Albany Creek Flood Study
Volume 1 of 2

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BRISBANE CITY

Dedicated to a better Brisbane

Albany Creek Flood Study

Volume 1 of 2

Prepared by Cardno Pty Ltd
Prepared for Brisbane City Council

July 2014

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EXECUTIVE SUMMARY

Albany Creek is located in the northern part of Brisbane. The catchment is split between the Brisbane City Council (BCC) and Moreton Bay Regional Council (MBRC) local government areas.

The total area of the catchment is approximately 888 hectares. The creek outlets into the South Pine River just downstream of Leitchs Crossing at Brendale.

Brisbane City Council (BCC) commissioned Cardno to carry out a flood study of Albany Creek, comprising:

- an XP-RAFTS hydrologic model; and
- a TUFLOW 1D/2D hydraulic model.

BCC defined the study area for the hydraulic model to be downstream of Streisand Drive in McDowall, and Country Club Drive and Sunningdale Court in Albany Creek, as shown in Figure A1 (in Appendix A).

Based on the available rainfall and maximum height gauge (MHG) data, the following three events were selected for the calibration of the hydrologic and hydraulic models:

- May 1996;
- May 2009; and
- October 2010.

Good agreement was generally achieved at all MHGs for all calibration events, with differences between the recorded and calculated flood levels within ±250 mm.

The calibrated hydrologic and hydraulic models were used to analyse flood events with an average recurrence interval (ARI) from 2 years to 2,000 years. Design event modelling was carried out using the Australian Rainfall and Runoff temporal patterns.

In addition, flood discharges for design events were estimated by undertaking a flood frequency analysis to determine flows in Albany Creek for a range of recurrence events. The discharges determined from the flood frequency analysis were then compared to the discharges calculated using duration independent storms synthesised as proposed in Morris (1996). Factors were derived for the synthetic storms to provide the best possible agreement between the peak flows predicted by the synthetic storms and the peak flows predicted by the flood frequency analysis.

The hydraulic model was used to determine the peak flood levels along the creek for two scenarios:

- existing waterway conditions; and

- ultimate waterway conditions, including waterway corridors and a minimum vegetated riparian corridor.

Maps are contained in Volume 2, showing:

- peak flood levels and extent of inundation for the 2 to 100 year ARI flood events; and
- peak flood depths for the 2 to 100 year ARI flood events.

PART A – MODEL CALIBRATION

1.0 INTRODUCTION

1.1 Purpose

The purpose of this report is to describe the calibration of the Albany Creek Flood Model to the selected historical flood events in the catchment.

1.2 General

Albany Creek is located in the northern part of Brisbane. The catchment is split between the Brisbane City Council (BCC) and Moreton Bay Regional Council (MBRC) local government areas.

The total area of the catchment is approximately 888 hectares. The creek outlets into the South Pine River just downstream of Leitchs Crossing at Brendale.

The catchment takes in the suburbs of McDowall, Bridgeman Downs, Bunya, Albany Creek, and Arana Hills.

The upper reaches of the catchment are located in the Bunyaville Forest Reserve, upstream of Old Northern Road. Only a small part of the catchment upstream of Old Northern Road is developed – a small residential development in Arana Hills off Collins Road, and large-lot residential areas bounded by Old Northern Road and the Jinker Track.

Between Old Northern Road and Darien Street, the catchment is almost fully urbanised.

Between Darien Street and Albany Creek Road, there are some small areas of residential development along the western side of the catchment (near Keong Road). However, the majority of the catchment in this reach is undeveloped, containing the Darien Street Sports Fields, the Albany Creek Crematorium and Memorial Gardens, and the Albany Creek Road Reserve (including a pony club and park).

Downstream of Albany Creek Road, the catchment comprises residential development on the western side of the creek (in the MBRC area) and rural-residential development on the eastern side of the creek (in the BCC area).

1.3 Study Elements

The Flood Study comprises two elements:

- a hydrologic model; and
- a hydraulic model.

BCC defined the study area for the hydraulic model to be downstream of Streisand Drive in McDowall, and Country Club Drive and Sunningdale Court in Albany Creek, as shown in Figure A1 (in Appendix A).

2.0 AVAILABLE DATA

2.1 Hydrographic Data

2.1.1 Continuous Rainfall Gauges

Continuous rainfall data was provided for four gauges located in the vicinity of the Albany Creek catchment. This data is summarised in Table 2.1. The locations of the gauges are shown in Figure A2 (in Appendix A).

Table 2.1: Continuous Rainfall Data

Rain Gauge	Period of Record
C_R572	25/5/1994 – present
LCR566	1/7/1994 – present
A_R842	22/3/2005 – present
SPR959	19/2/2010 – present

2.1.2 Maximum Height Gauges (MHGs)

Three maximum height gauges (MHGs) are located along Albany Creek. Details of these gauges are listed in Table 2.2. The locations of the gauges are shown in Figure A2 (in Appendix A).

Table 2.2: Maximum Height Gauge Data

Maximum Height Gauge	Approximate Location	Period of Record
100	600 metres upstream of creek mouth	2/11/1981 – present
110	Just upstream of Albany Creek Road	5/5/1980 – present
120	Extension of Darien St, Bridgeman Downs	4/4/1988 – present

2.2 Topographic Data

The following topographic data was provided for the flood study:

- 0.25 metre contour data of the catchment in the Moreton Bay Regional Council local government area;
- 0.50 metre contour data of the catchment in the Brisbane City Council local government area;
- Airborne Laser Scanning (ALS) survey data of the catchment provided by the Department of Environment & Resource Management (DERM, 2009);
- cross sections of the creek (from the 1991 Flood Study, and survey carried out by Council in 2012); and
- survey and as-constructed information of the creek crossings (refer Section 2.3).

2.3 Hydraulic Structures

The flood study area comprises four hydraulic structures. The details of these structures are shown in Table 2.3.

Table 2.3: Hydraulic Structure Details

Location	Details
Galaxy Street, Bridgeman Downs	2/1800 mm RCPs
Peterson Place, Bridgeman Downs	8/1200 mm RCPs
Wendon Way, Bridgeman Downs	7/2.4 x 2.4 metre RCBCs
Albany Creek Road, Albany Creek	Bridge Structure

Details of the three sets of culverts were provided by BCC.

Department of Main Roads' drawings of the bridge structure at Albany Creek Road were also provided by BCC. In addition, Council provided survey information of:

- the opening under the bridge;
- the road elevation; and
- handrails and road barriers on the upstream and downstream sides of the bridge.

Hydraulic Structure Reference Sheets for each of the four structures are contained in Appendix B.

3.0 MODEL DESCRIPTIONS

3.1 Introduction

The models utilised for the Flood Study were:

- an XP-RAFTS hydrologic model; and
- a TUFLOW 1D/2D hydraulic model.

3.2 Hydrologic Model

The hydrologic modelling of the runoff in the Albany Creek catchment was carried out using XP-RAFTS. XP-RAFTS is an urban and rural runoff routing model used to calculate flood hydrographs from rainfall, catchment and channel inputs.

For the hydrologic model, the Albany Creek catchment was subdivided into 36 subareas. The subcatchment delineation is shown in Figure A3 (in Appendix A).

The fraction impervious of each subarea was determined for three scenarios:

- 1996 conditions (for the May 1996 calibration event), based on historical aerial photography;
- 2009/2010 conditions (for the May 2009 and October 2010 calibration events), based on current aerial photography; and
- ultimate catchment development (for the design flood events), based on current planning information from both the Moreton Bay Regional Council and Brisbane City Council.

The fraction impervious value adopted for each land use is shown in Table 3.1.

Table 3.1: Land Use Types

Land Use	Fraction Impervious
ResA/Low Density Residential	60%
Emerging Community	60%
Special Facilities	60%
Special Residential	40%
Rural	20%
Community Use	20%
Park Residential	10%
Park/Open Space/Sport & Recreation	0%
Conservation	0%

Each subarea was divided into two parts to reflect the impervious and pervious sections of the subarea.

The fraction impervious value adopted for each subarea is shown in Tables A1 to A3 (in Appendix A). The average fraction impervious of the overall catchment for each scenario was:

- 1996 conditions – 18%;
- 2009/2010 conditions – 22%; and
- ultimate catchment development – 25%.

The distribution of land uses across the catchment, for each scenario, is shown in Figures A13 to A15 (in Appendix A).

Approximately 40% of the Albany Creek catchment is located upstream of Old Northern Road. Albany Creek and its tributaries cross under Old Northern Road via culverts at three locations. At each crossing, the level of Old Northern Road is approximately six metres higher than the invert level of the creek at that point. Consequently, the culverts at Old Northern Road attenuate the stormwater runoff discharging to areas downstream.

The detention provided at the three culvert crossings at Old Northern Road was therefore included in the hydrologic model of the catchment. Modelling confirmed that Old Northern Road is not overtopped for floods up to and including the 1 in 100 year average recurrence interval (ARI) event.

Stage-storage curves for the storage area just upstream of Old Northern Road at each culvert crossing were determined based on the 0.25 metre contour data provided by MBRC. Culvert details were also provided by MBRC. A summary of the characteristics of each crossing is shown in Table 3.2.

Table 3.2: Old Northern Road Culvert Crossings

Approximate Location	Catchment Area (ha)	Culvert Details
The Boulevard, Albany Creek	204.6	3/1650 mm RCPs
Allamanda Crescent, Albany Creek	111.6	3/1650 mm RCPs
De Vito Place, McDowall	8.3	1200 mm RCP

To provide an understanding of the indicative response time of the overall catchment, the Bransby-Williams equation was used to estimate the time of concentration of the catchment. The characteristics of the catchment were calculated as follows:

- Stream Length = 8.5 km
- Catchment Area = 888 ha
- Equal Area Slope = 0.7%

Using the Bransby-Williams equation, the time of concentration of the catchment was calculated to be approximately 270 minutes, or 4.5 hours. The Bransby-Williams equation is suitable for use in rural catchments, whereas the Albany Creek catchment contains large areas of urban development. It also includes areas which provide significant flow attenuation (as discussed above). However, this estimate of the time of concentration can be used as a reasonable indication of the response time of the catchment.

The XP-RAFTS model includes a number of routing parameters as follows:

- Channel Cross Section and Slope;
- Catchment Slope;
- Manning's n (PERN); and
- Storage coefficient multiplication factor (BX)

Routing in the XP-RAFTS links was completed by taking representative cross sections from the ALS data within each subcatchment. The slope along the channel within each reach was also obtained from the ALS data.

The catchment slope for each subcatchment was obtained from the ALS data supplied by Council. The slope was entered into the model as a uniform percentage within each subcatchment for both the impervious and pervious areas.

The Manning's n (PERN) values for the impervious and pervious subcatchment areas were 0.014 and 0.03 respectively. These values are consistent with the recommendations provided by XP-RAFTS Reference Manual.

In order to maintain consistency with previous modelling of the Albany Creek catchment, a storage coefficient multiplication factor (BX) has been adopted. This was determined from an iterative assessment of the peak discharges from the outlet of the model. The final value that has been adopted is 1.5.

3.3 Hydraulic Model

The flood flow in Albany Creek was modelled using the combined 1-dimensional/2-dimensional unsteady flow software TUFLOW (Build 2010-10-AC-iSP). TUFLOW models free-surface flows in one-dimensional links (such as open channels, pipes and culverts, bridges, etc) and two-dimensional domains.

For the flood study, a fine 4.0 metre grid was used to define flow in the 2-dimensional domain. 1-dimensional links were used to model the four hydraulic structures included within the study area defined by BCC.

A timestep of 2 seconds was used in the model.

Topographic information for the TUFLOW model was obtained from ALS data provided by BCC. The ALS data was collected by DERM in 2009. The vertical accuracy of ALS data is reportedly in the order of ±150 mm.

Following discussions with Council, it was decided to use the invert level data from the existing surveyed cross sections of the creek and the hydraulic structures to define a continuous creek invert along Albany Creek in the TUFLOW model.

The Manning's n roughness values used in the model are shown in Table 3.3.

Table 3.3: Manning's n Values

Land Use	Manning's n Value
Heavily Treed Areas	0.10
Riparian Vegetated Corridors	0.15
Residential Areas	0.20
Grassed Areas, Parks	0.035
Road Reserves	0.025

Inflows to the TUFLOW model were provided by the calibrated XP-RAFTS model.

4.0 CALIBRATION

4.1 Calibration Events

4.1.1 Hydrologic Model Calibration

Based on the available rainfall and maximum height gauge data, the following three events were selected for the calibration of the hydrologic and hydraulic models:

- May 1996;
- May 2009; and
- October 2010.

The cumulative rainfall from each event, including the rain in the week leading up to and following the peak of the event, are shown in Figures A4 to A6 (in Appendix A).

Intensity-Frequency-Duration (IFD) curves for each event are also shown in Figures A7 to A9 (in Appendix A). As discussed in Section 3.2, the time of concentration of the catchment is in the order of 4.5 hours.

No stream gauges are located within the Albany Creek catchment. Consequently, no direct calibration of the hydrologic model to recorded stream flows was possible. However, the flows calculated by the hydrologic model were input into the hydraulic model, for calibration to the recorded flood peaks.

As a further check of the hydrologic model, the peak design flows calculated by the hydrologic model were compared to the design flows determined from:

- the Rational Method; and
- the 1991 flood study of the catchment.

These results are discussed in Part B of the report.

For all calibration events, an initial loss of 0 mm was adopted, and a continuing loss rate of 0 mm/h and 2.5 mm/h was adopted in the impervious and pervious areas respectively. An initial loss of 0 mm was considered acceptable, as some lead-up rainfall was recorded for each storm event prior to the heaviest burst of rain which caused the flooding in the catchment.

For the three calibration events, the calculated flow hydrographs at the mouth of the creek are shown in Figures A10 to A12 (in Appendix A).

4.1.2 Hydraulic Model Calibration

The peak flood levels calculated by the hydraulic model were compared to the recorded maximum height gauge (MHG) readings, for each flood event. The results are discussed in the following sections.

Brisbane City Council advises that the recorded maximum height gauge levels are generally considered to have an accuracy of ±200 mm, due to gauge reading errors, problems with the operation of the gauge, flow patterns around the gauge, etc. Thus, calibration to the recorded levels is considered acceptable if the model results are within approximately 200 mm of the maximum height gauge levels.

A good calibration was generally achieved across all events, using a Manning's n roughness value in the heavily treed areas (i.e. along the waterways) of 0.10.

It is noted that the ALS data does not accurately define the low flow paths in heavily vegetated areas, as the laser scanning cannot penetrate to the true invert level of the creek. Consequently, survey cross sections of the creek were used to define the invert level continuously along the creek within the Study Area. The surveyed cross sections were obtained from:

- the 1991 Flood Study; and
- Council survey from 2012.

4.2 May 1996 Event

The May 1996 event was a relatively minor flood event. Based on the IFD curves of the recorded rainfall, the frequency of the event was in the order of 2 to 5 years.

A comparison of the recorded peak flood levels to the modelled peaks is shown in Table 4.1.

Table 4.1: Peak Flood Levels – May 1996 Event

Maximum Height Gauge	Recorded Peak Flood Level (mAHD)	Calculated Peak Flood Level (mAHD)	Difference (m)
100	10.32	10.34	+0.02
110	14.93	16.05	+1.12
120	25.05	25.12	+0.07

Good agreement was achieved at MHGs 100 and 120, with the differences less than or equal to 70 mm. A poor match was achieved at MHG 110. However, the recorded peak level at MHG 110 appears to be incorrect, as discussed below.

The magnitude of the May 1996 and May 2009 events are similar. Based on the IFD curves, both events had a frequency in the order of 2 to 5 years. The recorded peak flood levels in May 1996 were 180 mm and 90 mm higher than the recorded peak flood levels in May 2009 at MHGs 100 and 120 respectively. However, the recorded peak flood level in May 1996 at MHG 110 was 940 mm lower than that recorded in May 2009. It can therefore be concluded that the recorded level at MHG 110 in May 1996 is approximately one metre too low.

4.3 May 2009 Event

The May 2009 event was similar in magnitude to the May 1996 event. Based on the IFD curves of the recorded rainfall, the frequency of the event was in the order of 2 to 5 years.

As discussed above, the recorded flood level at MHG 110 in May 1996 appears to be incorrect. Thus, based on the recorded flood levels in May 2009, this event is the smallest of the three calibration events.

A comparison of the recorded peak flood levels to the modelled peaks is shown in Table 4.2.

Table 4.2: Peak Flood Levels – May 2009 Event

Maximum Height Gauge	Recorded Peak Flood Level (mAHD)	Calculated Peak Flood Level (mAHD)	Difference (m)
100	10.14	10.35	+0.21
110	15.87	16.07	+0.20
120	24.96	25.17	+0.21

Reasonable agreement was achieved at all three MHGs, with the differences less than or equal to 210 mm at all gauges.

4.4 October 2010 Event

The October 2010 event is the largest of the three calibration events. Based on the IFD curves of the recorded rainfall, the frequency of the event was in the order of 5 to 20 years.

A comparison of the recorded peak flood levels to the modelled peaks is shown in Table 4.3.

Table 4.3: Peak Flood Levels – October 2010 Event

Maximum Height Gauge	Recorded Peak Flood Level (mAHD)	Calculated Peak Flood Level (mAHD)	Difference (m)
100	10.94	10.70	-0.24
110	16.12	16.34	+0.22
120	25.15	25.36	+0.21

Good agreement was achieved at all MHGs, with the differences less than or equal to 240 mm.

5.0 VERIFICATION

5.1 Verification Events

No verification events were modelled.

To check for consistency between the hydrologic and hydraulic models, the discharge hydrographs calculated at the mouth of Albany Creek were compared. The results for the three calibration events are shown in Figures A10 to A12 (in Appendix A).

These hydrographs demonstrate a high degree of consistency between the two models, both in terms of the magnitude and the timing of the flow peaks.

5.2 Structure Verification

The TUFLOW model included three sets of culverts (at Galaxy Street, Peterson Place and Wendon Way) and the bridge crossing (at Albany Creek Road). The structure losses at these three crossings (as calculated by the TUFLOW model) were verified by checking the results with an independent hydraulic package, namely HEC-RAS (version 3.1.3).

The structure losses were compared for the three calibration events, and for the 10 year and 50 year ARI design events. The results of both hydraulic models for all events are shown in Table 5.1.

These results demonstrate that the structure losses calculated by the TUFLOW model are consistent with the results from the HEC-RAS model.

Table 5.1: Comparison of Hydraulic Model Structure Losses

Event	Upstream Level TUFLOW (mAHD)	Upstream Level HEC-RAS (mAHD)	Difference (m)
Galaxy Street			
May 1996	35.35[1]	35.36[1]	+0.01
May 2009	35.38[1]	35.38[1]	0
October 2010	35.72[1]	35.70[1]	-0.02
10 Year ARI	35.95[1]	35.95[1]	0
50 Year ARI	36.26[1]	36.27[1]	+0.01
Peterson Place			
May 1996	25.97[2]	25.98[2]	+0.01
May 2009	26.00[2]	26.01[2]	+0.01
October 2010	26.20[2]	26.23[2]	+0.03
10 Year ARI	26.29[2]	26.28[2]	-0.01
50 Year ARI	26.62[2]	26.61[2]	-0.01
Wendon Way			
May 1996	25.72[2]	25.73[2]	+0.01
May 2009	25.77[2]	25.76[2]	-0.01
October 2010	25.97[2]	25.95[2]	-0.02
10 Year ARI	26.07[2]	26.06[2]	-0.01
50 Year ARI	26.35[2]	26.36[2]	+0.01
Albany Creek Road Bridge			
May 1996	15.92	15.90	-0.02
May 2009	15.93	15.92	-0.01
October 2010	16.23	16.23	0
10 Year ARI	16.23	16.24	+0.01
50 Year ARI	16.38	16.41	+0.03

Notes: 1. Inlet Control
2. Outlet Control

5.3 Sensitivity to Model Parameters

A sensitivity analysis was carried out, examining the impact of the following factors on flood levels along Albany Creek:

- tailwater level in the South Pine River; and
- Manning's n value along the waterway.

The impact of the tailwater level in the South Pine River is discussed in Section 7.2.

As discussed in Section 3.3, a Manning's n value along the Albany Creek waterway of 0.10 was used in the calibration of the model. To examine the sensitivity of the model to changes in the Manning's n value, the 100 year 2 hour storm event (which produces the peak flood levels along most of the waterway within the study area) was re-run assuming a Manning's n value of 0.15 along the waterway.

The results showed that increasing the Manning's n in all heavily vegetated areas (which constitutes the vast majority of the inundation extent) from 0.10 to 0.15 causes an increase in flood level of up to 300 mm along most of the creek. A map showing the changes in flood level is contained in Figure A16 (in Appendix A).

5.4 Calibration and Verification Summary

The above results show that the model has achieved a good calibration, across all flood events.

The model verification confirmed that the hydrologic and hydraulic models are producing consistent results. It also demonstrated the structure losses calculated by the TUFLOW model are consistent with the losses calculated by HEC-RAS. Thus, the structure losses are acceptable.

PART B – DESIGN EVENT MODELLING

6.0 INTRODUCTION

The purpose of this part of the report is to describe the calculation of the design flood levels and discharges in Albany Creek, using the calibrated hydrologic and hydraulic models.

7.0 MODEL DATA

7.1 Design Rainfall

The calibrated hydrologic and hydraulic models were used to analyse flood events with an average recurrence interval from 2 years to 2,000 years.

For the 2 to 100 year ARI events, the rainfall intensities were obtained from the Brisbane City Council's *Subdivision and Development Guidelines.* For the 200 and 500 year ARI events, the rainfall intensities were determined in accordance with the CRC-FORGE methodology. For the 2,000 year ARI event, Brisbane City Council provided a 6 hour "superstorm" rainfall pattern.

Temporal patterns for all events were determined in accordance with Australian Rainfall and Runoff (AR&R).

For all design events, an initial loss of 0 mm was adopted, and a continuing loss rate of 0 mm/h and 2.5 mm/h was adopted in the impervious and pervious areas respectively. This is consistent with the losses used for the calibration events.

7.2 Tailwater Conditions

Albany Creek outlets into the South Pine River at Brendale. Design flood levels in the South Pine River were obtained from the *Lower Pine River Flood Study* (Worley Parsons, 2009). The design flood levels (assuming ultimate catchment development) are shown in Table 7.1.

Table 7.1: South Pine River Peak Flood Levels at Albany Creek Mouth

Average Recurrence Interval	Flood Level (mAHD)
10 years	6.78
20 years	7.15
50 years	7.45
100 years	7.75

The catchment area of the South Pine River to the mouth of Albany Creek is approximately 190 km². It therefore has a time of concentration significantly longer than the time of concentration of Albany Creek. The *Lower Pine River Flood Study* did not provide any details regarding the time of concentration for the South Pine River catchment. For the purposes of this analysis, an indicative time of concentration of the South Pine River to Albany Creek of 12 hours was adopted.

A coincident flooding assessment was carried out, in accordance with QUDM Section 8.03.4(c). This assessment indicated the following coincident floods:

- Albany Creek Q100 + South Pine River Q1; and
- South Pine River Q100 + Albany Creek Q5.

A sensitivity analysis was carried out to determine the impact of the downstream tailwater level on flood levels in Albany Creek. The analysis showed that a change in the tailwater level of more than two metres (between 7.43 mAHD and 9.48 mAHD) only affected flood levels in the final 250 metres of the creek because of the flood gradient along Albany Creek. Given this result, a uniform tailwater level in the South Pine River equivalent to the 100 year flood level (i.e. 7.75 mAHD) was adopted for all design flood events in Albany Creek.

7.3 Topographic and Structure Data

Airborne Laser Scanning (ALS) survey data of the study area, provided by BCC, was used to setup the TUFLOW model. The ALS data was collected in 2009.

As discussed in Section 4.1.2, the ALS data was supplemented by surveyed invert levels of Albany Creek.

Three culvert crossings are located within the study area. Details of these culverts were provided by BCC. Details of these structures are shown in Table 2.3.

One bridge crossing, at Albany Creek Road, is located within the study area. Department of Main Roads' drawings (dated 1986) of this two-span bridge were provided by BCC. In addition, Council provided survey information of the opening under the bridge, the bridge handrails and barriers, and the road level. The bridge was therefore modelled using the survey information provided by Council.

Fencing or barriers were included in the hydraulic model at Albany Creek Road and Wendon Way. At Peterson Place, the handrail comprises a post and rail fence with large open areas through it. Thus, the fence does not cause a significant obstruction to the flow. At Galaxy Street, there is no obstruction to flow occurring over the road.

The blockages at the hydraulic structures are summarised in Table 7.2

Table 7.2: Hydraulic Structure Blockages due to Fencing

Location	Blockage Details
Albany Creek Road	Concrete barriers
Wendon Way	Fencing with narrow openings
Peterson Place	Post & Rail Fencing
Galaxy Street	No blockage

7.4 Land Use

The hydrologic modelling of the catchment for the design flood events adopted land uses assuming ultimate catchment development (based on current planning). Ultimate land use maps were provided for both the BCC and MBRC local government areas. The adopted fraction imperviousness in each subarea is shown in Tables A1 to A3 (in Appendix A).

8.0 DESIGN EVENT MODELLING

8.1 Australian Rainfall and Runoff

Design event modelling was carried out using the Australian Rainfall and Runoff (AR&R) temporal patterns.

The 2, 5, 10, 20, 50, 100, 200, 500 and 2000 year Average Recurrence Interval (ARI) flood events were modelled.

The relationship between Average Recurrence Interval (ARI) and Annual Exceedance Probability (AEP) is shown in Table 8.1

Table 8.1: Design Event Frequency

Average Recurrence Interval	Annual Exceedance Probability
2 years	50%
5 years	20%
10 years	10%
20 years	5%
50 years	2%
100 years	1%
200 years	0.5%
500 years	0.2%
2,000 years	0.05%

Storm durations from 15 minutes to 24 hours were initially modelled. However, the peak flood levels and discharges within the study area were produced by storms of 1 to 3 hours' duration.

The hydrographs calculated using the AR&R hydrology were used in the TUFLOW model.

8.2 Duration Independent Storm (DIS)

Duration independent storms were developed for a given ARI event using intensity-frequency-duration (IFD) curves for Brisbane based on Australian Rainfall & Runoff (Institution of Engineers, 1987).

Flood discharges for design events were estimated by undertaking a flood frequency analysis to determine flows in Albany Creek for a range of recurrence events. The discharges determined from the flood frequency analysis were then compared to the discharges calculated using duration independent storms synthesised as proposed in Morris (1996). Factors were derived for the synthetic storms to provide the best possible agreement between the peak flows predicted by the synthetic storms and the peak flows predicted by the flood frequency analysis.

The factors were then applied to the duration independent storms in the calibrated XP-RAFTS model to determine discharges for all design events throughout the catchment.

A flood frequency analysis of the XP-RAFTS model flows is based on Brisbane Central Business District (CBD) rainfall from 1911 to 2011. The analysis assumed historically recorded Brisbane CBD rainfall was representative of rainfall in the Albany Creek catchment as a whole.

The most severe recorded rainfall events from each year between 1911 and 2011 (inclusive), for a range of standard durations were selected. The rainfall recorded at gauges located within the Brisbane CBD was used for the analysis rather than data collected within the Albany Creek catchment due to the long and continuous record available via the CBD gauges. Further, given the relatively close proximity of the Albany Creek catchment to the Brisbane CBD, it was considered that the use of the CBD data was acceptable.

A range of standard duration storms, from 30 minutes to 24 hours, was applied to the catchment to ensure that the peak discharge was calculated at all points along the creek. The standard duration storms used in the analysis are:

- 30 minutes
- 1 hour
- 2 hours
- 3 hours
- 4 hours
- 6 hours
- 12 hours
- 24 hours

The longer standard duration rainfall events, i.e. between 3 and 24 hours, were applied to the catchment to ensure that the rainfall events critical to the detention areas upstream of Old Northern Road were considered.

Discharges in Albany Creek were calculated for the standard duration rainfall events for each of the 101 years of rainfall data. One key representative location in the catchment, i.e. Albany Creek Road, was then selected to perform the flood frequency analysis.

The 101 annual peak discharges at Albany Creek Road were ranked from highest to lowest. The plotting position (P_i) (which provides an estimate of the average recurrence interval) of each calculated discharge was determined using Cunnane's formula (Institution of Engineers Australia, 1987):

$$P_i = \frac{r - 0.4}{N + 0.2}$$

where r = rank of discharge (the largest flood having r = 1)
N = number of annual peak discharges

The peak annual discharge series at Albany Creek Road (on a logarithmic scale) was plotted against the plotting position (average recurrence interval) of the storms (on a normal distribution scale). A line of best fit was drawn through these annual peak discharge series to determine the anticipated design discharge, for return periods ranging from 2 years to 100 years, as shown in Figure A17 (in Appendix A). A comparison to the peak discharges calculated using the AR&R storm events is also shown in this figure, and demonstrates a high level of consistency with the flood frequency analysis results, especially for the 5 year to 100 year ARI events.

The duration independent synthetic storm for a given average recurrence interval contains the highest intensity bursts of rainfall for all durations. Therefore, one rainfall event can be applied to the entire catchment to determine the peak discharge at all points along a waterway, rather than the large number of rainfall temporal patterns representing the range of standard storm durations presented in Australian Rainfall and Runoff.

However, hydrologic modelling of other catchments has shown that the magnitude of these synthetic storms needs to be factored down to achieve results consistent with the flood frequency analysis discharges.

Therefore, factored synthetic storms were applied to the Albany Creek catchment to achieve calculated discharges consistent with those determined from the flood frequency analysis.

Based on the peak flows predicted by the flood frequency analysis, factors were applied to the duration independent synthetic storms to provide flow rates which agreed as closely as possible with the results of the frequency analysis. The factor derived for each average recurrence interval event is shown in Table 8.2.

Table 8.2: Duration Independent Storm Factors

Average Recurrence Interval	Duration Independent Storm Factor
2 years	0.84
5 years	0.87
10 years	0.87
20 years	0.87
50 years	0.86
100 years	0.82

Based on these results, it is considered that a uniform factor of 0.85 could be applied across all average recurrence interval events.

The peak flow rates determined from the flood frequency analysis, the AR&R storm events, the un-factored duration independent synthetic storms, and the factored duration independent synthetic storms are presented in Figure A18 (in Appendix A).

9.0 HYDROLOGIC MODELLING

9.1 Comparison of Peak Discharges

The peak discharges calculated by the RAFTS hydrologic model were compared to the results from two independent sources: the Rational Method, and flows from the 1991 Albany Creek Flood Study.

As discussed in Section 3.2, Albany Creek (and its tributaries) cross under Old Northern Road via culverts at three locations. At each crossing, the level of Old Northern Road is approximately six metres higher than the invert level of the creek at that point. Consequently, the Old Northern Road culverts attenuate the runoff from the upstream catchments at these locations.

The Rational Method was therefore used to estimate the 100 year peak discharge to Old Northern Road in the two main catchments. The Rational Method cannot be used to calculate flows further downstream in the catchment, due to the flow attenuation which occurs at Old Northern Road (as discussed in Section 3.2).

The Bransby-Williams Method was used to calculate the time of concentration of the catchments. The results are shown in Table 9.1.

Table 9.1: Time of Concentration Calculations

Parameter	Catchments A – H	Catchments J – O
Area (ha)	204.6	111.6
Length (m)	3650	1930
Equal Area Slope (%)	1.3	1.6
Time of Concentration (min)	119	64

Times of concentration of 2 hours and 1 hour were therefore adopted for these two catchments respectively. The 100 year peak discharges from the catchments calculated using the Rational Method, compared to the results from RAFTS model, are shown in Table 9.2.

A comparison of the 100 year peak discharges from the 1991 Flood Study and the RAFTS model is also shown in Table 9.2.

Table 9.2: Comparison of 100 Year Peak Discharges

Location	100 Year Peak Discharge (m³/s)		
	Rational Method	1991 Flood Study	RAFTS Model
Old Northern Rd (Catchment A – H)	34.8	26.4	45.4
Old Northern Rd (Catchment J – O)	28.6	-	38.2
Darien Street	-	73.4	79.5
Albany Creek Road	-	105.9	103.4
Mouth	-	113.8	108.3

These results show that the RAFTS model results are consistent with both the Rational Method results and the 1991 Flood Study.

The Rational Method yields an estimate of the catchment flow at a point. However, the RAFTS model provides a much better definition of the catchment, including channel lengths, channel slopes, subcatchment areas, subcatchment slopes, and subcatchment fractions imperviousness. Thus, it is considered that the RAFTS model results are more robust than the Rational Method flow estimates.

9.2 Hydrologic Model Results

The 100 year peak discharges calculated by the RAFTS model, for both the AR&R storm events and the factored DIS event, are summarised in Table 9.3.

Table 9.3: RAFTS Model 100 Year Peak Discharges

Location (Subarea)	AR&R 100 Year Peak Discharge (m³/s)	AR&R Critical Storm Duration	Factored DIS 100 Year Peak Discharge (m³/s)
Old Northern Rd (H) – Upstream	45.4	2 hour	42.8
Old Northern Rd (H) – Downstream	33.8	2 hour	32.4
Country Club Drive(I)	35.1	2 hour	33.6
Old Northern Rd (O) – Upstream	38.2	1 hour	31.5
Old Northern Rd (O) – Downstream	30.9	1 hour	27.8
Sunningdale Court (P)	33.0	1 hour	29.9
Old Northern Rd (R) – Upstream	4.0	1 hour	3.2
Old Northern Rd (R) – Downstream	2.7	1 hour	2.4
Galaxy Street (T)	11.4	1 hour	9.9
Darien Street (W)	79.5	2 hour	72.2
Albany Creek Road (Z-AD)	103.4	2 hour	95.4
Mouth (AJ)	120.8	2 hour	113.5

A comparison of the 100 year peak discharges calculated by the RAFTS model and the TUFLOW model are summarised in Table 9.4. These results confirm that the two models are providing consistent routing of flows along the waterway.

Table 9.4: 100 Year Peak Discharges – RAFTS and TUFLOW

Location	RAFTS Peak Discharge (m^3/s)	TUFLOW Peak Discharge (m^3/s)
Galaxy Street	11.4	11.3
Darien Street	79.5	79.8
Albany Creek Road	103.4	101.6
Mouth	120.8	119.3

10.0 HYDRAULIC MODELLING

10.1 Model Setup

The setup of the TUFLOW model is described in Section 3.3.

10.2 Modelling Ultimate Waterway Conditions

10.2.1 Existing Waterway Corridors

Waterway corridors are an integral part of the Council's Planning Scheme for Brisbane. City Plan describes waterway corridors as:

> *"The corridors along a waterway indicated on the Planning Scheme maps. These corridors are defined by:*
>
> - *A flood regulation line (FRL)*
> - *A local plan environmental corridor or a waterway corridor (WC)*
> - *A waterway corridor defined in a stormwater management plan*
> - *A waterway corridor defined in a waterway management plan.*
>
> *If more than one of these is available for a particular waterway, the largest applies.*
>
> *If there is no FRL described in local plan, SMP or WMP, a 30 metre distance measured on each side from the centre line of the waterway would apply. (BCC 2000, vol. 1, ch. 3, p. 75)*

These corridors identify zones where water flow, water quality, ecology and open space, and recreational and amenity values are to be preserved and/or managed in an ecologically sustainable manner.

The presence of waterway corridors in the hydraulic model excludes the conveyance and/or storage characteristics of the watercourse beyond the limits of the waterway corridor. Essentially, this practice assumes that filling and development will ultimately occur beyond the boundary of the waterway corridors.

For areas in the Moreton Bay Regional Council area, the Waterway Corridor boundary was applied at the 50 year ARI Albany Creek flood level.

The waterway corridors were included in the Ultimate Scenario model of the creek.

10.2.2 Minimum Riparian Vegetated Corridor

The vegetation along the waterway is described as riparian vegetation and it is a key contributor to waterway health, acting as a buffer between the waterway and the adjacent lands. A well-vegetated riparian zone can improve water quality by filtering overland flow and reducing erosion along creek banks. Shady trees protect vulnerable organisms from extremes of temperature; root systems and woody debris become habitat for fauna; and organic matter sustains aquatic food webs. Vegetation also provides habitat and forage for fauna and adds to a waterway's recreational value.

This study calculates anticipated flood levels assuming a minimum vegetated riparian corridor width along the entire creek system. The hydraulic investigation does not in any way imply that Council is planning to establish a minimum riparian vegetated corridor width in the creek catchment. The minimum riparian vegetated corridor is modelled solely in recognition that at some specified time in the future, revegetation may occur, either through natural regeneration or as a result of planting programs. The results of this modelling are intended to ensure that the habitable floor levels of developments within the flood plain take account of possible future revegetation.

Minimum riparian corridors have been applied to main branches of all reaches modelled in the hydraulic model. The minimum riparian corridor was simulated as dense vegetation by applying a 'Manning's n' value of 0.15, extending from the top of the low flow banks for a minimum width of 15m on both sides of the creek. Where there was no obvious low flow channel, the vegetation was applied at the anticipated 2 year ARI flood level on the basis that this size event is generally contained within the bed and banks of the creek. Where the existing Manning's n value of the creek in the vicinity of the minimum riparian corridors was higher than 0.15, the existing value was not altered.

The hydraulic model was therefore revised to examine the impact of the proposed minimum riparian corridors and waterway corridors on flood levels.

It is noted that apart from a small number of isolated locations (i.e. just upstream and just downstream of Wendon Way, just upstream and just downstream of Albany Creek Road), Albany Creek already provides a heavily vegetated corridor along the majority of its length.

11.0 FINAL MODEL RESULTS

11.1 Flood Levels and Discharges

The peak flood levels and discharges along the creek, assuming Ultimate Development Scenario (i.e. ultimate catchment development, waterway corridors, and minimum riparian zone) are detailed for the 2 year to 100 year average recurrence interval events in Table 11.1 and Table 11.2. The Adopted Middle Thread Distance (AMTD) line is shown in Figure A20.

Table 11.1: Peak Flood Levels – Ultimate Scenario

AMTD (m) / Location	Peak Flood Level (mAHD)					
	2 Year	5 Year	10 Year	20 Year	50 Year	100 Year
0	7.75	7.75	7.75	7.75	7.75	7.75
500	9.91	10.14	10.30	10.44	10.60	10.74
MHG100	10.54	10.77	10.90	11.01	11.14	11.24
1000	11.89	12.09	12.22	12.33	12.48	12.59
1500	13.82	14.04	14.17	14.30	14.45	14.59
2000 D/S Albany Creek Rd	16.05	16.18	16.23	16.29	16.36	16.41
U/S Albany Creek Rd	16.14	16.29	16.35	16.41	16.47	16.52
MHG110	16.25	16.39	16.45	16.51	16.58	16.63
2500	18.45	18.75	18.87	19.02	19.22	19.34
3000	20.21	20.48	20.57	20.72	20.87	20.96
3500	22.59	22.80	22.87	22.98	23.10	23.16
4000	25.01	25.21	25.26	25.34	25.42	25.46
MHG120	25.28	25.50	25.56	25.66	25.77	25.82
D/S Wendon Way	25.86	26.08	26.16	26.30	26.44	26.59
U/S Wendon Way	25.91	26.17	26.27	26.46	26.69	26.88
4500	26.78	26.89	26.93	27.02	27.13	27.21
5000	29.86	30.09	30.16	30.31	30.45	30.54
D/S Peterson Pl	26.18	26.41	26.50	26.69	26.90	27.05
U/S Peterson Pl	26.22	26.47	26.60	26.82	27.02	27.12
D/S Galaxy St	35.54	35.65	35.72	35.80	35.88	35.95
U/S Galaxy St	35.70	35.90	36.03	36.19	36.38	36.54

Table 11.2: Peak Discharges – Ultimate Scenario

Location	Peak Discharge (m^3/s)					
	100 Year	50 Year	20 Year	10 Year	5 Year	2 Year
Outlet	38.6	55.6	68.3	81.4	99.1	113.9
Albany Ck Rd (total)	35.7	50.2	60.1	70.3	86.5	99.1
Darien Street	29.2	42.5	47.9	58.7	71.6	79.8
Wendon Way (culvert)	33.3	41.8	47.2	57.6	70.2	78.1
Peterson Pl (culvert)	7.0	9.9	11.7	14.3	17.1	18.4
Galaxy St (culvert)	4.0	5.5	6.7	8.1	9.8	11.2

The 200, 500 and 2000 year average recurrence interval events were also modelled. Rainfall data for the 200 and 500 year ARI events were determined using CRC-FORGE. The rainfall depths for each storm event are shown in Table 11.3. For the 2,000 year ARI event, a 6 hour duration “superstorm” was provided by Council for use in the analysis.

Table 11.3: CRC-FORGE Rainfall Depths

Duration (hours)	Rainfall Depth (mm)		
	200 Year ARI	500 Year ARI	2,000 Year ARI
1	126.6	147.3	182.2
1.5	145.0	168.7	208.7
2	159.7	185.8	229.8
3	182.9	212.8	263.3
4.5	208.4	242.5	300.0
6	228.6	266.1	329.1

The tailwater level adopted for these events was the same as that used for the other TUFLOW modelling.

The peak flood levels and discharges for the 200 and 500 year average recurrence interval events (assuming ultimate catchment development, minimum riparian zone and waterway corridors are shown in Table 11.4 and 11.5 respectively. The modelling assumed filling of areas outside the waterway corridor to a level of 300 mm above the ultimate 100 year ARI flood level.

Table 11.4: Peak Flood Levels – Ultimate Scenario

AMTD (m) / Location	Peak Flood Level (mAHD)	
	200 Year	500 Year
0	7.75	7.75
500	10.84	11.04
MHG100	11.33	11.48
1000	12.67	12.84
1500	14.68	14.85
2000 – D/S Albany Creek Rd	16.44	16.52
U/S Albany Creek Rd	16.55	16.63
MHG110	16.67	16.75
2500	19.43	19.62
3000	21.01	21.16
3500	23.22	23.32
4000	25.50	25.56
MHG120	25.87	25.95
D/S Wendon Way	26.67	26.76
U/S Wendon Way	27.01	27.12
4500	27.29	27.38
5000	30.60	30.71
D/S Peterson Pl	27.19	27.30
U/S Peterson Pl	27.25	27.38
D/S Galaxy St	36.02	36.10
U/S Galaxy St	36.73	37.10

Table 11.5: Peak Discharges – Ultimate Scenario

Location	Peak Discharge (m^3/s)	
	200 Year	500 Year
Outlet	128.2	152.3
Albany Ck Rd (total)	109.2	130.5
Darien Street	86.7	100.1
Wendon Way (culvert)	81.6	84.4
Peterson Pl (culvert)	19.4	20.3
Galaxy St (culvert)	13.0	15.0

11.2 Discussion of Results

The results of the flood study, assuming the Ultimate Development Scenario, show that there are only a few locations where flooding presents a problem. These locations are discussed below.

- ***Nursery/Landscaping Property downstream of Albany Creek Road***
  Flood inundation occurs within this property for all events greater than or equal to the 2 year ARI event. The flooding produces depth-velocity values less than 0.4 m^2/s through the majority of the site for the 100 year ARI event.

- ***Albany Creek Road***
  Albany Creek Road is overtopped for all events greater than or equal to the 2 year ARI event. The largest depth of flooding occurs just to the east of Albany Creek. This depth-velocity values are between 0.6 and 1.0 m^2/s in this area for the 100 year ARI event.

- ***Peterson Place***
  Peterson Place is overtopped for events greater than or equal to the 50 year ARI event. However, the depth of flooding over the road in the 50 year and 100 year ARI event is approximately 50 mm and 150 mm respectively. Thus, the road is still trafficable for these events.

The 100 year ARI water surface level along Albany Creek (assuming the Ultimate Development Scenario) was compared to the profile calculated as part of the 1991 Flood Study (which assumed existing waterway conditions and flood regulation lines). Thus, the 1991 Flood Study did not assume a minimum riparian vegetated corridor along the creek.

The water surface level comparison is shown in Figure A19 (in Appendix A). In summary, the TUFLOW model results are generally higher than the results from the 1991 Flood Study, with most differences in the range of 0 to 600 mm. However, as discussed above, the 1991 Flood Study did not assume a minimum riparian vegetated corridor along the length of Albany Creek.

11.3 Flood Mapping

The flood levels and depths for the 2 year to the 100 year average recurrence interval events are shown on Figures 1 to 12 (in Volume 2). The flood levels for the 200 year and 500 year average recurrence interval events are shown on Figures 13 and 14 (in Volume 2). These results are based on ultimate catchment development, minimum riparian zone and waterway corridors. However, for the purposes of the flood mapping, the waterway corridors were assumed to be removed and the flood inundation was stretched out over the adjacent topography until it intersected the ground level.

This was achieved by using Vertical Mapper (within MapInfo) to extrapolate the flood inundation extent beyond the waterway corridor boundary, then trimming the results at the point where the extrapolated flood level intersected the ground level. This methodology stretches the flood inundation over the adjacent topography, but the limitations of this approach include:

- flood levels in overland flowpaths outside the waterway corridors may not be properly represented, as they have not been hydraulically modelled; and
- flood levels near intersections of different waterways may not be properly represented, as the interaction of flow outside the waterway corridors has not been hydraulically modelled.

The extent of inundation for the 2,000 year average recurrence interval event (assuming ultimate catchment development, and existing waterway conditions) is shown in Figure 15 (in Volume 2). The extents of inundation for the 2 year to the 500 year average recurrence interval events are shown in Figures 16 to 23 (in Volume 2).

12.0 REFERENCES

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Appendix A Part A – Model Calibration

Figures

Tables

ALBANY CREEK FLOOD STUDY

Cardno

BRISBANE CITY

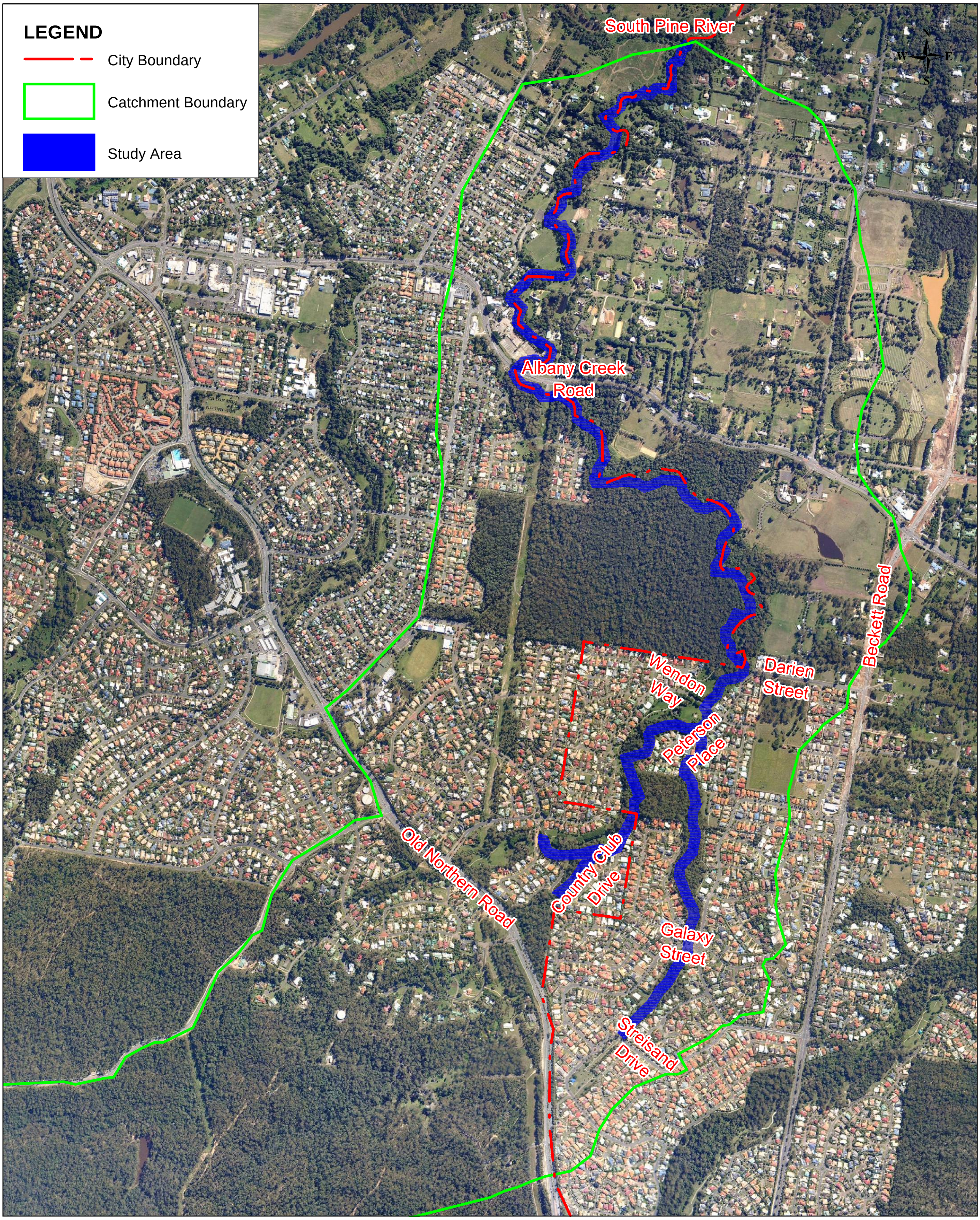

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FIGURE A1
EXTENT OF STUDY

Rev: Orig   Date: 06 July 2011

Project No: J11039

Water Resources Branch

Print Date: 30-8-2012

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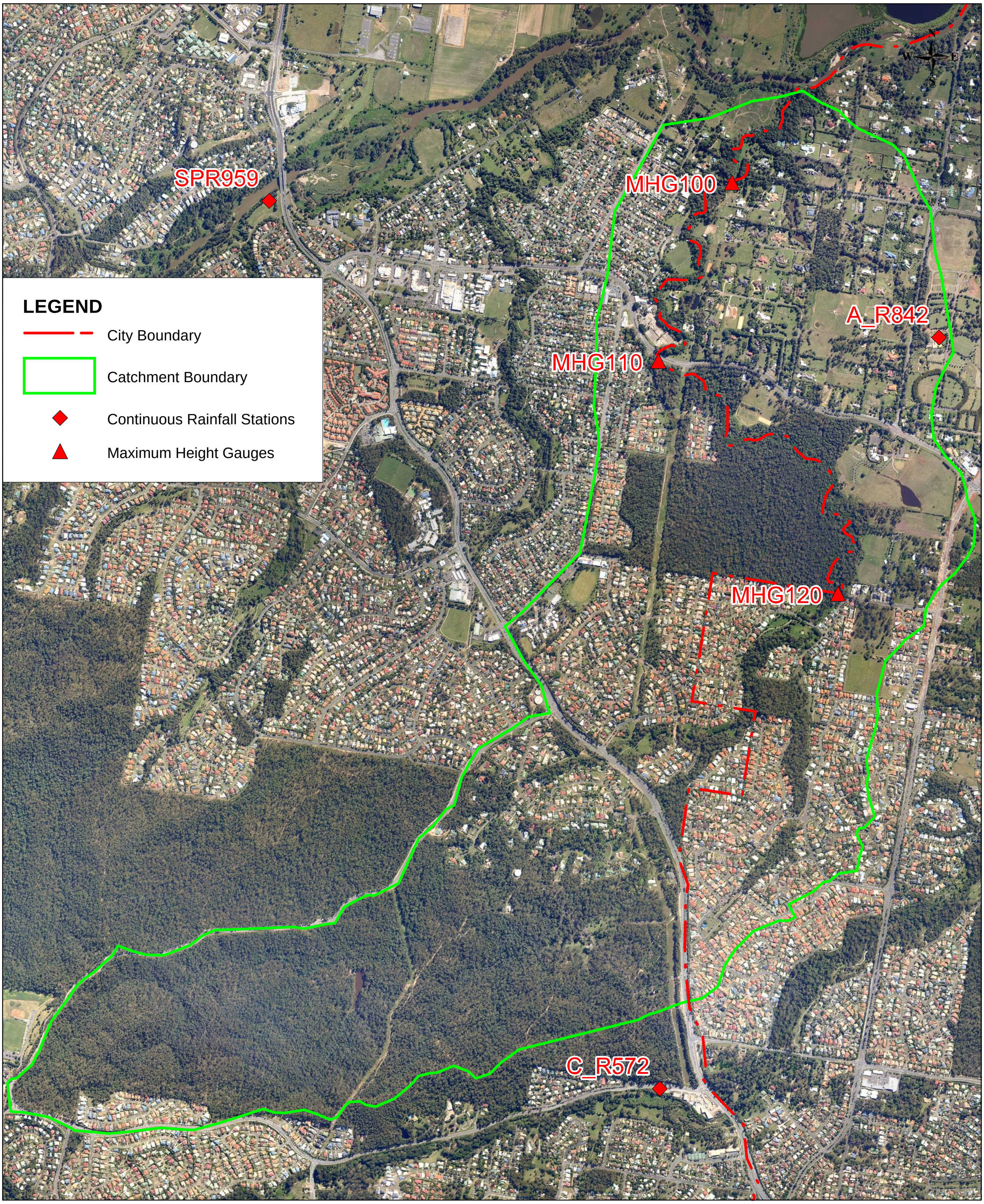

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FIGURE A2
GAUGE LOCATIONS

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Water Resources Branch

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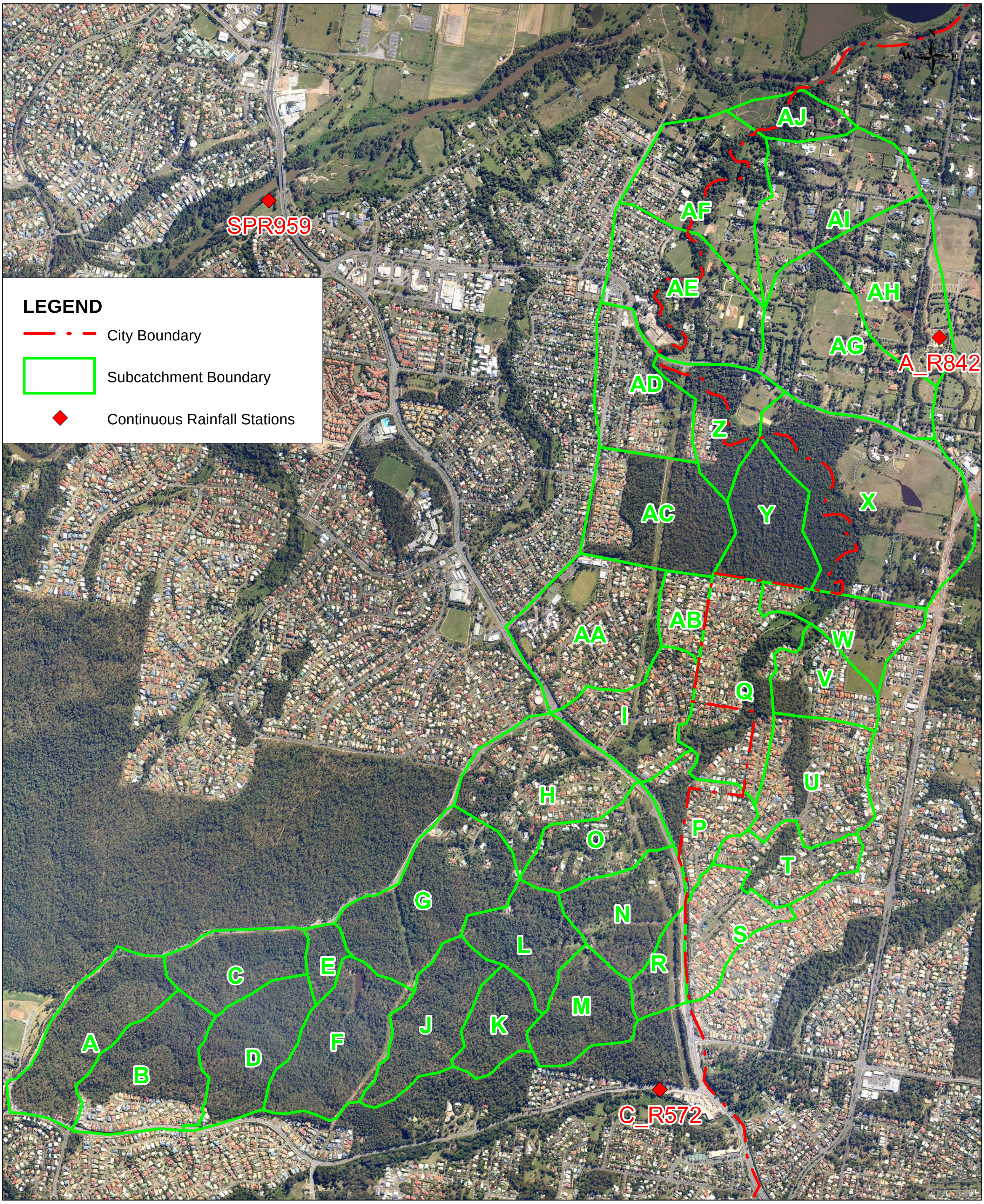

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FIGURE A3
RAFTS MODEL

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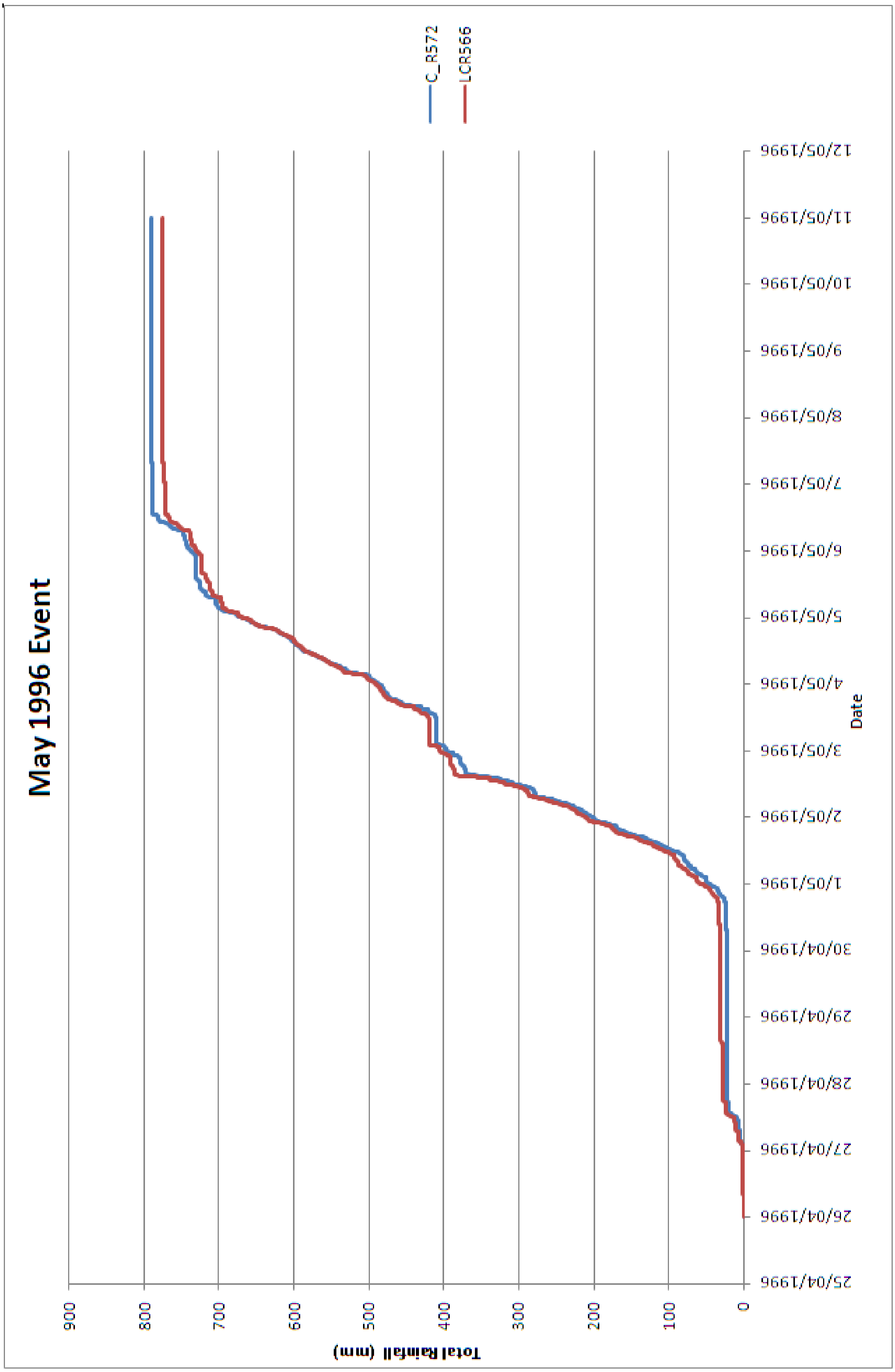


Figure A4. Cumulative Rainfall, May 1996

Figure A5. Cumulative Rainfall, May 2009

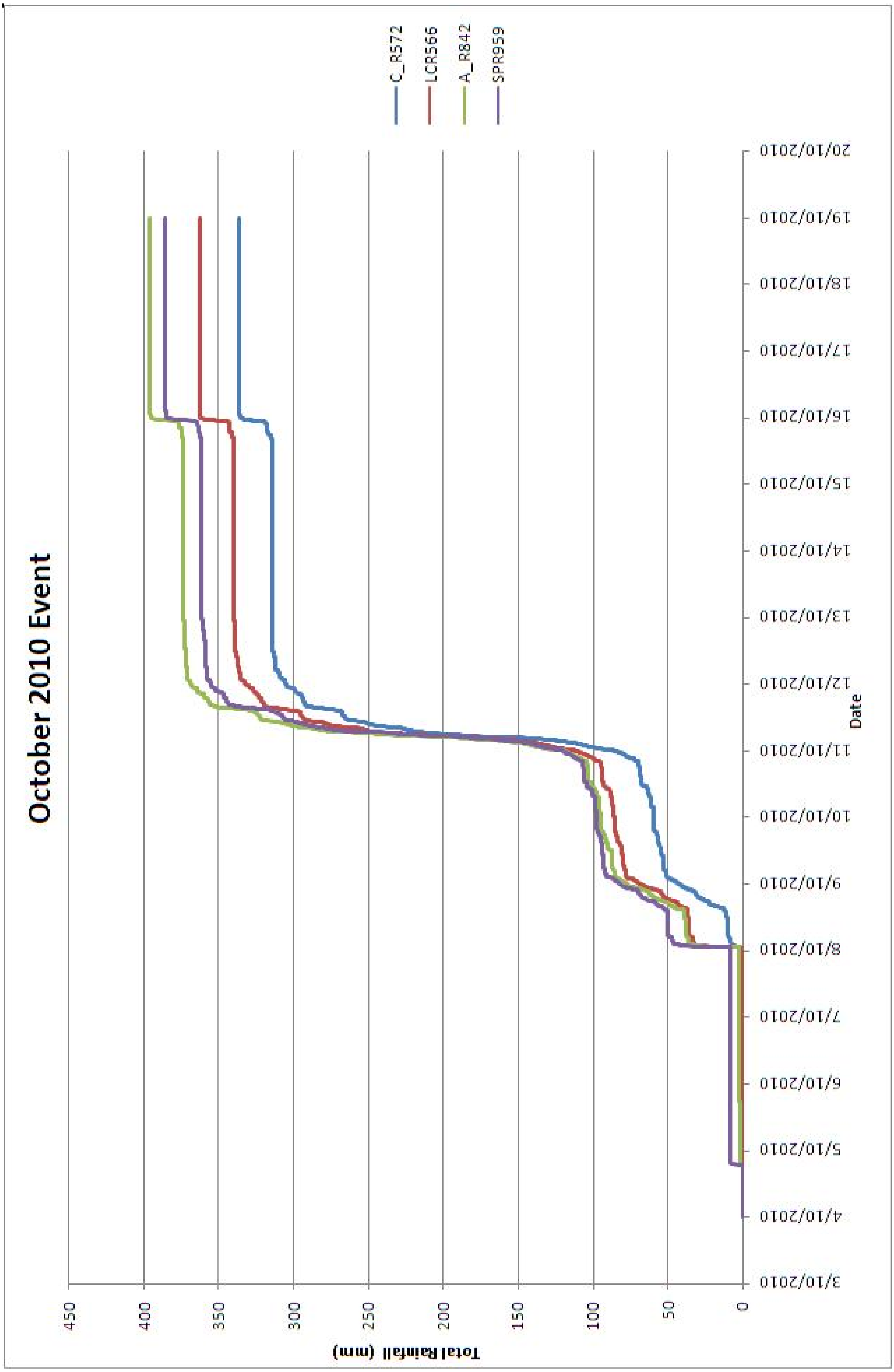


Figure A6. Cumulative Rainfall, October 2010

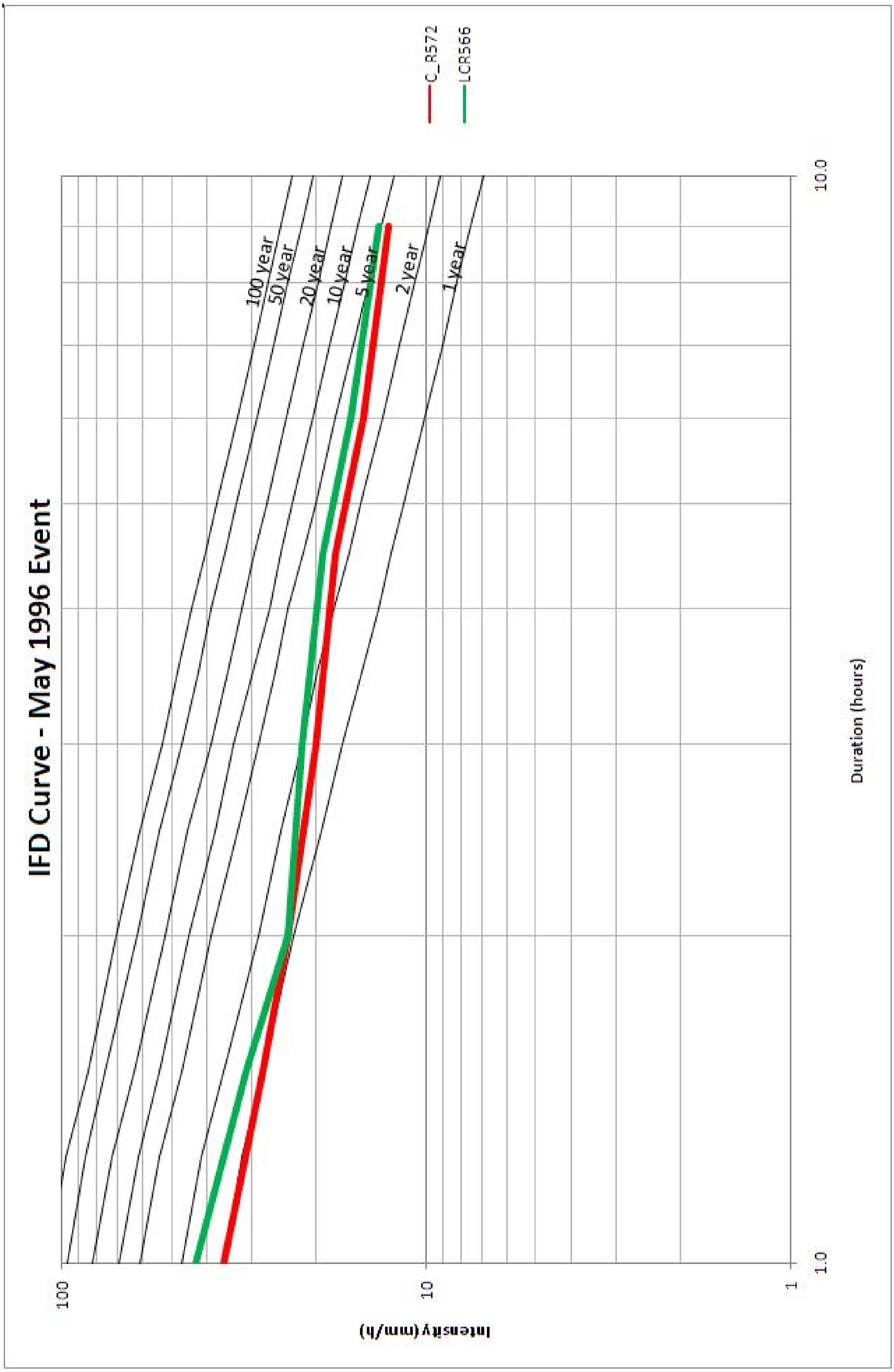


Figure A7. IFD Chart, May 1996

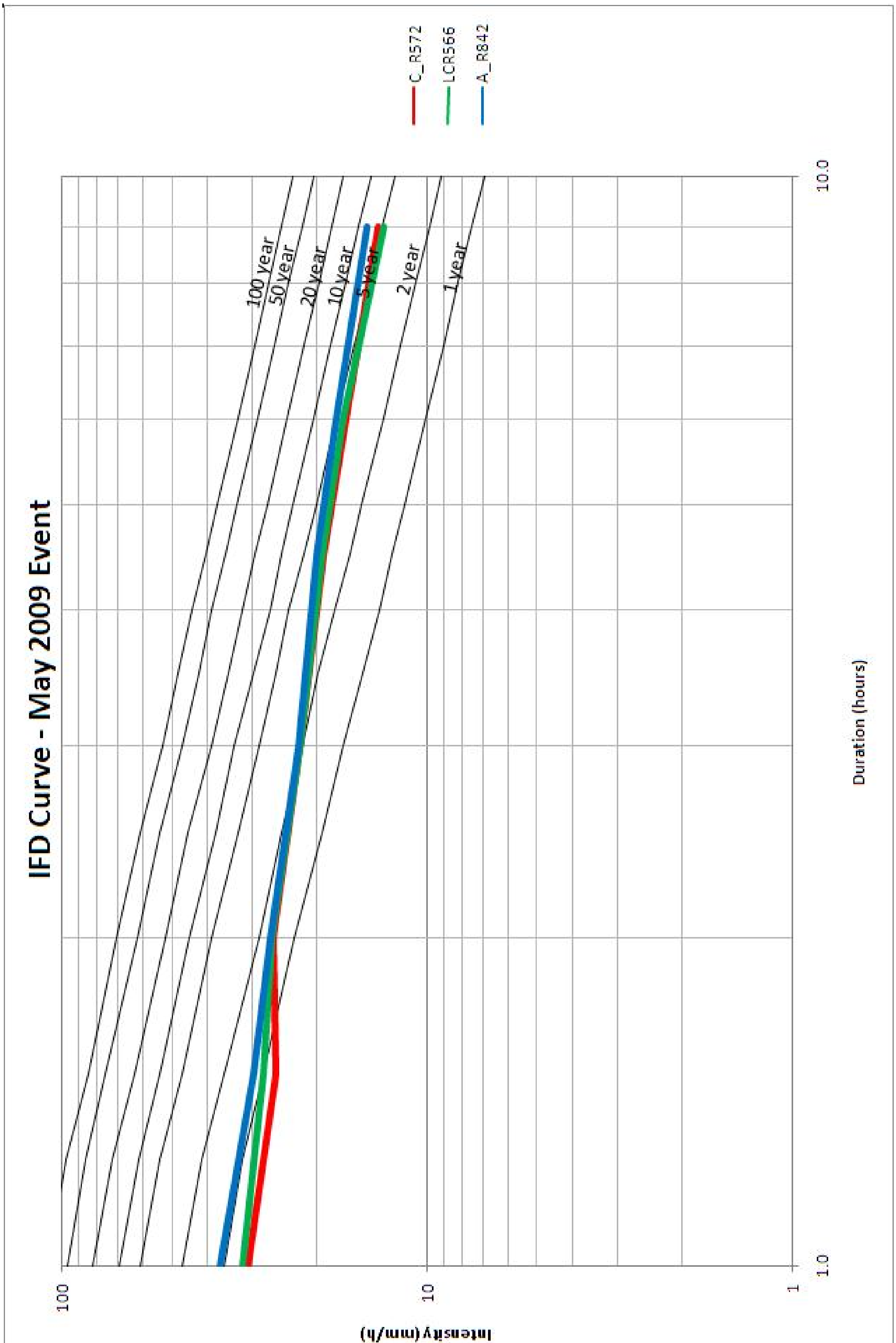


Figure A8. IFD Chart, May 2009

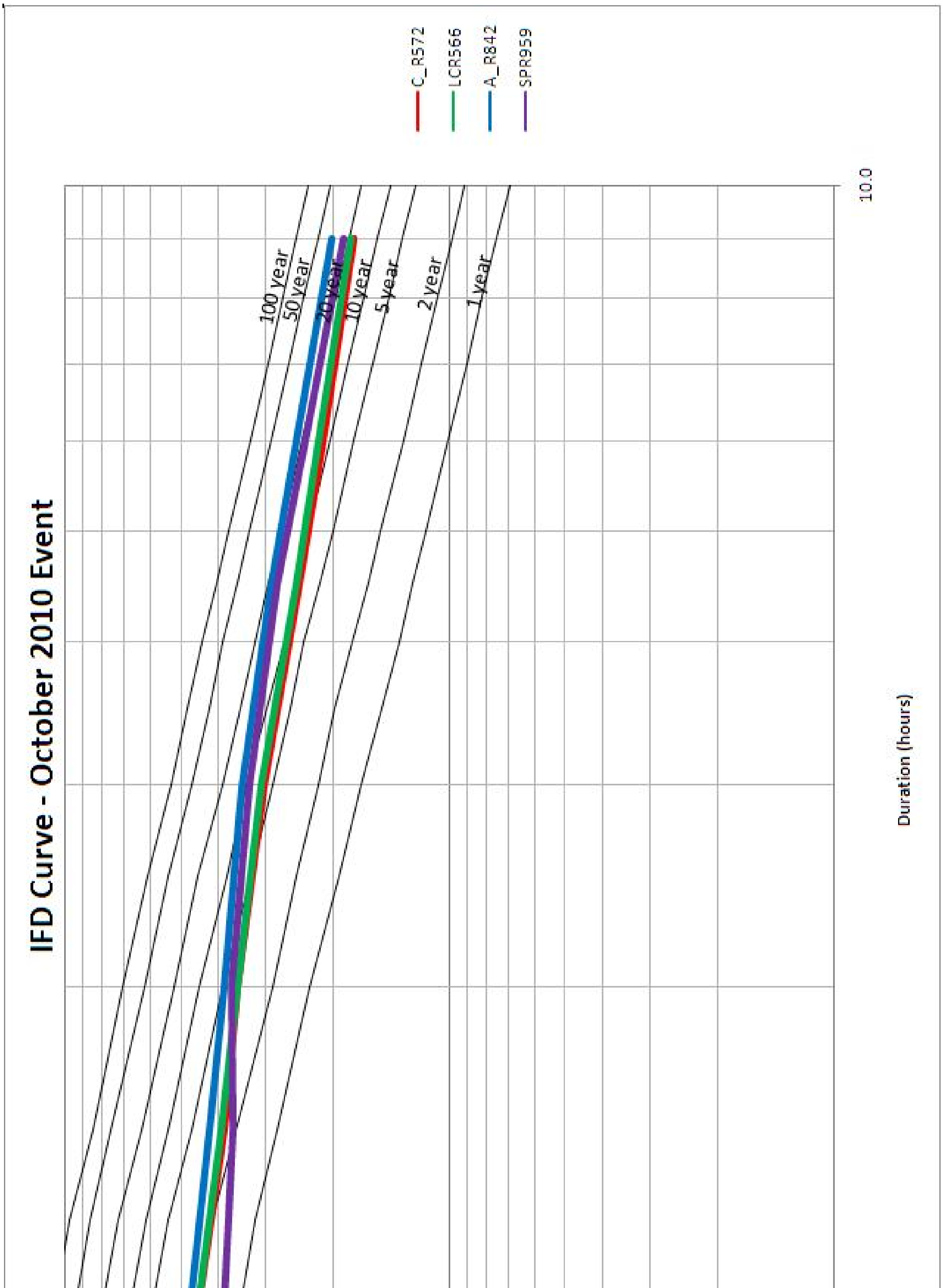


Figure A9. IFD Chart, October 2010

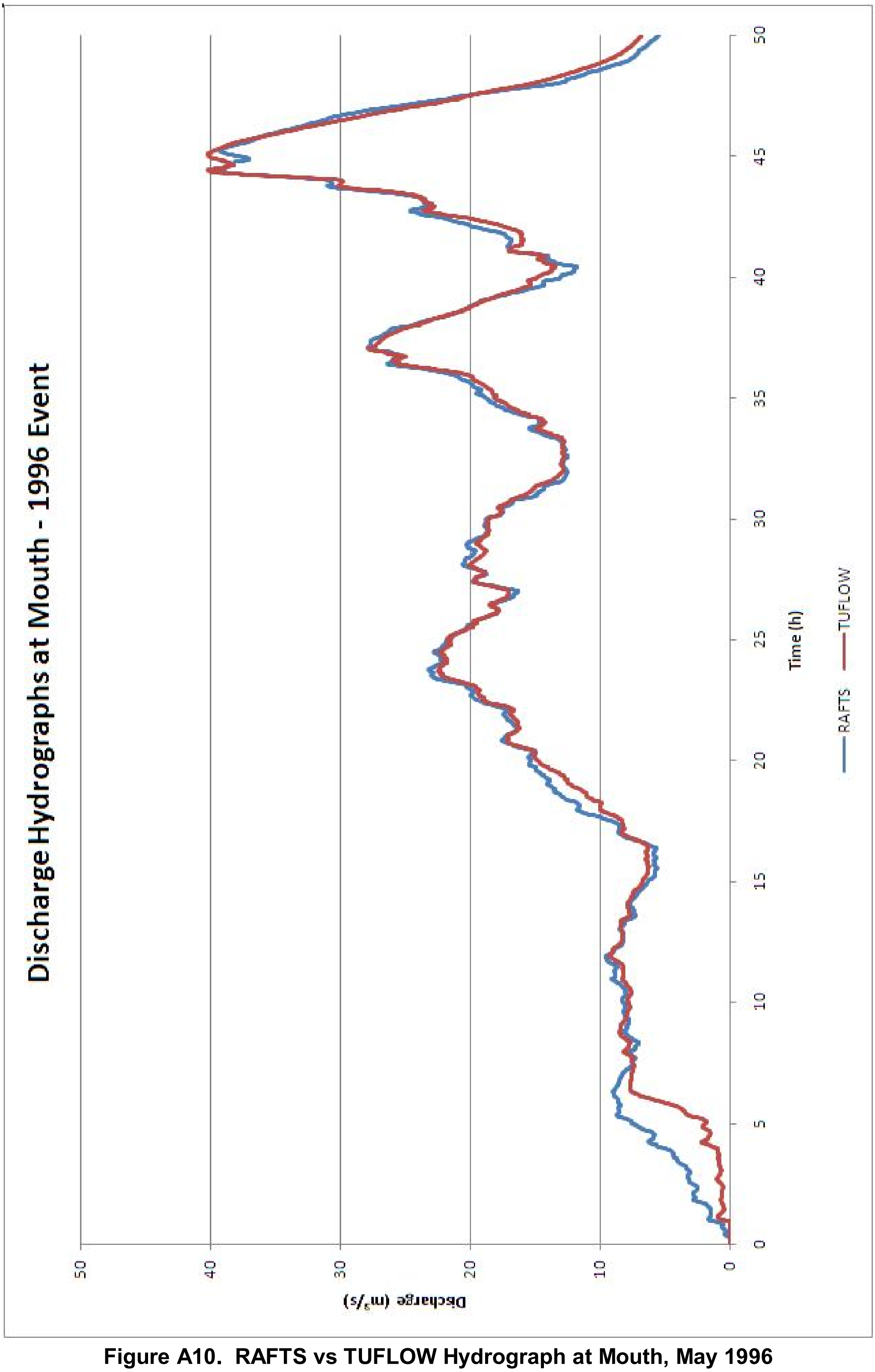


Figure A10. RAFTS vs TUFLOW Hydrograph at Mouth, May 1996

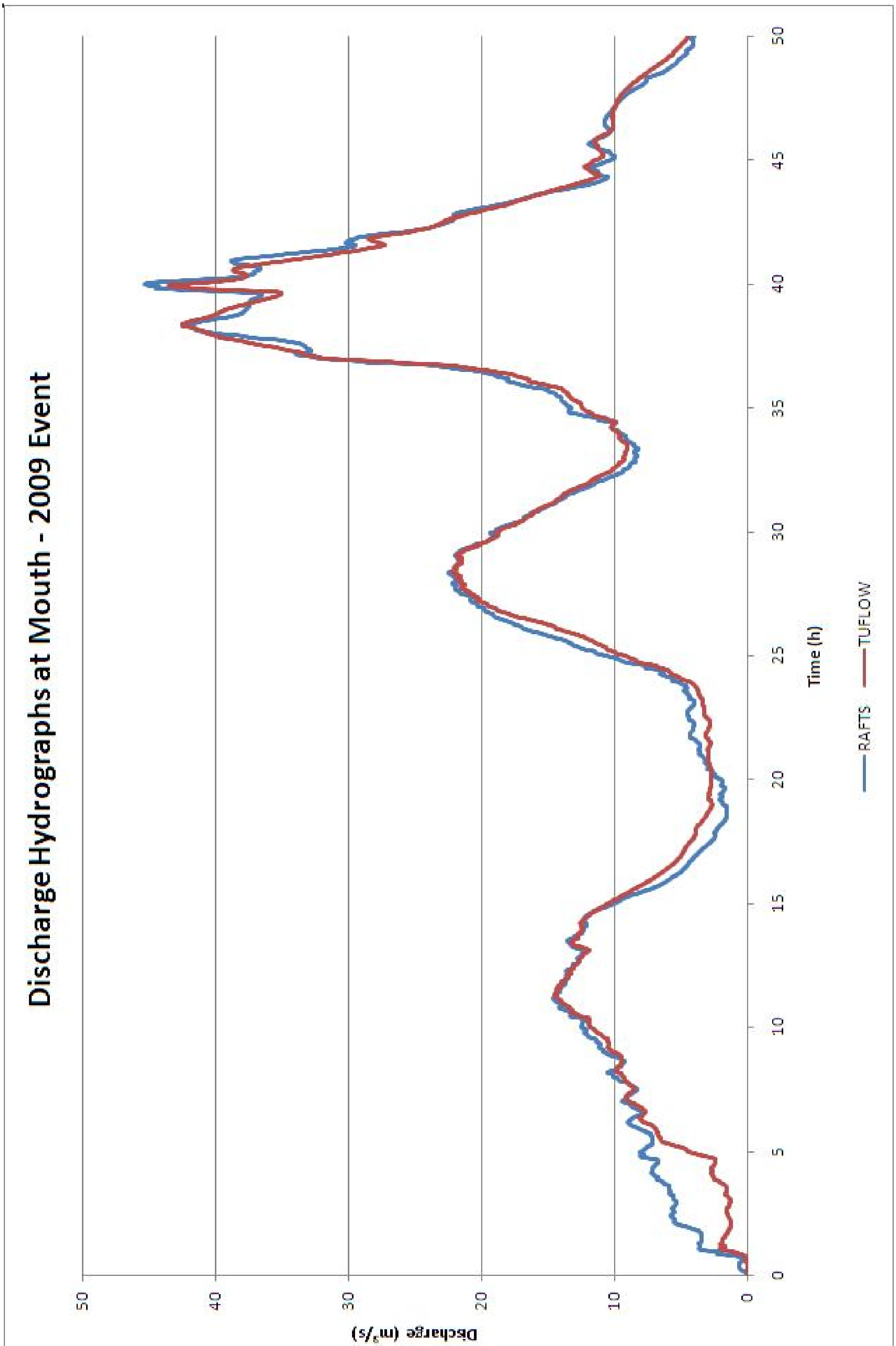


Figure A11. RAFTS vs TUFLOW Hydrograph at Mouth, May 2009

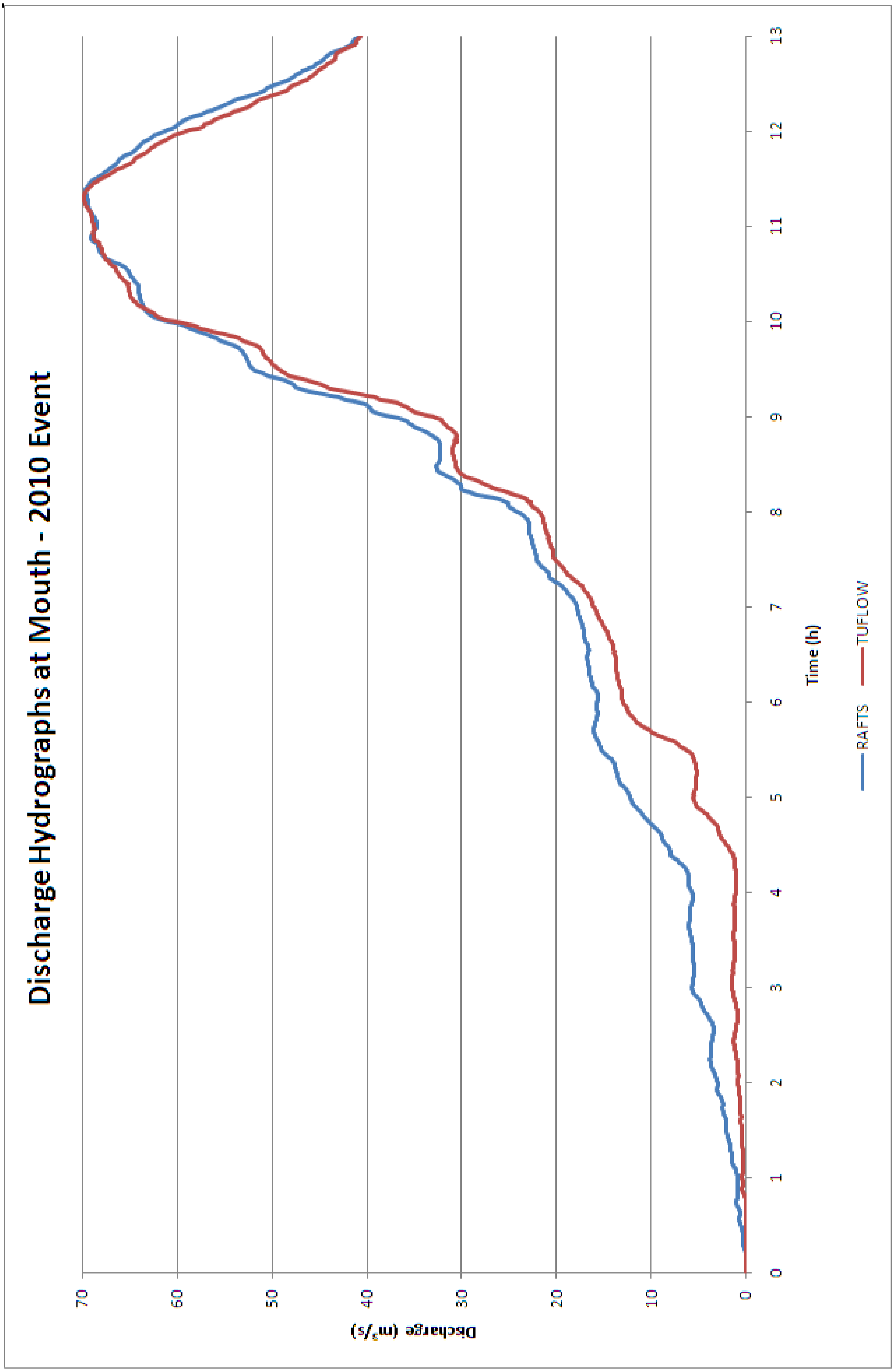


Figure A12. RAFTS vs TUFLOW Hydrograph at Mouth, October 2010

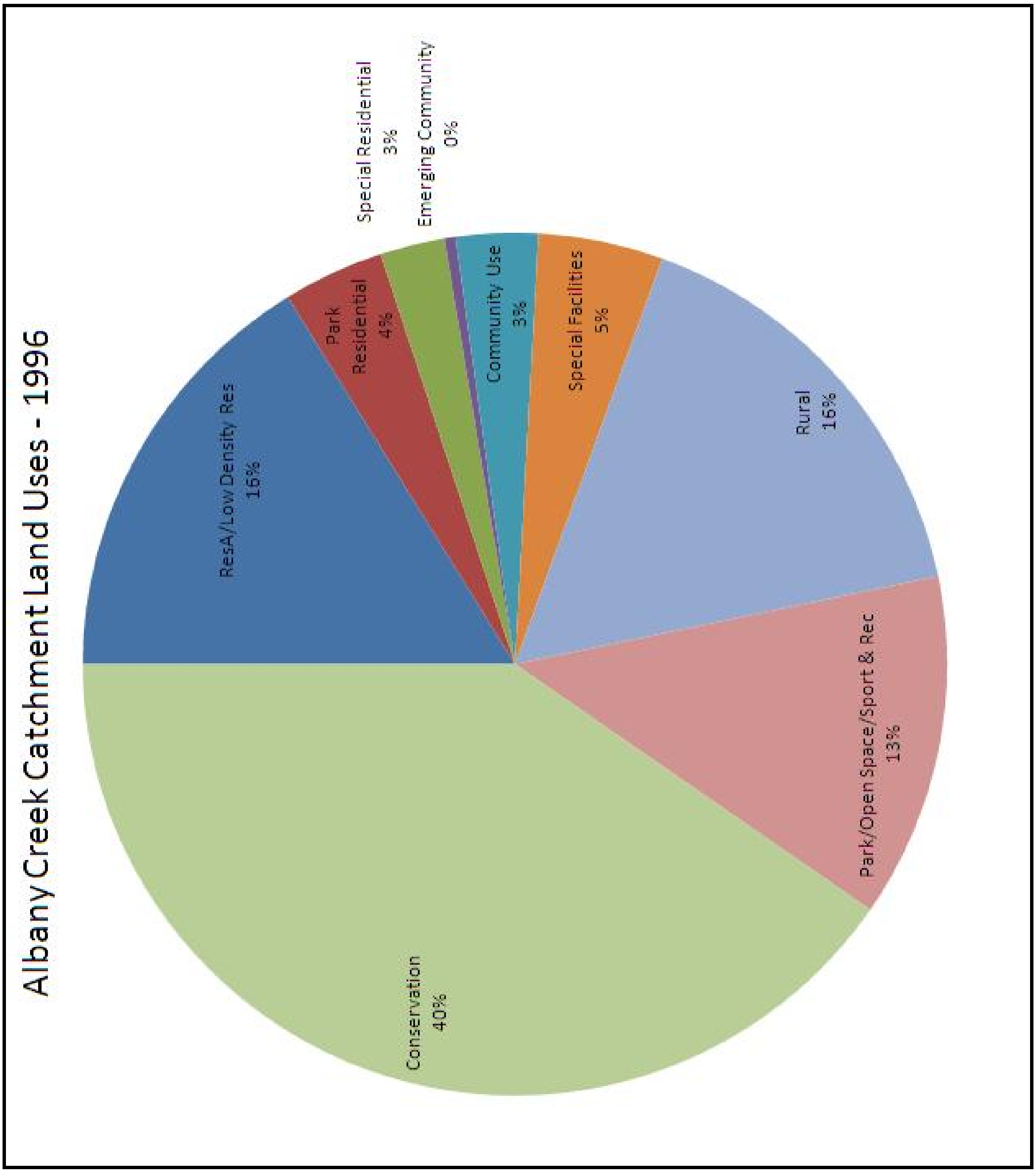


Figure A13. Albany Creek Catchment Land Uses – 1996

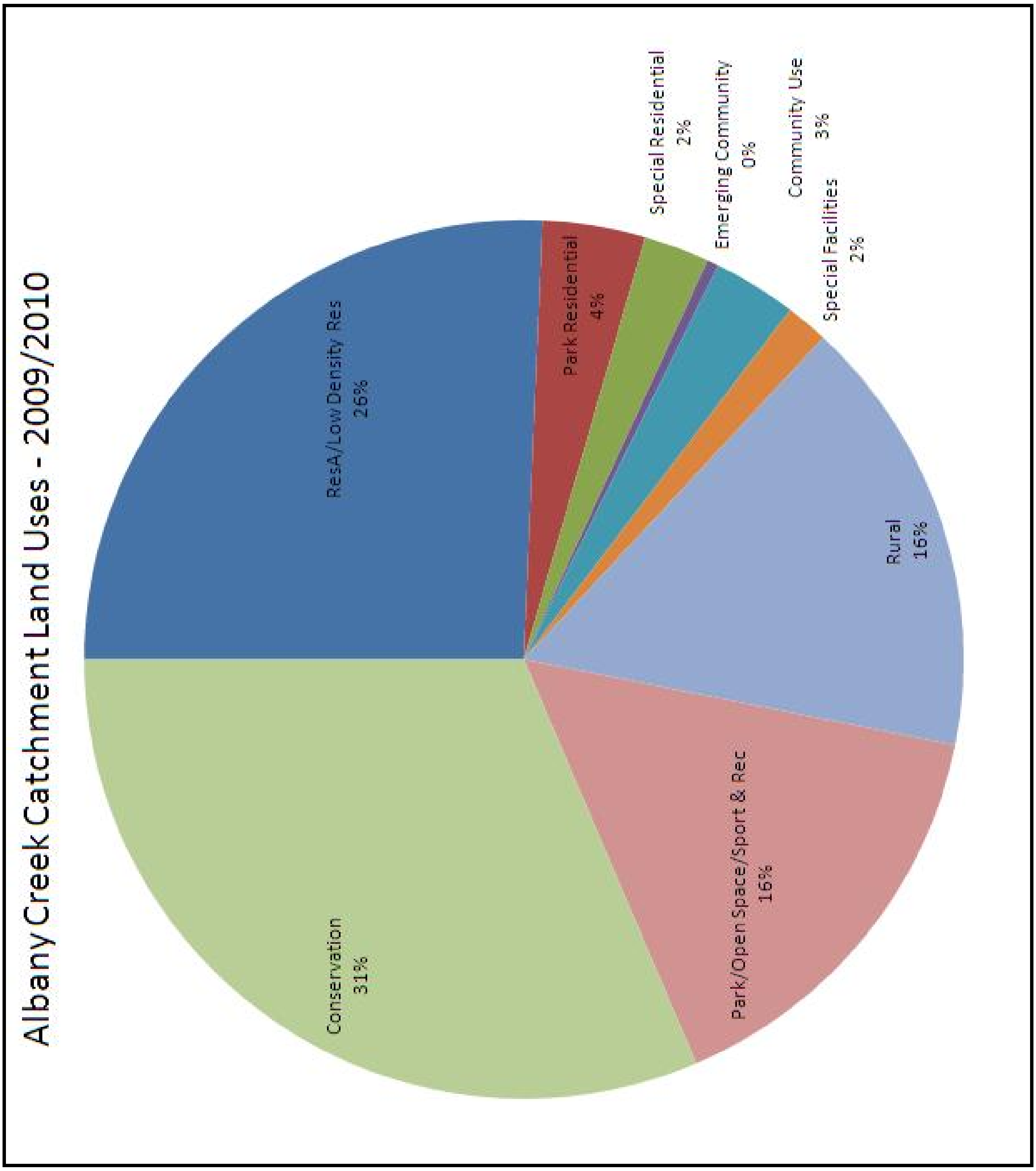


Figure A14. Albany Creek Catchment Land Uses – 2009/2010

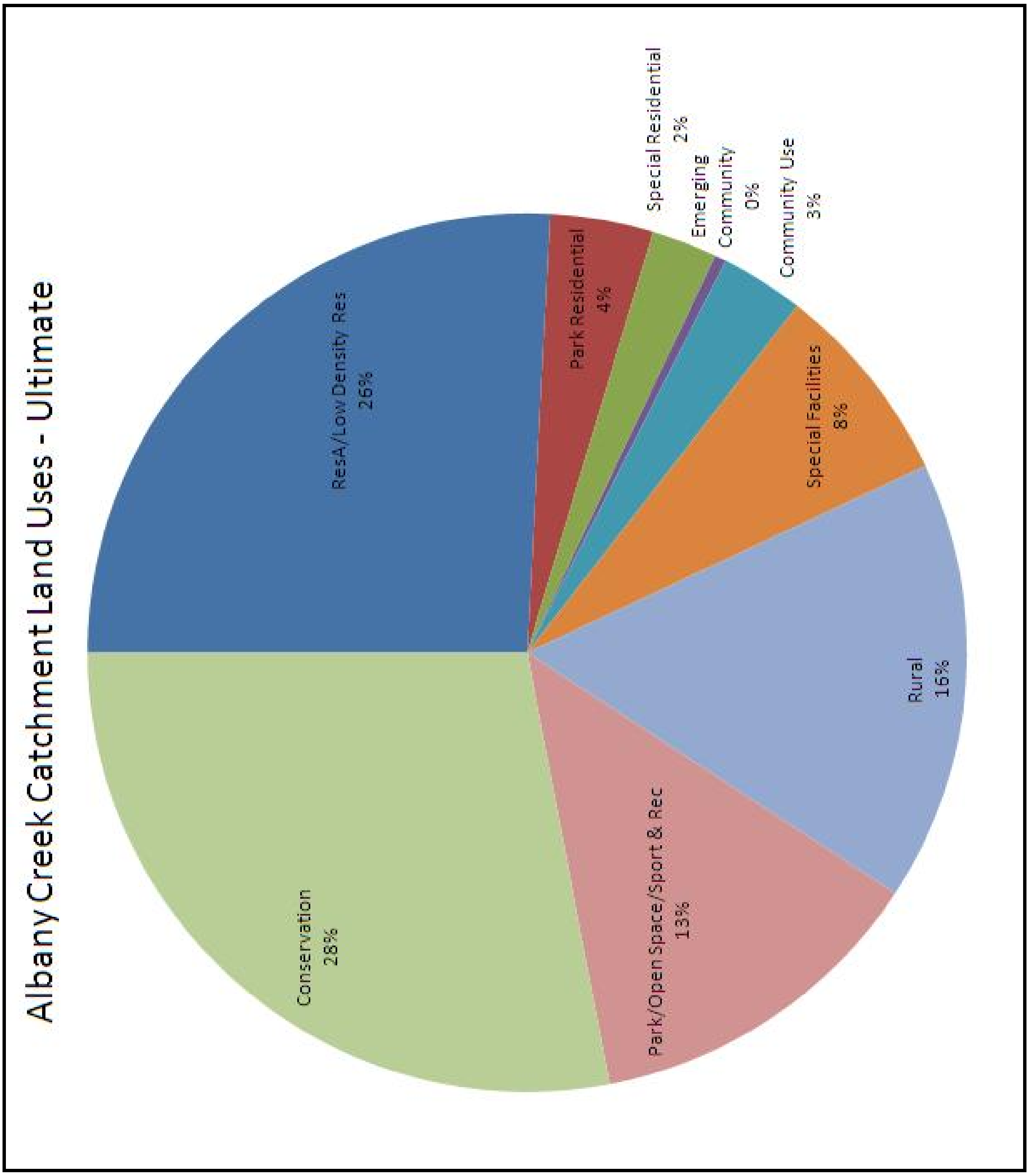


Figure A15. Albany Creek Catchment Land Uses – Ultimate

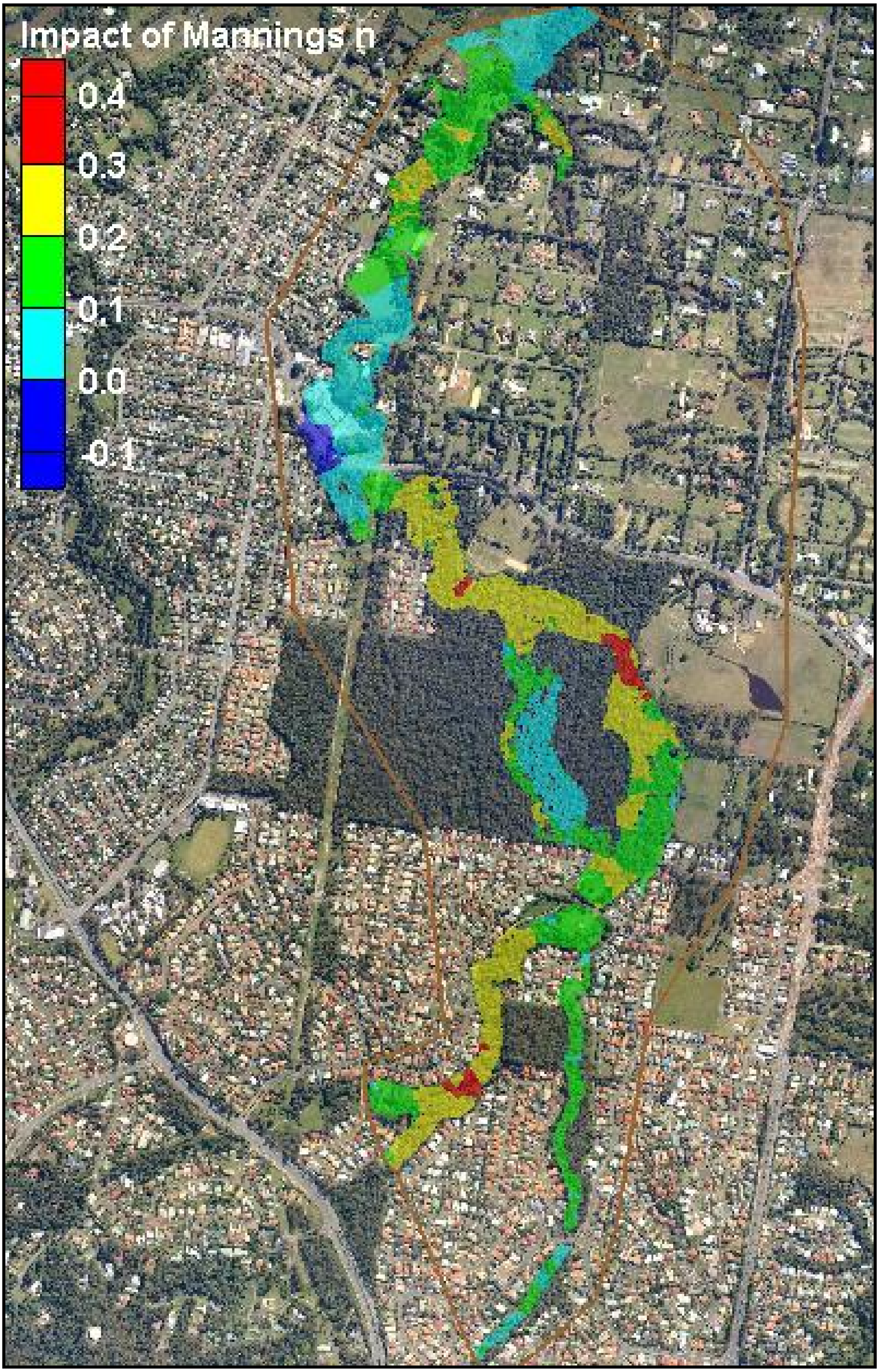


Figure A16. Impact of Manning's n on 100 Year Flood Levels

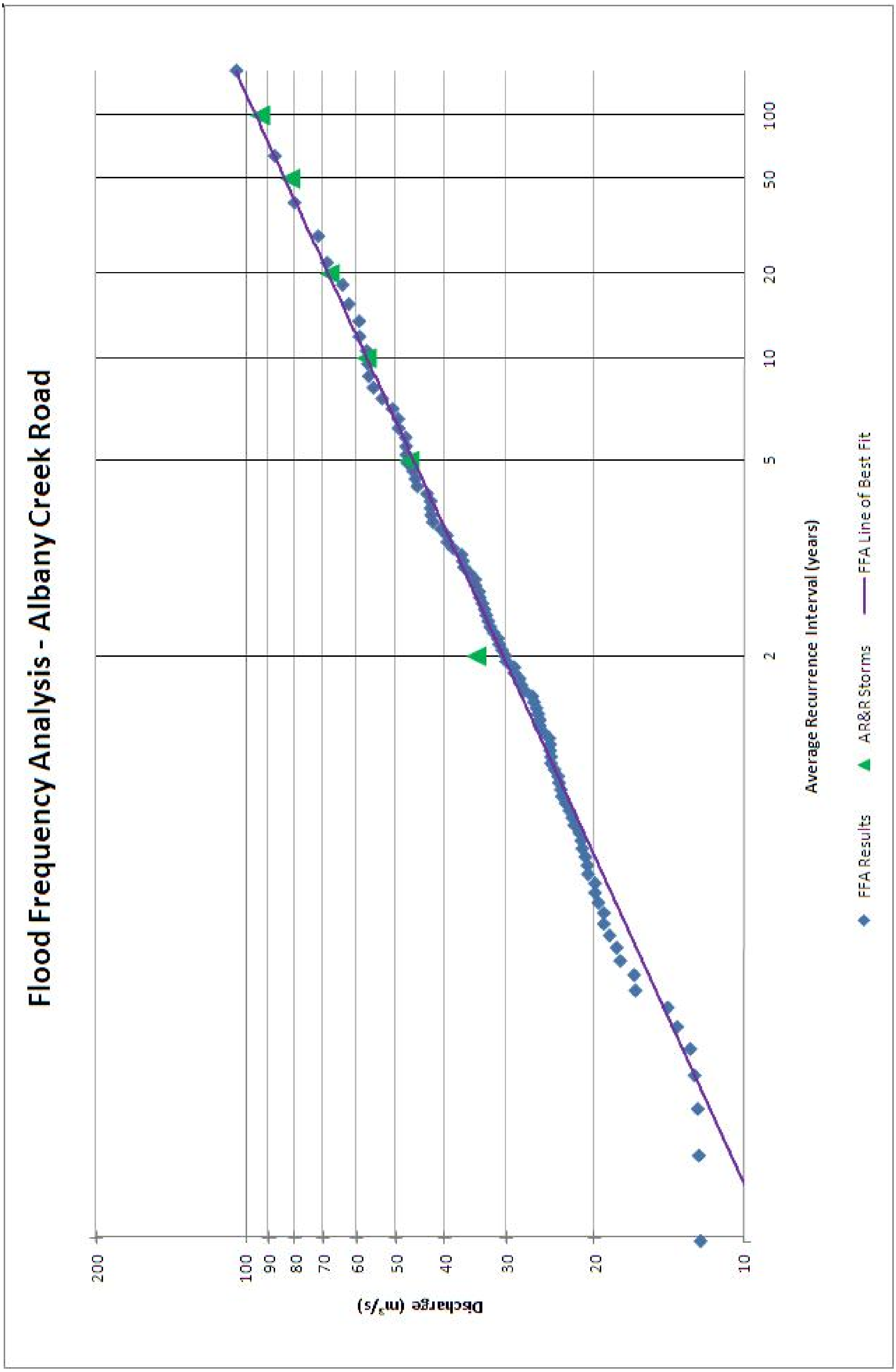


Figure A17. Flood Frequency Analysis – Albany Creek Road

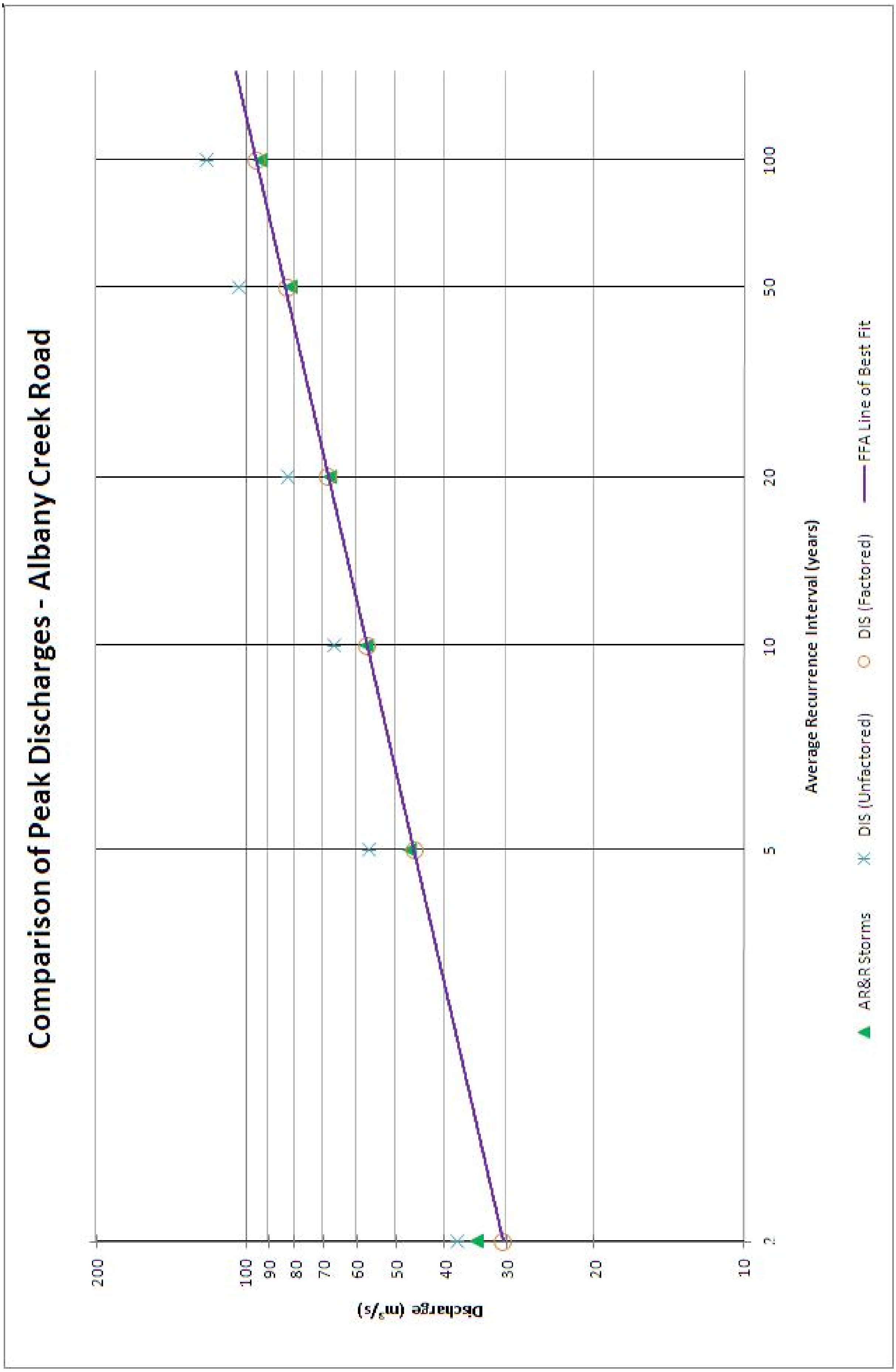


Figure A18. Comparison of Peak Discharges – Albany Creek Road

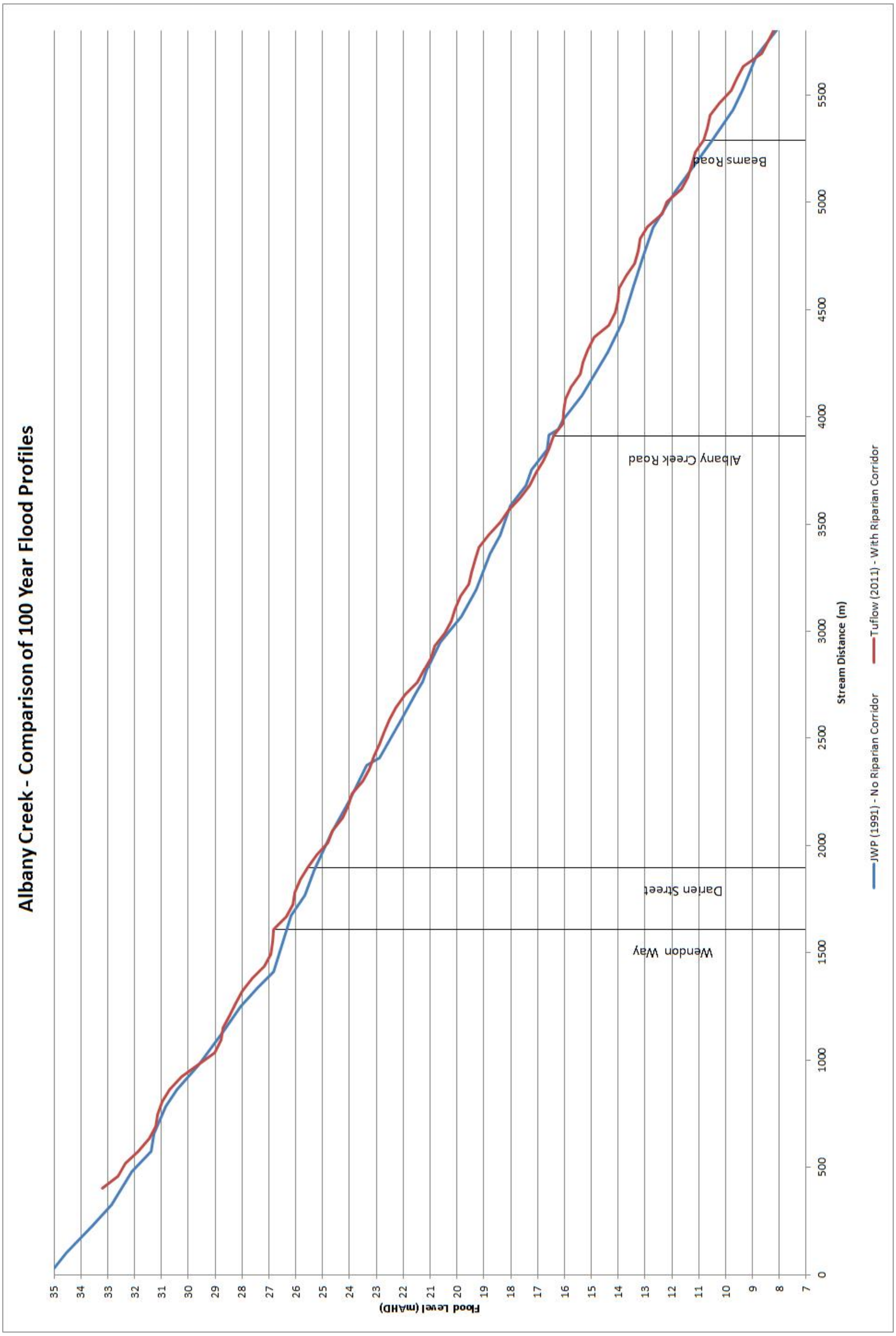
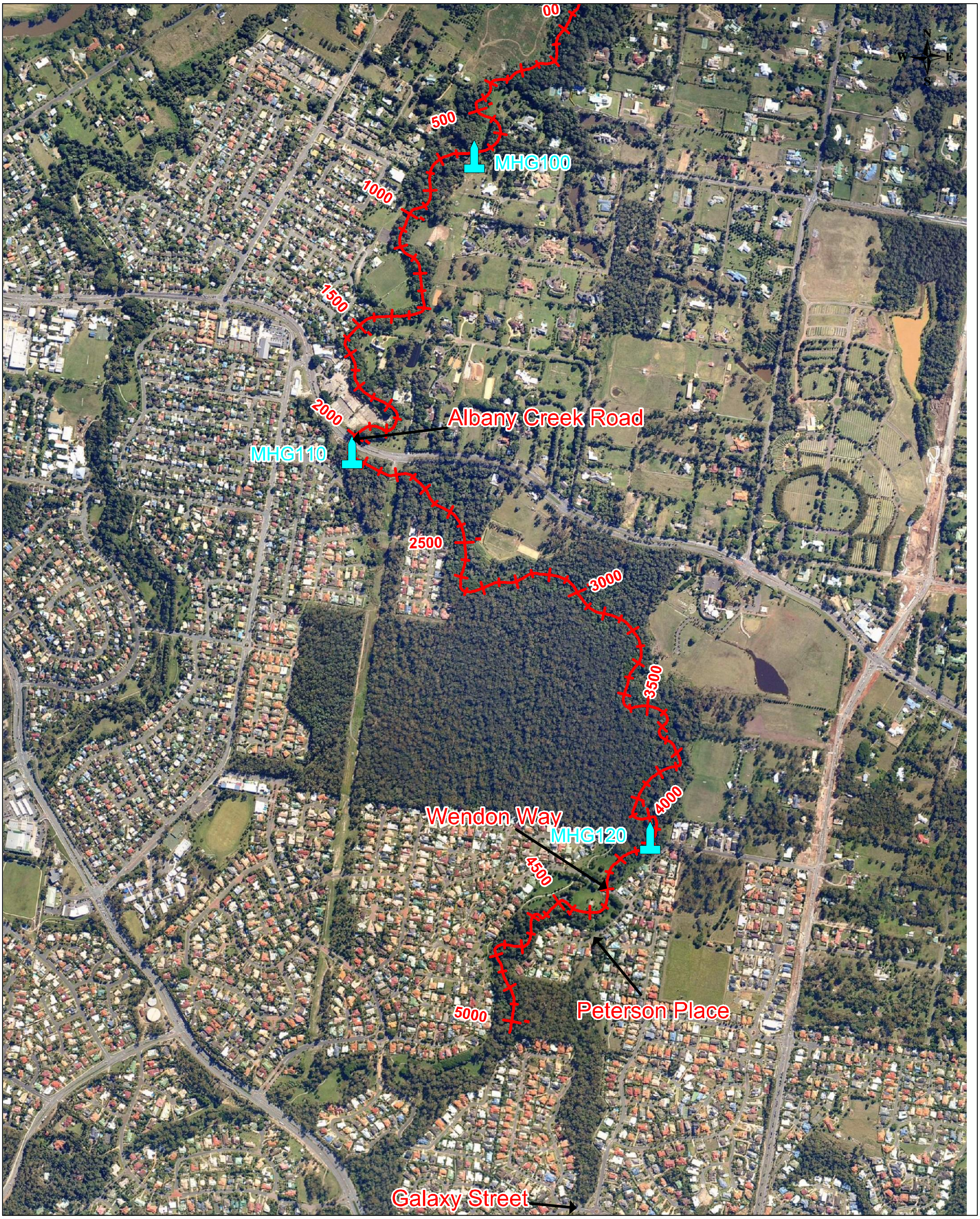


Figure A19. Comparison of 100 Year ARI Water Surface Profiles



Scale: 1:15,000

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FIGURE A20.
RESULT OUTPUT LOCATIONS

Rev: Orig Date: 06 July 2011

Project No: J11039

Table A1: RAFTS Model Details – 1996

Sub catchment Name	Area (ha)	Land Use Area (ha)									Fraction Impervious
		ResA/Low Density Res	Park Residential	Special Residential	Emerging Community	Community Use	Special Facilities	Rural	Park/Open Space/Sport & Rec	Conservation	
A	28.2									28.2	0.0%
B	29.6									29.6	0.0%
C	18.4									18.4	0.0%
D	23.7									23.7	0.0%
E	5.2									5.2	0.0%
F	28.5									28.5	0.0%
G	37.1		7.3						1.8	28.0	2.0%
H	33.9		5.8	20.2					7.9		25.5%
I	22.9	8.1							4.8	10.0	21.2%
J	22.2									22.2	0.0%
K	12.8									12.8	0.0%
L	18.1									18.1	0.0%
M	18.0									18.0	0.0%
N	21.8		7.0							14.8	3.2%
O	18.8		13.5	1.3					0.6	3.4	10.0%
P	16.9	13.4							3.5		47.4%
Q	36.0	18.2							10.8	7.0	30.3%
R	8.3									8.3	0.0%
S	16.8								0.4	16.4	0.0%
T	15.9	6.7							2.6	6.6	25.3%
U	27.7	16.1							4.2	7.4	34.9%
V	15.8	4.1			1.4				4.5	5.8	20.9%
W	16.9	4.2			2.3				6.4	4.0	23.0%
X	65.2	5.0				16.0	4.0	17.2	20.9	2.1	18.5%
Y	21.7						21.1		0.6		58.3%
Z	18.9	0.0					2.6		16.3		8.2%
AA	37.0	24.0					9.7		3.3		54.6%
AB	8.0	8.0									60.0%
AC	32.5									32.5	0.0%
AD	24.0	12.1							4.3	7.6	30.2%
AE	38.1	11.8					2.8	16.9	6.6		31.9%
AF	34.0	11.9					1.4	9.5	11.2		29.0%
AG	40.2					3.2		37.0			20.0%
AH	32.0					8.1		23.9			20.0%
AI	33.6							33.6			20.0%
AJ	9.2							5.6	3.6		12.2%
TOTAL	**887.8**	**143.6**	**33.6**	**21.5**	**3.7**	**27.3**	**41.6**	**143.7**	**114.4**	**358.5**	**18.0%**

Table A2: RAFTS Model Details – 2009/2010

Sub catchment Name	Area (ha)	Land Use Area (ha)									Fraction Impervious
		ResA/Low Density Res	Park Residential	Special Residential	Emerging Community	Community Use	Special Facilities	Rural	Park/Open Space/Sport & Rec	Conservation	
A	28.2									28.2	0.0%
B	29.6	8.9							1.0	19.7	18.0%
C	18.4									18.4	0.0%
D	23.7								1.2	22.5	0.0%
E	5.2									5.2	0.0%
F	28.5									28.5	0.0%
G	37.1		7.3						1.8	28.0	2.0%
H	33.9		5.8	20.2					7.9		25.5%
I	22.9	18.1							4.8		47.4%
J	22.2									22.2	0.0%
K	12.8									12.8	0.0%
L	18.1									18.1	0.0%
M	18.0									18.0	0.0%
N	21.8		7.0							14.8	3.2%
O	18.8		13.5	1.3					0.6	3.4	10.0%
P	16.9	13.4							3.5		47.4%
Q	36.0	25.2							10.8		42.0%
R	8.3									8.3	0.0%
S	16.8	16.4							0.4		58.6%
T	15.9	13.3							2.6		50.2%
U	27.7	23.5							4.2		50.9%
V	15.8	9.9			1.4				4.5		42.9%
W	16.9	8.2			2.3				6.4		37.3%
X	65.2	5.0				16.0		17.2	20.9	6.1	14.8%
Y	21.7								21.7		0.0%
Z	18.9	3.1							15.8		9.8%
AA	37.0	24.0					9.7		3.3		54.6%
AB	8.0	8.0									60.0%
AC	32.5	7.6								24.9	14.0%
AD	24.0	19.7							4.3		49.3%
AE	38.1	11.8					2.8	16.9	6.6		31.9%
AF	34.0	11.9					1.4	9.5	11.2		29.0%
AG	40.2					3.2		37.0			20.0%
AH	32.0					8.1		23.9			20.0%
AI	33.6							33.6			20.0%
AJ	9.2							5.6	3.6		12.2%
TOTALS	**887.8**	**227.9**	**33.6**	**21.5**	**3.7**	**27.3**	**13.9**	**143.7**	**137.2**	**279.0**	**21.8%**

Table A3: RAFTS Model Details – Ultimate Development

Sub catchment Name	Area (ha)	Land Use Area (ha)									Fraction Impervious
		ResA/Low Density Res	Park Residential	Special Residential	Emerging Community	Community Use	Special Facilities	Rural	Park/Open Space/Sport & Rec	Conservation	
A	28.2									28.2	0.0%
B	29.6	8.9							1.0	19.7	18.0%
C	18.4									18.4	0.0%
D	23.7								1.2	22.5	0.0%
E	5.2									5.2	0.0%
F	28.5									28.5	0.0%
G	37.1		7.3						1.8	28.0	2.0%
H	33.9		5.8	20.2					7.9		25.5%
I	22.9	18.1							4.8		47.4%
J	22.2	1.4								20.8	3.8%
K	12.8									12.8	0.0%
L	18.1									18.1	0.0%
M	18.0									18.0	0.0%
N	21.8		7.0							14.8	3.2%
O	18.8		13.5	1.3					0.6	3.4	10.0%
P	16.9	13.4							3.5		47.4%
Q	36.0	25.2							10.8		42.0%
R	8.3									8.3	0.0%
S	16.8	16.4							0.4		58.6%
T	15.9	13.3							2.6		50.2%
U	27.7	23.5							4.2		50.9%
V	15.8	9.9			1.4				4.5		42.9%
W	16.9	8.2			2.3				6.4		37.3%
X	65.2	5.0				16.0	4.0	17.2	20.9	2.1	18.5%
Y	21.7						21.1		0.6		58.3%
Z	18.9	3.1					2.6		13.2		18.0%
AA	37.0	24.0					9.7		3.3		54.6%
AB	8.0	8.0									60.0%
AC	32.5	7.6					24.9				60.0%
AD	24.0	19.7							4.3		49.3%
AE	38.1	11.8					2.8	16.9	6.6		31.9%
AF	34.0	11.9					1.4	9.5	11.2		29.0%
AG	40.2					3.2		37.0			20.0%
AH	32.0					8.1		23.9			20.0%
AI	33.6							33.6			20.0%
AJ	9.2							5.6	3.6		12.2%
TOTALS	**887.8**	**229.4**	**33.6**	**21.5**	**3.7**	**27.3**	**66.5**	**143.7**	**113.5**	**248.6**	**25.4%**

Appendix B Hydraulic Structure Reference Sheets

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK ALBANY CREEK	
LOCATION	**Galaxy Street, Bridgeman Downs**

DATE OF SURVEY: **21 February 1992**	UBD REF:
AERIAL PHOTO No:	STRUCTURE ID: **F12000001**
BCC XS No:	CHAINAGE (m):
STRUCTURE DESCRIPTION: **ROAD CULVERT**	
STRUCTURE SIZE **2 / 1.8 metre diameter RCPs** For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths	
UPSTREAM INVERT LEVEL: **34.650** DOWNSTREAM INVERT LEVEL: **34.545** For culverts give floor level.	UPSTREAM OBVERT LEVEL: **36.450** DOWNSTREAM OBVERT LEVEL: **36.345** For bridges give bed level
For Culverts LENGTH OF CULVERT BARREL AT INVERT (m): **22.0** LENGTH OF CULVERT BARREL AT OBVERT (m): **22.0** TYPE OF LINING: Sloping concrete channel **Concrete** (e.g. concrete, stones, brick, corrugated iron)	
IS THERE A SURVEYED WEIR PROFILE? **ALS** If yes give details ie. Plan number and/or survey book number. Note: This section should be at the highest part of the road eg crown, kerb, hand rails guard rails whichever is higher.	
WEIR WIDTH (m): **Varies** (In direction of flow, ie. distance from u/s face to d/s face)	LOWEST POINT OF WEIR (m AHD): **38.3**
HEIGHT OF GUARDRAILS: **None.** DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:	
The following should also be provided. Wingwall/Headwall details, entrance details eg. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.	
CONSTRUCTION DATE OF CURRENT STRUCTURE: **2/1992** PLAN NUMBER: **W7351S3** HAS THE STRUCTURE BEEN UPGRADED? **No** If yes, explain type and date of upgrade. Include plan number and location if applicable. ADDITIONAL COMMENTS: **Contains baffle blocks downstream – refer photograph**	

CREEK	ALBANY CREEK
LOCATION	Galaxy Street, Bridgeman Downs

ARI (years)	DISCHARGE (m^3/s)			U/S WATER LEVEL (mAHD)	AFFLUX AT MAX FLOW (m)	AREA (m^2)		VELOCITY (m/s)	
	Weir	Structure	Total			Structure	Weir	Structure	Weir
100	0	9.9	9.9	36.39	0.64	5.09	0	2.7	0
50	0	8.8	8.8	36.26	0.55	5.09	0	2.5	0
20	0	7.4	7.4	36.09	0.45	5.09	0	2.3	0
10	0	6.2	6.2	35.95	0.36	5.09	0	2.1	0
5	0	5.5	5.5	35.85	0.31	5.09	0	2.0	0
2	0	4.1	4.1	35.68	0.23	5.09	0	2.1	0

Galaxy Street – Upstream

Galaxy Street – Downstream

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK ALBANY CREEK	
LOCATION	**Peterson Place, Bridgeman Downs**

DATE OF SURVEY: **13 August 1992**	UBD REF:
AERIAL PHOTO No:	STRUCTURE ID: **F13000013**
BCC XS No:	CHAINAGE (m):

STRUCTURE DESCRIPTION: **ROAD CULVERT**

STRUCTURE SIZE **8 / 1.2 metre diameter RCPs**
For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

UPSTREAM INVERT LEVEL: **24.760** UPSTREAM OBVERT LEVEL: **25.960**

DOWNSTREAM INVERT LEVEL: **24.700** DOWNSTREAM OBVERT LEVEL: **24.900**
For culverts give floor level. For bridges give bed level

For Culverts
LENGTH OF CULVERT BARREL AT INVERT (m): **16.0**

LENGTH OF CULVERT BARREL AT OBVERT (m): **16.0**

TYPE OF LINING: Sloping concrete channel **Concrete**
(e.g. concrete, stones, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? **ALS**
If yes give details ie. Plan number and/or survey book number.
Note: This section should be at the highest part of the road
eg crown, kerb, hand rails guard rails whichever is higher.

WEIR WIDTH (m): **Varies** LOWEST POINT OF WEIR (m AHD): **26.85**

(In direction of flow, ie. distance from u/s face to d/s face)

HEIGHT OF GUARDRAILS: **Refer photograph for handrail description**

DESCRIPTION OF ALL HAND AND GUARD RAILS AND
HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:

The following should also be provided.
Wingwall/Headwall details, entrance details eg. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.
For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: **8/1992** PLAN NUMBER: **W8886**

HAS THE STRUCTURE BEEN UPGRADED? **No**
If yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

CREEK	**ALBANY CREEK**
LOCATION	**Peterson Place, Bridgeman Downs**

ARI (years)	DISCHARGE (m^3/s)			U/S WATER LEVEL (mAHD)	AFFLUX AT MAX FLOW (m)	AREA (m^2)		VELOCITY (m/s)	
	Weir	Structure	Total			Structure	Weir	Structure	Weir
100	0	19.0	19.0	26.80	0.18	9.05	0	2.1	0
50	0	16.8	16.8	26.62	0.14	9.05	0	1.9	0
20	0	14.2	14.2	26.43	0.10	9.05	0	1.6	0
10	0	11.8	11.8	26.29	0.07	9.05	0	1.3	0
5	0	10.2	10.2	26.16	0.07	9.05	0	1.2	0
2	0	7.6	7.6	26.01	0.04	9.05	0	1.2	0

Peterson Place – Upstream

Peterson Place – Downstream

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK **ALBANY CREEK**	
LOCATION **Wendon Way, Bridgeman Downs**	

DATE OF SURVEY: **11 August 1992**	UBD REF:
AERIAL PHOTO No:	STRUCTURE ID: **F12000002**
BCC XS No:	CHAINAGE (m):
STRUCTURE DESCRIPTION: **ROAD CULVERT**	
STRUCTURE SIZE **7 / 2.4 x 2.4 metre RCBCs** For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths	
UPSTREAM INVERT LEVEL: **23.56** DOWNSTREAM INVERT LEVEL: **23.50** For culverts give floor level.	UPSTREAM OBVERT LEVEL: **25.96** DOWNSTREAM OBVERT LEVEL: **25.90** For bridges give bed level
For Culverts LENGTH OF CULVERT BARREL AT INVERT (m): **18.0** LENGTH OF CULVERT BARREL AT OBVERT (m): **18.0** TYPE OF LINING: Sloping concrete channel **Concrete** (e.g. concrete, stones, brick, corrugated iron)	
IS THERE A SURVEYED WEIR PROFILE? **ALS** If yes give details ie. Plan number and/or survey book number. Note: This section should be at the highest part of the road eg crown, kerb, hand rails guard rails whichever is higher.	
WEIR WIDTH (m): **Varies** (In direction of flow, ie. distance from u/s face to d/s face)	LOWEST POINT OF WEIR (m AHD): **26.96**
HEIGHT OF GUARDRAILS: **Refer photograph** DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:	
The following should also be provided. Wingwall/Headwall details, entrance details eg. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.	
CONSTRUCTION DATE OF CURRENT STRUCTURE: **n/a** PLAN NUMBER: **W8964** HAS THE STRUCTURE BEEN UPGRADED? **No** If yes, explain type and date of upgrade. Include plan number and location if applicable. ADDITIONAL COMMENTS: **Note significant sediment build up upstream of culverts.**	

CREEK	ALBANY CREEK
LOCATION	Wendon Way, Bridgeman Downs

ARI (years)	DISCHARGE (m^3/s)			U/S WATER LEVEL (mAHD)	AFFLUX AT MAX FLOW (m)	AREA (m^2)		VELOCITY (m/s)	
	Weir	Structure	Total			Structure	Weir	Structure	Weir
100	0	71.8	71.8	26.50	0.26	40.32	0	1.8	0
50	0	62.7	62.7	26.35	0.20	40.32	0	1.8	0
20	0	52.8	52.8	26.20	0.14	40.32	0	1.9	0
10	0	45.5	45.5	26.07	0.10	40.32	0	1.9	0
5	0	40.1	40.1	25.91	0.07	40.32	0	1.9	0
2	0	39.8	39.8	25.76	0.05	40.32	0	1.8	0

Wendon Way - Upstream

Wendon Way – Downstream

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK ALBANY CREEK	
LOCATION	**Albany Creek Road, Albany Creek**

DATE OF SURVEY: **April 2012**	UBD REF:
AERIAL PHOTO No:	STRUCTURE ID:
BCC XS No:	CHAINAGE (m):

STRUCTURE DESCRIPTION: ROAD BRIDGE

STRUCTURE SIZE **2 x 11.85 metre spans**
For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

UPSTREAM INVERT LEVEL:	**12.73**	UPSTREAM OBVERT LEVEL:	**15.94**
DOWNSTREAM INVERT LEVEL:	**13.07**	DOWNSTREAM OBVERT LEVEL:	**15.54**
For culverts give floor level.		For bridges give bed level	

For Culverts
LENGTH OF CULVERT BARREL AT INVERT (m):

LENGTH OF CULVERT BARREL AT OBVERT (m):

TYPE OF LINING: Sloping concrete channel
(e.g. concrete, stones, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? Yes
If yes give details ie. Plan number and/or survey book number.
Note: This section should be at the highest part of the road eg crown, kerb, hand rails guard rails whichever is higher.

WEIR WIDTH (m):	**Varies**	LOWEST POINT OF WEIR (m AHD):	**16.24**
(In direction of flow, ie. distance from u/s face to d/s face)			

HEIGHT OF GUARDRAILS: Handrail:

DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:

The following should also be provided.
Wingwall/Headwall details, entrance details eg. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.
For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?
If yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

CREEK	ALBANY CREEK
LOCATION	Albany Creek Road, Albany Creek

ARI (years)	DISCHARGE* (m^3/s)			U/S WATER LEVEL (mAHD)	AFFLUX AT MAX FLOW (m)	AREA (m^2)		VELOCITY (m/s)	
	Weir	Structure	Total			Structure	Weir	Structure	Weir
100	n/a	n/a	81.2	16.54	n/a	Not relevant to 2D modelling Refer mapping			
50	n/a	n/a	71.9	16.49	n/a				
20	n/a	n/a	60.9	16.42	n/a				
10	n/a	n/a	52.0	16.34	n/a				
5	n/a	n/a	42.6	16.23	n/a				
2	n/a	n/a	31.3	16.06	n/a				

* Discharges under the bridge and over the road are not able to be separated, because the overall bridge structure has been modelled in the 2-dimensional domain.

Albany Creek Road – Upstream

Albany Creek Road – Looking Downstream

Appendix C Addendum Report – Climate Change Analysis

Albany Creek Flood Study

Addendum Report – Climate Change Analysis

Prepared by Brisbane City Council's, City Projects Office

June 2014

Dedicated to a better Brisbane

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1	04.07.13		MG		EC	10498	EC
3	11.06.14		HZ		EC	10498	

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1.0 Introduction

1.1 Catchment Overview

Albany Creek is located in the northern part of Brisbane. The catchment is split between the Brisbane City Council (BCC) and Moreton Bay Regional Council (MBRC) local government areas. The total area of the catchment is approximately 888 hectares. The creek outlets into the South Pine River immediately downstream of Leitchs Crossing at Brendale. The catchment takes in the suburbs of McDowall, Bridgeman Downs, Bunya, Albany Creek, and Arana Hills.

The upper reaches of the catchment are located in the Bunyaville Forest Reserve, upstream of Old Northern Road. Only a small part of the catchment upstream of Old Northern Road is developed – a small residential development in Arana Hills off Collins Road, and large-lot residential areas bounded by Old Northern Road and the Jinker Track. Between Old Northern Road and Darien Street, the catchment is almost fully urbanised. Between Darien Street and Albany Creek Road, there are some small areas of residential development along the western side of the catchment (near Keong Road). However, the majority of the catchment in this reach is undeveloped, containing the Darien Street Sports Fields, the Albany Creek Crematorium and Memorial Gardens, and the Albany Creek Road Reserve (including a pony club and park). Downstream of Albany Creek Road, the catchment comprises residential development on the western side of the creek (in the MBRC area) and rural-residential development on the eastern side of the creek (in the BCC area).

1.2 Study Background

In September 2012, the Albany Creek Flood Study project was completed. Hydrological and hydraulic models were developed and used to determine flood levels, discharges and velocities in Albany Creek and its tributaries for design events of 2, 5, 10, 20, 50 and 100 years ARI. In addition, the rare to extreme events 200, 500 and 2000 year were investigated for existing and ultimate catchment conditions. The two-dimensional TUFLOW hydraulic model of Albany Creek extended from downstream of Streisand Drive in McDowall, and Country Club Drive and Sunningdale Court in Albany Creek.

Council's Natural Environment, Water and Sustainability (NEWS) Branch required longer term planning horizons to be considered in their program of flood studies by considering extreme flood events and potential climate change impacts. At this time, State Planning Policy 3/11 (now superseded by the Coastal Protection State Planning Regulatory Provision) and the Inland Flood Study (DERM, 2010) had provided guidance on assessing the potential impacts on communities and development of projected climate change effects, including sea level rise and increased rainfall intensities.

The SPP 3/11 outlined the following factors to be used by local government to determine planning levels for appropriate planning horizons (2050, 2070 and 2100):

- A sea-level rise factor of 0.8 metres;
- An increase in the maximum cyclone intensity by 10 per cent; and
- Where a relevant storm-tide inundation assessment has not been completed in relation to a proposed development, the coastal hazard area is taken to be all land between high water mark and a minimum default 100-year Design Storm Tide Event level of 1.5 metres above the level of Highest Astronomical Tide for all developments in SEQ.

The Inland Flooding Study outlines the rationale for adopting an interim methodology for assessing flooding risk in Queensland:

1. *The proposed methodology is to factor a 5 per cent increase in rainfall intensity at Annual Exceedance Probabilities (AEP) of 1% (100 yr ARI), 0.5% (200 yr ARI) and 0.2% (500 yr ARI) per degree of global temperature increase for all rainfall events recommended in SPP 1/03 for the location and design of new development.*
2. *The following temperatures and timeframes should be used for the purposes of applying the climate change factor in Recommendation 1:*
    a) *2C by 2050*
    b) *3C by 2070*
    c) *4C by 2100*

1.3 Study Objectives

For this study, two planning horizons, 2050 and 2100, were considered. The methodology is detailed in Section 2.0.

1.4 Report Scope (Limitations)

This assessment of climate change has been based on the hydrologic and hydraulic models developed within the Albany Creek Flood Study (2012) without modification, except as described within this report.

The methodology described for representing ultimate conditions has made reference to the modelled 100 year ARI flood levels for ultimate condition. The methodology considers creek flooding only.

The results of this assessment should not be used without reference to this report and the Albany Creek Flood Study report and an understanding of the modelling extents.

2.0 Climate Change Sensitivity Analysis

2.1 Overview

The following climate change scenarios were modelled in accordance with the methodology detailed below as part of this addendum report.

Table 2-1: Modelled Climate Change Events

Timeframe	Floodplain Conditions	Design Event	Rainfall Intensity Increase	Tailwater Boundary
2050	Existing Conditions	100-year ARI	10% increase	South Pine River 7.75 mAHD
	Ultimate Conditions	100-year ARI	10% increase	South Pine River 7.75 mAHD
	Ultimate Conditions	200-year ARI	10% increase	South Pine River 7.75 mAHD
2100	Existing Conditions	100-year ARI	20% increase	South Pine River 7.75 mAHD
	Ultimate Conditions	100-year ARI	20% increase	South Pine River 7.75 mAHD
	Ultimate Conditions	200-year ARI	20% increase	South Pine River 7.75 mAHD
	Ultimate Conditions	500-year ARI	20% increase	South Pine River 7.75 mAHD

2.2 Hydrologic Modelling

For the climate change assessment the XP-RAFTS model from the 2012 study was used unmodified. Land use parameters were adopted unchanged and represented ultimate catchment development, based on projected planning schemes, for all scenarios.

Rainfall intensity factors were applied to account for climate change, as per the recommendations made in the Inland Flooding Study; 10% for the 2050 planning horizon and 20% for the 2100 planning horizon. The hydrological model was used to simulate these scenarios and the results extracted at the hydraulic model boundaries.

2.3 Hydraulic Modelling

The 2012 study involved the development of a two-dimensional (2D) TUFLOW model. The model was used for this study unmodified, except as discussed in this section. In order to simulate events larger than 100 yr ARI, some modifications were required to ensure the terrain extended beyond flood extents and appropriately represented floodplain conditions in their ultimate state. Further explanation is provided below.

The scaled inflow hydrographs were applied to the 'existing' and 'ultimate' case TUFLOW models to represent climate change conditions. Consistent with the 2012 study, a uniform tailwater level in the South Pine River equivalent to the 100 year flood level (i.e. 7.75 mAHD) was adopted for all design flood events in Albany Creek.

2.3.1 Floodplain Conditions

Traditionally, flood studies have generally considered design events up to and including the 100 year ARI event. This was considered the key event for flood impact assessment and planning purposes. For planning studies, the hydraulic model was developed to represent 'existing' conditions and 'ultimate' conditions. The objective of modelling 'ultimate' conditions is to consider future plans for the watercourse when defining development planning levels, including:

- Minimum riparian corridor (MRC) requirements - i.e. the riparian zone is assumed to be vegetated with a corresponding higher Manning's 'n' value
- Floodplain 'filling' outside of the waterway corridor – i.e. full development up to the waterway corridor is assumed, effectively removing that portion of the floodplain when assessing flood levels

The inclusion of waterway corridors within the hydraulic model has typically been simulated by 'walling off' the zone outside of the waterway corridor, as shown in Figure 2-1.

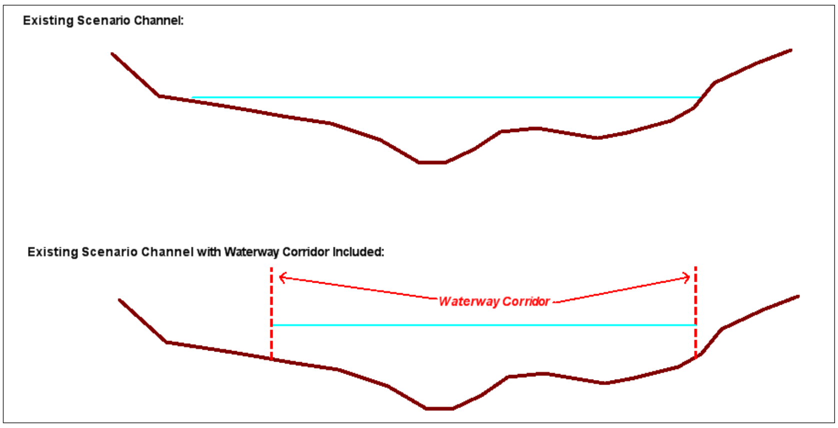

Figure 2-1: Implementation of Waterway Corridor using 'Walling Off' Method

2.3.2 Alteration to Terrain

The method described in the previous section is satisfactory when simulating 2 to 100 year ARI design flood events. However, when simulating the higher 'ultimate' design events, prior experience has shown that the waterway corridor 'walls' results in conservatively high water levels and stability issues in some hydraulic modelling software packages. For this study, the following method for simulating the waterway corridor was adopted for these events:

- Extend the terrain using BCC ALS data (2009) to sufficiently contain the anticipated 2000 year ARI flood extents, under existing floodplain conditions (i.e. no MRC defined and no floodplain filling)
- Take the 'ultimate' case 100 year ARI DIS flood levels from the previous study and add 300mm development freeboard (development level)

In areas outside the waterway corridor, raise the terrain model to this height until the natural surface level is intersected, as shown in Figure 2-2.

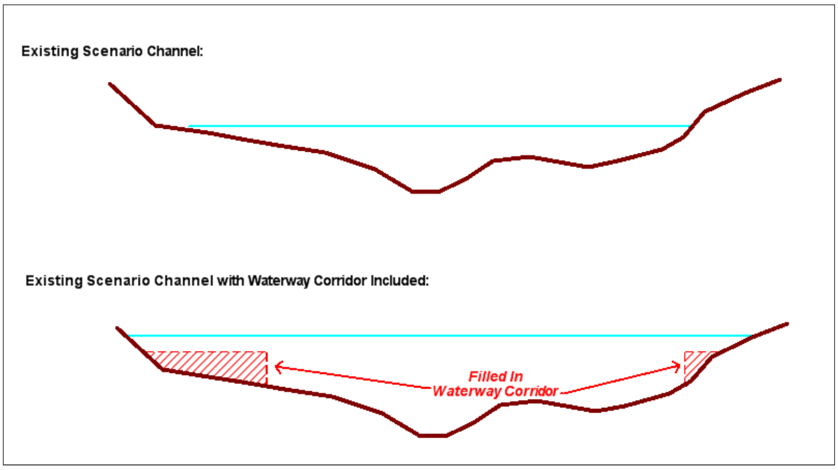

Figure 2-2: Implementation of Waterway Corridor using 'Filling' Method

Additional modification is required when implementing this method to ensure the application of the filling is hydraulically sensible.

2.3.3 Alteration to Structures

To model flood events larger than 100 year ARI, some modification to the representation of hydraulic structures is required to maintain model accuracy and stability. This includes the addition of weir sections (if previously not included) to represent overtopping, increased width of weir sections and/or removal of structures which do not incur significant head loss under extreme conditions. However, no modification to the structures was required for modelling of the climate change in Albany Creek.

2.4 Climate Change Results

Climate change events modelling results are available in digital format.

3.0 Discussion of Results

The climate change analyses have considered a range of flood events and varying topographic conditions representing the floodplain in its existing state and under a fully developed ‘ultimate’ state.

The objective of this assessment is to enable planners and decision makers to consider the impact of climate change predictions on the flood risk profile

A comparison of the ‘ultimate case’ 100 year ARI flood levels and extent from the 2012 study and the climate change scenarios shows only minor changes in flood extents. This suggests that that climate change impacts, as represented by current literature, will not significantly change the Albany Creek flood risk profile.

4.0 References

City Projects Office - Brisbane City Council '*101128 Albany Creek Flood Study*' Draft Report, Version 3, August 2012.

Department of Environment and Resource Management, *State Planning Policy 3/11: Coastal Protection*, 2012.

Pilgrim, DH, (ed*), Australian Rainfall & Runoff - A Guide to Flood Estimation*, Institution of Engineers, Australia, Barton, ACT, 1987.

Appendix D Peer Review of Albany Creek Flood Models

Dedicated to a better Brisbane

MEMORANDUM

Brisbane City Council

To:	Richard Yearsley	Date: 11/06/2014
CC:	Evan Caswell	
From:	Hanieh Zolfaghari	
Re:	Peer Review of Albany Creek Flood Models	

Flood Management

City Projects Office

Brisbane Infrastructure

GPO Box 1434
Brisbane Qld 4001

Phone: 07 30274686
Facsimile: 07 3334 0212
Email: Hanieh.zolfaghari@brisbane.qld.gov.au
Internet: www.brisbane.qld.gov.au

1 Introduction

This review has been undertaken to ensure:

- Council has received all required data associated with the Albany Creek Flood Study (Cardno 2012) to enable future adoption into Council systems
- The flood study has been delivered in accordance with Council procedures and methods at the time the study was undertaken
- The output is fit for purpose

This review includes a high level technical review of the models and results. It is assumed that Cardno have applied best-practise Quality Assurance in producing the flood study and that the work has been prepared under suitably qualified RPEQ supervision as is required by State law.

Reference is made to the documents saved under: 'G:\BI\CD\Proj10\101128_Albany_Creek_Flood_Study\FloodManagement\Documents\WPData\Reports' which was used to track and close out issues during the study.

A peer review check list is included in **Appendix A**.

1.1 Files reviewed

G:\BI\CD\Proj10\101128_Albany_Creek_Flood_Study\FloodManagement\Project Delivery-Cardno\October 2012-Final

2 Hydrology Model

An XP-RAFTS model version 2009 was developed for 2012 Albany Creek Flood Study.

2.1 Sub-catchment representation

- 36 sub-catchments for 888 ha is sufficient
- Delineation looks reasonable – similarly sized sub-catchments, appropriate resolution for this type of study
- Spot check of the catchment area, catchment slope, Pern values was undertaken. The adopted Pern values for the pervious surfaces seems a bit low.
- Muskingum Cunge method was used to route the flow within the XP-RAFTS model
- Land use assignment (pervious and impervious percentage) for calibration events and design events not checked.

- Spot check of routing lengths and slopes was undertaken. They were within an acceptable range.

- The local catchment flow (and total catchment flows at the upstream end of each branch) was used as inflows into hydraulic model. Hydraulic model is being used to route flow.
- The setup of the 3 detention basins and their outlet configuration in the hydrology model were checked and they all seem correct. The detention basin storage values were not checked.

2.2 Model parameters

- Storage Coefficient Multiplication Factor of 1.5 was adopted for all runs which is within a reasonable range.

2.3 Calibration

- The IL = 0 and CL = 2.5 were adopted for all calibration events which seems to be suitable as good agreement with recorded levels was achieved for all calibration events. A check on the antecedent rainfall for calibration events was not undertaken.
- Event rainfall data not checked (depths and spatial variability) but the report indicates that a thorough process has been applied.
- No stream gauge data was available for this catchment. The hydrology model was validated by the comparison of the hydrology and hydraulic model hydrographs (Joint calibration).
- The overall calibration looks reasonable – A comparison between hydrology and hydraulic model hydrograph at the mouth of Albany Creek shows a good agreement between the peak discharges and timing of the events. No additional check at a different location was undertaken.

2.4 Design rainfall data

- AR&R design storms simulated
- IFD data (up to 100yr) spot checked (2yr and 100yr) against IFD tool (AUS IFD Version 2.0 and online IFD creator in BoM website). The rainfall intensity for 50yr 60min storm seems a bit lower than the correct value (87mm/hr rather than 97mm/hr). It is understood that a different method was maybe used to extract the IFD data, however; the difference seems out of range.
  Cardno re-ran the models based on new intensity of 97mm/hr and updated all the tables and figures.
- 200 and 500 and 2000 year ARI rainfall depth and intensities were extracted from CRC FORGE by consultant. The extracted values were not checked against CRC FORGE tool. 2000 year ARI will be rerun during flood study finalisation using BCC super storm. (FLM - DESIGN - BCC Catchments 2000yr PMP Superstorm.xlsx)
- The acceptable values of IL = 0 and CL = 2.5 were used for all design events.
- No PMP simulation for Albany Creek catchment was undertaken.
- Q100yr6hr storm file set-up checked and was correct.
- AR&R temporal patterns not checked (zone3.pat) as its built in XP-RAFTS engine
- Model was run for 100yr event for all standard duration storm – ran successfully and outputs matched the table 6.3 in the report. However, Albany Creek Road in Table 9.3 should be linked to node Z-AD rather than node AD.
  Cardno updated the report.

2.5 Consistency check

- Check was made by Consultant: FFA undertaken at Albany Creek Road using City Gauge data (long term record) which demonstrates a high level of consistency between peak discharges calculated using the AR&R storm events and the flood frequency analysis results, especially for the 5 year to 100 year ARI events.

3 Hydraulic Model

A TUFLOW model build 2010-10-AC-iSP-w64 was developed for 2012 Albany Creek Flood Study.

3.1 Schematic

- Model includes the main Albany Creek and 2 other small tributaries
- 2D TUFLOW model developed in Mapinfo, 4m grid (have had channel invert defined within 2D domain) (The invert level against survey information was not checked)
- Model includes three 1D structures and one 2D structure.

3.2 Topography

- Model has used ALS (2009) and some 2012 ground survey as well as older survey data for the inclusion in the model (gully line to capture the invert level of the channel) and to validate the topographical data as a significant difference in flood levels were observed at several locations throughout the catchment compared to the flood levels from the 1992 Albany Creek Study.
- Major floodplain controls (road embankments) should be reasonably represented by a 4m grid; modifications have been made to correct issues and define inverts/crests
- Ultimate topography has referenced Waterway Corridor in BCC area and 50 yr ARI flood extent in MBRC area. Some minor changes have been made to the BCC waterway corridor for the purpose of modelling.
- Ultimate case topography has not been modified to incorporate filling outside WC (to 100yr-ult+300mm level). The study was undertaken in accordance with the 2012 flood study requirement which did not include filling the floodplain for the ultimate case scenario for extreme events. This work will be undertaken as part of Flood Study Finalisation and has not been included in this review.
- Spot check of the topography modifications (zsh, zln, etc) to the existing case dtm was not undertaken as digital data did not include check files; model not simulated for this review. However the inclusion of topography modifications (zsh, zln, etc) was checked and it seems correct.

3.3 Roughness

- Same material layer has been used for all calibration events, existing and ultimate scenarios as the land use for this area is mostly open space or rural. Manning's 'n' values tabulated in the report are within the range of industry accepted values. While, the hydraulic model material file (.tmf file) is not based on the BCC City Plan land use mapping and has been adopted based on the aerial photography, the adopted methodology still sounds valid.

3.4 Boundary conditions

- Check made and Rafts model outflows match TUFLOW model inflows
- Check confirmed correct boundary has been adopted for design studies: Q100 South Pine River(2009 Study) @ Albany Creek mouth = 7.75m AHD
  It is noted that due to the significant changes in ground level (sudden drop) immediately upstream of confluence of Albany Creek and South Pine River, the model is not sensitive to the changes in receiving water levels.

3.5 Structures

- The set-up of structures was checked and all adopted parameters seem to be within an acceptable range. HEC-RAS models were also developed to check the performance of the

structures and a comparison of the HEC-RAS and TUFLOW results is included in the report which shows a good agreement. HEC-RAS model was not checked as part of this review.
- Hydraulic Structure Reference Sheets summarise immunity of structures. The HSRS's were not checked.

3.6 Model performance

3.6.1 Calibration

- Calibration achievement discussed throughout study; calibration maps/graphs indicate a reasonable level of calibration has been achieved
- Good agreement between hydraulic and hydrology model discharge hydrographs was achieved
- Oct 2010, May1996 and May 2009 events achieved calibration target of within 300mm at all MHGs. Exception is made for gauge 110 during May 1996 event which is due to the incorrect recorded data.

3.6.2 Stability and mass errors

- _H.csv, _Q.csv and PO.csv hydrographs spot checked for flows up to 2000 yr ARI. Some instabilities were observed in the rising limbs of the discharge hydrographs at Wendon Way crossing. However, these instabilities happen at the early stages of the simulation and don't have any impacts on the peak discharges.
- Model mass errors checked and they all found to be within acceptable range (ie. less than 1%).

3.7 Quality Assurance

- Model log not included in digital data – this should be requested in future studies
- Model has utilised logical naming conventions and standard folder structure
- Model Handover Guides included

4 Outputs and Mapping

- Report format is standard and in accordance with the BCC report template provided at the time study was undertaken
- Explanatory note is required within the report as to how waterway corridor has been derived within the Moreton Bay area
- Clarity required for scenario labelling of maps. However, maps will be redone as per new flood study procedure version 5.2 as part of Flood Study Finalisation project
- WC not shown on any of the maps

5 Cardno Deliverable Data Gap

The following data gaps were noted:

- HEC-RAS model used to verify the losses within the structures
- TUFLOW model check files
- TUFLOW model log (this may not be necessary as only the final version of the model was provided to BCC)

6 Conclusion

In general it appears the models have been prepared diligently and are fit for purpose. Required input and output data has been handed over in a logical format.

It is important however to understand that the study up to this stage does not include filling the floodplain in developing the 'ultimate case' DTM and associated mapping (stretching the flood extent) and as such they have not been included in this review.

Approved By:

Hanieh

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Hanieh Zolfaghari
Flood Engineer

E R Caswell

....

Evan Caswell (RPEQ No.10498)
Principal Engineer, Flood Management

Appendix A

Peer Review Checklist

1.0 Project Details

Project Name:	Albany Creek Flood Study
Client:	NEWS- BCC
Project Job Number:	CD101128
Date:	26/05/2013
Modellers Name:	Michael Della
Modellers Organisation:	Cardno
Reviewers Name:	Hanieh Zolfaghari
Reviewers Organisation:	BCC - Flood Management, CPO
Major Catchment Name:	Pine River
Creek Name:	Albany Creek

Review Status

- [ ] Model Build
- [ ] Calibration / Verification
- [ ] Design Modelling
- [x] Final Handover
- [ ] Other (specify)

Purpose of Study

- [x] Flood Planning Levels (e.g. flood study)
- [ ] Flood Mitigation Design (e.g detention basin)
- [ ] Hydraulic Impact Assessment (e.g. bridge upgrade)
- [ ] Flood Hazard Mapping
- [ ] Flood Warning
- [ ] Other (specify)

Modelling software

x	RAFTS		MIKE 11
	URBS	x	HEC RAS
	WBNM	x	TUFLOW
	RORB		MIKE 21/FLOOD
	Other (specify)		Other (specify)

Further description of the modelling

2.0 Models

Hydrology model

Can model be opened and run?	[x] Yes [ ] No	[ ] N/A
Do results match accompanying report?	[x] Yes [ ] No	[ ] N/A

Hydraulics model

Can model be opened and run?	[x] Yes [ ] No	[ ] N/A
Do results match accompanying report?	[x] Yes [ ] No	[ ] N/A
Have all technical issues identified at hold points been addressed and resolved? (reference progress meeting minutes and any responses from draft interim reviews)	[x] Yes [ ] No	[ ] N/A

3.0 Documentation

Does handover documentation include?

Detailed report in required forma	[x] Yes [ ] No	[ ] N/A

Model handover guide, detailing:

- Model software and version/patch details	[ ] Yes [x] No	[ ] N/A
- Key data sources with date stamp	[ ] Yes [x] No	[ ] N/A
- Data file structure and naming format	[x] Yes [ ] No	[ ] N/A
- Instructions for model use	[ ] Yes [x] No	[ ] N/A
- Limitations and future use of model (incl. data requirements)	[ ] Yes [x] No	[ ] N/A
- Other instruction notes/'read me' files	[ ] Yes [x] No	[ ] N/A

Quality assurance documentation:

- Models logs	[ ] Yes [x] No	[ ] N/A
- Interval verification checklists	[ ] Yes [x] No	[ ] N/A
- Sign off by RPEQ	[ ] Yes [x] No	[ ] N/A
Is output considered to be 'fit for purpose'?	[x] Yes [ ] No	[ ] N/A

Other Comments / Issues

The report will be signed off by RPEQ as part of Flood Study Finalisation project

Refer Memorandum

3.0 Archiving

Models copied to central location?	☐ Yes ☒ No	☐ N/A
Master model catalogue completed with:		
- Brief history of model	☐ Yes ☒ No	☐ N/A
- Who worked on model and why	☐ Yes ☒ No	☐ N/A
- Model software and version/patch details	☐ Yes ☒ No	☐ N/A
- Key data sources (and date stamp)	☐ Yes ☒ No	☐ N/A
- Hydrology summary (e.g. URBS model developed/modified)	☐ Yes ☒ No	☐ N/A
- Hydraulics summary (e.g. TUFLOW model developed/modifiied)	☐ Yes ☒ No	☐ N/A
- Calibration and validation summary	☐ Yes ☒ No	☐ N/A
- Limitations and future use of model	☐ Yes ☒ No	☐ N/A

Other Comments / Issues

A model storage system is not currently in place. Final models have been saved to:

G:\BI\CD\Proj10\101128_Albany_Creek_Flood_Study\FloodManagement\Project Delivery-Cardno\October 2012-Final

Model handover guide is included in the report as one of the appendices

The draft final report is saved in TRIM 197/630/543/554 and the document number is CA13/576643