Hemmant-Lytton Flood Study
Volume 1 of 2

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Hemmant-Lytton Flood Study

Volume 1 of 2

Prepared by BMT WBM Pty Ltd
Prepared for Brisbane City Council

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Dedicated to a better Brisbane

BMT WBM

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Executive Summary

Catchment Overview

The Hemmant-Lytton Flood Study covers an investigation area of 18km² and includes the suburbs of Hemmant, Lytton and Wynnum West. Hemmant Drain and Lindum Creek are the primary waterways within the catchment. The Port of Brisbane Motorway and the Cleveland Railway line extend across the full width.

The catchment is highly urbanised, with most of the rainfall runoff being directed though a stormwater network before reaching the creeks. There are three distinct changes in land use throughout the study area. Low density residential zoning covers much of the upper catchment whilst green space, parkland and rural zoning cover much of the middle of the catchment. The lower regions of the catchment are heavily dominated by industry.

Aim of Study

The aim of this flood investigation was to determine flood levels for a range of design flood events and extreme events, along with the provision of flood inundation and depth x velocity mapping. The completed investigation will serve as the provision of flooding information to assist in the setting of Council planning policy and floodplain management.

A range of data including previous flood investigations, topographic survey data, hydrometric data and Maximum Height Gauge (MHG) data were used to undertake the investigation.

Hydrologic Model

A XP-RAFTS hydrologic model was used to simulate the rainfall-runoff process within the catchment. Previous hydrologic models were reviewed, combined, extended and updated to ensure they were fit for purpose for this investigation. The hydrologic model calibration was undertaken jointly with the TUFLOW hydraulic model. Two recent flood events were selected for the calibration; January 2013 and October 2010, with the December 2010 flood event chosen to validate the flood model.

Hydraulic Model

A TUFLOW hydraulic model was developed to route the runoff which was computed using the hydrologic model through the catchment. For this investigation, pre-existing hydraulic models were available. However, the Hemmant-Lytton flood model was created from scratch with many of the existing building blocks from the previous hydraulic models being incorporated into the new hydraulic model.

Hydraulic Model Calibration and Verification

Two recent flood events were selected for the calibration; January 2013 and October 2010. The model was also validated with the December 2010 flood event.

The January 2013 flood event was the largest of the three historical events. Some inconsistencies in the recorded levels were identified between gauges during the calibration process. It is likely that an anomaly occurred during the recording of peak level in particular at Gauge MHG 210. As a result of this, a substantial discrepancy between recorded and modelled flood level is reported for the flood event.

A good calibration was achieved to the October 2010 event, with the modelled peak water levels within a tolerance of +/-0.3m from the recorded levels at each MHG gauge.

The December 2010 flood event was used to verify the hydraulic model. For this event, the model under predicted the recorded levels at two gauges, which may be due to some blockage of hydraulic structures during the event not captured by the model. At Tingalpa Road under prediction may be due to a blockage of the structure.

The consistency of flow predictions in the hydrologic and hydraulic models was checked. Good consistency was only achieved in the upper Hemmant catchment. This was expected, as the complexity of the floodplain storage and conveyance through much of the catchment is beyond the predictive capability of the simple routing techniques within the hydrologic model; hence the need to develop a 2D hydraulic model.

Design Event Modelling

The hydraulic model was used to determine both discharges and flood levels for the 50%, 20%, 10%, 5%, 2% and 1% year AEP events. These events were simulated for durations from 30 minutes to 24 hours. The following design event scenarios were simulated in the hydrologic and hydraulic models:

Scenario 1: Existing Waterway Conditions

Scenario 2: Minimum Riparian Corridor (MRC)

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

AR&R was used to develop the design storms for the design events. One central location was used in the Hemmant-Lytton catchment to derive IFD data for the design storms. The hydrology model's percentage impervious and Manning's n values were updated to represent ultimate catchment conditions for all scenarios.

The calibrated hydraulic model was used as the basis for the design event modelling. The model was updated as follows:

- Scenario 1: No updates were made to the calibration model as it represented existing catchment conditions.
- Scenario 2: An additional MRC materials layer was added to the model. This layer covered 15m on either side of waterways and a Manning's n of 0.15 was used within the MRC.
- Scenario 3: The Scenario 2 model was updated by including a terrain modifier that filled all areas outside the waterway corridor.

The downstream boundary water level was set to a static Mean High Water Spring for design events up to 1% AEP.

Extreme events (0.5%, 0.2%, 0.05% AEP and Probable Maximum Flood (PMF) flood events) were simulated for Scenario 1. CRC Forge was used to determine the rainfall depths for the extreme events. The AR&R temporal patterns were adopted for the 0.5% and 0.2% AEP events. Council's 'superstorm' methodology was used to develop a six hour design storm for the 0.05% AEP and PMF event.

Sensitivity Analysis

Two climate change horizons were considered: 2050 and 2100. The adopted assumptions are in line with a state government report on climate change in Queensland (DERM et al., 2010). The adopted climate change assumptions are:

- 2050: 10% increase in rainfall intensity and 300mm increase in mean sea level
- 2100: 20% increase in rainfall intensity and 800mm increase in mean sea level.

A simplified approach was adopted for the inflow from Bulimba Creek. Whereby, the flow was increased by 10% and 20% for the 2050 and 2100 horizon respectively. This approach is considered suitable, as the flows in the lower Bulimba Creek are largely controlled by the water level at the Brisbane River.

The 1% and 0.5% AEP design events were simulated for the existing scenario for both the 2050 and 2100 horizons, and the 0.2% AEP design event was simulated for the 2100 horizon.

A structure blockage assessment was carried out in line with the provisional 2013 edition of QUDM (DEWS, 2013). QUDM recommends a culvert blockage of 20% for unscreened culverts with width of less than 5m and 10% for unscreened culverts with width of greater than 5m. Primary structures were selected for the assessment and grouped into three:

1. Blockage of culverts along Lytton Road and Wondall Road;
2. Blockage of culverts along Kianawah Road; and
3. Blockage of culverts along Cleveland Railway line, at Hemmant and Tingalpa Road and the northern Port of Brisbane Motorway.

The groups were selected to ensure that the additional attenuation caused by blockages did not influence the assessment of blockage at culverts further downstream. The assessment was undertaken on the existing scenario for the 1% AEP design flood event.

Summary of Study Findings

This flood investigation has estimated the hydraulic behaviour of flood waters through the study area associated with the design flood events and historical events that were assessed. The model is designed to assess large flood events originating from the watercourses, and was based on information provided at the time of the investigation. The following should be considered for future use of the model:

- Future development may influence the results presented in this study;
- Flooding from sources other than watercourses (such as overland flow) has not been simulated in this study;
- A review of the Bulimba Creek model was beyond the scope of this investigation. Therefore, the Bulimba Creek model has been assumed suitable for use in the current investigation; and
- The TUFLOW model has been based on pre-existing TUFLOW models. It has been assumed that the information on channel and structures in the pre-existing models is correct.

Deliverables

The following deliverables have been developed:

- Hydrological model (XP-RAFTS) input and output files;
- Hydraulic model (TUFLOW) input and output files;
- Spatial data files providing information on flood levels, extents, etc.;
- Flood Inundation Mapping (Volume 2 Report - separate A3 document);
- Tabulated Results (Appendix D);
- Flood Study Report;
- Hydraulic Structure Reference Sheets (Appendix C); and
- Model Handover Guide.

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Glossary of Terms

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.
ARI	The Average Recurrence Interval (ARI) is a statistical estimate of the average period in years between the occurrence of a flood of a given size. For example, the 10 year ARI event will occur on average once every 10 years
Flood model	Refers to both the hydrologic and hydraulic models
LiDAR	Refers to an aerial survey technique that uses a laser and analyses the reflected light
PMF	Probable Maximum Flood. The maximum flood that is reasonably estimated to not be exceeded. Derived from a PMP.
PMP	Probable Maximum Precipitation. The maximum precipitation (rainfall) that is reasonably estimated to not be exceeded.

AEP – ARI Conversion Table	
AEP (%)	**ARI (years)**
50	2
20	5
10	10
5	20
2	50
1	100
0.5	200
0.2	500
0.05	2000

List of Abbreviations

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)
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
QUDM	Queensland Urban Drainage Manual (2013)
WC	Waterway Corridor

1.0 Introduction

1.1 Catchment Overview

The Hemmant-Lytton study area covers an area of 22km² and includes the suburbs of Hemmant, Lytton and Wynnum West (see Figure 1-1). Hemmant Drain and Lindum Creek are the primary waterways within the catchment. Hemmant Drain drains into the lower reach of Bulimba Creek which, in turn, drains into the Brisbane River) and Lindum Creek drains into Bulimba Creek close to the Brisbane River. The catchment is divided into the following three distinct land uses:

- Low density residential zoning covers much of the upper catchment;
- Green space, parkland and rural zoning cover much of the middle of the catchment; and
- The lower region of the catchment is heavily dominated by industrial sites.

The Port of Brisbane Motorway and the Cleveland Railway line extend across the full width of the catchment from Bulimba Creek to the Caltex Oil Refinery.

The entire catchment lies within the Brisbane City Council (Council) jurisdiction. Figure 1.1 indicates the locality of the catchment.

1.2 Study Background

Council is in the process of updating all of its flood studies to reflect the current conditions of the catchments and best practice flood modelling techniques. In addition to this, Council is developing a Neighbourhood Plan for the Hemmant-Lytton area, and as such have expedited the implementation of this flood investigation. This will ensure that Council has the most up-to-date information for the Hemmant-Lytton catchment and the study will assist in floodplain management and planning purposes within the catchment.

The most recent flood investigation for the catchment is the "Hemmant-Wynnum West Master Drainage Plan and Flood Study, BCC, 1997" which includes the main channel and side tributaries in the Hemmant Drain and Lindum Creek catchments. Subsequent flood investigations were undertaken to assess flooding constraints related to proposed infrastructure in the catchment.

1.3 Study Objectives

The Hemmant-Lytton Flood Study has been prioritised for the following reasons:

- Changes to the watercourse and associated infrastructure have occurred since the last investigation was undertaken;
- New survey has been undertaken in a number of areas since the last investigation was undertaken;
- Inclusion of Lytton area and lower part of Bulimba Creek catchment in the study area;
- Extension of the hydraulic model coverage; and

- Flood modelling software has advanced and more sophisticated modelling methodologies are now available.

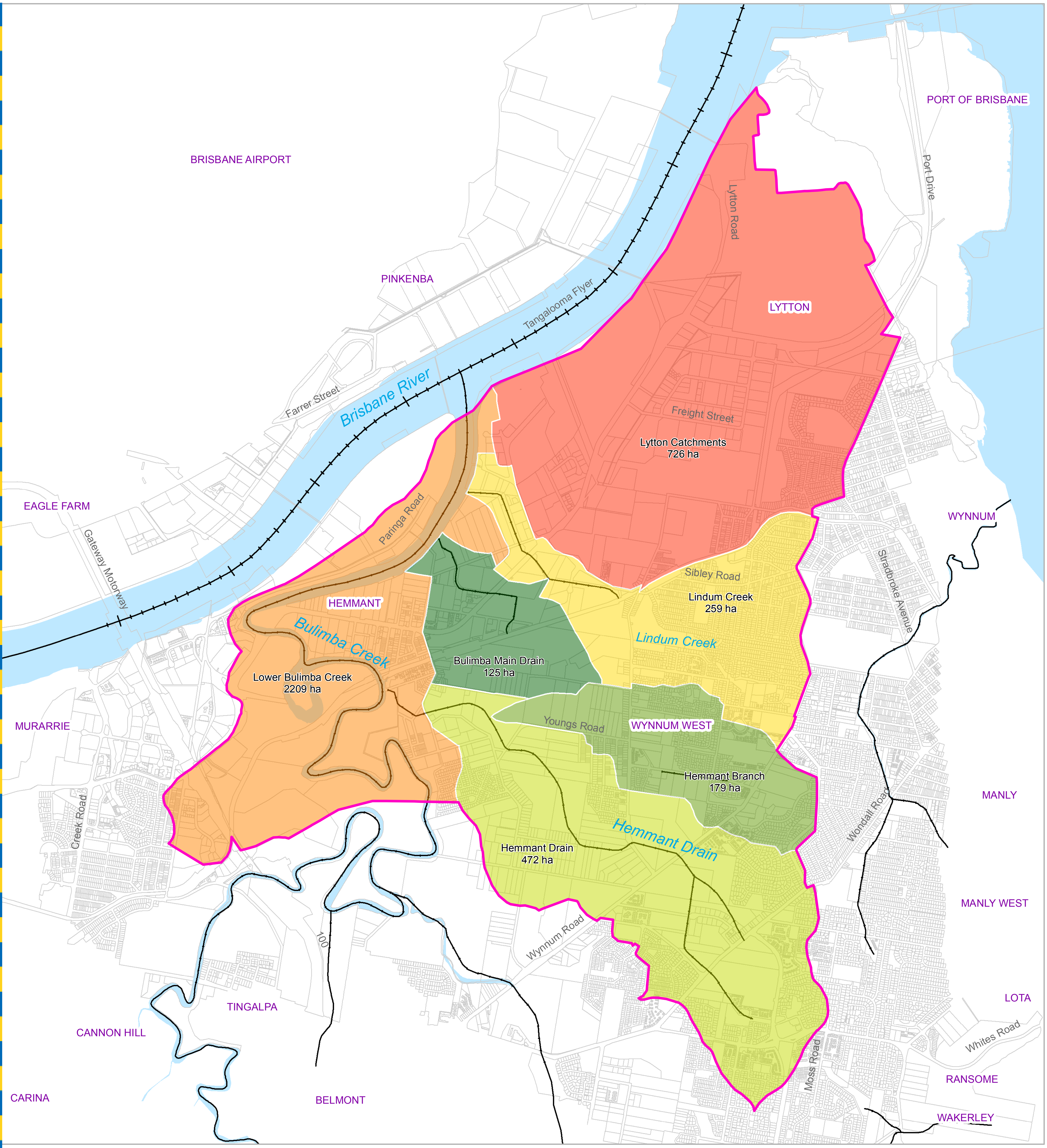
Council has revised documentation defining the required level of service/specifications for the studies in the BCC area from a planning and floodplain management perspective and the existing flood investigation does not meet these revised standards.

The aim of this flood investigation is to determine flood levels for a range of design flood events, along with the provision of flood inundation and depth x velocity mapping. The investigation will also contribute to ensuring consistency of flood models and reporting across all of Council's creek catchments. The completed investigation will serve as the provision of flooding information to assist in the setting of Council planning policy and floodplain management.

1.4 Report Scope and Limitations

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

- Review the existing XP-RAFTS hydrologic model within the catchment area and amend and extend as appropriate to incorporate the additional areas within the study area and to represent current catchment conditions.
- Review the existing hydraulic models (Tilley Road – Stage 2 extension model and Port of Brisbane Motorway model) and combine and extend these to develop a hydraulic model of the full study area and to represent the current catchment conditions and best practice flood modelling techniques.
- Undertake a joint calibration of the hydrologic and hydraulic models to the October 2010 and January 2013 historical flood events for the upper area of the catchment.
- Validate the hydrological and hydraulic models to the December 2010 historical flood event.
- Determine and model the design flood events for the full range of events up to the Probable Maximum Flood (PMF) for a broad range of storm durations for the existing catchment (Scenario 1).
- Simulate the minimum riparian corridor scenario (Scenario 2) for the 1% AEP design event.
- Simulate the ultimate development scenario (Scenario 3) for the full range of events up to the 1% AEP flood event for a broad range of storm durations.
- Undertake sensitivity testing on selected model parameters, blockage and climate change.
- Produce flood inundation mapping for a selected range of design and extreme events for the existing scenario.



Legend

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- Modelled Area
- AMTD Line
- Waterway/Waterbody

Catchment Boundary

- Bulimba Main Drain
- Hemmant Branch
- Hemmant Drain
- Lindum Creek
- Lower Bulimba Creek
- Lytton Catchments

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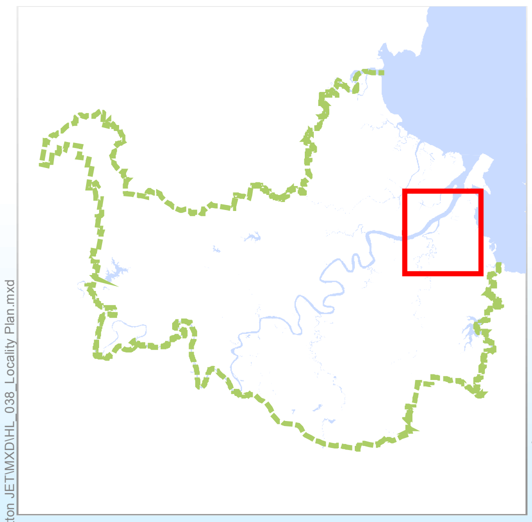

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Prepared : Jayesh.Patel
Checked : RS
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File : I:\B20669_I_BRH Hemmant Lytton JET\MXD\HL_038_Locality Plan.mxd

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.

Hemmant Lytton Locality Plan

Figure 1-1

MXD-038

2.0 Catchment Description

2.1 Catchment and Waterway Features and Characteristics

The Hemmant-Lytton catchment is situated in the eastern suburbs of the Brisbane City Council area, bordering Wynnum Creek and Crab Creek on the eastern and northern side and Bulimba Creek and Lota Creek on the western and southern side. The Hemmant-Lytton catchment covers the suburbs of Hemmant, Lytton and Wynnum West. Hemmant Drain and Lindum Creek are the major overland flow paths within the catchment. These have both been heavily modified throughout the catchment and exist as, for large portions, engineered open channels. Both Hemmant Drain and Lindum Creek flow into the lower reaches of Bulimba Creek through separate channels.

Due to the interaction between Bulimba Creek catchment and the Hemmant Drain and Lindum Creek catchment during flood events, the total catchment area is irregular in shape. As such the study area incorporates catchments for Lindum Creek, Hemmant Drain, the Lytton area as well as the lower reaches of Bulimba Creek. The study area extends from south of Manly Road to the Brisbane River. There are several key obstructions to flow across the catchment, including Wondall Road, Wynnum Road, the Port of Brisbane Motorway and the Cleveland Railway line.

2.2 Land Use

The catchment is highly urbanised, with most of the rainfall runoff being directed though a stormwater network before reaching the creeks. There are three distinct changes in land use throughout the study area. Low density residential zoning covers much of the upper catchment whilst green space, parkland and rural zoning cover much of the middle of the catchment. The lower regions of the catchment are heavily dominated by industry.

The south east of Kianawah Road and to the south of Wynnum Road is typically populated by low density residential areas and a network of urban roads. A network of underground stormwater assets provides a key linkage of rainfall runoff to the overland flow paths within the catchment, principally Lindum Creek and Hemmant Drain.

Significant areas of green space and rural land lie adjacent to the main overland flow paths within the catchment, in particular along the mid and lower reaches of Hemmant Drain. The majority of the catchment between Wynnum Road and the Cleveland Railway line, bordered by Kianawah Road to the east, exists as largely undeveloped open green space of very low density rural housing.

North of the Cleveland Railway line is where the most distinct change in catchment land use occurs. Here, the catchment is heavily industrialised, resulting in a high proportion of fraction of impervious area. Underground stormwater assets combined with engineered open channels are key flow paths through this area.

3.0 Available Information

3.1 Previous Studies

3.1.1 Hemmant-Wynnum West Area, Master Drainage Plan and Flood Study

The Hemmant-Wynnum West Area, Master Drainage Plan and Flood Study (hereafter referred to as the '1997 MDP') was the last major investigation undertaken for this area (L&T, 1997). It was undertaken by Lawson and Treloar (now Cardno) on behalf of Council and completed in 1997. This investigation included the main channel and side tributaries in the Hemmant Drain and Lindum Creek catchments. Since the development of the 1997 MDP model, the occurrence of flood events in the catchment has made available additional calibration data for use in this study. Additionally, some catchment development and channel modifications have also occurred. Several structures have been constructed/upgraded in the catchment.

The XP-RAFTS software was used to develop the hydrology model for the 1997 MDP investigation, though it existed in two separate models.

- Southern Catchment – the southern catchment model extends from Manly Road to Kianawah Road and Hemmant and Tingalpa Road.
- Northern Catchment – the northern catchment model extends from Lindum Creek to Bulimba Creek.

The models cover the full Hemmant Drain catchment. Council holds these RAFTS models in two formats, the first being the catchment in its existing condition at the time of the original investigation and the second being an ultimate development case. These models were used as a base for development of the hydrologic model for the current investigation.

The 1997 MDP used MIKE11 to develop a 1D hydraulic model of the catchment.

3.1.2 Tilley Road Extension Flooding Assessments

As part of a hydraulic assessment investigating the feasibility of extending Tilley Road, this model was updated by Council to a linked 1D-2D model using MIKEFLOOD. The MIKEFLOOD model was then updated and converted to TUFLOW by Aurecon (Aurecon, 2012) at the preliminary design stage of the Tilley Road extension – Stage 2 project, and used to assess potential flood mitigation options. The TUFLOW model was provided by Council along with the hydraulic assessment report (Aurecon, 2012).

3.1.3 Port of Brisbane Motorway – Stage 2

GHD developed a TUFLOW model of the Lytton area as part of the Port of Brisbane Motorway Stage 2 project. The model was provided by Council without any supporting information.

3.1.4 Bulimba Creek Flood Study

The Bulimba Creek Flood Study was undertaken by Council and completed (in draft form) in June 2011 (BCC, 2011). Since Hemmant Drain is a tributary to Bulimba Creek, the study area for this Hemmant-Lytton Flood Investigation encompasses the lower portion of Bulimba Creek. Council provided the WBNM hydrologic model and MIKE11 hydraulic model from the Bulimba Creek Flood Study. These models were used to extract flow data at the upstream boundary of the Hemmant-Lytton hydraulic model. Since the Bulimba Creek Flood Study used a Duration Independent Storm (DIS) approach, the WBNM and MIKE11 models have been rerun as part of this investigation to model each storm duration.

It was beyond the scope of this investigation to review the Bulimba Creek modelling. It has been assumed that the Bulimba Creek models are suitable for use in this investigation.

3.2 Topographic Survey Data

3.2.1 Field and Bathymetric Survey

Field and bathymetric survey of the waterways has been incorporated into the pre-existing hydraulic models developed by Aurecon and GHD.

3.2.2 Aerial Survey and Photography

Aerial imagery (1997, 2001, 2009, 2012) have been provided. 2002 and 2009 Airborne Laser Scanning (ALS) data has been provided as XYZ point format. The 2009 dataset has been used throughout the model to develop the hydrologic model catchment delineation. However, it does not extend north of Port Drive. In this area the 2002 dataset was used to supplement the 2009 dataset.

Neither the 2002 or 2009 ALS data contains the Port of Brisbane Motorway from Port Drive to Canberra Street. Topographic data for this infrastructure was extracted from the GHD TUFLOW model.

3.3 Hydrometric Data and Analysis

3.3.1 Recorded Rainfall

Recorded rainfall for the calibration flood events (October 2010 and January 2013) and the validation flood event (December 2010) were provided for three rainfall gauges that lie either inside or adjacent to the Hemmant-Lytton Catchment. Recorded rainfall depth at 10 minute intervals were provided for Wynnum Creek at Bowls Club (W_R837), Bulimba Creek at Hemmant (BMR527) and Watervale Parade at Wakerley Bio-retention (LTR759). The rainfall gauges are shown in Figure 4-1 and rainfall distribution for the three historical events in Figure A1 and A2.

3.3.2 Recorded Flood Levels

3.3.2.1 *Stream Gauge Data*

Stream gauge river height data was provided for the calibration and validation flood events for the stream gauge Bulimba Creek at Hemmant (BMA528). This is the only stream gauge in the study area, and is located on Bulimba Creek near the confluence with Hemmant Drain. The water levels at the gauge are controlled by the tidal levels in the nearby lower Brisbane River.

3.3.2.2 *MHG Data*

Maximum Height Gauge (MHG) data were provided for the calibration and validation flood events for:

- Hemmant Channel (HM110) – located upstream of Hemmant and Tingalpa Road crossing on Hemmant Drain 500m upstream of its confluence with Bulimba Creek.
- Hemmant Channel (HM130) – located downstream of Wynnum Road on Hemmant Drain.
- Hemmant Channel (HM210) – located upstream of Kianawah Road on a tributary to Hemmant Drain.

The location of these gauges is shown on Figure 4-1.

3.3.2.3 *Debris Marks*

No debris marks have been provided for this investigation.

3.3.3 Tidal Information

Tide levels at Brisbane bar were provided for the calibration and verification events. Additional tidal data was obtained from Maritime Safety Queensland's *Semidiurnal Tidal Planes – 2014*.

3.4 Hydraulic Structure Data

Hydraulic structure data were obtained from the pre-existing hydraulic models. As built drawings were provided for some structures and used to check the information obtained from the existing models.

3.5 Other Model Data

Cadastral data and City Plan Area Classifications have been provided in MapInfo format and have been used in conjunction with aerial photography to determine current and future land use.

3.6 Selection of Calibration and Verification Events

Council selected the January 2013 and October 2010 flood events as calibration events and December 2010 as a verification event. The January 2013 is the most recent and biggest (for durations of longer than 2 hours) of the historical events. By comparison with AR&R IFD curves (see Figure 3-1 to Figure 3-3) the January 2013 event has a magnitude of about 50% AEP. For locations of rainfall gauges see Figure 4-1.

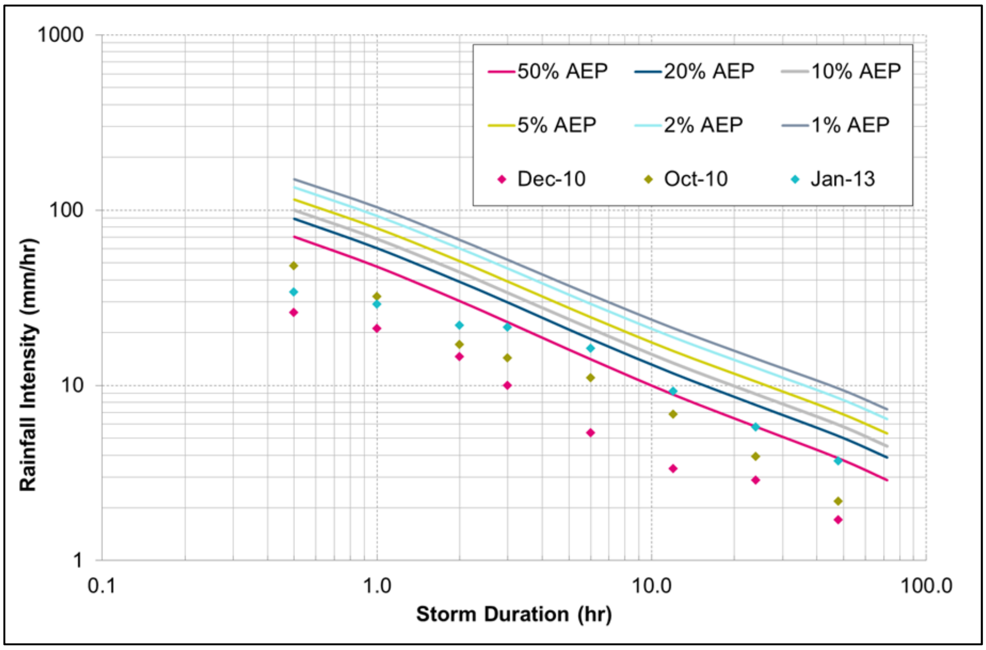


Figure 3-1: Rainfall IFD Curve for BMR527

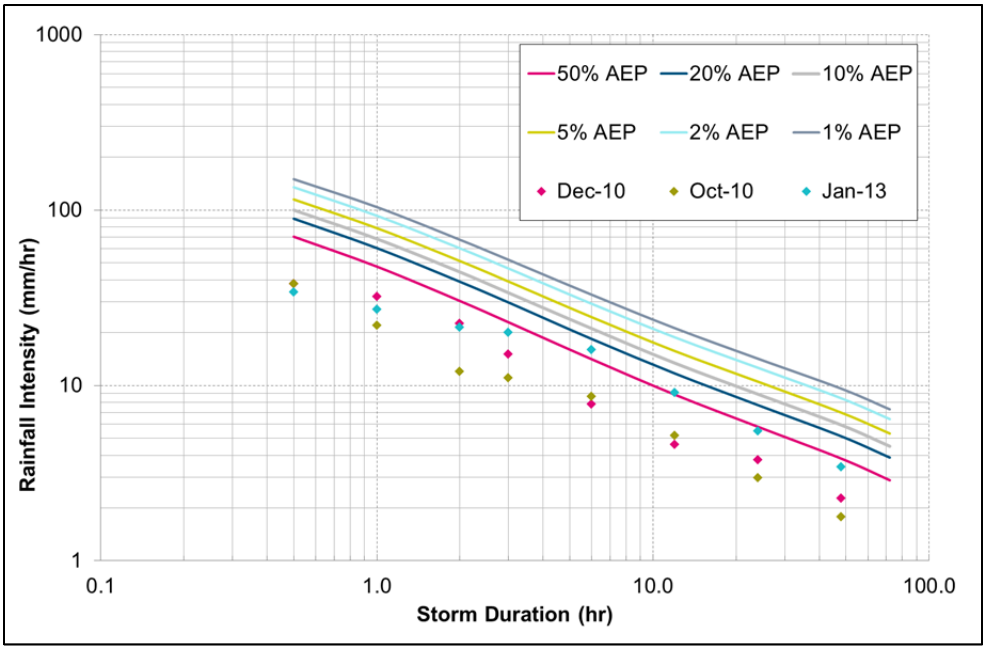


Figure 3-2: Rainfall IFD Curve for WR837

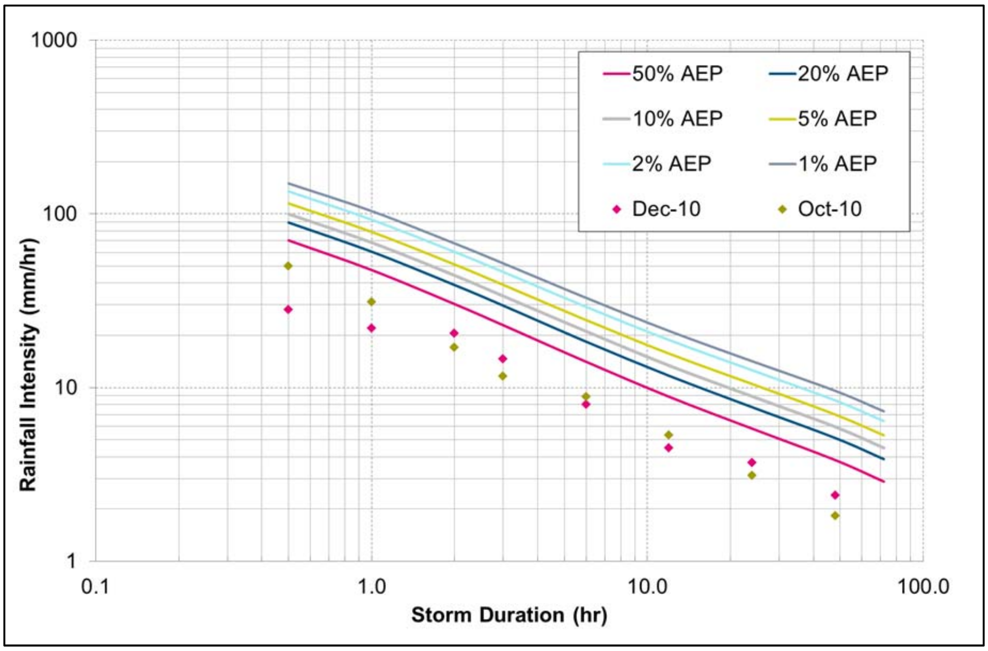


Figure 3-3: Rainfall IFD Curve for LTR759

The recorded flood level during the January 2013 flood event at MHG 130 shows the highest level on Hemmant Drain (see Table 3-1). In contrast, MHG 210 shows the January 2013 event to be the smallest of the three historical events. This may be due to the MHG 210being located in the upper catchment on a small tributary to Hemmant Drain, and very short storm durations being critical. Also, this gauge is located upstream of a structure and levels are sensitive to blockage. The recorded level at MHG 110 for December 2010 is considered to be unreliable (see further discussion in Section 5.4). Hence, the ‘trusted’ recorded levels at MHG 110 are considered to be consistent with the January 2013 event being the largest.

Table 3-1: List of Maximum Height Gauges with Recorded Level (mAHD)

Gauge ID	11-Oct-10	26-Dec-10	27-Jan-13
110	1.56	1.78	1.68
130	2.51	2.7	3.19
210	2.47	2.8	2.15

4.0 Hydrologic Model Development and Calibration

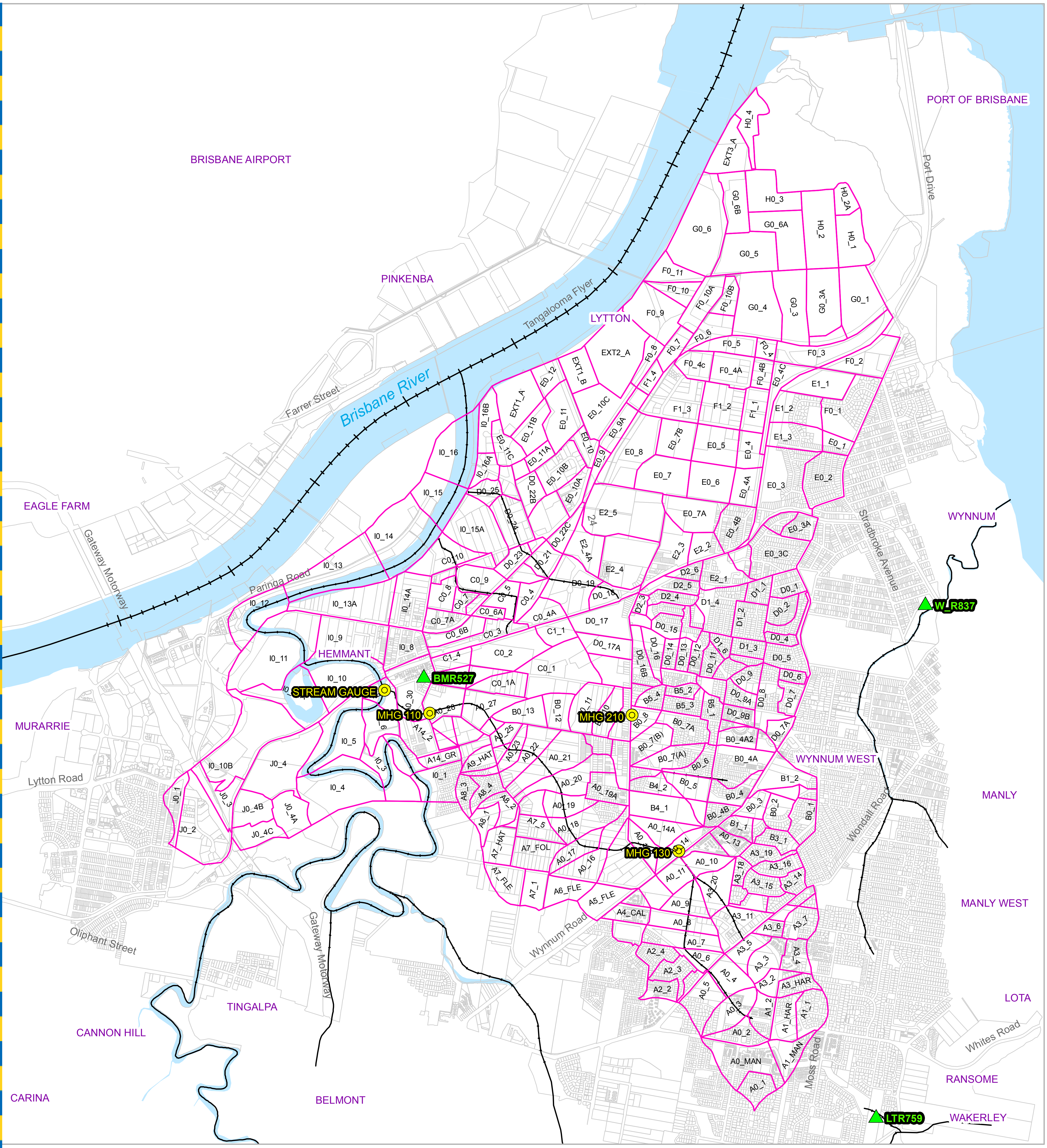
4.1 Overview

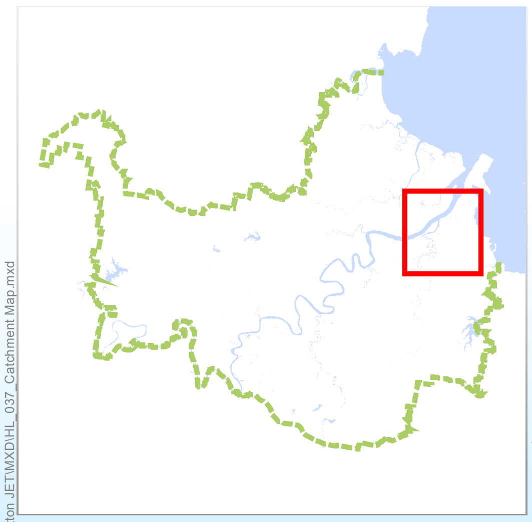
The hydrologic model simulates the rainfall-runoff process within the catchment and calculates the flow hydrograph at the outlet of each sub-catchment. The XP-RAFTS model for the Hemmant-Lytton catchment was initially developed as part of the 1997 Hemmant – Wynnum West Area, Master Drainage Plan and Flood Study (Council) and existed in two separate models as described in Section 3.1.

Review of the existing XP-RAFTS model indicated that the majority of the sub-catchment delineation upstream of the Cleveland Railway line was acceptable and was retained. However, catchment changes downstream of the railway required significant re-working of the sub-catchment boundaries. The separate existing northern and southern models were combined, extended and updated to address the following:

- Combining of the pre-existing northern and southern models.
- Extending model area to include lower reaches of Bulimba Creek and Lytton area. The extension into Bulimba Creek was done to simplify the hydraulic model inputs. Such that all local catchment inflows in the hydraulic model were derived from the XP-RAFTS model.
- Update the model to XP-RAFTS version 2009.
- Update of sub-catchment delineation as a result of new development.
- Review and update the catchment parameters (e.g. impervious percentage, PERN, catchment slope) to suit the revised sub-catchment delineation and current catchment conditions for the historical events and City Plan for the ultimate development scenario.

A catchment map is presented in Figure 4-1.





Legend

- Rain Gauges
- Gauge_Locations
- Sub-catchments
- AMTD Line
- Waterway/Waterbody

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Metres

N W E S

Prepared : Jayesh.Patel
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Hemmant Lytton CatchmentI Map

Figure 4-1

MXD-037

4.2 Model Set Up and Schematisation

The Hemmant-Lytton XP-RAFTS model comprises 302 sub-catchments and the layout is illustrated in Figure 4.1. It is recognised that the sub-catchment delineation is relatively fine scaled; this has perpetuated from the sub-catchment delineation resolution used in the 1997 XP-RAFTS model. Catchment and sub-catchment delineation downstream of the Cleveland Railway line and in a few other areas within the model were adjusted to better represent current topographic conditions.

The modelled sub-catchment slope was updated and a slope calculated for each sub-catchment. Sub-catchment slopes have been calculated from the topography by identifying indicative longest flow paths using the equal area method.

The Hemmant-Lytton catchment is considered to be heavily urbanised. The land use and impervious areas have been identified using aerial photography and City Plan Area Classifications. The adopted land use for the calibration and verification events is listed in Table 4-1. The total area of each sub-catchment was multiplied by the fraction impervious weighting to create two areas representing the total area of pervious area and total area of impervious area within each sub-catchment. This allows separate parameters to be applied to impervious and pervious areas throughout the model as required.

Table 4-1: Sub-catchment Fraction Impervious by Land-use

Land-use Type	% Impervious
Community Use Area Cemetery	50
Community Use Area Community Facilities	70
Community Use Area Education Purposes	70
Community Use Area Emergency Services	70
Community Use Area Health Care Purposes	70
Community Use Area Railway	75
Community Use Area Utility Services	75
Conservation	0
Emerging Communities	70
Environmental Protection	0
Future Industry	90
General Industry	90
Heavy Industry	95
High Density Residential	90
Light Industry	90
Low Density Residential	60

Land-use Type	% Impervious
Low-Medium Density Residential	70
Medium Density Residential	80
Multi Purpose Centre Convenience Centre	90
Multi Purpose Centre Suburban Centre	90
Park Land	5
Rural	20
Special Purpose Centre Major Hospital And Medical Facility	80
Special Purpose Centre Port	90
Sport And Recreation	20
Road Reserve	90
Creek and Other Pervious Area (Parks)	0

The hydrologic roughness parameter (PERN) is input as a Manning's 'n' representation of the average sub-catchment roughness. As each sub-catchment is divided into pervious and impervious portions, this allows a different hydrologic roughness to be applied to pervious and impervious areas within each sub-catchment.

The PERN value for the impervious component for each sub-catchment was determined to be a value of 0.015. This is consistent with other studies conducted by Council and is representative of typical materials, such as concrete, that create impervious areas.

The PERN value for the pervious component of each sub-catchment was simplified to three values:

- n = 0.04 – represented smooth terrain with little vegetation, and urbanised areas;
- n = 0.06 – moderately vegetated areas; and
- n = 0.08 – more densely vegetated areas.

Routing between sub-catchments has been developed using time lag links.

4.3 Calibration Procedure

Recorded data from each calibration and verification event were incorporated into the XP-RAFTS model using a standard RAFTS storm in the 'Global Databases'. The XP-RAFTS rainfall database comprised recorded rainfall at ten minute intervals. This enabled the full rainfall period for each of the events to be modelled.

Voronoi polygons were created between rainfall gauging stations to enable the recorded rainfall to be apportioned to each of the sub-catchments in the XP-RAFTS model (see Figure A-1 and A-2). The relevant pluviograph was assigned to each sub-catchment for which the centroid of the sub-catchment was located within the respective voronoi polygon.

An Initial Loss (IL) and Continuing Loss (CL) approach was adopted for rainfall losses. The IL (mm) is 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 capacity of the soil, whereby a dry soil has a larger capacity than a saturated soil. This loss can change across historical events to reflect different antecedent conditions. The CL (mm/hr) is the average loss rate throughout the remainder of the rainfall event and is predominantly dependant on the underlying soil type and porosity. This is a catchment characteristic loss rate and does not change across historical events unless warranted by changes in the catchment, such as development.

The IL and CL have been iteratively refined during calibration to improve the comparison between modelled and recorded levels.

Hydrologic model calibration and validation was undertaken jointly with the TUFLOW hydraulic model. Both models were calibrated to two events (October 2010 and January 2013) and verified against a third event (December 2010). There is no recorded flow data in the catchment, so it was not possible to calibrate the hydrologic model independent of the hydraulic model. As such, there are no calibration results to display from the hydrologic model. Comparisons between modelled and recorded data are presented in Section 5.4.

Rainfall distribution maps are located in Appendix A, and the final adopted hydrological modelling parameters for each sub-catchment are tabulated in Appendix B. The adopted rainfall losses are listed in Table 4-2.

Table 4-2: Adopted Rainfall Losses – Calibration and Design Events

Historical Flood Event	Pervious Initial Loss	Pervious Continuing Loss
January 2013	15	2
October 2010	15	2
December 2010	15	2

4.4 Comparison with Rational Method

The results from the hydrological model (XP-RAFTS) were compared to a certified hydrologic assessment method, namely, The Rational Method. The methodology outlined by the Queensland Urban Drainage Manual (DEWS, 2013) was followed in order to obtain these results.

In its general form, the Rational Method equation is:

$$Q_y = C_y \, {}^{t}_{y}IA$$

Where:

Q_y = peak flow rate (m^3/s) for annual exceedance probability (AEP) of 1 in 'y' years
C_y = coefficient of discharge (dimensionless) for AEP of 1 in 'y' years
A = area of catchment (m^2)
I_y = average rainfall intensity (mm/h) for a design duration of 't' hours and an AEP of 1 in 'y' years
t = time of concentration (hours)

The rational method was undertaken on the two largest portions of the Hemmant Lytton catchment which were not modelled hydraulically. These were identified to be on the Western face as shown in Figure 4-2.

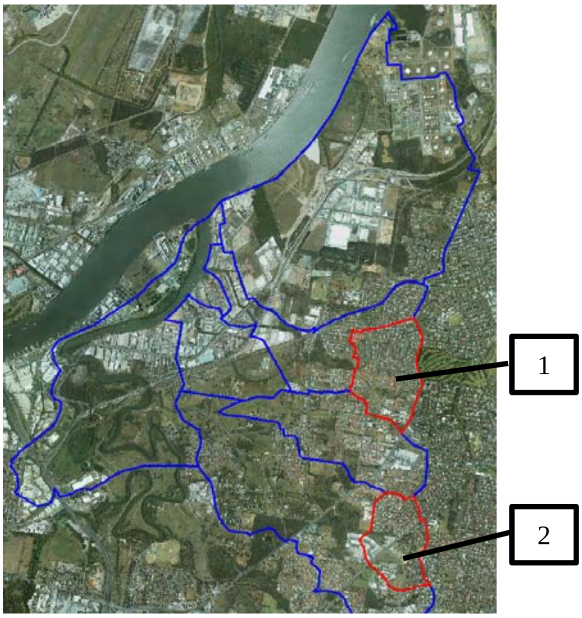


Figure 4-2: The Two Largest Portions of Hemmant Lytton Catchment

The QUDM process of the rational method was undertaken to determine the following parameters for sub-catchments 1 and 2 as labelled in Figure 4-2.

Table 4-3: Catchment Characterisations of the Rational Method

Parameter	Sub-catchment 1	Sub-catchment 2
Area	926, 000 m^2	767, 600m^2
Percent Impervious	56.01	67.57
C_{100}	0.969	0.983
${}^{0.5}_{100}I$	148 mm/h	148 mm/h

From these parameters, the peak flow was able to be calculated and is shown in Table 4-4.

Table 4-4: Comparison of RAFTS and 'The Rational Method' Using the Peak Flow Rate

Catchment ID	RAFTS Results	Rational Method Result
Sub-catchment 1	44.81 m^3/s	36.91 m^3/s
Sub-catchment 2	29.68 m^3/s	31.01 m^3/s

As evident from the results, the RAFTS hydrology model gives a sensible estimate as verified by the Rational Method. It should be noted that the Rational Method is an extremely lumped and approximate method, hence should only be used to verify the correctness of hydrology models by giving a 'ball-park' estimation of the peak flow rate. Considering the differences of the two methods give an error margin within the range of 4% - 20%, the RAFTS hydrology model is considered to give suitable results and is considered verified for the hydrologic model of choice for this study.

5.0 Hydraulic Model Development and Calibration

5.1 Overview

A hydraulic model of the catchment was developed to route the runoff – computed using the hydrologic model – through the catchment. The hydraulic model predicts information such as flow, depth and velocity throughout the catchment based on the input flows. Therefore, the hydraulic model is a tool for developing an understanding of flood risk in the catchment.

For this flood investigation, pre-existing hydraulic models were available (see Section 3.1). Nevertheless, the Hemmant-Lytton hydraulic model was built 'from scratch' using the TUFLOW hydraulic modelling software. TUFLOW is a 2D hydraulic modelling software, which simulates depth-averaged free surface flow across a regular square grid. It also simulates 1D free surface flow and flow across hydraulic structures.

The pre-existing hydraulic models were TUFLOW models. Therefore, many of the 'building blocks' of the pre-existing models were copied across to the new Hemmant-Lytton TUFLOW model. The composition of the model is discussed below.

5.2 Model Development

5.2.1 Model Schematisation

The hydraulic model covers an area of 22km², and is composed of two domains:

- A 1D domain; where waterways have been modelled in 1D due to a relatively narrow channel width compared to the 2D cells size; and
- A 2D domain; the remainder of the floodplain and waterways where the channel width is relatively wide compared to the 2D cell size or the flow conveyance through the waterways is adequately represented by the 2D grid.

The location and extent of these domains is shown in Figure 5-1. The 2D domain is based on a 4m x 4m regular square grid. The 1D domain was extracted from the pre-existing TUFLOW models, and comprises the main waterways through the study area. The 1D and 2D domains are linked, such that water can flow between the two domains during the simulation.

The model extent covers the lower reach of Bulimba Creek in order to capture the flood behaviour at the confluence with Hemmant Drain.

5.2.2 Topography

Each grid cell in the 2D domain comprises 5 points used in the 2D computation (Z points):

- A Z point in the centre of the cell; ZC – used to compute the water depth at the cell and determine if the cell is 'wet' or 'dry'; and

- A Z point on the centre of each cell side; ZU and ZV – used to compute the velocity across the side of each cell.

Elevations were assigned to each Z point using the following data:

- Topographic data from a LiDAR survey in 2002 was the only data available in the vicinity of the oil refinery in the northern extremity of the Lytton catchment. This data formed the basis for the topography in the 2D domain in this area.
- Topographic data from a LiDAR survey in 2009 formed the basis of the topography in the remainder of the 2D domain.
- The pre-existing 2D model of the Port of Brisbane Motorway included a GIS layer of the Z points representing the topography of the Port of Brisbane Motorway structure. This point object GIS layer was converted to a 2m ASCII grid (linear triangulation with smoothing using Vertical Mapper). The grid of the Port of Brisbane motorway was then read directly into the model.

TUFLOW software includes terrain modifiers (Z point, Z line and Z shape layers), which facilitate modification of the elevations of the Z points in the model. Terrain modifiers were used to:

- Modify the topography surrounding structures (see Section 5.2.4).
- Carve continuous 'gullies' into the 2D grid where small waterways were represented in the 2D domain.
- Carve a channel into the 2D domain along Bulimba Creek. Since the original survey could not be located, the topography modification focussed on approximating the flow area and invert level of the cross section in the pre-exiting MIKE11 model of Bulimba Creek.
- Cut 'lakes' into the topography where large water bodies had deformed the triangulation of the LiDAR data.
- Fix topographical irregularities that caused instability issues in the 2D domain.
- Fill the floodplain for the Ultimate Development Scenario (see Section 6.1).

The topography in the 1D domain was based on that in the pre-existing models. In the Hemmant-Lytton catchment, the topography originated from a survey of the watercourses provided by Council for the 1997 MDP. No information was provided on the source of the topography in the 1D domain in the Port of Brisbane Motorway model (developed by GHD).

5.2.3 Land Use

Land use across the floodplain has been delineated in order to define spatially varying hydraulic roughness in the 2D domain. Council's City Plan was used as a basis for the land use delineation. Land use defined in the pre-existing models was then overlayed. The resulting land use was then reviewed using aerial photography. Figure 1-1a and Figure 1-1b show the adopted land use categories for the existing and ultimate scenarios and the adopted Manning's n values are listed in Table 5-1.

TUFLOW uses the Manning's model to compute friction losses for flow. Therefore, each land use type was assigned a Manning's n value.

Table 5-1: Land Use Classification

Land Use ID	Manning's n	Description	Source
1	0.15	Urban/Residential Block	Tilley Road Extension
2	0.12	Industrial	Tilley Road Extension
3	0.015	Streets	Tilley Road Extension
4	0.04	Mowed grass	Tilley Road Extension
5	0.045	Long Grass Channel	Tilley Road Extension
6	0.04	Vegetated channel + Open trees	Tilley Road Extension
7	0.12	Dense Vegetation	Tilley Road Extension
8	0.08	Medium Density trees	Tilley Road Extension
9	0.15	Minimum Riparian Corridor	-
101	0.15	Low-Medium Density Residential	City Plan
102	0.1	Low Density Residential	City Plan
103	0.1	Conservation	City Plan
104	0.2	Emerging Communities	City Plan
105	0.045	Sport And Recreation	City Plan
106	0.15	Community Use Area Education Purposes	City Plan
107	0.05	Park Land	City Plan
109	0.2	General Industry	City Plan
110	0.2	Multi-Purpose Centre Suburban Centre	City Plan
111	0.2	Future Industry	City Plan
112	0.08	Environmental Protection	City Plan
113	0.2	Heavy Industry	City Plan
114	0.2	Community Use Area Health Care Purposes	City Plan
115	0.04	Community Use Area Railway	City Plan
116	0.045	Rural	City Plan
117	0.045	Community Use Area Cemetery	City Plan
119	0.2	Multi Purpose Centre Convenience Centre	City Plan

Land Use ID	Manning's n	Description	Source
120	0.2	Light Industry	City Plan
121	0.07	Community Use Area Utility Services	City Plan
122	0.1	Community Use Area Community Facilities	City Plan
124	0.1	Community Use Area Emergency Services	City Plan
125	0.15	Special Purpose Centre Major Hospital And Medical Facility	City Plan
203	0.03	Open space- mostly grass	Port of Brisbane Motorway
204	0.04	Open space- some bush	Port of Brisbane Motorway
206	0.05	Creek or Open space- mostly bush	Port of Brisbane Motorway
261	0.025	Smooth Waterway	Port of Brisbane Motorway
262	0.06	Medium Vegetated Waterway	Port of Brisbane Motorway
263	0.07	High Vegetated Waterway	Port of Brisbane Motorway
264	0.045	Low Vegetated Waterway	Port of Brisbane Motorway

5.2.4 Hydraulic Structures

There are 50 waterway crossings in the study area. One of these is a bridge (under the Port of Brisbane Motorway) and the remainder are culverts. These structures have been represented in the model using 1D structure channels – either rectangular or circular. Table 5-2 lists all the culverts in the model along with the source of structure details. Note that some structure details were not available, and parameters have been assumed based on the waterway size and topography in the vicinity and site inspection.

Overflow of the structures has generally been simulated by defining the crossing crest level in the 2D domain (using a TUFLOW terrain modifier where necessary), and allowing water to overtop the structures in the 2D domain. Note that railing on structures has been assumed to be 100% blocked for design events only. This was represented using a TUFLOW terrain modifier that adds a specified height to the underlying Z points.

A tidal gate near the outlet of Hemmant Drain was included in the model using unidirectional circular culverts (FB8673/1 and FB8673/2). This structure is located at a track crossing about 85m downstream of the Hemmant and Tingalpa Road crossing and comprises two 1.3m diameter culverts and one 1.5m diameter culvert. Overtopping of the structure is represented using a 1D weir channel. The structure details were extracted from the 1997 MDP report (L&T, 1997).

Table 5-2: List of Structure Crossings

Network ID	Crossing Name	Locality	Details	Source
C1868P & C1870B	Kianawah Rd	Upper Hemmant Drain	1No. 2.4m x 0.6m	Council Stormwater Data
C4543B	Beverly Road	Hemmant catchment	5No. 1.8m x 0.75m	Council Stormwater Data
C0403B	Ropley Road	Hemmant Branch	1No. 1.8m dia.	Council Stormwater Data
C2849P	South of Ropley Road	Hemmant Branch	3No. 1.05m dia.	Council Stormwater Data
C0516P	Pamela Street	Hemmant catchment	3No. 1.5m dia.	Council Stormwater Data
C3534B	Cleveland Railway near Ulagree Street	Lytton catchment	1No. 1.5m x 0.9m	Assumed dimensions
unknown09	Cleveland Railway near Pritchard Street	Lytton catchment	2No. 1.05m dia.	Assumed dimensions
Unknown10	Cleveland Railway near Pritchard Street	Lytton catchment	2No. 1.05m dia.	Assumed dimensions
Assumed	Private road off Pritchard Street	Drainage along Port of Brisbane Motorway	2No. 0.75m dia.	Assumed dimensions
unknown05	South Street	Drainage near Oil Refinery	2No. 2.7m x 0.9m	Assumed dimensions
unknown08	Oil Refinery road	Drainage near Oil Refinery	1No. 1.8m x 0.9m	Assumed dimensions
unknown07	Oil Refinery road	Drainage near Oil Refinery	1No. 0.9m dia.	Assumed dimensions
unknown06	Lytton Road	Drainage near Oil Refinery	1No. 0.9m dia.	Assumed dimensions
HEMDR_05	Kianawah Road	Hemmant Drain	3No. 1.25m dia.	Tilley Road Model
HEMDR_07	Near Foley Road	Hemmant Drain	1No. 7.7m x 1.75m	Tilley Road Model
HEMDR_08	Youngs Road	Hemmant Drain	3No. 1.2m dia.	Tilley Road Model
HEMDR_01	Wondall Road	Hemmant Drain	4No. 1.35m dia.	Tilley Road Model
HEMDR_02	Wynnum Road	Hemmant Drain	3No. 3m x 1.8m	Tilley Road Model
HEMDR_03	Wynnum Road	Hemmant Drain	1No. 6.7m x 1.5m	Tilley Road Model

Network ID	Crossing Name	Locality	Details	Source
HEMDR_04	Wynnum Road	Hemmant Drain	1No. 1.8m x 1.8m	Tilley Road Model
HEMDR_09	Hemmant and Tingalpa Road	Hemmant Drain	4No. 3m x 1.8m	Tilley Road Model
BRANCH1_01	Kianawah Road	Hemmant Branch	2No. 2.1m x 1.2m	Tilley Road Model
BRANCH1_02	Kianawah Road	Hemmant Branch	1No. 0.75m dia.	Tilley Road Model
BRANCH1_03	Kianawah Road	Hemmant Branch	1No. 1.2m x 0.45m	Tilley Road Model
MAINDR_01	Cleveland Railway	Main Drain	1No. 2.6m x 1.45m	Tilley Road Model
MAINDR_03	Canberra Street	Main Drain	1No. 1.5m dia.	Tilley Road Model
MAINDR_04	Lytton Road	Main Drain	3No. 1.5m dia.	Tilley Road Model
MAINDR_05	Gosport Street	Main Drain	2No. 3.6m x 1.8m	Tilley Road Model
MAINDR_06	Gosport Street	Main Drain	4No. 3.6m x 1.6m	Tilley Road Model
MAINDR_02	Port of Brisbane Motorway	Main Drain	1No. 6m x 1.45m	Tilley Road Model
LINDUM_01	Kianawah Road	Lindum Creek	5No. 1.6m dia.	Tilley Road Model
LINDUM_02	Cleveland Railway	Lindum Creek	2No. 3m x 1.5m	Tilley Road Model
LINDUM_03	Ingham Place	Lindum Creek	3No. 3.6m x 1.2m	Tilley Road Model
LINDUM_04	Port of Brisbane Motorway	Lindum Creek	1No. 10.25m x 2.191m	Tilley Road Model
LINDUM_05	Lytton Road	Lindum Creek	5No. 1.5m dia.	Tilley Road Model
LINDUM_06	Gosport Street	Lindum Creek	4No. 3m x 0.9m	Tilley Road Model
Chan4_POBM	Port of Brisbane Motorway	Lytton Catchment	Bridge Form Loss Coefficient 0.2	Port of Brisbane Motorway Model
Chan4Lytt	Lytton Road	Lytton Catchment	6No. 1.8m dia.	Port of Brisbane Motorway Model
Chan3Ex	Export Street	Lytton Catchment	3No. 2.4m x 1.2m	Port of Brisbane Motorway Model
Chan3Trade	Trade Street	Lytton Catchment	3No. 2.4m x 1.2m	Port of Brisbane Motorway Model

Network ID	Crossing Name	Locality	Details	Source
Ch3N_MOT	Port of Brisbane Motorway	Lytton Catchment	6No. 1.918m dia.	Port of Brisbane Motorway Model
Chan3Lytt	Lytton Road	Lytton Catchment	3No. 2.4m x 1.2m	Port of Brisbane Motorway Model
Ch3_NEW_Lyt	Lytton Road	Lytton Catchment	3No. 2.4m x 1.2m	Port of Brisbane Motorway Model
Ch2Ex	Export Street	Lytton Catchment	2No. 2.4m x 1.2m	Port of Brisbane Motorway Model
Chn2_NEW_CUL	Pritchard Street	Lytton Catchment	3No. 1.918m dia.	Port of Brisbane Motorway Model
Ch2_US	Pritchard Street	Lytton Catchment	2No. 0.75m dia.	Port of Brisbane Motorway Model
Ch2_N_Lyt	Lytton Road	Lytton Catchment	5No. 1.918m dia.	Port of Brisbane Motorway Model
Ch2_N_MOT_ds	Port of Brisbane Motorway	Lytton Catchment	3No. 1.918m dia.	Port of Brisbane Motorway Model
Ch2_N_MOT_u1	Port of Brisbane Motorway	Lytton Catchment	3No. 1.918m dia.	Port of Brisbane Motorway Model
Ch2_N_MOT_u2	Port of Brisbane Motorway	Lytton Catchment	3No. 1.918m dia.	Port of Brisbane Motorway Model

5.2.5 Boundary Conditions

Catchment Runoff

As part of this investigation, the 1997 MDP, sub-catchment delineation was adopted as a basis for this investigation. As such, the sub-catchment delineation is relatively fine with many small sub-catchments in the upper catchment. Hydraulically modelling the small upper sub-catchments would simulate overland flow rather than flooding originating from the waterways. In addition, a number of the small upper sub-catchments fall outside of the waterway corridor; i.e. they fall in parts of the catchment that have been 'filled' for the Ultimate Development Scenario.

To facilitate consistency in the way that flows are applied in the model across the three scenarios (existing case, minimum riparian corridor and ultimate development), the flow derived from many small upper sub-catchments outside the waterway corridor were not routed through the hydraulic model, instead they were routed through the hydrologic model. As such, the hydraulic model is designed primarily to simulate flooding derived from the waterways, and flooding caused from overland flow (before the runoff has reached the waterway) has not been represented.

The first step was to assess the sub-catchments to determine which sections of the catchment would be routed through the hydraulic model. Sub-catchments that intersected with the 1D domain of waterways were applied directly to the underlying 1D channels. Sub-catchments within the waterway area that did not intersect the 1D domain were applied as Source-Area (SA) boundaries in the 2D domain.

Bulimba Creek

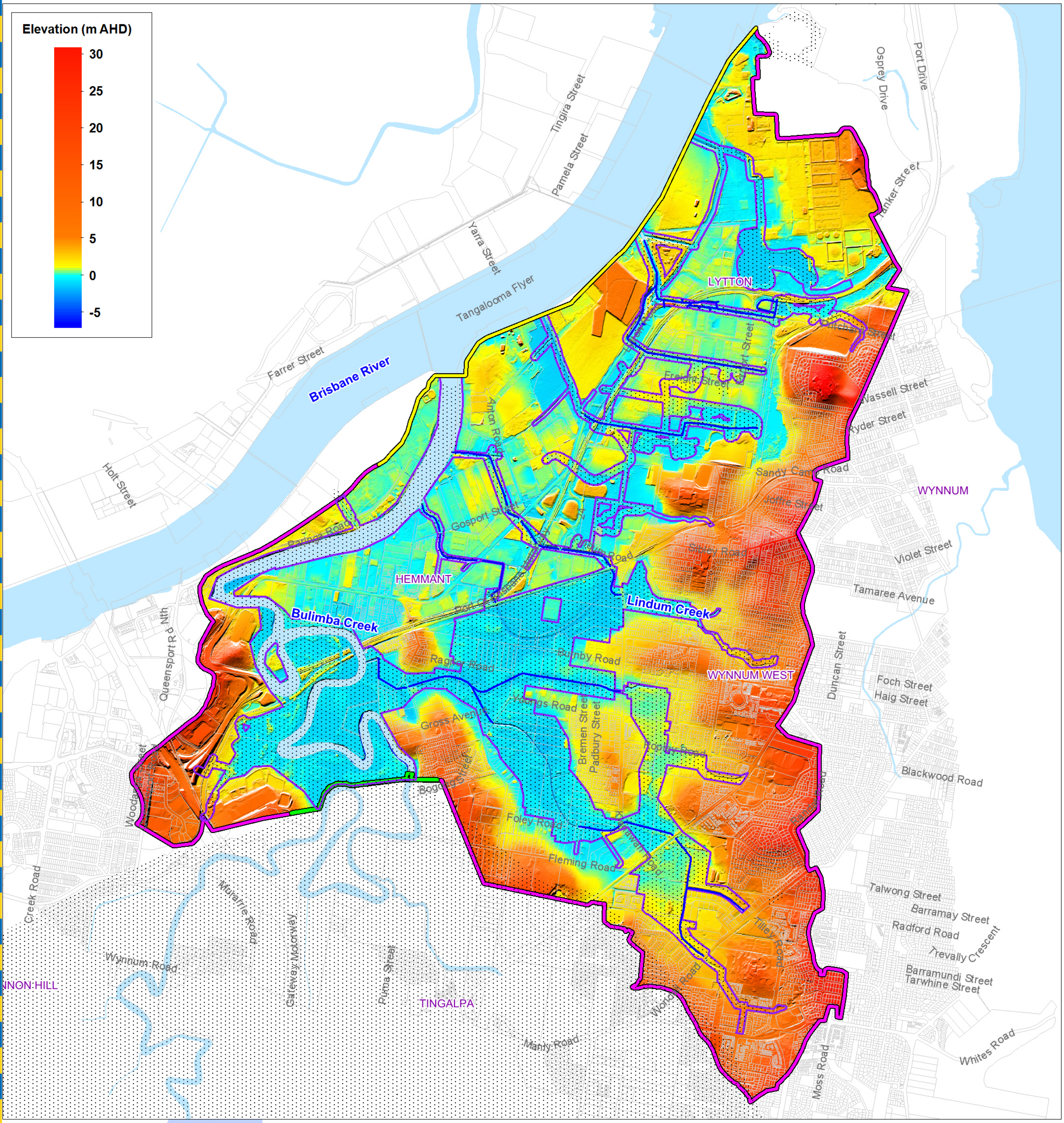
Hemmant Drain drains into the lower reach of Bulimba Creek. The lower reach of Bulimba Creek was included in the hydraulic model in order to simulate the flood behaviour at the confluence. Council provided a WBNM model of the Bulimba Creek catchment and a MIKE11 model of Bulimba Creek. These models were simulated for the same storm events to extract a flow hydrograph for each flood event from the MIKE11 model at the upstream boundary of the Hemmant-Lytton model.

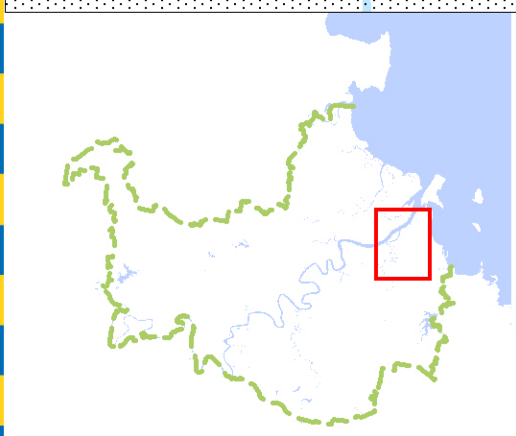
Downstream Boundary

The downstream boundary was located along the right bank of the Brisbane River, where Bulimba Creek and the other waterways outfall into the river. The boundary was set up to linearly interpolate the slight difference in timing (10 minute difference in phase between Pinkenba and Brisbane Bar) and amplitude (level at Pinkenba is 1.02 times level at Brisbane Bar) of the tidal conditions in the river along the boundary. The following boundary conditions were adopted:

- Historical events – recorded water levels at Brisbane Bar
- Design Events (up to 1% AEP) – static Mean High Water Spring (MHWS)
- Extreme Events – static Highest Astronomical Tide (HAT).

For the climate variability assessment sea level rise was added to the conditions listed above (see Section 8.1).





Legend

- Extent of 2D Domain
- Modelled Waterway Corridor
- Council's Waterway Corridor
- Waterway/Waterbody
- Downstream Boundary
- Bulimba Creek Inflow
- 1D Channel

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Prepared : JP
Checked : RS
Revision : 0
Publication Date : 10/10/14
Project Number : B20669

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Hemmant Lytton Hydraulic Model Layout

Figure 5-1

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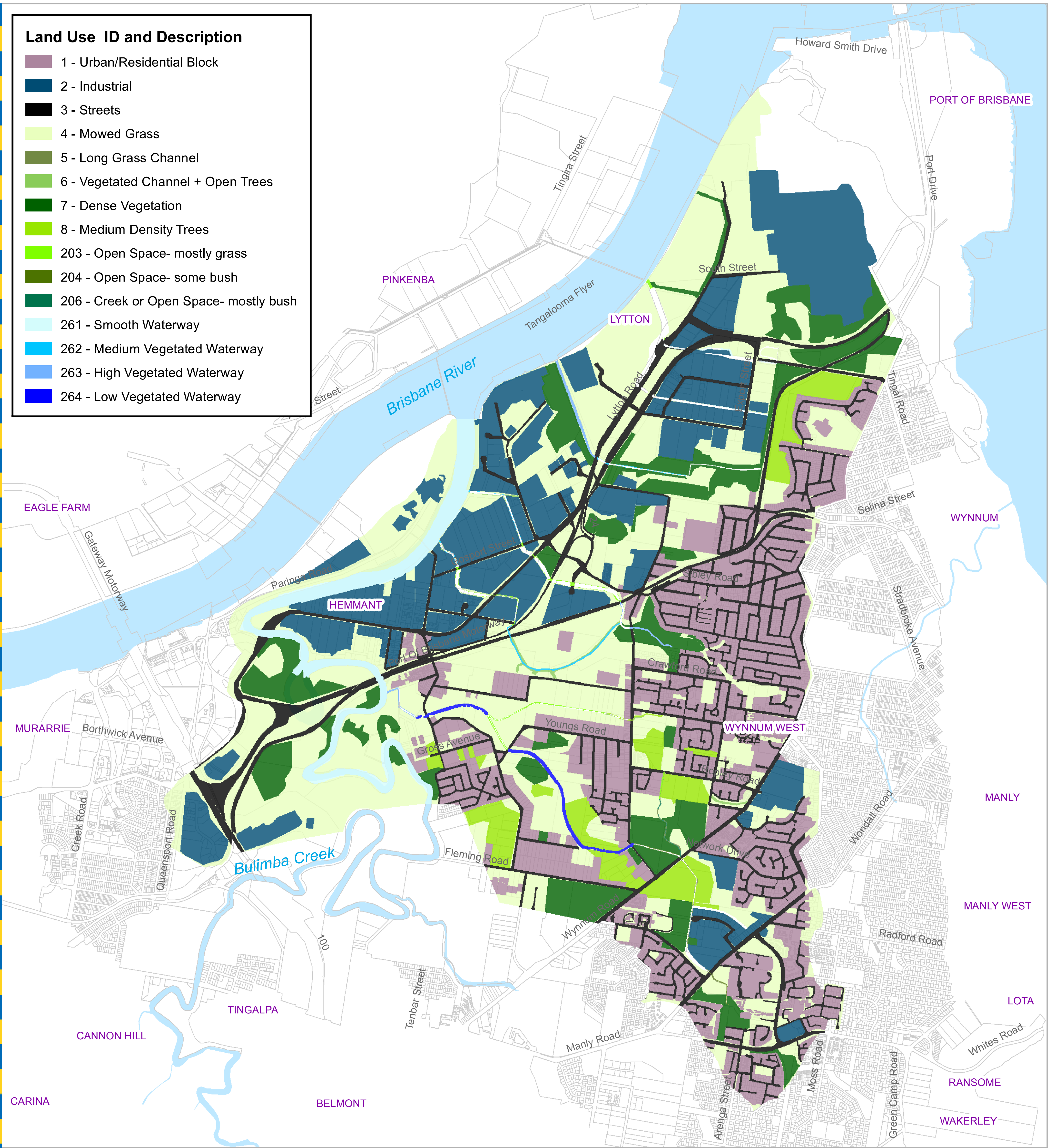

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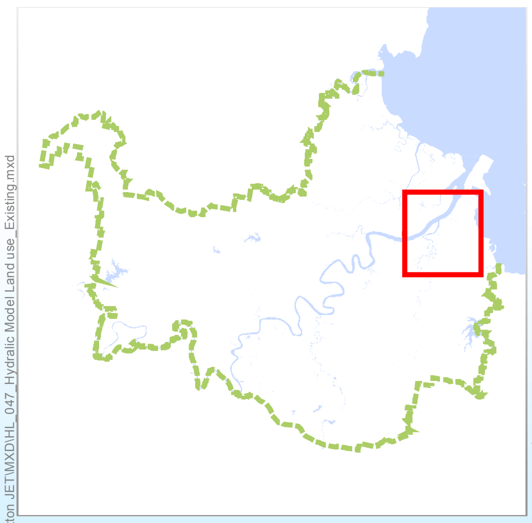

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Hemmant Lytton Hydraulic Model Land Use -Existing

Figure 5-2a

MXD-047

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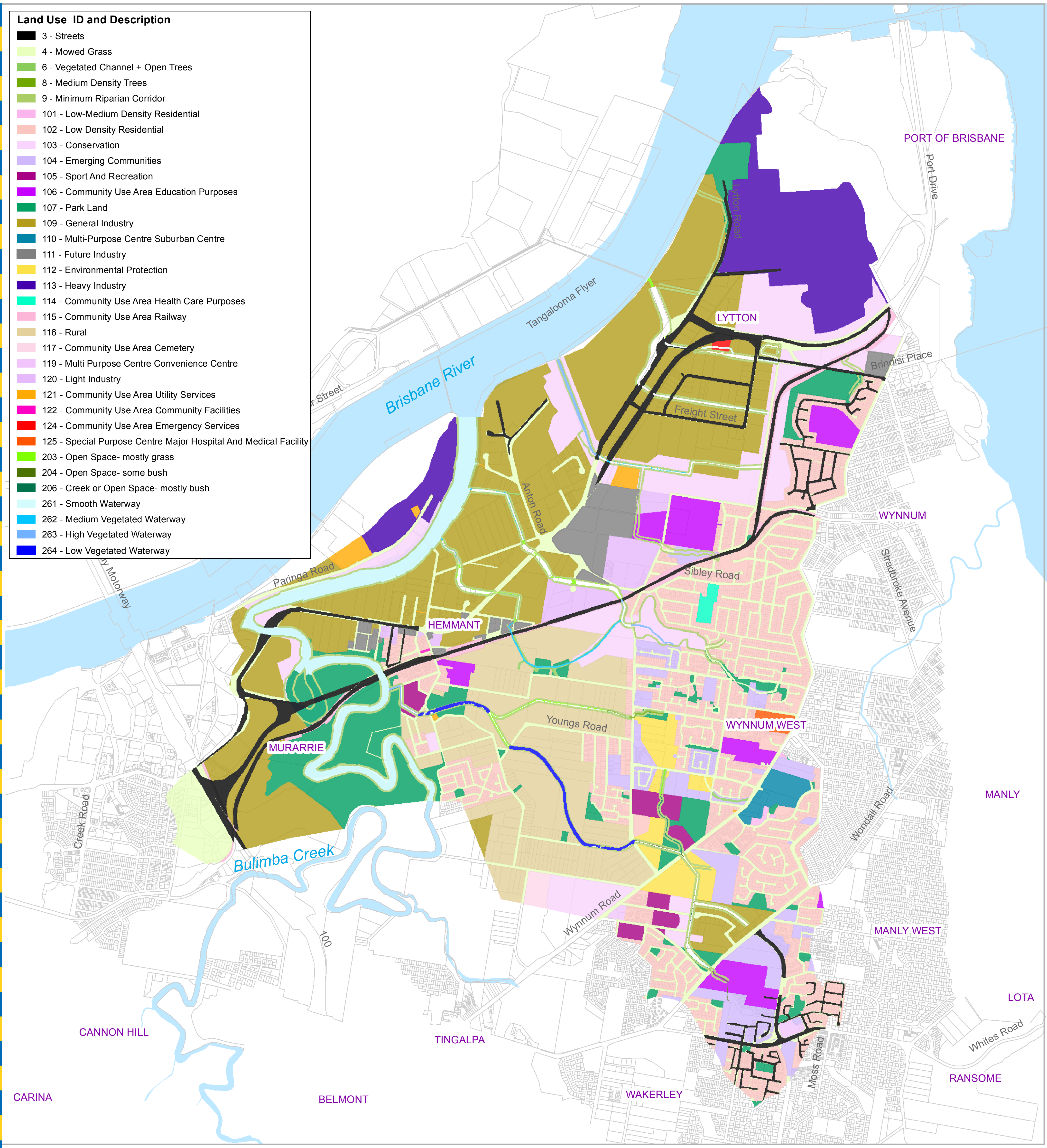

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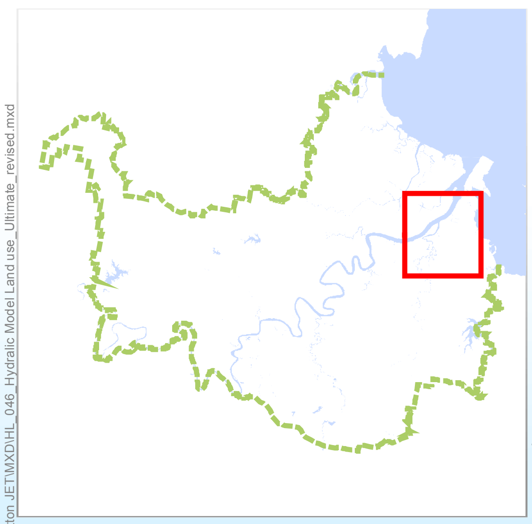

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Hemmant Lytton Hydraulic Model Land Use -Ultimate

Figure 5-2b

MXD-046

5.3 Calibration Procedure

The calibration was undertaken according to the following steps:

1. Make an initial estimate of the model parameters;
2. Run the model and compare the results with the recorded data;
3. Where the model results are beyond Council's tolerances (see below) investigate the potential cause;
4. Where the discrepancy can be attributed to model parameters or schematisation, make adjustments to the models (hydrologic and hydraulic);
5. Rerun the model and compare the results with recorded data; and
6. Return to Step 3.

When comparing the modelled results with recorded data the following tolerances are a guide from Council's flood study procedure:

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

5.4 Hydraulic Model Calibration and Verification Results

5.4.1 January 2013

The results for the January 2013 flood event are shown in Table 5-3.

Table 5-3: Peak Level Comparison for January 2013 Flood Event

	Recorded (mAHD)	Modelled (mAHD)	Difference (m)
MHG 130	3.19	2.82	-0.37
MHG 210	2.15	2.67	0.52
MHG 110	1.68	1.78	0.10
Stream Gauge	1.64	1.56	-0.08

The model under predicts water levels at MHG 130 by 0.37m, which is just outside the tolerance. This gauge is located on Hemmant Drain downstream of Wynnum Road. 400m downstream of this gauge is Kianawah Road. Kianawah Road was overtopped during the January 2013 flood event. The Kianawah Road embankment controls water levels (acting as a weir) in the vicinity of MHG 130 for this event, and:

- Water levels are insensitive to Manning's n.
- Much additional water volume would be required to make up for the 0.37m water level deficit, which cannot be achieved through reduced rainfall losses.

Therefore, it was not possible to adjust the model to reduce this under prediction. The higher recorded water level may be due to fencing along residential properties along Kianawah Road.

The January 2013 flood event is the largest of the three historical events, yet the recorded level at MHG 210 is much lower for the January 2013 event than the recorded levels for the other historical events. Therefore, the over prediction of 0.52m at MHG 210 is believed to be due to an anomaly with the recorded data.

A comparison of the recorded and modelled levels at the stream gauge is shown in Figure 5-3. The timing and peak levels match well. The model over predicts the low tidal levels. This is considered acceptable, since the design event modelling uses a static downstream water level.

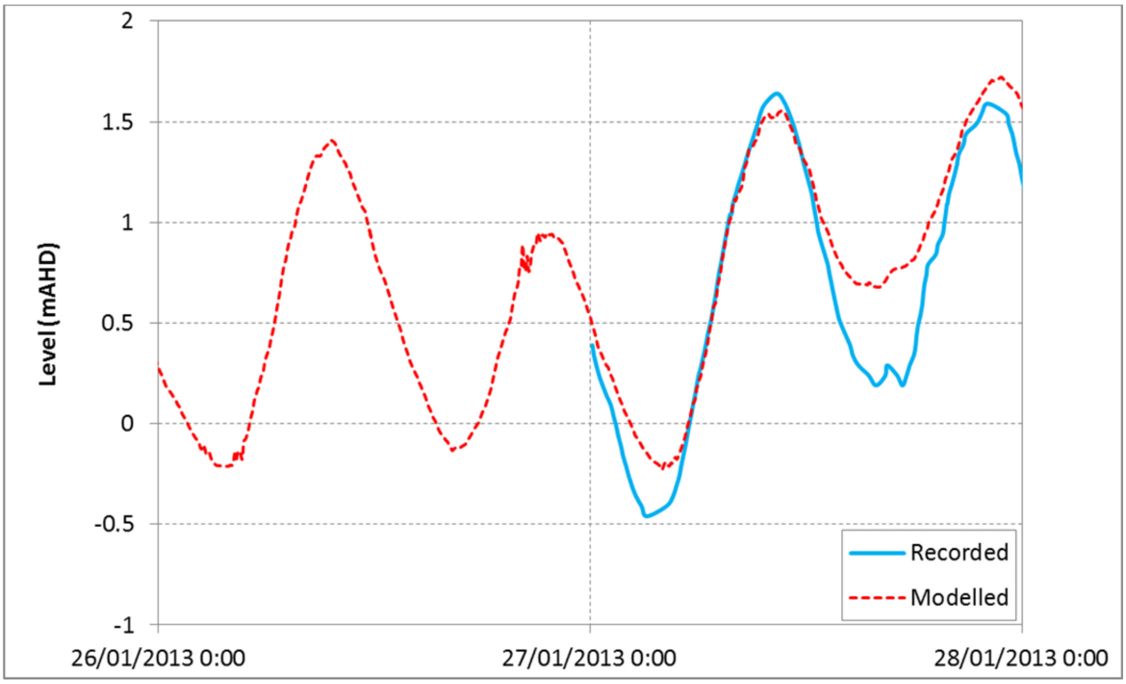


Figure 5-3: Stream Gauge Comparison – January 2013

5.4.2 October 2010

The results for the October 2010 flood event are shown in Table 5-4.

Table 5-4: Peak Level Comparison for October 2010 Flood Event

	Recorded (mAHD)	Modelled (mAHD)	Difference (m)
MHG 130	2.50	2.58	0.08
MHG 210	2.47	2.44	-0.23
MHG 110	1.56	1.67	0.11
Stream Gauge	1.42	1.62	0.20

The modelled peak water levels at the three MHG are within the tolerance of 0.3m. Note that a structure blockage of 20% was applied at Kianawah Road crossing on the Hemmant Drain branch immediately downstream of MHG 210. The modelled stream gauge level is slightly beyond the tolerance of 0.15m. However, the stream gauge level is controlled by the inflows on Bulimba Creek (which is extracted from an external model) and the downstream water level. Therefore, levels at the stream gauge are relatively insensitive to changes in the parameterisation in the Hemmant Drain catchment.

A comparison of the recorded and modelled levels at the stream gauge is shown in Figure 5-4. The timing and peak levels match well. The model over predicts the low tidal levels. This is considered acceptable, since the design event modelling uses a static downstream water level.

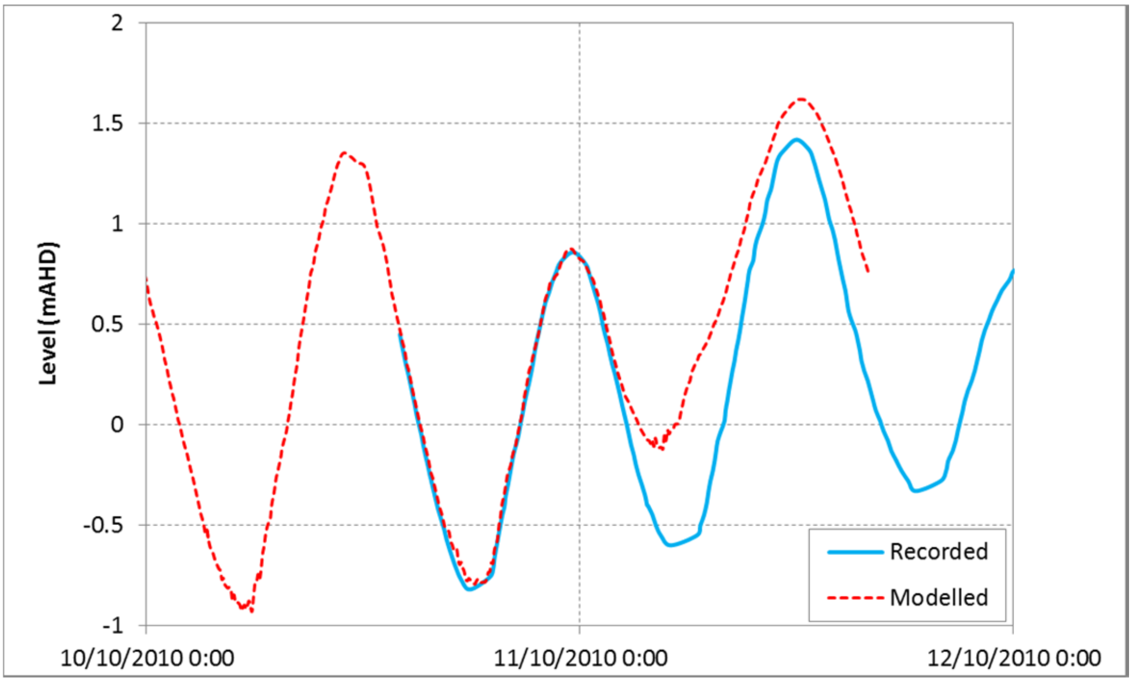


Figure 5-4: Stream Gauge Comparison – October 2010

5.4.3 December 2010

The results for the December 2010 flood event are shown in Table 5-5.

Table 5-5: Peak Level Comparison for December 2010 Flood Event

	Recorded (mAHD)	Modelled (mAHD)	Difference (m)
MHG 130	2.70	2.65	-0.05
MHG 210	2.80	2.24	-0.56
MHG 110	1.78	1.31	-0.47
Stream Gauge	Not captured	1.53	-

The modelled water level at MHG 210 is outside the tolerance. This gauge is located upstream of Kianawah Road on the Hemmant Drain branch. The road level is approximately 2.75mAHD. The under prediction may be due to a blockage of the structure.

The modelled water level at MHG 110 is outside the tolerance. The recorded level at this MHG of 1.78mAHD is higher than for the January 2013 event of 1.56mAHD. Yet the January 2013 event was a larger event. MHG 110 is upstream of Hemmant and Tingalpa Road, which has a road level of approximately 1.7mAHD. The high record at this MHG is may be due to a blockage of the structure.

A comparison of the recorded and modelled levels at the stream gauge is shown in Figure 5-5. The timing and peak levels match well. The model over predicts the low tidal levels. This is considered acceptable, since the design event modelling uses a static downstream water level.

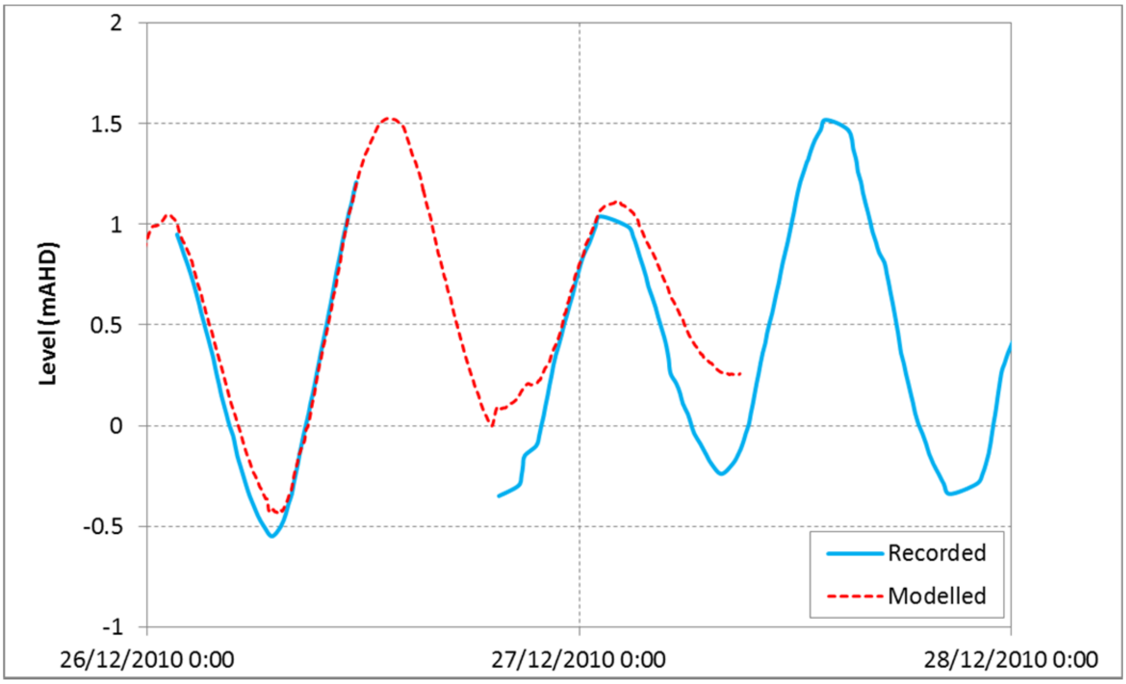


Figure 5-5: Stream Gauge Comparison – December 2010

5.5 Hydraulic Structure Head Loss Verification

It is typical to verify the model results for flow through bridges by comparison with an alternative modelling approach such as HEC-RAS. There is only one bridge in the Hemmant Lytton model. This bridge is located under the Port of Brisbane Motorway. The bridge details were not provided; details were obtained from the pre-existing model of this area. Therefore, it has been assumed that the bridge configuration, as extracted from the pre-existing model, is suitable for this investigation.

5.6 Hydrologic-Hydraulic Model Consistency Check

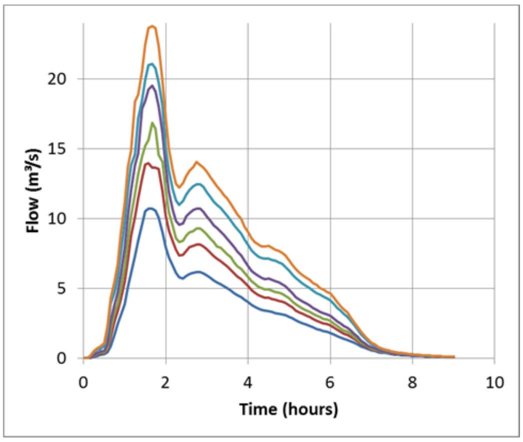
The catchment is highly urbanised with numerous road crossings affecting the hydraulic behaviour. In addition, there are low lying areas receiving water from multiple upper catchments. As such, the hydraulic behaviour of the catchment is complex, and beyond the predictive capability of the simple routing techniques within the hydrologic model. This is why a 2D hydraulic model of the catchment is required, and a good correlation between the hydrologic and hydraulic model flows should not be expected.

Nevertheless, a comparison of flows at selected locations has been made – see Figure 5-6. The following comments are made with respect to this comparison:

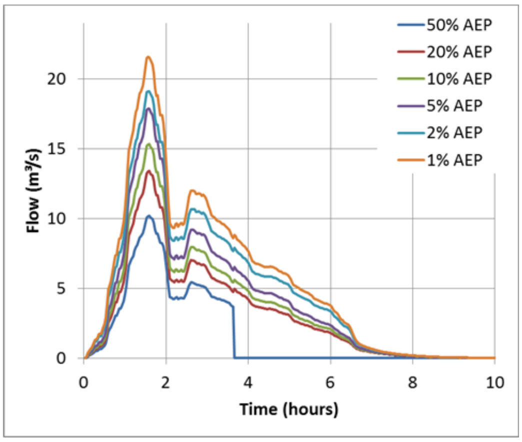
- The locality at Hemmant Drain at Wondall Road (Location 1) is relatively high in the catchment with less upstream hydraulic complexity compared to the other locations. Hence the relatively similar flows.
- In the vicinity of Wynnum Road, Kianawah Road, Hemmant and Tingalpa Road and the Cleveland train line (Locations 2, 3, 6, 7, and 8) the hydrological model has underestimated attenuation – TUFLOW flows are lower than the XP-RAFTS flows. This is expected, due to the large volume of floodplain storage upstream of the structures. Additional storage could be added to the hydrology model in an attempt to improve the comparison. This was not done due to the complexity of the storage area, which links multiple waterway catchments and has multiple outlets.
- In the vicinity of Kianawah Road on the Hemmant Drain Branch and Lindum Creek and at Export Street in the Lytton area (Locations 4, 5 and 9), the TUFLOW flows are larger than the XP-RAFTS model. Upstream of these locations the hydraulic behaviour is complex. These differences may be due to upstream flow ‘splits’ differing from that predicted in the hydrologic model.

1. **Hemmant Drain Downstream of Wondall Road**

TUFLOW

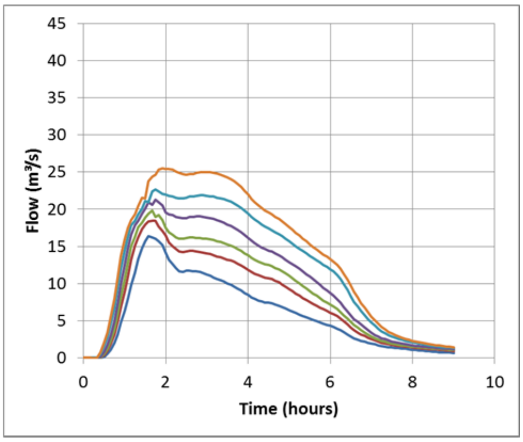


XP-RAFTS

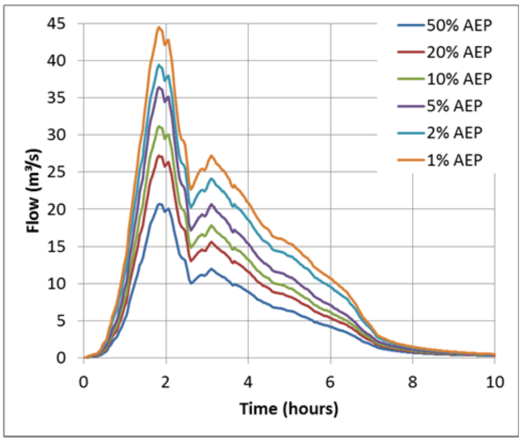


2. **Hemmant Drain Upstream of Wynnum Road**

TUFLOW

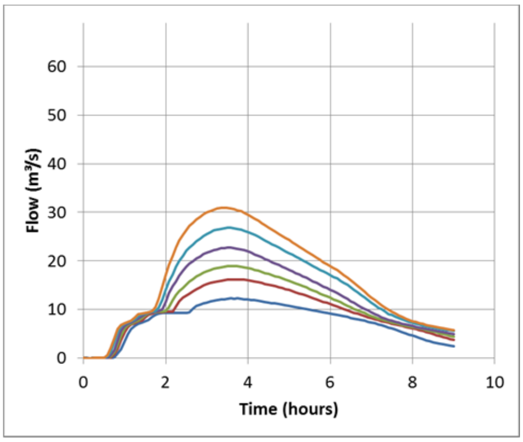


XP-RAFTS

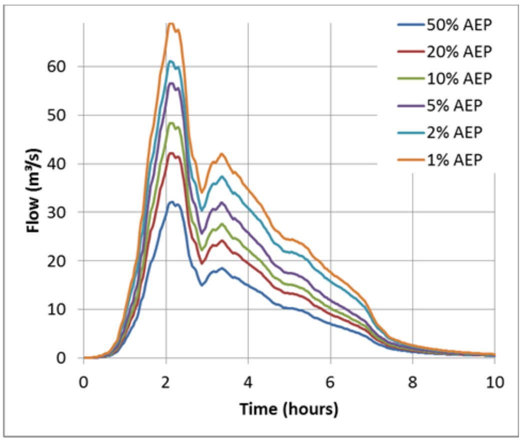


3. **Hemmant Drain at Kianawah Road**

TUFLOW

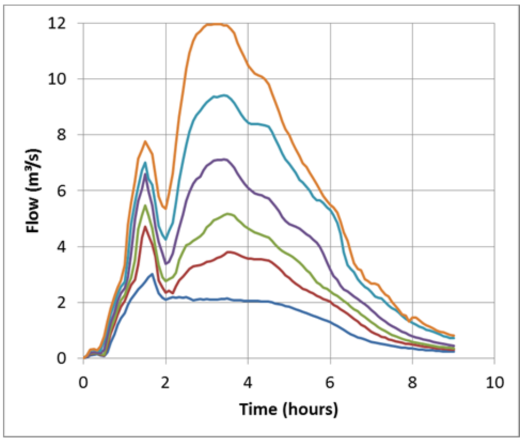


XP-RAFTS

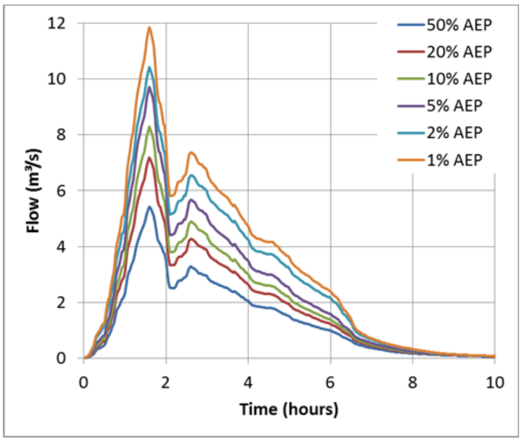


4. Hemmant Drain Branch Downstream of Kianawah Road

TUFLOW

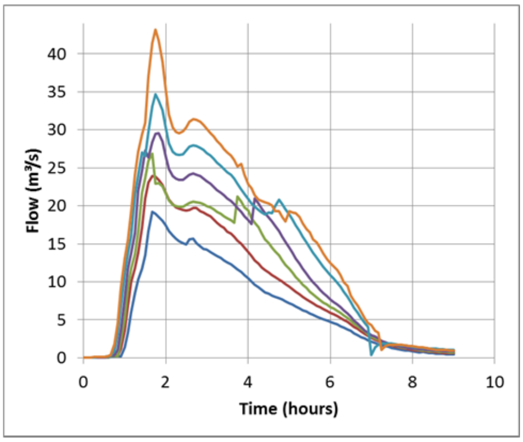


XP-RAFTS



5. Lindum Creek Upstream of Kianawah Road

TUFLOW



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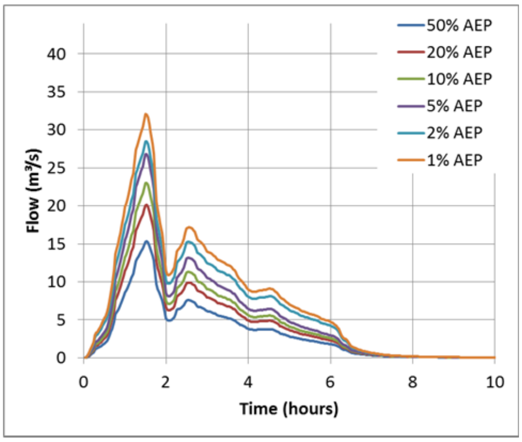
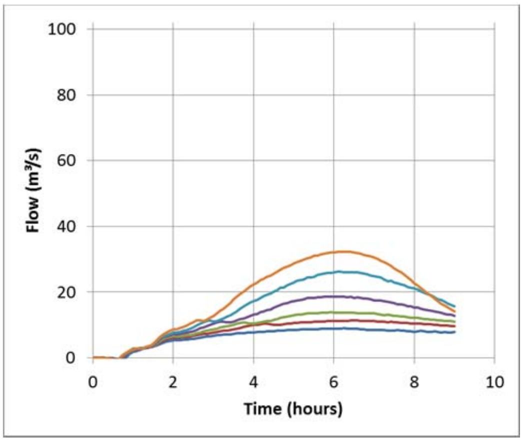
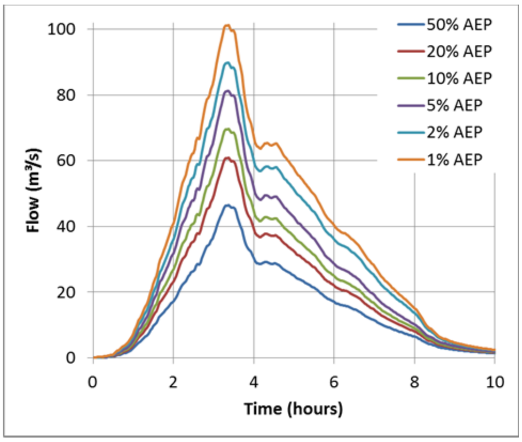


6. Hemmant Drain at Hemmant and Tinglapa Road

TUFLOW

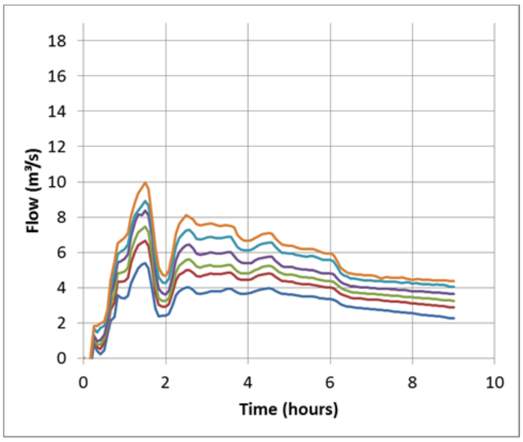


XP-RAFTS

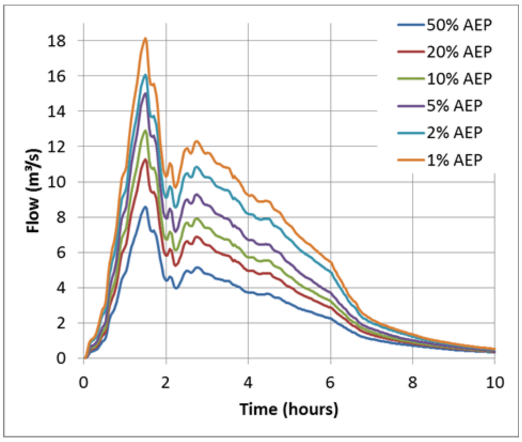


7. Main Drain at Gosport Street

TUFLOW

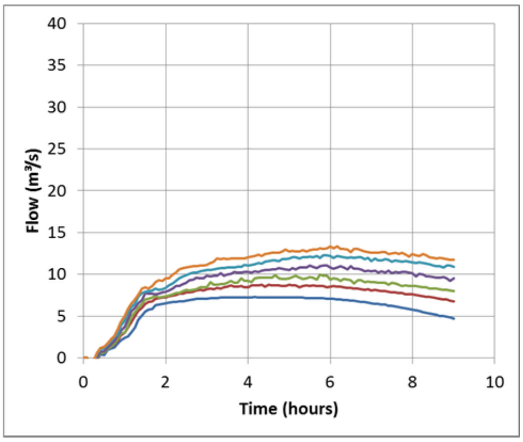


XP-RAFTS

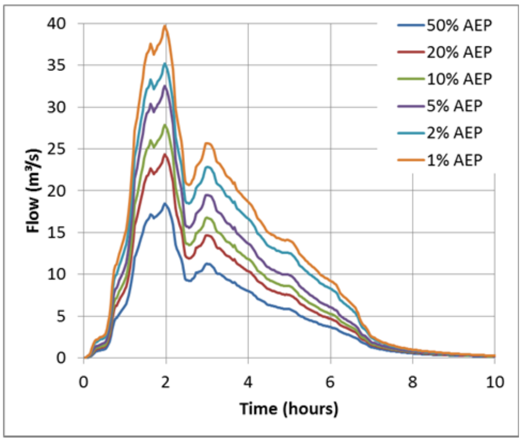


8. Lindum Creek at Gosport Street

TUFLOW

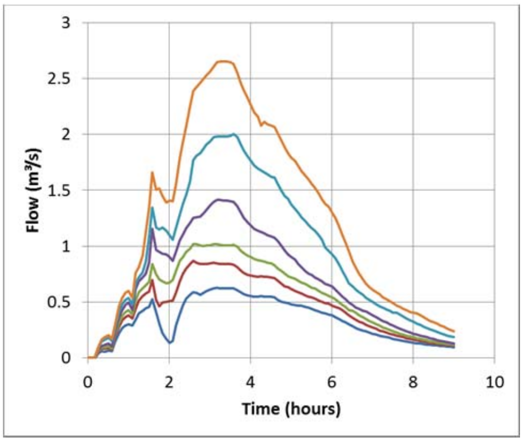


XP-RAFTS



9. Southern Lytton Drain at Export Street

TUFLOW



XP-RAFTS

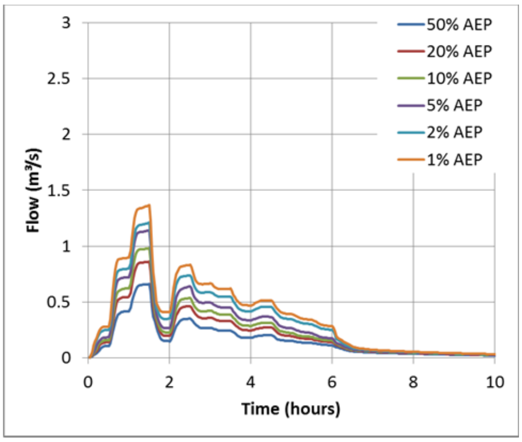


Figure 5-6: Hydrology and Hydraulic Models Consistency Check for 9 Locations

6.0 Design Event Analysis

6.1 Design Event Scenarios

The hydraulic model was used to determine both discharges and flood levels for the 50%, 20%, 10%, 5%, 2% and 1% year AEP events. These events were simulated for durations from 30 minutes to 24 hours.

The following design event scenarios were simulated in the hydrologic and hydraulic models:

- Scenario 1: Existing Waterway Conditions
- Scenario 2: Minimum Riparian Corridor (MRC)
- Scenario 3: Filling to the Waterway Corridor (WC) + Minimum Riparian Corridor (MRC).

6.2 Waterway Corridor

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

"The corridors along a waterway indicated on the Planning Scheme maps. These corridors are defined by:

- A flood regulation line (FRL)
- A local plan environmental corridor or a waterway corridor (WC)
- A waterway corridor defined in a stormwater management plan
- A waterway corridor defined in a waterway management plan.

If more than one of these is available for a particular waterway, the largest applies.

If there is no FRL described in local plan, SMP or WMP, a 30 metre distance measured on each side from the centre line of the waterway would apply" (Brisbane City Council Plan 2000, vol. 1, ch. 3, p. 75).

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

Waterway corridors are represented in the hydraulic model by the exclusion of the conveyance and/or water storage characteristics of the watercourse beyond the limits of the waterway corridor location. Essentially, this practice assumes that filling and development will ultimately occur beyond the boundary of the waterway corridors.

The waterway corridors have been included in the hydraulic models for the Ultimate Scenario flood events. Traditionally, the inclusion of waterway corridors within the hydraulic model was simulated by 'walling off' the zone outside of the waterway corridor, as shown in Figure 5.5.

Note: Best practise suggests that an appropriate Manning's roughness value be applied to these 'walls' (i.e. not assumed to be frictionless) to ensure correct calculation of wetted perimeter at each cross-section.

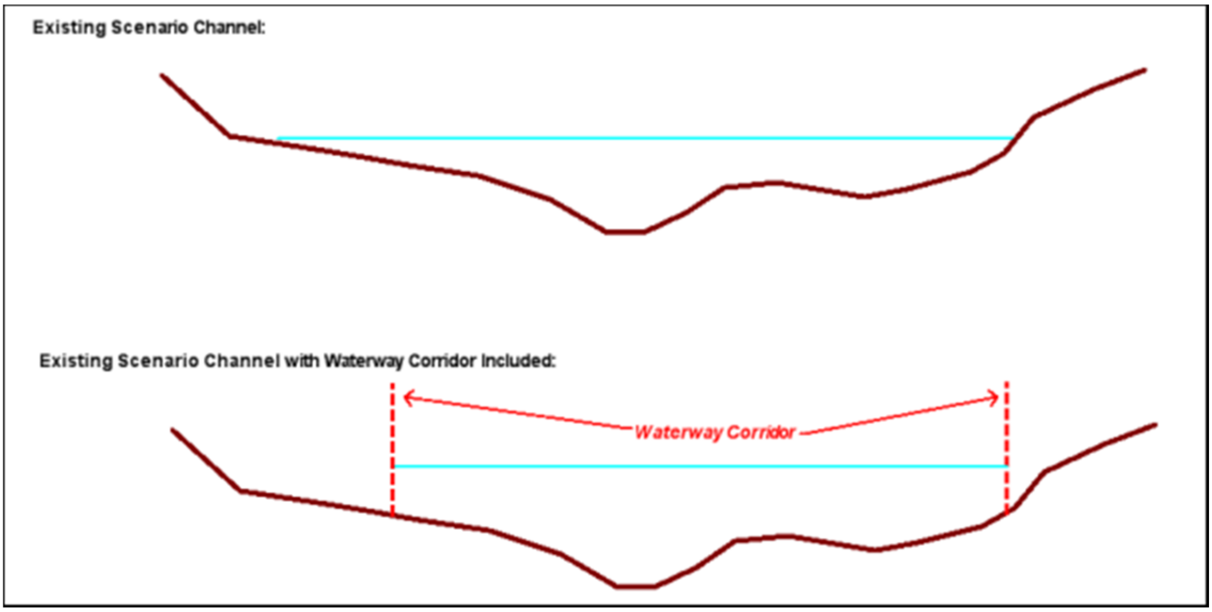


Figure 6-1: Implementation of Waterway Corridor using 'Walling Off' Method

6.3 Minimum Riparian (Vegetated) Corridor

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

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

Minimum vegetated riparian corridors have been applied to the main channels modelled in the hydraulic model. The minimum vegetated riparian corridors were simulated as dense vegetation (i.e. Manning's n value of 0.15) extending from the top of the low flow channel for a minimum width of 15 m on both sides of the creek. Where there is no obvious low flow channel, the vegetation was applied at the anticipated 50% AEP flood level on the basis that this size event is generally contained within the bed and banks of the low flow channel.

6.4 Design Hydrology

AR&R was used to develop the design storms for the design events. The Bulimba Creek model included six locations where IFD data were used to establish the design storms. One central location was used in the Hemmant-Lytton catchment to derive IFD data for the design storms.

The hydrology model's percentage impervious and Manning's n values were updated to represent ultimate catchment conditions for all design event scenarios. A conservative approach was adopted for the rainfall losses, whereby zero rainfall losses were applied for all design events for both initial and continuing losses.

6.5 Design Hydraulics

The calibrated model was used as the basis for the design event modelling. The model was updated as follows:

- Scenario 1: no updates were made to the calibration model as it represented existing catchment conditions.
- Scenario 2: An additional MRC materials layer was added to the model. This layer covered 15m on either side of waterways and a Manning's n of 0.15 was used within the MRC.
- Scenario 3: The Scenario 2 model was updated by including a terrain modifier that filled all areas outside the waterway corridor.

The downstream boundary water level was set to a static MHWS level for design events up to 1% AEP (0.93mAHD at Brisbane Bar).

6.6 Design Event Results and Mapping

A summary of all design events simulated for this investigation is presented in Table 6-1 to Table 6-3 for each Scenario (see Section 6.1). More details on the rare events setup is outlined in Section 7.

Table 6-1: Scenario 1 Design Simulation Summary

Event (AEP) \ Duration (mn)	30	60	90	120	180	270	360	540	720	1080	1440
50%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
20%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
10%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
5%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
2%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
1%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
0.5%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
0.2%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
0.05%	N.A	N.A	N.A	N.A	N.A	N.A	✓	N.A	N.A	N.A	N.A
PMF	N.A	N.A	N.A	N.A	N.A	N.A	✓	N.A	N.A	N.A	N.A

N.A: Not Applicable as not required.

Table 6-2: Scenario 2 Design Simulation Summary

Event (ARI) \ Duration (mn)	30	60	90	120	180	270	360	540	720	1080	1440
1%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

Table 6-3: Scenario 3 Design Simulation Summary

Event (ARI) \ Duration (mn)	30	60	90	120	180	270	360	540	720	1080	1440
50%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
20%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
10%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
5%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
2%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
1%	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

For the 50%, 20%, 10%, 5%, 2% and 1% AEP design events under Scenario 1, the mean peak flood levels were extracted along a number of cross-sections and results are presented in Appendix D.

A summary of the flood maps produced for this investigation is presented in Table 6-4. For each map, an envelope of the maximum results from all the simulated storm durations as outlined in Table 6-1 was performed.

Table 6-4: Design Events Mapping Summary

Map Reference	Design Event (AEP)	Output Type	Scenario
F.1a, F.1b, F.1c, F.1d	50%	Peak Water Level	1
F.2a, F.2b, F.2c, F.2d	20%	Peak Water Level	1
F.3a, F.3b, F.3c, F.3d	10%	Peak Water Level	1
F.4a, F.4b, F.4c, F.4d	5%	Peak Water Level	1
F.5a, F.5b, F.5c, F.5d	2%	Peak Water Level	1
F.6a, F.6b, F.6c, F.6d	1%	Peak Water Level	1
F.7a, F.7b, F.7c, F.7d	0.5%	Peak Water Level	1
F.8a, F.8b, F.8c, F.8d	0.2%	Peak Water Level	1

6.6.1 Return Periods of Historic Events

As outlined in Section 3.2, three Maximum Height Gauge (MHG) data were provided for the calibration and validation flood events, namely:

- Hemmant Channel (HM110) – located upstream of Hemmant and Tingalpa Road crossing on Hemmant Drain 500m upstream of its confluence with Bulimba Creek.
- Hemmant Channel (HM130) – located downstream of Wynnum Road on Hemmant Drain.
- Hemmant Channel (HM210) – located upstream of Kianawah Road on a tributary to Hemmant Drain.

The model was calibrated against recorded levels at the three gauges above mentioned, for three flood events: January 2013, October 2010 and December 2013. For each calibration event, Table 6-5 to 6-7 provides a comparison of recorded level to closest modelled level and associated design event.

Table 6-5: January 2013 Flood Event – MHG Return Period Comparison

	Recorded (mAHD)	Level Closest Event	Closest event (AEP)
MHG 130	3.19	3.17	1%
MHG 210	2.15	2.71	50%
MHG 110	1.68	1.66	50%

Table 6-6: October 2010 Flood Event – MHG Return Period Comparison

	Recorded (mAHD)	Level Closest Event	Closest event (AEP)
MHG 130	2.50	2.77	50%
MHG 210	2.47	2.71	50%
MHG 110	1.56	1.66	50%

Table 6-7: December 2010 Flood Event – MHG Return Period Comparison

	Recorded (mAHD)	Level Closest Event	Closest event (AEP)
MHG 130	2.70	2.77	50%
MHG 210	2.80	2.71	50%
MHG 110	1.78	1.79	20%

The MHG return period comparison show that all the three calibration events were generally below a 50% AEP event which is consistent with the analysis presented in Section 3.6 (comparison of recorded rainfall with AR&R IFD curves). For the January 2013 event however, a discrepancy is observed for Gauge MHG 130 where the recorded level is close to a 1% AEP design event, whereas the other two gauges are closer to a 50% AEP event. However, the calibration model under predicted levels by 0.37m at the MHG 130 gauge for the January 2013 flood event. Here the flood levels are controlled by the downstream Kianawah Road embankment overflow level. The same model was used to calibrate all of the three events, and the setup that provided the best overall fit was selected. Local changes for one particular event that may have altered the flood behaviour (such as fences blockage etc.) were not replicated by the model. This is also outlined in Section 5.4.1.

6.6.2 Flood Immunity of Existing Crossings

Hydraulic Structure Reference Sheets (HSRS) have been completed for 21 major structures with the Hemmant – Lytton catchment which present detailed information of the structures including their flood immunity. The HSRS are presented in Appendix C.

7.0 Rare and Extreme Event Analysis

The 0.5%, 0.2%, 0.05% AEP and PMF flood events have been simulated for Scenario 1. While Council usually simulate these events for Scenario 3, for this investigation this was not done due to the difficulty in the 'stretching' process that is needed to fill the floodplain to the 1% AEP level plus freeboard.

CRC Forge was used to determine the rainfall depths and the AR&R temporal patterns were used for the 0.5% and 0.2% AEP events (see Table 5-5 for list of the rainfall depths). For the 0.05% AEP and PMF event, Council supplied the design storm event based on their 6 hour superstorm methodology (Appendix G) (see Table 7-2 for the 'superstorm' rainfall depths and Figure 7-1 and Figure 7-2 for the rainfall profiles).

Highest Astronomical Tide (HAT) levels were adopted for the downstream boundary conditions on the Brisbane River (1.49mAHD at Brisbane Bar).

Table 7-1: Rare Event Rainfall Depths

Duration (hours)	0.5% AEP (mm)	0.2% AEP (mm)
0.5	84	95
1	118	134
1.5	137	156
2	153	174
3	177	201
4.5	204	233
6	226	259
9	228	262
12	290	333
18	350	404
24	401	463

Table 7-2: 'Superstorm' Rainfall Depths

	Total 6 hour 'Superstorm' Rainfall Depth (mm)
0.05% AEP	340
PMP	816

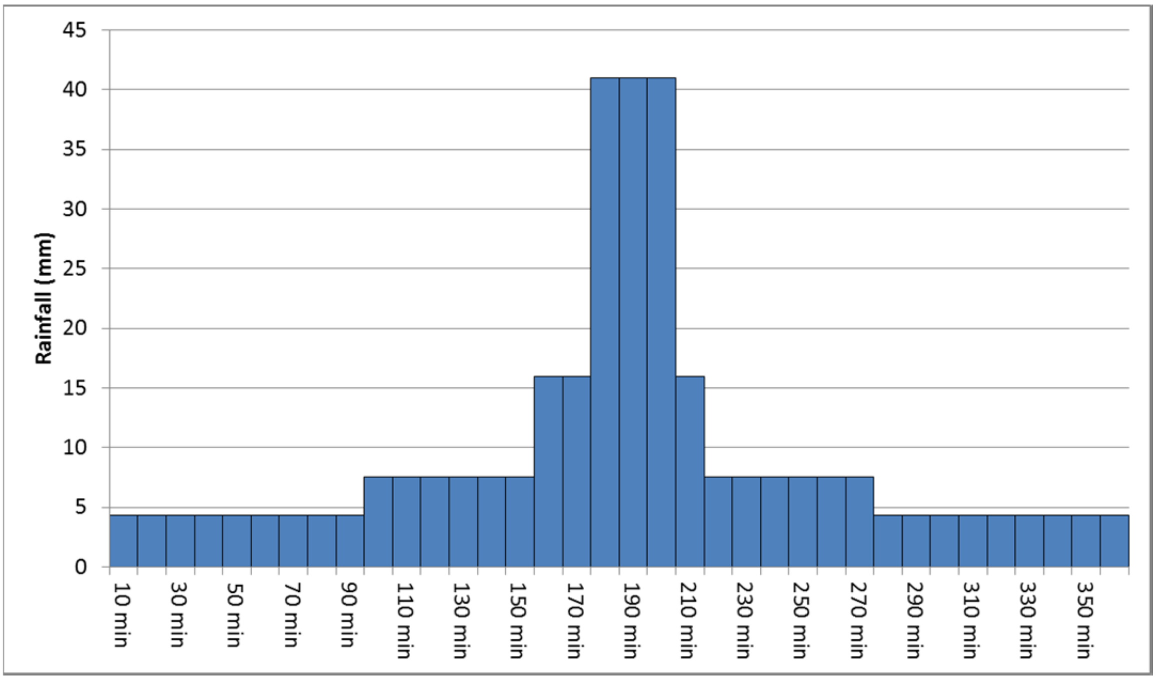


Figure 7-1: 0.05% AEP Rainfall Profile Plot

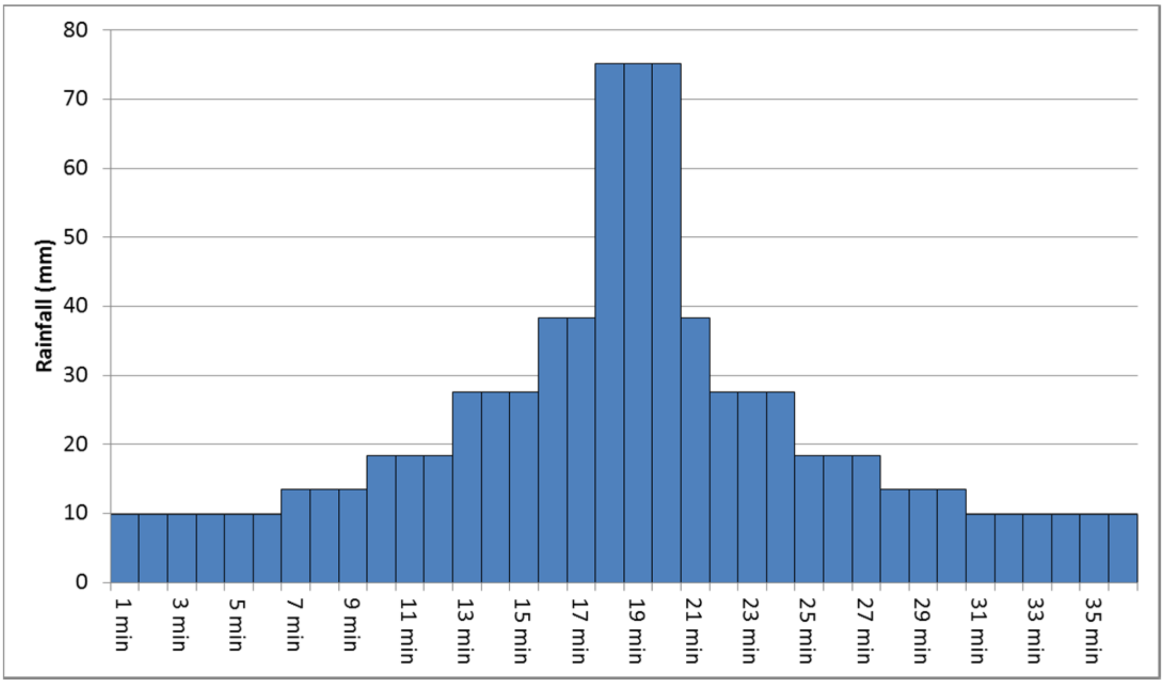


Figure 7-2: PMP Rainfall Profile Plot

8.0 Sensitivity Analysis

8.1 Climate Variability

Two climate change horizons have been considered: 2050 and 2100. The adopted assumptions are in line with a state government report on climate change in Queensland (DERM et al., 2010). The adopted climate change assumptions are:

- 2050: 10% increase in rainfall intensity and 300mm increase in mean sea level
- 2100: 20% increase in rainfall intensity and 800mm increase in mean sea level.

As the existing Bulimba Creek model had not been run with these assumptions for the design events, a simplified approach was adopted for the inflow from Bulimba Creek. Whereby, the flow was increased by 10% and 20% for the 2050 and 2100 horizon respectively. This approach is considered suitable, as the flows in the lower Bulimba Creek are largely controlled by the water level at the Brisbane River.

The 1% and 0.5% AEP design events were simulated for the existing scenario for both the 2050 and 2100 horizons, and the 0.2% AEP design event was simulated for the 2100 horizon.

8.2 Structure Blockage

A structure blockage assessment was carried out in line with the provisional 2013 edition of QUDM (DEWS, 2013). QUDM recommends a culvert inlet blockage of 20% for unscreened culverts with width of less than 5m and 10% for unscreened culvert inlets with width of greater than 5m. Primary structures were selected for the assessment and grouped into three:

1. Blockage of culverts along Lytton Road and Wondall Road;
2. Blockage of culverts along Kianawah Road; and
3. Blockage of culverts along Cleveland Railway line, at Hemmant and Tingalpa Road and the northern Port of Brisbane Motorway.

The groups were selected to ensure that the additional attenuation caused by blockages did not influence the assessment of blockage at culverts further downstream. All blockages were 20%, and the selected structures are listed in Table 8-1.

The assessment was undertaken on the existing scenario for the 1% AEP design flood event.

Table 8-1: List of Blocked Structures

Network ID	Crossing Name	Locality	Details	Group
HEMDR_01	Wondall Road	Hemmant Drain	4No. 1.35m dia.	1
BRANCH1_02	Kianawah Road	Hemmant Branch	1No. 0.75m dia.	1
BRANCH1_03	Kianawah Road	Hemmant Branch	1No. 1.2m x 0.45m	1
MAINDR_04	Lytton Road	Main Drain	3No. 1.5m dia.	1
LINDUM_05	Lytton Road	Lindum Creek	5No. 1.5m dia.	1
Chan4Lytt	Lytton Road	Lytton Catchment	6No. 1.8m dia.	1
Chan3Lytt	Lytton Road	Lytton Catchment	3No. 2.4m x 1.2m	1
Ch3_NEW_Lyt	Lytton Road	Lytton Catchment	3No. 2.4m x 1.2m	1
Ch2_N_Lyt	Lytton Road	Lytton Catchment	5No. 1.918m dia.	1
HEMDR_05	Kianawah Road	Hemmant Drain	3No. 1.25m dia.	2
BRANCH1_01	Kianawah Road	Hemmant Branch	2No. 2.1m x 1.2m	2
LINDUM_01	Kianawah Road	Lindum Creek	5No. 1.6m dia.	2
HEMDR_09	Hemmant and Tingalpa Road	Hemmant Drain	4No. 3m x 1.8m	3
MAINDR_01	Cleveland Railway	Main Drain	1No. 2.6m x 1.45m	3
LINDUM_02	Cleveland Railway	Lindum Creek	2No. 3m x 1.5m	3
Ch3N_MOT	Port of Brisbane Motorway	Lytton Catchment	6No. 1.918m dia.	3
Ch2_N_MOT_ds	Port of Brisbane Motorway	Lytton Catchment	3No. 1.918m dia.	3
Ch2_N_MOT_u1	Port of Brisbane Motorway	Lytton Catchment	3No. 1.918m dia.	3
Ch2_N_MOT_u2	Port of Brisbane Motorway	Lytton Catchment	3No. 1.918m dia.	3

9.0 Summary of Study Findings

This flood investigation has estimated the hydraulic behaviour of flood waters through the study area associated with the design flood events and historical events that were assessed. The model is designed to assess large flood events originating from the watercourses, and was based on information provided at the time of the investigation. The following should be considered for future use of the model:

- Future development may influence the results presented in this study. Before the model is used, any changes in land use and topography should be considered and the model adapted as required.
- Flooding from sources other than watercourses has not been simulated in this study (such as overland flow).
- The lower Bulimba Creek has been included in the model extent in order to resolve tail water conditions on Hemmant Drain, Main Drain and Lindum Creek. Limitations pertaining to the model results along Bulimba Creek are as follows:
    - A review of the Bulimba Creek model was beyond the scope of this investigation. Therefore, the Bulimba Creek model has been assumed suitable for use in the current investigation.
    - It is noted that the Bulimba Creek model is a 1D model and that the 1D cross-sections were not wide enough to accurately simulate large flood events. However, since the focus of this investigation is on the Hemmant Drain, and the lower Bulimba Creek levels are largely controlled by water levels in the Brisbane River, this fact is not considered to detract from the outcomes of the current investigation.
    - A general assumed channel profile along Bulimba Creek has been cut into the model topography based on the channel invert and cross-section area in the existing MIKE11 model.
    - The hydraulic model simulations have been set up to capture the peak flood levels in the investigation catchments. Bulimba Creek is not an investigation catchment, and for many design events the peak flood levels have not been reached along the full extent of Bulimba Creek.
- The TUFLOW model has been based on pre-existing TUFLOW models. It has been assumed that the information on channel and structures in the pre-existing models is correct.

10.0 References

Aurecon (2012), Tilley Road Extension – Stage 2, Flood Mitigation Strategy Hydraulic Assessment Report, prepared for Brisbane City Council by Aurecon, 18 May 2012

BCC (2011), Bulimba Creek Flood Study (Draft), Brisbane City Council, Water and Environment, City Design, June 2011

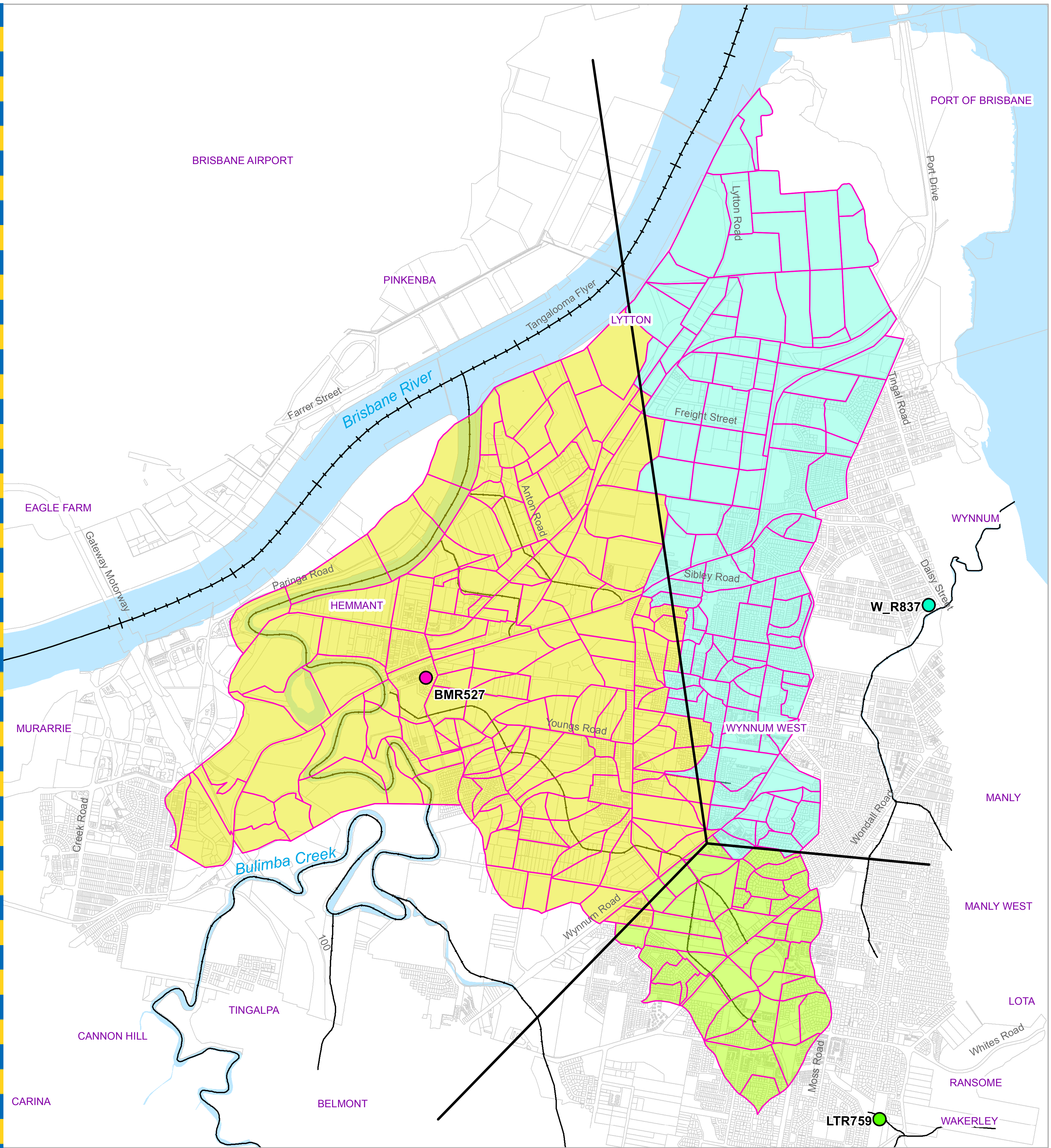
DERM et al. (2010), Increasing Queensland's resilience to inland flooding in a changing climate: Final report on the Inland Flooding Study, Queensland Government, 2010

DEWS (2013), Queensland Urban Drainage Manual, Third Edition 2013-provisional, Department of Energy and Water Supply, April 2013

L&T (1997), Master Drainage Plan & Flood Study, Hemmant-Wynnum West Area, Prepared for Brisbane City Council by Lawson and Treloar in association with John Wilson and Partners, August 1997

APPENDICES

APPENDIX A - Rainfall Distribution Maps



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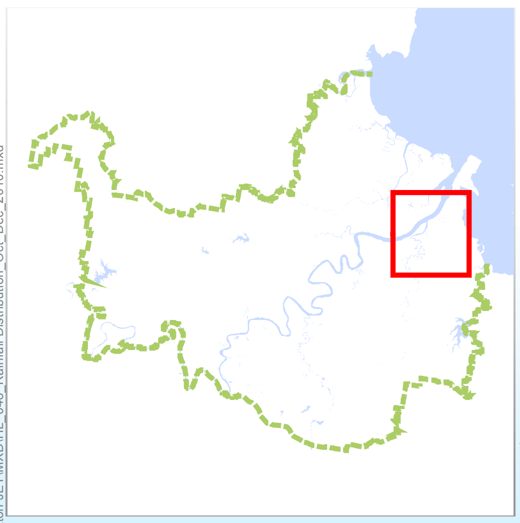

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Prepared : Jayesh.Patel
Checked : RS
Revision : 0
Publication Date : 25 Nov 2014
Project Number : B20669

File : I:\B20669_I_BRH Hemmant Lytton JET\MXD\HL_048_Rainfall Distribution_Oct_Dec_2010.mxd

Legend

- Rainfall Distribution Boundary
- Sub-catchments
- Sub-catchments for LTR759
- Sub-catchments for W_R837
- Sub-catchments for BMR527
- Waterway/Waterbody
- BMR527
- LTR759
- W_R837
- AMTD Line

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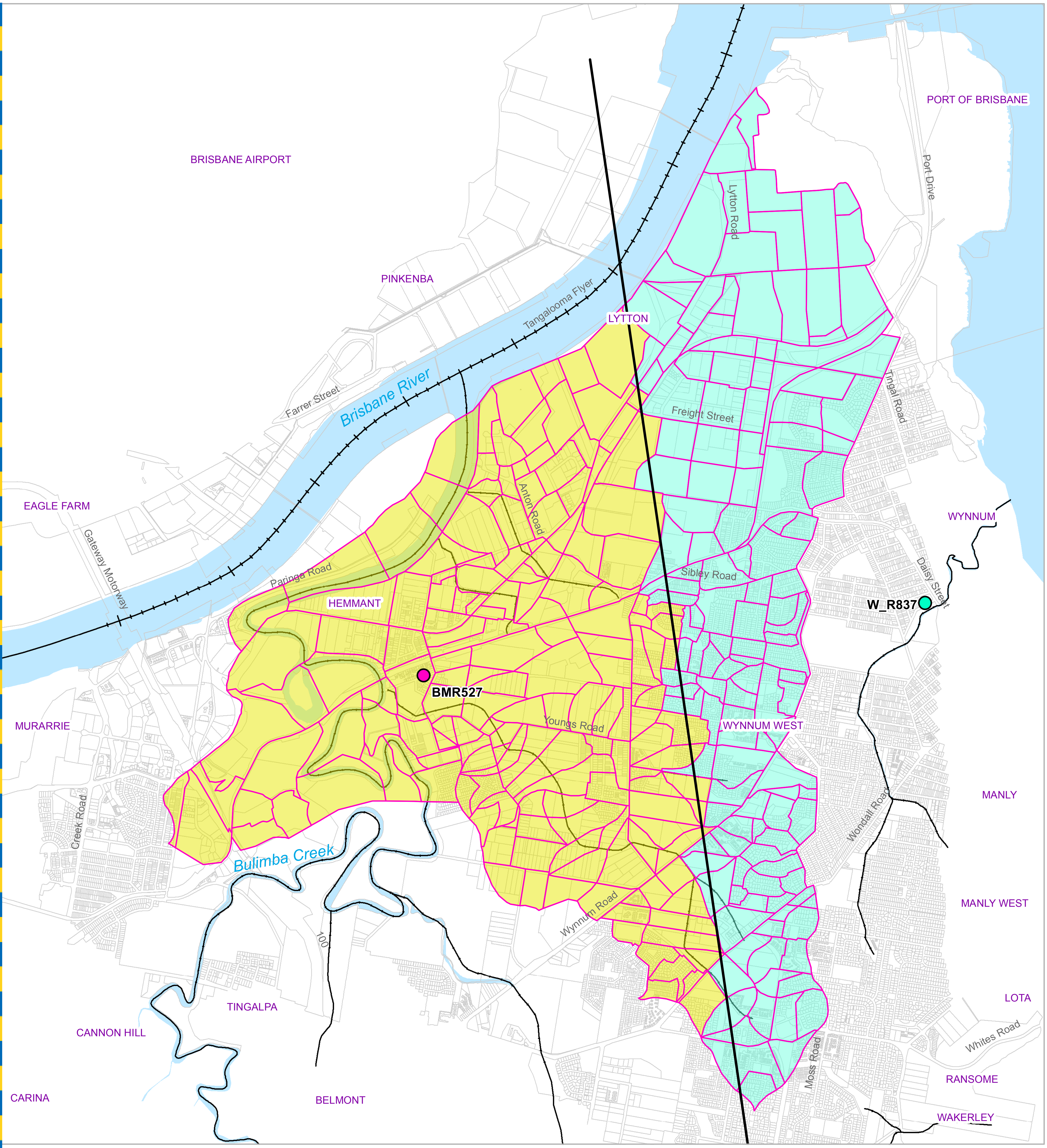
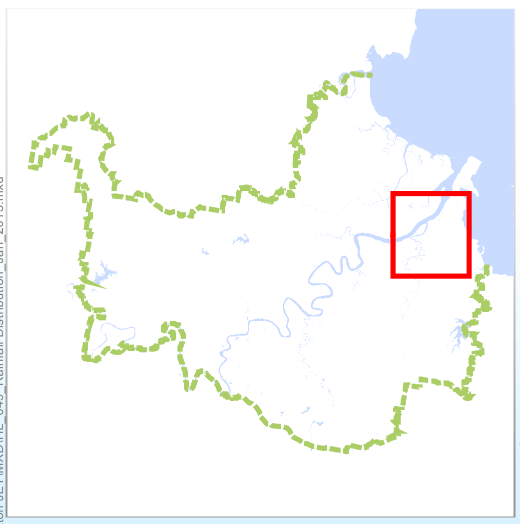
DATA INFORMATION

Hemmant Lytton Rainfall Distribution October 2010 and December 2010

Figure A1

MXD-048





Legend

- BMR527
- W_R837
- Rainfall Distribution Boundary
- Sub-catchments
- Sub-catchments for W_R837
- Sub-catchments for BMR527
- Waterway/Waterbody
- AMTD Line

0 125250375
Metres

N W E S

Prepared : Jayesh.Patel
Checked : RS
Revision : 0
Publication Date : 25 Nov 2014
Project Number : B20669

File : I:\B20669_I_BRH Hemmant Lytton JET\MXD\HL_049_Rainfall Distribution_Jan_2013.mxd

DATA INFORMATION

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Hemmant Lytton Rainfall Distribution January 2013

Figure A2

MXD-049

APPENDIX B - Hydrologic Model Parameters

Node	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
A0_1	1.9	0.04	3.7	0.015	1.9	0.04	3.7	0.015
A0_2	1.8	0.06	4.0	0.015	1.3	0.06	4.5	0.015
A0_3	5.1	0.08	3.9	0.015	2.7	0.08	6.3	0.015
A0_4	3.7	0.04	3.4	0.015	2.1	0.04	5.0	0.015
A0_5	5.4	0.06	3.8	0.015	3.0	0.06	6.1	0.015
A0_6	2.9	0.04	1.4	0.015	2.9	0.04	1.4	0.015
A0_7	3.7	0.08	5.1	0.015	3.7	0.08	5.1	0.015
A0_8	3.6	0.04	3.7	0.015	3.6	0.04	3.7	0.015
A0_9	5.3	0.08	1.7	0.015	5.3	0.08	1.7	0.015
A0_10	3.1	0.06	2.1	0.015	3.1	0.06	2.1	0.015
A0_11	4.9	0.06	0.3	0.015	4.9	0.06	0.3	0.015
A0_12	1.1	0.04	2.2	0.015	1.1	0.04	2.2	0.015
A0_13	1.9	0.04	2.3	0.015	1.9	0.04	2.3	0.015
A0_14	4.3	0.08	1.8	0.015	4.3	0.08	1.8	0.015
A0_14A	6.6	0.06	0.4	0.015	5.9	0.06	1.1	0.015
A0_14B	4.6	0.06	0.7	0.015	4.1	0.06	1.2	0.015
A0_15	8.9	0.06	4.3	0.015	8.9	0.06	4.3	0.015
A0_16	6.8	0.04	3.7	0.015	6.8	0.04	3.7	0.015
A0_17	3.8	0.04	2.6	0.015	3.8	0.04	2.6	0.015
A0_18	6.8	0.04	3.9	0.015	6.8	0.04	3.9	0.015
A0_19	5.1	0.04	1.4	0.015	5.1	0.04	1.4	0.015
A0_19A	2.1	0.04	4.7	0.015	2.1	0.04	4.7	0.015
A0_19B	2.3	0.04	1.3	0.015	2.3	0.04	1.3	0.015
A0_19C	0.5	0.04	0.3	0.015	0.5	0.04	0.3	0.015
A0_20	7.8	0.04	4.2	0.015	7.8	0.04	4.2	0.015
A0_21	10.4	0.06	4.5	0.015	10.4	0.06	4.5	0.015
A0_22	6.1	0.06	2.5	0.015	6.1	0.06	2.5	0.015
A0_23	2.8	0.06	1.1	0.015	2.8	0.06	1.1	0.015
A0_23A	0.6	0.04	0.7	0.015	0.4	0.04	0.9	0.015
A0_24	0.4	0.04	0.7	0.015	0.4	0.04	0.7	0.015
A0_25	2.3	0.04	1.0	0.015	2.3	0.04	1.0	0.015
A0_26	1.1	0.04	0.5	0.015	1.1	0.04	0.5	0.015
A0_27	6.3	0.04	2.5	0.015	6.3	0.04	2.5	0.015
A0_28	6.1	0.04	2.0	0.015	6.1	0.04	2.0	0.015
A0_30	7.2	0.04	6.4	0.015	6.6	0.04	7.0	0.015
A0_MAN	5.0	0.04	9.8	0.015	5.0	0.04	9.8	0.015

Node	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
A1_1	2.4	0.04	5.6	0.015	2.4	0.04	5.6	0.015
A1_2	2.0	0.06	3.8	0.015	1.5	0.06	4.2	0.015
A1_HAR	2.2	0.04	4.9	0.015	2.2	0.04	4.9	0.015
A1_MAN	2.1	0.04	4.6	0.015	2.1	0.04	4.6	0.015
A10_HAT	1.6	0.04	1.0	0.015	1.6	0.04	1.0	0.015
A11_HAT	2.5	0.04	1.0	0.015	2.5	0.04	1.0	0.015
A12_HAT	1.9	0.04	0.7	0.015	1.9	0.04	0.7	0.015
A13_HAT	1.1	0.04	0.5	0.015	1.1	0.04	0.5	0.015
A14_2	4.0	0.04	3.9	0.015	4.0	0.04	3.9	0.015
A14_GR	2.6	0.04	0.9	0.015	2.6	0.04	0.9	0.015
A2_1	0.4	0.04	1.5	0.015	0.4	0.04	1.5	0.015
A2_2	1.6	0.04	2.4	0.015	1.6	0.04	2.4	0.015
A2_3	1.7	0.04	4.0	0.015	1.7	0.04	4.0	0.015
A2_4	1.9	0.04	3.2	0.015	1.9	0.04	3.2	0.015
A3_2	2.1	0.04	1.9	0.015	1.1	0.04	2.9	0.015
A3_3	2.7	0.04	4.5	0.015	2.0	0.04	5.2	0.015
A3_4	2.1	0.04	3.3	0.015	2.1	0.04	3.3	0.015
A3_5	2.4	0.04	2.9	0.015	1.4	0.04	3.9	0.015
A3_6	0.7	0.04	1.9	0.015	0.7	0.04	1.9	0.015
A3_7	1.8	0.04	4.5	0.015	1.8	0.04	4.5	0.015
A3_8	0.3	0.04	0.6	0.015	0.3	0.04	0.6	0.015
A3_9	0.6	0.04	0.9	0.015	0.6	0.04	0.9	0.015
A3_10	0.5	0.04	1.2	0.015	0.5	0.04	1.2	0.015
A3_11	1.0	0.04	9.3	0.015	1.0	0.04	9.3	0.015
A3_12	2.0	0.06	0.7	0.015	2.0	0.06	0.7	0.015
A3_13	0.7	0.04	1.6	0.015	0.7	0.04	1.6	0.015
A3_14	0.9	0.04	2.4	0.015	0.9	0.04	2.4	0.015
A3_15	1.6	0.04	3.6	0.015	1.6	0.04	3.6	0.015
A3_16	1.4	0.04	3.4	0.015	1.4	0.04	3.4	0.015
A3_17	0.6	0.04	1.2	0.015	0.6	0.04	1.2	0.015
A3_18	0.7	0.04	1.7	0.015	0.7	0.04	1.7	0.015
A3_19	1.3	0.04	3.2	0.015	1.3	0.04	3.2	0.015
A3_20	3.7	0.04	4.7	0.015	3.7	0.04	4.7	0.015
A3_HAR	1.3	0.04	3.0	0.015	1.3	0.04	3.0	0.015
A4_5	0.8	0.04	1.1	0.015	0.8	0.04	1.1	0.015
A4_6	3.8	0.06	0.5	0.015	3.8	0.06	0.5	0.015
A4_BOG	2.3	0.04	1.9	0.015	2.3	0.04	1.9	0.015
A4_CAL	1.3	0.04	3.4	0.015	1.3	0.04	3.4	0.015
A5_FLE	6.6	0.08	1.1	0.015	6.6	0.08	1.1	0.015

	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
Node	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
A6_FLE	6.6	0.08	0.0	0.015	6.6	0.08	0.0	0.015
A7_1	1.7	0.04	4.1	0.015	1.7	0.04	4.1	0.015
A7_5	2.8	0.04	3.1	0.015	2.8	0.04	3.1	0.015
A7_FLE	5.3	0.06	2.5	0.015	5.3	0.06	2.5	0.015
A7_FOL	3.2	0.06	5.5	0.015	3.2	0.06	5.5	0.015
A7_HAT	5.4	0.06	1.7	0.015	5.4	0.06	1.7	0.015
A8_1	2.2	0.04	2.1	0.015	2.2	0.04	2.1	0.015
A8_2	2.9	0.04	1.7	0.015	2.9	0.04	1.7	0.015
A8_3	1.5	0.04	2.3	0.015	1.5	0.04	2.4	0.015
A8_4	1.7	0.04	3.1	0.015	1.5	0.04	3.3	0.015
A9_HAT	2.6	0.04	1.5	0.015	2.6	0.04	1.5	0.015
B0_1	1.9	0.04	4.6	0.015	1.9	0.04	4.6	0.015
B0_2	2.2	0.04	3.4	0.015	1.7	0.04	3.9	0.015
B0_3	1.1	0.04	3.1	0.015	0.5	0.04	3.7	0.015
B0_4	1.8	0.04	4.5	0.015	1.8	0.04	4.5	0.015
B0_4A	5.3	0.04	6.5	0.015	3.6	0.04	8.2	0.015
B0_4A1	0.7	0.04	1.3	0.015	0.7	0.04	1.3	0.015
B0_4A2	2.1	0.04	2.9	0.015	1.8	0.04	3.2	0.015
B0_4B	1.2	0.04	2.6	0.015	1.2	0.04	2.6	0.015
B0_4C	1.8	0.08	0.1	0.015	1.8	0.08	0.1	0.015
B0_5	5.2	0.06	0.7	0.015	3.4	0.06	2.5	0.015
B0_6	1.4	0.04	3.6	0.015	1.4	0.04	3.6	0.015
B0_7A	2.1	0.04	3.2	0.015	2.1	0.04	3.2	0.015
B0_7(A)	4.3	0.04	4.1	0.015	4.3	0.04	4.1	0.015
B0_7(B)	6.6	0.08	2.9	0.015	6.6	0.08	2.9	0.015
B0_8	2.9	0.04	1.4	0.015	2.9	0.04	1.4	0.015
B0_9	2.0	0.04	2.6	0.015	2.0	0.04	2.6	0.015
B0_10	4.5	0.04	1.5	0.015	4.5	0.04	1.5	0.015
B0_11	7.0	0.04	2.4	0.015	7.0	0.04	2.4	0.015
B0_12	4.7	0.04	1.4	0.015	4.7	0.04	1.4	0.015
B0_13	5.5	0.04	1.6	0.015	5.5	0.04	1.6	0.015
B1_1	1.3	0.04	2.6	0.015	1.1	0.04	2.8	0.015
B1_2	3.6	0.04	5.5	0.015	1.7	0.04	7.3	0.015
B2_1	0.3	0.04	1.1	0.015	0.3	0.04	1.1	0.015
B2_2	0.2	0.04	1.8	0.015	0.2	0.04	1.8	0.015
B3_1	1.4	0.04	3.2	0.015	1.4	0.04	3.2	0.015
B3_2	0.6	0.04	0.9	0.015	0.5	0.04	1.1	0.015
B4_1	9.0	0.06	1.7	0.015	9.0	0.06	1.7	0.015
B4_2	3.3	0.06	1.7	0.015	2.6	0.06	2.4	0.015

Node	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
B4_3	0.5	0.04	1.3	0.015	0.5	0.04	1.3	0.015
B5_1	1.5	0.04	3.4	0.015	1.5	0.04	3.4	0.015
B5_2	1.2	0.04	1.4	0.015	1.2	0.04	1.4	0.015
B5_3	1.7	0.04	3.0	0.015	1.7	0.04	3.0	0.015
B5_4	1.1	0.04	2.4	0.015	1.1	0.04	2.4	0.015
B5_5	0.5	0.04	1.0	0.015	0.5	0.04	1.0	0.015
B5_SCH	0.5	0.04	0.6	0.015	0.5	0.04	0.6	0.015
B6_YOU	1.6	0.04	3.6	0.015	1.6	0.04	3.6	0.015
B7_YOU	0.9	0.04	1.9	0.015	0.9	0.04	1.9	0.015
C0_1	15.4	0.04	5.3	0.015	15.4	0.04	5.3	0.015
C0_1A	4.9	0.04	1.7	0.015	4.9	0.04	1.7	0.015
C0_2	10.7	0.06	3.6	0.015	10.7	0.06	3.6	0.015
C0_3	1.4	0.04	6.8	0.015	1.0	0.04	7.2	0.015
C0_4	0.4	0.04	4.3	0.015	0.4	0.04	4.3	0.015
C0_4A	2.2	0.04	3.4	0.015	0.6	0.04	5.0	0.015
C0_5	0.9	0.04	4.5	0.015	0.5	0.04	5.0	0.015
C0_6	0.3	0.04	1.2	0.015	0.1	0.04	1.4	0.015
C0_6A	0.2	0.04	2.0	0.015	0.2	0.04	2.0	0.015
C0_6B	0.5	0.04	5.3	0.015	0.5	0.04	5.3	0.015
C0_7	0.5	0.04	2.1	0.015	0.2	0.04	2.4	0.015
C0_7A	0.5	0.04	4.9	0.015	0.5	0.04	4.9	0.015
C0_8	0.7	0.04	4.4	0.015	0.5	0.04	4.6	0.015
C0_9	1.7	0.04	5.8	0.015	0.7	0.04	6.7	0.015
C0_9A	0.2	0.04	1.9	0.015	0.2	0.04	1.9	0.015
C0_9B	0.2	0.04	1.3	0.015	0.1	0.04	1.3	0.015
C0_10	3.9	0.04	6.0	0.015	1.3	0.04	8.6	0.015
C1_1	3.0	0.04	2.2	0.015	3.0	0.04	2.2	0.015
C1_2	2.3	0.04	0.6	0.015	2.2	0.04	0.6	0.015
C1_3	3.0	0.04	0.9	0.015	1.2	0.04	2.7	0.015
C1_4	2.3	0.04	2.0	0.015	1.9	0.04	2.4	0.015
D0_1	1.5	0.04	3.6	0.015	1.5	0.04	3.6	0.015
D0_2	1.5	0.04	3.6	0.015	1.5	0.04	3.6	0.015
D0_3	1.3	0.04	2.8	0.015	1.3	0.04	2.8	0.015
D0_4	0.7	0.04	1.6	0.015	0.7	0.04	1.6	0.015
D0_5	2.2	0.04	5.7	0.015	2.2	0.04	5.7	0.015
D0_6	0.8	0.04	1.3	0.015	0.8	0.04	1.3	0.015
D0_7	1.2	0.04	3.3	0.015	1.2	0.04	3.3	0.015
D0_7A	0.8	0.04	2.7	0.015	0.8	0.04	2.7	0.015
D0_8	3.4	0.04	5.8	0.015	3.4	0.04	5.8	0.015

Node	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
D0_9	1.9	0.04	2.2	0.015	1.9	0.04	2.2	0.015
D0_9A	1.3	0.04	2.8	0.015	1.3	0.04	2.8	0.015
D0_9B	1.3	0.04	2.3	0.015	1.3	0.04	2.3	0.015
D0_9C	0.9	0.04	2.2	0.015	0.9	0.04	2.2	0.015
D0_10	1.1	0.04	1.7	0.015	1.1	0.04	1.7	0.015
D0_11	1.3	0.04	2.2	0.015	1.3	0.04	2.2	0.015
D0_12	1.7	0.04	1.8	0.015	1.7	0.04	1.8	0.015
D0_13	1.5	0.04	1.4	0.015	1.5	0.04	1.4	0.015
D0_13A	0.6	0.04	1.2	0.015	0.6	0.04	1.2	0.015
D0_13B	1.2	0.04	1.4	0.015	1.2	0.04	1.4	0.015
D0_14	1.2	0.04	1.6	0.015	1.2	0.04	1.6	0.015
D0_14A	1.1	0.04	2.3	0.015	1.1	0.04	2.3	0.015
D0_15	2.7	0.04	1.6	0.015	2.7	0.04	1.6	0.015
D0_16	2.8	0.04	2.2	0.015	2.8	0.04	2.3	0.015
D0_16A	1.6	0.04	0.1	0.015	1.6	0.04	0.1	0.015
D0_16B	2.7	0.04	3.8	0.015	2.7	0.04	3.8	0.015
D0_17	10.1	0.04	1.9	0.015	10.1	0.04	1.9	0.015
D0_17A	6.0	0.04	0.5	0.015	6.0	0.04	0.5	0.015
D0_17B	3.7	0.04	3.3	0.015	3.7	0.04	3.3	0.015
D0_18	1.6	0.04	3.5	0.015	0.6	0.04	4.4	0.015
D0_19	4.1	0.04	10.1	0.015	1.5	0.04	12.7	0.015
D0_20	0.2	0.04	2.5	0.015	0.2	0.04	2.5	0.015
D0_21	3.0	0.06	2.3	0.015	1.1	0.06	4.2	0.015
D0_22	0.1	0.04	0.9	0.015	0.1	0.04	0.9	0.015
D0_22A	1.8	0.04	6.4	0.015	0.8	0.04	7.4	0.015
D0_22B	0.5	0.04	4.8	0.015	0.5	0.04	4.8	0.015
D0_22C	2.0	0.06	2.4	0.015	0.4	0.06	3.9	0.015
D0_23	0.8	0.04	3.5	0.015	0.4	0.04	3.8	0.015
D0_24	2.3	0.04	7.5	0.015	1.0	0.04	8.9	0.015
D0_24A	0.5	0.04	2.7	0.015	0.3	0.04	2.9	0.015
D0_24B	0.1	0.04	1.5	0.015	0.1	0.04	1.5	0.015
D0_25	2.3	0.04	1.7	0.015	0.4	0.04	3.6	0.015
D1_1	1.2	0.04	2.8	0.015	1.2	0.04	2.8	0.015
D1_2	2.4	0.04	5.4	0.015	2.4	0.04	5.4	0.015
D1_3	1.3	0.04	2.6	0.015	1.3	0.04	2.6	0.015
D1_4	3.7	0.04	7.7	0.015	3.7	0.04	7.7	0.015
D1_5	0.5	0.04	1.1	0.015	0.5	0.04	1.1	0.015
D1_6	1.0	0.04	1.6	0.015	0.8	0.04	1.7	0.015
D2_1	1.4	0.04	1.5	0.015	1.3	0.04	1.5	0.015

Node	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
D2_3	1.3	0.04	0.4	0.015	1.3	0.04	0.4	0.015
D2_4	1.4	0.04	2.4	0.015	1.4	0.04	2.4	0.015
D2_5	1.3	0.04	3.7	0.015	1.3	0.04	3.7	0.015
D2_6	0.8	0.04	2.1	0.015	0.8	0.04	2.1	0.015
E0_1	1.4	0.04	2.1	0.015	1.0	0.04	2.5	0.015
E0_2	3.8	0.04	8.7	0.015	3.8	0.04	8.7	0.015
E0_3	11.5	0.06	10.6	0.015	11.5	0.06	10.6	0.015
E0_3A	1.4	0.04	3.3	0.015	1.4	0.04	3.3	0.015
E0_3B	0.8	0.04	1.9	0.015	0.8	0.04	1.9	0.015
E0_3C	4.2	0.04	9.8	0.015	4.2	0.04	9.8	0.015
E0_3D	1.0	0.04	2.0	0.015	1.0	0.04	2.0	0.015
E0_4	3.1	0.04	2.8	0.015	2.9	0.04	2.9	0.015
E0_4A	5.1	0.06	1.7	0.015	5.1	0.06	1.7	0.015
E0_4B	2.9	0.04	6.6	0.015	2.9	0.04	6.6	0.015
E0_4C	4.8	0.06	2.1	0.015	3.5	0.06	3.4	0.015
E0_5	6.5	0.04	4.4	0.015	1.1	0.04	9.9	0.015
E0_6	10.0	0.06	0.9	0.015	7.1	0.06	3.8	0.015
E0_7	11.2	0.08	0.9	0.015	11.2	0.08	0.9	0.015
E0_7A	3.2	0.04	8.1	0.015	3.2	0.04	8.1	0.015
E0_7B	0.6	0.04	8.5	0.015	0.6	0.04	8.5	0.015
E0_8	12.6	0.04	4.9	0.015	4.1	0.04	13.3	0.015
E0_9	1.1	0.04	1.9	0.015	0.9	0.04	2.1	0.015
E0_9A	3.1	0.06	2.2	0.015	0.5	0.06	4.8	0.015
E0_10	3.1	0.08	1.0	0.015	3.1	0.08	1.1	0.015
E0_10A	1.8	0.04	3.9	0.015	0.5	0.04	5.1	0.015
E0_10B	2.7	0.04	7.1	0.015	1.1	0.04	8.7	0.015
E0_10C	9.2	0.04	3.3	0.015	1.9	0.04	10.6	0.015
E0_11	11.2	0.08	0.6	0.015	11.2	0.08	0.6	0.015
E0_11A	0.3	0.04	2.9	0.015	0.3	0.04	2.9	0.015
E0_11B	0.7	0.04	6.6	0.015	0.7	0.04	6.6	0.015
E0_11C	0.6	0.04	5.5	0.015	0.6	0.04	5.5	0.015
E0_12	3.6	0.08	1.4	0.015	3.6	0.08	1.4	0.015
E1_1	8.1	0.06	2.0	0.015	8.1	0.06	2.0	0.015
E1_2	5.6	0.04	4.3	0.015	5.3	0.04	4.6	0.015
E1_3	4.8	0.04	4.3	0.015	4.6	0.04	4.4	0.015
E2_1	2.6	0.04	5.6	0.015	2.6	0.04	5.6	0.015
E2_2	1.5	0.04	3.8	0.015	1.5	0.04	3.8	0.015
E2_3	3.5	0.04	8.0	0.015	3.3	0.04	8.2	0.015
E2_4	9.0	0.04	7.3	0.015	2.2	0.04	14.1	0.015

Node	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
E2_4A	4.9	0.04	3.4	0.015	3.5	0.04	4.9	0.015
E2_5	13.0	0.08	12.1	0.015	4.0	0.08	21.2	0.015
EXT1	0.0	0.04	0.0	0.015	0.0	0.04	0.0	0.015
EXT1_A	1.3	0.04	12.0	0.015	1.3	0.04	12.0	0.015
EXT1_B	6.1	0.04	1.4	0.015	1.4	0.04	6.1	0.015
EXT2	0.0	0.04	0.0	0.015	0.0	0.04	0.0	0.015
EXT2_A	17.7	0.04	4.4	0.015	2.2	0.04	20.0	0.015
EXT3	0.0	0.04	0.0	0.015	0.0	0.04	0.0	0.015
EXT3_A	6.7	0.04	3.8	0.015	6.7	0.04	3.8	0.015
F0_1	6.2	0.04	9.3	0.015	5.3	0.04	10.1	0.015
F0_2	8.8	0.06	2.3	0.015	6.3	0.06	4.8	0.015
F0_3	9.3	0.06	5.3	0.015	9.3	0.06	5.3	0.015
F0_4	0.3	0.04	2.3	0.015	0.3	0.04	2.4	0.015
F0_4A	0.9	0.04	7.9	0.015	0.9	0.04	8.0	0.015
F0_4B	3.4	0.04	0.5	0.015	0.9	0.04	2.9	0.015
F0_4C	0.6	0.04	5.5	0.015	0.6	0.04	5.5	0.015
F0_5	3.2	0.04	3.0	0.015	1.2	0.04	4.9	0.015
F0_6	0.7	0.04	5.8	0.015	0.7	0.04	5.8	0.015
F0_10	4.2	0.04	0.7	0.015	0.5	0.04	4.4	0.015
F0_7	1.2	0.04	3.7	0.015	0.5	0.04	4.4	0.015
F0_10A	0.8	0.04	7.8	0.015	0.8	0.04	7.8	0.015
F0_8	2.4	0.04	1.8	0.015	0.4	0.04	3.8	0.015
F0_10B	0.6	0.04	4.0	0.015	0.6	0.04	4.0	0.015
F0_9	11.4	0.04	3.4	0.015	1.5	0.04	13.3	0.015
F0_11	5.0	0.04	1.3	0.015	0.6	0.04	5.6	0.015
F1_1	2.4	0.04	4.4	0.015	1.7	0.04	5.2	0.015
F1_2	1.8	0.04	9.8	0.015	1.2	0.04	10.5	0.015
F1_3	2.3	0.04	13.9	0.015	1.6	0.04	14.6	0.015
F1_4	0.2	0.04	2.7	0.015	0.2	0.04	2.7	0.015
F1_4A	0.1	0.04	1.7	0.015	0.1	0.04	1.7	0.015
G0_1	4.5	0.04	11.2	0.015	4.5	0.04	11.2	0.015
G0_2	1.6	0.04	1.0	0.015	1.6	0.04	1.0	0.015
G0_3	10.1	0.04	4.1	0.015	9.4	0.04	4.8	0.015
G0_3A	4.6	0.04	9.9	0.015	1.4	0.04	13.0	0.015
G0_4	16.2	0.06	2.5	0.015	16.2	0.06	2.5	0.015
G0_5	10.0	0.04	5.4	0.015	1.2	0.04	14.2	0.015
G0_6	21.5	0.04	4.9	0.015	5.3	0.04	21.0	0.015
G0_6A	0.6	0.04	13.9	0.015	0.6	0.04	13.9	0.015
G0_6B	8.9	0.04	0.8	0.015	2.7	0.04	7.0	0.015

	Existing Scenario (Historical Events)				Ultimate Scenario (Design Events)			
Node	Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)		Sub-catchment 1 (Pervious)		Sub-catchment 2 (Impervious)	
	Area	n	Area	n	Area	n	Area	n
H0_1	0.4	0.04	10.1	0.015	0.4	0.04	10.1	0.015
H0_2	0.7	0.04	16.3	0.015	0.7	0.04	16.3	0.015
H0_2A	0.2	0.04	3.9	0.015	0.2	0.04	3.9	0.015
H0_3	0.5	0.04	10.8	0.015	0.5	0.04	10.8	0.015
H0_4	0.2	0.04	7.0	0.015	0.2	0.04	7.0	0.015
I0_1	5.8	0.04	4.0	0.015	5.8	0.04	4.0	0.015
I0_2	4.7	0.04	0.8	0.015	4.6	0.04	0.8	0.015
I0_3	11.2	0.04	0.5	0.015	11.2	0.04	0.5	0.015
I0_4	22.8	0.04	1.7	0.015	21.4	0.04	3.1	0.015
I0_5	16.8	0.04	1.5	0.015	16.8	0.04	1.5	0.015
I0_6	7.5	0.04	0.4	0.015	7.5	0.04	0.4	0.015
I0_7	4.9	0.04	0.4	0.015	4.9	0.04	0.4	0.015
I0_8	2.1	0.04	7.0	0.015	2.1	0.04	7.0	0.015
I0_9	4.6	0.04	11.7	0.015	3.0	0.04	13.3	0.015
J0_1	1.3	0.04	4.0	0.015	1.3	0.04	4.0	0.015
J0_2	2.1	0.04	18.9	0.015	2.1	0.04	18.9	0.015
J0_3	2.8	0.04	5.0	0.015	2.8	0.04	5.0	0.015
J0_4	18.8	0.06	2.4	0.015	13.2	0.06	7.9	0.015
J0_4A	3.9	0.04	4.5	0.015	0.8	0.04	7.5	0.015
J0_4B	6.9	0.04	1.1	0.015	0.9	0.04	7.1	0.015
J0_4C	0.7	0.04	5.9	0.015	0.7	0.04	5.9	0.015
I0_10	15.5	0.04	3.9	0.015	15.5	0.04	3.9	0.015
I0_10A	11.8	0.06	3.6	0.015	10.2	0.06	5.2	0.015
I0_10B	8.3	0.04	19.9	0.015	8.3	0.04	19.9	0.015
I0_11	14.8	0.06	16.7	0.015	10.5	0.06	21.0	0.015
I0_12	8.0	0.04	4.9	0.015	8.0	0.04	4.9	0.015
I0_13	13.2	0.04	9.3	0.015	13.2	0.04	9.3	0.015
I0_13A	3.0	0.04	16.4	0.015	2.9	0.04	16.4	0.015
I0_14	12.6	0.04	11.5	0.015	12.5	0.04	11.5	0.015
I0_14A	2.2	0.04	10.5	0.015	1.3	0.04	11.4	0.015
I0_15	6.2	0.04	5.6	0.015	6.1	0.04	5.7	0.015
I0_15A	3.4	0.04	13.3	0.015	1.9	0.04	14.8	0.015
I0_16	8.9	0.04	9.3	0.015	8.9	0.04	9.3	0.015
I0_16A	0.4	0.04	3.9	0.015	0.4	0.04	3.9	0.015
I0_16B	0.6	0.04	5.9	0.015	0.6	0.04	5.9	0.015

APPENDIX C - Hydraulic Structure Reference Sheets

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**C4543B**
LOCATION:	**Beverly Road**

IMMUNITY RATING: **5%**

DATE OF SURVEY:	31/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	C4543B	AMTD (m):	

STRUCTURE DESCRIPTION: **A RCBC crossing Beverly Road on the Hemmant Catchment.**

STRUCTURE SIZE: 5No. 1.8m x 0.75m

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: 3.6 m

UPSTREAM OBVERT LEVEL: 4.35 m

DOWNSTREAM INVERT LEVEL: 3.52 m

DOWNSTREAM OBVERT LEVEL: 4.27 m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 21.1

LENGTH OF CULVERT BARREL AT OBVERT (m): 21.1

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 21.1

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

LOWEST POINT OF WEIR (m AHD): 4.75

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability greater than 0.02%. A major break occurs at an AEP of 0.05%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	C4543B
LOCATION	Beverly Road

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	18.524	5.403	4.671	732	21.1	86.3		2.744
0.2	18.299	5.367	4.651	716	21.1	86.3		2.711
1	17.277	5.225	4.597	628	16.4	56.6		2.56
2	16.393	5.131	4.56	571	15.4	42.4		2.429
5	15.335	4.964	4.462	502	-	-		2.272
10	13.637	4.826	4.428	398	-	-		2.02
20	12.09	4.718	4.406	312	-	-		1.791
50	9.573	4.559	4.363	196	-	-		1.418

NB: Results are based on existing stream conditions.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Ch2_N_Lyt**
LOCATION:	**Lytton Road**

IMMUNITY RATING:	**1%**

DATE OF SURVEY:	24/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	Ch2_N_Lyt	AMTD (m):	

STRUCTURE DESCRIPTION: **A circular culvert crossing Lytton Road.**

STRUCTURE SIZE: 6No. 1.8m dia.

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: -0.036 m

UPSTREAM OBVERT LEVEL: 1.764 m

DOWNSTREAM INVERT LEVEL: -0.45 m

DOWNSTREAM OBVERT LEVEL:1.755m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 24

LENGTH OF CULVERT BARREL AT OBVERT (m): 24

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 16

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

LOWEST POINT OF WEIR (m AHD): **-0.35m**

(Level at which water overtops road)

PIER WIDTH (m):

HYDRAULIC STRUCTURE REFERENCE SHEET

HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.5%. The entire lane on the road is inundated at this point.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Ch2_N_Lyt**
LOCATION	**Lytton Road**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	21.65	2.245	2.118	127	*	0.53		1.499
0.2	16.832	2.012	1.995	17	61.1	0.36		1.165
1	15.759	1.702	1.684	18	-	-		1.091
2	15.03	1.635	1.618	17	-	-		1.04
5	13.657	1.552	1.539	13	-	-		0.954
10	12.016	1.463	1.453	10	-	-		0.857
20	10.614	1.394	1.384	10	-	-		0.772
50	8.435	1.266	1.258	8	-	-		0.639

NB: Results are based on existing stream conditions.

*entire road inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**CH2_N_MOT_ds**
LOCATION:	**Port of Brisbane Motorway**

IMMUNITY RATING:	**1%**

DATE OF SURVEY:	31/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	CH2_N_MOT_ds	AMTD (m):	

STRUCTURE DESCRIPTION: **A circular crossing POBM on the Lytton Catchment.**

STRUCTURE SIZE: 5No. 1.8m x 0.75m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: 0.183 m		UPSTREAM OBVERT LEVEL:	0.933m
DOWNSTREAM INVERT LEVEL: 0.172 m		DOWNSTREAM OBVERT LEVEL:	0.922m
For culverts give floor level.		For bridges give bed level.	

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 58.7

LENGTH OF CULVERT BARREL AT OBVERT (m): 58.7

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 58.7		LOWEST POINT OF WEIR (m AHD):	1.1
(In direction of flow, i.e. distance from u/s face to d/s face)		(Level at which water overtops road)	

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an AEP greater than 0.01%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**CH2_N_MOT_ds**
LOCATION	**POBM**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	12.494	2.301	2.265	36	19.78	1.23		1.522
0.2	7.985	2.036	1.991	45	17	0.99		0.995
1	8.249	1.739	1.673	66	-	-		1.17
2	7.794	1.679	1.614	65	-	-		1.153
5	7.36	1.621	1.558	63	-	-		1.138
10	6.591	1.533	1.474	59	-	-		1.09
20	5.984	1.469	1.415	54	-	-		1.044
50	4.758	1.348	1.306	42	-	-		0.923

NB: Results are based on existing stream conditions.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Ch2_N_MOT_u1**
LOCATION:	**Port of Brisbane Motorway**

IMMUNITY RATING: **0.05%**

DATE OF SURVEY:	26/11/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	**Ch2_N_MOT_u1**	AMTD (m):	

STRUCTURE DESCRIPTION: **A circular culvert crossing POBM.**

STRUCTURE SIZE: 3No. 1.918m dia.

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: 0.209 m

UPSTREAM OBVERT LEVEL: 2.127

DOWNSTREAM INVERT LEVEL: 0.199 m

DOWNSTREAM OBVERT LEVEL: 2.117

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 60

LENGTH OF CULVERT BARREL AT OBVERT (m): 60

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 40

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

LOWEST POINT OF WEIR (m AHD): **3.06**

(Level at which water overtops road)

PIER WIDTH (m):

HYDRAULIC STRUCTURE REFERENCE SHEET

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.05%. It is not known what AEP its capacity will be exceeded as it was able to withstand this event.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Ch2_N_MOT_u1**
LOCATION	**POBM**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	11.489	2.381	2.364	17	-	-		1.326
0.2	6.613	2.12	2.109	11	-	-		0.827
1	7.164	1.846	1.829	17	-	-		0.944
2	6.81	1.782	1.764	18	-	-		0.931
5	6.483	1.72	1.703	17	-	-		0.921
10	5.837	1.624	1.607	17	-	-		0.882
20	5.371	1.553	1.536	17	-	-		0.858
50	4.235	1.417	1.404	13	-	-		0.763

NB: Results are based on existing stream conditions.

*entire road inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Ch2_N_MOT_u2**
LOCATION:	**Port of Brisbane Motorway**

IMMUNITY RATING:	**0.05%**

DATE OF SURVEY:	26/11/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	**Ch2_N_MOT_u2**	AMTD (m):	

STRUCTURE DESCRIPTION: **A circular culvert crossing POBM.**

STRUCTURE SIZE: 3No. 1.918m dia.

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: 0.199 m

UPSTREAM OBVERT LEVEL: 2.117

DOWNSTREAM INVERT LEVEL: 0.183 m

DOWNSTREAM OBVERT LEVEL: 2.101

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 87.9

LENGTH OF CULVERT BARREL AT OBVERT (m): 87.9

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 62.49

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

LOWEST POINT OF WEIR (m AHD): **2.3**

(Level at which water overtops road)

PIER WIDTH (m):

HYDRAULIC STRUCTURE REFERENCE SHEET

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.05%. It is not known what AEP its capacity will be exceeded as it was able to withstand this event.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	Ch2_N_MOT_u2
LOCATION	POBM

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	12.493	2.364	2.301	63	-	-		1.457
0.2	8.022	2.109	2.036	73	-	-		0.984
1	8.217	1.829	1.739	90	-	-		1.112
2	7.757	1.764	1.679	85	-	-		1.091
5	7.321	1.703	1.621	82	-	-		1.072
10	6.555	1.607	1.533	74	-	-		1.029
20	5.955	1.536	1.469	67	-	-		0.987
50	4.729	1.404	1.348	56	-	-		0.878

NB: Results are based on existing stream conditions.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Ch3N_MOT**
LOCATION:	**Hemmant and Tingalpa Road**

IMMUNITY RATING:	**0.05%**

DATE OF SURVEY:	17/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:		BCC ASSET ID:
MODEL ID:	HEMDR_09	AMTD (m):

STRUCTURE DESCRIPTION: **A circular culvert crossing Hemmant and Tingalpa Road. A main drain.**

STRUCTURE SIZE: 4No. 3m x 1.8m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.241 m

UPSTREAM OBVERT LEVEL: 1.559m

DOWNSTREAM INVERT LEVEL: -0.277 m

For culverts give floor level.

DOWNSTREAM OBVERT LEVEL: 1.523m

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 12.5 m

LENGTH OF CULVERT BARREL AT OBVERT (m): 12.5 m

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 12.5

LOWEST POINT OF WEIR (m AHD): **-0.12**

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

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

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the square ends of the culvert (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.05%. The water level never breaches the culvert onto the road.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Ch3N_MOT**
LOCATION	**Hemmant and Tingalpa Road**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	23.241	2.417	2.323	0.094	-	-		1.341
0.2	18.629	2.207	2.157	0.05	-	-		1.075
1	13.874	1.838	1.817	0.021	-	-		0.801
2	12.558	1.754	1.738	0.016	-	-		0.729
5	11.548	1.678	1.666	0.012	-	-		0.68
10	10.104	1.579	1.569	0.01	-	-		0.615
20	9.145	1.514	1.506	0.008	-	-		0.571
50	7.075	1.384	1.376	0.008	-	-		0.47

NB: Results are based on existing stream conditions.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Chan3Ex**
LOCATION:	**Export Street**

IMMUNITY RATING:	**0.2%**

DATE OF SURVEY: 31/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:	BCC ASSET ID:
MODEL ID: Chan3Ex	AMTD (m):

STRUCTURE DESCRIPTION: **A RCBC crossing Export Street on the Lytton Catchment.**

STRUCTURE SIZE: 3No. 2.4m x 1.2m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: 0.936 m

UPSTREAM OBVERT LEVEL: 2.136m

DOWNSTREAM INVERT LEVEL: 0.814 m

For culverts give floor level.

DOWNSTREAM OBVERT LEVEL: 2.014m

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 24.12

LENGTH OF CULVERT BARREL AT OBVERT (m): 24.12

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 24.12

LOWEST POINT OF WEIR (m AHD): 0.85

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

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

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability greater than 0.2%.
A catastrophic breach occurs at an AEP of 0.05%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Chan3Ex**
LOCATION	**Export Street**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	10.282	2.732	2.71	22	*	0.07		1.19
0.2	5.225	2.242	2.237	5	-	-		0.632
1	3.046	1.9	1.897	3	-	-		1.135
2	2.389	1.806	1.803	3	-	-		1.145
5	1.598	1.742	1.738	4	-	-		1.14
10	1.302	1.64	1.637	3	-	-		1.012
20	1.129	1.563	1.559	4	-	-		0.982
50	0.881	1.438	1.434	4	-	-		0.916

NB: Results are based on existing stream conditions.

*Road completely inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Chan3Lytt & Ch3_New_Lyt**
LOCATION:	**Lytton Road**

IMMUNITY RATING:	**0.5%**

DATE OF SURVEY: 31/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:	BCC ASSET ID:
MODEL ID: Chan3Lytt	AMTD (m):

STRUCTURE DESCRIPTION: **A RCBC crossing Lytton Road on the Lytton Catchment.**

STRUCTURE SIZE: 3No. 2.4m x 1.2m + 2No. 2.4m x 1.2m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.258 m	UPSTREAM OBVERT LEVEL: 0.942m
DOWNSTREAM INVERT LEVEL: -0.273 m	DOWNSTREAM OBVERT LEVEL: 0.927m
For culverts give floor level.	For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 29.28

LENGTH OF CULVERT BARREL AT OBVERT (m): 29.28

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 29.28	LOWEST POINT OF WEIR (m AHD): -0.27
(In direction of flow, i.e. distance from u/s face to d/s face)	(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS:

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	Chan3Lytt & Ch3_New_Lyt
LOCATION	Lytton Road

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	20.53	2.315	2.248	67	*	0.44		1.188
0.2	20.07	2.17	2.12	50	**	0.04		1.161
1	15.41	1.876	1.85	26	-	-		0.892
2	14.00	1.796	1.776	20	-	-		0.81
5	12.52	1.703	1.688	15	-	-		0.724
10	10.97	1.605	1.594	11	-	-		0.635
20	9.75	1.523	1.513	10	-	-		0.564
50	7.60	1.38	1.373	7	-	-		0.44

NB: Results are based on existing stream conditions.

*Road completely inundated

**Flow path is too complicated to give a single width.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Chan3Trade**
LOCATION:	**Trade Street**

IMMUNITY RATING:	**0.02%**

DATE OF SURVEY: 31/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:	BCC ASSET ID:
MODEL ID: Chan3Trade	AMTD (m):

STRUCTURE DESCRIPTION: **A RCBC crossing Trade Street on the Lytton Catchment.**

STRUCTURE SIZE: 3No. 2.4m x 1.2m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.132 m

UPSTREAM OBVERT LEVEL: 1.068m

DOWNSTREAM INVERT LEVEL: -0.085 m

DOWNSTREAM OBVERT LEVEL: 1.115m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 22.68

LENGTH OF CULVERT BARREL AT OBVERT (m): 22.68

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 22.68

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

LOWEST POINT OF WEIR (m AHD): 0m

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability greater than 0.02%. A major break occurs at an AEP of 0.05%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Chan3Trade**
LOCATION	**Trade Street**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	17.909	2.59	2.438	152	*	0.16		2.073
0.2	11.254	2.157	2.106	51	-	-		1.303
1	8.2	1.804	1.788	16	-	-		0.949
2	7.37	1.711	1.7	11	-	-		0.853
5	6.971	1.648	1.639	9	-	-		0.807
10	5.91	1.546	1.538	8	-	-		0.684
20	5.236	1.463	1.456	7	-	-		0.606
50	4.108	1.327	1.321	6	-	-		0.475

NB: Results are based on existing stream conditions.

*Road completely inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Chan4_POBM**
LOCATION:	**Port of Brisbane Motorway**

IMMUNITY RATING:	**0.05%**

DATE OF SURVEY:	25/11/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	Chan4_POBM	AMTD (m):	

STRUCTURE DESCRIPTION: **A bridge crossing The Port of Brisbane Motorway.**

STRUCTURE SIZE:

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.03 m

DOWNSTREAM INVERT LEVEL: -0.56 m

For culverts give floor level.

UPSTREAM OBVERT LEVEL: Unknown

DOWNSTREAM OBVERT LEVEL: Unknown

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 57

LENGTH OF CULVERT BARREL AT OBVERT (m): 57

TYPE OF LINING:

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 57

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

LOWEST POINT OF WEIR (m AHD): **0m**

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: Details of the bridge are unknown. For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.05%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Chan4_POBM**
LOCATION	**Port of Brisbane Motorway**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	28.159	2.42	2.414	6	-	-		0.856
0.2	12.802	2.27	2.257	13	-	-		0.389
1	9.107	2.157	2.15	7	-	-		0.277
2	6.157	2.099	2.091	8	-	-		0.187
5	5.292	2.038	2.033	5	-	-		0.161
10	4.654	1.984	1.98	4	-	-		0.141
20	4.049	1.96	1.956	4	-	-		0.123
50	3.428	1.917	1.913	4	-	-		0.104

NB: Results are based on existing stream conditions.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**Chan4Lytt**
LOCATION:	**Lytton Road**

IMMUNITY RATING:	**0.5%**

DATE OF SURVEY: 24/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:	BCC ASSET ID:
MODEL ID: Chan4Lytt	AMTD (m):

STRUCTURE DESCRIPTION: **A circular concrete culvert crossing Lytton Road.**

STRUCTURE SIZE: 6No. 1.8m dia.

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.23 m	UPSTREAM OBVERT LEVEL: 1.57m
DOWNSTREAM INVERT LEVEL: -0.24 m	DOWNSTREAM OBVERT LEVEL: 1.56m
For culverts give floor level.	For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 17.2

LENGTH OF CULVERT BARREL AT OBVERT (m): 17.2

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 17.2	LOWEST POINT OF WEIR (m AHD): 2
(In direction of flow, i.e. distance from u/s face to d/s face)	(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.5%. The entire lane on the road is inundated at this point.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**Chan4Lytt**
LOCATION	**Lytton Road**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	14.343	2.402	2.339	63	*	0.1		0.94
0.2	11.015	2.245	2.196	49	**	0.001		0.722
1	23.384	1.849	1.948	-99	-	-		1.532
2	21.511	2.092	2.17	-78	-	-		1.41
5	21.238	2.03	2.126	-96	-	-		1.392
10	21.072	1.995	2.098	-103	-	-		1.381
20	21.212	1.976	2.085	-109	-	-		1.39
50	21.143	1.914	2.027	-113	-	-		1.385

NB: Results are based on existing stream conditions.

*entire road inundated towards the upstream end.

** Flow path too complex to follow

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**HEMDR_02& HEMDR_03& HEMDR_04**	**IMMUNITY RATING: 39.35%**
LOCATION:	**Wynnum Road**	

DATE OF SURVEY: 24/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:	BCC ASSET ID:
MODEL ID: HEMDR_02	AMTD (m):

STRUCTURE DESCRIPTION: **A main RCBC crossing Wynnum Road. The road is two way, two lane.**

STRUCTURE SIZE: 3No. 3m x 1.8m + 1No.6.7m x 1.5m + 1No. 1.8m x 1.8m

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: 1.329m

UPSTREAM OBVERT LEVEL: 3.129m

DOWNSTREAM INVERT LEVEL: 1.309 m

DOWNSTREAM OBVERT LEVEL: 3.109m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 22

LENGTH OF CULVERT BARREL AT OBVERT (m): 22

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? Yes, DTM Survey. Project Name: Wynnum Road Bikeway. Surveyor: Norman Johnson. Project Number: 080018

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 22

LOWEST POINT OF WEIR (m AHD): 2.9

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

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

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the square ends of the culvert (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 39.35%. Only the lane towards the upstream end of the culvert is inundated, with the second lane experiencing inundation when the AEP of the event is less than 0.2%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	HEMDR_02& HEMDR_03& HEMDR_04
LOCATION	Wynnum Road

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	57.488	3.649	3.544	105	*	0.30		1.988
0.2	34.196	3.107	3.083	24	*	0.27		1.167
1	25.055	2.945	2.931	14	**	0.23		0.949
2	22.272	2.884	2.873	11	**	0.17		0.93
5	20.42	2.819	2.81	9	**	0.15		0.902
10	19.605	2.779	2.77	9	**	0.14		0.874
20	19.096	2.755	2.747	8	**	0.13		0.863
50	17.62	2.697	2.69	7	-	-		0.81

NB: Results are based on existing stream conditions.

*Entire road inundated

*Road not inundated, but flow width is indefinitely long on the bottom of the weir

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**HEMDR_09**
LOCATION:	**Hemmant and Tingalpa Road**

IMMUNITY RATING:	**1%**

DATE OF SURVEY:	17/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	HEMDR_09	AMTD (m):	

STRUCTURE DESCRIPTION: **A main RCBC crossing Hemmant and Tingalpa Road. A main drain.**

STRUCTURE SIZE: 4No. 3m x 1.8m

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: -0.241 m		UPSTREAM OBVERT LEVEL:	1.559m
DOWNSTREAM INVERT LEVEL: -0.277 m		DOWNSTREAM OBVERT LEVEL:	1.523m
For culverts give floor level.		For bridges give bed level.	

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 12.5 m

LENGTH OF CULVERT BARREL AT OBVERT (m): 12.5 m

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 22.9	LOWEST POINT OF WEIR (m AHD):	**2.26m**
(In direction of flow, i.e. distance from u/s face to d/s face)	(Level at which water overtops road)	

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the square ends of the culvert (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 1%. The flood extents significantly increase between an AEP of 1% and an AEP of 0.5%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**HEMDR_09**
LOCATION	**Port of Brisbane Motorway**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	55.77	2.91	2.92	22.9	*	0.60		2.582
0.2	45.472	2.54	2.55	0	*	0.37		2.105
1	32.317	2.16	2.1	40	0	0		1.496
2	26.322	2.06	2.03	40	0	0		1.219
5	18.75	1.94	1.92	20	0	0		0.868
10	14.004	1.84	1.83	10	0	0		0.648
20	11.384	1.76	1.75	10	0	0		0.527
50	8.967	1.61	1.60	10	0	0		0.415

NB: Results are based on existing stream conditions.

*Entire Road inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**LINDUM_01**
LOCATION:	**Kianawah Road**

IMMUNITY RATING:	**39.35%**

DATE OF SURVEY:	24/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	LINDUM_01	AMTD (m):	

STRUCTURE DESCRIPTION: **A main circular culvert crossing Kianawah Road.**

STRUCTURE SIZE: 5No. 1.6m dia.

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: 0.663 m

UPSTREAM OBVERT LEVEL: 2.263

DOWNSTREAM INVERT LEVEL: 0.626 m

DOWNSTREAM OBVERT LEVEL: 2.226

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 20

LENGTH OF CULVERT BARREL AT OBVERT (m): 20

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? Yes, DTM survey completed by Schlencker Mapping. Survey number 080434.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 22

LOWEST POINT OF WEIR (m AHD): 2.5m

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

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

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.5%.
The entire lane on the road is inundated at this point.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**LINDUM_01**
LOCATION	**Kiawanah Road**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	34.876	3.35	2.918	432	*	0.45		3.471
0.2	37.688	3.204	2.363	841	*	0.35		3.751
1	29.197	3.101	2.175	926	*	0.26		2.906
2	28.342	3.05	2.134	916	*	0.22		2.935
5	26.86	2.966	2.096	870	*	0.16		2.903
10	25.09	2.872	2.02	852	*	0.09		2.794
20	23.838	2.769	1.986	783	**	0.03		4.688
50	21.76	2.463	1.925	538	-	-		2.668

NB: Results are based on existing stream conditions.

*Entire road is completely inundated

**Upstream end of road inundated, no water downstream past culvert

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**LINDUM_02**
LOCATION:	**Cleveland Railway**

IMMUNITY RATING:	**1%**

DATE OF SURVEY:	24/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:		BCC ASSET ID:
MODEL ID:	LINDUM_02	AMTD (m):

STRUCTURE DESCRIPTION: **An RCBC crossing the Cleveland Railway.**

STRUCTURE SIZE: 2No. 3m x 1.5m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: 0.38 m

UPSTREAM OBVERT LEVEL: 1.88m

DOWNSTREAM INVERT LEVEL: 0.06 m

DOWNSTREAM OBVERT LEVEL: 1.56m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 20

LENGTH OF CULVERT BARREL AT OBVERT (m): 20

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? Yes, DTM survey completed by Schlencker Mapping. Survey number 080434.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 20

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

LOWEST POINT OF WEIR (m AHD): 2.60

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability greater than 1%. Railway partially inundated for these events.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	LINDUM_02
LOCATION	Cleveland Railway

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	12.998	2.895	2.758	137	*	0.04		1.444
0.2	11.775	2.5	2.399	101	*	0.12		1.308
1	10.143	2.164	2.091	73	-	-		1.127
2	9.765	2.069	2.012	57	-	-		1.085
5	9.53	1.94	1.895	45	-	-		1.059
10	8.545	1.847	1.811	36	-	-		0.95
20	8.18	1.736	1.704	32	-	-		0.909
50	6.614	1.571	1.562	9	-	-		0.751

NB: Results are based on existing stream conditions.

*Road on upstream end of culverts is completely inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**LINDUM_03**
LOCATION:	**Lindum Creek**

IMMUNITY RATING:	**1%**

DATE OF SURVEY:	24/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	LINDUM_03	AMTD (m):	

STRUCTURE DESCRIPTION: **A bridge crossing Lindum Creek.**

STRUCTURE SIZE: 3No. 3.6m x 1.2m

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: 0 m

UPSTREAM OBVERT LEVEL: 1.2

DOWNSTREAM INVERT LEVEL: 0 m

DOWNSTREAM OBVERT LEVEL: 1.2

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 14.6

LENGTH OF CULVERT BARREL AT OBVERT (m): 14.6

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 16

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

LOWEST POINT OF WEIR (m AHD): **2.5**

(Level at which water overtops road)

PIER WIDTH (m):

HYDRAULIC STRUCTURE REFERENCE SHEET

HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.5%. The entire lane on the road is inundated at this point.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**LINDUM_03**
LOCATION	**Lindum Creek**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	15.889	2.742	2.664	78	*	0.33		1.226
0.2	15.319	2.611	2.524	87	**	0.22		1.182
1	11.682	2.16	2.12	40	-	-		0.901
2	11.977	2.112	2.064	48	-	-		0.924
5	10.612	1.961	1.925	36	-	-		0.819
10	9.844	1.829	1.8	29	-	-		0.76
20	8.28	1.659	1.637	22	-	-		0.639
50	5.916	1.461	1.451	10	-	-		0.456

NB: Results are based on existing stream conditions.

*Road completely inundated

**Only upstream end of culverts is inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**LINDUM_04**
LOCATION:	**Port of Brisbane Motorway**

IMMUNITY RATING:	**0.2%**

DATE OF SURVEY:	17/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:		BCC ASSET ID:
MODEL ID:	LINDUM_04	AMTD (m):

STRUCTURE DESCRIPTION: **A main RCBC crossing the Port of Brisbane Motorway connecting Lindum creek.**

STRUCTURE SIZE: 1No. 10.25m x 2.191m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: 0m

UPSTREAM OBVERT LEVEL: 2.191m

DOWNSTREAM INVERT LEVEL: 0m

DOWNSTREAM OBVERT LEVEL: 2.191m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 12 m

LENGTH OF CULVERT BARREL AT OBVERT (m): 12 m

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 17

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

LOWEST POINT OF WEIR (m AHD): **2.5**

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the square ends of the culvert (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.2%. The flood extents significantly increase when for events with an AEP less than this.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	LINDUM_04
LOCATION	Port of Brisbane Motorway

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	19.19	2.66	2.63	30	*	0.11		0.854
0.2	15.88	2.52	2.5	20	-	-		0.707
1	12.48	2.12	2.11	10	-	-		0.597
2	12.61	2.06	2.05	10	-	-		0.609
5	10.88	1.92	1.91	10	-	-		0.565
10	10.05	1.80	1.79	10	-	-		0.567
20	8.62	1.63	1.62	10	-	-		0.523
50	6.21	1.45	1.44	10	-	-		0.425

NB: Results are based on existing stream conditions.

*Road not inundated, but low area of weir inundated for an indefinite length

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**LINDUM_05**
LOCATION:	**Lytton Road**

IMMUNITY RATING:	**0.05%**

DATE OF SURVEY:	31/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	LINDUM_05	AMTD (m):	

STRUCTURE DESCRIPTION: **A circular culvert crossing Lytton road, connecting Lindum Creek**

STRUCTURE SIZE: 5No. 1.5m dia.

For Bridges: Number of spans and their lengths

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: 0 m		UPSTREAM OBVERT LEVEL:	1.5m
DOWNSTREAM INVERT LEVEL: -0.07m		DOWNSTREAM OBVERT LEVEL:	1.43m
For culverts give floor level.		For bridges give bed level.	

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 22

LENGTH OF CULVERT BARREL AT OBVERT (m): 22

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 22	LOWEST POINT OF WEIR (m AHD):	2.3

HYDRAULIC STRUCTURE REFERENCE SHEET

(In direction of flow, i.e. distance from u/s face to d/s face) (Level at which water overtops road)

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.05%. Road is overtopped at an AEP of 0.05% but not from the culvert reaching capacity. It overtops due to upstream flooding along Lytton Road.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	LINDUM_05
LOCATION	Lytton Road

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	16.657	2.628	2.373	255	*	0.07		1.886
0.2	15.176	2.494	2.286	208	-	-		1.718
1	12.416	2.1	1.975	125	-	-		1.406
2	12.657	2.043	1.929	114	-	-		1.433
5	11.075	1.906	1.813	93	-	-		1.254
10	10.295	1.78	1.702	78	-	-		1.166
20	8.553	1.615	1.558	57	-	-		0.969
50	6.365	1.434	1.406	28	-	-		0.729

NB: Results are based on existing stream conditions.

*Road upstream of culvert completely inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**LINDUM_06**
LOCATION:	**Gosport Street**

IMMUNITY RATING:	**0.5%**

DATE OF SURVEY:	24/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:		BCC ASSET ID:
MODEL ID:	LINDUM_06	AMTD (m):

STRUCTURE DESCRIPTION: **An RCBC crossing Gosport street.**

STRUCTURE SIZE: 4No. 3m x 0.9m

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.132 m

UPSTREAM OBVERT LEVEL: 0.768m

DOWNSTREAM INVERT LEVEL: -0.137 m

DOWNSTREAM OBVERT LEVEL: 0.763m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 22

LENGTH OF CULVERT BARREL AT OBVERT (m): 22

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 22

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

LOWEST POINT OF WEIR (m AHD): 2.22

(Level at which water overtops road)

PIER WIDTH (m):

HYDRAULIC STRUCTURE REFERENCE SHEET

HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the circular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.5%. The entire lane on the road is inundated at this point.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	LINDUM_06
LOCATION	Gosport Street

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	17.478	2.362	2.217	145	*	0.11		1.618
0.2	16.457	2.274	2.116	158	**	0.03		1.524
1	13.628	1.962	1.856	106	-	-		1.262
2	12.996	1.916	1.828	88	-	-		1.203
5	11.566	1.798	1.727	71	-	-		1.071
10	10.216	1.69	1.635	55	-	-		0.946
20	8.148	1.546	1.513	33	-	-		0.754
50	6.19	1.396	1.378	18	-	-		0.573

NB: Results are based on existing stream conditions.

*Road upstream completely inundated, small amounts of flooding downstream

**Only road upstream is completely inundated

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**MAINDR_02**
LOCATION:	**Port of Brisbane Motorway**

IMMUNITY RATING:	**0.05%**

DATE OF SURVEY:	31/10/2014	UBD REF:	
SURVEYED CROSS SECTION ID:		BCC ASSET ID:	
MODEL ID:	MAINDR_02	AMTD (m):	

STRUCTURE DESCRIPTION: **A RCBC crossing POBM, and a main drainage system.**

STRUCTURE SIZE: 1No. 6m x 1.45m

For Culverts: Number of cells/pipes and sizes

For Bridges: Number of spans and their lengths

Where dimensions have been estimated, this should be clearly stated.

UPSTREAM INVERT LEVEL: 0.2 m

UPSTREAM OBVERT LEVEL: 1.65m

DOWNSTREAM INVERT LEVEL: 0.1m

DOWNSTREAM OBVERT LEVEL: 0.55m

For culverts give floor level.

For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 26

LENGTH OF CULVERT BARREL AT OBVERT (m): 26

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No.

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 26

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

LOWEST POINT OF WEIR (m AHD): 2.4

(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

PIER WIDTH (m):
HEIGHT OF GUARDRAILS (m AHD): DESCRIPTION OF ALL HAND AND GUARD RAILS AND HEIGHTS TO TOP AND UNDERSIDE OF GUARD RAILS:
ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the rectangular entry to the culvert due to abrupt entry (producing higher headwater levels). For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels. For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.
CONSTRUCTION DATE OF CURRENT STRUCTURE: PLAN NUMBER: HAS THE STRUCTURE BEEN UPGRADED? If yes, explain type and date of upgrade. Include plan number and location if applicable.
ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability greater than .05%. The lane closest to the upstream end of the culvert is overtopped at an AEP of 2% , but the lane at the downstream end remains immune until AEP events of 0.05%.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**MAINDR_02**
LOCATION	**POBM**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	12.083	2.779	2.623	156	*	0.11		1.389
0.2	7.239	2.494	2.447	47	*	0.005		0.832
1	5.88	2.156	2.133	23	-	-		0.676
2	5.323	2.033	2.023	10	-	-		0.612
5	4.054	1.906	1.896	10	-	-		0.466
10	3.639	1.794	1.786	8	-	-		0.418
20	3.286	1.7	1.691	9	-	-		0.378
50	2.662	1.534	1.528	6	-	-		0.318

NB: Results are based on existing stream conditions.

*Road not flooded, but lying area along weir is inundated for an indefinite length

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK:	**MAINDR_06 & MAINDR_05**
LOCATION:	**Gosport Street**

IMMUNITY RATING:	**0.5%**

DATE OF SURVEY:	17/10/2014	UBD REF:
SURVEYED CROSS SECTION ID:	**130170 Gosport St Culvert 20130709**	BCC ASSET ID:
MODEL ID:	MAINDR_06	AMTD (m):

STRUCTURE DESCRIPTION: **A main RCBC crossing Gosport street which connects the two waterway corridors together.**

STRUCTURE SIZE : 4No. 3.6m x 1.6m + 2 No 3.6 x1.8

For Culverts: Number of cells/pipes and sizes
Where dimensions have been estimated, this should be clearly stated.

For Bridges: Number of spans and their lengths

UPSTREAM INVERT LEVEL: -0.5m	UPSTREAM OBVERT LEVEL: 1.1m
DOWNSTREAM INVERT LEVEL:-0.68m	DOWNSTREAM OBVERT LEVEL: 0.92m
For culverts give floor level.	For bridges give bed level.

For Culverts

LENGTH OF CULVERT BARREL AT INVERT (m): 17m

LENGTH OF CULVERT BARREL AT OBVERT (m): 17m

TYPE OF LINING: Concrete

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

IS THERE A SURVEYED WEIR PROFILE? No

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
e.g. crown, kerb, hand rails, guard rails or whichever is higher.

WEIR WIDTH (m) 21.6	LOWEST POINT OF WEIR (m AHD): **1**
	(Level at which water overtops road)

HYDRAULIC STRUCTURE REFERENCE SHEET

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

PIER WIDTH (m):

HEIGHT OF GUARDRAILS (m AHD):

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

ADDITIONAL STRUCTURE DETAILS: An entry head loss factor of 0.5 was applied in the model due to the square ends of the culvert (producing higher headwater levels).

For culverts, wingwall/headwall details, entrance details e.g. pipe flush with embankment or projecting, socket or square end, entrance rounding, levels.

For bridges, details of piers and section under bridge including abutment details. Specify Survey Book No.

CONSTRUCTION DATE OF CURRENT STRUCTURE:

PLAN NUMBER:

HAS THE STRUCTURE BEEN UPGRADED?

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

ADDITIONAL COMMENTS: The structure has immunity to events with an Annual Exceedance Probability less than 0.5%. The model did not explore events of this magnitude hence the culvert has not reached capacity

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	**MAINDR_06&MAINDR_05**
LOCATION	**Gosport Street**

AEP (%)	DISCHARGE (m^3/s)	U/S WATER LEVEL (m AHD)	D/S WATER LEVEL (m AHD)	AFFLUX (mm)	FLOW WIDTH ABOVE STRUCTURE (m)	FLOW DEPTH ABOVE STRUCTURE (m)	VELOCITY (m/s)	
							Weir	Structure
0.05	26.611	1.952	1.94	8	46.9	0.05		0.736
0.2	24.842	1.925	1.915	10	16.1	0.03		0.644
1	16.156	1.494	1.486	8	0	0		0.459
2	16.156	1.437	1.429	8	0	0		0.419
5	15.043	1.388	1.382	6	0	0		0.385
10	13.702	1.327	1.321	6	0	0		0.31
20	11.825	1.287	1.281	6	0	0		0.284
50	9.416	1.205	1.2	5	0	0		0.222

NB: Results are based on existing stream conditions.

HYDRAULIC STRUCTURE REFERENCE SHEET

CREEK	
LOCATION	

Photograph looking upstream at structure

Photograph looking downstream at structure

APPENDIX D - Design Event Peak Flood Levels

The mean peak flood levels were extracted along a number of cross-sections and results are presented in this Appendix.

Scenario 3 Design Event Flood Levels (mAHD)

NAME	LABEL	50%	20%	10%	5%	2%	1%
HEMMANT DRAIN TRIBUTARY A	CH 0	3.07	3.15	3.19	3.25	3.34	3.42
HEMMANT DRAIN TRIBUTARY A	CH 100	-	3.25	3.31	3.36	3.4	3.43
HEMMANT DRAIN TRIBUTARY A	CH 200	-	-	-	-	-	-
HEMMANT DRAIN TRIBUTARY A	CH 300	-	-	-	-	-	-
HEMMANT DRAIN TRIBUTARY A	CH 400	-	-	-	-	-	-
HEMMANT DRAIN TRIBUTARY A	CH 500	-	-	-	-	-	-
HEMMANT DRAIN TRIBUTARY A	CH 600	-	-	-	-	-	-
BULIMBA MAIN DRAIN	CH 500	1.2	1.27	1.32	1.38	1.44	1.49
BULIMBA MAIN DRAIN	CH 900	1.23	1.32	1.37	1.43	1.49	1.56
LINDUM CREEK	CH 500	-	-	-	-	-	-
LINDUM CREEK	CH 700	-	-	-	-	-	-
LINDUM CREEK	CH 1100	1.58	1.74	1.83	1.96	2.13	2.26
HEMMANT DRAIN	CH 0	1.32	1.57	1.7	1.88	2.09	2.27
HEMMANT DRAIN	CH 100	-	1.62	1.76	1.96	2.18	2.35
HEMMANT DRAIN	CH 200	1.46	1.64	1.77	1.96	2.19	2.36
HEMMANT DRAIN	CH 300	1.49	1.66	1.79	1.97	2.2	2.37
HEMMANT DRAIN	CH 400	1.65	1.77	1.89	2.04	2.23	2.39
HEMMANT DRAIN	CH 500	1.66	1.78	1.91	2.06	2.26	2.42
HEMMANT DRAIN	CH 600	1.66	1.79	1.91	2.06	2.27	2.42
HEMMANT DRAIN	CH 700	1.67	1.79	1.92	2.07	2.27	2.43
HEMMANT DRAIN	CH 800	1.68	1.8	1.93	2.08	2.28	2.43
HEMMANT DRAIN	CH 900	1.68	1.81	1.93	2.08	2.28	2.44
HEMMANT DRAIN	CH 1000	1.69	1.82	1.94	2.09	2.29	2.44
HEMMANT DRAIN	CH 1100	1.7	1.83	1.95	2.1	2.29	2.44
HEMMANT DRAIN	CH 1200	1.72	1.85	1.97	2.12	2.3	2.45
HEMMANT DRAIN	CH 1300	2.03	2.18	2.27	2.36	2.43	2.5
HEMMANT DRAIN	CH 1400	2.04	2.19	2.28	2.37	2.44	2.5
HEMMANT DRAIN	CH 1500	2.05	2.2	2.28	2.37	2.45	2.51
HEMMANT DRAIN	CH 1600	2.05	2.2	2.29	2.37	2.45	2.51
HEMMANT DRAIN	CH 1700	2.05	2.2	2.29	2.37	2.45	2.51
HEMMANT DRAIN	CH 1800	2.05	2.2	2.29	2.37	2.45	2.51
HEMMANT DRAIN	CH 1900	2.05	2.2	2.29	2.37	2.45	2.51
HEMMANT DRAIN	CH 2000	2.05	2.2	2.29	2.38	2.45	2.51
HEMMANT DRAIN	CH 2100	2.05	2.2	2.29	2.38	2.45	2.51
HEMMANT DRAIN	CH 2200	2.05	2.2	2.29	2.38	2.45	2.51

NAME	LABEL	50%	20%	10%	5%	2%	1%
HEMMANT DRAIN	CH 2300	2.06	2.21	2.29	2.38	2.46	2.52
HEMMANT DRAIN	CH 2400	2.07	2.21	2.29	2.38	2.46	2.52
HEMMANT DRAIN	CH 2500	2.1	2.22	2.3	2.39	2.47	2.53
HEMMANT DRAIN	CH 2600	2.13	2.23	2.32	2.4	2.48	2.54
HEMMANT DRAIN	CH 2700	2.17	2.27	2.35	2.44	2.52	2.58
HEMMANT DRAIN	CH 2800	2.67	2.89	2.98	3.07	3.16	3.28
HEMMANT DRAIN	CH 2900	2.7	2.91	3	3.09	3.18	3.26
HEMMANT DRAIN	CH 3000	2.71	2.92	3	3.1	3.19	3.26
HEMMANT DRAIN	CH 3100	2.73	2.93	3.01	3.1	3.2	3.26
HEMMANT DRAIN	CH 3200	-	-	-	3.18	3.28	3.36
HEMMANT DRAIN	CH 3300	2.87	2.99	3.08	3.18	3.28	3.37
HEMMANT DRAIN	CH 3400	3.04	3.12	3.16	3.24	3.34	3.41
HEMMANT DRAIN	CH 3500	3.24	3.35	3.4	3.44	3.49	3.52
HEMMANT DRAIN	CH 3600	3.41	3.53	3.59	3.65	3.71	3.75
HEMMANT DRAIN	CH 3700	3.51	3.64	3.7	3.75	3.81	3.85
HEMMANT DRAIN	CH 3800	3.68	3.82	3.87	3.93	4	4.04
HEMMANT DRAIN	CH 3900	3.84	4	4.06	4.14	4.22	4.27
HEMMANT DRAIN	CH 4000	3.86	4.02	4.09	4.16	4.25	4.31
HEMMANT DRAIN	CH 4100	3.89	4.05	4.12	4.2	4.29	4.35
HEMMANT DRAIN	CH 4200	3.93	4.1	4.17	4.26	4.35	4.42
HEMMANT DRAIN	CH 4300	4.71	4.99	5.21	5.39	5.58	5.7
HEMMANT DRAIN	CH 4400	5.55	5.73	5.79	5.88	5.99	6.08
HEMMANT DRAIN	CH 4500	6.04	6.34	6.42	6.52	6.63	6.71
HEMMANT DRAIN	CH 4600	6.68	7.11	7.19	7.28	7.39	7.47
HEMMANT DRAIN	CH 4700	7.41	8.49	8.54	8.61	8.69	8.75
HEMMANT DRAIN	CH 4800	8.09	8.57	8.66	8.77	8.88	8.97
HEMMANT DRAIN TRIBUTARY B	CH 0	3.15	3.28	3.3	3.35	3.4	3.43
HEMMANT DRAIN TRIBUTARY B	CH 100	3.15	-	-	-	-	-
HEMMANT DRAIN TRIBUTARY B	CH 200	3.2	3.41	3.46	3.53	3.59	3.65
HEMMANT DRAIN TRIBUTARY B	CH 300	3.59	3.78	3.81	3.84	3.87	3.91
HEMMANT DRAIN TRIBUTARY B	CH 400	-	-	-	-	-	-
HEMMANT DRAIN TRIBUTARY B	CH 500	-	-	-	-	-	-
LINDUM CREEK	CH 0	1.02	1.03	1.05	1.08	1.11	1.13
LINDUM CREEK	CH 100	1.13	1.2	1.23	1.27	1.33	1.37
LINDUM CREEK	CH 200	1.21	1.31	1.34	1.39	1.46	1.51
LINDUM CREEK	CH 300	-	-	-	-	1.56	1.61
LINDUM CREEK	CH 400	-	-	-	-	-	-
LINDUM CREEK	CH 600	-	-	-	-	-	-
LINDUM CREEK	CH 800	1.52	1.67	1.73	1.83	1.95	2.05
LINDUM CREEK	CH 900	1.57	1.73	1.81	1.95	2.12	2.24
LINDUM CREEK	CH 1000	1.57	1.73	1.82	1.95	2.12	2.24

NAME	LABEL	50%	20%	10%	5%	2%	1%
LINDUM CREEK	CH 1200	1.59	1.74	1.83	1.96	2.14	2.26
LINDUM CREEK	CH 1300	1.6	1.76	1.87	2.01	2.19	2.33
LINDUM CREEK	CH 1400	1.6	1.76	1.87	2.01	2.2	2.33
LINDUM CREEK	CH 1500	1.6	1.76	1.87	2.01	2.2	2.34
LINDUM CREEK	CH 1600	1.6	1.77	1.87	2.01	2.2	2.34
BULIMBA MAIN DRAIN	CH 0	1	1.02	1.03	1.06	1.08	1.11
BULIMBA MAIN DRAIN	CH 100	1.13	1.18	1.21	1.25	1.28	1.32
BULIMBA MAIN DRAIN	CH 200	1.17	1.24	1.29	1.34	1.39	1.44
BULIMBA MAIN DRAIN	CH 300	1.18	1.26	1.3	1.36	1.41	1.47
BULIMBA MAIN DRAIN	CH 400	1.19	1.27	1.31	1.37	1.43	1.48
BULIMBA MAIN DRAIN	CH 600	1.2	1.28	1.33	1.39	1.45	1.5
BULIMBA MAIN DRAIN	CH 700	1.21	1.29	1.34	1.4	1.46	1.51
BULIMBA MAIN DRAIN	CH 800	1.22	1.3	1.35	1.42	1.48	1.54
BULIMBA MAIN DRAIN	CH 1000	1.26	1.35	1.4	1.47	1.54	1.6
BULIMBA MAIN DRAIN	CH 1100	1.29	1.39	1.44	1.51	1.59	1.64
BULIMBA MAIN DRAIN	CH 1200	-	1.73	1.84	1.97	2.14	2.28

APPENDIX E - Models Peer Review and Response

Dedicated to a better Brisbane

Brisbane City Council

MEMORANDUM

To:	**Richard Yearsley – Program Officer– Natural Environment, Water and Sustainability**	Date:	**20.11.2014**
Via:	**Evan Caswell – Principal Engineer, Flood Management**		
From:	**Hanieh Zolfaghari - Engineer, Flood Management**		
Re:	**Peer Review of Hemmant Lytton Flood Study**		

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1. Introduction

The purpose of this memorandum is to summarise the peer review undertaken by City Project's Office on the Hemmant Lytton Flood Study project. The study was undertaken by BMT WBM.

The peer review has been undertaken to ensure:

- Council has reviewed all required data associated with the Hemmant Lytton Flood Study (BMT WBM 2014) to enable future adoption into Council systems
- The flood study has been delivered in accordance with Council procedures and methods current at the time the study was undertaken
- The output is fit for purpose

The peer review includes a high level technical review of the models and results. It has been undertaken in four parts, namely;

- Base hydrology model review
- Calibrated hydrology and hydraulic model review, and,
- Design hydraulic model
- Extreme events and sensitivity analysis and report review

It is assumed that BMT WBM have applied best-practice Quality Assurance in producing the flood study and that the work has been prepared under suitably qualified RPEQ supervision as is required by State law.

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

2. Hydrology Model

The existing XP-RAFTS hydrologic model was checked and updated and extended by BMT WBM. The modelled catchment covers the whole catchment extents as specified in the project brief. The base XP-RAFTS hydrological model was reviewed by the Council. The following comments (in black) in relation to the review were provided by BCC to BMT WBM, with their response shown (in red) where applicable.

- Percentage impervious values for the hydrology model stated in the report are generally lower than expected. BMT WBM should use the Council adopted percentage impervious provided to them.

  BMT WBM Response: The percentage impervious values provided by Council were subsequently used. The final results presented in the report are based on the revised values.

- The sub-catchment names provided in MapInfo do not match the sub-catchment names in the RAFTS model in several locations.

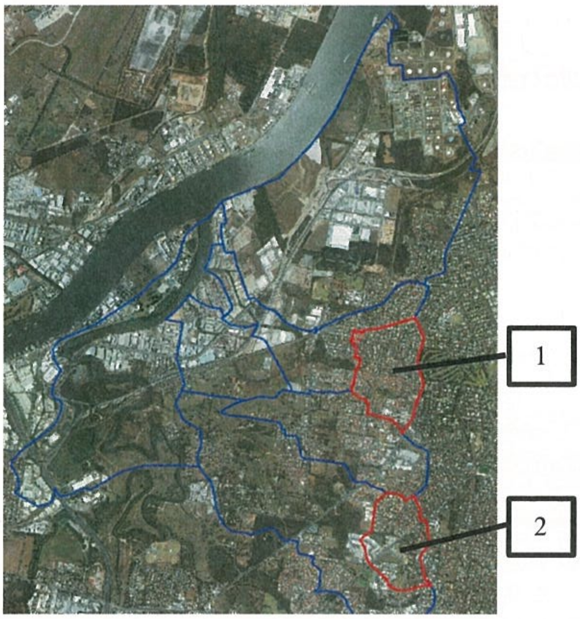
  BMT WBM Response: The sub-catchment names in the GIS layer have been amended to match the RAFTS model.

- The link routing and lag times should be updated in the hydrology model. As discussed all defined channel and defined flow path should be routed through the cross sections.

  BMT WBM Response: The catchment delineation is unusually fine, resulting in many sub-catchments. As such there are many links. Given the complexity of the floodplain dynamics, and the inherent limitations of the hydrological model to represent the floodplain dynamics, BMT WBM regarded the simple linking approach using lag times as the most efficient and practical method (rather than routing through cross-sections). Lag times have been estimated for all sub-catchment links in the RAFTS model.

- The total flows from hydrology model were not verified against any other method like rational method.

  BMT WBM Response: The catchment areas that were not hydraulically modelled are generally very small upper catchment areas. The hydrology model has been compared to the rational method for the largest two catchment areas that were not hydraulically modelled (see image below). The methodology outlined by the Queensland Urban Drainage Manual (DEWS, 2013) was followed in order to obtain these results.



The QUDM process of the rational method was undertaken to determine the following parameters for sub-catchments 1 and 2 as labelled in the above image.

Parameter	Sub-catchment 1	Sub-catchment 2
Area	926, 000 m^2	767, 600m^2
Percent Impervious	56.01	67.57
C_{100}	0.969	0.983
${}^{0.5}_{100}I$	148 mm/h	148 mm/h

From these parameters, the peak flow was calculated and is shown below.

Catchment ID	RAFTS Results	Rational Method Result
Sub-catchment 1	44.81 m^3/s	36.91 m^3/s
Sub-catchment 2	29.68 m^3/s	31.01 m^3/s

As evident from the results, the RAFTS hydrology model gives a sensible estimate as verified by the Rational Method. It should be noted that the Rational Method is an extremely lumped and approximate method, hence should only be used to verify the correctness of hydrology models by giving a 'ball-park' estimation of the peak flow rate. Considering the differences of the two methods give an error margin within the range of 4% - 18%, the RAFTS hydrology model is considered to give suitable results and is considered verified for the hydrologic model of choice for this study.

The Flood Management peer reviewer has noted some inconsistencies between the XP-RAFTS and the Rational Method results. However, the XP-RAFTS in general provides a more conservative result and therefore it is considered acceptable.

- The calibration hydrology model should be updated using existing landuse (Based on 2012 Aerial Photo).

  BMT WBM Response: This was done prior to finalisation of the calibration and verification.

- WBM is to ensure that the catchment delineation represents the current topography and drainage system in proximity of Port of Brisbane Motorway based on latest TUFLOW model of Port of Brisbane Motorway upgrade.

  BMT WBM Response: Council provided this model to BMT WBM. The topography was checked and matched the infrastructure observed in aerial photography that was used to delineate the catchment.

Based on the comments submitted by BMT WBM in response to Council's review (above), the model was accepted in its current form. However, for future work it is recommended to use coarser scale sub-catchments as the current sub-catchments are very refined which is unnecessary for the flood modelling at this scale.

3. Hydraulic Model

The calibrated hydrologic and hydraulic models were reviewed. BMT WBM has developed a TUFLOW hydraulic model for the catchment. The model includes the lower part of Bulimba Creek in order to properly represent the flood behaviour at the confluence of Bulimba Creek and Hemmant Drain and Lindum Creek. The following comments (in black) in relation to the review were provided by BCC to BMT WBM, with their response shown (in red) where applicable.

Model Calibration

- Please demonstrate that consistency between hydraulic and hydrology model has been achieved.

  BMT WBM Response: This consistency check has been done and is discussed in the report.

- Check made and Rafts model outflows match TUFLOW model inflows. However the following sub-catchments are missing from TUFLOW inflows: A2_4, A4_BOG, A4_Cal, A4_5, I0_13A, I0_14A and G0_6A.

  BMT WBM Response: This was due to some upper catchments being lumped in the hydraulic model using total flows rather than local flows.

- Please make sure all handrails are blocked for design event scenarios.

  BMT WBM Response: This was done using Z lines to add a specified fence height to the underlying road levels.

- The set-up of structures was checked and most adopted parameters seem to be within an acceptable range, however, the following needs to be clarified:

  - Adoption of very high form loss at structure C2325P. BMT WBM Response: This was a mistake and has been corrected. It is noted that this structure is redundant as it is outside the floodplain.
  - Inclusion of Dummy_xx structure in 1d_nwk_DSBDY_01 table. BMT WBM Response: This is a redundant channel that was added during debugging and is no longer required. It has now been removed
  - Adoption of "7" as UCS attributes for couple of structures. BMT WBM Response: This has been corrected.

- _H.csv, _Q.csv and PO.csv hydrographs spot checked for all three calibration events at random locations. Model instabilities were observed. BMT WBM should review the network elements with instabilities. The instabilities were specifically observed at the Lindum Creek structures and 1d channel and downstream of Kianawah Road. BMT WBM Response: This has been resolved.

- The inflow set up (local inflows) in the calibration model may cause some inconsistency in modelling of the ultimate scenario. Total inflows may be adopted in some areas within the waterway corridor in order to use the unchanged calibration model to run design events or alternatively waterway corridor should artificially be extended further upstream. The issue was also discussed with BMT WBM in progress meeting. BMT WBM Response: This has been resolved.

- Please clarify why the multiplier of 1.02 has been used for Pinkenba downstream boundary condition. BMT WBM Response: This was used for scaling of levels at Pinkenba compared to Brisbane Bar, as per the semidiurnal tidal planes from MSQ Queensland. This scaling is done prior to conversion to mAHD, thus ensuring that the low tide is lifted rather than lowered (due to being below zero in mAHD).

- The area between the rail line, Pritchard Street and Port Drive does not drain anywhere, WBM should inspect the area and deliver a solution. BMT WBM Response: A site inspection was undertaken, but the road controlling flow is on private land and not accessible. An assumption was made on the drainage across the private road.

Design Events Model

- There has been some flow leakage from the waterway corridor in Scenario 3 (ie, the flood extent leaks outside the waterway corridor). BMT WBM Response: The method used to represent the waterway corridor has been changed to prevent this 'leakage'.

- Review and revise the WLL lines to ensure the mapping of 1D channel is correct. In particular, WLL lines at upstream and downstream of the road crossings and structures. BMT WBM Response: This has been resolved.

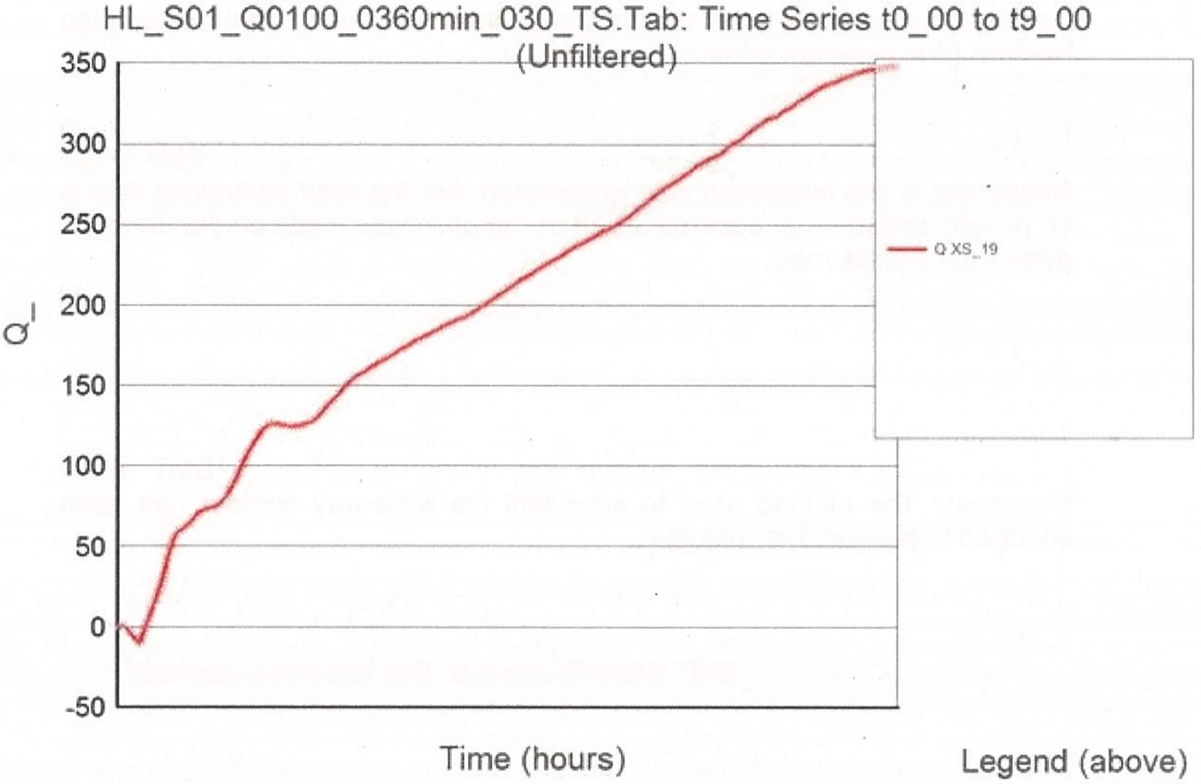
- Waterway Corridor should be connected along each waterway for the purpose of modelling to avoid isolated ponding areas. In Particular, area downstream of Pritchard Street and area upstream of School Road has to be connected to the rest of flood extent and areas upstream and downstream of roads should also be connected. BMT WBM Response: This has been resolved.

- Include PO line (Reporting line) at all road crossings. BMT WBM Response: This has been resolved.

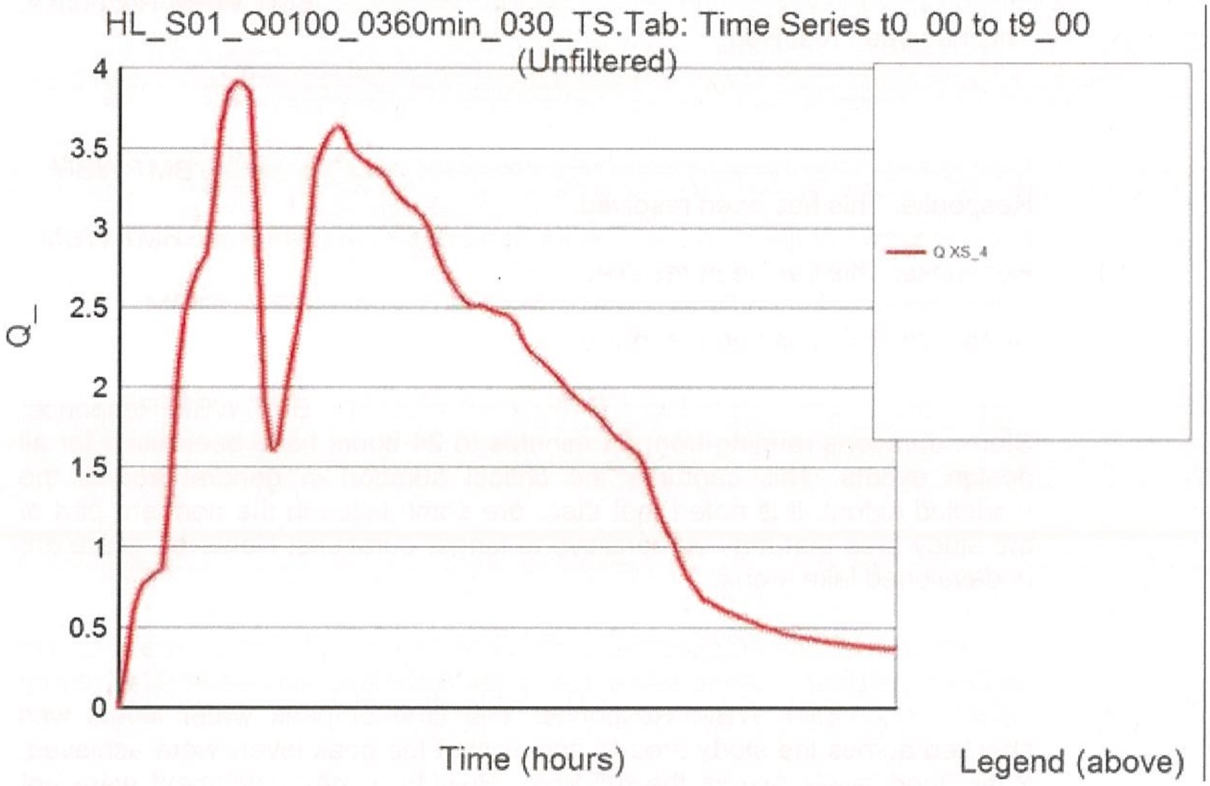
- Model instabilities were observed at the following locations:
  - East of Foley Road immediately downstream of the 1D channel. BMT WBM Response: This has been resolved.
  - Road crossing at the corner of Evelyn Road and Kianawah Road BMT WBM Response: This has been resolved.
  - Downstream of Lytton Road (downstream of Chan4Lytt) BMT WBM Response: This has been resolved.

- Please ensure the critical duration has been modelled. BMT WBM Response: Storm durations ranging from 30 minutes to 24 hours have been used for all design events. This captures the critical duration in general across the modelled extent. It is noted that there are some lakes in the northern part of the study area that may be sensitive to longer durations. However, these are undeveloped lake areas.

- In some of the runs it appears that the flood levels in Bulimba Creek has not reached the peak. Please make sure peak flood level has been captured by current runs. BMT WBM Response: The time of peak water levels was checked across the study area to ensure that the peak levels were achieved. Peak flood levels across the full lower Bulimba Creek catchment were not achieved for all storm durations. However, peak levels for the investigation areas for this flood study have been achieved for all design events.



- Please check the discharge and flood levels at the POline XS_4 to ensure it is inherited from the rainfall pattern and it's not model instability. BMT WBM Response: As per rainfall pattern.



Extreme Events Modelling and Sensitivity Analysis

- For blockage scenarios, blockage (%) adopted in the model is what has been referred to as % inlet blockage in QUDM and not % barrel blockage. Please clarify. BMT WBM Response: Barrel blockage from sedimentation is not a form of blockage that would occur suddenly on rising floodwaters (may occur as flood levels drop) and is not a form of blockage we have typically applied for flood studies. So the inlet blockage was adopted (considers debris in flood waters) and applied to the barrel as TUFLOW does not model the inlets independently.

Based on the comments submitted by BMT WBM in response to Council's review (above), the TUFLOW hydraulic model is considered to be fit for purpose with no observed errors.

4. Report Review and General Comments

- 'hazard' should be 'depth velocity product'. BMT WBM Response: This has been resolved.

- A conversion table for ARI and AEP is required in the report. BMT WBM Response: This has been resolved.

- Figure 1.1 shows an incorrect catchment boundary and it includes the lower part of the Bulimba Creek catchment. What has been shown is study area and not catchment boundary. Please revise the figure. BMT WBM Response: This has been resolved.

- Page 6, Sec 3.2.1: Does it mean that no survey from provided survey data was used in the model? Please clarify. BMT WBM Response: Correct, this has been clarified in the report. Survey had been incorporated by AURECON for the Tilley Road assessment.

- Add rainfall gauges to figure 4.1 and provide a reference to Figure 4.1 in section 3.3. BMT WBM Response: This has been resolved.

- Explain in the report what downstream boundary condition has been adopted for each scenario including standard design events, extreme events and climate change scenarios. BMT WBM Response: This has been resolved.

- As stated in the report, adopted methodology to model MRC in the hydraulic model (20m and 0.12 manning's n) for Scenarios 2 and 3 is different with what has been stated in the Council's Flood Study Procedure. Please clarify. BMT WBM Response: This was a typo in the report and has been corrected.

- Hydraulic model should be tidied up. All pipes and culverts outside modelled area should be removed from the model files. BMT WBM Response: This has been resolved in part. However, some redundant 1D elements still exist, which was only identified after the model runs had been completed.

5. Conclusion

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

CPO has made sure that the work has been undertaken in accordance with the required standards and procedures through the reviews documented above, and through regular communication and meetings with BMT WBM. We acknowledge that BMT WBM has appropriately addressed the issues/concerns as noted by Council throughout the review process.

Hanieh

………………..

Hanieh Zolfaghari
Flood Engineer
Flood Management

ERCaswell

………………..

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

Appendix A – Calibrated Hydraulic Model Peer Review Checklist

1.0 Project Details

Project Name:	Hemmant Lytton Flood Study
Client:	NEWS - BCC
Project Job Number:	BUD No AB07/AO29 -- Job No 140502/140516
Date:	27/03/2014 (model files dated 13/03/2014)
Modellers Name:	Richard Sharpe (Modeller) - Jo Tinnion (RPEQ 11395)
Modellers Organisation:	BMT WBM
Reviewers Name:	Hanieh Zolfaghari (Reviewer) - Evan Caswell (RPEQ 10498)
Reviewers Organisation:	Flood Management - BCC
Major Catchment Name:	Bulimba Creek, Hemmant Lytton Lindum Creek
Creek Name:	Hemmant Drain, Lindum Creek and Lytton Overland Area

Review Status

- ☑ Model Build
- ☑ Calibration / Verification
- ☑ Design Modelling
- ☑ Handover

Purpose of Model

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

Modelling software- Hydrology

- ☑ XP-RAFTS
- ☐ WBNM
- ☐ URBS
- ☐ Other (specify)

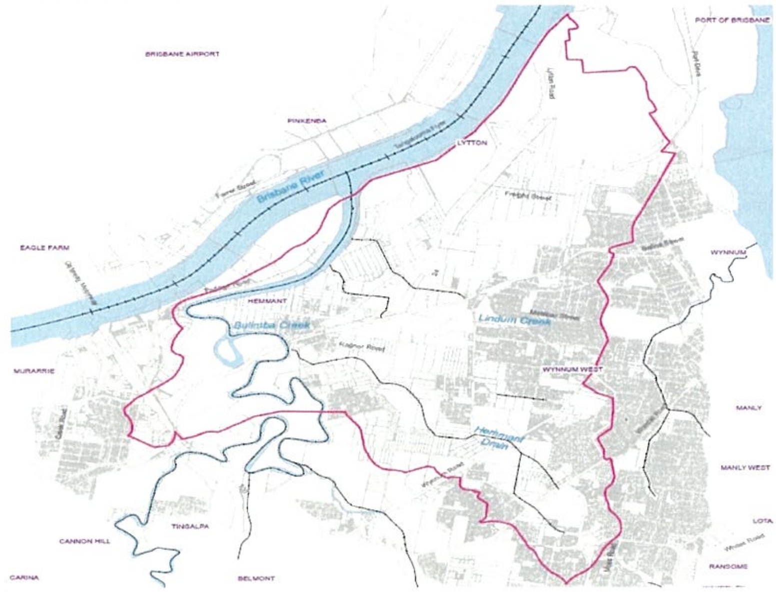
Modelling software- Hydraulic

- ☑ TUFLOW — ☑ 1D / 2D — ☐ 2D
- ☐ MIKE 11
- ☐ MIKEFLOOD — ☐ 1D / 2D — ☐ 2D
- ☐ HEC-RAS — ☐ Steady — ☐ Unsteady
- ☐ XP-SWMM / XP-STORM
- ☐ Other (specify)

Further description of the modelling

Catchment includes Hemmant Drain, Lindum Creek, Lytton overland flow areas. Downstream part of Bulimba Creek Has Also been included in the model to ensure coincident flooding from Bulimba Creek catchment has also been considered Bulimba Creek lower catchment was included and modelled coarsely in order to simulate the flood behaviour at the confluence Tilley Road Stage 2 model was used as a base of hydraulic model and Port of Brisbane Motorway - GHD model was also used to include Port of Brisbane Motorway in the model. The 1997 Hydrology model was used as a base for hydrology model and was combined and further developed and extended for this study. The hydrology model is very fine scale model It caused some issues and it was agreed with WBM to use the total inflows at some locations due to the location of WC and consistency between Existing and Ultimate Model.

The model has been reviewed at different stages of the project including review of the base hydrology model, calibration review, design event modelling review and extreme events and sensitivity runs and finally draft report



2.1 Hydraulic Model Build - Model Extents

Model extents as per the study brief?
☐ Yes ☐ N/A
☑ No — Scope of work changed and model extent changed (decreased) as well.

Are the extents of the model sufficient to prevent glass walling?
☑ Yes
☐ No

Model extends sufficiently upstream / downstream of the study area to negate boundary effects?
☑ Yes ☐ N/A
Bulimba Creek downstream catchment was included
☐ No

Model extents sufficient to capture potential afflux limits?
☑ Yes ☐ N/A
☐ No

2.2 Hydraulic Model Build - Channel Representation

Origin of bathymetry data
Source of topographic data described in consultancy brief and flood study report - ALS and ground survey; no new survey undertaken

Origin of each cross-section defined in the report?
☐ Yes ☐ N/A
☑ No

Precision of bathymetry data
The only bathymetry data available is from 1997 ground survey
Age of data reduces reliability.

Channel representation in the model
☑ 1D Channel
☑ 2D grid — Grid size = 4m
☐ Flexible mesh

Cross-sections geo-referenced?
☑ Yes ☐ N/A
☐ No

Cross-section spacing sufficient?
☑ Yes ☐ N/A — They are typically adequately spaced
☐ No

Cross-sections perpendicular to flow?
☑ Yes ☐ N/A
☐ No

Spacing of sections agree with chainage?
☑ Yes ☐ N/A
☐ No

Channel reach lengths represented adequately
☑ Yes ☐ N/A
☐ No

Cross-sections left to right when viewed in the downstream direction (BCC preferred)?
☑ Yes ☐ N/A
☐ No

Are interpolated cross-sections used?
☐ Yes ☐ N/A
☑ No

(If yes, state why)

Top of bank / section markers represented adequately?
☑ Yes ☐ N/A
☐ No

Manning's n for the channel represented adequately?
☑ Yes
☐ No

Manning's n categories defined in the report?
☑ Yes
☐ No

Most applicable radius type selected (MIKE11 only)?
☐ Yes ☑ N/A
☐ No

Conveyance checks undertaken?
☐ Yes ☑ N/A
☐ No

Is the channel generally represented adequately in the model?
☑ Yes — Refer comments below
☐ No

Other Comments / Issues

The original scope was to include overland flow area as the model was also to be used for neighbourhood planning project

However the scope changed as modelling overland flow caused some inconsistency between different hydraulic model scenarios. It was decided to apply the inflows at the start of waterway corridor or in some area waterway corridor was artificially extended further upstream

Model is a 1D/ 2D model using a 4m grid. 1D elements used for structures and open channels (low flow channel- bank to bank).

Generally 2009 ALS used with available channel survey (1997 cross sections of main channel) which was used in the Tilley Road Model

Lower part of Bulimba Creek was modelled using Zline instead of cross sections, however its considered suitable as it only simulates the downstream flood behaviour

No survey was undertaken as part of 2014 Hemmant Lytton Flood Study

Overall, the catchment topography is represented sufficiently for use in a catchment scale model assessing large and extreme events.

Survey info and cross sections used in the model were not checked

3.0 Hydraulic Model Build - Floodplain Representation

Origin of topographic data

ALS 2009

ALS 2002 for north of Port Drive to supplement the 2009 ALS data

Survey data used in the Tilley road model

No additional survey data has been included

Precision of topographic data

Standard ALS accuracy

Model in the lower bulimba creek has been represented coarsely and only by zline and not

the actual survey data. Should be ok for the purpose of simulate the boundary condition

Floodplain representation in the mode

- ☐ Extended 1D sections
- ☐ Quasi 2D
- ☑ 2D Grid — Grid size = 4m
- ☐ Multiple sized 2D Grids — Grid sizes =
- ☐ Flexible mesh

Is the floodplain representation consistent with the study objectives and data limitations	☑ Yes	☐ No	☐ N/A
Channel / floodplain interface represented adequately?	☑ Yes	☐ No	☐ N/A
Channel breakout flows represented adequately?	☑ Yes	☐ No	☐ N/A
Major obstructions (e.g buildings represented adequately?	☑ Yes	☐ No	☐ N/A
Floodplain storage adequately represented	☑ Yes	☐ No	☐ N/A
Cross-sections perpendicular to flow on floodplain (1D / Quasi 2D)?	☐ Yes	☐ No	☑ N/A
Floodplain reach lengths represented adequately	☐ Yes	☐ No	☑ N/A
Ineffective flow areas considered and represented adequately (if applicable)	☐ Yes	☐ No	☑ N/A
Manning's n for the floodplain represented adequately?	☑ Yes	☐ No	
Manning's n categories defined in the report?	☑ Yes	☐ No	
Is the floodplain generally represented adequately in the model?	☑ Yes	☐ No	

Other Comments / Issues

Most major road embankments represented in ALS and some TUFLOW modification tools were also used to improve the model resolution

Other major hydraulic controls are structures (represented as 1D structures and deck levels and handrails represented as Zline)

Manning's 'n' value represented in the model is based on City Plan land use maps which is standard for catchment scale models

4.0 Hydraulic Model Build - Bridges

Number of bridges in the model: 1 - Port Of Brisbane Motorway

Repeat the following for each bridge structure

Bridge name / bridge reference	Chan4Lytt	
River / creek name	Lytton Catchment	
Origin of bridge data	Port Of Brisbane Motorway Model - GHD model	
- Bridge structure		
- Upstream / downstream cross-sections		
- Road / weir profile		
Bridge modelling approach (e.g. Energy, WSPRO, USBPR, etc)	TUFLOW 1D bridge structure routine	
Is the bridge modelling approach the most applicable for the structure?	[x] Yes [ ] No	[ ] N/A
(If no, state why)		
Are there dual bridges (e.g. dual carriage motorway)	[ ] Yes [x] No	
If yes, are the bridges represented individually or combined?	[ ] Individual structures [ ] Combined structure	[x] N/A
If yes, is this representation considered the most appropriate?	[ ] Yes [ ] No	[x] N/A
(If no, state why)		
Is the bridge skewed to the normal flow direction?	[ ] Yes [x] No	
If yes, have the skew effects been represented adequately?	[ ] Yes [ ] No	[x] N/A
(If no, state why)		
Is this structure being modelled as part of a group of structures? (e.g. bridge plus floodplain relief culverts)	[ ] Yes [x] No	[ ] N/A
If yes, is this representation considered the most appropriate?	[ ] Yes [ ] No	[x] N/A
Cross-sections located at the most appropriate location for the modelling software and bridge routine?	[x] Yes [ ] No	[ ] N/A
Bridge dimensions (opening area/deck/ piers/abutments) correctly represented?	[ ] Yes [ ] No	Not checked as it was borrowed from GHD model and its assumed to be correct- DTMR recommended the model
Does the weir profile represent the highest elevation along the road?	[x] Yes [ ] No	[ ] N/A Not checked
Are the bridge coefficients reasonable? (e.g. weir, friction, pier, contraction / expansion, orifice, submergence, etc)	[x] Yes [ ] No	Checked but not recalculated.Its within the acceptable range
Handrail / guardrail blockage considered?	[x] Yes [ ] No	[ ] N/A
Handrail / guardrail represented adequately?	[x] Yes [ ] No	[ ] N/A
Headlosses at hydraulic structures appear logical / sensible?	[x] Yes [ ] No	[ ] N/A Report needs to tabulate losses
Headlosses at hydraulic structures checked by an alternate method?	[ ] Yes [x] No	[ ] N/A No , the bridge was borrowed from PoBM mode and it was assumed it is suitable to be used in the model
Is there a need to compare or calibrate the bridge modelling with another method?	[x] Yes [ ] No	[ ] N/A It could be beneficial
Hydraulic Structure Reference Sheets provided in report	[x] Yes [ ] No	[ ] N/A

Other Comments / Issues

Structure set up has not been checked in detail

Bridge has been modelled using TUFLOW 1D bridge structure

Pier losses represented by a form loss of 0.2 which is within acceptable range

5.0 Hydraulic Model Build - Culverts

Number of culverts in the model: 49

Repeat the following for each culvert

Culvert name / culvert reference: Culverts not checked in detail - However, spot checks of the culverts
River / creek name: information, including losses, dimensions, mannings n were undertaken to
Origin of culvert data: make sure they are with the acceptable range and no error has occurred
- Culvert details
- Upstream / downstream cross-sections
- Road / weir profile

Are there dual structures (e.g. dual carriage motorway)	☐ Yes ☐ No	
If yes, are the culvert structures represented individually or combined?	☐ Individual structures ☐ Combined structure	
If yes, is this representation considered the most appropriate?	☐ Yes ☐ No	☐ N/A
(If no, state why)		
Is the road/weir skewed to the normal flow direction?	☐ Yes ☐ No	
If yes, have the skew effects been represented adequately?	☐ Yes ☐ No	☐ N/A
(If no, state why)		
Is there a trash rack at the culvert inlet?	☐ Yes ☐ No	
If yes, have the headlosses been represented adequately?	☐ Yes ☐ No	☐ N/A
Is this structure being modelled as part of a group of structures? (e.g. culvert plus floodplain relief culverts)	☐ Yes ☐ No	☐ N/A
If yes, is this representation considered the most appropriate?	☐ Yes ☐ No	☐ N/A
Cross-sections located at the most appropriate location for the modelling software and bridge routine?	☐ Yes ☐ No	
Culvert dimensions correctly represented?	☐ Yes ☐ No	
Does the weir profile represent the highest elevation along the road?	☐ Yes ☐ No	☐ N/A
Are the culvert coefficients reasonable? (e.g. weir, friction, inlet / outlet, contraction / expansion, etc)	☐ Yes ☐ No	
Handrail / guardrail blockage considered?	☐ Yes ☐ No	☐ N/A
Handrail / guardrail represented adequately?	☐ Yes ☐ No	☐ N/A
Headlosses at hydraulic structures appear logical / sensible?	☐ Yes ☐ No	☐ N/A
Headlosses at hydraulic structures checked by an alternate method?	☐ Yes ☐ No	☐ N/A
Hydraulic Structure Reference Sheets provided in report	☐ Yes ☐ No	☐ N/A

Other Comments / Issues

Culvert structures not individually checked
Culverts modelled using TUFLOW 1d culvert structure routine
Reasonable loss factors applied
Handrails have been modelled as 100% blocked
No trashscreens have been considered

6.1 Hydraulic Model Build - Outflow Boundary Conditions (generic not run specific)

Downstream boundary

- [ ] Normal depth
- [ ] Rating Curve
- [x] Specified WL — For design even
- [x] Head v Time — For calibration even
- [ ] Other — For Bulimba Creek to be confirmed if H vs time used or Q vs tim

Origin / Derivation of downstream boundary

Calibration events have time series (water level) applied from event
using Brisbane Bar tide data
Design events have fixed level - MHWS or HAT
Climate Change events have fixed level MHWS+300/800mm or HAT+300/800mm

Will selection of the downstream boundary significantly influence results?

- [x] Yes
- [ ] N/A
- [ ] No — Its very flat in downstream are

Is the downstream boundary appropriate

- [x] Yes
- [ ] N/A
- [ ] No — Its in accordance with Councils requiremen

6.2 Hydraulic Model Build - Inflow Boundary Conditions (generic not run specific)

Inflow boundary(s)

- [x] Flow v Time
- [ ] Steady flow(s)
- [ ] Head v Time
- [ ] Direct rainfall
- [ ] Combination of the above
- [ ] Other

Origin / Derivation of inflow boundaries

RAFTS model - local inflows
RAFTS model - total inflows
Bulimba Creek Mike11 Model

Is there a need to check the inflows?

- [x] Yes
- [ ] N/A
- [ ] No — It was only checked during the calibration revie

Is the type of inflow the most appropriat for the analysis?

- [x] Yes
- [ ] N/A
- [ ] No

Inflow locations in the hydraulic mode at the most appropriate locations an consistent with the hydrologic analysis

- [x] Yes
- [ ] N/A
- [ ] No — To be consistent for all scenarios, total flows from RAFTS were use

Are there sufficient inflow locations to achieve the modelling objectives

- [x] Yes
- [ ] N/A
- [ ] No

Is the inflow distributed over a suitabl wide section to capture the flow width

- [x] Yes
- [ ] N/A
- [ ] No

Other Comments / Issues

Inflows applied as 2d_SA and 1d_bc tables as local and total inflows

The coincident flooding with Bulimba Creek was considered for this study by including the lower part of Bulimba Creek catchment in the TUFLOW model. Bulimba Creek FS originally uses DIS method for hydrology. Both hydrology and hydraulic model were run using AR&R for this study to simulate boundary condition for this study

7.0 Model Simulation (generic not run specific)

Run type	☐ Steady ☑ Unsteady (fixed time step) ☐ Unsteady (variable time step) ☐ Other	1D=0.25S and 2D=2S
Does the software use an implicit or explicit finite difference scheme?	☑ Implicit ☐ Explicit	☐ N/A
Initial conditions	☑ User input Q and WL / Depth ☐ Hotstart file ☐ Other	
Are there special items (e.g. reservoirs) which need unique initial conditions?	☑ Yes ☐ No	☐ N/A Ponded area/dam
Will selection of the initial conditions influence the modelling results?	☐ Yes ☑ No	☐ N/A
Are the initial conditions generally acceptable?	☑ Yes ☐ No	☐ N/A
What is the time step?	1D=0.25S and 2D=2S	
Courant conditions satisfied?	☑ Yes ☐ No	☐ N/A
Is the timestep appropriate? (note, if impact assessment a fixed timestep may be more appropriate)	☑ Yes ☐ No	☐ N/A
Have all warning and error messages been checked and resolved?	☑ Yes ☐ No	☐ N/A they were not checked
Has the model results / log file been provided and checked?	☑ Yes ☐ No	☐ N/A
Results / log file checked for mass balance?	☑ Yes ☐ No	☐ N/A
Hydrographs at selected locations in the model checked for visual instabilities?	☑ Yes ☐ No	☐ N/A there are some instabilities but overall its ok
Results / log file checked for non-convergence / max number of iterations exceeded?	☐ Yes ☑ No	☐ N/A
Model run time suitable for the intended use of the model?	☑ Yes ☐ No	☐ N/A
Model reporting intervals suitable to detect instabilities and satisfy modelling objectives?	☑ Yes ☐ No	☐ N/A

Other Comments / Issues

Model not rerun

Bulimba Creek inflows files were not provided for the check. The boundary is believed to be QT format.

Hydrographs provided for PO lines and 1D network - spot checked

Due to the model complexity there has been some instability in discharge hydrographs. It is considered acceptable if the stage hydrographs are stable

In the downstream area at the confluence with Bulimba Creek some of the hydrographs did not reach the peak for some of the durations. WBM was asked to check and rerun if required

8.0 Calibration (generic not run specific)

Number of calibration events	2	
Dates of calibration events	Oct-10	
	Jan-13	
Approximate ARI of smallest calibration event	less than 2 yr ARI	
Approximate ARI of largest calibration event	5 yr ARI	
Are the events selected suitable for calibration?	☑ Yes ☐ No	☐ N/A
(If no, why)	Very limited historical data was available for this catchment	
	Calibration to larger events was not possible due to the lack of historical data	
Specific details of each calibration event described in the report?	☑ Yes ☐ No	☐ N/A Limited information
Details of the historical catchment changes detailed in the report?	☑ Yes ☐ No	☐ N/A
Specifics and the limitations of the recorded gauged data detailed in the report	☑ Yes ☐ No	☐ N/A
Basis of the calibration? (e.g. joint hydrologic / hydraulic	☑ Joint ☐ Hydraulic only	☐ Other ☐ N/A at the upstream part of the catchment where possibl
Calibration tolerances specified?	☑ Yes ☐ No	☐ N/A
What parameters were calibrated?	Not specified - assumed to be roughness within hydraulic model	
	and losses within hydrology model and structure blockage	
Headlosses at hydraulic structures appear logical / sensible?	☐ Yes ☐ No	☑ N/A The losses were not recalculated but they seem acceptabl
Headlosses at hydraulic structures checked by an alternate method	☐ Yes ☑ No	☐ N/A Only one bridge in the mode
Fit to hydrograph timing adequately achieved?	☑ Yes ☐ No	☐ N/A
Fit to peak flood level adequately achieve acceptable ranges?	☑ Yes ☐ No	☐ N/A Overally yes
Fit to volume achieved in hydrological calibration?	☐ Yes ☐ No	☑ N/A
Are the calibrated parameters within acceptable ranges?	☑ Yes ☐ No	☐ N/A
Calibration produced a consistent set of parameters to use in verification	☑ Yes ☐ No	☐ N/A
Locations at which a good fit has not been achieved (and reasons why)	The report explains where calibration has not been achieved within targets.	

Other Comments / Issues

there has been a few erroneous recorded levels for historical events which has been explained in the report

Results and outcomes were also explained in the report

Overall, it seems model under-predicts at the MHGs however in most cases, it still is within acceptable tolerances

9.0 Verification (generic not run specific)

Number of verification events	1	
Dates of verification events	Dec-10	
Approximate ARI of smallest verification event	Less than 2 yr ARI	
Approximate ARI of largest verification event		
Are the events selected suitable for verification?	☑ Yes ☐ No	☐ N/A
(If no, why)		
Specific details of each verification event described in the report?	☑ Yes ☐ No	☐ N/A
Details of the historical catchment changes detailed in the report?	☑ Yes ☐ No	☐ N/A
Specifics and the limitations of the recorded / gauged data detailed in the report?	☑ Yes ☐ No	☐ N/A
Verification tolerances specified?	☑ Yes ☐ No	☐ N/A
Headlosses at hydraulic structures appear logical / sensible?	☐ Yes ☐ No	☑ N/A The losses were not recalculated but they seem acceptable
Headlosses at hydraulic structures checked by an alternate method?	☐ Yes ☑ No	☐ N/A
Fit to hydrograph timing adequately achieved?	☑ Yes ☐ No	☐ N/A
Fit to peak flood level adequately achieved acceptable ranges?	☑ Yes ☐ No	☐ N/A
Fit to volume achieved in hydrological verification?	☐ Yes ☐ No	☑ N/A
Does the verification give confidence the model is producing accurate results and is suitable for design runs	☑ Yes ☐ No	☐ N/A
Locations at which a good fit has not been achieved (and reasons why)	included in the report	

Other Comments / Issues

Refer calibration comments

10.0 Design - Flood Planning Levels (generic not run specific)

Range of design ARI events	50%,20%,10%,5%,2%,1%,0.5%,0.2%,0.05% AEP and PMF	
Does the design ARI consider joint probability of creek / river; creek / tide; etc; interaction?	[x] Yes [ ] No	[ ] Other [ ] N/A
If no, should it have been considered?	[ ] Yes [ ] No	[ ] N/A
(If yes, why)	Interaction of tide and Bulimba Creek and Hemmant Drain and Lindum Creek has been considered for the purpose of modelling the flood levels for this study	
	MHWS for standard design events and HAT for all extreme events For climate change scenarios, similar but +300mm for year 2050 and +800mm for year 2100	
Has the joint probability analysis been applied correctly?	[x] Yes [ ] No	[ ] N/A
(what methodology was used?)	Lower part of bulimba creek catchment was included in the model to simulate the flood levels at the confluence of Bulimba Creek and Hemmant Drain / Lindum Creek	
Type of hydraulic scenario(s)?	[x] Existing (Ex) [x] Ex + MRC	[x] Ex + MRC + WWC [ ] Other
Origin or the design hydrology?	[x] Hydrology model (AR&R) [ ] Hydrology model (DIS)	[ ] Rational Method [ ] Other
Is the design hydrology existing or ultimate catchment conditions?	[ ] Existing [x] Ultimate	[ ] Other
Is the design hydrology calibrated or un-calibrated?	[ ] Calibrated [x] Un-calibrated	[ ] Other [ ] N/A — Model is not calibrated, but its consistent with hydraulic model results at the upper part of the catchment
Are the design model parameters consistent with the calibrated model?	[x] Yes [ ] No	[ ] N/A
Has the Minimum Riparian Corridor been modelled adequately?	[ ] Yes [x] No	[ ] N/A — WBM was asked to modify and change the model
Has the Waterway Corridor been modelled adequately?	[x] Yes [ ] No	[ ] N/A
Have the design ARI flood level results been compared against previous results?	[ ] Yes [x] No	[ ] N/A
If yes, are the results comparable to these results?	[ ] Yes [ ] No	[x] N/A
(If no, why)	It wasn't part of scope of work	
Have the design ARI flood level results been compared against each other for consistency?	[x] Yes [ ] No	[ ] N/A
Headlosses at hydraulic structures appear logical / sensible?	[x] Yes [ ] No	[ ] N/A
Headlosses at hydraulic structures checked by an alternate method?	[ ] Yes [x] No	[ ] N/A
Has a sensitivity analysis been undertaken?	[x] Yes [ ] No	[ ] N/A
If yes, what parameters?	[ ] Manning's n [x] Blockages	[ ] Boundary conditions [x] Other — Climate Change
Has a climate change assessment been undertaken?	[x] Yes [ ] No	[ ] N/A
Has the climate change change assessment been modelled appropriately?	[x] Yes [ ] No	[ ] N/A
Modelling results appear to be accurate and suitable for the study objectives?	[x] Yes [ ] No	

Other Comments / Issues

The hydrology model is uncalibrated. However, the model results have been verified at couple of locations with the results from the hydraulic model at the upstream part of the catchment.

11.0 Project Details

Project Deliverables	Digital files including flood models (TUFLOW and XP-RAFTS), Report in .doc and .pdf format and GIS files including figures Hard Copy: 4 copies of Hemmant Lytton Flood Study Report Volume 1 and Volume 2 and Model handover guide		
Modelled Scenario	*Scenario 1: Existing Scenario- for 50% AEP to PMF + CC2100 for 1% AEP, 0.5% AEP and 0.5% AEP and CC 2050 for 1% AEP and 0.5% AEP + Blockage Scenario for 1% AEP *Scenario 2: Existing Scenario+ MRC- for 1% AEP *Scenario 3: Existing Scenario + MRC + WC- for 50% AEP to 1% AEP No stretching of flood levels for Scenario 3 was undertaken		
Model Output	Model Extent, Water Level Grid, Depth Grid, Depth*Velocity Grid, Velocity Grid		
Report Inclusion	Calibration Report and Design Event Report Flood Maps Hydraulic Structure Reference Sheet BCC Peer Review Memorandum Model Handover Guide		
Which maps have been included in the report?	50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP, 0.2% AEP, 0.05% AEP Model Extent for Scenario 1 only		
Does the report meet the standard requirement?	☑ Yes	☐ No	Its in accordance with the Council's requirement and provided report template
Has the report been signed off by an RPEQ Engineer?	☑ Yes	☐ No	As required

APPENDIX F (in Volume 2) - Design Flood Mapping

APPENDIX G Technical Memorandum for Adopted Methodology – Extreme Events Modelling (BCC, 2013)

Dedicated to a better Brisbane

MEMORANDUM

Brisbane City Council

To:	**Natural Environment Water and Sustainability Branch (NEWS)**	Date:	**15/03/2013**
Attn:	**Suba Subasing Gamachchige -** Project Owner, NEWS		
CC:	**Ellen Davidge -** Principal Engineering Officer, NEWS **Evan Caswell** - Principal Engineer, Flood Management		
From:	**Allan Herring -** Design Manager, Flood Management **Hanieh Zolfaghari –** Engineer, Flood Management		
Re:	**Technical Memorandum for Adopted Methodology - Extreme Events Modelling**		

Planning & Design Branch
Flood Management

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Brisbane Qld 4001

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1.0 Introduction

The Flood Management team, within the Planning and Design Branch of the City Projects Office, has been asked to provide a technical memorandum for the adopted methodology for the extreme events hydrologic modelling which has been undertaken with the intention to update Council's creek flood studies.

2.0 Background

The additional scenarios to be modelled as part of the flood studies include the 200, 500 and 2000 year average recurrence interval (ARI) events and the Probable Maximum Precipitation (PMP) event. This memorandum documents the methodology adopted as well as the limitations of the methodology.

3.0 Methodology

Events Up to 100 year ARI

The events up to the 100 year ARI are developed using the AR&R temporal pattern which involves running multiple model runs to simulate the various standard storm durations.

200 and 500 year ARI Events

For the 200 and 500 Year ARI events, the CRC-Forge rainfall data were derived and used for each catchment. The CRC-Forge method adopts the AR&R temporal pattern to simulate rainfall within the catchment, and also requires multiple model runs to simulate the various standard storm durations.

The durations modelled were 30min. 1hr, 3 hrs and 6 hrs.

A 9hr rainfall depth was interpolated for Kedron Brook and Bulimba Creek.

2000 year ARI Event

To analyse the 2000 Year ARI flood event, the CRC-Forge rainfall depths were adopted. However, to simplify the analysis over a large number of similarly sized catchments, (based on the average size of catchments in the Brisbane area) the adopted rainfall data was extracted for a catchment size of 60 km^2 located at the north-west part of Brisbane. Note that rainfall depth varies by less than 10% across the entire area.

To avoid running multiple storm patterns for different storm durations, a super-storm approach was adopted. This is a common practice adopted overseas for broad scale planning scenario flood mapping with the temporal pattern built up to reflect the extreme rainfall depths published by the BoM. The rationale for adopting this approach is that world-wide research shows that as storm rainfall depths increase for short duration storms, the rainfall intensity becomes more uniform. For this reason, the multi peaked temporal patterns for the 100 year from AR&R were not considered suitable for the analysis of the more extreme events.

For this analysis, a 6 hour super storm was developed in 30 min blocks to represent a number of shorter extreme events. Shorter durations than 30 minutes were not considered. The pattern developed is representative of the 30, 60, 90, 120, and 180 minute storm burst. The total rainfall depth and duration of the storm was set equal to 6 hours for all catchments except Kedron Brook and Bulimba Creek.

For these two catchments only, a nine hour pattern was developed and applied, with the central part of the storm replicating the six hour pattern. This was considered necessary to ensure that all catchment routing was complete by the end of the model run.

Reference: *The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (GSDM), BoM, June 2003.*

PMP

For the PMP scenario, the rainfall depth was derived from the 6 hour temporal pattern using the Generalised Short Duration Method (GSDM). For the tropical and subtropical coastal areas it is recommended that this method is to be used to estimate the PMP over areas up to 520km^2 and for durations up to 6 hours.

For the purpose of PMP estimation for the creeks and to be consistent across the Brisbane area, an average catchment size of 60 km^2 and moisture adjustment factor of 0.85 were adopted. This method is adopted for most of the creeks within the Brisbane area; however, exception is made to Oxley Creek due to the longer response time of the catchment. The adopted PMP temporal Pattern is shown in ***Appendix A***.

Other Durations and ARI's

No methodology or guidance is provided by the BoM or by AR&R for the estimation of PMP rainfall depths for durations longer than 6 hours or ARI's between 2000 years and PMP. One common method used by practitioners makes use of Log-Log interpolation. The challenge with this methodology is to provide an ARI for the PMP event and then to interpolate between the 2000 year ARI rainfall depths and the PMP rainfall depths. The method is approximate only but is considered reasonable considering the paucity of observed extreme rainfall observations in Australia and overseas. It is generally accepted that the probability of the PMP is in the order of 1 in 10^6 to 1 in 10^7.

All rainfall depths derived by the methods described were rounded to the nearest 10mm and they are shown in ***Appendix B***.

3.1 Verification

The storm pattern derived using methodology mentioned above was compared against 2 extreme storm events, which were the Carrara event and the Maroochydore event. The Maroochydore was in the order of 2000 year ARI and the Carrara event between 500 and 2000 year ARI respectively.

The comparison shows a good correlation and certified the adopted methodology.

3.2 Limitations

The assumptions and limitations of the adopted methodology to model extreme events include:

- The GSDM method is only valid for catchments with areas up to 520km^2; however, the majority of the catchments in Brisbane are smaller than 100 km^2 in size, with an average size of 60 km^2.

- Derived rainfall depths vary by less than 10% within the different catchments in the Brisbane area; however, the adoption of an average catchment size of 60km^2 is considered a reasonable approach considering the significant amount of rainfall during an extreme event.

- The adopted PMP pattern is well suited for catchments with a response time of half an hour up to 6 hours. This is the response time for the majority of the creeks in Brisbane with the exception of Oxley Creek.

For a better understanding of the limitations of this method, *The Estimation of Probable Maximum Precipitation in Australia: GSDM, June 2003* paper is attached to this memorandum (***Appendix C***).

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Appendix A

Adopted Temporal Pattern

Duration (%)	0	3	6	8	11	14	17	19	22	25
Rainfall (%)	0	1	2	4	5	6	7	9	11	12
Duration (%)	28	31	33	36	39	42	44	47	50	53
Rainfall (%)	14	17	19	22	26	29	34	39	48	57
Duration (%)	56	58	61	64	67	69	72	75	78	81
Rainfall (%)	66	71	74	78	81	83	86	88	89	91
Duration (%)	83	86	89	92	94	97	100			
Rainfall (%)	93	94	95	96	98	99	100			

Appendix B

200 and 500 Year ARI Event Rainfall Depth (mm)

Creek Name	Storm Events									
	200 Year ARI					500 Year ARI				
	30 min	1 Hour	3 Hour	6 Hour	9 Hour	30 min	1 Hour	3 Hour	6 Hour	9 Hour
Bulimba Creek	80	110	160	200	252	90	120	180	230	294
Kedron Creek	90	120	170	220	271	100	140	200	250	315
Lota Creek	80	110	160	210		90	130	190	240	
Norman Creek	80	120	170	210		100	130	190	240	
Breakfast Creek	90	130	180	230		100	150	210	260	
Perrin Creek	80	110	170	210		100	130	200	250	
Pine River Creek	90	120	180	220		100	140	200	260	
Albany Creek	90	130	180	230		110	150	210	270	
Cabbage Tree Creek	90	120	180	220		100	140	210	260	
Nundah Creek	90	120	180	220		100	140	200	260	

2000 Year ARI, PMP, Carrara and Maroochydore Events Rainfall Depth (mm)

Event	Storm Duration									
	0.5 hour	1 hour	1.5 hour	2 hour	2.5 hour	3 hour	4 hour	4.5 hour	5 hour	6 hour
2000 year ARI	120	170	190	220	240	260	290	300	310	340
PMP	230	340	440	510	570	620	700	730	770	820
Carrara	80	150	190	230	260	280	340	360	380	440
Maroochydore	60	120	160	200	220	260	310	330	350	350

Appendix C

The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method

HYDROMETEOROLOGICAL ADVISORY SERVICE
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JUNE 2003

The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method

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The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method (GSDM) offers guidance to those engaged in estimating the probable maximum precipitation for durations up to three or six hours in Australia. Despite careful preparation, it may contain typographical or other errors that affect use of the procedures and/or the numerical values obtained. Readers are encouraged to report suspected errors to the Hydrology Unit of the Bureau of Meteorology. Once confirmed, errors will be noted and, where circumstances allow, corrected. The Bureau will maintain a list of GSDM errata/corrigenda accessible via the World Wide Web. The location of the list will be advised through the Hydrometeorological Advisory Service section of the Bureau's web site: http://www.bom.gov.au/hydro/has. The Bureau of Meteorology does not give any commitment to communicate errors, whether suspected or confirmed. Nor is liability accepted from losses arising from use of the GSDM, its procedures, howsoever caused. The Bureau of Meteorology has not approved any instruction that use of the GSDM procedures be made mandatory for particular applications.

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1. INTRODUCTION

Probable Maximum Precipitation (PMP) is defined by the World Meteorological Organization (1986) as '*the greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of year*'.

Hydrologists use a PMP magnitude, together with its spatial and temporal distributions, for the catchment of a dam to calculate the probable maximum flood (PMF). The PMF is one of a range of conceptual flood events used in the design of hydrological structures. In the main, it is used to design a spillway that will minimise the risk of overtopping of the dam. Overtopping of a dam structure can result in damage to the dam wall or abutments through breaching. The risk of loss of life, cost of rebuilding the dam, cost of the additional flood damage downstream and cost to the community due to the loss of a water supply can thus be minimised.

The purpose of this publication is to provide a method that can be used to make consistent and timely estimates of probable maximum precipitation for catchment areas up to 1000 km^2. Estimates are limited to a duration of six hours along the tropical and subtropical coastal areas and three hours in inland and southern Australia. The method allows for two classes of terrain and takes into account the local moisture availability and the mean elevation of the catchment.

The low density of the raingauge networks, particularly the pluviograph network, has resulted in few severe short-duration rainstorms having been recorded or documented in Australia. This is particularly the case in the sparsely populated part of the continent away from the coastal fringe and is a severe limitation on the estimation of short duration probable maximum precipitation in Australia. For this reason, United States data and Australian data have been used in the development of the Generalised Short Duration Method for use in Australia. Areal rainfall data are provided for some major Australian rainstorms in Appendix 3 to support the PMP magnitudes derived.

Design temporal and spatial distributions of PMP based on average storm characteristics are also given. These facilitate the distribution of the PMP depth when used in hydrological models.

This document replaces 'Bulletin 53: The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method' (Bureau of Meteorology, December 1994), and should be used instead. It was considered that a new version was required as, since 1994, a revised method of spatial distribution has been introduced and the moisture factors updated.

2. HISTORY OF THE DEVELOPMENT OF PMP METHODOLOGY IN AUSTRALIA

The early methods used to estimate extreme floods, other than reliance on local knowledge, were statistical. Frequency analysis has been used in most parts of Europe where it is relatively effective due to the homogeneity of the storm population, the long length of records and the availability of historical flood marks. The original spillway designs of some Australian dams, such as the Warragamba Dam, were based on this method. In the tropics and subtropics (e.g. Australia), the lack of homogeneity in the storm population and relatively short length of records cause significant deficiencies in the severe storm rainfall sample available for frequency analysis. This led to the need to develop deterministic methods, which used the sample outliers to estimate the rainfall from the optimum storm mechanism and a maximisation factor to adjust the storm rainfall to that possible with the potential extreme moisture inflow.

The deterministic methods of estimating PMP have developed from '*in situ* maximisation' through 'storm transposition' to the current 'generalised' methods.

2.1 *In Situ* Storm Maximisation Method

Early estimates of PMP in Australia (1950s to 1970s) were based on *in situ* maximisation. Only storms that had occurred over the catchment were considered for maximisation. The rainfall depths from storms covering a range of durations were maximised for moisture and the maximum depth at a specified duration was taken as the PMP for that duration. The maximisation procedure consisted of the adjustment of the rainfall depth measured in a storm by the ratio of the highest observed atmospheric moisture content in the area of the catchment to that observed in the storm. In some cases, the rainfall was also maximised for potential wind speed and direction accompanying the rainfall, but in general there was insufficient information available to make this practical. Wind speed and direction are now considered to be part of the overall storm mechanism. Recorded temporal and spatial distributions of the individual storms were used as design patterns.

The occurrence or lack of occurrence of an outlier in the storm sample, within the length of rainfall records available for different catchments, led to inconsistencies between PMP estimates for catchments in the same general area.

2.2 Storm Transposition Method

During the late 1960s and early 1970s storm transposition was gradually introduced. This procedure increased the size of the sample of significant storms that could be maximised for a catchment. The larger sample improved the consistency of PMP estimates within regions of similar topography, and generally led to higher PMP estimates than those produced using *in situ* maximisation.

The method was limited to the transposition of storms that had occurred near the catchment in regions with similar topographic features to those of the catchment. No guidance was available on how to adjust storm depths for the response of rainfall to differing topography. Consequently, storms that occurred near the subject catchment could not be transposed if they had occurred over a region with different topography. In addition, the individual storm spatial patterns of the transposed storms reflected the topography of the storm area and were not always appropriate for use in the target catchment. The choice of storms for transposition introduced a significant level of subjectivity to the methodology.

A storm transposition method is used for catchments in southwestern Tasmania, as described in 'Development of the Method of Storm Transposition and Maximisation for the West Coast of Tasmania - HRS 7' (Xuereb et al., 2001); the extreme lack of data making it impractical to develop a generalised method for this region.

2.3 Generalised Methods

Generalised methods of estimating PMP have gradually been developed for various parts of Australia and were introduced from the mid-1970s onward. This follows the trend in the United States where they were gradually introduced from the early 1960s. Generalised methods differ from the *in situ* and transposition methods in that they use all available data over a large region and include adjustments for moisture availability and differing topographic effects on rainfall depth. These storm data are enveloped by smoothing over a range of areas and durations. Generalised methods also provide design spatial and temporal patterns of PMP for the catchment. These methods require a considerable investment of time to develop, but when completed, estimates for individual catchments can be made more easily and objectively.

The United States generalised methods for areas with minimal topographic enhancement were developed first as an extension of the limited transposition methods. This type of method was suitable for most of the United States east of the Rocky Mountains (United States National Weather Service, 1978). Variations on the basic method were then gradually developed for areas with significant topographic enhancement of the rainfall. The method of dealing with topographic effects varies considerably, reflecting the specific problems posed by the prevailing meteorological regime and the availability of meteorological information (World Meteorological Organization, 1986; United States Weather Bureau, 1961, 1965, 1969; United States National Weather Service 1977, 1984, 1988; Wang, 1986).

The use of generalised methods has tended to increase the PMP estimates for a given catchment, compared with those obtained using the '*in situ* maximisation' and 'storm transposition' methods due to the increased chance of the larger sample containing an outlier. This is discussed with respect to the Warragamba Dam Catchment in Pearce (1993). Generalised method estimates have a lower notional Annual Exceedance Probability (AEP). They also have the advantage of providing regionally consistent estimates, although the notional AEP may vary slowly across a large zone or differ between zones. In assessment of both comparative risk and cost-benefit analyses between dams within a region, generalised methods set a more uniform standard than *in situ* or limited transposition methods (where topographic effects made transposition subjective).

The generalised methods currently available in Australia are:

i) The Generalised Short Duration Method (GSDM) described in chapters 3 and 4.

(ii) The Generalised Southeast Australia Method (GSAM), which was finalised in 1992. This method is for use in catchments in southeast Australia and is described by Kennedy et al. (1988), Pearce and Kennedy (1993, 1994) and Minty et al. (1996). Figure 1 shows the two zones for application of the GSAM: inland and coastal. The maximum duration covered by this method ranges from 3 to 5 days

(iii) The revised version of the Generalised Tropical Storm Method (GTSMR), which was finalised in 2003. This method is applicable to those parts of Australia affected by tropical storms and divides the region into 3 parts: the coastal application zone (CAZ), the inland application zone (IAZ) and the southwest Western Australia application zone (SWAZ). Figure 1 shows these zones. The maximum duration covered by this method is 5 days in the coastal zone in summer and 4 days for all other zones and seasons. The method is described in Walland et al. (2003).

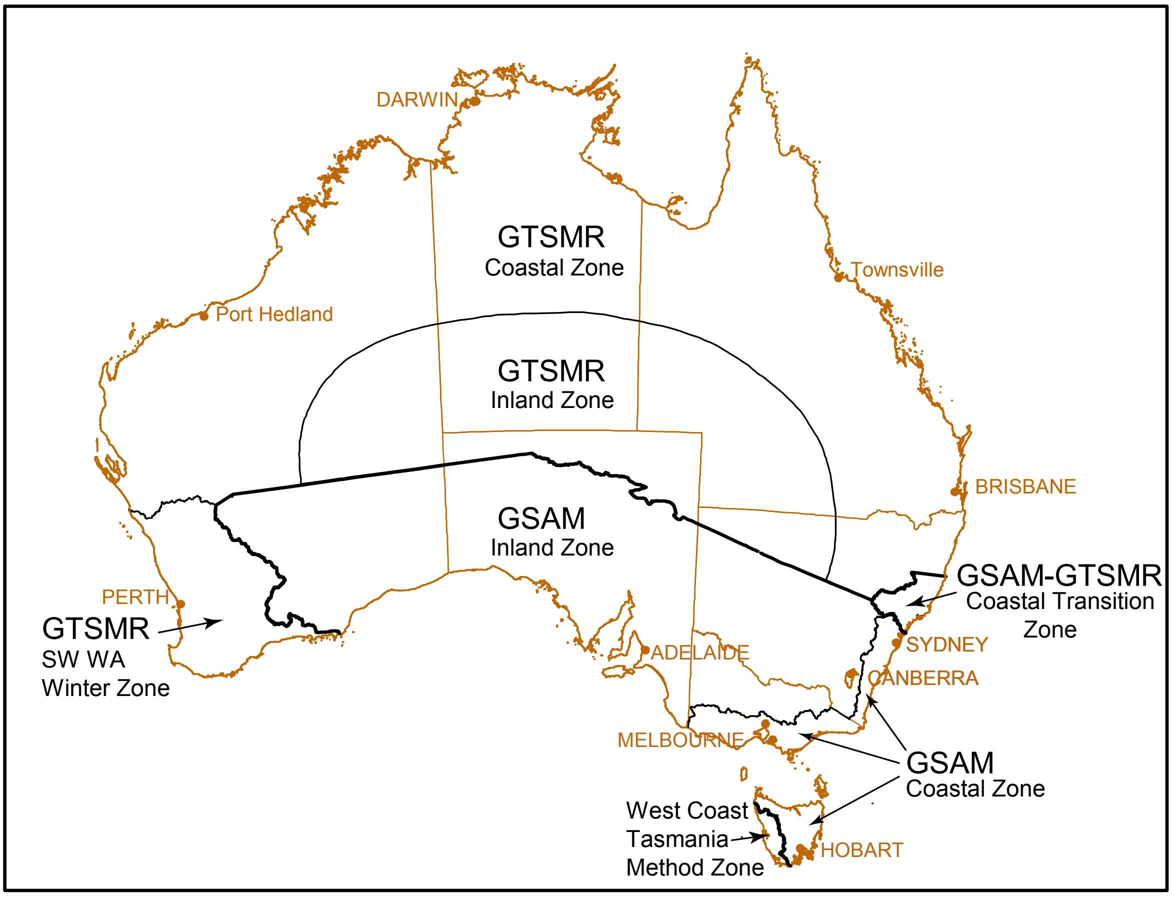

Figure 1: Generalised Tropical Storm Method and Generalised Southeast Australia Method Zones

2.4 Limitations and Restrictions on Generalised PMP Estimation Methods used in Australia

The accuracy and reliability of an estimate depends on the amount and quality of the data available for use in the estimating procedure and the maintenance of a balance in the degree of maximisation used in order to obtain realistic estimates. The transposition method was limited to the use of storms that occurred near the catchment in areas with similar topographic features. The generalised methods use a deterministic approach to adjust for topographic and moisture effects and thus increase the usable transposition area. However, even with these adjustments there are meteorological limitations on the transposability of some types of storms. The selection of meteorologically compatible zones in generalised PMP methodology requires that an equivalent optimum storm mechanism could occur anywhere in the transposition area; the frequency of occurrence is not important. The GTSMR, for example, is only applicable to those parts of Australia affected by tropical storms. The frequency of occurrence of the storm mechanisms varies considerably across the zones, but this does not necessarily affect the magnitude of the estimated PMP.

The restrictions on the GSAM and GTSMR PMP estimation methods for short durations are due to the limitations on availability and quality of short duration storm data. The development of these methods relied significantly on daily data in order to make the most effective use of record length and network density for the storm search procedures. These methods therefore need to be used in conjunction with the GSDM where appropriate (i.e. over small catchments where the critical duration is between that covered by the GSDM and the GSAM or GTSMR).

All three of the generalised methods are based on single storm events only, including single storms with multiple peaked temporal distributions. This means that the methods have an upper limit to the effective duration for which they can be applied to the catchment. The joint probability of a design sequence of two or more extreme rainfall events would be much lower than the probability of the generalised PMP event by itself.

None of the methods incorporates long-term climate change, other than climatic variability implicitly contained within the available years of records. However, climatic trends progress slowly so their influence on PMP is small compared to other uncertainties in estimating extreme values. This is consistent with the current practice described in World Meteorological Organization (1986).

3. BACKGROUND TO PMP ESTIMATION FOR SHORT DURATIONS

Methods for estimating PMP for small areas and short durations have been used by the Bureau of Meteorology since 1960. The first depth-duration-area (DDA) values used in Australia were those published by the United States Weather Bureau in 1945 (United States Weather Bureau, 1945).

The original method was known as the 'Thunderstorm Model' method because extreme rainfall totals for short durations and small areas are most likely to be produced by large, efficient convective cells. These cells may be either isolated thunderstorms or form part of a mesoscale or synoptic scale storm system. Later, the method became known as the 'method of adjusted United States data' (Kennedy, 1982). PMP estimation for short durations and small areas in Australia was based on the maximisation of United States thunderstorm depth-duration-area (DDA) data because of an inadequate supply of Australian short duration rainfall data. The Australian network of daily rainfall gauges has a far greater density and more effective years of record than the pluviograph network.

Initially it was recommended that the method be used to estimate PMP over areas up to 200 mi^2 (520 km^2) and for durations up to 6 hours for catchments in the tropical and subtropical coastal strips of the continent. The method was later extended to cover inland and southern Australia where the limit to the duration was 3 hours. The maximum area for application was also increased to 1000 km^2 for all areas.

In 1978 the DDA curves used by the Bureau of Meteorology were updated using information given in later hydrometeorological reports (United States Weather Bureau, 1960, 1969; United States National Weather Service, 1977, 1978) and by Wiesner (1970). At this time, terrain classifications of 'rough' and 'smooth' were introduced, with separate sets of DDA curves being provided for each category.

In 1984 a phenomenal storm occurred near Dapto in New South Wales (Shepherd and Colquhoun, 1985). For some areas and durations, the maximised rainfall from this storm exceeded the adjusted United States values. Areal rainfall depths recorded in this storm were added to the United States data when the method was published in 1985 as 'Bulletin 51: The Estimation of Probable Maximum Precipitation in Australia for Short Durations and Small Areas' (Bureau of Meteorology, 1985).

With the publication of *Bulletin 51,* the six-hour zone was broadened, especially in northern Australia, and an intermediate zone was introduced between the three and six hour zones. Subsequently, the definitions of 'rough' and 'smooth' terrain were altered, as described in 'Australian Rainfall and Runoff' (The Institution of Engineers, Australia, 1987). This and other adjustments were included in the next edition, published as *Bulletin 53* in 1994. Since then, the method has been referred to as the 'Generalised Short Duration Method' (GSDM), in line with the terms used to describe other generalised methods.

The GSDM is suitable for application to small catchments such as those of tailings dams and small reservoirs anywhere in Australia. Chapter 4 explains the GSDM procedure in detail and a worked example is found in Appendix 2. Additionally areal rainfall depths recorded in a number of severe Australian storms are given in Appendix 3.

4. GSDM PROCEDURE

This section describes in detail the steps to be followed in determining GSDM PMP estimates for a catchment. A sample calculation sheet to use with this procedure is given in Appendix 1 and an example covering all the steps is provided in Appendix 2.

4.1 Selection of Duration Limits

The first step is to establish the maximum duration for which the method is applicable to the catchment. Figure 2 shows the areas of Australia subject to the duration limits of three and six hours. There is also an intermediate zone where the maximum duration can be determined by using linear interpolation, setting the boundary values to three and six hours.

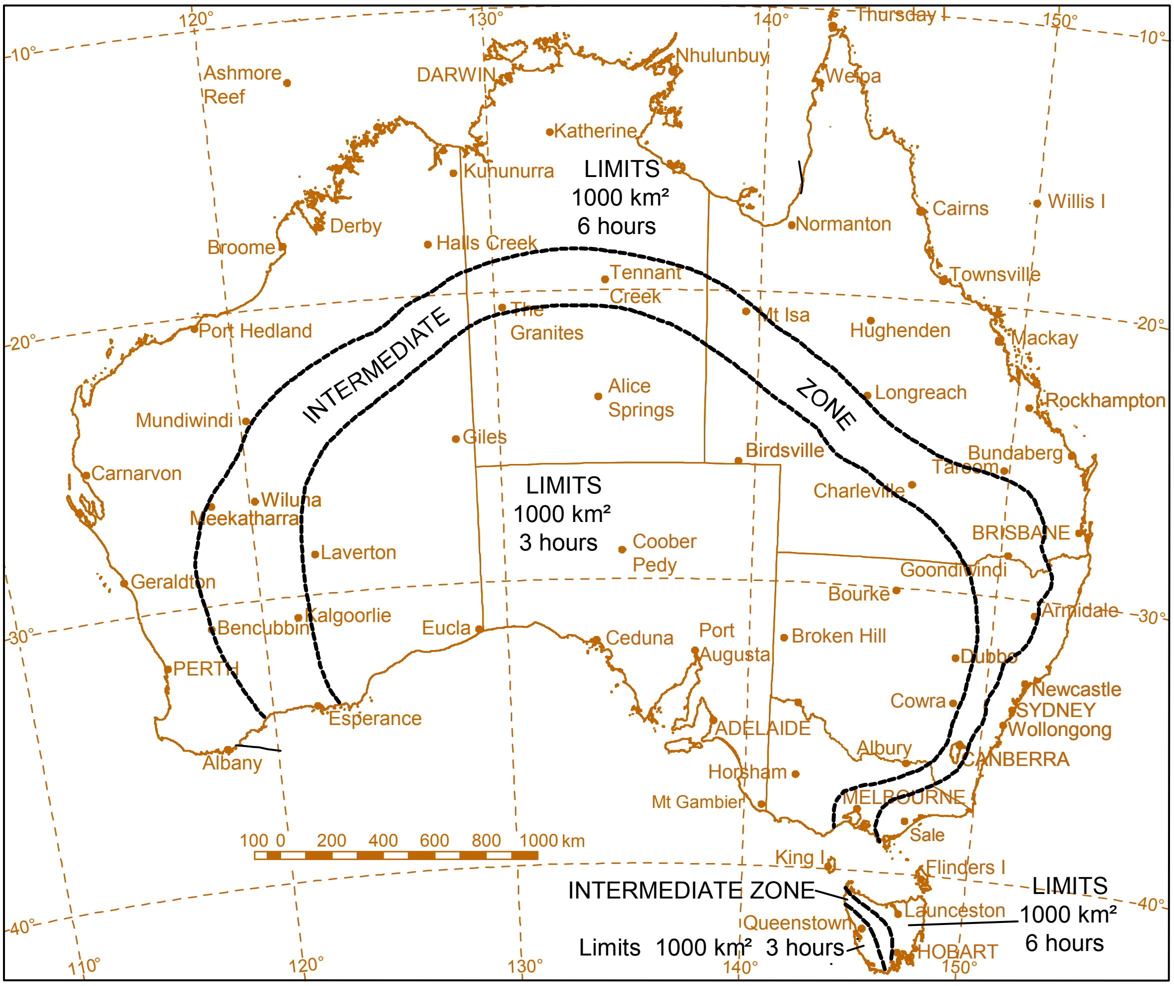

Figure 2: Generalised Short-Duration Method zones.

4.2 Selection of Terrain Category

Rainfall from single, short duration thunderstorm events is not significantly affected by the terrain. Therefore, it is not necessary to classify the terrain of the catchment for durations of an hour or less.

If durations longer than one hour are required, the next step is to establish the terrain category of the catchment and to calculate the percentages of the catchment that are 'rough' and 'smooth'. 'Rough' terrain is classified as that in which elevation changes of 50 m or more within horizontal distances of 400 m are common. 'Rough' terrain induces areas of low level convergence which can contribute to the development and redevelopment of storms, thereby increasing rainfall in the area over longer durations.

Terrain that is within 20 km of generally 'rough' terrain should also be classified as 'rough'. If there is 'smooth' terrain within the catchment that is further than 20 km from generally 'rough' terrain, an areally weighted factor of 'rough' (**R**) and 'smooth' (**S**) terrain should be calculated such that **R** plus **S** equals one. If a catchment proves difficult to classify under these guidelines then the whole catchment should be classified as 'rough'.

4.3 Adjustment for Catchment Elevation

The next step is calculation of the Elevation Adjustment Factor (**EAF**). The mean elevation of the catchment should be estimated from a topographic map. If this value is less than or equal to 1500 m the EAF is equal to one. For elevations exceeding 1500 m the EAF should be reduced by 0.05 for every 300 m by which the mean catchment elevation exceeds 1500 m. For most catchments in Australia the EAF will be equal to one.

4.4 Adjustment for Moisture

The moisture index used in PMP work is the precipitable water value corresponding to the 24-hour persisting dewpoint. By assuming a saturated atmosphere with a pseudo-adiabatic lapse rate during storm conditions, the precipitable water value can be estimated from the surface dew point temperature, a commonly measured quantity. The ratio of the extreme moisture index for a storm location to the moisture index at the time of the storm was used in the maximisation process.

The rainfall Depth-Duration-Area (DDA) curves in Figure 4 have been standardised to a moisture index equivalent to a surface dew point temperature of 28EC. An adjustment is required to allow for the potential moisture availability at the catchment. A map has been constructed based on the percentage adjustment for any locality and is given in Figure 3. The Moisture Adjustment Factor (**MAF**) for a catchment can be read from this map.

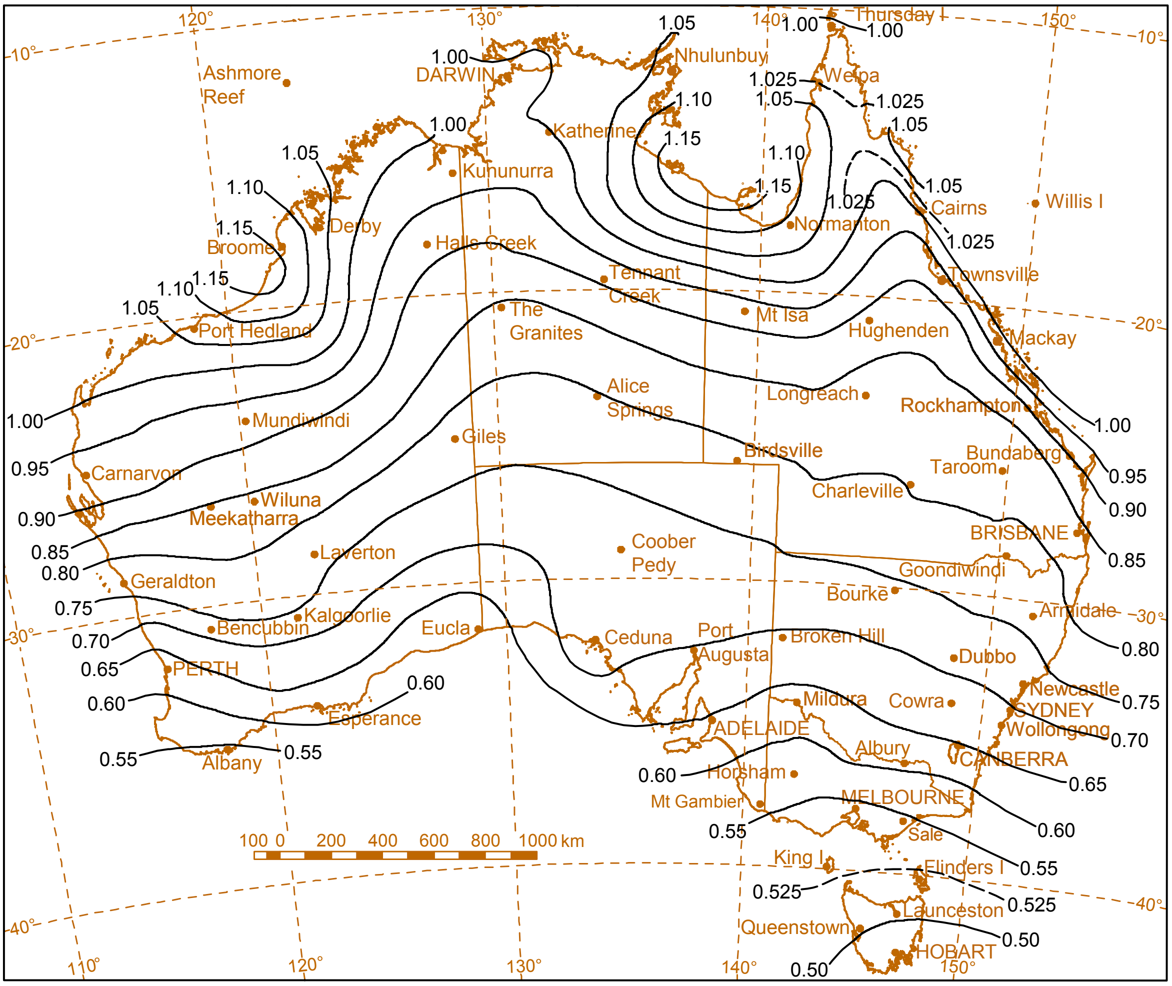

Figure 3: Moisture Adjustment Factor

4.5 Calculation of PMP Estimates

The DDA curves, given in Figure 4, were produced by drawing enveloping curves to the highest recorded United States and Australian rainfall depths, which had been adjusted to correspond to a common moisture index.

Also given in Figure 4 are PMP values applicable to a point, based on those given by Wiesner (1970). If a PMP value is required for an area smaller than 1 km^2 the value can be estimated by using linear interpolation between the 1 km^2 and the point values.

The initial rainfall depth for the 'smooth' (**D_S**) and/or 'rough' (**D_R**) terrain categories are read from the DDA curves for the required catchment area and storm duration. To obtain rainfall values for intermediate durations a plot of rainfall (log) versus duration (linear) can be used. The value for the specified duration can then be interpolated.

The PMP estimates for the catchment are calculated from:

PMP Value = (S H D_S + R H D_R) H MAF H EAF

This value should then be rounded to the nearest 10 mm.

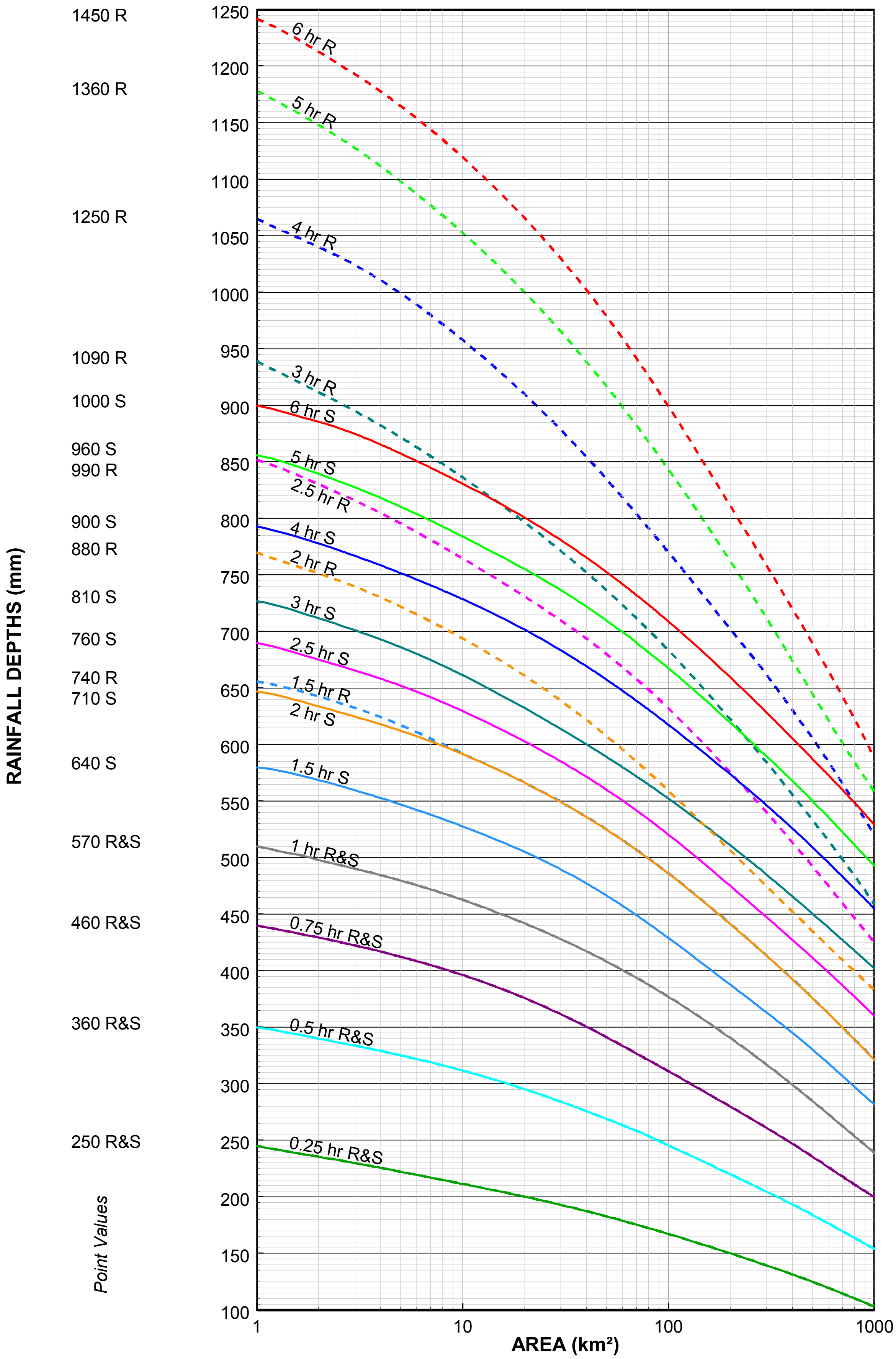

Figure 4: Depth-Duration-Area Curves of Short Duration Rainfall

5. DESIGN TEMPORAL DISTRIBUTION OF PMP

A design temporal distribution was derived using pluviograph traces recorded in major Australian storms. This pattern is shown in Table 1 with figures rounded to 1% and presented as a mass curve in Figure 9.

Table 1: Design Temporal Distribution of Short Duration PMP

% of time	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
% of PMP	0	4	10	18	25	32	39	46	52	59	64	70	75	80	85	89	92	95	97	99	100

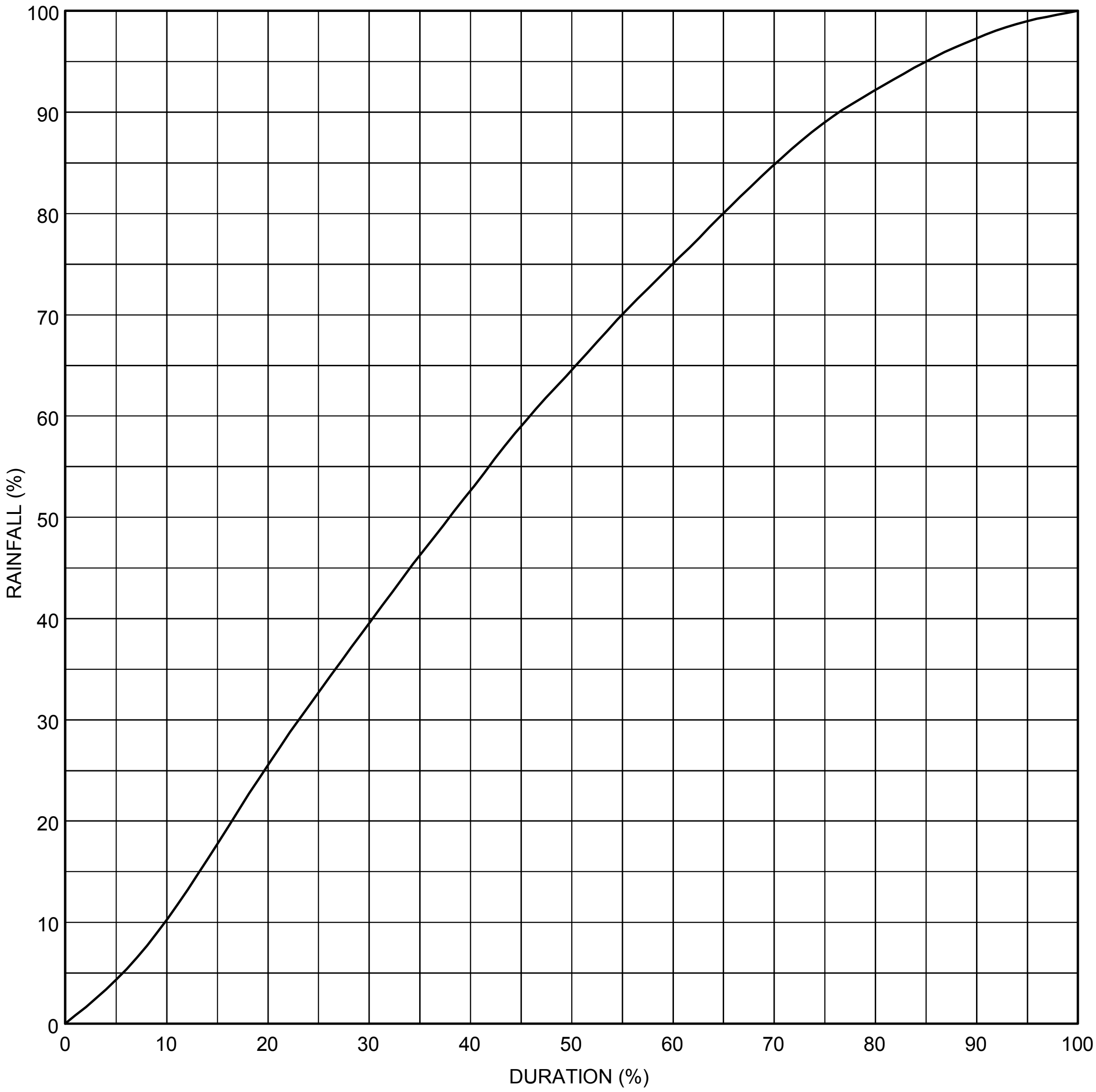

Figure 5: Generalised Short Duration Method Temporal Distribution

6. DESIGN SPATIAL DISTRIBUTION OF PMP

The design spatial distribution for convective storm PMP is given in Figure 6. It is based on the distribution provided by the United States Weather Bureau (1966) and the World Meteorological Organization (1986) but has been modified in light of Australian experience. It assumes a virtually stationary storm and can be oriented in any direction with respect to the catchment. Instructions for the application of the spatial distribution are given below and an example is given in Appendix 2.2.

For simplicity and consistency of application, it is recommended that PMP depth be distributed using a step-function approach. This means having a constant value at all points in the interval between consecutive ellipses (or within the central ellipse), and stepping to a new constant value at each new ellipse. This constant value between ellipses is the mean rainfall depth for that interval and is derived by the procedure described below. Further information on the rationale behind this method may be found in Taylor et al. (1998).

Instructions for the use of the spatial distribution diagram

Step 1 Positioning the spatial distribution diagram

Enlarge or reduce the size of the spatial distribution diagram (Figure 6) to match the scale of the catchment outline map. Overlay the spatial distribution diagram on the catchment outline and move it to obtain the best fit by the smallest possible ellipse. This ellipse is now the outermost ellipse of the distribution.

Step 2 Areas of catchment between successive ellipses

Determine the area of the catchment lying *between* successive ellipses ($CBtn_i$, where the i^{th} ellipse is one of the ellipses A to J).

Where the catchment completely fills both ellipses, this is just the difference between the areas enclosed by each ellipse as given in Table 2.3:

$$CBtn_i = Area_i - Area_{i-1}$$

Where the catchment only partially fills the interval between ellipses, use planimetering or a similar method to determine this area.

Step 3 Area of catchment enclosed by each ellipse

Determine the area of the catchment *enclosed by* each ellipse ($CEnc_i$):

$$CEnc_i = \sum_{k=A}^{i} CBtn_k$$

The area of the catchment enclosed by the outermost ellipse will be equal to the total area of the catchment.

Step 4 Initial mean rainfall depth enclosed by each ellipse

Obtain the x-hour initial mean rainfall depths ($IMRD_i$) for each of the areas enclosed by successive ellipses ($CEnc_i$) (Step 3).

Where the catchment completely fills an ellipse ($CEnc_i$=$Area_i$), determine the x-hour initial mean rainfall depth for this area from Table 2.3. Where the catchment only partially fills an ellipse ($CEnc_i$ < $Area_i$), determine the x-hour initial mean rainfall depth for that area from the appropriate Depth-Duration-Area (DDA) curves (Figure 4).

Table 2: Initial Mean Rainfall Depths Enclosed by Ellipses A-H in Figure 6

Ellipse label	*Area Enclosed ((km²)*	*Area between (km²)*	*Initial Mean Rainfall Depth (mm)*										
			Duration (hours)										
			0.25	*0.5*	*0.75*	*1*	*1.5*	*2*	*2.5*	*3*	*4*	*5*	*6*
SMOOTH													
A	2.6	2.6	232	336	425	493	563	628	669	705	771	832	879
B	16	13.4	204	301	383	449	513	575	612	642	711	765	811
C	65	49	177	260	330	397	453	511	546	576	643	695	737
D	153	88	157	230	292	355	404	459	493	527	591	639	679
E	280	127	141	207	264	321	367	418	452	490	551	594	634
F	433	153	129	190	243	294	340	387	422	460	520	562	599
G	635	202	118	174	223	269	314	357	394	434	491	531	568
H	847	212	108	161	208	250	293	335	373	414	468	506	544
ROUGH													
A	2.6	2.6	232	336	425	493	636	744	821	901	1030	1135	1200
B	16	13.4	204	301	383	449	575	672	742	810	926	1018	1084
C	65	49	177	260	330	397	511	590	663	717	811	890	950
D	153	88	157	230	292	355	459	527	598	647	728	794	845
E	280	127	141	207	264	321	418	480	546	590	669	720	767
F	433	153	129	190	243	294	387	446	506	548	621	664	709
G	635	202	118	174	223	269	357	417	469	509	578	613	656
H	847	212	108	161	208	250	335	395	441	477	541	578	614

Note that no initial mean rainfall depths are required for ellipses I and J because the areas of these ellipses are greater than 1,000 km^2 which is the areal limit of the DDA curves.

Step 5 Adjusted mean rainfall depth enclosed by each ellipse

Adjust the initial mean rainfall depths for moisture and elevation using the adjustment factors and procedure described in Section 4:

$$AMRD_i = IMRD_i \times MAF \times EAF$$

The adjusted mean rainfall depth (AMRD) for the area enclosed by the outermost ellipse will be equal to the (unrounded) PMP for the whole catchment (Section 4.5).

Step 6 Volume of rain enclosed by each oval

Multiply the area of the catchment enclosed by each ellipse ($CEnc_i$) (Step 3) by the corresponding adjusted mean rainfall depth for that area ($AMRD_i$) (Step 5) to obtain the volume of rainfall over the catchment and within each ellipse ($VEnc_i$):

$$VEnc_i = AMRD_i \times CEnc_i$$

Step 7 Volume of rainfall between successive ellipses

Obtain the volume of rainfall over the catchment and between successive ellipses ($VBtn_i$) by subtracting the consecutive enclosed volumes ($VEnc_i$) (Step 6):

$$VBtn_i = VEnc_i - VEnc_{i-1}$$

The volume of rainfall within the central ellipse has already been obtained in Step 6.

Step 8 Mean rainfall depth between successive ellipses

Obtain the mean rainfall depth over the catchment and between successive ellipses (MRD_i) by dividing the volume of rainfall between the ellipses ($VBtn_i$) (Step 7) by the catchment area between them ($CBtn_i$) (Step 2):

$$MRD_i = \frac{VBtn_i\,(Step 7)}{CBtn_i\,(Step 2)}$$

Step 9 Other PMP Durations

Repeat steps 1 to 8 for other durations.

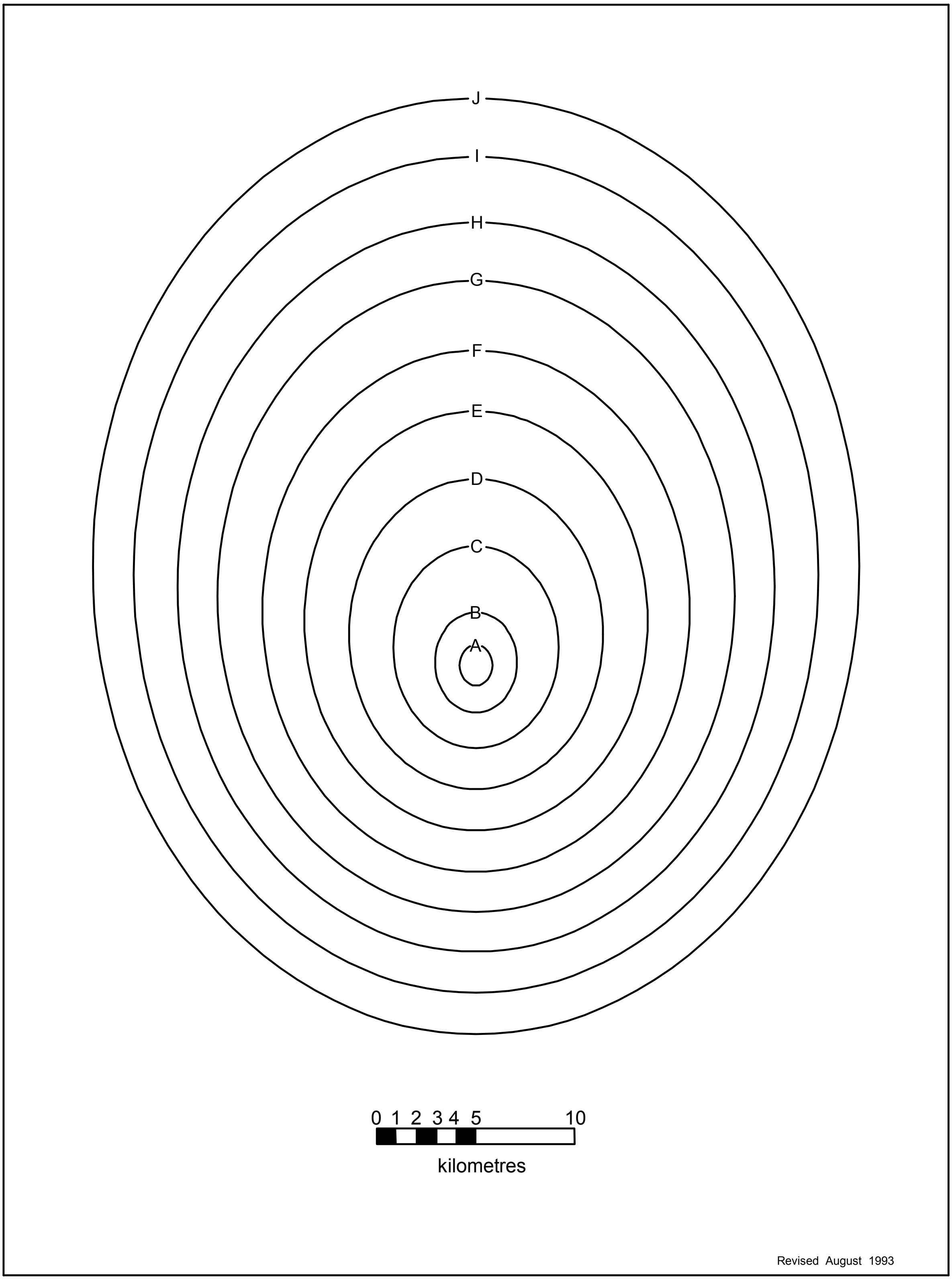

Figure 6: Generalised Short Duration Method Spatial Distribution

7. SEASONAL VARIATION OF PMP

The meteorological events associated with short duration, limited area PMP are most likely to be summer or early autumn convective storms. They may be isolated 'supercells', or they may consist of numerous convective cells embedded in a larger storm system. However, other seasonal factors, such as high antecedent rainfall, may cause greater floods to occur at other times of the year.

In some regions summers are mostly dry so very large catchment loss rates may be assumed in the calculation of the probable maximum summer flood. If the winters are wet, winter PMP values with low losses may produce a higher flood. This is sometimes the case in southwestern Australia.

The areal limit for short duration winter PMP estimates is taken as 500 km^2. It is reasonable to transpose smaller scale convective storms between seasons, as their basic structure is not considered to vary significantly with season. However, seasonal transposition of synoptic-scale storms to estimate PMP over large areas is not considered realistic.

For Australian catchments south of 30ES, Figure 7 can be used to convert the annual PMP to the PMP for a specific month. The monthly percentage moisture adjustment has been derived for a number of locations in southern Australia by calculating the extreme moisture index for each month as a percentage of the extreme annual moisture index. The highest monthly values are given in Figure 7. It is a straightforward procedure to calculate the annual PMP and convert it to a monthly PMP by multiplying by the appropriate percentage given in Figure 7.

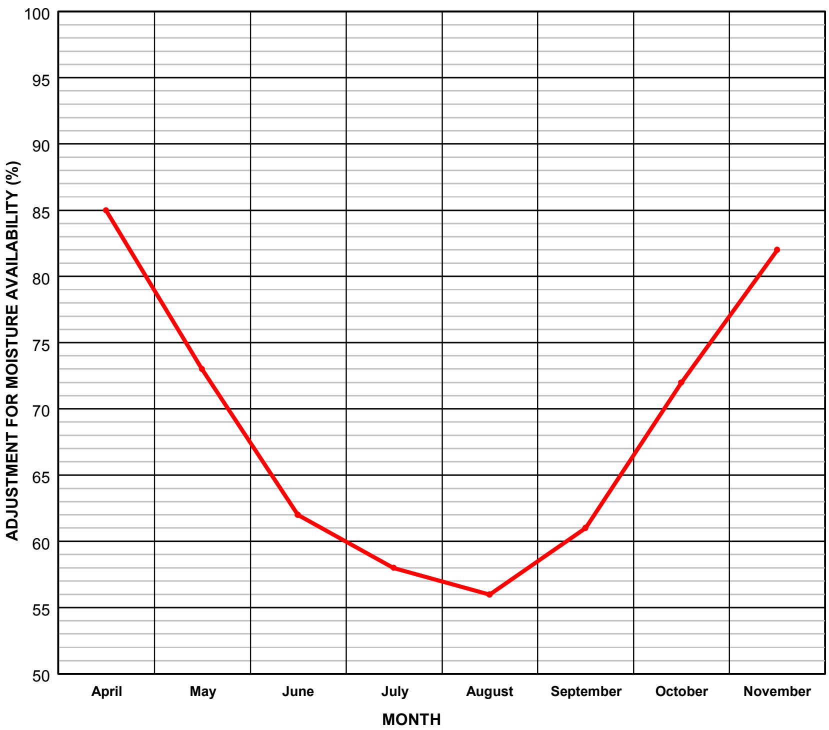

Figure 7: Monthly Percentage Moisture Adjustment for Southern Australia (south of 30ES) Note: The areal limit for winter is 500km^2

8. NOTIONAL AEP OF PMP DEPTHS DERIVED USING THE GSDM

In theory, the PMP concept, as defined in section 2, implies zero probability of exceedance. However, the estimates made by the various PMP methods have a non-zero probability of exceedance. For example, the '*in situ* maximisation' method PMP estimates for the Fortescue River catchment in Western Australia were exceeded by rainfall from Tropical Cyclone Joan in 1975 (Kennedy, 1982). The maximised storm depths from the Dapto 1984 storm (Shepherd and Colquhoun, 1985) near Wollongong in NSW exceeded the 'method of adjusted United States data' PMP estimates used at the time. Notional probabilities of exceedance can therefore be associated with the application of the method (i.e. the methodology plus the limitations of available data) used to estimate the PMP, but not with the concept of PMP itself.

Using deterministic methods of estimating PMP rather than statistical methods, means that the assignment of Annual Exceedance Probabilities (AEPs) to the PMP estimates is not straightforward. The uncertainties associated with any estimate of the exceedance probability of a PMP depth are very large. However, by using the same assumptions to estimate AEPs for each of the PMP methods, the results can provide useful guidance in a comparative sense (Pearce, 1994).

Estimates of PMP depth have been made using a variety of methods for some catchments (e.g. *in situ*, limited transposition, generalised), but the associated notional probabilities vary considerably. Generalised methods of PMP estimation, applicable to different meteorological regions, can also have different exceedance probabilities. Probabilities of variables such as temporal patterns, spatial patterns, antecedent rainfall, losses, reservoir levels, flood model assumptions etc. assumed in converting rainfall to floods will also affect the notional exceedance probability of the PMF with respect to that of the PMP estimates. However, as discussed above for the PMP, if similar assumptions and flood models are used in transforming the PMP to PMF, the resultant design flood can provide useful guidance in comparing safety between various dams.

Kennedy and Hart (1984) used notional AEPs for various PMP methods as a means of indicating the different security levels provided by the different methods. Laurenson and Kuczera (1999) issued interim estimates of the AEP which included a modification of Kennedy and Hart's (1984) figures. They recommended an AEP of 10^{-7} for areas of 100 km^2 and below, rising to 10^{-6} for an area of 1000 km^2. On the subject of confidence limits, they added:

- Recommended AEP values plus or minus two orders of magnitude of AEP be regarded as notional upper and lower limits for true AEPs;
- Recommended AEP values plus or minus one order of magnitude of AEP be regarded as confidence limits with about 75% subjective probability that the true AEP lies within the limits; and
- The recommended AEP values be regarded as the current best estimates of the AEPs.

9. CONCLUSION

The Generalised Short Duration Method of estimating Probable Maximum Precipitation described here enables design engineers to make estimates of PMP for small areas and short durations for any site in Australia. The method is based partly on United States data as only a few severe short duration rainstorms have been adequately documented in Australia. It should be noted, however, that the highest rainfall depths at some durations for the ‘rough’ terrain category were derived from depths recorded in a storm that occurred near Dapto, New South Wales in 1984.

This document included both the revised method of spatial distribution of GSDM depth estimates introduced in 1996 and the updated moisture data used by the Hydrometeorology Section of the Bureau of Meteorology since 2001. It supersedes ‘Bulletin 53: The Estimation of Probable Maximum Precipitation in Australia: Generalised Short Duration Method’ (Bureau of Meteorology, 1994), and should be used instead.

The notional AEP of the GSDM estimates is approximately 10^{-7} for an area of 100 km² rising to 10^{-6} for an area of 1000 km² for all durations covered by the method (Laurenson and Kuczera, 1999). The uncertainty attached to these estimates is discussed in Section 8.

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Appendix 1

GSDM CALCULATION SHEET

LOCATION INFORMATION

Catchment .... Area .... km²

State .... Duration Limit .... hrs

Latitude ....E..............’ S Longitude.....................E...............’ E

Portion of Area Considered:

Smooth , **S** = ... (0.0 - 1.0) Rough , **R** = ... (0.0 - 1.0)

ELEVATION ADJUSTMENT FACTOR (EAF)

Mean Elevation ....m

Adjustment for Elevation (-0.05 per 300m above 1500m) ....

EAF = ... (0.85 - 1.00)

MOISTURE ADJUSTMENT FACTOR (MAF)

MAF = ... (0.40 - 1.00)

PMP VALUES (mm)

Duration (hours)	**Initial Depth - Smooth (D_S)**	**Initial Depth - Rough (D_R)**	**PMP Estimate = (D_SHS + D_RHR) H MAF H EAF**	**Rounded PMP Estimate (nearest 10 mm)**
0.25				
0.50				
0.75				
1.0				
1.5				
2.0				
2.5				
3.0				
4.0				
5.0				
6.0				

Prepared by .... Date ..../.........../..........

Checked by .... Date ..../.........../..........

Appendix 2

EXAMPLE OF THE APPLICATION OF THE GSDM

A2.1 PMP Estimates for the Example Catchment

All calculations and relevant information are recorded on the GSDM Calculation Sheet, Table A2.1.

(i) Estimates of short duration PMP are required for a hypothetical catchment in New South Wales, centred around the coordinates 36E25’ S 148E15’ E. The catchment area is 110 km².

(ii) From Figure 2 it is determined that the catchment lies within the intermediate zone. Linear interpolation across the zone indicated a maximum duration of 5 hours.

(iii) From a suitably contoured map of the area, it was found that 10% of the catchment was considered ‘smooth’ and the remaining 90% ‘rough’. ‘Rough’ terrain is that in which elevation changes of 50 m or more within horizontal distances of 400 m are common. Terrain that was within 20 km of ‘rough’ terrain was classified as ‘rough’. ‘Smooth’ terrain within the catchment but further than 20 km from ‘rough’ terrain was classified as ‘smooth’.

$S = 0.1$ and $R = 0.9$

(iv) From Figure 4, the initial depths for both the ‘smooth’, D_S, and ‘rough’, D_R, categories were read, for a catchment area of 110 km^2 for each duration up to 5 hours.

(v) The average elevation of the catchment was found to be 1750 m.

Adjustment for Elevation	=	- 0.05 per 300 m above 1500m
	=	- ((1750-1500)/300) H (0.05)
	=	- 0.04

EAF = 1.0 - 0.04 = 0.96

(vi) From Figure 3, the moisture adjustment factor was found to be 0.60.

MAF = 0.60

(vii) PMP depth = $(S \text{ H } D_S + R \text{ H } D_R) \text{ H } EAF \text{ H } MAF$

= $(0.1 \text{ H } D_S + 0.9 \text{ H } D_R) \text{ H } 0.96 \text{ H } 0.60$

The estimates were then rounded to the nearest 10 mm.

Table A2.1: Example GSDM Calculation Sheet

LOCATION INFORMATION

Catchment ..... EXAMPLE .....

Area ..... 110 ..... km²

State ..... N.S.W. ......

Duration Limit ..... 5 ..... hrs

Latitude ..... 36..E ..... 25..’ S

Longitude ..... 148..E..... 15..’ E

Portion of Area Considered:

Smooth , **S** = ... 0.1 ..... (0.0 - 1.0)

Rough , **R** = ... 0.9 ..... (0.0 - 1.0)

ELEVATION ADJUSTMENT FACTOR (EAF)

Mean Elevation ..... 1750 ..... m

Adjustment for Elevation (-0.05 per 300m above 1500m) ....-0.04 .....

EAF = ... 0.96 ..... (0.85 - 1.00)

MOISTURE ADJUSTMENT FACTOR (MAF)

MAF = ... 0.60 ..... (0.40 - 1.00)

PMP VALUES (mm)

Duration (hours)	Initial Depth - Smooth (D_S)	Initial Depth - Rough (D_R)	PMP Estimate = (D_SHS + D_RHR) H MAF H EAF	Rounded PMP Estimate (nearest 10 mm)
0.25	164	164	94	90
0.50	242	242	139	140
0.75	306	306	176	180
1.0	372	372	214	210
1.5	423	480	273	270
2.0	480	552	314	310
2.5	514	624	353	350
3.0	546	675	381	380
4.0	611	760	429	430
5.0	661	832	469	470
6.0	-	-	-	-

Prepared by ....N. Smith.......................................

Date ....1...../....06....../...03.......

Checked by ....P. Citizen.....................................

Date ....3..../.....06....../.....03.......

A2.2 Spatial distribution over the example catchment

In this example, the distribution of only the three-hour PMP will be derived. Results are given in columns a-h of Table A2.2.

Step 1 Positioning the spatial distribution diagram

The scale of the spatial distribution diagram was altered to match that of the catchment outline map. The spatial distribution diagram was placed over the catchment outline to obtain the best fit by the smallest possible ellipse. Ellipse E encloses the catchment as shown in Figure A2.1.

Step 2 Areas of catchment between successive ellipses

The catchment areas *between* successive ellipses ($CBtn_i$) were determined. The results are listed in column b.

e.g. between ellipses A and B, $CBtn_B = 13.4\ km^2$ (from Table 2)
between ellipses B and C, $CBtn_C = 37.7\ km^2$ (by planimetering)

Step 3 Area of catchment enclosed by each ellipse

The catchment area *enclosed by* each ellipse ($CEnc_i$) (column c) was calculated by progressively accumulating the catchment areas between ellipses (column b).

e.g. for ellipse C, $CEnc_C = 2.6 + 13.4 + 37.7 = 53.7\ km^2$

As a check, the area enclosed by the outermost ellipse, ellipse E, which is 110 km^2, should equal the area of the catchment.

Step 4 Initial mean rainfall depth enclosed by each ellipse

Since the catchment completely fills ellipses A and B, the 3-hour initial mean rainfall depths ($IMRD_i$) at these areas may be determined from Table 2, weighting and summing the 'smooth' and 'rough' depths according to the proportions of 'smooth' and 'rough' terrain (Section A2.1).

i.e., 3 hr, ellipse A, 'smooth' = 705 mm
3 hr, ellipse A, 'rough' = 901 mm
$IMRD_A = (0.1 \times 705 + 0.9 \times 901) = 881$ mm

For ellipses C, D and E, the initial mean rainfall depths were determined from the 3-hour DDA curves in Figure 4.

e.g. for ellipse C, 3 hr, 53.7 km^2, 'smooth' = 585 mm
3 hr, 53.7 km^2, 'rough' = 731 mm
$IMRD_C = (0.1 \times 585 + 0.9 \times 731) = 716$ mm

The initial mean rainfall depths are listed in column d.

Step 5 Adjusted mean rainfall depth enclosed by each ellipse

The initial mean rainfall depths (column d) were adjusted for moisture and elevation (column e) by multiplying by the moisture and elevation adjustment factors (Section A2.1).

e.g. for ellipse C, $AMRD_C = 716 \times 0.60 \times 0.96 = 412$ mm

As a check, the adjusted mean rainfall depth for the area enclosed by the outermost ellipse, ellipse E, which is 382 mm, should approximately equal the 3-hour (unrounded) PMP for the catchment (Section A2.1).

Step 6 Volume of rainfall enclosed by each ellipse

The adjusted mean rainfall depths (column e) were multiplied by the areas of the catchment enclosed by each ellipse (column c) to give values for the volume of rainfall enclosed by each ellipse ($VEnc_i$) (column f).

e.g. for ellipse C, $VEnc_C = 412 \times 53.7 = 22{,}124$ mm.km^2

Step 7 Volume of rainfall between successive ellipses

Consecutive enclosed rainfall volumes (column f) were subtracted to obtain the rainfall volume between ellipses ($VBtn_i$) (column g).

e.g. between ellipses B and C, $VBtn_C = 22{,}124 - 7{,}312 = 14{,}812$ mm.km^2

Step 8 Mean rainfall depth between successive ellipses

The mean rainfall depths between successive ellipses (MRD_i) (column h) were obtained by dividing the rainfall volume between ellipses (column g) by the area between ellipses (column b).

e.g. between ellipses B and C, $MRD_C = 14{,}812 / 37.7 = 393$ mm

Step 9 Other PMP Durations

Repeat the above steps for other durations for which the spatial distribution of PMP is required.

Table A2.2: Calculation of the Spatial Distribution of 3-hour PMP over the Example Catchment

a Ellipse	b Step 2 Catchment area between ellipses (km^2)	c Step 3 Catchment area enclosed by ellipse (km^2)	d Step 4 Initial mean rainfall depth (mm)	e Step 5 Adjusted mean rainfall depth (mm)	f Step 6 Rainfall volume enclosed by ellipse (mm.km^2)	g Step 7 Rainfall volume between ellipses (mm.km^2)	h Step 8 Mean rainfall depth between ellipses (mm)
A	2.6	2.6	881	507	1,318	1,318	507
B	13.4	16	793	457	7,312	5,994	447
C	37.7	53.7	716	412	22,124	14,812	393
D	42.6	96.3	673	388	37,364	15,240	358
E	13.7	**110**	663	**382**	42,020	4,656	340

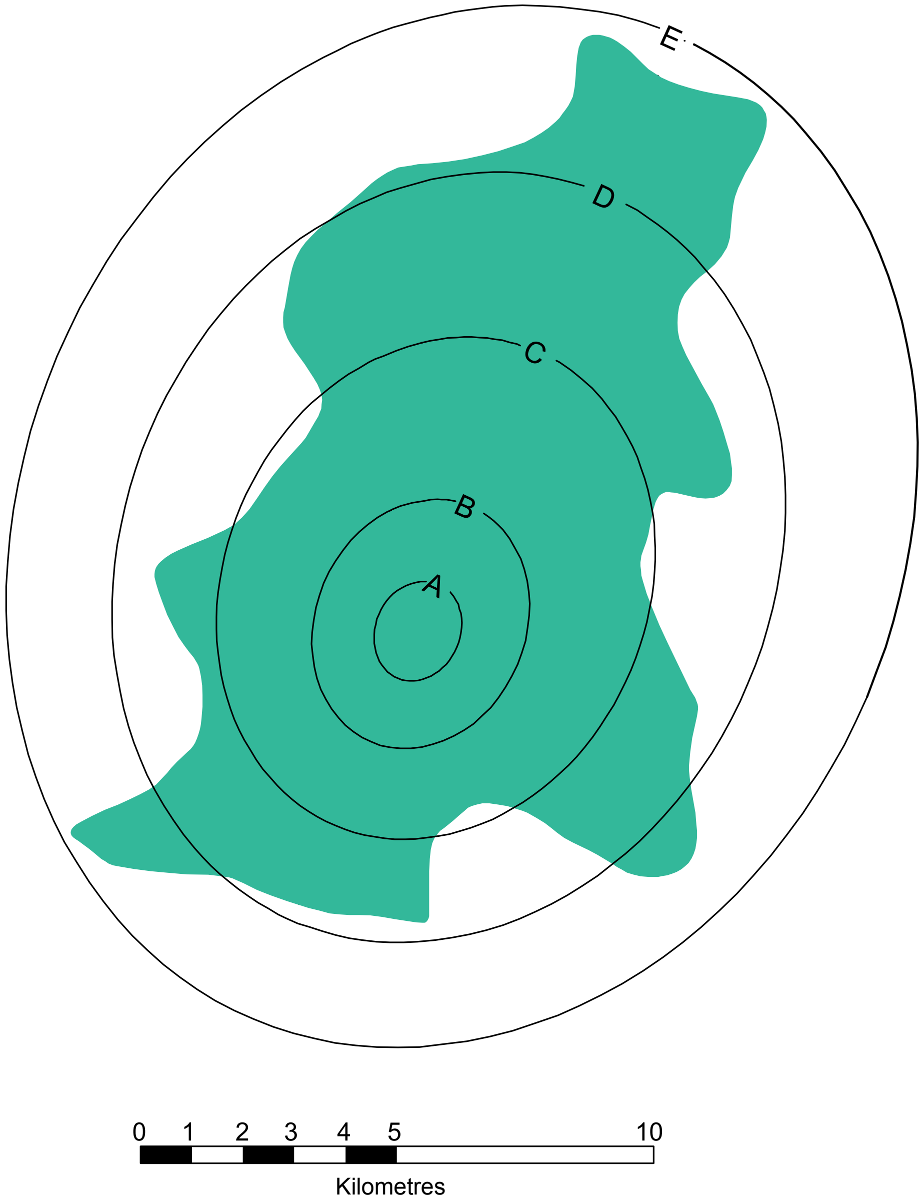

Figure A2.1: Spatial Distribution over Example Catchment

Appendix 3

NOTABLE SHORT DURATION AREAL RAINFALL EVENTS RECORDED IN INLAND AND SOUTHERN AUSTRALIA

A3.1 The Molong Storm of 20 March 1900

On 20 March 1900 a series of thunderstorms formed over a strip of country about 75 km wide extending from near Hungerford to the southeast near Moss Vale in New South Wales. The heaviest rainfall occurred in the Orange-Molong area. The information given by Russell (1901) indicates that the storm lasted for about three hours. The storm dew point temperature was estimated as 19EC. The recorded storm rainfall and the rainfall normalised for the moisture content corresponding to an extreme dew point temperature of 23.5EC are compared with the PMP estimates in Table A4.1.

Table A3.1: Depth-Area Data for the Molong Storm

Area (km²)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 23.5EC (mm)	3-hour PMP Estimate (mm)
10	205	300	450
50	195	290	400
100	190	280	380
500	180	260	310
1000	170	250	270

A3.2 The St Albans Storm of 8 January 1970

On 8 January 1970 between 1400 and 1730 EST an intense thunderstorm was located in the St Albans area about 15 km west-northwest of Melbourne. Near the centre of the storm rainfall totals exceeding 120 mm were recorded. The storm was studied by Finocchiaro (1970). Radar observations and information obtained from private raingauge readers indicate that about 90 per cent of the total rainfall fell within a period of 1.5 hours. The storm dew point was assessed to have been 13EC and the extreme dew point for the storm area for January is 20.4EC. The storm data are compared with the PMP estimates in Table A3.2.

Table A3.2: Depth-Area Data for the St Albans Storm

Area (km²)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 20.4EC (mm)	1.5-hour PMP Estimate (mm)
1	111	210	300
10	88	170	280
20	80	150	260
30	72	140	260
50	63	120	240

A3.3 The Woden Valley Storm of 26 January 1971

During the evening of 26 January 1971 extremely heavy rainfall associated with an almost stationary thunderstorm complex fell over the Canberra suburbs of Farrer and Torrens for about 90 minutes (Bureau of Meteorology, 1972). The resulting flood in the Woden Valley claimed several lives. The storm dew point temperature was assessed as 14EC and the extreme dew point is 22.8EC. The storm data are compared with the PMP estimates in Table A3.3.

Table A3.3: Depth-Area Data for the Woden Valley Storm

Area (km^2)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 22.8EC (mm)	1.5-hour PMP Estimate (mm)
1	102	220	370
10	99	210	340
50	87	190	300
100	78	170	270
250	62	130	240

A3.4 The Melbourne Storm of 17 February 1972

On the afternoon of 17 February 1972 an intense thunderstorm developed over the city of Melbourne and the suburbs immediately north of the city. The storm was observed by radar and three pluviograph traces were obtained from sites near the centre of the storm. This storm lasted for about 60 minutes and produced severe local flooding. Rainfall depths for this storm are given by Pierrehumbert and Kennedy (1982). The storm dew point was estimated as 12EC and the extreme dew point is 20.9EC. The storm depth-area values are compared with the PMP estimates in Table A3.4.

Table A3.4: Depth-Area Data for the Melbourne Storm

Area (km^2)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 20.9EC (mm)	1-hour PMP Estimate (mm)
2	83	180	270
20	73	160	240
50	68	150	220
100	60	130	200
250	49	110	180

A3.5 The Laverton Storm of 7 April 1977

A storm lasting for about 12 hours brought exceptionally heavy rain to areas to the west and north of Melbourne on 7 April 1977. The heaviest burst in the storm lasted for about 3 hours and affected areas from Laverton to Sunbury. The Melbourne and Metropolitan Board of Works (1979) gives details of the rainfall recorded over the entire storm area. The representative storm dew point temperature was 10EC and the extreme dew point is 20.1EC. The recorded and maximised storm depth-area data are compared with the PMP estimates in Table A3.5.

Table A3.5: Depth-Area Data for the Laverton Storm

Area (km^2)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 20.1EC (mm)	3-hour PMP Estimate (mm)
10	121	310	340
100	96	240	280
400	73	180	240
600	60	150	220
800	53	130	210
1000	51	130	200

A3.6 The Buckleboo Storm of 26 January 1981

On the afternoon of 26 January 1981 an intense and almost stationary thunderstorm produced some of the highest short-duration rainfalls ever recorded in South Australia. While the only quantitative data are daily totals, it is reliably reported that virtually all the rain fell in a period of about three hours. The representative storm dew point was estimated to have been 19EC. The recorded values were adjusted for a moisture content corresponding to a surface dew point temperature of 23.5EC for comparison with the PMP estimates in Table A3.6.

Table A3.6: Depth-Area Data for the Buckleboo Storm

Area (km^2)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 23.5EC (mm)	3-hour PMP Estimate (mm)
10	187	270	450
50	169	250	400
100	154	230	380
500	106	160	310
1000	77	110	270

A3.7 The Barossa Valley Storm of 2 March 1983

During the evening of 2 March 1983 numerous thunderstorm cells produced very heavy rainfall over the Adelaide Plains and the eastern part of the Mt Lofty Ranges. Nearly all the rain fell in a period of about three hours. The thunderstorms occurred in a moist airmass of tropical origin which was fed into the area from the northeast. The storm is described by Burrows (1983).

The rainfall produced severe flash flooding and extensive property damage, particularly in the Barossa Valley and around Dutton. An unofficial gauge on a farm 1 km north of Dutton recorded 330 mm during the storm. Several unofficial gauges recorded totals in excess of 200 mm, whereas the highest value recorded by an official gauge was 103 mm at Angaston. This illustrates the problem of detecting severe local storms with the sparse network of official gauges.

The representative storm dew point temperature was estimated as 20EC and the extreme dew point is 22.2EC. The storm rainfalls are compared with the PMP estimates for a duration of three hours in Table A3.7.

Table A3.7: Depth-Area Data for the Barossa Valley Storm

Area (km^2)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 22.2EC (mm)	3-hour PMP Estimate (mm)
1	300	360	440
10	222	270	400
50	190	230	350
100	173	210	340
500	129	150	270
1000	110	130	240

A3.8 The Dapto Storm of 18 February 1984

An extraordinary heavy rainfall event occurred near Dapto in New South Wales on 18 February 1984, as described by Shepherd and Colquhoun (1985). The rainfall was particularly heavy on and near the Illawarra escarpment. While rain fell for more than 24 hours most of the rain fell in a period of about 6 hours. For durations of around 6 hours and areas up to about 200 km^2 the normalised rainfall values exceed the adjusted United States data. The maximised rainfall values from the Dapto storm were used in deriving the `rough' terrain category DDA curves in Figure 2 in the first edition of *Bulletin 51* by the Bureau of Meteorology (1985). The storm dew point temperature was estimated to be 19EC. The extreme dew point temperature for February is 23.3EC. The 6-hour rainfall values for this storm are given in Table A3.8 where they are compared with the PMP estimates.

Table A3.8: Depth-Area Data for the Dapto Storm

Area (km²)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 23.3EC (mm)	6-hour PMP Estimate (mm)
10	520	750	750
50	450	650	650
100	410	590	600
500	250	360	460
1000	160	230	390

A3.9 The Sydney Storm of 4-7 August 1986

A low pressure centre which moved southwards close to the coast brought very heavy rainfall to the Sydney metropolitan area, the Blue Mountains and the Illawarra region, causing extensive local flooding. Six fatalities resulted from the storm. The Sydney rainfall for the 24 hours to 9 am on 6 August 1986 was a record 328 mm. There was a particularly heavy period of rain on the afternoon of 5 August 1986. Pluviograph data have been used to extract maximum 6 hour depths for that part of the storm which occurred over the metropolitan area. The storm dew point was 10EC and the extreme dew point is 16.7EC. The storm is described by the Bureau of Meteorology (1987). The depth-area rainfall values for the storm are compared with the PMP estimates in Table A3.9.

Table A3.9: Depth-Area Data for the Sydney Storm

Area (km²)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 16.6EC (mm)	6-hour PMP Estimate (mm)
50	133	250	320
200	124	230	270
500	112	210	240
1000	103	190	200

A3.10 The St Kilda Storm of 7 February 1989

On the afternoon of 7 February 1989, a severe thunderstorm brought torrential rainfall to the inner southern and southeastern suburbs of Melbourne (Board of Works, 1989). The storm was centred over the St Kilda area and caused flash flooding. The heavy rainfall part of the storm lasted for about one hour. The representative storm dew point temperature was estimated to have been 14EC and the extreme dew point for February is 20.9EC. The depth-area rainfall values for the storm are compared with PMP estimates in Table A3.10.

Table A3.10: Depth-Area Data for the St. Kilda Storm

Area (km²)	Recorded Storm Rainfall (mm)	Storm Rainfall Adjusted to 20.9EC (mm)	1-hour PMP Estimate (mm)
5	91	160	260
10	85	150	250
20	75	140	240
40	62	110	230
60	53	100	220
80	49	90	210

A3.11 References for Appendix 3

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Model Handover Guide (separate document)