Harrow

Flood Investigation

Draft Final Report

June 2017

DOCUMENT STATUS

Version	Doc type	Reviewed by	Approved by	Distributed to	Date issued
v01	Draft Report	Ben Hughes	Ben Tate	Tatjana Bunge	04/04/2017
v02	Final Report	Ben Hughes	Ben Hughes	Tatjana Bunge	31/05/2017

PROJECT DETAILS

Project Name	Harrow Flood Investigation
Client	Glenelg Hopkins CMA
Client Project Manager	Tatjana Bunge
Water Technology Project Manager	Ben Hughes
Report Authors	Ben Hughes, Emily Darlison
Job Number	4296-01
Report Number	R06
Document Name	4296-01R06V02a_Final_Report.docx

Cover Photo: Flooding in Harrow during January 2011 flood event, captured 10 December 2010, 7:41am (GHCMA)

Copyright

Water Technology Pty Ltd has produced this document in accordance with instructions from Glenelg Hopkins CMA for their use only. The concepts and information contained in this document are the copyright of Water Technology Pty Ltd. Use or copying of this document in whole or in part without written permission of Water Technology Pty Ltd constitutes an infringement of copyright.

Water Technology Pty Ltd does not warrant this document is definitive nor free from error and does not accept liability for any loss caused, or arising from, reliance upon the information provided herein.

15 Business Park Drive
Notting Hill VIC 3168

Telephone (03) 8526 0800
Fax (03) 9558 9365

ACN No. 093 377 283
ABN No. 60 093 377 283

Quality ISO 9001 SAI GLOBAL
Environment ISO 14001 SAI GLOBAL
Health & Safety AS 4801 SAI GLOBAL

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. EXECUTIVE SUMMARY

Glenelg Hopkins CMA commissioned Water Technology to undertake the Harrow Flood Investigation. The study included detailed hydrologic and hydraulic modelling of the Glenelg River, Salt Creek and several small tributaries near Harrow.

Harrow is in south western Victoria, approximately 75 km north west of Hamilton and 30 km south east of Edenhope. The township is located on the Glenelg River downstream of the Salt Creek confluence.

The Glenelg River begins in the Grampians National Park where it interacts with Moora Moora Reservoir via a diversion channel, and flows on to Rocklands Reservoir. Rocklands is a significant storage operated by GWMWater and its construction in 1953 has significantly altered the flow regime of the Glenelg River and the potential for flooding in Harrow. As such, streamflow records prior to 1953 are not reflective of potential flows today and were not considered relevant for calibration and design flow determination.

The Harrow community was actively involved in the investigation through community consultation sessions, social media and meetings with a Project Steering Committee which included several community members. The community consultation sessions were largely managed by Glenelg Hopkins CMA and West Wimmera Shire Council. The aims of the community consultation were to raise awareness of the study, to identify key community concerns, to provide information to the community and seek their feedback/input regarding the study outcomes including the existing flood behaviour and proposed mitigation options for the township.

Three major community meetings were held:

- Initial community meeting, Harrow Hermitage Hotel – 18th February 2016 – The first public meeting was held to outline the objectives of the study to the community, communicate what the community can expect from the study and gather input from the community on observed inundation and potential mitigation solutions.
- Second community meeting, Harrow Hermitage Hotel – 2nd June 2016 – The second community meeting presented calibration results for the September and December 2010 events and outlined a list of potential flood mitigation options identified to date. Community feedback was sought on the flood modelling results and their preference/suggestions for additional flood mitigation options.
- Third community meeting, Harrow Hermitage Hotel – 19th December 2016 –The final public meeting presented planning scheme layers, mitigation modelling and project outcomes. Community feedback was sought on potential levee design, location and appearance.

There are numerous streamflow gauges on the Glenelg River which can be reflective of potential flooding in Harrow, the most significant of these is Glenelg River at Rocklands, Glenelg River at Fulham Bridge and Glenelg River at Harrow. These gauges were used during the streamflow analysis for this project.

The primary aims of the streamflow analysis undertaken for this project included:

- Determine calibration events and flows to be used in the hydraulic model.
- Determine design event peak flow and hydrograph shape for input to the hydraulic model at the model boundaries. Design events included 0.2%, 0.5%, 1%, 2%, 5%, 10% and 20% AEP flood events, Probable Maximum Flood (PMF) and climate change scenarios.
- Test the impact of varying starting levels in Rocklands Reservoir on flows in the Glenelg River downstream of Rocklands.

To achieve these aims, the streamflow analysis was separated into two major components determining flows for the two major contributing catchment areas; downstream and upstream of the Fulham Bridge streamflow gauge. Flows for these areas were determined as follows:

- **Glenelg River tributary flows between Fulham Bridge and Harrow** – Inflows to the Glenelg River between Fulham Bridge and Harrow were determined using a RORB runoff routing model for both calibration and design. The inflows were then entered into a 1D hydraulic model of the Glenelg River between Fulham Bridge and Harrow, combining with the routed Fulham Bridge flow.
- **Upstream of the Glenelg River at Fulham Bridge** –
    - **Calibration** - Calibration flows for the catchment area upstream of Fulham Bridge were directly extracted from the Fulham Bridge gauge record. They were then used as an inflow boundary to the 1D model between Fulham Bridge.
    - **Design** - Peak flows for the catchment area upstream of Fulham Bridge were determined via an annual series peak flow Flood Frequency Analysis (FFA) at the Fulham Bridge gauge. The hydrograph shape and volume were determined by a RORB model of the catchment upstream of Fulham Bridge. The volume of the RORB generated Fulham Bridge hydrograph was then confirmed by using a volume based FFA at the Fulham Bridge gauge based on a four-day event duration. Four days was determined as the typical event duration in the Glenelg River at Fulham Bridge.

Each hydrology component was calibrated using the September 2010, December 2010 and January 2011 events.

The 2010 Dept. of Sustainability and Environment Index of Stream Condition LiDAR provided high accuracy topographic data for hydraulic modelling elements of the. A series of surveyed road crest and survey transects were used to verify the accuracy of the Index of Stream Conditions (ISC) LiDAR data available for the project. Glenelg River transects at Harrow captured during the 2003 Harrow Rehabilitation Survey were also compared to the ISC data as part of the verification process. During this processing, a 0.32 m systematic error in the post processing of the Glenelg Hopkins Region ISC data was found. This was consistent with the error found in the same data set for previous Glenelg Hopkins Region flood investigations (e.g.. Skipton Flood Investigation (BMT WBM, 2014) and Glenelg Regional Flood Mapping Project (Water Technology, 2014).

The LiDAR data was used as a basis for a detailed combined 1D-2D hydraulic model of the study area. The hydraulic modelling approach consisted of the following components:

- One dimensional (1D) hydraulic model of key waterways, drainage lines and hydraulic structures;
- Two dimensional (2D) hydraulic model of the broader floodplain; and
- Linked one and two dimensional hydraulic model to accurately model the interaction between in bank flows (1D) and overland floodplain flows (2D).

The hydraulic modelling suite, TUFLOW, was used in this study. TUFLOW is a widely used hydraulic model that is suitable for the analysis of overland flows in urban areas. TUFLOW has four main inputs:

- Topography and drainage infrastructure data;
- Inflow data (based on catchment hydrology);
- Roughness; and,
- Boundary conditions.

The hydraulic model was calibrated using the September 2010 and December 2010 flood events, using surveyed flood heights, stream height information and anecdotal community observations. The calibrated model was then used to produce design flood mapping. The design flood mapping showed there were two buildings flooded above floor and three buildings flooded below flood during a 1% AEP flood event, both on the south eastern side of Blair Street.

Several addition sensitivity tests were also undertaken, including:

- The impact of additional Rocklands Reservoir releases during flood events;
- Variable floodplain roughness;
- Blockage factors at the Salt Creek and Glenelg River bridges; and,
- The impact of climate change.

During the process of the investigation several structural mitigation options were suggested to reduce the impact of floods in Harrow. Water Technology reduced the number of options to be reviewed in detail using a prefeasibility assessment. The options that warranted further investigation were as follows:

- Build a levee to protect the township along the back of the buildings on Blair Street;
- Build levees/raised garden beds to protect individual properties;
- Build/alter the levee around Johnny Mullagh Memorial Park to the height of the road; and,
- Ensure no environmental releases are occurring at the same time as an expected flood event.

A levee along the back of the properties along Blair Street was modelled to assess any potential adverse impacts during all floods up to and including the 1% AEP event, modelling showed no building were flooded to a higher depth and no additional buildings were flooded. The levee could successfully remove inundation from all properties along Blair Street. During community meetings, the community were generally not in support of a broad scale levee option to protect these properties due to the potential aesthetic impacts of the levee and the limited number of properties impacted. Individual property protection with levees or raised garden beds was considered a more appropriate option for these properties.

A levee around the Johnny Mullagh Memorial Park protecting to above a 1% AEP flood level was shown to cause an increase in flood level at properties already flooded above floor. On discussion with the community, a lower levee height was modelled allowing overtopping during events rarer than a 5% AEP. This prevented frequent inundation but was shown to reduce the upstream water level increases enough so no adverse impacts on buildings were observed.

Non-structural mitigation measures were also assessed, including a review of the existing flood warning system, the implementation of Land Subject to Inundation Overlay (LSIO) and Flood Overlay (FO) within Harrow and updates to the Municipal Flood Emergency Plan (MFEP) to include specific detail around Harrow.

Due to the level of community concern, water level sensitivity testing was completed including the addition of a steady state flow to the design flows at Harrow. A steady state flow of 61.3 m^3/s increased water levels in Harrow by around 0.3 m, while steady state flows of 14.5 and 6.9 m^3/s increased levels by 0.075 m and 0.03 m respectively, these flows are representative of the maximum and typical environmental flow releases from Rocklands Reservoir. In the 6.9 m^3/s scenario there was no perceivable increase in inundation extent. This demonstrates that controlled releases are not likely to add significantly to natural flood levels at Harrow with the level of increase relatively minor.

The investigation made the following recommendations:

1. The West Wimmera Shire Council Municipal Flood Emergency Plan (MFEP) be updated with the information provided in the Harrow Flood Investigation Flood Intelligence Report.
2. The Land Subject to Inundation Overlay (LSIO) and Flood Overlay (FO) and associated planning scheme amendment documentation produced as part of this study be adopted in the West Wimmera Shire Council Planning Scheme.

3. The Victorian Flood Database (VFD) should be updated using the outputs of the Harrow Flood Investigation which have been formatted into the standard VFD outputs.
4. The Harrow Flood Investigation VFD deliverables should be uploaded to FloodZoom.
5. Bureau of Meteorology Flood Class Levels should be determined for the Glenelg River at Fulham Bridge and the Glenelg River at Harrow streamflow gauges and related to maps in the West Wimmera Shire Council Municipal Flood Emergency Plan.
6. A crowdsourcing flood information network for Salt Creek involving adjacent landholders should be created, including the installation of gauge boards as reference points.
7. An emergency flood plan for the Harrow RSL club should be created.
8. The local CFA brigade should be actively engaged in community preparedness education for flooding.
9. A levee around the Johnny Mullagh Memorial Park should be considered further with community groups and considered for funding. The identification of an aboriginal a scar tree at the Johnny Mullagh for which flooding is important may hinder this level of protection.

2. INTRODUCTION

Water Technology was commissioned by Glenelg Hopkins CMA to undertake the Harrow Flood Investigation. The study included detailed hydrologic and hydraulic modelling of the Glenelg River, Salt Creek and several small tributaries in the vicinity of Harrow.

This is the Final Study Report, combining all previous reports produced by Water Technology except for the Harrow Flood Investigation Flood Intelligence Report which was written for inclusion in the West Wimmera Shire Council Municipal Flood Emergency Plan. All previous reporting stages were reviewed by Glenelg Hopkins CMA and the project Steering Committee. Major reports underwent an independent peer review via a process managed by the Department of Environment, Land, Water and Planning (DELWP). This final report combines the comments received throughout the review process including the independent peer reviewers.

Two reporting stages were not completed by Water Technology, these are as follows –

- Harrow Flood Investigation – Flood Warning Recommendations (Molino Stuart)
- Harrow Flood Investigation – West Wimmera Shire Council, planning scheme amendment documentation (Planning and Environmental Design)

These reports are summarised in this report. Further detail can be sourced from them directly.

2.1 Study Area

Harrow is in south western Victoria, approximately 75 km north west of Hamilton and 30 km south east of Edenhope. The township is located on the Glenelg River with several small tributaries in close proximity, the most significant of these to Harrow is Salt Creek, flowing into the Glenelg immediately upstream of Harrow.

The Glenelg River begins in the Grampians National Park where it interacts with Moora Moora Reservoir via a diversion channel and flows on to Rocklands Reservoir, the largest storage in the system. Rocklands is a significant storage operated by GWMWater and its construction in 1953 has significantly altered the flow regime for the Glenelg River.

Harrow is located approximately 75 km downstream of Rocklands Reservoir. The major waterways are shown in Figure 2-1. The figure shows the Salt Creek catchment to the north flowing into the Glenelg River at Harrow.

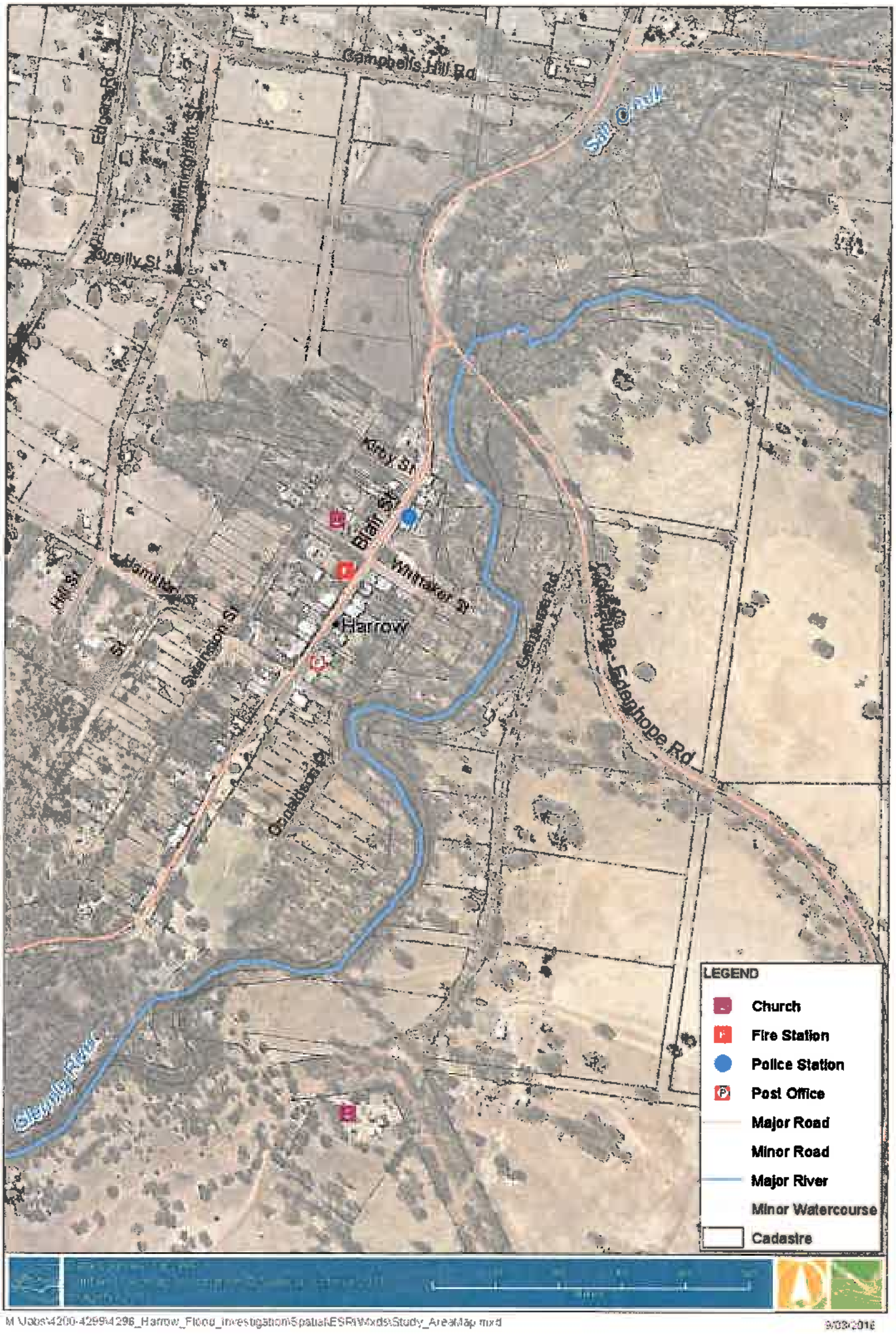


Figure 2-1 **Harrow – Major waterways within the township**

3. DATA COLLATION AND REVIEW

3.1 Overview

Data collation and review undertaken as part of this project documented previous flood related information for the study area, this included:

- Previous flood related studies
- Hydrological Data
  - Streamflow
  - Rainfall
  - Storages
- Flood Records
  - August 1956
  - October 1975
  - August 1981
  - September 1983
  - September 2010
  - December 2010
- Physical features
  - Topographic survey
  - Observed peak flood heights
  - Floor level and feature survey
- Site visit

3.2 Flood Related Studies

Several previous studies relevant to flooding of the Glenelg River were available, including:

- Glenelg Flood Investigations (Cardno Lawson and Treloar, 2008)
- Casterton Flood Investigation (Cardno, 2011)
- Review of Storage Operation During Floods Grampians Wimmera Mallee Water (Water Technology, 2011)
- Preparation of Glenelg Hopkins CMA Submission to the Review of 2010-11 Flood Warnings and Response (Water Technology, 2012)
- Casterton Flood Intelligence & Warning Improvements (WBM BMT, 2014)
- Glenelg Regional Flood Mapping Project (Water Technology, 2015)
- Glenelg River Technical Flows Study (Water Technology, 2015)

The most relevant of these was the Glenelg Regional Flood Mapping Project, these report is documented in detail in the following section.

3.2.1 Glenelg Regional Flood Mapping Project

The Glenelg Regional Flood Mapping Project[1] is the most recent relevant project to the Harrow Flood Investigation. The study included detailed one-dimensional and two-dimensional flood modelling of Harrow, reviewed all Glenelg River streamflow gauges, constructed and calibrated a RORB hydrological model of the catchment, undertook design flow estimates using the calibrated RORB model and Flood Frequency Analysis (FFA) at all gauges with a sufficient gauge record length.

[1] Water Technology, 2015 – Glenelg Regional Flood Mapping Project, report prepared for DELWP

Modelling completed during the Glenelg Regional Flood Mapping Project used the September 2010 and December 2010 events for calibration in the Harrow township due to the presence of surveyed flood heights and good flow and water level information captured at the Harrow gauge. The 1983 event was also modelled in both the 1D and 2D models, however limited calibration information was available at Harrow with the focus of the events elsewhere on the Glenelg River during these events.

There were seven flood marks surveyed of the September 2010 flood peak in Harrow. Unfortunately only two of these were referenced to AHD and one was referenced to a gauge board on the Glenelg River with an unknown gauge zero.

The 2D hydraulic model September 010 calibration achieved during the study is shown below in Figure 3-1, the calibration results show an excellent match to the observed data. The calibration was achieved using a uniform Manning's 'n' roughness of 0.06.

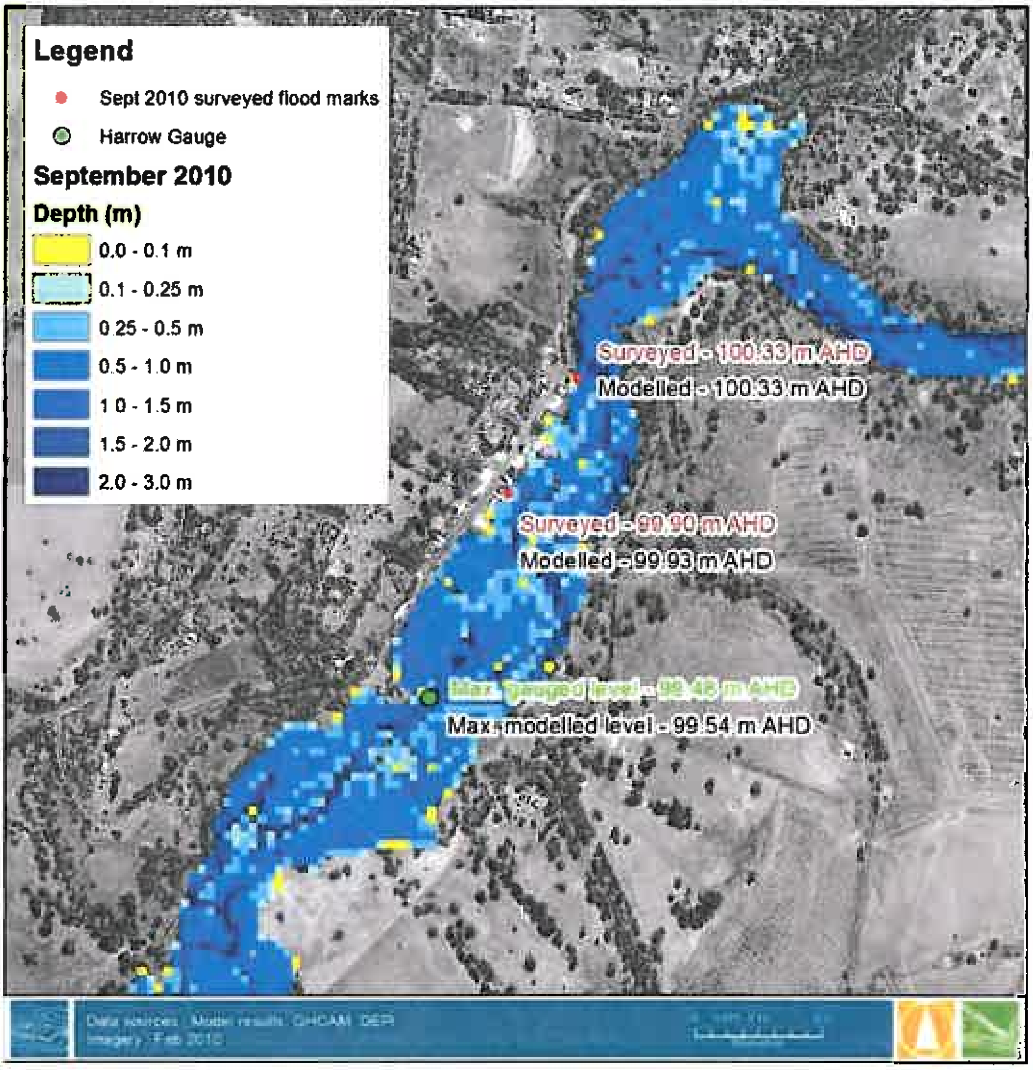


Figure 3-1 December 2010 2D model calibration – Glenelg Regional Flood Mapping Project

There were nine surveyed flood marks available for the December 2010 event in Harrow. The marks were surveyed and supplied by GHCMA. The 2D hydraulic model was run using a Manning's 'n' roughness of 0.06, as determined during the September 2010 calibration. The observed and modelled flood height comparisons are also shown in Figure 3-2.

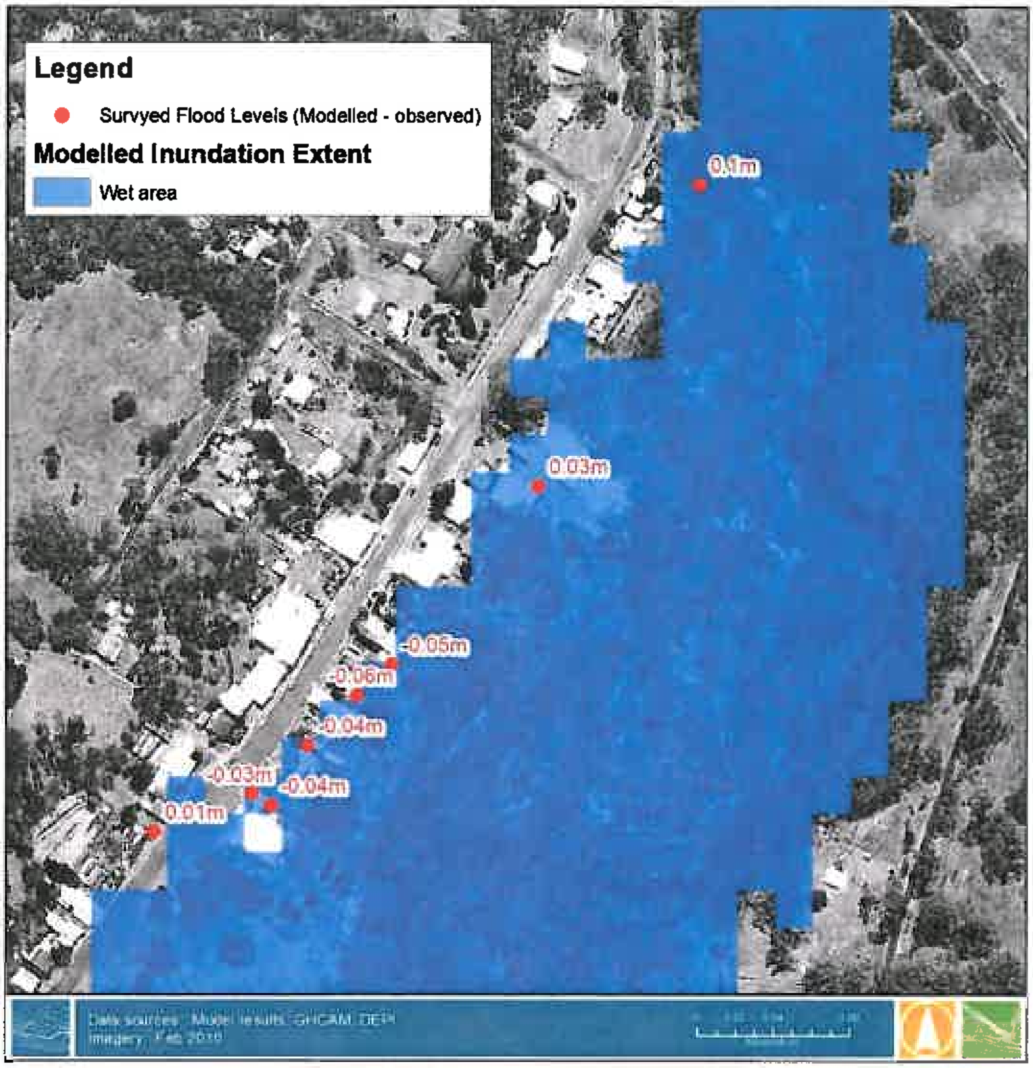


Figure 3-2 **December 2010 2D model results and surveyed flood marks**

The Glenelg Regional Flood Mapping[1] design flow estimates were developed using both a calibrated RORB model and a FFA at the Fulham Bridge gauge. Unfortunately, the Harrow streamflow gauge had insufficient gauge record for completion of a FFA, with more data available at the Fulham Bridge gauge. An analysis of the concurrent record shows only a small degree of change in peak flow between the Fulham Bridge and Harrow streamflow gauges.

A comparison of the design flow estimates at the Fulham Bridge gauge made using both RORB and FFA is shown in Table 3-1.

Table 3-1 **Glenelg Regional Flood Mapping Project - FFA and RORB model peak flows**

Design Event Annual Exceedance Probability (AEP)	Fulham Bridge Peak Flow Estimates (m^3/s)		RORB Critical Duration (hours)
	FFA	RORB	
20%	6,310	6,650	30
10%	8,730	8,900	36

5%	10,500	10,700	30
2%	11,800	12,000	30
1%	12,500	12,600	30
0.5%	12,960	12,960	30

3.3 Hydrological Data

3.3.1 Streamflow

Currently, there are four operational stream flow gauges upstream of Harrow. An additional gauge at Balmoral was discontinued in 1956. Each of these gauges is shown in Table 3-2, detailing the period of record and maximum flow recorded. The gauge locations are also shown in Figure 3-3.

Rocklands Reservoir has a large influence on flows in the Glenelg River, the reservoir finished construction in 1953. Therefore, events prior to 1953 are not reflective of streamflows that may be observed today and were omitted from the calibration and design flow determination.

Table 3-2 Study area gauge details

Location	Number	Start Date	Start instantaneous	End Date	Peak Flow (m^3/s)	Peak flow date
Big Cord	238231	24/04/1968	17/05/1979 15:00	Current	10.2	January 2011
Rocklands	238205	22/03/1941	21/07/1983 4:01	Current	77.9* 47.0^	September 1942 & March 1946 August 1956
Balmoral	238201	25/05/1889	-	1/10/1956	365.4	March 1946
Fulham Bridge	238224	06/03/1964	8/01/1976 13:00	Current	131.3	December 2010
Harrow	238210	30/11/2001	30/11/2001 14:58	Current	116.7	December 2010

* Maximum peak flow occurred prior to the construction of Rocklands Reservoir in 1953
^ Peak flow post the construction of Rocklands Reservoir

There have been no major spills from Rocklands Reservoir since construction, with the largest 47 m^3/s in 1956. The Fulham Bridge gauge has recorded much larger flows, indicating that the catchment downstream of Rocklands Reservoir can contribute significant flow that generate floods without requiring spills from Rocklands Reservoir. Floods could also be produced by large rainfalls in the upper catchment leading to Rocklands Reservoir filling and spilling in combination with runoff generated in the lower catchment. Given the capacity of Rocklands Reservoir, the current operational rules which mandate the storage must not exceed 80% capacity, and record of spills since 1953, future spills are unlikely to be frequent. For example, in the record wet years of 2010-12, Rocklands Reservoir only filled to around 40% of its operating capacity.

The Fulham Bridge and Harrow streamflow gauges have the highest value to this study. The Fulham Bridge gauge is located approximately 40 km upstream of Harrow while the Harrow gauge is located south of the Harrow township, immediately downstream of the Harrow Recreation Reserve.

It must be noted the water quality and gauge height measurements for the Harrow gauge are in different locations with the water quality recordings taken approximately 350 m downstream of

Coleraine-Edenhope Road (they are shown as the same location on the Department of Environment, Land, Water and Planning (DELWP) Water Measurement Information System[2]). The location of these gauges is shown in Figure 3-4.

The Harrow and Fulham Bridge gauges are discussed in detail in the following sections, while the remaining gauges are discussed more briefly.

[2] DELWP Water Measurement Information System - http://data.water.vic.gov.au/monitoring.htm

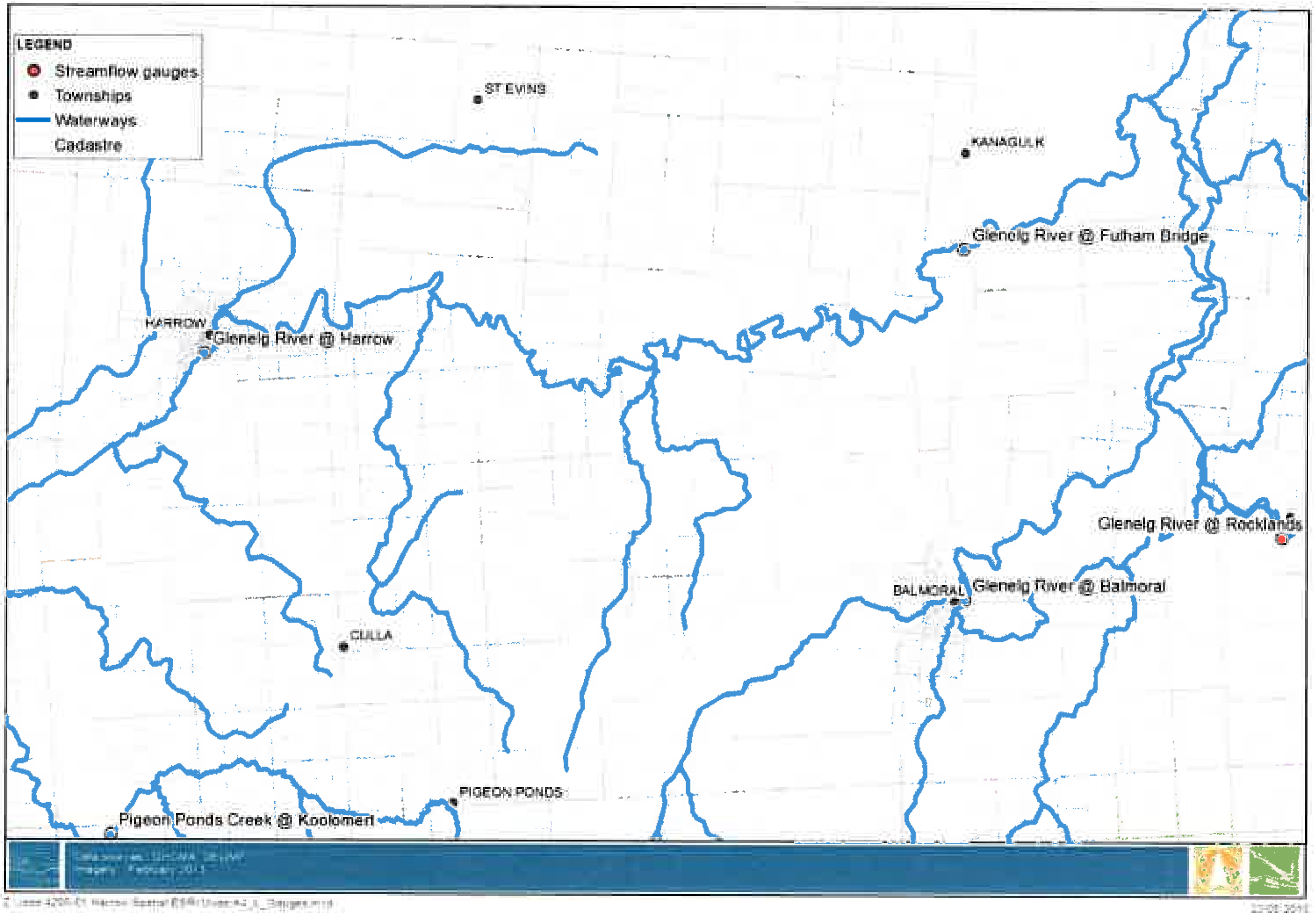


Figure 3-3 **Streamflow gauge locations**

Figure 3-4 **Harrow streamflow and water quality gauge locations**

Glenelg River at Fulham Bridge

The Fulham bridge gauge had 37 full years of record at the completion of this investigation. This was sufficient to determine design flow estimates using FFA. The current gauge rating curve along with all past gauging observations is shown in Figure 3-5.

The gauging measurements shown generally match the adopted rating curve, with the current rating curve slightly overestimating the flow in some cases. These older gaugings are not likely to be used to construct the current rating curve. Interestingly the Bureau of Meteorology's Water Data Online website[3] was checked and the rating curve currently being applied at Fulhum Bridge is different to the DELWP rating curve on the Water Information Measurement System[2]. There appears to be two distinct branches of the Glenelg River at the Fulham Bridge site, it is unknown how the gaugings are taken at these locations for generation of the rating curve.

The DELWP rating curve suggests that the gauge data is reliable up to a height of 2.4 m or 74 m^3/s (6,400 ML/d), beyond which it is extrapolated.

The stream height record at the Fulham Bridge gauge is shown in Figure 3-6, the gauge record shows a large number of high flow events prior to 1996, then a period of very low stream heights in the early 2000s, and several high flow events in 2010-2011. The December 2010 event is the only event outside of the reliable section of the rating curve, with a recorded level of 2.74 m. This event was used in the

[3] Water Data Online (Bureau of Meteorology), http://www.bom.gov.au/waterdata/

model calibration process, as discussed in Section 6.3.1. The streamflow estimates for the calibration events of September 2010 and December 2010 are all likely to be accurate.

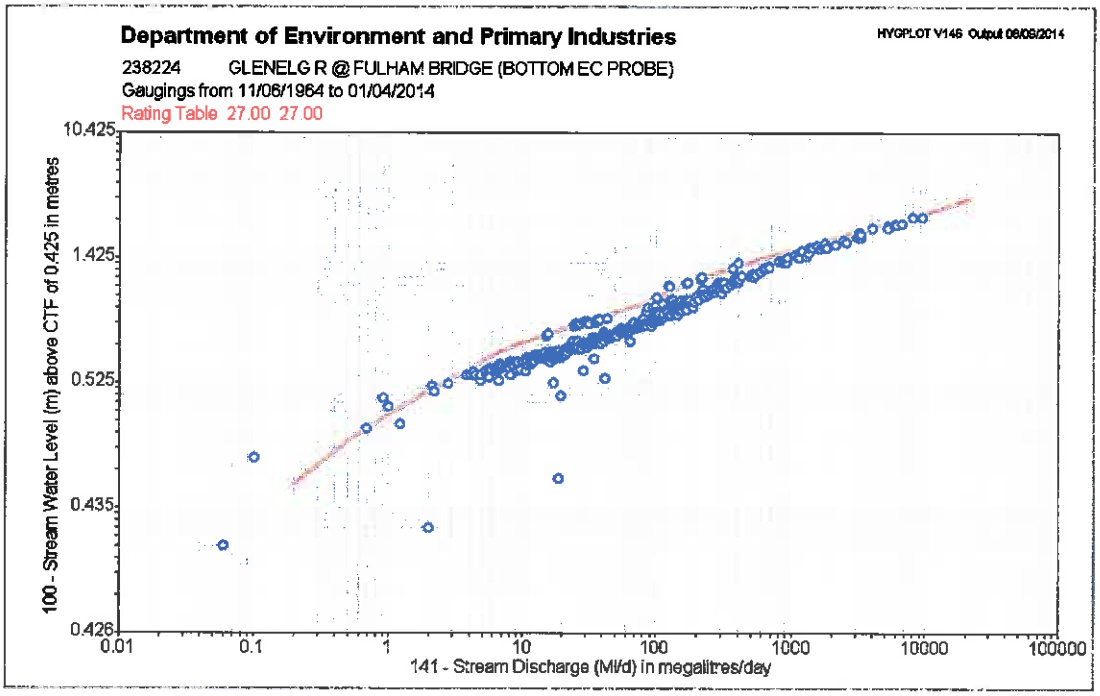


Figure 3-5 Comparison of the measured water levels and flows at Fulham Bridge[2]

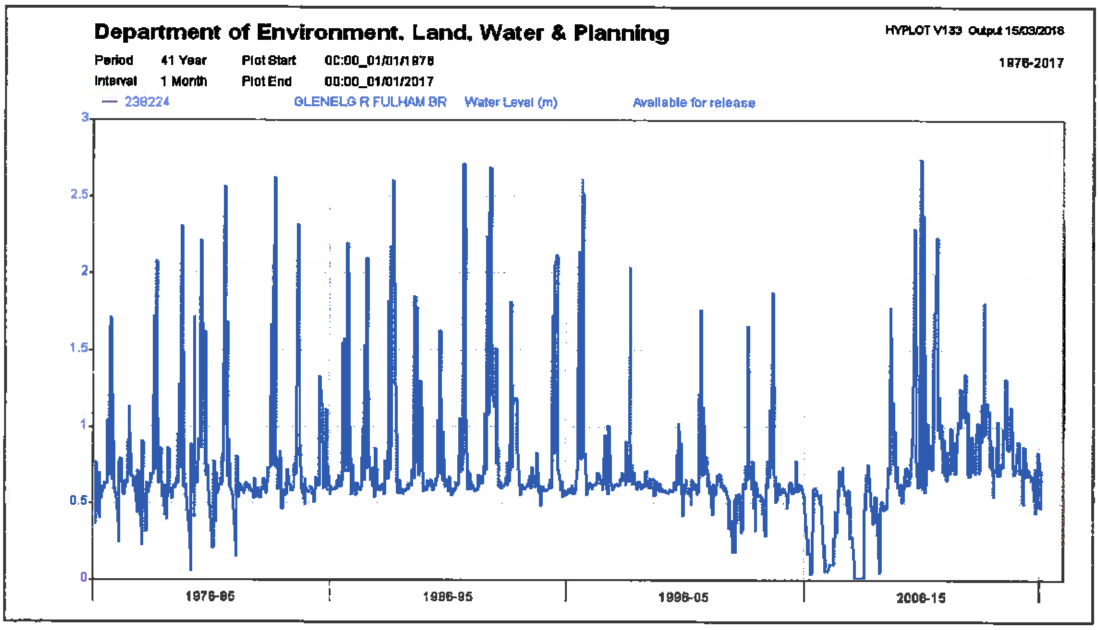


Figure 3-6 Glenelg River at Fulham Bridge Gauge Records[2]

Glenelg River at Harrow

At the time of this projects completion, the Harrow gauge had a record of 14 complete years, insufficient for use in estimating design flow estimates through FFA. The rating curve and all past gauging events are shown in Figure 3-7.

The Harrow gauge rating is not as accurate and doesn't cover the same flow range as the Fulham Bridge gauge. The gauge is only considered reliable over a very narrow range, between 0.54 m and 1.1 m, or 0.3 m^3/s and 8.8 m^3/s (28 ML/d and 760 ML/d). Despite the rating curve not being considered reliable for flows above 8.8 m^3/s (760 ML/d), the Glenelg River Regional Flood Mapping Project[1] showed that the extrapolated rating curve and observed flows matched the modelled flows very closely when routing Fulham Bridge observed flows and RORB modelled tributary inflows through a 1D model of the Glenelg River. This suggests the extrapolated rating curve is reasonably good for flows up to the calibrated December 2010 flow of nearly 54.4 m^3/s (4,700 ML/d).

The stream height record at the Harrow gauge is shown in Figure 3-8. The gauge record shows several high flow events in late 2010 and early 2011, which are all around 1 m or more above the reliable section of the rating curve. As described above, the extrapolated rating curve is considered reliable for events up to the December 2010 magnitude.

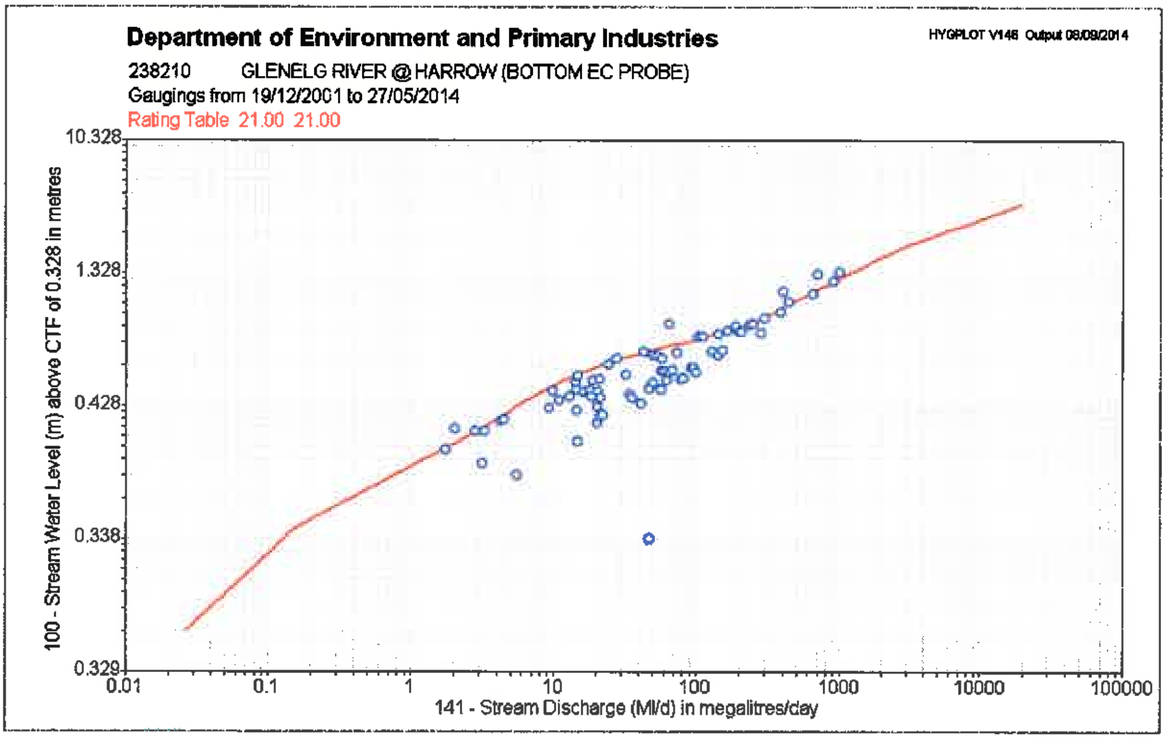


Figure 3-7 Comparison of the measured water levels and flows at Harrow[2]

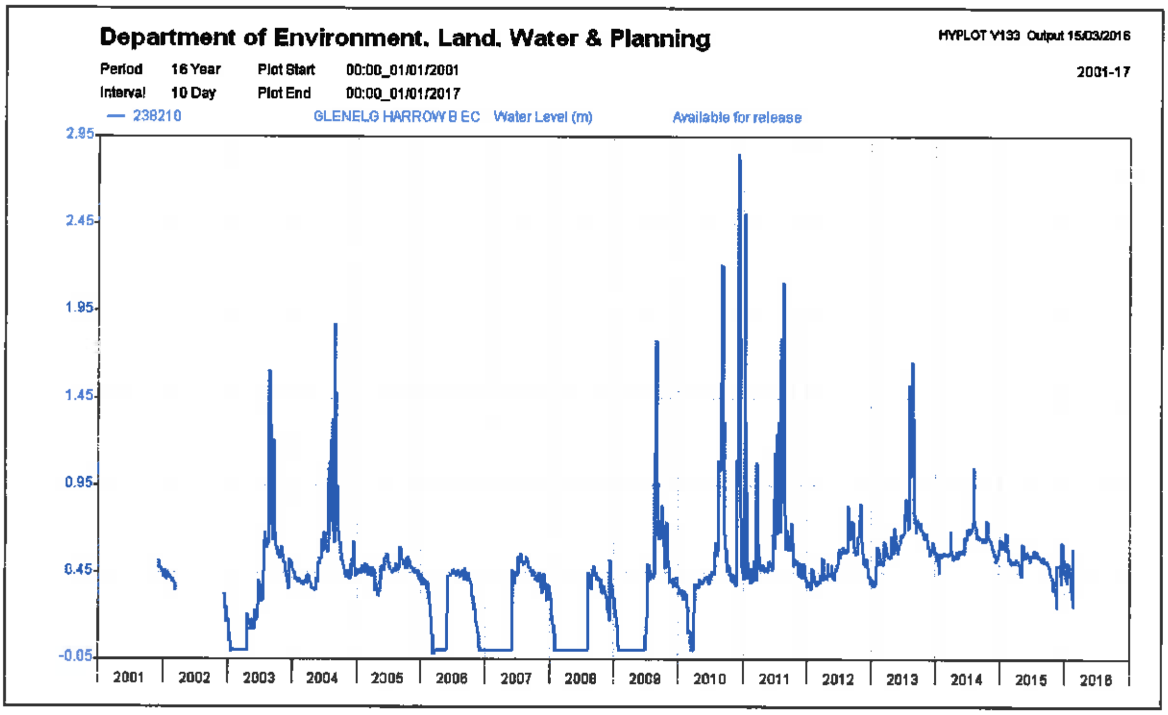


Figure 3-8 **Glenelg River at Harrow Gauge Records**[2]

Other gauges

In addition to the Fulham Bridge and Harrow streamflow gauges, upstream Glenelg River gauges are located at Balmoral, Rocklands and Big Cord.

The gauge at Balmoral has a streamflow record from 1889 to 1956, resulting in only three years of gauge record post the construction of Rocklands Reservoir.

The Rocklands gauge has flows from 1941 to current, the gauge is largely representative of outflows from Rockland Reservoir. The characteristics of Rocklands Reservoir are discussed further in Section 7.2.

The Big Cord gauge is upstream of Rocklands and has recorded flows from 1956 to the time of this projects completion. The gauge has a relatively small catchment area of 57 km^2 and isn't representative of the potential flows in the Glenelg River downstream of Rocklands Reservoir. Its rating curve is also quite limited, with flows spilling out of bank and across a wide flat valley floor in relatively frequent events.

Summary and Discussion

Assessing the reliability of streamflow gauges within a study area was a relatively fast and easily completed task. This is due to the availability of the gauge rating curves and base data on the DELWP online Water Measurement Information System[2]. It is important to understand a gauge rating curve, its limits and sections of the curve that are most likely to contain a higher degree of uncertainty.

The rating curves show that the Fulham Bridge gauge is reasonably accurate for the magnitude events used for calibration in this investigation. The Harrow gauge however has a very narrow range on the rating curve considered reliable, and the calibration events are all well beyond the reliable section of the rating curve. Previous work has demonstrated that the extrapolated rating curve at Harrow is reliable up to flows of the December 2010 magnitude.

Table 3-3 shows the ranked highest observed flows at the Fulham Bridge gauge and the corresponding peak flows at Harrow (although possibly inaccurate), where available.

Table 3-3 Highest ranked peak flows recorded at Fulham Bridge and Harrow Gauges

Year	Glenelg River at Fulham Bridge (m^3/s)	Glenelg River at Harrow (m^3/s)
2010	131	117
1991	128	-
1992	123	-
1983	116	-
1996	113	-
1988	112	-
1981	107	-
2011	78	80
1984	76	-
1979	76	-

Hydrographs of the December 2010 and January 2011 events are shown in Figure 3-9 and Figure 3-10 respectively. The hydrographs clearly show how flow in the Glenelg River changes between the Fulham Bridge and Harrow streamflow gauges. The Harrow streamflow gauge shows two defined peaks, one from tributary inflows between the gauges, the other the Glenelg River flow routed between them. In the case of January 2011, the tributary inflows between the gauges has provided a large peak flow, indicating their significance in generating large flood flows.

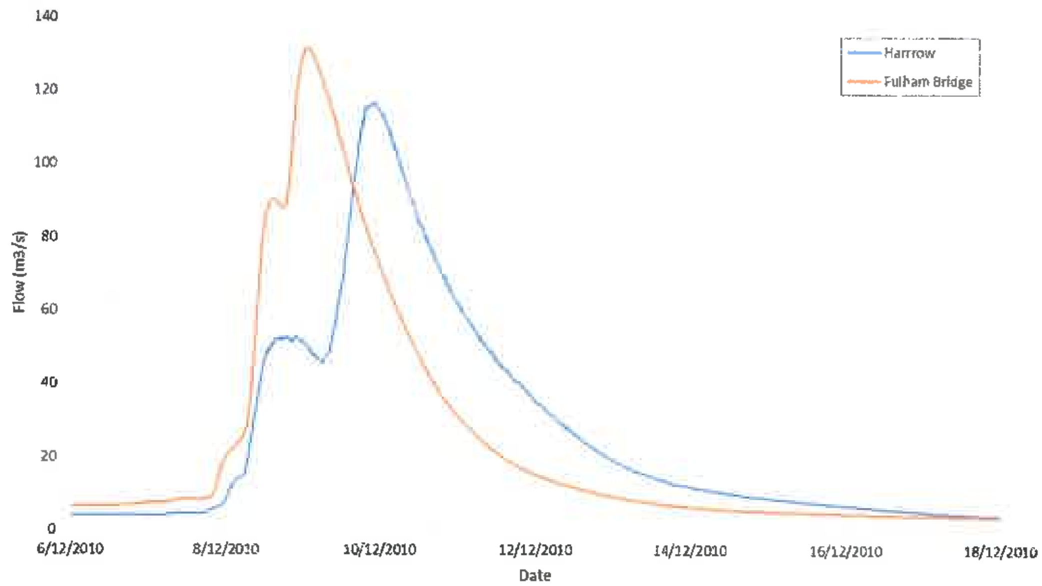


Figure 3-9 December 2010 – Hydrograph comparison at Fulham Bridge and Harrow

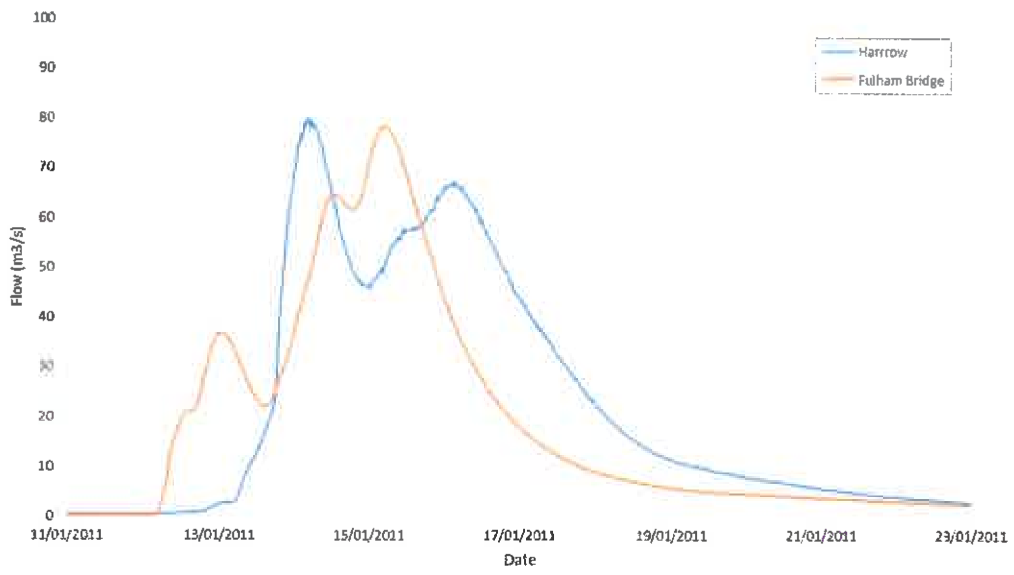


Figure 3-10 January 2011 – Hydrograph comparison at Fulham Bridge and Harrow

3.3.2 Rainfall

There are numerous daily rainfall gauges located across the Glenelg River catchment upstream of Harrow. There is also a sub-daily rainfall gauge located at Rocklands.

The daily and sub daily gauges considered relevant to this study are shown below in Table 3-4, detailing each gauge's period of record and maximum daily recording. The gauges within the Harrow catchment area are highlighted in **Bold**. Gauge locations are shown in Figure 3-11.

Table 3-4 Relevant rainfall gauges and their respective gauge record

Gauge Name	Gauge Number	Start of daily record	End of record	Max. Daily Recording (mm)	Year achieved
Clear Lake (Marlbro)	79008	1903	-	117.1	1957
Halls Gap (Post Office)	79074	1958	-	146.6	2011
Harrow (Post Office)	**79021**	**1908**	**-**	**108**	**1946**
Harrow (Pine Hills)	**79022**	**1884**	**2011**	**88.9**	**1952**
Rocklands Reservoir*	**79052**	**1948**	**2010**	**118.1**	**1957**
Telangatuk East (Milingimbi)	**79078**	**1968**	**-**	**95**	**2011**
Balmoral (Post Office)	89003	1884	-	104.1	1952
Mirranatwa (Bowacka)	89019	1901	-	124	1957
Willaura (Yarram Park	89037	1902	-	98	2010
Gatum (Orana)	89043	1953	-	88.4	1957
Coojar (Killara)	**90026**	**1939**	**-**	**90.4**	**1946**
Nareen	90140	1968	2005	68	1987
Wartook Reservoir	79046	1890	-	118.4	1941

* sub daily rainfall gauge

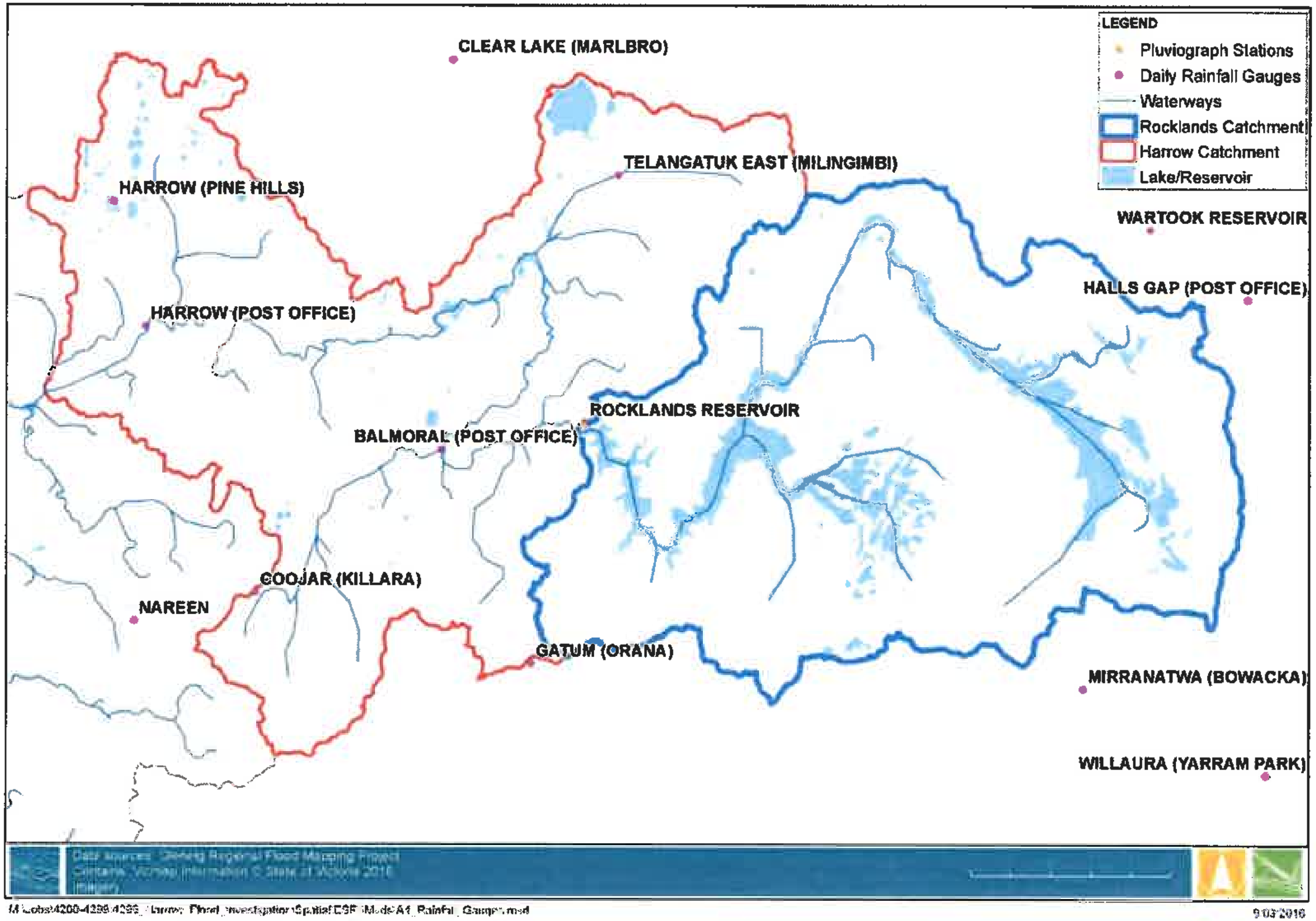


Figure 3-11 Rainfall gauge locations

3.3.3 Storages

There are two major water storages within the Glenelg River catchment upstream of Harrow, Rocklands Reservoir and Moora Moora Reservoir.

The following information was reproduced from the Glenelg Regional Flood Study[1] as the upstream storages are of relevance to this study. The impact of Rockland Reservoir, in particular, on flood behaviour at Harrow was raised by community members and was examined closely in this study.

"Moora Moora Reservoir is a relatively small reservoir upstream of Rocklands Reservoir, constructed in 1934. The reservoir has a Full Supply Volume of 6,300 ML and captures flows from Moora Moora Creek. The Reservoir is off line from Glenelg River. Moora Moora Reservoir Outlets to the Moora Channel which passes on to Distribution Heads.

Rocklands was finished construction in 1953, with a capacity of 348,000 ML. It is managed and maintained by GWMWater, the largest storage in their system. It was originally designed as a carry-over storage to be managed along with Toolondo Reservoir[4]. Due to its shape, Rocklands has much higher evaporation than Toolondo and therefore, water was transferred to and stored in Toolondo in preference to Rocklands. Inflow to Rocklands Reservoir averages 101,000 ML/year with much of the flow occurring during the period July to October[5].

In light of the Northern Mallee and Wimmera Mallee Pipeline Projects, Rocklands is used primarily to supply environmental flows and as a supplementary water source for Hamilton, suppling some irrigation and Supply by Agreement demands.

Approximately 40% of the water released by GWMWater for the environmental allocation each year is made as releases from Rocklands Reservoir into the Glenelg River to meet the Environmental Demands on the Glenelg River at Harrow[6]. The Reservoir is currently run with a maximum operating volume of 261,000 ML (or 75% capacity) at 194.1 m AHD, providing a de facto 87,000 ML of flood reserve. This reduced operating volume is in light of the storage being operated primarily for environmental flows but will also minimise flood overflows to the Glenelg River. The reduced operational level public consultation occurred during 2010 with the implementation occurring in early 2011. There was intention to change the operational capacity of Rocklands Reservoir to 85% in late 2014. The change had not occurred at the time of this reports production but was considered imminent[7]. The Rocklands Reservoir spillway is at 195.47 m AHD with a length of 154.5 m. The change in operational rules is unlikely to change the attenuation of flood flows.

The outlet capacity of Rocklands Reservoir is (14.5 m^3/s) 1,250 ML/d and releases from Rocklands Reservoir occur via the main outlet which connects to the Toolondo Channel and Glenelg River. Flows can be discharged to the Glenelg River at three locations: 5 Mile outlet, 12 Mile outlet and the wall. Transfers to Toolondo Reservoir are limited when the capacity of Rocklands exceeds 75% due to outlet constraints[5].

The GWMWater O&M Manual for Rocklands Reservoir states the dam has never passed a major flood, with the maximum outflow stated at 61.3 m^3/s (5,300 ML/d) in 1975[8]. Unfortunately, the data

[4] Barlow (1987) - Wimmera / Mallee Headworks System Reference Manual

[5] Water Technology (2011) - Review of Storage Operation During Floods Grampians Wimmera Mallee Water

[6] GHD (February 2011) - Report for the Wimmera-Glenelg REALM Model Update, produced for the Department of Sustainability and Environment

[7] GWMWater (March 2014) – Bulk and Environmental Entitlements Operations Review

[8] GWMWater (March 2010) - Rocklands Reservoir Operation, Inspection and Maintenance Manual (O&M Manual)

available via the DEPI Water Measurement Information System only shows the rising and falling limbs of the measured hydrograph on the Glenelg River at Rocklands. At what is assumed to be the peak flow the data quality code is listed as 254, Rating Table Exceeded.

The partial hydrograph recorded at the Rocklands streamflow gauge is shown in Figure 3-12.

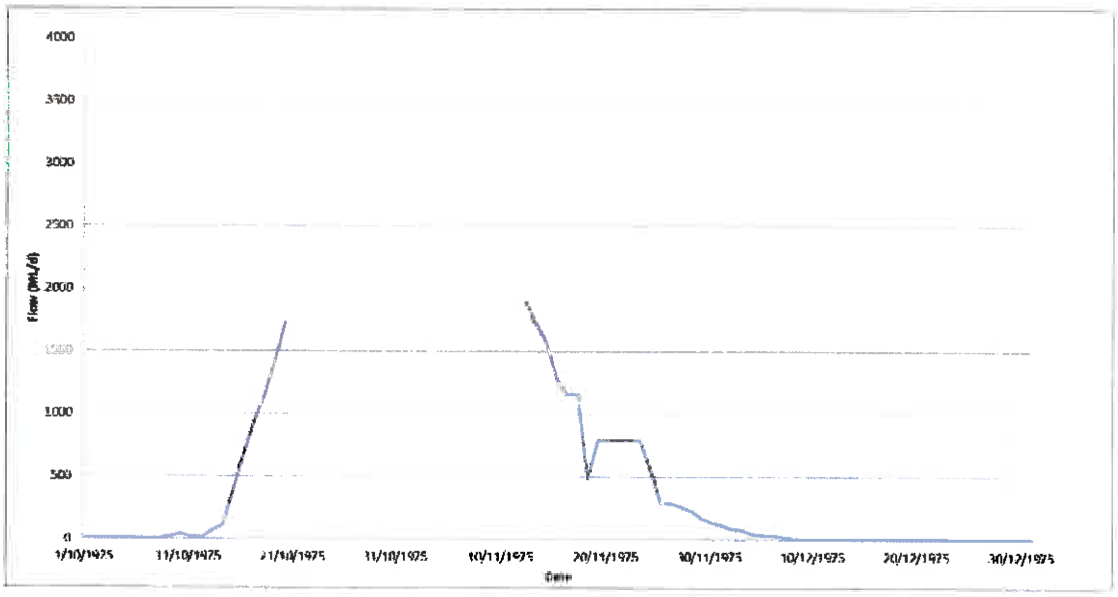


Figure 3-12 October 1975 flow on the Glenelg River at Rocklands

A review of the Rocklands Reservoir Head Gauge levels and discussion with former GWMWater staff[9] indicted reservoir spills have occurred in:

- *1953*
- *1955*
- *1956*
- *1958*
- *1960*
- *1974*
- *1975*
- *1988*
- *1989*
- *1990*
- *1992*
- *1993*
- *1996*

A number of these spills are not identified in the GWMWater reservoir level online record due to a re-rating of the reservoir volume which changed from 335,500 ML to 348,300 ML. In the years prior to 1988 the surcharge volume was also not recorded with the reservoir height only recorded as the spill way height. Of the spills that have occurred at Rocklands, only five have recorded flows greater than 23 m^3/s (2000 ML/d). The data and peak flow measured at the Glenelg River at Rocklands gauge for these spills is shown below in Table 3-5. No flood release procedures exist for Rocklands Reservoir[5]."

[9] Pers. Comm – John Martin (Former Executive Manager, Sustainable Water and Infrastructure)

Table 3-5 Rocklands Reservoir spill details

Spill Date	*Maximum discharge recorded on the Glenelg River at Rocklands*	
	ML/d	*m^3/s*
August 1956	*4060*	*47.0*
September 1974	*2250*	*26.0*
October 1975	*5300*	*61.3*
July 1983	*2605*	*30.2*
August 1988	*3280*	*38.0*
August 1992	*3540*	*41.0*

A key component of this project was to better understand the impact of Rocklands Reservoir on flooding at Harrow. A range of scenarios were modelled in the hydrological model and this is discussed further in Section 7.

3.3.4 Flood Records

Discussion of historic events focuses on events post the completion of Rocklands Reservoir in 1953, flood events prior to the construction of Rocklands Reservoir are of limited use in the model calibration with an aim to produce accurate design modelling. However, they are useful for community understanding and comparison to design mapping.

There have been several previous major flood events in Harrow, including September 1983 and most recently the September and December 2010 events. In Harrow, the December 2010 was the largest event since Rocklands construction in 1953.

The details of the historic events used in the model calibration are discussed in the hydrology (Section 5.4.3) and hydraulics (Section 6.3) sections of this report.

3.4 Topographic Data/Survey

3.4.1 LiDAR

High resolution LiDAR was available for the study area, ensuring the topography could be accurately represented in the hydraulic modelling. The Glenelg Regional Flood Mapping Project[1] used a series of surveyed road crest and survey transects to verify the accuracy of the Index of Stream Conditions (ISC) LiDAR data available for the project. Glenelg River transects at Harrow captured during the 2003 Harrow Rehabilitation Survey were also compared to the ISC data as part of the verification process. An example of these transects is shown in Figure 3-13 and Figure 3-14. The VicMap 20 m Digital Elevation (DEM) is also shown for comparison.

The surveyed transects showed a clear difference between the LiDAR and the surveyed transects, with the ISC LiDAR consistently higher than the survey. This was observed for survey data locations along the Glenelg River across all survey sources. The LiDAR verification process identified the difference between the survey and LiDAR data to be 0.32 m (ISC - Survey), meaning the ISC LiDAR data was 0.32 m higher than the survey. This was verified by the LiDAR verification undertaken during the Casterton Flood Investigation[13] and Skipton Flood Investigation[14]. which also found a uniform difference between the ISC LiDAR data and survey heights of 0.32 m. In both projects the ISC LiDAR data was lowered to accommodate for this difference. This shift in the LiDAR was used for the Harrow Flood Investigation, and no further control transects to verify the LiDAR datasets were required.

As shown in the below figures, the Glenelg River channel was generally not well represented by the LiDAR as it has captured the water surface at the time of survey. The available cross-section survey (shown in Figure 3-15) data was used to stamp in the channel to ensure its capacity was properly represented. Further transects focusing on the Glenelg River road crossings between Rocklands and Casterton are shown in Appendix A.

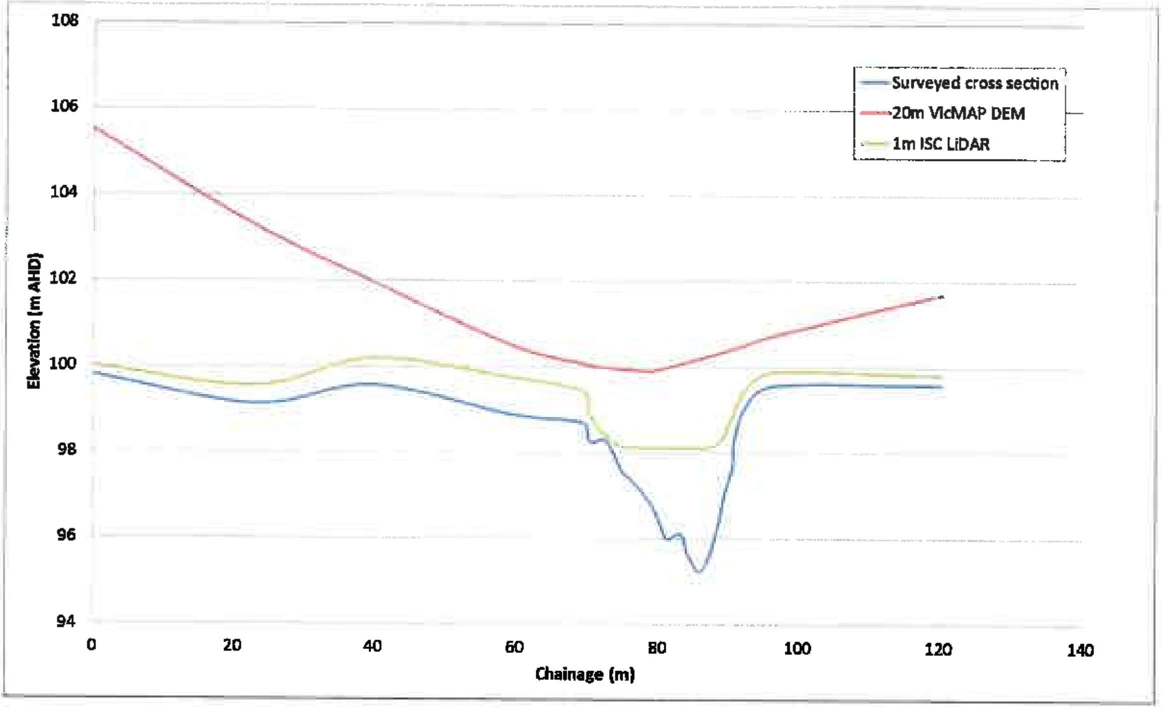


Figure 3-13 **Survey vs ISC LiDAR data cross section comparison at Harrow, Harrow Rehabilitation Survey – Chainage 1400 m**

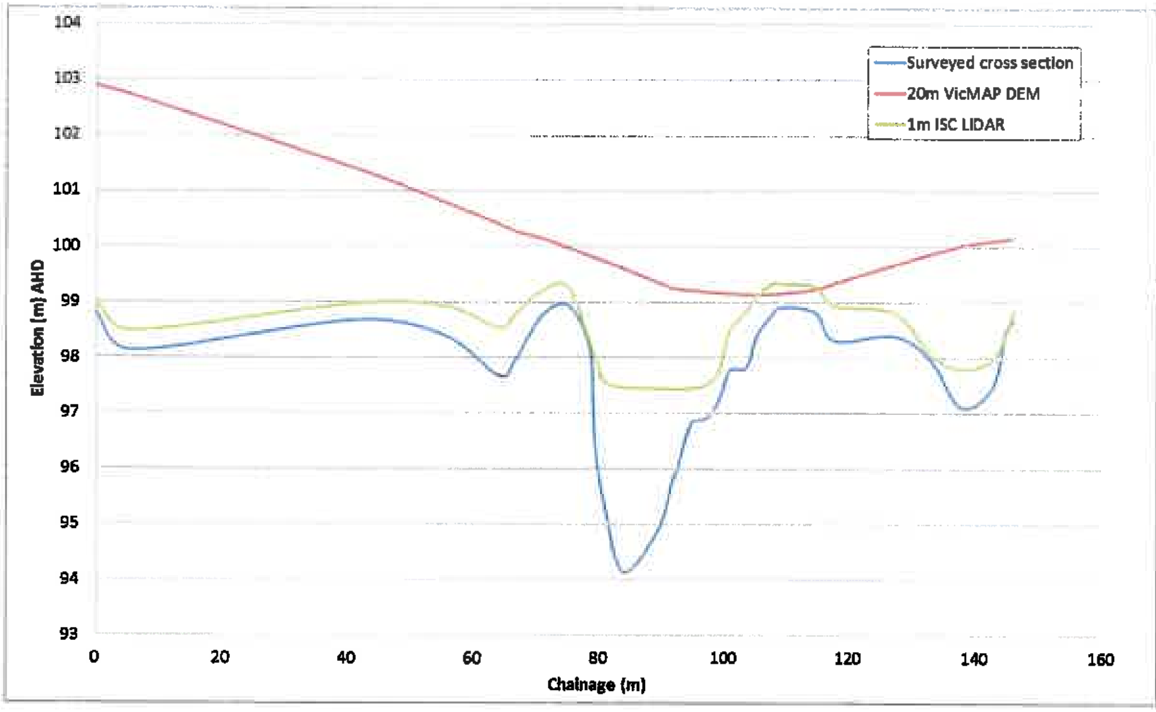


Figure 3-14 **Survey vs ISC LiDAR data cross section comparison at Harrow, Harrow Rehabilitation Survey – Chainage 2800 m**

Figure 3-15 Available cross-section survey transects[10]

[10] Glenelg Hopkins CMA, 2003 – Harrow Rehabilitation Survey

3.4.2 Observed peak flood heights and extents

A number of observed peak flood heights were available within the Harrow township. These surveyed levels area available for the following events

- 1946 (2)
- September 2010 (7) and
- December 2010 (9).

The location of the observed flood heights is shown in Figure 3-16.

Unfortunately, the only formal flood extents available for Harrow are for the 1946 event which was not preferred for calibration due to the construction of Rocklands Reservoir in 1953. However, a significant amount of community anecdotal evidence is available for the more recent events. This information was drawn on during the calibration process.

Figure 3-16 **Harrow - Observed peak flood heights**

3.5 Site Visit

A site visit was undertaken on 28th February 2016, prior to the inception meeting. A number of key floodplain features around the township were visited and photos taken. The images below show some of the key locations visited.

1946 flood level marked on pole at community hall

Outlet of small watercourse adjacent to Swanston Street

Weir located immediately downstream of stream flow gauge near football oval.

Location of streamflow gauge in weir pool adjacent to football oval.

4. PROJECT CONSULTATION

4.1 Overview

A key element in the development of the Harrow Flood Investigation was the active engagement of residents in the study area. This engagement was developed over the course of the study through community consultation sessions, social media and meetings with a Project Steering Committee including several members of the community. The community consultation sessions were largely managed by Glenelg Hopkins CMA and West Wimmera Shire Council. The aims of the community consultation were as follows:

- To raise awareness of the study and to identify key community concerns.
- To provide information to the community, seek their feedback/input regarding the study outcomes including the existing flood behaviour and proposed mitigation options for the township.

4.2 Stakeholder Advisory Group

The Harrow Flood Investigation was led by a Stakeholder Advisory Group consisting of representatives from Glenelg Hopkins CMA, West Wimmera Shire Council, Department of Environment, Land, Water and Planning (DELWP), State Emergency Service (SES), Bureau of Meteorology (BoM) Grampians Wimmera Mallee Water (GWMWater), Water Technology and the Harrow community.

The Steering Committee met on 3 occasions at key points throughout the study, to manage the development of the investigation. The meeting dates and basis for discussion was as follows:

- Thursday 18th February 2016 – Project introduction and overview
- Thursday 2nd June 2016 – Modelling methodology and calibration
- Tuesday 29th November 2016 - Mitigation options, planning scheme overlays, flood intelligence and warning

4.3 Community Consultation

All community meetings were supported by media releases to local papers and meeting notices advertising meetings well in advance. The following community meetings were held as part of the consultation process:

- Initial community meeting, Harrow Hermitage Hotel – 18th February 2016 – The first public meeting was held to outline the objectives of the study to the community, communicate what the community can expect from the study and gather input from the community on observed inundation and potential mitigation solutions;
- Second community meeting, Harrow Hermitage Hotel – 2nd June 2016 – The second community meeting presented calibration results for the September and December 2010 events and outlined a list of potential flood mitigation options identified to date. Community feedback was sought on the flood modelling results and their preference/suggestions for additional flood mitigation options; and
- Third community meeting, Harrow Hermitage Hotel – 19th December 2016 –The final public meeting presented planning scheme layers, mitigation modelling and project outcomes. Community feedback was sought on potential levee design, location and appearance.

4.4 Community Feedback

In general, the Harrow community was very pleased with the rigour and outcomes of the Harrow Flood Investigation. The community was generally not in favour of any general structural flood mitigation for buildings within the township aside from individual property protection measures which could be investigated by individual property owners.

There was interest in a levee protecting the John Mullagh Memorial Park to prevent repetitive inundation during minor floods. This is discussed in Section 8.

There was also numerous comments and discussion about environmental flows occurring during flood events, which was perceived to exacerbate flood levels. This is discussed further in Section 7.

4.5 DELWP Technical Review Panel Comments

During the Harrow Flood Investigation two reporting stages were submitted to a Technical Review Panel managed by the DELWP floodplain team. These reporting stages were:

- Hydrology Report (June 2016)
- Hydraulic Calibration Report (June 2016)

4.5.1 Hydrology Report Comments

Review of the Hydrology Report provided the following general summarised comments:

- *"The hydrology of flooding at Harrow is complex and Water Technology have developed a sophisticated approach to determining design flood events."*
- *"Overall the hydrologic analysis and modelling undertaken by Water Technology is of a suitable standard to provide guidance to the remainder of the project."*

There were also several specific issues that required further consideration, these issues were largely due to missing detail in the draft report or typos. These points are clarified in this report to improve reader understanding.

4.5.2 Hydraulic Calibration Report Comments

Review of the Hydraulics Report provided the following general summarised comments:

- *"A detailed combined 1D-2D hydraulic modelling approach was adopted for this study, within which a 1D hydraulic model replicated key waterways, drainage lines and hydraulic structures, a 2D hydraulic model was used for the broader floodplain, and a linked one and two dimensional hydraulic model was utilized to accurately model the interaction between in bank flows (1D) and overland floodplain flows (2D). The use of the TUFLOW modelling suite was specified by Reference 1 and was used for this study. This reviewer endorses this overall approach."*
- *"Overall, this reviewer considers that the model has been properly established." "...there is a lack of detail in how hydraulic structures are modelled, how Manning's n was adjusted to achieve a calibrated model, and the development of the downstream boundary condition."*
- *"This reviewer agrees with the calibration approach adopted. It is noted, however, that only finally determined Manning's n values are listed in the report. The values listed are completely reasonable, but this reviewer would like to see more detail on the calibration process itself. Calibration represents, in part, an opportunity to understand the key drivers in determining flood levels in different parts of the study area. While this reviewer is satisfied with the process, further detail would be a positive addition."*

- *"this reviewer notes that, under the circumstances, an excellent calibration has been achieved. Spot heights are generally well reproduced and flood event behaviour during the calibration event is generally consistent with the model results."*

There were also several specific issues that required further consideration, these issues were largely due to missing detail in the draft report or typos. These points are clarified in this report to improve reader understanding.

5. HYDROLOGY

5.1 Overview and Methodology

The primary aims of the hydrological analysis undertaken for this project included:

- Determine calibration events and flows to be used in the hydraulic model.
- Determine design event peak flow and hydrograph shape for input to the hydraulic model at the model boundaries. Design events included 0.2%, 0.5%, 1%, 2%, 5%, 10% and 20% AEP flood events, Probable Maximum Flood (PMF) and climate change scenarios.
- Test the impact of varying starting levels in Rocklands Reservoir on flows in the Glenelg River downstream of Rocklands.

To achieve these aims, the hydrological assessment was separated into two major components determining flows for the two major contributing catchment areas; downstream and upstream of the Fulham Bridge streamflow gauge. These contributing catchment areas were combined using a 1D model between Fulham Bridge and Harrow developed during the Glenelg Regional Flood Mapping Project[1]. A 1D model was used to route the flow from Fulham Bridge to Harrow rather than an inflow into the RORB model because routing along the Glenelg River reach in the RORB model can only be calibrated using the 'kc' value of a lag function. Whereas the 1D hydraulic model can be calibrated using Manning's 'n' and channel/floodplain geometry, resulting in more accurate routing.

- **Glenelg River tributary flows between Fulham Bridge and Harrow** – Inflows to the Glenelg River between Fulham Bridge and Harrow were determined using a RORB runoff routing model for both calibration and design. The inflows were then entered into the 1D model of the Glenelg River between Fulham Bridge and Harrow, combining with the routed Fulham Bridge flow.
- **Upstream of the Glenelg River at Fulham Bridge** –
  - **Calibration** - Calibration flows for the catchment area upstream of Fulham Bridge were directly extracted from the Fulham Bridge gauge record. They were then used as an inflow boundary to the 1D model between Fulham Bridge.
  - **Design** - Peak flows for the catchment area upstream of Fulham Bridge were determined via an annual series peak flow Flood Frequency Analysis (FFA) at the Fulham Bridge gauge, the hydrograph shape and volume were determined by a RORB model of the catchment upstream of Fulham Bridge developed during the Glenelg Regional Flood Mapping Project[1]. The volume of the RORB generated Fulham Bridge hydrograph was then confirmed by using a volume based FFA at the Fulham Bridge gauge based on a four-day event duration.

A schematic of how the flows were determined for each major catchment area is shown in Figure 5-1.

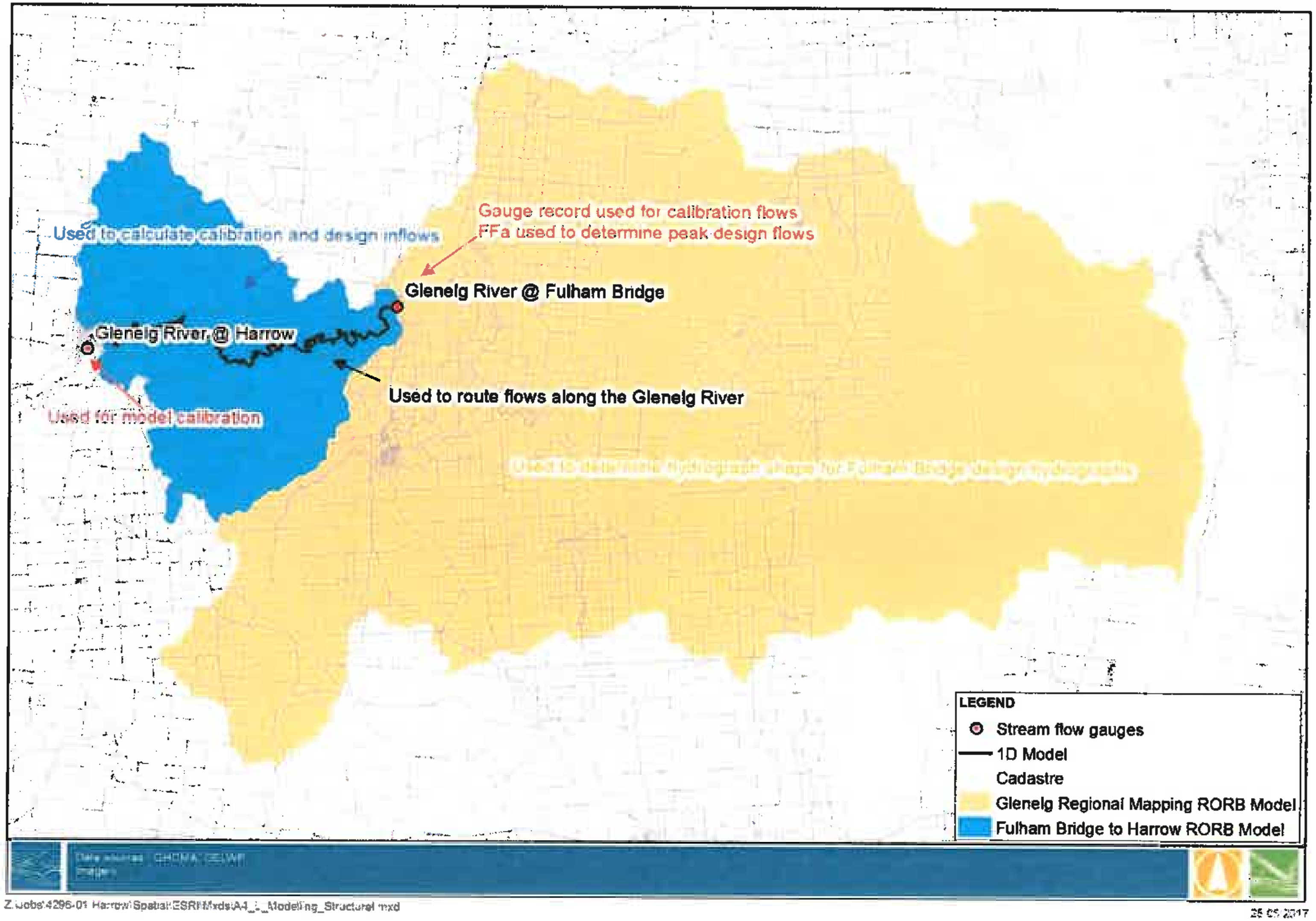


Figure 5-1 Modelling schematisation

5.2 Downstream of Fulham Bridge

5.2.1 Overview

A hydrologic model of the Glenelg River catchment was developed to determine the tributary flows between the Fulham Bridge gauge and Harrow. To generate inflows to the 1D hydraulic model between Fulham Bridge and Harrow or directly into 2D hydraulic model of Harrow in the case of Salt Creek. The rainfall-runoff program, RORB, was utilised.

RORB is a nonlinear rainfall runoff and streamflow routing model for calculation of flow hydrographs in drainage and stream networks. The model requires catchments to be divided into sub areas, connected by a series of conceptual reach storages. Observed or design storm rainfall is input to the centroid of each sub area. Specific losses are then deducted, and the excess routed through the reach network.

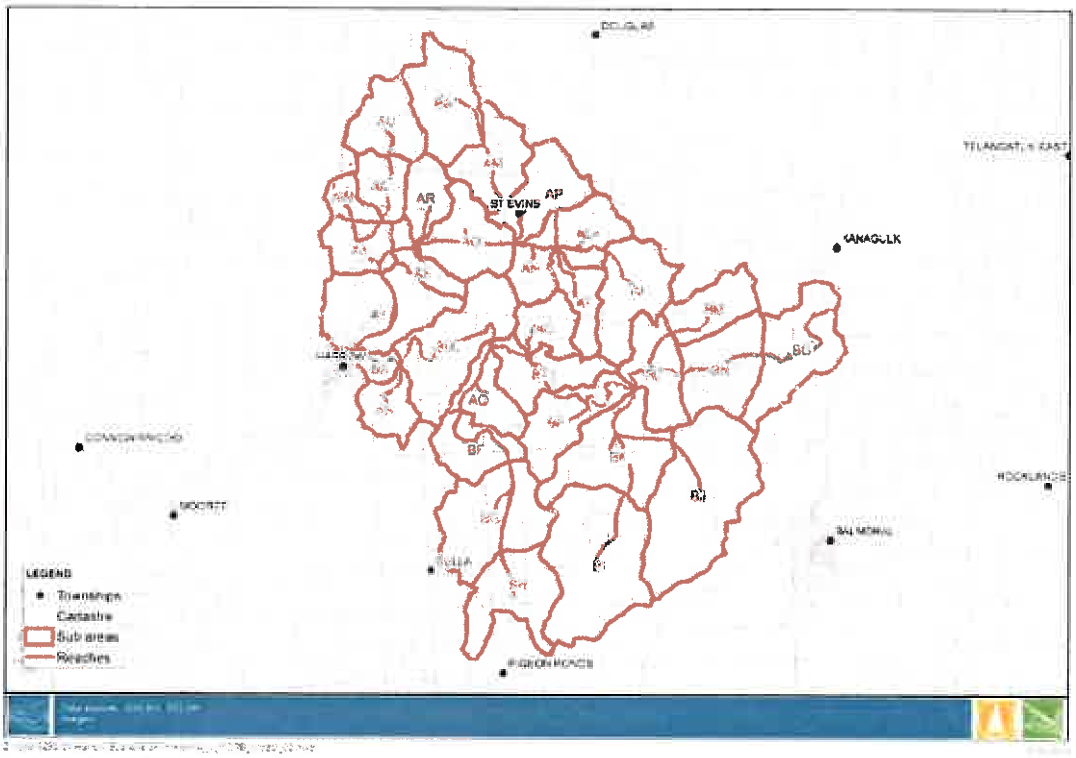


Figure 5-2 Revised RORB model structure – between Harrow and Fulham Bridge

The following methodology was applied for the RORB modelling:

- Glenelg River catchment upstream of Harrow was delineated
- The model catchment areas were divided based on the topography and required hydrograph print (result) locations.
- The RORB model was constructed using appropriately selected reach types, slopes and sub area fraction impervious values.
- Storm files for the chosen calibration events were constructed.
- RORB modelling was calibrated by modifying the RORB 'kc' and loss values with the 'kc' value compared to other regional estimates.

5.2.2 Model Structure

Sub-areas and Reaches

Sub-area boundaries and reaches were delineated using ArcHydro and revised as necessary to allow flows to be extracted at the points of interest. The RORB model was constructed using MiRORB (MapInfo RORB tools), RORB GUI and RORBWIN V6.15.

The sub areas and reaches were delineated from the 20 m VicMap Elevation Digital Terrain Model (DTM) of the area. Nodes were placed at areas of interest, the centroid of each sub-area and the junction of any two reaches. Nodes were then connected by RORB reaches, each representing the length, slope and reach type.

Reach types in the model were set to be consistent with the land use across the catchment. Five different reach types are available in RORB (1 = natural, 2= excavated & unlined, 3= lined channel or pipe, 4= drowned reach, 5= dummy reach). All reaches were set to natural, representative of the open grassed areas and natural waterways in the catchment.

Fraction Impervious

Fraction Impervious (FI) values were calculated using MiRORB. Default sub-area FI values were calculated based on the current Planning Scheme Zones (current July 2013), the fraction impervious values used for each zoning is shown in Table 5-1, with the zones mapped in

The area weighted average FI of the Glenelg River catchment was calculated to be 0.1, reflecting the predominantly rural/natural nature of the catchment. The spatial distribution of the weighted average FI for each sub-area is shown in Figure 5-4.

Table 5-1 RORB Model fraction impervious values and zones[11]

Zone	Description	Typical Fraction Impervious
FZ	Farming Zone	0.1
PCRZ	Protection of natural environment or resources.	0
PPRZ	Main zone for public open space, incl. golf courses.	0.1
PUZ1	Power lines, Pipe tracks and retarding basins	0.05
PUZ2	Schools and Universities	0.7
PUZ3	Hospitals	0.7
PUZ7	Museums	0.6
RDZ1	Major roads and freeways.	0.7
RLZ	Predominantly residential use in rural environment.	0.2
TZ	Small township with little zoning structure	0.55

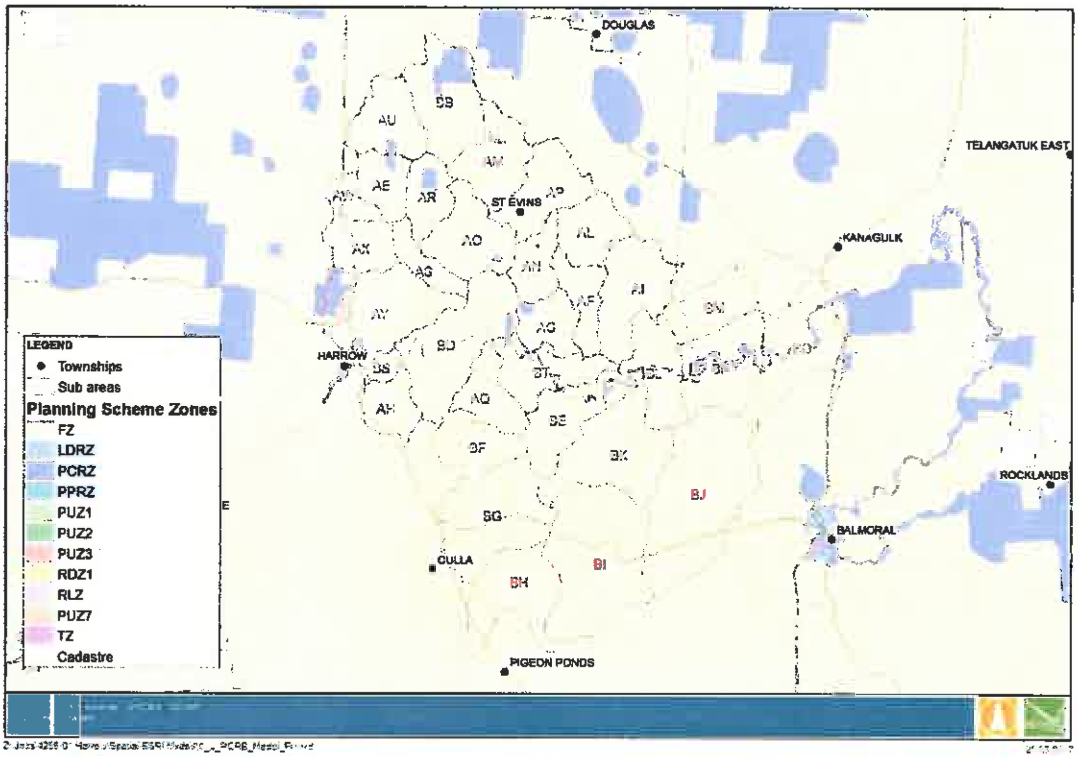


Figure 5-3 RORB model planning zones

[11] Melbourne Water, 2010 – Music Guidelines, Recommended input parameters and modelling approaches for MUSIC users

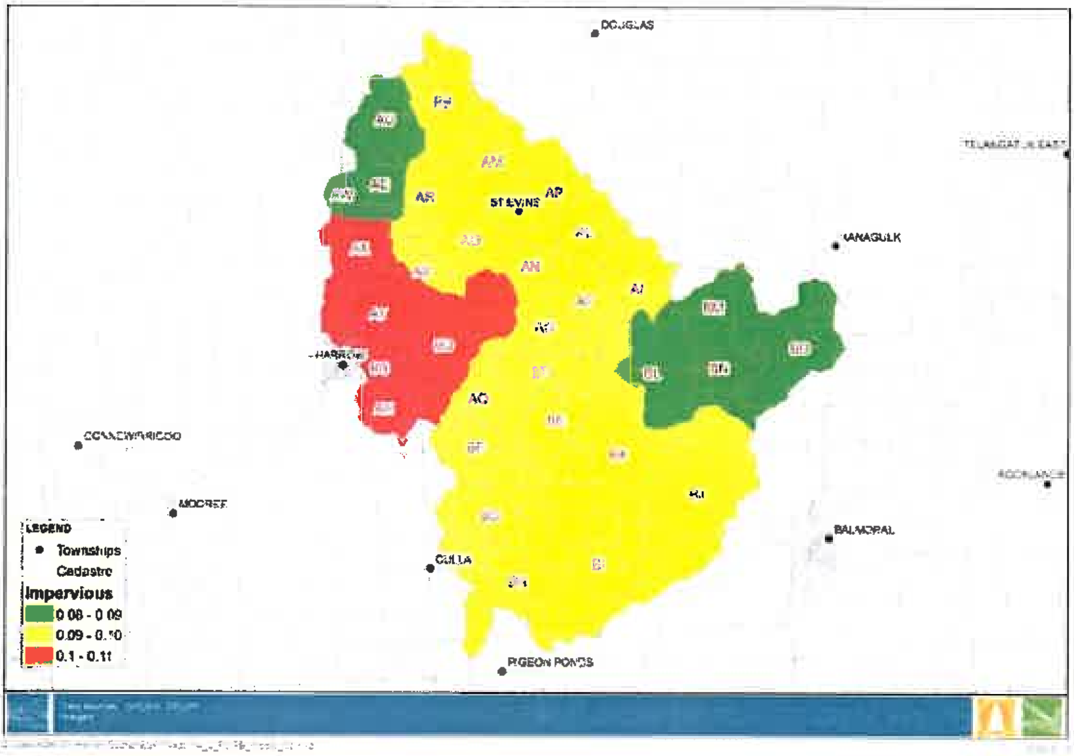


Figure 5-4 **RORB model fraction impervious calculated distribution – Fulham Bridge to Harrow**

5.3 Upstream of Fulham Bridge

5.3.1 Overview

As discussed in Section 3.3.1 there are four gauges on the Glenelg River upstream of Harrow. The flood investigation focussed on deriving accurate flood mapping for flood events ranging between 20% AEP to 0.2% AEP and the PMF. The Harrow gauge had an insufficient period of record to enable design flow estimation using Flood Frequency Analysis (FFA). In light of this, a FFA was undertaken for the Fulham Bridge gauge only.

When fitting a probability distribution in a FFA, small annual peaks with low flows that are not considered floods can skew the analysis. This is particularly the case in waterway systems with large dams on them like the Glenelg River. Low flow censoring was used to account to the effect of low flows on the analysis. Censoring was undertaken using the Multiple Grubbs Beck Test. Censoring of low flows is especially significant for gauges in the Glenelg River catchment due to the number of low flow years that are present in each gauge annual series.

The FFA for this project was undertaken in Flike[12] and multiple probability distributions were tested.

5.3.2 Peak Flow Analysis

The Fulham Bridge gauge record was comprised of instantaneous flow data for all years of the record, spanning from 1978 to 2015 including 37 annual peaks. The annual peak series contained one year with the flow extracted from an extrapolated rating curve recorded in 2010. All annual peaks were

[12] Flike - http://flike.tuflow.com/about/

considered of sufficient certainty for inclusion into the FFA. With censoring of low flow values, 15 low flows were removed from the analysis. The low flow threshold using the Multiple Grubbs Beck Test was 17.4 m^3/s.

The FFA was undertaken using a range of typical flood frequency distributions including Generalised Extreme Value (GEV), Log Normal and Log Pearson Type 3 (LP3). A LP3 distribution was found to be the best match for the dataset when considering the fit by eye produced by Flike.

Results for the Fulham Bridge gauge are shown in Table 5-2. The annual series, censored flows and FFA graph shown in Figure 5-5. Graphs of the other FFA distributions are shown in Appendix A.

Table 5-2 Glenelg River at Fulham Bridge Flood Frequency Analysis Peak Flow Estimation

AEP	Fulham Bridge FFA Results Peak Flow (m^3/s)		
	Raw annual series	Censored annual series (Adopted)	Censored annual series 5-95% Confidence Limits
20 %	75	74	57 - 103
10 %	107	106	85- 137
5 %	130	130	108 - 176
2 %	151	152	127 - 245
1 %	160	164	135 - 298
0.5 %	167	172	141 - 362
0.2 %	174	178	144 - 446

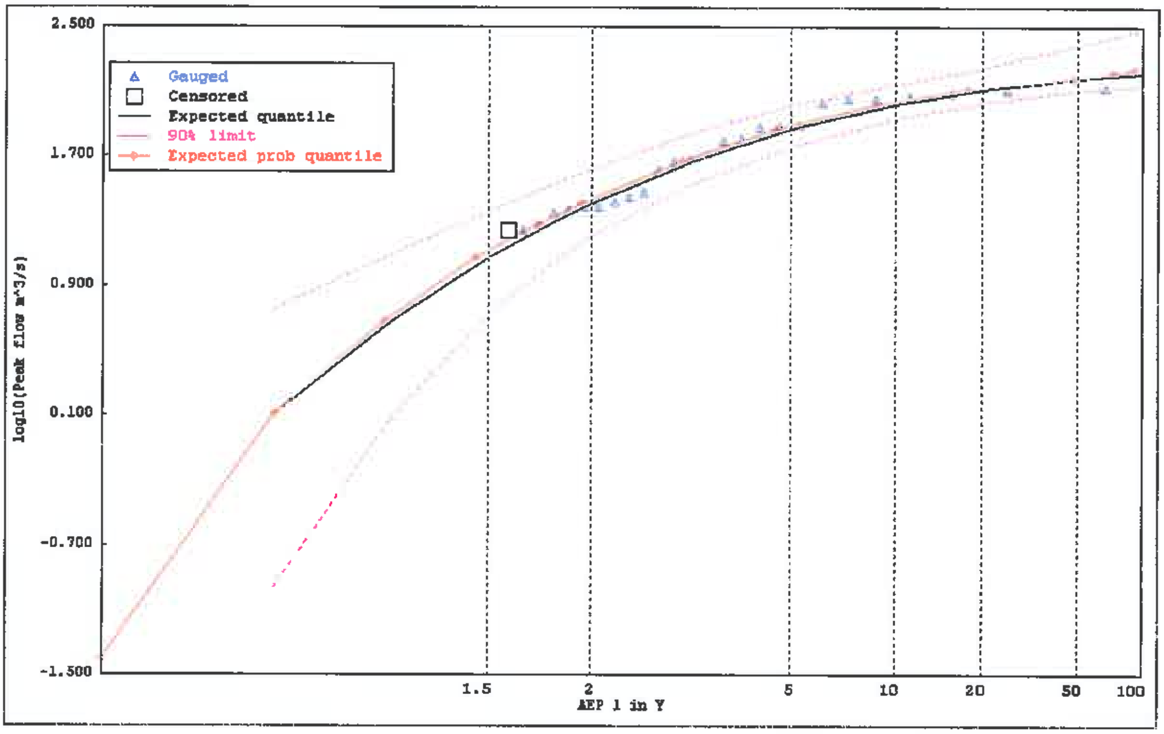


Figure 5-5 Glenelg River at Fulham Bridge Flood Frequency Plot

The estimated AEPs for the five highest flow events in the Fulham Bridge gauge record are shown below in Table 5-3.

Table 5-3 Fulham Bridge gauge observations and Flood Frequency Comparison

Year of Flood	Peak Flow (m^3/s)	AEP (%)	ARI (years)
December 2010	131.3	5	1 in 20
August 1991	127.7	6	1 in 17
October 1992	123.3	6.7	1 in 15
September 1983	115.9	9	1 in 11
September 1996	112.7	11	1 in 9

5.3.3 Design Hydrograph Shape

Overview

Design hydrograph shapes were determined from the RORB modelling of the upper Glenelg River completed during the Glenelg Regional Flood Mapping Project[1]. The RORB model shapes were scaled to match the peak flows determined by the FFA in this project, discussed in Section 5.3.2. This section provides a background to how the RORB design modelling was completed during the Glenelg Regional Flood Mapping Project[1] bearing in mind the RORB outputs were used for hydrograph shape only.

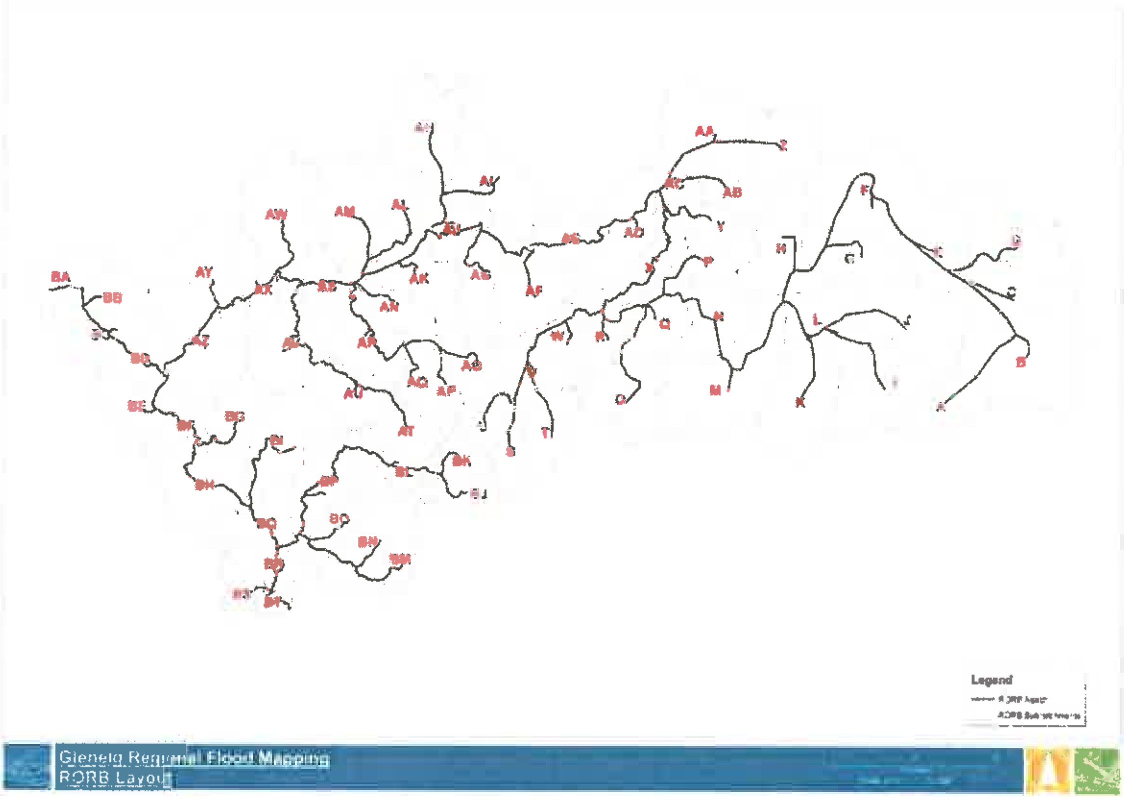


Figure 5-6 Glenelg Regional Flood Mapping RORB Model Structure[1]

Calibration

The RORB model was calibrated to the October 1975, September 1983 and December 2010 events. Calibration spatial patterns were developed using the daily rainfall record of surrounding gauges, with the temporal pattern developed using the Rocklands Reservoir sub daily gauge.

The modelled flow was compared to that observed at Fulham Bridge for the September 1983 and December 2010 events. Unfortunately, the Fulham Bridge gauge was not in operation during October

1975, however the model calibration was still undertaken for the lower Glenelg River gauge at Casterton.

Comparison of the observed and modelled hydrographs for September 1983 and December 2010 are shown in Figure 5-7 and Figure 5-8 respectively. A comparison of peak flow, volume and timing for each event is also shown in Table 5-4.

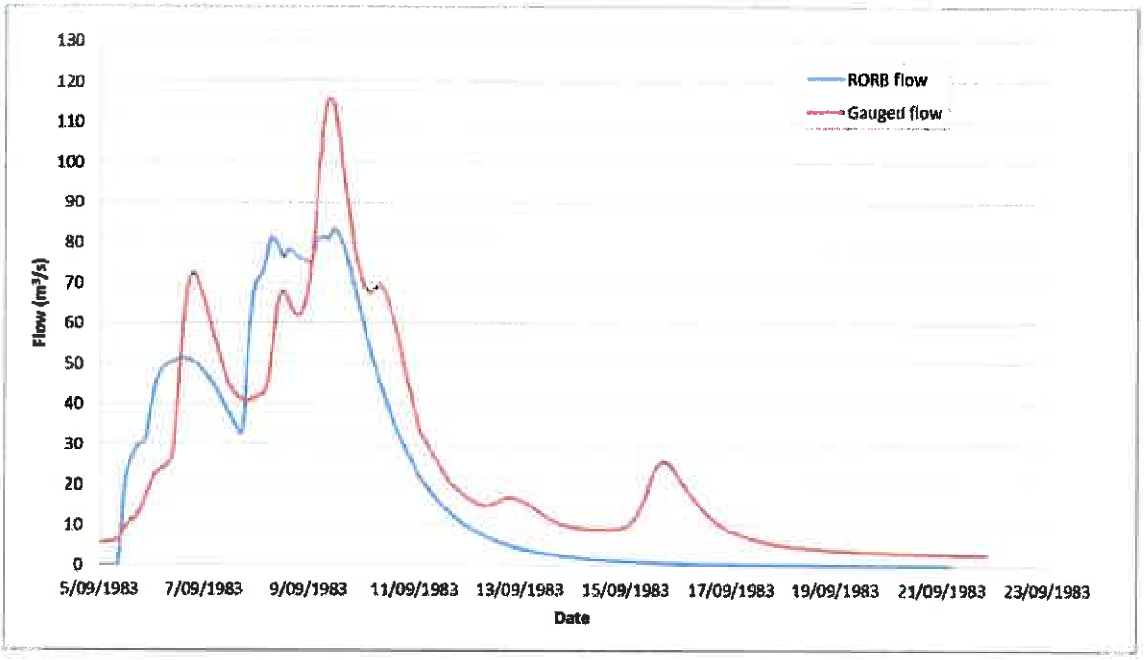


Figure 5-7 **Glenelg Regional Flood Mapping Project – September 1983 RORB model calibration[1]**

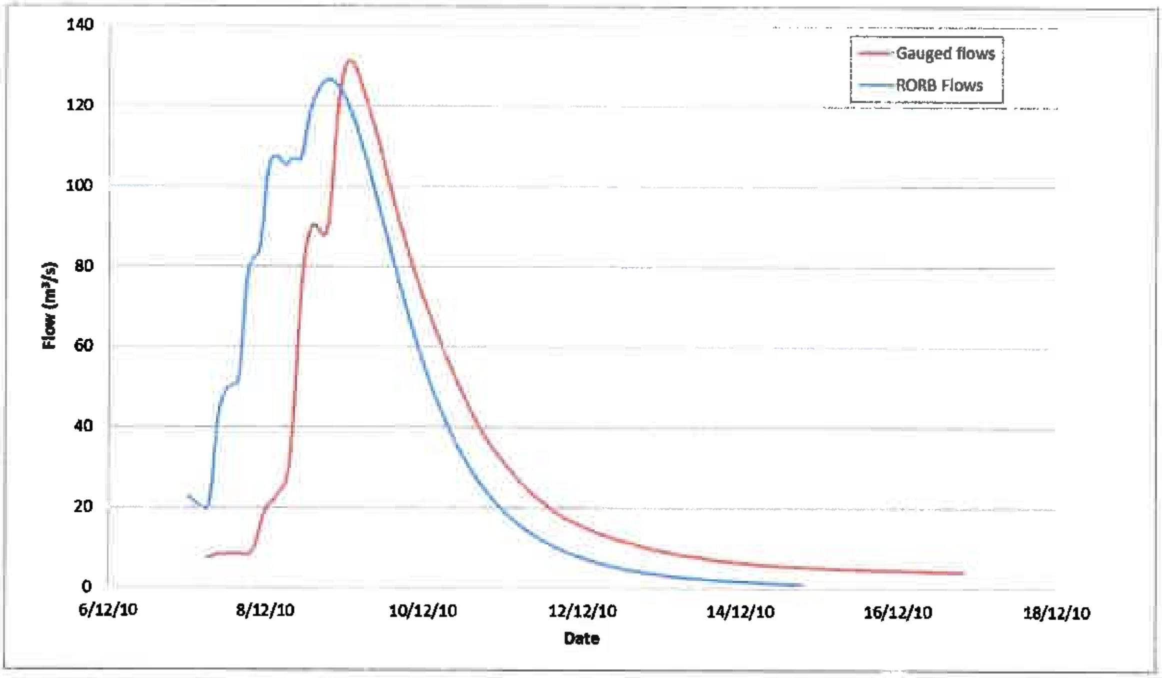


Figure 5-8 **Glenelg Regional Flood Mapping Project – December 2010 RORB model calibration[1]**

Table 5-4 September 1983 and December 2010 calibration summary at Fulham Bridge[1]

Streamflow Gauge	Peak discharge (m^3/s)	Peak timing	Event volume (ML)
		September 1983	
Gauged flow	116	9/09/1983 09:21	37,980
RORB Flow	83	9/09/1983 11:00	28,951
Difference	-33 (-28%)	1:39 hours	-9,029 (-24%)
		December 2010	
Gauged flow	131	8/12/2010 18:00	24,755
RORB Flow	127	9/12/2010 0:45	30,358
Difference	-4 (-3%)	6:45 hours	5,603 (23%)

The parameters adopted for the September 1983 and December 2010 events are shown in Table 5-5.

Table 5-5 Calibration parameters used during the Glenelg Regional Flood Mapping Project[1]

Calibration Parameter	September 1983	December 2010
kc	260	260
m	0.8	0.8
Initial Loss	10	20
Continuing Loss	0.9	3.5

The model calibration completed during the Glenelg Regional Flood Mapping Project[1] showed a reasonable match for peak flow and hydrograph shape for the December 2010 event. The September 1983 event however showed a less accurate fit. The modelled hydrograph is missing a distinct peak in hydrograph. The shape is generally well represented other than this sharp rise and fall. The missing peak may be associated with rainfall occurring in the catchment different to that recorded in the temporal pattern at Rocklands Reservoir. This difference was not considered significant to the outcomes of this study given the RORB model was used for hydrograph shape only.

Design

Design modelling completed during the Glenelg Regional Flood Mapping Project[1] was completed using a spatial pattern representing that observed during the September 1983 and October 1975. These are the two largest catchment wide events on record. A Zone 2 temporal pattern was adopted as it most closely represented the observed events. Further discussion on this is included in Section 5.5.1. A 'kc' value of 260 was adopted, the same as determined during the September and December 2010 events. An 'm' value of 0.8 was also adopted. The design 'kc' value was compared to other previous study and imperial estimates to confirm its applicability.

Table 5-6 shows a comparison between the Glenelg Regional Flood Mapping Project[1] adopted 'kc' value and 'm' value opposed to regional and other study 'kc' and 'm' values.

Table 5-6 Design model parameters

Source	m	kc
This study	0.8	260
Casterton Flood Investigation	0.96	115
Default RORB	0.8	151
Vic MAR<800 mm - Eq 3.22 ARR (BkV)	-	120
Victoria data (Pearse et al, 2002)	-	164
Aust. wide Dyer (1994) (Pearse et al 2002)	-	150
Aust. wide Yu (1989) (Pearse et al 2002)	-	126

Given the Glenelg Regional Flood Mapping Project[1] determined a 'kc' value much higher than previous studies or regional calculations was required. Further investigation as to why such a high 'kc' was required to calibrate the RORB model. The following discussion is a summarised excerpt from the Glenelg Regional Flood Mapping Project[1] report:

"The RORB manual offers a method for adjusting a 'kc' value should the m coefficient be changed. In the previous Cardno study a 'm' of 0.96 was used. The adjustment equation is provided below:

$$kc_{(new)} = kc_{(old)} \times (Qpeak/2)^{m1-m2'}$$ *(where 'm1' equals old 'm' and 'm2' equals new 'm')*

Using the adjustment equation and a peak flow of 302 m^3/s for the 1% AEP flow from flood frequency an adjusted 'kc' of 257 is determined. This is very close to that adopted in the study.

Several recent studies that used ArcHydro to delineate sub areas and reaches at a much finer resolution than determined in the past, has resulted in some catchments having very high 'kc' values in order to calibrated to observed streamflow.

The Water Technology Glenelg River RORB model included 8,600 km of reach length and 72 sub areas as compared to only 2,790 km of reach length and 25 sub areas in the Cardno RORB model. The Water Technology d_{av} was 131 compared to 118 in the Cardno RORB model.

Sensitivity testing of the 'kc' value was undertaken by comparing varying 'kc' values to the 1975 and 1983 gauge hydrographs at Casterton. Comparisons are shown in Figure 5-9 and Figure 5-10.

By modifying the 'kc' value to 200 the peak flow was considerably higher than the gauged flow in both the 1983 and 1975 events. The peak also occurred early, with hydrograph becoming peakier. This shows lowering the 'kc' value to a value more similar to calculated in the regional equations would not match either the peak flow rate or timing at Casterton. By modifying the 'kc' value to 300 the RORB model predicted peak flows lower and later than the gauge records".

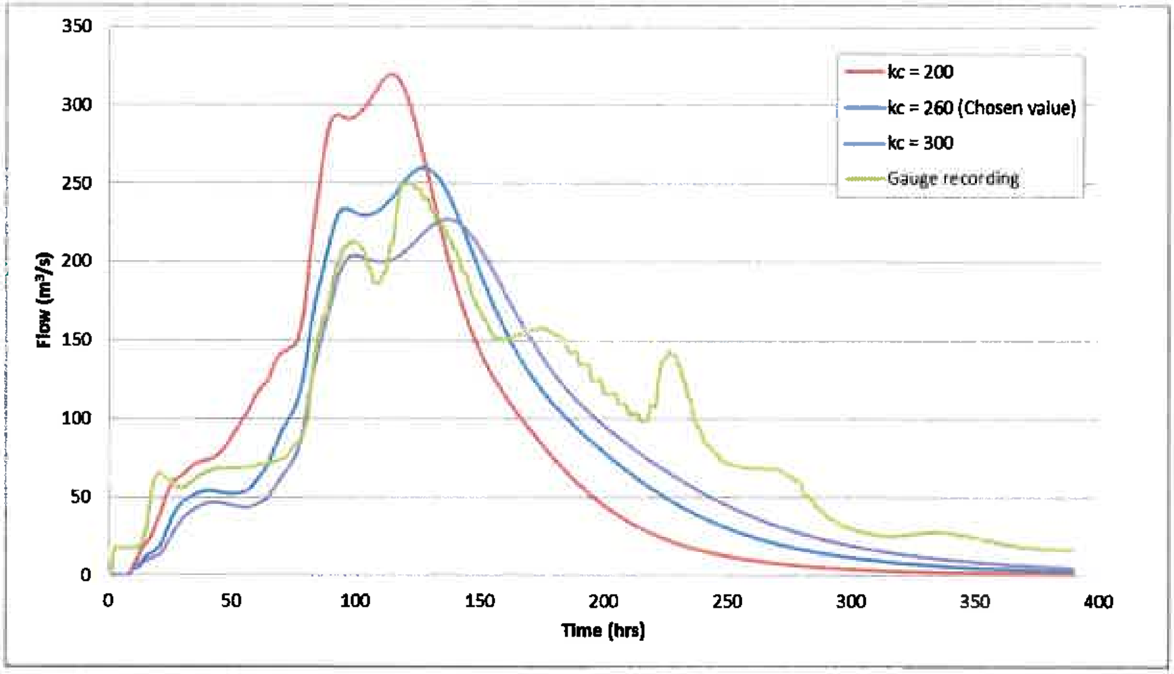


Figure 5-9 ***Gauged and modelled hydrographs for 'kc' values of 200, 260 and 300 for the 1983 event at Casterton***

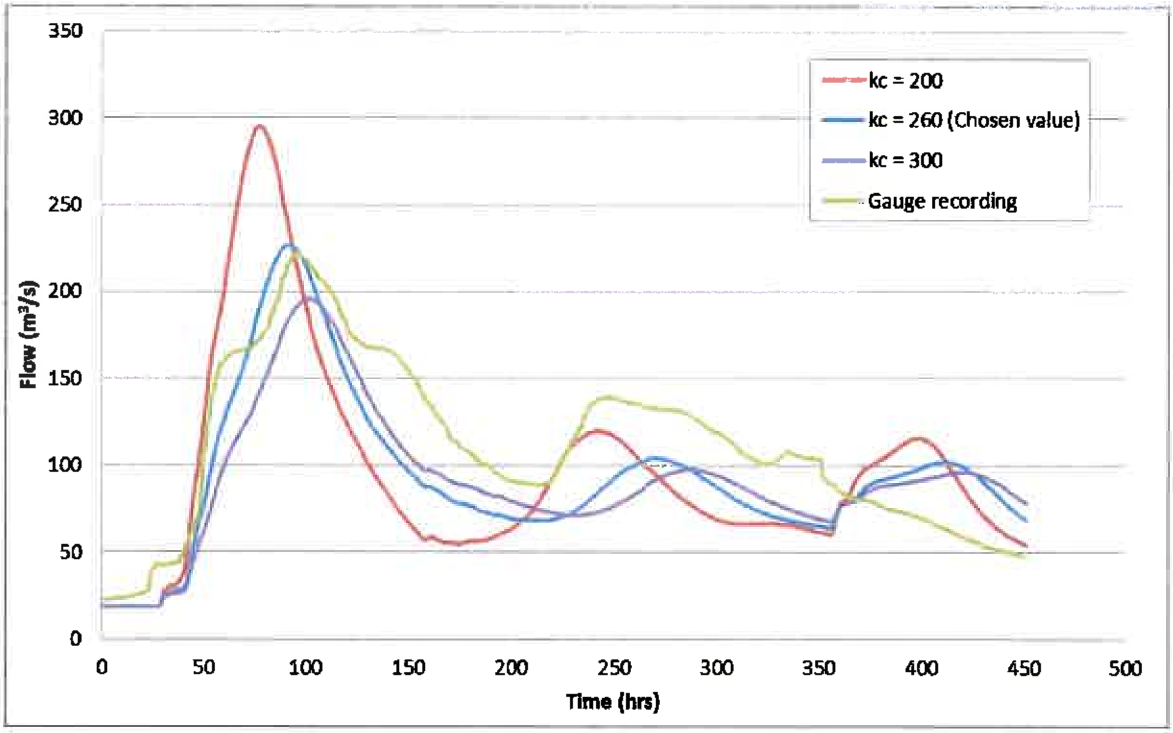


Figure 5-10 ***Gauged and modelled hydrographs for 'kc' values of 200, 260 and 300 for the 1983 event at Casterton***

Design modelling was completed varying the Initial and Continuing Losses with AEP. This was completed up and downstream of the Fulham Bridge gauge, matching the design flow peaks determined by FFA. The adopted losses up and downstream of Fulham Bridge are shown in Table 5-7. For the 20%-5% AEP events the same losses were adopted up and downstream of the Fulham Bridge gauge.

Table 5-7 Design losses adopted during the Glenelg Regional Flood Mapping Project.

Event AEP	Initial loss (mm)	Continuing loss (mm)	
		US Fulham Bridge	DS Fulham Bridge
20%	20	1 (both up and downstream of Fulham Bridge)	
10%	20	1.3 (both up and downstream of Fulham Bridge)	
5%	20	1.7 (both up and downstream of Fulham Bridge)	
2%	20	2.5	2.5
1%	25	3.0	2.9
0.5%	25	4.2	4

The adopted losses were compared to recommended and previously adopted loss values, as shown in Table 5-8. The adopted losses values were within the range of the design loss parameters as set out within AR&R 1987[20].

Table 5-8 Recommended and previously adopted design Losses

Source		Initial loss (mm)	Continuing loss (mm)
Casterton Flood Investigation (2011)[13]		20	2
Skipton Flood Investigation (2011)[14]		15.2	2.8
Halls Gap Flood Study (2008)[15]		20	2
Port Fairy Regional Flood Study (2008)[16]		15	1.3-1.85 (varying with duration)
South Warrnambool Flood Study (2007)[17]		20	1.7-3.9 (varying with AEP)
AR&R (1987)[20]	Cordery & Pilgrim (1983)[18]		2.5
	Melbourne and Metropolitan Board of Works[19]	15-20	
	Rural Water Commission[19]	25-35	

To give an indication of how the RORB model results were scaled, the Glenelg Regional Flood Mapping Project[1] 1% AEP hydrograph was compared to the adopted 1% AEP hydrograph, as shown in Figure 5-12. All AEP peak flows are compared in Table 5-9.

[13] Cardno (2011), Casterton Flood Investigation, Commissioned by Glenelg Hopkins CMA

[14] Skipton Flood Investigation (2011), Water Technology, Commissioned by Glenelg Hopkins CMA

[15] Halls Gap Flood Investigation, (2008), Water Technology, Commissioned by Wimmera CMA

[16] Water Technology (2008), Port Fairy Regional Flood Study, Commissioned by Glenelg Hopkins CMA

[17] Water Technology (2007), South Warrnambool Flood Study, Commissioned by Glenelg Hopkins CMA

[18] Cordery, I., & Pilgrim, D.H. (1983), On the lack of dependence of losses from flood runoff on soil and cover characteristics

[19] Government organisations listed as data sources in Australian Rainfall and Runoff - Volume 1, Book II Section 3

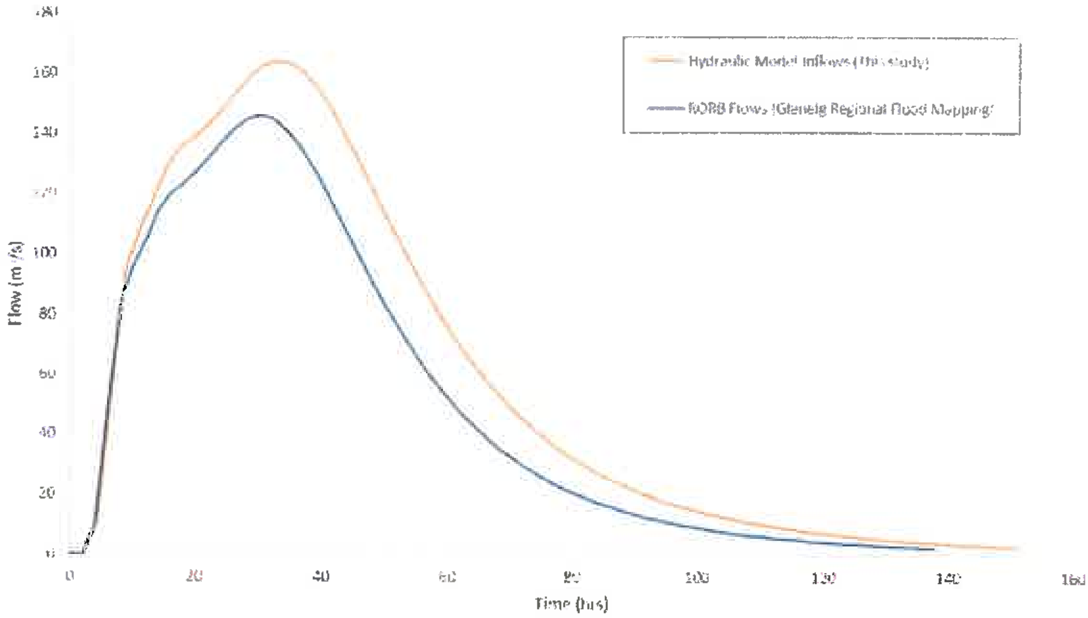


Figure 5-11 **Glenelg River at Fulham Bridge 1% AEP design flow hydrographs determined during the Glenelg Regional Flood Mapping Project and this project**

Table 5-9 **Glenelg Regional Flood Mapping Project[1] peak flows compared to this project's peak flows**

AEP (%)	RORB Peak Flow (Glenelg Regional Flood Mapping Project) (m^3/s)	This project (m^3/s)	Comparison
20	77	74	-3 (-4%)
10	103	106	3 (2%)
5	124	130	6 (5%)
2	139	152	13 (8%)
1	146	164	18 (11%)
0.5	150	172	22 (13%)
0.2	-	178	-

5.3.4 Design Hydrographs

The Fulham Bridge inflow hydrographs are shown in Figure 5-12.

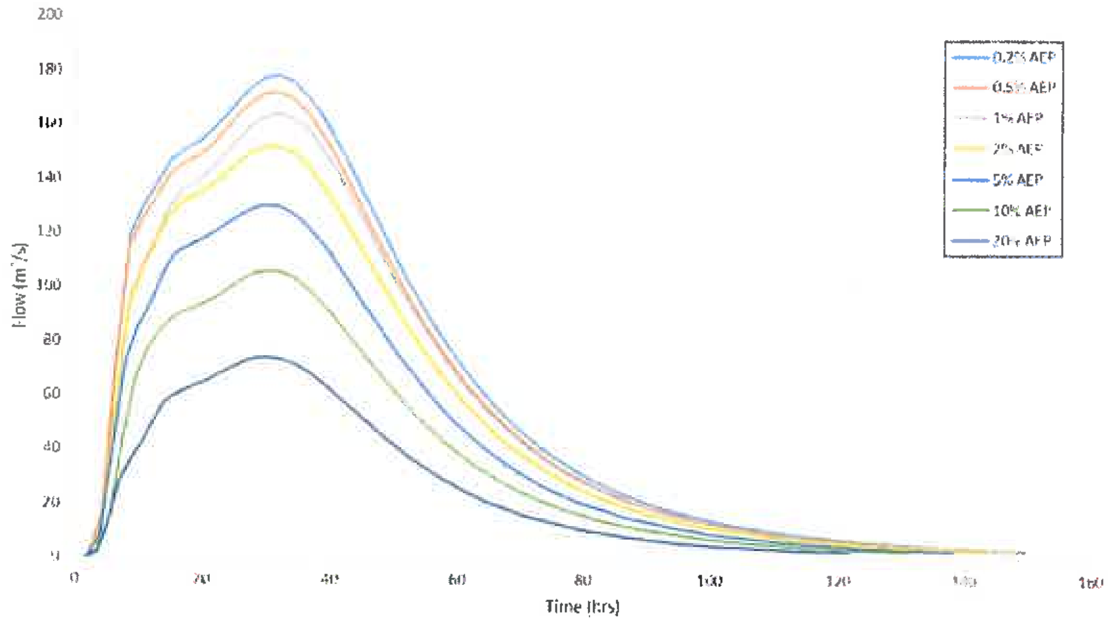


Figure 5-12 Glenelg River at Fulham Bridge Design flow hydrographs

To confirm the volume of the scaled hydrographs was suitable, a FFA on four-day volume was undertaken. Four days was determined as the typical hydrograph duration at Fulham Bridge based on previous high flow events.

The four day volumes determined by the FFA and RORB model for each AEP are shown in Table 5-10, along with the with the four-day volume. The RORB four day volume hydrographs were scaled to match the FFA determined volumes exactly.

Table 5-10 Fulham Bridge FFA peak flows, FFA 4 day volumes and RORB hydrograph volumes

AEP	FFA four-day volume (ML)	Scaled RORB hydrograph volume (ML)	Difference in volume (ML) (%)
20%	12,626	14,528	1,902 (13%)
10%	18,694	19,882	1,188 (6%)
5%	23,522	23,852	330 (1%)
2%	28,073	27,311	-762 (-3%)
1%	30,411	28,931	-1,480 (-5%)
0.5%	32,057	30,256	-1,801 (-6%)
0.2%	33,489	31,312	-2,177 (-7%)

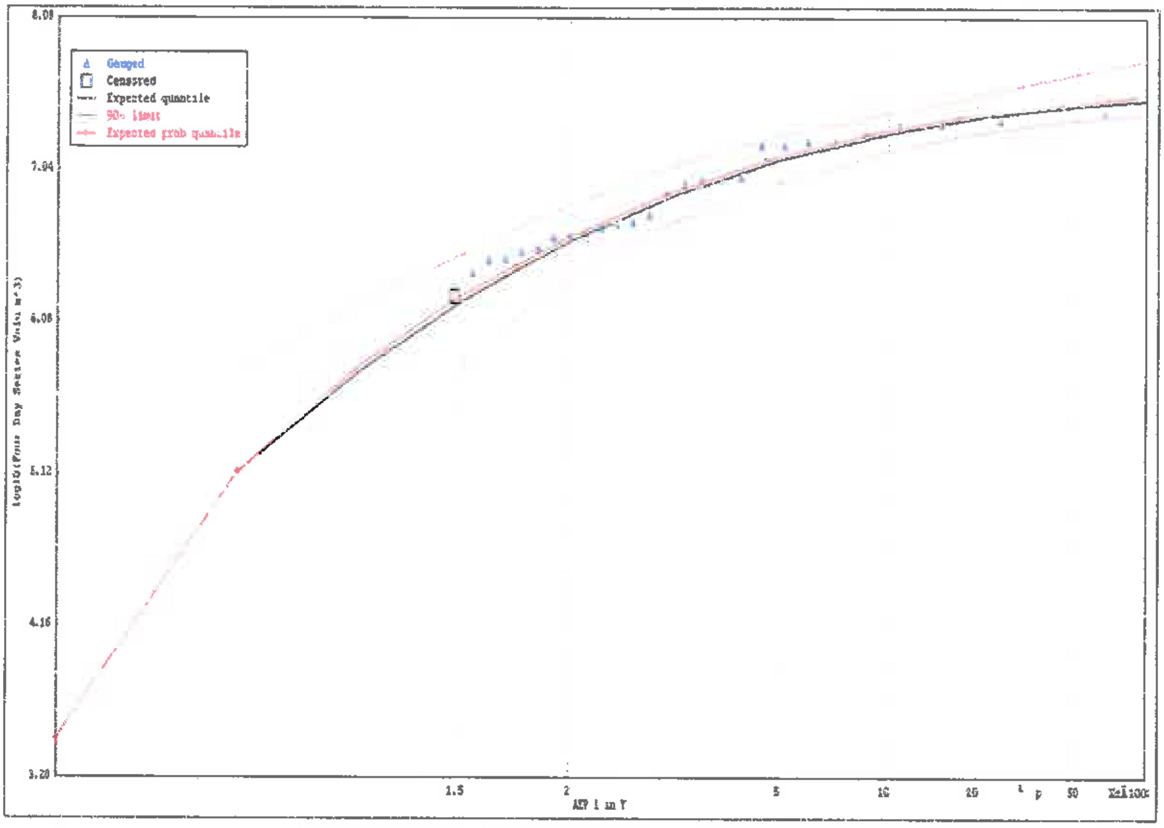


Figure 5-13 **Glenelg River at Fulham Bridge four-day volume FFA**

5.4 Model Calibration utilising the Glenelg River 1D model

5.4.1 Overview

The RORB model was calibrated by creating a spatial distribution map of recorded daily rainfall depths across the catchment area between Fulham Bridge and Harrow. The temporal pattern from the Rocklands sub daily rainfall gauge was used.

The RORB model flows were compared to the Harrow streamflow gauge. Glenelg River tributary inflows were extracted from the RORB model and added to the 1D model spanning from Fulham Bridge to Harrow, along with the gauged flow at Fulham Bridge. Flows in the 1D model were then compared to the gauge record at Harrow.

5.4.2 Calibration Parameters

Overview

There are several model parameters used in RORB that control the resulting peak flow rate and volume of runoff. These values are 'kc', 'm', initial and continuing losses. These parameters can be adjusted to fit the model to observed information.

Losses

The loss model chosen for the Glenelg River catchment was an initial and continuing loss model. This model was chosen because it is a predominantly rural/forested catchment. The catchment is likely to have high rainfall losses at the beginning of an event when the ground is dry, which will then reduce to a smaller loss rate over the remainder of the event.

As part of the calibration process several initial and continuing loss values were trialled for each calibration event, and the RORB model results were compared with gauge records at Harrow. These loss values are discussed in respect to each event below.

m

The RORB 'm' value is typically set at 0.8. This value remains unchanged and is an acceptable value for the degree of non-linearity of catchment response (Australian Rainfall and Runoff, 1987)[20]. There are alternate methods for determining m, such as Weeks (1980),[21] which uses multiple calibration events to select 'kc' and m. However, if retaining a value of 0.8 is possible it is best left unchanged.

kc

The RORB model 'kc' value was estimated using a range of prediction equations as shown below in Table 5-11. These equations use either catchment area or D_{av} (the average flow distance in the channel network of sub area inflows) and have been developed using different data sets (or subsets of the same data set). The parameter selected for design is based on consistency of prediction and resulting flows.

Based on the regional prediction equations, several 'kc' values were initially trialled, with calibration to the gauge records used to refine the 'kc' value for each of the selected calibration events.

Table 5-11 Various 'kc' calculated values

Method	Equation	Predicted kc
Default RORB	$kc = 2.2*A^{0.5}$	46.7
Vic MAR<800 mm - Eq 3.22 ARR (BkV)[20]	$kc=0.49*A^{0.65}$	26.01
Victoria data (Pearse et al, 2002)[22]	$kc=1.25*D_{av}$	29.07
Aust wide Dyer (1994) (Pearse et al 2002)[22]	$kc=1.14*D_{av}$	26.52
Aust wide Yu (1989) (Pearse et al 2002) [20]	$kc=0.96*D_{av}$	22.33

Manning's 'n'

The 1D model was calibrated by varying a uniform Manning's 'n' roughness value. In a 1D model Manning's 'n' is a representation of numerous components of the resistance to flow including:

- Riparian vegetation;
- Waterway sinuosity; and,
- Deep pools and riffles

The most appropriate roughness value was selected by matching peak flow and timing between the Fulham Bridge and Harrow gauging stations.

[20] AR&R, 1987 – Australian Rainfall and Runoff

[21] Weeks, W. D. (1980). Using the Laurenson model: traps for young players. Hydrology and Water Resources Symposium, Adelaide, Institution of Engineers Australia

[22] Pearse et al, 2002 – A Simple Method for Estimating RORB Model Parameters for Ungauged Rural Catchments, Water Challenge: Balancing the Risks: Hydrology and Water Resources Symposium, 2002

5.4.3 Event Calibration

Event Selection

The RORB model was calibrated using observed events in the Glenelg River focusing on the events available for both Glenelg River gauges at Fulham Bridge and Harrow. During the initial stages of the streamflow data review several large events were highlighted as potential calibration events. As discussed in Section 3.3.1, only events post construction of Rocklands Reservoir in 1953 were used. The events used in the calibration of the RORB model were September 2010, December 2010 and January 2011. These events were most recent and therefore represented the most current catchment conditions. There was also the largest amount of calibration information available for these events, with the Harrow streamflow gauge recording all three. Surveyed flood levels were also available for both 2010 events for the hydraulic model calibration. The December and September 2010 events have an estimated AEPs of 5% and less than 20% respectively.

September 2010

The September 2010 event was relatively minor in the upper Glenelg River with an AEP of approximately 20% at the Fulham Bridge gauge. The event began on the 4th with relatively small daily totals recorded on the 5th and 6th. The average total rainfall depth across the sub areas was 39.5 mm. The spatial pattern of the December 2010 event showing the total depth of rainfall for each sub area is shown in Figure 5-14.

The recorded rainfall resulted in moderate streamflow in the Glenelg River with the Fulham Bridge gauge recording a peak flow of 66 m^3/s and the Harrow streamflow gauge recording a double peak hydrograph with 47 m^3/s recorded in the initial peak generated by tributary flow in the morning of the 5th of September, and a second peak recording 54 m^3/s generated by the Glenelg River catchment upstream of Fulham Bridge in the morning of the 7th of September. The recorded hydrographs at Fulham Bridge and Harrow are shown in Figure 5-16. The recorded travel time between peaks from Fulham Bridge to Harrow was 28 hrs. This event clearly shows that the Glenelg River at Harrow may begin to rise well prior to the flood peak reaching Fulham Bridge gauge upstream. The tributary inflows downstream of Fulham Bridge, most notably Salt Creek, can contribute significant flows leading to rises in the river prior to the Fulham Bridge gauge rising. The time between the intense rainfall beginning and a rise in the Harrow gauge during the September 2010 event was approximately 36 hours.

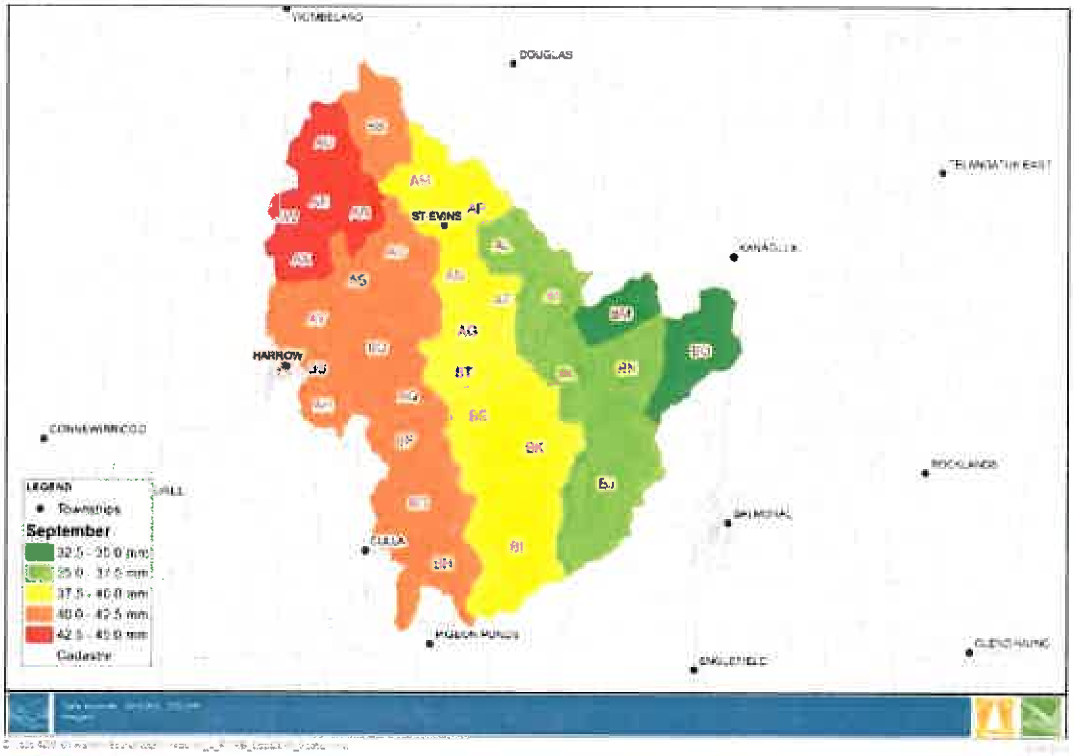


Figure 5-14 **September 2010 - Rainfall spatial pattern**

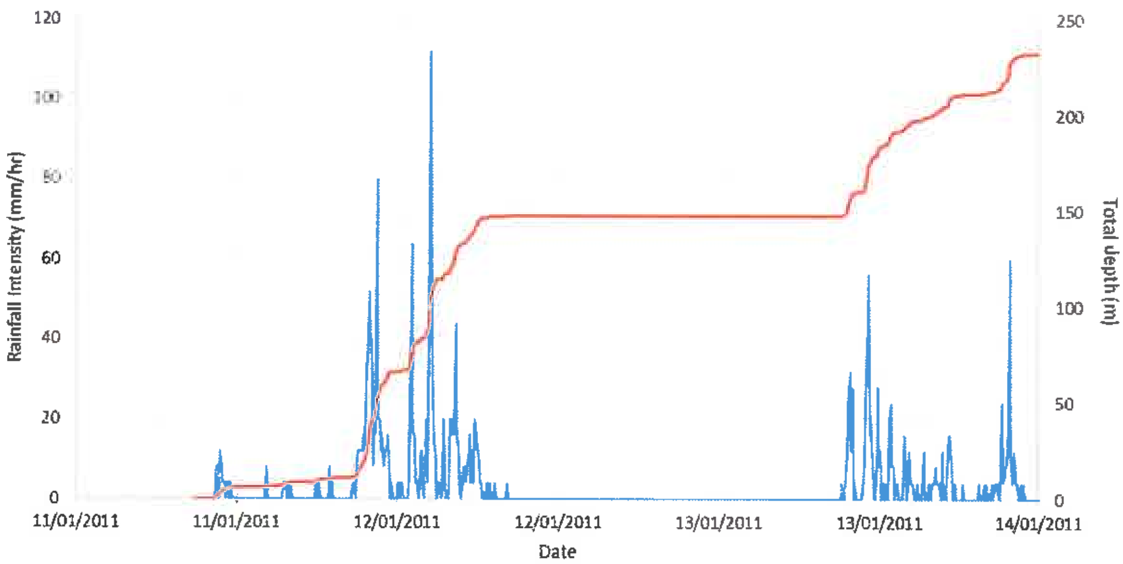


Figure 5-15 **September 2010 – Rocklands rainfall temporal pattern**

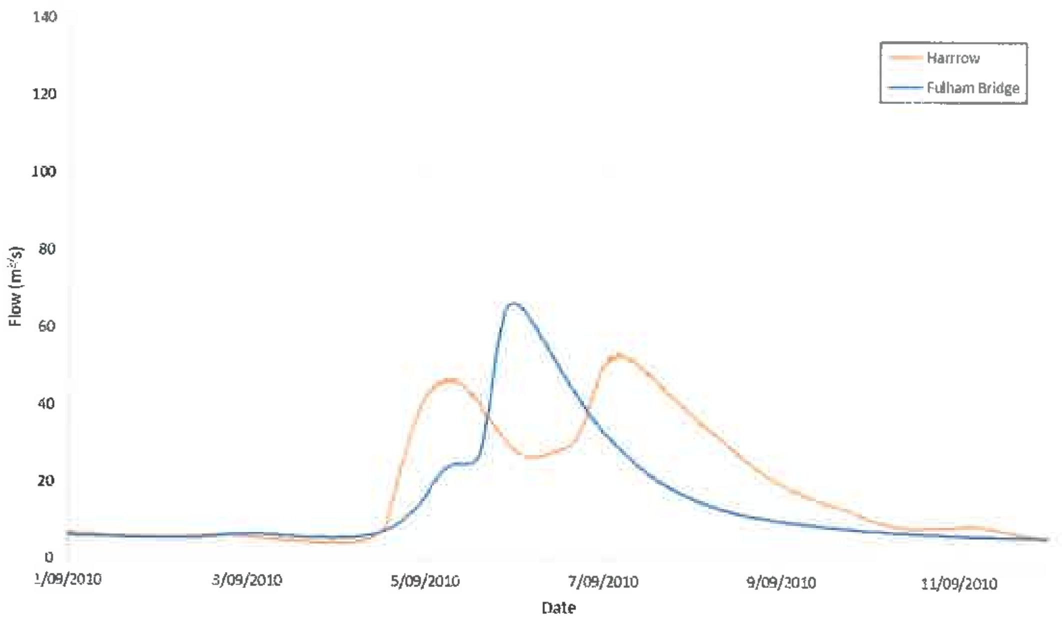


Figure 5-16 September 2010 – Fulham Bridge and Harrow recorded hydrographs

The RORB model was run using the recorded rainfall information, modelling was initially completed using a 'kc' value of 29, as estimated by the Pearce[22] equation and a preliminary estimate of an initial and continuing loss. The outflow hydrographs were then input into the Glenelg River 1D hydraulic model with the recorded Fulham Bridge hydrograph. The hydraulic model predicted flows at the Harrow streamflow gauge for comparison to the gauged flows.

'kc' and loss values were modelled iteratively varying each individually to test the impact on the modelled hydrograph by comparing to that recorded at the Harrow streamflow gauge. Of the numerous combinations of 'kc', initial loss and continuing loss, a 'kc' of 40, initial loss of 15 mm and continuing loss of 2.5 mm/hr showed the best match between modelled and observed hydrographs.

The 1D hydraulic model showed the best results with a Manning's 'n' roughness of 0.12, this is representative of very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush[36].

The model results are shown in terms of peak flow and timing in Table 5-12 and graphically in Figure 5-17.

Table 5-12 September 2010 – Model calibration peak flow and timing

	Observed	Modelled	Difference
Peak flow (first peak)	46.5 m^3/s	48.0 m^3/s	1.5 m^3/s (3.2%)
Timing (first peak)	05/09/1010 7:00 am	04/09/2010 7:00 pm	12 hrs
Peak flow (second peak)	54.1 m^3/s	59.9 m^3/s	5.8 m3/s (10.7%)
Timing (second peak)	07/09/2010 3:45 am	07/09/2010 1:00 am	2 hrs 45 mins

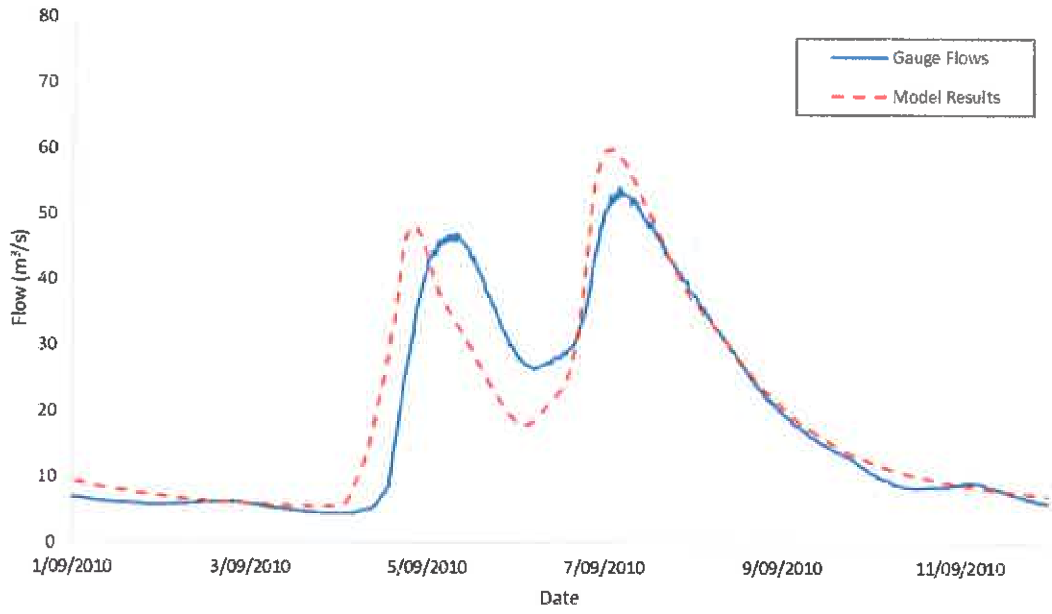


Figure 5-17 September 2010 – Harrow modelled and recorded hydrographs

With the pluviograph at Rocklands Reservoir providing the timing of the rainfall, it is likely that there would be a slight offset in the timing of modelled compared to observed flows. It is possible that a higher initial loss could be applied to delay the start of the rising limb generated from tributary flows.

The September 2010 event was relatively minor in regards to impacts at Harrow, however it provides an understating of flooding at the lower end of the design modelling completed during this project (20% AEP). The initial and continuing loss values determined for the event are a relatively high proportion of the total rainfall depth with an average sub area depth of less than 40 mm, an initial loss of 15 mm leaves only 25 mm of excess rainfall which is then further reduced by a continuing loss of 2.5 mm/hr.

December 2010

The December 2010 event was relatively isolated with the majority of the rainfall occurring in the Glenelg River catchment upstream of Fulham Bridge. The rainfall occurred from the 5th to the 9th of December. The average total rainfall depth across all RORB sub areas was 115.0 mm. The December 2010 spatial pattern showing the total rainfall for each sub area is shown in Figure 5-18.

The Rocklands sub daily record shows three discrete bursts of rainfall separated by periods of little to no rain. The first burst totalled 40.0 mm over 90 hours reaching a maximum intensity of just under 40 mm/hr, the second burst totalled 23.8 mm over 3.5 hours with a maximum intensity of 48 mm/hour, the third burst totalled 31.6 mm over a longer 19 hours with the highest intensity of 54 mm/hr. Given the duration of the event, and the timing of rises in the streamflow gauges, only the second and third bursts were modelled. The first burst contributes to the antecedent conditions and the selection of the loss parameters adopted.

The temporal pattern of the December 2010 event recorded at Rocklands Reservoir is shown in Figure 5-19.

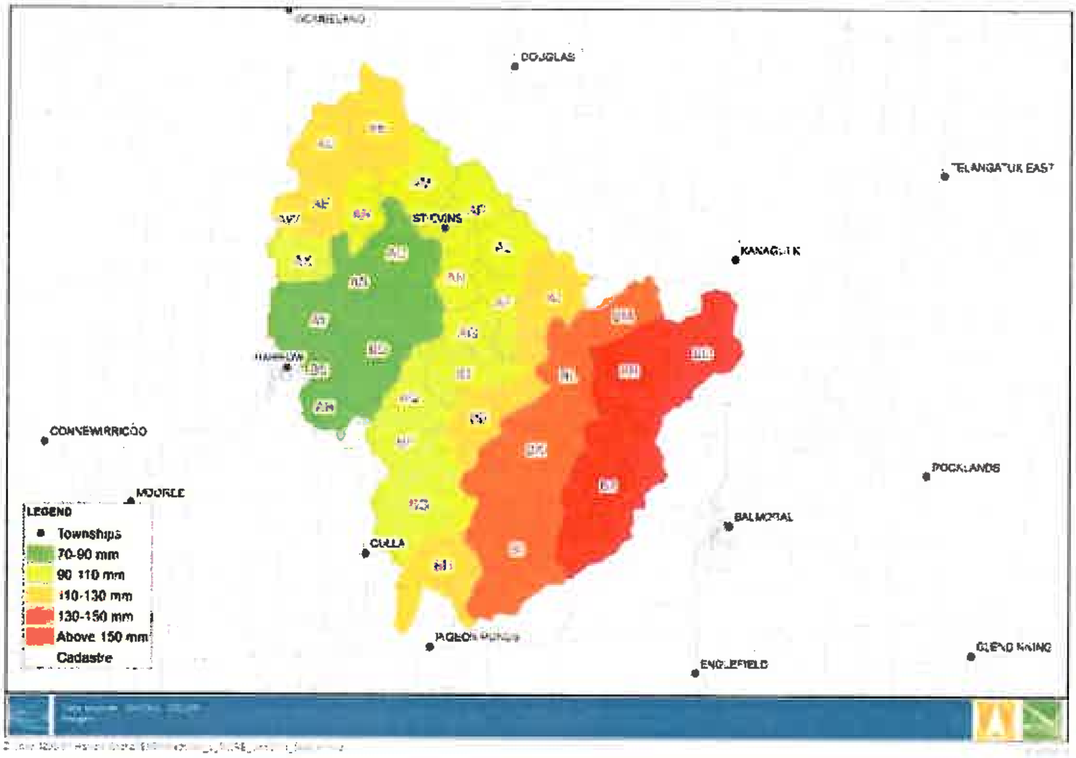


Figure 5-18 December 2010 -Rainfall spatial pattern

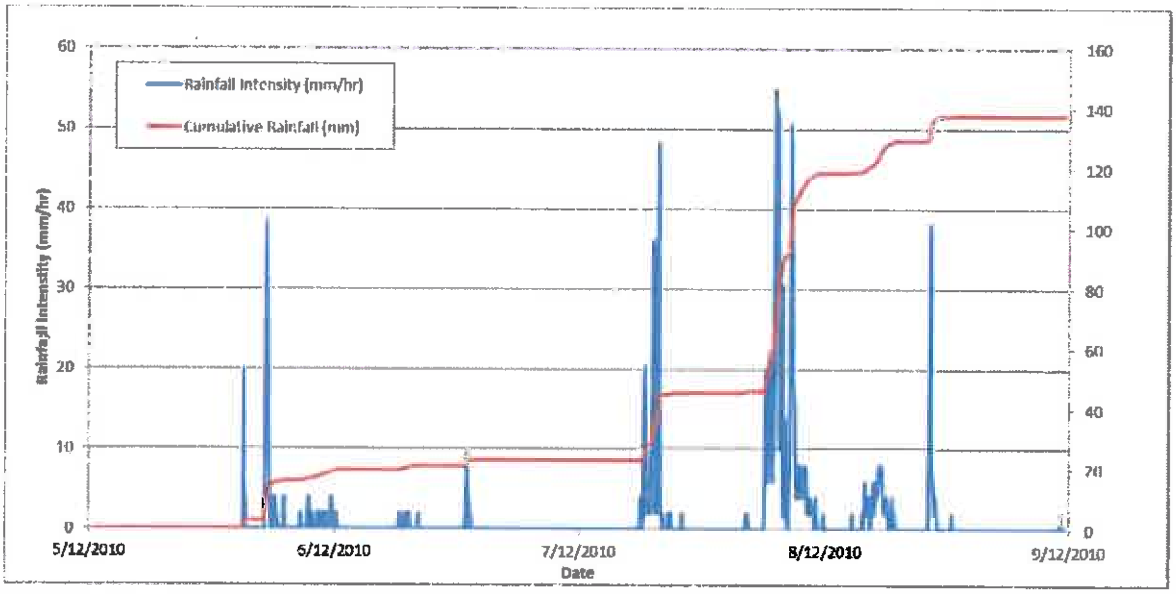


Figure 5-19 December 2010- Rainfall temporal Pattern

The Fulham Bridge gauge recorded a peak flow of 131 m^3/s recorded at 12am, 9th December 2010. Both the Fulham Bridge and the Harrow streamflow gauges began to rise just under 24 hours after the second burst of rainfall began in the morning of the 7th December. As per the FFA this is estimated to be around a 5% AEP event.

Similar to the September 2010 event, the December 2010 hydrographs recorded at the Fulham Bridge and Harrow showed an initial peak with localised catchment runoff generating an initial rise in Glenelg River flows which subsided slightly before the broader catchment area contributed runoff causing the peak flood flows. During December 2010 the recorded travel time between peaks at Fulham Bridge and Harrow was around 20 hrs.

The recorded streamflow hydrographs at Fulham Bridge and Harrow are shown in Figure 5-20.

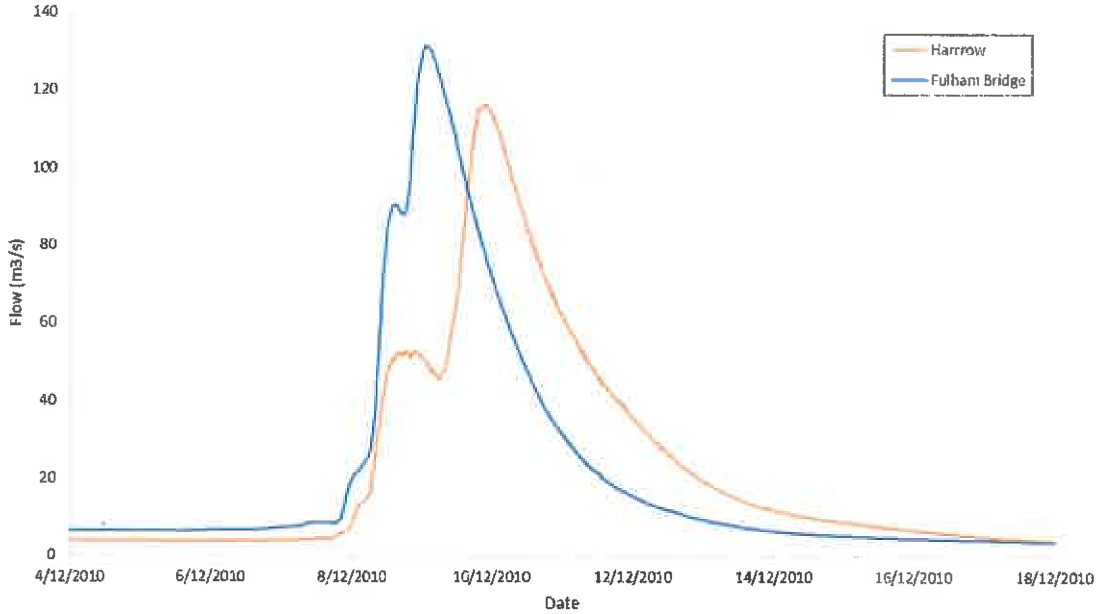


Figure 5-20 December 2010 recorded hydrographs at Fulham Bridge and Harrow

Significant inundation was observed in the Harrow township with several buildings flooded below floor. There were seven peak flood heights surveyed of the December 2010 event in Harrow, these points were used for the hydraulic model calibration. An aerial photo capturing the inundation in Harrow during December 2010 is shown in Figure 5-20.

Figure 5-21 Inundation in Harrow observed during December 2010 (source: Warrnambool Standard)

Similar to the September 2010 event the RORB model was run using the recorded rainfall information, modelling was completed starting with the 'kc' value of 40 and losses determined during the September 2010 calibration. Each parameter was then modified iteratively until the best match was determined.

Similarly, the 1D hydraulic model roughness was started at a Manning's 'n' of 0.12, this was determined as the best match of the December 2010 event as well.

Of the numerous combinations of 'kc', initial loss and continuing loss a 'kc' of 40, initial loss of 50 mm and a continuing loss of 6 mm/hr. The RORB model calibration results are shown in terms of peak flow and timing in Table 5-13 and graphically in Figure 5-22.

Table 5-13 December 2010 – Model calibration peak flow and timing

	Observed	Modelled	Difference
Peak flow (first peak)	54.1 m³/s	60.5 m³/s	6.4 m³/s (11.8%)
Timing (first peak)	08/12/2010 4:00 pm	08/12/10 10:00 pm	10 hrs
Peak flow (second peak)	116.7 m³/s	123.0 m³/s	6.3 m³/s (5.4%)
Timing (second peak)	09/12/10 10:00 pm	9/12/2010 10:00pm	-

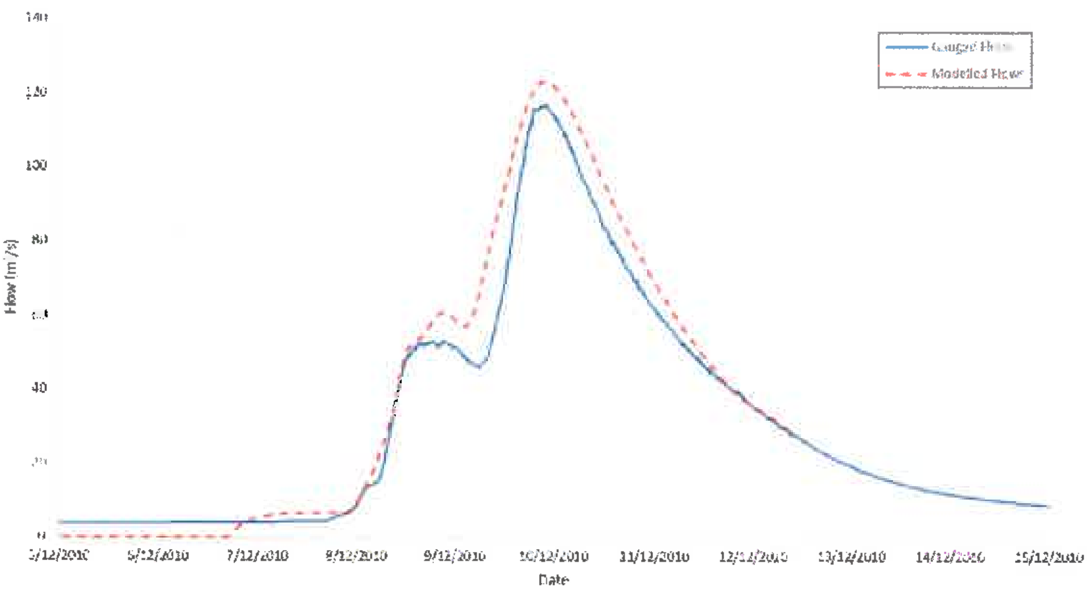


Figure 5-22 December 2010 – Harrow modelled and recorded hydrographs

Very high initial and continuing losses were adopted for the December 2010 calibration. This large initial and continuing loss was unexpected given the first rainfall burst days earlier was excluded from the RORB modelling. It was expected that losses would be lower considering the wet antecedent conditions. However, the calibration achieved was relatively good.

The modelled flows were consistently higher than that observed indicating the volume of the hydrograph is also slightly larger than that observed. The shape of the observed hydrograph was matched relatively closely.

January 2011

The January 2011 rainfall event was not significant in terms of flood impacts in the upper Glenelg River, but was very widespread across north-western and north-central Victoria. The event had two distinct rainfall bursts approximately 24hrs apart with large rainfall totals recorded to 9am on the 12th and 14th of January respectively. The average total rainfall depth across the RORB sub catchments was 126.4 mm. Whilst the January 2011 rainfall depth average was higher than December 2010, there was a more significant loss of runoff which caused significant attenuation of flooding at Harrow. This is discussed in more detail under Section 5.4.4 below. The January 2011 spatial pattern showing the total rainfall for each sub area is shown in Figure 5-23.

The Rocklands sub-daily rainfall gauge recorded two separate bursts of rainfall on the 12th and 14th, similar to indications made by the daily gauges around Harrow. The highest intensity was 56 mm/hr recorded in the early morning on the 12th. The temporal pattern of the January 2011 event recorded at Rocklands Reservoir is shown in Figure 5-24.

The Fulham Bridge gauge recorded a peak flow of 78.3 m^3/s, this was exceeded by the flow at Harrow, recording 79.8 m^3/s. This was due to the initial peak generated from the localised catchment area between Harrow and Fulham Bridge exceeding that of the broader catchment area upstream of Fulham Bridge. The reason for this can clearly be seen in the spatial distribution of rainfall, with the Salt Creek catchment receiving much higher rainfall totals than the broader catchment. The Fulham Bridge and Harrow streamflow gauges are shown in Figure 5-25.

The Glenelg River at Harrow began to rise around 36 hours after the first burst of rainfall, this initial rise was generated from the tributaries, particularly Salt Creek. The travel time between peaks from Fulham Bridge to Harrow was around 22 hours.

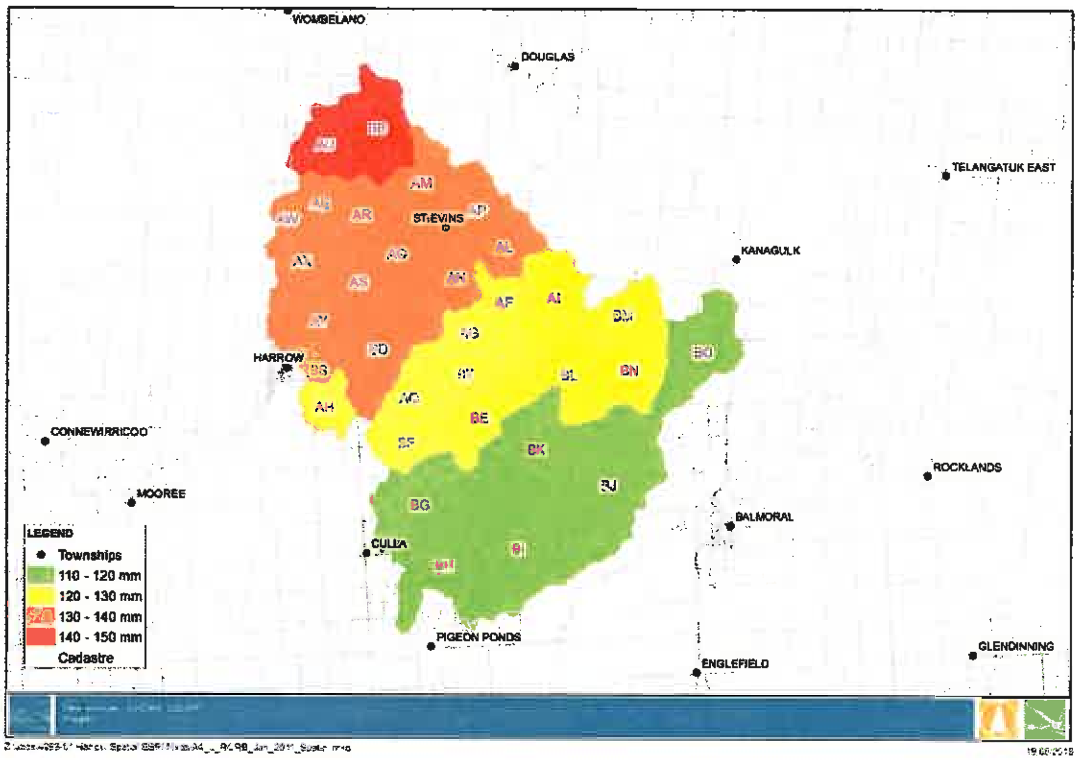


Figure 5-23 January 2011 – Rainfall spatial pattern

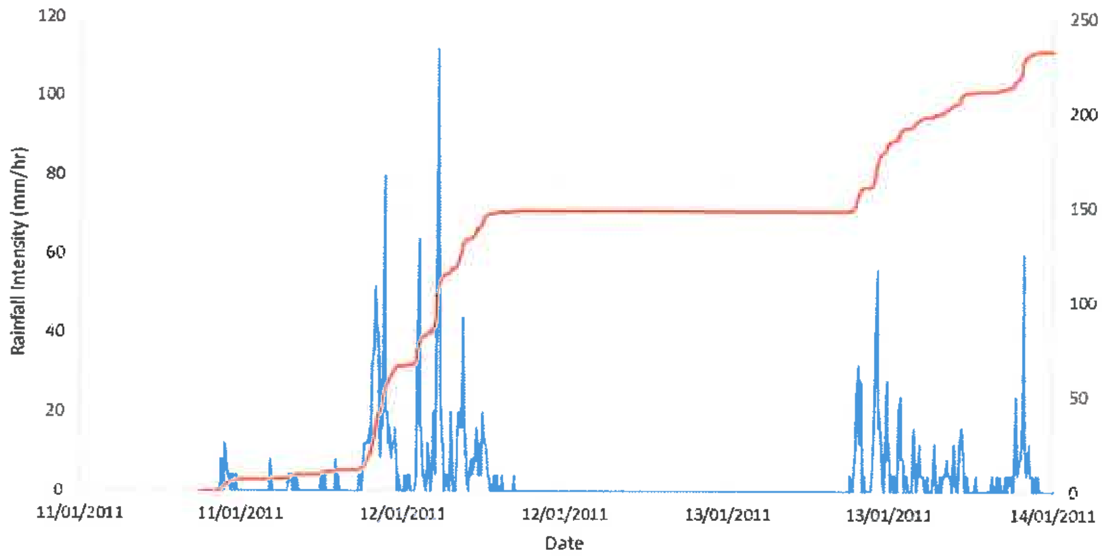


Figure 5-24 January 2011 – Rainfall temporal pattern

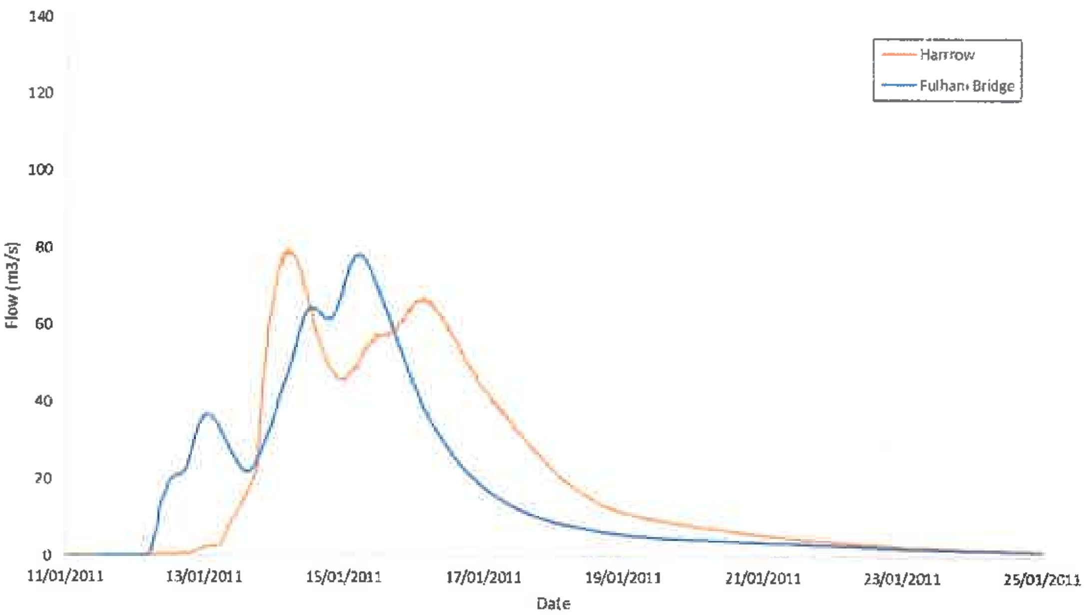


Figure 5-25 January 2011 recorded hydrographs at Fulham Bridge and Harrow

The RORB model was run for the January 2011 event using the recorded rainfall information, modelling was completed using a 'kc' value of 40, as it was shown as the best match during the September and December 2010 calibration modelling. The initial and continuing loss values were iteratively modified until the best match was determined.

The 1D hydraulic model roughness was maintained at a Manning's 'n' of 0.12 as determined during the previous 2010 events. The determined initial and continuing loss values were 50 mm and 10 mm/hr respectively.

The RORB model calibration results are shown in terms of peak flow and timing in Table 5-14 and hydrograph shape in Figure 5-26.

Table 5-14 January 2011 – Model calibration peak flow and timing

	Observed	Modelled	Difference
Peak flow	79.8 m^3/s	80.4 m^3/s	0.6 m^3/s (0.8%)
Timing	14/01/2011 4:00 am	14/01/2011 2:00 am	2 hrs

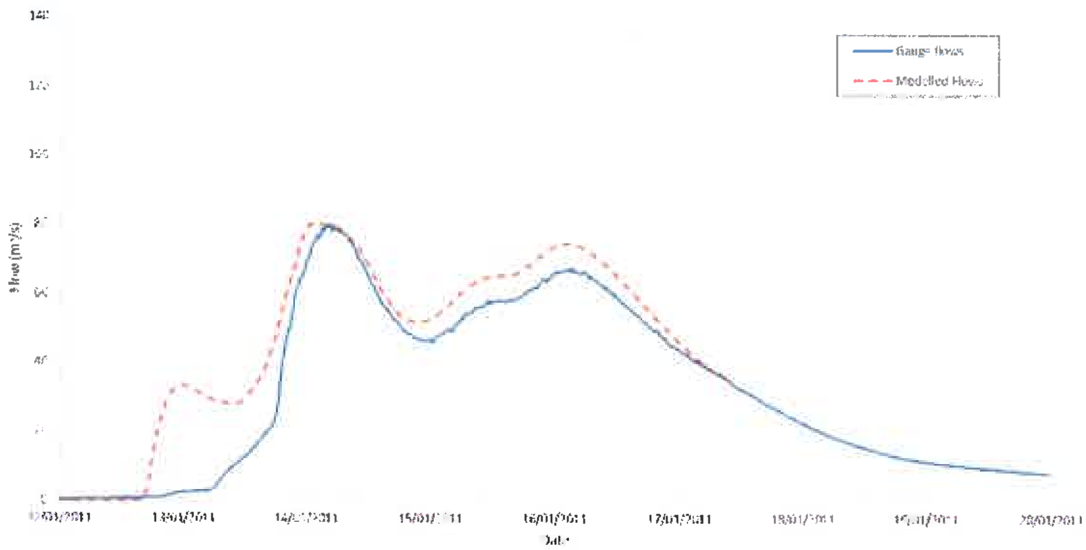


Figure 5-26 January 2011 – Harrow modelled and recorded hydrographs

The losses determined for the January 2011 event were very high. The recorded rainfall at the Rocklands Reservoir pluviograph and the daily streamflow gauges showed two rainfall bursts occurring prior to 9am on the 12th, and the second prior to 9am on the 14th. The RORB model results show a minor peak in the Glenelg River streamflow on the 13th, however no initial peak was actually recorded. Over the duration of the event the modelled volume and flow rates are slightly larger than the recorded event with a 10 mm/hr continuing loss. A larger initial loss would remove the early peak in the modelled results but the loss value is already very high.

5.4.4 Discussion

During the model calibration process the December 2010 and January 2011 events required very high losses to match the gauged flow at Harrow. Losses of this magnitude were surprising for the study team and further analysis was required. Analysis focused on separating the model components and confirming each of them. The model testing was completed on the December 2010 event and included:

- Running the M11 1D model separately without RORB model inflows to confirm routing along the Glenelg River was represented well. This was confirmed by the peak flows and timing matching well at Harrow.
- Modelling the December 2010 event for multiple periods, including all bursts or just the last two, which reduces the total rainfall depth across the event. This showed the first rainfall burst didn't really contribute to the streamflow at Harrow and should be removed from the RORB model event.

- Comparing the catchment rainfall volume and the increase in gauge hydrograph volume between Fulham Bridge and Harrow indicating large losses in the Glenelg River catchment downstream of Fulham Bridge.
- Confirming the Glenelg River catchment area between Fulham Bridge and Harrow using multiple methods. This allowed us to refine the catchment boundary and exclude some of the flat catchment that drains to a chain of terminal wetlands to the north-west of the Salt Creek catchment.
- Increasing the 'kc' value to increase attenuation, produced a better match to recorded peak flows and timing of rise and fall at Harrow.

Each of these tests is discussed in the following sections.

M11 1D model - Glenelg River routing test

Running the 1D model and routing the Fulham Bridge inflows without any tributary inflows showed the routed flow matched that of Harrow gauging station quite closely. The attenuation was matched well with a close match on timing and peak flow, with the modelled flows slightly higher than that observed. The observed and modelled flow comparison at Harrow is shown in Figure 5-27.

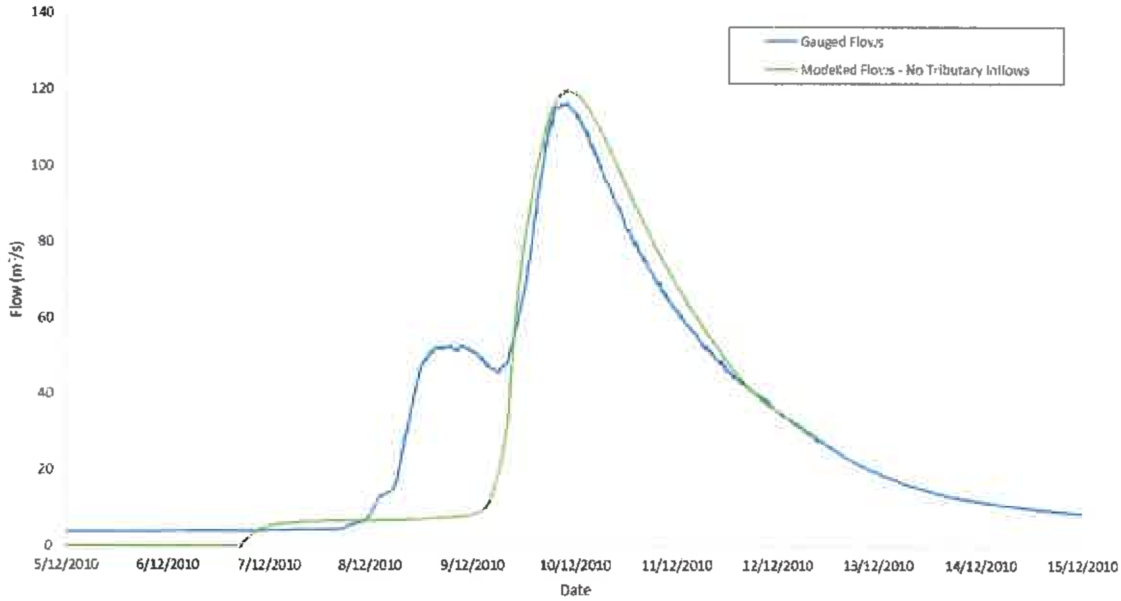


Figure 5-27 December 2010 – Modelled and recorded hydrographs with no RORB inflows

The modelled and observed flow comparison at Harrow indicates that the 1D model is accurately representing the routing of the Glenelg River. It also indicates that the effect of tributary inflows between Fulham Bridge and Harrow is highly variable in terms of the ultimate flood level attained at Harrow. The magnitude and distribution of rainfall events appear to be significant n terms of how significant the tributary inflows are likely to be.

December 2010 - Multiple durations

The December 2010 event has three separate rainfall bursts, as shown in Figure 5-19. The RORB model was run using all three, the second and third burst and the third burst alone. The December 2010 event peak flows occurred in the evening of the 9th and the largest daily totals in the catchment area between Fulham Bridge and Harrow occurred on the 8th with flooding peaking in Harrow on the 10th, it is clear the majority of the inundation was caused by rainfall occurring on the 8th.

Modelling of three, two and one burst required very similar losses for the Modelled RORB hydrograph to match that observed at Harrow. This is likely to be because high flow in the Glenelg River didn't begin to occur until early in the morning on the 8th after the third burst of rainfall. There was no real increase flow after the first burst occurring on the 6th.

Volume Comparison

The volume of each of the modelled events was calculated at the Fulham Bridge and Harrow gauging stations, this was completed over the full event hydrographs. The durations were 6 days for September 2010 and 12 days for December 2010 and January 2011.

The calculated hydrograph volumes were compared to determine the increase in volume between Fulham Bridge and Harrow and therefore the rainfall excess volume from each of the events, this was then converted to an average rainfall excess depth using the total catchment area and compared to the rainfall volume excess and depth determined in the calibrated RORB model results.

This comparison is shown below in Table 5-15.

Losses used in the RORB model compared to the observed losses calculated between Fulham Bridge and Harrow streamflow gauges and the recorded rainfall depths match relatively closely, with the RORB losses lower than that determined for each event using the rainfall and streamflow gauge information. During September 2010 the RORB model used a total loss of 26.4 mm while the gauge information indicated a loss of 29.7 mm. For the December 2010 event the RORB model adopted loss was 100.9 mm and the gauge information indicated 118.1 mm. RORB modelling of the January 2011 event adopted a total loss of 105.4 mm while the gauge information indicated a loss of 117.9 mm.

Table 5-15 Calibration Event Volume Comparison – Fulham Bridge to Harrow

	September 2010	December 2010	January 2011
Fulham Bridge streamflow volume (ML)	14,589	25,804	20,268
Harrow streamflow volume (ML)	18,331	28,125	23,607
Gained downstream volume (ML)	3,743	2,321	3,339
Recorded average rainfall depth (mm)	39.5	115.0	126.4
Recorded rainfall volume (ML)	14,539	42,364	46,547
Rainfall loss (ML) – (Gained downstream volume – recorded rainfall volume)	10,796	40,043	43,208
Average rainfall loss (mm) (Volume divided by catchment area)	29.3	108.7	117.3
RORB modelled loss volume (ML)	5,240	5,590	9,000
RORB modelled loss depth (mm) (Volume divided by catchment area)	26.4	100.9	105.4

As the above table shows, the rainfall losses across the catchment are indeed high. The modelled losses in RORB and those calculated by a simple water balance are reasonably close, providing justification of the loss values. It is understood that a simple water balance of loss values is not an accurate means to calculate loss values, but it does demonstrate the high losses are reasonable.

kc increases

The RORB model 'kc' value determined to best match the recorded data was 40. To test if lower losses could be adopted by increasing the RORB model attenuation, the 'kc' was increased to 80. The December 2010 event was used for the model testing. The result was a lowering of the RORB model peak flow, allowing lower loss values to be applied. However, the increased 'kc' resulted in the timing of the tributary flows being slowed down and coinciding closer with the routed Glenelg River flows from Fulham Bridge, increasing the peak flow at Harrow, and ruining the hydrograph shape. A demonstration of this is shown in Figure 5-28, where initial and continuing losses of 35 mm and 5 mm/hr were used.

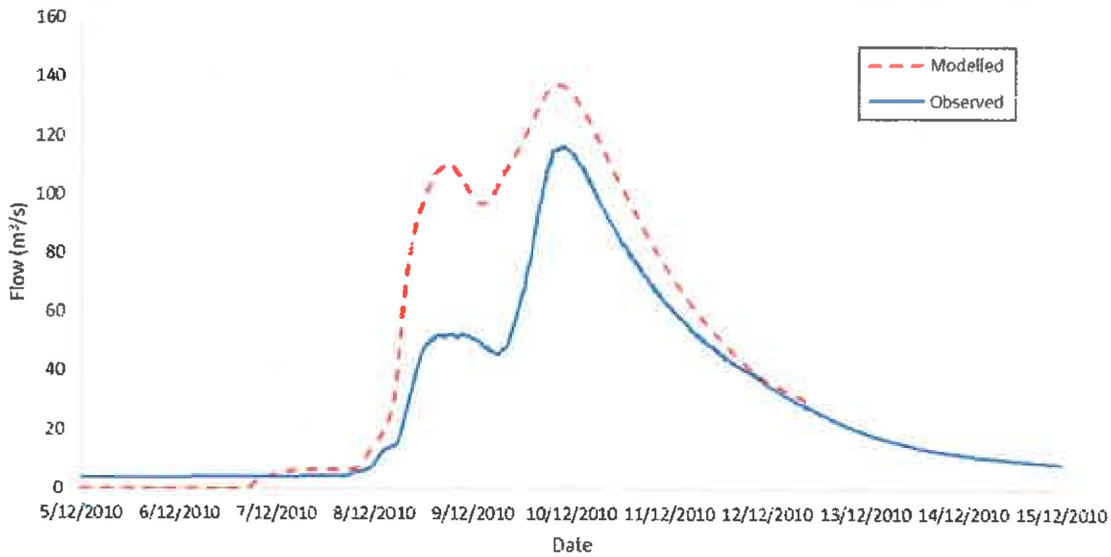


Figure 5-28 December 2010 – Initial loss of 35 mm and continuing loss of 5 mm/hr, 'kc' of 80

After a significant amount of testing the RORB model calibration values adopted for each of the calibration events seem reasonable and provide a good match to observed flows at Harrow. Model parameters are further discussed in regard to design modelling in Section 5.5.

5.5 Design Modelling

Modelling of a range of possible future design flood events was undertaken during this study. Flood modelling of the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% and PMF was required.

5.5.1 RORB Modelling

Design Rainfall Depths

Design rainfall depths were determined using the Bureau of Meteorology online IFD tool[23]. The rainfall Intensity Frequency Duration (IFD) parameters were generated for a location in the approximate centre of the Glenelg catchment area between Fulham Bridge and Harrow (37.15Lat, 141.650Long) and are shown in Table 5-16 below.

[23] BoM Online IFD Tool - http://www.bom.gov.au/hydro/has/cdirswebx/cdirswebx.shtml *Accessed: December 2011*

Table 5-16 Catchment IFD Parameters

$2I_1$ (mm/hr)	$2I_{12}$ (mm/hr)	$2I_{72}$ (mm/hr)	$50I_1$ (mm/hr)	$50I_{12}$ (mm/hr)	$50I_{72}$ (mm/hr)	G	F2	F50
17.65	3.33	0.87	33.92	6.22	1.63	0.46	4.38	14.76

Design Temporal Pattern

Design temporal patterns were taken from Australian Rainfall and Runoff[24]. In order to understand the sensitivity of the flood estimates to temporal pattern a number of patterns were first reviewed. The catchment area between Fulham Bridge and Harrow is located within Zone 6 of the temporal pattern map as defined in Australian Rainfall and Runoff[20] (1987); however, it is located close to the boundary between Zone 2 and Zone 6.

During the Glenelg Regional Flood Mapping Project[1], Zone 2 and Zone 6 temporal patterns were compared for a 48 hour duration storm. 48 hrs was approximately representative of the 1975 and December 2010 events, the largest observed events in the Glenelg River catchment. Figure 5-29 shows a comparison of the temporal patterns using percentage of storm duration and percentage of total rainfall. Given the observed events matched the Zone 2 pattern more closely it was adopted for the design modelling in this project.

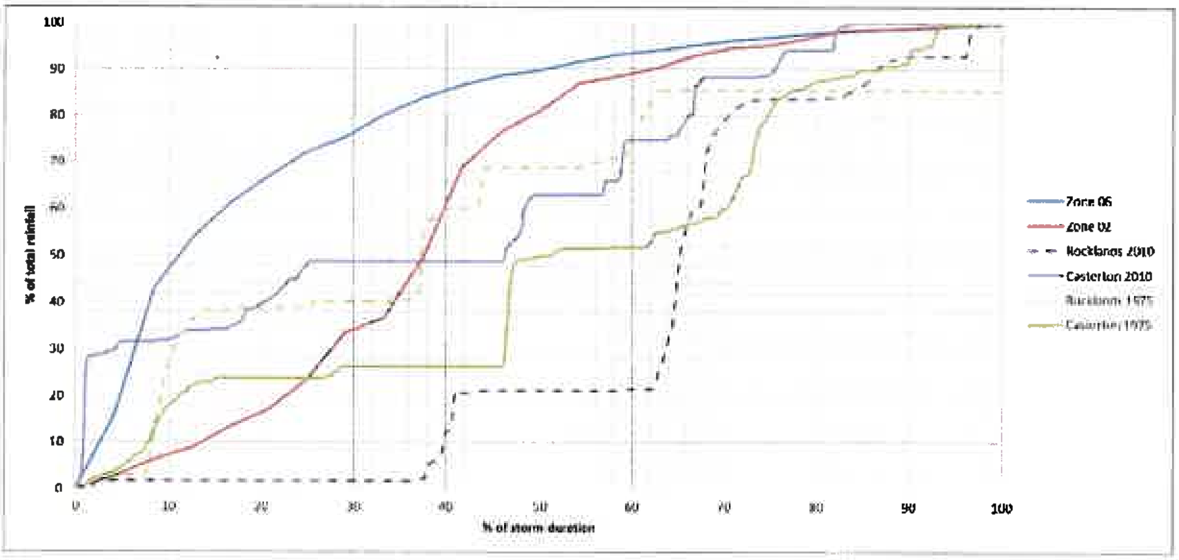


Figure 5-29 Zone 02, Zone 06 and historic temporal patterns over a 48 hour duration

Design Spatial Pattern

A varying spatial rainfall pattern (i.e. different rainfall depths applied to each sub area in the catchment) was adopted for the generation of design flood hydrographs for events up to the 0.2% AEP event. This is in line with ARR2016[25] recommendations that design modelling for catchments over 20 km^2 should consider spatially varying design rainfalls.

[24] Engineers Australia (1987) - Australian Rainfall and Runoff

[25] Engineers Australia (2016), Australian Rainfall and Runoff, Book 2 Section

Design spatial patterns were varied according to the IFD maps produced by the BoM and included in ARR87[26], with the total rainfall for each AEP event rainfall proportioned accordingly.

The percentage of mean catchment area rainfall applied to each subarea for each design event is shown in Figure 5-30.

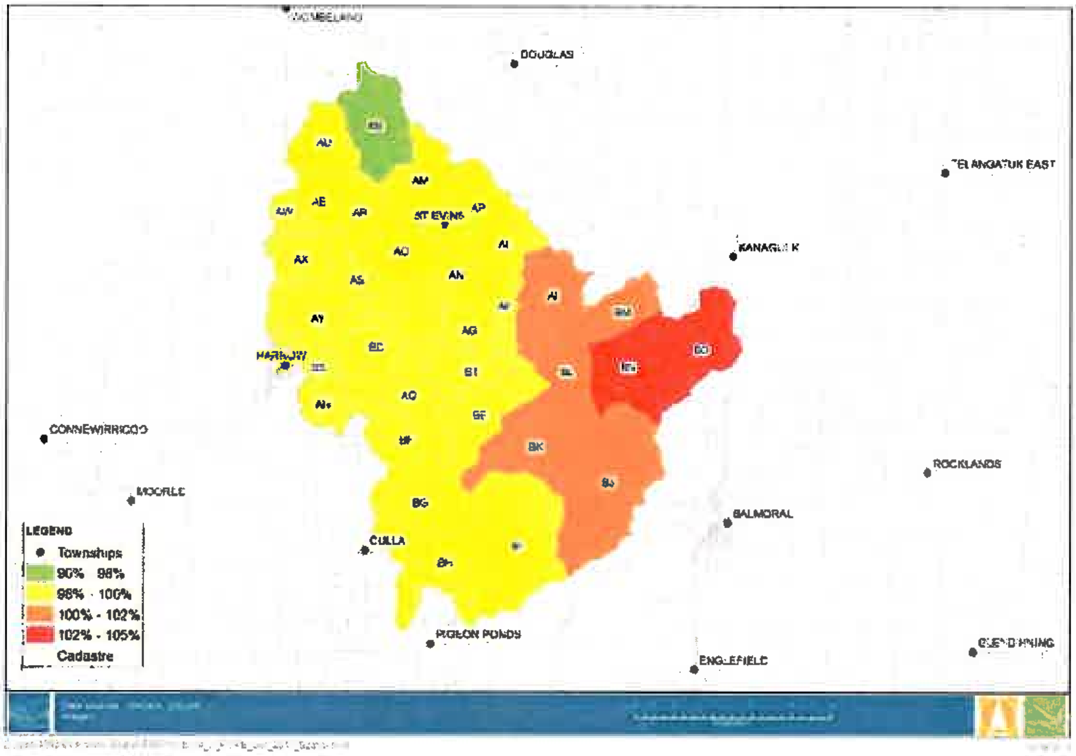


Figure 5-30 Design spatial pattern rainfall distribution

Areal Reduction Factors

Areal reduction factors were used to convert point rainfall to areal estimates and are used to account for the variation of rainfall intensities over a large catchment. Siriwardena and Weinmann (1996)[27] areal reduction factors were applied to the catchment area as recommended in Australian Rainfall and Runoff (1987)[28]. It is understood that these have not changed significantly for Victoria in the recent ARR edition[25].

Routing Parameters

Various regional 'kc' estimation equations were trialled during the model calibration, the model calibration determined a 'kc' of 40 matched each of the historic events well, and this was adopted in the design modelling.

[26] Bureau of Meteorology (1987), Australian Rainfall and Runoff

[27] Siriwardena and Weinmanm, 1996 - Derivation of Areal Reduction Factors For Design Rainfalls (18 - 120 hours) in Victoria

[28] Engineers Australia (1987), Australian Rainfall and Runoff

Design Losses

The calibration losses used for December 2010 (IL 50 mm, CL 6 mm/hr) and January 2011 events (IL 50 mm, CL 10 mm/hr) are higher than the recommended values in both Australian Rainfall and Runoff (1987 and the revised 2010 edition), while the September 2010 losses were closer to the expected vales (IL 15 mm, CL 2.5 mm/hr). ARR1987[29] recommends initial losses south of the Great Dividing Range ranging from 15-35mm and a continuing loss of 2.5 mm/hr, while the revised ARR2016 recommends a range of initial loss values from 15-40 mm for the Murray-Darling and south-east coast catchments. The continuing loss may range between 2.5 to 7 mm/hr in the Murray-Darling catchments and 1 to 3 mm/hr for Western Victorian catchments in the south-east coast region. The reason the Murray-Darling catchment values are also mentioned is that the local catchment is perhaps more indicative of the upper Wimmera than it is of the lower Glenelg. It should also be mentioned these new loss values are specific to temporal patterns that were not released at the time of this reports production.

The losses in the localised catchment area are highly dependent on the antecedent conditions and given both the December 2010 and January events occurred in summer, and had losses higher than those recommended an assessment of the most likely time a flood could occur on the Glenelg River was undertaken.

Figure 5-31 below shows the monthly mean and median mean daily flows for the entire length of record at the Fulham Bridge streamflow gauge. The months with the highest average daily flows are late winter/spring with July, August, September and October recording the highest mean values. This is also shown in the median daily streamflows. Large differences between the mean and median daily flows is an indication of the occurrence of extreme events, as they will statistically have a greater impact on the mean than the median. Larger differences between the mean and median daily peak flows are observed in the months of July, August, September and October, indicating those months have witnessed a greater proportion of extreme events. The highest ratio between monthly mean daily flow and monthly median daily flow were in September (1:2.1) and August (1:2.2), indicating these months were the most likely to have high flow events.

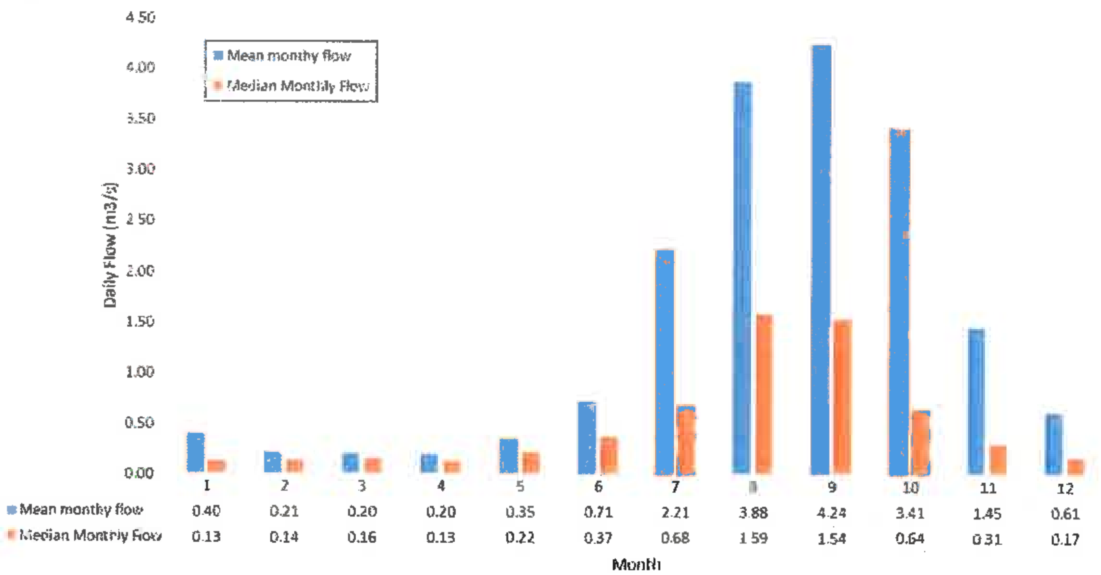


Figure 5-31 Fulham Bridge streamflow gauge – Monthly mean and median daily flows

[29] Engineers Australia (1987), Australian Rainfall and Runoff, Book 2, Section 3

The losses adopted for each of the calibration events along with the recommended ARR1987 and ARR2016 losses are shown in Table 5-17.

Table 5-17 Calibration and Recommended loss values

Source	Initial Loss (mm)	Continuing Loss (mm/hr)
September 2010 calibration	15	2.5
December 2010 calibration	50	6
January 2011 calibration	50	10
ARR1987	10-35	2.5
ARR2016	15-40	2.5-7

Sensitivity testing of design losses was undertaken using the calibrated RORB model. Testing was completed used a static initial loss of 35 mm and the continuing losses of 2.5 mm/hr and 5 mm/hr. The peak flows determined from these losses were then compared to the FFA determined peak flows at Fulham Bridge, considering the catchment area upstream of Fulham Bridge (downstream of Rocklands) and the RORB catchment area. The RORB catchment area was 368 km^2 and the catchment area between Fulham Bridge and Rocklands Reservoir was calculated at 864 km^2, the RORB model catchment area is 43% of the Fulham Bridge catchment area. A comparison of the peak flows for the modelled losses is shown in Table 5-18.

Table 5-18 Loss values – Sensitivity Testing

AEP (%)	FFA - Fulham Bridge Peak Flow (m^3/s)	RORB model flow (m^3/s)	
		IL35 CL5	IL35 CL2.5
20	74	2.1	7.8
10	106	4.6	18.8
5	130	10.75	37.4
2	152	25.3	72.3
1	164	57.6	116.8
0.5	172	96.9	169.5

The sensitivity testing is showing that the catchment area modelled by RORB has higher peak flows for both trialled continuing loss values when comparing to the FFA determined flows. Using a continuing loss of 2.5 mm/hr results in much higher flows coming from the RORB catchment than when a continuing loss of 5 mm/hr is used. In the 1% AEP event using a 2.5 mm/hr continuing loss, the RORB flow is 71 % of the FFA determined flow at Fulham Bridge, while using a continuing loss of 2.5 mm/hr the RORB flow is 35% of that determined by the FFA at Fulham Bridge. Using a continuing loss of 5 mm/hr results RORB model peak flows which match the up and downstream of Fulham Bridge catchment area ratio more closely for the 1% AEP event. Anecdotally, flooding in Harrow has been driven by flows generated by the broader catchment area with numerous community members confirming there are generally two flood peaks in Harrow, an initial small one then a larger second peak.

This study adopted an initial loss of 35 mm and a continuing loss of 5 mm as the design loss parameters. The loss parameters were applied across all AEP events and durations. The study team feel the adopted losses are a conservative estimate of rainfall losses in the catchment area. While the adopted losses are higher than those recommended by ARR1987 they are lower than the adopted December and September calibration losses by a reasonable amount. They are considered a reasonable estimate of what the losses could be during a flood event. The reality is the localised catchment contributions modelled by RORB only provide an initial flow in the Glenelg River prior to the larger catchment area routed from Fulham Bridge and do not provide the peak discharge at Harrow.

Probable Maximum Flood

Probable Maximum Precipitation (PMP) depths and temporal patterns were determined using the Generalised Southeast Australia Method (GSAM)[30]. The determined depths for each event duration are shown in Table 5-19.

Table 5-19 PMP depths

Method	Event Duration (hrs)	Depth (mm)
GSAM	24	470
	36	530
	48	560
	72	590

The RORB model encompassing the Rocklands upstream catchment developed during the Glenelg Regional Flood Mapping Project[1] was run for each event duration using the maximum operating level of Rocklands Reservoir (75%). The losses utilised for the 0.5% AEP event were also adopted for the PMF, as outlined in Table 5-7. The 36 hr event was shown to be the critical duration generating a peak flow of 3294 m^3/s.

5.5.2 1D Modelling

The 1D model was run using the design hydrographs determined for Fulham Bridge and the RORB determined inflows. Across the three modelled calibration events the Harrow gauge record shows that the localised catchment inflow to the Glenelg River peaked consistently 30-48 hrs before that of the flow routed from Fulham Bridge. A 30 hr spacing was used to separate the RORB generated and Fulham Bridge hydrographs at Harrow. This separation was made by iteratively running the Mike11 model varying the timing of the Fulham Bridge inflow.

The flow routed from Fulham Bridge was larger than that generated by the localised catchment area for between Fulham Bridge and Harrow for each of the modelled design flood events. The localised catchment area contributions modelled in RORB and input into the 1D MIKE11 model provided an initial peak in the Glenelg River prior to the Fulham Bridge routed flows, producing a hydrograph that looks much like those of the three calibration events considered.

The peak flows at Harrow for the modelled flood event are shown in Table 5-20.

Table 5-20 Modelled design event peak flows at Harrow

AEP	Harrow peak flow (m^3/s)	Fulham Bridge peak flow (m^3/s)
20 %	74	74
10 %	105	106
5 %	129	130
2 %	150	152

[30] BoM (2006), Guidebook to the Estimation of Probable Maximum Precipitation: GENERALISED SOUTHEAST AUSTRALIA METHOD

1 %	162	164
0.5 %	169	172
0.2 %	175	178

5.5.3 Localised Catchment area design estimation verification

Overview

Several comparisons were made between the RORB model 1% AEP peak flow and empirical peak flow estimation equations. These estimates were made for the catchment area between Fulham Bridge and Harrow with an area of 368 km^2. Catchment area is the major driver for peak flow in these equations.

Rational Method

Probabilistic Rational Method[31] calculations were performed as a comparison to the RORB generated peak flows. The Rational Method estimated a higher 1% AEP peak flow of 147 m^3/s. The method of calculation is shown below:

$$Q_{100} = C_y * I * \mathrm{A}$$

Where,

$C_y = F_y * C_{10}$

$\mathrm{I} = Rainfall\ intensity\ (\frac{mm}{hr})$

$\mathrm{A} = Area\ (km^2)$ = 368 km^2

And;

$F_y = 1.2$

$C_{10} = 0.9 * f + C_{10}^1 * (1 - f) = 10yr\ runoff\ coefficient = 0.10$

$F = Fraction\ Impervious$ = 0.1

$C_{10}^1 = the\ pervious\ area\ runoff\ coefficient = 0.126$

Regional Method

A regional method for estimating a 1% AEP peak flow in rural catchments (Grayson et al, 1996)[32] was applied to the Glenelg River catchment between Fulham Bridge and Harrow. The peak 1% AEP flow generated by the Glenelg River catchment between Fulham Bridge and Harrow was estimated as 424 m^3/s. The method of calculation is shown below, where the catchment area is 368 km^2:

$$Q_{100} = 4.67\ A^{0.763}$$

Regional Flood Frequency Estimation Model

[31] ARR 1987 – Australian Rainfall and Runoff

[32] Grayson et al, 1996 - Estimation Techniques in Australian Hydrology

The Regional Flood Frequency Estimation (RFFE) Model[33] developed by Australian Rainfall and Runoff was used to estimate the 1% AEP peak discharge from the catchment area between Fulham Bridge and Harrow for comparison to the RORB model output. The RFFE model produced a peak 1% AEP flow of 334 m^3/s.

[33] http://rffe.arr.org.au/

Flow Comparison

The equation based 1% AEP flow estimates were compared to the design 1% AEP flow generated from the localised catchment area between Fulham Bridge and Harrow generated by the RORB model and routed to Harrow using the 1D model. This comparison is shown in Table 5-21

Table 5-21 Design peak flow comparison

Method of calculation	Peak Flow (m^3/s)
This studies RORB model/1D model	73.2
Rational Method	147
Regional Method	424
RFFE	334

The RORB model is producing considerably lower flows than that of empirical flow estimate equations, this is primarily due to the high losses adopted during the design modelling, however as discussed in Section 5.5.1 these loss values are considered appropriate and are significantly less than the losses adopted during the calibration process. RORB is a far more accurate way to determined design flow than the empirical flow estimation equations.

6. HYDRAULICS

6.1 Overview

A detailed combined 1D-2D hydraulic modelling approach was adopted for this study. The hydraulic modelling approach consisted of the following components:

- One dimensional (1D) hydraulic model of key waterways, drainage lines and hydraulic structures;
- Two dimensional (2D) hydraulic model of the broader floodplain; and
- Linked one and two dimensional hydraulic model to accurately model the interaction between in bank flows (1D) and overland floodplain flows (2D).

The hydraulic modelling suite, TUFLOW, was used in this study. TUFLOW is a widely used hydraulic model that is suitable for the analysis of overland flows in urban areas. TUFLOW has four main inputs:

- Topography and drainage infrastructure data;
- Inflow data (based on catchment hydrology);
- Roughness; and,
- Boundary conditions.

This section defines the scope of the hydraulic analysis, details the hydraulic model construction, and discusses the hydraulic model calibration.

The construction of the model is discussed in Section 6.2. Calibration of the hydraulic model to observed flood information underpins a reliable hydraulic model. Details of the hydraulic model calibration are provided in Section 6.3.

6.2 Hydraulic Model Schematisation

The TUFLOW model was constructed using MapInfo V11.0 and text editing software. This section details key elements and parameters of the TUFLOW model which adhere to both the AR&R 2D Modelling Guidelines – Project 15 Report[25] as well as the Melbourne Water 2D Modelling Guidelines[34].

The double precision version of the latest TUFLOW release was used for all simulations (TUFLOW Version: 2012-05-AC).

6.2.1 2D Grid Size and Topography

A single-domain approach was utilised to ensure the small areas of interest were modelled at an appropriate scale, while achieving practical model run-times. A relatively fine grid size of 4 m was selected for the Harrow township area to ensure the local tributaries could be accurately represented and mapped. This was deemed an appropriate grid size to accurately flood map the larger watercourses through the surrounding flat floodplain whilst also sufficient for the areas in and around the township.

The 2D model extents are shown below in Figure 6-1.

The model topography adopted was based on the datasets as outlined Section 3.4. This is primarily based on the lowered Index of Stream Conditions (ISC) LiDAR and an incorporated lowered waterway channel which was based on the combination of the toe of bank ISC data and a visual assessment of the aerial imagery.

[34] Melbourne Water (2010), 2D Design Modelling Guidelines

Within the Glenelg River channel, the LiDAR was lowered to account for the water surface reflecting the survey. Uniform lowering of the channel by 1.0 m and 0.5 m was trialled with 0.5m showing a better match to observed flood levels. This is discussed further in Section 6.3.

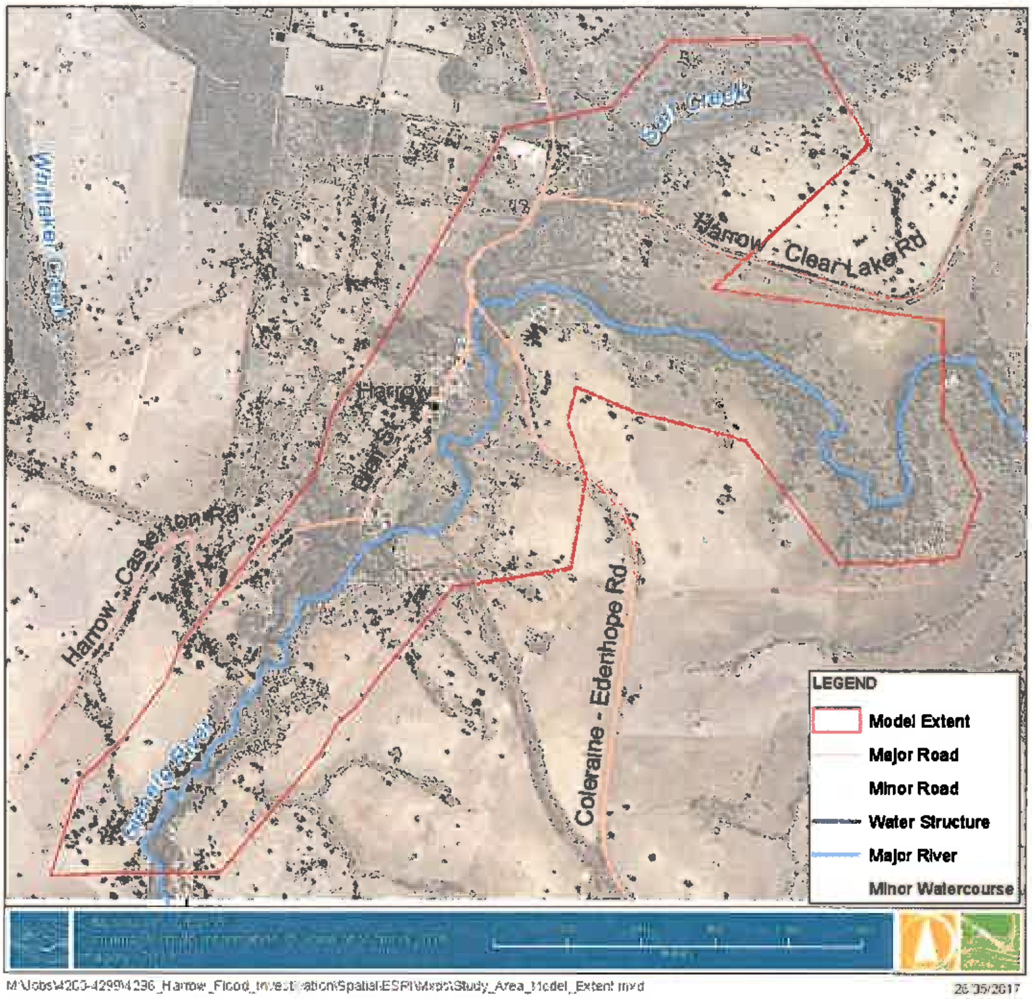


Figure 6-1 Extent of TUFLOW model

6.2.2 Roughness

The 2D model roughness values were produced based on Land Use Zones, with further refinement through the use of aerial photographs and site visits. The hydraulic model roughness values were also used as a mechanism for model calibration, adjusting the model roughness values to ensure the model results matched the observed flood information. This is discussed further in Section 6.3. The final adopted Manning's 'n' roughness values are listed in Table 6-1 and shown graphically in Figure 6-2.

Table 6-1 Manning's 'n' roughness values

Land Use	Manning's n Roughness Coefficient
Farmland/pasture/ Grassed	0.035
Residential	0.2
Industrial / Commercial zones	0.3

Paved Surface	0.02
Paved roads	0.02
Unpaved roads	0.03
Water bodies	0.03
Rural Residential/Township/Agricultural	0.06
Bushland/dense vegetation	0.1
Vegetated Creek	0.08

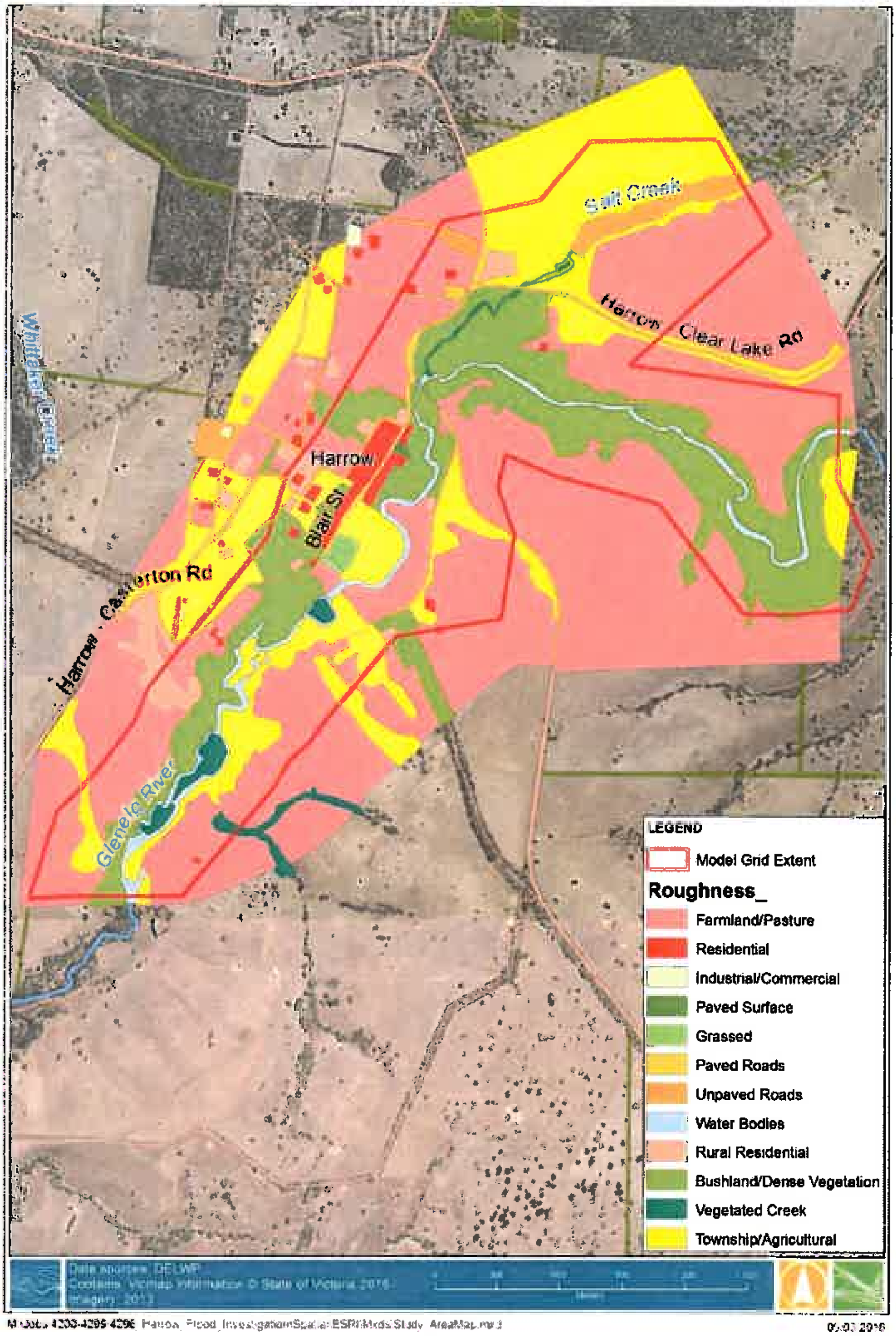


Figure 6-2 Adopted Manning's 'n' roughness values

6.2.3 Hydraulic Structures

Two brides were included in the hydraulic model. They were located on the Glenelg River at the Coleraine-Edenhope Road and on Salt Creek at the Harrow-Clear Lake Road. These bridges were modelled as layered flow constrictions as per design plans and site inspections. The modelled structures are shown in Figure 6-3.

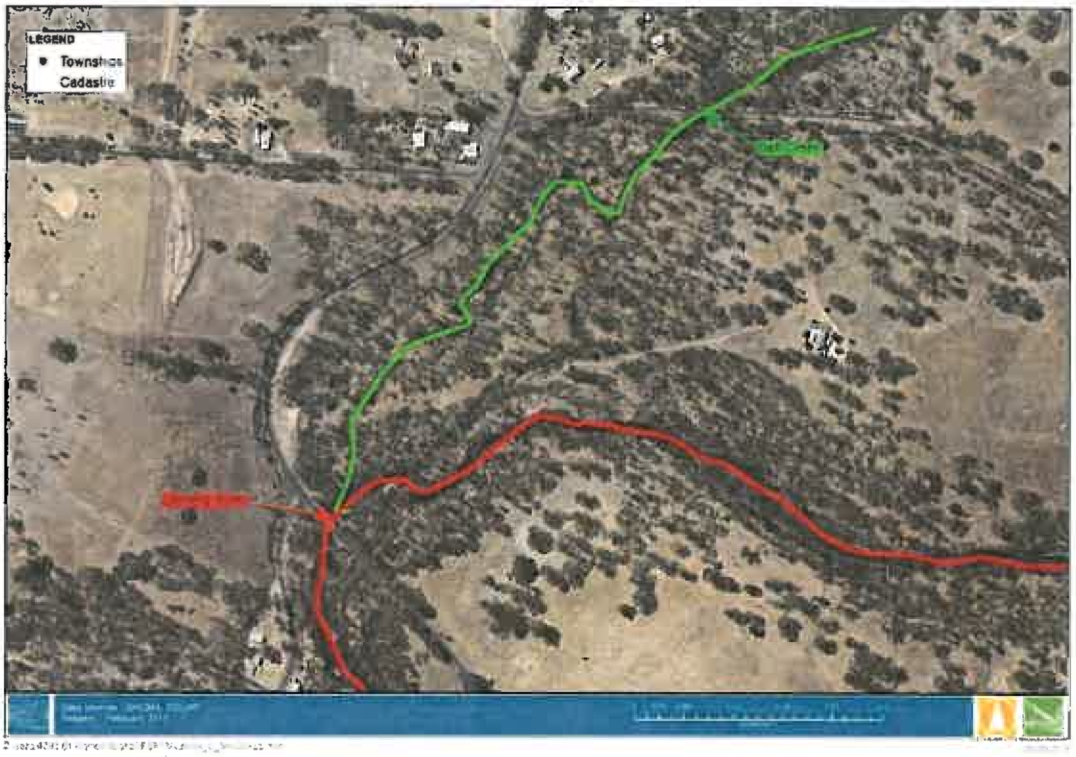


Figure 6-3 Structures included in the hydraulic model

6.2.4 Boundary Condition - Inlet boundaries

One of the principal considerations in constructing the model was the location of inflow boundaries to ensure all runoff from the catchment was being adequately represented in the modelling. The model boundaries for the Harrow model included the Glenelg River and Salt Creek. As outlined in this project's Hydrology Report, the Glenelg River inflows were determined by a combination of 1D routed flows from the Fulham Bridge combined with localised catchment inflows calculated in RORB. The Salt Creek inflows were determined by the calibrated RORB model.

6.2.5 Boundary Condition - Outlet boundaries

A 2D height flowrate (HQ) boundary was used at the downstream model boundary to convey Glenelg River flows from the model, HQ boundaries are a commonly used boundary type in TUFLOW which assign a water level based on the flow and topography.

The hydraulic model boundaries are shown in Figure 6-4.

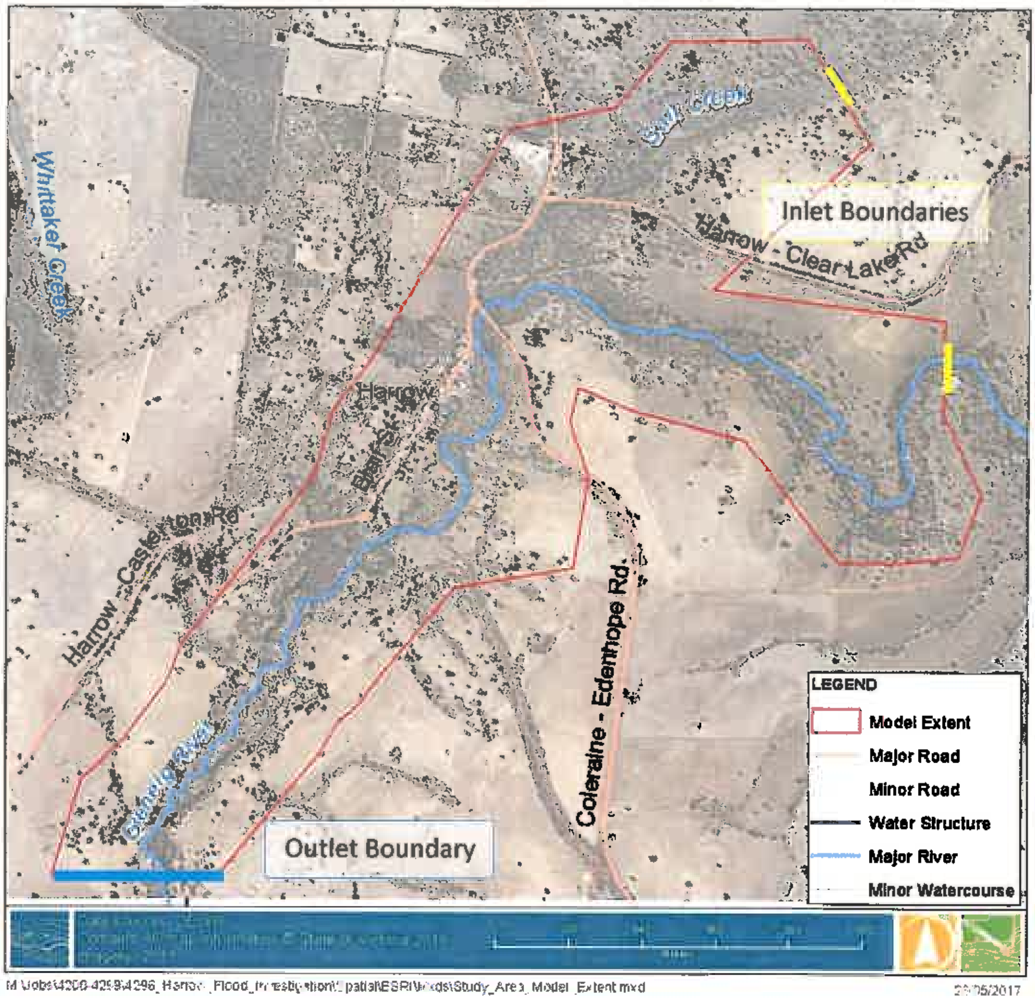


Figure 6-4 Hydraulic model boundaries

6.3 Hydraulic model calibration

Hydraulic model calibration was achieved through the comparison of modelled and observed flood heights (provided by Glenelg Hopkins CMA), observed gauge data and anecdotal community comments. December 2010 was used as the primary calibration event with September 2010 used as a secondary event. These events were chosen because of the available peak flood height information, gauge data at Harrow and available anecdotal evidence. Due to both events being within recent memory the community have expressed a good understanding and appreciation for the events.

It should be noted that while flood mark survey was available for the calibration events there is inherent inaccuracies in the collection of those levels. The levels are often based on flood debris marks which may be significantly higher or lower than the true peak due to a number of reasons such as debris piling up on the upstream side of an obstruction or debris being deposited during the recession of a flood.

A certain level of judgement is required in the collection of this data by the surveyor and inaccuracies in such data are common. As discussed below a two of the surveyed flood marks were found to be invalid due to obvious errors.

6.3.1 December 2010 Event Calibration

Nine surveyed flood marks were available for the December 2010 flood event. All of the reference points were surveyed to meters AHD and provide a reasonably reliable record for calibration of the event.

As can be seen from Figure 6-5 below, all the flood marks used for calibration were located in and around the township area.

Figure 6-5 Locations of December 2010 Surveyed Flood Marks

A number of simulations were modelled in order to develop a best fit with the recorded flood event data. Channel roughness was reduced from an initial adopted Manning's 'n' value of 0.1 to 0.08 to provide an appropriate calibration through this reach of the Glenelg River.

Figure 6-6 shows the modelled maximum water depth for the December 2010 event. A comparison of the surveyed flood levels and the modelled maximum water surface elevations was undertaken as part of the calibration process. Table 6-2 shows the difference between the modelled and surveyed levels at each respective location.

The model was able to replicate 7 of the 9 surveyed flood levels within 0.1 m. Surveyed levels at the northern extents, located on Blair Street showed the greatest difference to modelled levels. The northern most surveyed level (1) is described as a debris line on a corrugated iron fence. At this location the modelled level was around 0.40 m higher than that surveyed. Given how well the remainder of the survey marks matched the observed levels the landholder was contacted. Discussion revealed the modelled extents matched those observations more closely. Given the large difference in modelled and observed levels a large difference in extent would also be expected. It is likely that this survey point was in error.

The second northern most level (2) is described as a flood level mark taken on a 'Shed lean to'. The modelled level is approximately 0.23 m higher than that surveyed. Discussion with landholders indicated the modelled flood extent matched well with that observed in this area.

The model was shown to correlate well with the recorded results with 8 of 9 markers within 300 mm of the observed records and 7 of 9 within 100 mm. During the second round of community consultation there was general agreement the modelled levels and extents well replicated.

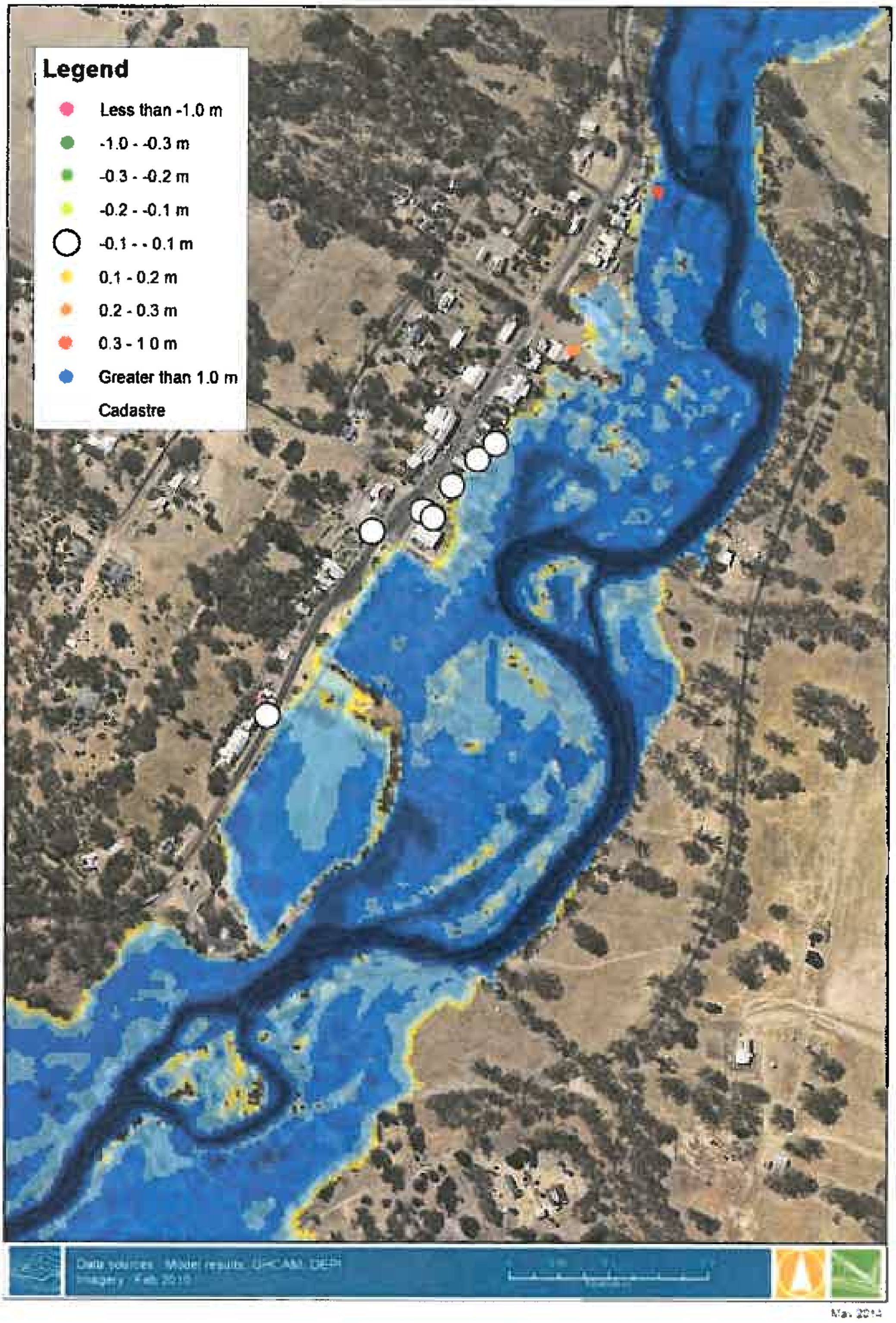


Figure 6-6 **Comparison of December 2010 model results against flood survey**

Table 6-2 Comparison of December 2010 flood marks and model results

Marker Number	Flood Marks (m AHD)	Model (m AHD)	Difference (m)
1	100.78	101.19	0.41
2	100.65	100.88	0.23
3	100.47	100.49	0.02
4	100.52	100.51	-0.01
5	100.51	100.52	0.01
6	100.46	100.44	-0.02
7	100.33	100.38	0.05
8	100.22	100.31	0.09
9	100.47	100.46	-0.01

Additional to comparison of the peak flood heights a water level comparison was made over the duration of the December 2010 event at the Glenelg River at Harrow streamflow gauge. This comparison is shown in Figure 6-7.

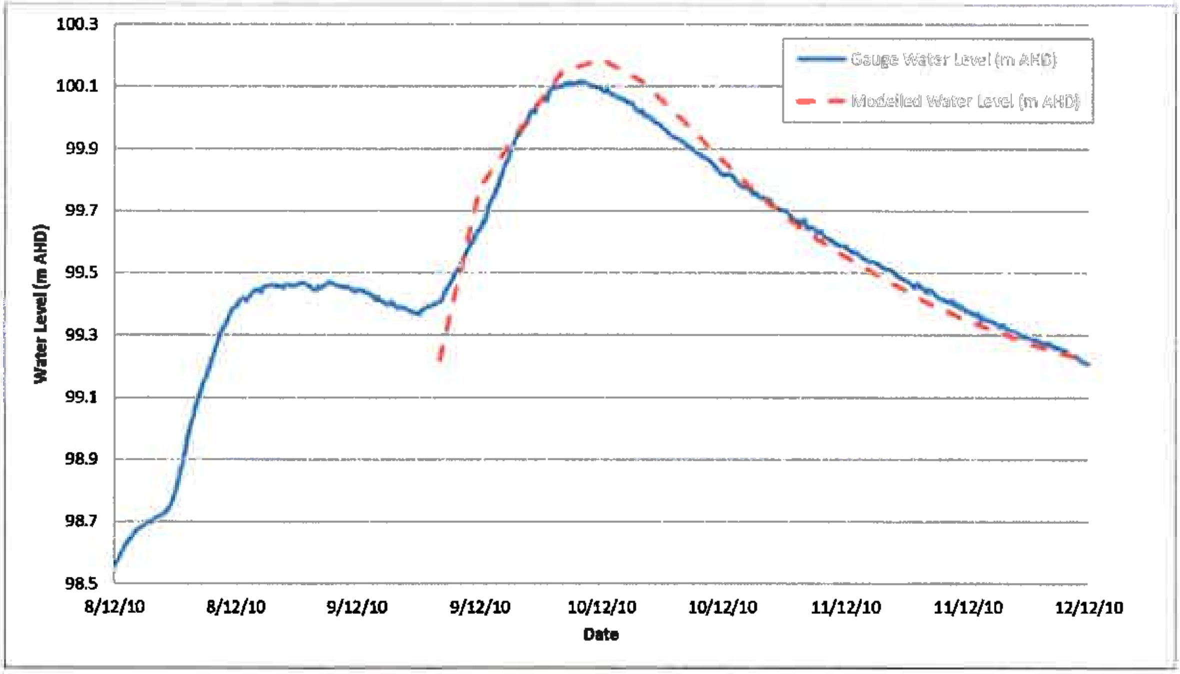


Figure 6-7 Comparison of December 2010 modelled and gauged water levels

The gauge reached a maximum water level of 100.12 m AHD, this compared to a modelled water level of 100.18 m AHD, a difference of 0.06 m. The shape of the water levels varying at the gauge also match quite closely.

6.3.2 September 2010 Event Calibration

Seven surveyed flood marks from the September 2010 flood event were made available by Glenelg Hopkins CMA with all points located within the Harrow township. A review of the survey marks found that several of the points were invalid with a number of them having no elevation information, indicative of flood extent only. Figure 6-8 shows the location of the available flood marks for the September 2010 event.

Figure 6-8 Location of September 2010 surveyed flood marks

Based on the model simulations undertaken for the December 2010 event calibration the refined hydraulic model was run for the September 2010 flood event. Only 3 of the recorded levels were surveyed to AHD a limited comparison of modelled and surveyed flood levels was available.

Of the 3 reliable surveyed flood marks two showed a difference between modelled and observed levels of less than 0.1 m, indicating a good calibration. The remaining survey marker, located on a power pole immediately upstream of the sporting oval is around 1.6 m higher than the modelled flood levels. Given that the available topographic information shows that the level is significantly higher than the surrounding streets and does not match with observed inundation extents from any historic events, it is likely that this survey point is in error.

The surveyed points matched the flood extent closely at most points. The modelled flooding of the September 2010 event was deemed an acceptable calibration result, albeit with limited calibration data available. A calibration plot for the September 2010 flood event is shown in Figure 6-9 below.

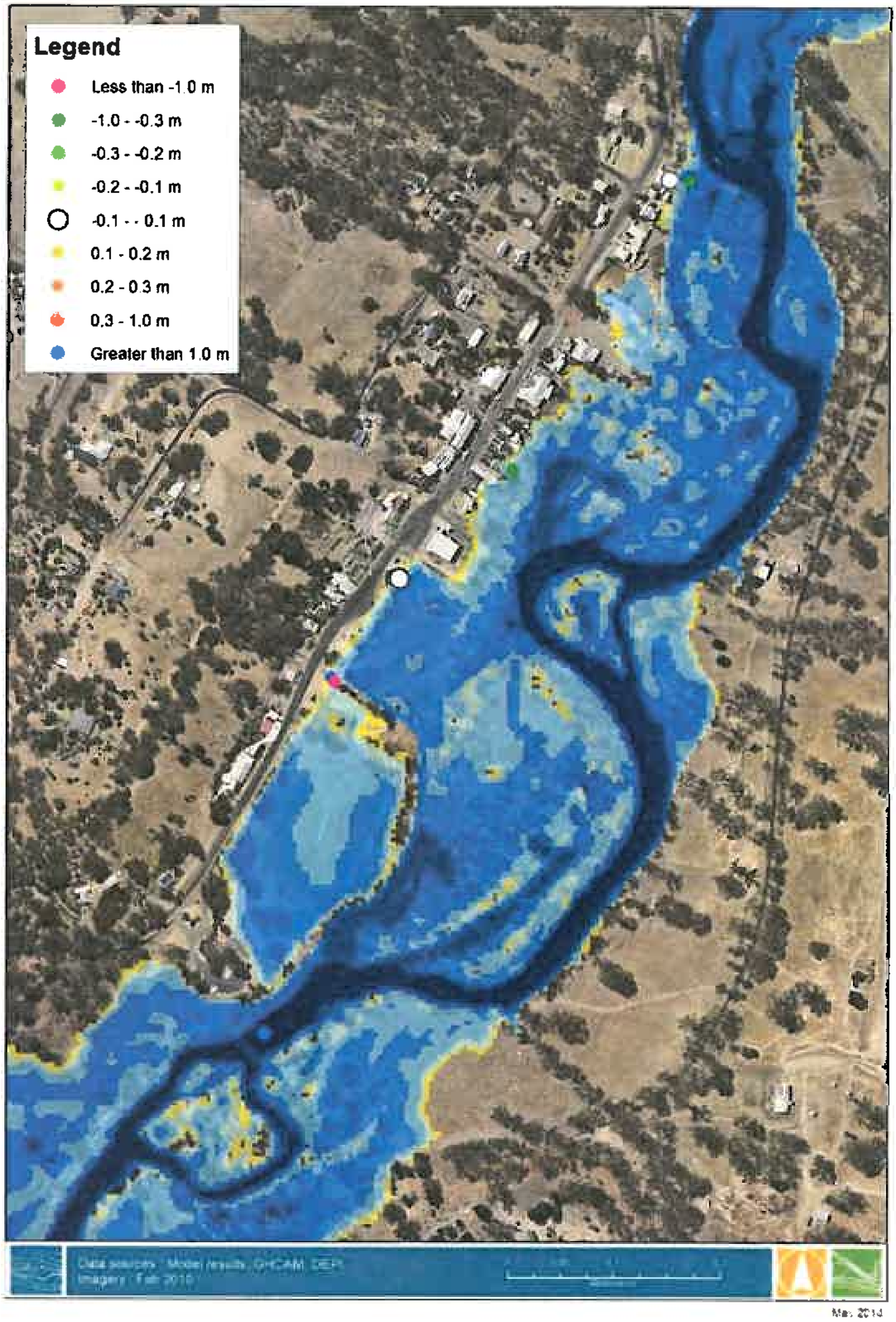


Figure 6-9 **Comparison of September 2010 model results against flood survey**

Table 6-3 Comparison of September 2010 flood marks and model results

Marker Number	Flood Marks (m AHD)	Model (m AHD)	Difference (m)
1	-	99.47	-
2	-	99.78	-
3	99.901	99.84	-0.058
4	-	100.37	-
5	100.326	100.37	0.047
6	-	99.75	-
7	101.321	99.69	-1.63

Additional to the survey point comparison the gauged and modelled heights were compared at the Glenelg River at Harrow streamflow gauge in the same fashion undertaken for the December 2010 event. This comparison is shown in Figure 6-10.

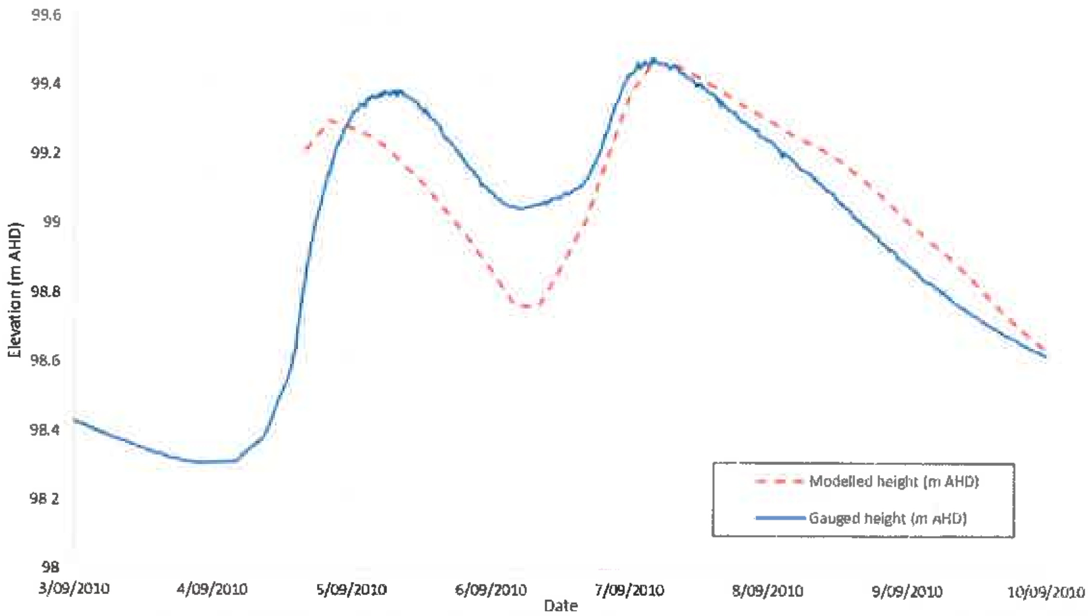


Figure 6-10 Comparison of September 2010 modelled and gauged water levels

The peak recorded water level at the gauge was 99.48 m AHD with the modelled water level 99.47 m AHD, showing a very close match. The shape of the water levels varying over the event is slightly different, this is likely to be due to differences in the inflows from RORB and perhaps the initial condition in the hydraulic model. The modelled hydrograph matches the observed flood behaviour well.

6.3.3 Anecdotal Comparison

Limited Imagery is available on which to base further validation of the flood levels and extents from the December 2010 and September 2010 events. Two images shown below in Figure 6-11, taken during the December 2010 event do however validate the significance of the event and show extents and heights within proximity of a high water mark. It is however likely that these photographs were taken following the peak of the flood during December 2010.

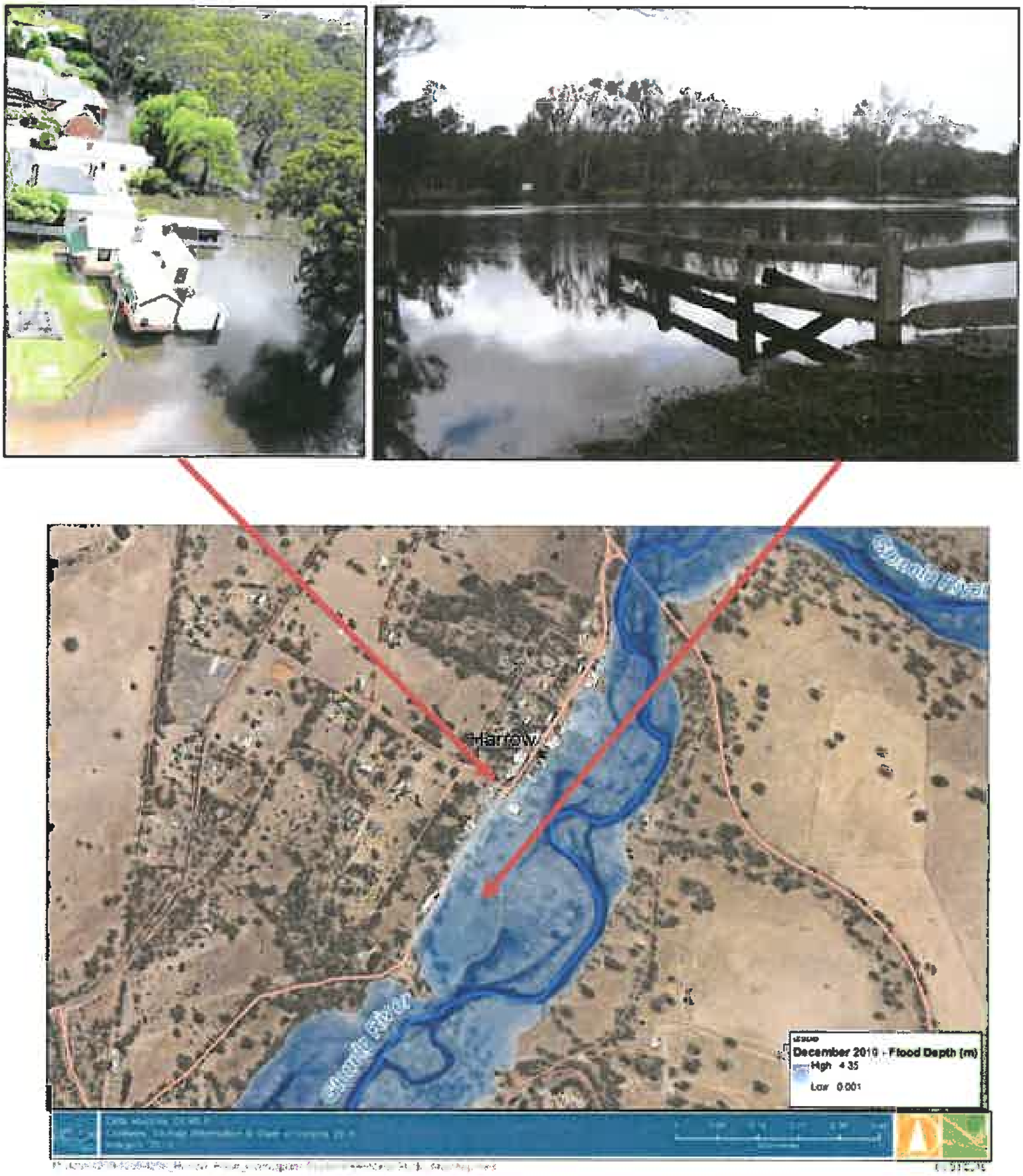


Figure 6-11 Comparison of December 2010 model results against flood photos

6.3.4 Discussion

Modelling of the December and September 2010 flood events has shown an excellent match to the observed data, using both a peak flood height and gauged water levels.

During the hydraulic model calibration, it was found modification of the Glenelg River channel had a reasonable impact on flood levels on the surrounding floodplain. For example, modification of the roughness between 0.08 and 0.1 caused around a 0.15 m increase in level. Modification to the channel invert to correct for the presence of water in the LiDAR, lowering from 1.0 m to 0.5 m was also shown to have a similar impact.

6.4 Design Hydraulic Modelling

Design hydraulic modelling was completed adopting the hydraulic model roughness values determined during the calibration phase, as discussed in Section 6.3. Modelling was completed for the full suite of design events including the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP events.

These events are overlayed in Figure 6-12, with a closer perspective of the Harrow township shown in Figure 6-13.

The inundation extents in Harrow don't vary much across design events, however the water levels between the 20% AEP and 0.2% AEP events increase by around 0.8 m at the gauge location, from 99.61 m AHD to 100.42 m AHD.

The PMF event was modelled with a single inflow boundary at the Glenelg River, this inflow combined all catchments upstream of Harrow including Salt Creek. Inundation depths for the PMF event are shown in Figure 6-14

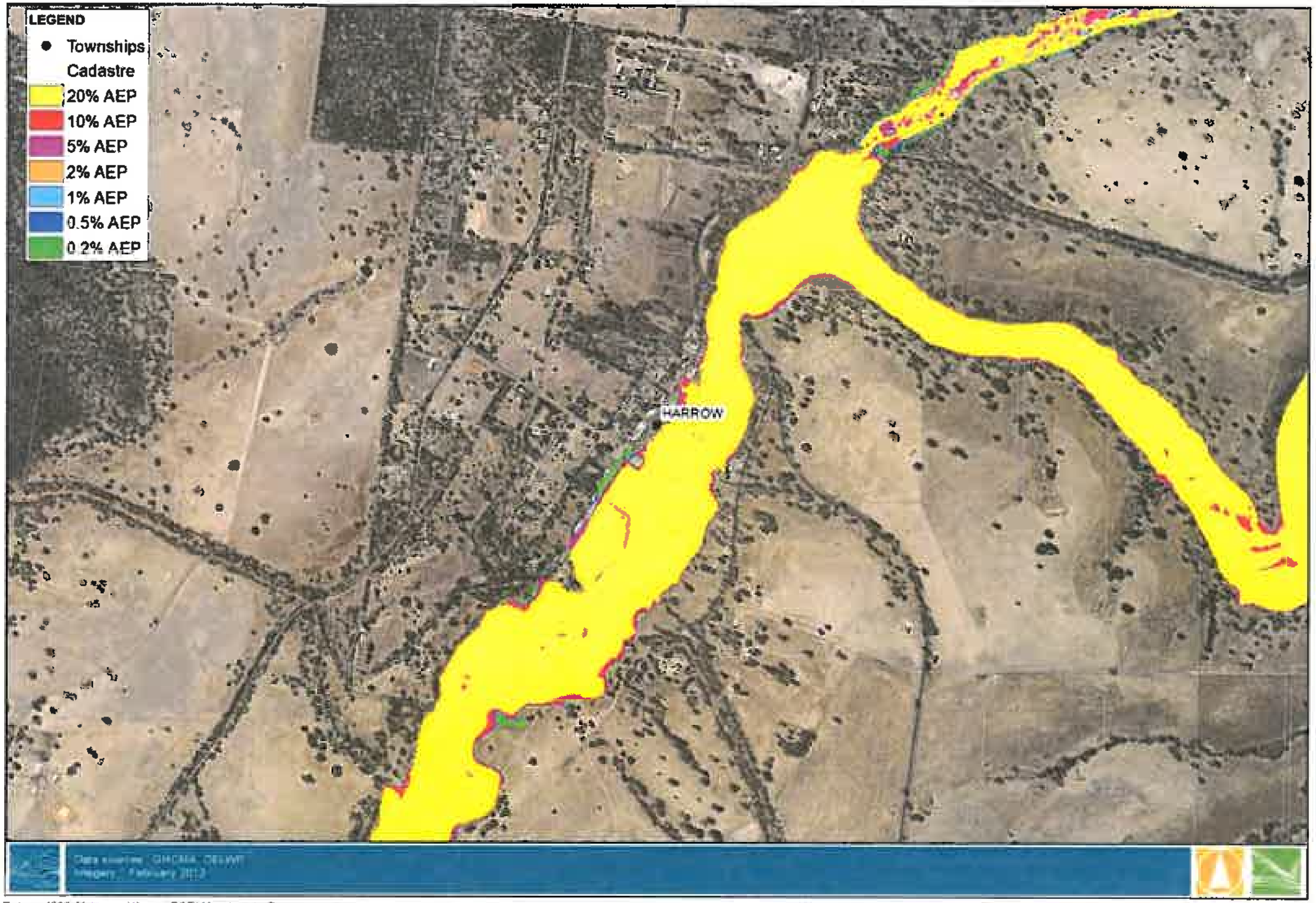


Figure 6-12 Design event flood mapping – All events overlayed

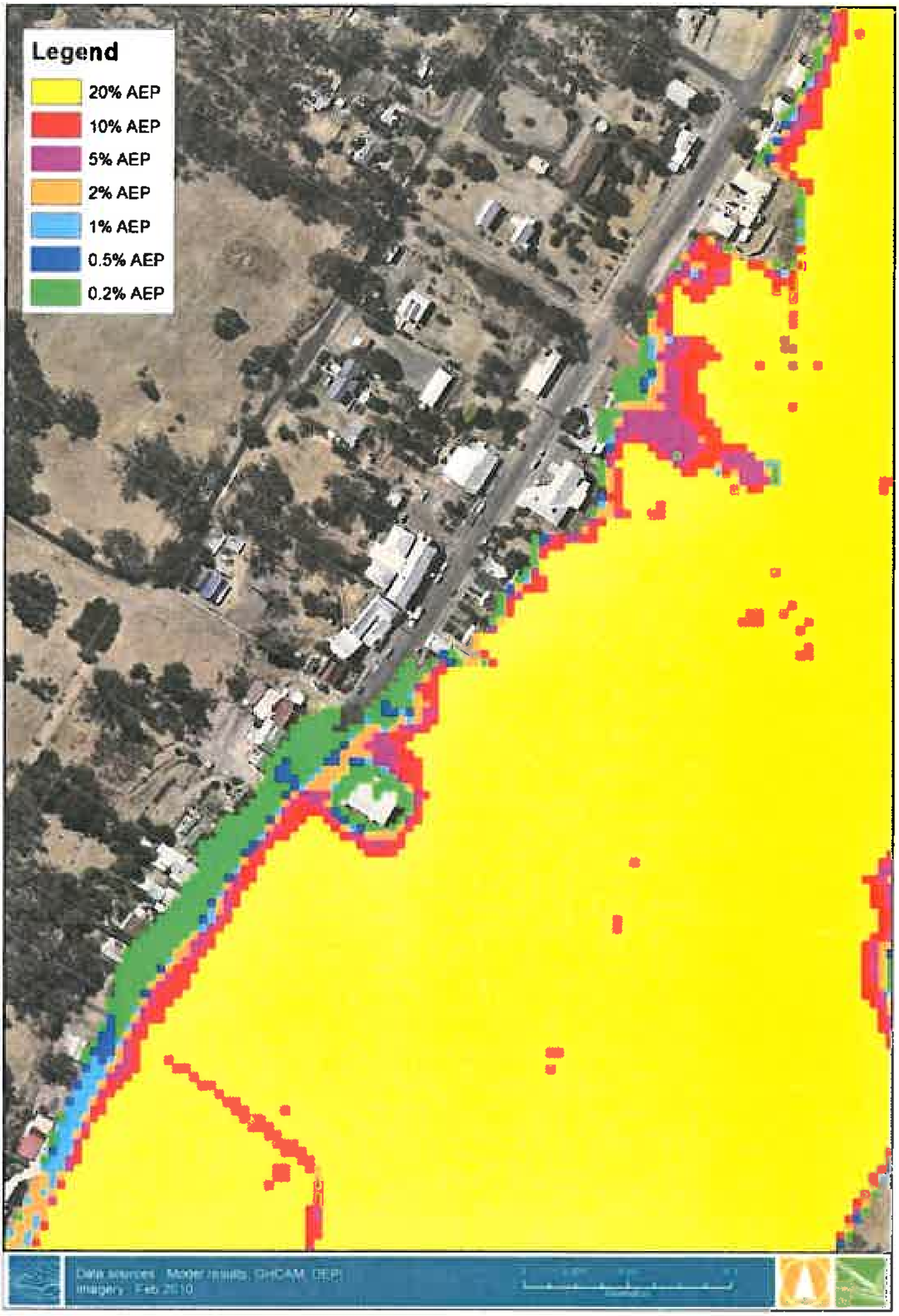


Figure 6-13 Design event flood mapping – All events overlayed (Harrow township)

Figure 6-14 PMF event flood depths

7. SENSITIVITY TESTING

7.1 Overview

The project brief required a number of sensitivity tests to be completed, these included:

- Three Rocklands Reservoir volume scenarios
- Variable roughness coefficients
- Blockage factors
- Boundary conditions
- Climate change scenarios

These tests were completed using both RORB and hydraulic modelling techniques.

7.2 Rocklands Reservoir

7.2.1 Overview

The impact of Rocklands Reservoir on flood behaviour at Harrow was raised by community members previous to this project, and during this projects community consultation process.

As discussed in Section 3.3.3, the outlet capacity of Rocklands Reservoir is 14.5 m^3/s (1,250 ML/d) and releases from Rocklands Reservoir occur via the main outlet which connects to the Toolondo Channel and Glenelg River. Flows can be discharged to the Glenelg River at three locations: 5 Mile outlet, 12 Mile outlet and the wall. The GWMWater O&M Manual for Rocklands Reservoir states the dam has never passed a major flood flow, with the maximum outflow stated at 61.3 m^3/s (5,300 ML/d) in 1975[35]. Small spills have occurred in the past, but they have been minor compared with flows generated from the catchment downstream of Rocklands.

Concern over the potential impact of Rocklands Reservoir outflows could have on inundation in Harrow is separated into spills and controlled releases. For this reason, modelling undertaken as part of this project has assessed three scenarios; a large spill from Rocklands, the maximum possible controlled release possible from Rocklands and a standard release rate. These event were modelled in the hydraulic model using the release/spill rate occurring at the same time as a 1% AEP event.

7.2.2 Hydrology

The impact of Rocklands Reservoir level on flood flows in the Glenelg River was tested using the RORB model of the entire catchment developed as part of the Glenelg Regional Mapping Project[1]. It is noted that the RORB model upstream of Rocklands was not calibrated well due to a lack data, however, to test the impact of starting levels in the storage, the volume into the reservoir is more important than peak flow. The calibration to peak flow is therefore not a major concern.

The 1% AEP flood event was run for Rocklands starting levels of 75% (historic operating level), 85% (current operating level) and 100% (maximum storage level prior to spilling). For each scenario, what spills from Rocklands Reservoir is purely dependent on the volume of water entering the reservoir. This makes the catchment conditions prior to rainfall, and therefore rainfall losses important in the estimation of inflows into the reservoir. Design modelling completed during the Glenelg Regional Mapping Project[1] used initial and continuing losses of 25 mm and 3 mm/hr respectively, these loss values were used upstream of the Fulham Bridge gauge to match the FFA completed at the gauge

[35] GWMWater (March 2010) - Rocklands Reservoir Operation, Inspection and Maintenance Manual (O&M Manual)

during the project. To maximise the event volume, the 72 hr event was used. Using these losses, no spills from Rocklands Reservoir occurred in scenarios with Rocklands starting at 75% and 85% full capacity. Peak discharge from Rocklands Reservoir in the 100% full starting level scenario was 24 m^3/s.

To test the sensitivity of lower losses, the RORB model was run using the initial and continuing loss values shown in Table 7-1. This was completed using the 100% initial starting capacity scenario. The peak outflow from Rocklands Reservoir for each scenario is also shown and hydrographs are shown in Table 7-1.

In addition to variable loss values, the 'kc' of the model was altered to test the impact of the peak inflow on Rocklands Reservoir outflows. By halving the 'kc' from 260 to 130 the peak flow was increased by 1.6 m^3/s or 7.7%. The Rocklands outflow is therefore not sensitive to the adopted Kc value and peak inflow into the reservoir.

Table 7-1 Sensitivity testing – Initial and continuing loss values and peak Rocklands Reservoir

Initial Loss (mm)	Continuing Loss (mm/hr)	Peak Rocklands Reservoir Outflow (m^3/s)
25	3	20
20	2	26
15	1	43
10	1	48

There are a number of possible reasons the Rocklands spills in 1956 and 1975 were larger than that shown in the sensitivity analysis. These could include:

- Rocklands has multiple high rainfall events over the spill, i.e. this may be an explanation for the 1956 and '75 events as these were both wet years.
- Operation and measurement of Rocklands occurred differently in the past to now

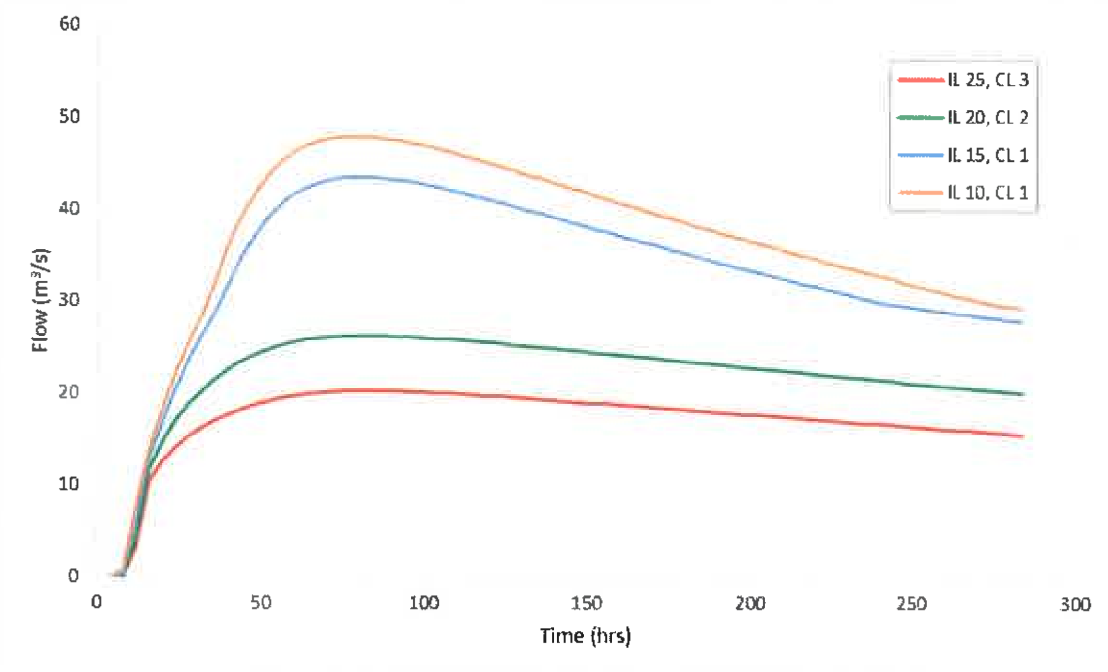


Figure 7-1 Variable Initial and continuing loss values - Rocklands Reservoir Outflow

The sensitivity testing to Rocklands Reservoir starting levels has shown that even using highly conservative starting water levels and low losses upstream of the reservoir, the peak flow likely to be generated from upstream of Rocklands Reservoir is only around one third of the 1% AEP flow at Fulham Bridge from Flood Frequency Analysis.

7.2.3 Hydraulics

To test the potential impact of Rocklands Reservoir spilling at the same time as a 1% AEP event occurring in the catchment area between Rocklands Reservoir and Harrow, the Harrow hydraulic model was run for the 1% AEP event plus an additional steady state flow of 61.3 m^3/s (5,300 ML/d). This is the same as the maximum overflow rate from Rocklands Reservoir, recorded in 1975. As discussed in Section 7.2.2, this is greater than the peak 1% AEP flow rate generated from a single event modelled in RORB with Rocklands Reservoir at 100% capacity at the beginning of the event and with very low rainfall losses of 10 mm initial loss and 1 mm/hr continuing loss. These circumstances are considered to have a probability far lower than a 1% AEP.

The difference in water levels and extent due to the additional steady state flow of 61.3 m^3/s (5,300 ML/d) are shown in Figure 7-2.

Figure 7-2 Difference in water level due to the 61.3 m^3/s Rocklands release depths at Harrow

Additional to a spill from Rocklands Reservoir, controlled releases from Rocklands Reservoir were also added as a steady state flow to the 1% AEP event. The maximum possible release rate, 14.5 m^3/s (1,250 ML/d) and a more standard release of 6.9 m^3/s were modelled as a steady state flow with the 1% AEP event hydrograph occurring concurrently. The differences in water level and extent due to the additional 14.5 m^3/s and 6.9 m^3/s are shown in Figure 7-3 and Figure 7-4.

Figure 7-3 Difference in water level due to the 14.5 m^3/s Rocklands release - Depths at Harrow

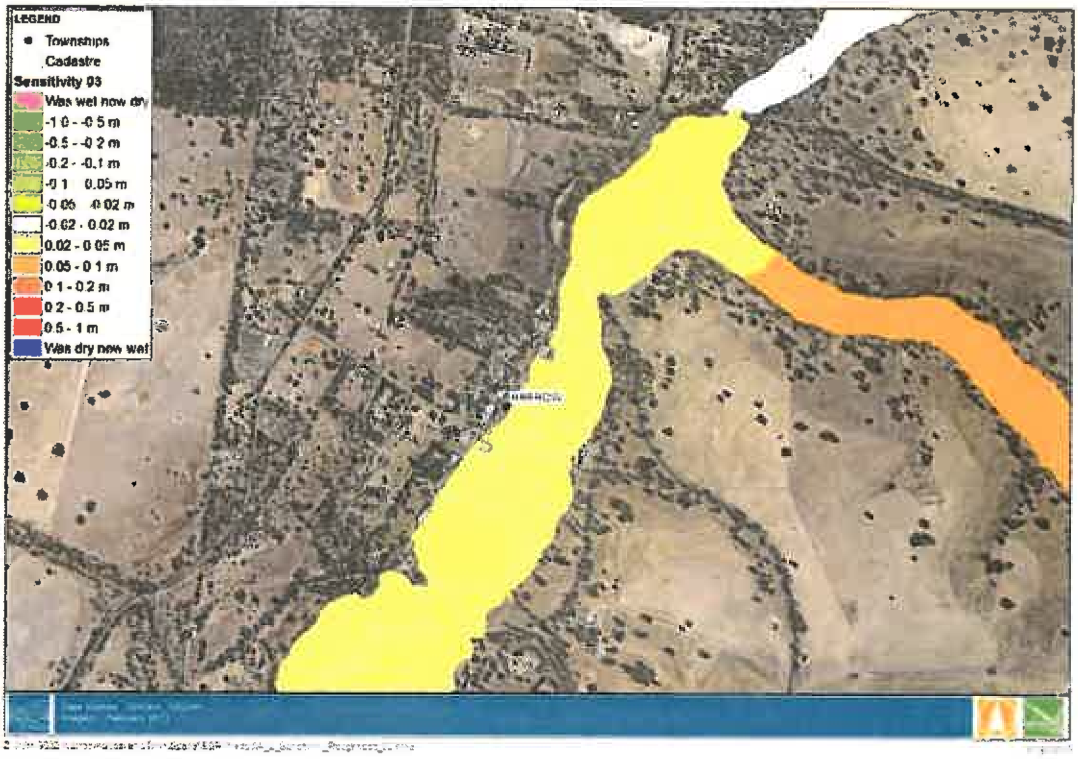


Figure 7-4 Difference in water level due to the 6.9 m^3/s Rocklands release - Depths at Harrow

7.2.4 Discussion

The inclusion of steady state flow additional to the design flows at Harrow has shown reasonable increases in water level but a very limited increase in inundation extent. This is similar to the increase between the design AEP events. A steady state flow of 61.3 m^3/s increased water levels in Harrow by around 0.3 m, while steady state flows of 14.5 and 6.9 m^3/s increased levels by 0.075 m and 0.03 m respectively. In the 6.9 m^3/s scenario there was no perceivable increase in inundation extent. This demonstrates that controlled releases are not likely to add significantly to natural flood levels at Harrow with the level of increase relatively minor.

7.3 Variable Roughness Coefficients

Variable roughness coefficients were used in the hydraulic model for the 1% AEP event to test their impact on water level. The Glenelg River channel roughness was found to have a significant impact on water levels during the calibration process, however, given there is limited ability to physically change the channel roughness it is unlikely to become a potential mitigation solution. During community consultation several community members voiced their concern that floodplain vegetation (all introduced species, predominantly phalaris) could "block flow" and cause increased flood levels. There is a current Glenelg River beautification project in Harrow which has been removing non-native species.

To test the impact of floodplain roughness on flood levels it was determined the potential to change the roughness through physical works and removal of non-native species would be approximated in the model.

The floodplain roughness determined during the calibration modelling process was a Manning's 'n' of 0.1. Two sensitivity tests were done; Scenario 1 - reducing the floodplain roughness to 0.03 (this roughness is equivalent to short grass[36]) this value is the lowest potential roughness for the Glenelg River floodplain and was used as a test not an indication of the what could be achieved. The Harrow community are very mindful of the Glenelg River's aesthetic appeal and the scenario was used to demonstrate a relatively limited impact even with the extreme example of removing all floodplain vegetation and the replacement with mown grass. Scenario 2 increased the roughness of all values by 10%.

The change in inundation extents and water levels as a result of the change in roughness for Scenario 1 and 2 are shown in Figure 7-5 and Figure 7-6 respectively.

[36] Chow (1959), Open Channel Hydraulics

Figure 7-5 **Change in water levels and extents due to an unrealistically decreased floodplain roughness**

Figure 7-6 **Change in water levels and extents due to a 10% increase to all roughness values**

7.3.1 Discussion

Roughness sensitivity modelling has shown floodplain roughness plays a very large part in water levels through Harrow. This is primarily due to the confined nature of the floodplain in this area.

A decrease in floodplain roughness from 0.1 to 0.03 has caused modelled water level changes of up to 0.8 m in the very confined areas upstream of Harrow, down to 0.07 m in the broader floodplain in Harrow. However, there is a limited change in inundation extent in any location. This decrease in roughness is physically unrealistic with the removal of all floodplain vegetation but demonstrates that a relatively small reduction of 0.07 m could be achieved.

A 10% increase in all roughness values has caused increases of 0.23 m upstream of Harrow and 0.14 m within Harrow. Similar to the decrease in roughness there is a limited change in inundation extent.

7.4 Blockage factors

ARR2016[37] provides guidance on blockage of hydraulic structures including determination of likely blockage levels and mechanisms. The guidelines provide a framework to assess the likelihood of blockage by assessing a series of factors.

These guidelines were used to assess the likelihood of blockage at the Glenelg River Bridge on the Coleraine-Edenhope Road and the Salt Creek Bridge on the Harrow Clear Lake Road, and the potential blockage percentage that could be used. The assessment criteria assigned ranking is shown in Table 7-2 and Table 7-3.

Table 7-2 Blockage assessment – Glenelg River

Assessment	Description	Outcome
Debris Type and Dimensions	Logs, sticks, branches	-
L_{10}	Average length of the longest 10% of the debris that could arrive at the site	3 m
Debris Availability	Thick vegetation, difficult to walk though, considerable fallen limbs	High
Debris Mobility	Medium response times, main debris source close to stream, steep debris source, streams frequently overtop their banks.	Medium
Debris Transportability	Wide stream, lots of meander	Medium
Site based Debris Potential (High/medium/low)	Based on Availability, Mobility and Transportability	High, Medium, Medium = DP Medium
AEP Adjusted Debris Potential	Observation of debris conveyed in streams strongly suggests a correlation between an event's magnitude and debris potential at a site	DPMedium and debris moving between 5% and 0.5% AEP event = Medium
Debris Blockage	Most likely inlet blockage	Medium AEP Adjusted Debris Potential and $W < L_{10} = 0\%$

[37] Engineers Australia (2016), Australian Rainfall and Runoff, Book 6, Chapter 6

Table 7-3 Blockage assessment – Salt Creek

Assessment	Description	Outcome
Debris Type and Dimensions	Logs, sticks, branches	-
L_{10}	Average length of the longest 10% of the debris that could arrive at the site	1.5 m
Debris Availability	Thick vegetation, difficult to walk though, considerable fallen limbs	Medium
Debris Mobility	Medium response times, main debris source close to stream, steep debris source, streams frequently overtop their banks.	Medium
Debris Transportability	Wide stream, lots of meander, lots of benches and bars to catch debris	Medium
Site based Debris Potential (High/medium/low)	Based on Availability, Mobility and Transportability	High, Medium, Medium = DP Medium
AEP Adjusted Debris Potential	Observation of debris conveyed in streams strongly suggests a correlation between an event's magnitude and debris potential at a site	DPMedium and debris moving between 5% and 0.5% AEP event = Medium
Debris Blockage	Most likely inlet blockage	Medium AEP Adjusted Debris Potential and $L_{10} \leq W \leq L_{10}$= 10%

The recommended debris blockage for the Glenelg River is 0% and Salt Creek 10%. As a sensitivity test 10% blockage was used to assess the sensitivity of a blockage at the Glenelg River and Salt Creek structures.

The change in water level due to blockage of the Glenelg River Bridge at the Coleraine Edenhope Road is shown in Figure 7-7.

Figure 7-7 Change in water level due to a 10% blockage of the Coleraine Edenhope Road

7.4.1 Discussion

10% blockage at the Glenelg River structure caused increases in modelled water level of less than 0.02 m, this is due to the size of the structure and available flow area. The modelled blockage at Salt Creek however, has caused increases of up to 0.06 m upstream of the structure. The effect of this decreases so that there is no change in level beyond 200m from the structure.

7.5 Climate change scenarios

7.5.1 Overview

The assessment of climate change was modelled in RORB for a range of rainfall intensity increases including 10 %, 20% and 30% to provide a range of potential flows that could occur at Harrow due to climate change.

The impacts of climate change were further tested using the hydraulic model using a 10% rainfall intensity increase. This was determined by using the prediction of a 5% rainfall intensity increase per degree of warming[38], and a scenario of 2°C of warming (i.e. 10% increase in rainfall intensity)[39].

[38] Engineers Australia (2014), Australian Rainfall and Runoff Discussion Paper: An Interim Guideline for Considering Climate Change in Rainfall and Runoff (Draft). Report No. ARR D3

[39] CSIRO. (2005). Climate Change in Eastern Victoria - Stage 1 Report: The effect of climate change on coastal wind and weather patterns. CSIRO.

The impact of climate change on flows was determined for the catchment area upstream and downstream of Fulham Bridge separately then modelled in the hydraulic model.

7.5.2 Hydrology

Upstream of Fulham Bridge

Given design flows for the catchment area upstream of Fulham Bridge was determined using FFA rather than a RORB model the following methodology for determining climate change sensitivity flows was used:

- Apply rainfall intensity increases to the RORB model developed during the Glenelg Regional Flood Mapping Project[1] using the 1% AEP, 30hr flood event
- Determine % increase in peak flow caused by each rainfall intensity increase
- Determine % increase in event volume caused by each rainfall intensity increase
- Apply the same % increases to the 1% AEP design event

The increase in peak flow and volume at Fulham Bridge for the 1% AEP event in each climate change sensitivity scenario is shown in Table 7-4.

Table 7-4 Climate change peak flow and volumes at Fulham Bridge

% increase in rainfall intensity	Fulham Bridge 1% AEP peak flow (m^3/s)	Fulham Bridge 1% AEP event volume (ML)
-	164	12,878
10 %	191 (16% increase)	16,268 (26% increase)
20 %	236 (44% increase)	20,470 (59% increase)
30 %	281 (71% increase)	24,275 (88% increase)

7.5.3 Downstream of Fulham Bridge

To determine the impact of climate change on the catchment area downstream of Fulham Bridge the RORB model was run for the 1% AEP event using increases to rainfall intensity of 10 %, 20 % and 30% as specified in the project brief. The inflows were routed through the 1D model to Harrow.

Table 7-5 Climate change peak flow at Harrow for the catchment area downstream of Fulham Bridge

% increase in rainfall intensity	Harrow catchment downstream of Fulham Bridge 1% AEP flow (m^3/s)
-	90
10 %	109 (21 %)
20 %	157 (74 %)
30 %	205 (127 %)

Hydraulics

The increase in flow at Harrow due to a 10% increase in rainfall intensity was modelled for the 1% AEP event, using the 30 hr event. The change in inundation extent and water levels is shown in Figure 7-8.

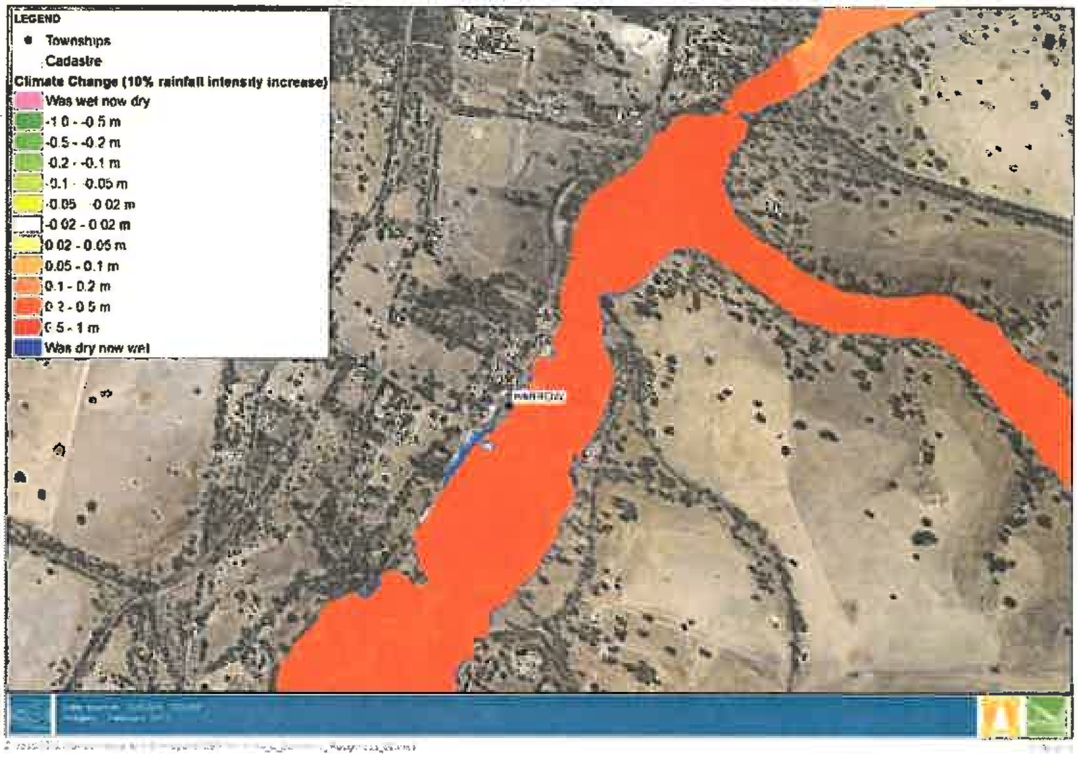


Figure 7-8 1% AEP - Change in water levels and extents due to climate change

7.5.4 Discussion

The increase in flows due to a 10% increase in rainfall intensity resulted in a 0.24 m increase in water level in the Harrow township. The highest water level increases within the hydraulic model were in the confined areas of the Glenelg River with up to 0.36m increases. As with the other sensitivity tests the inundation extent did not increase significantly, one additional building was flooded above floor and the depth of above floor flooding at was increased by around 0.2 m.

8. MITIGATION

8.1 Overview

Flood risk and flood damages in Harrow can be reduced via both structural and non-structural mitigation. Non- structural mitigation measures ensure that development doesn't occur in high flood risk areas and that the community is aware of the potential impact a given flood may have and how best to be prepared. Structural mitigation options are engineering solutions focused on reducing flood extent, depth and damage.

The 1% AEP flood inundation extent and properties flooded below and above floor for Harrow are shown in Figure 8-1.

Figure 8-1 Harrow - 1% AEP flood extent

8.2 Non-Structural Mitigation Options

8.2.1 Overview

There are a range of non-structural mitigation options possible to reduce flood damages, these include:

- Land use planning;
- Flood warning and response; and,
- Flood awareness.

During this project, sub-consultants Planning and Environmental Design and Molino Stuart were engaged to assist with reviewing the current non-structural flood mitigation arrangements for the land use planning and flood warning, response and awareness respectively.

The below sections summarise their individual reports, if further detail is required, please refer to:

- Planning and Environmental Design (2016), Planning Scheme Amendment Documentation – Harrow Flood Investigation
- Molino Stewart (2016), Harrow Flood Investigation - Flood Warning Assessment and Recommendations Report

8.2.2 Land Use Planning

The Victoria Planning Provisions (VPPs) contain a number of controls that can be employed to provide guidance for the use and development of land that is affected by inundation from floodwaters. These controls include the Floodway Overlay (FO), the Land Subject to Inundation Overlay (LSIO), the Special Building Overlay (SBO), and the Urban Floodway Zone (UFZ).

Section 6(e) of the Planning and Environment Act 1987 enables planning schemes to 'regulate or prohibit any use or development in hazardous areas, or areas likely to become hazardous'. As a result, planning schemes contain State planning policy for floodplain management requiring, among other things, that flood risk be considered in the preparation of planning schemes and in land use decisions.

Guidance for applying flood controls to Planning Schemes is available from the Department of Environment, Land, Water and Planning's (formerly Department of Planning and Community Development's (DPCD)) Practice Note on Applying Flood Controls in Planning Schemes.

Planning Schemes can be viewed online at http://services.land.vic.gov.au/maps/pmo.jsp. It is recommended that the planning scheme for this project's study area is amended to reflect the flood risk identified by this project.

This study has produced draft LSIO and FO layers for inclusion in the West Wimmera Shire Council Planning Scheme. The LSIO is representative of the 1% AEP extent of inundation, while FO represents a higher degree of flood risk combining 1% AEP flood depths and velocities. As specified by Glenelg Hopkins CMA the FO was defined by depths greater than 0.5 m and a velocity depth product greater than 0.4 m^2/s. Figure 8-2 shows the proposed FO for the entire study area, with a closer perspective of the central Harrow township shown in Figure 8-3.

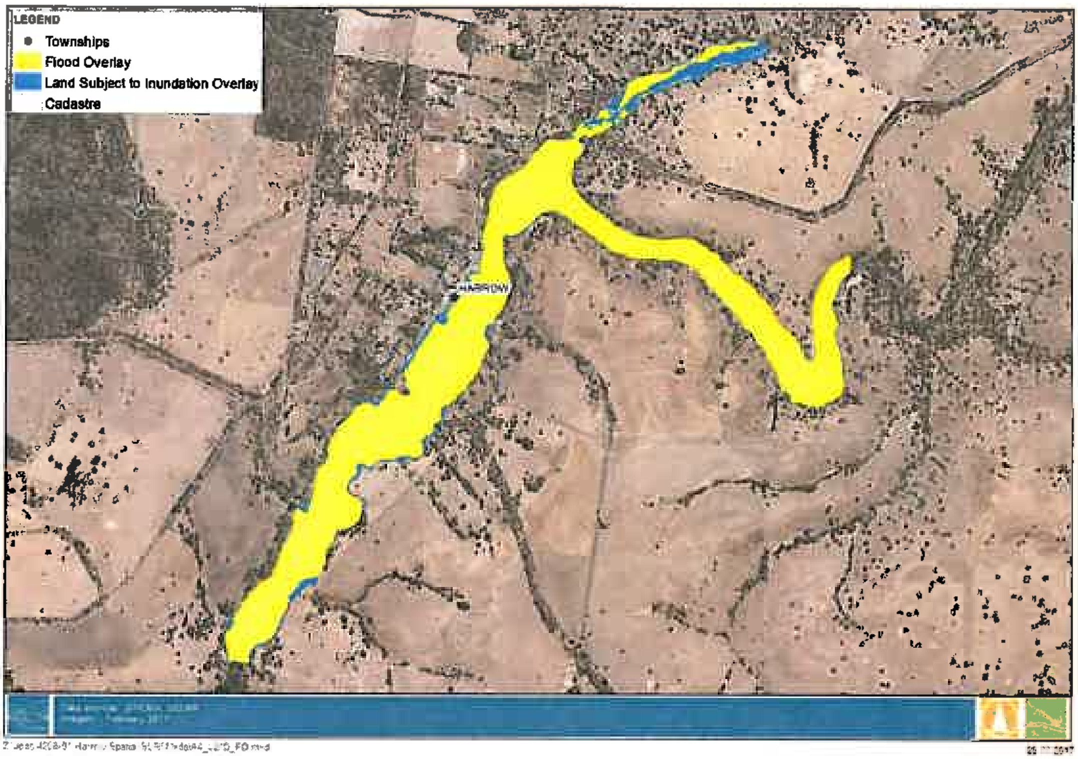


Figure 8-2 Flood Overland and Land Subject to Inundation Overlay covering the study area

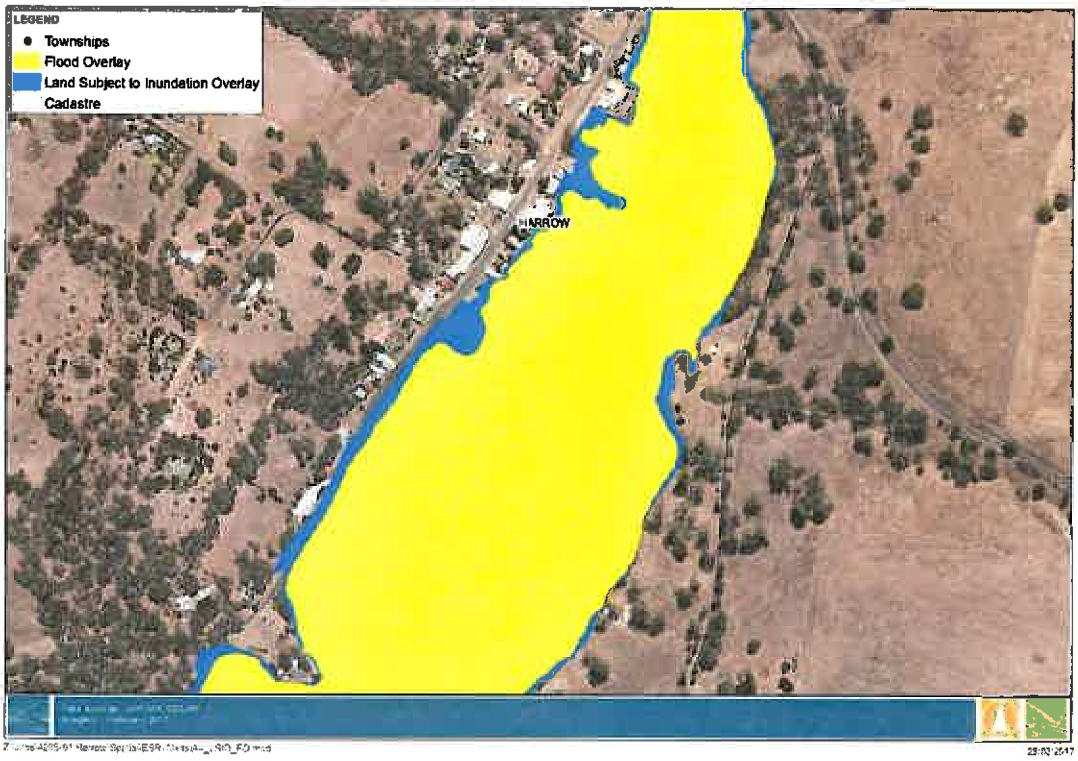


Figure 8-3 Flood Overland and Land Subject to Inundation Overlay in the central Harrow area

8.2.3 Flood Warning Recommendations

An objective of the Harrow Flood Investigation was to identify options for improved flood warning arrangements. elow is a summary of the full Harrow Flood Investigation – Total Flood Warning Assessment[40]. The review and identification of options for improvement was carried out during the study by:

- Assessing the area's flood warning service needs; and,
- Assessing the potential benefits of a Total Flood Warning System (TFWS) to reduce flood impacts for the community.

Molino Stewart was commissioned by Water Technology to conduct this part of the investigation. Consultation with stakeholders including the Victoria State Emergency Service (VICSES), Glenelg Hopkins Catchment Management Authority and West Wimmera Shire Council was undertaken. Data from the hydrology and hydraulics components of the flood investigation conducted by Water Technology was also used, along with demographic data sources such as the Australian Bureau of Statistics.

The review identified Harrow has a local streamflow gauge (Glenelg River at Harrow) and an upstream streamflow gauge (Glenelg River at Fulham Bridge) that provides ample warning lead time for flooding in the township. Along with the existing flood warning services provided by the BoM and VICSES and the existence of a CFA brigade to support emergency response, the existing configuration allows for the basis of a robust TFWS for Harrow.

However, the review identified some gaps and issues in the current warning provision for Harrow. It recommended the addition of the following components to enable an effective TFWS configuration:

1. The BoM consider enabling the streamflow gauges at Fulham Bridge and Harrow to have flood class levels and that this data is available online.
2. Crowdsourcing system for Salt Creek involving adjacent landholders requiring the installation of gauge boards as reference points. This would remove the uncertainty surrounding the potential contribution of flows in Salt Creek.
3. The preparation of a Municipal Flood Emergency Plan for Harrow based on the Flood Intelligence Cards produced as part of the flood investigation and detailed in this report.
4. An emergency flood plan for the Harrow RSL club which can experience above-floor flooding.
5. Involvement of the local CFA brigade in community preparedness education for flooding, helping the RSL club with sandbagging and doorknocking to support Harrow residents as a flood progresses.
6. Support for vulnerable people in the community particularly to stock up on food, water and medicines.
7. Community participation in the review and integration of the Harrow TFWS components.

A benefit-cost analysis was conducted for these additional components giving a ratio of 0.84, with the main benefits to people's safety, which were not factored into this analysis.

8.3 Structural Mitigation

8.3.1 Overview

A list of structural mitigation options was developed during community meetings, Project Steering Committee meetings and general discussion. Mitigation options were focused on the Harrow

[40] Molino Stewart (2017), Harrow Flood Investigation – Total Flood Warning System Assessment

township. Each and every mitigation option suggested over the course of the project was assessed based on its potential to reduce flood damages.

Given the number of mitigation options suggested the mitigation assessment was separated into five stages, these were as follows:

- Prefeasibility Assessment - to determine the potential for a mitigation option to reduce flood damage at reasonable cost and feasibility
- Detailed Hydraulic Modelling Assessment - to determine what reduction in flood levels and extents could be achieved
- Damages Assessment – to determine the reduction in damages that could be achieved by the chosen mitigation options
- Cost Benefit Analysis – to compare the reduction in flood damage and costs of the chosen mitigation options over a period of time to assess the economic performance of the options
- Concept design of the recommended mitigation option.

The following sections document each of these stages.

8.3.2 Prefeasibility Assessment

Overview

Each option was assessed to determine its feasibility and to highlight any property which may be negatively impacted by the construction of the option. Mitigation solutions using changes to existing infrastructure as well as construction of new infrastructure were suggested. The suggested mitigation measures are summarised below in Table 8-1.

Table 8-1 Suggested mitigation options

Option No.	Detail	Source
1	Ensure no environmental flow releases are occurring at the same time as an expected flood event	Community
2	Extract sand "chokes" from the Glenelg River	Community
3	Remove vegetation (weeds – phalaris) from the floodplain	Community
4	Put an embankment upstream of Harrow controlling the flow to a rate which doesn't cause damage	Community
5	Build/alter the levee around John Mullagh Memorial Park to the same height of the road	Steering Committee
6	Build a levee to protect the township along the back of the buildings	Community
7	Remove a choke downstream of Harrow at Deep Creek	Community
8	Build levees/raised garden beds to protect individual properties	Water Technology

Assessment Criteria

Each mitigation option was assessed against four criteria; potential reduction in flood damage, cost of construction, feasibility of construction and environmental impact. The score for each criteria was based on a ranking system of 1 to 5, with 1 being the worst score and 5 the best. Each criteria score was then weighted according to the weighting shown in

Table 8-2 below. The reduction in flood damage was the most heavily weighted criteria as this is the main objective for all flood mitigation. Table 8-3 reviews and scores each mitigation option against the four criteria and calculates a total score for each option. The options with the higher scores indicate the more appropriate mitigation solutions for each location. While these options were reviewed and recorded individually, it is important to consider a combination of options when developing a flood mitigation scheme.

Table 8-2 Prefeasibility assessment criteria

Score	Reduction in Flood Damages	Cost ($)	Feasibility/Constructability	Environmental Impact
Weighting	**2**	**1**	**0.5**	**0.5**
5	Major reduction in flood damage	Less than $50,000	Excellent (Ease of construction and/or highly feasible option)	None
4	Moderate reduction in flood damage	$50,000 – $100,000	Good	Minor
3	Minor reduction in flood damage	$100,000 – $500,000	Average	Some
2	No appreciable reduction in flood damage	$500,000 – $1,000,000	Below Average	Major
1	Increase in flood damage	Greater than $1,000,000	Poor (No access to site and/or highly unfeasible option)	Extreme

Assessment

Each of the suggested mitigation options was assessed using the outlined assessment criteria above, and is discussed in Table 8-3.

Table 8-3 Prefeasibility assessment criteria

No.	Mitigation Option	Criteria					Score
		Damages Reduction	Cost ($)	Feasibility	Enviro. Impact	Comments	
1	Ensure no environmental releases are occurring at the same time as an expected flood event	2	5	5	5	There were several sensitivity runs completed during the hydraulic modelling component of this study, two of these options assessed the potential impact of an environmental release occurring at the same time as a 1% AEP flood event. The release rates used during the analysis were 1,250 ML/d and 600 ML/d, these releases caused increased water levels of 0.08 m and 0.03 m respectively with a marginal impact on inundation extent in both scenarios. Figures of the change in water level are shown in Figure 7-3 and Figure 7-4. Ensuring environmental releases do not occur at the same time as a flood event should be relatively easy to achieve for virtually no cost.	15
2	Extract sand "chokes" from the Glenelg River	2	3	2	3	Sand accumulation is common in the Glenelg River due to the nature of the catchment and the previous land management practices. There have been several sand surveys completed in the Glenelg River with sand extraction being completed at several locations in the Glenelg River catchment on a commercial basis at the time of this reports production. Sand extraction requires significant earth moving equipment and is likely to cause a reasonable amount of environmental damage in the short term. In general, sand extraction creates pools and riffles that may	9.5

No.	Mitigation Option	Criteria					Score
		Damages Reduction	Cost ($)	Feasibility	Enviro. Impact	Comments	
						not necessarily increase the capacity of the channel, i.e. some areas might be increased, others might not. The capacity of the Glenelg River channel is relatively minor by comparison to the floodplain at Harrow as a whole.	
3	Remove vegetation (weeds – phalaris) from the floodplain	3	5	5	5	There is a significant amount of vegetation on the floodplain upstream of Harrow and in some areas through Harrow, however there has been recent efforts made to remove weeds and replace them with native grasses and shrubs. During roughness sensitivity modelling completed as part of this project a reduction in Manning's 'n' roughness from 0.1 to 0.03 was made. This is equivalent to reducing the existing vegetation to no trees and short grass. This caused a 6.5 cm reduction in water levels during the 1% AEP event with a negligible impact on the inundation extent. The change in water level due to the reduction in floodplain roughness is show in Figure 7-5 in Appendix A.	16
4	Put an embankment upstream of Harrow controlling the flow to a rate which doesn't cause damage.	5	2	1	2	There is a constructed area of floodplain on the Glenelg River approximately 8km upstream of Harrow. An embankment could be placed in this section of the river with an opening wide enough to allow low flows but preventing large enough flows to flood Harrow through. There is a likelihood some vegetation would need to be removed to	13.5

No.	Mitigation Option	Criteria					Score
		Damages Reduction	Cost ($)	Feasibility	Enviro. Impact	Comments	
						construct the embankment and ensuring the structural integrity of the embankment would be extremely important to ensure there was no risk of failure potential worsening inundation impact in Harrow. Causing a restriction to flow might also limit the potential/frequency of floodplain inundation in areas further downstream of Harrow. The option is likely to be relatively expensive.	
5	Build/alter the levee around John Mullagh Memorial Park to the same height of the road	4	5	5	5	Construction/modification of the levee around the John Mullagh Memorial Park would prevent inundation of the oval and assets immediately surrounding the oval. There may be a reasonable decrease in flood damage due to potential damage to the oval's playing surface. Cost would be relatively low given there is a levee currently in place. There may be some impact to upstream flood levels with potential restriction of the floodplain flow area.	18

No.	Mitigation Option	Criteria					Score
		Damages Reduction	Cost ($)	Feasibility	Enviro. Impact	Comments	
6	Build a levee to protect the township along the back of the buildings	5	5	3	5	A levee behind the buildings on the southern side of Blair Street a levee could protect all the buildings subject to below and above floor inundation. Given the buildings are higher on the floodplain than the potential levee location the levee may need to be relatively high. There would be a reasonable decrease in flood damages as a result of this option but at a reasonable cost. The environmental impact of the levee is not considered to be high, and there is limited potential for water levels to be increased elsewhere as a result of the levee. The potential for the option to be adopted would be largely dependent on the impact to the aesthetic appeal of the immediate area and view of the Glenelg River.	19
7	Remove a choke downstream of Harrow at Deep Creek	2	4	3	2	There is a constriction in the Glenelg River floodplain around 5.2 km downstream of Harrow at Deep Creek, the suggested mitigation option is to remove this restriction to flow and allow water faster down the Glenelg River. The choke is likely to be a result of deposition from Deep Creek catchment erosion accumulating over time. Given the distance downstream from Harrow any backwater from the Deep Creek area is likely to have dissipated. .	10.5

No.	Mitigation Option	Criteria					Score
		Damages Reduction	Cost ($)	Feasibility	Enviro. Impact	Comments	
8	Build levees/raised garden beds to protect individual properties	4	4	4	5	The use of individual property protection could be an effective way to decrease damages for properties that want it. This could be in the form of individual levees, raised garden beds or concrete fencing.	18.5

Using the prefeasibility assessment above, the eight mitigation options were ranked by weighted score. Their ranking is shown below in Table 8-4

Table 8-4 Weighted prefeasibility mitigation scores

Rank	Option No.	Mitigation Option	Weighted Score
1	6	Build a levee to protect the township along the back of the buildings	19
2	8	Build levees/raised garden beds to protect individual properties	18.5
3	5	Build/alter the levee around John Mullagh Memorial Park to the same height of the road	18
4	3	Remove vegetation (weeds – phalaris) from the floodplain	16
5	1	Ensure no environmental releases are occurring at the same time as an expected flood event	15
6	4	Put an embankment upstream of Harrow controlling the flow to a rate which doesn't cause damage.	13.5
7	7	Remove a choke downstream of Harrow at Deep Creek	10.5
8	2	Extract sand "chokes" from the Glenelg River	9.5

Discussion/Recommendations

The ranking showed construction of a levee around the back of the buildings along Blair Street, individual property flood protection and improving/increasing the height of the levee around John Mullagh Memorial Park as the most feasible options. All three have the potential to adversely impact surrounding properties, and require detailed flood modelling to demonstrate potential flood level increases due to the impediment to flood flow and design levee height

Other high ranking options were: ensuring no environmental releases occurred concurrently with a flood event and removing weeds from the floodplain. Both these options have been modelled previously during sensitivity testing and their potential impact is well understood.

The remaining three options are not considered to be viable for mitigation in Harrow due to the level of risk or lack of potential damage reduction.

It was determined that Options 6 (levee to the back of Blair Street properties) and 5 (John Mullagh Levee) all be modelled to demonstrate their viability and that discussion of the existing model results be used to assess the remaining options. Option 8 was not assessed as modelling of Option 6 will show the maximum potential afflux that could be caused by levees in this location and the exact nature of property specific protection is unknown. The hydraulic modelling completed is included in the following section.

8.3.3 Hydraulic Modelling

Hydraulic modelling was completed of the following mitigation options:

- Option 6 - Levee constructed behind the buildings to the south of Blair street.
- Option 5 - Increase the levee height around the John Mullagh Oval

The options were assessed using the calibrated hydraulic model to determine their impact on the properties they protect and those that remain unprotected.

The proposed levee alignments are displayed over the 1% AEP flood extent as modelled under existing conditions in Figure 8-4.

Figure 8-4 Assessed levee alignments in Harrow

Option 5 - Buildings Levees

Two levees were included into the hydraulic model to a height greater than the existing 1% AEP level flood levels. The modelling was used to determine the extent of potential adverse water level increases.

The addition of the two levees removed inundation from behind properties along Blair Street. The levee scenario was modelled using the 1% AEP flood event, the modelled extent and depths in proximity to the levee is shown in Figure 8-5. Figure 8-6 and Figure 8-7 show the change in water level as the result of including the north and south levees respectively.

Very little change to water levels upstream and downstream of the levee was observed, with a small increase on the upstream side of each levee. There were no flood level increases on developed blocks. The levee alignment provides complete protection for the houses behind the levee without increasing the risk of inundation for any surrounding properties.

Figure 8-5 Buildings Levee Alignment and 1% AEP depths

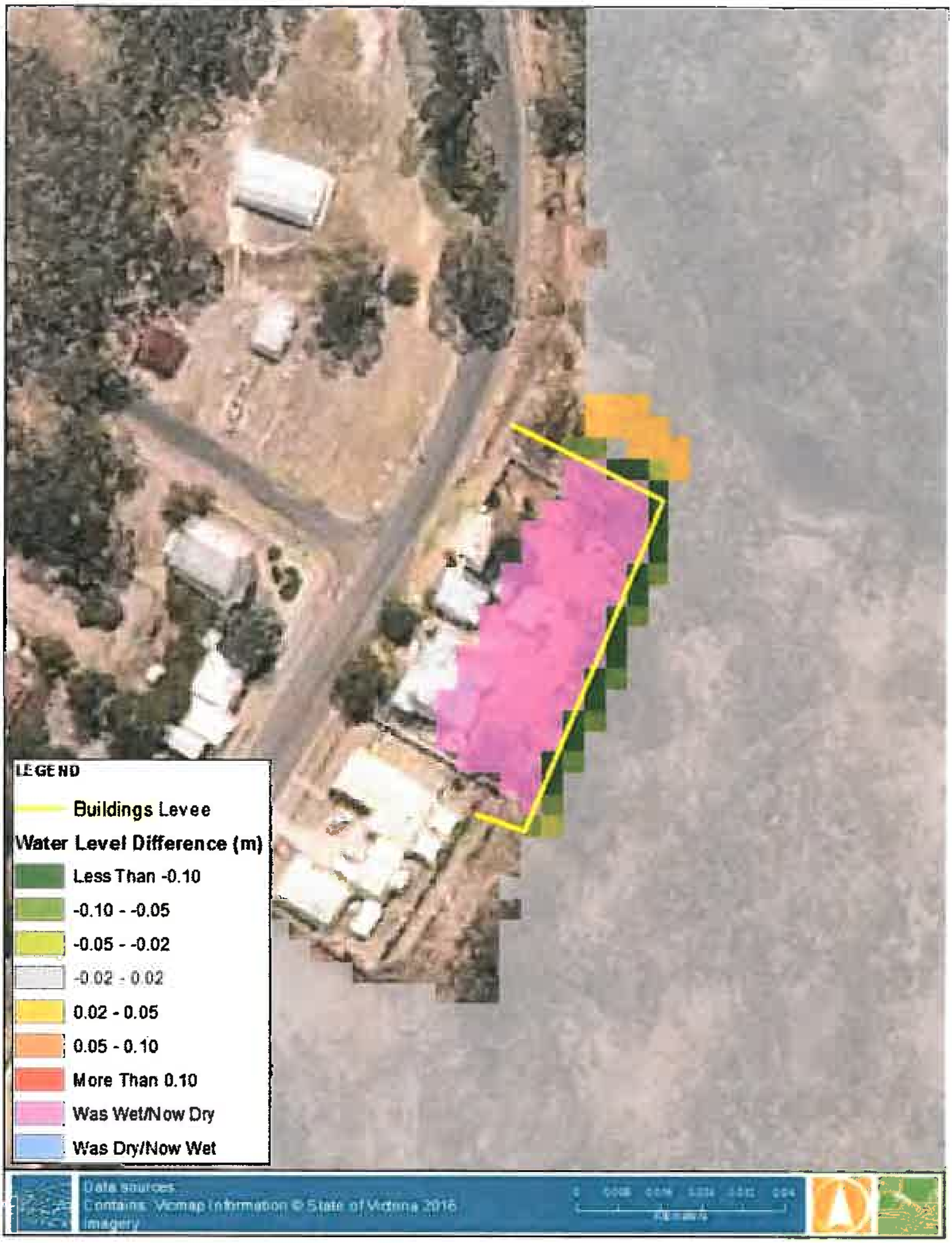


Figure 8-6 North Buildings Levee Alignment and Water Level Difference

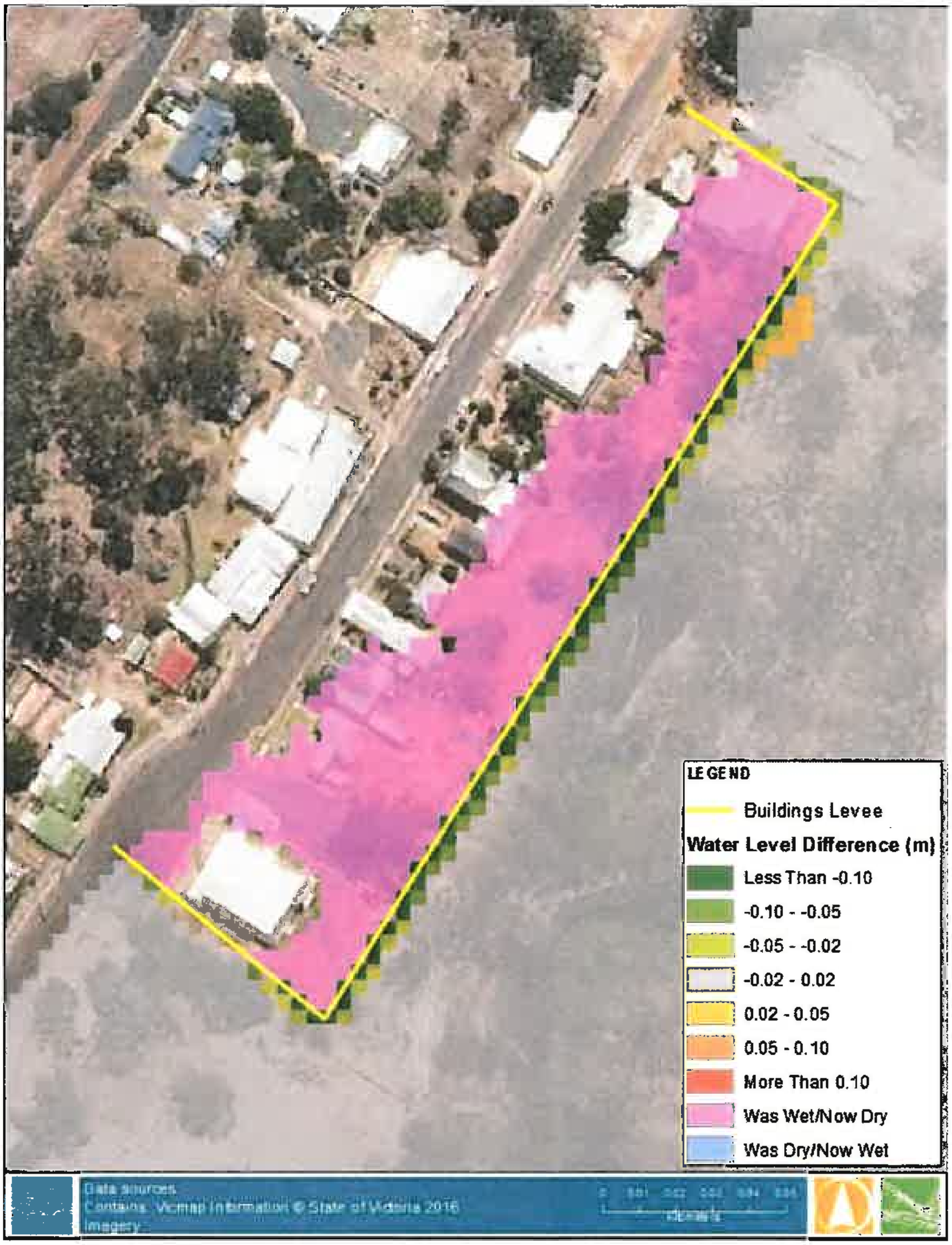


Figure 8-7 North Buildings Levee Alignment and Water Level Difference

Option 6 - John Mullagh Memorial Park Levee

The existing levee at the John Mullagh oval does not sufficiently protect the oval from inundation during 20% AEP events or greater. To assess the impact of protecting the oval against flood events the levee was modelled increasing it to above the 1% AEP flood level.

The levee increase was modelled for a 1% AEP flood event, the resulting depth and extent of inundation is shown in Figure 8-8, with the change in water levels as a result of the levee' construction shown in Figure 8-9.

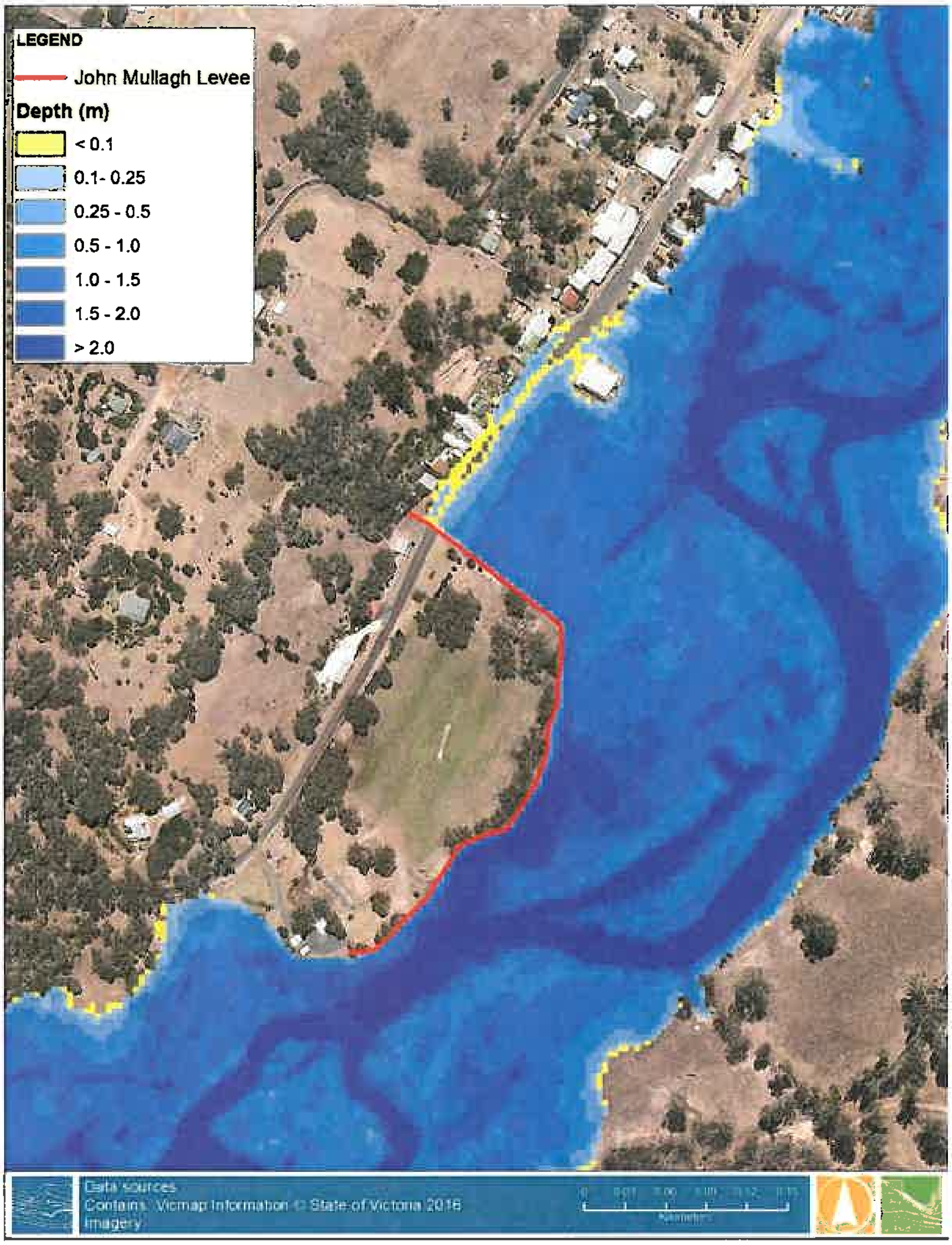


Figure 8-8 John Mullagh Oval Levee Option A Alignment and 1% AEP Depths

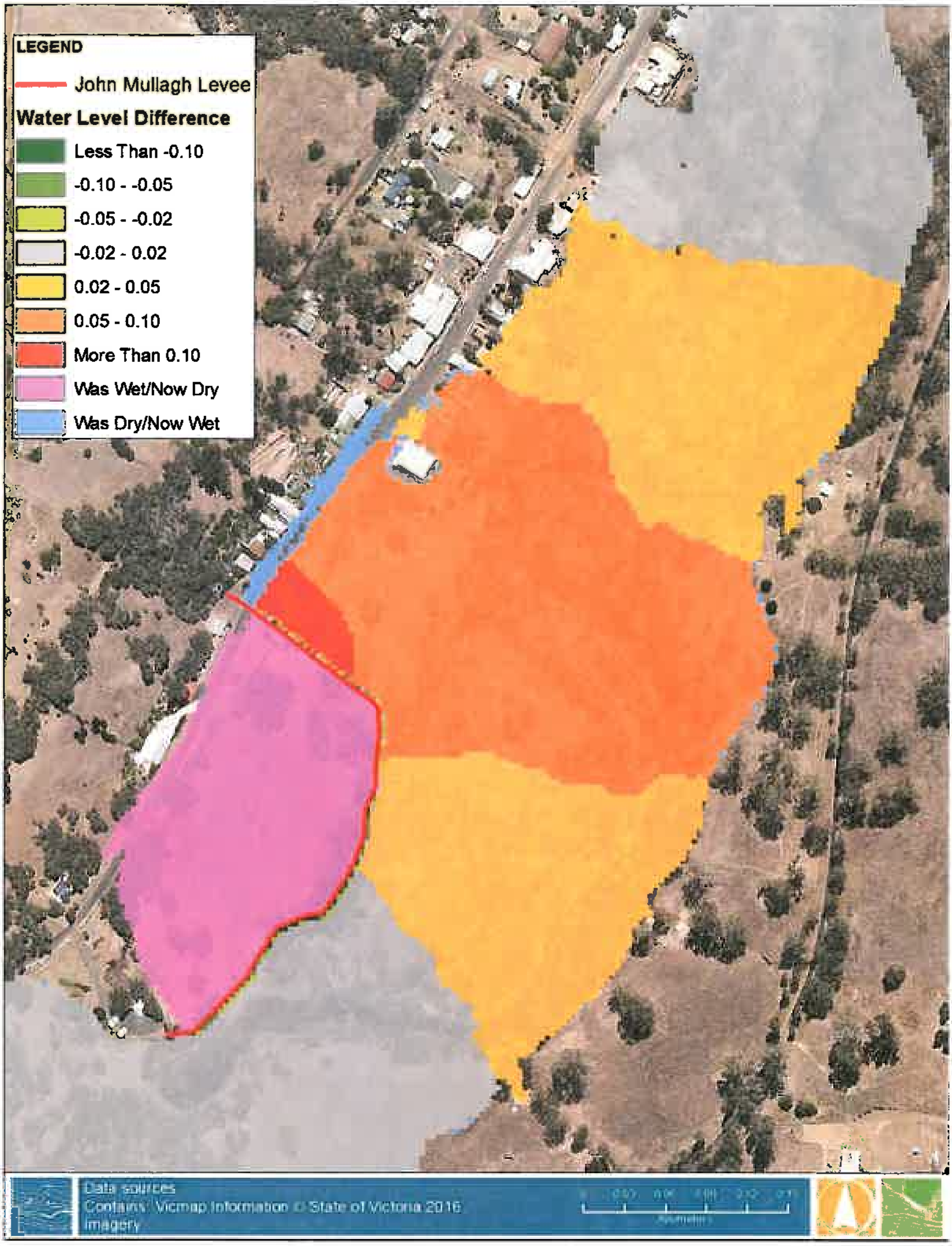


Figure 8-9 John Mullagh Oval Levee Option A, Change in Water Level from Existing Conditions

Results show the levee caused increased water levels for some distance upstream, impacting on buildings already inundated above and below floor

To reduce the impact of the levee a lower levee crest height was trialled, reducing the level of protection to a 5% AEP flood event. This was discussed with the community and would ensure that on average the oval would only be inundated once every 20 years, rather than more than once every 5 years in the existing scenario.

The model was re-run for the 1% AEP flood event, allowing the levee to overtop. The modelled depths are shown in Figure 8-10 with the change in water levels as a result of the levee shown in Figure 8-11.

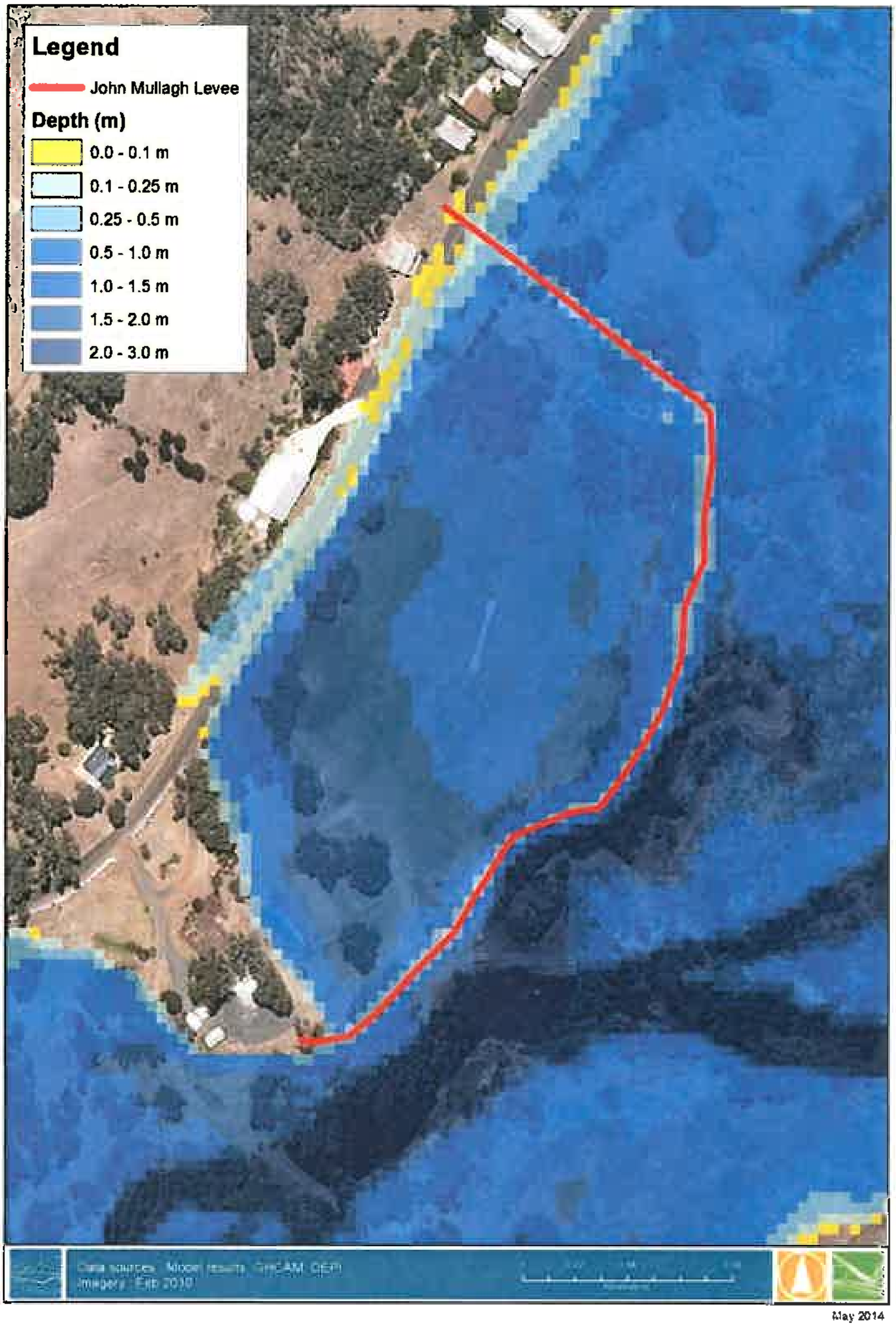


Figure 8-10 Water depths at John Mullagh Oval – 5% AEP protection

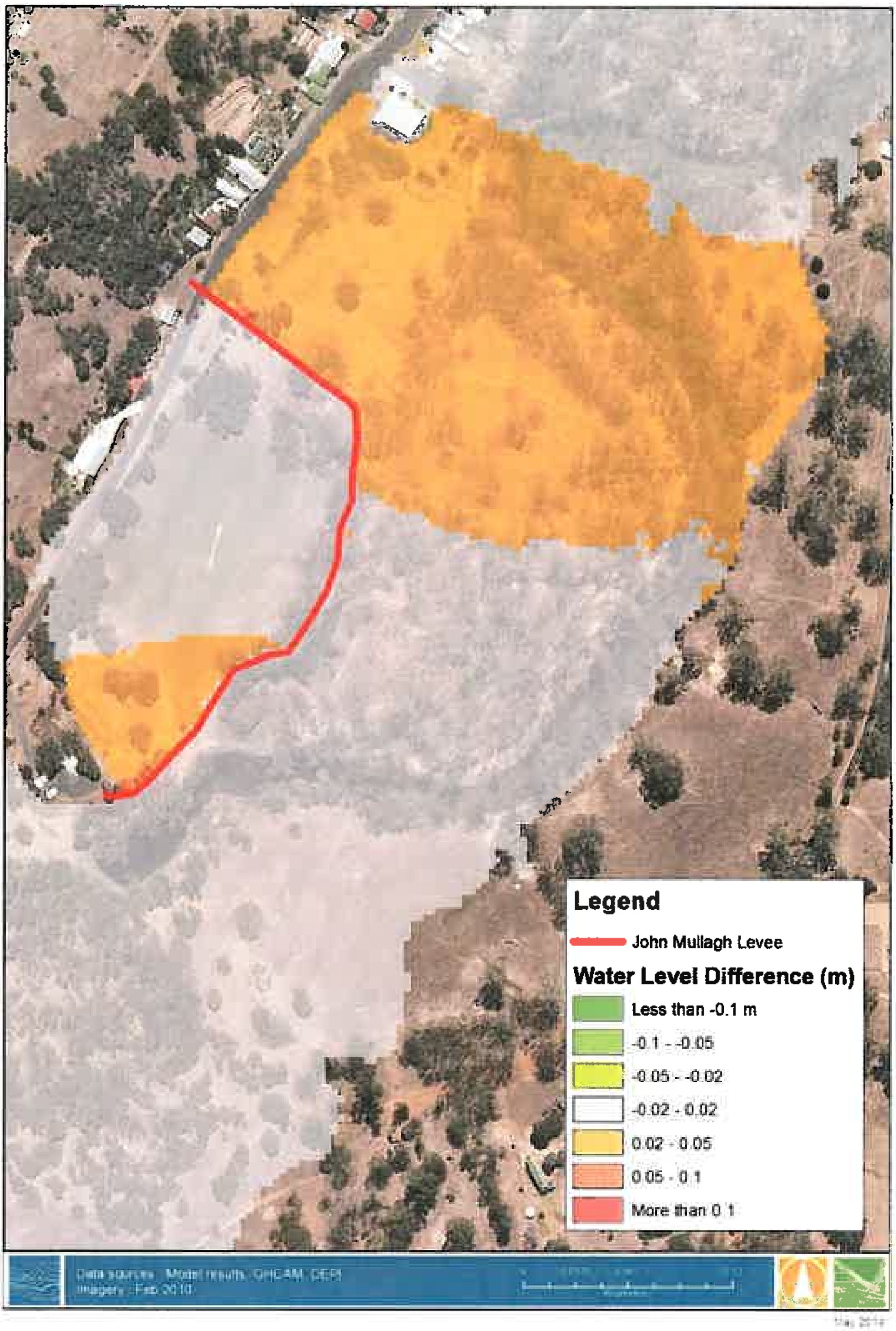


Figure 8-11 1 % AEP change in Water Level due to John Mullagh Oval Levee – 5% AEP protection

Mitigation Option Cost

Water Technology has undertaken many levee functional designs and costings, we have developed standard spreadsheets based on industry rates from Melbourne Water and Rawlinsons. A 30% contingency cost was included along with engineering and administration costs. It should be noted that these costs are based on estimated rates and should be checked during the detailed design phase.

The Victorian Levee Guidelines has standard recommendations for levee crest width (2 m), batter slopes (3:1 batter on water side, 2:1 on dry side) and clay core with cut-off trench requirements. The levee proposed meets these requirements with a 2m crest width, 3:1 batter slopes on both sides.

The buildings levee was designed to the 1% AEP level with the inclusion of a 300mm earthen freeboard.

The John Mullagh levee was increased to the height of 100.04 m AHD, matching the 5% AEP flood event level.

The costing rates were based on several references, including:

- Melbourne Water rates for earthworks and pipe construction costs;
- Melbourne Water rates for land acquisition; and
- Comparison to cost estimates for similar works for other flood studies.

An annual maintenance cost (3% of the total construction cost) was factored in for levee works. The cost of the levee has been separated into permanent and temporary portions. Permanent portions were costed with the inclusion of a clay core and cut-off trench, while temporary sections of levee were costed based on standard levee construction rates excluding topsoiling and grassing.

The estimated capital cost of sections of levee protecting the township (Option 5), was $101,000. The estimated cost of the increasing the John Mullah Memorial Park levee is $60,220. The breakdown of these estimates is shown in Table 8-5 and Table 8-6.

Table 8-5 Levee protecting the Harrow township – Option 5

Levee section	Length (m)	Average height (m)	Volume (m^3)	Estimated Construction Cost	Estimated Annual Maintenance Cost
Northern Levee	120	1.2	758	$32,441	$597
Southern Levee	391	1	1554	$68,738	$1,265
Sub-total 'A'				$62,090	
'A' x Engineering Fee @ 15%				$9,313	
Sub-total 'B'				$71,403	
'B' x Administration Fee @ 9%				$6,426	
Sub-total 'C'				$77,830	
'A' x Contingencies @ 30%				$23,349	
FORECAST EXPENDITURE				$101,179	$1,862

Table 8-6 Levee protecting the John Mullugh Memorial Park

Levee section	Length (m)	Average height (m)	Volume (m^3)	Estimated Construction Cost	Estimated Annual Maintenance Cost
Oval Option B	370	1.1	1334	$60,220	$1,109
Sub-total 'A'				$36,955	
'A' x Engineering Fee @ 15%				$5,543	
Sub-total 'B'				$42,498	
'B' x Administration Fee @ 9%				$3,825	
Sub-total 'C'				$46,323	
'A' x Contingencies @ 30%				$13,897	
FORECAST EXPENDITURE				$60,220	$1,109

8.3.4 Flood Damages Assessment

Overview

A flood damage assessment for the study area was undertaken using the range of design events modelled (20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP design events) for existing conditions. The damage assessment was used to determine the monetary flood damage for the design floods.

The flood damages assessment was also undertaken with the inclusion of the township levees (Option 5), to determine the potential reduction in damage that could result due to their construction.

Water Technology has developed an industry best practice flood damage assessment methodology that has been utilised for a number of studies in Victoria, combining aspects of the Rapid Appraisal Method, ANUFLOOD and other relevant flood damage literature. The NSW Office of Environment and Heritage stage damage curves are utilised, which represent far superior damage estimates at low depths above floor and below floor than earlier stage damage curves. Water Technology utilises WaterRide to undertake the property inspection and apply the appropriate stage damage curves.

The model results for all mapped flood events were processed to calculate the numbers and locations of properties affected. This included properties with buildings inundated above floor, properties with buildings inundated below floor and properties where the building was not impacted but the grounds of the property were. In addition to the flood affected properties, lengths and damages of flood affected roads for each event were also calculated.

The Average Annual Damage (AAD) was determined as part of the flood damage assessment. The AAD is a measure of the flood damage per year averaged over an extended period. This is effectively a measure of the amount of money that must be put aside each year in readiness for when a flood may happen in the future.

Existing Conditions

The flood damage assessment for existing conditions is shown below in Table 8-7. The Average Annual Damages (AAD) for existing conditions is estimated at approximately **$28,000.**

Mitigation Options/Package

Two levees protecting the buildings south west of Blair Street was used for an assessment of the potential a reduction in flood damages. The levees prevent all above floor and below floor inundation within the township during the 1% AEP flood event. This option was not generally supported by the community but it was determined a better understanding of the potential reduction in flood damage was necessary. The levee around the John Mullagh Memorial Park was not assessed in terms of its reduction to flood damages because of the lack of data available assess damages to the oval and impact on community. Generally, the damage is repaired through volunteer efforts which is largely undocumented.

The flood damage assessment for the Combined Mitigation Package within the Harrow township is shown below in Table 8-5. The Average Annual Damages (AAD) for existing conditions is estimated at approximately **$22,000.**

Non-economic Flood Damages

The previous discussion relating to flood damages has concentrated on monetary damages, i.e. damages that are easily quantified. In addition to those damages, it is widely recognised that individuals and communities also suffer significant non-monetary damage, i.e. emotional distress, health issues, etc.

There is no doubt that the intangible non-monetary flood related damage in and along the Glenelg River is high. The benefit-cost analysis presented in this report does not factor in this cost. Any decisions made that are based on the above benefit cost ratio need to understand that the true cost of floods in and along the Glenelg River is far higher than the economic damages alone. These intangible costs increase the benefit-cost ratio, improving the argument for approving a mitigation scheme at Harrow.

Table 8-7 Existing conditions damages

ARI (years) / AEP	500y 0.2	200yr 0.5%	100yr 1%	50yr 2%	20yr 5%	10yr 10%	5yr 20%
Residential Buildings Flooded Above Floor	0	0	0	0	0	0	0
Commercial Buildings Flooded Above Floor	3	2	2	2	0	0	0
Properties Flooded Below Floor	35	26	26	26	28	27	26
Total Properties Flooded		**296**	**180**	**81**	**9**	**6**	**4**
Direct Potential External Damage Cost	$306,859	$308,886	$304,114	$299,515	$297,185	$270,523	$230,001
Direct Potential Rural Damage Cost	$15,399	$15,043	$14,943	$14,797	$14,506	$14,100	$13,238
Direct Potential Residential Damage Cost	$0	$0	$0	$0	$0	$0	$0
Direct Potential Commercial Damage Cost	$91,300	$46,396	$36,411	$21,884	$0	$0	$0
Total Direct Potential Damage Cost	**$413,558**	**$370,325**	**$355,468**	**$336,196**	**$311,691**	**$284,623**	**$243,239**
Total Actual Damage Cost *(0.8*Potential)*	**$330,846**	**$296,260**	**$284,375**	**$268,957**	**$249,353**	**$227,698**	**$194,591**
Infrastructure Damage Cost	**$73,337**	**$55,006**	**$49,070**	**$41,184**	**$29,585**	**$26,547**	**$21,892**
Total Cost	**$404,183**	**$351,266**	**$333,445**	**$310,142**	**$278,938**	**$254,246**	**$216,483**
Average Annual Damage (AAD)	**$28,229**						

Table 8-8 Mitigation damages – Option 5

ARI (years) / AEP	500y 0.2	200yr 0.5%	100yr 1%	50yr 2%	20yr 5%	10yr 10%	5yr 20%
Residential Buildings Flooded Above Floor	0	0	0	0	0	0	0
Commercial Buildings Flooded Above Floor	2	0	0	0	0	0	0
Properties Flooded Below Floor	34	23	23	23	23	23	23
Total Properties Flooded	**36**	**23**	**23**	**23**	**23**	**23**	**23**
Direct Potential External Damage Cost	$290,927	$233,263	$231,856	$230,386	$225,937	$214,534	$180,762
Direct Potential Residential Damage Cost	$15,399	$15,043	$14,943	$14,797	$14,506	$14,100	$13,238
Direct Potential Commercial Damage Cost	$5,816	$0	$0	$0	$0	$0	$0
Total Direct Potential Damage Cost	**$312,142**	$0	$0	$0	$0	$0	$0
Total Actual Damage Cost *(0.8*Potential)*	**$249,714**	**$198,645**	**$197,439**	**$196,147**	**$192,354**	**$182,907**	**$155,200**
Infrastructure Damage Cost	**$72,963**	**$54,588**	**$48,158**	**$39,081**	**$27,953**	**$24,807**	**$21,487**
Total Cost	**$322,677**	**$253,233**	**$245,598**	**$235,228**	**$220,307**	**$207,714**	**$176,687**
Average Annual Damage (AAD)	**$22,049**						

8.3.5 Benefit-Cost Analysis

A benefit-cost analysis was undertaken to assess the economic viability of the Combined Mitigation Package. An indicative benefit-cost ratio was based on the construction cost estimates and Average Annual Damages calculated above.

The results of the benefit-cost analysis are shown below in Table 8-9. For this analysis, a net present value model was used, applying a 6% discount rate over a 30 year project life. The benefit cost ratio should ideally be equal to or greater than 1, meaning that the long term benefit of flood mitigation equals or exceeds the long term costs. In this analysis, the cost benefit ratio is 0.44, which indicates that the cost of mitigation exceeds the long term benefits. However, it is important to note that this analysis does not include social costs or benefits, some of which may be considered to be of greater value than the economic costs.

Table 8-9 Cost Benefit Analysis

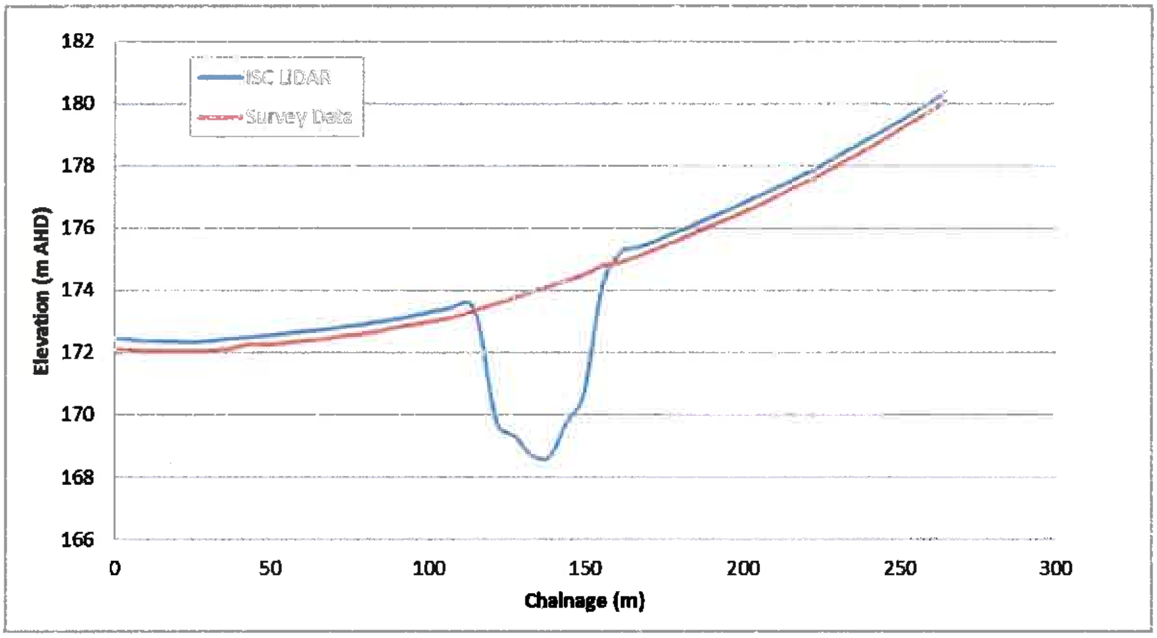
	Existing Conditions	Buildings Levees
Average Annual Damage	$28,229	$22,049
Annual Maintenance Cost	-	$3,035
Annual Cost Savings	-	$3,145
Net Present Value	-	$44,226
Cost of permanent mitigation		$50,358
Capital Cost of Mitigation	-	$101,179
Benefit-Cost Ratio	-	0.44

9. RECOMMENDATIONS

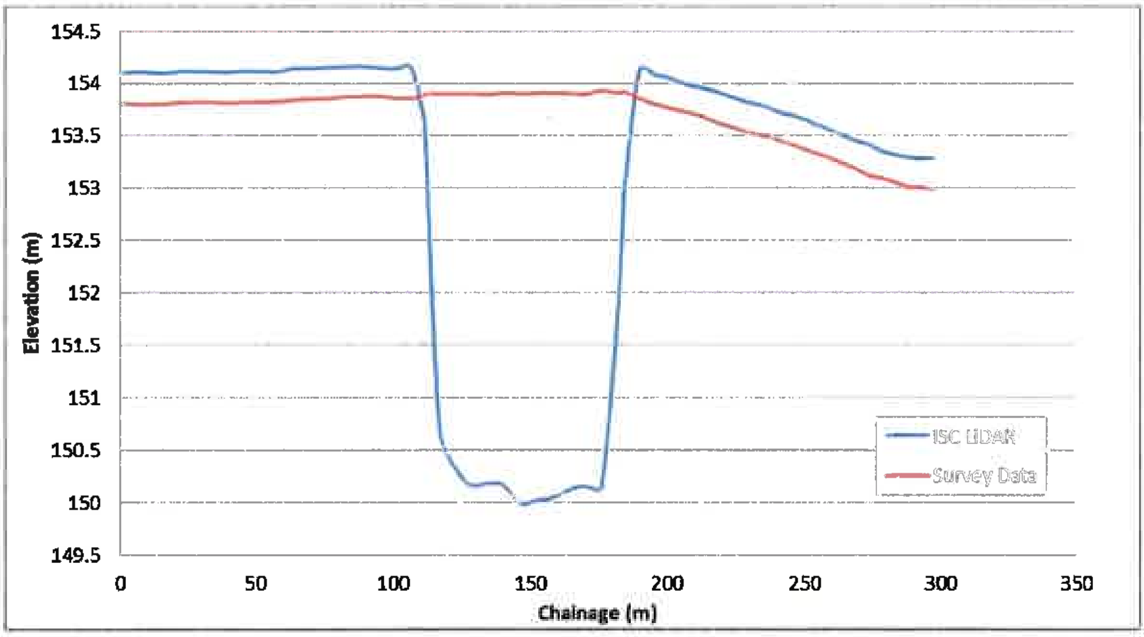
The following recommendations were made because of the findings of this study:

1. The West Wimmera Shire Council Municipal Flood Emergency Plan (MFEP) be updated with the information provided in the Harrow Flood Investigation Flood Intelligence Report.
2. The Land Subject to Inundation Overlay (LSIO) and Flood Overlay (FO) and associated planning scheme amendment documentation produced as part of this study be adopted in the West Wimmera Shire Council Planning Scheme.
3. The Victorian Flood Database (VFD) should be updated using the outputs of the Harrow Flood Investigation which have been formatted into the standard VFD outputs.
4. The Harrow Flood Investigation VFD deliverables should be uploaded to FloodZoom.
5. Bureau of Meteorology Flood Class Levels should be determined for the Glenelg River at Fulham Bridge and the Glenelg River at Harrow streamflow gauges and related to maps in the West Wimmera Shire Council Municipal Flood Emergency Plan.
6. A crowdsourcing flood information network for Salt Creek involving adjacent landholders should be created, including the installation of gauge boards as reference points.
7. An emergency flood plan for the Harrow RSL club should be created.
8. The local CFA brigade should be actively engaged in community preparedness education for flooding.
9. A levee around the John Mullagh Memorial Park should be considered further with community groups and considered for funding.

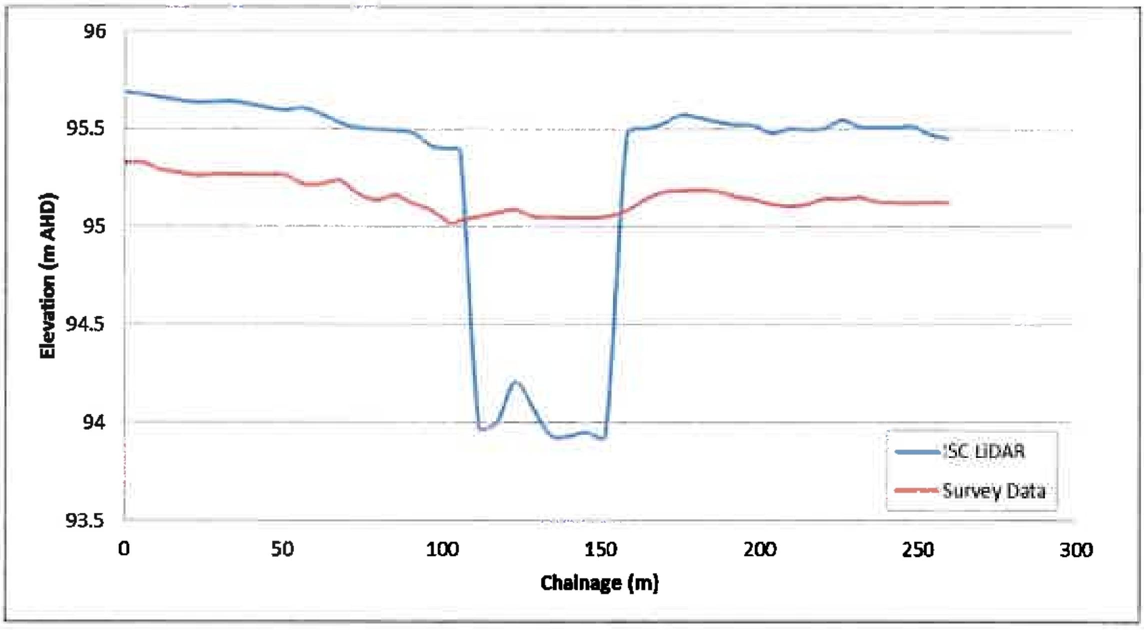
APPENDIX A – ROAD TRANSECTS



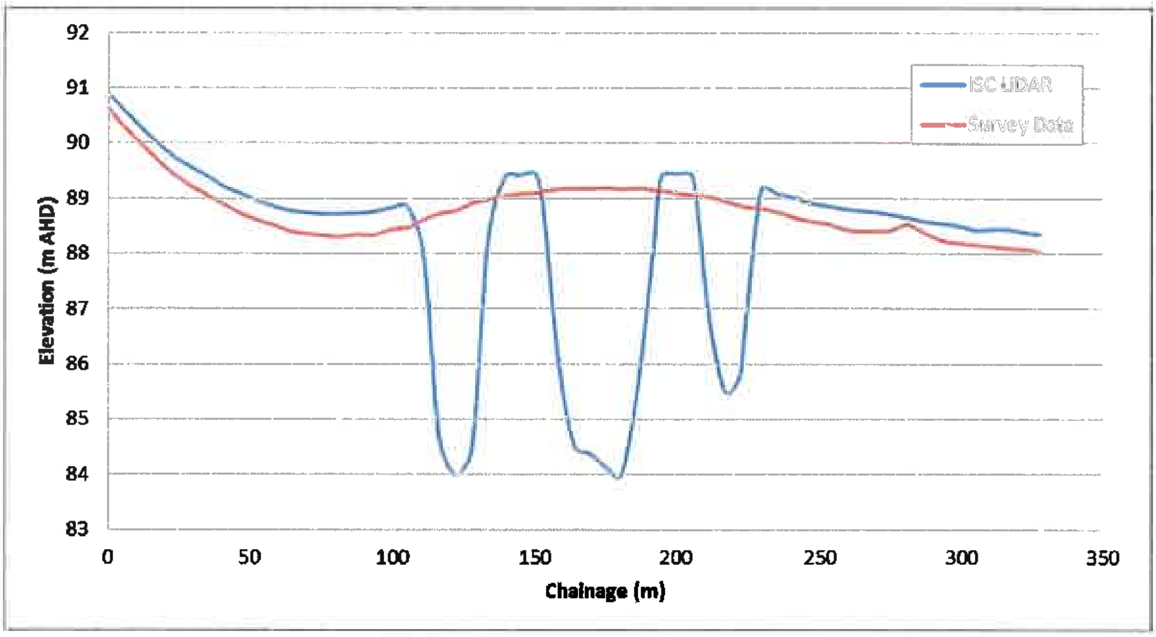
Natimuk Hamilton Road, Kanagulk



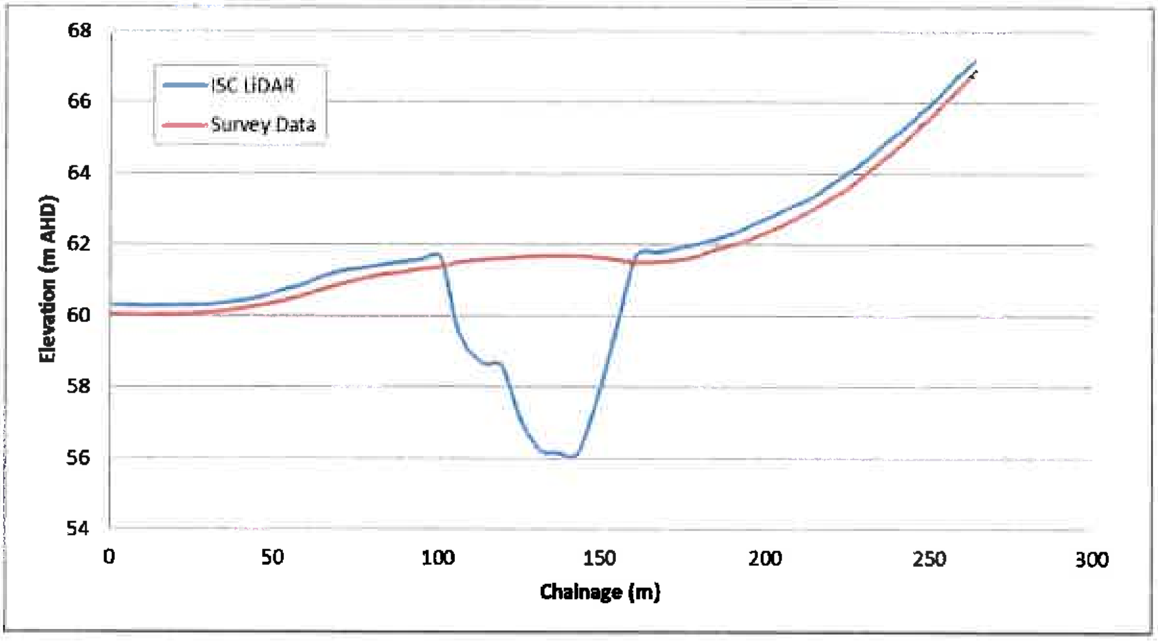
Coleraine Nareen-Moo Road - Culla



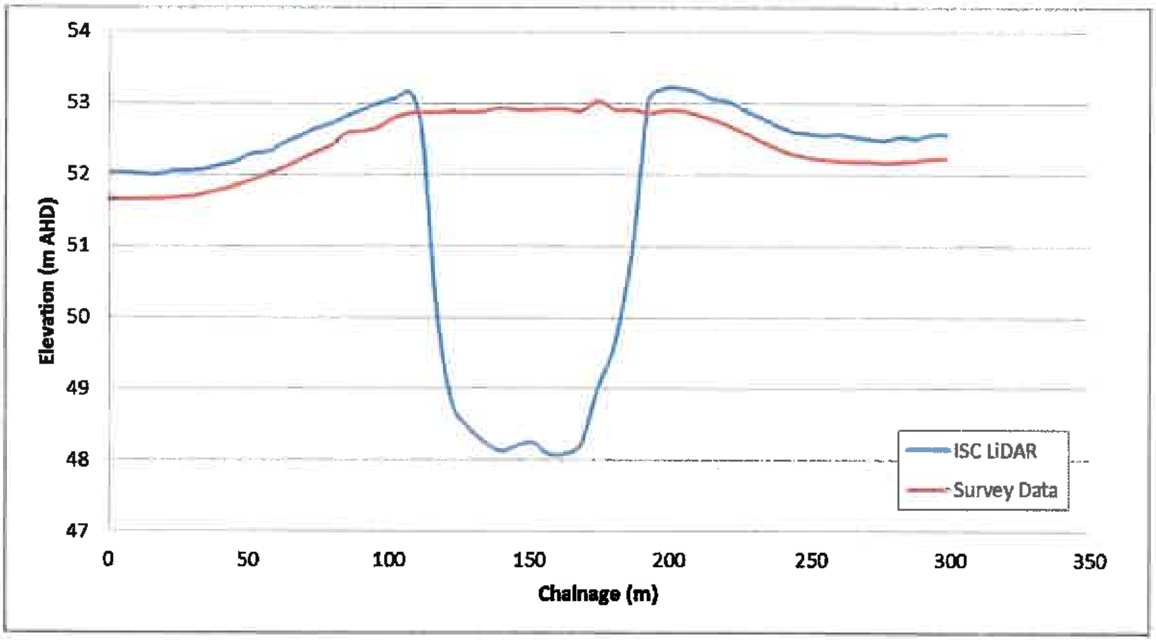
Casterton Edenhope Road - Chetwynd



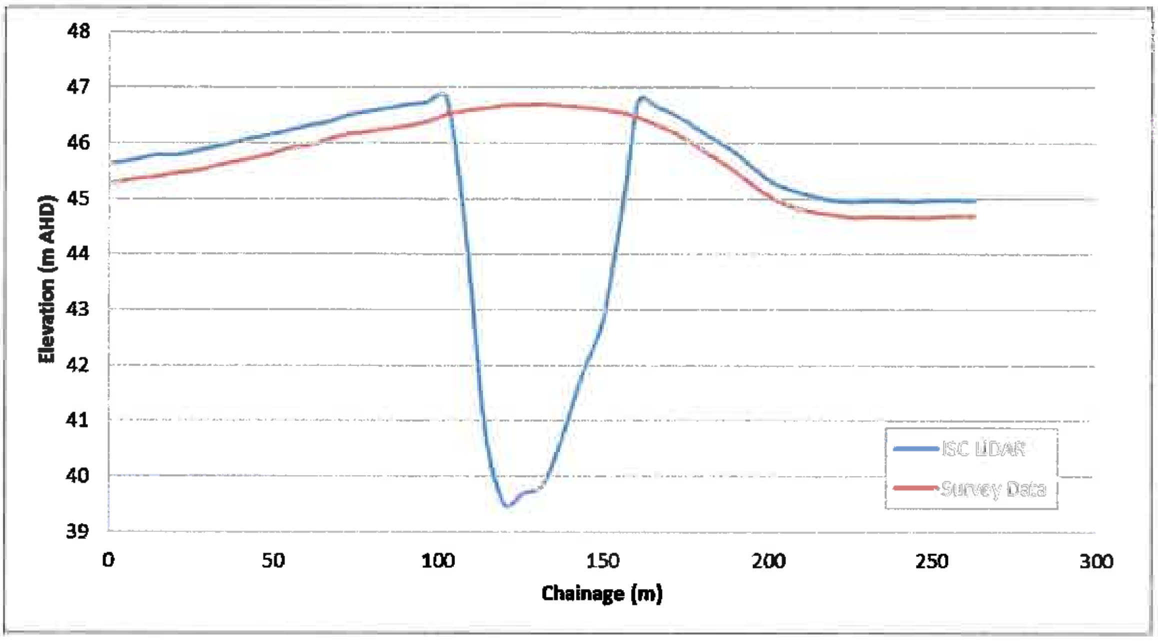
Warrock Road - Roseneath



Section Road - Dunrobin



Glenelg Highway - Casterton



Andersons Road - Casterton