Onkaparinga Catchment Water Management Board

Upper Onkaparinga River Catchment

Floodplain Mapping Report

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Upper Onkaparinga River Catchment

Floodplain Mapping Report

prepared for

Onkaparinga Catchment Water Management Board

in association with

Adelaide Hills Council
Bureau of Meteorology
City of Onkaparinga
District Council of Mount Barker
Transport SA

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Document History and Status

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Executive Summary

This report and the accompanying floodplain maps have been produced for the Onkaparinga Catchment Water Management Board (OCWMB). The extent of the study area includes the Upper Onkaparinga River, and sections of Aldgate Creek, Hahndorf Creek, Cox Creek (Bridgewater), Junction Creek (Balhannah), Echunga Creek, Lenswood Creek, Lobethal Creek and Dashwood Gully Creek (Kangarilla).

Peak flows used for derivation of the floodplain maps were taken from a hydrological investigation of the Upper Onkaparinga Catchment carried out by Transport SA (Dr David Kemp).

Hydraulic modelling for production of the maps was undertaken using HEC-RAS Ver 3.1. Geometric data for the hydraulic models was derived from a DTM produced using photogrammetric techniques by AEROmetrex Pty Ltd, existing ground survey, new ground survey undertaken for the study and site inspections. HECGeoRAS, an extension of ARCView GIS, was used to extract terrain information from the Digital Terrain Model (DTM) for input into the HEC-RAS geometry file.

Limited information was available for calibration of the hydraulic model. Validation of the modelled flood extents was largely based on a review of the flood maps by the project team and members of the Steering Committee. The flood inundation maps produced in this study were also found to be consistent with the approximate extent of flooding for the August 1992 flood event along the Onkaparinga River.

Flood inundation maps were produced for the study area for the 10, 50, 100, 200 and 500 year ARI and PMF flood events, while hazard maps were produced for the 50, 100 and 500 year ARI and PMF flood events. The Hazard maps were produced in accordance with the CSIRO publication, Floodplain Management in Australia (SCARM report 73).

An assessment of the 100 year ARI flood inundation extent indicated that over 150 dwellings / building structures are located within the floodplain. The townships of Balhannah, Hahndorf and Aldgate have the most significant number of structures at risk of flooding. It is recommended that further more detailed investigation be undertaken in these areas to assess the actual flood risk and to estimate the associated flood damages.

The final inundation and hazard maps have been produced using GIS and are available in both electronic and hardcopy formats.

1. Introduction

This report has been prepared to accompany the Upper Onkaparinga River Floodplain Maps, produced as a part of a study commissioned by the Onkaparinga Catchment Water Management Board (OCWMB). A Steering Committee comprising the OCWMB together with representatives from the Bureau of Meteorology, City of Onkaparinga, Adelaide Hills Council, District Council of Mount Barker and Transport SA provided guidance during the course of the project.

Floodplain maps for the following rivers and creeks were produced during the study:

- Onkaparinga River between Mylor and Charleston
- Aldgate Creek between the Onkaparinga River at Mylor and its branched upper reaches in Stirling West, Stirling and Stirling East
- Cox Creek between the township of Bridgewater and its upper reaches at Aldgate West and Arbury Park
- Lenswood Creek
- Dashwood Gully Creek at Kangarilla and Cut Hill Road
- Echunga Creeks (3 of)
- Hahndorf Creek between the Onkaparinga River and the branched upper reaches through the developed areas of Hahndorf
- Eastern and Western Branches of Lobethal Creek.

The floodplain maps produced during the study are intended to convey the broad risk of inundation and the degree of flood hazard along the various watercourses.

The intended users of the maps are expected to be:

- Planners seeking guidance on the risk of inundation and associated redevelopment requirements of land adjacent to the watercourses
- Emergency services personnel seeking to identify, assist and, if necessary, evacuate areas vulnerable to flooding
- Asset Managers (Local Government, State Government and private interest groups) seeking to plan/consider flood damage risks to infrastructure.
- Community members seeking to determine the likely extent of flooding on land that they potentially have an interest in or land that they own.

2. Data Sources

The study area comprises approximately 80 kilometres of rivers and creeks with more than 150 bridge and culvert structures. Data used in preparation of the floodplain maps is described below.

2.1 Hydrology

Hydrological data for the Floodplain Mapping Study was produced as part of a separate investigation carried out by Transport SA (Dr David Kemp). The hydrology report has been included in its entirety as Appendix B to this report.

During the course of this study, the study team liaised with Transport SA regarding the production of additional flow data at specific locations.

The hydrographs produced from the hydrological modelling were used to determine travel times for the peak of the 100 year ARI flood event along the various watercourses. These travel times were also found to be similar for the 50 and the 500 year ARI flood events. This data has been shown on the floodplain maps as an indication of the speed at which floods are expected to peak along the various reaches of the streams. In the case of the Onkaparinga River, a “time zero” has been established at Charleston and the maps show the travel time of the flood peak downstream of this point. For Aldgate and Hahndorf Creeks, travel times have been related to a time zero at the upstream most end of each tributary.

2.2 Survey Data

2.2.1 Photogrammetry

Photogrammetry was used as the primary means of gaining survey data for the floodplain mapping.

Photogrammetric survey and production of the digital terrain model was undertaken by AEROmetrex Pty Ltd.

Aerial photography of the entire study area was obtained in January 2002 at a scale of 1:8000. The aerial photographs were scanned at high resolution and rectified to produce a digital ortho-photo of the areas that were flown. The digital ortho-photo tiles (with geocoding), have been colour corrected to provide a seamless rectified digital aerial photograph of the study area and have been supplied in .ecw format on the study CD ROMs.

A digital terrain model was produced from the rectified photography using 'soft' photogrammetry techniques. The data set comprised a spread of land points together with break lines along defining terrain features. Break lines typically have been produced for edges and centrelines of roads in the vicinity of creek crossings. Break lines were also generally produced at distinct changes of grade.

2.2.2 Ground Survey

There is an extensive tree canopy along most of the watercourses within the study area. This canopy precludes the determination of accurate channel geometry in some areas using photogrammetry.

In specific areas, particularly within the various townships, it was deemed to be necessary to gain more accurate channel geometry data to enable flood levels to be determined with a higher accuracy. Ground based survey was used in these areas to supplement the photogrammetry. This additional survey generally involved the survey of channel cross sections.

2.2.3 Existing Survey and Cross Section Data

Tonkin Consulting and other engineering consultants have undertaken a number of flood investigation projects of various sizes in the study area. Each of these projects required the collection of survey data. Where possible, the existing survey was utilised to minimise the extent of new survey work undertaken for this current investigation.

In addition, the hydraulic modelling outputs from this previous work have been used to assist in the calibration and verification of modelling results undertaken for this broader study.

2.3 Bridge and Culvert Structures

Where available, design drawings for bridges and culverts along the watercourses in the study area were obtained from Transport SA and the various Councils. Information shown on the design drawings was verified by field inspection.

The basic dimensions of all major bridge and culvert openings were measured in the field. In addition, inverts were measured in relation to adjacent land features (such as the crown of the road) in order to facilitate correlation with the digital terrain model.

Where possible, bridge and culvert structures were photographed. Photographs have been numbered and the mapping shows photograph numbers as appropriate. The CD ROM which accompanies this report contains the numbered photographs for reference.

3. Methodology

3.1 Modelling Regime

The study area comprises well defined and incised channels and streams. Tributaries which flow to the Onkaparinga River such as Aldgate Creek and Cox Creek have very well defined valleys and, when in flood, these streams will be fast flowing and accommodate very little floodplain storage. Accordingly, the benefits of a 1D or 2D unsteady hydraulic model are largely inconsequential along these streams as the degree of floodplain storage and associated flood peak attenuation are negligible. As a result, a 1D steady flow model was used on the tributary streams such as Aldgate Creek, Leslie Creek, Cox Creek, Echunga Creek(s), Hahndorf Creek and the creeks through Lobethal.

Along the Onkaparinga River between Charleston and Mylor, considerable effort was made to run an unsteady model since it was viewed that there may be some benefit in accounting for floodplain storage effects. However, there were numerous unresolvable instabilities encountered in the course of the modelling. The instabilities were considered to be associated with:

- Frequent grade changes associated with the invert of the natural channel
- Numerous bridge and culvert structures
- Variation in cross-section conveyance (cross-sections can vary significantly between those having a deeply incised channel with very little storage to open flood plain areas).

The study team concluded that there would need to be significant compromises made in the hydraulic model in order to achieve a mathematically stable unsteady model. In short reaches where both steady and unsteady models were successfully run, it was evident that there were negligible differences in the model results and as such a 1D steady model was adopted along the entire length of the Onkaparinga River.

3.2 Computer Processes and Software

The hydraulic modelling package used throughout this project was HEC-RAS Version 3.1.

HEC-RAS is an industry standard one-dimensional hydraulic modelling package. Traditionally, HEC-RAS has been a steady state model only. However, Version 3 of the model includes the capability to carry out unsteady flow modelling.

Extensive use of the 'HecGeoRas' ArcView GIS extension has been made throughout this project for the preparation of data for the hydraulic model. In simple terms, 'HecGeoRas' provides an interface between HEC-RAS and ArcView that facilitates the production of inundation maps. The process is as follows:

1. Prepare a digital terrain model of the study area and mount the data in ArcView.
2. Using the 'HecGeoRas' extension, draw stream lines, bank lines, flow paths and cross sections within the Arcview GIS for the reach of river to be modelled.
3. Upon completion of the model preparation using 'HecGeoRas', extract a normal HEC-RAS geometry file using the software.
4. Import the geometry file into HEC-RAS and add bridge and culvert data.
5. Once the model has been prepared, run the hydraulic analysis and assess the results.
6. Export the water level data from HEC-RAS into ArcView for mapping.
7. Produce a water surface TIN in Arcview and intersect this TIN with the terrain model to calculate a flood inundation map.

Details of the modelling results in specific portions of the study area are presented in later sections of this report.

4. Hydraulic Modelling

4.1 Detailed Hydraulic Modelling Description

The study area was modelled in sections to reduce computation time and to provide reaches of a size that would facilitate systematic debugging and error checking. Furthermore, the extensive use of GIS applications for mapping of the floodplain required division of the area into smaller model sections.

The study area was divided into twenty separate HEC-RAS models, namely: Balhannah, Bridgewater, Echunga, Hahndorf, Kangarilla, Lenswood, Lobethal, four models covering Aldgate and Leslie Creeks (Aldgate, Stirling and Mylor), four models along the Onkaparinga River (Mylor to Charleston) and five models for the smaller tributaries along the Onkaparinga River. The extent of each of these models is illustrated in Figure 4.1.

In the case of the main river reaches, the adjacent HEC-RAS models overlap over a distance of several hundred metres to ensure satisfactory continuation of the backwater curve from model to model.

4.2 Boundary Conditions

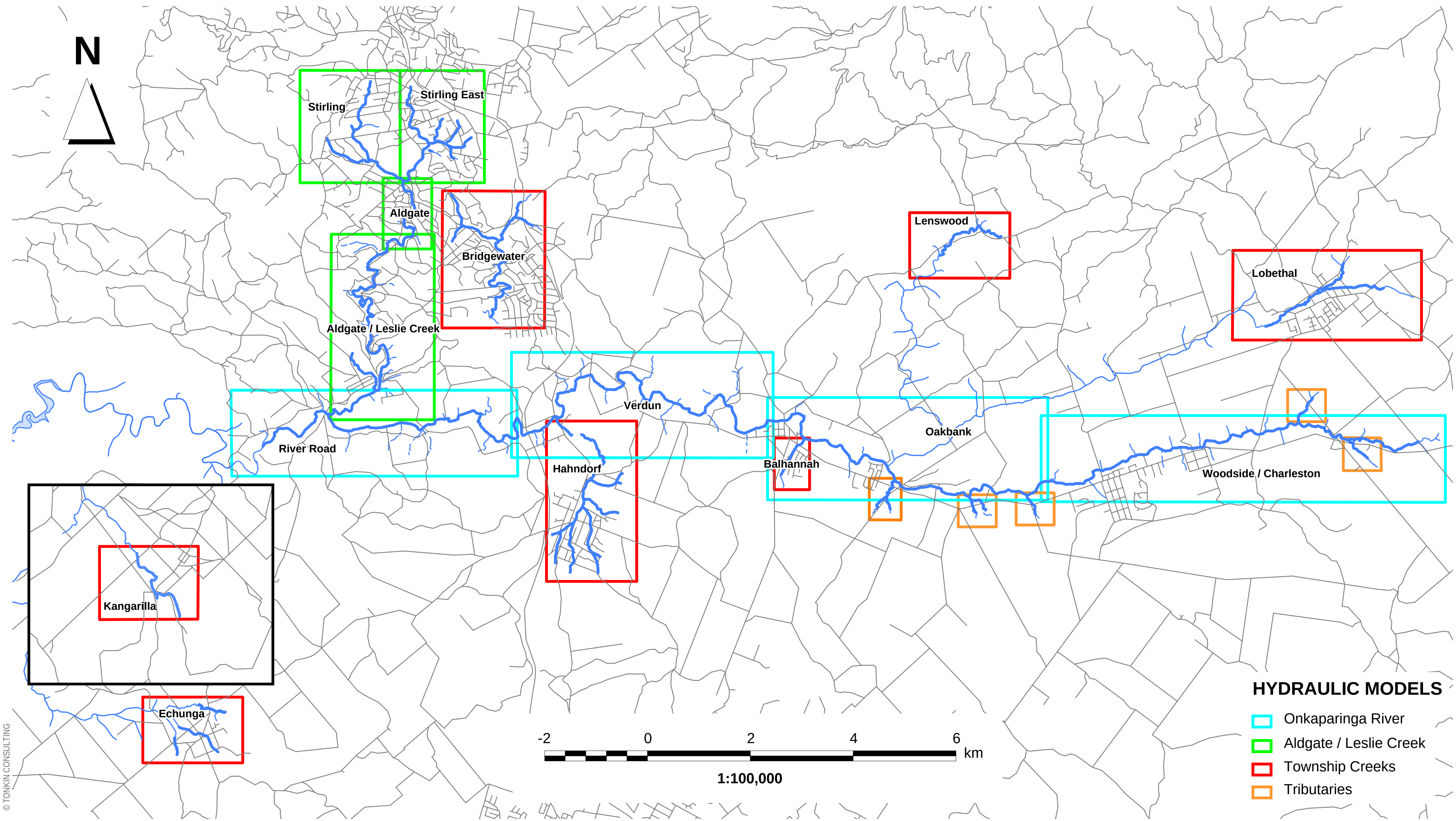
For the downstream most model of the Onkaparinga River, the downstream boundary condition was set to normal depth for the average channel slope in that reach of the model. This was considered to be appropriate since the flow regime was largely subcritical and selection of normal depth would result in a marginally conservative start depth.

The upstream models utilised the calculated water surface elevation from the adjacent downstream model as the downstream boundary condition.

Models of tributaries along the main river reach used the river water surface level as the downstream boundary condition.

4.3 Mannings Roughness Coefficients

Manning’s Roughness Coefficients for the channels and the floodplains were initially estimated from the vegetation shown on the aerial photography. Those areas having dense tree cover were estimated to have a roughness coefficient of approximately 0.08, while areas with short grass were estimated to have a roughness coefficient of approximately 0.03 - 0.035.



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MAP DETAILS

Road Data:	MapInfo Australia
River Data:	Onkaparinga Catchment Water Management Board
Job Number:	2002.0041
Filename:	Model_Boundaries.wor
Drawn:	Tricia de Vink
Date:	24/02/2004

Onkaparinga Catchment Water Management Board

Upper Onkaparinga River Floodplain Mapping

HYDRAULIC MODEL LAYOUT

Figure 4.1

Field inspections of several areas were carried out to confirm the estimates of Manning's Roughness Coefficients.

4.4 Cross-Sections

The distances between cross-sections along the Onkaparinga River were typically between 70 - 100 metres. However, along the smaller creeks and tributaries the distance between sections generally ranged from 20 - 50 metres. Greater cross-sectional spacing along the main river was considered satisfactory since the flows were much higher and the cross-sectional shape more consistent.

Within ArcView, cross-section cut-lines were positioned to ensure that the cross-sections were perpendicular to the flow so that the cross-sectional data in the model would closely represent the available cross-sectional flow area. Where possible, cross-sections were taken at locations clear of dense vegetation to utilise photogrammetry that would be unaffected by vegetation.

In most cases, the cross-sections were cut to fully span across the expected width of the Probable Maximum Flood (PMF).

In order to better control the invert level of the cross-sections, minor adjustments were made to the cross-section profiles using either field measurements of the channel depth or ground based survey data.

4.5 Bridges

Measurements of each bridge and culvert along the Onkaparinga River and its tributaries were gathered during field inspections and were tied in with the photogrammetry data in HEC-RAS.

4.6 Blocked Obstructions

In the more highly urbanised areas, especially in close proximity to the channel, blocked obstructions were utilised in HEC-RAS to account for buildings on the floodplain. The blocked obstructions were positioned using the Ineffective Flow Area theme in ArcView. The ineffective flow areas were then converted to blocked obstructions in the HEC-RAS model, as blocked obstructions better represent the presence of buildings and add wetted perimeter to the calculation set.

A comparison of models with and without blocked obstructions was undertaken in Hahndorf. The inclusion of blocked obstructions in the model resulted in the prediction of water surface elevations up to 400mm higher for the 500 year Annual Recurrence Interval (ARI) flood. From this it can be concluded that modelling buildings as blocked obstructions in highly developed areas is an important consideration since there is the potential for the water surface profile to be altered significantly for the larger ARI floods. Areas where blocked obstructions were utilised due to the higher development density included Hahndorf, Aldgate, Lobethal and Stirling.

4.7 Model Validation

Model calibration is ideally carried out using known flood levels together with a known flood flow. In the case of the watercourses examined for this Study, reliable data on which to calibrate the various models were not available.

Validation of the model was based on:

- A review of the floodplain maps by members of the Steering Committee, based on their knowledge and experience of flooding within the study area
- A review of the floodplain maps by senior members of the study team based on their knowledge and experience of flooding within the study area
- A review of maps showing the indicative extent of inundation along the Onkaparinga River in the August 1992 floods.

The review of the August 1992 flood data is discussed in more detail below.

4.7.1 Review of August 1992 Flood Data

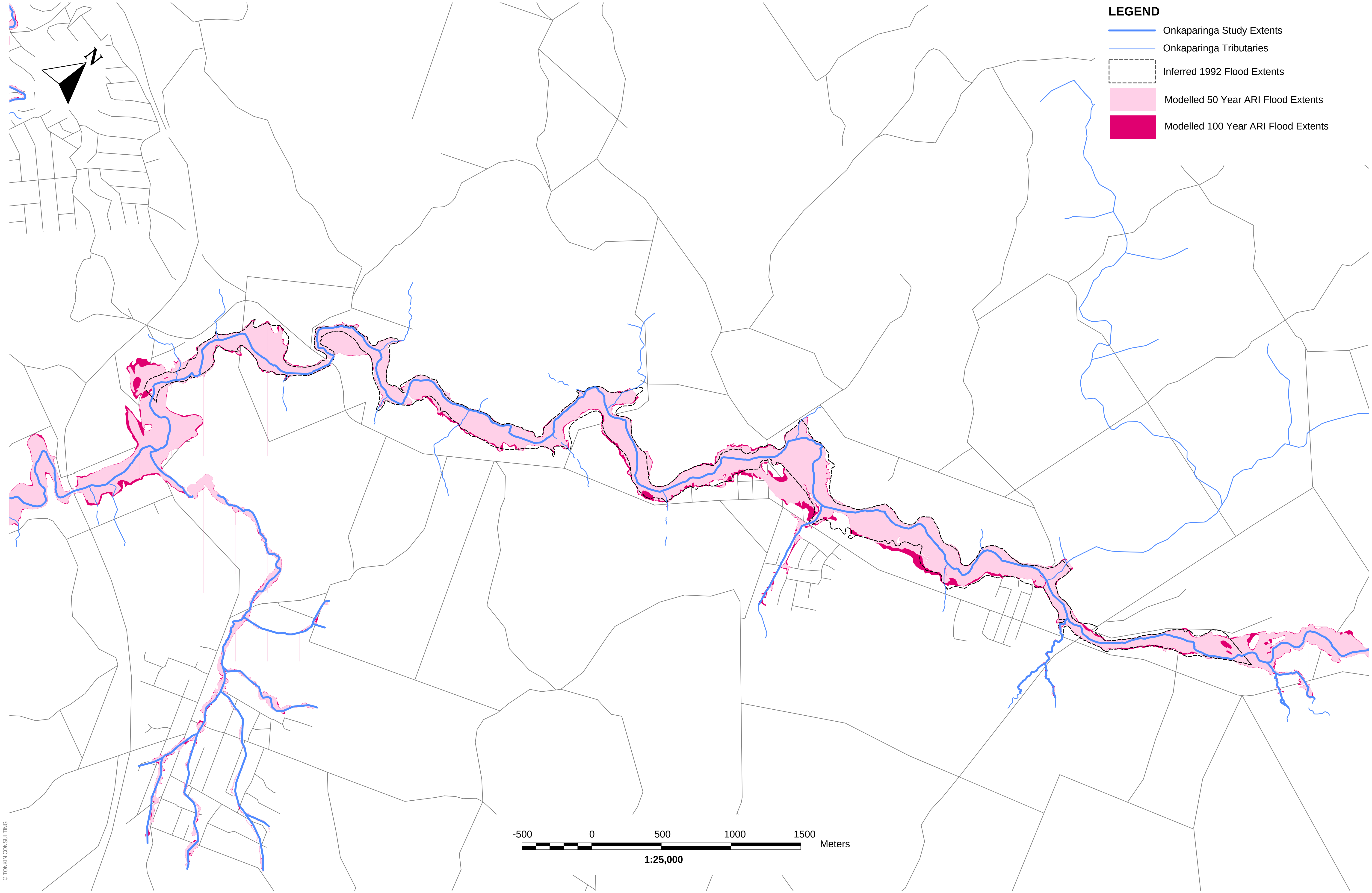
Following the flooding that occurred along the Onkaparinga River in August 1992, the Bureau of Meteorology collected data to identify the approximate extent of flooding. A map was produced using the field data that was collected showing the inferred extent of inundation. This map is shown in Figure 4.2.

At the time of the 1992 floods, the only flow gauge on the Onkaparinga River was situated at Houlgraves Weir. This gauge is outside the study area and is well downstream of the area in which the Bureau collected their flood inundation data. Peak flow data within the study area have therefore been derived from hydrological modelling carried out by Transport SA. This data is presented in Table 4.1.

Table 4.1 Peak Flows at Various Locations (Onkaparinga River)

Location	August 1992 Peak Flow (m^3/s)	50 year ARI Peak Flow (m^3/s)	100 year ARI Peak Flow (m^3/s)
Woodside	92	78	98
Oakbank	296	291	358
Verdun (u/s Hahndorf Creek)	350	369	449

Examination of the data in Table 4.1 indicates that in the vicinity of Woodside, the flood was close to a 100 year ARI event, while further downstream the peak corresponded more closely to a 50 year ARI event.



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MAP DETAILS

Flood Mapping:	Tonkin Consulting
Road Data:	MapInfo Australia
River Data:	Onkaparinga Catchment Water Management Board
Job Number:	2002.0041
Filename:	flood_comparison.wor
Drawn:	Tricia de Vink
Date:	25/02/2003

Onkaparinga Cathcment Water Management Board

Upper Onkaparinga River
Floodplain Mapping

HYDRAULIC MODEL VALIDATION

Figure 4.2

Comparison of the predicted flood extents for the 50 and 100 year ARI events with the extent of inundation produced by the Bureau shows that the predicted extents of inundation are close to those observed during the flooding (refer to Figure 4.2).

4.8 Model Details

4.8.1 Onkaparinga River (Mylor to Charleston)

The main Onkaparinga River was divided into four separate HEC-RAS models:

- Mylor to Verdun
- Verdun to Balhannah
- Balhannah to Woodside
- Woodside to Charleston.

It was determined that field survey was not required to supplement the photogrammetry along the Onkaparinga River. This was based on the fact that it was possible to cut cross-sections using the photogrammetry at sufficiently close spacings through gaps in the tree coverage.

4.8.2 Aldgate and Leslie Creeks (Aldgate, Stirling and Mylor)

Aldgate Creek was divided into four separate models:

- Mylor to Somerset Road, Aldgate (which includes Leslie Creek)
- Somerset Road, Aldgate to Kemp Road, Aldgate
- Kemp Road, Aldgate to Stirling East
- Euston Road, Aldgate to Stirling.

The Stirling area has many houses located near the various creek lines. The buildings located within the floodplain were modelled using blocked obstructions.

From the aerial photography, it was evident that there was a dense tree coverage along a considerable length of Aldgate Creek in the Stirling and Aldgate areas. From a closer review of the photogrammetric data in conjunction with site inspections, it was concluded that a field survey would be required to accurately represent the creek profile and invert and the surrounding floodplains.

Some field data was available from previous studies by Kinhill Engineers (Flood Study of Stirling Area, June 1993) and C.J. Ciccocioppo (Aldgate Creek Flood Study, June 1994). The field data from these reports covered the main areas of interest in Aldgate and Stirling. However, there were a few reaches in the model that were not included and a field survey was therefore required to define the channel in these areas. The extent of the additional field survey and the surveys covered by the Kinhill Engineers and C.J. Ciccocioppo reports are presented in Figure 4.3.

Another area of interest in the Aldgate Creek model was the Snows Road Dam. Detailed plans of the dam were obtained from Transport SA (TSA) and these provided embankment levels, spillway levels, weir elevation levels and ground elevation levels, which were incorporated into the model. For modelling purposes, it was assumed that the dam was holding water at the start of the simulation.

Downstream of the dam, at the Aldgate shops located on the corner of Mount Barker and Euston Roads, the 50, 100, 200 and 500 year ARI flood flows were found to break out of the channel upstream of Theodore Lane. This resulted in a split flow, where the majority of the flow continued down the channel with the remainder diverted away from the channel along Mount Barker Road.

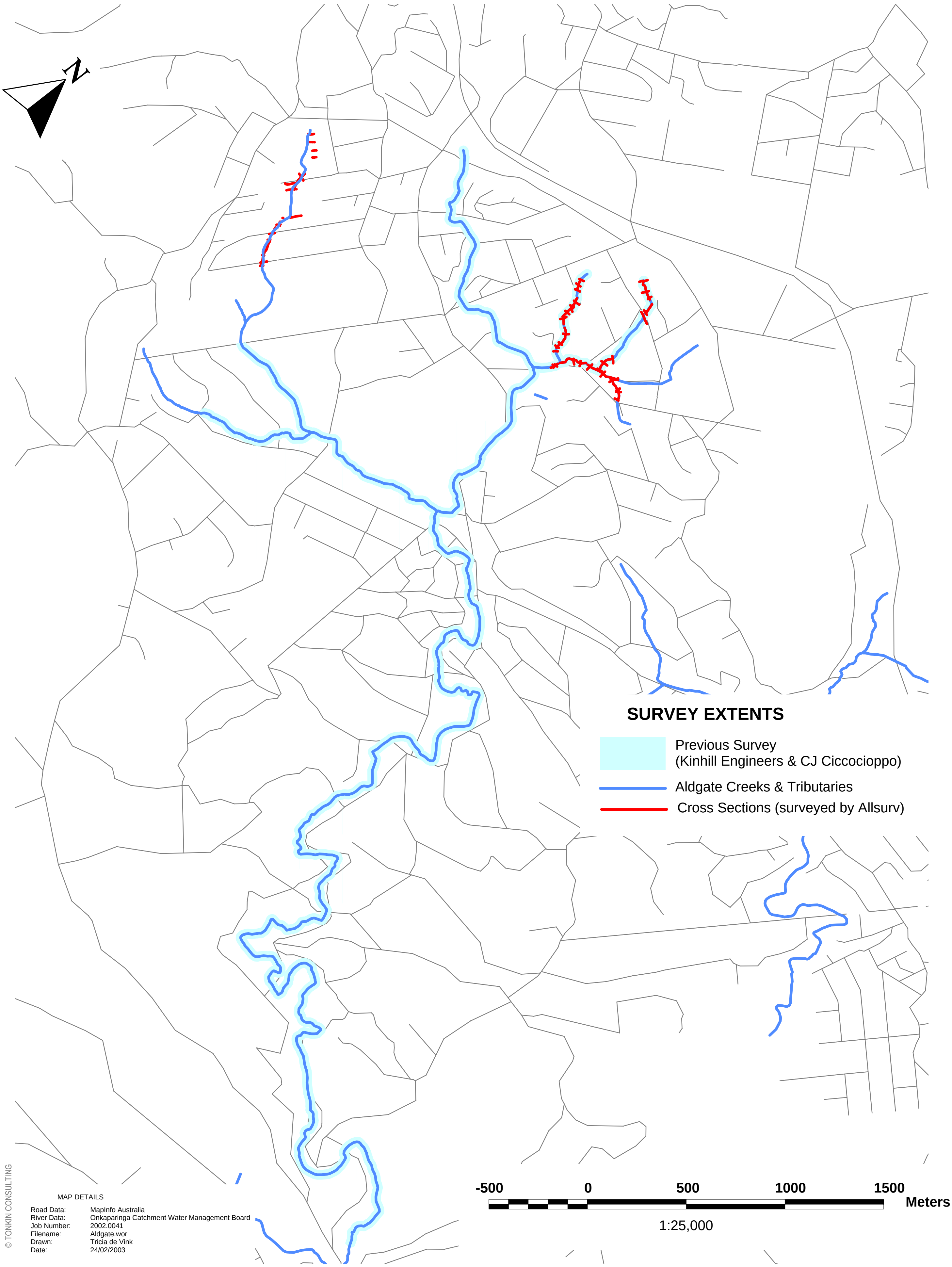
The point at which the flows split, between Theodore Lane and Euston Road, was modelled as a lateral weir in HEC-RAS. The flow continuing down the road was quantified and modelled in a separate HEC-RAS model of the area between the shops and the northern side of the road.

The open channel behind the shops connects into two box culverts that direct the flow under the car park adjacent to Kingsland Road. The roadway weir flow path between the chicken shop and the shops fronting the main street was also modelled as a lateral weir. The flows that spill over the car park at this point and the flows from Mount Barker Road were added back into the main channel approximately 80 metres downstream of Kingsland Road.

The resultant water levels derived in the main street adjacent to the Aldgate Hotel were corroborated by anecdotal evidence provided by Adelaide Hills Council, giving some confidence that the calculated flow splits were providing a reasonable depiction of the real behaviour of the system.

4.8.3 Cox Creek (Bridgewater)

Cox Creek and its tributary streams are crossed by a railway embankment. The embankment acts as a hydraulic control and results in the ‘heading up’ of waters upstream of the railway line. The 500 year ARI flood and the PMF have been estimated to overtop the eastern railway bridge (Bridge #128). Floodwaters that overtop the railway would tend to travel in an easterly direction, returning to the main creek line further downstream. The direction of flow for the 500 year ARI flood and the PMF are depicted on the Flood Inundation Maps A3.2.24 and A3.3.24 respectively.



MAP DETAILS

Road Data:	MapInfo Australia
River Data:	Onkaparinga Catchment Water Management Board
Job Number:	2002.0041
Filename:	Aldgate.wor
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Date:	24/02/2003

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Upper Onkaparinga River

Floodplain Mapping

GROUND SURVEY DATA

ALDGATE - STIRLING

Figure 4.3

The western railway bridge (Bridge #96) is overtopped by the PMF. Flood waters would pond upstream of the bridge and ultimately flow in an easterly direction along the railway, returning to the creek further downstream. The pattern of flows is shown on Flood Inundation Map A3.3.24.

4.8.4 Hahndorf Creek (Hahndorf)

A considerable length of Hahndorf Creek is under dense tree coverage. As a result, field survey was undertaken to obtain accurate creek cross-sections and floodplain levels. The areas in which field survey was undertaken are shown in Figure 4.4.

Additional ground survey data were obtained from a previous Study of the area (Hahndorf Township Watercourse Management Study, BC Tonkin & Associates, 1992). The data from this investigation covered areas in the vicinity of Strempel Avenue, English Street and Bernhardt Crescent. Culvert sizes from a number of driveways crossings were used in the development of the hydraulic model.

The creek floodplain within Hahndorf is highly developed. Buildings within the floodplain form a significant obstruction to flows. As a result, in order to properly model flood elevations, the impact of the buildings was modelled using blocked obstructions.

4.8.5 Junction Creek (Balhannah)

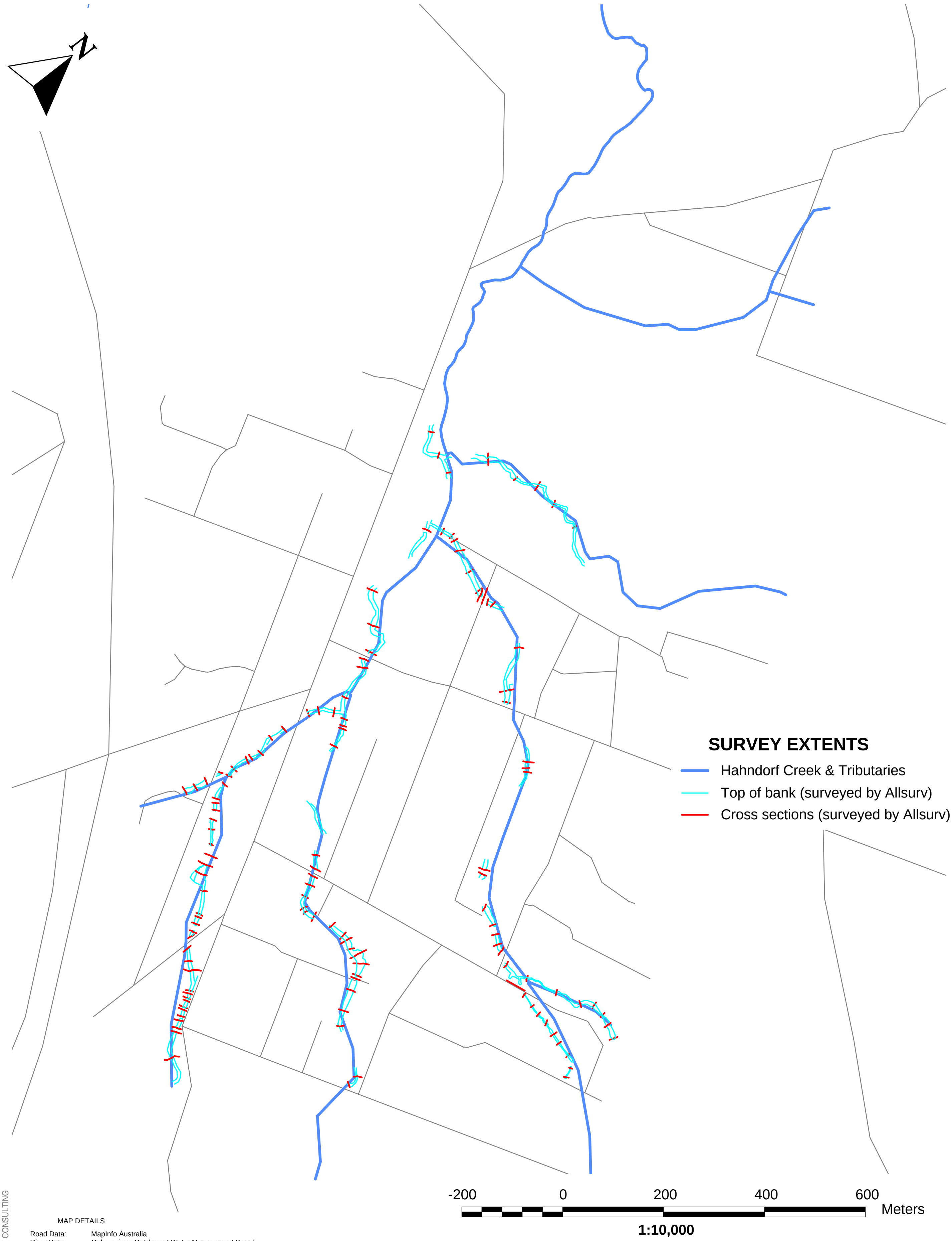
A levee has been constructed at the confluence between Junction Creek and the Onkaparinga River to protect the area to the west of Junction Creek from flooding. The levee runs approximately 400 metres along the southern side of the Onkaparinga River parallel with Bridge Street and approximately 200 metres along the western side of Junction Creek.

The current modelling shows that the levee would be overtopped in the area to the west of the confluence with the Onkaparinga River in a 50 year ARI event.

The flood levels produced by the current modelling are between 500 and 800 mm higher than the levels for design of the levee. The higher levels may be attributed to the increased vegetation cover within the main channel, the coarser section spacing within the current modelling and the lack of data on the true river invert produced by the photogrammetry.

4.8.6 Echunga

The study area covers three creeks which run through the township of Echunga. The northern most creek flows from the north-east and passes over the Mylor-Echunga road.



MAP DETAILS

Road Data:	MapInfo Australia
River Data:	Onkaparinga Catchment Water Management Board
Job Number:	2002.0041
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Onkaparinga Catchment Water Management Board

Upper Onkaparinga River Floodplain Mapping

GROUND SURVEY DATA

HAHNDORF

Figure 4.4

The central creek drains towards West Street where it enters a piped system which runs approximately 200m before it discharges into a dam and then paddocks downstream of the township. Modelling of the pipe system indicates it has approximately a 100 year ARI capacity. Design floods which exceed the 100 year ARI flow will break over West Terrace and flood through a group of houses between West Terrace and Adelaide Road.

Flooding along the southern most creek would not impact any buildings within the town.

4.8.7 Lobethal

Lobethal comprises two branches which converge upon each other in the middle of the town. To provide higher definition of the creek, survey cross sections were undertaken in the location shown in Figure 4.5.

The hydraulic modelling of the creeks revealed the following:

- culverts under the former woollen mills have a 200 year ARI capacity
- flood flows above the 200 year ARI event join at Bridge Street
- flood flows spill from the eastern branch to the western branch at Pioneer Avenue for events between a 10 year and a 500 year ARI (flood maps identify the flood spill rates).
- flood flows merge at Pioneer Avenue for the PMF flood event.

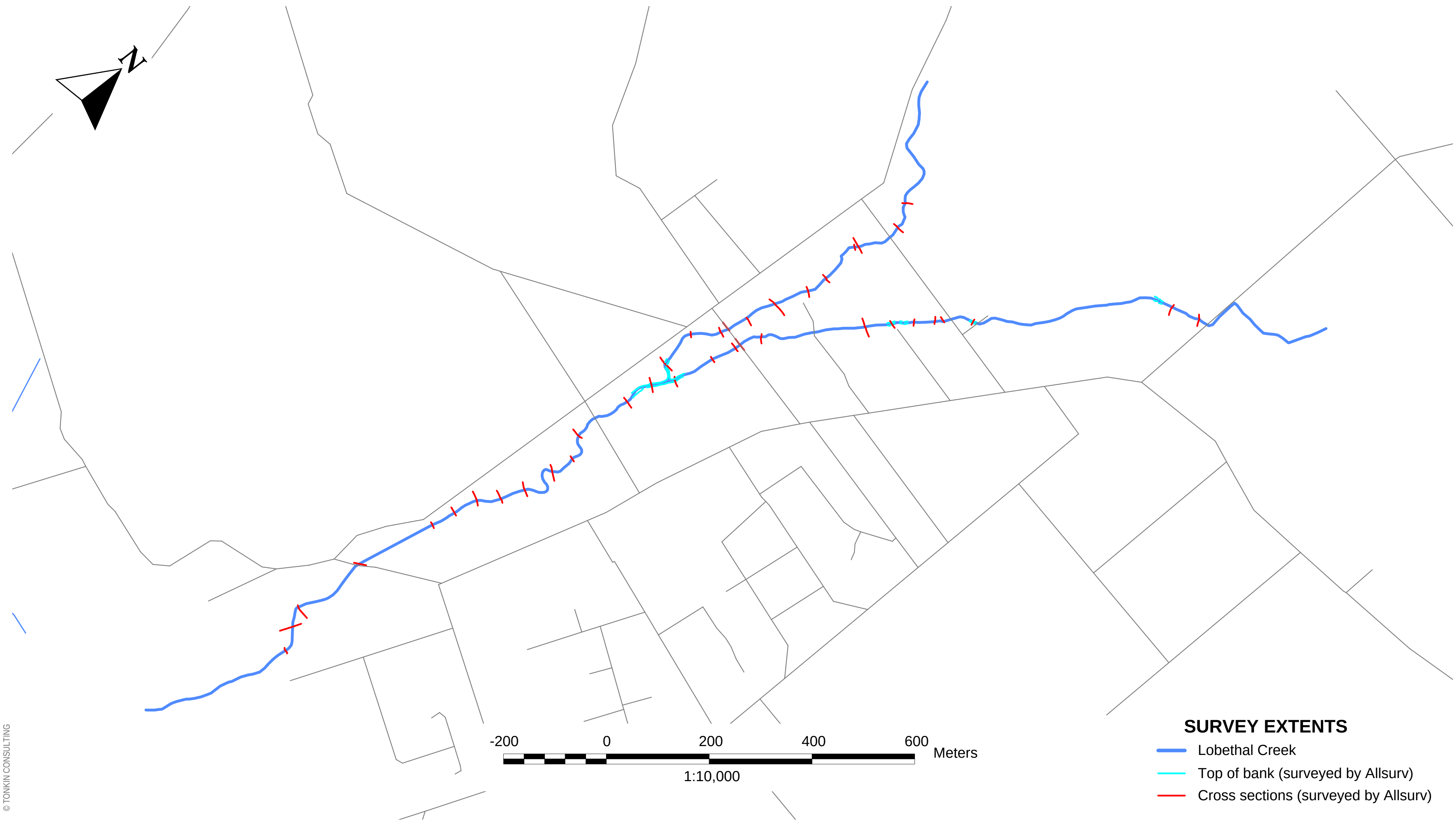
4.8.8 Lenswood

The section of creek through Lenswood includes two road bridges but otherwise does not contain any hydraulic features of particular note.

4.8.9 Kangarilla

Dashwood Gully Creek comprises the main portion of the model through Kangarilla. The lower reaches of the creek have flat floodplain areas with gently undulating terrain.

The upstream end of the model includes a small creek which passes beneath Cut Hill Road. The Cut Hill Road crossing is very old and small. Floods, exceeding approximately a 10 year ARI event, will break out of the channel and spill over the road and travel westward as shallow sheet flow. For planning purposes, the floodplain maps show the magnitude of the lateral spilling flows.



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MAP DETAILS

Road Data:	MapInfo Australia
River Data:	Onkaparinga Catchment Water Management Board
Job Number:	2002.0041
Filename:	Lobethal.wor
Drawn:	Tricia de Vink
Date:	24/02/2003

Onkaparinga Catchment Water Management Board

Upper Onkaparinga River Floodplain Mapping

GROUND SURVEY DATA
LOBETHAL

Figure 4.5

4.9 Hazard Mapping

Hazard Maps of the floodplain are generally useful to local planning agencies and emergency services agencies. Hazard maps can be used to provide an initial assessment of flood risk when considering new development and in developing flood management and evacuation plans for an area.

The hazard mapping has been carried out in accordance with the CSIRO publication, Floodplain Management in Australia (SCARM Report 73).

Factors that affect the hazard caused by a flood can be grouped into four broad categories:

- flood behaviour (ie severity, depth, velocity, rate of rise, duration)
- topography (ie evacuation routes, islands)
- population at risk (ie no. of people, land use, flood awareness)
- emergency management (ie flood forecasting, flood warning, evacuation plans, recovery plans).

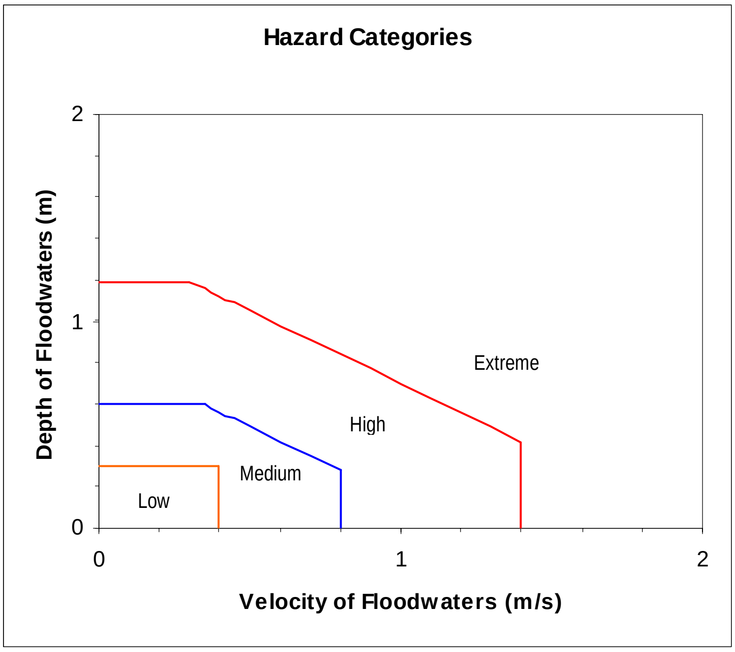
The flood hazard mapping has only considered the flood behaviour, principally flood depth and velocity to determine the degree of hazard.

The four degrees of hazard considered were:

- Low Hazard
- Medium Hazard
- High Hazard
- Extreme Hazard

Figure 4.6 illustrates the criteria for establishing the hazard category.

Figure 4.6 Hazard Categories



A description of the hazard categories is included below:

Low Hazard

If necessary, children and elderly people could wade to safety with little difficulty; maximum flood depths and velocities along evacuation routes are low. Evacuation is possible by a sedan-type motor vehicle, even a small vehicle.

Medium Hazard

Fit adults can wade safety, but children and elderly may have difficulty; maximum flood depths and velocities are greater. Evacuation by sedan-type vehicles may be possible, however 4WD vehicles or trucks should be used.

High Hazard

Fit adults have difficulty in wading to safety; maximum flood depths and velocities are greater (up to 1.0m and 1.5 m/s respectively). Motor vehicle evacuation may be possible by 4WD vehicles or trucks. Boats or helicopters may be required.

Extreme Hazard

Boats and helicopters are required for evacuation; wading is not an option because of the depth and velocity of floodwaters. Maximum flood depths and velocities are over 1.0 m and over 1.5 m/s respectively.

4.9.1 Hazard Map Calculations

In order to generate the hazard maps, velocity and water surface elevations were extracted from the floodplain modelling results. The velocity and flood depths were used to generate a range of 'pseudo flood levels' which represent the boundary lines between low and medium, medium and high and high and extreme hazard zones.

Using HEC-RAS and HecGeoRas, these pseudo flood levels were remapped to generate the four hazard zones within the floodplain.

Flood elevation contours for the recurrence interval of interest were generated within GIS for the water surface modelled and displayed on the Hazard maps.

4.10 Flood Risk Assessment

An assessment was carried out to identify buildings in the study area that may be at risk of inundation for a 100 year ARI flood event. This will enable future studies to be conducted in these areas to identify the potential flood risk and the associated flood damages estimate.

The 100 year ARI flood inundation maps were reviewed to determine the number of buildings located within the flood extents. Table 4.2 lists the number of buildings (includes dwellings, sheds, public, commercial, and industrial buildings) that are potentially at risk of flooding for specific locations within the study area. It is evident that the main areas of concern include Balhannah, Hahndorf and Aldgate with 68, 24 and 17 buildings affected respectively. More detailed assessment of these areas should be undertaken to assess the actual flood risk, the associated flood damages and possible mitigation strategies.

Table 4.2 Number of buildings within the 100 year floodplain

River	Location	No of Buildings Affected
Lobethal Creek	Lobethal	3
Onkaparinga River	Oakbank	8
Junction Creek	Balhannah	6
Onkaparinga River	Balhannah	68
Onkaparinga River	Verdun	10
Hahndorf Creek	Hahndorf	24
Onkaparinga River	Mylor	4
Aldgate Creek	Mylor	5
Aldgate Creek	Aldgate	17
Aldgate Creek	Stirling	3
Cox Creek	Bridgewater	1
Dashwood Gully Creek	Kangarilla	7

4.11 Flow - Stage Relationships at Gauging Station Locations

The flow versus stage relationship from the hydraulic model at the Aldgate, Verdun, Oakbank, Woodside and Charleston Gauging station locations are shown in Figure 4.7, Figure 4.8, Figure 4.9, Figure 4.10 and Figure 4.11, respectively. Rating tables for these stations are included in Appendix A.

Figure 4.7 Aldgate Gauging Station (AW503509)

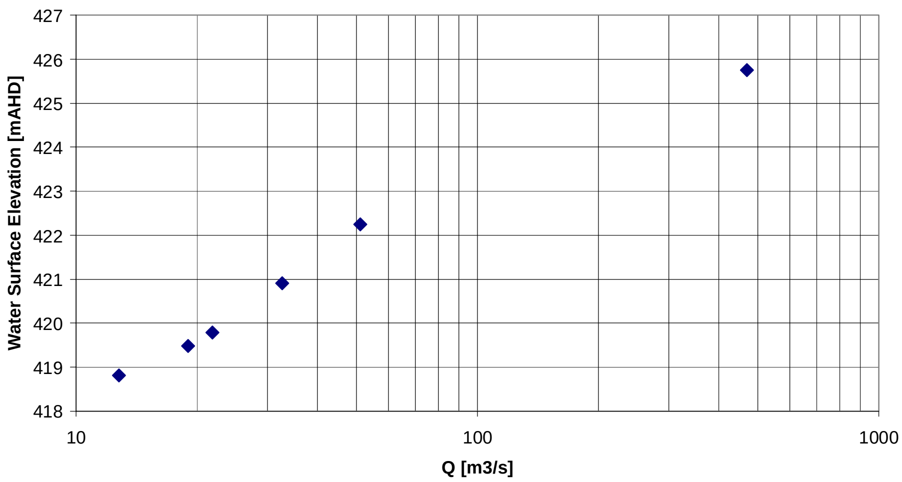


Figure 4.8 Verdun Gauging Station (midway between freeway bridges)

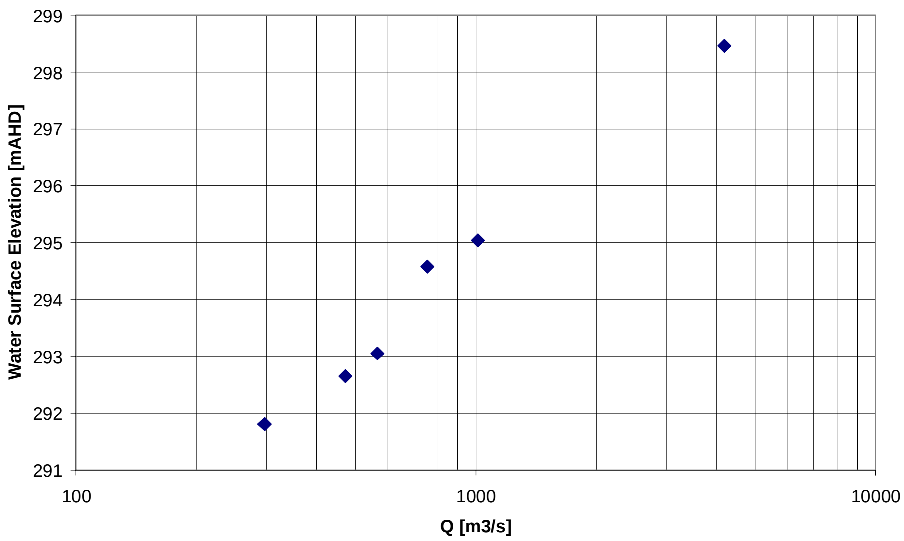


Figure 4.9 Oakbank Gauging Station (AW503902)

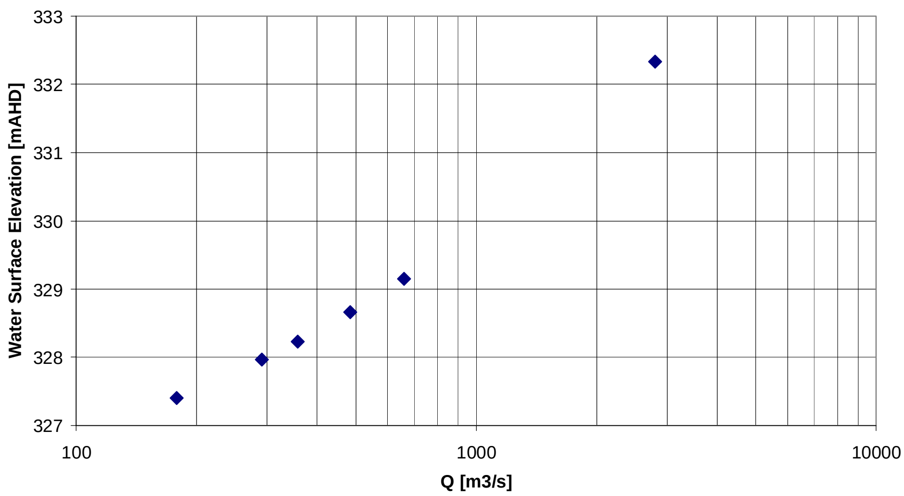


Figure 4.10 Woodside Weir Gauging Station (AW503903)

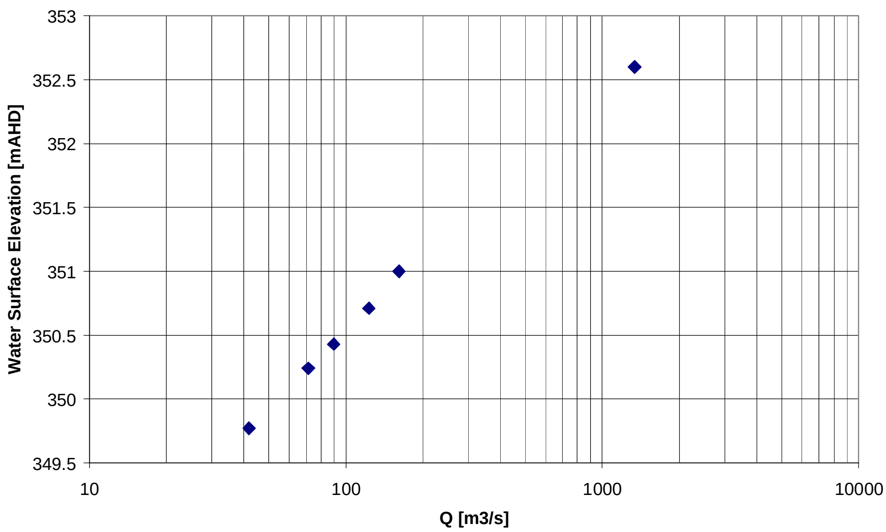
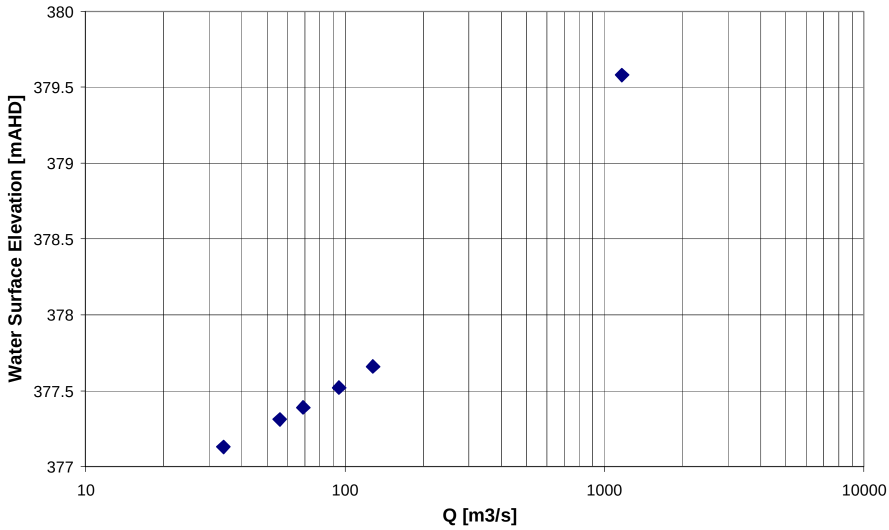


Figure 4.11 Charleston Gauging Station (Graebner Rd)



5. Study Outputs

5.1 Maps

Map production has been carried out using GIS tools with the finished maps being produced using ESRI, ArcGIS.

5.1.1 Map Colours

Colours used for the floodplain mapping were selected in consultation with representatives of the Steering Committee. The selected colours have been determined to enable printing of the maps in black and white if necessary.

5.1.2 Map Sheets

224 colour A3 maps have been produced as part of this Study. There are 32 maps providing coverage of the study area. For each map sheet there are 7 flood series maps as follows:

1 10/50 year ARI flood inundation
2 100/200/500 year ARI flood inundation
3 PMF flood inundation
4 50 year ARI hazard map
5 100 year ARI hazard map
6 500 year ARI hazard map
7 PMF hazard map.

5.2 CD ROM

Two CD ROMs have been produced that contain the Study outputs. The contents of the CDs are as follows:

CD1

- PDF outputs for the complete set of floodplain maps. The file _KEY.pdf shows the location of each map sheet.
- MapInfo GIS files containing the floodplain map data in Mapinfo format. The file _Readme.txt in the MapInfo GIS Files directory contains a list of files and their contents
- ArcView GIS files containing the floodplain map data in Arcview format. The file _Readme.txt in the Arcview GIS Files directory contains a list of the files and their contents

- Photographs taken at the various bridges. The photographs are referenced by number on the floodplain maps.

CD 2

- Digital ortho-photos for the study area (in .ecw format)
- Raw DTM data (including xyz point coordinates and breaklines)

Appendix A

Gauging Station Rating Tables

Upper Onkaparinga Gauging Station - Rating Table - Flow (Q) Vs Water Surface Elevation (WS Elev)

Event	**Aldgate**		**Verdun**		**Oakbank**		**Woodside**		**Charleston**	
AEP	Q (m3/s)	WS Elev (m AHD)	Q (m3/s)	WS Elev (m AHD)	Q (m3/s)	WS Elev (m AHD)	Q (m3/s)	WS Elev (m AHD)	Q (m3/s)	WS Elev (m AHD)
10	13	418.8	296	291.8	178	327.4	42	349.8	34	377.1
50	19	419.5	471	292.7	291	328.0	72	350.2	56	377.3
100	22	419.8	567	293.1	358	328.2	90	350.4	69	377.4
200	33	420.9	756	294.6	485	328.7	123	350.7	95	377.5
500	51	422.3	1012	295.0	660	329.2	161	351.0	128	377.7
PMF	470	425.8	4183	298.5	2800	332.3	1339	352.6	1170	379.6

Appendix B

Upper Onkaparinga River Hydrology Report

TRANSPORT SA

TRANSPORT SA

HYDROLOGICAL STUDY OF THE UPPER ONKAPARINGA RIVER CATCHMENT

December 2003

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Urban Planning and the Arts

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CONTENTS

1 INTRODUCTION

This report contains the results of hydrological modelling of the upper Onkaparinga River catchment. The modelling has been carried out using a new model, the RRR model. This model is calibrated separately at all the gauging stations within the catchment, and then used to predict flows for a wide range of flood frequencies, up to the Probable Maximum Flood (PMF).

The results of the model are compared with the long term record of historical flows at Mount Bold reservoir, and the results of an extension of this record by continuous simulation of flow over a very long period.

The results of this modelling are the best available at this point, but the accuracy of the predictions will improve as more data becomes available from stations within the catchment. There would also be benefits in the installation of more gauging stations, to improve the definition of the model in some locations.

2 CONTINUOUS SIMULATION

The continuous simulation system for design flood estimation, developed by Dr Walter Boughton was used at the Houlgraves station (AW503504) to determine flood frequency. A full description of the continuous simulation method is given in Boughton (1999), and the program and documentation is available from the CRC for Catchment Hydrology. The continuous simulation system consists of:

- A catchment water balance model (AWBM) for continuous simulation of losses which produces rainfall excess at hourly intervals,
- A flood hydrograph model (WBMOD) for converting rainfall excess to hydrographs of hourly flow at the catchment outlet,
- A data generation model for generating 2000 year sequences of daily rainfalls, and
- A model for disaggregating the generated daily rainfalls to hourly values, which are then converted to hydrographs, and thus peak flows and a flood frequency for the 2000 year period.

Several (normally 15) 2000 year periods are synthesised, and the flood frequency determined.

2.1 AWBM

The AWBM is a catchment water balance model that can relate runoff to rainfall with daily or hourly data, and calculate losses from rainfall for flood hydrograph modelling.

The model uses three surface stores to simulate partial areas of runoff. The water balance of each surface store is calculated independently of the others. The model calculates the moisture balance of each partial area at either daily or hourly time steps. At each time step, rainfall is added to each of the 3 surface moisture stores and evapotranspiration is subtracted from each store. The water balance equation is:

$$store_n = store_n + rain - evap \quad (n = 1 \text{ to } 3)$$

If the value of moisture in the store becomes negative, it is reset to zero. If the value of moisture in the store exceeds the capacity of the store, the moisture in excess of capacity becomes runoff and the store is reset to the capacity.

When runoff occurs from any store, part of the runoff becomes recharge of the baseflow store if there is baseflow in the streamflow. The fraction of the runoff used to recharge the baseflow store is BFI*runoff, where BFI is the baseflow index, ie. the ratio of baseflow to total flow in the streamflow. The remainder of the runoff, ie. (1.0 - BFI)*runoff, is surface runoff. The baseflow store is depleted at the rate of (1.0 - K)*BS where BS is the current moisture in the baseflow store and K is the baseflow recession constant of the time step being used (daily or hourly).

The surface runoff can be routed through a store if required to simulate the delay of surface runoff reaching the outlet of a medium to large catchment. The surface store acts in the same way as the baseflow store, and is depleted at the rate of (1.0 - KS)*SS, where SS is the current moisture in the surface runoff store and KS is the surface runoff recession constant of the time step being used.

The AWBM model is primarily a water balance runoff model, and only in a rudimentary fashion deals with the routing of flows to the catchment outlet.

2.2 Data Input

The data needed to calibrate the AWBM continuous simulation loss model are:

- A file of daily rainfalls
- A file of daily streamflows
- A file of monthly streamflows
- A file of evapotranspiration values

These files must all cover exactly the same period of time, and be in multiples of whole calendar years (1 January to 31 December).

The data needed to calibrate the WBMOD flood hydrograph model are:

- A file of hourly rainfalls for 1 to 10 flood events, each 5 days of 24 hours in length

- A file of hourly streamflows corresponding exactly with the hourly rainfall file

The data needed to calibrate the daily rainfall generation model are:

- A file of daily rainfalls, usually containing a longer period than the file of daily rainfalls used to calibrate the AWBM model
- Estimates of annual maxima daily rainfalls of large ARI, such as the CRC-FORGE estimates for ARIs 50 to 2000 years

The data needed for the disaggregation of daily to hourly rainfalls are the intensity-frequency-duration information from Australian Rainfall and Runoff that are normally used in rainfall based design flood estimation.

The generation program uses the same daily rainfall file that is used to calibrate the daily rainfall generation model and the same evaporation file that is used in the calibration of the AWBM model.

The AWBM model was calibrated on 13 years of data, from 1986 to 1998. The decision on the years to be used was made on the basis of data availability.

The daily rainfall data for the calibration of the AWBM model was obtained from the Department for Water Resources (DWR). Rainfalls were obtained for three stations (Inverbrackie Creek, Mount Lofty and Echunga Creek), and a Theissen mean daily rainfall determined.

The daily runoff data was obtained from the Department for Water Resources. Because of the pumping from the Murray River into the Onkaparinga the raw data at Houlgraves does not represent true catchment runoff. The data supplied had the pumped flows extracted.

Evaporation data was obtained from the Bureau of Meteorology. Data for the Lenswood research station for the period modelled by AWBM (1986 – 1998) was used. For the continuous simulation, estimates of evapotranspiration were required. The average annual pan evaporation for the period 1986 – 1998 was 1241mm, and the potential areal evapotranspiration is 1100mm (BOM, 2001), so the pan evaporation values were adjusted by a factor of 0.88 for calibration of the AWBM model.

A total of nine runoff events were used to calibrate the WBMOD flood hydrograph model. The same rainfall stations were used as for the daily rainfall. Runoff events lasting five days with the following start dates were used:

14/07/1987
22/05/1988
17/08/1990
28/08/1992
12/09/1992
06/10/1992
16/12/1992
01/08/1996

26/09/1996

The Australian Rainfall & Runoff intensity-frequency-duration information was determined for the catchment as follows:

2 year, 1 hour intensity	18.0mm/hour
2 year, 12 hour intensity	4.3mm/hour
2 year, 72 hour intensity	1.2mm/hour
50 year, 1 hour intensity	34.9mm/hour
50 year, 12 hour intensity	7.0mm/hour
50 year, 72 hour intensity	2.0mm/hour

Average Regional Skewness	0.55

Geographical Factor F2	4.47
Geographical Factor F50	14.95

CRC Forge 24 hour design rainfalls were obtained from the report on the application of the CRC forge method to South Australia (Sinclair Knight Mertz, 2000). Data for the Mount Bold Reservoir catchment was used as a basis, with an adjustment for catchment area. The catchment area of Mount Bold is $384km^2$, and Houlgraves is $321km^2$.

The rainfalls are given in Table 1.

Table 1 CRC Forge 24 Hour Rainfalls for Houlgraves

ARI (years)	AEP	Mt Bold (mm)	Houlgraves (mm)
50	0.02	105	105.6
100	0.01	123	123.6
200	0.005	145	145.7
500	0.002	181	181.9
1000	0.001	215	216
2000	0.0005	255	256.2

Long term daily rainfall data was also obtained from the Department for Water Resources. 107 years of record were available. The data was produced for the CRC – FORGE investigation (Sinclair Knight Mertz, 2000). The record at each station had been verified and missing data filled in by reference to adjacent stations. Theissen mean daily rainfalls were determined for the catchment to Houlgraves using the following stations:

023707	Bridgewater
023713	Echunga
023720	Hahndorf
023726	Lobethal
023750	Uraidla
023829	Woodside

The short term daily rainfalls for the calibration of the AWBM were adjusted so that there was consistency with the long term data. The short period (1986 – 1998) average annual rainfall was 787.5mm, compared with the long term rainfall data for the same period of 858.9mm. The short term rainfalls (daily and hourly) were multiplied by 1.091 to ensure consistency with the long term data.

2.3 Results

Once all the data files were assembled the recommended procedure was followed, and flows determined for a range of recurrence intervals up to 500 years. The AWBM model was fitted to the 1986 – 1998 period, and the rainfall generation to the 107 year period. Table 2 gives a summary of the events fitted by the flood hydrograph part of the system (WBMOD), and Figure 1 shows the generated annual rainfall for the Houlgraves catchment compared with the annual rainfall distribution for the 107 years of record.

Table 2 Flow Events Fitted by WBMOD

Event	Event Order		Ranked Order	
	Actual Flow m^3/sec	Calculated Flow m^3/sec	Actual Flow m^3/sec	Calculated Flow m^3/sec
1	151.5	123.4	428.5	239.7
2	158.4	120.3	258.6	237.8
3	22.8	20.6	193.1	180.9
4	428.5	149.2	183.1	149.2
5	96.2	98.2	158.4	123.4
6	193.1	237.8	156.4	120.3
7	183.1	239.7	151.5	98.2
8	156.4	70.8	96.2	70.8
9	258.6	180.9	22.8	20.6

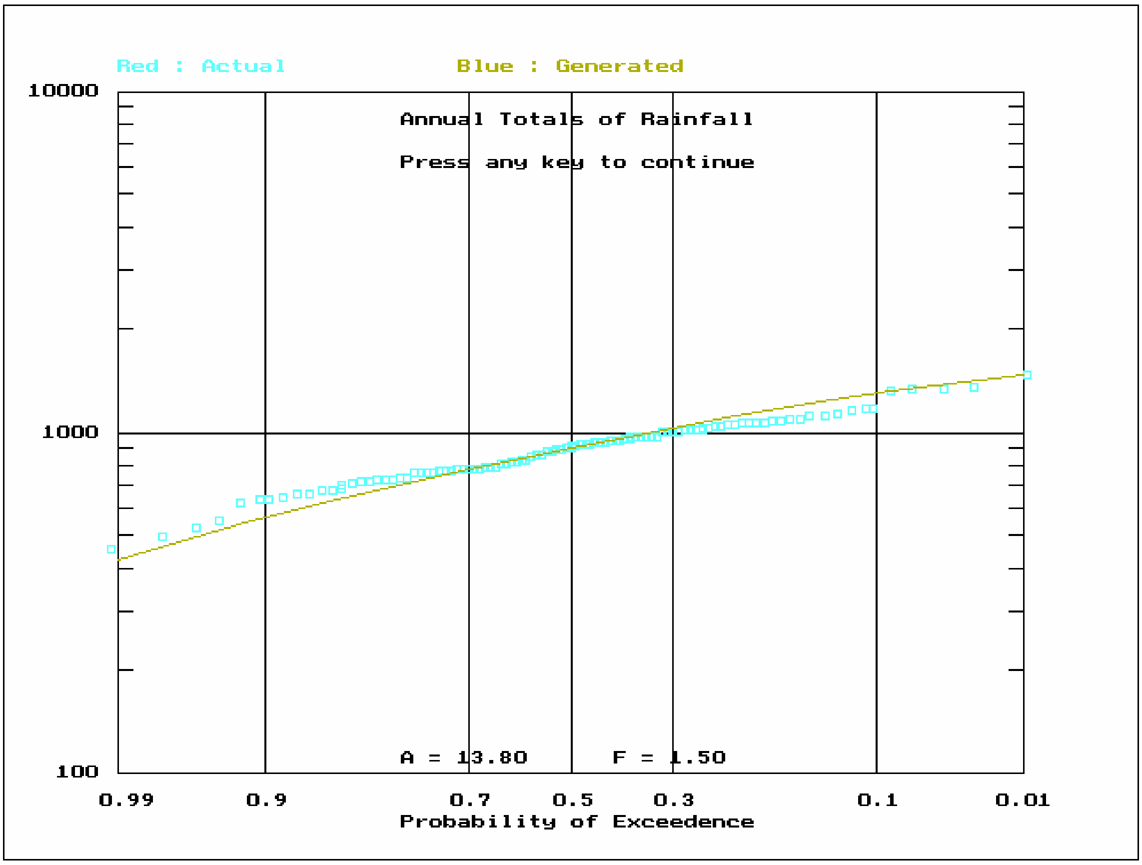

Figure 1 Generated Houlgraves Catchment Annual Rainfall Compared With Recorded Rainfall

The program was then run for 15 sets of 2,000 years of generated rainfall data, and median values of floods of a range of recurrence determined. Table 3 gives the results, together with a comparison to the flows determined by Daniell and Hill, extended by the extra years of record.

Table 3 Results of Continuous Simulation at Houlgraves

Average Recurrence Interval (years)	Daniell & Hill, updated Flow (m^3/sec)	Continuous Simulation Flow (m^3/sec)
2	84	101
5	179	242
10	294	345
20	395	457
50	516	650
100	657	817
200	819	1022
500	1289	1336

The flows predicted by the continuous simulation system are in general higher than the flows predicted by flood frequency analysis. Figure 2 shows a comparison of the flows predicted by the continuous simulation and the log-normal flood frequency at Houlgraves. Discussion with Walter Boughton, the developer of the continuous simulation method indicated that the method is sensitive to the baseflow index, and it was suggested that a higher baseflow index be trailed, to more directly match the flood frequency analysis, particularly for the more frequent events. This was not done, as the continuous simulation was used as a confirmation of the slope of the at-station flood frequency curve, and this has already been confirmed.

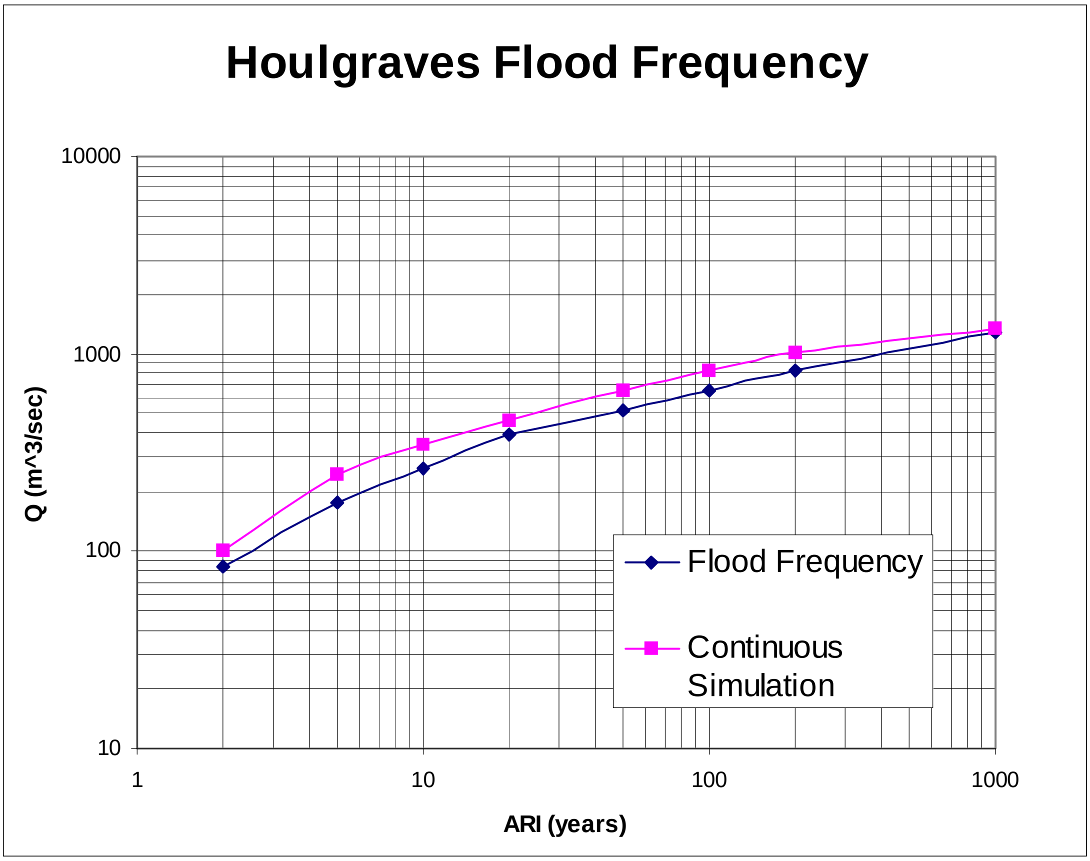

Figure 2 Comparison of Houlgraves Log - Normal and Continuous Simulation Flood Frequency

3 THE RRR MODEL

The RRR model (Kemp and Daniell, 1996, Kemp 2001) has been developed to overcome some of the limitations of previous runoff routing models, whilst maintaining the simplicity of the model by using a series of storages to represent the catchment response. It is able to model both baseflow and surface runoff.

In the case of a catchment having uniform rainfall input there is no need to perform manual catchment sub-division. The channel and hillside or processes are represented separately.

- The model represents the channel storage response by ten equal channel storage elements, each representing a reach length of d/10, where d is the longest flow path length in the catchment (km). It is assumed that the area contributing to each storage element is equal. Channel storage for each channel reach is modelled as a linear storage of the form $S = 3\,600\ k\ Q$;

- Contributions from any number of separate hydrological processes are added at the downstream end of each channel reach before routing through the channel storage. Examples of processes that could occur are baseflow and surface runoff.
- Any runoff processes occurring on the hillslope and contributing to a channel storage element is represented by ten equal storages in series with storage $S = 3\,600\ k_p\ Q^m$, k_p being a lag related to runoff process. The total area of each process storage series is the total catchment area/10,
- Each of the hydrological processes has an initial loss (IL) and a continuing (CL) or proportional loss (PL) associated with it. These losses are each related to the total catchment rainfall.

The use of ten elements for both the process and channel storages follows the Laurenson Runoff Routing Model, and provides for differing elements of rainfall excess to pass through different amounts of storage. The catchment is not however delineated with equal travel times, but with equal areas, as per the RAFTS model.

Laurenson (1964) reported that when using five sub-areas instead of ten less satisfactory results were obtained.

Figure 3 shows diagrammatically the structure of the RRR model. In a single sub-catchment model there is no actual catchment sub-division to be carried out, as must be carried out in the RORB model.

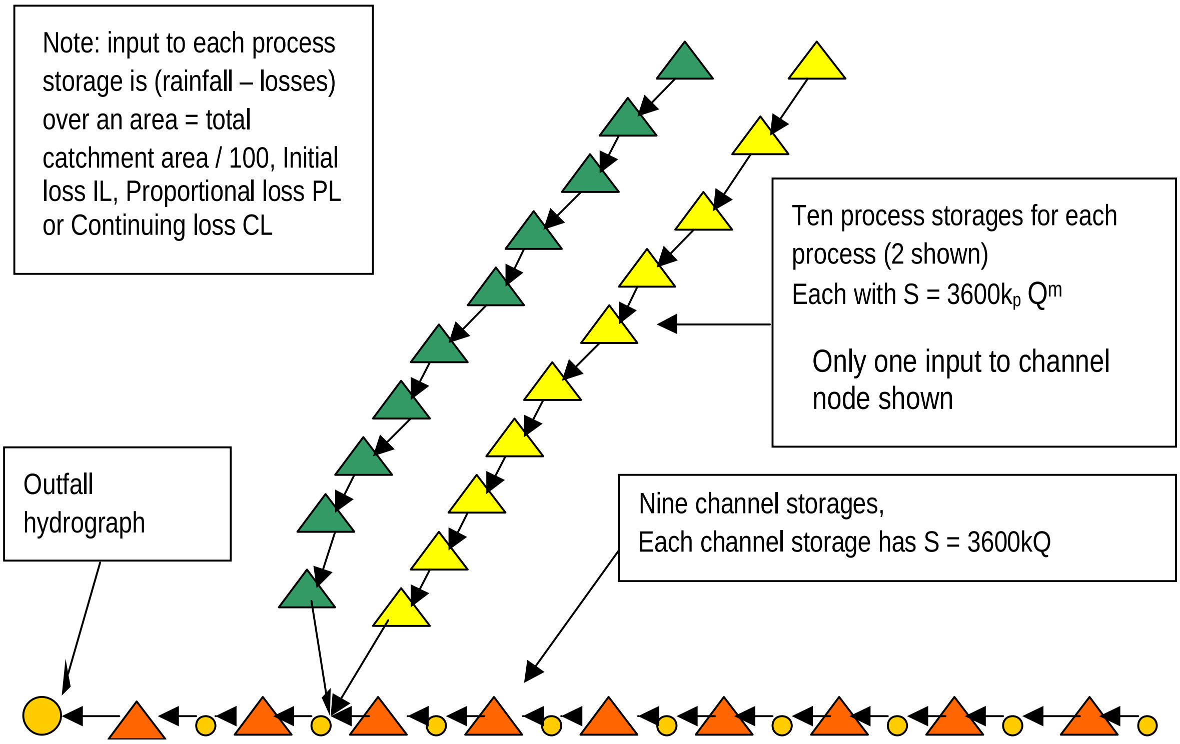

Figure 3 Structure of the RRR Model

Although the model may initially look complicated with 100 storages it is in effect simple as all elements are the same area, and storage parameters and losses need be input only once for the sub-catchment or node.

Evidence gathered during the development of the RRR model suggests that the processes modelled can be separated into three processes. However the boundaries between one process and another may be blurred due to the non-homogeneity of catchment soils and structure.

The three processes and the associated characteristics are as follows:

- Baseflow. This is the traditional concept of baseflow and is related to the steady state regional groundwater system. It is known that the lag between rainfall and runoff by discharge to streams can be substantial, due to the long flow path length in the groundwater system;
- Slow flow, most probably capillary fringe flow. This mechanism acts with a lag from rainfall to stream flow that is less than that of the baseflow above, due to the quicker response time from rainfall to runoff into the stream; and
- Fast flow, most probably similar to Hortonian overland flow, either from a part of the catchment area, or the full catchment area. The response time of this mechanism is short compared with the two above, as no groundwater flow is involved.

The above model structure can be used as one sub-catchment of a total catchment model. This allows the variation across the catchment of rainfall or model loss or storage parameters. To allow the use of the RRR model is this way generalised parameters are needed.

As the channel lag is linear it could be expected that for rural catchments the channel lag will be highly correlated with the mainstream length of the catchment. For the purposes of the derivation of a generalised parameter, a variable representing the characteristic flood wave velocity v_c is introduced. This can be related to channel lag k on the assumption of the ten channel reaches. Equation 1 relates v_c to k, allowing for the number of channel reaches and the conversion of lag time, which is in hours.

$$v_c = \frac{d}{36k} \qquad \text{Equation 1}$$

Where v_c is the channel characteristic flood wave velocity (m/sec)
d is the longest flow path length (km)
k is the channel storage lag parameter (hrs)

However the non-linearity of most process storages creates a problem in that the storage lag depends on the storage outflow, which is in turn dependent on the modelled catchment area.

For this reason a new variable is used, being the catchment characteristic lag parameter, c_p, where:

$$k_p = c_p A^{1-m} \qquad \text{Equation 2}$$

Where A is the catchment or sub-catchment area (km^2)
m is the exponent in the process storage relationship
k_p is the process storage parameter

The reason for the use of this parameter is as follows. The lag of a single process storage is given by the equation:

$$lag = k_p Q^{m-1} \qquad \text{Equation 3}$$

Where Q is the total flow into the channel storages. But it can be seen that the lag of the catchment process storages changes as the area of the modelled catchment changes, as Q is dependent on the area represented by the process storages. If c_p is used the lag is then:

$$\begin{aligned} lag &= c_p A^{1-m} Q^{m-1} \\ &= c_p \left[\frac{Q}{A}\right]^{m-1} \end{aligned} \qquad \text{Equation 4}$$

It can be seen that lag will not now depend on catchment area as Q/A is constant. This constant Q/A follows from the structure of the RRR model, which assumes a constant catchment width, meaning that flow into the channel (Q) is proportional to the channel length and thus the area represented by the series of process storages (A). Since the lag is for a single sub-catchment the effect of rainfall distribution or catchment topography need not be considered.

4 FITTING THE RRR MODEL

4.1 Previous Calibration

The RRR model was fitted to three catchments within the scope of this study as part of the PhD thesis to be presented by Kemp (2001). These catchments were Inverbrackie Creek (AW503508), Scott Creek (AW503502) and Echunga Creek (AW503506). Two adjacent catchments, the Torrens River at Mount Pleasant (AW504512) and Brownhill Creek at Mitcham (AW504901) have also been used for calibration.

For simplicity in the calibrations each process is given a number, starting with baseflow as process 1, and slow flow as process 2. Thus c_p for the first process is designated c_p1, and the proportional loss PL1. The c_p for the second process is c_p2, the initial loss IL2, and the proportional loss PL2.

Full details of the calibration and verification of the model is given in the thesis. For each catchment twelve storm events were selected, and the model fitted on six events. The other events were used for the verification of the model, with mean parameters from the calibration applied to the model. Table 4 and Table 5 give the results of the modelling. It can be seen that the characteristic velocity v_c remains relatively constant for each catchment, but the losses and storage parameters vary.

Table 4 RRR Model Storage Parameters

Location	Station	c_p1	c_p2	v_c (m/sec)
Torrens River	AW504512	0.66	0.21	0.97
Inverbrackie Creek	AW503508	0.77	0.20	0.86
Echunga Creek	AW503506	0.96	0.19	1.14
Scott Creek	AW503502	0.80	0.22	1.20
Brownhill Creek	AW504901	1.72	0.46	1.24

Table 5 RRR Model Loss Parameters

Location	Station	PL1	IL2 (mm)	PL2
Torrens River	AW504512	0.75	11.5	0.28
Inverbrackie Creek	AW503508	0.74	16.9	0.42
Echunga Creek	AW503506	0.89	8.7	0.73
Scott Creek	AW503502	0.78	21.6	0.76
Brownhill Creek	AW504901	0.82	17.5	0.77

4.2 Calibrations on Other Sub-Catchments

The RRR model was calibrated on sub-catchments of the Onkaparinga River where there were sufficient pluviometer and gauging data available.

The calibration of the RRR model was carried out using the parameter estimation program PEST. PEST can be applied to any model having ASCII text file input and output. The PEST program takes control of the model, by writing to the model data file before each run and then reading results from the model output file for use in the next iteration.

PEST proceeds to vary the parameters selected to minimise the difference between the observed and calculated values, in this case the hydrograph ordinates. It does this by minimising the sum of the squares of the differences between the observed and calculated values, designated phi by PEST. This is an objective function, to be minimised to provide the best fit.

There is the opportunity to provide a weighting to each observation, such that some observations are emphasised. In the case of fitting hydrographs this could be used to emphasise the fitting to the peak flow.

In all cases it was assumed that baseflow was occurring at the start of the event. The initial loss of the second process (IL2) was used in the calibration. The initial loss is defined as the loss that occurs between the start of the storm event and the start of the runoff that is modelled. However the definition of the start of the storm event is open to debate, particularly in the case of RRR which seeks to model all contributions to the event hydrograph. The initial loss for process 2 is clearly dependent on the time selected for the start of modelling.

For each catchment storms were selected for calibration by PEST based on the largest storm events for the period of record.

Once each of the selected storms had been calibrated using PEST the mean parameter values were determined. In order to determine mean values it is desirable to weight the calibrated parameters by a measure of how good a fit was achieved in the calibration run. A parameter calibrated from an event having a very good fit is obviously worthier of emphasis than one from an event that does not provide a good fit.

The weighting factor was chosen as follows:

A mean error of the estimate is defined as:

$$\text{Mean Error} = \sqrt{\frac{phi}{n}} \qquad \text{Equation 5}$$

where n is the number of observations, or hydrograph ordinates
phi Is the objective function used by PEST, being the sum of the squares of the differences between the observed and predicted ordinates at each time step

The mean error of estimate will not however provide a good measure of the overall fit that can be used for the weighting of calibrated values. The calibrated events are of varying magnitude, and account must be taken of this. Higher emphasis must be given to a good fit to an event having a higher peak flow. The weighting factor chosen was the observed peak flow divided by the mean error of estimate.

The weighting factor used was:

$$WF = \frac{\text{Observed peak flow}}{\text{Mean error}} = \left(\frac{n}{\sum_{1}^{n} (q_o - q_c)^2} \right)^{0.5} x\ Q_{op} \qquad \text{Equation 6}$$

where q_o is the observed flow at each time step
q_c is the modelled flow at the time step
n is the number of time steps or observations
q_{op} is the observed peak flow

Figure 4 shows the Onkaparinga catchment with the rainfall and stream gauging stations used in the calibration.

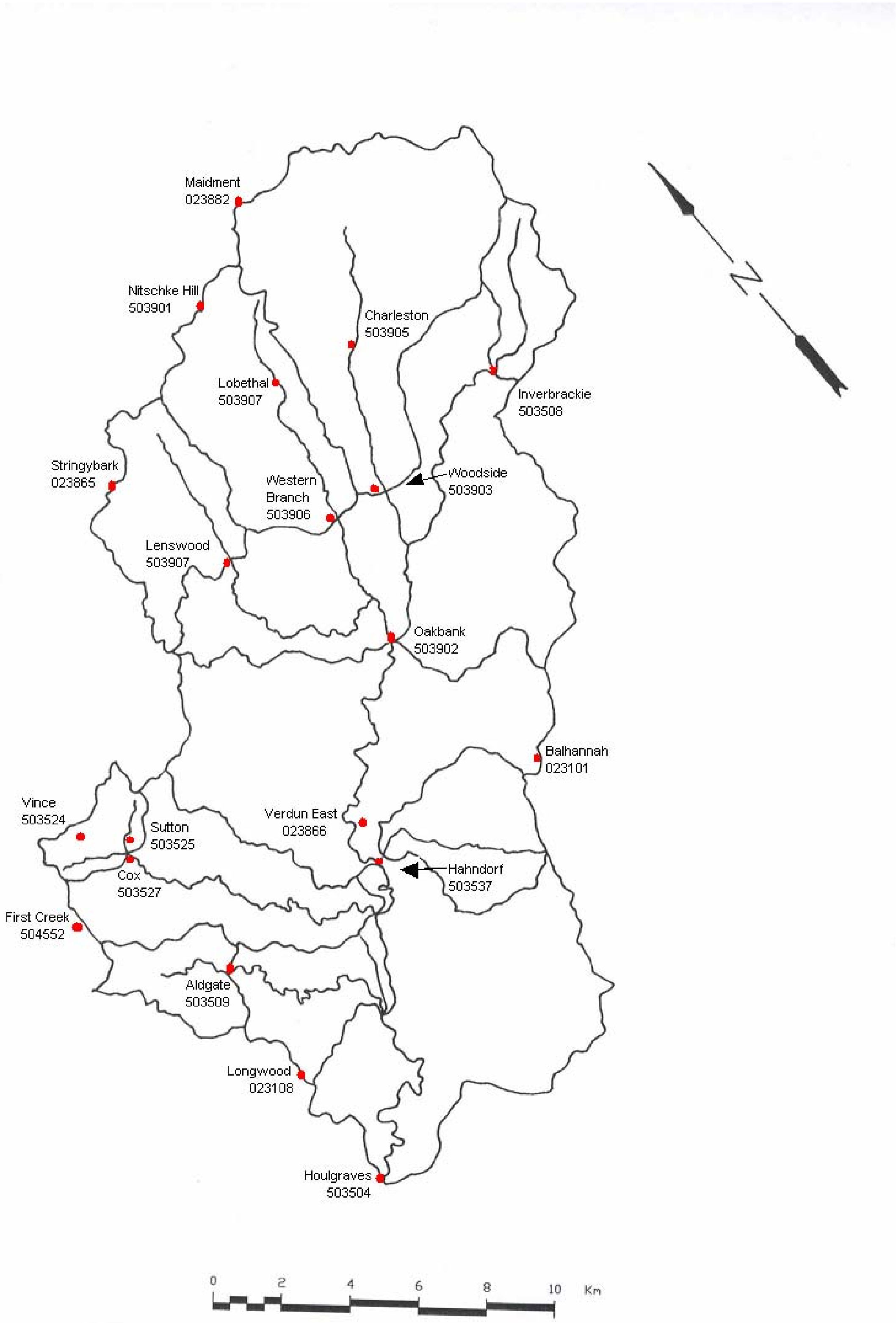

Figure 4 Onkaparinga Catchment with Rainfall and Stream Gauge Stations

4.2.1 Cox Creek

The Cox Creek catchment has a catchment area of 4.3km^2. It is located in the higher rainfall portion of the catchment, with an annual rainfall of approximately 1090mm/annum (Uraidla). Land use is dominated by horticulture, particularly viticulture. Underlying rock is predominantly sandstone.

Six events were chosen for the calibration of the model. Flow data at the Cox Creek station (AW50352), and rainfall data at either Vince (AW503524) or Sutton (AW503525) was used, depending on availability. Table 6 gives the results of the calibration.

Table 6 Cox Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	k_p1	k_p2
24/08/1983	0.82	8.80	0.80	0.036	0.589	0.0376
07/09/1983	0.79	3.39	0.74	0.133	0.396	0.0108
16/08/1984	0.90	6.05	0.80	0.089	0.524	0.1125
01/07/1986	0.77	0	0.84	0.072	1.354	0.0572
01/08/1986	0.74	10.3	0.75	0.093	1.073	0.0884
23/06/1987	0.73	0	0.58	0.308	0.874	0.0361
Mean	0.82	5.58	0.76	0.112	0.676	0.0660

The baseflow index (ie. the ratio of baseflow to total runoff) can be calculated from the mean proportional losses. For Cox Creek the baseflow index is 0.429.

4.2.2 Lenswood Creek

The Lenswood Creek catchment has an area of 16.5km^2, and an annual average rainfall of 1030mm (Lenswood). The catchment land use is horticulture, but a substantial amount of native vegetation remains. The predominant rock types are siltstones and shales.

Six events were chosen for the calibration of the model. Flow data at the Lenswood Creek gauging station (AW503507) was used, along with rainfall data from the Stringybark pluviometer (BM023865). This pluviometer was chosen as it had a longer period of record than the Lenswood Creek pluviometer (AW503507), and it is situated such that it probably best represents catchment rainfall.

Table 7 Lenswood Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	k_p1	k_p2
02/07/1995	0.73	6.38	0.65	0.066	2.263	0.368
21/07/1995	0.53	4.32	0.66	0.180	1.418	0.172
03/08/1996	0.47	4.22	0.36	0.141	2.190	0.445
28/09/1996	0.75	38.6	0	0.126	1.564	0.391
27/07/1998	0.84	26.5	0.73	0.187	1.975	0.231
07/09/2000	0.61	12.1	0.46	0.111	3.304	0.48
Mean	0.68	17.28	0.58	0.131	2.134	0.357

The baseflow index is 0.432.

4.2.3 Western Branch

The catchment to the Western Branch gauging station (AW503906) includes the town of Lobethal. Average annual rainfall is approximately 888mm (Lobethal). The major rock type within the catchment is mainly quartzite. It has a catchment area of 24.2km^2.

Six events from the period of record were chosen for calibration, using pluviometer data from the Lobethal station (BM023862). The Lobethal station was chosen as it was closest to the catchment centroid. Table 8 gives the results of the calibration. Baseflow was present in only one event modelled.

Table 8 Western Branch RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	k_p1	k_p2
03/08/1996	0.90	4.28	0.66	0.277	1.225	0.298
28/09/1996	%	14.3	0.63	0.345	%	0.462
27/07/1998	%	28.85	0.78	0.317	%	0.387
07/08/1999	%	19	0.82	0.264	%	0.400
15/09/1999	%	25.13	0.78	0.249	%	0.374
07/09/2000	%	9.16	0.70	0.299	%	0.413
Mean	0.90	18.04	0.73	0.292	1.225	0.391

Note %: No contribution was found from this process.

The baseflow index for the one event having baseflow is 0.227.

4.2.4 Woodside Weir

Six events were chosen for calibration at the Woodside Weir on the Onkaparinga River (AW503903). The catchment area to this point is 51.9km^2, and the average annual rainfall is 812mm (Woodside). Rainfall data from the Lobethal pluviometer (BM023862) was used, as this pluviometer is closer to the catchment centroid than the pluviometer at AW503903.

Table 9 gives the result of the calibration. Baseflow was present in two of the six events. In these two events the relative contribution of the second contribution (slow flow) was greater.

Table 9 Woodside Weir RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	k	k_p1	k_p2
21/07/1995	0.92	9.12	0.62	0.328	2.933	0.479
03/08/1996	%	7.58	0.59	0.109	%	0.358
26/08/1996	0.80	11.32	0.36	0.267	1.536	0.387
28/09/1996	%	13.65	0.52	0.309	%	0.556
27/07/1998	%	21.73	0.79	0.383	%	0.465
07/09/2000	%	5.96	0.68	0.382	%	0.672
Mean	0.85	13.39	0.68	0.347	2.092	0.567

Note %: No contribution was found from this process.

The mean baseflow index for the two events having baseflow is 0.215.

4.2.5 Aldgate Creek

The Aldgate Creek catchment is situated in the high rainfall part of the Onkaparinga catchment, with an average annual rainfall of 1190mm (Stirling). The catchment has a substantial amount of urban and commercial development within it, and for this reason the catchment was modelled with the assumption of 10% impervious area. The impervious area was assumed to have an initial loss of 1mm, and zero continuing loss.

Seven storm events were modelled. Only two of these (22/05/1999 and 07/09/2000) had rainfall data available from a pluviometer at the gauging station (AW503509). For the other events pluviometer data from Mount Lofty was used (AW504552). As this station is outside the catchment and in an area having steep rainfall gradients is was expected that it would be more difficult achieving a reasonable fit for most events.

Table 10 gives the result of the calibrations.

Table 10 Aldgate Creek RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k	kp1	kp2
07/09/2000	0.80	20.16	0.52			0.259	2.634	0.207
22/05/1999	0.73	18.13	0.87			0.242	5.712	0.129
22/09/1998	0.91	5.07	0.90			0.254	2.600	0.111
01/08/1995	0.63	0.00	0.68			0.190	0.823	0.152
22/05/1988	%	31.59	0.24			0.200	%	0.236
21/06/1987	%	20.27	0.45			0.062	%	0.284
01/07/1986	0.79	26.07	0.81	85.0	0.63	0.585	3.007	0.106
Mean	0.75	15.63	0.60			0.235	2.425	0.180

Note %: No contribution was found from this process.

The baseflow index for Aldgate Creek is 0.384.

4.2.6 Oakbank

Two storm events were chosen for modelling at Oakbank. The gauging station at Oakbank did not open until October 1996. The two events chosen were those having large flows in the sub-catchments (Lenswood, Western branch and the Onkaparinga at Woodside)

It was found that for the two events no good calibration could be carried out. This is most probably as a result of the position of the station, upstream of a ford having low flow pipes that block. When the pipes block they give falsely large flow readings.

4.2.7 Houlgraves Weir

Calibration of the RRR model to the gauged flows at the Houlgraves weir (AW 503504) was carried out. The model had a total of 11 sub-catchments, including all those previously calibrated within the catchment.

Seven events were chosen for calibration. These events included where possible those used for calibration of the sub-catchments.

Table 11 Events used for Calibration at Houlgraves

Event Start Date	Peak Flow (m^3/sec)	Calibrated Sub-Catchments
29/08/92	429.4	
07/10/92	192.4	
17/12/92	186.4	
03/08/96	156.4	Lenswood, West Branch, Woodside
28/09/96	259.1	Lenswood
27/07/98	86.7	Lenswood, West Branch, Woodside
07/09/00	110.8	West Branch, Woodside, Aldgate

Calibrated parameter values (storage and loss) were used for sub-catchments where possible, otherwise mean values were used. The calibration is thus only of the balance of the total catchment area without those catchments previously calibrated.

Table 12 Houlgraves RRR Calibration Results

Event Start Date	PL1	IL2 (mm)	PL2	IL3 (mm)	PL3	k	kp1	kp2
29/08/92	%	22.1	0.11	%	%	0.854	%	0.697
07/10/92	0.91	12.0	0.21	%	%	0.625	2.330	0.615
17/12/92	0.64	41.9	0.92	90.0	0.25	0.621	4.275	0.296
03/08/96	0.59	0	0.35	%	%	0.645	9.507	0.822
28/09/96	0.85	18.5	0.76	50.7	0.31	0.297	1.391	0.645
27/07/98	0.94	21.1	0.72	%	%	0.532	2.709	0.895
07/09/00	0.73	14.0	0.65	%	%	0.381	1.672	0.745
Mean	0.78	21.4	0.61	73.9	0.27	0.549	3.436	0.652

Note %: No contribution was found from this process.

4.3 Global Storage Parameters from the Calibrations

From the above calibrations global parameters can be determined. These parameters are used in application to ungauged catchments.

Table 13 Global RRR Storage Parameters for Calibrated Catchments

Catchment	v_c	c_p1	c_p2
Cox Creek	0.77	0.505	0.049
Lenswood Creek	1.38	1.215	0.203
Western Branch	0.78	0.648	0.207
Woodside Weir	1.10	0.949	0.257
Aldgate Creek	0.54	1.602	0.119
Houlgraves	0.88	1.469	0.279

5 FLOOD FREQUENCY ANALYSIS

Flood Frequency analysis was carried out on stations within the Onkaparinga catchment having more than ten years of data. The stations used are shown in Table 14.

Table 14 Stations for Flood Frequency Analysis

Station Name	Number	Years of Record
Scott Creek	AW503502	31
Echunga Creek	AW503506	27
Lenswood Creek	AW503507	28
Inverbrackie Creek	AW503508	28
Aldgate Creek	AW503509	28
Cox Creek	AW503526	25

For each station annual maximum flows were determined. For periods of missing record comparison was made with adjacent stations, and the year discounted if there was not certainty that the annual maximum would not have occurred during the period.

For all stations log-normal frequency distribution was used. This distribution fitted the recorded values in most cases, and was confirmed by the continuous simulation to be a reasonable distribution. In some cases low flows were censored from the data set if these would influence unduly the frequency distribution for the less common flows.

The high flow in Cox Creek is double any other flow recorded at the station. Comment was sought from Robin Leaney, Senior Hydrological Information Officer of the Department for Water Resources, who confirmed that there were no indications that the flow was in error. John Harrison of the Adelaide Hills Council was also contacted. However there are no records in Council indicating that significant flooding had occurred. Examination of the records also showed that the recorded level at the gauging station was only 200mm above the second highest flow.

In the end it was censored from the record for frequency analysis due to doubt about its accuracy. The results from frequency analysis were also compared with the design flows produced by the calibrated RRR model. This indicated that the frequency analysis and the RRR model flows are similar, supporting this decision.

Table 16 gives the result of the flood frequency analysis. Appendix 2 contains plots of the frequency distributions.

Table 15 Annual Maximum Flows (m^3/sec) used in Flood Frequency Analysis

Year	AW503502 (Scott)	AW503506 (Echunga)	AW503507 (Lenswood)	AW503508 (Inverbrackie)	AW503509 (Aldgate)	AW503526 (Cox)
1970	7.3	n/a	n/a	n/a	n/a	n/a
1971	10.8	n/a	n/a	n/a	n/a	n/a
1972	5.5	n/a	8.5	4.0	2.7	n/a
1973	10.6	n/a	25.0	7.4	9.7	n/a
1974	8.3	n/a	6.1	5.8	5.4	n/a
1975	5.8	12.5	5.2	9.6	6.6	n/a
1976	1.3	12.3	2.4	1.7	3.5	1.7
1977	0.5	5.0	2.4	7.4	7.3	3.4
1978	6.1	11.3	10.8	6.2	6.4	7.5
1979	8.6	17.8	15.7	4.5	10.5	14.5
1980	7.4	4.1	5.7	0.5	8.2	4.9
1981	18.3	22.1	48.4	20.7	23.0	6.2
1982	1.9	0.5	1.2	0.007	3.3	2.6
1983	8.8	9.3	19.2	4.3	6.6	4.4
1984	8.9	14.3	8.8	3.7	4.8	4.2
1985	5.4	7.2	5.8	2.3	3.7	2.9
1986	12.3	8.6	17.5	2.5	6.6	5.8
1987	15.8	30.3	16.4	8.0	8.6	5.4
1988	5.0	16.7	10.4	5.3	16.9	5.6
1989	7.8	6.2	n/a	3.1	n/a	n/a
1990	4.1	17.0	n/a	3.0	n/a	n/a
1991	7.9	8.4	n/a	1.5	n/a	n/a
1992	15.0	44.2	n/a	18.1	n/a	n/a
1993	3.6	13.9	n/a	0.9	n/a	n/a
1994	1.5	3.3	n/a	0.019	3.5	n/a
1995	10.2	27.6	12.9	4.7	6.3	4.6
1996	15.4	41.7	15.1	6.3	6.9	4.1
1997	5.0	5.8	2.2	0.5	8.6	2.9
1998	5.9	6.6	9.3	1.0	8.8	3.4
1999	2.8	3.4	5.1	0.4	10.0	3.5
2000	8.5	17.4	9.8	8.4	8.2	6.6

n/a indicates that the year was not available or used for analysis.

0.5 Flow censored (not used) - low flow

14.5 Flow censored – high outlier

Table 16 Results of Flood Frequency Analysis

Station	Area (km^2)	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)
Cox	4.3	6.65	7.49	8.55	9.35
Aldgate	7.96	13.2	15.9	19.7	22.6
Inverbrackie	8.4	12.3	16.2	22.0	27.0
Lenswood	16.5	25.9	35.8	51.6	65.9
Scott	26.8	15.6	20	26.4	31.7
Echunga	34.2	30.6	40.1	54.6	66.9

For the quick estimation of flows within the catchment a regional regression can be derived from the above information. If area is considered to be the significant variable affecting design flows then the following equations can be derived:

$Q10 = 1.92\ A^{0.83}$ ($r^2 = 0.93$) Equation 7

$Q20 = 2.37\ A^{0.84}$ ($r^2 = 0.93$) Equation 8

$Q50 = 2.77\ A^{0.88}$ ($r^2 = 0.93$) Equation 9

$Q100 = 3.08\ A^{0.91}$ ($r^2 = 0.92$) Equation 10

Figure 5 shows the plot of design flows versus catchment area, and also the fitted equations. In using these equations however care should be exercised due to the differences in individual catchment responses as evidenced by the RRR model calibration. There is no substitute for proper hydrological analysis.

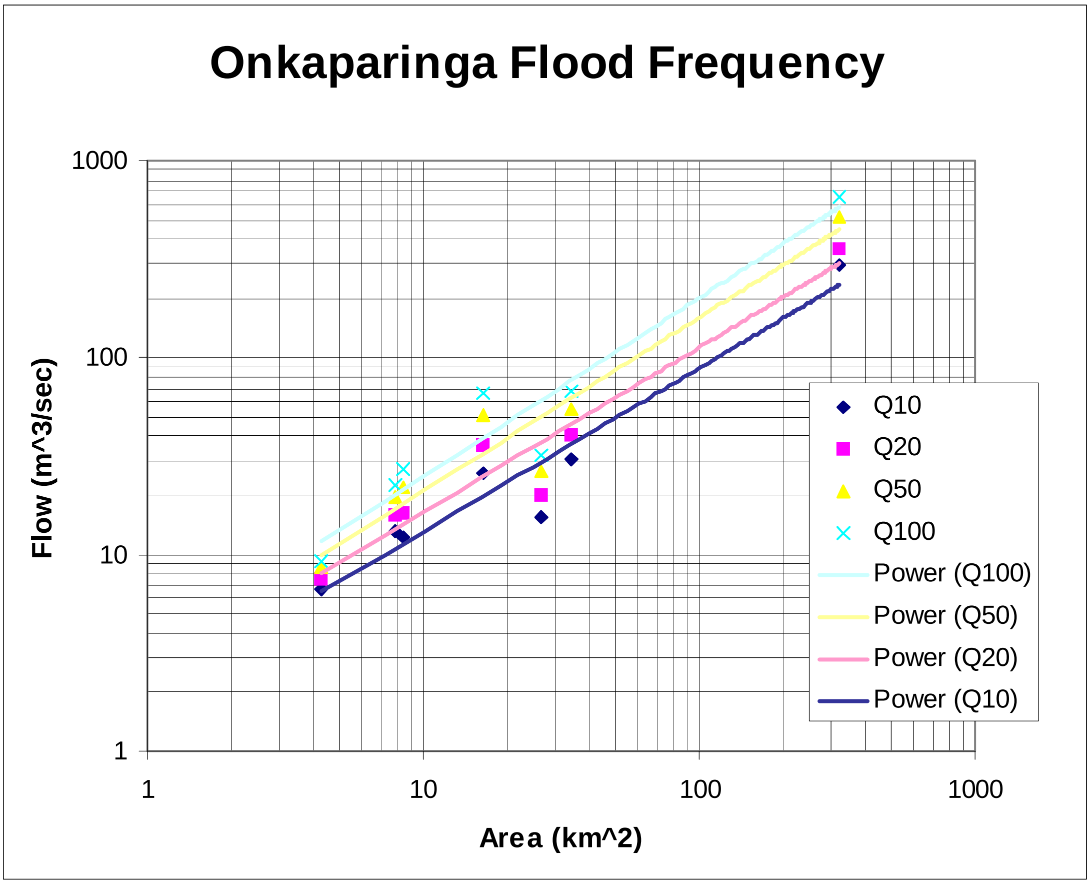

Figure 5 Onkaparinga Regional Flood Frequency

6 PARAMETERS FOR THE ESTIMATION OF DESIGN FLOWS

6.1 Comparison of RRR and Flood Frequency

Based on the calibration of the RRR model and the flood frequency analysis parameters must be chosen for the RRR model that will provide the best estimate of design flows at all locations within the catchment.

The first step is to apply design rainfalls and temporal patterns to the catchments where flood frequency analysis is available, to confirm that the model parameters determined from the calibration will predict similar flows to those determined from flood frequency analysis.

In all cases weighted mean values of the storage parameters and proportional losses were used, together with zero initial loss for process 1 (baseflow) and the weighted mean value of the initial loss for the second process. The initial loss for the third process was set at 100mm, with the proportional loss consistent with the proportional losses for the other two processes.

One problem with the estimation of design flows is that the initial and proportional loss for process 3 (fast flow) is not usually determined from calibration, as the process occurs rarely. In most cases PL3 must be estimated. From the calibrations undertaken that show runoff from process 3, the proportional loss is generally of the same order as that of process 1 and 2. Table 17 gives a summary of the proportional losses.

Care must be taken in the application of the RRR model as losses for all processes are related to the total rain falling on the catchment. Thus, with a low proportional loss applied to each process it is possible that the outflow volume from the catchment could exceed the rainfall input volume. If the initial and proportional loss for each of the three processes were zero, the volume outflow would be three times the rainfall volume.

The use of PL3 close to the value of PL1 sometimes leads to this problem, with more runoff occurring than rainfall during that part of the storm where 3 runoff processes are occurring. The value of PL3 to be used for design purposes must be reviewed in the derivation of design losses, to avoid the situation where runoff is exceeding rainfall for part of the storm.

Table 17 Proportional Losses Assumed for Comparison

Catchment	PL1	PL2	PL3	% Runoff with 3 processes operating
Cox	0.82	0.76	0.80 (estimated)	62%
Aldgate	0.75	0.60	0.65 (from 1 calibration)	100%
Inverbrackie	0.74	0.42	0.70 (estimated)	114%
Lenswood	0.68	0.58	0.60 (estimated)	114%
Scott	0.78	0.76	0.75 (estimated)	71%
Echunga	0.89	0.72	0.82 (from 1 calibration)	47%
Houlgraves	0.78	0.61	0.56	105%

The initial loss of process 3 is also unknown, but 100mm is assumed for initial comparison.

Table 18 gives the comparison, and confirms that there is no significant bias. However there are some differences, particularly significant being the Echunga Creek and the Houlgraves catchment.

Table 18 Comparison of Flood Frequency and Calibrated RRR Model

Catchment	Q10 RRR model (m^3/sec)	Q10 flood frequency (m^3/sec)	Q100 RRR model (m^3/sec)	Q100 flood frequency (m^3/sec)
Cox	5.7	6.7	9.3	9.4
Aldgate	14.4	13.2	24.4	22.6
Inverbrackie	13.2	12.3	22.9	27.0
Lenswood	24.2	25.9	61.3	65.9
Scott	18.5	15.6	31.3	31.7
Echunga	26.0	30.6	42.6	66.9
Houlgraves	212	294	509	657

6.2 Derivation of Design Losses

From the calibrated RRR models design losses must be determined. This is necessary because design storms represent bursts within longer duration storm events. In recent times work has been carried out by the CRC for Catchment Hydrology on the derivation of design losses for flood estimation (Hill et al, 1998).

Another problem is that the calibrated loss may not be truly representative of mean catchment conditions, to be used with design rainfalls. Examination of the calibrated proportional losses show wide variation (for example the proportional loss for process 2 on the Houlgraves catchment varies from 0.11 to 0.92 – in other words a runoff coefficient of 0.89 to 0.08 for the same catchment).

It is thus considered legitimate to vary the losses determined in the calibration to obtain design losses.

However the other issue is whether the RRR model flow or the station flood frequency analysis flow is more representative of the true flow for each recurrence interval. As the station flood frequency flow is based on recorded data, and in the case of Houlgraves is supported by the continuous simulation, emphasis should be given to at station flood frequency flows. It was thus decided to adjust the RRR model parameters to match the flood frequency analysis flows, where this was possible while keeping to reasonable parameter limits.

As a first step, the calibrated losses on the 6 catchments having flood frequency flows were adjusted so that the RRR model matched the flood frequency analysis flows. Houlgraves was excluded, as design loss parameters had to be determined for sub-catchments first. This was done as follows:

The PL2 was adjusted so that the 10 year ARI flows matched. This was done as it was expected that the slow flow contribution would be the most significant at this ARI.

The PL3 and IL3 were then adjusted to give good agreement with the 100 year ARI flow. The IL3 was kept at 100mm, and PL3 adjusted. If the total runoff occurring during that part of the storm having fast runoff exceeded the rainfall occurring, IL3 was adjusted.

Table 19 gives the results of the RRR model parameter adjustment for the 6 catchments having flood frequency analysis available.

Table 19 RRR Model Design Loss Parameters – Catchments with Frequency Analysis

Catchment	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3
Cox	5.6	100	0.82	0.76	0.80
Aldgate	15.6	100	0.75	0.63	0.65
Inverbrackie	16.9	50	0.74	0.46	0.80
Lenswood	17.3	70	0.68	0.55	0.77
Scott	21.6	96	0.78	0.80	0.75
Echunga	8.7	90	0.89	0.67	0.44

The RRR model was then run on the Western Branch and Woodside sub-catchments, and the losses adjusted as above so that a reasonable match was obtained to the regional flood frequency flows. These two catchments showed no baseflow for most of the calibration events, so an initial loss had to be determined for the baseflow. This was set at 75mm. Since the proportional loss for the baseflow was relatively high flows were not sensitive to the value.

Table 20 gives the result of the adjustment.

Table 20 RRR Model Design Loss Parameters - Western Branch and Woodside

Catchment	IL1 (mm)	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3
Western Branch	75	18.04	100	0.90	0.55	0.45
Woodside	75	13.39	75	0.85	0.50	0.65

The model was then set up for Houlgraves. Parameters for individual sub-catchments were set as above, and as before losses on the balance of the catchment were adjusted to match first the 10 year ARI flow and then the 100 year ARI flow.

It was found that the 100 year ARI flow at Houlgraves could not be matched in this way. The peak 100 year ARI flow predicted was 587 m^3/sec, compared with the flood frequency flow of 657 m^3/sec (from section 2.1). Table 21 gives the parameters. They are significantly different to the mean values determined by calibration. However two of the events calibrated (29/08/92 and 07/10/92) had similar loss values to those finally selected.

Table 21 RRR Design Loss Parameters - Houlgraves

Catchment	IL2 (mm)	IL3 (mm)	PL1	PL2	PL3
Houlgraves	21.4	75	0.78	0.25	0.97

6.3 Comparison of Preliminary Design Flows with Other Studies

It is of interest to compare the preliminary design flows produced above with flows used in other studies within the catchment. It should be noted however that these flows are preliminary, subject to review by the steering group.

Table 22 Comparison of RRR Preliminary Flows and Other Study Flows

Study	Q100 Previous Study (m^3/sec)	Q100 RRR (m^3/sec)
Verdun	400	452
Balhannah	295	356
Stirling Flood Study (Aldgate Creek)	31	23

7 HYDROGRAPHS FOR MAPPING

Flows can be produced at all points of interest within the catchment, using the above calibrated parameter values. The design storage and loss parameters will be used. Allowance is made for those sub-catchments having significant urban development. The influence of the urbanisation of the Aldgate Creek catchment has already been allowed in the calibration of the model to the Aldgate Creek gauging station. This is achieved by the inclusion of an impervious area assumed to be directly connected to the channel system, with appropriate losses (a small initial loss and no continuing loss). Storage lag parameters for the balance of the area, and the characteristic channel velocity are unchanged, as it can be assumed that the balance of the catchment will act like a rural catchment. Urban catchments (for example the Paddocks and Glenelg catchments, where RRR parameters were derived) do not retain the open channel system, and rural catchment runoff processes cannot occur.

It should be noted that the above analysis has not taken into account areal reduction factors for design rainfalls as recommended in Australian Rainfall & Runoff. Since the RRR model has been calibrated with design rainfalls to match the flood frequency flows at gauging stations within the catchment this areal reduction has been taken into account in the losses selected. A lower design rainfall could have been used (adjusted by the areal reduction factor), but this would have led to lower losses being selected to compensate.

To give an idea of the significance of the areal reduction factor, for the 36 hour 100 year ARI design storm on the Houlgraves catchment the design rainfalls should have been multiplied by a factor of 0.95. Smaller areas within the catchment will have a higher factor, in other words less reduction.

In order to keep the size of the models for the derivation of design flows manageable the catchment was split into separate models for parts of the total catchment.

These models are as follows:

- Lobethal.
- Woodside, including all the catchment above Verdun.
- Aldgate Creek, to the junction of the Onkaparinga.
- Cox Creek, to the junction of the Onkaparinga.
- Hahndorf.
- Downstream of Verdun.
- Kangarilla
- Echunga

In the case of the model downstream of Verdun hydrographs have been directly entered into the model at Verdun, the Aldgate Creek junction and the Cox Creek junction from the other models.

All models were run for a range of storm durations, to determine the maximum flow. In most cases for rural catchments the critical duration was 36 hours, apart from Aldgate Creek to the gauging station (48 hours) and Cox Creek at the gauging station (24 hours). The differences in critical duration are due to the different losses in the catchments affecting the excess rainfall with different storm durations.

Electronic files containing hydrographs for the locations required for floodplain mapping were created. Each location is referenced by an eight figure grid reference, scaled from 1:10,000 mapping.

Comments on the individual models are as follows:

7.1 Lobethal

The Lobethal model uses the storage, loss and characteristic channel velocity parameters derived by calibration on the Western Branch gauging station, as the Lobethal model is a sub-catchment of the Western Branch catchment, and has similar geology to the rest of the catchment.

The sub-catchments having significant impervious area directly connected to the channel system were modelled as stated above, with up to 25% being considered to be directly connected.

The resulting model shows relatively high peak flows for short duration storms (30 minutes) due to the directly connected impervious area, but the highest peak flows occur for the 36 hour storm, due to the contribution of the rural areas, and the balance of the area within the township.

The peak 100 year ARI flow at the southern end of the town is 9.95m^3/sec.

7.2 Woodside

The Woodside model extends from the northern end of the catchment to Verdun. Calibrated parameter values are used for all sub-catchments, and the model sub-divided so that hydrographs are produced at all points of interest. In all cases the peak flow was obtained with a 36 hour duration design storm event.

The peak flows obtained from the detailed model were generally similar to the broad scale model used for calibration, with 100 year ARI flows at the major points of interest being as follows:

Woodside	98m^3/sec
Inverbrackie Creek	74m^3/sec (at Onkaparinga Valley Road)
Oakbank	361m^3/sec
Balhannah	386m^3/sec
Verdun	449m^3/sec

7.3 Aldgate Creek

The Aldgate Creek model extends for the full length of Aldgate Creek to the Onkaparinga River. It uses parameter values calibrated to the Aldgate Creek gauging station, or Houlgraves (below the gauging station). The 48 hour design storm was critical upstream of the gauging station, and the 36 hour storm for the rest of the catchment.

100 year ARI flows at points of interest are:

Gauging station	23m^3/sec
Mylor	49m^3/sec
Leslie Creek at Mylor	7.6m^3/sec

7.4 Cox Creek

The Cox Creek model extends to the junction of the Onkaparinga. The calibrated parameter values for Cox Creek were used to the gauging station, and the Houlgraves calibrated values downstream, of that point. The 36 hour design storm event was critical for all the catchment.

Cox Creek had a predicted 100 year ARI flow of 9.3m^3/sec at the gauging station, and 77m^3/sec at Mount Barker Road in Bridgewater.

7.5 Hahndorf

The Hahndorf model uses the calibrated parameter values from Houlgraves, and the allowance for directly connected impervious area as was used for Lobethal.

100 year ARI flows at points of interest are as follows:

Creek 1 at junction of Hahndorf Creek	15.0m^3/sec
Creek 2 downstream Church Street	4.1m^3/sec
Creek 3 at English Street	8.8m^3/sec
Creek 4 at Mount Barker Road	12.6m^3/sec
Hahndorf Creek upstream Onkaparinga River junction	47.7m^3/sec

The creek naming is in accordance with that shown in the Hahndorf Township Watercourse Management Study (Tonkin & Associates, 1992).

7.6 Downstream of Verdun

The model downstream of Verdun uses the outflow hydrographs from the Woodside, Hahndorf, Cox and Aldgate models and combines these with local inflow to determine flows in the Onkaparinga River.

The predicted 100 year ARI flow at Houlgraves is 609m^3/sec. This is higher than the flow predicted by the model used for the calibration of design losses, but is still less than the 100 year ARI flood frequency flow (657m^3/sec), or the continuous simulation 100 year ARI flow (817m^3/sec). Figure 6 and Table 23 contain a comparison of the flows predicted by the RRR model with the flood frequency flows and the continuous simulation flows. The RRR model flows are close to the flood frequency flows for most recurrence intervals, but are higher for more frequent events. The flows are still less than the continuous simulation flows.

Table 23 Houlgraves Predicted Flow Comparison (to 100 years ARI)

Average Recurrence Interval (years)	Flood Frequency Flow (m^3/sec)	Continuous Simulation Flow (m^3/sec)	RRR Model Flow (m^3/sec)
10	261	345	322
20	395	457	397
50	516	650	508
100	657	817	609

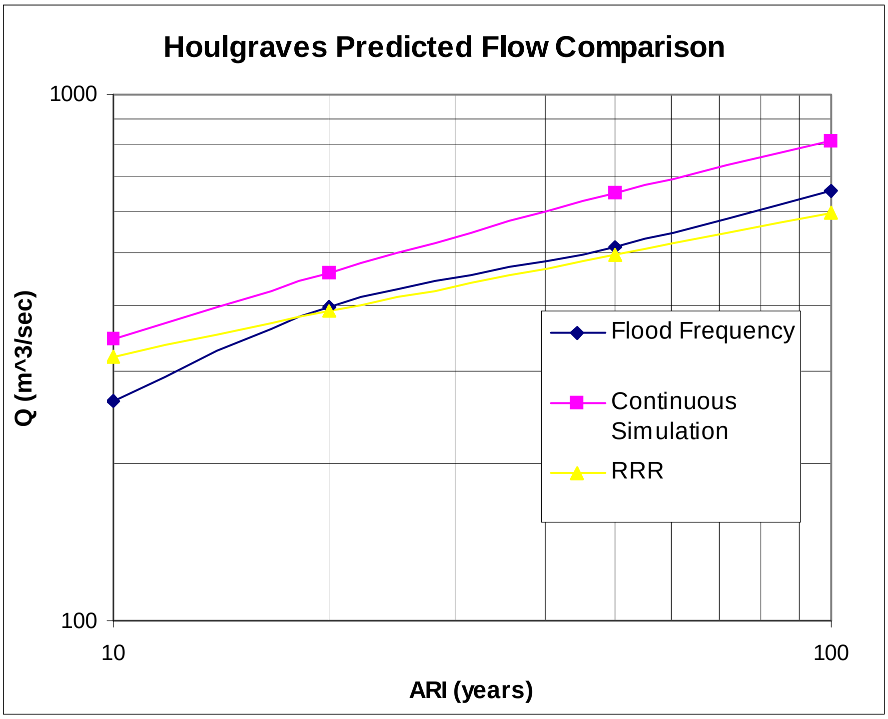

Figure 6 Comparison of Houlgraves Flood Frequency, Continuous Simulation and RRR Flows (to 100 year ARI)

In order to determine the sensitivity of the model to the loss parameters, and to see if the slope of the predicted flow line could be matched better to the flood frequency flows a series of runs were carried out using the Woodside model, with varying proportional losses.

The Woodside model was chosen as the design proportional losses for fast flow (process 3) for the Houlgraves calibrated part of the catchment was significantly higher than for other parts of the catchment, and the slow flow (process 2) lower. The comparison was carried out with flows at Verdun.

Table 24 Sensitivity of Verdun Q100 to Proportional Losses

PL2	PL3	Q10 (m^3/sec)	Q100 (m^3/sec)
0.25 (design)	0.97 (design)	223	449
0.37	0.85	212	437
0.47	0.75	200	427

The transfer of runoff from process 2 (slow flow) to process 3 (fast flow) has the effect of reducing the predicted peak flows at both the 10 and 100 year ARI. This is because of the high initial loss for the fast flow component. In general the predicted peak flow is not very sensitive, particularly at the 100 year ARI.

In the end it was decided to accept the flows predicted by the RRR model, as these compared reasonably well with the flows derived by other methods.

7.7 Kangarilla

A RRR model was set up to predict flows at Kangarilla. However there is no calibration available on which to base the design parameters. The model was run with parameter values from the Houlgraves calibration, and the Echunga Creek calibration. The results are given in Table 25.

Table 25 Predicted Flows for Kangarilla

Parameter Set or Regression	Q10 (m^3/sec)	Q100 (m^3/sec)
Parameter Sets		
Houlgraves Echunga	36.1 22.4	67.5 49.7
Regressions		
Eusuff (1995) Section 5, this report Woodside Model regression	21.5 26.8 30.2	38.6 55.3 54.5

The two parameter sets give quite different answers. In order to select which parameter set is appropriate comparison can be made with flows predicted by regional regressions such as Eusuff (1995), or the one derived by flood frequency analysis in Section 5, or the Woodside Model. It is recommended that the Houlgraves parameter set be used, as the Kangarilla catchment can be expected to have a short response time due to the relative steepness and lack of natural vegetation.

7.8 Echunga

The Echunga model uses the calibrated storage and loss parameter values from the Echunga Creek gauging station. The predicted 100 year ARI flow of Echunga Creek at Adelaide Road is 7.5m^3/sec.

8 FLOWS FOR THE CALIBRATION OF THE HYDRAULIC MODEL

During the mapping phase of the project it was decided to calibrate the hydraulic model to the flood event that occurred on 30 August 1992.

There were not as many pluviometers within the catchment in 1992 for the calibration of the model as for later events. The RRR model fit using the available pluviometers and the standard calibration method did not match the peak well at Houlgraves, as the methodology was designed to provide the best overall hydrograph fit.

It was decided to firstly review the rainfall inputs to the model and secondly to adjust the calibration to give a high weighting to matching the peak flow at Houlgraves. By this means the best estimate of flows could be derived for the Onkaparinga River from Woodside to Verdun.

The model was reviewed and the rainfalls in each sub-area upstream of Oakbank adjusted according to total rainfalls in the 3 day period to 9 am on 31 August 1992 for daily read stations compared with the pluviometers used in the model (Stringy bark and Inverbrackie Creek). This resulted in an increase in up to 35% in sub-area rainfalls. The model was then fitted using PEST with a large weighting given to the peak flow. Lower peaks were then not as well represented (Figure 7). However since the peak flow only is required this is considered to be a reasonable approach.

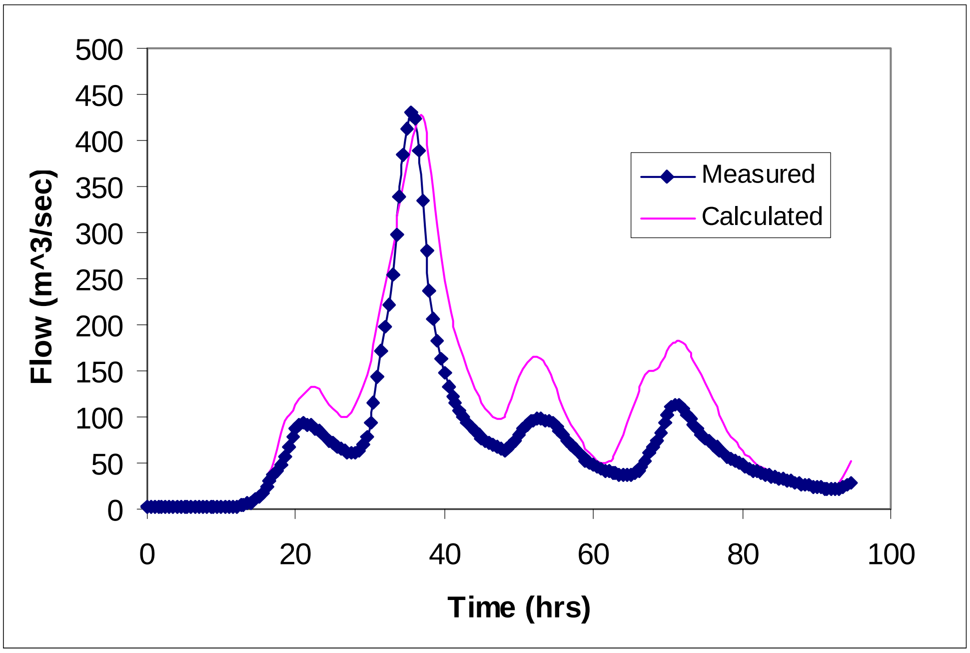

Figure 7 RRR Model Fit at Hougraves - Revised August 1992

The predicted peak flows at other points in the model are as follows:

Woodside	92 m^3/sec
Station AW503902 (Oakbank)	296 m^3/sec
Verdun - Upstream of Hahndorf Creek	350 m^3/sec
Verdun - Downstream of Hahndorf Creek	360 m^3/sec

These flows were used with the recorded flood extents, however there is likely to be some error in these predicted flows (say + or - 10 to 20 cumecs).

9 HYDROGRAPHS FOR RARE EVENTS

In accordance with Book 6 of Australian Rainfall & Runoff flood events in excess of 100 year ARI are termed rare events, and there are different requirements for design rainfalls and losses from large events (defined as 100 year ARI or more frequent).

For long duration rainfalls, which are critical on the upper Onkaparinga catchment the use of the GSAM temporal patterns are recommended. In addition the use of regional rainfall frequency analysis (CRC-FORGE) is recommended.

The CRC-FORGE rainfalls were derived for the same sites as were used for the design rainfalls for events up to 100 year ARI, and with both large event and GSAM temporal patterns. The GSAM temporal patterns were obtained from BOM (1993).

It was found when running the Woodside model for the 200 year ARI that the GSAM temporal pattern produced a much lower predicted peak flow, of 431m^3/sec at Verdun, compared with 627m^3/sec for the large event temporal pattern from Australian Rainfall & Runoff. The peak flow at Verdun is less than the 100 year ARI peak flow of 439m^3/sec predicted by the design rainfalls and temporal patterns for large events, so the use of the GSAM temporal pattern must be questioned. The use of losses interpolated between 100 year ARI and PMF losses, also as recommended by Book 6 did not produce a higher predicted flow.

The 200 year ARI flow with the CRC-FORGE rainfalls was also in excess of the flow predicted by the Australian Rainfall & Runoff intensities and temporal patterns for large events, this being 535m^3/sec. Consequently, the design rainfalls for the 36 hour duration were examined.

Table 26 summarises the rainfalls, and shows that although there is reasonable consistency at 100 year ARI, the 200 year CRC-FORGE rainfalls are generally greater than the Australian Rainfall & Runoff rainfalls, leading to the increase in peak flow.

Table 26 Design Rainfalls for the Upper Onkaparinga Catchment

Location	Australian Rainfall & Runoff 100 yr, 36hr (mm)	CRC-FORGE 100 yr, 36hr (mm)	Australian Rainfall & Runoff 200yr, 36hr (mm)	CRC-FORGE 200 yr, 36hr (mm)
Crafers	172.1	179.4	194.4	212.3
Bridgewater	162.0	156.1	181.1	184.2
Lenswood	142.2	147.3	156.6	173.4
Balhannah	145.1	149.0	161.6	175.0
Hahndorf	148.0	146.1	156.6	172.2
Echunga	148.7	145.1	167.4	171.1

In producing design hydrographs for large events the Australian Rainfall & Runoff temporal patterns were retained, together with CRC-FORGE rainfalls. This was in order to maintain consistency with flows of up to 100 years ARI. Table 27 and Figure 8 show the comparison of the flood frequency analysis at Houlgraves with the flows predicted by continuous simulation and the RRR model. The RRR model results show good agreement with the flood frequency analysis.

Table 27 Houlgraves Predicted Flow Comparison (to 500 years ARI)

Average Recurrence Interval (years)	Flood Frequency Analysis Flow (m^3/sec)	Continuous Simulation Flow (m^3/sec)	RRR Model Flow (m^3/sec)
10	261	345	322
20	395	457	397
50	516	650	508
100	657	817	609
200	819	1020	805
500	1090	1340	1076

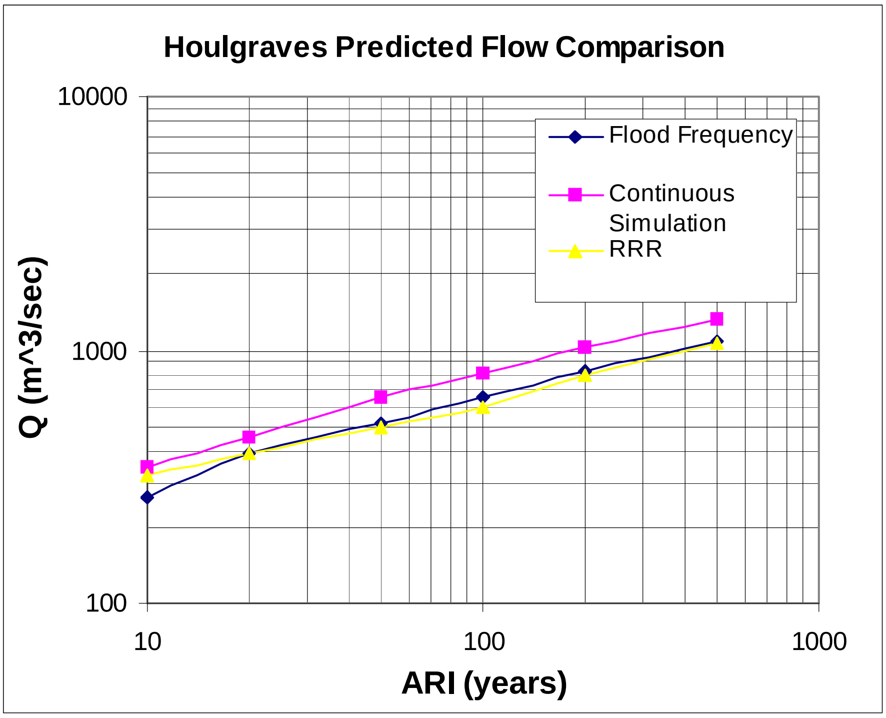

Figure 8 Houlgraves Flood Frequency (to 500 year ARI)

10 PEAK FLOWS FOR MAPPING

There were a number of sites where a peak flow only was required for the mapping. In order to reduce the effort involved in producing the estimate these flows were not produced directly by the RRR model.

For these sites flows were estimated using the approach of deriving a regression using predicted flows from the RRR model and catchment area, and applying this to the catchment area to the point of interest.

Three regressions were carried out, using the Aldgate Creek model to the gauging station, the Woodside model, and the Echunga model. The Aldgate Regression flows were applied to all small catchments within the Aldgate Creek and Cox Creek catchments, and the Woodside model to flows within the area of the Woodside and the Hahndorf model. Table 28 gives the equations used for the derivation of peak flows. All peak flows are calculated from the equation $Q = a A^b$.

Table 28 Regression Parameters for Design Peak Flows

Model	Q10		Q20		Q50		Q100		Q200		Q500	
	a	b	a	b	a	b	a	b	a	b	a	b
Aldgate	1.74	0.94	2.11	0.94	2.52	0.95	2.96	0.94	4.25	0.96	7.09	0.92
Woodside	1.13	0.99	1.42	0.99	1.89	0.98	2.72	0.95	4.10	0.92	5.39	0.93
Echunga	1.07	0.95	1.24	0.98	1.79	0.97	2.77	0.90	3.49	0.89	4.67	0.87

The flows are included in tables in Appendix 5, and labels as minor branch flows.

11 PROBABLE MAXIMUM FLOOD

An estimate of the probable maximum flood (PMF) is required for mapping. Due to the complexities of predicting the PMF at every location within the catchment it was decided that a simplified approach would be adopted.

The normal approach to the prediction of PMF requires that the storm producing the PMF be placed over the catchment to the point of interest so that the rainfall is maximised for the catchment. Since there are over 80 sites at which hydrographs are required this would be an extremely time consuming process. The suggested approach is to determine the PMF at the Houlgraves gauging station, and use a regional regression equation to determine PMF at all other sites.

The regression is based on Nathan et al (1994). Information was collected for a total of 68 catchments to dams and other storages within southeastern Australia for which PMF had been calculated. It was found that the main determinant of peak flow was catchment area, as follows:

$$Q_p = 129.1A^{0.616} \qquad (r^2 = 0.95) \qquad \text{Equation 11}$$

To confirm that the flows predicted by Nathan are reasonable for the Upper Onkaparinga catchment the PMF was calculated for the Houlgraves gauging station, using the calibrated RRR model. Rainfalls were calculated by the methodology outlined in the Bureau of Meteorology publication “The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method” (GSDM), published as Bulletin 53, and amended in 1996.

A storm with a standard areal rainfall distribution is placed over the catchment such that the total catchment rainfall is maximised. The storm pattern is defined by a series of ellipses representing rainfall depth. Mean rainfalls are calculated for each successive ellipse, from which sub-catchment rainfall is calculated. This procedure is repeated for a range of storm durations. The methodology is applicable for areas of up to 1000km^2, and 3 hour duration. For durations in excess of 3 hours reference was made to the CRC_FORGE design rainfalls for the Mount Bold dam, as published by Sinclair Knight Mertz for the SA Water Corporation (Sinclair Knight Mertz, 2000). The rainfalls were adjusted by a factor to account for the different catchment area to Houlgraves.

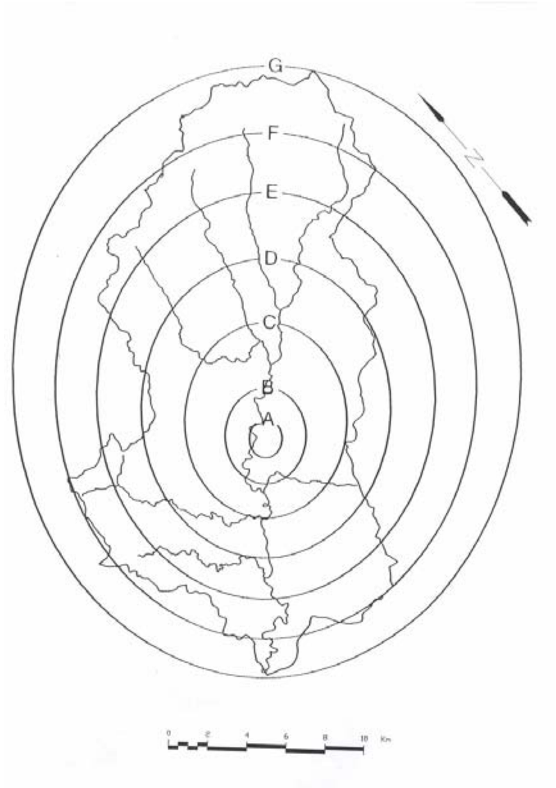

Figure 9 Onkaparinga Catchment with Short Duration PMP Pattern

In accordance with the recommendations of Book 6 of Australian Rainfall & Runoff losses in the calibrated RRR model were adjusted. Book 6 recommends the use of very low initial loss (usually zero), and also a low continuing loss. It is considered that for the PMF most runoff will be occurring as fast flow, and so the values of initial and proportional loss for the three processes were used as in Table 29.

Table 29 Losses for PMF

Process	Initial Loss (mm)	Proportional Loss
Process 1 (baseflow)	0	0.95
Process 2 (slow flow)	0	0.95
Process 3 (fast flow)	0	0.10

Application of the RRR model with storm durations of up to 3 hours gave an indication that the critical duration event for the downstream parts of the Onkaparinga River above Houlgraves was in excess of 3 hours.

To extend the analysis to durations in excess of 3 hours consideration has to be given to Generalised Southeast Australia Method (GSAM). Information on GSAM spatial and temporal patterns is given in BOM (1993). For storm durations between 3 hours and 24 hours both GSAM and GSDM spatial patterns must be applied to the catchment, and the maximum of these two flows becomes the PMF.

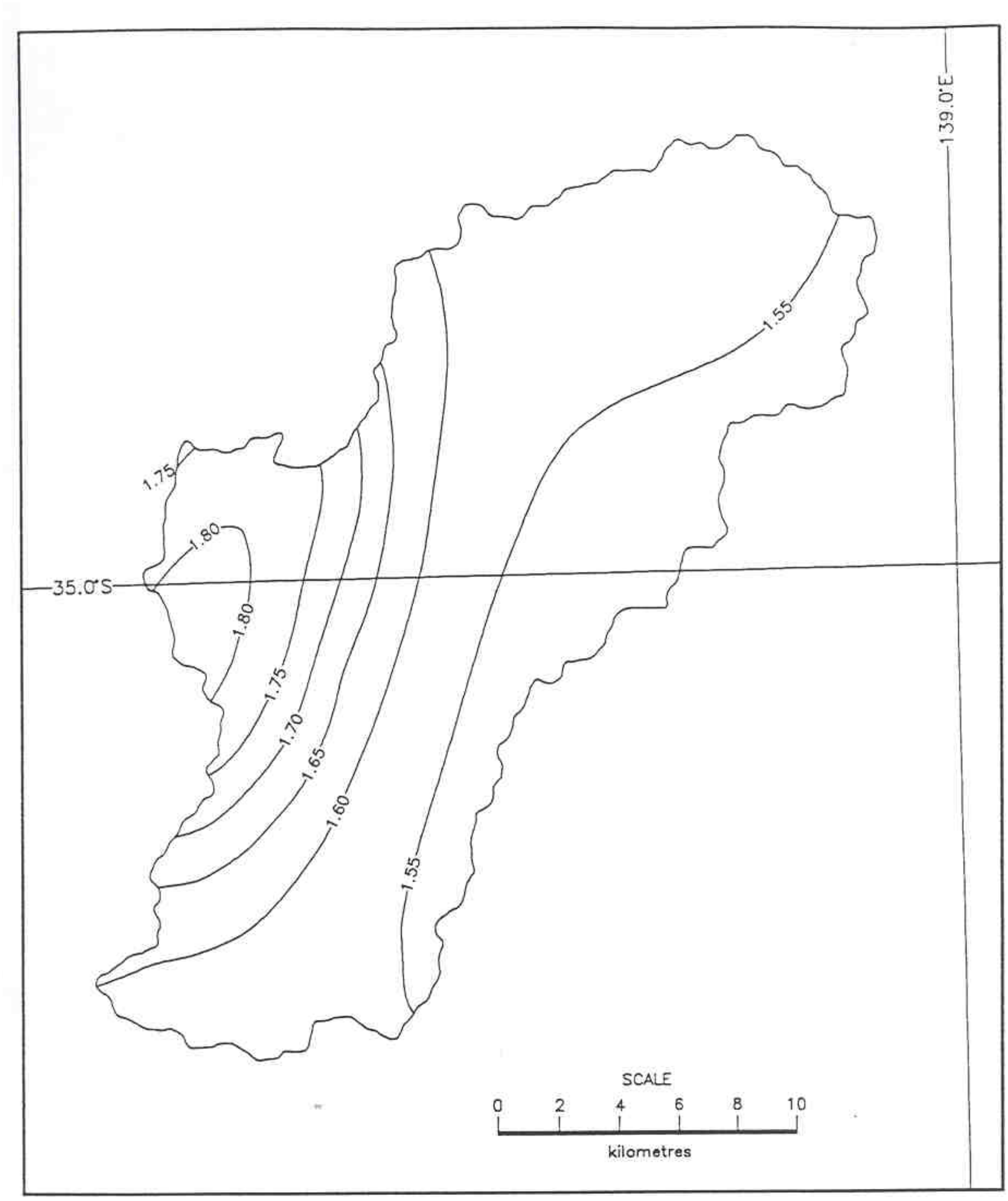

Figure 10 Mount Bold GSAM Long Duration PMP Spatial Pattern

Figure 10, from BOM (1993) shows the spatial patterns for the long duration GSAM rainfall, in terms of a ratio of the rainfall to a reference value. A procedure in the Bureau publication allows the determination of sub-catchment rainfalls, based on the catchment mean rainfall.

It is recommended also that both GSAM and GSDM temporal patterns be used, but it was decided to adopt the 24 hour pattern for the 12 and 18 hour duration, and the short duration (GSDM) patterns for the 6 and 9 hour durations.

In addition to the above analysis of the catchment to Houlgraves a PMF was calculated for sub-catchments with a catchment area less than $100km^2$, with a uniform rainfall applied. BC Tonkin in the analysis of the River Sturt (BC

Tonkin, 1996) found that for catchments less than 100km^2 there is no need to calculate spatial variations in PMP for input into a rainfall – runoff model to derive PMF, since resulting increases in the PMF are minimal.

The maximum flows at points within the Houlgraves model determined by the GSDM, GSAM or uniform rainfall methods were compared with Nathan et al (1994) flows. The result is given in Table 30 and shown on Figure 11.

Table 30 PMF for Onkaparinga Catchment

Location	Area (km^2)	RRR m^3/sec	Nathan m^3/sec
Cox Creek GS	4.3	280	320
Aldgate Creek GS	7.96	380	460
Hahndorf Creek	14.9	740	680
Lenswood Creek GS	16.5	940	730
Aldgate Creek at Onkaparinga junction	19.4	520	800
Western Branch GS	24.2	940	920
Cox Creek at Onkaparinga junction	28.2	770	1010
Onkaparinga at Woodside	51.9	1670	1470
Onkaparinga at Oakbank	162.4	3500	2700
Onkaparinga downstream Verdun	227.7	3600	3650
Onkaparinga downstream Cox Creek	260.1	3900	3970
Onkaparinga downstream Aldgate Creek	285.4	4100	4200
Houlgraves	321	4300	4520

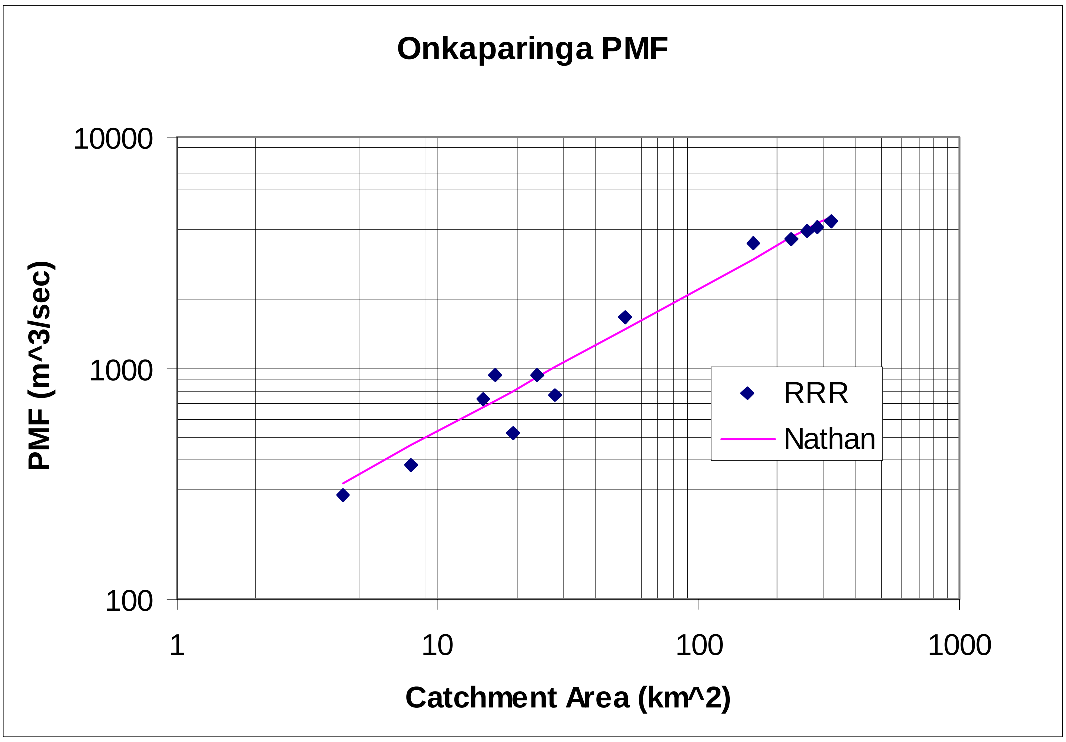

Figure 11 Onkaparinga Catchment PMF

It can be seen that there is reasonable agreement between the values derived by the application of Nathan's regression equation and the RRR model results. It is therefore acceptable to use the Nathan regression equation for the derivation of PMF values for the upper Onkaparinga river catchment, for the purposes of flood mapping.

12 SUMMARY

The RRR model has been used to predict flows throughout the upper Onkaparinga catchment. The model has been individually calibrated on a number of sub-catchments, and the resulting parameter values used with design rainfalls to predict design flows at a number of locations.

The values of the design flows have been checked where possible against at-station flood frequency analysis, to ensure consistency. In most cases the calibrated RRR model performs well.

The prediction of design floods has been extended to rare (200 and 500 year ARI) and extreme events, being the probable maximum flood (PMF). It has been confirmed that a regression equation can be used for the determination of PMF on the catchment. The predicted flows are consistent with the predictions on other catchments in southeastern Australia.

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APPENDIX 1

AWBM RESULTS
(Monthly runoff in mm)

YEAR		JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
1986	ACT	0	0	0	0	3	3	65	42	45	18	3	8	187
	EST	0	0	0	6	6	9	98	56	56	32	5	4	271
1987	ACT	0	0	0	0	10	65	84	43	9	11	1	1	225
	EST	1	1	1	1	14	59	66	43	6	7	1	4	203
1988	ACT	0	0	0	0	23	32	48	18	33	4	2	0	159
	EST	0	0	0	0	23	60	47	19	30	3	1	1	183
1989	ACT	0	0	0	1	3	12	34	58	36	11	4	2	161
	EST	0	0	0	0	7	34	50	50	21	4	2	2	169
1990	ACT	1	0	1	0	0	7	35	61	15	8	3	3	134
	EST	0	0	0	1	0	10	56	48	11	2	0	1	131
1991	ACT	0	0	1	0	0	5	17	56	82	5	2	1	169
	EST	0	0	0	1	0	9	34	50	46	5	1	0	146
1992	ACT	1	0	1	1	2	9	18	105	117	59	20	52	385
	EST	0	1	4	1	5	11	29	73	108	71	32	103	438
1993	ACT	3	0	1	0	2	4	12	10	10	8	2	2	56
	EST	12	1	0	0	1	5	22	18	19	11	1	2	93
1994	ACT	4	1	1	0	1	6	4	5	3	8	4	1	37
	EST	2	0	0	0	1	19	8	8	1	10	3	0	51
1995	ACT	0	0	0	1	3	10	134	32	6	4	1	0	192
	EST	0	1	0	1	6	27	106	23	3	1	0	0	168
1996	ACT	1	1	0	0	0	12	40	103	60	25	2	1	245
	EST	0	0	1	1	0	19	59	73	55	25	2	0	235
1997	ACT	1	0	0	0	1	1	1	16	17	4	4	1	46
	EST	0	0	0	0	3	2	1	13	22	5	3	0	50
1998	ACT	0	1	0	1	1	7	29	19	10	9	2	1	81
	EST	0	0	0	6	1	7	23	15	4	2	1	0	59
Total Actual = 2076.5				Total Calculated = 2196.9										

APPENDIX 2

FINAL CONTINUOUS SIMULATION PARAMETER FILE

```
Onkaparinga River at Houlgraves
     321.000
dayradj.onk
onkevapo.day
onkflow.day
onkflow.mon
hourradj.onk
hourflow.onk
  13
  1986
   11.0
  148.0
  473.0
  0.093
  0.385
  0.522
   12.0
   60.0
  300.0
    0.0
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 0.9228
 0.1431
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  1.50
  18.00
   4.30
   1.20
  34.90
   7.00
   2.00
   0.550
   0.00700
   2.56250
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 0.0005
 0.0010
 0.0027
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 0.4958
 0.0141
 0.3524
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0.0029
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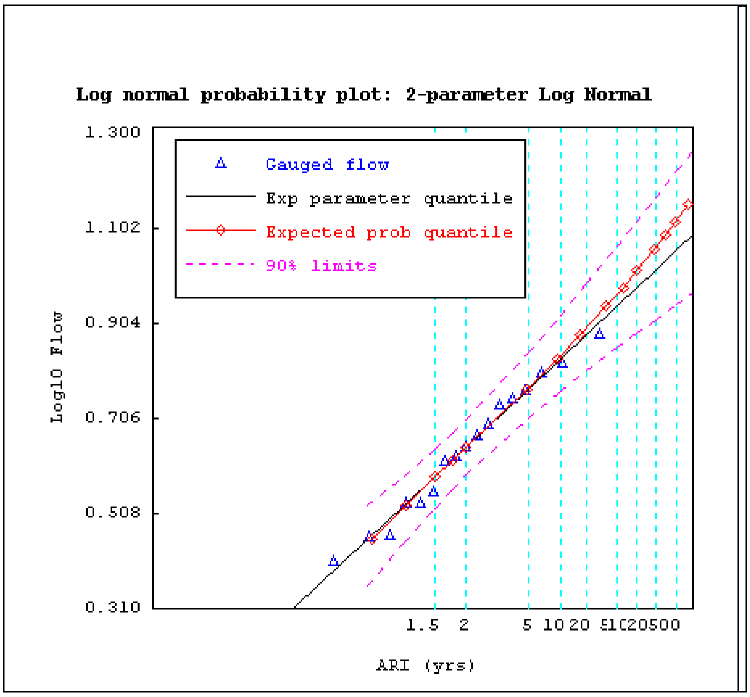
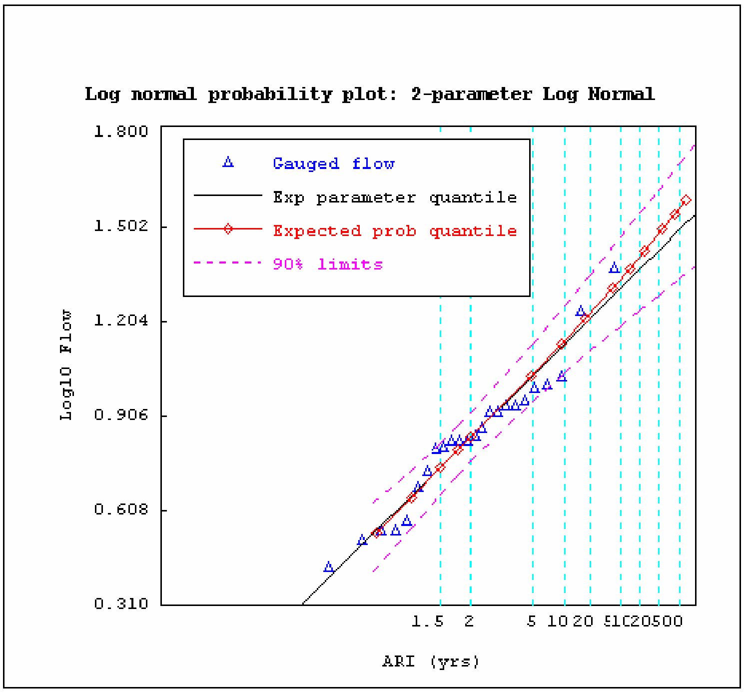
APPENDIX 3

FLOOD FREQUENCY ANALYSIS DISTRIBUTIONS

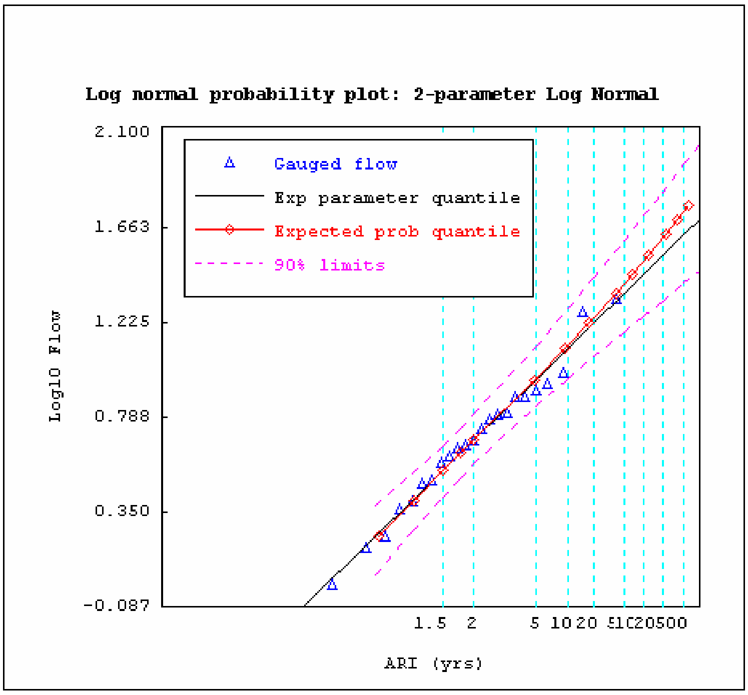
Cox Creek



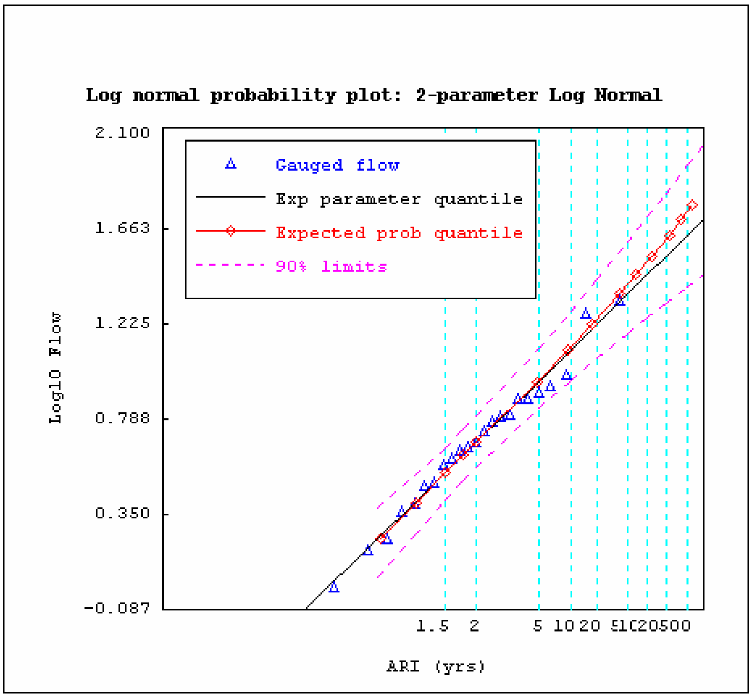
Aldgate Creek



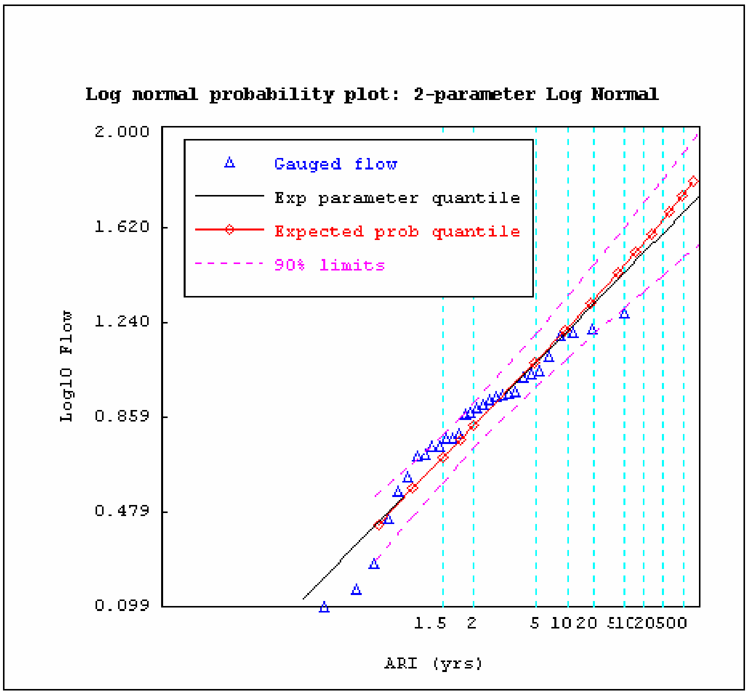
Inverbrackie Creek



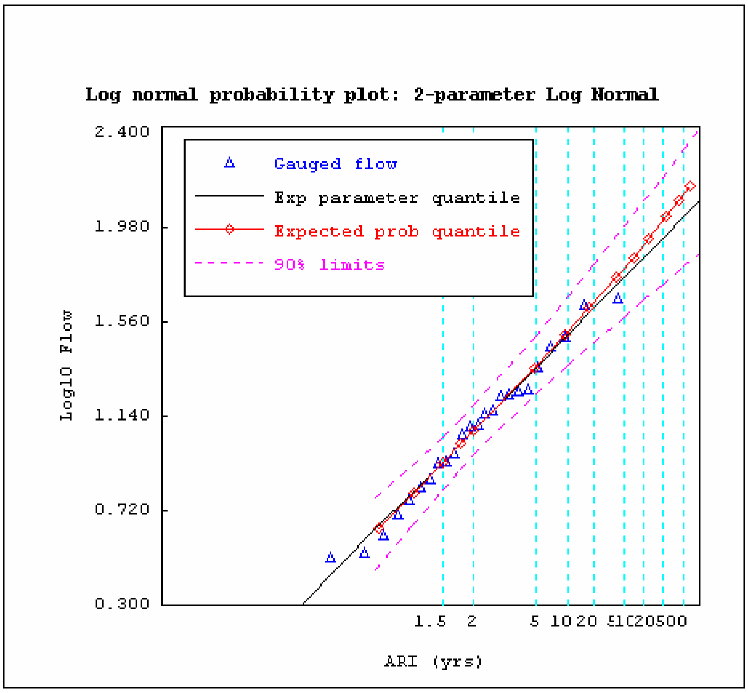
Lenswood Creek



Scott Creek



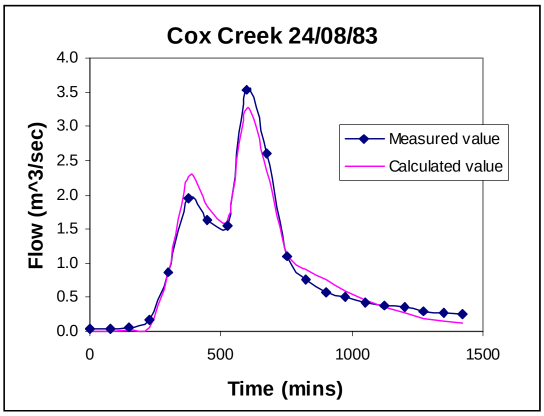
Echunga Creek

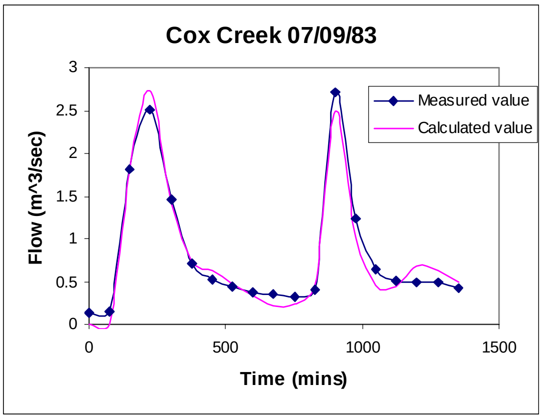


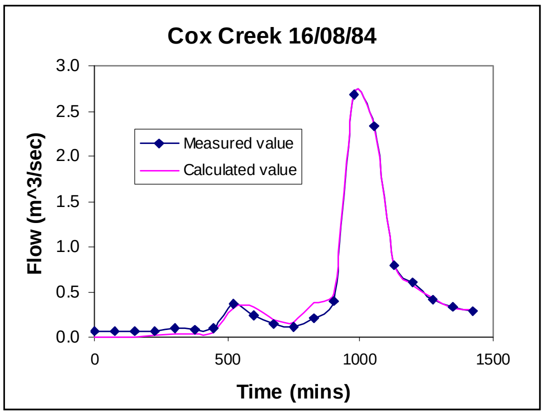
APPENDIX 4

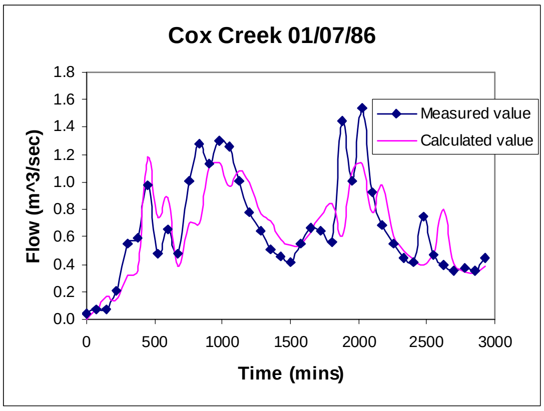
RRR MODEL CALIBRATIONS

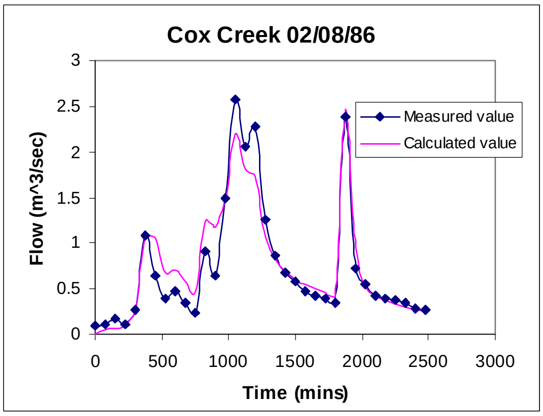
Cox Creek

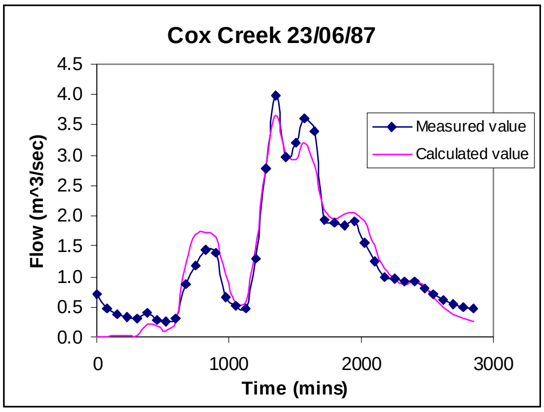




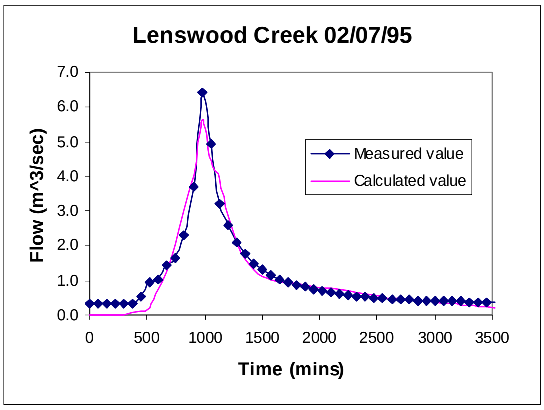


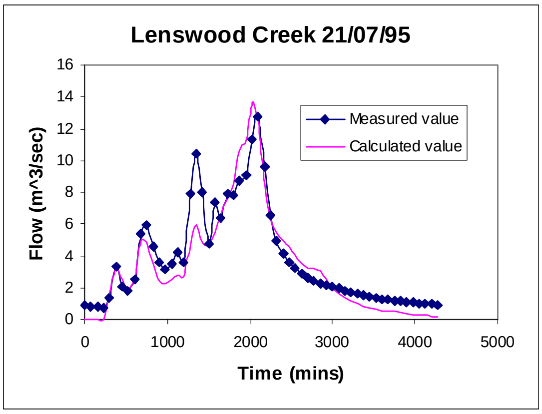


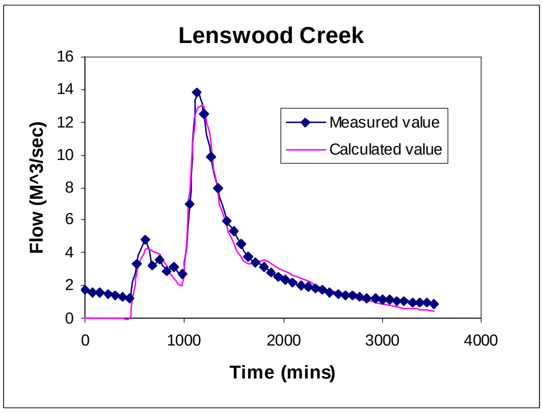


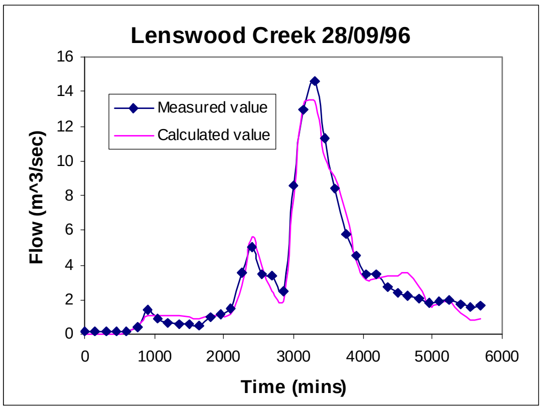


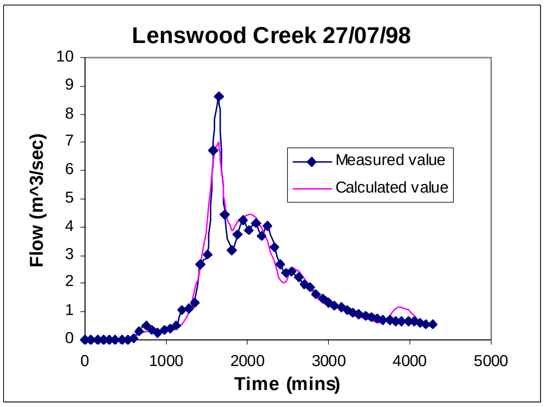
Lenswood Creek

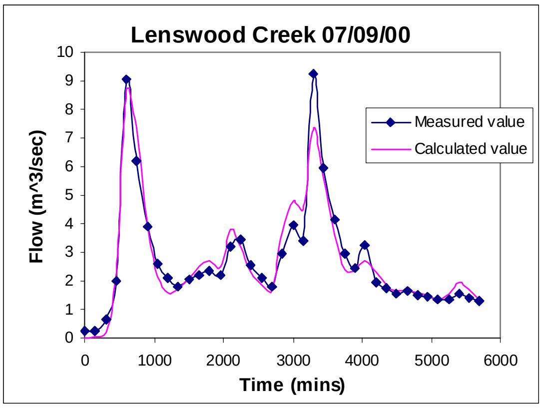




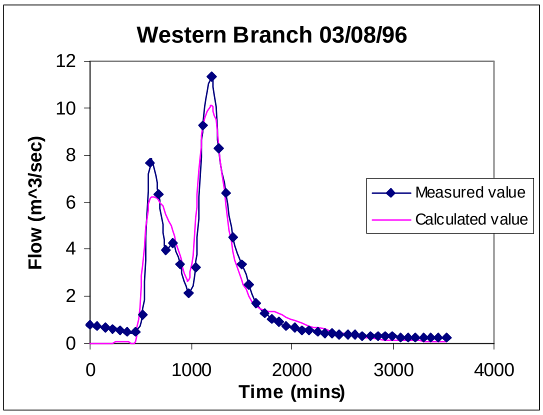


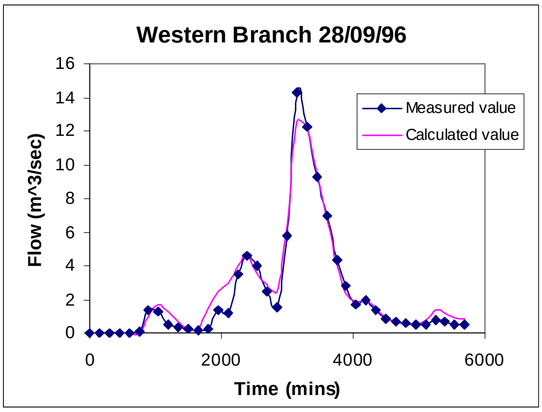


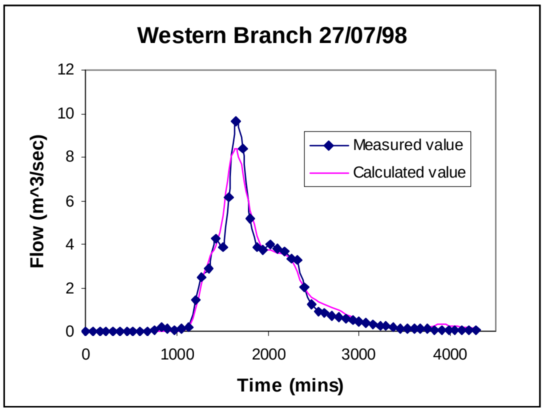


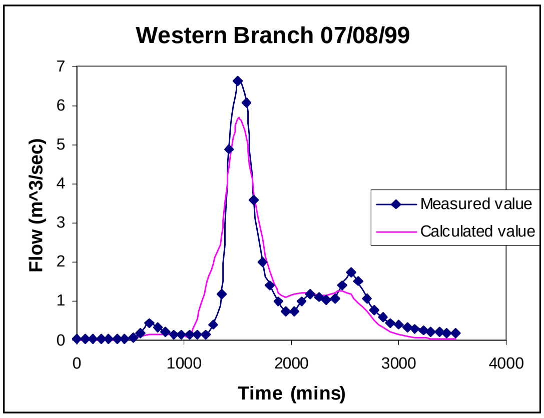


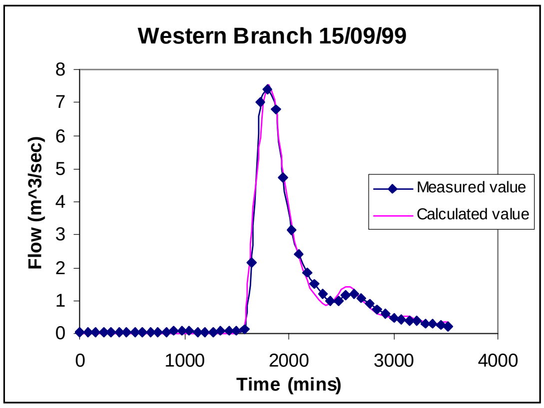
Western Branch

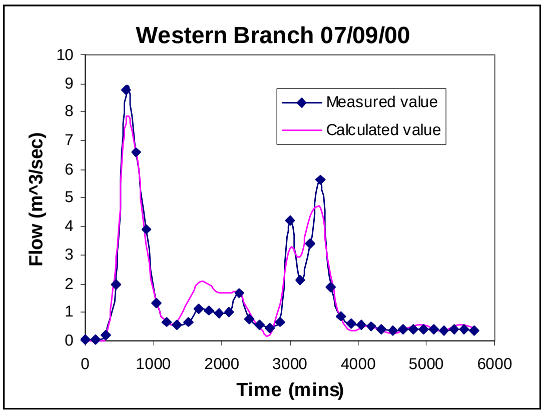












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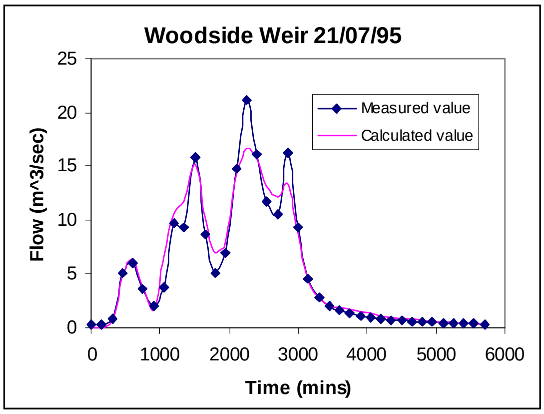
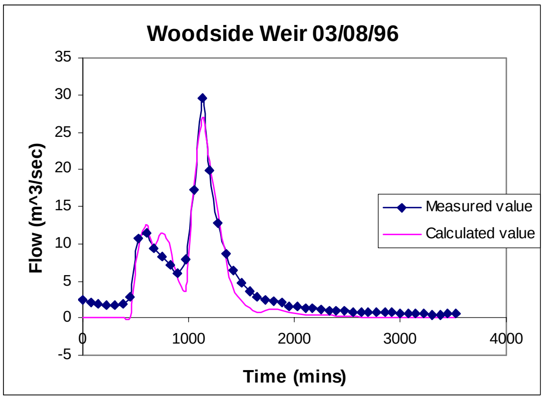
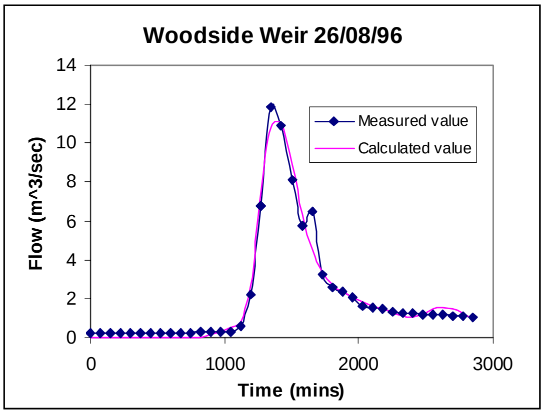
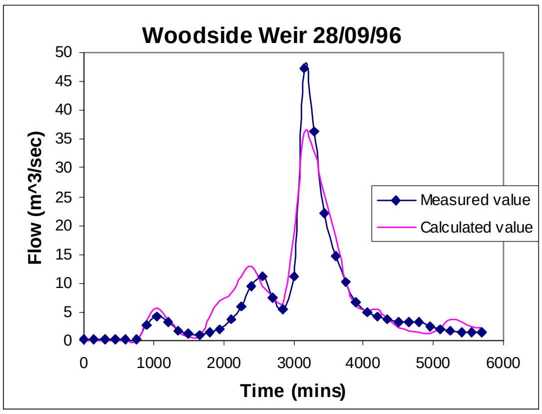
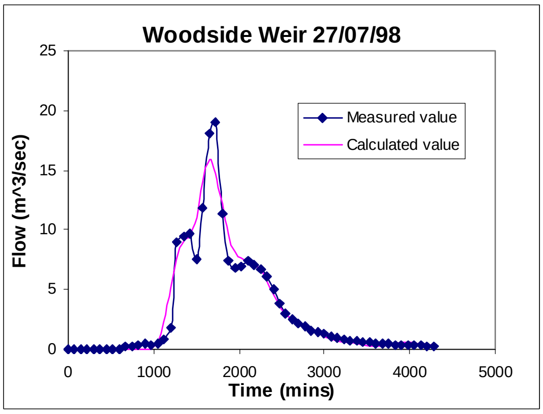
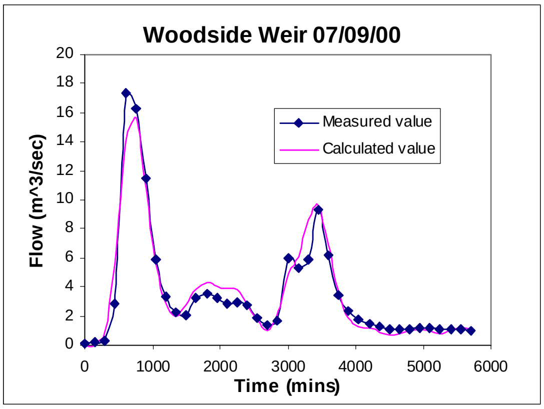




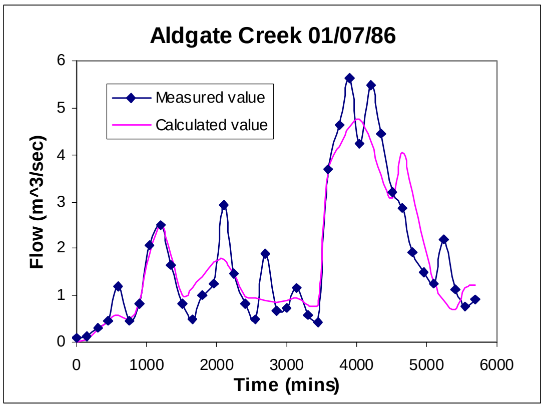


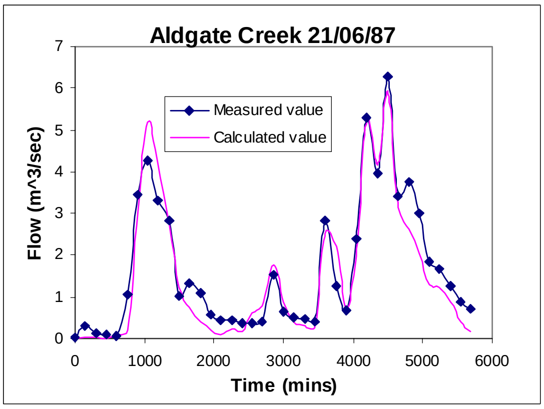


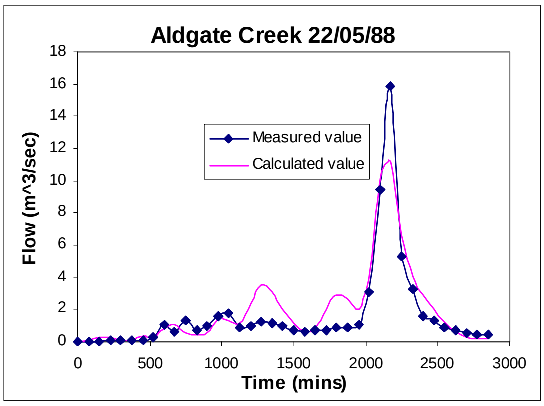


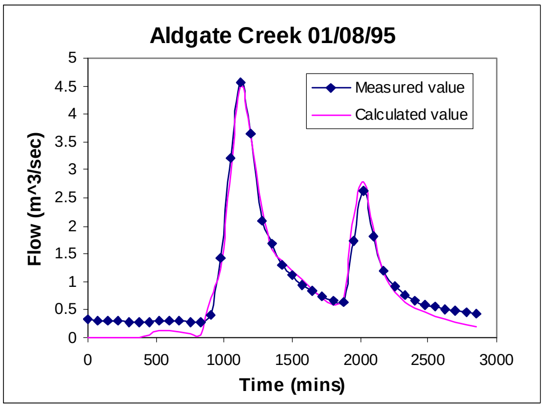


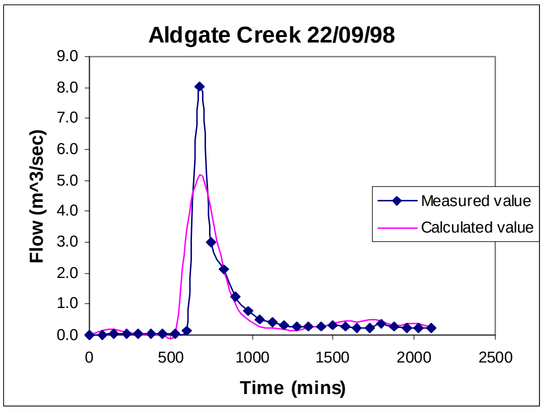
Aldgate Creek

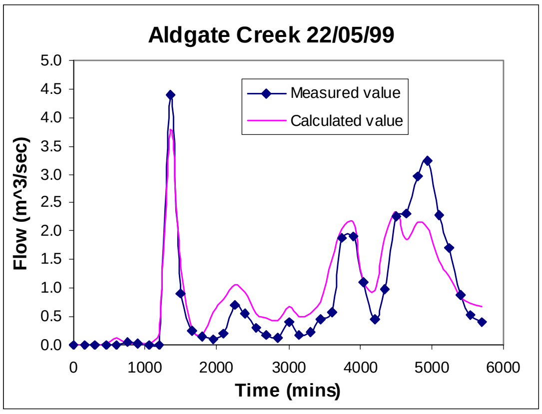


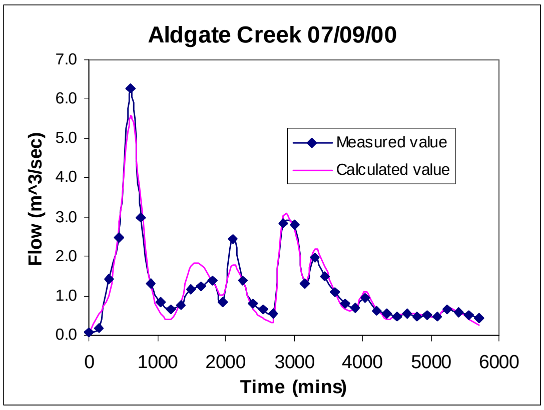




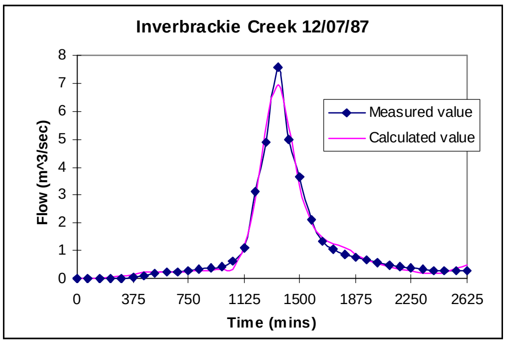


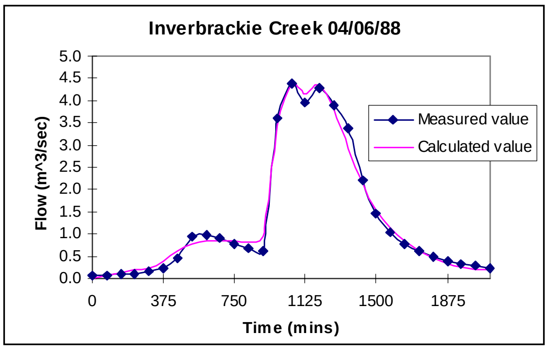


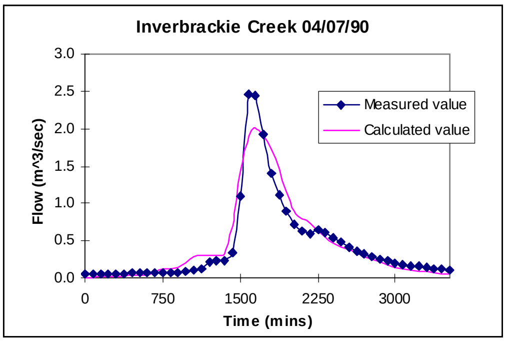


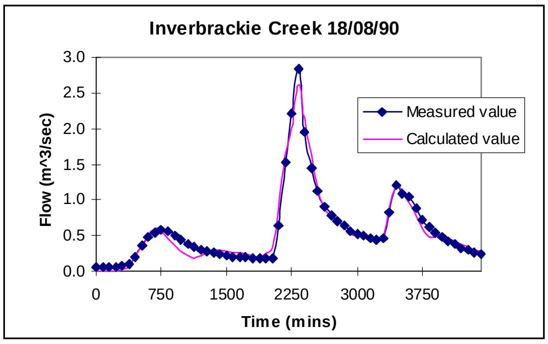


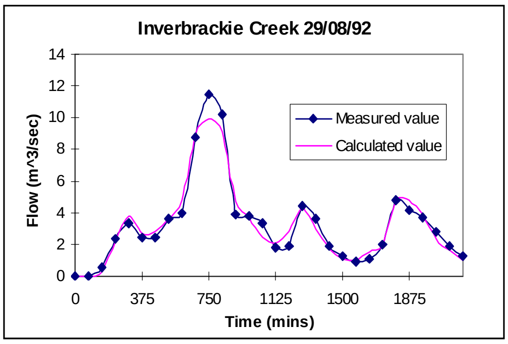
Inverbrackie Creek

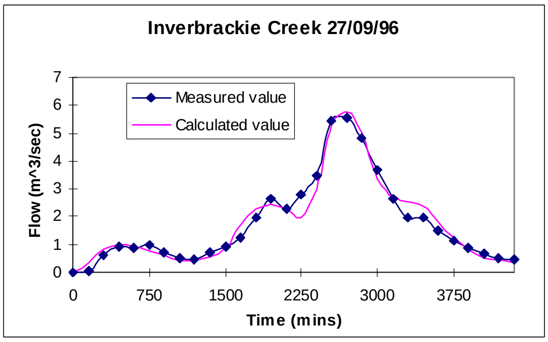




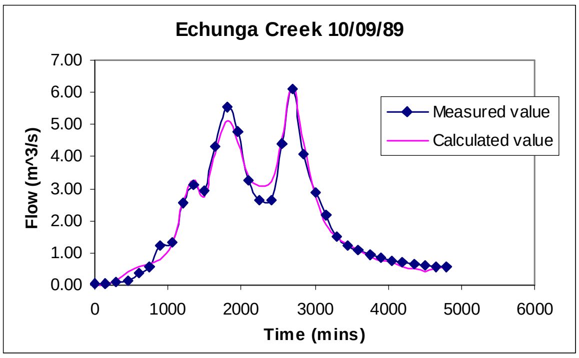


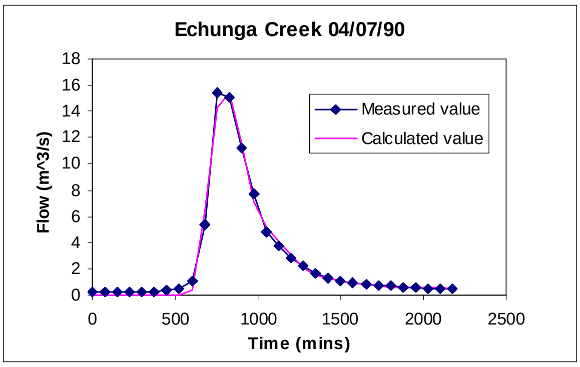


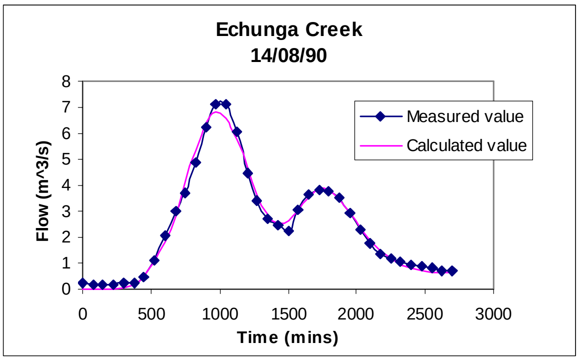


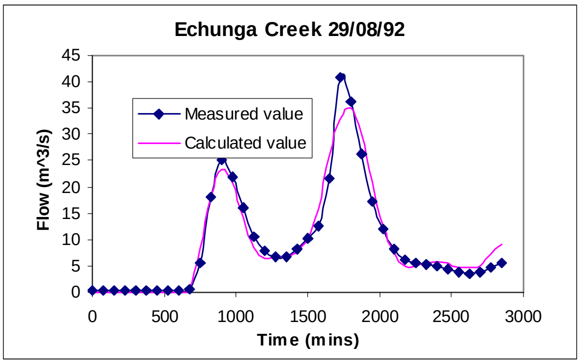


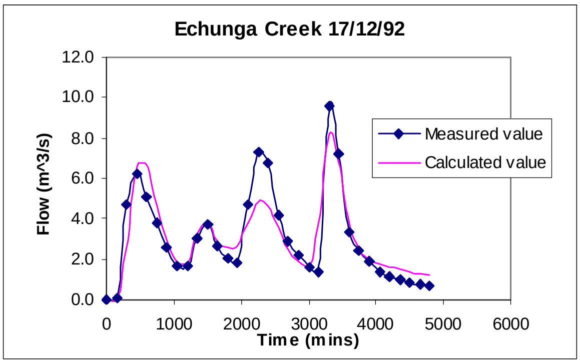
Echunga Creek

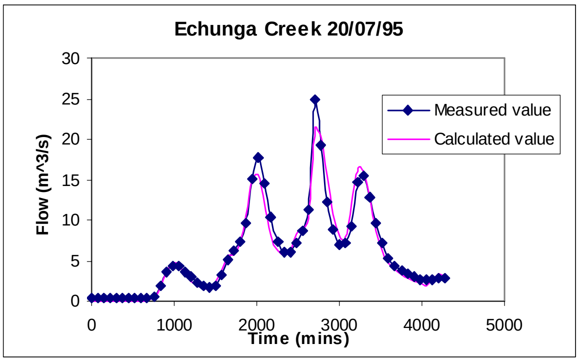




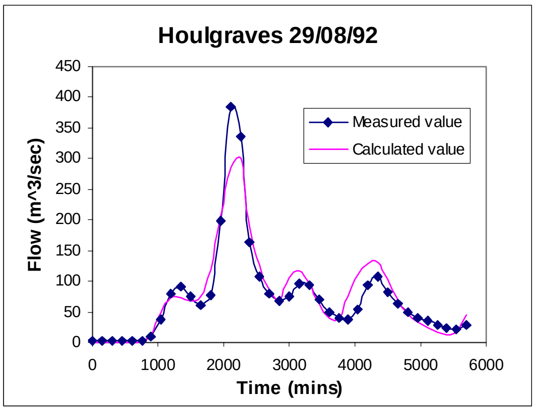


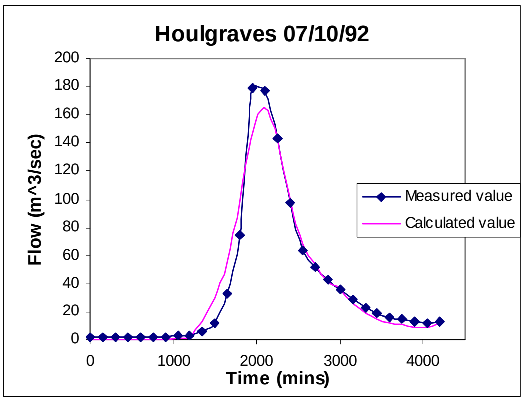


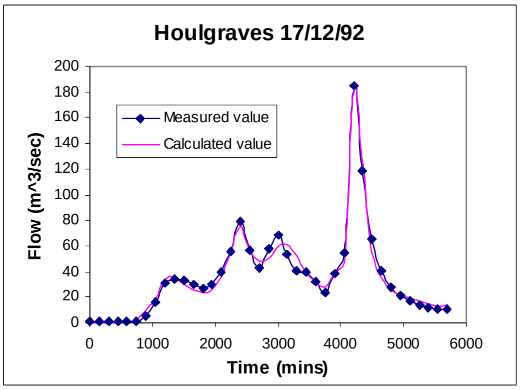


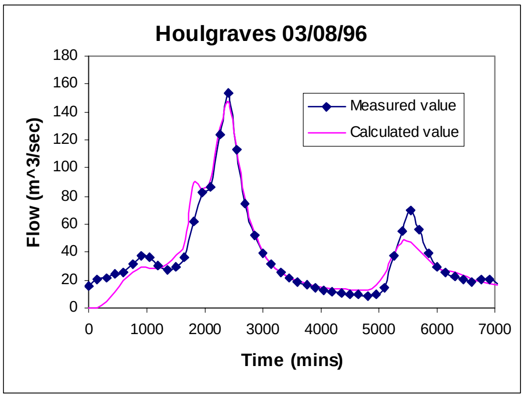


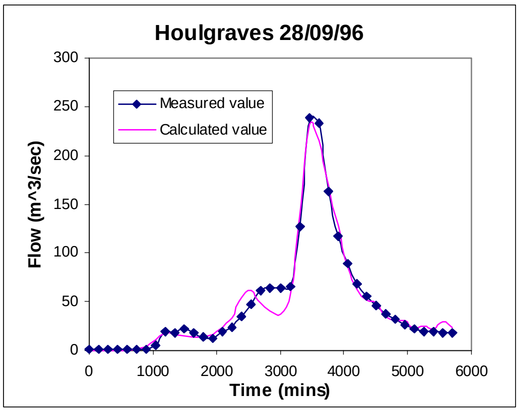
Houlgraves

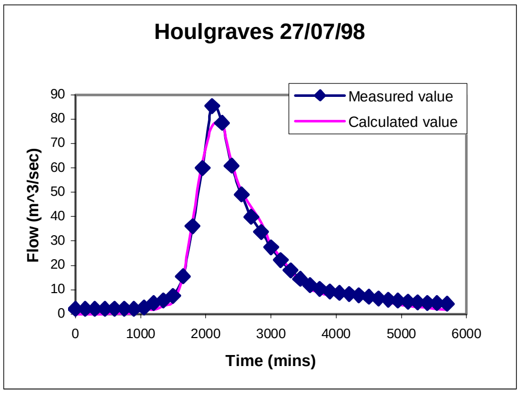

Houlgraves 29/08/92
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value

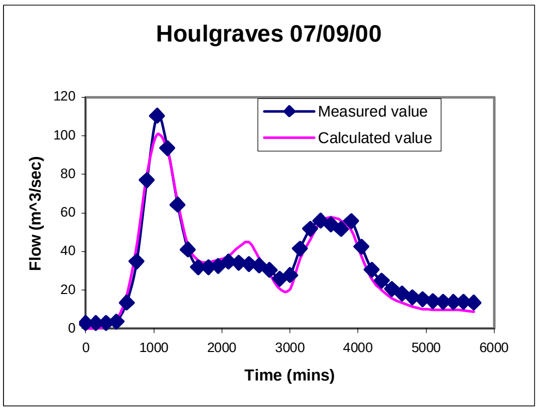

Houlgraves 07/10/92
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value


Houlgraves 17/12/92
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value


Houlgraves 03/08/96
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value


Houlgraves 28/09/96
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value


Houlgraves 27/07/98
Flow (m^3/sec)
Time (mins)
Measured value
Calculated value


Houlgraves 07/09/00
Measured value
Calculated value
Flow (m^3/sec)
Time (mins)
0
20
40
60
80
100
120
0
1000
2000
3000
4000
5000
6000

APPENDIX 5

PREDICTED FLOWS

Lobethal Model

Creek	Location	Easting	Northing	Area km²	Q10 (m³/sec)	Q20 (m³/sec)	Q50 (m³/sec)	Q100 (m³/sec)	Q200 (m³/sec)	Q500 (m³/sec)
East	Gumeracha Road	0612	3645	1.2	1.5	1.7	2	2.3	3.9	7.4
East	Pioneer Ave	0581	3602	1.53	1.9	2.2	2.6	3	4.9	9.4
West	Pioneer Ave	0560	3603	2.34	2.9	3.3	4	4.5	7.6	14.5
Lobethal	Post Office Road	0553	3485	4.65	5.9	6.9	8.1	9.2	14.5	27.3
Lobethal	Lenswood - Lobethal Road	0550	3432	5.16	6.3	7.4	8.7	10	15.7	29.7

Aldgate Creek Model

Creek	Location	Easting	Northing	Area km²	Q10 (m³/sec)	Q20 (m³/sec)	Q50 (m³/sec)	Q100 (m³/sec)	Q200 (m³/sec)	Q500 (m³/sec)
	Merrion Tce	9172	2380	2.35	3.9	4.7	5.8	6.7	9.7	15.5
	Gould Road	9217	2357	3.01	4.8	5.9	7.2	8.3	12.1	19.3
	Snows Road	9264	2365	4	6.1	7.4	8.9	10.4	15.6	23.7
	Kemp Road - East Branch	9294	2283	4.89	7.3	8.9	10.9	12.7	19.1	28.7
	u/s Milan Tce - south branch	9215	2252	1.27	2.2	2.7	3.2	3.7	5.4	9.0
	Mabel Street	9126	2304	0.56	0.98	1.2	1.4	1.7	2.4	4
	u/s Milan Tce - north branch	9214	2256	1.3	2.3	2.7	3.3	3.8	5.5	9.2
	u/s Milan Tce	9220	2260	2.57	4.5	5.4	6.6	7.5	10.8	18.4
	Kemp Road - West Branch	9290	2280	3.26	5.5	6.7	8.1	9.3	13.5	22.5
Aldgate	Gauging Station (AW503509)	9296	2278	8.15	12.8	15.6	19	21.9	32.6	51.1
Aldgate	Hampstead Hill Rd	9382	2230	9.23	14.9	18.2	22.4	26.0	37.1	57.4
Aldgate	Upstream Mylor	9502	2025	14.15	24.2	30	36.4	42.6	54.0	79.6
Aldgate	Strathalbyn Road	9564	2010	15.13	25.9	31.5	39	45.6	56.9	83.2
Leslie	Strathalbyn Road	9561	2006	2.58	3.8	4.8	6.0	7.0	8.3	11.1
Aldgate	d/s Leslie junction	9570	2005	17.71	29.6	36.2	44.9	52.6	65.1	93.5
Aldgate	u/s Onkaparinga	9555	1900	19.33	31.2	39.4	48.9	57.2	69.8	99.4
	Minor Branches	9261	2380	0.158	0.31	0.37	0.43	0.52	0.72	1.28
		9305	2413	0.172	0.34	0.40	0.47	0.56	0.78	1.39
		9311	2387	0.112	0.22	0.27	0.31	0.38	0.52	0.93
		9292	2396	0.373	0.69	0.84	0.98	1.17	1.64	2.84
		9267	2376	0.688	1.23	1.49	1.77	2.08	2.96	5.01
		9282	2397	0.232	0.44	0.53	0.63	0.75	1.04	1.83
		9280	2390	0.632	1.14	1.37	1.63	1.92	2.73	4.63

Woodside Model

Creek	Location	Easting	Northing	Area km^2	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)	Q200 (m^3/sec)	Q500 (m^3/sec)
Lenswood	Gauging Station (AW503907)	0010	3173	16.7	24.1	30.3	42.6	52.1	61.9	80.4
Western Branch	Gauging Station (AW503906)	0415	3060	24.2	26.7	32	38.7	45.1	67.2	109
Western Branch	u/s Onkaparinga	0305	2690	61.4	76.3	93.2	121.8	147	191	274
Inverbrackie	Onkaparinga Valley Rd	0522	2861	26.5	36	46.8	61.4	73.7	99.6	129
Charleston Tributary	Onkaparinga Valley Rd	0833	3424	4.33	4.6	5.7	7.6	11.3	18.2	23.8
Onkaparinga	Near Bell Ct, Charleston	0906	3528	28.3	27.8	34.8	44.2	52.7	72.4	96.3
Onkaparinga	just u/s Graebner Rd	0827	3436	31	30.1	37.4	48.6	58.7	80	106
Onkaparinga	u/s tributary	0738	3366	37	35.6	44.5	58.8	73.3	100	132
Tributary	u/s Onkaparinga	0730	3368	2.9	3.02	3.8	5.01	7.83	12.1	15.7
Onkaparinga	800m u/s Lobethal - Woodside Rd	0731	3363	39.9	38.1	47.6	63.8	80	109	144
Onkaparinga	near Naughton Road	0648	3198	44.6	42	52.2	71.6	89.7	123	161
Onkaparinga	Woodside GS (AW503903)	0565	3006	49	45.2	56.1	77.9	97.9	135	176
Onkaparinga	u/s Inverbrackie	0516	2865	50.9	46.5	57.7	80.3	101	139	182
Onkaparinga	d/s Inverbrackie	0510	2660	77.4	81.6	103.2	137.4	172	236	307
Onkaparinga	u/s Western Branch	0307	2682	97.9	105.4	133.4	175.6	218	300	393
Onkaparinga	Oakwood Road (AW503902)	0295	2583	159.3	179.1	219.5	294.5	361	491	665
Onkaparinga	Oval	0167	2593	161.8	181.6	227.7	298.4	366	497	672
Balhannah Branch	Balhannah	0165	2585	10.3	13.7	17.7	23.3	28.6	39.5	52
Onkaparinga	Balhannah	0155	2590	172.1	192.8	241.4	316.3	386	526	708
Onkaparinga	Verdun - Onkaparinga Valley Rd	9864	2375	205.5	224.3	280.5	364.3	444	603	812
Onkaparinga	Verdun - Mt Barker Rd	9837	2246	209.8	227.4	284.1	368.8	449	610	821
	Minor Branches	0475	2785	12.9	14.14	17.71	23.47	30.71	43.6	58.0
		0467	2765	1.33	1.49	1.88	2.50	3.56	5.3	7.0
		0455	2771	14.4	15.76	19.74	26.16	34.08	48.3	64.2
		0360	2640	0.96	1.08	1.36	1.81	2.61	4.0	5.2
		0350	2630	1.78	1.99	2.51	3.33	4.69	7.0	9.2
		0342	2666	2.8	3.12	3.92	5.20	7.21	10.6	14.0

Cox Creek Model

Creek	Location	Easting	Northing	Area km^2	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)	Q200 (m^3/sec)	Q500 (m^3/sec)
Cox	Gauging Station (AW503527)	9317	2740	4.3	5.2	6.1	7.3	9.3	12.0	17.2
Cox	Freeway	9468	2474	19.7	34.3	46.3	54.0	63.8	84.1	114
Cox	Mount Barker Road	9533	2340	23.4	40.9	50.8	64.3	76.7	98.5	132
Cox	Below Orphir Close	9597	2263	26	45.2	56	70.8	84.9	108.0	144
Cox	Above Onkaparinga River	9741	2125	29.2	50	61.6	77.7	93.5	117	157
	Minor Branches	9440	2283	0.082	0.17	0.20	0.23	0.28	0.38	0.70
		9428	2316	0.17	0.33	0.40	0.47	0.56	0.77	1.37
		9371	2340	0.14	0.28	0.33	0.39	0.46	0.64	1.15
		9427	2320	0.56	1.01	1.22	1.45	1.72	2.43	4.14
		9428	2321	0.73	1.30	1.57	1.87	2.20	3	5.30
		9492	2356	1.14	1.98	2.39	2.86	3.35	4.82	8.00
		9485	2416	1.43	2.44	2.96	3.55	4.15	5.99	9.87

Hahndorf Creek Model

Creek	Location	Easting	Northing	Area km^2	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)	Q200 (m^3/sec)	Q500 (m^3/sec)
Creek1	West end golf course	0055	2214	4.23	7.2	9	11.3	13.4	16.4	21.4
Creek1	Junction with Hahndorf Creek	9993	2180	4.51	8.1	10	12.5	15	18.3	24
Creek 2	English Street - northern creek	0084	2128	0.72	1.3	1.6	2	2.3	2.8	3.7
Creek 2	English Street - southern creek	0085	2125	0.18	0.32	0.4	0.5	0.6	0.7	0.95
Creek 2	Below junction	0080	2126	0.9	1.6	2	2.5	2.9	3.6	4.6
Creek 2	Downstream Church Street	0001	2169	1.28	2.2	2.7	3.4	4.1	5	6.5
Creek 3	East Windsor Ave near Valma Ave	0088	2070	2.5	4.4	5.3	6.8	8.1	9.7	12.8
Creek 3	Upstream junction	0020	2130	2.75	4.8	5.8	7.4	8.8	10.7	14
Pine Ave trib	Aauricht Rd	0013	2103	1.1	1.9	2.4	3	3.5	4.3	5.6
Creek 4	Near Windsor Ave	0054	2062	2.4	4.2	5.1	6.5	7.7	9.4	12.3
Creek 4	Upstream junction Pine Ave trib	0014	2105	2.58	4.4	5.5	7	8.3	10.1	13.2
Creek 4	Downstream Junction	0013	2109	3.68	6.3	7.8	10	11.8	14.4	18.8
Creek 4	Upsteam junction Creek 3	0018	2130	3.93	6.8	8.4	10.6	12.6	15.4	20.1
Hahndorf	Downstream junction Creek 3, 4	0017	2133	6.43	11.4	14.1	17.8	21.3	25.8	34
Hahndorf	Upstream junction Creek 2	0001	2167	6.86	12	14.9	18.8	22.5	27.3	36.1
Hahndorf	Downstream junction Creek 3	9998	2170	8.14	14.2	17.6	22.3	26.6	32.3	42.6
Hahndorf	Upstream junction Creek 1	9993	2177	8.39	14.7	18.2	23	27.4	33.3	43.9
Hahndorf	Downstream junction Creek 1	9990	2177	12.9	22.6	28	35.3	41.8	51.3	67
Hahndorf	Upstream junction Onkaparinga	9846	2231	14.77	25.6	31.8	40	47.7	58.2	76.4
	Minor Branches	9973	2215	0.47	0.53	0.67	0.90	1.33	2.0	2.7
		0003	2255	0.13	0.15	0.19	0.25	0.39	0.6	0.8
		0007	2253	0.14	0.16	0.20	0.27	0.42	0.7	0.9

Downstream Verdun Model

River	Location	Easting	Northing	Area km^2	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)	Q200 (m^3/sec)	Q500 (m^3/sec)
Onkaparinga	Downstream Mount Barker Rd	9832	2226	224.7	241	300	388	471	635	856
Onkaparinga	Upstream Cox Creek	9740	2120	228.9	244	304	392	477	641	864
Onkaparinga	Downstream Cox Creek	9735	2118	258.1	276	343	441	532	712	955
Onkaparinga	Downstream Aldgate Creek	9565	1885	283.3	296	366	471	567	756	1012
Onkaparinga	Houlgraves	9235	1535	320.2	322	397	508	609	805	1076

Kangarilla Model

Creek	Location	Easting	Northing	Area (km^2)	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)	Q200 (m^3/sec)	Q500 (m^3/sec)
		8700	0797	1.95	3.3	4.1	5.3	6.3	8	10.5
		8700	0778	21.9	33.2	41	52.5	62.1	79.7	104.1
		8688	0785	23.9	36.1	44.7	56.9	67.5	86.6	113.4

Echunga Model

Creek	Location	Easting	Northing	Area (km^2)	Q10 (m^3/sec)	Q20 (m^3/sec)	Q50 (m^3/sec)	Q100 (m^3/sec)	Q200 (m^3/sec)	Q500 (m^3/sec)
Echunga	Adelaide Road	9865	1354	3.0	3.1	3.7	5.2	7.5	9.6	12.7
	Meadows Road	9876	1259	1.3	1.35	1.6	2.3	3.5	4.3	5.7
		9903	1317	0.45	0.50	0.57	0.83	1.35	2	2.33

APPENDIX 6

CRC-FORGE RAINFALLS

Balhannah

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	113.2	133.1	156.8	195.6	232.0	275.8
30	120.5	141.6	166.9	208.2	247.0	294.0
36	126.8	149.0	175.6	219.1	259.9	309.8
48	137.4	161.4	190.3	237.6	281.8	336.5
60	145.5	170.6	200.9	250.6	297.4	356.1
72	152.5	178.5	210.0	261.9	310.9	372.9
96	161.0	188.4	221.3	275.9	327.8	394.1
120	166.5	194.9	228.6	285.1	338.9	408.3

Bridgewater

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	117.0	137.4	162.1	202.9	241.4	288.4
30	125.5	147.4	173.9	217.6	258.9	309.6
36	133.0	156.1	184.2	230.4	274.0	328.1
48	145.6	171.0	201.7	252.2	299.8	359.4
60	154.6	181.1	213.4	266.8	317.4	381.3
72	162.3	189.8	223.5	279.3	332.5	400.1
96	171.5	200.6	235.7	294.6	351.2	423.5
120	177.4	207.6	243.7	304.6	363.5	438.9

Crafers

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	131.9	155.0	183.3	231.0	276.8	333.2
30	143.4	168.5	199.4	251.3	301.2	362.8
36	153.6	180.5	213.7	269.2	322.8	388.9
48	171.1	201.2	238.2	300.1	360.0	434.0
60	183.4	215.1	254.1	319.8	383.6	462.8
72	194.0	227.1	267.9	336.9	404.0	487.7
96	206.8	242.0	284.8	357.6	428.8	517.8
120	215.2	251.8	295.8	371.0	444.6	537.2

Echunga

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	109.4	128.4	151.2	188.1	222.6	263.9
30	117.0	137.4	161.8	201.7	238.8	283.9
36	123.5	145.1	171.1	213.4	253.0	301.3
48	134.7	158.3	186.7	233.4	277.0	330.9
60	142.9	167.6	197.4	246.5	292.6	349.9
72	149.9	175.5	206.6	257.8	306.0	366.2
96	158.3	185.4	217.8	271.4	322.0	385.5
120	163.8	191.9	225.0	280.1	332.2	397.8

Hahndorf

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	111.0	130.4	153.6	191.4	226.8	269.4
30	118.2	138.8	163.6	204.0	241.9	287.8
36	124.3	146.1	172.2	214.9	254.9	303.8
48	134.8	158.4	186.8	233.3	276.9	331.0
60	142.8	167.5	197.2	246.2	292.3	349.9
72	149.7	175.3	206.2	257.2	305.4	366.2
96	158.0	185.0	217.3	270.9	321.8	386.4
120	163.5	191.4	224.5	279.7	332.4	399.6

Kangarilla

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	115.1	135.3	159.4	198.8	235.8	280.3
30	123.0	144.5	170.3	212.6	252.3	300.1
36	129.8	152.5	179.7	224.6	266.6	317.4
48	141.3	166.0	195.7	245.0	290.8	346.5
60	149.6	175.3	206.5	258.2	306.4	365.5
72	156.6	183.3	215.7	269.5	319.8	381.8
96	165.0	193.1	226.9	282.9	335.6	401.1
120	170.5	199.5	234.1	291.5	345.6	413.2

Lenswood

Duration (hrs)	1:50 (mm)	1:100 (mm)	1:200 (mm)	1:500 (mm)	1:1000 (mm)	1:2000 (mm)
24	112.0	131.6	155.0	192.9	228.0	270.1
30	119.2	140.0	164.8	205.2	242.6	287.7
36	125.4	147.3	173.4	215.8	255.2	302.9
48	135.8	159.5	187.7	233.8	276.4	328.5
60	143.7	168.5	198.2	246.7	291.8	347.7
72	150.6	176.2	207.1	257.7	305.0	364.1
96	158.9	185.9	218.2	271.5	321.5	385.1
120	164.3	192.3	225.5	280.6	332.5	399.1