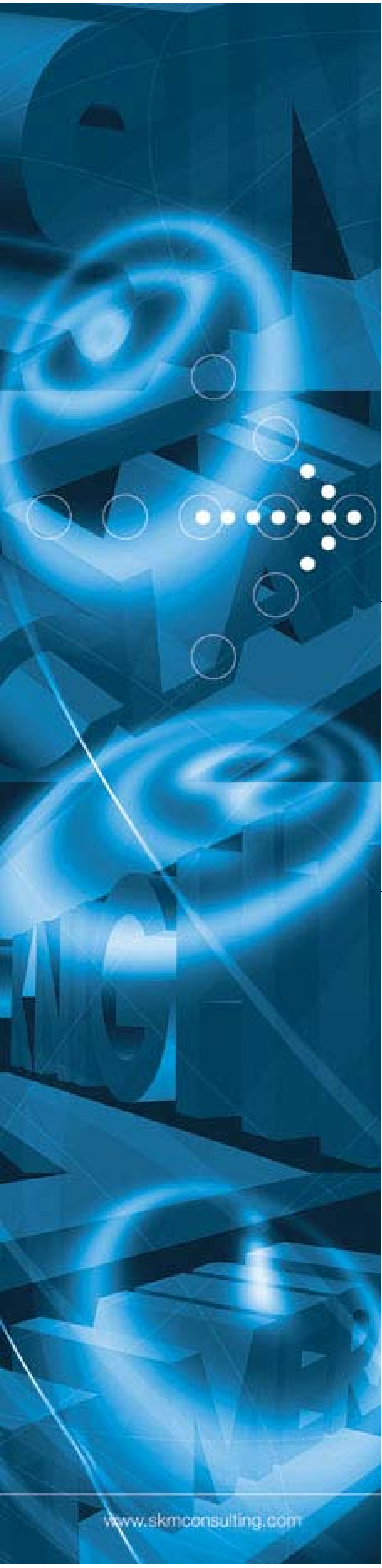

Shepparton Mooroopna Floodplain Management Study

Floodplain Management Plan

Stage 1 Technical Report

October 2002

LAWSON AND TRELOAR PTY LTD
Coastal, Ocean and Water Resources Consulting Engineers

SINCLAIR KNIGHT MERZ

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1. Introduction

Following the Spring 1993 floods, a Scoping Study was prepared (SKM, 1998) that identified the need for a comprehensive study for Shepparton-Mooroopna. In June 1999, Sinclair Knight Merz was commissioned by the Greater Shepparton City Council (GSCC) to undertake a comprehensive floodplain management study for Shepparton-Mooroopna. The Goulburn Broken Catchment Management Authority (GBCMA) has also played a lead role in managing this study. The study forms the basis on which the Floodplain Management Plan was developed.

The main objective of the floodplain management plan is to minimise the economic and social impacts of flooding on the community. It has been achieved through this study by investigating the existing nature of flooding and investigating a range of flood mitigation measures and their merits. The mitigation measures investigated included both structural (eg. levees, floodways) and non-structural options (land use planning, emergency response).

The study was coordinated and guided by a technical steering committee (TSC) comprising representatives from relevant agencies. The committee met throughout the course of the study. Its role was to review work to-date, provide guidance to the consultant, and make resolutions regarding the consultant's findings and study outcomes. A community reference group (CRG) consisting of residents nominated by the community was also formed. The CRG has played a pivotal role of providing feedback on the study direction and outputs during the course of the study.

The plan has been developed in two stages to enable the application of risk management principles. The advantage of this approach is it improves community understanding of existing risks (ie. likelihood and consequences) to allow the community to make informed decisions (eg. selection/approval of risk treatments or commonly known as flood mitigation options) to be made based on a sound understanding of flood risk principles. By streamlining the study, the approach also has the advantages of ensuring decisions are made with all necessary information and in an effective sequence.

The use of the risk management framework is in line with best practice principles as outlined in the Victoria Flood Management Strategy (DNRE/DoJ 1998). Key elements of the two stages are as follows:

- **Stage 1** - Investigation of flooding, determining the likelihood and consequences for existing conditions.
  - Data collection – collection of data relevant to study (eg. topographic information, historical flood levels, etc),
  - Community consultation - providing information to and seeking flood related information from the community,
  - Hydrologic analysis – analysis of streamflow information to assess the likelihood of the floods of a given size occurring (ie, flood peaks and volume),
  - Hydraulic analysis – computer modelling of flood behaviour to estimate flood extents and levels resulting from a given flood under existing conditions.

- Flood damage assessment – assessment of economic damages to the community from flooding under existing conditions,
- Flood mapping for emergency response – mapping on a cadastral base of a range of flood events (output from the hydraulic analysis) to enable improved emergency management and response during floods,
- Planning scheme information – providing GSCC and GBCMA with suitable outputs to aid the revision of the planning scheme related to flooding under existing conditions.

- **Stage 2** – Investigation of measures to reduce economic and social consequences from flooding
  - Community consultation - providing information to and seeking feedback from the community on the existing flooding risks (likelihood and consequences), and possible measures to reduce economic and social consequences from flooding,
  - Preliminary identification and assessment of possible mitigation measures – broad assessment of flood mitigation measures identified through community consultation,
  - Detailed assessment of mitigation measures – assessment includes hydraulic, economic, environmental and social impacts due to mitigation measures,
  - Development of a floodplain management plan for Shepparton Mooroopna.

This report documents the components of **Stage 1** of this study. Documentation of **Stage 2** is provided in a separate report (SKM, 2002a).

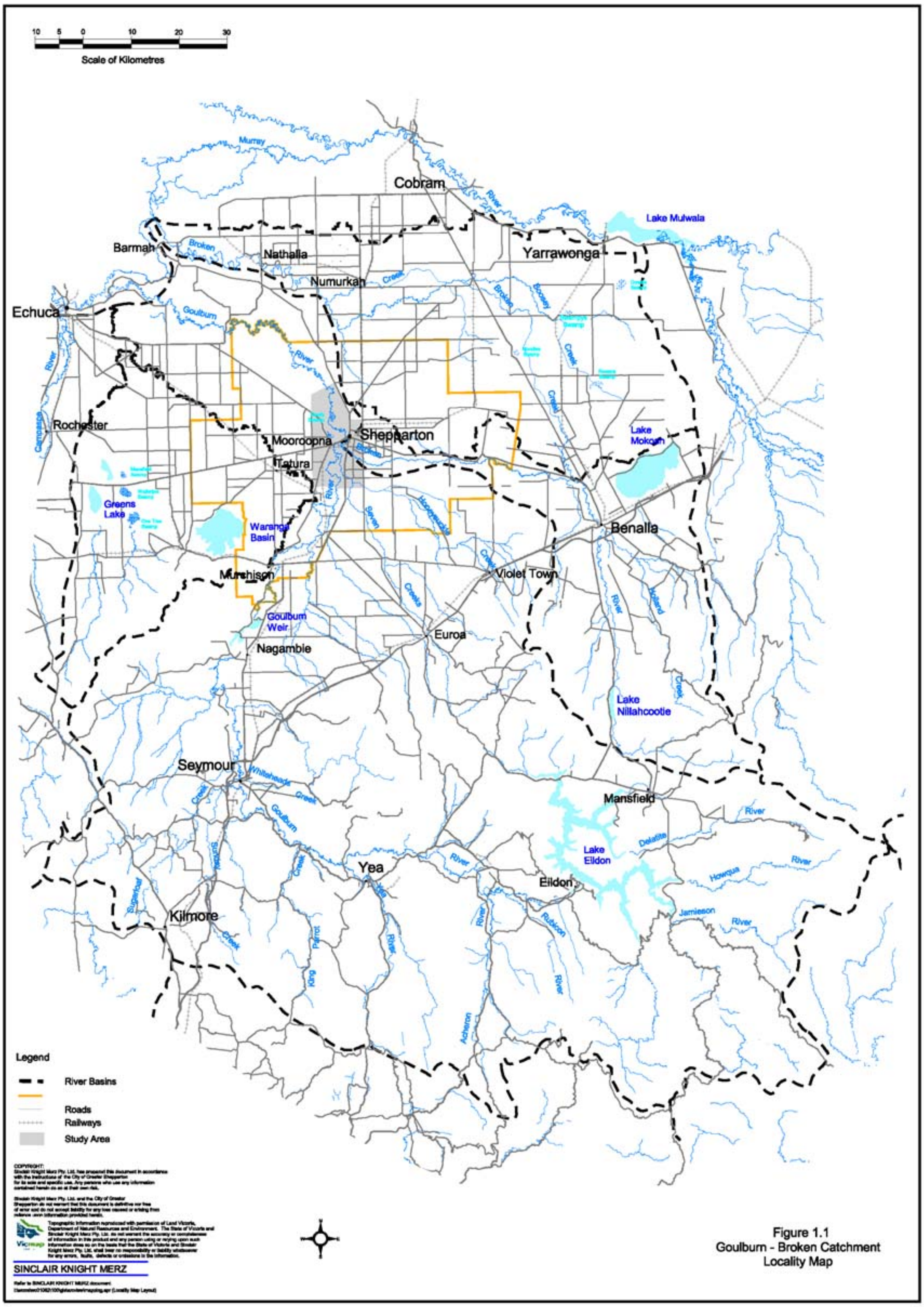
1.1 Description of the Goulburn-Broken Catchment

Shepparton-Mooroopna lies at the confluence of the three main river systems, the Goulburn River, the Broken River and Seven Creeks. Large floods can originate from any one of the three systems or from a combination of the three systems.

The total catchment area to Shepparton is 16,125 km^2. The total Goulburn-Broken catchment is shown in **Figure 1-1**.

The Goulburn River catchment at its confluence with Seven Creeks has an approximate catchment area of 12,000 km^2. The river rises in the Great Dividing Range above Jamieson. The upper catchment flows into Lake Eildon which has a storage capacity of 3,390,000 ML and provides irrigation supplies to a large part of northern and central Victoria. During floods, the storage may reduce flow peaks from the upper catchment. From Lake Eildon to Seymour, several tributaries including Rubicon, Acheron and Murrindindi Rivers join the Goulburn as it flows to the west. From Seymour the Goulburn River turns to flow in a northern direction to the Goulburn Weir near Nagambie. Downstream of the Goulburn Weir, the river continues to flow in a northern direction to Shepparton. Just upstream of Shepparton, Goulburn River is joined by Seven Creeks and the Broken River. Downstream of Shepparton at Bunbartha, the Goulburn flows in a north westerly direction to join the River Murray near Echuca.

- **Figure 1-1 Goulburn-Broken Catchment**



The Broken River rises in the Tolmie highlands and flows to the west before flowing to the north into Lake Nillahcootie. Lake Nillahcootie has a storage capacity of 39,800 ML and is not large enough to have a significant effect on major floods (HydroTechnology 1995a). Holland Creek joins the Broken River just upstream of Benalla. The river continues flowing north until downstream of Benalla where the river turns and flows west to join the Goulburn River. The catchment area of the Broken River at the Goulburn River confluence is 2,510 km^2. During large floods, the flow in the Broken River break out near Casey's Weir to the north and joins the Broken Creek. Further breakouts to the north and south occur during large floods along the Broken River between Casey's Weir and Shepparton. About 10 km upstream of the Broken River's confluence with the Goulburn River, the East Goulburn Main Channel passes under the Broken River via a siphon. The channel causes a constriction in the floodplain and during major floods this constriction results in a ponding of water upstream of the channel. Flood flow may break out upstream of the channel and flow to the south to join Honeysuckle Creek, a tributary of Seven Creeks or to the north to the Broken Creek via a number of tributaries including Pine Lodge, Congupna and Dainton Creeks. The breakouts and the floodplain storage result in a reduction of the peak flow for the Broken River from Benalla to its confluence with the Goulburn River.

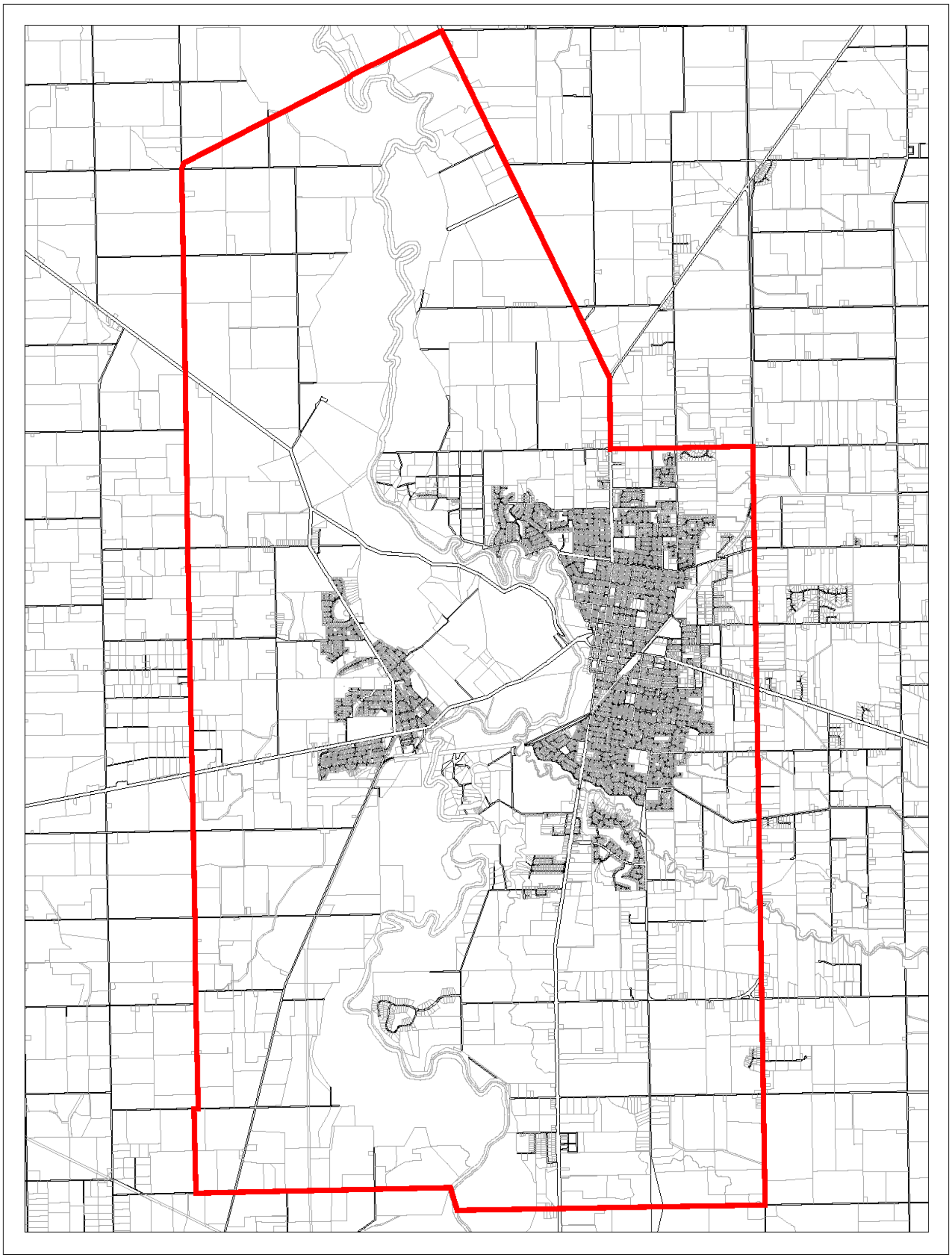
Seven Creeks flows to the north west from the Strathbogie Ranges through Euroa and to its confluence with the Goulburn River. The catchment area of Seven Creeks at the confluence is about 1,550 km^2. Honeysuckle Creek is a tributary of Seven Creeks and joins just upstream of Kialla West. During major flood events in the Broken River, the flow may break out of the Broken River and flow to the south joining Honeysuckle Creek. Some exchange of flow from Sevens Creek to the Broken River may occur during major floods. This exchange occurs downstream of Kialla West.

1.2 Description of the Study Area

The study area covered by the floodplain management plan is shown in **Figure 1-2**. It is bounded by Maneroo Road and Barmah-Shepparton Road to the north, Pogue Road and Moira Drive to the south, Turnball Road and Trotter Road to the west and Euroa-Shepparton Road, Doyles Road and Grahamsvale Road to the east.

The study area is centred on the urban areas and the confluence of the Goulburn and Broken Rivers. It extends north (downstream) to include the Sewage Treatment Lagoons and various irrigation drains in order to identify any role they might have in flood behaviour within the study area. In addition, this area is subject to consideration by VicRoads for a Shepparton-Mooroopna bypass highway. The models developed within the study were also able to assist in assessing the impact of bypass options. The study area extends south (upstream) to include the confluence of the Goulbourn River and Seven Creeks as well as expanding pockets of development in this area.

- **Figure 1-2 Study Area**



1.3 Historical Floods

At Shepparton, the largest floods this century have occurred in 1916, 1939, 1974 and 1993. These were ranked 1, 3, 2 and 4, respectively (HydroTechnology, 1995) (see **Table 1-1**). However, Big Eildon Dam was not present in 1916 and 1939, and would have had some effect in reducing the peaks of those floods. Given that the estimated peak discharges in 1939 and 1993 were very similar, allowance for the effect of Eildon would elevate the ranking of the 1993 flood to the third highest this century.

The effect of Eildon Reservoir in reducing flood peaks has been studied previously (SRWSC, 1981). It was estimated that at Shepparton the impact on flood peaks in large floods is approximately 7%. Nathan (1992) estimated a reduction of 27% in large floods in the Goulburn River at Murchison. The impact is larger in more frequent floods of smaller magnitude, and the impact is also greater further upstream near Eildon. The effect diminishes downstream because of the effect of unregulated tributary inflows.

There is also fairly clear indirect evidence that a flood larger than any this century occurred in 1870. Although there were no gauges operating on the Goulburn at that time, the Murray River at Echuca peaked much higher in 1870 (and in 1867) than in 1916. It should be noted that the effect on flooding at Echuca from the Murray River downstream of Barmah is restricted by the effect of the Bama Samdhills, so that little more than the "choke" capacity of approximately 35,000 ML/d can pass along the Murray without forcing additional flow north along the Edward River into NSW. Therefore, the magnitude of flood peaks at Echuca above this capacity is very dependent on the magnitude of flows received from the Goulburn and Campaspe Rivers, and to a lesser extent the Broken Creek.

A comparison of the highest ranked floods last century is presented in **Table 1-1** for the Goulburn River at Shepparton. A continuous recorder has operated at this location since 1939. A staff gauge was observed daily from 1921 to 1939.

- **Table 1-1 Magnitudes and Ranking of Major Floods at Shepparton**

Flood/Year	Peak Discharge	Rank
1916	233,300	1
1974	214,000	2
1939	161,000	3
1993	160,500	4
1956	121,000	5
1934	118,400	6
1975	105,000	7
1924	103,300	8
1958	103,000	9
1921	97,500	10

The May 1974 and October 1993 floods were the focus for historical data for this study.

1.4 Previous Studies

Attention to and analysis of the floodplain management problems in the Shepparton - Mooroopna area has occurred over an extended period. This commenced with a Flood Study undertaken by Sinclair Knight & Partners and reported in 1982 (SKP, 1982). Little effective action has been undertaken largely because of a lack of consensus and the lack of an integrated strategy for flood management amongst the then three local government jurisdictions. In some cases, alternative measures have been initiated such as informal levee construction in Kialla West, purchase of properties in Riverview Road and isolated structural works and floodway zoning in The Boulevard area, but these measures have not compromised elements of an integrated floodplain management plan.

The 1982 Flood Study arose after flooding problems in the flood of May 1974, which has been determined to have an Annual Exceedance Probability (AEP) of approximately 1.4% (ie. ~ 75 year ARI) in the Goulburn River at Shepparton. This current study arose from the flooding of October 1993 and the desire to complete the floodplain management process begun with the 1982 study. The flood in October 1993 has an AEP of approximately 3% (ie. ~33 year ARI) in the Goulburn River at Shepparton. Flooding from the Broken River and Seven Creeks was more severe in 1993, but flows from the mid-Goulburn were lower than in 1974.

2. Community Consultation

Community consultation was a significant component of the study process in Stage 1. The community consultation was conducted via the resident survey (ie. questionnaires), community reference group briefing sessions and media releases.

A resident survey/questionnaire was distributed in October 1999 to gather flood knowledge, information and preliminary feedback on the study outputs. The questionnaire was distributed to 18,000 properties with a target of approximately 12,000 residents. Responses were received from 941 residents (8% of total target).

As part of the questionnaire, residents were asked to provide information regarding the location of historical flood marks within the study area. Approximately 300 references to flood marks were provided. Where possible these flood marks were surveyed as part of the data collection phase. In addition, a collection of video, flood photography and resident interviews were carried out.

As part of the first questionnaire, residents were asked to nominate suitable persons for a community reference group (CRG). In total of around 135 nominations were received.

Appendix A contains a copy of the Stage 1 questionnaire.

A second questionnaire was distributed in June 2001, for Stage 2. This is discussed in the Stage 2 report.

There have been five briefing sessions conducted with the CRG during the study (both Stage 1 and Stage 2).

3. Data Collection

3.1 General

The execution of this type of study generally demands the collection of large amounts of data for both the catchment and the study area. Information was available from previous studies, surveys or external sources but was generally deemed unsuitable or dated. Therefore, a significant amount of new data was collected for this study via new field surveys.

3.2 Existing Data Sources

The primary data sources or types of existing data for this study are summarised as follows.

The key reports available were Documentation and Review of Victoria Floods - Broken River Catchment Floods October 1993, Volume 4 (HydroTechnology, 1995), Documentation and Review of 1993 Floods Lower Goulburn, Volume 5 (HydroTechnology, 1995), the Shepparton-Mooroopna Flood Study (SKP, 1982) and Shepparton-Mooroopna Flood Scoping Study (SKM 1998).

The key plans used are summarised in **Table 3-1**. These were obtained from Goulburn Broken Catchment Management Authority (CMA).

- **Table 3-1 Summary of Existing Plans**

Plan Title	Plan sub-title	Sheets	Date	Data Type
SR&WSC River Survey & Flood Study, Shepparton-Mooroopna Area	Goulburn River	1-14	1977	Cross Sections
SR&WSC River Survey & Flood Study, Shepparton-Mooroopna Area	Broken River	1-10	1977	Cross Sections
SR&WSC River Survey & Flood Study, Shepparton-Mooroopna Area	Seven Creeks	1-4	1977	Cross Sections

Note that existing 1981 100 mm contour plans of about one third to one half of the study area were available and considered. However, they were deemed unsuitable primarily due to their age, their limited coverage and the change in levels. It was concluded that the limited ground survey could no longer represent conditions that need to include current features such as laser grading which has occurred over much of the area, embankments and any other new works. The data available via photogrammetry of the entire study area, even at 200 mm accuracy in the outer areas, were considered superior to and used in preference to these plans.

Available flood level data were obtained in digital form from the Goulburn Broken CMA. These data had been collated under the Flood Data Transfer project conducted by the Department of Natural Resources & Environment (NRE). They covered a number of historical events over the last century from 1916 to 1993.

Aerial photograph mosaics of the 1993 floods were bound within the HydroTechnology (1995) report. Full scale sets of these photographs and photographs from the 1974 event were also provided by the Goulburn Broken CMA.

Through the community consultation process (discussed further in **Section 2**), a collection of miscellaneous photographs, videos and reports of the 1974 and 1993 floods was obtained via questionnaires and subsequent interviews.

These data provided an excellent source of historical and current data for the study.

The study also required the collection of a number of new types of data never previously collected. These requirements and the data are discussed in the following section.

3.3 New Data Sources

The new data sources or types collected for this study were as follows:

- aerial photography
- photogrammetry of the entire study area,
- structure data
- field data of the Causeway and key irrigation channel banks
- questionnaires,
- property data,
- flood level data,
- cadastral data,
- streamflow records from gauges within the catchment.

Aerial Photography was undertaken to provide an overview of the key features of the floodplain.

Photogrammetry, conducted by AAM Surveys, was undertaken to provide surface data (ie. ground surface information) across the entire study area. The study area was subdivided into “inner” (largely built up) and “outer” (largely rural) areas, primarily to define areas requiring ±100mm and ±200mm accuracy respectively. The area was flown and photographed on 10 and 11 September 1999. Processing of the photographs produced thousands of “contourable” and “non-contourable” data sets. The contourable data contain surface elevation data and include spot levels and “breaklines”, the latter defining linear features such as ridges, embankments, and drains. The “non-contourable” data are surface features that do not contain or need elevations and include water body outlines and building and crop boundaries.

Structure data was surveyed to capture information on culverts and bridges on rural roads and the rail embankment and on culverts and syphons under irrigation channels and drains. Importantly, this survey was used to capture only the size/configuration and location of these structures. Elevations for these structures were estimated from the surface data obtained from the photogrammetric survey. This survey did not overlap with data obtained from VicRoads, which only retains information for structures owned by VicRoads and with a cross sectional area greater than 6m^2. The survey of the irrigation channels and drains was guided by information obtained from Goulburn-Murray Water (G-MW) and thus was used to confirm and expand G-MW’s database records.

Field data was collected to provide more accurate surface information for three features with the potential to act as significant hydraulic controls – namely the Causeway, and the main Eastern and Western Irrigation Channels (in the northern reaches of the study area). It was believed that levels obtained for these features from photogrammetry alone would not be sufficiently accurate to properly represent their function as hydraulic (flow) controls. For the Causeway, both bridge opening dimensions and road levels were surveyed. For the irrigation channel, tops of channel banks were surveyed.

Questionnaires and associated information were developed and used as a means of communicating with the wider community. Their purpose, content and responses are further discussed in **Section 2**. For data collection they were used to identify the location of historical flood level data (ie. flood marks) for surveyors to then visit and record.

Property Data, was collected to establish a database of all properties expected to be affected by the 100 year ARI flood event (estimated at the time of the survey). The survey for each property included address, position, building type, size, condition, floor level and one representative ground level. Positions were determined using GPS units or by manual dead-reckoning via a digital cadastre. Positions were determined for the floor level (indicating building position) and the ground level (usually taken at the front yard of the property). A total of 9,554 properties were surveyed. The collected property data was collated and transformed into a Geographic Information Systems (GIS) based database. Importantly, a key feature of both the survey and this database is the use of identification numbers for each property that matched the existing identification numbers used in Council's rating database system. This will enable Council to automatically merge the two datasets and add the surveyed property data to its rating database.

Flood Level Data was necessary to enable calibration of the hydraulic model. Flood levels for the 1974 and 1993 floods were of particular interest for calibration. There were two sources for this data. First, pre-existing data (elevation and position) was available from the Flood Data Transfer Project (DNRE, 2000). That project catalogued a number of flood levels from a number of historical events. Secondly, via the questionnaire noted above, a new call for flood levels from the public. The flood levels were surveyed in tandem with the property data, using the same techniques. Additional records of each flood level (ie. a photograph and locality sketch) were compiled. **Figure 3-1** shows the new flood levels collected from this study.

Cadastral data is used primarily as a map base. A digital cadastre of the study area was supplied by the Greater Shepparton City Council in April 2002. The cadastral data included property boundaries, labels of selected local landmarks, street names and street numbers for each property.

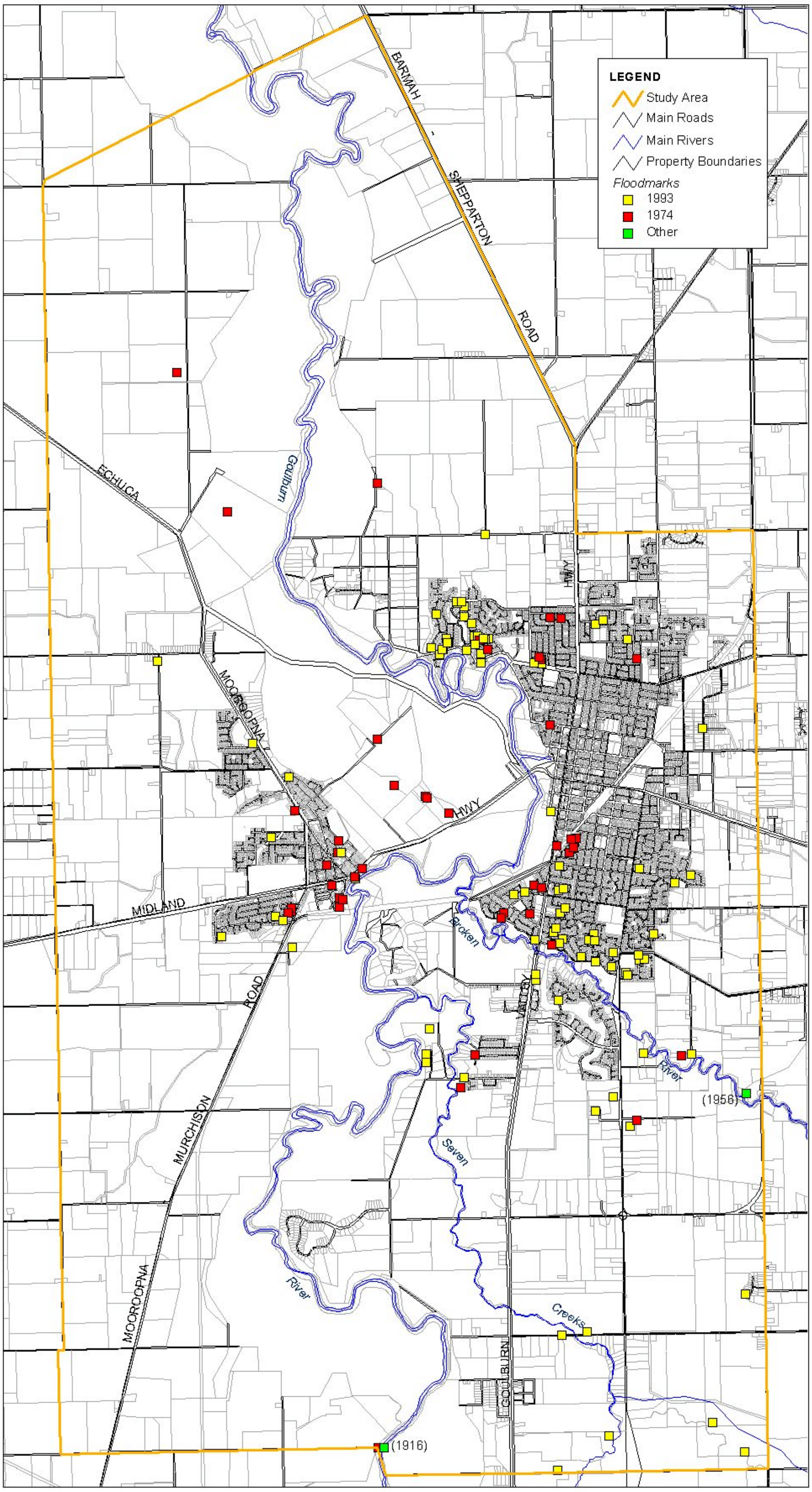
Finally, all available relevant **streamflow records** were collected from the Goulburn Broken catchment. These records were used in the hydrologic analysis to generate design (ie. 5 to 500 year ARI) flood hydrographs. These records and their uses are further discussed in **Section 4.**

3.4 Survey Documentation

The data collected and selected pre-existing data used for this study has been brought together into a single Project CD-ROM. This is an effective way to ensure data is not lost and remains readily accessible after completion of the study. The CD-ROM also includes study outputs, in particular flood inundation maps and associated listings (see **Section 9**). There is an opportunity to continue to update the CD-ROM as additional data or relevant flood information becomes available.

The CD-ROM contains a customised ArcView Project File which guides the user to the various data in contains. It also contains a User's Guide.

■ **Figure 3-1 New Flood Levels**



3.5 Digital Terrain Model (DTM)

The surface levels and breaklines obtained from photogrammetry and field survey (both described above) were the sources of data for the Digital Terrain Model (DTM) of the study area. The DTM simply links all the available surface data to form a computer representation or "model" of the elevation and form of the "terrain" across, in this case, the full study area. The DTM is fundamental to the hydraulic modelling (see **Section 5**) and flood inundation mapping (see **Section 9**).

The accuracy and reliability of the DTM is simply a function of the accuracy and reliability of the source data – the photogrammetry and field survey. Photogrammetry in particular, whilst highly effective, does have limitations in floodplain management applications which need to be recognised in using it as a source of data. First, there are limitations associated with visibility or lines of site to the ground. Tree cover can reduce the density of ground data points that can be extracted from the aerial photography. In Shepparton-Mooroopna, tree cover was most significant in the immediate floodplain areas. Fortunately, it was found to be relatively low-density cover and did not affect the extraction of ground data points. A related feature is high crop cover. In these areas, dense high crops obscure the ground more fully. As a result, a lower accuracy of extracted ground data must be expected and accepted. These areas were highlighted and coded in the source photogrammetry to flag this limitation, which would be carried through to the DTM. Fortunately there are not many such areas and they lie in outer, less critical areas. A final related feature is building cover. Obviously buildings obscure the true ground levels. This can become problematic for large buildings.

Secondly there are limitations associated with rivers and water features. River banks are too steep to be effectively picked up by photogrammetry. Furthermore, any water in the river (or any water body) will obscure bed levels. As a result, photogrammetry is rarely reliable within river banks. In these instances, defining levels along the top of the river bank is particularly important. For this study, river bed levels were obtained from existing cross sections (see **Table 3-1** above). The DTM was then modified during the hydraulic modelling to reflect true river levels and capacities.

In addition to such limitations, care needs to be taken in the development of DTMs from mixed data sources. Ground data sources that are similar at first glance can differ for reasons of survey technique, accuracy, age, and position. An example if this and critical finding from the survey work for this study was the difference between surface levels obtained via the photogrammetric survey and the property survey.

Both surveys were accurate and appropriate for the purposes for which they were intended. However, comparisons of the two data sets revealed significant differences. The source of these differences was partly the different level of accuracy specified for each survey, but more importantly, the different survey techniques used. Taking as an example a residential block (ie. the most common location where property data was collected), the photogrammetric survey typically extracted one surface level at the front and back boundaries of the block. Surface elevations within the block could then be derived only by interpolation between these levels. In contrast, the property survey captured a single level representative of the entire block, usually taken in the front yard. Comparisons of these levels often show significant differences, despite each level being legitimate for the purposes for which it was intended – photogrammetry to produce a DTM of the study area and property data to determine flood damages and representative flood depths at each property. The key conclusion from this finding was that the data sets should not be merged.

This study collected a massive amount of useful and reliable ground data from a variety of survey techniques. Clearly, however, care needs to be taken in the application and mixing of various data sets.

4. Hydrologic Analysis

4.1 Outline of Hydrological Analyses

This section details the hydrological analyses undertaken as part of this study. The hydrological analyses undertaken for this study involved the following tasks:

- Derivation of design flood hydrographs for the Goulburn and Broken Rivers, and Seven Creeks at the upstream study area boundary. Design hydrographs are required for the 100 year ARI event and events corresponding to gauge increments at Shepparton, approximately ranging from 5 to 500 year Average Recurrence Interval (ARI).
- Preliminary assessment of the Probable Maximum Flood (PMF).
- Construction of historical flood hydrographs at the upstream boundary of the study area for the Goulburn and Broken Rivers, and Seven Creeks for use in the hydraulic model calibration.

4.2 Adopted Design Flood Estimation Methodology

The adopted streamflow-based approach incorporates the following four components:

- Estimation of design peak flow for the required range of ARIs using flood frequency analysis,
- Estimation of design flood volume for the required range of ARIs using flood frequency analysis,
- Determination of design hydrographs for the required range of ARIs,
- Determination of critical combinations of flows from the Goulburn and Broken Rivers and Seven Creeks.

The details of the streamflow gauging stations used in this analysis are listed in **Table 4-1**. These streamflow gauging details include the period of continuous streamflow record for each gauge. The continuous period of record is the period of systematic recording of streamflow via a daily read staff gauge or a continuous recorder. For some streamflow gauges, records are available during flood events only.

Table 4-1 Streamflow Gauges used in this Analysis

Station Number	Station Name	Catchment Area (km^2)	Period of Continuous Streamflow Record
404200	Broken River at Casey's Weir (Goorambat) Tailwater Gauge	1924	July 1916 to June 1979
404216	Broken River at Casey's Weir (Goorambat) Headwater Gauge	1924	February 1888 to June 1916. July 1979 to date
404222	Broken River at Orrvale	2508	June 1977 to date
404203	Broken River at Benalla	1461	Oct 1977 to date
405200	Goulburn River at Murchison	10772	June 1881 to March 1967 November 1984 to date
405253	Goulburn River at Goulburn Weir	10627	March 1967 to October 1985
405270	Goulburn River at Kialla West	12038	June 1977 to August 1985
405204	Goulburn River at Shepparton	16125	June 1921 to date
405237	Seven Creeks at Euroa Township	332	May 1963 to date
405269	Seven Creeks at Kialla West	1505	June 1977 to date

Examination of available streamflow data showed the following streamflow gauging stations are located on the Goulburn and Broken Rivers and Seven Creeks near the upstream study area boundary:

- Goulburn River at Kialla West (gauge number 405270),
- Broken River at Orrvale (gauge number 404222),
- Seven Creeks at Kialla West (gauge number 405269).

The above gauges are well located for study purposes. However, the length of available streamflow record at the above stations is insufficient to employ flood frequency analysis for estimation of peak flows and flood volumes over the required range of ARIs.

There are four gauges located some distance upstream of the study area on the Goulburn and Broken Rivers and Seven Creeks. These gauges have long streamflow records and the details of these gauges are listed below:

- Goulburn River at Murchison (gauge number 405200),
- Broken River at Casey's Weir (gauge number 404200/404216)
- Broken River at Benalla (gauge number 404203)
- Seven Creeks at Euroa (gauge number 405237)

These stations have sufficient record to allow the use of flood frequency analysis for estimation of peak flows and volumes. As noted previously, the design estimates for the higher ARIs (greater than 50 years) will have an increased uncertainty.

Further analysis showed the inaccuracies in recorded high flows at Casey's Weir, which affect the results from the peak flow frequency analysis. Due to these inaccuracies the streamflow gauge on the Broken River at Benalla was used instead of the Casey's Weir gauge.

Further discussion of the streamflow gauges and the quantity and quality of available streamflow data is given in **Section 4.3**.

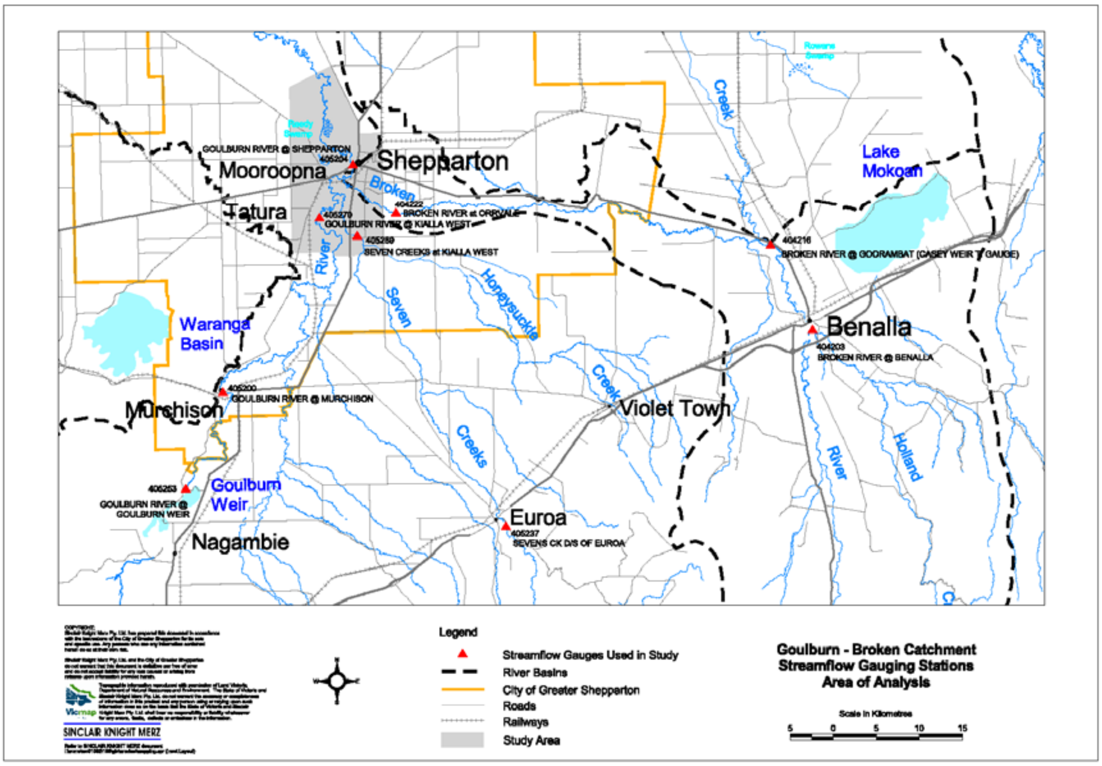
Figure 4-1 displays the location of the streamflow gauges used in this analysis.

4.2.1 Design Peak Flow Estimation

A flood frequency analysis is undertaken on the annual instantaneous peak flow series at the upstream gauges listed in **Section 4.2**. This analysis yields design peak flows for the required range of ARIs.

Previous studies for Benalla (Willing and Partners 1998) and Euroa (SKM 1997) employed calibrated runoff routing models with design rainfalls estimates to obtain design peak flows. ARR87 provides guidance on the relative accuracy of rainfall and streamflow based approaches. A formula is provided to determine the ARI above which the rainfall based estimates are considered more reliable. This ARI is known as the probability of indifference. The probabilities of indifference were determined for both Benalla and Euroa. The previous studies' rainfall based peak flow estimates were adopted for ARIs greater than the probability of indifference. For ARIs less than the probability of indifference, the design peak flow estimates were obtained from a frequency analysis.

■ **Figure 4-1 Location of Streamflow Gauges used in this Analysis**



Regression equations were developed between the peak flows at the upstream gauges and the gauges adjacent to the study boundary to transpose design peak flow estimates downstream. The reliability of the regression equations is dependant on the length and range of streamflow data used in their development.

Section 4.4 details the design flood volume estimation for each streams and for the required range of ARIs.

4.2.2 Design Flood Volume Estimation

Peak flow and flood volume are both important flood characteristics determining flooding behaviour within the study area. The analysis outlined in **Section 4.2.1** deals with peak flows only. It is necessary to undertake a similar procedure as adopted for the peak flow estimation to estimate flood volume for the required ARIs at the upstream study boundary. The design peak flow and flood volume estimates will then be combined to result in a design flood hydrograph (See **Section 4.2.3**)

As for the peak flows, the design flood volume at the upstream gauges were transposed to the study boundary using a regression equation. The regression equation were developed between the flood volumes at the upstream gauges and the gauges located adjacent to study boundary.

Section 4.5 details the design flood volume estimation for each streams and for the required range of ARIs.

4.2.3 Design Hydrograph Determination

To determine appropriate design hydrographs from the design peak flows and flood volumes, this study adopted an approach outlined in Australian Rainfall and Runoff 1987 (ARR87) (IE Aust, 1987) Chapter 10. The approach was developed by the Tasmanian Hydro-Electric Commission.

The approach is based on the ratio of peak flow to average flow over the flood duration. An assumption underlying the approach is that the peak flow and flood volume can be treated as coming from a single population. That is, the ARI of the combination of the peak flow and flood volume is equal to the ARI of the separate events. This assumption needs to be validated prior to the application of the approach. This validation is undertaken by checking for equally ranked events whether a flood used in the peak flow frequency analysis is the same flood as used in the flood volume frequency analysis. For this study, it was found that this underlying assumption was validated for the three contributing catchments.

For each ARI, the ratio of the peak flow to the flood volume is determined. From the historical streamflow record, an observed hydrograph with a similar peak flow to volume ratio is selected as the representative hydrograph. This hydrograph is then scaled to obtain the required peak flow and flood volume for the ARI.

4.2.4 Design Flood Event Combination

The previous sections outlined the methodology employed to determine the design flood hydrograph at the study boundaries. The brief requires the production of flood inundation maps for each 200 mm increment in gauge height at Shepparton. To achieve this, it is necessary to determine design flood event combinations that resulted in the required gauge height at Shepparton.

Flooding within the study area is due to the combinations of inflows from the three main contributing catchments. The maximum gauge height obtained at Shepparton for a given flood event is depended on the following factors:

- Flood event magnitudes (ARI of peak flow and flood volume) in the three contributing catchments,
- Relative timing of the peak flows from the three contributing catchments.

The relative timing of the peak flows from the contributing catchments is determined from the historical flood events.

Using the design flood hydrographs developed at the study boundary and the relative timing obtained from historical events, a preliminary assessment of design flood event combinations were undertaken using a coarse one dimensional hydraulic model. The hydraulic model routed the design flood hydrographs from the study boundary through the study area to the Shepparton gauge. For this assessment, the design flood hydrographs for the three streams are assumed to have the same ARI.

4.2.5 Preliminary Probable Maximum Flood Estimate

The study brief required a preliminary estimate of the probable maximum flood (PMF). The PMF will be routed through the study area to determine an indicative estimate of additional flood extent and flood level above the 100 year ARI event.

A relationship between the catchment area and the PMF has been developed for catchments in South Eastern Australia (Nathan et al 1994). This relationship is based on previous PMF studies and provides a preliminary estimate of the PMF. However this relationship is not suitable for use in this study due to the breakouts of flood flow from the Broken River to Broken Creek. Also, the catchment area to Shepparton is 16,125 km^2, which is larger than the largest catchment (10,000 km^2) used in the development of the prediction equation.

This analysis used a PMF estimate (SKM 1998) from the Hume Dam catchment to determine estimates of the PMFs for the Goulburn and Broken Rivers, and Seven Creeks at the upstream study boundary. The catchment area for the Hume Dam is of similar size to the Goulburn River to Shepparton (16,125 km^2).

The Hume Dam study estimated the 100 year ARI and PMF inflows to the Dam. The ratio of the PMF to the 100 year ARI inflow for Hume Dam is 8.9 to 1. The same ratio is used to convert this study's 100 year ARI design peak flow rates to PMF estimates. This procedure is used to obtain PMF estimates for the three contributing streams.

4.3 Available Data Review

4.3.1 Overview

For the streamflow gauges listed in **Section 4.3**, a review was undertaken to assess data quantity and quality. This review investigated the reliability of streamflow data from several sources including:

- Shepparton Mooroopna Flood Study (1982), Sinclair Knight and Partners (SKP 1982)
- Documentation and Review of 1993 Victorian Floods (1995) HydroTechnology, (HydroTechnology 1995a & HydroTechnology 1995b)
- Hydsys Streamflow Database - Data provided by Thiess
- Victorian Surface Water Information to 1987 (1990), Rural Water Corporation (RWC 1990)
- Euroa Floodplain Management Study (SKM 1997)

4.3.2 Goulburn River

Murchison

Continuous streamflow records are available from June 1881 to March 1967 and from November 1984 to date at Murchison. No streamflow data was available during the period 1967 to 1984. This period of data was infilled by data from the gauge at Goulburn Weir (405253).

For the Goulburn River at Murchison, there was good agreement between the annual peak flows from three available data sources: SKP (1982), Hydsys and RWC (1990). Data from Hydsys and RWC (1990) were identical for the concurrent period 1881 to 1987. In seven years, distinct differences existed between SKP (1982) data and the other two data sources for the medium range flows. The differences were investigated and found to be due to revision in the rating curves.

Advice from Theiss – Hydrographic Services (2000) indicates the flow measurements have been undertaken across the entire range of observed flows with the highest gauging at 106,000 ML/d. As a result, the rating curve may be considered as good.

Given the good rating curve and the small number of discrepancies, the Hydsys data was adopted for both peak flows and flood volumes by this study.

Kialla West
Streamflow data are available for the period June 1977 to August 1985. Hydsys is the only data source for streamflow at this site. The highest gauging undertaken at the site occurred in July 1981 with flow of 51,300 ML/d gauged. The Hydsys data was adopted for this study.

Shepparton
Four data sources for peak flows SKP (1982), Hydsys (2000), RWC (1990) and HydroTechnology (1995b) were available for the Goulburn River at Shepparton. There was a perfect agreement between RWC (1990) and HydroTechnology (1995b). For about six years in the period (1922–1995) significant differences between the RWC (1990) HydroTechnology (1995b) and the other data sources were found. Possible sources of these discrepancies include rating curve revisions, gauge datum correction and incorrect recording of data.

Advice was sought from Thiess – Hydrographic Services regarding the reliability of the Hydsys flows. Thiess (2000) advised a number of high flow gaugings have been undertaken since the RWC (1990) flows were published. The rating curve had been revised on the basis of these gaugings. As a result, Thiess recommended the use of Hydsys flow as the most reliable data source. This study adopted the Hydsys data for both peak flow and flood volumes.

4.3.3 Broken River

Casey's Weir
Streamflow records for Casey's Weir are available from two adjacent streamflow gauges, the tailwater gauge (404200) and headwater gauge (404216). The combined record from two gauges extends from 1888 to date.

At Casey's Weir, three data sources - SKP (1982), Hydsys, RWC (1990) were available for comparison of peak flows. Significant differences were found for about twenty years. The discrepancies are due to revisions of the rating curve and gauge datum corrections.

Advice was sought from Thiess Hydrographic Services regarding the reliability of the Hydsys flows. Thiess (2000) advised a number of high flow gaugings have been undertaken since the RWC (1990) flows were published. The rating curve had been revised on the basis of these gaugings. As a result, Thiess recommended the use of Hydsys flow as the most reliable data source. This study adopted the Hydsys data for both peak flow and flood volumes.

The rating table is extrapolated for higher flows greater than 29,000 ML/d. Also, at high flows an anabranch on the left bank operates with flow bypassing the gauges at Casey's Weir. This indicates the accuracy of streamflow data above 29,000 ML/d is limited.

Further analysis showed that the inaccuracies in the high flows at Casey's Weir have an impact on the design peak flow estimates (See **Section 4.4.2 *Broken River at***

Casey's Weir). Given these inaccuracies the peak flows from Casey's Weir were not used in the analysis.

Benalla
A staff gauge was located at the site from 1913 to 1977. Prior to 1955 readings were obtained during flood events only. Since 1955 systematic recording of streamflow has been undertaken. A recorder replaced the staff gauge in 1977.

The highest gauged flow was 107,900 ML/d during the October 1993 event. The peak flow in the October 1993 event was 112,000 ML/d.

HydroTechnology (1995a) contained a peak flow series from 1955 to 1993 plus estimates of the 1916 and 1921 peak flows. Hydsys peak flow data is available from 1977 but with several years having missing data. For the period of concurrent data, no difference existed between the peak flows from HydroTechnology (1995a) and Hydsys. Given the longer period of available data, the peak flows from HydroTechnology (1995a) were adopted.

Orrvale
Continuous streamflow data is available from June 1977 to date. Hydsys is the only readily available data source for this site. High flow measurements were taken during the October 1993 flood event with the highest measured flow of 42,900 ML/d, just under the peak flow of 43,850 ML/d. Given measurements have been taken across the entire range of observed flows, the gauge is considered well rated and the Hydsys data was adopted for this study.

4.3.4 Seven Creeks

Euroa
Continuous streamflow records are available for Seven Creeks at Euroa from May 1963 to date. The two available data sources for peak flows, SKM (1997) and Hydsys, display significant differences across the medium to high peak flows. Generally, the SKM (1997) flows are lower than the Hydsys flows. The differences result from the application of rating curve developed as part of the SKM (1997) study. The revised rating curve was based on hydraulic modelling and increased the flow for a given level in comparison to the Hydsys rating curves. Given the detailed development of the rating curve at Euroa, the SKM (1997) flows were adopted for this study.

Flood volume data was not available from SKM (1997). Hydsys data was used to obtain flood volumes. These flood volumes were then revised to allow for the rating curve developed by SKM (1997).

Kialla West
Continuous streamflow data is available from June 1977 to date. Hydsys data is the only readily available data source for streamflows. Streamflow measurements have been taken up to a flow of 51,300 ML/d. The peak flow recorded in October 1993 was 62,200 ML/d. There is potential for flows during the large flood to bypass the gauge and travel overland to the Broken River. The potential bypassing and the lack of high flow measurements during the October 1993 event limits the reliability of high flows for this site.

4.4 Design Peak Flow Estimation

4.4.1 Overview

As outlined in **Section 4.2.1**, the estimation of design peak flows consists of two tasks as follows:

1) Peak flow frequency analysis at upstream streamflow gauges,
2) Peak flow transposition from the upstream gauges to the study boundary.

This section summaries the results from the above two tasks.

4.4.2 Peak Flow Frequency Analysis

Goulburn River at Murchison

Due to progressive enlargement of Lake Eildon, three periods of streamflow records were analysed to assess the impact of Eildon at Murchison on peak flows. The periods considered were as follows:

- Entire period of Murchison record (1881-1999)
- Period of Murchison record post Big Eildon Dam (1956-1999)
- Period of Murchison record post Big Eildon Dam (1956-1999) plus 1916

The third of the above periods of streamflow record considered in the frequency analysis was the period 1956 to 1999 (post Big Eildon) plus the 1916 event. The 1916 event occurred prior to construction of any storage at Eildon. The peak flow at Murchison in 1916 was 196,000 ML/d and is considerably larger than the 1974 peak flow of 111,000 ML/d. The rainfall spatial pattern for the 1916 event (SKP 1982) indicates significant rainfall fell downstream of Eildon. The 1916 event occurred in September, a time of year where the storage level in Lake Eildon is usually high. Given the size, the spatial rainfall pattern and time of year the event occurred, it is considered reasonable to assume the presence of Big Eildon, if constructed, may have had little impact on the peak flow at Murchison for the 1916 event. As a result, the peak flow for the 1916 event is included in the frequency analysis without modification.

A Generalised Extreme Value (GEV) distribution was fitted by higher order L-moments (Wang 1997) to annual peak flow series from the first two of the above periods. This is the preferred method for estimate of larger average return interval flows. For the third period however, it was necessary, given the inclusion of the 1916 peak flow, to fit a LP3 distribution via the method outlined in ARR87. This is due to the procedures for the fitting a GEV distribution not providing for the inclusion of historical peak flows in the analysis. The historical period was taken from 1881 to 1999. **Table 4-2** displays the results from the three periods.

Table 4-2 Design Peak Flow Quantiles - Goulburn River at Murchison

ARI (years)	Entire Record (1881-1999) (ML/d)	Post Big Eildon (1956-1999) (ML/d)	Post Big Eildon (1956-1999 plus 1916) (ML/d)
5	58,300	52,300	51,900
10	75,600	65,200	68,400
20	93,300	77,000	87,000
50	117,600	91,600	114,000
100	137,000	102,000	134,000
200	157,000	112,000	158,000
500	186,000	125,000	192,000

Apart from the 500 year ARI estimate, the post Big Eildon plus 1916 estimates lie between the other two results. For this study, the peak flow estimates from the post Big Eildon period plus the 1916 peak were adopted as it is considered these estimates are the most representative of current conditions.

SKP (1982) undertook a peak flow frequency analysis for the Goulburn River at Murchison. In that analysis, a LP3 distribution was fitted to peak flows for the period 1881- 1978. A total of six low flows were eliminated from the analysis. **Table 4-3** provides a comparison of peak flow quantiles from this study and SKP (1982).

Table 4-3 Comparison of Design Peak Flow Quantiles - Goulburn River at Murchison

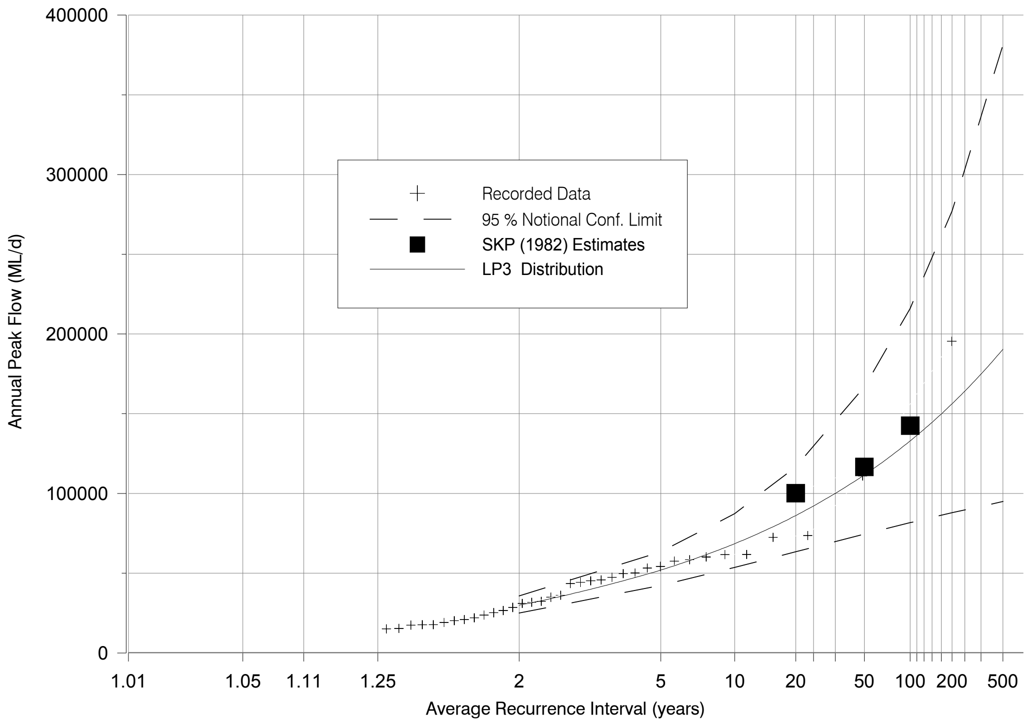
ARI (years)	Post Big Eildon (1956-1999 plus 1916) (ML/d)	SKP (1982) (1881-1978) (ML/d)
5	51,900	
10	68,400	
20	87,000	100,000
50	114,000	117,000
100	134,000	143,000
200	158,000	
500	192,000	

This study results in lower estimates for all ARIs than SK&P (1982). This is due to the use of a longer period of record post Big Eildon in this study's frequency analysis than in SK&P (1982).

Figure 4-2 presents the peak flow frequency curve for the post Big Eildon period plus the 1916 peak with 95% confidence limits shown. Also shown is the SKP (1982) design peak flow estimates. An extract of **Figure 4-2** between 5 and 100 year ARI is shown in **Section B-1** in **Appendix B**.

The May 1974 peak flow at Murchison was 111, 000 ML/d and this peak flow corresponds to an ARI of approximately 50 years. The October 1993 peak flow was 73,700 ML/d and corresponds to an approximate ARI of 13 years.

■ **Figure 4-2 Peak Flow Frequency Analysis - Goulburn River at Murchison**



Broken River at Casey's Weir

Peak flow frequency analysis was undertaken using the adopted Hydsys peak flows for the period 1889-1999 (111 years). A GEV distribution was fitted by higher order L moments to the annual peak flow series. SKP (1982) undertook a peak flow frequency analysis for the historical period 1903 – 1978 by fitting a LP3 distribution.

Peak flow estimates at Benalla were made by Willing & Partners (1998) as part of the Benalla Floodplain Management Study. A calibrated runoff routing model was employed to obtain these estimates. The estimates at Benalla provide a useful check on the estimates at Casey's Weir.

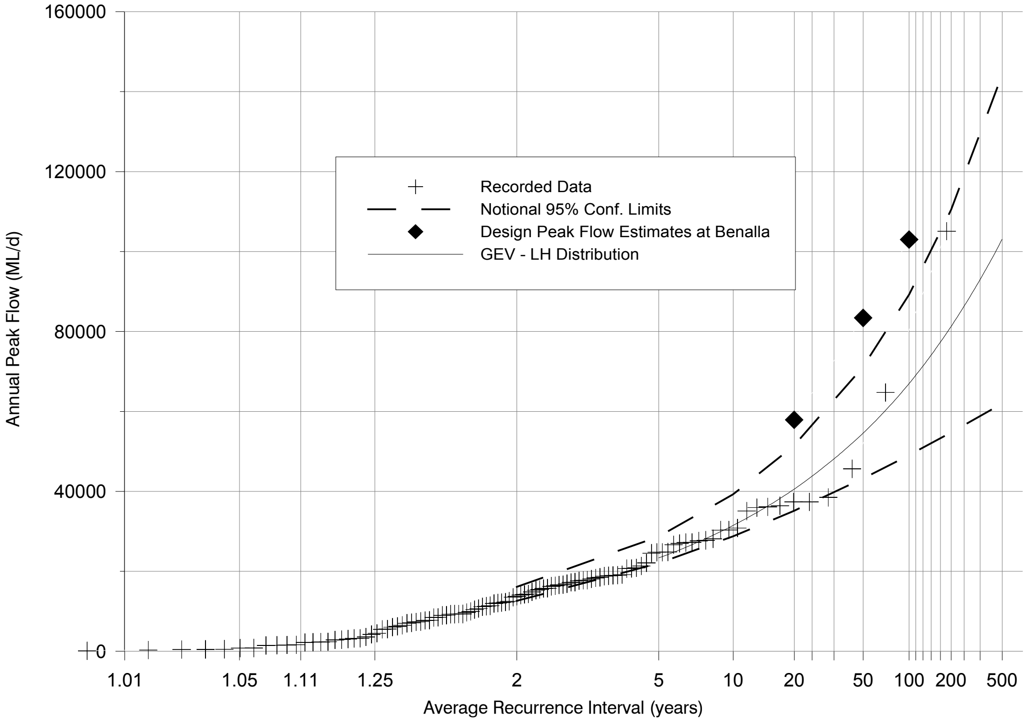
The design peak flow quantiles from three studies are presented in **Table 4-4**.

■ **Table 4-4 Comparison of Design Peak Flow Quantiles – Broken River at Casey's Weir and at Benalla**

ARI (years)	SKM (2000) Casey's Weir (1889-1999) (ML/d)	SK&P (1982) Casey's Weir (1903 –1978) (ML/d)	Willing & Partners (1998) Benalla (ML/d)
5	23,300		
10	31,400		
20	40,500	39,700	57,900
50	54,500	47,500	83,400
100	66,900	51,900	103,000
200	81,200		
500	103,000		

Figure 4-3 shows the peak flow frequency analysis for Casey's Weir and the design peak flow estimates for Benalla from Willing and Partners (1998). An extract of **Figure 4-3** between 5 and 100 year ARI is shown in **Section B-2** in **Appendix B**.

■ **Figure 4-3 Peak Flow Frequency Analysis - Broken River at Casey's Weir**



The result show significant differences between the peak flow estimates from the current study for Casey's Weir and the 1998 study for Benalla. These differences appear to be inconsistent with the expected behaviour of the floodplain and the catchment between Benalla and Casey's Weir.

The peak flow determined at Casey's Weir for the October 1993 event was 105,000 ML/d. This peak flow was determined from the recorded stage using the existing rating curve to obtain the flow. Based on the results of the current study frequency analysis, the 1993 event would have an ARI of approximately 500 years. Willing & Partners (1998) estimated the 1993 event had an approximately ARI of 100 years at Benalla. This indicates an inconsistency between Benalla and Casey's Weir design peak flow estimates.

As the existing rating curve of Casey's Weir is extrapolated for flows greater than 29,000 ML/d, the high flow rating curve is considered to be poorly defined. During large floods, flows can bypass the Casey's Weir gauge and therefore not be recorded. These two effects are contributing factors to the differences in design peak flow estimates.

Given the inconsistencies between Casey's Weir and Benalla and the inaccuracies in the high flow rating at Casey's Weir, an alternative approach using the design peak flow estimates at Benalla was adopted.

Broken River at Benalla

A peak flow frequency analysis was undertaken using the peak flows from 1955 to 1999 plus 1916 and 1921. The peak flows for the period 1955 to 1993 plus 1916 and 1921 were obtained from HydroTechnology (1995a). The peak flows for the period 1994 to 1999 were obtained from Hydsys. The log Pearson 3 (LP3) distribution was employed instead of the GEV distribution for the peak flow frequency analysis to accommodate the historical peak flows.

As previously mentioned, the Benalla Floodplain Management Study (Willing and Partners 1998) employed a calibrated runoff model to estimate rainfall-based design peak flows at Benalla.

ARR87 provides guidance on the relative accuracy of rainfall and streamflow based approaches. A formula is provided to determine the ARI above which the rainfall based estimates are preferred. This ARI is known as the probability of indifference. As the formula does not account for the inclusion of any historical peak flows, only the period 1955 to 1999 was used. Based on this period, an ARI of 35 years was calculated for the probability of indifference. Exclusion of the 1916 and 1921 floods is likely to result in an underestimate of the probability of indifference.

This study adopts the peak flow estimates from the current frequency analysis for ARIs up to the 20 year. For ARIs from 50 to 100 years, the peak flows from the rainfall-based calibrated runoff routing models (Willing and Partners, 1998) were adopted. No peak flow estimates for the 200 and 500 year ARIs were available from Willing and Partners (1998). The 200 and 500 year ARI peak flow estimates from the current frequency analysis were adopted.

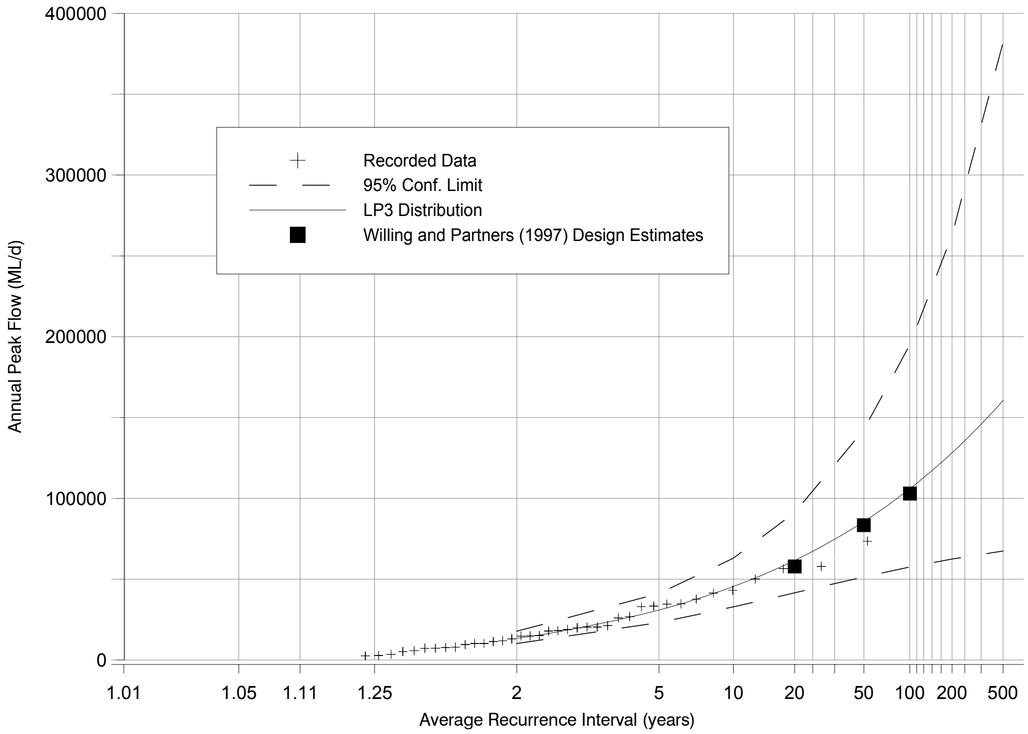
Table 4-5 shows the various available design peak flow estimates for Benalla and the estimates adopted by this study.

- **Table 4-5 Comparison of Design Peak Flow Quantiles – Broken River at Benalla**

ARI (years)	SKM (2000) (1955-1999 plus 1916 & 1921) (ML/d)	Willing and Partners (1998) (ML/d)	Adopted Peak Flows Estimates (ML/d)
5	30,900	-	30,900
10	45,500		45,500
20	61,600	57,900	61,600
50	85,600	83,400	83,400
100	106,000	103,000	103,000
200	128,000		128,000
500	161,000		161,000

Figure 4-4 displays the peak flow frequency analysis undertaken by this study and the design peak flow estimates from Willing and Partners (1998). An extract of **Figure 4-4** between 5 and 100 year ARI is shown in **Section B-3** in **Appendix B**.

■ **Figure 4-4 Peak Flow Frequency Analysis – Broken River at Benalla**



The peak flow in October 1993 was 112 000 ML/d and this peak flow corresponds to an approximate ARI of 100 years.

Seven Creeks at Euroa

The annual peak flows as listed in SKM (1997) were used in the peak flow frequency analysis (See **Section 4.3.4**) for the period 1963 to 1995. The peak flows for the period 1996-1999 were obtained by converting the peak stage recorded in Hydsys to peak flow via the rating curve developed by SKM (1997). The rating curve developed by SKM (1997) is considered more reliable than the current rating curve used in Hydsys as detailed hydraulic modelling was used in its' development. The annual peak flow series for the period 1963-1999 plus a estimate of the 1916 flood was employed in the frequency analysis. A LP3 distribution was fitted by method outlined in ARR87 to the annual peak flows.

The Euroa Floodplain Management Study (SKM 1997) employed a rainfall based approach to the estimation of design floods. The approach utilised a calibrated runoff routing model and design rainfall estimates.

The probability of indifference based on the period 1963 to 1999 is an ARI of about 30 years. As the formula does not account for the inclusion of any historical peak flows, the exclusion of the 1916 flood is likely to result in an underestimate of the probability of difference.

Accordingly, the streamflow based estimates from this current study were adopted for ARIs up to 20 years, while, the rainfall based estimates from SKM (1997) were adopted for ARIs above 50 years.

Table 4-6 lists the design peak flows estimates from this study's peak flow frequency analysis, SKM (1997), and the adopted design peak flows.

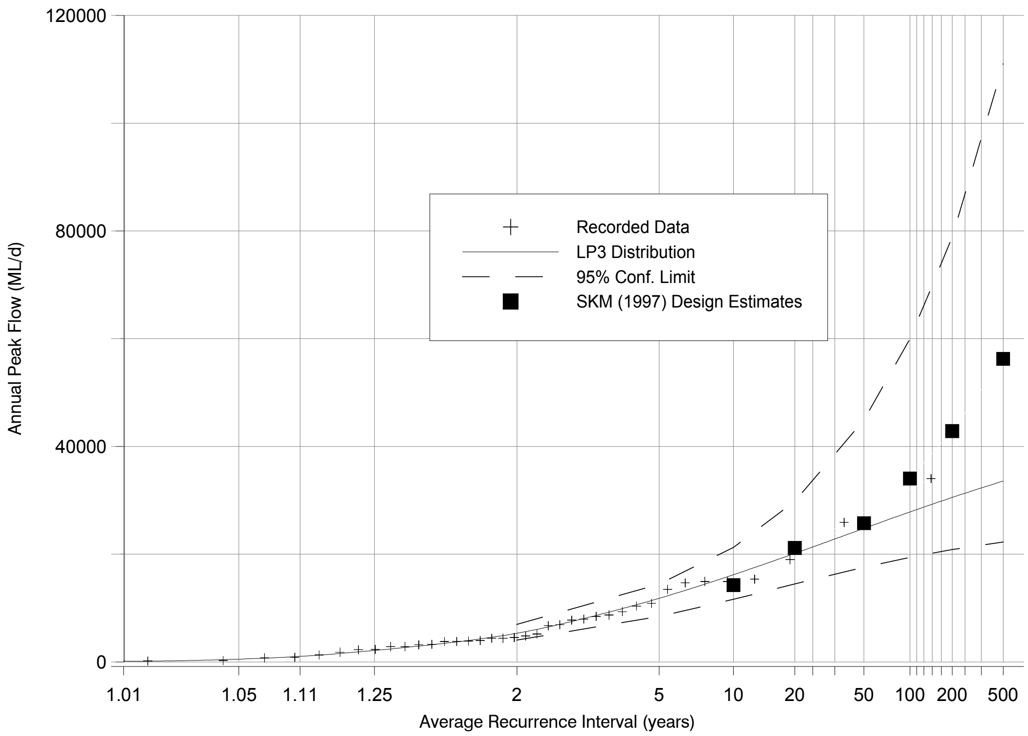
- **Table 4-6 Comparison of Design Peak Flow Quantiles – Seven Creeks at Euroa**

ARI (years)	Streamflow Based Estimates SKM (2000) (1963-1999 plus 1916) (ML/d)	Rainfall Based Estimates SKM (1997) (ML/d)	Adopted Peak Flows Estimates (ML/d)
5	11,800	-	11,800
10	16,200	14,200	16,200
20	20,200	21,200	20,200
50	24,800	25,800	25,800
100	27,800	34,000	34,000
200	30,500	42,900	42,900
500	33,600	56,300	56,300

Figure 4-5 displays the peak flow frequency curve for the period 1963-1999 plus 1916 and rainfall based estimates from SKM (1997). An extract of **Figure 4-5** between 5 and 100 year ARI is shown in **Section B-4** in **Appendix B**.

The peak flow in October 1993 at Euroa was 25,920 ML/d and this peak flow corresponds to an approximate ARI of 50 years.

- **Figure 4-5 Peak Flow Frequency Analysis - Seven Creeks at Euroa**



Goulburn River at Shepparton

The annual peak flows for the period 1921 – 1999 plus an estimate of the 1916 peak flow were utilised in the peak flow frequency analysis. A LP3 was fitted by the method outlined in ARR87. Three low flows were excluded from the analysis.

Two previous studies, SK&P (1982) and HydroTechnology (1995b), undertook peak flow frequency analyses for the Goulburn River at Shepparton. SK&P (1982) used the historical period 1921 – 1978 with three historical floods, 1870, 1916, and 1917. HydroTechnology (1995b) employed the period, 1921 – 1993 with only the 1916 flood as a historical event. The design peak flow quantiles from the three studies are presented in **Table 4-7**.

For the 20 year ARI, the peak estimate from this current study is less than the estimate from the two earlier studies, whilst, the estimates for the 50 and 100 year ARIs lie between the earlier studies. The differences between the current study and SKP (1982) are largely due to the increased streamflow record length and the revised annual peak flow series employed. Similarly, the differences with the HydroTechnology (1995b) are due to the increased record used.

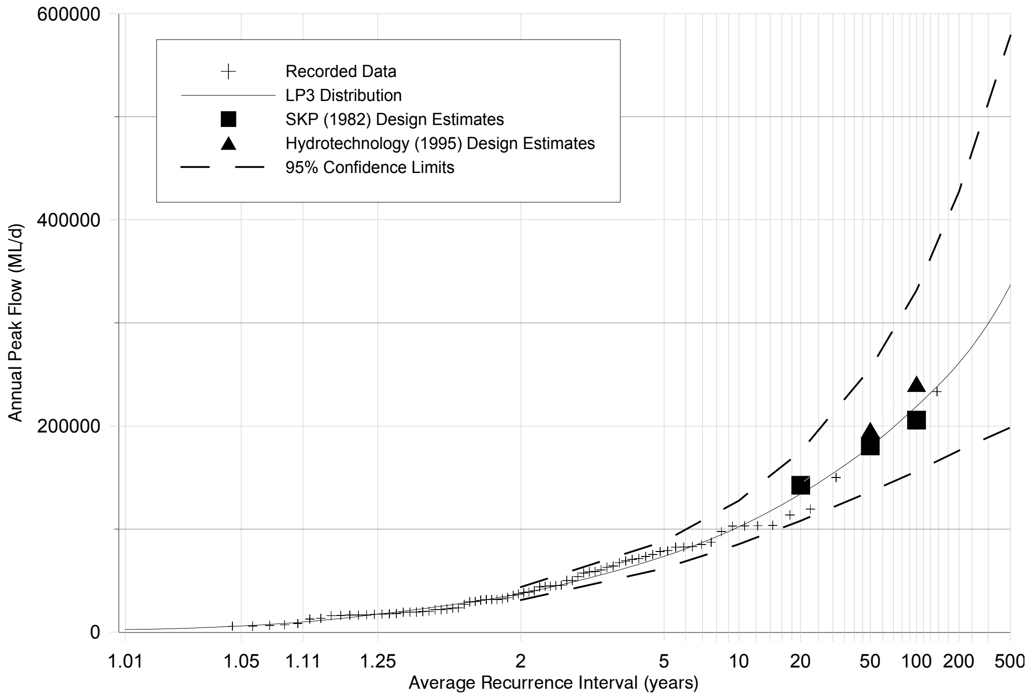
The design peak flow estimates from this current study are considered the best available estimates and have accordingly been adopted.

■ **Table 4-7 Comparison of Design Peak Flow Quantiles – Goulburn River at Shepparton**

ARI (years)	SKM (2000) (1921-1999 plus 1916) (ML/d)	SKP (1982) (1921-1978 plus 1870,1916, 1917) (ML/d)	HydroTechnology (1995b) (1921-1993 plus 1916) (ML/d)
5	73,400		76,600
10	102,000		108,000
20	137,000	142,000	143,000
50	180,000	181,000	196,000
100	219,000	206,000	241,000
200	261,000		
500	336,000		

Figure 4-6 displays this study's peak flow frequency curve for the Goulburn River at Shepparton and the design peak flow estimates from the current study, SKP (1982) and HydroTechnology (1995b). An extract of **Figure 4-6** between 5 and 100 year ARI is shown in **Section B-5** in **Appendix B**.

■ **Figure 4-6 Peak Flow Frequency Analysis - Goulburn River at Shepparton**



At Shepparton, the peak flow for the May 1974 was 191 000 ML/d and has approximately ARI of 75 years. For the October 1993 flood, the peak flow at Shepparton was 150,000 ML/d and has an approximate ARI of 35 years.

4.4.3 Peak Flow Transposition

To transpose the design peak flow from the upstream gauges to the study boundary, regression relationships were developed between the upstream gauges and the gauges adjacent to the study boundary. The relationships were based on maximum monthly instantaneous peak flows from the upstream gauges with the corresponding instantaneous peak flows at the gauges adjacent to the study boundary.

Goulburn River – Murchison to Kialla West

For the Goulburn River, the monthly peak flows for the period 1977-1980 and peak flows for the May 1974 and October 1993 events were employed in the development of the peak flow transposition.

The initial analysis showed that lower flows were exercising a considerable leverage on the regression equation developed for the transposition. Given the focus of the transposition was high flows, monthly peak flows below 1,000 ML/d were excluded from the regression. This threshold corresponds to an ARI of less than 1 year.

A regression was fitted to the peak flows at Murchison and Kialla West. This provided the following estimating equation:

$$\textit{Peak Flow at Kialla West} = 0.799 * \textit{Peak Flow at Murchison}^{1.024}$$

The coefficient of determination R^2 is 0.98. However, this R^2 value is not a true measurement of the goodness of fit of the regression due to the small number of peak flows used in the analysis.

Figure 4-7 displays the peak flow regression for the Goulburn River. Also included in **Figure 4-7** are the design peak flows for the Goulburn River at Murchison. These provide an indication of the range of flows used in the regression analysis. The short available concurrent record length and the lack of any large floods during this record adds considerable uncertainty to the extrapolation of the above transposition to large flood events.

Using the above regression equation, the 100 year ARI peak flow at Kialla West derived from the Murchison 100 year ARI peak flow of 134,000 ML/d (see **Table 4-3**) is 142 000 ML/d. The Kialla West 100 year ARI design peak flow is about 6% higher than at Murchison. This increase reflects the contribution made from Castle and Pranjip Creeks and the catchment downstream of Murchison. SKP (1982) estimated the increase in the 100 year ARI peak flow from Murchison to upstream of the Seven Creeks confluence (approximately Kialla West) was about 7%. The consistency in the increase of the peak flows from the two studies indicates the regression equation is suitable for this study's purposes.

A common relationship employed to transpose peak flow on the same stream from one site to another is as follows (Grayson et al 1996):

$$Q_{D/S} = Q_{U/S} * (A_{D/S}/A_{U/S})^{0.7}$$

The catchment area from Murchison to Kialla West increases from 10,772 km^2 to 12,038 km^2. Using the above relationship the ratio of the 100 year ARI peak flow at Kialla West and Murchison would be 1.08:1. This ratio is in good agreement with the regression equation developed above.

■ **Figure 4-7 Peak Flow Transposition - Goulburn River Murchison to Kialla West**

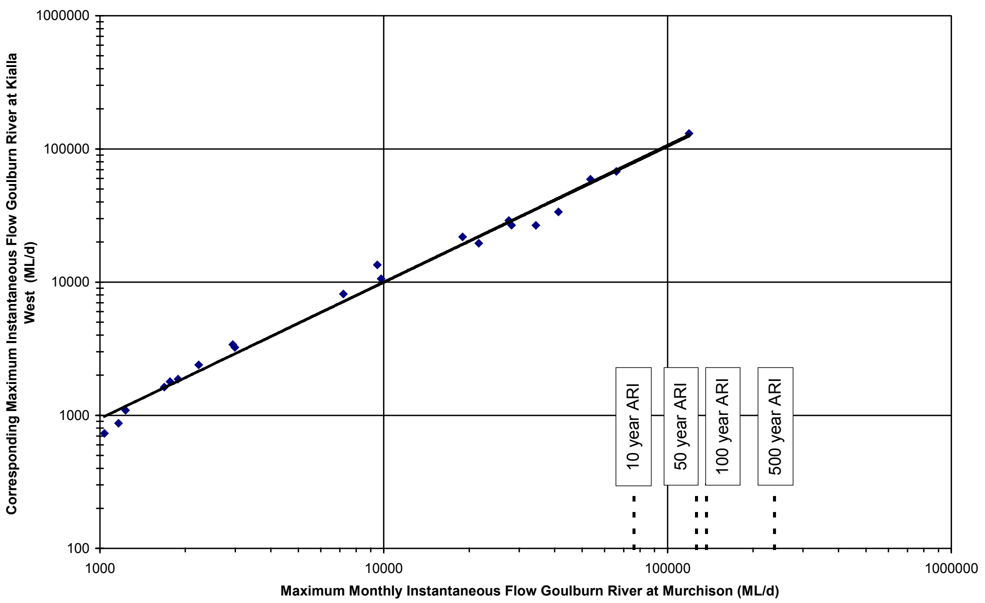


Table 4-8 displays the design peak flow estimates at Kialla West derived from the regression equation and the peak flows at Murchison (see **Table 4-3**). For comparison, the design peak flows determined by SKP (1982) for the Goulburn River upstream of the Seven Creeks confluence are included. The two locations can be considered representative of inflows at the upstream study area boundary. This study has resulted in a reduction in the design peak flow for the Goulburn River compared to SKP(1982). This is due to the lack of large flood events in the additional streamflow data used in the frequency analysis.

■ **Table 4-8 Design Peak Flow Quantiles – Goulburn River at Upstream Study Boundary**

ARI (years)	Goulburn River at Murchison (ML/d)	Goulburn River at Kialla West (ML/d)	Goulburn River U/S of the Seven Creeks SKP (1982) (ML/d)
5	51,900	53,800	
10	68,400	71,400	
20	87,000	91,300	99,400
50	114,000	120,000	128,000
100	134,000	142,000	152,000
200	158,000	168,000	
500	192,000	205,000	

Broken River – Benalla to Orrvale

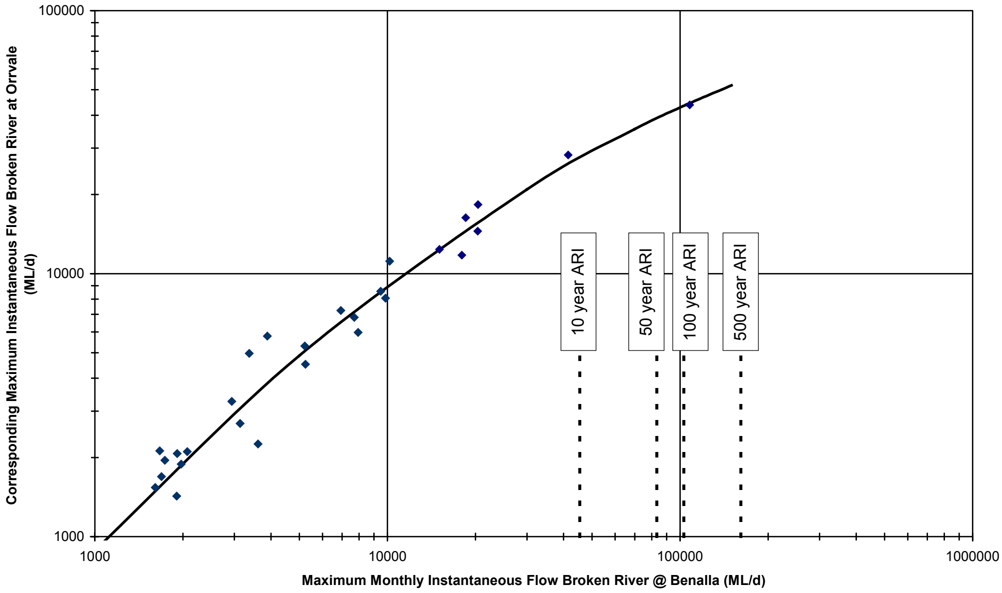
The period July 1977 to October 1993 was used in the development of the peak flow transposition for the Broken River. An initial analysis indicated that the two regression relationships were required to satisfactorily estimate the peak flow at Orrvale from the peak flow at Benalla. A linear relationship was applied to logs of the peak flows for flows at Benalla up to 10,000 ML/d. For higher flows, a second order polynomial relationship was fitted to the logs of the peak flows. These relationships reflect the effect of the breakouts at higher flows from the Broken River to Broken Creek. These breakouts results in a reduction in peak flow for the Broken River from Benalla to Orrvale.

Given the range of design peak flows at Benalla, only the regression equation for the higher flow is used in this study for the transposition of peak flows from Benalla to Orrvale. The regression equation for the higher flows has the following form:

$$Log(Peak\ Flow\ at\ Orrvale) = -0.168(Log(Peak\ Flow\ at\ Benalla))^2 + 2.184(Log(Peak\ Flow\ at\ Benalla)) - 2.095$$

The coefficient of determination (R^2) for the high flow regression equation is 0.954. **Figure 4-8** displays the peak flow regression for the Broken River and the design peak flows at Benalla. From **Figure 4-8**, it can be seen the regression used peak flows with ARIs from less than 10 years to 100 years. The inclusion of the October 1993 flood in the regression analysis aids in defining the transposition of large floods along the Broken River. The transposition is considered adequate for the purposes of this study.

■ **Figure 4-8 Peak Flow Transposition - Broken River Benalla to Orrvale**



The design peak flows at Orrvale are derived using the regression equation and the design peak flows at Benalla and are shown in **Table 4-9**. Also shown in **Table 4-9** is the SKP (1982) peak flow estimates for the Broken River upstream of the Goulburn River confluence. The two locations can be considered representative of inflows at the upstream study area boundary for the Broken River.

This study's estimates are lower than the SKP (1982) estimates. The reduction has occurred despite the use of the higher Benalla estimates instead of the lower Casey's Weir estimates as the upstream station design peak flows. The regression equation used to transpose the peak flow estimates has resulted in a significant decrease in the design peak flows from Benalla to Orrvale. This study's regression equation is considered more reliable for high flows than the method used in SKP (1982) due to the use of the October 1993 flood event in the development of the regression equation.

■ **Table 4-9 Design Peak Flow Quantiles – Broken River at upstream study boundary**

ARI (years)	Broken River at Benalla ML/d	Broken River at Orrvale (ML/d)	Broken River U/S of Goulburn River Confluence SKP(1982) ML/d
5	30,900	21,400	
10	45,500	27,500	
20	61,600	33,000	39,700
50	83,400	39,000	44,100
100	103,000	43,500	47,500
200	128,000	48,300	
500	161,000	53,600	

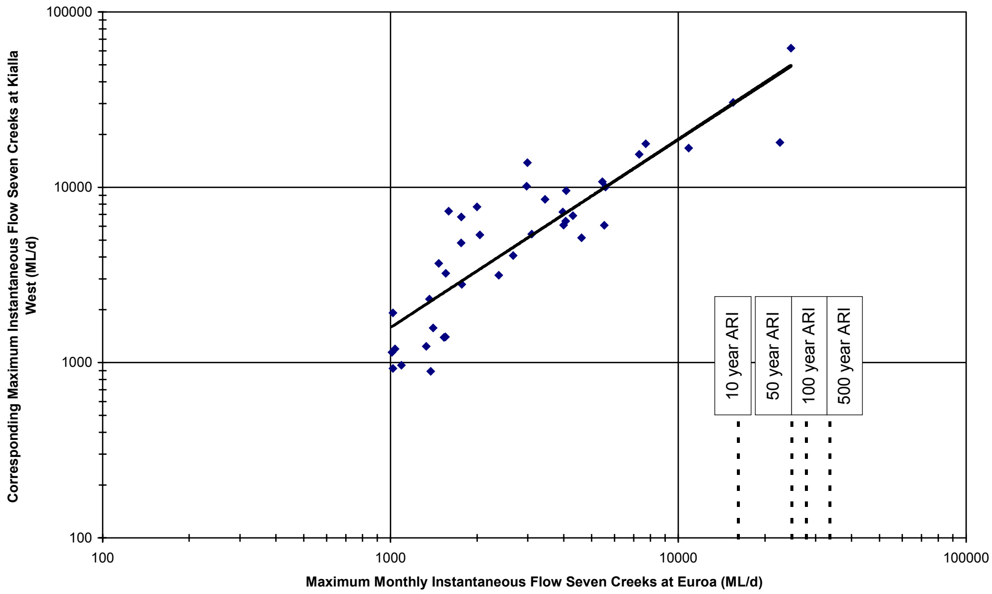
Seven Creeks – Euroa to Kialla West

For Seven Creeks, the maximum monthly instantaneous peak for the period July 1977 to May 1996 were used in the development of the peak flow transposition. A relationship fitted to the peak flows has the following form:

$$\textit{Peak Flow at Kialla West} = 0.953\ \textit{Peak Flow at Euroa}^{1.073}$$

Figure 4-9 displays the peak flow regression for the Seven Creeks and the design peak flows at Euroa. The regression analysis used peak flows up to approximately a 70 year ARI. The fit over the peak flow range is acceptable with a R^2 of 0.79. The lower R^2 value is due to the scatter of flows across the entire range.

■ **Figure 4-9 Peak Flow Transposition - Seven Creeks Euroa to Kialla West**



Using the above regression, the design peak flows at Kialla West were derived from the Euroa peak flow. The design peak flow estimates for Seven Creeks at Kialla West and the SKP (1982) estimates for Seven Creeks are shown in **Table 4-10**.

■ **Table 4-10 Design Peak Flow Quantiles – Seven Creeks at Upstream Study Boundary**

ARI (years)	Seven Creeks at Euroa Adopted Peak Flows Estimates (ML/d)	Seven Creeks at Kialla West (ML/d)	Seven Creeks U/S of Goulburn River confluence SKP(1982) (ML/d)
5	11,800	21,200	
10	16,200	27,500	
20	20,200	42,000	32,000
50	25,800	57,800	46,700
100	34,000	69,900	57,900
200	42,900	89,400	
500	56,300	120,000	

This study's estimates are significantly greater than the SKP (1982) estimates. The previous study adopted the calculated May 1974 peak flow as the 100 year ARI peak flow. This was based on a runoff routing model estimate for the May 1974 peak flow at the Goulburn confluence and the adoption of the May 1974 event as the 100 year ARI event at Euroa. The current study used more reliable design estimates at Euroa and included the October 1993 event in the development of the transposition relationship. As a result, it is considered the current study's peak flow estimates for Seven Creeks are more reliable than the SKP's (1982) study.

A common relationship employed to transpose peak flow on the same stream from one site to another is as follows (Grayson et al 1996):

$$Q_{D/S} = Q_{U/S} * (A_{D/S}/A_{U/S})^{0.7}$$

The catchment area from Euroa to Kialla West increases from 332 km^2 to 1505 km^2. Using the above relationship the ratio of the 100 year ARI peak flow at Kialla West and Euroa would be 2.9:1. This compares to a ratio of 2.1:1 obtained for this study. The lower ratio reflects the attenuation of the peak flow by floodplain storage downstream of Euroa.

4.4.4 Summary of Design and Historical Floods

Table 4-11 shows the design floods determined as part of the hydrologic analysis.

■ **Table 4-11 Design Floods**

ARI (years)	Peak Flow (ML/d)			
	Goulburn River at Upstream Limit	Broken River at Upstream Limit	Seven Creeks at Upstream Limit	Goulburn River At Shepparton
5	53,800	21,400	21,200	73,400
10	71,400	27,400	27,400	102,000
20	91,300	32,900	42,000	137,000
50	120,000	39,000	57,800	180,000
100	142,000	43,500	69,900	219,000
200	168,000	48,300	89,400	261,000
500	205,000	53,600	120,000	336,540

To put the above design floods into perspective, the ARIs and peak flow for three major historical floods at the Shepparton gauge are listed in **Table 4-12.**

■ **Table 4-12 Historical Floods at Shepparton**

Date of Historical Flood	ARI (years)	Peak Flow (ML/d)
October 1993	35	150,000
May 1974	75	192,000
September 1916	160	233,000

4.5 Design Flood Volume Estimation

4.5.1 Overview

Peak flow and flood volume are both important flood characteristics determining flooding behaviour within the study area. The analysis outlined in **Section 4.4** deals with peak flows only. It is necessary to undertake a similar procedure as adopted for the peak flow estimation to estimate flood volume for the required ARIs at the upstream study boundary. The design peak flow and flood volume estimates will then be combined to result in a design flood hydrograph.

Design flood volume estimation, like the design peak flow estimation, involves two tasks as follows:

1) Flood volume flow frequency analyses at the upstream gauges.
2) Flood volume transposition from the upstream gauges to the study boundary

This section presents the results for the above two tasks.

4.5.2 Five-Day Flood Volume Frequency Analysis

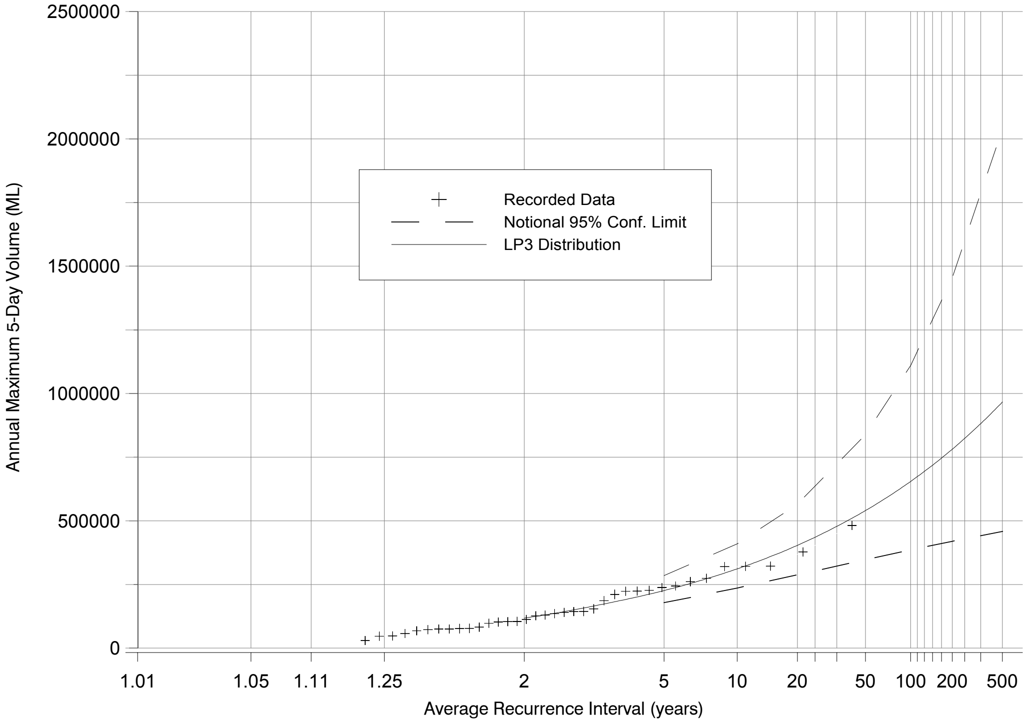
To undertake the flood volume frequency analysis, it is necessary to select a representative flood duration. A 5-day duration was considered adequate to define flood volume during large floods in the Goulburn – Broken catchment. Annual series of maximum 5-day volumes were constructed from the available data at the upstream gauges. It was not possible to estimate a 5-day volume for historical flood events prior to the commencement of the continuous streamflow gauging as only information on the peak flows was available. As a result, the streamflow data used in the flood volume frequency analysis was limited to the period of the continuous streamflow measurement. The periods of available continuous streamflow data for the gauges of interest are listed in **Table 4-1**.

Goulburn River at Murchison
To be consistent with the peak flow frequency analysis, the period of record post Big Eildon (1956-1999) plus 1916 event was employed in the flood volume frequency analysis. A LP3 distribution was fitted by the ARR87 method to annual peak 5-day volume series. The design 5-day volume quantiles are presented for the Goulburn River at Murchison in **Table 4-13**. **Figure 4-10** display the 5-day flood volume frequency curve at Murchison. The 1916 event is the largest recorded event plotted in and significantly larger than the other recorded events. No previous studies have undertaken frequency analysis of flood volumes and, thus no comparison of estimates is possible.

■ **Table 4-13 Five-Day Flood Volume Quantiles – Goulburn River at Murchison**

ARI (years)	Goulburn River at Murchison (1956-1999 Plus 1916) (ML)
5	220,000
10	301,000
20	391,000
50	525,000
100	640,000
200	770,000
500	965,000

■ **Figure 4-10 Five-Day Flood Volume Frequency Analysis - Goulburn River at Murchison**

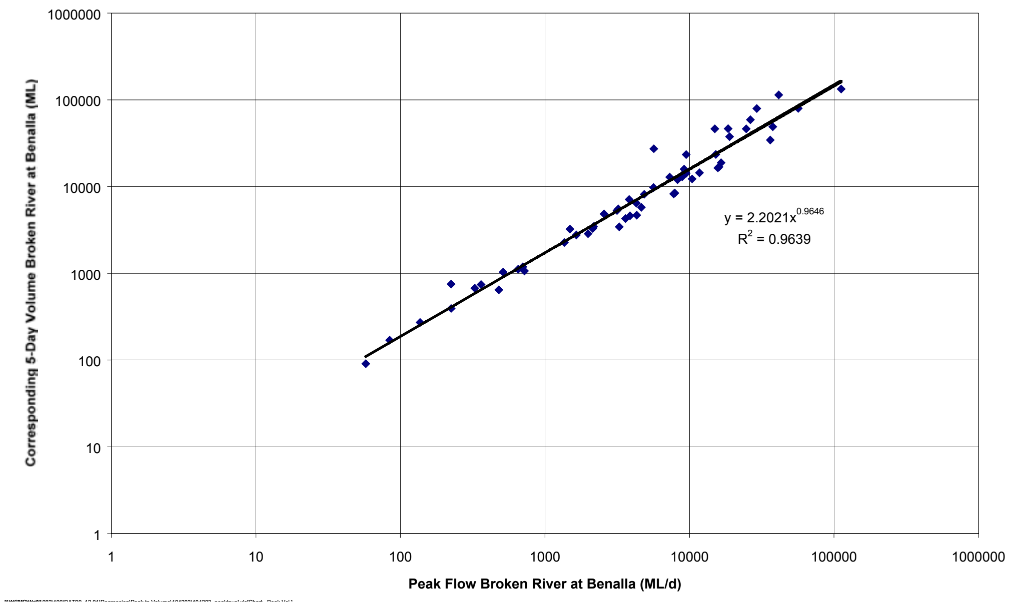


Broken River at Casey's Weir and Benalla

For the Broken River at Casey's Weir, a GEV distribution was fitted by higher order L-moments to annual peak 5-day volume series for the period 1889 – 1999. The design 5-day volume quantiles are presented in **Table 4-14** and **Figure 4-12** displays the 5-day flood volume frequency curve.

Given the inaccuracies in the Casey's Weir rating curve, it was consider necessary to check the 5-day volume design estimates for Casey's Weir against the corresponding estimates at Benalla. Flood volume data for the Broken River at Benalla is available from 1955 to date, however, there are considerable periods of missing data. As a result, there is an insufficient length of record for use in a frequency analysis. To extend the available flood volume data at Benalla, a relationship was developed between the peak flow and 5-day volume for the available data period and **Figure 4-11** shows the relationship developed.

■ **Figure 4-11 Peak flow to 5-day flood volume - Broken River at Benalla**

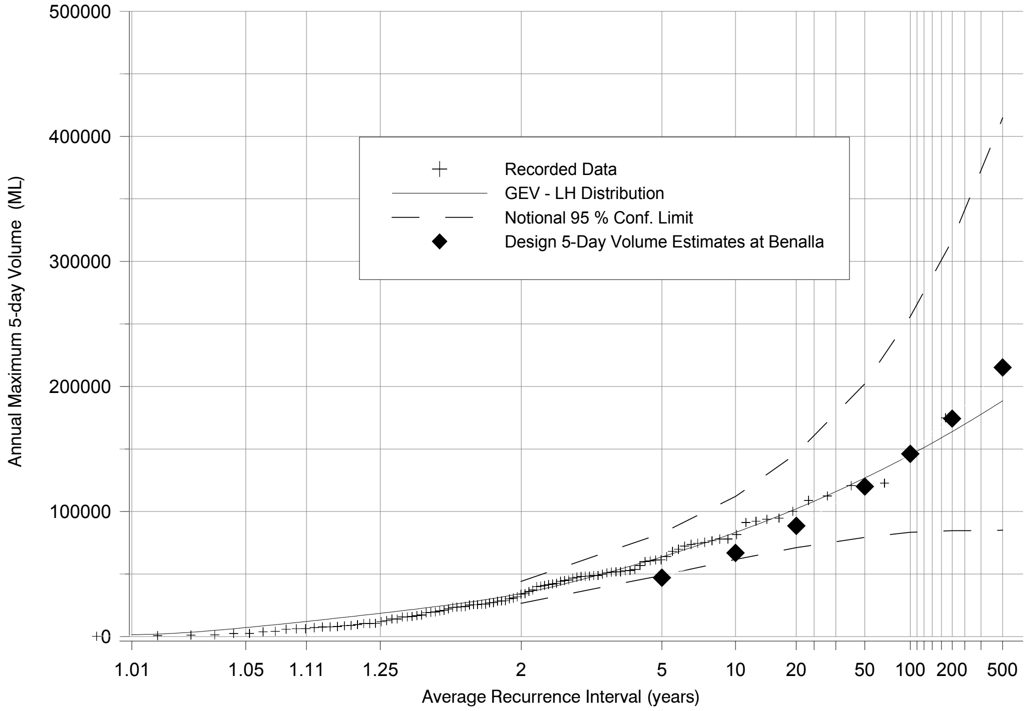


Using this relationship and the peak flows from 1955 to 1999 plus 1916 and 1921, 5-day volumes were estimated where observed data was not available. A LP3 distribution was fitted to the 5-day volumes series at Benalla from 1955 to 1999 plus 1916 and 1921. The design 5-day volume quantiles are presented in **Table 4-14** and **Figure 4-12** displays the 5-day flood volume design estimates in comparison to Casey's Weir design estimates.

■ **Table 4-14 Five-Day Flood Volume Quantiles – Broken River at Casey's Weir and Benalla**

ARI (years)	Broken River at Casey's Weir (1889-1999) (ML)	Broken River at Benalla (1955-1999 plus 1916 & 1921) (ML)
5	63,600	47,100
10	83,200	67,000
20	102,000	88,500
50	127,000	120,000
100	145,000	146,000
200	164,000	174,000
500	188,000	215,000

■ **Figure 4-12 Five-Day flood volume frequency analysis - Broken River at Casey's Weir and Benalla**



The 5-day volume estimates from Benalla are lower than the Casey's Weir estimates for the ARIs from 5 to 20 years. The estimates from the two sites are reasonably consistent for ARIs from 50 to 200 years with the Casey's Weir estimate being lower than Benalla for the 500 year ARI.

Insufficient concurrent flood volume data existed at Orrvale and Benalla to enable the development of a transposition relationship for flood volumes. Given this lack of concurrent flood volume data and the reasonable consistency between the design 5-day volumes estimates, it is considered to adequate use Casey Weir for the 5-day flood volume estimation.

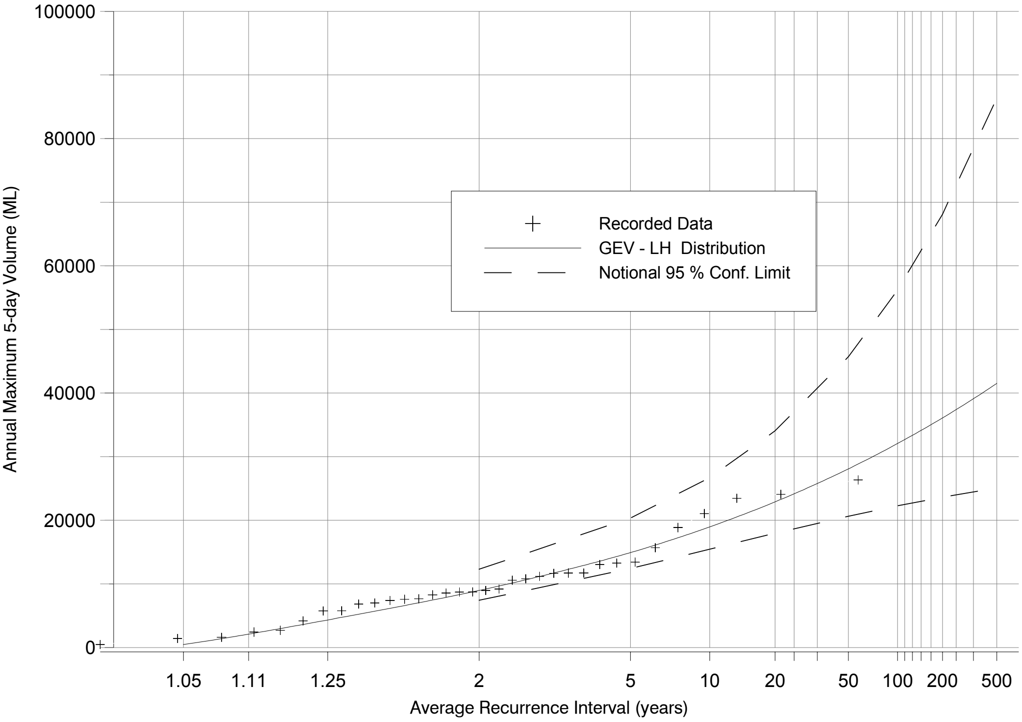
Seven Creeks at Euroa

A GEV distribution was fitted by higher order L-moments to the annual maximum 5-day volumes at Euroa for the period 1963 to 1999. The design 5-day volume quantiles are presented in **Table 4-15** and **Figure 4-13** displays the 5-day flood volume frequency curve.

■ **Table 4-15 Five-Day Flood Volume Quantiles – Seven Creeks at Euroa**

ARI (years)	Seven Creeks at Euroa (1963-1999) (ML)
5	14,900
10	18,900
20	22,900
50	28,100
100	32,000
200	36,100
500	41,500

■ **Figure 4-13 Five-Day Flood Volume Frequency Analysis - Seven Creeks at Euroa**



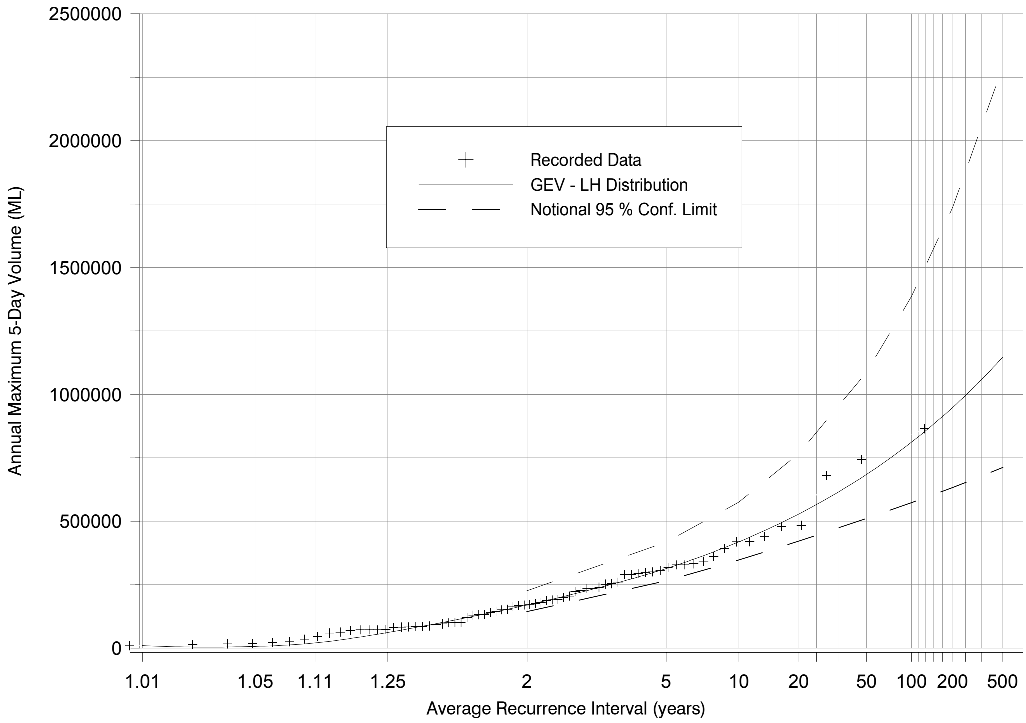
Goulburn River at Shepparton

A GEV distribution was fitted by higher order L-moments to the annual maximum 5-day volumes at Shepparton for the period 1921 to 1999. The design 5-day volume quantiles are presented in **Table 4-16** and **Figure 4-14** displays the 5-day flood volume frequency curve.

■ **Table 4-16 Five-Day Flood Volume Quantiles – Goulburn River at Shepparton**

ARI (years)	Goulburn River at Shepparton (1921-1999) (ML/d)
5	313,000
10	419,000
20	528,000
50	684,000
100	812,000
200	949,000
500	1,150,000

■ **Figure 4-14 Five-Day Flood Volume Frequency Analysis – Goulburn River at Shepparton**



4.5.3 Five-Day Flood Volume Transposition

To provide estimates of the design flood volumes at the study boundaries, transposition relationships were determined. These relationships were developed from the monthly maximum 5-day volumes at upstream gauges and the corresponding 5-day volume at the gauges adjacent to the study boundaries.

Goulburn River – Murchison to Kialla West

Like the peak flow transposition, only the period July 1977 to March 1980 (3 years) was found to be suitable for the 5-day flood volume regression analysis. Low volumes were found to exercise a considerable leverage on the regression equation. Monthly maximum 5-day volumes below 10000 ML were excluded from the regression analysis. This volume corresponds to less than 1 year ARI.

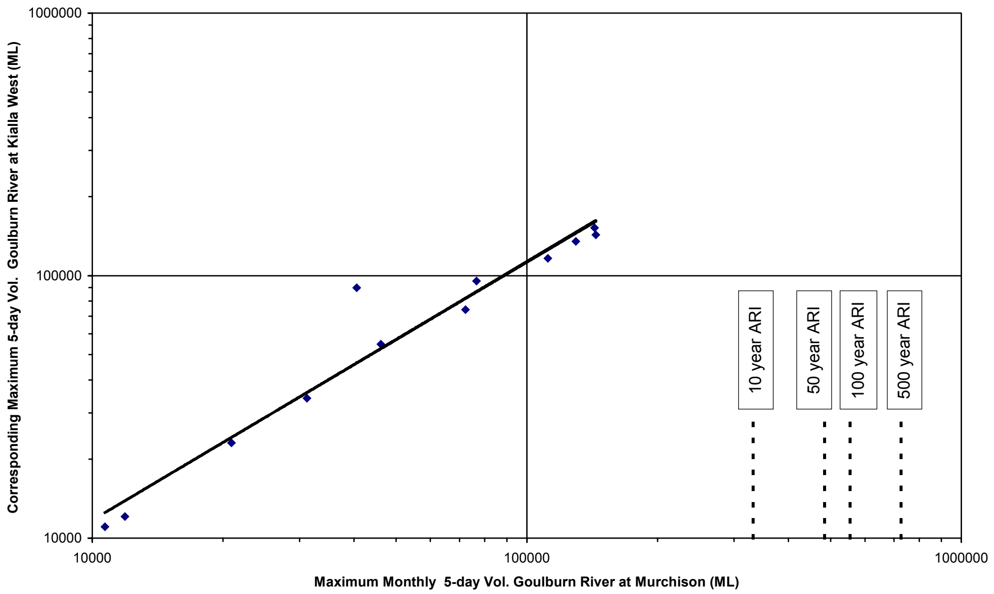
The regression equation was fitted to 5-day volumes at Murchison and Kialla West and has the following form:

*Five-Day Volume at Kialla West = 1.3602*5-Day Volume at Murchison$^{0.984}$*

The coefficient of determination (R^2) for the regression equation is 0.946. **Figure 4-15** displays the 5-day volume regression for the Goulburn River and the design 5-day volumes for the Goulburn River at Murchison. These provide an indication of the range of volumes used in the regression analysis.

Similarly to peak flow transposition, the short available concurrent record length and the lack of any large floods during this record adds considerable uncertainty to the extrapolation of the above transposition to large flood events.

- **Figure 4-15 Five-Day Flood Volume Transposition - Goulburn River Murchison to Kialla West**



Broken River – Casey's Weir to Orrvale

The period July 1977 to October 1993 was used in the development of the 5-day volume transposition for the Broken River. As for the peak flow transposition, two regression relationships were required to satisfactorily estimate the 5-day volume at Orrvale from the 5-day volume at Casey Weir. A linear relationship was applied to logs of the 5-day volumes at Casey Weir up to 10000 ML. For higher volumes, a second order polynomial relationship was fitted to the logs of the volumes. The higher volume relationship reflects the loss of flood volume from the Broken River to Broken Creek during large flood events, thus reducing the flood volume for the Broken River from Casey's Weir to Orrvale.

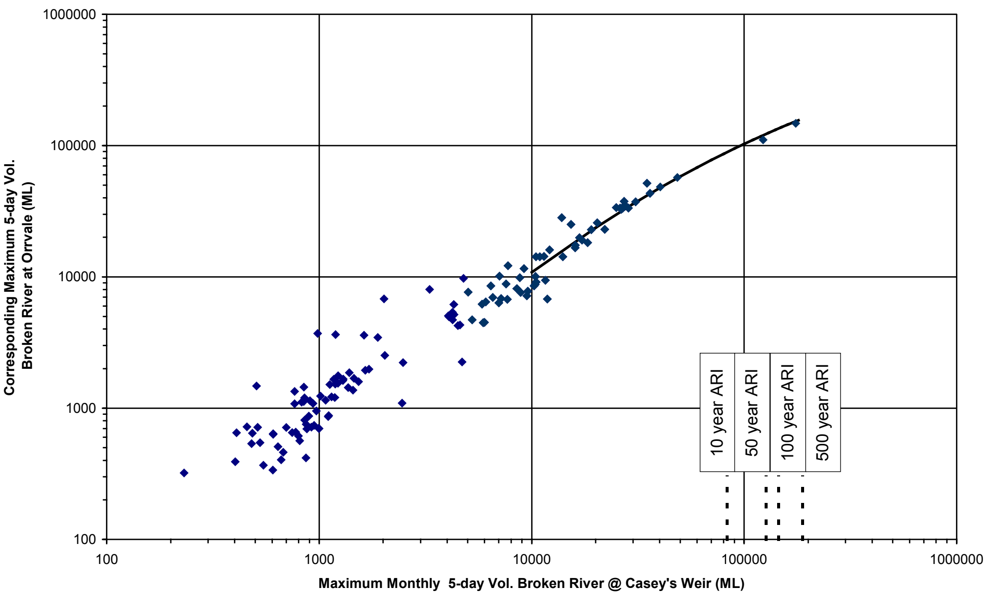
Given the range of design 5-day volumes of interest at Casey's Weir, only the regression equation for the higher volumes was used in this study. The regression equation for the higher flows has the following form:

Log(5-day volume at Orrvale) = -0.218(Log(5-day volume at Casey's Weir))^2 +2.945(Log(5-day volume at Casey's Weir)) – 4.25

The coefficient of determination (R^2) for the higher volume regression equation is 0.92. **Figure 4-16** displays the 5-day volume regression for the Broken River and the design 5-day volumes at Casey's Weir. From **Figure 4-16**, it can be seen the regression analysis used flood volumes with ARIs from less than 10 years to 100 years. The inclusion of the October 1993 in the regression analysis aids in defining the transposition of large floods along the Broken River.

From **Figure 4-16**, the reduction in flood volume from Casey's Weir to Orrvale for events where the 5-day volume is greater than 100,000 ML is due to the breakouts from the Broken River. For smaller events the breakouts do not operate and hence the 5-day volumes at Orrvale and Casey's Weir are similar.

- **Figure 4-16 Five-Day Flood Volume Transposition - Broken River Casey's Weir to Orrvale**



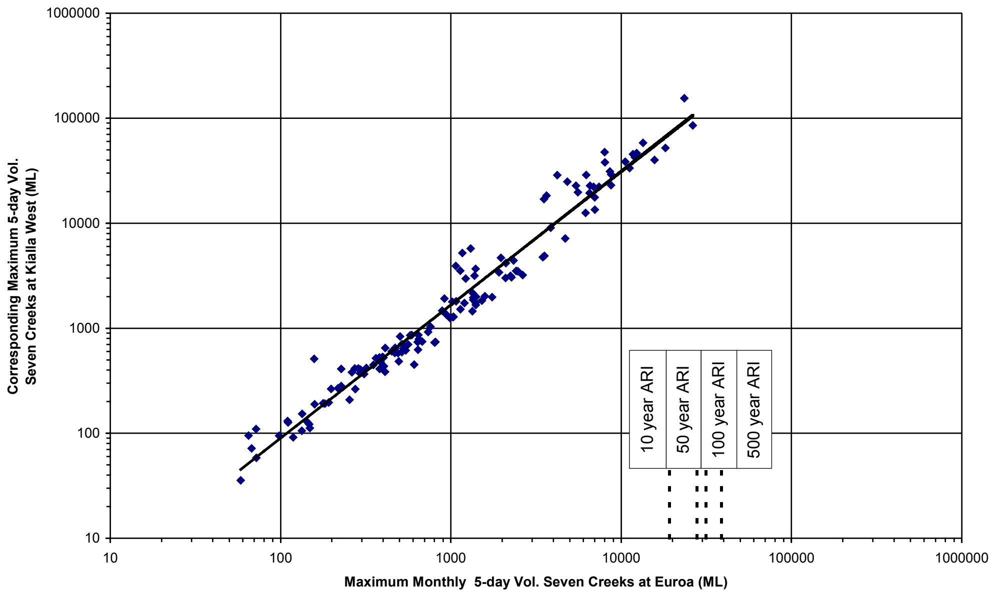
Seven Creeks – Euroa to Kialla West

For Seven Creeks, the period July 1977 to May 1996 was used in the 5-day volume regression analysis. A relationship was fitted to the 5-day volumes and has the following form:

$$\textit{Five-Day Volume at Kialla West} = 0.259 * \textit{5-Day Volume at Euroa}^{1.27}$$

Figure 4-17 displays the 5-day volume regression for the Seven Creeks and the design peak flows at Euroa. The regression analysis used 5-day volumes up to approximately 30 year ARI. The fit over the peak flow range is acceptable, with a R^2 of 0.96.

■ **Figure 4-17 Five-Day Flood Volume Transposition - Seven Creeks Euroa to Kialla West**



4.5.4 Five-Day Flood Volumes at Upstream Study Boundary

The design 5-day flood volume estimates were transposed from upstream long term gauges to downstream short term gauges using the regression equations detailed in **Section 4.5.3**. **Table 4-17** below displays the design 5-day volume estimates at the upstream study boundaries.

■ **Table 4-17 Design 5-day Volumes Quantiles – At Study Boundary and Shepparton**

ARI (years)	Goulburn River at Kialla West (ML)	Broken River at Orrvale (ML)	Seven Creeks at Kialla West (ML)	Goulburn River at Shepparton (ML)
5	245,000	71,800	51,600	313,000
10	333,000	89,500	70,000	419,000
20	431,000	105,000	89,000	528,000
50	576,000	123,000	115,000	684,000
100	700,000	135,000	136,000	812,000
200	839,000	147,000	159,000	949,000
500	1,050,000	161,000	190,000	1,150,000

To check whether the transposition relationships appear reasonable, the runoff volume per unit area for each catchment was determined at the various streamflow gauges used in the analysis. This involves dividing design flood volume estimates at each gauge for each ARI by the catchment area to that gauge.

Table 4-18 Unit Catchment Runoff Volumes

ARI (years)	Goulburn River			Broken River		Seven Creeks	
	Murchison (ML/km^2)	Kialla West (ML/km^2)	Shepparton (ML/km^2)	Casey Weir (ML/km^2)	Orrvale (ML/km^2)	Euroa (ML/km^2)	Kialla West (ML/km^2)
5	20.4	20.4	19.4	33.1	28.6	44.9	34.3
10	28.0	27.7	26.0	43.2	35.7	57.0	46.5
20	36.3	35.8	32.8	53.0	41.8	68.9	59.1
50	48.7	47.8	42.4	65.8	49.0	84.5	76.4
100	59.4	58.2	50.3	75.4	53.9	96.5	90.4
200	71.4	69.7	58.8	85.1	58.5	108.6	105.6
500	89.5	87.0	71.1	97.9	64.2	125.0	126.2

As the catchment area increases it is reasonable to expect the volume of runoff per square kilometre should decrease. This is due to the reduction in mean catchment rainfall from south-east to north-west across the catchment resulting in higher runoff volumes for the upper catchments.

For the Goulburn River, a slight reduction is seen in the runoff rates from Murchison to Kialla West with a further larger reduction to Shepparton. This behaviour is considered reasonable given the lack of any tributaries with high runoff volumes.

Runoff rates for the Broken River reflects the reduction in the rainfall downstream of Casey's Weir, the lack of any tributaries between Orrvale and Casey's Weir and the flood volume loss from the river due to the breakouts for the larger flood events.

The lower events (up to 50 year ARI) for Seven Creeks display a significant reduction in runoff rates from Euroa to Kialla West in line with the reduction in mean catchment rainfall. However, for the higher events, the reduction in the runoff rates is less marked with a small increase in the runoff rates for the 500 year ARI event. This increase is likely due to the increased uncertainty in the flood volume frequency analysis at Euroa and the transposition relationship for larger events. The increase indicates the 500 year ARI flood volume at Kialla West may be conservative. Given the uncertainty involved in the analysis, the adoption of a conservative estimate is considered appropriate.

4.6 Design Hydrograph Determination

4.6.1 Overview

The end product of the hydrological task is design flood hydrographs for the required range of ARIs at the upstream study boundary. The foregoing analyses have produced estimates of the peak flows and 5-day flood volumes for the required ARIs. The next step is the development of design flood hydrographs with the correct peak and volume characteristics.

This study has adopted the approach presented in ARR87 Chapter 12. The approach was developed by the Tasmanian Hydro-Electric Commission and the steps involved are presented below.

1) Select an appropriate duration for the calculation of flood volume
2) Rank annual peak flows and undertake an annual peak flow frequency analysis
3) Rank annual volumes and undertake an annual flood volume frequency analysis based on the duration selected in Step 1
4) Check whether the peak flow used in the peak flow frequency analysis is associated with the same flood event as the flood volume in the flood volume frequency analysis.
5) Check whether the peak flow and flood volume of a flood event have the same rank in the respective peak flow and flood volume series.
6) If the coincidence is acceptable from Step 4 & 5, the peak flow and volume may be treated as coming from a single population. That is the combination of the peak flow and flood volume results in the ARI of the joint event being equal to the ARI of the separate events. For each specified ARI, a peak flow to volume ratio is determined from the peak flow and flood volume estimates for the specified ARI.
7) A historical hydrograph with similar peak to volume ratio as the required ARI is selected. The selected historical event is then scaled by the ratio of the required ARI design flood volume and the historical volume. Some adjustment maybe required to achieve the correct peak discharge.
8) If the coincidence is not acceptable, the peak flow and flood volume must be treated as coming from two different populations. In this case, the peak flow and flood volume must be treated as coming from a bivariate population.

4.6.2 Selection of Representative Hydrographs

Goulburn River at Kialla West

Examination of the available streamflow data from Kialla West and Murchison showed the peak flows and flood volumes could be treated as a single population. That is, there was sufficient coincidence of given floods ranking equally for peak flows are for flood volume (see Section 10.12 of IEAust, 1987).

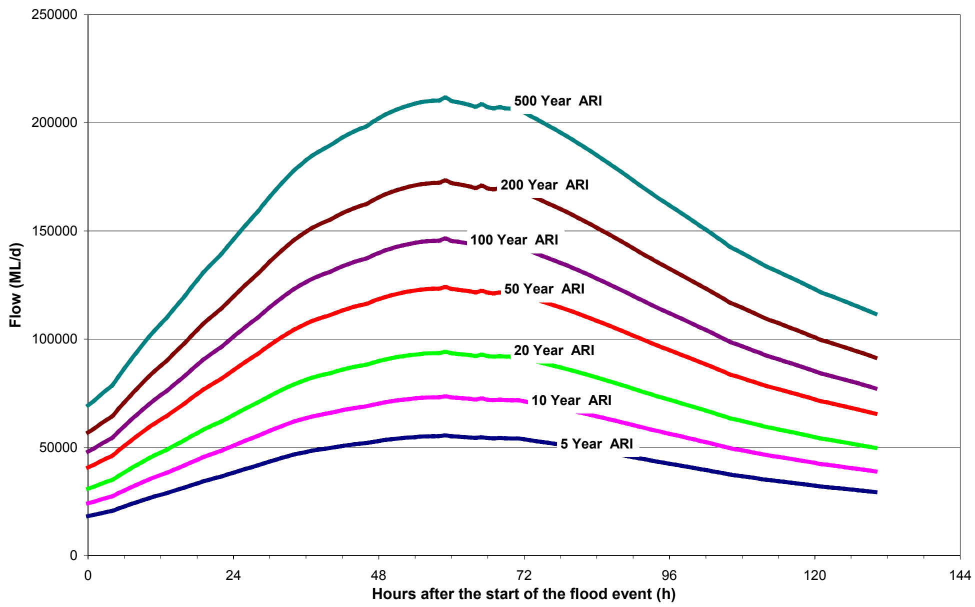
Table 4-19 displays the ratio of the peak flow to 5-day volume for the required ARIs.

■ **Table 4-19 Design Peak to 5-day Volume Ratios - Goulburn River at Kialla West**

ARI (years)	Goulburn River at Kialla West
5	0.22
10	0.21
20	0.21
50	0.21
100	0.20
200	0.20
500	0.20

The May 1974 flood hydrograph was selected as the representative hydrograph across the range of ARIs. The peak to 5-day volume ratio for the May 1974 event was 0.23. Design hydrographs were obtained by scaling the ordinates of the May 1974 hydrograph by the ratio of the design flood volume to the observed May 1974 flood volume. Some further manual adjustments to both peak flow and 5-day flood volume were made to achieve best-fit hydrographs. **Figure 4-18** displays the design flood hydrographs. The small differences in peaks between **Figure 4-18** and **Table 4-8** are the result of those manual modifications.

■ **Figure 4-18 Design Flood Hydrographs – Goulburn River at Kialla West**



Broken River at Orrvale

It was found that the peak flows and flood volumes for the Broken River could be treated as a single population. That is, there was sufficient coincidence of given floods ranking equally for peak flows are for flood volume (see Section 10.12 of IEAust, 1987). **Table 4-20** displays the ratio of the peak flow to 5-day volume for the required ARIs.

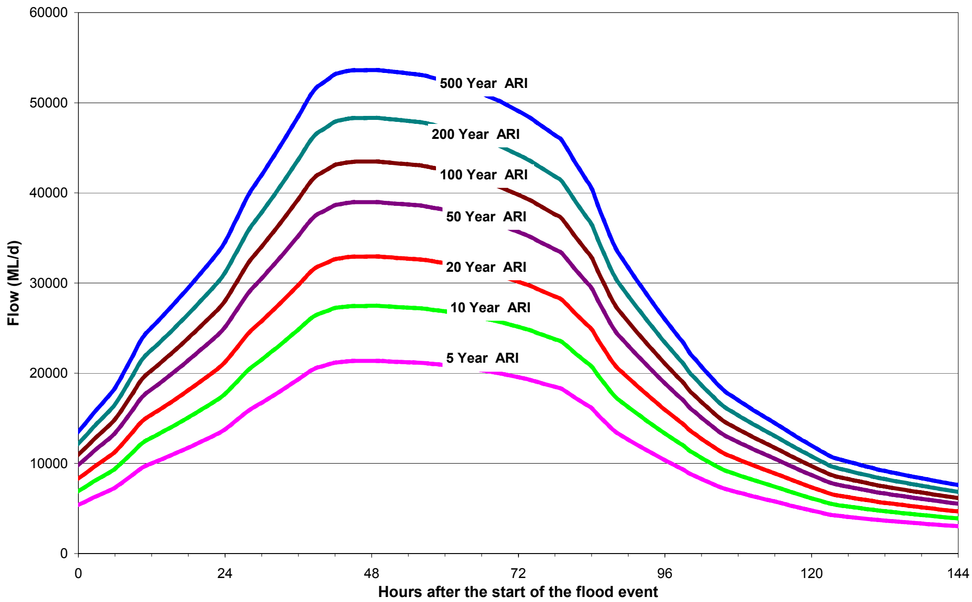
The October 1993 flood hydrograph was selected as the representative hydrograph across the range of ARIs. The peak to 5-day volume ratio for the October 1993 event was 0.30. Design hydrographs were obtained by scaling the ordinates of the October

1993 hydrograph by the ratio of the design flood volume to the observed October 1993 flood volume. **Figure 4-19** displays the design flood hydrographs.

- **Table 4-20 Design Peak to 5-day Volume Ratios: Broken River at Orrvale**

ARI (years)	Broken River at Orrvale
5	0.30
10	0.31
20	0.31
50	0.32
100	0.32
200	0.33
500	0.33

- **Figure 4-19 Design Flood Hydrographs – Broken River at Orrvale**



Seven Creeks at Kialla West

Examination of the available streamflow data from Kialla West and Euroa showed the peak flows and flood volumes could be treated as a single population.

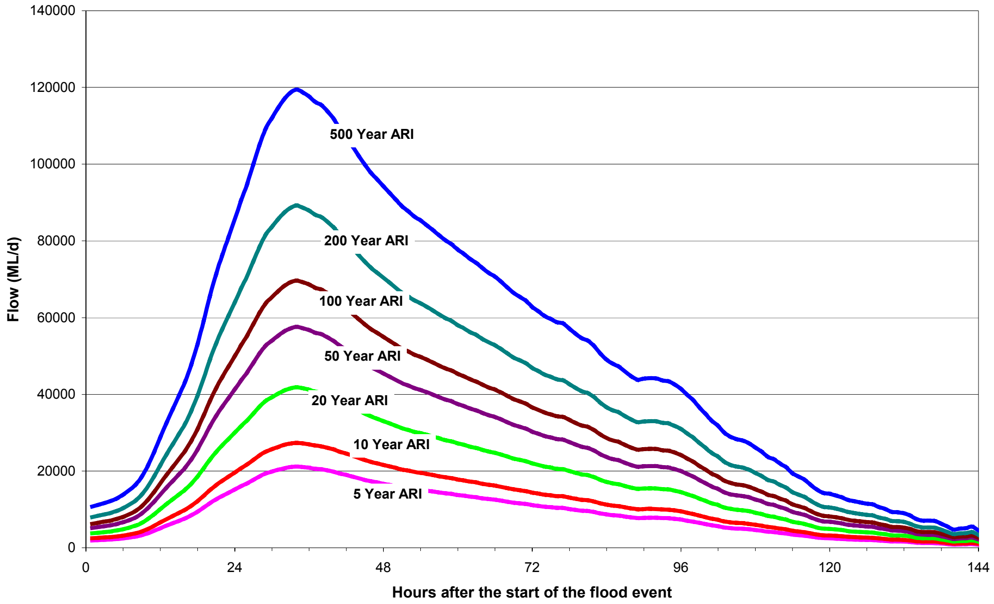
Table 4-21 displays the ratio of the peak flow to the 5-day volume for the required ARIs.

The October 1993 flood hydrograph was selected as the representative hydrograph across the range of ARIs. Design hydrographs were obtained by scaling the ordinates of the October 1993 hydrograph by the ratio of the design flood volume to the observed October 1993 flood volume. Some further manual adjustments to 5-day flood volume were made to achieve best-fit hydrographs. **Figure 4-20** displays the design flood hydrographs for the required range of ARIs.

■ **Table 4-21 Design Peak to 5-Day Volume Ratios - Seven Creeks at Kialla West**

ARI (years)	Seven Creeks at Kialla West
5	0.41
10	0.39
20	0.47
50	0.50
100	0.51
200	0.56
500	0.63

■ **Figure 4-20 Design Flood Hydrographs - Seven Creeks at Kialla West**



4.7 Design Flood Event Combinations

4.7.1 Overview

This section presents an assessment of the design flood event combinations. The assessment began with a coarse one-dimensional hydraulic model of the three rivers only and was then refined using the two-dimensional hydraulic model of the study area. The hydraulic modelling is discussed in detail in **Section 5**.

Flooding within the study area can result from a number of combinations of inflows from the three main contributing catchments. The maximum gauge height obtained at Shepparton for a given flood event is depended on the following factors:

- Flood event magnitudes (ARI of peak flow and flood volume) in the three contributing catchments,
- Relative timing of the peak flows from the three contributing catchments.

The assessment considered various combinations of flood magnitude (ARI) in each of the three contributing catchments, and the resulting flood magnitude (ARI) at the Shepparton gauge.

The relative timings of the peak flows from the three contributing catchments for this assessment were determined from observed floods. Details of the determination of relative timings are provided in the following section.

4.7.2 Relative Timing of Contributing Peak Flows

As previously mentioned the relative timings of the peak flows from the three contributing catchments have a major impact on the resulting flood at Shepparton.

The timing of the peak flow from a catchment depends on the catchment size, temporal and spatial rainfall patterns and the available storage within the catchment. It is reasonable to expect a smaller catchment would peak prior to a larger catchment if the rainfall commenced at the same time in both catchments.

Using the available streamflow records for the Broken River at Orrvale and Seven Creeks at Kialla West, a comparison of the time of peaks from both catchments was undertaken. The peak flow for Seven Creeks at Kialla West occurs between 6 hours and 24 hours prior to the peak flow for the Broken River at Orrvale. The median value of 15 hours was adopted as the relative timing for this assessment.

Little data is available on the relative timing of peaks for the Goulburn River, given the short period of record available at Kialla West and the lack of large floods during this available period. To obtain the relative timings for the Goulburn River at Kialla West, the times of peaks for the May 1974 and October 1993 events were estimated from the records at Murchison and Goulburn Weir.

For the May 1974 event, the time of the peak at Kialla West was estimated by lagging the peak flow at Goulburn Weir by 33 hours. This lag time was based on the lag times calculated in SKP (1982). The Goulburn River at Kialla West was found to peak approximately 15 hours after the peak for the Broken River at Orrvale.

A similar approach was employed for the October 1993 event. Based on the lag times calculated by SKP (1982), the peak flow at Murchison was lagged by 30 hours. The peak in Goulburn at Kialla West is then approximately 60 hours after the peak in the Broken River at Orrvale. The longer lag for the October 1993 flood, in comparison to the May 1974, is due to the effect of Eildon attenuating the inflows from the upstream catchment, and the small contribution to the flood event from the catchment downstream of Eildon.

For this assessment, a relative timing for the Goulburn River peak at Kialla West of 15 hours after the Broken River peak at Orrvale was adopted (from the May 1974 event). As part of the trials of design event combinations (see below), sensitivity analyses were undertaken to assess the impact of the relative timing of the Goulburn peak flow on those combinations. However, this did not lead to any change in the adopted 15 hour relative timing.

4.7.3 Design Event Combinations

The design flood hydrographs with above relative timings were input into the two dimensional hydraulic model. The hydraulic model routed the inflows through the study area and produced a resultant hydrograph at Shepparton. Combinations of design floods in the three streams were trialed to determine the resulting design flood at the Goulburn River at Shepparton gauge. Final combinations are shown in **Table 4-20.**

- **Table 4-22 Design Flood Combinations**

ARI at Shepparton Gauge (years)	ARI (years) of Design floods in Contributing Streams		
	Goulburn River at Upstream Limit	Broken River at Upstream Limit	Seven Creeks at Upstream Limit
5	5 2	2 5	2 5
10	10 5	5 10	5 10
20	20 10	10 20	10 20
50	50 20	20 50	20 50
100	100 50	50 100	50 100
200	200 100	100 200	100 200
500	500 200	200 500	200 500

For example, a flood with 100 year ARI at the Shepparton gauge can result from either:

- a 100 year ARI in Goulburn River in combination with 50 year ARI floods in the Broken River and Seven Creeks,
- or 100 year ARI floods in the Broken River and Seven Creeks in combination with a 50 year ARI flood in the Goulburn River.

4.8 Preliminary Probable Maximum Flood Determination

4.8.1 Overview

The probable maximum flood (PMF) is defined as the largest flood that can be reasonably expected to occur. The study brief requires a preliminary estimate of the PMF to be made. Rigorous analysis of the PMF is beyond the scope of this study and would require a more detailed study to be undertaken. The PMF was routed through the study area to determine an indicative estimate of additional flood extent and flood level above the 100 year ARI event.

This study utilised the following two methods to determine the PMF for each contributing streams at the upstream study boundary:

- Prediction equation based on catchment area (Nathan et al, 1994),
- Transposition of PMF estimates for the Hume Dam catchment.

A prediction equation has been developed relating the catchment area to the PMF (Nathan et al, 1994). The equation was based on PMF estimates from 56 catchments in South Eastern Australia ranging in areas from 1 km^2 to 10,000 km^2.

The PMF inflow to the Hume Dam was estimated via a detailed study (SKM, 1998). The catchment for the Hume Dam is a similar size to the Goulburn River catchment at Shepparton. From the Hume Dam study it is possible to determine the ratio of the PMF to the 100 year ARI design peak inflow. This ratio then can be used to factor the 100 year ARI design peak flows from this study to obtain PMF estimates for the three contributing streams.

4.8.2 PMF Estimates

Using the Prediction Equation

The PMF prediction equation developed has the following form:

$$Q_p = 129.1\, A^{0.616}$$

where Q_p - PMF peak flow (m^3/s)
A - catchment area (km^2)

Applying the above equation to the three contributing streams and at Shepparton results in the PMF estimates shown in **Table 4-23.**

Table 4-23 PMF Estimates – Prediction equation

Catchment	Catchment Area (km^2)	PMF Estimate from Prediction Equation (Nathan et al 1994) (ML/d)
Goulburn River to Kialla West	12,038	3,640,000
Broken River to Orrvale	2,508	1,380,000
Seven Creeks to Kialla West	1,505	1,010,000
Goulburn River to Shepparton	16,125	4,360,000

Using the Ratio of the PMF to 100 year ARI Peak Flow

The ratio of the PMF and 100 year ARI design peak inflows for the Hume Dam is 8.9:1. Using this ratio to multiply the 100 year ARI design peak flows for the three contributing catchments results in the PMF estimates at the upstream study boundary shown in **Table 4-24**. The PMF estimate at Shepparton was obtained by routing the PMFs from the contributing catchments to Shepparton using a coarse one dimensional hydraulic model and is shown in **Table 4-24**.

- **Table 4-24 PMF Estimates – via PMF : 100 year ARI Ratio**

Catchment	100 year ARI Design Peak Flow (ML/d)	PMF Estimate from Ratio of PMF to 100 year ARI peak flow (ML/d)
Goulburn River to Kialla West	150,000	1,330,000
Broken River to Orrvale	43,500	388,000
Seven Creeks to Kialla West	69,900	622,000
Goulburn River to Shepparton	227,000	2,120,000

Conclusions

Comparing the PMF estimates in **Table 4-23** and **Table 4-24** shows significant differences in the two PMF estimates. The estimates obtained by the prediction equation are considerably higher than the estimates derived from the ratio of the PMF to the 100 year ARI peak flow. Furthermore, the prediction equation results in PMF to 100 year ARI peak flow ratios greater than 15. These ratios are larger than what would be normally expected.

The prediction equation (Nathan et al, 1994) does not account for the breakouts along the Broken River. Furthermore, as noted above, the predictive equation is based on work for catchments up to 10,000 km^2. The Goulburn Broken catchments are significantly larger.

Given these limitation in the use of the prediction equation, it is considered the PMF estimates obtained from the ratio of the PMF to the 100 year ARI peak flows in the River Murray catchment to Hume Dam are adequate for the purposes of this study.

4.9 Historical Flood Hydrograph Determination

4.9.1 Overview

The hydraulic model was calibrated against the recorded flood levels for both the May 1974 and October 1993 flood events (see **Section 5**). To enable the hydraulic model calibration, flood hydrographs for the Goulburn and Broken Rivers and Seven Creeks are required at the upstream study boundaries.

Details of the determination of the flood hydrographs for May 1974 and October 1993 are provided in the following sections.

4.9.2 May 1974 Flood Hydrographs

At the time of the May 1974 event no streamflow gauging stations adjacent to the upstream study boundary were operating. It is thus necessary to estimate the hydrographs for the May 1974 event at the upstream study boundary. As part of the SKP (1982) study, flood hydrographs for the Goulburn and Broken Rivers at the upstream study boundaries were derived for the May 1974 event. For Seven Creeks, only a peak flow was estimated in that study (a complete flood hydrograph was not required). This current study has adopted the May 1974 flood hydrographs from the 1982 study for the Goulburn and Broken Rivers. Details of the derivation of the 1974 flood hydrographs for the Goulburn and Broken Rivers from SKP (1982) and Seven Creeks are provided below.

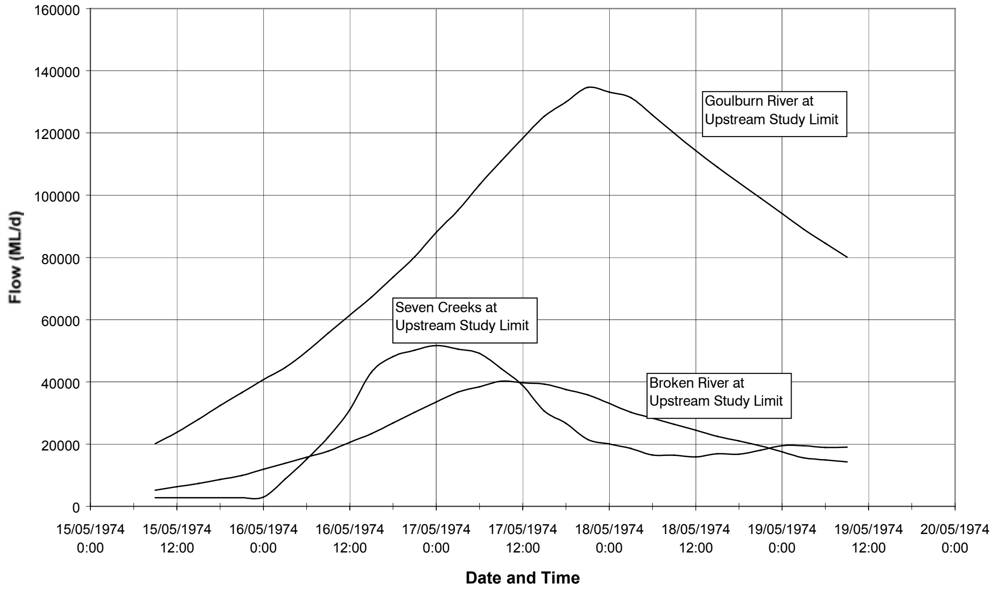
For the Goulburn River, the recorded hydrograph at Murchison was translated to the study boundary by lagging the recorded hydrograph by 30 hours. Similarly, recorded hydrographs for Castle Creek at Arcadia and Pranjip Creek at Moorilim were lagged to the study boundary. Details on the determination of the lags can be obtained from SKP (1982). The three lagged hydrographs were summed to obtain the Goulburn River hydrograph at the study boundary.

The Broken River hydrograph at the study boundary was obtained by the routing of the recorded hydrograph at Casey's Weir. Details of the routing model employed can be found in SKP (1982).

The Seven Creeks hydrograph was obtained by the subtraction of the Goulburn and Broken River hydrographs from the recorded hydrograph at Shepparton.

Figure 4-21 displays the May 1974 hydrographs at the study boundaries for the Goulburn and Broken River and Seven Creeks.

- **Figure 4-21 May 1974 flood hydrographs**



4.9.3 October 1993 Flood Hydrographs

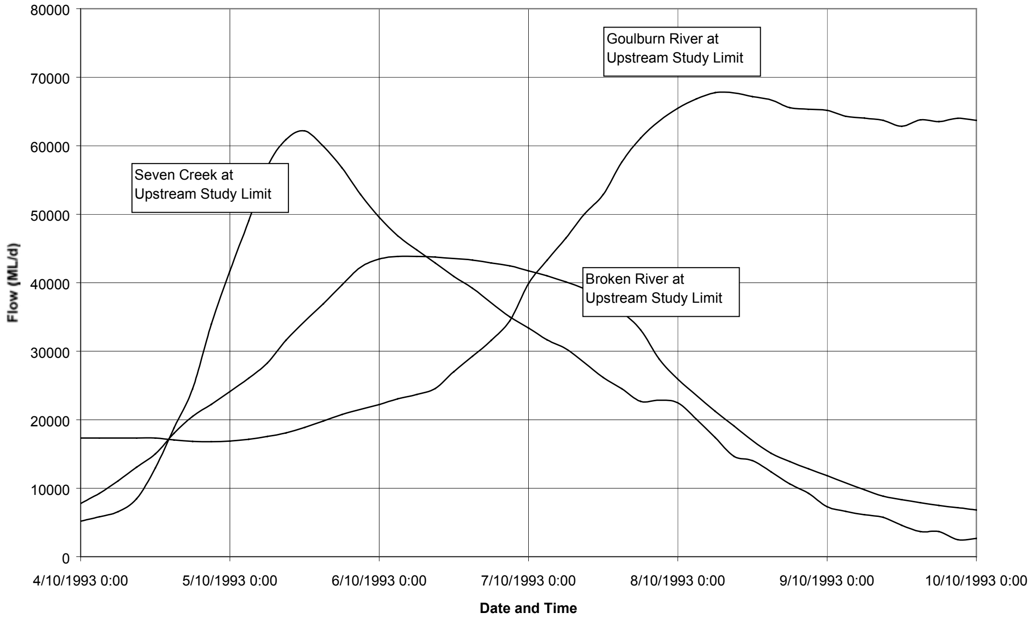
For the October 1993 event, no recorded hydrograph was available for the Goulburn River at the upstream study boundary. This study adopted the same approach as the 1982 study to determine the flood hydrograph for the Goulburn River at the upstream study boundary. The recorded hydrographs for the Goulburn River at Murchison, Castle Creek at Arcadia and Pranjip Creek at Moorilim were translated to the study boundary. The three translated hydrographs were then summed to obtain the hydrograph for the Goulburn River at the upstream study boundary.

The available recorded hydrograph for the Broken River at Orrvale was used for the Broken River hydrograph at the upstream study boundary.

For Seven Creeks, the recorded hydrograph at Kialla West is available only during the peak (5/10/93 0:00 to 6/10/93 14:00) due to backwatering from the Goulburn River. The remainder of the Seven Creeks hydrograph was determined by subtracting the Goulburn and Broken River hydrograph from the recorded hydrographs at Shepparton.

Figure 4-22 displays the October 1993 hydrographs at the study boundaries for the Goulburn and Broken Rivers and Seven Creeks.

■ **Figure 4-22 October 1993 flood hydrographs**



5. Hydraulic Analysis

5.1 Introduction

This section summarises the development and calibration of the hydraulic model used for this study. This work, including associated plots included in this section, was undertaken by Lawson and Treloar.

Once developed, this model provided the means to simulate the floodplain processes in detail for both historic (calibration) events and design events. Investigations consisted of the following components:

- Data gathering,
- Review of previous investigations,
- Establishment of hydraulic models of the study area,
- Calibration of the hydraulic models to historic flood events.

There have been numerous investigations into the Goulburn River and major tributaries over the years. The study brief provides a comprehensive list of investigations into the study area. The principal references used specifically (although not exclusively) for the hydraulics component of this current investigation are listed below.

- Shepparton – Mooroopna Flood Study Main Report and Appendices, Sinclair Knight & Partners P/L and Kinhill P/L, 1982.
- Shepparton – Mooroopna Flood Mitigation Scheme – Discussion Document on Flood Mitigation Options, Sinclair Knight and Partners P/L, 1987.
- Shepparton Bypass Planning Study and EES Phase 2, Subconsultancy Report on Hydrology and Hydraulics– Discussion Document on Flood Mitigation Options, Ian Drummond and Associates P/L, 1996.
- Documentation and Review of 1993 Victorian Floods, HydroTechnology P/L, 1995.
- Lower Goulburn Waterway and Floodplain Management Plan, Sinclair Knight Merz P/L, 1998.

The Greater Shepparton City Council (GSCC) and the Goulburn Broken Catchment Management Authority (GBCMA) have also provided significant documentation of historic flood events gained through authority investigations and also media attention. This information will be discussed in following sections.

5.2 Overview of Modelling Approach

Hydraulic modelling of the study area has been undertaken utilising the MIKE 21 modelling system. MIKE 21 solves the full non-linear equations describing conservation of mass and momentum in two horizontal dimensions. It is commonly referred to as a full two dimensional or 2D hydraulic model. MIKE 21 has been developed by the Danish Hydraulic Institute for modeling two-dimensional flows in estuaries, bays and coastal seas. Recent developments have broadened its application to complex two-dimensional flows in river and floodplain systems.

The use of a fully two-dimensional model enables the following:

- The 2-D model computes water levels and velocities at each grid point as a function of the local ground level, bed resistance, hydraulic grade and any shear stresses from flow in adjacent grid points. As such, the model can readily describe major and minor flow paths down to the same scale as the model grid. No prior assumptions need be made as to the path the flow will take or its direction.
- The 2-D model can accurately represent flow around individual structures (such as buildings, bridges, etc.) and the formation of any eddies or flow separation zones along with their associated head losses. These are included explicitly in the model formulation, and do not need to be incorporated in the bed-friction term.

The 2-D model can provide details of water levels and velocities throughout the model domain. This detailed information can be provided on the same scale as the model grid. While the computing power is now available to model areas of the scale of the Shepparton - Mooroopna Floodplain in great detail, these hydraulic models are computationally intensive. Simulation times for individual events can vary from hours to days of computer time and are a function of:

- Processor speed,
- Number of computational points, and
- Simulation timestep.

For this reason, the study area was modelled using two separate model set-ups. Covering the entire study area, the “outer” model is based on a 25m grid. The “inner” model provides a higher level of accuracy and is based on a 12.5m grid. Note that the “inner” model uses boundary conditions derived from the results of the outer model.

In order to establish a hydraulic model of the study area the floodplain must be accurately described through the creation of a Digital Terrain Model (DTM) based on survey information as described in **Section 3.4**.

5.3 Model Establishment

The basic requirement of the hydraulic model is a Digital Terrain Model (DTM). This has been constructed from a number of sources as discussed below.

5.3.1 Photogrammetry

Photogrammetric survey of the study area was supplied by AAM Surveys P/L, based on photography flown in September '99. Data was supplied at two levels of accuracy. The two levels of accuracy will be referred to as the "General Study Area" and "Higher Accuracy Area". This information is presented and discussed in detail in **Section 3.3**. General Characteristics of these two areas are provided in **Table 5-1** below.

■ **Table 5-1 Photogrammetric Survey Characteristics**

	General Survey Area	Higher Accuracy Area
Date	Sept '99	Sept '99
Terrain Model	3D features, breaklines and spot heights on a 50m grid	3D features, breaklines and spot heights on a 25m grid
Standard error (1 sigma)	Horizontal data – 0.25m Vertical data – 0.18m	Horizontal data – 0.10m Vertical data – 0.10m

As noted in the study brief, there is a significant amount of survey information available from previous sources. However, much of this information is now quite dated in an area where significant development has occurred. Where possible, the photogrammetric data has been utilised as the primary source of topographic information as it represents the most current data source. In certain areas (specifically Mooroopna), relatively recent and accurate ground survey is available from previous investigations, specifically the Mooroopna levee investigations and associated survey (August 1984) undertaken by the Department of Natural Resources and Environment. Where possible, comparisons between these two surveys were made, with favourable results.

Note that photogrammetry is subject to the following limitations:

- No data below water level can be provided, e.g. river cross sections,
- In certain areas, vegetation obscures the ground preventing natural surface levels being derived,
- Similarly, natural surface levels are provided in the areas surrounding buildings with no levels provided on the actual buildings themselves,
- Natural surface levels beneath high crops should be viewed as indicative only,
- Ground survey is still necessary to supply accurate details for key hydraulic features e.g. levees, bridges, culverts etc.
- Ground features smaller that the resolution of the photogrammetry may not be fully represented in the resulting Digital Terrain Model (DTM). Small local levees in the south of Mooroopna are an example. To ascertain the impact of such features generally, a specific check of the impact of these levees (ie. by manually including them in the DTM) showed that these and similar sub-photogrammetry scale features have negligible impact on the hydraulic analysis and results.

Other data sources are necessary to provide additional terrain information to overcome the limitations of photogrammetry, and these are discussed below.

5.3.2 Structure Survey

Natural features that define the hydraulic characteristics of the Goulburn River floodplain include:

- Capacity of the Goulburn River, Broken River and Sevens Creek main channels,
- Capacity of the "riparian zone" in the immediate vicinity of the floodplain,
- General topography of the broader floodplain,
- Geological features such as the sandhills between Shepparton and Mooroopna.

However, the Goulburn River floodplain in the vicinity of Shepparton and Mooroopna has been significantly altered from its natural state. Such alterations include:

- Road embankments (specifically the Midland and Goulburn Valley highway), bridges and culverts,
- The Railway embankments, bridges and culverts,
- Irrigation channels,
- Drainage channels and local area drainage works,
- The urban areas of Shepparton and Mooroopna townships,
- The (bunded) wastewater treatment ponds downstream of Shepparton and Mooroopna.

Many of these features form hydraulic (flow) "controls" during floods and it is necessary to accurately define their hydraulic characteristics. For this reason, "structure" surveys were conducted to enhance the photogrammetry for key embankments or levees, and to provide information that photogrammetry could not ie bridge and culvert details. As in the case of the photogrammetry details, this information is presented and discussed in detail in **Section 3.3**.

5.3.3 Waterway Cross Sections

In order to accurately define the "in-bank" capacity of the major waterways through the study area, it is necessary to obtain information describing the main or "low flow" channel. This information was principally sourced from:

- SR&WSC River Survey & Flood Study Shepparton – Mooroopna Area, Goulburn River Cross Sections, Sheets 1-14, 1977
- SR&WSC River Survey & Flood Study Shepparton – Mooroopna Area, Broken River Cross Sections, Sheets 1-10, 1977
- SR&WSC River Survey & Flood Study Shepparton – Mooroopna Area,
- Sevens Creek Cross Sections, Sheets 1-4, 1977

5.3.4 Other Data Sources

Other data sources that were used in the formulation of the DTM and/or the hydraulic model included:

- Structure details gained through a search of the VicRoads database,
- Drainage and subdivisional details provided by GSCC,
- Miscellaneous survey details

5.4 Model Calibration

"Calibration" refers to the process whereby the raw hydraulic models are refined to adequately represent observed flooding behaviour through the study area. This process may incorporate gauged stream flows, observed maximum (or peak) flood levels, areas of inundation as shown in aerial photographs and residents or observers recollections of flooding patterns. Where the model does not adequately represent what was observed, the reason for the discrepancy is identified, the model adjusted and the additional simulations undertaken until adequate representation of the historical event is reached.

5.4.1 Historic Event Information

In terms of quantitative data, both the 1974 and the 1993 flood events are well documented and suitable for calibration of the hydraulic model. **Table 5-2** summarises the information available for each event.

- **Table 5-2 Historic (calibration) Event Data within Study Area**

Information	1974 Event	1993 Event
Observed level (and derived flow) history at Gauges	Shepparton Gauge Record	Shepparton Gauge Record
Flow gauging information	Thiess gauging along Midland highway structures	Thiess gauging along Midland highway structures
Aerial flood photography	Full study area	Full study area
Observed maximum flood levels	62	95
Other information	• Various photo's videos etc provided by the public and featured in the media	• Various photo's videos etc provided by the public and featured in the media • Opportunistic level gauging along Broken River Drive, Lincoln Drive and the Boulevard. • "Comments and thoughts" of the Manager, Engineering, Shire of Shepparton following the 1993 event.

The data describing the 1974 and 1993 events has associated varying degrees of accuracy. The aerial flood photography provides an exceedingly valuable source of information for subsequent interpretive and calibration work. While often available only at one particular time, at that time it provides information on flooding patterns, extent and thus flood level. Gauge records provide a detailed (and usually accurate) description of the time history of flooding at a particular point. Generally the calibration process aims to reproduce the recorded (gauged) levels to within +/- 100 mm. Note however, that the associated flow record is derived from the observed level history via the gauge-rating curve. Due to the difficulties in rating gauges at high flow rates, there is often an increased level of uncertainty associated with high-level derived flows. However, as presented in **Table 5-2** above, high-level flow gaugings have been obtained from Thiess for both the 1974 and the 1993 flood events at the Shepparton gauge.

There are numerous maximum flood-height observations available for the 1974 and 1993 flood events. While providing a geographical spread of data, these observations are to varying levels of accuracy and need to be treated with caution.

Where the actual water level was observed and marked either during the flood event or relatively quickly afterwards, the flood height provides a reliable observation. However, there can be uncertainties associated with:

- Debris levels providing flood heights rather than actual observations,
- Flood heights indications based on recollections years after the event.
- The impact of temporary features (eg. sandbags) or landforms which existed at the time of the flood, but have since been removed or changed and thus not reflected in the current photogrammetric data.

Furthermore, flood marks should be most reliable when marked on fixed structures such as buildings. More remote marks such as on fences in outer areas would be expected to be less reliable.

For these reasons, maximum flood height observations would be given an indicative accuracy of say +/-50 to 200mm, but on occasions accuracy could be +/-500mm.

However, for the current study it is typical to have a number of flood level observations available in a particular area. "Unreliable" maximum flood levels tend to become apparent when compared to other observed levels in the immediate vicinity.

5.4.2 General Calibration Process

The general calibration process has been to ensure appropriate representation of the total area, followed by detailed modelling of the inner area. **Table 5-3** provides some detail as to the calibration steps taken for the inner and outer area models. Note that the run numbers presented in **Table 5-3** below are indicative of the stages or steps in the calibration process. Within each step, numerous runs or sensitivity tests have been conducted. **Table 5-3** is not intended to present in detail the calculations/analysis undertaken at each step.

- **Table 5-3 Calibration Run History**

Run Number	Event	Comments/Modifications
Pre-calibration sensitivity testing		• Spreadsheet model establishment • one dimensional model establishment (based on ID&A, 1996) • Numerous 2D grid trials
Inner Area Model		
1	1993	Initial 12.5 m, 1993 grid
2	1993	Broad scale modification to topography to rectify obvious discrepancies. Hydrograph timing sensitivity tests
3	1974 1993	Broad scale modifications to topography and 1974 event simulation.
4	1993	Specific attention to floodways through Mooroopna, the Boulevard areas and South Shepparton.
5	1993	Modifications to roughness map
6	1974 1993	Further modifications to the Boulevard area, based on additional data from ID&A. Further modifications in Mooroopna flowpaths
Outer Area Model		
1	1974 1993	Initial Runs
2	Calibration (1974 & 1993) Design	Hydrograph and downstream boundary sensitivity testing
3	Calibration (1974 & 1993) Design	Modification to Wanganui floodway area
4	Calibration (1974 & 1993) Design	Further modifications to downstream boundary.

Calibration Results for the inner and outer area models are discussed in more detail in the following sections.

5.4.3 Inner-Area Calibration Results

Figure 5-1 presents the inner area hydraulic model topography.

As a result of the calibration process, the following parameters have been adopted in the inner area MIKE 21 model of the Goulburn River floodplain.

- Computational grid size 12.5m
- Time step 5s
- Eddy viscosity 0.1 m^2/s

■ **Figure 5-1 Inner Area Hydraulic Model Topography**

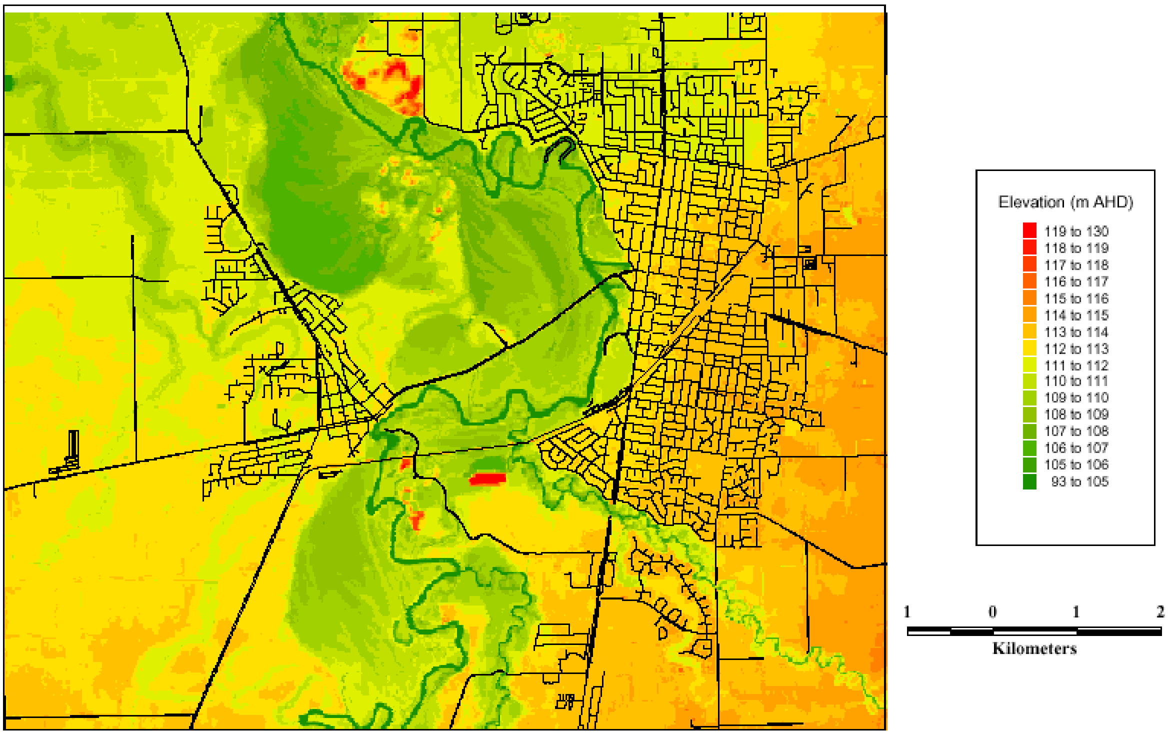


Table 5-4 details in general terms, values of hydraulic roughness adopted to represent various floodplain elements. **Figure 5-2** presents the inner area hydraulic model roughness map.

■ **Table 5-4 Adopted Hydraulic Roughness Parameters**

Floodplain Element	Manning's M	Manning's n
General floodplain(clear)	14.3	0.07
Main channel roughness	20 – 25	0.04 – 0.05
Riparian Zone	10 – 12.5	0.08 – 0.10
Urban Area	5	0.20
Clear, paved areas (streets)	50	0.02

Note the roughness adopted for urban areas and streets. Through previous experience, it has been found that the most appropriate way to model the significantly reduced flow capacity of residential areas (houses, fences etc), while accurately accounting for the floodplain storage in these areas, is to adopt an extremely high hydraulic roughness for these areas. The hydraulic model exhibits inundation in these urban areas but with higher velocities (due to the higher conveyance) in the adjacent streets. Values adopted through previous projects were utilised for this investigation and are considered a realistic representation of the complex process.

■ **Figure 5-2 Inner Area Hydraulic Model Roughness**

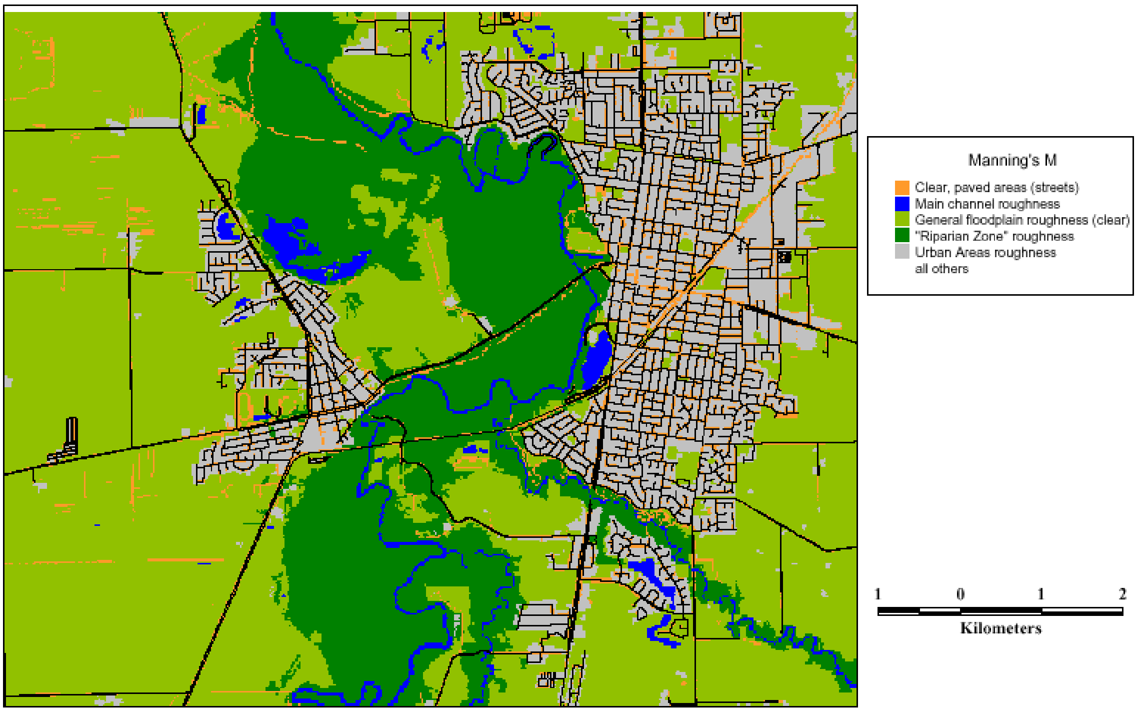
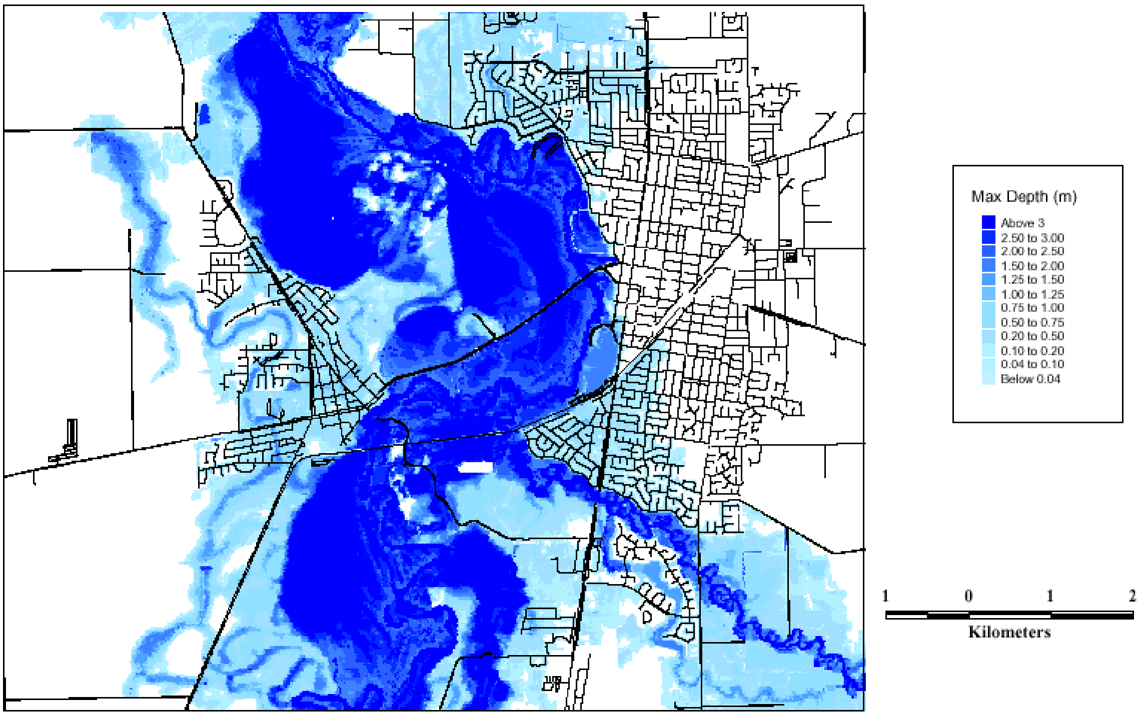
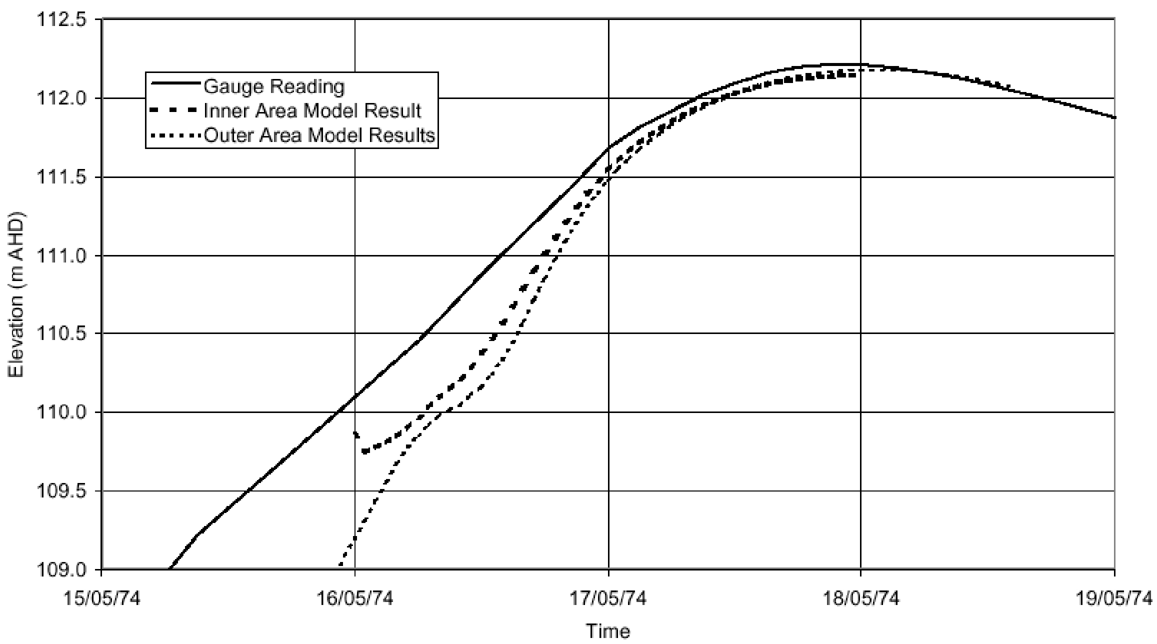


Figure 5-3 presents the predicted inundation depths across the inner model area associated with the 1974 event, while **Figure 5-4** presents the simulated time history of elevation at the gauge.

■ **Figure 5-3 Inner Area Hydraulic Model - May 1974 Predicted Inundation**



■ **Figure 5-4 Inner Area Hydraulic Model – May 1974 Simulated Gauge Height Time-Series**



Similarly, **Figure 5-5** and **Figure 5-6** present inundation depths and time histories of elevations at the gauge for the 1993 event. Note that the Shire of Shepparton undertook opportunistic gauging at a number of locations during the 1993 event. These are presented in **Figure 5-7** and **Figure 5-8**.

Table 5-5 presents a quantitative comparison of predicted and observed peak levels at the Shepparton Gauge.

■ **Table 5-5 Comparison of peak flood elevations at Shepparton gauge**

Flood level	**1974**	**1993**
Observed	112.21 m AHD	111.84 m AHD
Modelled (Inner Area)	112.15 m AHD	111.86 m AHD
Comparison	-0.06 m	+0.02 m

Outer model results are also reported at the gauge location. As for the inner model, the outer model results are within 0.06m of the observed gauge elevations. The rising limb of the predicted hydrographs at the gauge is under-predicted at the gauge as exhibited in **Figure 5-4**. This is a function of initial conditions for the model. Due to the significant model simulation times required, the initial conditions used represent a compromise between simulation effort and accurate definition of inundation of the floodplain at lower levels.

Thiess undertook gaugings during both the 1974 and 1993 events along the Midland Highway (represented by the discharge rating at the Shepparton Gauge). This information has been obtained, and is presented in **Table 5-6** below along with the discharges derived from the hydraulic model.

■ **Table 5-6 Comparison of discharges through Midland Highway Causeway openings**

Causeway opening	**Thiess Gaugings (m^3/s)**		**Model Results (m^3/s)**		**Comparison**	
	1974	1993	1974	1993	1974	1993
Geraghty's Bridge	54	73	69	50	+29%	-32%
AH Wong Bridge	154	127	149	112	-3%	-12%
Boolbadah Fldwy	272	133	241	187	-11%	+40%
Daishes Bridge	150	208	164	131	+9%	-37%
McGuires Bridge	382	402	505	418	+32%	+4%
Dainton Bridge	1,068	795	1,010	831	-5%	+5%
Total	**2,080**	**1,740**	**2,138**	**1,729**	**+3%**	**-1%**

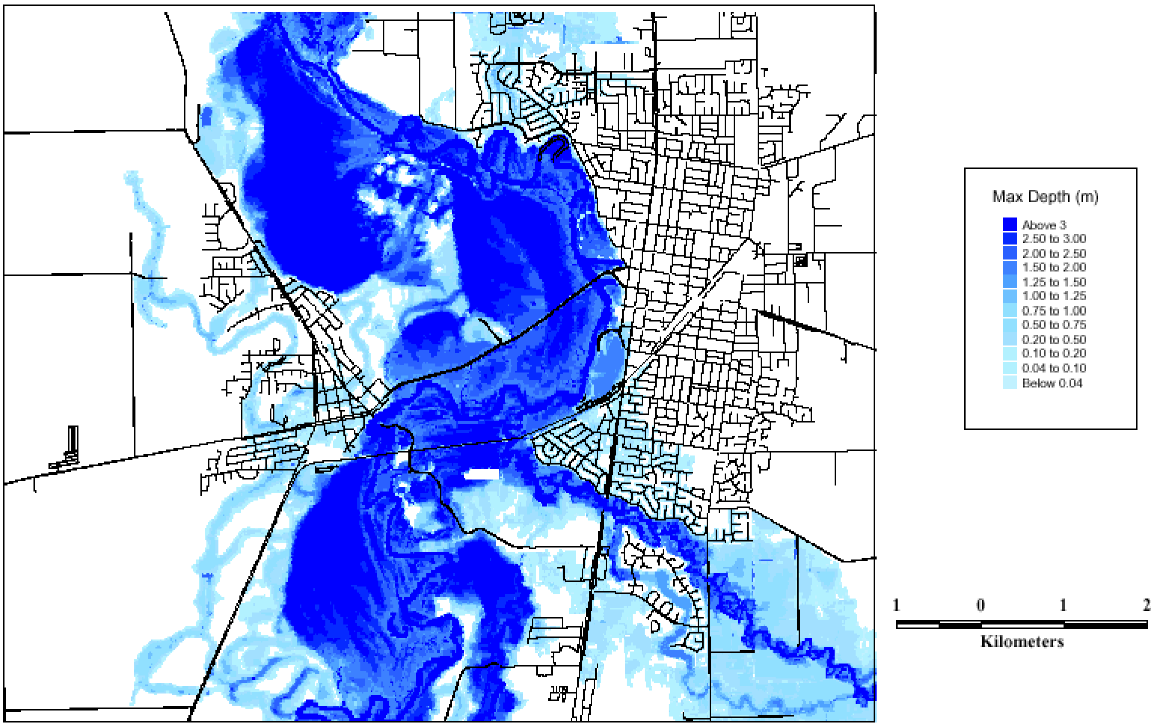
There is a deal of spread in the discharge results which is to be expected given the number of variables present in both discharge calculation and measurement. These results indicate that the predictions are generally of the right order, and do not indicate that systematic errors are present in the representation of flows across the floodplain.

When combined with the property survey information, an overall picture of the model's ability to replicate the observed property inundation is gained. **Appendix D** presents the observed maximum flood levels as gathered during the course of this study. **Appendix C** presents the comparison of predicted levels and the observed levels. The comparisons are in the form of:

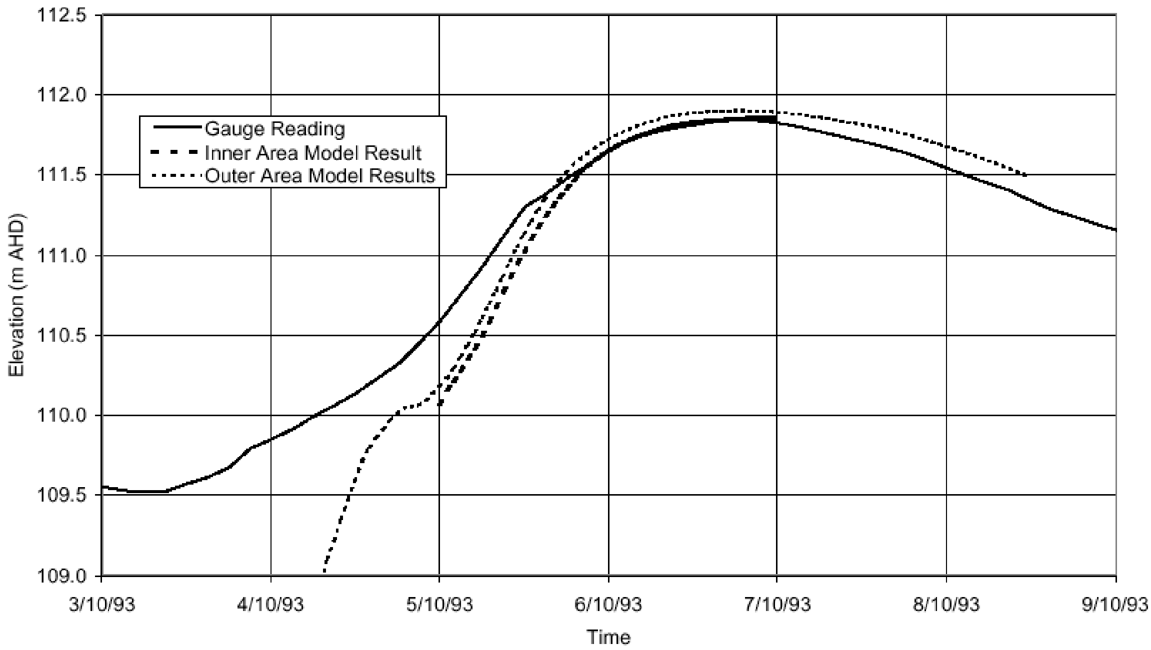
Comparison = Predicted level – Observed level.

A value of 0.1m indicates that the predicted level is 0.1m higher than the observed level. A value of –0.1 indicates that the predicted level is 0.1m lower than the observed level. Details are presented in the Mooroopna, South Shepparton and North Shepparton areas.

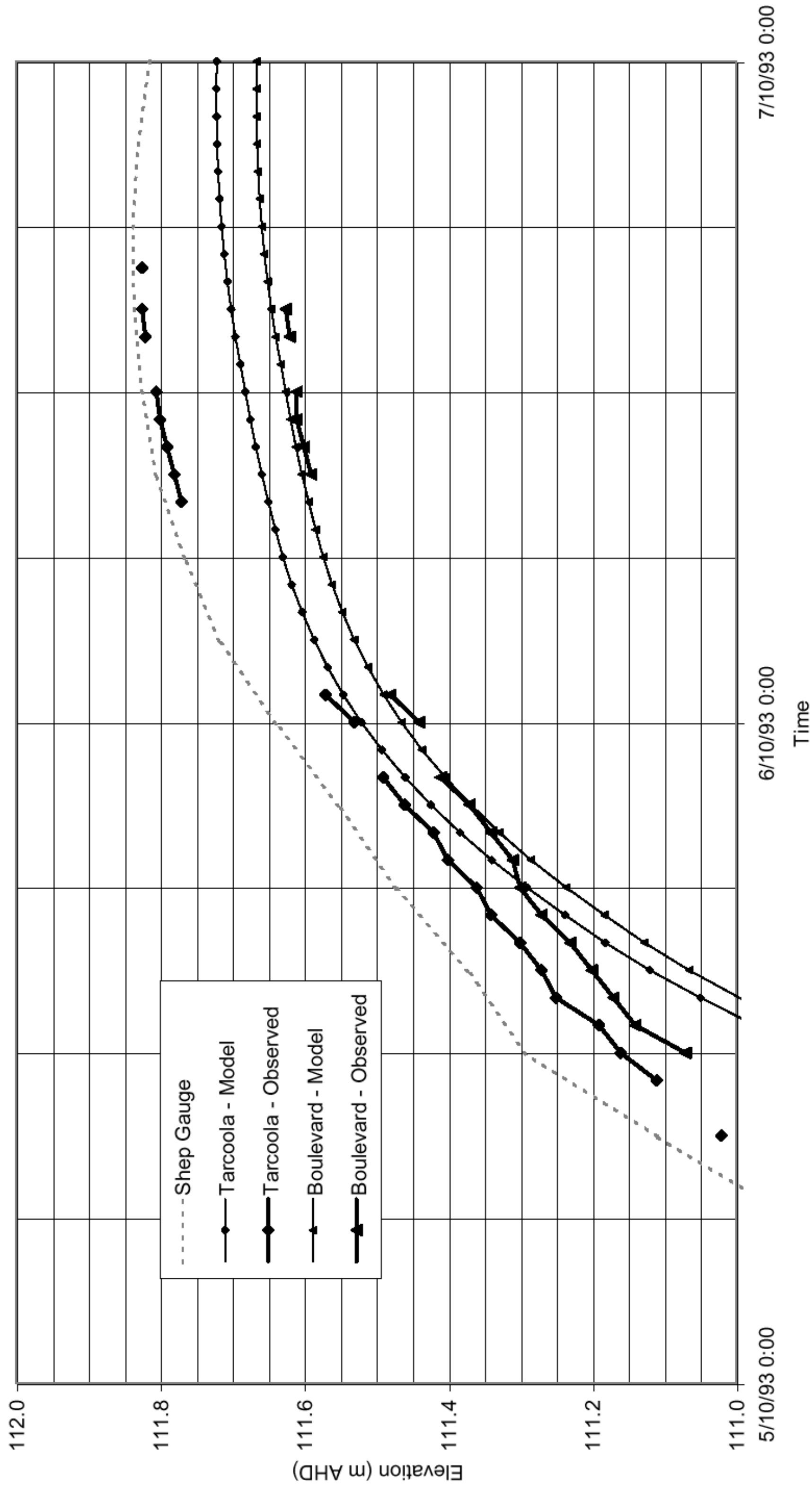
- **Figure 5-5 Inner Area Hydraulic Model – October 1993 Predicted Inundation**



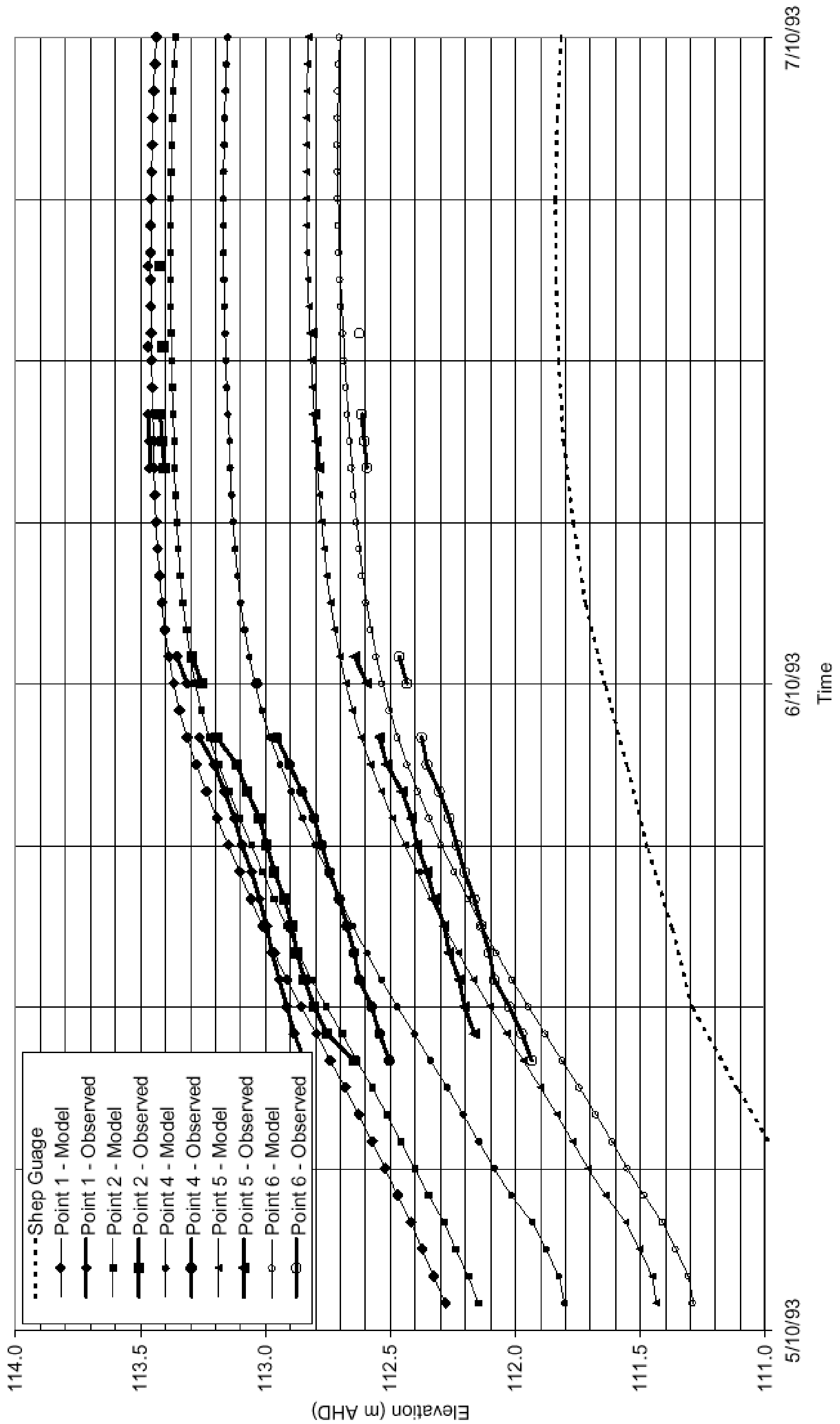
- **Figure 5-6 Inner Area Hydraulic Model – October 1993 Simulated Gauge Height Time-Series**



■ **Figure 5-7 Inner Area Hydraulic Model – October 1993 Tarcoola/Boulevard Gaugings vs Model Predictions**



■ **Figure 5-8 Inner Area Hydraulic Model – October 1993 Broken River/Lincoln Drive Gaugings vs Model Predictions**



In general the results compare well for the three areas albeit with a large spread. The spread of comparisons presented in **Appendix D** needs to be interpreted in conjunction with the spread of observed levels in **Appendix C**. In general, the results in South Shepparton and North Shepparton agree with the observed flood levels.

This is confirmed by comparison with the aerial flood photography for these areas. The comparisons indicate that there may be some overestimation in areas of Mooroopna. This is thought to be principally a function of flood transmission through the rail embankment as detailed representation of temporary flood protection works has not been undertaken in these areas.

It is also likely that actions taken in Mooroopna, in particular sandbagging, to protect property during the floods will have influenced flood patterns and reduced actual flood levels at some properties.

Summary
These results indicate general consistency with the observed flooding behaviour for the historic events. The comparison of model predictions and observed flood levels in terms of above or below floor flooding is affected by a number of variables. Some of these areas of uncertainty are:

- Accuracy of flood marks,
- Location of nominated result points (eg on the high or low side of relatively steeply sloping blocks),
- The extremely complex and localised flow patterns that impact on flow through urbanised areas (eg local fences, walls, debris blockages etc).

It is not possible to accurately model all of these effects at the scale at which this investigation has been undertaken. However, as a result of the calibration process, the model reasonably reproduces not only the observed peak flood levels along the major watercourses, but the pattern of above-floor flooding as observed by residents.

5.4.4 Outer-Area Calibration Results

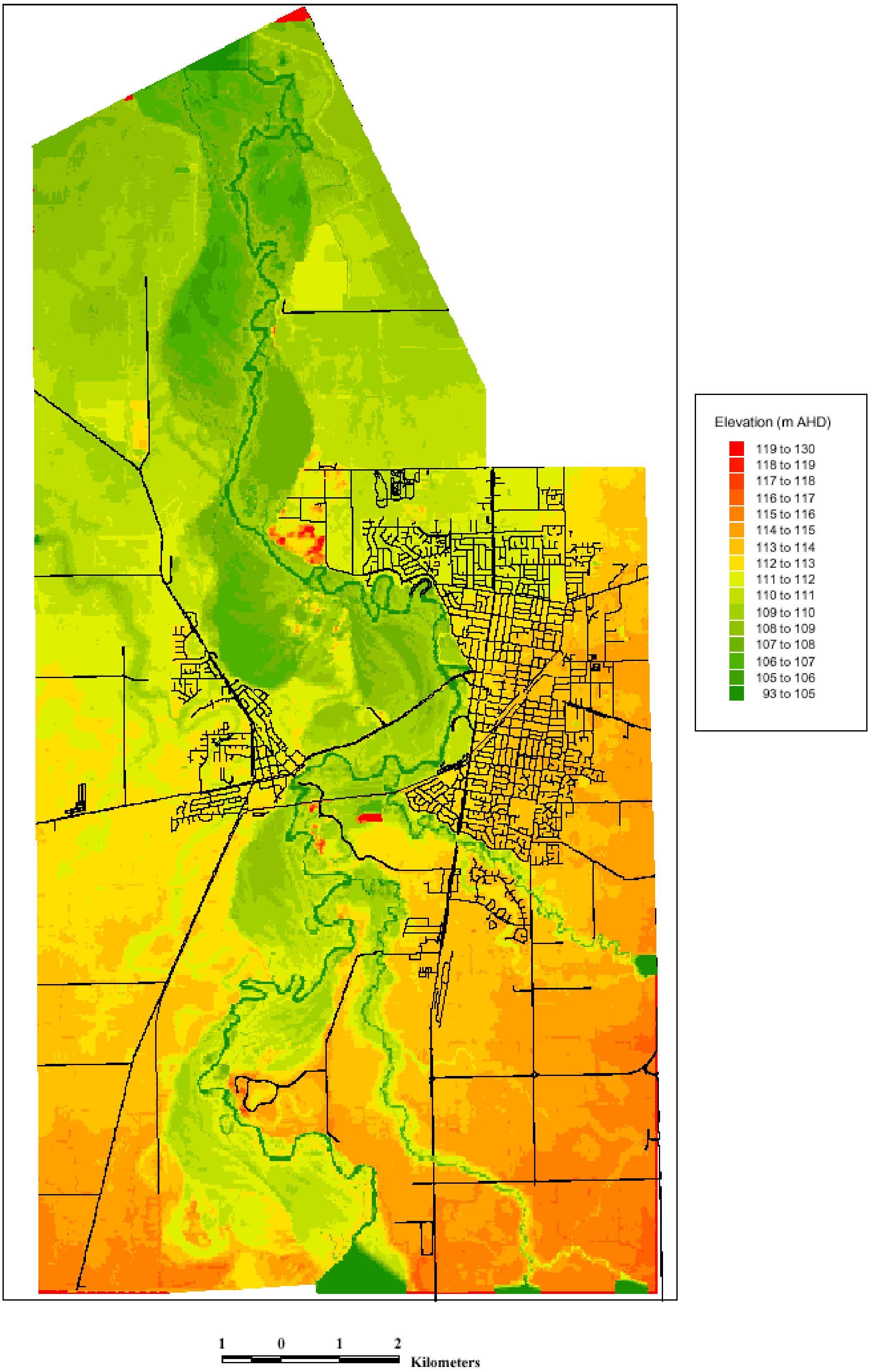
Figure 5-9 presents the outer area hydraulic model topography. As a result of the calibration process, the following parameters have been adopted in the inner area MIKE 21 model of the Goulburn River floodplain.

- Computational grid size 25m
- Time step 5s
- Eddy viscosity 0.1 m^2/s

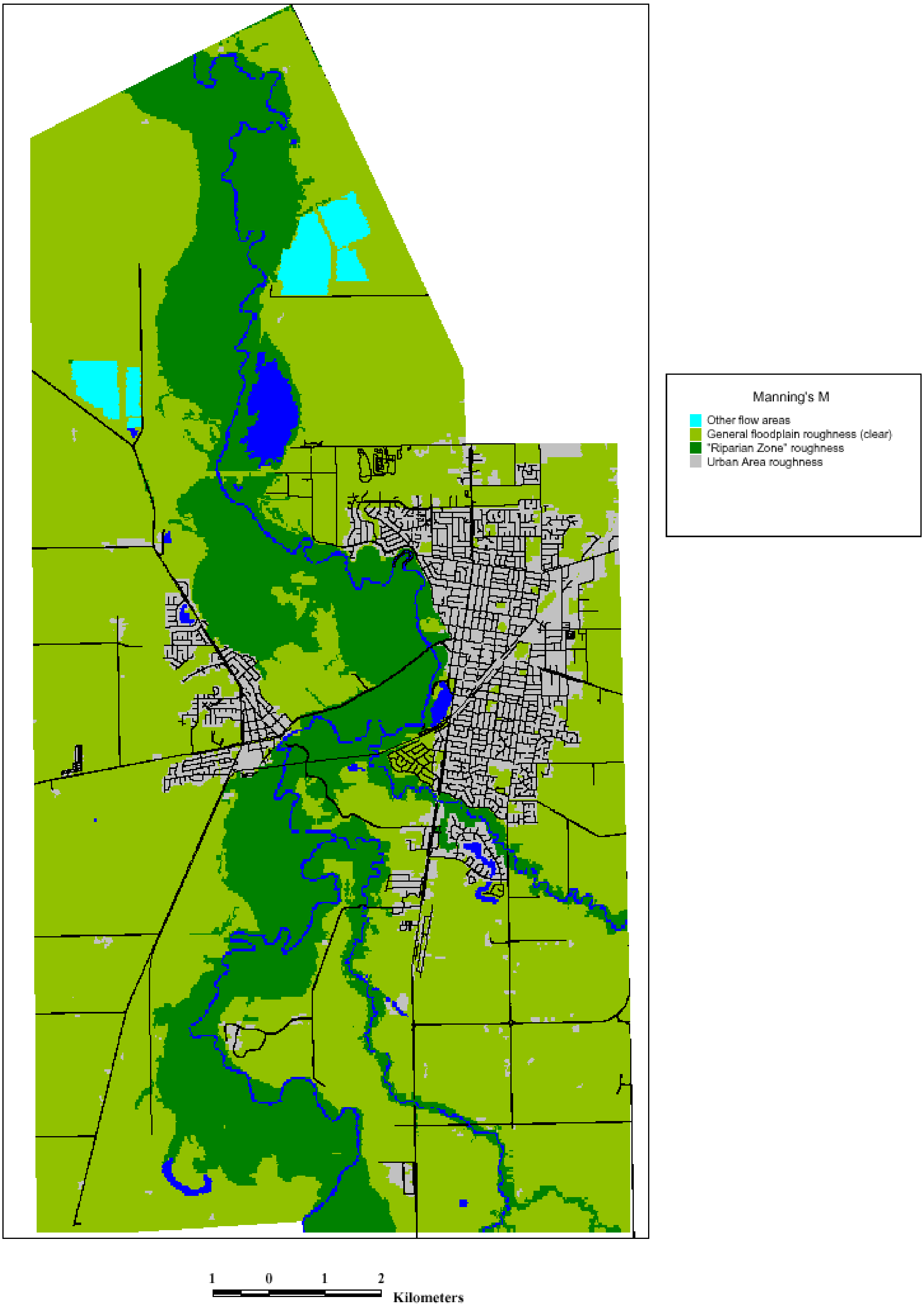
In general, the same hydraulic roughness parameters as utilised in the inner area model were adopted for the outer area model. **Figure 5-10** presents the hydraulic roughness map adopted for the outer area model. The chief difference between the outer and inner area roughness maps is in the urban areas where the outer area model does not differentiate between built up areas and streets at the same resolution as the inner area model does. For this reason, inner area model results should be utilised in these areas.

Figure 5-11 and **Figure 5-12** present the predicted inundation depths for the May 1974 flood event, while **Figure 5-13** and **Figure 5-14** present predicted inundation depths for the October 1993 event.

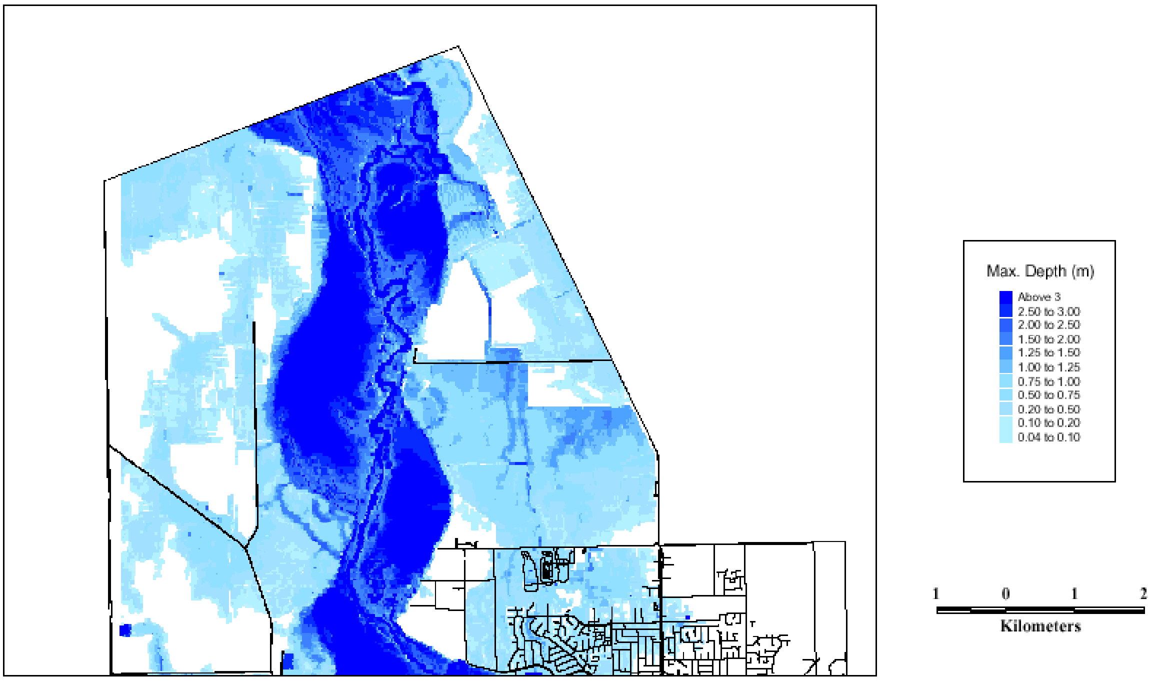
■ **Figure 5-9 Outer Area Hydraulic Model Topography**



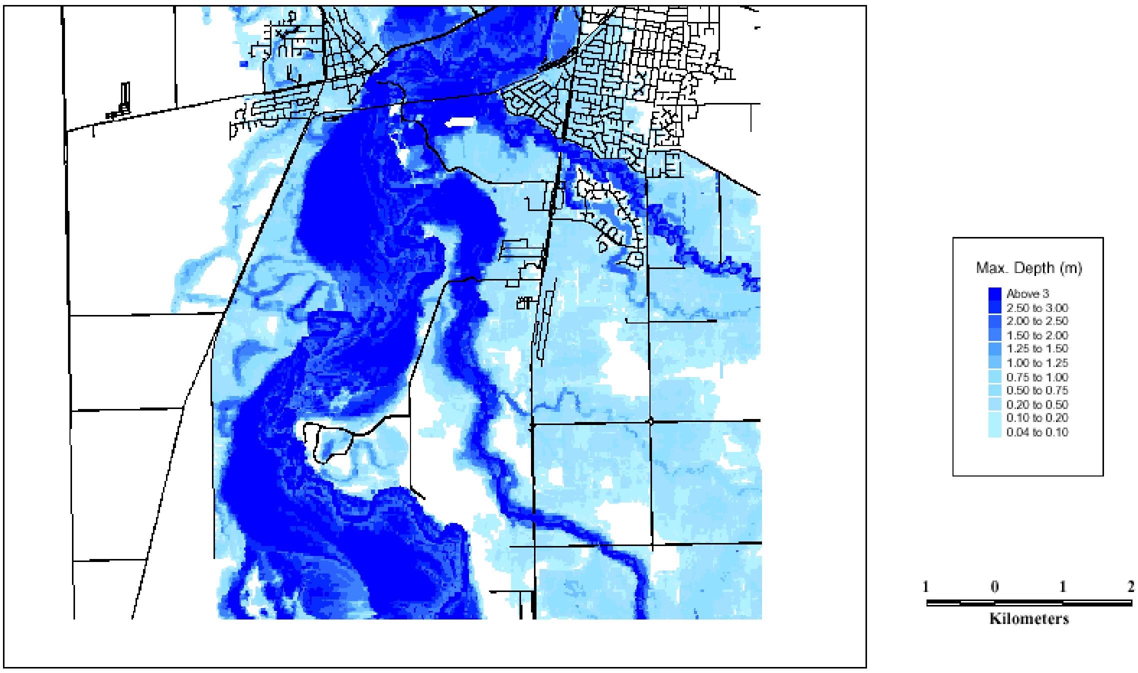
■ **Figure 5-10 Outer Area Hydraulic Model Roughness**



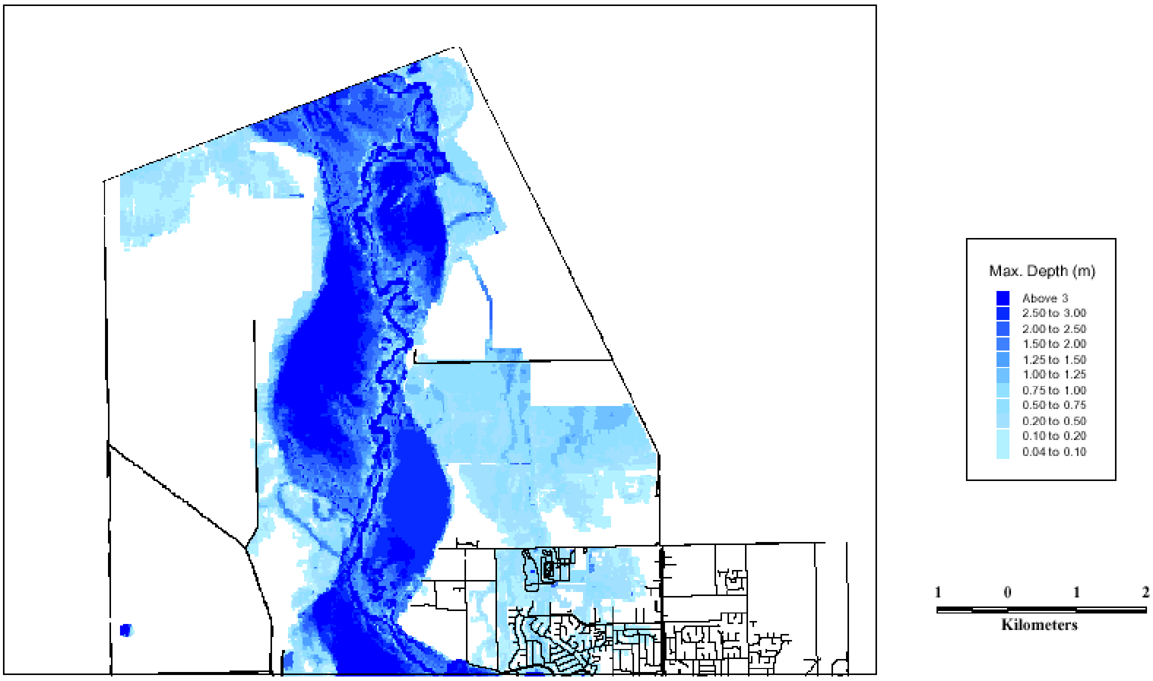
■ **Figure 5-11 Outer Area Hydraulic Model – May 1974 Predicted Inundation Northern Portion**



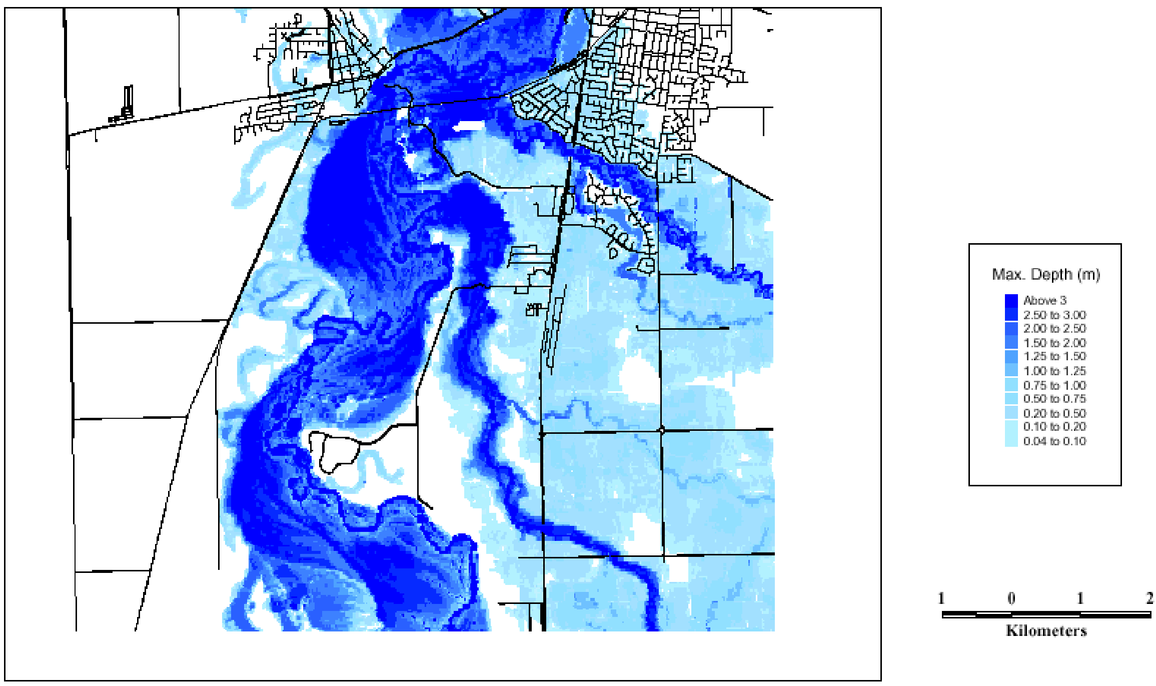
■ **Figure 5-12 Outer Area Hydraulic Model – May 1974 Predicted Inundation Southern Portion**



- **Figure 5-13 Outer Area Hydraulic Model – October 1993 Predicted Inundation Northern Portion**



- **Figure 5-14 Outer Area Hydraulic Model – October 1993 Predicted Inundation Southern Portion**



Note that no attempt has been made to modify the existing situation topography to replicate the condition of the northern portion of the floodplain as it was at the time of the 1974 event. In particular, the 1974 event has been run with the second water treatment pondage in its current configuration.

Note that the results from this model were used to generate boundary conditions for the inner area model.

5.5 Model Outputs

Once calibrated, the models were used to generate flood levels for various design event combinations derived in **Section 4.7** and for the probable maximum flood derived in **Section 4.8**. A theoretical rating curve at the location of the Shepparton Gauge was also constructed for comparison with the current Shepparton Gauge on the Goulburn River, Number 405204 (see **Table 4-1**), discussed in **Section 4.3.2**.

5.5.1 Design Events

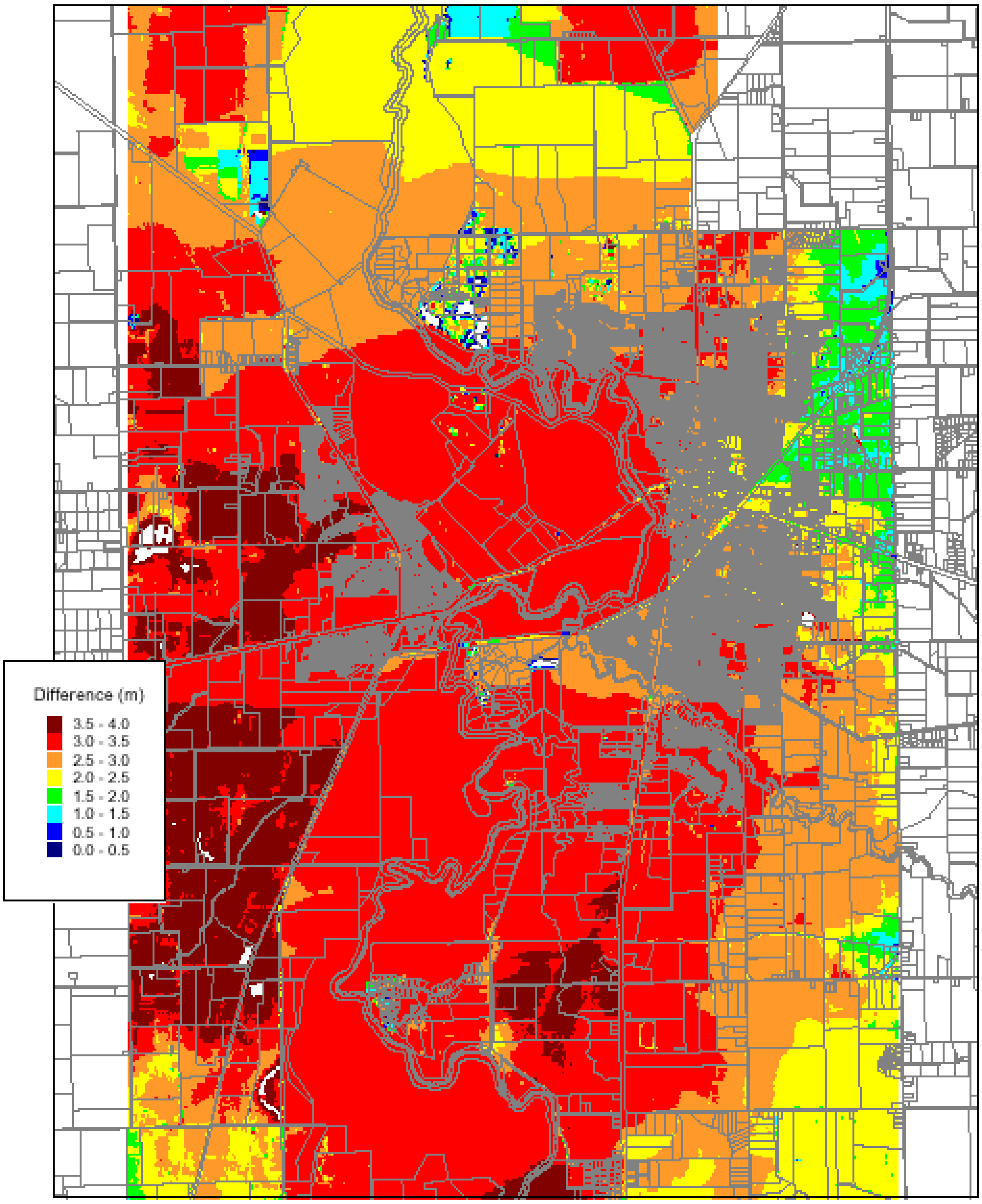
Design event were used for the flood damage analysis (**Section 6**), including determining the existing level of risk (**Section 7**) and for land use planning mapping (see **Section 8**) and flood inundation mapping (see **Section 9**). Planning and flood inundation maps are bound in separate volumes.

5.5.2 Probable Maximum Flood

Given the magnitude of this event and the simplified methods used to determine probably maximum flood (PMF) inflows, formal mapping was not appropriate. Rather, the model was used to provide indicative expected flood depths, in particular above the 100 year ARI flood. **Figure 5-15** shows the *difference* between the PMF flood levels and the 100 year ARI flood levels (for existing conditions) in the central section of the study area.

It is important to emphasis that these estimates are indicative. **Figure 5-15** clearly demonstrates that the model boundaries rather than the natural surface levels are constraining the flood, most noticeable at the western limit of the model. This leads to an overestimate of the actually flood level, as an actual PMF would cause flood waters to spread further west in particular, lowering levels within Shepparton Mooroopna. There is also some uncertainty as to the appropriate downstream model boundary water level (which must be set for any model run). **Figure 5-15** suggests that the PMF level is around 3 to 4 m deeper than the 100 year ARI flood level across the study area. The true difference may well be much less.

■ **Figure 5-15 Probably Maximum Flood Impact – Water Surface Level Difference (above the 100 year ARI event levels)**



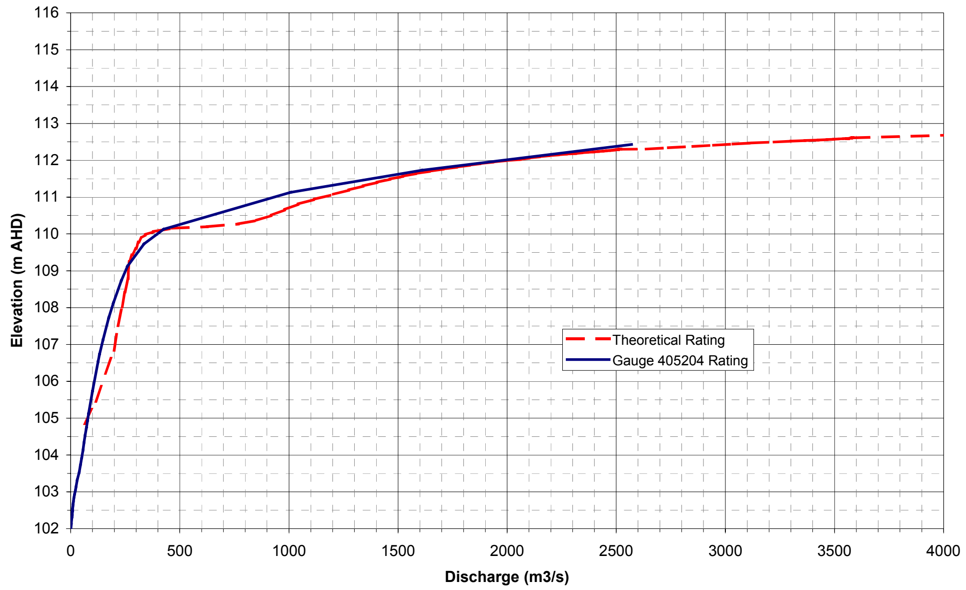
5.5.3 Shepparton Gauge - Theoretical Rating Curve

The MIKE 21 model provides the means to construct a theoretical rating curve (ie. a stage-discharge relationship) along any section within the model domain. When taken along the line used for Shepparton Gauge, Number 405204, it provides an opportunity to compare and assess the actual Gauge rating curve.

Figure 5-16 shows a comparison this theoretical rating curve with the current Shepparton Gauge (405204) rating curve. The theoretical curve has been constructed for a section that runs along and parallel to the Midland Highway extending through Mooroopna.

It demonstrates very close correlation up to the limit of the current rating curve (approximately a 100 year ARI flood peak flow). Note that whilst the model was run for the Probable Maximum Flood (PMF), **Section 5.5.2** notes that PMF water level are likely to be overestimated. For this reason, the theoretical rating has been truncated at around 4000 m^3/s, just in excess of the 500 year ARI flood peak flow.

- **Figure 5-16 Comparison of theoretical rating with actual rating at the Shepparton Gauge, Number 405204**



6. Flood Damages Assessment

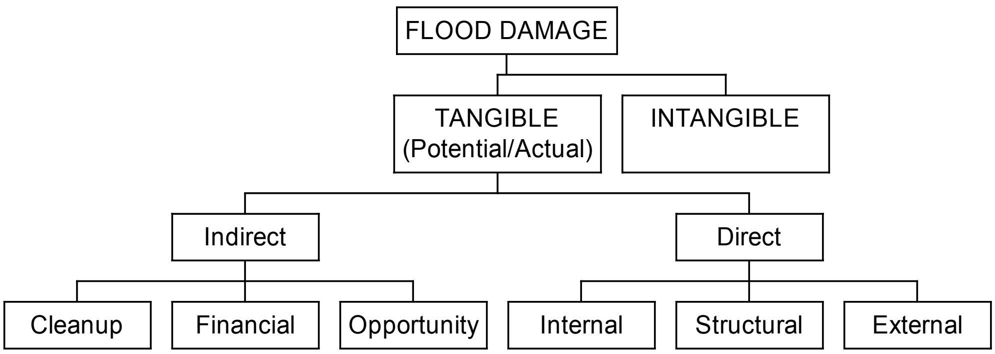
6.1 General

A flood damages assessment has been undertaken for Shepparton Mooroopna to determine the scope of damages under existing conditions and to assess the merits of various flood mitigation options in reducing those damages. The assessment is a key component in the determination of existing flood risk and a preferred flood mitigation scheme, via estimates of damages and as input into a financial benefit-cost assessment.

The term "flood damage" generally refers to the cost (monetary or otherwise) of damage resulting from flooding. However, it can also refer to the extent of damages (ie. number of affected properties and how they are affected), an equally important factor in a flood damage assessment.

Damages from flooding can be sub-divided into a number of categories. **Figure 6-1** shows the various categories commonly used in flood damage assessments.

- **Figure 6-1 Categories of Flood Damage**



Tangible flood damages are those to which a monetary value can be assigned and include property damages, business losses and recovery costs. Intangible flood damages are those to which a monetary value cannot be assigned and include anxiety, inconvenience and disruption of social activities. Both are a function of flood magnitude. This flood damages assessment focuses on the tangible flood damages. Intangible damages are important and are considered, but under the broader assessment of existing conditions and flood mitigation options.

Tangible damages can be sub-divided into direct and indirect damages. Direct damages are those financial costs caused by the physical contact of flood waters and include damage to property, roads and infrastructure.

Property damages can be sub-divided into internal and external damages. Internal damages include damage to carpets, furniture and electrical goods. External damages include damages to building structures, vehicles and in rural areas, crops, fencing and machinery.

Indirect damages are those additional financial costs generally incurred after the flood during clean-up and include the cost of temporary accommodation, loss of wages, loss of production for commercial and industrial establishments and the opportunity loss caused by the closure or limited operation of business and public facilities.

Tangible damages can also be treated as potential or actual damages. Potential damages are the maximum damages that could occur for a given flood event. In determining potential damages, it is assumed that no actions are taken (whether months or hours) prior to or during the flood to reduce damage by, for example, lifting or shifting items to flood free locations, shifting motor vehicles or sandbagging. Actual damages, in this context, are the expected damages for a given flood event. Their value - a proportion of potential damages - is based on the community's flood preparedness, a function of community awareness and the lead-time of flood warnings.

6.2 Damage Assessment Methodology

Central to the assessment is the collection of property data for all potentially flood prone properties for the range of flood events being considered. Flood level data for each event of interest are applied to property data to determine flood depths and from this, flood damages at each property. Total damages are simply a summation of damages for each property, combined with estimates for other community-wide damages including infrastructure and services.

The primary tool for the assessment was a flood damage model developed in-house. The model was constructed in a GIS environment using the ArcView software package. The GIS environment provides a visual representation of the assessment. It can show building locations, flood extents and flood affected properties as well as list property data and calculate resulting damages. The methods and damage data used in the model are largely those of a model called ANUFLOOD (Smith and Greenaway, 1992) developed by the Centre for Resource and Environmental Studies (CRES) at Australian National University. A recent study, the Rapid Appraisal Method (RAM) for Floodplain Management (NRE, 2000), has provided additional damage data and recommendations on appropriate adjustments to the ANUFLOOD data. These data and adjustments have been used in this assessment.

The assessment involved determining total actual damages (both extent and cost) for each of a range of flood events under given floodplain conditions (ie. existing conditions or some flood mitigation scheme conditions). Actual damages are derived by applying a damage reduction factor (DRF) to potential damages.

The range of total damage estimates can be combined to determine an average annual damage (AAD) or annual damage cost to the community for a given floodplain condition. The AAD is commonly used in floodplain management studies, as it is a useful single value indicator of the financial vulnerability of a community to flooding in existing conditions and of the benefit of proposed mitigation schemes.

6.3 Flood Damage Assessment Data

The data requirements for the flood damages assessment are significant. They fall into two broad categories – physical data and damage costs. The types and sources of data are discussed in the following sections.

6.3.1 Physical Data

<u>Property Data</u>
Fundamental to the assessment is the compilation of property data for properties potentially affected by flood. Data were collected as described in **Section 3** and input

into a property database within the GIS flood damage model. For the assessment methods adopted in this study, the following property data are required. A brief description of why these data are important and how they are used is also provided.

Building Location
The building location must be defined by both property address (Street Number and Street Address) and ground coordinates. The address is required to identify physical building location as used in the flood affected property listings (See **Section 9**). The building coordinates are used by the GIS model to identify the flood level at each building and to map the building location.

Building Type
The building type for each property is a major factor in determining the expected damage for a given flood depth. Building type includes residential, commercial, industrial and public. The property survey also made a distinction between urban residential and rural residential buildings. Damages will differ with building type. The flood damage model considers two building types, residential and commercial. The latter is used to cover all non-residential building types, namely commercial, public and industrial.

Property Damage or Value Class
Property value is an important determinant in flood damages. “Property” refers to buildings and their contents. The flood damage model requires each building to be assigned to a damage or value class. The class determines which flood depth-versus-damage data are used for each building (see **Section 6.3.2**). Class categories differ for residential and commercial buildings. For residential buildings, damage class is a function of building material and condition. Each residential building is assigned to one of three residential damage classes. The classes are intended to represent dwellings of respectively poor, normal or excellent value from a flood damage viewpoint.

For commercial (ie. “non-residential”) buildings, value class is primarily a function of building contents, although building material and condition are also factors. Building size is very important and as such is considered separately in the model (see Commercial Building Size Class below). There are five commercial value classes. The “very low” class typically includes offices, sports pavilions and churches, the “medium” class typically includes libraries and clothes businesses and the “very high” class typically includes electronics and camera businesses. However value class is also a function of building contents. For example, a furniture store with extensive stock of lower quality furniture may be assigned to a higher value class than a furniture store with limited stock of high quality furniture because the value of its contents is greater. It is the value of the business and its contents that determines its value class.

Commercial Building Size Class
Commercial buildings are assigned to one of three size classes based on the building areas as follows.

- Small: < 186 m^2
- Medium: 186 - 650 m^2
- Large: > 650 m^2

For commercial buildings, it is the value and size classes that determine which flood depth-versus-damage data are used for each property (see **Section 6.3.2**).

Ground and Floor Levels
In order to determine a flood depth at each building, ground and floor level data including location (ie. coordinates) must be obtained for each building.

Property Data Summary
Property data was collected for the total study area, and targeted all buildings then estimated to lie within the 100 year ARI flood extent.

Table 6-1 provides a summary of property data by building type for the total study area.

- **Table 6-1 Property data summary**

Building Type	Total Study Area
Urban Residential	8,598
Rural Residential (including farm buildings)	415
Commercial	453
Recreational	20
Industrial	27
Public	82
TOTAL	**9,595**

Infrastructure Data
Infrastructure includes all roads and services. For this assessment, infrastructure was represented by road length. This is a reasonable assumption as much of the service infrastructure follows the paths of road reserves and the quantity of other infrastructure might be expected to be broadly a function of the length of road.

Roads were subdivided into three categories as used in the RAM report (NRE, 2000) – highway, sealed road and unsealed road. Each was determined using the cadastral information supplied by GSCC and by inspection of aerial photos. These data were used to determine inundated road lengths for given flood scenarios.

Flood Data
Flood data for the study area were determined via hydraulic analysis, described in detail in **Section 5**. For study area, the MIKE 21 model produced results as a grid of flood levels.

By overlaying the flood data onto the property data, a flood level can be assigned to each flood affected building. Flood depths (ie. above ground and above floor) for each building were then calculated. Inundated areas can be calculated directly from the flood data and lengths of inundated road can be calculated by overlaying the flood data onto the road data.

6.3.2 Damage Costs

Property Damages
Property damage data are used to convert flood depths into monetary damages. Data are required for both direct and indirect damages. Direct damages can be further sub-

divided into internal and external damages. The data must also cover the different types or categories of buildings/properties in the floodplain.

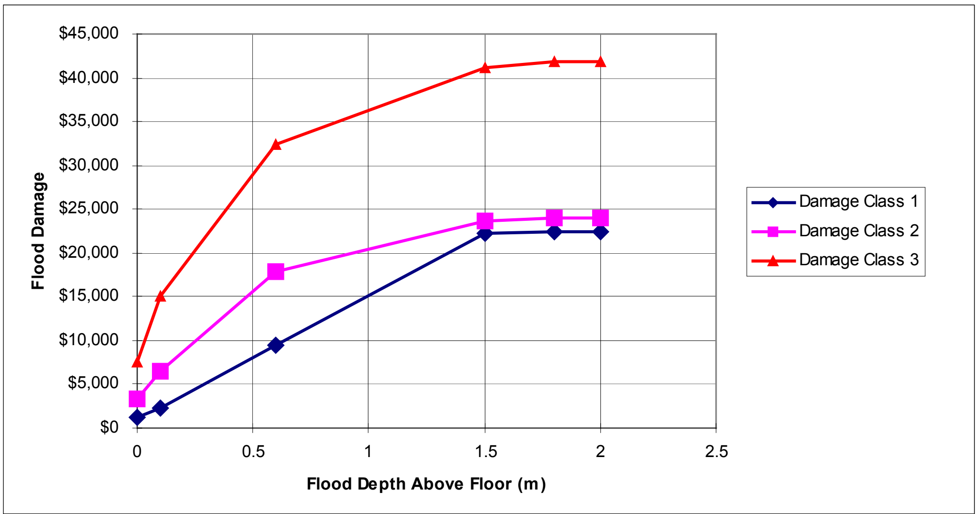
Following is a brief description of the property damage data used for each damage type or category. All data are embedded in the GIS flood damage model.

Direct Damages
Direct damage curves have been taken from the ANUFLOOD program. There are eighteen curves, three for residential properties (for 3 damage classes) and fifteen (for 3 size classes by 5 value classes) for commercial properties. Each relates flood depth above floor with monetary cost of the internal damage.

The RAM report (NRE, 2000) notes that the ANUFLOOD data underestimates potential damages by 60%, primarily due to the age of the data. Note however that this also includes an allowance for external damages, which is not part of the ANUFLOOD data. (The RAM report does not provide separate data for external damages). For properties flooded above floor (ie. where ANUFLOOD data is available), the 60% factor has been applied and separate external damage calculations have not been necessary.

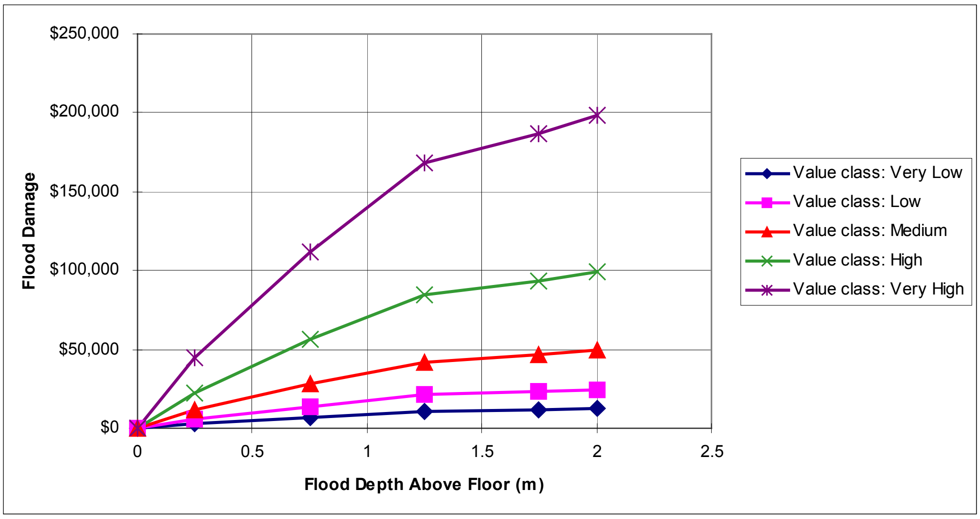
Figure 6-2 reproduces the adjusted direct damage curves used for this flood damages assessment for residential buildings.

■ **Figure 6-2 Residential Total Damage Curves**

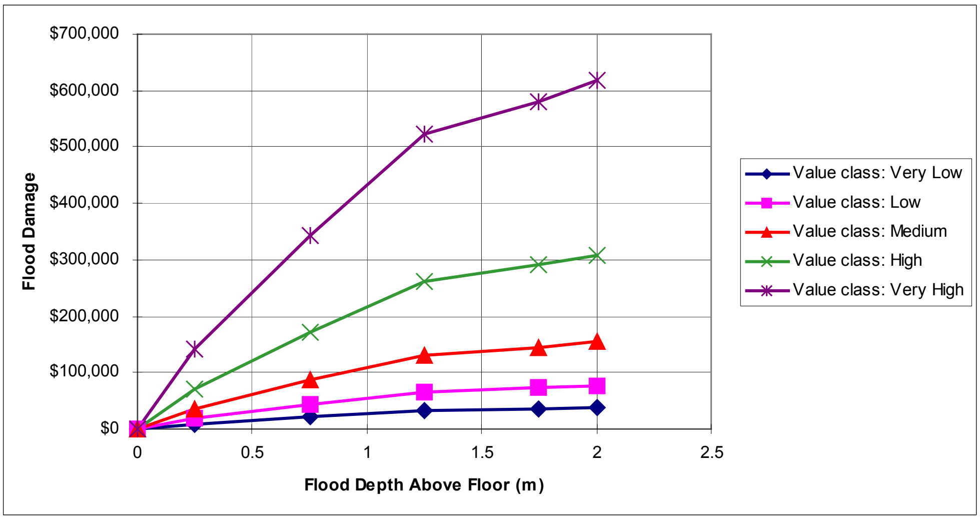


Figures 6-3 to 6-5 reproduces the RAM adjusted internal direct damage curves used for this flood damages assessment for the three size classes of commercial properties. Note that damages for large commercial properties (**Figure 6-5**) are a function of floor area.

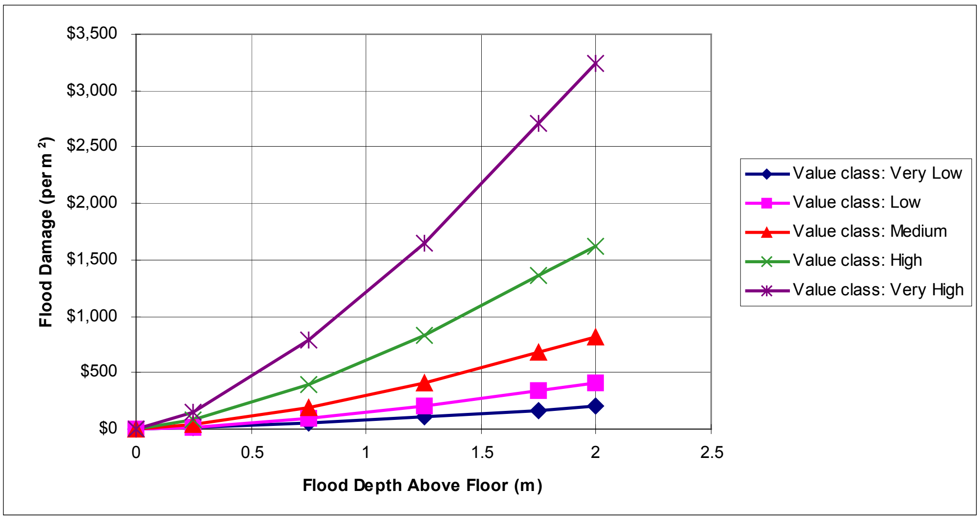
■ **Figure 6-3 Small (<186 m²) Commercial Total Damage Curves**



■ **Figure 6-4 Medium (186 - 650 m²) Commercial Total Damage Curves**



■ **Figure 6-5 Large (>650 m²) Commercial Total Damage Curves**

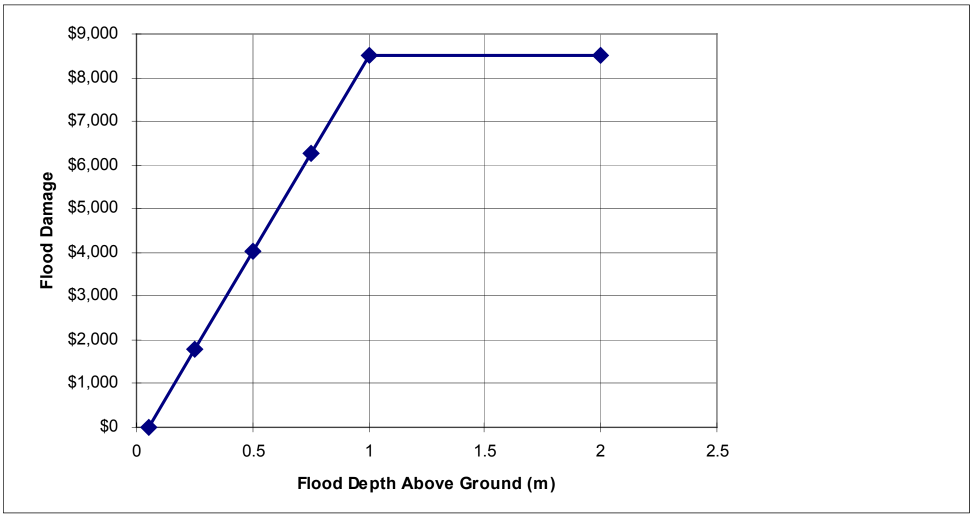


External Damages

For properties flooded below floor only (ie. where ANUFLOOD data is not available), separate external damages have been calculated. An external direct damage curve has been developed using data from Floodplain Management in Australia, Volume 2 (DPIE, 1992). It assumes that external damages commence at a flood depth above ground of 0.05 m and vary linearly to an upper limit of $8 500 at a flood depth above ground of 1 m. No distinction is made between residential and commercial properties.

Figure 6-6 shows the resulting external direct damage curve used for this flood damages assessment for all properties.

- **Figure 6-6 External Damage Curve**



Indirect Damages

Indirect damages are calculated as a percentage of total direct damages. The percentage values used in ANUFLOOD are assumed a function of land use only. **Table 6-2** lists these percentages used by ANUFLOOD.

- **Table 6-2 Indirect Damages (as a percentage of Direct Damages)**

Residential	Commercial	Industrial
15%	55%	70%

However, the RAM report (NRE, 2000) suggests that "in most cases" indirect cost be calculated as simply 30% of the total direct damage. This approach would usually net estimates that exceed the ANUFLOOD estimates. The RAM approach was adopted for this assessment.

Damage Reduction Factors

As the above damage data is based on potential damages, damage reduction factors (DRFs) must be applied to reflect expected actual damages. The DRF is simply a ratio of actual damage to potential damage. DRFs can range from 0.9 for inexperienced communities with less than 2 hours flood warning to 0.4 for experienced communities with more than 12 hours flood warning (NRE, 2000). For Shepparton-Mooroopna, a DRF of 0.7 was adopted (inexperienced community, warning time greater than 12 hours).

Infrastructure Damages

Damage to infrastructure includes street and road repairs (including restoration of weakened subgrades), bridge repairs, telephone and telecommunications facilities, electrical connections, water supply and sewerage infrastructure and resulting higher maintenance costs.

The RAM report (NRE, 2000) provide infrastructure data for “roads and bridges”. It does not provide any damage estimate for other infrastructure but notes that “damages for other regional infrastructure (telecommunications, electricity, water, sewerage and other underground services) are small relative to roads and bridges”.
In the absence of “other” infrastructure damage data, the “road and bridges” has been used as representative of all infrastructure.

Table 6-3 summarises the adopted monetary damages for the infrastructure represented by inundated road length found in the study area.

- **Table 6-3 Inundated Infrastructure Damages (via Road Lengths)**

Road Type	Damage ($/km)
Highway	59,000
Sealed Road	18,500
Unsealed Road	8,400

Note that the analysis did not consider the influence of flood depth, flow velocity or inundation time on infrastructure damages.

7. Flood Risks under Existing Conditions

The impact of flooding on a community can be usefully summarised in terms of Flood Risk. Flood Risk is defined as the product of likelihood of flooding and consequence of flooding. That is:

*Flood Risk = Likelihood * Consequence*

Likelihood of flooding is represented by the average return interval (ARI) of a given flood depth. Consequence of flooding is represented by the resulting monetary flood damages resulting from that given flood depth. Total flood risk is then derived as the summation of flood risk at all points across the study area for each of a range of flood events.

Flood risk can be compared under existing conditions and under a given mitigation option to determine the benefits of the mitigation option.

7.1 Existing Likelihood of Flooding

The likelihood of flooding for Shepparton-Mooroopna has been determined via hydrologic and hydraulic modelling (see **Sections 4** and **5**). Likelihood can be represented graphically by flood maps. A number of flood maps have been produced for this study (see **Section 9**). Likelihood is demonstrated within **Figure 7-2**, represented by the three flood events and the depths and extents these floods reach. For each of these and any other event, the depth and extent of flooding will have a particular and net consequence (see below).

7.2 Existing Consequence of Flooding

The consequence of flooding for Shepparton-Mooroopna has been determined via the flood damage assessment (see **Section 6**). Consequence is represented as both numbers of properties flood affected (both above and below floor) and associated monetary damages (including infrastructure damages) for any given flood event. **Table 7-1** provides a summary of existing consequences for the total study area for a Goulburn River dominant event.

- **Table 7-1 Existing Consequences of Flooding (Goulburn Dominant Event)**

Flood Damage Data	**ARI (years)**					
	10	**20**	**50**	**100**	**200**	**500**
Properties Flooded Above Floor	20	103	831	2,160	3,654	5,599
Properties Flooded Below Floor	21	307	3,106	4,412	4,292	3,120
Total Flooded Properties	**41**	**410**	**3,937**	**6,572**	**7,946**	**8,719**
Total Direct Damages	$0.254 mil	$1.80 mil	$19.6 mil	$51.2 mil	$83.1 mil	$125 mil
Indirect Damages (30% direct)	$0.076 mil	$0.540 mil	$5.87 mil	$15.4 mil	$24.9 mil	$37.7 mil
Potential Damages	$0.33 mil	$2.34 mil	$25.4 mil	$66.6 mil	$108 mil	$163 mil
Actual Damages (DRF at 0.7)	**$0.231 mil**	**$1.64 mil**	**$17.8 mil**	**$46.6 mil**	**$75.6mil**	**$114 mil**
Total Inundated Roads (km)	197.9	232.8	350.9	431.1	488.0	525.0
Total Infrastructure Damages	**$2.02 mil**	**$2.78 mil**	**$5.64 mil**	**$7.73 mil**	**$9.34 mil**	**$10.4 mil**
TOTAL DAMAGES (DRF at 0.7)	**$2.25 mil**	**$4.42 mil**	**$23.4 mil**	**$54.3 mil**	**$85.0 mil**	**$125 mil**

Similarly, **Table 7-2** provides a summary of existing consequences for the total study area for a Broken River/Seven Creeks dominant event.

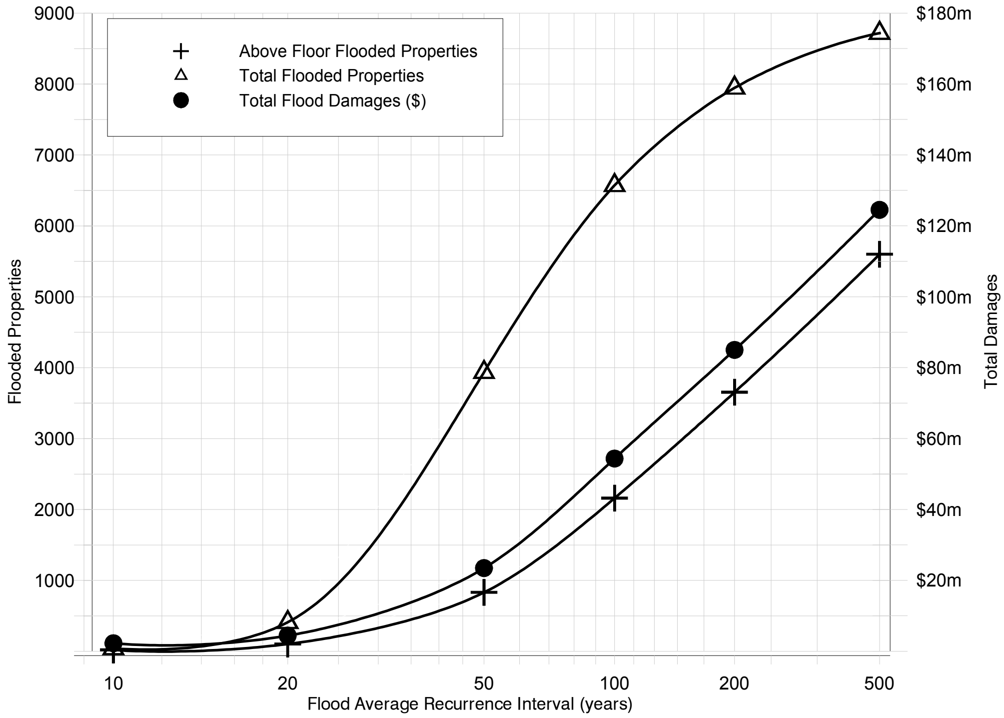
- **Table 7-2 Existing Consequences of Flooding (Broken/Seven Dominant Event)**

Flood Damage Data	ARI (years)					
	10	20	50	100	200	500
Properties Flooded Above Floor	16	77	552	1,971	3,727	5,598
Properties Flooded Below Floor	16	341	2,888	4,483	4,073	3,066
Total Flooded Properties	**32**	**418**	**3,440**	**6,454**	**7,800**	**8,664**

Total damages have not been calculated for a Broken River/Seven Creeks dominant event. A comparison of the data in **Table 7-1** and **Table 7-2** demonstrates that the Goulburn dominant event is a worse case for existing consequences of flooding.

Figure 7-1 shows graphical representation of consequences, including properties flooded above floor, total flooded properties flooded and damages for the entire study area, for a Goulburn River dominant event.

- **Figure 7-1 Existing Consequences of Flooding (Goulburn River Dominant)**



Consequence is also demonstrated on **Figure 7-2**. The upper table in **Figure 7-2** provides a summary of consequences for a range of historical and design events. These are consistent with the summary in **Table 7-1**. The lower table in **Figure 7-2** provides in the first three columns consequences by sub-area within the total study area for a 100 year ARI event. The subdivision by sub-areas provides a means of identifying areas of particular concern and a snapshot of urban and rural consequences. The sub-area boundaries are based on the "inner" and "outer" survey area boundaries (see **Section 3.3**).

- **Figure 7-2 Existing Flood Risk**

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7.3 Existing Flood Risk

Existing flood risk provides an indication of the significance of the existing threat associated with flooding to the community.

Flood risk, as a function of likelihood *and* consequence, can be determined for a given flood or can be integrated over a range of floods, to provide a single indicator of the risk to the community. This indicator is known as Average Annual Damage (AAD) and represents the cost to the community each year due to flooding. Average annual damage is calculated as the area under a curve of total monetary damages versus flood ARI (in **Figure 7-1**). The AAD or flood risk for Shepparton-Mooroopna under existing conditions is estimated at **$1.09 million** (ie. $1.09m/annum) to a 100 year ARI event and **$1.75 million** to a 500 year ARI event.

Flood risk is also demonstrated on **Figure 7-2**. The lower table in **Figure 7-2** provides in the last column flood risk by sub-area within the total study area for a 100 year ARI event. The subdivision by sub-areas again provides a means of identifying areas of particular concern and a snapshot of urban and rural flood risk.

Understanding and quantifying flood risk becomes invaluable in assessing the economic merit of mitigation options. Mitigation options reduce flood risk and AAD. A comparisons of AAD for existing conditions with AAD of a given mitigation option represents the benefit (ie. reduction in AAD) of the option. Comparing the benefit with the cost of implementation of the option provides a benefit-cost ratio, which assists in assessing and ranking of options on economic grounds. Discussion and assessment of mitigation options for Shepparton-Mooroopna is provided in the **Stage 2** report (SKM, 2002a)).

8. Planning Scheme Information

8.1 Background

Planning controls and building regulations provide mechanisms for ensuring appropriate use of land and building construction given the physical constraints of flooding from rivers and streams.

As part of ongoing municipal reform, the State Government recently introduced a consistent planning scheme format for application across the State. The Victoria Planning Provisions (VPPs) have been adopted, to incorporate local requirements, by all Victorian municipalities and will help prevent the escalation of future flood problems.

In Victoria, there are Building Regulations which specify that floor levels should be 300mm above a nominated flood level. The nominated flood level is the level of the 100 year ARI flood, or if that has not been determined for a particular area, it is that level nominated by the floodplain management authority usually on the basis of historical flooding. If land is subject to flooding, the municipal council may set conditions that require particular types of construction or particular types of construction materials.

In this section, the structure of the Victoria Planning Provisions is outlined, the basis of the delineation of land subject to inundation and floodways is explained.

8.2 Victoria Planning Provisions (VPPs)

In respect of floodplain management, the VPPs aim to achieve consistency in the application of planning controls for areas subject to flooding throughout the State. The stated objectives are to protect life, property and community infrastructure from flood hazard, and to preserve flood conveyance capacity, floodplain storage and natural areas of environmental significance.

In the preparation of municipal strategic planning statements or planning schemes, account must be taken of, *inter alia*, any Floodplain Management Plan adopted by the responsible authority.

Under the Victoria Planning Provisions (DoI 2000) there is provision for two overlays and one zone associated with mainstream flooding which are relevant to Shepparton-Mooroopna. These are:

- Land Subject to Inundation Overlay (LSIO),
- Floodway Overlay (FO),
- Urban Floodway Zone (UFZ).

Generally the LSIO identifies land in flood storage or flood fringe areas which are subject to inundation during a 100 year ARI flood, or some other nominated flood if the 100 year ARI flood has not been determined. For Shepparton-Mooroopna, the 100 year ARI flood has been employed to delineate LSIO.

The floodway zone and overlay (UFZ and FO) identify main flood paths and flood storage areas and/or flood prone areas having a high hazard. Such areas are usually associated with significant flood depths and/or velocities, frequent flooding, or are important for conveying significant flood flows or storing significant flood volumes.

In general, certain precautions should be undertaken in the flood affected areas, and development should be regulated by the system of building and planning permits. For example, all floor levels should be elevated at least 300 mm above the nominated flood level according to the Building Regulations and the erection of significant buildings, works and structures should be actively discouraged in floodways by the relevant planning authorities. The local planning authority, in this case Greater Shepparton City Council, may specify exemptions in some instances, having regard for specific conditions appropriate to the locality.

The VPPs proceed to specify for each of the relevant zone or overlays the appropriate types of land uses and developments which are to be regulated through a system of permits. These are intended to achieve consistency throughout the State, but local variations to these guidelines are allowed for through planning permit exemptions that may be declared in a schedule and applied to each of the overlays by the local authority.

8.2.1 Land Subject to Inundation Overlay (LSIO)

This overlay is used to identify land liable to inundation by overland flow in a flood storage or flood fringe area affected by the 100 year ARI flood.

The permit requirements of LSIO are intended:

- to ensure that development maintains the free passage and temporary storage of floodwaters,
- to minimise flood damage,
- to be compatible with the flood hazard and local drainage conditions,
- not to cause any significant rise in flood level or flow velocity,
- to protect water quality in accordance with relevant State Environment Protection Policies (SEPPs).

In general, emergency facilities (hospitals, etc) should be excluded from this area, together with developments or land uses which involve the storage or disposal of environmentally hazardous chemicals or wastes, and other dangerous goods.

Permits are required to construct buildings or carry out works including fencing and works which increase the length or height of embankments or roads. Permits are also required to subdivide land.

These restrictions do not apply to limited categories of buildings or works, such as:

- buildings or works exempted in the schedule declared by the local planning authority;
- works carried out by the floodplain management authority;
- routine repairs or maintenance to existing buildings or works;
- post and wire, and rural type fencing; and
- underground services, and telephone and power lines, provided they do not alter the land surface topography or involve the construction of towers or poles, and provided they are undertaken in accordance with approved plans.

8.2.2 Floodway Overlay (FO)

The purpose of this overlay is to identify waterways, main flood paths, drainage depressions and high hazard areas within rural and urban areas which have the greatest risk and frequency of flooding.

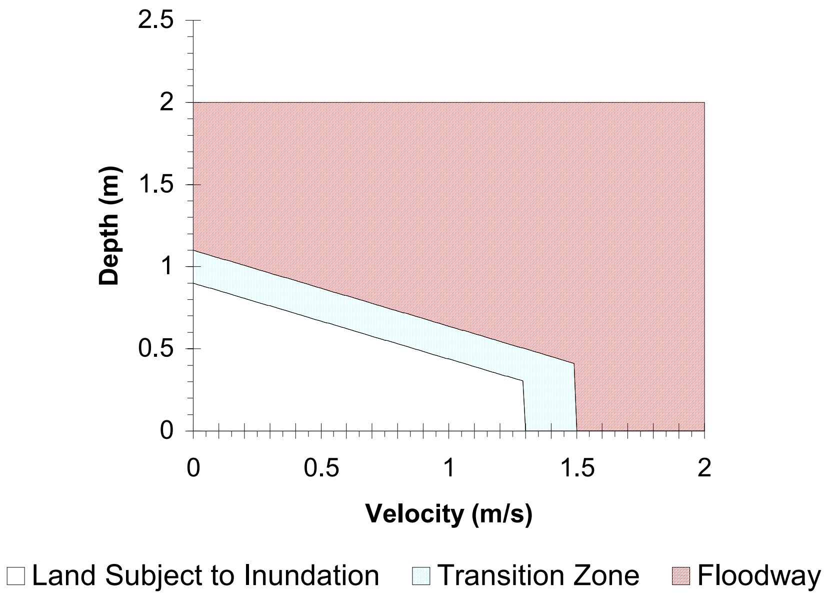
The identification of floodways was based on NRE's "Advisory Notes for Delineating Floodways." (NRE 1998). These give considerable scope in floodway delineation. The process adopted for this study focused on flood depth, flow velocity (which combined represent flood hazard) and flood frequency.

When using velocity-depth considerations to define floodway, the advisory notes suggest the use of a graph, reproduced in **Figure 8-1**.

When considering flood frequency, Appendix A1 of the advisory notes suggest areas which flood frequently and for which the consequences of flooding are moderate or high, should generally be regarded as floodway. In the case of Shepparton-Mooroopna, a threshold flood frequency of 10 years is appropriate.

It is possible that the use of these criteria alone can omit areas of quite deep flooding which might best be classed as floodway. Therefore, consideration should also be given to classifying as floodway selected areas where depth is greater than 0.5m. Furthermore, flow paths with clear links that allows flooding to transfer from one area to another should be selected as important floodway areas.

- **Figure 8-1 Assessment of Floodway Based on Depth and Velocity (Source: NRE (1998))**



8.2.3 Urban Floodway Zone (UFZ)

This zone is used to identify waterways, main flood paths, drainage depressions, and high hazard areas within urban areas that have the greatest risk and frequency of flooding. Unlike the flood overlays, which provide for additional controls over and above the underlying land use, this zone places restrictions on the use of the land.

Within this zone, there is a specified table of uses, as follows. Section 1 (Permit not required) includes apiculture, extensive animal husbandry, natural systems, informal outdoor recreation, mineral exploration, or (subject to conditions) mining or stone quarrying. Section 2 (Permit required), including agriculture, leisure and recreation, and roads. Section 3 (Prohibited) includes indoor recreation facilities, motor racing tracks, and other use not in Sections 1 and 2.

Permits are required to construct buildings or carry out works including fencing and roadworks, except for limited categories of buildings or works. These are identical to those stipulated in the LSIO clauses in the VPPs, except that there are no schedule exclusions other than for advertising signs.

UFZ also has very strict controls on subdivisions. Unless a local floodplain development plan specifically provides otherwise, land may only be subdivided to:

- realign lot boundaries,
- excise land to be transferred to the floodplain management authority for public purposes.

8.2.4 Decision Guidelines

The VPPs also stipulate numerous decision guidelines that must be considered by the responsible authority (the Greater Shepparton Greater Shepparton) when deciding on applications for permits. For UFZ and FO, unless the responsible authority has adopted a local floodplain development plan, the applicant is required to prepare a flood risk report, which *inter alia*, helps identify the flood impacts at the site and for adjoining areas. The flood risk report (or the local floodplain development plan where applicable) is incorporated into the decision guidelines.

For LSIO a flood risk report is not required. However, the responsible authority is required to assess each application having regard to the same considerations required for the flood risk reports for the floodway zone and overlay, and the application has to be consistent with any local floodplain development plan approved for the area.

While the list of matters to consider in the decision guidelines are similar for the flood zone and overlays, it is the clear intention of the VPPs that planning applications in floodway areas (and especially UFZ areas) are subject to more stringent controls, conditions and/or restrictions than those in LSIO.

8.2.5 Referrals

Applications must be referred to the relevant floodplain management authority, ie the Goulburn Broken Catchment Management Authority for independent assessment.

8.2.6 Local Floodplain Development Plans

The primary purpose of a local floodplain development plan is to simplify and streamline the consideration of planning permit applications for particular areas. This is achieved by providing specific development requirements , having regard for the floodplain management issues in the area of interest. In doing so, Local Floodplain Development Plans reduce the requirements on the applicant for supporting information and provides all stakeholders including GSCC referral authority and developers with a set of performance-base criteria for decision making.

8.3 Application of Planning Controls to Shepparton-Mooroopna

Flooding delineation option maps have been produced to assist GSCC and GBCMA in the definition of new land use flood zones and overlays, and the designation of flood levels. These maps have been prepared for existing conditions for Shepparton-Mooroopna. From these option maps, GSCC and GBCMA have developed the planning maps in accordance with the Victorian Planning Provisions Practice Notes – Applying the Flood Provisions in Planning Schemes (DoI 2000). The flooding delineation maps are bound in a separate volume.

The designated flood levels were derived by enveloping the peak flood levels from the two design flood combinations listed in **Table 4-22** that result in a 100 year ARI flood at the Shepparton gauge ie. 100 year ARI Goulburn River with 50 year ARI Broken River and Seven Creeks, and 100 year ARI Broken River and Seven Creeks with 50 year ARI Goulburn River.

The GBCMA as the floodplain management authority has advertised its intention to declare the flood levels in accordance with the requirements of the Water Act, 1989.

8.4 Guidelines for Buildings on Floodplains

Building development on floodplains must be managed in a controlled and coordinated way not only to maintain the natural flow patterns and the environmental values of floodplains but also to minimise the risk to life, health and safety of occupants. Some key reasons why it is important to plan and control buildings are:

- the cumulative effects of unplanned and uncoordinated development can have long term detrimental effects, including changes to flooding patterns, particularly if waterways are obstructed or modified;
- the construction of dwellings below the design flood level may result in significant flood damages and associated personal trauma;
- inappropriately located buildings or dwellings can place occupants in life threatening situations, as floods aren't always preceded by advance warnings; and
- through its planning scheme, the Greater Shepparton City Council is well placed to ensure that building construction is carried out in accordance with local and regional flooding conditions, and established planning guidelines.

These guidelines are primarily intended to help the floodplain management authority, architects and builders when considering or preparing building applications for planning approval. Further guidelines can be found in the VPP Planning Practice Notes (DoI, 2000). Note that this document is currently in draft form and permission for its use from the Department of Infrastructure will be required.
Buildings that are subject to these guidelines are:

- habitable dwellings and extensions;
- commercial and industrial buildings and extensions;
- sheds; and
- pergolas, carports and garages.

8.4.1 Planning Permit Exemptions

Not all buildings require a planning permit. The VPPs allow for exemptions from planning permits to apply to certain types of buildings and works, which are included as schedules to the land subject to inundation and floodway overlays. It is suggested that the following types of buildings be exempted from planning permits:

- non-habitable buildings or extensions to non-habitable buildings with total floor areas less than 100 m^2;
- an extension to a habitable dwelling including contiguous garages, provided that the floor area of the extension is less than 20 m^2 for areas subject to FO and less than 40 m^2 for areas subject to LSIO, and is less than 50% of the existing floor area;
- a pergola, carport, hay shed or in-ground swimming pool associated with an existing dwelling.

These recommendations are preliminary and should be finalised by the CoGS in consultation with the GBCMA. In particular the two authorities need to be satisfied that in allowing these types of development to proceed, important flowpaths are not obstructed. The justification for these exemptions is that these works are likely to have a low or negligible increase in flood risk from the perspective of the threat to life, health and safety, and will generally have a low impact on flood velocities, depths and flow distribution.

8.4.2 Other Buildings

All other buildings not specified above require a planning permit from the Greater Shepparton City Council

Residential Buildings

All new residential dwellings and extensions must be built at least 300 mm above the 100-year ARI flood level. For those extensions that are permitted to be built at the existing floor level, the building permit should require water resistant materials are to be used up to 300 mm above the 100-year ARI flood level or to a level specified by the floodplain management authority.

Free standing garages should be assessed by the same criteria, except that flood level requirements can be relaxed so that garage floor levels are at the 100 year ARI flood level.

Vacant allotments do not necessarily have an existing entitlement to build. There may be areas where subdivisions have proceeded without significant regard to the flood risk and where it is desirable for building densities to be kept at a low level.

Generally the floodplain management authority will be required to provide flood level advice, although for some areas GSCC can perform this role if in accordance with a written agreement between the GSCC and GBCMA or by Local Floodplain Development Plans.

In deciding the appropriateness of building proposals, GSCC and GBCMA should consider:

- imposing minimum and average house density requirements;

- whether a proposed dwelling should be sited on higher ground to minimise flood obstructions and facilitate evacuation;
- in the case of extensions, whether it is *bone fide* for the purposes of accommodating the existing family;
- whether the location of a proposed dwelling is appropriate, having regard for flood risk factors such as the frequency, duration, extent, depth and velocity of flooding at the site and accessway; the flood warning time available; and the danger to the occupants of the development, other floodplain residents and emergency personnel if the site or accessway is flooded;
- whether the house foundations are compatible with the flood risk (slab-on ground floors take longer to dry out than floors elevated to above ground level);
- limiting further building extensions ;
- requiring amalgamation of smaller lots as a condition of the planning permit; and
- whether the owner should enter a Section 173 Agreement stating the floor level and 100-year ARI flood level, for the benefit of future purchasers.

Commercial and Industrial Buildings

To allow buildings to be constructed in land liable to flooding, municipal councils must comply with Regulation 6.2 (4) of the Building Regulations, 1996. This requires the relevant municipal council to refuse consent unless there is no significant danger to the life, health or safety of the occupants of the building due to flooding of the site.

Wherever possible buildings should be located on flood free land. However there are instances where this is not possible, and suitable construction sites may exist only on land liable to inundation.

In such cases, commercial and industrial buildings should be located as far as practicable on land where the risk of property damage and harm to occupants is low, and where building development has minimal impacts on adjoining properties. The construction of buildings on the high hazard floodway land should be avoided wherever possible.

Generally, the minimum floor level of industrial or commercial buildings shall be at least 300 mm above the 100-year ARI flood level, defined as the “nominal protection level” (NPL) in the Building Regulations (1994), unless the floodplain management authority consents to a lower level. The floodplain management authority will determine this, unless agreement has been reached with the relevant municipality for it to provide this advice on the floodplain management authority’s behalf.

There are however, some instances where relaxing this floor level requirement is warranted. For example, floor levels at or above the NPL may be unacceptably high in relation to surrounding development such as street footpaths, adjoining properties, or create access problems for disabled people or people with prams. These must be assessed on a case by case basis, in conjunction with the floodplain management authority, and having full regard for the flood risk. Conditions that must be met before approval to construct a building below the NPL are listed below.

Where a relaxation in floor levels is warranted, the authority will normally require floor levels to be set at least to the 100 year ARI flood level. However, as indicated above, there will be special cases where further relaxation is desirable.

In considering whether or not to approve a planning permit application for an industrial or commercial building, GSCC and GBCMA need to be mindful of any likely changes to the proposed use or likely change in ownership of the proposed building. Consideration also needs to be given to the potential flood impacts. The applicant should be required to produce a commercial flood risk report, in which the economic and social risks and consequences of flooding are identified and evaluated.

No relaxation of floor levels will be permitted unless the following conditions are met:

- water resistant building materials are used up to at least the NPL;
- flood proofing measures are incorporated into the design and details are incorporated on the plans accompanying the building application, if required by the floodplain management authority (see below);
- all electrical fittings are fixed above the nominal protection level and to the requirement of the relevant power authority;
- sewer fixtures are located to the requirements of the relevant responsible authority with all inlets above the flood level;
- adequate drainage is provided for;
- on issue of a building permit, the owners undertake to notify occupants of the flood risk, and of the need for contingency planning when a flood occurs.

Where required by the floodplain management authority, flood proofing measures will need to be incorporated into the design of the building. These require:

- provision for sealing of openings, ideally by provisions for gates or bulkheads with effective seals, but sandbagging may be acceptable in some instances ;
- window sills to be located above the NPL; and
- a written undertaking by the owner to maintain all flood proofing measures incorporated into the building or required to protect the building from flooding.

In deciding if flood proofing is required the floodplain management authority shall have regard for the intended use of the building and the potential flood damages (stock, fittings, structural damages, etc). In many cases, particularly industrial buildings with metal cladding, effective flood proofing is impractical if not impossible.

Referral to the Floodplain Management Authority

The floodplain management authority is a referral authority under Section 55 of the Planning and Environment Act, 1987, unless there is a referral exemption specified in the schedule to the relevant flood overlay or where GSCC has made a written agreement with the floodplain management authority. Referral to the floodplain management authority is also appropriate where:

- the proposed works are within a proclaimed water supply catchment;
- the proposed works are within 100 metres of a waterway;
- the proposed works may impact on drainage behaviour on adjoining properties; and
- the proposed works may impact on water quality in the catchment.

Notification

If GSCC determines that the proposed works have the potential to create an unreasonable flow or interfere with the reasonable flow of water or may result in a material detriment to any party, notification may be required pursuant to Section 52 of the Planning and Environment Act.

GSCC may also consult with VicRoads, any relevant Rural Water Authority and/or the Department of Natural Resources and Environment.

9. Flood Inundation Mapping for Emergency Response

9.1 Overview

A number of flood maps have been produced for existing conditions, primarily for the purposes of flood emergency planning and response.

Flood inundation maps have been produced for the design flood combinations outlined in **Section 4.7**. Two design event scenarios, Goulburn River dominant and Broken River/Seven Creeks dominant, were considered. These design flood combinations result in peak gauge heights for the Goulburn River at Shepparton corresponding to the 10, 20, 35, 50, 100, 200 and 500 year ARI design events.

For each design event combination two map sheets have been produced, an inner area map showing the urban area of Shepparton-Mooroopna and an outer area showing the entire study area. Each map sheet includes flood extents, shaded flood depth zones and flood contours. The location of existing buildings is also shown and this information is colour coded to identify whether flooding occurs above or below floor level. Key features and buildings are also highlighted. For clarity, some features within the inner map area have been omitted from the outer area map.

Complementing each inner area flood inundation map, separate correlation tabulations have been produced. Each tabulation documents the design event combination and ARI and gauge height at Shepparton for the corresponding map. Combined with each of these correlation tabulations are detailed listings of flood affected buildings (including address, floor level and flood depth for each).

For the 100 year ARI event, velocity maps have also been produced for the two design scenarios and two map areas. These show shaded flow speed zones and flow direction arrows.

Two composite maps have been developed from the seven mapped events for each of the Goulburn River and Broken River/Seven Creeks dominant design event combinations. This map shows the progression of the flood extent with increasing gauge height. It provides, on the one map, flood breakout patterns and associated affected areas.

Inner area maps have been produced on single B1 sheets at 1:10,000. Total area map has been produced on a single B1 sheet at 1: 25,000.

The maps and tabulations will provide an invaluable tool in emergency planning and response in particular.

Flood inundation maps are provided in separate volumes. Tabulations are also provided in a separate volume. Copies of the maps and listings are also provided on the Project CD-ROM.

9.2 Mapped Flood Events

For the design flood combination inner map area, the study brief required that inundation maps be produced from the threshold of overbank flow to the 500 year ARI at 200 mm gauge increments. The design events were used directly for flood inundation mapping as they were broadly representative of the progress of flood depth in Shepparton at around 200 mm increments. However, the design events alone left a 400 mm gap between 11.6m (ie. a 20 year ARI event) and 12.0m (ie. a 50 year ARI flood event) on the Shepparton gauge. Therefore an additional flood event was derived to produce a map for 11.8m at the Shepparton gauge. This gauge height equates to a 35 year ARI flood event.

The data for the flood maps were derived from flood surfaces extracted from the hydraulic models.

Table 9-1 summarises the mapped events including gauge height and design ARI for both Goulburn River dominant and Broken River/Seven Creeks dominant floods (shown in upper and lower lines respectively against each Shepparton gauge height).

- **Table 9-1 Summary of Flood Maps**

Gauge height at Shepparton gauge (m)	ARI at Shepparton Gauge (years)	ARI of Floods in Contributing Streams* (years)		
		Goulburn River (at study boundary)	Broken River (at study boundary)	Seven Creeks (at study boundary)
11.3	10	**10**	5	5
		5	**10**	**10**
11.6	20	**20**	10	10
		10	**20**	**20**
11.8	35	**35**	17	17
		17	**35**	**35**
12.0	50	**50**	20	20
		20	**50**	**50**
12.2	100	**100**	50	50
		50	**100**	**100**
12.3	200	**200**	100	100
		100	**200**	**200**
12.5	500	**500**	200	200
		200	**500**	**500**

*Dominant contributing stream(s) for each map shown in bold.

9.3 Flood Inundation Mapping

9.3.1 Flood Inundation Data

The flood inundation maps were developed to show both flood elevation (in the form of flood surface contours) and flood depth (in the form of shaded depth zones). Flood surface contours were produced readily by contouring the flood surface data. A contour interval of 0.2 m was adopted.

Shaded depth zones were derived by subtracting each ground surface grid point (ie. the DTM) from each flood surface grid point to determine a grid of flood depth throughout the study area. Each flood depth grid point was then colour-coded to produce the following flood depth zones on the maps:

- Less than 0.25 m
- 0.25 m to 0.5 m
- 0.5 m to 1.0 m
- Greater than 1.0 m

9.3.2 Flood Affected Properties

On the flood inundation maps, small markers within property boundaries identify each building. Property data was derived from the property database developed for the flood damage assessment. In most cases they represent the approximate building location.

Buildings affected by above floor flooding for each flood event are coloured red. Buildings affected by below floor flooding can be inferred from the flood extent.

9.3.3 Map Base

The main feature of the map base is a cadastre obtained from GSCC. The cadastre is discussed in **Section 3.3**. Other important landmarks and locations are also labelled. Both inner and outer area map bases also highlight, using large colour symbols, locations of significance for emergency planning and response, namely the:

- Ambulance
- Fire Station
- Police
- Shire Offices
- Hospital

9.3.4 Gauge Correlations

Each flood inundation map represents a specific gauge at Shepparton and is derived from given inflows from the three contributing streams, summarised by ARI in **Table 9-1**. For emergency management purposes, knowledge of the magnitude of approaching floods in the contributing streams will assist emergency response in Shepparton-Mooroopna. However, ARIs in **Table 9-1** serves no practical use. Therefore, for effective flood response, gauge heights at Shepparton gauge for each event must be linked to gauge heights at upstream gauges for contributing events. **Table 9-2** provides this link to gauges at Murchison, Orrvale and Kialla West for both Goulburn River dominant and Broken River/Seven Creeks dominant floods (shown in upper and lower lines respectively against each Shepparton gauge height). These gauge heights are shown on each flood inundation map.

- **Table 9-2 Correlations of Upstream Gauges to the Shepparton Gauge**

Gauge height at Shepparton gauge (m)	ARI at Shepparton Gauge (years)	Gauge Height in Contributing Streams* (m)		
		Goulburn River at Murchison Gauge	Broken River at Orrvale Gauge	Seven Creeks at Kialla West Gauge
11.3	10	**10.4**	7.5	6.2
		9.6	**7.9**	**6.6**
11.6	20	**10.9**	7.9	6.6
		10.4	**8.1**	**7.2**
11.8	35	**11.1**	8.0	6.9
		10.8	**8.2**	**7.5**
12.0	50	**11.3**	8.1	7.2
		10.9	**8.3**	**7.6**
12.2	100	**11.6**	8.3	7.6
		11.3	**8.4**	**7.9**
12.3	200	**11.8**	8.4	7.9
		11.6	**8.5**	**8.0**
12.5	500	**12.2**	8.5	8.0
		11.8	**8.6**	**8.2**

*Dominant contributing stream(s) for each map shown in bold.

Refer to **Appendix E** for relevant rating tables to determine corresponding flow rates for gauge heights listed in **Table 9-2**.

9.4 Velocity Maps

The velocity maps were developed for the 100 year ARI flood event to show both maximum flow speed (in the form of shaded flow speed zones) and indicative flow direction (in the form of arrows).

Shaded flow speed zones were derived by extracting flow speed at each model grid point from the model. Each flow speed grid point was then colour-coded to produce the following flow speed zones on the maps:

- Less than 0.25 m/s
- 0.25 m/s to 0.5 m/s
- 0.5 m/s to 0.75 m/s
- 0.75 m/s to 1.0 m/s
- 1.0 m/s to 1.5 m/s
- Greater than 1.5 m

The arrows show indicative directions of flow and their size represent maximum flow speed to the nearest 0.25m/s. Arrow sizes will be consistent with the underlying flow speed zones. For clarity arrows have been produced at every 4th and 10th grid square for the inner (1:10,000) and outer (1:25 000) map sheets respectively.

9.5 Incremental Inundation Maps

The study brief required the correlation tabulations to include “critical locations where important flood flow breakouts begin to occur” as well as “critical access points where access might be cut off and an indication of which areas would be affected” for each inundation map.

The documentation of this information can be achieved more effectively on a single plan by layering the seven flood surfaces from each flood inundation map. The layering proceeded from the largest event to the smallest event (each overlain by the next) with each flood surface shaded a different colour. The resulting map identifies incremental flood extent for the range of design events mapped.

For any point or area on the map, its colour defines the gauge height and hence ARI at which the point or area is inundated. This immediately identifies “critical access points” and “which areas would be affected” for any given gauge height. Flood flow breakout characteristics at any point or into any area can also readily be inferred from local shading patterns.

The Goulburn River dominant and Broken River/Seven Creeks dominant events have been treated on separate maps.

9.6 Correlation Tabulations

For each flood inundation map produced, correlation tabulations and listings have been compiled from the flood surface data and the property database.

The correlations provide peak flow, ARI and gauge height at Shepparton (and Murchison, Orrvale and Kialla West) for each flood inundation map (see **Table 9-2**). The detailed listings provide a number of property related data. They include, for each event:

- number of above floor affected properties
- number of below floor affected properties
- total number of flood affected properties

Then, within each listing, for each property:

- street address, type (ie. commercial, public or residential), description (primarily for commercial and public buildings)
- ground level, floor level, flood elevation, flood depth above ground, flood depth above floor.

The listings are sub-divided into above floor and below floor listings.

Again, separate listings have been prepared for Goulburn River dominant and Broken River/Seven Creeks dominant events.

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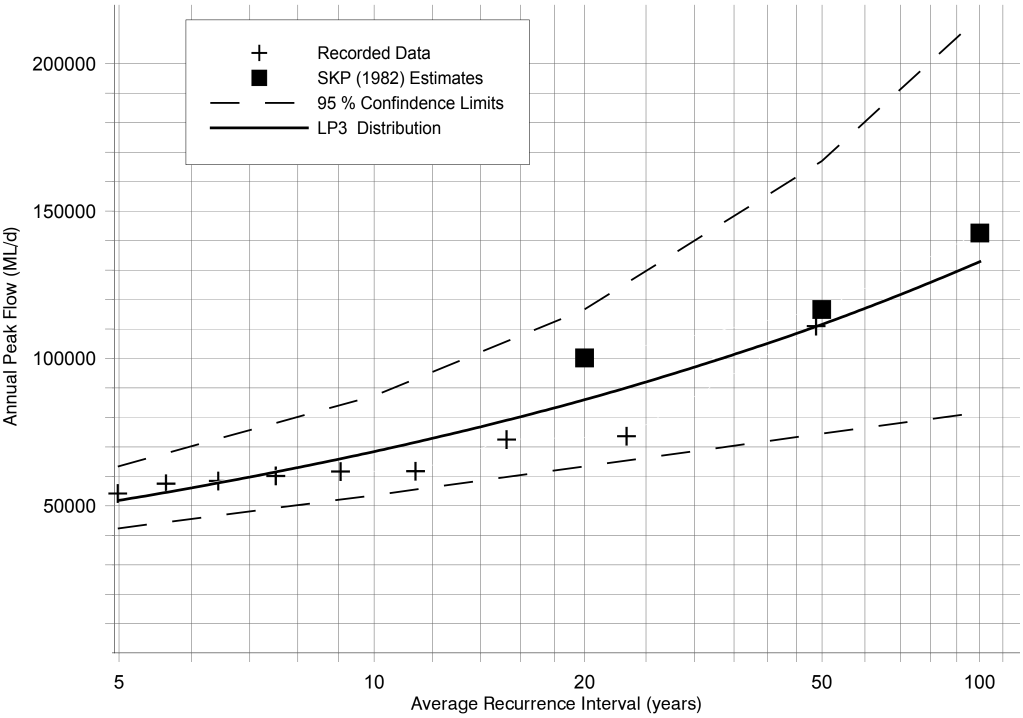
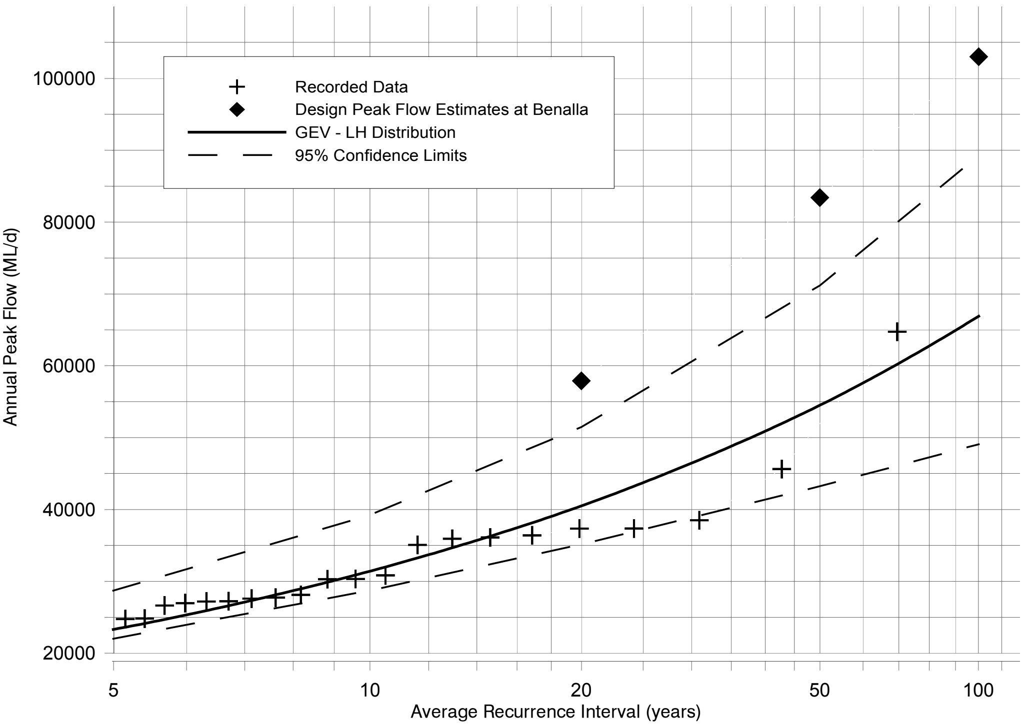
Appendix A Stage 1 Community Consultation

Appendix B Flood Frequency Curves

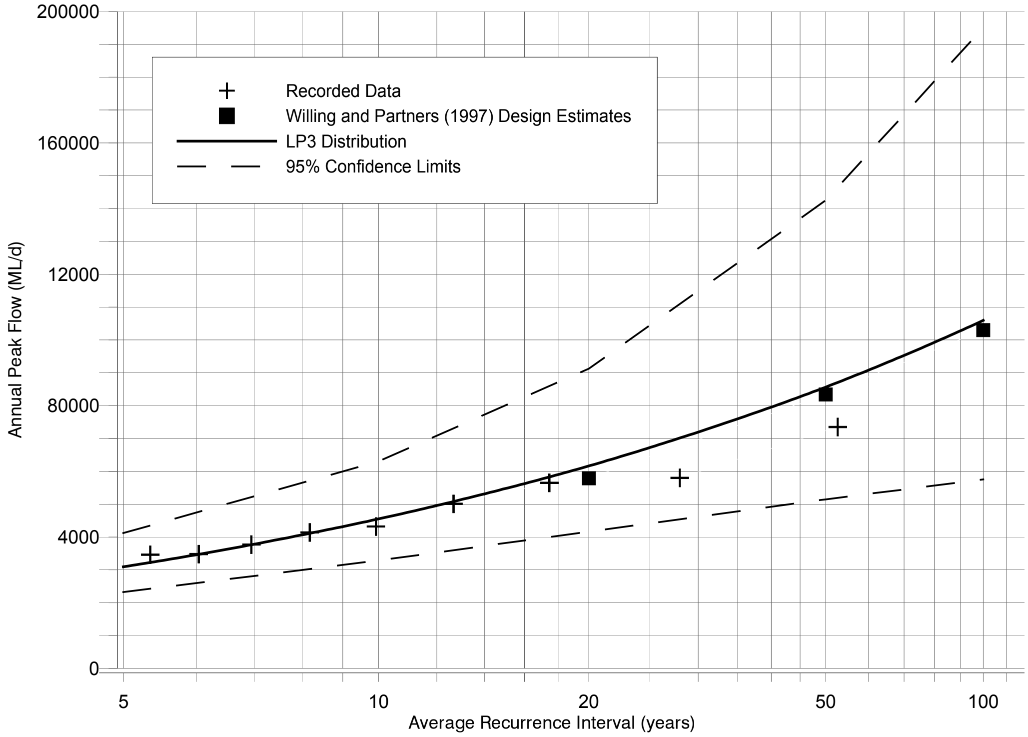
B.1 Goulburn River at Murchison (from Figure 4-2)



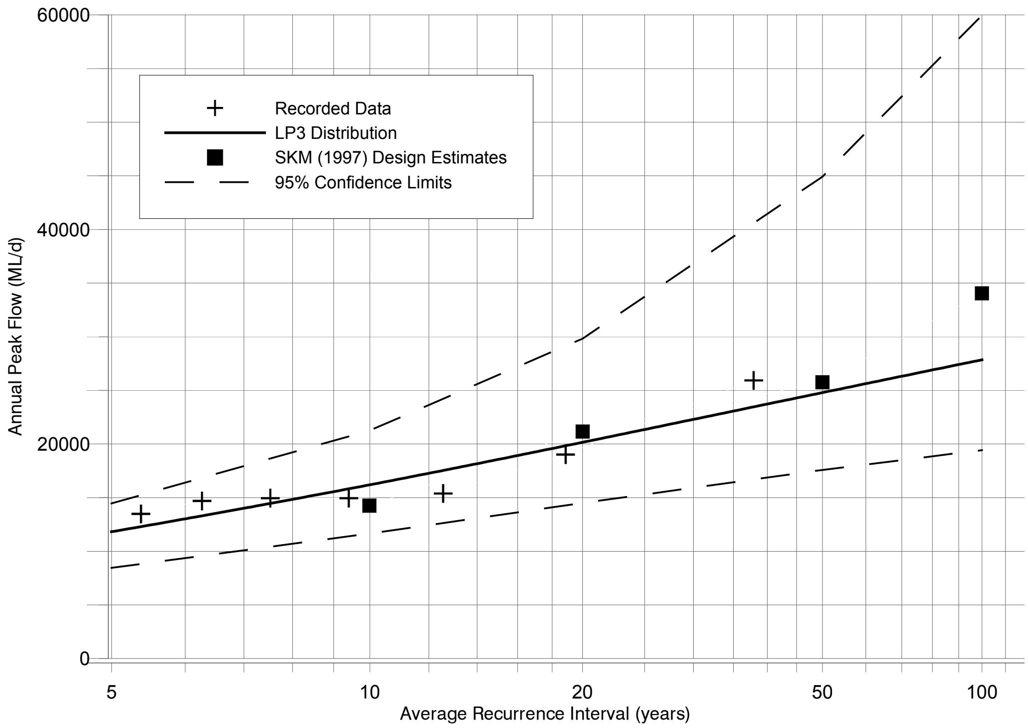
B.2 Broken River at Casey's Weir (from Figure 4-3)



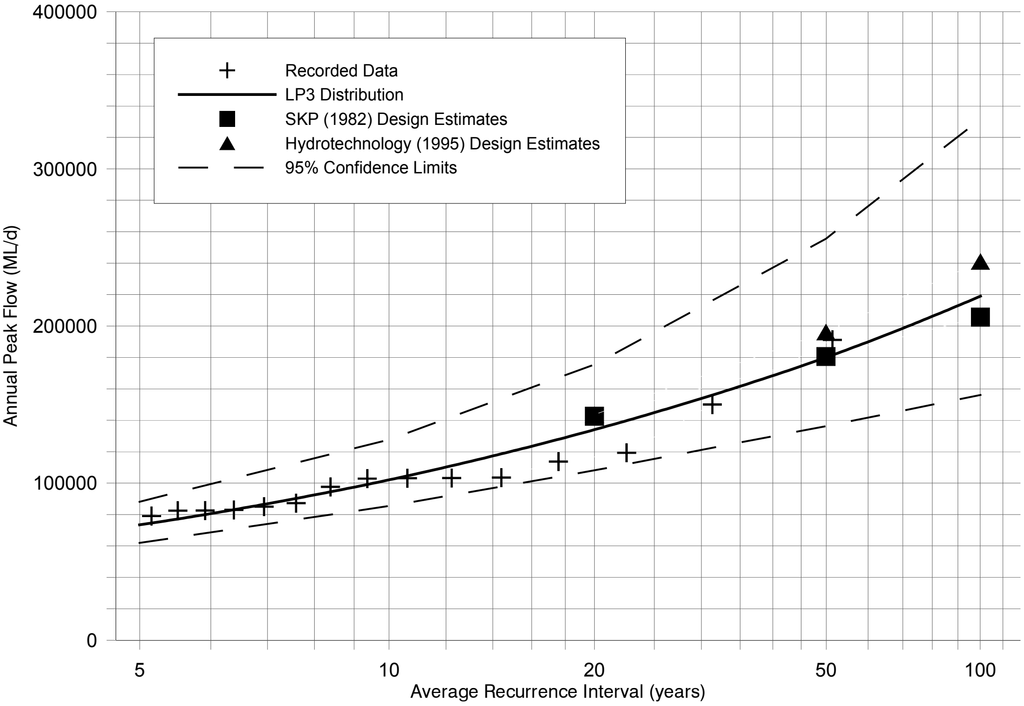
B.3 Broken River at Benalla (from Figure 4-4)



B.4 Seven Creeks at Euroa (from Figure 4-5)



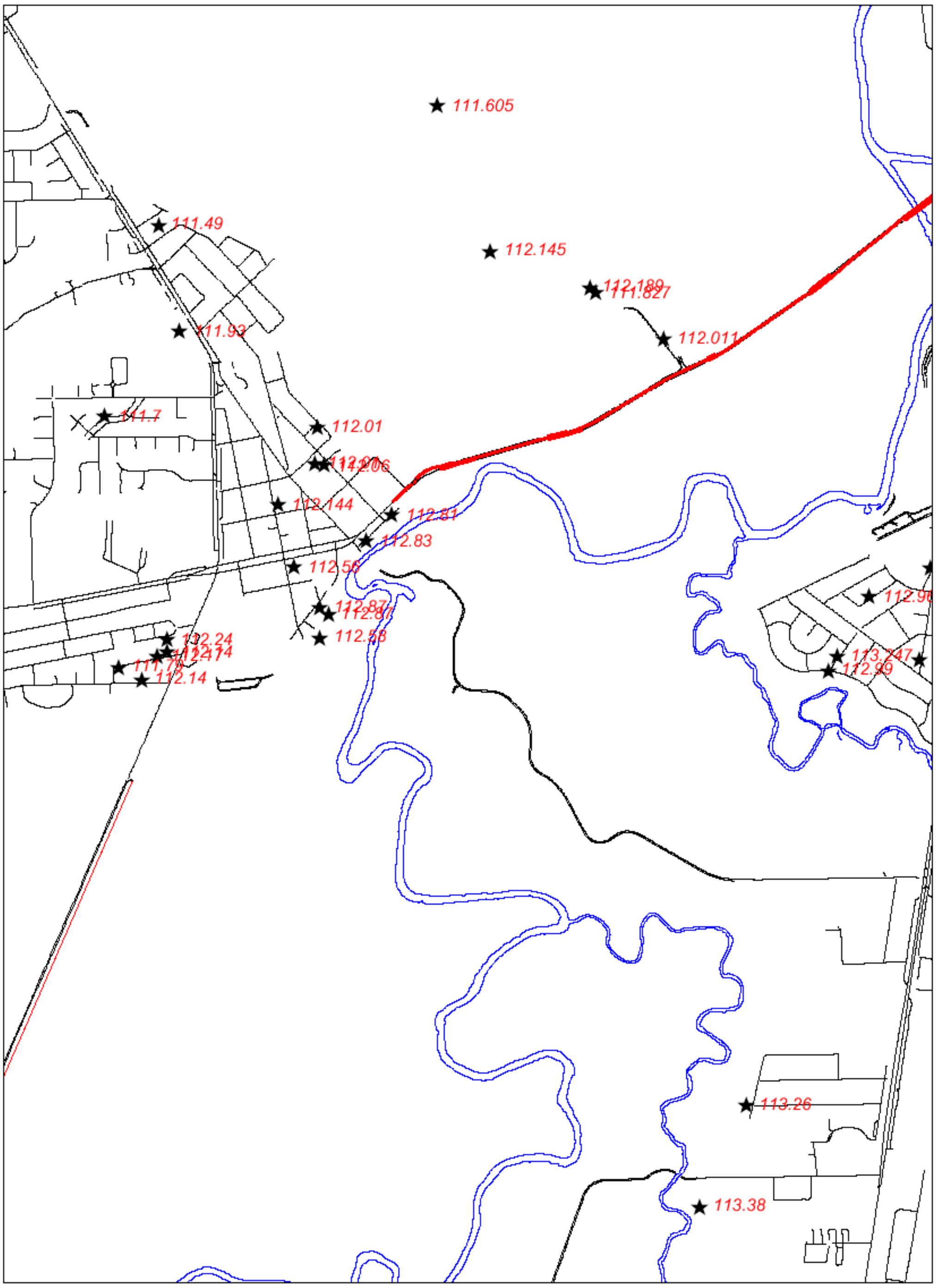
B.5 Goulburn River at Shepparton (from Figure 4-6)



Appendix C Observed Maximum Flood Levels

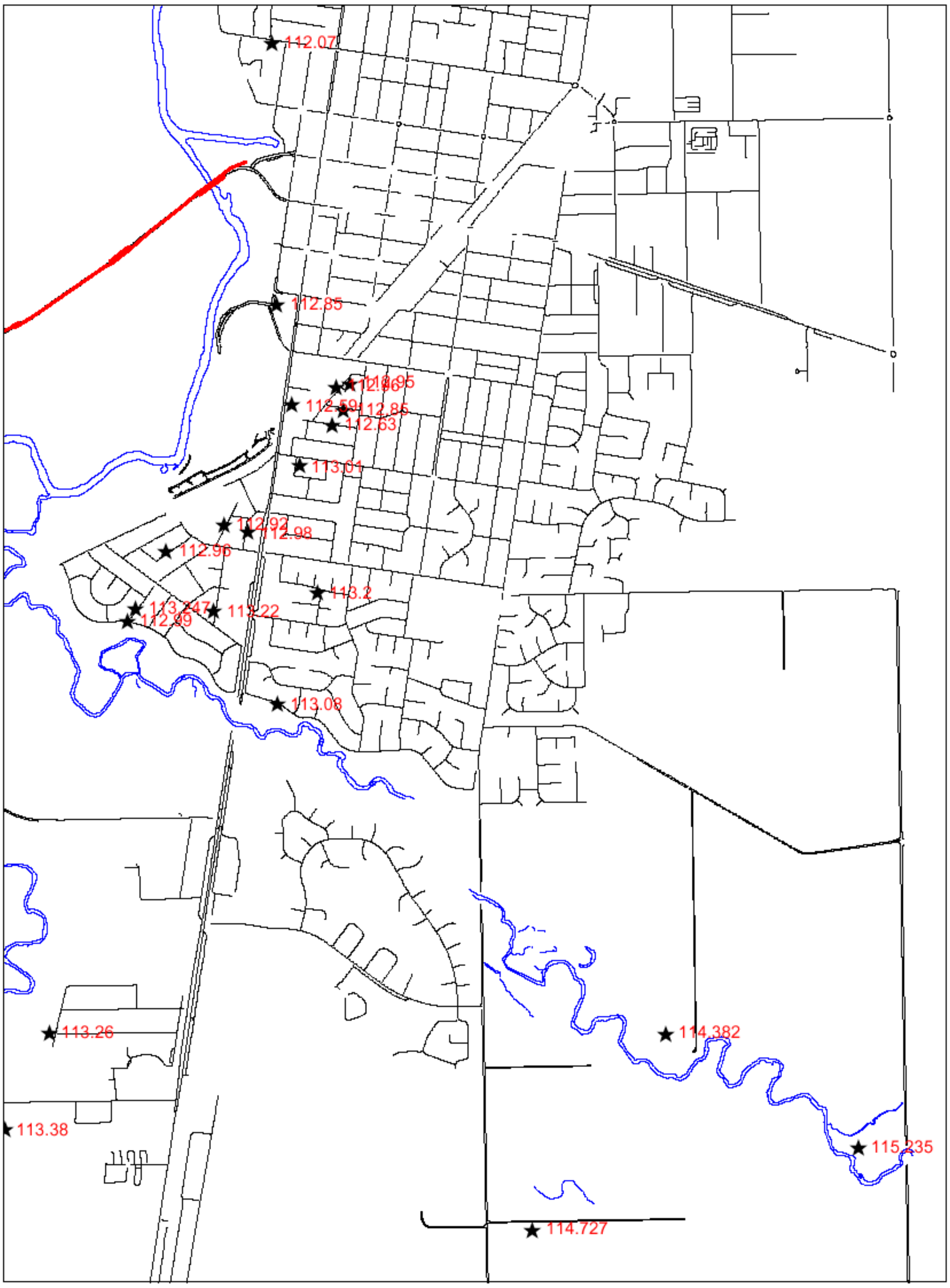
C.1 May 1974 Observed Flood Levels

C.1.1 Mooroopna



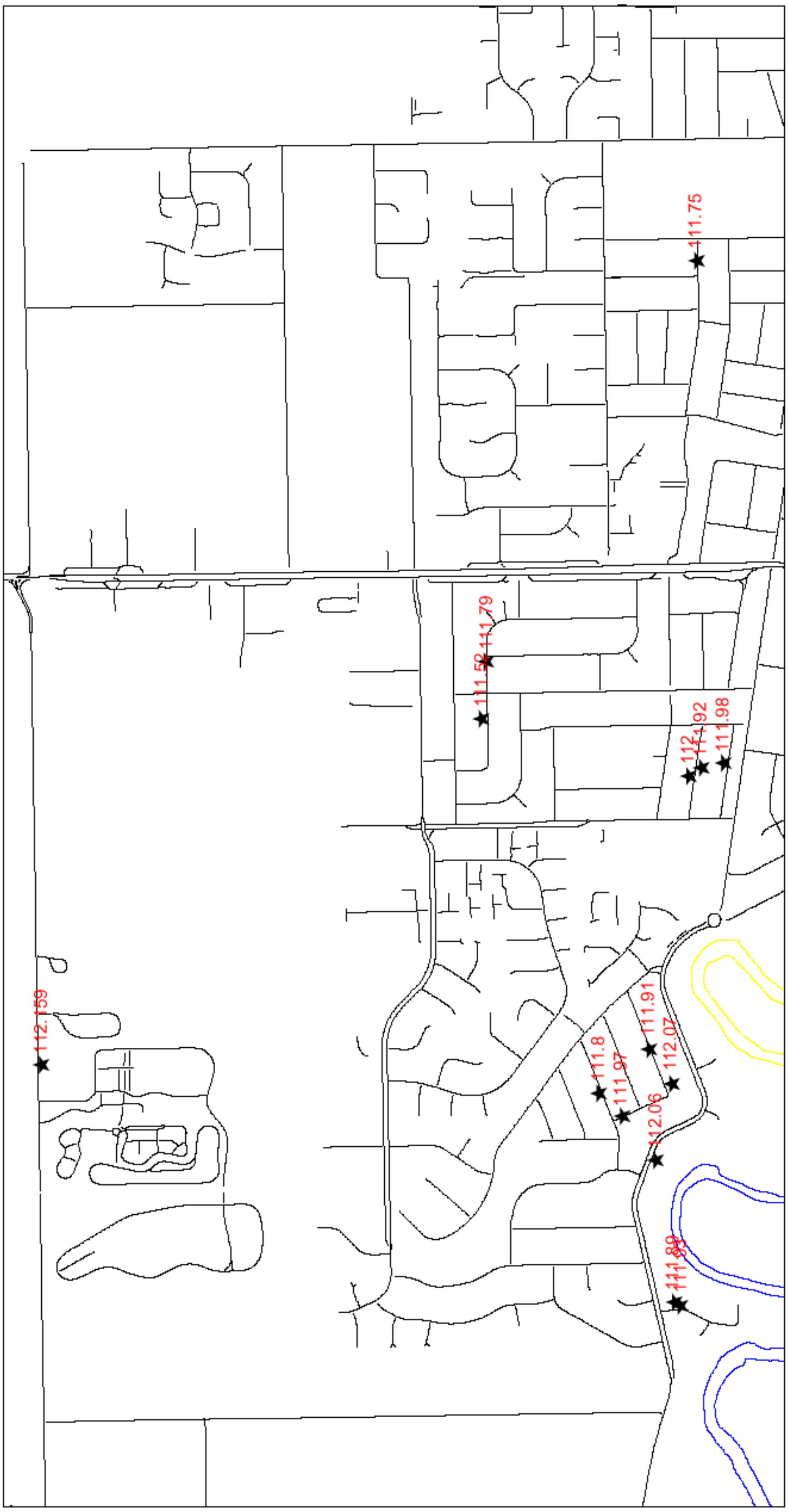
1974 Event Observed Maximum Flood Levels (m AHD)

C.1.2 Shepparton



1974 Event Observed Maximum Flood Levels (m AHD)

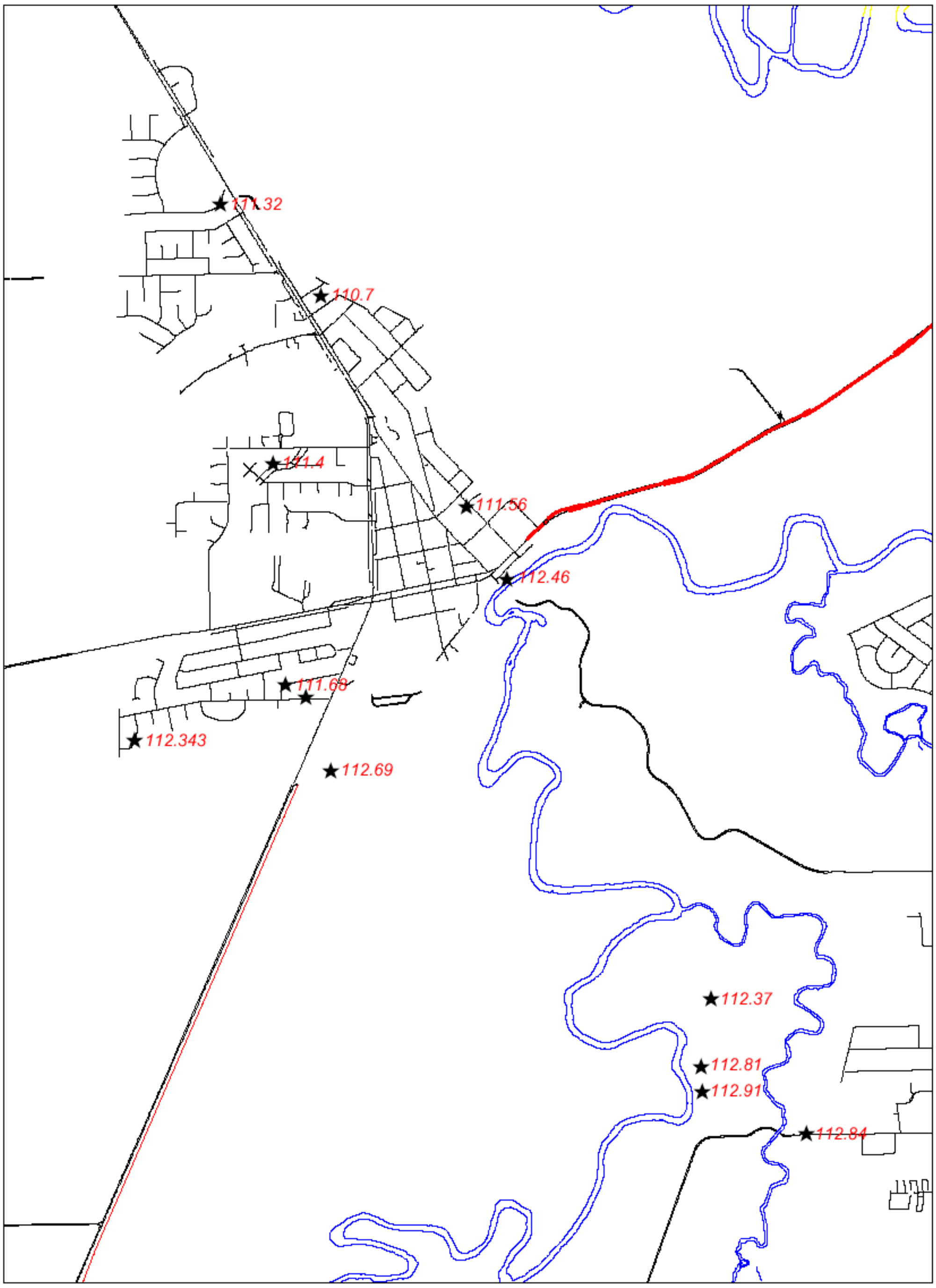
C.1.3 North Shepparton



1974 Event Observed Maximum Flood Levels (m AHD)

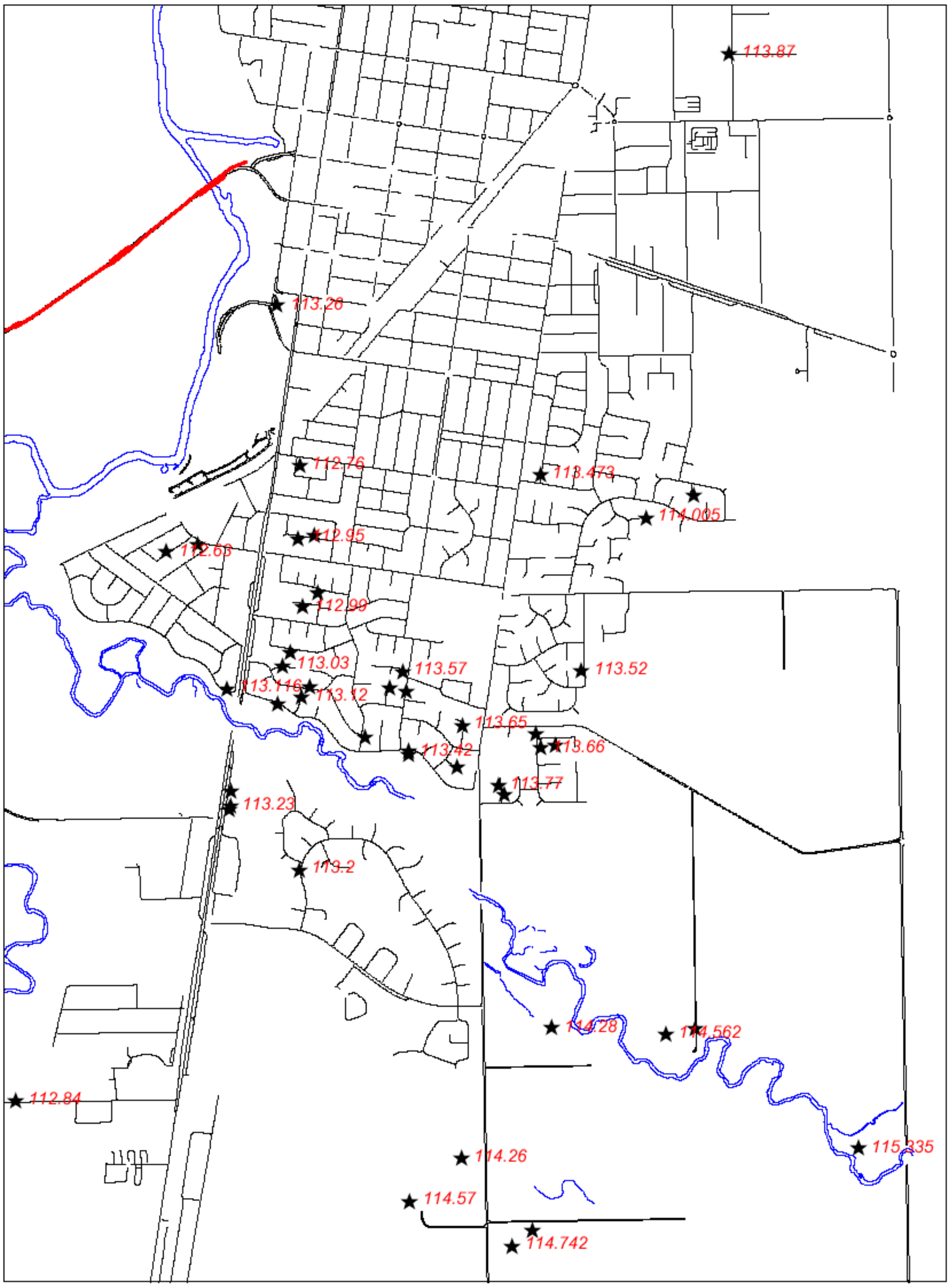
C.2 October 1993 Observed Flood Levels

C.2.1 Mooroopna



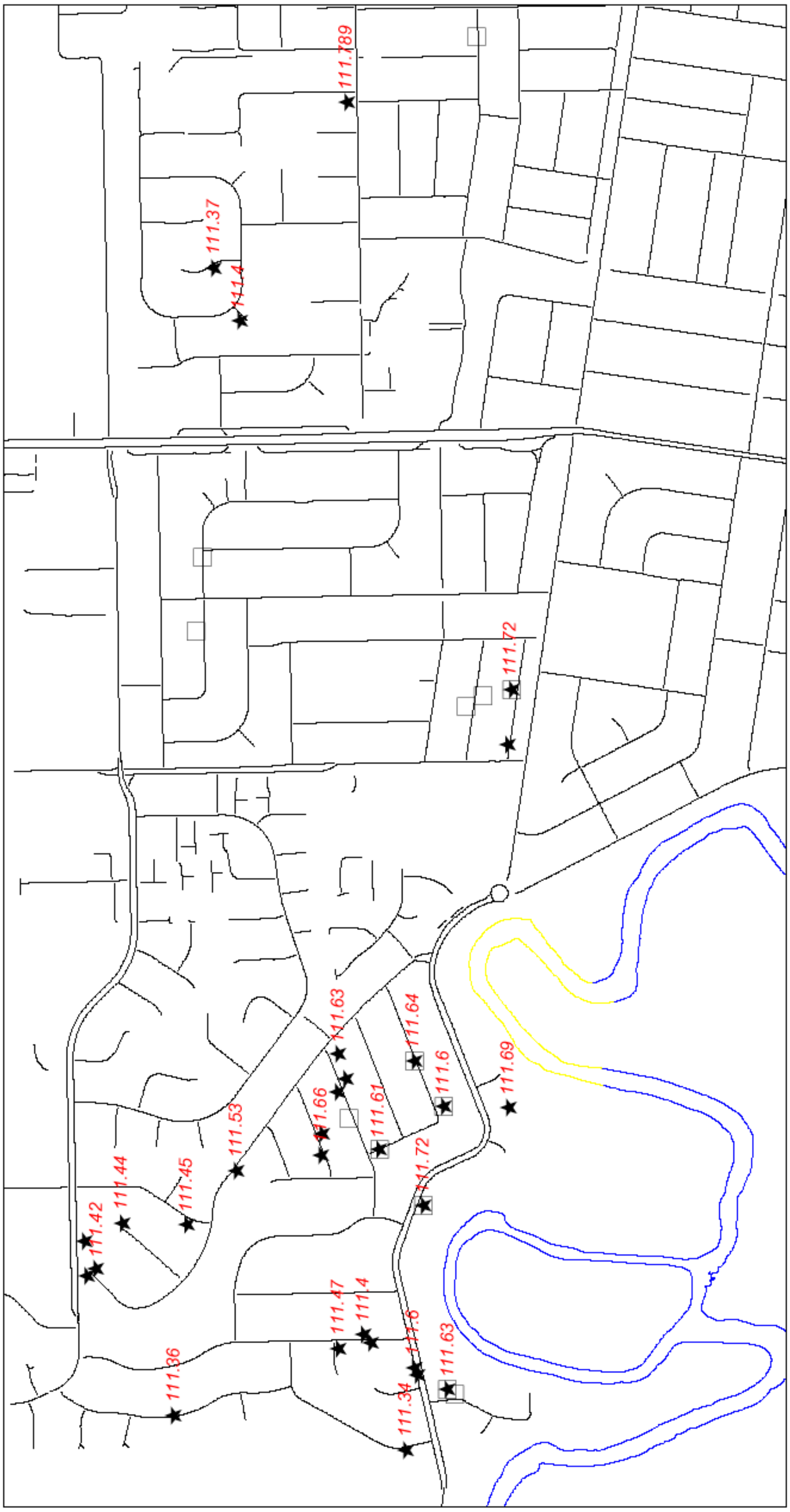
1993 Event Observed Maximum Flood Levels (m AHD)

C.2.2 Shepparton



1993 Event Observed Maximum Flood Levels
(m AHD)

C.2.3 North Shepparton

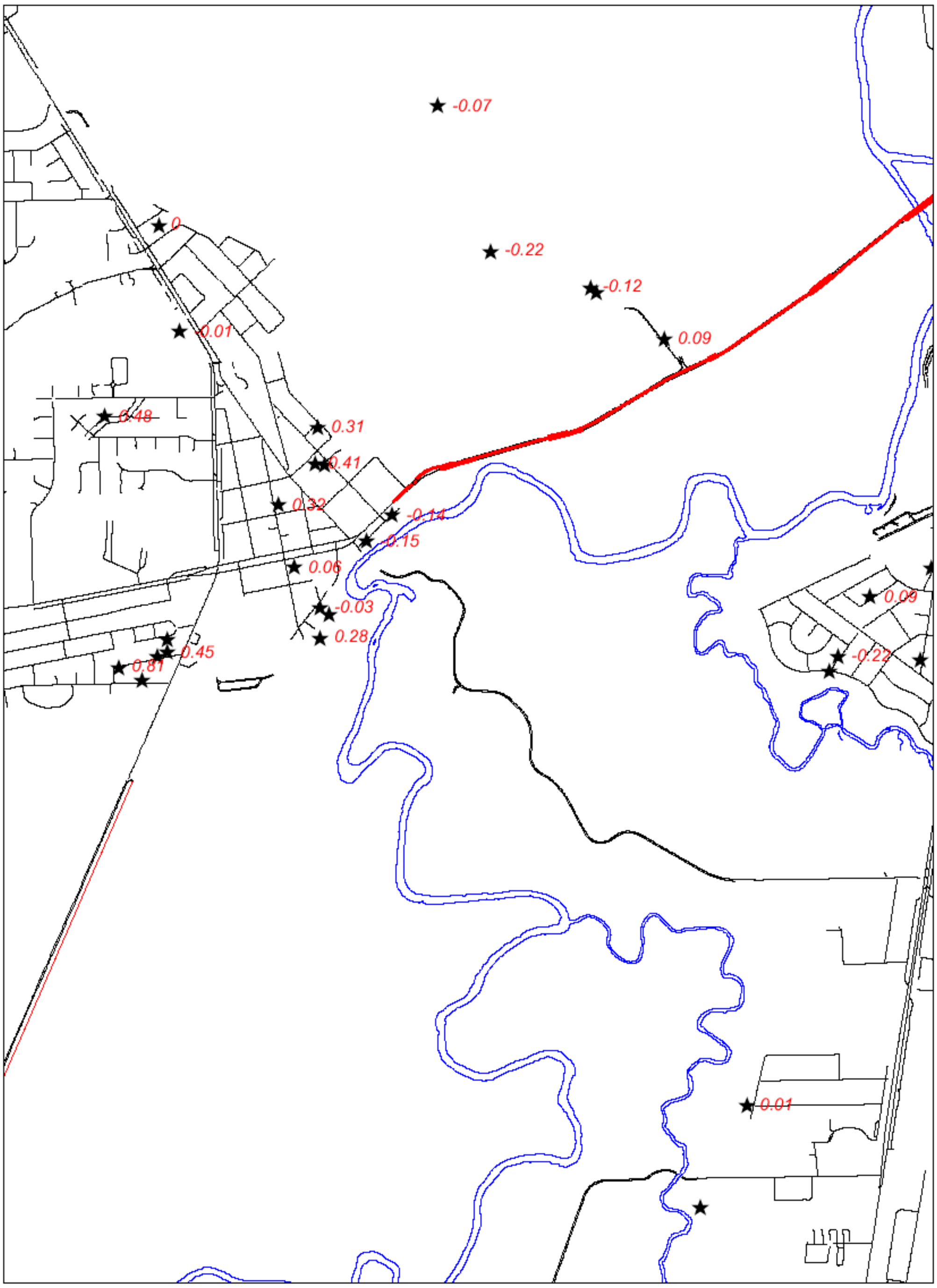


1993 Event Observed Maximum Flood Levels (m AHD)

Appendix D Comparison of Maximum Flood Levels

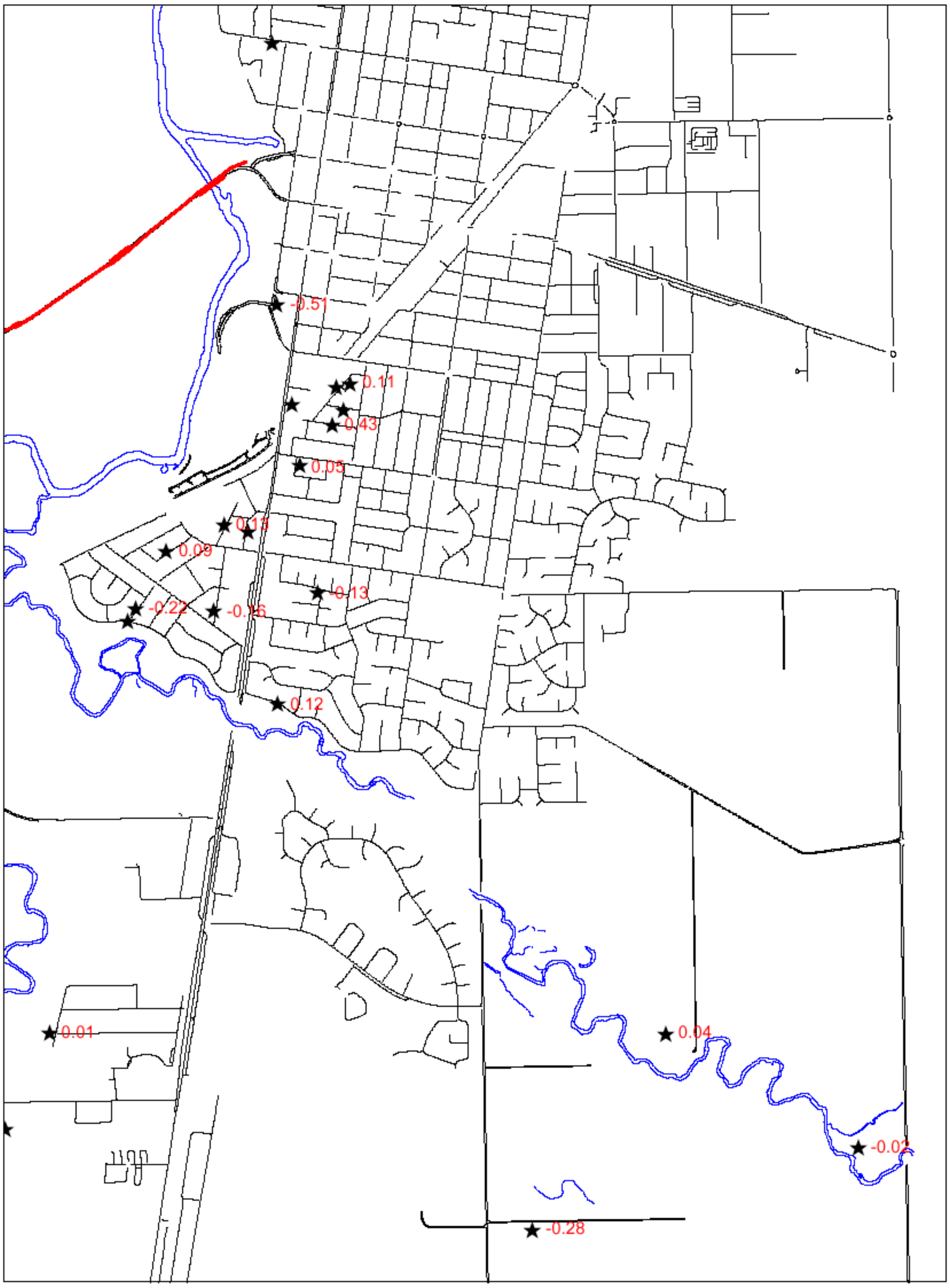
D.1 May 1974 Flood Level Comparison

D.1.1 Mooroopna



1974 Event Maximum Flood Level Comparison
(Predicted - Observed) (m)

D.1.2 Shepparton



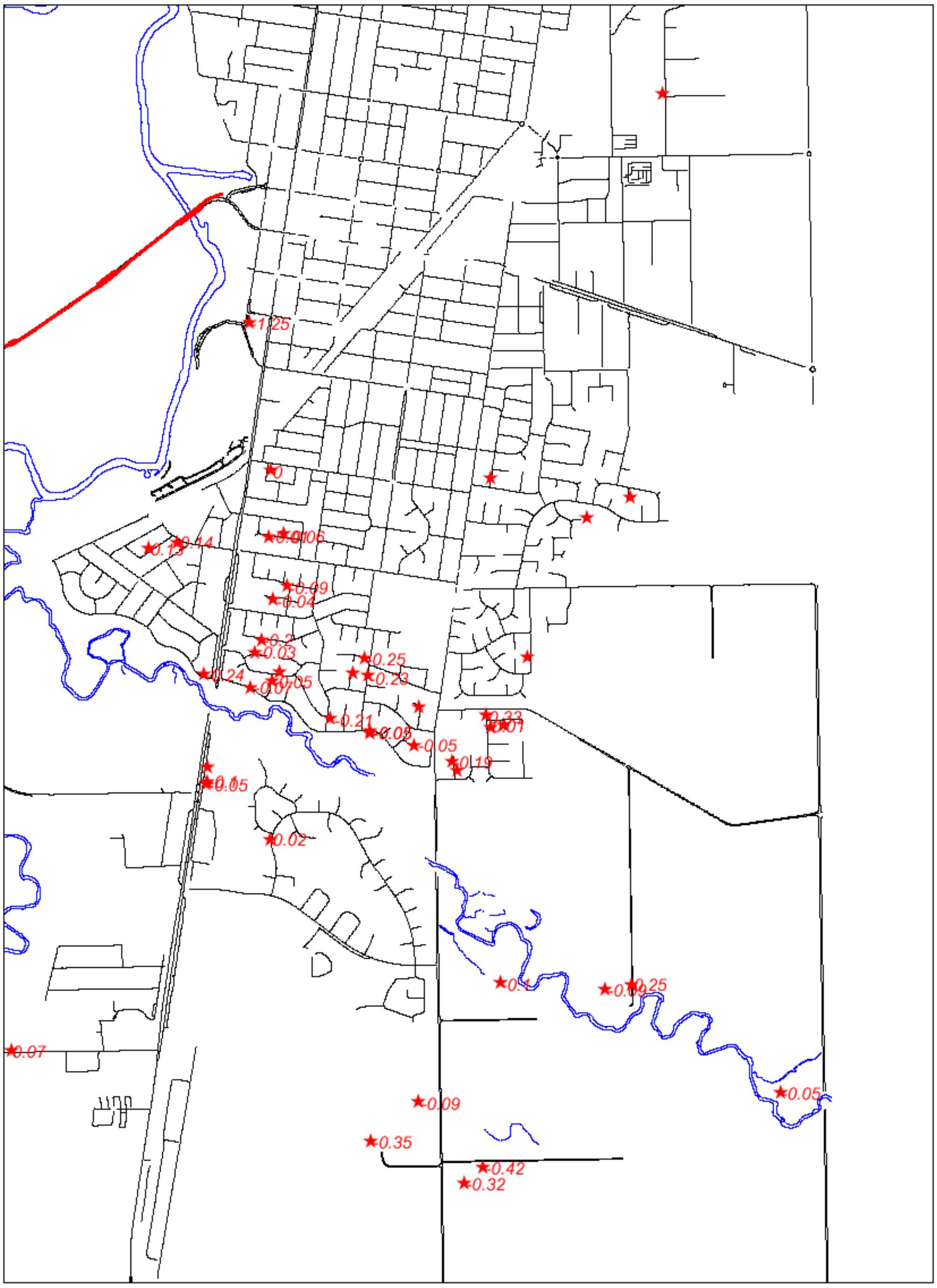
1974 Event Maximum Flood Level Comparison
(Predicted - Observed) (m)

D.1.3 North Shepparton

1974 Event Maximum Flood Level Comparison (Predicted - Observed) (m)

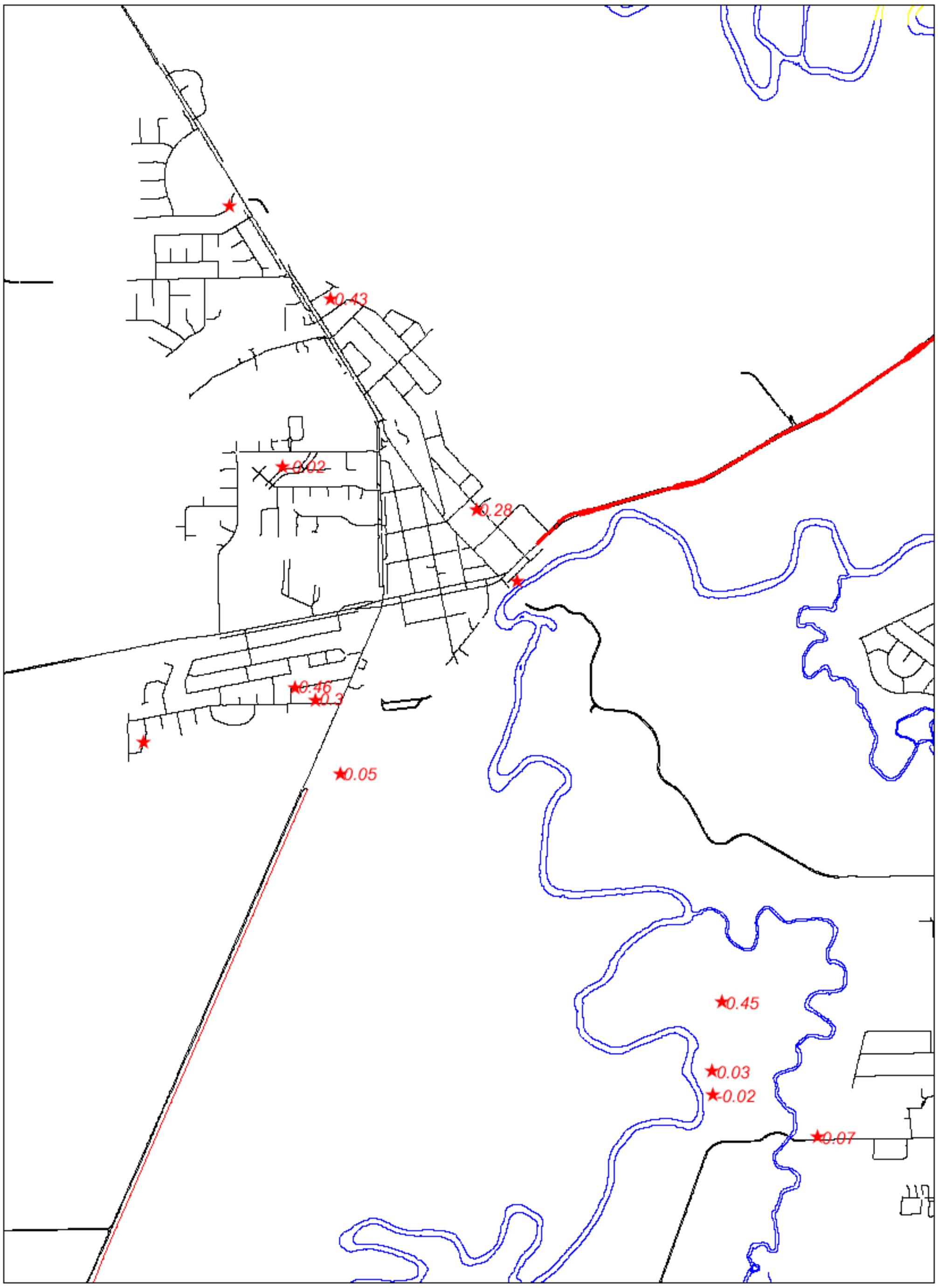
D.2 October 1993 Flood Level Comparison

D.2.1 Shepparton



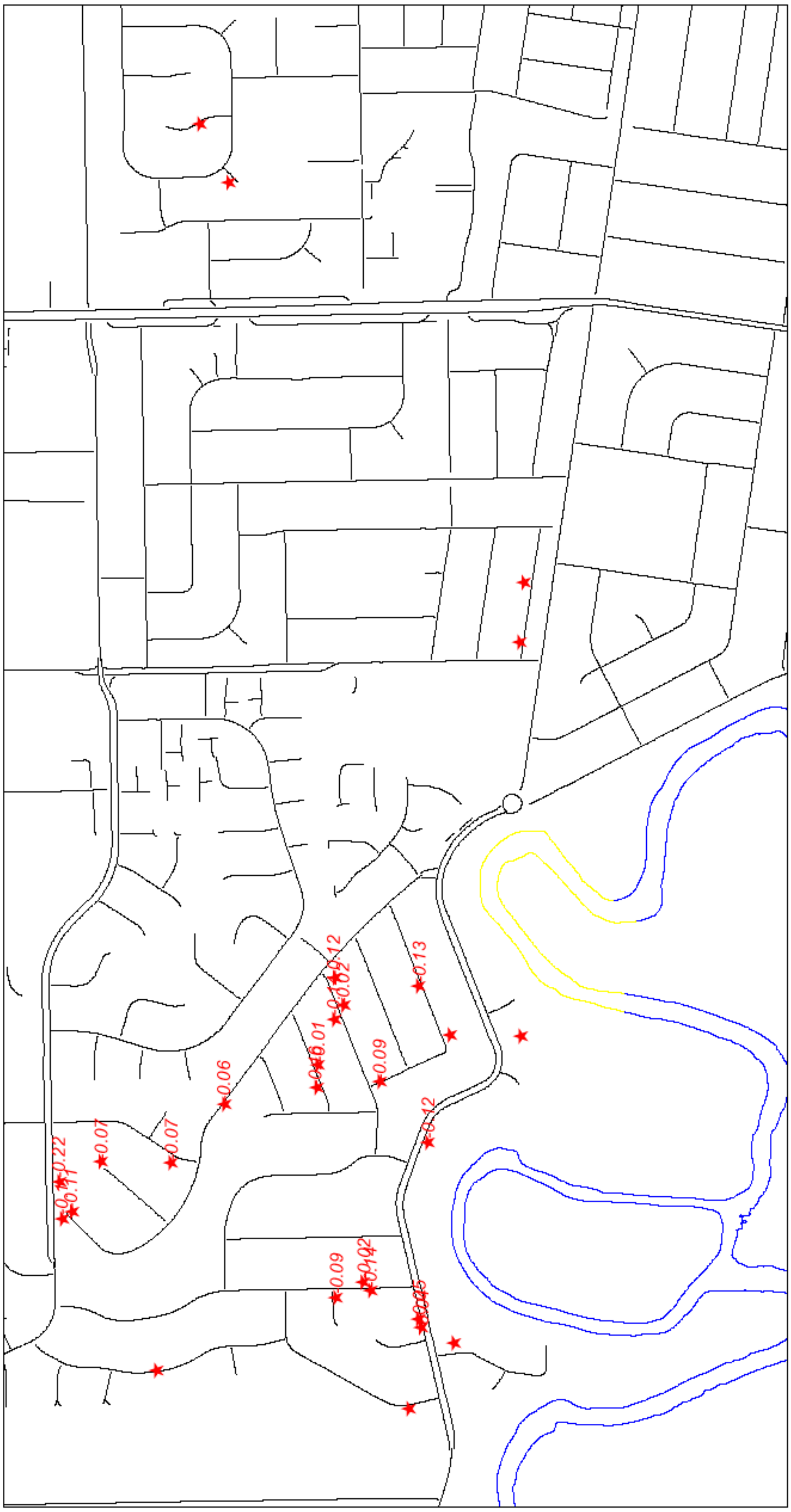
1993 Event Maximum Flood Level Comparison
(Predicted - Observed) (m)

D.2.2 Mooroopna



1993 Event Maximum Flood Level Comparison
(Predicted - Observed) (m)

D.2.3 North Shepparton



1993 Event Maximum Flood Level Comparison
(Predicted - Observed) (m)

Appendix E Rating Tables

Stream Name	**GOULBURN RIVER**	Station Name	**SHEPPARTON**
Station Number	**405204**	C.M.A.	**Goulburn Broken**
Catchment Area	**16125 km²**	Station Location	**8 m upstream Midlands Highway bridge**
Description	**Flat farmland**	Station Operating Period	**Site A – June 1921 to July 1967**
			Site B – October 1962 to December 1968
			Site C – December 1968 to date
		Gauge Zero	100.127 m AHD

GOULBURN RIVER @ SHEPPARTON

405204C

Rating Table 32.00

G.H.	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
2	407	653	919	1222	1503	1805	2105	2405	2705	2985
3	3250	3505	3745	3990	4235	4480	4720	4962	5210	5455
4	5705	5944	6187	6434	6685	6918	7154	7393	7634	7878
5	8125	8376	8630	8886	9145	9406	9670	9960	10253	10550
6	10850	11150	11454	11761	12071	12384	12700	13034	13371	13712
7	14057	14405	14757	15113	15471	15834	16200	16657	17122	17593
8	18070	18555	19046	19544	20049	20561	21080	21830	22598	23383
9	24187	25009	25849	26708	27586	28484	29400	31124	32930	34821
10	36800	40300	44000	48100	52400	57200	62300	67800	73700	80100
11	87000	94231	101990	110311	119228	128778	139000	148671	158924	169789
12	181296	193478	206367	220000						

Station 405200A GOULBURN RIVER at MURCHISON

Rating Table 53.00 01/01/1994 to Present C.T.F. = -0.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
-0.10	[ ]									
-0.00	54.0	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
0.00	54.0	62.0	70.4	79.0	88.0	96.2	105	113	122	132
0.10	141	150	158	167	175	183	191	198	206	213
0.20	220	227	233	240	246	252	258	264	270	276
0.30	283	289	296	303	310	318	325	333	341	348
0.40	356	365	374	382	391	400	409	418	427	436
0.50	445	455	465	475	486	496	506	517	527	537
0.60	548	560	572	585	597	609	622	635	647	660
0.70	673	686	699	713	726	739	753	766	780	794
0.80	807	821	835	849	863	877	892	906	921	935
0.90	950	964	979	994	1009	1024	1039	1054	1069	1085
1.00	1100	1116	1133	1150	1167	1183	1200	1217	1235	1252
1.10	1269	1287	1304	1322	1339	1357	1375	1393	1411	1429
1.20	1448	1466	1484	1503	1522	1540	1559	1578	1597	1616
1.30	1635	1654	1674	1693	1712	1732	1752	1771	1791	1811
1.40	1831	1851	1872	1892	1912	1933	1953	1974	1994	2015
1.50	2036	2057	2078	2099	2120	2142	2163	2184	2206	2228
1.60	2249	2271	2293	2315	2337	2359	2381	2403	2426	2448
1.70	2471	2493	2516	2539	2561	2584	2607	2630	2653	2677
1.80	2700	2722	2744	2767	2789	2811	2834	2856	2879	2902
1.90	2924	2947	2970	2993	3016	3039	3062	3085	3109	3132
2.00	3155	3179	3202	3226	3249	3273	3297	3321	3344	3368
2.10	3392	3416	3440	3465	3489	3513	3537	3562	3586	3611
2.20	3635	3660	3685	3709	3734	3759	3784	3809	3834	3859
2.30	3884	3910	3935	3960	3986	4011	4037	4062	4088	4114
2.40	4139	4165	4191	4217	4243	4269	4295	4321	4347	4374
2.50	4400	4425	4451	4476	4501	4527	4553	4578	4604	4629
2.60	4655	4681	4707	4733	4759	4785	4811	4837	4863	4889
2.70	4915	4941	4968	4994	5020	5047	5073	5100	5126	5153
2.80	5180	5206	5233	5260	5287	5314	5340	5367	5394	5421
2.90	5449	5476	5503	5530	5557	5585	5612	5640	5667	5695
3.00	5722	5750	5777	5805	5833	5861	5888	5916	5944	5972
3.10	6000	6027	6055	6082	6109	6137	6164	6192	6219	6247
3.20	6274	6302	6330	6357	6385	6413	6441	6469	6496	6524
3.30	6552	6580	6608	6636	6665	6693	6721	6749	6777	6806

Station 405200A GOULBURN RIVER at MURCHISON

Rating Table 53.00 01/01/1994 to Present C.T.F. = -0.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
3.40	6834	6863	6891	6919	6948	6976	7005	7034	7062	7091
3.50	7120	7148	7177	7206	7235	7264	7293	7322	7351	7380
3.60	7409	7438	7467	7496	7525	7555	7584	7613	7642	7672
3.70	7701	7731	7760	7790	7819	7849	7879	7908	7938	7968
3.80	7997	8027	8057	8087	8117	8147	8177	8207	8237	8267
3.90	8297	8327	8357	8388	8418	8448	8478	8509	8539	8570
4.00	8600	8629	8658	8686	8715	8744	8773	8802	8831	8860
4.10	8889	8918	8947	8976	9005	9034	9063	9093	9122	9151
4.20	9180	9210	9239	9268	9298	9327	9356	9386	9415	9445
4.30	9474	9504	9533	9563	9593	9622	9652	9682	9711	9741
4.40	9771	9801	9830	9860	9890	9920	9950	9980	10010	10040
4.50	10070	10100	10130	10160	10190	10220	10250	10280	10311	10341
4.60	10371	10401	10432	10462	10492	10523	10553	10583	10614	10644
4.70	10675	10705	10736	10766	10797	10828	10858	10889	10919	10950
4.80	10981	11012	11042	11073	11104	11135	11166	11197	11227	11258
4.90	11289	11320	11351	11382	11413	11444	11475	11507	11538	11569
5.00	11600	11631	11661	11692	11723	11753	11784	11815	11845	11876
5.10	11907	11938	11969	11999	12030	12061	12092	12123	12154	12185
5.20	12216	12247	12278	12309	12340	12371	12402	12433	12464	12496
5.30	12527	12558	12589	12620	12652	12683	12714	12746	12777	12808
5.40	12840	12871	12903	12934	12965	12997	13028	13060	13091	13123
5.50	13155	13186	13218	13249	13281	13313	13344	13376	13408	13440
5.60	13471	13503	13535	13567	13599	13631	13662	13694	13726	13758
5.70	13790	13822	13854	13886	13918	13950	13982	14014	14047	14079
5.80	14111	14143	14175	14207	14240	14272	14304	14336	14369	14401
5.90	14433	14466	14498	14531	14563	14595	14628	14660	14693	14725
6.00	14758	14790	14823	14856	14888	14921	14953	14986	15019	15051
6.10	15084	15117	15150	15182	15215	15248	15281	15314	15346	15379
6.20	15412	15445	15478	15511	15544	15577	15610	15643	15676	15709
6.30	15742	15775	15808	15841	15874	15908	15941	15974	16007	16040
6.40	16074	16107	16140	16174	16207	16240	16274	16307	16340	16374
6.50	16407	16441	16474	16507	16541	16574	16608	16642	16675	16709
6.60	16742	16776	16809	16843	16877	16910	16944	16978	17012	17045
6.70	17079	17113	17147	17181	17214	17248	17282	17316	17350	17384
6.80	17418	17452	17486	17520	17554	17588	17622	17656	17690	17724
6.90	17758	17792	17826	17860	17895	17929	17963	17997	18031	18066

Station 405200A GOULBURN RIVER at MURCHISON

Rating Table 53.00 01/01/1994 to Present C.T.F. = -0.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
7.00	18100	18138	18176	18213	18251	18289	18327	18365	18403	18441
7.10	18479	18517	18555	18593	18631	18669	18707	18745	18784	18822
7.20	18860	18898	18937	18975	19013	19052	19090	19129	19167	19205
7.30	19244	19282	19321	19360	19398	19437	19475	19514	19553	19592
7.40	19630	19669	19708	19747	19786	19824	19863	19902	19941	19980
7.50	20019	20058	20097	20136	20175	20214	20254	20293	20332	20371
7.60	20410	20450	20489	20528	20568	20607	20646	20686	20725	20765
7.70	20804	20844	20883	20923	20962	21002	21042	21081	21121	21161
7.80	21200	21240	21280	21320	21360	21399	21439	21479	21519	21559
7.90	21599	21639	21679	21719	21759	21799	21839	21879	21920	21960
8.00	22000	22058	22116	22174	22233	22291	22349	22408	22466	22525
8.10	22584	22643	22701	22760	22819	22879	22938	22997	23057	23116
8.20	23176	23235	23295	23355	23415	23475	23535	23595	23655	23715
8.30	23776	23836	23897	23957	24018	24079	24139	24200	24261	24323
8.40	24384	24445	24506	24568	24629	24691	24752	24814	24876	24938
8.50	25000	25080	25160	25240	25321	25401	25482	25563	25644	25726
8.60	25807	25889	25970	26052	26134	26217	26299	26382	26464	26547
8.70	26630	26714	26797	26881	26964	27048	27132	27216	27301	27385
8.80	27470	27555	27640	27725	27811	27896	27982	28068	28154	28240
8.90	28327	28413	28500	28587	28674	28761	28849	28936	29024	29112
9.00	29200	29335	29471	29608	29744	29882	30020	30158	30297	30436
9.10	30576	30716	30856	30998	31139	31282	31424	31567	31711	31855
9.20	32000	32145	32291	32437	32584	32731	32878	33027	33175	33325
9.30	33474	33625	33775	33927	34078	34231	34384	34537	34691	34845
9.40	35000	35239	35479	35720	35963	36207	36453	36700	36948	37198
9.50	37450	37702	37957	38212	38469	38728	38988	39249	39512	39777
9.60	40043	40310	40579	40850	41122	41395	41670	41947	42225	42505
9.70	42786	43069	43353	43639	43927	44216	44507	44799	45093	45389
9.80	45686	45985	46286	46588	46892	47198	47505	47814	48124	48437
9.90	48751	49067	49384	49704	50025	50347	50672	50998	51326	51656
10.00	51988	52321	52656	52994	53332	53673	54013	54344	54677	55012
10.10	55348	55686	56026	56367	56711	57056	57402	57751	58101	58453
10.20	58807	59163	59521	59880	60241	60604	60969	61335	61704	62074
10.30	62446	62820	63196	63574	63954	64335	64719	65115	65518	65923
10.40	66330	66739	67151	67564	67980	68397	68817	69240	69664	70090
10.50	70519	70950	71383	71819	72256	72696	73138	73583	74030	74479
10.60	74930	75383	75839	76298	76758	77221	77686	78160	78660	79163

Sinclair Knight Merz HYRATAB V102 Output 24/05/2002

Station 405200A GOULBURN RIVER at MURCHISON

Rating Table 53.00 01/01/1994 to Present C.T.F. = -0.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
10.70	79669	80178	80689	81203	81720	82239	82762	83287	83815	84346
10.80	84880	85417	85957	86499	87045	87593	88145	88699	89256	89817
10.90	90380	90946	91516	92088	92664	93242	93824	94409	94996	95587
11.00	96182	96779	97379	97983	98590	99200	99813	100429	101049	101672
11.10	102298	102928	103561	104197	104836	105479	106125	106775	107428	108085
11.20	108744	109408	110074	110745	111418	112096	112776	113460	114148	114836
11.30	115528	116223	116922	117625	118331	119041	119755	120472	121192	121917
11.40	122645	123377	124113	124852	125595	126342	127093	127847	128605	129367
11.50	130133	130903	131677	132455	133236	134022	134811	135604	136402	137203
11.60	138009	138818	139632	140449	141271	142096	142926	143760	144598	145441
11.70	146287	147138	147993	148852	149715	150583	151455	152331	153212	154097
11.80	154986	155880	156778	157681	158588	159499	160415	161335	162260	163176
11.90	164064	164957	165853	166754	167659	168568	169481	170399	171320	172246
12.00	173176	174110	175049	175992	176939	177890	178846	179806	180770	181739
12.10	182712	183690	184672	185659	186650	187645	188645	189649	190658	191672
12.20	192690	193712	194740	195771	196808	197849	198895	199945	201000	202060
12.30	203124	204194	205268	206347	207430	208519	209612	210710	211813	212921
12.40	214033	215151	216274	217401	218534	219671	220814	221961	223114	224272

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All rated data has been coded as reliable
except where the following tags are used...
[ ] Data Not Recorded

Sinclair Knight Merz HYRATAB V102 Output 24/05/2002

Station 404222A BROKEN RIVER at ORRVALE

Rating Table 2.00 22/02/1992 to Present C.T.F. = 0.8100

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
-1.00	0.0									
....										
0.80	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
0.90	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]	[ ]
1.00	37.0	39.4	41.9	44.4	46.9	49.4	52.0	54.6	57.2	59.8
1.10	62.5	67.3	72.2	77.4	82.8	88.3	94.1	100.0	106	112
1.20	119	124	130	135	141	147	153	159	165	171
1.30	177	183	189	195	202	208	214	221	228	234
1.40	241	248	255	262	269	277	284	292	299	307
1.50	314	322	330	337	344	352	359	367	374	382
1.60	389	397	404	412	420	428	435	443	451	458
1.70	466	474	482	490	498	506	514	522	530	538
1.80	547	555	563	572	580	588	597	605	614	623
1.90	631	640	649	658	666	675	684	693	702	711
2.00	720	729	738	747	757	766	775	785	794	804
2.10	813	822	832	842	851	861	871	880	890	900
2.20	910	920	929	939	949	959	969	980	990	1000
2.30	1010	1020	1030	1041	1051	1061	1072	1082	1092	1103
2.40	1113	1124	1134	1145	1156	1166	1177	1188	1198	1209
2.50	1220	1231	1242	1253	1264	1275	1286	1297	1308	1319
2.60	1330	1342	1354	1367	1379	1391	1404	1416	1429	1441
2.70	1454	1466	1479	1492	1504	1517	1530	1543	1556	1569
2.80	1582	1595	1608	1621	1634	1647	1660	1674	1687	1700
2.90	1714	1727	1741	1754	1768	1781	1795	1809	1822	1836
3.00	1850	1864	1878	1892	1906	1920	1935	1949	1963	1978
3.10	1992	2006	2021	2035	2050	2064	2079	2094	2108	2123
3.20	2138	2153	2168	2182	2197	2212	2227	2242	2258	2273
3.30	2288	2303	2318	2334	2349	2364	2380	2395	2411	2426
3.40	2442	2458	2473	2489	2505	2521	2536	2552	2568	2584
3.50	2600	2617	2634	2652	2669	2686	2704	2721	2739	2757
3.60	2774	2792	2810	2827	2845	2863	2881	2899	2917	2935
3.70	2953	2972	2990	3008	3026	3045	3063	3082	3100	3119
3.80	3137	3156	3175	3194	3212	3231	3250	3269	3288	3307
3.90	3326	3345	3365	3384	3403	3423	3442	3461	3481	3500
4.00	3520	3541	3561	3582	3603	3623	3644	3665	3686	3707

Sinclair Knight Merz HYRATAB V102 Output 24/05/2002

Station 404222A BROKEN RIVER at ORRVALE

Rating Table 2.00 22/02/1992 to Present C.T.F. = 0.8100

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
4.10	3728	3749	3770	3791	3813	3834	3855	3877	3898	3920
4.20	3941	3963	3985	4007	4028	4050	4072	4094	4116	4138
4.30	4160	4183	4205	4227	4250	4272	4294	4317	4340	4362
4.40	4385	4408	4430	4453	4476	4499	4522	4545	4568	4591
4.50	4615	4638	4661	4685	4708	4732	4755	4779	4803	4826
4.60	4850	4874	4898	4923	4947	4971	4996	5020	5045	5070
4.70	5094	5119	5144	5169	5194	5219	5244	5269	5294	5319
4.80	5344	5370	5395	5420	5446	5471	5497	5523	5548	5574
4.90	5600	5626	5652	5678	5704	5730	5756	5782	5808	5835
5.00	5861	5887	5914	5940	5967	5994	6020	6047	6074	6101
5.10	6128	6155	6182	6209	6236	6263	6290	6318	6345	6373
5.20	6400	6431	6463	6495	6526	6558	6590	6622	6654	6686
5.30	6718	6750	6782	6815	6847	6880	6913	6945	6978	7011
5.40	7044	7077	7110	7144	7177	7210	7244	7277	7311	7345
5.50	7379	7412	7446	7481	7515	7549	7583	7618	7652	7687
5.60	7721	7756	7791	7826	7861	7896	7931	7966	8002	8037
5.70	8073	8108	8144	8180	8215	8251	8287	8323	8359	8396
5.80	8432	8468	8505	8542	8578	8615	8652	8689	8726	8763
5.90	8800	8850	8901	8951	9002	9053	9104	9155	9207	9259
6.00	9310	9363	9415	9467	9520	9573	9626	9679	9733	9786
6.10	9840	9894	9948	10003	10057	10112	10167	10222	10278	10333
6.20	10389	10445	10501	10557	10614	10671	10728	10785	10842	10900
6.30	10957	11015	11073	11132	11190	11249	11308	11367	11427	11486
6.40	11546	11606	11666	11726	11787	11848	11909	11970	12031	12093
6.50	12155	12217	12279	12341	12404	12467	12530	12593	12657	12720
6.60	12784	12848	12913	12977	13042	13107	13172	13237	13303	13369
6.70	13435	13501	13567	13634	13701	13768	13835	13903	13970	14038
6.80	14107	14175	14244	14312	14381	14451	14520	14590	14660	14730
6.90	14800	14901	15003	15105	15207	15310	15414	15518	15623	15728
7.00	15834	15940	16047	16155	16263	16371	16480	16590	16700	16811
7.10	16922	17034	17146	17259	17373	17487	17601	17717	17832	17949
7.20	18066	18183	18301	18420	18539	18659	18779	18900	19022	19144
7.30	19267	19390	19514	19639	19764	19890	20016	20144	20271	20399
7.40	20528	20658	20788	20918	21050	21182	21314	21448	21581	21716
7.50	21851	21987	22123	22260	22398	22536	22675	22815	22955	23096
7.60	23238	23380	23523	23666	23810	23955	24101	24247	24394	24542
7.70	24690	24839	24988	25139	25290	25441	25594	25747	25900	26055

Sinclair Knight Merz HYRATAB V102 Output 24/05/2002

Station 404222A BROKEN RIVER at ORRVALE

Rating Table 2.00 22/02/1992 to Present C.T.F. = 0.8100

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
7.80	26210	26366	26522	26679	26837	26996	27155	27315	27476	27638
7.90	27800	28044	28290	28537	28787	29038	29291	29546	29803	30061
8.00	30322	30584	30848	31115	31383	31653	31925	32199	32475	32753
8.10	33032	33314	33598	33884	34172	34462	34754	35049	35345	35643
8.20	35944	36246	36551	36858	37167	37478	37791	38107	38425	38745
8.30	39067	39391	39718	40047	40378	40712	41048	41386	41727	42070
8.40	42415	42762	43112	43465	43820	44177	44537	44899	45263	45630
8.50	46000									

------------------ Notes -------------------
All rated data has been coded as reliable
except where the following tags are used...
[ ] Data Not Recorded

Sinclair Knight Merz HYRATAB V102 Output 24/05/2002

Station 405269A SEVEN CREEKS at KIALLA WEST

Rating Table 5.00 03/11/1997 to Present C.T.F. = 1.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
1.10	0.0	0.325	0.650	0.975	1.30	1.71	2.15	2.70	3.40	4.42
1.20	5.58	6.90	8.25	9.74	11.4	13.2	15.3	17.6	20.0	22.4
1.30	25.0	27.8	30.7	33.8	37.1	40.5	43.9	47.4	51.0	54.3
1.40	57.7	61.2	64.8	68.4	72.2	76.0	80.0	84.0	87.9	91.8
1.50	95.9	100.0	104	108	113	117	122	126	131	135
1.60	140	144	148	152	156	161	165	169	173	178
1.70	182	187	191	196	200	206	212	219	225	232
1.80	238	245	252	259	266	273	280	288	295	302
1.90	310	318	325	333	341	349	357	365	374	382
2.00	390	399	408	416	425	434	443	452	461	471
2.10	480	490	499	509	519	529	539	549	559	569
2.20	579	590	600	611	622	633	644	655	666	677
2.30	688	699	711	722	734	746	758	770	782	794
2.40	806	818	831	843	856	868	881	894	907	920
2.50	933	946	960	973	986	1000	1014	1028	1042	1056
2.60	1070	1084	1099	1113	1128	1142	1157	1172	1187	1202
2.70	1217	1232	1248	1263	1279	1294	1310	1326	1342	1358
2.80	1374	1390	1406	1422	1439	1455	1472	1489	1506	1523
2.90	1540	1557	1574	1591	1609	1626	1644	1661	1679	1697
3.00	1715	1733	1751	1770	1788	1806	1825	1844	1862	1881
3.10	1900	1919	1938	1957	1976	1996	2015	2034	2054	2074
3.20	2093	2113	2133	2153	2173	2194	2214	2234	2255	2275
3.30	2296	2317	2338	2359	2380	2401	2422	2444	2465	2487
3.40	2508	2530	2552	2574	2596	2618	2640	2662	2684	2707
3.50	2729	2752	2775	2798	2821	2844	2867	2890	2913	2937
3.60	2960	2984	3007	3031	3055	3079	3103	3127	3151	3176
3.70	3200	3225	3249	3274	3299	3324	3349	3374	3400	3425
3.80	3450	3476	3502	3527	3553	3579	3605	3631	3657	3684
3.90	3710	3737	3763	3790	3817	3844	3870	3898	3925	3952
4.00	3979	4007	4034	4062	4090	4117	4145	4173	4201	4230
4.10	4258	4286	4315	4343	4372	4401	4430	4458	4487	4517
4.20	4546	4575	4605	4634	4664	4693	4723	4753	4783	4813
4.30	4843	4873	4904	4934	4965	4995	5026	5057	5088	5119
4.40	5150	5186	5223	5260	5297	5334	5371	5408	5446	5483
4.50	5521	5559	5597	5635	5673	5712	5751	5789	5828	5867
4.60	5907	5946	5985	6025	6065	6105	6145	6185	6226	6266
4.70	6307	6348	6389	6430	6472	6513	6555	6597	6639	6681

Sinclair Knight Merz

HYRATAB V102 Output 24/05/2002

Station 405269A SEVEN CREEKS at KIALLA WEST

Rating Table 5.00 03/11/1997 to Present C.T.F. = 1.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
4.80	6723	6765	6808	6851	6893	6936	6980	7023	7066	7110
4.90	7154	7198	7242	7286	7330	7375	7420	7465	7510	7555
5.00	7600	7656	7713	7769	7826	7884	7941	7999	8057	8116
5.10	8174	8233	8292	8352	8412	8472	8532	8593	8653	8715
5.20	8776	8838	8900	8962	9025	9087	9151	9214	9278	9342
5.30	9406	9471	9535	9601	9666	9732	9798	9864	9931	9998
5.40	10065	10132	10200	10268	10337	10405	10474	10543	10613	10683
5.50	10753	10824	10894	10965	11037	11108	11180	11253	11325	11398
5.60	11471	11545	11619	11693	11767	11842	11917	11992	12068	12144
5.70	12220	12297	12374	12451	12528	12606	12684	12763	12841	12921
5.80	13000	13101	13202	13304	13406	13509	13613	13717	13821	13927
5.90	14032	14139	14246	14353	14462	14570	14680	14790	14900	15011
6.00	15123	15235	15348	15462	15576	15691	15806	15922	16039	16156
6.10	16274	16392	16511	16631	16751	16872	16994	17116	17239	17362
6.20	17486	17611	17737	17863	17989	18117	18245	18373	18503	18633
6.30	18763	18895	19027	19159	19293	19426	19561	19696	19832	19969
6.40	20107	20245	20383	20523	20663	20804	20945	21087	21230	21374
6.50	21518	21663	21809	21955	22102	22250	22399	22548	22698	22849
6.60	23000	23157	23316	23475	23634	23795	23956	24119	24282	24445
6.70	24610	24776	24942	25109	25277	25445	25615	25785	25956	26129
6.80	26301	26475	26650	26825	27001	27178	27356	~~27535~~	27715	27895
6.90	28076	28259	28442	28626	28810	28996	29183	29370	29558	29748
7.00	29938	30129	30321	30513	30707	30902	31097	31294	31491	31689
7.10	31888	32088	32289	32491	32694	32898	33102	33308	33514	33722
7.20	33930	34140	34350	34561	34773	34986	35201	35416	35632	35849
7.30	36067	36285	36505	36726	36948	37171	37395	37620	37845	38072
7.40	38300	38537	38776	39016	39256	39498	39741	39985	40230	40476
7.50	40724	40972	41222	41473	41725	~~[illegible]~~	42232	42487	42744	43001
7.60	43260	43520	43781	44043	44307	44571	44837	45104	45372	45641
7.70	45912	46183	46456	46730	47005	47282	47559	47838	48118	48400
7.80	48682	48966	49251	49537	49825	50113	50403	50694	50987	51280
7.90	51575	51871	52169	52467	52767	53069	53371	53675	53980	54286
8.00	54594	54903	55213	55525	55838	56152	56467	56784	57102	57422
8.10	~~57742~~	58065	58388	58713	59039	59366	59695	60025	60357	60690
8.20	61024	61359	61696	62035	62374	62715	63058	63402	63747	64094
8.30	64442	64791	65142	65494	65848	66203	66560	66918	67277	67638

Station 405269A SEVEN CREEKS at KIALLA WEST

Rating Table 5.00 03/11/1997 to Present C.T.F. = 1.1000

Converting 100 Stream Water Level in Metres
Into 141 Stream Discharge in Megalitres/Day

G.H.	0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
8.40	68000									

------------------ Notes -------------------
All rated data has been coded as reliable