Glenelg-Hopkins Catchment Management Authority

North Warrnambool Flood Study for Merri River and Russell Creek

Flood Study Report

Final Report

December 2003

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1 Introduction and Background

1.1 Study Area

Glenelg Hopkins Catchment Management Authority commissioned GHD to conduct a flood study for Russell Creek and part of the Merri River floodplain, within the City of Warrnambool. The study area includes Russell Creek between Aberline Road and the Merri River confluence, and the Merri River between Grange Road (the westerly extension of Wangoom Road) and Caramut Road.

This report describes the work undertaken for Russell Creek and the Merri River within the study area.

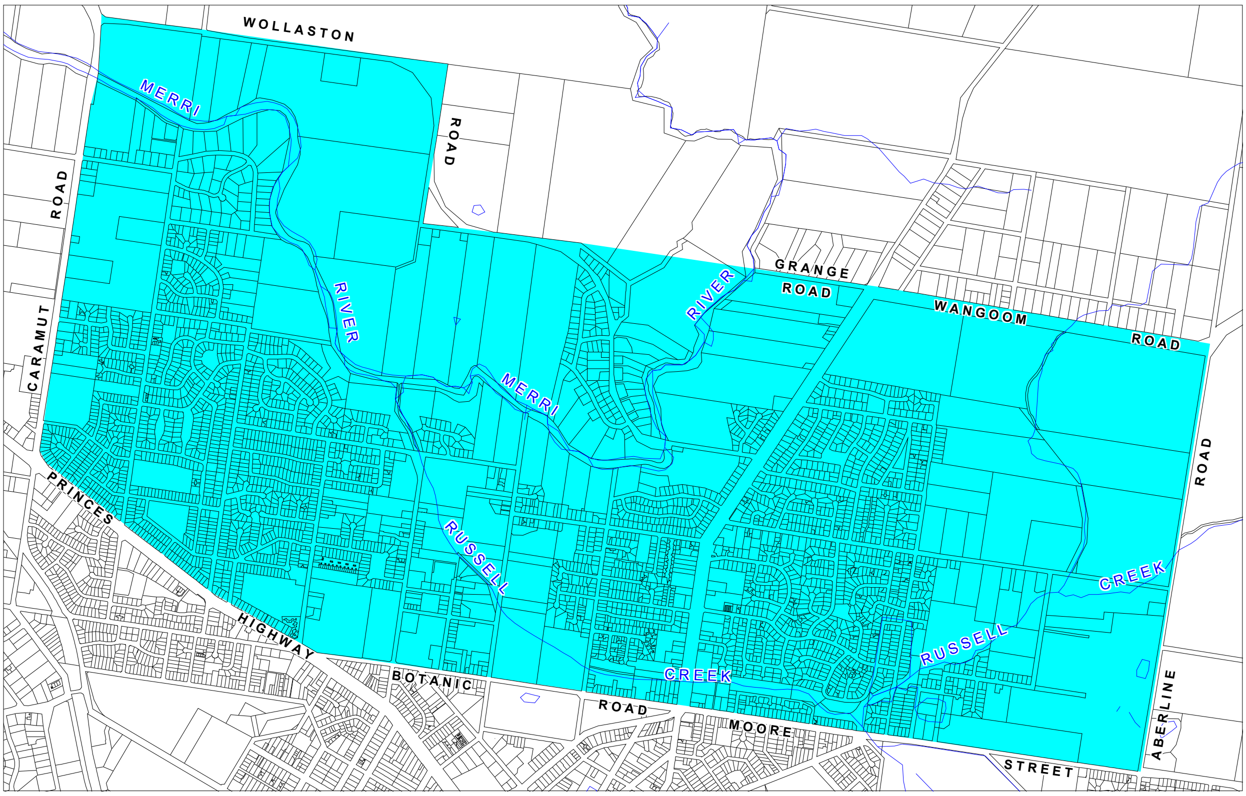

Figure 1.1 Study Area

1.2 Introduction to Hydrology

The hydrology part of the study has been done to estimate flood flows for a range of historical and theoretical flood events. Available data has been used to obtain estimates of the 5, 20, 50 and 100 year Average Recurrence Interval flows.

A preliminary Flood Frequency Analysis (FFA) had been undertaken by Thiess for the Merri River from 1966 - 2000. The preliminary FFA estimated flood flows for a range of design flood probabilities.

The purpose of the hydrology part of this study was to reconsider the flood estimates to include information from floods prior to 1966, including the 1946 flood event.

Anecdotal evidence suggests that the 1946 flood was an extremely large event. However, there was no streamflow data available for this event. Daily rainfall records are available at a number of locations in the region, going back to 1900 and including the 1946 flood.

GHD developed a daily rainfall runoff model to generate stream flows for the Merri River for the period 1900-2000. The model was able to provide an indication of the relative magnitude of the 1946 flood. The 100 years of data, together with 1946 flood estimates were incorporated into the FFA to provide a more comprehensive assessment of the 5, 10, 20, 50 and 100 year Average Recurrence Interval flows for the Merri River.

In the absence of any measured flows on Russell Creek, design flows were prepared using the RORB runoff routing model.

1.3 Introduction to hydraulics

Hydraulic modelling has been done to estimate flood levels for the 5, 10, 20, 50 and 100 year Average Recurrence Interval floods. The hydraulic models are based on aerial survey, from photographs taken in 2002 plus ground survey of bridges and culverts.

Recorded flood levels and flow estimates from the 1946 flood were used to calibrate the hydraulic models to produce reliable flood levels and flood extents. Maps of the flood extents have been produced.

1.4 Consultation

Consultation was undertaken with a range of individuals and organisations during the course of this project. Steering Committee meetings were held in Warrnambool and attended by the three key organisations (additional personnel attended some of the meetings):

- Glenelg Hopkins Catchment Management Authority (GHCMA), represented by Brad Henderson;
- Warrnambool City Council, represented by Peter Robertson; and
- GHD, represented by Andrew Prout and Andre Vanderputt.

Advertisements were placed in local papers to advise residents of the study and to seek information via a questionnaire. Questionnaires were returned via Council and a number of photographs were also forwarded.

At one of the Steering Committee meetings on 29 November 2002 we received extensive anecdotal descriptions from Mr Ken Boyle regarding flooding, particularly in the lower reaches of Russell Creek in the vicinity of Wentworth Street.

We also undertook site inspections on 29/11/2002 with Mr. J. Malik of Moore Street, Warrnambool who also provided a significant amount of information.

Ian Gauntlett from the Floodplain Management Unit of the Department of Natural Resources and Environment (DNRE) provided verbal advice regarding historic flooding and previous investigations.

We would like to thank all parties who participated in the study.

1.5 Definitions

The 100-year Average Recurrence Interval (ARI) flood event has a 1% chance of being equalled or exceeded in any year. Similarly, the 50, 20, 10 and 5 year average recurrence interval flood events have 2%, 5%, 10%, 20% chances respectively of being equalled or exceeded in any year.

Flood frequency analysis (FFA) is a method for determining the flow rate for various theoretical floods, eg a 100 year ARI flood. The method involves using the highest flow in each year, ranking the flows in order from largest to smallest and assigning an ARI or probability of exceedence to various flows using statistics.

2 Flood Frequency Analysis for Merri River

2.1 Results From Preliminary FFA

GHCMA had a preliminary flood frequency analysis done for the Merri River by Thiess. The analysis was based on recorded streamflow data at Woodford from 1966-2000. The peak design flows are shown in Table 2.1.

Table 2.1 Preliminary FFA

	Gumbel Distribution		**LP3 Distribution**	
Design Flood ARI (years)	Peak Design Flow (ML/day)	5% & 95% Confidence Limits	Peak Design Flow (ML/day)	5% & 95% Confidence Limits
2	7,200	5,600-8,800	6,500	5,100-8,300
5	13,000	10,000-15,900	12,000	9,100-16,300
10	16,800	12,700-20,900	16,100	12,000-23,500
25	21,600	16,100-27,200	21,900	15,600-34,700
50	25,200	18,500-31,900	26,500	18,300-44,500
100	28,800	20,900-36,600	31,300	21,000-55,400

Note: Results exclude low flow years defined as those years with peak flow less than 1,000 ML/day

2.2 Limitations in Preliminary FFA

The preliminary flood frequency analysis was based on 34 years of recorded data (1966 –2000). The length of record is sufficient to produce a reasonable flood estimate for the 2 year ARI to 20 year ARI design floods. For rarer floods there are two limitations in using the preliminary FFA:

- Confidence limits are quite wide and may be improved by a longer record of flood flows; and
- The 1946 flood event was not included in the preliminary FFA.

Anecdotal evidence suggests that the 1946 flood was an extremely large event. However, there is no streamflow data available for this event. The absence of the 1946 event from the flood frequency analysis may have resulted in a lower estimate of the 100-year ARI event.

3 Rainfall Frequency Analysis

3.1 Correlation between Streamflow and Rainfall

The simplest method of estimating streamflows in the Merri River would be to directly relate the daily rainfall records to streamflow.

An attempt was made to find a correlation between streamflow and rainfall. This simplified approach was abandoned due to its inability to produce any meaningful results. A more rigorous method would have to be applied to take into consideration important catchment parameters, such a storage, antecedent moisture conditions, base flow, surface flow and evaporation.

3.2 ARI of the 1946 & 1978 Rainfall Events

A rainfall frequency analysis was undertaken for the 1946 and 1978 storms as described below.

The largest known flood event for Warrnambool occurred in March 1946. The rainfall for this event occurred over a three-day period, from the 16th to the 18th. The total rainfall depths over the 72-hour period are shown in Table 3.2.

Table 3.2 Rainfall for 1946 Flood

Duration (h)	Hawkesdale (mm)	Woolsthorpe (mm)	Warrnambool (mm)
24	158.2	128.0	167.1
48	200.4	160.3	226.8
72	234.4	187.5	238.5

The second largest known flood event for the Merri River at Warrnambool occurred in August 1978. The rainfall for this event occurred over a three-day period, from the 7th to the 9th. The total rainfall depths over the 72-hour period are shown in Table 3.3.

Table 3.3 Rainfall for 1978 Flood

Duration (h)	Hawkesdale (mm)	Woolsthorpe (mm)	Warrnambool (mm)
24	39.8	29.5	20.0
48	51.2	35.0	23.2
72	78.8	63.8	58.8

A comparison of the rainfalls for 1946 and 1978 over a 72-hour duration shows that the rainfall at Hawkesdale and Woolsthorpe was approximately three times greater in 1946 compared with the rainfall in 1978. The rainfall at Warrnambool for 1946 was approximately four times the magnitude of the 1978 event.

Using the Intensity-Frequency-Duration table (see Appendix A) the Average Recurrence Interval (ARI) for the 1946 and 1978 rainfalls were estimated. Table 3.4 Warrnambool IFD contains a summary of the IFD table, converted to total depth in millimetres. A comparison of the rainfall during March 1946 and IFD for Warrnambool shows that the rainfall at Woolsthorpe was approximately equal to the 500-year ARI rainfall event. However, the rainfall in 1946 at Hawkesdale and Warrnambool was greater than the 500-year ARI rainfall event.

Table 3.4 Warrnambool IFD

Duration (h)	100 Year ARI (mm)	500 Year ARI (mm)
24	106.8	139.7
48	130.6	170.4
72	142.6	185.8

4 Water Balance Model for Merri River

There is only 36 years of gauged streamflow data for the Merri River at Woodford from 1966 to 2002. However, rainfall data exists within the catchment back to the late 1800's. Using this rainfall data the streamflow could be artificially extended. It was decided that a period starting from January 1900 would be sufficient to provide a more detailed flood frequency analysis. Thus providing approximately 100 years of synthesised streamflow, including estimates of the 1946 flood.

Extending the streamflow record was done using the Australian Water Balance Model (AWBM). This computer based modelling package is able to model daily runoff from a catchment by applying rainfall and evaporation to a series of three hypothetical storages covering the entire catchment. When a storage overflows, direct runoff is produced. Some of this direct runoff enters the main waterway and becomes streamflow (wet weather surges), the rest empties into groundwater stores, which slowly releases the water to the stream as baseflow (dry weather flows).

4.1 AWBM Parameters

The parameters used for the AWBM models are defined below.

Each hypothetical storage is defined in terms of its depth '**d**' (mm), and the proportion of the catchment that it covers '**a**'.

When a storage overflows, the proportion of the overflow, which becomes groundwater is '**B**'.

The rate at which groundwater is fed into streams depends on the baseflow recession constant '$\mathbf{K_b}$'.

The length of time required for water in a stream to reach the catchment outlet depends on the surface recession constant '$\mathbf{K_s}$'.

4.2 Climate Data

4.2.1 Rainfall

Daily rainfall data was available for a number of sites around and within the Merri River catchment. Three sites were selected due to their geographic distribution and to provide the best coverage of the catchment. Refer to Figure 4.1 showing the location of the gauges.

90045	Hawkesdale Post Office	Sep 1884 to Sep 2002
90084	Woolsthorpe	Oct 1884 to Jun 2001
90081	Warrnambool Shire Office	Apr 1867 to Apr 1998

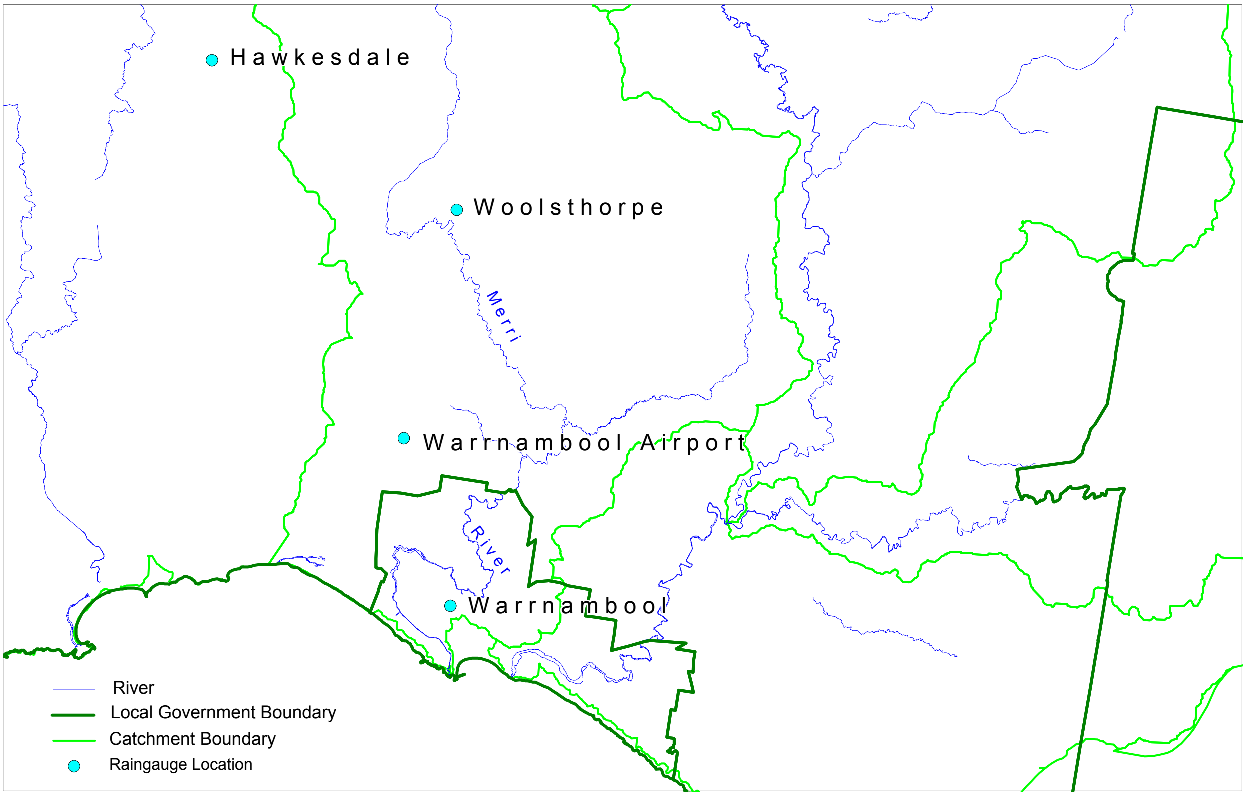

Figure 4.1 Rain gauge Locations

Data after 1998 for Warrnambool has been collected at the airport, which is approximately 10 km to the north. Due to the distance the data at the airport was not combined with the data from the Shire Office to extend the record.

The data for Warrnambool from 1867 to 1998 appeared to be complete, however there were several gaps in the Hawkesdale and Woolsthorpe records. Generally, there were only a few months of missing data over the whole period. The longest gap appears in the Hawkesdale record between 1924 and 1930. This was not considered to be significant as there was no historical record of large flood events during this period.

4.3 Evaporation

Gauged evaporation data was not available within the catchment. Monthly averages were obtained from the Bureau of Meteorology's Average Point Potential Maps. The figures were then divided by the number of days in the month to produce daily estimates.

The average monthly evaporation used has been shown in Table 4.1 AWBM Evaporation Data.

Table 4.1 AWBM Evaporation Data

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Evap (mm)	180	160	130	70	40	35	40	60	70	120	140	160

4.4 AWBM Calibration

The model was initially calibrated using a number of key factors:

- The total volume of recorded flow was compared with the total volume from the model. The storage capacities were adjusted until a balance was achieved.
- The daily hydrographs for the recorded streamflow were compared against the calculated flow. The model was adjusted to match the shape of the recorded hydrograph and thus simulate a similar catchment response.
- A comparison was made between the recorded daily maximum flows and the recorded instantaneous peaks. It was found that there was an average increase of 20.8% from the daily peak to the instantaneous maximum peak flow. Therefore the yearly maximum flows were factored up by 20.8% to convert them to instantaneous maximum peaks.

Following the initial calibration of the AWBM model a flood frequency analysis was undertaken and the magnitudes of the peak flows and the 1946 flow were reviewed. These initial results were discussed with Brad Henderson of the Glenelg Hopkins CMA and Ian Gauntlett of the Department of Natural Resources and Environment. The initial results were not considered a good fit to the peak flood flows and therefore the following adjustments were made to achieve a better fit to the observed peak flows:

- A rainfall frequency analysis was undertaken as described in Section 3.2.
- The soil storage parameters in AWBM were adjusted to achieve a better fit for the peak flows (with a less accurate fit to total flow volumes).
- The flood frequency analysis was redone and the frequency of the predicted 1978 and 1946 flows were compared with the rainfall frequency results.

Note: The calibration of the model was based on streamflow data from 1974 to 2000. Data prior to 1974 was not used due to a lack of continuous data.

The parameters used for each AWBM model are shown in Figure 4.2, Figure 4.3 and Figure 4.4.

4.5 Comparison of Modelled Flows with Actual Flow

4.5.1 Hawkesdale

Figure 4.2 shows the comparison between the modelled flows using the rainfall at Hawkesdale and the observed flows at the Woodford gauge. The modelled results provided a reasonable fit for the period shown, although the corresponding peaks differed in magnitude. However, the model failed to predict the flood in 1996. This may be due to the fact that the storm cell was concentrated along the coast and did not travel as far north as Hawkesdale.

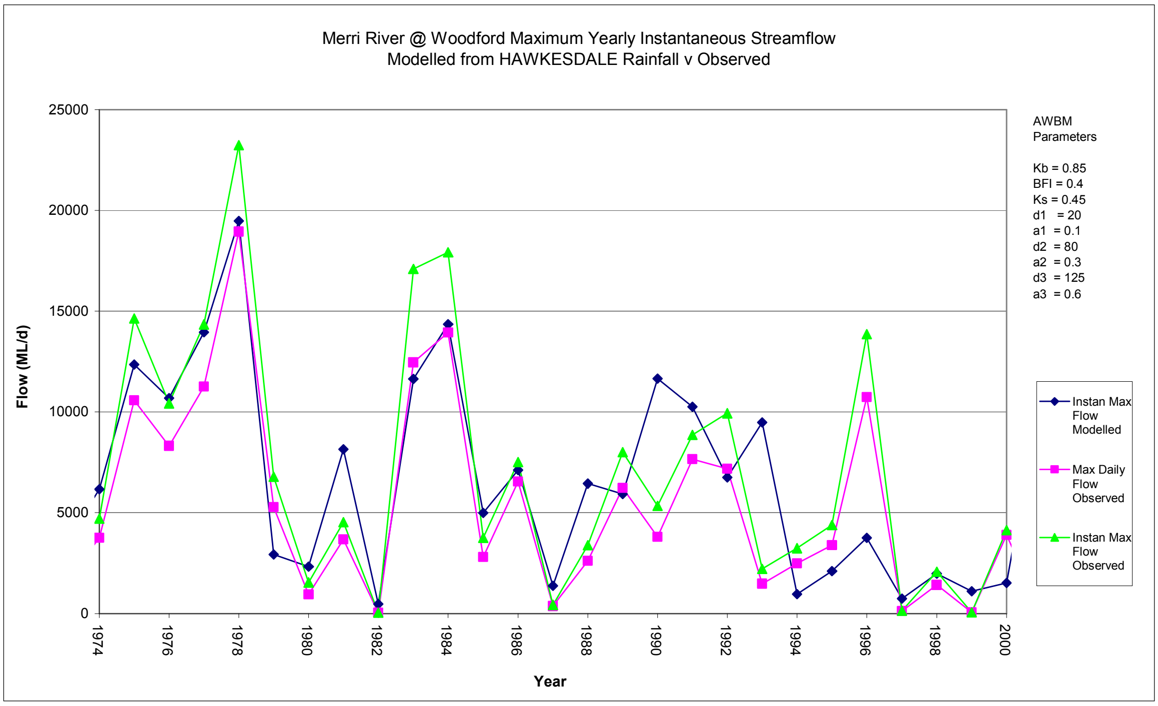

Figure 4.2 Modelled Flows (Hawkesdale Rainfall)

4.5.2 Woolsthorpe

Figure 4.3 shows the comparison between the modelled flows using the rainfall at Woolsthorpe and the observed flows at the Woodford gauge. The modelled results provided a reasonable fit, although not as good as the result using the Hawkesdale rainfall. A number of the peaks coincide, although their magnitudes differ.

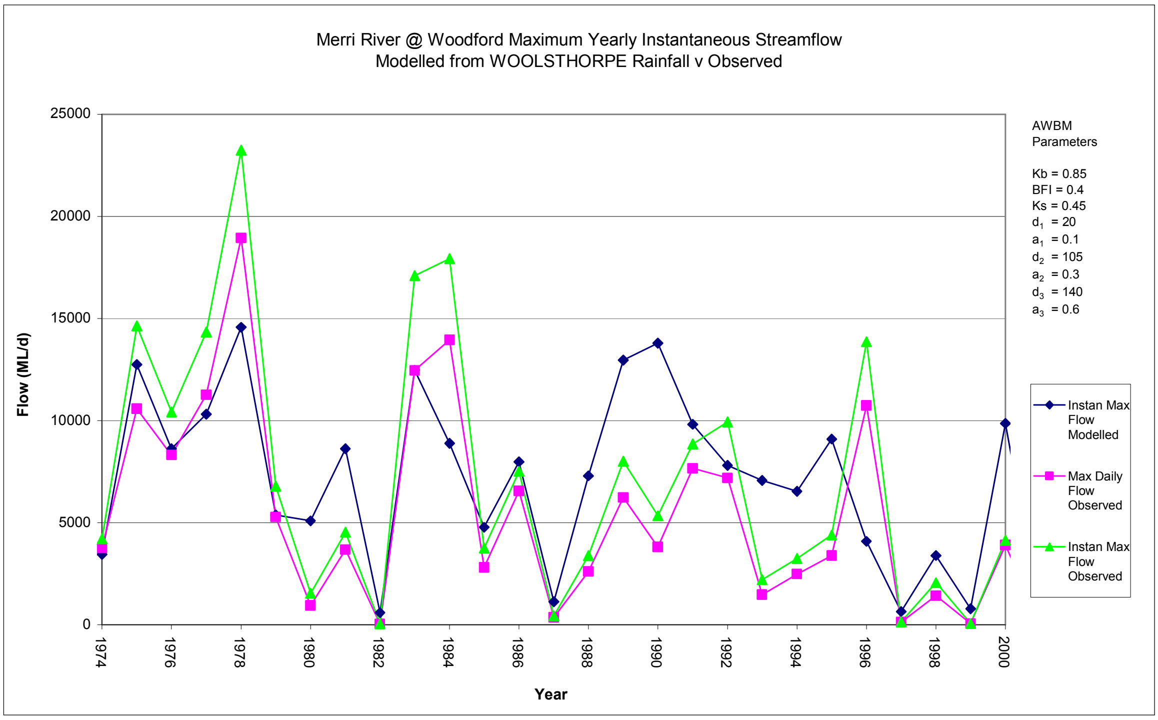

Figure 4.3 Modelled Flows (Woolsthorpe Rainfall)

4.5.3 Warrnambool

Figure 4.4 shows the comparison between the modelled flows using the rainfall at Warrnambool and the observed flows at the Woodford gauge. The model failed to predict the large flood in 1978 and underestimated the floods in 1983 and 1984. This may be due to the fact that the storm cells were more concentrated further inland and did not travel as far south as Warrnambool. The lack of heavy rainfall along the coastal region resulted in the model's inability to accurately predict the floods.

The model tended to overestimate flows in the following years, 1981, 1990 and 1991. In these cases the storm cells may have been more concentrated along the coastal regions, resulting in larger modelled flows.

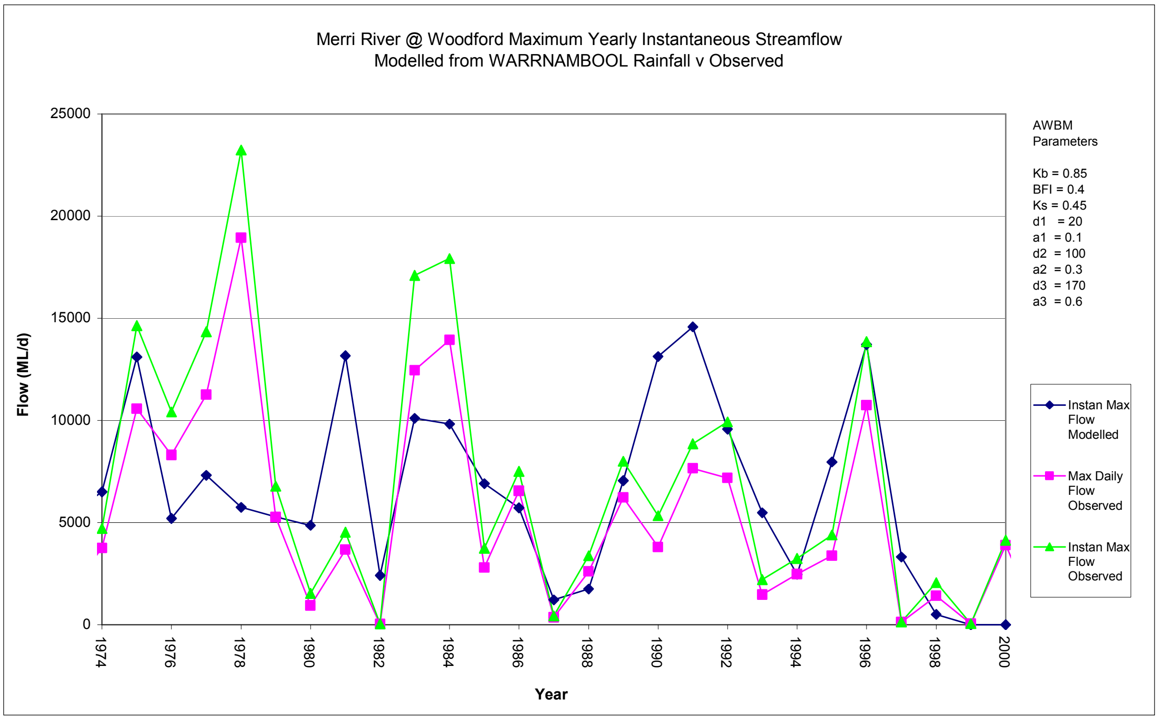

Figure 4.4 Modelled Flows (Warrnambool Rainfall)

4.6 Extending the Length of Streamflow Record to 100 Years

The calibrated AWBM models were then used to synthesise 100 years of streamflow at Woodford using the rainfall data at each location. The results for each location are shown in Appendix B. This extended length of record would provide a more detailed flood frequency analysis, which would take into consideration the 1946 flood.

4.6.1 Estimate of the 1946 Flood

Using the rainfall data at the three locations the AWBM model was able to estimate the magnitude of the 1946 flood at Woodford (see Table 4.3 Estimate of 1946 Flood at Woodford).

Table 4.3 Estimate of 1946 Flood at Woodford

Rainfall Location	Estimate of 1946 Flood (ML/d)
Hawkesdale	31,900
Woolsthorpe*	25,000*
Warrnambool	40,500

* Woolsthorpe results not used, see following page for discussion.

4.7 Flood Frequency Analysis Using Synthesised Data

A flood frequency analysis was undertaken for annual instantaneous maximum peak flows at Woodford using rainfall data at the following locations:

- Hawkesdale 1900 – 2002;
- Woolsthorpe 1900 – 2001; and
- Warrnambool 1900 – 1998.

The datasets were fitted with Log Pearson Type III (LP3) frequency distribution in accordance with techniques described in AR&R. Graphs of the Flood Frequency Analysis are contained in Appendix C.

Table 4.4 FFA Using Synthesised Data

	Hawkesdale		Woolsthorpe*		Warrnambool	
Design Flood ARI (Years)	Peak Design Flow (ML/day)	5% & 95% Confidence Limits	Peak Design Flow (ML/day)	5% & 95% Confidence Limits	Peak Design Flow (ML/day)	5% & 95% Confidence Limits
5	10,918	**9,203 - 12,953**	10,737	**9,328 - 12,359**	10,667	**9,076 - 12,538**
10	14,869	**12,170 - 18,167**	13,750	**11,835 - 15,974**	14,681	**12,163 - 17,722**
20	18,905	**14,795 - 24,156**	16,430	**13,704 - 19,698**	18,943	**15,057 - 23,831**
50	24,359	**17,693 - 33,537**	19,645	**15,376 - 25,098**	24,996	**18,536 - 33,706**
100	28,564	**19,469 - 41,907**	21,842	**16,160 - 29,523**	29,886	**20,887 - 42,763**

<u>Note:</u> Results exclude low flow years defined as those years with peak flow less than 1,000 ML/day

* Woolsthorpe results not used, see following page for discussion.

The preliminary FFA (see Table 2.1) indicated that the 100-year ARI flood was 28,800 ML/d using the Gumbel distribution and 31,300 ML/d using the LP3 distribution. The results of the FFA using the 100 years of data for Hawkesdale and Warrnambool seem to confirm the initial estimates (see Table 4.4 FFA Using Synthesised Data). The use of 100 years of data provided tighter confidence limits using the LP3 distribution, when compared to the 36 years of data.

The results at Woolsthorpe were disregarded due to the lower estimate of the 100-year ARI flood. This was due to the fact that the rainfall at Woolsthorpe during the 1946 event was substantially lower than at the other two locations (see Table 3.2). This in turn provided a lower estimated peak streamflow for 1946 and affected the FFA.

4.8 Adopted Flows at Woodford

The synthesised data from Hawkesdale and Warrnambool was used to estimate the flood magnitude. It was decided to take an approximate average of the two figures to provide the final estimate.

- Adopted 100-year ARI Flow = 29,200 ML/d
- Adopted 50-year ARI Flow = 24,700 ML/d
- Adopted 20-year ARI Flow = 18,900 ML/d
- Adopted 10-year ARI Flow = 14,800 ML/d
- Adopted 5-year ARI Flow = 10,800 ML/d

5 Adopted Merri River Flows

It was necessary to extend the flow estimates for the Merri River from Woodford to within the study area. An additional 26 km^2 of catchment contributes to flows between the Woodford gauge and Russell Creek (compared with a total catchment at the Woodford gauge of 899 km^2).

A further 32 km^2 of catchment contributed from Russell Creek. The adopted design flows in the Merri River within the Study area are given in Table 5.1 below.

Table 5.1 Adopted Design Flows

ARI	Merri River Design Flow Upstream of Russell Creek (ML/d)	Merri River Design Flow Downstream of Russell Creek (ML/d)
5	11,100	12,400
10	15,202	17,252
20	19,500	21,600
50	25,400	29,300
100	30,000	35,400

5.1 1946 Flood

The peak flood flows for the Merri River for the 1946 flood were estimated at a number of locations. An estimate of the 1946 flow in the lower reach of Russell Creek was also made as detailed below.

5.1.1 ARI of the Merri River 1946 Flood

The AWBM analysis using the rainfall at Hawkesdale and Warrnambool the estimated 1946 flood to be between 31,900 – 40,500 ML/d at the gauge at Woodford. From the flood frequency curves (see Appendix C) both rainfall locations estimate the 1946 event to have an ARI >500 years.

The 1946 flood may have been produced by a different weather pattern than the smaller floods. The frequency of occurrence of this weather pattern is not known and therefore assigning an ARI to the 1946 flood is imprecise.

5.1.2 Adopted Flow for 1946 Flood at Woodford

It was decided to take an approximate average of the above flows to provide the final estimate at Woodford.

- Adopted 1946 Merri River Flow at Woodford = 36,200 ML/d.

5.1.3 Adopted Flow for 1946 Merri River Flood upstream of Russell Creek

It was necessary to extend the flow estimates for the Merri River from Woodford to within the study area. An additional 26 km^2 of catchment contributes to flows between the Woodford gauge and Russell Creek (compared with a total catchment at the Woodford gauge of 899 km^2).

- Adopted 1946 Merri River Flow upstream of Russell Creek = 37,200 ML/d.

5.1.4 Adopted Flow for 1946 Flood in the lower reach of Russell Creek

The additional flows for the Russell Creek catchment were added to the Merri River based on the method outlined in Hydrological Recipes Section 6.4.

$$F = (A_C / A_G)^{0.7}$$

Where

F	multiplier function
A_C	catchment area of Russell Creek (km^2)
A_G	catchment area of Merri River (km^2)

$$F = (32 / (899+26))^{0.7}$$

$$= 0.0949$$

Therefore:

- Adopted 1946 Flow for Russell Creek = 0.0949 x 37,200 ML/d
  = 3,530 ML/d.

This flow rate is only considered applicable for the lower reaches of Russell Creek, where the flooding is dominated by the backwater from the Merri River. The lack of flow or flood level data for most of Russell Creek for 1946 makes predictions of 1946 flows and flood levels less certain for the upper reaches of Russell Creek, particularly given possible variations in storm pattern and the timing of peak flows from Russell Creek compared with the Merri River.

5.1.5 Adopted flow for 1946 Flood in Merri River downstream of Russell Creek

Downstream of Russell Creek the estimated peak flow for the Merri River was determined based on adding the additional flow from the Russell Creek catchment.

- Adopted 1946 Merri River Flow downstream of Russell Creek = 40,730 ML/d.

5.2 Estimated PMF at Woodford

Using the Generalised Short-Duration Method (Bulletin 53) together with AR&R the Probable Maximum Flood (PMF) was calculated to have an ARI of 1,000,000 years. The flood frequency curves for Hawkesdale and Warrnambool rainfalls were extrapolated to produce the following results:

- PMF (Hawkesdale rainfall) = 160,000 ML/d; and
- PMF (Warrnambool rainfall) = 200,000 ML/d.

It was decided to take an average of the two figures to provide the final estimate.

- Adopted PMF in Merri River at Woodford = 180,000 ML/d.

5.3 Estimated PMF within Study Area

It was necessary to extend the flow estimates for the Merri River from Woodford to within the study area. An additional 26 km^2 of catchment contributes to flows between the Woodford gauge and Russell Creek.

- Adopted PMF in Merri River at Russell Creek = 185,000 ML/d.

6 Catchment Hydrology for Russell Creek

Russell Creek is a partly urban catchment of approximately 32 km^2 north of Central Warrnambool. No stream flow data exists for Russell Creek and the most appropriate method for determining design flood flows is by calculation using rainfall/runoff techniques.

6.1 RORB Model Establishment

A RORB model was established for the Russell Creek Catchment, based on the current Planning Scheme, to predict the peak flows for the 20%, 10%, 5%, 2%, 1% and PMP floods. RORB is a runoff routing computer model developed at Monash University and is a recognised industry standard program in Australia.

The hydrological analysis was applied in general accordance with the procedures outlined in Australian Rainfall and Runoff published in 1999 by the Institution of Engineers, Australia (AR&R).

6.2 RORB Data Requirements

- Rainfall intensity frequency duration relationship for return periods up to 100 years from AR&R Volume 1;
- PMP estimates derived as per AR&R for short duration storms;
- Storm temporal patterns from AR&R Volume 1;
- Catchment areas, stream lines and slopes from base topographic mapping at 1:25,000 scale;
- Land use from aerial photography and Planning Schemes; and
- Runoff coefficients from AR&R Volume 2.

6.3 Catchment Subdivision

Application of RORB required the catchment to be divided into a number of sub areas. Nodes were placed on the main stream at the centroid of each sub area. The rainfall excess hydrograph for the sub area is assumed to be concentrated at this node. Nodes are also placed at stream confluences and at key locations within the study area. Conceptual storages between each pair of nodes are used to model the storage effects of river reaches.

The conceptual storages have the form:

$$S = k_c \, k_r \, Q^m$$

where: S is the volume of water in temporary storage;

k_c is the catchment parameter;

k_r is a reach parameter, usually taken as proportional to reach length;

Q is the discharge from storage; and

m is an exponent controlling the non-linearity of catchment response.

The parameter k_c was calculated from Equation 3.22, ARR 99, Book 5. The relationship for k_c is for a value of *m* of 0.8 and for a region where the mean annual rainfall is less than 800 mm.

$$k_c = 0.49A^{0.65}$$

where: A is the catchment area (km^2)

The Russell Creek catchment was sub-divided into 14 sub areas. Details of the model sub areas are shown in Appendix D. The RORB catchment file is provided in Appendix E.

6.4 RORB Parameters

RORB's initial loss/constant continuing loss model was used throughout the study. The parameters adopted were based on the method outlined in AR&R 1999 Book 6.

Table 6.1 RORB loss parameters

ARI	Initial Loss (mm)	Continuing Loss (mm/h)
100	26.3	3.5
50	26.3	3
20	25	3
10	25	3
5	20	3

**Note: For lower ARI events the Initial and Continuing Losses were reduced to produce realistic flow rates for minor storms.*

- m = 0.8
- k_c = 4.7
- Fraction impervious for Rural Zones = 5%
- Fraction impervious for Residential Zones = 50%

6.5 RORB Results

Table 6.2 Russell Creek Design Flows contains a summary of the peak flows at key locations within the Russell Creek catchment for various design floods and over a number of storm durations.

Table 6.2 Russell Creek Design Flows

Design Flood ARI (Years)	Flow at Aberline Road (ML/d)	Flow at Mortlake Road (ML/d)	Flow at Queens Road (ML/d)	Flow at Merri River / Russell Creek Confluence (ML/d)
100	3,000	5,100	5,400	5,400
50	2,300	3,800	4,000	4,100
20	1,300	2,200	2,400	2,400
10	1,100	1,900	2,000	2,100
5	600	1,300	1,400	1,500

7 Hydraulic Modelling and Determination of Flood Levels

7.1 Overview

Hydraulic models use flood flow and terrain data as model inputs to determine flood levels. There is a wide range of hydraulic models that vary in complexity. In general, as they increase in complexity, they require more data and take longer to prepare and run, but the reliability of the output also increases. Engineering judgement is required to determine what degree of model complexity is required. In general, engineers try to use the simplest (and therefore quickest and least costly) model that will provide an adequate degree of certainty. A description of various types of hydraulic models is provided in Appendix K.

On this project, two types of hydraulic models were used:

- HEC-RAS for the Merri River and most of Russell Creek; and
- Delft Flood Level System (FLS) for the “breakaway area” at Russell Creek.

The hydraulic modelling of Russell Creek was undertaken in two stages:

- Stage 1: HEC-RAS model of Merri River and Russell Creek; and
- Stage 2: FLS model of Russell Creek “breakaway area”.

The hydraulic modelling is described below.

7.2 HEC-RAS model

A HEC-RAS computer model was established for Merri River and Russell Creek in three separate reaches (see Figure 7.1 on the following page and the plan in Appendix G):

- Merri River from Caramut Road upstream to Russell Creek;
- Merri River from Russell Creek upstream to the continuation of Grange Road; and
- Russell Creek from Merri River upstream to Aberline Road.

The model was used to determine flood levels for a range of flood events. HEC-RAS Version 3.1 allows the inclusion of the following features in the model:

- One dimensional flow along the Merri River and Russell Creek (refer to Appendix K for further explanation of a one dimensional model);
- Consideration of ineffective flow areas;
- Flow blockouts for buildings and other obstructions; and
- Bridge routines to model flow through bridges.

Parts of the HEC-RAS model were calibrated to observed flood levels where these were available. The HEC-RAS model runs quickly and sensitivity analyses on the hydraulic parameters, and the assumed downstream water levels, were undertaken.

Cross sections of Merri River and Russell Creek, with their respective floodplains, for use in the HEC-RAS model floodplain were extracted from the Digital Terrain Model (DTM) supplied by Qasco. Bathymetry data was not available, however the Merri River cross section below water level is expected to have an insignificant capacity in relation to the larger flood flows modelled in this study. For the smaller floods (5 year ARI), the effect of not having bathymetry is expected to be more significant. However the 5 year ARI flood is not used for planning purposes and so the cost of bathymetry is not warranted.

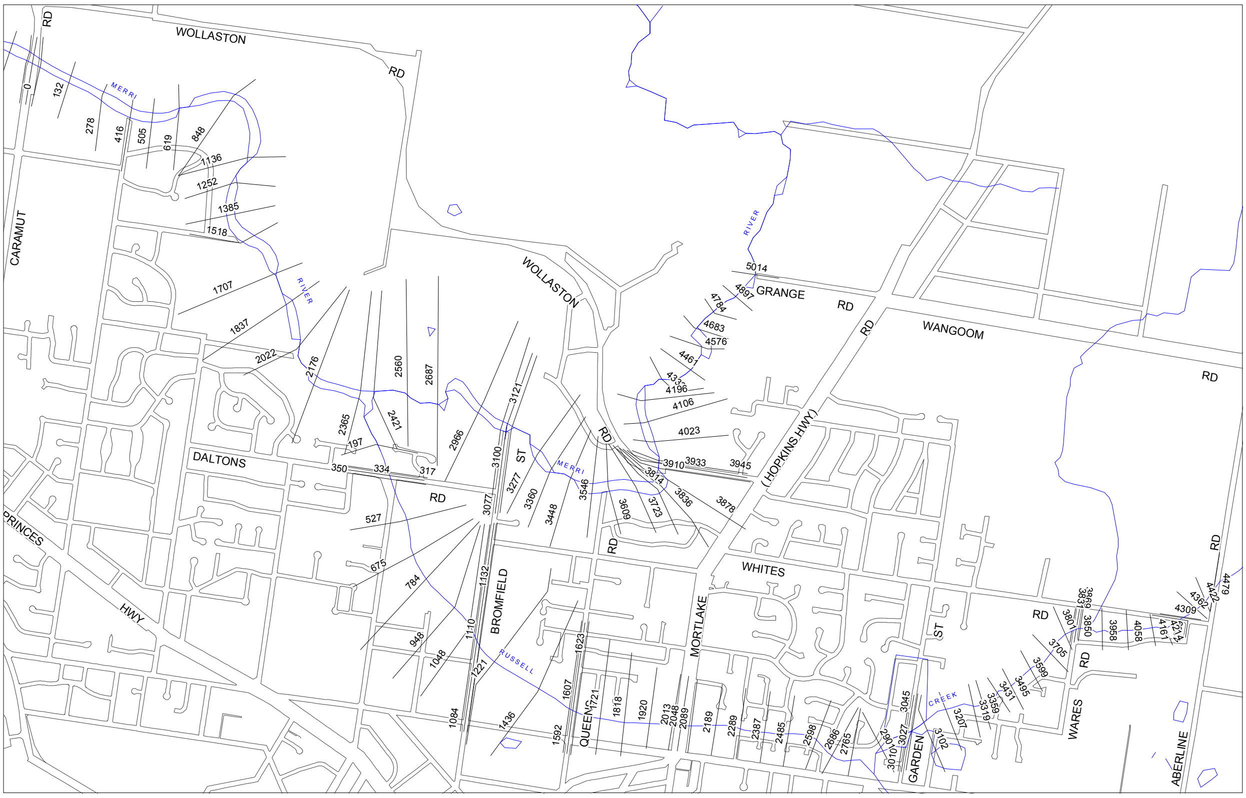

Figure 7.1 HEC-RAS model layout

7.3 Calibration of the HEC-RAS model

The 1946 flows were input into the HEC-RAS model and calibrated to the observed 1946 flood levels sourced from the Flood Data Transfer Project, i.e. hydraulic roughness (Manning “n”) values were selected with a view to obtaining agreement between the observed and modelled flood levels. There was some inconsistency in the observed flood levels and therefore some judgement had to be applied. The downstream boundary condition for the 1946 flood event was based on the observed flood level of 6.58 metres AHD on the upstream side of Caramut Road (to allow for losses with the old timber bridge) refer to Figure 7.2. Boundary conditions for a range of flows were estimated from other historic floods and by review of hydraulic parameters, including the hydraulic slope, Froude Number, velocity and conveyance. Using this data a rating curve was established for the boundary conditions for use with the design flood event flows.

Blockage factors at other bridges were found to have a negligible effect on the calibration. For the lower Russell Creek bridges and culverts and for the Wollaston Bridge the head losses at the bridges were very low due to most of the flood flow occurring over the roads rather than through the bridges.

Observed 1946 flood levels in the vicinity of the Merri River / Russell Creek confluence were in the range of 7.714 to 8.004 metres AHD (refer to Figure 7.2). An average of the two above mentioned flood levels was taken to produce a flood level of 7.86 metres AHD, which was used for the calibration of the HEC-RAS model.

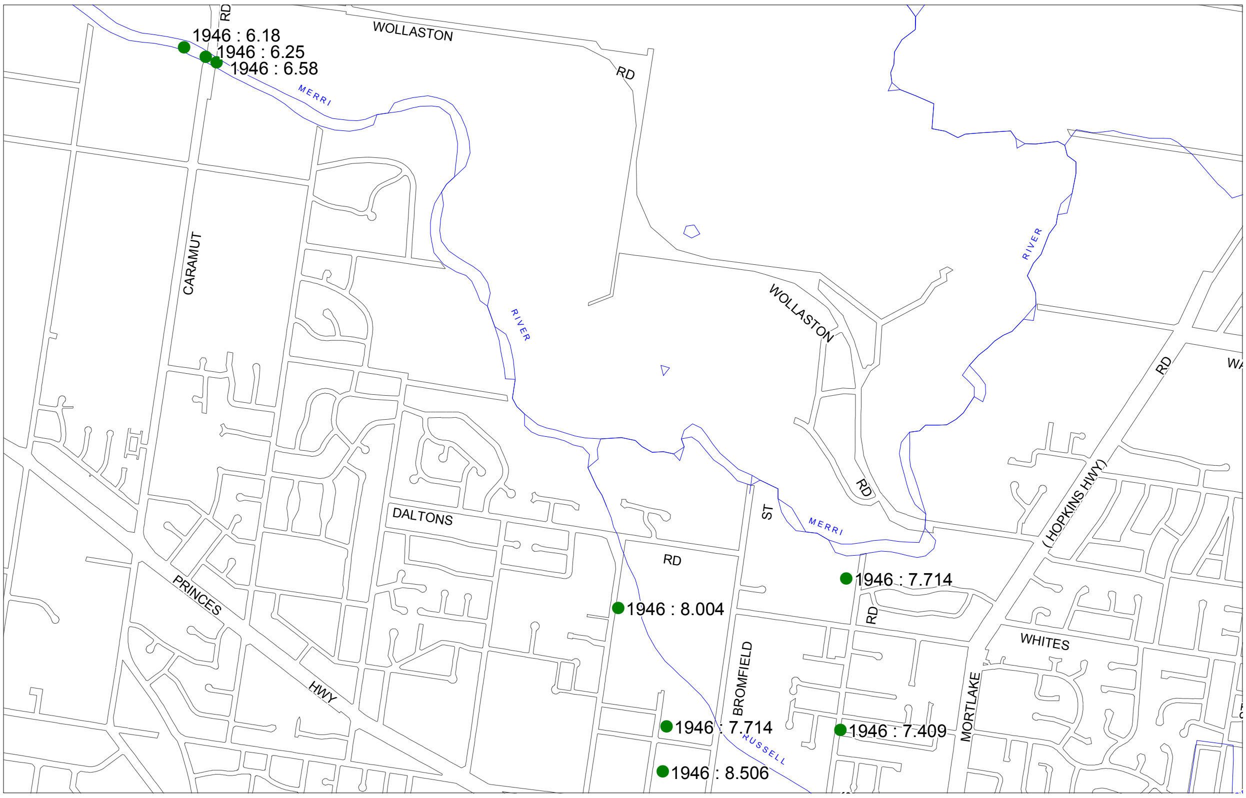

Figure 7.2 1946 Flood Levels

Typical values for Manning's n for the Merri River and Russell Creek are shown below.

Table 7.1 Adopted Manning's "n" values for Merri River and Russell Creek

Location	Main Channel	Floodplain	Description
Merri River – Caramut Road to Russell Creek	0.08	0.15	Calibrated to 1946 flood. Adopted values are at the high end of the design range due to the narrow valley, debris and rows of trees across floodplain.
Merri River upstream of Russell Creek	0.06	0.12	Calibrated to 1946 flood. Adopted values in upper part of design range.
Russell Creek	0.06	0.12	No calibration data available, except in lower reaches where backwater effect from Merri River dominates flood levels rather than Manning's "n" values.

7.4 HEC-RAS Flood Profiles

Table 7.2 below contains the flood levels at selected locations obtained from the HEC-RAS model for the design events and the 1946 flood. It should be noted that the 1946 flood levels upstream of Mortlake Road are based on a rough estimate of the 1946 flood flow in Russell Creek and should be used with caution. The HEC-RAS flood levels in the Russell Creek "breakaway area" are not the levels adopted by this study and should not be used. A plot of each HEC-RAS cross-section and a complete set of water surface levels are shown in Appendices G and H.

Table 7.2 Water Surface Levels (WSL) from HEC-RAS model

Stream Reach	Reach Chainage (m)	WSL 5 yr ARI	WSL 10 yr ARI	WSL 20 yr ARI	WSL 50 yr ARI	WSL 100 yr ARI	1946 Flood
Merri river upstream of Caramut Road	0	3.05	3.83	4.22	5.18	5.92	6.59
Merri River downstream of Russell Creek confluence	2365	5.15	5.78	6.08	6.80	7.34	7.85
Merri River downstream of Russell Creek confluence	2421	5.16	5.79	6.09	6.80	7.34	7.85
Merri River downstream of Wollaston Bridges	3878	5.58	6.16	6.46	7.09	7.59	8.10
Merri River upstream of Wollaston Bridges	3933	5.68	6.37	6.50	7.13	7.62	8.15
Merri River at upstream end of study area	5014	6.24	6.91	7.11	7.68	8.12	8.61
Russell Creek upstream of Daltons Road	350	5.24	5.81	6.12	6.83	7.37	7.85
Russell Creek upstream of Bromfield Street	1132	5.42	5.90	6.17	6.84	7.38	7.86
Russell Creek upstream of Queen Street	1623	6.74	6.83	6.88	7.05	7.46	7.88
Russell Creek upstream of Mortlake Road	2089	8.19	8.54	8.79	9.76 (1)	10.54 (1)	N/A
Russell Creek upstream of Garden Street	3045	11.91	12.07	12.10	12.25	12.31	N/A
Russell Creek upstream of Whites Road	4309	21.54	21.63	21.68	21.81	21.88	N/A

1. These results have been superseded by 2-dimensional modelling, refer to Section 7.7 below.

The following diagrams show the flood profiles for the Merri River and Russell Creek.

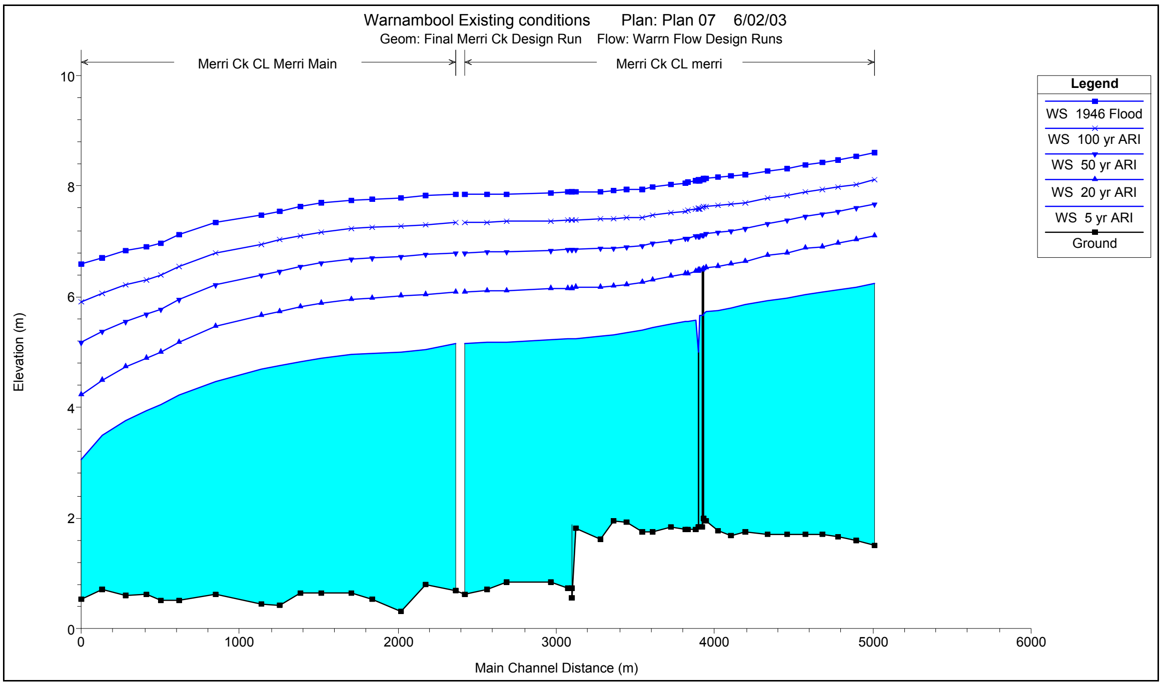

Figure 7.3 Merri River Flood Profile Long Section

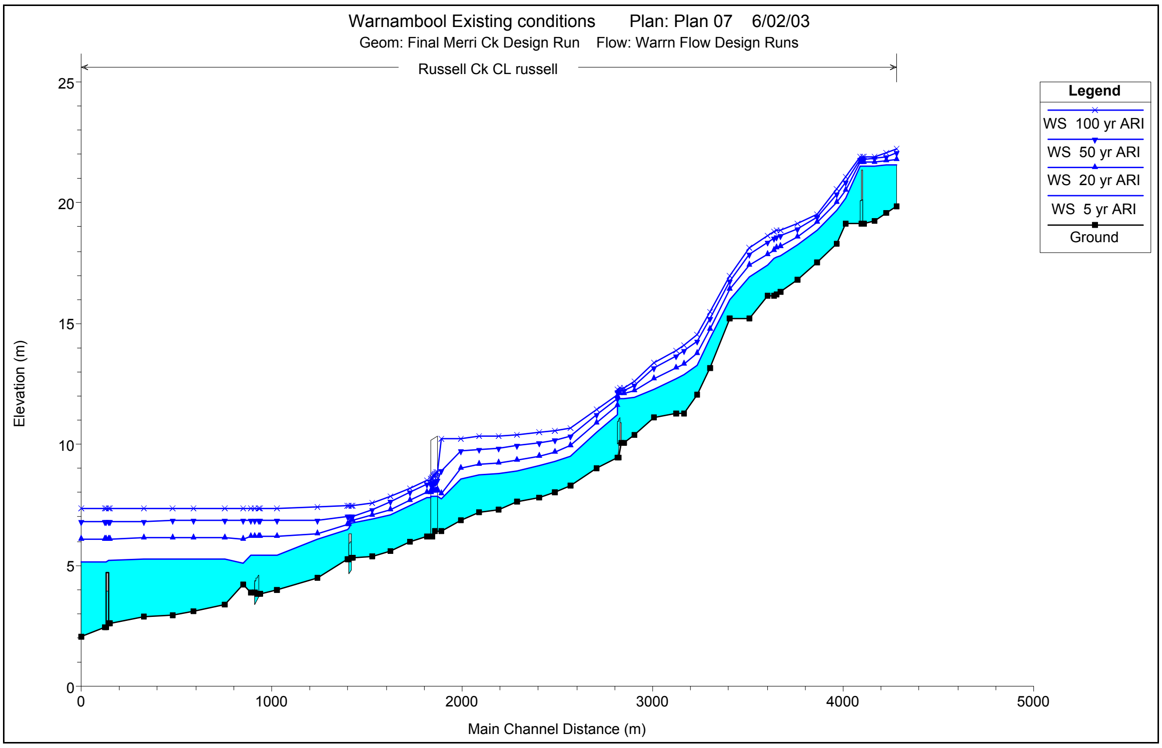

Figure 7.4 Russell Creek Flood Profile Long Section

7.5 Comparison of 1946 and 100 year ARI floods

For the Merri River and the lower reaches of Russell Creek (downstream of Queen Street, refer to Figure 7.1) the modelled 100 year ARI flood levels are approximately 500 mm below the observed flood levels from the 1946 flood. The 1946 flood was estimated to be greater than a 500 year ARI flood event as discussed in Section 5.1.

Approval of development at levels below the 1946 flood may not be appropriate and therefore the Glenelg Hopkins CMA and Warrnambool City Council may choose to apply a freeboard that results in fill levels in the flood fringe areas that are at or above the 1946 flood levels. Further discussion of this issue and recommendations could be developed in a Floodplain Management Plan.

7.6 Limitations of HEC-RAS Model

The flood levels initially determined using HEC-RAS were used to plot the extent of inundation in Russell Creek. It became apparent that the 100 year and 50 year ARI flood flows would probably not be confined to the creek. In a 100 year ARI flood, water would breakaway from Russell Creek approximately 200 m upstream of Mortlake Road and flow generally north and north-west along Labella Court, Evelyn Crescent, Breton Street and Hayley Drive. Flood water would overtop Mortlake Road and flow generally west before rejoining Russell Creek.

This finding posed a dilemma to the project because, on the one hand, HEC-RAS, being a 1-dimensional model (refer to Appendix K for an explanation of hydraulic models) cannot model the Russell Creek breakaway, and yet inundation maps were required. Initially maps were produced using the HEC-RAS levels. The extent of inundation in the breakaway area was determined based on the flood level in Russell Creek. The extents presented in May were approximate and could be considered a “first approximation”.

Two-dimensional modelling is much more time consuming than HEC-RAS. However, it was decided that a 2-dimensional model was required to model the breakaway and the shallow flow through the streets within the breakaway area.

7.7 2-Dimensional Modelling of the Russell Creek Breakaway

The 2-dimensional model called Delft Food Level System (FLS) was selected for Russell Creek. FLS uses a regular square grid as the basis for its computations and has a maximum size of 800 x 800 grid points. A grid size of 2.5 m was selected because this dimension is small enough to model the flow in the gutters and yet big enough that the extent of the model, i.e. 2 km x 2 km (2 km =2.5 m x 800) would cover the entire area affected by the breakaway. Refer to Figure 7.5 for the extent of the FLS model.

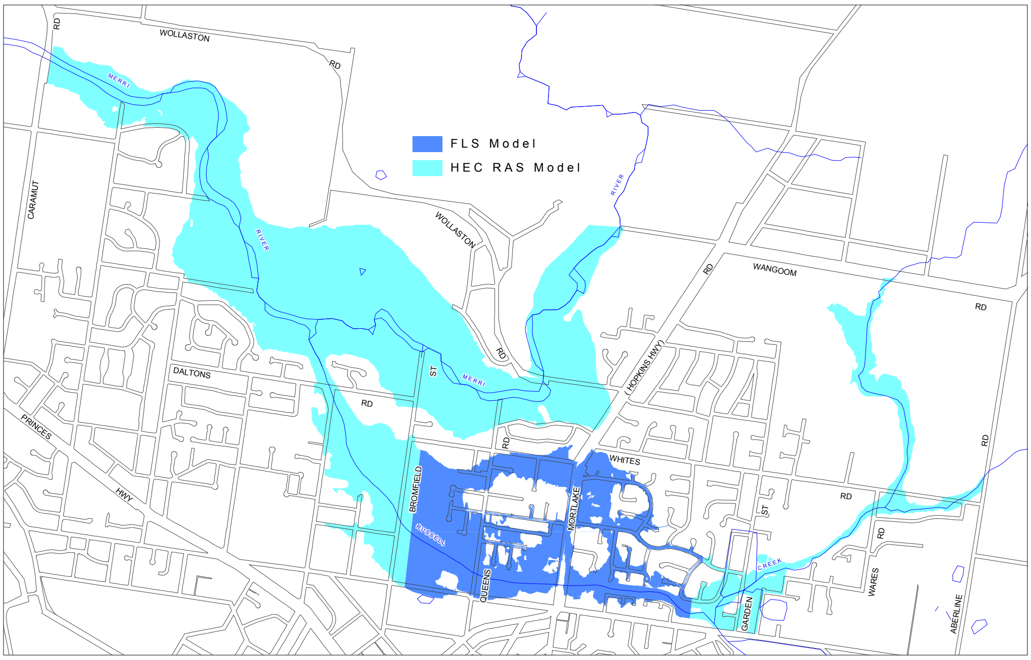

Figure 7.5 Extent of 2-dimensional FLS model

The DTM obtained from Qasco was used as the geometric basis for the model. Other inputs to FLS include a flood flow at the upstream end (eastern end) of the model and a downstream boundary condition, which in this case was taken from the HEC-RAS model. It should be noted that the flood flow in Russell Creek decreases with distance upstream. In the area covered by the FLS model the 100 year ARI flood flow decreases from 62.6 m^3/s at the downstream boundary to 52.6 m^3/s at the upstream boundary. In FLS it is difficult to change the flow midway through the model and the FLS model was run with a constant flow of 59.6 m^3/s, which is the flow at Mortlake Road. This flow was chosen because it is most relevant in terms of determining the breakaway flow, but it should be noted that the FLS flow at the upstream end of the model was larger than the HEC-RAS flow, i.e. 59.6 m^3/s rather than 52.6 m^3/s. This anomaly was accounted for in the mapping by adopting the FLS modelled levels for much of the FLS modelled area, and adopting the HEC-RAS levels in the area where the flows are smaller, refer to Figure 7.1. It should be noted that, once the breakaway flow was known (from the FLS model) the HEC-RAS model was re-run with only the creek flow at the Mortlake Road culvert.

FLS can model bridges, but not as well as in HEC-RAS. The flood levels upstream of Mortlake Road are heavily influenced by the culvert under the road, which is an arch with a bike path in it. In order to ensure that the culvert was properly modelled, it was modelled in FLS as twin boundary conditions. The relationship at each boundary was the elevation-discharge relationship of the culvert derived using HEC-RAS.

FLS was run for the 100 and 50 and 20 year ARI flood flows. In was found that in the 20 year ARI flood, flows do not breakaway. This means that the HEC-RAS results for the 20 year ARI and smaller flows are valid and were adopted for mapping purposes.

7.8 Differences Between HEC-RAS and FLS 100 year ARI Flood Levels

Flood maps of Russell Creek, based on HEC-RAS were produced in May 2003. As described earlier, HEC-RAS is not able to model the breakaway and the FLS model results were different in many places. A comparison of flood level results is provided below. The comparison is based on 5 regions (refer to Figure 7.6).

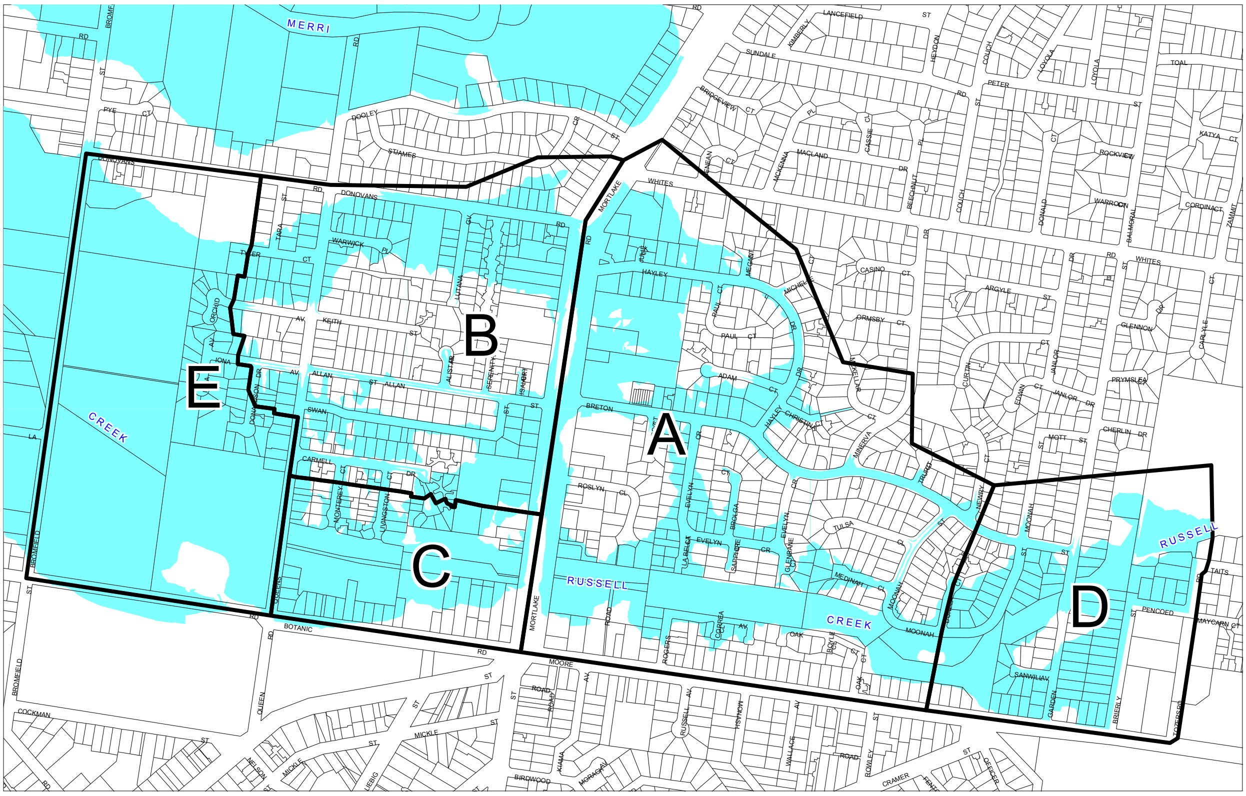

Figure 7.6 Regions of FLS Model

- Region A: Flood levels in region A are lower for two reasons. Firstly, FLS has included the flow over Mortlake Road and thereby decreased the flow through the Mortlake Road culvert. This leads to lower flood levels in Russell Creek upstream of the culvert. Secondly, the flood levels in the breakaway area have been modelled with the 2.5 m grid model and should be considered reliable. Previously the flood levels in Russel Creek, as modelled using HEC-RAS were adopted. The change varies from approximately 1.8 m lower in Hayley Drive just upstream of Mortlake Road to approximately 0.6 m lower in Labella Court.
- Region B: Flood levels in Region B are higher because they now account for the flow over Mortlake Road, which HEC-RAS could not account for. Flood levels are higher by approximately 150 to 500 mm.

- Region C: Flood levels in region C are lower because the flow in Russel Creek in this location is lower. This region is located upstream of where the Mortlake Road overflow returns to the creek. Flood levels are lower by approximately 60 to 200 mm.
- Region D: Flood levels in this region remain unchanged for two reasons: (1) the flows in the creek are not affected by the breakaway so they are unchanged, and (2) the effect of the lower flood levels just upstream of Mortlake Road have "died out" in this region because it is too far upstream.
- Region E: Flood levels in this region remain unchanged because the flood flows in the breakaway have returned to the creek, i.e. they are the same as those used in HEC-RAS. Flood levels in this region are governed by the backwater effect from the Merri River.

8 Interpretive Mapping

8.1 Land Subject to Inundation Delineation

The 100-year ARI flood extent was adopted as the land subject to inundation overlay (LSIO) in accordance with the Guidelines for Delineating Floodway (NRE, July 1998). The LSIO shows:

- Significant flood storage areas that should be subject to planning control;
- Effluent streams that should be subject to planning control; and
- Areas where inappropriate works, which might be damaged by floods, or may aggravate the impact of floods, and should come under adequate planning control.

8.2 Floodway Delineation

The delineation of Floodway was in accordance with the criteria set out in Guidelines for Delineating Floodways (NRE, July 1998). The floodway is generally defined as follows:

- Significant flow path that conveys the majority of the flood waters;
- Significant flood storages;
- Area where higher velocities and depths are experienced compared with the rest of the floodplain (depths of greater than 0.3 to 0.9 metres in a 100 year ARI flood, depending on velocity and other factors);
- Hazardous areas where the potential for flood risk to people or flood damage to property is great; and
- Areas that flood relatively frequently (the 10 year ARI event is often used, depending on other factors). For the Merri River and Russell Creek the adopted criteria used to define floodway are described in Table 8.1 and were selected following discussions with Council and the CMA.

Where there is little delineation between LSIO and Floodway, the LSIO was adopted as Floodway, based on Council's recommendations.

In the various reaches of the Merri River and Russell Creek the following criteria have been used for the purposes of preparing the Planning Maps showing the delineation of the floodway areas.

Table 8.1 Floodway delineation criteria

Stream reach	Adopted Criteria	Comments
Merri River	50 year ARI flood extent	Depths at floodway boundary of 0.5 to 0.7 metres in a 100 year ARI flood. Depth at floodway boundary of 1.0 metre in a repeat of the 1946 flood. 20 year ARI flood line is closer to current floodway area, but depths would be 1.1 to 1.3 metres in a 100 year ARI flood.
Russell Creek – Main Channel downstream of Queen Street	50 year ARI flood extent	Depth at floodway boundary of 0.55 metre in a 100 year ARI flood. Very low velocities, backwater from Merri River. 50 year ARI extent is close to the current floodway area.
Russell Creek – Main Channel Queen Street to Mortlake Road	20 year ARI flood extent	Depths at floodway boundary of 0.25 to 0.6 metres in a 100 year ARI flood. Low velocities
Russell Creek – Main Channel Mortlake Road to Racecourse Drain	20 year ARI flood extent	Depths at floodway boundary of 0.45 to 0.65 metres in a 100 year ARI flood. Backwater from Mortlake Road.
Russell Creek – Breakway Area	Generally 50 year ARI flood extent, but limited to road reserve and significant overland flow paths.	The road reserve takes the majority of the flood conveyance and experiences higher flow velocities and flood depths when compared to adjacent properties. FLS model accounted for minimal flood conveyance through properties.
Russell Creek – Main Channel Racecourse Drain to Aberline Road	20 year ARI flood extent	Depths at floodway boundary of 0.5 to 0.6 m in a 100 year ARI flood. Proposed floodway is just contained by existing creek reserve.

9 Flood Risk

9.1 Overview

The flood risk to properties and buildings in the study was assessed with the use of GIS software (MapInfo) linked to the 100 year ARI flood extent and level results from the hydraulic models. Two databases have been prepared:

- The property polygon database which is primarily aimed at providing data to support planning controls; and
- The building polygon database which is primarily aimed at assessing the existing risk of flood damage.

9.2 Property Polygon Database

The property polygon database is based on cadastral property polygons as supplied by Warrnambool City Council. It lists all properties that are affected by (i.e. “touched” by) the 100 year ARI flood and provides representative flood levels for each property. In broad terms, the approach to determining the flood affected properties was as follows:

- The extent of inundation in the 100 year ARI flood was defined as a closed polygon;
- The 100 year ARI flood extent polygon was compared to the property polygons; and
- If the 100 year ARI extent of flooding touched any part of a property polygon it was included in the property polygon database.

Each property was assigned one representative flood level for the 100 year ARI flood, this being the maximum, flood level within the property polygon. The weighted average and minium 100 year ARI flood levels have been included in the data base to indicate the range of modelled flood levels over the property. The database also contains the percentage of the property that is above the 100 year ARI flood.

The property polygon database is primarily used for planning purposes.

The property database has been supplied in an Excel spreadsheet and GIS format.

9.3 Building polygon database

The building polygon database is based on the outline or “footprint” of the building and contains a representative 100 year ARI flood level for each of the identified building polygons. The floor level for all the identified buildings were surveyed. In broad terms, the approach to identifying the buildings at risk was as follows :

- Each building located on a flood affected property polygon (refer to section 9.2 above) was assigned one representative level for the 100 year ARI flood, this being the maximum flood level within the building polygon;
- The floor level for each of the buildings was compared to the representative 100 year ARI flood level to determine which floors are flood prone in the 100 year ARI flood

- The weighted average and minimum 100 year ARI flood levels have been included in the database to indicate the range of modelled flood levels across the building footprint.

The building database has been supplied in an Excel spreadsheet and GIS format.

9.4 Risks to Properties and Buildings

If a property is inundated by the 100 year ARI flood extent, it could mean several things:

- The property slopes and only a small portion of the property is touched by the 100 year ARI flood with the remainder of the property and the building being above the 100 year ARI flood;
- The property is generally flat and is subject to shallow inundation in a 100 year ARI flood, with the building floor above the 100 year ARI flood;
- The property is subject to moderately deep flooding and the building is lower than the 100 year ARI flood; or
- A variety or combination of the above.

Results from the flood risk assessment are provided in Table 9.1 below. It should be noted that the reason the number of flood prone buildings is smaller than the number of flood prone properties is because many of the flood prone properties are vacant.

Table 9.1 Risk to Properties and Buildings

Description of area	Number of Properties at Risk of Flooding	Number of Buildings in the Building Database	Number of Buildings at Risk of Flooding i.e. with Floors below the 100 year ARI Flood
Region A – Russell Creek[1]	244	152	34
Region B – Russell Creek[1]	223	190	83
Region C – Russell Creek[1]	77	61	26
Region D – Russell Creek[1]	92	79	29
Region E – Russell Creek[1]	58	55	33
Russell Creek sub-area (excluding Region A – E)	127	37	16
Merri River sub-area	180	84	27
Study area total	**1001**	**658**	**248**

1. Refer to Figure 7.6 for the Russell Creek Regions

Figure 9.1 Russell Creek Breakaway sub-area boundary

It should be noted that the **number of buildings** reported as being at risk has changed since the May 2003 version of this report (ref. 1). In May 668 buildings were in the building database and there are now 658 buildings in the building database. The reasons for this change is that, on average, the representative flood levels in the database have decreased.

It should be noted that:

- There are buildings in Region B (refer to Figure 7.6) that are now in the building database that were not previously in the building database because flood levels in the region increased, refer to section 7.8 above; and
- Flood mitigation works at Mortlake Road have the potential to limit the breakaway flow through Evelyn Crescent. If the 100 year flood level could be reduced by approximately 0.6 m, this would prevent breakaway flows up to approximately the 100 year ARI event. Several consequences would arise : (1) the 286 buildings and 383 properties in the sub-area shown above could be removed from the 100-year extent for Russell Creek, and (2) the 106 buildings located in the breakaway area with floor levels below the 100 year ARI flood level would no longer be flood prone, and (3) properties located in region B (refer to Figure 7.6) would be subject to slightly higher flood levels because all the flow would flow down Russell Creek and none in the breakaway, and (4) properties located on the left (south) bank of Russell Creek that are not in the breakaway area, would be subject to lower flood levels.

9.5 Changes to Representative Levels in Databases since May 2003 Report

Since the May 2003 version of this report was issued, there have been several changes to the assessment of flood risk. Changes have occurred because flood levels have changed (refer to section 7.8 above) and a property database has been introduced. A description of changes on a region by region basis is provided below.

Regions A and C (refer to Figure 7.6): Modelled flood levels in regions A and C have decreased, and all representative flood levels in the building database have decreased. Where properties are vacant (no building present) they were not and are not in the building database, but where such properties are flood prone they have now been included in the property database.

Region B: Modelled flood levels in region B have increased slightly, resulting in an increase in representative flood levels in the building database. The extent of inundation for the 100 year ARI flood has increased resulting in additional properties, and hence buildings, being identified as potentially flood prone. Additional floor survey has been carried out in this region to determine whether building floors are above or below the 100 year ARI flood. Where properties are vacant (no building present) they were not and are not in the building database, but where such properties are flood prone they have been included in the property database.

Regions D and E, the Merri River and the remainder of the Russell Creek study area: Modelled flood levels in these regions have not changed, therefore the representative flood levels in the building database will remain the same as presented in the May 2003 version. Where properties are vacant (no building present) they were not and are not in the building database, but where such properties are flood prone they have been included in the property database.

9.6 Risks at Road Crossings

This section of the report provides details of the expected frequency or depth of flooding for each road that crosses the Merri River or Russell Creek within the study area. Photographs are also provided for a number of locations.

9.6.1 Caramut Road Bridge

Caramut Road Bridge crosses the Merri River at the downstream end of the study area. In 1946 the previous timber bridge (Cassady's Bridge) at this location was swept away by floodwaters. The existing bridge is above the 1946 and 100 year ARI flood levels.

9.6.2 Wollaston Bridges

The Wollaston Bridges consist of two structures across the Merri River. The old suspension bridge is no longer used for road traffic. A new bridge has been constructed upstream of the old bridge.

The road on the east side of the bridges acts as a causeway in flood events. The hydraulic modelling indicates that the road would be inundated in any flood greater than a 5 ARI flood. The new bridge deck is higher than the road and would not be expected to be flooded until a 20 year ARI flood event. The deck of the old bridge is slightly lower than the new bridge and would be flooded in a 10 to 20 year ARI flood.

Anecdotal reports and nearby recorded flood levels indicate that the flood in 1946 had flood water approximately 2.0 metres above the deck of the old suspension bridge. Only the upper parts of the suspension cable support towers were reported as being visible in 1946.

Old Wollaston Bridge in 2002

In August 2001 a minor flood occurred that resulted in floodwaters getting close to the deck of the Wollaston Bridges as shown in the photograph below. The photograph below was supplied by Rosalie Hay of Warrnambool.

A flooded Merri River – Wollaston Bridge August 2001

9.6.3 Daltons Road

Daltons Road crosses Russell Creek just upstream of the confluence with the Merri River. Backwater effects from the Merri River can flood the road, with floodwater expected to inundate the road for floods greater than a 5 year ARI flood.

9.6.4 Bromfield Street

Bromfield Street is constructed close to ground level across a wide floodplain in the lower reaches of Russell Creek. A six cell culvert conveys flow under the road.

The hydraulic modelling and anecdotal reports indicate that the road would be inundated more frequently than once every 5 years. Flood water would be approximately 3.0 metres deep over the road in a 100 year ARI flood.

9.6.5 Queen Street

Queen Street is constructed close to ground level across a wide floodplain in the lower reaches of Russell Creek. A twin culvert conveys flow under the road. The hydraulic modelling indicates that the road would be inundated in any flood greater than a 5 year ARI flood. Flood water over 1.0 metres deep could be expected over the road in a 100 year ARI flood.

9.6.6 Mortlake Road (Hopkins Highway)

Mortlake Road is an elevated road embankment with a large corrugated metal culvert to convey Russell Creek under the road. The FLS hydraulic modelling (refer to section 7.7) indicates that the road would be inundated in a 100 year ARI flood approximately 500 m north of where the road crosses the creek.

Possible issues with this road crossing include the risk of blockage, the height of the road embankment compared with upstream properties and safety risks associated with the access paths through the culvert and across the creek.

The 100 year ARI flood level upstream of the Hopkins Highway is approximately 1.35 metres higher than the downstream flood level. This large increase in flood level is due to the limited culvert capacity and high road embankment. The adverse impacts of the higher flood level include flooding of existing residential properties and breakaway flows to the north. Further consideration of the culvert capacity should be considered in any floodplain management study or flood mitigation works.

The breakaway flow to the north of Russell Creek was modelled using Delft FLS together with HEC-RAS, refer to section 7.7.

9.6.7 Garden Street

Garden Street crosses Russell Creek just upstream of the Warrnambool Racecourse drain. Four culverts convey water under the road.

The hydraulic modelling indicates that the road would be inundated in any flood greater than a 5 year ARI flood.

9.6.8 Whites Road

Whites Road is at the upstream end of the urban areas in the Russell Creek catchment. A single culvert exists under the road. The hydraulic modelling indicates that the road would be inundated in any flood greater than a 5 year ARI flood.

Downstream subdivision has created a low point on the corner of White Road and Kielli Drive. Any flow over Whites Road may affect residential properties.

10 Mapping Output

10.1 Hard Copy Mapping

The flood mapping consists of both **Flood Data** and **Flood Planning** maps, which are listed in Table 10.1. The maps have been produced in colour on A1 sized sheets and an A3 copies has been included in Appendices I and J.

The information shown on the two sets of maps is listed below:

Flood Data Maps

The flood data maps are intended for use in showing the design flood extents and other technical information for interpretation by the responsible authorities. They include the following information:

- 5, 10, 20, 50 and 100 year ARI flood extents;
- Flood extent for repeat of 1946 flood (with existing surface levels);
- HEC-RAS model cross section locations and 100-year flood levels; and
- Ortho photograph (aerial photographs adjusted to fit survey coordinates).

Flood Planning Maps

The flood planning maps are intended for incorporation into the Council's Planning Scheme. They show the following information:

- Property details;
- Proposed Land Subject to Inundation area; and
- Proposed Floodway area.

Table 10.1 List of Maps Produced

Type	Map No	Title	Scale
Flood Data	530 000-001	Key Map	125 000
Flood Data	530 000-002	Merri River – Warrnambool	5 000
Flood Data	530 000-003	Lower Russell Creek & Merri River - Warrnambool	5 000
Flood Data	530 000-004	Upper Russell Creek & Tributary - Warrnambool	5 000
Flood Planning	530 001-001	Key Map	125 000
Flood Planning	530 001-002	Merri River - Warrnambool	5 000
Flood Planning	530 001-003	Lower Russell Creek & Merri River - Warrnambool	5 000
Flood Planning	530 001-004	Upper Russell Creek & Tributary - Warrnambool	5 000

10.2 Digital Data

Digital mapping and other digital data from the Study was supplied to Glenelg Hopkins CMA as follows:

- MapInfo data, including flood extents, water surface level (WSL) contours, property and building databases;
- Data in DXF format;
- Ortho photographs;
- Survey Data (plus hardcopy survey maps);
- Survey Contours at 200 mm intervals;
- Property Database;
- Building Database;
- PDFs of Maps and Report;
- HEC-RAS models; and
- RORB model for Russell Creek.

11 Conclusions

This report concludes that:

1. The 1946 flood in the Merri River had an Average Recurrence Interval of at least 500 years.
2. Hydraulic modelling of the Merri River has been able to reproduce the observed flood levels from the 1946 flood.
3. Modelling has been completed to determine design flow rates and flood levels for the Merri River and Russell Creek for 5,10, 20, 50 and 100 year ARI floods.
4. Estimated 100 year ARI flood levels along the Merri River are approximately 0.5 metres lower than the 1946 flood levels.
5. Detailed flood data maps and hydraulic models have been produced as part of this study and supplied to the Glenelg Hopkins Catchment Management Authority.
6. Flows and flood levels in the breakaway from Russell Creek located upstream and downstream of Mortlake Road have been modelled using a 2D hydraulics model.
7. Flood planning maps have been produced for the Glenelg Hopkins Catchment Management Authority and Warrnambool City Council for possible inclusion in Council's Planning Scheme.
8. 1001 **properties** within the study have been identified as being affected by the 100 year ARI flood extent. Each property has been assigned a representative flood level, this being the maximum flood level with the property polygon. All flood affected properties have been included in the **property database**, which has been supplied in an Excel spreadsheet and GIS format.
9. 658 **buildings** within the study area have been identified as being potentially at flood risk, however only 248 buildings have floor levels below the 100 year ARI flood. All buildings at risk of flooding have been included in the **building database**, which has been supplied in an Excel spreadsheet and GIS format.

It should be noted that :

- In a 100 year ARI flood, flooding may occur beyond the extents shown for a variety of reasons including local drainage and landscaping details;
- Similarly, a 100 year ARI flood may not inundate all the areas shown as flood prone;
- The databases provide a valuable assessment of the mainstream flooding issues. Assessment of individual properties with respect to flooding should take into account a range of issues, not just those associated with the mainstream flood.

Appendix A

Intensity Frequency Duration Table

Appendix B

Synthesised Streamflow Graphs

Appendix C

Flood Frequency Analysis Graphs

Appendix D

RORB Catchment Diagram

Appendix E

RORB Catchment File

Appendix F

Plan of HEC-RAS model

Appendix G

HEC-RAS Cross Sections

Appendix H

HEC-RAS results tables

Appendix I

Flood Data Maps – A3

Appendix J

Flood Planning Maps – A3

Appendix K

Hydraulic Models

Hydraulic Models

GHD has several hydraulic models at its disposal that could be used on any given project. A hydraulic model is one that determines flood levels from geometric and flow data. A selection of these are described below.

HEC-RAS is a 1-dimensional hydraulic model that uses cross-section data and a flood flow to determine the flood level in a waterway. It works best in situations where the flow is confined to one flow path, such as a defined creek with a defined floodplain. HEC-RAS is relatively easy to use, runs very quickly (in seconds), has excellent graphics and presentation of results. HEC-RAS has limited ability to model splits in the flow path and is therefore not suitable for complex floodplains where there are multiple splits or bifurcations, such as in the as the Loddon River, Victoria. In order for the flood levels to be reliable, a HEC-RAS model should be calibrated using a recorded flow and associated flood levels. Until recently, HEC-RAS was only able to run in steady state mode and was therefore unable to determine attenuation of a flood hydrograph. HEC-RAS can now be run in unsteady state mode and, in theory, could be used to determine attenuation of a flood hydrograph and to evaluate the effects of changes in floodplain storage.

Extran UDD32.XP (UDD) is a branched 1-dimensional hydraulic model that uses cross-section data of a creek or flow path and flow data to determine flood levels. UDD is similar to HEC-RAS but has additional features including the ability to model flow in pipes, run in unsteady state mode, advanced flow splits, etc. Because it can model flow splits well, it can be considered to be a quasi-2-dimensional model. By running in unsteady state, UDD is able to evaluate the attenuation of a flood hydrograph as it flows down a floodplain. Calibration of a UDD model requires recorded flow data and the associated recorded flood levels. GHD has used UDD extensively to flood map urban areas in Melbourne.

Delft FLS (FLS) is a 2-dimensional hydraulic model that uses a regular (square) grid of terrain data coupled with flow data to determine flood levels. Because FLS is a 2-D model, it automatically works out where the flow paths are and where the water flows within the area being modelled. Calibration of an FLS model requires a recorded flow at the upstream boundary of the model and recorded flood levels. Where flood levels are not available, an FLS model can be calibrated using a recorded flood flow and aerial photography of the extent of inundation. In practise, effective calibration can be difficult because the models tend to be large with long run times. FLS can be run in unsteady state so it is able to determine the attenuation of a flood hydrograph down a floodplain. FLS works well in situations where the area being modelled is moderately uniform in terms of its relevant detail. For example, FLS is well suited to modelling a large river and floodplain such as the River Murray, but can also be successfully applied to urban situations if the grid resolution is fine enough to reflect the streets and houses. FLS is at a disadvantage in situations where the creek is narrow and the floodplain is very wide because of its regular square grid, i.e. to accurately model the creek, a fine grid is needed but the software has an upper limit to how many grid points it can accommodate. The larger the model, the longer it takes to run and large models can take up to 24 hours to run. GHD has used, and is currently using, FLS on several projects, both urban and rural.

Deflt SOBEK Overland Flow (SOBEK) is a combined 1D / 2D model. It combines the advantages of 2D models, i.e. the ability to automatically determine how the flow splits in a floodplain, with the advantages of a 1D model, i.e. the ability to robustly determine flood levels in a simple creek or river using cross-sections. SOBEK can run in unsteady state and can therefore determine attenuation. As a hydraulic model, SOBEK is well suited to the Loddon River floodplain because of the large area and complex channels and floodplain, i.e. the creeks could be modelled using cross-sections and the floodplains could be modelled as a course grid. However SOBEK requires extensive terrain data, (terrain in the floodplain and cross-sections in the creeks) which may be costly to obtain. Significant modelling effort is required to establish and calibrate a SOBEK model.

GHD Pty Ltd ABN 39 008 488 373

20 Business Park Drive
Notting Hill VIC 3168
T: (03) 9558 8333 F: (03) 9558 8444 E: nghmail@ghd.com.au

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		Name	Signature	Name	Signature	Date