A part of BMT in Energy and Environment

Herbert River Flood Mapping Update

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In association with

Herbert River Flood Mapping Update

Prepared for:	Hinchinbrook Shire Council
Prepared by:	BMT WBM Pty Ltd (Member of the BMT group of companies) In association with Venant Solutions Pty Ltd

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1 Introduction

1.1 Background

In 2003 WBM Oceanics Australia (now BMT WBM) completed the *Herbert River Flood Study* (WBM Oceanics Australia, 2003) for the Herbert River Improvement Trust (HRIT). A deliverable of this study was the production of flood level, depth and hazard mapping, which was subsequently incorporated into Hinchinbrook Shire Council's (HSC) planning scheme. The mapping was generated using a calibrated TUFLOW two-dimensional hydraulic model developed by BMT WBM for the Herbert River Improvement Trust. During the course of the flood study the model was calibrated to two flood events and subject to scrutiny by stakeholders and the community. It was considered to be best practice and based on the best available information at the time.

However, since the completion of the study there have been changes on the floodplain that have affected flooding and additional more accurate data has become available. In work undertaken by BMT WBM for the HRIT and the Department of Transport and Main Roads (DTMR) there have been updates to the flood model to reflect the changes on the floodplain and to use the more accurate data. These updates have resulted in changes to the flood level, depth and hazard predicted by the model. These updates are not reflected in the mapping referred to in the planning scheme.

BMT WBM has been commissioned by HSC to incorporate the numerous flood model updates from the subsequent studies into an updated flood model and produce revised flood level, depth and hazard mapping. The project was undertaken in association with Mark Jempson from Venant Solutions.

1.2 Flooding History in the Lower Herbert River Floodplain

Flooding within the Lower Herbert River floodplain can be a result of river flooding or local catchment flooding. The Herbert River catchment covers nearly 10,000 km^2 with its headwaters near Herberton on the Dividing Range. The River drains much of the western areas of the Atherton Tablelands west of the Wet Tropics. The Lower River traverses an alluvial distributary plain formed by frequent and major flooding and associated sediment deposition. The river is perched above the floodplain and so when the river breaks its banks the floodwater flows away from the river eventually reaching the ocean through a myriad of distributary channels. These breakout flows flood extensive areas of the floodplain in many locations in even small flood events (< 5 year average recurrence interval (ARI).

In living memory, flooding in the Lower River has predominantly been the result of high rainfalls in the lower part of the catchment below Gleneagle in combination with rainfall in the upper catchment. One exception is the 1967 flood which was a result of high rainfall in the upper catchment only. This flood is well remembered in the community for its size and lack of rainfall in the lower catchment. Flooding in the Lower River can be influenced by the magnitude and timing of flows from the Stone River, a tributary with its confluence upstream of Trebonne. In large floods, the Stone River will break its banks upstream of Trebonne and flow into Trebonne Creek and Cattle Creek.

The local catchments in the lower floodplain themselves have significant catchments which can produce sufficient runoff to also overtop and close the highway. The major local catchments include Seymour River, Arnot Creek, Ripple Creek, Cattle Creek and Frances Creek.

This mixture of river flooding, distributary channels and large local catchments means that the flooding patterns on the Lower Herbert River floodplain are highly complex.

1.3 Previous Studies

Since the completion of the original flood study in 2003, there has been numerous studies completed within the Lower Herbert River floodplain, resulting in incremental updates been made to the flood model. A summary of the major studies undertaken, along with updates made to the flood model is provided in the following Sections.

1.3.1 Herbert River Flood Study (WBM Oceanics Australia, 2003)

In 2001 the Herbert River Improvement Trust engaged WBM Oceanics Australia (now BMT WBM) to undertake the Herbert River Floodplain Management Plan. As part of the Herbert River Floodplain Management Plan a detailed flood study was undertaken to define the flooding characteristics of the Lower Herbert River Floodplain. This flood study is detailed in the *Herbert River Flood Study Main Report – Volume 1 of 2* (WBM Oceanics Australia, 2003). As part of this study a calibrated flood model of the Lower Herbert River Floodplain was developed.

The flood model linked a calibrated URBS hydrologic model developed by the Bureau of Meteorology (BoM) with a TUFLOW 2D/1D linked hydraulic model covering more than 900 km^2. The flood model was calibrated/validated against the February 1991 and the March 1967 historical flood events.

The TUFLOW model was calibrated to river gauges and to surveyed peak flood marks in the floodplain. For the 1991 flood 399 surveyed flood marks were available and for the 1967 verification event 797 surveyed flood marks were available.

In addition to the calibration/verification to river gauges and survey marks, the model was also extensively scrutinised by stakeholders and the community. Animations and flood extents were shown as part of consultation sessions and feedback was sought as to the model's representation of flooding patterns locally in the area the participants lived. The feedback further supported the reliability of the modelling.

1.3.2 Bruce Highway Upgrade Rutledge St to Cardwell Range Highway Planning Study Hydraulic Assessment (BMT WBM, 2011)

The *Bruce Highway Upgrade Rutledge St to Cardwell Range Highway Planning Study Hydraulic Assessment Final Report* (BMT WBM, 2011) was prepared for DTMR to undertake preliminary design and assessment of the drainage and grade level requirements for the preliminary highway alignment options to ensure flood immunity and flood impact goals were achieved.

To improve model definition within the Ingham area where flood impacts as a result of the highway alignments assessed were sensitive, a fine mesh (10 m) nested model domain was incorporated into the model that covered the Ingham area.

Also during the course of this study, the region was subjected to a significant flood in February 2009. BMT WBM estimated the flood to be generally in the 10 to 20 year ARI range, although of substantially longer duration than historically has occurred. As part of the study the model was tested using this event as a validation exercise. This same historical flood event has been used to validate the model for this current study as described in Section 2.1.

1.3.3 Herbert River Levee Modelling Study (BMT WBM, 2012[a]) and Herbert Levees' Studies – Halifax Levee Overtopping Risk Assessment (BMT WBM, 2014[b])

In 2012 the HRIT commissioned BMT WBM to undertake the *Herbert River Levee Modelling Study* (BMT WBM, 2012[a]) to; understand the likely further impacts to flood levels resulting from on-going uncontrolled levee construction; raise awareness of the impacts generated by continuing levee development and; move towards an acceptable process to control such growth.

In order to successfully complete this study new LiDAR data flown in 2009 covering the majority of the model extent was included in the model along with top of bank and levee survey along the Herbert River from approximately the Ingham Pump Station to Halifax. This data represents the basis of the topography representation in the updated model.

Following the completion of the *Herbert River Levee Modelling Study* (BMT WBM, 2012[a]), BMT WBM undertook *Herbert Levees' Studies – Halifax Levee Overtopping Risk Assessment* (BMT WBM, 2014[b]). The purpose of this study was to investigate why the Halifax levee, which was designed with a spillway crest at the 100 year ARI flood level was, after a few years, overtopping during flood events that were smaller than the 100 year ARI further upstream.

The model developed for the first Levee Modelling Study was used as the basis of this study with updates made to ensure that representation of flood levels at the Halifax levee was improved. The main updates included imbedding a fine mesh (10 m) nested model domain along the Herbert River and adjacent floodplain from approximately the Ingham Pump Station to the downstream model extent, including the entire township of Halifax. The Halifax levee spillway levels and wall levels were updated with survey to match the as-constructed levels rather than the design levels.

1.3.4 Bruce Highway Helens Hill to Rutledge Street Link Study Hydraulic Assessment (BMT WBM, 2012[b]) and Bruce Highway Cattle and Frances Creeks Upgrade Business Case Hydraulic Assessment (BMT WBM, 2014[a])

In 2012 DTMR undertook a Link Study for the highway from Helens Hill to Rutledge Street, which focussed on the Frances Creek and Cattle Creek crossings. As part of the Link Study BMT WBM completed a hydraulic assessment, *Bruce Highway Helens Hill to Rutledge Street Link Study Hydraulic Assessment Final Report* (BMT WBM, 2012b).

The existing TUFLOW model (last updated in BMT WBM, 2012a) only extended as far south as Pennas Road. In order to undertake the hydraulic assessment of the proposed highway upgrades, which extended further south than Pennas Road, the model extent was extended approximately another 4 km south. Other updates were made to model in this area including; imbedding a 10 m fine mesh grid, improving definition in the URBS hydrology model, improved definition of hydraulic structure and the addition of survey information of Frances and Cattle Creeks, the highway and Pennas and Pappins Roads.

As part of the subsequent study; the *Bruce Highway Cattle and Frances Creeks Upgrade Business Case Hydraulic Assessment* (BMT WBM, 2014a) the model was further upgraded and additional validation was undertaken, including validating against the 2009 flood and seeking feedback on model outputs from local residents.

1.4 Study Purpose and Scope

The purpose of this study is to incorporate the numerous flood model updates from the subsequent studies into an updated flood model and produce revised flood level, depth and hazard mapping to be used by HSC to revise the planning scheme.

The scope of works for the mapping update includes:

- Update the hydraulic (TUFLOW) model (refer to Section 2), to:
  - Update the base topography with new LiDAR data captured in 2009;
  - Nest a finer resolution hydraulic computational grid (10 m compared with 40 m used over the rest of the model), that covers the townships of Ingham and Halifax, the Herbert River and immediate floodplain downstream of Ingham and the Bruce Hwy south of Ingham;
  - Extend the hydraulic model to the south to better represent Frances Creek;
  - Include ground survey, mainly of river bank levees from Ingham to Halifax and road centrelines in selected areas;
  - Run the model with an up to date version of the TUFLOW modelling software.
- Update the flood frequency analysis (FFA) (refer to Section 3), analysis using the additional 13 years of catchment flow data now available;
- Update the hydrology to include an allowance for climate change (refer to Section 4);
- Update the ocan boundary to include an allowance for 0.8 m of sea level rise;
- Prepare revised flood mapping using the updated model (refer to Section 5);
- Assessment of; the impact of non-assessable filling in low hazard areas and “permissible fill” mapping for medium and high hazard areas (Section 6) to assist in the updating of controls within the planning scheme;
- Prepare this summary report documenting model updates and supply to HSC the updated model and mapping products.

2 Model Updates

To ensure that the flood model was fit for purpose for the many flooding assessments undertaken by BMT WBM since the completion of *Herbert River Flood Study* (WBM Oceanics Australia, 2003), numerous incremental updates to the flood model have been made. This section summarises the major updates made to the TUFLOW hydraulic model, and consolidated from previous assessments described in Section 1.3, for this flood mapping update.

The major updates made to the model, and consolidated for this study include:

- Nest a finer resolution hydraulic 10m computational grid (Figure 2-1), that covers the townships of Ingham and Halifax, the Herbert River and immediate floodplain downstream of Ingham and the Bruce Hwy south of Ingham;
- Extend the hydraulic model to the south to better represent Frances Creek (Figure 2-1);
- Update the base topography with new LiDAR data captured in 2009 (Figure 2-2);
- Include additional ground survey, mainly of river bank levees from approximately Ingham Pump Station to Halifax and road centrelines in selected areas;
- Adjustments to the distribution of local catchment runoff on the model to reflect the finer resolution 10 m grid;
- Run the model with an up to date version of the TUFLOW modelling software to take advantages of advancements in the software package.

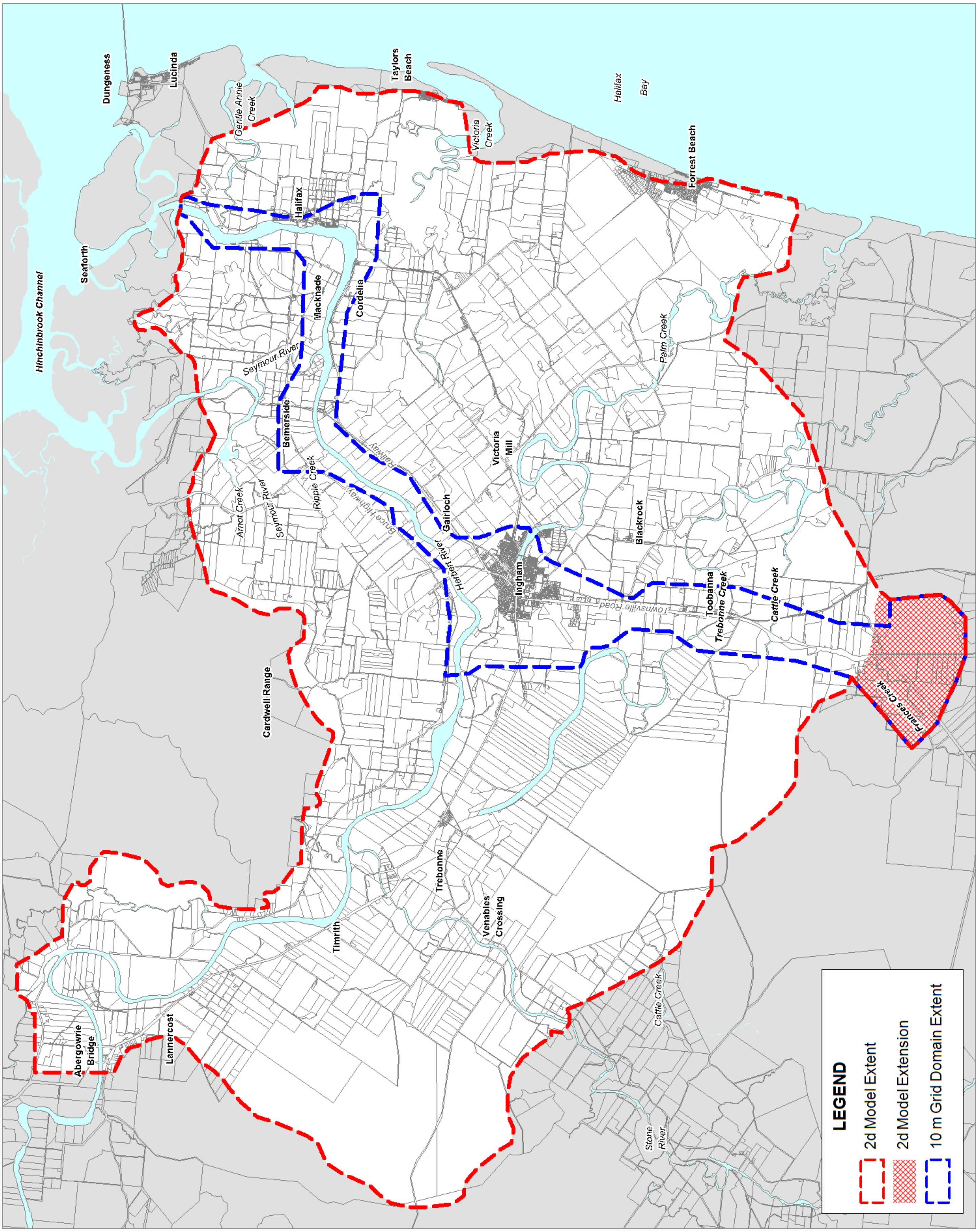
The original model was based on a 40 m calculation grid, which was best practice when the model was originally developed, but is relatively coarse when looking at localised flood levels around in areas of interest, particularly major townships. Software updates and improved computer hardware now allows for 'nesting' of finer grids within a coarser grid so as to improve the resolution of the model in areas of interest while maintaining appropriate run times.

For this model update, a higher definition 10 m grid domain, covering Ingham, the Herbert River and adjacent floodplain including Halifax downstream approximately the Ingham Pump Station, and the Bruce Highway south of Ingham was nested into the broader 40 m grid as shown in Figure 2 1. This 10 m domain extent represents a consolidation of the 10 m nested domains established for BMT WBM (2009), BMT WBM (2012[b]) and BMT WBM (2014[b]). It should be noted that although better representation of floodplain characteristics is provided by the 10 m grid domain, the TUFLOW model is still "entire catchment" model and is limited in its representation of topography and hydraulic structures in urban areas (townships).

The TUFLOW original model extended as far south as Pennas Road, and the Frances Creek catchment inflow was applied at this location. The model has been extended approximately a further 4 km to the south of Pennas Road to provide better representation of Frances Creek and the adjacent floodplain in the Helens Hill area. The extension of the hydraulic model is shown in Figure 2-1. Minor modifications were made to the URBS hydrological model to reflect the altered location of the hydraulic model boundaries, and to improve the resolution of the model's representation of the Frances Creek catchment.

In mid-2009, the Department of Environment and Resource Management captured LiDAR data covering the majority of the model extent as shown in Figure 2-2. In areas where there was no new LiDAR available such as the upper reaches of Lannercost and Cattle Creeks the Digital Elevation Model (DEM) developed for the original study was used. The new LiDAR data does not represent river bed levels where water was present during the time of capture, so in these areas, the lower reaches of the Herbert and Seymour Rivers, the bed level data from the original model was retained as shown in Figure 2-2.

To support the new LiDAR data in defining the model topography, a large amount of ground survey has been taken, mainly by HSC and DTMR to fulfil the purposes of individual studies. All of this survey data was included in the updated model. Most notably, top of bank/levee survey along Herbert River from Ingham to Halifax has been included. As described in Section 2.1, additional top of bank/levee survey was captured along the south bank of the Herbert River from Palm Creek to the John Roe Bridge for the current model update.



Title:

TUFLOW Hydraulic Model Extent

Figure: **2-1**

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N

0 3.75 7.5km

Approx. Scale

BMT WBM

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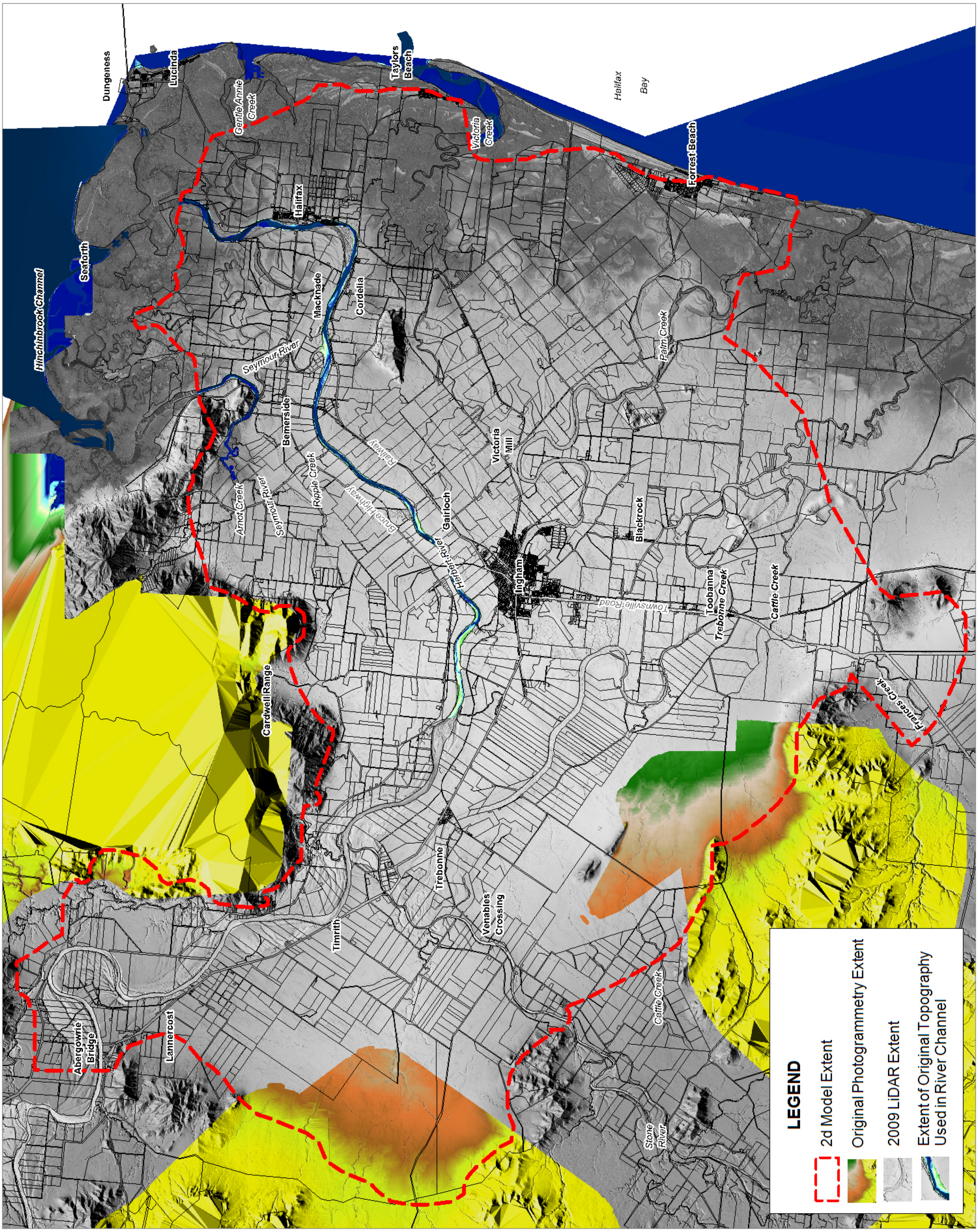

Title:
LiDAR Dataset Extents

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N

0 3.75 7.5km

Approx. Scale

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2.1 Model Validation

When the TUFLOW model was originally developed it was calibrated to the February 1991 flood and validated against the March 1967 flood (WBM, 2003). During the course of the Ingham to Range Planning Study being undertaken by DTMR, the region was subjected to a significant flood in February 2009. BMT WBM estimated the flood to be generally in the 10 to 20 year ARI range, although of substantially longer duration than historically has occurred. As part of the *Bruce Highway Upgrade Rutledge St to Cardwell Range Preliminary Hydraulic Assessment* (BMT WBM, 2011) the model was tested using this event as a validation exercise. The validation was revisited as part of the *Herbert River Levee Modelling Study* (BMT WBM, 2012[a]) because the model was substantially updated using a new DEM (digital elevation model). To incorporate updated model setups and new flood event information, the flood model was further validated to the February 2009 flood event as part of the; *Bruce Highway Helens Hill to Rutledge Street Link Study Hydraulic Assessment Final Report* (BMT WBM, 2012b)*; Bruce Highway Cattle and Frances Creeks Upgrade Business Case Hydraulic Assessment* (BMT WBM, 2014a)*; and the Herbert Levees' Studies – Halifax Levee Overtopping Risk Assessment* (BMT WBM, 2014b). . While these validations further supported the reliability of model it is recognised that the extent of model updates made for the current study warrants re-validation of the flood model.

The flood event and validation process for the current study, described below, focuses on confirming the performance of the model updates, focusing on the townships of Ingham and Halifax.

2.1.1 Description of Event

The Herbert River catchment received significant rainfall in January and February 2009 leading to major flooding (Bureau of Meteorology (BOM), 2009). During this period there were two separate flood events, the first from 12th to 13th January 2009 (January 2009 event) and the second between 29th January and 8th February 2009 (February 2009 event). Only the second of these events is considered here as it was the more severe of the two events.

According to the BOM (2009) the February 2009 event was the result of 3 separate periods of heavy downpours occurring on:

- 29th and 30th January 2009;
- 1st and 2nd February 2009; and
- 6th and 7th February 2009.

The BOM states that the first period of rainfall resulted in moderate flooding with the second and third periods of rainfall resulting in widespread and prolonged major flooding.

The BOM (2009) reports that the rainfall on the 29th – 30th January 2009 was the result of a monsoon trough that developed to the north east of Cairns on the 29th January 2009. This trough moved southward during the 30th January 2009 developing into Tropical Cyclone Ellie early on the 1st February 2009. The rainfall on the 1st and 2nd February 2009 was due to the approach of Tropical Cyclone Ellie which weakened to a rain depression during the latter part of the 1st February 2009. The rainfall during the 6th and 7th of February 2009 was caused by a weak low developing to the east of Cairns which drifted northward producing rain on the Herbert catchment over this period.

Rainfall totals and Intensity - Frequency - Duration (IFD) analysis have been completed by BOM (2009). This work reports that significant rainfall totals occurred across the catchment, with higher rainfall totals on the lower catchment and lower rainfall totals on the upper catchment. The total rainfall from the 29th January to 10th February 2009 on the lower catchment was in excess of 1000mm in several locations. The maximum reported rainfall depth was at Ingham Pump Station with a depth of 1265mm.

IFD analysis was completed by BOM (2009) who report the Annual Recurrence Intervals (ARI) for the recorded rainfall depths against a variety of durations at a number of stations during the period 29th January to 10th February 2009. The durations ranged from the 5 minute event to the 72 hour event. Longer durations were not considered as the methodology outlined in Australian Rainfall and Runoff (AAR) (Institution of Engineers Australia, 1987) does not extend beyond the 72 hour event. The following stations were reported:

- Gleneagle Alert;
- Abergowrie Alert;
- Gairloch Alert; and
- Halifax Alert

In general the ARI for shorter duration storms of less than 1 hour were in the 1-2 year range or less. Durations greater than 1 hour and less than 6 hours had ARI of less than 5 years with the exception of the Gleneagle Alert which had ARI of up to 50 years. The 12 hour ARI had the highest recurrence interval at all stations with Gleneagle having an ARI greater than 100 year, Abergowrie and Gairloch Alerts having an ARI in the range 10-20 years and Halifax between 5 and 10 years. The ARI for the longer durations were less than 10 years at all stations except Gleneagle.

A rainfall event of a particular magnitude does not necessarily translate to the same magnitude flood event. Other factors that will influence the magnitude of the flood event include the antecedent catchment conditions and the temporal and spatial variability of rainfall across the catchment. Therefore BMT WBM investigated the magnitude of the flood event across the Lower Herbert River region by comparing the recorded peak flood levels with the design flood levels in WBM (2003). The recorded peak flood levels were surveyed flood marks picked up by DTMR and Hinchinbrook Shire Council and levels from river gauging stations. This analysis found that across much of the floodplain the ARI of the February 2009 event was in the range of 10 to 20 years ARI. At the Bruce Highway crossing of Cattle Creek the flood is also estimated to be in the range of 10 to 20 years ARI.

As noted earlier the second and third rainfall periods resulted in prolonged periods of flooding. The flooding was unusually long for the Herbert River as indicated in Figure 2-3. In this figure the flood height-time series at Abergowrie Bridge are plotted for the 1967, 1991, 1997 and 2009 flood events: these are the flows from the upper catchment. As documented earlier there was also significant rainfall in the lower catchment over a similar period. As reported in WBM (2003) the

1991 height-time history, and hence duration, was considered to be typical for the catchment and was used as a basis for the development of the design floods.

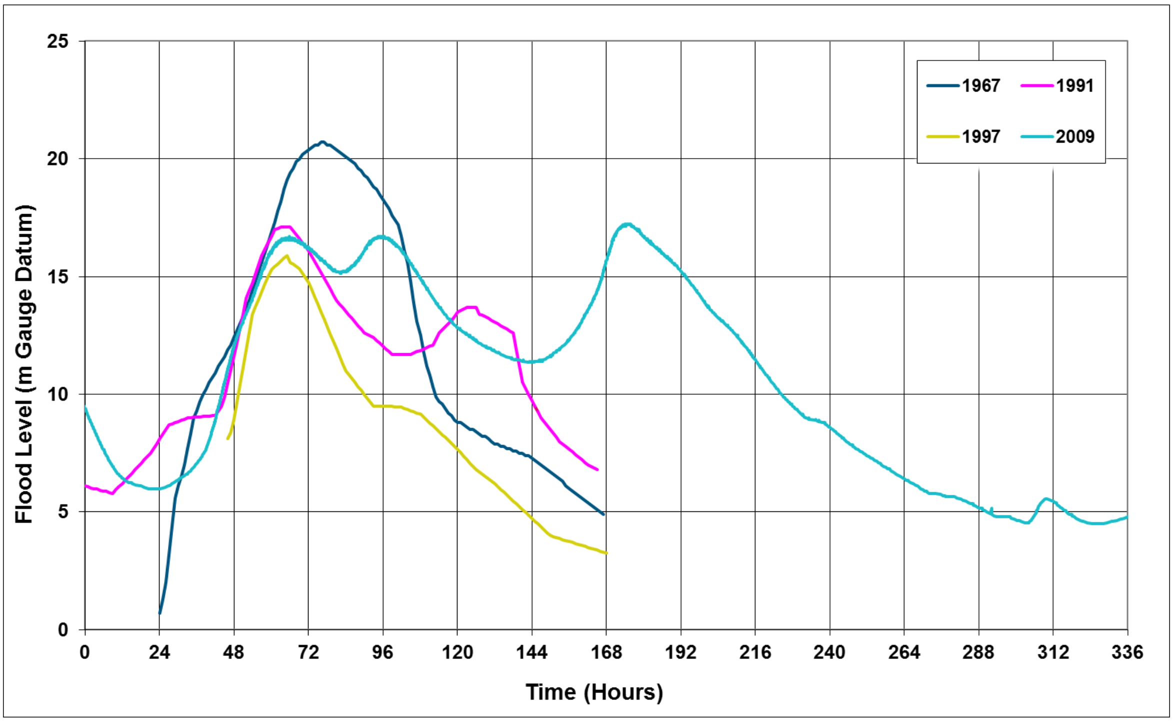


Figure 2-3: Historical flood height-time plots at Abergowrie Bridge

2.1.2 Boundary conditions

Inflow and ocean boundaries were required for the model validation. To develop these boundaries the following recorded data was obtained:

- ocean tide levels from the gauge at Lucinda;
- a river height-time series was obtained for the gauge at Abergowrie bridge;
- rainfall data as described in Section 2.1.1.

The upstream boundary of the model used for calibration is at the Abergowrie Bridge: the model used for design events assessments, with the exception of the PMF, extends up to Abergowrie. The model boundary was located at Abergowrie Bridge because there is a river gauge and the river is contained at that location meaning that a more reliable estimate of flow rate can be made. In WBM (2003) the model was successfully calibrated by specifying this boundary as a height-time series and allowing TUFLOW to generate the flow rate. The same approach was adopted for this validation.

Historically, flooding in the lower Herbert floodplain has been dominated by runoff from the catchment upstream of the lower Herbert floodplain. This was still the case in this event, but the rainfall over the floodplain was significant in that it impacted on peak flood levels. Inflow boundaries for TUFLOW model for the lower catchment (below Abergowrie Bridge) were derived using the URBS hydrological model, which required input of the event rainfall data. As described in Section 2.1.1, rainfall was available from pluviograph and daily total stations. Only pluviograph data was applied to the model to improve temporal resolution. To take advantage of the semi-distributed nature of the URBS model a Trade Area Analysis was used to distribute rainfall to the URBS sub-catchments. In a limited number of cases pluviograph stations were noted to have failed during the event but are reporting reliable daily totals. In these cases the daily records were disaggregated based on weighted temporal patterns determined through a Trade Area Analysis based on the surrounding pluviographs. In the context of generating catchment runoff across the lower Herbert, the rainfall data was relatively scarce and so there is some uncertainty as to the reliability of the runoff generated by URBS.

2.1.3 Validation Model Updates

The purpose of re-validating the model for the current study was to remove some uncertainty surrounding the model updates and improve on previous validation results. This was achieved by updating the model and consolidating the rainfall data and flood marks compiled during previous studies as follows:

- Revised rainfall in the mountain ranges north of Ingham.
- Lowering the Manning's roughness coefficient used in the Herbert River channel upstream of the Gairloch Washaway.
- Including updated top of bank/levee survey captured by Council surveyors on the southern bank of the Herbert River between Palm Creek and the John Row Bridge.

Historically in some of the past studies, flood levels surveyed in the northern part of the Lower Herbert floodplain in the vicinity of the Seymour River and Arnott Creek have been underestimated by the model. The rainfall totals applied to the model in the mountain ranges north of Ingham, specifically Mount Hawkins, Gardiner Mountain and South Gardiner Mountain at the upstream reaches of Ripple Creek, the Seymour River and Arnot Creek, were smaller than those recorded on the adjacent floodplain. As it is likely that rainfall over the ranges, where there are no rainfall stations, would be equal to or greater than those recorded on the floodplain, the rainfall totals were revised to match those in recorded over the floodplain.

It was also observed in some of the past studies that flood levels modelled along the Herbert River upstream of Ingham were generally higher than the recorded flood marks, while flood levels in the lower reaches of the floodplain were too low. In order to lower flood levels upstream and transfer more water to the lower floodplain via the main channel, the Mannings roughness coefficient in the river channel upstream of the Gairloch Washaway was lowered from 0.35 to 0.3.

During the current validation, flood levels were been underestimated in parts of the township of Ingham to the north/north-east of Palm Creek. This suggested that there was not enough flow breaking out of the Herbert River between Palm Creek and the John Roe Bridge. Council undertook additional top of bank/levee survey along this stretch of river to determine the ground elevations controlling flow out of the river at this location. An analysis of this survey, suggested that the underlying model topography (DEM) was too high in this area. Accordingly the DEM levels were lowered to ensure the new survey levels acted as the hydraulic control. This resulted in a significant improvement to the calibration in this part of Ingham

2.1.4 Validation Results

For the current study the model was validated by comparing model results with river gauge time series at the Gairloch Washaway, and by comparing model flood levels with survey flood marks.

River gauges within the model extent include Abergowrie Bridge, Trebonne, Ingham Pump Station, Gairloch Washaway and Halifax. The Abergowrie Bridge was used at the model boundary so cannot be used. The Trebonne and Ingham gauges were both damaged during the flood and the Halifax gauge gave faulty readings. Therefore the only available gauge is Gairloch Washaway. A comparison between the Gairloch gauge readings and the model levels is presented in Figure 2-4. Across most of the 10 day flood, excellent agreement is achieved in timing and flood level.

230 surveyed flood marks were provided by Hinchinbrook Shire Council and DTMR. These were a mixture of debris levels picked up by the surveyors, typically along roads, and flood marks provided by landowners. There is always uncertainty associated with such levels as the debris or mud line may not represent the peak flood level. Therefore a tolerance is put around each calibration point. In the Herbert River Flood Study a tolerance of ± 0.2m was adopted and so the same tolerance was adopted for this validation exercise.

The 230 surveyed peak flood level are compared to the model levels in Figure 2-5. Rather than display numbers, the difference between the modelled peak flood level and the surveyed level is colour coded according to the legend in the drawing. A positive number indicates that the modelled level is higher than the recorded level. The yellow colour indicates where the model is within the ±0.2 m tolerance.

Overall the flood levels predicted by the model are still higher upstream of Ingham and slightly lower within the lower floodplain but generally still within tolerance. Overall the validation against recorded peak flood levels is an improvement in comparison to the 1991 calibration (WBM, 2003) as indicated by the statistical analysis of the 1991 calibration and the 2009 validation in Table 2-1.

In the township of Ingham flood levels are generally within tolerance, however as shown in in Figure 2-5 levels in the centre of Ingham adjacent to Palm Creek they are over estimated at some points, and some of the levels east of Townsville Road are underestimated. Reasons for these differences were explored but the calibration at these points could not be improved without further detailed investigations. With regards the points around Palm Creek there are a number of drainage structures and buildings obstructing flow the flow path in this area and so it may be that a more detailed representation of these would improve calibration in this localised area. With regards the points to the east of Townsville Road they are located in area where there is a very flat hydraulic gradient which means the water is pooling and moving slowly through the area. The area drains to the south-east into Palm Creek. Possible reasons for the levels being underestimated include an underestimation of the local rainfall and/or the drainage path into Palm Creek operating too efficiently.

As shown in Figure 2-5, the flood levels modelled in Halifax are generally in good agreeance with those recorded.

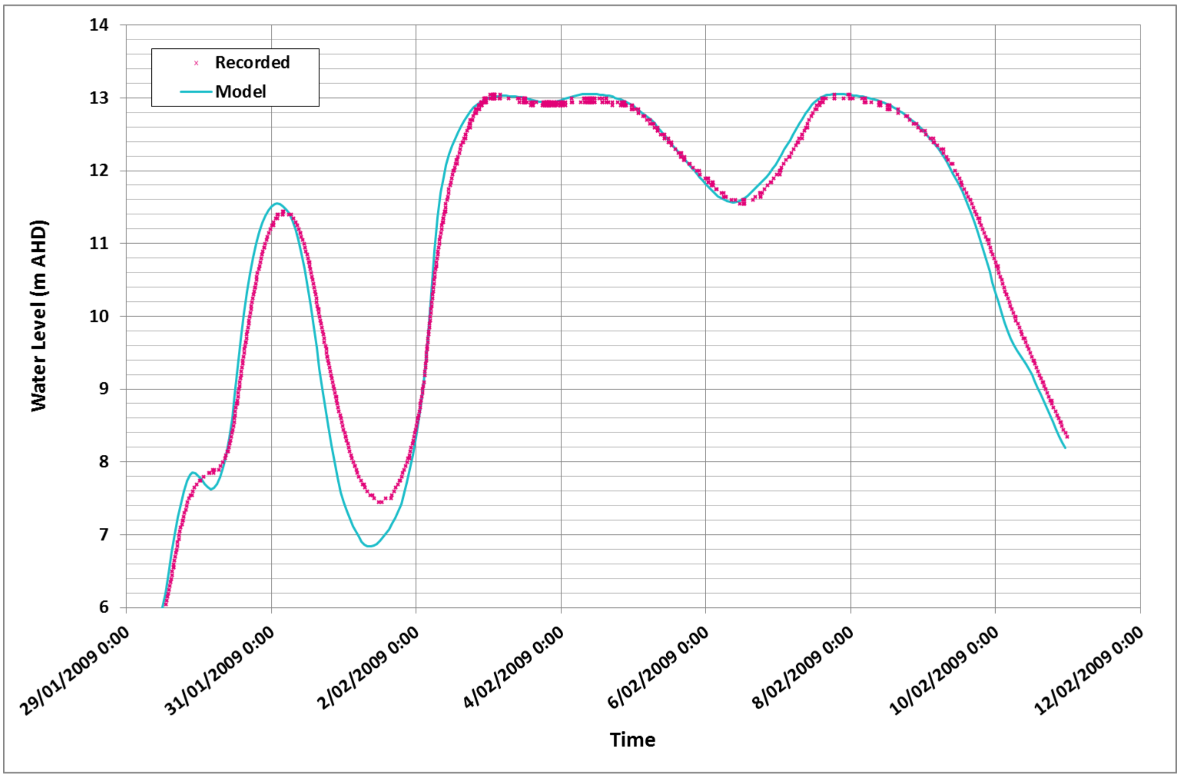


Figure 2-4 Recorded and model levels at Gairloch Gauge - February 2009

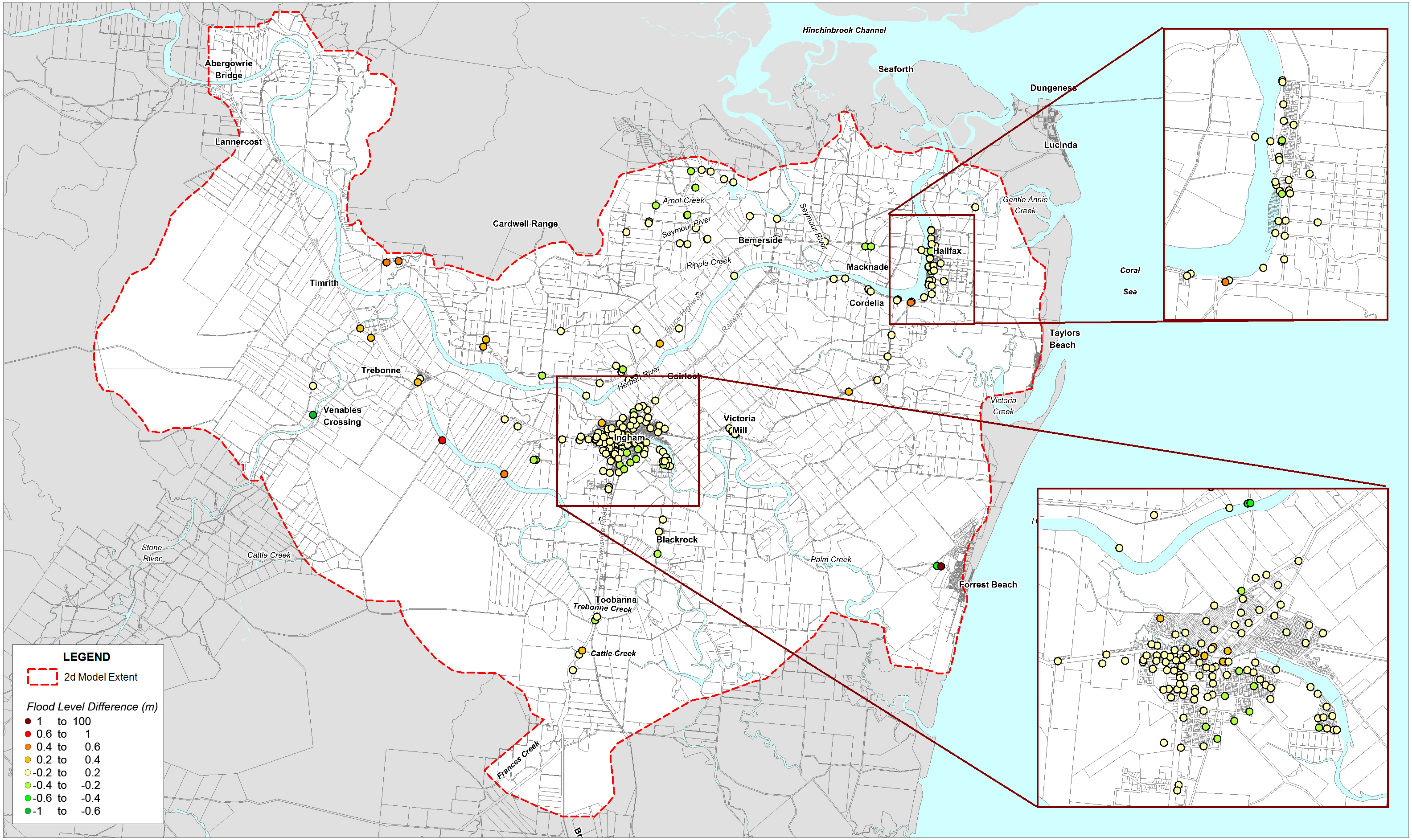
Table 2-1 Statistical Analysis of 1991 Calibration & 2009 Validation

Range	Percentage of Calibration Points within Range (%)	
	1991	2009[1]
< 1.0 m	0	0
-1.0 m to –0.6 m	1	0
-0.6 m to –0.4 m	3	1
-0.4 m to –0.2 m	13	11
-0.2 m to +0.2 m	72	78
0.2 m to 0.4 m	8	6
0.4 m to 0.6 m	2	3
0.6 m to 1.0 m	1	0
> 1.0 m	1	0

1. There are four calibration points located outside of the modelled flood extent. These points are not included in the percentage analysis and fall beyond the modelled extent as a result of limitations in representing localised topography and rainfall.

2.1.5 Summary

In summary it was considered that the validation exercise supported the reliability of the model in reproducing flood extent, level and flooding patterns and represented an improvement over the existing model. It is considered that the model is fit-for-purpose for updating Council's flood mapping.



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Model Validation Results - February 2009 Flood

Figure: 2-5

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3 Flood Frequency Analysis

3.1 Introduction

A flood frequency analysis (FFA) was undertaken as part Herbert River Flood Study (WBM 2003). Since the completion of WBM (2003), an additional 13 years of gauge data has been captured. The additional data represents an increase in the instrumental record length of 15% at the Ingham Pump Station River Gauge and 38% at the Abergowrie River Gauge. Importantly, the increase in the instrumental record captures notable flood events that occurred in 2009 and 2014, which will impact upon the flood frequency analysis.

The Probable Maximum Flood (PMF) is determined independently from the FFA and is not influenced by the additional gauge data. Therefore the PMF flows determined in WBM (2003) were adopted.

3.2 Approach

Flood frequency analysis (FFA) has been undertaken using the methods outlined in the draft version of Australian Rainfall and Runoff (ARR) Book IV Estimation of design peak discharges (Kuczera and Franks, 2006). The FFA of the Herbert River at Ingham Pump Station Gauge (116001) and the Herbert River at Abergowrie Gauge (116006) has been undertaken using the Flike FFA software (Kuczera, 1999). This package provides a Bayesian framework for comprehensive at-site flood frequency estimation that allows the inclusion of ungauged historical events.

The fitting of flood frequency distributions using Flike was undertaken with the following steps:

- Prepare data:
  - Collect gauged streamflow data
  - Undertake standard data checks on the stream flow data including checking error codes, cataloguing data gaps and undertaking visual inspections;
  - Extract the annual maxima series and check peaks for independence; and
  - Collect historic data
- At-site FFA: Fit an extreme value distribution using Flike, this involves:
  - Incorporating historic data;
  - Censoring low flows: low flows were systematically removed using a multiple Grubbs Beck test from the data to ensure that the distributions are 'aware' of the full length of record as opposed to block censoring the data; and
  - Prior Parameters Information: Distribution parameter estimates from the RFFE model were applied to Flike as prior information.

3.2.1 Background on Approach

The ARR technical committee recommends that Bayesian methods are used in preference to the methods outlined in previous versions of ARR. Specifically published on the ARR website, the following Practice Advice is given:

- Log Pearson 3 (LP3) is no longer specifically recommended - the user should select the distribution which best fits the data. In many locations research has found the best fit is either the Generalised Extreme Value (GEV) or LP3, but other distributions are not precluded.
- The log space moment fitting technique recommended in ARR87 is no longer recommended as other techniques have been shown to be more efficient. The preferred technique uses Bayesian methods as described in the draft flood frequency chapter mentioned above.

The approach adopted for this Investigation is consistent with the advice published on the ARR website and repeated above.

3.3 Data

3.3.1 Collect the data

Streamflow data was collated for two locations within the catchment, namely Herbert River at Abergowrie and Herbert River at Ingham Pump Station. This data was sourced from the Water Monitoring Portal maintained by the Queensland Government's Department of Natural Resources and Mines.

There are 47 complete years (1967 to 2014) and 97 complete years (1917 to 2014) of instantaneous flow record length for the Abergowire and Ingham Pump Stations gauges respectively. No average daily flows beyond this period have been recorded which could have been used to extend the stream gauging record.

3.3.2 Rating Curve Review

WBM (2003) derived a rating curve for the Ingham Pump Station gauge that was based on the TUFLOW hydraulic model results. This derived rating curve was considered to be more reliable than the DRNM and BoM rating curves for the gauge. In light of the more recent survey data being used in the hydraulic model, the rating curve adopted in WBM (2003) was reviewed against the hydraulic model results from the updated hydraulic model.

The rating curve derived from the updated hydraulic model was compared to that generated as part of WBM (2003). The revised rating curve did not appear to accurately represent the in-bank flows as accurately as WBM (2003). Upon review of the terrain data from WBM (2003) and the current study, it was determined that the photogrammetry was providing a better representation of the in-bank geometry. Therefore the adopted rating curve was based on WBM (2003) for the in-bank flows (below 12 m AHD) and the revised rating curve was adopted for the out of bank flows. The revised rating curve is shown in Figure 3-1 and Table 3-1.

The rating curve adopted in WBM (2003) for Abergowrie was not amended as part of the current study.

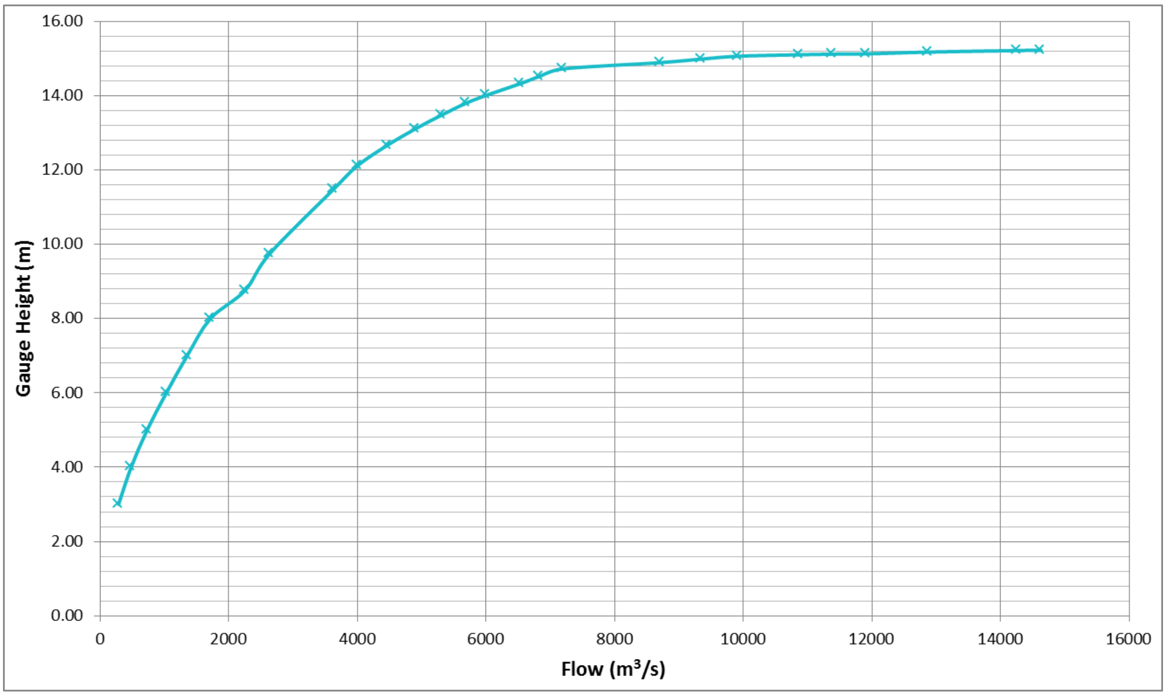


Figure 3-1 Adopted Rating at Ingham Pump Station

Table 3-1 Adopted Rating at Ingham Pump Station Values

Gauge Height (m)	Gauge Height (m AHD)	Flow (m^3/s)	Gauge Height (m)	Gauge Height (m AHD)	Flow (m^3/s)
3.00	4.25	287	14.52	15.76	6825
4.00	5.25	483	14.72	15.97	7197
5.00	6.25	731	14.89	16.14	8715
6.00	7.25	1030	14.99	16.23	9351
7.00	8.25	1354	15.06	16.31	9913
8.00	9.25	1715	15.10	16.35	10866
8.75	10.00	2250	15.12	16.37	11386
9.75	11.00	2640	15.13	16.37	11904
11.48	12.73	3630	15.17	16.42	12879
12.11	13.36	3999	15.22	16.46	14254
12.67	13.91	4467	15.22	16.47	14629
13.11	14.35	4900			
13.48	14.73	5309			
13.80	15.05	5689			
14.01	15.26	6003			
14.32	15.57	6528			

3.3.3 Annual Maximum Data

The annual maximum data for the Abergowrie and Ingham Pump Station gauging stations was extracted from the streamflow data collected and is listed in Table 3-2 and Table 3-3 respectively. A plot of this data is presented in Figure 3-2 (Abergowrie) and Figure 3-3 (Ingham Pump Station).

Table 3-2 Annual Maximum Series: Abergowrie

Year	Peak Flow (m^3/s)	Year	Peak Flow (m3/s)
1967	13500	1991	9105
1968	2100	1992	985
1969	990	1993	462
1970	823	1994	2301
1971	1734	1995	384
1972	3657	1996	1834
1973	2140	1997	6529
1974	1939	1998	9037
1975	987	1999	5576
1976	953	2000	5135
1977	12043	2001	1750
1978	1164	2002	858
1979	2592	2003	373
1980	3034	2004	1906
1981	2182	2005	999
1982	1489	2006	2702
1983	1424	2007	4090
1984	1198	2008	2153
1985	563	2009	9799
1986	7417	2010	1086
1987	286	2011	6037
1988	1025	2012	1781
1989	2434	2013	4683
1990	5020	2014	7357

Table 3-3 Annual Maximum Series: Ingham Pump Station

Year	Peak Flow (m^3/s)	Year	Peak Flow (m3/s)	Year	Peak Flow (m3/s)
1916	2174	1950	3239	1984	2406
1917	3239	1951	2309	1985	1404
1918	5490	1952	3107	1986	6917
1919	0	1953	4827	1987	0
1920	2848	1954	0	1988	1458
1921	1862	1955	9124	1989	3104
1922	4906	1956	6841	1990	6218
1923	0	1957	2573	1991	8167
1924	3239	1958	3107	1992	1493
1925	0	1959	3016	1993	1203
1926	0	1960	2510	1994	4093
1927	7999	1961	0	1995	1155
1928	4167	1962	2794	1996	2649
1929	4285	1963	2573	1997	5082
1930	4474	1964	1685	1998	7665
1931	1200	1965	3963	1999	5554
1932	4176	1966	0	2000	6282
1933	2080	1967	12518	2001	2897
1934	6843	1968	3641	2002	1909
1935	2214	1969	2143	2003	0
1936	4113	1970	2327	2004	2995
1937	2502	1971	3549	2005	1417
1938	2798	1972	6218	2006	3408
1939	3203	1973	3385	2007	4764
1940	0	1974	4529	2008	2358
1941	3584	1975	1604	2009	7631
1942	2657	1976	2573	2010	1456
1943	1825	1950	3239	2011	6106
1944	1532	1978	2676	2012	3444
1945	5835	1979	3082	2013	5475
1946	7999	1980	3947	2014	7014
1947	3384	1981	3492		
1948	6027	1982	2165		
1949	3184	1983	3195		

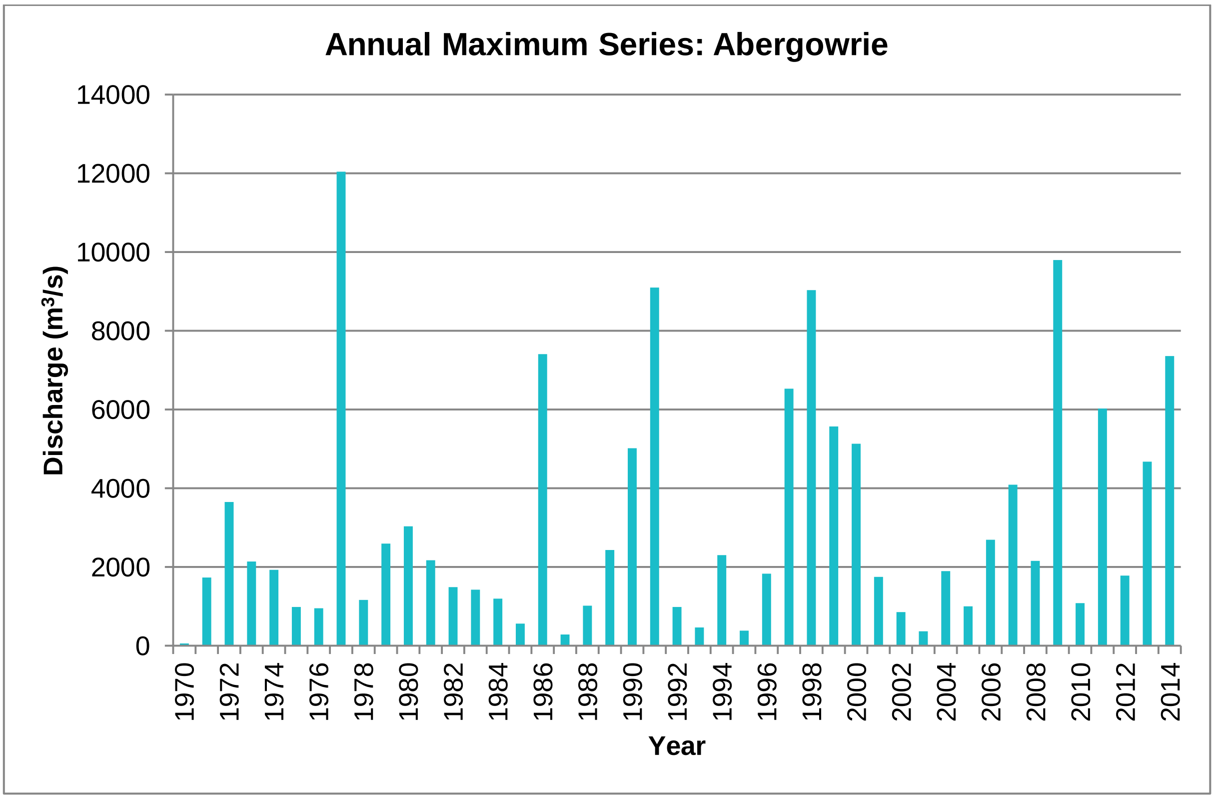


Figure 3-2 Annual Maximum Series: Abergowrie

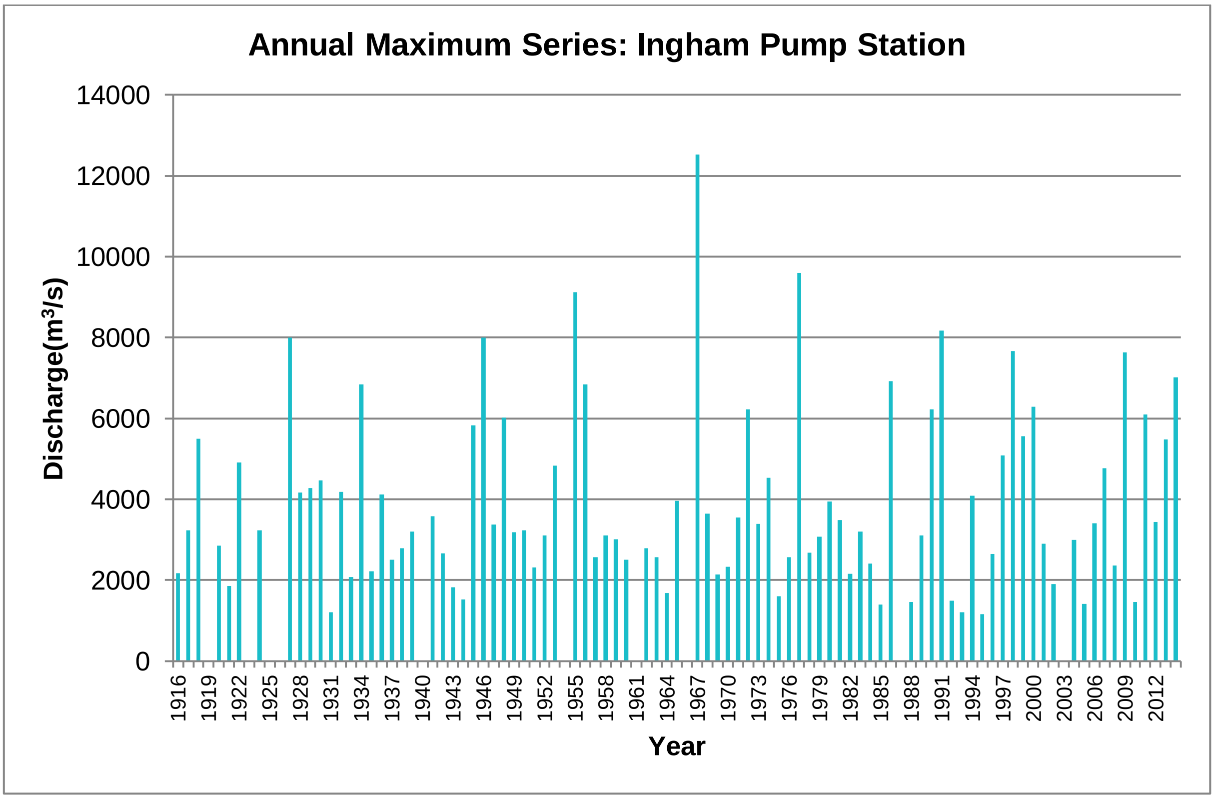


Figure 3-3 Annual Maximum Series: Ingham Pump Station

3.3.4 Collect Historic Data

WBM (2003) noted that although the 1927 flood recorded a gauge height 0.34 metres lower of the Ingham Pump Station gauge when compared to the 1967 flood; there is sufficient anecdotal evidence that the 1927 flood was larger on the floodplain as shown on historical flood markers in Figure 3-4 and Figure 3-5. This evidence is also supported by local residents who confirmed this on numerous occasions to the project team. The Bayesian framework of Flike allows the incorporation of historic events into the FFA. Figure 3-4

The flood record was adjusted to allow the incorporation of the 1927 event as the largest in the instrumental flood record.

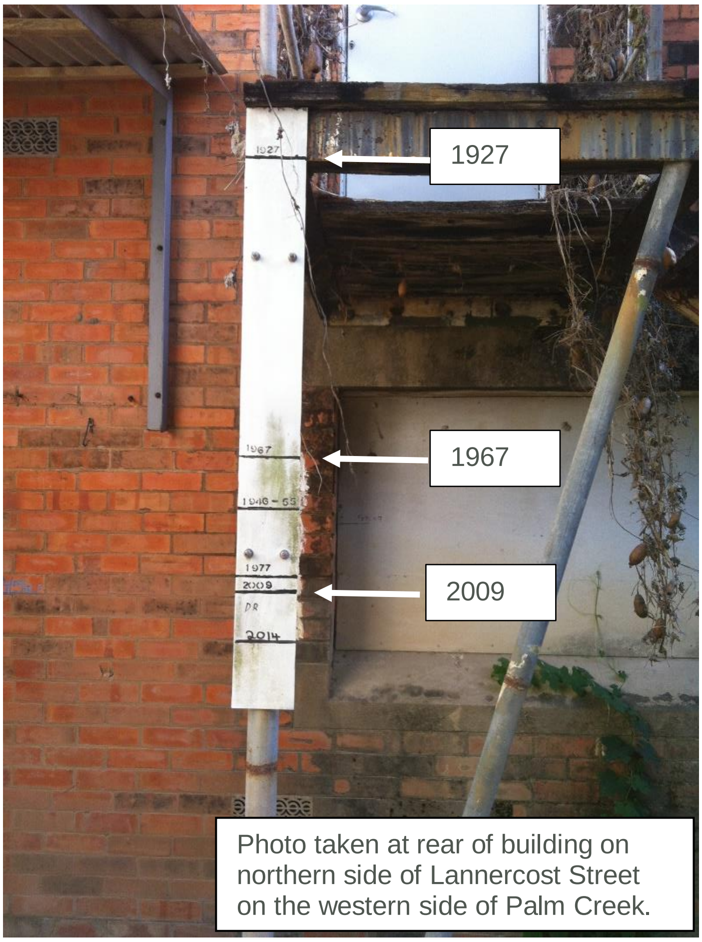


Figure 3-4 Flood Marks in Ingham

Figure 3-5 Flood Marks at Bemerside Hotel

3.4 Flood Frequency Analysis

The FFA was undertaken using the Flike software program which uses either a Bayesian inference framework or LH moments. The software uses global search to determine the most probable values of the parameters and calculates a second-order approximation of the posterior distribution. Confidence limits are then calculated together with flood quantiles and expected probability flood distributions.

3.4.1 Historic Information

The historic events outlined previously were incorporated in to Flike. That is, the 1927 flood event being the largest in the gauging record.

3.4.2 Prior Parameters Information

The higher order Log Pearson Type III parameters derived from the RFFE analysis were used as prior information to the Bayesian framework in Flike, i.e., the standard deviation (log flow) and skew (log flow) parameters. As well as the parameter values the standard deviations of the determined parameters, were used as prior information. The mean (log flow) parameter was determined from the streamflow data. The parameters used are shown in Table 3-4 (Ingham Pump Station) and Table 3-5 (Abergowrie).

Table 3-4 RFFE parameters for Ingham Pump Station

Parameter	Mean	St Dev	Correlation		
Mean (loge flow)	7.592	0.437	1.000		
St dev (loge flow)	1.230	0.190	-.210	1.000	
Skew (loge flow)	-0.672	0.186	-0.040	-0.410	1.000

Table 3-5 RFFE parameters for Abergowrie

Parameter	Mean	St Dev	Correlation		
Mean (loge flow)	7.491	0.437	1.000		
St dev (loge flow)	1.230	0.190	-0.210	1.000	
Skew (loge flow)	-0.672	0.186	-0.040	-0.410	1.000

Data Analysis

The flood frequency analysis was undertaken for the Ingham Pump Station using two distinct timeframes; the entire record (1916 to 2014) and a subset of the record (1967 to 2014), a period which coincides with the record length of the Abergowrie gauge. The FFA of the Ingham Pump Station Gauge showed that expected flows for the 1967 to 2014 period were approximately 5% lower than the expected flows when the analysis was undertaken for the entire record length. This finding was consistent with the observations of WBM (2003).

Consequently, the FFA flows derived for the Abergowrie gauge were increased by 5% (a process consistent with WBM (2003)).

3.5 Results

The results for the FFA are shown in Table 3-6, and presented graphically in Figure 3-6 (Abergowrie) and Figure 3-7 (Ingham Pump Station). This table lists the 100 year ARI peak discharge as 17,500 m^3/s at Abergowrie and 14,800 m^3/s at the Ingham Pump Station.

Table 3-6 FFA Results

ARI	Peak Discharge (m^3/s)	
	Abergowrie	Ingham Pump Station
2	2300	3300
5	5500	5500
10	8000	7300
20	10700	9200
50	14500	12200
100	17500	14800

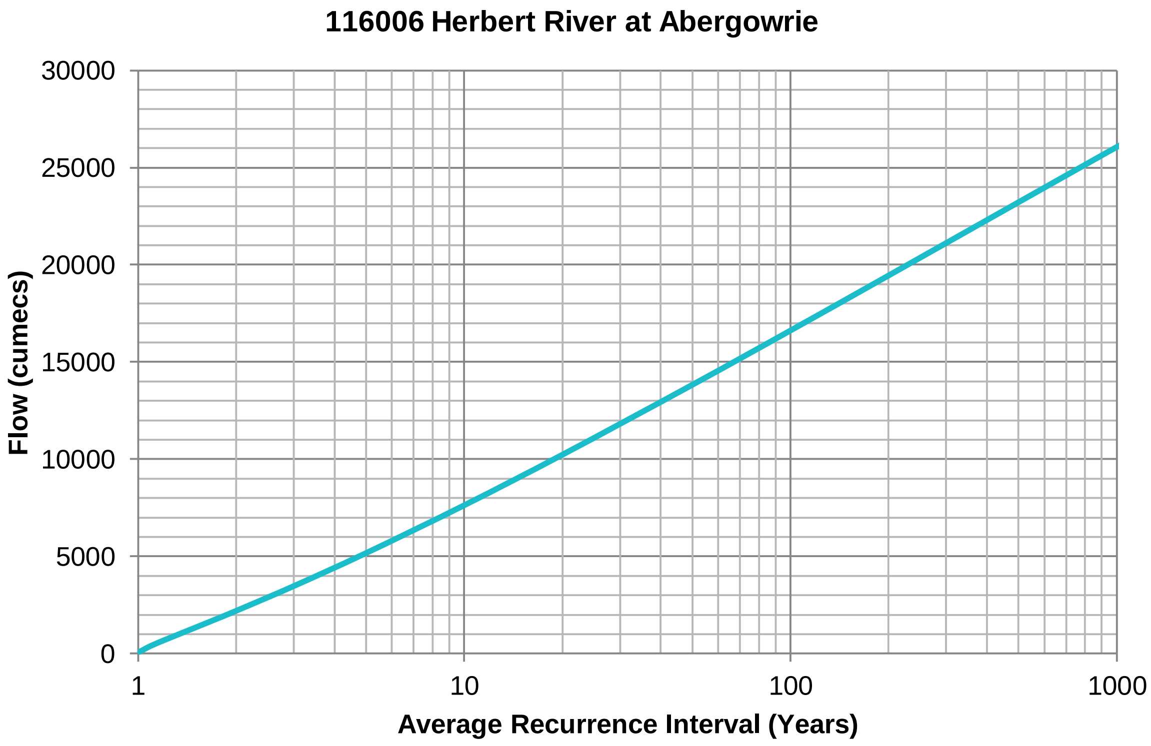


Figure 3-6 FFA Results: 116006 Herbert River at Abergowrie – Log Pearson Type III

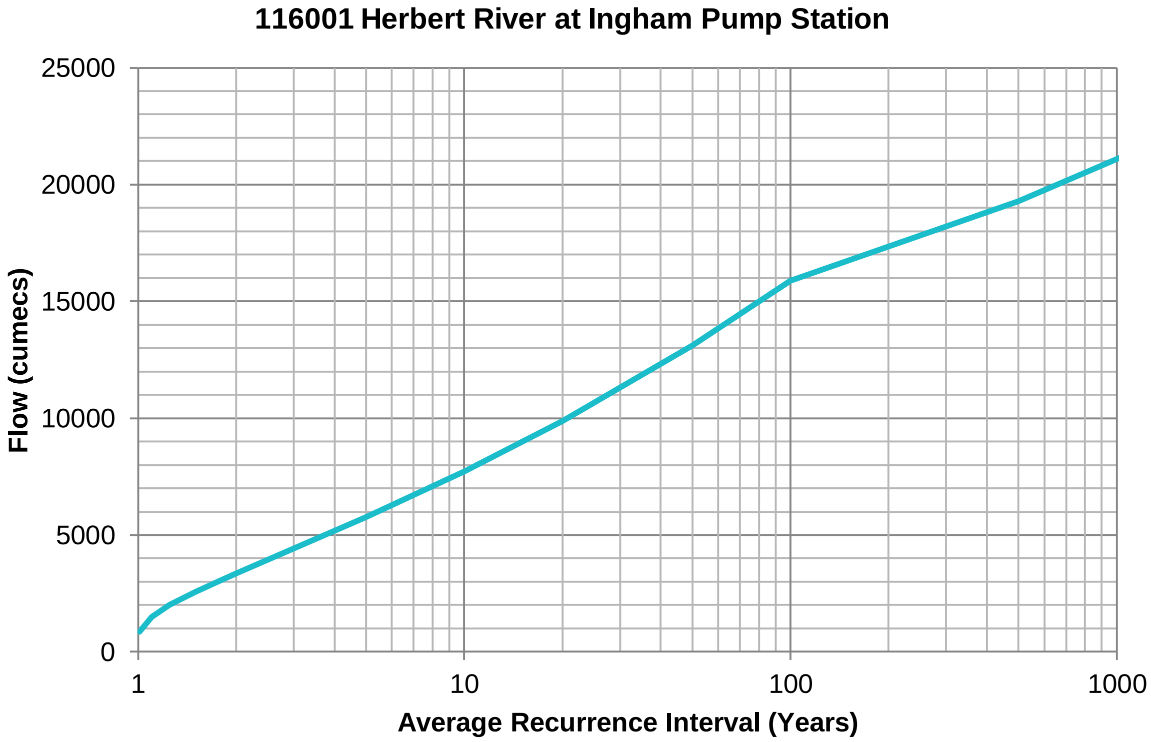


Figure 3-7 FFA Results: 116001 Herbert River at Ingham Pump Station – Log Pearson Type III

3.6 Comparison to Previous FFA

The results for the adopted FFA are compared to those originally presented in WBM (2003) in Table 3-7. This comparison shows that the peak flows at Abergowrie have generally increased by about 5 to 10%. The changes observed in the Abergowrie results are greater than those at Ingham Pump Station, a reflection of the significantly increased gauging record at Abergowrie utilised by the current assessment when compare to WBM (2003).

Table 3-7 Comparison of FFA Results

ARI	Peak Discharge (m^3/s)			
	Abergowrie		Ingham Pump Station	
	Adopted	WBM 2003	Adopted	WBM 2003
2	2300	2200	3300	2450
5	5500	5500	5500	5500
10	8000	7900	7300	7700
20	10700	10300	9200	9800
50	14500	13300	12200	12600
100	17500	15400	14800	14500
500	24400	NA	18300	NA

4 Climate Change Assessment

Following discussion with HSC, it was agreed that the design flood events would include allowances for the predicted effects of climate change.

To model climate change conditions, the rainfall intensity in the URBS hydrologic model was increased by 20%. This is based on a 5% increase in rainfall intensity per degree of global warming with predicted temperature increases of 4°C by 2100 (State of Queensland, 2010). It is important to note that the rainfall intensity of the PMF event was not increased as the PMF event already represents the largest probable rainfall.

To provide revised flows at Abergowrie (upstream hydraulic model boundary) the peak discharges from the existing URBS model and the 20% increase in rainfall intensity models were compared for each ARI at Abergowrie to provide a multiplication factor to apply to the adopted FFA results. The revised peak discharges at Abergowrie for each ARI event is shown in Table 4-1.

The downstream tidal boundary was also raised by 800mm to account for predicted sea level rise as advised by HSC.

Table 4-1 Climate Change Peak Discharge Comparisons at Abergowrie

ARI	Peak Discharge (m^3/s)	
	FFA Adopted	20% Increase in Rainfall Intensity
5	5500	7100
10	8000	10200
20	10700	13600
50	14500	18300
100	17500	22000
500	24400	30300

5 Flood Mapping

This section provides an overview of the flood mapping process used to update the flood mapping for the lower Herbert floodplain. The following flood mapping products were produced:

- **Flood Extent** – Peak flood inundation extent.
- **Flood Level** – Peak flood level contour lines at 0.2 mAHD increments.
- **Flood Depth** – Peak flood depth at the following increments:
  - 0.0m – 0.5m
  - 0.5m – 1.0m
  - 1.0m – 2.0m
  - 2.0m – 3.0m
  - 3.0m – 4.0m
  - > 4.0m
- **Flood Hazard** – Low, Medium and High peak flood hazard (refer to Section 5.1 for a description of adopted flood hazard criteria).

Each of the products listed above has been mapped for the 5y, 10y, 20y, 50y, 100y, 500y ARI and PMF design flood events.

The full suite of mapping products was provided to HSC in ArcGIS (.shp) format for future use and incorporation into databases.

5.1 Flood Hazard Criteria

In consultation with the HSC, the flood hazard criteria were developed by revising the criteria used by the Department of Natural Resources and Mines (DNRM).

The theory behind the flood hazard classification used by DNRM is documented in (Australian Emergency Management Institute, 2014). To summarise, the criteria comprises six vulnerability thresholds, each of which considers the severity of the flood condition in terms of depth, velocity and depth x velocity product, together with the vulnerability of people, buildings and infrastructure. The adopted flood hazard classification and associated vulnerability thresholds are presented in Figure 5-1.

For the purposes of the HSC flood mapping, the DNRM hazard criteria were combined to represent three hazard categories, Low, Medium and High, as detailed in Table 5-1 below.

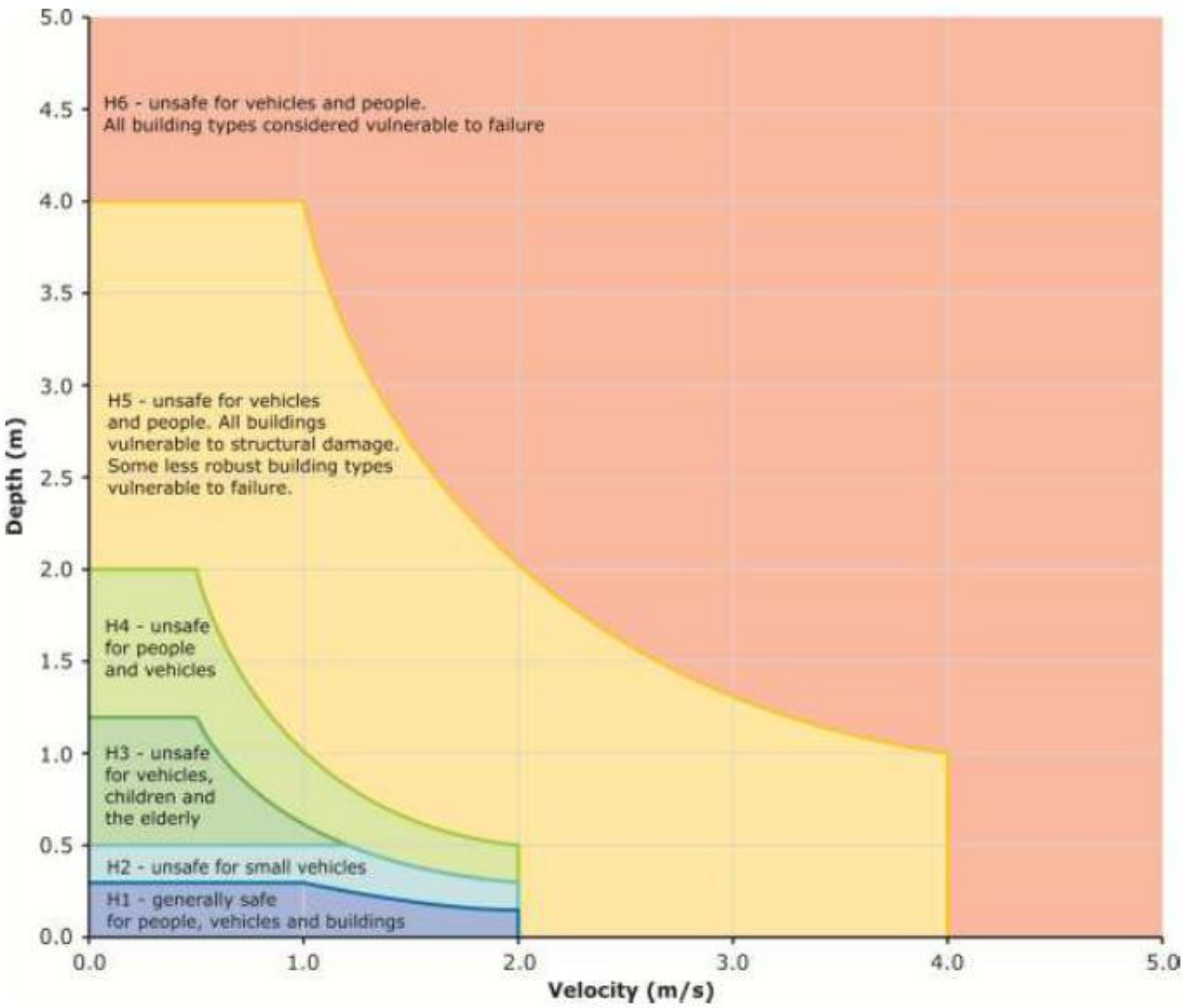


Figure 5-1 Flood Hazard Criteria (Australian Emergency Management Institute, 2014)

Table 5-1 Flood Hazard Classification – Vulnerability Thresholds

Hazard Classification	DNRM Criteria	Description
Low	H1	Generally safe for vehicles, people and buildings.
	H2	Unsafe for small vehicles.
Medium	H3	Unsafe for vehicles, children and the elderly
	H4	Unsafe for vehicles and people.
High	H5	Unsafe for vehicles and people. All building types vulnerable to structural damage. Some less robust building types vulnerable to failure.
	H6	Unsafe for vehicles and people. All building types considered vulnerable to failure.

6 Self-Assessable Development

HSC is considering incorporating into their planning scheme limited filling for self-assessable residential developments in the Ingham area. To understand the implications of this on flood levels, a flood impact assessment has been undertaken. They are also considering incorporating an overlay for areas where the Australian Building Code would require a structural assessment because the velocity exceeds 1.5 m/s. Mapping is provided for this overlay.

The investigation was primarily focussed on residential filling, but HSC asked that the assessment include filling on the smaller lots in the commercial precinct to the south of the airport.

For planning purposes the 100 year ARI design flood event was used to flood hazard and velocity for this assessment.

6.1 Permissible Fill Area Assessment

Initially two permissible fill options were assessed separately for:

(1) filling in low hazard areas;

(2) filling in medium and high hazard areas.

A fill depth of 650 mm was used in the assessment as this is the maximum fill that would be required in the low hazard areas to satisfy the planning scheme requirement of a 300 mm freeboard above the 100 year ARI flood level for habitable floor levels. By definition the maximum depth of flooding a low hazard area is 500 mm. The Australian Building Code requires that habitable floors be set 150 mm above the fill level. Therefore to achieve the 300 mm freeboard above the 100 year ARI flood level, a fill level of up to 650 mm would be required.

A fill depth of 650 mm was also adopted for the medium and high hazard area to be consistent with the low hazard areas. In the medium and high hazard areas this depth of filling will not be sufficient to allow a slab-on-ground type construction which satisfies the 300 mm freeboard in the 100 year ARI event. Rather it is intended to allow limited filling for say the construction of a shed or an under house area that provides some additional protection against smaller floods.

For the medium and high hazard areas it was expected that the flow velocity would be a significant factor in achieving an acceptable outcome. Initial sensitivity testing indicated that an acceptable outcome would not be achieved if filling occurred in areas where the velocity is greater than 0.5 m/s and hence the assessment focussed on areas where the velocity is ≤ 0.5 m/s. With the exception of a few larger areas such as along Palm Creek, through the Tyto Wetlands, and other smaller isolated patches, flow velocities in Ingham are less than 0.5 m/s.

Initial testing for both of the low hazard and medium/high hazard areas assumed filling on all lots, but this resulted in unacceptable impacts. To reduce the flood impacts it was agreed in discussion with HSC that, in the long-term, it is improbable that all lots would be filled and that a more realistic but still conservative uptake would be 1 in 3 lots.

Testing assuming a 1 in 3 lot uptake significantly reduced the impacts, but they were still considered to unacceptable. It was then decided limit the fill area to 500 m^2 on an individual lot. This gave a good outcome for the medium and high hazard assessment, but for the low hazard areas there was still some small areas of impact that was unacceptable to Council. These areas were found to have localised areas of velocity greater than 0.5 m/s, noting that the low hazard assessment up until this point did not exclude areas based on velocity.

At this point it was decided that velocity should be used to define the overlay and hence in the planning scheme there would not be separate overlays for the different hazard areas.

The results of the final assessment showed that the flood impacts of permissible fill area assessment are acceptable based on the following requirements:

- Lots are filled to a maximum depth of 650 mm.
- It has been assumed that 1 in 3 lots within the self-assessable development fill area would be ultimately be filled.
- Each lot filled has a maximum fill platform of 500 m^2.

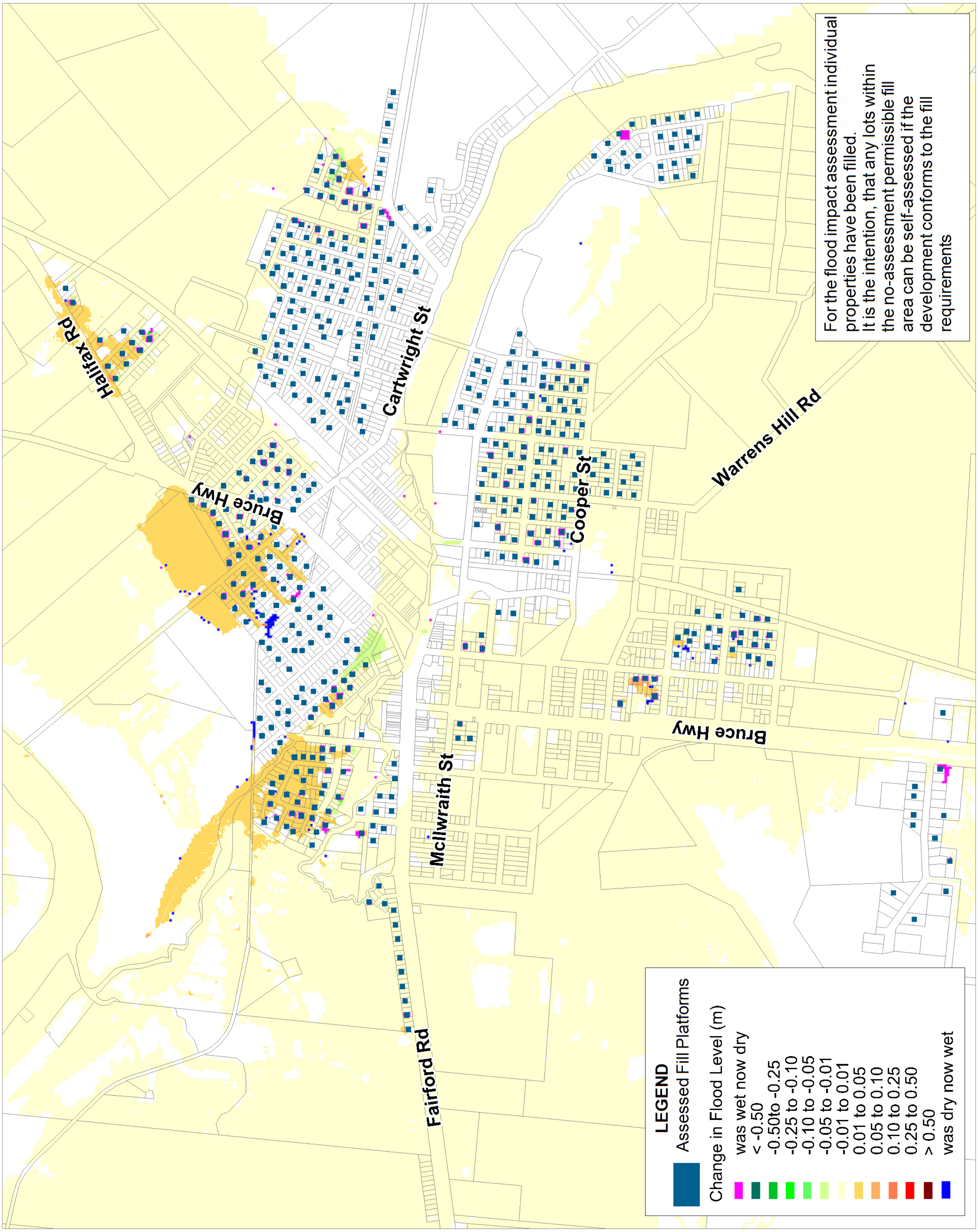
Whilst not specifically modelled in the final assessment it is also recommended that the planning scheme include a requirement that there be a setback from the property boundary to the fill platform. The intention of this is to minimise the effects of localised changes in flooding patterns on neighbour's properties. It is suggested that the setback be in the range 1.5 m to 2.0 m.

The resulting impacts on peak flood levels are shown in Figure 6-1 to Figure 6-5 for the 5 to 100 year ARI design events respectively. These figures show that for all design flood events the impacts on peak flood levels are restricted below 0.05 m (50 mm).

It is important to note that for the flood impact assessment individual properties have been filled, represented by the blue areas on the flood impact maps. However, the selection of these lots was for flood modelling purposes and it is the intention, that any lots within the no-assessment permissible fill area can be self-assessed if the development conforms to the fill requirements.

6.2 High Velocity Structure Assessment

The Australian building code requires that buildings located in areas with flood velocity greater than 1.5 m/s be assessed by structural engineers. GIS mapping defining areas of flow velocity greater than 1.5 m/s has been provided to HSC for inclusion into the planning scheme.



Title:

Self-Assessment Permissible Fill Area Impacts on 5 Year ARI Peak Flood Levels

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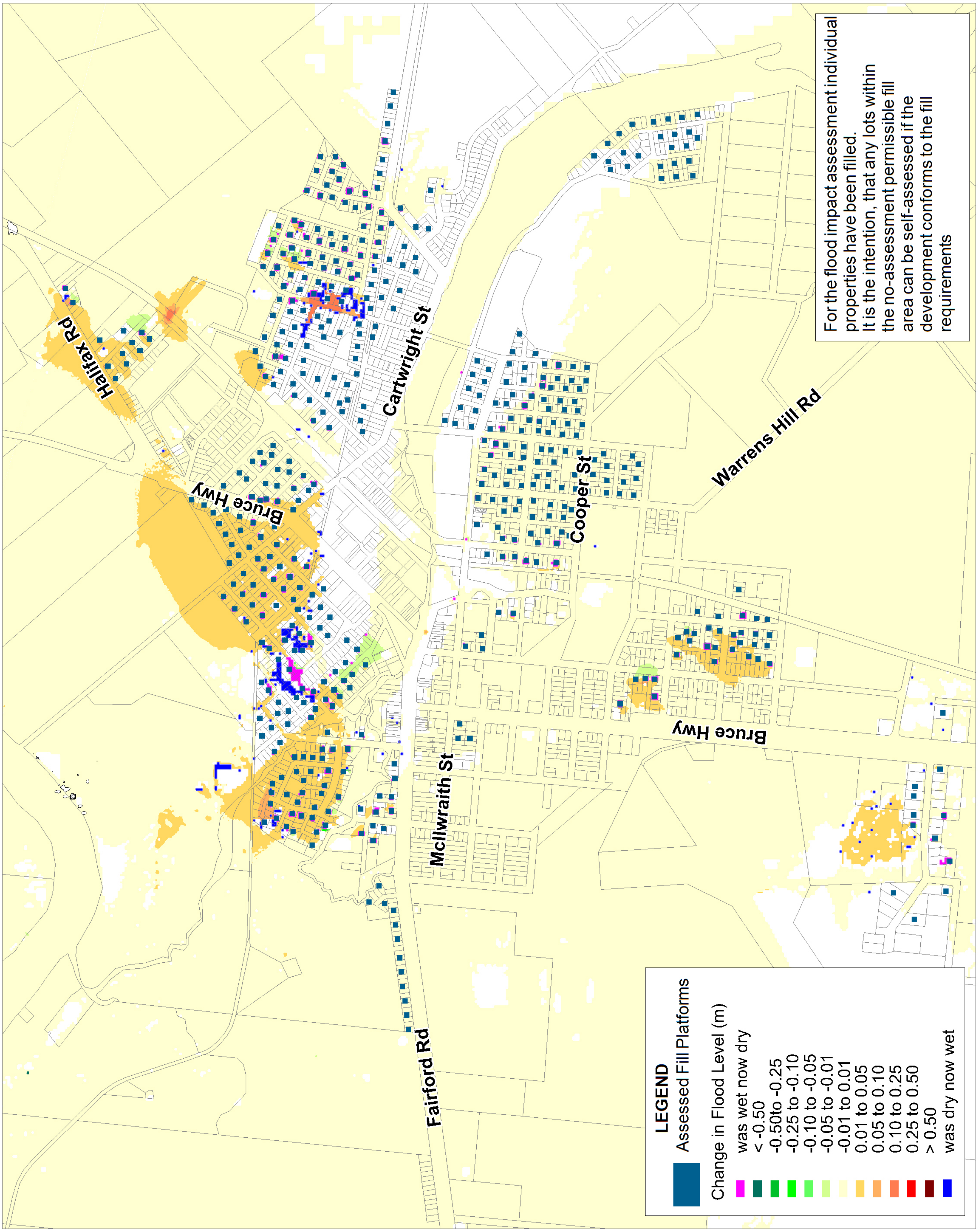

Title:

Self-Assessment Permissible Fill Area Impacts on 10 Year ARI Peak Flood Levels

Figure: **6-2**

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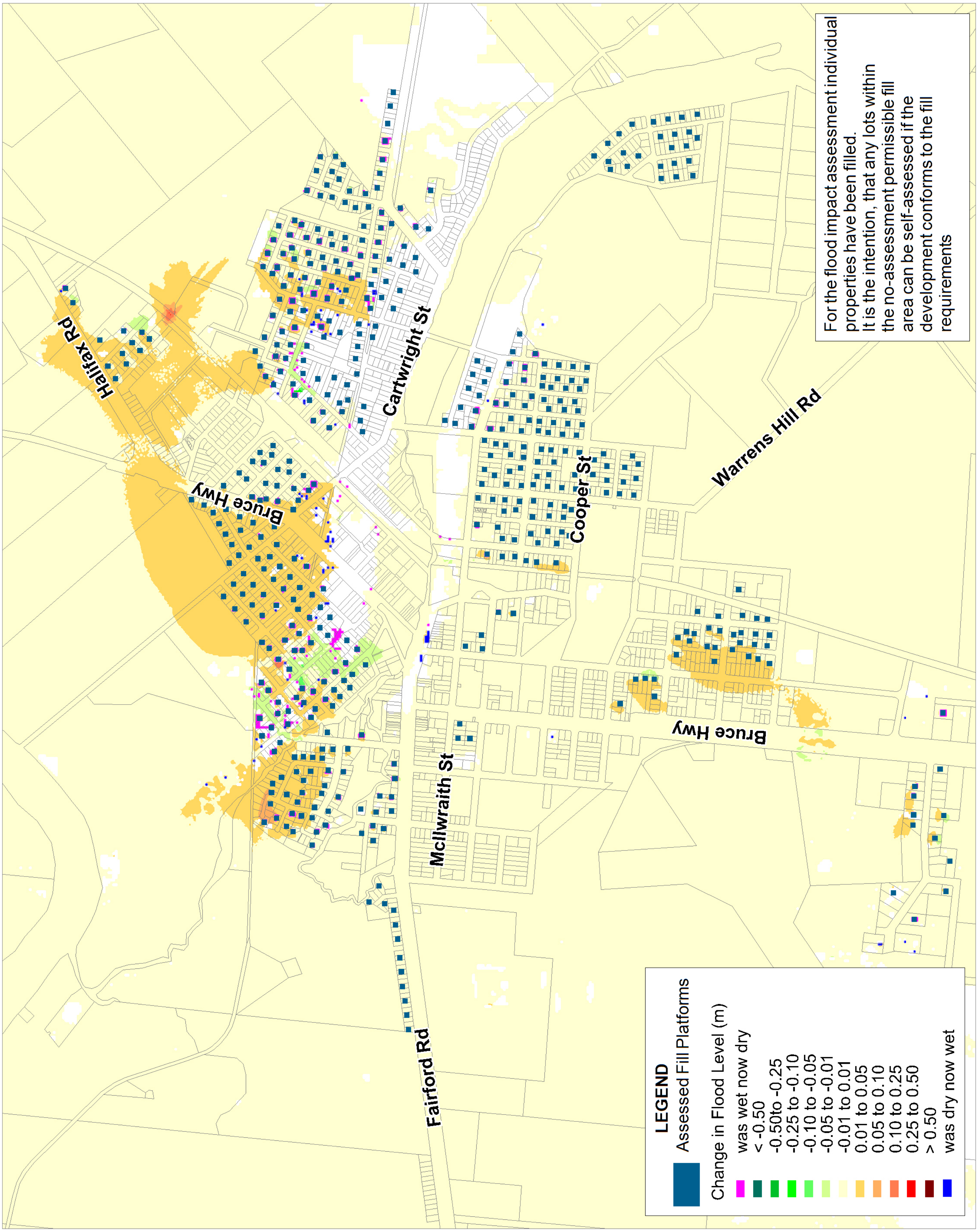

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Self-Assessment Permissible Fill Area Impacts on 20 Year ARI Peak Flood Levels

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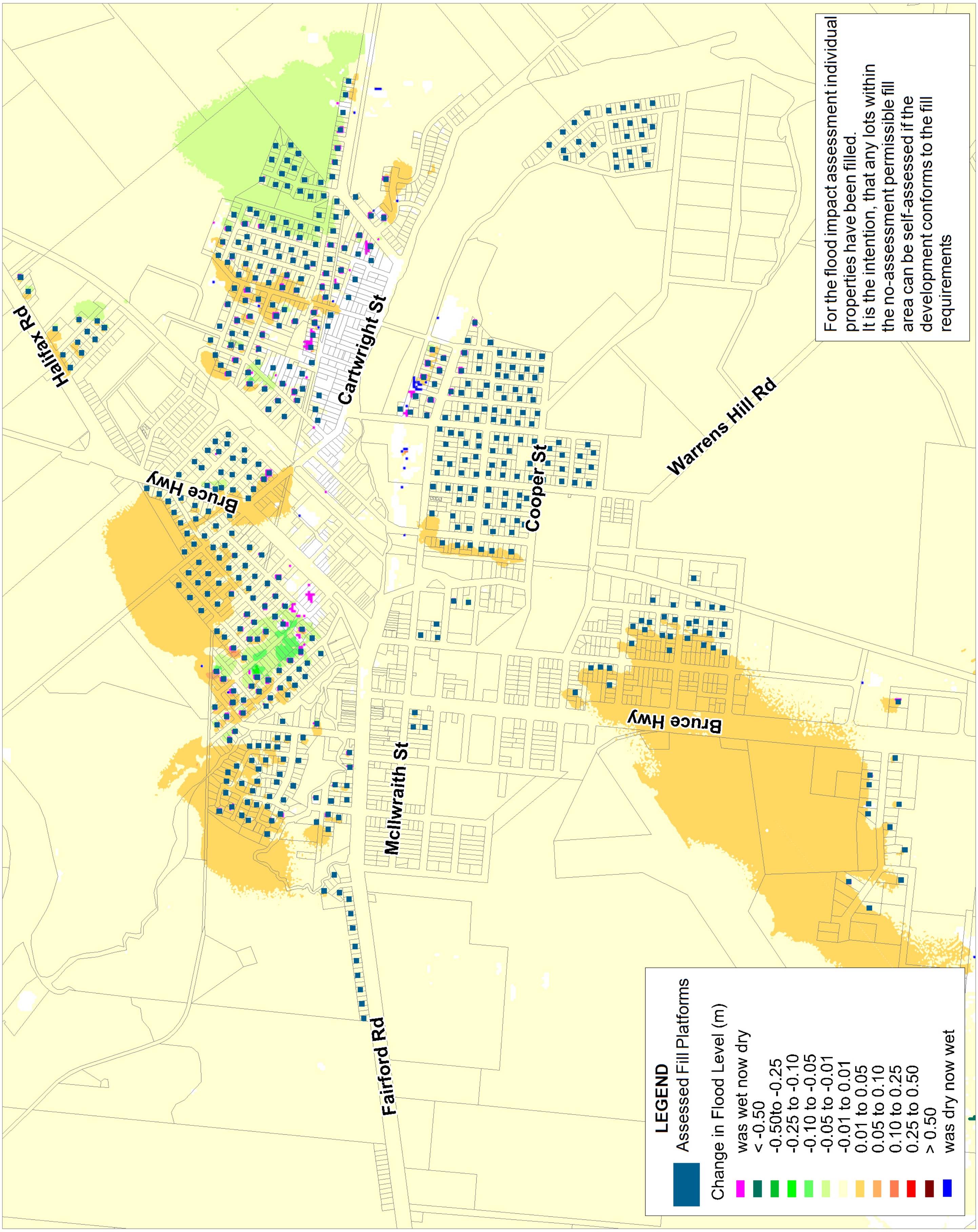

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Self-Assessment Permissible Fill Area Impacts on 50 Year ARI Peak Flood Levels	**6-4**	**A**

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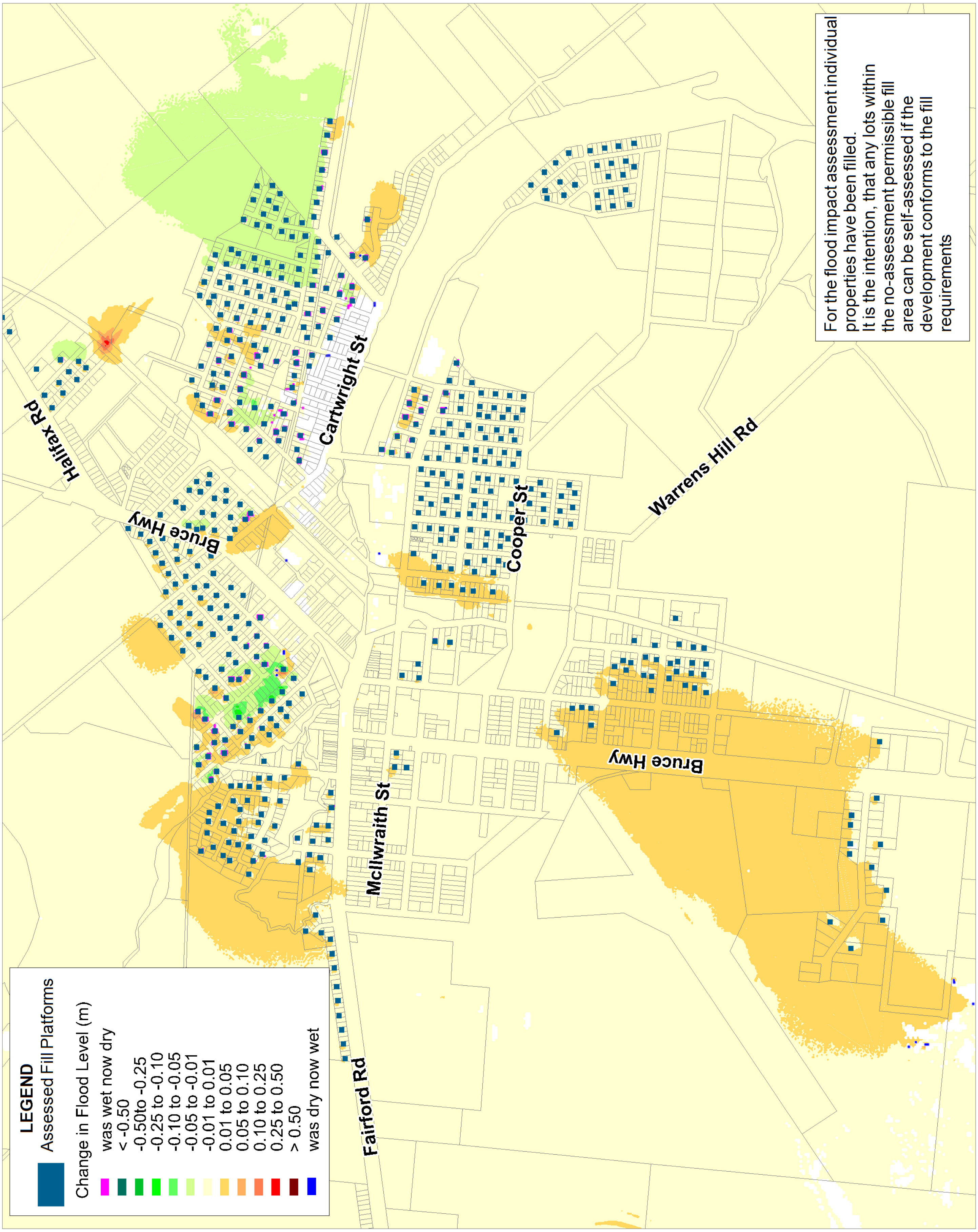

Title:

Self-Assessment Permissible Fill Area Impacts on 100 Year ARI Peak Flood Levels

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7 Data Handover

This report accompanies a DVD containing the updated flood mapping and the flood model.

7.1 Flood Mapping

The flood extent, level, depth and hazard mapping products, detailed in Section 5, provided to HSC in ArcGIS (.shp) format for future use and incorporation into databases.

A Read Me text file has been supplied with the MAPPING detailing naming conventions.

7.2 Flood Model

All of the model inputs for the final revision of the TUFLOW and URBS models updated for the Herbert River Flood Mapping Update study have been supplied. A Read Me text file has been supplied with the models detailing required model parameters and naming conventions.

The model scenarios supplied include:

- Existing Conditions: 5y, 10y, 20y, 50y, 100y, 500y and PMF events.
- Climate Change (20% increased rainfall intensity and 0.8 m sea level rise): 5y, 10y, 20y, 50y, 100y and 500y events.

8 Summary

The flood model and mapping for the Lower Herbert River Floodplain has been updated, including an allowance for climate change. This process was undertaken to provide HSC with updated flood mapping that includes all of the modelling developments made since the completion of the original flood mapping in 2003.

An investigation into the possibility of allowing limited fill in self-assessable development applications was investigated. After a number of iterations it was found that filling up to 650 mm depth and a maximum footprint of 500 m^2 would limit the magnitude and extent of flood impacts across the floodplain to a degree that would be acceptable to HSC. The assessment was based on the assumption that such filling may occur on up to 1 in 3 lots over time.

Accompanying this report, the flood model and updated mapping are provided in digital format (DVD).

9 References

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