: Assumes a 4 m wide raft of debris lodges on the upstream side of each bridge pier over the full height of the clear opening.
- **Blockage Mode 3**: Analysis based on a combination of Modes 1 and 2.

Afflux diagrams showing the sensitivity of results to blockage for the 100 year ARI critical storm are shown in **Chapters 3** to **5**, with discussion of the results at each village given in relevant sections of those Chapters.

2.7.4 Sensitivity to Levee Failure

For flood levels greater than that of the IFF of the levee, there is the potential for overtopping to occur which could lead to a partial failure of the embankment due to scour in the case of earth levees. The IFF is the threshold flood with a peak level which encroaches into the freeboard nominated for the levee when specifying its hydrologic level of protection. Freeboard is a factor of safety equal to the difference between the elevation of the levee crest and the peak flood level. For the present investigation a freeboard of 500 mm was adopted. Accordingly, floods which exceeded that level were assumed to cause the levee to partially fail.

The approach adopted for assessing the impact of the levee failure on the design flood envelopes was to run the TUFLOW model for floods greater than the IFF of each levee, with an embankment elevation which varied (due to scour) over the duration of the flood.

In accordance with current practice and OEH advice it was assumed that the levees would scour along approximately 10 per cent of their length to an elevation corresponding with half the depth of inundation of the peak of the design flood. The levees were assumed to fail over a 30 minute period commencing at the time when the peak water level on the unprotected side reached the IFF level (i.e. the level equivalent to a freeboard of 500 mm on the crest).

The Old Tarcutta Inn is protected by two levees: an outer levee of earth construction which was assumed to scour as discussed above and an inner levee of block wall construction which was removed from the model when flood levels reached its IFF level. The IFF's for the earth and block wall levees were 5 year ARI and 20 to 25 year ARI respectively.

Partial failure of the levees locally changed peak flood levels and flow patterns within the protected areas. In more remote areas, these effects were not significant. However, the design flood modelling results have been presented as an envelope of the higher values of flood level or flow at each location in the modelled area resulting from the two scenarios when the IFF is exceeded: i.e. partial failure and no failure.

Using the above principles, preliminary analyses of the impacts of levee failure were undertaken for the levees at Tarcutta and Uranquinty and results for the two villages are discussed in **Section 3.4** and **5.4** respectively.

2.8 Climate Change Sensitivity Analysis

2.8.1 General

Scientific evidence shows that climate change will lead to sea level rise and potentially increase flood producing rainfall intensities. The significance of these effects on flood behaviour will vary depending on geographic location and local topographic conditions. Climate change impacts on flood producing rainfall events show a trend for larger scale storms and resulting depths of rainfall to increase.

OEH recommends that its guideline *Practical Considerations of Climate Change, 2007* be used as the basis for examining climate change induced increases in rainfall intensities in projects undertaken under the State Floodplain Management Program and in accordance with the FDM, 2005. The guideline recommends that until more work is completed in relation to the climate change impacts on rainfall intensities, sensitivity analyses should be undertaken based on increases in rainfall intensities ranging between 10 and 30 per cent. On current projections the increase in rainfalls within the service life of developments or flood management measures is likely to be around 10 per cent, with the higher value of 30 per cent representing an upper limit. Under present day climatic conditions, increasing the 100 year ARI design rainfall intensities by 10 per cent would produce a 200 year ARI flood; and increasing those rainfalls by 30 per cent would produce a 500 year ARI event.

The impacts of climate change and associated effects on the viability of floodplain risk management options and development decisions may be significant and will need to be taken into account in the *FRMS&P* investigation.

At the present flood study stage, the principal issue regarding climate change is the potential increase in flood levels and extents of inundation throughout the study area. In addition it is necessary to assess whether the patterns of flow will be altered by new floodways being developed for key design events, or whether the provisional flood hazard will be increased.

In the *FRMS&P* it will be necessary to consider the impact of climate change on flood damages to existing development. Consideration will also need to be given both to setting floor levels for future development and in the formulation of works and measures aimed at mitigating adverse effects expected within the service life of development.

Afflux diagrams showing the sensitivity of results to increases in rainfall intensity resulting from climate change for the 100 year ARI critical storm are shown in **Chapters 3** to **5**, with discussion of the results at each village given in relevant sections of those Chapters.

2.9 Selection of Interim Flood Planning Area and Levels (Main Stream)

This study has determined flooding patterns under present day conditions. The assessment of flood damages, specifying the *FPL* and investigating flood mitigation measures will be undertaken in the *FRMS&P.*

As part of the *FRMS&P,* a *draft Flood Policy* will need to be formulated to cater for future development. The *Flood Policy* will need to consider residential and commercial development and assign *FPL's* based on a flood magnitude (the 100 year ARI event for residential land use and typically that event for commercial/industrial development), with appropriate freeboard.

A higher level of protection is usually given to essential services, critical utilities and flood vulnerable developments such as aged persons' accommodation, child care centres, etc. Flood related controls such as conformance with minimum floor levels and safety criteria for those categories of development could be applied for floods ranging between the residential *FPL* and the PMF.

After consideration of the TUFLOW results and the findings of sensitivity studies for each village outlined in **Chapters 3** to **5**, a freeboard allowance of 500 mm on 100 year ARI flood levels is suggested as the interim *FPL* and for defining the extent of the *Flood Planning Area (FPA)* pending the completion of the *FRMS&P.*

Interim *FPL* contours developed on that basis and the associated interim *FPA* for Main Stream flooding only are shown in **Volume 2** as **Figure 3.18** for Tarcutta, **Figure 4.17** for Ladysmith and **Figure 5.18** for Uranquinty. Assessment of corresponding data for the urban areas in each village, subject to MOF, will need to be undertaken during the *FRMS&P* (refer to discussion in the next section).

2.10 Flood Planning Area and Levels (Major Overland Flow Paths)

In the preparation of the *draft Flood Policy*, there will be two types of flooding to consider:

- Main Stream Flooding in the creeks and tributaries, which flow in incised and well defined channels. Definition of the interim *FPA* for those waterways is relatively straightforward as it is a matter of mapping the extent of the 100 year ARI flood plus the freeboard allowance (typically the 500 mm provisionally recommended above).
- MOF which typically represents relatively slow moving and shallow flow over the natural surface and in the ill-defined watercourses and gullies which eventually join the main streams.

It is not appropriate to define interim *FPL* and *FPA* data in areas subject to MOF during the *Flood Study,* pending further consideration of appropriate freeboard, which is usually undertaken in the *FRMS&P* with the benefit of the *final determination* of flood hazard and after consideration of the likely impacts of flooding on development. In recognition of the shallow and slow moving nature of overland flow, a lesser freeboard than the 500 mm adopted for Main Stream flooding may be justified. As previously noted, the definition of high and low hazard conditions in the *Flood Study* is *provisional* only and is subject to adjustment in the *FRMS&P*, based on a range of criteria in addition to the depth and velocity of inundation.

3 TARCUTTA

3.1 Presentation and Interpretation of Results

3.1.1 Water Surface Profiles and Extents of Inundation

Figure 3.1, **Sheets 1** and **2** show design water surface profiles on Tarcutta Creek and along the extents of the levees. Reference to these diagrams allows the IFF to be estimated. The IFF is the threshold flood event which rises to a peak level within the adopted freeboard of 500 mm of the levee crest (refer also to the discussion in **Section 2.7.4**). Based on this definition, the IFF's for the levees are approximately:

- Tarcutta Levee – 20 to 25 year ARI
- Old Tarcutta Inn Levee (inner block wall) – 20 to 25 year ARI
- Hambledon Levee – 20 year ARI

Discharge and stage hydrographs at key locations on Tarcutta Creek are shown on **Figure 3.2**, **Sheets 1** and **2**. The hydrographs apply for the design storm of 18 hours duration, which is "critical" in terms of maximising flows at these locations and do not allow for scour of the levees when overtopped. (**Table A1** of **Appendix A** also shows peak discharges at locations within the village area for the relatively short duration storms which are critical for those small local catchments.)

Table 3.1 over page summarises historic and design flood peak discharges, elevation and gauge heights at the Tarcutta flood gauge at Sydney Street.

Design flood levels on the main arm of Tarcutta Creek generally peak around 15 hours after the commencement of rainfall, whereas the local catchments are flash flooding generally peaking about one hour after the commencement of the respective critical storm. **Figure 3.2** at **Sheet 2** (location Q6) also shows the discharge hydrographs for flow which overtops the Tarcutta Levee. Two cases are shown:

- Assuming no scour when the levee is overtopped (solid line)
- Assuming partial failure for floods greater than the IFF (dashed line)

As expected, the flow entering the protected area increases once the levee commences to fail.

Figures 3.3 to **3.10** are plans showing the TUFLOW model results for the 5, 10, 20, 50, 100, 200 and 500 year ARI floods and the PMF. These diagrams show the indicative extents and depths of inundation along the main arm of the creek, as well as along the overland flow paths in the urban area of the village. Flood levels and extents of inundation are an "envelope" of maximum values applying for the critical storm duration at each location within the modelled area and for the two scenarios: partial levee failure and no failure due to scour.

3.2 Flood Hazard Zones and Floodways

Following the approach outlined in **Section 2.6**, the hazard and hydraulic categorisation of the floodplain for the 100 year ARI event is presented on **Figure 3.11**. The floodplain has been divided into five hazard categories. There is a High Hazard Floodway which is continuous along the line of Tarcutta Creek, with areas of Low Hazard Floodway and Flood Fringe near the limits of the flooded area. Some of the deeper flooded areas near the fringe have been categorised a High Hazard Flood Storage.

Most of the flooded area in the overland flow paths in the urban part of the village located above the eastern floodplain area located in Low Hazard Flood Fringe areas.

TABLE 3.1
DESIGN AND HISTORIC FLOOD DATA AT TARCUTTA FLOOD GAUGE

Flood Event [A]	Peak Flow(1,2) (m^3/s) [B]	Peak Flood Level(1,5) (m AHD) [C]	Gauge Height(3,4,5) (m) [D]
5 year ARI	151	227.45	2.77
10 year ARI	350	228.10	3.42
20 year ARI	526	228.51	3.83
March 2012	528	228.53 [228.54]	3.85 [3.86]
October 2010 (6)	903	228.97 [229.17]	4.29 [4.49]
50 year ARI	853	229.10	4.42
100 year ARI	1035	229.32	4.64
200 year ARI	1207	229.51	4.83
500 year ARI	1435	229.78	5.10

1. Peak flood levels and flows extracted from TUFLOW model results. Flows include all flow passing Sydney Street, including flow in Tarcutta Creek plus flow overtopping Tarcutta Levee and then conveyed overland across Sydney Street.
2. Peak flows are for design storm of 18 hours duration, which is generally critical for maximising flow in Tarcutta Creek at Tarcutta.
3. Gauge zero = RL 224.68 m AHD. (Source: Survey undertaken by WWCC). Refer **Annexure B** of L&A, 2014 for details.
4. Gauge heights based on computed peak flood levels in **Column C**.
5. Values in [ ] are actual gauge heights recorded at flood gauge for each historic flood.
6. Hume Highway under construction during time of event and reduced riparian vegetation cover Sydney Street to Hume Highway, compared with March 2012 (March 2012 roughness values adopted for design - **Table 2.3**).

3.3 Sensitivity Studies

3.3.1 General

The sensitivity of the hydraulic model was tested to variations in model parameters such as hydraulic roughness and potential blockage of hydraulic structures. The main purpose of these studies was to obtain guidance on the appropriate freeboard to be adopted when setting floor levels of development in flood prone areas in the *FRMS&P*. The results are summarised in the following sections.

3.3.2 Sensitivity to Hydraulic Roughness

Figure 3.12 shows the “afflux” (i.e. increase in flood levels) for the critical 100 year ARI 18 hour duration storm resulting from a 20 per cent increase in roughness, compared with design values of roughness shown in **Table 2.3**. The afflux is given in colour coded increments in metres and is shown along Tarcutta Creek, as well as in the urban area on the protected side of the levees. This figure also identifies areas where land is rendered flood free, or where additional areas of land are flooded.

The higher roughness provides additional resistance to the passage of flow resulting in increases in flood levels in Tarcutta Creek on the upstream side of Sydney Street which are in the range 100 - 200 mm along the frontage of the Tarcutta Levee. Due to the increase in flows overtopping the levee (compared with the discharge hydrograph shown at location Q6 in **Figure 3.2**, **Sheet 2**), water levels within the protected area could increase by 200 – 300 mm compared with design conditions.

3.3.3 Sensitivity to Blockage of Hydraulic Structures

Section 2.7.3 set out the procedure adopted for assessing the impacts of partial blockage of the bridges. On Tarcutta Creek, it is more likely that the two bridges on Sydney Street would experience a partial blockage during future flood events than the new Hume Highway bridges.

Accordingly, the impact of an accumulation of debris on the Sydney Street Bridges was assessed for the three modes of blockage described in **Section 2.7.3**. However, the two bridges over Tarcutta Creek at the new Hume Highway are high level structures and therefore only **Mode 2** was modelled (i.e. assuming a 4 m wide raft of debris lodges on the upstream side of each bridge pier over the full height of clear opening).

Figure 3.13 is an envelope of the maximum increase in 100 year ARI flood levels (i.e. the afflux) arising from the above scenarios. As expected the maximum values of afflux occur upstream of the Sydney Street Bridges, where creek flood levels could increase by 300 – 500 mm along the frontage of the Tarcutta Levee, with consequent reduction in the flood security afforded by the levee. Within the protected area 100 year ARI flood levels could increase by a similar amount compared with design conditions.

Increases in 100 year ARI flood levels of 300 – 500 mm could occur within the area protected by the Hambledon Levee, and 200 – 300 mm within that of the Old Tarcutta Inn Levee.

3.4 Sensitivity to Levee Failure

The procedure adopted for design flood modelling with partial failure of the levees is described in **Section 2.7.4**.

Figure 3.14 shows the effects of levee failure for the 100 year ARI flood in terms of afflux (i.e. peak water levels with levee failure compared with flood levels assuming the levees do not scour). In the case of the Tarcutta Levee there would be an increase in peak levels in parts of the protected area exceeding 500 mm as a result of failure. However, due to the comparatively greater depths of overtopping under design conditions, the afflux in the protected areas for the other levees is considerably smaller; around 20 – 50 mm for the Old Tarcutta Inn Levee and not significant for the Hambledon Levee.

3.5 Climate Change Sensitivity Analysis

3.5.1 Sensitivity to Increased Rainfall Intensities

Figure 3.15 shows the afflux resulting from an increase of 10 per cent in 100 year ARI rainfall intensities. The average increase in peak flood levels across the study area is around 200 to 300 mm, increasing to between 300 and 500 mm in the developed part of the village on the eastern bank of Tarcutta Creek.

Figure 3.16 shows the afflux for a 30 per cent increase in 100 year ARI rainfall intensities. The increase in peak flood levels along the main arm of Tarcutta Creek is greater than 500 mm in village areas protected by the levee.

The impact of increased rainfall intensities on flooding patterns may be summarised as follows (ref. also to **Figure 3.17** which shows the increase in the extent of inundation from the two rainfall scenarios):

- The extent of inundation along the length of the main arm of Tarcutta Creek does not widen significantly, owing to the relatively steep nature of the surrounding overbank areas.
- The flood security of the existing levees will reduce significantly. From **Figure 3.1, Sheet 2**, the 200 year ARI flood levels (the future 100 year ARI event with a 10 per cent increase in rainfall intensities) along the line of the Tarcutta Levee is about 200 mm higher than the present day 100 year ARI. The 500 year ARI (the future 100 year ARI event with a 30 per cent increase in rainfall intensities) is about 500 mm higher. As the IFF of the levee is 20 - 25 year ARI under present day conditions, a major upgrade would be required to combat the effects of future climate change. A similar situation exists for the Hambledon Levee and Old Tarcutta Inn Levee. A detailed investigation of requirements for levee upgrades is required in the *FRMS&P*.
- Depths and widths of inundation in the overland flow paths in the urban areas will increase, resulting in an increase in the incidences of above-floor inundation in residential properties and flood damages.
- There may be a reduction in the time of rise of the floodwaters. This may not be significant on the main arm of Tarcutta Creek which has a time of rise of around 15 hours. However, the local catchments discharging through the village area are flash flooding with little warning time available to residents (the critical duration for major storm events is 60 minutes under 100 year ARI conditions). Effective flood warning may not be achievable even with the benefit of future technical improvements in such systems. Therefore on-going community education via WWCC and NSWSES is required to limit risks to people and property. Further consideration of flood warning arrangements and strategies will need to be undertaken in the *FRMS&P*.

3.6 Selection of Interim Flood Planning Area and Levels (Main Stream)

After consideration of the TUFLOW results and the findings of sensitivity studies outlined in **Sections 3.3** to **3.5**, a freeboard allowance of 500 mm on 100 year ARI flood levels is suggested as the interim *FPL* and for defining the extent of the interim *FPA* pending the completion of the *FRMS&P.*

Interim *FPL* contours developed on that basis and the associated interim *FPA* for Main Stream flooding only are shown on **Figure 3.18**. As noted in **Section 2.9**, assessment of corresponding data for the urban area of Tarcutta, subject to overland flooding, will be undertaken during the *FRMS&P* (refer to preliminary discussion on the approach in **Section 2.10**.)

3.7 Summary of Flood Affectation and Issues for Further Investigation in the *FRMS&P*

Following is a brief summary of flood affectation at Tarcutta and the issues which will need to be further investigated during the preparation of the *FRMS&P*.

- At the 100 year ARI, floodwaters on Tarcutta Creek extend over a width of 800 - 900 m along the extent of the study reach. The urban area of the village is also subject to MOF flooding from the local sub-catchments which drain westwards to Tarcutta Creek. Significant overland flows commence at the 10 year ARI level of flooding due to surcharges of the trunk drainage system.
- The time of rise of Tarcutta Creek under design flood conditions is around 15 hours. (**Figure 3.2**). The response time of the MOF paths through the urban area is shorter, with a time of rise generally limited to less than one hour.
- The hydraulic analysis for the design flood estimation has included partial failure of the levees for floods greater than the IFF (defined at **Section 2.7.4**). The IFF for the levees ranges between 20 year ARI for the Hambledon Levee and 20 to 25 year ARI for the Old Tarcutta Inn Levee (inner block wall) and the Tarcutta Levee.
- A preliminary assessment of the effects of partial levee failure was also carried out (increases in peak flood levels for the 100 year ARI compared with the case of no scour of the levees are shown on **Figure 3.14**). Peak 100 year ARI water levels within the protected parts of the village could be increased by values in excess of 500 mm.
- Mitigation of existing flooding problems will require the upgrading of the levees. This will be one of the important management measures to be considered in the *FRMS&P*. Selection of appropriate crest levels will require consideration of potential rises in design flood levels due to increased hydraulic roughness in the flood plain, partial blockage of downstream bridge waterways and future climate change.
- The floodway and flood hazard extents shown in **Figure 3.11** are based on depth and velocity of flow considerations and should be regarded as provisional. A final determination of hazard should be made in the *FRMS&P* in areas subject to both main Stream and MOF, based on additional considerations summarised in **Section 2.6.2**.
- Matching the extents of inundation determined in this flood study with the footprints and floor levels of existing residential development will be undertaken in the *FRMS&P* to estimate damages resulting from a range of flood events. The hydraulic and economic analysis of potential works will be undertaken at the strategic level of detail in the *FRMS&P*. This will enable a priority list of flood management measures to be prepared.

4 LADYSMITH

4.1 Presentation and Interpretation of Results

4.1.1 Water Surface Profiles and Extents of Inundation

Figure 4.1 shows design water surface profiles on Kyeamba Creek along the frontage of Ladysmith.

Discharge and stage hydrographs at representative locations on Kyeamba Creek, within the urban portion of the village and at several crossings of Tumbarumba Road are shown on **Figure 4.2**, **Sheets 1** and **2**. The hydrographs apply for the design storm of 6 hours duration, which is "critical" in terms of maximising flows on Kyeamba Creek and for the shorter duration storms (1 to 2 hours duration) which are critical for the minor tributaries and overland flow paths. (**Table A2** of **Appendix A** shows peak discharges.)

Table 4.1 summarises historic and design flood peak discharges, elevation and gauge heights at the Ladysmith gauge.

TABLE 4.1
DESIGN AND HISTORIC FLOOD DATA AT LADYSMITH STREAM GAUGE (GS 410048)

Flood Event [A]	Peak Flow(1,2) (m^3/s) [B]	Peak Flood Level(1,5) (m AHD) [C]	Gauge Height(3,4,5) (m) [D]
5 year ARI	77	200.13	4.91
10 year ARI	177	200.57	5.34
20 year ARI	274	200.97	5.75
March 2012	383	201.33 [201.35]	6.11 [6.13]
50 year ARI	491	201.74	6.52
October 2010(6)	474	201.88 [201.89]	6.66 [6.67]
100 year ARI	614	202.06	6.84
200 year ARI	737	202.25	7.03
500 year ARI	920	202.45	7.23

1. Peak flood levels and flows extracted from TUFLOW model results.
2. Peak flows are for design storm of 6-9 hours duration, which is generally critical for maximising flow in Kyeamba Creek at Ladysmith.
3. Gauge zero = RL 195.224 m AHD.
4. Gauge heights based on computed peak flood levels in **Column C**.
5. Values in [ ] are actual gauge heights recorded at flood gauge for each historic flood.
6. Assumed Railway Bridge No. 2 scoured on receding limb of October 2010 Event.

Design flood levels on Kyeamba Creek generally peak around 9 to 10 hours after the commencement of rainfall, whereas the local tributary catchments are flash flooding with a corresponding time to peak of less than two hours.

Figures 4.3 to **4.10** are plans showing the TUFLOW model results for the 5, 10, 20, 50, 100, 200 and 500 year ARI floods and the PMF. These diagrams show the indicative extents and depths of inundation along Kyeamba Creek and its tributaries, as well as the overland flow paths in the urban part of the village. Flood levels and extents of inundation are an "envelope" of maximum values applying for the critical storm duration at each location within the modelled area.

4.2 Flood Hazard Zones and Floodways

The provisional hazard and hydraulic categorisation diagram for the 100 year ARI event is presented on **Figure 4.11**. Following the approach outlined in **Section 2.6**, the floodplain has been divided into five hazard categories. There is a continuous High Hazard Floodway along the central thread of Kyeamba Creek and its western floodplain, with zones of Low Hazard Floodway and Flood Fringe near the limits of the inundated area. Some of the deeper flooded areas on the creek, where milder flow velocities prevail, have been categorised as Flood Storage areas.

Several MOF paths which drain westwards through the urban part of Ladysmith act as Low Hazard Floodways. However, most of the flood affected urban area is situated in the Low Hazard Flood Fringe.

4.3 Sensitivity Studies

4.3.1 General

The sensitivity of the hydraulic model was tested to variations in model parameters such as hydraulic roughness and potential blockage of hydraulic structures. The main purpose of these studies was to obtain guidance on the appropriate freeboard to be adopted when setting floor levels of development in flood prone areas in the *FRMS&P.* The results are summarised in the following sections.

4.3.2 Sensitivity to Hydraulic Roughness

Figure 4.12 shows the "afflux" (i.e. increase in flood levels) in the Ladysmith study area for the 100 year ARI 6 hour duration storm resulting from a 20 per cent increase in roughness, compared with design values of roughness shown in **Table 2.3**. This figure also identifies areas where land is rendered flood free, or where additional areas of land are flooded.

The higher roughness provides additional resistance to the passage of flow resulting in an increase in flood levels on Kyeamba Creek generally in the range of 100 - 200 mm, with lesser values up to 50 mm in the tributaries and the urban part of the village.

4.3.3 Sensitivity to Blockage of Hydraulic Structures

Figure 4.13 shows the afflux under 100 year ARI conditions resulting from a partial blockage of the following bridges on Kyeamba Creek:

- The Railway Bridge (Openings Nos 1 and 2).
- Tywong Street Bridge.
- Mona Vale Road Bridge

The analysis was carried out according to principles set out in **Section 2.7.3,** with all bridges assumed to be blocked simultaneously. Maximum afflux was modelled immediately upstream of the two Railway Bridges, where 100 year ARI flood levels would increase by 300 - 500 mm, with the afflux reducing to zero on the downstream side of the Tywong Street Bridge, about 1.3 km upstream of the railway. Blockage of the low level Tywong Street and Mona Vale Road Bridges would not have a significant effect on design levels for major flood events. No increases in flood levels would be experienced within the urban area of the village due to partial blockage of these structures.

4.3.4 Sensitivity of Flood Levels to Failure of Railway Embankment

While there is considerable likelihood of failure of the disused railway embankment during major flood events, it was considered that peak flood levels downstream of the railway embankment were not likely to increase greatly as a result. The drop in peak water surface elevations across the embankment is only about 500 mm, indicating that a rapid failure of a section of the railway line would not result in a large pulse of water travelling downstream of the crossing.

Accordingly hydraulic analyses of embankment failure were not undertaken.

4.4 Climate Change Sensitivity Analysis

4.4.1 Sensitivity to Increased Rainfall Intensities

Figure 4.14 shows the afflux resulting from an increase of 10 per cent in 100 year ARI rainfall intensities. The maximum increase in peak flood levels in Kyeamba Creek occurs on the downstream side of the Railway Bridge and is around 200 to 300 mm. In the developed part of the village the increase is no greater than 50 mm, with a very small increase occurring in the inundated area.

Figure 4.15 shows the afflux for a 30 per cent increase in 100 year ARI rainfall intensities. The increase in peak flood levels in Kyeamba Creek floodplain is generally around 300 to 500 mm. In the developed part of the village the increase is no greater than 100 mm, with a small increase in inundated area.

The impact of increased rainfall intensities on the extent of inundation flooding patterns may be summarised as follows (ref. **Figure 4.16** which shows the increase in inundation from the two rainfall scenarios):

- The extents of inundation along the length of the main arm of Kyeamba Creek and its tributaries do not widen significantly, owing to the relatively steep nature of the surrounding overbank areas.
- Depths and widths of inundation in the overland flow paths in the urban areas will increase, by up to 100 mm. Any resulting increase in the incidences of above-floor inundation in residential properties and flood damages will be evaluated in the *FRMS&P*.
- There may be a reduction in the time of rise of the floodwaters. This may not be as significant on the main arm of Kyeamba Creek, which has a time of rise of around 9 to 10 hours. However, the local catchments discharging through the village area are flash flooding with little warning time available to residents (the critical duration for major storm events is 60 minutes and they peak between 1 to 2 hours after the commencement of heavy rainfall. As for Tarcutta, effective flood warning may not be achievable even with the benefit of future technical improvements in such systems. Therefore on-going community education via WWCC and NSWSES is required to limit risks to people and property. Further consideration of flood warning arrangements and strategies will be undertaken in the *FRMS&P*.

4.5 Selection of Interim Flood Planning Area and Levels (Main Stream)

After consideration of the TUFLOW results and the findings of sensitivity studies outlined in **Sections 4.3** and **4.4**, a freeboard allowance of 500 mm on 100 year ARI flood levels is suggested as the interim *FPL* and for defining the extent of the interim *FPA* pending the completion of the *FRMS&P.*

Interim *FPL* contours developed on that basis and the associated interim *FPA* for Main Stream flooding only are shown on **Figure 4.17**. As noted in **Section 2.9**, assessment of corresponding data for the urban area of Ladysmith subject to overland flooding, will be undertaken during the *FRMS&P* (refer to preliminary discussion on the approach in **Section 2.10**.)

4.6 Summary of Flood Affectation and Issues for Further Investigation in the *FRMS&P*

Following is a brief summary of flood affectation at Ladysmith and the issues which will need to be further investigated during the preparation of the *FRMS&P*.

- At the 100 year ARI floodwaters on Kyeamba Creek extend over a width of 500 - 700 m along the extent of the study reach. Although the urban part of the village is not affected by main stream flooding, even at the 100 year ARI, it is affected by MOF from the local sub-catchments which drain westwards to Kyeamba Creek. Significant overland flows commence at the 10 year ARI level of flooding due to surcharges of the trunk drainage system.
- The time of rise of Kyeamba Creek under design flood conditions is around 9 to 10 hours. (**Figure 4.2**). The response time of the MOF paths is shorter with a time of rise generally limited to less than two hours.
- The floodway and flood hazard extents shown in **Figure 4.11** are based on depth and velocity of flow considerations and should be regarded as provisional. A final determination of hazard should be made in the *FRMS&P* in areas subject to both main Stream and Major Overland Flow, based on additional considerations summarised in **Section 2.6.2**.
- Matching the extents of inundation determined in this flood study with the footprints and floor levels of existing residential development will be undertaken in the *FRMS&P* to estimate damages resulting from the range of flood events. Mitigation of existing flooding problems in the urban parts of the village subject to MOF could require measures such as levees, channel improvements or detention basins. Hydraulic and economic analysis of potential works will be undertaken at the strategic level of detail in the *FRMS&P*. This will enable a priority list of flood management measures to be prepared.

5 URANQUINTY

5.1 Presentation and Interpretation of Results

5.1.1 Water Surface Profiles and Extents of Inundation

Figure 5.1 shows design water surface profiles on Sandy Creek and along the extents of the levees. Reference to these diagrams allows the IFF to be estimated. The IFF is the threshold flood event which rises to a peak level within the adopted freeboard of 500 mm of the levee crest (refer also to the discussion in **Section 2.7.4**). Based on this definition, the IFF's for the levees are approximately:

- Connorton Street Levee Less than 5 year ARI.
- Town Levee (South) – 10 year ARI
- Town Levee (North) – 5 year ARI

Discharge and stage hydrographs at key locations on Sandy Creek and within the urban portion of the village are shown on **Figure 5.2**, **Sheets 1** and **2**. The hydrographs apply for the design storm of 6 hours duration, which is "critical" in terms of maximising flows at these locations and show the effects of partial failure of the levees when the IFF is exceeded. (**Table A3** of **Appendix A** shows peak discharges.) Design flood levels on Sandy Creek generally peak around 7 to 8 hours after the commencement of rainfall. The discharge hydrograph at Yarragundry Street (Q4) represents the flow through the culvert and does not include the flow that surcharges the culvert and flows over the western floodplain. The flow at location Q6 represents the discharge over the railway line at the low point on the eastern side of the creek, where a small surcharge occurs under 100 year ARI conditions. **Figure 5.2** at **Sheet 2** (location Q5) also shows discharge hydrographs for flow which overtops the Town Levee South at the low point in Deane Street. Two cases are shown:

- Assuming no scour when the levee is overtopped (solid lines).
- Assuming failure for floods greater than the IFF (dashed lines).

As expected, the flow entering the protected area increases once the levee commences to fail.

Figures 5.3 to **5.10** are plans showing the TUFLOW model results for the 5, 10, 20, 50, 100, 200 and 500 year ARI floods and the PMF. These diagrams show the indicative extents of inundation along the main arm of the creek, as well as the overland flow paths in the urban area and the depths of inundation. Flood levels and extents of inundation are an "envelope" of maximum values applying for the critical storm duration.

5.2 Flood Hazard Zones and Floodways

Following the approach outlined in **Section 2.6**, the hazard and hydraulic categorisation of the floodplain for the 100 year ARI event is presented on **Figure 5.11**. The floodplain has been divided into five hazard categories. There is a High Hazard Floodway which is continuous along the central thread of Sandy Creek, with areas of Low Hazard Floodway and Flood Fringe near the limits of the flooded area. Some of the deeper flooded area on the upstream side of the railway has been categorised a High Hazard Flood Storage.

Most of the flooded area within the village is Low Hazard Flood Fringe.

5.3 Sensitivity Studies

5.3.1 General

The sensitivity of the hydraulic model was tested to variations in model parameters such as hydraulic roughness and potential blockage of hydraulic structures. The main purpose of these studies was to obtain guidance on the appropriate freeboard to be adopted when setting floor levels of development in flood prone areas in the *FRMS&P.* The results are summarised in the following sections.

5.3.2 Sensitivity to Hydraulic Roughness

Figure 5.12 shows the "afflux" (i.e. increase in flood levels) for the 100 year ARI 6 hour duration storm resulting from a 20 per cent increase in roughness, compared with design values of roughness shown in **Table 2.3**. The afflux is given in colour coded increments in metres and is shown along Sandy Creek, as well as in the urban area of Uranquinty. This figure also identifies areas where land is rendered flood free, or where additional areas of land are flooded.

The higher roughness provides additional resistance to the passage of flow resulting in an increase in flood levels on Sandy Creek which is up to 100 - 200 mm along the extent of the levee upstream and downstream of the Olympic Highway crossing. The increase in roughness leads to an average increase in flood levels in the area protected by the levee upstream of Olympic Highway of 20 to 50 mm. Downstream of the highway crossing the increase in levels in the protected area reaches up to 300 – 500 mm and there is a significant increase in inundated area.

5.3.3 Sensitivity to Blockage of Hydraulic Structures

Figure 5.13, **Sheets 1** and **2** are envelopes of the maximum increase in 100 year ARI flood levels (i.e. the afflux) arising from the blockage modes presented in **Section 2.7.2**.

- **Sheet 1** applies for partial blockage of the three main waterway openings across Sandy Creek: Railway Bridge, Olympic Highway Bridge and the Yarragundry Street Culverts.
- **Sheet 2** applies for partial blockage of the Ryan-Key Street Culvert only.

As expected, the maximum values of afflux shown on **Sheet 1** occur upstream of the Olympic Highway Bridge where creek flood levels could increase by more than 500 mm along the frontage of the Town Levee (South), with a major reduction in the flood security afforded by the levee. Within the protected area, 100 year ARI flood levels could increase by 300 – 500 mm compared with design conditions. On the downstream side of the railway, there would be a major increase in both depths and extents of inundation in the area protected by the Town Levee (North).

5.4 Sensitivity to Levee Failure

The approach to modelling the partial failure of the levees was as described in **Section 2.7.4**, apart from the assumption of total failure of the Connorton Street levee, given its minor nature. Failure of that levee was simulated by its removal from the ground model. For the remaining levees, the following locations were adopted for breaches:

- A breach at the eastern end of Deane Street, near the start of the earthfill section of the Town Levee (South). The assumed length of the breach was 40 m, representing about 10 per cent of the total length of Deane Street.
- A breach at the downstream end of the Town Levee (North) at the location of the spillway (low point). The assumed length of the breach was 150 m, which represents about 10 per cent of the total length of the Town Levee (North).

The Town Levees (North and South) were assumed to fail over a 30 minute period following the time at which the peak water level on the unprotected side first reached the IFF level (corresponding with freeboard allowance of 500 mm on the embankment crest elevation).

Figure 5.14 shows the effects of levee failure for the 100 year ARI flood in terms of afflux (i.e. peak water levels with levee failure compared with flood levels assuming the levees do not scour). Flood levels in the protected area upstream of the Olympic Highway increase by between 100 – 200 mm, with similar increases on the downstream side of the railway, along with a considerable increase in the additional area of flooded land.

5.5 Climate Change Sensitivity Analysis

5.5.1 Sensitivity to Increased Rainfall Intensities

Figure 5.15 shows the afflux resulting from an increase of 10 per cent in 100 year ARI rainfall intensities. Upstream of the Olympic Highway levels in Sandy Creek increase by 200 – 300 mm, resulting in increases between 50 – 100 mm in the area protected by the Town Levee (South). On the downstream side of the railway levels in the area protected by the Town Levee (North) increase by up to 300 – 500 mm with a corresponding increase in the additional area of flooded land.

Figure 5.16 shows the afflux for a 30 per cent increase in 100 year ARI rainfall intensities. The increase in peak flood levels along the main arm of Sandy Creek is greater than 500 mm immediately upstream of the Olympic Highway resulting in a large ingress of water into the protected areas and increases in flood affectation both upstream and downstream of the railway.

The impact of increased rainfall intensities on flooding patterns may be summarised as follows (ref. also to **Figure 5.17** which shows the increase in the extent of inundation from the two rainfall scenarios):

- The extent of inundation along the length of the main arm of Sandy Creek does not widen significantly, owing to the relatively steep nature of the surrounding overbank areas.
- The flood security of the existing levees will reduce significantly. From **Figure 5.1**, **Sheet 2**, the 200 year ARI flood levels (the future 100 year ARI event with a 10 per cent increase in rainfall intensities) along the line of the Town Levee (South) is about 200 mm higher than the present day 100 year ARI. The 500 year ARI (the future 100 year ARI event with a 30 per cent increase in rainfall intensities) is about 500 mm higher. As the IFF of the levee system is 5 year ARI under present day conditions, a major upgrade would be required to combat the effects of future climate change. A detailed investigation of requirements for levee upgrades is required in the *FRMS&P*.
- Depths and widths of inundation in the overland flow paths in the urban areas will increase, resulting in an increase in the incidences of above-floor inundation in residential properties and flood damages.
- There may be a reduction in the time of rise of the floodwaters. The main arm of Sandy Creek has a time of rise of around 7 to 8 hours and under present day conditions. Floodwaters overtop the Town Levee (South) in the vicinity of Deane Street about 4 to 5 hours after the commencement of heavy rainfall during major flood events. Reductions in this time due to climate change may have an adverse impact on emergency management. Therefore on-going community education via WWCC and SES is required to limit risks to people and property. Further consideration of flood warning arrangements and strategies will be undertaken in the *FRMS&P*.

5.6 Selection of Interim Flood Planning Area and Levels (Main Stream)

After consideration of the TUFLOW results and the findings of sensitivity studies outlined in **Sections 5.3** to **5.5**, a freeboard allowance of 500 mm on 100 year ARI flood levels is suggested as the interim *FPL* and for defining the extent of the interim *FPA* pending the completion of the *FRMS&P.*

Interim *FPL* contours developed on that basis and the associated interim *FPA* for Main Stream flooding only are shown on **Figure 5.18**. As noted in **Section 2.9**, assessment of corresponding data for the urban area of Uranquinty subject to overland flooding will be undertaken during the *FRMS&P* (refer to preliminary discussion on the approach in **Section 2.10**).

5.7 Summary of Flood Affectation and Issues for Further Investigation in the *FRMS&P*

Following is a brief summary of flood affectation at Uranquinty and the issues which will need to be further investigated during the preparation of the *FRMS&P*.

- At the 100 year ARI, floodwaters on Sandy Creek extend over a width of 500 - 1200 m along the extent of the study reach. The time of rise of Sandy Creek under design flood conditions is around 7 to 8 hours (**Figure 5.2**).
- The hydraulic analysis for the design flood estimation has included partial failure of the levees for floods greater than the IFF (defined at **Section 2.7.4**).The IFF for the Connorton Street and Town Levee (South) systems is 5 years ARI. A preliminary assessment of the effects of partial levee failure was also carried out (increases in peak flood levels for the 100 year ARI compared with the case of no scour of the levees are shown on **Figure 5.14**). Peak water levels within the protected part of the village would be increased by up to 500 mm, with a corresponding increase in inundated area. A detailed hydraulic analysis involving a rigorous assessment of the implications of levee failure is warranted during the preparation of the *FRMS&P*.
- Mitigation of existing flooding problems will require the upgrading of the levees. This will be one of the important management measures to be considered in the *FRMS&P*. Selection of appropriate crest levels will require consideration of potential rises in design flood levels due to increased hydraulic roughness in the floodplain, partial blockage of downstream bridge waterways and future climate change.
- The floodway and flood hazard extents shown in **Figure 5.11** are based on depth and velocity of flow considerations and should be regarded as provisional. A final determination of hazard should be made in the *FRMS&P* in areas subject to both main Stream and Major Overland Flow, based on additional considerations summarised in **Section 2.6.2**.
- Matching the extents of inundation determined in this flood study with the footprints and floor levels of existing residential development will be undertaken in the *FRMS&P* to estimate damages resulting from a range of flood events. The hydraulic and economic analysis of potential works will be undertaken at the strategic level of detail in the *FRMS&P*. This will enable a priority list of flood management measures to be prepared.

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7 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).