Floodplain Development Strategy

Murray Drainage and Water Management Plan and Associated Studies

This report was prepared for the Department of Water

September 2010

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Executive Summary

Background and scope

The coastal plain south of Perth is predominantly rural-zoned land that is coming under increasing speculation for urban development. Structure planning is not yet complete and many places now being targeted by developers have environmental constraints. The area has relatively flat terrain, high water tables and surface water ponding in winter, wetlands of significance and experiences periodic riverine flooding.

A key step in the structure plan process is the preparation of a drainage and water management plan (DWMP) that embraces water sensitive urban design, best management practices and provides a framework for more site specific water management plans. The DWMP will provide guidance on how water quantity and quality can be managed to minimise any adverse impacts on the environment and to ensure that the proposed development of areas is sustainable.

This study has prepared the tools and recommended development strategies that will form the basis of the floodplain management component of the DWMP. This study supersedes previous work for the Murray River (PWD, 1984), Serpentine River (WAWA, 1991), Peel Inlet and Harvey Estuary (WAWA, 1985). This study uses the recent findings of the Serpentine River Flood Study (SKM, 2010) and the draft report on the Review of Available Information and Policies relating to development in flood prone areas for the Mandurah Region (Damara, 2009)

Catchment characteristics and history of flooding

The flood study area is traversed by the lower reaches of the Serpentine and Murray Rivers and bordered to the west by the Peel Inlet / Harvey Estuary. There are a number of other smaller rivers and streams that flow into or through the study area, including Nambeelup Brook; the Dandalup River system, including the North and South Dandalup Rivers; Conjurunup Creek; Oakley and Marrinup Brooks; and a number of small streams that enter the flood study area from the east and drain into the Murray River. In addition, there are many small drains on farmland within the flood study area, particularly in and south of the Nambeelup Brook catchment. These have generally been constructed by landholders to drain wetlands and low areas.

There are three major water supply dams operated by the Water Corporation on the Serpentine, North Dandalup and South Dandalup Rivers. The Water Corporation also operate small pipe head dams on the Serpentine River and Conjurunup Creek. Alcoa has a small reservoir and a number of storage ponds within the Oakley and Barritt Brook catchments, east of Pinjarra.

The Murray River, and its major tributaries the Hotham and Williams Rivers, is the largest of the catchments draining into the Peel Inlet / Harvey Estuary. The Murray River has a substantial floodplain, extending from south of Pinjarra to its mouth at the Peel Inlet. Peak flows in the Serpentine River are smaller than the Murray River due to its smaller catchment and the considerable storage in the river's floodplain as it traverses the coastal plain. Accordingly, its floodplain is restricted to close to the river itself and nearby low-lying lakes, including Goegrup and Black Lakes. The Serpentine River discharges into the Peel Inlet just north of the Murray River's mouth and the two rivers form a broad delta.

The Murray area is subject to relatively regular flooding events resulting from the passage of storm fronts through the catchment during the winter months. Events observed in June 1945, August 1964 and July 1996 are examples of major winter floods of this nature.

During the summer months major flooding may occur as result of significant rainfall volumes and intensities associated with rainfall from tropical origins, including the passage of an ex-tropical cyclone through the catchment. Floods in February 1955 and January 1982 are such examples of major summer floods related to the passage of ex-tropical systems.

The Peel Inlet / Harvey Estuary and the lower reaches of the Serpentine and Murray Rivers are also subject to flooding from major tidal and storm surge events. In May 2003 there was a major storm surge event that resulted in a peak water level within the estuary of approximately 1.0 m AHD.

Hydrologic assessments

A flood frequency analysis was undertaken on the available gauging station data for the streams that pass through the study area to determine flow estimates for a range of event probabilities. To account for the two differing rainfall mechanisms (winter frontal and summer ex-tropical lows) that are known to generate major floods in the catchments, a seasonal flood frequency approach was adopted. This approach involved determining flood frequency curves for summer and winter separately, then combining the curves to produce an annual curve. This analysis identified that the relatively infrequent but significant summer events affect the design peak flow estimates for events of 1 in 100 AEP and larger on the Murray River.

A frequency analysis on the annual gauged maximum event volumes was undertaken to assist in the determination of runoff coefficients within the Murray River catchment.

Runoff routing models were developed for the Murray River and a number of catchments along the eastern side of the flood study area (the Hills Catchments). Parameters in the Murray River model were derived from calibration against data from observed events and predictions for design events verified against the flood frequency data. The RORB models of the Hills Catchments were parameterised using a regional method (Water Corporation, 2006) and verified using available streamflow data.

The results of a recent flood study for the Serpentine River (SKM, 2010) have been extracted and applied within the modelling of the Murray DWMP study area.

The design peak discharges for the Murray and Serpentine Rivers are lower than the previous estimates due to the different methodologies employed and the additional years of gauged record available (Table E1).

Table E1 Comparison of estimated 1 in 100 AEP peak flows for the Murray and Serpentine Rivers with previous estimates (m^3/s)

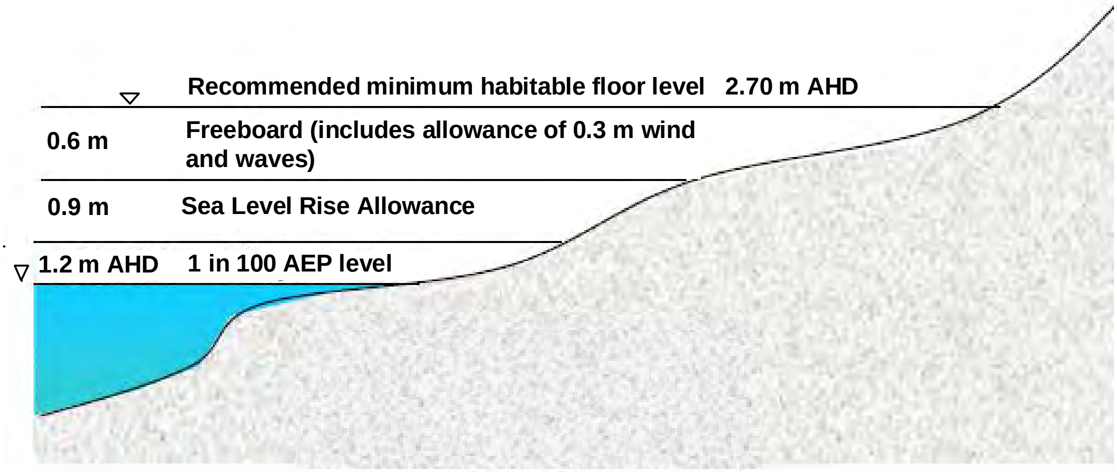
Location	This study	Murray River Flood Study (PWD, 1984)	Serpentine River Flood Study (WAWA, 1991)
Murray River at Baden Powell gauging station	1030	1250	-
Murray River at Pinjarra	830	1273	-
Serpentine River at Lakes Road	120	-	270

With regard to tidal flooding, Damara (2009) compared the peak water levels within the Peel Inlet / Harvey Estuary with the long term record available for Fremantle and undertook a frequency analysis on the data. The analyses indicate that the design 1 in 100 AEP peak water level within the Peel Inlet / Harvey Estuary is approximately 1.20 m AHD.

A hydrodynamic model of the Peel Inlet / Harvey Estuary was also prepared using bathymetry data (both with and without the Dawesville Channel) and calibrated using the observed tide and wind information. The model was used to simulate tide and storm surge levels in the estuary near the Murray River mouth for a number of design event scenarios and these formed a boundary condition for the riverine model. Applying the estimated design 1 in 100 AEP storm surge centred on the average high tide (Mean High High Water, 0.17 m AHD) gave consistent results to the analyses of Damara (2009). The modelling was then used to investigate the sensitivity of peak water levels in the Peel Inlet / Harvey Estuary should the estimated surge occur at Highest Astronomical Tide (0.42 m AHD) and coincident with major river flooding.

Based on a recent decision of the Western Australian Planning Commission (WAPC Minutes of Ordinary Meeting 173, Item 12.3, 25 May 2010), an allowance of 0.90 metre for sea level rise by the year 2110 has been adopted for establishing the peak still water level for planning. The hydrodynamic modelling results suggest the impact of sea level rise is applicable to the design water levels in the Peel Inlet / Harvey Estuary. An additional freeboard of 0.6 metre is recommended to allow for the impact of wind, waves and run up on the surface of Peel Inlet / Harvey Estuary. This information is summarised in Figure E1.

Figure E1 Components of the design inundation level for the Peel Inlet / Harvey Estuary



Floodplain mapping

A two-dimensional hydraulic model of the flood study area was developed based on detailed ground survey (Lidar) data and incorporates the hydrology and tide data as inputs. Observed flood levels for a number of historic events were used to calibrate the model and verify flood level predictions. Runoff generation within the flood study area was simulated within the hydraulic model using a regional method (Water Corporation, 2006) and verified against available streamflow data. The calibrated model was then used to produce flood extent, depth, velocity and flood hazard mapping for a range of design events.

The existing floodway and flood fringe mapping for the Murray and Serpentine Rivers was reviewed based on the new 1 in 100 AEP floodplain mapping results and detailed survey information (Figure E2). The reduction in design peak flows has resulted in minimal reductions to the Murray (PWD, 1984) and Serpentine River (WAWA, 1991) defined floodways. There are no areas where the floodway has been increased within the Murray and Serpentine River floodplains.

Regional scale floodplain mapping for the North and South Dandalup Rivers, Nambeelup Brook and other smaller tributaries that pass through the study area has also been prepared. The mapping provides indicative information on flood extent and levels to assist land use planning. In addition, peak discharge information for a number of locations within the study area has been determined.

Figure E2 1 in 100 AEP floodplain mapping for the Murray Area

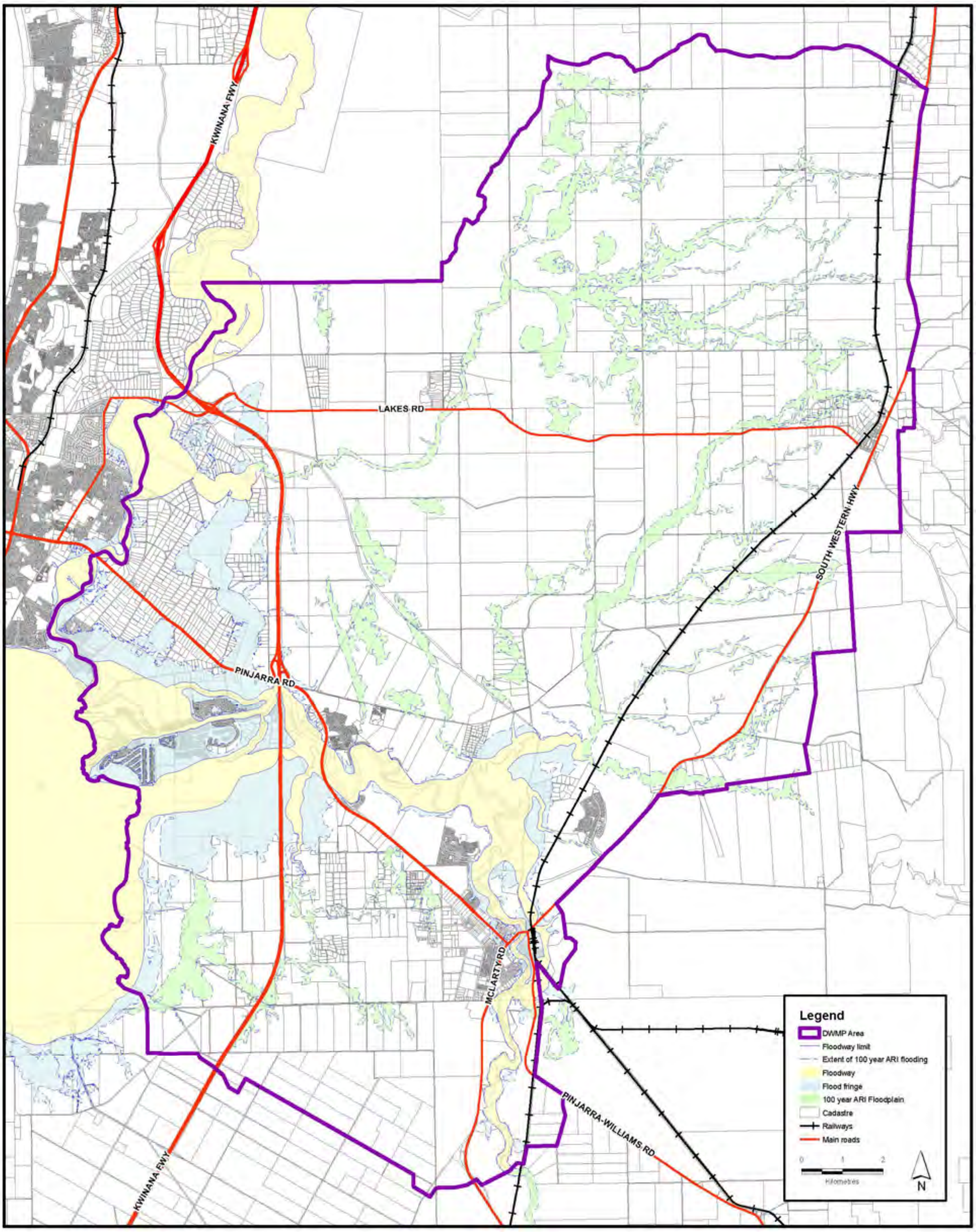


Figure prepared by the Department of Water.

Floodplain Development Strategy

The following general principles were used in developing the recommended floodplain development strategy:

- Proposed development has an adequate level of flood protection.
- Proposed development does not detrimentally impact on the existing flooding regime of the general area.
- The public has adequate protection from flood hazard (e.g. flow depth and velocity, frequency and duration of overtopping of road crossings).

A broad range of flood mitigation options, including both structural and non-structural measures, were considered in preparing the recommended floodplain management strategy for the Murray area.

There are currently no broad-scale, structural mitigation measures within the flood study area. While there has been discussion of channel/diversion works for breakout flows from the Murray River just south of Pinjarra, no definite plans have been developed. This study has found that the breakout does not become active until the flows exceed the 1 in 100 AEP event. There are numerous constructed drains within the area which reduce ponding in low-lying areas.

Existing floodplain development strategies (PWD, 1984 and WAWA, 1991) promote the use of non-structural measures and are based around appropriate land use planning and building and development controls. These form the basis of the existing Peel Region Scheme Floodplain Management Policy (WAPC, 2002). No change to this strategy for the Peel Inlet / Harvey Estuary and Serpentine River and Murray River floodplains is recommended except that reference should be made to the results of the revised 1 in 100 AEP floodplain mapping from this study.

In order to conform with the floodplain development strategy and utilising the revised floodway and flood fringe mapping for the Murray and Serpentine Rivers and Peel Inlet / Harvey Estuary, the following are recommended building and development controls:

- Proposed development (i.e. filling, building, etc) that is located outside of the floodway is considered acceptable with respect to major flooding. However, a minimum habitable floor level of 0.50 m above the adjacent 1 in 100 AEP flood level is recommended to ensure adequate flood protection.

  However, in addressing other planning issues (such as aesthetic, streetscaping and privacy issues associated with possible high fill levels) for new dwellings in existing subdivided / developed areas, a reduction in the freeboard from 0.50 m to 0.15 m above the 1 in 100 AEP flood level can be considered.

  Similar consideration should also be given to proposed commercial properties.

- Proposed development (i.e. filling, building, etc) that is located outside of the floodway and adjacent to the Peel Inlet / Harvey Estuary is considered acceptable with regard to major flooding. The 1 in 100 AEP flood level for this area is estimated to be 2.10 m AHD and this includes a 0.90 m sea level rise projection over the next century. A freeboard of 0.60 metre above the 1 in 100 AEP flood level is recommended to account for factors such as the result of wind and wave action and local variations in flood levels. Consequently, a minimum habitable floor level of 2.70 m AHD for proposed development is recommended to ensure adequate flood protection.

  However, in addressing other planning issues (such as aesthetic, streetscaping and privacy issues associated with possible high fill levels) for new dwellings in existing subdivided / developed areas, a reduction in the freeboard from 0.60 m to 0.15 m above the 1 in 100 AEP flood level can be considered, i.e. a minimum habitable floor level of 2.25 m AHD.

- Proposed development (i.e. filling, building, etc) that is located within the floodway and is considered obstructive to major flows is not acceptable as it would as it would detrimentally impact on the existing flooding regime of the general area. No new dwellings are acceptable within the floodway.

  However, in certain circumstances, proposed development within the floodway may be considered based on its merit. The major factors that should be considered include depth of flooding, velocity of flow, its obstructive effects on flow, possible structural and potential flood damage, and difficulty in evacuation during major floods and its regional benefit. These circumstances include the re-development of an existing dwelling, public works (i.e. bridges) or community facilities (i.e. picnic facilities) that are considered of significant regional benefit.

  The replacement of an existing dwelling within the floodway may be considered provided:

  - existing dwelling is demolished or relocated.
  - effective width of obstruction of the new dwelling to major flows is no greater than the effective width of the existing dwelling.
  - proposed dwelling achieves a minimum habitable floor level of at least 0.50 m above the 1 in 100 AEP flood level.

  It is the responsibility of the proponent to undertake the necessary reviews, assessments and modelling to demonstrate, to the satisfaction of the Department of Water, that proposed development within the floodway is consistent with the floodplain management principles.

- Regional scale floodplain mapping for the North and South Dandalup Rivers, Nambeelup Brook and other smaller tributaries that pass through the study area has also been prepared. The mapping provides indicative information on flood extent and levels to assist land use planning. In addition, peak discharge information for a number of locations within the study area has been determined.

  For watercourses where the 1 in 100 AEP floodplain mapping has not been delineated into floodway and flood fringe areas, proposed development that is located within the 1 in 100 AEP floodplain will be assessed on a case-by case basis until such time as detailed structure planning that considers major flooding has been undertaken for the area.

  It is the responsibility of the proponent to undertake the necessary reviews, assessments and modelling to demonstrate, to the satisfaction of the Department of Water, that proposed development within the floodplain is consistent with the floodplain management principles.

Flood emergency planning should focus on the development and maintenance of emergency response and recovery plans, maintain and improve existing flood prediction and warning systems and invest further in community awareness and preparation

The study has identified that a number of population centres may be affected during major flooding as a result of inundation of properties or access roads to the centres.

The floodplain mapping and development strategy outlined in this study provides guidance for future planning and development decisions in accordance with the Better Urban Water Management framework (WAPC, 2008).

Review

This study and recommended strategy has required the development of detailed hydrologic and hydraulic modelling tools. These tools should become the basis for assessment of future proposals for development with the floodplain. The Department of Water will be the custodian of the modelling tools and will maintain the hydraulic model and information from this study.

The strategy and tools developed as part of this study should be reviewed following any large flood event, as more detail on expected climate change impacts become available or after a period of time no greater than 5 years.

1. Introduction

1.1 Background

The coastal plain south of Perth is predominantly rural zoned land that is coming under increasing speculation for urban development. Structure planning is not yet complete and many places now being targeted by developers have environmental constraints. The area has relatively flat terrain, high water tables and surface ponding in winter, wetlands of significance and experiences periodic riverine flooding.

The Western Australian Planning Commission in consultation with local government authorities have identified the area as a high priority for structure planning, which will provide guidance for future development and management of environmental issues.

A key step in the structure plan process is the preparation of a drainage and water management plan (DWMP) that embraces water sensitive urban design, best management practices and provides a framework for more site specific water management plans. The DWMP will provide guidance on how water quantity and quality can be managed to minimise any adverse impacts on the environment and to ensure that the development of areas identified for development is sustainable.

The DWMP study area extends from the Nambeelup Brook catchment in the north to the Fauntleroy Drain catchment in the south and from the Lower Serpentine River and Peel-Harvey Estuary in the west to the Murray River and Darling Range foothills in the east.

1.2 Study scope

The floodplain development study will contribute to the DWMP by defining the flood prone land and outline a recommended strategy for development of the flood prone land. The DWMP includes the following major components:

- Floodplain development study, including inundation mapping and local catchment stormwater modelling.
- Groundwater studies, including regional pre-development groundwater levels, climate impacts and extent of current waterlogged areas.
- Preparation of the DWMP for the Murray DWMP study area.
- Development of detailed stormwater drainage strategies for selected areas of proposed development as required.

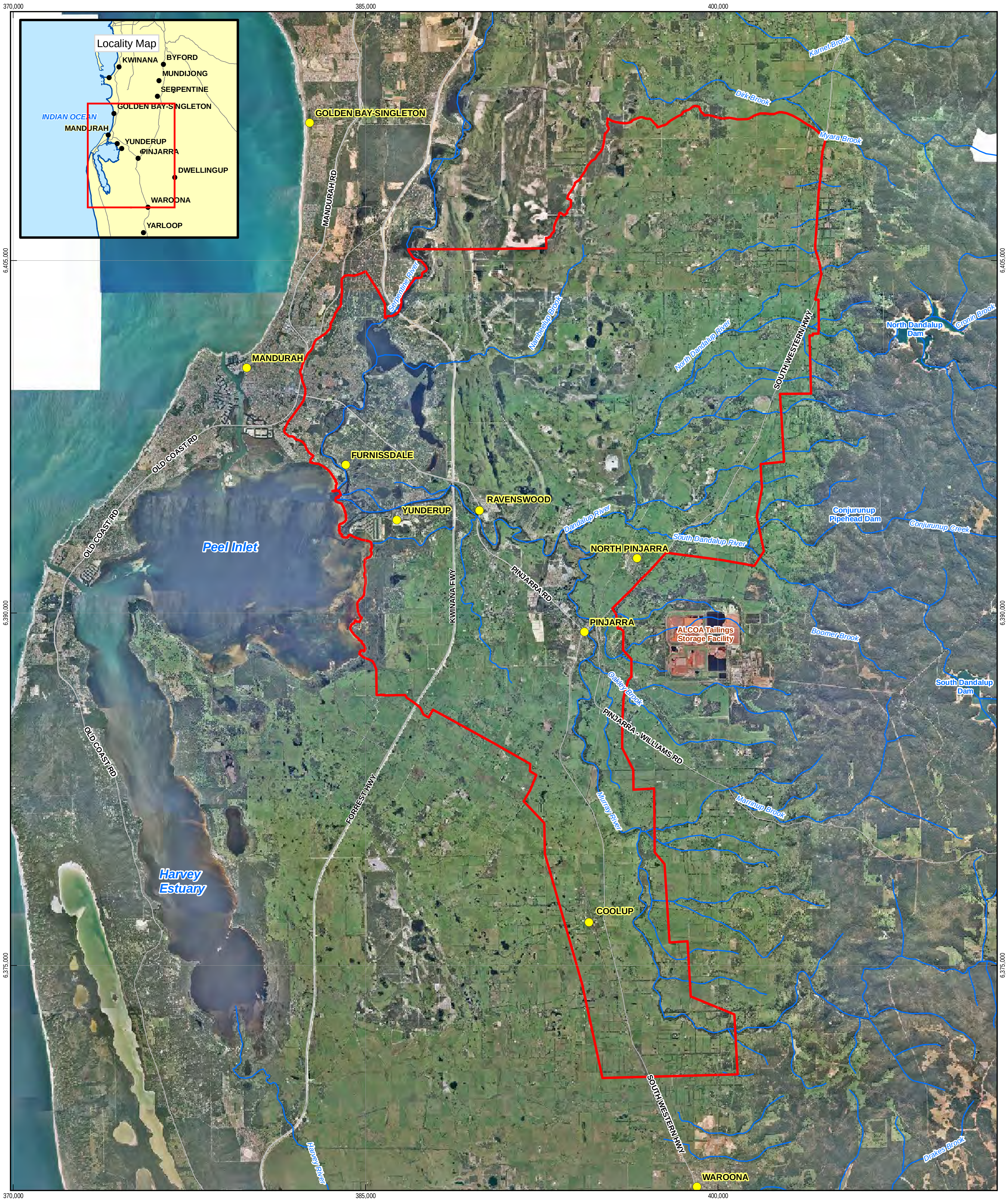
The floodplain development study has the following components:

1. Catchment hydrology.
2. Review of available tidal/storm surge water level information.
3. Floodplain mapping of watercourses within the study area.
4. Floodplain development strategy.

The flood study area (Figure 1) includes the lower Murray River, the lower Serpentine River, Peel-Harvey Estuary, Mandurah Channel and Dawesville Channel, Nambeelup Brook, North Dandalup and South

Dandalup Rivers and numerous watercourses and drains. The flood study area extends further south than the DWMP, following the Murray River in its course along the coastal plain.

This report presents the floodplain development strategy, including a summary of the catchment hydrology, tide/storm surge and floodplain mapping studies. The other aspects of the DWMP study are described separately to this report.



LEGEND

- ● Locality
- ▭ Flood Study Area
- — Watercourse
- — Roads

1:150,000 (at A3)

1,500 750 0 1,500 3,000 4,500

Metres

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Government of Western Australia
Department of Water

Department of Water
Murray Drainage and Water Management Plan
and Associated Studies

Job Number	61-2393701
Revision	2
Date	12 FEB 2010

Flood Study Area

Figure 1

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Data Source: Geoscience Australia: WatercourseLines - 200605, Lakes - 200605; MRWA: Roads - 20090409; DoW: Murray Drainage Study Area - 20090105; Landgate: Swan Coastal Plain 2008 Mosaic - 2008. Created by: croach2, mludovico, wdavis, xntan

1.3 Flooding mechanisms

The study area is located in an area that is subject to relatively regular flooding events resulting from the passage fronts through the catchment during the winter months. Events observed in June 1945, August 1964 and July 1996 are examples of major winter floods of this nature.

During the summer months major flooding may occur as result of significant rainfall volumes and intensities associated with rainfall from tropical origins, including the passage of an ex-tropical cyclone through the catchment. Floods in February 1955 and January 1982 are examples of major summer floods related to the passage of ex-tropical systems through the catchment.

The Peel-Harvey Estuary and the lower reaches of the Serpentine and Murray Rivers are also subject to flooding from major tidal and storm surge events.

1.4 Limitations

GHD recognises the potential for third party liability and seeks to limit our exposure as far as reasonable. GHD requires that appropriate limitations/disclaimers relating to flood extent mapping and other results from the study are included with the maps, reports and digital data generated in this project in the future. These limitations/disclaimers include but are not limited to the following:

- Explanation of probability and the chance of a flood of a certain size being received.
- Factors that affect the accuracy of predicted extent of flooding.
- Chance of floods occurring greater than the mapped/design size.
- Effects of climate change and extreme events on design predictions.
- Flood extent/predictions coverage/validity; effects of local conditions, blockages, etc.
- Incidence of coincident flooding downstream; shallow flooding.
- Reference to reports/studies/supporting information.
- Data sources, data accuracy and checking.
- No warranty that individual properties will not be flooded.

In addition, to the maximum extent permitted by law, any liability of GHD for loss or damage, whether arising under or in connection with or for breach of this contract, or in connection with the Services, whether such liability arises in contract, in tort (including negligence), under statute or otherwise, and whether arising in connection with one or more events, is limited to $2,000,000 (two million dollars). To protect both the Department of Water and GHD we recommend that, when investigation outputs (e.g. flood extents, model outputs, etc) are used, adequate reference is made to the main report and the qualifications and explanations that it contains. We insist on such referencing whenever the work is attributable to GHD through the use of our logo, name or otherwise.

This report has been prepared for the Department of Water for use in preparation of the Murray Drainage and Water Management Plan.

1.5 Definitions and assumptions

The following terms to describe probability were adopted, consistent with terminologies presented in Pilgrim (2001):

- Average Recurrence Interval (ARI) – to describe rainfall that has been generated mainly from a partial series analysis of rainfall data.
- Average Recurrence Interval (ARI) – to describe results of the tide and storm surge study.
- Annual Exceedance Probability (AEP) – to describe streamflow that has been generated mainly from an annual series analysis of streamflow data.

It is acknowledged that the source of rainfall and streamflow estimates may vary but these terms were adopted for consistency.

2. Catchments and data

2.1 Introduction and background

This section summarises past studies and information, describes the general characteristics of catchments contributing to the flood study area and presents key streamflow and rainfall data that were used in the preparation and calibration of the various models developed for the study.

The flood study area and contributing catchments cover a large area, some 10,000 km^2. There is a large body of information available describing the hydrology of the area and a number of streamflow and rainfall stations are located within the catchments. Not all of the information from these sites is suitable for direct use in this study; this section presents the data that were used.

2.2 Prior studies

A number of prior studies have been completed in the study area. The most relevant are reviewed below.

2.2.1 1984 Public Works Department Murray River flood study

A flood study for the Murray River was undertaken by the Public Works Department in 1984 (PWDWA 1984). This study mapped flood levels along the Murray River from its mouth at the Peel Inlet to Coolup Bridge. Flood levels along the Murray River floodplain were modelled using the one-dimensional models IRWASP and AFLUXM based on scaled hydrographs calculated from a flood frequency analysis at gauging station 614006 Murray River Baden Powell.

A log normal distribution fitted to an annual maximum flow distribution was selected for the flood frequency analysis of the Baden Powell station data. A total of 39 years of data (1940-1978) were available at the time. The adopted design peak discharges, including data scaled from Baden Powell to downstream locations, are shown in Table 1.

The study was undertaken before the Dawesville Channel, which connects the Harvey Estuary with the ocean, was constructed and so assumes only one exit from the estuary in the simulations. Effects of a proposed new highway (the recently constructed Kwinana Freeway and Forrest Highway) were considered.

The report documents flooding of Pinjarra associated with various floods. The June 1945 flood was modelled and a flood extent mapped. A flood in 1926 was reported to cause little damage. The flood of July 1862 entirely flooded Pinjarra, overtopped bridges and St John’s Church had several feet of water in it.

This study mapped flood extent for the 1 in 100 AEP flood and provides long sections for 1 in 10, 20, 25, 50 and 100 AEP floods and 1945, 1964 and 1982 events.

The study delineated floodways, based on the 1 in 100 AEP modelling, which have been used to assist planning and development on the Murray River floodplain to the present day. The report suggests that floodwaters would affect Pinjarra in floods greater than a 1 in 20 AEP and that larger floods are generally contained within the main Murray River channel upstream of Blythewood. The study estimated that about 20% of the peak 1 in 100 AEP flow at Pinjarra passed through the Tate Gully floodway.

Table 1 Peak discharge estimates from prior studies

Probability (%)	AEP (1 in *Y*)	Peak discharge 614006 Murray River Baden Powell (m^3/s)	Peak discharge Murray River at Pinjarra (m^3/s)	Peak discharge Murray River at Peel Inlet (m^3/s)
1984 Public Works Department:				
50	**2**	**115**	**-**	**-**
10	**10**	**430**	**438**	**476**
4	**25**	**690**	**703**	**763**
2	**50**	**960**	**978**	**1,062**
1	**100**	**1,250**	**1,273**	**1,382**
0.1	**1,000**	**2,763**	**-**	**-**
1999 SKM flood study:				
1	**100**	**1,013**	**930**	**940**

2.2.2 1990 Water Authority of Western Australia Revised Lower Serpentine Flood Study

A flood study for the Serpentine River catchment was undertaken in 1990 by the Water Authority of WA (1990). This study considered the entire Serpentine River catchment, including the area above the Serpentine Dam. The studied provided the following information:

- Flood frequency analyses.
- Runoff routing modelling (RORB).
- Design flood estimation.

The study was reviewed as part of the Serpentine River Floodplain Management Study by SKM (in prep).

2.2.3 1991 Serpentine River Flood Study (Water Authority of Western Australia)

Floodplain mapping for the 1 in 100 year AEP event in the lower reaches of the Serpentine River, within the Shire of Murray, was undertaken by the Water Authority of WA in 1991. Based on the design flow estimates in Water Authority of WA (1990), flood levels along the Serpentine River floodplain were modelled using cross-section information within one-dimensional models IRWASP and AFLUXM. The 100 year ARI floodplain mapping was based on 1 metre interval contour information. A floodplain development strategy was developed based on the floodplain mapping.

2.2.4 1999 SKM Murray River flood study

The flood mapping and hydrology for the 1 in 100 AEP event developed in the 1984 Public Works Department was reviewed by SKM in 1999 (SKM 1999) for the Shire of Murray. Flood hydrographs for the whole Murray River were simulated using the RORB model, calibrated to streamflow data from Station 614006 Murray River at Baden Powell.

A summary of revised peak discharges is given in Table 1. The revised 1 in 100 AEP peak flow at Pinjarra and the Peel Estuary were found to be are some 30% lower than the 1984 estimates. The revised flood frequency analysis of the Baden Powell gauging station data, which included an additional 20 years of data, gave a 1 in 100 AEP peak 20% lower than the 1984 estimate.

Peak water levels along the Murray River were modelled using the HEC-RAS model. Predicted flood levels using the revised peak discharge are 0.3 m lower than the 1984 study level at Pinjarra Bridge and approximately 1 metre lower immediately south of the Pinjarra town site.

2.2.5 2002 WAPC Peel Region Scheme Floodplain Management Policy

The Peel Region Scheme Floodplain Management Policy provides guidance on appropriate land use and development within floodplains. Its objectives include minimising damage during major flooding and maintaining the flood carrying capacity of floodplains. More details of the policy are given in Section 7.2.

2.2.6 2004 CRC Forge Estimation for Western Australia

The WA CRC-FORGE Extract is a rainfall database and computer program developed to facilitate the extraction of statistics for large to rare rainfall events for Western Australia. The program will allow generation of design rainfall totals and areally reduced totals for AEP's from 1 in 20 to 2000 and durations from 24 to 120 hours at any point in Western Australia. The program and analysis used are described in DoE (2004).

2.2.7 2006 Water Corporation small dams study

The Water Corporation of WA has developed a method for predicting flood hydrographs for small catchments in the South West of Western Australia (Pearce 2006). The method is applicable for events between a 50-year average recurrence interval (ARI) and the probable maximum flood (PMF) and for catchment areas between 1 and 100 km^2. The method is based on the RORB runoff routing model with the model parameters k_c and runoff coefficient, C, estimated using regional relationships with catchment area and rainfall amount respectively.

This method was used in the Murray flood study to help derive RORB parameters for the smaller, ungauged catchments contributing to the flood study area.

2.2.8 2009 Shire of Boddington Floodplain Management Study

A flood study for the Hotham River within the Shire of Boddington was undertaken for the Shire by SKM in 2009. This study investigated the catchment hydrology, using both a flood frequency and runoff routing modelling approach and prepared floodplain mapping of the 1 in 10, 25 and 100 AEP events. A floodplain development strategy was developed based on the mapping.

2.2.9 SKM Lower Serpentine flood study (in preparation)

Sinclair Knight Merz Pty. Ltd. (SKM) are currently finalising a flood study for the Lower Serpentine River through the Serpentine, Karnup and Keralup areas. This study provides design flow estimates and floodplain mapping for the Serpentine River between Lakelands Road and South West Highway. The downstream boundary of the study abuts the Murray flood study boundary at Yalbanberup Pool on the Serpentine River.

A major event in July 1987 was used to calibrate the model and design flood estimates for 1 in 10, 25, 100 and 500 AEP events are presented. The outflows from the Lower Serpentine River Flood Study were used as input to the Murray flood study; these data are descried in Section 4.4.

2.3 Catchments and drainage

The Peel-Harvey Estuary lies at the confluence of three major rivers – the Serpentine, Murray and Harvey. The catchments for these rivers are shown in Figure 2. Drainage features within the flood study area and near surrounds are shown in Figure 3.

The Harvey River enters the southern end of the Harvey Estuary and the river itself does not lie in the flood study area.

The flood study area is traversed by the lower reaches of the Serpentine and Murray Rivers. The Murray River, and its major tributaries the Hotham and Williams Rivers, is the largest of the catchments draining into the Peel-Harvey Estuary. The Murray River has a substantial floodplain, extending from south of Pinjarra to the Peel Inlet.

Flows in the Serpentine River are smaller than the Murray due to its smaller catchment and considerable storage in the river's floodplain as it traverses the coastal plain. Accordingly, the floodplain in the Murray flood study area is restricted to close to the river itself and nearby low-lying lakes, including Goegrup and Black Lakes. The river discharges into the Peel Inlet just north of the Murray River's mouth and the two rivers form a broad delta.

There are a number of other smaller rivers and streams that flow into or through the study area, including:

- Nambeelup Brook, which is almost fully enclosed in the flood study area and drains into the Serpentine River.
- The Dandalup River system, including the North and South Dandalup Rivers which enter the flood study area from the east, draining into the Murray River north of Pinjarra.
- Conjurunup Creek, which is part of the Dandalup River system, enters the flood study area from the east and drains into the South Dandalup River.
- Oakley and Marrinup Brooks and a number of small streams that enter the flood study area from the east and drain into the Murray River.

Characteristics of these catchments are summarised in Table 2.

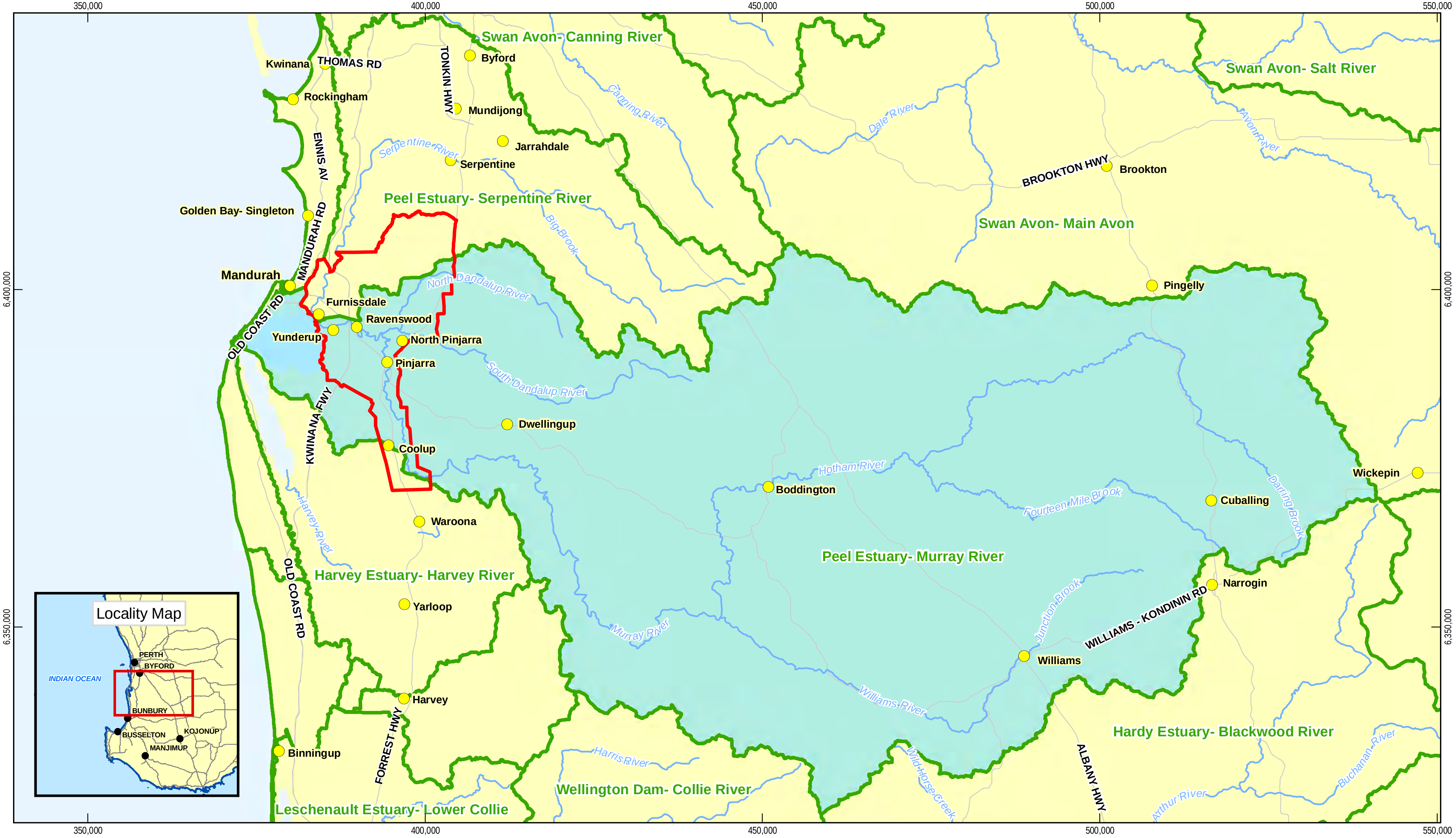
There are many small drains on farmland within the flood study area, particularly in and south of the Nambeelup Brook catchment. These have generally been constructed by landholders to drain wetlands and ponded areas.

Two Water Corporation main drains - Greenlands and Fauntelroys - traverse the lower flood study area south west of Pinjarra, discharging toward the Peel Inlet. Buchanans Drain flows into the Murray River at Ravenswood.

There are three major water supply dams operated by the Water Corporation on the Serpentine, North Dandalup and South Dandalup Rivers. The Water Corporation also operates small pipe head dams on Conjurunup Creek and on the Serpentine River. The Alcoa site, just east of Pinjarra, has tailings and water supply storage facilities, which have some storage capacity for direct rainfall. The impact of these storages on catchment runoff was investigated and is described in Section 4.3.2.

Table 2 Summary of major catchments

Catchment	Area (km^2)	Description
Hotham River at the Murray River confluence	4,301	Major tributary to the Murray River. The Maradong Bridge gauging station on the Hotham River (Station 614224) has a catchment area of 3,967 km^2.
Williams River at the Murray River confluence	1,436	Major tributary to the Murray River. The streamflow gauging station at Saddleback Road Bridge (Station 614196) has a catchment area of 1,408 km^2.
Murray River at Baden Powell streamflow gauging station (614006)	6,758	Key location to assist the determination of design flows for the Murray River.
Murray River at flood study boundary	6,867	Some 15 km downstream of the Baden Powell gauging station
Murray River at the Peel Inlet	8,117	The entire Murray River, including the Hills Catchments.
Serpentine River at Serpentine Dam	664	SKM (in prep.) found that this catchment did not contribute to flooding downstream.
Serpentine River at flood study boundary	810	Some 11 km upstream of the Peel Inlet and including the area above Serpentine Dam.
Serpentine River at Peel Inlet	1,684	The entire Serpentine River (including above Serpentine Dam). The catchment area below the dam is approximately 1,020 km^2.
North Dandalup River at North Dandalup Dam	151	Catchment above the dam.
South Dandalup River at South Dandalup Dam	313	Catchment above the dam.
Dandalup River at the confluence with the Murray River	706	Including the catchments above the dams on the North and South Dandalup Rivers. The catchment area below the dam is 242 km^2.
Marrinup Brook at the Murray River	78	A streamflow gauging station (614003) monitors flow from the upper catchment (45 km^2).
Nambeelup Brook at the Serpentine River	119	Largely contained within the flood study area. The catchment area to the Kielmann streamflow gauging station (614063) is 115 km^2



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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

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LEGEND

- 🟡 Locality
- Watercourse
- Roads
- 🟥 Flood Study Area
- 🟩 Hydrographic Catchments

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River Catchments

Figure 2

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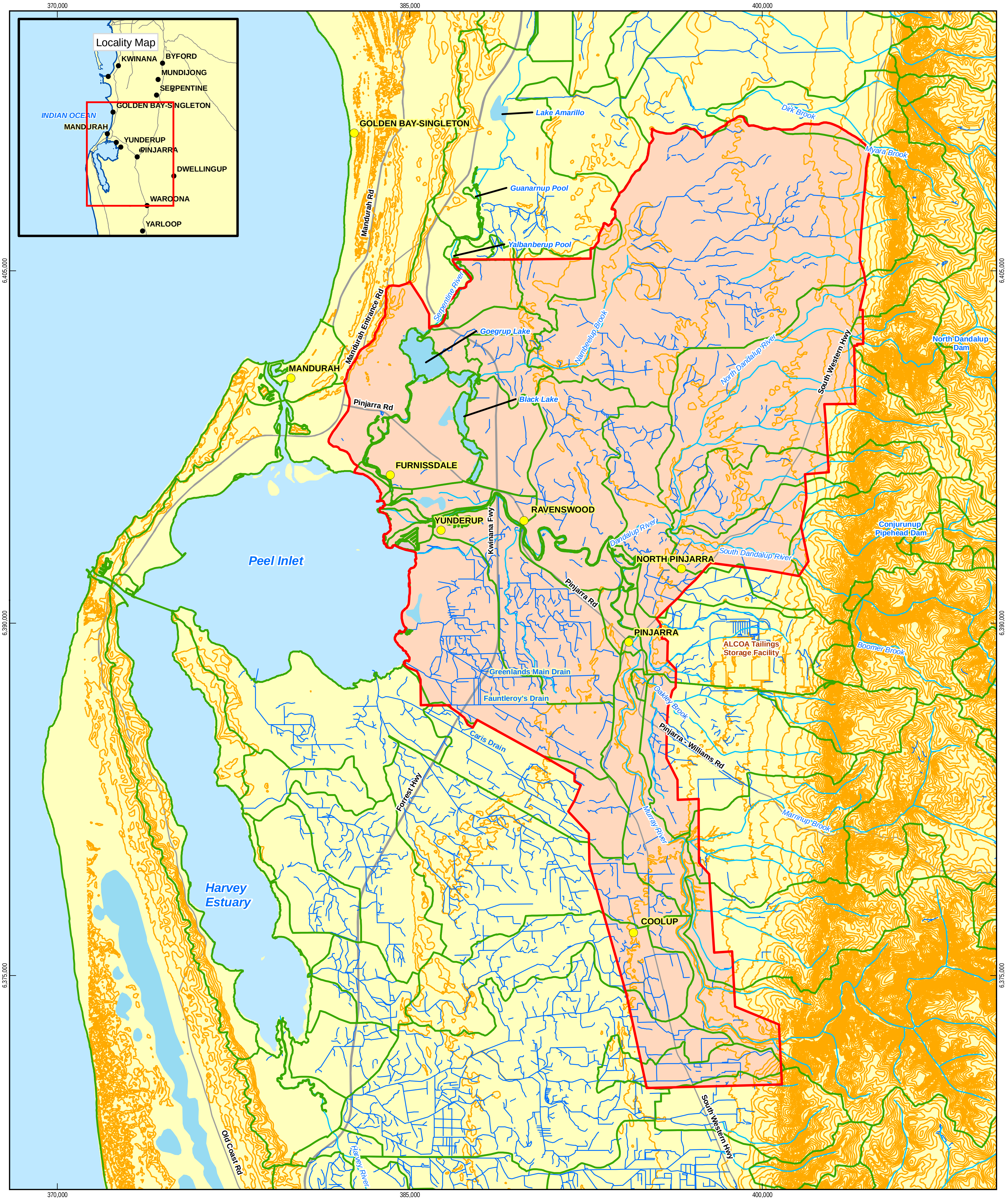
Data Source: Geoscience Australia: WatercourseLines - 200605, Lakes - 200605, Populated Places - 200605; MRWA: Roads - 20090409; DoW: Murray Drainage Study Area - 20090105. Created by: croach2, mludovico, xntan, mdalton2



LEGEND

●	Locality	—	Watercourses	—	Topographic Contours (10 m)
🟥	Flood Study Area	—	Hydrography Linear (Major Drains)		
☐	Subcatchments	—	Roads		

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1,500 750 0 1,500 3,000 4,500

Metres

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

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Drainage in the Flood Study Area

Figure 3

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GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: Geoscience Australia: Lakes - 200605; MRWA: Roads - 20090409; DoW: Murray Drainage Study Area - 20090105; DoW: Linear Hydrography - 20090428; Landgate: Contours - ; DoW: Subcatchments - 20090428; GA: Watercourses - 200905. Created by: croach2, mludovico, wdavis, xntan

2.4 Rainfall and streamflow data

There are a large number of streamflow and rainfall observation stations throughout the study area. However, many do not collect the required data or cover the relevant period of time. Station details were reviewed and a subset of key stations selected for use in the modelling. These stations are shown in Figure 4 and details are given in Tables 3 and 4.

Key rainfall stations used in the RORB modelling of the Murray River included 009538 Dwellingup, 009509 Boddington, 009742 Bannnister North, 509082 Harris River, 509308 Hotham River Marradong Rd, 010614 Narrogin, 010626 Pingelly, 010655 Williams. These stations either have rainfall intensity data or have daily data but were strategically located to capture regional variation in rainfall across the Murray River catchment.

Stations 009538 Dwellingup and 508387 Hopelands were used to define the rainfall within the Hills Catchments.

Stations 009572 Mandurah Park, 009596 Pinjarra and 508387 Hopelands were used to define the variation in rainfall within the flood study area.

Table 3 Key rainfall stations

Station Number	Name	Data	Period of record#
009111	Karnet	Daily	1963-present
009509	Boddington Shire	Daily	1915-present
009526	N Dandalup	Daily	1921-1955
009538	Dwellingup Forestry	Daily, rainfall intensity	1934-present
009572	Mandurah Park	Daily	1889-2001
009596	Pinjarra	Monthly, Daily	1876-present
009614	Waroona	Daily	1935-present
009073	Serpentine pipehead	Daily, rainfall intensity	1959-1991
009742	Bannnister North	Daily	1963-1978
009749	Fairbridge	Daily	1921-present
009873	S Dandalup Dam	Daily	1989-2006
009874	North Dandalup Dam	Daily	1953-2006
010614	Narrogin	Daily, 15-minute	1963-2005
010626	Pingelly	Daily	1890-present
010648	Wandering Comparison	Daily	1887-2003
010655	Williams	Daily	1884-present
010658	Wonnaminta	Daily	1904-present
509082	Harris River	Daily, 15-minute	1965-present
509244	Mayfield	Daily, 15-minute	1983-1999
509306	Williams River Saddleback Rd Bridge	Daily, 15-minute	1975-present
509308	Hotham River Marradong Rd	Daily, 15-minute	1975-present
509387	Hopelands Road	Daily, 15-minute	1979-1999

The key streamflow gauging stations used were 614006 Murray River Baden Powell, 614224 Hotham River Marradong Rd Bridge and 614196 Williams River Saddleback Road Bridge. The Murray River site was the primary RORB calibration location and the Hotham and Williams River sites were subsidiary calibration stations.

The Marrinup Brook Brookdale Siding gauging station (614003) was used during the verification of the regional parameterisation method used in the RORB models for the small catchments to the east of the flood study area. Other streamflow gauging stations were also used during the verification of the regional parameterisation method. These sites include:

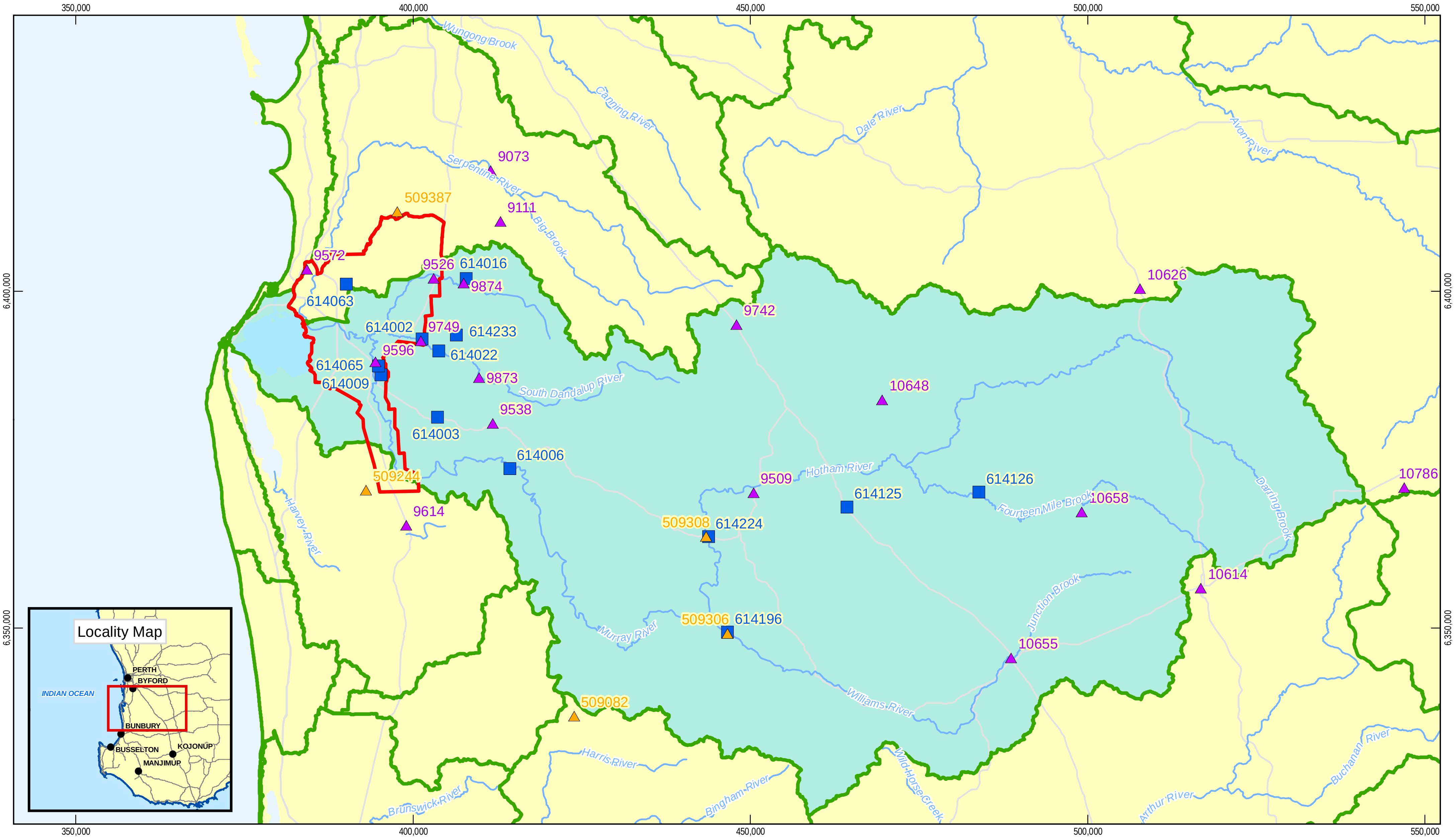
- Station 614025 - A small, partially cleared tributary of Marrinup Brook.
- Station 614009 Oakley Brook Pinjarra South Railway Bridge.
- Station 614233 Conjurunup Creek.
- Station 614068 North Dandalup River at North Dandalup.

The gauging station on Oakley Brook at the Pinjarra South Railway Bridge is affected by backwater from floodwaters in the Murray River. The Department of Water gauging station history file notes that this has occurred at least three times (1974, 1982 and 1983) during its record. However, it was possible to use the data for this site to help verify the regional parameterisation adopted within the runoff routing modelling.

Streamflow gauging stations on the Murray River at Pinjarra (614065) and on Nambeelup Brook at Kielman (614063) were used during the calibration and verification of the hydraulic (Mike 21) modelling.

Table 4 Key streamflow stations

Station Number	Name	Data available/used	Period of record
614002	South Dandalup River Fairbridge Farm	Hourly	1942-1958
614003	Marrinup Brook Brookdale Siding	Hourly	1969-present
614006	Murray River Baden Powell	Peaks, Daily, Hourly	1939-present
614009	Oakley Brook Pinjarra South Railway Bridge	Hourly	1970-1985
614016	North Dandalup River Scarp Road dam site	Hourly	1939-1992
614022	South Dandalup River, West	Hourly	1954-1996
614025	Marrinup Brook Tributary Umbucks Catchment	Hourly	1978-1999
614063	Nambeelup Brook Kielman	Hourly	1990-1998, 2000-present
614065	Murray River Pinjarra	Hourly	1991-present
614068	North Dandalup River North Dandalup	Hourly	1992-1996
614196	Williams River Saddleback Road Bridge	Hourly	1966-present
614224	Hotham River Marradong Rd Bridge	Hourly	1965-present
614233	Conjurunup Creek L Dandalup Scarp Road	Hourly	1966-1993



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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

LEGEND

🟥 Flood Study Area
🟩 Hydrographic Catchments
— Watercourse
— Roads
▲ DoW Rainfall Stations
▲ BOM Rainfall Stations
■ Streamflow Stations

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Key Rainfall and Streamflow Gauging Stations

Figure 4

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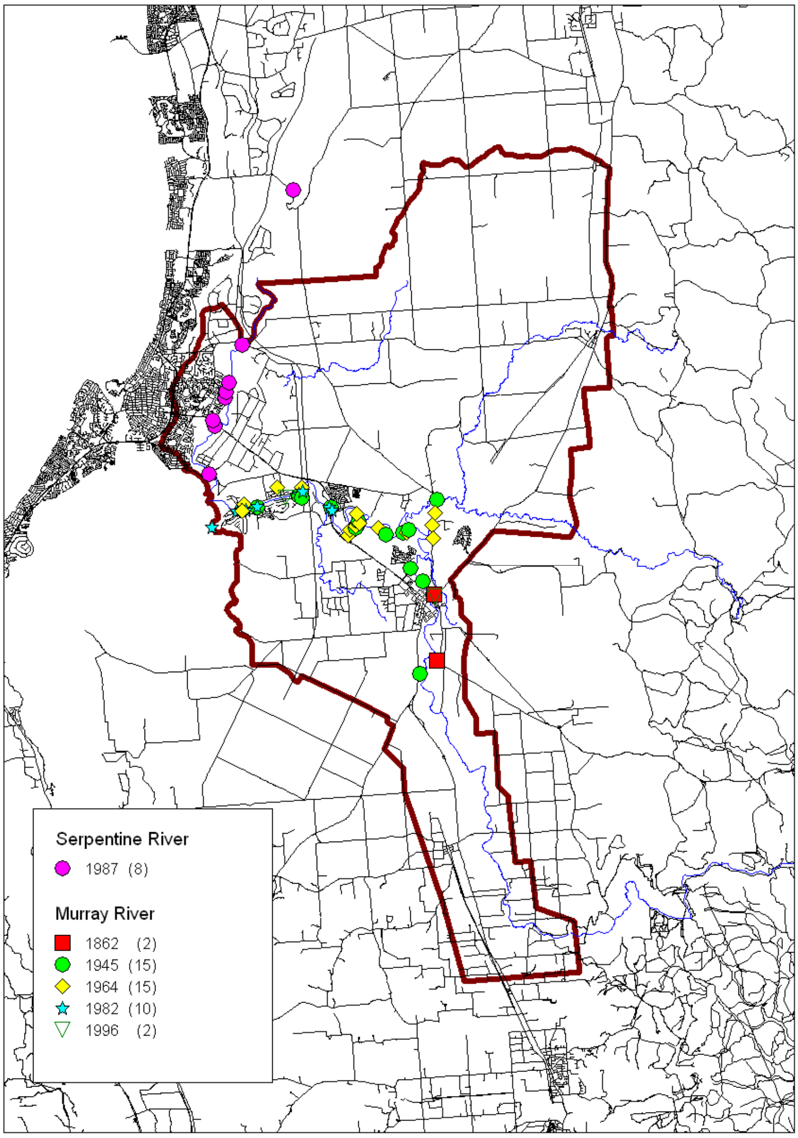
Data Source: Geoscience Australia: WatercourseLines - 200605; MRWA: Roads - 20090409; DoW: Murray Drainage Study Area - 20090105; DoW: Rainfall Sations - 20090527; DoW: Streamflow Gauging stations - 20090527; BOM: Rainfall Stations - 20090527. Created by: croach2, mludovico, wdavis, xntan

2.5 Historic flood level information

Surveyed peak level information for a number of large events on the Murray and Serpentine Rivers was available. This location of observed historic flood levels are shown in Figure 5.

The gauging stations on the Murray River at Pinjarra (614065) and Nambeelup Brook at Kielman (614063) were used during calibration of the 1996 event.

Figure 5 Location of observed historic flood levels



2.6 Survey data

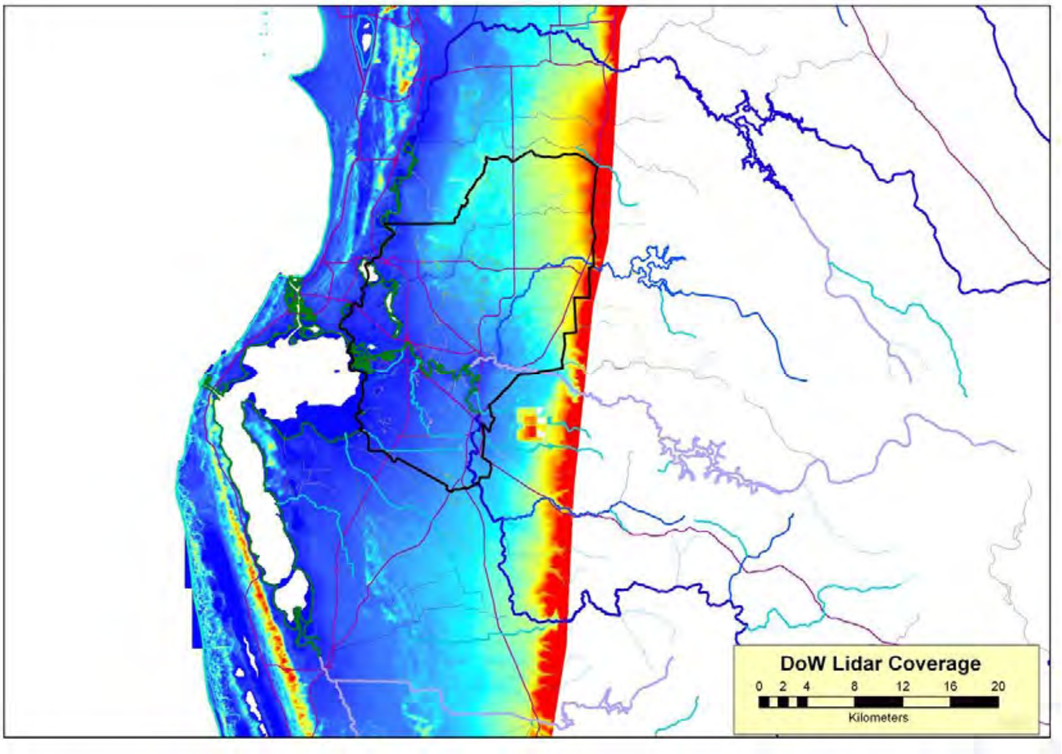
2.6.1 Lidar data

Topography for the Mike 21 modelling was based on Lidar data provided by the Department of Water (Fugro 2008). A representation of the extent of coverage of the data is shown in Figure 6.

These data were captured on 25th February 2008 by Fugro Spatial Solutions Pty Ltd using a Leica ALS50 II MPIA airborne laser sensor on board a fixed wing aircraft. Flying height was 2170 m, with a point density of 1 point per square metre and accuracy of 0.15 m at 67% confidence (Fugro 2008).

The laser sensor does not penetrate water, so did not provide data for sections of the rivers, lakes and estuaries underwater at the time of the flight.

Figure 6 Lidar data coverage



2.6.2 Cross-section information

The Department of Water supplied the cross-sectional information for the main channel in sections of the Murray and Serpentine Rivers that was used during the previous flood studies in 1984 and 1991, respectively. These, along with additional information collected by GHD were used to infill lidar dataset for lengths of river below water-level.

The DoW cross-sections cover the Serpentine River in the flood study area, i.e. between the top of Goegrup Lake and near the outlet to the Peel Inlet. The cross-sections cover the Murray River from upstream of Pinjarra Road to Coolup Road. Spacing between cross-sections is smallest near Pinjarra (about 100 m), increasing to 1-3 km apart upstream and downstream.

GHD collected additional cross-sections to supplement the DoW cross-sections in parts of the river system with ponded water at the time the Lidar data was collected. The GHD cross-sections are located mainly in the upper reaches of the Murray River and in the Dandalup River.

2.6.3 Survey of major structures

In May 2009 GHD collected survey details of 20 bridges and 40 culverts. Smaller culverts were identified visually.

2.6.4 Bathymetry

The terrestrial hydraulic model needed to extend some way into the Peel Inlet. The bathymetry of this area was taken from bathymetry information collected over the last 5 years and provided by the Department of Transport. However, some of the data for the broader estuary was collected by the Public Works Department in the early 1970's.

Bathymetry information for Geogrup and Black Lakes and Yalbanberup Pool were taken from mapping data supplied by DoW.

2.7 Design information for major roads

2.7.1 New Perth Bunbury Highway

Design information for the road profile, culverts and bridges along the New Perth Bunbury Highway were sourced from the Southern Gateway Alliance. Details of a total of four bridges and 21 culverts were used. The top of the road profile was used to represent the road embankment within the digital elevation model used in the hydraulic modelling. The Lidar survey was taken prior to completion of construction the New Perth Bunbury Highway, so the Lidar survey was not accurate for the road surface itself.

2.7.2 Greenlands Road

Road design information for the newly upgraded Greenlands Road was used to provide a road surface and details of one stream crossing. The road was upgraded after the Lidar survey was collected so the road profile data were used to update the digital elevation model used in the hydraulic model.

2.8 Tide and wind data

2.8.1 Water levels

Water levels from the Fremantle, Mandurah, Peel Inlet, Harvey, and Bouvard (Dawesville Channel) tide stations were provided by the Department of Transport. These data were used in the storm surge modelling.

2.8.2 Wind

Wind data sets from three Bureau of Meteorology stations in the study region (Mandurah BoM 009977 Mandurah BoM 009887, Dwellingup BoM 009538) were evaluated. These stations captured the sea and landward wind patterns. A summary of the wind stations is provided in Table 5.

Table 5 Wind stations

Station	General Description	Period	Resolution
009977	Mandurah, at Mandurah Ocean Marina	Nov 1987 – Oct 2001	Hourly
009887	Mandurah, at the train terminal, about 2 km inland.	Oct 2001 – May 2009	Hourly
009538	Dwellingup, about 40 km inland from Mandurah	Feb 1999 – May 2009	3-hr

3. Murray River flood frequency analysis

3.1 Introduction

A flood frequency analysis was undertaken on gauging station data for the Murray River to determine flow estimates for a range of probabilities. The Murray River is the largest contributor of flood flows to the study area, so the characterisation of its hydrology is important. There are long-term streamflow records available for the Murray River at Baden Powell, near the flood study boundary, so a detailed analysis was possible.

Flood frequency analysis on the peak annual flow data series is the most common type of frequency analysis undertaken in Australia. However, as discussed in Section 1.3, the south west of Western Australia is subject to flooding from different mechanisms during different times of the year - ex- tropical cyclones during summer and frontal systems during winter. The differences in timing of the two types of event and the large separation time between the events, during which the catchment antecedent wetness conditions can reduce, suggest the events can be considered independent.

As a result, a seasonal flood frequency approach was adopted, to assess if the relatively infrequent but significant summer events affect the design flow estimates from flood frequency analyses within the range of probabilities examined (1 in 5 to 1 in 500 AEP). This approach involved determining flood frequency curves for summer and winter separately, than adding the curves to produce an annual curve.

3.2 Methodology

The flood frequency analysis was undertaken for the Murray River catchment using gauging stations at 614006 Murray River Baden Powell, 614224 Hotham River Marradong Rd Bridge and 614196 Williams River Saddleback Road Bridge.

The annual and seasonal (summer and winter) maximum flows were extracted from the gauged record. The summer period was defined as October to March and winter as April to September. These summer and winter periods were defined to be consistent with CRC FORGE (DoE 2004) design rainfall estimates.

It is recognised that parts of the winter elevated flows may occur over the summer period with the definition of an October start to the season. As only discrete events resulting from rainfall in the summer period were considered within the summer analyses, any residual flows from the winter period were censored from the summer series and a peak below a censored threshold assumed.

Hydrographs were inspected for completeness and to ensure that events running across a season boundary did not produce two peaks from the same event. Data that could distort the analysis, outliers (i.e. those that fell outside the 90% confidence limits) and low values not representing a significant flow event were also censored. The resultant summer data series for the Murray River Baden Powell site contained 25 flow events from the 68 years of data. The winter peak flows dataset contained valid data for all 68 years of record.

Log Normal, Log Pearson type III (LPIII) and Generalised Extreme Value distributions were fitted to the annual and seasonal data series. The LPIII distribution was found to adequately represent the distributions from the three gauging stations.

An annual probability was then calculated from the summer and winter distributions by summing probabilities using the method described in Pilgrim (2001). This method assumes that the two series are independent. The high flow years within the summer and winter series for each of the gauges tend to occur in different years, suggesting that this assumption is valid.

A frequency analysis was also undertaken on the annual 5 day maximum volume at the Baden Powell gauging station on the Murray River to help evaluate design event volumes. This information will assist in preparing the design hydrographs for the Murray River for input to the hydraulic model.

3.3 Results

A summary of the adopted flood frequency results for the three gauging stations is given in Table 6. Curves and annual data for the Murray River Baden Powell station are plotted in Figure 7. Details of the flood frequency data are given in Appendix A.

Design flow estimates from the annual data series analysis, using the streamflow record (1940 to 2007) gives a 1 in 100 AEP discharge of about 1,000 m^3/s. As no large events have been received since 1996, the additional data since the previous studies for the Murray River has resulted in a reduction in the 1 in 100 AEP discharge. However the seasonal analysis adopted in this GHD study has found that summer events do impact on the annual frequency estimates for events of 1 in 100 AEP and larger. The net result for the 1 in 100 AEP event on the Murray River is comparable to the prior studies (Table 1).

The estimated summer 1 in 500 AEP flows from the flood frequency analysis are considered doubtful due to the limited number of true flows available within the summer dataset. It is expected that the slope of the curve is flatter at higher flows than indicated by the available data. However, summer events are likely to define the 1 in 500 year AEP event for the Murray River. The 10 and 90 percent confidence interval curves for the larger events (see Appendix A) indicate considerable uncertainty in these estimates. The calibrated RORB Model was used to investigate the design flood estimates from the flood frequency analyses.

There was some evidence to suggest that the June 1945 peak of 793 m^3/s at the Murray River Baden Powell station may have been up to 880 m^3/s, some 11% higher. This event is the largest event on record so the sensitivity of this alternate value on the flood frequency results was investigated. The annual 1 in 100 AEP discharge increased by only 3% (35 m^3/s) using the higher streamflow value indicating that the flood frequency results are relatively insensitive to variation or uncertainty in the observed data.

The 1 in 100 AEP annual discharge for the Murray River at Baden Powell that is presented here is of a similar order to previous studies, but this appears to be largely coincidental. The flood frequency analysis presented here accounts for additional data than available to the earlier studies and uses a different approach (the seasonal analysis).

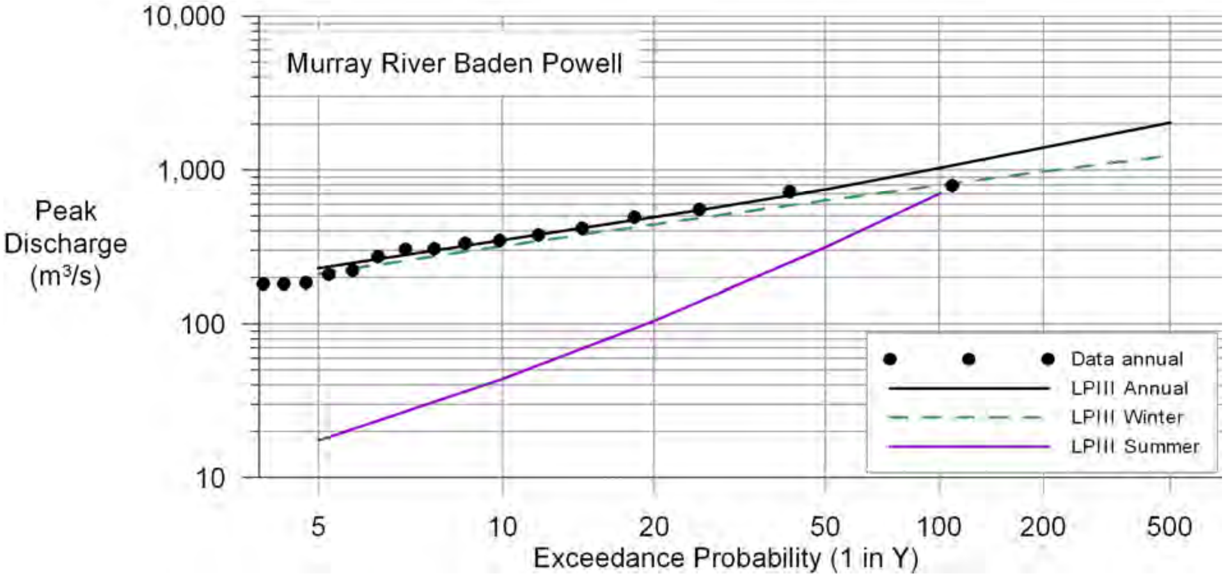
A flood study for the Hotham River by SKM (SKM 2009) produced discharges larger than those determined by flood frequency analyses on the observed flows at the Maradong Road Bridge gauging station. The SKM (2009) study, though, included flow estimates from observed flood levels for the February 1955 and 1945 floods that pre-date the gauge record at the Maradong Bridge gauging station on the Hotham River. Therefore, the difference is attributable to the different methodologies employed. The flow estimates for the Hotham River do not impact the results of the Murray River at Baden Powell design flow estimates.

Table 6 Seasonal flood frequency analysis

AEP	Prob-ability	Peak discharge (m^3/s)								
(1 in Y)	(%)	614006 Murray River Baden Powell Water Spout			614196 Williams River, Saddleback Road Bridge			614224 Hotham River Marradong Rd Bridge		
		Annual	Summer	Winter	Annual	Summer	Winter	Annual	Summer	Winter
5	20	230	20	210	90	10	80	120	10	110
10	10	340	45	320	135	30	110	165	30	160
25	4	550	170	490	210	110	170	230	70	230
100	1	1,030	790	800	480	440	260	360	180	330
500	0.2	2,050*	-**	1,250	950*	-**	380	610	510	470

* reduced from 5,380 and 1,800 m^3/s for the Murray and Williams Rivers respectively; ** ignored.

Figure 7 Flood frequency analysis – Murray River Baden Powell



A frequency analysis on the maximum 5 day volumes for the Murray River at Baden Powell (see Appendix A) determined that the 1 in 100 AEP volume is of the order 210 – 240 GL. The 1 in 10 AEP 5 day flow volume is estimated to be 95-100 GL. It should be noted that 24 of the 57 years analysed had periods of missing data. Of these years, half had missing data in winter. No attempt was made to infill missing data and as such, the results of the frequency analysis are likely to be an underestimate.

4. Hydrology studies

4.1 Introduction

The hydrology studies generated hydrographs at the edge of the flood study area for input to the hydraulic model. The hydrographs were derived from a number of different sources – RORB models were developed for the Murray River and the catchments to the east of the flood study area, contributions from the Serpentine River were taken from a study by SKM, and runoff generation within the flood study area was simulated within the hydraulic model.

RORB is a general runoff and streamflow routing program commonly used in Western Australia to calculate flood hydrographs. The model is described in Laurenson et al. (2007). The model was calibrated using observed streamflow data and verified using the results of the flood frequency analysis (see Section 3).

Separate models were used to generate hydrographs for the Murray River and the smaller catchments along the eastern edge of the flood study area (termed the "Hills Catchments"). Aside from the Murray and Serpentine Rivers, there are a total of 18 external catchments that contribute to the flood study area. It was considered desirable to model these separately so as to more accurately represent initial conditions, the early stage of the Murray River hydrograph and flooding away from the main Murray River floodplain.

Inflows to the study area from the Serpentine River were derived from the Lower Serpentine Floodplain Management Study (SKM, in prep.).

Runoff generation and flood routing within the flood study area was undertaken as part of the hydraulic model; the method used is summarised in Section 4.5.

4.2 Murray River RORB model

4.2.1 Model setup

The Murray River catchment to the flood study boundary was split into 18 sub-areas. Sub-area delineation was based on the location of gauging stations and the geometry of the main stream network. The catchments were defined to enable calibration to the streamflow gauging stations on the Hotham and Williams Rivers and at the Baden Powell gauging station on the Murray River. The model extended to the edge of the flood study area.

Total catchment area represented in the model, at the flood study boundary, was 6,867 km^2.

A 6 hour time step was used. While a time step of this length could lead to some loss of hydrograph resolution, it was chosen considering the large amount of data that needed to be collated for the calibration and application of the model, the likely hydrograph rise time, size of the catchment at the point of interest (i.e. the flood study boundary) and model computational requirements. A 1 hour time step for the 1982 event was evaluated to test the sensitivity to the time step length. It was concluded that the choice of time step did not influence the model calibration.

4.2.2 Calibration

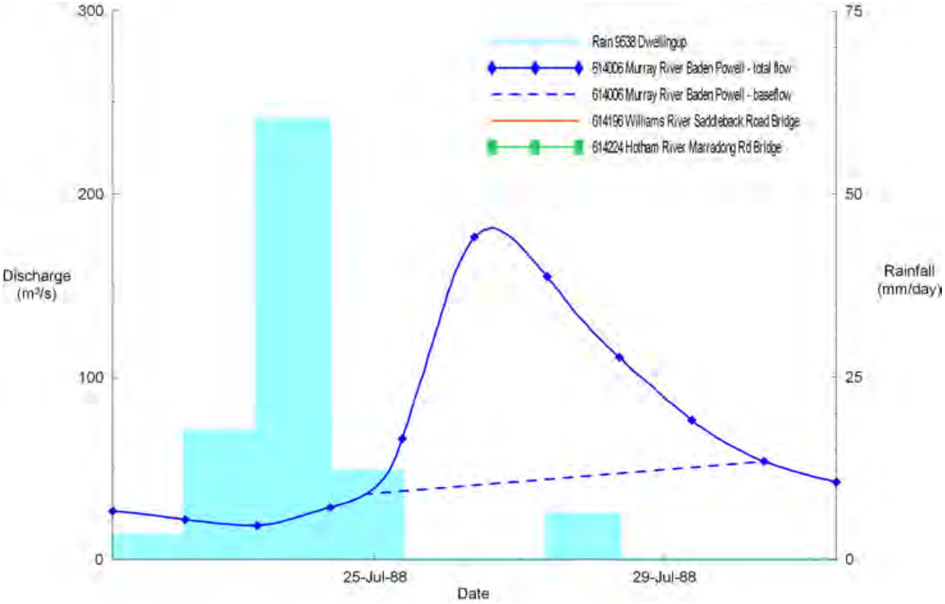
The model was calibrated initially to a selection of observed streamflow events, to set a value for the parameter k_c and runoff coefficient, *C*. A summary of the calibration results is given in Table 7.

Values of the runoff coefficient, *C*, was verified by calibration to the flood frequency results (see Section 4.2.3).

Observed streamflow at stations 614006 Murray River Baden Powell, 614224 Hotham River Marradong Rd Bridge and 614196 Williams River Saddleback Road Bridge were used to calibrate the model. While three stations within the catchment were used, the focus of the calibration was on the Baden Powell station, which was closest to the outlet and where the application hydrographs were required.

Direct runoff was used in the calibration simulations (i.e. base flow was not simulated in RORB). The baseflow for observed streamflow events was separated from direct runoff using the constant slope method. An example of the baseflow separation is shown in Figure 8.

Figure 8 Baseflow separation for the 1988 event at the Murray River at Baden Powell gauging station



Baseflow is an important contribution to total event flow on the Murray, particularly for winter events. Median baseflow, calculated as a percentage of total flow at the peak, was 6 and 28% for summer and winter events respectively. Median values of baseflow as a percentage of total flow volume were 14 and 59% for summer and winter events.

During calibration, the model was run in fit mode using the proportional loss approach, in accordance with local conventional practice (Pearce, 2006). The volume of direct runoff was calculated during the baseflow separation process, prior to calibrating initial loss and k_c. The parameter *m* was set to 0.85 for all events, in accordance with current practice for the South West of Western Australia (Pearce, 2006). Initial loss was varied until the observed hydrograph rise was. The model determined the runoff coefficient to match runoff volume for each of the events. The parameter k_c was varied until the hydrograph shape and peak were matched. A uniform k_c for the entire catchment was adopted.

Five events were used to calibrate a catchment k_c. The calibrated k_c values ranged from 65 to 150 with a median value of 90. This value of k_c is consistent with a regional relationships with catchment area found in a Water Corporation study (Pearce 2006). There was no consistent relationship between k_c and event size (Figure 9). Consequently, a k_c of 90 was adopted for the design flood simulations.

Derived runoff coefficients for the calibration events, split into summer and winter, are shown in Figure 9.

Runoff coefficient at the Baden Powell station was found to vary with peak discharge and appeared to be slightly lower for summer events than equivalently sized winter events (Figure 10).

Calibrated initial loss is shown in Figure 11. There is no obvious relationship between initial loss and event magnitude. However, the initial loss for summer events did vary between the predominantly cleared catchments of the Hotham and William Rivers, east of the Darling Range, compared to the largely forested catchments with the Darling Range to the Baden Powell gauge on the Murray River.

Table 7 RORB calibration to observed events

	Baseflow separation					Calibration		
Catchment	**Rainfall (mm)**	**Direct runoff (mm)**	**Base-flow (% total flow at peak)**	**Base-flow (% total flow volume)**	**Peak Q (m^3/s)**	k_c	**Initial Loss (mm)**	**Runoff Coeffic., *C*, calib. *IL***
Winter events:								
Aug 1964 event:								
614224 Hotham River								
614196 Williams River								
614006 Murray River	72.0	14.3	28%	56%	553.6	90	22	0.29
Jul 1988 event:								
614224 Hotham River	45.8	4.0			85.0	110	20	0.16
614196 Williams River	46.2	9.1			100.7		0	0.20
614006 Murray River	53.7	5.1	19%	59%	181.8		30	0.21
Aug 1996 event:								
614224 Hotham River	41.6	6.9	39%		217.0	120	15	0.26
614196 Williams River	52.4	5.2	45%		92.3		20	0.16
614006 Murray River	49.0	5.8	44%	75%	304.1		25	0.24
Summer events:								
Feb 1955 event:								
614224 Hotham River						90		
614196 Williams River								
614006 Murray River	230.3	17.5	0%	4%	721.7		160	0.25
Jan 1982 event:								
614224 Hotham River	151.0	5.0	15%		188.6	65	95	0.09
614196 Williams River	182.8	15.0	15%		285.0		85	0.15

	Baseflow separation					**Calibration**		
Catchment	**Rainfall (mm)**	**Direct runoff (mm)**	**Base-flow (% total flow at peak)**	**Base-flow (% total flow volume)**	**Peak Q (m^3/s)**	k_c	**Initial Loss (mm)**	**Runoff Coeffic., *C*, calib. *IL***
614006 Murray River	168.9	7.3	12%	24%	345.5		190	0.18

* no streamflow data.

Figure 9 Seasonal K_c, Murray River catchment, calibration events

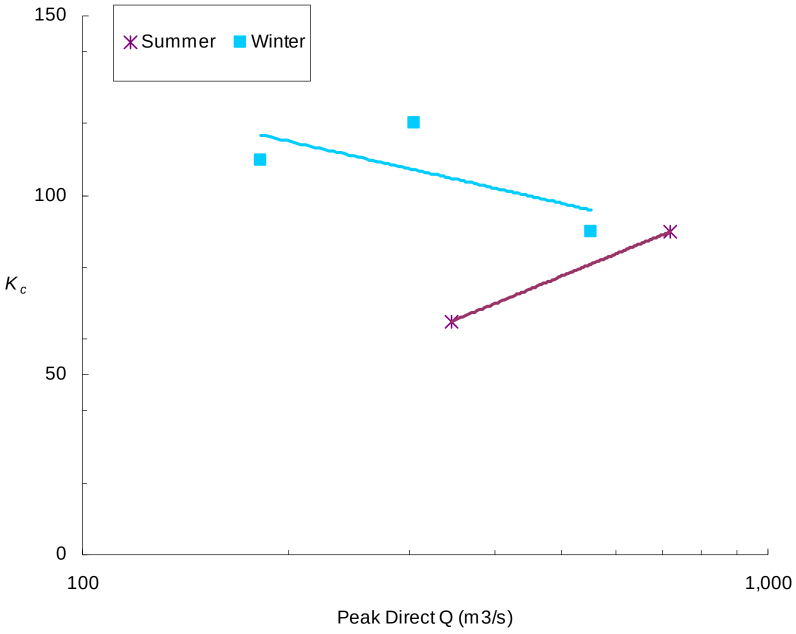


Figure 10 Seasonal runoff coefficient, Murray River catchment, calibration events

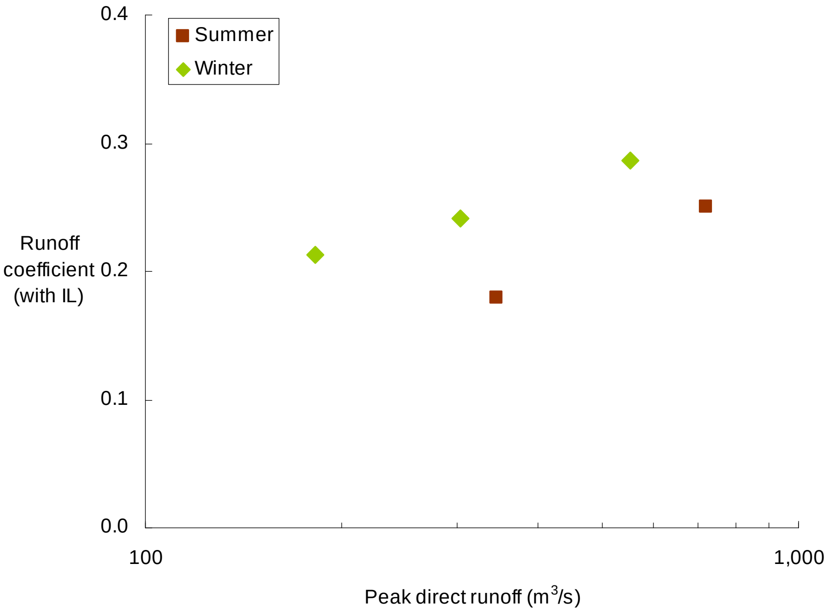
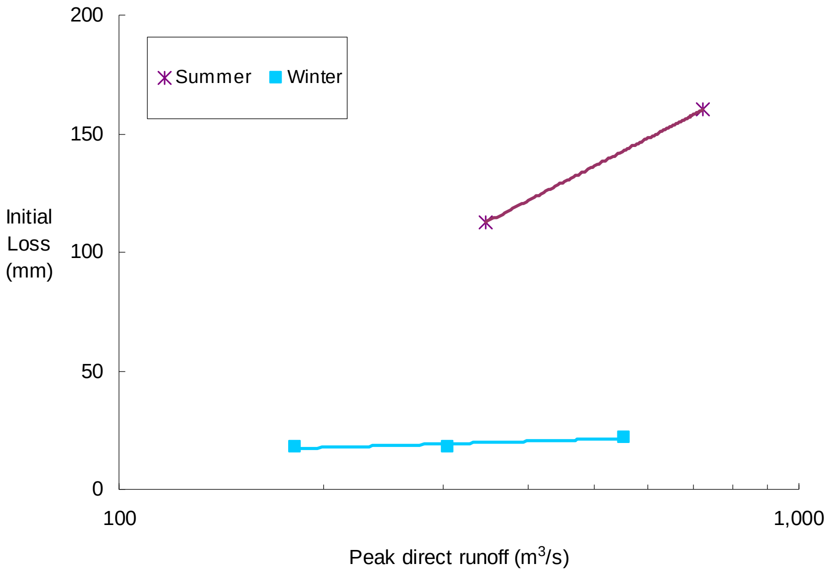


Figure 11 Initial loss, Murray River catchment, calibrated to events



4.2.3 Design flow estimation

Design rainfall estimates

IFD information was calculated using the methods outlined in Australian rainfall and Runoff (Pilgrim 2001) and CRC FORGE (DoE 2004). Rainfall was generated for three locations, at Dwellingup, Boddington and Wonnaminta, and distributed across the catchment using thessien polygons. Temporal patterns were derived using the methods given in Australian Rainfall and Runoff (Pilgrim 2001) and the program AUSIFD (Jenkins 2001).

Critical rainfall duration for the catchment to 614006 Murray River Baden Powell was determined to be 48 hours for all design flow simulations.

Seasonal considerations

For annual simulations, winter and summer processes contributed differently depending on the AEP of the event. Based on the contributions of the seasonal components, the following methodology was adopted:

- For events from 1 in 5 to 1 in 50 AEP, winter baseflow and initial loss values were adopted.
- For the 1 in 100 AEP event, a 50-50 contribution of winter and summer baseflow and initial loss was used.
- For the 1 in 500 AEP event, summer baseflow and initial loss was used.

Baseflow

Baseflow was not simulated in the RORB model (i.e. direct runoff was simulated) and the flood frequency data were adjusted to exclude an estimated value of baseflow. Baseflow was set at 5% of the peak total flow for summer and 30% for winter, based on the calibration events.

Baseflow was added to direct runoff using the formula:

$Qb1 = BR*Qbi-1 + BC(Qr)^{\wedge}BM$

Where $Qb1$ = the baseflow at time step 1, $Qbi-1$ = the baseflow at time step $i-1$, BR, BC and BM = calibrated parameters, and Qr = direct runoff.

A constant proportion of total flow (30 % for winter, 5% for summer) has been used in other studies (e.g. Pearce 2006). The above method, however, is considered to more accurately match the behaviour of observed events. It has the peak baseflow contribution lagged from the direct runoff peak, a higher contribution of baseflow in the falling limb and better matches total event baseflow contribution.

Figure 12 illustrates the contribution of baseflow to the total hydrograph for the constant-value method and for the method adopted here. A winter dominant event (1 in 5 AEP) and a summer dominant event (1 in 500 AEP) are shown. The increased baseflow contribution to the falling limb of the winter 1 in 5 AEP (winter) event versus the 1 in 500 AEP (summer) event is clearly visible.

The baseflow model was parameterised by calibration to the flood frequency peak (annual) and to match the baseflow contribution to total event volume from the observed event calibration. The total flow hydrograph was calculated externally to RORB by adding direct runoff (modelled using RORB) to the baseflow contribution.

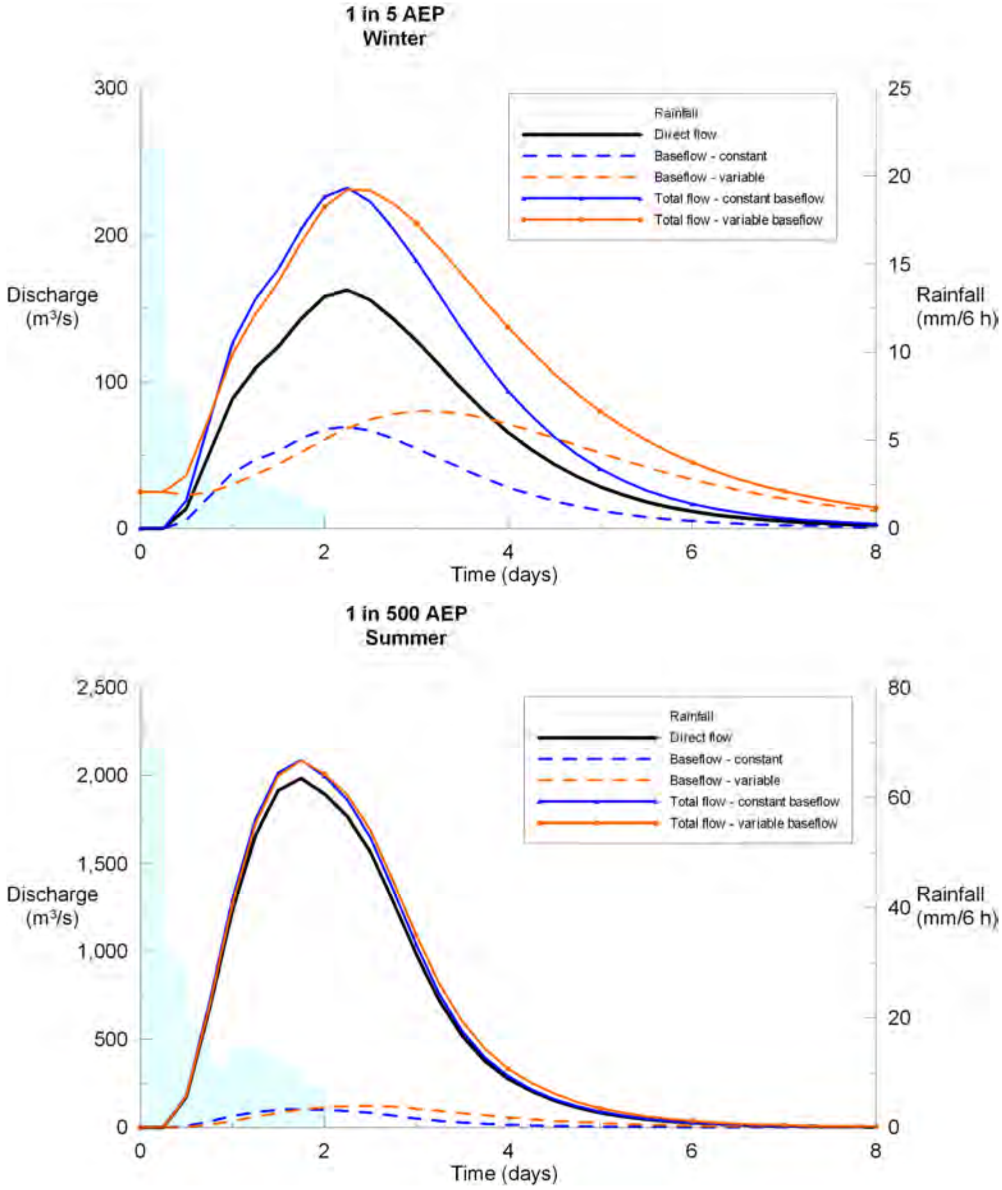
Baseflow in the river prior to the event was based on the median daily flow in February (summer) and August (winter).

The derived baseflow parameter values are shown in Table 8. The fraction of total baseflow of the event volume was kept lower than the observed events to keep the distribution of baseflow late in the event to what appeared reasonable values. The shape of the function used did not allow more baseflow near the peak, which is likely, and less in the tail of the event. Likewise, both parameters BR and BC needed to be varied with AEP to give an acceptable temporal variation in baseflow.

Table 8 Calibrated baseflow parameter values

Parameter	**AEP (1 in Y)**				
	5	**25**	**50**	**100**	**500**
Baseflow (% total flow volume)	45	43	47	29	7
Baseflow (% total flow at peak)	35	34	38	23	6
BR	0.86	0.90	0.86	0.80	0.75
BC	0.100	0.170	0.120	0.080	0.020
BM	1.00	0.90	1.00	1.00	1.00
Initial baseflow (m^3/s)	25	25	25	0	0

Figure 12 Baseflow components, Murray River design hydrographs



Verification to flood frequency data

Runoff coefficients for the design events were verified by reproducing the estimated peak discharges from the flood frequency analyses using the calibrated RORB model. Winter, summer and annual rainfall was applied to the model and the runoff coefficient varied until the corresponding flood frequency peak discharge was matched.

A range of event durations were investigated and 48 hours was found to be the critical duration for events between 1 in 5 and 1 in 500 AEP.

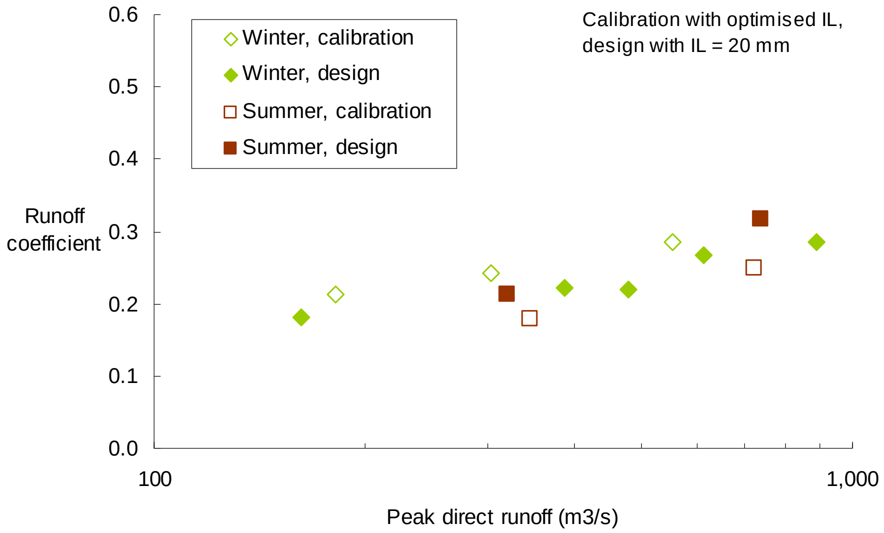
Initial loss was set at 50 mm for summer events and 20 mm for winter events. The median initial loss determined from calibration to the summer observed events was found to be too high for the design events. Using the median initial loss from calibration resulted in runoff coefficients well above those observed in calibration and for the winter design events. This result is consistent with design rainfall information being for rainfall bursts rather than complete storms as was the case during calibration. The observed summer events tend to have a higher total rainfall in comparison to the design rainfall bursts.

The calibrated winter initial loss of 20 mm produced an acceptable range of runoff coefficients, although slightly lower than those found during calibration (Figure 13). This result is consistent with the concept of AEP neutrality, since calibration events tend to result from smaller rainfall events on wetter than average catchments. For example, the expected probability of the July 1996 flow event is estimated to be about 1 in 10 AEP, despite the observed rainfall that caused the event being less than a 1 in 5 AEP.

Summer runoff coefficients were only derived for events greater than 1 in 50 AEP, as smaller events were found to be winter dominant in the flood frequency analysis (see Section 3). Adoption of a runoff coefficient for the 1 in 500 AEP event for summer events (90 mm initial loss and 0.5 runoff coefficient) produced a peak discharge of 2,300 m^3/s. This supports the assumption that the summer flood frequency result for the 1 in 500 AEP event were too high (see discussion in Section 3.3).

The modelled 5 day design flow volumes are consistent with the results of a frequency analysis on the gauged information for the Baden Powell monitoring station (see Section 3.3 and Appendix A). The modelled 5 day design 1 in 10 AEP and 1 in 100 AEP flow volumes from the RORB modelling of the Murray River were found to be 95 GL and 225 GL, respectively. These values are consistent with the frequency results on the annual maximum 5 day gauged volumes of 95-100 GL (1 in 10 AEP) and 210-240 GL (1 in 100 AEP) quoted in Section 3.3. This consistency in results between the two methods provides additional support for the adopted design loss rates and baseflow representation within the RORB modelling.

Figure 13 Seasonal runoff coefficient, Murray River Baden Powell, event and calibration



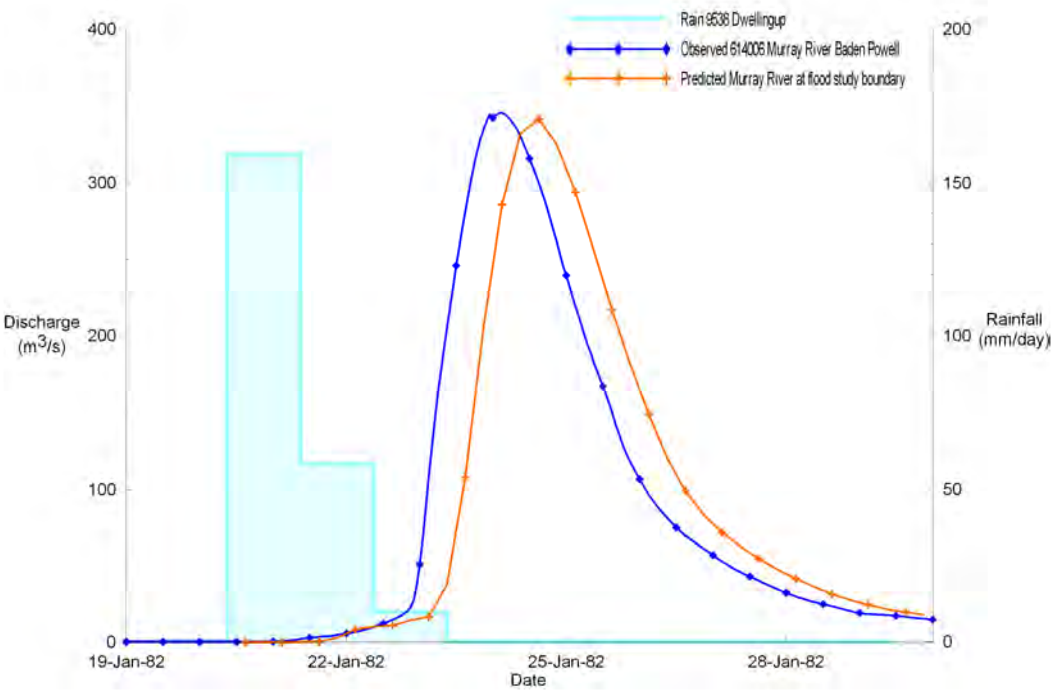
4.2.4 Application to the hydraulic model

The calibrated runoff routing model was used to produce hydrographs at the edge of the flood study area, for use in the hydraulic modelling. Hydrographs were generated both for observed events needed for calibration of the hydraulic model and for design events.

For the observed events, the gauged streamflow at the Murray River Baden Powell station was input and calibrated parameters for each observed event were used to simulate direct runoff for the relatively small catchment area between the Baden Powell gauging station and the flood study boundary. Baseflow was derived by scaling using a catchment area ratio from the baseflow derived by separation at the Baden Powell station and input directly into the model at the outlet.

An example of an observed hydrograph at the Baden Powell gauging station and the simulated hydrograph at the flood study boundary is shown in Figure 14 for the 1982 event. The simulation indicates that there is little change in the hydrograph shape between Baden Powell and the flood study boundary and a temporal shift of about 12 h.

Figure 14 Observed and predicted hydrographs, Murray River, 1982 event



For the design events, the calibrated model was used to generate hydrographs at the flood study area boundary. Design hydrographs are shown in Figure 15. A summary of characteristics of the design events is given in Table 9.

Figure 15 Predicted design event hydrographs, Murray River at the flood boundary

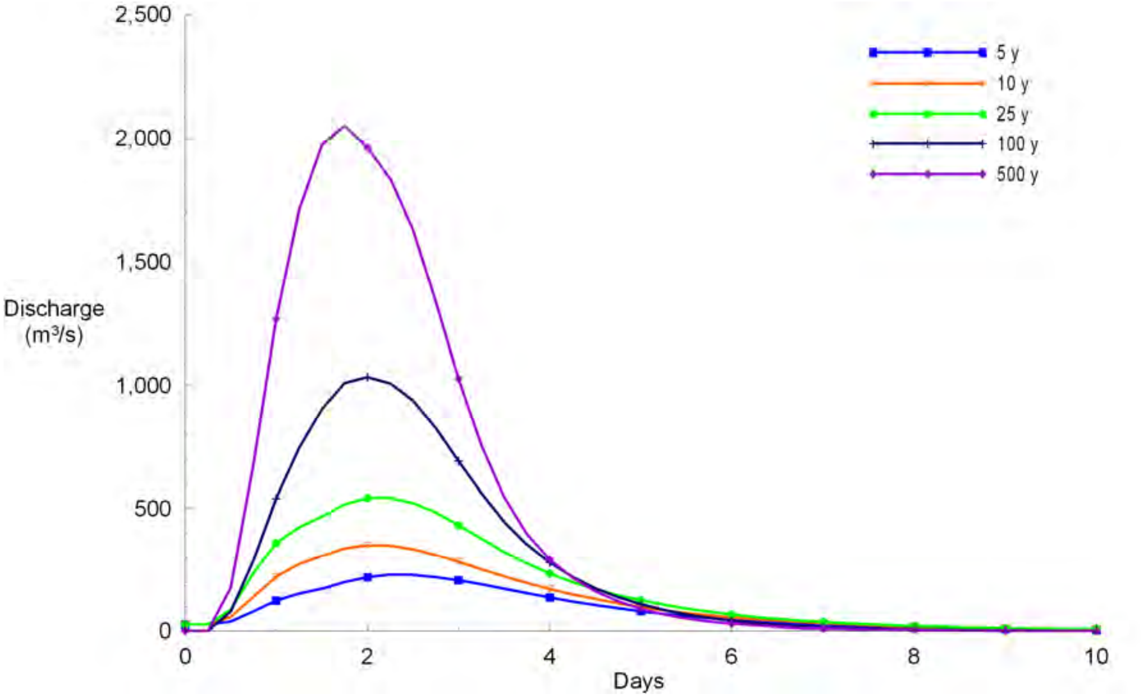


Table 9 Predicted design event characteristics, Murray River at the flood study boundary

AEP	Probability (%)	Peak discharge (m^3/s)	Event volume (GL)*
5	20	230	75
10	10	350	105
25	4	540	160
100	1	1,030	230
500	0.2	2,050	400

* calculated over 10 days.

4.3 Hills Catchments RORB models

4.3.1 Catchment description and representation in modelling

Of the 18 Hills Catchments, the nine larger catchments were simulated using individual RORB models. The models were setup with between 5 and 10 sub-areas, depending on catchment size and complexity. For catchments with a gauging station (Marrinup and Oakley Brooks), sub-area delineation was setup to allow calibration to these stations. A time step of 1 h was used.

Hydrographs at the outlets of the smaller catchments, which were less than 10 km^2 in area, were derived by scaling on an area basis from a RORB model for a nearby catchment. While it is acknowledged that peak discharge is not generally considered to vary linearly with area, runoff volume for catchments of a similar size and physical characteristic is more likely to be linearly variable. A direct area scaling is appropriate here for input to the broad floodplain hydraulic model as the runoff volume is more important than peak discharge.

Characteristics of the Hills Catchments are summarised in Table 10.

Table 10 Characteristics of the Hills Catchments

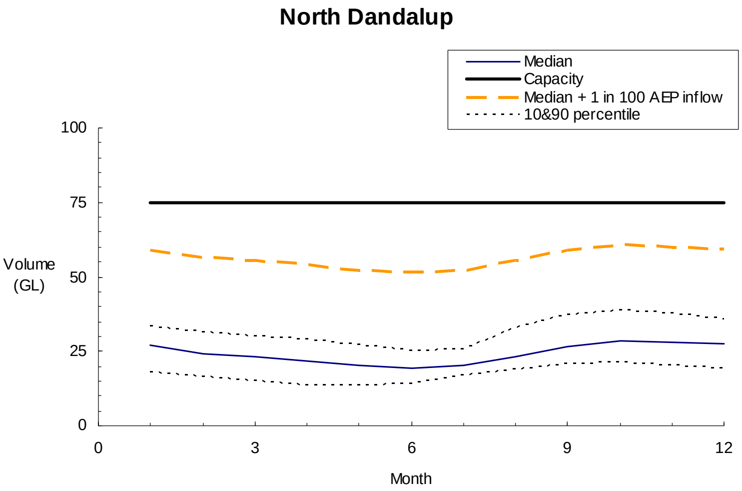
Catchment Name	Catchment Code	Area (km^2)	Fraction cleared (%)	Description
Conjurunup Creek	CC	56.2	19	Simulated using a RORB model.
Marrinup Brook	MB	77.7	44	Simulated using a RORB model, with verification against gauging station data.
Murray River banks, south of S Pinjarra	MR2 banks	25.9	84	Simulated using a RORB model.
Murray River tributary	MR2T1	30.6	87	Simulated using a RORB model.
Murray River tributary	MR2T2	8.4	100	Scaled from MR2T1 RORB model hydrograph.
Murray River tributary	MR2T3	4.2	98	Scaled from MR2T1 RORB model hydrograph.
Nambeelup Brook tributary	NBT3	3.2	88	Scaled from NDR RORB model hydrograph.
Nambeelup Brook tributary	NBT3T1	2.8	97	Scaled from NDR RORB model hydrograph.
Nambeelup Brook tributary	NBT4	2.3	73	Scaled from NDR RORB model hydrograph.
North Dandalup River, lower	NDR	14.6	33	Section below dam simulated using a RORB model. Dam assumed to not overtop.
North Dandalup River tributary	NDRT1	7.2	68	Scaled from NDR RORB model hydrograph.
North Dandalup River tributary	NDRT2	10.2	44	Simulated using a RORB model.
North Dandalup River tributary	NDRT3	2.4	89	Scaled from NDR RORB model hydrograph.
North Dandalup River tributary	NDRT4	7.5	45	Scaled from NDR RORB model hydrograph.
Oakley Brook	OB	47.0	54	Simulated using a RORB model. Gauging station affected by backwater effects during major flows on the Murray River.
South Dandalup River, lower	SDR	44.3	18	Catchment below dam simulated using a RORB model. Dam assumed to not overtop.
South Dandalup River tributary	SDRT1	1.6	95	Scaled from NDR RORB model hydrograph.
Tate Gully	TG	18.9	87	Simulated using a RORB model.

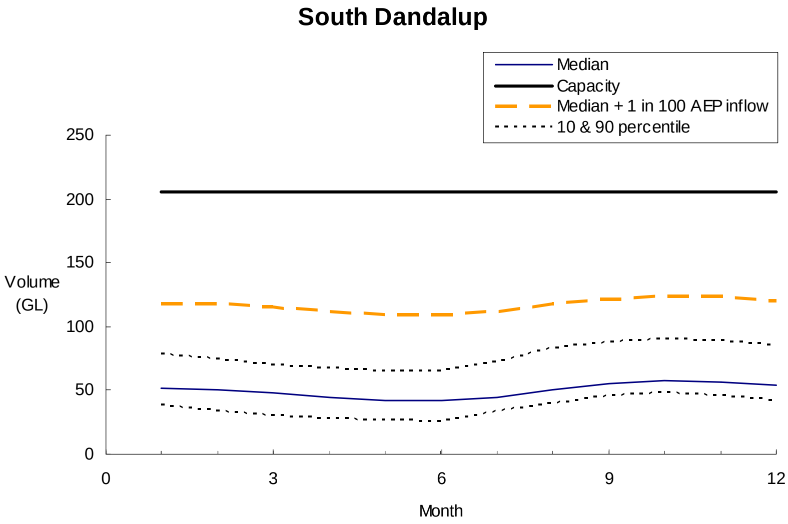
4.3.2 Storages

The Hills Catchments include the North Dandalup and South Dandalup Rivers. Major water supply dams have been constructed on these Rivers. These dams have a large capacity relative to the average catchment flows. An assessment of the potential for the catchments upstream of these dams to contribute to the flooding within the flood study area during the range of design events was investigated.

Historical water levels for the North Dandalup and South Dandalup Dams were obtained from the Water Corporation. The median monthly storage levels were calculated as representative of average antecedent storage levels prior to the onset of a flood event (Figure 16). Adopting these median values as a starting water level and applying a 1 in 100 AEP design inflow (100% runoff) from the upstream catchment did not result in an overtopping of the spillways as there was sufficient storage available within the reservoirs. It is considered unlikely under the current extraction patterns that these dams will overtop for the events up to the 1 in 500 year AEP that are the focus of this study. Therefore, the catchments upstream of the dams on the North Dandalup and South Dandalup Rivers have been excluded from further investigations.

Figure 16 Historical dam water levels





Conjurunup Creek also contains a small pipehead dam operated by the Water Corporation of WA as part of their water supply scheme. This storage is generally operated at a full level and is small relative to the flows from the upstream catchment. Accordingly, the dam capacity was ignored and the upstream catchment assumed to contribute flow to the study area.

The Tate Gully catchment includes a large tailings storage facility, operated by Alcoa. The tailings storage facility was assumed to not contribute streamflow (i.e. not overtop) for all the calibration events and the design events, except for the 1 in 500 AEP design event. It was assumed that during the 1 in 500

AEP design event the tailings facility was near full (100 mm freeboard) and there was 100% runoff from the area of the facility. This increased streamflow at the flood study boundary, particularly later in the hydrograph.

4.3.3 Regional runoff parameterisation method

There are insufficient gauging station data in the Hills Catchments to adequately calibrate RORB models for each of the 18 catchments that flow into the study area. Therefore, RORB models were parameterised using a regional method for estimating k_c and C developed by the Water Corporation for the south west of Western Australia (Pearce 2006). The regional parameter values were verified using the available gauging station data.

Values of k_c were derived based on catchment area using the following relationships:

- Foothills (i.e. cleared areas on the coastal plain, e.g. Nambeelup): $k_c = 1.07*A^{0.76}$
- Fully forested in Darling Range: $k_c = 0.49*A^{0.76}$
- 50% or more clearing in Darling Range: $k_c = 0.22*A^{0.76}$

where A = area in km^2.

Note that this relationship is valid for catchments <100 km^2 in the south west of Western Australia. A constant value of k_c is applied to each catchment.

Values of runoff coefficient, C, were derived based on event rainfall using the following relationships:

- Foothills (i.e. cleared coastal plain): $C = -3.55E-07R^2 + 6.71E-04R + 3.71E-01$
- Fully forested in Darling Range: $y = -2.33E-07R^2 + 6.35E-04R + 8.49E-02$
- 50% or more clearing in Darling Range: $y = -3.09E-07R^2 + 6.57E-04R + 2.64E-01$

Where R = areally reduced event rainfall in mm.

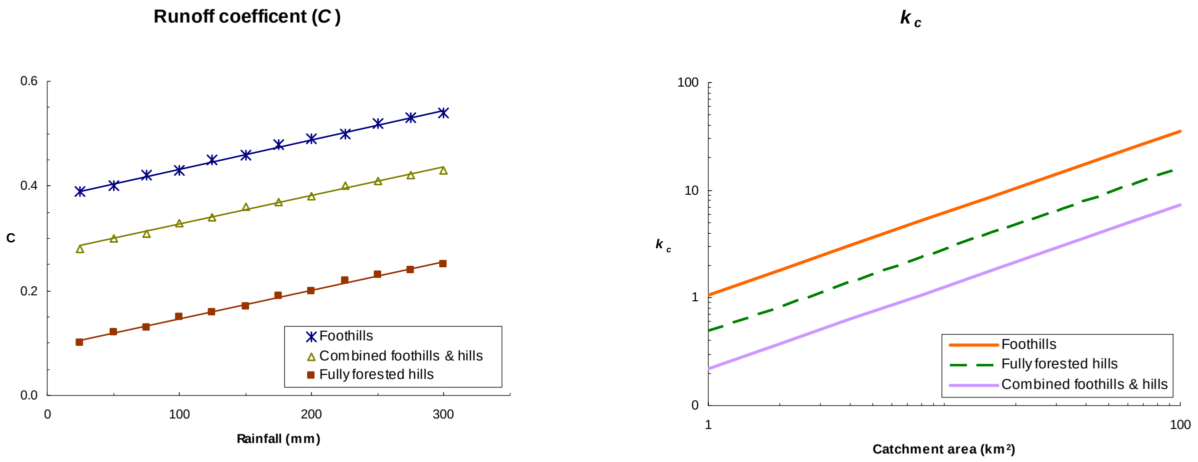
This relationship is valid for catchments <100 km^2 in the south west of Western Australia. A constant value of C is used for each catchment. The calculated runoff coefficient excludes baseflow.

These relationships are plotted in Figure 17.

Baseflow of 30% was added to direct runoff.

A value of m = 0.85 was adopted for all simulations.

Figure 17 Regional RORB parameter relationships



Runoff coefficent (*C*)

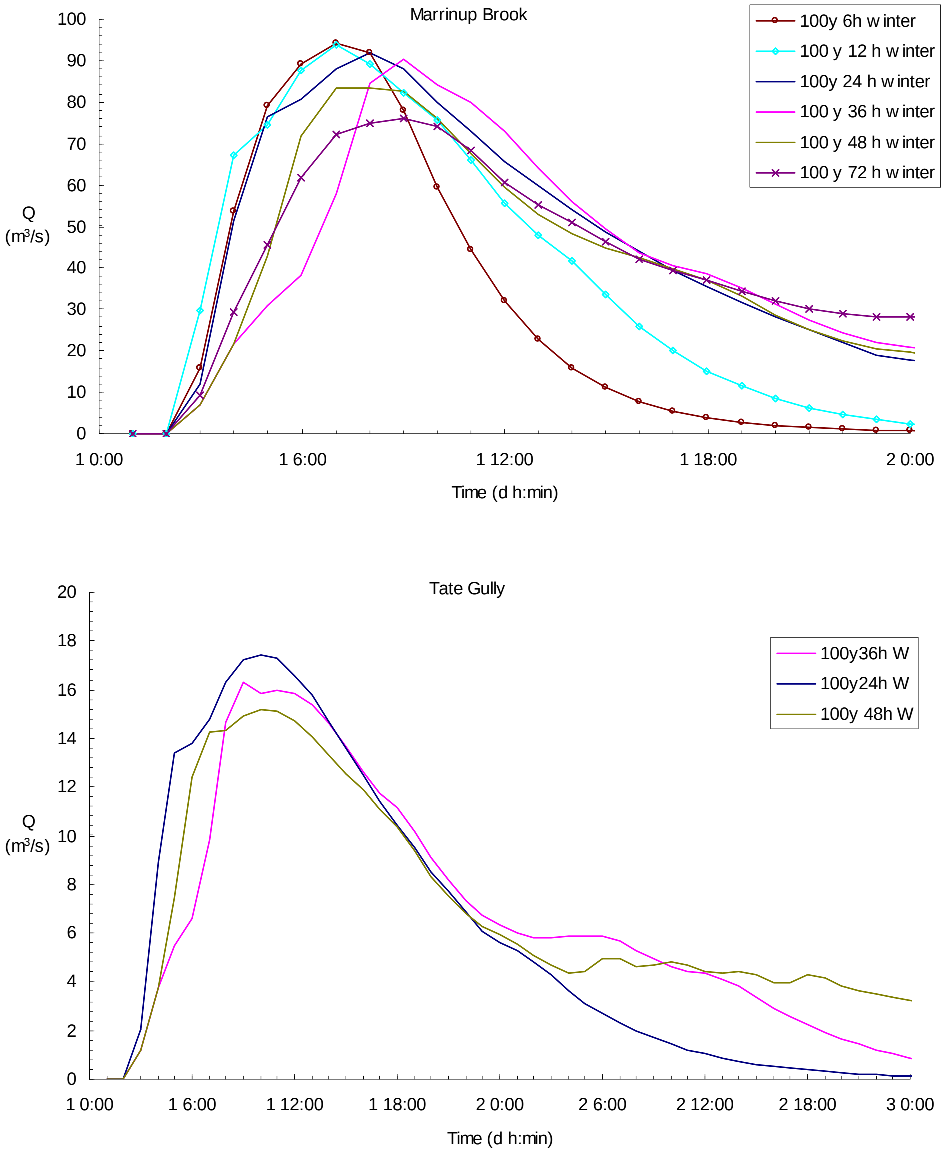
k

CRC FORGE winter rainfall for a 36 hour duration, areally reduced for the whole Hill and flood study area (800 km^2) was used for the design event simulations. Critical duration for the Hills Catchments will vary from this, depending on catchment size and location (foothills or forested), however a 36 h duration is a reasonable compromise between capturing a realistic local peak discharge and an appropriate volume contribution to the flood study area. Effects of varying rainfall duration on hydrograph shape is illustrated for several Hills Catchments in

Figure 18. Peak discharge is relatively insensitive but volume for shorter duration events is considerably less than for the 36 h event. Sensitivity of adopting a 36 h duration on predicted flood extent and water levels throughout the flood study area is evaluated in Section 4.5.

The RORB models ended at the hydraulic model boundary but many of the catchments continued into the flood study area, hence critical duration for the whole catchment may be longer. There are also smaller catchments contained fully within the flood study area that may have shorter critical durations. It is suggested that the critical duration for high priority, individual catchments be evaluated in more detail.

Figure 18 Effect of varying rainfall duration on hydrograph shape for several Hills Catchments



4.3.4 Verification

Peak discharge predicted using the regional method was verified by comparison with the available stream flow data (Table 11). The RORB predictions agreed reasonably closely with the observed data.

Data for stations 614003 Marrinup Brook Brookdale Siding, 614009 Oakley Brook and 614068 North Dandalup River North Dandalup were used. The regional curves used for each station were selected based on the amount of clearing in each catchment. Fully forested parameters were used for Marrinup Brook and the 50% clearing curves for Oakley Brook and North Dandalup River.

The period of record at the Oakley Brook and North Dandalup sites are short and the flood frequency results at these sites, particularly for the larger events, is not considered reliable.

Gauged streamflow information for a number of small (< 3 km^2) catchments is available but is of limited use as the catchments are completely forested and most of the Hills Catchments flowing into the flood study have all been subjected to relatively high levels of clearing.

The parameterised model was used to produce hydrographs at the hydraulic modelling area boundary for the both the calibration and design events.

Table 11 Hills Catchments model verification

AEP for station (1 in Y)	Area (km^2)	Period of record	Fraction cleared (%)	Peak discharge (m^3/s)	
				Flood frequency	RORB
614003 Marrinup Brook Brookdale Siding:	45.6	1969-present	2		
1 in 10				14	11
1 in 100				25	22
614009 Oakley Brook	46.2	1970-1985	54		
1 in 10				15-30	39
1 in 100				20-70	67
614068 North Dandalup River North Dandalup	14.6	1992-1996	33		
1 in 10				10	14
1 in 100				-	24
614025 Marrinup Brook Tributary Umbucks Catchment	3.3	1978-1990	0		
1 in 10				4	11
1 in 100				10	20

4.3.5 Application

The parameterised model was used to produce hydrographs at the flood study boundary for both the calibration and design events.

CRC FORGE winter rainfall for a 36 hour duration, areally reduced for the whole Hills Catchments and flood study area (800 km^2) was used for the design event simulations. Baseflow of 30% was added to direct runoff.

Table 12 summarises the design flood estimates for the Hills Catchments.

Table 12 Predicted design event inflows to the flood study area

Catchment Name	Catchment Code	Area (km^2)	Peak discharge (m^3/s) for AEP (1 in Y)					Event volume (GL) for AEP (1 in Y)				
			5	**10**	**25**	**100**	**500**	**5**	**10**	**25**	**100**	**500**
Conjurunup Creek	CC	56.2	8.4	8.4	15.5	22.5	33.2	0.68	0.93	1.19	2.54	3.37
Marrinup Brook	MB	77.7	38.5	51.8	65.2	90.3	124.7	2.24	2.93	3.62	4.88	6.57
Murray River banks	MR2 banks	25.9	9.6	12.7	15.8	22.2	30.1	0.98	1.28	1.57	2.15	2.85
Murray River tributary	MR2T1	30.6	10.5	14.0	17.6	24.9	34.2	1.16	1.51	1.85	2.54	3.37
Murray River tributary	MR2T2	8.4	6.1	8.1	10.1	13.8	13.8	0.24	0.31	0.39	0.52	0.52
Murray River tributary	MR2T3	4.2	1.4	1.9	2.4	3.4	4.7	0.16	0.21	0.26	0.35	0.47
Nambeelup Brook tributary	NBT3	3.2	1.1	1.5	1.9	2.6	3.6	0.12	0.16	0.20	0.27	0.36
Nambeelup Brook tributary	NBT3T1	2.8	1.0	1.3	1.6	2.3	2.7	0.11	0.14	0.17	0.23	0.27
Nambeelup Brook tributary	NBT4	2.3	1.7	2.2	2.7	3.7	3.7	0.06	0.08	0.10	0.14	0.14
North Dandalup River, lower	NDR	14.6	10.8	14.3	17.8	24.3	33.2	0.42	0.55	0.68	0.91	1.23
North Dandalup River tributary	NDRT1	7.2	5.3	7.0	8.7	11.9	11.9	0.21	0.27	0.33	0.45	0.45
North Dandalup River tributary	NDRT2	10.2	8.1	10.8	13.4	18.3	25.0	0.29	0.38	0.47	0.64	0.86
North Dandalup River tributary	NDRT3	2.4	0.8	1.1	1.4	2.0	2.7	0.09	0.12	0.15	0.20	0.27
North Dandalup River tributary	NDRT4	7.5	5.5	7.3	9.1	12.4	12.4	0.21	0.28	0.35	0.47	0.47
Oakley Brook	OB	47.0	29.2	39.1	49.0	67.4	78.9	1.35	1.77	2.18	2.94	3.41
South Dandalup River, lower	SDR	44.3	6.7	9.5	12.5	18.1	26.8	0.54	0.73	0.94	1.31	1.87
South Dandalup River tributary	SDRT1	1.6	0.6	0.7	0.9	1.3	1.8	0.06	0.08	0.10	0.13	0.18
Serpentine River	SR	1,684	39	66	79	120.7	158	11.8	19.2	24.2	40.7	53.5
Tate Gully	TG	18.9	7.5	9.8	12.0	16.3	22.2	0.60	0.76	0.91	1.19	1.77

4.4 Serpentine River

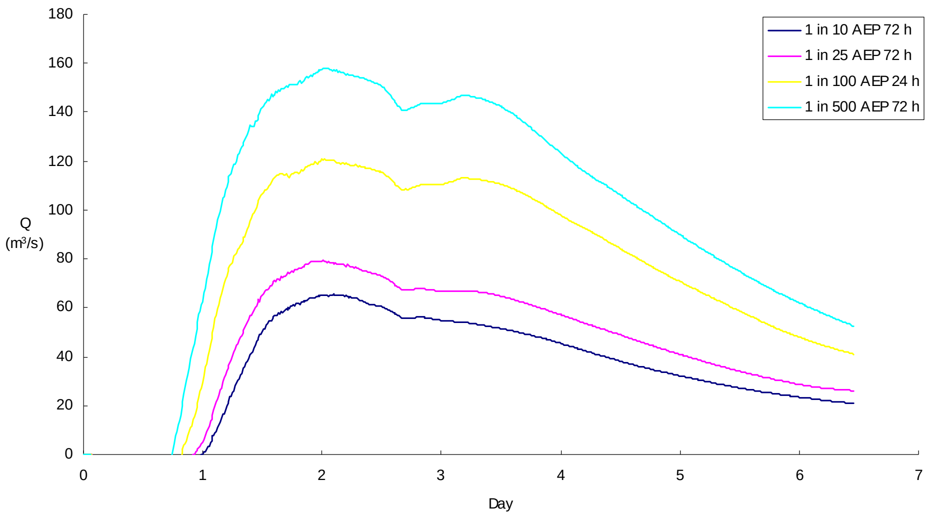
The Serpentine River enters the Murray flood study area at Yalbanberup Pool, upstream of the Kwinana Freeway crossing. A floodplain management study for the lower Serpentine River was recently undertaken for the Department of Water (SKM in prep.). Inflows to the Murray Area flood study area were taken directly from the SKM study.

The Serpentine River Floodplain Management Study found:

- The catchment are above Serpentine Dam is not expected to contribute to major flooding up to the 1 in 500 AEP event.
- Runoff routing modelling (RORB) was used to determine 1 in 10, 25 and 100 AEP flow estimates.
- A Mike 21 hydraulic model of the floodplain was developed and calibrated to an event in Jul 1987
- Hydraulic modelling illustrated that the existing drainage network has significant influence on flows in the area.
- Existing levees in the Baldivis area are expected to contain the 1 in 25 AEP flood event but will not fully contain the 1 in 100 AEP event.
- Design flood estimates are generally smaller than the previous mapping by the WA Water Authority, prepared in 1993.

The SKM study (in prep.) produced hydrographs at the Murray Area flood study boundary for a number of design events (1 in 25, 100 and 500 AEP) and for an observed event in 1987. Complete hydrographs for these events were not available from the SKM modelling but the duration available was adequate for determining peak water levels in the Murray flood study area so were used as is. The SKM study did not produce 1 in 5 nor 1 in 50 AEP predictions; hydrographs for these events were extrapolated using a log-normal model fitted to the peaks. The adopted hydrographs are shown in Figure 19 and a summary of events characteristics is given in Table 12.

Figure 19 Serpentine River design event hydrographs



4.5 Design flow estimates within the flood study area

Typically in flood studies runoff from rainfall within the study area is ignored because the magnitude of flooding from local stormwater is negligible compared with flooding caused by a much larger upstream catchment. In this study generation of local runoff was considered important because of the large flood study area, the diffuse nature of the flood prone areas within the study area, and a requirement for definition of flood extent away from the Murray and Serpentine Rivers for input to the DWMP. Runoff within the hydraulic modelling area was simulated within the Mike 21 model as part of the flood hydraulics simulation (see Section 6).

Streamflow generation through most of the flood study area was simulated by entering a rainfall excess into the Mike 21 simulation. The hydraulic model simulated flow hydraulics, routing the excess rainfall overland and in channels through the flood study area, mixing with flood hydrographs input at the edges of the model.

No rain was applied in the flood study area south of Pinjarra South as this area has a defined catchment and was readily simulated using RORB, as described in Section 4.3.

The rainfall excess was generated by subtracting a proportional loss with the parameters derived using the regional parameterisation method described in Section 4.3.3. Parameters for the foothills was used.

Winter rainfall with a duration of 36 h, areal reduced for the area of the flood study plus and Hills Catchment (800 km^2) was used. It is recognised that the critical duration for points within the flood study area will vary depending on their contributing catchments but a consistent duration across the whole area was thought to be more robust than varying duration across the flood study area and Hills catchments. A number of different durations were evaluated and 36 h was found to provide a realistic representation of the contribution of the Hills catchments to flooding in the study area. Peak water levels along key streamlines did not vary noticeably for rainfall durations varying from 24 to 48 h.

Groundwater levels in the area are relatively shallow and the general area can be subject to periodic inundation at certain times of the year. A parallel study is currently underway to define the surface water/groundwater interaction within the flood study area. The range and average proportion of the hydraulic modelling area that is subject to inundation will provide guidance on likely runoff rates for the area. These results will be used to help verify/refine the rainfall loss model adopted. Further work utilising the results of the SW/GW study has commenced and will be incorporated into the final report.

More detailed discussion of the hydraulic modelling is provided in Section 6.

4.6 Climate change

Climate change is an emerging issue, and although its existence is widely accepted, the effects on design rainfall and flood estimates are unclear and unquantifiable at the present time.

Potential climate change impacts may affect this project and some of these impacts could be significant. These could include, without limitation, impacts on the physical, climatic, commercial and/or social setting of the project. They may vary in magnitude, timing, duration and distribution and may have specific, cumulative and/or collateral impacts. These effects may impact on the operation, functionality, performance and durability of the project beyond what can be reasonably predicted with the current available knowledge.

Global climate change models (e.g. as summarised in Climate Change 2009), depending on the choice of model, tend to predict a reduction in annual rainfall in the southwest of Western Australia. The median scenario indicates a reduction of 5 to 10 % for the southwest of Western Australia. This reduction could affect flood flows (particularly in winter) by decreasing the antecedent wetness of the catchment at the onset of a major rainfall event. Summer events occur on catchments that are already relatively dry and hence the impact of a reduction in annual rainfall is likely to have a lesser impact.

Daily precipitation intensity and extreme precipitation intensity (99th percentile daily) is predicted to decrease in the southwest of Western Australia (Climate Change 2009). The decrease is predicted to be more significant in winter/spring while for annual, summer and autumn the decrease is significantly less pronounced.

The seasonal flood frequency approach adopted showed that both winter and summer events have similar weighting at about 1 in 100 AEP. A decrease in winter flooding is possible with drier catchments even with no change in design rainfall estimates. However, no change or a slight increase in summer estimates is possible. Based on the current research, there is unlikely to be a significant change in design 1 in 100 AEP floods under a changing climate. However, the more frequent design floods (smaller than the 1 in 50 AEP), that are currently winter dominant may reduce in line with the predicted reductions in 99th percentile precipitation intensities during the winter /spring.

The results for the range of design events investigated provides insight into the likely sensitivity of the results to the impact of climate change with the 1 in 50 AEP 1 and in 500 AEP possible representing the bounds of potential changes in the design 1 in 100 AEP flow under a changing climate.

This project has been undertaken in accordance with the guidelines given in Australian Rainfall and Runoff (Pilgrim 2001) and CRC FORGE (DoE 2004). The design rainfall and flood estimates provided within this report do not include an additional allowance for the possible effect of climate change into the future.

5. Tide and storm surge

5.1 Introduction and background

5.1.1 Characteristics of the Peel Inlet / Harvey Estuary

The study area includes the shallow coastal plain around the perimeter of the Peel Inlet / Harvey Estuary from the Murray River floodplain in the east to the Dawesville and Mandurah Channels in the west and north, respectively. The estuary is comprised of an approximately circular basin of the Peel Inlet (10 km fetch) and the narrow and elongated Harvey Estuary. Combined they form a P-shape estuary. The two basins are linked by a shallow constriction that includes a narrow navigation channel. Though the maximum depth in the Peel Inlet is 2.5 m, most of the basin is shallower than 1 m. Depth in the Harvey Estuary is approximately 2 m.

Freshwater enters the system predominantly in winter from three rivers: Murray, Serpentine, and Harvey rivers.

The estuary is connected to the Indian Ocean through two channels, namely:

- The natural Mandurah Channel (4 km long, 1-2 m depth, and 100 m width) that is dredged to accommodate recreational boats; and
- The man-made Dawesville Channel (2.2 km long, >3 m depth, 200 m width), dredged in 1994 to improve the flushing characteristics of the system. This channel is often referred to as ‘the cut’.

5.1.2 Components of flooding in the Peel Inlet / Harvey Estuary

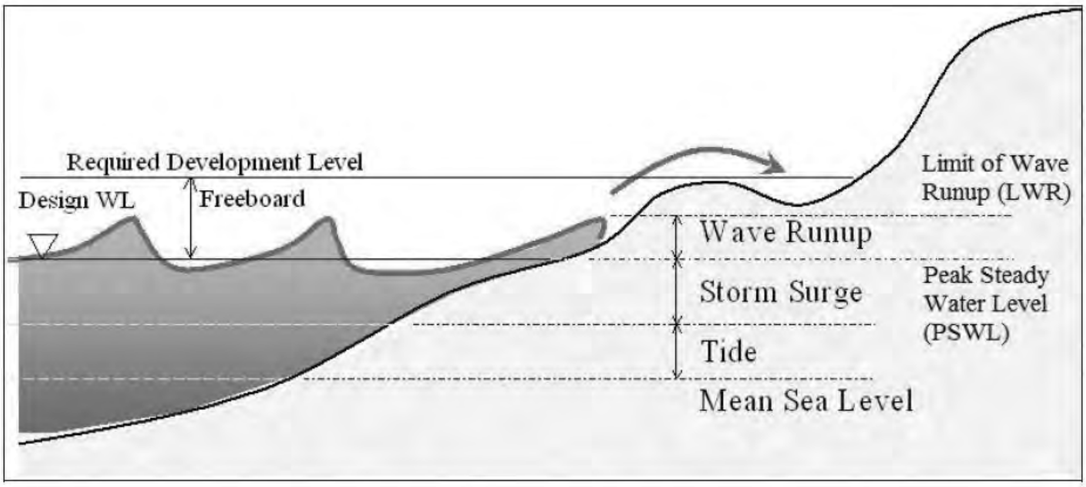
Water level fluctuations in coastal areas are mainly caused by the combined effects of astronomical tides, storm surge, tsunamis, basin oscillations, and other climate effects such as long-term sea level rise.

The non-tidal component of the water level, estimated as the difference between measurements and astronomical tides, is called ‘residual’. If a particular event, such as a storm, generates a large non-tidal component, the residual is called ‘storm surge’. For this reason, tsunamis and cyclones may be classified as severe storm surges. However, the effect of these events is considered rarer than the frequency bounds set for this project and consequently have not been assessed.

In estuaries, such as the Peel Inlet / Harvey Estuary, additional factors relating to the local wind and waves and inflow from streams can also affect water levels.

Simplistically, the three key features that contribute to flooding on the Peel Inlet /Harvey Estuary are the tide, storm surge and the wave runup (Figure 20).

Figure 20 Key Components contributing to flooding on the Peel Inlet / Harvey Estuary



Tide

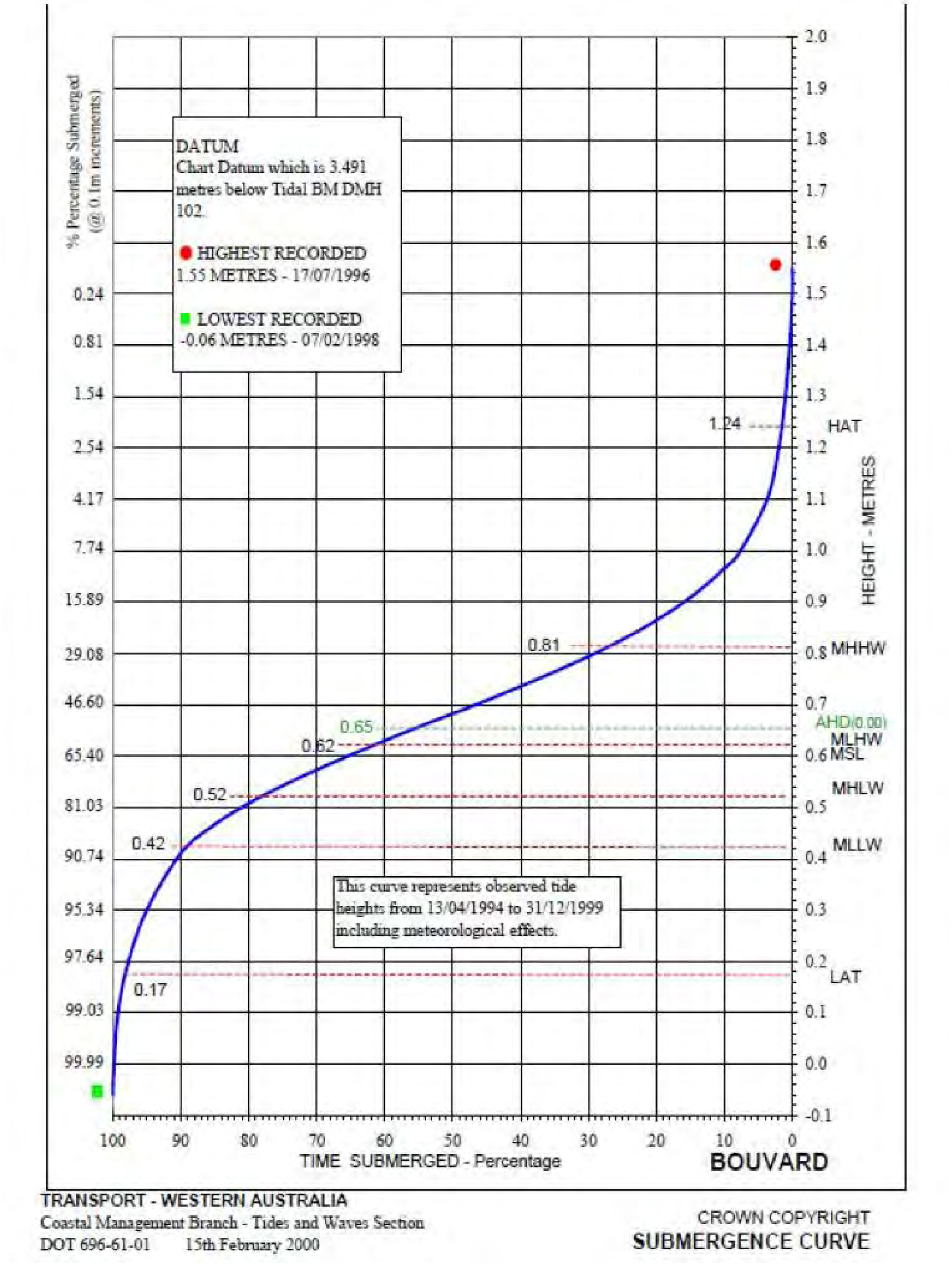
Astronomical tides are the response of the ocean to the gravitational attraction forces existing in the earth-moon-sun system. Although this response is complex with features varying in the estuary spatially and seasonally, sufficient information is available to predict the astronomical tide.

Tides in the Indian Ocean, outside of the estuary, are mainly diurnal with typically one high and one low tide in a 24 hour cycle. Tides in the area are also very similar to those at Fremantle (Hearn et al. 1994; Damara 2009) and are classified as micro-tides due to low range of approximately 0.8 m.

The tides affecting the Peel Inlet / Harvey Estuary vary seasonally with winter tides approximately 0.22 m higher than those in summer.

The tidal submergence curve for the Bouvard tide gauge within the Peel Inlet / Harvey Estuary shows the variation in the predicted tide level expressed as a percentage of time a given water level is expected to be exceeded (Figure 21). The submergence curve illustrates that the median water level (50 %) is approximately 0.67 metres above the chart datum (or 0.02 m AHD). The mean high tide (MHHW) and highest predicted tide expected are 0.81 m and 1.24 m above chart datum, respectively.

Figure 21 Tidal submergence curve for the Bouvard tide gauge



Source: Department of Transport.

Storm Surge

Storm surge is the temporary increase, at a particular locality, in the height of the sea due to extreme meteorological conditions (low atmospheric pressure and/or strong winds). The storm surge is defined as being the excess above the level expected from the tidal variation alone at that time and place.

Low atmospheric pressure causes the water level to increase while high atmospheric pressure reduces water levels. The increase in water level is about 10 mm for each millibar drop in atmospheric pressure.

Generally, the relative magnitude of storm surge depends on the time of the year and on the local bathymetry. Furthermore, meteorological disturbances are greater in winter and have greatest effect in shallow areas (i.e. wave run up and wind set up). Rising regional sea levels due to climate change will worsen the impact of these events by increasing the base sea level on which they occur.

The maximum storm surge observed in the Peel Inlet / Harvey Estuary occurred on 16 May 2003 and resulted in observed water levels of approximately 0.7 metres above the predicted tide.

Wave run up

Within the Peel Inlet / Harvey Estuary the prevailing winds act on the water surface to produce higher water levels at the downwind shore, and lower water levels at the upwind shoreline. This process is known as wind set-up.

In addition to the wind set-up there is a second process related to the waves generated within the water body, known as wave runup. This is the vertical extent of wave uprush on a shore or structure above the still water level. In the Peel Inlet / Harvey Estuary the relatively short fetch distance (open water distance) and shallow depth limit the magnitude of the combined impact of wind setup and wave runup.

5.1.3 Previous Studies

Murray River Flood Study (PWD 1984)

The study used estimated 1 in 25 AEP and 1 in 100 AEP design water levels based on the application of a Murray River hydrograph to the Estuary. Discharges from the Estuary were estimated using a hydraulic analysis on a representative cross-section, within the Mandurah Entrance Channel, with the downstream level controlled by tide.

There was no tide information available for Mandurah at the time of the study and information from Bunbury was used. The water level in the Peel Inlet / Harvey Estuary was found to be relatively insensitive to the tide. Increasing the tide by 0.5 metre resulted in only a 0.05 metre increase within the Peel Inlet / Harvey Estuary.

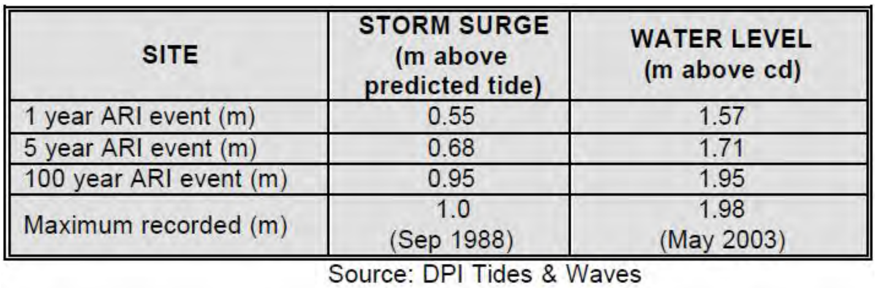
The study did not include inflows / discharges through the Dawesville Channel as this was not constructed until the 1990s. Inflows from the Serpentine River and Harvey River were not included directly but were incorporated through adopting a slightly higher starting water level.

The study estimated the 1 in 100 AEP water level in the Peel Inlet / Harvey Estuary at 1.60 m AHD. To ensure adequate flood protection, the study recommended a minimum habitable floor level of 2.20 m AHD for proposed developments adjacent to the Peel Inlet / Harvey Estuary.

Port Beach Coastal Erosion Study (DPI 2004)

This study evaluated the extent and cause of erosion at Port Beach and identified longer-term coastal management options. An evaluation of tide and storm surge at Fremantle is also given.

Astronomical tides at Fremantle are predominantly diurnal and relatively small, with a daily range of 0.5 and 0.3 m during spring and neap tides, respectively. The highest astronomical tide at Fremantle is 1.27 m (referred to Chart Datum, CD). Interannual variations of 0.15 m in mean sea level occurred in response to El Niño (e.g. MSL = 0.7 m CD in 1997) and La Niña (e.g. MSL = 0.85 m CD in 1999) effects. Particularly high water levels may result from the coincidence of storm surge and high tides, as occurred during a storm in May 2003 when a surge of 0.8 m was coincident with a high tide of 1.2 m CD. DPI (2004) return period estimates of extreme events at Fremantle of relevance to this project are shown below:



SITE	STORM SURGE (m above predicted tide)	WATER LEVEL (m above cd)
1 year ARI event (m)	0.55	1.57
5 year ARI event (m)	0.68	1.71
100 year ARI event (m)	0.95	1.95
Maximum recorded (m)	1.0 (Sep 1988)	1.98 (May 2003)

Source: DPI Tides & Waves

The design water level estimated by DPI (2004) adopted a mean sea level rise due to climate change of 0.38 m for planning purposes in Western Australia.

Mandurah Region, Development in Floodprone Areas, Review of Available Information and Existing Policies (Damara 2009)

This draft report for the Department of Transport summarises the relevant information and policies for the various flooding mechanisms that affect the Mandurah Region. The study includes discussion on coastal, estuarine and riverine flooding.

The study examined the available water level data for the Peel Inlet / Harvey Estuary and contains a detailed comparison of the data to the long term record available at Fremantle.

Importantly, the study found that the coherence between tides at Peel Inlet / Harvey Estuary and Fremantle suggest that the processes identified in the Fremantle record are relevant to the Mandurah region with the following key features (Damara 2009):

- The micro-tidal diurnal cycle presents solstitial peaks in June and December (Davies 1980).
- Significant meteorological surges are associated with low barometric pressure systems and westerly storm events.
- Continental shelf waves are generated by tropical cyclones that result in minor surges.
- Seasonal mean sea level ranges over approximately 0.22 m in association with the Leeuwin Current (Pattiaratchi & Buchan, 1991).
- The inter-annual mean sea level variability responds to ENSO phenomenon.
- The 18.6-year lunar nodical tidal cycle peaked during 2007-2008.

Frequency analyses on the available annual peak water level data were undertaken. In addition, an analysis of the recorded data for the Peel Inlet and Harvey Estuary plotted against the estimated return interval at Fremantle. Based on the analyses, the recommended design 10 year ARI (1 in 10 AEP) and 100 year ARI (1 in 100 AEP) peak water level within the Peel Inlet / Harvey Estuary are approximately

1.0 m AHD and 1.20 m AHD, respectively. The study recommends using a vertical increase in water level of 0.4 metre to account for sea level rise.

Sea Level Change in Western Australia: Application to Planning (Department of Transport, 2010)

The report summarises the latest tide gauge and satellite altimeter data from around the world that show that the sea level is generally increasing at approximately 1.5 mm/year over the last century. The rate is consistent with the high emission scenarios provided in the latest IPCC report (2007). The report also examines the latest projections of global and regional sea level rise that suggest there is a greater likelihood that the rate of sea level rise will exceed the majority of the projections provided in IPCC (2007). This report underpins a recent decision of the Western Australian Planning Commission (WAPC Minutes of Ordinary Meeting 173, Item 12.3, 25 May 2010), to recommend the adoption of a vertical sea level rise of 0.90 metre within the State Coastal Planning Policy (SPP 2.6) to allow for the impact of coastal processes over a 100 year planning timeframe (2010 to 2110).

5.2 Hydrodynamic modelling

A hydrodynamic model of the Peel Inlet / Harvey Estuary was prepared to assist in understanding how the peak water level may vary with location within the estuary and assess the sensitivity of water levels to the astronomic tide and river discharge.

Because of the shallow depth in the Peel Inlet and Harvey Estuary the waters are typically vertically well-mixed so a two-dimensional (2D), vertically-averaged hydrodynamic model is appropriate for water level modelling.

Simulations were undertaken with DHI's MIKE 21 HD-FM numerical model to provide the water levels in the region of the confluence with the Murray River. MIKE 21 HD-FM is part of MIKE 21, a comprehensive vertically-averaged coastal modelling package developed and supported by DHI Group, Denmark. The model is a numerical tool for simulation of water levels and flows in estuaries, bays and coastal areas that is based on a flexible mesh approach. The flexible mesh approach allows for grid elements of varying dimensions to be combined to readily construct model domains with complex shorelines and intricate channels as in this study.

MIKE 21 is based on the numerical solution of the two-dimensional incompressible Reynolds-Averaged Navier-Stokes equations with Boussinesq and of hydrostatic pressure assumptions. It simulates unsteady two-dimensional flows in a vertically homogenous water body.

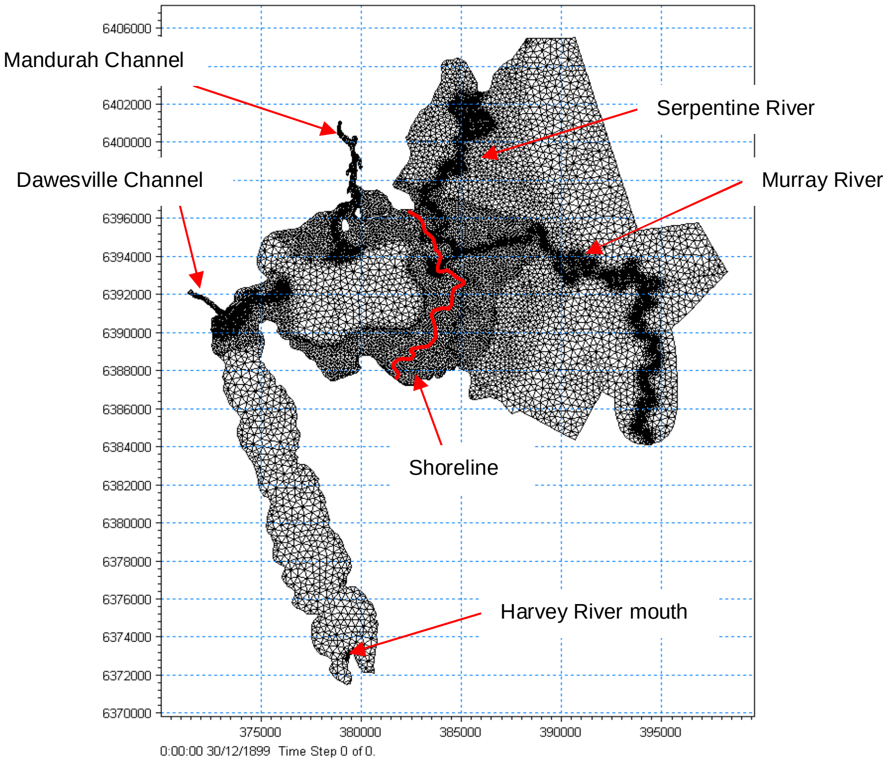
The advantages of utilising a hydrodynamic model of the Peel-Harvey Estuary to determine tailwater conditions for the hydrological model are three-fold:

- The dynamic nature of numerical modelling studies enables time-varying conditions such as flooding and drying of inundated areas to be accounted for.
- The water level signal at any point within the model domain can be extracted from the model at any given time or period. This gives flexibility in the selection of tailwater conditions at the confluence between the hydrodynamic and hydrological models.
- Processes that are difficult to uncouple in a desktop study such as basin scale oscillations and wind set up are explicitly accounted for in the hydrodynamic model.

5.2.1 Model setup

The hydrodynamic model uses a flexible mesh grid with elements of varying sizes (Figure 22). Higher spatial resolution (i.e. smaller element area) was utilised in river channels and along the coast in regions requiring higher spatial resolution. Element sizes range from 5,000 m^2 in the channels to 80,000 m^2 at the centre of the inlet. The grid has 21909 elements and 11450 nodes.

Figure 22 Flexible mesh of the Peel-Harvey Estuary and Murray and Serpentine River floodplains

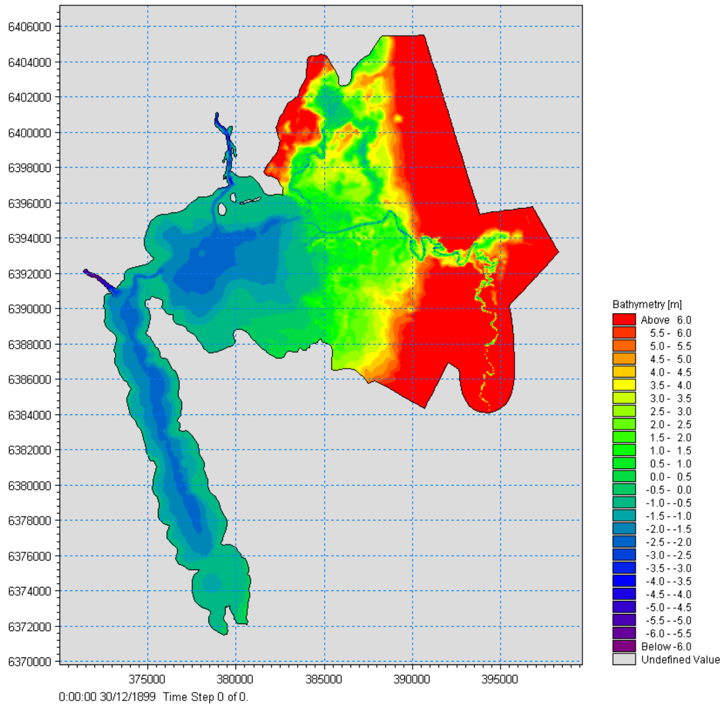


Bathymetric data for the project was derived from three sources including:

- Mike C-Map data (DHI's digital Global ocean depth repository) – digital land boundaries and water points for the estuary region at equivalent resolution to that available on navigation charts;
- Navigation Charts – used to verify the land boundaries and to increase the resolution of the Mike C-Map data at regions of interest (e.g. dredged channels); and
- Lidar data – Contours at 1 m intervals for the Murray and Serpentine River floodplains.

The bathymetry was then interpolated to the horizontal flexible mesh. The resulting digital elevation model, referred to mean sea level, is shown in Figure 23.

Figure 23 Model bathymetry



5.2.2 Boundary conditions

Tidal forcing and storm surge

Water elevations at hourly intervals were forced at the entrances of the Mandurah and Dawesville channels. Water levels consisted of the combination of predicted tides constructed for the five main tidal harmonic constituents as reported in Ranasinghe and Pattiaratchi (2000) and storm surge levels for each risk scenario.

Wind forcing

Three wind conditions were considered by adopting a spatially uniform wind field over the surface of the whole model domain, namely:

1. No wind condition, representative of calm days.
2. Winds from Mandurah Ocean Marina station, for calibration runs.
3. Maximum wind event on a 7.5-year record (from November 2001) at Mandurah Ocean Marina to consider maximal wind set up effects.

River inflows

The 1 in 100 AEP hydrographs of the Murray and Serpentine Rivers served as inputs to the model in the scenario that considered the coexistence of storm surge and river flooding. For this scenario the inflow from the Harvey River was set to its mean discharge. This assumption is consistent with Murray and Serpentine River inflows being out of phase with respect to the Harvey River inflow at the time they enter the basin.

Other model setup parameters

Seasonal sea level fluctuations due to ENSO were not specifically accounted for.

In spite of the Peel Inlet / Harvey Estuary's relatively short north-south extension, the effect of the earth's rotation on currents (i.e. Coriolis force) was simulated in the model.

Because of the relatively small extent in which barometric set up in the basin can occur and its secondary importance relative to storm surge and wind set up, a constant atmospheric pressure was applied in all the simulations; and

The model wetting and drying capabilities were switched on to accommodate the dynamic nature of storm surge and river flooding.

Two mean sea level rise cases from climate change were accounted for in a subset of the scenarios - 0.4 m and 0.9 m. These scenarios were based on the current advice in the State Coastal Planning Policy (SPP 2.6, WAPC 2003 Amended Dec 2006) that recommends consideration of 0.38 metre sea level rise for planning and a recent decision of the Western Australian Planning Commission (WAPC Minutes of Ordinary Meeting 173, Item 12.3, 25 May 2010), to recommend the adoption of a vertical sea level rise of 0.90 metre within the State Coastal Planning Policy (SPP 2.6) to allow for the impact of coastal processes over a 100-year planning timeframe (2010 to 2110).

5.2.3 Model calibration

The model was initialised with zero currents and fluxes and the water level over the entire model domain was set to mean sea level (i.e. 'cold' start). A time step of 60 seconds was implemented, which is appropriate to simulate the low frequency motions investigated here (i.e. tidal and wind driven currents). The time step satisfied the CFL condition over the whole domain. The initial 4 days of the simulation (4 tidal cycles) was sufficient for the tidal and circulation patterns to become established from the 'cold' start initial condition after which simulated results were used for calibration and as inputs for subsequent catchment modelling.

The calibration modelling period spans over the month of December 2005 during the occurrence of the highest predicted tidal range for model calibration. A high tidal range yields the largest flood and ebbing current speeds and constitutes the greatest degree of wetting and drying, thereby representing the most challenging conditions for the model to capture. December is also a dry month when river discharges are marginal, hence the model can be calibrated when tidal and wind forcing are the dominant processes.

The hydrodynamic tidal model was calibrated with measured tidal data at the Peel Inlet and Harvey Estuary stations.

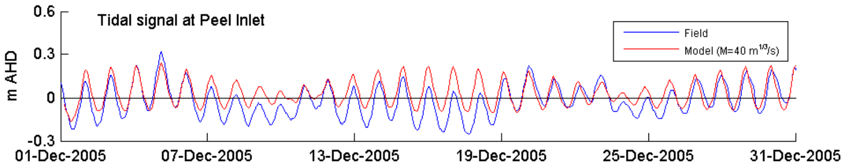
Calibration consisted of adjusting the bed friction coefficient in the standard quadratic friction law from within practical ranges. Model runs were tested with the Manning's number (M) in the range of 30-50 $m^{1/3}/s$ over the December 2005 simulation period. The calibration runs were forced with predicted tidal elevations at the channel entrances and a uniform wind field of Mandurah Ocean Marina measurements. No river inflows were inputted to the model during this calibration run.

Comparison of the simulated and recorded water levels at the Peel Inlet (Figure 24) show that the tidal phase is well captured in both sub-basins (Peel Inlet and Harvey Estuary), but there appears to be a periodic wave with a period of approximately 4 days that is not captured. This is attributable to an oceanic surge component that is not accounted for in the predicted tidal inputs. Calibration of such mesoscale processes requires the measured boundary conditions to serve as inputs. As the objective of the calibration was to acquire reasonable calibration with astronomical tides and to superimpose storm surge at the boundaries, the model calibration analysis emphasis was on accurate representation of the tidal variation (i.e. tidal range), as discussed next.

A range of bed friction coefficients were simulated and the accuracy of the model in capturing tidal ranges assessed with correlations between modelled and measured tidal ranges. The best agreement between simulated and measured tidal range was obtained with an M of 40 $m^{1/3}/s$ with a root-mean-square-error (RMSE) of 0.0533 m and 0.0648 m at Peel Inlet (Figure 25) and Harvey Estuary (not shown), respectively. The model was deemed calibrated as the RMSE of measured versus modelled tidal range during the period of highest tidal range was sufficiently low to be acceptable.

Figure 24 Comparison between simulated and measured tidal levels

a) Peel Inlet



b) Harvey Estuary

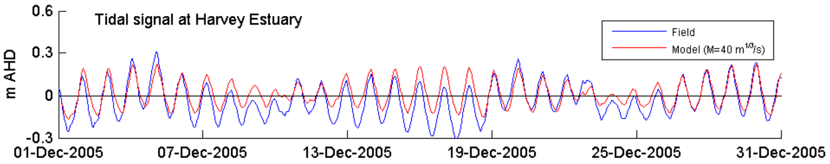
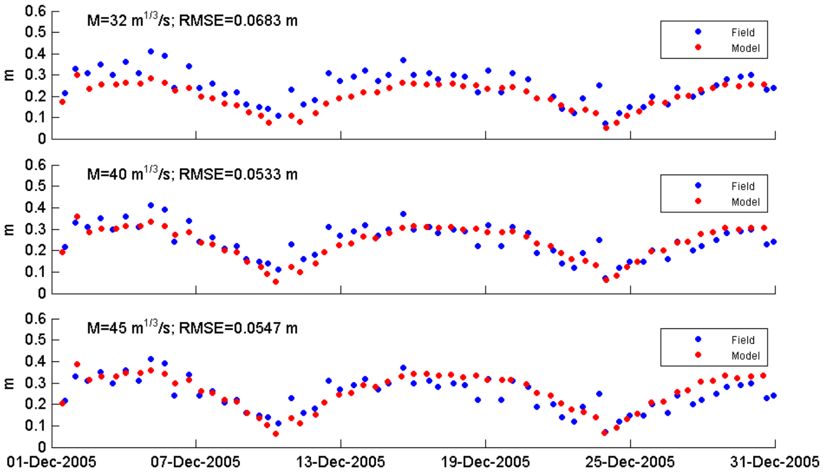


Figure 25 Comparison between simulated (red) and measured (blue) tidal range for an M of 32, 40, and 45 $m^{1/3}/s$ at Peel Inlet



5.3 Design scenario modelling

Predicted oceanic tides on the ocean side of the estuary were combined with surge and mean sea level rise to define the input water levels for the calibrated hydrodynamic model for a set of risk scenarios.

The modelling period was over 18 days (including 4 days for spin up) and was centred on the Mean High High Water (3 February 1991) and HAT (2 June 2007).

5.3.1 Development of design surge

Three important characteristics describe a surge, namely:

- Maximum residual (i.e. the maximum difference between measured and predicted tides).
- Duration.
- Shape (i.e. surge level as a function of time).

Maximum residual

The surge heights at Fremantle by DPI (2004) are illustrated in Figure 26. To a first approximation, surge heights for ARIs that were not reported by DPI (2004) were estimated through log-linear interpolation or extrapolation (red crosses in Figure 26). According to Pugh (1987), a valid extrapolation of risk scenarios is limited to 3-4 times the record length, hence use of the 500 year ARI surge level must be treated with caution. The resulting maximum residuals for a set of risk scenarios are presented in Table 13.

Figure 26 Surge levels above predicted tides

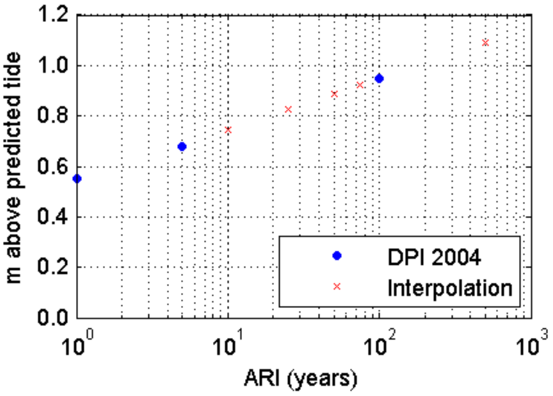


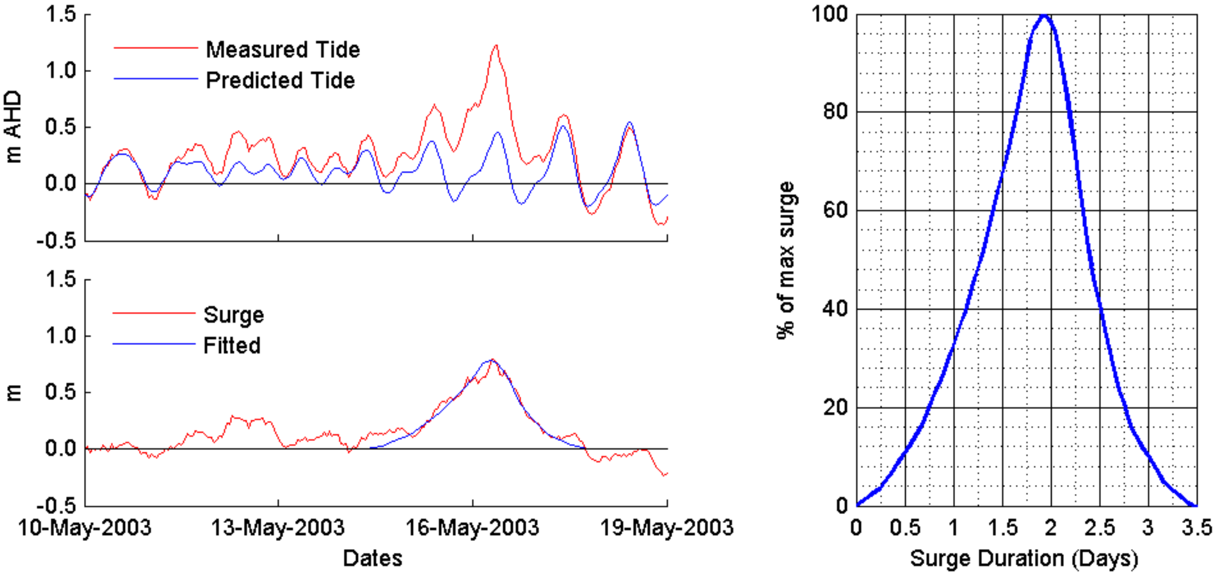
Table 13 Surge levels above predicted tides

ARI (y)	Height above predicted tide (m)	Source / Comment
5	0.68	DPI (2004).
10	0.75	Interpolation from DPI (2004).
25	0.83	Interpolation from DPI (2004).
50	0.92	Interpolation from DPI (2004).
100	0.95	DPI (2004).
500	1.08	Extrapolation from DPI (2004).

Duration and shape

An example of a storm surge at Fremantle during May 2003 is shown in Figure 27. This storm was catalogued as an approximate 100 year ARI event on the basis of total water depth and a 5-10 year ARI event on the basis of its residual. The shape of this surge was characterised by fitting a line to the surge (bottom panel of Figure 27). A non-dimensional version of the synthetised surge is shown in the right panel of the same figure. Firstly, the surge appears to have a Gaussian distribution, which is in agreement with general observations of surges (Pugh 1987). Secondly, the maximum surge height occurred 2 days after the surge began while the surge duration was 3.5 days. As other surges had a similar shape and duration (not shown), this was utilised to construct the design surges.

Figure 27 Recorded storm surge (upper left) and residual (lower left) at Fremantle on May 2003 (left) with a magnified view of the design surge characteristics (right).



5.3.2 Summary of scenarios

The list of scenarios that were run with the numerical model is given Table 14.

Table 14 Scenarios modelled

Simulation	Scenario	Surge at Fremantle [m above HAT]	Wind	Mean sea level rise [m]	Comments
D0	Propagation of predicted tides	0	No Wind	0.0	This scenario was to determine the time phase lag of the HAT in the absence of wind, surge or inflows.
D1	5 year ARI	0.68	No Wind	0.0	
D2	10 yr ARI	0.75	No Wind	0.0	
D3	25 yr ARI	0.83	No Wind	0.0	
D4	50 yr ARI	0.92	No Wind	0.0	
D5a	100 yr ARI	0.95	No Wind	0.0	Base condition for the 100-yr ARI

Simulation	Scenario	Surge at Fremantle [m above HAT]	Wind	Mean sea level rise [m]	Comments
D5b	100 yr ARI	0.95	Strongest in 10-yr record	0.0	The strongest wind in the 7 yr record at Mandurah Ocean Marina was uniformly applied to the whole domain
D5c	100 yr ARI	0.95	Strongest in 10-yr record	0.4	This scenario is equivalent to D5b but accounting for a mild sea level rise by 2090 (SRES B1)
D5d	100 yr ARI	0.95	Strongest in 10-yr record	0.9	As per D5c for a high sea level rise by 2090.
D5e	100 yr ARI	0.95	No Wind	0.0	Corresponding to simulation D5a but including inflow discharge through Murray and Serpentine Rivers with a The 100 yr ARI hygrograph.
D6	500 yr ARI	1.08	No Wind	0.0	

5.3.3 Scenario modelling results

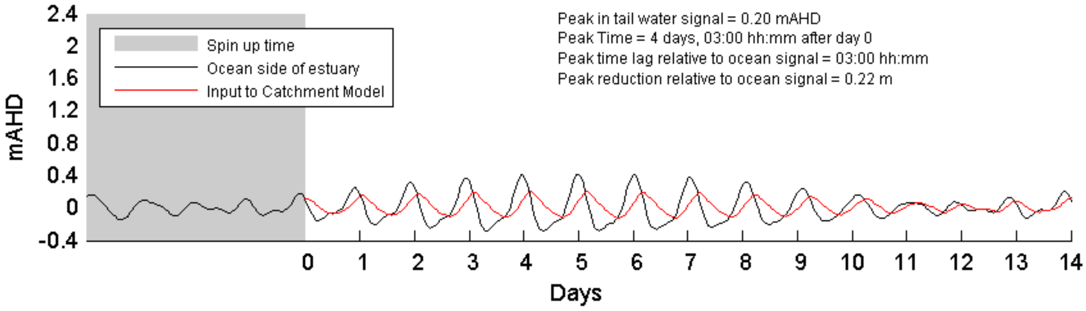
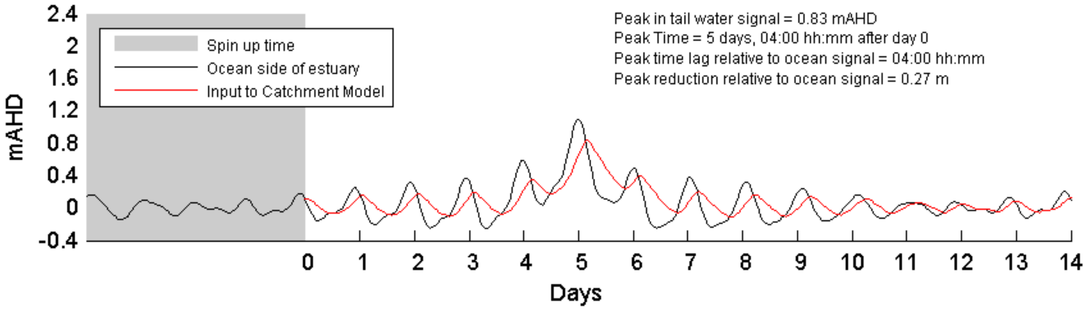
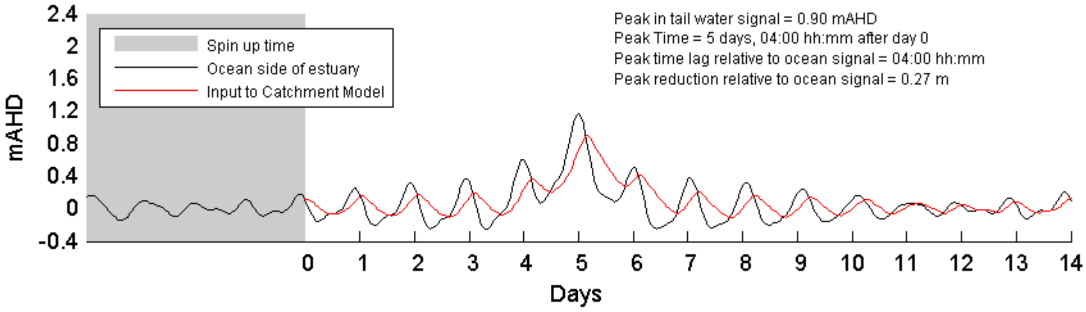
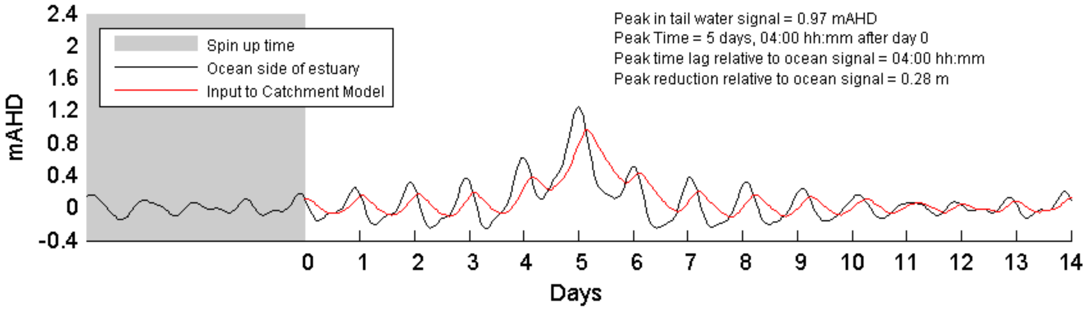
Simulations of the scenarios indicate attenuation of the maximum surge level in the inlet by approximately 20% as well as a 3-4 hour phase lag in the propagation of high water levels from the ocean to the downstream boundary of the catchment model (Table 15). These attenuation time scales are consistent with energy dissipation that occurs through the Dawesville and Mandurah Channels and along the shallow estuary.

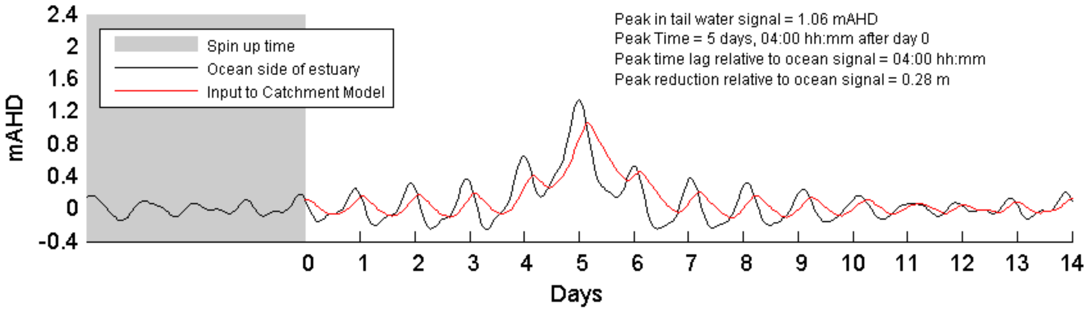
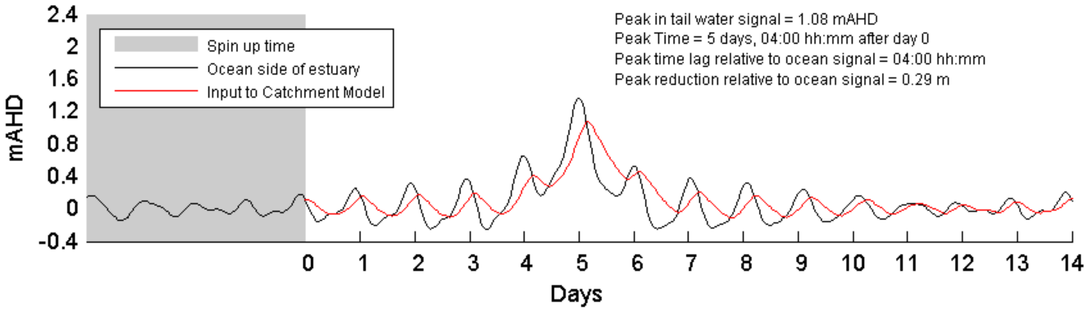
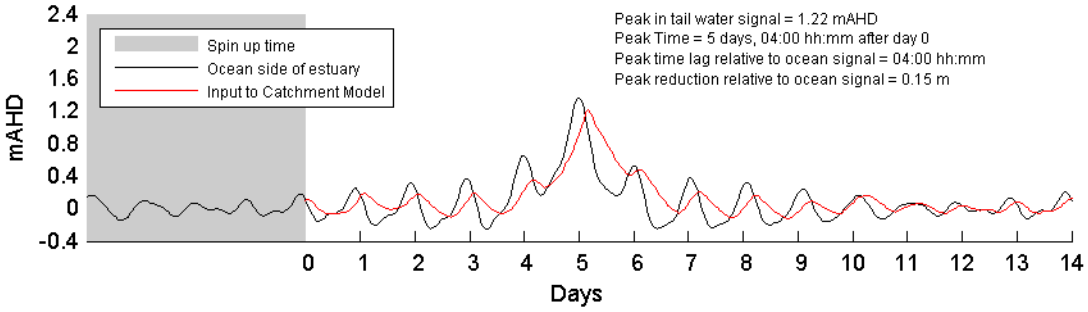
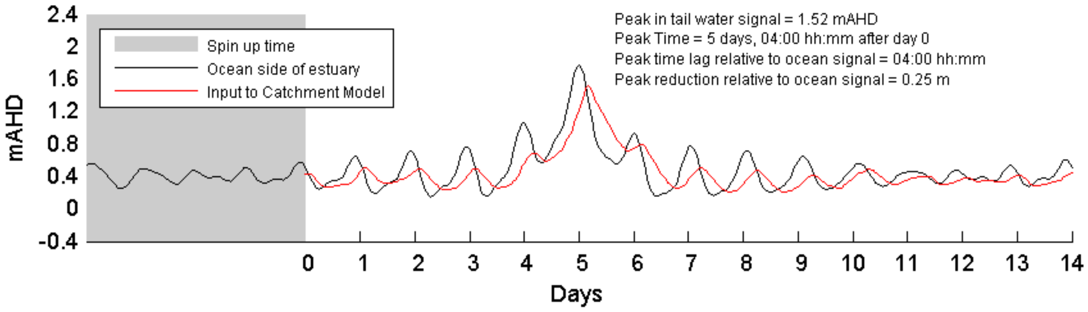
Table 15 Water level attenuation in the basin

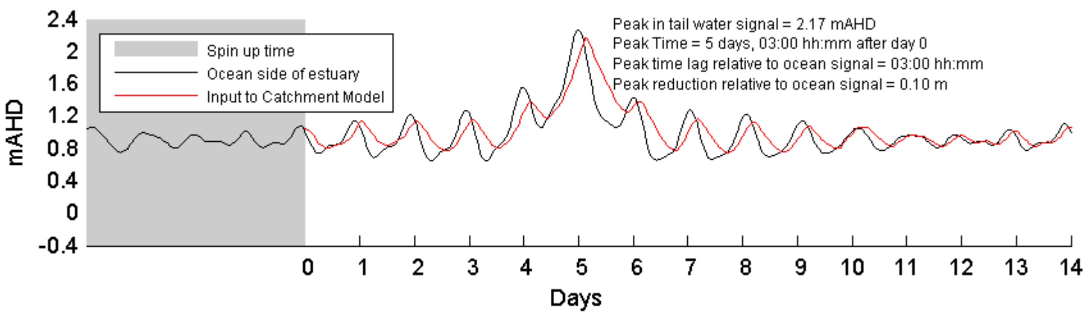
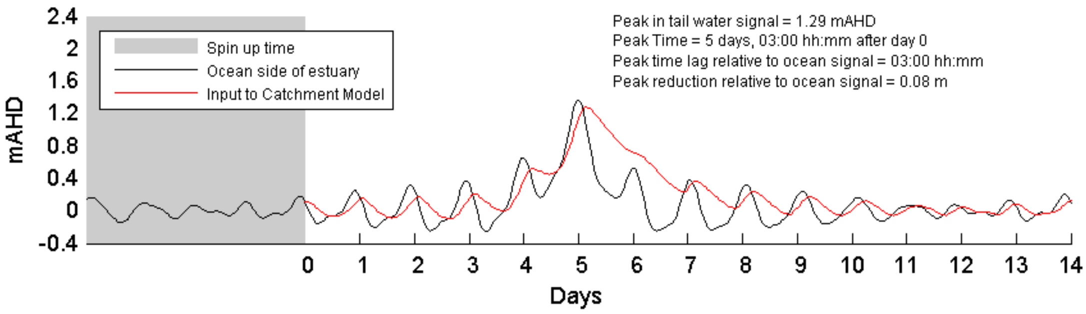
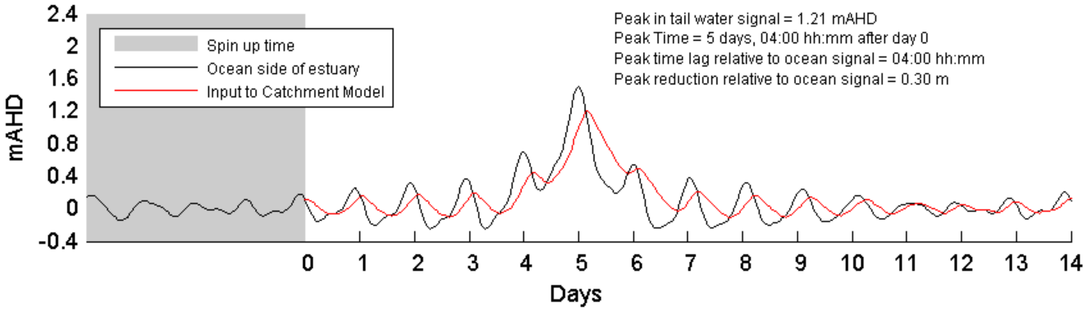
Simulation	Peak ocean water level (m AHD)	Peak Peel Inlet (m AHD)	Peak reduction (m)	Peak reduction (%)	Phase lag (hr)
D0	0.42	0.20	0.22	52	3
D1	1.10	0.83	0.27	25	4
D2	1.17	0.90	0.27	23	4
D3	1.25	0.97	0.28	22	4
D4	1.34	1.06	0.28	21	4
D5a	1.37	1.08	0.29	21	4
D5b	1.37	1.22	0.15	11	4
D5c	1.77	1.52	0.25	14	4
D5d	2.27	2.17	0.10	04	3
D5e	1.37	1.29	0.08	06	3
D6	1.51	1.21	0.30	20	4

Hydrodynamic modelling results were provided to the hydrology modelling team as a time-varying water level signal at the open boundary of the catchment model. Table 16 summarises the scenario modelling results graphically.

Table 16 Predicted estuarine water levels

Simulation	Results
D0	 Peak in tail water signal = 0.20 mAHD Peak Time = 4 days, 03:00 hh:mm after day 0 Peak time lag relative to ocean signal = 03:00 hh:mm Peak reduction relative to ocean signal = 0.22 m
D1	 Peak in tail water signal = 0.83 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.27 m
D2	 Peak in tail water signal = 0.90 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.27 m
D3	 Peak in tail water signal = 0.97 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.28 m

Simulation	Results
D4	 Peak in tail water signal = 1.06 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.28 m
D5a	 Peak in tail water signal = 1.08 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.29 m
D5b	 Peak in tail water signal = 1.22 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.15 m
D5c	 Peak in tail water signal = 1.52 mAHD Peak Time = 5 days, 04:00 hh:mm after day 0 Peak time lag relative to ocean signal = 04:00 hh:mm Peak reduction relative to ocean signal = 0.25 m

Simulation	Results
D5d	
D5e	
D6	

5.4 Discussion of results

The hydrodynamic modelling described above has been used to assess the sensitivity of the water level within the Peel Inlet / Harvey Estuary to a number of issues. As the peak water levels in the Peel Inlet / Harvey Estuary are affected by the complex interaction of a number of features, the modelled results are not expected to provide accurate results on expected impacts. However, the modelling results provide indicative information that can be used to assess the possible impact of the individual parameters.

5.4.1 Wind waves, wind set up and wave runup

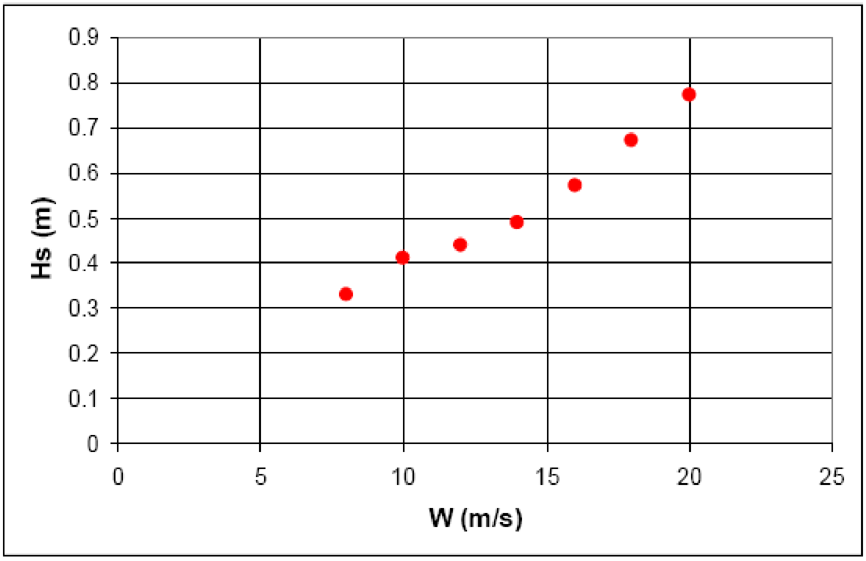
Estimates of the significant wind wave height in the Inlet were undertaken with the SMB method. The SMB method is based on a wave energy growth model calibrated empirically (Wallingford 1980). The method relates the dimensionless significant wave height (gH_s/W^2) and period ($gT_s/2\pi W$) to the dimensionless fetch (gF/W^2), duration ($g\,t_d/W$), and depth (gd/W^2).

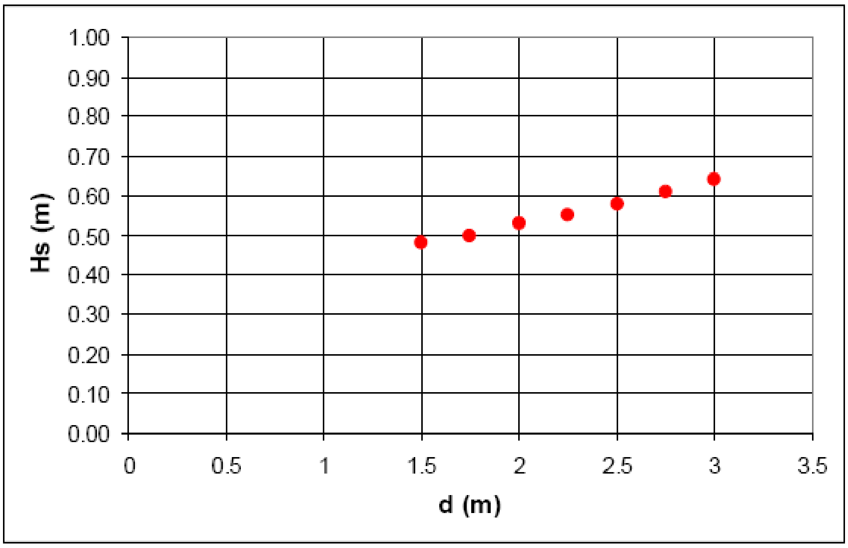
To estimate the significant wind wave height (H_s), representative values of wind speed, fetch and wind duration were selected. Since the strongest winds are predominantly from the west, and as the west-east axis coincides with the longest fetch in the Peel Inlet (~12 km), H_s was estimated for westerly winds at the eastern shoreline. The wind data from Mandurah Ocean Marina was discretized into northerly (315° to 45°), easterly (45° to 135°), southerly (135° to 225°) and westerly (225° to 315°) winds and an exceedance frequency analysis applied to the westerlies. The one percentile exceedance frequency (i.e. strong winds) during the windy period (May to October) corresponds to an hourly average intensity of approximately $W = 15$ m/s. This wind intensity was adopted for the calculations.

For a wind intensity $W = 15$ m/s, a fetch $F = 12$ km, a wind duration $t_d \geq 2$ hr, and a mean water depth $d = 2$ m, the SMB method yields $H_s = 0.55$ m at the eastern shore of the Inlet.

The sensitivity of the wave height estimate to wind speed and depth was investigated and is summarised in Table 17.

Table 17 Sensitivity of the significant wind wave height to wind speed and depth





By balancing the wind forcing and the hydrostatic pressure terms in the momentum equation along the west-east axis of the Inlet, the wind set up in the eastern shoreline for a westerly wind of 15 m/s, water depth of 2 m and fetch of 12 km was found to be approximately $\eta = 0.13$ m.

Comparison of the results for Scenarios 5a and 5b (refer to Table 14), which exclude and include strong winds, respectively suggest the wind set up is approximately 0.15 metre within the Peel Inlet / Harvey Estuary. The difference in modelled peak water level for a location on the downwind margin of the Peel Inlet is only slightly higher (< 0.1 m) than the modelled peak water level on the upwind margin. Modelled peak water levels are highest at the southern end of the Harvey Estuary but are not markedly higher (< 0.1m above the level within the centre of the Peel Inlet).

The modelled peak water levels in the Dawesville and Mandurah Entrance Channels are not affected by the application of wind within the hydrodynamic model. While the peak water levels within these channels were generally slightly higher than within the Peel Inlet / Harvey Estuary the difference was not considered significant enough to warrant specifying individual design peak water levels.

While it is unlikely that the peak storm surge and maximum wind will occur simultaneously this assessment has shown that the impact of wind setup on peak water levels is a relatively minor issue within the Peel Inlet / Harvey Estuary.

5.4.2 Astronomic tide and major river flooding

It is unlikely that the peak storm surge event will coincide with the highest astronomic tide. However, the impact on the expected flood level within the Peel Inlet / Harvey Estuary of such an occurrence requires consideration.

The astronomic tidal range within Peel Inlet / Harvey Estuary is relatively small (refer to Figure 21) and the difference between an average high tide (MHHW) and the highest astronomical tide (HAT) is approximately 0.25 metre. The hydrodynamic modelling suggests that while the actual level within the Estuary is lower than the outside ocean level, the difference in level within the Peel Inlet / Harvey Estuary is consistent with the difference in the outside ocean.

Similar to the above discussion, it is unlikely that the peaks of a major river flooding event and major storm surge event will coincide. However, the sensitivity to this occurrence was investigated using the hydrodynamic model. Comparison of scenarios 5a and 5e, suggest the peak flood levels in Peel Inlet / Harvey Estuary increase by approximately 0.2 metre in the event that major river flooding coincides with the peak storm surge.

5.4.3 Potential Impact of a changing climate

Two main potential impacts have been considered:

- Increased storminess
- Sea level rise

Potential impacts include increased storminess and sea level rise. Increased storminess could result in increased storm surges (maximum residual) but the likelihood and magnitude of any change is currently not well understood. However, as the estimated 500 year ARI storm surge (extrapolated from the work of DPI, 2004) is approximately 0.13 metre larger than the 100 year ARI estimate (refer to Table 13), increased storminess may not significantly increase the design water levels in the Peel Inlet / Harvey Estuary.

The allowance for projected rises in sea level over the next century are substantially larger than the sensitivity of water levels to the other possibilities (i.e. coinciding with higher tide than average, coinciding with major river flooding and increased storminess) discussed above. The recommended allowance for sea level rise over the next century of 0.9 metre results in a 75% increase in the peak water level in the Peel Inlet / Harvey Estuary. This increase is more significant than the combined indicative impact of wind setup, coinciding with the highest astronomical tide and major river flooding, and increased storminess.

5.4.4 Recommended design water levels in Peel Inlet / Harvey Estuary

The detailed analyses of Damara (2009) are considered to provide the most reliable design water level estimates for the Peel Inlet / Harvey Estuary. An allowance of 0.90 metre for sea level rise over the 100 year planning timeframe (2010 to 2110) is recommended to be consistent with the recent decision of the Western Australian Planning Commission (WAPC Minutes of Ordinary Meeting 173, Item 12.3, 25 May 2010). Hydrodynamic modelling suggests peak water levels within the Peel Inlet / Harvey Estuary do not vary significantly with location and the levels in the Dawesville and Mandurah Entrance Channels are not significantly different to those within the Peel Inlet / Harvey Estuary.

The recommended design flood estimates for the Peel Inlet / Harvey Estuary and Dawesville and Mandurah Entrance Channels are summarised in Table 18

Table 18 Recommended design peak water levels for the Peel Inlet / Harvey Estuary and Dawesville and Mandurah Entrance Channels

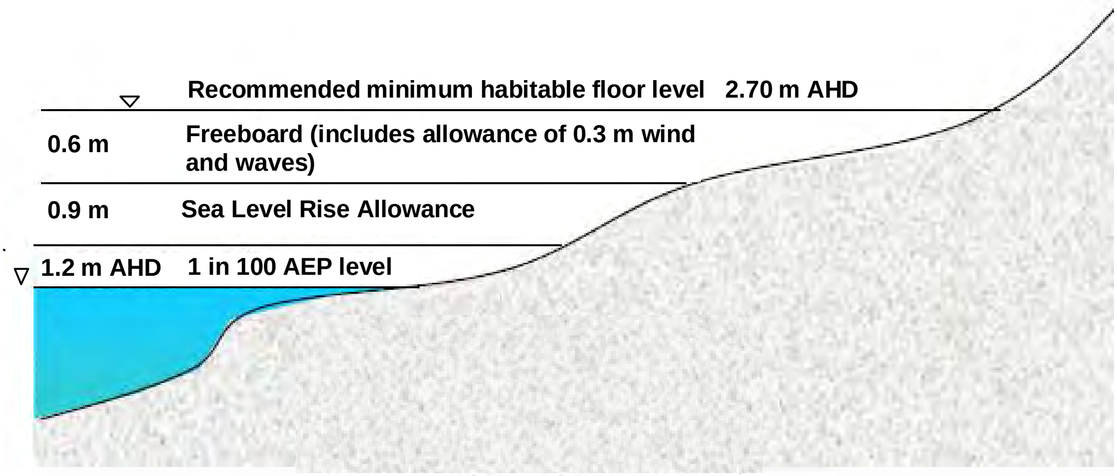
Design Event (*y*-year ARI)	Design water level (m AHD)	Design water level + 0.9 m for sea level rise (m AHD)
5	0.95	1.85
10	1.00	1.90
25	1.08	1.98
50	1.14*	2.04*
100	1.20*	2.10*
500	1.30*	2.20*

* the estimate is extrapolated from 12 years of observed water levels and is beyond the recommended limits for the available observation period (3 times the observation period).

A freeboard of 0.60 metre above the 100-year ARI peak water level is recommended for future development to ensure adequate protection against flooding during this event. This freeboard includes an allowance of 0.3 metre for wind setup and wave runup.

Components of the design inundation level are illustrated below.

Components of the design inundation level for the Peel Inlet / Harvey Estuary



6. Floodplain mapping

6.1 Introduction

The floodplain mapping component of the project involved developing a two-dimensional hydraulic model of the flood study area, using the model to simulate flood extent across the area, and extracting water level and discharge information. This information was used to inform the floodplain development strategy (Section 6.6) and provide input to the DWMP.

The model development process involved developing a hydraulic modelling framework that had as input runoff hydrographs from the Murray River, Hills Catchments and Serpentine River and rainfall-excess over the flood study area (described in Section 4) and estuarine water level (Section 5) as a boundary condition. Lidar data and resistance (land use) mapping formed the foundation of the hydraulic model. Observed flood levels for a number of historic events were used to calibrate the model and verify predictions. The calibrated model was then used to prepare floodplain mapping for a range of design events.

6.2 Model development

6.2.1 Model structure

A 20 m, rectangular grid was used for all simulations. Mike version 2009 software was used.

A time step of 1.5 seconds, eddy viscosity of 2 and flooding and drying depths of 2 and 4 mm respectively were used.

The hydraulic model was developed in two forms:

- A Mike 21 model for the calibration calibrations.
- A Mike Couple model for the design simulations.

The Mike Couple model was used to allow representation of the effect of culverts on flow, particularly across the New Perth Bunbury Highway. This is discussed in more detail in Section 6.2.7.

The model had a single boundary - the Peel inlet.

Inflow hydrographs and rainfall excess were connected as source inputs.

6.2.2 Topography

The 1 m Lidar data was clipped to the defined study area. A raw, or unprocessed, 20 m grid was interpolated in Mike from the 1 m Lidar grid.

This interpolated 20 m grid was then modified based on surveyed bathymetry as Lidar does not accurately measure the ground surface elevation beneath water. Lake and stream channel bathymetry was interpolated from the available and additional information (see Section 2.6). Channel shape was corrected using cross-sections of the Murray River surveyed by GHD in May 2009, and of the Murray, Serpentine and Dandalup Rivers, provided by Department of Water. Riffles, or raised sections of channel, in the upper Murray River were inserted based on observations of aerial imagery. Bathymetry of the Peel Inlet was incorporated from the storm surge study (GHD 2009 b), based on C-MAP data.

Bathymetry of Goegrup and Black Lakes was estimated based on the available survey information and topographic mapping.

The grid containing bathymetry was then modified to produce three versions;

- For design simulations – this included the New Perth Bunbury Highway, upgraded (i.e. raised) Greenlands Road, South Western Highway and the railway.
- For calibration to the 1996, 1982, 1964 and 1945 events – as for the design runs but excluding the New Perth Bunbury Highway and without the Greenlands Road upgrade.
- For the 1862 event – without the New Perth Bunbury Highway, Greenlands Road, South Western Highway or the railway.

The design topography was intended to represent a present day land surface and used for modelling all design events.

The calibration topography was developed for use with model runs for observed events from 1945 to 1996. It was primarily based on the Lidar survey. As the New Perth to Bunbury Highway was under construction at the time the Lidar survey was flown, this was removed from the calibration topography.

The 1862 topography had the New Perth to Bunbury Highway and the existing Greenlands Road removed. Other man-made features removed from the Lidar data for this topography were the railway, built in 1892, and the South Western Highway.

6.2.3 Resistance/roughness

A resistance map was developed based on land use. Minor modifications to resistance for some land use categories were made as part of the calibration process.

The land use map was developed from Landgate cadastre (2008) and DLI aerial imagery (2008) and considering prior land use mapping by DoW. Land was divided into seven land use/resistance categories, as shown in

Table 19, based on its zoning and the dominant land cover type. Each land use was then assigned a Manning's roughness coefficient, with reference to Chow (1959). The resistance used within the model is M, which is equal to the inverse of Manning's n.

Two versions of the resistance map were developed:

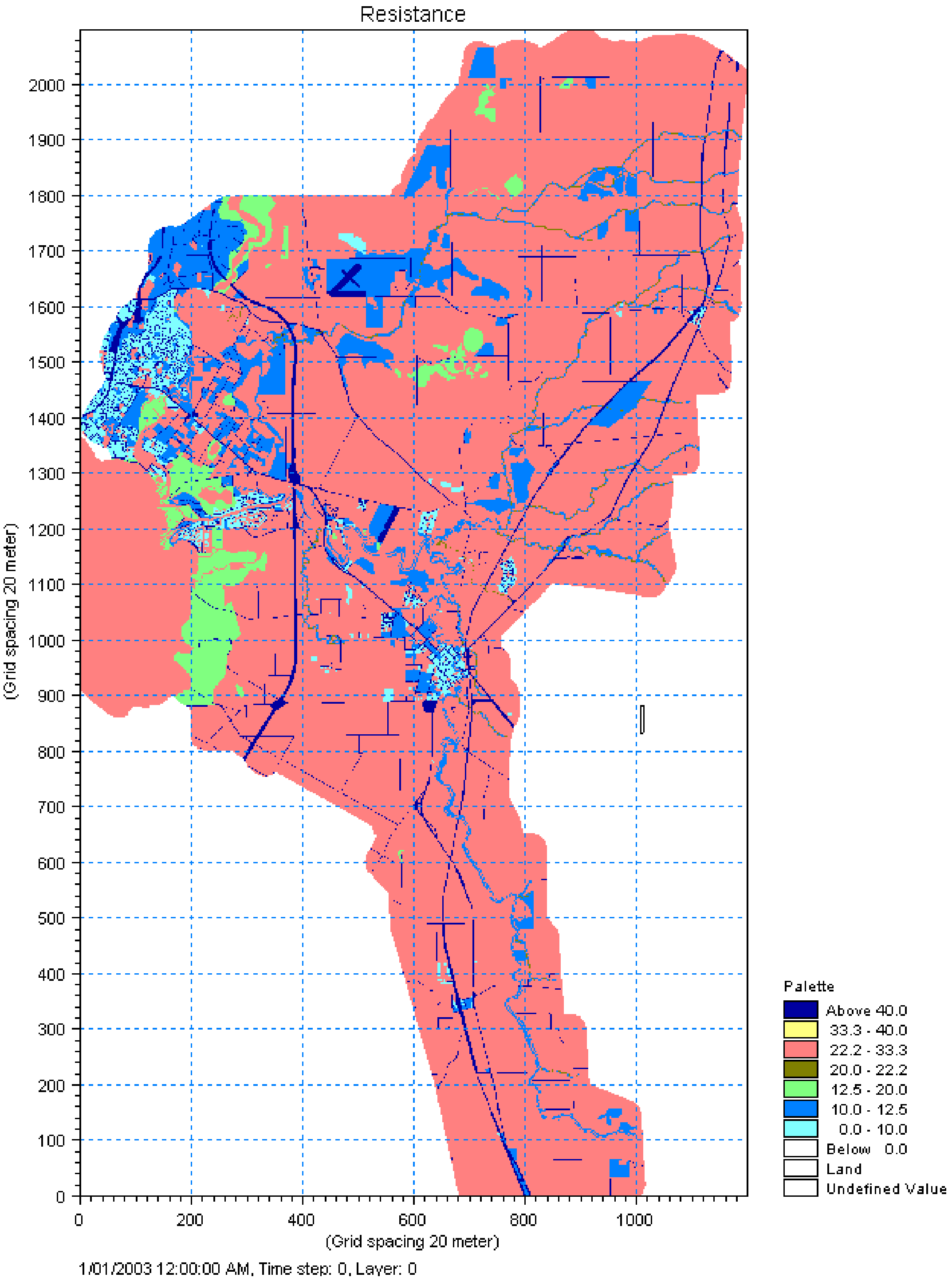
- Design and recent calibration events, which was based on current land use including extensive clearing for agriculture.
- For the 1862 event – with a reduced level of clearing.

The resistance map used in the design and recent calibration simulations is shown in Figure 28.

Table 19 Land use and surface roughness

Description	Manning's n	M	Comment
Road reserve, hardstand	0.020	50.0	Pilgrim (2001).
Urban	0.100	10.0	
Cleared	0.030	33.3	SKM serpentine used .05, Chow Floodplain D-2 a 1 short grass
Trees - fringing channels	0.050	20.0	SKM serpentine used .08, Chow floodplain D-2 c 3 light brush & trees summer, between min & normal
Wetland	0.050	20.0	Chow D-2 c 1 floodplains scattered brush heavy weeds
Wet channel/water	0.024	41.0	Chow major streams D-3 a regular
Dry channel	0.025	40.0	SKM serpentine used .025, Chow minor streams D-1 a 1 clean, straight

Figure 28 Resistance – design and recent calibration events



6.2.4 Initial water level

Initial catchment wetness, stream baseflow levels at the start of the event, and initial water levels in the Peel Inlet were set using an initial condition grid. For most simulations, ground above the estuary starting water level was set dry (i.e. initial water level equal to bathymetry) and lower areas set equal to the starting water level in the Peel Inlet.

6.2.5 Boundary conditions

Water level in the Peel Inlet formed the boundary condition for the model. Water levels were used from the storm surge and tidal study (see Section 5).

Observed tide level data were available for the 1996 event and this was input to the model as is.

The following AEP's tide traces were used for the calibration events:

- 1982 1 in 10 AEP.
- 1964 1 in 25 AEP.
- 1945 1 in 50 AEP.
- 1862 1 in 100 AEP.

Highest astronomical tide was used for all design event simulations.

6.2.6 Source inputs

Inflow hydrographs

Inflow hydrographs, derived in the hydrology study (see Section 4), were input as point sources into the model grid.

For the Serpentine River, hydrographs were derived from the SKM hydraulic model of the lower Serpentine. (SKM 2008). Data are available for an event in 1987 and for some of the required design events (1 in 25, 100 and 500 AEP). Hydrographs for the 1 in 5 and 50 AEP events were extrapolated from the 1 in 25 and 1 in 100 AEP hydrographs (see Section 4).

Complete hydrographs were not available for either the design events or the 1987 event but were used as is. It was considered that the duration of the available hydrographs were adequate to simulate peak flood levels through the GHD flood study area and for comparison with historical water level data.

For the 1987 event, no Murray or Nambeelup inflow data were calculated in the GHD hydrology study, so the 1987 simulation had only the Serpentine River hydrograph as input.

There are no SKM predictions for the other calibration events simulated in the GHD study, so no inflow was simulated for the Serpentine River for these events.

Rainfall

Rainfall excess was input as a distributed, time varying file (i.e. a dfs2). Rainfall excess was derived by applying the regional, proportional runoff coefficient to design rainfall, as described in Section 4.3.3. Parameters for the foothills were used.

Up to three stations in the flood study area were used for the calibration events with spatial distribution determined using a thiessien polygon approach. One station, derived for the centre of the flood study area, was used for the design simulations.

No rainfall was applied in the upper parts of the Murray River within the flood study area or outside the natural topographic divide for the Murray and Serpentine River systems.

6.2.7 Structures

An evaluation of afflux associated with important bridges on the Murray and Serpentine Rivers was undertaken to determine the level of detail that bridges and culverts need to be represented in the hydraulic model. The bridges were simulated with HEC-RAS and Mike 11 models and afflux (i.e. difference between upstream and downstream peak water levels) compared with prior studies. Afflux was generally small, less than 0.2 m. This is small compared with other inaccuracies in the model and the effect of terrain, so it was not considered necessary to represent bridges and culverts for the calibration simulations. Bridges and culverts were represented as lowered cells in the topography.

However it was considered necessary to represent culverts, in particular, along the New Perth Bunbury Highway as this crosses the lower part of the Murray and Serpentine River floodplains where there is extensive flow outside the main channels. The road has a large number of relatively small culverts which, if represented as lowered cells with a 20 m cell size, would overestimate the rate of conveyance across the road alignment. Accordingly these culverts were represented in the model. Three bridges on the New Perth Bunbury Highway and a culvert on Greenlands Road were also represented.

Other bridges on the Murray and Serpentine Rivers and Nambeelup Brook were also represented in the design simulation, for completeness.

Bridges and culverts were represented in the design runs in a Mike 11 model coupled to Mike 21 using Mike Couple.

6.3 Calibration and verification

6.3.1 Observed maximum water levels

The model was calibrated and partially verified by comparing predicted and observed peak water levels for the following historical events:

- 1996, 1982 and 1964 for points along the Murray River.
- 1987 for points along the Serpentine River.

The locations of the observed water levels are shown in Figure 5.

Observed versus predicted water levels for the calibration events are shown in Figure 29 and values are listed in Table 20. There is consistent correlation between observed and predicted water levels in the upper reaches of the Murray River for the 1996, 1982 and 1964 events. In the lower reaches some points appear to be affected by assumptions made regarding tail-water conditions, i.e. tides in the estuary. This effect can be seen in the 1982 data for observed levels less than 2 m. There are no observed data for the older calibration events, so a design tail-water with an AEP matching the Murray River inflow hydrograph was used. This assumption probably contributes to the difference between observed and predicted water levels. For the 1982 event a 1 in 10 AEP tail water was used. This tide trace has a peak of 0.9 m which is higher than about half the observed points in the lower reaches. The lowest observed water level is 0.11 m AHD, which if this point is accurate, suggests a peak tide closer to 0.1 m AHD, much lower than the 0.9 m used.

Observed water levels in the Serpentine River (1987 event only) were accurately reproduced when a tail water value of 0.5 m AHD was used.

Figure 29 Observed versus predicted floodwater levels for calibration events

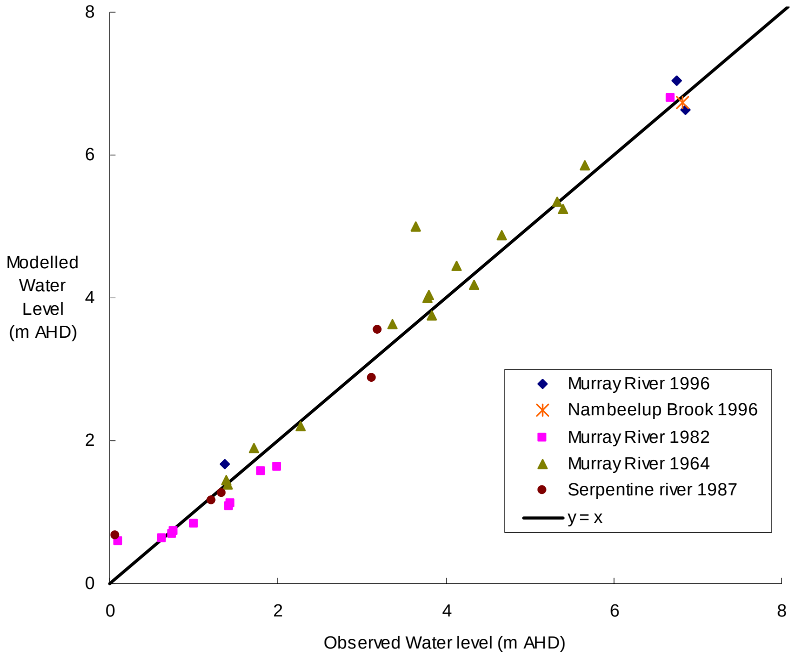


Table 20 Observed and predicted calibration water levels

Event	Location	Water level (m AHD)	
		Observed	Modelled
1996	Just upstream of SW Hway bridge at Pinjarra	6.86	6.64
1996	On Pateman St Yunderup	1.37	1.67
1996	614065 gauging station	6.75	7.04
1996	814063 Nambeelup	6.83	6.73
1982	Just upstream of SW Hway bridge at Pinjarra	6.68	6.80
1982	Murray River upstream Pinjarra Rd	1.99	1.63
1982	Murray River downstream Pinjarra Rd	1.80	1.58
1982	Murray River downstream now NPBH	1.44	1.12
1982	Murray River downstream now NPBH	1.42	1.09
1982	Pelican St	1.00	0.83
1982	Cooper St	0.77	0.73
1982	Strain St	0.74	0.70
1982	Rivergum St	0.63	0.64
1982	MR outlet S	0.11	0.60
1964	MR Paterson Rd S	5.66	5.86
1964	Ravenswood Bridge to Blythewood	5.39	5.25
1964	Ravenswood Bridge to Blythewood	5.32	5.35
1964	West Murray Area	4.66	4.87
1964	MR N Pinjarra	4.33	4.18
1964	West Murray Area	4.13	4.45
1964	West Murray Area	3.84	3.75
1964	West Murray Area	3.80	4.04
1964	West Murray Area	3.78	4.00
1964	West Murray Area	3.64	4.99
1964	West Murray Area	3.37	3.64
1964	Murray River downstream now NPBH	2.27	2.21
1964	West Murray Area	1.71	1.90
1964	West Murray Area	1.41	1.39
1964	Strain St	1.39	1.44
1987	Serpentine River Lakelands Rd	1.34	1.26
1987	Serpentine River Koolyunga St	1.21	1.17
1987	Serpentine River Wanda St	1.16	1.14
1987	Serpentine River Redcliffe St	1.09	1.13
1987	Serpentine River Pinjarra Rd ustr	0.98	0.96
1987	Serpentine River Pinjarra Rd dstr	0.78	0.95
1987	Serpentine River Bertram Rd	0.48	0.65

6.3.2 Comparison with the 1984 study results

The model agreed reasonably well with water levels predicted in the 1984 study (PWDWA 1984). The data, methods and flows used in the 1984 study are different to the GHD (2010) study, so the results are not directly comparable. Nevertheless, a comparison helps better understand the flood characteristics of the area.

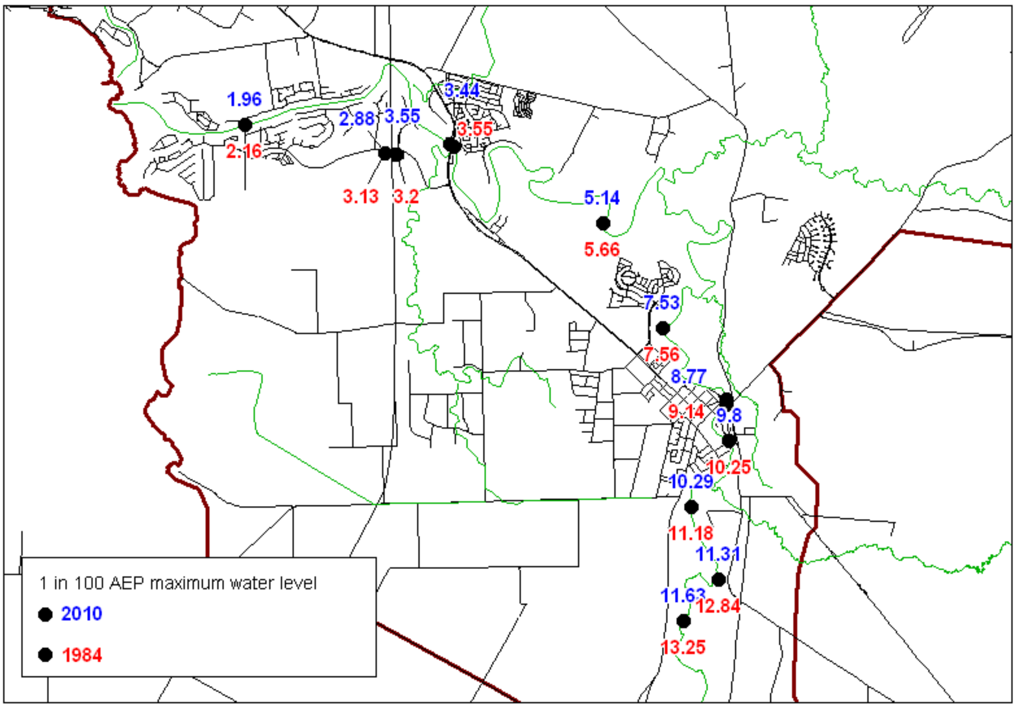
Generally, the GHD model produced slightly lower water levels to the 1984 study. GHD's modelled water levels in the upper reaches of the Murray, above Blythwood, tend to be lower than the 1984 study and the difference increases with distance upstream. This is largely because of lower peak discharge in the GHD study. Routing of the Baden Powell hydrograph to Pinjarra and the contribution of local catchments to the flood peak are handled differently in the two studies, affecting predicted peak discharge through the flood study area. The more accurate ground surface representation associated with the two-dimensional model may have also affected predicted water levels.

In some areas the 1984 water levels vary sharply from the GHD predictions. The 1984 water levels in the Blythewood area are substantially higher than the GHD levels. The 1984 study predicted an afflux of 0.75 m at the Pinjarra Road Bridge that could not be reproduced within the revised hydraulic modelling. Afflux for the current bridge arrangement is predicted to be 0.3 metre.

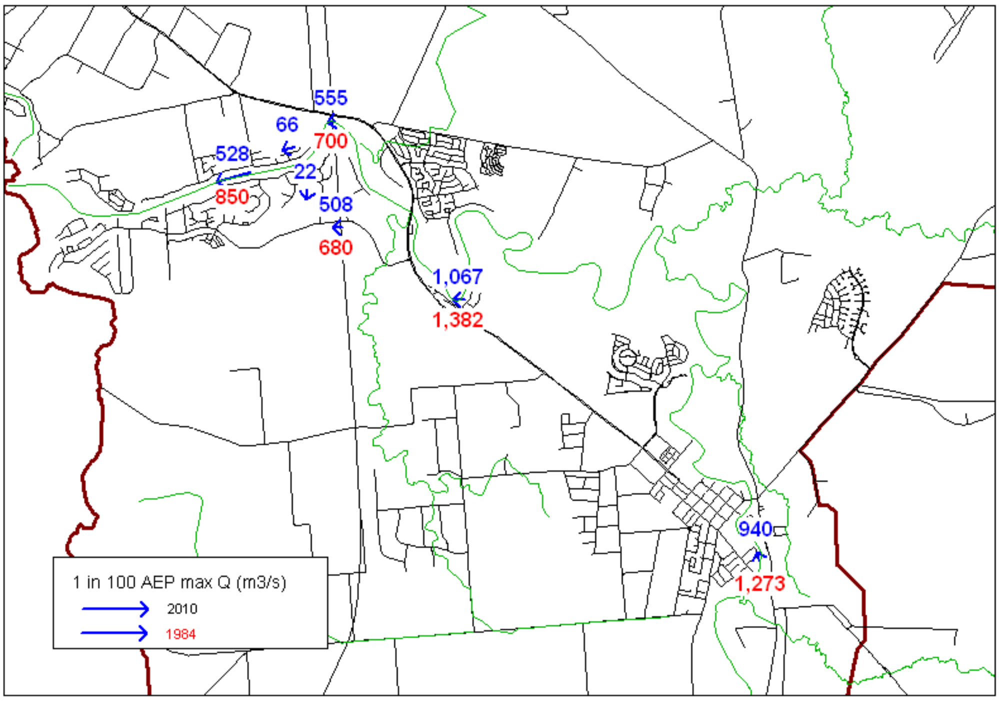
Predicted afflux at the South West Highway Bridge at Pinjarra is similar to the 1984 results (0.2 m for the 1984 study, 0.3 m for the GHD study).

Figure 30 Comparison with the 1984 PWDWA study

Peak water level



Peak discharge



6.3.3 Comparison with 1996 event data

Observed and modelled hydrographs for streamflow gauging stations on the Murray River at Pinjarra (DoW Station 614065) and Nambeelup Brook Kielman (Station 614063) are shown in Figure 31. Modelled depth for the 1996 event is plotted against discharge in Figure 32 and compared with the rating curves provided by DoW for these two stations.

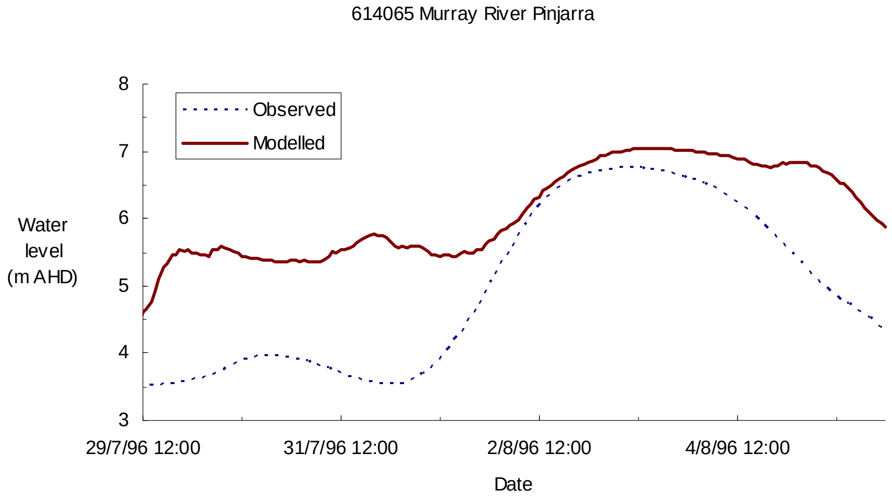
The Murray River data indicate some variation in the depth-discharge relationship between the model and the DoW rating, particularly for lower discharges. Gaugings by DoW of moderate flows, between 50 m^3/s and 150 m^3/s, in August 2008 and 2009 suggest the rating curve is incorrect in this range. Lower flows will be controlled by a low level weir which has not been incorporated within the hydraulic model. Despite these problems with the stage discharge relationships at the site, the observed peak discharge, peak level and hydrograph shape is reasonably well reproduced.

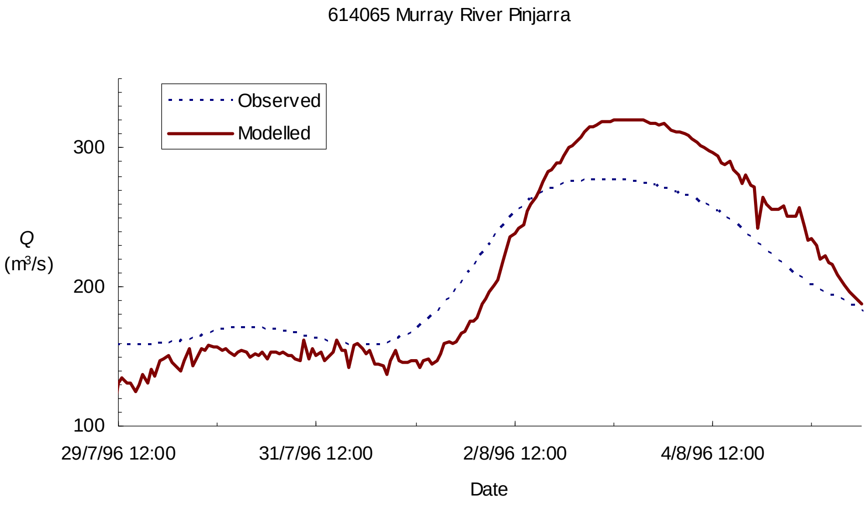
The Nambeelup Brook modelled data indicate a tendency toward an elevated recession hydrograph. This could be due to loss of channel resolution with the modelled grid size, varying effective depth-resistance and effects of using a coarse rainfall time step and inadequately represented seepage loss. Peak discharge and level are reasonable well predicted, though.

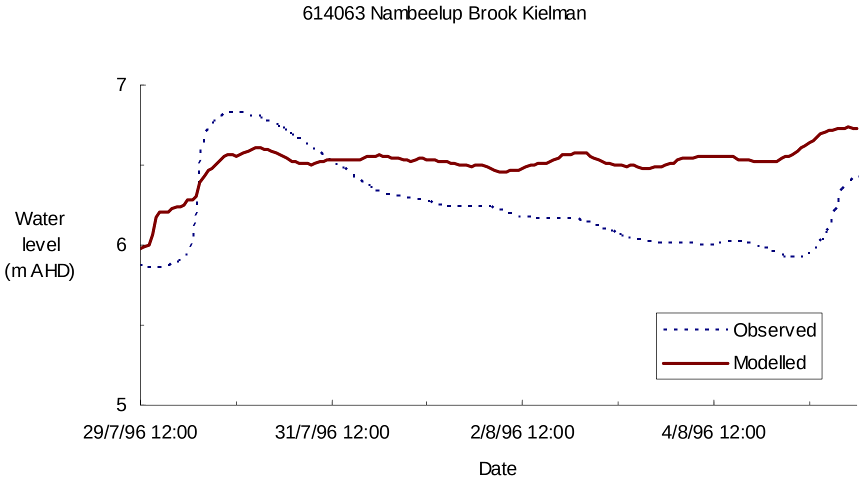
The depth-discharge comparison for the Murray River shows reasonable agreement between the model and the DoW rating information for higher levels. Note that the DoW rating information does not extend as high as the level data.

There is some discrepancy between the Nambeelup depth-discharge data and the DoW rating, probably as the weir at the site is not represented in the model.

Figure 31 Observed and predicted hydrographs, 1996 event




614065 Murray River Pinjarra


614063 Nambeelup Brook Kielman

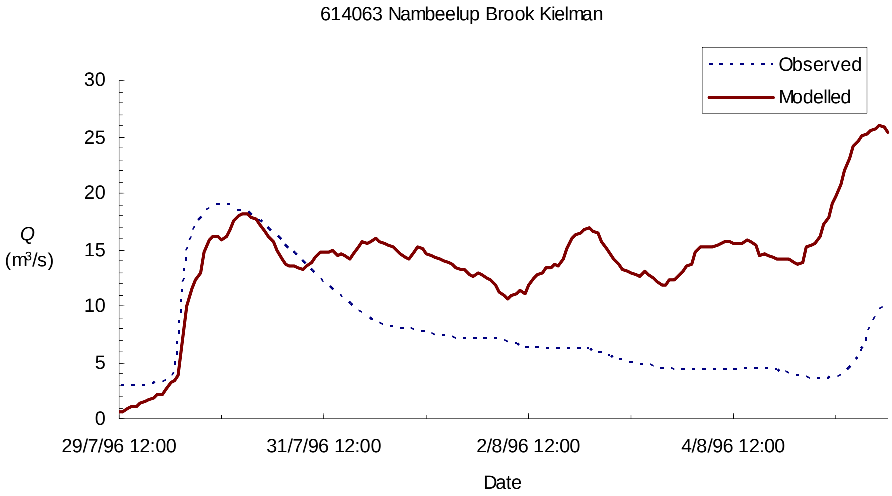
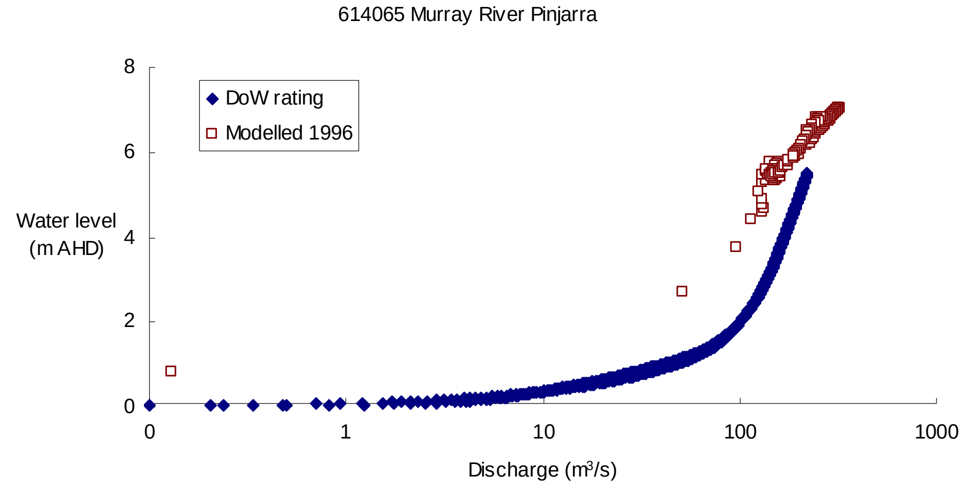
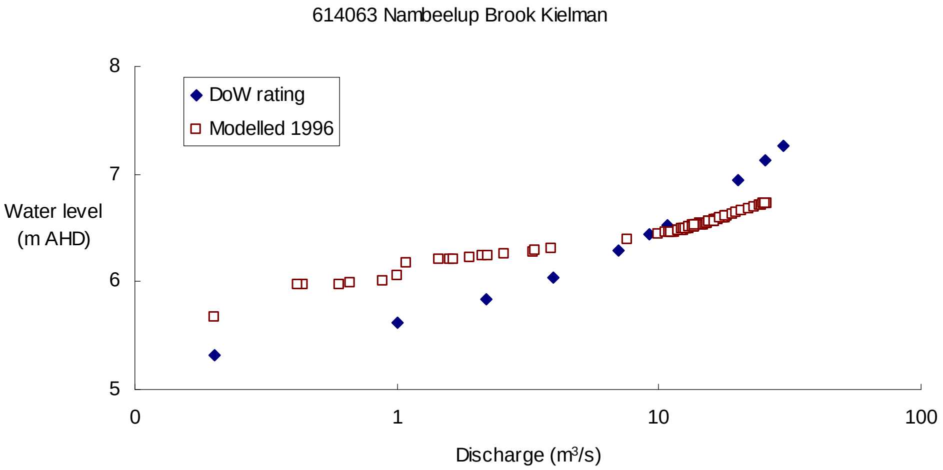

614063 Nambeelup Brook Kielman

Figure 32 Depth-discharge relationships

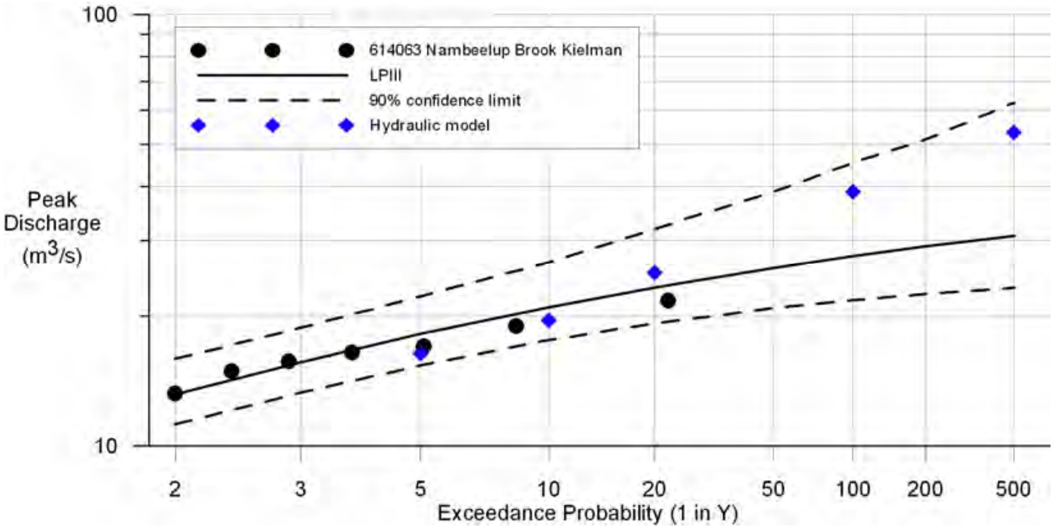




6.3.4 Comparison with flood frequency data

There is only one gauging station within the flood study area with a sufficiently long record (13 years) to develop a flood frequency curve, Station 614063. A comparison of the observed flood frequency curve and the equivalent modelled peak discharges are shown in Figure 33. The model reproduces the flood frequency curve for lower AEP accurately but tends to over predict for higher AEP's. This could be because the observed record is short and does not include any years with particularly large events, resulting in an underestimation of the peak discharge estimates from the flood frequency analysis at higher AEP's.

Figure 33 Observed and predicted flood frequency, Nambeelup Brook Kielman



6.4 Floodplain mapping

6.4.1 Methodology

The calibrated MIKE 21 hydraulic model described in Section 6.3 with a Mike 11 couple to represent bridges and culverts was used to prepare floodplain mapping for the design events. The model was run of 1 in 5, 10, 25, 100 and 500 AEP events. Preliminary revised floodways were delineated based on the current floodway delineation, observation of flow characteristics and the afflux associated with development of the flood fringe areas.

The design ground surface and resistance maps were used, as discussed in Section 6.2, and an initial condition accounting for starting water level in the Peel Inlet was used.

Design inflow hydrographs to the two-dimensional model domain for 1 in 5, 10, 25, 100 and 500 AEP events were taken from the hydrology study (see Section 4) and added to the hydraulic model as point inputs. Rainfall excess on the model domain was applied as a source input (see Section 4.5). Tide data, for the boundary condition, were taken from the GHD tide and storm surge study (Section 5).

6.4.2 Flood depth and extent

Streams and floodplains

Floodplain mapping for the 1 in 2, 10, 25, 100 and 500 AEP design event are shown in Figures 34 to 38. The floodplain mapping indicates the extent of predicted flooding along the main streamlines and the distributed nature of ponding and streamflow through the model domain.

Historical flood paths and extents observed in past events are reproduced.

The flow breakout to the west over South West Highway and Greenlands Road just south of Pinjarra at Blythewood (as identified in the PWDWA 1984 study) was not found to be active during the revised 1 in 100 AEP mapping. However, this breakout was found to be active during the 1 in 500 AEP event.

The substantial breakouts from the Murray River at Pinjarra via Tate Gully, at the confluence with the Dandalup River and at Yunderup identified in PWDWA (1984) were found to be active during the 1 in 100 AEP event. An additional shallow-flow breakout from Marrinup Brook connecting with Oakley Brook is evident during the 1 in 500 AEP event.

A breakout from the Murray River inundates the lower sections of Buchanan's Drain. Upstream of the influence of the Murray River the flood is mainly contained within the main channel of the drain.

Flooding in the Serpentine River is largely contained within the river's floodplain and major lakes, including Black Lake. Flow from the Murray River floodplain also connects with Black Lake. The floodplain in the Furnissdale area, near the Serpentine River outlet to the Peel inlet, is affected by flooding from and the Murray River and from a major storm surge in Peel Inlet.

Flooding in Nambeelup Brook is generally contained within the main streamline as a result of the significant floodplain storage in the upper reaches.

Roads

In a 1 in 100 AEP event, Pinjarra Road is inundated on the southern side of the Murray River near Ravenswood and Old Mandurah Road in places along its length.

South Yunderup Road immediately east of the Forrest Highway underpass is inundated to some depth. South Yunderup Road is inundated at this location during events as frequent as 1 in 10 AEP. Other local roads will be affected by inundation from a 1 in 100 AEP event to varying degrees.

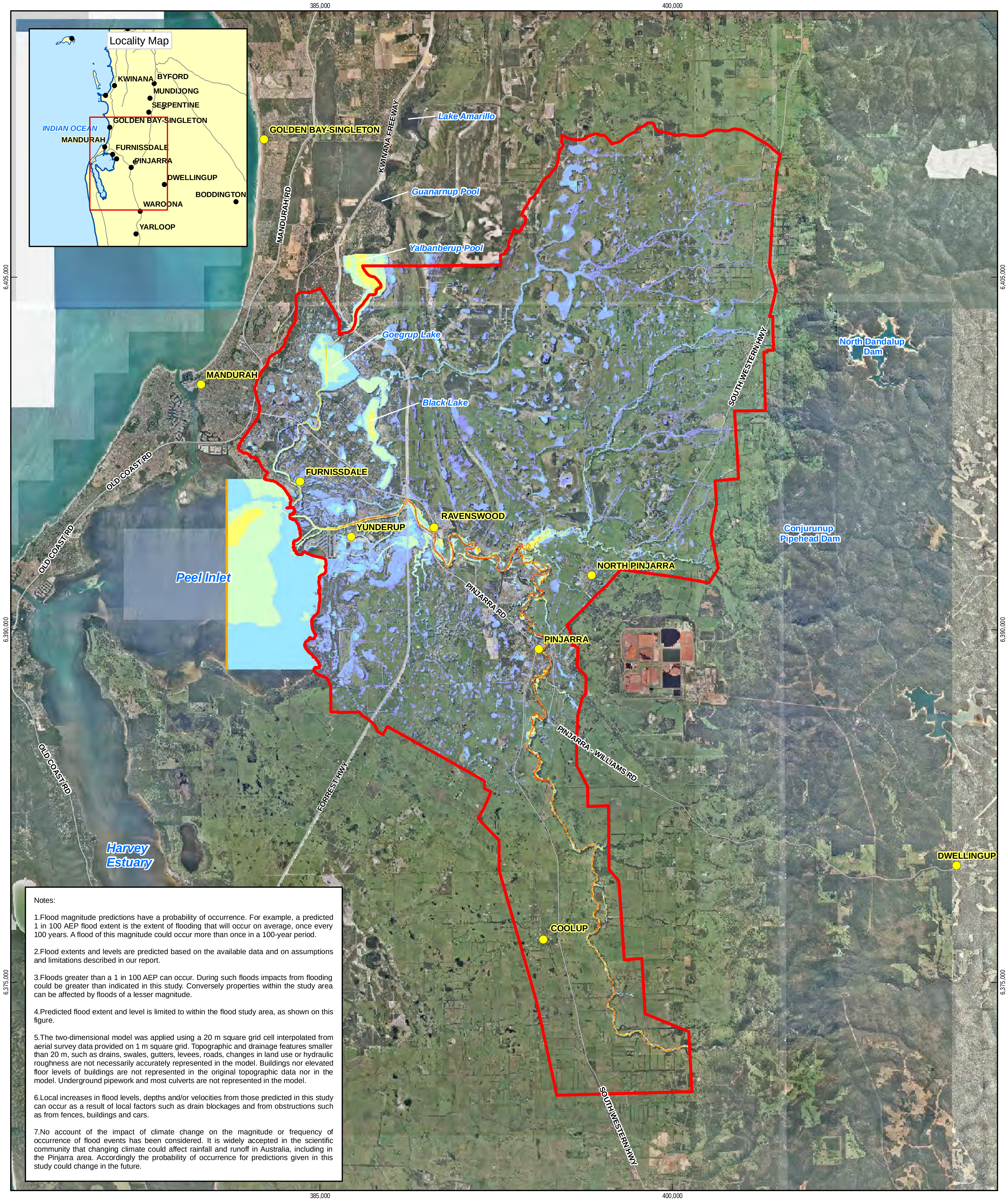
The Forrest Highway and Kwinana Freeway are flood free in a 1 in 100 AEP event.

Floodwaters in a 1 in 100 AEP event overtop with shallow flows the South West Highway north of Pinjarra.

Towns

In Pinjarra, some ponding occurs on the north western edge near the Murray River in larger events, as a result of backwater from the Murray River and local stormwater. Most of the town itself is not affected by floodwaters from the Murray River in a 1 in 100 AEP event, but in a 1 in 500 AEP event floodwaters do pass through the town from the south east to north west.

Most of the more densely urbanised areas in Ravenswood, Yunderup and Furnisdale are not directly flooded by floodwater from the Murray or Serpentine Rivers. However, some areas in Yunderup on the northern bank of the Murray River have flooding indicated up to 1 m deep.



Notes:

1.Flood magnitude predictions have a probability of occurrence. For example, a predicted 1 in 100 AEP flood extent is the extent of flooding that will occur on average, once every 100 years. A flood of this magnitude could occur more than once in a 100-year period.

2.Flood extents and levels are predicted based on the available data and on assumptions and limitations described in our report.

3.Floods greater than a 1 in 100 AEP can occur. During such floods impacts from flooding could be greater than indicated in this study. Conversely properties within the study area can be affected by floods of a lesser magnitude.

4.Predicted flood extent and level is limited to within the flood study area, as shown on this figure.

5.The two-dimensional model was applied using a 20 m square grid cell interpolated from aerial survey data provided on 1 m square grid. Topographic and drainage features smaller than 20 m, such as drains, swales, gutters, levees, roads, changes in land use or hydraulic roughness are not necessarily accurately represented in the model. Buildings nor elevated floor levels of buildings are not represented in the original topographic data nor in the model. Underground pipework and most culverts are not represented in the model.

6.Local increases in flood levels, depths and/or velocities from those predicted in this study can occur as a result of local factors such as drain blockages and from obstructions such as from fences, buildings and cars.

7.No account of the impact of climate change on the magnitude or frequency of occurrence of flood events has been considered. It is widely accepted in the scientific community that changing climate could affect rainfall and runoff in Australia, including in the Pinjarra area. Accordingly the probability of occurrence for predictions given in this study could change in the future.

LEGEND

● Locality
▭ Flood Study Area
— Roads

Flood Depth (m)

▨	0.05 - 0.15
▨	0.15 - 0.5
▨	0.5 - 1
▨	1 - 2
▨	2 - 3
▨	3 - 4
▨	>5

1:150,000 (at A3)
0 0.75 1.5 3 4.5 6 7.5
Kilometers
Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Murray Drainage & Water Management
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Predicted Maximum Flood Depth
1 in 5 AEP

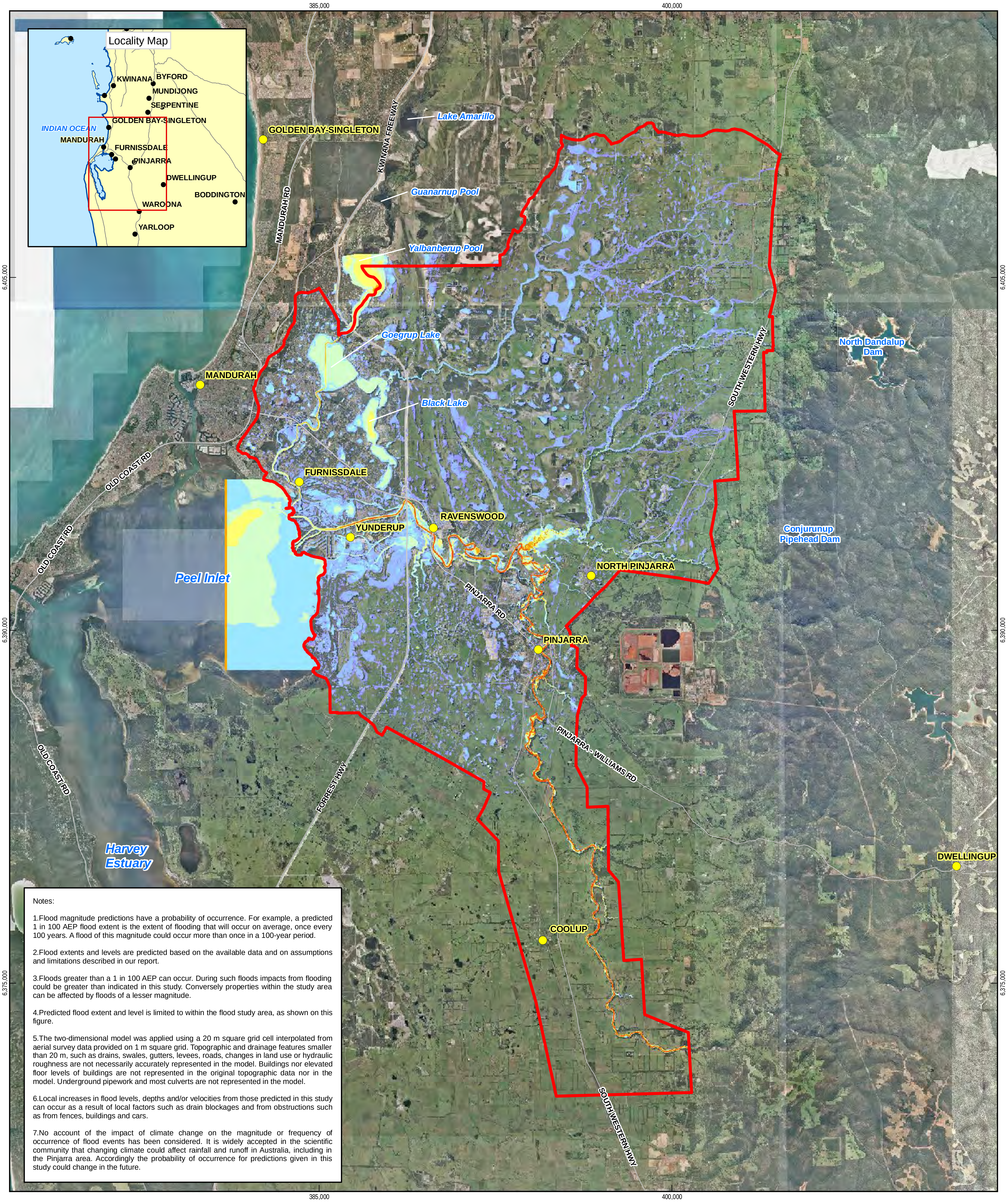
Job Number	61-2393701
Revision	0
Date	01 JUN 2010

Figure 34

G:\61\2393706\GIS\mxds\612393701-G006.mxd

GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: GA: Populated Places - 2006; DOW: Swan Coastal Plain Imagery - 2008; MRWA: Road Centreline - 20090501; GHD: Flood Study Area - 20090501, Predicted Maximum Flood Depth - 20100202. Created by: xntan, kdiralu



Notes:

1.Flood magnitude predictions have a probability of occurrence. For example, a predicted 1 in 100 AEP flood extent is the extent of flooding that will occur on average, once every 100 years. A flood of this magnitude could occur more than once in a 100-year period.

2.Flood extents and levels are predicted based on the available data and on assumptions and limitations described in our report.

3.Floods greater than a 1 in 100 AEP can occur. During such floods impacts from flooding could be greater than indicated in this study. Conversely properties within the study area can be affected by floods of a lesser magnitude.

4.Predicted flood extent and level is limited to within the flood study area, as shown on this figure.

5.The two-dimensional model was applied using a 20 m square grid cell interpolated from aerial survey data provided on 1 m square grid. Topographic and drainage features smaller than 20 m, such as drains, swales, gutters, levees, roads, changes in land use or hydraulic roughness are not necessarily accurately represented in the model. Buildings nor elevated floor levels of buildings are not represented in the original topographic data nor in the model. Underground pipework and most culverts are not represented in the model.

6.Local increases in flood levels, depths and/or velocities from those predicted in this study can occur as a result of local factors such as drain blockages and from obstructions such as from fences, buildings and cars.

7.No account of the impact of climate change on the magnitude or frequency of occurrence of flood events has been considered. It is widely accepted in the scientific community that changing climate could affect rainfall and runoff in Australia, including in the Pinjarra area. Accordingly the probability of occurrence for predictions given in this study could change in the future.

LEGEND

● Locality
▭ Flood Study Area
— Roads

Flood Depth (m)
- 🟪 0.05 - 0.15
- 🟦 0.15 - 0.5
- 🟦 0.5 - 1
- 🟩 1 - 2
- 🟨 2 - 3
- 🟧 3 - 4
- 🟥 >5

1:150,000 (at A3)
0 0.75 1.5 3 4.5 6 7.5
Kilometers
Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Department of Water
Murray Drainage & Water Management
and Associated Studies

Job Number	61-2393701
Revision	0
Date	01 JUN 2010

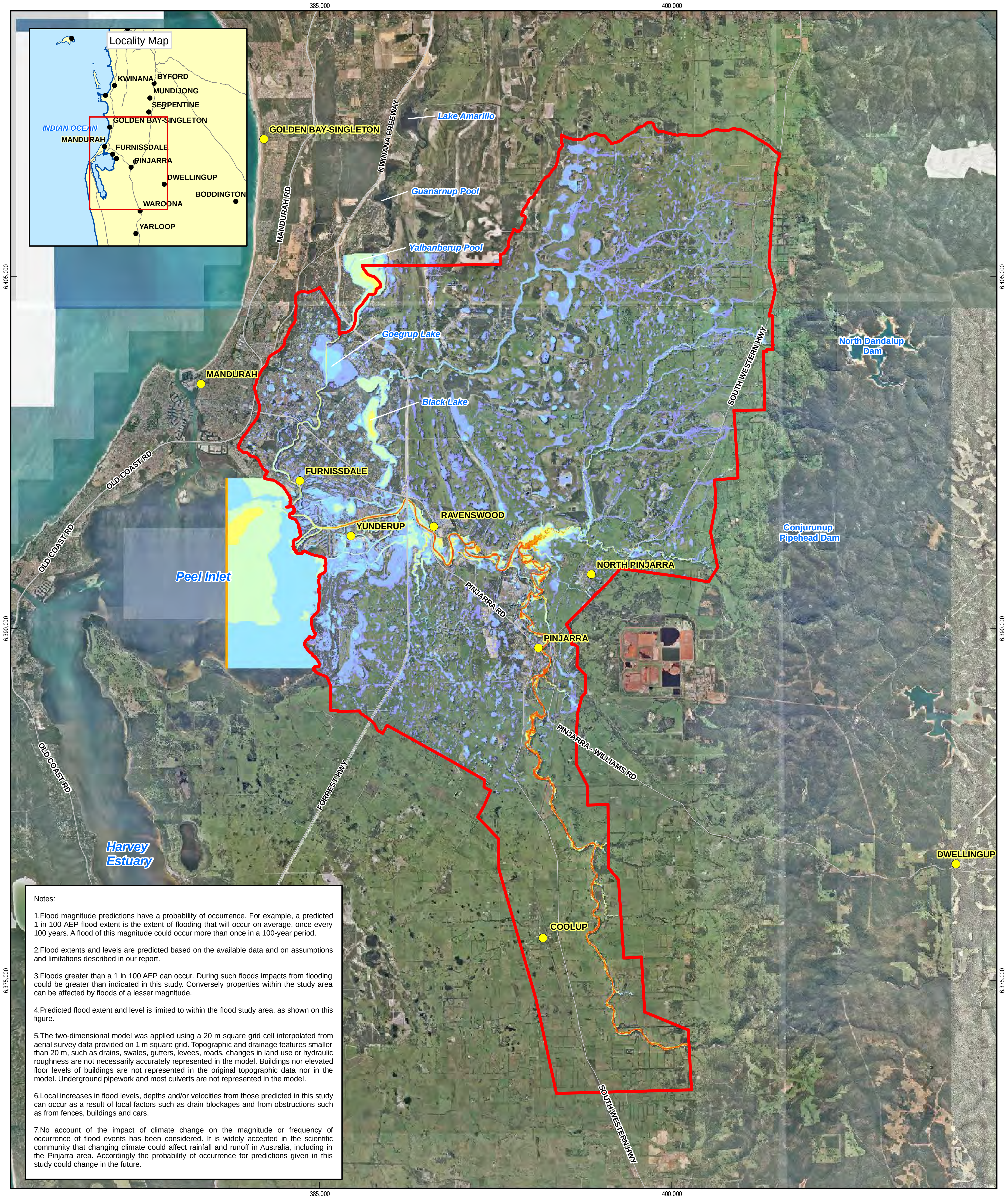
Predicted Maximum Flood Depth
1 in 10 AEP Flood Extent

Figure 35

G:\61\2393706\GIS\mxds\612393701-G007.mxd

GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: GA: Populated Places - 2006; DOW: Swan Coastal Plain Imagery - 2008; MRWA: Road Centreline - 20090501; GHD: Flood Study Area - 20090501, Predicted Maximum Flood Depth - 20100202. Created by: xntan, kdiralu



Notes:

1.Flood magnitude predictions have a probability of occurrence. For example, a predicted 1 in 100 AEP flood extent is the extent of flooding that will occur on average, once every 100 years. A flood of this magnitude could occur more than once in a 100-year period.

2.Flood extents and levels are predicted based on the available data and on assumptions and limitations described in our report.

3.Floods greater than a 1 in 100 AEP can occur. During such floods impacts from flooding could be greater than indicated in this study. Conversely properties within the study area can be affected by floods of a lesser magnitude.

4.Predicted flood extent and level is limited to within the flood study area, as shown on this figure.

5.The two-dimensional model was applied using a 20 m square grid cell interpolated from aerial survey data provided on 1 m square grid. Topographic and drainage features smaller than 20 m, such as drains, swales, gutters, levees, roads, changes in land use or hydraulic roughness are not necessarily accurately represented in the model. Buildings nor elevated floor levels of buildings are not represented in the original topographic data nor in the model. Underground pipework and most culverts are not represented in the model.

6.Local increases in flood levels, depths and/or velocities from those predicted in this study can occur as a result of local factors such as drain blockages and from obstructions such as fences, buildings and cars.

7.No account of the impact of climate change on the magnitude or frequency of occurrence of flood events has been considered. It is widely accepted in the scientific community that changing climate could affect rainfall and runoff in Australia, including in the Pinjarra area. Accordingly the probability of occurrence for predictions given in this study could change in the future.

LEGEND

● Locality

▭ Flood Study Area

── Roads

Flood Depth (m)

▨ 0.05 - 0.15	▨ 1 - 2
▨ 0.15 - 0.5	▨ 2 - 3
▨ 0.5 - 1	▨ 3 - 4
	▨ >5

1:150,000 (at A3)

0 0.75 1.5 3 4.5 6 7.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Department of Water
Murray Drainage & Water Management
and Associated Studies

Job Number	61-2393701
Revision	0
Date	01 JUN 2010

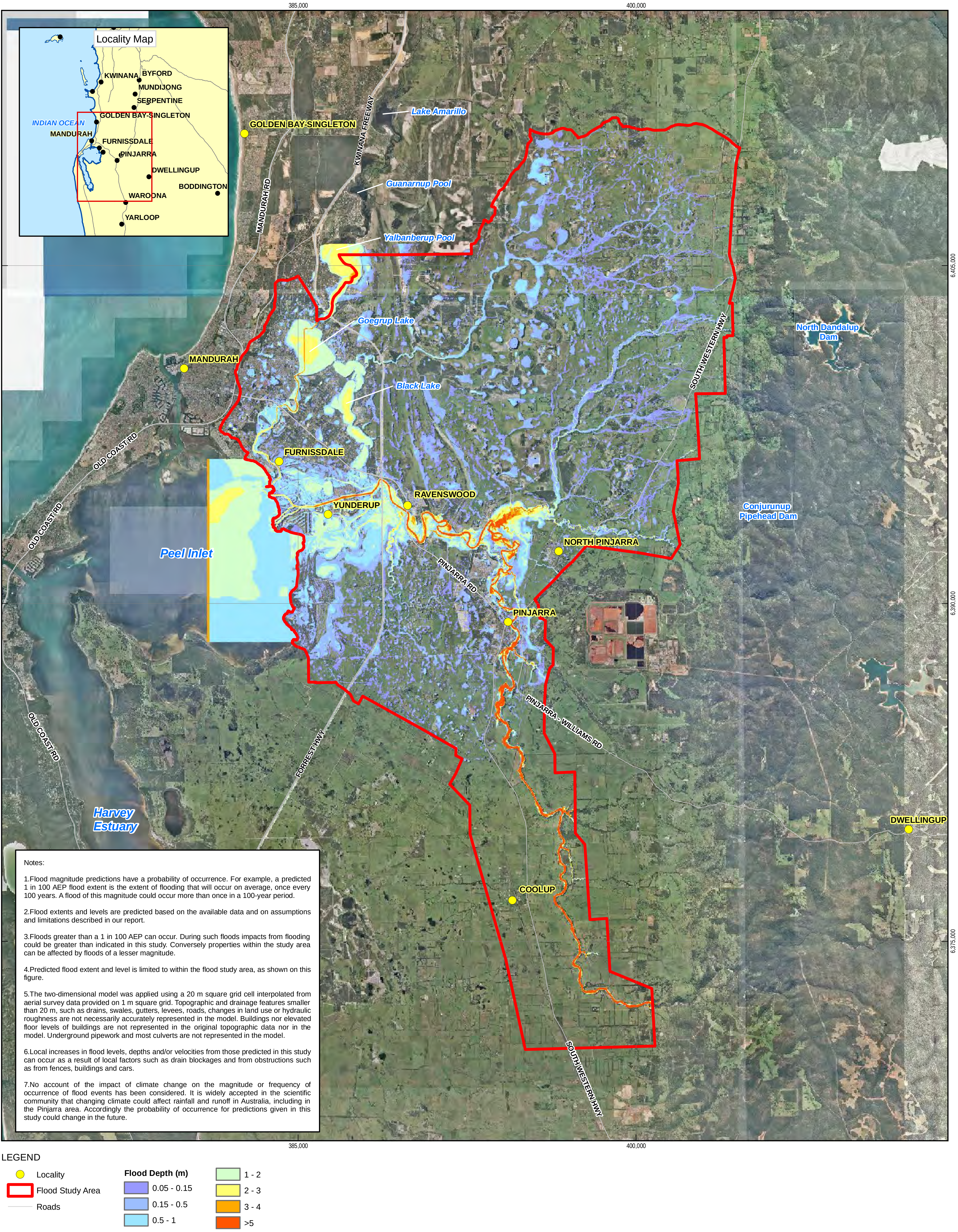
Predicted Maximum Flood Depth
1 in 25 AEP Flood Extent

Figure 36

G:\61\2393706\GIS\mxds\612393701-G008.mxd

GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: GA: Populated Places - 2006; DOW: Swan Coastal Plain Imagery - 2008; MRWA: Road Centreline - 20090501; GHD: Flood Study Area - 20090501, Predicted Maximum Flood Depth - 20100202. Created by: xntan, kdiralu



Notes:

1.Flood magnitude predictions have a probability of occurrence. For example, a predicted 1 in 100 AEP flood extent is the extent of flooding that will occur on average, once every 100 years. A flood of this magnitude could occur more than once in a 100-year period.

2.Flood extents and levels are predicted based on the available data and on assumptions and limitations described in our report.

3.Floods greater than a 1 in 100 AEP can occur. During such floods impacts from flooding could be greater than indicated in this study. Conversely properties within the study area can be affected by floods of a lesser magnitude.

4.Predicted flood extent and level is limited to within the flood study area, as shown on this figure.

5.The two-dimensional model was applied using a 20 m square grid cell interpolated from aerial survey data provided on 1 m square grid. Topographic and drainage features smaller than 20 m, such as drains, swales, gutters, levees, roads, changes in land use or hydraulic roughness are not necessarily accurately represented in the model. Buildings nor elevated floor levels of buildings are not represented in the original topographic data nor in the model. Underground pipework and most culverts are not represented in the model.

6.Local increases in flood levels, depths and/or velocities from those predicted in this study can occur as a result of local factors such as drain blockages and from obstructions such as from fences, buildings and cars.

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1:150,000 (at A3)
0 0.75 1.5 3 4.5 6 7.5
Kilometers
Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Murray Drainage & Water Management
and Associated Studies

Job Number	61-2393701
Revision	0
Date	01 JUN 2010

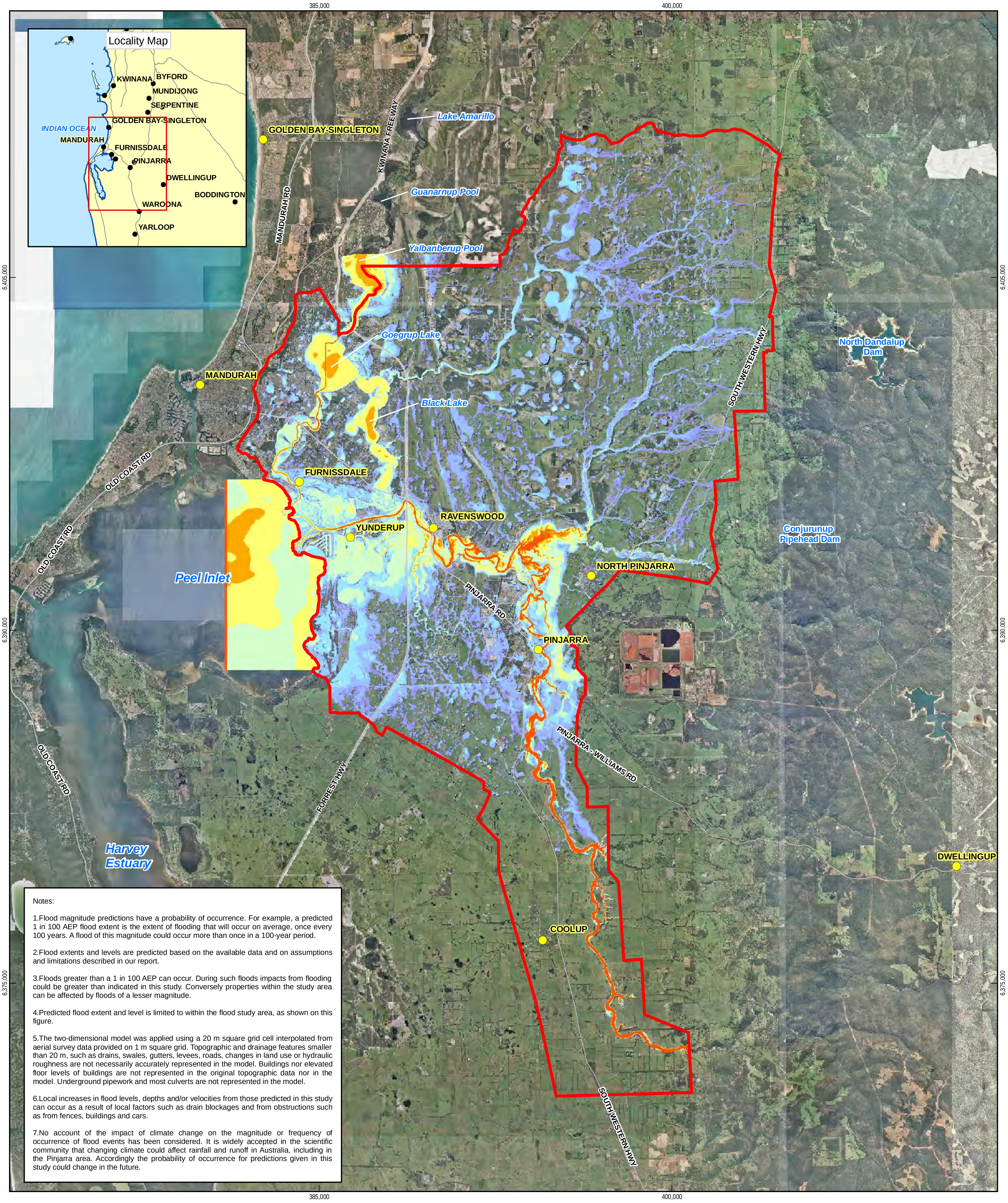
Predicted Maximum Flood Depth
1 in 100 AEP

Figure 37

G:\61\2393706\GIS\mxds\612393701-G005_Rev0.mxd

GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: GA: Populated Places - 2006; DOW: Swan Coastal Plain Imagery - 2008; MRWA: Road Centreline - 20090501; GHD: Flood Study Area - 20090501, Predicted Maximum Flood Depth - 20100202. Created by: xntan, kdiralu



Notes:

1.Flood magnitude predictions have a probability of occurrence. For example, a predicted 1 in 100 AEP flood extent is the extent of flooding that will occur on average, once every 100 years. A flood of this magnitude could occur more than once in a 100-year period.

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LEGEND

● Locality
▭ Flood Study Area
— Roads

Flood Depth (m)

🟪	0.05 - 0.15
🟦	0.15 - 0.5
🟦	0.5 - 1
🟩	1 - 2
🟨	2 - 3
🟧	3 - 4
🟥	>5

1:150,000 (at A3)

0 0.75 1.5 3 4.5 6 7.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Predicted Maximum Flood Depth
1 in 500 AEP Flood Extent

Job Number	61-2393701
Revision	0
Date	01 JUN 2010

Figure 38

G:\61\2393706\GIS\mxds\612393701-G009.mxd

GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

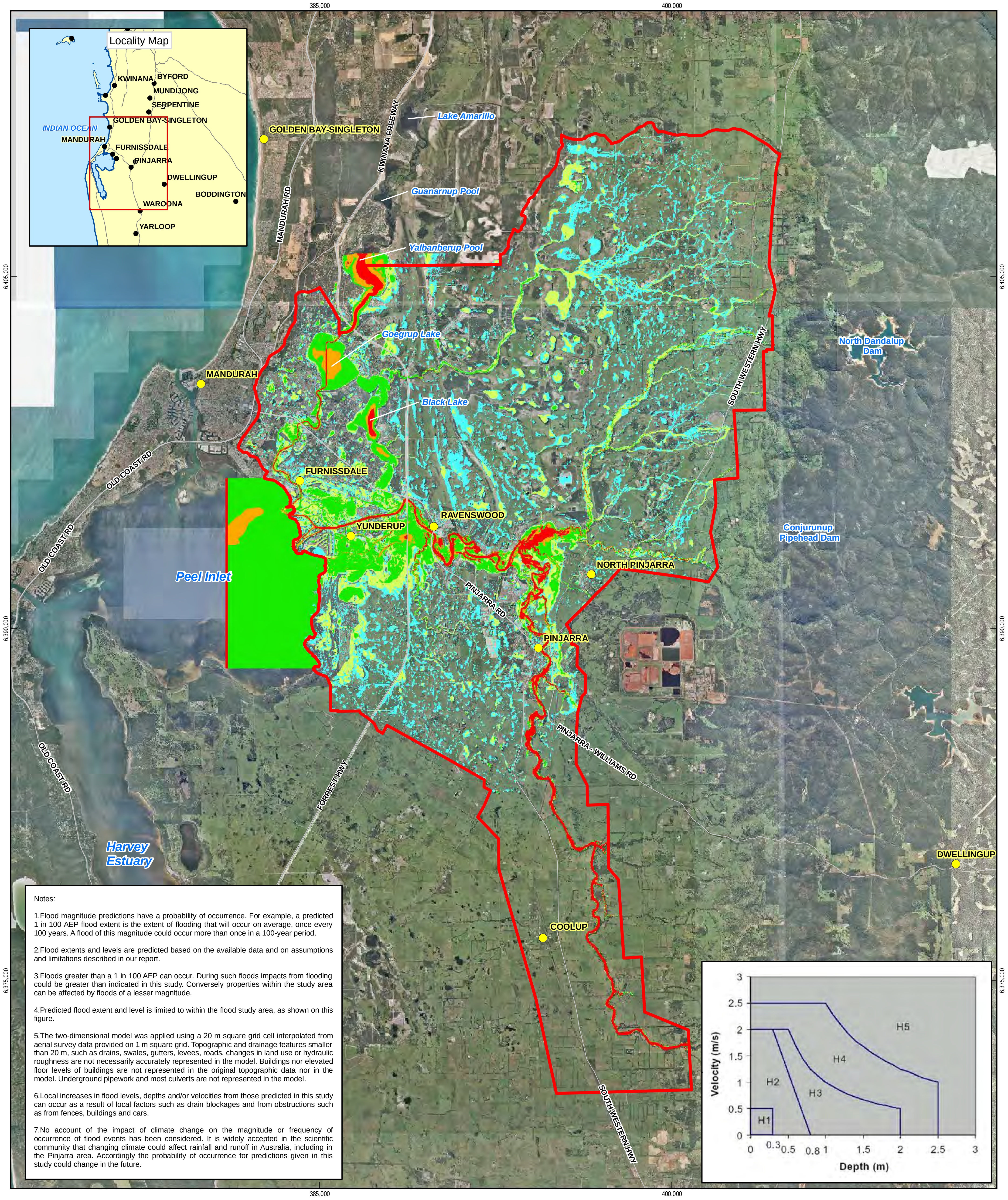
Data Source: GA: Populated Places - 2006; DOW: Swan Coastal Plain Imagery - 2008; MRWA: Road Centreline - 20090501; GHD: Flood Study Area - 20090501, Predicted Maximum Flood Depth - 20100202. Created by: xntan, kdiralu

6.4.3 Velocity and flood hazard mapping

Flood hazard mapping helps to define the risk that floodwaters pose to humans. Flood hazard is calculated as the product of flow velocity and depth and varies with characteristics of the population at risk. Maximum flood hazard can be mapped as hazard categories based on thresholds of depth and velocity.

Flood hazard for the 1 in 100 AEP event is mapped in Figure 39. Hazard categories are calculated according to the inset figure, which is based on guidelines developed by the Newcastle City Council (NCC 2003).

The mapping indicates areas of high hazard in main flow paths and where there is deep water with hazard reducing away from these areas.



Notes:

1.Flood magnitude predictions have a probability of occurrence. For example, a predicted 1 in 100 AEP flood extent is the extent of flooding that will occur on average, once every 100 years. A flood of this magnitude could occur more than once in a 100-year period.

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LEGEND

● Locality

▭ Flood Study Area

— Roads

Hazard Categories

🟥 5

🟧 4

🟩 3

▨ 2

▨ 1

1:150,000 (at A3)

0 0.75 1.5 3 4.5 6 7.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

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Predicted Flood Hazard
1 in 100 AEP

Job Number	61-2393701
Revision	0
Date	01 JUN 2010

Figure 39

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GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: GA: Populated Places - 2006; DOW: Swan Coastal Plain Imagery - 2008; MRWA: Road Centreline - 20090501; GHD: Flood Study Area - 20090501, Predicted Maximum Flood Depth - 20100202. Created by: xntan, kdiralu

6.4.4 Revised Floodway definition

Since the late 1970's the Department of Water and its predecessors have been preparing floodplain mapping that delineates the 1 in 100 AEP floodplain into areas of floodway and flood fringe.

The floodway is defined as those areas of the floodplain where significant discharge or storage of water occurs during major flood. They are often aligned with naturally defined channels and include areas which, if filled or even partially blocked, would cause a significant flood hazard by redistributing of flood flow, and/or by detrimentally increasing flood levels in the general area.

The flood fringe is the area of the floodplain, outside of the floodway, which is affected by flooding but where development could be considered provided appropriate measures are taken. These areas are generally covered by still or very slow moving waters during a 1 in 100 AEP event.

Floodway/flood fringe mapping has previously been undertaken previously for the Murray and Serpentine Rivers (PWDWA 1984 and Water Authority of Western Australia 1991). This information has been revised as part of this study. The preliminary floodplain mapping results highlighted areas where the current floodway delineation required review.

The impact of full development of the flood fringe increases flood levels by up to 0.3 m. The maximum increase in flood level occurs in South Yunderup and around the Forrest Highway south of the Murray River.

6.5 Hydrographs and long sections

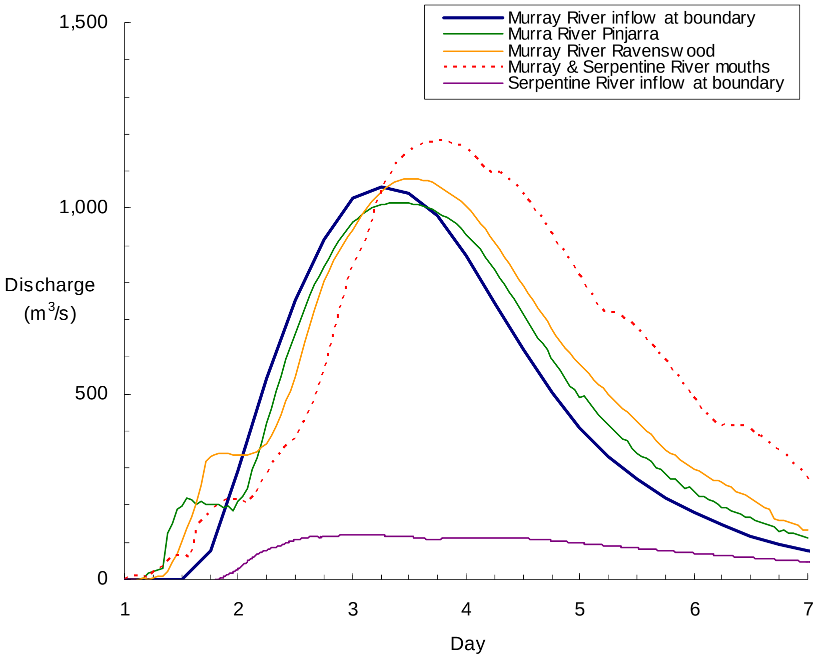
6.5.1 Hydrographs

Hydrographs at key locations along the Murray and Serpentine Rivers are shown in Figure 40. More details of predicted peak discharge and water level are given in Appendixes B and C.

The Murray River hydrograph shows some attenuation between the inflow point at the model boundary and Pinjarra. Effects of local catchments can be seen early in the hydrograph. The peak and volume of the hydrograph then increase with distance downstream, a reflection of inflows from Tate Gully and the Dandalup River.

The Serpentine River inflow is considerably smaller than the Murray River, and flood impact is appropriately smaller. Nevertheless, flows in the Serpentine River contribute some 100 m^3/s at the peak of the combined Murray and Serpentine River hydrograph at the Peel Inlet.

Figure 40 Predicted hydrographs in the flood study area

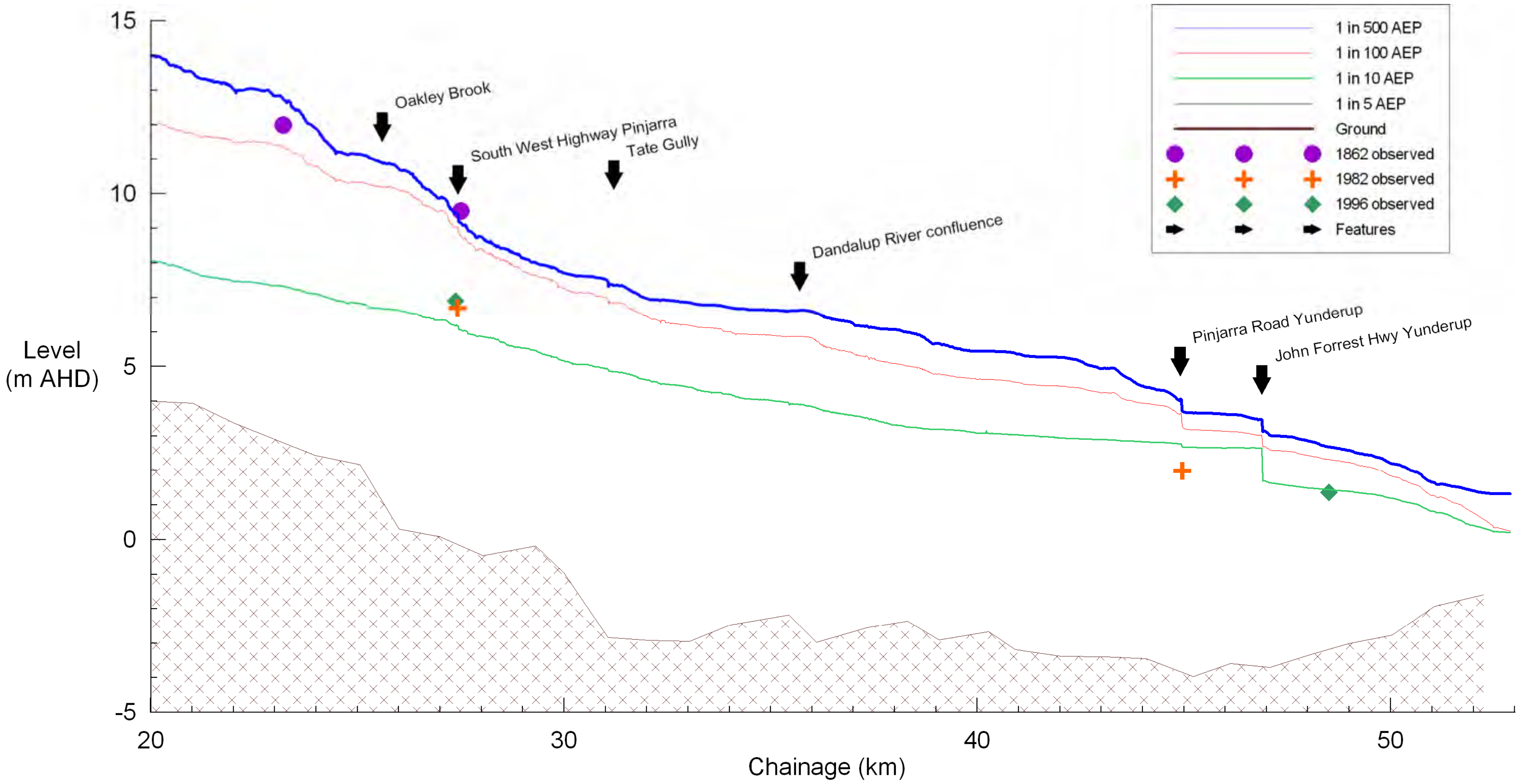


6.5.2 Long sections

A long section of the water levels in the Murray River with comparison to some historical events is shown in Figure 41. These results suggest that the event in 1862 had a magnitude greater that 1 in 100 AEP, but less than 1 in 500 AEP. The 1996 event is approximately 1 in 10 AEP, though there is variation along the length of the river.

Long sections for the larger tributaries within the flood study will be prepared as part of the final reporting. Examples for the Murray River and Nambeelup Brook are given in Appendix B.

Figure 41 Murray River long section



6.6 Detailed modelling

Detailed modelling for two tributaries of the Murray River (Winter Brook and Buchanans Drain) was undertaken to assess the sensitivity on flood levels and peak discharges to some of the assumptions inherent within the regional scale modelling. In particular, the modelling investigated the impact of event duration, channel geometry and additional infrastructure, such as culvert crossings on minor roads, on peak flood levels and flows. A description of the modelling is given in Appendix D.

The modelling indicates that peak flood levels and flow even for these small catchments is strongly linked to event volume. Accordingly, critical duration for the two catchments is consistent with that used in the regional model (i.e. 36 h).

The more detailed representation of channel geometry and infrastructure, mainly road culverts, allows better representation of flows at and downstream of obstructions, such as roads. It also allows a more accurate representation of peak flow rate in small drains, which is important for urban drainage planning in the area.

For example, flows in Buchanans Drain downsteam of Old Mandurah Road increase in the detailed model compared with the regional flood model. The Old Mandurah Road culvert and the drain itself, which is small at that point, was not represented in the regional flood model so the road tends to block flow. As the flow at this point is largely confined to the drain, general maximum flood levels in the area are not greatly affected.

In the Buchanan’s Drain catchment, an overland flow path across Curtis Lane is reduced in depth in the detailed model due to interception by Drain 41. This reduces the predicted flooding compared with the regional model but flow still occurs in the 1 in 100 AEP event.

The impact of debris partially or fully blocking culverts will affect flooding and should be considered as part of further investigations.

The detailed modelling study concluded that the regional model provides reasonable baseline flood extent information at a regional scale. It is recommended that detailed modelling is undertaken for the coastal plain tributaries to refine these regional estimates.

6.7 Climate Change

The impact of climate change on flood levels is linked to the inputs to the hydraulic modelling (level in Peel Inlet and the upstream hydrographs) and not to the modelling itself. However, potential changes in the roughness parameters within the model may occur as a result of climate induced changes in vegetation complexes. This possibility has not been examined as part of the modelling.

Sections 4 (hydrologic modelling) and 5 (tide and storm surge analyses) provide detail on the potential impact of climate changes on these inputs to the hydraulic model.

The design flood levels for the range of design events investigated provides insight into the likely sensitivity of the results to the impact of climate change with the 1 in 50 AEP 1 and in 500 AEP possibly representing the upper bound of potential changes in the design 1 in 100 AEP flow under a changing climate.

The 1 in 500 AEP flood levels are generally between 0.25 and 0.75 m higher than the 1 in 100 AEP levels but up to 2 m occurring in the Murray River upstream of Pinjarra. The 1 in 25 AEP flood levels are approximately 1 to 2 m lower than the design 1 in 100 AEP levels.

This illustrates that in general flood levels are likely to be relatively insensitive under a changing climate.

7. Floodplain development strategy

7.1 Background and objectives

Floodplains should be managed for the benefit of the whole community such that the risk and damages are minimised and environmental values are protected.

Accordingly, the objectives of sound floodplain development strategy are to:

- Ensure land use minimises flood risk and damage costs.
- Ensure all levels of government and the local community accept their responsibilities in floodplain management.
- Ensure appropriate floodplain mitigation measures minimise damage and are acceptable to the local community.
- Promote the use of non-structural rather than structural mitigation measures where possible.
- Ensure floodplain management measures have economic, social and environmental outcomes.
- Ensure flood forecasting and warning systems and emergency management arrangements cope with the impact of flooding.

The 1 in 100 AEP flood has been generally adopted in Australia and overseas as the basis for floodplain management planning.

Non-structural measures aim to reduce or avoid the susceptibility of new developments to flood damage as well as reducing the impact of flooding on existing developments. They include appropriate land use and building controls, acquisition of land and relocation, effective flood forecasting and flood warning, creation of public awareness and flood insurance.

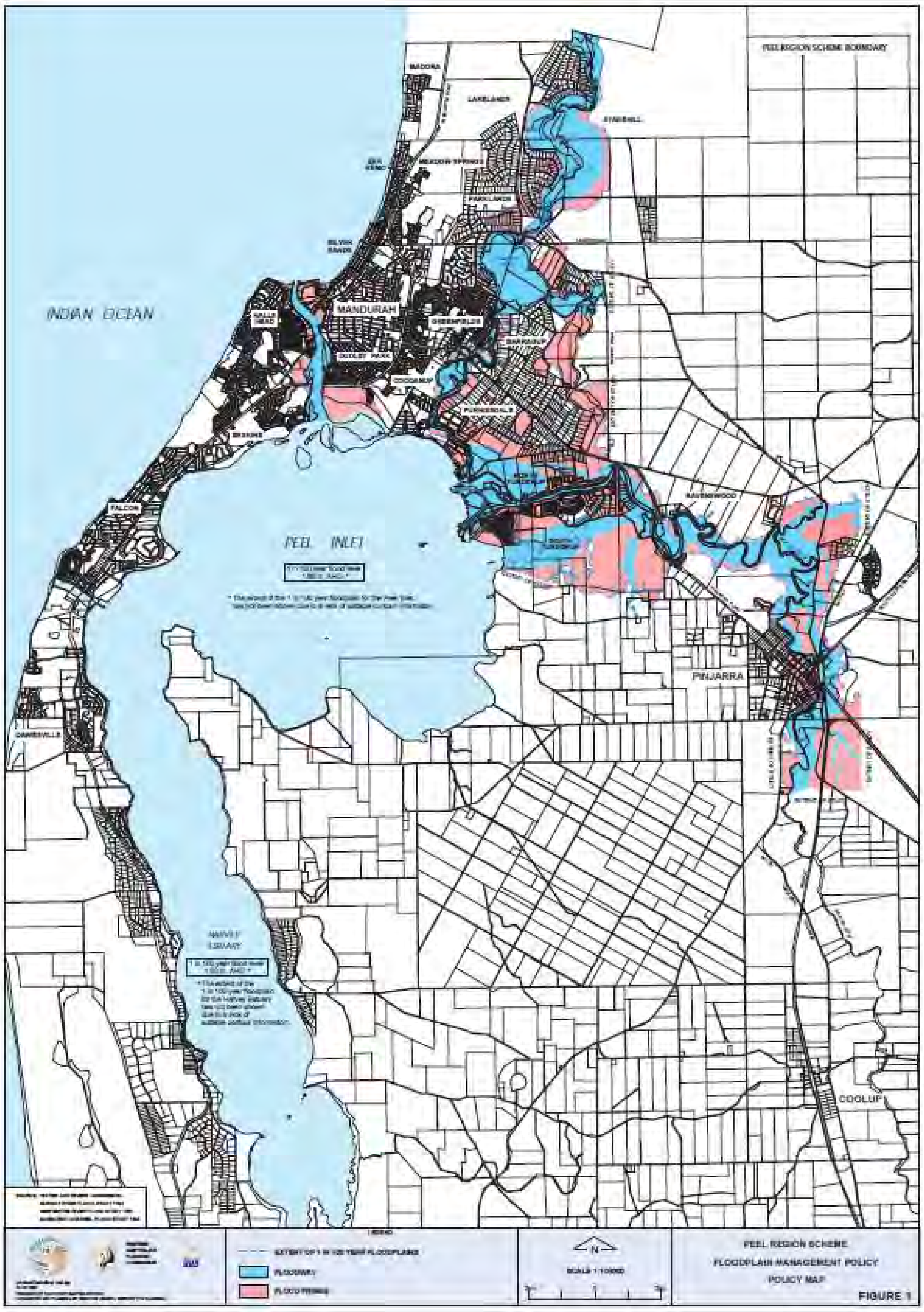
Structural flood mitigation measures physically modify the natural behaviour of flooding and are designed to reduce the frequency or impact of flooding on existing developments. They include levee banks, channel improvements, river diversions, retarding basins and flood mitigation dams.

7.2 Existing floodplain management strategies and policy

Floodplain development strategies currently exist for the Murray (PWDWA 1984) and Serpentine Rivers (Water Authority of Western Australia 1990). These strategies form the basis of the Peel Region Scheme Floodplain Management Policy (WAPC 2002). The policy provides guidance on appropriate land use and development within floodplains to minimise damage during major floods and to help maintain the natural flood carrying capacity of floodplains. The policy applies to all water courses within the Scheme area and around the Peel Inlet and Harvey Estuary.

The current Peel Region Scheme Floodplain Management Policy map is shown in Figure 42.

Figure 42 Peel Region Scheme Floodplain Management Policy



7.3 Flood mitigation measures

Floodplain mitigation measures fall into two broad categories – non-structural and structural. Non-structural measures can be broken into three sub-categories – land use planning; building and development controls; and flood warning and emergency planning.

7.3.1 Non-structural measures

Land use planning

Planning scheme controls are the most effective means of controlling developments in floodplains so as not to increase future flood damages or incur unnecessary risk to life, health and safety on occupants of floodplains. This measure is recognised as the most cost effective through controlling development on flood prone land.

Land use planning measures are based on the explicit recognition of flood prone land and identification of areas of high flood risk and important flow areas as "floodway" in planning scheme maps. This requires consultation and cooperation between the Department of Water, planning authorities and local government councils.

Voluntary land acquisition in more hazardous flood areas is another, although rarely utilised, example of a land use planning measure.

Building and development controls

Building and development controls involve works that make property (e.g. houses and roads) located within the floodplain less susceptible to flooding.

These controls include:

- Building floor level controls – specified minimum floor levels for new buildings in flood prone areas, based on predicted flood levels plus a freeboard.
- Flood proofing buildings – modifications to existing buildings (e.g. house raising, modifying wall finishes, doors) and building controls on new construction.
- Design guidelines – including addressing development setbacks, types of fencing and drainage.

More information on property modification is given in SCARM (2000) and NSW Government (2005).

Flood warning and emergency planning

This measure involves modifying the community's response to a flood threat. Emergency management planning needs to occur at the local level, with local stakeholders taking responsibility for implementation. The specific roles and responsibilities of the key players must be identified, integrated and coordinated

Unless designed for a probable maximum flood, all flood protection measures will at some time be overwhelmed by a larger flood than the design, and the implementation of appropriate flood warning, evacuation and cleanup plans can be effective in managing flood impacts. In some cases, emergency management planning may be the only economic or socially acceptable flood management option.

Emergency management planning measures include:

- Community awareness – including community involvement in floodplain management practices and education.

- Community preparedness – ability of the community to defend their properties and well-being from flood threat using appropriate preparatory and evacuation measures.
- Flood prediction and warning – warnings of impending major flows based on weather and/or streamflow observations or predictions.
- Emergency response plans – including evacuation planning and definition of tasks and responsibilities.
- Emergency recovery plans – including cleanup planning and definition of tasks and responsibilities.
- Flood insurance – including awareness of exclusions in general insurance policies relating to flood damage and potentially for locally funded speciality insurance schemes.

Further information on development of flood emergency response and flood warning plans is given in Commonwealth of Australia (1999), which includes a description of a total warning system, and SCARM (2000). The Bureau of Meteorology (BoM 2009) is a key agency for the provision of severe weather and flood warnings.

7.3.2 Structural measures

The purpose of structural measures is to modify the behaviour of a flood by reducing flood levels or velocities or by excluding floodwaters from areas at risk. Structural measures by their nature may have environmental and ecological impacts (positive or negative), so any proposal for such works must be subject to strict and detailed assessment in accordance with planning and assessment legislation.

Structural measures options include:

- Infiltration at source – use of soak wells, swales, basins, and other structures to encourage infiltration and reduce the volume of stormwater generation at the source. Infiltration is generally only an option in areas with high infiltration rates (i.e. sandy soils).
- Detention near source – use of basins (e.g. swales, small dams, and wetlands) widely disbursed throughout mainly urban areas to temporarily hold back stormwater, thus reducing downstream peak flow rates and water levels. Storage volumes need to be large enough to contain a suitable amount of stormwater and need to have suitably regulated outlets to control outflow rates.
- Regional detention – larger storages strategically located to provide temporary flood storage.
- Flood protection levees – use of levees to modify or control flow paths. These generally should be used for protecting existing development in urban areas, preferably in conjunction with non-structural measures. They should be designed, constructed and maintained to appropriate standards.
- Channel modification – modifying watercourses to safely convey flow, particularly reducing the effects of man-made structures in creating ponding (afflux), such as undersized culverts and constructed drains. A component of this option is maintenance and upgrade of drainage infrastructure.
- Maintenance – appropriate maintenance is important to prevent levee failure, culvert blockage, scour, etc. which reduces channel capacity and to minimise erosion and deposition issues. Upgrade of undersized infrastructure should also be included as part of a maintenance/upgrade programme.

7.4 Identified issues

The following flood management issues have been identified, considering input from the community and in discussion with the Department of Water:

- Pressure for development of flood prone land.
- Pressure for a reduction in the land defined as floodways.
- Clarification of the boundaries of defined floodways and building controls within these floodways.
- Clarification of minimum floor levels required for flood protection, as this affects fill levels and development cost.
- Community expectation that appropriate responses to climate change impacts will be adopted.
- Updated 1 in 100 AEP flood levels, accounting for an increased data record and changes to the Peel Inlet / Harvey Estuary since the Dawesville Channel was built.
- Historical encroachment of development into floodways (e.g. North Yunderup and Tate Gully).
- Street landscaping issues associated with elevated floor levels of new buildings in older developed areas.
- Importance of maintaining Wilgie Creek as a functioning floodway.

7.5 Recommended floodplain development strategy

7.5.1 Principles

The following general principles were used in developing the recommended floodplain development strategy:

- Proposed development has an adequate level of flood protection.
- Proposed development does not detrimentally impact on the existing flooding regime of the general area.
- The public has adequate protection from flood hazard (e.g. flow depth and velocity, frequency and duration of overtopping of road crossings).
- Environmental impact issues, both as a result of flood mitigation works and resulting from increased flood flows/inundation following development, are important.
- Proposed flood management works (both structural and non-structural) need to be economically acceptable. For example, the benefit of flood management works should be weighed against the cost of implementing or not implementing the works.

Existing floodplain development strategies (PWD, 1984 and WAWA, 1991) promote the use of non-structural measures and are based around appropriate land use planning and building and development controls. These form the basis of the existing Peel Region Scheme Floodplain Management Policy (WAPC, 2002). No change to this strategy for the Peel Inlet / Harvey Estuary and Serpentine River and Murray River floodplains is recommended except that reference should be made to the results of the revised 1 in 100 AEP floodplain mapping from this study.

Based on the above principles and the revised 1 in 100 AEP floodplain mapping, a recommended floodplain development strategy is presented. The strategy has been developed in consultation with staff from the Department of Water and input from key stakeholders and the community.

7.5.2 Non structural measures

Land use planning

Land use planning and development controls are the primary flood management measures currently in place and these remain appropriate for the study area.

Land use zoning is the key planning instrument to deliver floodplain development strategies. In the study area both the Peel Region Scheme (PRS) and several Local Planning Schemes (LPS) are in effect. These planning instrument are administered by the Department of Planning (DoP) and the relevant local government, with technical advice provided by the Department of Water.

In order to avoid increasing the impacts of flooding through inappropriately located land use and development, flood risk must be considered in preparing both Regional and Local planning schemes and any amendments to such schemes to inform land use planning and development approval decisions.

The existing floodway and flood fringe mapping for the Murray and Serpentine Rivers was reviewed based on the new 1 in 100 AEP floodplain mapping results and detailed survey information. The reduction in design peak flows has resulted in minimal reductions to the defined floodways of the Murray River (PWD, 1984) and Serpentine River (WAWA, 1991). There are no areas where the floodway has been increased within the Murray and Serpentine River floodplains. There have also been some changes associated with the actual location of bridge openings of the Kwinana Freeway and John Forrest Highway crossing the rivers.

Floodway delineation of the Murray River floodplain has not been extended south of the railway crossing approximately 8 kilometres upstream of Pinjarra. However, floodplain mapping for this section of the Murray River and the existing smaller watercourses throughout the study area has been prepared and will be used by the Department of Water when assessing planning proposals in these areas in order to conform to the floodplain development strategy principles.

Building and development controls

In order to conform with the floodplain development strategy, and utilising the revised floodway and flood fringe mapping for the Murray and Serpentine Rivers and Peel Inlet / Harvey Estuary, the following are recommended building and development controls:

- Proposed development (i.e. filling, building, etc) that is located outside of the floodway is considered acceptable with respect to major flooding. However, a minimum habitable floor level of 0.50 m above the adjacent 1 in 100 AEP flood level is recommended to ensure adequate flood protection.

  However, in addressing other planning issues (such as aesthetic, streetscaping and privacy issues associated with possible high fill levels) for new dwellings in existing subdivided / developed areas, a reduction in the freeboard from 0.50 m to 0.15 m above the 1 in 100 AEP flood level can be considered.

  Similar consideration should also be given to proposed commercial properties.

- Proposed development (i.e. filling, building, etc) that is located outside of the floodway and adjacent to the Peel Inlet / Harvey Estuary is considered acceptable with regard to major flooding. The 1 in 100 AEP flood level for this area is estimated to be 2.10 m AHD and this includes a 0.90 m sea level rise projection over the next century. A freeboard of 0.60 metre above the 1 in 100 AEP flood level is recommended to account for factors such as the result of wind and wave action and local variations in flood levels. Consequently, a minimum habitable floor level of 2.70 m AHD for proposed development is recommended to ensure adequate flood protection.

  However, in addressing other planning issues (such as aesthetic, streetscaping and privacy issues associated with possible high fill levels) for new dwellings in existing subdivided / developed areas, a reduction in the freeboard from 0.60 m to 0.15 m above the 1 in 100 AEP flood level can be considered, ie, a minimum habitable floor level of 2.25 m AHD.

- Proposed development (i.e. filling, building, etc) that is located within the floodway and is considered obstructive to major flows is not acceptable as it would as it would detrimentally impact on the existing flooding regime of the general area. No new dwellings are acceptable within the floodway.

  However, in certain circumstances, proposed development within the floodway may be considered based on its merit. The major factors that should be considered include depth of flooding, velocity of flow, its obstructive effects on flow, possible structural and potential flood damage, and difficulty in evacuation during major floods and its regional benefit. These circumstances include the re-development of an existing dwelling, public works (i.e. bridges) or community facilities (i.e. picnic facilities) that are considered of significant regional benefit.

  The replacement of an existing dwelling within the floodway may be considered provided:

  - existing dwelling is demolished or relocated.
  - effective width of obstruction of the new dwelling to major flows is no greater than the effective width of the existing dwelling.
  - proposed dwelling achieves a minimum habitable floor level of at least 0.50 m above the 1 in 100 AEP flood level.

  It is the responsibility of the proponent to undertake the necessary reviews, assessments and modelling to demonstrate, to the satisfaction of the Department of Water, that proposed development within the floodway is consistent with the floodplain management principles.

- Regional scale floodplain mapping for the North and South Dandalup Rivers, Nambeelup Brook and other smaller tributaries that pass through the study area has also been prepared. The mapping provides indicative information on flood extent and levels to assist land use planning. In addition, peak discharge information for a number of locations within the study area has been determined.

  For watercourses where the 1 in 100 AEP floodplain mapping has not been delineated into floodway and flood fringe areas, proposed development that is located within the 1 in 100 AEP floodplain will be assessed on a case-by case basis until such time as detailed structure planning that considers major flooding has been undertaken for the area.

  It is the responsibility of the proponent to undertake the necessary reviews, assessments and modelling to demonstrate, to the satisfaction of the Department of Water, that proposed development within the floodplain is consistent with the floodplain management principles.

Figure 43 Revised floodway, flood fringe and 1 in 100 AEP flood extent

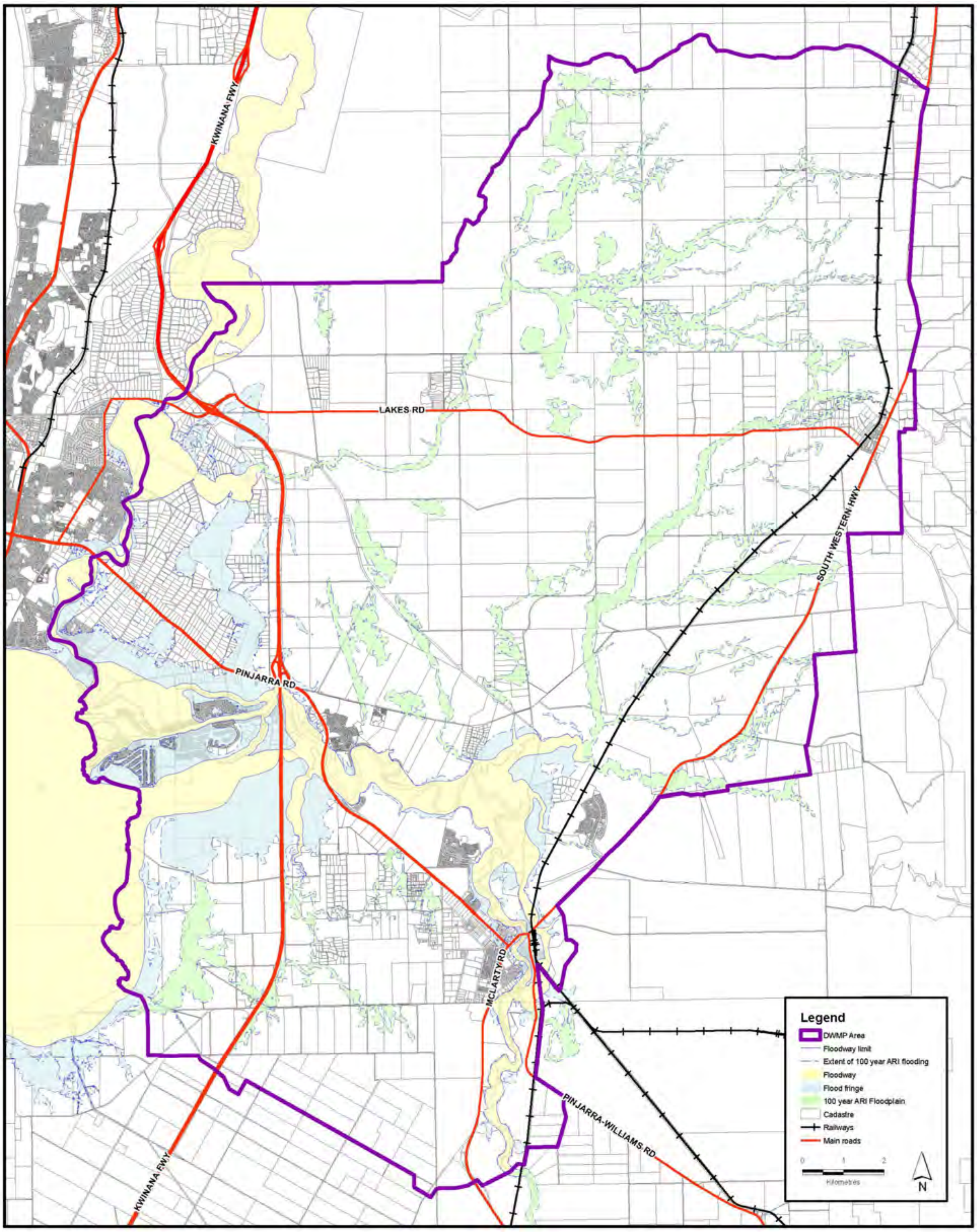
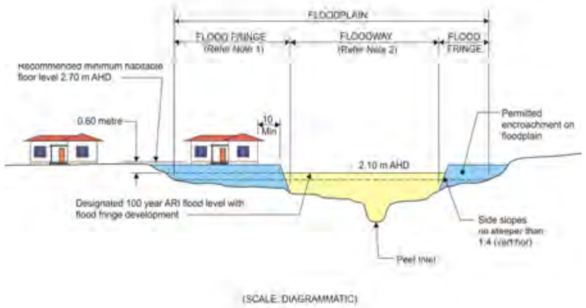


Figure prepared by Department of Water.

Figure 44 Recommended floodplain development strategy

RECOMMENDED FLOODPLAIN DEVELOPMENT STRATEGY - Type 1
Peel Inlet / Harvey Estuary



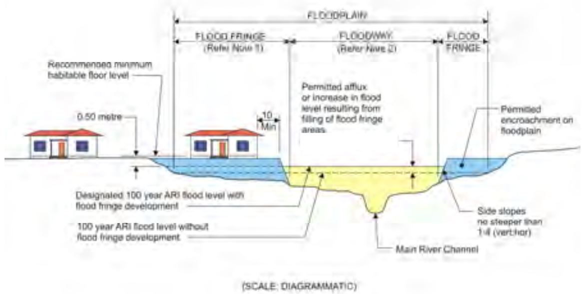
GENERAL NOTES

1. Proposed development (ie, filling, building, etc) that is located within the *flood fringe* is considered acceptable with respect to major flooding. However, a minimum habitable floor level of 2.70 m AHD is recommended to ensure adequate flood protection.

2. Proposed development (ie, filling, building, etc) that is located within the *floodway* and is considered obstructive to major flows is not acceptable as it would increase flood levels upstream. No new dwellings are acceptable within the floodway.

3. A failure to properly adhere to these recommendations will result in a greater exposure to risks of flood damage. This advice is related to major flooding only and other planning issues, such as environmental and ecological considerations, may also need to be addressed.

RECOMMENDED FLOODPLAIN DEVELOPMENT STRATEGY - Type 2
Murray River and Serpentine River



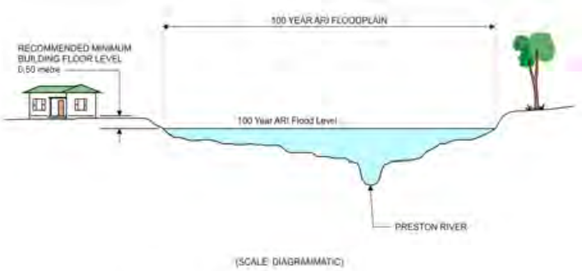
GENERAL NOTES

1. Proposed development (ie, filling, building, etc) that is located within the *flood fringe* is considered acceptable with respect to major flooding. However, a minimum habitable floor level of 0.50 metre above the adjacent 100 year ARI flood level is recommended to ensure adequate flood protection.

2. Proposed development (ie, filling, building, etc) that is located within the *floodway* and is considered obstructive to major flows is not acceptable as it would increase flood levels upstream. No new dwellings are acceptable within the floodway.

3. A failure to properly adhere to these recommendations will result in a greater exposure to risks of flood damage. This advice is related to major flooding only and other planning issues, such as environmental and ecological considerations, may also need to be addressed.

RECOMMENDED FLOODPLAIN DEVELOPMENT STRATEGY - Type 3
Murray Area



GENERAL NOTES

1. The 100 year ARI flood level is expected to occur, on average, once every 100 years. Floods higher than this level will occur but, on average, will be less frequent.

2. To ensure adequate flood protection is provided to future development the recommended floodplain management strategy is:

- For proposed development located outside of the 100 year ARI floodplain, a minimum habitable floor level of 0.50 metre above the adjacent 100 year ARI flood level is recommended.
- For proposed development located within the 100 year ARI floodplain, the Department of Water will provide advice on each proposal based on its merits. Factors that will be examined are depth of flow, velocity of flow, its obstructive effect to floodwaters, potential flood damage, etc. If development is considered acceptable, a minimum habitable floor level of 0.50 metre above the adjacent 100 year ARI flood level would be recommended.

3. A failure to properly adhere to these recommendations will result in a greater exposure to risks of flood damage. This advice is related to major flooding only and other planning issues, such as environmental and ecological considerations, may also need to be addressed.

Flood warning and emergency planning

Flood emergency planning should focus on the development and maintenance of emergency response and recovery plans, maintain and improve existing flood prediction and warning systems and invest further in community awareness and preparation. The Fire and Emergency Services Authority of WA, Bureau of Meteorology and Department of Water has an operational flood warning program in place for towns along the Murray River, The importance of the telemetered rainfall and streamflow sites that underpin the flood warning network forms an important part of emergency flood management for the area.

The floodplain mapping presented in this report will assist in the preparation of flood emergency response plans for the area. The mapping indicates that the following major population centres have flood prone areas:

- Pinjarra townsite and access roads – located across the Murray River and major floodway Tate Gully and access from the north crosses the Dandalup River system.
- North Pinjarra – access from the north is across the South Dandalup River.
- North Dandalup – access from the north is across the North Dandalup River and from the south across the Murray and South Dandalup Rivers.
- Ravenswood and Yunderup – located close to the Murray River with access affected by flooding in the Murray and Serpentine Rivers.
- Furnissdale – located close to the Serpentine River with access affected by flooding from the Serpentine and Murray Rivers.

The mapping also provides information on whether key transport routes through the area will be trafficable during the range of flood events examined.

In a 1 in 100 AEP event, Pinjarra Road will be inundated on the southern side of the Murray River near Ravenswood and Old Mandurah Road in places along its length. South Yunderup Road immediately east of the John Forrest Highway underpass will be inundated for a significant period cutting access to South Yunderup. South Yunderup Road is inundated at this location during events as frequent as 1 in 10 AEP. Other local roads will be affected by inundation from a 1 in 100 AEP event to varying degrees.

7.5.3 Structural measures

There are currently no broad-scale, structural mitigation measures within the flood study area. While there has been discussion of channel/diversion works for breakout flows from the Murray River just south of Pinjarra, no definite plans have been developed. This study has found that the breakout does not become active until the flows exceed the 1 in 100 AEP event. There are numerous constructed drains within the area which reduce ponding in low-lying areas.

Future roads must consider flood impacts as part of the design process and as such are not structural works. Some potential exists for upgrades to existing roads to better manage flood impacts, mainly in areas away from the Murray and Serpentine River channels. For example, an upgrade of the South West Highway near Greenlands Road would provide additional flood protection to area west of the Highway. This would enable planning for this area to continue without a requirement to manage an overflow from the Murray River through the area.

Due to the impact that road and rail embankments have on flood flows, particularly away from the main Murray and Serpentine River channels, it is recommended that new roads be designed to maintain existing surface water flow paths for the 1 in 100 AEP event and that existing roads be reviewed and upgraded as required.

Infiltration at source for roof and road runoff is routinely required as part of development approvals through the study area but the focus of these works is more toward maintaining existing streamflow and water quality regimes, not flood mitigation.

Routine maintenance of flood management structures, particularly culverts, bridges and drains is and will continue to be important.

7.5.4 Relationship with the Better Urban Water Management Framework

The WAPC (2008) Better Urban Water Management (BUWM) document provides a framework for how water resources should be considered at each planning stage by identifying actions and investigations required to support the particular planning decision being made. Information derived at each planning stage can then be carried through to inform the subsequent planning stage(s).

Application of the recommended approach aims to ensure consideration of issues which are relevant to the site at a level of detail which is appropriate to the planning decision being made and the degree of risk associated with the proposal, in terms of ecological and community impacts.

The recommended floodplain development strategy for the Murray Area (Section 7.5.4) has been developed in accordance with the principles of the BUWM framework. The following summarises the floodplain management information requirements at the six stages of the planning and development process (WAPC, 2008).

Regional / Sub-Regional

The floodplain mapping (Figure 43) adequately defines the land that may be subject to inundation during major events at a Regional level. This mapping and the discussion on proposed re-zoning of land within the recommended floodplain development strategy may suitably address the floodplain management requirements at this level of planning. Additional supporting information may be required when considering significant areas of floodplain. Additional information may include:

- Detailed topographic survey information.
- Conceptual drainage design that outlines how floodwaters (based on the estimates of peak discharge provided in this report) will be managed.
- Additional flood modelling illustrating revised flood discharge/extent/levels and/or impacts relating to the proposal.
- Conceptual development and building controls (refer to Section 7.3.1).

District

The floodplain mapping (Figure 43) adequately defines the land that may be subject to inundation during major events at a District level. This mapping and the discussion on proposed re-zoning of land within the recommended floodplain development strategy may suitably address the floodplain management requirements at this level of planning. Additional supporting information may be required when considering significant areas of floodplain. Additional information may include:

- Detailed topographic survey information.
- Conceptual drainage design that outlines how floodwaters (based on the estimates of peak discharge provided in this report) will be managed.
- Additional flood modelling illustrating revised flood discharge/extent/levels and/or impacts relating to the proposal.
- Conceptual development and building controls (refer to Section 7.3.1).

Local

The floodplain mapping (Figure 43) adequately defines the land that may be subject to inundation during major events at a local level. This mapping and the discussion on proposed rezoning and sub-division of land within the recommended floodplain development strategy may suitably address the floodplain management requirements at this level of planning.

Additional supporting information will be required when considering local structure plans or schemes that include encroachment into a defined floodplain not identified as flood fringe within the floodplain mapping. Proponents will need to demonstrate, to the satisfaction of the DoW, that the proposal is consistent with the floodplain management principles set out in Section 7.5.1. Additional information may include:

- Detailed topographic survey information.
- Detailed drainage design that outlines how floodwaters will be managed.
- Additional flood modelling illustrating revised flood discharge/extent/levels and/or impacts relating to the proposal.
- Detailed information on appropriate building and development controls (refer to Section 7.3.1).

Subdivision

Additional supporting information will be required when considering a sub-division proposal that encroaches into a defined floodplain that has not been identified as flood fringe within the floodplain mapping. Proponents will need to demonstrate, to the satisfaction of DoW, that the proposal is consistent with the floodplain management principles set out in Section 7.5.1. Additional information may include:

- Detailed topographic survey information
- Detailed drainage design that outlines how floodwaters will flow through the proposed sub-division
- Additional flood modelling illustrating revised flood discharge/extent/levels and/or impacts relating to the proposed subdivision
- Appropriate building and development controls (refer to Section 7.3.1)

Development

The floodplain mapping identifies the areas within which proposed development is considered acceptable with regard to major flooding.

Additional supporting information will be required when considering a development application that encroaches into a defined floodplain that has not been identified as flood fringe within the floodplain mapping. Proponents will need to demonstrate, to the satisfaction of DoW, that the proposal is consistent with the floodplain management principles set out in Section 7.5.1. Additional information may include:

- Detailed topographic survey information.
- Additional flood modelling illustrating revised flood discharge/extent/levels and/or impacts relating to the proposed subdivision.
- Appropriate building and development controls (refer to Section 7.3.1).

When proposed development is considered acceptable a minimum habitable floor level above the estimated 100 year ARI flood level is recommended to ensure adequate flood protection.

Post development

The floodplain mapping will assist in the preparation of flood emergency plans for the area and can assist in increasing the community awareness and preparation for major flooding. Further discussion on flood emergence planning measures is provided in Section 7.3.1.

7.5.5 Review

This study and recommended strategy has required the development of detailed hydrologic and hydraulic modelling tools. These tools should become the basis for assessment of future proposals for development with the floodplain. The Department of Water will be the custodian of the modelling tools and will maintain the hydraulic model and information from this study.

The ongoing management of the floodplain models will be critical in ensuring that development occurs without causing detrimental impacts. The strategy for managing the tools aims to maintain knowledge of the systems within the Department of Water whilst the greatest development pressure is managed.

The Department of Water can assist proponents by providing guidance on appropriate methodologies and the acceptable impact of the proposed development with regard to major flooding. The Department of Water can provide the relevant floodplain mapping, existing hydraulic model for the 1 in 100 AEP event and available survey information for the area.

The strategy and tools developed as part of this study should be reviewed following any large flood event, as more detail on expected climate change impacts become available or after a period of time no greater than 5 years.

8. Recommendations for further work

8.1 Recommendations

The information from the gauging station at Baden Powell on the Murray River has been extremely important in the preparation of this strategy. It is recommended that streamflow monitoring at this site be continued to enable the hydrologic analyses to be reviewed in future.

The design flood estimates for the catchments on the western slopes of the Darling Scarp are based on regional information. It is recommended that suitable streamflow gauging sites are operated to verify/improve the estimates for these streams.

To ensure the best planning outcome, structure planning for broad areas that considers flooding is encouraged in preference to small scale development occurring in piecemeal fashion.

The stakeholders and community should provide input to the floodplain development strategy

8.2 Further work

It is recommended that further detailed modelling of individual catchments be undertaken as a part of the overall DWMP project.

9. References

Australian Government (2009). Connected Water. http://www.connectedwater.gov.au/framework/baseflow_separation.php

BoM (2009). Bureau of Meteorology. http://www.bom.gov.au/.

Chow (1959). Open-Channel Hydraulics. McGraw-Hill: London.

Climate Change (2009), Climate Change in Australia, http://www.climatechangeinaustralia.gov.au.

Commonwealth of Australia (1999). Australia Emergency Manuals Series. Volume III – Emergency Management Practice Volume 3 – Guidelines. Commonwealth of Australia: Canberra.

Damara WA Pty Ltd. (October 2009). Mandurah Region, Development in Floodprone Areas, Review of Available Information and Existing Policies. Draft report.

Davies, J.L. (1980) “Geographic Variation in Coastal Development.” Longmans, London.

DoE (2004). User Manual for the “WA CRC-FORGE EXTRACT” computer program. Department of Environment, Government of Western Australia, Surface Water Hydrology Series Report No. HY20.

DPI (2004). Port Beach Coastal Erosion Study. Technical Report, Report No. 427, July 2004, Government of Wester Australia, Department for Planning and Infrastructure.

Fugro Spatial Solutions Pty Ltd. (2008). Metadata Report for Department of Water WA J221571 Armadale – Dunsborough North Metro & 3 Towns Lidar Survey.

GA (2009). Geoscience Australia. http://www.ga.gov.au/ Australian Government.

GHD (2009a). Murray Drainage and Water Management Plan and Associated Studies - Storm Surge and Tide Study. Unpublished report prepared by GHD Pty. Ltd. For the Department of Water: Perth.

Hearn, C., Lukatelich, R., and McComb, A.J. (1994). “Coastal lagoon ecosystem modeling.” *In:* Kjerfve, B. (ed), Coastal Lagoon Processes, Elsevier, Amsterdam, 471-502.

HR Wallingford (1980). Design of Seawalls for Wave Overtopping. Report EX924.

IPCC, (2007). Climate Change 2007: The Physical Science Basis. Summary for Policymakers, Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change. The United Nations Environment Program and the World Meteorological Organisation. 5th February 2007.

Jenkins (2001). AUS-IFD. Version 2. University of Wollongong.

Laurenson, E.M., Mein, R.G. and Nathan, R.J. (2007). RORB Version 6. Runoff routing program. User Manual. Monash University, Department of Civil Engineering, Sinclair Knight Merz Pty. Ltd. And Melbourne Water Corporation.

Melbourne Water (2003). Guidelines for Development in Flood-Prone Areas. Melbourne Water: Melbourne.

NCC (2003), Flood Policy 2003 – Part A Principals, Newcastle City Council, Newcastle.

NSW Government (2005). Floodplain development manual, the development of flood liable land. Department of Infrastructure, Planning and Natural Resources: Sydney.

Pattiaratchi, C. and Buchan, S. (1991). “Implications of long-term climate change for the Leeuwin Current.” *Journal of the Royal Society of Western Australia,* 74, 133-140.

PWDWA (1984). Murray River Flood Study. Drawing series No’s: PWDWA 51796 and PWDWA 54586. Engineering Division, Public Works Department, Western Australia.

Ranasinghe, R. and Pattiaratchi, C. (2000). “Tidal inlet velocity asymmetry in diurnal regimes.” *Continental Shelf Research*, 20, 2347-2366.

Pearce, L. (2006). Small Dams Flood Study – Regional Analysis. Water Corporation.

Pilgrim D.H. (ed.) (2001). Australian rainfall and runoff. A guide to flood estimation*. Vol. 1.* The Institution of Engineers Australia: Barton.

Pugh, D.T. (1987). Tides, Surges, and Mean Sea-level, John Wiley & Sons, New York.

SCARM (2000). 'Floodplain Management in Australia. Best Practice Principals and Guidelines.' Agriculture and Resource Management Council of Australia and New Zealand. Standing Committee on Agriculture and Resource Management. CSIRO Publishing: Collingwood, Victoria.

SKM (2010). Serpentine River Floodplain Management Study. Draft unpublished report prepared by SKM Pty. Ltd. for the Government of Western Australia, Department of Water.

SKM (1999). Murray River Flood Study Review. Unpublished report prepared by SKM Pty. Ltd. for the Shire of Murray.

SKM (2009). Shire of Boddington Floodplain Management Study. Unpublished report prepared by SKM Pty. Ltd. for the Shire of Boddington.

Wallingford, H.R. (1980). Design of Seawalls for Wave Overtopping. Report EX924.

WAPC (2002). Peel region Scheme. Floodplain Management Policy. Western Australian Planning Commission: Perth.

Water Authority of Western Australia (1990). Revised Lower Serpentine Flood Study. Water Authority of Western Australia, Water Resources Directorate Report No. WS20R1: Perth.

Water Authority of Western Australia (1991). Lower Serpentine Flood mapping. Series BL06. Water Authority of Western Australia: Perth.

WRC (2000). Floodplain Management. Water Facts 14. Department of Water: Perth.

10. Acknowledgements

The following people's contribution to the floodplain development study is gratefully acknowledged: Leonie Joyce, Simon Rodgers, Rick Bretnall, Casey Skaleski, Christian Hoermann, Chris Roach, Kate Roehner, Nikki Tan, Leanne Pearce, Robin Connolly, Casey Skaleski, Nicholas Deeks, Christian Hoermann, Helen Brookes and Chris Roach.

Appendix A

Flood frequency data

614006 Murray River Baden Powell

Summer (Oct-Mar)				Winter (Apr-Sept)				Annual (Apr-Mar)			
Year	Month	Adopted max Q (m^3/s)	Comment	Year	Month	Adopted max Q (m^3/s)	Comment	Year	Month	Adopted max Q (m^3/s)	Comment
1940	`	0.0	No notable event	1940		0.0	No notable event.	1940		0.0	No notable event.
1941		0.0	No notable event	1941		64.1		1941		64.1	
1942		0.0	No notable event	1942		138.0		1942		138.0	
1943		0.0	No notable event	1943		72.7		1943		72.7	
1944		0.0	No notable event	1944		19.1		1944		19.1	
1945		0.0	No notable event	1945		793.0	There is some information that suggests this flow rate is ~ 880 m3/s.	1945		793.0	
1946	11	10.0	Nov	1946		419.0		1946		419.0	
1947	10	12.9	Oct	1947		110.0		1947		110.0	
1948	10	46.8	2/10/48, close to winter peak on 30/9/48	1948	9	45.4	30/9/1948, close to annual/summer peak.	1948		46.8	
1949	10	8.5	Oct	1949		156.0		1949		156.0	
1950	10	11.8	Oct	1950		26.0		1950		26.0	
1951	12	28.6	Dec	1951		46.0		1951		46.0	
1952	10	0.0	Censored as part of winter recession	1952	8	0.0	Deleted, peak missing, trace got to about 60 m^3/s.	1952	8	0.0	Deleted, peak missing.
1953	10	0.0	Censored as part of winter recession	1953	7	210.6		1953	7	210.6	
1954	10	0.0	Censored as part of winter recession	1954	8	124.5		1954	8	124.5	

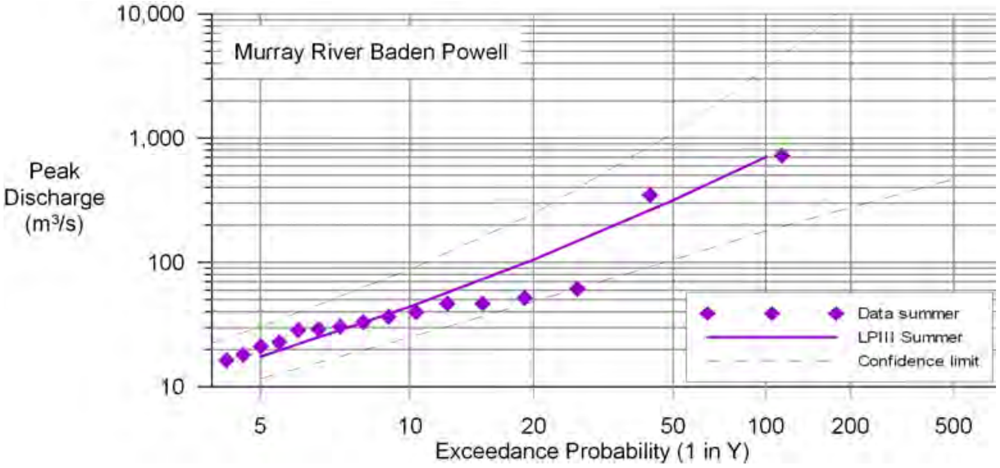
Summer (Oct-Mar)				Winter (Apr-Sept)				Annual (Apr-Mar)			
1955	10	722.0	*** Feb 55 estimate. Second event in August estimated at 478 m3/s	1955	8	478.1		1955	8	722.0	*** Feb 55 estimate. Second event in August estimated at 478 m^3/s.
1956	10	0.0	Censored as part of winter recession	1956	8	181.3		1956	8	181.3	
1957	10	0.0	Censored as part of winter recession	1957	6	182.5		1957	6	182.5	
1958	10	0.0	Censored as part of winter recession	1958	7	492.9		1958	7	492.9	
1959	10	0.0	Censored as part of winter recession	1959	8	106.7		1959	8	106.7	
1960	10	0.0	Censored as part of winter recession	1960	7	0.0	Deleted, peak missing, trace got to about 200.	1960	7	0.0	Deleted, peak missing.
1961	10	0.0	Censored as part of winter recession	1961	7	70.2		1961	7	70.2	
1962	10	0.0	Censored as part of winter recession	1962	6	57.6	100 m^3/s from DoW.	1962	6	57.6	100 m3/s from DoW.
1963	10	0.0	Censored as part of winter recession	1963	8	376.8		1963	8	376.8	
1964	10	0.0	Censored as part of winter recession	1964	8	553.6		1964	8	553.6	
1965	10	51.9	only 2 months, retained	1965	8	120.5		1965	8	120.5	
1966	10	0.0	Censored as part of winter recession	1966	7	185.4		1966	7	185.4	
1967	10	0.0	Censored as part of winter recession	1967	6	332.6		1967	6	332.6	
1968	10	0.0	Censored as part of winter recession	1968	6	130.1		1968	6	130.1	

Summer (Oct-Mar)				Winter (Apr-Sept)				Annual (Apr-Mar)			
1969	2	0.0	Censored as part of winter recession	1969	6	20.1		1969	6	20.1	
1970	10	39.8		1970	6	169.6		1970	6	169.6	
1971	10	46.5		1971	8	35.7		1971	10	46.5	
1972	10	0.0	Censored as part of winter recession	1972	8	65.4		1972	8	65.4	
1973	10	0.0	Censored as part of winter recession	1973	8	167.3		1973	8	167.3	
1974	10	13.0		1974	8	273.4		1974	8	273.4	
1975	10	0.0	Censored as part of winter recession	1975	7	134.5		1975	7	134.5	
1976	10	0.0	Censored as part of winter recession	1976	8	58.3		1976	8	58.3	
1977	11	0.0	Censored as part of winter recession	1977	8	44.3		1977	8	44.3	
1978	10	22.9		1978	7	221.5		1978	7	221.5	
1979	10	0.0	Censored as part of winter recession	1979	7	21.8		1979	7	21.8	
1980	10	9.0		1980	8	17.3		1980	8	17.3	
1981	1	347.3		1981	8	124.8		1981	1	347.3	
1982	10	9.4		1982	8	15.4		1982	8	15.4	
1983	10	0.0	Censored as part of winter recession	1983	7	304.4		1983	7	304.4	
1984	11	0.0	Censored as part of winter recession	1984	9	51.0		1984	9	51.0	
1985	2	0.0	Censored as part of winter recession	1985	8	69.3		1985	8	69.3	
1986	10	0.0	Censored as part of	1986	8	57.4		1986	8	57.4	

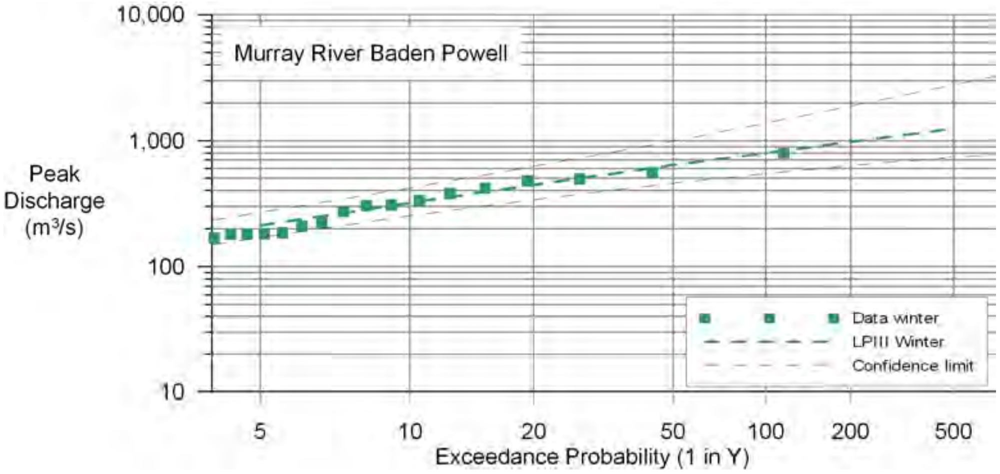
Summer (Oct-Mar)				Winter (Apr-Sept)			Annual (Apr-Mar)		
			winter recession						
1987	10	0.0	Censored as part of winter recession	1987	8	97.6	1987	8	97.6
1988	10	36.6		1988	7	181.8	1988	7	181.8
1989	2	61.2		1989	8	48.3	1989	2	61.2
1990	10	0.0	Censored as part of winter recession	1990	7	105.7	1990	7	105.7
1991	10	0.0	Censored as part of winter recession	1991	8	166.0	1991	8	166.0
1992	3	0.0	Censored as part of winter recession	1992	6	104.0	1992	6	104.0
1993	10	8.6		1993	8	48.5	1993	8	48.5
1994	10	0.0	Censored as part of winter recession	1994	8	119.4	1994	8	119.4
1995	10	30.6		1995	7	129.2	1995	7	129.2
1996	10	29.0		1996	8	306.1	1996	8	306.1
1997	10	0.0	Censored as part of winter recession	1997	8	32.27	1997	8	32.3
1998	10	16.3		1998	9	155.427	1998	9	155.4
1999	10	33.2		1999	8	64.152	1999	8	64.2
2000	10	0.0	Censored as part of winter recession	2000	7	116.454	2000	7	116.5
2001	10	0.0	Censored as part of winter recession	2001	8	23.867	2001	8	23.9
2002	10	0.0	Censored as part of winter recession	2002	9	49.438	2002	9	49.4
2003	10	16.0		2003	7	72.896	2003	7	72.9
2004	10	0.0	Censored as part of	2004	8	71.592	2004	8	71.6

Summer (Oct-Mar)				Winter (Apr-Sept)				Annual (Apr-Mar)			
			winter recession								
2005	10	18.1		2005	8	106.462		2005	8	106.5	
2006	10	0.0	Censored as part of winter recession	2006	8	33.428		2006	8	33.4	
2007	10	21.1	2008 deleted partial year	2007	8	64.985		2007	8	65.0	2008 deleted partial year.
				2008	8	113.146					

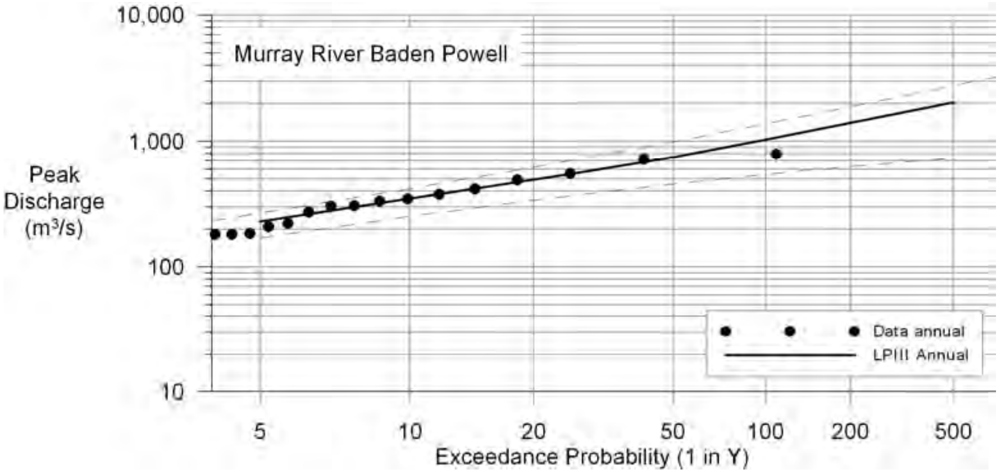
Flood frequency analysis, Murray River Baden Powell, summer



Flood frequency analysis, Murray River Baden Powell, winter



Flood frequency analysis, Murray River Baden Powell, annual



Government of **Western Australia**
Department of **Water**

Memorandum

TO: Senior Engineer
FROM: Engineer
CC:
DATE: 1 February 2010
RE: Baden Powel Flood Frequency Analysis
FILE NO:

Objective

To perform a flood frequency analysis based on annual maximum events at Baden Powell Gauging Station (614006)

Methodology

Daily flows were extracted in megalitres per day (ML/d) using Hydstra, for the period 1/1/1952 to 31/12/2008 (57 years).

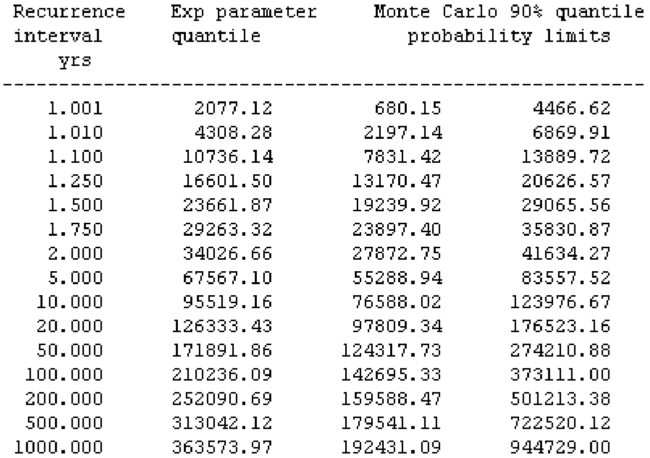
In order to estimate event flows, five day totals were calculated across the entire period and the maximum event total was extracted for each year.

The annual maximum event totals were entered into FLIKE and a flood frequency analysis was performed using a Log Pearson III (LPIII) distribution.

It should be noted that 24 of the 57 years analysed had periods of missing data. Of these years, half had missing data in winter. As such, the results of the flood frequency analysis are likely to be an underestimate.

Results

The table below shows the results of the analysis with all volumes shown in ML. The estimated 100 year ARI event total is approximately 210,000 ML (or 210 GL). The graph on the following page shows the LPIII distribution that was fit to the Baden Powell data.



```
 Recurrence    Exp parameter     Monte Carlo 90% quantile
 interval      quantile             probability limits
     yrs
----------------------------------------------------------
     1.001        2077.12         680.15        4466.62
     1.010        4308.28        2197.14        6869.91
     1.100       10736.14        7831.42       13889.72
     1.250       16601.50       13170.47       20626.57
     1.500       23661.87       19239.92       29065.56
     1.750       29263.32       23897.40       35830.87
     2.000       34026.66       27872.75       41634.27
     5.000       67567.10       55288.94       83557.52
    10.000       95519.16       76588.02      123976.67
    20.000      126333.43       97809.34      176523.16
    50.000      171891.86      124317.73      274210.88
   100.000      210236.09      142695.33      373111.00
   200.000      252090.69      159588.47      501213.38
   500.000      313042.12      179541.11      722520.12
  1000.000      363573.97      192431.09      944729.00
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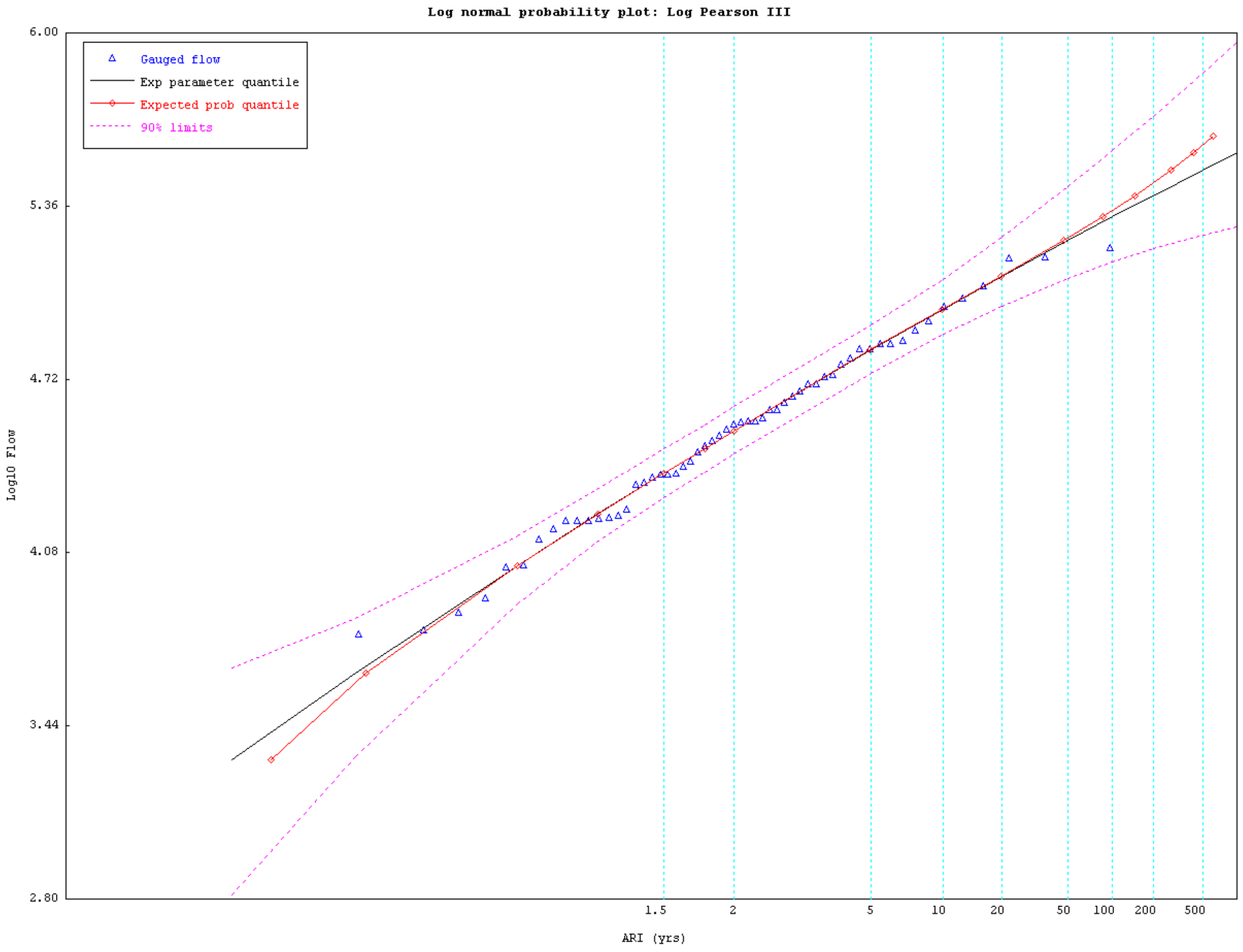


Historical data for Baden Powell dates back to 1940, with1945 estimated to have had the largest event on record. The table below shows the results obtained when this information is incorporated into the flood frequency analysis using the censored flow tool in FLIKE. The 100 year ARI event total is estimated to be approximately 240,000 ML (or 240 GL).

```
 Recurrence     Exp parameter     Monte Carlo 90% quantile
 interval       quantile             probability limits
     yrs
-------------------------------------------------------
     1.001        2279.81        766.34       4703.08
     1.010        4489.94       2321.53       7017.18
     1.100       10772.82       7884.06      13821.75
     1.250       16549.87      13111.11      20564.18
     1.500       23603.51      19144.72      29073.42
     1.750       29280.95      23873.57      36015.08
     2.000       34164.66      27903.14      41972.64
     5.000       69882.99      57127.16      86777.38
    10.000      101216.30      80945.35     131877.94
    20.000      137180.82     105447.17     194937.97
    50.000      192765.62     136999.70     311265.62
   100.000      241549.46     160125.86     435392.31
   200.000      296697.23     181767.25     603010.81
   500.000      380259.14     208770.50     901309.88
  1000.000      452242.70     228141.81    1212399.88
```

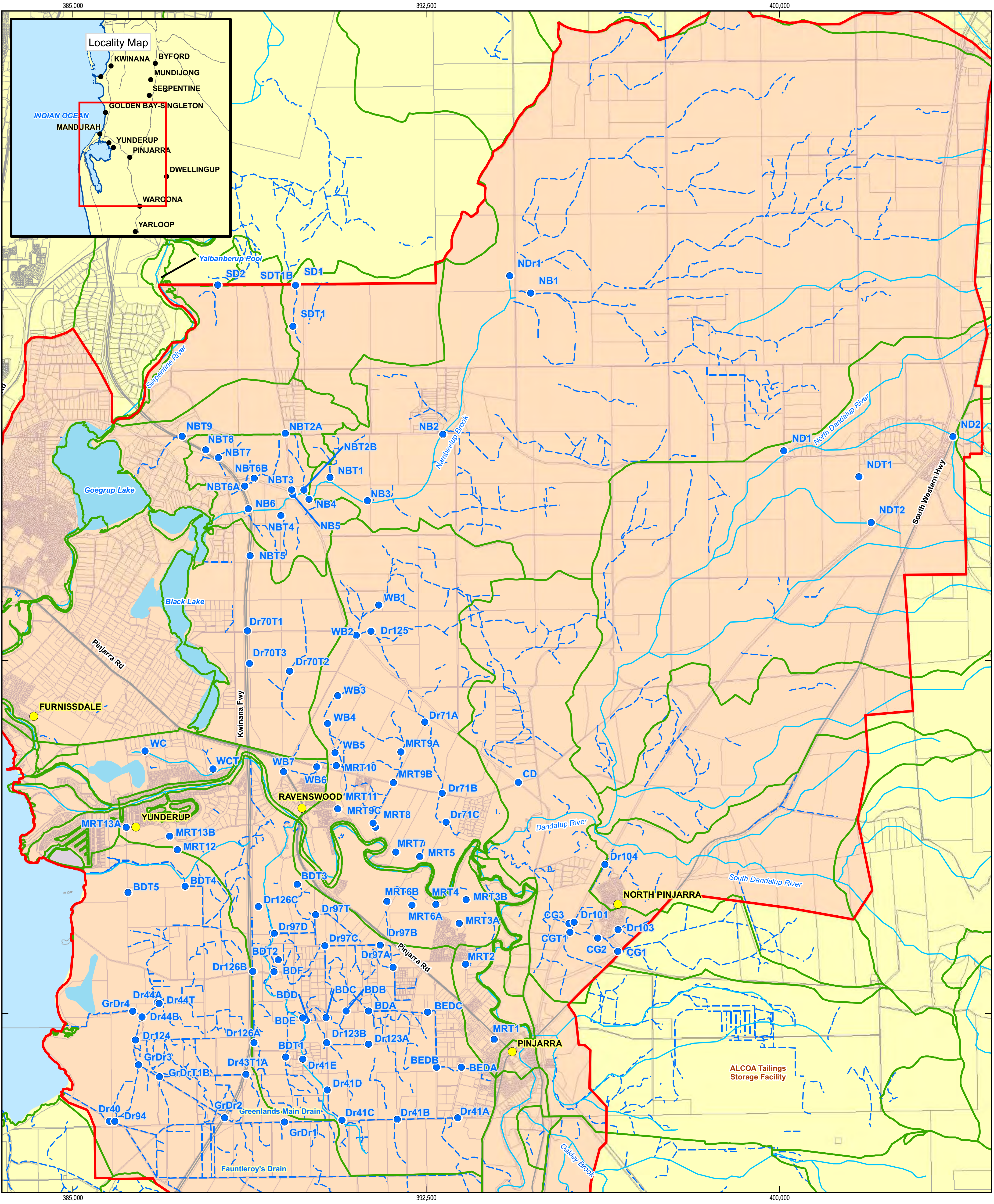
614006 Murray River Baden Powell

Year	Annual maximum volume (m^3 over 5 days)
1960	59,889
1961	23,552
1962	14,742
1963	116,926
1964	161,724
1965	37,822
1966	63,253
1967	86,873
1968	36,628
1969	7,238
1970	53,943
1971	15,845
1972	23,464
1973	47,591
1974	104,622
1975	43,340
1976	16,462
1977	15,781
1978	71,462
1979	6,224
1980	6,011
1981	45,440
1982	71,630
1983	79,717
1984	16,076
1985	25,158
1986	15,713
1987	26,198
1988	50,716
1989	13,516
1990	40,687
1991	54,827
1992	35,837
1993	17,448
1994	32,567
1995	36,805
1996	97,528
1997	10,587
1998	50,655
1999	21,484
2000	40,790
2001	8,144

2002	16,220
2003	23,406
2004	28,458
2005	29,898
2006	10,858
2007	21,753
2008	36,903

Appendix B

Peak discharge and water level extracts



LEGEND

● Extraction Points

● Locality

— Watercourses

- - Hydrography Linear (Major Drains)

— Roads

🟥 Flood Study Area

🟩 Subcatchments

☐ Cadastre

1:75,000 (at A3)

0.75 0.375 0 0.75 1.5 2.25 3

Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: Geocentric Datum of Australia (GDA)
Grid: Map Grid of Australia 1994, Zone 50

N

GHD

CLIENTS | PEOPLE | PERFORMANCE

Government of Western Australia
Department of Water

Department of Water
Murray Drainage and Water Management Plan
and Associated Studies

Job Number	61-2393701
Revision	0
Date	12 AUG 2010

Extract Locations

G:\61\2393706\GIS\mxds\612393701-G011.mxd

GHD House, 239 Adelaide Terrace Perth WA 6004 **T** 61 8 6222 8222 **F** 61 8 6222 8555 **E** permail@ghd.com.au **W** www.ghd.com.au

Data Source: Geoscience Australia: Lakes - 200605; MRWA: Roads - 20090409; DoW: Murray Drainage Study Area - 20090105; DoW: Linear Hydrography - 20090428; DoW: Subcatchments - 20090428; GA: Watercourses - 200905; GHD: Extraction Points - 20100303; Landgate: Cadastre - 200806. Created by: croach2, mludovico, wdavis, xntan, KDIRALU

ID	Catchment	Easting (m)	Northing (m)	Peak discharge (m3/s) for AEP (1 in Y)			Peak water level (m AHD) for Aep (1 in Y)			Comment*
				5	10	100	5	10	100	
BDA	Buchanan's Drain	391,279	6,390,056	0.92	0.97	1.28	6.80	6.80	6.82	BD detailed model
BDB	Buchanan's Drain	390,807	6,390,055	0.90	1.10	1.90	6.01	6.06	6.15	BD detailed model
BDC	Buchanan's Drain	390,378	6,389,916	1.11	1.42	2.30	5.37	5.40	5.46	BD detailed model
BDD	Buchanan's Drain	389,959	6,389,859	1.50	1.72	2.93	4.35	4.35	4.36	BD detailed model
BDE	Buchanan's Drain	389,883	6,389,911	3.31	3.77	6.89	4.00	4.03	4.19	BD detailed model
BDF	Buchanan's Drain	389,275	6,390,886	3.27	3.59	7.09	2.96	2.96	3.04	BD detailed model
BDT1	Buchanan's Drain Tributary	389,520	6,389,080	0.18	0.24	0.53	5.72	5.73	5.75	BD detailed model
BDT4	Buchanan's Drain Tributary	387,389	6,392,707	0.18	0.21	0.39	3.77	3.77	3.77	BD detailed model
BDT5	Buchanan's Drain Tributary	386,174	6,392,569	0.81	0.99	1.43	2.67	2.72	3.04	BD detailed model
BEDA	Buchanan's East Drain	393,247	6,388,862	0.04	0.04	0.07	9.08	9.20	9.43	BD detailed model
BEDB	Buchanan's East Drain	392,723	6,388,865	0.21	0.25	0.53	9.24	9.23	9.24	BD detailed model
BEDC	Buchanan's East Drain	392,533	6,390,034	0.20	0.24	0.52	8.41	8.40	8.41	BD detailed model
CD	Corio Drain	394,463	6,394,908	0.01	1.66	3.49	6.77	6.79	6.86	
CG1	Cornish Gully	396,570	6,391,323	-	-	-	11.13	11.13	11.14	ND, BW or I
CG2	Cornish Gully	396,144	6,391,604	5.95	0.00	13.74	dry	10.13	10.13	
CG3	Cornish Gully	395,538	6,391,912	5.93	7.88	13.43	7.44	7.47	7.55	
CGT1	Cornish Gully Tributary	395,554	6,391,729	-	-	-	dry	8.95	8.95	ND, BW or I
Dr101	Drain 101	395,641	6,391,945	-	-	-	8.66	8.66	8.68	ND, BW or I
Dr103	Drain 103	396,575	6,391,784	-	-	-	dry	11.85	11.85	ND, BW or I
Dr104	Drain 104	396,299	6,393,171	-	-	-	9.80	9.80	9.80	ND, BW or I
Dr123A	Drain 123	391,281	6,389,350	0.25	0.27	0.44	7.66	7.66	7.66	BD detailed model
Dr123B	Drain 123	390,389	6,389,383	0.38	0.41	0.83	5.89	5.91	5.93	BD detailed model
Dr124	Drain 124	386,336	6,389,437	0.21	0.30	0.59	2.01	2.05	2.23	
Dr125	Drain 125	391,330	6,398,124	0.40	0.51	0.83	8.18	8.09	8.23	WB couple
Dr126A	Drain 126	388,845	6,389,384	0.40	0.41	0.54	4.81	4.82	4.90	BD detailed model
Dr126B	Drain 126	388,820	6,390,897	0.56	0.66	0.95	3.42	3.42	3.43	BD detailed model
Dr126C	Drain 126	388,945	6,392,272	0.53	0.59	0.97	2.82	2.84	3.04	BD detailed model

ID	Catchment	Easting (m)	Northing (m)	Peak discharge (m3/s) for AEP (1 in Y)			Peak water level (m AHD) for Aep (1 in Y)			Comment*
				5	10	100	5	10	100	
Dr40	Drain 40	385,783	6,387,714	-	-	-	2.07	dry	2.24	ND, BW or I
Dr41A	Drain 41	393,173	6,387,789	0.56	0.50	0.69	10.43	10.43	10.45	BD detailed model
Dr41B	Drain 41	391,888	6,387,759	0.58	0.50	0.65	9.21	9.20	9.21	BD detailed model
Dr41C	Drain 41	390,715	6,387,738	0.85	0.94	1.86	7.47	7.48	7.49	BD detailed model
Dr41D	Drain 41	390,406	6,388,378	0.99	1.11	2.56	6.43	6.43	6.43	BD detailed model
Dr41E	Drain 41	389,884	6,389,031	1.04	1.17	2.52	5.57	5.58	5.64	BD detailed model
Dr43T1A	Drain 43 Tributary	388,676	6,388,707	0.22	0.31	0.91	4.49	4.56	4.70	
Dr43T1B	Drain 43 Tributary	386,844	6,388,661	0.02	0.03	0.29	dry	dry	2.26	
Dr44A	Drain 44	386,797	6,390,235	-	-	-	1.98	2.04	2.22	ND, BW or I
Dr44B	Drain 44	386,471	6,389,927	-	-	-	1.98	2.04	2.22	ND, BW or I
Dr44T	Drain 44 Tributary	386,832	6,390,214	-	-	-	dry	2.04	2.22	ND, BW or I
Dr70T1	Drain 70 Tributary	388,705	6,398,127	0.37	0.38	0.70	3.50	3.50	3.50	
Dr70T2	Drain 70 Tributary	389,604	6,397,269	0.10	0.28	1.07	5.71	5.73	5.77	
Dr70T3	Drain 70 Tributary	388,751	6,397,436	0.48	0.55	1.66	3.33	3.33	3.38	
Dr71A	Drain 71	392,478	6,396,192	0.09	0.32	0.98	8.43	8.47	8.52	
Dr71B	Drain 71	392,845	6,394,680	0.09	0.36	1.41	8.16	8.16	8.17	
Dr71C	Drain 71	392,927	6,394,063	0.14	0.40	1.54	6.97	6.99	7.06	
Dr94	Drain 94	385,894	6,387,712	0.08	0.11	0.15	2.10	2.11	2.24	
Dr97A	Drain 97	391,801	6,390,983	0.07	0.07	0.27	7.41	7.41	7.43	BD detailed model
Dr97B	Drain 97	391,528	6,391,454	0.18	0.19	0.52	6.14	6.14	6.45	BD detailed model
Dr97C	Drain 97	390,356	6,391,442	0.19	0.22	0.69	4.67	4.68	4.68	BD detailed model
Dr97D	Drain 97	389,282	6,391,702	0.11	0.15	0.41	2.80	2.80	3.04	BD detailed model
Dr97T	Drain 97 Tributary	390,156	6,392,100	0.01	0.01	0.10	4.12	4.12	4.12	BD detailed model
GrDr1	Greenlands Drain	389,497	6,387,698	0.14	0.19	0.32	6.08	6.08	6.09	
GrDr2	Greenlands Drain	388,228	6,387,789	0.62	0.95	1.83	dry	dry	dry	
GrDr3	Greenlands Drain	386,397	6,388,911	0.68	0.98	1.84	2.02	2.07	2.24	
GrDr4	Greenlands Drain	386,275	6,390,047	-	-	-	2.25	2.25	2.25	ND, BW or I
MRT1	Murray River Tributary	393,946	6,389,453	0.12	0.24	0.84	7.94	7.96	8.20	
MRT10	Murray River Tributary	390,597	6,395,263	-	-	-	7.17	7.17	7.17	ND, BW or I
MRT11	Murray River Tributary	390,627	6,394,349	0.06	0.07	0.14	5.87	5.87	5.89	

ID	Catchment	Easting (m)	Northing (m)	Peak discharge (m3/s) for AEP (1 in Y)			Peak water level (m AHD) for Aep (1 in Y)			Comment*
				5	10	100	5	10	100	
MRT12	Murray River Tributary	387,223	6,393,479	-	-	-	1.64	1.68	2.33	ND, BW or I
MRT13A	Murray River Tributary	386,137	6,393,954	-	-	-	1.48	1.52	2.14	ND, BW or I
MRT13B	Murray River Tributary	387,057	6,393,764	-	-	-	1.62	1.67	2.35	ND, BW or I
MRT2	Murray River Tributary	393,336	6,391,047	-	-	-	7.97	7.97	7.97	ND, BW or I
MRT3A	Murray River Tributary	393,204	6,391,916	0.36	0.29	0.51	8.49	8.47	8.49	
MRT3B	Murray River Tributary	393,352	6,392,418	-	-	-	dry	6.89	6.89	ND, BW or I
MRT4	Murray River Tributary	392,707	6,392,317	-	-	-	6.40	6.40	6.41	ND, BW or I
MRT5	Murray River Tributary	392,369	6,393,334	0.02	0.04	0.09	6.71	6.71	6.71	
MRT6A	Murray River Tributary	392,199	6,392,301	0.06	0.05	0.10	5.77	5.77	5.77	
MRT6B	Murray River Tributary	391,666	6,392,383	0.13	0.22	0.33	5.11	5.12	5.13	
MRT7	Murray River Tributary	391,858	6,393,429	-	-	-	dry	7.14	7.14	ND, BW or I
MRT8	Murray River Tributary	391,427	6,393,954	0.21	0.26	0.42	3.32	3.37	4.51	
MRT9A	Murray River Tributary	391,970	6,395,556	0.10	0.14	0.31	7.07	7.07	7.07	
MRT9B	Murray River Tributary	391,805	6,394,900	0.02	0.03	0.06	6.69	6.71	6.85	
MRT9C	Murray River Tributary	391,377	6,394,044	-	-	-	5.65	5.64	5.64	ND, BW or I
NB1	Nambeelup Brook	394,725	6,405,302	11.75	12.80	26.94	17.12	17.13	17.29	
NB2	Nambeelup Brook	392,853	6,402,304	15.49	17.86	36.69	12.05	12.08	12.29	
NB3	Nambeelup Brook	391,257	6,400,893	15.93	18.69	37.90	9.41	9.47	9.78	
NB4	Nambeelup Brook	390,019	6,400,926	16.26	19.54	38.85	6.48	6.54	6.77	
NB5	Nambeelup Brook	389,674	6,401,027	16.00	19.40	38.39	5.43	5.51	5.84	
NB6	Nambeelup Brook	388,729	6,400,721	16.88	20.82	40.08	2.64	2.72	3.04	
NBT1	Nambeelup Brook Tributary	390,464	6,401,386	0.11	0.14	0.20	8.65	8.66	8.66	
NBT2A	Nambeelup Brook Tributary	389,516	6,402,321	-	-	-	7.40	7.40	7.40	ND, BW or I
NBT2B	Nambeelup Brook Tributary	389,908	6,401,120	0.03	0.02	0.05	6.35	6.35	6.37	
NBT3	Nambeelup Brook Tributary	389,647	6,401,118	0.02	0.04	0.15	6.81	6.81	6.81	
NBT4	Nambeelup Brook Tributary	389,421	6,400,574	0.29	0.50	0.79	5.46	5.48	5.50	
NBT5	Nambeelup Brook Tributary	388,766	6,399,726	0.49	0.75	1.23	4.53	4.55	4.65	
NBT6A	Nambeelup Brook Tributary	388,857	6,401,367	0.11	0.14	0.28	5.45	5.45	5.45	
NBT6B	Nambeelup Brook Tributary	388,653	6,401,202	0.16	0.22	0.41	4.58	4.58	4.60	
NBT7	Nambeelup Brook Tributary	388,090	6,401,809	-	-	-	3.27	3.15	3.45	ND, BW or I

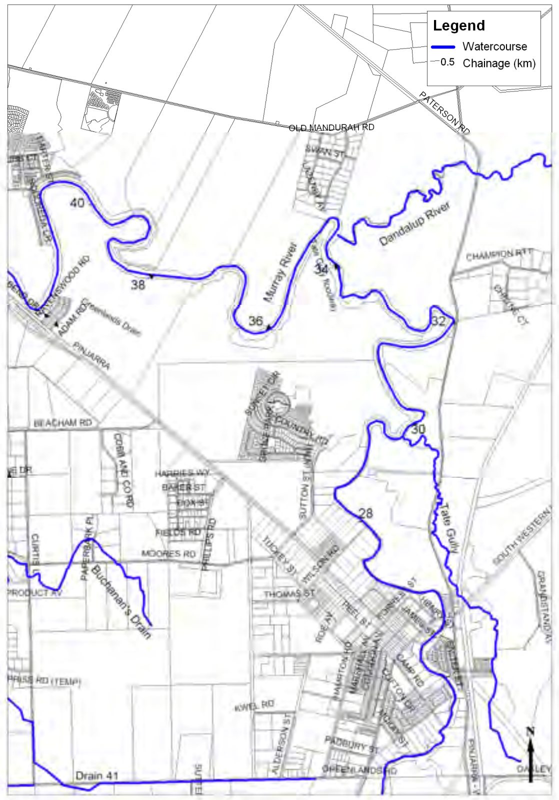
ID	Catchment	Easting (m)	Northing (m)	Peak discharge (m3/s) for AEP (1 in Y)			Peak water level (m AHD) for Aep (1 in Y)			Comment*
				5	10	100	5	10	100	
NBT8	Nambeelup Brook Tributary	387,827	6,401,971	0.39	0.47	0.71	3.27	3.15	3.45	
NBT9	Nambeelup Brook Tributary	387,327	6,402,254	-	-	-	3.67	3.63	3.80	ND, BW or I
ND1	North Dandalup	400,097	6,401,953	14.59	19.55	35.33	26.79	26.85	26.96	
ND2	North Dandalup	403,682	6,402,250				dry	dry	dry	ND, BW or I
NDr1	Nambeelup Drain	394,277	6,405,667	2.77	3.56	7.95	16.81	16.82	16.93	
NDT1	North Dandalup Tributary	401,691	6,401,402	0.38	0.60	1.40	30.98	30.98	30.98	
NDT2	North Dandalup Tributary	401,950	6,400,425	1.13	1.44	2.49	33.02	33.03	33.05	
SD1	Serpentine Drain	389,740	6,405,491	0.51	0.65	1.09	8.96	8.98	9.00	
SD2	Serpentine Drain	388,079	6,405,472	0.17	0.21	0.39	dry	2.17	2.39	
SDT1	Serpentine Drain Tributary	389,676	6,404,596	0.23	0.37	0.63	9.23	9.24	9.26	
SDT1B	Serpentine Drain Tributary	389,731	6,405,464	0.46	0.59	1.17	8.96	8.98	9.00	
WB1	Winter Brook	391,500	6,398,673	0.95	1.08	1.39	8.51	8.51	8.52	WB couple
WB2	Winter Brook	391,023	6,398,035	1.37	1.59	2.58	8.03	8.09	8.23	WB couple
WB3	Winter Brook	390,625	6,396,749	0.83	0.95	2.45	7.13	7.13	7.18	WB couple
WB4	Winter Brook	390,408	6,396,160	0.18	0.25	1.02	6.79	6.81	6.89	WB couple
WB5	Winter Brook	390,570	6,395,539	0.32	0.32	1.10	6.75	6.86	6.84	WB couple
WB6	Winter Brook	390,180	6,395,241	1.10	1.11	1.46	5.11	5.11	5.11	WB couple
WB7	Winter Brook	389,480	6,395,139	1.38	1.33	1.65	3.01	3.02	3.09	WB couple
WC	Wilgie Creek	386,539	6,395,577	-	-	-	1.27	1.42	1.95	ND, BW or I
WCT	Wilgie Creek Tributary	387,979	6,395,196	-	-	-	1.58	1.76	2.43	ND, BW or I

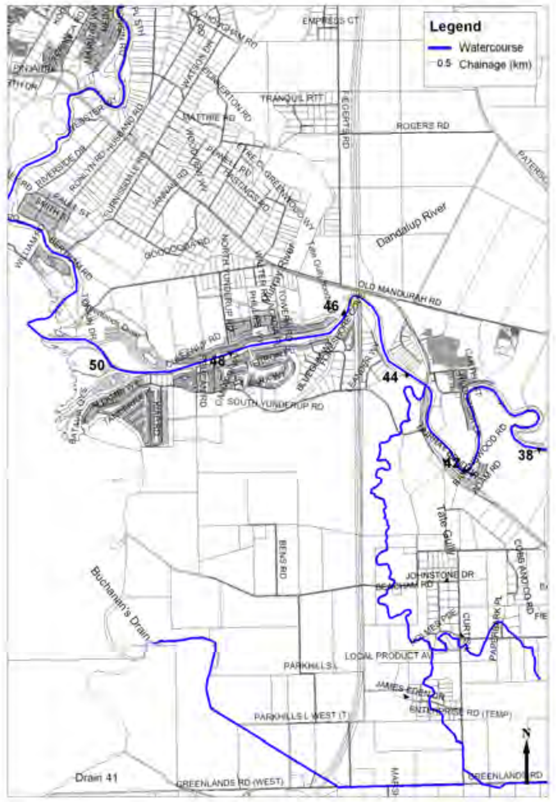
* ND = non-draining, BW = backwater, I = inundated

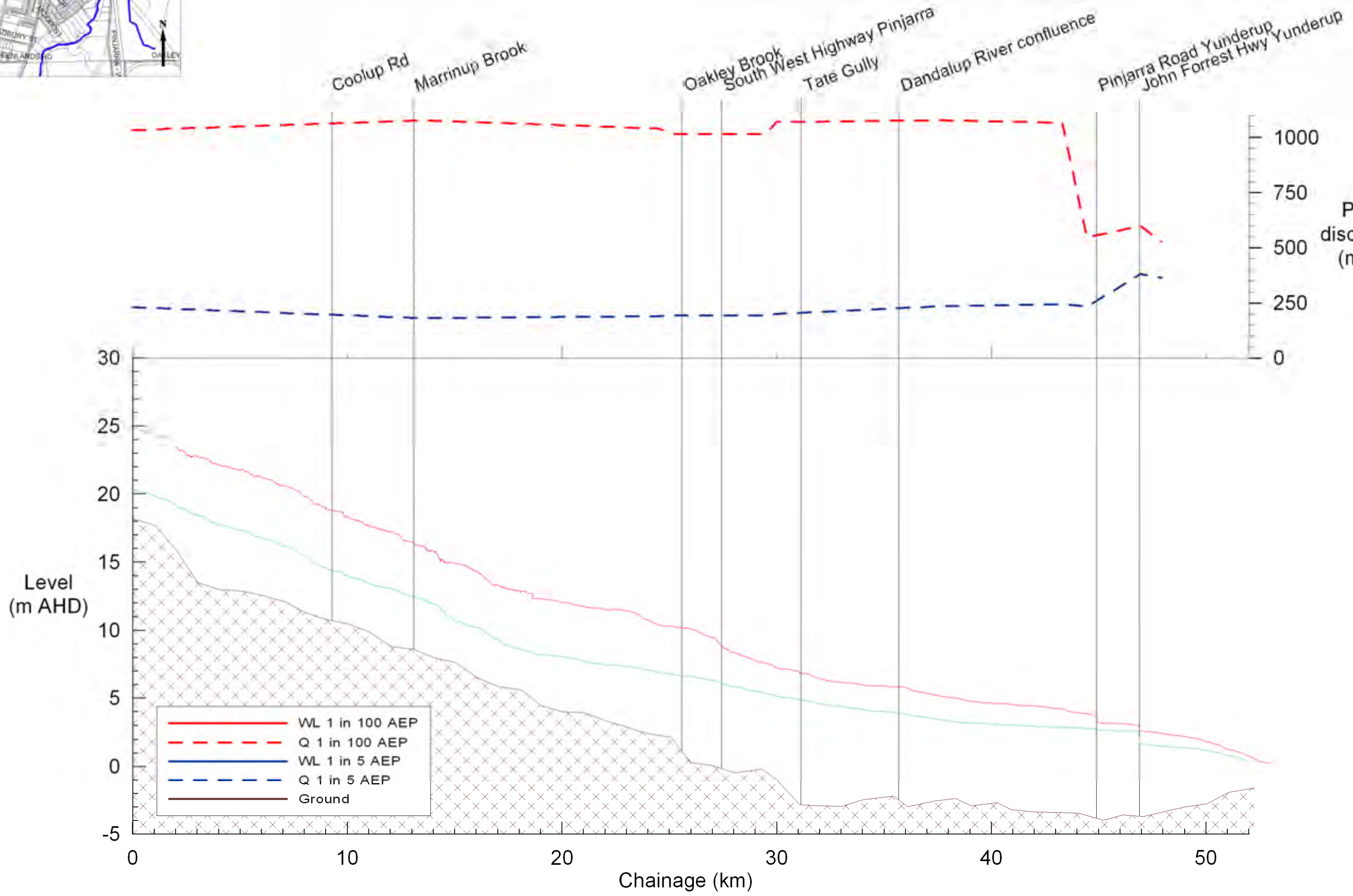
Appendix C

Long sections

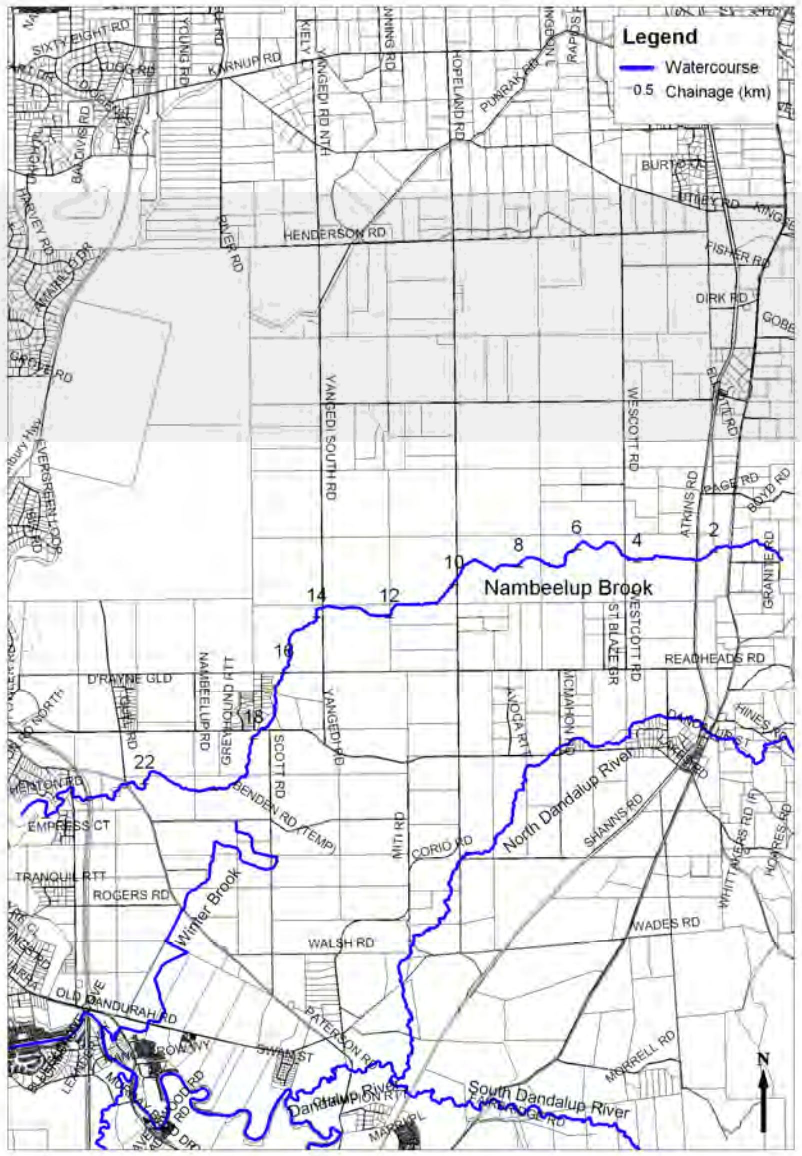
Murray River

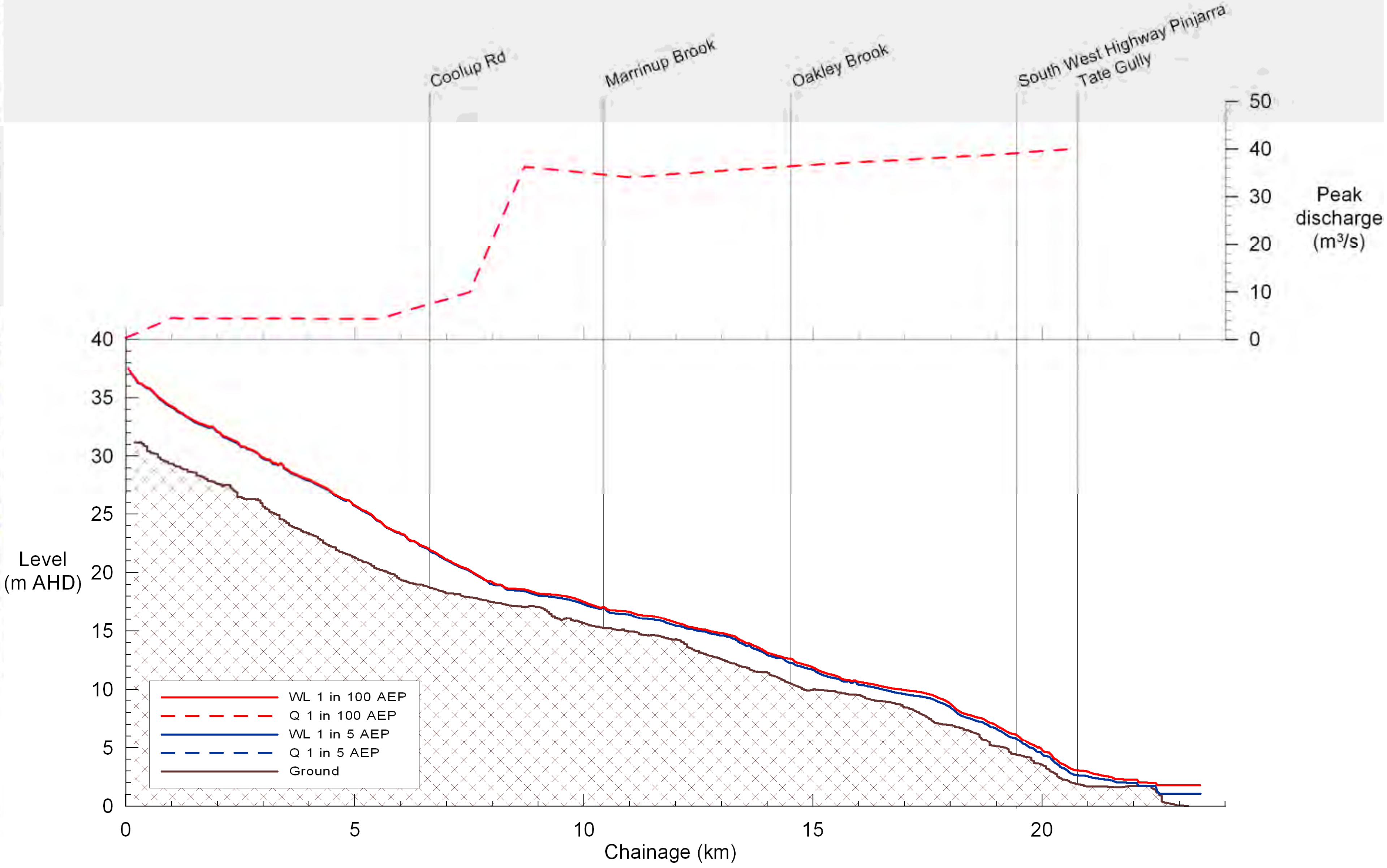




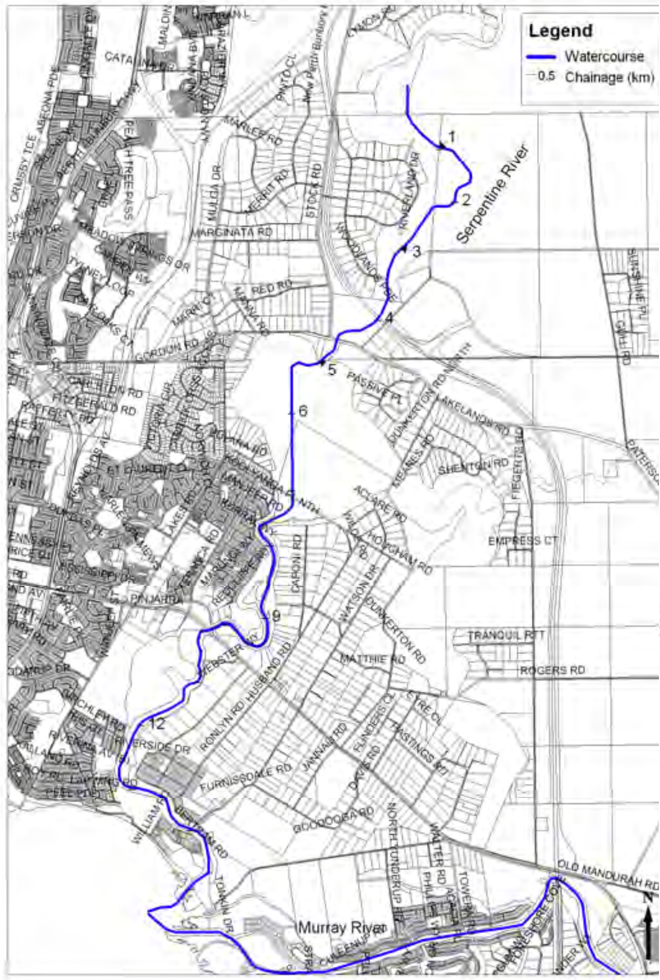


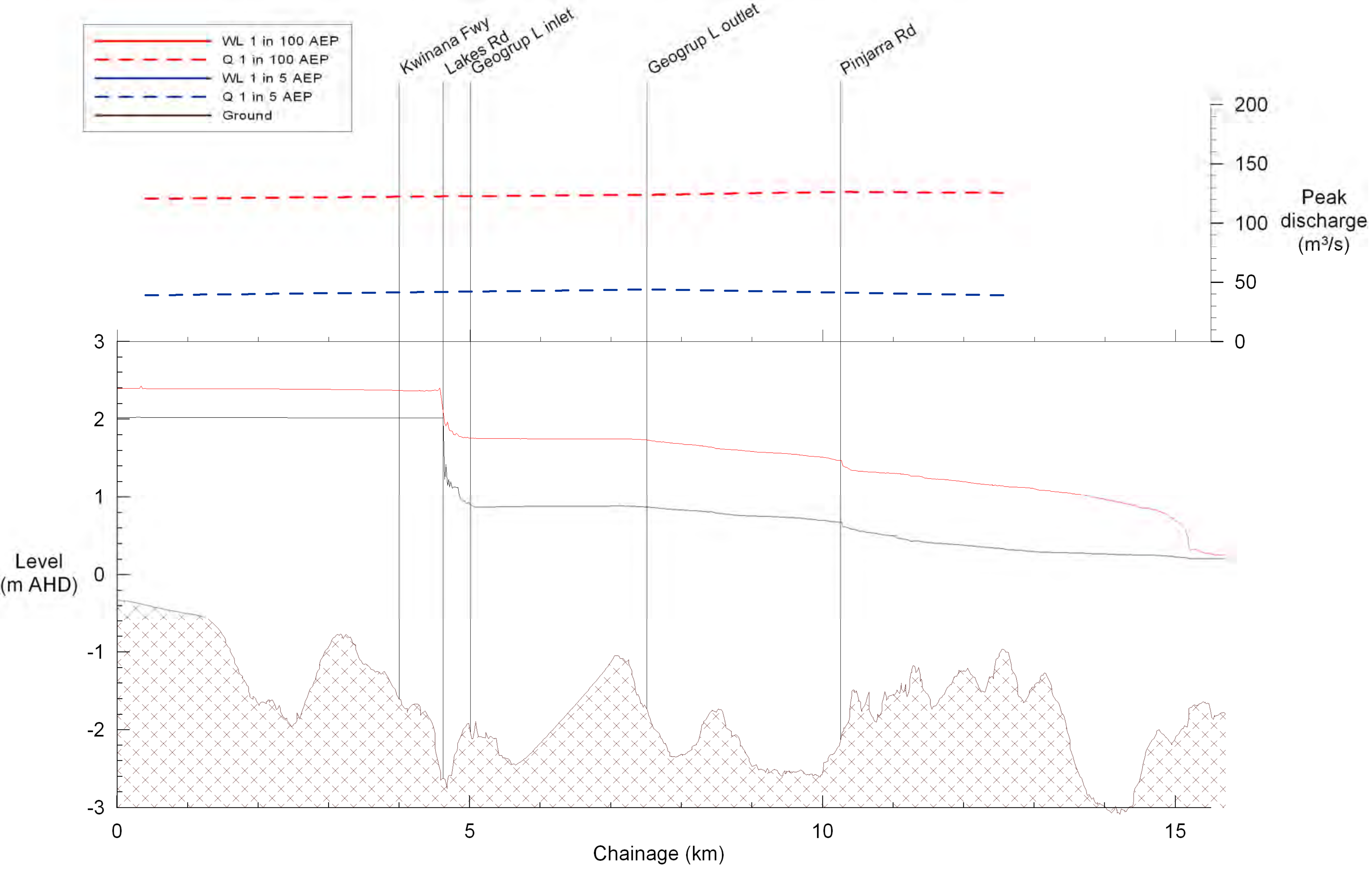
Nambeelup Brook



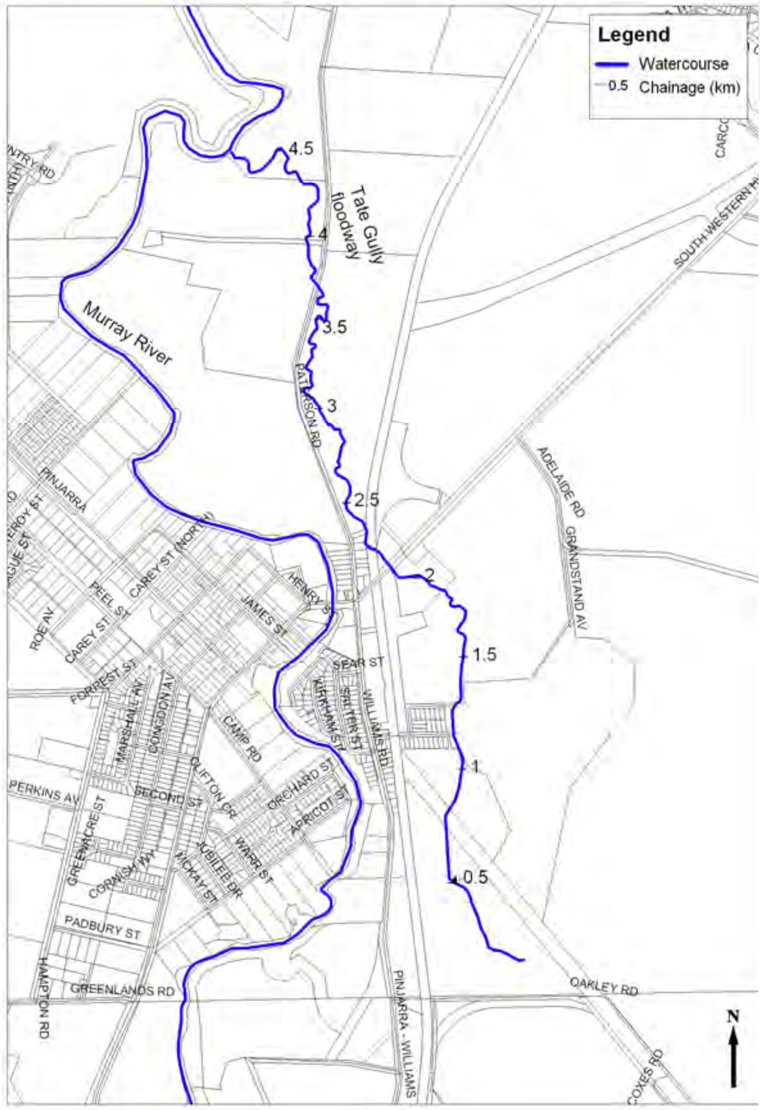


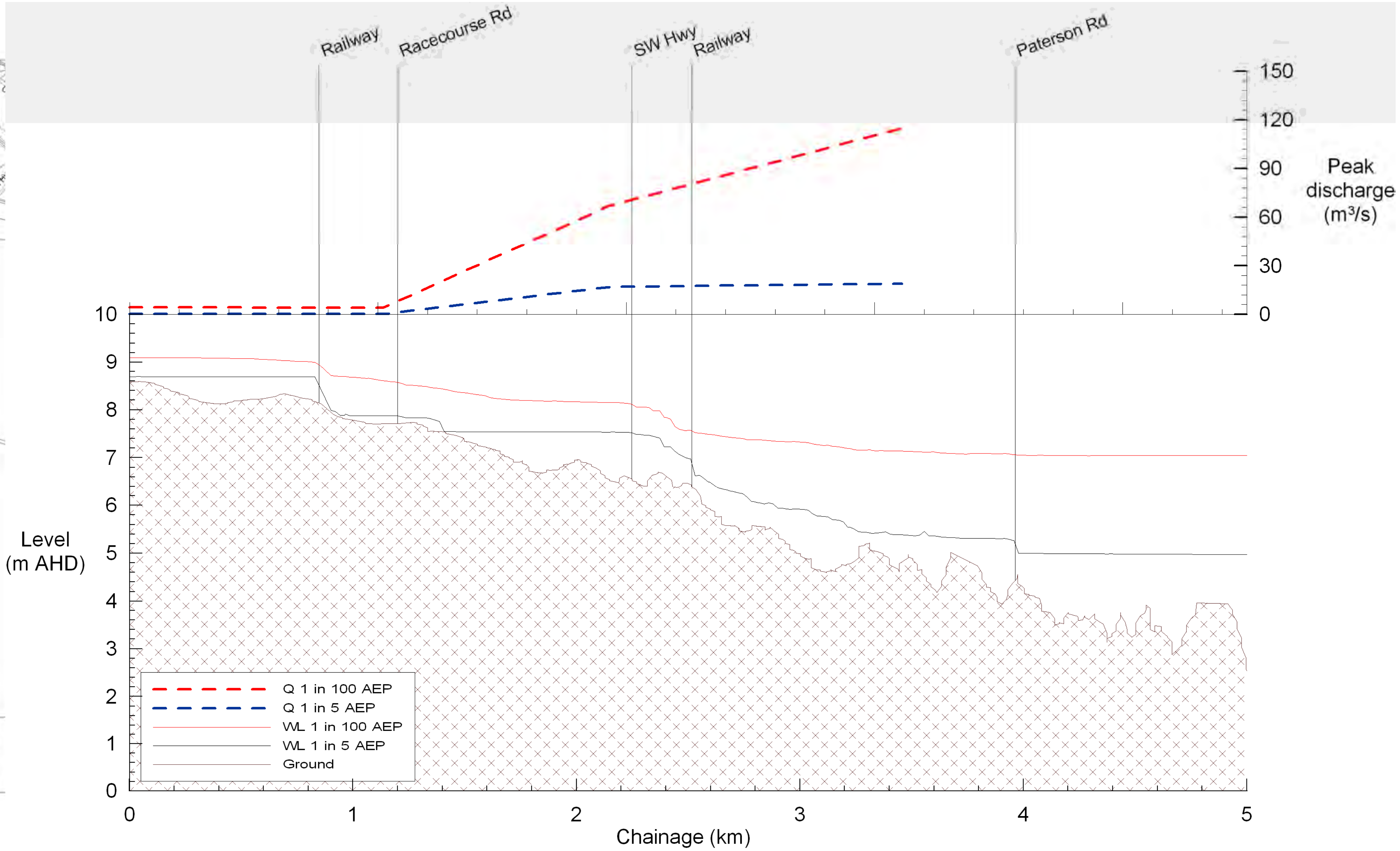
Serpentine River



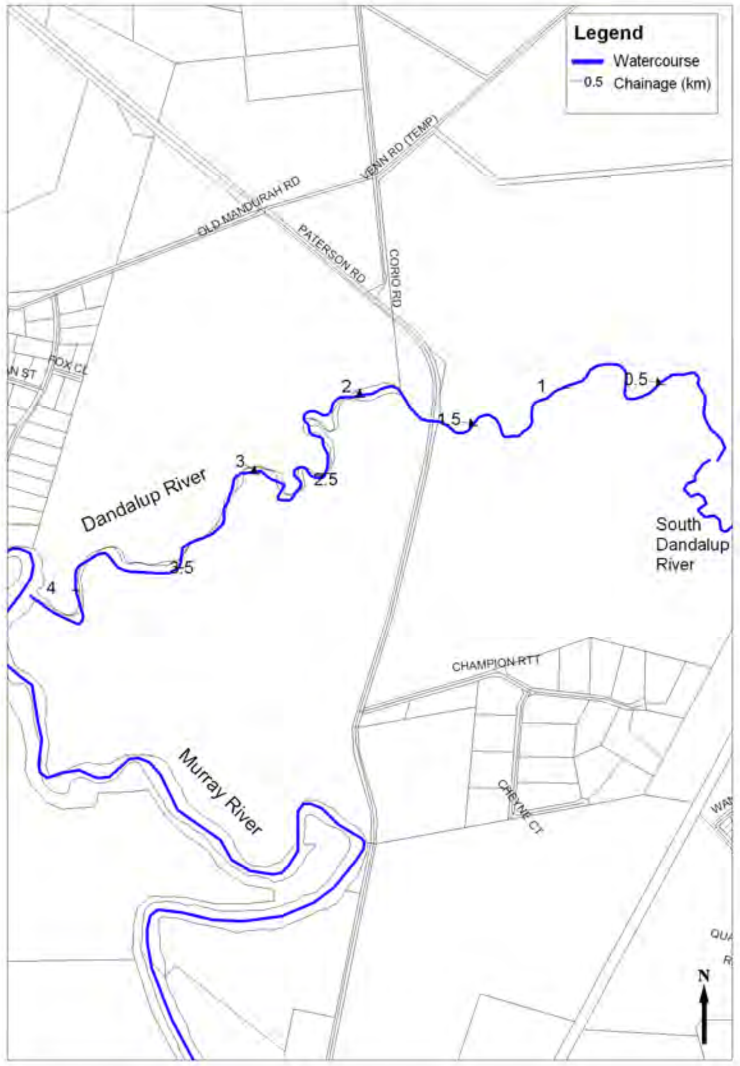


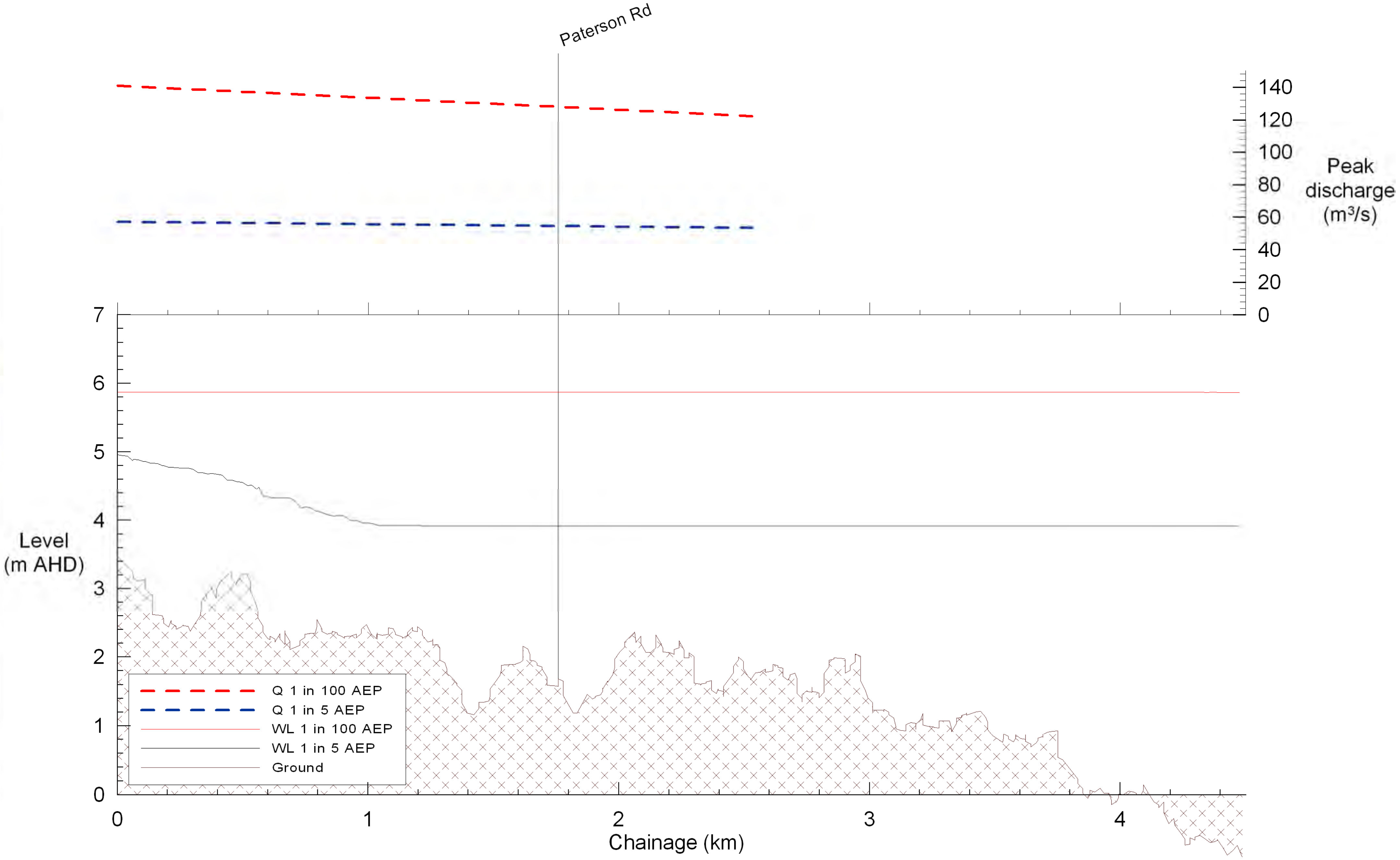
Tate Gully floodway



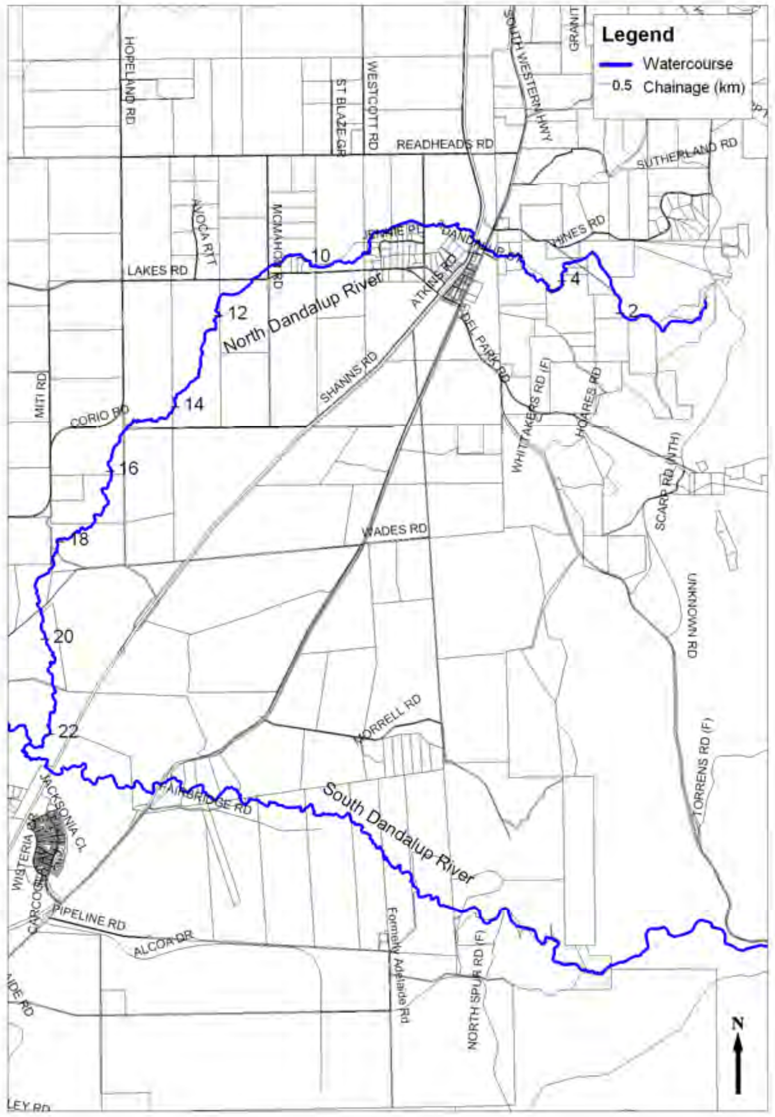


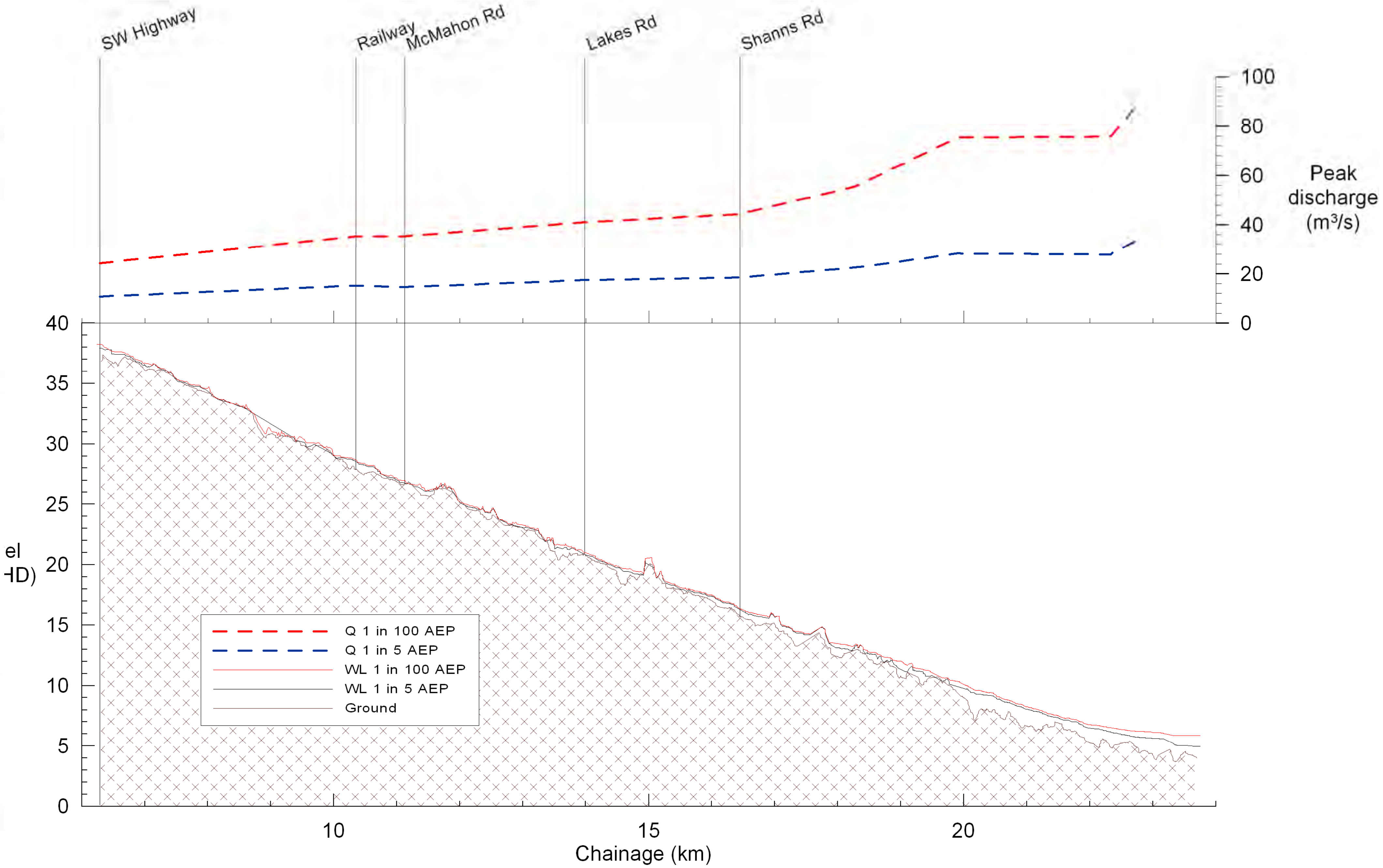
Dandalup River



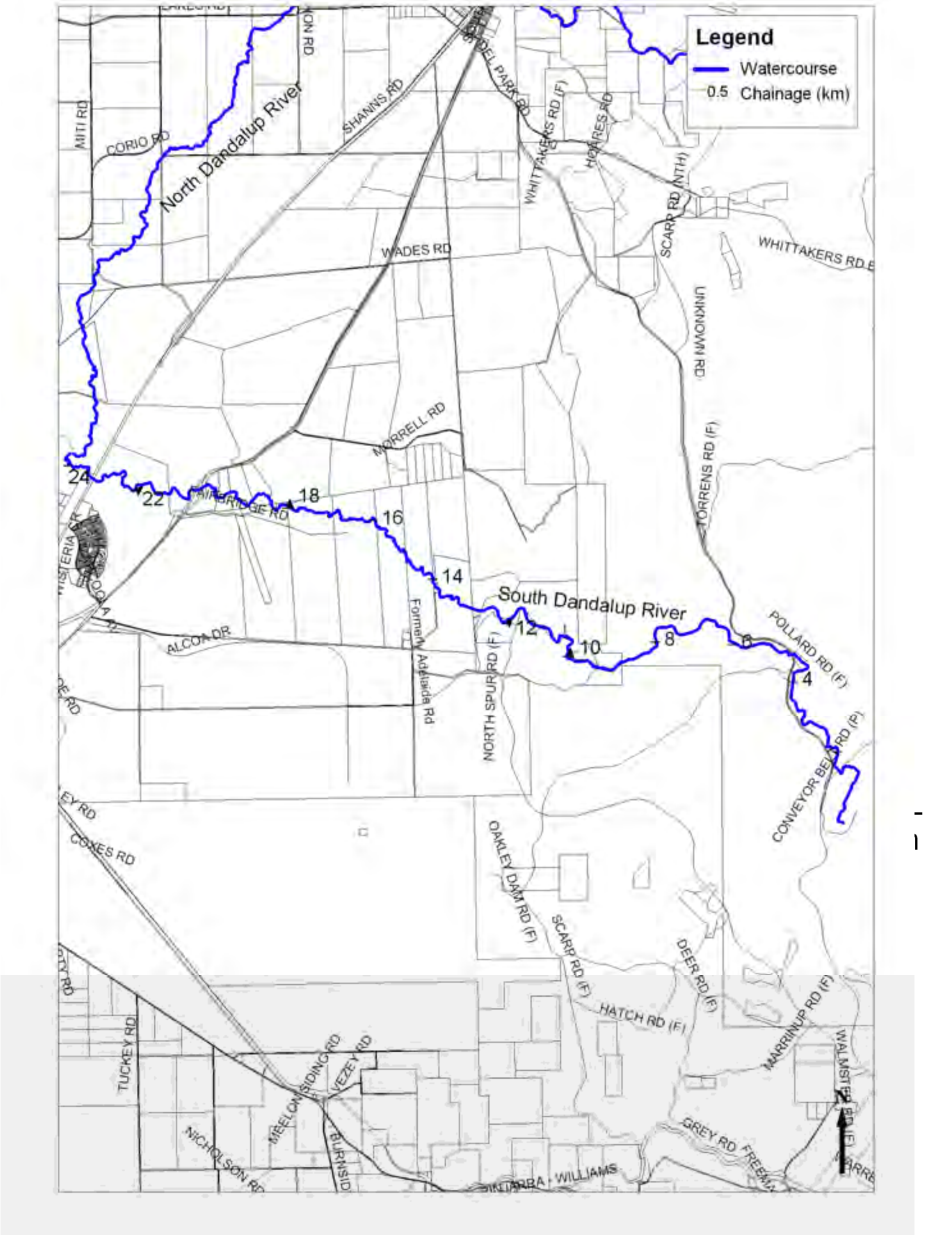


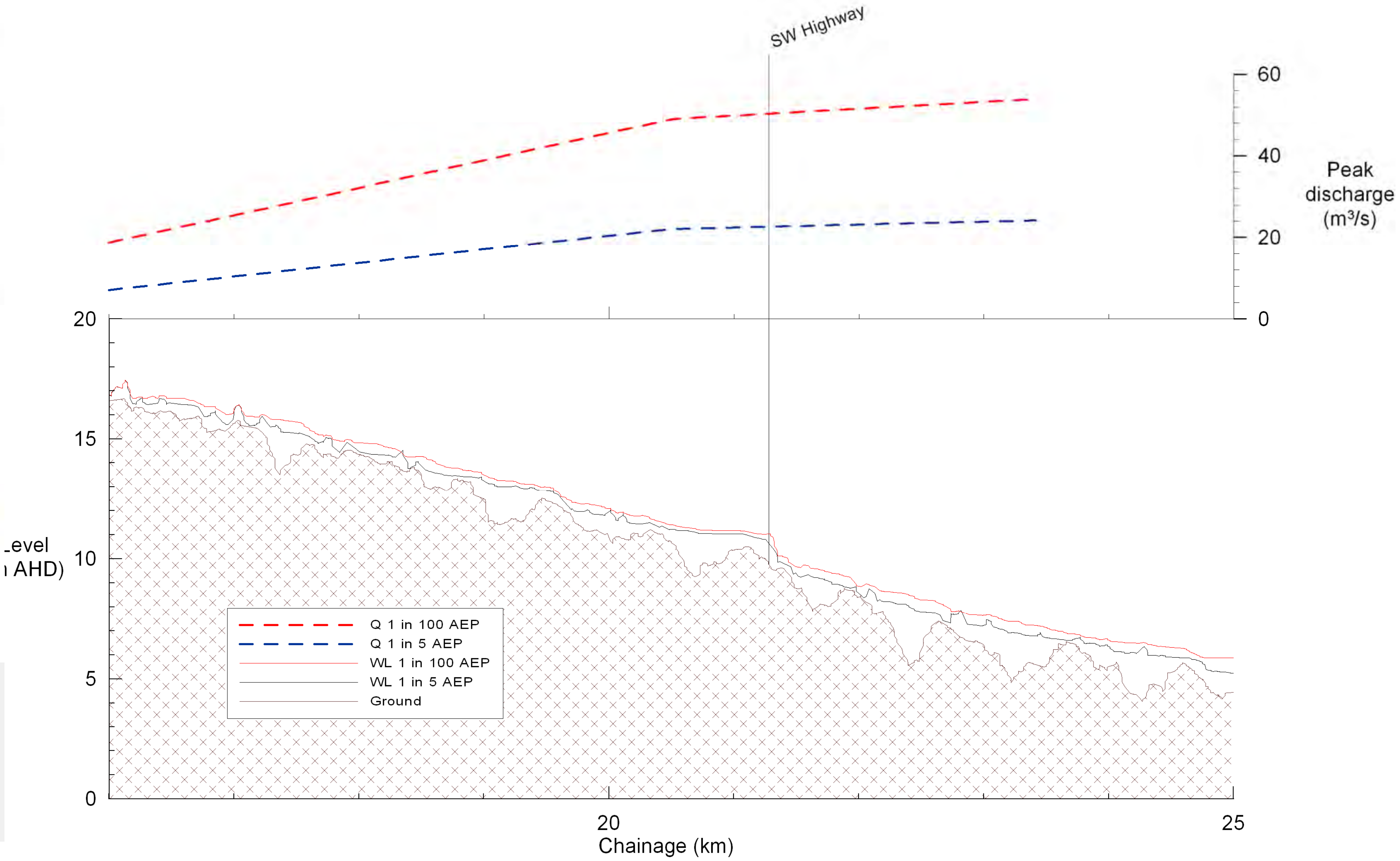
North Dandalup River



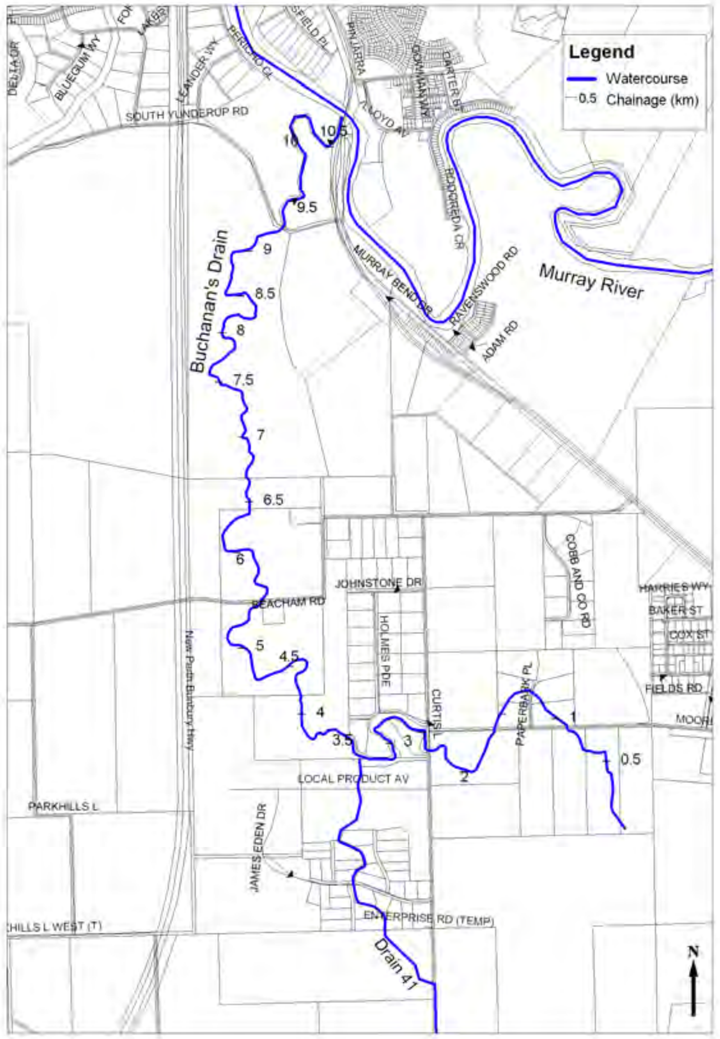


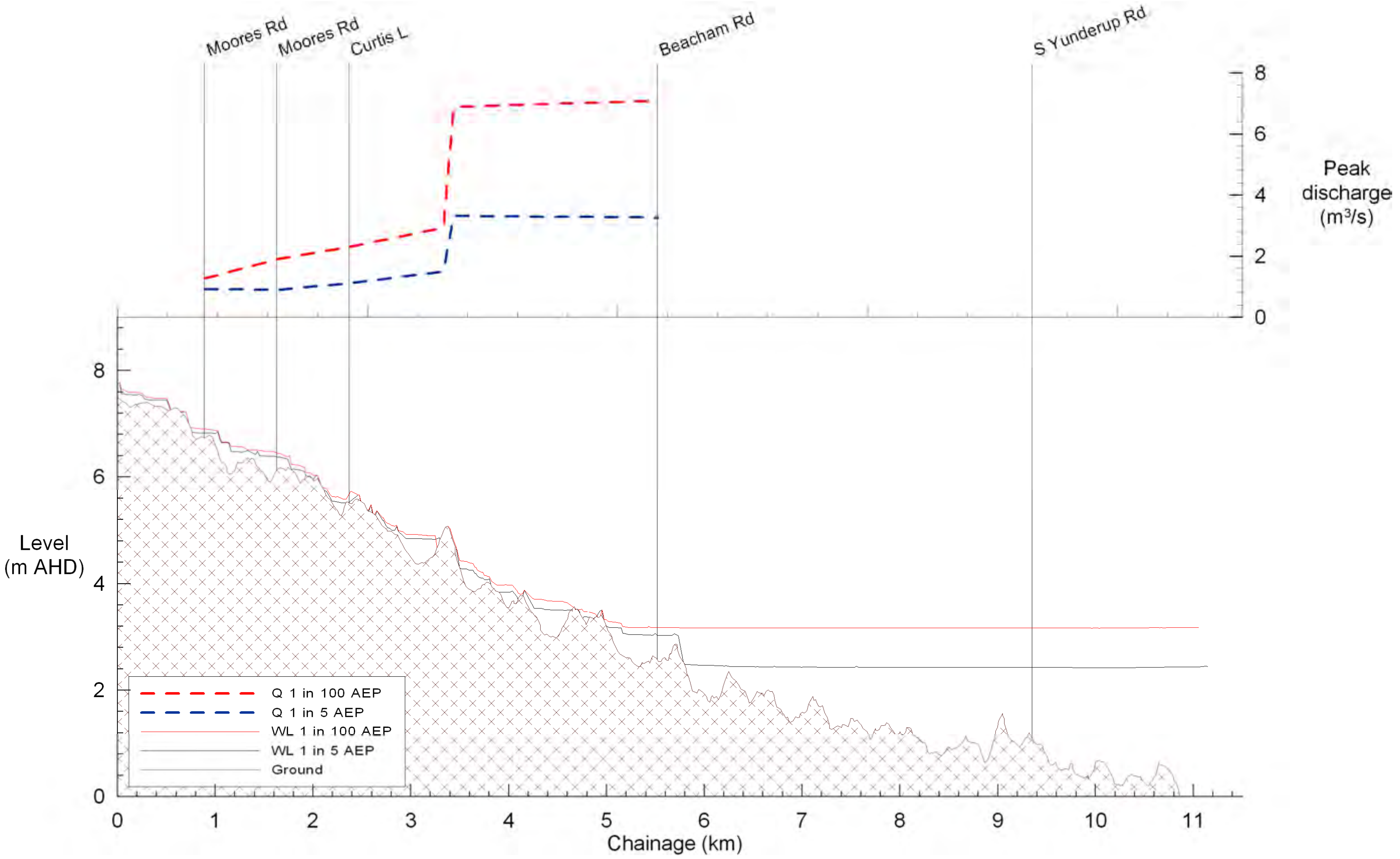
South Dandalup River



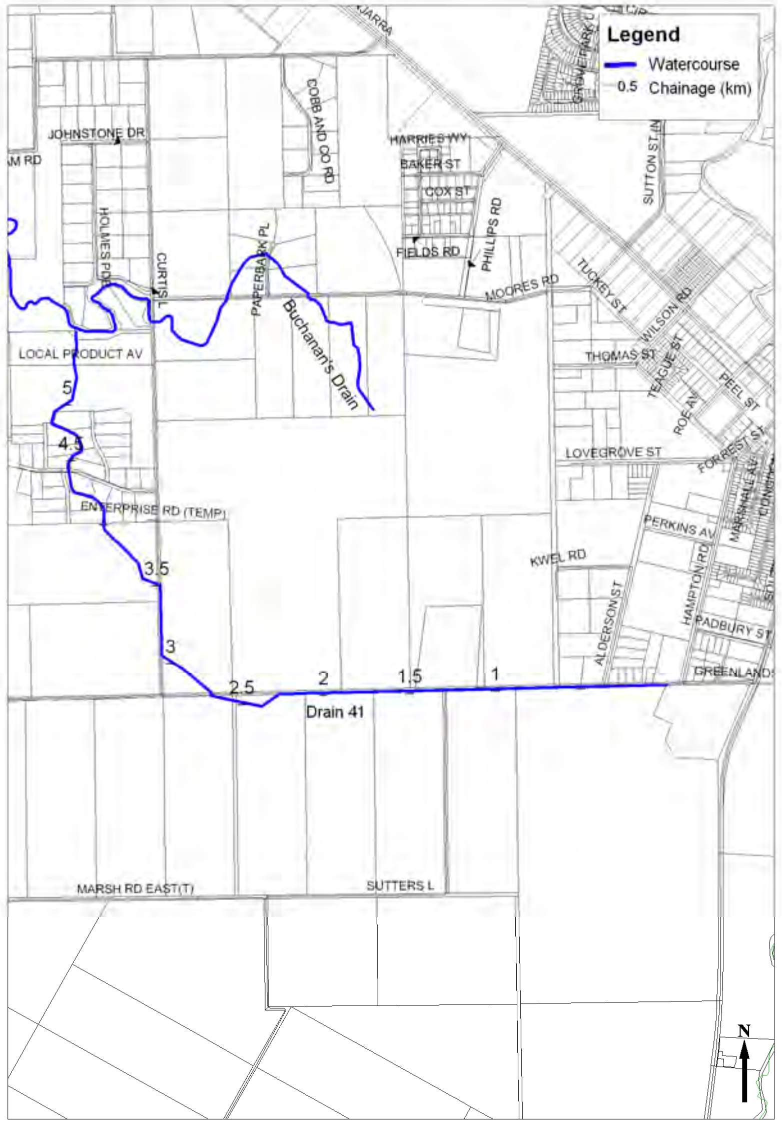


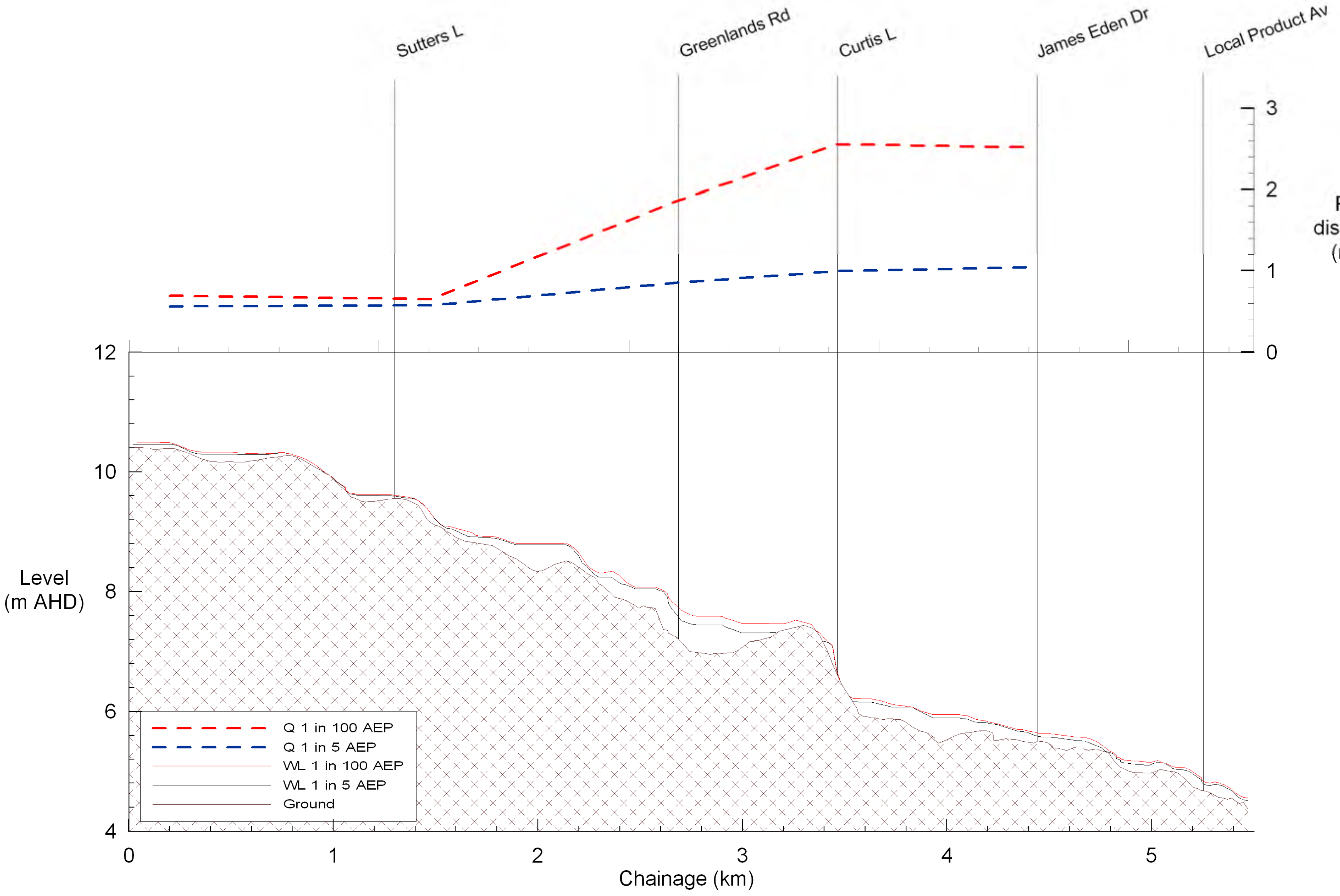
Buchanan's Drain



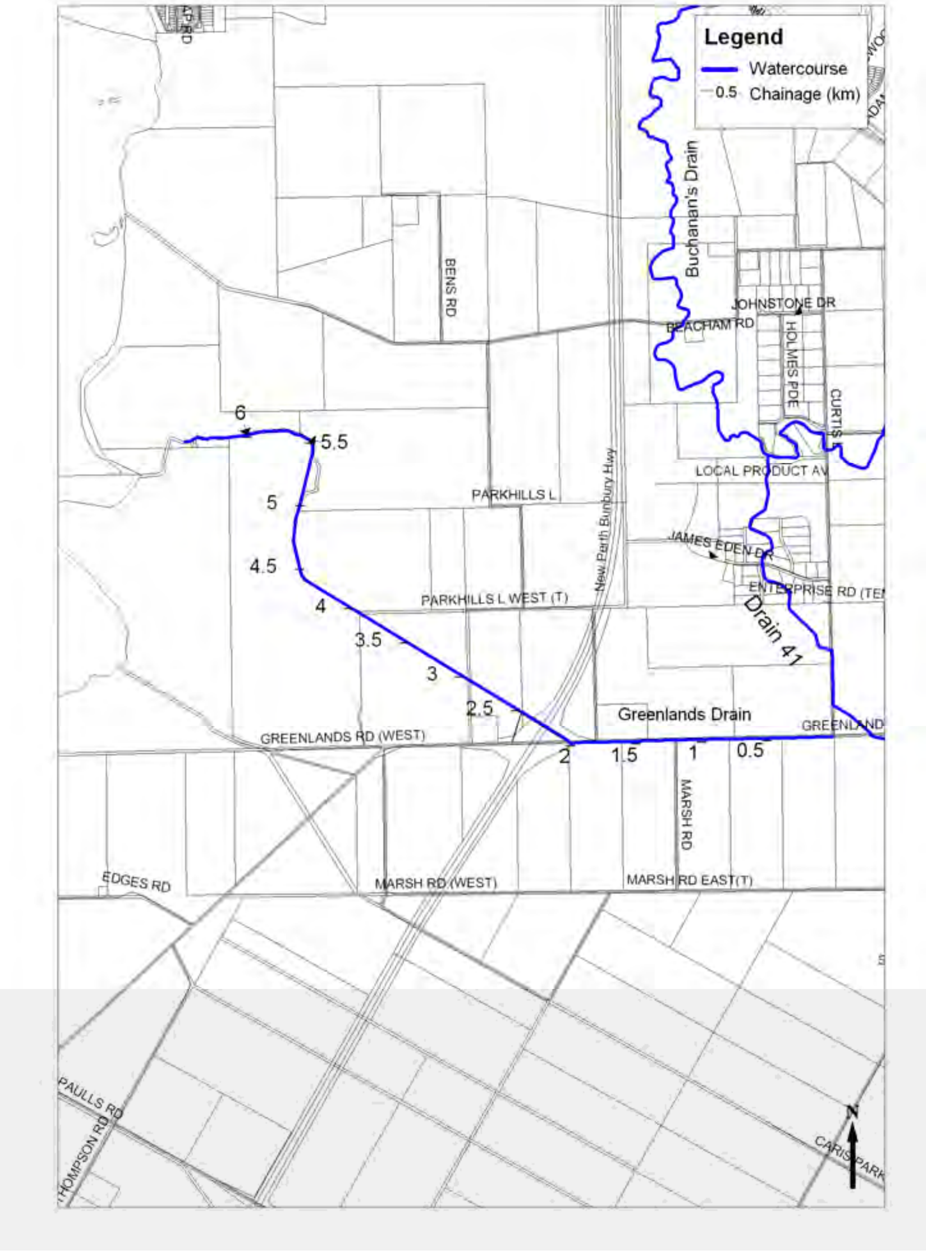


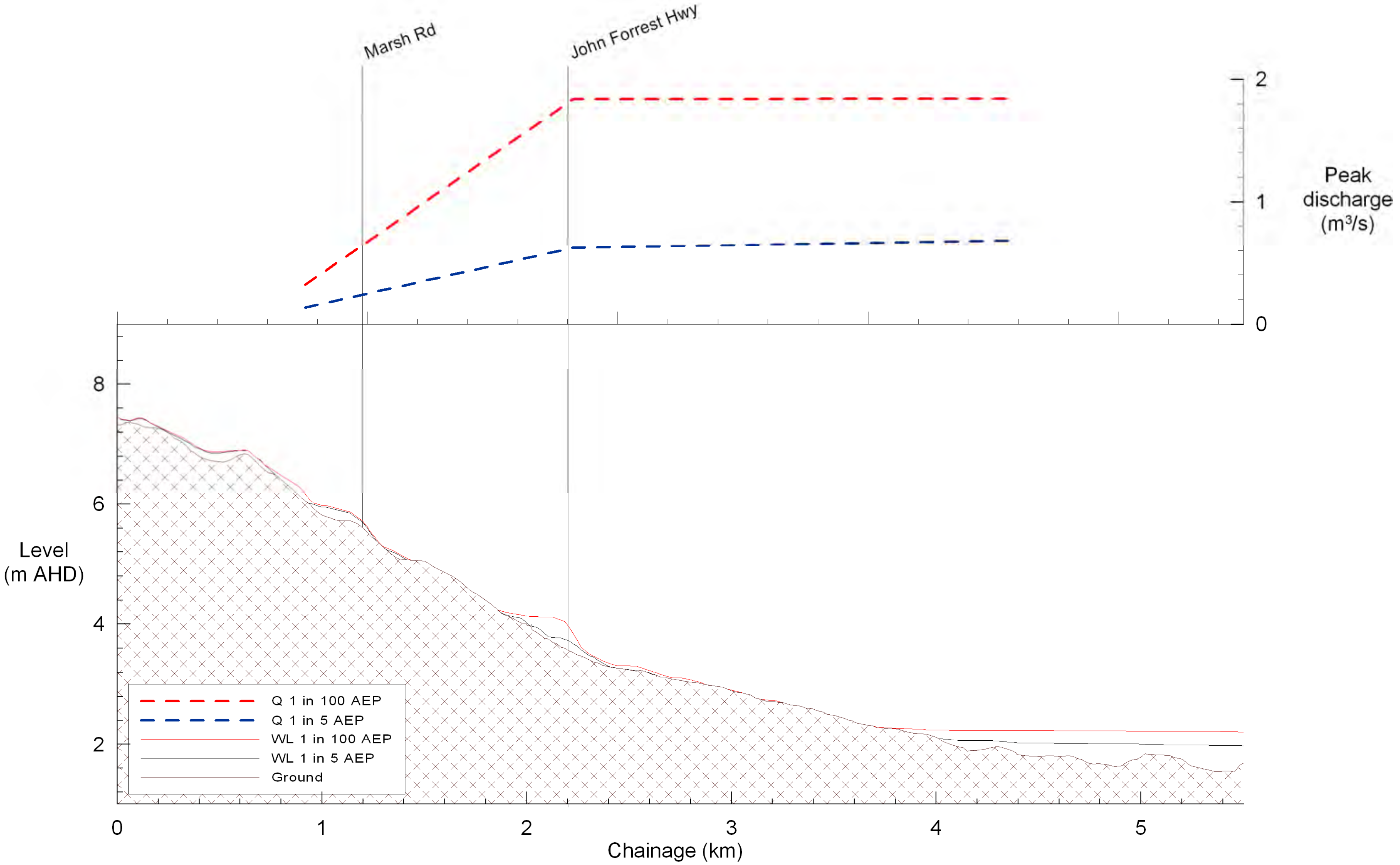
Drain 41


Legend
Watercourse
0.5 Chainage (km)
JARRA
JOHNSTONE DR
HOLMES PDE
CURTIS L
COBB AND CO RD
HARRIES WY
BAKER ST
COX ST
FIELDS RD
PHILLIPS RD
SUTTON ST
PAPERBARK PL
Buchanan's Drain
MOORES RD
TUCKEY ST
WILSON RD
THOMAS ST
TEAGUE ST
PEEL ST
ROE AV
LOCAL PRODUCT AV
LOVEGROVE ST
FORREST ST
ENTERPRISE RD (TEMP)
PERKINS AV
KWEL RD
HAMPTON RD
ALDERSON ST
PADBURY ST
GREENLANDS
5
4.5
3.5
3
2.5
2
1.5
1
Drain 41
MARSH RD EAST(T)
SUTTERS L
N

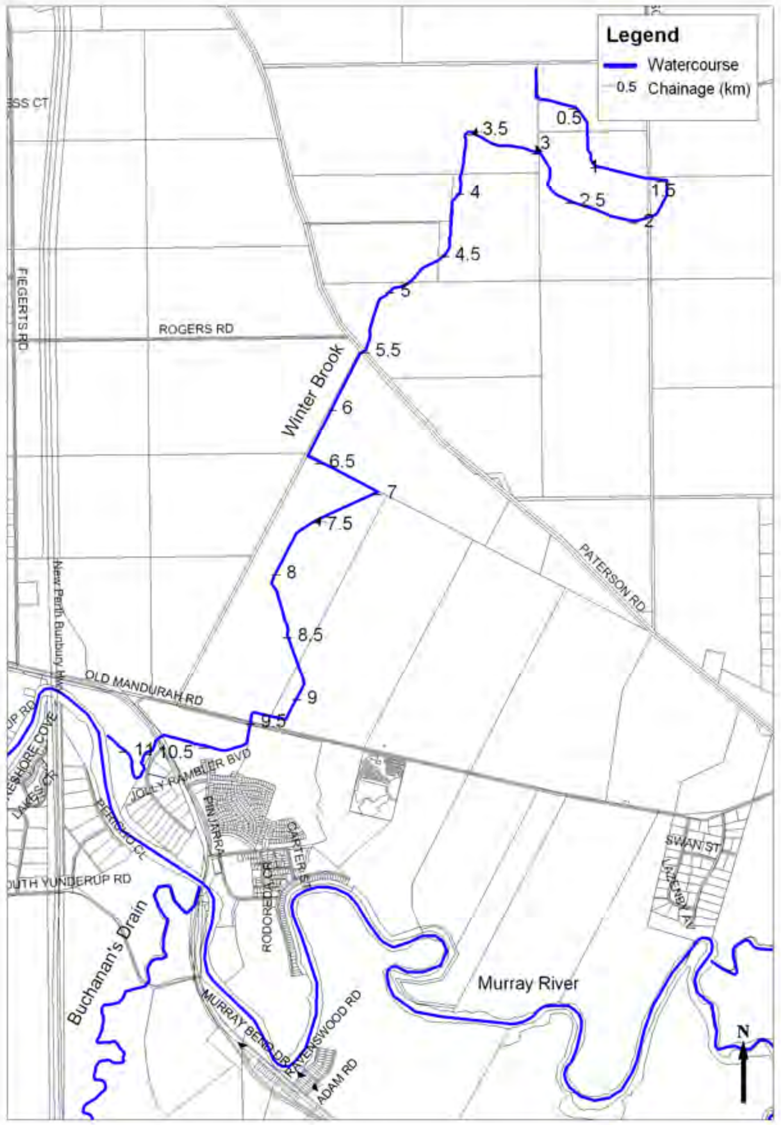

Sutters L
Greenlands Rd
Curtis L
James Eden Dr
Local Product Av
Peak discharge (m³/s)
0
1
2
3
Level (m AHD)
4
6
8
10
12
Chainage (km)
0
1
2
3
4
5
Q 1 in 100 AEP
Q 1 in 5 AEP
WL 1 in 100 AEP
WL 1 in 5 AEP
Ground

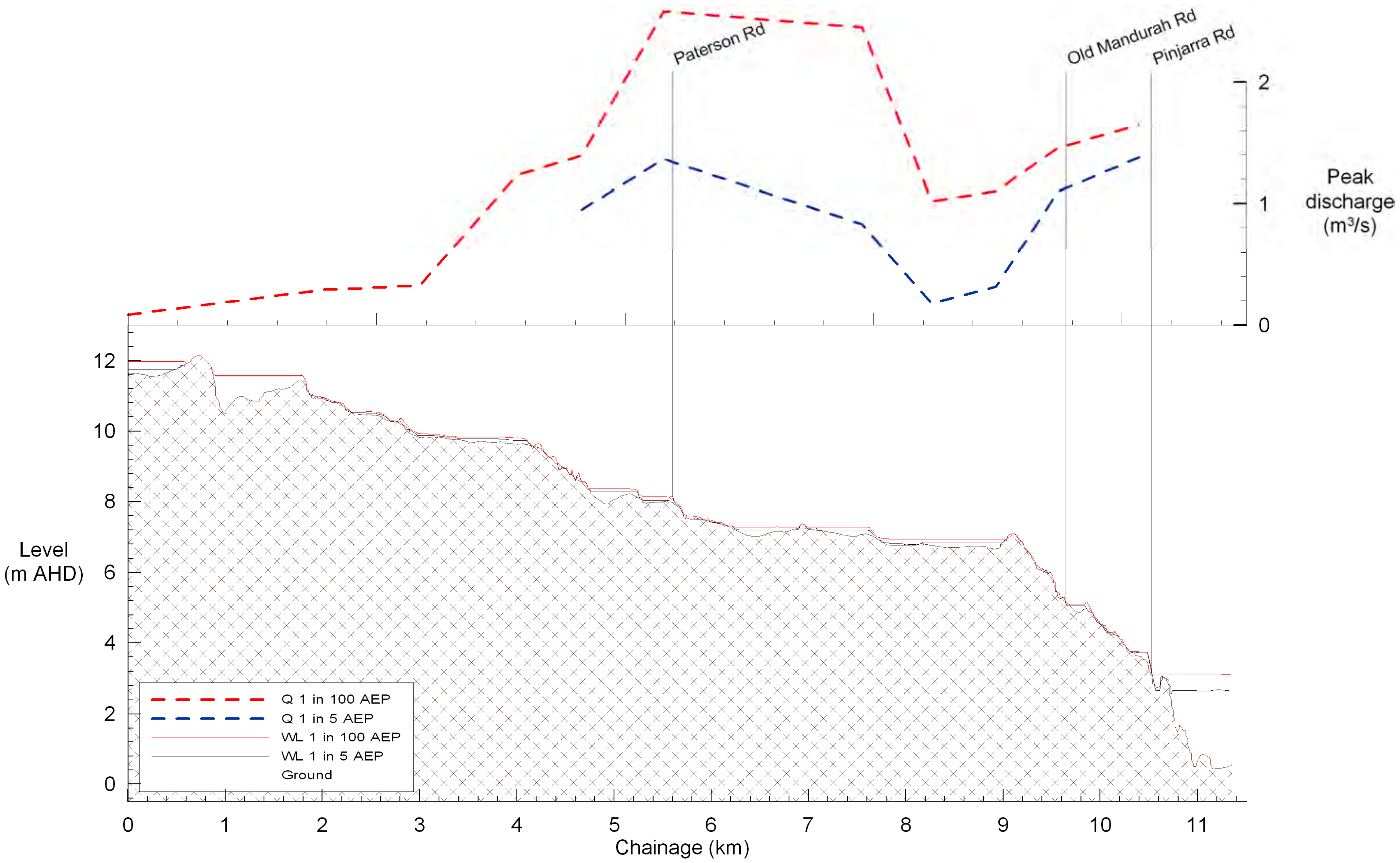
Greenlands Drain





Winter Brook





Appendix D

Detailed modelling

Background

Detailed models of two parts of the flood study area, Winter Brook and Buchanan's Drain were developed to evaluate the impact of small drainage lines on flooding patterns and to provide greater definition for peak discharge and water level estimates. Both catchments lie in the lower reaches of the Murray system, in the Ravenswood area, and discharge directly to the Murray River. The area is flat with ponding in winter and a large number of small mainly farm drains installed mainly to reduce ponding.

As flagged in Section 9, additional detailed modelling was recommended for individual areas to provide more detailed input to the DWMP planning process. This Appendix presents results of detailed modelling of Winter Brook and Buchanan's Drain and a comparison against the regional model's flood predictions.

Methods

Two drainage systems were simulated in more detail – Winter Brook and Buchnan's Drain. The two drainage systems were modelled separately. The models were based on the larger Mike Couple flood study model (referred to here as the regional flood model), as presented in Section 6 of this report, with additional detail to represent key smaller drains.

The following process was used in the simulation:

- The ground surface and resistance from the regional flood model was clipped to the study areas. The 20 m grid, as used in the regional model, was retained.
- Alignments and cross-sections for key drains were defined using the Lidar data.
- Culverts and bridges were characterised by site observation.
- Characteristics of the catchments were investigated and design simulations (1 in 5, 10 and 100 AEP) run. Peak flood level and discharge was then extracted and are reported in Appendix C.

Gauging station data for Winter Brook (Station 6124127) and Buchanan's Drain (614128) and pluviograph data (509646) were available for the area. Locations of the stations are given in Figure 1. The data were reviewed to help confirm the runoff coefficient used and help understand the hydrology of the catchment. Baseflow was separated using the straight line method.

Design rainfall for Pinjarra, was used. Rainfall was reduced by an area of 15 km^2 using the CRC FORGE reduction parameters.

Figure 1 Location of gauging stations

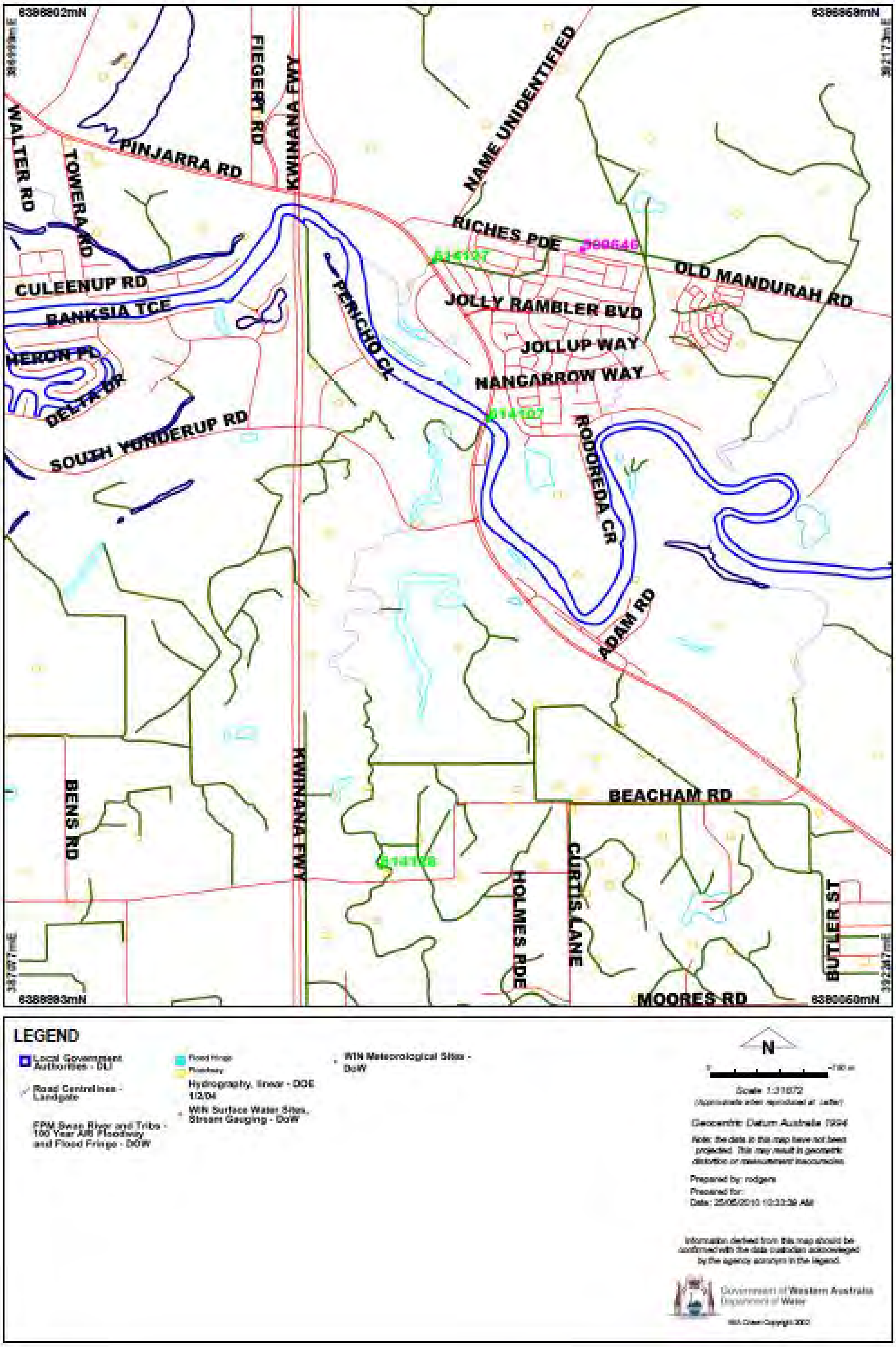


Figure 2 Winter Brook model layout

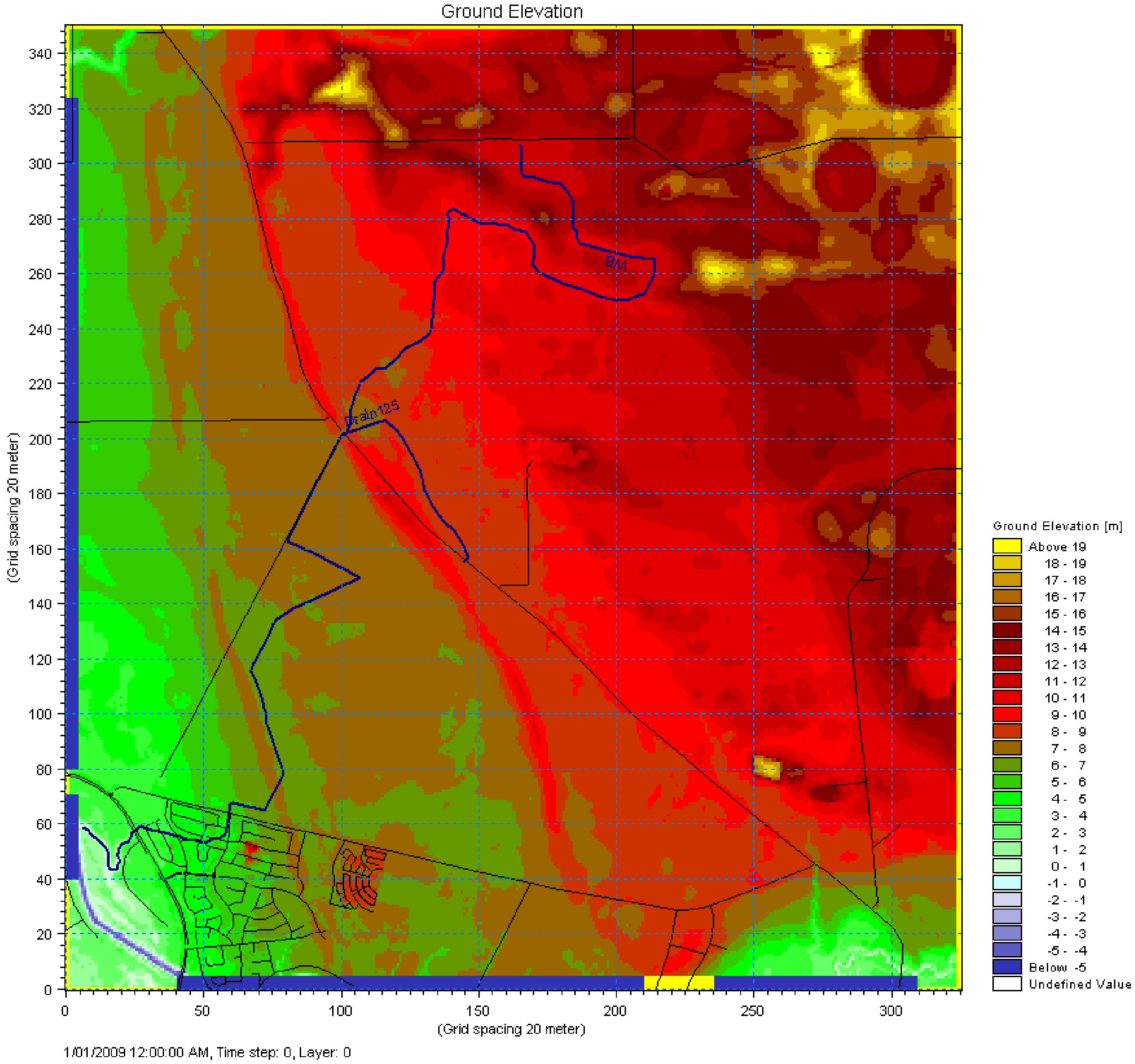
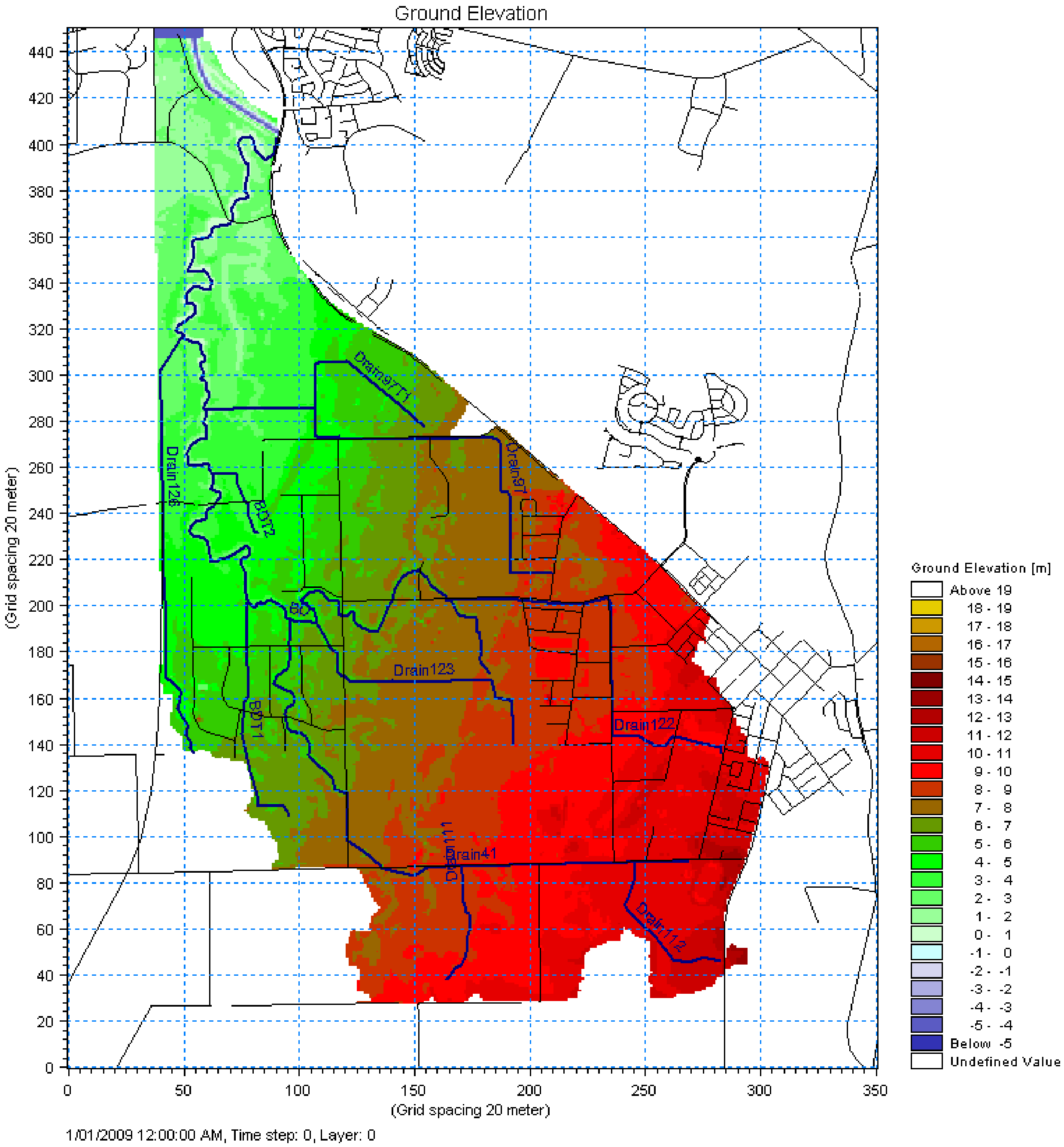


Figure 3 Buchanan's Drain model layout



Simulation results

Observed flow data

The observed data were used to help characterise the catchment's characteristics. The observed data captured three small flow events over winter 2009. These events were too small to adequately model using the Mike Couple model, but did confirm the general runoff characteristics of the catchments. Table 1 summarises characteristics of the flow events for Winter Brook.

For Winter Brook, some 60 mm of rain occurred before direct runoff commenced. Baseflow varied from 11 to 23 % of total flow in the three events. Direct flow varied from 15 to 33 % of rain.

For the design simulations, based on the Water Corporation (Pearce 2006) method, a proportional runoff coefficient varying from 42% (1 in 5 AEP) to 46% (1 in 100 AEP) for a 36 h event was adopted. These values seem reasonable given that the design events are more intense than the observed events, so were adopted for the design simulations.

Table 1 Summary of flow events, Winter Brook

Event	Date*	Rain (mm)	Total flow			Baseflow			Direct flow			
			(m^3)	(mm**)	% total	(m^3)	(mm)	% total	(m^3)	(mm**)	% total	% total rain
1	15-31 Jul	92.6	275,566	17.9	19	55,353	3.6	20	220,212	14.3	80	15
2	11-28 Aug	68.7	475,215	30.9	45	121,552	7.9	26	353,663	23.0	74	33
3	7-29 Sept	71.3	363,592	23.7	33	193,462	12.6	53	170,130	11.1	47	16

* date is from start of rainfall event to end of direct runoff; ** assuming catchment area of 15.4 km^2.

Critical duration

A range of duration storm events were tested for the Winter Brook model. While short duration events lead to marginally higher peak water levels in some areas, the 36 h duration event generally produced the highest water levels. Accordingly, a 36 h duration was adopted for the design simulations.

Design simulations

The Mike Couple model was run for the two catchments for three 36 h design events (1 in 5, 10 and 100 AEP). Peak discharge and water level are reported in Appendix B.

A comparison between peak water levels for the 1 in 100 AEP event for the two catchments is given in Figures 4 and 5. These show maximum water level predicted with the detailed model minus the regional flood study results. Therefore, a negative value indicates a reduction in water level with the detailed model.

The results indicate water levels generally across the catchments are relatively unchanged, except in small drains or at restrictions, normally roads. Water levels in these areas fell by up to 0.25 m. This indicates the impact of a more detailed representation of drains and culverts available in the detailed models.

Flows in Buchanans Drain downsteam of Old Mandurah Road increased in the detailed model compared with the regional flood model. The Old Mandurah Road culvert and the drain itself, which is small at that point, was not represented in the regional flood model so the road tended to block flow. Predicted 1 in 100 AEP flow just downstream of Old Mandurah Road (at DWMP extraction point WB6, Appendix B) for example, increased to 1.5 m^3/s from 0.05 m^3/s.

As the flow at this point is largely confined to the drain, general maximum flood levels in the area are not greatly affected. Figure 5 is a cross-section at this point, showing the Lidar and Mike 21 representations of the ground surface and the regional and detailed model's predictions of 1 in 100 AEP water level. The model's predicted water levels are similar but the detailed model includes the lidar representation of the drain and so better represents flow in the drain. In this case the drain is cutting across the direction of overland flow.

In parts of the Buchanan's Drain catchment, levels fell by up to 0.3 m (at Greenlands Road/Drain 111 and at Alderson Street/Drain 122).

An overland flow path across Curtis Lane is reduced in depth due to interception by Drain 41, but still flows in the 1 in 100 AEP event. Figure 6 shows a cross-section of Buchanans Drain Tributary downstream of the Curtis Lane overtopping point, just upstream of James Eden Drive (at DWMP extraction point BDT1, Appendix B). Reduced overtopping combined with increased flow across James Eden Drive (via the drain) in the detailed model reduces peak water levels from the regional model's predictions.

Note that Winter Brook's catchment boundaries, particularly on the north east and along Pinjarra Road, are variable. Flow directions tend to reverse at times during the event, effectively varying the catchment boundary as a function of ponding and runoff rate.

Note that water levels in the lower reaches of Buchanan's Drain, Drain 126 and Winter Brook are affected by differences in the treatment of flooding from the Murray River in the two models and should be ignored. Predictions close to the boundary of the Winter Brook model should also be ignored.

Figure 4 Difference in maximum water level, Winter Brook, 1 in 100 AEP

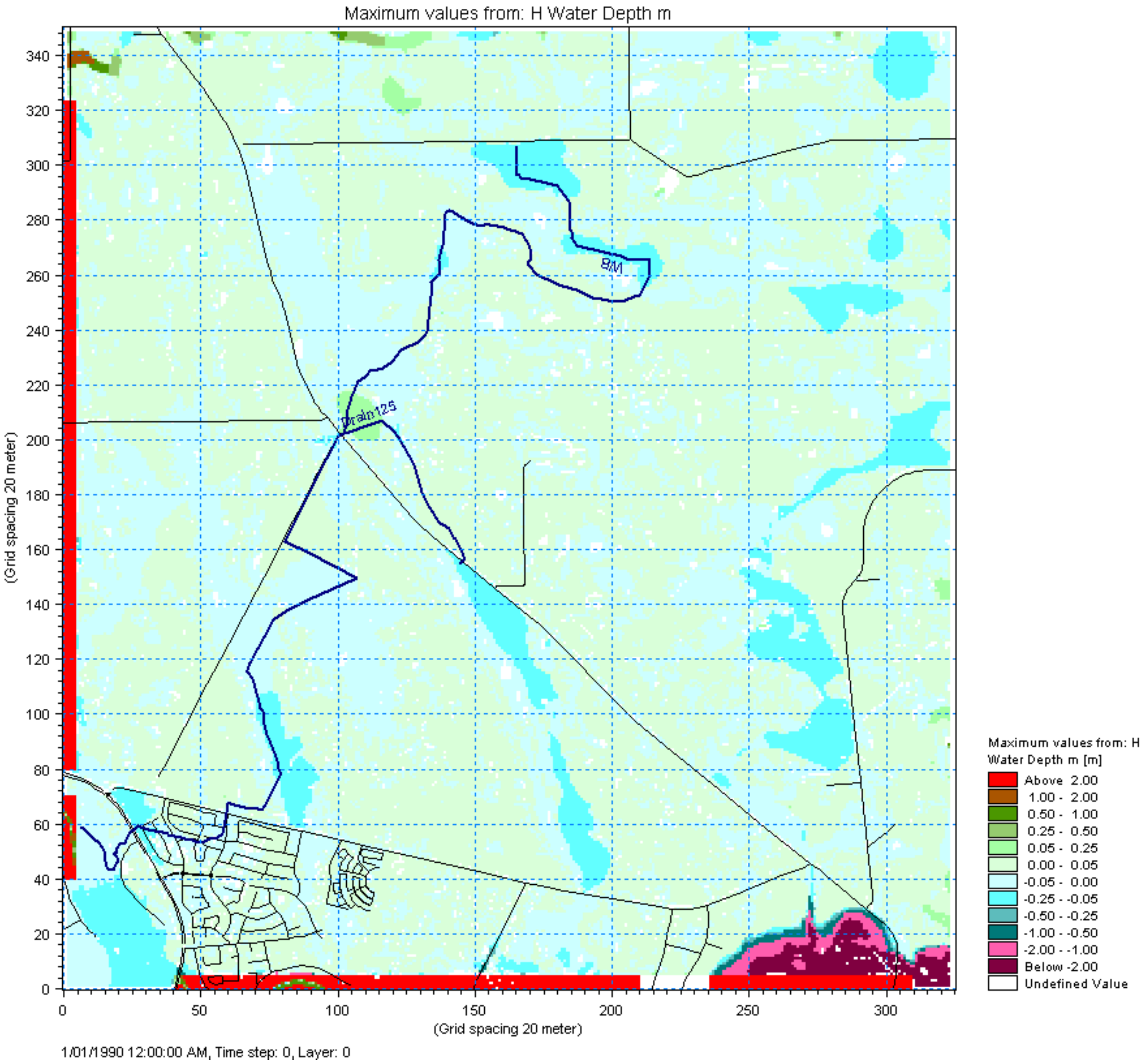


Figure 5 Difference in maximum water level, Buchanan's Drain, 1 in 100 AEP

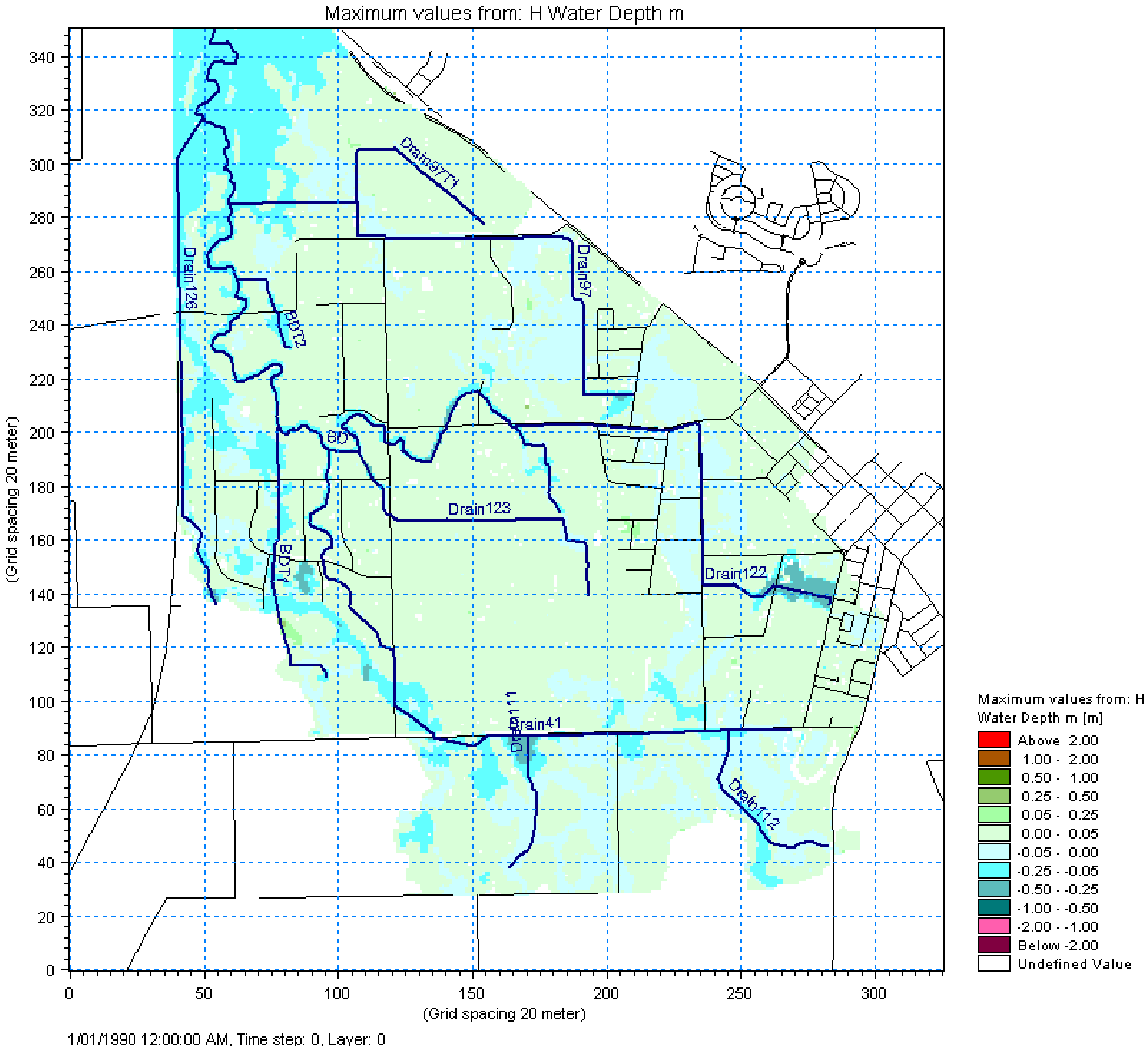


Figure 5 Cross-section of Winter Brook downstream of Old Mandurah Road

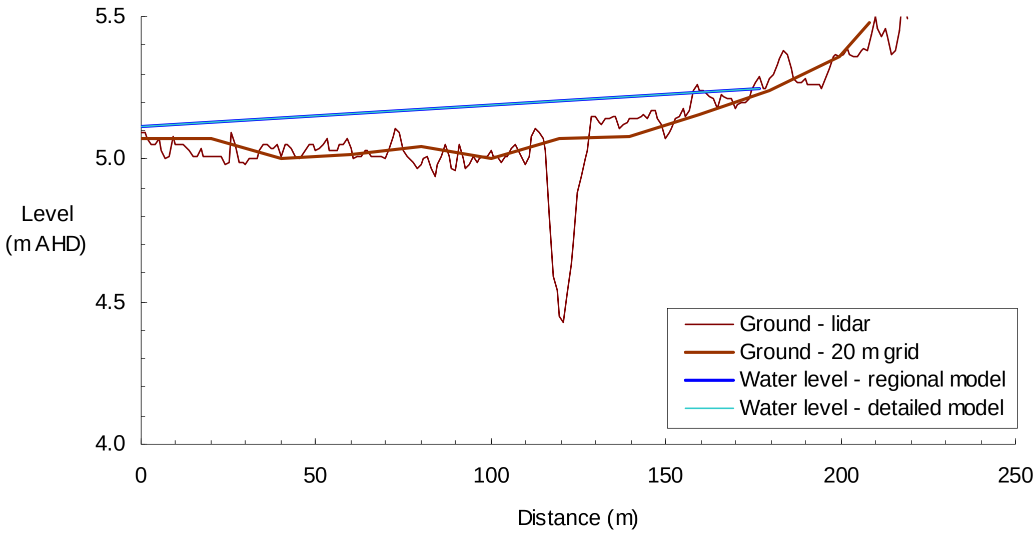
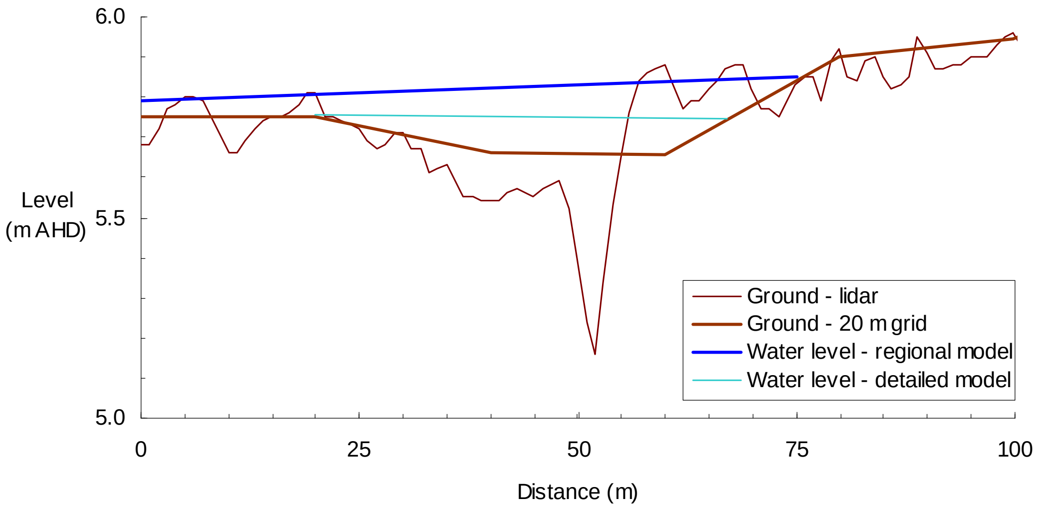


Figure 6 Cross-section of Buchnans Drain tributary upstream of James Eden Drive



Conclusions

From the comparison of the detailed modelling with the regional modelling, it is clear that the more detailed model allows better quantification of water levels and discharge in drains that are too small to be adequately represented with a 20 m grid. The detailed model predicts reductions in the 1 in 100 AEP peak flood level by up to 0.3 m compared with the regional model. Away from the small drains the detailed and regional models produce similar maximum water levels.

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