Wynnum Creek Flood Study
Volume 1 of 2

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Wynnum Creek Flood Study

Volume 1 of 2

Prepared by Brisbane City Council's, City Projects Office

July 2014

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Executive Summary

Introduction

Brisbane City Council (BCC) is in the process of updating all of its flood studies to reflect the current conditions of the catchment and best practice flood modelling techniques. The most recent flood study for the Wynnum Creek catchment was undertaken on behalf of BCC by Kinhill Engineers (now KBR) and Gutteridge Haskins & Davey (GHD) Engineers in 1997. A number of minor studies have been undertaken since this time, with the most significant being the 2004 Water Quantity Assessment undertaken by BCC City Design.

Wynnum Creek is a small catchment draining into Moreton Bay. It is located approximately 15 km south-east of the Brisbane CBD. The catchment has an area of 7.5 km^2 and encompasses the bayside suburbs of Wynnum, Wynnum West and Manly West. The entire catchment lies within the Brisbane City Council (BCC) jurisdiction. The catchment is effectively fully developed with the primary land use being residential development.

Project Objectives

The primary objectives of the project were as follows:

- Update the Wynnum Creek Catchment flood models (hydrologic and hydraulic) to represent the current catchment conditions and best practice flood modelling techniques.
- Adequately calibrate and verify the hydrologic and hydraulic models to historical storm events to confirm that the models are suitable for the purposes of simulating design flood events.
- Estimation of design and extreme flood magnitudes.
- Determination of design flood levels for the full range of design and extreme events up to the Probable Maximum Flood (PMF).
- Quantify the impacts of Minimum Riparian Corridor (MRC) and filling outside the Waterway Corridor (WC).
- Produce flood inundation and flood depth mapping for the selected range of design and extreme events up to the PMF (as applicable).
- Quantify the impacts of climate change as well as hydraulic structure blockages on flooding within the catchment.

Project Elements

The Wynnum Creek Flood Study consists of two main components, as follows:

Calibration and Verification Modelling

Hydrologic and hydraulic models of the Wynnum Creek Catchment have been developed using the XP-RAFTS and TUFLOW modelling software, respectively.

The hydrologic model simulates the catchment rainfall-runoff and runoff-routing processes. The hydrologic model also utilises high-level routing methodology to simulate the flow of floodwater in the major waterways within the catchment. The hydraulic model uses more sophisticated routing to simulate the movement of this floodwater through these waterways in order to predict flood levels, flood discharges and velocities. The hydraulic model takes into account the effects of the channel / floodplain topography; downstream tailwater conditions and hydraulic structures.

Calibration is the process of refining the model parameters to achieve a good agreement between the modelled results and the historical / observed data. Model calibration is achieved when the model simulates the historical event to within specified tolerances. Verification is then undertaken on additional flooding events to confirm the calibrated model is suitable for use in simulating synthetic design storm events.

Calibration of the XP-RAFTS and TUFLOW models was undertaken utilising two historical storms; namely 2nd March 2013 and the 11th December 2010. Verification of the XP-RAFTS and TUFLOW models utilised the 20th May 2009 and 11th December 1995 historical storm events.

A good agreement was achieved between the simulated and historical records for both of the calibration events. At the Maximum Height Gauges (MHGs), the simulated peak levels were within the specified tolerance of ± 0.3 m. At the Byrneside Terrace continuous recording stream gauge, the simulated results indicated a good replication of the rising and receding limbs as well as the magnitude and timing of the peak flood level.

Utilising the adopted parameters from the calibration process, the verification was undertaken. Similar to the calibration results, the verification achieved a good agreement between the simulated and historical records for both of the verification events.

Given the results of the calibration and verification process were quite reasonable, the XP-RAFTS and TUFLOW models were considered acceptable for use in the second part of the flood study, in which design flood levels were estimated.

Design and Extreme Event Modelling

The calibrated hydrologic and hydraulic models were then used to simulate a range of synthetic design flood events. Design and extreme flood magnitudes were estimated for the full range of events from 2-yr ARI to PMF. These analyses assumed ultimate catchment development conditions.

Three waterway scenarios were considered, as follows:

- Scenario 1 – Existing Waterway Conditions: Based on the current waterway conditions. Some minor modifications were made to the TUFLOW model developed as part of the calibration / verification phase.
- Scenario 2 – Minimum Riparian Corridor (MRC): Includes an allowance for a riparian corridor along the edge of the channel.
- Scenario 3 – Ultimate Conditions: Includes an allowance for the minimum riparian corridor (as per Scenario 2) and also assumes development infill to the boundary of the Waterway Corridor (WC) in order to simulate potential development outside the WC.

The results from the TUFLOW modelling were used to determine / produce the following:

- Peak flood discharges
- Critical storm durations at selected locations
- Peak flood levels
- Peak flood extent mapping

- Peak flood depth mapping
- Hydraulic structure flood immunity

A sensitivity analysis was undertaken to understand the impacts of the following:

- Climate change for two planning horizons; namely 2050 and 2100.
- Hydraulic Structure Blockages

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Glossary of Terms

Term	Definition
Annual Exceedance Probability(AEP)	The probability that a given rainfall total or flood flow will be exceeded in any one year.
Average Recurrence Interval (ARI)	The long-term average number of years between the occurrence of a flood as big as (or larger than) the selected event. For example, floods with a discharge as great as (or greater than) the 20 year ARI design flood will occur on average once every 20 years.
AHD	Australian Height Datum (AHD) is the reference level for defining reduced levels adopted by the National Mapping Council of Australia. The level of 0.0 m AHD is approximately mean sea level.
Brisbane Bar	Location at the mouth of the Brisbane River
Catchment	The area of land draining through the main stream (as well as tributary streams) to a particular site. It always relates to an area above a specific location.
Digital Elevation Model (DEM)	A three-dimensional model of the ground surface elevation.
Design Event, Design Storm	A hypothetical flood/storm representing a specific likelihood of occurrence (for example the 100 year ARI).
ESTRY	TUFLOW 1D engine.
Floodplain	Area of land subject to inundation by floods up to and including the probable maximum flood (PMF) event
Flood Frequency Analysis (FFA)	Method of predicting flood flows at a particular location by fitting observed values at the location to a standard statistical distribution.
HEC-RAS	Hydrodynamic modelling software package.
Hydrograph	A graph showing how the discharge or stage/flood level at any particular location varies with time during a flood.
Manning's 'n'	The Gauckler–Manning coefficient, used to represent roughness in 1D/2D flow equations.
MIKE11	Hydrodynamic modelling software package.
Minimum Riparian Corridor (MRC)	An area of (maximum) 15m width either side of the main flow channel.
Pluviograph	An instrument for measuring the amount of water that has fallen (ie. Rain gauge), with a feature to register the data in real time to demonstrate rainfall over a short period of time, often an automated graphing instrument.
Probable Maximum Flood (PMF)	An extreme flood deemed to be the largest flood that could conceivably occur at a specific location.
Probably maximum Precipitation (PMP)	Probable Maximum Precipitation. The maximum precipitation (rainfall) that is reasonably estimated to not be exceeded.
RAFTS	Hydrologic modelling software package.

List of Abbreviations

Abbreviation	Definition
1d	One dimensional, in the context of hydraulic modelling
2d	Two dimensional, in the context of hydraulic modelling
AMTD	Adopted Middle Thread Distance
ALS	Airborne Laser Scanning
AR&R	Australian Rainfall and Runoff (1999)
BCC	Brisbane City Council
CBD	Central Business District
CL	Continuing rainfall loss (mm/hr)
IFD	Intensity Frequency Duration
IL	Initial rainfall loss (mm)
m AHD	metres above AHD
MHG	Maximum Height Gauge
MRC	Minimum Riparian Corridor
MSQ	Maritime Safety Queensland
POT	Peak Over Threshold
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
QUDM	Queensland Urban Drainage Manual (2013)
WC	Waterway Corridor
WQA	Water Quantity Assessment

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1.0 Introduction

1.1 Catchment Overview

Wynnum Creek is a small catchment draining into Moreton Bay, located approximately 15 km south-east of the Brisbane CBD. The catchment has an area of 7.5 km^2 and encompasses the bayside suburbs of Wynnum, Wynnum West and Manly West. The entire catchment lies within the Brisbane City Council (BCC) jurisdiction. Figure 1.1 indicates the locality of the catchment.

The catchment is effectively fully developed with the primary land use being residential development.

1.2 Study Background

BCC is in the process of updating all of its flood studies to reflect the current conditions of the catchment and best practice flood modelling techniques.

The most recent flood study for the catchment was undertaken on behalf of BCC by Kinhill Engineers (now KBR) and Gutteridge Haskins & Davey (GHD) Engineers in 1997[1]. A number of minor studies have been undertaken since this time, with the most significant being the 2004 Water Quantity Assessment (WQA) undertaken by BCC City Design[2].

1.3 Study Objectives

The primary objectives of the project are as follows:

- Update the Wynnum Creek Catchment flood models (hydrologic and hydraulic) to represent the current catchment conditions and best practice flood modelling techniques.
- Adequately calibrate and verify the hydrologic and hydraulic models to historical storm events to confirm that the models are suitable for the purposes of simulating design flood events.
- Estimation of design and extreme flood magnitudes.
- Determination of design flood levels for the full range of design and extreme events up to the Probable Maximum Flood (PMF).
- Quantify the impacts of Minimum Riparian Corridor (MRC) and filling outside the Waterway Corridor (WC).
- Produce flood inundation and flood depth mapping for the selected range of design and extreme events up to the PMF (as applicable).
- Quantify the impacts of climate change as well as hydraulic structure blockages on flooding within the catchment.

[1] Brisbane City Council 1997, *Wynnum Creek Flood Study*, prepared by GHD and Kinhill Engineers, Brisbane

[2] Brisbane City Council 2004, *Wynnum Creek Water Quantity Assessment Technical Report,* prepared by Water & Environment, City Design, Brisbane City Council, Brisbane.

1.4 Scope of the Study

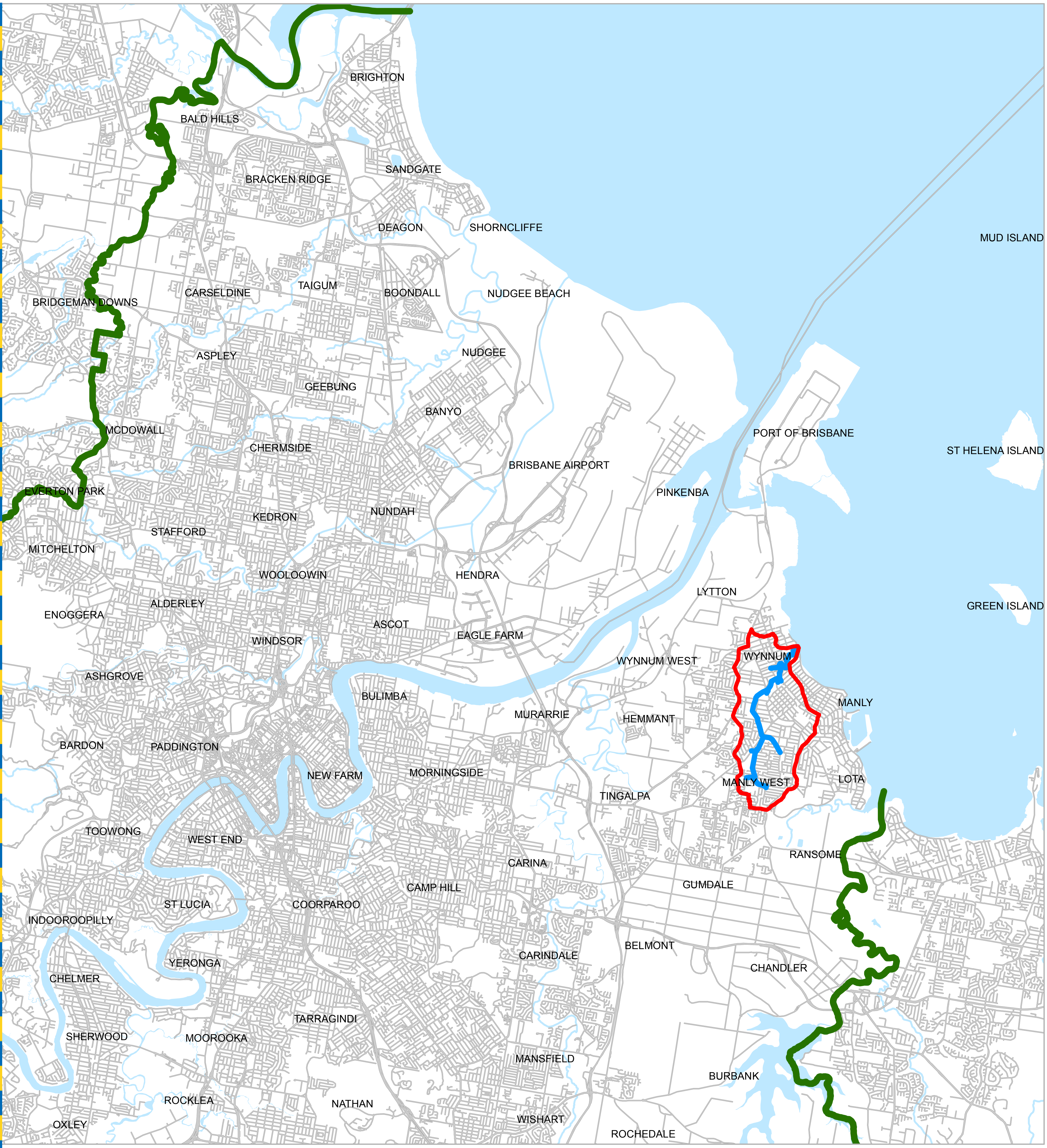
The following tasks were undertaken to achieve the project objectives as outlined in Section 1.3:

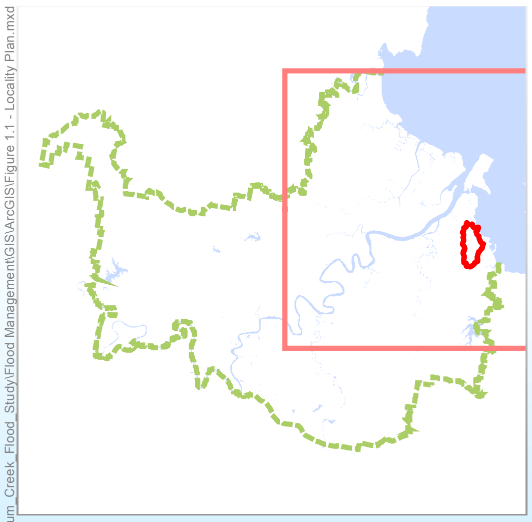
- Develop / upgrade the existing XP-RAFTS hydrologic model of the catchment, representing a refinement of the previous flood study.
- Develop a 1-dimensional / 2-dimensional hydraulic model of the creek system to replace the existing 1-dimensional MIKE11 hydraulic model.
- Calibrate the hydrologic and hydraulic models to the March 2013 and December 2010 historical flood events.
- Verify the hydrologic and hydraulic models to the May 2009 and December 1995 historical flood events.
- Estimation of design and extreme flood magnitudes for the full range of events from 2-yr ARI to PMF.
- Simulate synthetic Australian Rainfall and Runoff (AR&R) design storms for multiple durations to determine the critical duration at various locations within the catchment.
- Utilise the calibrated flood models to determine peak design flood levels for the full range of design and extreme events up to the PMF.
- Make adjustments to the hydraulic model to simulate the impacts of MRC and filling outside the WC.
- Combine the modelling results for the various storm durations to produce peak results throughout the catchment for each ARI.
- Produce peak mapping results for flood inundation and flood depth for the selected range of design and extreme events up to the PMF.
- Undertake climate change modelling for the 100-yr, 200-yr and 500-yr ARI events to determine the impacts.

1.5 Study Limitations

In utilising the flood models it is important to be aware of their limitations which can be summarised as follows:

- The models have only been calibrated / verified at locations where stream gauge and MHG records exist. This should be taken into account when considering the accuracy of results outside the influence of the gauge locations.
- These models are catchment scale and have been developed to simulate the flooding characteristics at a broad scale. As a result, smaller more localised flooding characteristics may not be apparent in the results.
- BCC 2009 ALS data has been used to represent the hydraulic model floodplain topography. Detailed checks have not been undertaken on the accuracy of the ALS data, it is assumed that the data is representative of the topography and “fit for purpose.”
- The accuracy of the model results is directly linked to the following:
    - The accuracy limits of the data used to develop the model (e.g. ALS, survey information, bridge data, etc).
    - The accuracy and quality of the hydrometric data used to calibrate / verify the models.
    - The number of historical stream gauge / MHG locations throughout the catchment.
    - The purpose of the study (i.e. catchment / broad-scale or detailed).





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Prepared : 077900
Checked : SG
Revision : 0
Publication Date : 16 Jul 2014
Project Number : 130115

Legend

Brisbane City Boundary

Wynnum Creek Catchment

Streets

Brisbane City Council
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Figure 1.1: Locality Plan

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File : G:\BI\CD\Proj13\130115_Wynnum_Creek_Flood_Study\Flood Management\GIS\ArcGIS\Figure 1.1 - Locality Plan.mxd

2.0 Catchment Description

2.1 Catchment and Waterway Characteristics

The Wynnum Creek Catchment has an area of 7.5 km^2 and is reasonably elongated and uniform in shape, with a length to width ratio of greater than 2.5. The catchment sides generally fall towards the creek at slopes of between 2 and 4 %. Because of the elongated shape of the catchment, there are no major tributaries within the catchment. The tributaries generally consist of large stormwater drainage pipes / box culverts, with the exception of the East Drain, which is an engineered open channel 650 m in length.

The highest point in the catchment is approximately 50 m AHD in the vicinity of Manly Road, along the boundary with the Lota Creek Catchment. The Wynnum Creek Catchment headwaters are in the residential areas of Manly West, where the creek is piped. Wynnum Creek is an open waterway from just downstream of Amberjack Street to its outfall at Moreton Bay; a length of approximately 5.1 km. The average grade of the creek over the full length is approximately 0.35 %.

The upper to middle reaches of the creek (upstream of Chandos Street) are within a valley confinement, resulting in limited floodplain on both sides of the creek. Between Chandos Street and Tingal Road the creek valley widens resulting in more expansive floodplain areas.

Upstream of Tingal Road, for a length of approximately 3.6 km, the creek has been heavily engineered and modified, with the majority of work occurring in the 1960s and 1970s. This has resulted in only isolated areas of riparian vegetation remaining. The various channel types are as follows:

- Amberjack Street to Graduate Street (0.1 km length) – engineered trapezoidal channel (concrete lined).
- Graduate Street to Stradbroke Avenue (2.9 km length) – engineered grass-lined channel of various shape and size.
- Stradbroke Avenue to Tingal Road (0.6 km length) – engineered trapezoidal channel (concrete lined).

The lower reaches of the creek are tidal, with interaction occurring from Moreton Bay. Downstream of Tingal Road, the waterway is in a more natural condition and is quite incised for a length of approximately 0.8 km. Further downstream, the creek opens out into the Moreton Bay foreshore area, with scattered mangroves occurring towards Moreton Bay.

2.2 Land Use

The Wynnum Creek Catchment is effectively fully developed with the primary land-use being low density residential development. The catchment experienced rapid development following the Second World War, with further urban development from the 1970s onwards, changing the catchment into primarily a residential area.

In the Wynnum Central area, close to where the creek crosses under the railway line, there are small pockets of low-to-medium residential; medium residential; commercial and light industrial development.

There are scattered green space areas (e.g. parks) throughout the catchment, which are predominantly adjacent to the creek. The most significant of these areas is the Wynnum Golf Course, which occupies an area of 36 hectares within the mid to lower catchment.

3.0 Hydrometric Data and Storm Selection

3.1 Selection of Historical Storm Events

Table 3.1 indicates the more significant flooding events which have occurred within the catchment over the previous 46 years, including information on the size of the event and the availability of stream gauge / MHG information.

Table 3.1 – Historical Peak Levels at Tingal Road (MHG W100)

Event	Peak Flood Level (m AHD)	Rank for All Events	Rank for Selected Events	Recorded Hydrograph at Stream Gauge	Number of MHGs and/or recorded levels
June 1967	6.28 (1)	1	-	No	6
January 1974	5.61 (1)	2	-	No	9
November 1979	4.49	11	-	No	6
May 1980	4.46	12	-	No	3
December 1982	-	-	-	No	1
June 1983	4.98	5	-	No	5
April 1988	-	-	-	No	3
December 1995	4.71	8	3	Yes	7
May 1996	4.66	10	-	Yes	6
March 1998 (2)	4.78	6		Yes	7
February 2008	5.94	3	-	Yes	2
May 2009	4.69	9	4	Yes	3
December 2010	5.20	4	1	Yes	4
March 2013	4.72	7	2	Yes	4

1. Estimate from 1997 Wynnum Creek Flood Study

The selection of specific historical events for calibration and verification was based upon the following criteria:

- Higher priority for those events with consistent rainfall throughout the catchment.
- Higher priority for those events which had readily available recorded hydrograph data at the Byrneside Terrace Stream Gauge.
- Higher priority for events where the catchment / creek conditions are similar to the present.
- Higher priority for larger events.
- Higher priority for events which had the greatest number of MHGs in operation.

On the basis of these selection criteria, the events as indicated in Table 3.2 were used for this study.

Table 3.2 – Events selected for Calibration and Verification

Calibration	Verification
2nd March 2013	20th May 2009
11th December 2010	11th December 1995

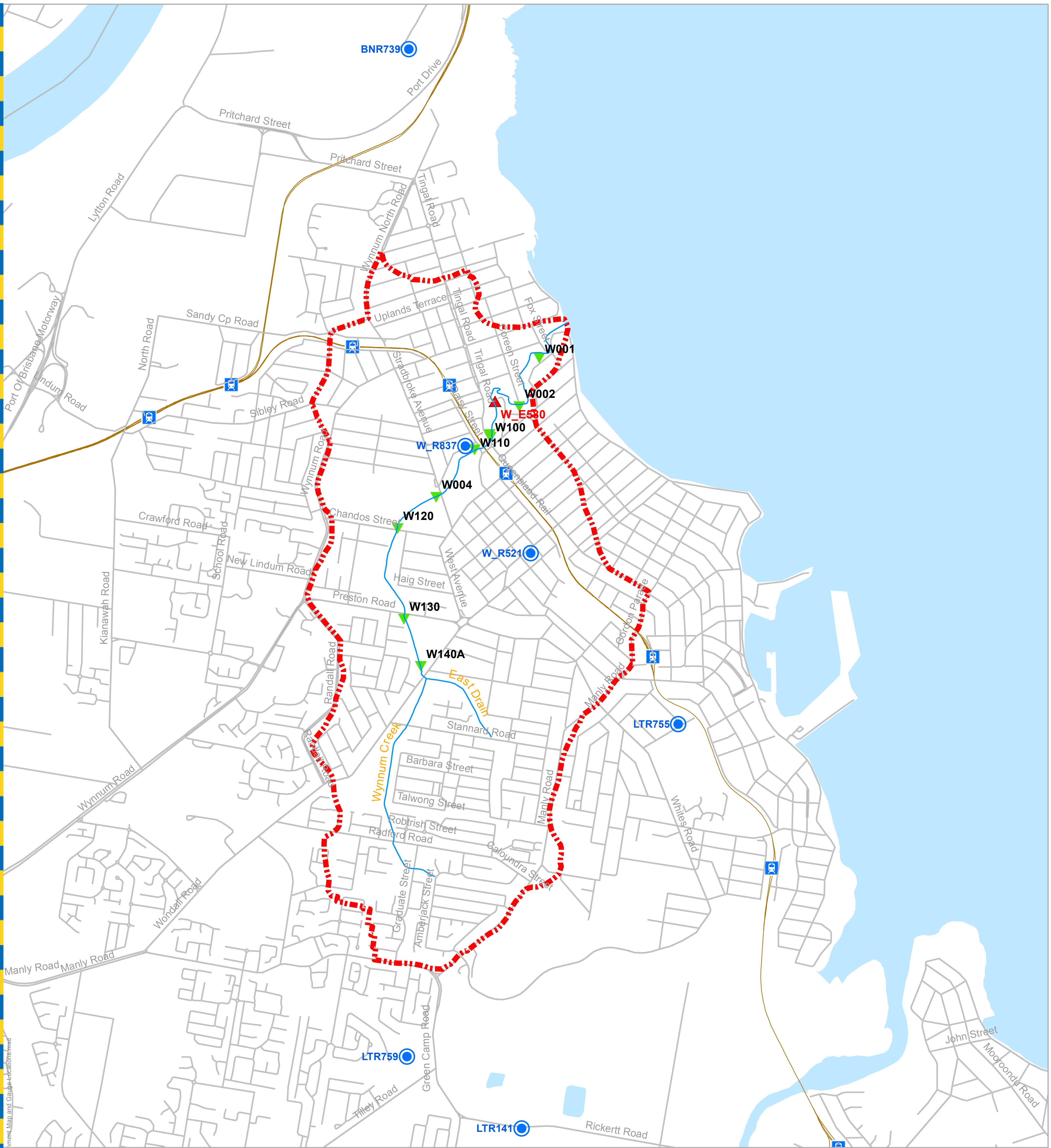
The larger 1967 and 1974 events were not included as numerous channel modifications (including new hydraulic structures) have occurred since this time. Also, the absence of a recorded hydrograph at the Byrneside Terrace stream gauge meant that the calibration / verification of the hydrograph shape would not be possible.

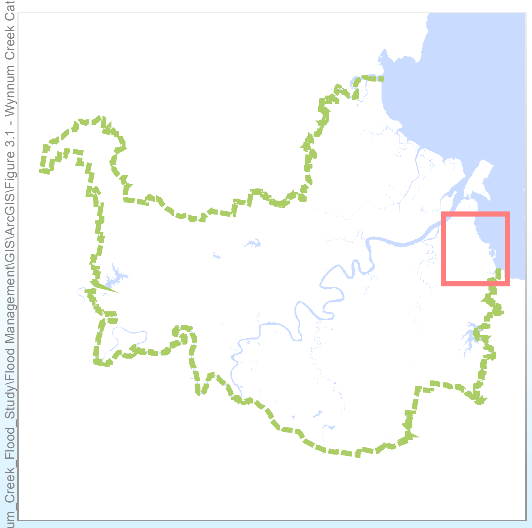
3.2 Availability of Historical Data for Selected Storms

3.2.1 Continuous Recording Rainfall (Pluviograph) Stations

There are six BCC owned rainfall (pluviograph) stations that were utilised for this study. Two are located within the Wynnum Creek Catchment, three within the Lota Creek Catchment and one near the Brisbane River mouth. The locations of the gauges are indicated in Figure 3.1.

Two of these gauges (LTR759 and W_R521) are currently closed, however they were operational for some of the historical events.





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File : G:\BI\CD\Proj13\130115_Wynnum_Creek_Flood_Study\Flood Management\GIS\ArcGIS\Figure 3.1 - Wynnum Creek Catchment Map and Gauge Locations.mxd

Legend

- Pluviograph Stations
- Continuous Recording Stream Gauge
- Maximum Height Gauge
- Wynnum Catchment
- Wynnum Creek Centreline
- Streets
- Railway Station
- Railway Line

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Figure 3.1: Wynnum Creek Catchment Map and Gauge Locations

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Table 3.3 indicates the commencement and closure dates (if applicable) of the pluviograph stations in the vicinity of the catchment.

Table 3.3 – Rainfall gauge period of record

Gauge ID	Location	Commencement Date	Closure Date	Period of Record (years)
LTR755	Harman Recreation Reserve	1 December 1999	Still active	14
LTR141	Rickertt Rd, Ransome	2 June 1999	Still active	14.5
LTR759	Watervale Pde Wakerley	26 August 2008	6 September 2012	4
W_R521	Wynnum Works Depot	1 January 1994	7 February 2001	7
W_R837	Wynnum Bowls Club	31 October 2001	Still active	12
BNR739	Wynnum Sewage Treatment Plant	15 January 1997	Still active	17

Table 3.4 indicates the availability of the rainfall gauge data for each of the selected storm events.

Table 3.4 – Rainfall gauge data availability

Gauge ID	Location	Calibration		Verification	
		2 March 2013	11 December 2010	20 May 2009	11 December 1995
LTR755	Harman Recreation Reserve	✓	✓	✓	×
LTR141	Rickertt Rd, Ransome	✓	✓	✓	×
LTR759	Watervale Pde Wakerley	×	✓	✓	×
W_R521	Wynnum Works Depot	×	×	×	✓
W_R837	Wynnum Bowls Club	✓	✓	✓	×
BNR739	Wynnum Sewage Treatment Plant	✓	✓	✓	✓

3.2.2 Continuous Recording Stream Gauges

Continuous recording stream height gauges collect instantaneous water level information over time. There is one stream gauge (W_E580) operational in the Wynnum Creek Catchment; this gauge is located near Byrneside Terrace in the lower section of the catchment. This gauge has been in operation since 1977 and was originally operated by the Irrigation and Water Supply Commission before being taken over by BCC in 1982.

This gauge has a small weir (crest level ~1.5 m AHD) downstream of the gauge, which results in the gauge readings not being subject to the normal tidal range. The location of the stream gauge is indicated in the previous Figure 3.1.

Digital stream gauge data is available for all calibration and verifications events used in this study, apart from the 11th December 1995 event. For this event, it was required to manually extract (digitise) the gauge readings from paper records. During this process it was also required to manually perform a time shift, as the time on the paper records was incorrect.

3.2.3 Maximum Height Gauges (MHGs)

Maximum Height Gauges (MHGs) record the maximum water level experienced in a flood event at the gauge location. Numerous MHGs exist in the Wynnum Creek catchment for which data availability is summarised in Table 3.5 and their locations are indicated in Figure 3.1.

Table 3.5 – Maximum Height Gauge data availability

Gauge ID	Location	Data Availability			
		2 March 2013	11 December 2010	20 May 2009	11 December 1995
W002	Coreen Street	-	-	-	✓
W100	d/s Tingal Road	✓	✓	✓	✓
W110	u/s Daisy Street	-	✓	✓	✓
W004	u/s Stradbroke Av.	-	-	-	✓
W120	d/s Chandos Street	✓ (1)	-	-	✓
W130	u/s Preston Road	✓	✓	-	✓
W140	d/s Wondall Road	✓	✓	✓	✓

1. Level from debris height, not used

3.2.4 Tidal Information

As there is no tide gauge at the mouth of Wynnum Creek, historic tidal information was obtained from the Brisbane Bar tide gauge; operated by Maritime Safety Queensland (MSQ). A shift was required for both the level and time data to account for the differences between the gauge location and the mouth of Wynnum Creek. Level and timing shifts were applied based on available data in the QLD Tide Tables (MSQ) booklet[3] for the year corresponding to each flood event. The tidal gauge data was available for all calibration and verification events.

The maximum tidal level recorded during each event is as follows:

- 2nd March 2013: 1.17 m AHD
- 11th December 2010: 1.01 m AHD
- 20th May 2009: 1.27 m AHD
- 11th December 1995: 0.84 m AHD

[3] Maritime Safety Queensland 2013, *Queensland Tide Tables*, MSQ, Queensland Government, Brisbane.
Maritime Safety Queensland 2010, *Queensland Tide Tables*, MSQ, Queensland Government, Brisbane.
Maritime Safety Queensland 2009, *Queensland Tide Tables*, MSQ, Queensland Government, Brisbane.
Maritime Safety Queensland 1995, *Queensland Tide Tables*, MSQ, Queensland Government, Brisbane.

3.3 Characteristics of Historical Events

3.3.1 March 2013 event

This event was a relatively small flooding event which produced a flood level of 4.72 m AHD just downstream of Tingal Road. The floodwater was generally limited to the channel with bank overtopping occurring in some localised areas.

Table 3.6 indicates the 4-day and 14-day antecedent rainfall as well as the total event rainfall at the three pluviographs. The catchment would have been quite saturated at the time of the event as it experienced approximately 50 mm of rainfall in the 4-day lead up to the event. The total event rainfall was reasonably consistent over the entire catchment, with between 115 and 141 mm falling on the 2nd March 2013. The most intense burst occurred over 3 hours between 1 pm and 4 pm, where approximately 80 mm of rainfall was recorded at LTR 755. The cumulative rainfall recorded by each rainfall gauge is plotted in Appendix A. Figure 3.2 presents a comparison of the Intensity-Frequency-Duration (IFD) curve for each pluviograph station against the IFD curves for Wynnum Creek catchment.

Table 3.6 - Rainfall characteristics (2nd March 2013 event)

Gauge ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
		14-day	4-day	2nd March	1st to 3rd March
LTR755	Harman Recreation Reserve	229	54	141	160
LTR141	Rickertt Rd, Ransome	291	65	115	136
W_R837	Wynnum Bowls Club	215	42	140	161

The equivalent design rainfall ARI based on the pluviograph at Wynnum Bowls Club (W_R837) was as follows:

- 1 hour rainfall: 1 in 1 year
- 2 hour rainfall: 1 in 1 year
- 6 hour rainfall: 1 in 2 years
- 12 hour rainfall: 1 in 3 years

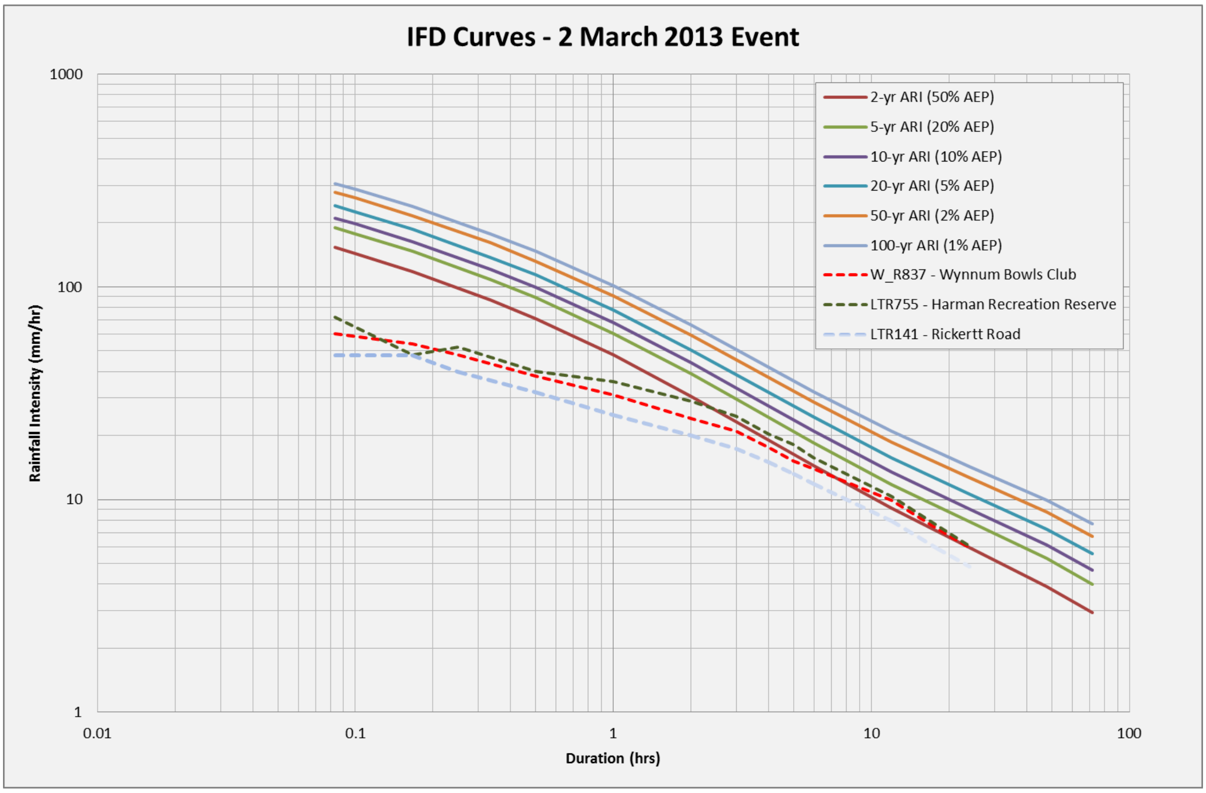


Figure 3.2: IFD Curve for March 2013 event.

3.3.2 December 2010 event

This event produced a level of 5.2 m AHD just downstream of Tingal Road, which was the fourth largest recorded flood level since 1967. The floodwater was generally limited to the channel with bank overtopping occurring in some localised areas.

Table 3.7 indicates the 4-day and 14-day antecedent rainfall as well as the total event rainfall at the three pluviographs. The catchment experienced approximately 10 to 15 mm of rainfall in the 4-day lead up to the event. The event was highly spatial with more intense rainfall falling in the lower section of the catchment, where approximately double the rainfall fell compared with the upper and middle sections of the catchment. The most intense burst occurred over 3 hours between 2 pm and 5 pm on the 11th December 2010, where approximately 80 mm of rainfall was recorded in the lower section of the catchment and 40 mm in the middle and upper sections of the catchment. The cumulative rainfall recorded by each rainfall gauge is plotted in Appendix A.

An IFD plot for each rainfall pluviograph is indicated in Figure 3.3. The IFD curves indicate that there is an uneven distribution of rainfall throughout the catchment, especially in the upper section of the catchment, where lower rainfall intensity is recorded.

Table 3.7 - Rainfall characteristics (11th December 2010 event)

Gauge ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
		14-day	4-day	11th December	10th to 12th December
LTR755	Harman Recreation Reserve	196	16	91	100
LTR759	Watervale Pde Wakerley	180	11	46	54
W_R837	Wynnum Bowls Club	173	11	106	116

The equivalent design rainfall ARI based on the pluviograph at Wynnum Bowls Club (W_R837) was as follows:

- 1 hour rainfall: 1 in 9 years
- 2 hour rainfall: 1 in 12 years
- 3 hour rainfall: 1 in 8 years
- 6 hour rainfall: 1 in 4 years
- 12 hour rainfall: 1 in 2 years

However as noted previously, the equivalent total catchment design rainfall ARI would be less than these values because of the spatial nature of the rainfall throughout the catchment.

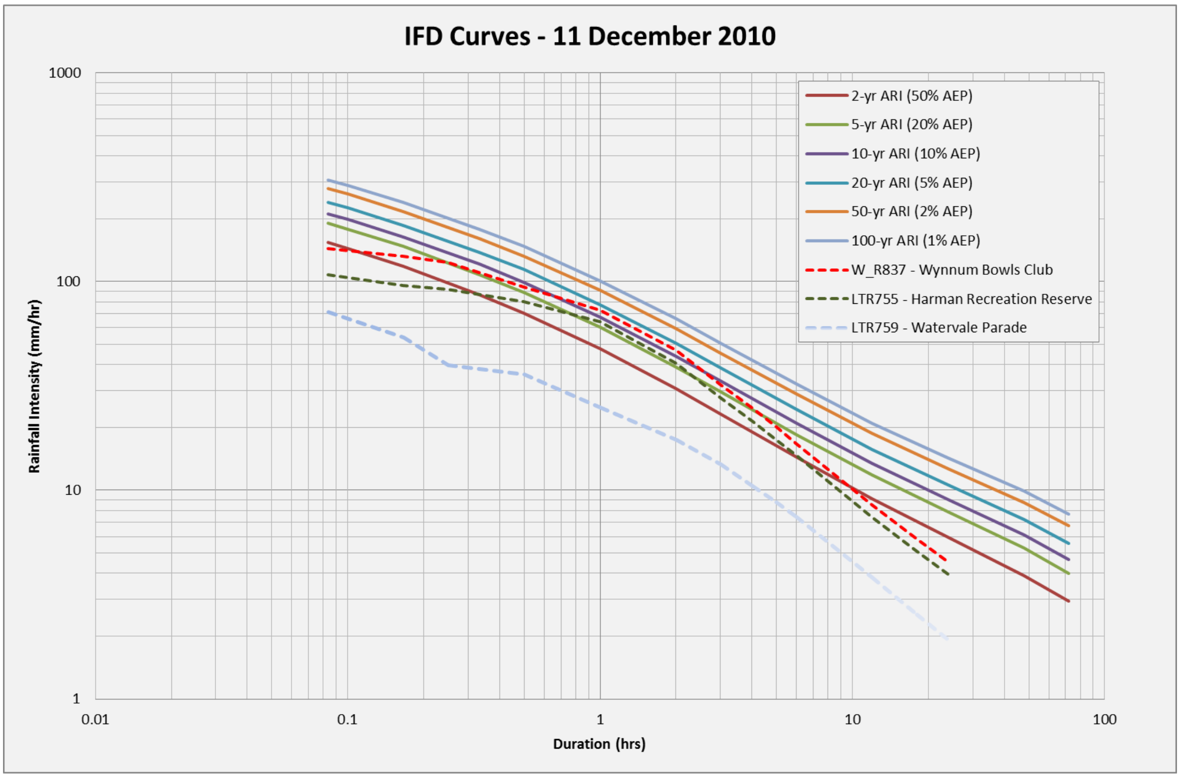


Figure 3.3: IFD Curve for December 2013 event.

3.3.3 May 2009 Event

This event was a relatively small flooding event which produced a flood level of 4.69 m AHD just downstream of Tingal Road. The floodwater was generally limited to the channel with bank overtopping occurring in some localised areas.

Table 3.8 indicates the 4-day and 14-day antecedent rainfall as well as the total event rainfall at the three pluviographs. The catchment would have been quite saturated at the time of the event as it experienced approximately 100 mm of rainfall in the 4-day lead up to the event. The total event rainfall was reasonably consistent over the entire catchment with between 186 and 206 mm falling between the 19th May and the 21st May 2009. Consistent rainfall totalling approximately 130 mm fell in the 14 hour period between 9:30 pm 19th May and 11:30 am 20th May 2009. The cumulative rainfall recorded by each rainfall gauge is plotted in Appendix A.

Table 3.8 - Rainfall characteristics (20th May 2009 event)

Gauge ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
		14-day	4-day	20th May	19th to 21th May
LTR755	Harman Recreation Reserve	99	98	122	206
LTR759	Watervale Pde Wakerley	88	85	124	201
W_R837	Wynnum Bowls Club	97	97	104	186

The equivalent design rainfall ARI based on the pluviograph at Wynnum Bowls Club (W_R837) was as follows:

- 1 hour rainfall: < 1 in 1 year
- 2 hour rainfall: 1 in 1 year
- 6 hour rainfall: 1 in 1 year
- 12 hour rainfall: 1 in 2 years

An IFD plot for each rainfall pluviograph is indicated in Figure 3.5. The IFD curves indicate that there are minor variations in the distribution of rainfall throughout the catchment

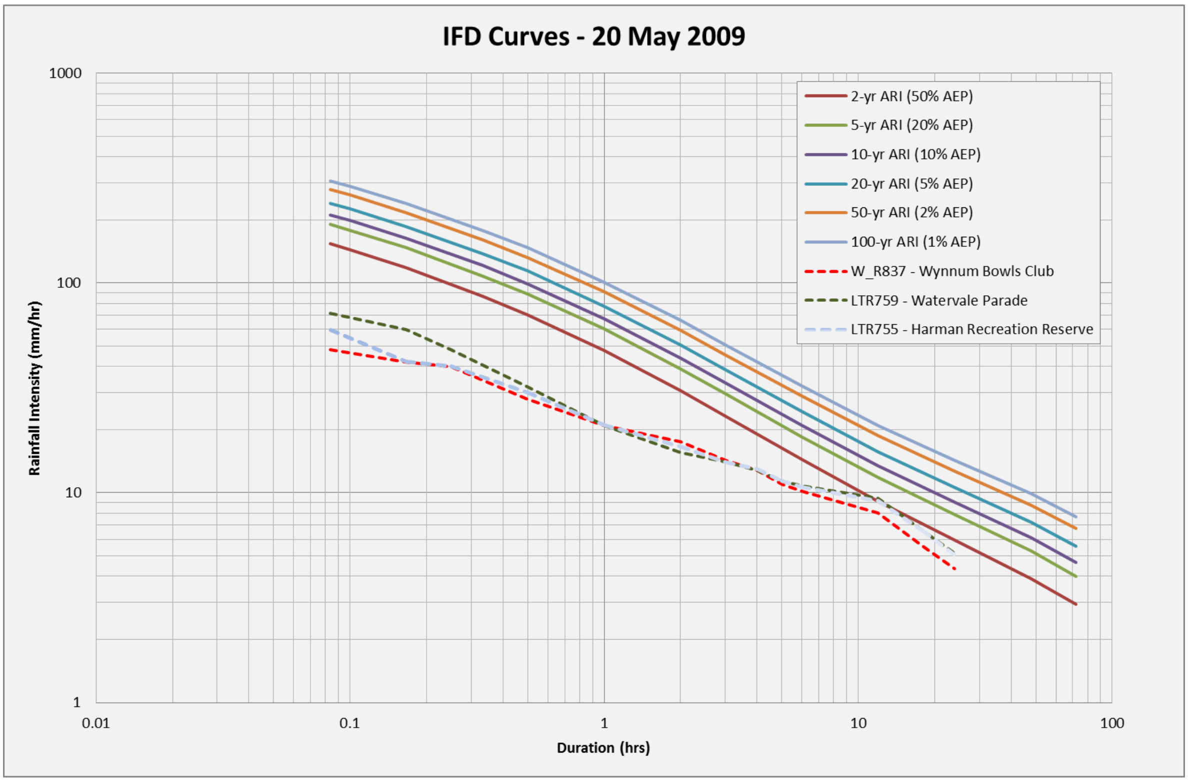


Figure 3.4: IFD Curve for May 2009 event.

3.3.4 December 1995 event

This event was not large in flooding terms and produced a flood level of 4.71 m AHD just downstream of Tingal Road. The floodwater was generally limited to the channel with bank overtopping occurring in some localised areas.

Table 3.9 indicates the 4-day and 14-day antecedent rainfall as well as the total event rainfall at the single pluviograph. The catchment would have been dry as there was virtually no rainfall in the weeks preceding the event. A short heavy burst of approximately 70 mm fell for one hour between 5:00 pm and 6:00 pm on the 11th December 1995. The cumulative rainfall recorded by the rainfall gauge is plotted in Appendix A. An IFD plot for the rainfall pluviograph is indicated in Figure 3.5.

Table 3.9 - Rainfall characteristics (11th December 1995)

Gauge ID	Location	Antecedent Rainfall (mm)		Event Rainfall (mm)	
		14-day	4-day	11th March	10th to 12th December
W_R521	Wynnum Works Depot, Pine St	19	0	83	85

The equivalent design rainfall ARI based on the pluviograph at Wynnum Bowls Club (W_R837) was as follows:

- 1 hour rainfall: 1 in 10 years
- 2 hour rainfall: 1 in 5 years
- 6 hour rainfall: 1 in 2 year
- 12 hour rainfall: 1 in 1 year

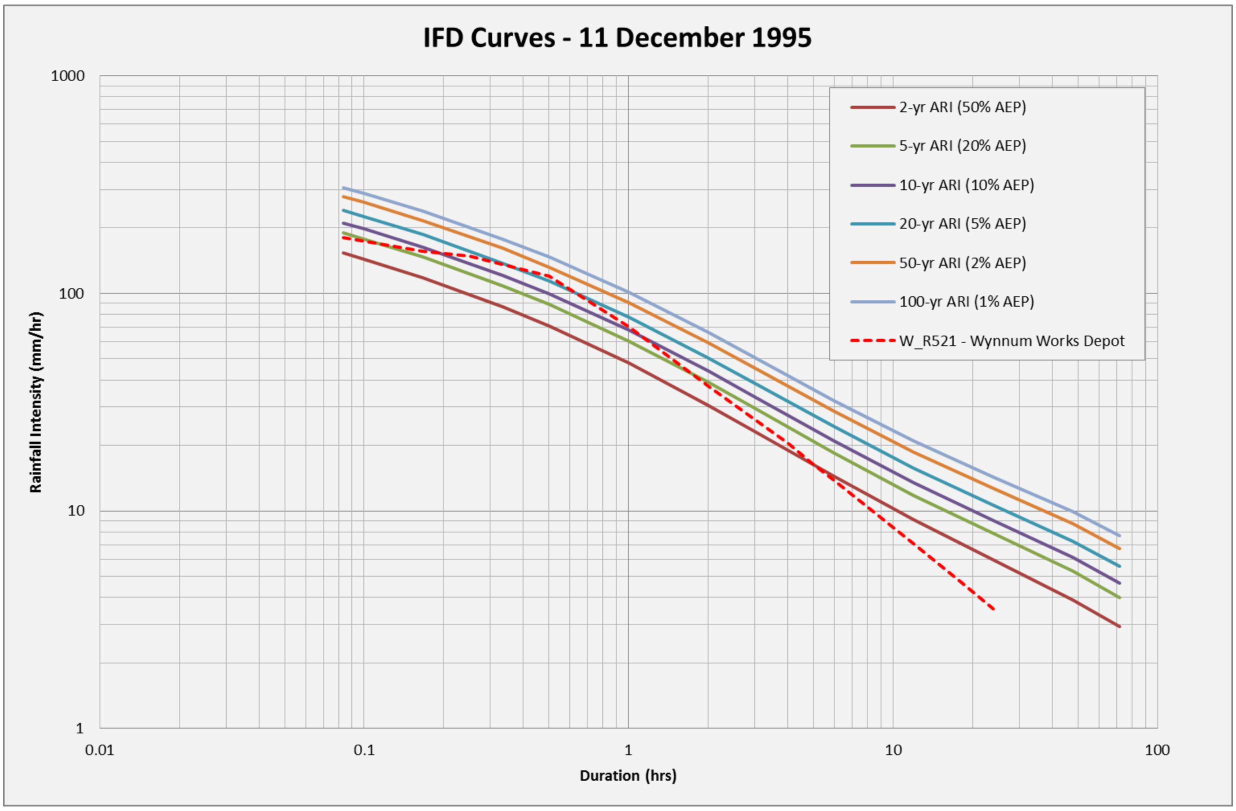


Figure 3.5: IFD Curve for December 1995 event.

4.0 Hydrologic Model Development and Calibration

4.1 Overview

The hydrologic model simulates the rainfall-runoff process within the catchment and calculates the flow hydrograph at the outlet of each sub-catchment. The XP-RAFTS model for the Wynnum Creek Catchment was initially developed as part of the 1997 Wynnum Creek Flood Study, by GHD Engineers and Kinhill Engineers.

Preliminary assessment of the 1997 XP-RAFTS model indicated that the model was required to be modified to address the following:

- Update the model to the latest version of the software.
- Update of sub-catchment delineation as a result of new development.
- Update of sub-catchment delineation to produce better definition in the hydraulic model.
- Update the model to incorporate sub-catchment specific slopes, rather than global values.
- Review and update the catchment parameters (e.g. impervious percentage and PERN) to suit the revised sub-catchment delineation.
- Review and update the Byrneside Terrace Gauge rating curve (as required).
- Review the requirement to include the node storage (detention basin) at Kitchener Park. Review the stage versus storage relationship and head versus discharge relationship for this storage and confirm whether it is adequate to adopt for this study.

4.2 Sub-catchment Data

4.2.1 General

This section describes the sub-catchment parameters used in the XP-RAFTS model. The adopted sub-catchment parameters for the calibration and verification events are presented in Appendix B. The same sub-catchment parameters have been used for all events due to the relatively recent age of the calibration and verification events and the minimal changes in catchment / channel topography and development during this period.

4.2.2 Sub-catchment Delineation

The Wynnum Creek XP-RAFTS model comprises 21 sub-catchments and the layout is indicated in Figure 4.1. Total catchment and sub-catchment delineation was adjusted to better represent current conditions and to produce better definition for the inflows into the TUFLOW hydraulic model.

4.2.3 Sub-catchment Slope

As noted above, the review of the 1997 XP-RAFTS model revealed that a single (global) value had been used to represent the sub-catchment slope for all sub-catchments. Common practice is to determine a unique slope for each sub-catchment, therefore the model was updated and a slope calculated for each sub-catchment. Sub-catchment slopes have been calculated from the topography by identifying indicative flow paths and associated equal area slopes.

4.2.4 Percentage Impervious

The Wynnum Creek catchment is considered to be fully urbanised. The land-use and impervious areas have been identified using BCC aerial photography and BCC City Plan[4]. The adopted land-use for the calibration and verification events is shown on a catchment map in Appendix C.

Table 4.1 indicates the percentage impervious values adopted for the various land-use types. Where XP-RAFTS sub-catchments contained more than one type of land-use, weighted averages of the percentage imperviousness were applied for each sub-catchment.

Table 4.1 – Sub-catchment parameter by land-use

Land-use Type	% Impervious
Emerging Communities	35
Low Density Residential	55
Low-Medium Density Residential	65
Medium Density Residential	75
High Density Residential	85
Community Use Area Community Facilities	45
Community Use Area Education Purposes	50
Community Use Area Health Care Purposes	80
Community Use Area Utility Services	55
Community Use Area Emergency Services	70
Community Use Area Railway	70
Multi-purpose Centre Suburban Centre	95
Multi-purpose Centre Convenience Centre	80
Light Industry	90
Sport And Recreation	10
Park Land	0
Creek and Other Pervious Area (Parks)	0
Road Reserve	55

4.2.5 Hydrologic Roughness (PERN)

The hydrologic roughness parameter (PERN) is input as a Manning's 'n' representation of the average sub-catchment roughness. To calculate the PERN value for each sub-catchment, a weighted average value was determined using a value of n = 0.015 for the impervious areas and a value of n = 0.04 for the pervious area. This generally produced a PERN value of approximately 0.03 for the

[4] Brisbane City Council 2000, *Brisbane City Plan 2000*, BCC, Brisbane

majority of sub-catchments, which would be considered representative of low density residential, which is the dominant land-use.

4.2.6 Link and Routing Parameters

Routing of the open channel links (i.e. Wynnum Creek and the East Drain) was undertaken using the Muskingum-Cunge methodology. The program calculates the Muskingum K and X values based on the channel cross-sectional and longitudinal characteristics. The cross-sectional shape was reviewed and adopted from the 1997 XP-RAFTS model.

Links representing below ground stormwater drainage conduits were modelled using the link-lag approach. This approach translates the base of the hydrograph (without attenuation) based on the input lag time. The lag time was initially calculated assuming an average travel time of 1 m/s.

4.3 Event Rainfall

4.3.1 Observed Rainfall

Recorded data from each calibration and verification event was incorporated into the XP-RAFTS model using a standard HYDSYS database format. The HYDSYS rainfall database which was used in the hydrological modelling, comprises recorded rainfall at five minutes intervals, noting that the pluviograph only records information when 1 mm or more of rain has fallen. This enabled the full rainfall period for each of the events to be modelled using a fast and reliable method.

Thiessen Polygons were utilised for each event to enable the gauged rainfall to be apportioned to each of the sub-catchments in the XP-RAFTS model. Those sub-catchments which fell totally within a polygon were fully assigned to the respective pluviograph. Those sub-catchments which bridged across two of more polygons were proportioned to the respective pluviograph based on the proportion of area within each polygon. The Thiessen Polygon distributions for the four events are presented in Appendix A for reference.

Three of the four events experienced consistent rainfall across the entire catchment, with only the December 2010 event having significant spatial variation, as previously mentioned in Section 3.3.2. During the calibration process for the 2010 event, it was established that the Thiessen Polygon distribution was not representative in the middle to upper parts of the catchment; therefore some minor adjustments to the pluviograph weightings were undertaken.

4.3.2 Rainfall Losses

The Initial Loss (IL) and Continuing Loss (CL) methodology was used to simulate the rainfall losses.

The IL (mm) is known to be the amount of rainfall that occurs before the start of surface runoff. The initial loss comprises factors such as interception storage (e.g. tree leaves); depression storage (e.g. ditches, surface puddles, etc.) and the initial infiltration capacity of the soil, whereby a dry soil has a larger capacity than a saturated soil.

The CL (mm/hr) is assumed to be the average loss rate throughout the remainder of the rainfall event and is predominantly dependant on the underlying soil type and porosity.

4.4 Calibration and Verification Procedure

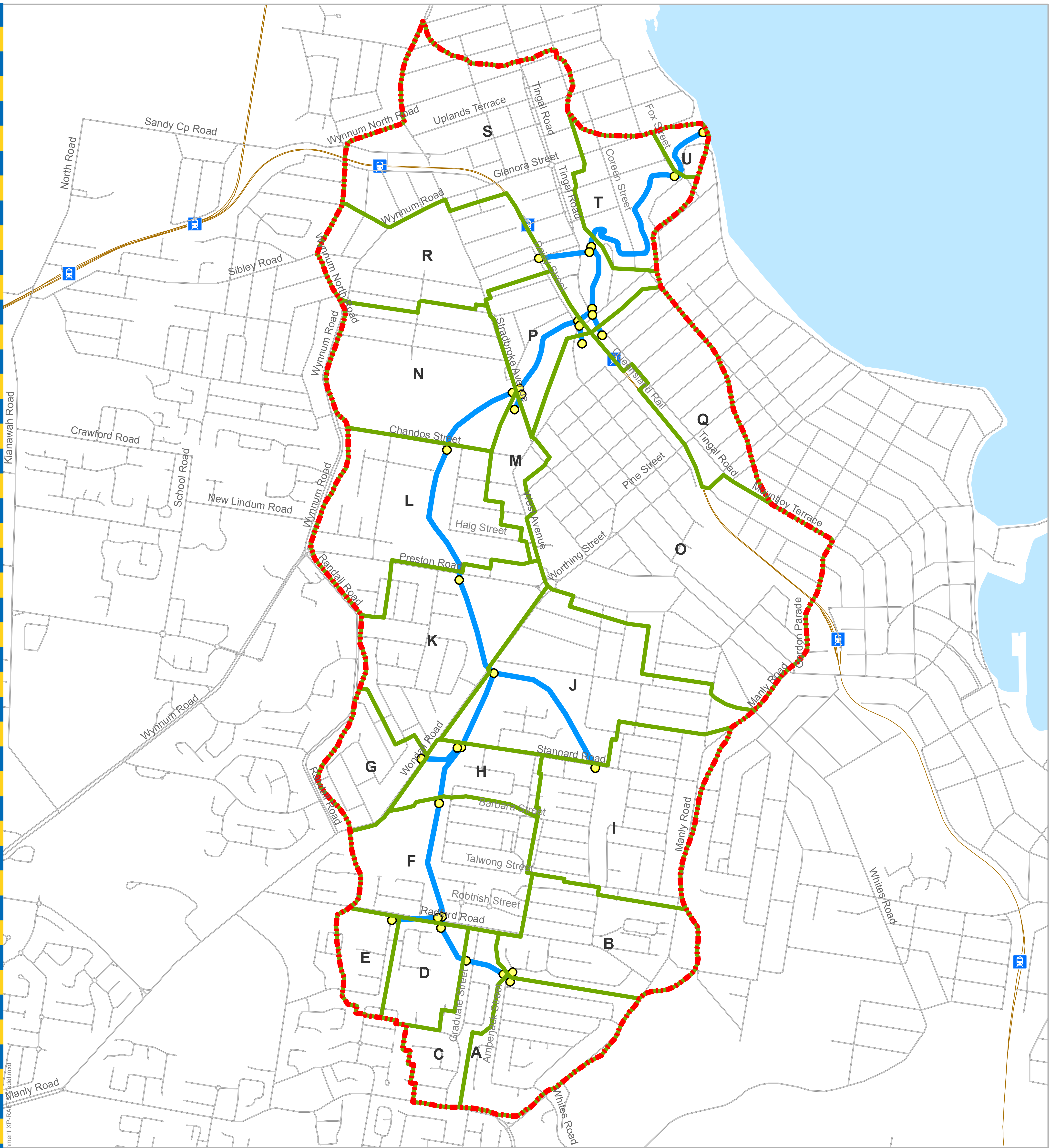
4.4.1 General

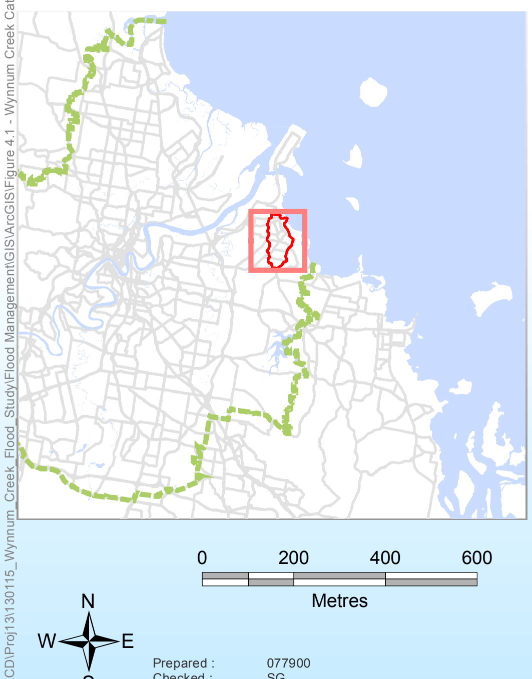
The calibration and verification process was adopted to suit the study objectives and requirements. The initial requirement was to produce a hydrologic model sufficiently robust to accurately predict design discharges without the need to run the hydraulic model. This requirement meant that the approach adopted was to undertake a separate hydrologic calibration to ensure the XP-RAFTS model was suitable to be used as a "standalone" model. The general approach adopted for the calibration and verification is as outlined in Section 4.4.2.

4.4.2 Methodology

The methodology applied to the calibration and verification of the XP-RAFTS model was as follows:

1) Input the observed rainfall gauge data and apportion the rainfall to each sub-catchment. This was undertaken using the Thiessen Polygon methodology as described in Section 4.3.
2) Establish an appropriate rating curve at the stream gauge and convert the stage recordings to flow. This is detailed further in Section 4.4.3.
3) Run the calibration events (i.e. March 2013 and December 2010) through the XP-RAFTS model and compare the simulated results against the observed flow records.
4) Iteratively adjust the model parameters and re-run the model to achieve a good fit with the observed data. The predominant model parameters adjusted included the IL (mm), CL (mm/hr) and the storage delay time coefficient multiplier (Bx). However, the link-lag timing and also the Manning's 'n' value of the routing link were also adjusted, if considered appropriate.
5) Adopt model parameters (typically CL and Bx) based on the calibration results.
6) Run the verification events (i.e. May 2009 and December 1995) through the calibrated XP-RAFTS model and compare the simulated results against the observed flow records.
7) Make adjustments to the initial loss (as required) to represent the event specific rainfall lost at the start of the event.
8) Repeat steps 2 to 7 (as necessary) following the results of the hydraulic model simulations. Refer to Section 5 for more detail on the hydraulic modelling.





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Project Number : 130115

File : G:\BI\CD\Proj13\130115_Wynnum_Creek_Flood_Study\Flood Management\GIS\ArcGIS\Figure 4.1 - Wynnum Creek Catchment XP-RAFTS Model.mxd

Legend

- Wynnum Catchment
- RAFTS Subcatchments
- RAFTS Nodes
- RAFTS Links/Routes
- Streets
- Railway Stations
- Railway Line

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Figure 4.1: Wynnum Creek Catchment XP-RAFTS Model

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4.4.3 Derivation of the Rating Curve at Byrneside Terrace Gauge

In order to undertake the hydrological calibration and verification to the stream gauge in the vicinity of Byrneside Terrace, it was necessary to establish an appropriate rating curve. BCC Hydrometrics does not keep records of rating curves for stream gauges; therefore it was required to generate the rating curve using the TUFLOW hydraulic model, developed for this flood study. For further discussion on the TUFLOW model, refer to Section 5.

The adopted rating curve at the Byrneside Terrace Gauge is shown in Figure 4.2. Checks were undertaken to establish whether the selection of the tidal boundary level influenced the rating curve. It was found that the rating curve was not influenced by tidal levels up to 1.6 m AHD. Checks with a tidal level above 1.6 m AHD were not undertaken.

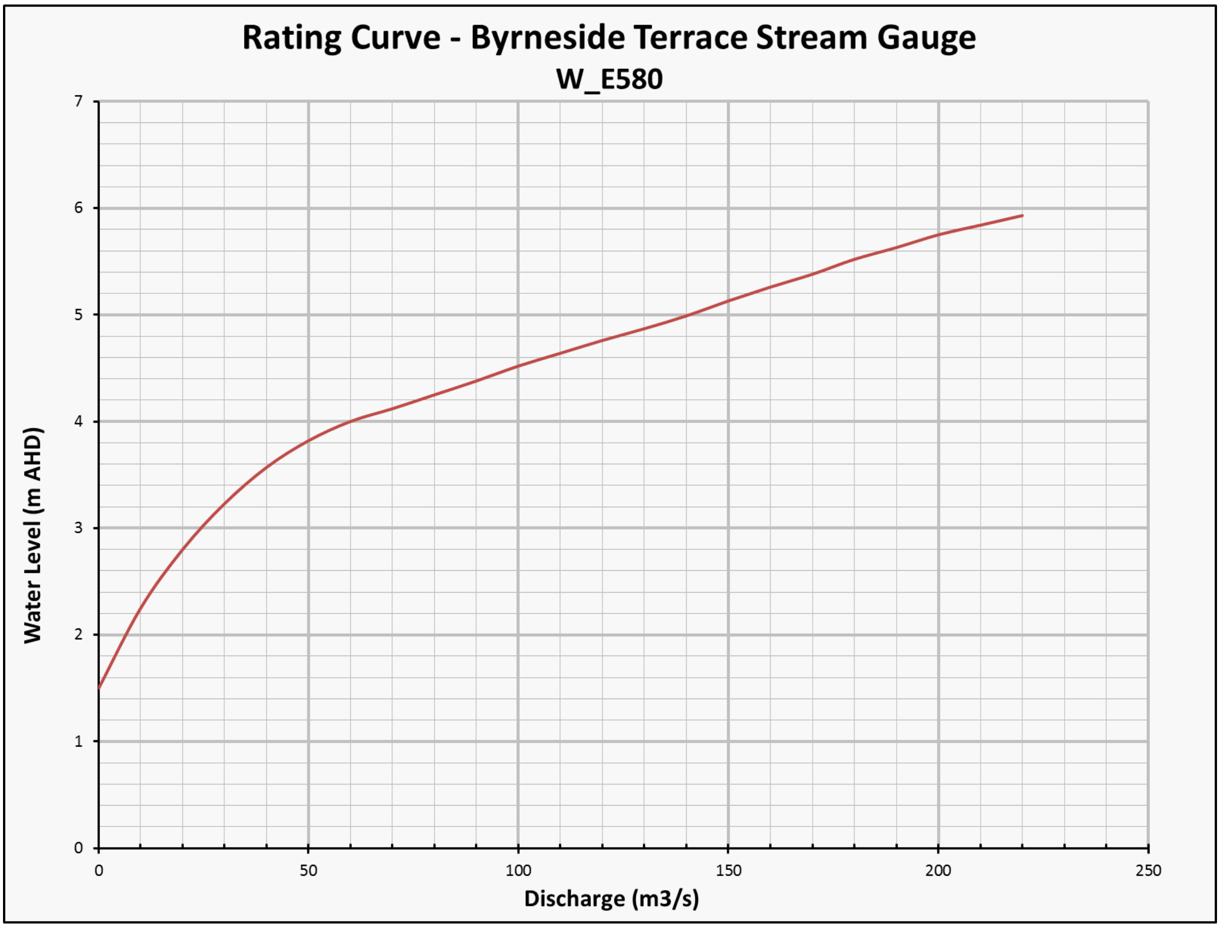

Figure 4.2: Adopted Rating Curve - Byrneside Terrace Stream Gauge (W_E580)

4.5 Hydrologic Model Calibration Results

4.5.1 March 2013

Figure 4.3 provides a comparison of the XP-RAFTS results and the gauged flows (established using the adopted rating curve) at the Byrneside Terrace Stream Gauge for the March 2013 event. The results indicate a reasonable fit to the complex multi-peaked event, with good timing of the peak / troughs. The XP-RAFTS model has over predicted the peak flow by approximately 25 % at the stream gauge, when compared to the rated flow.

The adopted XP-RAFTS parameters as part of the calibration were as follows:

- IL = 0 mm
- CL = 0 mm/hr
- Bx = 2.5

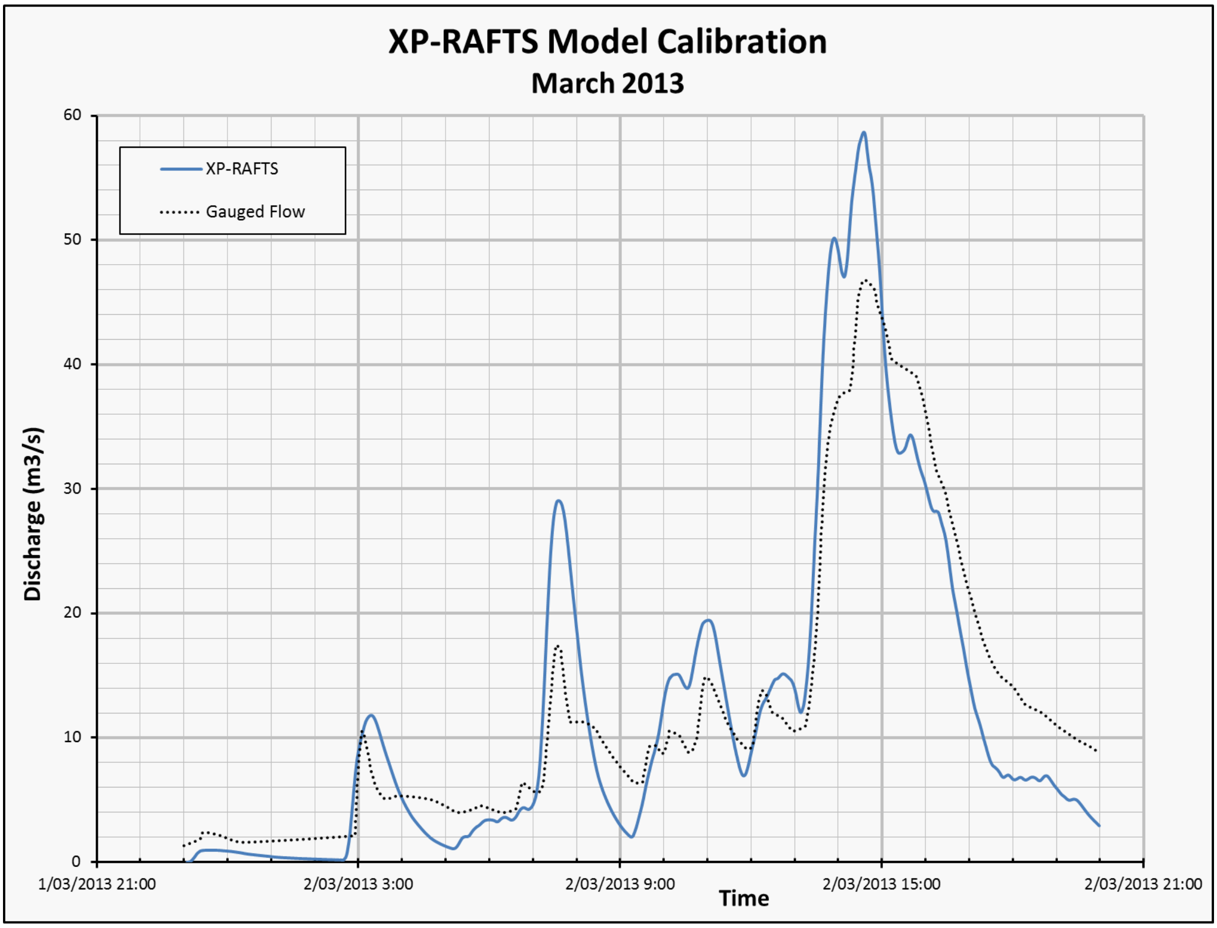

Figure 4.3: XP-RAFTS Model Calibration (March 2013)

4.5.2 December 2010

Figure 4.4 provides a comparison of the XP-RAFTS results and the gauged flows at the Byrneside Terrace Stream Gauge for the December 2010 event. The results indicate a reasonable fit to the single peaked event, with a good replication of the rising limb and the timing of the peak flow. The XP-RAFTS model has over predicted the peak flow by approximately 13 % at the stream gauge, when compared to the rated flow.

The adopted XP-RAFTS parameters as part of the calibration were as follows:

- IL = 20 mm
- CL = 0 mm/hr
- Bx = 2.5

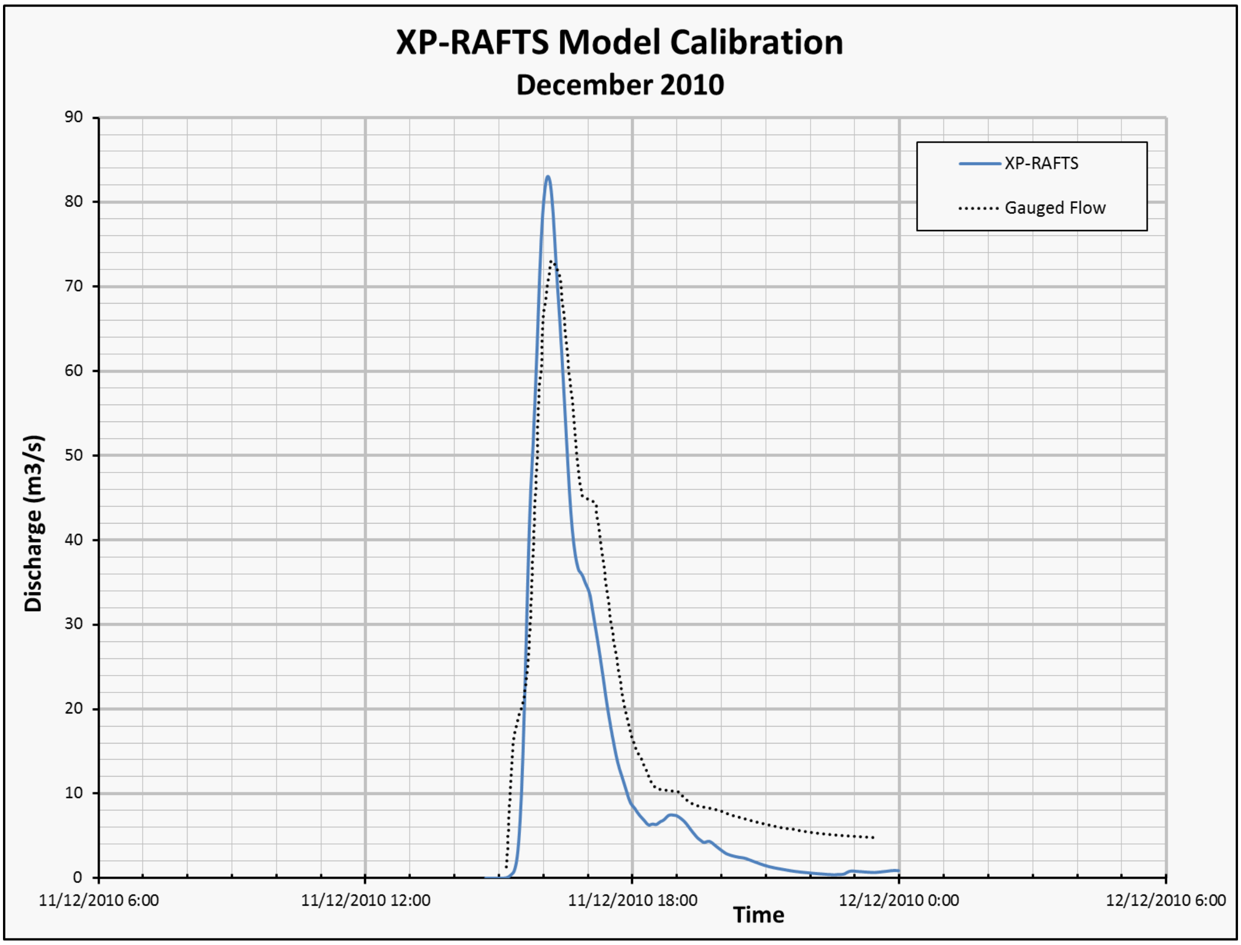

Figure 4.4: XP-RAFTS Model Calibration (December 2010)

4.6 Hydrologic Model Verification Results

4.6.1 Adopted model parameters

Table 4.2 indicates the parameters adopted from the hydrologic calibration of the two historical events. These parameters were used to verify the XP-RAFTS model to the two verification events (i.e. May 2009 and December 1995).

Table 4.2 – Adopted XP-RAFTS parameters

Parameter	Description	Adopted Value
n	Storage non-linearity exponent	-0.285
Bx	Storage delay time coefficient multiplier	2.5
CL	Continuing Loss (mm / hr)	0

4.6.2 May 2009

Figure 4.5 provides a comparison of the XP-RAFTS results and the gauged flows at the Byrneside Terrace Stream Gauge for the May 2009 event. The results indicate a good fit to the complex multi-peaked event, with a good replication of the timing of the peaks / troughs and overall hydrograph shape. The XP-RAFTS model has over predicted the peak flow by approximately 13 % at the stream gauge, when compared to the rated flow.

The adopted XP-RAFTS parameters as part of the verification were as follows:

- IL = 0 mm
- CL = 0 mm/hr
- Bx = 2.5

4.6.3 December 1995

Figure 4.6 provides a comparison of the XP-RAFTS results and the gauged flows at the Byrneside Terrace Stream Gauge for the December 1995 event. The results indicate a good fit to the single peaked event, with a good replication of the timing and magnitude of the peak flow.

The adopted XP-RAFTS parameters as part of the verification were as follows:

- IL = 44 mm
- CL = 0 mm/hr
- Bx = 2.5

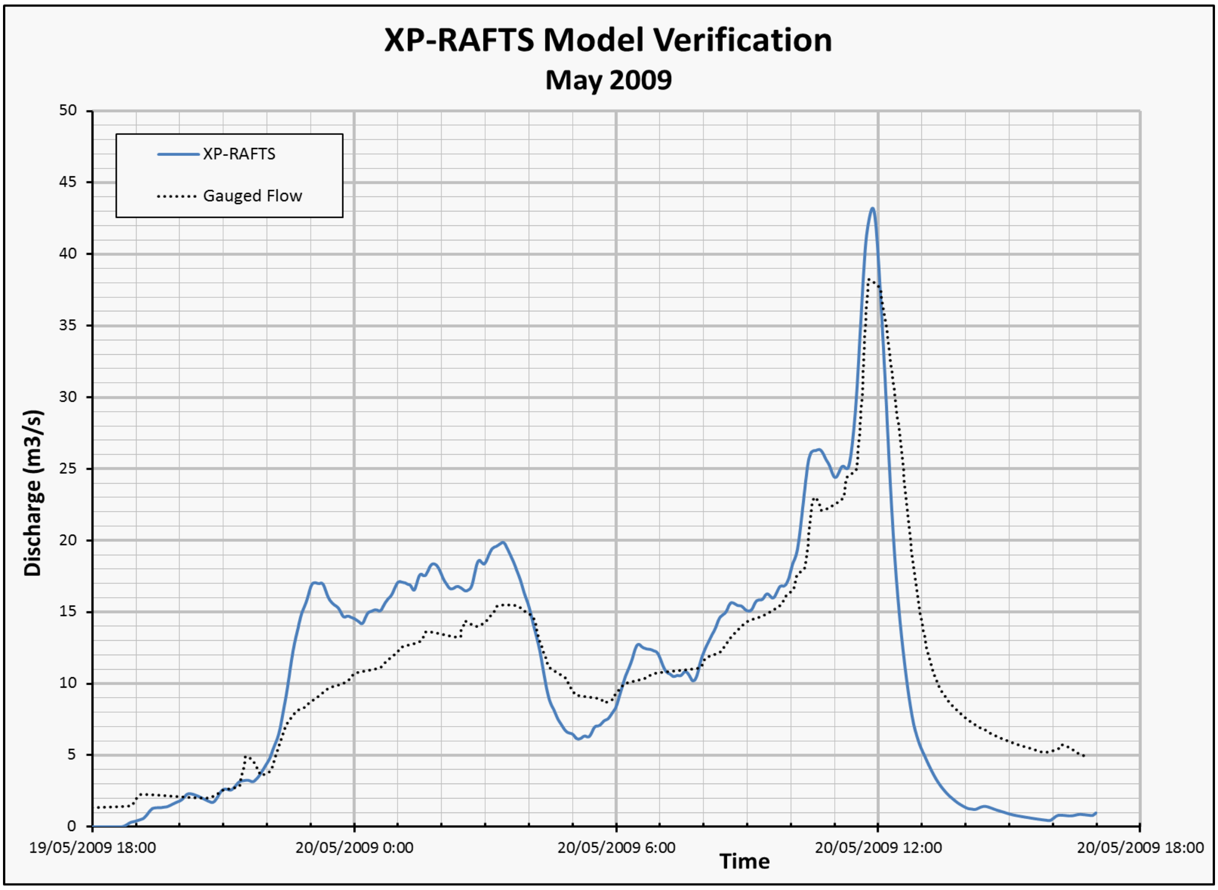

Figure 4.5: XP-RAFTS Model Verification (May 2009)

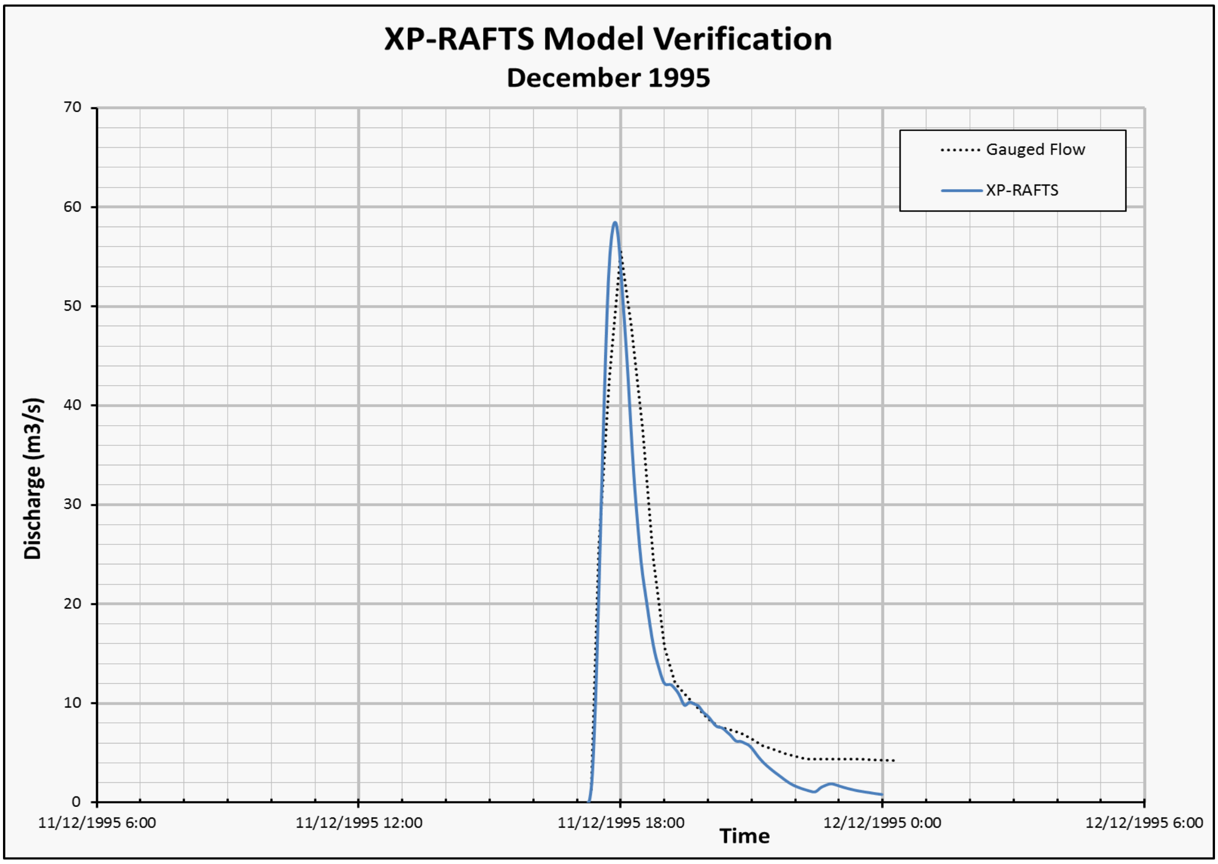

Figure 4.6: XP-RAFTS Model Verification (December 1995)

4.7 Discussion on the Hydrologic Calibration and Verification Results

The results of the hydrologic calibration and verification indicate that the XP-RAFTS model was able to achieve a reasonable correlation to the rated flows at the Byrneside Terrace Stream Gauge for all events modelled.

Peak Flow

The XP-RAFTS model generally over-predicted the peak flow for each event, with the difference ranging from 4 % to 25 % and the mean value being 14 % above the rated peak flow. The timing of the peak flow was generally very good for all events modelled.

General timing of peak and troughs

The model was able to adequately reproduce the timing of the single peak and multiple peak events.

Rising Limb

The model was able to adequately reproduce the timing and slope of the hydrograph rising limb.

Recession Limb

The model was consistently unable to accurately reproduce the timing of the recession limb of the hydrograph. In all instances the XP-RAFTS model receded quicker than the gauged hydrograph. This consistent trend would tend to indicate that there are some storage effects in the system that the XP-RAFTS model was unable to replicate. As all the flooding events were typically quite small (where the flows were generally confined to the channel) this cannot be attributed to a deficit in floodplain storage. The most likely cause would be the minor attenuation caused by hydraulic structures such as those between Daisy Street and Tingal Road as well as the access road culvert at Wynnum Leagues Club.

5.0 Hydraulic Model Development and Calibration

5.1 Overview

The previous hydraulic model of Wynnum Creek was a 1-dimensional MIKE11 model, developed for the 1997 Flood Study. To achieve best practice, it was considered appropriate to upgrade the 1-dimensional model to a 1-dimensional / 2-dimensional model. This would provide better representation of the floodplain flooding characteristics in the middle to lower sections of the creek as well as a more efficient tool to produce flood mapping products.

The 2-dimensional TUFLOW hydrodynamic model (version 2012-05-AE) was selected for the hydraulic analysis of Wynnum Creek.

5.2 Available Data

The following data was utilised in the development of the TUFLOW model:

- MIKE11 model – 1997 Wynnum Creek Flood Study
- BCC 1997 cross-section survey
- BCC aerial photography – 2012, 2011, 2009, 2007, 2005, 2001, 1999, 1997 and 1995
- BCC 2009 Airborne Laser Scanning (ALS) data
- Current version of BCC City Plan
- Hydraulic structure drawings / reference sheets. Refer to Appendix G for further details.
- BCC Cadastre and GIS databases

5.3 Model Development

5.3.1 Model Schematisation

Figure 5.1 indicates the extents of the TUFLOW model, as well as the inflow locations and the hydraulic structures included in the model. The hydraulic model includes the full length of the open waterway of Wynnum Creek, as well as the open channel tributary, referred to as the East Drain.

Wynnum Creek (Upper) refers to the section of the creek from of Amberjack Street to Wondall Road. Wynnum Creek (Lower) refers to the section of the creek from Wondall Road to its outfall at Moreton Bay. The East Drain extends from Stannard Road to Wondall Road, where it joins Wynnum Creek, upstream of the Wondall Road Culvert.

The model consists largely of a 1d-2d linked schematisation, with the 1d section modelled in ESTRY and the 2d section in TUFLOW. The 1d-2d schematisation extends from Graduate Street to Daisy Street and includes the East Drain.

There are two areas where the model is fully 2d. The first section is between Amberjack and Graduate Streets at the upper end of the open waterway. Initially, this section was modelled as 1d-2d, however because of numerous model instabilities was changed to fully 2d. The second area extends from downstream of Daisy Street to the creek mouth at Moreton Bay.

5.3.2 Topography

<u>1d Domain</u>

The 1d channel was generally represented by utilising the cross-sectional information from the 1997 MIKE11 model, with reference to the 2009 ALS data. These cross-sections were surveyed in 1997 for the purposes of this previous flood study. In many sections of the 1d channel, the ALS represented the channel bathymetry well, as the channel consisted of an engineered open waterway in which there was no standing water. At these locations, the 1997 MIKE11 model cross-sectional information was cross-referenced against the ALS and the most appropriate data utilised, which at times consisted of a merge between the data sets.

<u>2d Domain</u>

The 2d bathymetry was created using the 2009 BCC ALS data. The triangulated ALS data was converted to a 2 m grid digital elevation model (MGA Zone 56) for use with the TUFLOW model. Detailed checks have not been undertaken on the accuracy of the ALS data. It is assumed that the data is representative of the topography and "fit for purpose."

From Daisy Street to just upstream of Fox Street, the 2d channel bathymetry was manually adjusted utilising the 1997 MIKE11 cross-sectional information.

From Fox Street to the creek mouth, the ALS data was only representative of the water surface level in the creek channel. This required substantial adjustment to the 2d grid bathymetry for the full width of the water surface, which was based on the 1997 MIKE11 cross-sectional information.

At the 2d bridge crossings, localised adjustments have been made to ensure the bridge opening area is representative.

5.3.3 Land Use

The Manning's 'n' values shown in Table 5.1 were adopted within the 2d section of the TUFLOW model. The assignment of the appropriate roughness values to the land-use / topographical feature was based upon experience with similar studies and relevant hydraulic literature.

The discretation of the land-use and topographical areas was undertaken utilising a combination of BCC aerial photography, BCC City Plan and a number of site visits.

In the 1d ESTRY section, the Manning's 'n' values ranged from 0.013 to 0.05, depending on the type of channel material and degree of vegetation.

5.3.4 Hydraulic Structures

<u>Culverts and Bridges</u>

The major bridge and culvert structures within the model domain were represented in the TUFLOW model. These structures generally consisted of road crossings and the more significant footbridge crossings. Table 5.2 indicates the location and details of the structures as well as the modelling approach used.

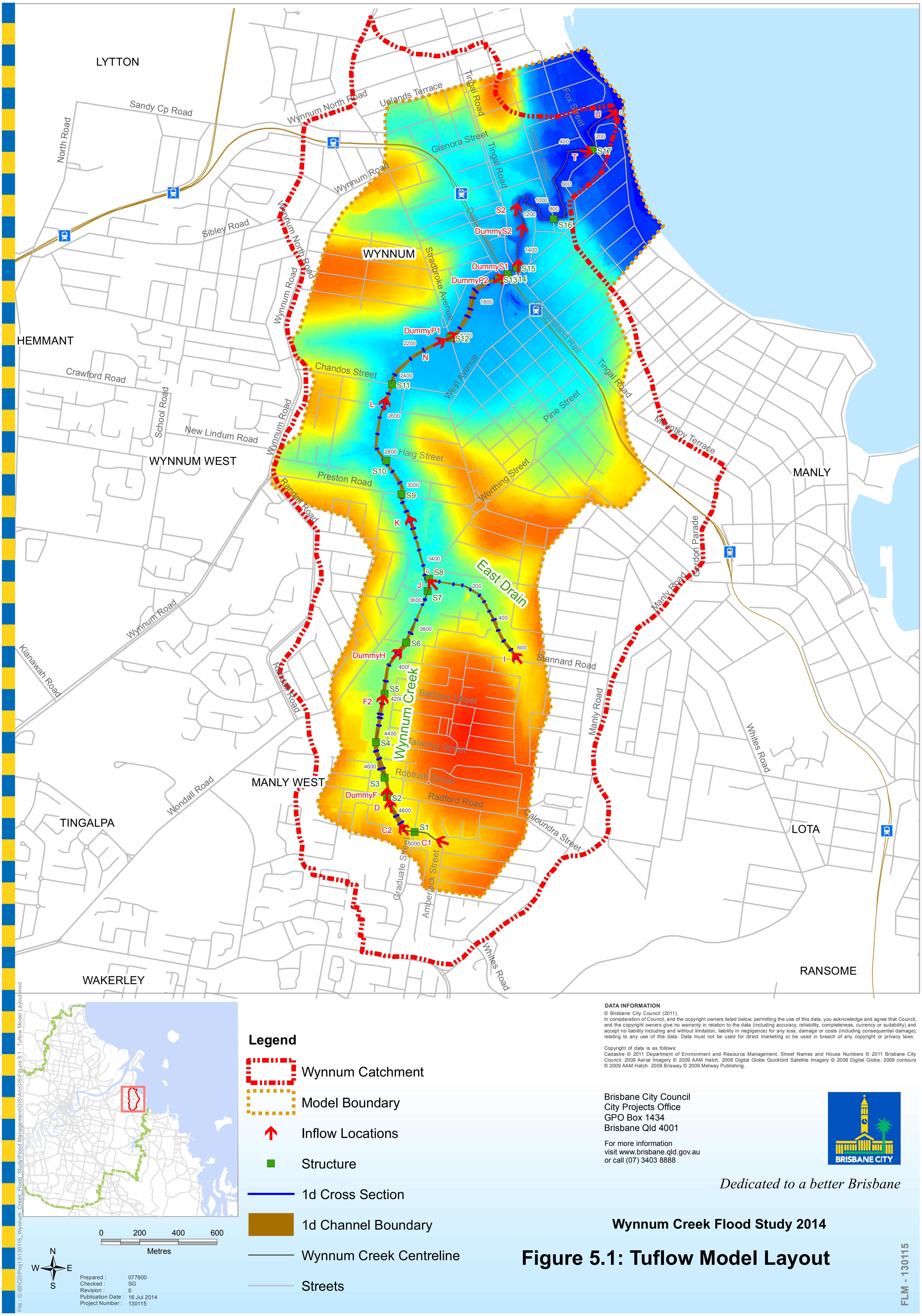

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Figure 5.1: Tuflow Model Layout

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File : G:\BICD\Proj\13\130115_Wynnum_Creek_Flood_Study\Flood Management\GIS\ArcGIS\Figure 5.1 - Tuflow Model Layout.mxd

Table 5.1 – Adopted roughness parameters

Topographical feature / Land-use	Adopted Manning's 'n'
City Plan Land-use	
Community Use Area Community Facilities	0.10
Community Use Area Education Purposes	0.10
Community Use Area Emergency Services	0.15
Community Use Area Health Care Purposes	0.15
Community Use Area Railway	0.04
Community Use Area Utility Services	0.04
Emerging Communities	0.06
Sport And Recreation	0.035
High Density Residential	0.15
Low – Medium Density Residential	0.15
Low Density Residential	0.12
Light Industry	0.15
Multi-Purpose Centre Convenience Centre	0.15
Multi-Purpose Centre Suburban Centre	0.15
Park Land	0.04
Sports and Recreation	0.04
Additional Roughness	
Roads	0.02
Channel – smooth (e.g. concrete)	0.015
Channel – smooth to medium	0.025
Channel - medium	0.035
Channel – medium to rough	0.05
Vegetation – little or none (e.g. grass)	0.035
Vegetation – light density	0.05
Vegetation – medium density	0.08
Vegetation – medium to high density	0.12
Vegetation – high density	0.15
Buildings	1.00
Minimum Riparian Corridor (MRC)	0.15

Table 5.2 – Hydraulic Structures represented in the TUFLOW model

Reach	Structure ID	Structure location	Structure details	Modelled structure representation	Origin of data used for coding the structure
Upper	S1	Graduate Street	2 / 1650 RCP	1d culvert / 2d weir	BCC Stormwater database
Upper	S2	Radford Road	2 / 525 RCP + 3 / 1800 RCP	1d culvert / 2d weir	1997 MIKE11 model plus onsite measurements
Upper	S3	Robtrish Street	Single span footbridge	1d bridge / 1d weir	Design drawings plus onsite measurements
Upper	S4	Talwong Street	Single span footbridge	1d bridge / 1d weir	Design drawings plus onsite measurements
Upper	S5	Barbara Street	Single span footbridge	1d bridge / 1d weir	Design drawings plus onsite measurements
Upper	S6	Stannard Road	3 / 1800 RCP	1d culvert / 2d weir	1997 MIKE11 model
Upper	S7	Leagues club access	1 / 750 RCP + 2 / 1200 RCP	1d culvert / 1d weir	1997 Flood Study plus onsite measurements
Upper	S8	Wondall Road	4 / 1800 RCP	1d culvert / 2d weir	1997 MIKE11 model
Lower	S9	Preston Road	Single span bridge	1d bridge / 2d weir	Design drawings plus onsite measurements
Lower	S10	Adjacent Haig Street	3 / 1800 RCP	1d culvert / 2d weir	1997 MIKE11 model
Lower	S11	Chandos Street	6 / 1800 RCP	1d culvert / 2d weir	1997 MIKE11 model
Lower	S12	Stradbroke Avenue	Single span bridge	1d bridge / 2d weir	Design drawings plus onsite measurements
Lower	S13	Daisy Street	2 / 2750 x 2500 RCBC	1d culvert / 2d weir	Design drawings plus onsite measurements
Lower	S14	Railway	Single span bridge	2d layered flow constriction	Design drawings
Lower	S15	Tingal Road	Two span bridge	2d layered flow constriction	Design drawings plus onsite measurements
Lower	S16	Coreen Street	Single span footbridge	2d layered flow constriction	Design drawings
Lower	S17	Fox Street	Two span bridge	2d layered flow constriction	Design drawings

In the 1d-2d linked section of the model, either of the following two approaches was used:

- 1d representation of the waterway opening with a 1d representation of the overtopping.
- 1d representation of the waterway opening with a 2d representation of the overtopping.

In the 2d section of the model, the bridges were represented using the fully 2d "layered flow constriction" approach.

The modelled head-loss across selected structures was checked utilising the HEC-RAS modelling software, as recommended in the TUFLOW manual. Refer to section 5.7 for further details.

There are approximately five fairway bridges spanning the creek within the Wynnum Golf Course, between Chandos Street and Stradbroke Avenue. These bridges are single span, with no handrails and would appear to have a very minor impact on flood flows. Rather than code each of the bridges into the model within a short length of creek (potentially causing model instabilities); the hydraulic roughness of the creek has been slightly increased from Chandos Street to Stradbroke Avenue to account for the minor impact of these structures.

<u>Weirs</u>

There are a number of minor weirs along the creek, of which the most significant would be the low-flow diversion weir between the Railway Bridge and Tingal Road. This structure is in the fully 2d section of the model and has been represented simplistically by increased hydraulic roughness, due to the existing complexity of the reach from Daisy Street to Tingal Road. Head-loss checks against the HEC-RAS model (in which the weir was represented as a 1d structure) indicate a very good correlation for all range of flows; refer to Section 5.7 for further details.

The weir at the Byrneside Terrace Stream Gauge was represented using the "z-line" approach, whereby the 2d bathymetry is raised to the level of the weir crest.

The small weir within the Wynnum Golf Course was represented by increased hydraulic roughness, similar to the minor fairway bridges.

5.3.5 Boundary Conditions

Inflows to the hydraulic model were taken from the XP-RAFTS hydrologic model. All inflows were represented as a discharge v time (Q-T) relationship, with the inflow locations as indicated in Figure 5.1. The inflow locations were generally adopted to match the XP-RAFTS model sub-catchment schematisation.

A varying water level versus time (H-T) downstream boundary was used to represent the downstream boundary conditions at the mouth of Wynnum Creek. As noted previously, this information was obtained from the Brisbane Bar tide gauge (operated by MSQ) and adjusted for the known difference between the locations

The 1d-2d linked model was joined to the fully 2d model at Daisy Street using an "SX" type flow boundary condition. Within the 1d-2d linked section of the model, the 1d channel was linked to the 2d domain using the "HX" type boundary condition

5.3.6 Run Parameters

Time Step

The 1d ESTRY component was run using a 0.5 second time step and 2d TUFLOW component using a 0.5 second time step.

Eddy Viscosity

The Smagorinsky method was used for specifying the eddy viscosity in the 2d domain. This method is recommended in the TUFLOW manual and the default approach, in lieu of the Constant method. This method uses the Smagorinsky formula with a "Constant Coefficient" of 0.1 and "Smagorinsky Coefficient" of 0.2.

5.4 Calibration Procedure

5.4.1 Tolerances

BCC flood studies aim to achieve the following tolerances with regard to the hydraulic model calibration / verification:

- Continuous recording stream gauges - within ± 0.15 m of the peak flood level.
- MHGs - within ± 0.30 m of the peak flood level.
- Debris marks - within ± 0.40 m of the peak flood level.
- Good timing of peaks and troughs.

5.4.2 Methodology

The methodology applied to the calibration and verification of the TUFLOW model was as follows:

1) Run a large slowing increasing flow through the TUFLOW model to enable hydraulic structure head-loss checks to be undertaken against the HEC-RAS model(s).
2) Iteratively adjust the bridge loss parameters (as required) and re-run the model to establish a reasonable correlation with the HEC-RAS model(s).
3) Using the flow inputs from the XP-RAFTS model, run the calibration events (i.e. March 2013 and December 2010) through the TUFLOW model and compare the simulated results against the observed flood levels at both the stream gauge and the MHGs.
4) Iteratively adjust the model parameters and re-run the model to achieve a good fit with the observed data. The predominant model parameters adjusted included Manning's 'n' and the hydraulic structure losses.
5) Adopt model parameters based on the calibration results.
6) Using the flow inputs from the XP-RAFTS model, run the verification events (i.e. May 2009 and December 1995) through the calibrated TUFLOW model and compare the simulated results against the observed flood levels at both the stream gauge and the MHGs.

As the creek conditions for all historical events are generally similar, the exact same model schematisation and parameters have been used for all four historical events. The only difference between the hydraulic modelling of the historical events is with the hydrologic flow inputs. This methodology ensures that the TUFLOW model is sufficiently robust to be utilised for the design and extreme event modelling.

5.5 Hydraulic Model Calibration Results

5.5.1 March 2013

Figure 5.2 provides a comparison between the TUFLOW results and the gauged flood level at the Byrneside Terrace Stream Gauge for the March 2013 event. The results indicate a reasonable fit to the complex multi-peaked event, with good timing of the peak / troughs as well as a reasonable fit to the peak flood level at the stream gauge.

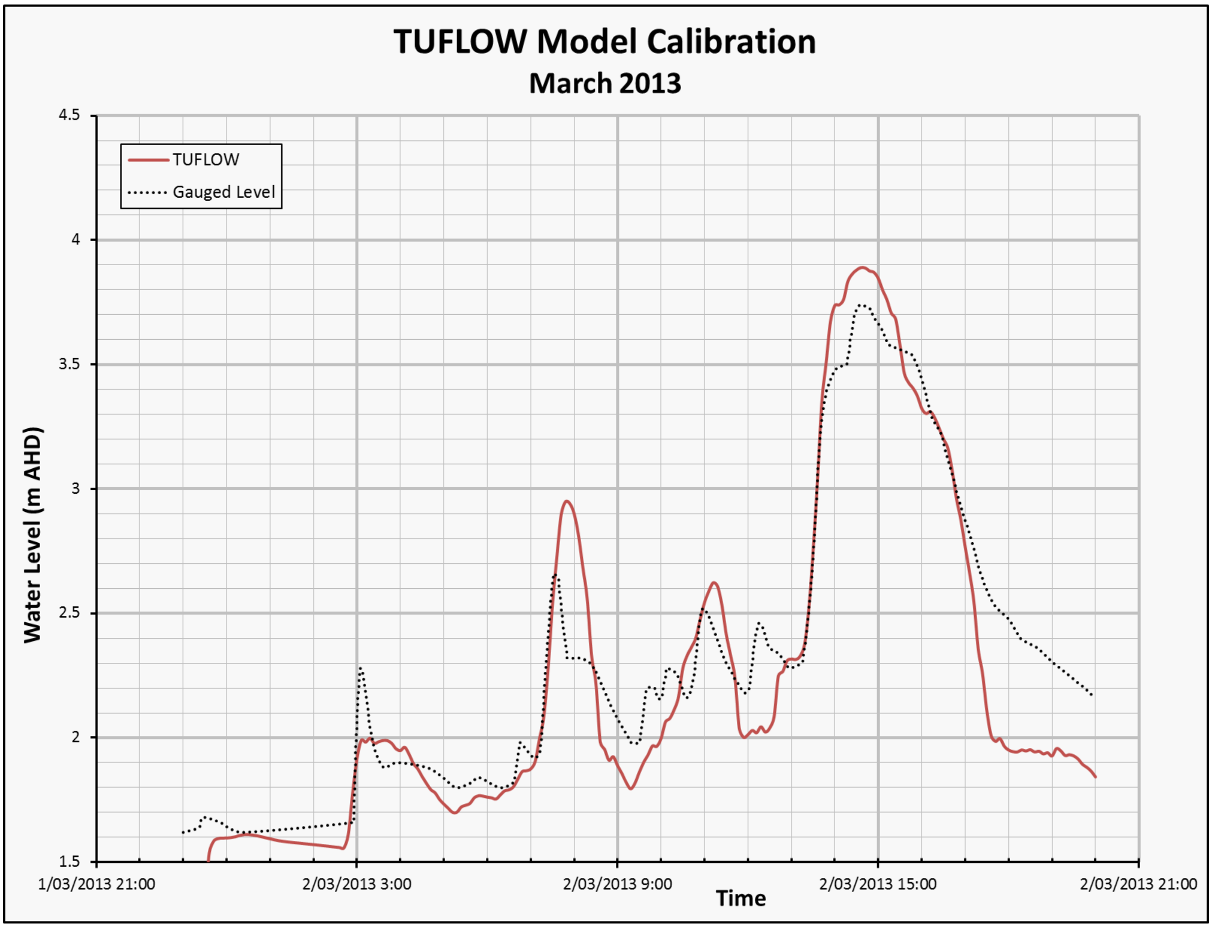


Figure 5.2: TUFLOW Model Calibration (March 2013)

Table 5.3 provides a comparison between the TUFLOW results and the recorded peak flood levels at the Stream Gauge and MHGs which were working during the event. The results indicate that the simulated peak flood levels were within the specified tolerance at all the MHGs. At the Byrneside Terrace Stream Gauge, the difference of 0.15 m was also within the ideal ± 0.15 m tolerance.

Table 5.3 – Calibration to Peak Flood Level Data (March 2013)

Reach	Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Wynnum Creek	W002	Coreen Street	-	2.82	-
	W_E580	Stream Gauge	3.74	3.89	0.15
	W100	d/s Tingal Road	4.72	4.84	0.08
	W110	u/s Daisy Street	-	5.85	-
	W004	u/s Stradbroke Av.	-	7.57	-
	W120	d/s Chandos Street	-	8.37	-
	W130	u/s Preston Road	10.38	10.42	0.04
	W140	d/s Wondall Road	11.54	11.35	-0.19

5.5.2 December 2010

Figure 5.3 provides a comparison between the TUFLOW results and the gauged flood level at the Byrneside Terrace Stream Gauge for the December 2010 event.

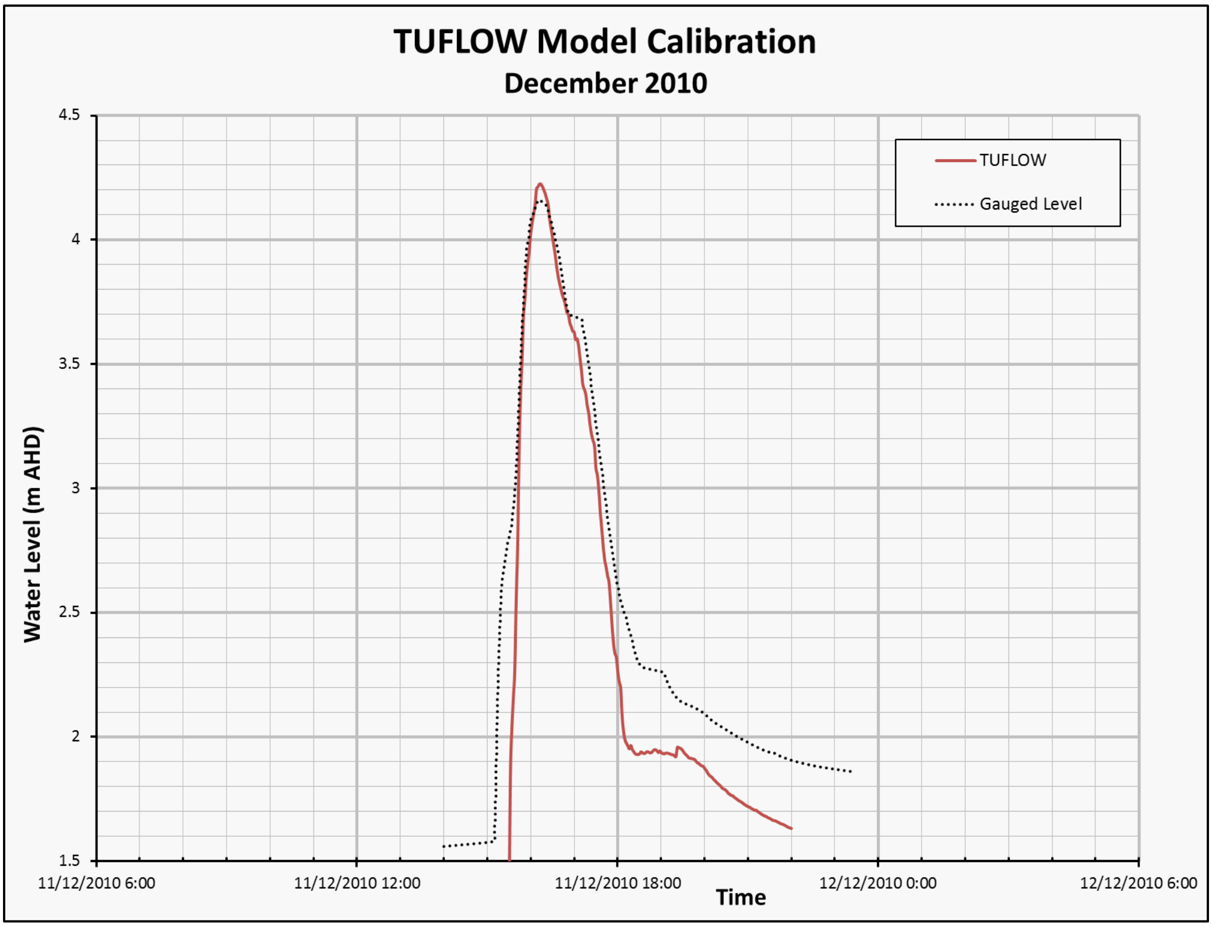


Figure 5.3: TUFLOW Model Calibration (December 2010)

The results indicate a good fit to the single peaked event, with a good replication of the rising and receding limbs as well as the magnitude and timing of the peak flood level.

Table 5.4 provides a comparison between the TUFLOW results and the recorded peak flood levels at the Stream Gauge and MHGs which were working during the event. The results indicate that the simulated peak flood levels were within the specified tolerance at all gauges.

Table 5.4 – Calibration to Peak Flood Level Data (December 2010)

Reach	Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Wynnum Creek	W002	Coreen Street	-	3.18	-
	W_E580	Stream Gauge	4.16	4.22	0.06
	W100	d/s Tingal Road	5.20	5.24	0.04
	W110	u/s Daisy Street	5.67	5.91	0.24
	W004	u/s Stradbroke Av.	-	7.28	-
	W120	d/s Chandos Street	-	8.12	-
	W130	u/s Preston Road	10.34	10.09	-0.25
	W140	d/s Wondall Road	11.39	11.13	-0.26

5.6 Hydraulic Model Verification Results

5.6.1 May 2009

Figure 5.4 provides a comparison between the TUFLOW results and the gauged flood level at the Byrneside Terrace Stream Gauge for the May 2009 event.

Table 5.5 provides a comparison of the TUFLOW results and the recorded peak flood levels at the Stream Gauge and MHGs which were working during the event.

At the Byrneside Terrace Stream Gauge, the results indicate a reasonable fit to the complex multi-peaked event, with good timing of the peak / troughs as well as a good fit to the peak flood level. The results indicate that the simulated peak flood levels were within the specified tolerance at all gauges.

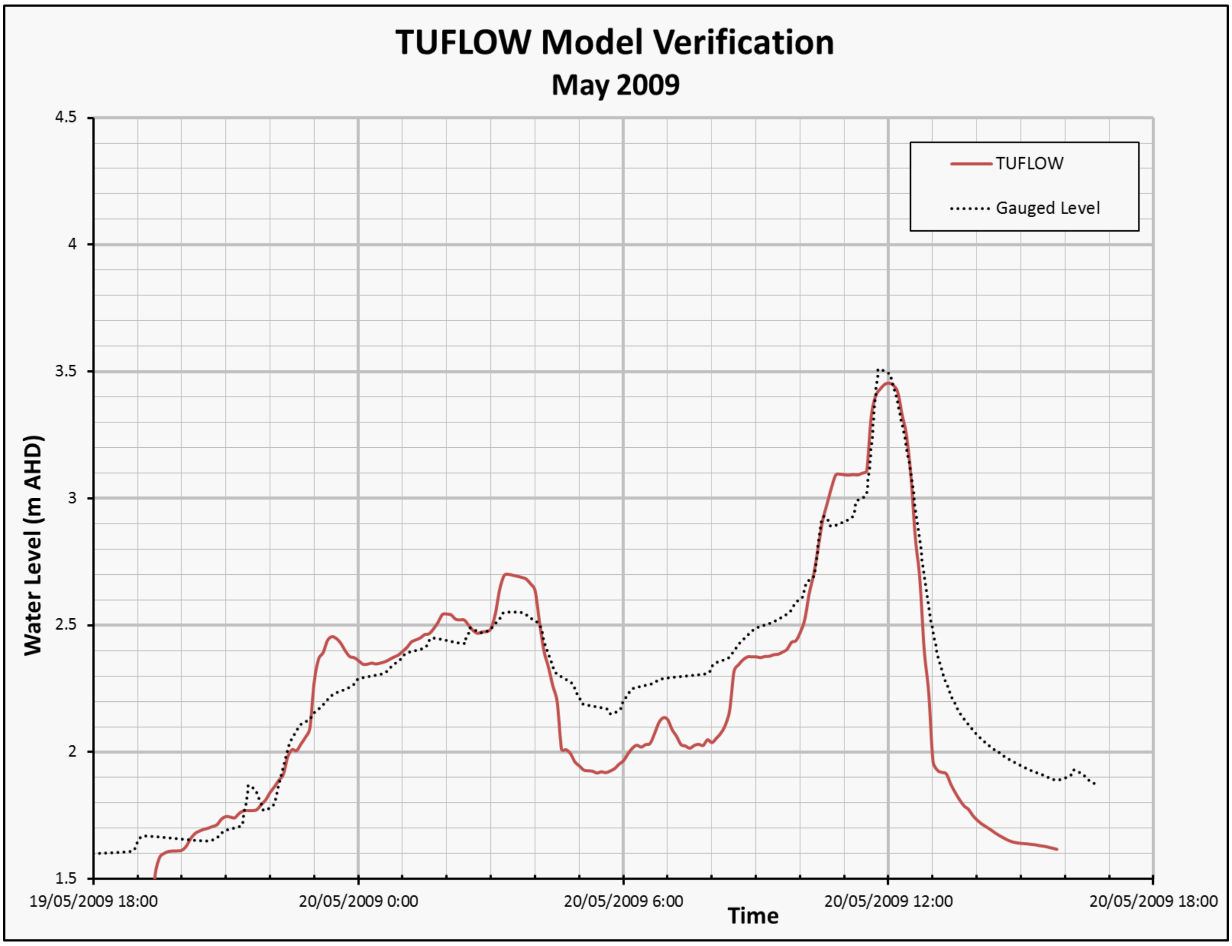


Figure 5.4: TUFLOW Model Verification (May 2009)

Table 5.5 – Verification to Peak Flood Level Data (May 2009)

Reach	Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Wynnum Creek	W002	Coreen Street	-	2.39	-
	W_E580	Stream Gauge	3.51	3.47	-0.04
	W100	d/s Tingal Road	4.69	4.42	-0.27
	W110	u/s Daisy Street	5.49	5.30	-0.19
	W004	u/s Stradbroke Av.	-	7.12	-
	W120	d/s Chandos Street	-	8.05	-
	W130	u/s Preston Road	-	9.87	-
	W140	d/s Wondall Road	11.00	11.02	0.02

5.6.2 December 1995

Figure 5.5 provides a comparison between the TUFLOW results and the gauged flood level at the Byrneside Terrace Stream Gauge for the December 1995 event. The results indicate a good fit to the single peaked event, with a good replication of the total stage hydrograph.

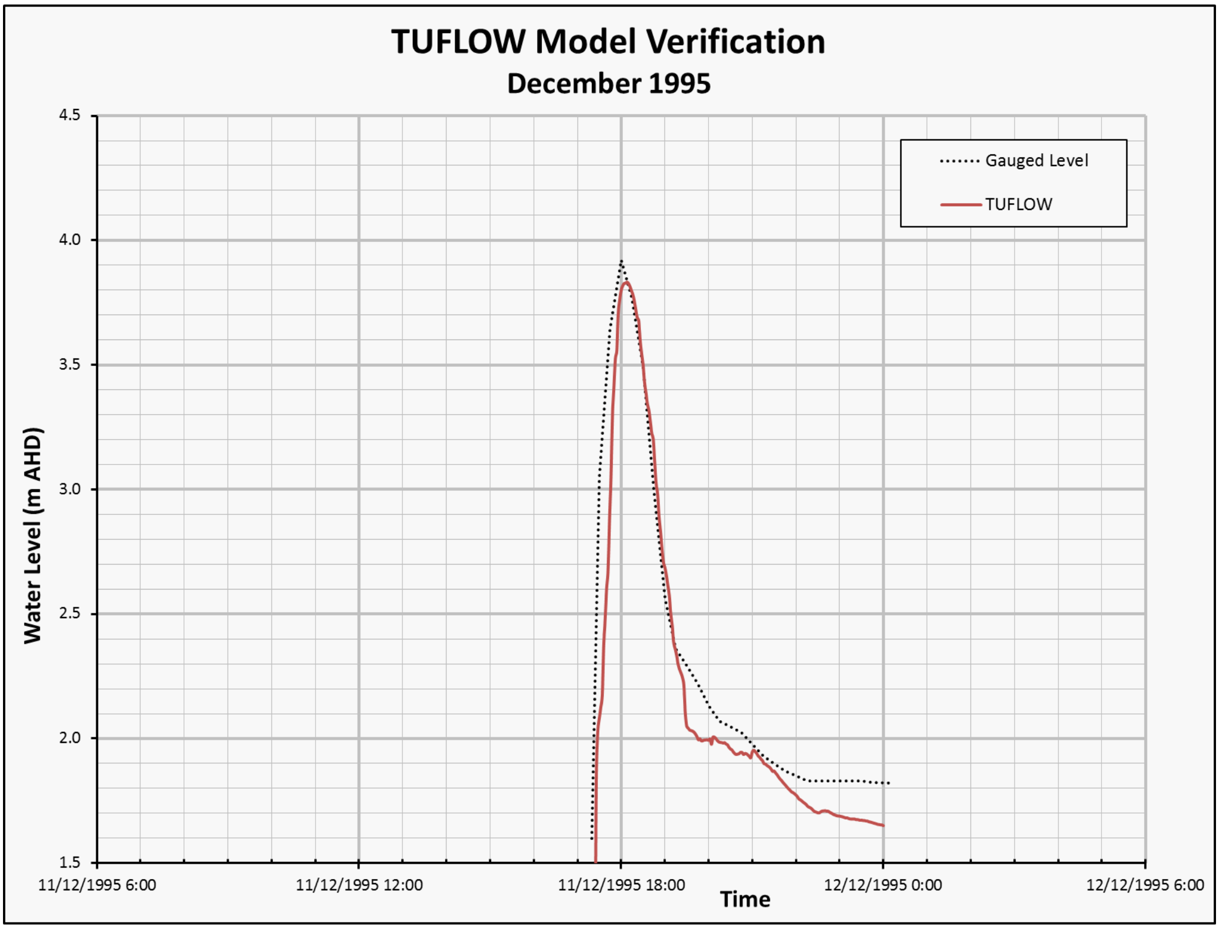


Figure 5.5: TUFLOW Model Verification (December 1995)

Table 5.6 provides a comparison between the TUFLOW results and the recorded peak flood levels at the Stream Gauge and MHGs which were working during the event. The results indicate that the simulated flood levels were within the specified tolerance at seven of the eight gauges.

The results at MHG W110 are considerably higher than the recorded level, which is expected as there are a number of significant changes to the channel between Daisy Street and Tingal Road since 1995, which include:

- The low flow diversion weir for water quality purposes
- The new Cleveland Rail Bridge and associated earthworks

The TUFLOW verification model has both of these features included, which would result in greater head-loss during smaller flood flows than the actual conditions in 1995.

Table 5.6 – Verification to Peak Flood Level Data (December 1995)

Reach	Gauge ID	Location	Recorded Peak WL (m AHD)	Simulated Peak WL (m AHD)	Difference (m)
Wynnum Creek	W002	Coreen Street	2.73	2.67	-0.06
	W_E580	Stream Gauge	3.92	3.82	-0.10
	W100	d/s Tingal Road	4.71	4.69	-0.02
	W110	u/s Daisy Street	5.21	5.80	0.59*
	W004	u/s Stradbroke Av.	7.61	7.47	-0.14
	W120	d/s Chandos Street	8.46	8.30	-0.16
	W130	u/s Preston Road	10.33	10.36	0.03
	W140	d/s Wondall Road	11.47	11.29	-0.18

*Please refer to earlier comments

5.7 Hydraulic Structure Verification

The TUFLOW manual recommends confirming the head-loss across hydraulic structures as follows:

It is strongly recommended that the losses through a structure be validated through:

- *Calibration to recorded information (if available).*
- *Cross-checked using desktop calculations based on theory and/or standard publications (e.g. Hydraulics of Bridge Waterways, US FHA 1973).*
- *Cross-checked with results using other hydraulic software.*

It is common practice in BCC flood studies to cross-check structure head-losses against results from the HEC-RAS hydraulic modelling software. Generally, HEC-RAS is regarded as one of the better hydraulic modelling packages when it comes to accurately representing hydraulic structures such as bridges.

The majority of the hydraulic structures within the Wynnum Creek Catchment are culverts, of which the TUFLOW and HEC-RAS algorithms are similar. Therefore, it was considered more important to check the head-loss at a number of the bridge structures.

The bridge structures where checks were undertaken included:

- Talwong Street Footbridge
- Preston Road Bridge
- Daisy Street – Railway – Tingal Road Crossing (1 culvert, 1 weir and 2 bridges)
- Coreen Street Footbridge
- Fox Street Bridge

Table 5.7 provides a comparison of the head-loss across the structure between TUFLOW and the HEC-RAS model. Generally, the TUFLOW head-losses for the hydraulic structures (which were checked) were within ± 0.3 m of the HEC-RAS values for the full range of flows at which checks were undertaken. This is considered reasonable and gives credence to the TUFLOW results.

Table 5.7 – Bridge Modelling Checks

Flow (m^3/s)	TUFLOW Head-loss (m)	HEC-RAS Head-loss (m)	Difference (m)
Structure S4 – Talwong Street Footbridge			
5	0.01	0.01	0.00
15	0.01	0.01	0.00
30	0.02	0.06	-0.04
60	0.02	0.11	-0.09
90	0.07	0.10	-0.03
120	0.09	0.11	-0.02
240	0.20	0.17	0.03
Structure S9 – Preston Road			
5	0.14	0.19	-0.05
15	0.21	0.43	-0.22
30	0.42	0.58	-0.16
60	1.12	1.06	0.06
90	1.10	1.04	0.06
120	1.05	0.92	0.13
220	0.97	N/A	N/A
Structure(s) S13, S14, S15 Daisy Street + Railway + Tingal Road			
5	1.10	1.37	-0.27
15	1.11	1.20	-0.09
30	0.96	0.84	0.12
60	0.80	0.85	-0.05
90	0.57	0.62	-0.05
120	0.38	0.25	0.13
220	0.34	0.36	-0.02
Structure S16 – Coreen Street			
5	0.01	0.01	0.00
15	0.03	0.02	0.01
30	0.03	0.03	0.00
60	0.04	0.03	0.01
90	0.05	0.05	0.00
120	0.07	0.04	0.03
220	0.06	0.05	0.01
Structure 17 – Fox Street			
5	0.00	0.00	0.00
15	0.01	0.02	-0.01
30	0.03	0.03	0.00
60	0.06	0.07	-0.01
90	0.05	0.12	-0.07
120	0.07	0.17	-0.10

5.8 Hydrologic-Hydraulic Model Consistency Check (Historical Events)

5.8.1 General

Checks were undertaken between the flows derived by the XP-RAFTS and the TUFLOW models in the middle and lower sections of the catchment, to understand how closely the hydrologic and hydraulic models were matching.

Figures 5.6 to 5.9 provide comparative plots of the XP-RAFTs and TUFLOW flow results for the calibration and verification events at the following two locations:

(i) Middle to Upper Catchment - immediately downstream of Wondall Road; and

(ii) Lower Catchment – at the Coreen Street Footbridge.

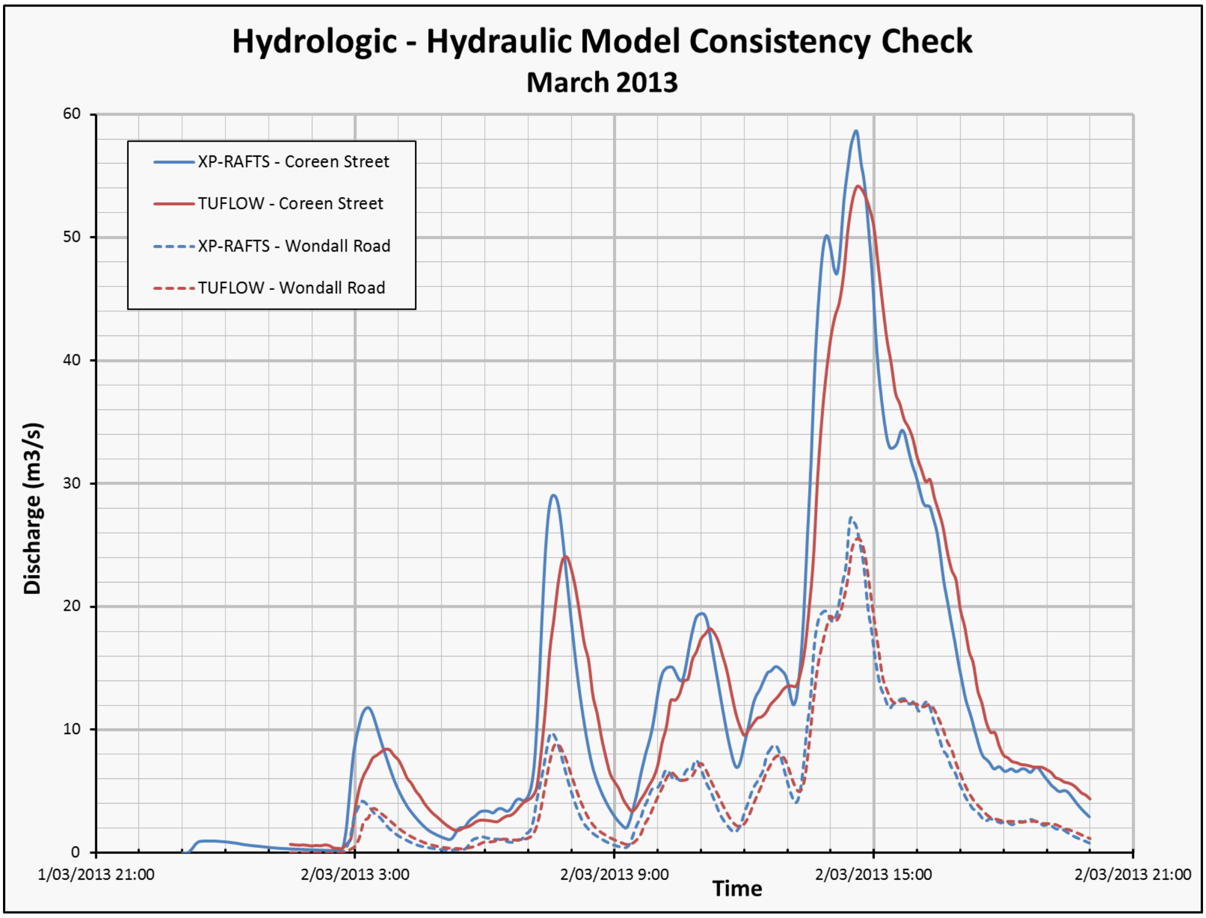


Figure 5.6: Model Consistency Check (March 2013)

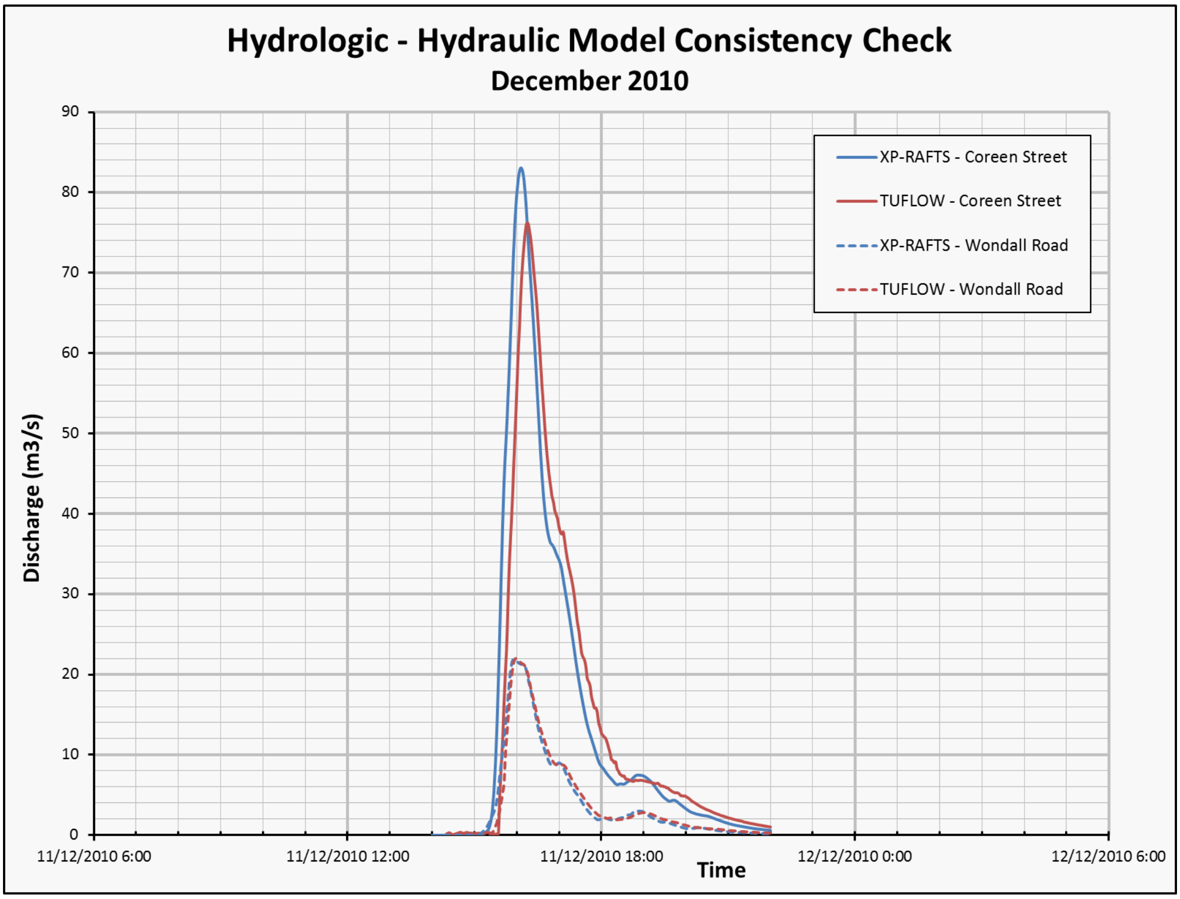


Figure 5.7: Model Consistency Check (December 2010)

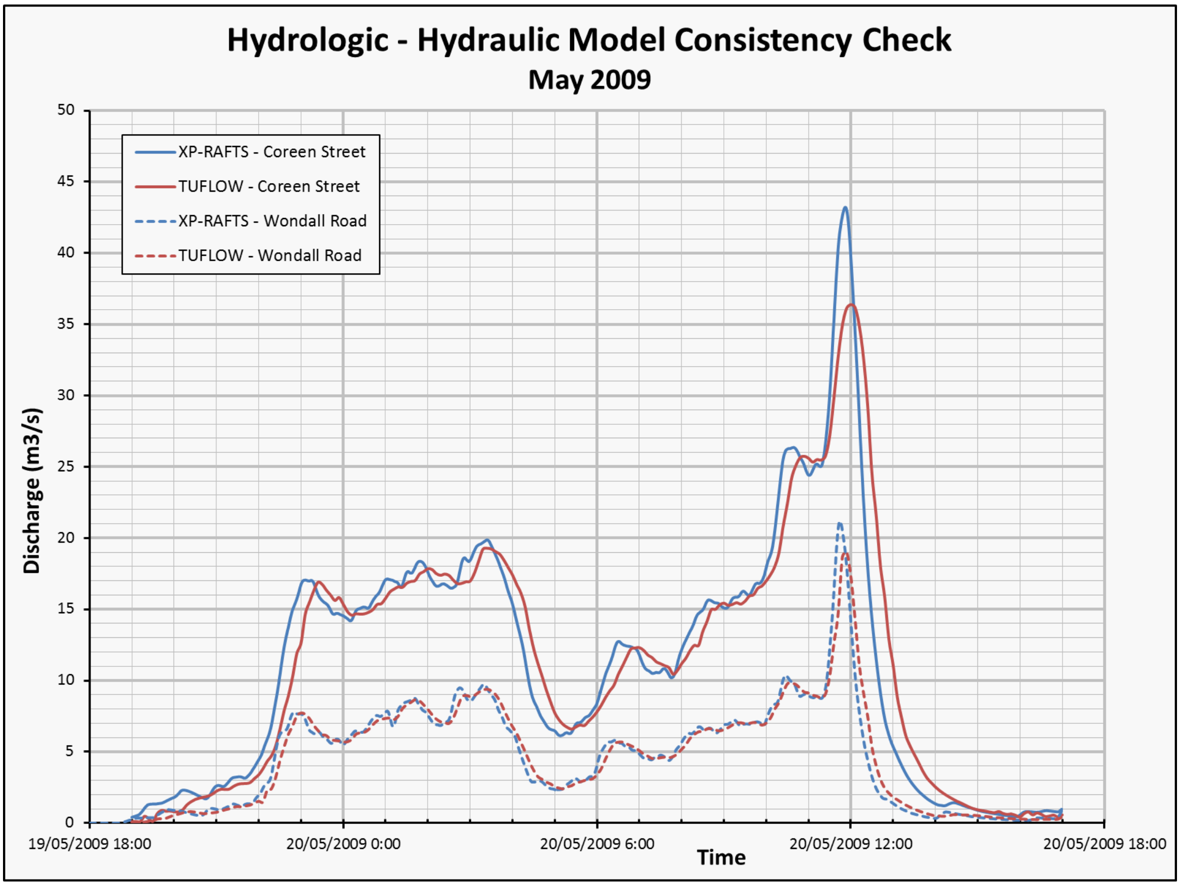


Figure 5.8: Model Consistency Check (May 2009)

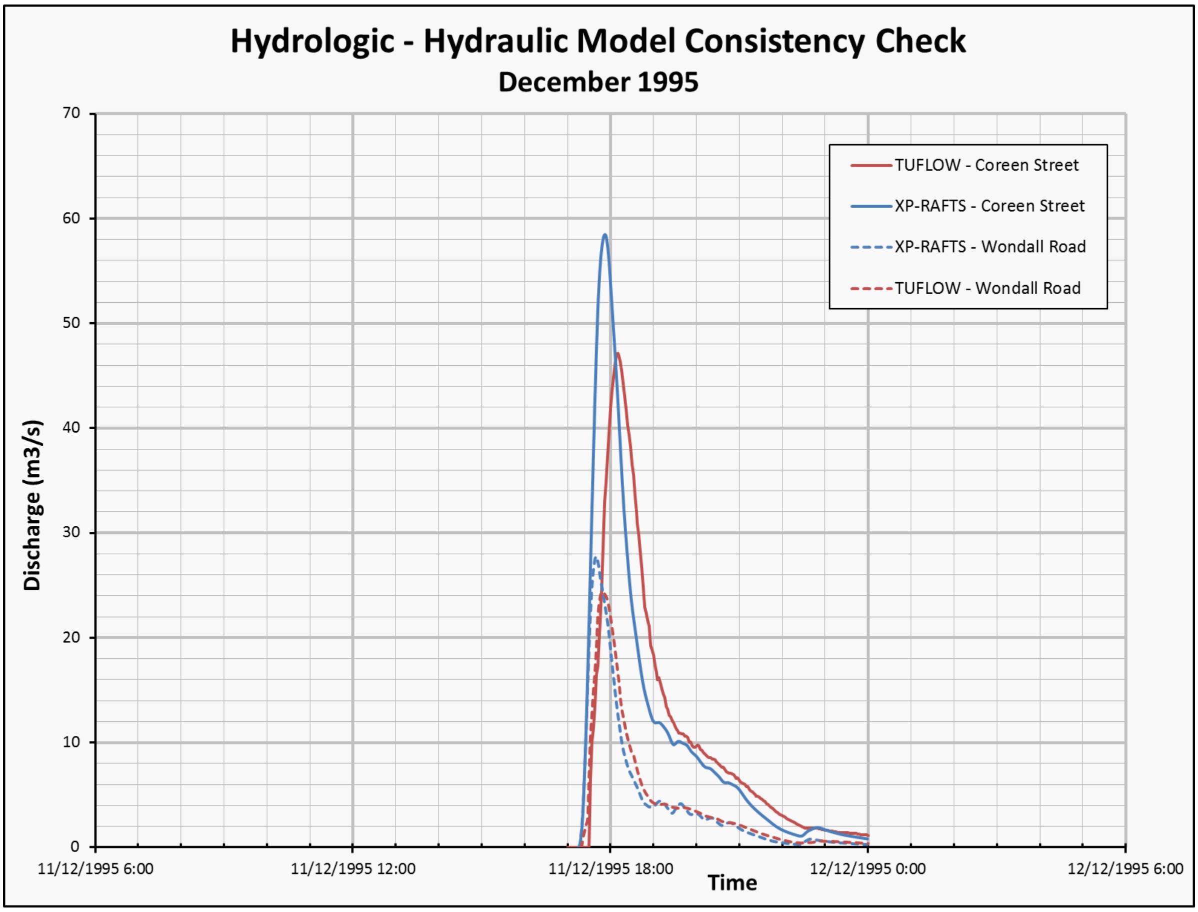


Figure 5.9: Model Consistency Check (December 1995)

The comparative plots for the four historical events indicate a reasonable correlation between the XP-RAFTS and TUFLOW models.

At Wondall Road, the correlation between the two models is very good for all four events, with the peak flow and timing matching very well.

At the Coreen Street Footbridge, the comparative plots indicate that the XP-RAFTS model is over predicting the TUFLOW model for all historical events. The timing and shape of the hydrographs are quite similar; however the peak flow is higher in the XP-RAFTS model. This trend is consistent with the results of the hydrologic calibration, in which the XP-RAFTS peak flows were consistently higher than the rated flood flows at the Byrneside Terrace Stream Gauge. This would tend to suggest that the simplistic XP-RAFTS hydrologic reach / channel routing is unable to accurately represent the channel routing at the downstream of the model, unless significantly more storage is introduced into the model.

As part of the 1997 Flood Study, a significant amount of additional storage was introduced at Kitchener Park into the XP-RAFTS model, to allow it to replicate the hydraulic and historical results.
The stage-storage-discharge curve that was used in the 1997 XP-RAFTS model was reviewed and was considered not appropriate to adopt, as there were some issues identified. For this current study, the initial methodology was to apply this same methodology and introduce additional storage into the hydrologic model. However, during the course of this study this methodology was changed and it was decided that the most critical issue was to ensure that the hydraulic model results were accurate and

that the consistency between hydrologic and hydraulic results was not essential. This change of methodology has resulted in the removal of the requirement to ensure the hydrologic model results matched the hydraulic model results.

5.9 Discussion

The results of the hydraulic calibration and verification indicated that the TUFLOW model was able to accurately simulate the historical flooding events to within the tolerances imposed on the study. All four historical events were able to be accurately simulated by the TUFLOW model. On this basis, it was concluded that the XP-RAFTS and TUFLOW models were sufficiently robust to use together to accurately simulate design flood events.

When used as a standalone model (in lieu of together with the TUFLOW model), the XP-RAFTS model consistently over predicted flows in the lower section of the creek. As the calibration and verification events were only small events, it would be expected that in the larger design events the XP-RAFTS model would significantly over predict the flow in the downstream section of the creek. Therefore, it is <u>not</u> recommended that the XP-RAFTS model be used as a standalone model to simulate design floods. Rather, the XP-RAFTS model should be used in conjunction with the TUFLOW model to produce accurate flood flows and levels.

As the calibration and verification events were small events, it was not possible to determine how accurately the TUFLOW model was able to simulate large flooding events. Therefore, it is recommended that the TUFLOW model be verified against any large flooding events that occur subsequent to this flood study.

6.0 Design Event Analysis

6.1 Design Event Terminology

The use of the terms "*recurrence interval*" and "*return period*" has been criticised as leading to confusion in the minds of some decision-makers and members of the public. Therefore, the current update of AR&R will utilise different terminology.

Generally, for the larger flood magnitude discharges, the term AEP (%) is now preferred by AR&R, in lieu of ARI.

Table 6.1 indicates the equivalent AEP value (rounded to a whole number) with respect to ARI. The relationship can be expressed by the following equation:

AEP = 1 – exp (-1 / ARI)

Table 6.1 – ARI versus AEP

ARI (year)	AEP (%)
2	39
5	18
10	10
20	5
50	2
100	1

It is common to see the 50 % AEP being equated to the 2-yr ARI and also the 20 % AEP being equated to the 5-yr ARI. This is not technically correct; however the use of *AEP = 1 / ARI* is very prevalent within the industry and often used for simplicity.

For the purpose of this technical report, the correct values indicated in Table 6.1 will be utilised. The flood probability will be firstly expressed firstly in ARI and then secondly in the equivalent AEP, for example 2-yr ARI (39 % AEP).

However, as the mapping products in Appendix J will likely be viewed by a wider audience, for ease of common understanding the simplified *AEP = 1 / ARI* will be utilised. The 2-yr ARI and 5-yr ARI will be referred to as 50% AEP and 20% AEP respectively.

6.2 Design Event Scenarios

Table 6.2 indicates the three scenarios utilised in the modelling of the design events, noting that all design event scenarios were modelled using ultimate hydrological conditions.

For the purpose of this report, the term “design events” refers to those events from 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP).

Table 6.2 – Design Event Scenarios

ARI (year)	AEP (%)	Scenario 1	Scenario 2	Scenario 3
2	39	✓	×	✓
5	18	✓	×	✓
10	10	✓	×	✓
20	5	✓	×	✓
50	2	✓	×	✓
100	1	✓	✓	✓

The following describes the design event scenarios:

Scenario 1: Existing Waterway Conditions

Scenario 1 is based on the current waterway conditions. Some minor modifications were made to the TUFLOW model developed as part of the calibration / verification; refer to Section 6.4 for further details.

Scenario 2: Minimum Riparian Corridor (MRC)

Scenario 2 includes an allowance for a riparian corridor along the edge of the channel. This involved firstly reviewing the existing vegetation and land-use adjacent to the channel to determine an appropriate Manning's 'n' roughness value for the riparian corridor. In most locations the default value of n = 0.15 was used, however where the existing manning's 'n' is higher than n = 0.15, the manning's 'n' was left unchanged.

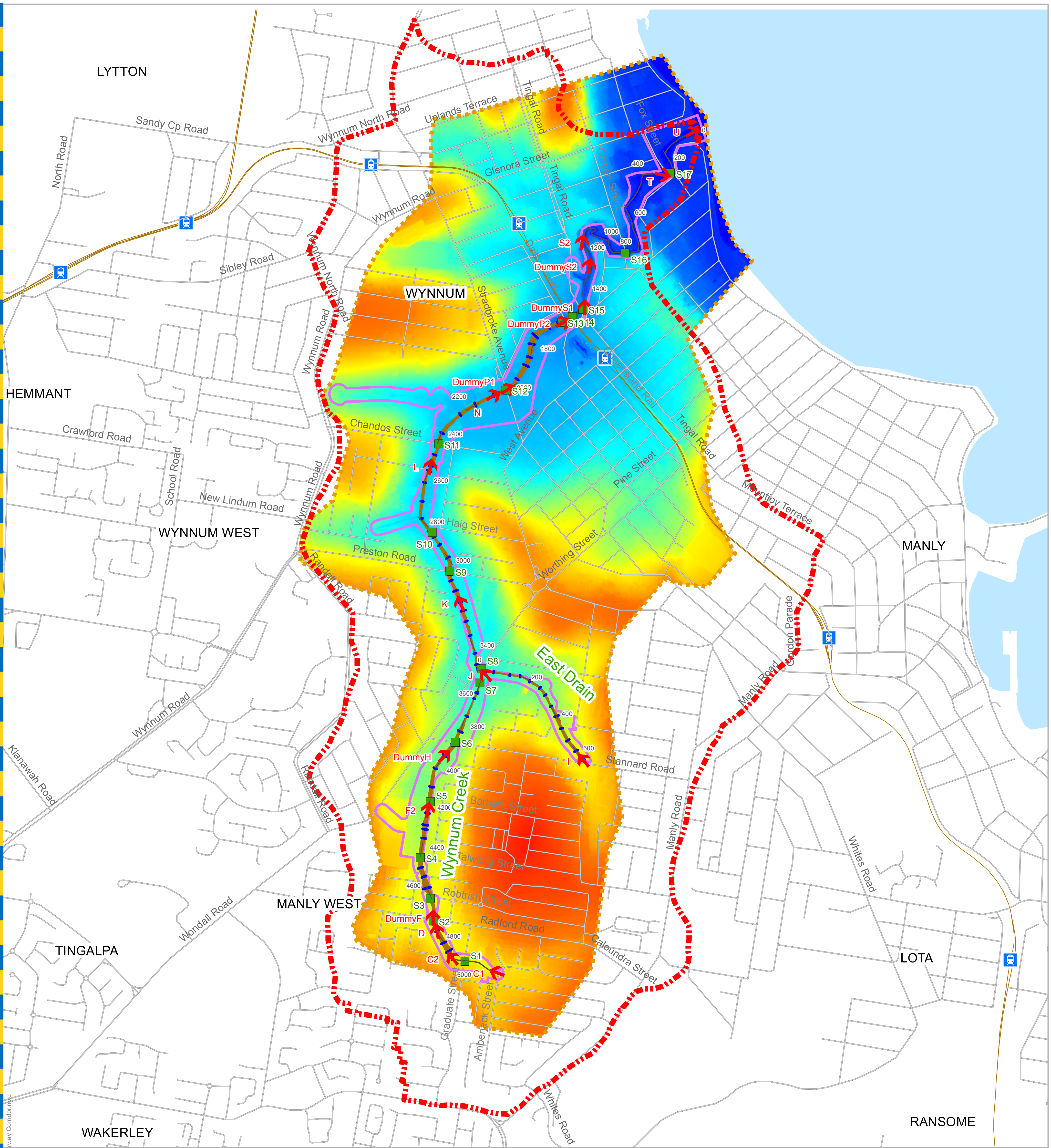
A 30 m wide corridor (15m wide each side from the low flow channel) was defined by changing the Manning's n of the 1d cross sections and a new materials layer within the TUFLOW model. In areas where the 15 m width was not available, the MRC was set to the maximum possible width (i.e. less than 15 m) up to the boundary of the waterway corridor.

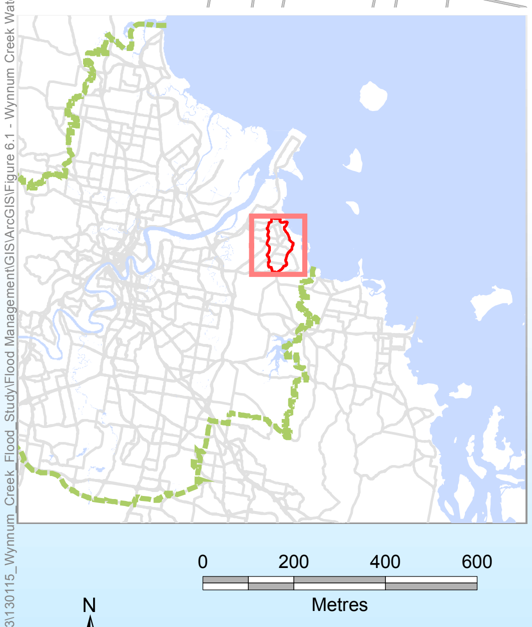
Scenario 3: Filling to the Waterway Corridor (WC) + Minimum Riparian Corridor (MRC)

Figure 6.1 indicates the current WC for the Wynnum Creek Catchment. Scenario 3 assumes filling to the WC boundary to simulate potential development outside the WC. In the design events, 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP), the filling acts as a barrier and the WC can be modelled simplistically as a glass-wall of infinite height. For modelling purposes, the WC lines near Graduate Street have been linked.

For the modelling of events greater than 100-yr ARI (1 % AEP), the fill height outside of the WC is set to the Scenario 3 100-yr ARI (1 % AEP) flood level plus an additional height allowance of 0.3 m.

Figure 6.2 illustrates the assumed filling outside of the WC. This is a simple and conservative assumption used to develop design planning levels. It does not necessarily reflect allowable development assumptions under City Plan.





Legend

- Wynnum Creek Waterway Corridor
- Wynnum Catchment
- Model Boundary
- Inflow Locations
- Structure
- 1d Cross Section
- 1d Channel Boundary
- Wynnum Creek Centreline
- Streets

Brisbane City Council
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Figure 6.1: Waterway Corridor Tuflow Model Layout

Prepared : 077900
Checked : SG
Revision : 0
Publication Date : 16 Jul 2014
Project Number : 130115

File : G:\BI\CD\Proj13\130115_Wynnum Creek Flood Study\Flood Management\GIS\ArcGIS\Figure 6.1 - Wynnum Creek Waterway Corridor.mxd

FLM - 130115

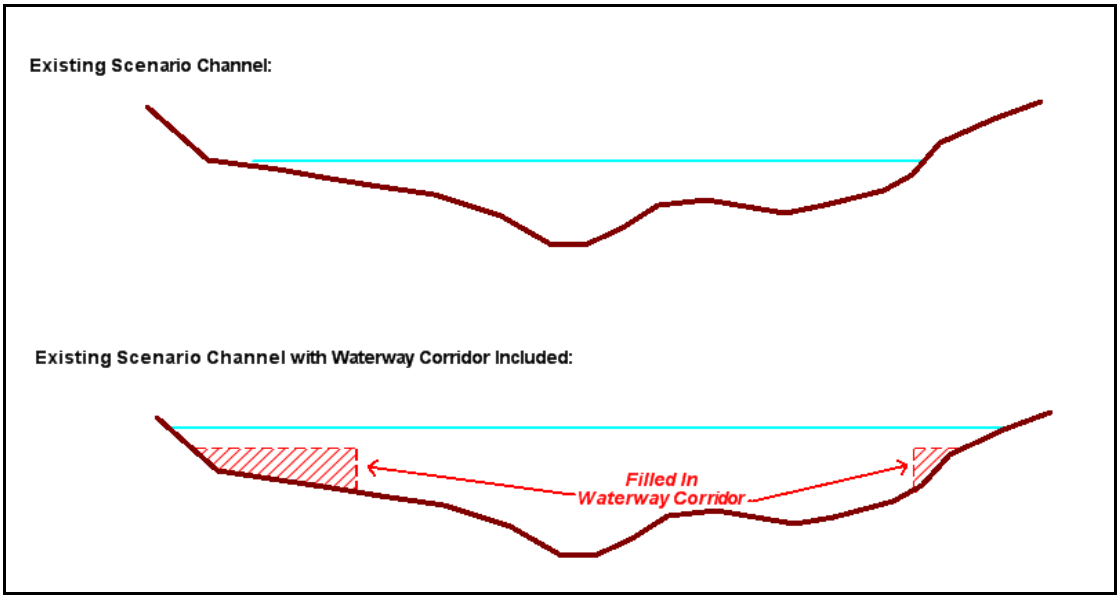

Figure 6.2: Filling Outside the Waterway Corridor

6.3 Design Event Hydrology

6.3.1 Overview

Design flood estimation is typically best determined by undertaking a flood frequency analysis of annual maximum and / or peak over threshold series from observed long-term stream flow records. However, in the Brisbane City Council region, the period of record is typically insufficient to enable sufficient confidence to warrant undertaking flood frequency methods. Table 6.3 [5] indicates some guidance for length of record versus expected error rate for flood frequency analysis.

On the basis that the one continuous recording stream gauge on Wynnum Creek (W_E580) has only approximately 36 years of records it has been deemed unsuitable to undertake flood frequency analysis for this study.

Table 6.3 – Guidance for Length of Record versus Expected Error Rate

ARI (year)	Required Length of Record (years)	
	± 10% Error Level	± 25% Error Level
10	90	18
25	105	31
50	110	39
100	115	48

[5] University Corporation for Atmospheric Research 2010, *Flood Frequency Analysis*, UCAR, USA

This study utilises the synthetic design storm concept from AR&R (1987) to estimate the design ARI flood in Wynnum Creek. This methodology is as follows:

- Design Intensity Frequency Duration (IFD) estimates are determined from AR&R for the full range of storm ARIs (2-yr to 100-yr) and durations (1 hour to 3 hours).
- Design temporal patterns are determined and design hyetographs produced for the full range of ARIs and durations.
- Appropriate design rainfall loss parameters are adopted by reference to the calibration and industry standard techniques.
- Using the calibrated models, design storms are simulated and the peak discharges and critical durations established within the model domain.

6.3.2 XP-RAFTS Model Set-up

The calibrated XP-RAFTS model was used to simulate the design storm rainfall-runoff and sub-catchment routing process. The following describes the adjustments made to the model in order to simulate the design events.

Catchment Development

The design events were modelled using ultimate catchment hydrological conditions. These conditions assume that the state of development within the catchment is at its ultimate condition, with reference to the current adopted planning scheme. Depending on the developed state of the catchment, an increase in development will generally affect the percentage impervious and the PERN hydrologic roughness values.

Appendix B presents the XP-RAFTS catchment parameters that were adopted for the design event modelling scenarios. The current adopted version of BCC City Plan (2000) was used to establish the ultimate catchment hydrological conditions. The adopted land-use for the ultimate catchment development is shown on a catchment map in Appendix C.

Rainfall Losses

The Initial Loss (IL) and Continuing Loss (CL) approach was used to simulate the rainfall losses in order to determine the rainfall excess.

An IL of 0 mm was adopted for the design events modelling. This value is typically used in BCC flooding studies and is considered a conservative approach.

A CL of 0 mm/hr was adopted for the design events modelling. This value was determined from the results of the calibration and verification process. As noted previously, a CL of 0 mm/hr has been used for a number of recently completed flood studies such as Norman Creek, Cabbage Tree Creek and Oxley Creek.

Design hyetographs

Design hyetographs were derived from the techniques in AR&R (1987). Hyetographs were created for the 2-yr ARI (39 % AEP), 5-yr ARI (18 % AEP), 10-yr ARI (10 % AEP), 20-yr ARI (5 % AEP), 50-yr ARI (2 % AEP) and 100-yr ARI (1 % AEP) events, considering durations of 1 hour, 1.5 hour, 2 hour and 3 hours.

6.4 Design Event Hydraulic Modelling

6.4.1 Overview

The TUFLOW model was used to determine design flows and flood levels for those scenarios as detailed in Table 6.2 for the 2-yr ARI (39 % AEP) to the 100-yr ARI (1 % AEP) events. These events were simulated for durations from 30 minutes to 3 hours.

6.4.2 TUFLOW model roughness

The hydraulic roughness in the calibrated TUFLOW model was updated as required to represent the ultimate catchment conditions.

6.4.3 TUFLOW model boundaries

The design inflow (Q-T) boundaries to the TUFLOW model were taken from the XP-RAFTS model for each ARI and duration. The inflow locations did not change from the calibrated TUFLOW model.

The TUFLOW model utilised a fixed water level (H-T) boundary at its downstream extent (i.e. Moreton Bay). A Mean High Water Springs (MHWS) value of 0.95 m AHD was adopted for all design events.

It should be noted that the joint probability of fluvial and tidal events has not been considered in the modelling.

6.5 Results and Mapping

6.5.1 Peak Discharge Results

The provision of tabulated peak flow information throughout the creek extents is not a requirement for this flood study. However, it is good practice to provide peak flows at the major hydraulic structures within the creek extents. The following Table 6.4 provides peak flows at the major hydraulic structures for the Scenario 1 conditions.

In the vicinity of a number of crossings, the flood extents are quite wide for some or all of the flood events. At these locations, it is difficult to determine an appropriate single discharge value representative at the structure. These locations include:

- Wondall Road
- Stradbroke Avenue
- Daisy Street, and
- Fox Street

Table 6.4 – Design Event Peak Discharge at Major Structures (Scenario 1)

Structure Location	Peak Discharge (m^3/s)					
	2-yr ARI (39 % AEP)	5-yr ARI (18 % AEP)	10-yr ARI (10% AEP)	20-yr ARI (5 % AEP)	50-yr ARI (2 % AEP)	100-yr ARI (1 % AEP)
Graduate St.	12.2	14.9	15.9	17.7	21.0	24.3
Radford Rd.	18.5	22.8	24.7	27.0	28.8	29.8
Stannard Rd.	30.5	35.1	37.3	43.1	49.2	55.0
Wondall Rd.	44.2	55.7	60.8	68.9	79.9	89.7
Preston Rd.	46.7	55.8	62.2	74.7	85.1	95.5
Chandos St.	48.9	58.4	62.9	70.4	82.4	92.8
Stradbroke Av.	52.2	66.0	73.0	80.2	91.8	104.2
Daisy St.	49.8	60.8	66.8	74.0	85.7	95.6
Railway	59.6	71.8	80.5	89.5	103.1	113.4
Tingal Rd.	59.7	72.2	80.4	89.7	103.2	113.5
Fox Street	77.0	91.0	100.2	112.8	127.3	140.6

6.5.2 Critical Durations

A full range of event durations (30 minutes, 1 hour, 1.5 hour, 2 hour and 3 hour) were simulated from which the critical duration for the 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) events at key locations within the catchment is provided in Table 6.5.

6.5.3 Peak Flood Levels

Tabulated peak flood level results for Scenario 3 are provided in Appendix D for Wynnum Creek and the East Drain. These results are presented for the 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) events.

Tabulated peak flood level results for Scenarios 1 and 2 are provided in the Model Handover Guide. These results are presented for the 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP) for Scenario 1 and the 100-yr ARI (1 % AEP) event for Scenario 2.

The peak flood levels were extracted at regular intervals using the peak envelope of flood levels, which considered all durations. The peak flood levels were extracted along the AMTD line.

Table 6.5 – Critical Durations at Major Structures

Structure Location	Critical Duration (minutes)					
	2-yr ARI (39 % AEP)	5-yr ARI (18 % AEP)	10-yr ARI (10% AEP)	20-yr ARI (5 % AEP)	50-yr ARI (2 % AEP)	100-yr ARI (1 % AEP)
Graduate St.	60	60	60	60	60	60
Radford Rd.	60	60	60	60	60	60
Robtrish St.	60	60	60	60	60	60
Talwong St.	60	60	60	60	60	60
Barbara St.	60	60	60	60	60	60
Stannard Rd.	60	60	60	60	60	60
Leagues Club	60	60	60	60	60	60
Wondall Rd.	60	60	60	60	60	60
Preston Rd.	60	60	60	60	60	60
Adj. Haig St.	60	60	60	60	60	60
Chandos St.	60	60	90	60	60	60
Stradbroke Av.	60	90	90	90	60	60
Daisy St.	90	90	120	120	90	90
Railway	60	90	120	120	90	90
Tingal Rd.	60	90	120	120	90	90
Coreen St.	60	60	90	90	60	90
Fox Street	90	90	90	90	90	90

6.5.4 Return Periods of Historic Events

In order to estimate the return period of the historical events modelled, a flood frequency curve was developed at both MHG W100 (downstream of Tingal Road) and MHG W140 (downstream of Wondall Road). These flood frequency curves were based on the Scenario 1 modelling and are indicated in Figures 6.3 and 6.4.

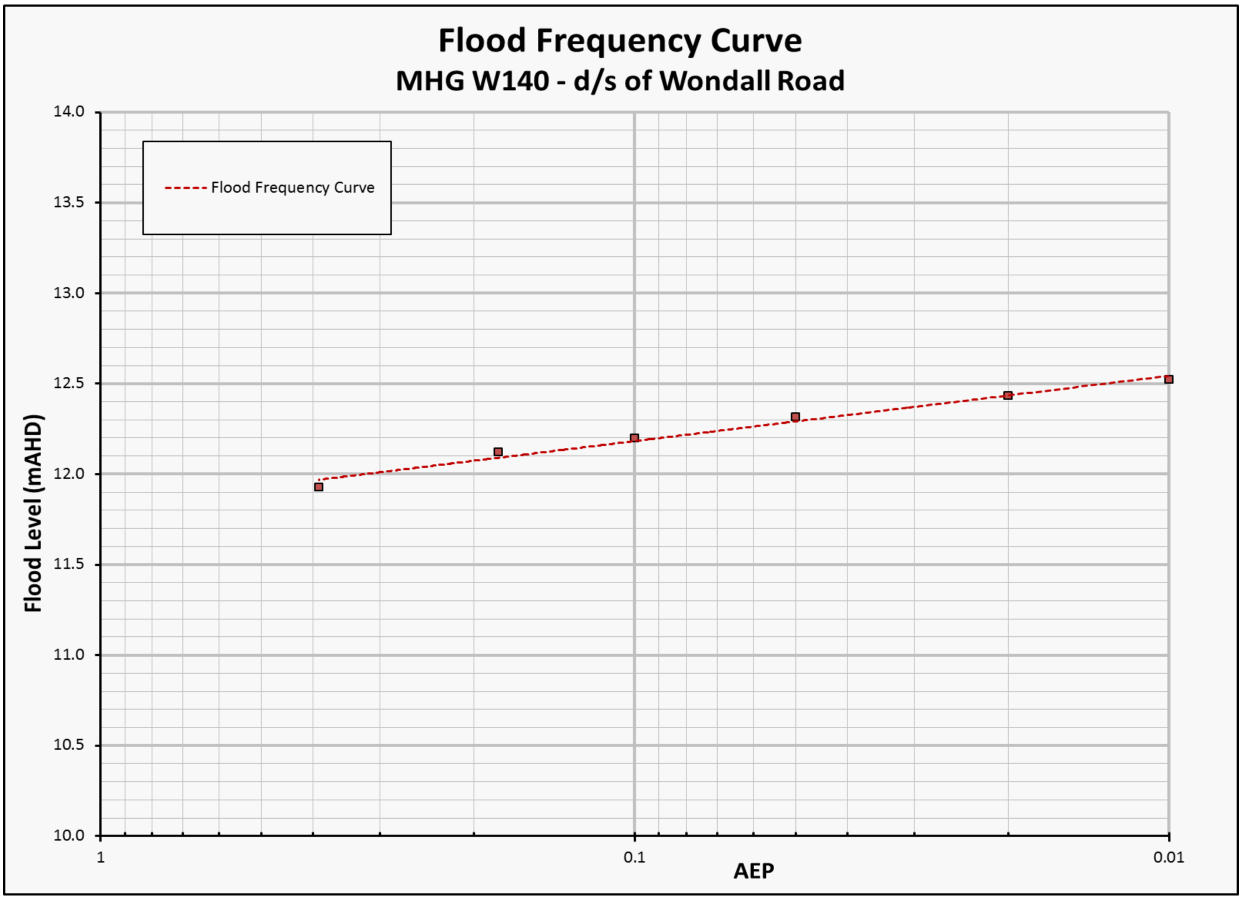

Figure 6.3: Flood Frequency Curve – Downstream of Tingal Road

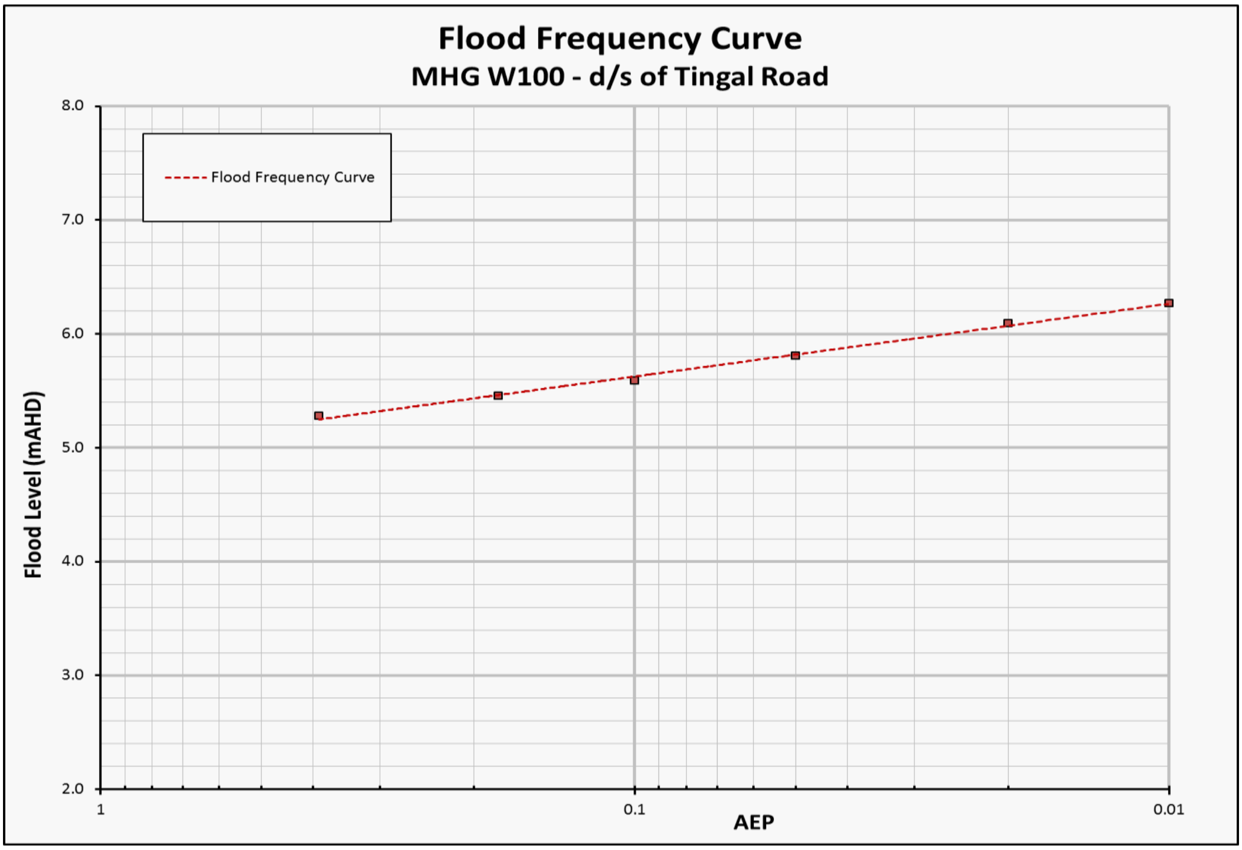

Figure 6.4: Flood Frequency Curve – Downstream of Wondall Road

Table 6.6 indicates the estimated return period of the historical events; based on the flood frequency curves, with the inclusion of estimates for the 1967 and 1974 events.

Table 6.6 – Estimate Return Period of Historical Events

Historical Event	Return Period (ARI)	
	MHG W140	MHG W100
1967	~100-yr ARI (1 % AEP)	~100-yr ARI (1 % AEP)
1974	~2-yr ARI (39 % AEP)	~10-yr ARI (10 % AEP)
1995	< 1-yr ARI (100 % AEP)	< 1-yr ARI (100 % AEP)
2009	< 1-yr ARI (100 % AEP)	< 1-yr ARI (100 % AEP)
2010	< 1-yr ARI (100 % AEP)	~2-yr ARI (39 % AEP)
2013	< 1-yr ARI (100 % AEP)	< 1-yr ARI (100 % AEP)

6.5.5 Rating Curves

Rating curves (H-Q) have been derived at a number of locations along the creek and have been provided as Figures 6.5 to 6.10. These locations are generally in the vicinity of hydraulic structures and include:

- Radford Road
- Wondall Road
- Preston Road
- Chandos Street
- Stradbroke Avenue
- Daisy Street

For locations at which there were significant hysteresis effects, the curve resulting in higher flood levels (for a given discharge) was adopted.

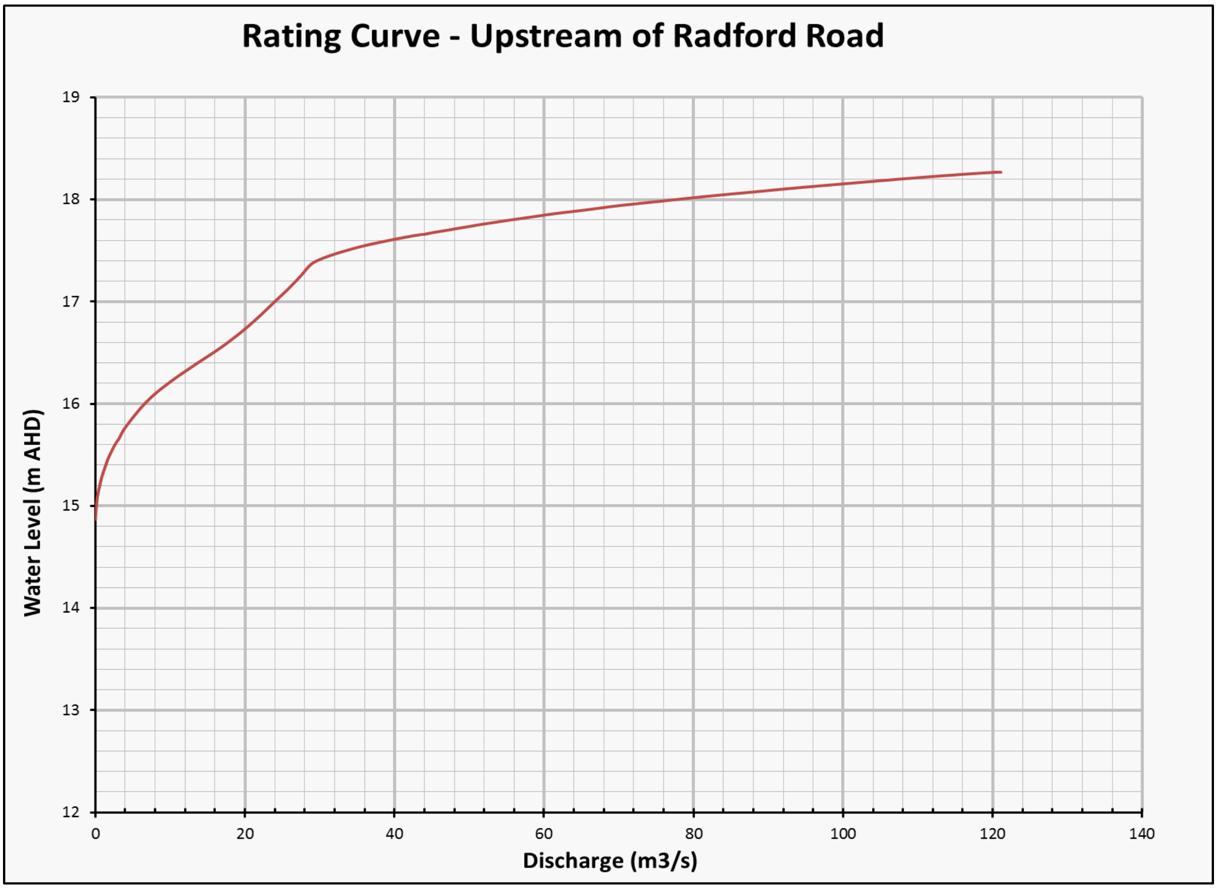

Figure 6.5: Rating Curve – Upstream of Radford Road

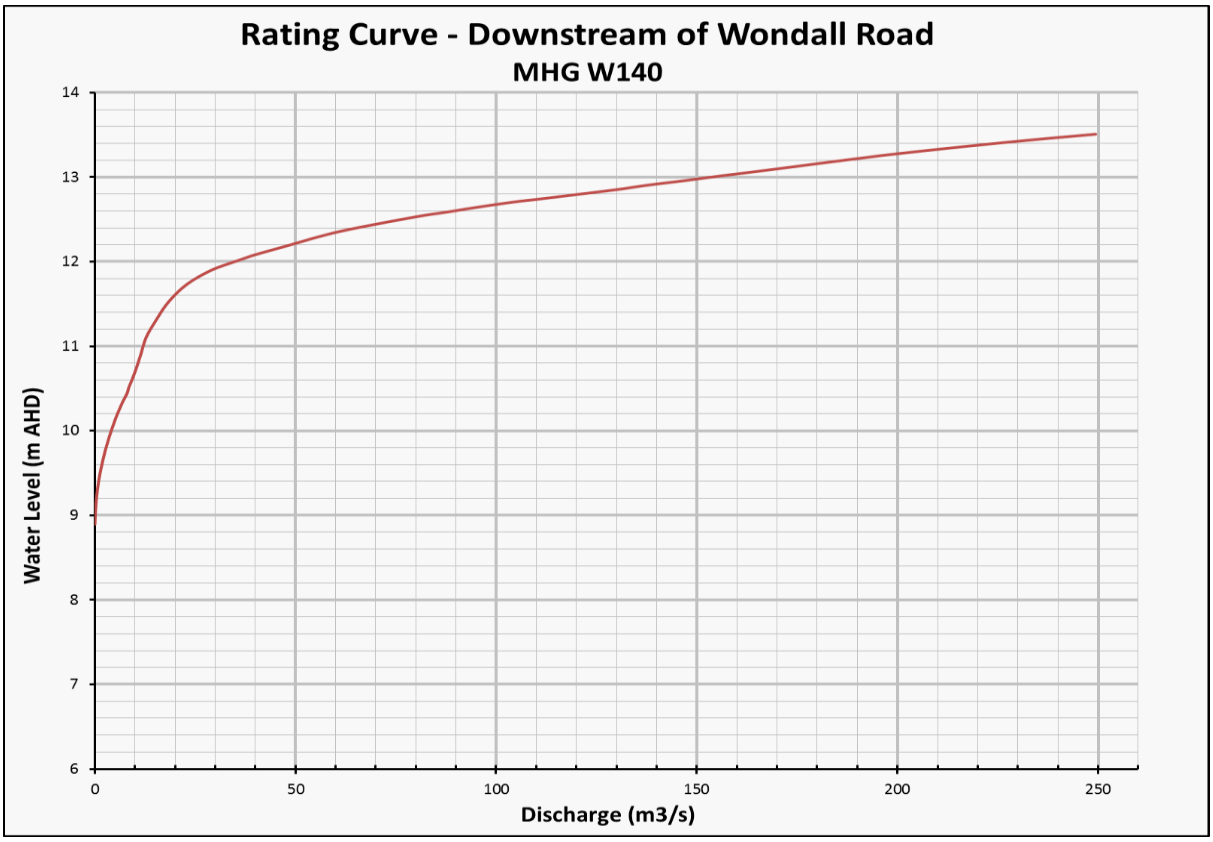

Figure 6.6: Rating Curve – Downstream of Wondall Road

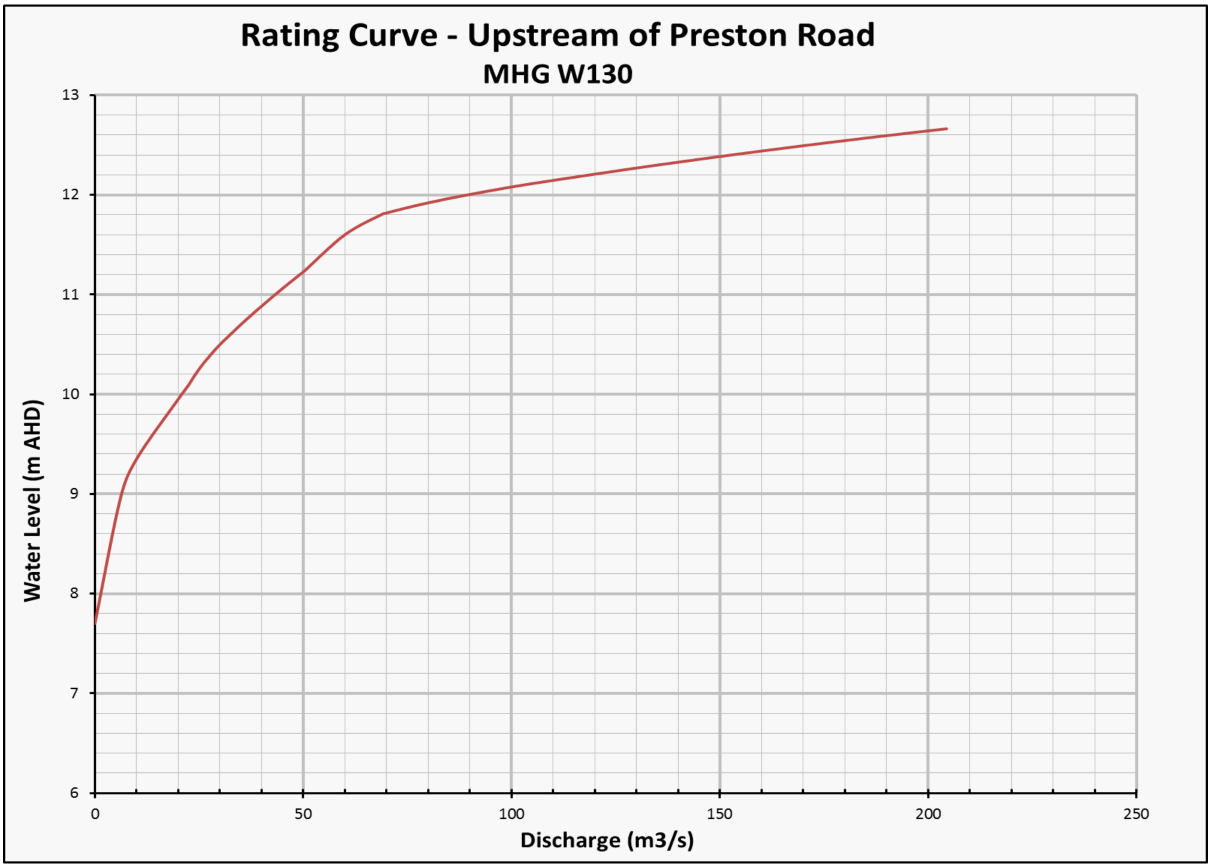

Figure 6.7: Rating Curve – Upstream of Preston Road

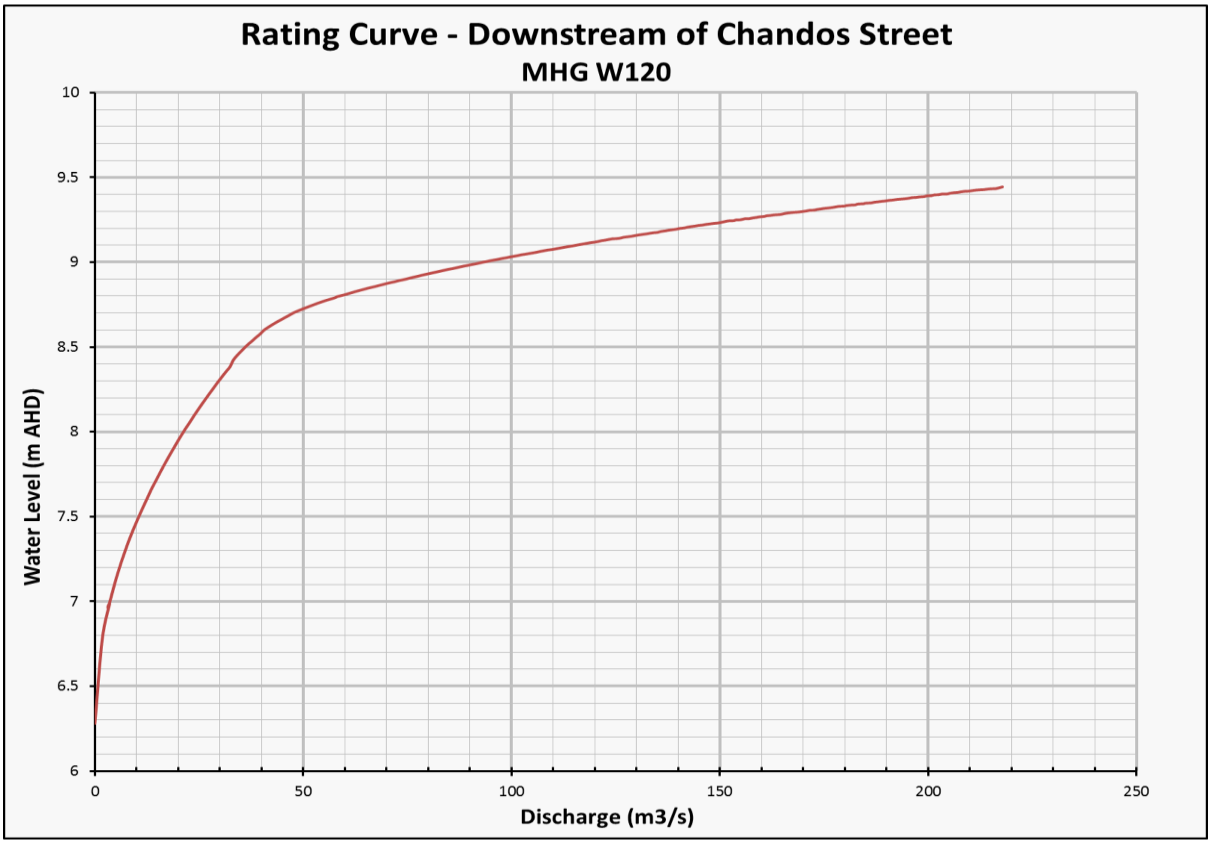

Figure 6.8: Rating Curve – Downstream of Chandos Street

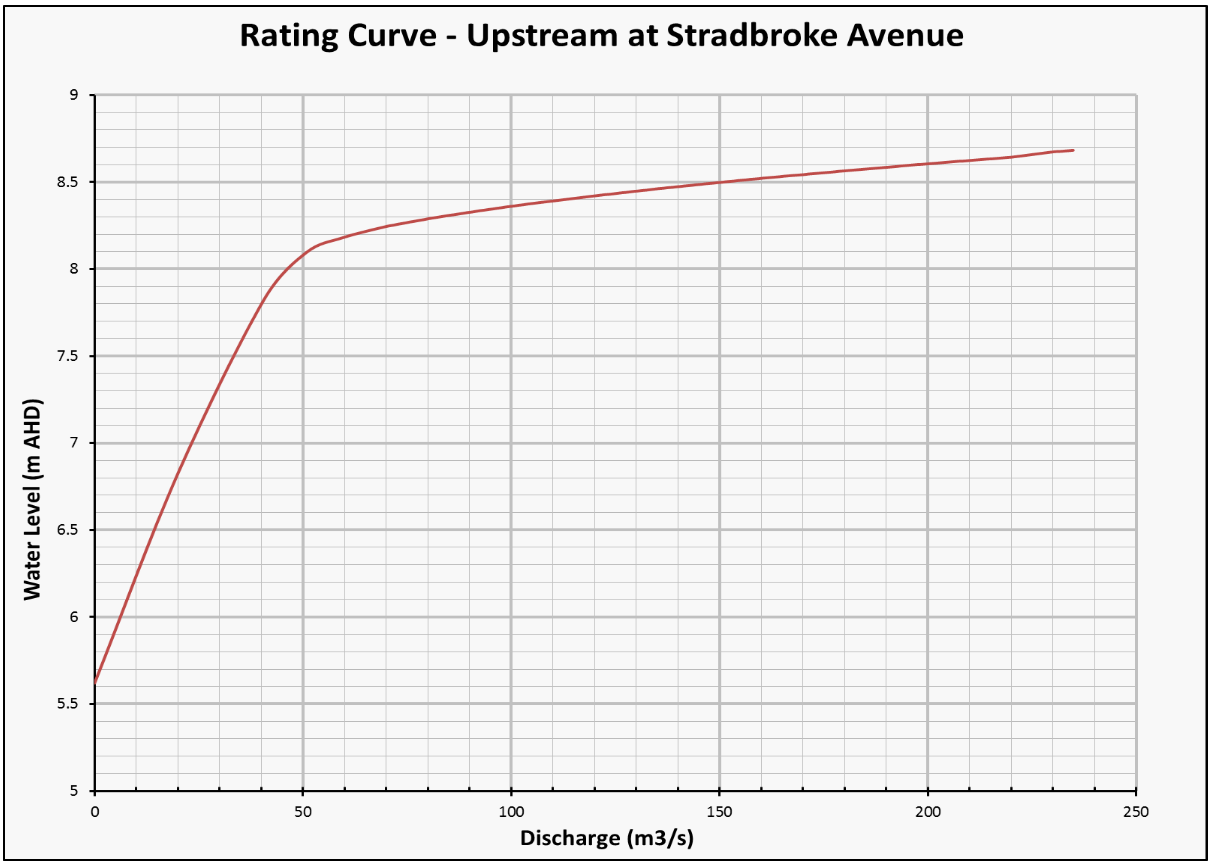

Figure 6.9: Rating Curve – Upstream of Stradbroke Avenue

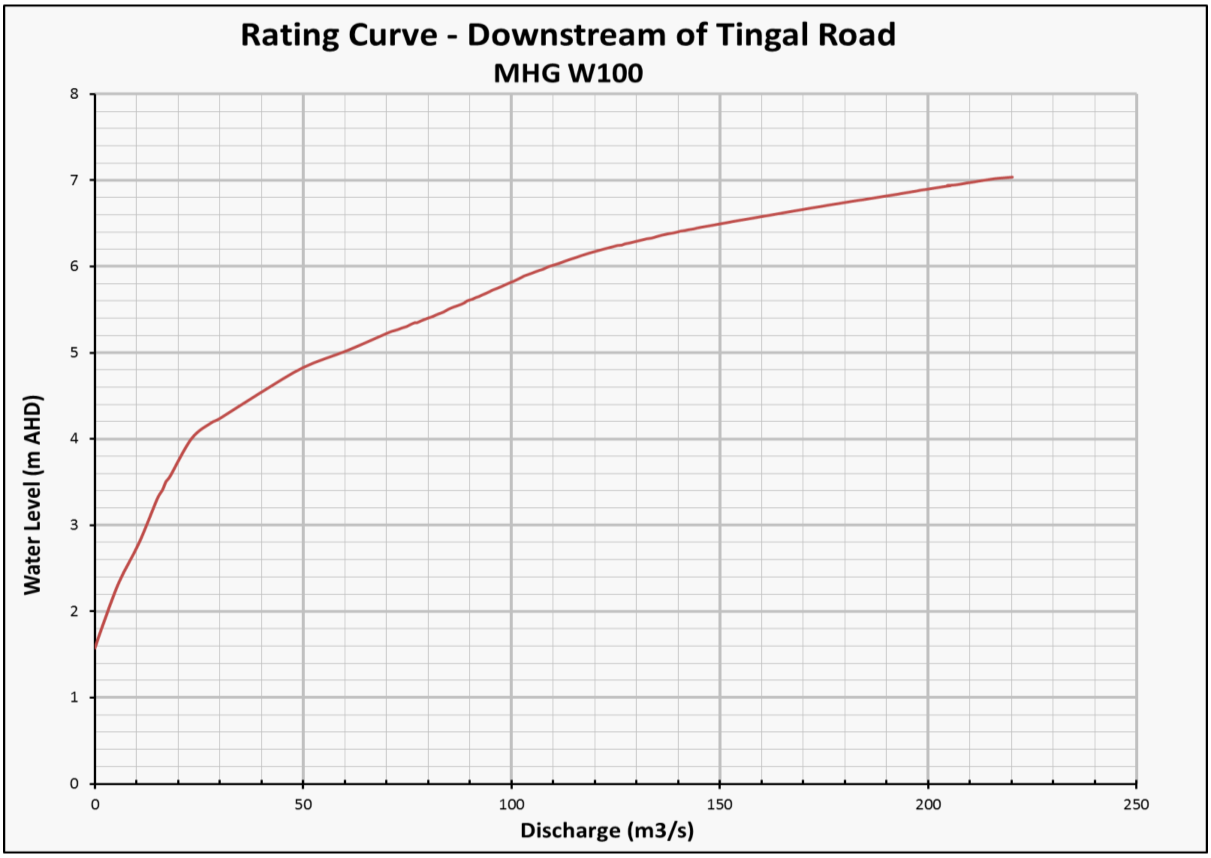

Figure 6.10: Rating Curve – Downstream of Tingal Road

6.5.6 Flood Immunity of Existing Crossings

The flood immunity of the structures under Scenario 3 was determined for each crossing by comparing peak flood levels upstream of the crossing with the minimum overtopping levels. The estimated structure immunities are presented in Table 6.7, where the minimum event considered was the 2-yr ARI (39% AEP) and the maximum was the 100-yr ARI (1% AEP).

Table 6.7 – Flood Immunity at Major Structures

Structure Location	Flood Immunity (ARI)
Graduate Street	10-yr (10% AEP)
Radford Road	20-yr (5% AEP)
Robtrish Street	50-yr (2% AEP)
Talwong Street	100-yr (1% AEP)
Barbara Street	10-yr (10% AEP)
Stannard Road	5-yr (20% AEP)
Wynnum Mainly Leagues Club Access Road	<2-yr (<39% AEP)
Wondall Road	<2-yr (<39% AEP)
Preston Road	2-yr (39% AEP)
Adjacent Haig Street	<2-yr (<39% AEP)
Chandos Street	2-yr (39% AEP)
Stradbroke Avenue	<2-yr (<39% AEP)
Daisy Street	<2-yr (<39% AEP)
QLD Railway	>100-yr (>1% AEP)
Tingal Road	2-yr (39% AEP)
Coreen Street	<2-yr (<39% AEP)
Fox Street	20-yr (5% AEP)

6.5.7 Hydraulic Structure Reference Sheets

Details of flood level and flow data derived for the hydraulic structure crossings modelled are summarised in the Hydraulic Structure Reference Sheets and included in Appendix G.

6.5.8 Flood Mapping

The flood mapping products provided in Appendix J (Volume 2) and include the following:

- Flood Level Mapping
  - Scenario 3: 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP)
- Flood Depth Mapping
  - Scenario 3: 2-yr ARI (39 % AEP) to 100-yr ARI (1 % AEP)

Scenario 3 “ultimate” flood level planning surfaces were required to be generated and mapped. Within the flood modelling context, the ultimate scenario involves modifying the flood model topography to represent a fully developed floodplain in accordance with City Plan and in most instances applying an allowance for a riparian corridor. This process generally results in design flood levels being increased, when compared with Scenario 1 “existing” flood levels. Council requires these increased levels to then be mapped against the current floodplain topography thus providing a flood extent that is conservative, extends beyond the “existing” flood extent and ‘flags’ the additional properties that could potentially be at flood risk in the future and should have development controls (planning levels) applied.

With the move to ‘two-dimensional’ flood models, the production of flood levels, extents and depth-velocity products is inherent in simulating a model, i.e. a flood map is a direct output from a model simulation removing the requirement to apply a separate process. For the Scenario 1 “existing” simulations, the model is run and the direct output is able to be mapped or referenced in a GIS environment. In order to simulate the “ultimate” scenario, the model topography must be modified to represent filling associated with development. This in turn affects the resulting flood mapping with the flood extent limited to the edge of the filled floodplain. Post processing of the model output is required to represent the modelled flood levels against the current floodplain conditions.

The WaterRide stretching tool was selected for the purpose of processing the Scenario 3 “ultimate” case results and producing the planning flood levels and surfaces. The stretching calculation starts at the north-easterly corner where it identifies each “dry cell” which is located immediately adjacent to the “wet cells”. It then calculates a water level for the dry cell by interpolating the neighbouring flood levels. If the assigned flood level is higher than the ground level for that cell, then the cell will be identified as wet. If this condition is not met (i.e. water level is less than ground level) then this cell will be identified as dry. This is an iterative process and continues counter clockwise until there is no wet cell left in a single revolution. To better control the process, a tolerance is adopted in the determination of a wet cell, being a water depth of 300 mm.

From experience to date, it is known that the WaterRide stretching tool alone cannot provide robust surface and level information in all conditions. Therefore, a thorough review of each surface produced by the tool was undertaken and manual intervention applied to the process to ensure suitable outcomes. To help with the initial review process, a comparison of the stretched extent with calculated flood extents including existing scenarios and larger events was undertaken. To modify the stretched surface, break lines were used to limit the expansion of the surface and to stop the “leakage” (upstream higher water level projecting to the downstream lower area) of the surface in problematic areas. Applying break lines at the right place enhances the produced flood levels and surfaces and minimises the anomalies across the flood extent.

In general, the modified areas are mostly observed around tight bends; at structures with high head losses; steep areas where the water can leak; stream junctions where cross-flow is likely; parallel channels; secondary paths and breakout areas. Specific application of the break lines for this flood study is detailed in Appendix H.

Despite the review of the stretched surfaces and the inclusion of break lines to manipulate the stretching process, the process and outputs are still subject to limitations as follows:

- The application of break lines will result in significant steps in the generated surface in some locations.
- The application of break lines is highly subjective in some locations.

- The application of break lines will not necessarily be consistent across all design events (i.e. they will change in number and location depending on the magnitude of the design event considered).
- The stretching process may not be readily repeatable (i.e. the output has not come directly from a model simulation and if model outputs change, it cannot be guaranteed that the process will not need further refinement to produce acceptable results).

Difficult areas to apply the stretching process, which could benefit from further refinement, are highlighted in Appendix H.

7.0 Rare and Extreme Event Analysis

7.1 Extreme Event Scenarios

Table 7.1 indicates the events and scenarios modelled as part of the extreme event analysis. These scenarios have been previously described in Section 6.2. All extreme event modelling was undertaken using ultimate hydrological conditions.

Table 7.1 – Extreme Event Scenarios

ARI (year)	AEP (%)	Scenario 1	Scenario 2	Scenario 3
200	0.5	✓	×	✓
500	0.2	✓	×	✓
2000	0.05	✓	×	×
PMF		✓	×	×

7.2 Extreme Event Hydrology

7.2.1 Overview

Extreme event flood hydrology was determined for the following events, as detailed further in Sections 7.2.2 to 7.2.4.

(i) 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events

(ii) 2000-yr ARI (0.05 % AEP) event, and

(iii) Probable Maximum Precipitation (PMP)

7.2.2 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) Events

The 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) design IFD rainfall data was obtained using the CRC-Forge method for the events.

Table 7.2 indicates the adopted 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) design rainfall intensities with comparison to the adopted 100-yr ARI (1 % AEP). The 1.5 hour, 2-hour and 4.5-hour values were interpolated as CRC-Forge does not produce results for these intermediate values. The interpolation was based by plotting a graph (i.e. 200-yr and 500-yr ARI) and estimating the values at the time of interest.

The 100-yr ARI (1 % AEP) AR&R design temporal pattern was adopted for both these events to create the hyetograph.

Table 7.2 – Adopted IFD (200-yr ARI and 500-yr ARI)

Duration (hr)	Rainfall Intensity (mm/hr)		
	100-yr ARI (1 % AEP)	200-yr ARI (0.5 % AEP)	500-yr ARI (0.2 % AEP)
0.5	147	161.8	188.1
1	101	112.6	130.8
1.5	79.1	90 [1]	104 [1]
2	66.2	75 [1]	87 [1]
3	50.8	56.0	65.0
4.5	39.1	42 [1]	49 [1]
6	32.2	35.7	41.5

1. Interpolated value

7.2.3 2000-yr ARI (0.05 % AEP)

The 2000-yr ARI (0.05 % AEP) IFD rainfall was determined using the CRC-Forge method. To avoid the need to simulate all of the different storm durations, a simplified super-storm method was used. This same methodology has also been used on other BCC flood studies currently being undertaken.

The rationale for adopting this approach is that world-wide research indicates that as storm rainfall depths increase during short duration storms, the rainfall intensity becomes more uniform. For this reason, the multi-peaked AR&R temporal pattern (as used for the 200-yr ARI and 500-yr ARI) was not considered suitable for the analysis of this more extreme event.

A 6-hr super-storm was developed to represent all storm durations up to 6 hours. The super-storm was developed in 30 minute blocks and incorporates the 0.5-hr, 1-hr, 1.5-hr, 2-hr and 3-hr storm bursts. Durations less than 30 minutes were not considered. The total rainfall depth of the super-storm was set equal to the 6-hr 2000-yr ARI (0.05 % AEP) CRC-Forge rainfall depth (representative across the Brisbane Region) which was determined as 340 mm.

7.2.4 PMP

For the PMP scenario, the 6-hr super-storm approach was also undertaken using the same temporal pattern as the 2000-yr ARI (0.05 % AEP) event.

The total PMP depth was derived from the 6-hr storm duration using the Generalised Short Duration Method (GSDM). For the tropical and sub-tropical coastal areas it is recommended that this method is to be used to estimate the PMP over areas up to 520 km2 and for durations up to 6 hours. To apply a consistent methodology across the majority of BCC an average catchment size of 60 km2 and moisture adjustment factor of 0.85 were adopted.

The total rainfall depth of the super-storm was set equal to the 6-hr GSDM PMP rainfall depth, which was determined as 816 mm. Table 7.3 indicates the adopted super-storm temporal pattern and hyetographs for the 2000-yr ARI (0.05 % AEP) and the PMP.

Table 7.3 – Adopted Super-storm Hyetographs

Time (hr)	Rainfall (%)	Rainfall (mm)		Time (hr)	Rainfall (%)	Rainfall (mm)	
		2000-yr ARI (0.05 % AEP)	PMP			2000-yr ARI (0.05 % AEP)	PMP
0.00	0	0.00	0.00	3.17	58	41.00	75.08
0.17	1	4.33	9.92	3.33	70	41.00	75.08
0.33	3	4.33	9.92	3.50	75	16.00	38.25
0.50	4	4.33	9.92	3.67	77	7.58	27.63
0.67	5	4.33	9.92	3.83	80	7.58	27.63
0.83	6	4.33	9.92	4.00	82	7.58	27.63
1.00	8	4.33	9.92	4.17	84	7.58	18.42
1.17	9	4.33	13.46	4.33	86	7.58	18.42
1.33	10	4.33	13.46	4.50	89	7.58	18.42
1.50	11	4.33	13.46	4.67	90	4.33	13.46
1.67	14	7.58	18.42	4.83	91	4.33	13.46
1.83	16	7.58	18.42	5.00	92	4.33	13.46
2.00	18	7.58	18.42	5.17	94	4.33	9.92
2.17	20	7.58	27.63	5.33	95	4.33	9.92
2.33	23	7.58	27.63	5.50	96	4.33	9.92
2.50	25	7.58	27.63	5.67	97	4.33	9.92
2.67	30	16.00	38.25	5.83	99	4.33	9.92
2.83	34	16.00	38.25	6.00	100	4.33	9.92
3.00	46	41.00	75.08				

7.3 Hydraulic Modelling

7.3.1 General

The TUFLOW model was used to simulate the scenarios as detailed in Section 7.1 to enable design flood levels and flood mapping products to be determined / produced.

7.3.2 TUFLOW model grid

For the 2000-yr (0.05% AEP) and PMP events, the TUFLOW model was run with a 4 m grid in lieu of the 2 m grid adopted for the design events and 200-yr and 500-yr events. This enabled greater model stability in the vicinity of the complex hydraulic arrangement between Daisy Street and Tingal Road, as well as Redford Road.

7.3.3 TUFLOW model roughness

No changes were made from the design event TUFLOW model(s).

7.3.4 TUFLOW model boundaries

The extreme event inflow (Q-T) boundaries to the TUFLOW model were taken from the results of the XP-RAFTS model for each ARI and duration. The inflow locations did not change from the design event TUFLOW model.

The TUFLOW model utilised a fixed water level (H-T) boundary as the downstream boundary at Moreton Bay. A HAT value of 1.52 m AHD has been used for all extreme events including the following:

- 200-yr ARI (0.5 % AEP)
- 500-yr ARI (0.2 % AEP)
- 2000-yr ARI (0.05 % AEP)
- PMF

7.3.5 Hydraulic Structures

All extreme event TUFLOW models incorporated the same hydraulic structures as the design event TUFLOW models.

7.4 Results and Mapping

7.4.1 Peak Flood Levels

Tabulated peak flood level results are provided in Appendix E for both Wynnum Creek and the East Drain. These tabulated flood levels are provided for the following events and scenarios:

- 200-yr ARI (0.5 % AEP) – Scenario 3
- 500-yr ARI (0.2 % AEP) – Scenario 3

7.4.2 Flood Mapping Products

Flood mapping products for the extreme events are provided in Appendix J (A3 Booklet) and include the following mapping products:

- Flood Level / Extent Mapping
  - Scenario 1: 2000-yr ARI (0.05 % AEP)
  - Scenario 3: 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP)

Refer to Section 6.5.8 Flood Mapping for discussion of mapping process.

7.4.3 Discussion of Results

A longitudinal plot of the 100-yr ARI (1% AEP), 200-yr (0.5% AEP) and 500-yr (0.2% AEP) events is presented in Figure 7.1 and 7.2 for the main Wynnum Creek and East Drain, respectively, to aid in the discussion of the results. The average increase in flood depth for the main branch of Wynnum Creek when compared to the 100-yr ARI (0.1% AEP) (Scenario 3) flood profile is:

- 200 year ARI (Scenario3) event: 0.18 m
- 500 year ARI (Scenario3) event: 0.38 m

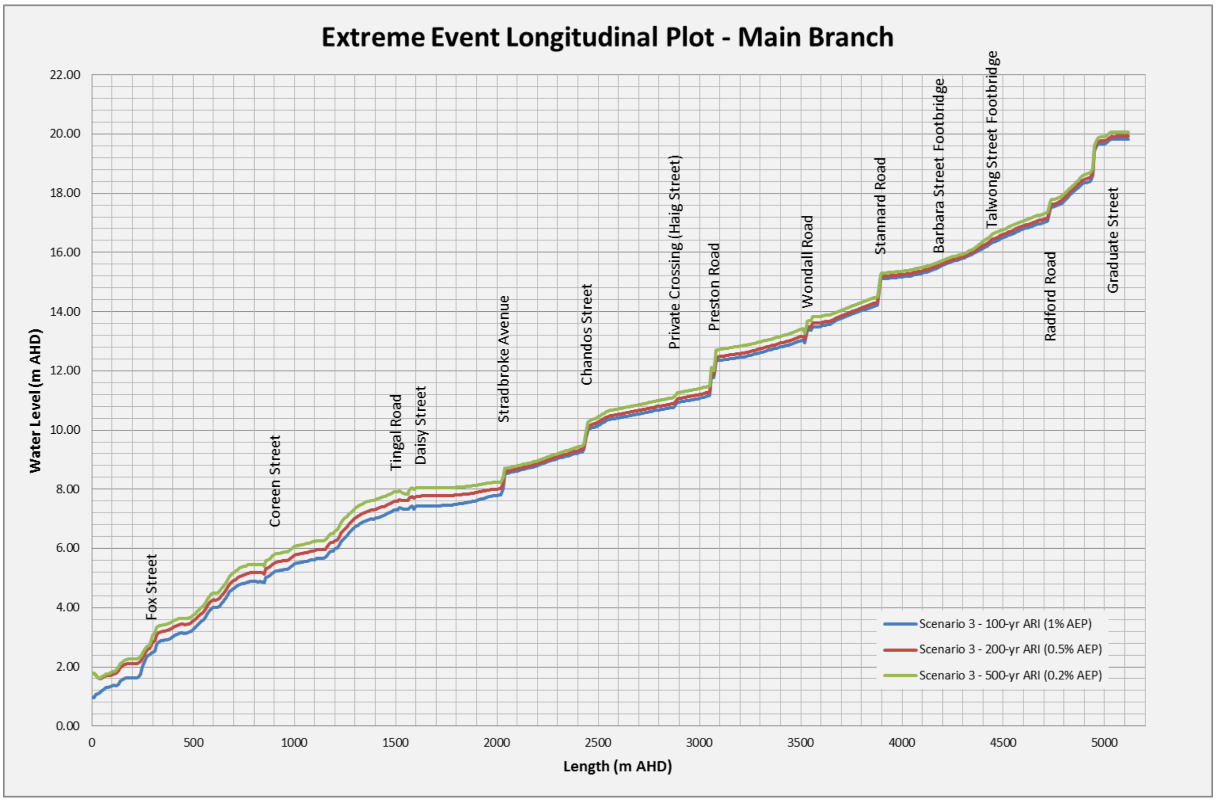

Figure 7.1: Longitudinal Profile 100-yr (1% AEP), 200-yr (0.5% AEP) and 500-yr (0.2% AEP) – Main Branch (Scenario 3)

The flood profile for both the 200-yr ARI (0.5% AEP) and 500-yr ARI (0.2% AEP) events are observed to follow a very similar trend when compared to the 100-yr ARI (1% AEP) flood profile along the main and east branch of Wynnum Creek (refer to Figure 7.1 and 7.2)

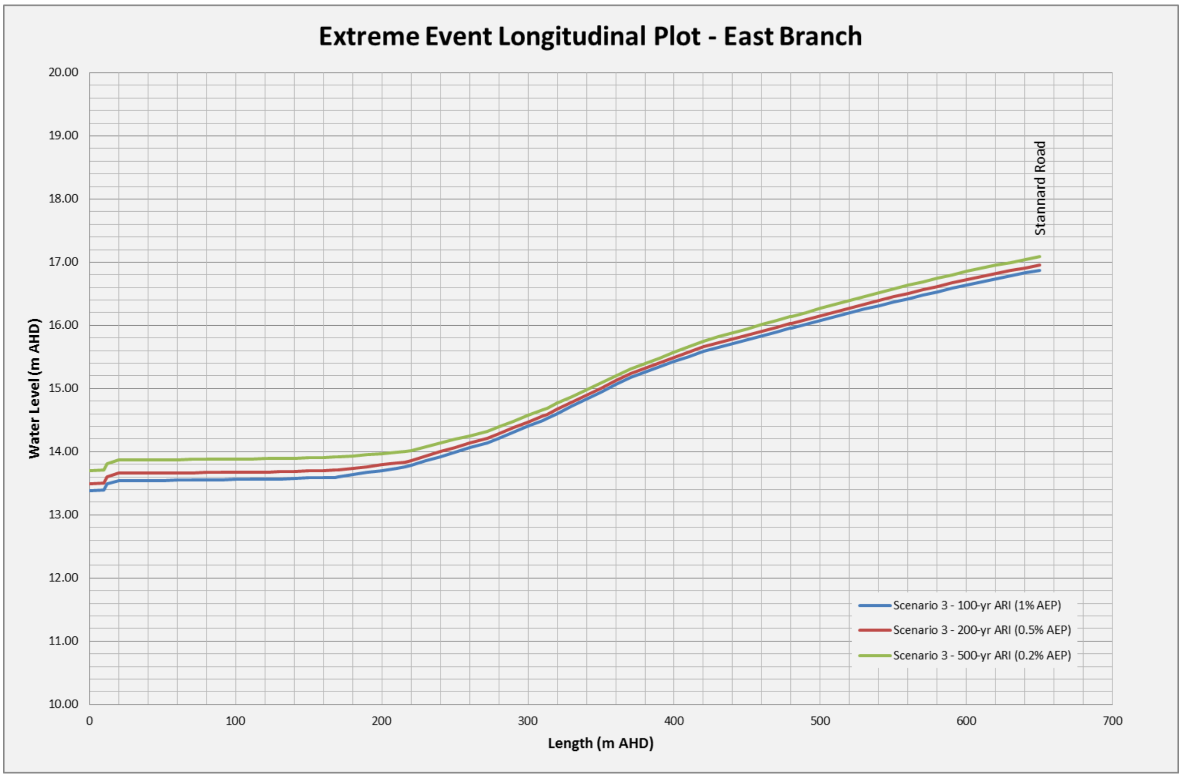

Figure 7.2: Longitudinal Profile 100-yr (1% AEP), 200-yr (0.5% AEP) and 500-yr (0.2% AEP) – East Branch (Scenario 3).

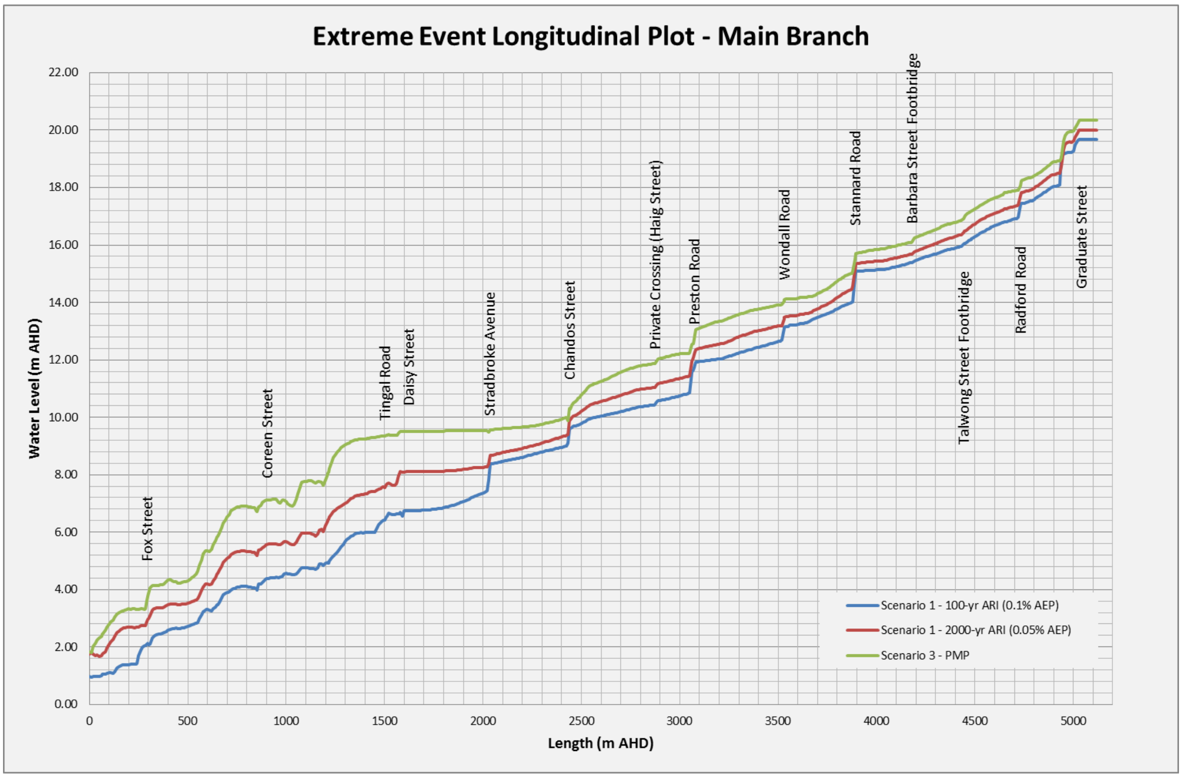

Figure 7.3: Longitudinal Profile 100-yr (1% AEP), 2000-yr (0.05% AEP) and PMF – Main Branch (Scenario 1)

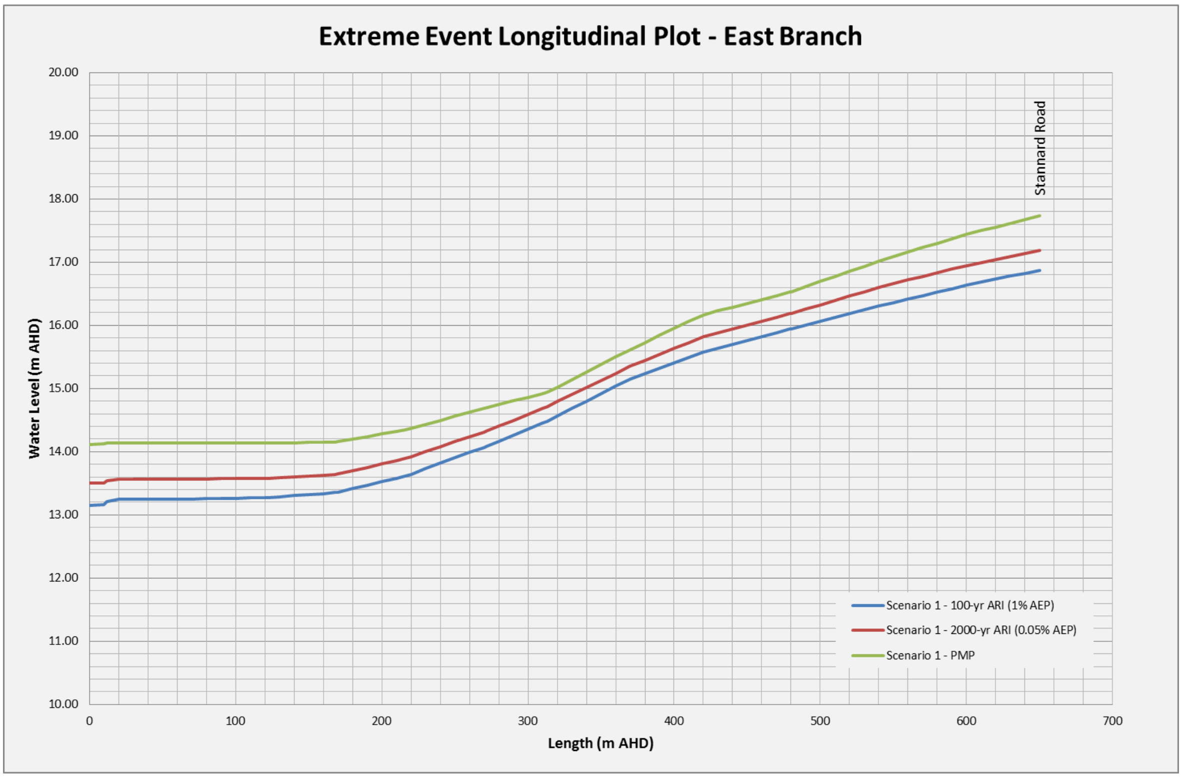

Figure 7.4: Longitudinal Profile 100-yr (1% AEP), 2000-yr (0.05% AEP) and PMF – East Branch (Scenario 1)

The flood profiles for 2000-yr ARI (0.05% AEP) and PMF indicate a consistent flood level difference upstream of Chandos Street (refer Figure 7.3). The railway embankment immediately downstream of Daisy Street poses as a constriction to the flow, resulting in increased water levels immediately upstream of this location. Downstream of Tingal Road, the natural channel, together with the lack of floodplain storage shows a significant increase in levels for larger events.

The average increase in flood depth in the main channel when compared to the 100-yr ARI (Scenario 1) flood profile is:

- 2000 year ARI (Scenario1) event: 0.70 m
- PMF (Scenario1) event: 1.59 m

8.0 Climate Change and Structure Blockage

8.1 Overview

To enable comprehensive planning to be undertaken, BCC flood studies are required to undertake a sensitivity analysis to address the following:

- Climate change
- Hydraulic structure blockage

The following sections provide the details of these analyses.

8.2 Climate Change

8.2.1 Overview

To allow BCC to intelligently undertake future land-use planning, there is a requirement to understand the impacts of climate change on flooding. BCC flood studies are therefore required to utilise the latest statutory guidelines in order to assess the impacts of climate change.

To enable BCC to understand and plan for the impacts of climate change on flooding in the Wynnum Creek Catchment, a number of climate change scenarios were undertaken, as outlined below. These scenarios are consistent with those undertaken in recently completed BCC flood studies and the latest statutory guidelines.

- 2050 Planning Horizon
    - 10 % increase in rainfall intensity
    - 0.3 m increase in mean sea level

- 2100 Planning Horizon
    - 20 % increase in rainfall intensity
    - 0.8 m increase in mean sea level

8.2.2 Modelled Scenarios

Modelling was used to determine climate change impacts for the 100-yr ARI (1 % AEP), 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events. Table 8.1 indicates the events modelled and the respective climate change modifications undertaken.

The rainfall intensity in the XP-RAFTS model was increased by 10 % (or 20 %) and simulations undertaken to determine the climate change hydrographs. These hydrographs were then input into the TUFLOW model and simulations undertaken for all climate change scenarios. A 4 metre grid has been used for 500-yr ARI (0.2% AEP) as it produced a more stable solution than the 2 metre grid.

Table 8.1 – Climate Change Modelling Scenarios

ARI (year)	AEP (%)	Planning horizon	Rainfall Condition	Tailwater Condition	Scenario 1	Scenario 3
100	1	2050	+ 10 %	MHWS + 0.3 m	✓	✓
		2100	+ 20 %	MHWS + 0.8 m	✓	✓
200	0.5	2050	+ 10 %	HAT + 0.3 m	✓	×
		2100	+ 20 %	HAT + 0.8 m	✓	×
500	0.2	2100	+ 20 %	HAT + 0.8 m	✓	×

8.2.3 Impacts of Climate Change

Tables 8.2 to 8.4 indicate the peak flood level climate change comparison for Scenario 1. The flood level results are provided at selected locations along the creek for the 100-yr ARI (1 % AEP), 200-yr ARI (0.5 % AEP) and 500-yr ARI (0.2 % AEP) events. The results indicate the greatest change in flood level is in the lower reach downstream of Tingal Road, where the dominant impact mechanism is the increase in water level in Moreton Bay.

Tabulated peak flood level results for the 100-yr ARI Scenario 3 events are provided in Appendix F for both Wynnum Creek and the East Drain.

Table 8.2 – 100-yr ARI (1% AEP) Climate Change Impacts at Selected Locations (Scenario 1)

Structure Location	100-yr ARI (1% AEP) Flood Level (m AHD)		
	Existing	2050	2100
Graduate Street	19.68	19.75	19.81
Radford Road	17.45	17.52	17.59
Stannard Road	15.05	15.09	15.14
Wondall Road	13.14	13.19	13.25
Preston Road	11.73	11.76	11.79
Chandos Street	9.61	9.67	9.73
Stradbroke Avenue	8.34	8.38	8.41
Tingal Road	6.56	6.72	6.85
Fox Street	2.11	2.25	2.51

Table 8.3 – 200-yr ARI (0.5% AEP) Climate Change Impacts at Selected Locations (Scenario 1)

Structure Location	200-yr ARI (0.5% AEP) Flood Level (m AHD)		
	Existing	2050	2100
Graduate Street	19.76	19.83	19.89
Radford Road	17.53	17.61	17.67
Stannard Road	15.10	15.16	15.20
Wondall Road	13.20	13.27	13.32
Preston Road	11.76	11.80	11.83
Chandos Street	9.68	9.75	9.81
Stradbroke Avenue	8.38	8.42	8.46
Tingal Road	6.74	6.89	7.03
Fox Street	2.3	2.56	2.83

Table 8.4 – 500-yr ARI (0.2% AEP) Climate Change Impacts at Selected Locations (Scenario 1)

Structure Location	500-yr ARI (0.2% AEP) Flood Level (m AHD)	
	Existing	2100
Graduate Street	19.86	19.99
Radford Road	17.65	17.77
Stannard Road	15.19	15.28
Wondall Road	13.3	13.41
Preston Road	11.82	12.11
Chandos Street	9.78	9.86
Stradbroke Avenue	8.44	8.55
Tingal Road	6.97	7.40
Fox Street	2.52	2.94

8.3 Hydraulic Structure Blockage

8.3.1 Overview

Blockage of hydraulic structures is a common cause of increasing flood risk over and above the risk due to the intensity and duration of the rainfall. Current guidance recommends that designers of hydraulic structures should make allowances for the risk of blockage in the design. However, current guidance does not stipulate that blockage is required to be included as part of the determination of the overall design flood level.

BCC has taken the approach to include the blockage of selected hydraulic structures as part of a sensitivity analysis. This approach will allow BCC to understand the potential impacts should the selected hydraulic structures become blocked during an event.

8.3.2 Selection of Hydraulic Structures

The following hydraulic structures were selected for the blockage analysis:

- Graduate Street – 2 / 1650 RCP
- Radford Road - 2 / 525 RCP + 3 / 1800 RCP
- Stannard Road - 3 / 1800 RCP
- League Club Access - 1 / 750 RCP + 2 / 1200 RCP
- Wondall Road – 4 / 1830 RCP
- Preston Road – Bridge
- Adjacent Haig Street – 3 / 1800 RCP
- Chandos Street - 6 / 1800 RCP
- Stradbroke Street – Bridge
- Daisy Street – 2 / 2750 x 2520
- Tingal Road – Bridge
- Coreen Street Footbridge – Bridge
- Fox Street - Bridge

These structures were primarily selected based on limiting the size of the bridge / culvert dimensions. However, other factors were considered including the following:

- the predominant upstream catchment use;
- availability of woody debris;
- existing submergence of the inlet;
- flood risk of upstream properties; and
- flooding characteristics of the reach

8.3.3 Blockage Scenarios

The blockage analysis has been carried out with the existing case scenario (Scenario 1) for the 100-yr ARI (1% AEP) design event only. Individual structures were blocked and modelled separately to ensure that the blockage impacts would not be masked by the effect of blocking other crossings. A total of thirteen separate runs have been conducted.

The Queensland Urban Drainage Manual (QUDM) was used as guidance for the degree of blockage for each structure. QUDM recommends that culverts of the size found in Wynnum Creek adopt 25% sediment blockage for the culvert barrel and 20% blockage for the culvert inlet.

For the modelling of box culvert blockages, this has been achieved by raising the invert level to account for a sediment blockage of 25% and further reducing the culvert width to account for an additional 20 % of inlet blockage. For concrete pipe culverts, a reduction of 40% to the size of the culvert was applied and the invert raised accordingly.

This approach is considered to be conservative and assumes both inlet blockage and culvert barrel blockage are incremental and occur together.

8.3.4 Impacts of Structure Blockage

Table 8.5 indicates the flood level differences immediately upstream of the hydraulic structure for each of 13 blockage simulations.

Table 8.5 – 100-yr ARI Blockages (Scenario 1)

Blockage Simulation #	Structure Location	Flood Level (m AHD)		Difference (m)
		Existing	Blockage Analysis	
1	Graduate Street	19.68	19.91	0.23
2	Radford Road	17.45	17.71	0.26
3	Stannard Road	15.05	15.18	0.13
4	Leagues Club Access	13.29	13.29	0.00
5	Wondall Road	13.14	13.28	0.14
6	Preston Road	11.73	11.78	0.05
7	Adjacent Haig Street	10.53	10.57	0.04
8	Chandos Street	9.61	9.84	0.23
9	Stradbroke Avenue	8.34	8.39	0.05
10	Daisy Street	6.74	6.74	0.00
11	Tingal Road	6.56	6.58	0.02
12	Coreen Street	4.37	4.51	0.14
13	Fox Street	2.11	2.18	0.07

The Wynnum-Manly Leagues Club access structure is located in a relatively flat area, immediately upstream of the Wondall Road crossing. This structure comprises of 2 / 1200 mm diameter and 1 / 750 mm diameter concrete pipes and overtops during a 2-yr ARI (39 % AEP) event. The relatively flat area and associated wide floodplain results in there being little impact in the 100-yr ARI (1 % AEP) event when this structure is modelled partially blocked.

At Daisy Street, overtopping of the structure occurs as a result of the 2-yr ARI (50 % AEP) event. The flood level at this location appears to be controlled by downstream conditions with significant submergence occurring in the larger flooding events. Therefore, in the 100-yr ARI event, the partial blockage of the structure results in no difference in flood levels.

9.0 Summary of Study Findings

This report details the calibration and verification events, design events, extreme events and sensitivity modelling for the Wynnum Creek Catchment. Hydrologic and hydraulic models of the Wynnum Creek Catchment have been developed using the XP-RAFTS and TUFLOW modelling software respectively.

Hydrometric data was sourced from the available recorded rainfall data. A number of MHG's are available in Wynnum Catchment, however only one continuous stream gauge exists. Calibration of XP-RAFTS and TUFLOW was undertaken with data from the March 2013 and December 2010 events. Verification of the XP-RAFTS and TUFLOW utilised storm events from May 2009 and December 2010.

The results of the hydraulic calibration and verification indicated that the TUFLOW model was able to accurately simulate the historical flooding events to within the tolerances imposed on the study. All four historical events were able to be accurately simulated by the TUFLOW model. On this basis, it was concluded that the XP-RAFTS and TUFLOW models were sufficiently robust to use together to accurately simulate design flood events.

Cross-checks of the TUFLOW structure head-losses were undertaken at selected structures using the HEC-RAS software, from which it was confirmed that the model was representing the structures adequately.

Design and extreme flood magnitudes were estimated for the full range of events from 2-yr ARI (39% AEP) to PMF. These analyses assumed hydrologic ultimate catchment development conditions in accordance with BCC City Plan (2000).

Three waterway scenarios were considered as follows:

- Scenario 1 is based on the current waterway conditions. No further modifications were made to the TUFLOW model developed as part of the calibration / verification phase.
- Scenario 2 includes an allowance for a riparian corridor along the edge of the channel.
- Scenario 3 includes an allowance for the riparian corridor (as per Scenario 2) and also assumes filling to the WC boundary to simulate potential development outside the WC.

The results from the TUFLOW modelling were used to produce the following:

- Peak flood discharges at selected locations
- Critical storm durations at selected locations
- Peak flood levels at cross section reporting points
- Peak flood extent mapping
- Peak flood depth mapping
- Hydraulic structure flood immunity data

As part of the required sensitivity analysis a climate change analysis was then undertaken to determine the impacts for two planning horizons; namely 2050 and 2100. This included making allowances for increased rainfall intensity and increased mean sea level rise. This analysis was undertaken for the 100-yr ARI (1% AEP), 200-yr ARI (0.5% AEP) and 500-yr ARI (0.2% AEP) events.

The sensitivity analysis also included analyses of blockages on significant hydraulic structures. Thirteen structures in the Wynnum Creek Catchment were blocked as per the recommendations in QUDM. Each structure was run independently with its own model simulation to ensure no interference from other structures.

Hydraulic Structure Reference Sheets (HSRS) for all major crossings within the TUFLOW model area were also prepared. The HSRS provide data for each hydraulic structure and include data relating to the structure description, location, hydraulic performance and history.

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APPENDICES

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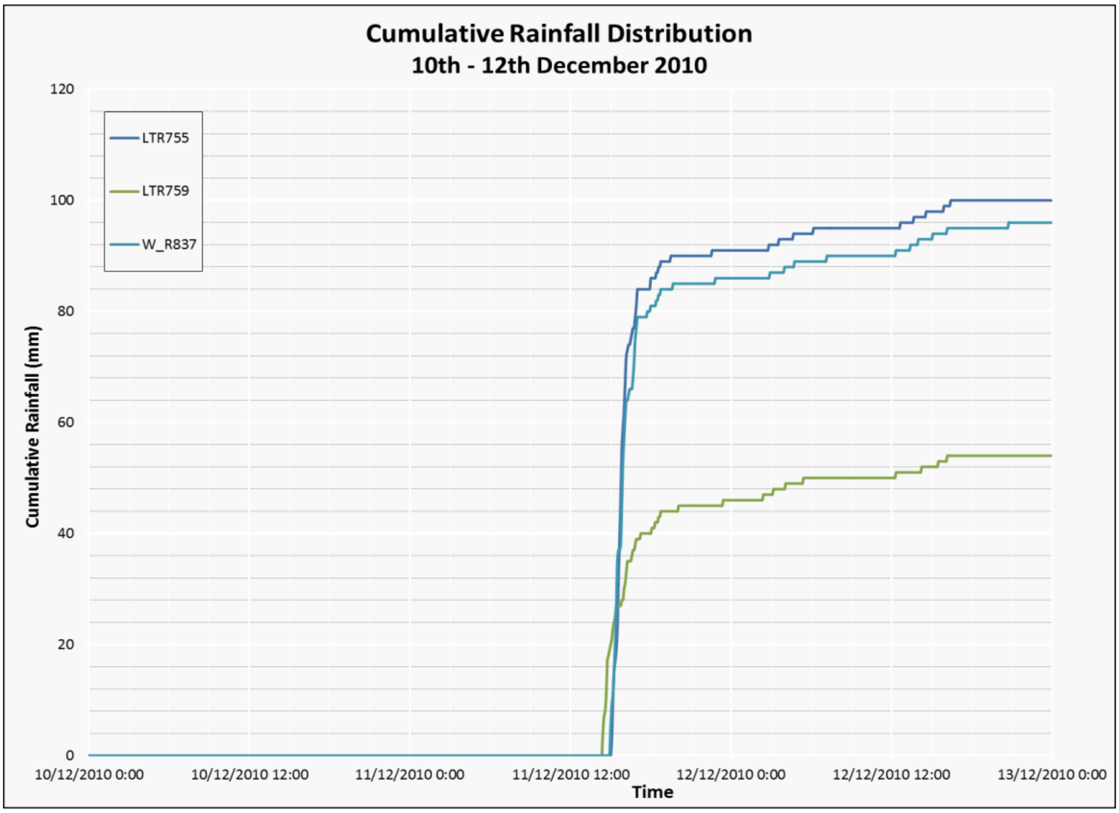
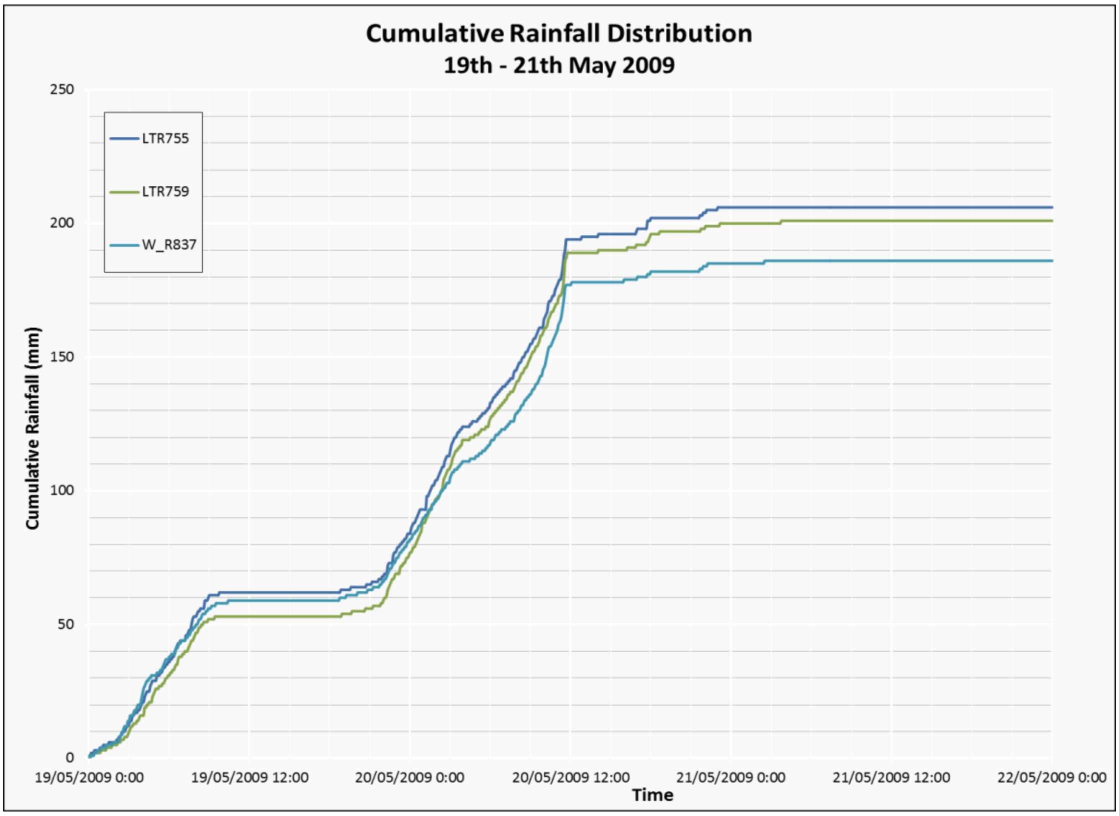
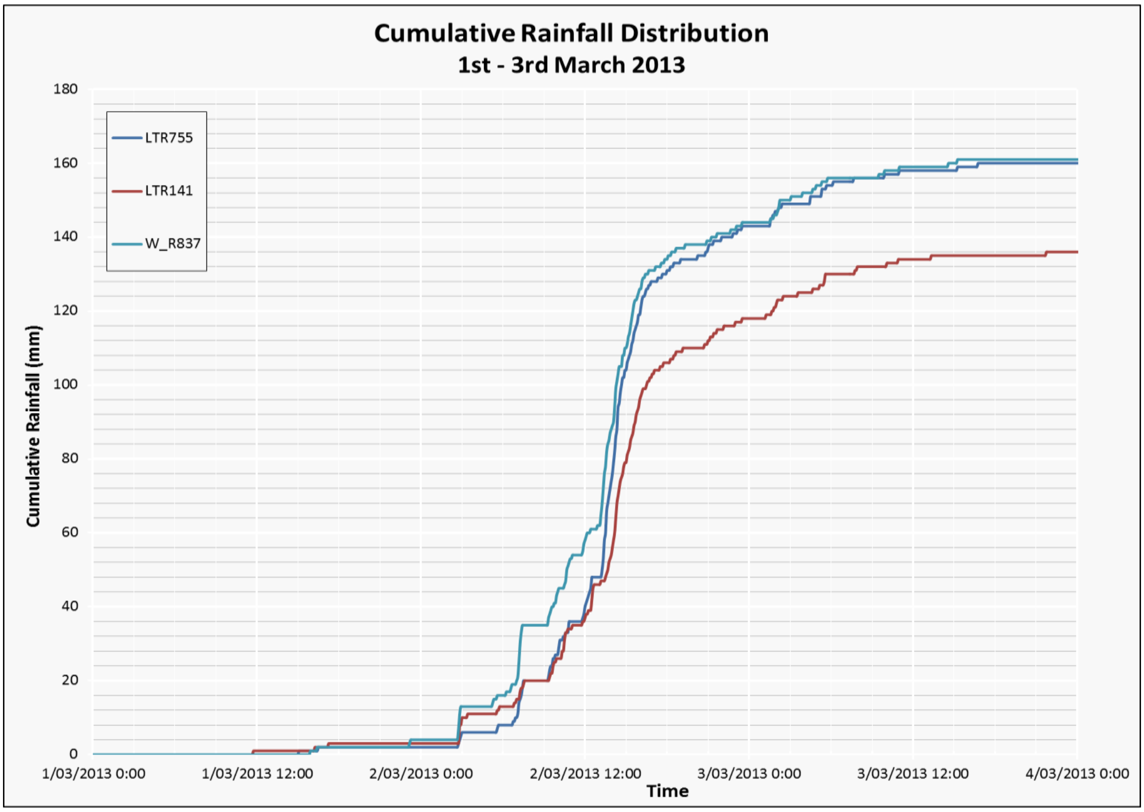
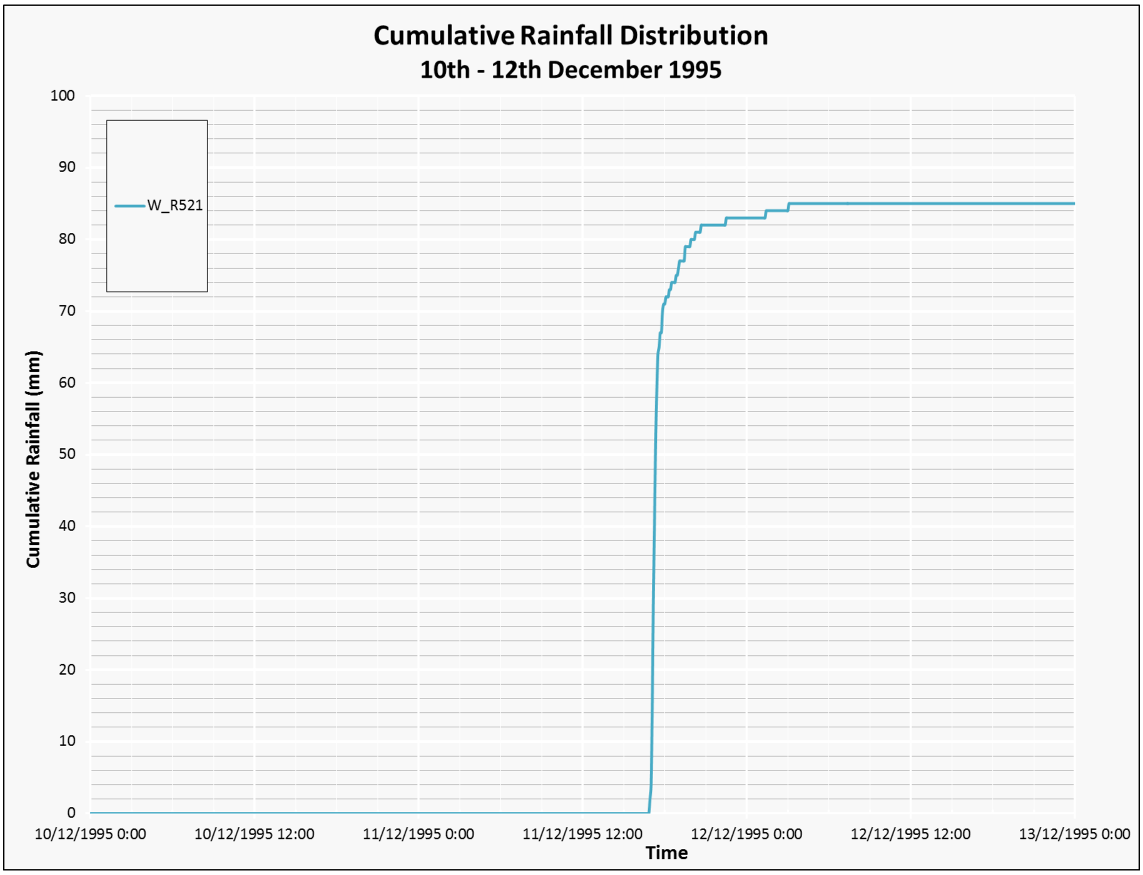
Appendix A – Rainfall Distribution



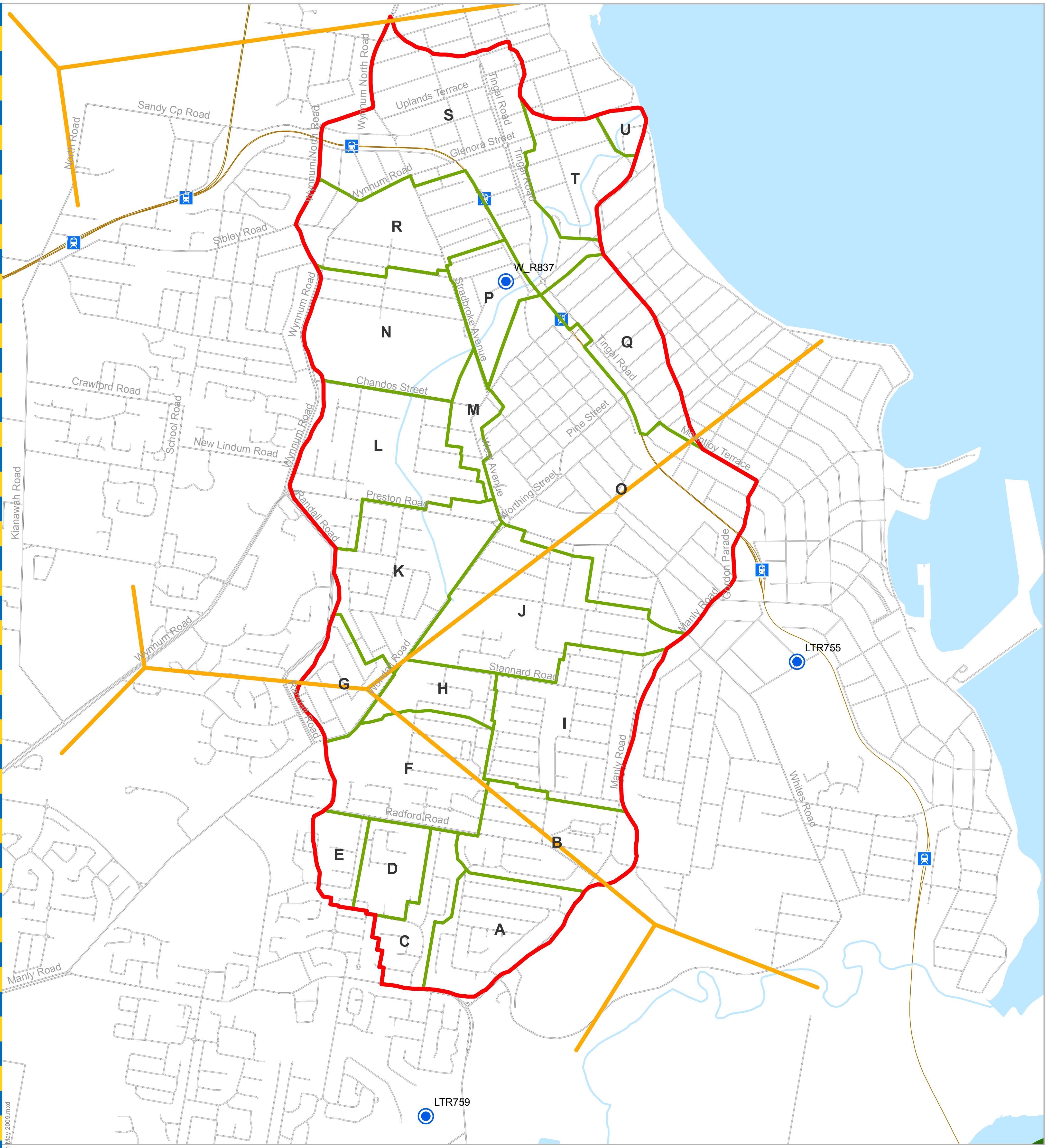


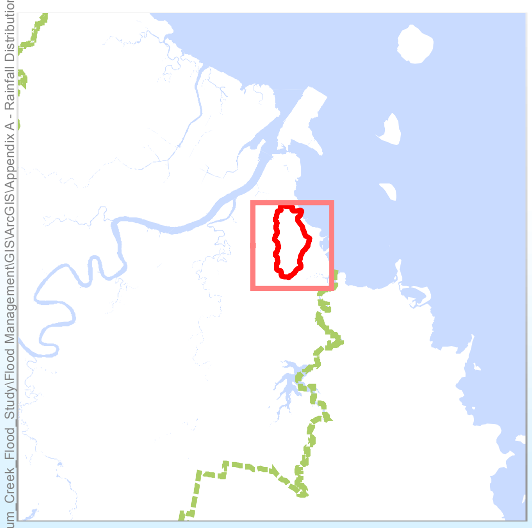


Cumulative Rainfall Distribution
1st - 3rd March 2013
LTR755
LTR141
W_R837
Cumulative Rainfall (mm)
0
20
40
60
80
100
120
140
160
180
1/03/2013 0:00
1/03/2013 12:00
2/03/2013 0:00
2/03/2013 12:00
3/03/2013 0:00
3/03/2013 12:00
4/03/2013 0:00
Time



Cumulative Rainfall Distribution
10th - 12th December 1995
W_R521
Cumulative Rainfall (mm)
0
10
20
30
40
50
60
70
80
90
100
10/12/1995 0:00
10/12/1995 12:00
11/12/1995 0:00
11/12/1995 12:00
12/12/1995 0:00
12/12/1995 12:00
13/12/1995 0:00
Time





Legend

- Rainfall Distribution - May 2009
- Rainfall Gauges - May 2009
- Wynnum Creek Catchment
- RAFTS Subcatchments
- Streets
- Railway Station
- Railway Line

0 150 300 450
Metres

N W E S

Prepared : 077900
Checked : SG
Revision : 0
Publication Date : 16 Jul 2014
Project Number : 130115

File : G:\BI\CD\Proj13\130115_Wynnum_Creek_Flood_Study\Flood Management\GIS\ArcGIS\Appendix A - Rainfall Distribution May 2009.mxd

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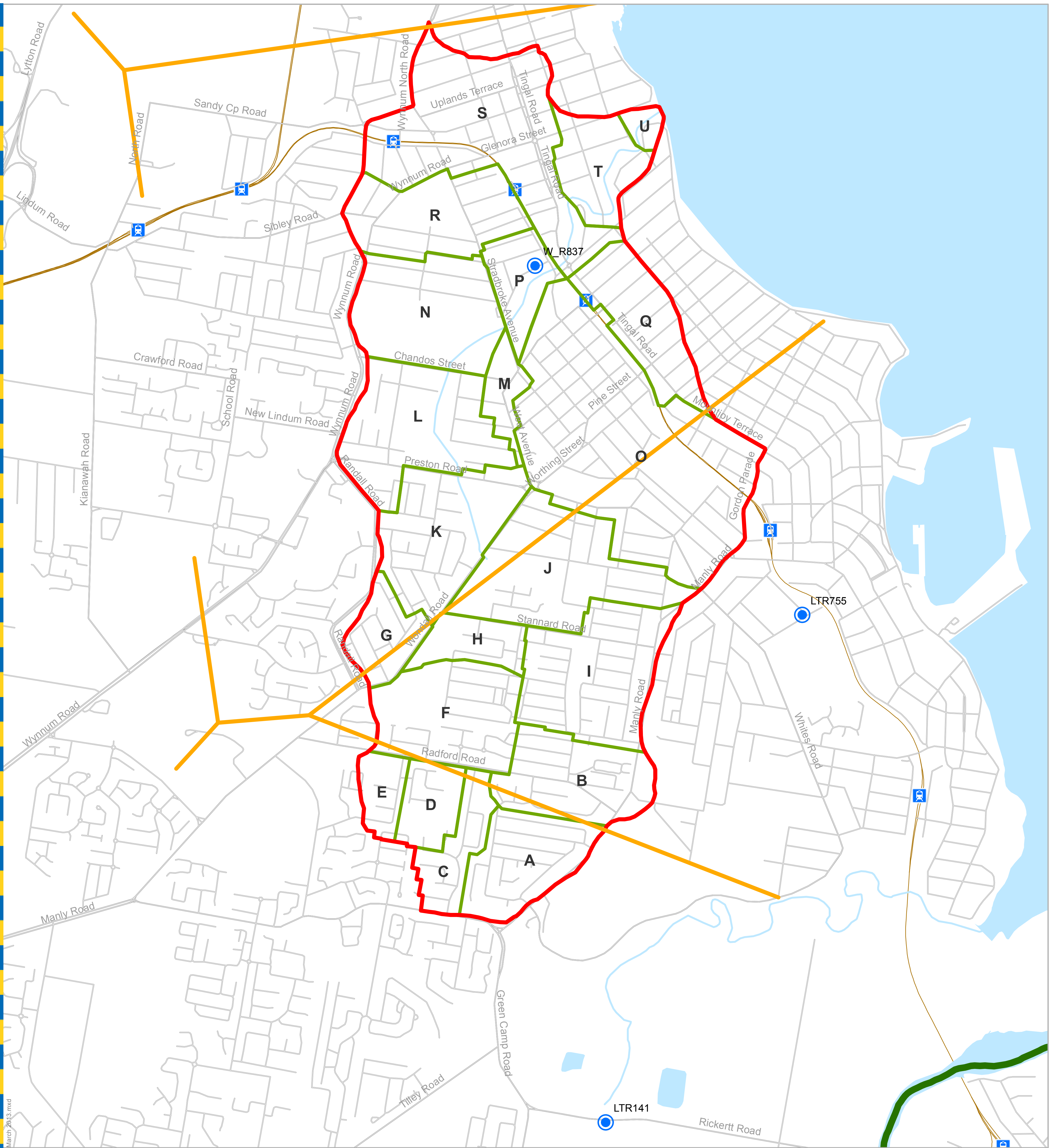
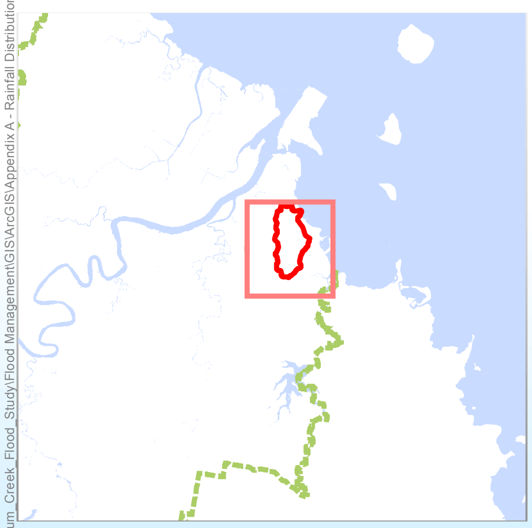
Thiessen Polygon 20th May 2009

FLM - 130115





0 150 300 450

Metres

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Legend

- Rainfall Distribution - March 2013
- Rainfall Gauges - March 2013
- Wynnum Creek Catchment
- RAFTS Subcatchments
- Streets
- Railway Station
- Railway Line

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Thiessen Polygon 2nd March 2013

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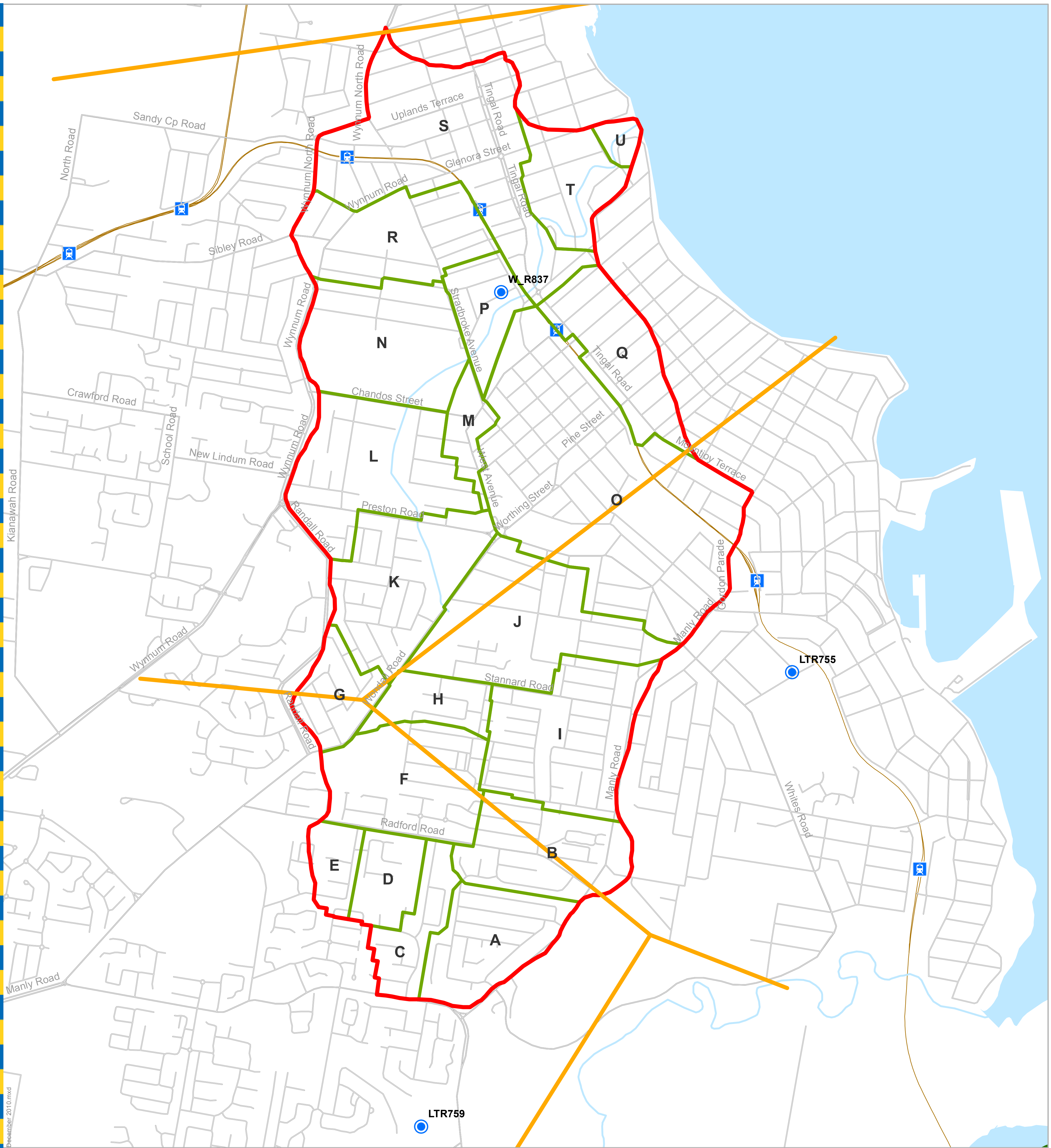

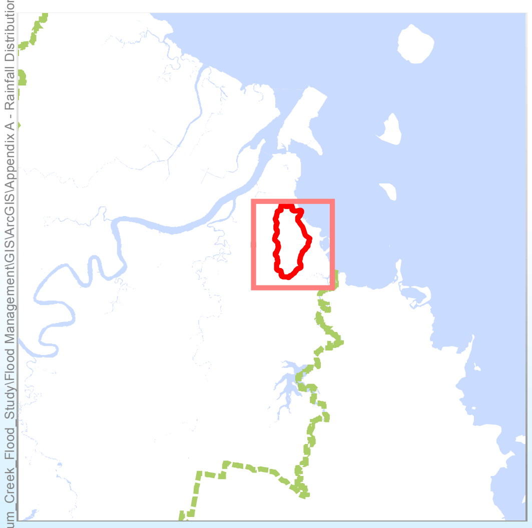

0 150 300 450

Metres

N W E S

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Legend

- Rainfall Distribution - December 2010
- Rainfall Gauges - December 2010
- Wynnum Creek Catchment
- RAFTS Subcatchments
- Streets
- Railway Station
- Railway Line

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Thiessen Polygon
11th December 2010

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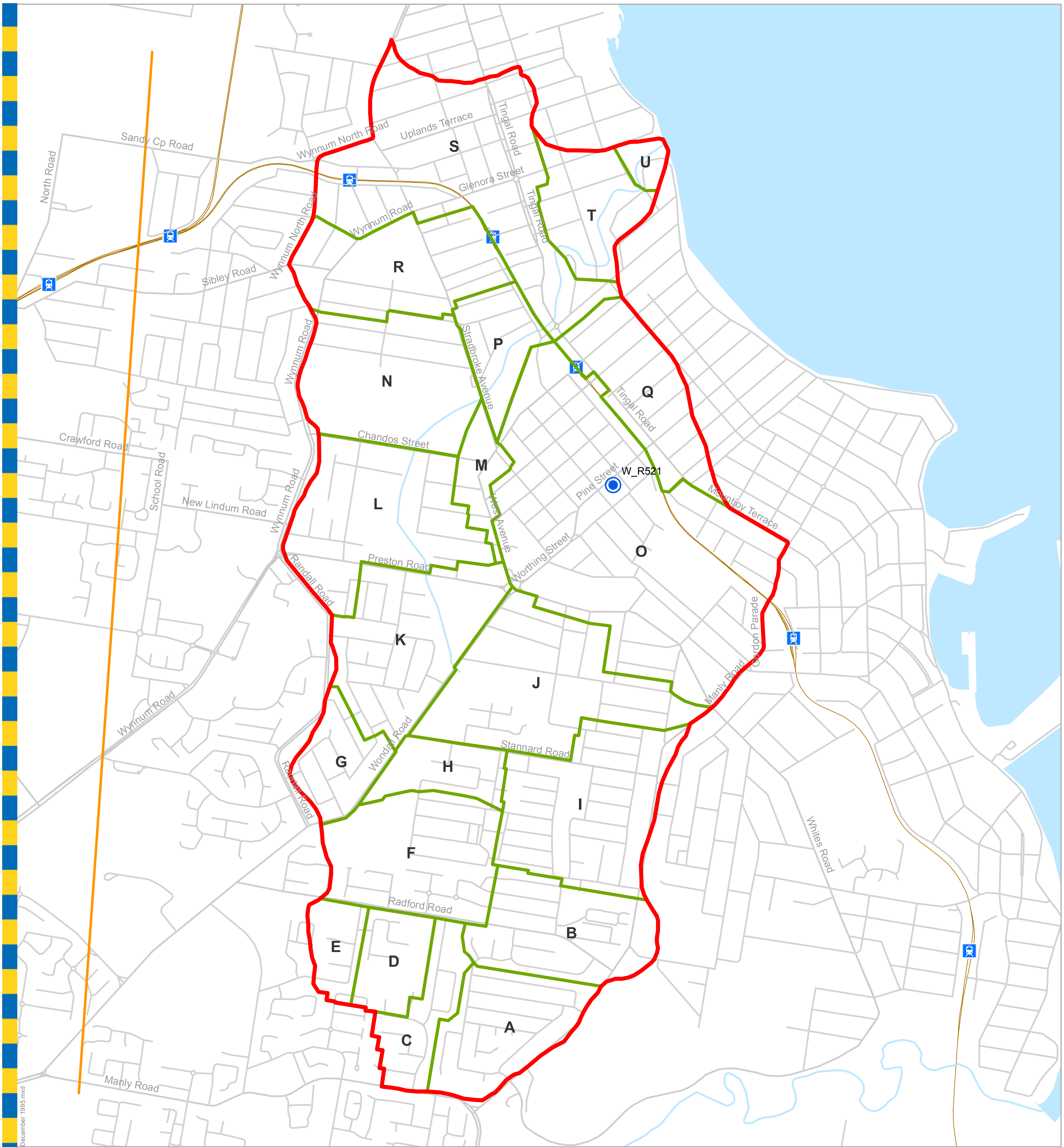

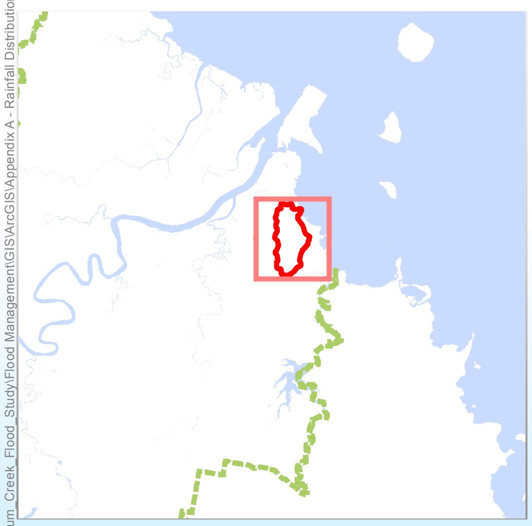

Legend

- Rainfall Distribution - December 1995
- Rainfall Gauge - December 1995
- Wynnum Creek Catchment
- RAFTS Subcatchments
- Streets
- Railway Station
- Railway Line

0 150 300 450
Metres

N W E S

Prepared : 077900
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11th December 1995

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Appendix B – XP-RAFTS Sub-catchment Parameters

Catchment	Total Area [ha]	Percentage Impervious [%]	Catchment Mannings 'n'	Catchment Slope [%]
A	29.78	53.1	0.03	1.8
B	34.67	53.4	0.03	2.1
C1	0.001	0	0.03	2.6
C2	16.74	51	0.03	2.6
D	14.38	41.9	0.03	2.9
E	10.18	49.8	0.03	2.6
Dummy_F	0.001	0	0.03	2
F1	0.001	0	0.03	2
F2	40.92	48.3	0.03	2
G	16.11	55	0.03	2
Dummy_H	0.001	0	0.03	3.7
J	53.64	47.2	0.03	1.7
I	49.09	54.8	0.03	2.5
K	44.09	53.4	0.03	2.1
L	48.69	51.7	0.03	1.5
M	9.72	44.2	0.03	1.6
P1	0.001	0	0.03	3.2
O	125.36	54.5	0.03	1.3
P2	16.17	35.4	0.03	3.2
Q	30.15	69.6	0.025	0.8
S1	0.001	0	0.03	1.1
R	37.91	52.6	0.03	1.5
Dummy_S2	0.001	0	0.03	1.1
S2	77.13	58.3	0.03	1.1
T	22.88	51	0.03	1.7
U	4.04	27.1	0.035	1.1
H	14.15	48.9	0.03	3.7
N	44.76	26.6	0.035	1.6
Dummy_S1	0.001	0	0.03	0.8
Dummy_P2	0.001	0	0.03	1.3
Dummy_P1	0.001	0	0.03	1.6

Appendix C – Land-use Mapping

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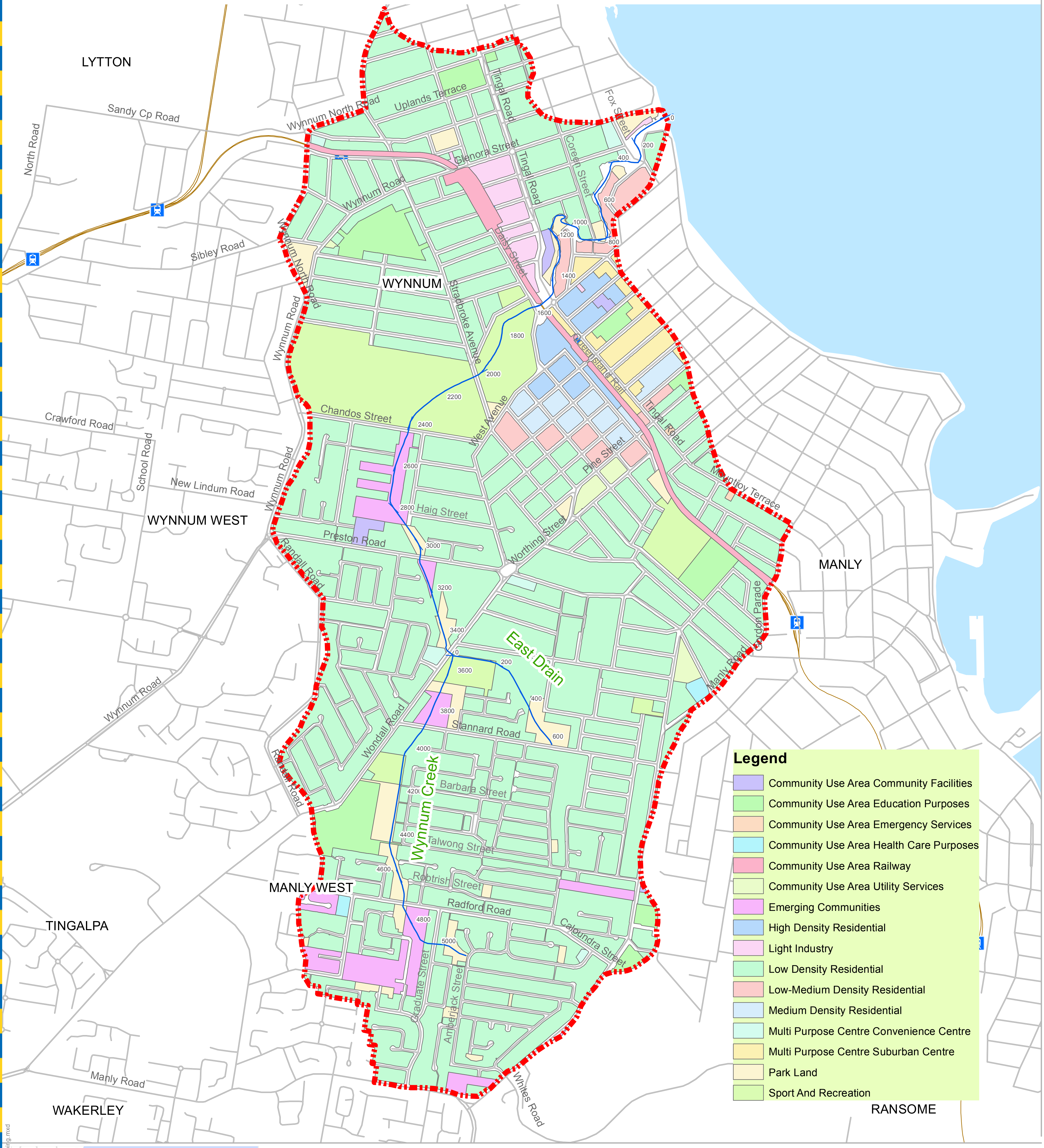

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0 200 400 600
Metres

N W E S

Legend

Wynnum Catchment

Wynnum Creek Centreline

Streets

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Appendix C: Land Use Mapping

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0 200 400 600
Metres

N W E S

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Legend

Wynnum Catchment

Wynnum Creek Centreline

Streets

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Appendix C: Wynnum Creek Catchment Aerial Image

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Appendix D – Design Event Peak Flood Levels

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Wynnum Creek

AMTD (m)	Cross Section ID	**Design Event - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)**					
		2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
0		0.96	0.96	0.96	0.97	0.97	0.97
66		1.00	1.03	1.05	1.11	1.19	1.28
100		1.03	1.07	1.10	1.17	1.27	1.36
166		1.10	1.16	1.23	1.33	1.48	1.62
200		1.11	1.20	1.26	1.34	1.50	1.64
239		1.14	1.28	1.34	1.43	1.62	1.76
285		1.57	1.81	1.95	2.13	2.28	2.40
Fox Street							
295		1.59	1.84	1.98	2.16	2.34	2.49
300		1.57	1.81	1.96	2.14	2.32	2.49
327		1.68	1.99	2.17	2.39	2.62	2.82
398		1.81	2.13	2.32	2.56	2.80	3.01
400		1.83	2.15	2.33	2.58	2.81	3.02
453		2.02	2.31	2.48	2.71	2.94	3.14
500		2.22	2.49	2.65	2.86	3.08	3.27
543		2.46	2.74	2.90	3.12	3.35	3.54
595		2.90	3.17	3.34	3.57	3.79	3.99
600		2.92	3.19	3.36	3.58	3.81	4.01
652		3.10	3.37	3.54	3.78	4.03	4.24
700		3.41	3.72	3.91	4.17	4.43	4.65
702		3.41	3.72	3.92	4.18	4.44	4.66
774		3.57	3.89	4.09	4.36	4.64	4.87
800		3.59	3.91	4.11	4.38	4.65	4.89
848		3.60	3.90	4.11	4.37	4.64	4.87
Coreen Street							
900		3.87	4.20	4.42	4.70	4.98	5.22
932		3.91	4.24	4.46	4.74	5.02	5.26
1000		4.10	4.44	4.66	4.95	5.24	5.48
1100		4.24	4.58	4.80	5.09	5.38	5.62
1104		4.26	4.59	4.82	5.10	5.39	5.64
1200		4.67	4.99	5.21	5.49	5.76	5.98
1212		4.69	5.02	5.23	5.52	5.80	6.02
1214		4.69	5.02	5.24	5.53	5.81	6.03
1260		4.99	5.33	5.57	5.88	6.20	6.44
1300		5.27	5.61	5.85	6.16	6.48	6.73
1308		5.30	5.65	5.89	6.20	6.52	6.77
1400		5.58	5.92	6.16	6.46	6.77	7.02
1405		5.59	5.93	6.16	6.47	6.78	7.03
1470		5.73	6.12	6.37	6.67	6.97	7.22
1499		5.90	6.31	6.53	6.79	7.07	7.30
Tingal Road							
1512		6.05	6.38	6.57	6.82	7.09	7.32
1563		6.18	6.45	6.62	6.84	7.10	7.34
1575		6.19	6.45	6.62	6.84	7.11	7.40

Wynnum Creek Flood Study 2014

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)					
		2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
1599		6.24	6.50	6.66	6.87	7.13	7.42
1600	XS1600	6.25	6.51	6.68	6.89	7.14	7.42
Daisy Street							
1628	XS1628	6.26	6.52	6.68	6.89	7.14	7.43
1671	XS1671	6.26	6.52	6.68	6.90	7.14	7.43
1700		6.28	6.53	6.69	6.91	7.15	7.44
1745	XS1745	6.30	6.55	6.71	6.92	7.17	7.45
1785	XS1785	6.33	6.59	6.74	6.95	7.20	7.48
1800		6.35	6.61	6.76	6.97	7.21	7.49
1846	XS1846	6.43	6.68	6.83	7.03	7.28	7.54
1886	XS1886	6.54	6.78	6.92	7.11	7.37	7.59
1900		6.61	6.82	6.96	7.13	7.40	7.62
1960	XS1960	6.93	7.05	7.13	7.22	7.51	7.74
2000		7.07	7.18	7.25	7.33	7.57	7.79
2020	XS2020	7.15	7.26	7.32	7.39	7.61	7.81
Stradbroke Avenue							
2038	XS2038	8.05	8.21	8.26	8.32	8.39	8.49
2063	XS2063	8.12	8.28	8.33	8.39	8.47	8.56
2091	XS2091	8.19	8.33	8.38	8.44	8.52	8.61
2100		8.21	8.34	8.39	8.45	8.53	8.62
2183	XS2183	8.37	8.49	8.54	8.61	8.68	8.76
2200		8.41	8.53	8.58	8.65	8.72	8.80
2263	XS2263	8.54	8.66	8.73	8.80	8.89	8.96
2300		8.60	8.72	8.79	8.87	8.96	9.03
2380	XS2380	8.72	8.85	8.91	9.00	9.10	9.19
2400		8.75	8.88	8.94	9.03	9.14	9.23
2422	XS2422	8.78	8.91	8.98	9.07	9.18	9.27
Chandos Street							
2445	XS2445	9.11	9.41	9.50	9.62	9.75	9.86
2484	XS2484	9.19	9.50	9.65	9.81	9.97	10.10
2500		9.28	9.58	9.71	9.87	10.03	10.16
2539	XS2539	9.53	9.79	9.90	10.03	10.19	10.32
2561	XS2561	9.60	9.84	9.95	10.08	10.24	10.36
2600		9.67	9.89	10.00	10.13	10.28	10.41
2616	XS2616	9.69	9.91	10.01	10.15	10.30	10.43
2700		9.84	10.03	10.13	10.26	10.41	10.54
2702	XS2702	9.84	10.04	10.13	10.26	10.41	10.54
2782	XS2782	9.98	10.16	10.26	10.38	10.53	10.66
2800		10.01	10.18	10.28	10.40	10.55	10.68
2845	XS2845	10.05	10.23	10.32	10.45	10.60	10.73
2872	XS2872	10.08	10.25	10.35	10.47	10.63	10.76
Private Crossing (Haig Street)							
2882	XS2882	10.23	10.38	10.47	10.58	10.73	10.86
2900		10.32	10.47	10.55	10.67	10.81	10.94
2925	XS2925	10.34	10.49	10.58	10.69	10.84	10.97
2993	XS2993	10.42	10.58	10.66	10.78	10.93	11.06

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)					
		2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
3000		10.43	10.59	10.67	10.79	10.94	11.07
3047	XS3047	10.50	10.67	10.76	10.88	11.02	11.16
Preston Road							
3073	XS3073	11.18	11.62	11.67	11.74	11.84	11.93
3100	XS3100	11.22	11.75	11.85	12.00	12.19	12.36
3150	XS3150	11.29	11.79	11.89	12.05	12.23	12.40
3200		11.37	11.83	11.94	12.10	12.28	12.45
3208	XS3208	11.39	11.84	11.95	12.10	12.29	12.46
3246	XS3246	11.48	11.91	12.02	12.17	12.35	12.51
3295	XS3295	11.62	12.02	12.14	12.29	12.46	12.61
3300		11.63	12.04	12.15	12.30	12.47	12.62
3361	XS3361	11.81	12.16	12.27	12.42	12.58	12.73
3400		11.91	12.24	12.35	12.49	12.65	12.80
3444	XS3444	12.03	12.32	12.43	12.57	12.73	12.89
3495	XS3495	12.16	12.43	12.54	12.69	12.85	13.01
3500		12.17	12.44	12.55	12.70	12.87	13.02
3510	XS3510	12.18	12.45	12.56	12.72	12.88	13.03
Wondall Road							
3532	XS3532	12.79	12.97	13.04	13.14	13.25	13.37
3548	XS3548	12.78	12.96	13.03	13.14	13.26	13.38
3550	XS3550	12.77	12.96	13.03	13.14	13.26	13.38
3587	XS3587	12.89	13.07	13.14	13.25	13.37	13.49
Wynnum Manly Leagues Club Crossing							
3605	XS3605	13.01	13.13	13.19	13.29	13.41	13.52
3638	XS3638	13.12	13.20	13.24	13.33	13.44	13.56
Wynnum Manly Leagues Club Pedestrian Crossing							
3643	XS3643	13.13	13.21	13.25	13.33	13.45	13.56
3670	XS3670	13.30	13.38	13.42	13.49	13.59	13.68
3700		13.39	13.48	13.52	13.59	13.68	13.78
3755	XS3756	13.51	13.61	13.65	13.72	13.82	13.92
3800		13.60	13.71	13.74	13.82	13.93	14.03
3830	XS3830	13.66	13.77	13.80	13.88	14.00	14.11
3875	XS3877	13.73	13.85	13.89	13.97	14.09	14.20
Stannard Road							
3894	XS3894	14.24	14.62	14.79	14.90	15.00	15.08
3900		14.24	14.62	14.79	14.91	15.02	15.11
3959	XS3959	14.30	14.66	14.82	14.95	15.06	15.15
4000		14.38	14.70	14.85	14.98	15.09	15.17
4030	XS4030	14.43	14.74	14.88	15.00	15.11	15.19
4100		14.64	14.88	14.99	15.10	15.21	15.30
4116	XS4116	14.69	14.91	15.02	15.13	15.24	15.33
4175	XS4175	14.98	15.17	15.25	15.33	15.40	15.48
Barbara Street Footbridge							
4180	XS4180	15.00	15.19	15.27	15.35	15.42	15.49
4200		15.07	15.25	15.35	15.42	15.49	15.56
4234	XS4234	15.19	15.37	15.46	15.53	15.59	15.66

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)					
		2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
4288	XS4288	15.32	15.50	15.58	15.65	15.71	15.77
4300		15.34	15.52	15.60	15.68	15.73	15.80
4308	XS4308	15.36	15.54	15.62	15.70	15.75	15.82
4348	XS4348	15.47	15.65	15.73	15.81	15.87	15.95
4400		15.68	15.83	15.90	15.98	16.04	16.14
4423	XS4423	15.77	15.91	15.98	16.05	16.12	16.23
4440	XS4440	15.86	16.00	16.06	16.13	16.20	16.31
Talwong Street Footbridge							
4446	XS4446	15.89	16.03	16.09	16.16	16.23	16.34
4482	XS4482	15.98	16.13	16.19	16.26	16.33	16.44
4500		16.04	16.18	16.24	16.31	16.38	16.50
4525	XS4525	16.11	16.26	16.32	16.39	16.46	16.57
4578	XS4578	16.27	16.41	16.48	16.55	16.62	16.73
4600		16.33	16.47	16.53	16.61	16.67	16.79
4631	XS4631	16.40	16.54	16.61	16.68	16.75	16.86
Robtrish Street Footbridge							
4639	XS4639	16.41	16.56	16.62	16.70	16.77	16.88
4700		16.51	16.67	16.74	16.82	16.89	17.02
4715	XS4715	16.53	16.70	16.77	16.85	16.92	17.05
Radford Road							
4736	XS4736	16.64	16.90	17.03	17.19	17.39	17.52
4780	XS4780	16.78	17.03	17.15	17.30	17.49	17.62
4800		16.94	17.17	17.28	17.42	17.59	17.73
4841	XS4841	17.38	17.55	17.62	17.72	17.88	18.02
4885	XS4885	17.69	17.84	17.91	17.99	18.15	18.28
4900		17.75	17.90	17.97	18.05	18.20	18.34
4921	XS4922	17.79	17.95	18.01	18.10	18.25	18.39
Graduate Street							
5034		18.53	19.05	19.29	19.54	19.70	19.82
5100		18.56	19.06	19.30	19.54	19.70	19.82
5117		18.57	19.06	19.30	19.54	19.70	19.82

East Tributary

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)					
		2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
0		12.78	12.96	13.04	13.15	13.26	13.38
10	EXS12	12.79	12.98	13.05	13.16	13.28	13.40
12	EXS14	12.89	13.07	13.14	13.25	13.37	13.49
48	EXS50	12.95	13.13	13.20	13.31	13.43	13.55
100		12.97	13.15	13.22	13.33	13.45	13.56
122		12.98	13.17	13.23	13.34	13.45	13.57
168	EXS170	13.04	13.22	13.30	13.39	13.50	13.60

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)					
		2-yr ARI (50% AEP)	5-yr ARI (20% AEP)	10-yr ARI (10% AEP)	20-yr ARI (5% AEP)	50-yr ARI (2% AEP)	100-yr ARI (1% AEP)
200		13.13	13.33	13.41	13.51	13.61	13.70
216	EXS218	13.18	13.38	13.47	13.58	13.67	13.76
300		13.92	14.06	14.14	14.24	14.33	14.40
313	EXS314	14.06	14.20	14.27	14.36	14.45	14.53
369	EXS371	14.74	14.87	14.94	15.02	15.10	15.17
400		15.00	15.13	15.19	15.28	15.36	15.43
420	EXS421	15.16	15.29	15.36	15.44	15.52	15.59
479	EXS481	15.49	15.63	15.70	15.79	15.87	15.95
500		15.60	15.74	15.81	15.91	16.00	16.08
540	EXS541	15.81	15.96	16.04	16.14	16.23	16.31
600	EXS601	16.14	16.29	16.36	16.46	16.55	16.64

Appendix E – Extreme Event Peak Flood Levels

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Wynnum Creek

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
0		0.97	1.77	1.77
66		1.28	1.68	1.74
100		1.36	1.74	1.83
166		1.62	2.09	2.24
200		1.64	2.11	2.27
239		1.76	2.18	2.33
285		2.40	2.61	2.72
Fox Street				
295		2.49	2.78	2.97
300		2.49	2.82	3.06
327		2.82	3.14	3.37
398		3.01	3.32	3.53
400		3.02	3.34	3.54
453		3.14	3.43	3.64
500		3.27	3.54	3.74
543		3.54	3.80	4.01
595		3.99	4.25	4.48
600		4.01	4.27	4.50
652		4.24	4.50	4.75
700		4.65	4.91	5.18
702		4.66	4.91	5.19
774		4.87	5.17	5.45
800		4.89	5.18	5.46
848		4.87	5.16	5.44
Coreen Street				
900		5.22	5.52	5.80
932		5.26	5.56	5.84
1000		5.48	5.77	6.06
1100		5.62	5.92	6.23
1104		5.64	5.94	6.24
1200		5.98	6.25	6.58
1212		6.02	6.30	6.66
1214		6.03	6.31	6.67
1260		6.44	6.74	7.09
1300		6.73	7.03	7.36
1308		6.77	7.07	7.40
1400		7.02	7.32	7.66
1405		7.03	7.33	7.67
1470		7.22	7.52	7.83
1499		7.30	7.59	7.91
Tingal Road				
1512		7.32	7.61	7.92

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
1563		7.34	7.64	7.88
1575		7.40	7.74	8.01
1599		7.42	7.76	8.03
1600	XS1600	7.42	7.76	8.04
Daisy Street				
1628	XS1628	7.43	7.77	8.04
1671	XS1671	7.43	7.77	8.04
1700		7.44	7.77	8.04
1745	XS1745	7.45	7.78	8.05
1785	XS1785	7.48	7.79	8.06
1800		7.49	7.80	8.07
1846	XS1846	7.54	7.83	8.09
1886	XS1886	7.59	7.87	8.12
1900		7.62	7.89	8.14
1960	XS1960	7.74	7.98	8.21
2000		7.79	8.00	8.23
2020	XS2020	7.81	8.02	8.25
Stradbroke Avenue				
2038	XS2038	8.49	8.58	8.69
2063	XS2063	8.56	8.63	8.73
2091	XS2091	8.61	8.67	8.78
2100		8.62	8.69	8.79
2183	XS2183	8.76	8.83	8.93
2200		8.80	8.87	8.97
2263	XS2263	8.96	9.02	9.12
2300		9.03	9.10	9.21
2380	XS2380	9.19	9.27	9.39
2400		9.23	9.30	9.43
2422	XS2422	9.27	9.35	9.48
Chandos Street				
2445	XS2445	9.85	9.95	10.09
2484	XS2484	10.10	10.21	10.38
2500		10.16	10.27	10.44
2539	XS2539	10.32	10.43	10.61
2561	XS2561.	10.36	10.48	10.66
2600		10.41	10.53	10.72
2616	XS2616	10.43	10.55	10.74
2700		10.54	10.66	10.85
2702	XS2702	10.54	10.66	10.86
2782	XS2782	10.66	10.78	10.98
2800		10.68	10.80	11.00
2845	XS2845	10.73	10.86	11.06
2872	XS2872	10.76	10.88	11.09
Private Crossing (Haig Street)				
2882	XS2882	10.86	10.98	11.18

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
2900		10.94	11.07	11.27
2925	XS2925	10.97	11.10	11.30
2993	XS2993	11.06	11.19	11.39
3000		11.07	11.20	11.40
3047	XS3047	11.16	11.28	11.48
Preston Road				
3073	XS3073	11.91	12.02	12.17
3100	XS3100	12.36	12.48	12.73
3150	XS3150	12.40	12.54	12.78
3200		12.45	12.58	12.83
3208	XS3208	12.46	12.59	12.84
3246	XS3246	12.51	12.65	12.89
3295	XS3295	12.61	12.74	12.97
3300		12.62	12.75	12.98
3361	XS3361	12.73	12.86	13.08
3400		12.80	12.93	13.16
3444	XS3444	12.89	13.02	13.26
3495	XS3495	13.01	13.15	13.39
3500		13.02	13.16	13.40
3510	XS3510	13.03	13.17	13.42
Wondall Road				
3532	XS3532	13.37	13.47	13.66
3548	XS3548	13.38	13.49	13.70
3550	XS3550	13.38	13.49	13.70
3587	XS3587	13.49	13.61	13.82
Wynnum Manly Leagues Club Crossing				
3605	XS3605	13.52	13.64	13.85
3638	XS3638	13.56	13.67	13.89
Wynnum Manly Leagues Club Pedestrian Crossing				
3643	XS3643	13.56	13.68	13.89
3670	XS3670	13.68	13.78	13.97
3700		13.78	13.87	14.05
3755	XS3756	13.92	14.02	14.20
3800		14.03	14.13	14.32
3830	XS3830	14.11	14.21	14.40
3875	XS3877	14.20	14.31	14.49
Stannard Road				
3894	XS3894	15.07	15.14	15.23
3900		15.11	15.18	15.29
3959	XS3959	15.15	15.22	15.34
4000		15.17	15.25	15.37
4030	XS4030	15.19	15.27	15.39
4100		15.30	15.38	15.50
4116	XS4116	15.33	15.41	15.53
4175	XS4175	15.48	15.54	15.65

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
Barbara Street Footbridge				
4180	XS4180	15.49	15.56	15.66
4200		15.56	15.63	15.74
4234	XS4234	15.66	15.72	15.82
4288	XS4288	15.77	15.83	15.93
4300		15.80	15.85	15.95
4308	XS4308	15.82	15.87	15.97
4348	XS4348	15.95	16.01	16.11
4400		16.14	16.23	16.36
4423	XS4423	16.23	16.33	16.48
4440	XS4440	16.31	16.42	16.58
Talwong Street Footbridge				
4446	XS4446	16.34	16.45	16.61
4482	XS4482	16.44	16.55	16.72
4500		16.50	16.60	16.78
4525	XS4525	16.57	16.68	16.85
4578	XS4578	16.73	16.84	17.01
4600		16.79	16.90	17.07
4631	XS4631	16.86	16.97	17.14
Robtrish Street Footbridge				
4639	XS4639	16.88	16.99	17.17
4700		17.02	17.13	17.31
4715	XS4715	17.05	17.16	17.35
Radford Road				
4736	XS4736	17.52	17.63	17.76
4780	XS4780	17.62	17.73	17.88
4800		17.73	17.84	17.98
4841	XS4841	18.02	18.13	18.24
4885	XS4885	18.28	18.40	18.55
4900		18.34	18.46	18.62
4921	XS4922	18.39	18.51	18.67
Graduate Street				
5034		19.82	19.92	20.07
5100		19.82	19.92	20.07
5117		19.82	19.92	20.07

East Tributary

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
0		13.38	13.49	13.70

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	200-yr ARI (0.5% AEP)	500-yr ARI (0.2% AEP)
10	EXS12	13.40	13.51	13.71
12	EXS14	13.49	13.60	13.81
48	EXS50	13.55	13.66	13.87
100		13.56	13.67	13.88
122		13.57	13.68	13.89
168	EXS170	13.60	13.71	13.91
200		13.70	13.79	13.97
216	EXS218	13.76	13.84	14.00
300		14.40	14.47	14.57
313	EXS314	14.53	14.60	14.69
369	EXS371	15.17	15.22	15.29
400		15.43	15.49	15.57
420	EXS421	15.59	15.66	15.75
479	EXS481	15.95	16.03	16.14
500		16.08	16.15	16.27
540	EXS541	16.31	16.40	16.52
600	EXS601	16.64	16.72	16.86

Appendix F – Climate Change Peak Water Levels

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Wynnum Creek

AMTD (m)	Cross Section ID	**Design Event - Scenario 3 Ultimate Case (m AHD)**	**Climate Change - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)**	
		100-yr ARI (1% AEP)	100-yr ARI (1% AEP) Climate Change 2050	100-yr ARI (1% AEP) Climate Change 2100
0		0.97	1.47	2.24
66		1.28	1.47	1.90
100		1.36	1.55	1.97
166		1.62	1.95	2.30
200		1.64	1.98	2.32
239		1.76	2.08	2.38
285		2.40	2.58	2.74
Fox Street				
295		2.49	2.75	3.00
300		2.49	2.79	3.09
327		2.82	3.13	3.42
398		3.01	3.31	3.59
400		3.02	3.32	3.60
453		3.14	3.42	3.68
500		3.27	3.53	3.77
543		3.54	3.80	4.02
595		3.99	4.25	4.46
600		4.01	4.26	4.47
652		4.24	4.50	4.71
700		4.65	4.92	5.13
702		4.66	4.92	5.13
774		4.87	5.15	5.37
800		4.89	5.17	5.39
848		4.87	5.15	5.37
Coreen Street				
900		5.22	5.50	5.71
932		5.26	5.54	5.75
1000		5.48	5.76	5.98
1100		5.62	5.91	6.15
1104		5.64	5.92	6.16
1200		5.98	6.25	6.48
1212		6.02	6.29	6.53
1214		6.03	6.30	6.54
1260		6.44	6.73	7.02
1300		6.73	7.03	7.32
1308		6.77	7.06	7.36
1400		7.02	7.32	7.62
1405		7.03	7.33	7.63
1470		7.22	7.50	7.82
1499		7.30	7.58	7.89
Tingal Road				
1512		7.32	7.59	7.89

AMTD (m)	Cross Section ID	Design Event - Scenario 3 Ultimate Case (m AHD)	Climate Change - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)	
		100-yr ARI (1% AEP)	100-yr ARI (1% AEP) Climate Change 2050	100-yr ARI (1% AEP) Climate Change 2100
1563		7.34	7.62	7.84
1575		7.40	7.71	7.99
1599		7.42	7.74	8.02
1600	XS1600	7.42	7.74	8.02
Daisy Street				
1628	XS1628	7.43	7.75	8.03
1671	XS1671	7.43	7.75	8.03
1700		7.44	7.75	8.03
1745	XS1745	7.45	7.76	8.04
1785	XS1785	7.48	7.77	8.05
1800		7.49	7.78	8.05
1846	XS1846	7.54	7.81	8.07
1886	XS1886	7.59	7.85	8.10
1900		7.62	7.87	8.11
1960	XS1960	7.74	7.96	8.18
2000		7.79	7.99	8.19
2020	XS2020	7.81	8.01	8.20
Stradbroke Avenue				
2038	XS2038	8.49	8.57	8.65
2063	XS2063	8.56	8.62	8.69
2091	XS2091	8.61	8.67	8.73
2100		8.62	8.68	8.75
2183	XS2183	8.76	8.82	8.88
2200		8.80	8.86	8.92
2263	XS2263	8.96	9.02	9.08
2300		9.03	9.10	9.16
2380	XS2380	9.19	9.26	9.34
2400		9.23	9.30	9.38
2422	XS2422	9.27	9.34	9.42
Chandos Street				
2445	XS2445	9.85	9.94	10.03
2484	XS2484	10.10	10.21	10.32
2500		10.16	10.27	10.38
2539	XS2539	10.32	10.43	10.54
2561	XS2561	10.36	10.48	10.59
2600		10.41	10.53	10.64
2616	XS2616	10.43	10.55	10.66
2700		10.54	10.66	10.78
2702	XS2702	10.54	10.66	10.78
2782	XS2782	10.66	10.78	10.90
2800		10.68	10.80	10.92
2845	XS2845	10.73	10.85	10.98
2872	XS2872	10.76	10.88	11.00
Private Crossing (Haig Street)				
2882	XS2882	10.86	10.97	11.10

AMTD (m)	Cross Section ID	**Design Event - Scenario 3 Ultimate Case (m AHD)**	**Climate Change - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)**	
		100-yr ARI (1% AEP)	100-yr ARI (1% AEP) Climate Change 2050	100-yr ARI (1% AEP) Climate Change 2100
2900		10.94	11.06	11.18
2925	XS2925	10.97	11.09	11.22
2993	XS2993	11.06	11.18	11.30
3000		11.07	11.19	11.31
3047	XS3047	11.16	11.27	11.40
Preston Road				
3073	XS3073	11.91	11.98	12.06
3100	XS3100	12.36	12.51	12.67
3150	XS3150	12.40	12.56	12.72
3200		12.45	12.61	12.77
3208	XS3208	12.46	12.61	12.78
3246	XS3246	12.51	12.66	12.82
3295	XS3295	12.61	12.75	12.90
3300		12.62	12.76	12.91
3361	XS3361	12.73	12.87	13.01
3400		12.80	12.94	13.09
3444	XS3444	12.89	13.02	13.17
3495	XS3495	13.01	13.15	13.30
3500		13.02	13.16	13.31
3510	XS3510	13.03	13.17	13.32
Wondall Road				
3532	XS3532	13.37	13.46	13.57
3548	XS3548	13.38	13.49	13.60
3550	XS3550	13.38	13.49	13.60
3587	XS3587	13.49	13.60	13.72
Wynnum Manly Leagues Club Crossing				
3605	XS3605	13.52	13.63	13.75
3638	XS3638	13.56	13.67	13.78
Wynnum Manly Leagues Club Pedestrian Crossing				
3643	XS3643	13.56	13.67	13.79
3670	XS3670	13.68	13.77	13.87
3700		13.78	13.86	13.96
3755	XS3756	13.92	14.01	14.11
3800		14.03	14.13	14.23
3830	XS3830	14.11	14.20	14.31
3875	XS3877	14.20	14.30	14.40
Stannard Road				
3894	XS3894	15.07	15.13	15.18
3900		15.11	15.18	15.24
3959	XS3959	15.15	15.22	15.28
4000		15.17	15.25	15.31
4030	XS4030	15.19	15.27	15.33
4100		15.30	15.37	15.44
4116	XS4116	15.33	15.40	15.46
4175	XS4175	15.48	15.54	15.59

AMTD (m)	Cross Section ID	**Design Event - Scenario 3 Ultimate Case (m AHD)**	**Climate Change - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)**	
		100-yr ARI (1% AEP)	100-yr ARI (1% AEP) Climate Change 2050	100-yr ARI (1% AEP) Climate Change 2100
Barbara Street Footbridge				
4180	XS4180	15.49	15.55	15.61
4200		15.56	15.62	15.68
4234	XS4234	15.66	15.71	15.76
4288	XS4288.	15.77	15.82	15.87
4300		15.80	15.85	15.90
4308	XS4308	15.82	15.87	15.92
4348	XS4348	15.95	16.01	16.06
4400		16.14	16.22	16.30
4423	XS4423	16.23	16.32	16.40
4440	XS4440	16.31	16.40	16.50
Talwong Street Footbridge				
4446	XS4446	16.34	16.44	16.53
4482	XS4482	16.44	16.54	16.63
4500		16.50	16.59	16.69
4525	XS4525	16.57	16.67	16.76
4578	XS4578	16.73	16.83	16.92
4600		16.79	16.89	16.98
4631	XS4631	16.86	16.96	17.06
Robtrish Street Footbridge				
4639	XS4639	16.88	16.98	17.08
4700		17.02	17.12	17.22
4715	XS4715	17.05	17.15	17.25
Radford Road				
4736	XS4736	17.52	17.62	17.84
4780	XS4780	17.62	17.72	17.79
4800		17.73	17.83	17.91
4841	XS4841	18.02	18.12	18.21
4885	XS4885	18.28	18.39	18.49
4900		18.34	18.45	18.55
4921	XS4922	18.39	18.50	18.60
Graduate Street				
5034		19.82	19.92	20.01
5100		19.82	19.92	20.01
5117		19.82	19.92	20.00

East Tributary

AMTD (m)	Cross Section ID	**Design Event - Scenario 3 Ultimate Case (m AHD)**	**Extreme Events - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)**	
		100-yr ARI (1% AEP)	100-yr ARI (1% AEP) Climate Change 2050	100-yr ARI (1% AEP) Climate Change 2100
0		13.38	13.49	13.60

AMTD (m)	Cross Section ID	**Design Event - Scenario 3 Ultimate Case (m AHD)**	**Climate Change - Scenario 3 Ultimate Case - Peak Water Levels (m AHD)**	
		100-yr ARI (1% AEP)	100-yr ARI (1% AEP) Climate Change 2050	100-yr ARI (1% AEP) Climate Change 2100
10	EXS12	13.40	13.50	13.62
12	EXS14	13.49	13.60	13.71
48	EXS50	13.55	13.65	13.77
100		13.56	13.67	13.78
122		13.57	13.67	13.79
168	EXS170	13.60	13.70	13.81
200		13.70	13.79	13.88
216	EXS218	13.76	13.83	13.91
300		14.40	14.46	14.53
313	EXS314	14.53	14.59	14.65
369	EXS371	15.17	15.22	15.26
400		15.43	15.48	15.53
420	EXS421	15.59	15.65	15.70
479	EXS481	15.95	16.02	16.08
500		16.08	16.14	16.21
540	EXS541	16.31	16.39	16.46
600	EXS601	16.64	16.71	16.79

Appendix G – Hydraulic Structure Reference Sheet

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Creek:	Wynnum Creek	**Immunity Rating:**	10-yr ARI
Location:	Graduate St		10 % AEP

DATE OF SURVEY: N/A	UBD REF: 163 E11
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID: N/A
MODEL ID: ID1	AMTD (m): 4978

STRUCTURE DESCRIPTION: Pipe Culvert

STRUCTURE SIZE: Entrance 2/1650; Exit 2/1800

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m)	16.41m AHD	U/S OBVERT LEVEL (m)	18.06m AHD
D/S INVERT LEVEL (m)	15.62m AHD	D/S OBVERT LEVEL (m)	17.42m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 89m

LENGTH OF CULVERT AT OBVERT (m): 89m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 89m PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~19.2m AHD (~18.7m AHD at Graduate Street)

HEIGHT OF GUARDRAIL/HANDRAIL: N/A

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Underside RL = ~17.4m Top RL = 18.7m Steel Handrail

PLAN NUMBER: W5065

BRIDGE OR CULVERT DETAILS:

The pipe diameter changes from 1.65m at the entrance of the culvert to 1.8m at the exit. The 1.8m diameter pipe incorporates the local catchment flows coming through Graduate Street.

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1976

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Lowest point of weir taken from Graduate Street.

Creek:	Wynnum Creek
Location:	Graduate St

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	44.0	20.00	18.49	1510	190	0.8	0.9	4.1
500-yr (0.2%)	33.6	19.86	18.28	1580	170	0.7	1.0	4.2
100-yr (0.1%)	24.3	19.68	18.07	1610	150	0.5	0.8	4.1
50-yr (0.2%)	21.0	19.58	17.98	1600	140	0.4	0.7	4.1
20-yr (5%)	17.7	19.43	17.92	1510	120	0.2	0.5	4.0
10-yr (10%)	15.9	19.19	17.86	1330	-	-	-	3.7
5-yr (20%)	14.9	18.98	17.81	1170	-	-	-	3.5
2-yr (50%)	12.2	18.47	17.69	780	-	-	-	2.9

* value can vary

Creek:	Wynnum Creek
Location:	Graduate St

Graduate Street culvert looking downstream

Graduate Street culvert looking upstream

Creek:	Wynnum Creek		**Immunity Rating:**	20-yr ARI
Location:	Radford Rd			5% AEP

DATE OF SURVEY: N/A

UBD REF: 163 D10

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID: C0696P

MODEL ID: ID2

AMTD (m): 4725

STRUCTURE DESCRIPTION: Pipe Culvert

STRUCTURE SIZE: 2/525 and 3/1800 RCP

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 16.06 and 14.64m AHD

U/S OBVERT LEVEL (m) 16.56 and 16.44m AHD

D/S INVERT LEVEL (m) 16.0 and 14.58m AHD

D/S OBVERT LEVEL (m) 16.53 and 16.38m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 21m

LENGTH OF CULVERT AT OBVERT (m): 21m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 21m

PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~17.2m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.3m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Handrail

PLAN NUMBER: W5065

BRIDGE OR CULVERT DETAILS:

Two different sets of culvert sizes with different invert levels. At the time of inspection, one of the 3/1800mm culvert was half blocked. No blockage applied to the model.

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1973

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Radford Rd

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	61.4	17.82	17.38	440	250	0.6	2.1	3.9
500-yr (0.2%)	44.4	17.65	17.15	500	230	0.4	1.4	3.9
100-yr (0.1%)	29.8	17.45	16.94	510	170	0.3	0.6	3.8
50-yr (0.2%)	28.8	17.35	16.88	470	130	0.2	0.1	3.6
20-yr (5%)	27.0	17.22	16.83	390	-	-	-	3.4
10-yr (10%)	24.7	17.05	16.75	300	-	-	-	3.1
5-yr (20%)	22.8	16.92	16.68	240	-	-	-	2.9
2-yr (50%)	18.5	16.66	16.53	130	-	-	-	2.4

* value can vary

Creek:	Wynnum Creek
Location:	Radford Rd

Radford Road culvert looking downstream

Radford Road culver looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	50-yr ARI
Location:	Robtrish St Footbridge		2% AEP

DATE OF SURVEY: N/A	UBD REF: 163 D10
SURVEYED CROSS SECTION ID: W970	BCC ASSET ID: B6018
MODEL ID: ID3	AMTD (m): 4635

STRUCTURE DESCRIPTION: Footbridge

STRUCTURE SIZE: 1 Span Bridge - 16.8m

For Culverts: Number of cells/pipes & sizes   For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m)	14.87m AHD	U/S OBVERT LEVEL (m)	~16.9m AHD
D/S INVERT LEVEL (m)	14.73m AHD	D/S OBVERT LEVEL (m)	~16.9m AHD

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2.6m   PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~16.7m AHD (~16.9m AHD at Structure)

HEIGHT OF GUARDRAIL/HANDRAIL: 1.25m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS:

PLAN NUMBER: W5924

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1988

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Robtrish St Footbridge

ARI (AEP %)	**DISCHARGE (m3/s)***	**U/S Water Level***	**D/S Water Level***	**AFFLUX (mm)**	**FLOW WIDTH ABOVE STRUCTURE (m)***	**FLOW DEPTH ABOVE STRUCTURE (m)***	**VELOCITY (m/s)***	
		(m AHD)					**Weir**	**Structure**
2000-yr (0.05%)	67.8	17.22	17.19	30	110	0.5	0.4	2.7
500-yr (0.2%)	49.4	16.99	16.97	20	60	0.3	0.5	2.3
100-yr (0.1%)	36.2	16.77	16.75	20	40	0.1	0.5	1.9
50-yr (0.2%)	33.4	16.72	16.70	20	-	-	-	1.9
20-yr (5%)	31.0	16.67	16.65	20	-	-	-	1.8
10-yr (10%)	28.1	16.60	16.58	20	-	-	-	1.7
5-yr (20%)	25.8	16.54	16.52	20	-	-	-	1.7
2-yr (50%)	20.8	16.40	16.38	20	-	-	-	1.6

* value can vary

Creek:	Wynnum Creek
Location:	Robtrish St Footbridge

Robtrish Road footbridge looking downtream

Robtrish Road footbridge looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	100-yr ARI
Location:	Talwong St Footbridge		1% AEP

DATE OF SURVEY: N/A

UBD REF: 163 D9

SURVEYED CROSS SECTION ID: W920

BCC ASSET ID: B1971

MODEL ID: ID4

AMTD (m): 4443

STRUCTURE DESCRIPTION: Footbridge

STRUCTURE SIZE: 1 Span Bridge - 16.8m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 14.48m AHD U/S OBVERT LEVEL (m) ~16.7m AHD

D/S INVERT LEVEL (m) 14.48m AHD D/S OBVERT LEVEL (m) ~16.7m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2.6m PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~16.2m AHD (~16.6 m AHD at Structure)

HEIGHT OF GUARDRAIL/HANDRAIL: 1.25m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Handrail

PLAN NUMBER: W5924

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1990

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Talwong St Footbridge

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	65.7	16.48	16.43	50	70	0.3	0.4	2.8
500-yr (0.2%)	48.5	16.26	16.22	40	50	0.1	-	2.4
100-yr (0.1%)	35.6	16.05	16.02	30	-	-	-	2.1
50-yr (0.2%)	33.1	16.00	15.97	30	-	-	-	2.0
20-yr (5%)	30.9	15.97	15.93	40	-	-	-	1.9
10-yr (10%)	28.0	15.91	15.87	40	-	-	-	1.8
5-yr (20%)	25.8	15.86	15.83	30	-	-	-	1.8
2-yr (50%)	20.8	15.75	15.72	30	-	-	-	1.6

* value can vary

Creek:	Wynnum Creek
Location:	Talwong St Footbridge


Talwong Street footbridge looking downstream

Talwong Street footbridge looking upstream

Creek:	Wynnum Creek
Location:	Barbara St Footbridge

Immunity Rating:	10-yr ARI
	10 % AEP

DATE OF SURVEY: N/A	UBD REF: 163 D8
SURVEYED CROSS SECTION ID: W880	BCC ASSET ID: B1970
MODEL ID: ID5	AMTD (m): 4178

STRUCTURE DESCRIPTION: Footbridge

STRUCTURE SIZE: 1 Span Bridge - 16.8m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 13.46m AHD U/S OBVERT LEVEL (m) ~15.5m AHD

D/S INVERT LEVEL (m) 13.46m AHD D/S OBVERT LEVEL (m) ~15.5m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2.6m PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~15.2m AHD (~15.4m AHD at Structure)

HEIGHT OF GUARDRAIL/HANDRAIL: 1.25m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Handrail

PLAN NUMBER: W5924

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1988

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Barbara St Footbridge

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*^	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	84.3	15.69	15.68	10	160	0.5	0.8	3.4
500-yr (0.2%)	68.2	15.54	15.54	0	140	0.3	0.8	3.2
100-yr (0.1%)	52.6	15.39	15.38	10	110	0.2	-	3.0
50-yr (0.2%)	47.8	15.34	15.33	10	80	0.1	-	2.9
20-yr (5%)	43.5	15.27	15.26	10	60	0.1	-	2.8
10-yr (10%)	38.9	15.18	15.16	20	-	-	-	2.7
5-yr (20%)	35.3	15.10	15.08	20	-	-	-	2.6
2-yr (50%)	27.9	14.93	14.91	20	-	-	-	2.3

* value can vary

^flow breaks through lower sections of the floodplain (values represetative of the floodplain)

Creek:	Wynnum Creek
Location:	Barbara St Footbridge

Barbara Street footbridge looking downstream

Barbara Street footbridge looking upstream

Creek:	Wynnum Creek		**Immunity Rating:**	5-yr ARI
Location:	Stannard Rd			20 % AEP

DATE OF SURVEY: N/A	UBD REF: 163 E7
SURVEYED CROSS SECTION ID: W830	BCC ASSET ID: C0186B
MODEL ID: ID6	AMTD (m): 3886

STRUCTURE DESCRIPTION: Box Culvert

STRUCTURE SIZE: 3/1800x1800

For Culverts: Number of cells/pipes & sizes   For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m)	11.62m AHD	U/S OBVERT LEVEL (m)	13.42m AHD
D/S INVERT LEVEL (m)	11.53m AHD	D/S OBVERT LEVEL (m)	13.33m AHD

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 17m

LENGTH OF CULVERT AT OBVERT (m): 17m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 17m   PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 14.5m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: 1.5m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS:
Underside RL = ~14.8m
Top RL = ~16.3m Steel Handrail

PLAN NUMBER: W5279

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1974

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Stannard Rd

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	104.0	15.32	14.47	850	130	0.8	3.1	7.2
500-yr (0.2%)	75.0	15.19	14.21	980	120	0.7	3.0	7.1
100-yr (0.1%)	55.0	15.05	14.00	1050	110	0.6	1.7	6.8
50-yr (0.2%)	49.2	15.00	13.93	1070	100	0.5	1.3	6.7
20-yr (5%)	43.1	14.92	13.85	1070	80	0.4	1.1	6.6
10-yr (10%)	37.3	14.77	13.76	1010	60	0.3	0.3	6.4
5-yr (20%)	35.1	14.54	13.72	820	-	-	-	6.0
2-yr (50%)	30.5	14.14	13.63	510	-	-	-	4.2

* value can vary

Creek:	Wynnum Creek
Location:	Stannard Rd

Stannard Road culvert looking downstream

Stannard Road culvert looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	<2-yr ARI
Location:	Leagues Club Access Road		<50% AEP

DATE OF SURVEY: N/A	UBD REF: 163 D9
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID: N/A
MODEL ID: ID7	AMTD (m): 3596

STRUCTURE DESCRIPTION: Pipe Culvert

STRUCTURE SIZE: 2/1200 and 1/750 RCP

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 10.98m AHD U/S OBVERT LEVEL (m) 11.73 and 12.18m AHD

D/S INVERT LEVEL (m) 10.97m AHD D/S OBVERT LEVEL (m) 11.72 and 12.17m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 10m

LENGTH OF CULVERT AT OBVERT (m): 10m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 10m PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 12.5m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.0m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Handrail

PLAN NUMBER: N/A

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: -

HAS THE STRUCTURE BEEN UPGRADED? -

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Leagues Club Access Road

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	104.8	13.56	13.53	30	270	1.1	1.1	3.7
500-yr (0.2%)	78.8	13.40	13.37	30	240	0.9	1.1	3.6
100-yr (0.1%)	59.9	13.24	13.21	30	210	0.7	1.1	3.6
50-yr (0.2%)	54.0	13.19	13.15	40	200	0.7	1.1	3.7
20-yr (5%)	47.2	13.12	13.07	50	190	0.6	1.1	3.7
10-yr (10%)	40.7	13.06	13.01	50	170	0.6	1.1	3.6
5-yr (20%)	38.1	13.03	12.96	70	160	0.5	1.1	3.7
2-yr (50%)	32.4	12.96	12.83	130	130	0.5	1.1	3.7

* value can vary

Creek:	Wynnum Creek
Location:	Leagues Club Access Road

Wynnum Mainly League Club Access Road Looking Downstream

Wynnum Mainly League Club Access Road Looking Upstream

Creek:	Wynnum Creek	**Immunity Rating:**	<2-yr ARI
Location:	Wondall Rd		<50 % AEP

DATE OF SURVEY: N/A

UBD REF: 163 E6

SURVEYED CROSS SECTION ID: W730

BCC ASSET ID: C0056P

MODEL ID: ID8

AMTD (m): 3521

STRUCTURE DESCRIPTION: Pipe Culvert

STRUCTURE SIZE: 4/1800 RCP

For Culverts: Number of cells/pipes & sizes

For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 9.59m AHD

U/S OBVERT LEVEL (m) 11.39m AHD

D/S INVERT LEVEL (m) 9.4m AHD

D/S OBVERT LEVEL (m) 11.2m AHD

For culverts give floor level

For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 19m

LENGTH OF CULVERT AT OBVERT (m): 19m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 19m

PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 12.5m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: 1.0m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS:

Underside RL = ~12.7m
Top RL = ~13.7m
U/S - Mesh Steel Fence, D/S Guardrail

PLAN NUMBER: W5279

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: -

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Wondall Rd

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	170.1	13.47	13.19	280	360	1.0	4.6	3.8
500-yr (0.2%)	122.2	13.30	12.87	430	330	0.8	3.9	3.9
100-yr (0.1%)	89.7	13.14	12.65	490	320	0.6	3.5	3.8
50-yr (0.2%)	79.9	13.08	12.56	520	310	0.6	3.4	3.8
20-yr (5%)	68.9	13.00	12.44	560	300	0.5	3.1	3.8
10-yr (10%)	60.8	12.93	12.33	600	290	0.4	2.6	3.8
5-yr (20%)	55.7	12.88	12.26	620	280	0.4	2.3	3.8
2-yr (50%)	44.2	12.73	12.08	650	270	0.2	1.8	3.7

* value can vary

Creek:	Wynnum Creek
Location:	Wondall Rd

Wondall Road culvert looking downstream


Wondall Road culvert looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	2-yr ARI
Location:	Preston Rd		50 % AEP

DATE OF SURVEY: N/A

UBD REF: 163 E4

SURVEYED CROSS SECTION ID: W620

BCC ASSET ID: B1610

MODEL ID: ID9

AMTD (m): 3060

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: 1 Span Bridge - ~7.2m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 8.12m AHD U/S OBVERT LEVEL (m) 10.59m AHD

D/S INVERT LEVEL (m) 7.84m AHD D/S OBVERT LEVEL (m) 10.59m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 18m PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 11.22m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: 1.3m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Underside RL = ~12.3m Top RL = ~13.6m Steel Handrail

PLAN NUMBER: W2324

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1960

HAS THE STRUCTURE BEEN UPGRADED? Yes W8184 Footpath bridge widening in 1988

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Preston Rd

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	184.9	12.19	11.43	760	180	1.0	2.7	4.3
500-yr (0.2%)	130.5	11.82	11.09	730	170	0.6	2.4	4.5
100-yr (0.1%)	95.5	11.73	10.84	890	140	0.5	2.0	4.4
50-yr (0.2%)	85.1	11.69	10.75	940	130	0.5	1.8	4.4
20-yr (5%)	74.7	11.63	10.64	990	110	0.4	1.6	4.3
10-yr (10%)	62.2	11.52	10.54	980	80	0.3	1.2	4.3
5-yr (20%)	55.8	11.39	10.50	890	30	0.2	0.7	4.2
2-yr (50%)	46.7	11.09	10.10	990	-	-	-	3.8

* value can vary

Creek:	Wynnum Creek
Location:	Preston Rd

Preston Road bridge looking downstream

Preston Road bridge looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	<2-yr ARI
Location:	Haig St Private Property		<50 % AEP

DATE OF SURVEY: N/A	UBD REF: 163 D3
SURVEYED CROSS SECTION ID: W570	BCC ASSET ID: N/A
MODEL ID: ID10	AMTD (m): 2877

STRUCTURE DESCRIPTION: Pipe Culvert

STRUCTURE SIZE: 3/1800m RCP

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) ~7.11m AHD U/S OBVERT LEVEL (m) ~8.91m AHD

D/S INVERT LEVEL (m) ~7.06m AHD D/S OBVERT LEVEL (m) ~8.86m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 5m

LENGTH OF CULVERT AT OBVERT (m): 5m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 5m PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~9.4m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: N/A

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: N/A

PLAN NUMBER: N/A

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: -

HAS THE STRUCTURE BEEN UPGRADED? - -

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

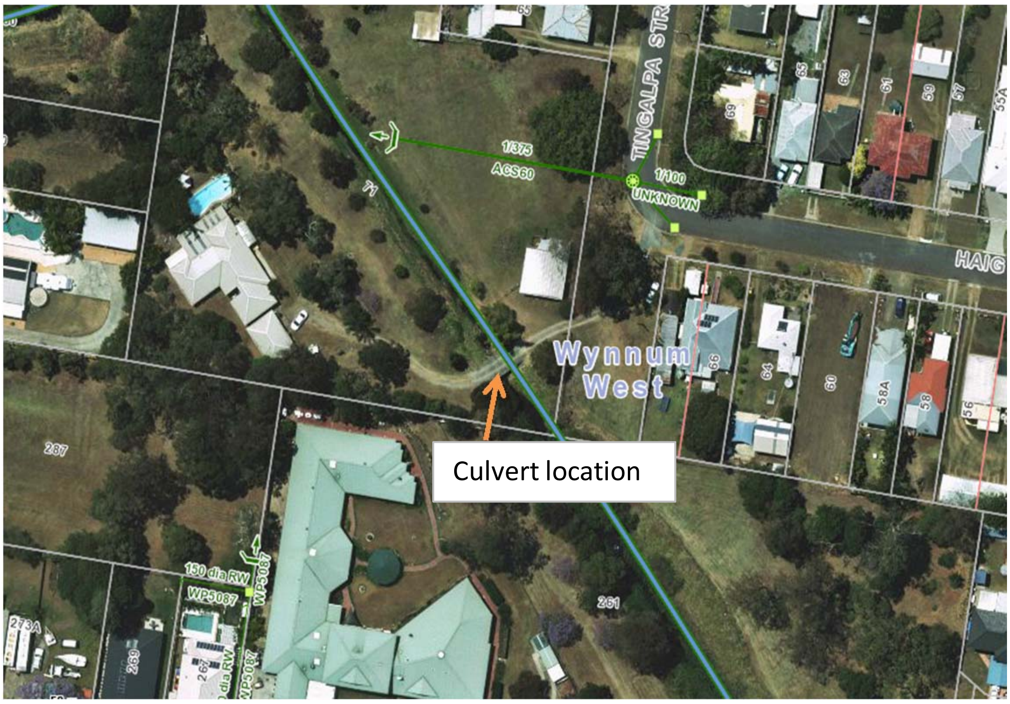
Culvert information obtained from old hydraulic model

Creek:	Wynnum Creek
Location:	Haig St Private Property

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	172.6	11.11	11.05	60	100	1.7	3.0	3.3
500-yr (0.2%)	122.2	10.79	10.69	100	100	1.4	2.9	3.5
100-yr (0.1%)	91.3	10.53	10.43	100	90	1.1	2.5	3.5
50-yr (0.2%)	82.1	10.44	10.34	100	90	1.0	2.4	3.5
20-yr (5%)	71.7	10.34	10.22	120	80	0.9	2.2	3.5
10-yr (10%)	63.5	10.25	10.12	130	80	0.9	2.1	3.5
5-yr (20%)	60.2	10.21	10.07	140	80	0.8	2.0	3.5
2-yr (50%)	50.3	10.09	9.91	180	70	0.7	1.7	3.4

* value can vary

Creek:	Wynnum Creek
Location:	Haig St Private Property



Haig Street private property culvert

Haig Street private property culvert looking downstream

Creek:	Wynnum Creek
Location:	Chandos St

Immunity Rating:	2-yr ARI
	50 % AEP

DATE OF SURVEY: N/A	UBD REF: 162 D2
SURVEYED CROSS SECTION ID: W490	BCC ASSET ID: C0128P
MODEL ID: ID11	AMTD (m): 2434

STRUCTURE DESCRIPTION: Pipe Culvert

STRUCTURE SIZE: 6/1800 RCP

For Culverts: Number of cells/pipes & sizes — For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 6.45m AHD — U/S OBVERT LEVEL (m) 8.25m AHD

D/S INVERT LEVEL (m) 6.42m AHD — D/S OBVERT LEVEL (m) 8.22m AHD

For culverts give floor level — For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 13m

LENGTH OF CULVERT AT OBVERT (m): 13m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 13m — PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~9.0m AHD (~9.3m AHD at Structure)

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.3m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Timber Handrail

PLAN NUMBER: W1469

BRIDGE OR CULVERT DETAILS:

Woody debris found blocking part of the entrace of the culvert during inspection.

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: -

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Chandos St

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	186.4	9.92	9.36	560	260	0.9	3.6	4.6
500-yr (0.2%)	127.2	9.78	9.14	640	250	0.8	2.3	4.4
100-yr (0.1%)	92.8	9.61	8.99	620	230	0.6	1.8	4.2
50-yr (0.2%)	82.4	9.55	8.94	610	230	0.6	1.6	4.1
20-yr (5%)	70.4	9.46	8.87	590	220	0.5	1.3	4.0
10-yr (10%)	62.9	9.37	8.83	540	120	0.4	1.2	3.8
5-yr (20%)	58.4	9.29	8.79	500	100	0.3	1.0	3.7
2-yr (50%)	48.9	9.04	8.71	330	-	-	-	3.2

* value can vary

Creek:	Wynnum Creek
Location:	Chandos St

Chandos Street culvert looking downstream

Chandos Street culvert looking upstream

Creek:	Wynnum Creek		**Immunity Rating:**	<2-yr ARI
Location:	Stradbroke Ave			<50 % AEP

DATE OF SURVEY: N/A

UBD REF: 163 F1

SURVEYED CROSS SECTION ID: W380

BCC ASSET ID: B1940

MODEL ID: ID12

AMTD (m): 2029

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: 1 Span Bridge - 9.6m

For Culverts: Number of cells/pipes & sizes

For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 5.62m AHD

U/S OBVERT LEVEL (m) 7.08m AHD

D/S INVERT LEVEL (m) 5.62m AHD

D/S OBVERT LEVEL (m) 7.08m AHD

For culverts give floor level

For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 13m

PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 7.9m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.0m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Top RL = 8.5m
Steel Handrail

PLAN NUMBER: W2384

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1963

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Stradbroke Ave

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	219.4	8.65	8.26	390	360	0.8	2.6	4.0
500-yr (0.2%)	142.7	8.44	7.57	870	320	0.5	1.9	4.0
100-yr (0.1%)	104.2	8.34	7.44	900	280	0.4	1.7	4.0
50-yr (0.2%)	91.8	8.30	7.37	930	260	0.4	1.6	4.0
20-yr (5%)	80.2	8.25	7.30	950	240	0.4	1.5	4.0
10-yr (10%)	73.0	8.22	7.26	960	230	0.3	1.4	4.0
5-yr (20%)	66.0	8.18	7.22	960	220	0.3	1.3	4.0
2-yr (50%)	52.2	8.05	7.15	900	170	0.2	0.8	4.0

* value can vary

Creek:	Wynnum Creek
Location:	Stradbroke Ave

Stradbroke Avenue looking downstream

Stradbroke Avenue looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	<2-yr ARI
Location:	Daisy St		<50 % AEP

DATE OF SURVEY: N/A

UBD REF: 143 G20

SURVEYED CROSS SECTION ID: W290

BCC ASSET ID: C0104B

MODEL ID: ID13

AMTD (m): 1614

STRUCTURE DESCRIPTION: Box Culvert

STRUCTURE SIZE: 2/2.75x2.45

For Culverts: Number of cells/pipes & sizes

For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) 2.75m AHD

U/S OBVERT LEVEL (m) 5.27m AHD

D/S INVERT LEVEL (m) 2.7m AHD

D/S OBVERT LEVEL (m) 5.22m AHD

For culverts give floor level

For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): 11m

LENGTH OF CULVERT AT OBVERT (m): 11m

TYPE OF LINING: Precast Concrete

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 11m

PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 5.64m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.3m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Handrail

PLAN NUMBER: W4662

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1971

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Daisy St

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	194.2	8.10	8.10	0	440	2.5	4.0	1.8
500-yr (0.2%)	138.6	7.14	7.14	0	300	1.5	3.9	1.9
100-yr (0.1%)	95.6	6.74	6.74	0	120	1.1	3.4	1.9
50-yr (0.2%)	85.7	6.65	6.65	0	110	1.0	3.2	1.9
20-yr (5%)	74.0	6.52	6.51	10	110	0.9	3.0	1.9
10-yr (10%)	66.8	6.40	6.40	0	100	0.8	2.9	1.9
5-yr (20%)	60.8	6.29	6.29	0	100	0.7	2.9	1.9
2-yr (50%)	49.8	6.04	6.04	0	90	0.4	2.9	2.0

* value can vary

Creek:	Wynnum Creek
Location:	Daisy St

Daisy Street culvert looking downstream

Daisy Street culvert looking upstream

Creek:	Wynnum Creek		**Immunity Rating:**	>100-yr ARI
Location:	QLD Rail			>1% AEP

DATE OF SURVEY: N/A	UBD REF: 143 G20
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID: W0042 W0043
MODEL ID: ID14	AMTD (m): 1569

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: 2 Span Bridge - 24.5m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) ~2.7m AHD U/S OBVERT LEVEL (m) 6.91m AHD

D/S INVERT LEVEL (m) ~2.7m AHD D/S OBVERT LEVEL (m) 6.91m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 12m PIER WIDTH (m): ~2.0m

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~7.91m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: ~2.0m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Concrete

PLAN NUMBER: S28725

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 2003

HAS THE STRUCTURE BEEN UPGRADED? Yes Upgraded from a 12 opening timber bridge in 2003

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	QLD Rail

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	235.2	8.04	7.75	290	330	0.1	1.25	4.3
500-yr (0.2%)	165.2	7.04	7.01	30	-	-	-	3.0
100-yr (0.1%)	113.4	6.64	6.64	0	-	-	-	3.0
50-yr (0.2%)	103.1	6.55	6.55	0	-	-	-	3.0
20-yr (5%)	89.5	6.43	6.42	10	-	-	-	3.0
10-yr (10%)	80.5	6.31	6.30	10	-	-	-	3.0
5-yr (20%)	71.8	6.19	6.17	20	-	-	-	3.0
2-yr (50%)	59.6	5.80	5.78	20	-	-	-	3.0

* value can vary

Creek:	Wynnum Creek
Location:	QLD Rail

Queensland rail bridge looking downstream

Queensland Rail bridge looking upstream

Creek:	Wynnum Creek		**Immunity Rating:**	2-yr ARI
Location:	Tingal Rd			50 % AEP

DATE OF SURVEY: N/A	UBD REF: 143 G19
SURVEYED CROSS SECTION ID: N/A	BCC ASSET ID: B2030
MODEL ID: ID15	AMTD (m): 1506

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: 2 Span Bridge - ~4.6m

For Culverts: Number of cells/pipes & sizes   For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) ~2.2m AHD   U/S OBVERT LEVEL (m) 5.14m AHD

D/S INVERT LEVEL (m) ~2.1m AHD   D/S OBVERT LEVEL (m) 5.06m AHD

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 13m   PIER WIDTH (m): 0.36 to 0.71m

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): 5.9m AHD

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.07m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Underside RL = 6.1m Top RL = 7.2m Steel Handrail

PLAN NUMBER: A-A-62

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1960

HAS THE STRUCTURE BEEN UPGRADED? Yes L-5-31

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Tingal Rd

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	239.5	7.68	7.56	120	320	1.78	2.5	3.3
500-yr (0.2%)	164.7	6.97	6.9	70	170	1.07	2.6	2.9
100-yr (0.1%)	113.5	6.56	6.42	140	140	0.66	2.4	2.4
50-yr (0.2%)	103.2	6.44	6.27	170	130	0.54	2.3	2.7
20-yr (5%)	89.7	6.26	6.05	210	110	0.36	2.1	2.7
10-yr (10%)	80.4	6.11	5.87	240	100	0.21	1.8	2.7
5-yr (20%)	72.2	5.96	5.68	280	90	0.06	1.3	2.7
2-yr (50%)	59.7	5.52	5.46	60	-	-	-	2.7

* value can vary

Creek:	Wynnum Creek
Location:	Tingal Rd

Tingal Road looking downstream

Tingal Road looking upstream

Creek:	Wynnum Creek	**Immunity Rating:**	<2-yr ARI
Location:	Coreen St Footbridge		<50 % AEP

DATE OF SURVEY: N/A

UBD REF: 143 H18

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID: N/A

MODEL ID: ID16

AMTD (m): 874

STRUCTURE DESCRIPTION: Footbridge

STRUCTURE SIZE: 2 Span Bridge

For Culverts: Number of cells/pipes & sizes   For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) -0.1m AHD   U/S OBVERT LEVEL (m) ~2.4m AHD (min)

D/S INVERT LEVEL (m) -0.1m AHD   D/S OBVERT LEVEL (m) ~2.4m AHD (min)

For culverts give floor level   For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 2m   PIER WIDTH (m): N/A

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~2.6m AHD (~2.8m AHD at Structure)

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.1m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Steel Handrail

PLAN NUMBER: W4344

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1970

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Coreen St Footbridge

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*^	FLOW DEPTH ABOVE STRUCTURE (m)*	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	276.3	5.57	5.21	360	100	2.97	2.4	4.0
500-yr (0.2%)	200.8	4.93	4.57	360	90	2.33	1.9	3.7
100-yr (0.1%)	140.5	4.37	4.02	350	80	1.77	1.6	3.1
50-yr (0.2%)	128.4	4.22	3.86	360	80	1.62	1.5	2.9
20-yr (5%)	112.5	4.04	3.68	360	70	1.44	1.4	2.8
10-yr (10%)	100.3	3.86	3.51	350	70	1.26	1.2	2.7
5-yr (20%)	91.7	3.74	3.39	350	60	1.14	1.1	2.7
2-yr (50%)	77.6	3.51	3.19	320	50	0.91	0.9	2.5

* value can vary

^flow breaks through lower sections of the floodplain (values represetative of the floodplain)

Creek:	Wynnum Creek
Location:	Coreen St Footbridge

Coreen Street footbridge looking from right hand bank from upstream side

Coreen Street footbridge looking upstream

Creek:	Wynnum Creek
Location:	Fox Street

Immunity Rating:	20-yr ARI
	5 % AEP

DATE OF SURVEY: N/A

UBD REF: 143 H17

SURVEYED CROSS SECTION ID: N/A

BCC ASSET ID: B0790

MODEL ID: ID17

AMTD (m): 290

STRUCTURE DESCRIPTION: Bridge

STRUCTURE SIZE: 2 Span Bridge - ~11.0m

For Culverts: Number of cells/pipes & sizes For Bridges: Number of Spans and their lengths

U/S INVERT LEVEL (m) -1.3m AHD U/S OBVERT LEVEL (m) ~2.2m AHD

D/S INVERT LEVEL (m) -1.6m AHD D/S OBVERT LEVEL (m) ~2.2m AHD

For culverts give floor level For bridges give bed level

For culverts:

LENGTH OF CULVERT AT INVERT (m): N/A

LENGTH OF CULVERT AT OBVERT (m): N/A

TYPE OF LINING:

(e.g. concrete, stone, brick, corrugated iron)

IS THERE A SURVEYED WEIR PROFILE? No survey conducted for this study

If yes give details i.e plan number and/or survey book number. Note: this section should be at the highest part of the road eg. Crown, kerb, hand rails whichever is higher

WEIR WIDTH (m): 12m PIER WIDTH (m): ~0.7m

In direction of flow, i.e distance from u/s face to d/s face

LOWEST POINT OF WEIR (m AHD): ~2.1m AHD (2.6m AHD at Low Point of Structure)

HEIGHT OF GUARDRAIL/HANDRAIL: ~1.10m

DESCRIPTION OF HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERISDE OF GUARD RAILS: Underside RL = 6.1m Top RL = 7.2m Steel Handrail

PLAN NUMBER: W602

BRIDGE OR CULVERT DETAILS:

Wingwall/Headwall details e.g Pipe flusk with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge inclucing abutment details. Specific survey book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE: 1955

HAS THE STRUCTURE BEEN UPGRADED? No

If, yes, explain type and date of upgrade. Include plan number and location if applicable.

ADDITIONAL COMMENTS:

Creek:	Wynnum Creek
Location:	Fox Street

ARI (AEP %)	DISCHARGE (m3/s)*	U/S Water Level*'	D/S Water Level*	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)*^	FLOW DEPTH ABOVE STRUCTURE (m)*`	VELOCITY (m/s)*	
		(m AHD)					Weir	Structure
2000-yr (0.05%)	277.8	2.97	2.74	230	420	0.87	3.0	3.4
500-yr (0.2%)	183.8	2.52	2.39	130	300	0.42	2.2	3.2
100-yr (0.1%)	140.6	2.11	2.04	70	90	0.01	1.2	3.2
50-yr (0.2%)	127.3	1.99	1.93	60	80	-	0.9	3.1
20-yr (5%)	112.8	1.86	1.81	50	-	-	-	3.0
10-yr (10%)	100.2	1.72	1.68	40	-	-	-	2.8
5-yr (20%)	91.0	1.61	1.57	40	-	-	-	2.7
2-yr (50%)	77.0	1.46	1.42	40	-	-	-	2.4

* value can vary

^flow breaks through lower sections of the floodplain (values represetative of the floodplain)

`water level through breakthrough area of floodplain is higher

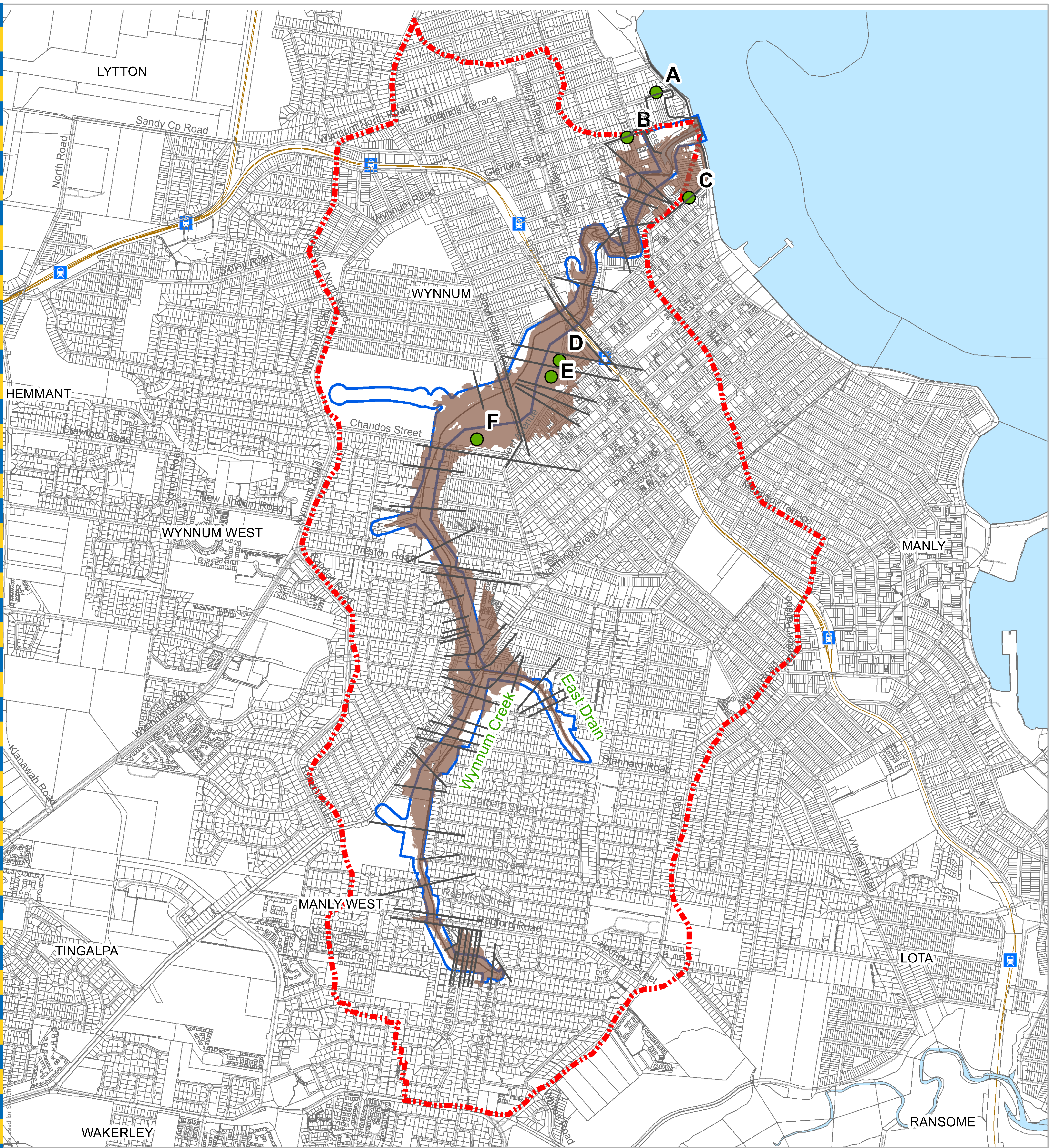
Creek:	Wynnum Creek
Location:	Fox Street

Fox Street looking Downstream

Fox Street Looking Upstream

Appendix H – Break Lines and Limitations

Limitation ID	Limitation Type	Locations Description	Additional Comments
A	Assume no overtopping of road	Alkoomie Street	Upstream water level breaks out and travels downstream. Extrapolated water levels restrict to road centre line to prevent unrealistinc inundation along coastline
B	Assume no overtopping of road	Glenora Street	
C	Assume no overtopping of road	Agnes Street	
D	Area allowed to infill from upstream break line for up to 100-yr ARI (1% AEP)	Kitchener Park near Colina Street	Area allowed to infill from expected overtopping upstream, however flood surface levels may be exaggerated
E	Area allowed to infill from upstream break line for up to 100-yr ARI (1% AEP)	Kitchener Park near Colina Street	
F	5-yr ARI (20% AEP) and 10-yr ARI (10% AEP) allowed to infill area as overtopping upstream of Chandos Street would occur naturally	Chandos Street	



0 200 400 600

Metres

N W E S

Prepared :	077900
Checked :	SG
Revision :	0
Publication Date :	17 Jul 2014
Project Number :	130115

Legend

- WaterRide Break Lines
- WaterRide Limitation Areas
- 1% AEP Ultimate Case Scenario Flood Inundation Extent
- Wynnum Catchment
- Wynnum Creek Centreline
- Wynnum Creek Waterway Corridor
- Waterway/Waterbody

DATA INFORMATION

The flood maps must be read in conjunction with the flood study report and interpretedby a qualified professional engineer. The flood maps are based on the best data available to Brisbane City Council ("Council") at the time the maps were developed. Council, and the copyright owners listed below, give no warranty in relation to the data (including accuracy, reliability, completeness, currency or suitability) presented in these maps and the user uses and relies upon the data in the maps at its own sole risk and liability. Council is not liable for errors or omissions in the flood maps. To the full extent that it is able to do so in law, the Council disclaims all liability (including without limitation, liability in negligence) for any loss, damage or costs (including indirect and consequential loss and damage), caused by or arising from anyone using or relying on the data contained in the flood maps for any purpose whatsoever.

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For more information
visit www.brisbane.qld.gov.au
or call (07) 3403 8888

BRISBANE CITY

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Wynnum Creek Flood Study 2014

Appendix H: WaterRide Break Lines and Limitations

FLM - 130115

Appendix I - External Peer Review Documentation

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Our Ref: L.B20679.001.Wynnum_Creek.docx

15 July 2014

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505 St Pauls Terrace
Fortitude Valley
Qld 4006

Attention: Erico Saito

Dear Erico

RE: WYNNUM CREEK FLOOD MODELLING PEER REVIEW

Background

BMT WBM was commissioned by Council to undertake a peer review of the Wynnum Creek flood modelling prepared as part of the Wynnum Creek Flood Study. This letter documents the outcomes of BMT WBM's review.

At the commencement of the review process, Council submitted the following data to BMT WBM:

- Hydrological models
- Hydraulic models including all model output files (the models were not rerun by BMT WBM)
- HEC-RAS verification models
- GIS data
- Site photographs
- Initial reporting
- Other background calculations

These data were reviewed and initial feedback provided to Council by email (dated 20th February 2014). BMT WBM's recommendations were subsequently adopted before finalisation of the calibration and design event modelling.

Overview of the Modelling Approach

Hydrological models were developed using the XP-RAFTS software. These models were based on an existing model developed in a previous study. Significant revision was made by Council to the existing model to ensure that it represents current catchment conditions.

Hydraulic models of Wynnum Creek were developed using the TUFLOW software with a 2m computational grid cell size for design events up to 500 year ARI and 4m for the 2000 year ARI and PMF events. The upper and middle reaches of the creek were modelled in 1D, and linked to a 2D model domain of the floodplain. This is a typical modelling methodology that is used when the width of the creek channel is small relative to the size of the 2D computational grid cells.

The lower reach of the creek was modelled within the 2D model domain. It is noted that the creek width is relatively narrow for this approach, which was adopted to mitigate instability issues that can arise across the 1D-2D links under high flow conditions. For this lower reach of Wynnum Creek, Council undertook a comprehensive assessment to represent the bathymetry as best as possible within the limitations of the 2D model resolution. The approach was further validated by comparison of the model results with that of an equivalent, alternative 1D modelling package, HEC-RAS, and the original model cell size was reduced from 4m to 2m to improve the resolution of the topography. As such, BMT WBM accepts that the adopted approach is suitable for the purposes of the study.

An existing MIKE11 model, developed for a previous study, was used to inform the channel cross-section data in the TUFLOW model. The floodplain topography was based on a 2009 Aerial Laser Survey (ALS).

Model Performance

The model performance has been checked in relation to: mass balance error, negative depth warnings, and instability. Only minor instability and negative depth warnings occur, and the mass balance errors are within the normal acceptable range. As such, the model performance is considered satisfactory.

It is noted that Council has also assessed the model performance in relation to replication of historical events (calibration and verification) and bridge structures have been compared to equivalent HEC-RAS models. Council’s acceptable tolerance for calibration is 0.15m variance for peak flood levels at stream gauges and 0.3m variance for peak flood levels at maximum height gauges. This correlates with standard industry practice.

Limitations of the Review

This review was undertaken at a relatively high level, and focussed on scrutinising model results and other performance indicators to assess the performance of the model. It was not possible within the budget constraints of the review to undertake a comprehensive assessment of all the information used to develop the model and BMT WBM has relied upon information and data supplied by Council. For example, the accuracy of the topographic data, land use mapping and structure details has not been explicitly checked. If supplied information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions may change.

The review is limited to the modelling component of the study, and does not consider the broader flood study methodology adopted by Council.

Conclusion

The flood modelling undertaken as part of the Wynnum Creek Flood Study complies with current industry practice, and is considered suitable for the purposes of the study.

Yours Faithfully
BMT WBM

Richard Sharpe
Senior Flood Engineer

Jo Tinnion RPEQ (11395)

Supervising Engineer[1]:

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[1] *Supervising engineer signoff is based on information provided by Richard Sharpe and confidence in Richard's ability to undertake the review. Trust has been placed in the validity and completeness of the information provided by Richard.*