

Macalister River Flood Study Stage 2

Study report



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EXECUTIVE SUMMARY

West Gippsland CMA commissioned Water Technology to undertake the second stage of the Macalister River flood study. This study involves the construction and calibration of a hydraulic (flood) model of the Macalister River floodplain downstream of Lake Glenmaggie to the Thomson River confluence. Based on the results of the hydraulic modelling, the study prepared flood mapping for flood response and land use planning, and undertook preliminary assessment of mitigation measures. This study was supported by Southern Rural Water (SRW) through a contribution to the collection of topographic data and to understanding flood behaviour impacts of potential changes to Southern Rural Water channels within the Macalister River floodplain.

This report addresses study Tasks 1 to 11 except for Task 8 (SRW proposed channel modification), which is provided as a separate stand alone report.

Topographic Data Review

The Macalister River Flood Study Stage 1 (WBM, 2008) concluded that the available topographic data was not of sufficient reliability to construct and calibrate a robust hydraulic model. Subsequently additional topographic data was collected by Fugro (LiDAR and aerial photography) of the Macalister Irrigation District (MID) during July and August 2008 for Southern Rural Water (SRW) and the Department of Sustainability and the Environment (DSE).

A detailed review of the LiDAR data against other independently sourced topographic data was undertaken as the first stage of this project. The independent topographic data included field survey data of waterway cross sections, levee/embankment crest strings and flood levels captured by the Stage 1 study (WBM, 2008). Two versions of the LiDAR data were supplied by DSE and SRW. The interim LiDAR data was supplied in January 2009, and a revised final data set provided in June 2009.

This report details the review of both the interim and final LiDAR data sets provided. A bias was identified in the interim LiDAR data compared to field survey i.e. the interim LiDAR data appeared to be generally higher than the field survey by 100 mm to 200 mm. Further review of the final LiDAR data set against the available field survey showed much better correlation between the LiDAR and field survey levels and no clear bias across the study area. This final LiDAR data set was therefore adopted for use in the development of the hydraulic model for the flood study.

Hydrologic Analysis

The study brief required the determination of design flood hydrographs for the 20, 50 and 100 year event plus the Probable Maximum Flood (PMF) event plus three other events for flood response purposes. The three other flood events adopted were the 5, 10 and 200 year events.

The hydrological analysis approach adopted for the present study builds on the previous BoM, WBM (2008) and SMEC (2007) investigations. The key elements of the present approach are as follows:

- Peak flow flood frequency analysis of observed flows downstream of Lake Glenmaggie,
- Flood volume flood frequency analysis of observed flows downstream of Lake Glenmaggie,
- URBS runoff routing modelling for design rainfall events with model parameters adjusted to reconcile URBS model peak flow estimates and flood frequency analysis ,
- Design flood hydrograph estimation using the URBS model to determine 5, 10, 20, 50, 100 and 200 year ARI events plus indicative PMF.
- Climate change sensitivity scenarios.

To compare the revised hydrologic analysis with previous work, the estimated average recurrence interval (ARI) of the June 2007 flood event was compared. This event had an estimated peak flow of 147,640 ML/d (WBM, 2008). The present study's peak flow frequency analysis assessed the

indicative ARI of this event at ~ 190-200 years, while WBM (2008) and SMEC (2007) give an estimated ARI of around 100 years.

SMEC (2007) assessed the June 2007 rainfall event to have an approximate ARI of 1 in 50 year, and estimated the ARI of the peak inflow to Lake Glenmaggie as between 1 in 100 year and 1 in 300 years.

The key reasons for the differences between this study and previous work are:

- The use of instantaneous peak daily flows for the entire flow record in the present study rather than mean daily flows (which likely underestimate the actual peak flow value on any given day). It is unclear whether the mean daily flow data used in WBM (2008) was adjusted to reflect the estimated instantaneous peak values. SMEC (2007) used mean daily flow data in their analysis.
- For floods towards the extreme end of the flood frequency distribution the best fit curve is relatively “flat” with only a small range of flows between the 100 and 200 year ARI flow estimates. The present study is considered to show a good fit between the observed data and the flood frequency curve (Figure 4-5). Observed peak flows were not plotted on the flood frequency curve in WBM (2008) and therefore the goodness of fit cannot be assessed.
- Flood volume frequency analysis has been included in the present study. WBM (2008) and SMEC (2007) considered peak flows without consideration of flood volumes. Given the extensive nature of the floodplain, flood volume in addition to peak flow will influence flood behaviour.

Revisions were also made to the existing URBS hydrologic model. The resultant design event hydrograph peak flows are summarised in Table 1. A comparison of the design peak flow estimates showed that for the 2 to 20 year ARIs, this study’s estimates were higher than WBM (2008). The difference varies from 26% for the 2 year ARI to 3% for the 20 year ARI. For the 50 and 100 year ARI, this study’s peak flow estimates were lower than WBM (2008). The difference varies from 4% for the 50 year ARI to 9% for the 100 year ARI.

As no recalibration of the URBS model was undertaken for the present study, the key difference between WBM (2008) and the present design event analysis is the consideration of rainfall losses. Losses in WBM (2008) were reconciled for the 100 year peak flow estimates using the peak flow frequency analysis, whereas the present study reconciled the 20, 50 and 100 year peak flows estimates. For the present study, losses are higher for 100 year event than in WBM (2008), but lower for the 20 year ARI event. This is reflected in the revised design peak flow estimates

Table 1 **Design peak flow estimates for the Macalister River downstream of Lake Glenmaggie**

ARI (years)	Design peak flow estimates Macalister River downstream of Lake Glenmaggie (ML/d)
5	52,900
10	64,800
20	90,800
50	115,900
100	133,400
200	151,900

Climate change impacts on flood behaviour were assessed as a sensitivity analysis of design rainfall intensities. Increases in design rainfall intensities by 10 % and 20% were assessed for the 20, 50 and 100 year ARI events. For a 10 % rainfall intensity increase, the peak flow estimates increased by ~ 17-18% , and for the 20 % design rainfall increase peak flows increased by 36-38%. This equates to the 20 year ARI peak flow increases to a ~ a 40 year ARI with 10% rainfall increase, and to ~ 80 year ARI with a 20% increase.

It should be noted as part of this sensitivity analysis, the design rainfall losses remained unchanged. It is reasonable to expect that the rainfall losses may increase with climate change (general drier climate). Any increase in design rainfall losses would offset (in part) the increases in rainfall intensities.

Hydraulic Analysis

The hydraulic model developed for this study was required to simulate the flow behaviour for large flood events (5 year to 200 year ARI plus PMF) across the Macalister River floodplain. Different model configurations were tested during the model development, with the final model being a 2D MIKE 21 hydraulic of the complete study area.

Two flood events, June 2007 and June 1978, were chosen for calibration and validation of the model respectively, as they are both relatively large magnitude flood events with a reasonable abundance of observed flood information. The June 2007 event had a peak flow of 147,640 ML/day and an estimated ARI of ~ 190-200 years, while the June 1978 event had a peak flow of 88,900 ML/day and an estimated ARI of ~20 years.

Detailed comparisons have been made between the observed flood levels and extents for both events, with considerable investigation of any locations where discrepancies were identified. Overall, given the complex array of topographic features (irrigation infrastructure, road/rail embankments) across the Macalister River floodplain downstream of Lake Glenmaggie, which strongly influence the flood behaviour, the calibrated hydraulic model is considered fit for purpose.

Flood Response and Land Use Planning Mapping

The hydrologic and hydraulic models described previously were used to simulate and map flood behaviour (extents, depth, velocities) for a range of flood magnitudes. The hydrologic analysis provided design event flood inflow hydrographs downstream of Lake Glenmaggie. These inflow hydrographs were routed through the calibrated hydraulic model. This enabled the modelling and mapping of flood depths, extents, velocities over a range of flood magnitudes.

To provide a suite of flood maps across the range of flood magnitudes of interest, flood mapping was undertaken for the peak flows of 7,500 ML/d, 20,000 ML/d, 30,000 ML/d and 40,000 ML/d, in addition to the 5,10,20,50,100 and 200 year ARI events plus the PMF event.

The mapping outputs can be applied for flood response planning, and land use planning purposes. All flood mapping data was prepared to the VFD metadata specifications.

Flood Damages Assessment

The results of the flood damages assessment are summarised in Table 7-4. The 1% AEP or 100 year ARI flood damage was calculated to be over \$3.5 million and the AAD was calculated to be approximately \$515,765 per year.

It should be noted that all damages calculations are based on current development, and flooding conditions and take no account of potential impacts of future climate change or change in development patterns.

It is important to recognise that the flood damages assessment developed here, whilst consistent with best current practice, has a number of uncertainties and could be expected to vary within a range of at least +/- 20%.

Flood Risk Mitigation

Structural mitigation measures are physical works to reduce the likelihood of flooding in a given location. Potential structural mitigation measures identified include the following:

- Upstream flood storage
- Levees
- Floodways
- Waterway management works
- Improvements to waterway structures (culverts etc)

Discussions with the West Gippsland CMA identified the mitigation of flooding impacts in Tinamba as the key study priority. To this end, the structural mitigation measures canvassed by this study focused on Tinamba. The following three structural mitigation options were assessed.

- Option 1: Raise the farm channel and laneway (refer Figure 8-1 in red). The levee would need to link up with SRW's irrigation channel at its western end and with the Tinamba-Newry road at its eastern end.
 - Option 1 prevented flooding of properties in Tinamba. The increases in flood levels associated with the re-distribution of flood flows do not adversely affect any dwellings. There were modelled increases at several farm sheds/outbuildings. Flood modelling indicated that the existing farm channel required rising by 1.0 m to 1.4 m to achieve a 600 mm freeboard in the 100 year event. Temporary barriers would be required across the Tinamba – Seaton Road, between the channel headwalls. This preliminary assessment only considered the hydraulic impacts of this option. Further functional and detailed design is required.
- Option 2: Option 1 plus increase the floodway capacity of the SRW Siphon and lower farm channel to south of Tinamba Seaton Road (refer Figure 8-1 in blue)
 - Option 2 provided similar reduction in flood levels and extents through Tinamba as Option 1. The increased opening in the SRW channel leads to a slightly lower increase in flood levels upstream of the proposed levee when compared to Option 1. Discussions with SRW, indicated difficulties associated with the hydraulic performance of a longer siphon.
- Option 3 Build a new levee around the town (Figure 8-1 in yellow) and increase flood conveyance through the open land beside the tennis courts.
 - Option 3 reduced flooding depths and extents in the Tinamba, north of the Tinamba Seaton Road. As discussed, some flooding still occurred in Tinamba due to the outflanking of the proposed levee. To achieve a similar reduction in flooding, as Option 1 and 2, the Option 3 levee needs to be extended to south of Tinamba Seaton road, and road levels along the Tinamba Newry (north of the roundabout) to be raised.

Non-structural measures are floodplain management activities aimed at reducing future flood damages. Non-structural measures aim to reduce existing flood risk by lowering flood damages (consequences) at a given location (as opposed to structural measures which tend to reduce frequency or likelihood of flooding). Non-structural measures include:

- Catchment management
- Flood awareness, preparedness, warning and response – discussed in the flood warning section.

- Land use planning – through the flood mapping outputs of the hydraulic modelling.

Flood Warning

The existing flood warning system for the Macalister River downstream from Lake Glenmaggie has been summarised against each of the eight Total Flood Warning System (TFWS) building blocks. The summary reflects current arrangements which include a number of organisational and procedural changes implemented as a result of recommendations contained in key reports arising from the 2007 floods in the region. A consideration of the summary revealed four major issues:

- Additional warning lead time is required, particularly for the Newry and Tinamba communities;
- Inundation extent maps are required to facilitate an understanding within the at-risk community and within the stakeholder agencies, of likely flood impacts, particularly during a severe event;
- A flood emergency plan that details likely flood consequences and critical timings as well as the triggers for damage reducing actions needs to be developed; and
- Flood intelligence in relation to flood impacts and required actions needs to be shared with the at-risk community.

Actions aimed at addressing these issues have been recommended. While the scope of recommendations could have been wider and remedies more comprehensive, the authors of this report are acutely aware that the Comrie-led Victorian Floods Review will be delivering a comprehensive report in late 2011 following a wide ranging examination of the flooding that occurred from September 2010 to January 2011 in relation to the TFWS and existing legislation and policies. That report is expected to contain a number of substantial policy, procedural and technically orientated recommendations that will have application to all river basins within the State, including the Macalister.

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1. INTRODUCTION

West Gippsland Catchment Management Authority (WGCMA) commissioned Water Technology to undertake the second stage of the Macalister River flood study. This study involves the construction and calibration of a hydraulic (flood) model of the Macalister River floodplain downstream of Lake Glenmaggie to the Thomson River confluence. Based on the results of the hydraulic modelling, the study prepared flood mapping for flood response and land use planning, and undertook preliminary assessment of mitigation measures. The study area is shown in Figure 1-1.

This study was supported by Southern Rural Water (SRW) through a contribution to the collection of topographic data and the assessment of potential changes to SRW channels within the floodplain.

The study brief outlines the following project tasks:

- Review available information and consult key stakeholders (Task 1)
- Review and verify available topographic survey data (Task 2)
- Facilitate the installation of flood marker posts (Task 3)
- Review previous hydrologic analysis and determine required historical and design flood hydrographs (Task 4)
- Construct and calibrate hydraulic model to assess flood behaviour across the study area (Task 5)
- Prepare report outlining hydrologic and hydraulic analyses from Tasks 4 & 5. (Task 6)
- Prepare flow correlation for use in flood response (Task 7)
- Assess the flood behaviour impacts of Southern Rural Water's proposed channel modifications (Task 8)
- Prepare flood mapping outputs for land use planning and flood response purposes (Task 9)
- Assess current flood risks and assess potential flood mitigation measures (Task 10)
- Prepare study report (Task 11)

This report addresses the above tasks except for Task 8 (SRW proposed channel modification). This task is detailed in a stand alone report.

The structure of this report is as follows:

- Section 2 reviews available data and previous investigations/studies,
- Section 3 outlines the hydrologic analysis ,
- Section 4 details the hydraulic model construction and calibration,
- Sections 5 & 6 discuss the application of the hydrologic and hydraulic model to mapping of flood characteristics for flood response and land use planning purposes,
- Section 7 summarises the assessment of flood damages,
- Section 8 outlines flood risk mitigation measures,
- Section 9 provides a review an overview of flood warning with application to the Macalister River downstream of Lake Glenmaggie, and
- Section 10 summarises the outcomes and recommendations from this study.

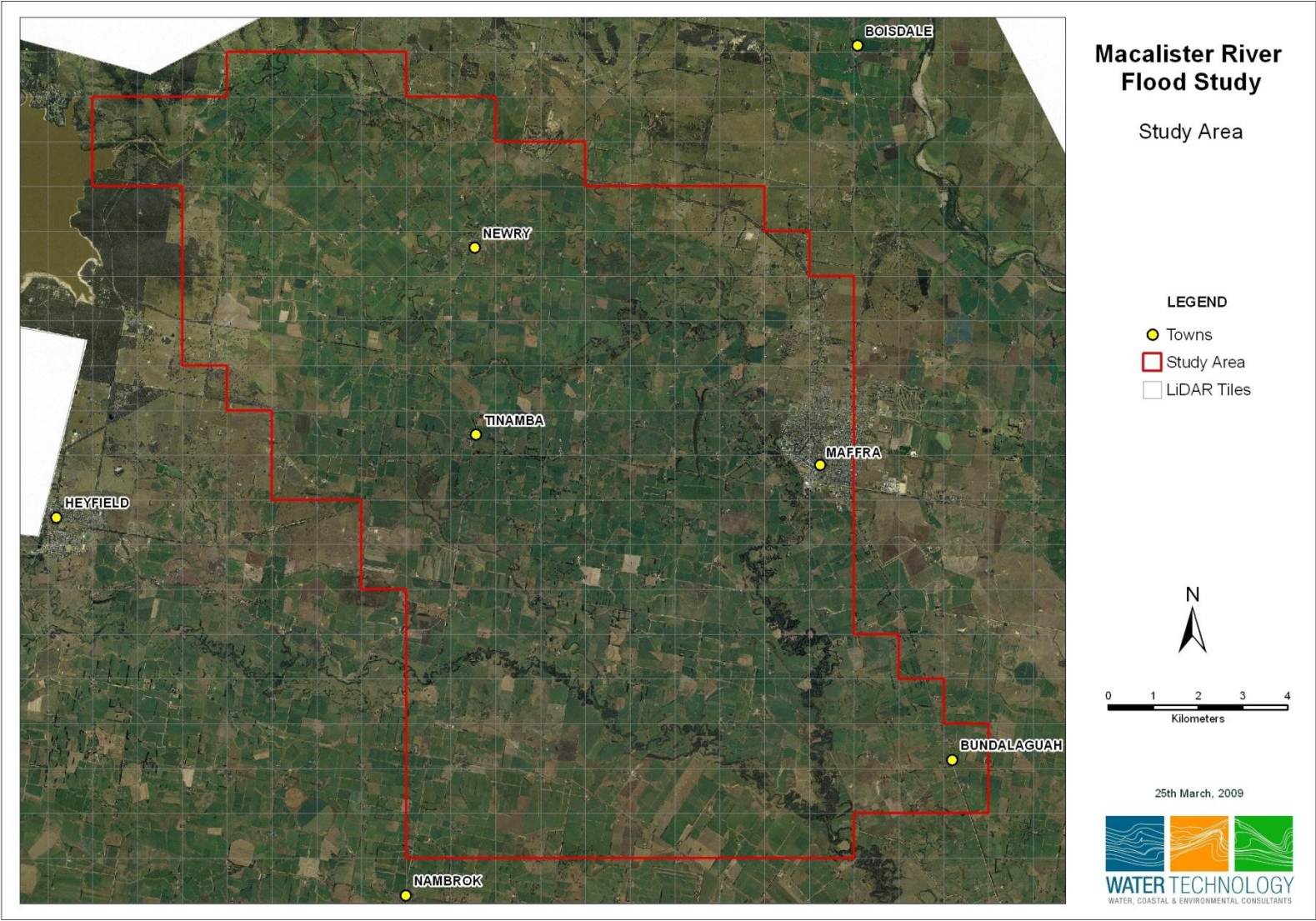


Figure 1-1 Study Area

2. AVAILABLE DATA REVIEW

2.1 Overview

This section identifies and briefly reviews the available data and information relevant to flooding from the Macalister River. Sources of background data and information collated have included:

- Previous waterway and floodplain studies
- Historical flood information - photographs, flood marks, etc
- Hydrologic data – observed and design flows, methodology, results

A review and verification of the available topographic data was documented in “Digital terrain model construction and verification” report dated April 2009 (Water Technology, 2009), which is summarised and extended in Section 3.

2.2 Previous Waterway and Floodplain Studies

A useful summary of the previous investigations is contained in the “Macalister River Flood Risk Assessment Study” SKM (2003). Elements of this summary relevant to this study are provided as follows:

- Ian Drummond and Associates (1992). Macalister River Flood Hydraulics
This study assessed the flood behaviour for the Macalister floodplain downstream of Mewburn Park, with a focus on the interaction between the river and the Serpentine drainage system. The assessment centred on flood events up to about the 20 year ARI event. A key conclusion was that the flood behaviour is controlled by channel, roads and natural embankment thresholds. The study indicated that small changes of the key controlling features (i.e. minor raising/lowering) have considerable impact on flood behaviour.
- State Rivers and Water Supply Commission (1978). Latrobe River, Thomson river, Macalister River, City of Sale, Township of Seaspray. Flood Report May-June 1978
This report discussed the flooding across the Gippsland Region in May-June 1978. For the Macalister River floodplain, the report noted breakouts occurred at Greenvale to the Boggy Creek system. These breakouts commenced for outflows from Glenmaggie Dam at 22,000 ML/d. The report highlighted landholders’ concern regarding the operation of Lake Glenmaggie. Peak flood levels were surveyed.

The above elements were extracted from SKM (2003). A full list of previous investigations (up to 2003) is provided in SKM (2003).

In addition, to provide a summary of previous investigations, the “Macalister River Flood Risk Assessment Study” SKM (2003) documented the flood behaviour and flood risk. The key findings included:

- River and flood characteristics:
 - Generally, the river channel flow capacity diminishes from Lake Glenmaggie to the Thomson River confluence.
 - Key effluents: Newry Creek, Boggy Creek, Serpentine Drain and Bundalaguah Drain.
 - Flooding affecting landholders occurred “fairly frequently”.
- Economic, social and environmental impacts:
 - Average annual damage estimated at \$1.35 million (at 2003).

- Flood mapping can inform potential mitigation measures to alleviate flood impacts with potential environmental benefits (e.g. improve flow connectivity for wetlands).

A key outcome from SKM (2003) was to recommend a flood study to assess flood behaviour and potential mitigation measures.

Following on from the SKM (2003) recommendation, WBM (2008) undertook the Macalister River Flood Study Stage 1. This study comprised both hydrologic and hydraulic analyses to assess flood behaviour. During the course of the Stage 1 flood study, a major flood event occurred in June 2007. A number of June 2007 flood levels were surveyed, refer to Section 2.3 for further details. These surveyed flood levels were employed as calibration data for the hydraulic model. WBM (2008) concluded that a reasonable hydraulic model calibration to the June 2007 event could not be achieved. WBM (2008) considered that the inadequacy of topographic data available at the time limited the hydraulic model's ability to simulate the June 2007 flood event. The subsequent collection of additional topographic data, as outlined in Section 3, underpinned the development of the present study's hydraulic model.

Hydrologic analysis undertaken by WBM (2008) determined design flood hydrographs for the Macalister River downstream of Lake Glenmaggie. This analysis employed a runoff routing model (URBS) and flood frequency analysis.

Further technical details of the WBM (2008) hydrologic and hydraulic analyses are discussed in Section 3.

2.3 Historical Flood Information

2.3.1 Streamflow Data

Streamflow data is available for the Macalister River downstream of Lake Glenmaggie (Station 225204). The period of available streamflow data is from 1919 to present. Mean daily flows are available for this entire period, and instantaneous flows are consistently available from March 1960 onwards. There are occasional instantaneous peak flows prior to March 1960 (RWCV 1990). The Glenmaggie Dam was completed in 1926, with additional raising and installation of radial gates in late 1940's and early 1950's. WBM (2008) noted 20 flood events with flows exceeding 30,000 ML/d downstream of Lake Glenmaggie since the record began (1919), and two events, January 1971 and June 2007, with flows exceeding 100,000 ML/d. Furthermore, WBM (2008) noted the streamflow data was considered generally reliable but with some missing during flood events. It should be noted the gauge suffered equipment malfunction during the June 2007 event. Southern Rural Water provided an estimate of the peak outflow for June 2007 event of 147,600 ML/d (WBM, 2008). Table 2-1 displays the flood events exceeding 30,000 ML/d downstream of Lake Glenmaggie.

The use of this available streamflow flow data in the hydrologic analysis is discussed in Section 3.

Table 2-1 Summary of floods exceeding 30,000 ML/d (after WBM, 2008)

Month & Year	Flow (ML/d)		Towns affected
	Maximum mean daily	Instantaneous Maximum	
April 1935	65,700	-	
April 1950	59,700	-	Tinamba
August 1951	42,900	-	
June 1952	70,600	77,800	Tinamba & Newry
October 1953	36,700	-	
January 1956	32,800	-	
August 1958	35,200	44,800	
September 1959	49,100	63,700	
June 1970	40,000	44,700	
January 1971	57,200	110,000	Tinamba & Newry
August 1974	23,800	42,100	
October 1976	38,200	48,800	
June 1978	36,900	88,900	
October 1985	43,300	51,300	
September 1993	-	75,500	
October 1993	-	57,000	Tinamba
October 1995	-	66,500	Tinamba
November 1998	-	42,000	Tinamba
June 2007	-	147,640	Tinamba & Newry
November 2007		59,100	Tinamba

2.3.2 Observed Flood Levels and Extents

SKM (2003) notes the availability of observed flood information for the May-June 1978 event. This available information consisted of surveyed flood levels and aerial flood photography.

A total of 248 flood levels were surveyed following the June 2007 event. West Gippsland CMA also provided aerial photography and video for the June 2007 event.

3. TOPOGRAPHIC DATA REVIEW

3.1 Overview

As discussed in Section 2.2, the Macalister River Flood Study Stage 1 (WBM, 2008) concluded that the available topographic data was not of sufficient reliability to construct and calibrate a robust hydraulic model.

Subsequent additional topographic data was collected using LiDAR. Fugro captured LiDAR and aerial photography of the Macalister Irrigation District (MID) during July and August 2008 for Southern Rural Water (SRW) and the Department of Sustainability and the Environment (DSE).

Given the WGCMA's concern over the reliability of topographic data, a detailed review of the LiDAR data against other independently sourced topographic data was undertaken. The independent topographic data included field survey data of waterway cross sections, levee/embankment crest strings and flood levels captured by the Stage 1 study (WBM, 2008). Two versions of the LiDAR data were supplied by DSE and SRW. The interim LiDAR data was supplied in January 2009, and the revised final data provided in June 2009.

An initial review of the interim LiDAR data set was undertaken in April 2009 (refer to Water Technology, 2009). The preliminary conclusion of this review was the interim LiDAR data displayed a bias when compared to field survey i.e. the LiDAR data appears to be generally higher than the field survey by 100 mm to 200 mm.

This section builds on the initial review, and discusses the review of the revised LiDAR data set against the available field survey.

3.2 LiDAR Dataset and Processing

The LiDAR which was originally provided to Water Technology, by Southern Rural Water, at the beginning of the project, the interim LiDAR, was re-issued in October 2009 after some anomalies were amended. The processing procedure outlined in this section was completed on both the interim and the final LiDAR datasets.

3.2.1 Metadata

Fugro were contracted by DSE to capture LiDAR and aerial photography of the Macalister Irrigation District (MID). This LiDAR was flown between 28 July and 3 August 2008. Fugro provided one kilometre tiles of the following data; unthinned and thinned ground data, non-ground data, first and last return points, and 0.5 m contours. The metadata provided by Fugro is shown in Table 3-1.

Table 3-1 Metadata supplied by Fugro

Spatial Accuracy – Horizontal	0.25 m at 67% Confidence Interval
Spatial Accuracy – Vertical	0.10 m at 67% Confidence Interval
Surface Type	Various
Average Point Separation	Target spacing 0.5 m ²
Laser Return Types	All Returns
Laser Footprint Size	0.32 m
Data Tile Size	1 km

The hydraulic model construction required the development of a Digital Elevation Model (DEM). Of most interest for creating a DEM was the 10 cm thinned ground points. This dataset represented a subset of the ‘unthinned’ ground points. The unthinned points is processed by ground filtering algorithms to remove any points which are considered ‘non-ground returns’ (such as tree tops and roofs of buildings). The thinned data set is then extracted from the unthinned point set, where all points which do not contribute to the shape of the terrain have been removed. The thinning process was undertaken by Fugro.

3.2.2 Processing Approach

The processing of the LiDAR data, discussed in the following sections, was undertaken for both the interim and final LiDAR data sets.

Although Southern Rural Water completed their own processing of the LiDAR to produce a DEM, Water Technology undertook further processing. This further processing provided precise knowledge of the LiDAR used in the DEM construction, as the accuracy of the DEM was critical to the construction and calibration of a hydraulic model. This section documents the processing of the LiDAR data so that any error propagation can be traced and quantified.

Only LiDAR tiles which lie within the study area were used in generating the DEM. In all, this amounted to around 200 tiles, where each tile contained between 60,000 and 300,000 points.

The steps required to move from xyz point files to a DEM involved the conversion of the text files to 3-dimensional shapefiles, construction of a TIN from these point shapefiles, and then conversion of the TIN to a grid. This grid is the final DEM.

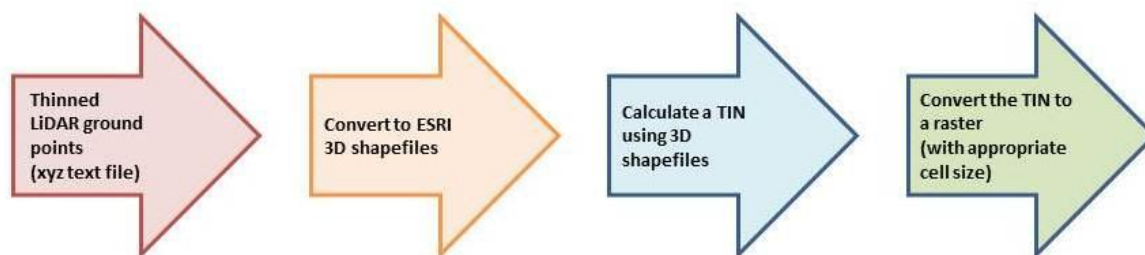


Figure 3-1 Terrain Data Processing Methodology

3.2.3 Formatting the LiDAR text files for use in ArcGIS

The 10 cm thinned ground points were considered the most appropriate available LiDAR data for the DEM construction.

The ‘.thn’ files were formatted so they could be read into ArcMap, using ET GeoWizards. This process did not alter the data. It merely added a header and footer to the file, and a unique identifier for each point. An example of an unformatted and formatted text file is shown in Figure 3-2.

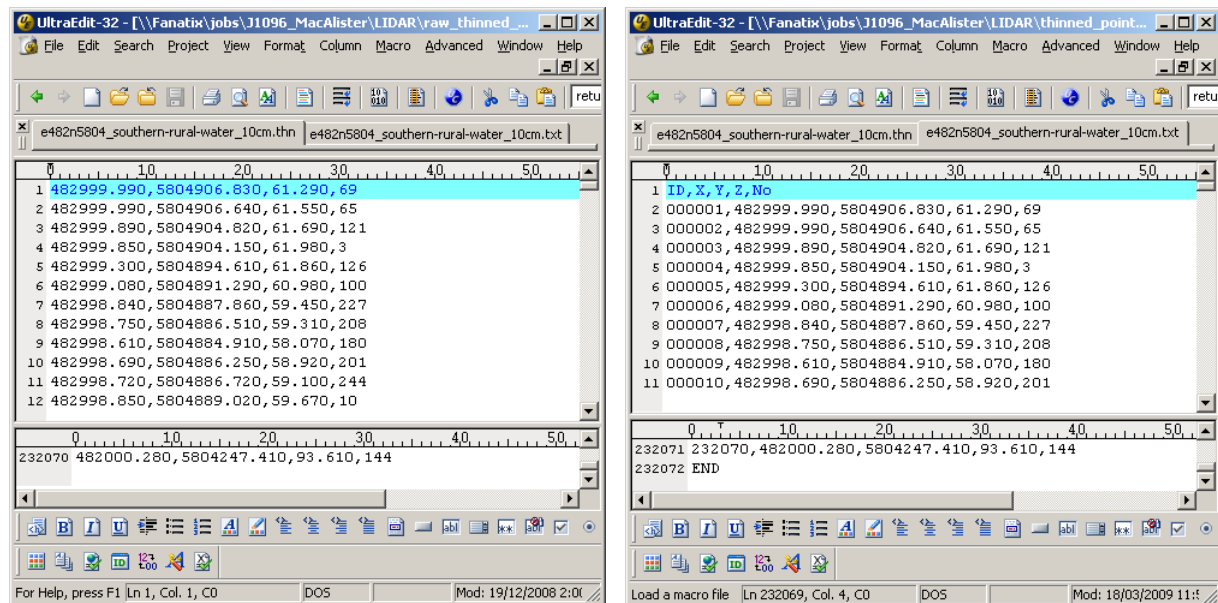


Figure 3-2 Comparison between Fugro's .thn files and the edited .txt file for ArcMap

Each of the 200 tiles in the study area were formatted in this way, and converted to 3-dimensional shapefiles using ETGeoWizards.

3.2.4 Generating a DEM from Multiple TINs

Once all the LiDAR had been imported into ArcGIS and saved as 3D shapefiles, a TIN was constructed. Due to the study area size ($\sim 200 \text{ km}^2$), nine separate TINs were generated. Figure 3-3 shows the extent of the nine TINs.

Sufficient overlapping of the adjoining TINs, ensured a 'smooth' finish, and that no errors or discontinuities occurred in the DEM as a result of interpolation errors across tile edges.

Each of the nine TINs were converted into rasters with a 1 m grid spacing. Some loss of data occurs at this stage of the process, and is discussed in more detail in Section 3.2.5. Finally, the nine rasters were joined together to form one, seamless DEM.

To ensure the final DEM was seamless, without errors along any boundaries, ArcGIS's mosaic method, 'Last', was chosen. This means that wherever two rasters overlap, the elevation would be taken from the next raster down the list. This is illustrated below in Figure 3-4.

The final DEM, for the revised LiDAR data, is shown below in Figure 3-5.

3.2.5 Discussion of Errors

The main source of error in the DEM generation process occurs in the conversion from TIN to raster. A TIN can be defined as a data structure that describes a three-dimensional surface as a series of irregularly shaped triangles derived from irregularly spaced points and breakline features. Whereas a raster can be considered a 2-dimensional representation of a 3-dimensional surface. It is made up of evenly spaced cells, where each cell contains one elevation value only.

Importantly, a TIN is the most accurate way to describe terrain information, and so migrating from a TIN to a raster results in some loss of data. The amount of data which is lost will depend on the grid size, and the degree and interval of any terrain changes. A raster will effectively smooth any sudden and large changes in elevation.

In this case a one metre grid spacing was chosen. This was deemed appropriate, as a smaller grid size would result in the final DEM being very large, and difficult to work with. It is likely that a smaller grid size would not be possible to create without running out of computational memory, and would be excessive based on the scale of the study.

This one metre DEM was further resampled to a 12.5m cell size for the hydraulic modelling component of the study. This results in a much larger loss of precision in the data, however is necessary for the model to run at a reasonable speed, that is days, rather than weeks. To overcome this loss of precision, significant hydraulic controls are stamped onto the DEM to ensure important features such as rivers, channels and levees are not smoothed out of the final model topography.

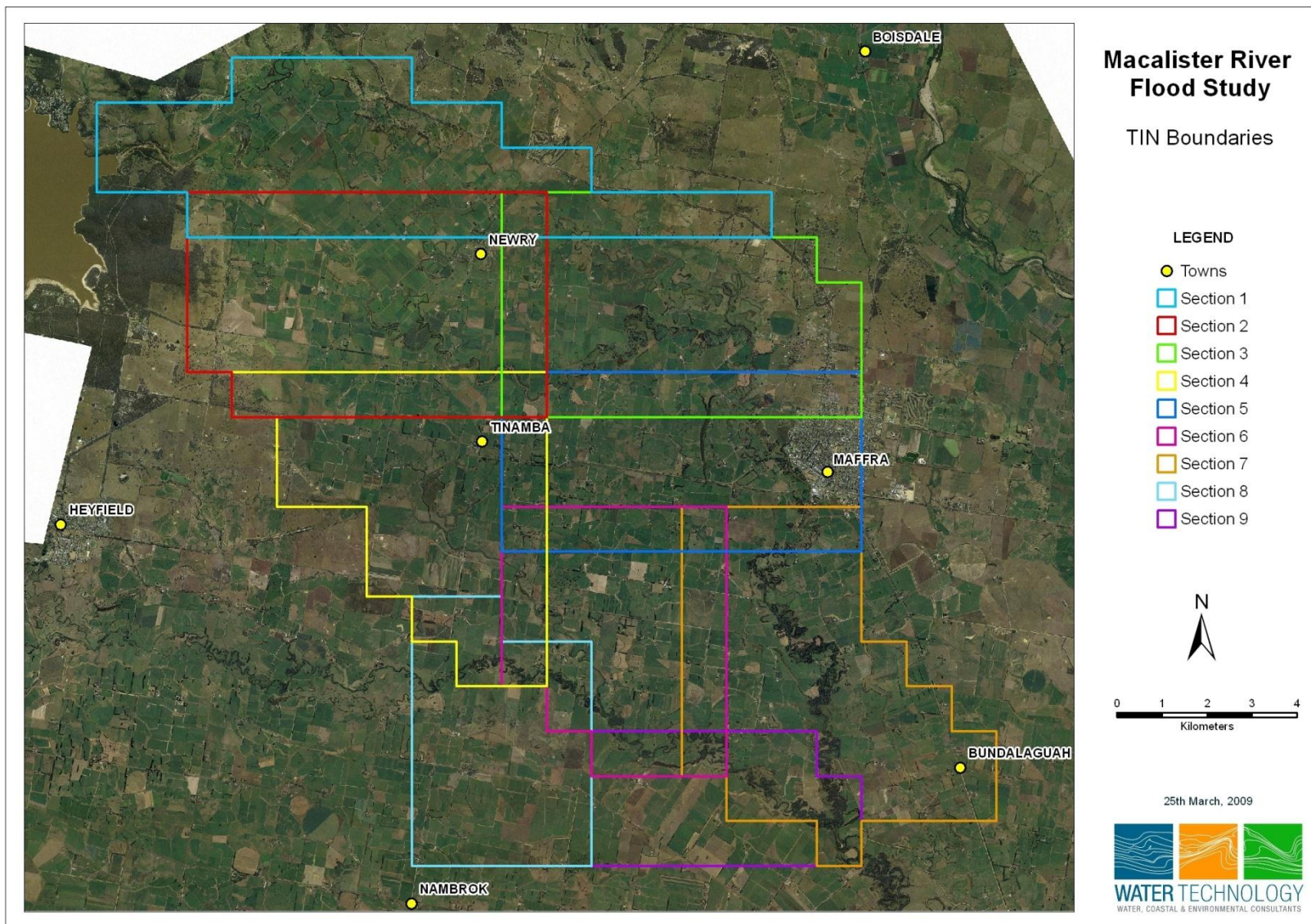


Figure 3-3 Bounding polygons for the nine TINs created of the Macalister River study area

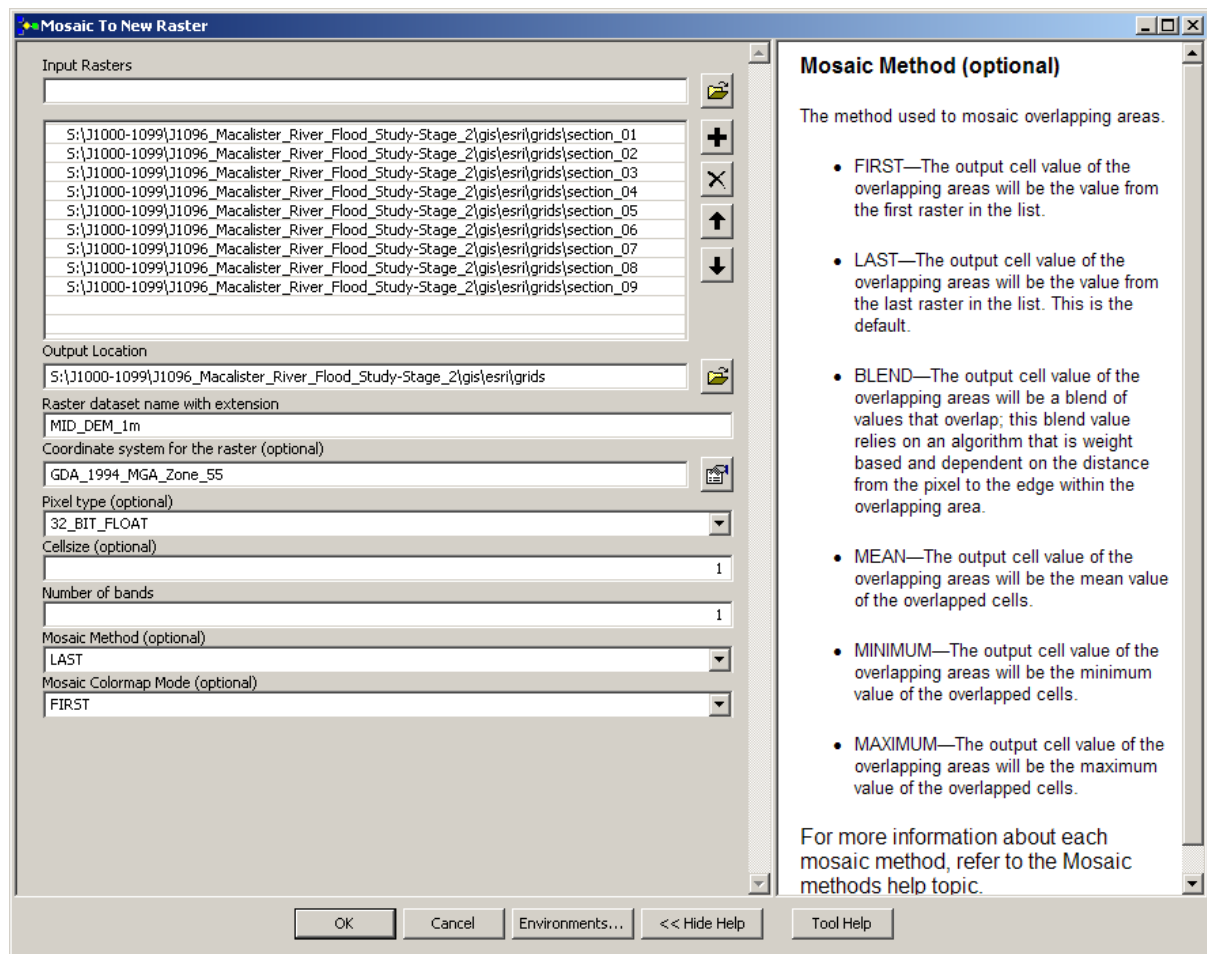


Figure 3-4 Creating the final DEM by mosaicing nine rasters together using the 'last' technique

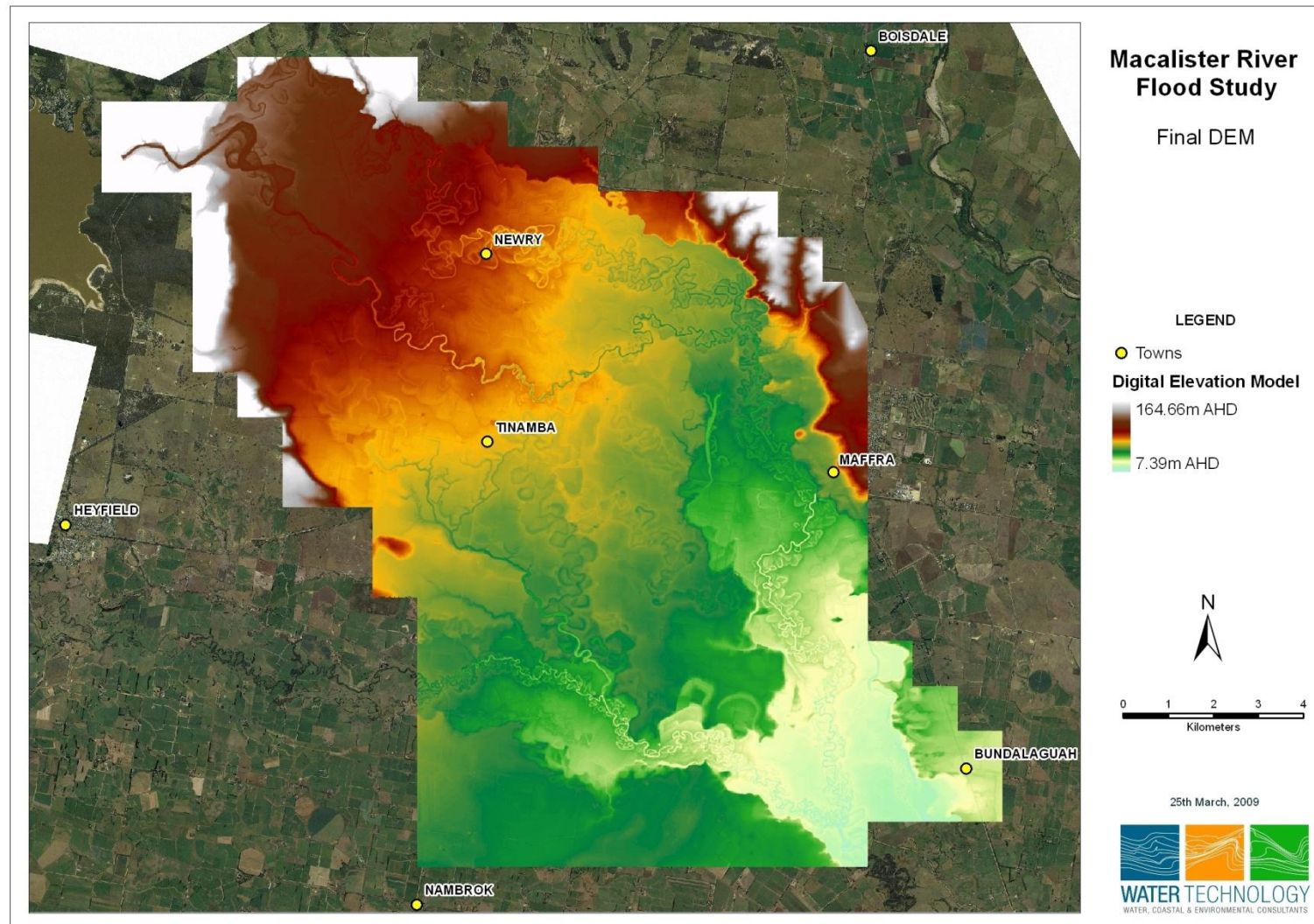


Figure 3-5 Final DEM of the study area

3.3 Topographic Data Comparisons

3.3.1 Overview

The available topographic data included both the LiDAR and LiDAR derived DEM, as well as field survey. The field survey was used to verify the accuracy of the LiDAR. Verification of topographic data from various sources provided guidance on the suitability of the data for use in the hydraulic modelling. The verification was undertaken for both the interim and final LiDAR datasets.

3.3.2 Available Field Survey Data

Field survey data was available from two sources. SKM completed a field survey as part of the Macalister River Flood Study Stage 1 (WBM, 2008). The second field survey dataset was provided to Water Technology by Southern Rural Water. It is believed this survey was completed in 2005. The available field survey data can be classified as strings or point datasets.

The following two sets of string data were available:

- *Waterway Cross Sections*: Cross sections were surveyed bank to bank along the Macalister River. These stretch from just downstream of Lake Glenmaggie, to just north of the Thomson River. There were eight cross sections surveyed.
- *Levee Strings*: Long sections were surveyed along selected levee banks, irrigation channels, and road sections.

The following two sets of point data were available:

- *Flood level spot heights*: These were measured during the flooding in 2007. Only levels denoted natural surface were used for comparison.
- *Point grid*: A small area, about 1 by 1.5km, was densely surveyed in an approximate grid pattern. This area is about halfway between Maffra and Bundalaguah.

Both the string and point data were used to verify the LiDAR.

To compare elevations, three dimensional strings and spot heights were extracted directly from the field survey data. TIN elevations, based on the interim and final LiDAR datasets, were stamped on top of them. This process ensured all field survey and LiDAR TIN elevations were located in exactly the same position, and no offset errors could be introduced.

3.3.3 Waterway Cross Section Comparisons

Eight cross sections were compared with the LiDAR. The results of elevation comparisons are shown in Figure 3-7. Figure 3-8 indicates the locations of these cross sections along the river.

It was expected that the LiDAR and field survey would match quite closely. The only expected areas of significant discrepancy were likely to be below the water level within the river channel, as LiDAR cannot penetrate the water column.

From these cross sections the interim LiDAR appeared to be consistently higher than the survey data. On close inspection, the average difference between surveyed and LiDAR elevations was approximately 150mm. However, the final LiDAR appears to correlate with the survey far more closely. In some areas the final LiDAR is still slightly higher, which could be attributed to vegetation and grasses. There are also some discrepancies where there are sharp changes in elevation, which is expected given the smoothing nature of the TIN data, and may also be a factor of the coarser field survey.

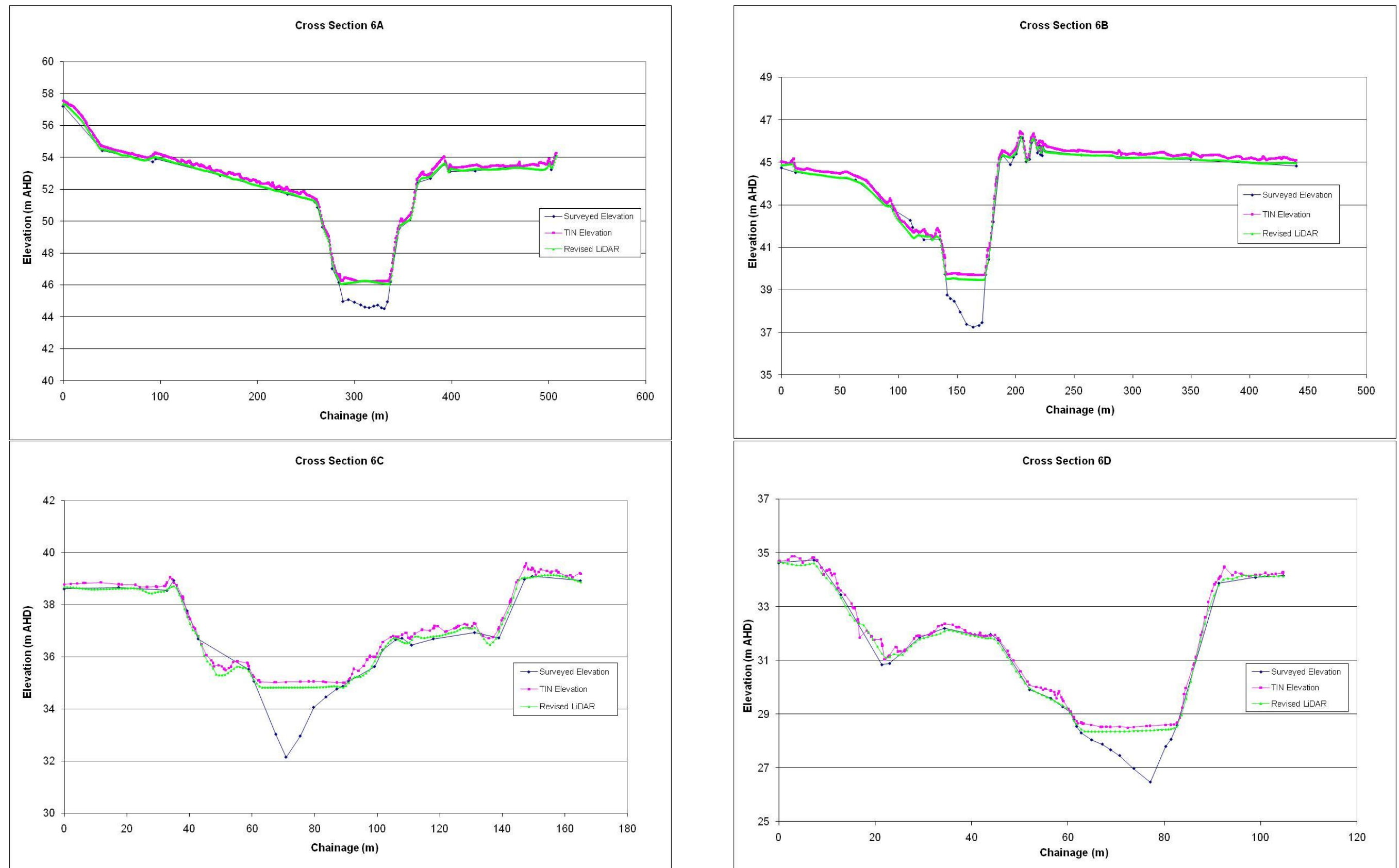


Figure 3-6 Comparison plots between SKM's surveyed cross sections and LiDAR (Cross sections 6A to 6D)

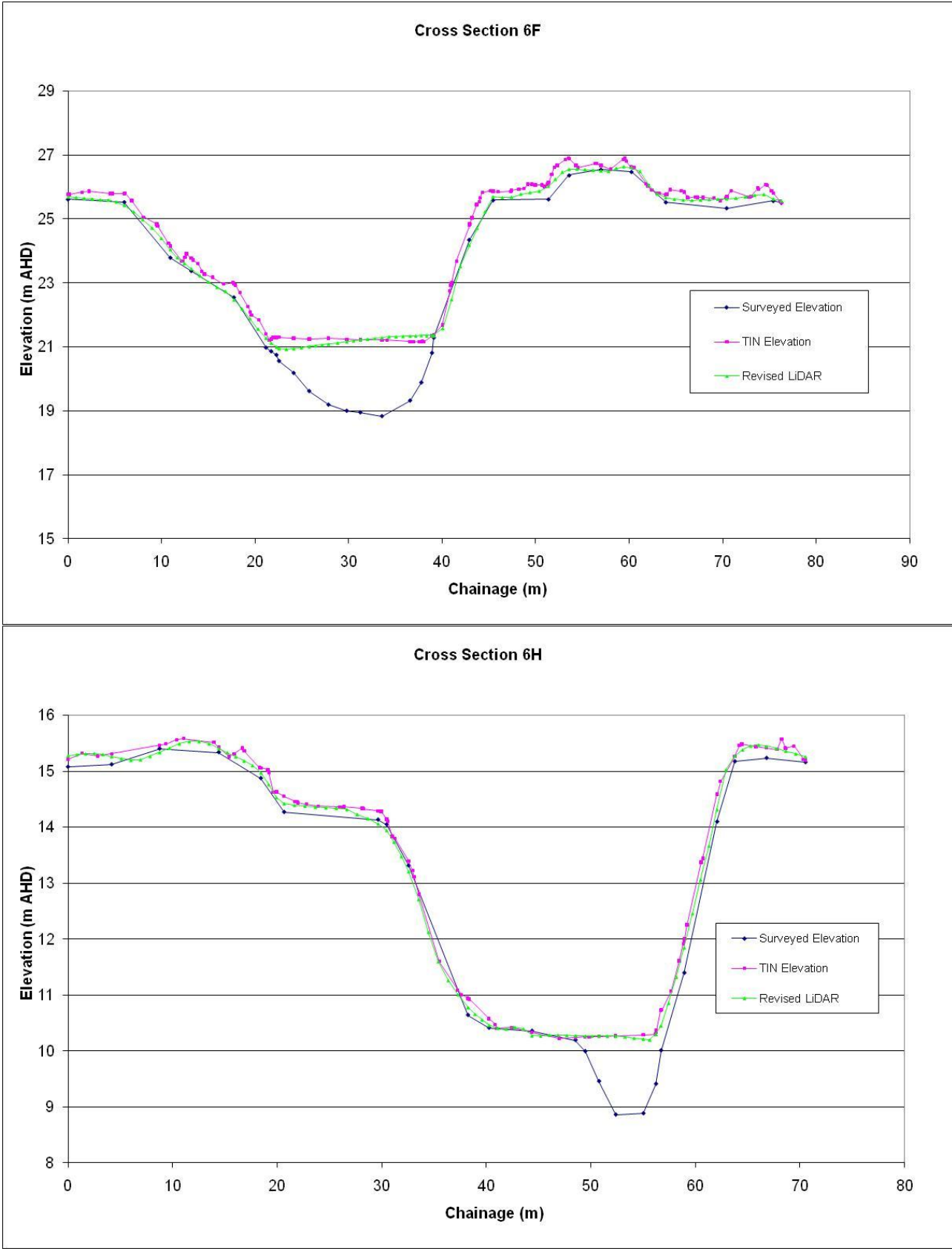
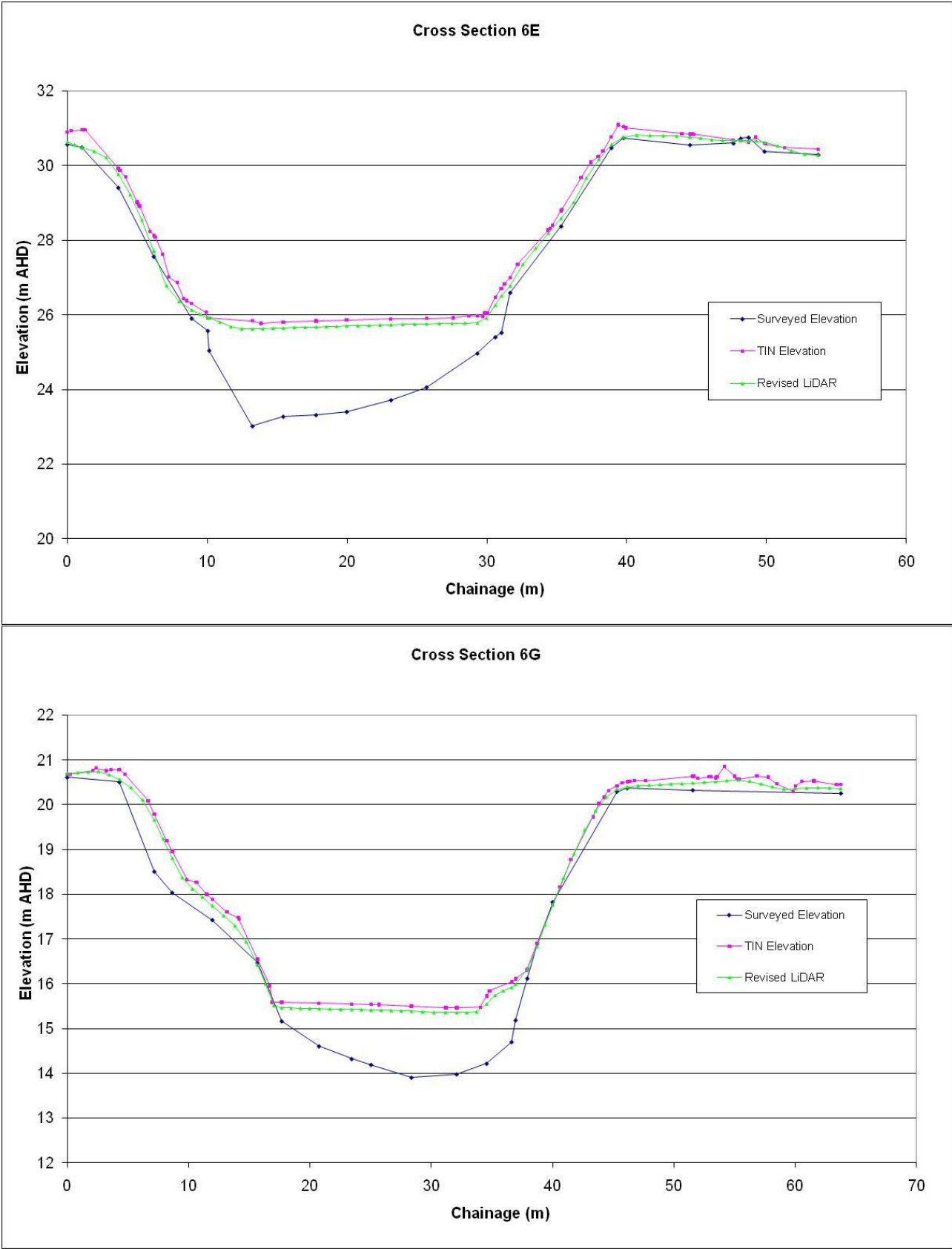


Figure 3-7 Comparison plots between SKM’s surveyed cross sections and LiDAR (Cross sections 6E to 6H)



Figure 3-8 Surveyed cross section locations

3.3.4 Levee Strings Comparison

Comparisons were made between surveyed levee and road long sections in the same manner as for the waterway cross sections. Field survey was available for levee crests, natural surface elevations along the levee banks, some lengths of road crown, and for irrigation channels. The results of these comparisons are shown below in Figure 3-9. Locations of the long sections are shown in Figure 3-10.

As for the cross sections, these comparisons indicated the interim LiDAR was consistently around 100 mm higher than field surveyed elevations. There was considerably less variation along the road string. The final LiDAR showed no consistent trend.

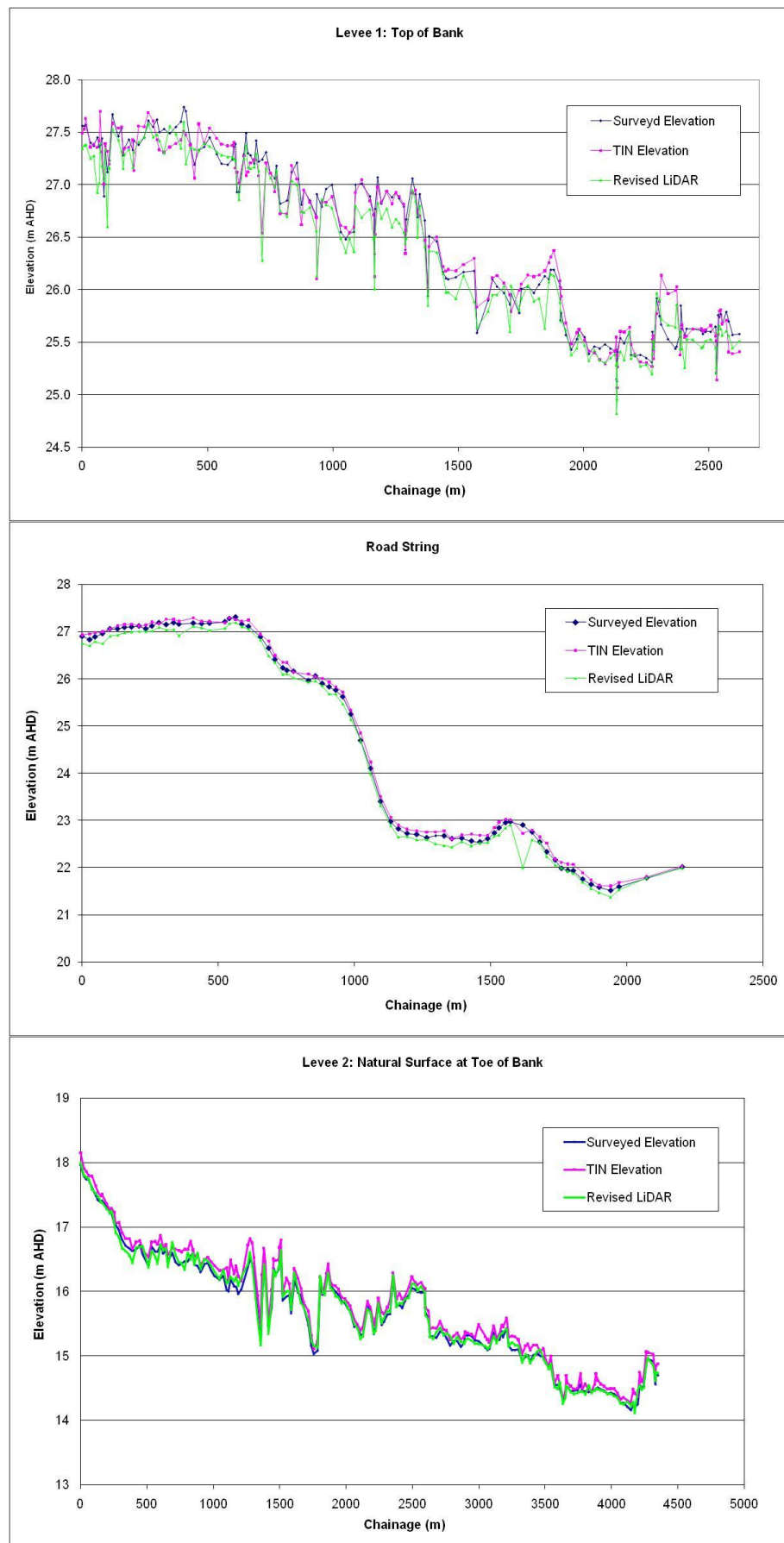


Figure 3-9 Comparison plots between surveyed longsections and revised LiDAR

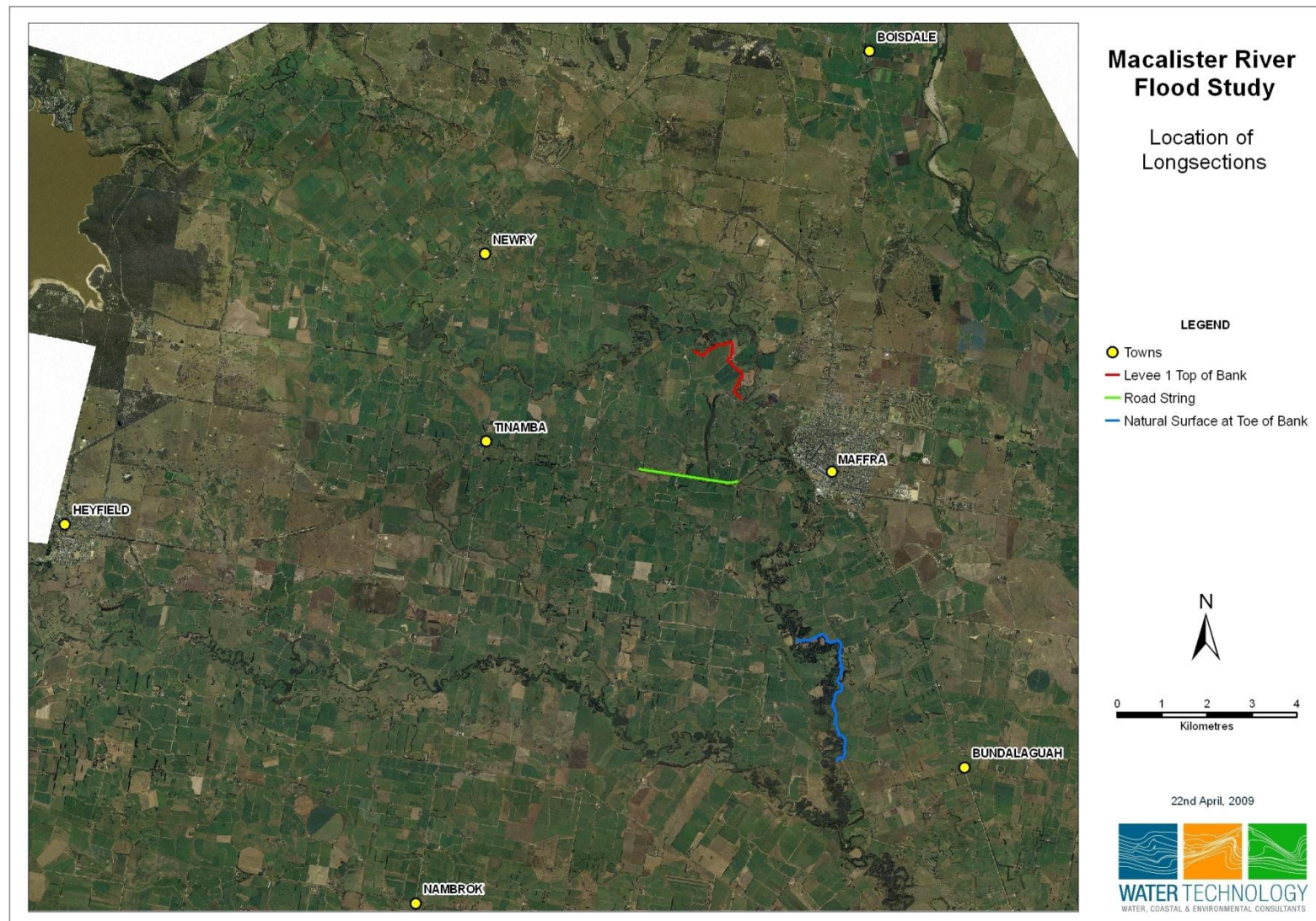


Figure 3-10 Location of longsection strings

3.3.5 Flood Level Comparison

Comparisons were undertaken for the flood levels noted as “natural surface”. In all, there were 148 points available for comparison.

Table 3-2 compares a selection of surveyed flood levels to the interim and revised LiDAR. Figure 3-11 indicates the location and ID number of each spot height within the MID. Figure 3-12 provides a spatial representation of the differences between the field survey and interim LiDAR, as shown by the colour coding. A negative notes where the LiDAR data was lower than the field survey. Figure 3-13 shows the same comparison for the revised LiDAR.

Table 3-2 Flood level elevation comparison

Flood Level ID	Easting	Northing	Surveyed Elevation	Interim LiDAR Elevation	Revised LiDAR Elevation	Difference Interim-Surveyed	Difference Revised-Surveyed
1	497592.54	5798898.38	23.59	23.73	23.67	0.14	0.09
2	494850.78	5799344.38	27.50	27.53	27.48	0.03	-0.02
3	492362.29	5799701.05	31.88	32.02	31.88	0.14	0.00
4	488379.81	5799382.39	33.35	33.34	33.08	-0.01	-0.27
5	495601.31	5797742.92	22.35	22.43	22.42	0.08	0.07
6	496502.06	5795009.73	19.59	19.66	19.53	0.07	-0.06
7	492437.96	5794706.44	25.33	25.43	25.26	0.11	-0.07
8	494453.45	5799704.68	28.21	28.30	28.23	0.09	0.02
9	492285.53	5799166.55	30.68	30.72	30.61	0.03	-0.07
10	490291.71	5795985.16	27.18	27.24	27.10	0.06	-0.07
11	495860.91	5796000.19	20.49	20.54	20.52	0.04	0.03
12	491167.47	5797020.76	26.93	26.97	26.88	0.04	-0.05
13	495314.54	5796558.46	20.31	20.32	20.23	0.00	-0.08
14	491286.77	5796553.94	26.91	27.09	26.90	0.18	0.00
15	496077.16	5799563.77	25.60	25.82	25.67	0.22	0.07
16	494807.11	5799014.08	27.54	27.61	27.39	0.07	-0.15
17	488912.39	5796654.18	28.92	29.03	28.86	0.11	-0.07
18	492132.97	5799625.89	32.14	32.20	31.97	0.06	-0.17
19	495969.05	5796210.96	20.52	20.64	20.43	0.12	-0.09
20	491492.11	5794069.04	24.80	24.86	24.79	0.07	-0.01

Similar to the cross section comparison, the flood level comparison found that the interim LiDAR was consistently higher than the field survey. Figure 3-13 shows that these differences appear to vary in magnitude, without any strong spatial trend.

The final LiDAR is, generally, lower than the interim and closer to the surveyed ground spot heights. Varying local differences can be expected when comparing spot heights, as it is unlikely the elevation value in each cell will correspond to the precise feature which was surveyed. An example may be surveyors measuring the elevation of the edge of bitumen, but the LiDAR cell indicating the top of the kerb. As with the interim LiDAR, there does not appear to be a strong spatial trend in these differences.

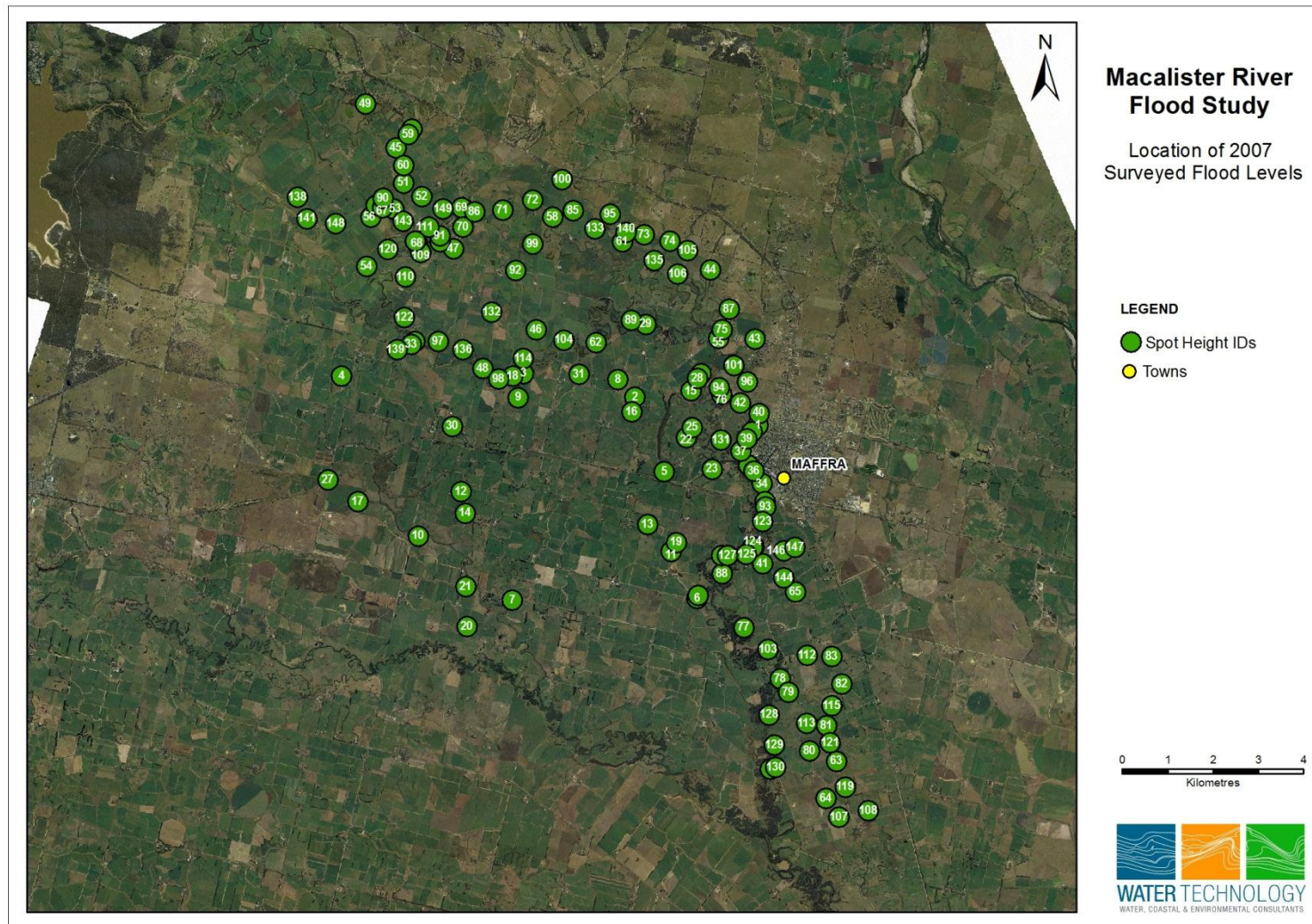


Figure 3-11 Location of 2007 flood level spot heights

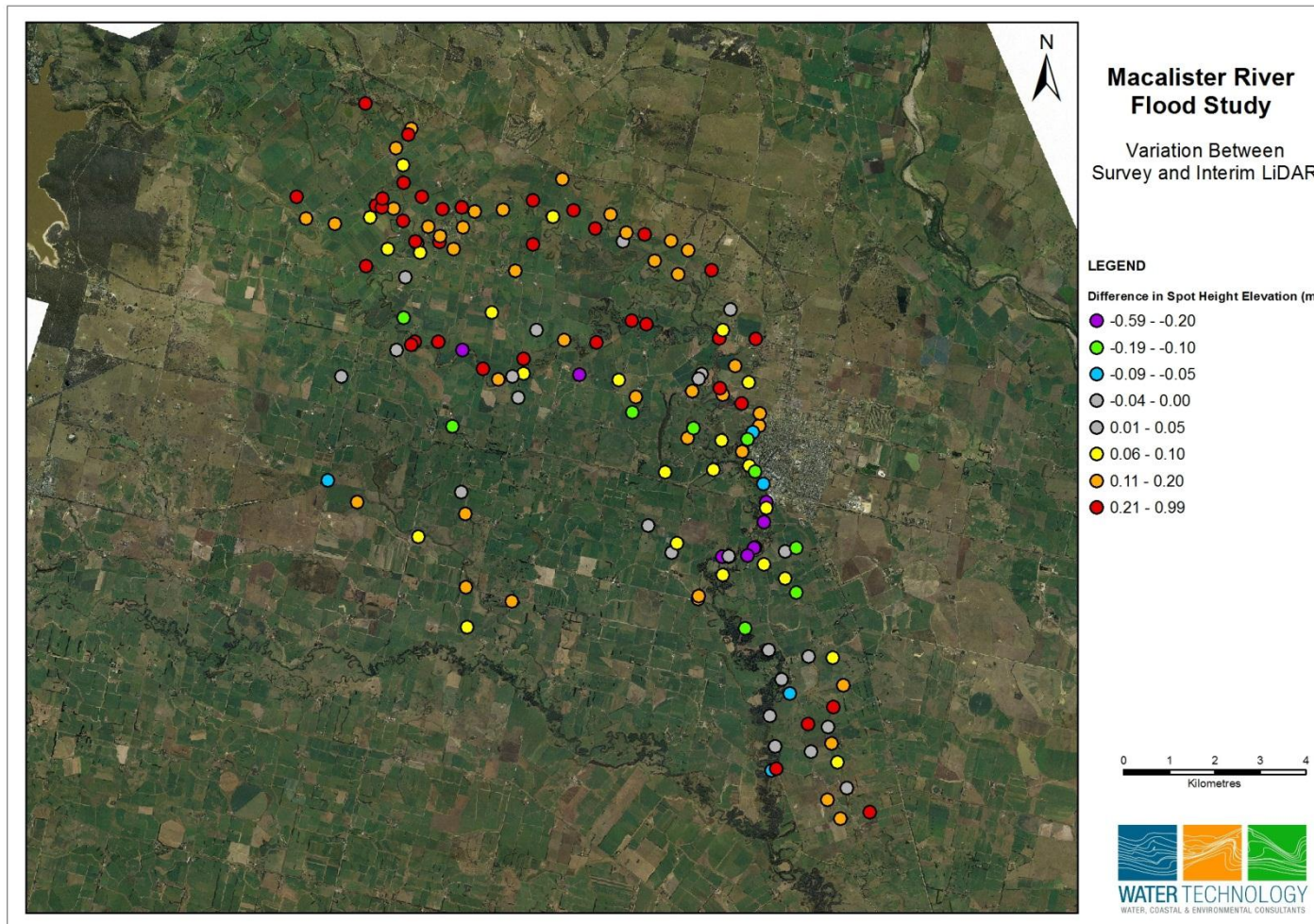


Figure 3-12 Difference in elevation between 2007 surveyed flood levels and Interim LiDAR

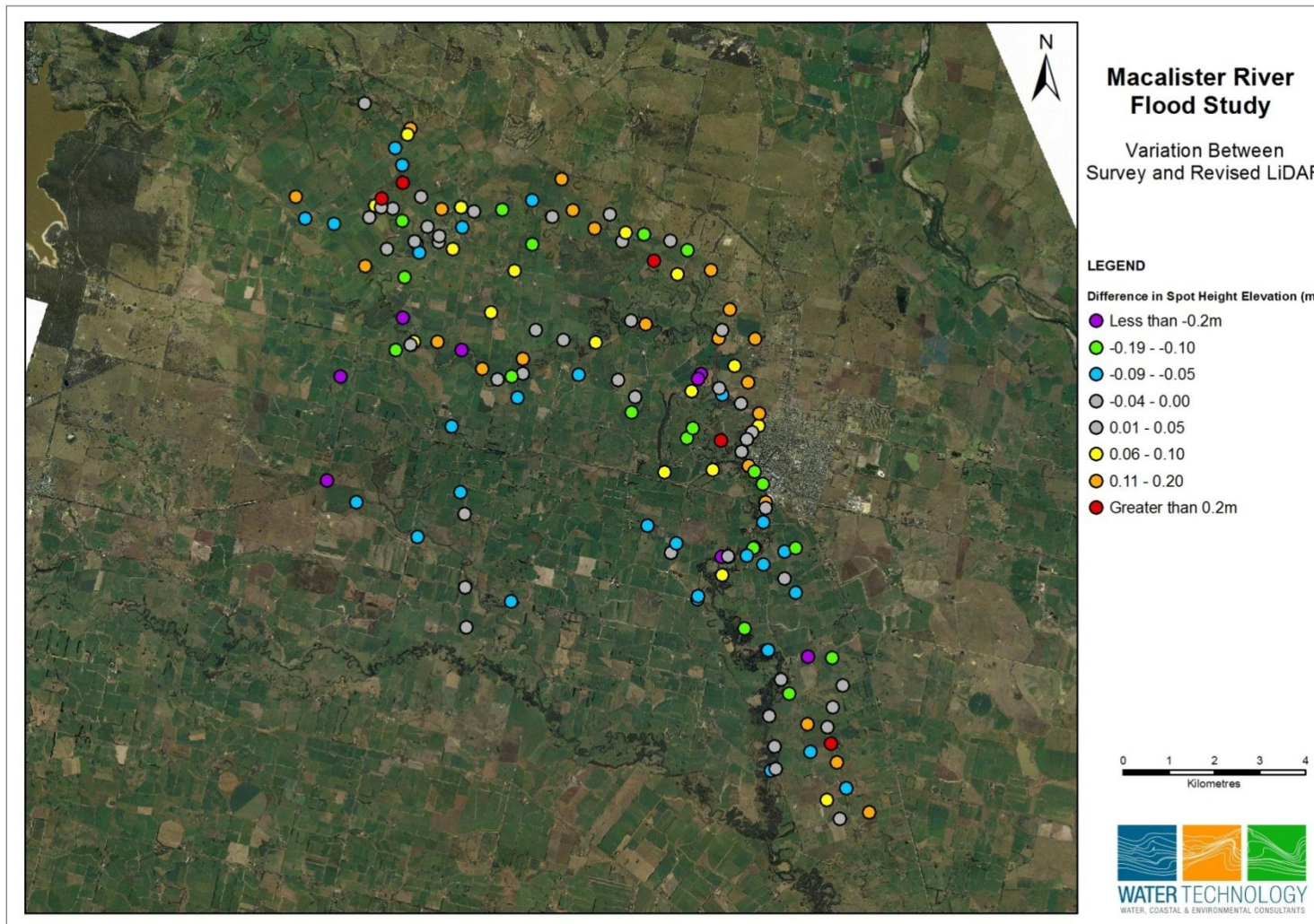


Figure 3-13 Difference in elevation between 2007 surveyed flood levels and revised LiDAR

Figure 3-14 shows a histogram of the difference, which confirms the interim LiDAR data was consistently higher than the field survey, but that the revised LiDAR, overall, was closer to the surveyed values. The frequency of the differences centering around zero in a roughly normal shaped distribution indicates the improvement in results.

Statistics across the entire flood data set are below in Table 3-3.

Table 3-3 Flood Level Comparison Statistics

Item	Interim LiDAR	Final LiDAR
Mean difference (m)	0.162	-0.012
Median difference (m)	0.137	-0.008
Standard Deviation (m)	0.139	0.139
No. points within $\pm 0.1\text{m}$ of surveyed level	79 out of 148	118 out of 145

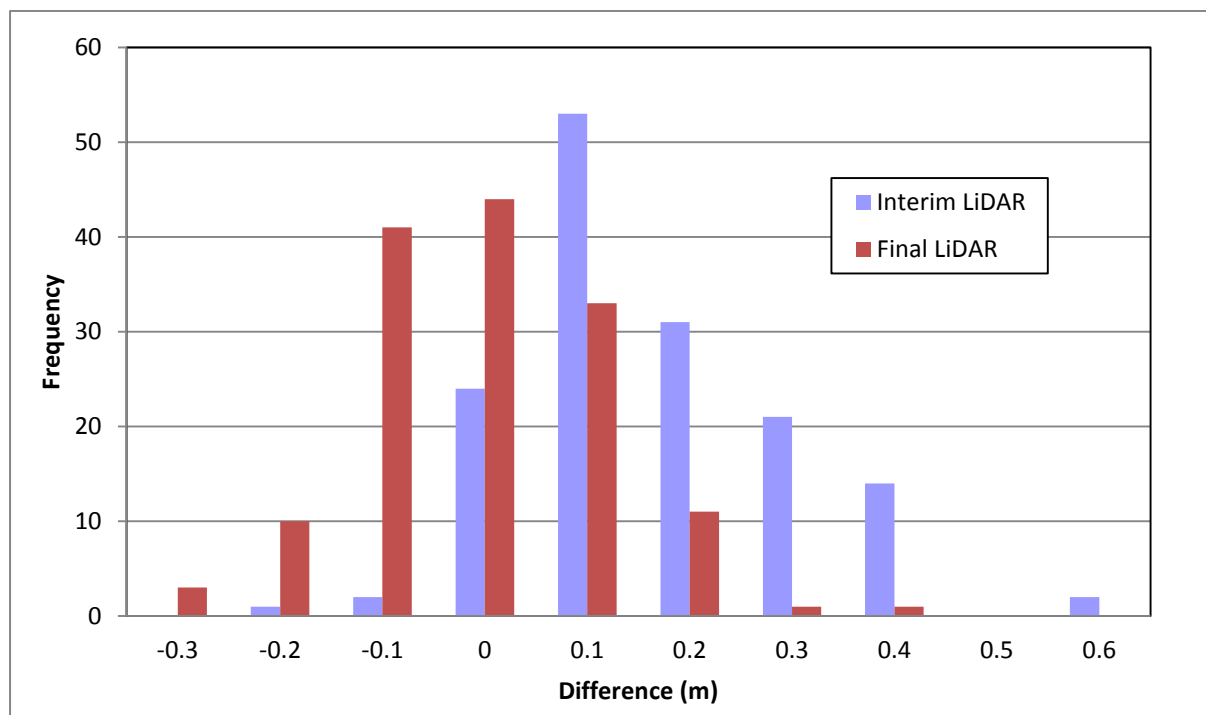


Figure 3-14 LiDAR verification histogram of natural surface flood levels

3.3.6 Point Grid Comparison

As for the flood levels, a small area of point field survey was compared to the LiDAR. Given that the surveyed area was far smaller in this case, it was hoped there would be a more consistent difference in elevation between the two data sources (as opposed to points being compared across a large floodplain). However this was not the case.

Table 3-4 provides a small sample of the comparison between point and LiDAR elevations. Again, the elevation differences varied randomly throughout the survey area, which is shown Figure 3-15.

Table 3-4 Point grid elevation comparison

Point ID	Easting	Northing	Surveyed Elevation	Interim LiDAR Elevation	Revised LiDAR Elevation	Difference Interim-Surveyed	Difference Revised-Surveyed
0	500049.29	5794513.74	21.20	21.25	21.22	0.052	0.022
1	500053.63	5794512.75	21.07	21.13	21.04	0.060	-0.029
2	500065.92	5794475.64	21.01	21.04	21.15	0.032	0.139
3	500077.76	5794438.71	21.00	21.12	21.09	0.122	0.087
4	500090.18	5794402.61	20.94	21.14	21.06	0.199	0.124
5	500102.28	5794365.82	20.94	21.28	21.24	0.339	0.296
6	500114.02	5794330.04	21.19	21.29	21.20	0.099	0.010
7	500123.76	5794299.35	21.45	21.62	21.49	0.174	0.039
8	500088.47	5794293.96	21.24	21.40	21.27	0.157	0.029
9	500076.21	5794327.56	21.20	21.35	21.22	0.153	0.015
10	500064.97	5794360.87	21.02	21.08	21.04	0.058	0.021
11	500054.21	5794393.89	21.00	21.03	20.99	0.029	-0.009
12	500044.56	5794427.78	21.04	21.09	21.03	0.047	-0.009
13	500034.40	5794465.84	21.24	21.29	21.23	0.052	-0.009
14	500017.62	5794429.03	21.21	21.39	21.17	0.180	-0.038
15	500020.49	5794392.98	21.10	21.21	21.16	0.110	0.057
16	500023.11	5794357.56	21.03	21.09	21.03	0.057	0.003
17	500023.25	5794323.36	21.35	21.41	21.37	0.064	0.022
18	500015.78	5794296.66	21.59	21.71	21.64	0.124	0.050
19	499981.48	5794292.50	21.91	22.01	21.92	0.096	0.008
20	499972.57	5794313.04	21.62	21.74	21.63	0.123	0.010

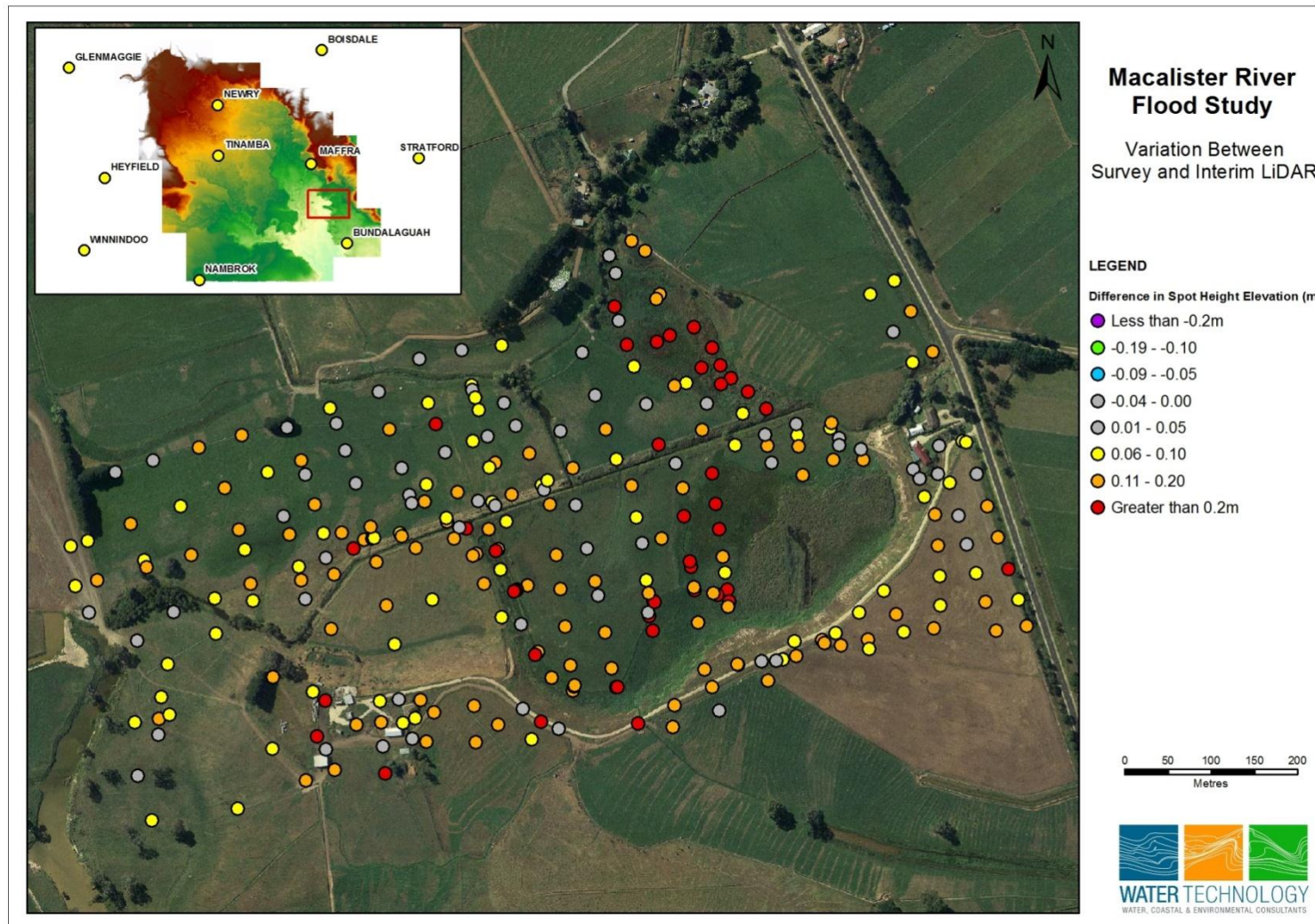


Figure 3-15 Difference in elevation between 2007 surveyed spot height elevations and interim LiDAR

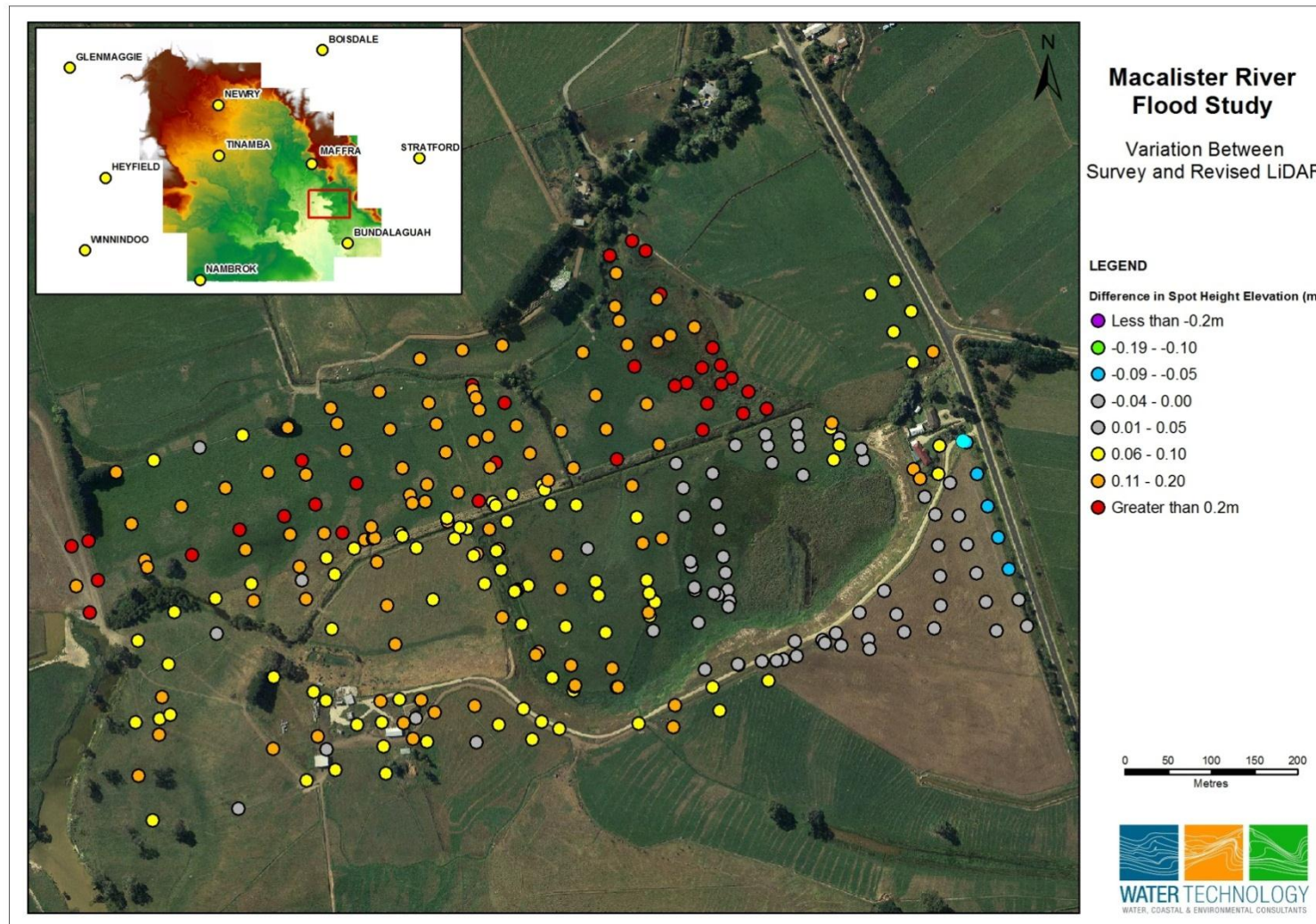


Figure 3-16 Difference in elevation between 2007 surveyed spot height elevations and revised LiDAR

This point set was more densely located than the flood levels, so differences in elevation expected to be more similar than a comparison made across the flood plain. While there was still a large spatial variation in the amount of difference, the statistics for this point set was more correlated.

Figure 3-17 shows a histogram of the difference, which again highlighted the higher interim LiDAR data, and the more consistently correlated revised LiDAR.

Statistics across the entire data set are shown in Table 3-5.

Table 3-5 Point Grid Comparison Statistics

Item	Interim LiDAR	Final LiDAR
Mean difference (m)	0.126	0.018
Median difference (m)	0.105	0.0
Standard Deviation (m)	0.125	0.118
No. points within $\pm 0.1\text{m}$ of surveyed level	210 out of 291	260 out of 291

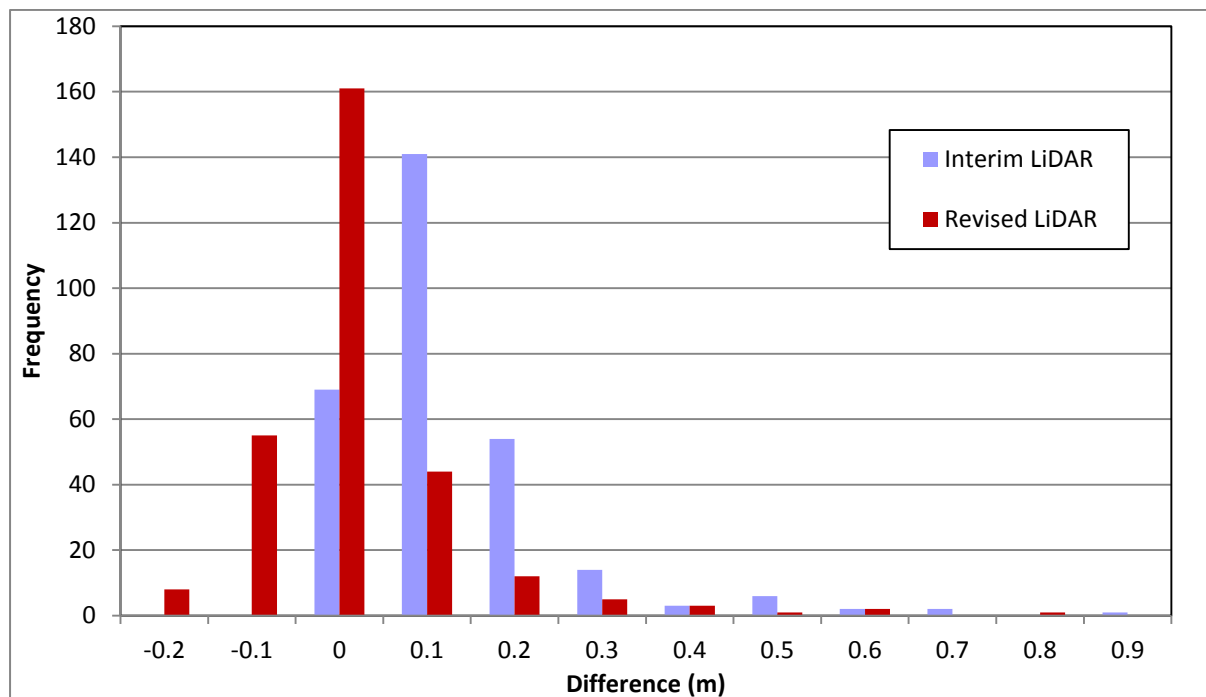


Figure 3-17 LiDAR verification histogram of spot heights near Maffra

3.4 Summary

The above verification suggests that the interim LiDAR is generally higher than the field survey, to a spatially varying and differing amount, across the entire floodplain. On natural surfaces, the variation is in the order of 100mm. However along flat, built assets such as roads, the field and LiDAR data sets were found to be in closer agreement.

The final, reissued LiDAR, was found to agree with the field survey much more closely than the interim LiDAR. Small local variations are to be expected due to the nature of comparing field survey to a topographic raster, no matter how fine the resolution of the raster.

As a result of this analysis, the final revised LiDAR was deemed acceptable for use in the hydraulic modelling of the Macalister floodplain.

4. HYDROLOGIC ANALYSIS

4.1 Overview

This section defines the scope of the required hydrologic analysis, reviews the previous hydrologic investigations, and details estimation of the required design flood hydrographs.

The study brief required the determination of design flood hydrographs for the 20, 50 and 100 year event plus the Probable Maximum Flood (PMF) event. Furthermore, the study brief required three other events for flood response purposes. As proposed in the tender response, the three other flood events are the 5, 10 and 200 year events.

4.2 Review of Previous Hydrologic Analysis

4.2.1 BoM – URBS Modelling

The Bureau of Meteorology (BoM) has undertaken runoff routing modelling for the Macalister River catchment. This modelling applied the runoff routing model URBS. The modelling is part of the flood forecasting framework for the Thomson-Macalister catchment. BoM provided a suite of URBS model files to this study.

The Macalister River Catchment URBS model provided by BoM for the catchment covers the catchment upstream of Lake Glenmaggie. The Macalister River catchment below Lake Glenmaggie is part of a separate URBS model covering the lower reaches of Macalister and Thomson Rivers.

The BoM Macalister River catchment URBS model consists of 37 sub-catchments above Lake Glenmaggie. The BoM URBS outputs modelled hydrographs at the following locations:

- Macalister River at Glencairn
- Macalister River at Licola
- Macalister River at Stringybark Creek
- Glenmaggie Creek at the Gorge
- Macalister River downstream of Glenmaggie

BoM has undertaken calibration of the URBS models to a number of large flood events. Unfortunately, due to malfunction of the gauges a calibration to the June 2007 event was not possible. BoM provided URBS input and output files for a number of calibration and verification historical flood events. The calibration has been undertaken to gauges upstream of Lake Glenmaggie: Macalister River at Stringybark Creek and Glenmaggie Creek at the Gorge. These upstream gauges have a combined upstream catchment area of 1681 km². This compares to the catchment area to the gauge downstream of Glenmaggie of 1891 km². The operation of the radial gates at Lake Glenmaggie adds complexity to the routing of flows through the storage, and in turn the modelling of flow hydrographs downstream of Lake Glenmaggie.

To assess the performance of the BoM URBS model, observed and modelled flow hydrographs for the upstream gauges were compared for a selection of three large flood events: June 1978, October 1985 and September 1993. There are other flood events used as calibration events by BoM. Figure 4-1, Figure 4-2 and Figure 4-3 show the comparison of modelled and observed flow hydrographs, and Table 4-1 provides the BOM URBS model parameters.

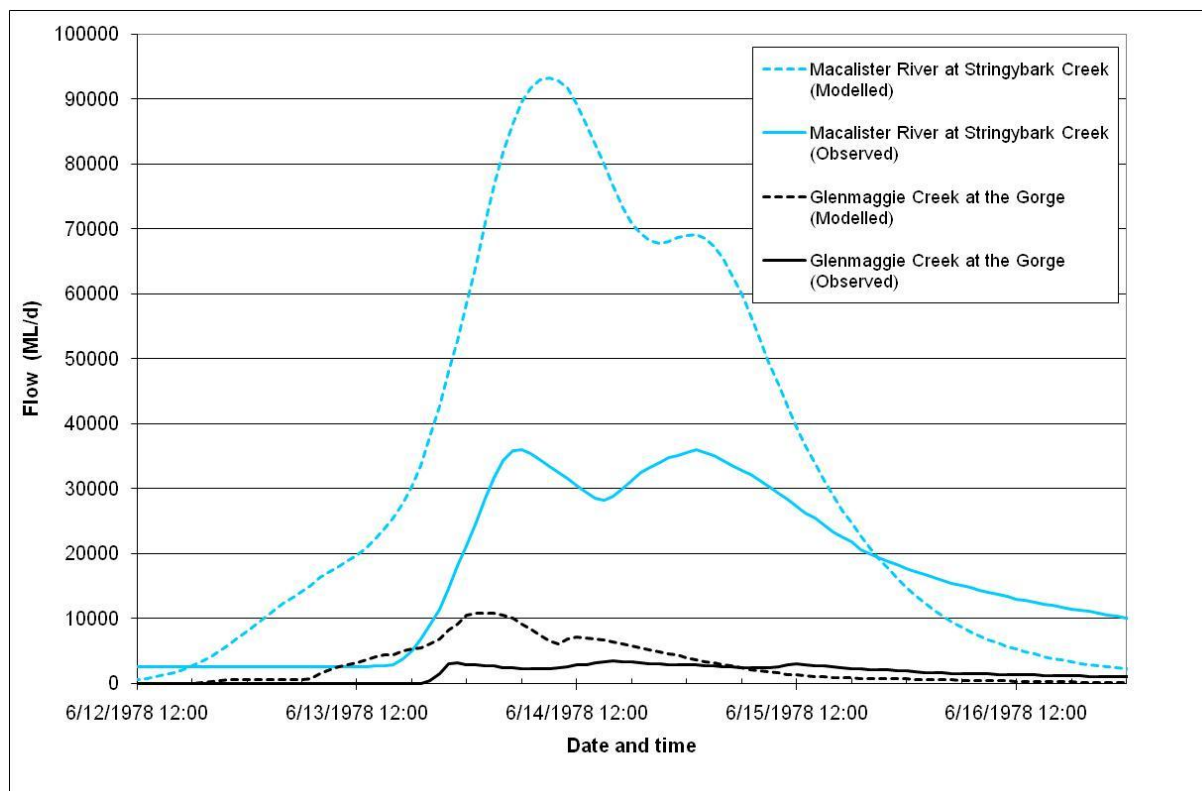


Figure 4-1 BoM URBS model calibration – June 1978

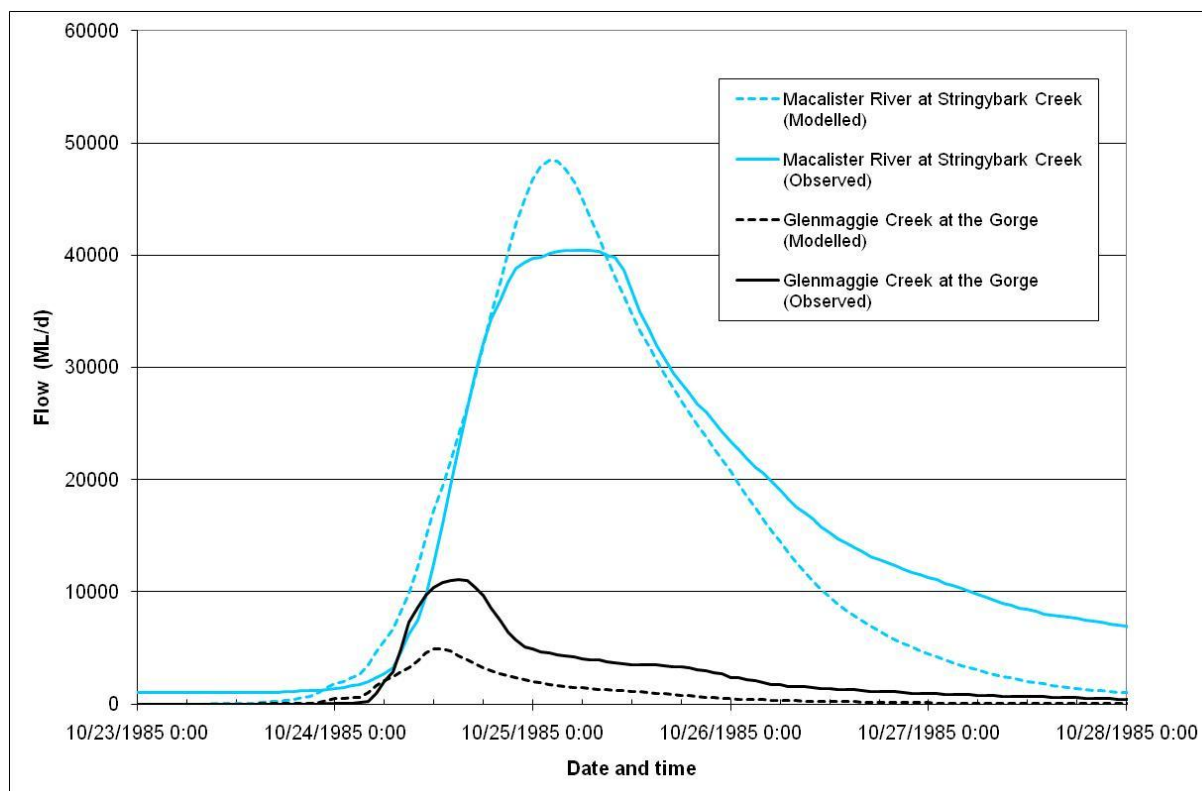


Figure 4-2 BoM URBS model calibration – October 1985

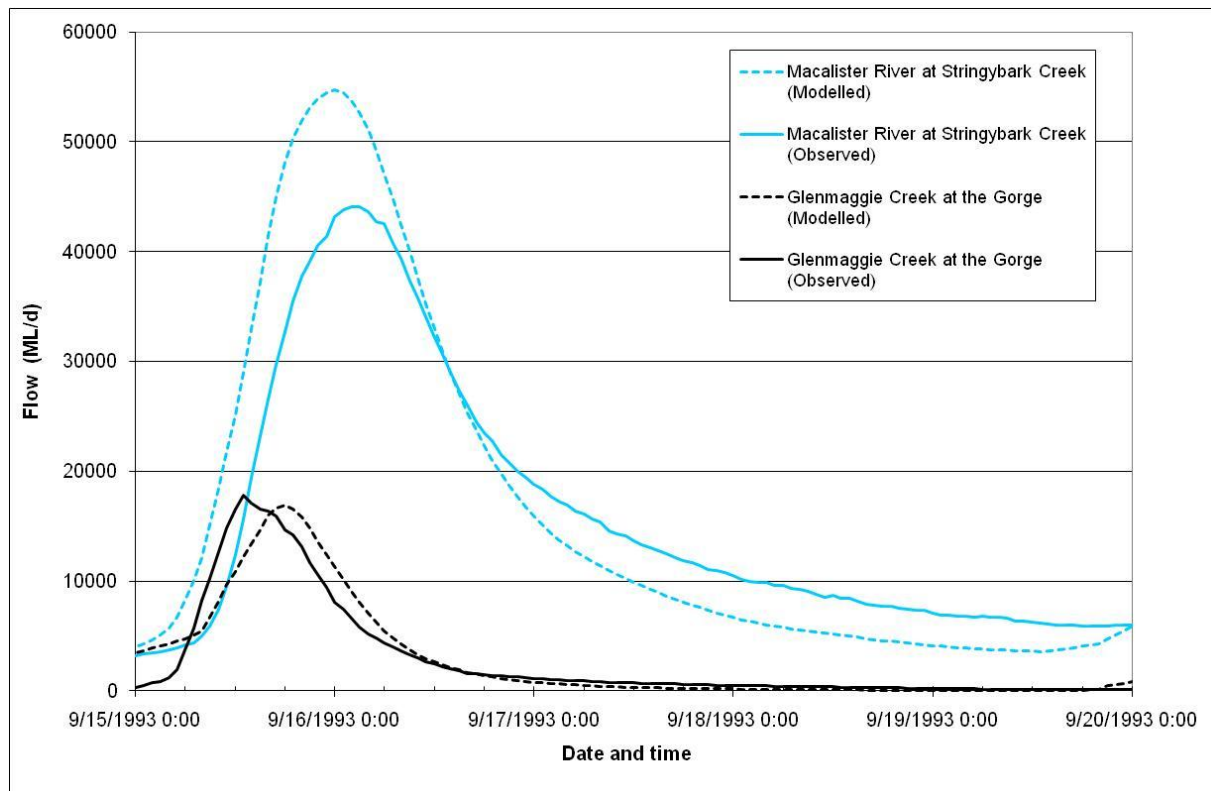


Figure 4-3 BoM URBS model calibration – September 1993

Table 4-1 BoM URBS model calibration – URBS model parameters

Event	URBS routing parameters			URBS rainfall loss	
	alpha	beta	m	Initial loss (mm)	Continuing loss (mm/h)
June 1978	0.2	2.5	0.8	0.0	0.99
October 1985	0.27	2.5	0.8	0.0	1.98
September 1993	0.23	2.5	0.8	14.9	1.98

The comparison of the modelled and observed hydrographs for the selected calibration events showed mixed results. For the June 1978 event (largest available event), the BoM URBS model considerably over-estimated the observed peak flows at both Stringybark Creek and the Gorge. Furthermore, the observed hydrograph shapes were poorly re-produced. Similar over-estimation of the peak flows occurred at Stringybark Creek for both October 1985 and September 1993 events. However, the observed hydrograph shapes were reasonably re-produced for the October 1985 and September 1993 events at Stringybark Creek. At the Gorge, the October 1985 peak flow was under-estimated, and the September 1993 peak flow was modelled well.

From the selected calibration events, the calibration performance of the BoM URBS model was poor, with peak flows and hydrograph shape generally not well simulated. Section 4.3 discusses the refinements undertaken to the BoM URBS model for use in estimating design floods for this study.

Examination of the BoM URBS model indicated that gate operation at Lake Glenmaggie was considered only in a simplified fashion. The model incorporated Lake Glenmaggie as an ungated structure (i.e. free overflow weir).

The following advice regarding gate operations were provided by SRW
:

'Gate operating rules' as such only apply to the opening sequence to pass any given flow. Gate openings are dependent on inflows and storage level, in particular ensuring the storage does not go above Full Supply Level. The Storage Operators and the Incident Management team make decisions on releases based around these factors as well as being aware of downstream impacts and trigger points. Ideally flows are increased steadily on the rising leg of a flood, however as every flood is different and flow patterns differ dramatically from one event to the next, outflows can alter and therefore release 'rules' as such can only be a guide.

For design flood purposes, I would imagine the conservative approach would be to assume the storage is full at the commencement of the flood. The storage will then be operated so that outflows are as close to inflows as possible.

4.2.2 WBM (2008) - Stage 1 Flood Study

The key elements of the WBM (2008) hydrologic analysis included:

- Flood frequency analysis of available streamflow data downstream of Lake Glenmaggie.
- Runoff routing modelling (URBS) for design rainfall events.

The flood frequency analysis considered two data periods, 1960–2007 (period of available instantaneous data), and 1919–2007 (entire record). The frequency analysis was undertaken using both annual peak flow and partial peak flow series. A log Pearson Type III distribution fitted by maximum likelihood was employed. Table 4-2 displays the design peak flow estimates for the Macalister River downstream of Lake Glenmaggie from the analysis.

The flood frequency analysis outlined in WBM (2008) followed the general principles contained in Australian Rainfall and Runoff, ARR87 (I.E. Aust 1987). However, there are several aspects of the approach that were not clearly reported. These aspects include the following:

- Figure 5-2 (WBM 2008) displays a flood frequency curve for the Macalister River downstream of Lake Glenmaggie. However, the observed peak flows are not plotted, and hence the goodness of fit of the frequency curve is unable to be assessed.
- It is unclear when using the entire period of streamflow data in the frequency analysis, whether the mean daily flow data (for the period prior to 1960) was adjusted to reflect an estimated instantaneous peak flow.

The effect of the above aspects on the peak flow estimates is difficult to assess without additional analysis.

Further to the above aspects, WBM (2008) considered peak flows without consideration of flood volumes. Given the extensive nature of the floodplain, flood volume in addition to peak flow will influence flood behaviour.

The runoff routing modelling undertaken by WBM (2008) employed a modified version of the BoM URBS model. The WBM (2008) URBS model extended the BoM URBS model to include the Macalister River between Lake Glenmaggie and the Thomson river confluence. WBM (2008) noted the adoption of the BoM's URBS model parameters from the BoM's calibration ($\alpha = 0.2$ m = 0.8 $\beta = 2.5$). As discussed previously, the BoM URBS model calibration was considered poor.

The BoM URBS model applied ARR87 design rainfall depths and temporal patterns. WBM (2008) does not provide the location where the design rainfall depths were determined. However, from the URBS model files it appears that the design rainfall depths were evaluated at the centroid of

each subarea. It is also unclear whether Areal Reduction Factors (ARF) were applied to the rainfall depths.

For the design flood hydrograph estimation, WBM (2008) adjusted the initial and continuing losses in URBS to reconcile the URBS model peak flows with the flood frequency analysis. Table 4-2 displays the WBM (2008) design peak flow estimates for the Macalister River downstream of Lake Glenmaggie based on their extended URBS model.

Table 4-2 Macalister River downstream of Lake Glenmaggie – WBM (2008) Design peak flow estimates

ARI (years)	Design peak flow estimates Macalister River downstream of Lake Glenmaggie (WBM 2008) (ML/d)	
	Flood frequency analysis	URBS model
2	12,960	
5	39,744	
10	63,072	
20	88,128	88,128
50	120,960	118,800
100	146,016	147,744
PMPDF	N.A	1,105,920

As noted previously, WBM (2008) did not assess flood volumes in their flood frequency analysis. This results in no comparison of the flood volumes in the adopted design flood hydrographs against the flood frequency estimates.

As per the BoM URBS modelling, it appears that the treatment of the gate operation at Lake Glenmaggie was considered in a simplified fashion. The WBM URBS model modelled Lake Glenmaggie as an ungated structure (i.e. free overflow weir).

4.2.3 SMEC (2007) – Review of the June 2007 Event

SMEC was engaged by Southern Rural Water to undertake a review of the hydrology of the Glenmaggie Dam catchment following the June 2007 event. The review included:

- Development of an inflow flood frequency curve for Glenmaggie Dam;
- Development of an outflow flood frequency curve for Glenmaggie Dam;
- Development of a rainfall frequency curve for the Glenmaggie Dam catchment;
- Estimate of the rainfall and runoff frequency of the storm event which occurred in late June 2007;
- Review of the observed range of runoff parameters associated with the RORB rainfall runoff model of the catchment;
- Calibration of the RORB model to two flood events; and
- Development of a revised flood frequency curve for the inflows and outflows associated with Glenmaggie Dam.

SMEC (2007) provided an update to and additional analysis for the hydrology modelling undertaken as part of the Glenmaggie Dam Design Review (SMEC, 1999). The work focused on the derivation of inflow frequency curves rather than outflow frequency curves.

Inflow Frequency

The flood frequency curve for the Glenmaggie Dam catchment inflow was predominately generated using a summation of gauged flow data from the Glenmaggie Dam catchment. It was estimated that the June 2007 inflow flood event has an Annual Return Interval (ARI) of between 1 in 100 to 1 in 300. The data record used to generate the inflow flood frequency curve involves a data infilling process which is subject to inaccuracies. The estimate of the return period of the June 2007 inflow flood event was indicative only and may be considered to have a medium level of accuracy.

Outflow Frequency

The study determined a flood frequency curve for the Glenmaggie Dam outflow using the downstream gauging station. They used a streamflow data set which extended back to 1919. Again, the streamflow data record between 1919 and 1964 was a mixture of instantaneous record and mean daily record. It was noted that the mean daily flow record was represented by a single record that was measured at the same time each day and as such may underestimate the peak flow that occurred that day. Therefore, the resulting flood frequency curve may plot with lower flow values than would be case if the entire record was instantaneous daily flow data. Overall, it was considered that a better representation of the flood frequency relationship would be achieved through the use of a longer period of record than would be the case with a shorter period of record incorporating the instantaneous flow record only.

The June 2007 outflow flood event was estimated to have an ARI of 1 in 100. This estimate was considered to have a high degree of accuracy. The RORB model developed for the study was not applied to estimate design flood hydrographs downstream of Lake Glenmaggie.

SMEC (2007) did not consider estimating daily peak instantaneous flows by establishing a relationship between the mean daily and peak daily instantaneous flows. In addition, the flood frequency analysis approach adopted is not specified.

Rainfall Frequency

The rainfall frequency curve for the entire Glenmaggie Dam catchment was generated using standard procedures recommended by the Bureau of Meteorology. The recorded rainfall across the catchment was used to establish which annual return period best represented the June 2007 rainfall event. It was estimated that the June 2007 rainfall event has an annual return period of around 1 in 50.

Rainfall Runoff Modelling

Calibration of the RORB model to the June 2007 flood event indicated that the runoff rate from the catchment was observed to be substantially higher than average for typical Glenmaggie Dam catchment conditions. This provided support for the view that the bushfires have altered the catchment runoff characteristics with respect to typical conditions.

Additional RORB model calibrations and analysis of a range of observed runoff routing data indicated that there was support for revising the catchment routing parameters and losses to be applied when modelling design flood events. It was recommended that a RORB kc value of 70 be adopted for use in developing design floods. It was considered that current conditions are likely to be better represented by a kc value of 35.

4.3 Current Hydrological Analysis Approach

The hydrological analysis approach adopted for the present study builds on the BoM, WBM (2008) and SMEC (2007) investigations. The key elements of the present approach were as follows:

- Peak flow flood frequency analysis of observed flows downstream of Lake Glenmaggie,
- Flood volume flood frequency analysis of observed flows downstream of Lake Glenmaggie,
- URBS runoff routing modelling for design rainfall events with model parameters adjusted to reconcile URBS model peak flow estimates and flood frequency analysis ,
- Design flood hydrograph estimation using the URBS model to determine 5, 10, 20, 50, 100 and 200 year ARI events plus indicative PMF.

Further details of these elements is provided.

4.3.1 Peak Flow Flood Frequency Analysis

Long term streamflow is available downstream of Lake Glenmaggie (Station 225204). This observed streamflow data set consists of mean daily flow data (1919 to 1960) and instantaneous flow data (1960 to date).

To establish an annual peak flow series, a relationship was developed between the mean daily flows and daily instantaneous peak flow for the concurrent period of record. This relationship was then applied to mean daily flows to yield an estimated daily instantaneous peak flow.

A flood frequency analysis was then undertaken using the revised annual peak flow series, and evaluated the goodness of fit of a number of probability distributions. This approach caters for the exclusion of low peak flows that can be considered as non-flood events. The exclusion of low peak flows is particularly applicable downstream of a dam (e.g. Lake Glenmaggie) where the dam operations strongly influence the flow behaviour.

The design peak flow estimates from this approach are considered to be in line with best practice as outlined in the updated draft flood frequency chapter in Australian Rainfall and Runoff (Kuzcrea and Franks 2006).

4.3.2 Flood Volume Flood Frequency Analysis

Similar to the above peak flow frequency analysis, an annual series of peak flood volumes was developed. In the development of the flood volume series, a representative flood duration was selected as 4 days.

A flood volume analysis was undertaken using the annual flood volume, and evaluated a number of probability distributions. Exclusion of low flow peak flood volumes was considered in the same way as for peak flows.

4.3.3 Revised URBS Modelling and Parameter Reconciliation

Formal re-calibration of the URBS model was considered to be outside the scope of this project. The URBS model parameters from BoM and WBM (2008) were therefore adopted. Rainfall losses applied for design flood estimation were adjusted to reconcile the URBS peak flows and flood volumes with the revised flood frequency analysis estimates.

In line with SRW advice, Lake Glenmaggie was assumed full at the commencement of a rainfall event for design flood estimation purposes. Furthermore, the gate operation was assumed to be equivalent to a free overflow (i.e. ungated spillway), in line with previous studies.

4.3.4 Design Flood Hydrograph Estimation

Following re-calibration of the URBS model, it was then used to develop design flood hydrographs for the following events: 5, 10, 20, 50, 100 and 200 year ARI events plus an indicative PMF. These event hydrographs were compared to the previous work.

4.4 Revised Flood Frequency Analysis

4.4.1 Streamflow Sequences

To extend the instantaneous streamflow record for use in the peak flow frequency analysis, a relationship was developed between the monthly maximum mean daily flows and the corresponding monthly maximum instantaneous flow for the period after March 1960. This allowed the annual instantaneous peak flow series to be extended to 1920, giving an additional 40 years of data to be included in the peak flow frequency. Figure 4-4 displays the relationship between mean daily and instantaneous peak flows.

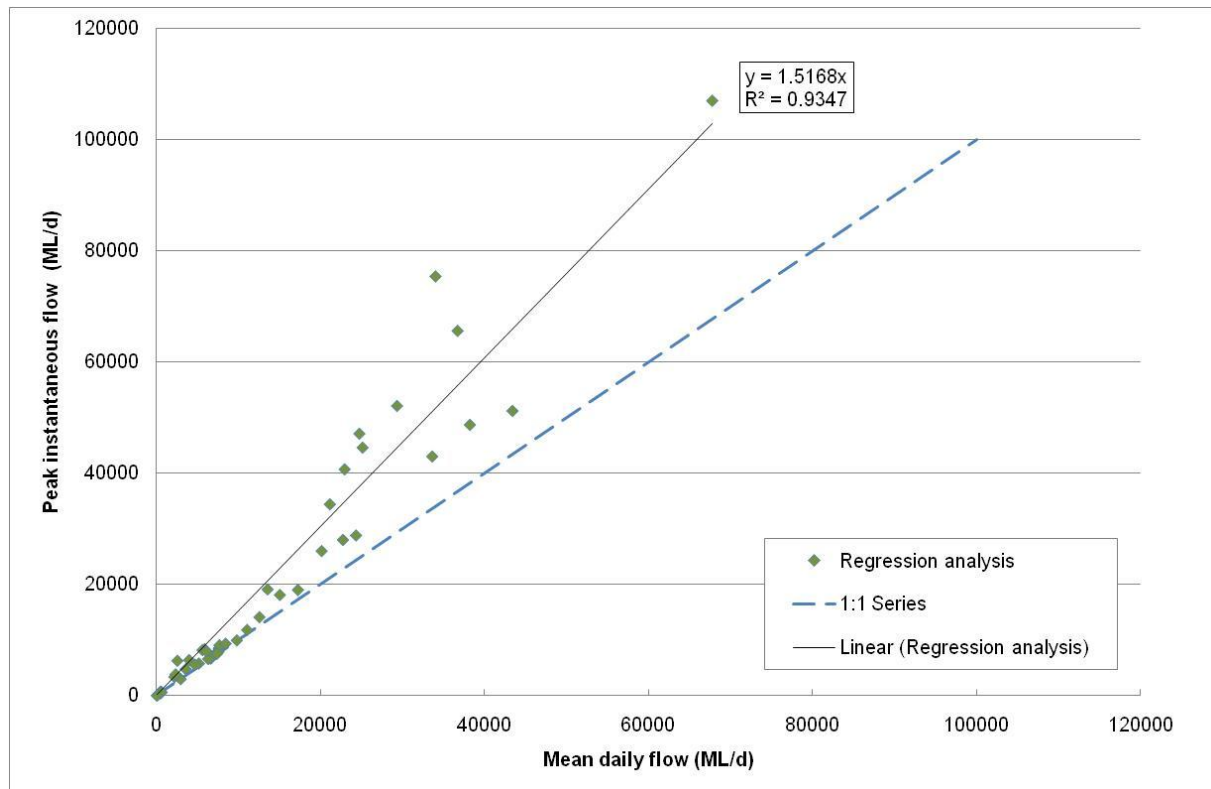


Figure 4-4 Mean daily flows – instantaneous peak flows relationship

4.4.2 Peak Flow Frequency Analysis

The revised peak flow frequency analysis used the extended peak flow series. Three probability distributions were trialled: Log Pearson 3 (LP3); Generalised Extreme Value (GEV); and Generalised Pareto (GP). The three probability distributions were fitted by a Bayesian inference framework. As part of the assessment a variety of low flow thresholds was assessed, in line with Kuzcrea & Franks (2006). The goodness of fit was assessed visually by a comparison against observed flood data.

A log Pearson 3 distribution, with no low flow threshold, yielded the best fit to the observed flood data. Figure 4-5 displays the peak flow flood frequency curves using a LP3 distribution with the observed flood data.

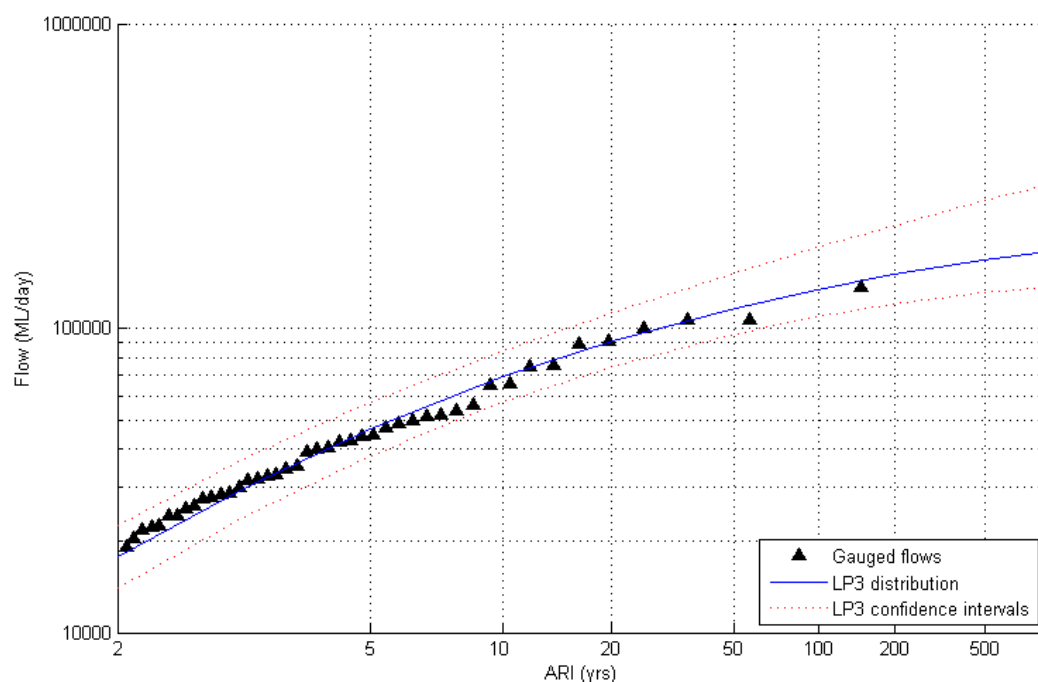


Figure 4-5 Peak flow flood frequency analysis – Log Pearson 3 (no low flow excluded) - Downstream of Lake Glenmaggie

Table 4-3 displays the peak flow estimates from the present study and previous estimates from WBM (2008).

Table 4-3 Peak flow flood frequency analysis – peak flow design estimates

ARI (years)	Design peak flow estimates Macalister River downstream of Lake Glenmaggie) (ML/d)	
	Present study	WBM (2008)
2	17,680	12,960
5	46,720	39,744
10	69,190	63,072
20	90,680	88,128
50	116,600	120,960
100	134,100	146,016
200	149,700	Not available

A comparison of the flood frequency design peak flow estimates shows that for the 2 to 20 year ARIs, this study's estimates are higher than WBM (2008). The difference varies from 26 % for the 2 year ARI to 3 % for the 20 year ARI.

For the 50 and 100 year ARI, the present study's peak flow estimates were lower than WBM (2008). The difference varies from 4 % for the 50 year ARI to 9 % for the 100 year ARI.

The June 2007 event had an estimated peak flow of 147,640 ML/d (WBM 2008). The present study's peak flow frequency analysis assessed the indicative ARI of the June 2007 event at ~ 190-200 years.

This indicative ARI of the June 2007 event compared to about 100 years from WBM (2008) and SMEC (2007).

Further discussion of the peak flow frequency results are provided in Section 4.9.

4.4.3 Flood Volume Frequency Analysis

The flood volume flow frequency analysis used an annual maximum 4 day volume series. Three probability distributions were trialled: Log Pearson 3 (LP3); Generalised Extreme Value (GEV); and Generalised Pareto (GP). The three probability distributions were fitted by a Bayesian inference framework. A variety of low flow thresholds were assessed. The goodness of fit was assessed visually by a comparison against observed flood data.

As with the peak flow frequency analysis, a Log Pearson 3 distribution, with no low flow threshold, yielded the best fit to the observed flood data. Figure 4-6 displays the flood volume frequency curves using a LP3 distribution with the observed flood data.

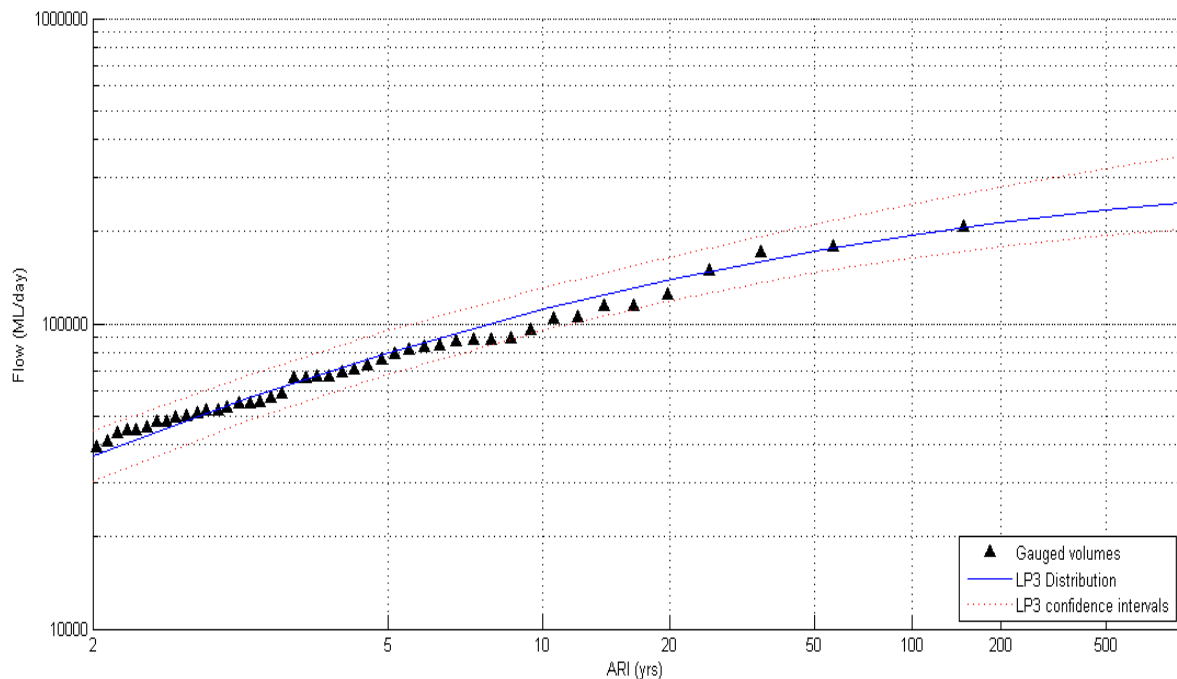


Figure 4-6 Flood volume frequency analysis – Log Pearson 3 (no low flow excluded) - Downstream of Lake Glenmaggie

Table 4-4 displays the 4-day flood volumes estimates from this study and WBM (2008). The design volumes were derived for the WBM (2008) from the design flood hydrographs for the 20, 50 and 100 year event (Storm duration 36 hour) provided in the study. No specific flood volume frequency analysis was undertaken by WBM (2008).

Table 4-4 4-day flood volume design estimates – Present study and WBM (2008)

ARI (years)	Design 4-day volumes estimates Macalister River downstream of Lake Glenmaggie) (ML/d)	
	This study	WBM (2008) - Derived estimate
2	36,772	Not available
5	80,755	Not available
10	111,477	Not available
20	139,551	144,832
50	172,430	190,477
100	194,185	236,044
200	213,430	Not available

A comparison of the flood frequency design flood volume estimates shows that the present study's estimates are lower than the WBM (2008) estimates for 20, 50 and 100 year ARIs.

Further discussion of the peak flow frequency results is provided in Section 4.9

4.5 URBS Runoff Routing Modelling – Parameter Reconciliation

As discussed, BoM has previously developed and calibrated an URBS runoff routing model for the Macalister River Catchment. This BoM URBS model was modified and employed in the Stage 1 study (WBM, 2008).

The present study adopted the previous URBS routing parameters estimates ($\alpha = 0.2$, $\beta = 2.5$ & $m = 0.8$). However, it was determined that the rainfall loss parameters (Initial loss & continuing loss) should be reconciled with the 20, 50 and 100 year peak flows estimates from the URBS model using the peak flow frequency analysis. Table 4-5 shows the revised design rainfall losses parameters adopted in the revised URBS model. The rainfall loss parameters described in WBM (2008) were determined through calibration of the 100 year peak flow only, which gave values of 40mm and 3.5 mm/hr for the initial and continuing losses respectively.

Table 4-5 Adopted design rainfall losses

ARI (years)	Rainfall loss		URBS peak flow (ML/d)
	Initial loss (mm)	Continuing loss (mm/h)	
20	40	3.3	90,800
50	40	3.7	115,900
100	45	4.2	133,400

4.6 Design Flood Hydrograph Estimation

4.6.1 Frequent to Large Flood Events

The revised URBS model was applied to estimate design flood hydrographs for 5, 10, 20 50, 100 and 200 year ARI events using the parameters detailed in Section 4.5. The critical storm duration for all ARIs was 36 hours, as per WBM (2008). The rainfall loss parameters for the 20 year ARI event (as

shown in Table 4-5) were applied to the 5 and 10 year events, and the design rainfall losses for the 100 year event were applied to the 200 year ARI event. As discussed in Section 4.3.3, the spillway at Lake Glenmaggie was assumed to be ungated (i.e. free overflow) for design flood estimation. This assumption is in line with advice received from SRW

Table 4-6 displays the design peak flow estimates, and Figure 4-7 displays the design flood hydrographs.

Table 4-6 Design peak flow downstream of Lake Glenmaggie – Present study and WBM (2008)

ARI (years)	Design peak flow estimates Macalister River downstream of Lake Glenmaggie) (ML/d)	
	Present study	WBM (2008)
5	52,900	Not available
10	64,800	Not available
20	90,800	88,100
50	115,900	118,800
100	133,400	147,700
200	151,900	Not available

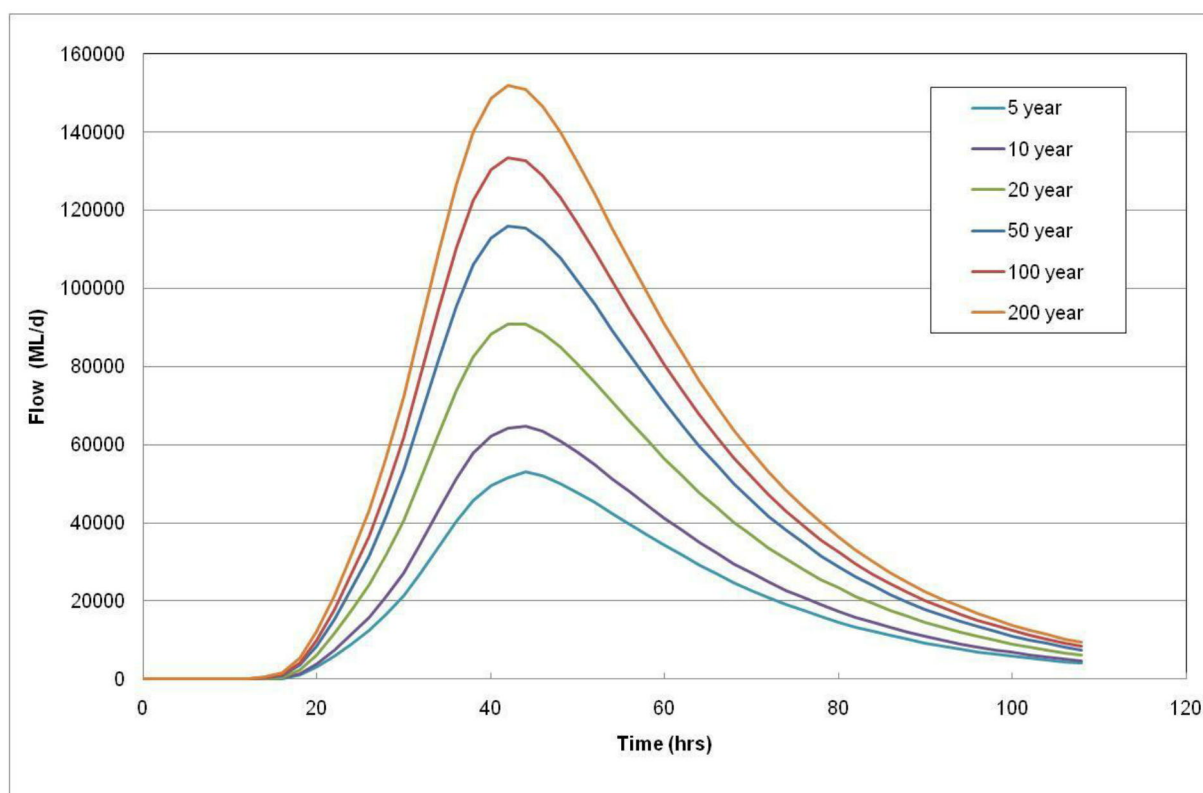


Figure 4-7 Design flood hydrographs – Downstream of Lake Glenmaggie

4.6.2 Probable Maximum Precipitation Design Flood

An indicative estimation of the probable maximum precipitation design flood (PMPDF) or PMF was required for this study. A rigorous assessment of the PMPDF was considered to be outside the scope

of this study. For this study's purposes, the indicative PMPDF estimate provides the general magnitudes of likely increases in flood depths and extent downstream of Lake Glenmaggie.

SMEC (2007) assessed large to extreme inflow events to Lake Glenmaggie. The estimates of the peak inflow to Lake Glenmaggie for the PMPDF ranged from 748,000 ML/d to 1,700,000 ML/d. This range reflected a range of routing parameters employed in the runoff routing model (RORB).

The present study's URBS model was applied to estimate the PMFDF with an initial loss of 0 mm and a continuing loss of 1 mm/h. The resultant indicative peak inflow for the PMPDF event was 1,243,000 ML/d. This peak inflow lay within the range of peak flows provided by SMEC (2007). The indicative URBS model PMPDF peak outflow from Lake Glenmaggie was 1,242,000 ML/d. For the PMPDF event, the comparison of the peak inflow and peak outflow shows that Lake Glenmaggie has a minimal attenuation effect.

Given the present study's requirement was for an indicative estimate of the PMFDF, this estimate is considered fit for purpose. Figure 4-8 shows the PMPDF outflow hydrograph.

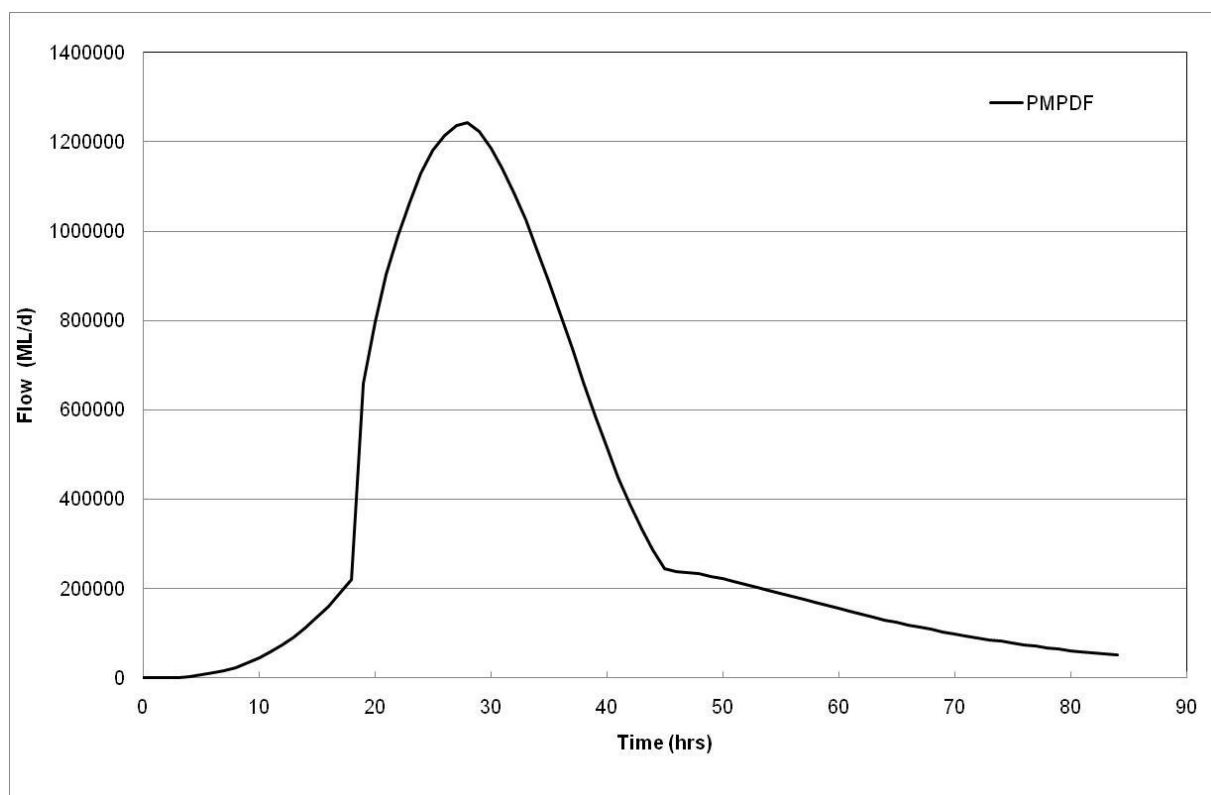


Figure 4-8 Design flood hydrograph for PMFDF event – Downstream of Lake Glenmaggie

4.7 Climate Change Sensitivity

To assess the sensitivity of the design flood estimates to potential changes associated with climate change, design rainfall intensities were scaled up by 10 % and 20%. These up-scalings were applied to the 20, 50 and 100 year ARI events. The design temporal patterns from Australian Rainfall and Runoff were retained.

The URBS models were re-run for the above up-scaled design rainfall, with all other model parameters (routing parameters and rainfall losses) unchanged. Table 4-7 displays the design peak flows for the base case and two climate change scenarios, and percentage increase is provided in brackets. Figure 4-9, Figure 4-10 and Figure 4-11 display the base case and climate change design flood hydrographs for the 20, 50 and 100 year events.

Further discussion of the climate change scenarios results are provided in Section 4.9.

Table 4-7 Design peak flow downstream of Lake Glenmaggie– Base case and climate change scenarios

ARI (years)	Design peak flow estimates Macalister River downstream of Lake Glenmaggie) (ML/d)		
	Base case	Design rainfall +10%	Design rainfall +20%
20	90,800	108,000 (18%)	126,000 (38 %)
50	115,900	136,000 (18%)	158,000 (36%)
100	133,400	156,000 (17%)	181,000 (36%)

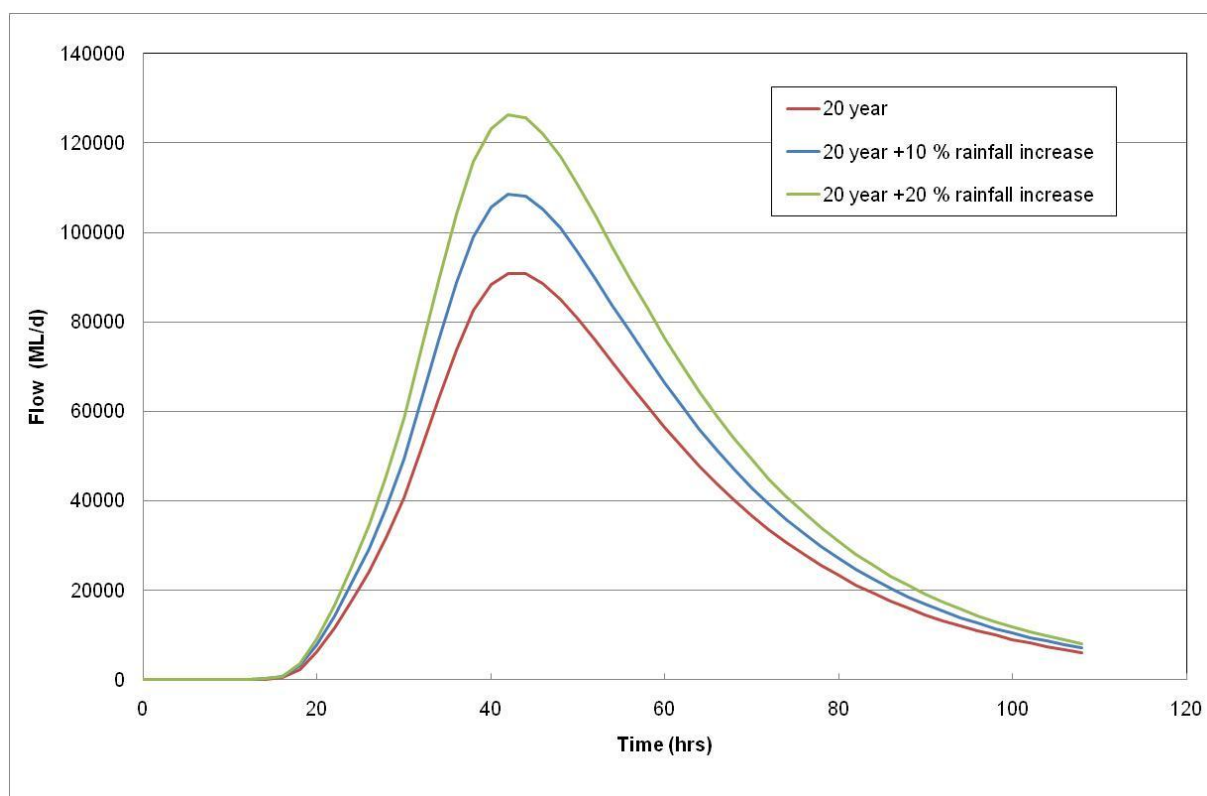


Figure 4-9 Design flood hydrographs – 20 year base case and climate change scenarios

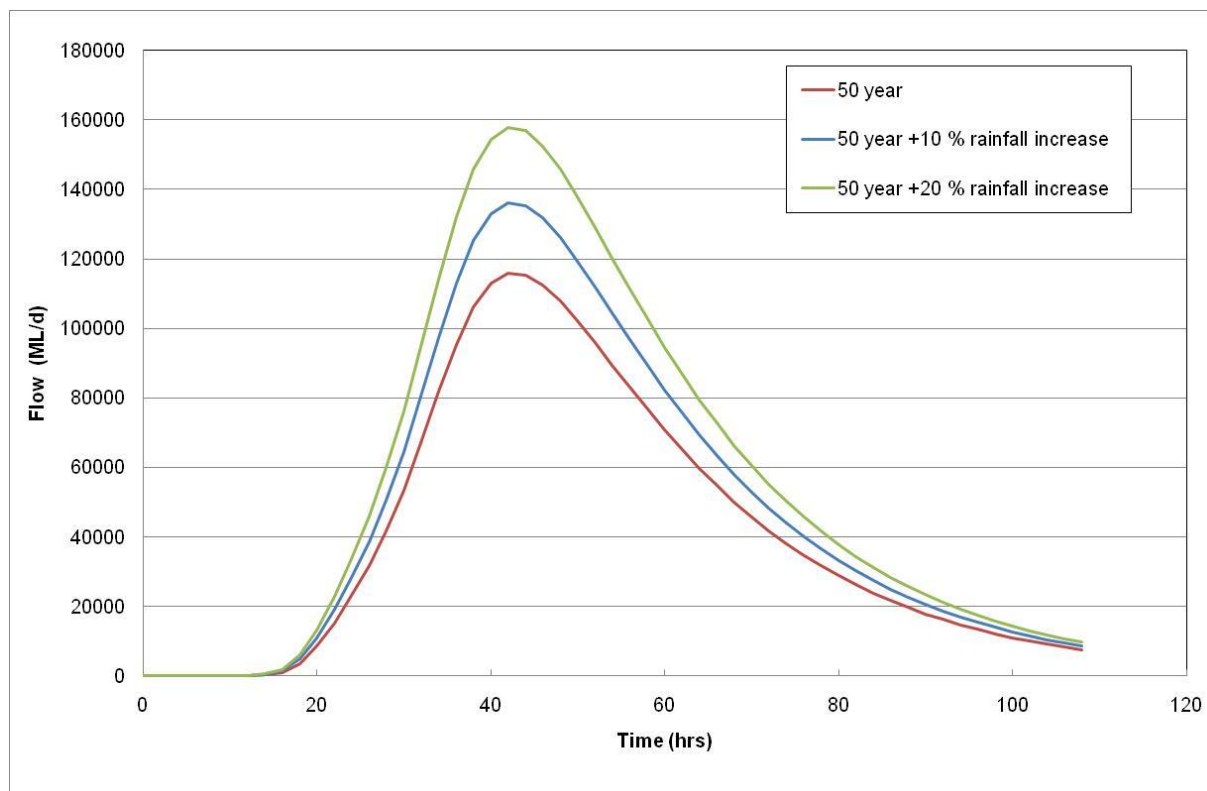


Figure 4-10 Design flood hydrographs – 50 year base case and climate change scenarios

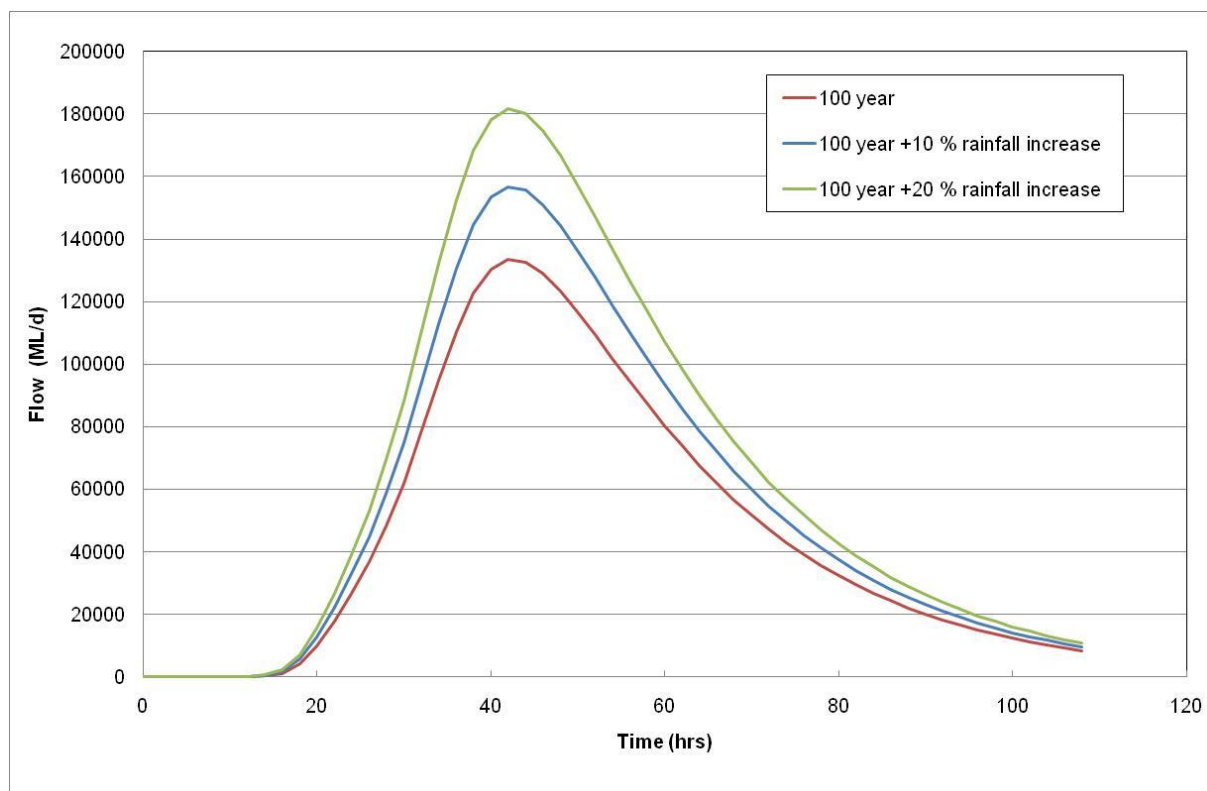


Figure 4-11 Design flood hydrographs – 100 year base case and climate change scenarios

4.8 Concurrent Thomson River Flow Assessment

The flood behaviour in the lower reaches of the Macalister River is potentially influenced by the concurrent flow in the Thomson River. An assessment of concurrent flows in the Macalister and Thomson Rivers was undertaken to assess the concurrency of peak flows.

The approach employed available streamflow from the Macalister River downstream of Lake Glenmaggie (Station 225204), and the Thomson River at Wandocka (Station 225212). The monthly maximum instantaneous peak flows for the two gauges were plotted for the period of concurrent data (January 1969 to June 2009). Figure 4-12 displays the scatter plot of the concurrent peak flow data. Also shown in Figure 4-12, are the design peak flow estimates for the Macalister River downstream of Lake Glenmaggie (as outlined in Table 4-3).

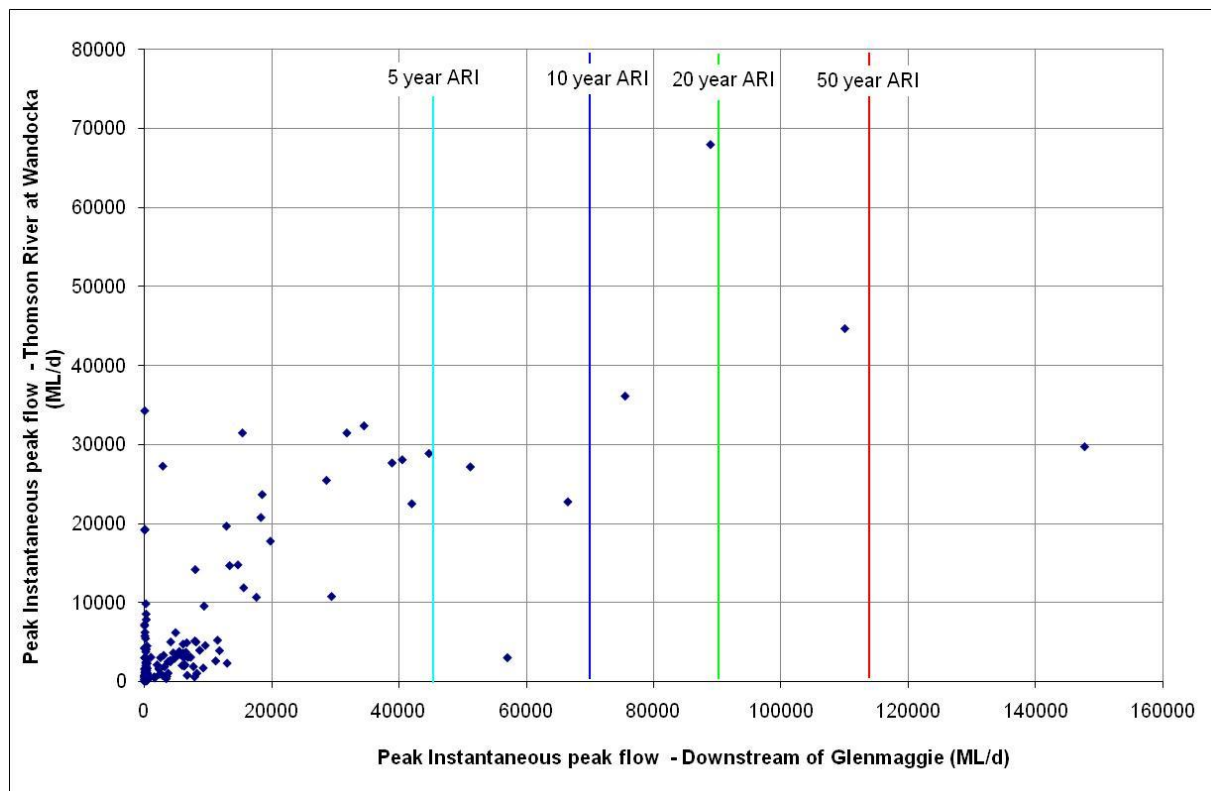


Figure 4-12 Concurrent flow assessment – Macalister River DS Lake Glenmaggie and Thomson River at Wandocka

Figure 4-12 shows a reasonable correlation between the Macalister and Thomson River flow up to the 5 year ARI (46,720 ML/d). Above the 5 year event, there are a limited number of observed flow events. Furthermore, there appears to be little correlation between the Macalister River and Thomson River peak flows.

The flow data at both gauges is influenced by the presence of upstream storages (Lake Glenmaggie and Thomson Dam).

For the purposes of establishing appropriate downstream boundary conditions in the hydraulic modelling (i.e. concurrent flow in the Thomson River), recommended peak flow combinations are presented in Table 4-8 based on the above assessment.

Table 4-8 Design peak flow combination - Downstream of Lake Glenmaggie and Wandocka

ARI (years)	Design peak flow estimates in the Macalister River downstream of Lake Glenmaggie) and Thomson river at Wandocka (ML/d)	
	Macalister River downstream Lake Glenmaggie	Thomson River at Wandocka
5	52,900	30,000
10	64,800	40,000
20	90,800	45,000
50	115,900	50,000
100	133,400	50,000
200	151,900	50,000

4.9 Hydrologic Analysis Discussion

This hydrologic analysis has prepared revised design flood hydrographs downstream of Lake Glenmaggie. It has employed flood frequency analysis and runoff routing modelling.

The flood frequency analysis utilised a best practice technique, as outlined by Kuczera and Franks (2006). The application of this technique underpins the changes in design peak flow estimates when compared to WBM (2008) and SMEC (2007).

4.9.1 Flood Frequency Analysis

The June 2007 event had an estimated peak flow of 147,640 ML/d (WBM, 2008). This study's peak flow frequency analysis assessed the indicative ARI of this event at ~ 190-200 years, while WBM (2008) and SMEC (2007) give an estimated ARI of around 100 years.

SMEC (2007) assessed the June 2007 rainfall event to have an approximate ARI of 1 in 50 year, and estimated the ARI of the peak inflow to Lake Glenmaggie as between 1 in 100 year and 1 in 300 years.

The key reasons for the differences are:

- Use of instantaneous peak daily flows for the entire flow record in the present study rather than mean daily flows which likely underestimates the actual peak flow value on any given day. It is unclear whether the mean daily flow data used in WBM (2008) was adjusted to reflect the estimated instantaneous peak values. SMEC (2007) used mean daily flow data in their analysis.
- For floods towards the extreme end of the flood frequency distribution the best fit curve is relatively "flat" with only a small range of flows between the 100 and 200 year ARI flow estimates. The present study is considered to show a good fit between the observed data and the flood frequency curve (Figure 4-5). Observed peak flows were not plotted on the flood frequency curve in WBM (2008) and therefore the goodness of fit cannot be assessed.
- Flood volume frequency analysis has been included in the present study. WBM (2008) and SMEC (2007) considered peak flows without consideration of flood volumes. Given the extensive nature of the floodplain, flood volume in addition to peak flow will influence flood behaviour.

4.9.2 Design Event Analysis

A comparison of the flood frequency design peak flow estimates showed that for the 2 to 20 year ARIs, this study's estimates were higher than WBM (2008). The difference varies from 26% for the 2 year ARI to 3% for the 20 year ARI. For the 50 and 100 year ARI, this study's peak flow estimates were lower than WBM (2008). The difference varies from 4% for the 50 year ARI to 9% for the 100 year ARI.

As no recalibration of the URBS model was undertaken for the present study, the key difference between WBM (2008) and the present design event analysis is the consideration of rainfall losses.

Losses in WBM (2008) were reconciled for the 100 year peak flow estimates using the peak flow frequency analysis, whereas the present study reconciled the 20, 50 and 100 year peak flows estimates. For the present study, losses are higher for 100 year event than in WBM (2008), but lower for the 20 year ARI event. This is reflected in the design peak flow estimates.

4.9.3 Climate Change

Climate change impacts on flood behaviour were assessed as a sensitivity analysis of design rainfall intensities. Increases in design rainfall intensities by 10 % and 20% were assessed for the 20, 50 and 100 year ARI events. For a 10 % rainfall intensity increase, the peak flow estimates increased by ~ 17-18% , and for the 20 % design rainfall increase peak flows increased by 36-38%. This equates to the 20 year ARI peak flow increases to a ~ 40 year ARI with 10% rainfall increase, and to ~ 80 year ARI with a 20% increase.

It should be noted as part of this sensitivity analysis, the design rainfall losses remained unchanged. It is reasonable to expect that the rainfall losses may increase with climate change (general drier climate). Any increase in design rainfall losses would offset (in part) the increases in rainfall intensities.

5. HYDRAULIC MODEL CONSTRUCTION AND CALIBRATION

5.1 Overview

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

The study brief required the determination of design flood levels and extents for the 20, 50 and 100 year event plus the Probable Maximum Flood (PMF) event. Further, the study brief required three other events for flood response purposes. As proposed in the tender response, the three other flood events adopted were the 5, 10 and 200 year events.

The principal tool for the determination of the design flood levels and extents was a robust and calibrated hydraulic modelling framework. The complexity of the flood behaviour required a flexible hydraulic modelling framework.

The construction of the hydraulic modelling framework is discussed in Section 5.2. A comprehensive hydraulic modelling framework has been employed in this study. However, the outcomes of the hydraulic modelling must be viewed in the light of the hydraulic models' capabilities, limitations and uncertainties. These aspects are also discussed in Section 5.2.

Calibration of the hydraulic modelling framework to observed flood information underpins a robust and reliable hydraulic model. Details of the hydraulic model calibration are provided in Section 5.3.

The base topographic data for the hydraulic analysis was the available ALS data. Water Technology (2009) noted a systematic bias in the supplied ALS topographic data when compared to available field surveyed data sources. At the time, based on advice from DSE, West Gippsland CMA advised to adopt the supplied ALS data for use in this study. Subsequent further review of the ALS data resulted in a revision of the ALS data set. The revision was in line with the systematic bias found in Water Technology (2009). The model calibration was undertaken using both ALS data sets. Comparison of the model calibration outcomes from the two ALS data sets provided some general insight into the relative importance of the topographic data accuracy in hydraulic modelling. Details of the model calibrations are provided in Section 5.3.

5.2 Hydraulic Model Construction

5.2.1 Hydraulic Modelling Framework

The hydraulic modelling framework was required to simulate the flow behaviour for large flood events (5 year to 200 year ARI plus PMF) across the Macalister River floodplain. A key aspect of the hydraulic modelling framework was to balance excessive model run times and the topographic resolution.

The hydraulic modelling framework comprised the following component models:

- One dimensional (1D) hydraulic models: Key waterways and anabranches for in-channel flows
- Two – dimensional (2D) hydraulic models: Broad scale floodplain features for large flood events
- Linked one – two dimensional hydraulic (1D/2D) model: Combines the 1D model with the broad 2D floodplain models to assess just overbank events.

As discussed in the hydraulic model calibration (Section 5.3), a linked 1D-2D was initially trialled. However, unsatisfactory calibration outcomes lead to the adoption of a standalone 2D model.

The hydraulic modelling suite, MIKE11, MIKE21 and MIKE FLOOD, developed by the Danish Hydraulic Institute (DHI) has been applied in this study. MIKE FLOOD is a state-of-the-art tool for floodplain modelling that combines the dynamic coupling of the one-dimensional MIKE 11 river model and

MIKE 21 fully two-dimensional model systems. Further details on the capabilities of the MIKE FLOOD modelling system can be found at <http://www.dhisoftware.com/mikeflood>.

5.2.2 One Dimensional (1D) Hydraulic Model Elements

Initially a linked 1D/2D model was created to model the Macalister River and surrounding floodplain. The 1D model components represent the Macalister River channel for flows up to bankfull. This allows accurate representation of the flow capacity of the river channel.

The 1D model has the key following elements.

- Branches:
 - Macalister River
- Cross Sections:
 - Surveyed Macalister River cross sections: Eight surveyed cross sections were available along the Macalister River. These surveyed cross sections were included in the 1D model. Details of the surveyed cross sections were provided in Topographic Data Review (Water Technology 2009).
 - ALS extracted cross sections: Nominal 300-400 m spacing. As discussed in Topographic Data Review (Water Technology 2009), the ALS extracted cross sections do not include the cross section below the water level at the time of the ALS data capture. The ALS extracted cross sections were revised to account for the “missing” area using the available surveyed cross sections. From the surveyed cross section inverts, a uniform bed profile was assumed between the surveyed cross sections. The ALS extracted cross sections were revised to reflect the assumed invert as shown in Figure 5-1.
 - Total surveyed and ALS extracted cross-sections – 205
- Boundaries:
 - Macalister River downstream of Lake Glenmaggie:
 - Observed streamflows: Macalister River downstream of Lake Glenmaggie (Gauging station 225204)
 - Downstream of Thomson River confluence:
 - Observed water levels: Observed peak flood levels from adjacent locations
- Roughness
 - Floodplain and river channel hydraulic roughness within the 1D model was expressed as Manning’s n. This study assessed Manning’s n via comparison of modelled and observed water levels. Further discussion of Manning’s n is provided in 5.3.

Figure 5-2 displays the 1D model elements.

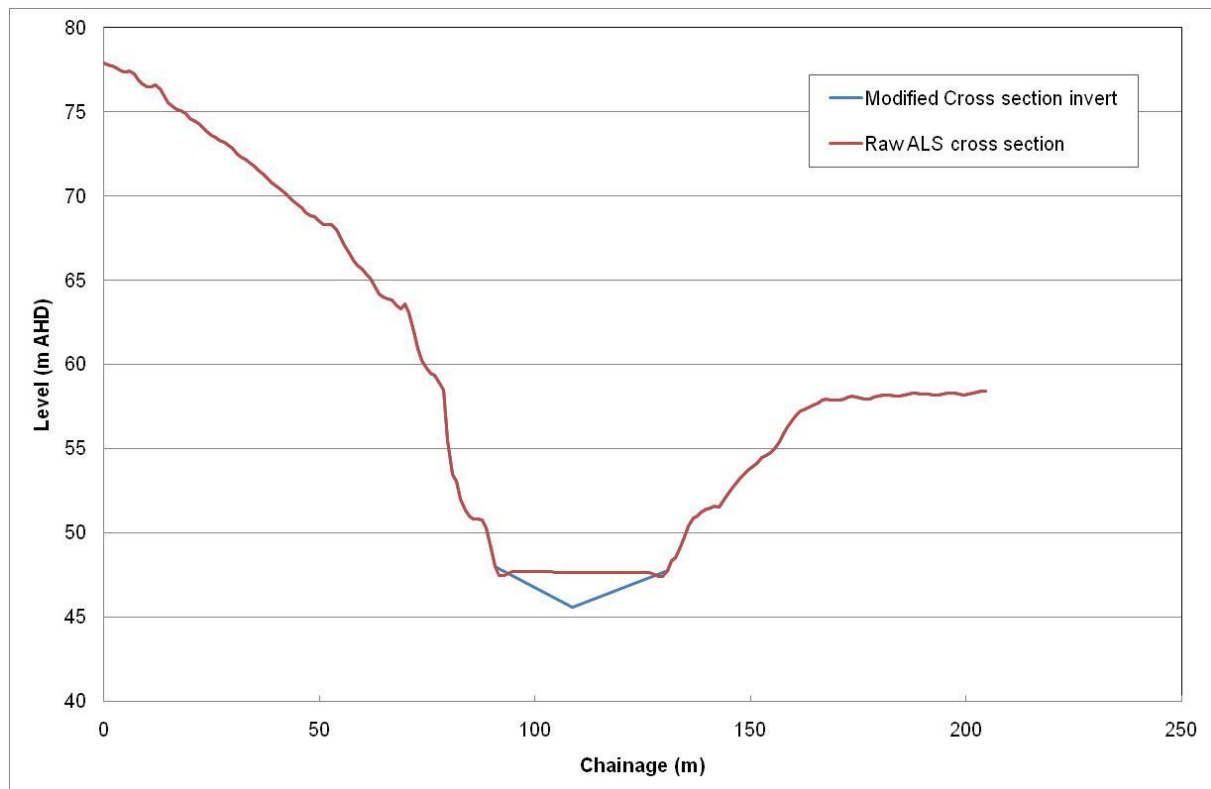


Figure 5-1 Macalister River Cross Section – ALS extracted and revised cross sections

5.2.3 Two Dimensional (2D) Hydraulic Model Elements

Two modelling approaches were applied, a linked 1D-2D model and standalone 2D model. The linked model represented the Macalister River channel as a 1D channel, refer to Section 5.2.2. The use of 1D channel enabled a coarser grid resolution for the 2D model component in the linked model than in the standalone 2D model, as the Macalister River channel geometry was represented by cross sections in the 1D model, rather than in a grid. Two alternative 2D grid resolutions were adopted, 25 m in the linked model, and 12.5 m in the standalone 2D model.

The 2D models consisted of the following elements:

- Grid extent ,resolution and features
 - Linked model

The study area was converted from the raw ALS points into a 25 m grid resolution
 - 2D standalone model

The study area was converted from the raw ALS points into a 12.5 m grid resolution
 - Topographic features

The study areas is crossed by numerous irrigation channel, road and rail embankments. These features control flood behaviour across the floodplain. Accurate representation of these features in the model topography was essential to the construction of a robust hydraulic model. The key topographic features, such as road and channel embankments, were stamped into the model grids.
- Boundaries:

- Macalister River downstream of Lake Glenmaggie:
Observed streamflows: Macalister River downstream of Lake Glenmaggie (Gauging station 225204)
- Downstream of Thomson River confluence:
Observed water levels: Observed peak flood levels from adjacent locations
- Roughness
 - Hydraulic roughness within the 2D model was expressed as Manning's n. For the estimation of the floodplain Manning's n, this study assessed land use and vegetation cover. The evaluation of Manning's n was undertaken through the calibration of the modelled and observed flood levels, and extents. The range of Manning's n values employed is discussed in Section 5.3.

Figure 5-2 displays the 2D model elements.

5.2.4 Linked One-Two dimensional (1D/2D) Hydraulic Model Elements

The linked 1D-2D model employed the 1D and 2D model elements discussed above. The linking between the 1D and 2D models occurred at 2D grid cells bordering the Macalister River channel. The linking was dynamic in nature, and allowed the transfer of mass and momentum in and out of the channel. The use of the linking slowed the model run time in comparison to a standalone 2D model. The use of a larger grid resolution in the linked model offset the increased model run times.

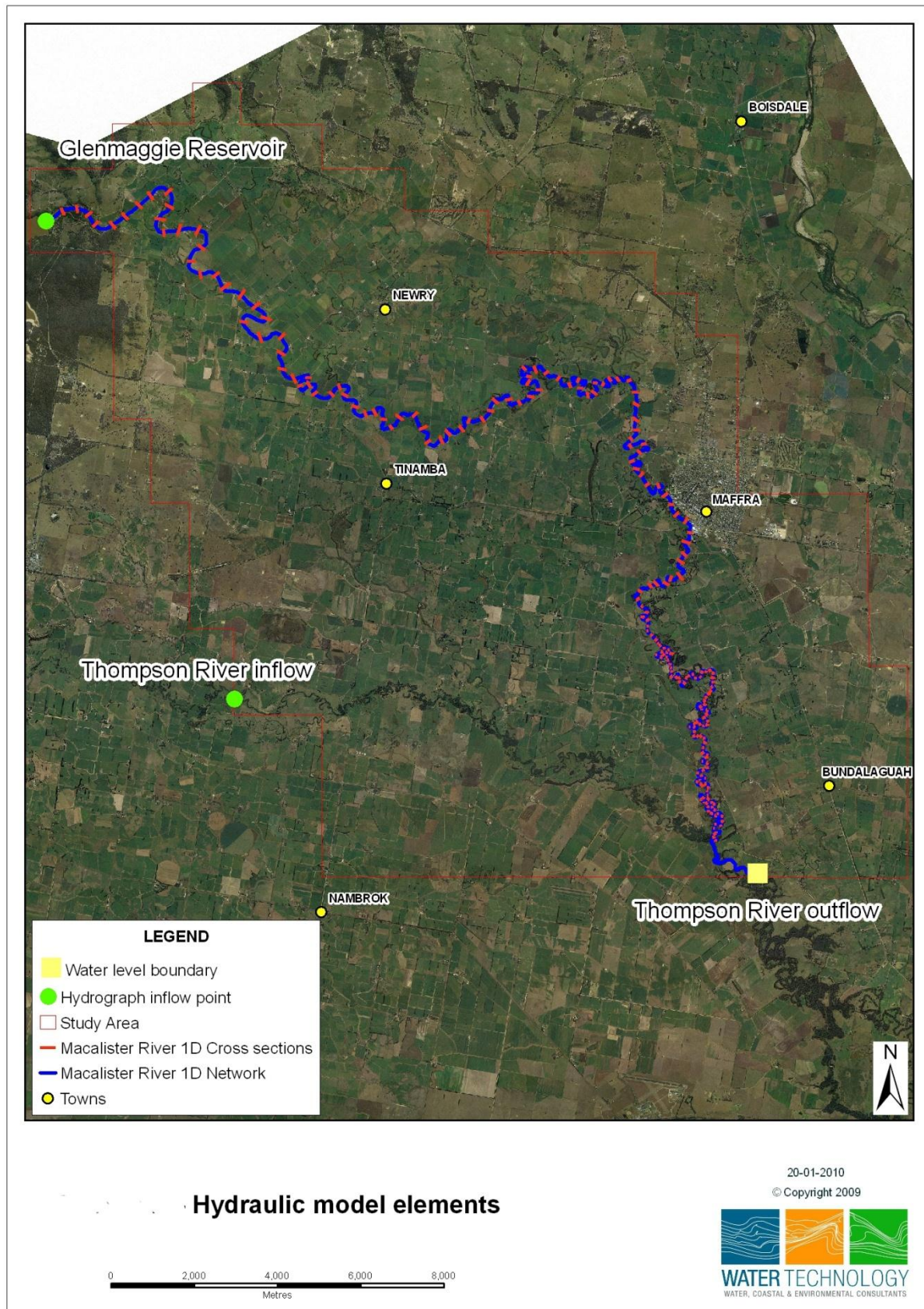


Figure 5-2 Hydraulic model elements

5.2.5 Hydraulic Model Capabilities and Uncertainties

There are numerous contributing factors to uncertainty associated with the modelled flood level in a complex hydraulic modelling exercise such as that undertaken for this study. Some of the uncertainties relate to the data inputs, whilst others are dependent on the numerical modelling process itself. Sources of output uncertainty related to the input data for the hydraulic modelling include:

- ALS data
- Bathymetry and cross section survey
- Definition of hydraulic controls/structures
- Observed flows for model input
- Observed flows and water levels for model calibration

Sources of uncertainty related to the hydraulic modelling process include:

- Model numerical and computational schemes – these relate to the ability of the model to replicate the physics of free-surface flow in channels and over land.
- Floating point accuracy of computing resources (truncation error)
- Model schematisation and set-up (location and spacing of cross-sections, grid resolution)
- Model parameters such as computational time-steps, surface-friction and other energy-loss parameters (expansion/contraction coefficients and eddy viscosity for example).

There is a wide variation in the magnitude of the impact associated with each source of uncertainty. In order to identify the most significant sources of uncertainty it is possible to consider items as either first or second order magnitude, where second order items are of a significantly smaller magnitude compared to first order items and can generally be ignored. A listing of the main sources of the modelling uncertainty and their approximate magnitudes is provided in Table 5-1.

Due to the complexity of the relationships between the input data and modelling outputs, there is no direct correlation between input and output data accuracy. Further, the error bounds on the data inputs are generally not cumulative. For example, inaccuracies in survey data inputs may be compensated for through adjustment of calibration parameters to achieve output hydraulic results that are nominally more accurate than the sum of the errors in the input data. Hence there are inferred relationships between model inputs and output accuracy that are typically developed through hydraulic modelling project experience.

The model development process can only address uncertainties arising from the following aspects:

- Definition of hydraulic controls/structures
- Model schematisation and set-up (location and spacing of cross-sections, grid resolution)
- Model parameters such as computational time-steps, surface-friction and other energy-loss parameters

The remaining aspects from Table 5-1 are constrained by the available data sources.

As discussed, the model calibration was undertaken using two ALS data sets. Comparison of the model calibration outcomes from the two ALS data sets provided some general insight into the relative importance of the topographic data accuracy in hydraulic modelling. Details of the model calibrations are provided in Section 5.3.

Table 5-1 Comparisons of Sources of Uncertainty

Scenario/Data/Process	Order of Accuracy	Approximate Impact on Results
ALS data and DEM	First	Change in floodplain levels/depths 0.1 m
Cross-section survey	First	Minimal direct impact, location and spacing of sections is more critical to model outputs
Definition of hydraulic controls/structures	First	Change in floodplain levels/depths 0.1 to 0.2 m
Observed flows for model input	First	Depends on available data, aim for observed/calibration accuracy +/- 10 % for flows
Observed flows and water levels for model calibration	First	Depends on available data, +/- 10 % for flows & +/- 0.15 m for observed flood levels.
Model numerical and computational schemes – these relate to the ability of the model to replicate the physics of free-surface flow in channels, wetlands and over land.	Second	N/A
Floating point accuracy of computing resources (truncation error)	Second	N/A
Model schematisation and set-up (location and spacing of cross-sections, grid resolution)	First	Difficult to quantify, aim for overall accuracy of +/- 0.1 m for levels and +/- 10 % for flows
Model parameters such as computational time-steps, surface-friction and other energy-loss parameters	First	Change in floodplain levels/depths +/- 0.1 m
Level/accuracy of model calibration	First	Depends on availability of calibration data, aim for +/- 0.1 m for levels and +/- 10 % for flows

5.3 Hydraulic Model Calibration and Validation

5.3.1 Approach

This section discusses the refinement of a hydraulic models' parameters through calibration against observed flood levels, water level and streamflow data, and outlines the validation of adopted model parameters.

The calibration process consisted of systematic comparison of observed and modelled flow/flood. This process incorporated comparisons between observed flood levels and flood extents. The models' parameters were adjusted to minimise the differences between the modelled and observed data. The validation process applied the calibrated model parameters to another flood event to assess model performance.

A robust calibration required the comparison of modelled and observed flood behaviour across a range of flow magnitudes. The following observed data was required for a historical event to be considered in the model calibration:

- Well defined inflows and outflows (boundary conditions).
- Flow and level measurements over time (temporal distribution) at discrete points of interest within and along the river such as effluent points and control structures.
- Flood extent and/or depth measurements (spatial distribution) at multiple times.

The historical flow/flood events used to calibrate/validate the models were chosen on the basis of available observed flood information. Section 5.3.2 details the selection and available data for the calibration/validation events.

As discussed, model calibration was undertaken for the three hydraulic model configurations:

- Linked one – two dimensional hydraulic (1D/2D) model using the original supplied (interim) ALS topographic data. Refer to Section 5.3.3.
- Two – dimensional (2D) hydraulic model using the original supplied (interim) ALS topographic data. Refer to Section 5.3.4
- Two – dimensional (2D) hydraulic model using the revised (final) ALS topographic data. Refer to Section 5.3.5.

The model validation was only undertaken for the 2D hydraulic model using the revised (final) ALS topographic data.

The calibration of the models was centred on the determination of floodplain and river channel hydraulic roughness values (Manning's n values) to achieve a reasonable agreement between observed and modelled flood levels. Initial Manning's n values were assigned to various land uses based on previous modelling experience, and where necessary to achieve a reasonable agreement, these initial Manning's n values were refined.

Furthermore, the key topographic features (such as irrigation channels) in the model grid were refined to improve the comparison of observed and modelled flood levels.

Some difficulty was encountered calibrating the models at locations where overtopping of irrigation channels/road embankments occurred. At these locations, irrigation channels/road embankments effectively act as weirs. A general pattern of under-estimating of observed flood levels upstream of these features was found. This pattern suggested that the flow over the feature was too high. To resolve this pattern, the roughnesses along the crest of these features were altered.

5.3.2 Available Observed Flood Data and Calibration/Validation Event Selection

The focus of the model calibration and validation was the general flood behaviour during large flood events, in line with the study's interests (5 year to 200 year events). Hence, the selection of calibration and validation events reflected large flood events with adequate available observed flood data suitable for this purpose. The June 2007 and June 1978 events were selected, given the relatively large flood magnitude and reasonable abundance of observed flood information.

Table 5-2 displays the peak flows and associated approximate ARI for the model calibration /validation events, and general nature of available observed flood information.

Table 5-2 Hydraulic model calibration/validation events – Peak flows, approximate ARI, and available observed flood information

Event	Peak flow (ML/d) and approximate ARI		Available flood information	Use
	Peak Flow (ML/d)	Approximate ARI (years)		
June 2007	147,640	~ 190-200	Observed flood levels: 239 Observed flood aerial photos and flood extent	Calibration
June 1978	88,900	~20	Observed flood levels: 72	Validation

5.3.3 Linked 1D-2D Model Calibration (Interim ALS topography)

The linked 1D-2D model calibration considered the June 2007 event. The performance of the model calibration was assessed by the comparison of modelled and observed flood levels. The differences were calculated by subtracting the modelled flood levels from the observed flood levels. Hence, a negative difference indicated that the modelled flood level was lower than the observed flood level i.e. under-estimation. Following each calibration run, revisions/refinements to the roughness and model topography were undertaken, with the aim of improving the model calibration. Table 5-3 displays the number and percentage of flood level differences within each category.

Table 5-3 Linked 1D-2D model calibration (June 2007) – flood level differences

Calibration run	Number of flood levels comparison					Comments
	Below -0.2 m	-0.2m to -0.1 m	-0.1 m to +0.1 m	+0.1m to +0.2 m	Above 0.2 m	
01 & 02	Initial model runs to resolve any instability					
03	162 (65%)	9 (4%)	37 (15%)	9 (4%)	32 (12%)	Channel roughness n 0.04. Generally too low along Newry Creek system
04	151 (60%)	15 (6%)	37 (15%)	16 (6%)	30 (12%)	Channel n 0.05. Marginal improvement. Increased marginally flow across overall floodplain, and along Newry Creek
05	153 (60%)	14 (6%)	32 (13%)	12 (5%)	28(11%)	Included several major channels adjacent to Newry. Marginal improvement.
06	147 (59%)	16 (6%)	35 (14%)	11 (4%)	30 (12%)	Include several major channels downstream of Newry and at Tinamba. Marginal improvement.
07	93 (36%)	21 (8%)	72 (28%)	15 (6%)	48 (19%)	Channel roughness n 0.06. Considerable improvement in matching observed flood levels. Observed flood extent exceeded immediately downstream of Glenmaggie on left bank.
08	144 (56%)	19 (7%)	44 (17%)	11 (4%)	31 (12%)	Channel roughness 0.05 roughness. Revised downstream conditions adjacent to Bundalagwah to reflect observed flood level pattern. Lower general flood levels. Underestimating flows to Newry Creek
09	143 (55%)	18 (7%)	46 (18%)	11 (4%)	31 (12%)	Channel roughness 0.05 roughness. Included secondary channels /embankments.
10	144 (56%)	18 (7%)	45 (17%)	11 (4%)	31 (12%)	Channel roughness 0.05 roughness. Included further secondary channels /embankments.

The best performed linked model calibration run was Run 07. Figure 5-3 displays the flood level differences. The colours of flood level differences dots reflect the differences in the modelled and observed flood levels. The dark and light green dots indicates observed flood levels were under-

estimated, the orange dots indicate the modelled and observed flood levels are within 0.1 m and the red and purple dots indicates observed flood levels were over-estimated.

Across the linked 1D-2D calibration runs, there was a general trend for the modelled flood levels to be lower than the observed flood levels. This was evidenced by the high proportion of flood level differences falling in the “below -0.2 m” category. This trend, in part, was influenced by the systematic bias in the originally supplied (interim) ALS data set. However, it was considered the linked 1D -2D model was unlikely to perform significantly better with the revised (final) ALS data set. On this basis, a standalone 2D model was explored.

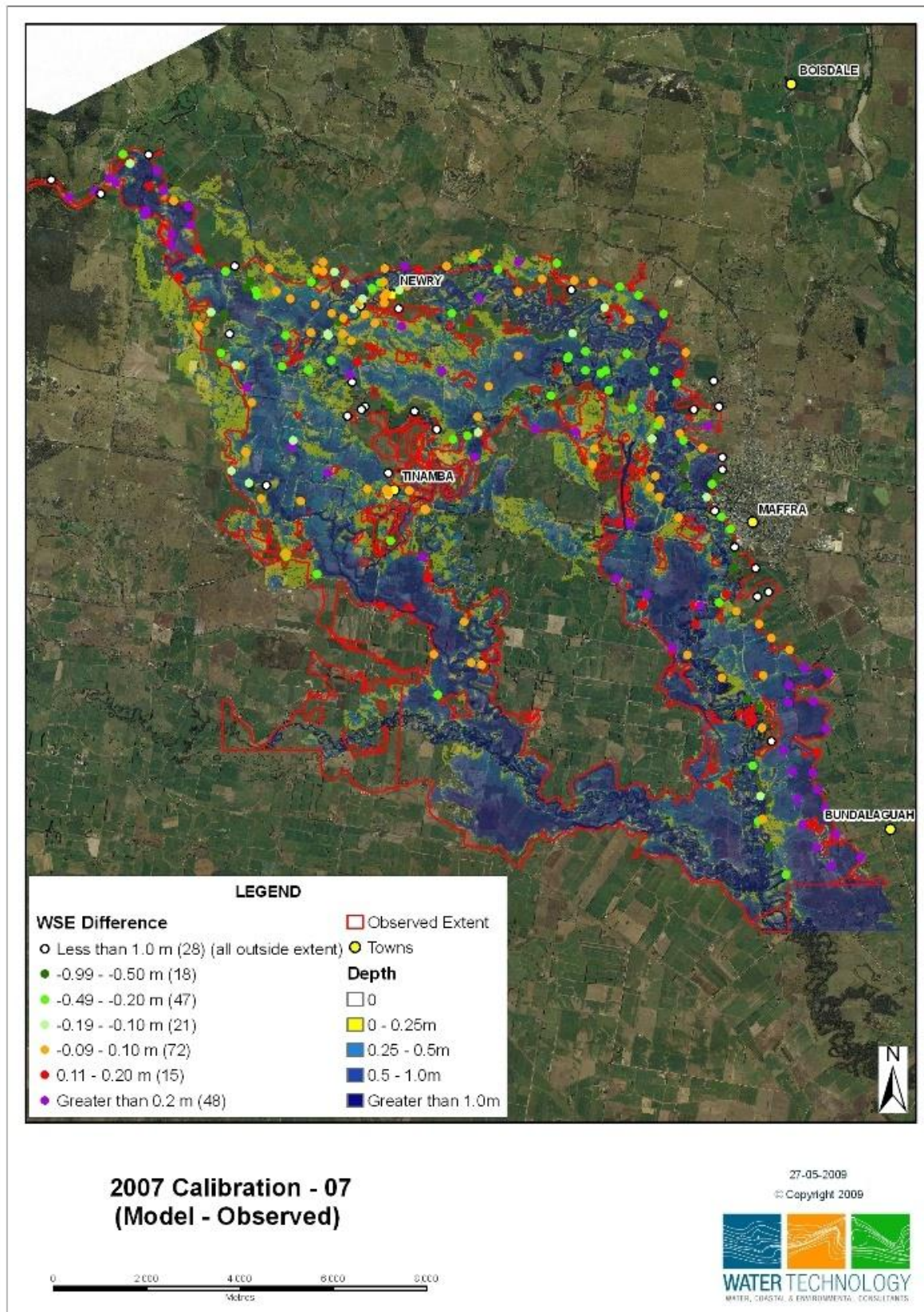


Figure 5-3 Linked 1D-2D model calibration – June 2007 – flood level differences (Run 07)

5.3.4 2D model Calibration (Interim ALS topography)

A standalone 2D model was explored to improve upon the calibration results of the linked model. To more accurately reflect the Macalister River channel geometry and capacity (without incorporating 1D river channel elements), the grid resolution was reduced to 12.5 m in the standalone 2D model. The interim ALS data was used as the base topographic data and calibration was again based on June 2007 event. A number of calibration runs were undertaken prior to the revision of the ALS data.

Similar to the linked model, the performance of the standalone 2D model calibration was assessed by the comparison of modelled and observed flood levels. Following each calibration run, revisions/refinements to the roughness and model topography were undertaken, with the aim of improving the model calibration. Table 5-4 displays the number and percentage of flood level differences within each category for each calibration run.

Table 5-4 Standalone 2D model calibration (interim ALS), June 2007 – flood level differences

Calibration run	Number of flood levels comparison					Comments
	Below - 0.2 m	-0.2m to -0.1 m	-0.1 m to + 0.1 m	+0.1m to +0.2 m	Above 0.2 m	
11 (2D model only)	48 (20%)	15 (6%)	51 (21%)	37 (15%)	93 (38%)	Revised topography to include several key embankments
12 (2D model only)	31 (12%)	17 (7%)	59 (24%)	36 (14%)	106 (43%)	Revised topography to include several key embankments
13 (2D model only)	37 (15%)	21 (8%)	66 (27%)	34 (14%)	91 (37%)	Revised topography and roughness
14 (2D model only)	Not applicable due to model instability					
15 (2D model only)						
16 (2D model only)	35 (14%)	15 (6%)	63 (25%)	39 (16%)	97 (39%)	Revised topography and roughness
17 (2D model only)	35 (14%)	20 (8%)	66 (27%)	34 (14%)	94 (38%)	Revised topography and roughness
18 (2D model only)	32 (13%)	22 (9%)	73 (29%)	32 (13%)	90 (36%)	Revised topography and roughness
19 (2D model only)	35 (14%)	20 (8%)	73 (29%)	39 (16%)	82 (33%)	Revised topography and roughness
20 (2D model only)	47 (19%)	12 (5%)	77 (31%)	37 (15%)	76 (31%)	Revised topography and roughness

The best performed standalone 2D (interim ALS) model calibration run was Run 20. Figure 5-4 displays the flood level differences. Across the 2D (interim ALS) calibration runs, there was a general trend for the modelled flood levels to be higher than the observed flood levels. This was evidenced by the high proportion of flood level differences falling in the “above +0.2 m” grouping (red, purple & yellow points). This outcome contrasts with the linked model runs.

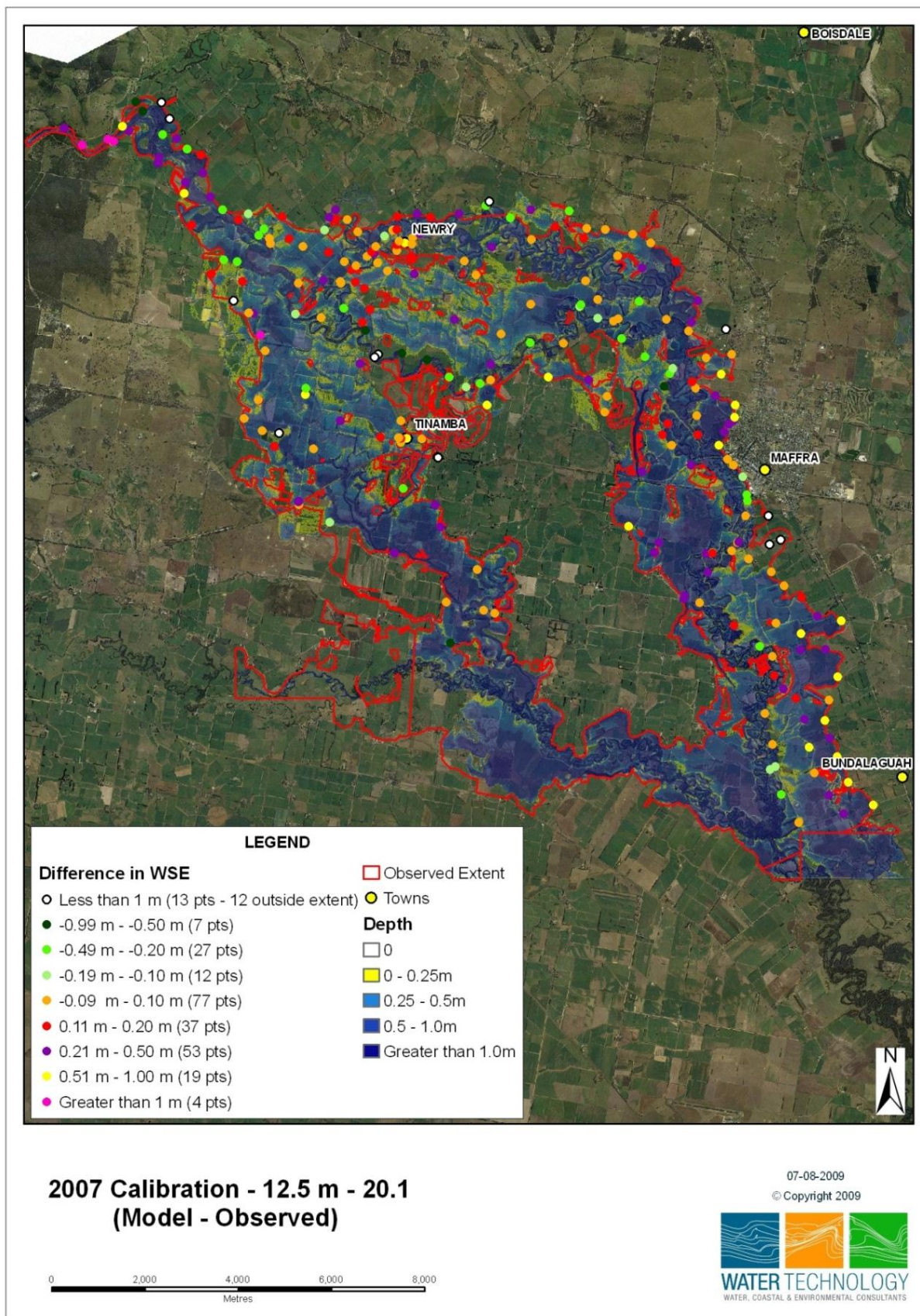


Figure 5-4 Standalone 2D model calibration (Interim ALS data), June 2007 – flood level differences (Run 20)

5.3.5 2D model calibration (Revised ALS topography)

Following review by DSE, a revised (final) ALS data was supplied. As the ALS data is the principal input to the hydraulic model topography, the supply of the revised ALS data necessitated the re-construction of the standalone 2D model. In line with the standalone 2D model using the interim ALS, a 12.5 m grid was adopted.

The standalone 2D model calibration and validation, using the final ALS data, again considered the June 2007 and June 1978 events.

June 2007 Event – Overall Performance

Following each calibration run, revisions/refinements to the roughness and model topography were undertaken, with the aim of improving the model calibration. Table 5-5 displays the number and percentage of flood level differences within each category for each model run.

Table 5-5 Standalone 2D model calibration (final ALS) – June 2007 – flood level differences

Calibration run (Final ALS)	Number of flood levels comparison					Comments
	Below - 0.2 m	-0.2m to - 0.1 m	-0.1 m to + 0.1 m	+0.1m to +0.2 m	Above 0.2 m	
01	99 (40%)	38 (15%)	65 (26%)	19 (8%)	28 (11%)	Roughness as per Interim ALS Run 20
02	89 (36%)	34 (14%)	73 (29%)	14 (6%)	39 (16%)	Revised roughness
03-06	Not applicable due to model instability					
07	61 (24%)	45 (18%)	77 (31%)	21 (8%)	45 (18%)	Revised topography and revised along embankments where outflow occurs
08	61 (24%)	50 (20%)	75 (30%)	21 (8%)	42 (17%)	Revised topography and revised along embankments where outflow occurs
09-12	Not applicable due to model instability					
13	65 (26%)	49 (20%)	77 (31%)	18 (7%)	40 (16%)	Revised roughness along the Macalister River and Newry Creek floodplain
14	52 (21%)	30 (12%)	86 (35%)	31 (12%)	50 (20%)	Run 13 with the inflow hydrograph increased by 10%
15-16	Not applicable due to model instability					
17	64 (26%)	31 (12%)	91 (37%)	24 (10%)	39 (16%)	Revised topography to better define embankment crests

A comparison of the Run 01 (final ALS) and Run 20 (interim ALS, refer to Table 5-4) reflects the influence of the revised ALS data. The general trend was a lowering of the modelled flood levels, with an increase proportion of modelled flood levels lower than the observed flood levels.

Discussion of Results

Revisions and refinements to the topography and roughness were undertaken in Runs 02-08. As discussed, the flow behaviour across the Macalister River floodplain is primarily influenced by irrigation channels/other embankments. The correct representation of these features in the hydraulic model drives the model performance.

Runs 07 and 08 show a marginal change in the spread of modelled and observed flood level differences. Comparing the number of modelled flood levels within ± 0.2 m, revised run 01 122 points (49%), revised run 07 143 points (57 %) and revised run 08 146 (59%).

However, the Runs 07 and 08 display a bias, with some 44% of modelled flood level lower than observed flood levels. This compared to 25% of modelled flood levels higher than observed flood levels.

Further revisions to the roughness were undertaken in Runs 09 to 14. Due to the malfunction of the streamflow gauge downstream of Lake Glenmaggie, the June 2007 inflow hydrograph to the hydraulic model was estimated by Southern Rural Water (WBM 2008). There is uncertainty surrounding the reliability of the estimated June 2007 hydrograph. To assess the sensitivity of the modelled flood behaviour to the inflow hydrograph, the estimated June 2007 hydrograph was scaled up to 10 % (Run 14).

Runs 13 and 14 has resulted in a marginally change in the spread of modelled and observed flood level differences. Comparing the number of modelled flood levels within ± 0.2 m, revised run 08 146 points (59%), revised run 13 144 points and revised run 14 147 (59%). The increased flow (by 10%) has resulted in a mean increase of modelled flood levels by ~ 0.04 m.

The spread of points above and below ± 0.1 m, shows for Run 13, 46% were less than -0.1 m and 23% were greater than $+0.1$ m, and for run 14 33% were less -0.1 m and 32% were greater than $+0.1$ m. Run 14 shows a more centred distribution of flood levels differences.

Run 13 display differences in observed and modelled flood extents. The principal differences occurred in the following locations:

- North-western edge of Boggy Creek: Modelled extent overestimates. The modelled flood levels appears to be reasonable (a number of orange points)
- Macalister River - North of Tinamba: Modelled extent underestimates. Modelled levels are low up to 0.7 m (green points).
- Macalister River adjacent to Mewburn Park: Modelled extent underestimates. Modelled levels are low up to 0.2 m (light green points). Changes in the roughness in this reach have little impact on modelled flood levels.

The lower modelled flood levels in the last two locations may suggest a higher bed level, and that erosion during the June 2007 subsequently lowered the bed level. WGCMA

advised that there were isolated spots of bank erosion, and some erosion on the floodplain, but no significant erosion of the bed.

Further, WGCMA advised that observed flood levels may have been elevated due to debris banked up on fences during the 2007 flood.

Further revisions to the model topography, particularly to capture irrigation channel crests were taken in Runs 15 to 17.

Run 17 has resulted in a marginal improvement in the spread of modelled and observed flood level differences. Comparing the number of modelled flood levels within +/- 0.2 m, revised run 13 144 points (58%) and revised run 17 146 (59%).

Other key outcomes from Run 17 were as follows:

- North-western edge of Boggy Creek: Improved agreement between modelled and observed extents. Some minor modelled overtopping of the channel banks.
- Macalister River - North of Tinamba: Modelled extent underestimates similar to Run 13. Modelled levels are low up to 0.7 m (green points).
- Macalister River adjacent to Mewburn Park: Modelled extent underestimates similar to Run 13.

The best performed standalone 2D (final ALS) model calibration run was Run 17. Figure 5-5 displays the modelled June 2007 flood extents, and flood level differences (Run 17).

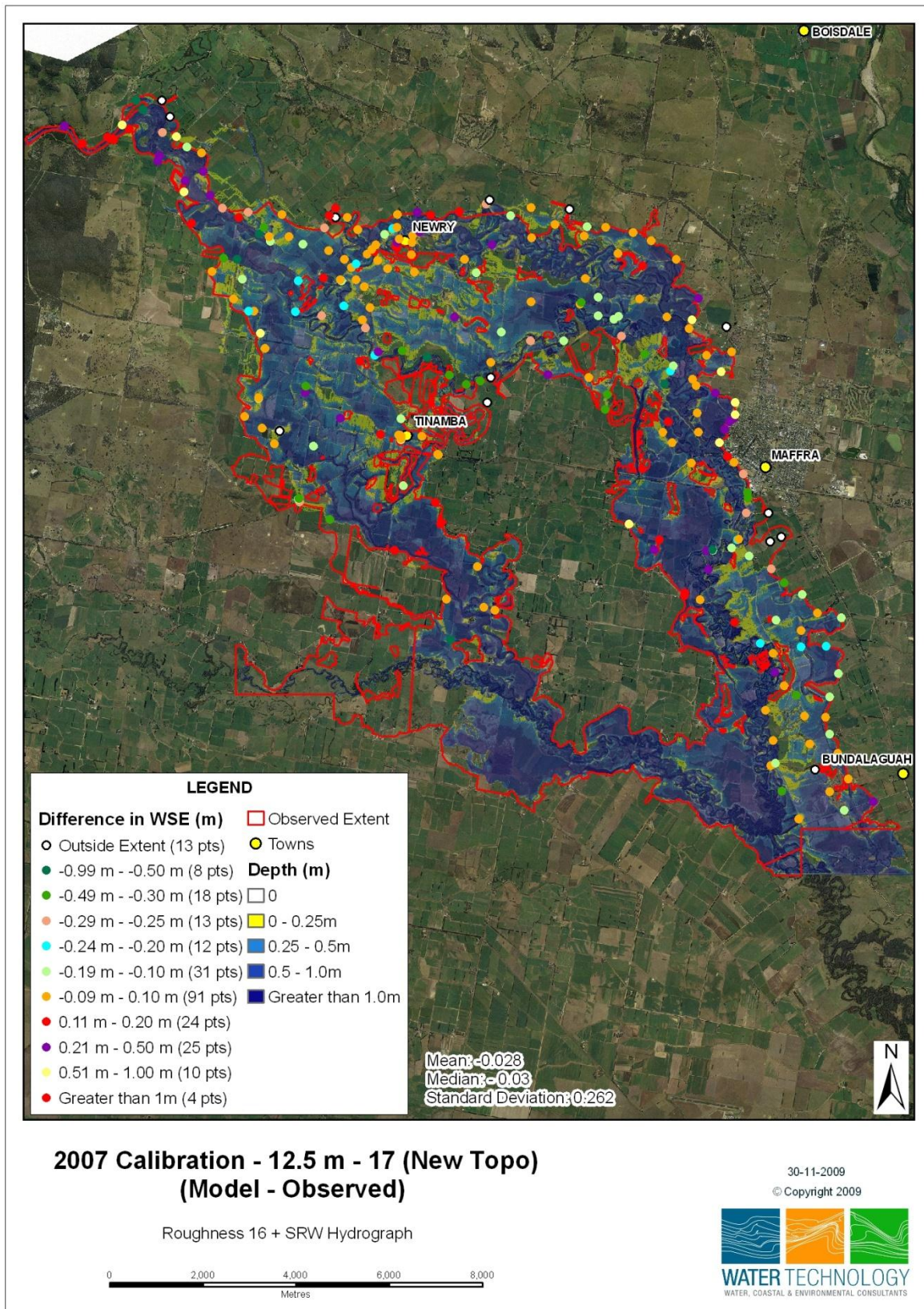


Figure 5-5 June 2007 – model calibration (Revised ALS) – Flood level differences (Run 17)

5.3.6 Discussion of Calibration Results

The following detailed discussion is provided of the 2D standalone model calibration outcomes for the June 2007 event using the revised ALS data.

Macalister River: Lake Glenmaggie to Hagens Bridge

Between Lake Glenmaggie and 300 m upstream of Hagens Bridge, the flow was generally confined to the channel and the immediate surroundings. Calibration results were mixed, with modelled flood levels both considerably lower and higher (± 0.2 m) than observed flood levels. Figure 5-6 displays the calibration results in this reach. Refer to Figure 5-5 for the legend of coloured flood level differences.

Examination of modelled flood levels showed a relatively steep flood slope (~ 1 in 610) and associated higher velocities (~ 2 -3 m/s). Given the flood slope and velocities, small discrepancies in the locations of observed flood level marks can contribute to considerable changes in the modelled flood levels. Hence, flood level differences can be amplified, and the model calibration results affected.

This reach contained 4 of the 8 modelled flood levels (dark green points), which were lower than the observed flood levels by less than -0.5 m. Further, the reach contained 3 of the 10 modelled flood levels (yellow points), which were higher than the observed flood levels by greater than $+0.5$ m.

Comparison of the observed flood levels at these 7 locations revealed some inconsistency with adjacent observed flood levels. This inconsistency combined with the potential errors discussed above, have contributed to the large differences found at these locations.

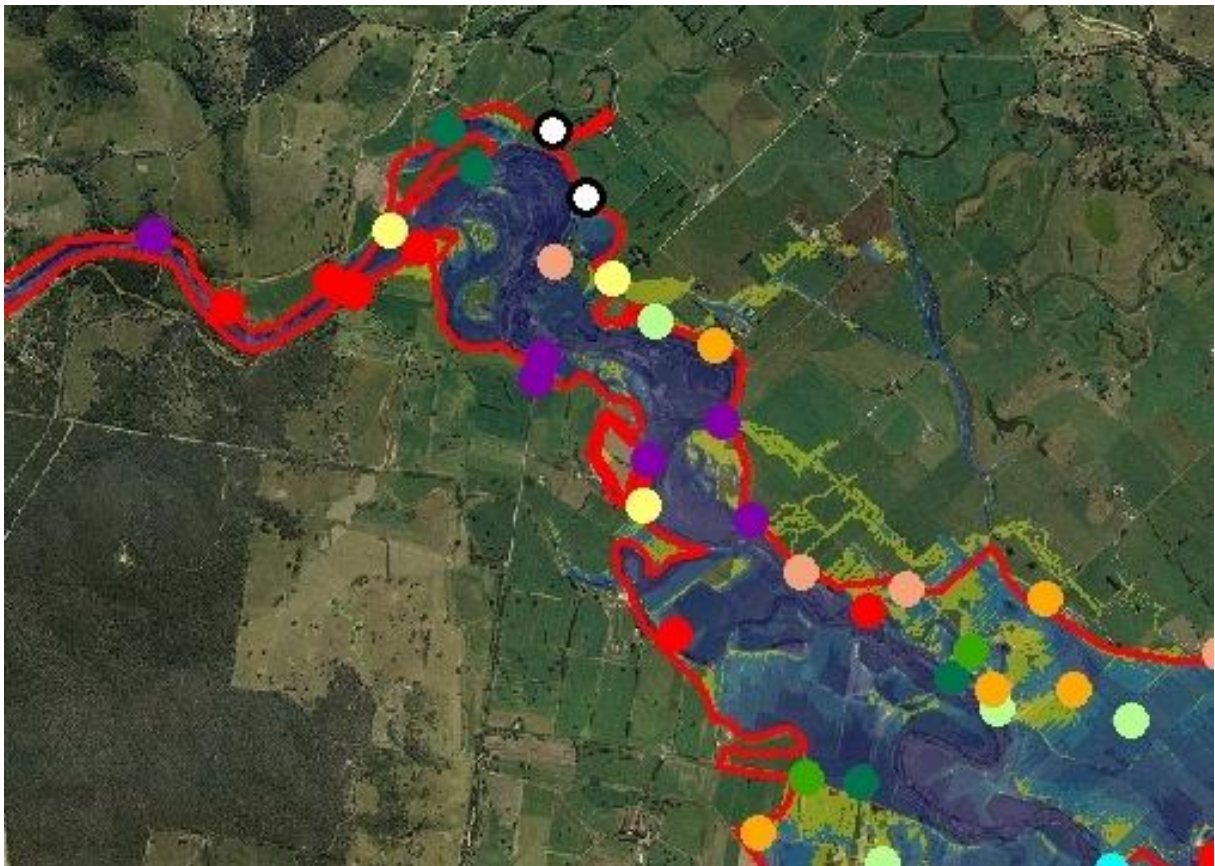


Figure 5-6 **June 2007 – model calibration (final ALS) – Macalister River: Lake Glenmaggie to Hagens Bridge**

Macalister River: Hagens Bridge to Mewburn Park

Through this reach, considerable breakouts from the Macalister River occurred, Newry Creek to the north, and Boggy Creek to the south. This discussion focuses on the calibration results along the river, and further discussion of Newry and Boggy Creeks is provided later.

Figure 5-7 shows the calibration results along this reach. Along the river, the modelled flood levels were generally lower than the observed flood levels (green, blue and pink points). Immediately downstream of the Mansons Bridge, the modelled flood levels were considerably lower than the observed flood levels, with 3 medium green (-0.3 to -0.5 m) points, and 3 dark green points (> -0.5 m). Significant attention has been paid to revising the model topography and roughness in this area. However, the differences could not be resolved. It is unclear what were the principal reasons underpinning these differences.

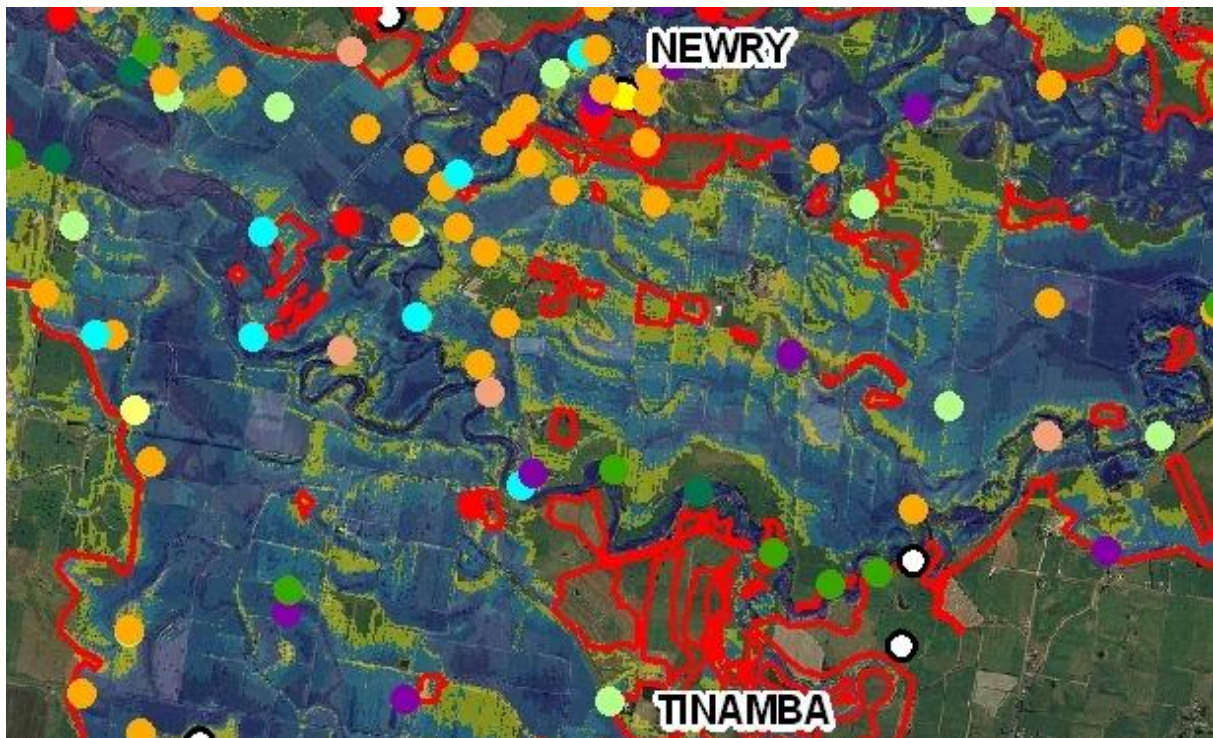


Figure 5-7 **June 2007 – model calibration (final ALS) – Macalister River Hagens Bridge to Mewburn Park**

Macalister River: Mewburn Park to Bellbird Corner

In this reach, overflows to the south occurred across the Tinamba Northern Channel at Ganon's, Gargan's and Corida's. These overflows continued into the Serpentine Lagoon and drainage system. At Bellbird Corner, Newry Creek joins the Macalister River.

Figure 5-8 shows the calibration results along this reach. Generally, the modelled flood levels were lower than the observed flood levels (light green and pink points). These lower modelled flood levels were offset by several modelled flood levels (orange points) within 0.1 m of the observed flood levels. Refinements made in the course of the model calibration emphasised the key control on flood levels in this reach was the crest level of the Tinamba Northern Channel.

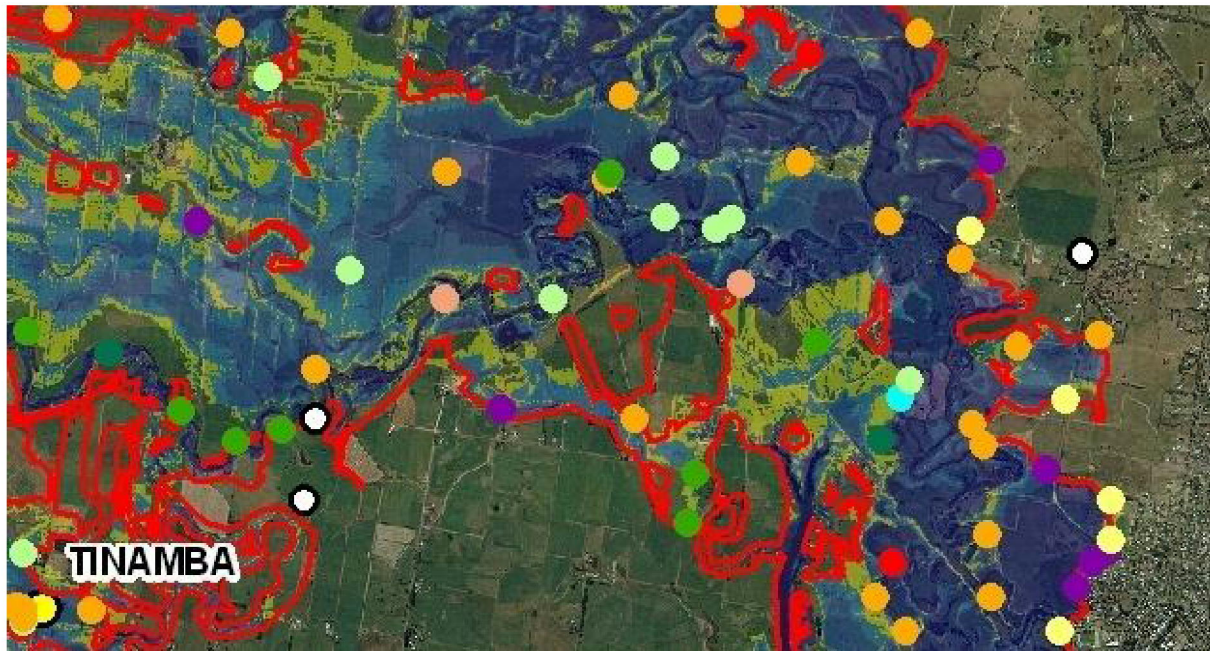


Figure 5-8 **June 2007 – model calibration (final ALS) – Macalister River Mewburn Park to Bellbird Park**

Macalister River: Bellbrid Corner to Maffra Weir

At the upstream limit of this reach, as discussed, breakouts occurred across the Tinamba Northern Channel at Ganon's, Gargan's, Corida's and Waugh's. These breakouts contributed overland flows to the Serpentine Lagoon. Levees along the Macalister River, adjacent to Maffra, controlled breakouts from the Macalister River, and controlled return of the breakout flows back to the river.

Figure 5-9 shows the calibration results along this reach.

There are numerous (20) modelled flood levels within 0.2 m of the observed flood levels (light green, orange and red points). These points were spatially well distributed across the reach, and this indicated a reasonable calibration result for this reach.

Immediately north-west of Maffra, adjacent to The Crescent and the Maffra-Newry Road, there were 6 modelled flood levels, where the modelled flood levels were higher than the observed flood levels by greater than +0.2 m (3 purple and 3 yellow points). Examination of the modelled flood levels indicated that the flood levels in this area were controlled by the levee crest along Laura Street. A relatively constant modelled flood level of ~ 24.2 m AHD occurred across this area. This compared to the observed flood level of ~ 23.95 m AHD. The levee crest levels along Laura Street in the hydraulic model topography were reviewed. However, no further improvements in the modelled flood levels were achieved.

Two of the three yellow points were located to the east of the Newry – Maffra Road (adjacent to the football oval). At these two points, the modelled flood levels were some 0.6 m higher than the observed. It appears the flow was obstructed by the Newry-Maffra Road.

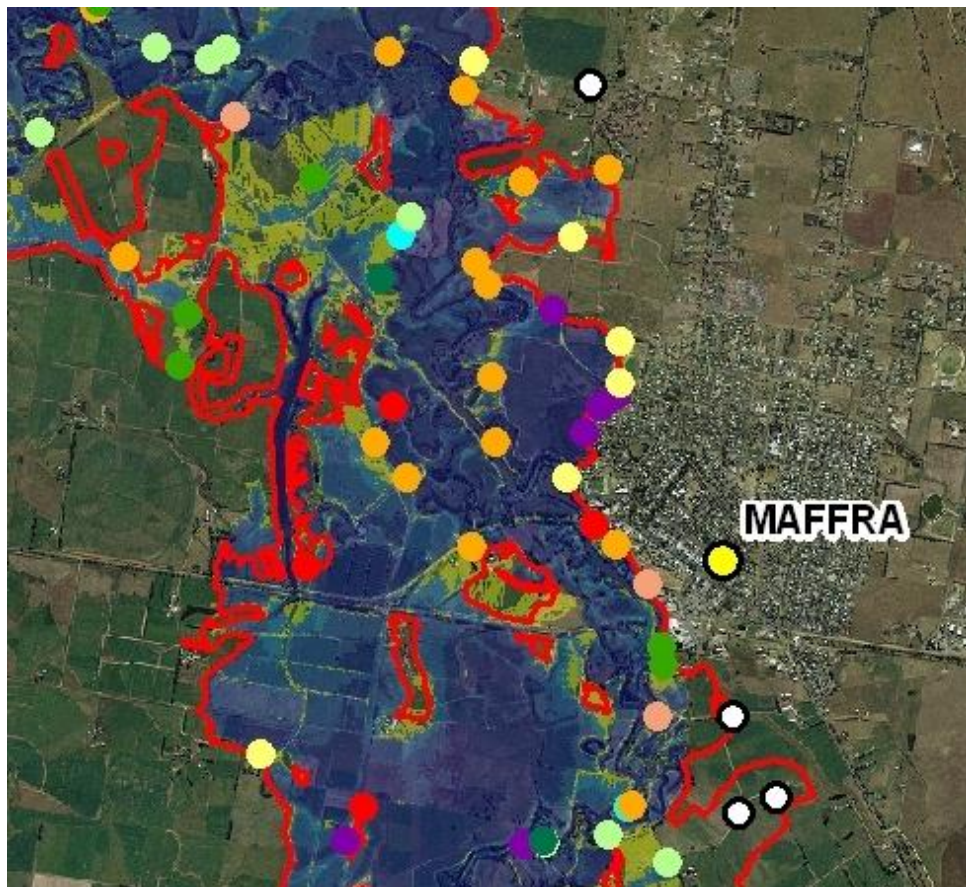


Figure 5-9 **June 2007 – model calibration (final ALS) – Macalister River Bellbird Park to Maffra Weir**

Macalister River: Maffra Weir to Thomson River

Extensive levees along the Macalister River downstream of Maffra Weir controlled breakout flows. Downstream of Maffra Weir, breakouts occurred across the left bank to the south. These breakouts were further controlled by the Bundalaguah Road. These breakouts continued south through the Bundalaguah Drainage system to rejoin the river adjacent to the Thomson River confluence. Along the western floodplain, overland flows from the Serpentine Drainage system return to the Macalister River.

Figure 5-10 shows the calibration results along this reach.

There are numerous (32) modelled flood level within 0.2 m of the observed flood levels (light green, orange and red points). These points are spatially well distributed across the reach. This indicated a reasonable calibration result for this reach. Further, the calibration results provide confidence in the hydraulic model's ability to reflect the observed flood behaviour, arising from the various hydraulic controls discussed above.

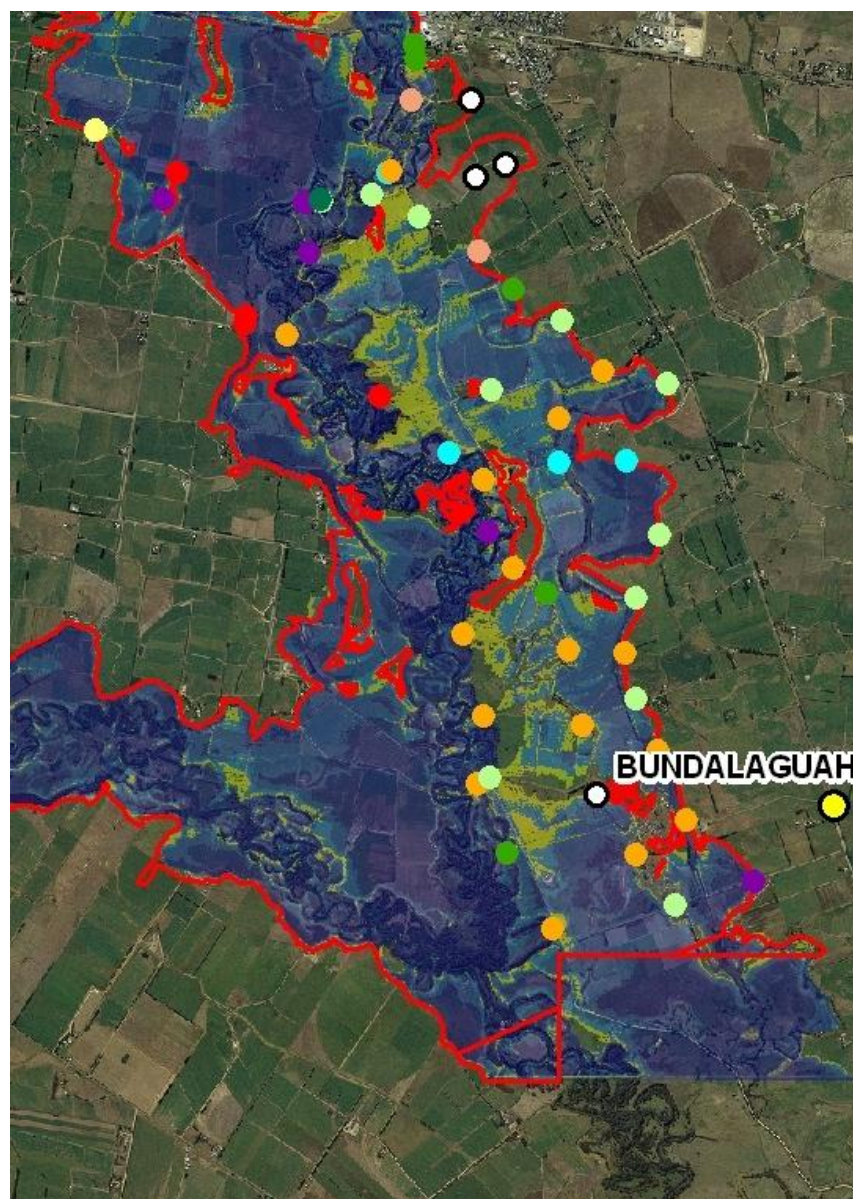


Figure 5-10 **June 2007 – model calibration (final ALS) – Macalister River Maffra Weir to the Thomson river confluence**

Newry Creek Floodplain

Breakouts to Newry Creek occurred along the left bank of the Macalister River immediately upstream of Hagen's Bridge through Mason's Bridge. The Newry River Channel was the key control on breakouts. Once the channel was overtopped, overland flows continued to east within the Newry Creek channel and across the floodplain. The flow across the floodplain was controlled by numerous irrigation channels, banks and drains.

Figure 5-11 shows the calibration results along this reach.

A high proportion of modelled flood levels (light green, orange and red) were within 0.2 m of the observed flood levels. Adjacent to Newry, there was reasonable agreement between modelled and observed flood levels, with a number of modelled levels within 0.1 m (orange points).

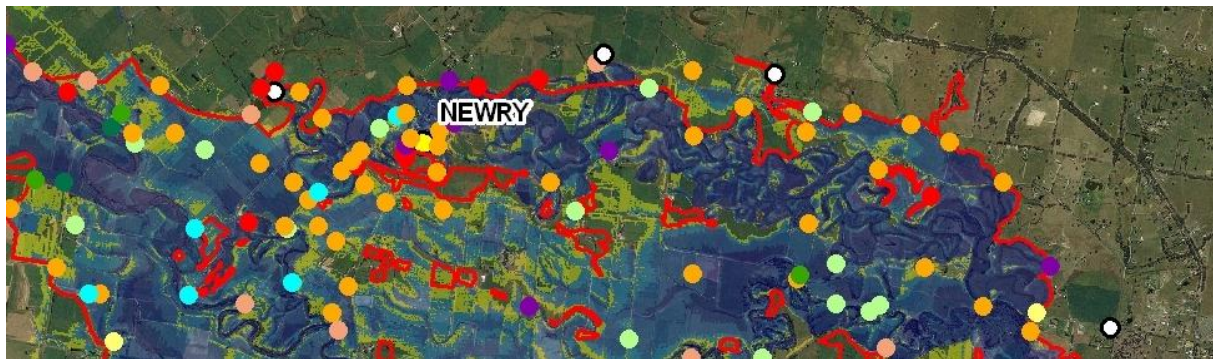


Figure 5-11 June 2007 – model calibration (final ALS) – Newry Creek

Boggy Creek Floodplain

Breakouts to the Boggy Creek system occurred along the right bank of the Macalister River in vicinity of Greenvale. These breakout flows were controlled by the Tinamba Northern channel and two floodways through the channel.

The flood extent along the western limit of the Boggy Creek floodplain was controlled by irrigation channels (Southern Channel). Comparison of observed and modelled extents showed a reasonable agreement. The modelled flood extent displayed some minor overtopping of the channel banks, not seen in the observed flood extents. Along the western edge, the modelled flood levels were generally within 0.1 m (orange points) of the observed flood levels.

Adjacent to Tinamba, the modelled flood levels were generally within 0.2 m (light green, orange and red). To the north of Tinamba, along the Tinamba-Newry Road, the modelled flood extent underestimated the observed flood extent.

To the south of Tinamba, the modelled flood levels were generally within 0.2 m (light green, orange and red).

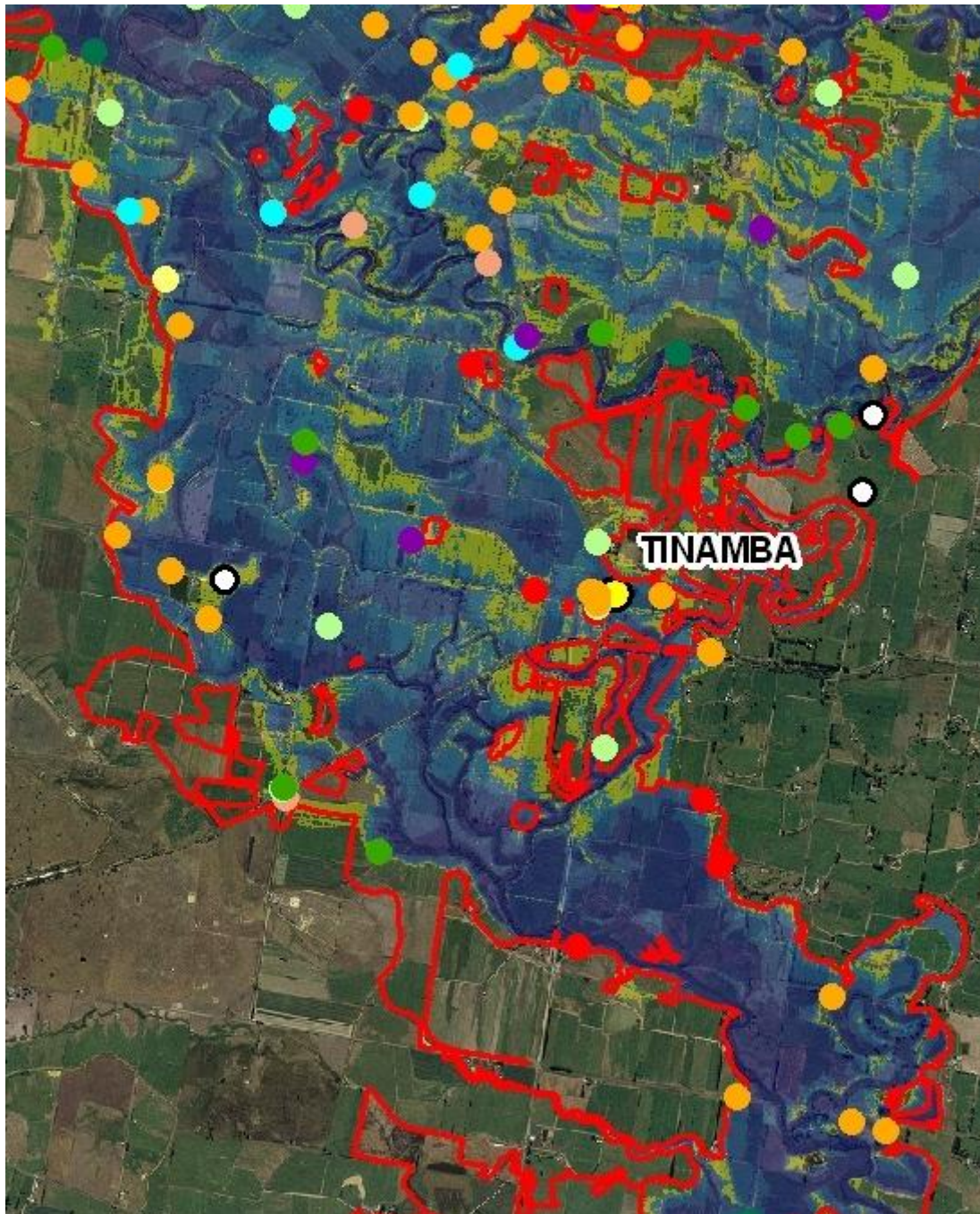


Figure 5-12 June 2007 – model calibration (final ALS) –Boggy Creek

Hydraulic roughness (Manning's n) was a key parameter resolved in the model calibration. The adopted Manning's values were:

- Macalister River channel and adjacent floodplain:
 - Regular section & light vegetation: 0.055 -0.06
 - Irregular section & heavy vegetation: 0.08 -0.11
- General floodplain (cleared & irrigated pasture): 0.04 -0.05
- Levee/embankment crest: 0.2

The above adopted Manning's n values were considered in line with typical values applied. As discussed, an increased Manning's n values were applied to levee/embankment crests. This

increased value was required to reflect the “weir” behaviour exhibited when overtopping occurred. Figure 5-13 shows the adopted Manning’s n values.

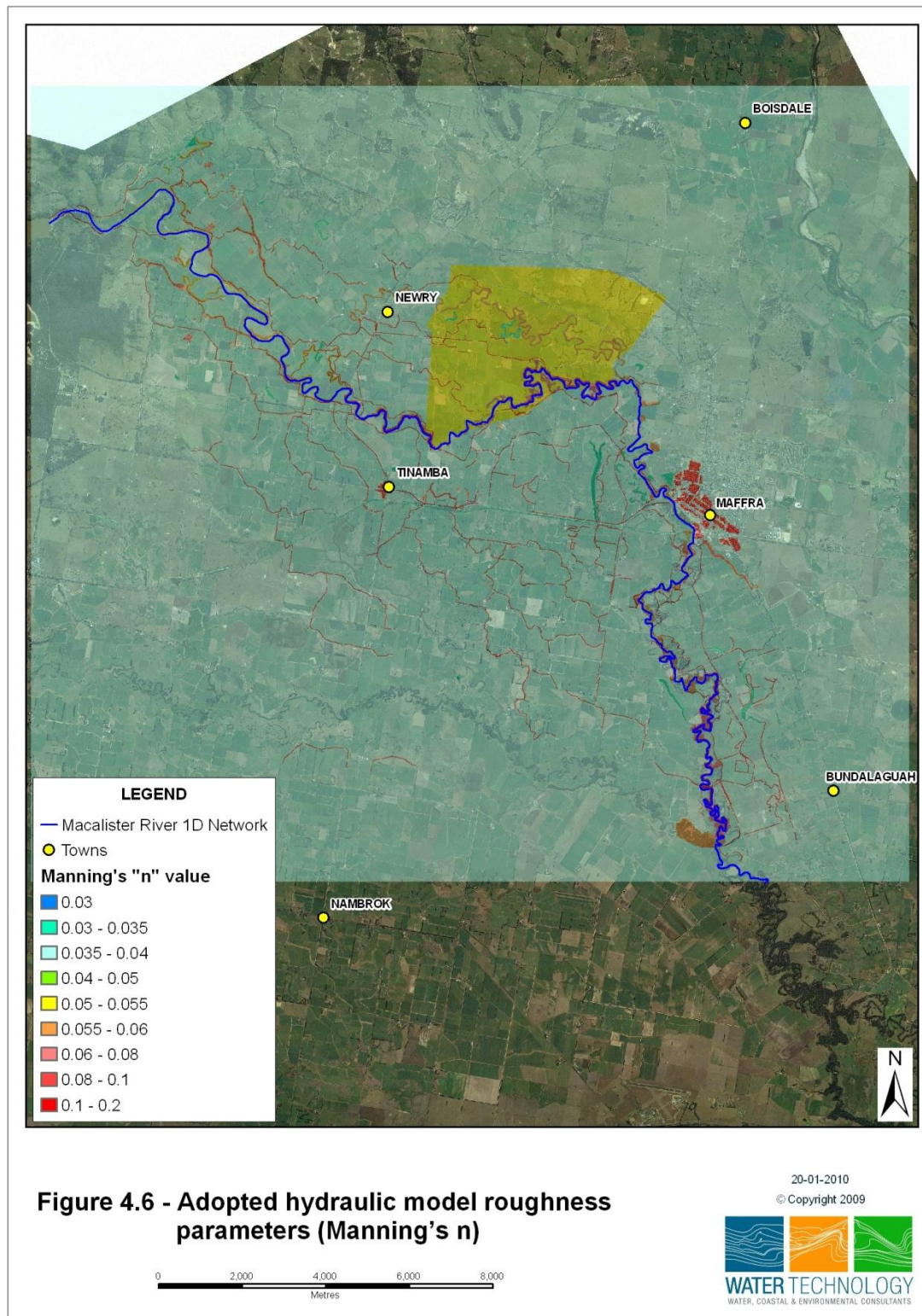


Figure 5-13 Adopted hydraulic model roughness parameters (Manning's n)

June 1978 Event

The June 1978 is the third largest flood since streamflow records began in 1919. The peak flow was 88,900 ML/d (downstream of Lake Glenmaggie), approximately ~ 20 year ARI.

The State Rivers and Water Supply Commission (SRWSC 1978) surveyed 72 observed flood level for the June 1978 event from downstream Lake Glenmaggie to the Macalister – Thomson River confluence. SRWSC (1978) provided the following comments on the accuracies of the surveyed flood levels:

“The elevations of the flood profiles shown in Appendix 5 are given to the first decimal place for three reasons. First, there is doubt as to the identification of some of those marks which were up to seven weeks old at the time of the survey. Second, the precision of an elevation shown to two decimal places can be misinterpreted. Third, the elevation refers to a small area around each point which is generally close to the river course. It would be dangerous to extrapolate these elevations across the direction of flow to the edge of the floodplain.” P. 13 SRWSC 1978.

With the above comments in mind, a review of the observed flood levels was undertaken to ensure a consistent data set. The review compared adjacent flood levels within a reach, and compared the observed 1978 flood levels against the neighbouring June 2007 flood levels. The observed June 1978 flood levels were revised using the following criteria:

- observed levels lower than adjacent downstream flood levels were removed
- observed levels higher than adjacent upstream flood levels were removed
- observed June 1978 flood levels higher than adjacent June 2007 were removed.

Following this revision, 32 out of 72 June 1978 were considered suitable for use in the model validation.

The model configuration (topography and roughness) employed in Run 17 (from the June 2007 calibration) were applied for the June 1978 validation event.

Figure 5-14 displays the modelled June 1978 flood extents, and flood level differences. The colours of dots reflect the differences in the modelled and observed flood levels. The dark and light green dots indicates observed flood levels were under-estimated, the orange dots indicate the modelled and observed flood levels are within 0.1 m and the red and purple dots indicates observed flood levels were over-estimated.

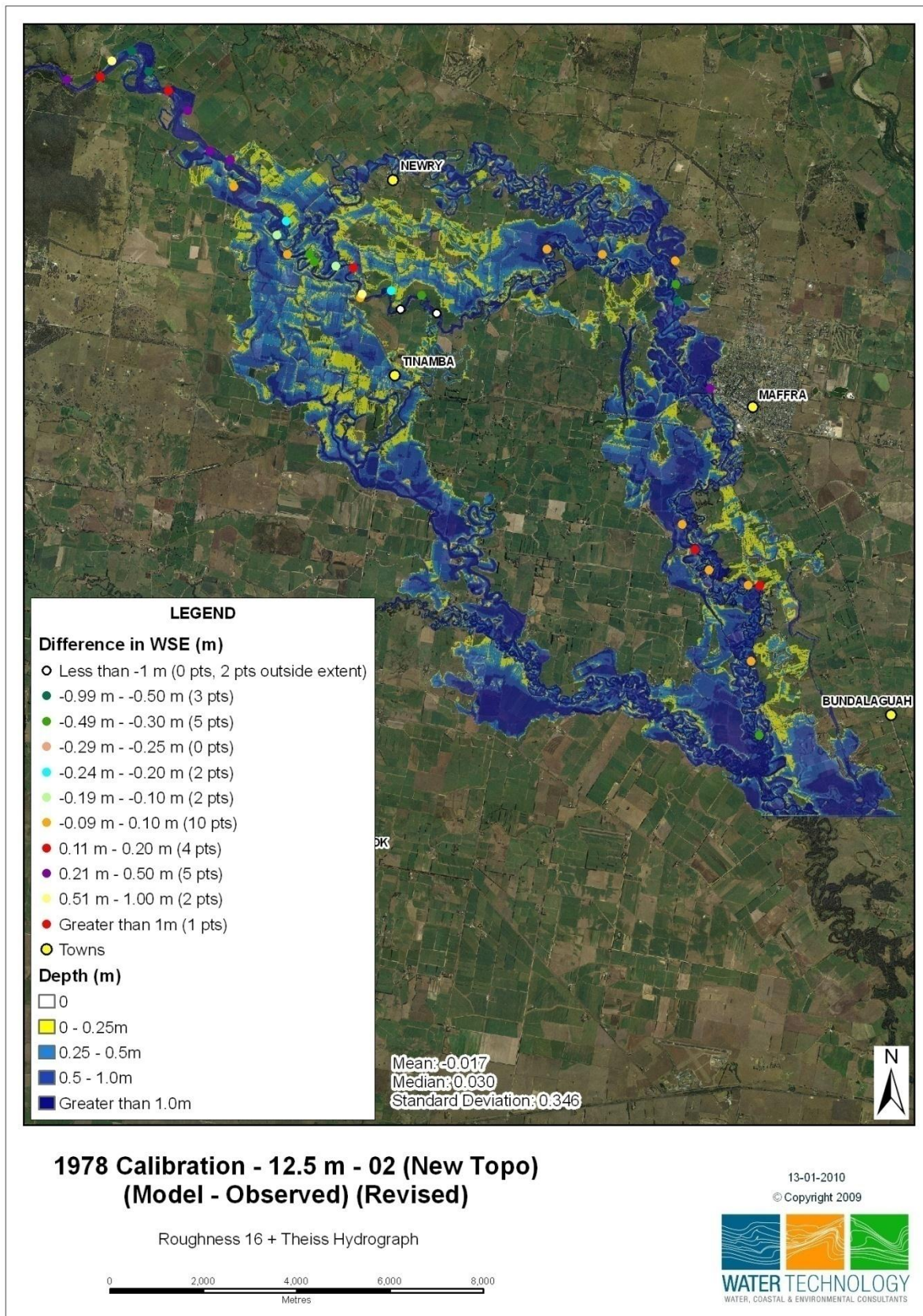


Figure 5-14 June 1978 – model validation – Flood level differences

The following provides a discussion of the model validation outcomes for the June 1978 event. The comments on the model validation were limited due to the small sample of observed flood levels, and the inherent uncertainties surrounding the observed flood levels discussed above.

Macalister River: Lake Glenmaggie to Hagens Bridge

Similar to the June 2007 calibration event, the flow was generally well confined to the channel and the immediate surroundings between Lake Glenmaggie and 300 m upstream of Hagens Bridge. Figure 5-15 displays the validation results in this reach.

Further, a similar pattern as the June 2007 calibration event was seen, with the validation results mixed. The modelled flood levels were both considerably lower and higher (± 0.2 m) (dark green, purple and yellow points) than observed flood levels.

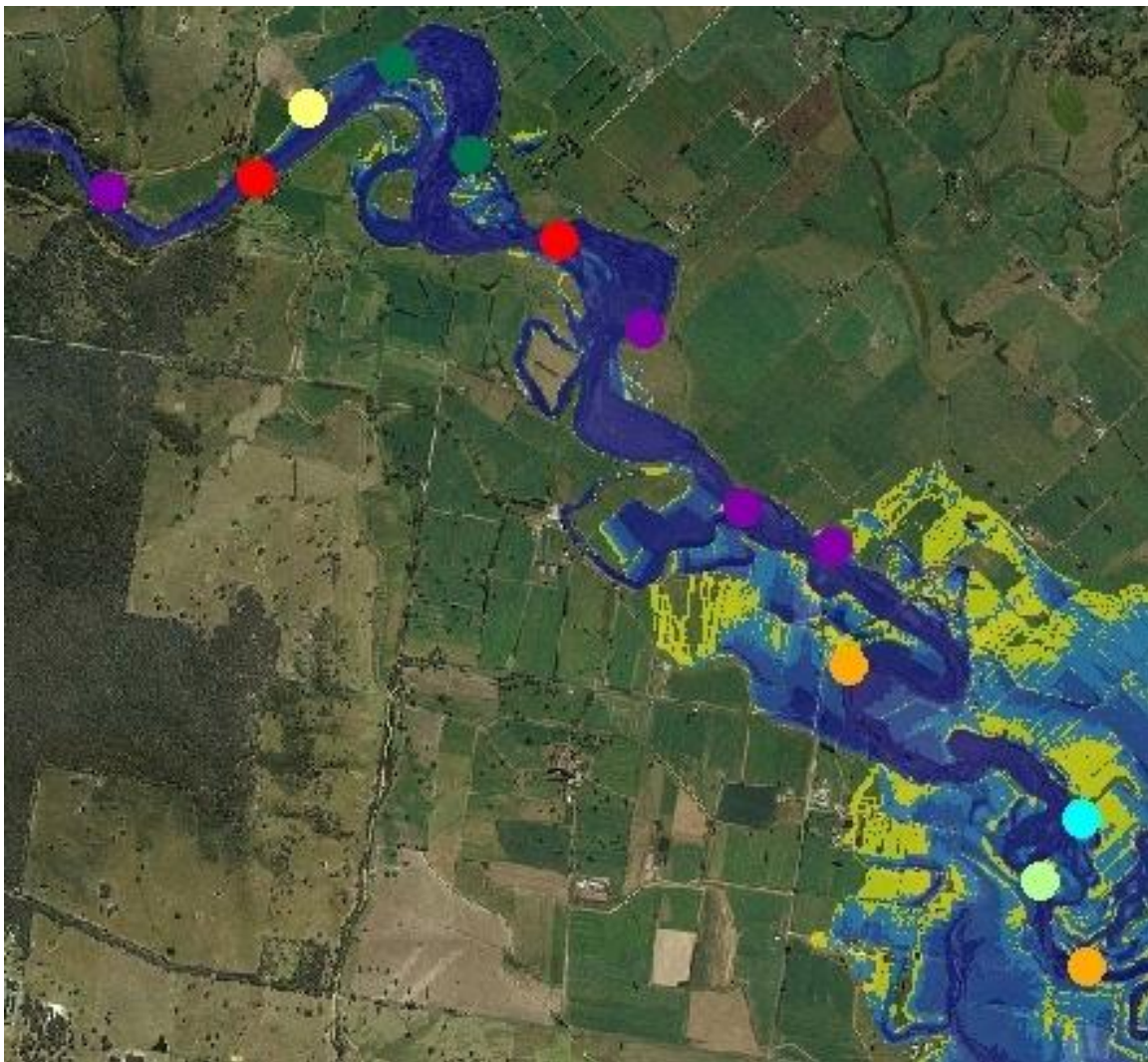


Figure 5-15 June 1978 – model validation – Macalister River Lake Glenmaggie to Hagens Bridge

Macalister River: Hagens Bridge to Mewburn Park

Figure 5-16 shows the validation results along this reach. As per the June 2007 calibration event, along the river, the modelled flood levels were generally lower than the observed flood levels (green and blue points).

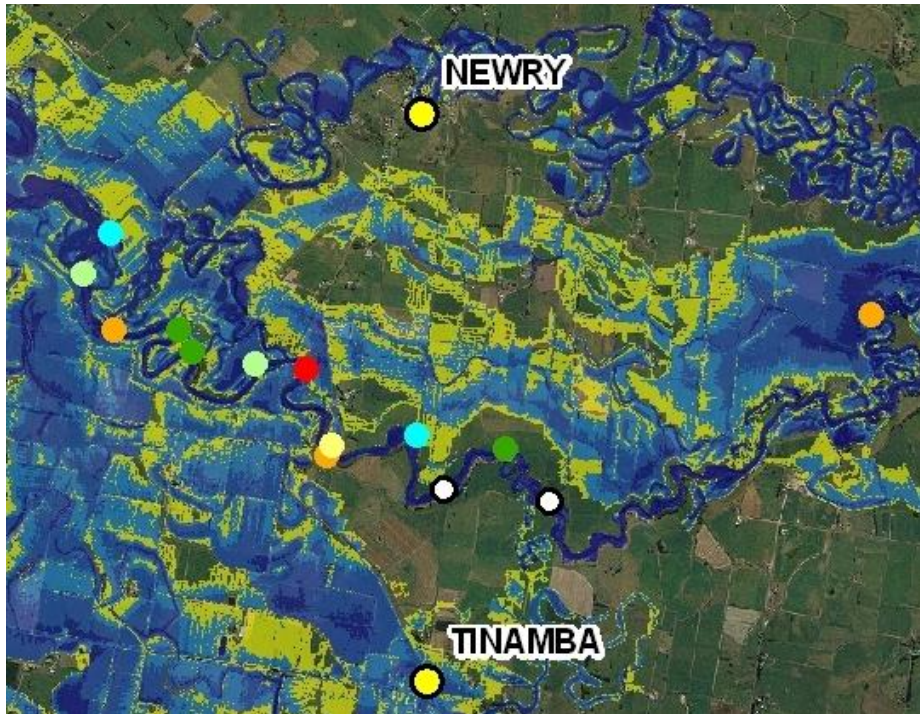


Figure 5-16 June 1978 – model validation – Macalister River Hagens Bridge to Mewburn Park

Macalister River: Mewburn Park to Bellbird Corner

Figure 5-17 shows the validation results along this reach. Good agreement, within 0.1 m (orange points) between the modelled and observed flood levels occurred at three locations. There were two other locations where the modelled flood level was lower than the observed flood levels (green points).

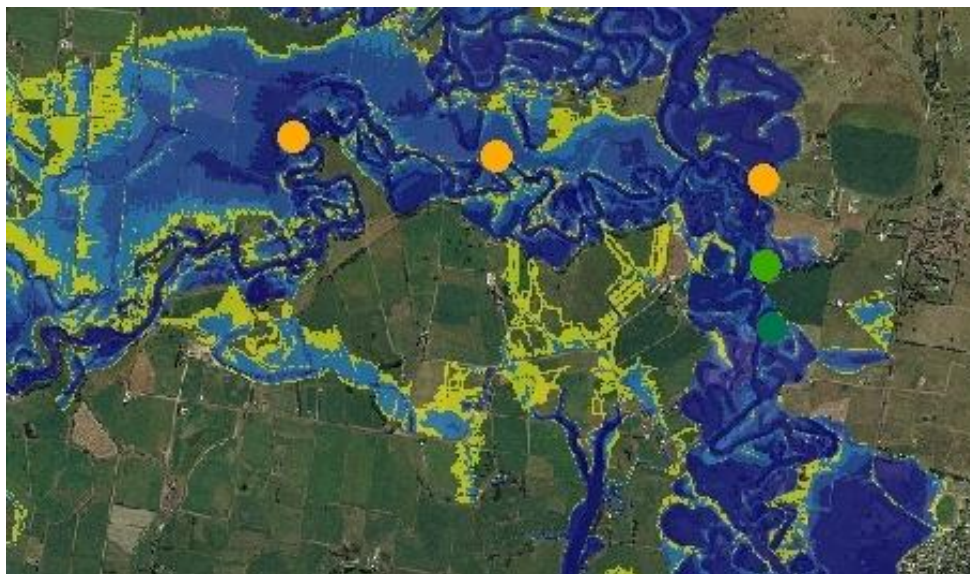


Figure 5-17 June 1978 – model validation – Macalister River Mewburn Park to Bellbird Park

Macalister River: Bellbird Corner to Maffra Weir

Figure 5-18 shows the validation results along this reach. There were only three observed flood levels available in this reach. The comparison of modelled flood levels at these three locations was mixed. At two locations considerably lower by at least -0.2 m (green points), and considerably higher by greater than +0.2 m (purple point). Given small sample of observed flood levels, comments on the model validation were constrained.

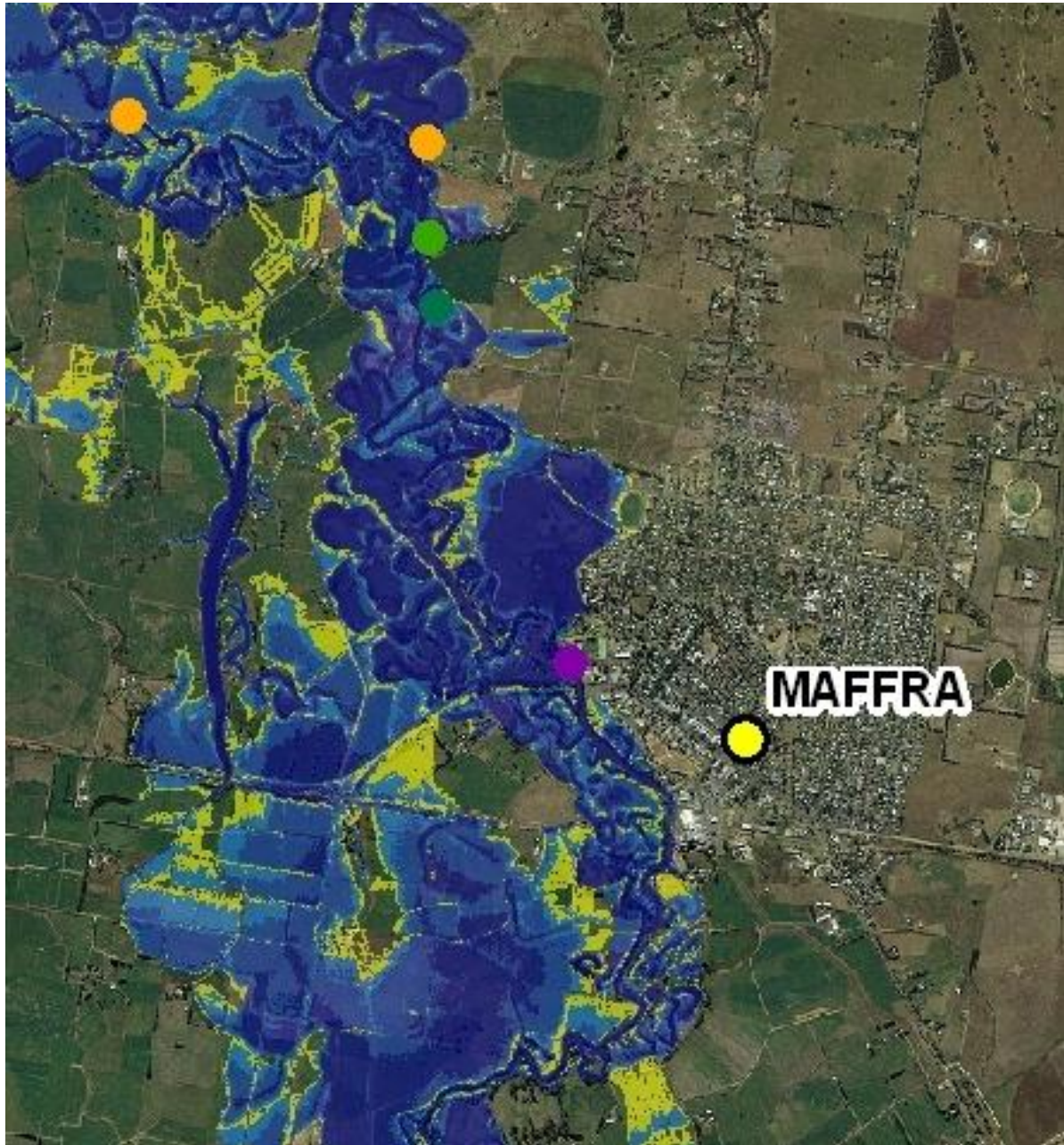


Figure 5-18 June 1978 – model validation – Macalister River Bellbird Park to Maffra Weir

Macalister River: Maffra Weir to Thomson River

Figure 5-19 shows the validation results along this reach. A comparison of modelled and observed flood levels (orange and red points) showed a reasonable agreement, with modelled levels within 0.2 m of observed flood levels.

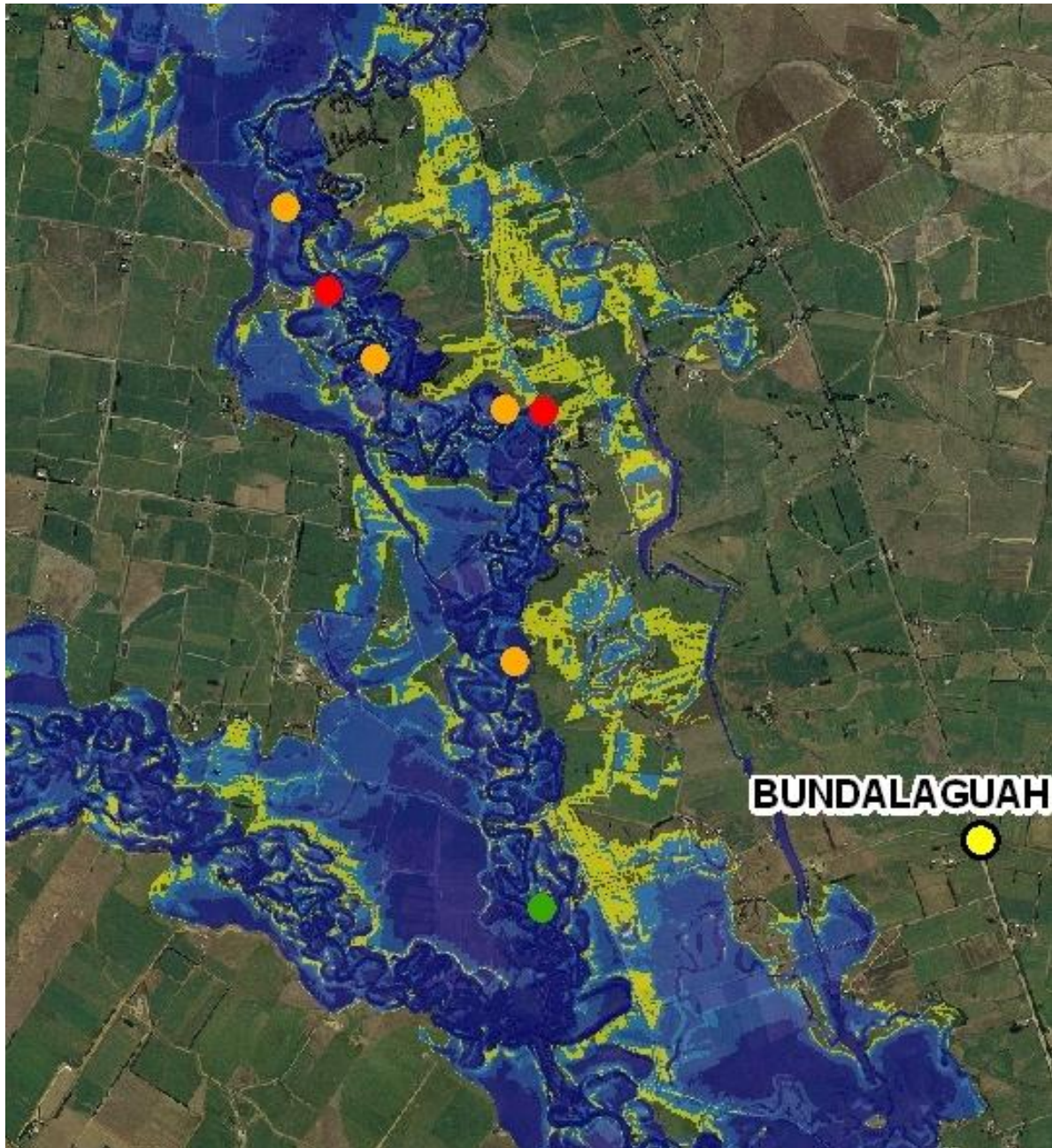


Figure 5-19 **June 1978 – model validation – Macalister River Maffra Weir to the Thomson river confluence**

Newry Creek and Boggy Creek Floodplains

No observed flood levels were available for either the Newry or Boggy Creek floodplains.

5.3.7 Overview of Calibration Approach

The hydraulic model calibration involved three distinct phases:

- Linked 1D-2D model calibration using *the interim ALS*
- Standalone 2D model calibration using *the interim ALS*
- Standalone 2D model calibration using *the final ALS*

Through each phase, a number of model calibration runs were undertaken, with the June 2007 modelled and observed flood levels compared. The flood level differences were grouped to allow the assessment of the model performance. Figure 5-20 displays the flood level difference grouping for the best calibration run from the three phases, with the flood level differences from the WBM (2008) calibration.

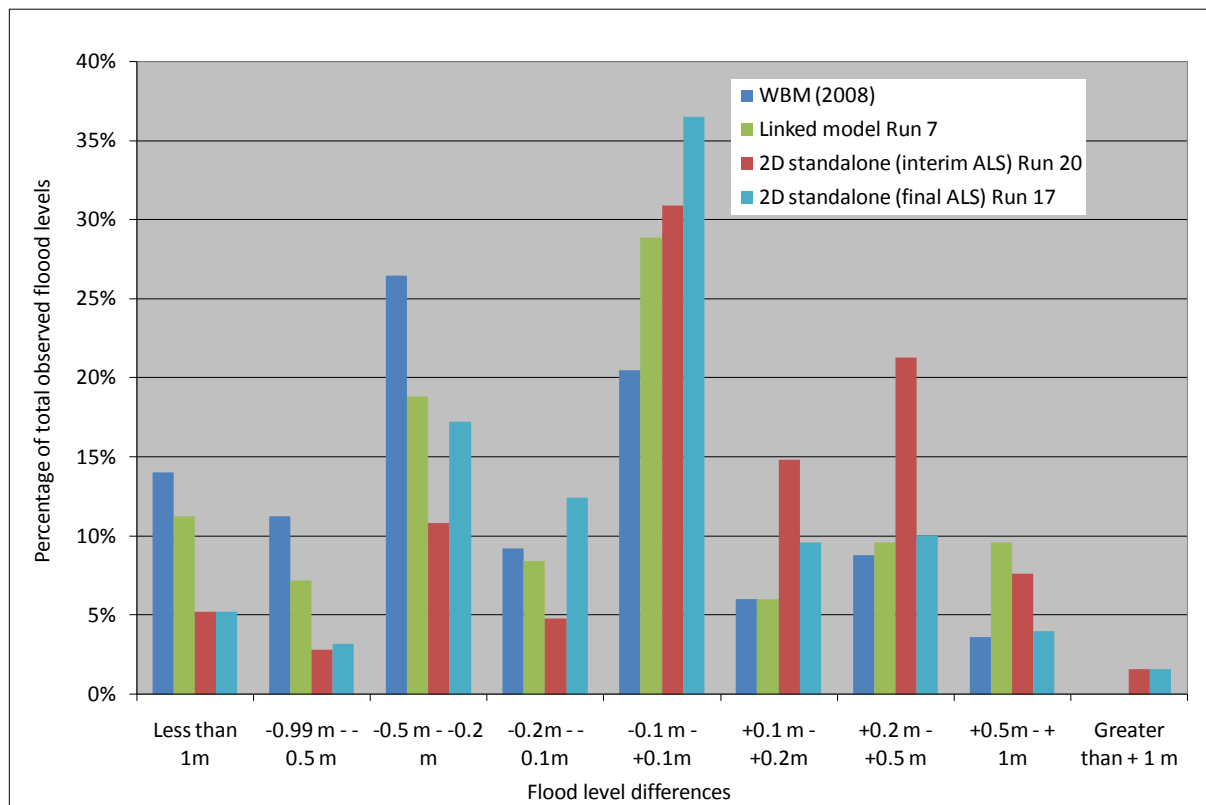


Figure 5-20 Hydraulic model calibration – Comparison of model performance

The WBM (2008) calibration shows a somewhat scattered pattern of flood level differences. Some 63% of flood levels were less than -0.2 m or greater than +0.2 m, and there was a general bias to the under-estimation of observed flood levels.

The linked 1D-2D model from the present study also showed a bias toward the underestimation of observed flood levels .

The 2D model using the interim ALS, show a more centred distribution of flood levels differences. However, there was a general bias to the over-estimation of observed flood levels. The use of the final ALS data in the standalone 2D model (Run 17) provided the strongest calibration result. A greater proportion of flood level differences lie within 0.1 m (36%). Furthermore, there is no significant bias in the modelled flood levels, with the mean difference of -0.03 m.

The discussion of the June 2007 calibration for the final ALS standalone 2D model noted several locations where the model calibration was weaker. Considerable investigation of these locations was

unable to improve the model calibration. The reasons for the flood level differences in these locations may include uncertainty associated with the following aspects:

- Input data for the hydraulic modelling
 - ALS data
 - Bathymetry and cross section survey
 - Definition of hydraulic controls/structures
 - Observed flows for model input
 - Observed flows and water levels for model calibration
- Hydraulic modelling process:
 - Model numerical and computational schemes – these relate to the ability of the model to replicate the physics of free-surface flow in channels and over land.
 - Floating point accuracy of computing resources (truncation error)
 - Model schematisation and set-up (location and spacing of cross-sections, grid resolution)
 - Model parameters such as computational time-steps, surface-friction and other energy-loss parameters (expansion/contraction coefficients and eddy viscosity for example).

The Macalister River floodplain downstream of Lake Glenmaggie contains a complex array of topographic features (irrigation infrastructure, road/rail embankments). These features strongly influence the flood behaviour. The model construction and calibration undertaken as part of this study, has yielded a hydraulic model fit for purpose.

6. HYDRAULIC MODEL APPLICATION AND FLOOD MAPPING

6.1 Overview

This section discusses the application of the hydrologic and hydraulic models to simulate and map flood behaviour (extents, depth, velocities) for a range of flood magnitudes.

The hydrologic analysis provided flood inflow hydrographs downstream of Lake Glenmaggie. These inflow hydrographs were routed through the calibrated hydraulic model. This enabled the modelling and mapping of flood depths, extents, velocities over a range of flood magnitudes.

The mapping outputs can be applied for flood response planning, and land use planning purposes.

6.2 Design Flood Behaviour and Mapping

The hydrologic analysis estimated design flood hydrographs for the 5,10,20,50,100 and 200 year ARI events plus the PMF event. The peak flow estimate for the 5 year was 46,720 ML/d. This compared to the minor, moderate and major flood class triggers of 7,500 ML/d, 20,000 ML/d and 40,000 ML/d respectively. To provide a suite of flood maps across the range of flood magnitudes of interest, flood mapping was undertaken for the peak flows of 7,500 ML/d, 20,000 ML/d, 30,000 ML/d and 40,000 ML/d, in addition to the 5,10,20,50,100 and 200 year ARI events plus the PMF event.

All flood mapping data was prepared to the VFD metadata specifications. The following VFD layers were delivered:

- Flood extents: Statistical – FLXT_STAT9 (20%,10%, 5%, 2% 1% & 0.5% AEP events)
- Limit of study: FLD_LOS
- Flood heights: Statistical - FLHITE_ST9 (20%,10%, 5%, 2% 1% & 0.5% AEP events)

6.3 Flood Response Mapping and Flow Correlation

6.3.1 Flood Response Mapping

A suite of ten flood response plans were developed corresponding to the following peak flows downstream of Lake Glenmaggie:

- 7,500 ML/d : Minor flood level
- 20,000 ML/d: Moderate flood level
- 30,000 ML/d
- 40,000 ML/d: Major flood level
- 46,720 ML/d: 5 year event
- 69,190 ML/d: 10 year event
- 90,680 ML/d: 20 year event
- 116,600 ML/d:50 year event
- 134,100 ML/d: 100 year event
- 149,700 ML/d 200 year event

The flood response plans included the following:

- Flood extent
- Flood level contour at 0.2 m interval
- Flood flow vectors (quantity and direction)
- Velocity at key locations (i.e. road crossings and other points required by the VICSES)
- Depth of inundation
- Identification of essential services
- Properties affected below and above floor level
- Road/street names
- SRW channels
- Major and minor streams aligned and mapped relative to the cadastral base
- Land marks, including all physical man-made features particularly those affecting flood flows and distribution

Soft copies were provided as JPEGs and PDF, hard copy maps were prepared at 1: 40,000 for A1 sheet. Related GIS files were provided in ARCGIS format.

6.3.2 Flow Correlations

Flood intelligence tabulations were prepared showing general behaviour linked to flood marker height, as shown in Figure 6-1 for Newry and Tinamba. Locations were chosen that were flood prone, central to the community and locations on public land with easy access. Flood marker height is taken from the natural surface.

For the Jones Street site in Newry the flood marker height reached a value of 3.6 m (indicating a flood depth of 3.6 m in the 2007 event. The Jones Street site was chosen because it was in a natural depression which is more flood-prone than the rest of Newry. The specific location shown in the figure is on the southern side of the road, where the surface is lower, rather than the middle. Across the middle of the road the modelled flood depth was about 2.4 m. Once a final location is selected for the flood marker in Newry a survey will be required to ascertain the true natural surface level and finalise the flood marker heights.

At present, the flow correlation for Tinamba is at the tennis courts which are away from the main focus of town. It would also be possible to establish a site around the main intersection in the town as the whole town is inundated for the 5 year ARI event and above. The flood marker heights will need to be recalculated once a specific flood marker site has been adopted.

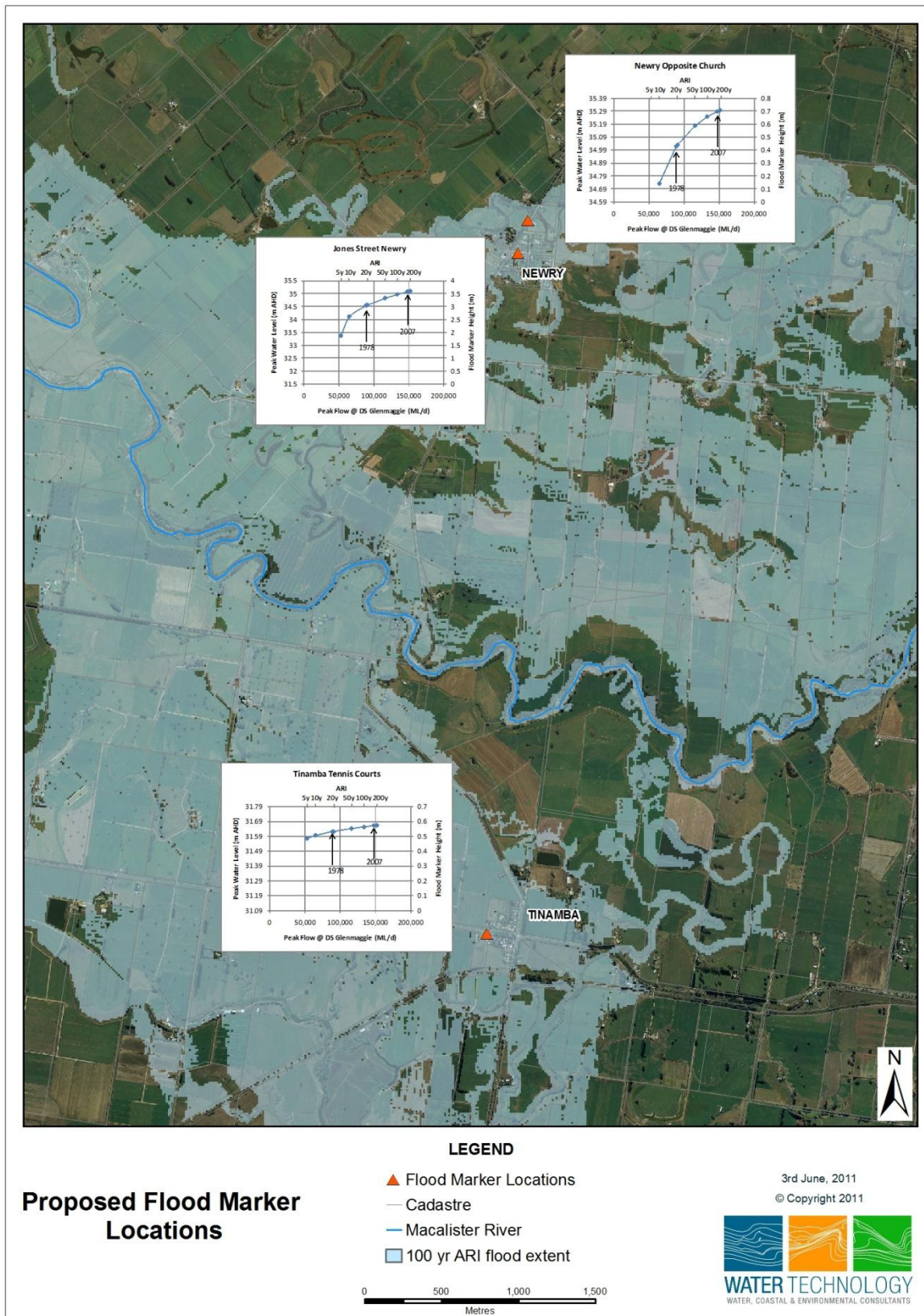


Figure 6-1 Proposed flood level correlations

6.4 Land Use Planning Mapping

Victorian Planning Provisions – Applying the Flood Provisions in Planning Schemes DOI (2000) sets out guidelines for the delineation of potential urban floodway zones (UFZ), floodway overlays (FO) and land subject to inundation overlays (LSIO) for land use planning purposes. The delineation of the UFZ, FO and LSIO are based on the following considerations:

- Flooding characteristics – frequency, velocity-depth (flood hazard), duration, storage, strategically significant sites (breakouts) and warning time.
- Socio-economic characteristics – population at risk, properties affected, damages and ease of evacuation.
- Environmental characteristics – areas of significant habitat, wetlands, riparian vegetation.

Model outputs (flood extents, flood depth and flow velocity) from the hydraulic analysis were employed to provide information on the flooding characteristics over the study area. Floodway options maps (at the same scale as the flood inundation mapping) were produced showing areas of potential UFZ, FO and LSIO based on flooding characteristics.

The floodway options maps were revised and amended in consultation with the Wellington Shire and West Gippsland CMA. Through this consultation, due consideration was given to local social, economic and environmental issues.

7. FLOOD DAMAGES ASSESSMENT

7.1 Overview

The purpose of the flood damages assessment is to estimate the likely damage bill associated with flood events in the Macalister floodplain downstream of Glenmaggie Weir. Flood damages estimates have been calculated for a range of flood events from a 5 year ARI to the PMF event.

Key datasets that were required for the assessment process included:

- Topographic data (as described in Section 3) including aerial imagery and LiDAR,
- Location and description of buildings with floor levels and details of other infrastructure
- Design flood levels (from Section 6)
- Flood damage cost from available literature.

The information presented in this Section is for the existing conditions on the Macalister floodplain. It could in future be used to undertake cost-benefit analyses between different flood mitigation options.

7.1.1 Topographic Data

The detailed terrain model developed for this study was used for the assessment, as detailed in Section 3.

7.1.2 Property Survey Data

A floor survey was undertaken by Beveridge Williams for the study area, comprising approximately 67km². A total of 897 buildings (which includes rural, residential, commercial and farm buildings) were identified within this area, and due to budget constraints a total 96 properties were surveyed. The surveyed properties were selected as they were located within the 200 year ARI flood extent in the townships of Newry and Tinamba. The remaining properties were taken into account during the damages assessment through manually locating buildings using high resolution aerial imagery. Where floor survey could not be obtained, a freeboard of 200mm was assumed above the topographic elevation derived from LiDAR.

7.1.3 Design Flood Levels

Design flood levels predicted by the hydraulic modelling (Section 6) were used to assess the damage risk at each property.

Due to the widespread nature of the study area the design flood level for each flood event depends on the location under consideration. Detailed hydraulic model outputs were used to assess the site specific flood level for each location in the damage assessment.

7.1.4 Flood Damage Cost Information

Six key sources were used for flood damage cost estimation methodology:

- ANUFLOOD cost curves, from CRES (1992)
- 'Rapid Appraisal Method (RAM) For Floodplain Management' (Read Sturges & Associates, 2000);
- 'Economic Costs of Natural Disasters in Australia' (Bureau of Transport Economics, 2001); and,
- 'Guidance on the Assessment of Tangible Flood Damages' (Queensland Government Department of Natural Resources and Mines, 2002).

- Impact of velocity on flood damage assessments in a recent study by Geoscience Australia (Middelmann-Fernandes, 2010)
- 'www.abs.gov.au' (The Australian Bureau of Statistics).

7.2 Assessment of Flood Damages Cost

7.2.1 Existing Conditions

For the purposes of this flood damage assessment, the study area was defined as the properties within the 200 year flood extent as shown in Figure 7-1, with economic costs and benefits considering both rural and urban areas.

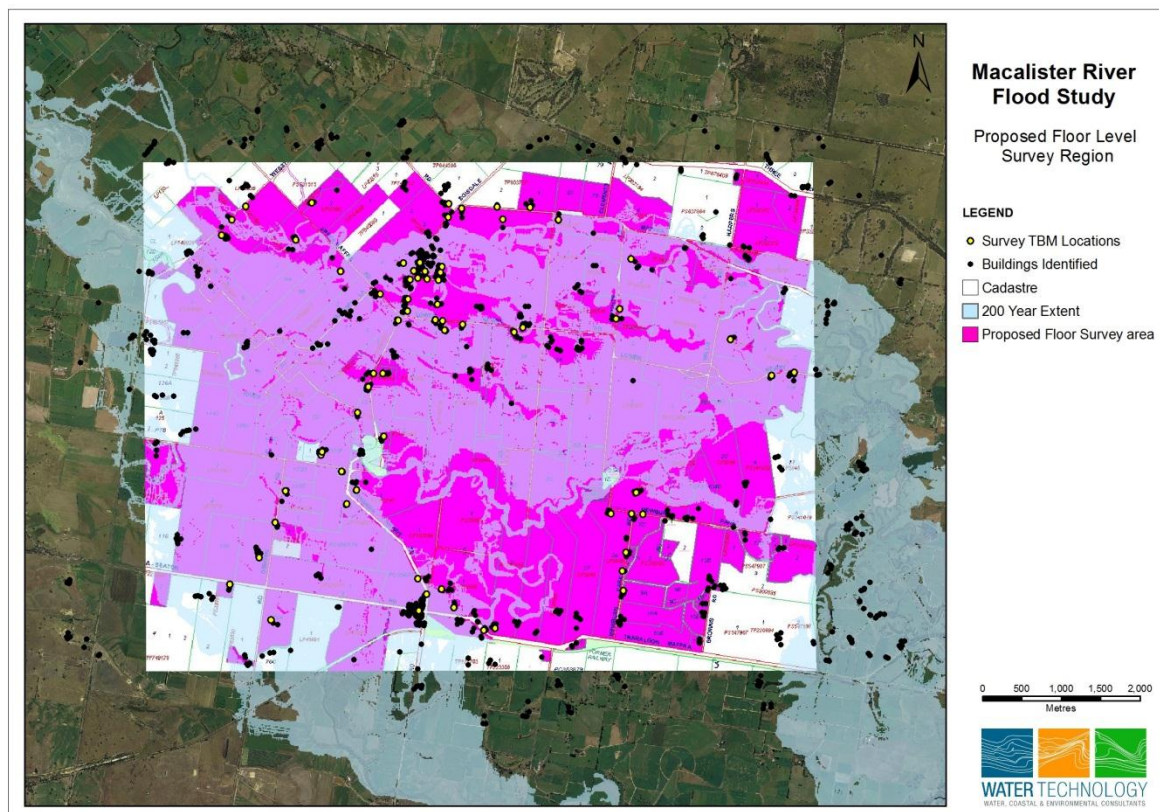


Figure 7-1 Damages Assessment Area and Floor Level Survey

7.2.2 Direct Damages to Buildings (commercial, residential, rural)

Floods can potentially cause a high level of damage to buildings, including structural damage (eg. walls, floors, doors, etc.), contents damage (eg. carpets, furniture, etc.) and external damage (eg. gardens, etc.).

For each building, a depth of above floor inundation was calculated under existing conditions for the design flood levels adopted from Section 6.2, using the floor levels from the property data described in Section 7.1.2. External damage was incorporated using the properties flooded below floor level for the same design events and property data.

Stage-damage curves estimate the relationship between the depth of above floor inundation of a building and the potential flood damage cost. This relationship is typically calculated by post-flood survey. ANUFLOOD stage-damage curves for residential and commercial buildings (NRM, 2002), were factored up by 60% to bring them up to a representative 1999 flood damage cost level, as

recommended by Read Sturgess & Associates (2000). They were then factored up to a June 2011 flood damage cost level using Building Price Index (BPI) and Consumer Price Index (CPI) from the Australia Bureau of Statistics website (www.abs.gov.au).

The total potential flood damage cost for existing conditions was then calculated by applying the updated stage-damage curves to each building and summing the individual potential flood damage costs.

The total potential flood damage cost represents the flood damage cost if no remedial action is taken. In reality, communities at risk of flooding will usually have some warning and will be able to take steps toward reducing the cost of flood damage (i.e. evacuation, doorstep sandbagging or removing valuable items to a safe level above flood waters). Read Sturgess & Associates (2000), estimated that for communities such as Newry and Tinamba, having prior flood experience and some warning time (2-4 hours), the ratio of actual to potential flood damage cost could be around 0.8. With increased warning time this ratio can be reduced to around 0.4. In this study a ratio of 0.8 was applied to the total potential flood damage cost as a conservative estimate of the total actual flood damage cost.

The total actual flood damage cost along with the number of residential, commercial and agricultural buildings inundated for the adopted existing condition design flood levels are presented in Table 7-1.

As presented in Table 7-1 the estimated actual flood damage cost for buildings under existing conditions is approximately **\$918,227** for the 100 year ARI event.

7.2.3 Direct Damages to Infrastructure and Agricultural Land

Floods can potentially cause significant damage to roads and other inundated infrastructure such as agricultural land. Roads can suffer initial damage from flooding as well as accelerated deterioration due to water intrusion under the pavement. While for agricultural land, the type of land use (e.g. crop type) is important for estimating likely damages.

The RAM method (Read Sturgess & Associates, 2000) includes costings for roads and bridges, and agricultural enterprises in addition to the direct building costs. The costs for roads, bridges and agricultural land from the RAM method were adopted and updated in the NRM (2002) report. These values were applied to the present study and factored up by CPI to June 2011 dollars.

The length of inundated major, minor and unsealed roads was calculated for the adopted existing condition design flood levels, and used to estimate the total cost of flood damage to, Table 7-2.

For agricultural land the land area per crop type was calculated and the appropriate cost per hectare applied in order to estimate the total cost of flood damage. The costs include the cost of clean-up of the agricultural land.

7.2.4 Indirect Damages

Indirect flood damages are damages sustained as a consequence of a flood but are not due to the direct impact of a flood (eg. emergency services, clean-up costs, alternative accommodation, disruption to business, etc.). Indirect costs are much harder to quantify than direct costs, so only the more readily estimated costs are usually included.

Read Sturgess & Associates (2000) recommend estimating indirect costs as 30% of total direct costs (depending on population density). This is a fairly coarse approximation and has not been adopted in this case. Instead a more detailed analysis has been undertaken, using methodology from BTE (2001).

Included in the estimate of indirect flood damage costs are residential and commercial clean-up, alternative accommodation and relocation of household goods, and emergency response costs, Table 7-3.

7.2.5 Total Existing Conditions Damages

The total flood damage cost for the Macalister study area under existing conditions is a sum of the actual flood damage cost of buildings, the road flood damage cost and the indirect flood damage cost, Table 7-4. The total existing conditions flood damage cost for the 100 year ARI event is **\$3,561,862**

This gives an annual average damage (AAD) cost of **\$515,765**. The AAD is a measure of the flood damage per year averaged over an extended period. It is calculated by the area under the flood frequency and total flood damage curve.

Table 7-1 Total Actual Flood Damage Cost to Buildings for Existing Conditions

Parameter	Average Recurrence Interval						
	PMF	200	100	50	20	10	5
Buildings Flooded Above Floor	549	111	96	75	51	35	28
Properties Flooded Below Floor	23	42	34	33	23	25	23
Total Properties Flooded	572	153	130	108	74	60	51
Direct Potential External Damage Cost	\$65,693	\$69,900	\$60,835	\$75,940	\$44,241	\$47,067	\$40,719
Direct Potential Residential Damage Cost	\$6,445,158	\$896,925	\$775,938	\$612,234	\$436,651	\$348,166	\$318,879
Direct Potential Commercial Damage Cost	\$8,231,323	\$421,749	\$311,011	\$219,656	\$140,778	\$96,881	\$79,789
Total Direct Potential Damage Cost	\$14,742,173	\$1,388,574	\$1,147,784	\$907,830	\$621,670	\$492,114	\$439,387
Total Actual Damage Cost (0.8*Potential)	\$11,793,738	\$1,110,860	\$918,227	\$726,264	\$497,336	\$393,691	\$351,510

*Note that these costs are for property damage only and do not include road repairs or indirect clean up costs.

Table 7-2 Total Actual Flood Damage Cost to Infrastructure and Agricultural Land for Existing Conditions

Parameter	Average Recurrence Interval						
	PMF	200	100	50	20	10	5
Farmland inundated (ha)	6043	3398	3187	3014	2767	2375	2087
Direct Actual External Rural Damage Cost (<i>0.8*Potential</i>)	\$2,668,633	\$1,523,894	\$1,428,111	\$1,349,359	\$1,237,715	\$1,060,650	\$929,901
Infrastructure Damage Cost	\$2,826,396	\$1,077,126	\$991,386	\$885,334	\$746,060	\$584,807	\$495,366
Total Actual Flood Damage Cost	\$5,495,029	\$2,601,021	\$2,419,496	\$2,234,694	\$1,983,775	\$1,645,456	\$1,425,268

Table 7-3 Total Indirect Damage Cost for Existing Conditions

Parameter	Average Recurrence Interval						
	PMF	200	100	50	20	10	5
Indirect Clean Up Cost	\$810,933	\$196,580	\$178,731	\$153,478	\$122,783	\$99,491	\$94,048
Indirect Residential Relocation Cost	\$101,532	\$23,665	\$22,139	\$19,085	\$15,268	\$12,978	\$12,214
Indirect Emergency Response Cost	\$23,269	\$23,269	\$23,269	\$18,615	\$13,961	\$9,308	\$4,654
Total Indirect Damage Cost	\$935,733	\$243,514	\$224,138	\$191,178	\$152,012	\$121,776	\$110,916

Table 7-4 Total Flood Damage Cost for Existing Conditions

Parameter	Average Recurrence Interval						
	PMF	200	100	50	20	10	5
Total Indirect Damage Cost	\$18,224,501	\$3,955,395	\$3,561,862	\$3,152,136	\$2,633,122	\$2,160,923	\$1,887,693

8. FLOOD RISK MITIGATION

8.1 Overview

Mitigation measures provide a means to reduce the existing flood risk. Mitigation measures can reduce existing flood risk by lowering the likelihood of flooding and/or lowering the flood damages (consequences) for a given flood depth. Mitigation measures can be broken into:

- **Structural:** Physical barriers or works designed to prevent flooding up to a specific design flood standard. Structural measures aim to reduce existing flood risk flood by lowering flood likelihood at given locations. Structural works include levees, floodways, waterway works, improvements to hydraulic structures.
- **Non-structural:** Management and planning arrangements between relevant authorities designed to reduce flood related damages. Non-structural measures aim to reduce existing flood risk flood by lowering flood damage. Non-structural measures include land use planning, flood warning, flood response and flood awareness.

The following discussion outlines the preliminary assessment of structural mitigation measures for the study area. Non-structural mitigation measures are discussed in detail in Section 8.3.

8.2 Structural Mitigation Measure Assessment

8.2.1 Overview

Structural mitigation measures are physical works to reduce the likelihood of flooding in a given location. Potential structural mitigation measures identified include the following:

- Upstream flood storage
- Levees
- Floodways
- Waterway management works
- Improvements to waterway structures (culverts etc)

Discussions with the West Gippsland CMA identified the mitigation of flooding impacts in Tinamba as the key study priority. To this end, the structural mitigation measures canvassed by this study focused on Tinamba.

Previous internal investigations of structural mitigation measures for Tinamba by the West Gippsland CMA, identified the potential use of existing embankments (irrigation channels and road) as levees. Also, the widening of opening (siphons) in irrigation channels as floodways was explored.

Through discussions with the West Gippsland CMA, three structural mitigation options were scoped for further assessment. Details of the three options follow and are shown in Figure 8-1:

- **Option 1:** Raise the farm channel and laneway (shown in red). The levee would need to link up with SRW's irrigation channel at its western end and with the Tinamba-Newry road at its eastern end.
- **Option 2:** Option 1 plus increase the floodway capacity of the SRW Siphon and lower farm channel to south of Tinamba Seaton Road (shown in pink)
- **Option 3** Build a new levee around the town (shown in Yellow) and increase flood conveyance though the open land beside the tennis courts.

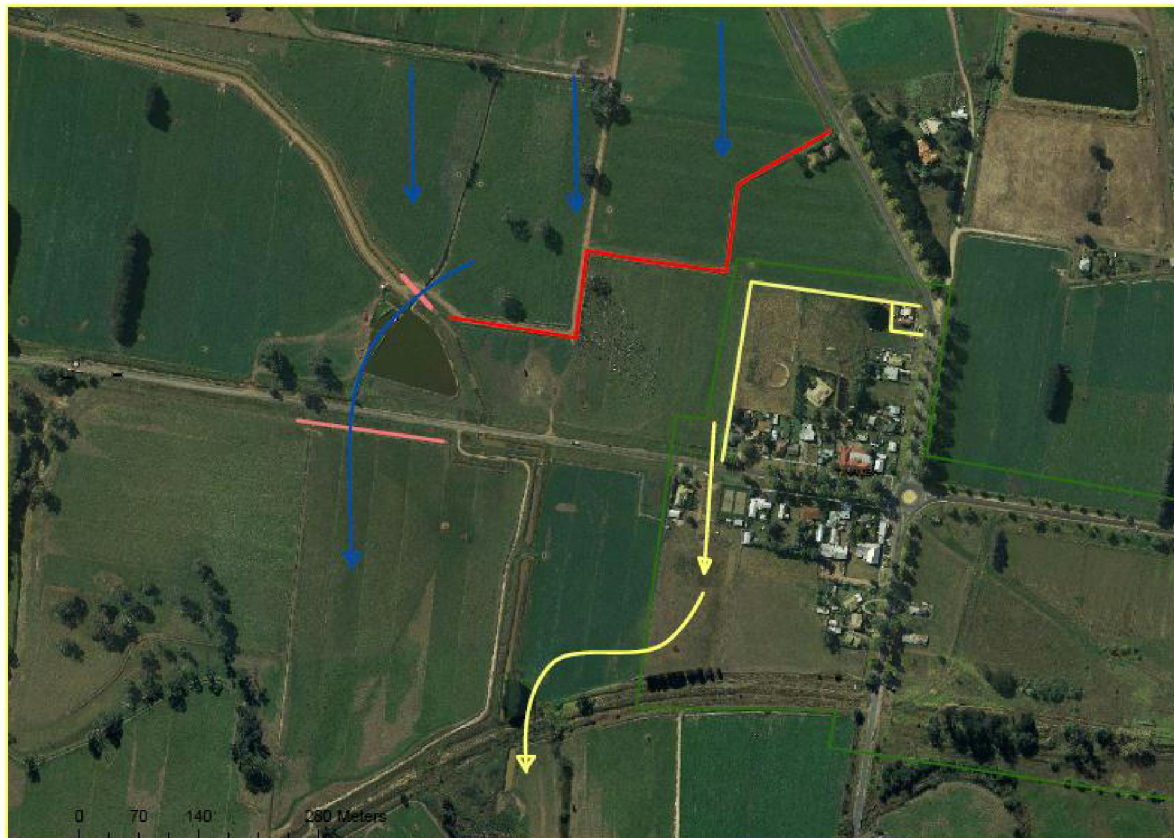


Figure 8-1 Tinamba – Structural mitigation options assessed

The impacts on flood behaviour for the above three options were modelled using the calibrated hydraulic model. For this preliminary assessment, only the flood behaviour impacts of the mitigation options were considered. Future assessments will need to consider the economic, social, cultural heritage and environmental aspects of any proposed mitigation measures.

8.2.2 Flood Behaviour Impacts Assessment

Option 1

Initial flood modelling showed that flood waters can over top the channel to the north of the Tinamba - Seaton Road, the road crest at the channel crossing, and across the channel to the south at the siphon opening. These flood waters can enter Tinamba, and affect properties to the south of the Tinamba Seaton Road. To prevent this flooding, the channel was raised and the opening at the siphon was filled. A temporary barrier will be required across the Tinamba Seaton Road. In past flood events, sandbags were placed between the channel headwalls across the road

. Figure 8-2 shows the general arrangement for Option 1.

Figure 8-3 shows the 100 year flood extent and depths with the Option 1 works. Figure 8-4 shows the differences in 100 year flood levels compared to the existing conditions. A positive difference (shown in red) reflects an increase in flood levels with works compared to the existing conditions.

Discussion of the changes in flood behaviour is provided in Section 8.2.3

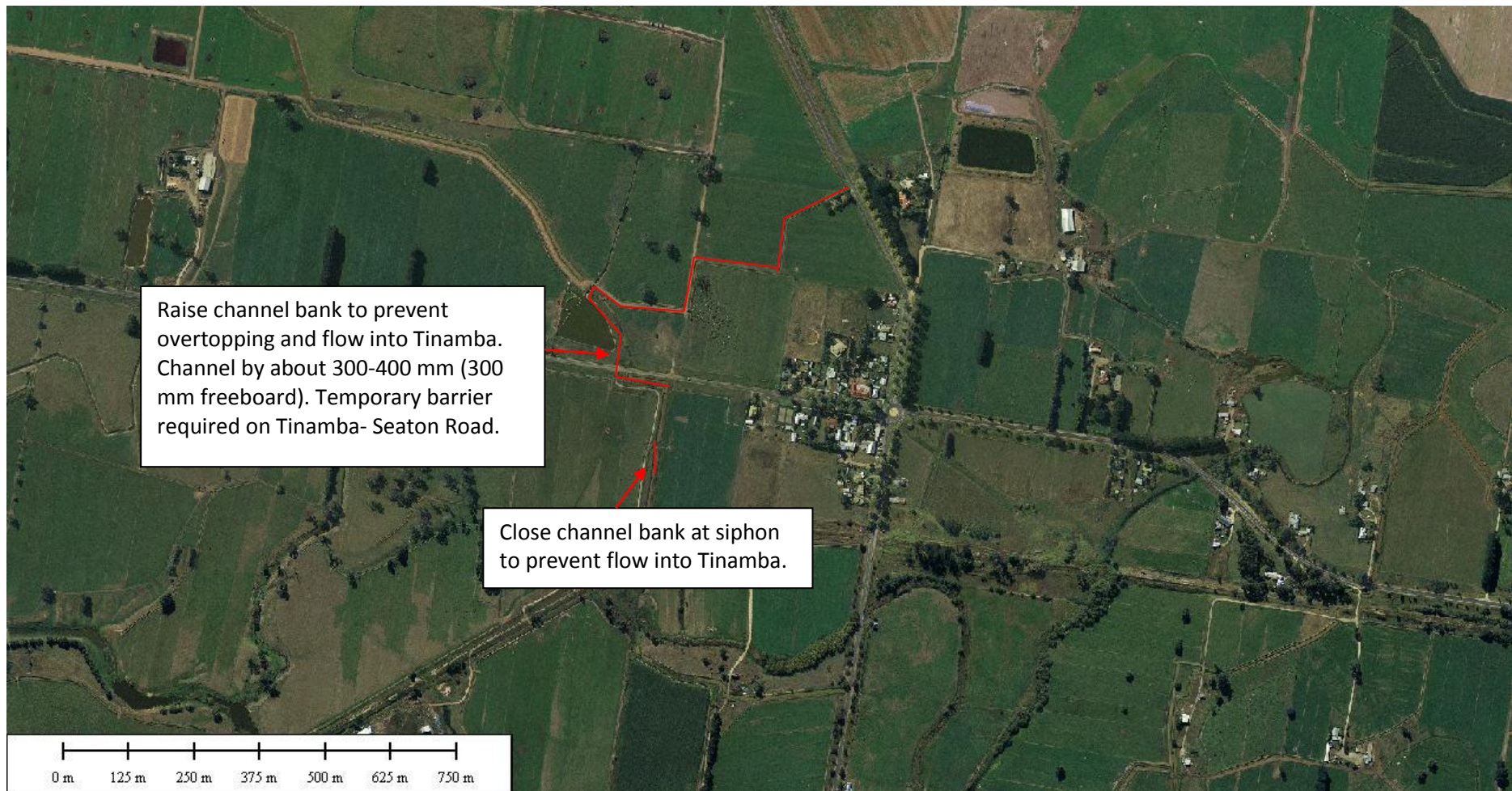


Figure 8-2 Tinamba Option 1 – General arrangements

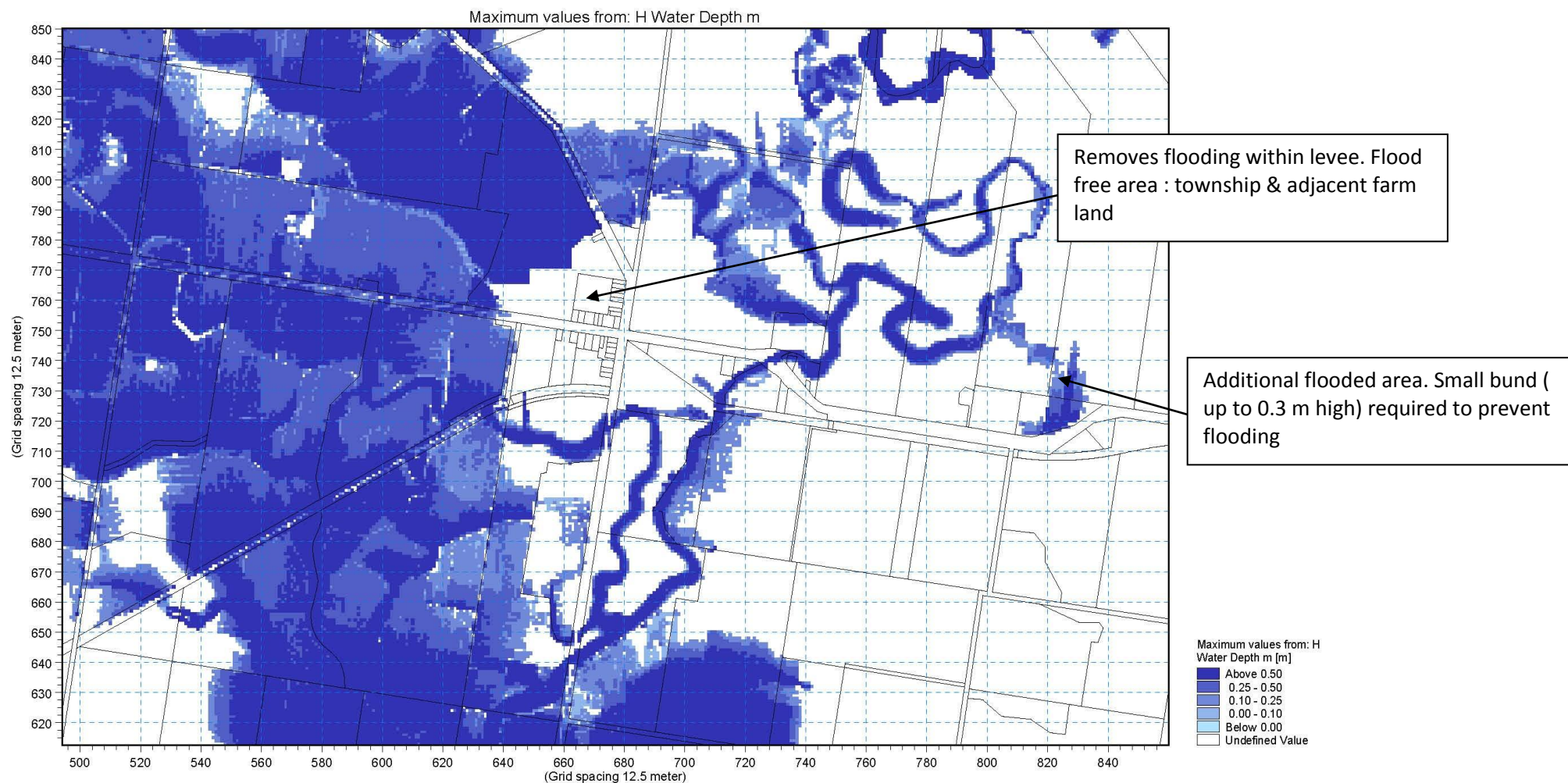


Figure 8-3 Tinamba Option 1 – 100 year flood depths

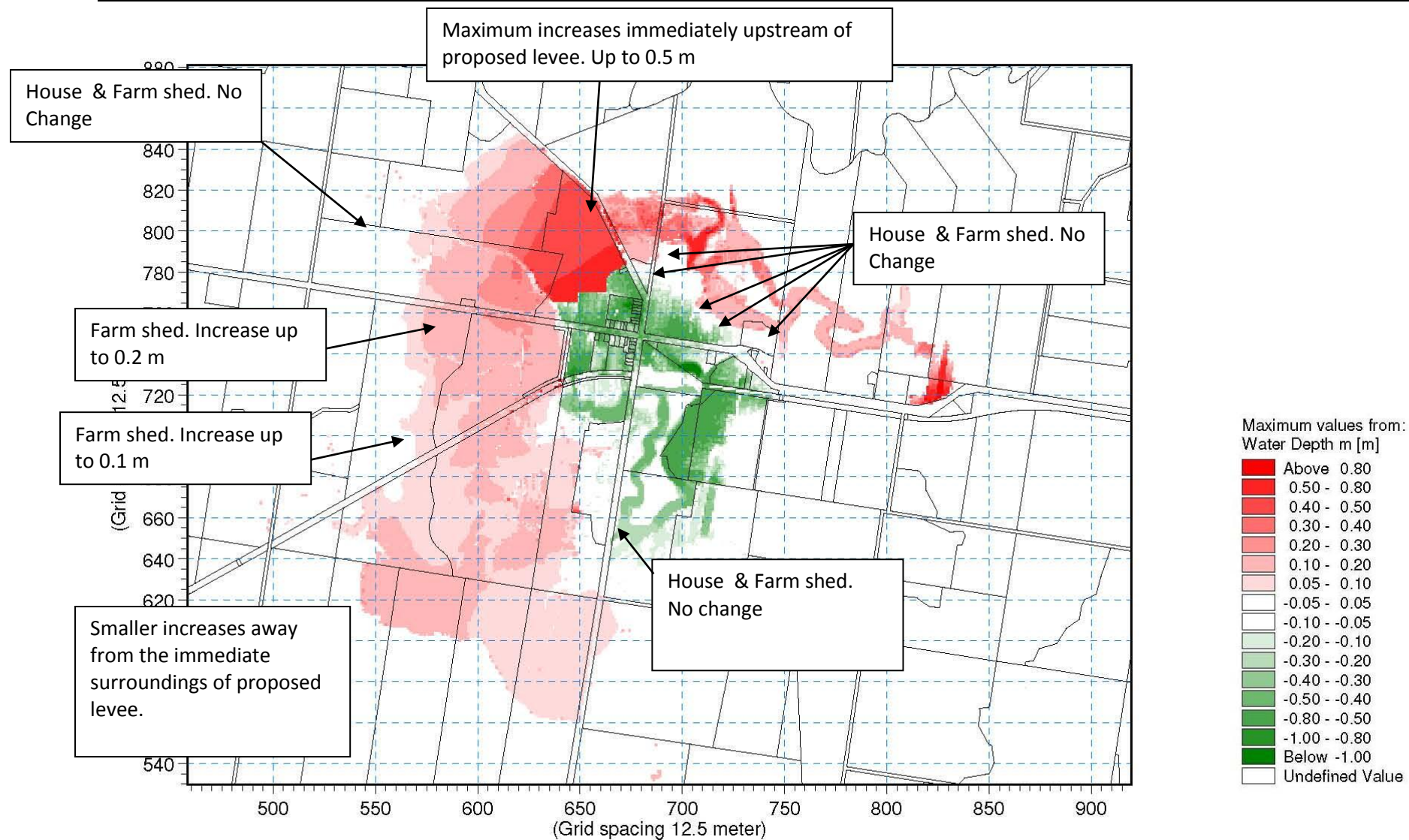


Figure 8-4 Tinamba Option 1 – 100 year flood levels differences

Option 2

Option 2 combined the Option 1 levee with a widening of the existing opening in the SRW channel to 100 m from 35 m. Figure 8-5 shows the general arrangement for Option 2.

Discussions with WGCMA suggested that the farm channel to south of the Tinamba Heyfield Road influenced local flood behaviour, and may need modification as part of mitigation works. A comparison of Tinamba Heyfield Road and farm channel crest shows that the farm channel is generally 100 -200 mm higher. The flood modelling suggested that the farm channel is not a significant control on neighbouring flood levels. As such, no changes were proposed to the farm channel.

Figure 8-6 shows the 100 year flood extent and depths with the Option 2 works. Figure 8-7 shows the differences in 100 year flood levels compared to the existing conditions. A positive difference (shown in red) reflects an increase in flood levels with works compared to the existing conditions.

Discussion of the changes in flood behaviour is provided in Section 8.2.3

Option 3

Option 3 consisted of a new levee around the north west corner of the township. As proposed in Figure 8-1, the levee abuts the Tinamba Newry Road at the north east limit, and the Tinamba Seaton Road at the south west end.

Figure 8-8 shows the 100 year flood extent and depths with the Option 3 works. Figure 8-9 shows the differences in 100 year flood levels compared to the existing conditions. A positive difference (shown in red) reflects an increase in flood levels with works compared to the existing conditions.

As seen in Figure 8-8, the Option 3 as proposed reduced flood depths in Tinamba. The levee is outflanked to the south of the Tinamba Seaton Road, and across the Tinamba Newry Road.

Discussion of the changes in flood behaviour is provided in Section 8.2.3

8.2.3 Discussion of Results

Option 1 prevented flooding of properties in Tinamba. The increases in flood levels associated with the re-distribution of flood flows do not adversely affect any dwellings. There were modelled increases at several farm sheds/outbuildings. Flood modelling indicated that the existing farm channel required rising by 1.0 m to 1.4 m to achieve a 600 mm freeboard in the 100 year event. As discussed, temporary barriers would be required across the Tinamba – Seaton Road, between the channel headwalls. This preliminary assessment only considered the hydraulic impacts of this option. Further functional and detailed design is required.

Option 2 provided similar reduction in flood levels and extents through Tinamba as Option 1. The increased opening in the SRW channel leads to a slightly lower increase in flood levels upstream of the proposed levee when compared to Option 1. Discussions with SRW, indicated difficulties associated with the hydraulic performance of a longer siphon.

Option 3 reduced flooding depths and extents in the Tinamba, north of the Tinamba Seaton Road. As discussed, some flooding still occurred in Tinamba due to the outflanking of the proposed levee. To achieve a similar reduction in flooding, as Option 1 and 2, the Option 3 levee needs to be extended to south of Tinamba Seaton road, and road levels along the Tinamba Newry (north of the roundabout) to be raised.

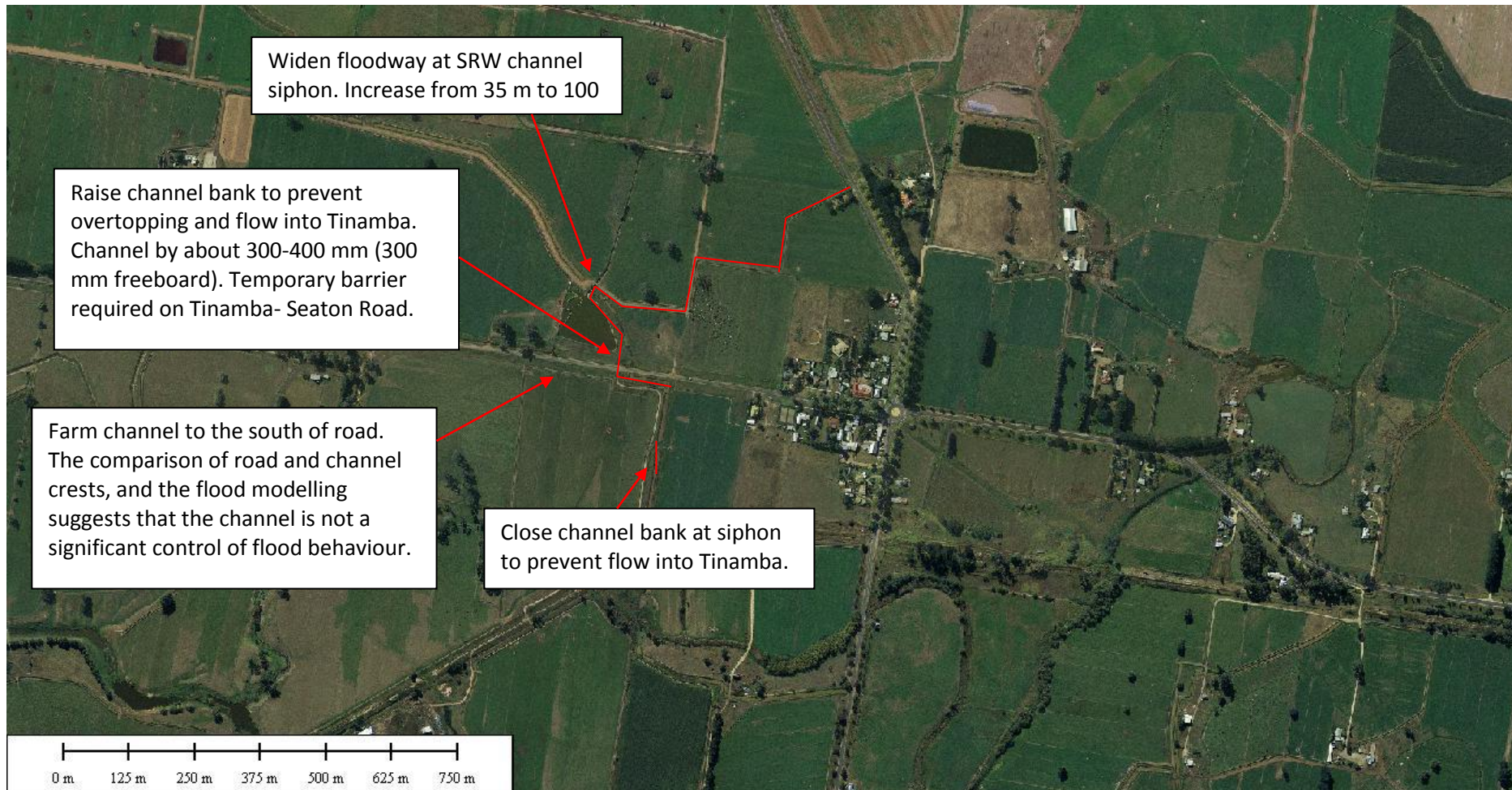


Figure 8-5 Tinamba Option 2 – General arrangements

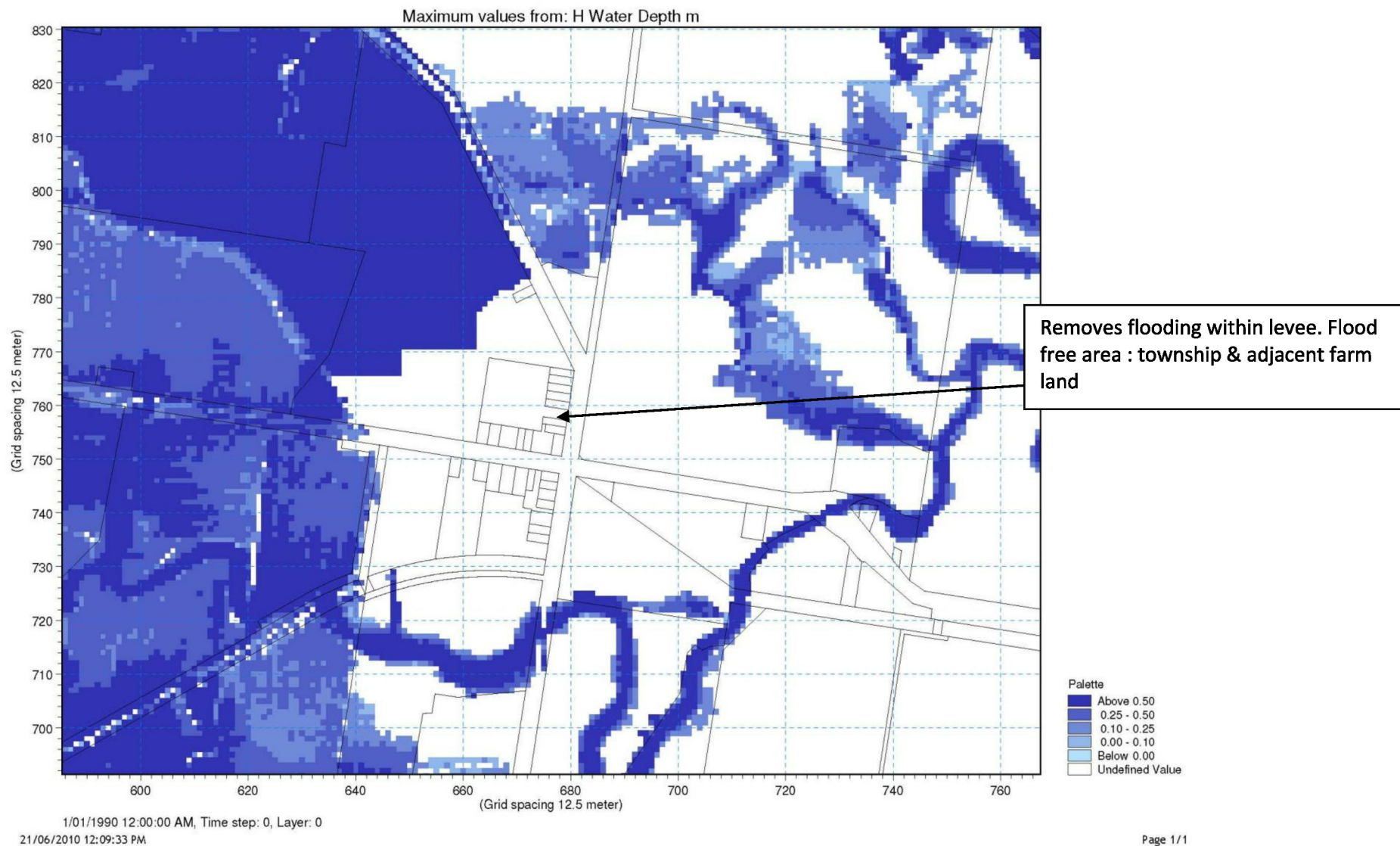


Figure 8-6 Tinamba Option 2 – 100 year flood depths

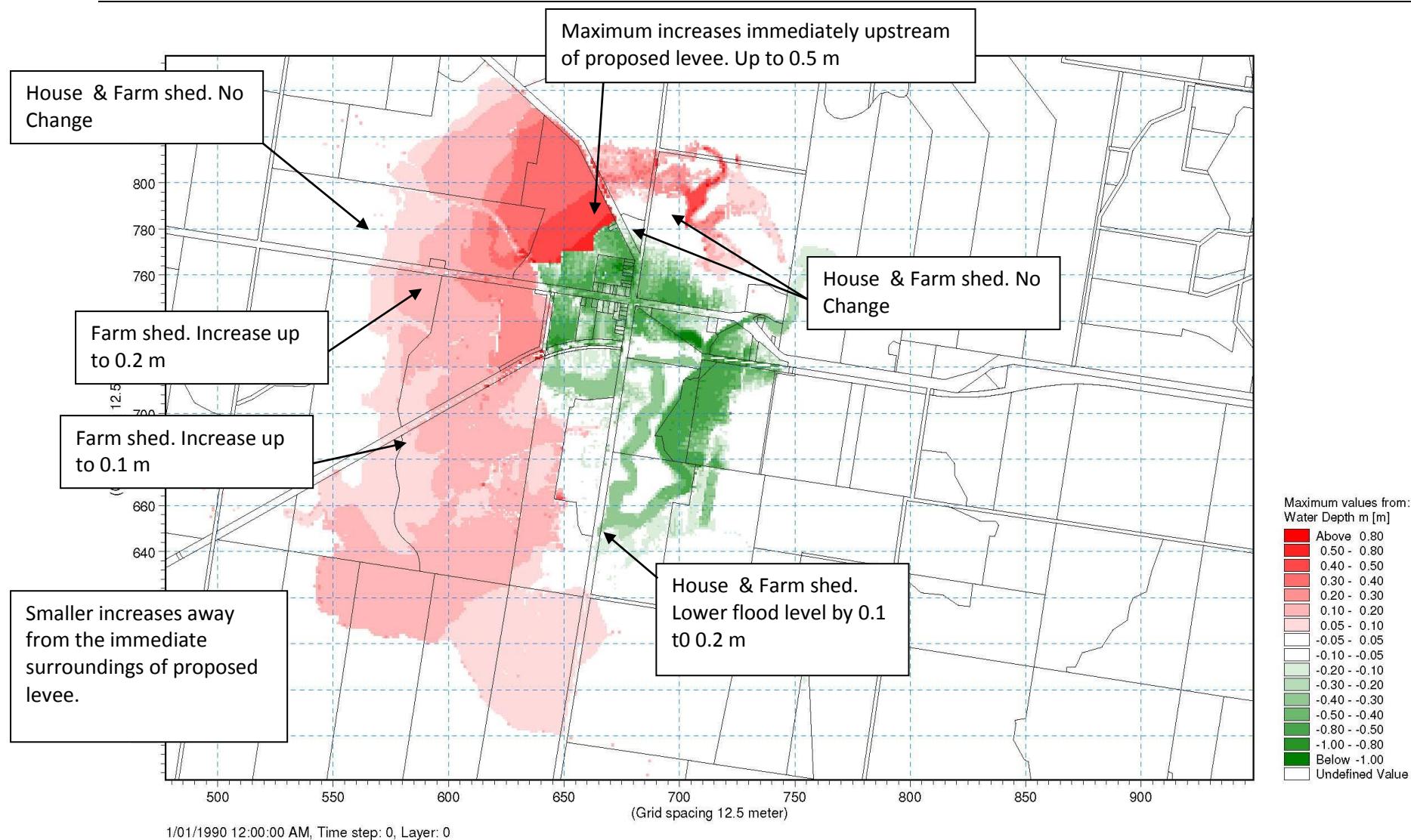


Figure 8-7 Tinamba Option 2 – 100 year flood level differences

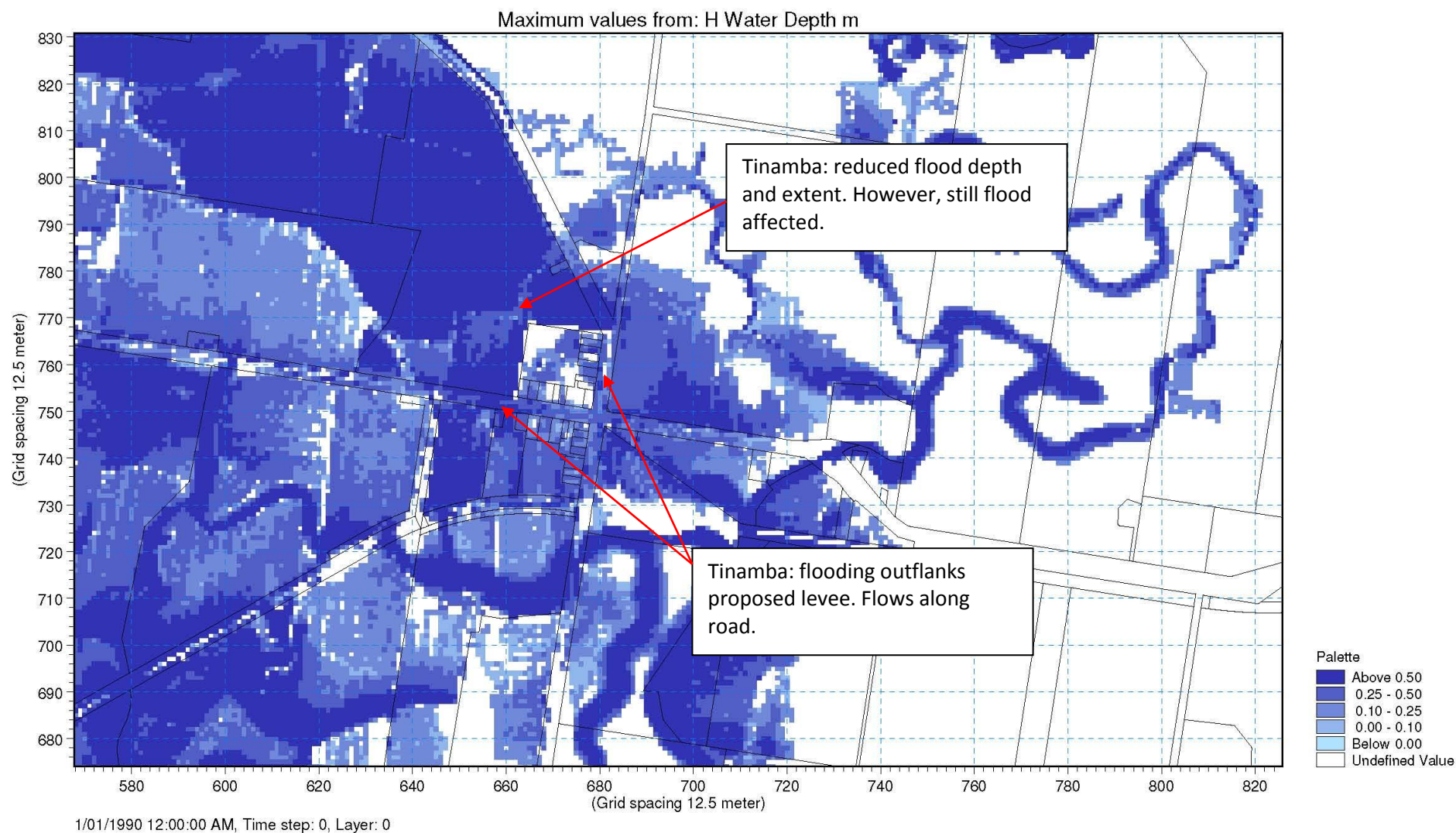


Figure 8-8 Tinamba Option 3 – 100 year flood depths

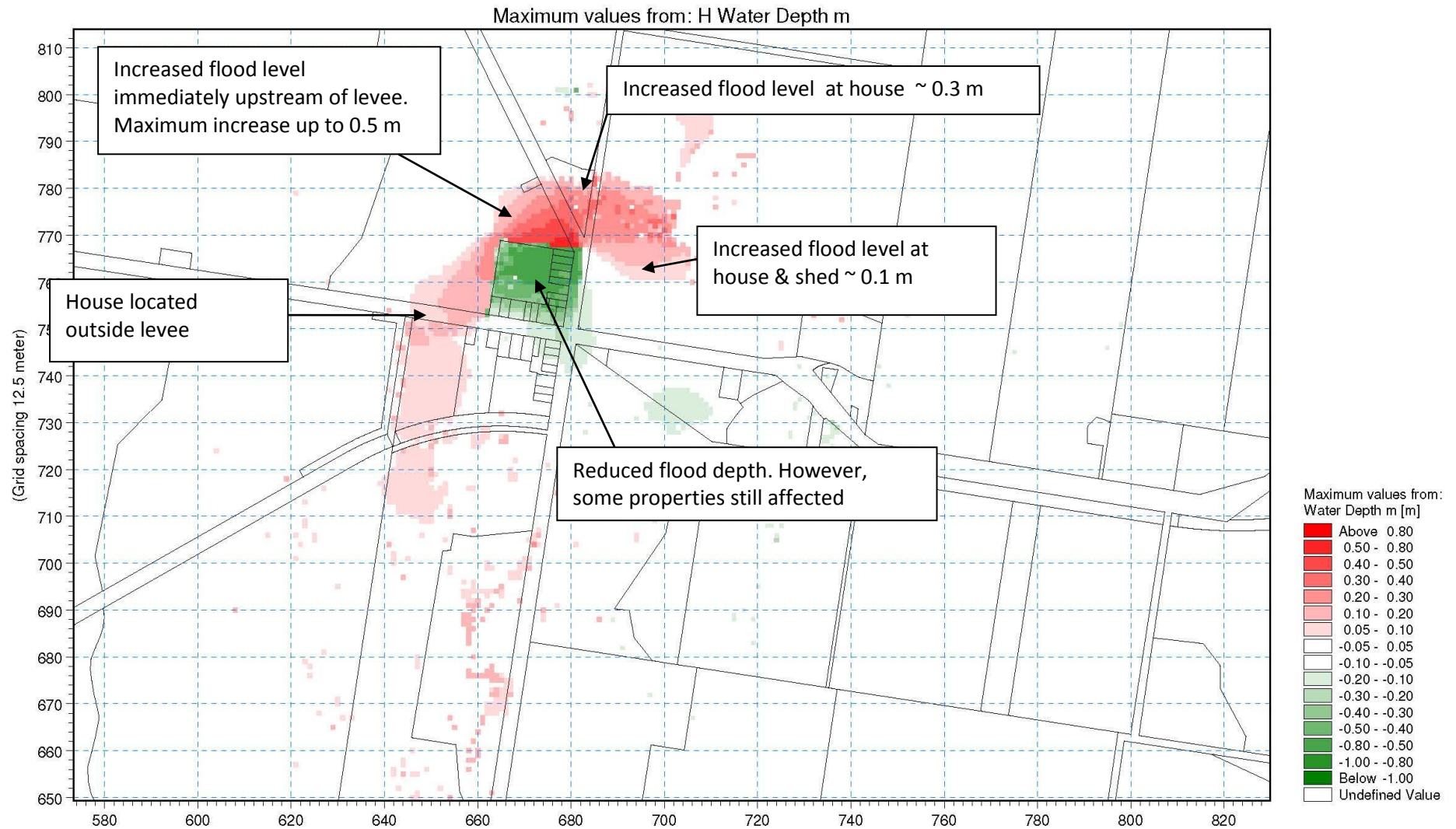


Figure 8-9 Tinamba Option 3 – 100 year flood level differences

8.3 Non-Structural Mitigation Measures

Non-structural measures are floodplain management activities aimed at reducing future flood damages. Non-structural measures aim to reduce existing flood risk by lowering flood damages (consequences) at a given location (as opposed to structural measures which tend to reduce frequency or likelihood of flooding). Non-structural measures include:

- Catchment management
- Flood awareness, preparedness, warning and response
- Land use planning

Catchment management activities in the upstream catchment can influence the existing catchment runoff characteristics (flood peaks and volumes). Flood volumes and peaks are a function of the vegetation cover and land use within a catchment (in addition to topography). Land clearing and drainage works have significantly altered flood response in the catchment. Further drainage works may increase flood peaks and flood volumes resulting from significant rainfall events. Increases in peak flows and flood volumes in turn result in a higher flooding likelihood and flood risk. Catchment revegetation, over the longer term may reduce flood volumes. However, in major floods reductions in peak flow would be expected to be minimal.

In the Macalister catchment the operation of Lake Glenmaggie will also influence flood characteristics.

Flood awareness, preparedness, warning and response aims to reduce the growth in future flood damages by improving community awareness of flooding and emergency services response. Flood awareness within a community reflects the frequency of significant flooding i.e. infrequent insignificant flooding leads to lower community flood awareness. The most recent significant flooding events occurred in the mid 2007. Given relatively recent occurrence of significant flooding with associated damages to property, the community awareness of flooding in communities on the Macalister Floodplain (e.g. Newry and Tinamba) is expected to be medium.

Further discussion of flood warning and response arrangements, and community flood awareness is provided in Section 9.

Land use planning aims to reduce flood damages by providing appropriate guidelines/controls for land use and development. The Victoria Planning Provisions (VPPs) allow for zoning of land and the application of controls on the type of land use and permitted activities in areas prone to flooding. The VPPs provide for the following zone and two overlays:

- Land Subject to Inundation Overlay (LSIO)
- Floodway Overlay (FO)
- Urban Floodway Zones (UFZ)

The VPPs provide guidelines for the appropriate uses and/or development of land in LSIO, UFZ and FO areas. A more detailed discussion of land-use controls is provided in the Volume 6 Datasets and Mapping Report.

9. FLOOD WARNING

9.1 The Total Flood Warning System

In 1995 the Australian Emergency Management Institute, following a national review of flood warning practices after disastrous flooding in the eastern states in 1990, published a best-practice manual entitled 'Flood Warning: an Australian Guide' (AEMI, 1995). In describing practices for the design, implementation and operation of flood warning systems in Australia, the manual introduced the concept of the 'total flood warning system' (TFWS). It also re-focused attention on flood warning as an effective and credible flood mitigation measure but made it clear that successful system implementation required the development of some elements that hitherto had been given little attention as well as the striking of an appropriate balance between each of the elements. In particular, it was noted that more attention needed to be given to risk communication and the education of communities about the flood risk, the measures which people could take to alleviate the problems that flooding causes and the place of warnings in triggering appropriate actions and behaviours.

While the 1995 manual has been revised and is now known more generally as Manual 21 of the Australian Emergency Manual Series (EMA, 2009), the concepts and terminology introduced in 1995 remain valid. There is evidence that the manual has had considerable influence on flood warning system development in various parts of Australia, and indeed in a number of other countries with significant flood problems, including the United Kingdom, the Netherlands and Bangladesh.

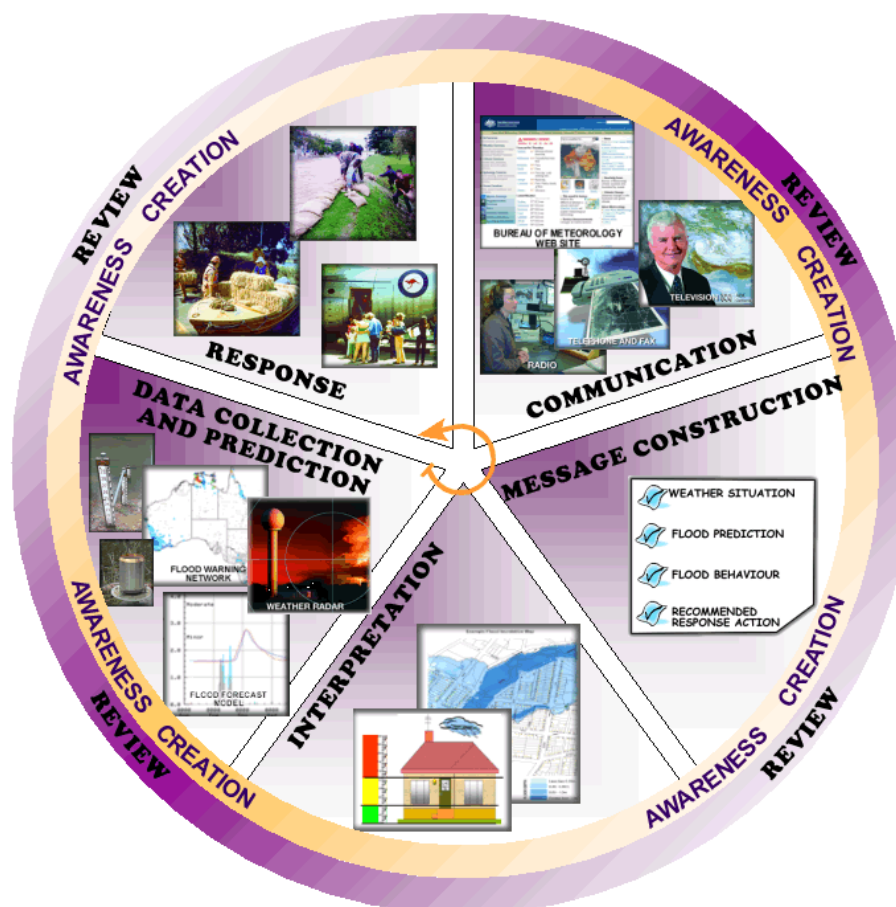


Figure 9-1 The Total Flood Warning System (derived from EMA, 2009)

The TFWS concept is shown in diagrammatic form in Figure 9-1. It represents the many interests that must be brought together to create an effective flood warning system: no single agency is likely to be able to provide all the skills required to deliver an effective flood warning service. It therefore makes clear the need for several agencies to play a part, with clearly-defined roles and with the various elements carefully integrated, and for the members of flood liable communities to be involved. Put another way, “effective warning systems rely on the close cooperation and coordination of a range of agencies, organisations and the community” (DoTARS, 2002).

The philosophy that underlies the TFWS concept coupled with the need for a coherent set of linked operational responsibilities and overlapping functions are documented and discussed in the context of guiding principles for effective early warning in IDNDR (1997).

9.2 Total Flood Warning System Building Blocks

As discussed, a flood warning system comprises much more than a data collection network, forecasting model and flood level (or flow) prediction.

An effective flood warning system requires all system components to be in place and for stakeholders, including members of the flood liable communities, to be working together.

An effective flood warning system is made up of eight building blocks. Each building block represents an element of the TFWS. The blocks (derived from EMA, 2009) along with the basic tools to facilitate delivery against each of the TFWS elements are represented in the Table 9-1.

9.3 TFWS for Macalister River Downstream from Lake Glenmaggie

A flood warning system has been operating for the Macalister River downstream from Lake Glenmaggie since at least the early 1970’s¹. In its infancy, the system consisted of advice from the then State Rivers and Water Supply Commission (SRWSC) to volunteer community nominated flood wardens in different parts of the Macalister Irrigation District, of outflows from Lake Glenmaggie once outflows were expected and again on (expected) exceedance of threshold flow rates. The wardens passed this advice on to landholders downstream of the Lake via a telephone tree.

While the system has undergone some strengthening over the years, the volunteer community based flood warden system remains a critical element or backbone.

A review of the role of VicSES in flood warning following the June 2007 flooding in Gippsland (VicSES, 2007) included documentation and discussion of flood warning, response and related activities across the region during that event (see also Molino Stewart, 2008)². While informative from an historical perspective, neither report (VicSES, 2007 and Molino Stewart, 2008) was aimed specifically at documenting or reviewing the flood warning system for downstream from Lake Glenmaggie although the role of the flood wardens was covered in both reports. In addition, the VicSES report included notes from meetings with flood warning system stakeholders that also refer to the flood wardens but that demonstrate some differences in understanding of their role and relationship with other system stakeholders.

¹ The flood warden system was established for the community, by the community after the 1971 floods and focuses on farmers and equipment in low lying areas. In the 1971 event Tinamba flooded before Newry. In June 2007, Newry flooded before Tinamba. The outflow from Lake Glenmaggie peaked at 148,000ML/d in June 2007, well in excess of the 1971 peak of 110,000ML/d.

² The report presents a chronology of the event and warnings issued together with a discussion of what each of the flood warning system stakeholder agencies did during the event. It also considers the flood awareness of residents before, during and after the flood, identifies information dissemination tools and reviews how the warnings and information flow influenced resident’s preparedness, decision making and behaviour. The methodology included agency interviews, media monitoring and community surveys. A raft of recommendations was delivered by the report.

It is understood that the Emergency Services Commissioner also undertook a review of the June 2007 flooding in Gippsland. The focus of that review was on examining the performance of the emergency management arrangements in terms of preparedness for, response to and recovery from the flood and at capturing and understanding learnings. It also aimed to look at the adequacy of warning systems and arrangements as well as organisational roles and responsibilities of emergency services under the draft State Flood Plan with attention on the (then) current flood policy documentation and municipal emergency management plans for the areas affected. The authors of this report have not seen a copy of the OESC review report³.

A summary of the existing flood warning system for the Macalister River downstream from Lake Glenmaggie is provided in Table 9-1 below, against each of the eight TFWS building blocks. The table was populated following discussions with key stakeholders over an extended period of some fourteen months.

³ The Terms of Reference for the review indicated that it would be completed by the end of November 2007. While the review has been completed and the report tabled in Parliament, at the time of writing this report, OESC were in the process of editing the report in order to place it into an appropriate contemporary context prior to public release. The editing is required as the review report makes a number of recommendations, many of which have been actioned. With changes, many of the matters raised in the review report no longer attract the same concern.

Table 9-1 Flood Warning System Building Blocks and current arrangements for the Macalister River downstream from Lake Glenmaggie
[Based on the EMMV, Commonwealth-State arrangements for flood warning service provision (Bureau of Meteorology, 1987), VFWCC (2001) and EMA (2009)]

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
DATA COLLECTION & COLLATION	BoM provides real time data for flood warning from the national rain gauge network and provides technical assistance for improved data collection networks to support flood warning systems. MW provides real time river and additional rain data for flood warning for the Port Phillip and Westernport region. River and other rain data availability assured through the DSE-managed Regional Surface Water Monitoring Partnerships (involve BoM, DSE, RWAs, CMAs, LG, etc). LG (as the prime beneficiary) has O&M funding responsibilities for upgraded flood warning networks if gauges have been installed primarily for flood warning purposes (VFWCC, 2001).	Data collection network (eg. rain & stream gauges, weather radar, satellite images).	The existing data collection network (rain and river gauges) was augmented following the June 2007 flood using capital funds provided by the State Government. The majority of on-going costs associated with these new sites are being picked up by SRW and Wellington Shire with the balance being funded by BoM. Not all flood producing storms are captured by the data collection network as there are a variety of flood producing mechanisms. Further, Lake Glenmaggie is small in volume compared with the size of the contributing catchment: this means it can fill quickly.
		System to convey data from field to forecast centre (eg. radio or phone telemetry).	A number of sites are radio telemetry while others are telephone based. SRW and BoM have established a mutual data access agreement to facilitate the sharing of data. However, a number of technical issues remain to be resolved to enable the intent of the agreement to be fully implemented.
		Data management system to check, correct, store, display data.	Data from all available rain and river sites within and adjacent to the Macalister catchment are available through the BoM website and are also available to SRW at Lake Glenmaggie.
		Information on water storage levels, inflows and operations.	SRW provides verbal (telephone) advice to: • BoM on storage level for Lake Glenmaggie as required as well as calculated inflows and Outflows beginning at minor flood flow (7,500ML/d); • VicSES (via established procedures through ESTA) at the same trigger levels and flows;

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
			• Volunteer community flood wardens when inflows are likely to lead to a gate opening and at predefined flows beginning at 3,500ML/d. Last advice is provided at major flood flow (35,000ML/d); and • VicPol beginning at an outflow of 60,000ML/d.
		Arrangements and facilities for system / equipment maintenance and calibration. For example, the Regional Surface Water Monitoring Partnerships, data warehousing, etc.	On-going costs associated with the rain and river data collection network that supports the flood warning service for the Macalister River are funded primarily by SRW, Wellington Shire and BoM through the Regional Surface Water Monitoring Partnership.
FLOOD DETECTION & PREDICTION (ie. Forecasting)	BoM prepares flood forecasts for rural areas and provincial centres. MW prepares flood forecasts for the main streams in the Port Phillip and Westernport region.	Information on critical levels / effects at key and other locations.	SRW's Eastern Storages Flood Plan includes information on critical flows in relation to the management of irrigation channels and related outfall structures and the flows at which Tinamba, Newry and main roads in the area are likely to be threatened by flooding. Some of the latter information is also provided in the Macalister catchment FloodSafe brochure produced by VicSES during 2009. SRW SitReps are forwarded to the ICC during a flood event.
	BoM provides predictions of weather conditions likely to lead to flash flooding for the whole State. LG is primarily responsible for flash flood forecasting but likely to be assisted by MW in the Port Phillip and Westernport region.	Appropriately representative flood class levels at key locations.	Representative flood class levels and flows have been established at the Lake Glenmaggie tail gauge for some considerable time as follows: minor 7,500ML/d; moderate 22,000ML/d; major 35,000ML/d. There are no flood warning gauges between the tail gauge at Lake Glenmaggie and the gauge at Bundalaguah near the confluence of the Macalister and Thomson Rivers. There are no flood forecast locations on the Macalister River downstream from Lake Glenmaggie. The next downstream flood warning gauge, complete with flood class levels, is located on the

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
			Thomson River at the Sale Wharf.
		Flood forecast techniques (ie. hydrologic and rainfall-runoff models, stream flow and height correlations, simple nomograms based on rainfall). URBS models developed for most of the larger Victorian catchments.	BoM have fitted an URBS model to the whole Macalister catchment. When a significant rainfall event is forecast, this model is used to predict the possible range and scope of flooding from a suite of likely rainfall depths assumed to be spatially and temporally uniformly distributed over the catchment. Model results are shared with the SRW, VicSES and other agencies to assist pre-flood planning and are indicative only. All flood warnings issued by BoM for the Macalister River downstream from Lake Glenmaggie are based on advice from SRW of outflows from Lake Glenmaggie. BoM does not forecast inflows to Lake Glenmaggie and, because it is a gated structure, does not forecast outflows.
		Meteorological analyses and data.	In order to enable safe operation of the gates on the Lake Glenmaggie spillway, SRW calculate inflows to Lake Glenmaggie. This calculation is based on change in storage level as per standard storage flood routing techniques. SRW also use the upstream rain and river gauges to gain an appreciation of the likely size of the expected inflow. However, not all flood producing storms are captured by the existing data collection network. SRW provides verbal (telephone) advice to BoM of Lake Glenmaggie storage level as required as well as calculated inflows and outflows beginning at minor flood flow (7,500ML/d). Similar advice is provided to VicSES. SRW also provides verbal (telephone) advice to volunteer community flood wardens when inflows are likely to lead to a gate opening and at predefined flows beginning at 3,500ML/d. Last advice is provided at major flood flow (35,000ML/d). Trigger levels and contact details are contained in SRW's Eastern Storages Flood Plan. BoM inputs Lake Glenmaggie outflows into the URBS (hydrologic) model for downstream of the Lake which together with Thomson River flows

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
			enables a forecast to be determined for Sale Wharf.
MESSAGE CONSTRUCTION	Warning messages are prepared by: ➢ BoM for weather conditions likely to lead to flash flooding for the whole State; ➢ BoM for flooding in rural areas and provincial centres; ➢ MW for flooding in the Port Phillip and Westernport region but disseminated through BoM system; ➢ LG for flash flooding in municipal areas. Opportunity exists for enhancement of messages by VicSES / LG through inclusion of local impacts and related information.	Warning messages / products and message dissemination system.	Flood watches are issued by BoM to notify communities and other stakeholders within broad areas (eg. West Gippsland rather than the Macalister catchment) of the potential flood threat from a developing weather situation. They provide a 'heads up' of likely flooding. Flood watches are based on an assessment of the developing weather situation and indicators of current catchment wetness. They provide generalised statements about expected forecast rainfall totals, the current state of the catchments within the target area and indicate the streams at risk from flooding. Instructions for obtaining rain and stream level observations and access to updated Watches and Warnings are also included. Normally, the BoM would issue a Flood Watch 24 to 36 hours in advance of any likely flooding and issue updates as required. If at any time during that period there was an imminent threat of floods occurring, the Flood Watch would be upgraded to a Flood Warning. Flood Warnings for the Macalister River downstream from Lake Glenmaggie are firm predictions of flooding based on actual outflows from Lake Glenmaggie and with regard for Lake levels as well as rainfall and river height information from upstream. Flood warnings are categorised as 'minor', 'moderate' or 'major' (see BoM website for an explanation of these terms and current flood class levels) and indicate the expected severity of the flood downstream from the Lake. Flood warnings also include where available: • Rainfall amounts for selected locations within and adjacent to the catchment; • River heights and trends (rising, steady, falling) at river gauge

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
			locations within the catchment; • Outflows (in ML/d) from Lake Glenmaggie; • A weather outlook and the likely impact of expected rainfall on flooding; and • A warning re-issue date and time. Note: The term “significant rises” may be used in the early stages of an event when it is clear that river levels will rise but it is too early to say whether flood will be reached. Flood warning lead times are significantly affected by notice of changes in outflows from Lake Glenmaggie. During a large flood, outflows usually increase quickly with the result that the warning may be upgraded a number of times in only a few hours. This means that Newry and Tinamba may receive less than 2 to 4 hours warning lead time respectively.
MESSAGE DISSEMINATION (ie. flood alerting and notification: communicating the warning message and information)	BoM to VicSES, LG, VicPol, CMAs and media. VicSES alerts relevant agencies and organisations when BoM issues flood warning(s) and may enhance flood warning(s) by issuing community safety information and action statements. VicSES may refer	Formal media channels ⁴ – TV, radio and print.	SRW provides verbal (telephone) advice to the volunteer community flood wardens when inflows are likely to lead to a gate opening and at predefined flows beginning at 3,500ML/d. Advice is repeated at increasing flows including at minor, moderate and major flood levels up to 35,000ML/d (major flood). The early advice includes suggestions that pumps and stock should be removed from near the river. Flood wardens relay information received from SRW to local landholders via telephone tree.
		Internet (eg. email, BoM website, VicSES website, social media).	
		Tape message services (eg. VicSES’ Flood and Storm Information Line for key messages and action	

⁴ ABC Radio has entered into a formal agreement with the Victorian Government and the Bureau of Meteorology to broadcast, in full, weather related warnings including those for flood. The agreement provides for the interruption of normal programming at any time to allow the broadcast of warning messages. This agreement will ensure that flood (and other) warnings issued by the Bureau are broadcast in their entirety and as soon as possible after they are received in the ABC’s studio.

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
	relevant agencies and organisations to BoM website or to VicSES website and / or Flood and Storm Information Line (when activated) for key messages and action statements. VicSES is not required to disseminate flood watches or warnings. LG disseminate information further. Not clear that messages are disseminated sufficiently to at-risk communities. BoM provides ALERT system co-operators with ENVIROMON software to collate and display data and initiate flood alerts that are based on exceedance of criteria such as rainfall volumes or rates and / or river levels or rates of rise.	statements). Other channels - fax / faxstream, phone / pager (eg. SMS such as offered by StreetData, voice, local communication ‘trees’), voice messaging systems (eg. Xpedite is in use for Maribyrnong, Shepparton-Mooroopna, Euroa, Benalla, Traralgon and Moolap in Geelong and being considered for other communities, the national Emergency Alert), community radio (eg. FM-88). Doorknocking. Other opportunities for at-risk communities to confirm warning details.	Flood wardens and the telephone tree were established for and are maintained by the Macalister Irrigation District community. It is understood however that the flood wardens are covered through VicSES insurance in a similar manner to SES volunteers. Flood Watches and Warnings (the latter including advice from SRW on outflows from Lake Glenmaggie) are issued by BoM via a mix of fax and email to VicSES, VicPol, the media (radio and TV - ABC and local), and a range of other stakeholder organisations including Wellington Shire Council, West Gippsland CMA, Department of Transport, DSE, SRW, Thiess Services, Red Cross, VicRoads and Telstra. While the BoM’s warning dissemination system monitors the delivery of faxes (a delivery failure trips an alert which results in a follow up telephone call from a BoM staff member), it does not monitor the success or otherwise of email delivery. VicSES provide follow-up advice to Councils when the BoM issues a Flood Watch or Warning and distribute additional information on flood impacts to local media. During larger floods, VicSES, in conjunction with SRW, host local community (town hall) meetings aimed at reinforcing flood warning messages, providing additional information and better targeting assistance. VicSES initiate the Emergency Alert during severe flooding and when evacuations are required. SRW brief the media about Lake Glenmaggie and outflows. Flood Bulletins are issued by VicSES to the media and other stakeholders.
INTERPRETATION (ie. what does	LG and community but is spread across LG, VicSES and	Interpretative tools (ie. flood inundation maps	SRW’s Eastern Storages Flood Plan includes information on critical flows in relation to the management of irrigation channels and related outfall

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
the forecast height mean for me or you)	CMAs, none of whom consider it core business, although VicSES adopted a lead role in recent 2010/11 events and employed specialist expertise to assist in this regard. Opportunity for MW and CMAs to assist LG / VicSES through provision of flood related expertise and experience re impacts, etc – both during planning for and responding to flood.	from experience, studies, VFD and related databases; flood information cards; flood histories; local knowledge; flood emergency plans that have tapped community knowledge and experience as well as flood related studies and other sources; etc).	structures including Maffra Weir and the flows at which Tinamba, Newry and main roads in the area are likely to be threatened by flooding. Some of the latter information is also provided in the Macalister catchment FloodSafe brochure produced by VicSES during 2009. WGCMA provide specialist expertise to the ICC / VicSES to assist with the identification of likely flood impacts (in terms of water depths, extents and assets at risk) and necessary response actions. Becomes difficult for the CMA to provide resources to fulfil this role during a large event because of the need to meet statutory obligations with respect to flood data collection and collation and related matters.
RESPONSE	VicSES are the Control Agency for flood response. Strong involvement from LG, VicPol and community. Should be driven by flood emergency plans (MEMP Sub-Plans) that include local flood intelligence gained from experience and extracted from flood study deliverables. Should also be driven by personal and business flood response plans. Ideally Flood Emergency Plans would be “owned” and maintained by LG but would be	Flood management tools (eg. MEMP Flood Emergency Plans complete with inundation maps and past ‘intelligence’, effective public dissemination of flood information, local flood awareness, individual and business flood action plans, etc).	Flood intelligence on which to base response activities is currently sparse as there are no flood inundation extent maps available (other than contained in the VFD datasets: some historic spot flood levels within the floodplain and the approximate extent of the estimated 1% AEP flood) and the Wellington Shire MEMP Flood Emergency Plan is not functional. Information on the consequence / impact of past floods (the last big flood occurred in 1971) is almost non-existent. Due to the time since the last big flood, staff in stakeholder agencies have little or no experience of flooding through the Macalister Irrigation District and downstream. While farmers are more aware of likely flood impacts on their own properties due to community “memory”, many residents in Newry and Tinamba had not lived through a large flood before June 2007. Road closures by Wellington Shire Council, VicRoads and / or VicPol as required.

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
	a joint VicSES / LG document that captures CMA input.		ICC and MECC established as required with VicPol fulfilling the coordination role.
		Standard operating procedures.	SRW initiate a number of activities to, closing regulators and outfalls to prevent flow along irrigation channel system, open the Tinamba Northern Channel with a backhoe,
		Community flood education and flood awareness raising, flood response guidelines and related information – all those tools that together work to build flood resilient communities (see the Awareness building block below).	Community Emergency Plans were produced for Newry & District and Tinamba & District in November 2009 as community projects, with input from Wellington Shire and VicSES, as a direct consequence of the June 2007 flood.
		Personal and business flood action plans (see EMA website, VicSES tool kit, etc).	The Newry & District and Tinamba & District Community Emergency Plans produced in November 2009 provide guidance on the preparation of personal flood plans along with a template. It is not clear how many residents on the floodplain downstream from Lake Glenmaggie have prepared such plans.
		Comprehensive use of available experience, knowledge and information.	WGCMA provide specialist expertise to the ICC / VicSES to assist with the identification of likely flood impacts and necessary response actions. SRW also provide input and advice if and as required or able.
REVIEW	All stakeholder entities including the VFWCC and communities potentially have	Post-event debriefs (agency, community), etc	VicSES facilitate community debriefs after severe flood events. Stakeholder agencies also engage in an incident and / or individual debriefs.

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
	opportunity to provide review comments. LG, MW, CMAs and VicSES have a role in collecting post-flood data (hydrologic, flood extent, impacts, damages, etc).	Review and update of personal, business and other flood action plans. Collection of flood 'intelligence' and flood damage data during and after the event (eg. rapid impact assessments, CMA, DPI and LG activities, etc).	The WGCMA undertake a comprehensive data collection and collation exercise during and immediately after significant flood events (aerial photography, flood extents, peak flood levels/depths, impacts) consistent with their Statement of Obligations. This data is available to be shared with other stakeholder agencies (eg. to update flood intelligence records and related materials). VicSES undertake rapid impact assessments during a flood event and share key information through regular SitReps. DPI undertakes damage assessments after significant flood events. Generally, data collected during flood events is available to be shared with other stakeholder agencies.
AWARENESS	VicSES has adopted a lead role with the roll-out of the FloodSafe program. Involvement from LG, MW and CMAs (and RWAs in some instances). Evidence that VicSES is taking a lead role.	Identification of vulnerable communities and properties (ie. flood inundation maps, information on flood levels / depths and extents, property-specific flood depths, etc). Activities and tools (eg. participative community flood education, flood awareness raising, flood risk communication) that aim to build flood resilient	The VFD datasets provide some historic spot flood levels within the floodplain and the approximate extent of the estimated 1% AEP flood. SRW's Eastern Storages Flood Plan includes flood intelligence that would be useful to VicSES and Wellington Shire. Some of the information has been captured to the Community Emergency Plans prepared by the Newry & District and Tinamba & District communities in 2009 and also to the FloodSafe brochure for the Macalister catchment. As a direct consequence of the June 2007 flood and with input from Wellington Shire and VicSES, Community Emergency Plans were produced for Newry & District and Tinamba & District in November 2009 as community projects. The plans contain limited information on the consequences and characteristics of flooding from the Macalister River but do promote the preparation of and provide guidance on the

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
		communities (ie. communities that can anticipate, prepare for, respond to and recover quickly from floods while also learning from and improving after flood events).	preparation of personal flood plans along with a template. It is not clear: • Where responsibility for maintaining and / or reproducing the Community Emergency Plans resides; • How many residents on the floodplain downstream from Lake Glenmaggie have prepared personal flood plans.
		VicSES' FloodSafe and StormSafe (flash / stormwater flooding) programs.	VicSES produced a FloodSafe brochure for the Macalister catchment during 2009. SRW's website includes information on SRW's (and others') role in the flood warning system as well as on managing floods at Lake Glenmaggie.
		Local flood education plans – developed, implemented and evaluated locally (eg Cities of Maroondah, Whitehorse, Wodonga, Benalla and Greater Geelong).	
		Flood response guidelines, residents' kits, flood level information, flood inundation maps, flood markers, property-specific flood charts (eg. Glenorchy, Horsham, Dimboola,	

Building Blocks of a Flood Warning System	Entities Involved in Victoria	Basic Tools	Current arrangements for the Macalister River downstream from Lake Glenmaggie
		Warracknabeal), flood levels in meter boxes (eg. Benalla, Traralgon) and on rate notices, etc.	

9.4 Discussion

The West Gippsland Regional Floodplain Management Plan (SKM, 2000) identified the need to upgrade the Macalister flood warning system downstream from Lake Glenmaggie and assigned a medium priority. The WGCMA was identified as the lead agency for this activity with input required from BoM, NRE, VicSES, Wellington Shire and SRW. Development (and pressure for further development) has continued on the floodplain downstream from the Lake with the result that there is an increasing need to improve the warning system.

VicSES (2007) and Molino Stewart (2008) contain a number of recommendations relating to the TFWS that are relevant to the Macalister River downstream from Lake Glenmaggie. A number of these have been addressed to varying degrees in relation to the Macalister catchment as part of organisational and procedural changes implemented since the reports were produced and are reflected in Table 9-1.

It is apparent that the volunteer community flood wardens and the associated telephone trees are an essential element of the existing flood warning system but who does what, when and why is not universally understood. Furthermore, it is apparent that the TFWS is not fully functional in the study area and that current deficiencies generally arise from the lack of data and related flood intelligence. The absence of a fully functioning TFWS has significant ramifications for the ability of the community to understand the likely consequences and appropriate responses to flooding. A further consequence is reduced community confidence in warnings that are provided, with a consequential “do nothing” or “wait and see” approach to impending flood events. It is therefore equally apparent that the deliverables from this study offer the potential for significant improvements to the system.

The \$445,000 allocated for improving flood warning systems in the Avon, Macalister and Thomson Rivers and the Gippsland Lakes by the State Government from the \$60 million flood recovery package following the June 2007 flooding has been used to improve the data network coverage in the headwaters areas of the Macalister, Avon and Mitchell rivers. This work has been completed and on-going funding arrangements agreed under the auspices of the Central Gippsland Flood Warning Group which is chaired by the WGCMA and includes representation from flood warning system agency stakeholders.

While the WGCMA interprets the liaison with VicSES and LG on flooding and emergency management matters function within the Statement of Obligations as indicating an operational role supporting the ICC / VicSES during periods of flooding, during large floods it becomes difficult for the CMA to provide the resources to do this because of the need to meet other statutory obligations relating to the collection and collation of flood extents, peak levels, aerial photography and other flood data.

There are no forecast locations on the Macalister River downstream from Lake Glenmaggie other than the Glenmaggie tail gauge. The reason for this is that the floodplain is complex and flows (and thus levels) are influenced by a variety of breakouts, channels and other infrastructure. However, the communities of both Newry and Tinamba have a significant flood and isolation risk. The high quality and reliable flood inundation maps produced by this study provide excellent guidance on likely flood extents, depths, high risk areas and assets likely to be flooded across the floodplain and together with the flood markers proposed for both locations, provide the necessary support to enable a flood forecast for each location.

Consideration of the above review reveals four major issues:

- Additional warning lead time is required, particularly for the Newry and Tinamba communities;
- Inundation extent maps are required to facilitate an understanding within the at-risk community and within the stakeholder agencies, of likely flood impacts, particularly during a severe event;

- A flood emergency plan that details likely flood consequences and critical timings as well as the triggers for damage reducing actions needs to be developed; and
- Flood intelligence in relation to flood impacts and required actions needs to be shared with the at-risk community.

9.1 Recommendations relating to the TFWS

The authors of this report are acutely aware that the Comrie-led Victorian Floods Review will be delivering a comprehensive report in late 2011 following a wide ranging examination of the flooding that occurred from September 2010 to January 2011 in relation to the TFWS and existing legislation and policies. That report is expected to contain a number of substantial policy, procedural and technically orientated recommendations that will have application to all river basins within the State, including the Macalister. Consequently and in order to avoid any overlap or conflict with those recommendations, the following recommendations relate only to matters addressed directly by this study.

9.5.1 Flood Forecasting and Additional Lead Time

In order to make best use of the flood inundation maps and related intelligence delivered by this study and improve the lead time of flood forecasts and warnings provided for downstream from Lake Glenmaggie, it is recommended that after the Lake fills, initial forecasts aimed at predicting the exceedance of key levels (flows) as well as the peak level (flow) could be based on an hydrologic model of the catchment with an appropriate routing routine to accommodate the effects of a full and spilling Lake Glenmaggie. While it is acknowledged that the initial forecast peak is unlikely to be as accurate as current forecasting arrangements, it will provide a more timely estimate of the peak and an earlier indication of the likely scale of the event which in turn will provide additional lead time for communities to prepare for inundation. The forecast could be refined based on advice of outflows from SRW. Issues associated with uncertainty, accuracy and messaging would need to be addressed before revised arrangements could be implemented.

It is suggested that the hydrologic model and its likely forecasting performance (in terms of its ability to predict peak outflows and the additional warning lead time achieved) could be tested on the 1995 and 2007 flood events; and

There are no forecast locations on the Macalister River downstream from Lake Glenmaggie other than the Glenmaggie tail gauge. One of the reasons for this is that the floodplain contains a complex array of topographic features (irrigation infrastructure, road/rail embankments, breakouts, etc) that strongly influence flood behaviour and thus both flows and levels. However, the communities of both Newry and Tinamba have a significant flood and isolation risk. The high quality and reliable flood inundation maps produced by this study provide excellent guidance on likely flood extents, depths, high risk areas and assets likely to be flooded across the floodplain and together with the flood markers proposed for both locations, provide the necessary support to enable a flood forecast for each location. It is therefore recommended that flood forecasts and warnings should be provided for Newry and Tinamba using the relationships developed between outflows from Lake Glenmaggie and the proposed flood markers.

As an extension to the above, minor, moderate and major flood class levels should be determined for both Newry and Tinamba.

9.5.2 Interpretation and Response

The flood inundation maps delivered by this study (for peak flows of 7,500 ML/d, 20,000 ML/d, 30,000 ML/d and 40,000 ML/d as well as the 5, 10, 20, 50, 100 and 200 year ARI events and PMF event) together with the lists of essential services, properties affected below and above floor level

and roads affected by flooding also produced by this study enable impacted assets and areas to be identified before flood waters arrive. As a minimum their use during future floods will allow for a more structured emergency response: a move from a reactionary to a planned response

Further, the flood markers proposed for installation at Newry and Tinamba together with the correlations developed through this study between outflows from Lake Glenmaggie and local flood depths will also assist residents determine whether they are likely to be impacted by flooding without having to rely on advice from VicSES.

While the flood inundation maps, property and related lists and flood markers in Newry and Tinamba will assist identification of what assets are likely to be affected by flooding, the challenge will be to implement response actions ahead of access being compromised. In larger floods, this will require additional lead time on likely peak outflows from Lake Glenmaggie (ie. an increased forecast lead time – see Section 9.5.1 above).

It is recommended that the intelligence delivered as part of this study is refined, extended and captured to a Wellington Shire MEMP Flood Emergency Plan. The VicSES Flood Emergency Plan template should be used. Assistance from appropriately skilled consultants could be sought on this task.

It is further recommended that the Plan be shared with the at-risk community.

As an additional assistance to the Newry and Tinamba communities, it is recommended that the peak level and date of previous significant floods should be added to the flood markers proposed by this study. It is also recommended that responsibilities for the long term maintenance of the markers is agreed and documented prior to installation.

9.5.3 Community Awareness

It is recommended that the Community Emergency Plans produced for Newry & District and Tinamba & District in November 2009 and the FloodSafe brochure are all updated with the intelligence delivered as part of this study. The update should include information about the flood markers and how they can be used.

It is further recommended that responsibility for long term responsibility for maintaining and / or reproducing the Community Emergency Plans is resolved and that details are provided within the Plans.

10. SUMMARY AND RECOMMENDATIONS

10.1 Overview

The Macalister Flood Study provides a comprehensive analysis and review of existing and future potential flood risk across the Macalister floodplain downstream of Lake Glenmaggie. The study has involved:

- Collection and review of a range of data relevant to the investigation of flooding within the study area.
- A survey analysis to develop a detailed description of the study area topography as a basis for hydrologic/hydraulic analysis and mapping.
- A rigorous hydrologic analysis to develop robust design flood estimates for the study area.
- Development of a detailed hydraulic model that is capable of predicting flood impacts across the Macalister floodplain under a range of conditions.
- Quantification of flood risk in terms of flood damages.
- Thorough sensitivity testing of the hydraulic results under both existing conditions and for a range of potential climate change scenarios.
- A preliminary examination of potential flood mitigation options.

10.2 Key Outcomes

Key outcomes from the study are summarised as follows:

10.2.1 Macalister Catchment Hydrology

Revised design peak flows have been developed for the Macalister River downstream of Lake Glenmaggie, as detailed below.

Table 10-1 Design peak flow estimates for the Macalister River downstream of Lake Glenmaggie

ARI (years)	Design peak flow estimates Macalister River downstream of Lake Glenmaggie (ML/d)
5	52,900
10	64,800
20	90,800
50	115,900
100	133,400
200	151,900

10.2.2 Hydraulic Characteristics of the Macalister River

The complex array of topographic features (irrigation infrastructure, road/rail embankments) across the Macalister River floodplain downstream of Lake Glenmaggie, strongly influence the flood behaviour. Considerable effort has been made to compare observed flood levels and extents for a range of flood events and the calibrated hydraulic model is considered fit for purpose.

10.2.3 Flood Response and Land Use Planning Mapping

The hydrologic and hydraulic models described previously were used to simulate and map flood behaviour (extents, depth, velocities) for a range of flood magnitudes. The hydrologic analysis provided design event flood inflow hydrographs downstream of Lake Glenmaggie. These inflow hydrographs were routed through the calibrated hydraulic model. This enabled the modelling and mapping of flood depths, extents, velocities over a range of flood magnitudes.

To provide a suite of flood maps across the range of flood magnitudes of interest, flood mapping was undertaken for the peak flows of 7,500 ML/d, 20,000 ML/d, 30,000 ML/d and 40,000 ML/d, in addition to the 5,10,20,50,100 and 200 year ARI events plus the PMF event.

The mapping outputs can be applied for flood response planning, and land use planning purposes. All flood mapping data was prepared to the VFD metadata specifications.

10.2.4 Flood Damages

The results of the flood damages assessment are summarised in Table 7-4. The 1% AEP or 100 year ARI flood damage was calculated to be over \$3.5 million and the AAD was calculated to be approximately \$515,765 per year.

It should be noted that all damages calculations are based on current development, and flooding conditions and take no account of potential impacts of future climate change or change in development patterns.

It is important to recognise that the flood damages assessment developed here, whilst consistent with best current practice, has a number of uncertainties and could be expected to vary within a range of at least +/- 20%.

10.2.5 Flood Risk Mitigation

Mitigation measures provide a means to reduce the existing flood risk. Mitigation measures can reduce existing flood risk by lowering the likelihood of flooding and/or lowering the flood damages (consequences) for a given flood depth. Mitigation measures can be broken into:

- Structural: Physical barriers or works designed to prevent flooding up to a specific design flood standard. Structural measures aim to reduce existing flood risk flood by lowering flood likelihood at given locations. Structural works include levees, floodways waterway works, improvements to hydraulic structures.
- Non-structural: Management and planning arrangements between relevant authorities designed to reduce related flood damages. Non-structural measures aim to reduce existing flood risk flood by lowering flood damage. Non-structural measures include land use planning, flood warning, flood response and flood awareness.

Three structural mitigation measures were identified and a preliminary hydraulic assessment undertaken. The mitigation options focussed on the townships of Newry and Tinamba.

Non-structural mitigation measures are also discussed.

10.2.6 Flood Warning

The existing flood warning system for the Macalister River downstream from Lake Glenmaggie has been summarised against each of the eight Total Flood Warning System (TFWS) building blocks. The summary reflects current arrangements which include a number of organisational and procedural changes implemented as a result of recommendations contained in key reports arising from the 2007 floods in the region. A consideration of the summary revealed four major issues:

- Additional warning lead time is required, particularly for the Newry and Tinamba communities;
- Inundation extent maps are required to facilitate an understanding within the at-risk community and within the stakeholder agencies, of likely flood impacts, particularly during a severe event;
- A flood emergency plan that details likely flood consequences and critical timings as well as the triggers for damage reducing actions needs to be developed; and
- Flood intelligence in relation to flood impacts and required actions needs to be shared with the at-risk community.

Actions aimed at addressing these issues have been recommended. While the scope of recommendations could have been wider and remedies more comprehensive, the authors of this report are acutely aware that the Comrie-led Victorian Floods Review will be delivering a comprehensive report in late 2011 following a wide ranging examination of the flooding that occurred from September 2010 to January 2011 in relation to the TFWS and existing legislation and policies. That report is expected to contain a number of substantial policy, procedural and technically orientated recommendations that will have application to all river basins within the State, including the Macalister.

10.3 Recommendations

Following the investigations undertaken for the study and the conclusions reached it is recommended that:

- The WGCMA and Wellington Shire Council adopt the determined design flood levels and in turn proceed with a declaration process.
- The WGCMA and Wellington Shire Council continue to engage the community in the treatment of flood risks through the development of a full Floodplain Management Plan for the Macalister River floodplain that involves broad community involvement and consultation with stakeholders.
- The WGCMA and Wellington Shire Council explore options for enhanced flood response measures through co-operation with SES and Police utilising the flood inundation maps produced from the study.
- The WGCMA and Wellington Shire Council explore options for the development of a flood warning system for the township of Tinamba and Newry in conjunction with the BoM and SES.

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