HERVEY BAY CITY COUNCIL



Pulgul Creek Catchment Flood Risk Reduction Study

Final Report

December 2006

Hervey Bay City Council

Pulgul Creek Catchment Flood Risk Reduction Study

December 2006

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Appendices

1 Introduction

John Wilson & Partners (JWP) has been commissioned by Hervey Bay City Council (HBCC) to undertake a Flood Risk Reduction Study for the Pulgul Creek Catchment. The purpose of the analysis is to document existing flooding characteristics within the catchment, assess potential mitigation options for reducing flood risk and meet HBCC design standards in the area. The study will be used for managing both existing and future development within the catchment based upon the reduction of flood risk.

This study represents the first comprehensive study of the entire Pulgul Creek catchment and includes a broadscale hydrologic and hydraulic analysis of the catchment.

The major components of works undertaken for this study have included:

- Definition of sub-catchment boundaries;
- The identification of existing drainage patterns including both piped systems (trunk drainage) and major overland flows;
- Construction of an extensive XP-RAFTS model for the catchment;
- Construction of a fully 2-dimensional (2D) TUFLOW hydraulic model;
- Hydrological and hydraulic model analysis to define flood levels, flow directions and drainage problems in selected regions of the catchment for the Q10, Q20, Q50 and Q100 design flood events;
- Analysis of mitigation options for the purposes of flood risk reduction;
- Preliminary construction cost estimates for the mitigation options; and
- Documentation of the study methodology and outcomes as part of a formal report on the investigation including a risk management report.

The following sections of this report aim to fully document the analysis works undertaken as part of this investigation.

2 Study Area

The Pulgul Creek catchment is bounded by Tavistock Street in the west, Torquay Terrace in the north, Shore Road in the south and Hervey Bay bay to the east. The catchment is approximately 850 hectares. Within the catchment there has been significant infill development and reconfiguration from park residential to low density residential. A previous drainage study has been undertaken for the catchment in 1996 with hydraulic modelling downstream of Elizabeth Street. As a result of the significant changes to land use and the coverage of the previous study, a revised flood study covering the entire catchment is required.

Figure 2.1 shows the Pulgul Creek catchment boundary and study area.

N

LEGEND

Study Boundary

200 0 200 400 600

Metres

Scale: 1:20,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 2.1

LOCALITY

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3 Study Data

The works undertaken as part of this study, particularly the establishment of the TUFLOW model of the catchment have been prepared based upon a compilation of data sources provided by Hervey Bay City Council and additional survey. Specifically, the models have been developed using a range of data sources and information, each of which are outlined and discussed separately below.

3.1 Topography Data

Topographical data for the catchment was provided in the form of contour information at 0.5m increments. As the contour data represents the only available topographical information provided for the study, this information has been adopted. JWP have prepared a Digital Terrain Model (DTM) using all of the contour data supplied to accommodate data extraction for the various modelling tasks undertaken as part of this study. The DTM as prepared using the MapInfo Vertical Mapper software is illustrated in Figure 3.1, clearly showing the catchment location and topographical variation.

The 2D modelling approach utilises the entire DTM across the model area. The DTM is sampled at increments corresponding to the 2D hydraulic model grid size chosen. The grid size is described in detail in Section 5.4. Contour information at 0.5m is coarse and it is likely that many key drainage features are poorly defined in the DTM. When this data is sampled for the hydraulic model, the definition is further lost. Thus, it is recommended that DTMs be created from the raw ALS data rather than contours produced from the raw data. The data provided is suitable for a broadscale flood study. Any areas that have been identified as needing further ground level information for more detailed modelling have been highlighted throughout this report.

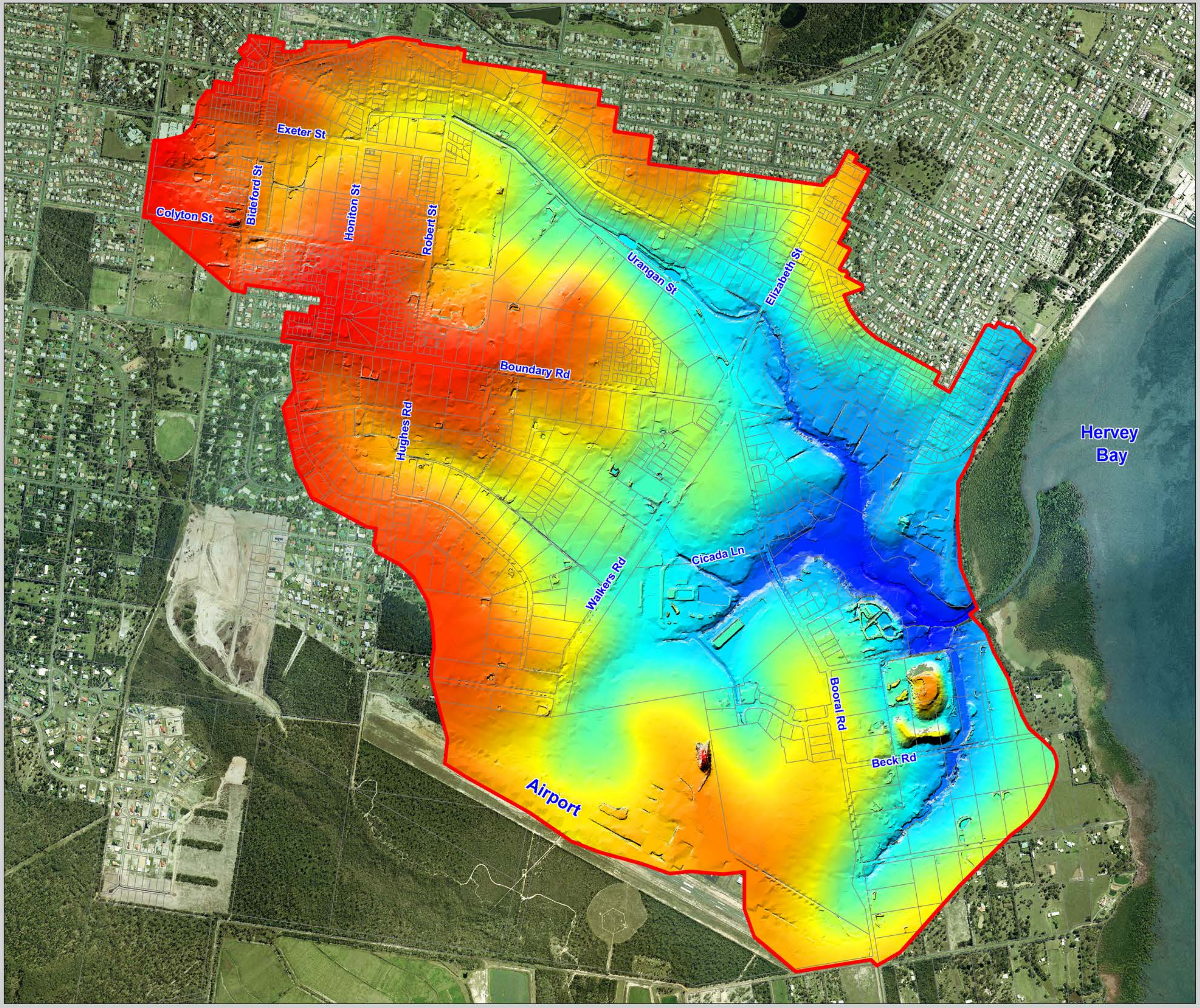
JWP note that the DTM prepared for the purposes of this study represents an interpolated topographic surface based upon contour information which itself represents an interpolated surface. As such, the DTM prepared for this study is unlikely to be sufficiently accurate to enable more detailed flood analysis works to be performed. The DTM is however suitable and appropriate for the purposes of a broad flood risk study as is the purpose of this study.

3.2 Survey Data

Collection of limited field survey data for the catchment has been undertaken as part of this project. The intent of the field survey data collection was to infill missing information, obtain structure details and to provide more detailed topographical information at discrete and critical locations throughout the study area. Areas where more detailed survey information was required were defined by JWP following a detailed assessment and review of the information initially provided. As part of the study, JWP were responsible for management of these works which included the preparation of detailed survey briefs, calling of survey tenders and managing the field collection data.

All detailed survey works collected for this project were undertaken by Surveyors@Work, a locally based and independent survey company in Hervey Bay. This information was collected using both traditional and GPS survey techniques and was provided in a digital AutoCAD format. JWP utilised this information to update the various drainage network details within the models.

The survey data included spot levels and cross-sections at specific locations in the catchment. As the survey data was only collected at discrete locations the data was not used to update or compile a more accurate DTM for the catchment however, was included in the TUFLOW model to enhance the hydraulic representation of the catchment.



N

LEGEND

Study Boundary

DTM

30

12

00

METRE

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 3.1

DIGITAL TERRAIN MODEL (DTM)

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Projection Trans. Merc. MGA56(GDA 94)

3.3 Pipe Data

Existing pipe and culvert crossing data throughout the catchment was provided by Council. Details of the existing pipe information were provided through Council supplied GIS data. This data was supplemented using detailed survey information collected at discrete areas, as discussed in Section 3.2.

All the available information as supplied for the study was consolidated to prepare the existing pipe system details within the TUFLOW model. Through liaison with Council it was agreed to model culverts larger than 600mm in diameter. In some cases pipes smaller than 600mm were included where the nature of the system meant that the infrastructure was critical for flood level determination.

3.4 Site Inspection

As part of the works for this study, JWP have undertaken a detailed and comprehensive site inspection of the Pulgul Creek catchment. The site inspection also included an extensive project inception meeting with HBCC.

The site inspection was documented by way of site notes and photographs. Outcomes from the inspection included:

- Assessment of physical catchment parameters including appropriate roughness parameters;
- Verification of crossings and existing hydraulic structures;
- A comprehensive investigation of the waterway; and
- Understanding of the flow dynamics of the catchment area and major waterway systems.

4 Catchment Modelling

4.1 Hydrological Modelling

The analysis of runoff from the catchment was performed using the non-linear runoff routing program RAFTS. Hydrographs for design events were produced by routing rainfall through a series of sub catchment storages and along channel links to enable flow determination at critical inflow locations to the hydraulic model.

The total catchment area contributing discharges was found to be 854 hectares. The RAFTS analysis involved division of the catchment into fifteen (15) separate sub-catchments, derivation of various physical properties of the sub catchments and assembly of the sub catchments by way of a nodal network. Figure 4.1 illustrates the catchment sub-division and RAFTS schematisation as used for the hydrological modelling of the catchment.

The storage routing parameter and nonlinearity exponent have been estimated by the relationship developed by Aitken (1986). Model parameters for sub catchment storages have been selected from recommended design values from the following data sources:

- Impervious and pervious areas – Strategic planning information for the catchment as obtained from HBCC;
- Sub-catchment slopes – Contour and topographic data provided by HBCC throughout the catchment;
- Catchment roughness values – Determined in accordance with the hydrologic parameter values recommended by Council for various landuse classifications, site visit and aerial photography; and
- Intensity-Frequency-Duration (IFD) values – HBCC Development Manual.

4.2 Design Rainfall

The design rainfall Intensity-Frequency Duration (IFD) data for Hervey Bay was used in the RAFTS model. Design IFD data for Hervey Bay was extracted from Hervey Bay City Council Development Manual. A copy of the HBCC IFD table is attached in Appendix A.

Rainfall temporal patterns used for the RAFTS analysis were also prepared in accordance with the procedures outlined in AR&R.

N

LEGEND

Study Boundary

Rafts Subcatchments

Rafts Link

Rafts Node

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 4.1

XP-RAFTS HYDROLOGIC MODEL

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Projection Trans. Merc. MGA56(GDA 94)

4.3 RAFTS Model Parameters

The adopted percentage impervious and corresponding Manning's roughness parameters for the various land use classifications were supplied by HBCC and are summarised in Table 4.1 below.

Table 4-1 Catchment Land Use Parameters

Description	% Impervious	Manning's "n"
Industrial	100%	0.015
Commercial	100%	0.015
Water Body	100%	0.015
Park Residential	20%	0.040
Residential Low Density	45%	0.025
Residential Medium Density	60%	0.025
Residential High Density	90%	0.020
Conservation	0%	0.100
Active Open Space	10%	0.040
Road	80%	0.020
Rural	2%	0.070
Utilities	50%	0.025
Educational Facilities	20%	0.025

On the basis of HBCC landuse strategic plan and the supplied landuse hydrologic parameters, the areas of various landuse classifications used in the development of the RAFTS model are summarised below in Table 4.2.

Table 4-2 Summary of Areas for Various Landuse Classifications

Landuse Classification	Pervious Area (ha)	Impervious Area (ha)
Open Space	1.15	0.13
Park Residential	28.81	7.20
Residential Low Density	192.5	157.50
Road	5.69	22.77
Utilities	0.10	0.10
Total	**187.54**	**228.41**

Rainfall loss on each sub-catchment was applied using an initial and continuing rainfall loss model. Design loss parameters for the RAFTS model were based on guideline values as recommended Hervey Bay City Council. These loss rates are consistent with AR&R (2000) which recommends a continuing loss of 2.5 mm/hr and an initial loss of between 15-35 mm be applied in eastern Queensland.

The adopted loss parameters applied for the RAFTS model are presented in Table 4-3 below.

Table 4-3 Adopted Rainfall Loss Parameters used in the RAFTS Model

Initial Loss (mm) / Continuous Loss (mm/hr)			
Pervious Area		**Impervious Area**	
Initial Loss (mm/hr)	Continuing Loss (mm)	Initial Loss (mm/hr)	Continuing Loss (mm)
15	2.5	0	0

The storage coefficient multiplication factor (Bx) in the RAFTS model uniformly modifies the basic calculated sub-catchment storage delay values. The factor represents the dampening effect of the catchment on the excess rainfall. The default Bx value of one (1) was adopted in the RAFTS models.

4.4 Hydrological Flow Verification

The hydrologic assessment indicates that a critical storm duration of 60 minutes dominates throughout the catchment for 100 year ARI flood event. As a result, the 60 minutes storm duration has been adopted for the generation of boundary conditions for incorporation into the hydraulic model.

The estimated peak discharges for the catchment are summarised in the in Appendix B of this report.

As calibration of the hydrological models to recorded data was not possible, rational method flow calculations have been undertaken at key locations for comparison and verification with the RAFTS results. The comparison of results for the 100 Year ARI storm event at key catchment locations are summarised in Table 4-4 below under ultimate catchment conditions. The locations of these RAFTS nodes are shown on Figure 4.1.

Table 4-4 Comparison of Hydrologic Results for the 100 Year ARI Storm Event (Existing Development Conditions)

RAFTS Node	Area (ha)	Rational (m^3/s)	RAFTS (m^3/s)	Percentage Difference (%)
D_3	143.5	47	47.9	1.8 %
D_4	204.3	60	59.5	-0.8 %
D_8	92.53	37	39.7	6.8 %
Out	415.83	110	117.8	6.6 %

Comparisons between peak flows at a range of locations throughout the catchment calculated using the RAFTS model and the Rational Method have identified that the difference of peak flows between the two methods is generally within the order of 10%. Therefore, the RAFTS model is considered to predict flows to an acceptable level of accuracy and has been adopted for the purposes of this assessment.

5 Hydraulic Modelling

5.1 General Overview

JWP have undertaken detailed 2 Dimensional (2D), hydrodynamic flood modelling of the catchment in order to provide accurate and detailed flood information for the entire waterway system. The following information provides details of the software, topographic data and other parameters used in the hydraulic modelling of the catchment.

5.2 Modelling Software

Hydraulic analysis of the study area has been undertaken using the coupled one dimensional (1D) and two dimensional (2D) finite difference model TUFLOW, an industry accepted, Australian owned and commercially available software package highly suited to the investigation of flood behaviour in complex flow scenarios. The model can simulate unsteady hydrodynamic flow in two directions on a rectilinear grid as well as one dimensional unsteady hydrodynamic flow through waterway structures such as culverts. The model is based on a robust finite difference solution scheme able to compute both sub critical and supercritical flow regimes.

The TUFLOW 1D/2D model is suited to simulation of dynamic hydraulic behaviour of overland flow in urban areas. Based on this and TUFLOW's ability to couple hydraulic structures such as culverts and bridges at road crossings in a stable and verified manner, the modelling system was considered the most appropriate investigative tool for the characteristics of Pulgul Creek and the surrounding urban areas.

Major advantages of a combined 1D and 2D modelling approach over traditional 1D approaches include:

- Full topographic survey terrain models are used, rather than selected, discrete cross sections;
- Flow patterns are dictated by the influence of topography and surface roughness conditions rather than by 'forced' flow paths, as used in quasi-two-dimensional networks;
- Flow directions and paths can vary with stage and flow conditions (c/f 'rigid' networks forcing flow paths in quasi-two-dimensional models); and
- Production of detailed output of flow patterns, flood rise and fall animations, and output suitable for direct GIS interfacing. This allows production of accurate depth of flooding, velocity and hazard maps as well as area of influence maps.

Major advantages of a combined 1D and 2D approach over an exclusive 2D approach include:

- Regions lying outside the area of interest that have to be modelled (e.g. to apply boundary conditions) do not necessarily have to be modelled in full 2D. This dramatically decreases the required computational time, leaving more room for detailed modelling of the area of interest; and
- The behaviour of hydraulic structures such as culverts and bridges can be simulated in a more detailed and robust way using a traditional 1D approach. Also, overtopping of bridges can be reliably modelled as the weir flow component of flood flow including the blockage of handrails at crossings can take place within the 2D environment whereas the flow through the hydraulic structure can be modelled in 1D.

5.3 Model Construction

The 2 dimensional TUFLOW model constructed for Pulgul Creek consists of the following elements, each of which are described in more detail in the sections of this report which follow:

- A two-dimensional (2D) curvilinear grid representing the topographic levels within the area of interest extracted directly from the DTM constructed using supplied contour data as outlined in Section 3.1.
- One-dimensional (1D) elements within the 2D grid extent that represent hydraulic structures and fine scale drainage elements;
- Downstream water level boundary applied at the model outlet; and
- Rainfall boundary conditions simulating the local sub-catchment response within the 2D modelling area.

Figure 5.1 illustrates the layout and extent of the hydraulic model constructed for this study.

5.4 2D Topographic Grid

The 2D model topography was derived using the Digital Terrain Model (DTM) constructed from contour data as supplied by Council (Refer Section 3.1). A balance between the number of computational points, level of modelling detail and model run times was required to deliver suitably accurate outcomes in a timely and efficient manner. The decision on the grid spacing to adopt in the 2D model is critical and is based on knowledge of the catchment, the pertinent drainage areas, and previous experience. A grid size of 4m was selected. This grid spacing allows for sufficient detail to be achieved in the urban environment whilst maintaining realistic model run times. The extent of hydraulic modelling is shown in Figure 5.1.

Real world co-ordinate systems have been used for all modelling. The 2D hydraulic model is based on **MGA94 Zone 56** horizontally and **AHD** vertically.

5.5 1D Hydraulic Structure Elements

In a full 2D modelling environment it is often not possible to accurately describe the hydraulic behaviour of structures such as culverts and bridges. This is due to the fact that grid cell sizes often exceed the dimensions of various structures in addition with the grid cells only representing bottom friction and consequently no roof friction or specific hydraulic structure losses. As a result, hydraulic structures are typically more accurately modelled in a 1D modelling environment, thus allowing prescriptive modelling of the exact characteristics of the various structures.

Within the 2D model, 1D elements have been introduced in order to enable the prescriptive modelling of various floodplain structures. The following waterway crossings have been included in the model as 1D elements throughout the study area (Refer Figure 5.1 for structure locations):

ID	Road Name
1	Bideford Street
2	Booral Road
3	Booral Road Bridge
4	Cicada Lane 1
5	Cicada Lane 2
6	Colyton Street
7	Elizabeth Street 1
8	Exeter Street
9	Honiton Street
10	Robert Street 1
11	Robert Street 2
12	Walkers Road 1
13	Walkers Road 2

Table 5-1 summarises the structure details at each of the above locations.

Table 5-1 Structure Details

Culvert ID	Invert Level (mAHD)	Description
1	16.42	4 x 0.45m circular pipe
2	6.54	2 x 1.05m circular pipes
3	2.10	5 x 1.8m circular pipes
4	4.34	2 box culverts 1.8m wide x 1.2 high
5	3.11	2 x 0.6m circular pipes
6	22.37	1 box culvert 0.6m wide x 0.3 high
7	3.85	5 x 1.8m circular pipes
8	11.55	1 box culvert 2.1m wide x 1.5 high
		1 box culvert 2.4m wide x 1.5 high
9	12.71	1 box culvert 1.2m wide x 0.45 high
10	15.05	2 x 0.9m circular pipes
11	16.08	2 x 0.6m circular pipes
12	7.24	5 x 0.825m circular pipes
13	7.16	3 box culverts 0.9m wide x 0.6 high

Each of the structures detailed in Table 5-1 have been prescriptively modelled based on the following data sources:

- Council's pipe network information GIS layers; and
- Verification of structure details and configurations by way of a detailed site inspection including photographic records compiled by JWP.

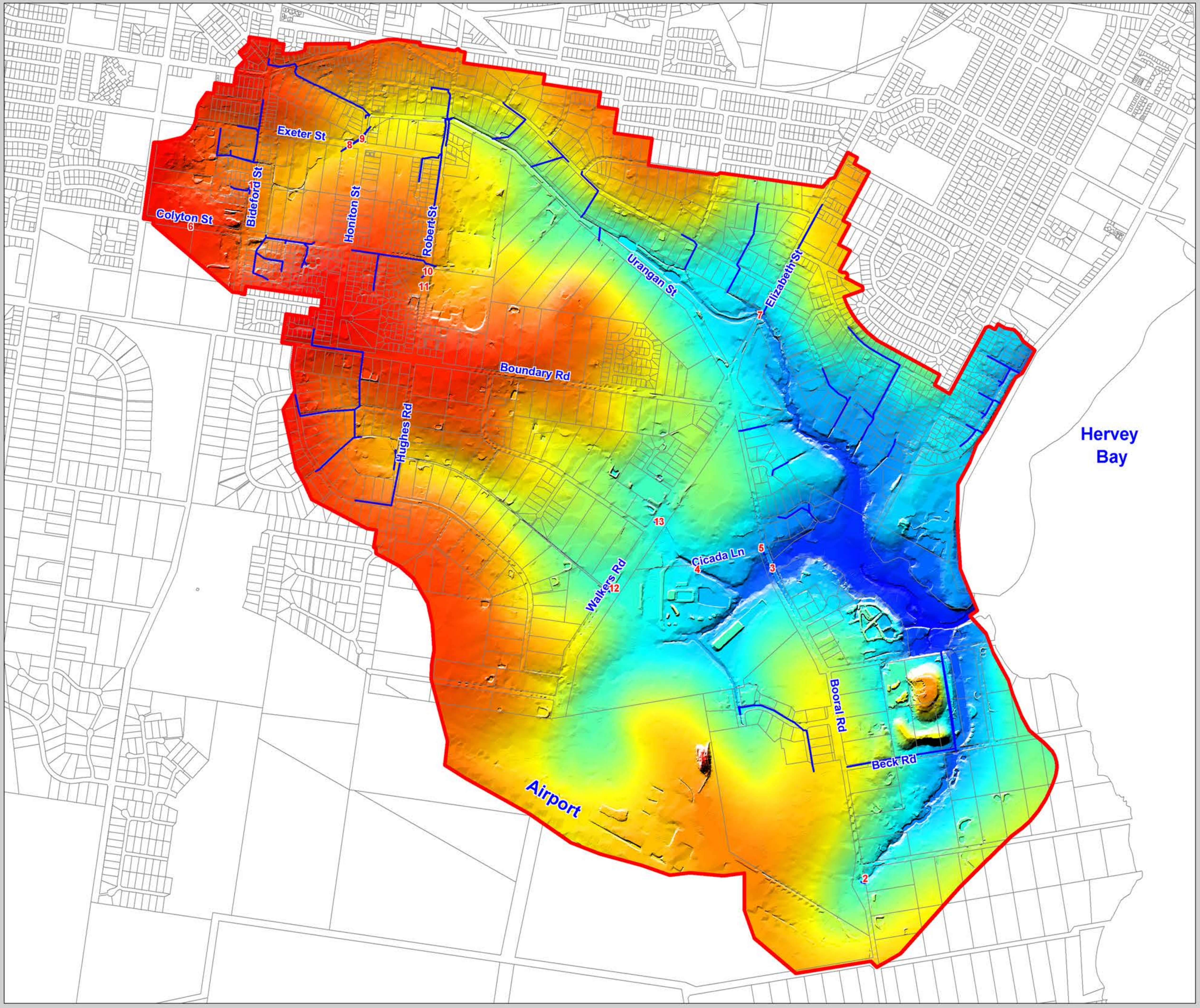
All Structures have been represented using a combination of 1D and 2D domains. Flow through the culverts is modelled using the 1D component of TUFLOW whilst the overland and weir flow, both around the structure and over the road, are modelled purely in the 2D scheme. Pipe networks (drainage) are not included in the description of structures provided above but are shown in Figure 5.1.

5.6 Model Boundary Conditions

Two types of boundaries have been applied to the model. The first, flows derived from the RAFTS model, are applied across the model in various locations representing the sub catchments defined from the hydrologic analysis. Flows have been applied across areas (rather than at point locations). This allows TUFLOW to assign the flow to the lowest cell in each area first and then distribute the flow to wet cells as the flood event progresses.

The second, tail water boundaries have been applied to the downstream edge of the model. For all design events, a Highest Astronomical Tide (HAT) level of 2.12 m AHD has been applied and was determined from review of the supplied Hervey Bay Storm Tide Study and the James Cook University Website. The Hervey Bay Storm Tide Study final report as prepared by Lawson & Treloar Pty Ld, 2002, states *"few historical cyclones have caused significant storm tide levels in the Hervey Bay region during the period of record, since they have been generally less than the HAT."*

A sensitivity analysis of the model boundary condition was required as part of this study and this has been undertaken and discussed separately in Section 6 of this report.



N

LEGEND

2D Model Boundary

Pipes Network

1 Road Crossings

DTM 30 12 00 METRE

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 5.1

HYDRAULIC MODEL LAYOUT

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Projection Trans. Merc. MGA56(GDA 94)

5.7 2D Model Roughness

A GIS land use map covering the entire study area was created for the purposes of defining the hydraulic roughness across the floodplain. Each grid cell is assigned a Manning's 'n' roughness value based upon land use defined on the map. The GIS layer of existing land use was generated using a combination of aerial photography, the HBCC DCDB, and utilising observations as well as oblique photography from the detailed site inspection.

Roughness values for each land use type were assigned based on site observations and using previous experience in 2D hydraulic modelling applications. The Manning's "n" roughness parameters adopted in the model ranged from 0.015 for open water bodies through to 0.75 for residential areas with ineffective flow paths blocked by buildings, fences and other obstructions. These values are typical of those adopted for floodplain roughness for studies of this nature, and are in accordance with those supplied by HBCC in the brief for this study. Table 5-2 documents roughness parameters assigned to each land use.

Table 5-2 Adopted Roughness Parameters

Land Use Type	Manning's "n" Roughness Parameter
Industrial	0.035
Commercial	0.035
Water Body	0.015
Park residential	0.050
Residential low density	0.750
Residential medium density	0.750
Residential high density	0.750
Conservation	0.100
Active open space	0.035
Road	0.025
Rural	0.040
Utilities	0.025
Educational facilities	0.120
creek 1d/2d (open drains)	0.030

5.8 Model Design Runs

The TUFLOW hydraulic model was analysed for the 10, 20, 50 and 100 year ARI design flood events for the critical 60 minute storm event. The results from the analysis for the existing case (current conditions) model are discussed separately in the following section of this report.

6 Existing Case Analysis Results

6.1 Verification of Model to Previous Study Results

A process of verification was undertaken to ensure that the model constructed as part of this assessment was behaving in an appropriate manner. Peak flood levels for the 100 year ARI event with a MHWS downstream boundary were compared at a number of key locations throughout the study area to previous modelling undertaken by GHD in 1996 using RAFTS and the HEC-RAS 1D modelling package. Table 6-1 presents comparisons between flood levels reported in the "Pulgul Creek Catchment Drainage Study" Report, November 1996 and levels obtained using the 2D hydraulic model constructed for use in this study. Figure 6.1 shows the location of the comparison points.

Table 6-1 Comparison of Peak Flood Level, 100 Year ARI

Location	100 Year ARI Flood Levels (m AHD)		
	GHD Pulgul Catchment Drainage Study (1996)	Existing TUFLOW Model	Difference
1	1.55	1.54	-0.01
2	2.27	2.29	0.02
3	2.39	2.43	0.04
4	2.63	2.52	-0.11
5	2.92	2.94	0.02
6	3.26	2.76	-0.50
7	3.47	3.06	-0.41
8	3.72	3.50	-0.22
9	4.16	4.14	-0.02
10	5.16	5.26	0.10
11	6.01	5.99	-0.02

Table 6-1 demonstrates that flood levels obtained using the newly constructed model compare well with the previously reported levels. Some difference is always expected due to the nature of the models used as the previous modelling adopted a one-dimensional approach whilst the current model uses a fully two-dimensional scheme.

JWP consider that the comparison in flood levels between the previous work and that carried out in this study shows satisfactory results. As such, the 2D TUFLOW model has been adopted for the determination of flooding behaviour within the study area.

6.2 Boundary Condition Sensitivity Analysis

A sensitivity analysis has been undertaken on the tidal levels adopted at the downstream boundary of the models.

The sensitivity analysis was undertaken on both the 10 and 100 year ARI events using the HAT tidal level and a ±0.3m variation (RL 2.12 m and RL 2.42 / 1.82m AHD respectively) based on the 60 minute storm event. Figures 6.2 to 6.5 show the extent of impact predicted as a result of varying the ocean boundary condition

The modelling demonstrates that the impacts from varying the tail water levels extend upstream in Pulgul Creek to the Booral Road crossing and to approximately behind Kruger Court.

As can be seen from Figure 6.2 to Figure 6.5, the difference in water levels which result from the variation of the tail water level does not cause impacts a significant distance upstream of the boundary, and does not effect flooding on properties.

N

LEGEND

Study Boundary

Flood Extent

2 Point ID

0.02 Difference (m) from Previous Study (GHD, 1996)

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6-1

STUDY COMPARISON

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Projection Trans. Merc. MGA56(GDA 94)

LEGEND

Study Boundary

Difference (m)

- -1 to -0.5
- -0.5 to -0.20
- -0.2 to -0.15
- -0.15 to -0.1
- -0.1 to -0.05
- -0.05 to -0.02
- -0.02 to 0.02
- 0.02 to 0.05
- 0.05 to 0.1
- 0.1 to 0.15
- 0.15 to 0.2
- 0.2 to 0.5
- 0.5 to 10

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6-2

TAILWATER SENSITIVITY ANALYSIS

10 YEAR ARI 1.82M BOUNDARY

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N

LEGEND

Study Boundary

Difference (m)

- -1 to -0.5
- -0.5 to -0.20
- -0.2 to -0.15
- -0.15 to -0.1
- -0.1 to -0.05
- -0.05 to -0.02
- -0.02 to 0.02
- 0.02 to 0.05
- 0.05 to 0.1
- 0.1 to 0.15
- 0.15 to 0.2
- 0.2 to 0.5
- 0.5 to 10

150 0 150 300 450
Metres
Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6-3

TAILWATER SENSITIVITY ANALYSIS

10 YEAR ARI 2.42M BOUNDARY

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Projection Trans. Merc. MGA56 (GDA 94)

N

LEGEND

Study Boundary

Difference (m)

- -1 to -0.5
- -0.5 to -0.20
- -0.2 to -0.15
- -0.15 to -0.1
- -0.1 to -0.05
- -0.05 to -0.02
- -0.02 to 0.02
- 0.02 to 0.05
- 0.05 to 0.1
- 0.1 to 0.15
- 0.15 to 0.2
- 0.2 to 0.5
- 0.5 to 10

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6-4

TAILWATER SENSITIVITY ANALYSIS

100 YEAR ARI 1.82M BOUNDARY

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Projection Trans. Merc. MGA56(GDA 94)

N

LEGEND

Study Boundary

Difference (m)

- -1 to -0.5
- -0.5 to -0.20
- -0.2 to -0.15
- -0.15 to -0.1
- -0.1 to -0.05
- -0.05 to -0.02
- -0.02 to 0.02
- 0.02 to 0.05
- 0.05 to 0.1
- 0.1 to 0.15
- 0.15 to 0.2
- 0.2 to 0.5
- 0.5 to 10

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6-5

TAILWATER SENSITIVITY ANALYSIS

100 YEAR ARI 2.42M BOUNDARY

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Projection Trans. Merc. MGA56(GDA 94)

6.3 Results

The TUFLOW, 2D model was adopted for the purposes of estimating flood levels and flood inundation throughout the study area under the 10, 20, 50 and 100 year ARI design flood events. These analyses were undertaken using the existing site topography which excluded any proposed flood mitigation works. The results therefore represent the "existing case" model results.

The results of the existing case (ultimate catchment conditions) model are provided in this report. The results provided include the following flood reporting information:

- Flood depths for all events;
- Flood level contours for all events;
- Flood inundation extents;
- Flood levels at key locations (Figure 6.6); and
- Velocity arrows for key areas of interest for the 100 year ARI flood event.

Mapping for all events is presented in Figures 6.7 to 6.18.

6.3.1 Flood Levels

Water levels are calculated at the cell centre and cell sides for all cells within the 2D model, which equates to some 533,000 points within this modelling area. It is therefore not practical to tabulate flood levels for all computation points throughout the model. Flood levels for the 2D scheme are commonly best presented using flood surface and extent maps created in a GIS environment.

For the purposes of this report, a summary table has been generated detailing peak water levels directly upstream of all major road crossings and throughout the waterway between crossings. Figure 6.6 illustrates the locations of the flood reporting locations summarised as part of this study. Peak water levels for each location illustrated in Figure 6.6 are presented in Table 6-2. This table documents a small distribution of flood levels in the area and it should be noted that detailed flood level information is available through the GIS mapping provided to HBCC. The GIS information enables flood level queries to be undertaken at any location within the flow path for all of the events analysed.

Table 6-2 Peak Design Flood Levels (mAHD)

Reporting Location	Design Flood Event (ARI)			
	10	20	50	100
1	2.15	2.16	2.18	2.20
2	2.22	2.26	2.33	2.39
3	2.35	2.43	2.53	2.62
4	2.64	2.74	2.85	2.94
5	4.95	5.05	5.17	5.27
6	6.14	6.25	6.38	6.47
7	7.13	7.22	7.31	7.38
8	8.02	8.13	8.23	8.30
9	9.66	9.70	9.76	9.80
10	10.83	10.87	10.92	10.97
11	11.86	11.91	11.96	11.99
12	12.49	12.53	12.58	12.63
13	13.84	13.85	13.88	13.90
14	16.67	16.68	16.71	16.74
15	3.90	3.95	3.99	4.03
16	8.80	8.82	8.84	8.85
17	3.66	3.83	4.01	4.10
18	8.24	8.24	8.26	8.28
19	12.05	12.06	12.07	12.08
20	4.29	4.38	4.49	4.58
21	6.89	6.94	7.00	7.04
22	8.66	8.68	8.71	8.73
23	11.32	11.37	11.43	11.47
24	14.73	14.76	14.79	14.80
25	9.09	9.12	9.16	9.19
26	13.13	13.14	13.16	13.17
27	7.17	7.26	7.33	7.39
28	-	-	-	17.40

Z:\42-Wways\050300-001_Pulgul_Creek_Flood_Risk\TUFLOW\results\ Flood_Level_Results_010c.xls

N

LEGEND

Study Boundary

Reporting Location ID

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.6

REPORTING LOCATIONS

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6.3.2 Flood Mapping

Figures 6.7 to 6.18 display the results for the design event analysis. A total of twelve (12) flood plans have been prepared as part of this study. The plans are presented to illustrate the 10, 20, 50 and 100 year ARI anticipated extent of flooding, flood depths and flood levels over the study area for the existing case (ultimate development).

The flood inundation mapping prepared as part of this study is subject to the following notations:

1. The flood extent and associated flood data prepared as part of this study is based on available survey data as supplied by Hervey Bay City Council. This includes aerial photogrammetric survey, limited field validation survey and stormwater pipe and pit information. The flood extents and flood results will therefore be subject to the accuracy and detail of the background study information. Drainage conditions may also have changed since the collection of the survey information;

2. A buffer of 0.1m has been applied to the derivation of the flood extent such that, depths less than 0.1m are not shown. This has been done in agreement with Council to remove the local drainage and sheet flow that is outside the scope and detail of the study.

3. All flood extents prepared as part of this study have been prepared based upon the DTM formed for the study area. Where critical information such as open channels have not been adequately represented in the DTM as a result of the original photogrammetric data captured, calculated flood extents may vary from those on the ground . The accuracy of the flood extents prepared from this study is subject to the accuracy of the topographical representation contained within the DEM.

N

LEGEND

Study Boundary

Q10 Flood Extent

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.7

10 YEAR ARI FLOOD EXTENT

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N

LEGEND

Study Boundary

Q20 Flood Extent

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.8

20 YEAR ARI FLOOD EXTENT

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N

LEGEND

Study Boundary

Q50 Flood Extent

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.9

50 YEAR ARI FLOOD EXTENT

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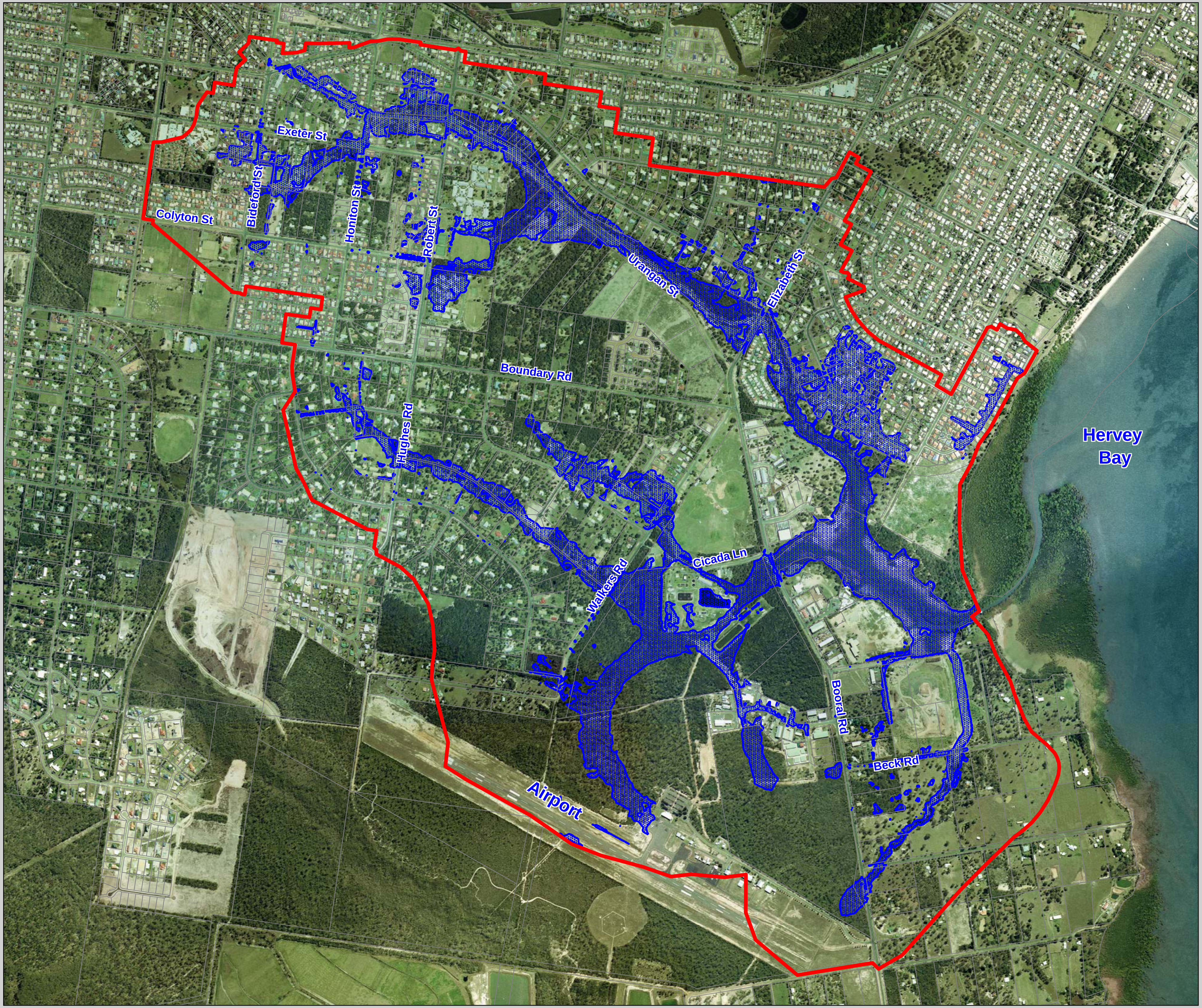
Projection Trans. Merc. MGA56(GDA 94)



N

LEGEND

- Study Boundary
- Q100 Flood Extent

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.10

100 YEAR ARI FLOOD EXTENT

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Projection Trans. Merc. MGA56(GDA 94)

N

LEGEND

Study Boundary

Depth (m)

- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.40
- 0.40 to 0.50
- 0.50 to 1.00
- 1.00 to 1.50
- 1.50 to 2.00
- 2.00 to 2.50
- 2.50 to 3.00

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.11

10 YEAR ARI PEAK FLOOD DEPTHS

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N

LEGEND

Study Boundary

Depth (m)

- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.40
- 0.40 to 0.50
- 0.50 to 1.00
- 1.00 1.50
- 1.50 to 2.00
- 2.00 to 2.50
- 2.50 to 3.00

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.12

20 YEAR ARI PEAK FLOOD DEPTHS

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N

LEGEND

Study Boundary

Depth (m)

- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.40
- 0.40 to 0.50
- 0.50 to 1.00
- 1.00 to 1.50
- 1.50 to 2.00
- 2.00 to 2.50
- 2.50 to 3.00

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.13

50 YEAR ARI PEAK FLOOD DEPTHS

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N

LEGEND

Study Boundary

Depth (m)

- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.40
- 0.40 to 0.50
- 0.50 to 1.00
- 1.00 to 1.50
- 1.50 to 2.00
- 2.00 to 2.50
- 2.50 to 3.00

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.14

100 YEAR ARI PEAK FLOOD DEPTHS

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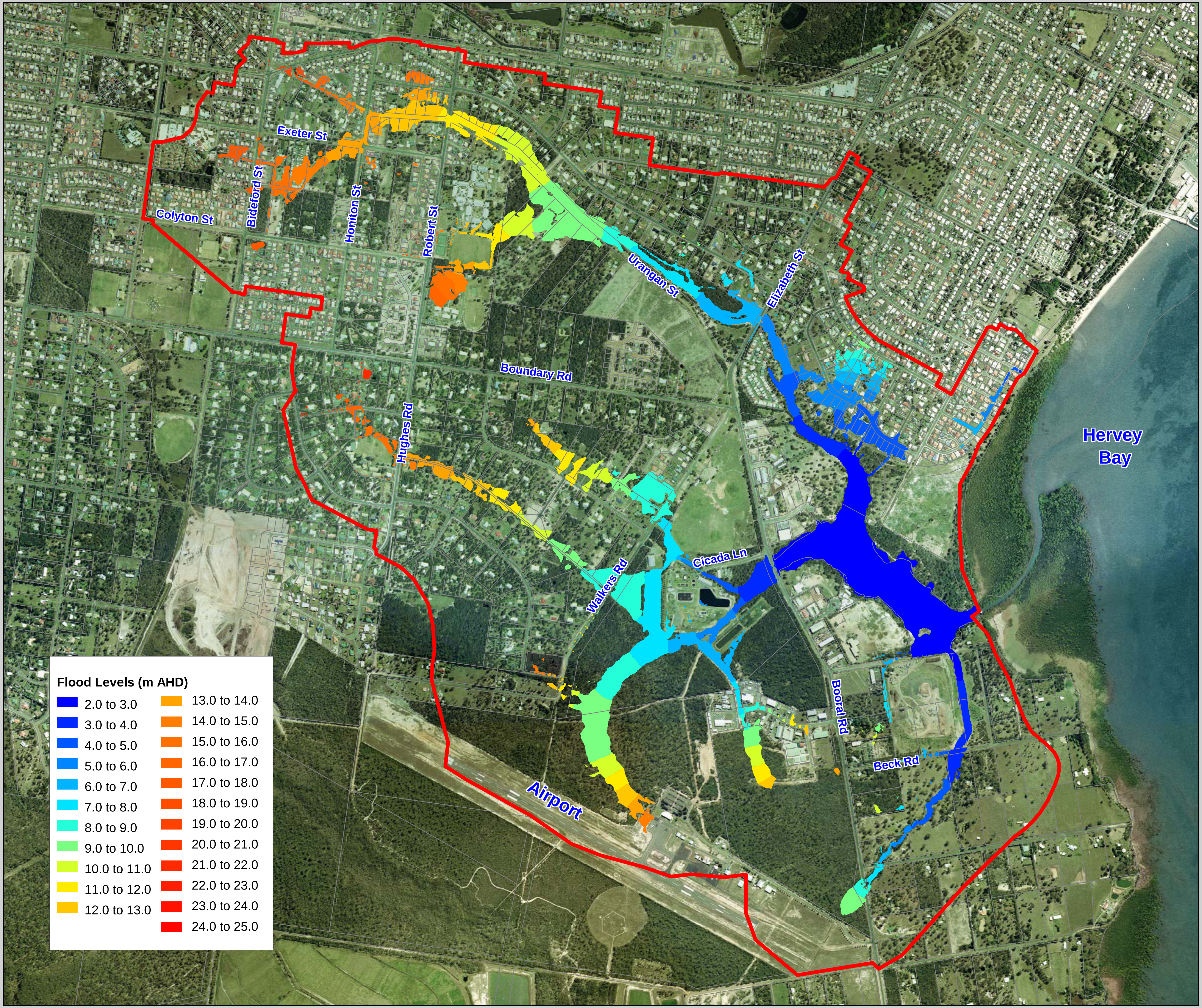

N

LEGEND

Study Boundary

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.15

10 YEAR ARI PEAK FLOOD LEVELS

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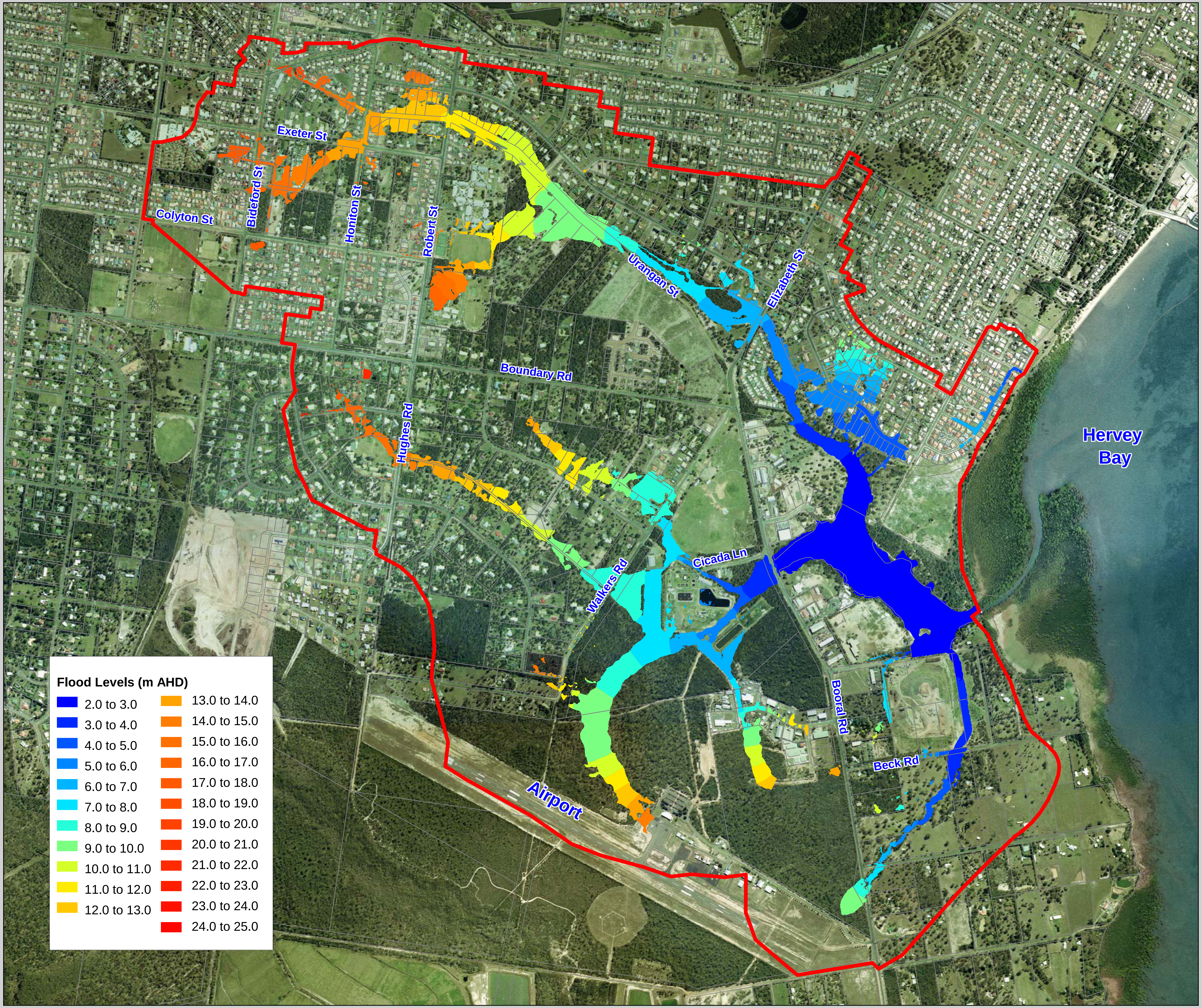

N

LEGEND

Study Boundary

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.16

20 YEAR ARI PEAK FLOOD LEVELS

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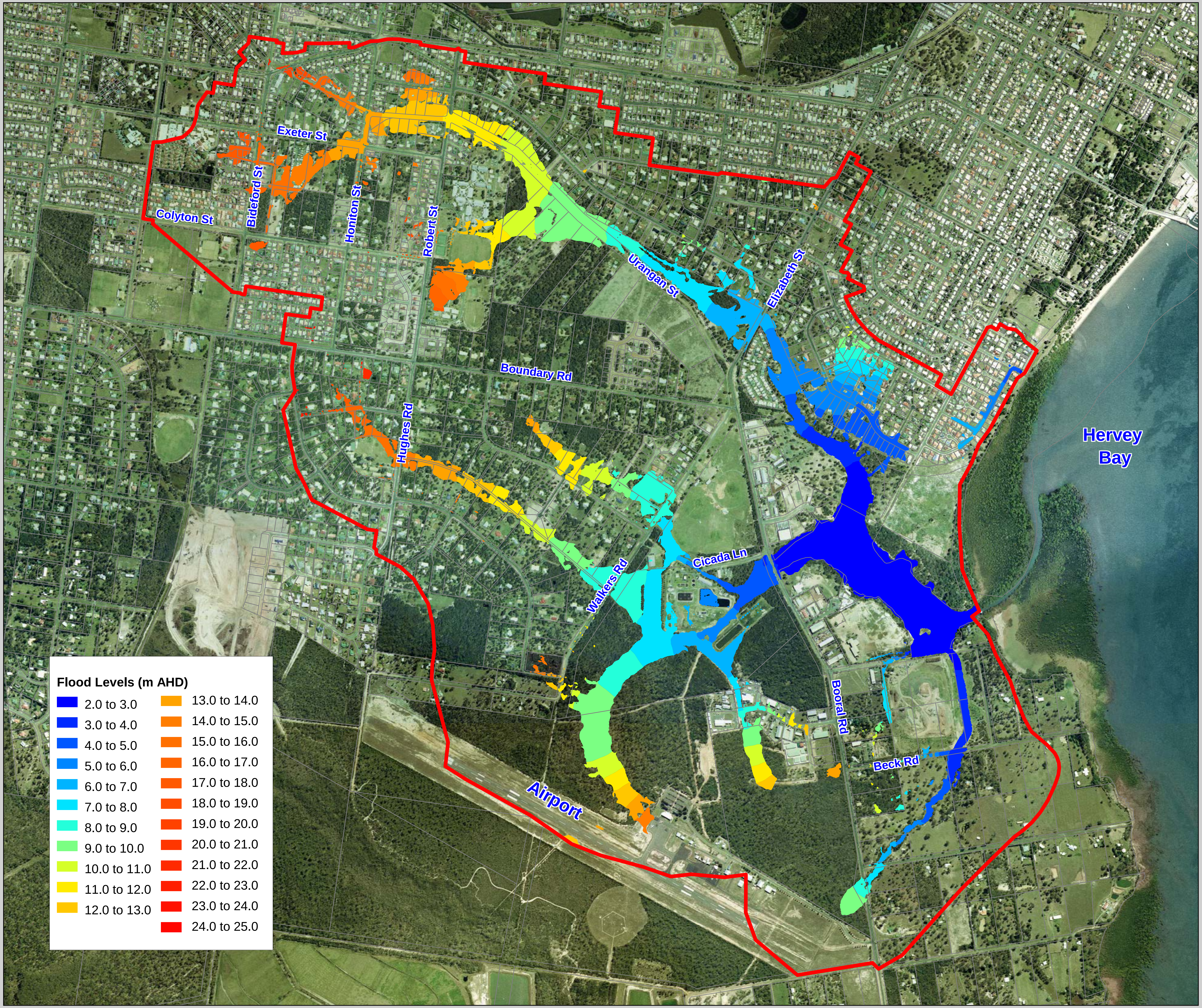

N

LEGEND

Study Boundary

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.17

50 YEAR ARI PEAK FLOOD LEVELS

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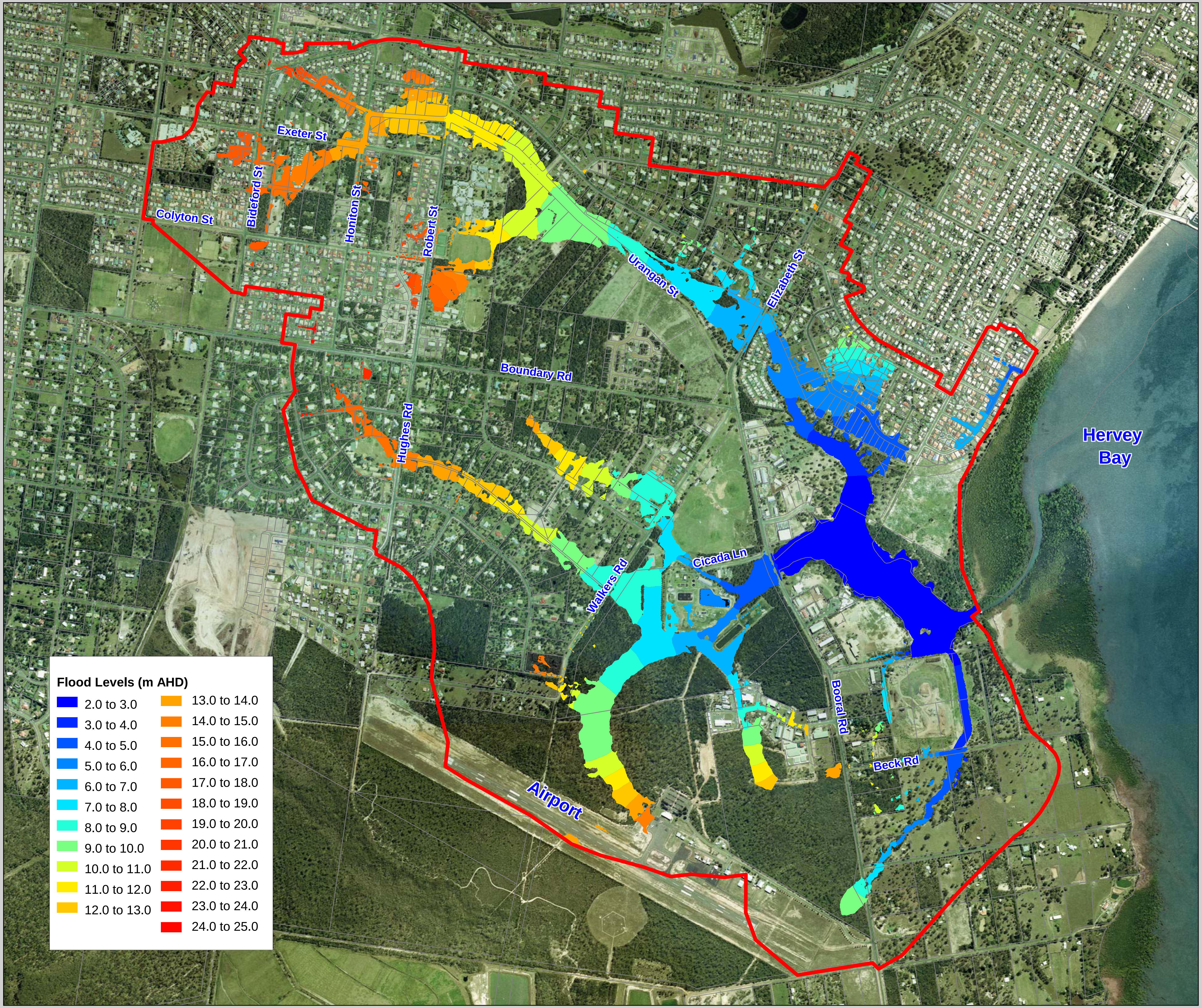
Projection Trans. Merc. MGA56(GDA 94)



N

LEGEND

Study Boundary

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 6.18

100 YEAR ARI PEAK FLOOD LEVELS

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7 Existing Scenario Risk Identification & Prioritisation

7.1 Risk Identification Methodology

In liaison with HBCC a procedure for the evaluation and prioritisation of risks was developed. Risks are evaluated and prioritised using two methodologies, Queensland Urban Drainage Manual (QUDM) and the risk ranking matrix. Identification of overtopping and hazard at road crossings was defined using the QUDM design criteria for roads as shown in Table 7-1 below.

Table 7-1 QUDM design criteria for roads

Criteria	Limit
For Vehicle and Pedestrian Safety	$0.6m^2/s$ ($0.4\ m^2/s$ if the area is known to have high pedestrian usage or has safety issues)
Maximum depth of flow on any Road	300mm

Prioritisation and risk for the identified crossings was evaluated using the risk ranking matrix. The risk ranking matrix considers the likelihood and consequence of the risk occurring and defines a risk ranking for each risk. Table 7-2 and 7-3 provides the classification of likelihood and consequence respectively. Table 7-4 shows the resulting risk ranking derived from the relationship of likelihood and consequence.

Table 7-2 Likelihood parameters

Almost certain	A 99.5% chance of a hazard being exceeded in a 50 year period – a 1 in 10 year event
Likely	Probability of exceedance is greater than 50% in a 50 year period, but less than 99.5% - a 1 in 50 year event
Possible	Probability of exceedance is greater than 20% in a 50 year period, but less than 50% - a 1 in 100 - 200year event
Unlikely	Probability of exceedance is greater than 5% in a 50 year period. but less than 20% - a 1 in 500 year event
Rare	Probability of exceedance is less than 5% in a 50 year period - a 1 in 500 year event

Table 7-3 Consequence parameters (based on 2000 AU$)

Insignificant	Natural hazards are experienced and cause some stress on community lifelines. Community agencies cope with some effort and total community financial loss is less than $1.0m
Minor	No disaster is officially declared and effects lead to temporary failure of lifelines other than energy supply for up to 24 hours. Total community financial loss is less than $10m
Moderate	Disruption lasts for more than 5 days including energy disruption. Recovery takes 14 – 21 days. Vulnerable elements are severely affected and all major agencies are involved. Hospitalisation of victims occurs and total community financial loss is less than $50m. State of emergency is declared during the event.
Major	All lifelines affected. Energy is disrupted for up to 14 days. Recovery takes 4 – 6 weeks. At least one death is suffered and temporary evacuation of area is required. State of Disaster is declared and total community loss is up to $200m.
Catastrophic	Effects are severe and all lifelines are affected. No energy for up to 8 weeks and recovery takes 6 – 24 months. At least 10 deaths suffered and significant evacuation required. Total community financial loss in hundreds of millions.

Table 7-4 Risk Ranking

Return period	Consequence / Likelihood	Insignificant	Minor	Moderate	Major	Catastrophic
10	Almost certain	H	H	E	E	E
50	Likely	M	H	H	E	E
100/200	Possible	L	M	H	E	E
500	Unlikely	L	L	M	H	E
1000	Rare	L	L	M	H	H

Where: E = extreme risk H = high risk M = moderate risk L = low risk

In addition to infrastructure lifelines, risk parameters for people, buildings, economic loss and loss of the natural environment are proposed as shown in Table 7-5.

Table 7-5 Risk Parameters for People, Buildings, Ecomonic Loss and Natural Environment

Risk element	Extreme (unacceptable) risk
People	Vulnerability to natural hazards is generally measured by the risk to life and property from known hazards. An area may be prone to a known hazard, but if there is no possible risk to life or property, the vulnerability is low. Where life and property are at risk, the magnitude and likelihood of the hazard combine to create a measure of vulnerability. **Unacceptable risks are death, serious injury and major health hazard.**
Buildings	The built environment is at risk from a number of known hazards in Hervey Bay. Various regulations have been developed locally (e.g. Local Laws) and at a wider scale (e.g. the Building Code of Australia) to minimise the risk of damage to the built environment. All of these regulations are based on an acceptable level of risk which has been determined either by Council or a wider community of interest (e.g., 1:100 flood immunity). Inevitably there will be extreme events which go beyond the acceptable level of immunity and the only possible way to immunise against these events is avoidance. **Unacceptable risks are collapse or damage to buildings requiring demolition.**
Economic loss	In all disaster events there is bound to be some form of economic loss. The Federal Government under the Natural Disaster Relief Arrangements provides funding to victims of disaster events. This funding is generally short term and designed to minimise immediate suffering and loss. Businesses need to make their own assessment of potential economic loss through a natural disaster event and make plans accordingly. These would range from building construction, to choice of location to insurance. **Unacceptable risks are loss of livelihood for more than 10% of the working community.**
Natural environment	The natural environment is at risk from a number of known hazards in Hervey Bay. **Unacceptable risks are loss of ecological systems, major habitats or conservation areas. Significant disruption to natural drainage systems.**
Risk escalation Risk escalation is likely to happen when initial risk minimisation programs or event response mechanisms do not achieve their intended purpose. The risks outlined in this document may have follow-on or secondary effects (e.g. an earthquake may lead to a dam break, which may	

lead to flooding, which may lead to injury or isolation). **Unacceptable risks arise from the failure of initial risk minimisation and response mechanisms.**
Risk frequency Risks to physical infrastructure are usually incorporated in design parameters (e.g. bridges are designed to withstand certain loads; drains are designed to accommodate mathematically derived flood levels). These are generally based on industry standards of acceptable levels of risk. These standards have until recently had very little legislative basis. The recent adoption of *State Planning 1/03 - Mitigating the adverse impacts of Flood, Bushfire and Landslide* introduces risk frequency levels (e.g., 1:100 years) which are required to be accommodated in planning and design documents (e.g. planning schemes and infrastructure codes). **Unacceptable risks are events which occur within the design capacity of infrastructure or industry accepted measures.**
Legal and social justice implications Risk management is applied by Council across all parts of its jurisdiction in an equal manner and includes all persons. Council is required to make decisions on an annual basis about prioritising its expenditure on various competing items. Expenditure on risk minimisation is incorporated in most capital works projects by way of an in-built design standard. **Unacceptable risks are deliberate inequality of expenditure against any one group, or any one part of the city.**
Political implications Council's decisions are subject to scrutiny and influence from various elements and sectors of the community. It is Council's role to make informed and un-biased decisions. **Unacceptable risks are decisions made which reflect unlawful political bias.**

For the Pulgul Creek Flood Risk Identification Study, specific flood risks were identified through use of the above risk matrix and examination of modelling results as discussed in Chapter 6. Where modelling identified a hazard, an analysis of the various risk elements was undertaken using the risk matrix above. A risk ranking for the hazard was determined based on the likelihood and the consequences of the hazard occurring. For elements such as people, not only the potential to suffer injury or death as a result of property inundation was analysed, but also the ease of egress from the property through the determination of Velocity x Depth products and road overtopping as defined in the (QUDM). These parameters are shown in Table 7-1.

A risk ranking for each specific flooding risk was determined. This risk ranking can be used to prioritise mitigation options within the total catchment and is discussed in more detail in Chapter 8. A description of flooding and risk ranking for key areas across the catchment is provided in the following text. Based on the derived risk ranking and the flooding characteristics of each location the upgrade and immunity requirements are presented in Tables 7-8.

Risk elements were further defined by flood hazard and road overtopping. The following sections provide existing case flooding information for key areas across the catchment. These areas include road overtopping and areas of inundation. Road overtopping has been assessed in accordance with the Queensland Urban Drainage Manual (QUDM), 1992. The following parameters were used in the assessment of road crossings:

7.2 Risk Identification

Risks have been identified based on the QUDM road design guidelines and the risk ranking matrix. Key risk elements are defined in the following sections.

7.2.1 QUDM Classification

As discussed in the previous section, QUDM defines road trafficability based on the depth of inundation and the velocity x depth product across the road. All crossings in the catchment have been assessed against these parameters and the outcomes are shown in Table 7-6.

7.2.2 Risk Ranking Matrix

All crossings and key areas were assessed to define the risk ranking. Table 7-7 shows the risk ranking matrix.

Table 7-6 QUDM Road Trafficability Assessment

Name	Road Type	Q10 Depth	Q20 Depth	Q50 Depth	Q100 Depth	Q10 vxd	Q20 vxd	Q50 vxd	Q100 vxd
Bideford St 1	Major	0.19	0.21	0.22	0.24	0.18	0.21	0.24	0.27
Bideford St 2	Major	0.11	0.12	0.13	0.14	0.03	0.02	0.03	0.02
Boat Harbour Drive	Major	0.09	0.10	0.10	0.11	0.07	0.10	0.10	0.12
Exeter Street 1	Minor	0.28	0.29	0.33	0.34	0.08	0.10	0.12	0.14
Honiton Street	Minor	0.40	0.41	0.47	0.47	0.44	0.39	0.46	0.43
Robert Street	Major	0.31	0.36	0.40	0.44	0.24	0.32	0.38	0.46
Garden Drive	Major	0.19	0.27	0.28	0.33	0.04	0.06	0.10	0.11
Elizabeth Street	Major	0.07	0.26	0.31	0.46	0.01	0.19	0.29	0.52
Booral Road 1	Major	0.15	0.16	0.16	0.18	0.06	0.00	0.06	0.06
Walkers Road 1	Minor	0.19	0.21	0.22	0.25	0.09	0.10	0.13	0.15
Walkers Road 2	Minor	0.10	0.12	0.13	0.15	0.08	0.11	0.16	0.19
Hughes Road	Major	0.09	0.13	0.13	0.17	0.05	0.12	0.13	0.21
Windermere Road	Minor	0.25	0.31	0.32	0.36	0.11	0.16	0.17	0.18
Walkers Road 2	Minor	0.15	0.17	0.17	0.20	0.11	0.13	0.15	0.17
Ellengowan Street	Minor	0.36	0.42	0.43	0.49	0.44	0.52	0.57	0.65
Booral Road 2	Major	0.20	0.23	0.23	0.26	0.18	0.21	0.21	0.23
Island View Drive	Minor	0.20	0.26	0.27	0.33	0.11	0.19	0.21	0.28
Beck Road	Minor	0.40	0.46	0.48	0.54	0.29	0.35	0.30	0.41

Table 7-7 Flood Risk Analysis

Location	Risk Element	Acceptable standard	Currently meets desired risk standard	Likelihood	Consequence	Risk Ranking	Upgrade recommended
Exeter Street Caravan Park	People - drowning	No resultant deaths, injuries or major health hazards	×	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	×	Likely	Minor	High	✓
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Bideford Street 1 - South of Exeter Street (Major Road)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	×	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Bideford Street 1 - Approx. Woodland Close (Major Road)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Boat Harbour Drive (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×

Location	Risk Element	Acceptable standard	Currently meets desired risk standard	Likelihood	Consequence	Risk Ranking	Upgrade recommended
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Honiton Street (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Robert Street/Urangan Street Intersection (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	×	Possible	Insignificant	Low	×
	Buildings	Q100 immunity	×	Likely	Minor	High	✓
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Garden Drive (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×

Location	Risk Element	Acceptable standard	Currently meets desired risk standard	Likelihood	Consequence	Risk Ranking	Upgrade recommended
The Esplanade	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Elizabeth Street (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	×	Likely	Minor	High	✓
	Buildings	Q100 immunity	×	Possible	Minor	Moderate	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Booral Road - Approx. Cicada Lane (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Booral Road - Approx. Island View Drive (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	×	Likely	Insignificant	Moderate	×

Location	Risk Element	Acceptable standard	Currently meets desired risk standard	Likelihood	Consequence	Risk Ranking	Upgrade recommended
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	**Low**	×
	Natural environment	N/A	✓	Unlikely	Insignificant	**Low**	×
Walkers Road Approx. Sunline Cresent (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	**Low**	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	**Low**	×
	Buildings	Q100 immunity	×	Unlikely	Insignificant	**Low**	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	**Low**	×
	Natural environment	N/A	✓	Unlikely	Insignificant	**Low**	×
Walkers Road Downstream Senorita Parade (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	**Low**	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	**Low**	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	**Low**	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	**Low**	×
	Natural environment	N/A	✓	Unlikely	Insignificant	**Low**	×
Hughes Road (Major)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	**Low**	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	**Low**	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	**Low**	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	**Low**	×
	Natural environment	N/A	✓	Unlikely	Insignificant	**Low**	×
Windermere Road (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	**Low**	×

Location	Risk Element	Acceptable standard	Currently meets desired risk standard	Likelihood	Consequence	Risk Ranking	Upgrade recommended
	People - ease of egress	DV Product <0.6 D<300mm	✓	Possible	Minor	Moderate	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Walkers Road Approx Seno Drive (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Ellengowan Street (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×
Island View Drive (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	Low	×
	People - ease of egress	DV Product <0.6 D<300mm	✓	Unlikely	Insignificant	Low	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	Low	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	Low	×
	Natural environment	N/A	✓	Unlikely	Insignificant	Low	×

Location	Risk Element	Acceptable standard	Currently meets desired risk standard	Likelihood	Consequence	Risk Ranking	Upgrade recommended
Beck Road (Minor)	People - drowning	No resultant deaths, injuries or major health hazards	✓	Unlikely	Insignificant	**Low**	×
	People - ease of egress	DV Product <0.6 D<300mm	×	Possible	Insignificant	**Low**	×
	Buildings	Q100 immunity	✓	Unlikely	Insignificant	**Low**	×
	Economic loss	Loss of livelihood for less than 10% of working community	✓	Unlikely	Insignificant	**Low**	×
	Natural environment	N/A	✓	Unlikely	Insignificant	**Low**	×

7.2.3 Key Risk Elements

Modelling of the Pulgul Creek catchment identified a number of key risk elements as outlined in Sections 7.2.4 – 7.2.7 as follows. Based on this preliminary assessment, it was determined whether further analysis of the areas is warranted.

7.2.4 Booral Road

Minor overtopping of Booral Road adjacent to the airport is predicted to occur in flood events larger than the 10 year ARI.

Figure 7.1 shows flood depth contour and velocity vectors for the 100 year ARI flood event. At the peak of the 100 year ARI depths across the road are predicted to be less than 250mm. The velocity x depth product is predicted to be less than 0.3 m^2/s. These values are within the acceptable limits of QUDM.

Discussions with HBCC have determined that a local drainage study for the airport predicted no overtopping of Booral Road. It is expected that this difference has resulted from the definition of the channel in the DTM in this area (for discussion on the DTM see Section 3.1). It is recommended that if this issue is of concern to Council, detailed survey for the area should be collected and included in the 2D hydraulic model.

7.2.5 Elizabeth Street

Overtopping of Elizabeth Street is predicted in the 10 year ARI event and greater. Table 7-8 shows peak depths of inundation across Elizabeth Street for each design event. Figure 7.2 shows flood depth contours and velocity arrows in the 100 year ARI flood event.

Table 7-8 Inundation depths on Elizabeth Street

Design Event	Depth of Inundation (mm)
10	100
20	200
50	330
100	430

Table 7-8 demonstrates that based on QUDM, Elizabeth Street is not trafficable in events greater than and including the 50 year ARI.

The velocity x depth product is predicted to be up to 0.7m^2/s in the 100 year ARI event. At the peak of the 100 year ARI flood event 39.5 m^3/s is predicted the flow across Elizabeth Street. 19.5m^3/s is carried by the existing pipe network (5 x 1800mm diameter pipes), and 20m^3/s flows across the road.

Elizabeth Street is classified as a major road, and forms part on the major north-south connector through the catchment. As such, further assessment of this crossing is warranted.

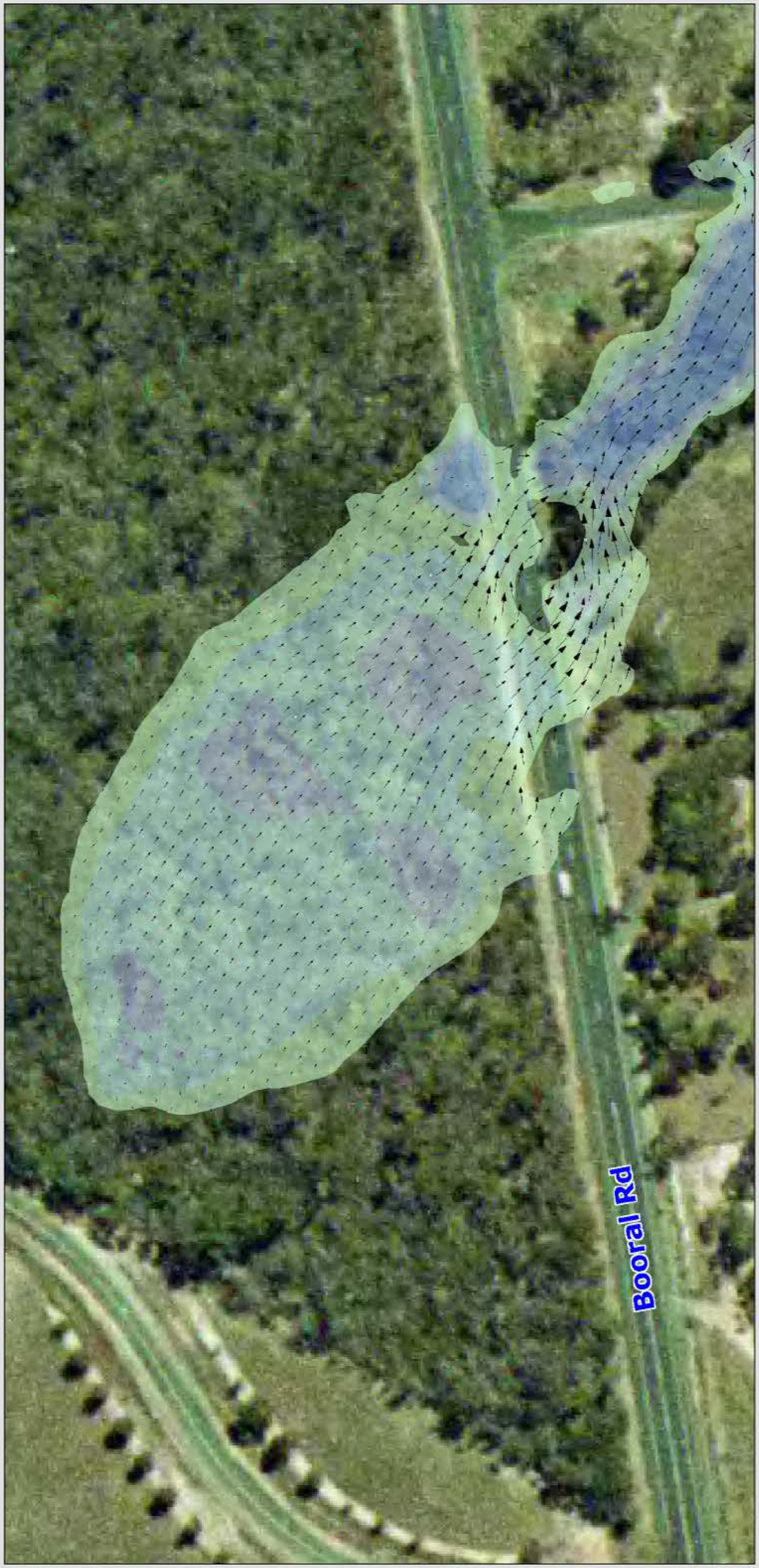

FIGURE 7.1

Booral Road Overtopping 100 Year ARI

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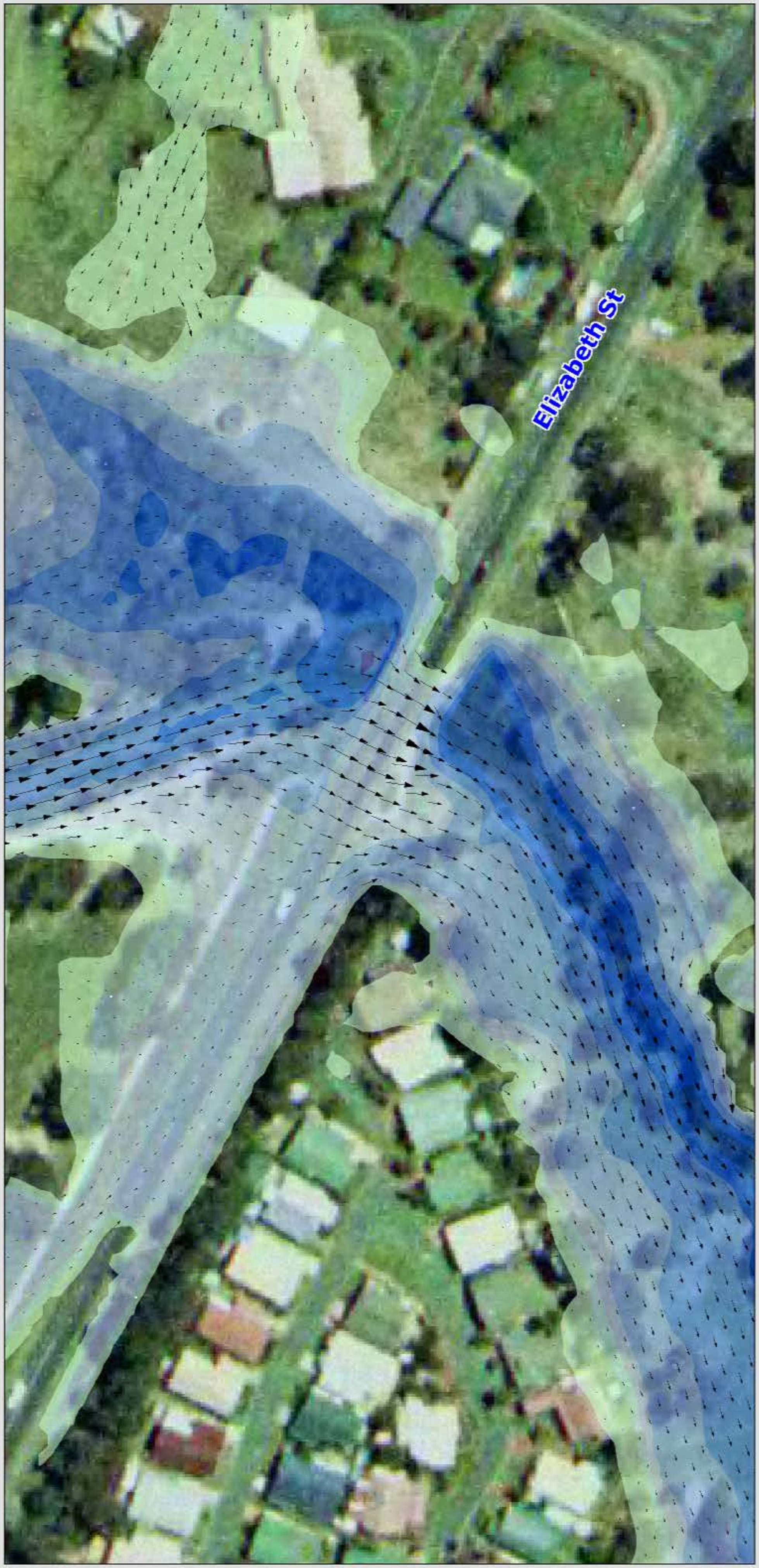
Projection Trans. Merc. MGA56(GDA 94)



N
W E
S

11 0 11 22 33
metres

Scale: 1:1,100

LEGEND

Depth (m)

- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.40
- 0.40 to 0.50
- 0.50 to 1.00
- 1.00 to 1.50
- 1.50 to 2.00
- 2.00 to 2.50
- 2.50 to 3.00
- Velocity m/s

FIGURE 7.2

Elizabeth Street Overtopping 100 Year ARI

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Projection Trans Merc MGA56(GDA 94)

7.2.6 Exeter Street Caravan Park

The modelling predicts that this area is subject to nuisance flooding from the 10 year ARI and larger. Inundation is predicted in the caravan park on Exeter Street as a result of breakouts from the channel at the rear of the property. Average peak inundation depths for the design flood events are shown in Table 7-9. Figure 7.3 shows flood depth contours and velocity arrows in the 100 year ARI flood event.

Table 7-9 Inundation depths in Caravan Park

Design Event	Depth of Inundation (mm)
10	290
20	300
50	320
100	340

In the 100 year ARI flood event velocity x depth product across the site is predicted to be less than $0.2m^2/s$. The modelling has predicted that the hazard from flooding is not high. However due to land use characteristics in the area and the frequency of nuisance flooding, further assessment of this area is warranted.

7.2.7 Urangan Street

Widespread inundation of Urangan Street is predicted in all design events. Figure 7.4 shows a long section profile of Urangan Street from Roberts street to Elizabeth Street for the 10, 20, 50 and 100 year ARI flood event. In the 10 year ARI event flood depths are predicted up to 700mm at the peak of the flood. Flood depths are predicted to be up to 750mm at the peak of the 100 year flood, across Urangan Street. Velocity x depth products of up to $1m^2/s$ are predicted in the 100 year ARI event.

Several factors contribute to the flooding of Urangan Street. These include the size of the drainage channel running parallel to the road, and the height of the road. Of particular interest to HBCC is the Urangan Street and Roberts Street intersection due to its proximity to the School. At this intersection flood depths across Roberts Street are predicted to be 0.33m in the 10 year ARI flood event. Velocity x Depth products of up to $0.4m^2/s$ are predicted to occur in the 10 year ARI flood event. Figure 7.5 illustrates flood depth contours and velocity arrows in the 100 year ARI flood event.

Urangan Street is categorised as a major road in the catchment. Therefore Urangan Street warrants further investigation. Due to the proximity to the school, the Urangan Street/Robert Street intersection is also considered to warrant further investigation.

N
W E
S

14 0 14 28 42
metres

Scale: 1:1,400

LEGEND

Depth (m)

- 0.10 to 0.20
- 0.20 to 0.30
- 0.30 to 0.40
- 0.40 to 0.50
- 0.50 to 1.00
- 1.00 to 1.50
- 1.50 to 2.00
- 2.00 to 2.50
- 2.50 to 3.00
- Velocity m/s

FIGURE 7.3

Exeter Street Carvan Park 100 Year ARI

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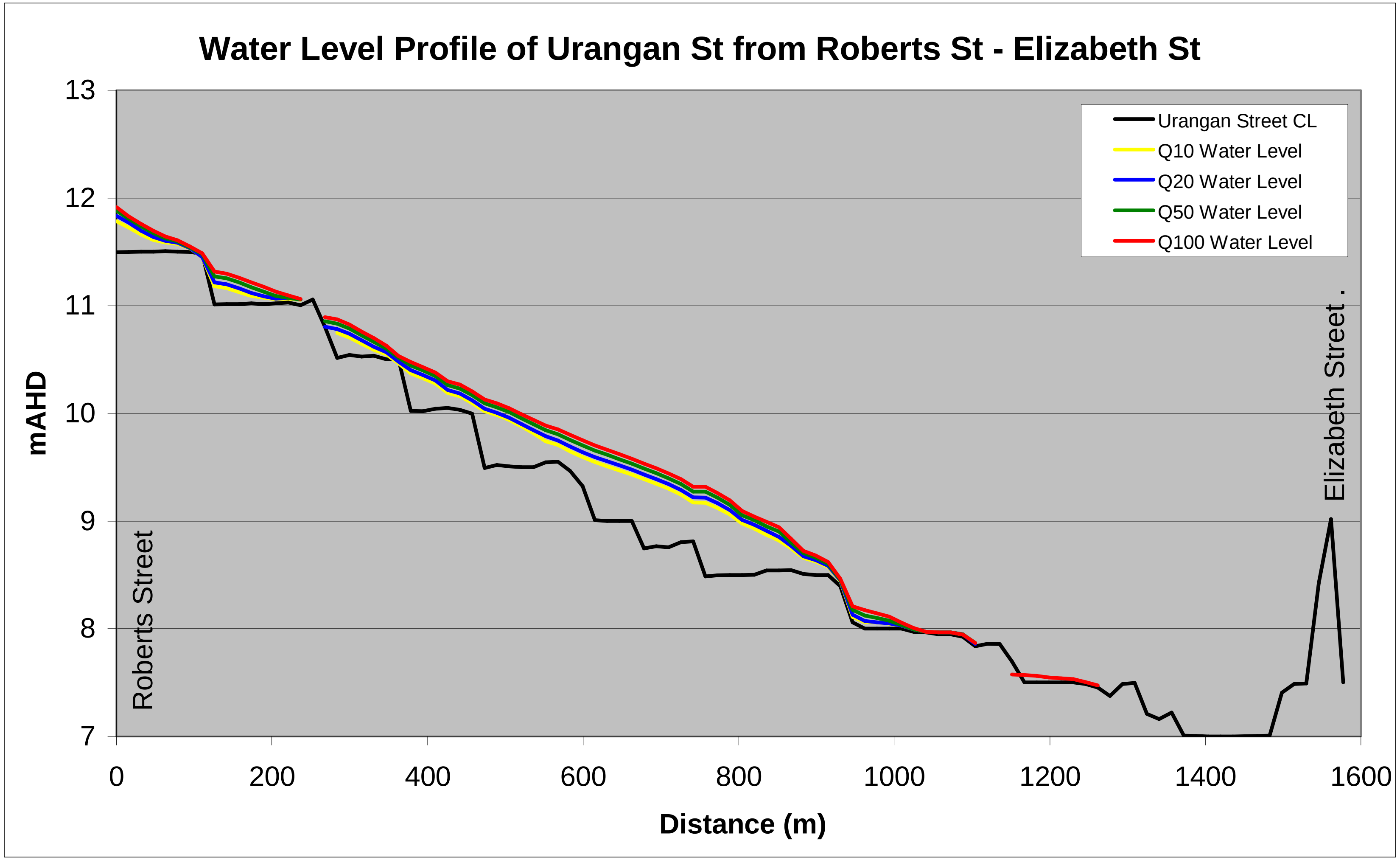
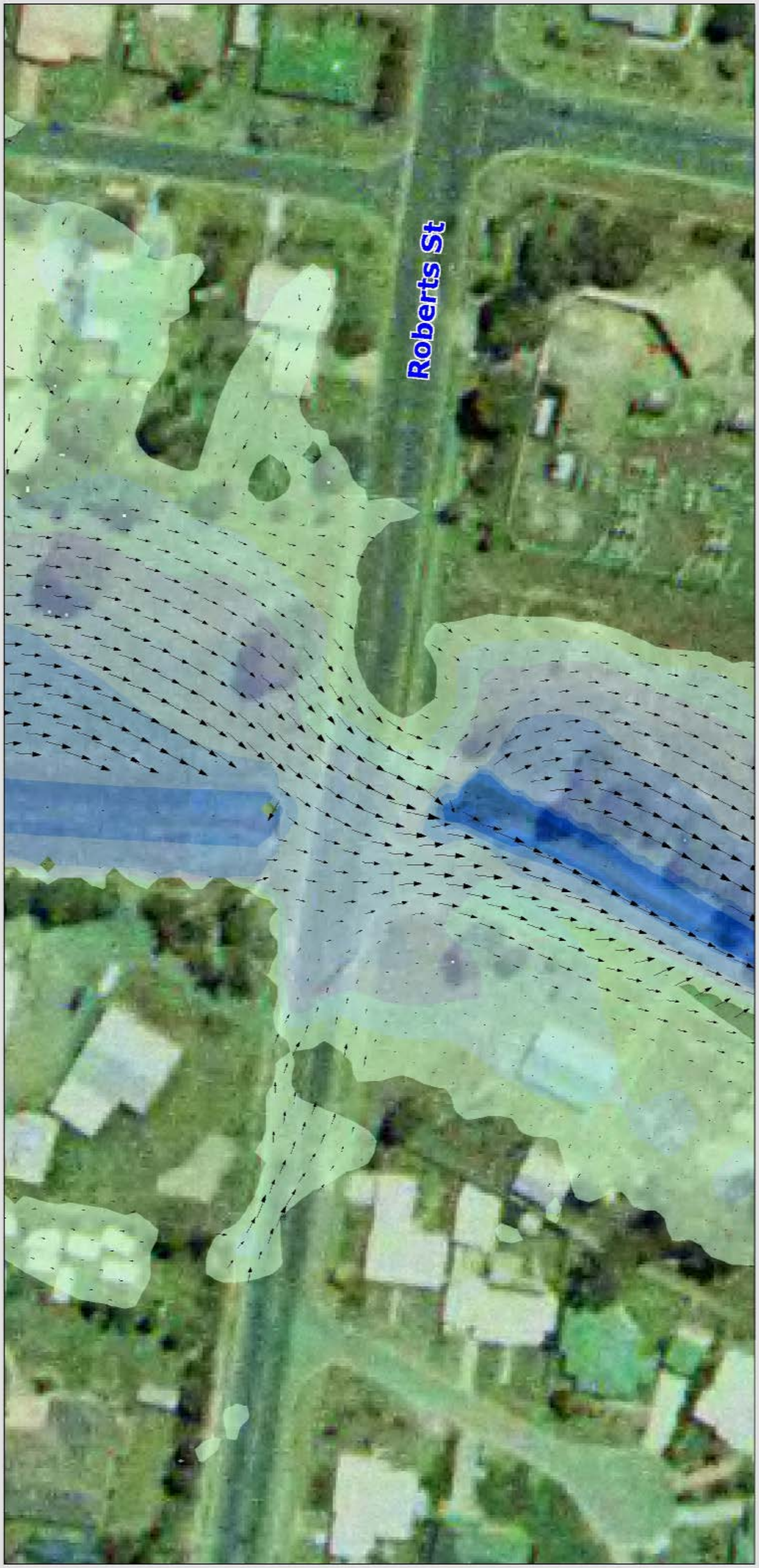
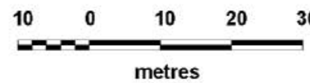
Projection Trans. Merc. MGA56(GDA 94)

Figure 7.4 Long Section Profile of Urangan Street



Urangan St Profile all evetns_010.xls





Scale: 1:1,000

LEGEND

Depth (m)

0.10 to 0.20
0.20 to 0.30
0.30 to 0.40
0.40 to 0.50
0.50 to 1.00
1.00 to 1.50
1.50 to 2.00
2.00 to 2.50
2.50 to 3.00
Velocity m/s

FIGURE 7.5

Roberts Street Overtopping 100 Year ARI

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Projection Trans. Merc. MGA56(GDA 94)

8 Risk Treatment and Flooding Mitigation

Treatment of flooding risks in each model as identified in Chapter 7 of this report has been investigated and is summarised below. Specifically, flooding areas that were identified as high risk were mitigated by means of drainage augmentation or other forms of mitigation works with the aim of an overall reduction of the flooding risk. Where flow depths were identified as failing to meet Council design guidelines, mitigation options have been suggested to alleviate flooding depths and ensure compliance to Council design requirements (QUDM).

Mitigation options have only been considered where necessary and to provide a beneficial outcome in terms of reducing flooding and flood risks. At this point, limited consideration has been given to the likely cost implications associated with these options. Figure 8.1 illustrates the locations of the drainage mitigation options investigated as part of this study. A brief description of each of these options is provided below. Detailed design measures are not included and are outside the scope of works of this study.

8.1 Elizabeth Street

To meet QUDM design requirements Elizabeth Street should be trafficable in a 50 year ARI event. HBCC have advised that they require an option that will satisfy Elizabeth Street being trafficable (i.e., flood depth less than 300mm and V x D product < 0.6m2/s) in a 100 year ARI event. An analysis has been carried out to determine the number of additional culverts required to convey flow beneath Elizabeth Street in the 100 year ARI event. Preliminary analysis indicates that two additional 1950mm diameter culverts will be required to carry the flow. In addition to the culverts the road was raised by 400mm at the lowest point. The modelling demonstrates that these mitigation works provide road immunity in the 100 year ARI event with peak depths less than 130mm and velocity depth products less than 0.1m2/s. Preliminary cost estimates for the mitigation works are $45,000.00.

8.2 Exeter Street Caravan Park

This mitigation option was investigated to provide a reduction to the inundation of the Caravan Park and neighbouring properties. Several options were investigated in isolation and together to achieve a solution. The final option assessed comprised of channel improvements including channel excavation, widening and formalising of the existing channel. To assess the impact of increasing the channel conveyance, the channel was lowered by approximately 1m widened to 12m and formalised behind the properties along Exeter Street. The modelling predicted that this mitigation option did lower levels slightly in the caravan park however, would not be successful in alleviating flooding problems in the caravan park.

While extensive analysis was carried out for this area a solution was not achieved. Significant factors resulting in the flooding of the caravan park appear to be the placement of the bund (existing) and the definition of the ground surface in the DTM. Neither of these are well defined in the existing available topography. To find a solution in this area a more detailed study would be required with detailed survey of the area is recommended. This option was not costed as further consideration is required to achieve a solution.

8.2.1 Urangan Street/Roberts Street Intersection

Robert Street is classified as major road. Therefore, the road should be trafficable in a 50 year ARI event. Trafficability in the 50 year was investigated for this crossing with a culvert upgrade scenario. An analysis has been carried out to determine the number of additional culverts required to convey flow beneath Robert Street. Preliminary analysis indicated that an additional culvert (2.1m x 1.5m) combined with road raising of approximately 500mm would be required to carry the flow and improve road immunity. Modelling of this scenario predicted that this option would result in the road being trafficable in a 50 year ARI with depths across the road of approximately 200mm. Preliminary cost estimates for the mitigation works are $60,000.00.

8.2.2 Urangan Street

Urangan Street, classified as a major road suffers from flooding from minor events and greater. Upgrading of the channel running parallel to Urangan Street has been investigated as an option to reduce flooding issues on Urangan Street. For this assessment, the existing channel was replaced in the model with a 1.5m deep, 10m wide channel. Based on the DTM and aerial photos, a 10m wide channel would fit within the existing road reserve.

The modelling predicted that this option alone was not sufficient to significantly improve flooding problems along Urangan Street. The DTM representation of Urangan Street is very coarse and as such, it was not considered feasible to continue assessing improvement options based on this data. Detailed survey of Urangan Street is required for this option to be assessed in more detail. This option was not costed as further consideration is required to achieve a solution.

Based on the preliminary assessment of Uranagn Street flooding, it is expected that raising the height of Urangan Street in conjunction with channel improvements (including assessment of an addition channel on the opposite side of Urangan Street) would be required to improve flooding along Urangan Street.

N

LEGEND

- Study Boundary
- Channel Upgrade
- Culvert Upgrade

150 0 150 300 450

Metres

Scale: 1:15,000

PULGUL CREEK FLOOD RISK REDUCTION STUDY

FIGURE 8.1

UPGRADE OPTION LOCATIONS

HERVEY BAY CITY COUNCIL

"Success through Partnership"

JWP

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Projection Trans. Merc. MGA56(GDA 94)

8.3 Risk Treatment Summary

Generally, a vast majority of the flooding / inundation experienced throughout the Pulgul Creek catchment is largely nuisance flooding i.e., Exeter Street caravan park. However, this study has identified several areas in the catchment where the flooding risk (from road overtopping) is considered high. Mitigation options have been assessed to lower the risk. Culvert upgrade options for Elizabeth Street and Robert Street are predicted to reduce the flood risk and provide trafficability in the 50 year ARI event for Roberts Street and the 100 year ARI for Elizabeth Street.

Channel upgrade options were assessed for the Exeter Street Caravan Park and Urangan Street. A number of achievable options were assessed. However, these options showed little benefit for the cost to construct. It is recommended that further analysis be carried out in these areas, utilising detailed ground survey.

9 Conclusions

This study has been successful in quantifying key risk areas and providing mitigation options in the the Pulgul Creek Catchment for the primary purposes of reducing existing flood risks in the area. Specifically, the works completed have included:

- The assessment and identification of existing drainage capacities, flow paths and flood information for the 1 in 10, 20, 50 and 100 year ARI design flood events;
- Preparation of detailed flood data outputs to fully document the outcomes from the analysis works including flood summary data and flood extent plans;
- A sensitivity analysis on the starting tail water level from the catchment including the analysis of a HAT ± 0.3m as requested in the project brief.
- Identification of potential drainage augmentation options for the catchment;
- Formal hydrological and hydraulic assessment of the agreed drainage augmentation options for the catchment including the preparation of detailed outputs to fully document the outcomes from the mitigation works;
- Identification of a preferred augmentation options for the catchment which has be shown to provide a beneficial outcome for the study in terms of lowering flood levels, reducing flood inundation and consequently flood risk;
- Preparation of preliminary establishment cost estimates for the preferred work options;
- Assessment of flood risk and the preparation of flood risk summaries; and
- Preparation of summary tables, models, flood extents, GIS mapping to formally document the outcomes of the study.

JWP recommends that Council utilises the outcomes from this Flood Risk Assessment Study for the Pulgul Creek catchment in the management of existing and future development within the catchment in terms of reducing flood risk to an acceptable and manageable standard. In addition, it is also recommended that further works be instigated to proceed with the detailed design of the preferred mitigation works such that flood risks throughout the catchments can be significantly reduced. This would also include programming these works and securing future allocations under Council's Capital Works Program or alternatively through other funding arrangements.

10 References

1. The Queensland Urban Drainage Manual (QUDM);
2. Australian Rainfall and Runoff (AR&R - 2001 edition);
3. James Cook University – Storm Surge Water Level Return Period website : http://mmu.jcu.edu.au/water_level_return_periods/SEQ/hervey_bay/PointVernonWest.html
4. Pulgul Creek Catchment Drainage Study, GHD, 1996

11 Qualification

1. In preparing the report and estimate of costs JWP has exercised the degree of skill and care and diligence normally exercised by members of the engineering profession and has acted in accordance with accepted practices of engineering design principles.

2. JWP has used all reasonable endeavours to inform itself of the parameters and requirements of the project and has taken all reasonable steps to ensure that the report and costs estimate is as accurate and comprehensive as possible given the information upon which it is based.

3. It is not intended that this report and costs estimate represent a final assessment of the feasibility of the project.

4. JWP reserves the right to review and amend all calculations, cost estimates and/or opinions included or referred to in the report if:

   (a) additional sources of information not presently available (for whatever reason) are provided or become known to JWP; or

   (b) JWP considers it prudent to revise the estimate in light of any information which becomes known to it after the date of submission.

5. JWP does not give any warranty nor accept any liability in relation to the completeness or accuracy of the report and cost estimate.

6. If any warranty would be implied whether by law, custom or otherwise, that warranty is to the full extent permitted by law excluded.

7. All limitations of liability shall apply for the benefit of the employees, agents and representatives of JWP to the same extent that they apply for the benefit of JWP.

8. This report and cost estimate is for the use of the party to whom it is addressed and for no other persons. No responsibility is accepted to any third party for the whole or part of the contents of this report and cost estimate.

9. If any claim or demand is made by any person against JWP on the basis of detriment sustained or alleged to have been sustained as a result of reliance upon the report and cost estimate or information therein, JWP will rely upon this provision as a defence to any such claim or demand.

APPENDIX A

Rainfall IFD Table

Rainfall Intensity Frequency Duration data for; Hervey Bay Qld
Geographic
Location: 22 Deg. South 153 Deg. East

AUSIFD Version 2.0 10-Feb 2006

Duration (mins)	1 Year ARI (mm/hour)	2 Year ARI (mm/hour)	5 Year ARI (mm/hour)	10 Year ARI (mm/hour)	20 Year ARI (mm/hour)	50 Year ARI (mm/hour)	100 Year ARI (mm/hour)
5	115	148	186	209	239	280	311
5.5	112	143	180	202	232	270	300
6	108	139	175	196	224	262	291
6.5	105	135	170	190	218	254	282
7	102	131	165	185	212	247	275
7.5	100	128	161	180	206	241	267
8	97	125	157	176	201	235	261
8.5	95	122	153	171	196	229	254
9	93	119	150	168	192	224	248
9.5	91	116	146	164	188	219	243
10	89	114	143	160	184	214	238
11	85	109	138	154	176	206	228
12	82	105	132	148	170	198	220
13	80	102	128	143	164	191	212
14	77	98	124	138	158	185	205
15	75	95	120	134	153	179	198
16	72	93	116	130	149	174	193
17	70	90	113	126	145	169	187
18	69	88	110	123	141	164	182
19	67	85	107	120	137	160	177
20	65	83	105	117	134	156	173
21	64	81	102	114	131	152	169
22	62	80	100	112	128	149	165
23	61	78	98	109	125	145	161
24	60	76	95	107	122	142	158
25	58	75	94	105	120	139	154
26	57	73	92	102	117	136	151
27	56	72	90	100	115	134	148
28	55	70	88	99	113	131	145
29	54	69	87	97	111	129	143
30	53	68	85	95	109	127	140
32	51	66	82	92	105	122	135
34	49.8	64	80	89	102	118	131
36	48.3	62	77	86	99	115	127
38	46.9	60	75	84	96	111	123
40	45.6	58	73	81	93	108	120
45	42.8	55	68	76	87	101	112
50	40.4	52	64	72	82	96	106
55	38.3	48.9	61	68	78	90	100
60	36.5	46.5	58	65	74	86	95
75	31.7	40.5	51	57	65	75	83
90	28.2	36	45.2	51	58	67	75
105	25.5	32.6	41	45.9	53	61	68
120	23.4	29.9	37.7	42.2	48.3	56	63
135	21.6	27.7	34.9	39.1	44.9	52	58
150	20.2	25.9	32.7	36.6	42	49	54
165	19	24.3	30.7	34.5	39.5	46.2	51
180	17.9	23	29	32.6	37.4	43.8	48.6

195	17	21.8	27.6	31	35.6	41.6	46.3
210	16.2	20.8	26.3	29.5	33.9	39.7	44.2
225	15.5	19.9	25.2	28.3	32.5	38	42.3
240	14.8	19	24.1	27.1	31.2	36.5	40.6
270	13.7	17.6	22.4	25.2	28.9	33.9	37.8
300	12.8	16.4	20.9	23.5	27.1	31.7	35.4
360	11.4	14.6	18.6	21	24.1	28.3	31.6
420	10.3	13.2	16.8	19	21.9	25.7	28.7
480	9.4	12.1	15.5	17.4	20.1	23.7	26.4
540	8.7	11.2	14.3	16.2	18.7	22	24.5
600	8.12	10.5	13.4	15.1	17.5	20.6	23
660	7.63	9.83	12.6	14.3	16.5	19.4	21.7
720	7.21	9.29	11.9	13.5	15.6	18.4	20.5
840	6.52	8.43	10.9	12.4	14.4	17	19.1
960	5.97	7.75	10.1	11.5	13.4	15.9	17.9
1080	5.53	7.19	9.44	10.8	12.6	15	16.9
1200	5.15	6.72	8.88	10.2	11.9	14.3	16.1
1320	4.84	6.32	8.4	9.68	11.4	13.6	15.4
1440	4.57	5.98	7.98	9.22	10.8	13	14.8
1800	3.93	5.17	6.99	8.13	9.62	11.6	13.2
2160	3.46	4.57	6.25	7.32	8.7	10.6	12.1
2520	3.11	4.12	5.68	6.69	7.98	9.76	11.2
2880	2.82	3.75	5.22	6.17	7.39	9.08	10.4
3240	2.58	3.45	4.83	5.74	6.89	8.5	9.79
3600	2.39	3.19	4.5	5.37	6.47	8.01	9.24
3960	2.22	2.97	4.22	5.04	6.1	7.57	8.76
4320	2.07	2.78	3.97	4.76	5.77	7.19	8.34

APPENDIX B

XP-RAFTS Results

NODE	Maximum Flow	Critical Duration	10 Years 10 mins	10 Years 15 mins	10 Years 30 mins	10 Years 45 mins	10 Years 60 mins	10 Years 90 mins	10 Years 120 mins	10 Years 180 mins	10 Years 270 mins	10 Years 360 mins
PC01	4.96	60 mins	4.55	4.85	4.89	4.04	4.96	4.56	4.11	3.41	3.08	2.52
PC02	16.80	15 mins	15.13	16.80	16.20	14.09	16.03	14.12	13.18	9.96	8.83	7.67
D_1	21.65	15 mins	19.28	21.65	21.09	18.13	20.99	18.68	17.28	13.37	11.90	10.18
PC03	8.32	60 mins	7.43	8.20	8.21	6.87	8.32	7.54	6.89	5.82	5.20	4.42
D_2	26.82	30 mins	23.42	26.77	26.82	23.01	25.99	23.39	21.37	18.55	16.44	14.24
PC05	12.16	15 mins	11.01	12.16	11.98	10.13	12.04	10.96	9.96	8.26	7.31	6.57
PC04	13.82	60 mins	12.25	13.60	13.57	11.34	13.82	12.54	11.48	9.92	8.70	8.17
D_3	42.25	60 mins	29.73	35.38	42.15	36.34	42.25	37.07	35.16	33.89	30.17	27.48
PC06	23.80	15 mins	21.61	23.80	23.37	19.96	23.40	20.90	19.31	15.81	13.80	12.67
D_4	49.74	60 mins	25.23	33.17	45.73	46.72	49.74	46.81	44.22	40.65	36.91	36.37
PC07	15.65	15 mins	14.04	15.65	15.50	13.02	15.64	14.08	12.94	10.84	9.58	8.53
D_5	53.70	60 mins	22.99	32.45	45.35	48.43	53.70	51.61	49.29	44.68	41.04	37.11
PC13	21.39	15 mins	19.18	21.39	20.61	17.89	20.49	18.14	16.85	12.79	11.29	9.99
PC10	8.92	60 mins	7.88	8.75	8.80	7.32	8.92	8.20	7.38	6.59	5.79	5.57
PC11	5.34	60 mins	4.56	5.12	5.20	4.28	5.34	4.92	4.45	4.45	4.22	4.19
D_6	10.52	60 mins	6.69	7.91	9.98	9.76	10.52	10.39	9.41	9.24	8.22	8.02
D_7	29.68	60 mins	23.54	26.63	27.57	22.71	29.68	26.67	24.56	20.69	18.53	17.83
PC12	14.68	15 mins	13.46	14.68	14.36	12.29	14.28	12.82	11.76	9.43	8.34	7.55
PC14	22.48	15 mins	20.60	22.48	21.64	18.85	21.38	18.83	17.53	12.66	11.36	8.99
D_8	62.95	60 mins	52.99	62.53	61.66	52.26	62.95	55.58	50.99	41.71	37.53	33.89
PC15	15.87	15 mins	14.60	15.87	15.40	13.32	15.23	13.51	12.57	9.60	8.55	7.21
PC20	27.64	15 mins	24.86	27.64	26.43	23.13	25.92	22.65	21.20	15.42	13.68	11.61
D_9	71.51	60 mins	41.48	50.16	65.92	64.84	71.51	64.97	59.67	52.98	49.65	47.06
PC08	7.71	15 mins	7.03	7.71	7.48	6.47	7.46	6.61	6.15	4.79	4.21	3.75
D_10	110.96	60 mins	51.29	64.74	95.84	99.44	110.96	109.07	101.19	91.23	85.60	81.29
Out	110.96	60 mins	51.29	64.74	95.84	99.44	110.96	109.07	101.19	91.23	85.60	81.29