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Gladstone City Council

Auckland Creek Flood Study

Report

August 2006

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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Abbreviations

1D	One-dimensional
2D	Two-dimensional
ABS	Australian Bureau of Statistics
AEP	Average Exceedence Probability
AGSO	Geoscience Australia
AHD	Australian Height Datum
ARF	Areal Reduction Factors
ARI	Average Recurrence Interval
BoM	Bureau of Meteorology
Ch	Chainage
DAF	Decay Amplitude Factor
DEM	Digital Elevation Model
DES	Department of Emergency Services
DFE	Design Flood Event
DNRM	Department of Natural Resources and Mines
DoT	Department of Transport (Maritime Division)
EAF	Elevation Adjustment Factor
EPW	Extreme Precipitable Water
GCC	Gladstone City Council
GIS	Geographic Information System
GSDM	Generalised Short Duration Method
GTSMR	Generalised Tropical Storm Method Revised
HAT	Highest Astronomical Tide
IFD	Intensity-Frequency-Duration
LAT	Lowest Astronomical Tide
LGA	Local Government Area
MAF	Moisture Adjustment Factor
MHWS	Mean High Water Springs
MLWS	Mean Low Water Springs
MSL	Mean Sea Level
PAR	Population At Risk
PIP	Priority Infrastructure Plan
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
TAF	Topographical Adjustment Factor

tc	Time of concentration
v.d	Velocity depth product

Executive Summary

Purpose

The Auckland Creek Flood Risk Study has been prepared to provide an overview of flooding across the Auckland Creek catchment and waterways, including a number of tributary systems. In keeping with this type of study, the emphasis has been to determine the extent of flooding for a range of design events, along with an assessment of people and property at risk for each of the considered events.

The results of the study provide a planning and risk management tool in the form of flood inundation and hazard maps. The use of these will minimise the future exposure of residents, property and key infrastructure to flood hazard.

Data, Property and Key Infrastructure

Little data pertaining to historic flood levels was available for the Auckland Creek system. Instead, a combination of standard flood assessment methodology, catchment data and previous studies has been relied upon. Catchment data exists in the form of land use maps, strategic plans, topography and aerial photographs. This was supplemented by cross-section survey, bathymetric data of the lower (tidal) reaches, details of culverts and bridges, and physical inspection of the catchment's waterways.

The lack of flood data has been addressed through the installation of two flood gauges along Auckland Creek / Police Creek. These are located at the Haddock Drive crossing of Police Creek and directly upstream of the Dawson Highway Bridge crossing of Auckland Creek.

Previous Studies

Seven previous studies have been referred to. Typically, each of these focussed on a smaller area than the current study, and hence comparisons of peak flow and level estimates were not always practical. Where comparisons have been made, differences are evident. These are attributable to factors such as the use of different design rainfall data, techniques, and model parameters (in particular, assumed rainfall losses were higher in previous studies). In the absence of a sound data set for model calibration, it was not possible to defend the selection of high loss rates for the current study.

Hydrology

Hydrologic modelling has been undertaken using the URBS software. A total of five flood events have been modelled, consisting of the 20, 50, 100, 500 year and PMP (probable maximum precipitation) events. Each of these events was assessed for existing and ultimate catchment development scenarios.

A key finding of the study is that at many locations, peak flow estimates for the ultimate development scenario show little increase in comparison to those for existing conditions. This appears to be a function of several factors, including the steepness of the upper catchment, and the extent of existing development in many of the sub-catchments.

Flood Levels

Flood flows within Auckland Creek and its tributaries tend to exhibit one-dimensional (1D) characteristics within most of the waterways, and two-dimensional (2D) characteristics across many of the floodplains, particularly towards the downstream end of the study area. In keeping with this, flood levels have been predicted using the hydrodynamic TUFLOW software package, rather than the originally proposed HECRAS software.

The model has been established using topographic and cross section data, with inflows generated using the URBS hydrologic model. All key structures (culverts, bridges, weirs and basins) have been represented in the model.

Flood levels have been predicted for each of the design events nominated above, with the PMP rainfall used to generate the probable maximum flood (PMF). Water levels at the downstream boundary of the model (tailwater levels) have been defined on the basis of extreme tides. These include the highest astronomical tide (HAT) and the 100 yr ARI storm tide level. As a rule, tailwater levels influence only that part of the catchment below Lake Callemondah.

The critical duration for the catchment (based on consideration of peak flows, volumes and predicted levels) was selected as 3 hours.

Performance

Existing or potential problem areas were identified through a number of processes, including:

- Locations previously noted by Council;
- Inspection of the study area;
- Reference to the results of modelling, which indicate:
    - areas with significant velocity; and
    - structures that are predicted to overtop.

Overall, approximately sixteen areas were identified as potential problem areas, with six roads predicted to overtop during the 100 yr ARI event.

Peak flood height and velocity maps have been produced for each of the nominated ARI events for both the existing and the ultimate cases. In addition, hazard maps have been produced for the 50 and 100-year ARI existing cases, forming the basis of the risk vulnerability assessment within the overall Risk Management framework.

Flood hazard was categorised based on the NSW Floodplain Development Manual (2005). The flood hazard is broken into three categories listed below:

- Low Hazard;
- Intermediate Hazard (dependent on site conditions); and
- High Hazard.

Low hazard is defined qualitatively as inundated locations where trucks could evacuate people and their possessions if necessary and able-bodied adults would have little difficulty in wading to safety.

High hazard is defined when depth exceeds 1.0 m, or velocity exceeds 2.0 m/s, or the depth-velocity product (v.d) exceeds 0.6.

Population and Property at Risk

The assessment of population and property at risk has been based on the 100 yr flood event. For this event (for existing conditions), it is estimated that 268 properties are at risk of inundation up to a depth of 2.2 metres, with most at substantially lower depths. Through the application of a factor of 2.8, this number of properties equates to 345 people at risk of injury, or 84 people at risk of death. However, in most areas the depth of inundation is low, and a range of factors (e.g. shelter within inundated buildings) suggests that the actual risk of death is somewhat lower.

Mitigation

A range of mitigation measures has been tested using the hydrologic and hydrodynamic models. Initially, two mitigation scenarios were tested independently to assess the relative benefit of different approaches. Option 1 was founded on the addition of several detention basins, whilst Option 2 focussed more on structural change (upgrades) to waterway crossings and the addition of levee banks.

Overall the most benefit was gained from the Option 1 retarding basins, which demonstrated significant beneficial effect on flooding along Tigalee Creek. The reduction in flood height was accompanied by lower velocities through downstream structures and waterways alleviating many of the predicted problems within the tributary.

The Option 2 (culvert upgrade) measures had a less beneficial effect with most augmented structures showing minimal reduction in upstream flood heights, despite the increase in flow capacity. Levee banks tested in Option 2 performed well in protecting targeted areas.

Following a review of results, and discussion with Council, a final mitigation scenario, consisting of a preferred combination of mitigation measures, was adopted. This consists of:

- A retarding basin upstream of Glenlyon Road adjacent to the Moura Short railway line;
- A retarding basin upstream of Glenlyon Road adjacent to Hurley Street.
- A levee bank along Tigalee Creek between Mercury Street and Witney Street;
- A levee band along Cathurbie Creek adjacent to Sandpiper Avenue;
- Levee banks along Phillip Street around the Kin Kora Shopping complex;
- A levee bank adjacent to Shaw Street downstream of the Penda Avenue crossing of Briffney Creek;

- Culvert upgrades for Cockatoo Drive, Mercury Street, Parksville Drive & Callemondah Ave; and
- A reduction in capacity (choking) of the Kirkwood Road Crossing #6 at Cathurbie Creek

Infrastructure Charges and Construction Plan

Capital and maintenance costs estimates have been prepared for each of the preferred mitigation options. These have been grouped in accordance with sub-catchment, with the combined costs then apportioned across the entire catchment in order to determine the infrastructure charge for flooding.

It is stressed that the estimated charges relate to flooding only, and do not address the provision of stormwater trunk drainage. The preferred option evenly distributes the infrastructure charges, estimated at $1,259 /ha impervious, across the entire catchment. The derivation of stormwater quality costs is provided in the Catchment Management report, which is presented separately.

Finally, an estimate of five and ten year construction plans has been completed. This was based on the assignment of a priority level to each of the proposed mitigation measures. It is estimated that approximately $ 500,000 is required for each five year period.

1. Introduction

1.1 Context and Scope

This report forms part of the overall Auckland Creek Flood Risk Management Study & Catchment Management Plan. The aim of this component of the study is to better understand flood risk to the community, and to prepare appropriate responses.

In keeping with the above, the objectives of the report are to:

- Provide a better understanding of flood risk to existing properties and to aid the decision process in future development planning;
- Better management flood impacts;
- Identify mitigation strategies and priorities;
- Prepare an infrastructure-charging plan for identified flood mitigation measures. (note that the charges do not allow for trunk drainage);
- Prepare a new flood model and generate flood risk mapping for the Auckland Creek catchment;

The output from this study aims to;

- Raise public awareness and preparedness for flooding;
- Direct future town planning;
- Direct counter disaster planing and mitigation strategies; and
- Provide a tool for better coordination of risk management response and mitigation measures between Gladstone City Council (GCC) and Calliope Shire.

1.2 Study Structure

The study utilised current generation software, along with reference to appropriate guidelines including:

- AS/NZS 4360:2004
- *Disaster Risk management*, Zamecka & Buchanan, 1999, Department of Emergency Services
- *Natural Disaster Risk Management: Guidelines for Reporting*, 2001, Department of Emergency Services

Additional guidance was gained from but not limited to the following references:

- *Local Counter Disaster Plan for the City of Gladstone and Shire of Calliope* (DES), 2003;
- *Community Risk in Gladstone* (AGSO & BoM, 2000);
- *Gladstone City Council Natural Disaster Mitigation Plan* (Earthtech & Sargent Consulting, 2003).

1.3 Model Extent

The extent of the hydraulic model covers Auckland / Police Creek, and the main tributaries of Briffney Creek, Tigalee Creek, Tondoon Creek and Cathurbie Creek. The model extends from the outlet of Auckland Creek (Gladstone Harbour) up to the point where the various waterways become steep from a hydraulic perspective. The development of the model (particularly with respect to assumptions made) has been undertaken in consultation and agreement with Council.

The extent of the Auckland Creek catchment and its location in relation to Gladstone City and Calliope Shire is shown in Figure 1. Overall, a total of 28 km of waterways has been simulated, as summarised below:

Table 1 Hydraulic Model Extent

Creek	Extents	Distance (km)
Auckland Creek	Harbour to confluence with Tondoon Creek	12.9
Police Creek	Auckland Creek confluence with Tondoon Creek to 700 m upstream of Haddock Drive.	3.5
Briffney Creek	Briffney Creek confluence with Auckland Creek to 400 m upstream of proposed Kirkwood Road alignment.	4.3
Cathurbie Creek	Cathurbie Creek confluence with Auckland Creek to 100 m upstream of Kirkwood Road.	1.5
Tigalee Creek	Tigalee Creek confluence with Auckland Creek to Glenlyon Road	2.9
Tondoon Creek	Tondoon Creek confluence with Auckland Creek to adjacent to Glenlyon Estate.	2.8

1.4 Design Flood Events Modelled

Mathematical modelling, both hydrologic and hydraulic, was undertaken to determine the 20 yr, 50 yr, 100 yr, 500 yr ARI and PMP flood extents within the Auckland Creek catchment.

Each of the nominated flood events was first simulated in the hydrologic model for design storm durations of 30, 45, 60, 90, 120, 180, 270 minutes and 6, 12, 24 and 48 hours, as described in Chapter 4. Through comprehensive analysis of these results and in consultation with Council, the critical duration was determined to be 3 hours.

Inflows into the hydraulic model were defined at the upstream end of each tributary, and at several locations within the model extent. Each of the nominated flood events was run in the hydraulic model for several scenarios, which were defined as follows:

- Existing development;
- Ultimate development;
- Ultimate mitigated case (several different mitigated cases were tested in the hydraulic model as described in Chapter 9)

1.5 Kirkwood Road

Kirkwood Road is an incomplete major arterial road connecting the Dawson Highway to Glenlyon Road through the middle reaches of the Auckland Creek catchment. It has, the potential to exert significant effects on the hydraulics of the catchments drainage system.

Following discussions with Council, Kirkwood Road was modelled in its current alignment and extent for both the existing and ultimate scenarios. This enabled accurate gauging of the effect of the change of land use between the two scenarios.

For the mitigated scenario, however, the full extent and ultimate alignment of Kirkwood Road was represented. This allowed for modelling of all future culverts and testing of those mitigation measures which involved crossings of Kirkwood Road.

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North

LEGEND

- ▭ Auckland Creek Catchment
- — Highway
- +-+-+ Railway
- 🟩 Gladstone State Development Area (Approx. Extent)
- 🟨 Calliope Shire
- 🟧 Gladstone City

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AUCKLAND CREEK FLOOD STUDY

FIGURE 1

AUCKLAND CREEK LOCALITY MAP

2. Data Collection

2.1 Project Data

A significant amount of data was assembled for this study. This data is described in the following sections and includes:

- Previous reports
- Copies of the Counter Disaster Plan and the Disaster Risk Management Plan for Gladstone
- Aerial photographs
- Digital data (eg. ground elevations and cadastral boundaries)
- Topography
- Survey data
- Site observations
- Rainfall data
- Land use information
- Details of waterway structures
- Location details of key infrastructure within flood prone areas.
- Stream gauges
- Historic flood data (minimal available).

2.2 Previous Studies

Gladstone City Council provided a number of reports for this study, as listed below:

- *Auckland Creek Hydraulic Study* (Pak-Poy & Kneebone Pty Ltd., 1986)
- *Major Stormwater Drainage Systems* (W. J. Reinhold & Partners, 1976)
- *Tondoon Creek Drainage Study* (JWP, 1996)
- *Tigalee Creek Drainage Study* (Cox Andrews, 2001)
- *Briffney Creek Drainage Study* (Cox Andrews, 2000)
- *Cathurbie Creek Drainage Study* (Cox Andrews, 2000)
- *Police Creek Drainage Study* (Cox Andrews, 2000)

A review of these reports is provided in Chapter 3.

2.3 Aerial Photographs

Aerial photographs of the Gladstone region taken in 1998 were provided by Council. These photographs provided information on the extent of development and vegetation patterns that existed at the time.

The extent of development and vegetation were used to determine appropriate runoff coefficients and Manning's roughness values in the hydrologic and hydraulic models. Additionally, they were used to verify the locations of various waterway structures, as derived from other sources.

2.4 Digital Data

Council made the following list of digital data available for use in this study.

- GIS land use maps for the existing and ultimate development cases,
- Cadastre,
- Stormwater and sewage infrastructure details,
- 1 m and 5 m contours for the Gladstone City and Calliope Shire Local Government areas, and
- Census 2001 GIS database, which provided demographic data for use in the flood disaster risk analysis.

2.5 Catchment Topography and Condition

The upper portion of the Auckland Creek catchment consists of large areas of native bushland that remain in generally good condition. Several residential developments are currently under construction within the upper catchment area, resulting in ongoing clearing of native vegetation. Creeks within the upper catchment are generally ephemeral, with flows only occurring during larger rainfall events significant enough to generate substantial overland flow. Few rainfall events have been experienced in the region in the past couple of years, allowing grasses and weeds to become well established in creek beds and channels.

Land uses in the mid-catchment include residential development, recreational land and light industrial areas, with some small areas currently under construction. This area also includes the Tondoon and Callemondah Lakes. The topography in this area is generally flatter than in the upper catchment, which has promoted the development of residential estates in particular.

In general, native tall trees, shrubs, grasses and herbs within the riparian zone have been extensively cleared with many areas now consisting primarily of grass. At several locations throughout the middle of the catchment, the natural creek channel has also been altered, either by straightening, deepening or by being replaced with a concrete drainage channel. The creeks within this area are generally lined with loose gravel and dirt.

Weirs have been placed at two locations within the Auckland Creek system forming Lake Tondoon and Lake Callemondah.

The lower catchment area consists of tidally influenced waterways and tidal flats below Lake Callemondah. The surrounding land use consists of industrial areas, land reclamation, various maritime businesses (including a marina and yacht club), a power station and a waste management facility.

The waterways of the lower catchment area are typically saline and tidally influenced, with the weir at the downstream end of Lake Callemondah representing the divide between the freshwater and saltwater sections of Auckland Creek.

2.6 Survey Data

Gladstone City Council provided a digital copy of cross section survey at selected points along Auckland Creek starting at the Lake Callemondah Weir. Council staff commenced the survey in February 2005. The location of cross-sections is illustrated in Figure 8.

The initial survey was supplemented by additional survey conducted in May 2005, which covered the area downstream of the Lake Callemondah Weir. This survey data was adapted by GHD into a GIS format for use in the hydraulic model.

The Department of Transport (Maritime Division) also provided hydrographic survey, which was provided in hardcopy form as, “Gladstone Auckland Creek Inlet, Hydrographic Survey (May/June, 1990). The 1990 hydrographic survey was supplemented with digital hydrographic survey data from a later, 1996 survey by the same department. Figure 8 shows the different sources of survey data used in the DEM.

2.7 Site Inspection

GHD undertook a detailed field inspection of the Auckland Creek Catchment on the 4th and 5th of April 2005. The inspection had the following objectives:

- Identify and detail key dimensions of structures such as culverts, bridges, and pipelines that would significantly affect the hydraulics of Auckland Creek.
- Characterise creek and channel hydraulic parameters for use in the relevant hydrologic and hydraulic models.
- Characterise water quality aspects of the Auckland Creek catchment.
- Identify any areas likely to be affected by or during significant flood events.
- Gather photographic and anecdotal evidence of previous flood heights.
- Allow project staff to gain an appreciation of the nature of the Auckland Creek catchment.

The inspection targeted 35 specific sites within the catchment and the sub-catchments of Tondoon, Tigalee, Briffney, Police and Cathurbie Creeks. These sites were identified in consultation with Gladstone City Council, desktop studies and the review of previous reports. The sites, listed in Table 3, were chosen on the basis of their perceived importance in terms of the hydrology and hydraulics of Auckland Creek and tend to correspond to waterway structures such as bridges and culverts.

2.8 Rainfall

The following design data rainfall sets have been used in this study:

- Probable maximum precipitation (PMP) based on the generalised Short Duration Method (GSDM) and Generalised Tropical Storm Method Revised (GTSMR) procedures;
- Intensity-Frequency-Duration (IFD) data derived from data sets provided by Gladstone City Council using the procedures described in Australian Rainfall and Runoff (IEAust, 1999).

Details of the rainfall data sets used in this study are included as Appendix A.

2.9 Land Use

Details of existing and future land uses in the catchment have been gathered from GCC and through discussions with Council GIS officers with respect to the current Town Plan Zones and the Gladstone Strategic Plan, as shown in Figure 3 and Figure 4.

The type and extent of land use in the area has a significant impact on the hydrological and hydraulic characteristics of the catchment. Management of such land use through the planning process is a key aspect of integrated catchment management and flood risk mitigation. The fully developed catchment, without mitigation measures, is referred to as the ultimate case.

Table 2 summarises existing and ultimate land use within the Auckland Creek catchment.

Table 2 Land Use Breakdown For Existing And Ultimate Cases

Land Use	**Fraction Impervious**	**Existing (ha)**	**Ultimate (ha)**
Urban	0.4	1413	3811
Open Space	0.1	677	980
Commercial	0.9	30	50
Industrial	0.9	194	656
Special Use	0.5	623	44
Rural	0.2	2644	39
Total		5580	5580

2.10 Waterway Structures

A list of the structures represented in the hydraulic model and key locations is presented in Table 3.

Table 3 Key Locations and Structures

Structure #	Description	Chainage (m)	Type
Auckland Creek			
1	Marina Bridge Creek Outlet	800	Bridge
3	Hanson Road (Clinton) Bridge	3840	Bridge
4	Ash Pond Causeway		Weir
5	Blain Drive Bridge	7060	Bridge
6	Lake Callemondah Weir	7550	Weir
7	North Coast Railway Crossing	8300	Bridge
8	Dawson Highway Bridge (Golf Course)	9480	Bridge
9	Moura Railway Crossing (#1)	10340	Bridge
42	Blain Park Pedestrian Bridge	8600	Bridge
Tigalee Creek			
10	Witney St Crossing	1250	Box Culvert
11	Mercury Street Crossing	2160	Box Culvert
12	Glenlyon Road Crossing (#1)*	2860	Pipe Culvert
36	Links Court Bridge	1000	Bridge
44	Moura Rail Crossing (#2)	1300	Pipe Culvert
45	Moura Rail Crossing (#3)	1950	Pipe Culvert
46	Moura Rail Crossing (#4)	2,300	Pipe Culvert
47	Glenlyon Road Crossing (#2)*	2300	Pipe Culvert
Emmadale Creek			
14	Cockatoo Drive Crossing	200	Pipe Culvert
39	Emmadale Drive	900	Pipe Culvert
Cathurbie Creek Tributary			
16	Parksville Drive (#2)	200	Box Culvert
19	Kirkwood Road Crossing (#5)	500	Pipe Culvert
Cathurbie Creek			
50	Kirkwood Road Crossing (#6)	1300	Pipe Culvert

Structure #	Description	Chainage (m)	Type
38	Parksville Drive (#1)	600	Pipe Culvert
Tondoon Creek			
21	Tondoon Reservoir Outlet	350	Dam wall with spillway
22	Glenlyon Road Crossing (#3)	1580	Box Culvert
Police Creek			
27	Kirkwood Road Crossing (#7)	14800	Box Culvert
28	Haddock Drive Crossing	15600	Box Culvert
41	Dixon Road Crossing*	12800	Box Culvert
Briffney Creek			
29	Moura Railway Crossing (#5)	400	Bridge
30	Callemondah Ave. (Bebo Arch)	850	Bebo Arch
31	Dawson Hwy Road Bridge	1180	Bridge
32	Penda Avenue (Bebo Arch)	1450	Bebo arch & box culvert
34	Kirkwood Road Crossing (#1)*	3700	Box Culvert
35	Kirkwood Road Crossing (#2)	2800	Pipe Culvert

* Future culverts modelled in mitigation case only.

2.11 Stream Gauges

As part of the flood study, and to facilitate Gladstone City Council's ongoing operations, two stream gauges have been installed within the Auckland Creek system. The gauges determine flood levels by measuring changes in pressure.

The two gauges, shown in Figure 2, are located:

- Approximately 30 m upstream of the Dawson Highway Bridge on Auckland Creek.
- Immediately upstream of the Haddock Drive culvert on Police Creek.

At the time of writing, there had been no rainfall events of significant magnitude since installation of the gauges, and hence no gauge data has been utilised in the study. A report detailing the installation of the gauges is included in this report as Appendix C.

Golf Course Station

Haddock Dr

Figure 2 Auckland Creek Stream Gauges

It is recommended that the gauges continue to be monitored and the data used in the future to update the hydraulic model.

2.12 Historical Flood & Cyclone Data

Limited historic flood level data was available, though Gladstone City Council provided flood heights and anecdotal evidence in relation to Cyclone Beni (6th February, 2003), which IFD analysis has shown to have an approximate 50 year ARI for the 48 hour duration (BoM 2003).

2.13 Known Problem Areas

Gladstone City Council has provided advice on previously identified water quality and quantity problems within the Auckland Creek catchment. Locations of identified problems, including sites of erosion and creek bank instability, road overtopping and flooding of properties are presented in Figure 5.

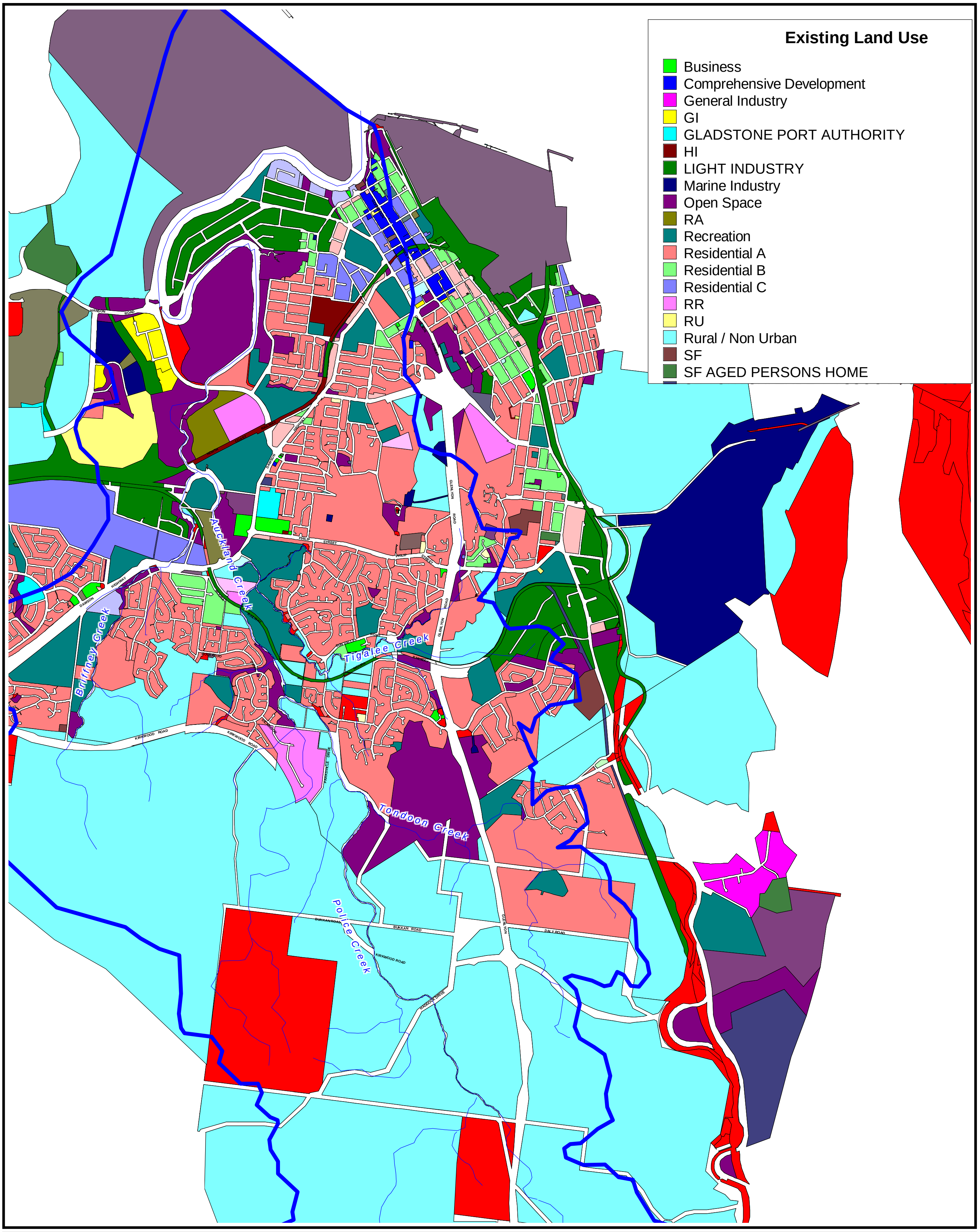
Figure 5 shows that the majority of the noted problem areas lie within the middle reaches of the catchment, which coincides with where the majority of recent urban development has occurred. Flooding has been identified as a problem at the following locations:

- Along Briffney Creek upstream of the proposed Kirkwood Road crossing and adjacent to Shaw Street. Potential flooding of properties adjacent to Shaw Street in the 100 yr ARI storm event was also noted in the previous Cox Andrews drainage study for Briffney Creek;

- Properties in the vicinity of Parksville Drive, Cathurbie Creek;
- Properties upstream of the Moura Short Railway line and downstream of the confluence of Cathurbie Creek and Police Creek;
- Several locations along Tigalee Creek adjacent to Sun Valley Road, including upstream of the Witney and Mercury Street culverts; and
- Upstream of the Glenlyon road crossing of Tondoon Creek.

Several areas of creek instability and high erosion potential were also identified. These are located:

- Downstream of the Links Court Bridge over Tigalee Creek;
- Within a tributary of Briffney Creek near the end of Penda avenue; and
- Adjacent to Allunga Drive in Tondoon Creek.



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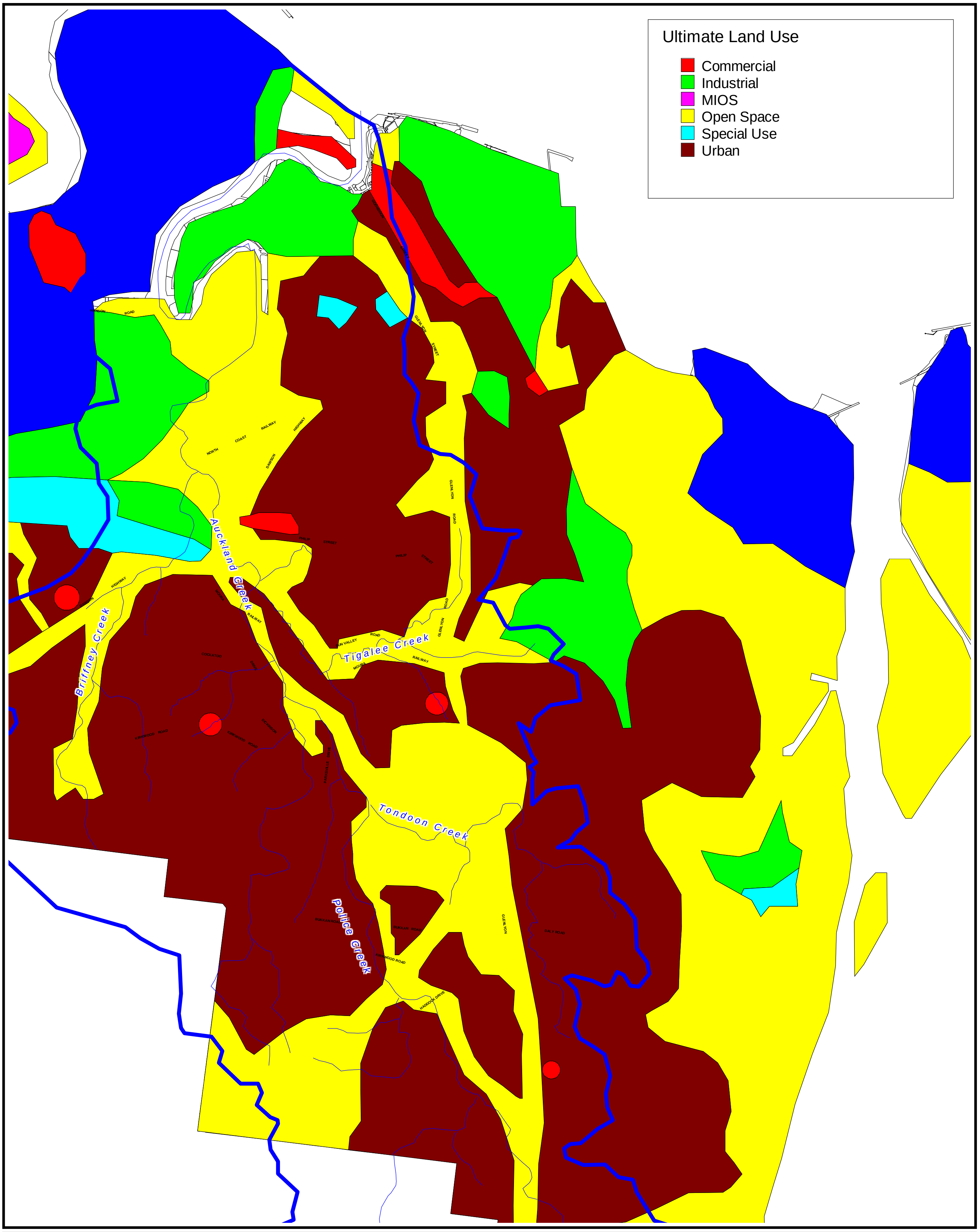
North

AUCKLAND CREEK FLOOD STUDY

FIGURE 3

EXISTING LAND USE



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North

AUCKLAND CREEK FLOOD STUDY

FIGURE 4

ULTIMATE LAND USE

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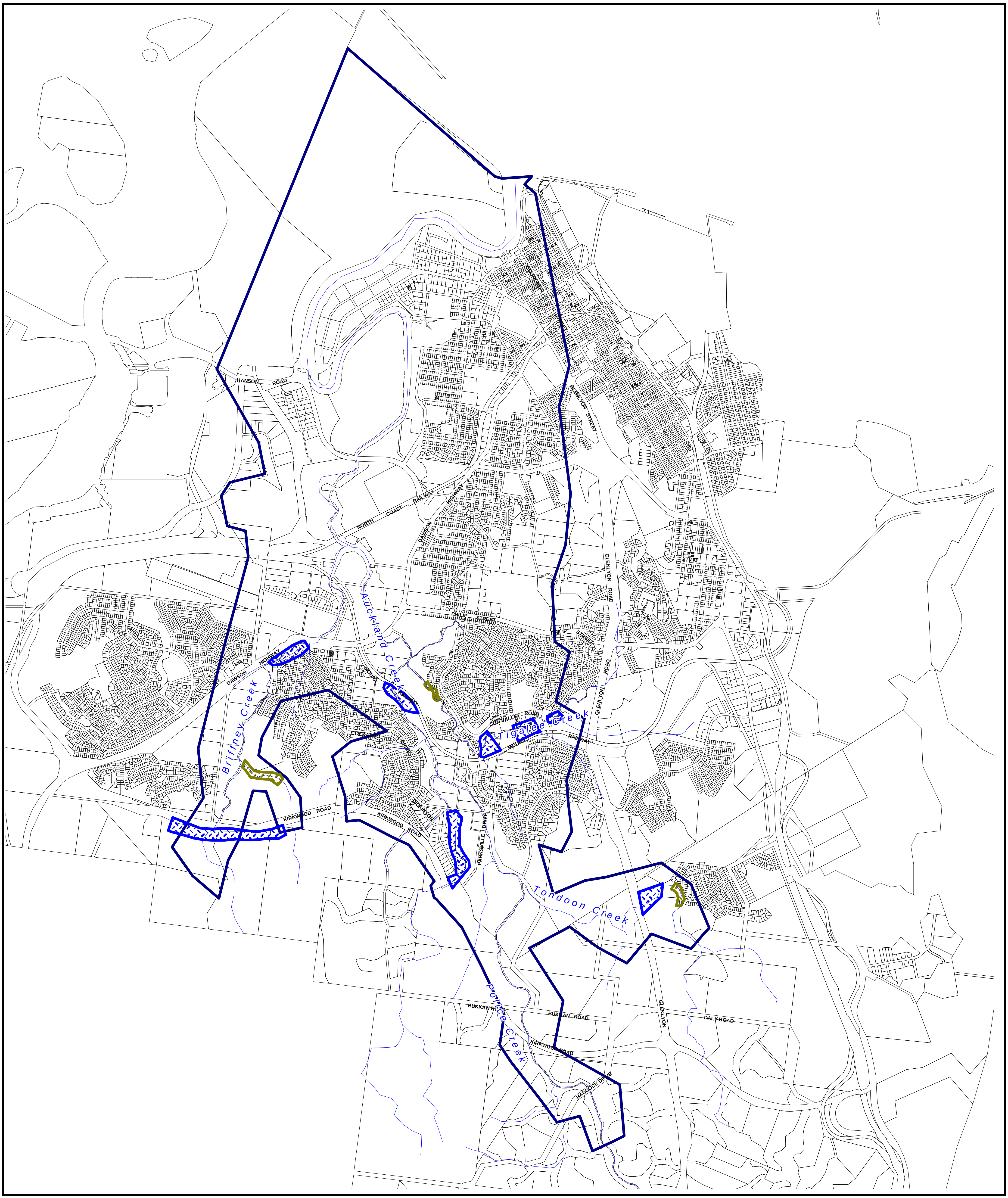

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LEGEND

▭	Region
—	Waterways
▨	Flooding
▨	Bank Erosion

North

0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 5

Known Problem Areas

3. Previous Studies

3.1 Introduction

The section provides a review of previous flood studies within the Auckland Creek catchment, in the form of a summary of conclusions and recommendations relevant to the current study. The reports date from 1976 through to 2001 and provide a history of drainage and stormwater management within Gladstone over the last 30 years.

Specifically, the reviewed reports comprised the following:

- “Auckland Creek Hydraulic Study”, (Pak-Poy & Kneebone Pty Ltd., 1986)
- “Major Stormwater Drainage Systems”, (W. J. Reinhold & Partners, 1976)
- “Tondoon Creek Drainage Study”, (JWP, 1996)
- “Tigalee Creek Drainage Study”, (Cox Andrews, 2001)
- “Briffney Creek Drainage Study”, (Cox Andrews, 2000)
- “Cathurbie Creek Drainage Study”, (Cox Andrews, 2000)
- “Police Creek Drainage Study”, (Cox Andrews, 2000)

Direct comparisons with the current study are difficult owing to changes in catchment characteristics and differences in the applied assessment methodology. However, these reports do provide a database of sorts with respect to the extent of Auckland Creek stormwater knowledge, and therefore provide a useful point of reference for the current study.

3.2 W. J. Reinhold & Partners

The earliest reviewed report, “Major Stormwater Drainage Systems”, (Reinhold & Partners, 1976), examined all major drainage catchments within Gladstone. It reported on the hydrology of the catchments and the impact of existing and future development on the hydraulics of major drainage system infrastructure. The report also assessed the environmental impact of construction and operation of the stormwater drainage system.

The report is of limited specific significance within the scope of this report owing to the changes in method of calculating hydrologic and hydraulic impacts of stormwater drainage systems and the considerable changes in development within Gladstone since 1976.

The report recommended that future planning for stormwater include a need for developers along the mid-reaches of the catchment to preserve natural creek flood corridors sufficient to convey the 50 yr ARI design storm flow. The current study has confirmed that where this recommendation was adhered to, flooding of properties is minor, and mitigation of the adverse effects of future development on stormwater quantity and quality, achievable. The successful implementation of this recommendation is a salient message that highlights the importance of future and ongoing developmental control in the implementation of cost effective stormwater management.

3.3 Pak-Poy & Kneebone Pty Ltd

In 1986, GCC, the Queensland Electricity Commission and the Gladstone Harbour Board commissioned consulting engineers Pak-Poy and Kneebone to undertake an hydraulic study of the Auckland Creek Catchment. This report had two major aspects, river and creek flooding whilst also considering siltation. The latter provided an assessment of the long-term effects of siltation rates on the lower reaches of Auckland Creek.

Many of the findings in the PPK report are not relevant to the current study due to substantial changes in stormwater drainage infrastructure over the last 20 years and differences in analysis methodology. However, the report also made a number of conclusions that do remain relevant in the context of the current study. These comprise the following:

- Flooding occurs at a number of houses at the downstream end of the Breslin Street Drain. This is caused mainly by extreme sea levels for which no mitigation work was recommended.
- During extreme events, flooding occurs at Ferguson Park Racecourse, Blain Park and Lions Park. No mitigation works were recommended for these sites.
- Flooding of Briffney Creek inundates houses in the vicinity of Shaw Street and Wilson Street. Mitigation in the form of channel augmentation and realignment was recommended for this site.
- Flood levels along Auckland Creek downstream of Blain Drive are predominantly determined by extreme sea level. Therefore recommendations with respect to flood levels are based on the Blain Bremer and Williams Surge and Tide analysis report with an additional allowance for freeboard recommended to account for possible water level rises due to simultaneous rainfall events.
- In agreement with the earlier W. J. Reinhold & Partners 1976 report, this report recommended future developmental control to limit intrusions into designated flood plains.

GHD’s flooding analysis in general concurs with these conclusions. This study has found similar flooding problems as those reported in the 1986 study.

3.4 John Wilson and Partners Pty Ltd

The scope of the *‘Tondoon Creek Drainage Study’* (JWP, 1996) was limited to the Tondoon Creek catchment, which is a major tributary of Auckland Creek.

The objectives of the study were to determine detailed flood levels for the 5yr and 50yr ARI design storms in the area extending from the spillway structure of the Tondoon Reservoir in the botanic gardens to the proposed Kirkwood Road, and to provide recommendations on the most appropriate drainage structures for the arterial road crossing at Glenlyon Road and Kirkwood Road.

The report provided 50 yr ARI design flows for Tondoon Creek at several locations, which were loosely used as a point of comparison with flood levels and flows produced in GHD's study. However, a detailed comparison of flows was not possible due to differences in methodology.

The JWP report provided recommendation for flood mitigation for the existing residential dwellings in the Allunga drive area of Glen Eden and cross drainage details for Glenlyon and Kirkwood Roads.

3.5 Cox Andrews Engineers Pty Ltd

Between 2000 and 2001 Gladstone City Council commissioned Cox Andrews Engineers Pty Ltd to undertake a series of drainage studies for several major sub-catchments of the Auckland Creek catchment including Tigalee Creek, Cathurbie Creek, Briffney Creek and Police Creek.

The reports were limited to small study areas, using the RORB hydrologic software to determine design flows for each of the sub catchments. Where required, development of drainage options and the preliminary design of culverts was provided.

The similarity between the hydrologic modelling package, RORB (used in these studies), and URBS (as used in the current study) allowed for a comparison of some of the Cox Andrews reported flood flows and those predicted by the GHD study. The comparison is provided in Section 4.7. Generally, the predicted design flood flows compare well despite the relatively high continuing loss values used in the Cox Andrews RORB model of 5 mm/hr for Tigalee Creek and 10 mm/hr for Police, Cathurbie and Briffney Creeks. These values are above those recommended for un-gauged catchments by Australian Rainfall & Runoff, which gives as a median value 2.5 mm/hr.

The Cox Andrews series of reports focussed on the design of future creek crossings and developing existing drainage upgrade options in specific locations and therefore did not account for interactions between sub catchments.

The current study, which assesses all waterway within the Auckland Creek catchment, provides context for the Cox Andrews reports and will allow them to be used more effectively within a catchment wide stormwater management plan.

3.6 Sargent Consulting

The Calliope Shire Council and Gladstone City Council commissioned Sargent Consulting to undertake the Calliope River Flood Risk Study Assessment Study in parallel to the GHD study of Auckland Creek. Both Calliope River and Auckland Creek share common receiving waters and consequently adopted tailwater levels are relevant to both studies.

Results from the Calliope River Flood Risk Study show that flood inundation from the river does not extend to the Auckland Creek catchment in the 100 yr ARI event, but may have a minor influence in some of the western industrial areas of the Auckland Creek catchment for the PMF event.

4. Hydrology

4.1 Overview

Design flood flows have been determined through the use of a commonly used hydrologic model, which desirably would have been calibrated to historic data. However, insufficient data existed to allow calibration. The design process therefore involved the adoption of appropriate design rainfall distributions, rainfall depths, rainfall losses and temporal patterns.

In this study, the uncalibrated GHD URBS model has been used to simulate design flood events and to determine appropriate design flood flows. A description of the key hydrologic design parameters adopted in the model is provided in Sections 4.3 to 4.6.

A range of factors affects the accuracy of hydrologic predictions. These include:

- The length and quality of historic stream flow and rainfall records;
- Natural climatic variability;
- Climate change associated with the Greenhouse Effect; and
- Changes to the catchment and waterway.

With a basis in statistics, each and every event may therefore alter future predictions from that point in time.

4.2 Design Flood Estimation

A design flood is a hypothetical flood that has been determined for the purpose of floodplain management and planning. Design floods are typically assigned a probability of occurrence that is specified as an Average Recurrence Interval (ARI) or as an Annual Exceedence Probability (AEP). The AEP is expressed as a percentage whilst ARIs are expressed in years. Table 4 provides a description of the design floods assessed in this study.

Table 4 Descriptions of Design Events

AEP	ARI	Description
50%	2 years	This flood is likely to occur on average once every 2 years.
20%	5 years	This flood is likely to occur on average once every 5 years.
5%	20 years	This flood is likely to occur on average once every 20 years.
2%	50 years	This flood is likely to occur on average once every 50 years.
1%	100 years	This flood is likely to occur on average once every 100 years.

AEP	ARI	Description
0.2%	500 years	This flood is likely to occur on average once every 500 years.
	PMF	The Probable Maximum Flood (PMF) is the limiting value of flood that could reasonably be expected to occur.

It is important to note that (for example) the 100 year ARI event occurs on average, once every 100 years but may occur more than once in any 100-year period.

4.3 Design Rainfall

The following design data rainfall sets have been used in this study:

- Probable Maximum Precipitation (PMP) based on the Generalised Short Duration Method (GSDM) and Generalised Tropical Storm Method Revised (GTSMR) procedures;
- Intensity-Frequency-Duration (IFD) data derived from data sets provided by Gladstone City Council using the procedures described in Australian Rainfall and Runoff (IEAust, 1998).

This section presents the relevant data for each rainfall set, areal reduction factors, and the adopted design rainfall for the Auckland Creek Catchment.

4.3.1 PMP Estimation Methods

The Bureau of Meteorology (BoM) has developed two methods for estimating PMP rainfalls depending on storm duration in the tropical storm zone region:

- GSDM - Generalised Short Duration Method (BoM, 2003); and
- GTSMR – Generalised Tropical Storm Method, Revised (Walland et al., 2003).

Details of these methods and the estimates are given below.

This section presents the relevant data for each rainfall set, and the adopted design PMPs for the Auckland Creek Catchment.

4.3.2 AEP of PMP

The Annual Exceedence Probability (AEP) of the PMP for the Auckland Creek Catchment was estimated to be 1x 10^{-7}, as determined from Book VI of Australian Rainfall & Runoff. This is equivalent to a 1 in 10,000 year event.

4.3.3 Spatial Distribution

Due to the relatively small catchment area for Auckland Creek, it has been assumed there is no spatial variability in rainfall distribution.

4.3.4 Temporal Patterns

The hydrologic model was run for design storm durations of 30, 45, 60, 90, 120, 180, 270 minutes and 6, 12, 24 and 48 hours. The design temporal patterns corresponding to each of these durations were extracted from Australian Rainfall and Runoff (IEAust, 1998).

4.3.5 GSDM Adjustment Factors

Adjustment factors and the 2005 PMP estimates for Auckland Creek Catchment are given in Table 5.

Table 5 GSDM Adjustment factors

Parameter	Auckland Creek Catchment
Duration limit	6 hours
Topography Classification	8% rough
Moisture Adjustment Factor (MAF)	0.87
Elevation Adjustment Factor (EAF)	1.0

4.3.6 GTSMR Adjustment Factors

The Auckland Creek Catchment Area is located within the GTSMR Coastal Zone region of applicability, which covers those regions of Australia where tropical storms are the source of the greatest depths of rainfall. In the coastal zone, the maximum duration covered by the method is 120 hours in summer and 96 hours for all other seasons.

The relevant GTSMR adjustment parameters estimated for the Auckland Creek Catchment are given in Table 6. A map illustrating the spatial variability across the Auckland Creek Catchment is provided in Appendix A.

Table 6 GTSMR Adjustment Factors

Parameter	Auckland Creek Catchment
Topographical Adjustment Factor (TAF)	1.43
Decay Amplitude Factor (DAF)	1.0
Extreme Precipitable Water (EPW)	94.93
Annual Moisture Adjustment Factor (MAF_a)	0.79

Preliminary PMP estimates have been calculated by multiplying the initial depths by the catchment adjustment factors. The formulas used are:

- Summer:
  - Preliminary PMP Estimate = Initial Depth x TAF x DAF x MAF_a

- Winter:
  - Preliminary PMP Estimate = Initial Depth x TAF x DAF x MAF_w

The PMP estimate based on winter data was not assessed because winter rainfall is low and the dominant wet season occurs in summer.

The final annual estimates are determined from the enveloping curve drawn to fit the depths in the “Preliminary PMP Estimate”. The adopted values are shown in Table 7, with the estimates plotted in Figure 6.

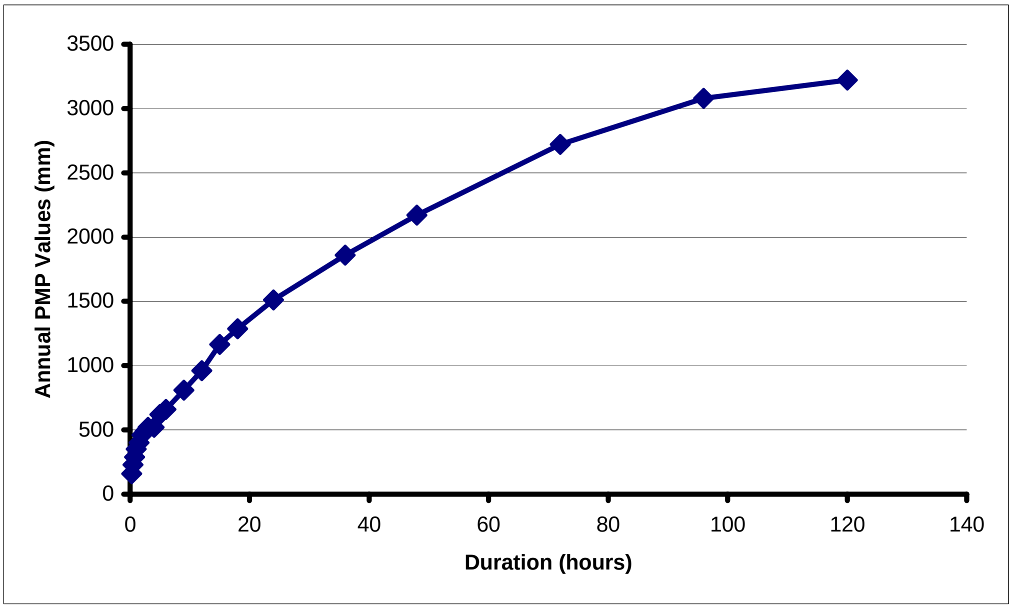

Figure 6 PMP Depths for Auckland Creek Catchment

4.3.7 PMP Estimates

Table 7 PMP Estimates for Auckland Creek catchment

Storm Duration (Hours)	PMP Estimate (mm)
15 minutes[1]	160
30 minutes[1]	230
45 minutes[1]	290
1 hour[1]	350
1.5 hours[1]	400

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[1] PMP Estimate based on GSDM

Storm Duration (Hours)	PMP Estimate (mm)
2 hours[1]	460
2.5 hours[1]	490
3 hours[1]	520
4 hours[1]	580
5 hours[1]	620
6 hours[2]	660
12 hours[2]	960
24 hours[2]	1510
36 hours[2]	1860
48 hours[2]	2170
72 hours[2]	2720
96 hours[2]	3080
120 hours[2]	3220

4.3.8 Intensity Frequency Duration Rainfall

Intensity-Frequency-Duration (IFD) rainfall is derived using the procedures described in Australian Rainfall and Runoff (IEAust, 1998). The IFD table for the Auckland Creek Catchment is listed in Appendix A.

4.3.9 Areal Reduction Factors

There are two sources of Areal Reduction Factors (ARF) for large and rare rainfall events:

- those described in Australian Rainfall and Runoff (IEAust, 1998), and
- those estimated by the Department of Natural Resources and Mines (DNRM) (2002), as part of the FORGE dataset creation.

The Australian Rainfall and Runoff factors are based on graphs plotting the reduction in rainfall versus storm duration. This data is based on limited information collected in the United States.

DNRM have estimated ARFs based on storm durations ranging from 1 to 5 days. This estimate is a function of storm duration and catchment area. DNRM consider extrapolation to durations smaller than 18 hours to be risky without the benefit of relevant data and recommend using the 18-hour figure for smaller durations. The

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[2] PMP Estimate based on GTSMR

DNRM-based estimates have been employed here and the values are summarised in Table 8.

Table 8 Areal Reduction Factors, Auckland Creek Catchment

Storm Duration	ARF
24	0.948
48	0.972
72	0.983
96	0.988
120	0.992

4.3.10 Adopted Design Rainfall Totals

The adopted design rainfall totals are shown in Appendix A.

4.3.11 Design Rainfall Losses

Table 9 indicates the initial and continuing losses adopted for design runs. These values fall within the ranges identified in Australian Rainfall and Runoff (IEAust, 1998).

Table 9 Design Rainfall Losses for Auckland Creek

ARI (years)	Losses	
	Initial Loss (mm)	Continuing Loss(mm/hr)
2 year	**20**	**2.5**
5 year	**15**	**2.5**
10 year	**10**	**2.5**
20 year	**5**	**2.5**
50 year	**0**	**2.5**
100 year	**0**	**2.5**
500 year	**0**	**2.5**
PMP	**0**	**2.5**

4.4 Model Description

This study uses the URBS SPLIT routing model. The SPLIT model separates the channel and catchment storage components of each sub-catchment for routing purposes. Each storage component is conceptually represented as a non-linear reservoir.

URBS main strength is its technical capability with regard to modelling land use effects and the impacts of urbanisation. However computationally there is little to distinguish between many of the available modelling packages.

URBS can be run with a command line prompt means that it is amenable to being set up in a batch mode, which facilitates the generation of multiple runs.

4.5 Model Development

The Auckland Creek catchment was subdivided into 7 major sub catchments representing Tondoon, Police, Tigalee, Cathurbie, Briffney and Auckland Creeks along with a tributary of Cathurbie Creek. These were further delineated into 62 sub-catchments based on the following factors:

- Natural topographical features such as creek confluences;
- The location of hydraulic structures;
- Areas of interest to Council;
- The location of problem areas identified through site inspection and the review of available data.

Table 10 provides a summary of the catchment area of each of the major sub catchments. Figure 7 shows the URBS sub catchment delineation for the Auckland Creek Catchment.

Table 10 Auckland Creek URBS Model Catchment Areas

Model	Catchment Area (km^2)	Number of sub-catchments
Auckland Creek	14.9	13
Police Creek	16.3	12
Cathurbie Creek	4.4	9
Cathurbie Creek Tributary	1.0	1
Tondoon Creek	5.8	8
Briffney Creek	7.0	9
Tigalee Creek	5.5	9
Kin-Kora Creek	0.9	1
Total Catchment	**56**	**62**

A listing of the adopted model parameters for the GHD URBS model is given Table 11.

Table 11 Adopted URBS Model Parameters

Model Parameter	Value
Channel lag (α)	0.055
Catchment lag (β)	0.04
Catchment non linearity (m)	0.80
Muskingham non linearity exponent (η)	1.00
Muskingham translation (x)	0.00
ULI (fraction impervious for low density urban)	0.1
UMI (fraction impervious for medium density urban)	0.4
UHI (fraction impervious for high density urban)	0.5

The fractions impervious, used for both the Rational Method and the URBS model, were derived from the Gladstone town plan zones and the strategic plan land use databases, supplied by GCC. To compare existing and developed cases, the 15 town plan zones were rationalised to the 8 land use zones as used in the strategic plan. Table 12 gives the strategic land use zone codes, designation and fraction impervious. Additionally, within the URBS model urbanisation parameters, low-density urbanisation (ULI) and medium density urbanisation (UMI) were used to account for changes in urbanisation within the catchments.

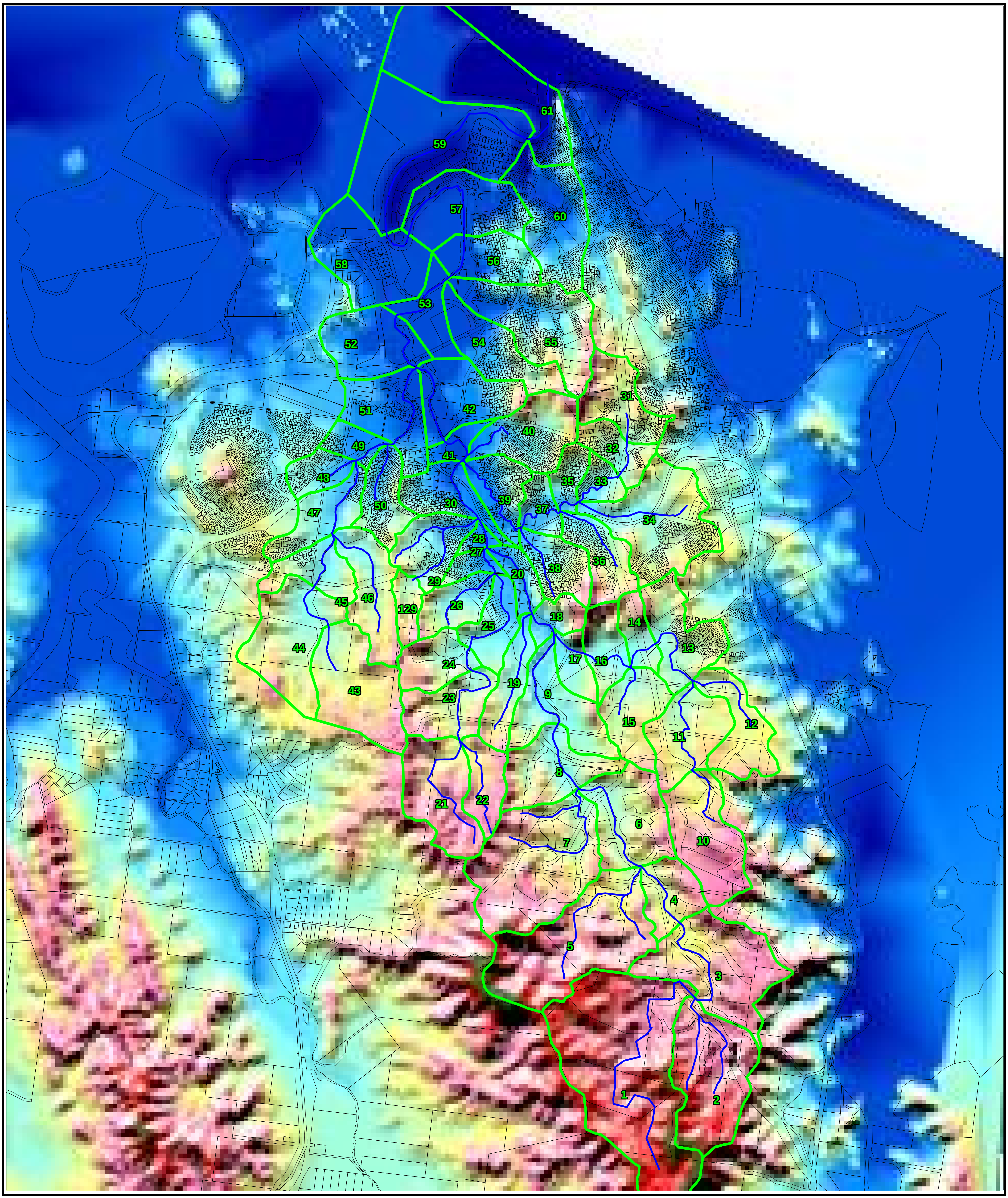
Table 12 Land Use Zones And Fraction Impervious

Code	Designation	% Impervious
OS	Open Space	0.1
SU	Special Use	0.5
U	Urban	0.4
RR	Road Reserve	0.9
I	Industry	0.9
C	Commercial	0.9
MI	Major Industry	0.9
RU	Rural	0.2

Default URBS model parameters are used in conjunction with the catchment parameters, as shown in Table 13.

Table 13 Selected Sub Catchment Parameters

Catchment ID	Area of contributing catchment	Tc	Slope	Fraction Impervious	
	(ha)	(Minutes)	(%)	Existing	Developed
1	373	81	1.75	0.20	0.40
9	1509	126	0.7	0.25	0.32
17	577	104	0.53	0.17	0.42
20	2207	133	0.64	0.35	0.43
24	301	30	1.32	0.22	0.40
27	429	71	1.1	0.32	0.38
32	122	29	1.35	0.37	0.41
39	548	77	0.13	0.33	0.37
40	22	51	0.55	0.22	0.25
44	269	34	1.33	0.22	0.36
50	59	28	0.48	0.36	0.39
51	704	106	0.25	0.51	0.62
61	5579	490	0.27	0.47	0.71



GHD
MANAGEMENT
ENGINEERING
ENVIRONMENT

LEGEND

▬	Waterway
🟩	URBS sub catchment
☐	Cadastre

North

↑

AUCKLAND CREEK FLOOD STUDY

FIGURE 7

URBS CATCHMENT DELINEATION

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4.6 Critical Duration

Peak flow estimates from the URBS model for the 1 hr, 3 hr and 24 hr duration storms were compared in order to determine the most appropriate critical duration to adopt for the Auckland Creek catchment.

The catchment has many contributing sub catchments, with correspondingly varying times to peak. Table 14 and Table 15 present the peak flows for the 1 hr, 3 hr and 24 hr duration 100 yr ARI storms. Several trends are clear from these tables. In the upper reaches of the creek, represented here by sub catchments 1 through 12, the 1 hr storm generally produced the peak flows while in the lower reaches (sub catchments 57 to 61) 24 hours was the critical duration. In the middle reaches of the catchment, where most new development is occurring and the majority of new stormwater infrastructure and stormwater flood risk mitigation measures are proposed, peak flows are still generated by the 1 hour duration storm which are marginally larger than the 3 hour duration peak flows.

Table 14 Comparisons of 1 hr, 3 hr and 24 hr 100 yr ARI Existing Peak Flows for Various Sub-Catchments

Catchment	1 hr		3 hr		24 hr		Peak Flows
	(m^3/s)	% of Peak	(m^3/s)	% of Peak	(m^3/s)	% of Peak	(m^3/s)
1	104	100%	90	86%	80	77%	104
2	67	100%	49	72%	35	52%	67
8	235	89%	253	96%	258	98%	264
9	227	85%	256	96%	267	100%	267
10	46	100%	35	76%	27	58%	46
11	63	100%	54	85%	46	73%	63
12	27	100%	21	77%	17	62%	27
16	89	87%	98	96%	99	97%	102
17	88	84%	100	96%	103	99%	104
18	312	84%	357	96%	371	100%	371
28	339	77%	417	95%	438	100%	438
37	124	100%	105	85%	95	77%	124
38	31	100%	21	66%	14	46%	31
36	30	100%	21	71%	15	50%	30
42	316	65%	450	93%	485	100%	485

Catchment	1 hr		3 hr		24 hr		Peak Flows
43	66	100%	46	69%	33	50%	66
44	64	99%	64	99%	58	91%	64
47	79	87%	86	94%	91	100%	91
48	18	100%	12	68%	10	57%	18
57	267	53%	421	84%	504	100%	504
58	264	52%	417	83%	504	100%	504
59	211	48%	345	79%	439	100%	439
61	207	47%	340	77%	442	100%	442

Table 15 Comparisons of the 1 hr, 3 hr and 24 hr 100 yr ARI Ultimate Peak Flows for various sub catchments

Catchment	1 hr		3 hr		24 hr		Peak Flows
	(m^3/s)	% of Peak	(m^3/s)	% of Peak	(m^3/s)	% of Peak	(m^3/s)
1	120	100%	104	86%	81	68%	120
2	88	100%	56	64%	41	47%	88
8	259	92%	268	95%	267	95%	282
9	240	87%	267	96%	267	96%	278
10	54	100%	39	73%	27	51%	54
11	66	100%	55	85%	47	71%	65
12	28	100%	22	79%	17	61%	28
16	95	90%	101	95%	103	97%	106
17	94	86%	104	96%	105	97%	108
18	331	86%	371	96%	375	97%	386
28	353	80%	431	98%	440	100%	440
36	31	100%	21	70%	15	50%	31
42	331	67%	466	94%	495	100%	495
43	92	100%	51	56%	41	45%	92
47	102	97%	98	93%	100	95%	105

Catchment	1 hr		3 hr		24 hr		Peak Flows
48	18	100%	12	67%	10	56%	19
57	274	54%	432	84%	511	100%	511
58	271	53%	428	84%	512	100%	512
59	215	48%	352	79%	446	100%	446
61	211	47%	347	77%	449	100%	449

However, flood risk is manifested by both peak flows and flood volume. Larger flood volumes cause a greater inundation extent and longer times of inundation, while higher peak flows will result in greater depth of flow and higher velocities through stormwater structures and overtopping of roads.

When flood risk including flood volume was considered, the 3-hour storm gave the highest combined risk. Thus 3 hours was adopted as the critical storm duration for use in the hydraulic modelling of flood risk in the Auckland Creek catchment.

4.7 Model Validation

In the absence of stream gauging data, the Rational Method (IEAUST, 1998) was used to verify the peak flows predicted by the URBS model.

The time of concentration (tc) was calculated using several methods including Bransby-Williams, Modified Friends, and a Combined Friends Inlet time and Channel flow time method as outlined in QUDM (1992). The latter method was found to provide the most reasonable estimate for tc. Rainfall intensities were derived using the Gladstone City rainfall IFD.

Table 16 shows a comparison of the URBS peak flow model output against peak flows calculated using the Rational method for the 100 yr ARI storm event.

Table 16 Comparison of predicted peak flows between URBS and the Rational Method for the ultimate development case

Catchment ID	Locality	URBS (m^3/s)	Rational Method (m^3/s)	Difference
9	Police Creek D/S of Tondoon confluence	267	238	11%
17	Tondoon Creek Outlet	105	99	5%
20	Police Creek Outlet	374	347	7%
27	Cathurbie Creek Outlet	100	97	3%
39	Tigalee Creek Outlet	121	118	2%

51	Briffney Creek Outlet	120	135	-13%
61	Auckland Creek Outlet	442	432	2%

Table 17 shows a comparison between the URBS peak flow model output against peak flows reported for similar locations reported by Cox Andrews for the 50 yr and the 100 yr ARI storm events.

Table 17 Comparison of predicted flows between GHD URBS model and the Cox Andrews RORB model for the ultimate development case

Catchment ID		**50 Year ARI**			**100 Year ARI**		
GHD	**Cox Andrews**	**C. A.**	**GHD**	**Diff.**	**C. A.**	**GHD**	**Diff.**
	Briffney Creek	**(m^3/s)**	**(m^3/s)**	**(%)**	**(m^3/s)**	**(m^3/s)**	**(%)**
51	B-3*	90	111	23%	106	131	24%
49	B-4	80	96	20%	94	114	21%
47	B-6	78	89	14%	92	105	14%
47	Dawson Hwy B-5	82	89	9%	96	105	9%
	Tigalee Creek						
37	Tig-1	86	103	20%	113	123	9%
39	Tig-5	86	102	19%	116	121	4%

Differences in the results between the GHD URBS model and the Cox Andrews RORB model may be due to the continuing loss values used in the Cox Andrews model of 5 mm/hr for Tigalee Creek and 10 mm/hr for Police, Cathurbie and Briffney Creeks. These values are well above those recommended by Australian Rainfall & Runoff, which gives as a median value 2.5 mm/hr.

Another source of differences is the likelihood of different catchment parameter definition both within the runoff rainfall model and the Rational Method calculations used to “calibrate” the respective models.

4.8 Peak Design Flow Estimates

Design event flows for Auckland Creek have been estimated for each of the durations listed in Section 4.3.4.

Peak flows for the existing and ultimate development cases at key locations in the catchment are presented in Table 18 and Table 19 respectively.

Table 18 Existing Development Peak Flows For Selected Locations

Catchment ID	Locality	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI	PMP
6	Haddock Drive	160	185	219	310	928
8	Kirkwood Rd. Police Creek	192	223	264	374	1090
9	Police Creek Outlet	196	226	267	376	1108
17	Tondoon Creek Outlet	76	88	104	148	430
20	Police Creek u/s of Cathurbie creek confluence	274	316	374	524	1553
27	Cathurbie Creek Outlet	73	84	100	144	444
32	Glenlyon Rd Tigalee Creek	52	55	64	88	236
37	Whitney St. Tigalee Creek	89	103	124	175	529
39	Tigalee Creek Outlet	89	102	121	170	521
44	Kirkwood Rd. Briffney Creek	50	55	64	89	251
47	Penda Ave. Briffney Creek	68	77	91	127	366
49	Dawson Highway Briffney Creek	74	85	99	138	402
51	Briffney Creek Outlet	88	101	120	168	500
61	Auckland Creek Outlet	303	371	442	633	2038

Table 19 Ultimate Development Peak Flows For Selected Locations

Catchment ID	Locality	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI	PMP
6	Haddock Drive	173	199	235	332	1016
8	Kirkwood Rd Police Creek	205	239	282	399	1192
9	Police Creek Outlet	199	234	278	394	1149
17	Tondoon Creek Outlet	79	92	108	154	445
20	Police Creek u/s of Cathurbie Creek confluence	280	323	383	546	1606
27	Cathurbie Creek Outlet	73	84	99	141	445
32	Glenlyon Rd. Tigalee Creek	52	55	65	89	236

Catchment ID	Locality	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI	PMP
37	Whitney St. Tigalee Creek	90	103	123	175	528
39	Tigalee Creek Outlet	89	102	121	171	520
44	Kirkwood Rd. Briffney Creek	79	89	105	145	395
47	Penda Ave. Briffney Creek	78	89	105	148	442
49	Dawson Highway Briffney Creek	83	96	114	160	476
51	Briffney Creek Outlet	96	111	131	185	536
61	Auckland Creek Outlet	310	377	449	642	2061

Table 20 shows the predicted increase in peak flows resulting from the development process. Increases were derived using the values provided in Table 18 and Table 19.

Table 20 Increase in Peak Flows for selected Locations

Catchment ID	Locality	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI	PMP
6	Haddock Drive	8%	7%	7%	7%	9%
8	Kirkwood Rd. Police Creek	6%	7%	6%	6%	9%
9	Police Creek Outlet	2%	4%	4%	4%	4%
17	Tondoon Creek Outlet	4%	4%	4%	4%	3%
20	Police Creek u/s of Cathurbie Creek confluence	2%	2%	2%	4%	3%
27	Cathurbie Creek Outlet	0%	-1%	-1%	-2%	0%
32	Glenlyon Rd. Tigalee Creek	0%	0%	0%	0%	0%
37	Whitney St. Tigalee Creek	0%	0%	0%	0%	0%
39	Tigalee Creek Outlet	0%	0%	0%	0%	0%
44	Kirkwood Rd. Briffney Creek	37%	37%	39%	39%	36%
47	Penda Ave. Briffney Creek	13%	13%	13%	14%	17%
49	Dawson Highway Briffney Creek	11%	12%	13%	14%	16%
51	Briffney Creek Outlet	9%	9%	9%	9%	7%

Catchment ID	Locality	20 Year ARI	50 Year ARI	100 Year ARI	500 Year ARI	PMP
61	Auckland Creek Outlet	2%	2%	1%	1%	1%

The following trends are evident:

- The Auckland Creek catchment has many contributing sub catchments, with correspondingly varying times to peak, resulting in the hydrograph at the creek outlet (61) being well distributed with little increase in peak flow evident.
- Briffney Creek flows increase by approximately 10% to 15% at the outlet compared to 0% for Tigalee Creek. This may be due to the difference in the extent of urbanisation in the existing development case where the Tigalee Creek catchment is almost completely urbanised compared to the mostly rural upper reaches of the Briffney Creek catchment.
- Similarly most of the flow increases occur in the upper reaches of the various catchments where the most dramatic changes to land uses are evident.

5. Hydraulic Model Development

5.1 Selection of Software

The main purposes of hydraulic modelling are to calculate flood levels and velocities and to determine flow patterns within watercourses (creeks, rivers and floodways) and their floodplains. In addition, models allow the prediction of the likely effect of future development activities on flood risk in the catchment. Generally the flow patterns in the Auckland Creek study area are one-dimensional (1D) within the creek channels, and two-dimensional (2D) across floodplains, particularly towards the downstream end of the study area. Therefore, the hydrodynamic modelling software package TUFLOW (Syme, 2005), which can simulate both 1D and 2D flow characteristics, was considered appropriate for this study.

5.2 Digital Elevation Model

The topography of the floodplain and bathymetry of the waterways form the basis of any hydraulic model. Generally, they are represented by a digital elevation model (DEM) for a 2D model and distance-elevation cross-sections perpendicular to the direction of flow for a 1D model.

Topographic data used in the development of the hydraulic model was obtained from the following sources:

- Digital contours (1m interval) from Gladstone City Council;
- Cross-sections survey by Gladstone City Council for the waterways upstream of Lake Callemondah;
- Digital and hardcopy data of hydrographic survey from Queensland Transport; and
- Additional cross-section survey to fill in the missing data.

Reference should be made to Figure 8, which shows the location of each data source.

A DEM for the hydraulic modelling area was created primarily using the above mentioned 1 m contours. The hydrographic data and cross-sections for the lower reaches of Auckland and Briffney Creeks were also incorporated into the DEM.

5.3 1D/2D Model Development

The extent of the 2D hydrodynamic modelling area is approximately 31 km^2. In the upper reaches of the modelling area, narrow streams were represented by a 1D-modelling network, which was dynamically linked to the 2D modelling domain. The floodplains of the upper reaches, and the relatively flat portion of Auckland Creek downstream of the North Coast Rail line, were represented by the 2D modelling domain.

Figure 9 shows the schematic representation of the 1D/2D model set-up.

The grid size of the 2D domain was 10 m, providing good representation of the floodplain of Auckland Creek and its tributaries. However, the narrow channels of the upper reaches could not be well represented by this grid, and the grid size could not be reduced any further without incurring excessive computation time for the model runs. Therefore the upper reaches streams were modelled using the 1D component of TUFLOW. The surveyed cross-sections were used to represent mainly bank-to-bank portion of the flow paths with the rest of the flowpath outside the stream banks represented by the 2D component of TUFLOW. The 1D network was dynamically linked to the 2D domain.

The channels downstream of the North Coast rail line were relatively wide and were well represented by the 2D modelling grids (generally 3 or more grids within the bank-to-bank flow paths).

5.4 Roughness of Channels & Floodplains

Table 21 summarises various waterway and floodplain conditions found in the catchment and the associated roughness values adopted in the hydraulic model. The spatial distribution of roughness is shown in Figure 10 (also listed in Table 21).

Table 21 Adopted Roughness Values for TUFLOW Model

Material	Assigned Roughness (Manning's n) Value
Water without any significant vegetation	0.03
Mangroves/swamp/medium to high density vegetation	0.10
Grassed areas	0.05
Ephemeral creek beds	0.04 to 0.10
Bush & trees	0.15
Properties (Residential, commercial and business)	0.20
Road reserve	0.02
Bare soil, ash deposit, etc	0.03

5.5 Boundary Conditions

Boundary conditions are required to drive hydrodynamic models. Typically, these consist of flow boundaries and water level boundaries. Flow boundaries are applied at the upstream end of each tributary, whilst a single downstream water level (tailwater level) defines the receiving water. The tailwater level can be constant or dynamic, with the selection typically based on the type of model being used, and the duration of the storm events under consideration.

5.5.1 Inflows

Inflows have been defined at the upstream end of each tributary, and at several locations within the model extent. These are defined as external and internal inflows respectively. Inflows for each event were generated using the URBS model, with locations illustrated in Figure 9.

5.5.2 Tide

The maximum tidal range for Gladstone is (4.69 m). Details of tidal planes are provided in the following table.

Table 22 Tidal Planes for Gladstone (Queensland Transport, 2001)

Tidal Plane	Description	Value (m AHD)
LAT	Lowest Astronomical Tide	-2.27
MLWS	Mean Low Water Spring	-1.60
MSL	Mean Sea Level	0.08
MHWS	Mean High Water Springs	1.64
HAT	Highest Astronomical Tide	2.42

5.5.3 Storm Tide

Most coastal areas of Queensland are subject to cyclone activity, which can result in storm tide levels that exceed those of the HAT. The predicted 100 yr ARI storm tide levels for Gladstone (Queensland Climate Change and Community Vulnerability to Tropical Cyclones, Qld Gov) are as follows:

- 2.82 m AHD (without Greenhouse effect);
- 3.33 m AHD (with Greenhouse effect).

5.5.4 Adopted Tailwater Levels

The Brief called for the assessment of flooding for “normal tide” and a range of water levels at 0.5m intervals. However, owing to the agreed change in model platform (i.e. use of the dynamic TUFLOW model, rather than the steady state HEC-RAS model), a reduced set of tailwater conditions was agreed.

The adopted tailwater level used during modelling was 2.42 m AHD (HAT) for the 20 year, 50 year, 100 year ARI and PMF flood events. This is consistent with the approach taken in the Calliope River Flood Study concurrently undertaken for Calliope Shire Council.

In addition, as detailed in Section 6.5, an analysis was undertaken to determine the sensitivity of the predicted flood inundation depths and velocities to changes in tailwater level. It was concluded that the tailwater level significantly affected results only as far upstream as the Callemondah Weir, which acts as a barrier to tidal inundation.

5.6 Modelling Waterway Structures

Waterway structures may be defined as a man-made feature that passes through or over a waterway and, in most cases, causing a constriction to flow. In this case, the primary structures consist of culverts, bridges, drop structures, weirs and pipe crossings. Of these, the drop structures are generally drowned out during flood flows (i.e. are of little relevance during high flow events). Pipe crossings can be similarly regarded in most cases, though they do tend to exert an influence for lower ARI design events. A list of structures was provided in Chapter 2, with full details provided in Appendix D.

The simulation of these structures can be complicated, with careful consideration of invert levels, obvert levels, road levels, and the potential for bypass required. Model instabilities are also relatively common in the vicinity of structures, necessitating an additional level of checking.

Waterway structures have been modelled within the 1D domain for relatively small structures (eg, culverts, smaller bridges) and within the 2D domain for relatively large structures (eg, bridges and spillways). Figure 11 illustrates indicative locations of the structures modelled.

Within the model culverts can be represented as either rectangular or circular. A range of different flow regimes is simulated with flow possible in either direction. Adverse slopes are accounted for and flow may be subcritical or supercritical.

Road crests and spillways are represented in TUFLOW as weirs. Weirs modelled in 1D use a standard weir flow formula (“Hydraulics of Bridge Waterways, 1978”). The weir is assumed to be broad-crested, such as a causeway or an embankment. Weirs have three flow regimes consisting of zero flow (dry), upstream controlled flow (unsubmerged) and downstream controlled flow (submerged).

5.7 Model Verification

Model verification typically requires a comparison of predicted flood levels to those recorded historical flood events. However, in this case, the only recorded data related to Cyclone Beni, with approximate levels at no more than five locations provided. This was insufficient to allow a calibration of the hydraulic model, though a comparison of predicted 50 yr levels to Cyclone Beni flood levels and to road crossings considered likely to be overtopped was made. Table 23 provides a summary of the predicted 50 yr ARI flood and the corresponding Cyclone Beni levels, which have been estimated as consistent with a 1 in 50 year ARI 48 hour storm.

Table 23 Comparisons between estimated 50 Yr ARI and Cyclone Beni recorded flood levels

Structure	Structure ID	Weir Level	TUFLOW 50 yr Flood Level (existing)	Cyclone Beni Flood Level	Difference
		(m AHD)	(m AHD)	(m AHD)	(m)
Witney St Crossing	10	13.89	13.70	11.73	1.79
Mercury St. Crossing	11	16.2	17.25	17.36	-0.11
Kirkwood Rd Crossing #5	19	22.21	19.09	19.5	-0.41
Kirkwood Rd Crossing #6	50	25.24	20.95	19.2	1.75
Penda Ave. Bebo Arch	32	11.3	9.75	9.15	0.60

Table 23 shows that the predicted TUFLOW flood heights compare well with the recorded Cyclone Beni flood heights with the exception of the Witney St Crossing and Kirkwood Rd #6. However, there are a number of uncertainties that qualify direct comparison between the recorded Cyclone Beni flood heights and the predicted flood heights:

- Possible reduction of culvert capacity due to debris blockage;
- Recorded Cyclone Beni flood heights were based on flood and debris marks, which may not record actual maximum flood heights.
- Rainfalls used in modelling have no relation to those that occurred during Cyclone Beni.

Table 24 provides a comparison between the TUFLOW predicted flood levels to those reported by Cox Andrews, using the HECRAS hydraulic model, in previous flood studies. The comparisons have been made where culvert dimensions have remained similar. Whilst results are also similar in most locations, the following points qualify the comparison:

- It is unknown what storm duration was used in the HECRAS models, therefore differences in peak flood heights may be expected;
- The Haddock Road culvert details were significantly different between the TUFLOW (GHD) and HECRAS (Cox Andrews) models. Upstream culvert invert levels were 26.96m AHD and 26.3m AHD respectively; with adopted road crest levels 30m AHD and 29.5m AHD, respectively). Additionally the TUFLOW model predicted a peak

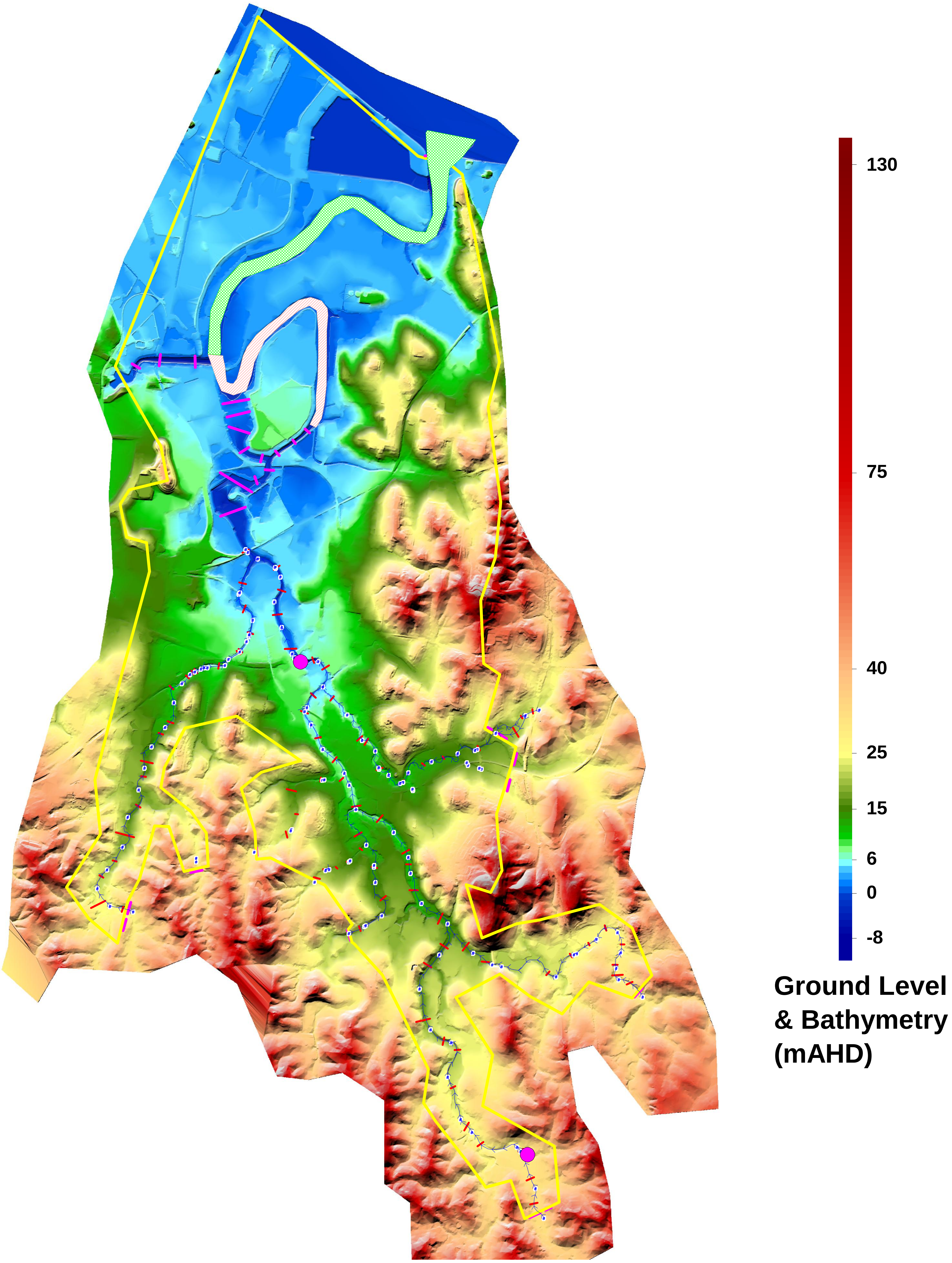
downstream flood level of 30.8, effectively drowning the culvert, with the HECRAS model tailwater being significantly lower than this.

Table 24 Comparison of Predicted Flood Levels

Structure ID	Location	Chainage	Description		TUFLOW	HECRAS
					(mAHD)	(mAHD)
19	Kirkwood Road Crossing (#5)	400, 500	Pipe Culvert	4/ 1.35	19.33	19.78
50	Kirkwood Road Crossing (#6)	1300	Pipe Culvert	4/ 2.1	21.12	21.09
10	Witney St Crossing	1250	Box Culvert	2/ 3.0 x 3.6	13.7	13.11
11	Mercury Street Crossing	2160	Box Culvert	5/ 2.1w x 1.8H	17.25	16.92
28	Haddock Road Crossing	15600	Box Culvert	3/ 3.6w x 2.1h	31.53	29.95
34	Kirkwood Road Crossing (#1)	3700	Box Culvert	3/ 3.0w x 2.1h	23.23	23.1

*All flood heights in the above table are 100 yr ARI, unless otherwise specified.

From the above comparisons we can conclude that the TUFLOW model, whilst different in nature (i.e. it is 2 dimensional, dynamic, larger in extent and is subject to different inflows), does appear to have consistency with previous studies



GHD
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LEGEND

- Original Cross-Section Survey by GCC
- Additional Cross-Section Survey by GCC
- 🟩 Digital Hydrographic Data from QT
- ☐ Hard Copy Hydrographic Data from QT
- ● Stream Gauge
- 1D/2D Model Boundary
- ☐ 2D Modelling Area
- 1D Node
- 1D Flowpath

North

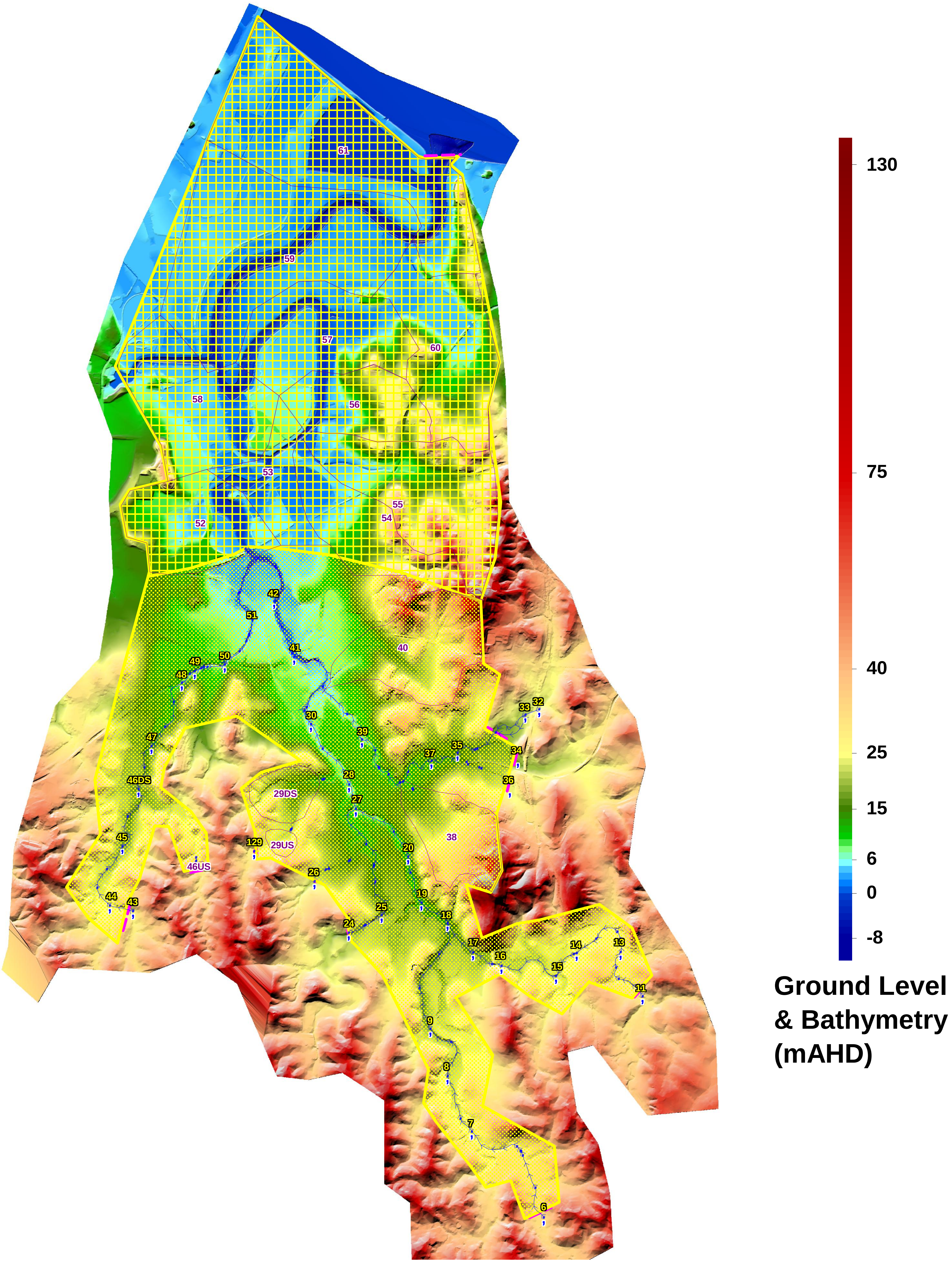
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 8

Data Map



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LEGEND

▭ Inflow to 2D Domain

, Inflow to 1D Network

• 1D Network Node

→→→ 1D Network Flowpath

1D/2D Modelling Areas

— — Flow and Level Boundaries

▨ 1D/2D Combined

▦ 2D Modelling Only

North

0 0.5 1 1.5 2

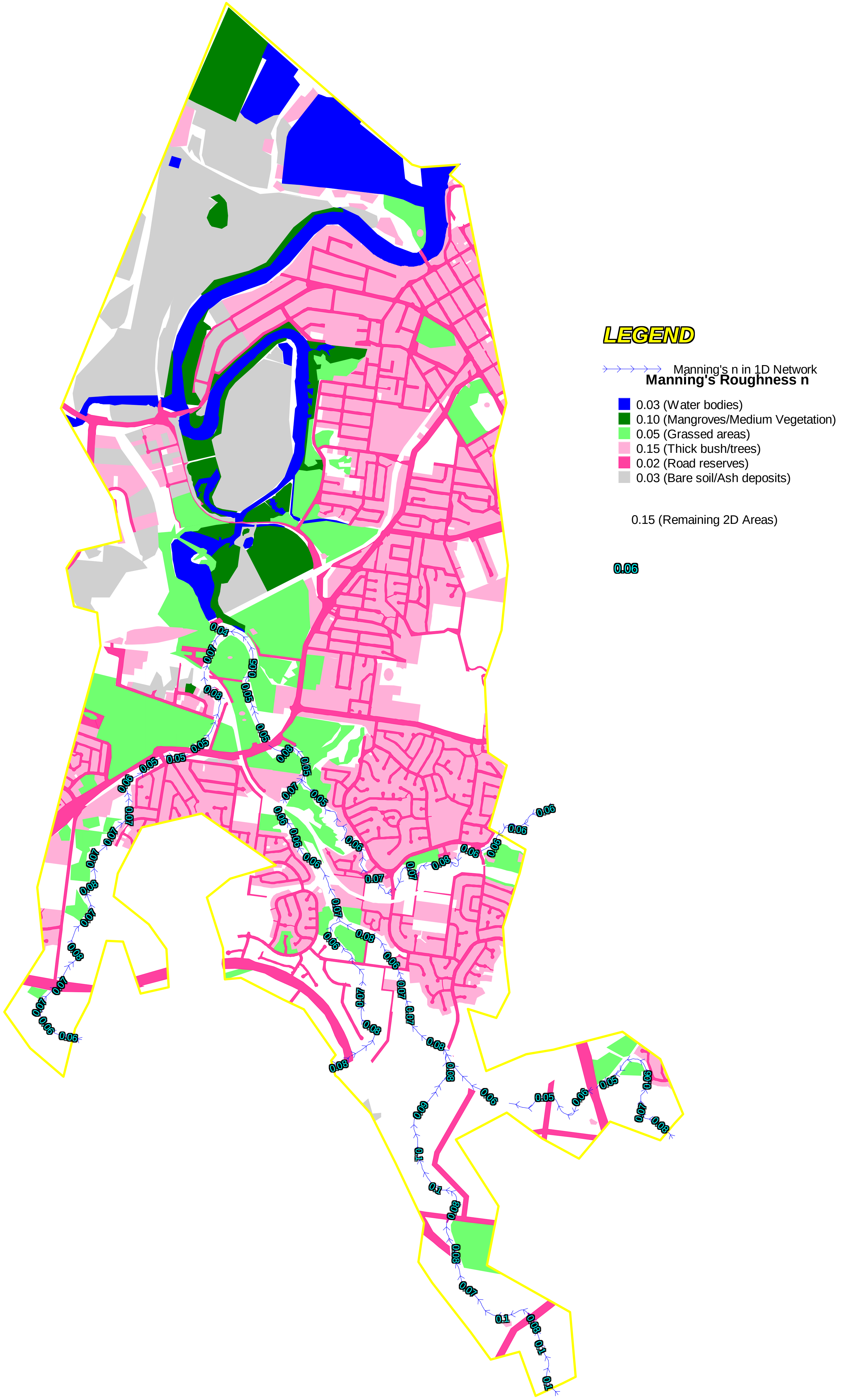
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 9

TUFLOW 1D/2D Model Configuration



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North

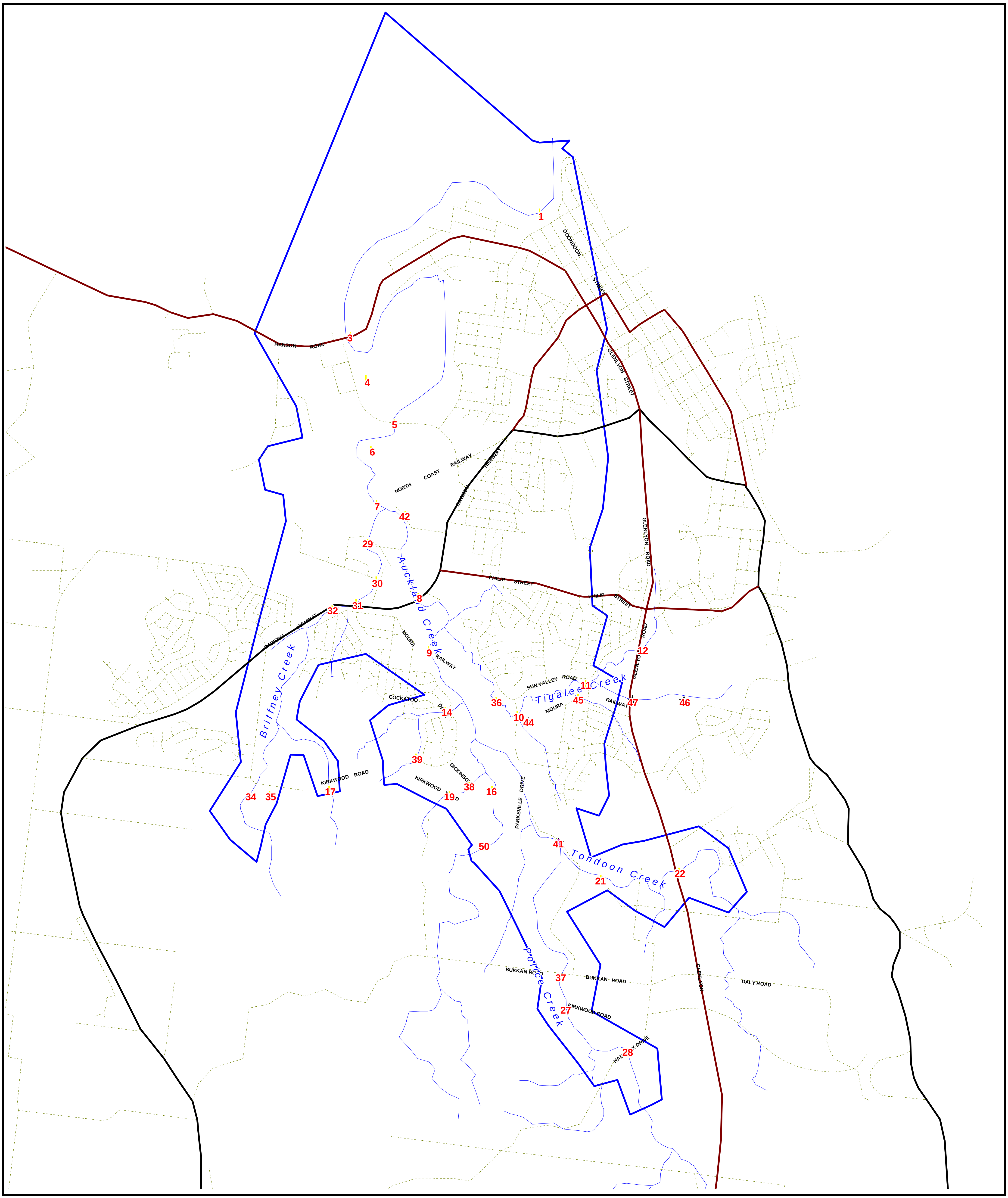
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 10

Manning's Roughness for Hydraulic Model



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LEGEND

- Waterways
- ! Waterway Structures Modelled
- ▭ Region

North

0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 11

Waterway structures modelled Existing & Ultimate Hydrology (Refer to Table 3 for Structure description)

6. Flood Modelling Results

6.1 Introduction

This chapter provides a summary of the predicted flood levels arising from simulation of the Auckland Creek system. The emphasis of results relates to predictions for existing and ultimate catchment conditions, with a discussion on mitigation options and the mapping of predicted flood levels provided in subsequent chapters.

6.2 Modelled Events

A large number of model runs have been completed, as defined below:

- 5 design events (20 yr, 50 yr, 100 yr, 500 yr, and PMF)
- 2 catchment conditions (existing and ultimate development)
- 2 tailwater conditions (HAT and storm tide). However, HAT was adopted as the tailwater level for all events after consideration of the results of sensitivity analysis (refer section 6.5) and following consultation with Council.

In addition, several mitigation options have been modelled, primarily focusing on the 100 yr ARI event.

6.3 Model Results

Modelling results at key locations are summarised in this section, with flooding maps provided in Chapter 10.

6.3.1 Existing Conditions

Predicted peak flood levels, for each of the five design events, are presented in Table 25.

Table 25 Predicted existing peak flood levels

Site #	Description	Ch	20 yr ARI	50 yr ARI	100 yr ARI	500 yr ARI	PMP
		(m)	(mAHD)	(mAHD)	(mAHD)	(mAHD)	(mAHD)
Auckland Creek							
1	Marina Bridge Creek Outlet	800	2.44	2.45	2.46	2.48	2.80
3	Hanson Road (Clinton) Bridge	3840	2.61	2.66	2.74	2.99	4.54
5	Blain Drive Bridge	7060	3.38	3.57	3.81	4.34	5.92
6	Lake Callemondah Weir	7550	3.86	4.08	4.33	4.77	6.04

Site #	Description	Ch	20 yr ARI	50 yr ARI	100 yr ARI	500 yr ARI	PMP
7	North Coast Railway Crossing	8300	4.12	4.48	4.78	5.41	6.96
8	Dawson Highway Bridge (Golf Course)	9480	5.56	5.86	6.12	6.75	8.56
Tigalee Creek							
10	Witney St Crossing	1250	13.50	13.70	14.00	14.45	15.78
11	Mercury Street Crossing	2160	17.17	17.24	17.36	17.61	18.51
36	Links Court Bridge	1000	10.47	10.62	10.87	11.54	14.11
Emmadale Creek							
14	Cockatoo Drive Crossing	200	11.88	11.88	11.99	12.26	14.03
39	Emmadale Drive	900	21.49	21.47	21.55	21.59	22.23
Cathurbie Creek Tributary							
16	Parksville Drive (#2)	200	14.98	15.04	15.09	15.20	15.62
19	Kirkwood Road Crossing (#5)	500	18.98	19.08	19.31	19.83	21.98
Cathurbie Creek							
50	Kirkwood Road Crossing (#6)	1300	20.68	20.90	21.08	21.38	22.93
38	Parksville Drive (#1)	600	14.80	14.90	15.07	15.36	16.66
Tondoon Creek							
21	Tondoon Reservoir Outlet	350	23.09	23.23	23.40	23.78	25.26
22	Glenlyon Road Crossing (#3)	1580	24.81	24.91	25.04	25.34	26.36
Police Creek							
28	Haddock Drive Crossing	15600	31.10	31.27	31.46	31.82	33.12
Briffney Creek							
30	Callemondah Ave. (Bebo Arch)	850	5.59	6.00	6.89	7.18	7.69
31	Dawson Hwy Road Bridge	1180	8.44	8.55	8.68	8.96	9.60

Site #	Description	Ch	20 yr ARI	50 yr ARI	100 yr ARI	500 yr ARI	PMP
32	Penda Avenue (Bebo Arch)	1450	9.40	9.57	9.81	10.27	12.21

6.3.2 Ultimate Catchment Development

As discussed previously, changes to runoff have been predicted based on the ultimate extent of development, as indicated by Council's Strategic Plan. The revised hydrographs have been entered into the TUFLOW model, and rerun for each of the 5 design events. Table 26 provides a summary of the revised flows. A comparison of these predictions with those for the existing conditions case is provided in the following section.

Table 26 Predicted ultimate peak flood levels

Site #	Description	Ch (m)	20 yr ARI	50 yr ARI	100 yr ARI	500 yr ARI	PMP
			(mAHD)	(mAHD)	(mAHD)	(mAHD)	(mAHD)
Auckland Creek							
1	Marina Bridge Creek Outlet	800	2.44	2.45	2.46	2.49	2.81
3	Hanson Road (Clinton) Bridge	3840	2.62	2.67	2.76	3.02	4.59
5	Blain Drive Bridge	7060	3.41	3.61	3.85	4.38	5.96
6	Lake Callemondah Weir	7550	3.9	4.12	4.36	4.8	6.14
7	North Coast Railway Crossing	8300	4.21	4.52	4.82	5.45	7.09
8	Dawson Highway Bridge (Golf Course)	9480	5.66	5.91	6.17	6.84	8.62
Tigalee Creek							
10	Witney St Crossing	1250	13.50	13.70	14.00	14.45	15.78
11	Mercury Street Crossing	2160	17.18	17.25	17.37	17.62	18.52
36	Links Court Bridge	1000	10.49	10.63	10.87	11.55	14.12
Emmadale Creek							
14	Cockatoo Drive	200	11.88	11.89	12.01	12.23	14.11

Site #	Description	Ch (m)	20 yr ARI (mAHD)	50 yr ARI (mAHD)	100 yr ARI (mAHD)	500 yr ARI (mAHD)	PMP (mAHD)
	Crossing						
39	Emmadale Drive	900	21.48	21.53	21.54	21.62	22.24
Cathurbie Creek Tributary							
16	Parksville Drive (#2)	200	14.99	15.02	15.04	15.22	15.65
19	Kirkwood Road Crossing (#5)	500	19.00	19.09	19.33	19.88	22.01
Cathurbie Creek							
50	Kirkwood Road Crossing (#6)	1300	20.81	20.95	21.12	21.41	23.11
38	Parksville Drive (#1)	600	14.80	14.91	15.08	15.40	16.76
Tondoon Creek							
21	Tondoon Reservoir Outlet	350	23.11	23.25	23.43	23.81	25.30
22	Glenlyon Road Crossing (#3)	1580	24.85	24.94	25.08	25.39	26.41
Police Creek							
28	Haddock Drive Crossing	15600	31.19	31.34	31.53	31.88	33.20
Briffney Creek							
30	Callemondah Ave.(Bebo Arch)	850	6.09	6.83	7.01	7.25	7.75
31	Dawson Hwy Road Bridge	1180	8.56	8.63	8.77	9.06	9.71
32	Penda Avenue (Bebo Arch)	1450	9.59	9.75	9.95	10.44	12.30

6.3.3 Impact of Ultimate Development

Table 27 presents the difference between the existing and ultimate flood levels for the nominated design events.

Table 27 Difference between predicted existing and ultimate peak flood levels

Site #	Description	Ch (m)	20 yr ARI	50 yr ARI	100 yr ARI	500 yr ARI	PMP
Auckland Creek							
1	Marina Bridge Creek Outlet	800	0.00	0.00	0.00	0.01	0.01
3	Hanson Road (Clinton) Bridge	3840	0.01	0.01	0.02	0.03	0.05
5	Blain Drive Bridge	7060	0.03	0.04	0.04	0.04	0.04
6	Lake Callemondah Weir	7550	0.04	0.04	0.03	0.03	0.10
7	North Coast Railway Crossing	8300	0.09	0.04	0.05	0.04	0.12
8	Dawson Highway Bridge (Golf Course)	9480	0.10	0.05	0.06	0.09	0.06
Tigalee Creek							
10	Witney St Crossing	1250	0.00	0.01	0.01	0.00	0.01
11	Mercury Street Crossing	2160	0.01	0.01	0.00	0.01	0.01
36	Links Court Bridge	1000	0.02	0.00	0.01	0.01	0.01
Emmadale Creek							
14	Cockatoo Drive Crossing	200	0.00	0.00	0.01	-0.03	0.07
39	Emmadale Drive	900	-0.02	0.06	-0.01	0.03	0.01
Cathurbie Creek Tributary							
16	Parksville Drive (#2)	200	0.01	-0.02	-0.05	0.02	0.03
19	Kirkwood Road Crossing (#5)	500	0.01	0.02	0.02	0.05	0.02
Cathurbie Creek							
50	Kirkwood Road Crossing (#6)	1300	0.12	0.05	0.04	0.03	0.18

38	Parksville Drive (#1)	600	0.00	0.01	0.01	0.04	0.09
Tondoon Creek							
21	Tondoon Reservoir Outlet	350	0.02	0.02	0.02	0.03	0.05
22	Glenlyon Road Crossing (#3)	1580	0.04	0.04	0.03	0.05	0.04
Police Creek							
28	Haddock Drive Crossing	15600	0.09	0.07	0.06	0.06	0.09
Briffney Creek							
30	Callemondah Ave. (Bebo Arch)	850	0.50	0.83	0.12	0.08	0.05
31	Dawson Hwy Road Bridge	1180	0.12	0.08	0.09	0.10	0.10
32	Penda Avenue (Bebo Arch)	1450	0.19	0.18	0.14	0.18	0.10

It can be seen that at most locations, peak water levels show little change. This is attributable to a combination of factors, including:

- Catchment and waterway characteristics. The upper parts of the catchment are relatively steep, resulting in quick travel times for all events, whilst the lower part of the catchment (i.e. downstream of Lake Callemondah) is dominated by the receiving water level of Gladstone Harbour.
- Several sub catchments (i.e. Tigalee, Emmadale Creek, and Auckland Creek), show little increase in imperviousness from the existing to fully developed scenarios. This is reflected in limited increases in peak flow predictions (Refer Chapter 4) and carries through to a low increase in predicted flood levels.

6.4 Flood Velocities

Table 28 compares the peak 100-year ARI flood velocities for the existing and ultimate cases.

Table 28 Peak 100 yr ARI flood velocities for existing and ultimate cases

Site #	Description	Ch (m)	Existing	Ultimate
			(m/s)	(m/s)
Auckland Creek				
1	Marina Bridge Creek Outlet	800	0.82	0.84

Site #	Description	Ch (m)	Existing	Ultimate
			(m/s)	(m/s)
3	Hanson Road (Clinton) Bridge	3840	1.44	1.48
5	Blain Drive Bridge	7060	2.06	2.06
6	Lake Callemondah Weir	7550	1.18	1.19
7	North Coast Railway Crossing	8300	1.90	1.95
8	Dawson Highway Bridge (Golf Course)	9480	1.87	1.92
Tigalee Creek				
10	Witney St Crossing	1250	3.40	3.42
11	Mercury Street Crossing	2160	1.53	1.53
36	Links Court Bridge	1000	3.61	3.63
Emmadale Creek				
14	Cockatoo Drive Crossing	200	1.01	0.99
39	Emmadale Drive	900	2.02	1.95
Cathurbie Creek Tributary				
16	Parksville Drive (#2)	200	1.08	1.08
19	Kirkwood Road Crossing (#5)	500	4.87	4.87
Cathurbie Creek				
50	Kirkwood Road Crossing (#6)	1300	3.73	3.77
38	Parksville Drive (#1)	600	4.26	4.26
Tondoon Creek				
21	Tondoon Reservoir Outlet	350	2.53	2.54
22	Glenlyon Road Crossing (#3)	1580	4.19	4.23
Police Creek				
28	Haddock Drive Crossing	15600	2.79	2.82
Briffney Creek				
30	Callemondah Ave. (Bebo Arch)	850	3.94	3.90
31	Dawson Hwy Road Bridge	1180	1.84	1.98
32	Penda Avenue (Bebo Arch)	1450	2.91	3.18

Peak 100 yr ARI flood velocities compared in Table 28 demonstrates negligible difference between the existing and ultimate cases. However several trends may be noted:

- Peak velocities generally decreased from the upper to the lower reaches of the Auckland Creek waterway. This follows the flattening of gradient as it progresses downstream from the steeper upper reaches to the flatter lower reaches.
- Peak velocities are low in the tidally influenced zone of the catchment i.e. below the Lake Callemondah weir.
- High velocities occurred in waterway crossings (culverts, bridges) where flow has been constricted.
- Lower than expected velocities in some of the ephemeral upper catchment reaches may be due to the retarding influence of the heavy shrub and undergrowth present, thereby countering the effect of steep waterway gradients.

6.5 Sensitivity Analysis

A sensitivity analysis was conducted with respect to the assumed tailwater levels. In this case, a higher tailwater level, attributable to a major storm tide, has been assumed. The resultant water levels in the lower parts of the catchment are tabulated below.

Table 29 Comparison between 100-year ARI peak flood levels modelled with storm tide and HAT tailwater levels

Site #	Description	Ch (m)	HAT	Storm Tide
			(2.42 m AHD)	(3.33 m AHD)
Auckland Creek				
1	Marina Bridge Creek Outlet	800	2.46	3.34
3	Hanson Road (Clinton) Bridge	3840	2.76	3.58
5	Blain Drive Bridge	7060	3.85	4.26
6	Lake Callemondah Weir	7550	4.36	4.55
7	North Coast Railway Crossing	8300	4.82	4.96
8	Dawson Highway Bridge (Golf Course)	9480	6.17	6.19
Tigalee Creek				
10	Witney St Crossing	1250	14.00	13.97
11	Mercury Street Crossing	2160	17.37	17.36
36	Links Court Bridge	1000	10.87	10.83

Site #	Description	Ch (m)	HAT (2.42 m AHD)	Storm Tide (3.33 m AHD)
Emmadale Creek				
14	Cockatoo Drive Crossing	200	12.01	12.00
39	Emmadale Drive	900	21.54	21.55
Cathurbie Creek Tributary				
16	Parksville Drive (#2)	200	15.08	15.11
19	Kirkwood Road Crossing (#5)	500	19.33	19.33
Cathurbie Creek				
38	Parksville Drive (#1)	600	15.08	15.04
Tondoon Creek				
21	Tondoon Reservoir Outlet	350	23.43	23.43
22	Glenlyon Road Crossing (#3)	1580	25.08	25.07
Police Creek				
28	Haddock Drive Crossing	15600	31.53	31.52
Briffney Creek				
30	Callemondah Ave. (Bebo Arch)	850	7.01	7.01
31	Dawson Hwy Road Bridge	1180	8.77	8.77
32	Penda Avenue (Bebo Arch)	1450	9.95	9.95

Table 29 shows that the predicted peak flood levels only vary noticeably downstream of the North Coast Railway. This is attributable to Callemondah Weir effectively acting as the upstream extent of tidal influence, hence reducing the model's sensitivity to tailwater conditions upstream of the weir. Lake Callemondah is drowned out for the case based on an elevated (storm-tide) tailwater.

7. Performance of Waterway Structures

7.1 Objectives

One of the key aims of the flood risk study is to address evacuation routes, and to determine which road crossings may be cut by flood waters. This has been based on a review of overtopped structures, and the depth of overtopping.

7.2 Desired Standard of Service

Different classifications of roads are generally required to have different design immunity levels. In the Gladstone area, these have been defined as follows:

Major Road	50 year ARI cross drainage
Minor Road	10 year ARI cross drainage

Exceptions to this would be where a road is overtopped, and provides the only access for a given area. In this case, a higher level of immunity may be desired for reasons of safety (e.g. access during a flooding event); and where 100 year ARI backwaters inundate upstream properties. In this case a higher cross drainage capacity may be required to ensure flooding does not occur.

7.3 Structure Performance

The flood levels, velocities and flow rates across various waterway structures for the 50 and 100 year ARI flood events are listed in Table 30 and Table 31 respectively. A total of 35 structures have been included in the hydraulic model. Of these, five are overtopped during the 50 yr ARI event, and a further one is overtopped during the 100 yr ARI event.

Table 32 then provides a summary for each of the overtopped structures for the 100 yr ARI event, in terms of whether or not a road upgrade is likely to be needed.

Note that in the tables, peak flows are reported for each of the flow paths in operation, namely:

- Flow through the structure
- Flow over the structure, and
- Flow bypassing (around) the structure.

Given that these do not necessarily occur at the same time, the combined peak flow may be less than the sum of the individual elements.

Peaks relating to flows through and over the structures have been provided from the 1D part of the flood model, whereas flows bypassing the structure are generated within the 2D domain.

Furthermore, flood levels are reported immediately upstream and downstream of each structure.

Table 30 50-Year ARI structure performance

Site #	Description	Ch (m)	Type	Dimension (m)	Weir/Crest Level[3] (mAHD)	U/S Peak Flood Level[4] (mAHD)	D/S Peak Flood Level (mAHD)	Flow Through Structure (m^3/s)	Flow Over Structure (1D) (m^3/s)	Flow Around Structure (2D) (m^3/s)	Over Topping Depth (m)	Outlet Velocity (m/s)
1	Marina Bridge Creek Outlet	800	Bridge	Arched 140m, 8 Spans	n/a	2.45	2.43	467	n/a	n/a	n/a	0.7
3	Hanson Road (Clinton) Bridge	3840	Bridge	100m, 4 Span	n/a	2.68	2.65	457	n/a	n/a	n/a	1.3
4	Ash Pond Causeway		Weir	200m long, 6m wide*	n/a	3.46	3.44	163	n/a	n/a	n/a	0.5
5	Blain Drive Bridge	7060	Bridge	60m, 4 Span	n/a	3.73	3.56	436	n/a	n/a	n/a	2.2
6	Lake Callemondah Weir	7550	Weir	200 m long	XS centre IL 2.603	4.1	4.05	472	n/a	n/a	n/a	1.3
7	North Coast Railway Crossing	8300	Bridge	60m, 5 Span	7.52	4.52	4.31	479	n/a	n/a	n/a	1.6
42	Blain Park Pedestrian Bridge	8600	Bridge	40m 1 Span	4.25	4.75	4.59	203	24	39	0.5	1.8

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[3] Weir/crest levels for major bridges downstream of Lake Callemondah significantly higher than PMF levels.

[4] Reported flood levels apply immediately upstream and downstream of structure.

Site #	Description	Ch (m)	Type	Dimension (m)	Weir/Crest Level[3] (mAHD)	U/S Peak Flood Level[4] (mAHD)	D/S Peak Flood Level (mAHD)	Flow Through Structure (m^3/s)	Flow Over Structure (1D) (m^3/s)	Flow Around Structure (2D) (m^3/s)	Over Topping Depth (m)	Outlet Velocity (m/s)
8	Dawson Highway Bridge (Golf Course)	9480	Bridge	Twin 5 Span Bridges Total length 65m	7.386	5.91	5.87	397	n/a	n/a	n/a	1.8
36	Links Court Bridge	1000	Bridge	15 m, 1 Span	12.45	10.63	10.51	80	n/a	n/a	n/a	3.3
10	Witney St Crossing	1250	Box Culvert	2/ 3.0 x 3.6	13.89	13.7	12.64	71	n/a	7	n/a	3.2
11	Mercury Street Crossing	2160	Box Culvert	5/ 2.1w x 1.8H	16.183	17.25	17.06	28	40	12	1.07	1.7
14	Cockatoo Drive Crossing	200	Pipe Culvert	3/ 1.8 dia	12.08	11.88	11.58	16	n/a	1	n/a	2
39	Emmadale Drive	900	Pipe Culvert	4/ 1.2	22.62	21.54	21.46	5	n/a	5	n/a	1
16	Parksville Drive (#2)	400, 200	Pipe Culvert	2/ 1.65	14.7	14.91	14.08	39	2	25	0.21	4.1
19	Kirkwood Road Crossing (#5)	400, 500	Pipe Culvert	4/ 1.35	22.21	19.09	18.29	9	n/a	n/a	n/a	4.9
50	Kirkwood Road Crossing (#6)	1300	Pipe Culvert	4/ 2.1	25.24	20.95	19.69	51	n/a	n/a	n/a	3.7

Site #	Description	Ch (m)	Type	Dimension (m)	Weir/Crest Level[3] (mAHD)	U/S Peak Flood Level[4] (mAHD)	D/S Peak Flood Level (mAHD)	Flow Through Structure (m^3/s)	Flow Over Structure (1D) (m^3/s)	Flow Around Structure (2D) (m^3/s)	Over Topping Depth (m)	Outlet Velocity (m/s)
38	Parksville Drive (#1)	600	Box Culvert	3/ 3.048w x 2.134h	15.4	15.02	14.95	5	n/a	6	n/a	1.1
21	Tondoon Reservoir Outlet	350	Dam Wall with Spillway	5m wide, 30 m long	XS centre IL 21.14	23.25	21.48	60	n/a	n/a	n/a	2.4
22	Glenlyon Road Crossing (#3)	1580	Box Culvert	3/ 3.6w x 2.1h	25.81	24.94	24.48	49	n/a	5	n/a	2.3
28	Haddock Drive Crossing	15600	Box Culvert	3/ 3.6w x 2.1h	30	31.34	30.58	63	65	n/a	1.34	2.8
30	Callemondah Drive (Bebo Arch)	850	Bebo Arch	L12	6.78	6.83	5.58	107	11	n/a	0.05	3.9
31	Dawson Hwy Road Bridge	1180	Bridge	2 x 30 m, 3 Span	11.436	8.63	8.52	109	n/a	n/a	n/a	1.8
32	Penda Avenue (Bebo Arch)	1450	Bebo Arch & Box Culvert	M12	11.3	9.75	9.67	100	n/a	n/a	n/a	2.8

Table 31 100-Year ARI structure performance

Site #	Description	Ch	Type	Dimension	Weir/Crest Level	U/S Peak Flood Level[5]	D/S Peak Flood Level	Flow Through Structure	Flow Over Structure (1D)	Flow[#] Around Structure (2D)	Over Topping Depth	Outlet Velocity
		(m)		(m)	(mAHD)	(mAHD)	(mAHD)	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m/s)
1	Marina Bridge Creek Outlet	800	Bridge	Arched 140 m 8 Span	n/a	2.46	2.43	568	n/a	n/a	n/a	0.9
3	Hanson Road (Clinton) Bridge	3840	Bridge	100m, 4 Span	n/a	2.76	2.72	552	n/a	n/a	n/a	1.5
4	Ash Pond Causeway		Weir	200m long 6m wide	n/a	3.64	3.6	205	n/a	n/a	n/a	0.6
5	Blain Drive Bridge	7060	Bridge	60m, 4 Span	n/a	3.99	3.76	508	n/a	n/a	n/a	2.2
6	Lake Callemondah Weir	7550	Weir	200 m long	XS centre IL 2.603	4.33	4.3	544	n/a	n/a	n/a	1.6
7	North Coast Railway Crossing	8300	Bridge	60m, 5 Span	7.52	4.82	4.53	571	n/a	n/a	n/a	2

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[5] Reported flood levels apply immediately upstream and downstream of structure.

Accounts for all flow not part of ID flow.

Site #	Description	Ch	Type	Dimension	Weir/Crest Level	U/S Peak Flood Level[5]	D/S Peak Flood Level	Flow Through Structure	Flow Over Structure (1D)	Flow[#] Around Structure (2D)	Over Topping Depth	Outlet Velocity
		(m)		(m)	(mAHD)	(mAHD)	(mAHD)	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m/s)
8	Dawson Highway Bridge (Golf Course)	9480	Bridge	Twin 5 Span Total length 65m	7.386	5.01	4.89	198	44	45	n/a	1.8
42	Blain Park Pedestrian Bridge	8600	Bridge	40m 1 Span	4.25	6.17	6.13	460	n/a	n/a	0.76	1.9
36	Links Court Bridge	1000	Bridge	15 m, 1 Span	12.45	10.87	10.74	98	n/a	n/a	n/a	3.5
10	Witney St Crossing	1250	Box Culvert	2/ 3.0 x 3.6	13.89	14	12.84	73	2	n/a	0.11	3.4
11	Mercury Street Crossing	2160	Box Culvert	5/ 2.1w x 1.8H	16.183	17.37	17.13	29	47	16	1.19	1.5
14	Cockatoo Drive Crossing	200	Pipe Culvert	3/ 1.8 dia	12.08	12.01	11.71	15	n/a	3	n/a	2
39	Emmadale Drive	900	Pipe Culvert	4/ 1.2	22.62	21.54	21.46	5	n/a	8	n/a	1
16	Parksville Drive (#2)	600	Box Culvert	3/ 3.048w x 2.134h	14.7	15.08	14.3	43	4	31	0.38	4.3
19	Kirkwood Road Crossing (#5)	500	Pipe Culvert	4/ 1.35	22.21	19.33	18.35	11	n/a	n/a	n/a	4.9

Site #	Description	Ch	Type	Dimension	Weir/Crest Level	U/S Peak Flood Level[5]	D/S Peak Flood Level	Flow Through Structure	Flow Over Structure (1D)	Flow[#] Around Structure (2D)	Over Topping Depth	Outlet Velocity
		(m)		(m)	(mAHD)	(mAHD)	(mAHD)	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(m/s)
50	Kirkwood Road Crossing (#6)	1300	Pipe Culvert	4/ 2.1	25.24	21.12	19.85	52	n/a	31	n/a	3.8
38	Parksville Avenue (#1)	200	Pipe Culvert	2/ 1.65	15.4	15.04	15.01	5	n/a	7	n/a	1.1
21	Tondoon Reservoir Outlet	350	Dam Wall with Spillway	5m wide, 30 m long	XS centre IL 21.14	23.43	21.57	71	n/a	n/a	n/a	2.5
22	Glenlyon Road Crossing (#3)	1580	Box Culvert	3/ 3.6w x 2.1h	25.81	25.08	24.55	52	n/a	12	n/a	4.2
28	Haddock Drive Crossing	15600	Box Culvert	3/ 3.6w x 2.1h	30	31.53	30.8	64	79	n/a	1.53	2.8
30	Callemondah Ave. (Bebo Arch)	850	Bebo Arch	L12	6.78	7.01	2.91	106	28	n/a	0.23	3.9
31	Dawson Hwy Road Bridge	1180	Bridge	2/ 30 m, 3 Span	11.436	8.77	8.73	128	n/a	n/a	n/a	2
32	Penda Avenue (Bebo Arch)	1450	Bebo Arch & Box Culvert	M12	11.3	9.95	9.85	117	n/a	n/a	n/a	3.2

Table 32 Culvert upgrade assessment

Site #	Description	Ch (m)	Type	Flow Through Structure Q100 (m³/s)	Flow Over Structure Q100 (m³/s)	Overflow Velocity Q100 (m/s)	Depth Velocity Product Q100 (m²/s)	Required Cross Drainage	Meets Cross Drainage Criteria	Meets Q100 v.d. Safety Criteria	Alternative Access	Culvert Upgrade Priority
10	Witney St Crossing	1250	Box Culvert	73	2	0.46	0.5	Q10	Yes	Yes	Yes	5
11	Mercury Street Crossing	2160	Box Culvert	29	47	1.8	1.7	Q10	No	No	Yes	3
16	Parksville Drive (#2)	200	Box Culvert	43	4	1.2	1.8	Q10	Yes	No	Limited	4
28	Haddock Drive Crossing	15600	Box Culvert	64	79	2.1	4.3	Q10	Yes	No	Limited	5
30	Callemondah Drive Crossing	850	Bebo Arch	106	28	1.3	2.1	Q10	Yes	No	No	5

8. Risk Management

8.1 Gladstone/Calliope Counter Disaster Plan Review

A review of the Gladstone/Calliope Counter Disaster (CD) Plan has been undertaken. Much of this relates to roles and responsibilities, with a generic overview of the types of risk that may occur. However, Council may consider updating several sections of the plan in response to the findings of this study.

In particular, the Evacuation Sub-Plan has the most relevance to this study. The review has therefore focused on the implications of a river or creek flood event on the execution of this sub-plan.

The following queries have been addressed:

- Should there be any changes to roles and responsibilities?
- Does the existence of detailed maps necessitate changes to the CD plan?
- Are the nominated evacuation centers and evacuation routes adequate for the various flood events considered?

Responses to the above items are embedded within the suggestions below:

Main Plan

- The results of this study do not necessitate any changes to “The Threats and Responsibly to Respond” section of the CD plan as it relates to river flooding.

Section B Sub Plans

Evacuation Sub Plan Number 3

In the Evacuation Sub Plan, two options are proposed for consideration:

- Section 9 – Transport Options – Road. The existence of detailed mapping and flood height predictions allows a revision of the proposed evacuation routes in this section. In particular the main evacuation route west, the Dawson Highway, is potentially briefly inundated in extreme floods (refer Table 33). Therefore consideration needs to be given to altering evacuation routes or increasing the flood immunity of the Dawson Highway.

Section D Maps

- Consider referencing maps in the Evacuation Sub Plan. These could consist of either the water surface or inundation depth maps, which show the extent of flooding for various flood events. These may be useful to emergency services in prioritizing their disaster relief efforts.

Welfare Sub Plan

- Of the nominated Welfare (evacuation) Centres only the Gladstone Port Authority Building will be inundated in any of the modeled flood events. It is predicted that it will be inundated to approximately a depth of 0.66 m during the PMF.

- It is recommended that an alternative location be nominated to replace the Gladstone Port Authority Building.

8.2 Emergency Services

The vulnerability to flooding from Auckland Creek of the following emergency services premises was assessed:

- Police;
- Fire and Rescue;
- Gladstone Base Hospital;
- Mater Hospital;
- Gladstone Port Authority Building;
- Ambulance;
- State Emergency Service.

The bases for each of the emergency services listed above are located outside the PMF inundation extent. However, flooding associated with the Design Flood Event (DFE) and the PMF event may affect the operational capability of some of these services by limiting access to parts of Gladstone during the inundated period. Table 33 gives the predicted inundation depths of the major arterial roads within the Auckland Creek catchment that are inundated by these events. The specific location of the road inundation for the 100 yr ARI and the PMF events can be seen in Figure 19 and Figure 22, respectively.

Table 33 Major road inundations in DFE and PMF

Road	100 yr ARI flood inundation depth (m)	PMF inundation depth (m)
Dawson Highway	0.31	1.68
Glenlyon Road	0.04	1.45
Hanson Road	0.28	1.09

8.3 Natural Disaster Risk Management Review

The Natural Disaster Risk Management review component of this study identifies and analyses the flooding related risks to life, property and the environment within the Auckland Creek catchment.

This study was undertaken in accordance with the following guidelines:

- *Natural Disaster Risk Management: Guidelines for Reporting*, Department of Emergency Services; and
- *AS/NZ 4360:1999 Risk Management*, Standards Australia, Standards New Zealand.

It is intended that the findings of the flood risk management study be included in the Council's *Natural Disaster Mitigation Plan* (GCC, 2003).

8.3.1 Context of Study

The study has been undertaken within the context of the following:

- The Council's *Natural Disaster Mitigation Plan* (GCC, 2003) defines treatment of the risks of river and creek flooding as a high priority.
- The Design Flood Event (DFE) has an average recurrence interval (ARI) of 100 years.

A major outcome of this study is to enable the determination of the most appropriate level of community and infrastructure protection from creek flooding. Updating of the *Natural Disaster Mitigation Plan* with river and creek flooding risk management analysis, will help decision makers determine the most appropriate structural and non-structural mitigation measures.

Previously noted flooding problems have been identified in the Auckland Creek catchment as:

- Along Cathurbie Creek immediately downstream of Kirkwood Road;
- In Briffney Creek adjacent to the Dawson Highway;
- Tondoon Creek immediately upstream of Glenlyon Road; and
- Tigalee Creek in the reach between the Witney Street and the Mercury Street culverts.

Further areas of flood risk have been identified in the earlier chapters of this report.

8.3.2 Key Risk Criteria

The flood risk in identified areas has been assessed based on the following criteria:

- Flood risk to public health and safety;
- Flood risk to critical infrastructure including roads; rail; power; telecommunications; sewage; potable water supply;
- Flood risk to emergency services operations;
- Flood risk to industry and commercial operations.

Flood hazard is categorised based on the NSW Floodplain Development Manual (2005). The flood hazard is broken into three categories listed below:

1. Low Hazard;
2. Intermediate Hazard (dependent on site conditions); and
3. High Hazard.

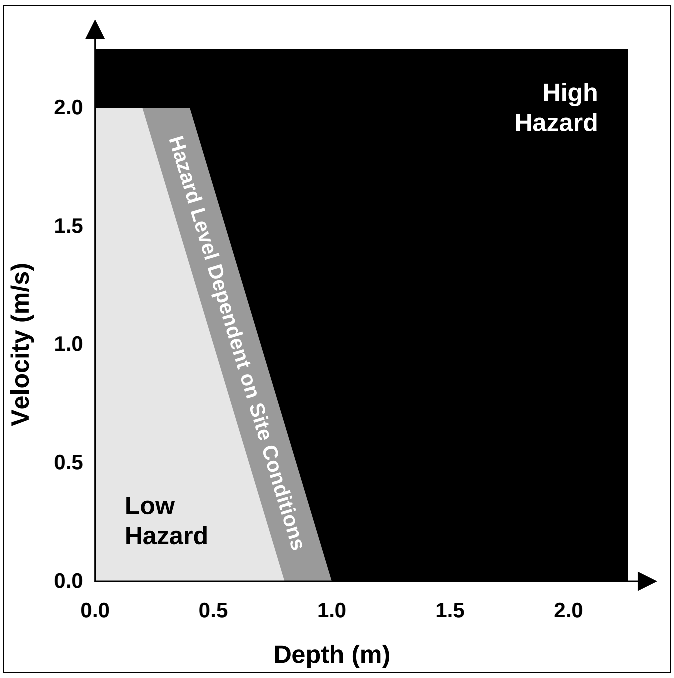

Figure 12 Flood hazard categorisation (Dept. Housing NSW)

8.4 Properties at Risk in the Design Flood Event

An analysis of the properties at risk (PAR) in the design flood event (DFE) was completed through a GIS interrogation of the flood hazard mapping output from TUFLOW and Gladstone City Council's GIS database, which contained the necessary property details.

The analysis provided the relative level of risk, as defined above, for the target land use types of residential, commercial and industrial properties. Table 34 gives a summary of the properties at risk analysis.

Table 34 Properties at risk in the 100 yr ARI flood event

Land Use	Low risk	Intermediate Risk	High Risk	Total
Residential	86	37	30	**153**
Commercial	12	3	1	**16**
Industrial	88	5	6	**99**
Total	**186**	**45**	**37**	**268**

A majority of the effected properties for each land use type fall in the low risk category, which is associated with generally manageable flood hazards.

The intermediate flood hazard category contains those properties where knowledge of specific site characteristics such as topography and vegetative cover type is necessary to determine the actual level of hazard. Therefore caution must be used when determining the final number of properties in each of the hazard categories.

The high-risk category contains those properties lying within areas determined to have flood risk high hazard. Flood hazard has been defined in accordance with the NSW Floodplain Development Manual (2005).

This defines high hazard as when:

- Depth exceeds 1.0 m, or
- Velocity exceeds 2.0 m/s, or
- Depth-velocity product (v.d) exceeds 0.6.

In terms of PAR a location is defined as high risk when there is possible danger to personal safety; evacuation by trucks may be difficult; able-bodied adults would have difficulty in wading to safety; or there is potential for significant structural damage to buildings.

Low hazard is defined as when: if it should it be necessary, trucks could evacuate people and their possessions; and able-bodied adults would have little difficulty in wading to safety.

Several properties are located within the areas defined as high hazard including:

- Properties immediately downstream of the Mercury Street crossing on Tigalee creek, and
- Properties in the region of Melbourne St.

8.5 Population at Risk in the Design Flood Event

An estimation of the potential “Population At Risk” (PAR) for the Auckland Creek catchment within the Gladstone City Local Government Area (LGA) has been made based on DCDB and Australian Bureau of Statistics (ABS) 2001 Census data.

The PAR analysis was completed through a GIS interrogation of the estimated flood depths and Gladstone City Council’s GIS database, which contained the necessary property details. Only those residential properties or commercial properties likely to house people during an event were considered. The Australian Bureau of Statistics (ABS) 2001 Census gives the average number of people per residential property in the Gladstone region as 2.8. Therefore the PAR is equal to 2.8 times the number of properties within the nominated hazard category zone.

The PAR has been estimated for the three levels of risk: low, intermediate, and high, as defined by the velocity depth (v.d) product. It has been assumed that residential properties inundated by the 100 yr ARI flood are lying in an area where the v.d product would expose the resident population to risk of serious injury or death.

The total numbers of residential properties inundated by the 100 yr ARI flood event is 153. This corresponds to a PAR from the nominated flood event of 429.

Residential properties in Gladstone estimated to be in the intermediate or high-risk categories in a 100 yr ARI flood event number 37 and 30, respectively. This gives the estimated PAR at intermediate or high risk of death or serious injury for the Auckland Creek catchment as 104 and 84, respectively.

It is noted that the actual PAR may be significantly lower than estimated here. This is due to fact that the analysis does not take into account the following factors:

- The protection people gain by being inside buildings, even when inundated;
- The community will usually have access to a communication and warning system that enables people to prepare for or avoid the flood event; and
- People have the ability to move out of harms way and therefore reduce their exposure to flood hazard.

8.6 Infrastructure at Risk

Critical and community infrastructure in the Gladstone City area at risk from Auckland Creek flooding have been identified and assessed with respect to the estimated peak flood levels. The following items of infrastructure have been assessed:

- Water supply and sewerage infrastructure;
- Transport infrastructure;
- Power and telecommunications infrastructure;
- Critical community infrastructure such as hospitals and emergency services.

Critical infrastructure was identified through the following actions:

- Review of the Gladstone City Council GIS databases.
- Review of *as-constructed* information held by the Council Engineering Department.
- Review of the Gladstone/Calliope Counter Disaster Plan

A Risk Register (Table 35) has been prepared that lists the risks elements vulnerable to river and creek flooding within the Auckland Creek catchment and the likely consequences should inundation occur. The Risk Register has been compiled in accordance with the “Natural Disaster Risk Management – Guidelines for Reporting” (DES, 2001).

8.6.1 Risk Register Part A: Risk Description

Table 35: Risk Register Part A – Risk Description

Vulnerable Elements	Risks	Consequences
People	Risk to people due to being in residences subject to flooding. Risk of people inadvertently entering waterways that have high flow depths and velocities. Risk to people from debris being carried downstream by floodwaters.	Fatality or serious injury. Particularly vulnerable groups are the aged, very young and disabled. PAR for the Auckland Creek catchment is 429. This can be broken into low, intermediate and high risk categories as 241, 104, and 84 respectively.
Buildings	Risk to buildings due to water damage to interiors during inundation. Building damage to exterior due to force of floodwaters and impact of debris on 150 properties at risk from inundation in the 100 yr ARI flood.	The level of damage caused to buildings by river and creek flooding will vary depending on a number factors including house type (shape, window size, cladding, age and methods of construction); shelter from surrounding structure; and local topographical features. Repair and reconstruction costs.
Environment	Potential for spills of fuel, oil and other hazardous chemicals. Risk to waterway flora and fauna due to increased sediment and nutrient mobilisation during flooding events, caused by bank erosion.	Potential for damage to fuel storage facilities, which could result in oil/fuel spillage. Pollution of waterways, flora and fauna impacts. Suffocation of fish and other aquatic life.

Vulnerable Elements	Risks	Consequences
Business	Flooding of commercial premises; Interruption of normal business activity; GIS analysis shows that 7 businesses are at risk of inundation from the 100 yr ARI flood.	Economic loss of capital and income within tourism industry. Temporary/permanent job losses depending on scale; loss of income due to direct closure or access closure. Inability to service customers. Impact on wider business community due to disruption, loss of business.
Lifelines	Lifeline damage should be restricted to the inundated area.	Where water supply reticulation is exposed to flooding, damage may occur. However, water mains are generally protected (buried) from flood events. Water mains attached to road bridges, of major creek crossings, may sustain damage in high velocity areas. Pump stations with exposed switch boards will be damaged when flood depths exceed switchboard heights.
	Road and rail transport within the inundated area could be disrupted, but will generally not impact on major through routes as depths of overtopping are low.	Road access may be progressively cut off as inundation level rises; structural damage to roads, rail and bridges; disruption of access to/from inundated areas. Major roads affected in Gladstone from Auckland Creek flooding during the DFE are the Dawson Highway (0.31 m), Glenlyon Road (0.04 m) and Hanson Road (0.28 m).

8.7 Community Awareness

Community awareness of this study and its implications on the Gladstone community has been facilitated by the posting of a brief description of the project on the Council website and inclusion in a locally distributed newsletter.

Objectives of the awareness campaign included:

- Eliciting a public response to the natural disaster mitigation initiative in Gladstone; and

- Raising and gauging public awareness and concern regarding flood hazard in the region.

Notably, the website and newsletter postings only elicited a single response from the Gladstone public. A low level of response may be interpreted to indicate a low level of awareness. Alternatively, if it is considered that there is sufficient awareness of the issue in the community, there would appear to be only a low perceived risk. This perception is likely to be strengthened by the lack of flood events during the last 5 to 10 years.

The maps and findings generated by this study should therefore be used in any long-term flood risk or natural disaster awareness programs.

9. Development of Flood Mitigation Measures

9.1 Introduction

Flood mitigation options developed in this study aim to:

- Eliminate, or limit to acceptable levels, the effect of flooding on the well-being, health and safety of flood prone individuals and communities;
- Eliminate, or limit to acceptable levels, damage caused by flooding to private and public property;
- Maintain or facilitate the natural function of the floodplain (i.e. to convey and store floodwaters during a flood) and where necessary, enhance floodplain function along with any flood –dependant ecosystems;
- Encourage planning and use of floodplains as a valuable and sustainable resource capable of multiple, but compatible uses of benefit to the community.

The costed mitigation options form the basis of the Infrastructure Charges Schedule (refer section 10) for future inclusion in the Gladstone City Council Priority Infrastructure Plan (PIP).

9.2 Definition of Problem Areas

Peak flood levels, peak velocities and flow rates for the ultimate development case were analysed and compared against those for the existing development case for the entire Auckland Creek catchment. The analysis was based on the 100 yr ARI storm event with a critical duration of 3 hrs.

Typical problems identified included:

- Problem areas previously identified by council;
- Problem areas highlighted in the Waterway Condition report (GHD, 2005);
- Areas of high velocity;
- Significant increases in flow rates and flood levels as a result of ultimate development;
- Overtopping structures;
- Existing inundation of properties; and
- Areas likely to be inundated as a result of ultimate land use.

9.3 Available Mitigation Options

Potential mitigation options were considered on the basis of cost, aesthetics, effectiveness and practicality, environmental and social concerns, and flooding impact on the surrounding area. The range of options considered included:

- Catchment management procedures / future developmental control;
- Detention basins;
- Structure upgrades;
- Creek and channel augmentation;
- Creek stabilisation;
- Flood proofing;
- Acquisitions;
- Source control measures;
- Investigation of floor levels to see if further site specific mitigation may be required;
- Levee banks.

The following abbreviations have been used to describe individual proposed treatment measures:

RB	Retarding (Detention) Basin;
SI	Site Investigation;
CU	Culvert Upgrade;
LB	Levee bank;
CA	Channel Augmentation;
BR	Bio Retention or Wetland

9.4 Mitigation Option Assessment

9.4.1 Preliminary Option Definition

The preliminary mitigation options presented in the section were based on the controls and procedures listed above. Consideration of site-specific aesthetics and a preliminary flood risk cost benefit analysis was used to narrow the list of available options to those most appropriate for each site.

Flood velocities and heights presented in Table 36 are based on the 100-year ARI ultimate flood event. The locations of the mitigation measures listed in Table 36 are shown in Figure 13.

Table 36 Identified Problem Areas and Mitigation Options Considered

Creek Chainage	Location[6]	Problem	Proposed Mitigation Options[7]
Briffney Ck. 850 m	Callemondah Dr. Bebo Arch (Site 30) and adjacent Industrial area including Neil St.	Structure overtops by approx. 0.25 m	Accept flood immunity lower than Q100. Culvert Upgrade (CU 061)
		High outlet velocity (approx. 4 m/s)	Downstream protection if evidence of scour
		Flooding of adjacent industrial area	Site investigation into floor levels along Callemondah Ave. and Neil St. to determine level of flood immunity and provide recommendations (SI 011)
Tigalee Ck. 1000 m	Immediately downstream of “Links Court” Bridge (Site 36).	High outlet velocity (approx. 3.5 m/s)	Bank Stabilisation of Tigalee Ck. d/s of Links Court Bridge (BS 021)
		Council noted problem area (erosion)	
Tigalee Ck. 2160 m	Mercury St. culvert (Site 11)	Structure overtops by approx. 1.0 m	Retarding Basin u/s of Glenlyon Rd. (RB 031), adjacent to Moura Railway line;
			Retarding Basin u/s of Glenlyon Rd. (RB 033) adjacent to Hurley St.;
		Structure has low flood immunity Council noted problem area (flooding)	Culvert Upgrade of Mercury St. crossing (CU 033)
		Flooding of properties along Pacific Ct.	Levee Bank (LB 131).

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[6] Site ID refers to structures identified in Figure 11

[7] Mitigation Option ID refers to options identified in Figure 13

Creek Chainage	Location[6]	Problem	Proposed Mitigation Options[7]
Tigalee Ck. 1250 m	Witney St. culvert (Site 10)	Structure overtops by approx. 0.15 m	Retarding Basin u/s of Witney St. (RB 032);
		Council noted problem area (flooding)	
Tondoon Ck. 1580 m	Glenlyon Rd. Crossing (Site 22)	Glenlyon Rd. adjacent to culvert overtops Council noted problem area (flooding)	Retarding Basin u/s of Glenlyon Rd. (RB 051);
			Culvert upgrade if 100 yr flood immunity required in near future.
			Installation of depth gauges and warning signs.
Tondoon Ck. 2100 m	Adjacent to Allunga Dr.	Medium velocities approx. 1.5 m/s Council noted problem area (erosion)	Bank stabilisation (BS 061) of Tondoon Ck u/s of proposed retarding basin (RB 051)
Police Ck. 15,600 m	Haddock Rd. Crossing (Site 28)	Structure overtops by approx. 1.5 m; Structure has low flood immunity;	Culvert upgrade of Haddock Rd. crossing (CU 072).
			Installation of depth gauges and warning signs.
		High outlet velocity (approx. 3.0 m/s);	Downstream protection if evidence of scour
		6% increase in flows from existing to ultimate case.	Retarding Basin u/s of Haddock Rd. (RB 071).
			Catchment based Stormwater Management to reduce increase in flows

Creek Chainage	Location[6]	Problem	Proposed Mitigation Options[7]
Cathurbie Ck. 1300 m	Kirkwood Rd. Crossing (Site 50)	Road adjacent to structure overtops (under construction and predicted overtopping may be occurring at the end of road embankment?);	Choking of Kirkwood Rd. #6 (CU 092)
			Additional culverts are recommended for future extension completion of Kirkwood Rd.
		High outlet velocity (approx. 2.8 m/s);	Outlet protection at existing and future culvert aprons.
Cathurbie Ck. 600 m	Parksville Dr. Crossing. (Site 16)	Structure overtops by approx. 0.4 m; Council noted problem area (partial flooding of large blocks)	Culvert Upgrade (CU 091).
		High outlet velocity (approx. 4.0 m/s);	Outlet protection if evidence of scour
Auckland/ Police Ck. 10,400 – 10,900 m	Upstream of Moura Railway Crossing (Site 9)	Potentially minor flooding of properties on the creek side of Sandpiper Ave. Council noted problem area (flooding)	Site investigation into floor levels along Sandpiper Ave. to determine level of flood immunity. (SI 101).
			Levee bank along property line (LB 101)
Auckland/ Police Ck 9,600 – 10,300 m	Downstream of Moura Railway Crossing (Site 9) confluence with Tigalee Ck.	Potentially minor flooding of properties along Olsen Ave	Site investigation into floor levels along Olsen Ave. to determine level of flood immunity. (SI 111)

Creek Chainage	**Location[6]**	**Problem**	**Proposed Mitigation Options[7]**
Auckland/ Police Ck 9,600 – 10,300 m	Phillip St.	Phillip St may overtop and cause subsequent flooding of shopping centre	Levee bank along Phillip St. (LB 121)
Auckland/ Police Ck 9,600 – 10,300 m	Between the. Dawson Highway Bridge and Phillip St.	Dawson Hwy overtops in this area	It is noted that this is a Department of Main Roads Issue and should be brought to their attention.
Auckland Ck. 2,000 – 5,000 m	Light Industrial/ Commercial area around Beckinsale St., Chapple St. and Hilliard Dr.	Flooding potentially due to adopted high storm surge tailwater level 3.33 m	Investigation into floor levels in flood affected area to determine level of flood immunity. (SI 141) Accept flood immunity lower than Q100.
Cathurbie Tributary 200 m	Cockatoo Dr. culvert (Site 14)	Cockatoo Dr overtops adjacent to culvert	Augmentation of channel upstream of culvert (CA 151)
			Upgrade of Cockatoo Dr. culvert (CU 153)
			Retarding basin upstream of Kirkwood road
		Minor flooding of houses upstream of culvert along Emmadale Dr.	Site investigation into floor levels in flood affected area to determine level of flood immunity. (SI 152)
Briffney Ck. 1,500 m	Shaw St. (Site 32)	Flooding of houses along Shaw St. Council noted problem area (flooding)	Levee bank along top of drainage channel (LB 161)

Creek Chainage	Location[6]	Problem	Proposed Mitigation Options[7]
Sewage Pump Station off Cemetery Rd. Auckland Ck	Flooding		Check floor levels and switch levels. Flood proofing if necessary
Water Booster Pump Station	Flooding		Check floor levels and switch levels. Flood proofing if necessary
Melbourne St.	Flooding		Check floor levels and switch levels. Flood proofing if necessary

9.4.2 Option Selection

An analysis of the above-recommended measures requiring modelling was carried out to determine the suitability compared to the previously listed criteria.

Discussions were held with Council during the initial review with a reduced list of measures design event in terms of then modelled to ascertain the benefit (if any) on the 100 yr ARI ultimate flood levels and velocities.

The full extent of Kirkwood Road and additional culverts associated with ongoing development in this area were included in each of the mitigation scenarios. These were represented by:

- Kirkwood Rd Crossing #7 (Site 27);
- Dixon Rd Crossing (Site 41); and
- Kirkwood Rd Crossing #1 (Site 34)

Two mitigation scenarios were run independently of each other to determine the relative benefit of the measures tested. In broad terms, Mitigation Option 1 tested the benefit of implementing retarding basins while Mitigation Option 2 tested the relative benefit of culvert upgrades and levee banks.

Mitigation Option 1 assessed the following measures:

- RB 033 – Retarding Basin upstream of Glenlyon Road adjacent to the Moura Short railway line;
- RB 031 - Retarding Basin upstream of Glenlyon Road adjacent to Hurley St.

Mitigation Option 2 assessed the following measures:

- Levee banks along Tigalee Creek between Mercury St and Witney St;

- Levee banks along Phillip St around the Kin Kora Shopping complex;
- Levee banks adjacent to Shaw St downstream of the Penda Ave. crossing of Briffney Creek;
- Culvert upgrades for Cockatoo Dr., Mercury St., Parksville Dr., & Callemondah Dve;
- Reducing capacity (choking) of Kirkwood Rd. crossing #6 Cathurbie Creek

The following tables Table 37 to Table 39 provide the dimensions of the proposed mitigation measures, which were modelled in the mitigated scenarios.

Table 37 Proposed Detention Basins

Parameter	**Units**	**RB 031**	**RB 033**
Location		Upstream of Glenlyon Rd. adjacent to Moura Railway Line	Upstream of Glenlyon Rd. adjacent to Hurley St.
Area	(ha)	5.97	3.15
Volume	(m^3)	200,000	44,600
Weir Crest Level	(mAHD)	23.86	24.97
Maximum Depth*	(m)	3.86	1.97
Outlet Configuration			
Pipe Diameter	(m)	3 x 1.35	4 x 1.5
Upstream IL	(mAHD)	20	23.3
Downstream IL	(mAHD)	19.75	23
Hydrology			
Peak Flow in (100 yr ARI)	(m^3/s)	44.9	45.3
Peak Flow out (100 yr ARI)	(m^3/s)	17.4	17.3
Peak Velocity (100 yr ARI)	(m/s)	5	1.75

*Maximum depth of retarding basins occurs in existing flood-prone area.
Average depth will be considerably lower in graded slopes of remainder of basin.

The proposed culvert upgrades, shown here in Table 38, are designed to cater for the 50 yr ARI cross flow. Checks have been done to ensure the proposed culvert configuration is suitable for each location (waterway width, invert levels, velocities). However, the designs are preliminary and may be subject to considerable refinement during the detailed design phase.

Table 38 Proposed Culvert Upgrades

Location	Existing Structure	Proposed Upgrade
Mercury St	5/2.1 (RCP)	4/3.6 x 2.1 (RCBC)
Cockatoo Drive	3/1.8 (RCP)	3/2.1 x 1.8 (RCBC)
Parksville Drive	3/3.0 x 2.1 (RCBC)	4/3.0 x 2.1 (RCBC)
Kirkwood Rd.	4/2.1 (RCP)	3/2.1 (RCP)

The proposed levee banks, given in Table 39, have been designed as a defence, for specific locations, against the 100 yr ARI flood. The have a common design configuration of a top width of 2m and a batter slope of 1H:4V.

The proposed channel; augmentation upstream of Cockatoo Drive is estimated to be 400 m long and require a volume of approximately 10,000 m^3 to be excavated.

Table 39 Proposed Levee Banks

Location	Length	Average Height	Fill Volume
	(m)	(m)	(m^3)
Phillip Street	470	1.2	1316
Kin Kora Mall	550	1.2	1540
Shaw Street	160	0.6	208
Pacific Court	370	1.2	1036
Sandpiper Avenue	520	3	5200

9.4.3 Preliminary Mitigation Option Results

This section presents the predicted flood heights for the two preliminary mitigation options tested against the unmitigated existing and ultimate 100 yr ARI cases. The results are summarised in Table 40.

Table 40 Predicted Mitigated Flood Heights

Site #	Description	Existing (m AHD)	Ultimate (m AHD)	Mitigation Option 1 (m AHD)	Mitigation Option 2 (m AHD)
1	Marina Bridge Creek Outlet	2.34	2.38	2.38	2.38
3	Hanson Road (Clinton) Bridge	2.65	2.67	2.66	2.67

Site #	Description	Existing (m AHD)	Ultimate (m AHD)	Mitigation Option 1 (m AHD)	Mitigation Option 2 (m AHD)
4	Ash Pond Causeway	3.54	3.57	3.54	3.56
5	Blain Drive Bridge	3.85	3.88	3.85	3.88
6	Lake Callemondah Weir	4.32	4.35	4.32	4.35
7	North Coast Railway Crossing	4.78	4.83	4.79	4.84
8	Dawson Highway Bridge (Golf Course)	6.11	6.17	6.14	6.16
10	Witney St Crossing	13.96	13.97	13.31	13.96
11	Mercury Street Crossing	17.36	17.36	16.72	17.51
36	Links Court Bridge	10.83	10.83	10.24	10.83
14	Cockatoo Drive Crossing	11.99	12.00	12.00	12.13
39	Emmadale Drive	21.5	21.55	21.55	21.55
16	Parksville Drive (#2)	15.07	15.08	15.11	14.93
19	Kirkwood Road Crossing (#5)	19.31	19.33	19.33	19.33
50	Kirkwood Road Crossing (#6)	20.42	20.45	20.45	22.01
38	Parksville Drive (#1)	15.1	15.04	15.04	15.04
21	Tondoon Reservoir Outlet	23.4	23.43	23.43	23.43
22	Glenlyon Road Crossing (#3)	25.04	25.07	25.07	25.07
28	Haddock Drive Crossing	31.46	31.52	31.52	31.52
30	Callemondah Dr. (Bebo Arch)	6.89	7.01	6.98	6.95

Site #	Description	Existing (m AHD)	Ultimate (m AHD)	Mitigation Option 1 (m AHD)	Mitigation Option 2 (m AHD)
31	Dawson Hwy Road Bridge (Briffney Creek)	8.68	8.77	8.75	8.69
32	Penda Avenue (Bebo Arch)	9.81	9.95	9.92	9.83

With Mitigation Option 1, retarding basins RB 033 and RB 031 demonstrated significant beneficial effect on the flooding along Tigalee Creek. Table 40 shows that the flood height is reduced by 0.65 m upstream of Mercury Street. Similarly, the upstream flood height at the Witney Street crossing is reduced by 0.68 m. This reduction in flood height, due to the retarding effects of the basins, is accompanied by lower velocities through downstream structures and waterways and carries through to the rest of Tigalee Creek, alleviating much of the predicted problems within the tributary.

The Mitigation Option 2 culvert upgrade measures had a less beneficial effect with most augmented structures augmented showing minimal reduction in upstream flood heights, despite the increase in flow capacity.

For the 100 yr ARI design event, flow through the Callemondah Drive & Cockatoo Drive culverts may be tailwater controlled and therefore not responsive to increasing capacity. Table 40 shows that the Cockatoo Drive flood height increased by 0.12 m after the culvert upgrade increased its capacity. Callemondah Drive shows a minimal decrease of just 0.05 m following the culvert upgrade.

Mitigation Option 2 results in the peak flood height increasing in the Mercury Street crossing despite an increase in culvert capacity. This is due to the culvert upgrade being modelled in conjunction with raising of the road level from 16.2 m AHD to 17 m AHD. This had the benefit of utilising the existing storage of the sports field adjacent to Tigalee Creek. While this had some downstream benefit, as evidenced in Table 43, the retarding basins modelled in Option 1 show a much greater benefit.

The choking of the Kirkwood Road crossing # 6 (CU 092) in Mitigation Option 2 had some benefit in reducing flooding downstream (at the Parksville Drive crossing #2) by 0.2 m. The benefit of the flood height reduction needs to be considered in conjunction with the future planning issues south of Kirkwood Road and the potential for locating a retarding basin in the area where the results showed that reduction in culvert capacity increased the flood height upstream of Kirkwood Road significantly (1.6 m).

Peak flood heights tabled for Mitigation Option 1 and 2 show reductions in several areas where the retarding effect of the additional culverts has had some effect. This is evident at the Briffney Creek crossings where the addition of the Kirkwood Road crossing #1 culvert has had beneficial effects downstream.

Where levee banks have been modelled, the results show a net decrease in flood height in the adjacent waterways despite an expected increase. This is due mainly to the effect of the mitigation measures (e.g. culvert upgrades) which were modelled simultaneously. The levee bank along Phillip St. (LB 121) modelled in Mitigation Option 2 had the desired effect of eliminating inundation of the Kin Kora Plaza and Mall shopping complexes. Similarly, the other levee banks modelled, Briffney Creek (LB 161); Auckland Creek (LB 101), and Tigalee Creek (LB 131)., protected the targeted properties.

9.5 Preferred Mitigation Option

A final suite of mitigation measures representing the ‘Preferred Case’ was tested in the TUFLOW model. The composition of this scenario was determined in consultation with Council and after consideration of the relative benefit of the mitigation measures tested in the preliminary two options. The locations of the final mitigation measures are shown in Figure 13.

In essence, the final suite of mitigation options adopted for assessment consisted of a combination of the two preliminary options, i.e. retarding basins and culvert upgrades in combination with the flood defence levee banks.

Specifically, the mitigation measures tested in the final Mitigation Option scenario consisted of:

- A retarding basin upstream of Glenlyon Road adjacent to the Moura Short railway line;
- A retarding basin upstream of Glenlyon Road adjacent to Hurley Street.
- A levee bank along Tigalee Creek between Mercury Street and Witney Street;
- Levee banks along Phillip Street around the Kin Kora Shopping complex;
- A levee bank adjacent to Shaw Street downstream of the Penda Avenue crossing of Briffney Creek;
- A levee bank adjacent to Sandpiper Avenue along Cathurbie Creek;
- Culvert upgrades for Cockatoo Drive, Mercury Street, Parksville Drive, & Callemondah Drive; and
- A reduction in capacity (choking) of the Kirkwood Rd. Crossing #6 Cathurbie Creek

Table 41 shows the effect of the tested mitigation measures on the 100 yr ARI flood heights at strategic locations throughout Auckland Creek.

Table 41 Predicted Final Mitigation Option Flood Heights

Site #	Description	Weir level (mAHD)	Existing (mAHD)	Ultimate (mAHD)	Final Mitigation (mAHD)
1	Marina Bridge Creek Outlet	8.165 in centre	2.46	2.46	2.46

Site #	Description	Weir level (mAHD)	Existing (mAHD)	Ultimate (mAHD)	Final Mitigation (mAHD)
3	Hanson Road (Clinton) Bridge	4.935	2.74	2.76	2.74
5	Blain Drive Bridge	5.5 m Approx.	3.81	3.85	3.92
6	Lake Callemondah Weir	2.6	4.33	4.36	4.28
7	North Coast Railway Crossing	8	4.78	4.82	4.75
8	Dawson Highway Bridge (Golf Course)	7.386	6.12	6.17	6.12
10	Witney St Crossing	13.89	14.00	14.00	13.35
11	Mercury Street Crossing	16.2	17.36	17.37	16.71
36	Links Court Bridge	12.2	10.87	10.87	10.26
14	Cockatoo Drive Crossing	12.2	11.99	12.01	11.36
39	Emmadale Drive	22.62	21.55	21.54	21.56
16	Parksville Drive (#2)	14.7	15.09	15.04	14.97
19	Kirkwood Road Crossing (#5)	22.21	19.31	19.33	19.33
50	Kirkwood Road Crossing (#6)	25.24	21.08	21.12	21.25
38	Parksville Drive (#1)	15.4	15.07	15.08	15.04
21	Tondoon Reservoir Outlet	21.14(1)	23.40	23.43	23.43

Site #	Description	Weir level (mAHD)	Existing (mAHD)	Ultimate (mAHD)	Final Mitigation (mAHD)
22	Glenlyon Road Crossing (#3)	25.81	25.04	25.08	25.06
28	Haddock Drive Crossing	30	31.46	31.53	31.53
30	Callemondah Ave. (Bebo Arch)	6.45	6.89	7.01	7.02
31	Dawson Hwy Road Bridge	11.07	8.68	8.77	8.77
32	Penda Avenue (Bebo Arch)	11.3	9.81	9.95	9.98

(1) Survey XS centre IL

Table 42 to Table 43 show details of mitigation modelling results for various affected structures throughout the catchment. Preliminary analysis of waterways around culvert upgrades shows that the proposed culverts should fit.

Table 42 Mercury Street Culvert Upgrade and Results

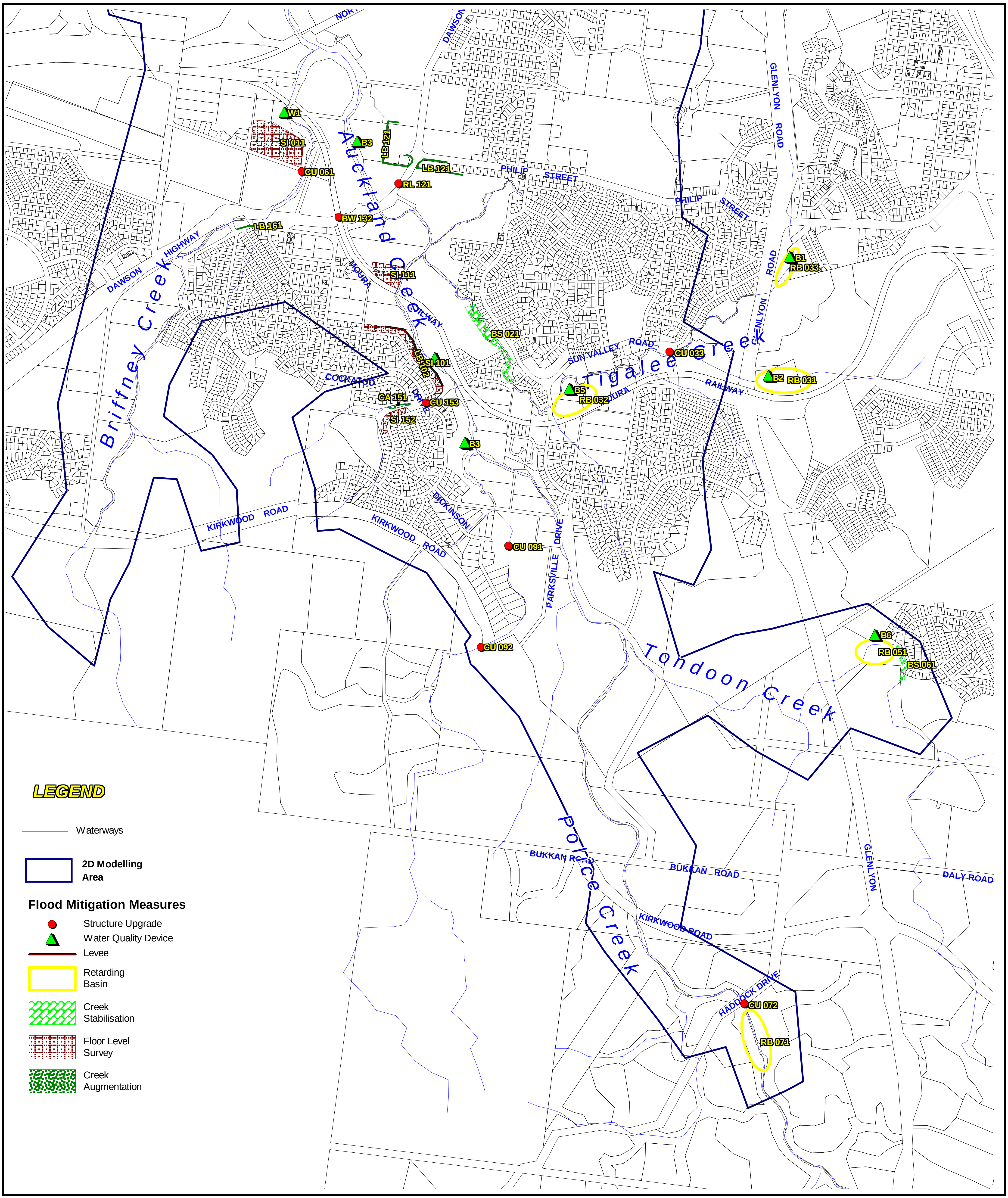
Parameter	Unit	Unmitigated (Ultimate Development)	Option 1	Option 2	Final Mitigation Option
Road (weir) Level	m AHD	16.2	16.2	17	17
Culvert Structure		5/ 2.1	5/ 2.1	4/ 3.6 x 2.1	4/ 3.6 x 2.1
Culvert Waterway Area	m^2	18.9	18.9	30.24	30.24
100 yr ARI Flood Levels					
Headwater Height	m AHD	17.36	16.72	17.51	16.71
Outlet Velocity	m/s	1.5	1.4	2	1.3
Flow Depth over Road	m	1.08	0.52	0.51	na
Flow Velocity over Road	m/s	1.8	1.2	1.2	na
v.d over road	m^2/s	1.9	0.6	0.6	na

Table 43 Parksville Drive Crossing #2 Upgrade and Results

Parameter	Unit	Unmitigated (Ultimate development)	Option 1	Option 2	Final Mitigation Option
Road (weir) Level	m AHD	14.7	14.7	14.7	14.7
Creek bed Level	m AHD	12.3	12.3	12.3	12.3
Culvert Structure		3/3.05x2.13	3/3.05x2.13	4/3.0 x 2.1	4/3.0 x 2.1
Culvert Waterway Area	m^2	19.51	19.51	25.2	25.2
100 yr ARI flood levels					
Headwater Height	m AHD	15.08	no change	14.97	14.97
Outlet velocity	m/s	4.3	no change	4.2	4.2
Flow depth over road	m	0.4	no change	0.3	0.3
Flow velocity over road	m/s	1.2	no change	1.0	1.0
v.d over road	m^2/s	0.5	no change	0.3	0.3

Table 44 Kirkwood Road Crossing #6 Upgrade and Results

Parameter	Unit	Unmitigated (Ultimate development)	Option 1	Option 2	Final Mitigation Option
Road (weir) Level	m AHD	25.24	25.24	25.24	25.24
Creek bed Level	m AHD	17.1	17.1	17.1	17.1
Culvert Structure		4/ 2.1	3/ 2.1	4/ 2.1	3/ 2.1
Culvert Waterway Area	m^2	13.85	10.39	13.85	10.39
100 yr ARI Flood Levels					
Headwater Height	m AHD	21.12	21.25	21.12	21.25
Outlet velocity	m/s	2.8	3.8	2.8	3.8
Flow depth over road	m	na	na	na	na
Flow velocity over road	m/s	na	na	na	na
v.d over road	m^2/s	na	na	na	na



MANAGEMENT
ENGINEERING
ENVIRONMENT

G:\41\14340\GIS\MapInfo\Report_Figures\Final Report\Fig13_Mitigation_Measures.WOR

North

0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 13

Mitigation Options Considered

9.6 Costing

This section of the report provides a summary of the costs involved to construct the flood risk mitigation infrastructure recommended in this study. The full cost breakdown for each item is provided as Appendix E.

9.6.1 Basis of Cost Estimates

Capital costs for each of the recommended mitigation measures were determined from the following sources:

- Capital costs for culvert upgrades and earthworks were based on 2004 unit rates for Brisbane sourced from the Rawlinsons Australian Construction Handbook 2005. The Brisbane unit rates were multiplied by a regional factor of 1.07 to convert to Gladstone prices (Rawlinsons, 2005). A factor of 1.06 was also applied to allow for price escalation between December 2004 and December 2005 (factor based on the published rise in the Australian Bureau of Statistics General Construction Industry Price Index for Road and Bridge Construction). The overall multiplier adopted for the 2004 Brisbane unit rates was therefore 1.13. Where possible, these unit rates were verified against product manufacturers data and recent civil works tender information.
- Land acquisition costs have not been considered in this study. It is likely that the drainage reserves required for stormwater management would be included in the minimum areas of land that developers are required to dedicate for Council use under the requirements of Council's Planning Scheme.
- The cost has been calculated in terms of net present value, assuming a 7% discount rate over a 50yr life cycle.

9.6.2 Statement of Accuracy of Cost Estimates

The accuracy of estimated costs is not expected to be better than about ± 25% for the scope of work described in this report. Contingencies of 15% for engineering and 20% for general contingencies have been added to all capital costs. A detailed design is recommended if a more reliable estimate is required.

9.6.3 Summary of Cost Estimates

Table 45 presents the cost estimates recommended by this study for the water quantity mitigation measures. Water quality mitigation measures costs are detailed in the separate Auckland Creek Catchment Management Plan (GHD, 2006).

Table 45 Water Quantity Mitigation Cost Estimates

Item	Description	Capital Costs	Annual Maintenance Cost	Net[#] Present Value
CHANNEL AUGMENTATION				
3.1	Emmadale Creek	$ 399,415	$ 3,333	$445,413
CULVERT UPGRADES				
4.1	Mercury Street	$ 272,310	$ 8,000	$ 382,716
4.2	Cockatoo Drive	$153,981	$ 8,000	$ 264,387
4.3	Parksville Drive	$ 86,971	$ 8,000	$ 97,377
4.4	Kirkwood Road #6	$ 27,000	$ 2,000	$ 54,601
RETARDING BASINS				
5.1	Retarding Basin RB 031	$ 104,915	$ 1,669	$ 127,949
5.2	Retarding Basin RB 033	$ 860,763	$ 4,607	$ 924,343
LEVEES				
6.1	Phillip Street (Auckland Ck.)	$ 65,273	$ 1,499	$ 85,960
6.2	Kin Kora Mall (Auckland Ck.)	$ 76,383	$ 1,606	$ 98,547
6.3	Shaw Street (Briffney Ck.)	$ 23,112	$ 796	$ 34,097
6.4	Pacific Court (Tigalee Ck.)	$ 51,385	$ 1,349	$ 70,002
6.5	Sandpiper Avenue (Cathurbie Ck.)	$ 185,350	$ 1,980	$212,675
	TOTALS	$2,307,000	$42,800	$2,898,000

 # Assumes 7% discount rate and 50yr life.

10. Infrastructure Charges

10.1 Infrastructure Charges Definition

Infrastructure charges have been determined for the proposed stormwater flood mitigation measures for the Auckland Creek catchment. The charges are based on the previously reported cost estimates, with costs calculated in terms of net present value and inclusive of the following components:

- Capital costs,
- Maintenance costs,
- Study costs.

Excluded from the infrastructure charges are:

- Trunk and subdivision scale drainage schemes, which Council may need to make allowance for; and
- Source control costs associated with future development, which should be incorporated into future planning and development regulations.

Infrastructure costs for flood mitigation measures were estimated for each of the following major sub-catchments:

- Auckland/Police Creek,
- Tigalee Creek,
- Briffney Creek,
- Cathurbie Creek,
- Tondoon Creek.

Infrastructure charges for the overall Auckland Creek catchment are then presented in terms of $/impervious hectare. Table 46 gives the existing and ultimate impervious areas for each of the major sub catchments used in preparing the infrastructure charges schedule.

Table 46 Existing and Ultimate Impervious Areas for Major Sub Catchments

Catchment	Total Area (ha)	Existing Impervious Area (ha)	Ultimate Impervious Area (ha)
Auckland/Police Creek	3120	990	1437
Tigalee Creek	640	209	231
Briffney Creek	700	161	213
Cathurbie Creek	540	123	204
Tondoon Creek	580	211	296
Total	**5,580**	**1,694**	**2,381**

10.2 Infrastructure Charges Schedule

Table 47 provides the Flood Mitigation Infrastructure Charges Schedule, as defined above, for each of the agreed sub-catchments. Infrastructure costs were determined in accordance with the ultimate impervious area of each sub-catchment.

Table 47 Infrastructure Costs by Sub-Catchment

Sub-Catchment	Infrastructure Cost	Study Cost[#]	Net Present Value	Ultimate Area Impervious (ha)
Auckland/ Police Creek	$ 649,160	$59,183	$708,343	1437
Tigalee Creek	$ 1,505,010	$ 9,495	$ 1,514,505	231
Tondoon Creek		$ 8,760	$ 8,760	213
Cathurbie Creek	$ 709,800	$ 8,387	$ 718,187	204
Briffney Creek	$ 34,097	$ 12,175	$ 46,272	296
TOTALS	$2,898,067	$98,000	$2,996,067	2381

The resultant infrastructure charge of @1,259 per impervious hectare makes use of the approach adopted for water quality infrastructure charges where it is assumed that improvements in the stormwater drainage system benefit the entire community within the Auckland Creek Catchment.

10.3 Construction Program

10.3.1 Prioritisation

Prioritisation of the stormwater infrastructure charges was based on the following principles, which have been applied qualitatively:

- The reduction in flood hazard to be directly derived from implementation of the infrastructure item;
- Benefit provided to emergency services during a flood hazard event; and
- Net benefit to the community;

Table 48 lists the assigned priority rating for each of the costed infrastructure items. The list forms the basis of the five and ten-year construction plans presented in Table 50.

Table 48 Stormwater infrastructure prioritisation

Item	Description	Priority
CHANNEL AUGMENTATION		
3.1	Emmadale Creek	1
CULVERT UPGRADES		
4.1	Mercury Street	3
4.2	Cockatoo Drive	3
4.3	Parksville Drive	4
4.4	Kirkwood Road #6	5
RETARDING BASINS		
5.1	Retarding basin RB 031	3
5.2	Retarding basin RB 033	4
LEVEES		
6.1	Phillip Street (Auckland Creek)	2
6.2	Kin Kora Mall (Auckland Creek)	2
6.3	Shaw Street (Briffney Creek)	1
6.4	Pacific Court (Tigalee Creek)	1
6.5	Sandpiper Avenue (Cathurbie Creek)	1

Table 49 presents the infrastructure costs broken down by order of priority.

Table 49 Stormwater infrastructure cost by priority

Priority level	Cost
1	$ 762,187
2	$ 184,507
3	$ 775,052
4	$1,121,720
5	$ 54,601
TOTAL	$2,898,067

10.3.2 5 &10 Year Construction Plan

A 10-year construction plan for the infrastructure items costed and prioritised above is presented in Table 50. The five-year construction plan is embedded within the 10-year plan.

Table 50 5 & 10 year construction plan

Year	Mitigation Item	Associated Cost
1	3.1	$ 445,413
2	6.3, 6.4	$ 104,099
3	6.5	$ 212,675
4	6.2, 6.1	$ 184,507
5	4.1	$ 382,716
6	4.2	$ 264,387
7	5.1	$ 127,949
8	5.2	$ 924,343
9	4.3	$ 197,377
10	4.4	$ 54,601

11. Flood Mapping

11.1 Overview

A series of flood inundation maps have been prepared in order to delineate the full extent of flooding. Flood maps have been produced for existing conditions, with all design events represented. Initially, flood maps have been produced for the cases involving a tidal (HAT) tailwater, as illustrated in Figure 14 to Figure 23. However, a map for the 100 yr event has also been produced for the high tailwater case (i.e. with storm tide), presented as Figure 24.

11.2 Existing Flooding Characteristics

11.2.1 Flow pattern in upper reaches

The flow in the upper reaches of the creeks appears to be confined within the creek channels, except at the following locations:

- Tondoon Reservoir;
- Tondoon Creek upstream of Glenlyon Road; and
- Police Creek at Haddock Drive;

11.2.2 Flow pattern in lower reaches

All tributaries join Auckland Creek upstream of Lake Callemondah, with the lower reaches and the area around Lake Callemondah, (and further downstream) consisting predominantly of floodplain. The flow in these lower reaches becomes two dimensional in nature.

11.3 Peak Velocity Maps

Peak velocity maps have been produced for the same set of events, and are provided in Figure 25 to Figure 32. It should be noted that in rare cases, peak velocity values are attributable to numerical modelling spikes, and are to be ignored. A list of such locations is provided below:

- Witney Street culvert (site 10) peak model velocity 5.4m/s, actual 3.2m/s;
- Links Court Bridge (site 36) peak model velocity 6.7m/s, actual 3.3m/s;
- Glenlyon Road (site 22) peak model velocity 4.2m/s, actual 2.3m/s.

11.4 Flood Hazard

Flood hazard has been defined in accordance with (NSW Floodplain Development manual, 2005).

High hazard is defined as when the:

- Depth exceeds 1.0 m, or

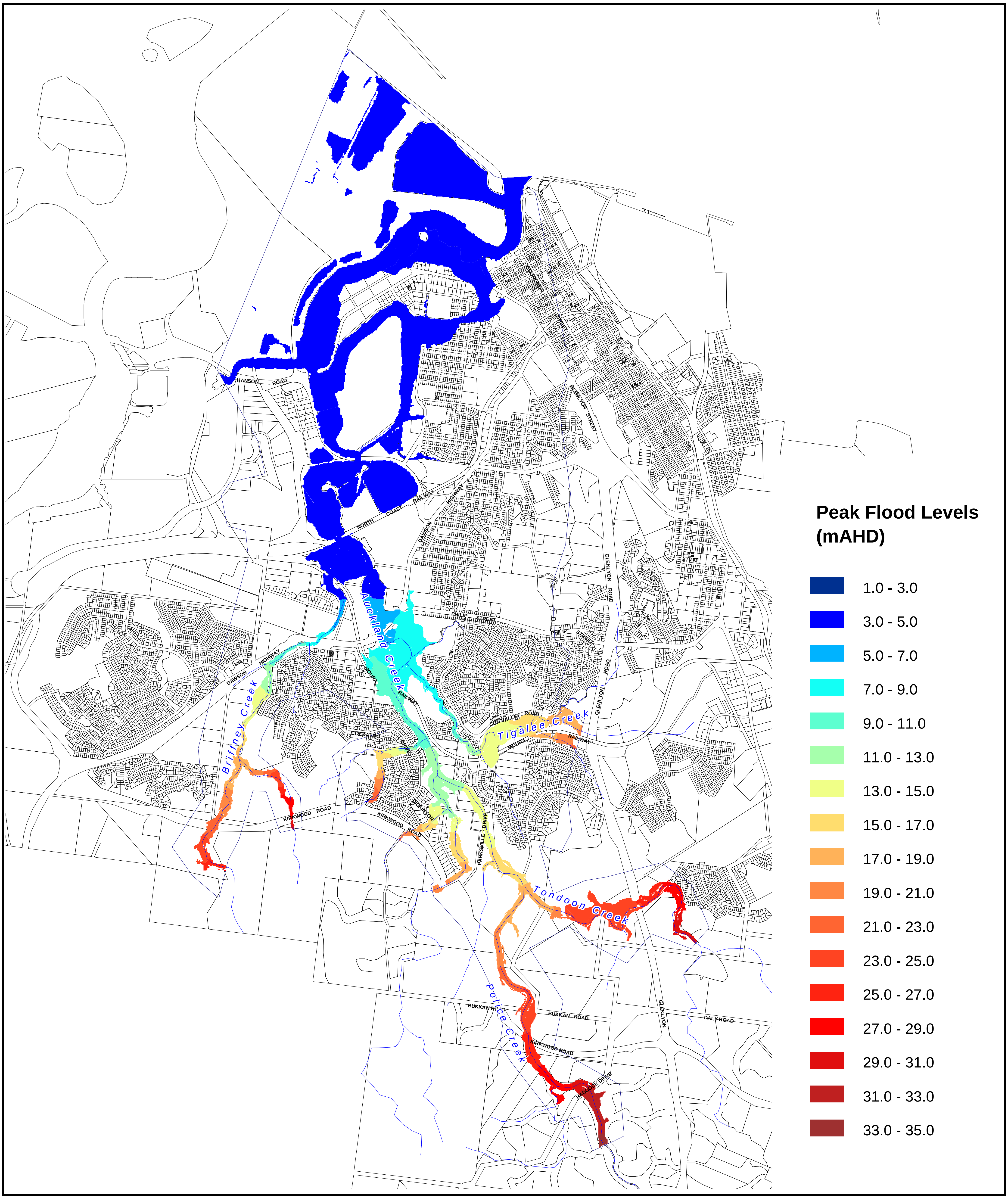
- Velocity exceeds 2.0 m/s, or
- Depth-velocity product (v*d) exceeds 0.6.

Maps of the depth-velocity product have been produced for existing conditions for the 50 and 100 yr ARI events (refer Figure 35 and Figure 36). Each of these hazard maps exhibits a similar pattern. Much of the inundated area is high hazard, regardless of low maximum velocities, owing to the depth being greater than 1.0 m.

The majority of floodplain areas, where flooding spills out of the waterway channels, fall into the low hazard category. The exceptions to this are the areas around the confluence of Auckland, Tigalee and Kin Kora Creeks; the confluence of Cathurbie and Police Creeks and floodplains upstream of the North Coast railway line.

Several properties are located within the areas defined as high hazard including:

- Properties immediately downstream of the Mercury Street crossing on Tigalee creek, and
- Properties in the region of Melbourne St.



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LEGEND

Waterways
2D Modelling Boundary

North

0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

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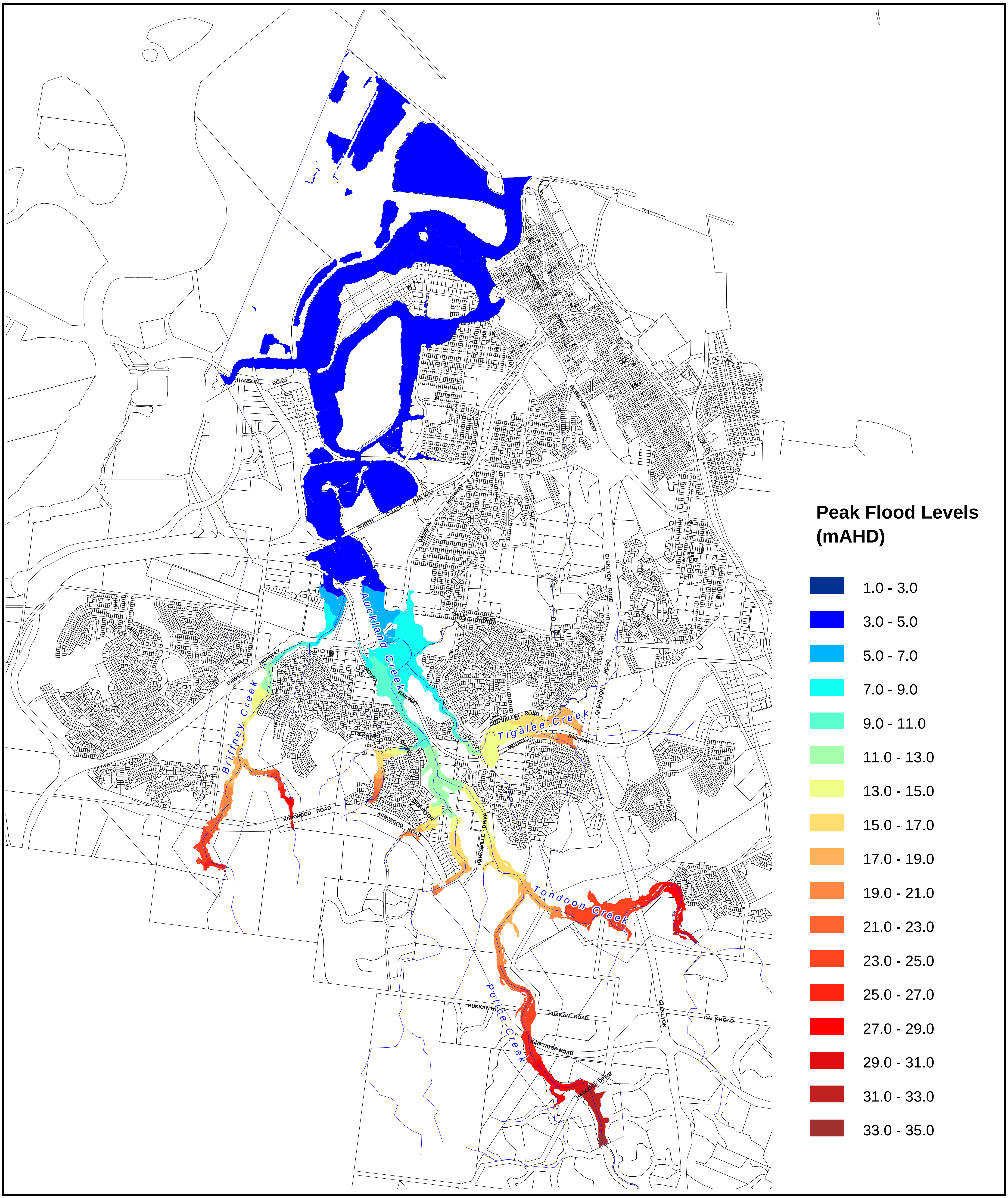
Auckland Creek Flood Study

FIGURE 14

20yr ARI Peak Flood Level
Existing Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

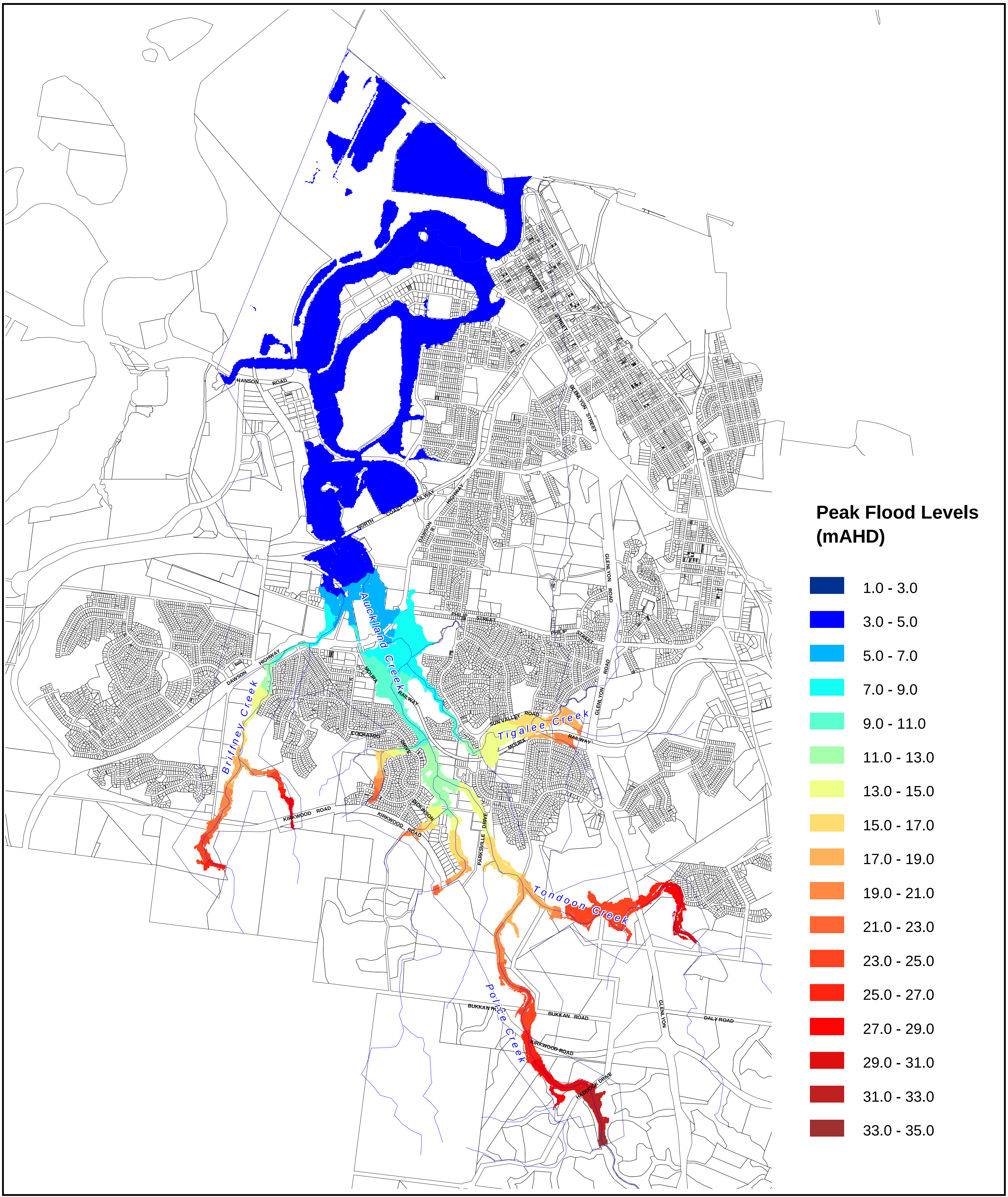
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 15

20yr ARI Peak Flood Level Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

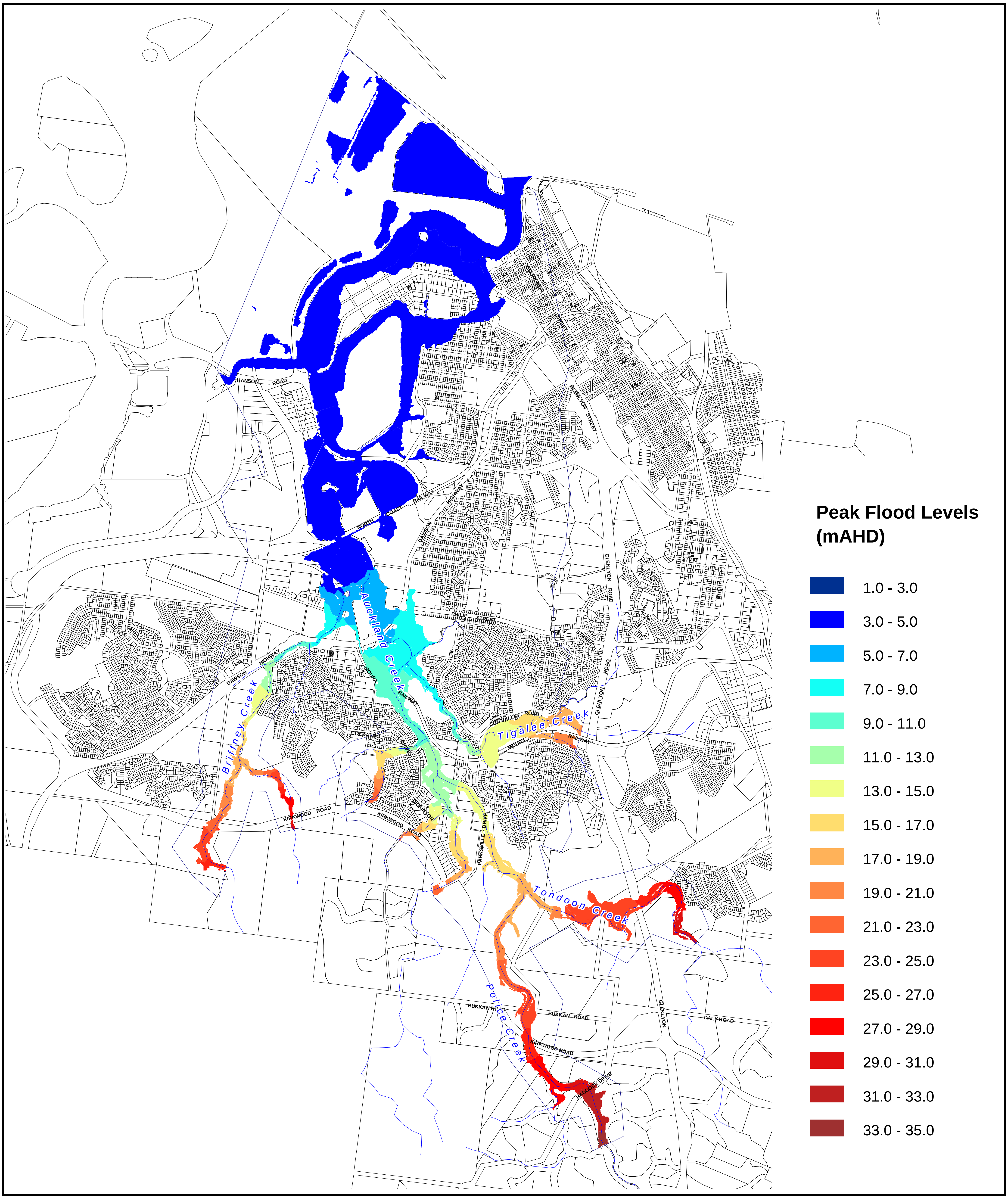
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour,
Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 16

50yr ARI Peak Flood Level
Existing Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour,
Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 17

50yr ARI Peak Flood Level
Ultimate Land Use

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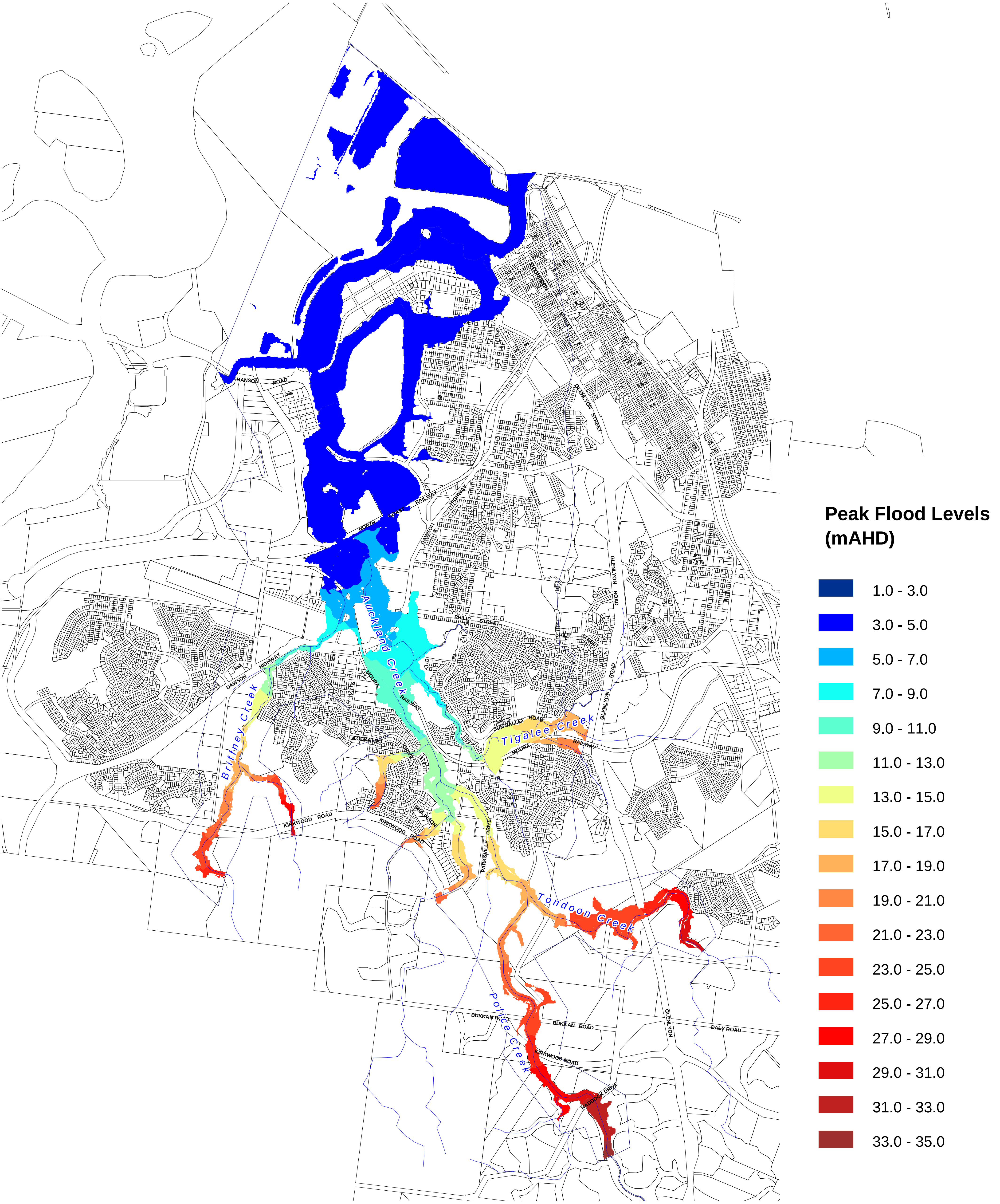

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LEGEND

Waterways

2D Modelling Boundary

North

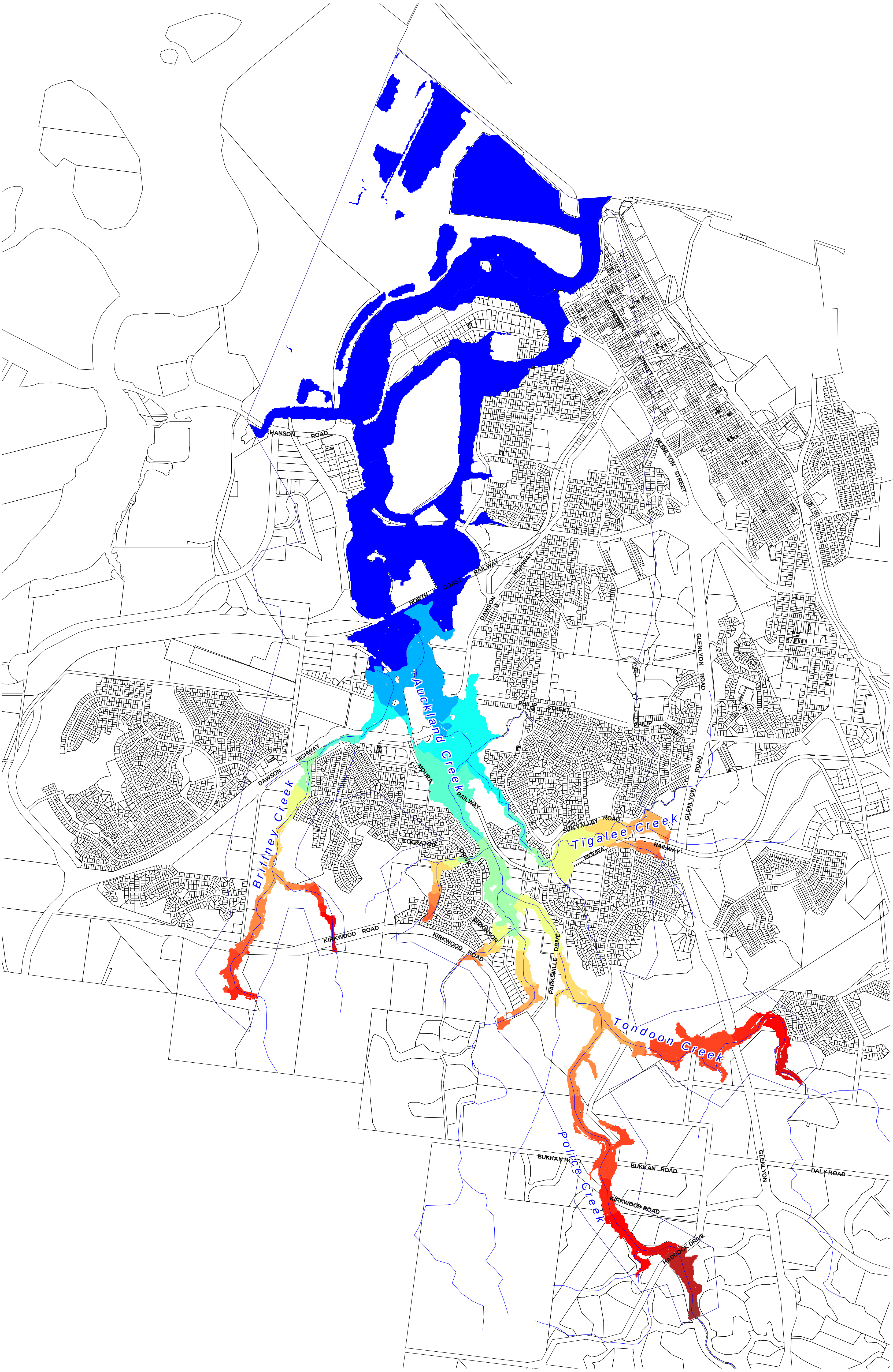
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 18

100 yr ARI Peak Flood Level Existing Land Use



Peak Flood Levels (mAHD)

🟦	1.0 - 3.0
🟦	3.0 - 5.0
🟦	5.0 - 7.0
🟦	7.0 - 9.0
🟩	9.0 - 11.0
🟩	11.0 - 13.0
🟨	13.0 - 15.0
🟨	15.0 - 17.0
🟧	17.0 - 19.0
🟧	19.0 - 21.0
🟧	21.0 - 23.0
🟥	23.0 - 25.0
🟥	25.0 - 27.0
🟥	27.0 - 29.0
🟥	29.0 - 31.0
🟫	31.0 - 33.0
🟫	33.0 - 35.0

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LEGEND

—	Waterways
▭	**2D Modelling Boundary**

North

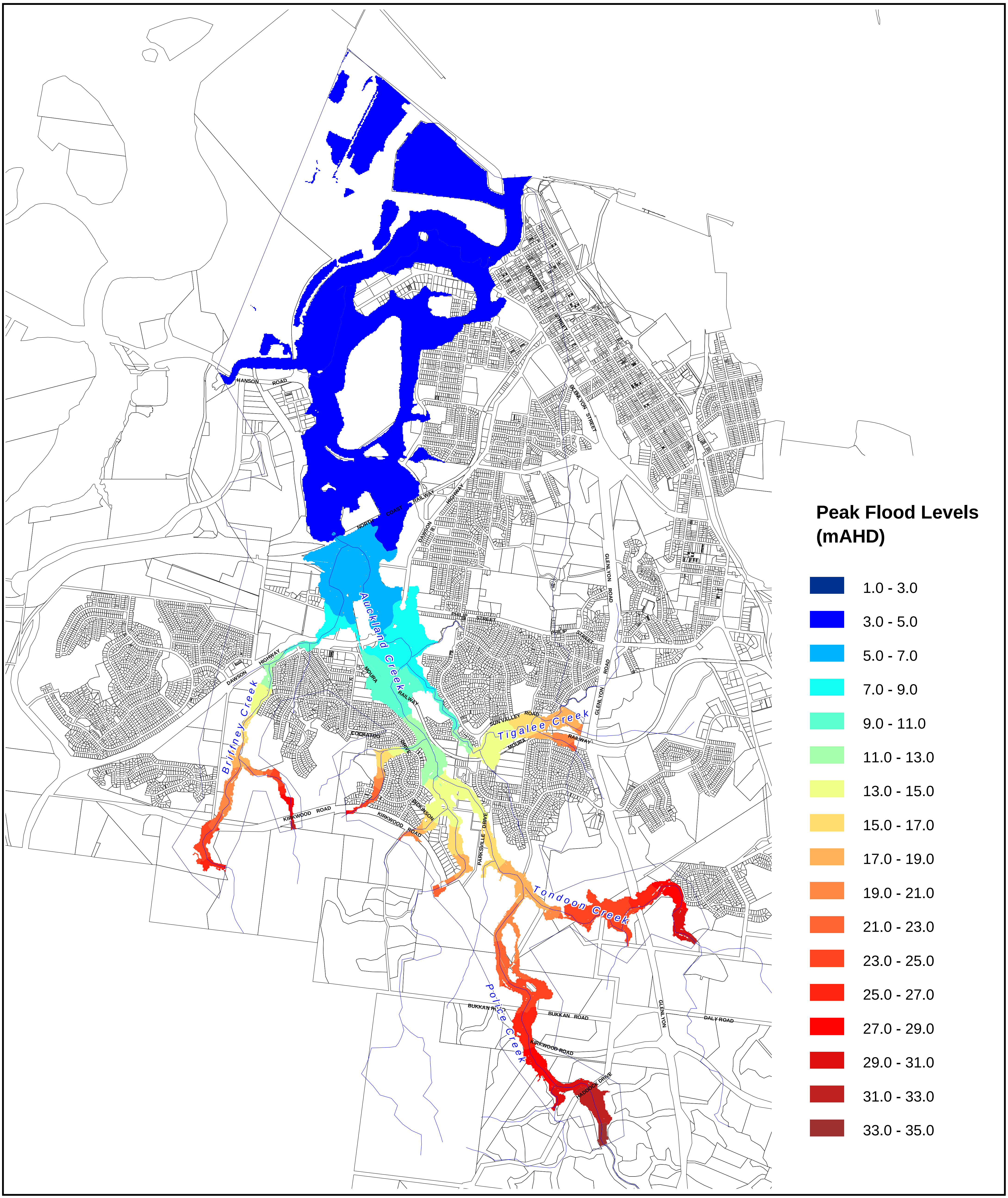
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 19

100 yr ARI Peak Flood Level
Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

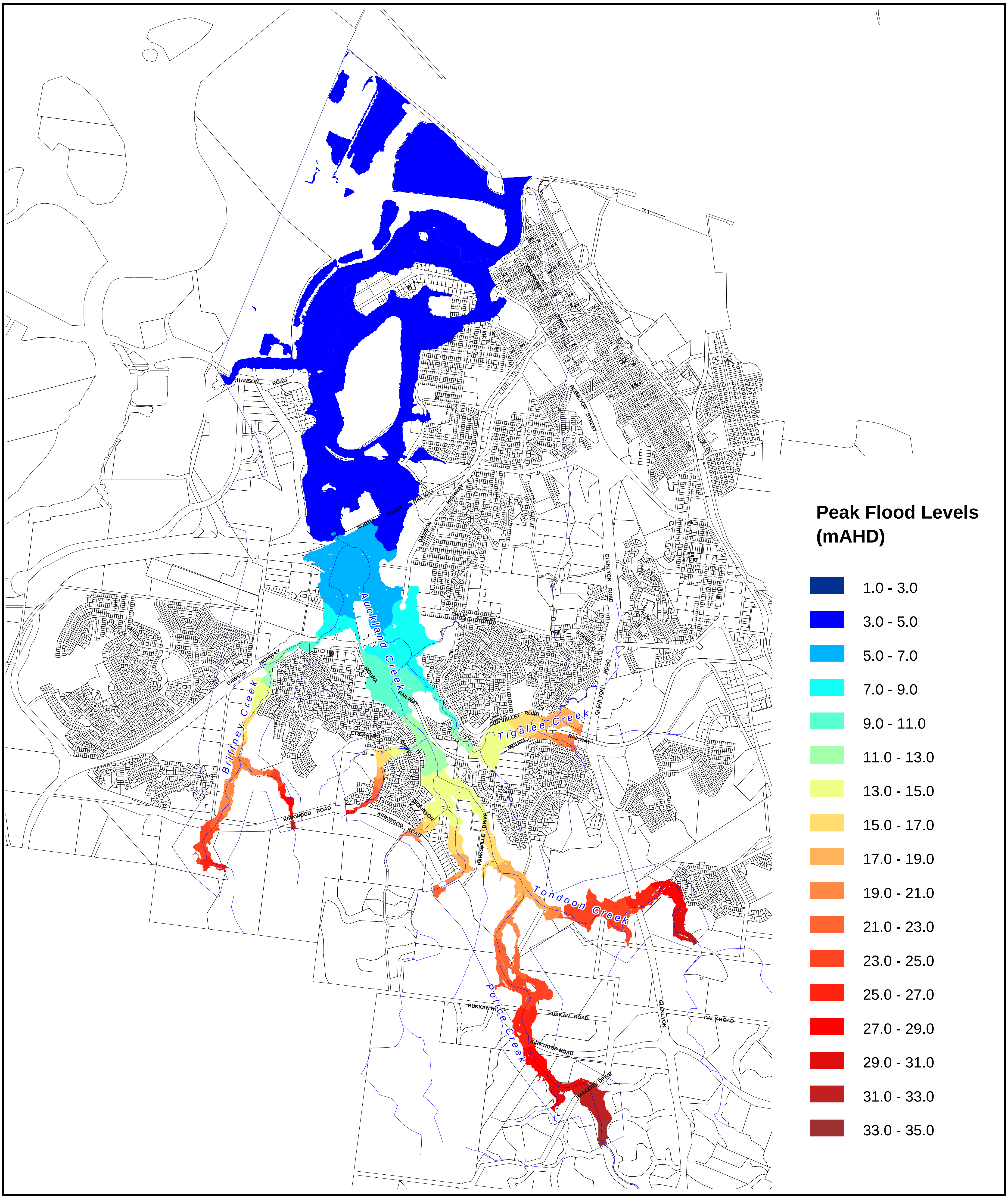
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 20

500 yr ARI Peak Flood Level
Existing Land Use



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Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

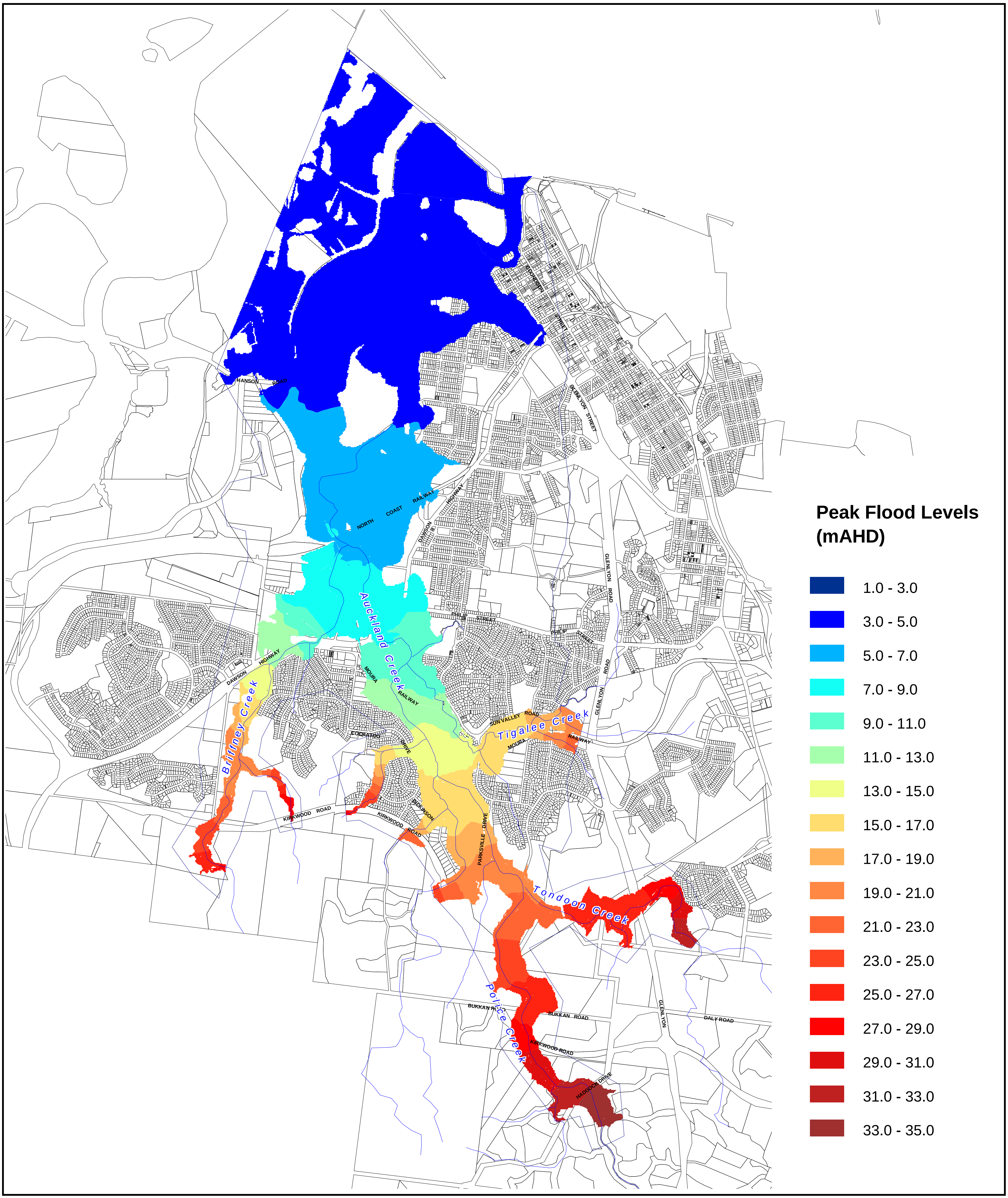
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 21

500 yr ARI Peak Flood Level
Ultimate Land Use



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Waterways

▭ **2D Modelling Boundary**

North

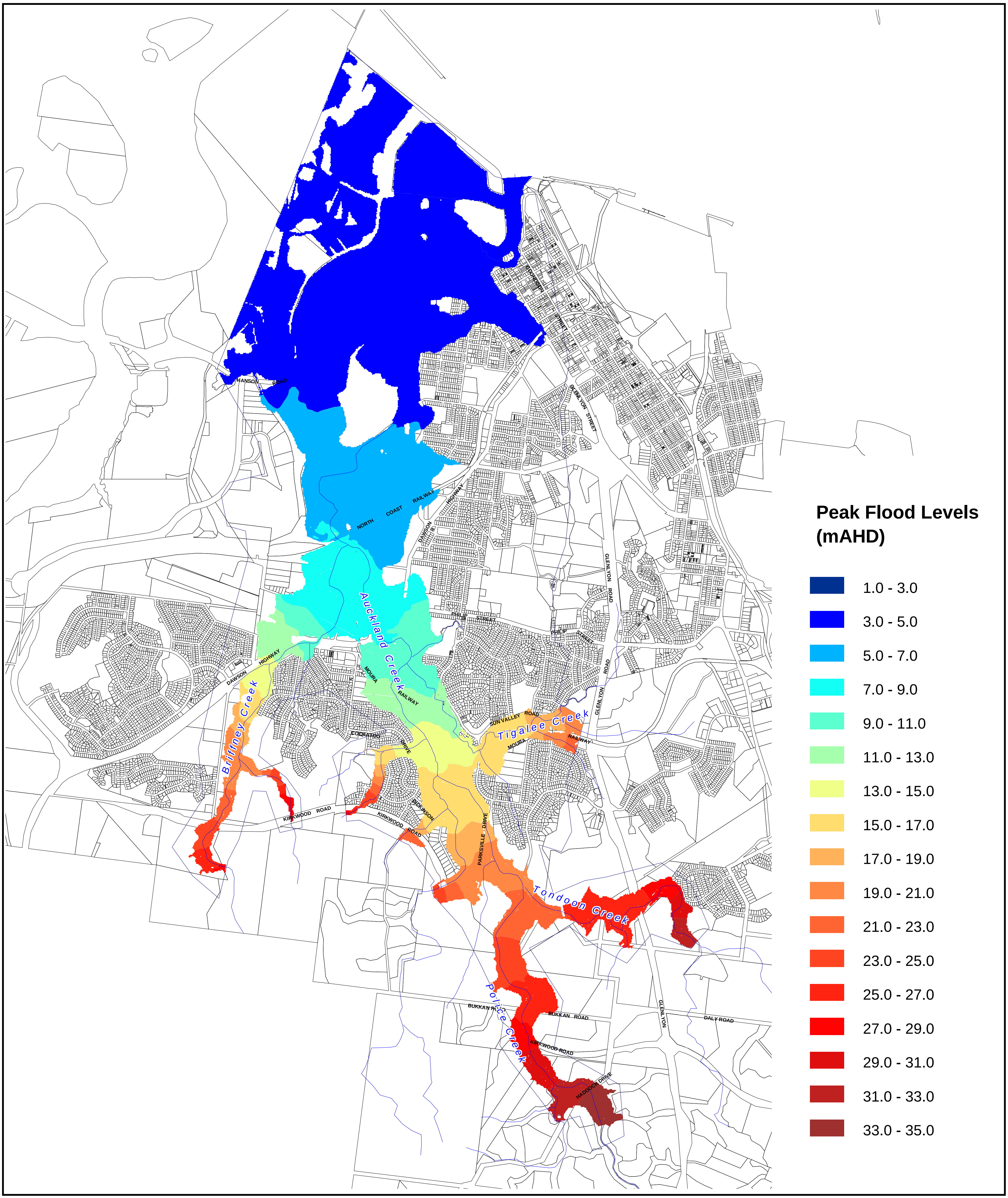
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 22

PMF Peak Flood Level
Existing Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

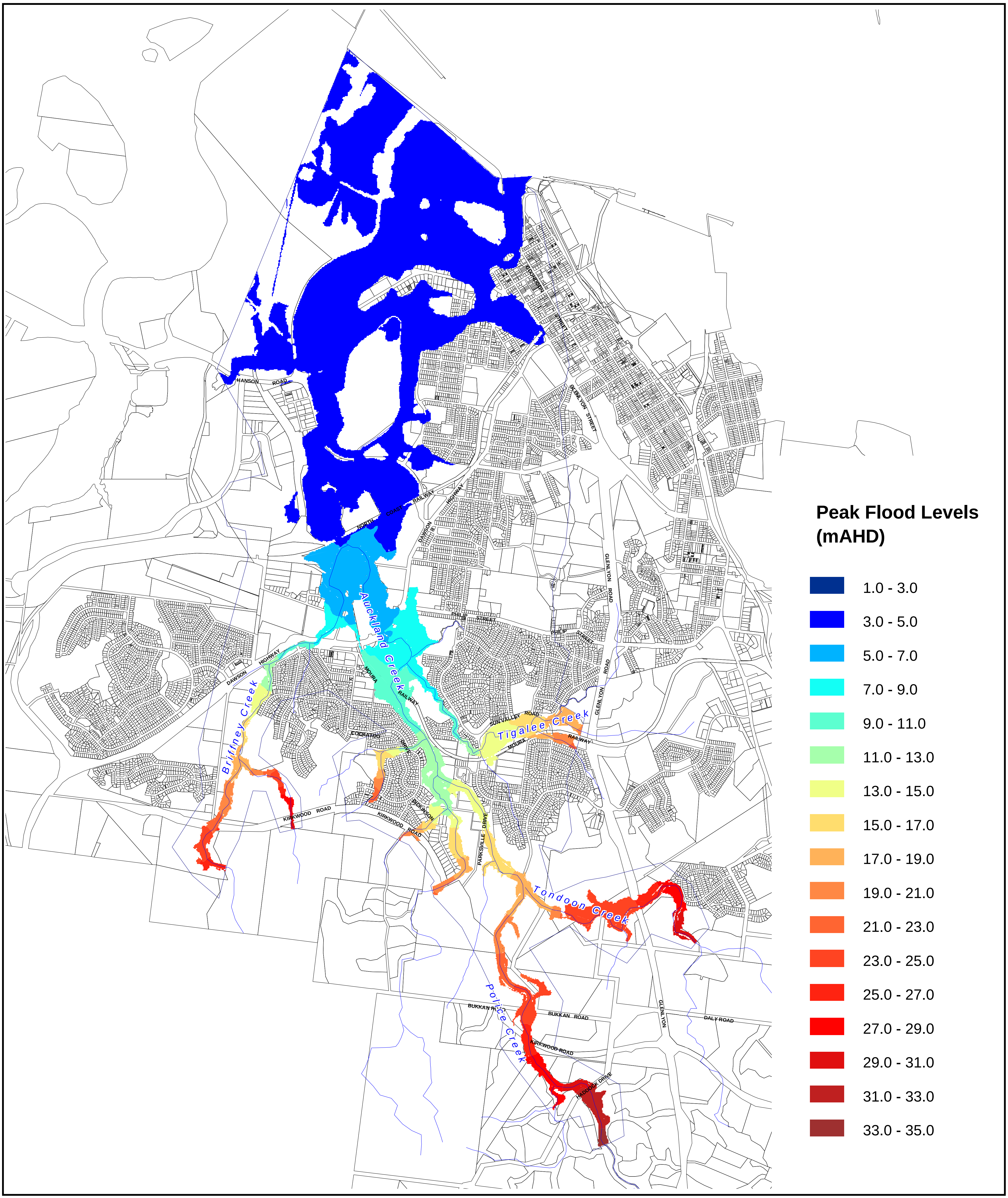
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 23

PMF Peak Flood Level
Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

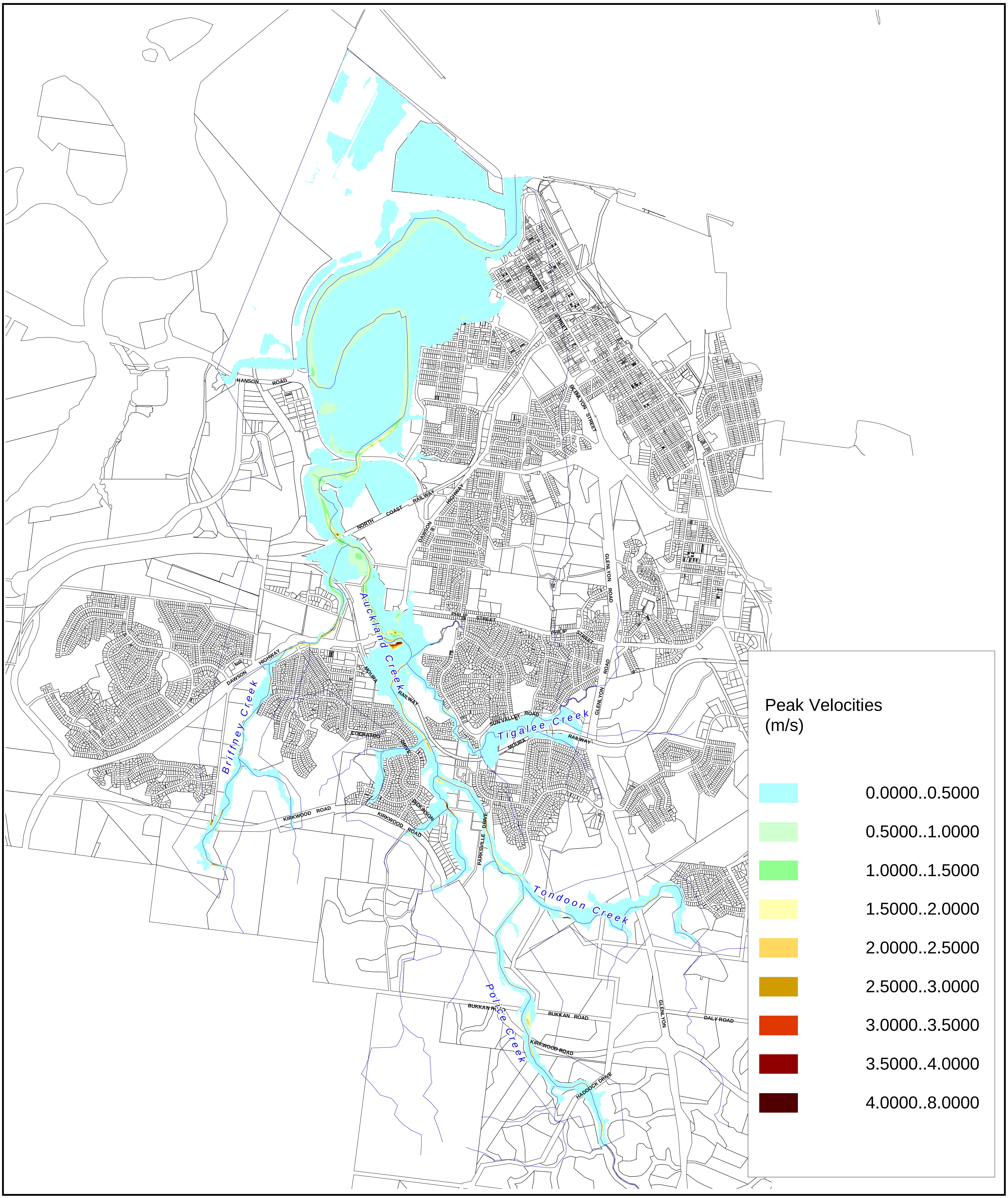
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 24

100 yr ARI Peak Flood Level
Ultimate Land Use
TWL (3.33mAHD)



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

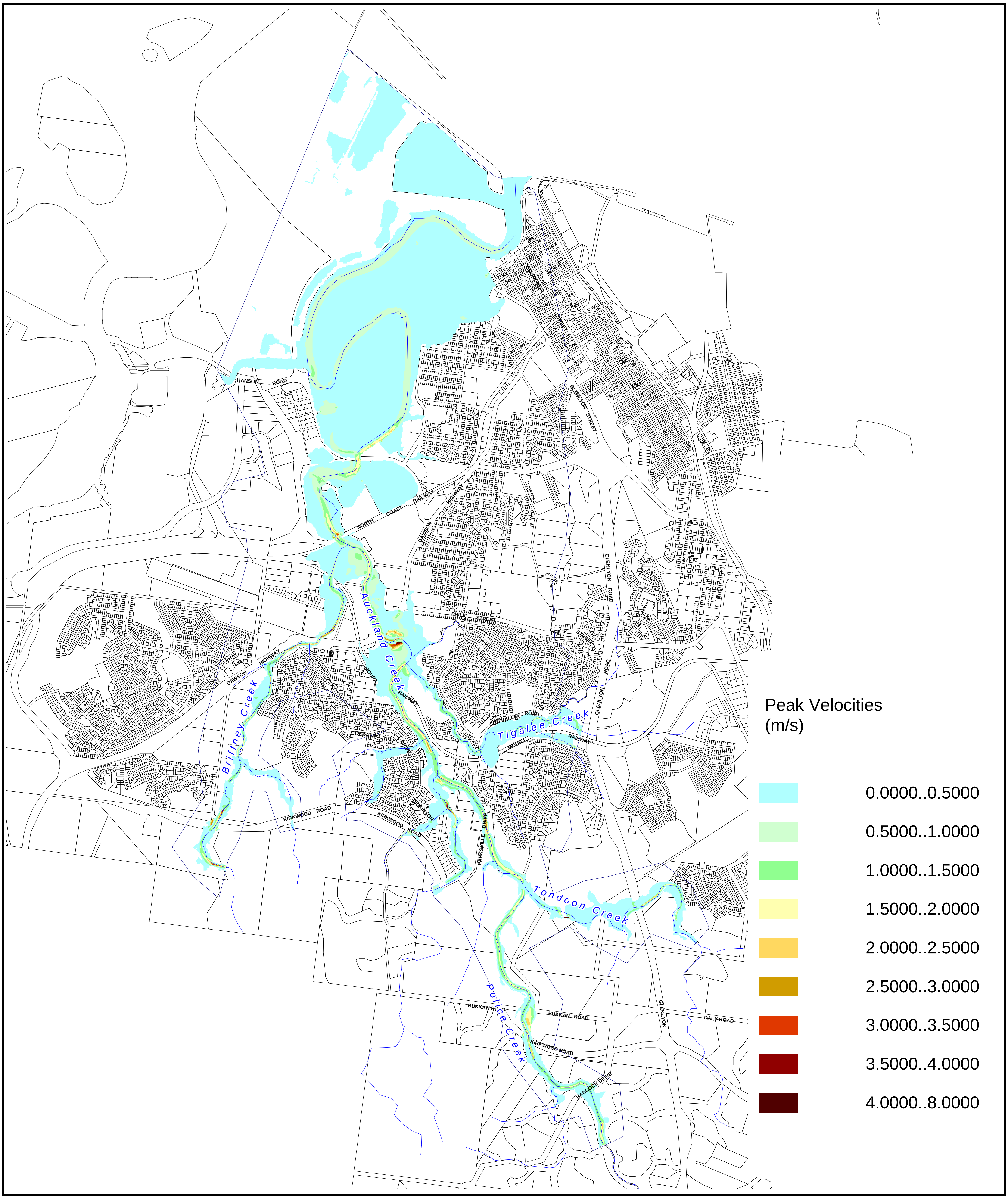
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 25

20yr ARI Peak Flood Velocity Existing Land Use



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Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

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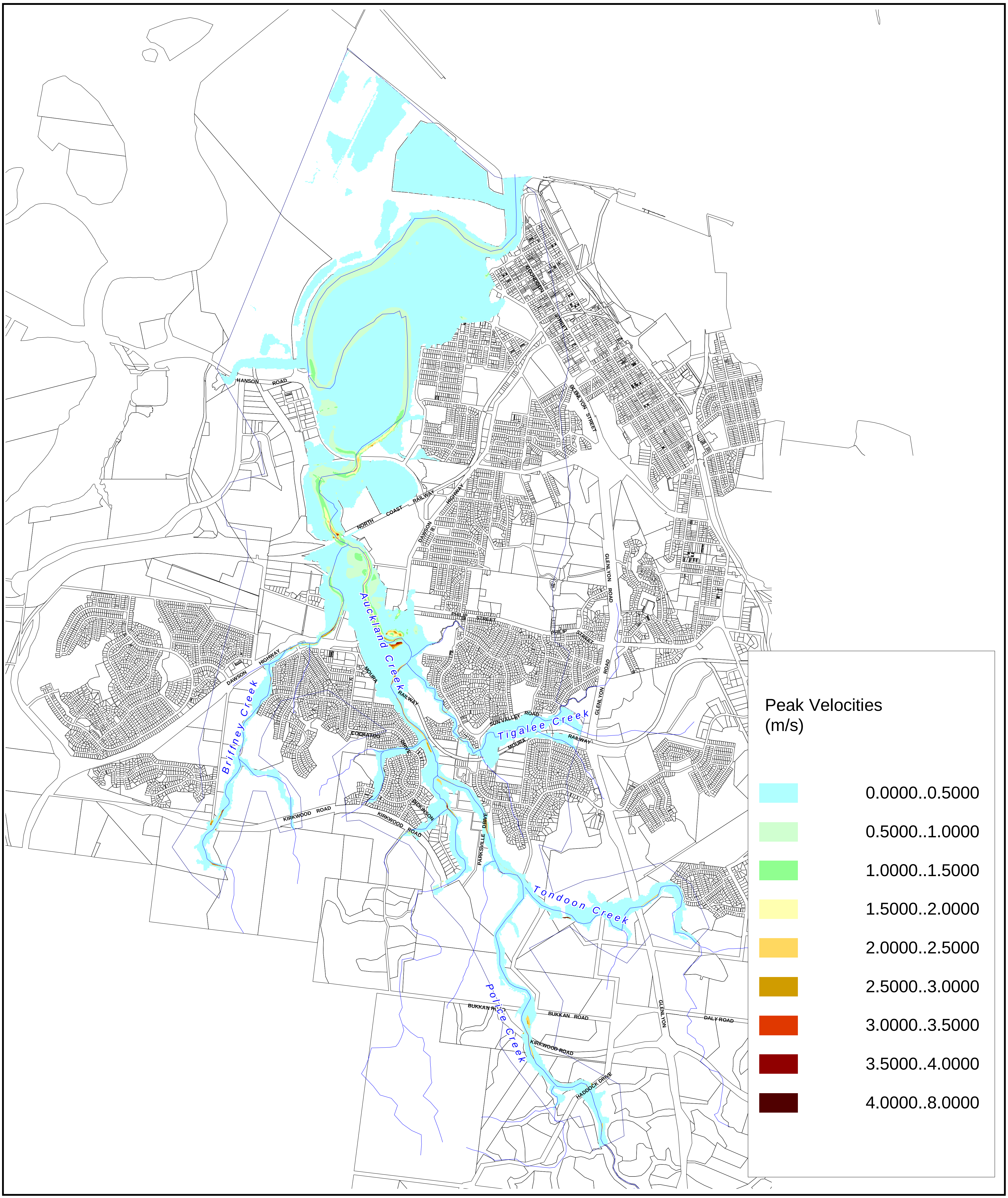
Auckland Creek Flood Study

FIGURE 26

20yr ARI Peak Flood Velocity Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

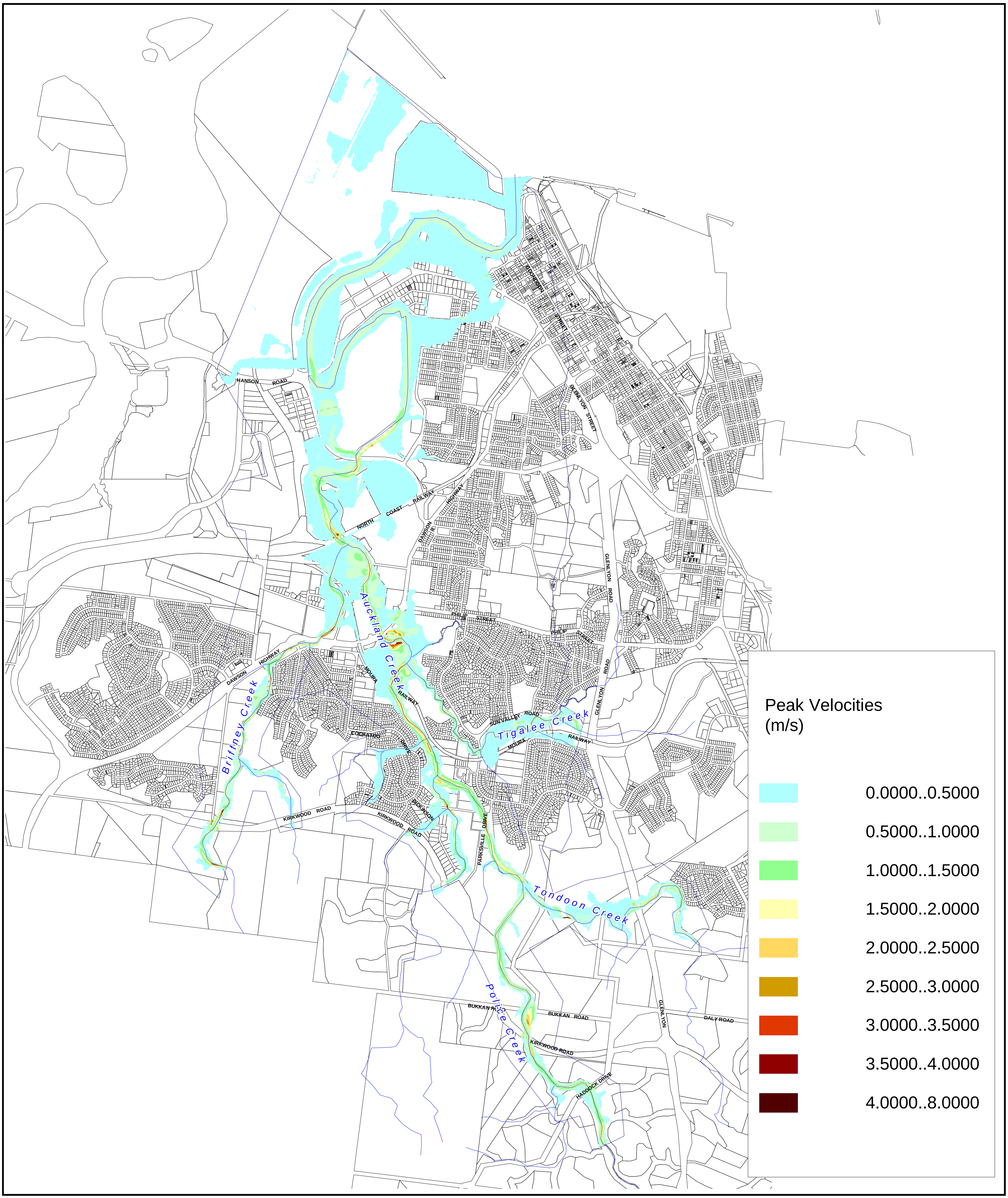
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 27

50yr ARI Peak Flood Velocity Existing Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

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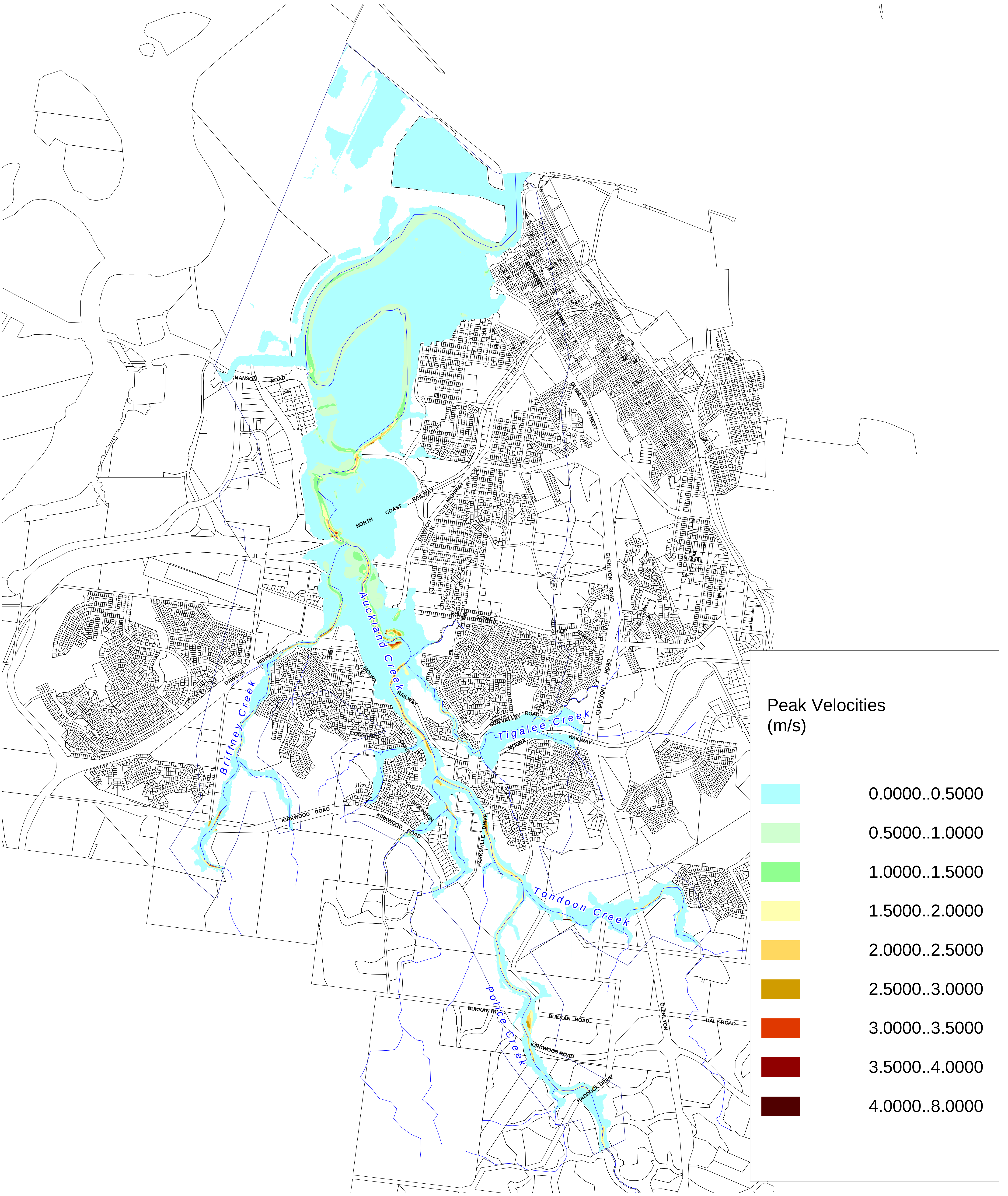
Auckland Creek Flood Study

FIGURE 28

50yr ARI Peak Flood Velocity Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

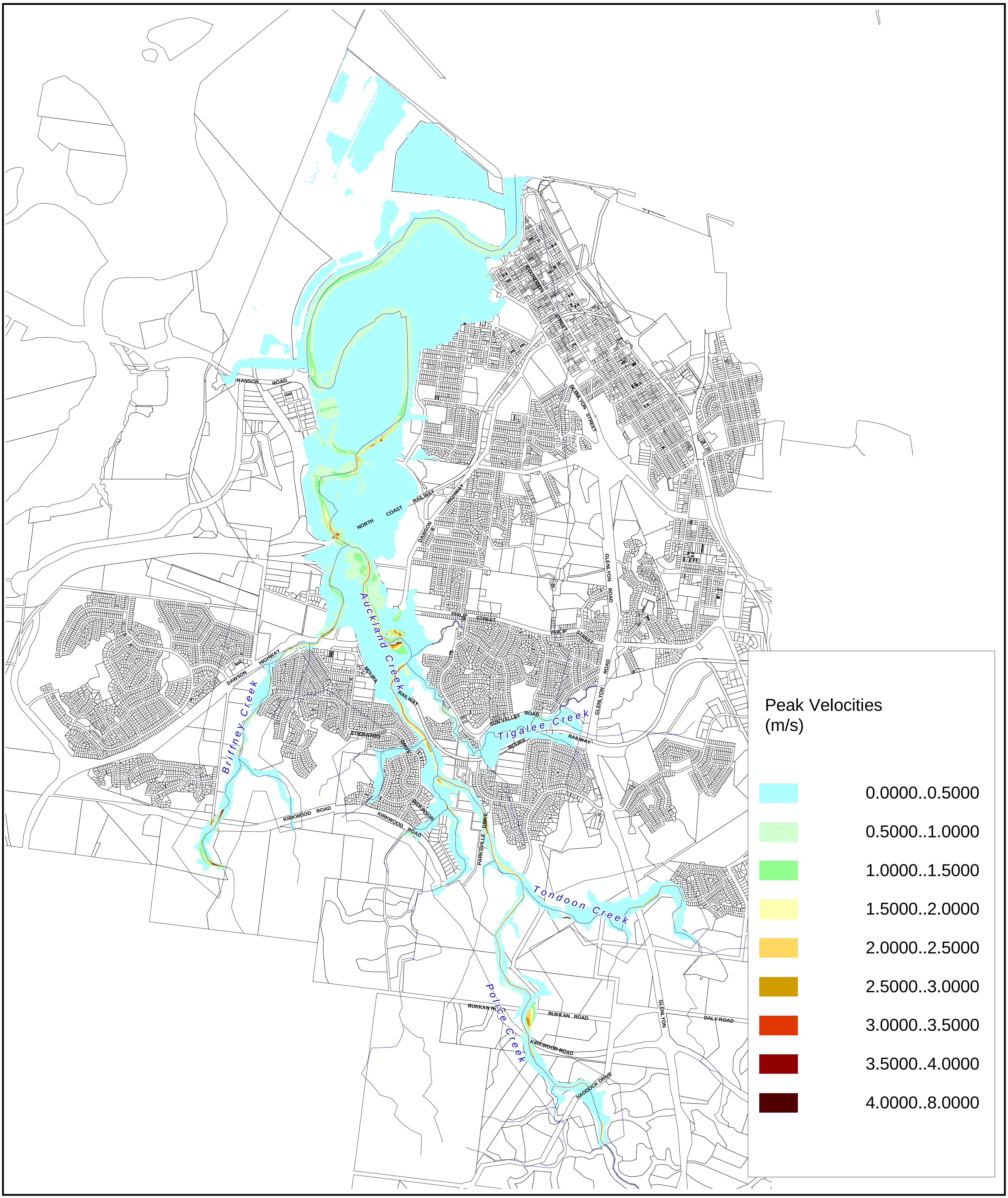
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 29

100 yr ARI Peak Flood Velocity Existing Land Use



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M:\41\14340\GIS\MapInfo\Report_Figures\Final Report\Fig29_100y3h_E_063_V_Max.WOR

LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

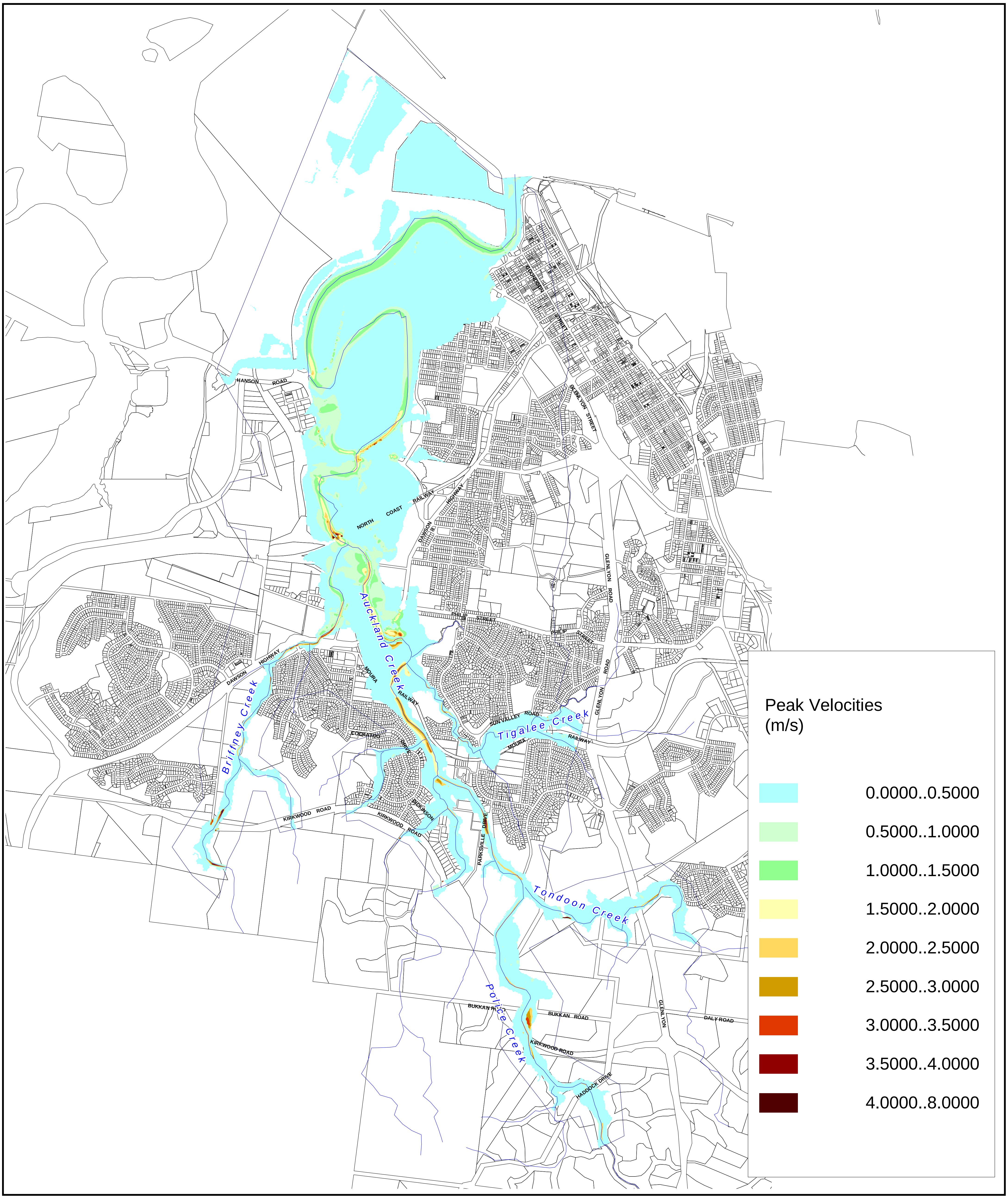
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 30

100 yr ARI Peak Flood Velocity Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

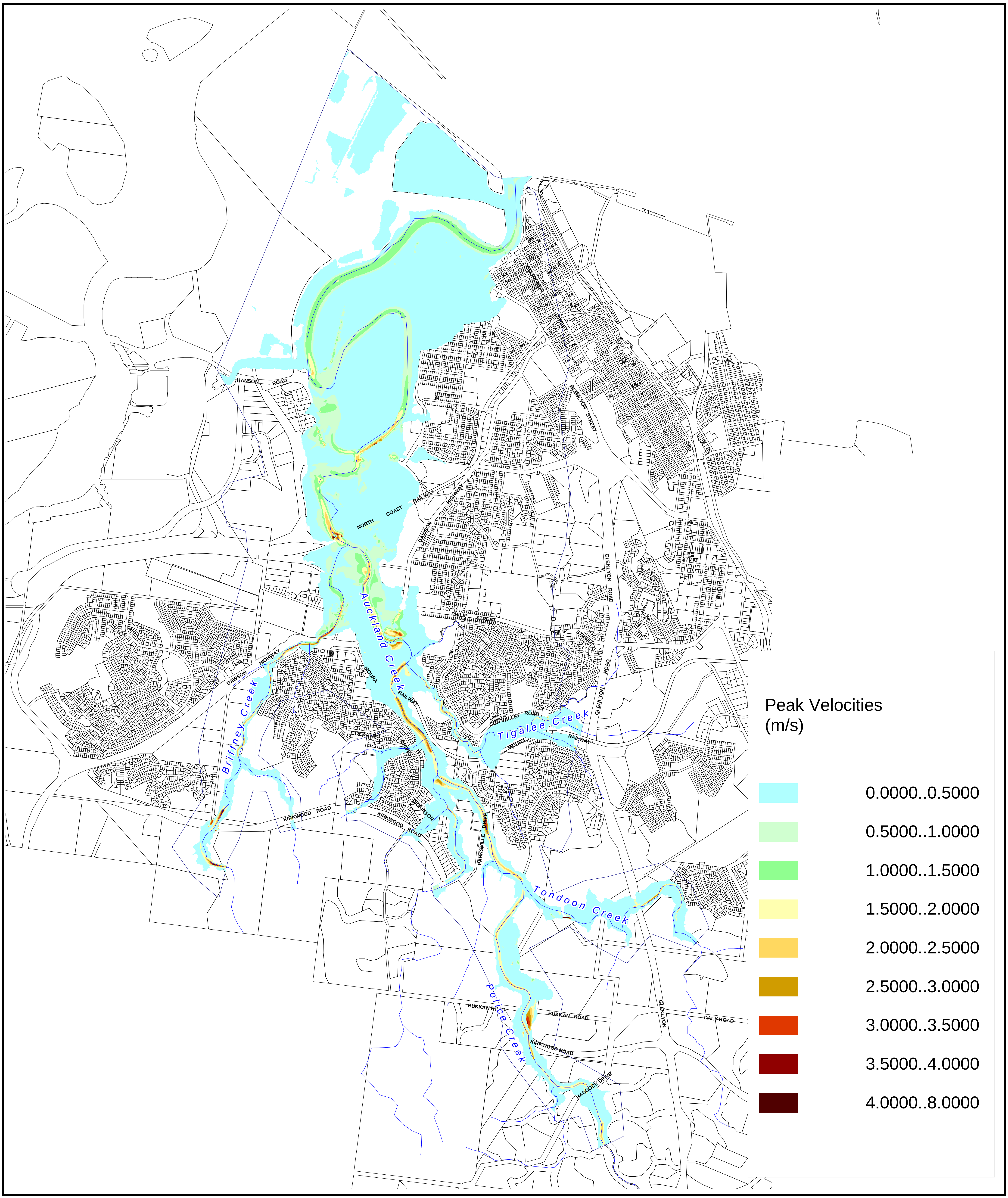
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 31

500 yr ARI Peak Flood Velocity
Existing Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

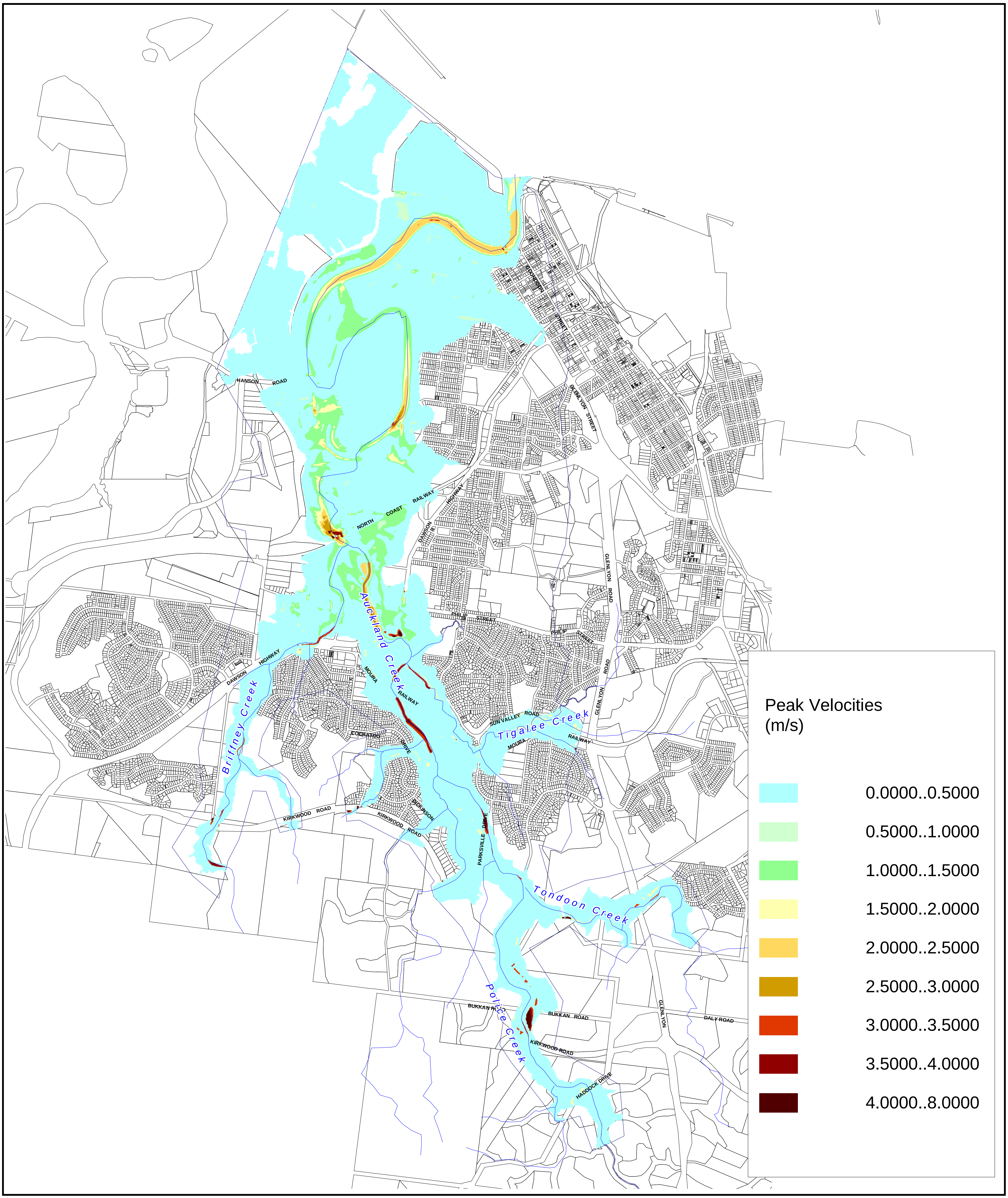
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 32

500 yr ARI Peak Flood Velocity
Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

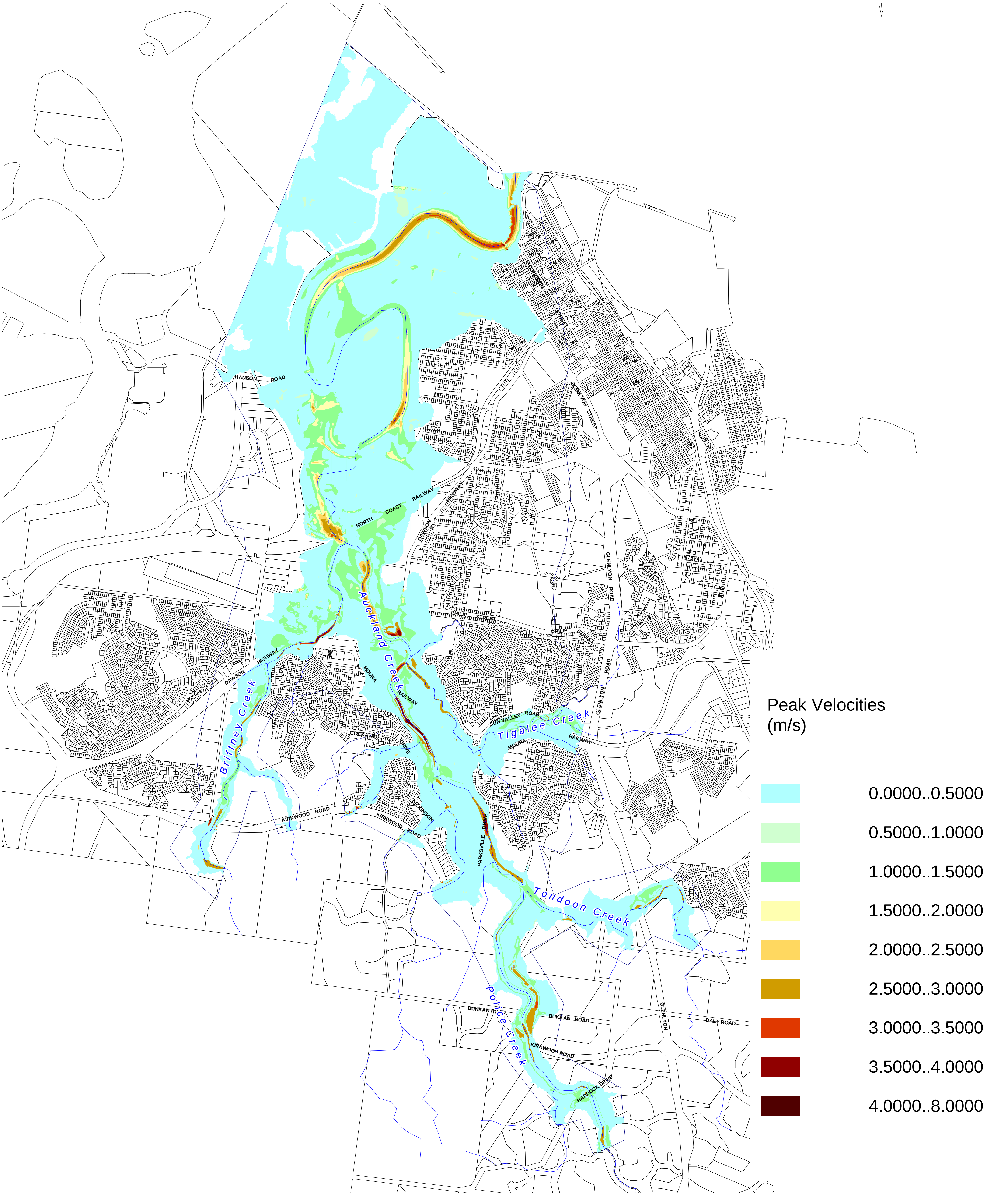
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour,
Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 33

PMF Peak Flood Velocity
Existing Land Use



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ENVIRONMENT

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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

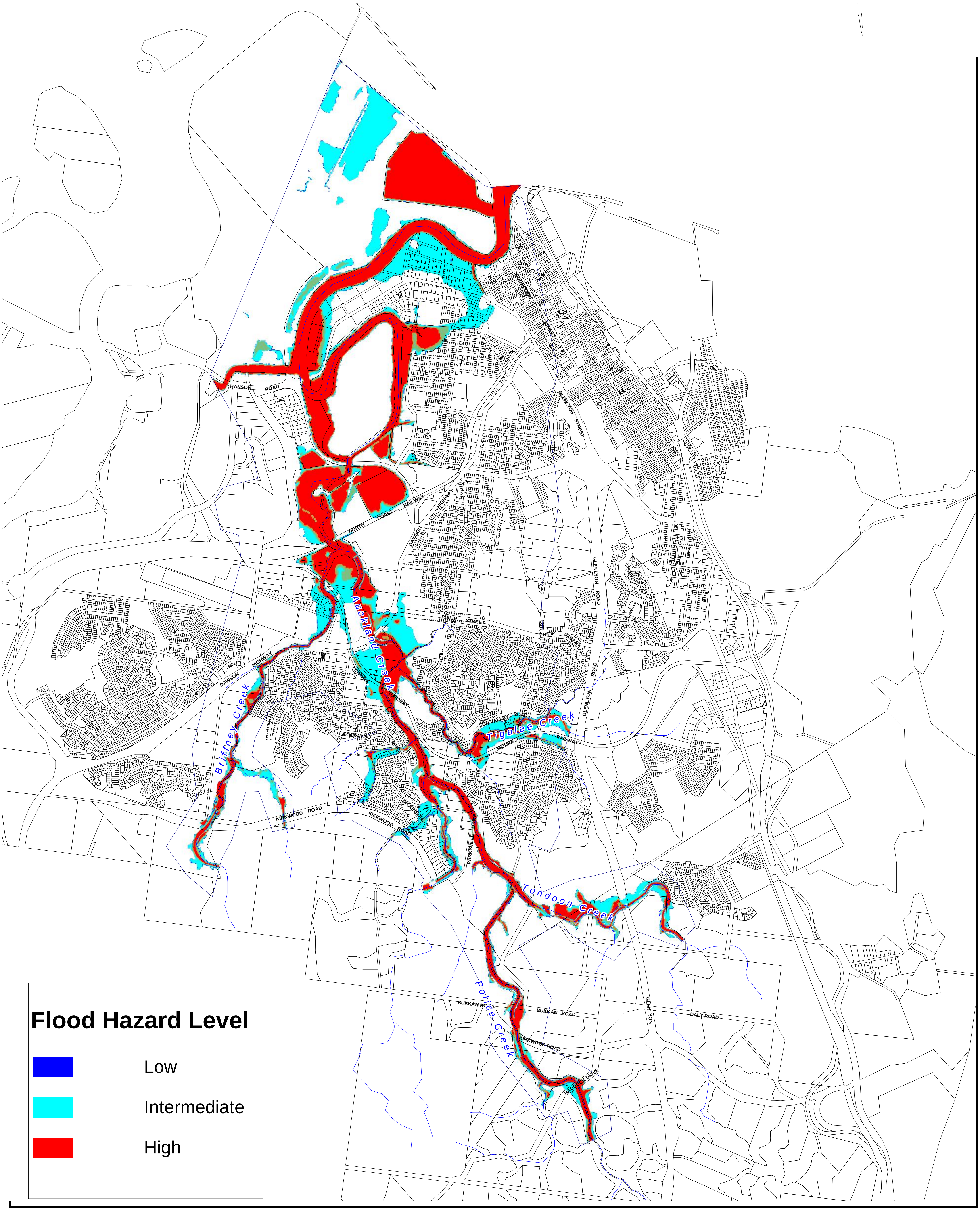
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 34

PMF Peak Flood Velocity Ultimate Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

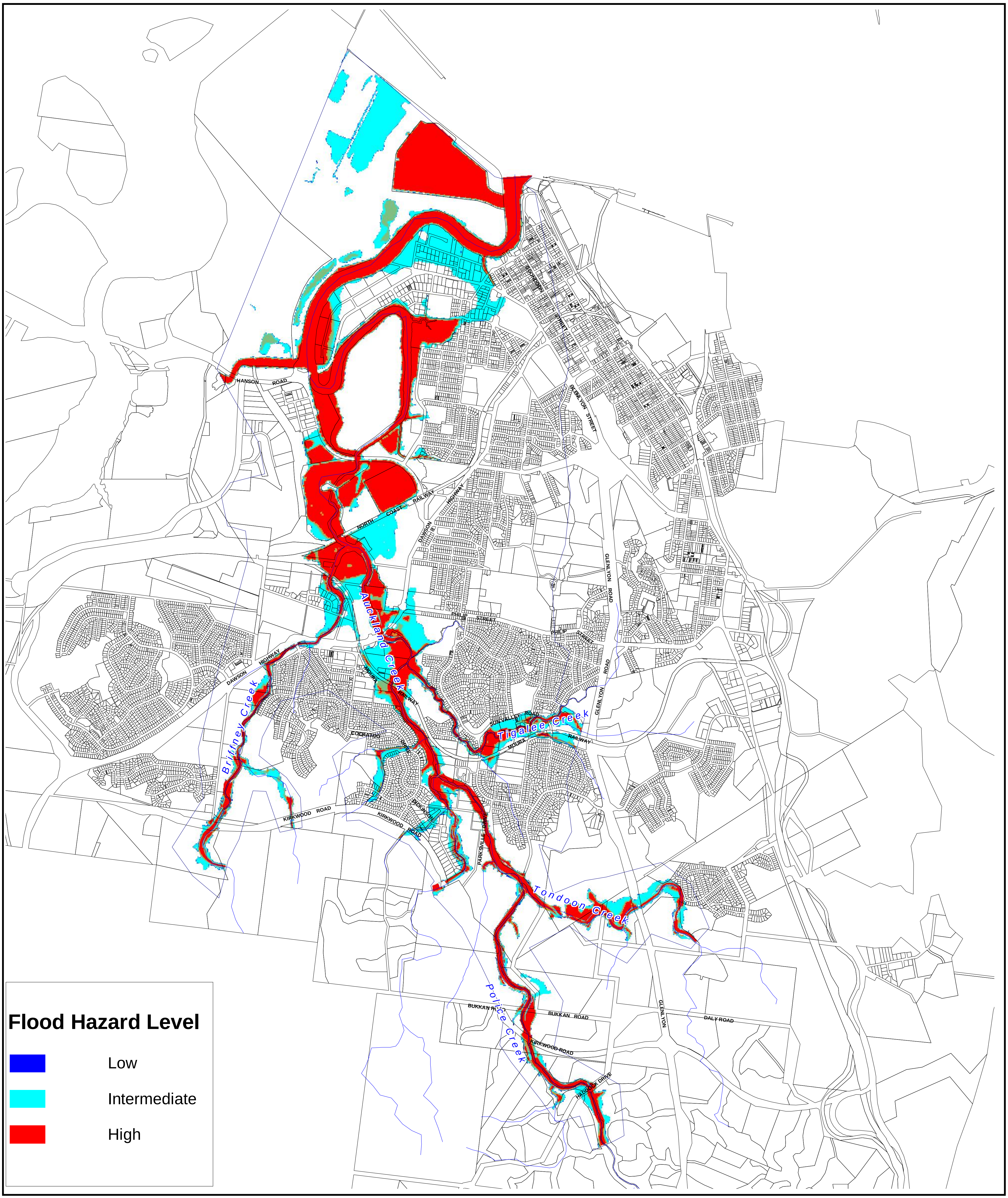
0 0.5 1 1.5 2
Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 35

50 yr ARI Flood Hazard Existing Land Use



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LEGEND

Waterways

2D Modelling Boundary

North

0 0.5 1 1.5 2

Scale in kms (1:35,000 at A3)

Source Information: DTM is based on 1m contour, Cross-section survey, QT Provided Bathymetric Data

Auckland Creek Flood Study

FIGURE 36

100 yr ARI Flood Hazard Existing Land Use

12. Conclusions and Recommendations

12.1 Summary and Conclusions

A flood risk study has been completed for the Auckland Creek catchment. This encompasses all of the main tributaries of the system, including Police Creek, Briffney Creek, Tigalee Creek, Tondoon Creek and Cathurbie Creek.

Hydrologic and hydraulic modelling were undertaken, utilising the URBS and Tuflow modelling packages. With little calibration data available, modelling efforts have focussed on the use of standard parameters, with a form of verification to the Rational Method. Use of the Tuflow model allowed flooding to be assessed in two dimensions, and dynamically (rather than the steady state model of the originally proposed HECRAS model).

Topography varies significantly within the catchment, from steep upper reaches at elevations of more than 100m AHD, to the lower tidally influenced reaches. The nature of the catchment results in the time to peak varying across the catchment. Storm durations of between 1 hr to 24 hours were compared, with the 3-hour storm considered to offer the highest flood risk in the majority of sub-catchments. However, it is worth noting that the 24 hr storm produced higher peak floods in the lower portion of the catchment below the Lake Callemondah weir and the 1 hr storm higher peaks in the upper reaches of the catchment.

Significantly, the comparison between the ultimate and the existing development cases shows only a small increase in flooding due to the development process. This is due to the relatively small increase (approximately 10%) in impervious area in the upper reaches of the catchment and may also be attributable to catchment shape.

The tailwater sensitivity analysis confirmed that flooding downstream of Lake Callemondah is controlled by the tide, whereas flooding upstream of Callemondah Weir is essentially unaffected by the tidal influence.

The performance of a large number of waterway structures has been assessed for the 50 yr and 100 yr ARI events revealing six structures that overtop. Furthermore, the assessment showed that several roads within the catchment have low flood immunity but also have low priority for upgrade.

The probable maximum flood (PMF) produces flood levels significantly higher than the 100yr event in places, with a corresponding increase in the extent of inundation.

Measures designed to mitigate the identified problem areas have been identified and tested within the hydrodynamic model to determine their relative benefit. The model results reveal that the greatest benefit comes from strategically placed retarding basins. A final suite of mitigation measures was tested following discussion and agreement with council. This was a combination of the most effective measures tested in the preliminary options.

Peak flood height and velocity maps, in conjunction with previously identified problem areas, have been used to assess the flood risk to the community. The assessment of population and property at risk has been based on the 100 yr flood event. For this event (existing conditions), it is estimated that 268 properties are at risk of some inundation, up to a depth of 2.2 metres, with most at substantially lower depths.

An infrastructure charges schedule has been proposed based on the costs involved to construct the proposed stormwater flood risk mitigation infrastructure recommended in this study. The infrastructure charge based on evenly distributing costs across the entire catchment would be $1,259 per impervious hectare.

Finally, an estimate of five and ten year construction plans has been completed. This was based on the assignment of priority to each of the proposed mitigation measures. It is estimated that approximately $ 500,000 will be required for each period.

12.2 Recommendations

Preliminary analysis of the results of the two mitigation scenarios suggests that the following mitigation measures, be implemented:

- Two detention basins (RB 031 and RB 033) within the Tigalee Creek sub-catchment which will reduce peak flow rates by approximately 60 percent in their respective reaches, providing significant flood reduction benefits downstream;
- Levee banks along Tigalee Creek (LB 131) between Mercury St and Witney St, which provide flood protection for properties in Pacific Way and Pacific Court;
- Levee banks along Phillip St (LB 121) around the Kin Kora Shopping Complex, providing flood protection for properties along Phillip St and both major shopping centres in Kin Kora;
- Levee banks adjacent to Shaw St (LB 161) downstream of the Penda Ave. crossing of Briffney Creek;
- Culvert upgrades for Cockatoo Dr., Mercury St., and Parksville Dr. designed for 50 yr ARI cross drainage;
- Addition of a culvert, which will lead to “choking” of the waterway upstream of Kirkwood Road crossing #6 over Cathurbie Creek, or provision of a retarding basin upstream of this location;
- Design of Kirkwood Road crossing #1 over Briffney Creek to maximise retarding effect or a retarding basin upstream of this crossing;
- Site investigation/survey of floor levels in the flood affected areas detailed in Table 36. Flood proofing or other site specific flood mitigation measures may be required at these sites;

Furthermore:

- It must be noted that all sizes of mitigation measures are indicative. Detailed design will be required before implementation;
- Council may consider requiring developers to restore any exposed waterway banks where development has encroached on waterway;

- Installation of depth gauges and warning signs may be required at any creek crossing still predicted to be inundated during a 100 yr ARI event following implementation of all other mitigation measures.
- Implementation of a catchment management plan in conjunction with more rigorous future developmental controls (These issues are presented in the separate Catchment Management Plan report);
- Infrastructure charges may be modified by Council to include the following: trunk and sub-divisional scale drainage; and source control costs associated with future development; implementation costs (e.g. updating the Planning Scheme for future non-engineered controls).

13. References

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Appendix A

Gladstone Rainfall

IFD Auckland Creek, Gladstone, Queensland

ARI (years)	Duration (hours)	Intensity (mm/h)
2	6 min	137
2	1	**46.9**
2	12	**9.1**
2	72	**2.8**
50	6 min	261
50	1	**87**
50	12	**20.3**
50	72	**8**

2.00E-01	Skewness (G)
4.28	Geographical factor for 6 min, 2 yr storm
17.9	Geographical factor for 6 min, 50 yr storm

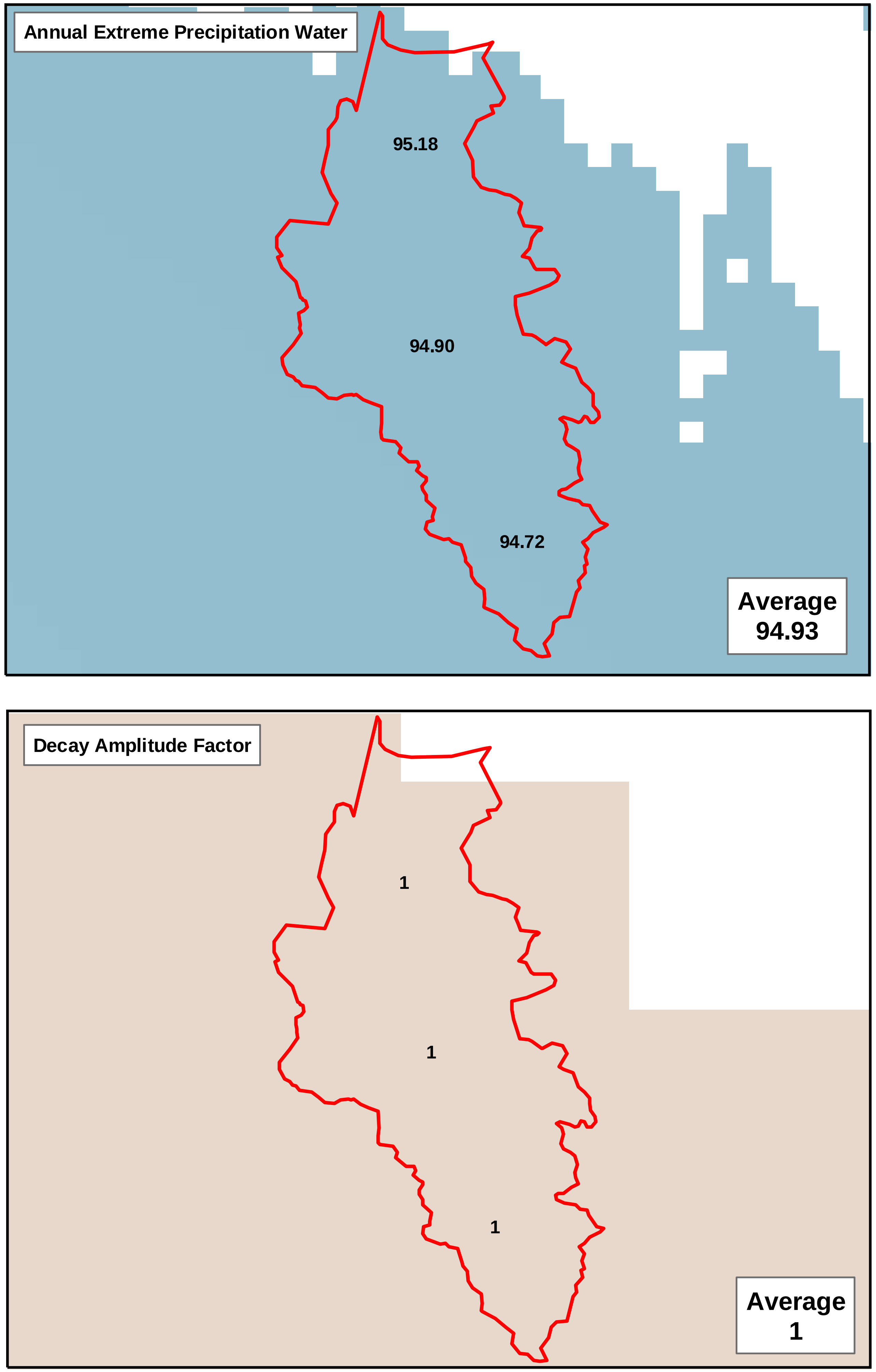
Rainfall Intensities (mm/h)

Duration	**Average Recurrence Interval (years)**							
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**
6 mins	**134.9**	**173.3**	**196.9**	**228.5**	**271.5**	**305.3**	**340.5**	**340.5**
7	127.6	163.8	186.0	215.9	256.5	288.2	321.5	367.6
8	121.4	155.7	176.8	205.2	243.6	273.7	305.2	348.9
9	116.0	148.7	168.8	195.8	232.4	261.1	291.1	332.7
10	111.2	142.5	161.7	187.5	222.5	250.0	278.6	318.4
11	106.9	137.0	155.4	180.2	213.8	240.1	267.6	305.7
12	103.1	132.0	149.7	173.6	205.9	231.2	257.6	294.3
13	99.6	127.5	144.6	167.6	198.7	223.1	248.6	284.0
14	96.5	123.4	139.9	162.1	192.2	215.8	240.4	274.6
15	93.6	119.6	135.6	157.1	186.3	209.1	232.9	265.9
16	90.9	116.2	131.7	152.5	180.8	202.9	226.0	258.0
17	88.4	113.0	128.0	148.3	175.7	197.2	219.6	250.7
18	86.1	110.0	124.6	144.3	171.0	191.9	213.7	243.9
20	81.9	104.6	118.5	137.2	162.5	182.3	203.0	231.6
25	73.5	93.8	106.1	122.8	145.4	163.0	181.4	206.9
30	67.1	85.5	96.7	111.8	132.3	148.3	165.0	188.1
35	62.0	78.9	89.2	103.1	121.9	136.6	151.9	173.1
40	57.8	73.5	83.0	95.9	113.4	127.0	141.2	160.9
45	54.2	68.9	77.9	90.0	106.3	119.0	132.3	150.7
50	51.2	65.1	73.5	84.9	100.2	112.2	124.7	142.0
55	48.6	61.7	69.7	80.4	95.0	106.3	118.1	134.5
60	**46.3**	**58.8**	**66.3**	**76.6**	**90.4**	**101.2**	**112.4**	**127.9**
75	40.2	51.3	58.1	67.3	79.7	89.4	99.6	113.6
90	35.7	45.8	52.1	60.5	71.8	80.8	90.1	103.0
2 hours	29.5	38.3	43.7	51.0	60.8	68.6	76.8	88.1
3	22.6	29.6	34.0	39.9	48.0	54.4	61.1	70.5
4	18.6	24.7	28.5	33.5	40.5	46.1	51.9	60.1
4.5	17.2	22.9	26.5	31.2	37.8	43.0	48.6	56.4
5	16.0	21.4	24.8	29.3	35.5	40.5	45.8	53.2
6	14.2	19.1	22.2	26.3	31.9	36.5	41.3	48.1
7	12.8	17.3	20.2	23.9	29.2	33.4	37.8	44.2
8	11.7	15.9	18.6	22.1	27.0	30.9	35.1	41.0
9	10.9	14.7	17.3	20.6	25.2	28.9	32.9	38.5
10	10.1	13.8	16.2	19.3	23.7	27.2	31.0	36.3
12	**9.0**	**12.3**	**14.5**	**17.3**	**21.3**	**24.5**	**28.0**	**32.8**
14	8.1	11.3	13.3	16.0	19.8	22.8	26.1	30.8
15	7.8	10.8	12.8	15.4	19.1	22.1	25.3	29.9
16	7.5	10.4	12.4	14.9	18.5	21.5	24.6	29.1
18	7.0	9.8	11.6	14.1	17.5	20.3	23.4	27.7
21	6.3	8.9	10.7	13.0	16.3	18.9	21.8	25.9
24	5.8	8.3	10.0	12.1	15.2	17.8	20.5	24.5
30	5.0	7.3	8.8	10.8	13.6	16.0	18.5	22.2
36	4.5	6.5	7.9	9.8	12.4	14.6	17.0	20.5
48	3.7	5.5	6.7	8.3	10.7	12.6	14.8	18.0
60	3.1	4.7	5.9	7.3	9.4	11.2	13.2	16.1
72	**2.7**	**4.2**	**5.2**	**6.5**	**8.5**	**10.2**	**12.0**	**14.7**

IFD Auckland Creek, Gladstone, Queensland

Rainfall Totals (mm)

Duration	**Average Recurrence Interval (years)**							
	2	**5**	**10**	**20**	**50**	**100**	**200**	**500**
6 mins	**13.5**	**17.3**	**19.7**	**22.9**	**27.2**	**30.5**	**34.1**	**34.1**
7	14.9	19.1	21.7	25.2	29.9	33.6	37.5	42.9
8	16.2	20.8	23.6	27.4	32.5	36.5	40.7	46.5
9	17.4	22.3	25.3	29.4	34.9	39.2	43.7	49.9
10	18.5	23.8	26.9	31.3	37.1	41.7	46.4	53.1
11	19.6	25.1	28.5	33.0	39.2	44.0	49.1	56.0
12	20.6	26.4	29.9	34.7	41.2	46.2	51.5	58.9
13	21.6	27.6	31.3	36.3	43.1	48.3	53.9	61.5
14	22.5	28.8	32.6	37.8	44.9	50.4	56.1	64.1
15	23.4	29.9	33.9	39.3	46.6	52.3	58.2	66.5
16	24.2	31.0	35.1	40.7	48.2	54.1	60.3	68.8
17	25.0	32.0	36.3	42.0	49.8	55.9	62.2	71.0
18	25.8	33.0	37.4	43.3	51.3	57.6	64.1	73.2
20	27.3	34.9	39.5	45.7	54.2	60.8	67.7	77.2
25	30.6	39.1	44.2	51.2	60.6	67.9	75.6	86.2
30	33.5	42.7	48.3	55.9	66.1	74.1	82.5	94.0
35	36.1	46.0	52.0	60.1	71.1	79.7	88.6	101.0
40	38.5	49.0	55.3	64.0	75.6	84.7	94.2	107.3
45	40.7	51.7	58.4	67.5	79.7	89.3	99.2	113.0
50	42.7	54.2	61.2	70.7	83.5	93.5	103.9	118.3
55	44.6	56.6	63.9	73.7	87.1	97.5	108.3	123.3
60	**46.3**	**58.8**	**66.3**	**76.6**	**90.4**	**101.2**	**112.4**	**127.9**
75	50.2	64.1	72.6	84.1	99.6	111.8	124.5	142.1
90	53.5	68.8	78.1	90.7	107.8	121.1	135.1	154.6
2 hours	59.1	76.6	87.4	101.9	121.6	137.2	153.5	176.2
3	67.7	88.8	102.1	119.8	143.9	163.1	183.3	211.5
4	74.5	98.7	114.0	134.2	162.1	184.2	207.7	240.6
4.5	77.5	103.0	119.2	140.6	170.1	193.7	218.6	253.6
5	80.2	107.0	124.1	146.6	177.7	202.5	228.8	265.9
6	85.2	114.4	133.0	157.6	191.6	218.8	247.7	288.5
7	89.8	121.0	141.1	167.5	204.2	233.6	264.9	309.2
8	93.9	127.1	148.5	176.7	215.9	247.3	280.9	328.3
9	97.7	132.7	155.4	185.2	226.7	260.1	295.7	346.2
10	101.2	137.9	161.8	193.1	236.8	272.1	309.7	363.1
12	**107.6**	**147.5**	**173.6**	**207.7**	**255.5**	**294.1**	**335.4**	**394.2**
14	114.1	157.7	186.5	224.0	276.8	319.6	365.5	431.1
15	117.1	162.5	192.6	231.7	286.9	331.7	379.9	448.7
16	120.0	167.2	198.5	239.2	296.7	343.4	393.7	465.8
18	125.5	175.9	209.6	253.3	315.3	365.8	420.3	498.6
21	132.9	187.9	224.9	273.0	341.3	397.2	457.7	544.8
24	139.7	198.9	239.0	291.1	365.3	426.4	492.5	588.1
30	151.3	218.3	264.1	323.5	408.8	479.2	555.9	667.2
36	161.2	234.9	285.9	351.9	447.1	526.2	612.6	738.4
48	176.9	262.3	322.2	399.7	512.5	606.9	710.7	862.8
60	188.8	283.8	351.3	438.5	566.6	674.2	793.3	968.6
72	**197.8**	**300.9**	**374.9**	**470.6**	**611.9**	**731.4**	**864.0**	**1060.2**



Gladstone City Council

Auckland Creek Flood Study

PMP Estimation Parameters

Legend

▭ Catchment Boundary

0 0.5 1 2 3 4
Kilometers

1:125,000

Source Information: BOM, 2004.
Projection: Geographic, GDA 94
Date Printed: 24/02/05
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Appendix B

Peak Flow Estimates

Auckland Creek Existing Flows

Table 51 Estimated Flows for Auckland Creek Existing Case (URBS)

Catchment	"Q2" (m^3/s)	"Q5" (m^3/s)	"Q10" (m^3/s)	"Q20" (m^3/s)	"Q50" (m^3/s)	"Q100" (m^3/s)	"Q500" (m^3/s)	"QPMP" (m^3/s)
1	27	45	58	75	87	104	147	442
2	19	31	40	50	56	67	95	274
3	46	75	96	121	139	167	239	723
4	46	75	96	122	141	167	235	717
5	18	31	41	52	60	71	101	297
6	59	97	126	160	185	219	310	928
7	19	31	41	51	59	71	101	299
8	70	116	150	192	223	264	374	1090
9	70	114	150	196	226	267	376	1108
10	13	22	28	35	39	46	66	194
11	17	28	36	46	53	63	91	276
12	7.9	13	16	21	23	27	38	109
13	24	39	49	62	71	84	119	357
14	25	40	50	64	74	87	123	358
15	9.2	15	19	23	26	30	41	113
16	29	46	58	75	86	102	145	418
17	30	47	59	76	88	104	148	430
18	99	160	209	272	313	371	523	1538
19	5.2	8.7	11	14	16	19	26	75
20	99	160	211	274	316	374	524	1553
21	13	21	27	35	39	46	65	192
22	5.9	10	14	17	20	24	33	100
23	19	32	41	53	62	74	104	310
24	23	37	48	61	71	85	121	364
25	23	38	50	64	74	89	127	390
26	3.7	6.1	7.9	10	12	14	20	59

Catchment	**"Q2" (m^3/s)**	**"Q5" (m^3/s)**	**"Q10" (m^3/s)**	**"Q20" (m^3/s)**	**"Q50" (m^3/s)**	**"Q100" (m^3/s)**	**"Q500" (m^3/s)**	**"QPMP" (m^3/s)**
27	27	44	57	73	84	100	144	444
28	114	184	243	320	370	438	612	1809
129	4.7	8.7	11	13	14	17	23	67
29	9.9	16	20	25	29	34	48	142
30	112	184	245	318	372	442	619	1853
31	19	32	37	43	45	53	74	215
32	22	36	43	52	55	64	88	236
33	17	27	33	42	47	56	79	232
34	18	28	35	43	49	58	82	239
35	35	56	70	88	99	117	166	483
36	8.3	14	18	23	25	30	42	126
37	37	57	72	89	103	124	175	529
38	11	17	21	25	27	31	44	124
39	37	57	71	89	102	121	170	521
40	0.9	1.5	2	2.6	3	3.5	4.9	15
41	129	215	286	374	438	520	733	2243
42	120	199	264	347	409	485	690	2113
43	19	32	41	51	56	66	91	250
44	21	32	40	50	55	64	89	251
45	19	31	39	49	57	66	92	248
46	8.2	14	18	23	26	30	43	127
47	25	40	52	68	77	91	127	366
48	6.3	9.5	12	14	15	18	25	69
49	27	43	57	74	85	99	138	402
50	6.9	11	14	17	19	22	31	90
51	32	50	66	88	101	120	168	500
52	137	226	300	395	469	558	792	2435
53	133	219	292	385	457	544	775	2378

Catchment	"Q2" (m^3/s)	"Q5" (m^3/s)	"Q10" (m^3/s)	"Q20" (m^3/s)	"Q50" (m^3/s)	"Q100" (m^3/s)	"Q500" (m^3/s)	"QPMP" (m^3/s)
54	4.7	7.6	9.7	12	14	16	23	69
55	12	19	23	29	33	40	56	169
56	133	221	295	389	467	554	788	2444
57	119	197	264	351	422	504	720	2259
58	119	196	262	349	421	504	721	2272
59	101	170	227	301	367	439	630	2020
60	11	17	21	26	30	35	50	148
61	102	172	228	303	371	442	633	2038

Auckland Creek Ultimate Flows

Table 52 Estimated Flows for Auckland Creek Ultimate Case (URBS)

Catchment	"Q2" (m^3/s)	"Q5" (m^3/s)	"Q10" (m^3/s)	"Q20" (m^3/s)	"Q50" (m^3/s)	"Q100" (m^3/s)	"Q500" (m^3/s)	"QPMP" (m^3/s)
1	37	58	72	89	101	120	170	520
2	25	44	55	68	74	88	123	357
3	56	88	111	139	160	191	272	828
4	56	88	110	138	156	187	268	818
5	26	40	50	62	70	83	120	367
6	71	110	138	173	199	235	332	1016
7	20	34	44	55	63	75	107	312
8	83	130	163	205	239	282	399	1192
9	80	125	157	199	234	278	394	1149
10	16	27	34	41	45	54	74	218
11	19	30	38	48	55	65	94	293
12	8.8	14	17	21	24	28	39	114
13	27	41	52	64	73	87	122	379
14	28	43	53	66	76	90	127	384
15	10	17	20	25	27	32	43	118
16	31	49	61	77	90	106	150	438
17	32	50	62	79	92	108	154	445
18	112	174	218	280	325	386	547	1603
19	6.9	11	14	17	19	23	32	93
20	111	174	217	280	323	383	546	1606
21	16	26	34	43	47	56	79	234
22	8.1	15	18	22	24	29	41	118
23	23	37	46	58	66	79	111	339
24	26	42	53	67	77	93	131	389
25	26	40	51	63	72	86	123	385

Catchment	"Q2" (m^3/s)	"Q5" (m^3/s)	"Q10" (m^3/s)	"Q20" (m^3/s)	"Q50" (m^3/s)	"Q100" (m^3/s)	"Q500" (m^3/s)	"QPMP" (m^3/s)
26	4.3	6.9	8.6	11	12	15	21	67
27	30	46	58	73	84	99	141	445
28	127	199	252	327	374	440	624	1845
129	5.9	9.7	12	14	15	18	25	72
29	11	17	21	25	28	34	49	155
30	126	198	253	327	377	445	627	1865
31	20	32	37	44	46	54	75	218
32	22	36	44	52	55	65	89	236
33	17	27	33	42	47	56	79	228
34	19	29	35	44	49	58	82	238
35	36	56	70	87	98	117	167	489
36	9	15	19	23	26	31	43	129
37	38	58	72	90	103	123	175	528
38	12	18	21	25	27	32	44	126
39	38	58	72	89	102	121	171	520
40	0.9	1.5	2	2.6	3	3.5	4.9	15
41	146	231	298	386	446	527	749	2267
42	130	210	275	358	417	495	700	2138
43	32	53	62	74	77	92	128	367
44	34	53	64	79	89	105	145	395
45	24	38	47	59	67	80	113	338
46	10	16	20	25	28	33	46	142
47	32	50	62	78	89	105	148	442
48	6.4	9.9	12	15	16	19	26	71
49	34	53	66	83	96	114	160	476
50	7.2	12	15	18	20	24	34	97
51	39	60	75	96	111	131	185	536
52	148	237	310	404	475	562	800	2450

Catchment	**"Q2" (m^3/s)**	**"Q5" (m^3/s)**	**"Q10" (m^3/s)**	**"Q20" (m^3/s)**	**"Q50" (m^3/s)**	**"Q100" (m^3/s)**	**"Q500" (m^3/s)**	**"QPMP" (m^3/s)**
53	144	229	301	392	464	550	782	2415
54	4.7	7.6	9.7	12	14	16	23	69
55	12	19	23	29	33	40	56	170
56	145	232	305	399	473	563	800	2473
57	130	208	273	358	430	511	731	2277
58	129	207	272	357.2	429	512	732	2298
59	111	179	235	309	374	446	639	2043
60	11	17	21	27	30	36	50	148
61	112	182	237	310	377	449	642	2061

Appendix F

Critical Duration Analysis

Appendix F Critical Duration Analysis
Subject: TUFLOW Flood Height Analysis

Node	1 hr		3 hr		24 hr		Design Duration of Maximum Flood Height	
	Hmax	difference to Max	Hmax	difference to Max	Hmax	difference to Max		Hmax
34	19.44	0.00	19.35	0.10	19.23	0.21	1 hr	19.44
36	22.74	0.00	22.71	0.03	22.56	0.18	1 hr	22.74
ACK&PL_001	35.00	0.09	35.09	0.00	35.09	0.00	3 hr	35.09
ACK&PL_002	32.42	0.11	32.53	0.00	32.52	0.01	3 hr	32.53
ACK&PL_003	31.54	0.08	31.61	0.00	31.60	0.01	3 hr	31.61
ACK&PL_003a	31.45	0.08	31.52	0.00	31.51	0.01	3 hr	31.52
ACK&PL_003b	30.70	0.10	30.80	0.00	30.78	0.02	3 hr	30.80
ACK&PL_004	30.27	0.13	30.39	0.00	30.37	0.02	3 hr	30.39
ACK&PL_005	27.39	0.13	27.52	0.00	27.52	0.00	3 hr	27.52
ACK&PL_006	27.16	0.10	27.26	0.00	27.26	0.00	3 hr	27.26
ACK&PL_007	24.45	0.35	24.79	0.00	24.80	0.00	24 hr	24.80
ACK&PL_008	23.50	0.29	23.79	0.00	23.80	0.00	24 hr	23.80
ACK&PL_009	21.89	0.31	22.18	0.02	22.20	0.00	24 hr	22.20
ACK&PL_010	20.00	0.35	20.32	0.02	20.35	0.00	24 hr	20.35
ACK&PL_011	18.23	0.39	18.59	0.03	18.62	0.00	24 hr	18.62
ACK&PL_012	17.21	0.41	17.57	0.05	17.61	0.00	24 hr	17.61
ACK&PL_013	15.62	0.44	16.02	0.04	16.06	0.00	24 hr	16.06
ACK&PL_014	14.93	0.43	15.31	0.04	15.35	0.00	24 hr	15.35
ACK&PL_015	13.99	0.33	14.29	0.03	14.32	0.00	24 hr	14.32
ACK&PL_016	13.71	0.28	13.97	0.02	14.00	0.00	24 hr	14.00
ACK&PL_017	12.01	0.50	12.43	0.07	12.50	0.00	24 hr	12.50
ACK&PL_018	11.66	0.50	12.09	0.07	12.16	0.00	24 hr	12.16
ACK&PL_019	10.78	0.61	11.31	0.08	11.39	0.00	24 hr	11.39
ACK&PL_020	10.10	0.57	10.60	0.07	10.68	0.00	24 hr	10.68
ACK&PL_021	9.60	0.49	10.04	0.06	10.10	0.00	24 hr	10.10
ACK&PL_021a	9.44	0.51	9.89	0.06	9.95	0.00	24 hr	9.95
ACK&PL_022	9.16	0.36	9.47	0.05	9.52	0.00	24 hr	9.52
ACK&PL_023	8.37	0.29	8.60	0.05	8.65	0.00	24 hr	8.65
ACK&PL_024	8.36	0.28	8.59	0.05	8.64	0.00	24 hr	8.64
ACK&PL_025	5.65	0.61	6.17	0.09	6.27	0.00	24 hr	6.27
ACK&PL_025b	5.62	0.59	6.13	0.09	6.22	0.00	24 hr	6.22
ACK&PL_026	5.23	0.56	5.71	0.08	5.79	0.00	24 hr	5.79
ACK&PL_027	5.03	0.57	5.51	0.08	5.60	0.00	24 hr	5.60
ACK&PL_028	4.67	0.56	5.15	0.08	5.23	0.00	24 hr	5.23
ACK&PL_028a	4.55	0.58	5.05	0.09	5.13	0.00	24 hr	5.13
ACK&PL_028b	4.38	0.65	4.94	0.09	5.03	0.00	24 hr	5.03
ACK&PL_029	4.36	0.66	4.92	0.09	5.01	0.00	24 hr	5.01
ACK&PL_029a	4.32	0.65	4.87	0.09	4.97	0.00	24 hr	4.97
ACK&PL_029b	4.19	0.46	4.58	0.06	4.65	0.00	24 hr	4.65
Briff_001	28.05	0.00	27.91	0.14	27.77	0.28	1 hr	28.05
Briff_002	25.74	0.00	25.66	0.07	25.65	0.09	1 hr	25.74
Briff_003	23.74	0.03	23.76	0.00	23.59	0.17	3 hr	23.76
Briff_004	22.91	0.17	23.08	0.00	22.94	0.14	3 hr	23.08
Briff_005	20.58	0.00	20.46	0.12	20.27	0.31	1 hr	20.58
Briff_006	18.92	0.12	19.03	0.00	18.85	0.18	3 hr	19.03
Briff_007	18.28	0.19	18.47	0.00	18.28	0.19	3 hr	18.47
Briff_008	15.98	0.11	16.09	0.00	15.97	0.13	3 hr	16.09
Briff_009	14.66	0.22	14.88	0.00	14.81	0.07	3 hr	14.88
Briff_010	13.78	0.13	13.90	0.00	13.86	0.04	3 hr	13.90
Briff_011	11.76	0.13	11.89	0.00	11.84	0.05	3 hr	11.89
Briff_012	10.75	0.27	11.03	0.00	10.99	0.04	3 hr	11.03
Briff_013	10.31	0.29	10.60	0.00	10.57	0.04	3 hr	10.60
Briff_014	10.08	0.30	10.38	0.00	10.35	0.03	3 hr	10.38
Briff_014a	9.62	0.32	9.95	0.00	9.92	0.03	3 hr	9.95
Briff_014b	9.55	0.29	9.85	0.00	9.82	0.03	3 hr	9.85
Briff_015	9.42	0.30	9.72	0.00	9.69	0.03	3 hr	9.72
Briff_016	8.54	0.22	8.76	0.00	8.76	0.00	3 hr	8.76
Briff_017	8.51	0.21	8.72	0.00	8.72	0.00	3 hr	8.72
Briff_018	7.44	0.30	7.75	0.00	7.74	0.00	3 hr	7.75
Briff_019	6.10	1.00	7.10	0.00	7.10	0.00	3 hr	7.10
Briff_019a	5.69	1.27	6.96	0.00	6.96	0.00	3 hr	6.96
Briff_019b	5.48	0.35	5.83	0.00	5.83	0.00	24 hr	5.83
Briff_020	5.09	0.30	5.39	0.01	5.39	0.00	24 hr	5.39
Briff_021	4.37	0.70	4.97	0.11	5.07	0.00	24 hr	5.07
Briff_021b	4.37	0.68	4.95	0.10	5.05	0.00	24 hr	5.05

Node	1 hr		3 hr		24 hr		Design Duration of Maximum Flood Height	
	Hmax	difference to Max	Hmax	difference to Max	Hmax	difference to Max		Hmax
Cath_001	21.18	0.00	21.10	0.08	21.03	0.15	1 hr	21.18
Cath_001a	20.51	0.00	20.45	0.06	20.38	0.14	1 hr	20.51
Cath_001b	19.86	0.00	19.84	0.02	19.78	0.08	1 hr	19.86
Cath_002	18.19	0.00	18.08	0.11	17.91	0.28	1 hr	18.19
Cath_003	15.96	0.00	15.83	0.13	15.71	0.25	1 hr	15.96
Cath_003a	15.20	0.00	15.11	0.08	15.01	0.18	1 hr	15.20
Cath_003b	14.48	0.00	14.35	0.13	14.13	0.35	1 hr	14.48
Cath_004	12.07	0.48	12.47	0.08	12.55	0.00	24 hr	12.55
CTrib2_001	24.45	3.65	27.66	0.44	28.10	0.00	24 hr	28.10
CTrib2_001a	21.57	0.00	21.50	0.07	21.46	0.11	1 hr	21.57
CTrib2_001b	21.50	0.00	21.46	0.04	21.41	0.09	1 hr	21.50
CTrib2_004a	12.14	0.00	12.00	0.13	11.95	0.19	1 hr	12.14
CTrib2_004b	11.82	0.00	11.71	0.12	11.71	0.12	1 hr	11.82
CTrib_001	21.80	0.00	21.79	0.01	21.77	0.03	1 hr	21.80
CTrib_002a	19.33	0.00	19.33	0.00	19.33	0.00	24 hr	19.33
CTrib_002b	18.35	0.00	18.35	0.00	18.35	0.00	24 hr	18.35
CTrib_004a	15.06	0.04	15.04	0.06	15.10	0.00	24 hr	15.10
CTrib_004b	15.02	0.04	15.01	0.04	15.05	0.00	24 hr	15.05
Kirk_cul17a	30.98	0.00	30.67	0.31	30.32	0.66	1 hr	30.98
Kirk_cul17b	29.47	0.00	29.42	0.05	29.35	0.12	1 hr	29.47
Mercury_2_1	19.00	0.00	18.98	0.01	18.89	0.11	1 hr	19.00
Mercury_2_2	18.58	0.00	18.57	0.01	18.52	0.06	1 hr	18.58
MSRail2_1	13.99	0.13	14.04	0.08	14.12	0.00	24 hr	14.12
MSRail2_2	13.98	0.09	13.98	0.08	14.07	0.00	24 hr	14.07
MSRail3_1	18.48	0.00	18.47	0.01	18.43	0.05	1 hr	18.48
MSRail3_2	16.98	0.00	16.98	0.00	16.96	0.02	1 hr	16.98
TIG_001	24.14	0.00	23.88	0.26	23.61	0.54	1 hr	24.14
TIG_002	21.06	0.00	20.84	0.22	20.68	0.39	1 hr	21.06
TIG_003	18.68	0.00	18.48	0.20	18.30	0.38	1 hr	18.68
Tig_003a	17.41	0.00	17.36	0.05	17.15	0.25	1 hr	17.41
Tig_003b	17.17	0.00	17.13	0.04	16.95	0.22	1 hr	17.17
TIG_004	16.14	0.00	16.13	0.01	15.96	0.18	1 hr	16.14
TIG_005	14.22	0.04	14.20	0.06	14.26	0.00	24 hr	14.26
TIG_006	13.99	0.08	14.00	0.08	14.07	0.00	24 hr	14.07
Tig_006a	13.96	0.08	13.96	0.08	14.04	0.00	24 hr	14.04
Tig_006b	12.80	0.07	12.81	0.07	12.87	0.00	24 hr	12.87
TIG_007	12.25	0.07	12.26	0.07	12.33	0.00	24 hr	12.33
TIG_007a	10.82	0.10	10.83	0.09	10.92	0.00	24 hr	10.92
TIG_007b	10.70	0.09	10.70	0.08	10.78	0.00	24 hr	10.78
TIG_008	10.12	0.08	10.12	0.07	10.20	0.00	24 hr	10.20
TIG_009	9.25	0.10	9.24	0.11	9.35	0.00	24 hr	9.35
TIG_010	8.43	0.40	8.74	0.08	8.82	0.00	24 hr	8.82
Ton_001	31.08	0.00	30.92	0.16	30.73	0.35	1 hr	31.08
Ton_002	30.40	0.00	30.27	0.13	30.14	0.26	1 hr	30.40
Ton_003	30.17	0.00	30.04	0.13	29.91	0.26	1 hr	30.17
Ton_004	28.57	0.00	28.53	0.04	28.47	0.10	1 hr	28.57
Ton_005	27.94	0.00	27.92	0.02	27.87	0.07	1 hr	27.94
Ton_006	26.80	0.00	26.74	0.06	26.71	0.09	1 hr	26.80
Ton_007	25.09	0.00	25.08	0.01	25.00	0.09	1 hr	25.09
Ton_007b	24.55	0.00	24.55	0.00	24.53	0.02	1 hr	24.55
Ton_008	23.78	0.14	23.87	0.06	23.92	0.00	24 hr	23.92
Ton_009	23.30	0.16	23.43	0.04	23.47	0.00	24 hr	23.47
Ton_009b	21.58	0.00	21.57	0.01	21.53	0.06	1 hr	21.58
Ton_010	17.41	0.41	17.76	0.05	17.81	0.00	24 hr	17.81

Appendix F **Critical Duration Analysis**

Subject: **TUFLOW Flow Analysis**

Node	1 hr		3 hr		24 hr		Design Duration of Maximum Flood Flow	Value
Channel	**Qmax**		**Qmax**		**Qmax**			**(m^3/s)**
ACK&PL_001	202	94%	216	100%	216	100%	3 hr	216
ACK&PL_002	205	93%	219	100%	218	99%	3 hr	219
ACK&PL_003	139	96%	145	100%	144	100%	3 hr	145
ACK&PL_003a	64	100%	64	100%	64	99%	1 hr	64
ACK&PL_003b	136	96%	141	100%	141	100%	3 hr	141
ACK&PL_003w	73	92%	79	100%	78	99%	3 hr	79
ACK&PL_004	203	92%	220	100%	217	99%	3 hr	220
ACK&PL_005	220	87%	250	100%	251	100%	24 hr	251
ACK&PL_006	214	90%	238	100%	238	100%	24 hr	238
ACK&PL_007	186	88%	210	100%	210	100%	24 hr	210
ACK&PL_008	219	85%	259	100%	259	100%	24 hr	259
ACK&PL_009	234	85%	273	99%	275	100%	24 hr	275
ACK&PL_010	232	84%	272	99%	275	100%	24 hr	275
ACK&PL_011	230	85%	268	99%	271	100%	24 hr	271
ACK&PL_012	291	82%	348	98%	356	100%	24 hr	356
ACK&PL_013	304	81%	370	98%	377	100%	24 hr	377
ACK&PL_014	303	80%	369	98%	377	100%	24 hr	377
ACK&PL_015	278	80%	340	98%	346	100%	24 hr	346
ACK&PL_016	261	84%	308	99%	312	100%	24 hr	312
ACK&PL_017	303	79%	372	97%	383	100%	24 hr	383
ACK&PL_018	312	76%	394	96%	409	100%	24 hr	409
ACK&PL_019	330	74%	427	96%	444	100%	24 hr	444
ACK&PL_020	330	75%	426	96%	442	100%	24 hr	442
ACK&PL_021	259	82%	311	98%	316	100%	24 hr	316
ACK&PL_021a	318	82%	385	99%	390	100%	24 hr	390
ACK&PL_021w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
ACK&PL_022	283	81%	342	98%	349	100%	24 hr	349
ACK&PL_023	82	71%	111	95%	117	100%	24 hr	117
ACK&PL_024	219	84%	253	97%	260	100%	24 hr	260
ACK&PL_025	335	70%	454	95%	477	100%	24 hr	477
ACK&PL_025a	338	71%	454	96%	473	100%	24 hr	473
ACK&PL_025w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
ACK&PL_026	340	78%	418	96%	434	100%	24 hr	434
ACK&PL_027	313	74%	407	97%	421	100%	24 hr	421
ACK&PL_028	246	89%	273	99%	275	100%	24 hr	275
ACK&PL_028a	173	100%	163	94%	164	95%	1 hr	173
ACK&PL_028b	211	95%	218	99%	221	100%	24 hr	221
ACK&PL_028w	11	21%	46	87%	53	100%	24 hr	53
ACK&PL_029	361	67%	511	95%	540	100%	24 hr	540
ACK&PL_029a	388	65%	566	94%	600	100%	24 hr	600
ACK&PL_029w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0

Node	1 hr		3 hr		24 hr		Design Duration of Maximum Flood Flow	Value
Channel	Qmax		Qmax		Qmax			(m^3/s)
Briff_001	63	100%	50	80%	39	61%	1 hr	63
Briff_002	75	93%	80	100%	64	80%	3 hr	80
Briff_003	63	93%	68	100%	55	82%	3 hr	68
Briff_004	60	76%	79	100%	63	79%	3 hr	79
Briff_005	90	100%	88	98%	71	79%	1 hr	90
Briff_006	72	94%	76	100%	64	83%	3 hr	76
Briff_007	86	88%	98	100%	86	88%	3 hr	98
Briff_008	86	88%	97	100%	86	88%	3 hr	97
Briff_009	82	84%	97	100%	92	95%	3 hr	97
Briff_010	59	86%	69	100%	66	95%	3 hr	69
Briff_011	75	84%	89	100%	84	94%	3 hr	89
Briff_012	89	80%	111	100%	107	96%	3 hr	111
Briff_013	89	79%	112	100%	108	96%	3 hr	112
Briff_014	92	78%	117	100%	114	97%	3 hr	117
Briff_014a	89	79%	114	100%	111	97%	3 hr	114
Briff_014b	92	78%	117	100%	114	97%	3 hr	117
Briff_014c	3	67%	4	100%	4	97%	3 hr	4
Briff_014w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
Briff_015	91	80%	114	100%	111	98%	3 hr	114
Briff_016	97	76%	128	100%	127	100%	3 hr	128
Briff_016W	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
Briff_017	96	80%	121	100%	121	100%	3 hr	121
Briff_018	97	74%	131	100%	127	97%	3 hr	131
Briff_019	97	78%	124	100%	124	100%	24 hr	124
Briff_019a	97	89%	108	100%	108	100%	3 hr	108
Briff_019b	97	76%	128	100%	128	100%	24 hr	128
Briff_019w	0	0%	22	100%	22	100%	24 hr	22
Briff_020	92	76%	121	100%	119	99%	3 hr	121
Briff_021	96	74%	129	100%	129	100%	24 hr	129
Briff_021b	56	83%	68	100%	66	97%	3 hr	68
Briff_021w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
Cath_001	71	100%	65	92%	58	83%	1 hr	71
Cath_001a	40	100%	39	98%	38	95%	1 hr	40
Cath_001b	79	100%	72	90%	65	82%	1 hr	79
Cath_001w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
Cath_002	82	100%	75	91%	66	80%	1 hr	82
Cath_003	81	100%	74	92%	67	83%	1 hr	81
Cath_003a	46	100%	44	96%	42	91%	1 hr	46
Cath_003b	76	100%	71	93%	61	80%	1 hr	76
Cath_003w	7	100%	5	76%	3	50%	1 hr	7
Cath_005	99	100%	89	90%	71	71%	1 hr	99
CTrib2_001a	5	100%	4	93%	4	93%	1 hr	5
CTrib2_001w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
CTrib2_004a	15	96%	15	95%	16	100%	24 hr	16
CTrib2_004w	0	100%	0	0%	0	0%	1 hr	0
CTrib_002a	11	99%	11	100%	11	100%	24 hr	11
CTrib_002w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
CTrib_004a	5	100%	5	100%	5	100%	24 hr	5
CTrib_004w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
Kirk_cul17a	14	100%	12	89%	10	74%	1 hr	14
Mercury_2	1	100%	1	100%	0	80%	3 hr	1
MSRail2	1	67%	2	94%	2	100%	24 hr	2
MSRail3	10	100%	10	100%	10	96%	3 hr	10

Node	1 hr		3 hr		24 hr		Design Duration of Maximum Flood Flow	Value
Channel	**Qmax**		**Qmax**		**Qmax**			**(m^3/s)**
TIG_001	65	100%	45	70%	33	51%	1 hr	65
TIG_002	25	100%	15	60%	9	36%	1 hr	25
TIG_002FP	93	100%	57	62%	36	38%	1 hr	93
TIG_003	47	100%	39	85%	34	74%	1 hr	47
Tig_003a	33	100%	29	87%	27	80%	1 hr	33
Tig_003b	72	100%	70	97%	58	80%	1 hr	72
Tig_003w	50	100%	47	94%	35	71%	1 hr	50
TIG_004	72	100%	70	98%	63	87%	1 hr	72
TIG_005	76	100%	75	98%	65	85%	1 hr	76
TIG_006	73	100%	70	96%	70	96%	1 hr	73
Tig_006a	73	99%	73	99%	74	100%	24 hr	74
Tig_006b	94	93%	95	94%	101	100%	24 hr	101
Tig_006w	1	31%	1	34%	4	100%	24 hr	4
TIG_007	88	94%	88	95%	93	100%	24 hr	93
TIG_007a	95	93%	95	94%	101	100%	24 hr	101
TIG_007b	94	93%	95	94%	101	100%	24 hr	101
TIG_007w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
TIG_008	95	94%	95	94%	101	100%	24 hr	101
TIG_009	105	95%	104	94%	110	100%	24 hr	110
TIG_010	99	69%	125	88%	142	100%	24 hr	142
Ton_001	85	100%	74	88%	63	74%	1 hr	85
Ton_002	79	100%	71	90%	62	79%	1 hr	79
Ton_003	72	100%	66	92%	60	83%	1 hr	72
Ton_004	88	100%	85	96%	79	90%	1 hr	88
Ton_005	75	100%	73	98%	70	93%	1 hr	75
Ton_006	90	100%	86	96%	80	89%	1 hr	90
Ton_007	52	100%	52	99%	49	94%	1 hr	52
Ton_007a	53	100%	52	99%	50	94%	1 hr	53
Ton_007w	0	#DIV/0!	0	#DIV/0!	0	#DIV/0!	24 hr	0
Ton_008	50	92%	52	95%	55	100%	24 hr	55
Ton_009	3	100%	3	93%	2	59%	1 hr	3
Ton_009w	63	86%	71	96%	74	100%	24 hr	74
Ton_010	83	84%	95	96%	99	100%	24 hr	99

Appendix F Critical Duration Analysis
Subject: TUFLOW Velocity Analysis

Node	1 hr		3 hr		24 hr		Design Duration of Maximum Flood Velocity	
Channel	**Vmax**		**Vmax**		**Vmax**			**(m/s)**
ACK&PL_001	1.6	98%	1.65	100%	1.7	100%	24 hr	1.7
ACK&PL_002	1.2	100%	1.12	96%	1.1	96%	1 hr	1.2
ACK&PL_003	1.3	79%	1.67	100%	1.3	76%	3 hr	1.7
ACK&PL_003a	2.8	100%	2.82	100%	2.8	99%	1 hr	2.8
ACK&PL_003b	1.9	100%	1.87	99%	1.9	99%	1 hr	1.9
ACK&PL_003w	2.0	97%	2.08	100%	2.1	100%	3 hr	2.1
ACK&PL_004	1.3	98%	1.33	100%	1.3	99%	3 hr	1.3
ACK&PL_005	1.2	100%	1.19	99%	1.2	99%	1 hr	1.2
ACK&PL_006	1.9	98%	1.93	100%	1.9	97%	3 hr	1.9
ACK&PL_007	1.3	98%	1.34	100%	1.3	99%	3 hr	1.3
ACK&PL_008	1.5	95%	1.59	100%	1.6	100%	3 hr	1.6
ACK&PL_009	1.3	96%	1.40	100%	1.4	100%	24 hr	1.4
ACK&PL_010	1.5	96%	1.58	100%	1.6	100%	3 hr	1.6
ACK&PL_011	1.5	95%	1.60	100%	1.6	100%	3 hr	1.6
ACK&PL_012	1.6	96%	1.67	100%	1.6	99%	3 hr	1.7
ACK&PL_013	1.5	92%	1.59	99%	1.6	100%	24 hr	1.6
ACK&PL_014	2.3	90%	2.56	99%	2.6	100%	24 hr	2.6
ACK&PL_015	1.4	100%	1.35	98%	1.4	99%	1 hr	1.4
ACK&PL_016	1.4	96%	1.49	100%	1.5	99%	3 hr	1.5
ACK&PL_017	1.2	91%	1.30	99%	1.3	100%	24 hr	1.3
ACK&PL_018	2.0	94%	2.07	99%	2.1	100%	24 hr	2.1
ACK&PL_019	1.8	88%	2.02	98%	2.1	100%	24 hr	2.1
ACK&PL_020	1.7	86%	1.99	98%	2.0	100%	24 hr	2.0
ACK&PL_021	1.3	96%	1.31	100%	1.3	97%	3 hr	1.3
ACK&PL_021a	3.8	96%	3.91	98%	4.0	100%	24 hr	4.0
ACK&PL_021w	0.0	0%	0.00	0%	0.0	100%	24 hr	0.0
ACK&PL_022	1.8	89%	2.01	99%	2.0	100%	24 hr	2.0
ACK&PL_023	0.8	100%	0.78	96%	0.7	89%	1 hr	0.8
ACK&PL_024	1.9	93%	2.03	99%	2.0	100%	24 hr	2.0
ACK&PL_025	1.3	85%	1.53	97%	1.6	100%	24 hr	1.6
ACK&PL_025a	1.6	85%	1.89	98%	1.9	100%	24 hr	1.9
ACK&PL_025w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
ACK&PL_026	1.3	91%	1.43	98%	1.5	100%	24 hr	1.5
ACK&PL_027	1.6	87%	1.77	98%	1.8	100%	24 hr	1.8
ACK&PL_028	1.5	100%	1.54	100%	1.5	99%	3 hr	1.5
ACK&PL_028a	1.5	100%	1.40	94%	1.4	95%	1 hr	1.5
ACK&PL_028b	0.8	100%	0.76	93%	0.8	93%	1 hr	0.8
ACK&PL_028w	0.9	61%	1.46	97%	1.5	100%	24 hr	1.5
ACK&PL_029	1.1	78%	1.41	96%	1.5	100%	24 hr	1.5
ACK&PL_029a	1.3	65%	1.93	94%	2.0	100%	24 hr	2.0
ACK&PL_029w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0

Briff_001	1.4	100%	1.33	93%	1.2	85%	1 hr	1.4
Briff_002	1.3	100%	1.27	100%	1.2	94%	1 hr	1.3
Briff_003	1.2	100%	1.10	90%	0.9	76%	1 hr	1.2
Briff_004	1.3	90%	1.40	100%	1.3	91%	3 hr	1.4
Briff_005	1.1	100%	0.98	93%	0.9	81%	1 hr	1.1
Briff_006	1.1	100%	1.08	95%	1.0	86%	1 hr	1.1
Briff_007	1.8	96%	1.90	100%	1.8	96%	3 hr	1.9
Briff_008	1.4	98%	1.41	100%	1.4	96%	3 hr	1.4
Briff_009	1.4	93%	1.49	100%	1.4	97%	3 hr	1.5
Briff_010	1.2	95%	1.31	100%	1.3	98%	3 hr	1.3
Briff_011	1.2	98%	1.25	100%	1.2	98%	3 hr	1.2
Briff_012	1.8	93%	1.97	100%	1.9	98%	3 hr	2.0
Briff_013	1.8	95%	1.88	100%	1.8	98%	3 hr	1.9
Briff_014	2.2	98%	2.25	100%	2.2	98%	3 hr	2.3
Briff_014a	2.7	84%	3.18	100%	3.1	98%	3 hr	3.2
Briff_014b	1.7	98%	1.72	100%	1.7	99%	3 hr	1.7
Briff_014c	0.8	83%	0.99	100%	1.0	98%	3 hr	1.0
Briff_014w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
Briff_015	2.0	100%	1.97	97%	2.0	97%	1 hr	2.0
Briff_016	2.0	92%	1.99	93%	2.1	100%	24 hr	2.1
Briff_016W	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
Briff_017	2.4	93%	2.58	100%	2.5	99%	3 hr	2.6
Briff_018	2.5	84%	3.05	100%	3.0	100%	3 hr	3.0
Briff_019	2.2	97%	2.28	100%	2.2	98%	3 hr	2.3
Briff_019a	3.6	91%	3.97	100%	4.0	100%	3 hr	4.0
Briff_019b	1.6	90%	1.71	100%	1.7	100%	24 hr	1.7
Briff_019w	0.0	0%	1.22	100%	1.2	100%	24 hr	1.2
Briff_020	1.1	87%	1.25	100%	1.2	93%	3 hr	1.3
Briff_021	1.2	79%	1.56	100%	1.4	88%	3 hr	1.6
Briff_021b	0.4	88%	0.49	100%	0.5	93%	3 hr	0.5
Briff_021w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
Cath_001	1.0	100%	0.95	95%	0.9	89%	1 hr	1.0
Cath_001a	2.9	100%	2.84	98%	2.7	95%	1 hr	2.9
Cath_001b	1.4	100%	1.17	85%	1.2	85%	1 hr	1.4
Cath_001w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
Cath_002	1.4	100%	1.35	97%	1.3	90%	1 hr	1.4
Cath_003	1.4	100%	1.39	97%	1.3	94%	1 hr	1.4
Cath_003a	4.4	100%	4.29	99%	4.2	97%	1 hr	4.4
Cath_003b	1.7	100%	1.75	100%	1.7	99%	3 hr	1.7
Cath_003w	1.3	100%	1.21	91%	1.1	79%	1 hr	1.3
Cath_005	1.1	100%	1.03	91%	1.1	95%	1 hr	1.1
CTrib2_001a	1.0	100%	0.93	93%	1.0	100%	1 hr	1.0
CTrib2_001w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
CTrib2_004a	2.0	96%	1.97	95%	2.1	100%	24 hr	2.1
CTrib2_004w	0.4	100%	0.00	0%	0.1	25%	1 hr	0.4
CTrib_002a	4.9	100%	4.87	100%	4.9	100%	24 hr	4.9
CTrib_002w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
CTrib_004a	1.1	98%	1.09	99%	1.1	100%	24 hr	1.1
CTrib_004w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0

Kirk_cul17a	3.9	100%	3.47	89%	3.0	77%	1 hr	3.9
Mercury_2	1.8	99%	1.86	100%	1.8	99%	3 hr	1.9
MSRail2	1.4	91%	1.55	100%	1.5	99%	3 hr	1.5
MSRail3	3.1	100%	3.12	100%	3.1	99%	1 hr	3.1
TIG_001	2.3	100%	2.04	88%	2.0	86%	1 hr	2.3
TIG_002	1.2	100%	1.00	83%	0.8	70%	1 hr	1.2
TIG_002FP	0.9	100%	0.78	84%	0.7	72%	1 hr	0.9
TIG_003	2.1	100%	1.90	91%	1.7	84%	1 hr	2.1
Tig_003a	1.8	100%	1.53	87%	1.5	86%	1 hr	1.8
Tig_003b	1.5	99%	1.49	100%	1.4	94%	3 hr	1.5
Tig_003w	1.8	100%	1.81	98%	1.6	89%	1 hr	1.8
TIG_004	1.2	100%	1.18	100%	1.1	97%	1 hr	1.2
TIG_005	0.9	100%	0.87	97%	0.8	91%	1 hr	0.9
TIG_006	0.6	92%	0.59	96%	0.6	100%	24 hr	0.6
Tig_006a	5.9	99%	5.95	99%	6.0	100%	24 hr	6.0
Tig_006b	1.1	97%	1.14	97%	1.2	100%	24 hr	1.2
Tig_006w	0.4	67%	0.46	70%	0.7	100%	24 hr	0.7
TIG_007	1.8	98%	1.76	98%	1.8	100%	24 hr	1.8
TIG_007a	7.0	100%	6.44	93%	6.9	99%	1 hr	7.0
TIG_007b	1.3	98%	1.32	98%	1.3	100%	24 hr	1.3
TIG_007w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
TIG_008	1.6	100%	1.57	100%	1.6	100%	24 hr	1.6
TIG_009	1.6	100%	1.41	90%	1.3	82%	1 hr	1.6
TIG_010	1.3	100%	1.06	82%	1.0	80%	1 hr	1.3
Ton_001	1.1	100%	0.99	88%	0.9	79%	1 hr	1.1
Ton_002	0.8	100%	0.74	91%	0.7	88%	1 hr	0.8
Ton_003	1.8	100%	1.75	98%	1.7	93%	1 hr	1.8
Ton_004	1.4	100%	1.36	98%	1.3	94%	1 hr	1.4
Ton_005	1.6	100%	1.57	100%	1.5	96%	1 hr	1.6
Ton_006	2.0	100%	1.94	97%	1.9	94%	1 hr	2.0
Ton_007	1.3	100%	1.27	97%	1.2	92%	1 hr	1.3
Ton_007a	4.2	100%	4.23	100%	4.1	98%	3 hr	4.2
Ton_007w	0.0	#DIV/0!	0.00	#DIV/0!	0.0	#DIV/0!	24 hr	0.0
Ton_008	1.0	74%	1.11	81%	1.4	100%	24 hr	1.4
Ton_009	1.0	100%	0.96	97%	0.8	84%	1 hr	1.0
Ton_009w	2.5	96%	2.54	99%	2.6	100%	24 hr	2.6
Ton_010	1.2	100%	1.20	97%	1.2	99%	1 hr	1.2

GHD Pty Ltd ABN 39 008 488 373

201 Charlotte Street Brisbane Qld 4000
GPO Box 668 Brisbane Qld 4001 Australia
T: (07) 3316 3000 F: (07) 3316 3333 E: bnemail@ghd.com.au

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
A	DCT, KA, TL	R Fryar / C Berry		R Fryar		16/05/2006
0	DCT	R Fryar		R Fryar		08/08/2006