Wollondilly
Shire Council

Attachment Booklet Part 2

Monday 11 December 2017

IN1

IN1 Attachments

1. Minutes of Wollondilly Floodplain Risk Management Committee October 2017
2. Stonequarry Creek Final Draft Flood Study

Monday 11 December 2017

IN1 – Stonequarry Creek – Final Draft Flood Study (Advisian 2017) – Public Exhibition

Wollondilly Shire Council

Minutes of Wollondilly Floodplain Risk Management Committee

Tuesday 31 October 2017 at 2pm
Council's Boardroom, Wollondilly Shire Council Admin Building

The meeting was chaired by Mike Nelson, Wollondilly Shire Council and declared open at 2.10pm.

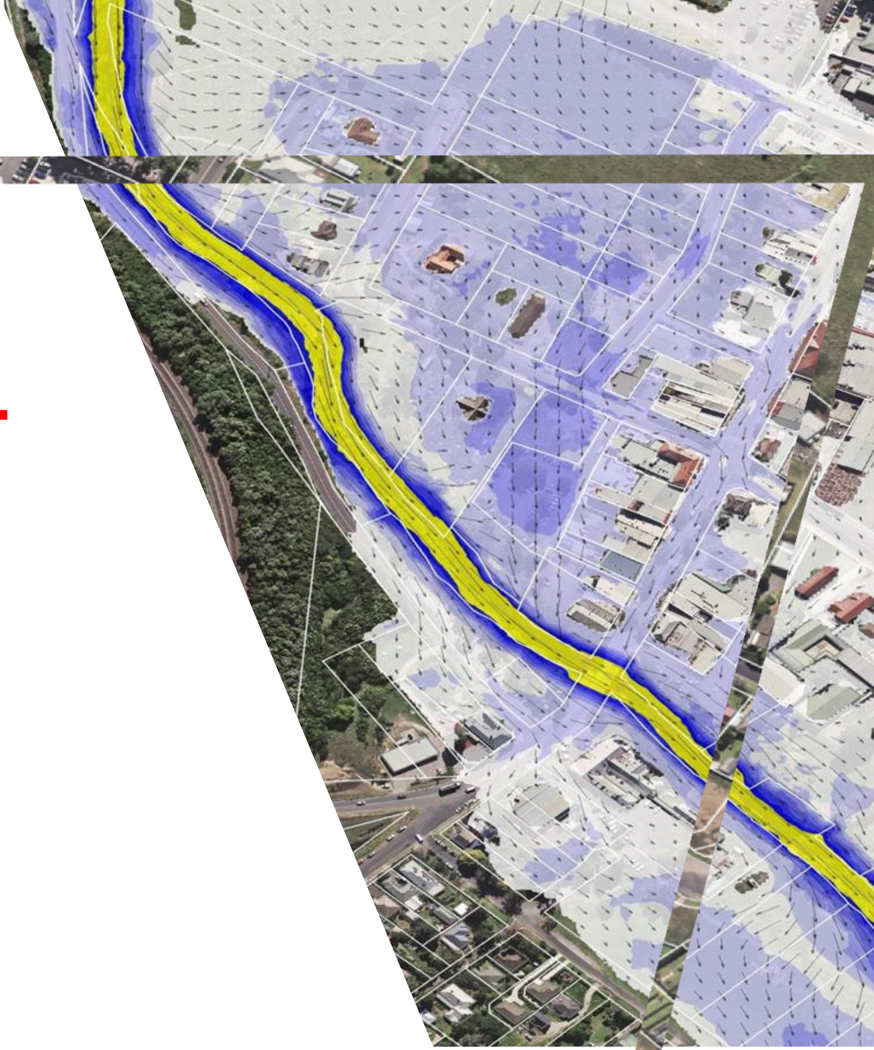
	Item	Action/Officer
1	**Acknowledgement of Country**	
	Acknowledgement of Country was undertaken out by Mike Nelson	
2	**Disclosure of Interest**	
	No declarations of interest were made	
3	**Attendance and Apologies**	
	Attendance: Cr Matthew Deeth participated in the meeting via phone conference Rod Wonson – SES Member Norman Dent – Community Member Trent Noonan - Community Member Jack Wilton – Community Member Roger Palmer – Community Member Chris Hughes – Community Member Leonie Gray - Community Member Wafaa Wasif – OEH Roy Golaszewski – Advisian Chris Thomas - Advisian Mike Nelson - Wollondilly Shire Council Ian Berthon – Wollondilly Shire Council David Henry - Wollondilly Shire Council Peter Wright – Wollondilly Shire Council Carolyn Whitten - Wollondilly Shire Council Robyne Ryan – Wollondilly Shire Council **Apologies**: Michael Silm – Community Member Garry Barnott-Clement – SES Cr Robert Khan	
4	**Presentation – Wafaa Wasif - OEH**	
	A presentation was made by Wafaa Wasiff from OEH on the Introduction and Overview of Floodplain Management in NSW.	Presentation will be made available in the Committee cloud link

5	**Stonequarry Creek – Final Draft Flood Study**	
5a	**Stonequarry Creek – Final Draft Flood Study** **Roy Golaszewski - Advisian**	
	A presentation was made by Roy Golaszewski from Advisian on the methodology and outcomes of the Draft Flood Study.	Presentation will be made available in the Committee cloud link
5b	**Recommendation to Council**	
	The committee discussed and agreed to recommend that Council place the Flood Study on public exhibition.	Recommendation from the Committee will be reported to Council in December
6	**Update – FRM Study & Plan**	
	• The Request for Quotes closed on Friday 27 October and Council is intending to engage a consultant to carry out the Floodplain Risk Management Study. The Committee will be notified when successful consultant has been engaged. • Council is hoping to run the early consultation for the Floodplain Risk Management Study together with the Draft Flood Study exhibition.	
7	**Setting Future Meeting Dates**	
	• The Committee noted that it would meet again at the end of the exhibition period which is anticipated to be in about March 2018, once the feedback from the Draft Flood Study is received. • The Committee discussed future meeting schedules and further discussion will take place. Monday afternoons, either 2pm or 3.30pm appears to suit for most members to attend.	
8	**Tasks/Actions**	
	• Report to Council in December to consider placing the Draft Flood Study on exhibition early in 2018 for a period of at least 28 days.	
9	**General Business**	
	• The Committee was advised that the focus of this Committee is the management of flooding in Wollondilly. Stonequarry is priority 1 and other floodplains will follow. • The Committee was advised that OEH have a Floodplain Management Program where there are grants available to Local Councils to fund Floodplain Risk Management. • The Committee was advised that commissioning of the study will show that Council has acted in good faith which affords protection under sec 733 of the Local Government Act. • The Committee discussed and noted that one of the options to be considered in future may be vegetation management/clearing. It was highlighted that the effect is likely to be marginal and must be balanced against many other costs/issues including the environmental aspects. • The Committee discussed the effects of pipe drainage and the technical advice was that it is limited due to the relative scale of the flood flows. • The Committee were advised that when the Study is placed on public exhibition it will include an engagement strategy that is planned to include a kiosk where a Council officer will be available to answer enquiries and also display the interactive mapping	

	system. The Study along with its plans and maps will also be available on Council's website. • The Committee were advised that future Development Applications will use the Flood Study to assist and determine applications within the Floodplain. • The Committee discussed the frequency of a review of a Flood Study and was advised that once adopted the Flood Study should be not need to be reviewed for about 10 Years. • The Committee noted that there is a rezoning proposal that is in progress within the Floodplain for Stonequarry Creek. The Committee may be asked to comment. • The Committee members agreed to setting up a group email for future correspondence.	
10	**Workplace Health and Safety**	
	No items raised.	

There being no further business to discuss, the meeting was declared closed at 4.45pm.

Next Meeting Date: Committee will be contacted for confirmation of meeting date to be convened after the exhibition period of the Stonequarry Creek – Final Draft Flood Study.



FINAL
DRAFT

Picton / Stonequarry Creek Flood Study

Level 17, 141 Walker St
North Sydney NSW 2060
Australia

rp301015-03199rg_crt170915-Stonequarry Ck Flood Study.docx

Issue 2 – Updated Final Draft

Wollondilly Shire Council

Advisian
WorleyParsons Group

www.**advisian**.com

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PROJECT: 301015-03199 – PICTON / STONEQUARRY CREEK FLOOD STUDY

REV	DESCRIPTION	AUTHOR	REVIEWER	WORLEY-PARSONS APPROVAL	DATE
0	**Preliminary Draft** *(Issued for Internal Review)*	RG	WJH		14/07/2014
1	**Draft Report** *(Issued for Client Review)*	RG	WJH		22/08/2014
2	**Final Draft Report** *(Updated following June 2016 Floods)*	RG	CRT		15/09/2017

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Acknowledgements

The Stonequarry Creek Flood Study was prepared by Advisian Pty Ltd on behalf of Wollondilly Shire Council with the financial assistance from the New South Wales Government through its Floodplain Management Program.

Foreword

The State Government's Flood Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the Government's Floodplain Development Manual (2005).

Under the Policy, the management of flood liable land remains the responsibility of Local Government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Local Government in the discharge of their floodplain risk management responsibilities.

The Policy provides for technical and financial support by the State Government through the following four sequential stages:

Stages of Floodplain Risk Management

	Stage	Description
1.	Flood Study	Determines the nature and extent of the flood problem.
2.	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed developments.
3.	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain.
4.	Implementation of Plan	Results in construction of flood mitigation works to protect existing development and the application of environmental and planning controls to ensure that new development it compatible with the flood hazard.

Wollondilly Shire Council commenced this process in 2005, when it engaged Advisian (*then Patterson Britton & Partners*) to develop a two-dimensional flood model of Stonequarry Creek and its floodplain. The model and its results were later used as the basis for the '*Stonequarry Creek – 2D Modelling and Climate Change Assessment*' (*WorleyParsons, 2011*).

With the availability of more reliable topographic data, Wollondilly Shire Council requested Advisian (*then WorleyParsons*) review and update the existing flood model. The updated model and its results are to be used to prepare a standalone Flood Study report titled '*Picton / Stonequarry Creek Flood Study*'.

Issue 1 of the '*Picton / Stonequarry Creek Flood Study*' was issued to Council and the Office of Environment and Heritage (*OEH*) in August 2014. Following reviews by Council and OEH, the report was recommended to be peer reviewed prior to being placed on public exhibition. The peer review was completed by Manly Hydraulic Laboratory in late 2016 prior to being finalised in mid-2017.

Over the time that the peer review was completed, further investigations were undertaken to validate the XP-RAFTS hydrologic model and RMA-2 hydrodynamic models to the June 2016 flood event. The validation was undertaken to recorded High Water Marks (*HWM*) and rainfall data and streamflow data.

The MHL peer review and the 2016 validation have been incorporated into the Flood Study report as '*Issue 2 – Final Draft*'.

Preparation of this Flood Study represents the first of the four stages in the process shown above. It has been prepared to assist Council and the community to understand and define the existing flood behaviour. The modelling developed for the Flood Study will subsequently be used to assess potential flood damage reduction options and future development scenarios as part of the Floodplain Risk Management Study and Plan.

1 INTRODUCTION

Stonequarry Creek is a tributary of the Nepean River that drains an 84 km^2 catchment located to the south-west of Sydney. As shown in **Figure 1**, Stonequarry Creek has its headwaters to the west of Picton, and is fed by four major tributaries namely, Racecourse Creek from the east, Crawfords Creek from the north and Cedar and Mathews Creek to the west of Picton. The tributaries rise to the east and west of Picton, with the highest elevations occurring to the east along the Razorback Range.

The township of Picton is located along Stonequarry Creek downstream of its confluence with all of its major tributaries. As shown in **Figure 1**, Stonequarry Creek generally flows in a southerly direction as it passes through Picton towards the Nepean River. Stonequarry Creek discharges to the Nepean River approximately 4.5 kilometres downstream of Picton.

This flood study covers the Stonequarry Creek catchment including parts of Racecourse Creek, Crawfords Creek and an unnamed Tributary, all of which join Stonequarry Creek upstream of the Picton CBD. The railway viaduct forms the downstream boundary to the south. The extent of the study area is overlayed on **Figure 1**.

Flooding of Stonequarry Creek can occur as a result of local catchment runoff breaking out of the main channel of Stonequarry Creek and its tributaries and inundating the surrounding floodplain. Although flooding in the vicinity of Picton is largely confined to areas adjacent to the creek, there are areas where 'breakouts' occur which can cause flooding across a wider floodplain area. During these larger flood events, floodwaters can overtop the banks of Stonequarry Creek and inundate parts of the Town Centre and surrounding urban areas. The floodplain drastically narrows at the railway viaduct where it enters a very steep sided gorge.

Since flood recordings began in 1956 there have been approximately nine (*9*) serious floods that have impacted Picton. The two largest of these flood events occurred in February 1956 and April 1969. The April 1969 flood is reported to have been the largest, with anecdotal information suggesting the flood peaked approximately one (*1*) metre above the Argyle Street Bridge.

In 1989, the NSW Department of Water Resources (*DWR*) completed a flood study for the Stonequarry Creek catchment titled the '*Picton Flood Study Report*' (*DWR, 1989*). The study involved the development of hydrologic and hydraulic computer models and their application to define flood behaviour across the floodplain of Stonequarry Creek. Flood modelling undertaken using HEC-2 predicted the 1969 event to be in the order of the 2% Annual Exceedance Probability (*AEP*) flood. In addition to reporting peak flood levels, hazard and hydraulic category mapping, the study also estimated flood damages for Picton and investigated flood monitoring and potential mitigation measures.

Flood discharges throughout the Stonequarry Creek catchment were determined using a hydrologic model that was developed using the RAFTS (*Runoff Analysis & Flow Training System*) software package. The flood behaviour across the floodplain was defined using the HEC-2 hydraulic modelling software with a total of 23 cross-sections defining the creek channel and adjoining floodplain. Due to limited historical data DWR was not able to calibrate either model.

Since the 1989 Flood Study there have been considerable changes within the catchment and across the floodplain. This has included land clearing and increased urban development. In recognition of the changes, Wollondilly Shire Council engaged Advisian (*then WorleyParsons*) in 2005 to update the hydrologic and hydraulic models to better represent existing conditions. This involved a review of the RAFTS model which led to it being updated to the latest version of the XP-RAFTS software.

In addition, a two-dimensional RMA-2 hydrodynamic flood model was developed for the floodplain areas extending upstream from the Picton CBD. At first the RMA-2 model covered the same extent as the HEC-2 model that was developed for the 1989 Flood Study. However, as more topographic data became available the model was extended further upstream.

The outcomes of Advisians previous investigations are documented in the following two reports:

- '*Stonequarry Creek – 2D Modelling & WaterRIDE Application*' (*2006*); and,
- '*Stonequarry Creek – 2D Modelling & Climate Change Assessment*' (*2011*).

In 2012, Wollondilly Shire Council obtained detailed LiDAR (*Light Detection and Ranging*) survey data for the study area. In recognition of the improved accuracy of this topographic data, Council re-engaged Advisian for the purpose of updating the existing RMA-2 model and application of it to re-simulate the 20%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods and the Probable Maximum Flood (*PMF*). In addition, Council requested that Advisian investigate the potential for climate change to impact on predicted peak 1% AEP flood levels.

In June of 2016, following completion of the RMA-2 flood modelling for all design events, sensitivity and climate change scenarios, Picton and much of the NSW coast experienced widespread heavy rainfall which led to major flooding along Stonequarry Creek. The major flooding that occurred in Picton resulted in significant damage to commercial and residential properties. Properties throughout the study area, including many along Argyle Street in the centre of town, experienced significant inundation with depths in excess of 1.5 metres recorded. A large number of trees and other in-bank vegetation were up-rooted during the flood and were conveyed downstream.

In the aftermath of the event Council collected High Water Mark (*HWM*) information for 76 locations along the creek system and across the floodplain. The HWM data, in conjunction with recorded rainfall data from nearby rainfall and streamflow gauges, was used to validate the XP-RAFTS and RMA-2 models relied upon for the Flood Study.

This report presents the work that has been undertaken to review and update the modelling of mainstream flood behaviour for Stonequarry Creek and its tributaries. This has been achieved by defining flows, flood levels, flood depths, velocities, and provisional hydraulic and hazard categories under current catchment and floodplain conditions. The report also includes the findings of the June 2016 validation of the XP-RAFTS hydrologic model and the RMA-2 flood models.

A peer review of Issue 1 of the '*Picton/Stonequarry Creek Flood Study*' (*WorleyParsons, 2014*) was completed by Manly Hydraulics Laboratory (*MHL*) at the request of Wollondilly Shire Council. (*refer* **Appendix A**). The objective of the peer review was to assess the key assumptions, procedures and conclusions made in the hydrologic and hydraulic modelling elements of the study and in the delineation of hazard and hydraulic categories.

As the peer review was prepared following finalisation of the hydrologic and hydraulic models and all modelling of design events, not all recommendations could be addressed in the Flood Study; particularly those that required re-modelling. On this basis Council requested that Advisian provide comment on the recommendations made by MHL within the Flood Study. These comments are included within **Appendix A**.

2 STUDY METHODOLOGY

2.1 GENERAL

Floodplain risk management in New South Wales generally follows the guidelines documented in the NSW Government's '*Floodplain Development Manual*' (*2005*). The Manual outlines the steps involved in the process and the activities required to be undertaken to successfully develop a Floodplain Risk Management Plan for flood affected areas.

A description of the inter-relationship between the various stages involved in the preparation of a Floodplain Risk Management Plan is provided overleaf. This flow chart also shows the link between the various outcomes of the studies involved in the floodplain risk management process and the implementation of measures to reduce flood damages (both planning and structural).

The formulation and implementation of floodplain risk management plans is the cornerstone of the Government's *Flood Prone Land Policy*. The primary objective of the Flood Prone Land Policy is to reduce the impacts of flooding on individual owners and occupiers of flood prone land, and to reduce private and public losses caused by flooding.

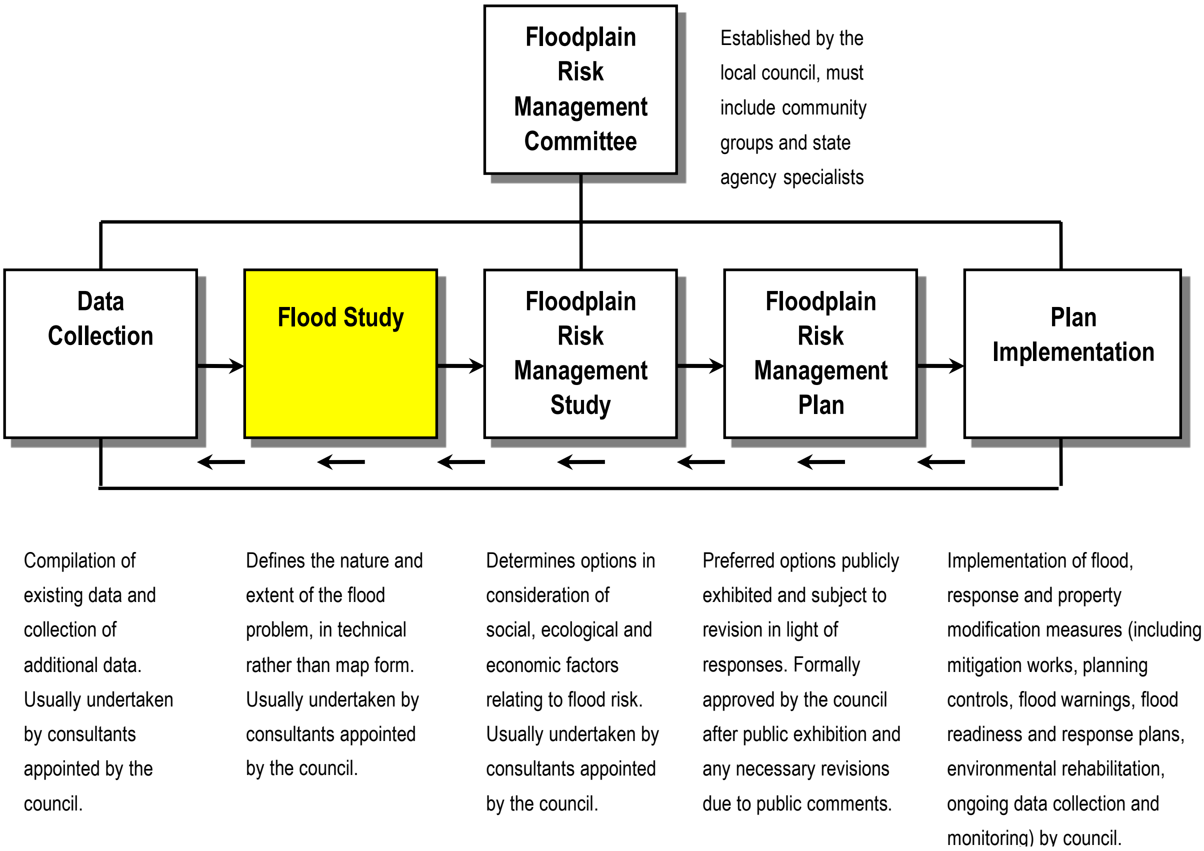
In this regard, the Policy recognises:

- that flood prone land is a valuable resource that should not be sterilised by unnecessarily precluding its development; and,
- that if all applications for development on flood prone land are assessed according to rigid and prescriptive criteria, some proposals may be unjustifiably disallowed or restricted, and equally, quite inappropriate proposals could be approved. (*NSW Government, 2005*)

One of the key steps involved in formulating a floodplain risk management plan is the recognition, definition and quantification of the principal factors associated with flooding. This information is presented in a Flood Study, which becomes a baseline document summarising flood related data which can be used to resolve floodplain risk management issues.

Wollondilly Shire Council initiated the process for the Stonequarry Creek by commissioning this study.

The aim of the study is to produce information on flood flows, velocities, peak flood levels, flood extents, and hydraulic and hazard category mapping for a range of flood events under existing floodplain and catchment conditions.



Source: *'Floodplain Development Manual'* (*2005*)

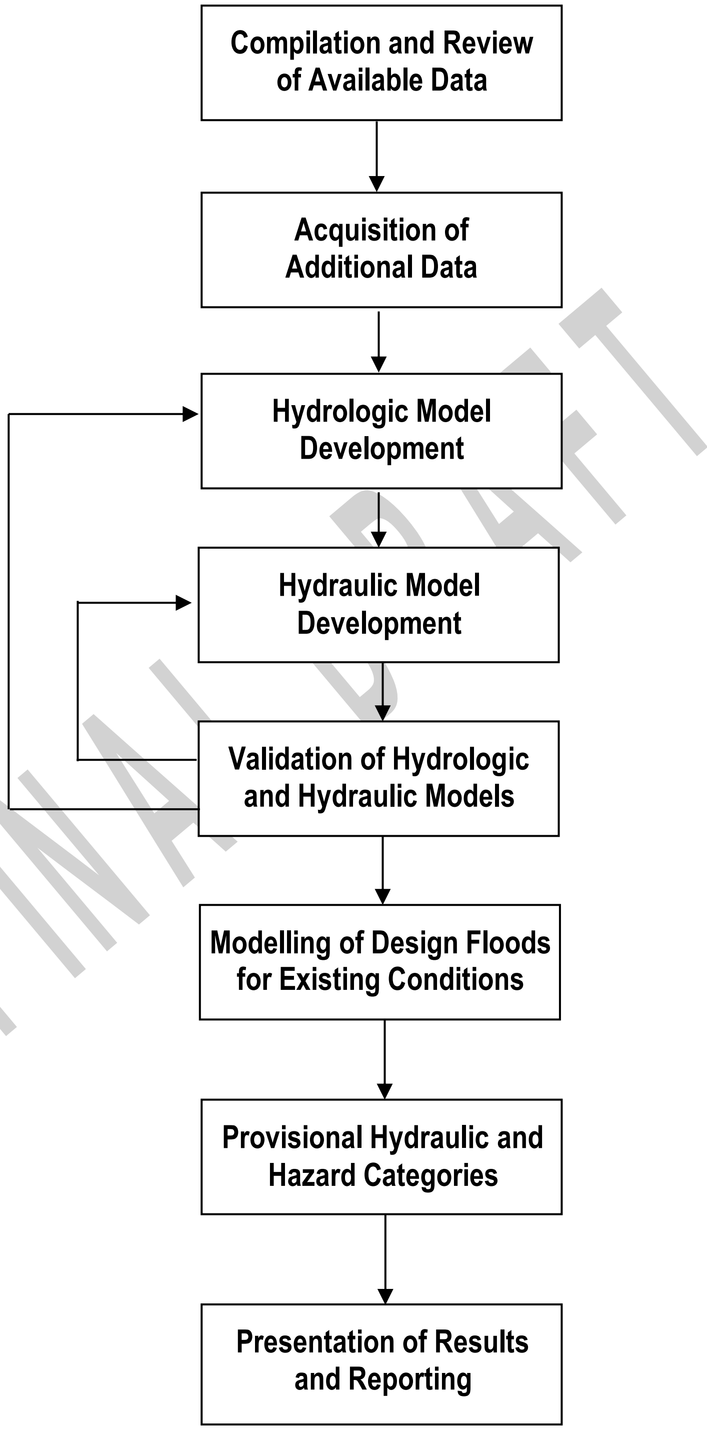
2.2 ADOPTED APPROACH

The general approach and methodology employed to achieve the study objectives involved:

- compilation and review of available information, including previously completed flood studies, streamflow gauge records, rainfall records, topographic mapping of the floodplain, hydrographic surveys of creek channels and details of bridge crossings;
- site inspections to establish catchment roughness, slope, and land-use, and to identify additional survey needs and critical hydraulic controls such as bridges and weirs;
- the collection of historical flood information, including records of peak flood levels for historical floods (*such as occurred in 1956, 1969 and recently in 2016*);
- the development of a computer based <u>hydrologic model</u> to simulate the transfer of rainfall into runoff and its concentration in streams during the flood;
- the development of a computer based <u>hydraulic model</u> to simulate the movement of floodwaters through the lower reaches of the floodplain;
- validation of the models against results from the 1989 Flood Study and against the June 2016 rainfall and flood event;
- the determination of peak water levels, flood flows, depths and flow velocities along Stonequarry Creek and its tributaries for the 20%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods and the Probable Maximum Flood (*PMF*);

- the determination of hazard and hydraulic category mapping for the 1% AEP flood; and
- modelling of climate change scenarios to predict potential changes in peak 1% AEP flood levels.

The flow chart shown below outlines the key steps and the sequence of work that has been undertaken in preparing this Flood Study.



2.3 COMPUTER MODELS

Computer models are the most reliable cost-effective tools available to simulate flood behaviour in rivers and streams. Two types of computer models were developed as part of the Flood Study for use in assessing and quantifying flooding characteristics within the Stonequarry Creek catchment. These are:

- a hydrologic model, covering the entire area of the Stonequarry Creek catchment and that of its tributaries; and,
- a hydraulic model, extending downstream of the Bakers Lodge Road crossing along Stonequarry Creek, and along a substantial portion of the major tributaries of Racecourse and Crawfords Creeks.

The **hydrologic model** simulates catchment runoff following a particular rainfall event. The main outputs from the hydrologic model are discharge hydrographs which define the quantity of runoff as well as the rate of rise, timing and magnitude of peak discharges resulting from the rainfall event. The discharge hydrographs are utilised as inputs into the hydraulic model.

The **hydraulic model** simulates the passage of floodwater along waterway reaches and across floodplain areas. The hydraulic model calculates key flooding characteristics such as flood levels, flow velocities, floodwater depths and flood hazard at selected points of interest throughout the study area.

Information on the topography and characteristics of the catchments, and the watercourses and their floodplains, is built into the models. For each historic flood, data on rainfall, flood levels and river flows can be used to simulate and validate (*calibrate and verify*) the models.

Development of the computer models involves:

- discretisation of the catchment, creek, floodplain, etc;
- incorporation of physical characteristics (*catchment areas, creek cross-sections, etc.*);
- setting up of hydrologic and hydraulic databases (*rainfall, creek flows, flood levels*) for historic events;
- calibration to one or more historic floods (*calibration is the adjustment of parameters within acceptable limits to reach agreement between modelled and measured values*); and,
- verification to one or more other historic floods (*verification is a check on the model's performance without adjustment of parameters*).

Once model development is complete, it may then be used for:

- establishing design flood conditions;
- setting flood standards for planning, so that future land-use is controlled to minimise potential losses/damage due to flooding;
- developing flood hazard mapping;
- hydraulic categorisation of the floodplain; that is, delineating floodway, flood storage and flood fringe;

- assessment and quantification of the impacts of climate change on design flood characteristics; and,
- the modelling of "what-if" management scenarios to assess the hydraulic impacts of structural mitigation measures; e.g., changes to a bridge structure to reduce upstream flood levels or the potential benefits of constructing a levee.

3 REVIEW OF AVAILABLE DATA

3.1 AVAILABLE DATA

A range of data is required to develop a flood model and for that model to be applied to simulate flood behaviour. Typically, contours of the land surface and cross-sections of the river and creek system are required to represent the floodplain topography and channel bathymetry. Details of critical hydraulic controls such as bridges and roadway embankments also need to be defined as they can influence flood behaviour. In addition, surface roughness parameters are required to reflect the influence that land features may have on the way floodwaters travel overland. These are usually based on consideration of vegetation density and soil type.

Calibration and verification of the model requires the collection of stream flows and flood level information for calibration and verification for a series of historic floods. Design flood simulation requires that the peak flows entering the modelled area have been established. This requires hydrologic modelling to be undertaken to determine design discharges for the creek.

The data for this study has been obtained from Wollondilly Shire Council and from previous investigations such as the 1989 Flood Study.

A detailed description of the data available to this study is provided in the following sections.

3.2 PREVIOUS INVESTIGATIONS

A number of previous hydrologic and hydraulic investigations have been undertaken to examine the nature and extent of flooding along Stonequarry Creek. These include the following reports:

- *'Flood Study Report, Stonequarry Creek'* (*Department of Water Resources, 1989*)
- *'Stonequarry Creek – 2D Modelling and WaterRIDE Application'* (*Patterson Britton & Partners, 2006*)
- *'Stonequarry Creek – 2D Modelling and Climate Change Assessment'* (*WorleyParsons, 2011*)

These investigations provide useful information and flood related data that is of use for this study. A brief synopsis of each is presented in the following sections.

3.2.1 Flood Study Report, Stonequarry Creek (NSW Department of Water Resources, 1989)

This report (*referred to hereafter as the "1989 Flood Study"*) was prepared by the NSW Department of Water Resources. The study was commissioned in May 1986 by Wollondilly Shire Council in recognition of the increasing demand for development to occur in areas that were thought to be flood liable.

The primary objectives of the study were to define design flood conditions (*levels, velocities, hazards and hydraulic categories*) throughout the study area. The study also aimed to assess several channel improvement scenarios and to quantify the potential flood damages that could occur under current floodplain conditions.

Flood discharges throughout the Stonequarry Creek catchment were determined using a RAFTS hydrologic model of the catchment (*refer* **Figure 2**). Due to limited availability of reliable historic data, the RAFTS model was not able to be readily calibrated and/or validated. The model was used to simulate the 5%, 2% and 1% AEP events.

Flood behaviour along Stonequarry Creek and its floodplain was defined using theHEC-2 software. A HEC-2 steady-state model was developed and used to simulate flooding along the section of Stonequarry Creek extending downstream from the its confluence with Racecourse Creek. The model extended downstream to the Main Southern Railway Viaduct crossing of Stonequarry Creek. In total, the HEC-2 model consisted of 23 cross-sections, the locations of each being shown in **Figure 3**.

The hydraulic model was used to simulate the 5%, 2% and 1% AEP events. The flow hydrographs for each event were defined using results generated from the RAFTS hydrologic model.

The report outlines design flood characteristics for the 5%, 2% and 1% AEP events. This data includes peak flood levels, flow velocities and flows at each of the cross-sections within the hydraulic model. The peak 1% AEP flood levels determined as part of the study are shown in **Table 1** for the natural creek scenario and a channel clearing scenario. Where applicable, corresponding locations have been identified, such as adjacent road crossings.

Table 1 DESIGN 1% AEP FLOOD LEVELS FOR STONEQUARRY CREEK FROM THE 1989 FLOOD STUDY

HEC-2 CROSS-SECTION NO. & LOCATION (*refer* **Figure 3**)	PREDICTED 1% AEP LEVEL (mAHD)	
	1989 HEC-2 (*Natural Scenario*)	**1989 HEC-2 (Channel Clearing)**
23 (Upstream End of HEC-2 Model)	161.47	160.80
17 (Elizabeth Street)	158.54	158.36
14 (Upstream Argyle Street)	158.11	157.87
13 (Upstream Argyle Street)	157.99	157.85
9 (Baxters Lane)	157.12	156.82
3 (Upstream Railway Line)	155.78	155.47
2 (Downstream Railway Line)	155.42	155.16
1 (Downstream End of HEC-2 Model)	154.85	154.85

3.2.2 Stonequarry Creek – 2D Modelling and WaterRIDE Application" (*Patterson Britton & Partners, 2006*)

In 2006 Wollondilly Shire Council engaged Patterson Britton and Partners (*now part of WorleyParsons*) to update the 1989 Flood Study using current and two-dimensional modelling techniques. This involved updating the 1989 hydrologic model to current catchment conditions in order to better reflect the increased urbanisation that had occurred since 1989. Instead of updating the 1989 HEC-2 model, a new two dimensional RMA-2 model was developed, covering the same extent as the 1989 HEC-2 model.

Updates to the RAFTS hydrologic model included increases to the impervious area for certain catchments where the extent of urban development had increased since 1989. RAFTS Model Nodes 2.00, 2.01, 6.04, 1.09 and 1.10 were updated to reflect a higher portion of impervious area (*refer* **Figure 2**).

The RMA-2 model was developed based on a Digital Terrain Model (*DTM*) developed from the digitised HEC-2 cross-sections. Roughness parameters were initially assigned to each of the element types based on those values adopted in the HEC-2 model. However, following verification of the model the roughness values and distribution were adjusted according to a review of aerial photography and water level comparisons.

Due to limitations in available historic flood data the RMA-2 model could not be calibrated and as such was only verified against the 1989 HEC-2 modelling results. **Table 2** on the following page provides a comparison of water levels generated by each model. As shown, the RMA-2 model generally predicted flood levels that were higher than those predicted by the 1989 HEC-2 model.

The RMA-2 model underwent further updates in order to incorporate any floodplain changes that had occurred since 1989. The most significant update was the Davies Place Development which was incorporated using "as built" drawings provided by Council.

The updated RMA-2 model was then used to re-simulate design flood conditions for the 5%, 2% and 1% AEP floods as well as for an Extreme Flood; the Probable Maximum Flood. The report documents peak flood levels and velocities throughout the study area as well as provides detailed flood extent mapping and depth and velocity mapping.

Updated hazard and hydraulic category mapping is also provided for the study area based on the detailed two-dimensional model results.

Table 2 COMPARISON OF 1% AEP 1989 HEC-2 AND 2006 RMA-2 LEVELS

HEC-2 CROSS-SECTION No. (*Refer* Figure 3)	PREDICTED 1% AEP FLOOD LEVEL (*mAHD*)		
	1989 HEC-2 MODEL (*mAHD*)	2006 RMA-2 Model (*mAHD*)	DIFFERENCE (*m*)
23	160.93	161.12	+ 0.19
22	159.95	160.40	+ 0.45
21	159.59	160.38	+ 0.79
20	159.43	159.90	+ 0.47
19	159.19	159.36	+ 0.17
18	158.64	158.90	+ 0.26
17	158.44	158.71	+ 0.27
16	158.30	158.44	+ 0.14
15	158.05	157.94	- 0.11
14	157.98	157.84	- 0.14
13	157.71	157.76	+ 0.05
12	156.94	157.54	+ 0.60
11	156.92	157.37	+ 0.45
10	156.75	156.96	+ 0.21
9	156.69	156.83	+ 0.14
8	156.54	156.74	+ 0.20
7	156.43	156.59	+ 0.16
6	156.29	156.37	+ 0.08
5	156.05	155.91	- 0.14
4	155.54	155.51	- 0.03
3	154.82	154.93	+ 0.11
2	154.08	153.63	- 0.45
1	153.62	154.10	+ 0.48

3.2.3 Stonequarry Creek – 2D Modelling and Climate Change Assessment (*WorleyParsons, 2011*)

This study was commissioned by Wollondilly Shire Council in order to extend the 2006 RMA-2 flood model and to carry out a climate change assessment for the study area.

The RMA-2 model was extended further upstream along Stonequarry, Racecourse and Crawfords Creeks based on a combination of detailed survey data and 2 metre contours provided by Council. Four upstream boundaries / inflow locations were adopted for the extended model, located along Stonequarry, Racecourse and Crawfords Creeks as well as an unnamed creek.

Updated hydrologic modelling identified that a critical duration of 9 hours applied to the study area generating the greatest discharge at the furthermost downstream model node (*i.e., RAFTS node 1.10, refer* **Figure 2**). This critical duration was longer than the 6 hour duration identified by the NSW Department of Water Resources as part of the 1989 Flood Study and adopted in the 2006 investigations. The change in critical duration to 9 hours was found to increase peak discharges by approximately 15% to 20% at Node 1.10.

A comparison of peak discharges generated at RAFTS Model Node 1.10 (*refer* **Figure 2**) is reproduced below as **Table 3**.

Table 3 PEAK DISCHARGE AT RAFTS MODEL NODE 1.10

DESIGN FLOOD	EXISTING (1989) DISCHARGE/ CRITICAL DURATION (m^3/s)	UPDATED MODEL DISCHARGE/ CRITICAL DURATION (m^3/s)	DIFF (%)
1% AEP	*494 (6hr)*	*578 (9hr)*	*+17%*
2% AEP	*424 (6hr)*	*509 (9hr)*	*+20%*
5% AEP	*345 (6hr)*	*431 (9hr)*	*+25%*

Updated flood model results in the form of flood level, depth and velocity, flood hazard and hydraulic categories mapping were presented based on the extended RMA-2 model and the updated discharge information (*based on a 9 hour critical duration*).

An assessment of Climate Change conditions was completed based on adoption of the methods outlined in the NSW Department of Environment and Climate Change's (*DECC, now OEH*), guideline document entitled '*Practical Consideration of Climate Change*'. In accordance with the guideline document, a sensitivity analysis was carried out by increasing 1% AEP rainfall intensities by 10%, 20% and 30% in the RAFTS hydrologic model. The RMA-2 model was then re-run using the RAFTS results to determine the impact on peak flood levels.

The maximum increase in peak 1% AEP flood levels for a 10%, 20% and 30% increase in rainfall intensity was 0.5 metres, 0.9 metres and 1.3 metres, respectively. These maximum increases occurred immediately upstream of the Railway Viaduct. Throughout the Picton CBD, the increases were substantially less; approximately 0.2 metres, 0.4 metres and 0.6 metres, respectively.

3.3 SUMMARY OF AVAILABLE DATA

3.3.1 Topographic / Hydrographic Data

As part of the data collection and review phase for the study, all available survey along Stonequarry Creek and its tributaries, and across the broader floodplain was compiled. This involved a review of the survey data that was collected for the previous studies outlined above.

The topography of the study area can be defined using the following sources:

- Digital Elevation Model (*DEM*) data for the floodplain developed from LiDAR (*Light Detection and Ranging*) data gathered in 2012;
- DEM data developed from site specific survey;
- Previously surveyed creek cross-sections collected for the 1989 Flood Study;

These data sources are described in the following sections.

Surveyed Cross-sections from the 1989 Flood Study

The location and extent of the 23 cross-sections from the original HEC-2 modelling is shown in **Figure 3**.

Light Detection and Ranging (*LiDAR*) Data

Light Detection and Ranging (*LiDAR*) data is available for much of the study area and for the entire extent of the hydraulic model. The data contains thousands of points defining the existing ground surface elevations.

The latest available data was collected by AAM Pty Ltd in August 2012 with a nominal vertical accuracy of 0.15 metres across clear areas. The extent of the available LiDAR data is shown in **Figure 4**.

LiDAR capture is unable to penetrate through water, and so the data does not typically include hydrographic features important for flood modelling, such as the bed level of streams that carry water under normal flow conditions.

However, Stonequarry Creek and its tributaries were not carrying significant flow during the periods when the LiDAR data was collected. Moreover, the definition of the creek beds and banks was compared to the surveyed cross-sections collected for the 1989 Flood Study and it was determined that the LiDAR data adequately defines the bed and banks within the study area. Accordingly, the LiDAR data has been used to define the channel and floodplain for the Stonequarry Creek system.

Site Specific Survey

Site specific survey information was available for the Davies Place development to the north of the Picton Town Centre. The "*as built*" design drawings had been provided by Council to be incorporated into the original RMA-2 model developed between 2006 and 2011.

Given the availability of the recently collected LiDAR data the Davies Place survey was only used as a check to confirm the LiDAR elevations. This was particularly helpful along the drainage channel which runs along the western edge of the site.

3.3.2 Historic Flood Levels

Historic flood level information along Stonequarry Creek is available for the 1969 and June 2016 floods. The flood level data available for the 1969 flood has been extracted from the 1989 Flood Study and is reproduced in **Table 4**. Unfortunately, the data is of limited us as the exact times when the flood levels were recorded and the actual locations where they were recorded are not specified.

Table 4 OBSERVED 1969 HISTORIC FLOOD LEVELS

LOCATION	RECORDED 1969 FLOOD LEVEL *(mAHD)*
Picton Plaza	157.14
Middletons Store	157.04
Picton Hotel	157.52
Westpac Bank	157.56
Travel Pac Travel Agency	156.95
Cottage (Elizabeth Street)	158.03
Residence (Abbotsford Road)	161.56
Residence (Menangle Street West)	157.12
Residence (Opposite Showgrounds)	156.58

Source - 'Picton Flood Study' (1989)

The flood level data available for the June 2016 flood is much more comprehensive with a total of 76 High Water Marks (*HWMs*) recorded throughout the study area. The information and photos accompanying the data indicates that the majority of flood level recordings were based on debris lines observed on fences, trees and buildings (*both externally and internally*).

Where possible the HWMs were surveyed to determine a peak flood level elevation relative to Australian Height Datum (*AHD*). Approximately 30% of all 2016 HWMs (*23 HWMs*) were surveyed.

A height above ground measurement was taken for the remaining HWMs. This height information was translated to an elevation in metres above AHD by adding the measurement to a ground elevation extracted from available LiDAR data. This approach is less reliable than field survey (*refer above*) but is expected to provide a vertical accuracy of +/- 0.2 metres.

Where HWM heights were measured inside buildings, the elevation in metres above AHD was determined based on floor level heights surveyed by Council.

The location of all June 2016 HWMs are shown in **Plate 1**.

Elevations for the all available 2016 HWMs are shown in **Figures 5**, **6** and **7**. A note is included with each HWM record indicating whether the reliability of the HWM is considered to be good, average or poor. This rating is based on notes provided by Council which document the source of the HWMs (*i.e., distinct debris line versus debris scattered in a tree or anecdotal*) and the collection method (*surveyed versus inferred height*).

Table 5 provides a comparison between flood levels recorded for the 1969 and 2016 events. The comparison is made relative to the available 1969 HWMs only and has an estimated accuracy of +/- 0.2 metres. The accuracy takes into consideration the source of the data and the uncertainty surrounding the location of many of the 1969 HWMs.

Table 5 COMPARISON BETWEEN OBSERVED 1969 AND 2016 HISTORIC FLOOD LEVELS

LOCATION	RECORDED FLOOD LEVEL (mAHD)	
	1969 HISTORIC FLOOD ^	2016 HISTORIC FLOOD ^^
Picton Plaza	157.14	158.60
Middletons Store	157.04	/
Picton Hotel	157.52	158.40
Westpac Bank	157.56	158.70
Travel Pac Travel Agency	156.95	/
Cottage (Elizabeth Street)	158.03	158.90
Residence (Abbotsford Road)	161.56	162.85
Residence (Menangle Street West)	157.12	158.30
Residence (Opposite Showgrounds)	156.58	157.20

^ *1969 Flood levels are based on HWMs extracted from the 'Picton Flood Study' (1989)*

^^ *2016 Flood levels are based on levels recorded at the nearest HWMs (refer* ***Plate 1****). Levels have been provided to the nearest 0.05 m recognising that HWM locations do not correlate exactly at the noted locations.*

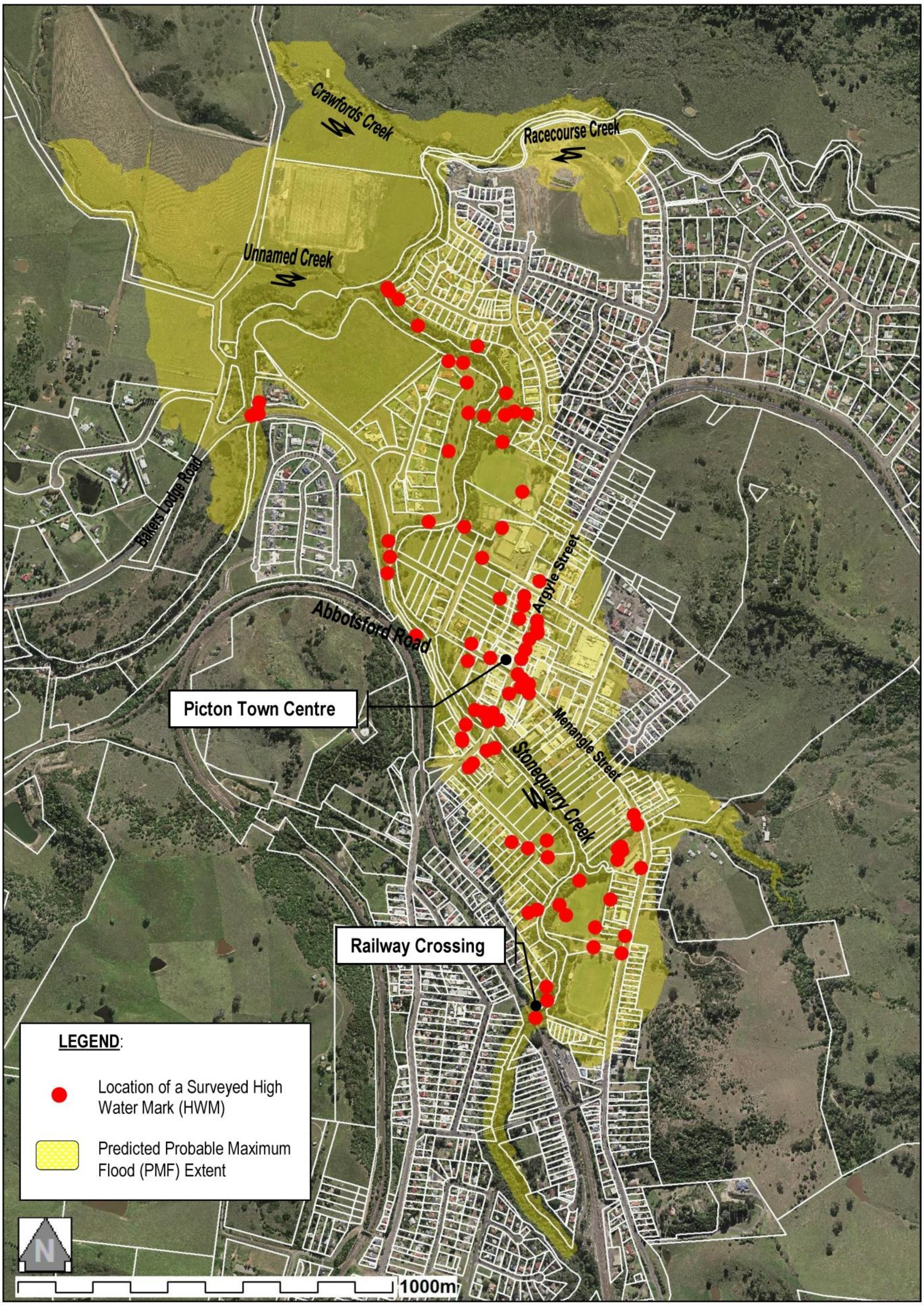


Plate 1 Location of Collected June 2016 High Water Marks (*HWMs*)

As shown in **Table 5**, the June 2016 event led to flood levels throughout Picton that were higher than those recorded for the 1969 flood at all locations. The difference in flood levels varies substantially with a range of 1.46 metres (*refer Picton Plaza*) to 0.52 metres (*refer Residence opposite Showground*).

A comparison between the recorded 1969 and 2016 flood levels to flood model predictions indicates that the historic events are in the order of a 2% Annual Exceedance Probability (*AEP*) event and rarer than a 1% AEP event, respectively.

3.3.3 Streamflow Data

The nearest river level gauge on Stonequarry Creek is located approximately 950 metres downstream (*to the south*) of Argyle Street and a short distance upstream of the Railway Crossing (*refer* **Figure 8**). The gauge is operated by the NSW Office of Water with a Gauge Number of 212053.

The gauge was commissioned on 4th December 1990 and has continuously recorded rainfall and river levels since that date. A rating curve has also been derived for the site to estimate discharges (*ML/day*) based on recorded gauge levels. The gauge was operational during the June 2016 event.

The magnitude of the June 2016 event relative to floods recorded since December 1990 is shown in **Plate 2** (*on the following page*) in terms of the gauge level reached (*Level, metres*) and the corresponding discharge (*ML/day*). The June 2016 flood resulted in a peak flood level that reached twice the gauge height of any flood over the preceding 25 years. The average daily flow during the June 2016 event was more than 5 times the average daily flow over this period.

3.3.4 Rainfall Data

Several rainfall gauges are located within or on the periphery of the study area and the Stonequarry Creek catchment. However, only one of these is a pluviometer. This It is operated by the NSW Office of Water and is located a short distance upstream of the Railway Crossing (*Gauge No. 212503*).

No other pluviometers are located within the Stonequarry Creek catchment, although Lake Nerrigorang (NOW Gauge No. 212063) and Thurns Road (NOW Gauge No. 568296), are close to the western and eastern catchment boundaries, respectively.

One daily-read rainfall gauge is located in Picton at the Council Depot (*BOM Gauge No. 68052*).

The locations of all available rainfall gauges relative to the Stonequarry Creek catchment are shown on **Figure 8**.

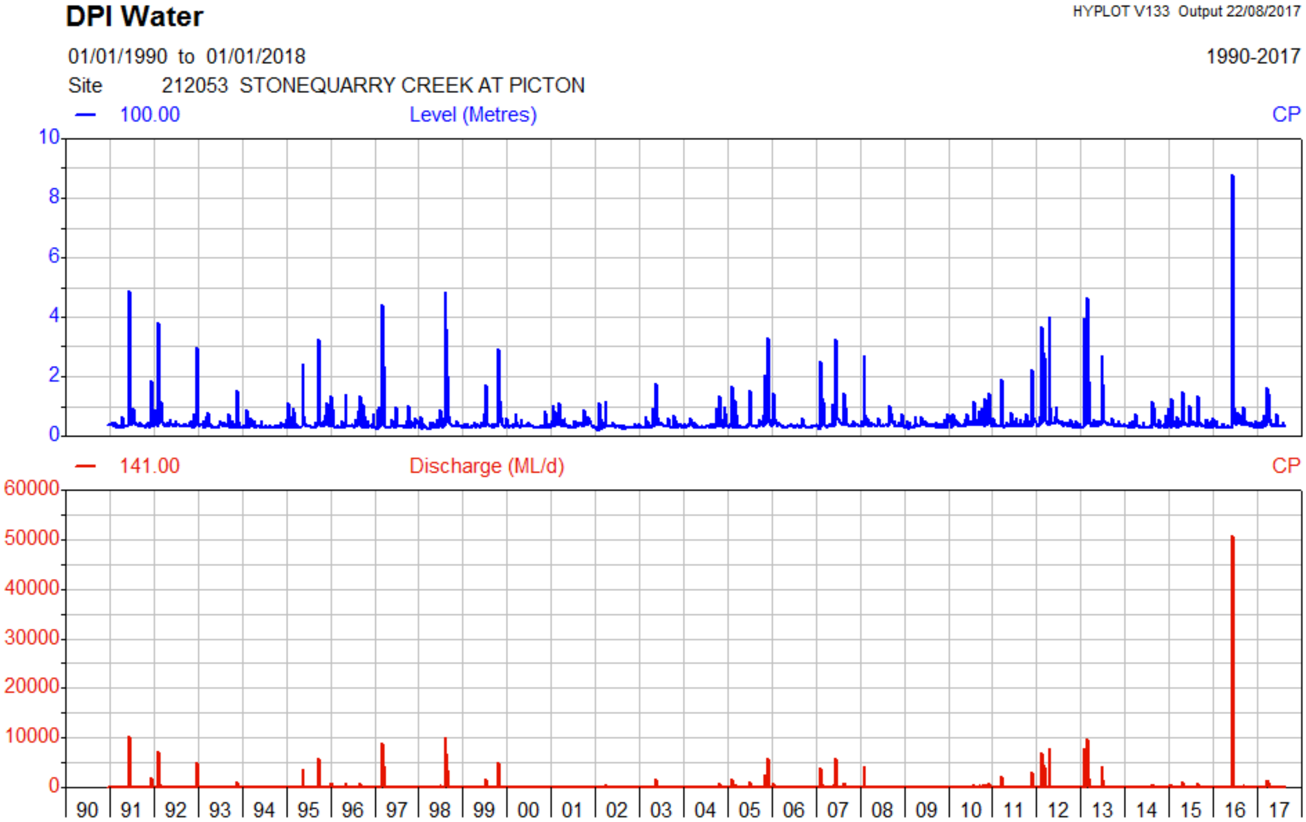


Plate 2 Recorded river level and discharge data for the Stonequarry Creek at Picton Gauge (Gauge No. 212503) operated by the NSW Office of Water

Source: http://realtimedata.water.nsw.gov.au

4 HYDROLOGIC MODELLING

4.1 GENERAL

The hydrology adopted for this study was largely based on the hydrologic modelling completed in 2011 as part of the report titled, '*Stonequarry Creek – 2D Modelling and Climate Change Assessment*' (*WorleyParsons, 2011*). As discussed in **Section 3.2.3**, the modelling completed in 2011 included an update of the RAFTS hydrologic model that was developed for the '*Picton Flood Study*' (*1989*).

The hydrologic modelling for this study is based on the previous RAFTS (*Runoff Analysis and Flow Training Simulation*) hydrologic modelling that was developed by the Department of Water Resources for the *'Picton Flood Study'* (*1989*). As part of this study, the RAFTS model of the Stonequarry Creek catchment has been updated to Version 7.00 (*2008*) XP-RAFTS.

XP-RAFTS can be used to develop a deterministic runoff routing model that simulates catchment runoff processes by incorporating a number of common catchment parameters into its calculation procedures. It is recognised in *'Australian Rainfall and Runoff – A guide to Flood Estimation'* (*1987*) (*ARR 87 herein*), as one of the available tools for use in flood routing within Australian catchments.

XP-RAFTS has the following attributes:

- it can account for spatial and temporal variations in storm rainfalls across a catchment;
- it can accommodate variations in catchment characteristics;
- it can be used to estimate discharge hydrographs at any location within a catchment; and,
- it has been widely used across eastern NSW and therefore, where suitable calibration data is not available, the results from modelling of other similar catchments can be used as a guide in the determination of model parameters.

Design storm conditions for this study were based on rainfall intensities and temporal patterns derived using standard procedures outlined in ARR 87. The design storm rainfall data was generated by applying the principles of rainfall intensity estimation described in Chapter 2 of ARR 87.

A new edition of '*Australian Rainfall and Runoff*' was released in December 2016 and revised IFD Data was also made available at this time by the Bureau of Meteorology. As the flood study was essentially completed prior to the new release of ARR, including the hydrologic and hydraulic modelling runs based on ARR 87, Wollondilly Shire Council decided that it was not necessary to revisit the modelling for the flood study at this time. Considerations of the revised ARR 16 will need to be carried out in future studies.

4.1.1 RAFTS Model Developed for 1989 Flood Study

A RAFTS hydrologic model of the Stonequarry Creek catchment and its tributaries was developed as part of the 1989 Flood Study. The model includes the entire catchment of

Stonequarry Creek downstream to the Main Southern Railway Viaduct crossing, which is located downstream of Picton (*refer* **Figure 2**).

The Stonequarry Creek catchment was delineated into 30 sub-catchments covering a total catchment area of 84 km^2 upstream from the Main Southern Railway Viaduct.

Intensity-Frequency-Duration (*IFD*) was obtained from the Bureau of Meteorology and used for the estimation of design rainfall intensities. The adopted rainfall inputs are listed in **Table 6** for the 5%, 2% and 1% AEP floods.

Table 6 RAINFALL INPUTS ADOPTED FOR THE 1989 RAFTS MODEL

STORM DURATION (*hours*)	RAINFALL INTENSITY (*mm/hr*)		
	5% AEP	2% AEP	1% AEP
1	51.4	60.6	67.7
2	34.2	40.3	44.9
3	26.8	31.5	35.2
6	17.7	20.7	23.1
12	11.7	13.8	15.4
18	9.24	10.9	12.2
24	7.81	9.27	10.4
48	5.11	6.17	6.98
72	3.88	4.73	5.37

Due to the absence of any stream flow data the hydrologic model could not be calibrated to recorded data. As a result, particular care was taken in the selection of appropriate initial and continuing loss rates and the storage delay co-efficient 'B_X'.

Following consideration of procedures including Cordery and Webb (*1974*) and Laurenson and Pilgrim (*1963*), an initial loss rate of 15 mm and a continuing loss rate of 1.5 mm/hr was adopted.

The peak discharges for each sub-catchment for the 5%, 2% and 1% AEP floods were determined using the RAFTS model described above. The full range of design storms between 1 and 72 hours were simulated (*refer* **Table 6**) and the results analysed to determine the critical storm duration for Picton. This was determined to be 6 hours and therefore the 6 hour storm was adopted as the design storm.

4.1.2 RAFTS Model Adopted for this Study

As discussed in **Section 3.2.3**, the RAFTS model of the Stonequarry Creek catchment that was developed for the 1989 Flood Study was updated in 2006 and 2011 as part of previous investigations completed by WorleyParsons. The primary modifications that were incorporated into the hydrologic model are as follows:

- The model was updated to a recent version of RAFTS (*Version 7.0, 2008*);
- The model was updated to present-day catchment conditions based on a review of aerial photography. Where increased urbanisation was evident the percent imperviousness was increased for the respective sub-catchment. Sub-catchments 2.00, 2.01, 6.04, 1.09 and 1.10 were all updated to reflect a higher proportion of impervious area.
- The critical duration was determined to be 9 hours (*not 6 hours*) following further review of peak discharges derived from further simulation of the various storm durations.
- Separate infiltration loss rates were incorporated for urban areas with initial loss of 2.5 mm and continuing losses of 0.5 mm/hr adopted.
- IFD parameters were reviewed and updated to be more catchment centric (*refer* **Table B1** *of* **Appendix B**).

No further changes were made to the 2011 RAFTS model as part of this study. A summary of the adopted sub-catchment parameters is provided in **Table B2** of **Appendix B**.

4.2 COMPARISON TO THE ORIGINAL 1989 RAFTS MODEL

The hydrologic model was validated against the original RAFTS model developed for the 1989 Flood Study and against recorded discharge data for the June 2016 event. Validation to the June 2016 event was only possible late in the Flood Study and following completion of all hydrologic model updates and simulations.

4.2.1 Original 1989 RAFTS Model

A comparison of the updated XP-RAFTS modelling results was made with the peak discharges produced by the original RAFTS model developed for the 1989 Flood Study (*refer* **Table 7**).

The comparison has been undertaken at each of the upstream and downstream limits of the study area for the 5% and 1% AEP floods. Peak discharges have been extracted from Table 5.2 of the 1989 Flood Study.

As shown in **Table 7**, the peak discharges predicted by the updated RAFTS hydrologic model are generally 10 to 30% higher than those predicted by the 1989 RAFTS model.

Differences of this magnitude are not surprising given the numerous model updates that were incorporated in 2006 and 2011 to update the hydrologic model according to current catchment conditions. Perhaps most notable of these updates are the change in critical duration from 6 hours to 9 hours, updated IFD parameters and the increase in urbanised areas; i.e., increased impervious areas with lower infiltration rates.

Table 7 COMPARISON WITH PEAK DISCHARGES FROM THE 1989 FLOOD STUDY

TRIBUTARY	XP-RAFTS MODEL NODE[1]	PEAK DISCHARGE[2] (m^3/s) 1% AEP			5% AEP		
		1989 Flood Study	RMA-2 (2014)	DIFF	1989 Flood Study	RMA-2 (2014)	DIFF
Stonequarry Creek (*Inflow*)	1.06	273	305	**12%**	194	230	**19%**
Racecourse Creek (*Inflow*)	6.04	99	117	**18%**	67	85	**27%**
Crawfords Creek (*Inflow*)	5.01	58	68	**17%**	40	51	**28%**
Unnamed Creek (*Inflow*)	4.02	48	60	**25%**	33	44	**33%**
Downstream Extent of Study Area	1.10	494	574	**16%**	345	431	**25%**

1. *For node and catchment locations refer to* ***Figure 2***
2. *Peak discharges adopted in the 1989 Flood Study taken from Table 5.2 of that Report*
3. *Peak discharges listed do not necessarily occur simultaneously*

A basic Rational Method calculation was completed for the 1% AEP flood as a check on the peak flows. The peak flow at the Stonequarry Creek inflow was calculated to be 244 m^3/s, which is 20% lower than the corresponding peak discharge derived using the latest XP-RAFTS model. A similar calculation at the downstream end of the study area provided a peak 1% AEP flood discharge of 403 m^3/s, which is about 40% lower than the corresponding discharge derived from the XP-RAFTS model.

It is not uncommon for the Rational Method to provide lower peak flows than detailed hydrologic modelling due to the Rational Method not accounting for the urbanised portions of the catchment which lead to increased runoff.

4.3 HYDROLOGIC MODEL VALIDATION TO THE JUNE 2016 EVENT

The updated XP-RAFTS model was validated to the June 2016 historic event based on comparison to recorded discharges at the Picton Gauge (*NOW Gauge 212053, refer* **Section 3.3.3**).

The following sections detail the findings of the XP-RAFTS model validation including discussion on the severity of the June 2016 event and the recorded discharge and rainfall data on which the validation was based.

4.3.1 June 2016 Event Overview

During the first week of June 2016 an upper level trough developed over central and eastern Australia along with an accompanying low pressure surface trough. The system intensified on Friday 3rd June and moved across south-east Queensland bringing with it persistent rainfall and high winds.

Early on Sunday 5th June 2016, the system moved off the coast and developed into an East Coast Low causing heavy rain, strong winds and large waves along the NSW coast. The low pressure system brought widespread heavy rainfall to the northern coast and ranges, before the main rainfall focus shifted southwards to impact the south coast and ranges of NSW. Rain persisted through both Saturday and Sunday and many locations reported their wettest June on record in the first week of the month.

The major flooding that occurred in Picton resulted in damage to commercial and residential properties. Properties throughout the study area, including many along Argyle Street in the centre of town, experienced significant inundation with depths in excess of 1.5 metres recorded. Trees and other in-bank vegetation were up-rooted during the flood. This debris was conveyed downstream reflecting the significant velocity of floodwaters carried along Stonequarry Creek and its tributaries.

4.3.2 June 2016 Rainfall Data

Rainfall data was obtained from the Bureau of Meteorology (*BOM*) and NSW Office of Water (*NOW*) from a range of pluviometers and daily read rain gauges. Data from the following rainfall gauges was used in the model validation:

- Picton Council Depot Gauge (*BOM Gauge No. 68052*) – *daily read gauge*
- Stonequarry Creek at Picton (*NOW Gauge No. 212053*) – *pluviometer*
- Thurns Road TBRG (*NOW Gauge No. 568296*) – *pluviometer*
- Nerrigorang at Thirlmere (*NOW Gauge No. 212063*) – *pluviometer*

The rainfall data was compiled and is presented in **Plate 3**. The pluviograph data shows a consistent pattern of rainfall in the area. Rain began on the morning of Saturday 4th June and continued until about 20:00 on Sunday 5th June.

The total rainfall recorded at the daily-read gauge at the Picton Council Depot appears low compared to the rainfall recorded for the corresponding period at surrounding gauges. There is potential that the gauge may have overflowed based on anecdotal reports that it had not been emptied in the two days prior. Therefore, it is possible that the total rainfall recorded by this gauge is underestimating the rainfall that fell at Picton during the June 2016 event.

The gauge at Stonequarry Creek recorded the greatest depth of rainfall with 334 mm recorded over the duration of the event (*36 hours*). The most intense rainfall occurred over a 9 hour duration from 10:00 to 19:00 on the 5th June 2017. For all pluvios in the vicinity of Picton, rainfall totals in the order of 150 mm were recorded over this period. Based on the BOM's Intensity-Frequency-Distribution (*IFD*) data, the rainfall exceeded a 1% Annual Exceedance Probability (AEP) event over a 12 and 24 hour storm duration.

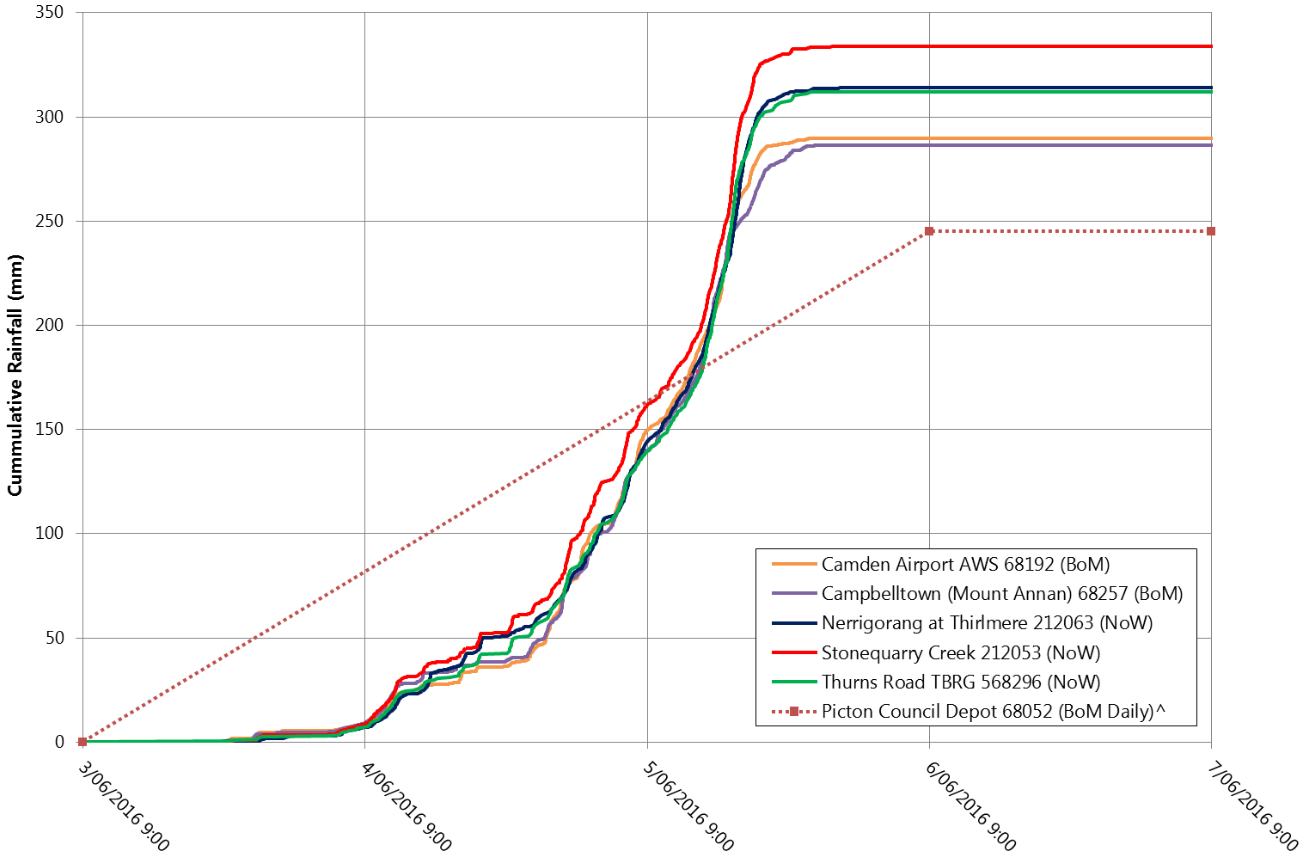


Plate 3 Cumulative Rainfall Data

^ *The gauge at Picton Council Depot is a daily read gauge usually recorded at 09:00 every morning. The gauge was not read on 4th and 5th June and the data for 6th June is accumulated over the 2 days prior. It is possible that the reading at this gauge is an underestimate of the actual total rainfall as it may have filled to capacity and overflowed.*

4.3.3 June 2016 Level and Discharge Data

The nearest river level gauge on Stonequarry Creek is located approximately 950 m downstream (*to the south*) of Argyle Street and a short distance upstream of the Railway Crossing. Recorded river level and flow data for this gauge was obtained from the NSW Office of Water.

River level and rainfall data for the June 2016 event as recorded by the Stonequarry Creek Gauge (*NOW Gauge No. 212053*) is presented in **Plate 4**.

The gauge data shows the creek began to respond at about 10:00 on 4th June with floodwaters rising relatively slowly for the first 15 hours. From the early hours of Sunday 5th June water levels in the creek began to rise more rapidly at about 0.3 m per hour. From around 14:00 on Sunday 5th June, as the rainfall intensified, water levels rose even more rapidly at a rate of 1.3 m/hr to the peak recorded level of 8.8 m which was recorded at 18:30. This equates to an elevation of about 156.6 mAHD.

The rainfall began to ease from around 19:00 and water levels dropped rapidly over the next 12 hours.

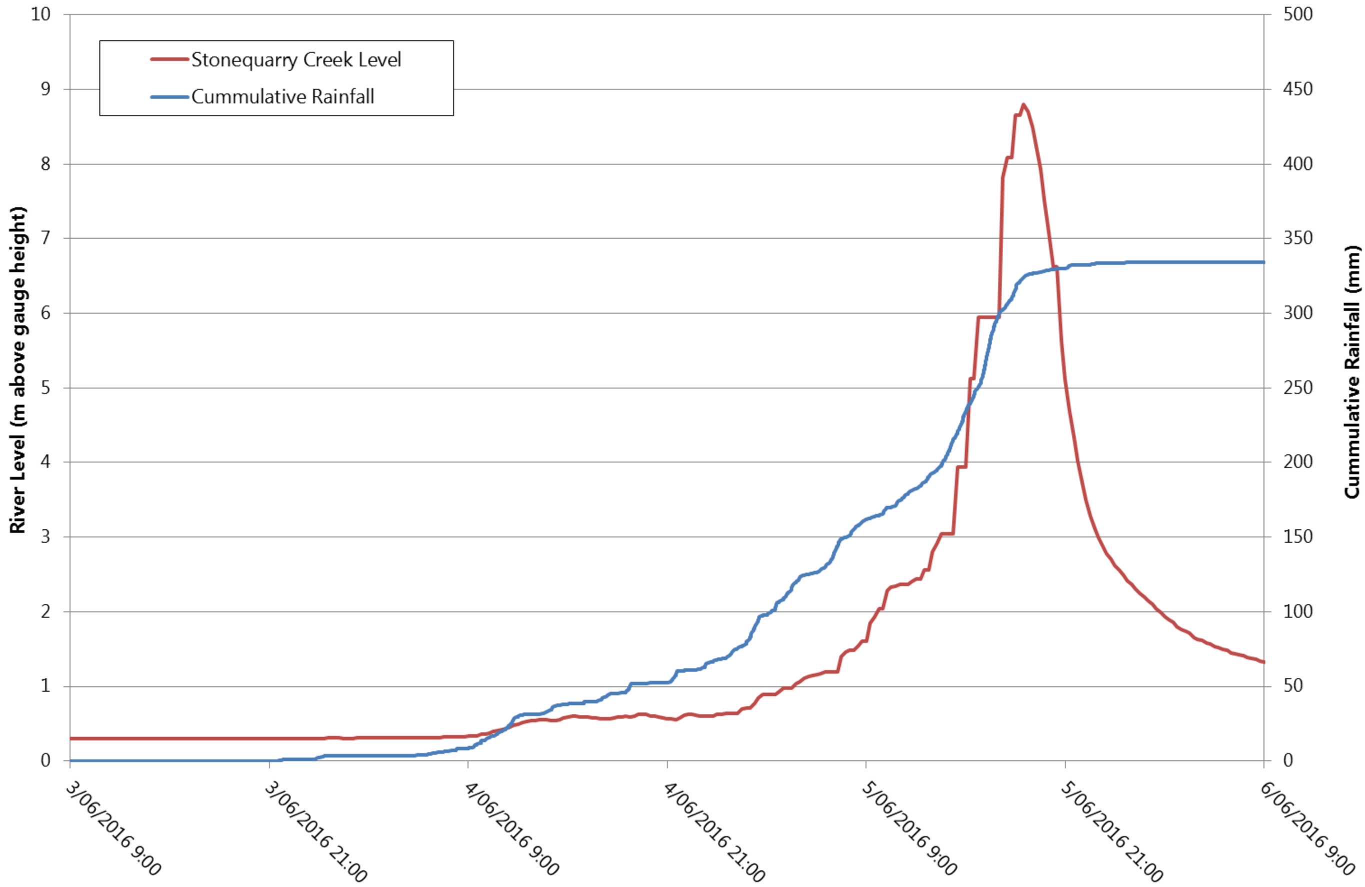


Plate 4 Recorded river levels and rainfall data at Stonequarry Creek Gauge

A rating curve has been developed for the river level gauge by NOW to allow conversion of recorded flood levels to estimates of flood discharge. The discharge hydrograph for the June 2016 event was exported directly from the NOW website at 15 minute intervals. The rating curve indicates that flows along Stonequarry Creek peaked at approximately 575 m^3/s during the June 2016 event. A plot of recorded levels and corresponding flows is presented in **Plate 5** on the following page.

4.3.4 Validation Results

Discharge hydrographs can be estimated for the June 2016 event using the XP-RAFTS hydrologic model and the recorded rainfall data. As there are multiple rainfall gauges within the catchment rainfall was applied to each sub-catchment based on its proximity to a rainfall gauge. A figure showing the distribution of catchments relative to the applied rainfall data is shown in **Figure 9**.

As shown, only three of the rainfall gauges were adopted to represent the June 2016 rainfall event across the study area. This is based on the proximity of the gauges relative to the catchment and their spread across the centre and perimeters of the catchment. Analysis of the recorded rainfall for each of the adopted gauges also indicates that the recorded rainfall intensities (*mm/hr*) and total cumulative rainfall (*mm*) was similar for each. It is therefore unlikely that the modelling would be sensitive to any variation in the application of gauge data to the catchments (*refer* **Figure 9**).

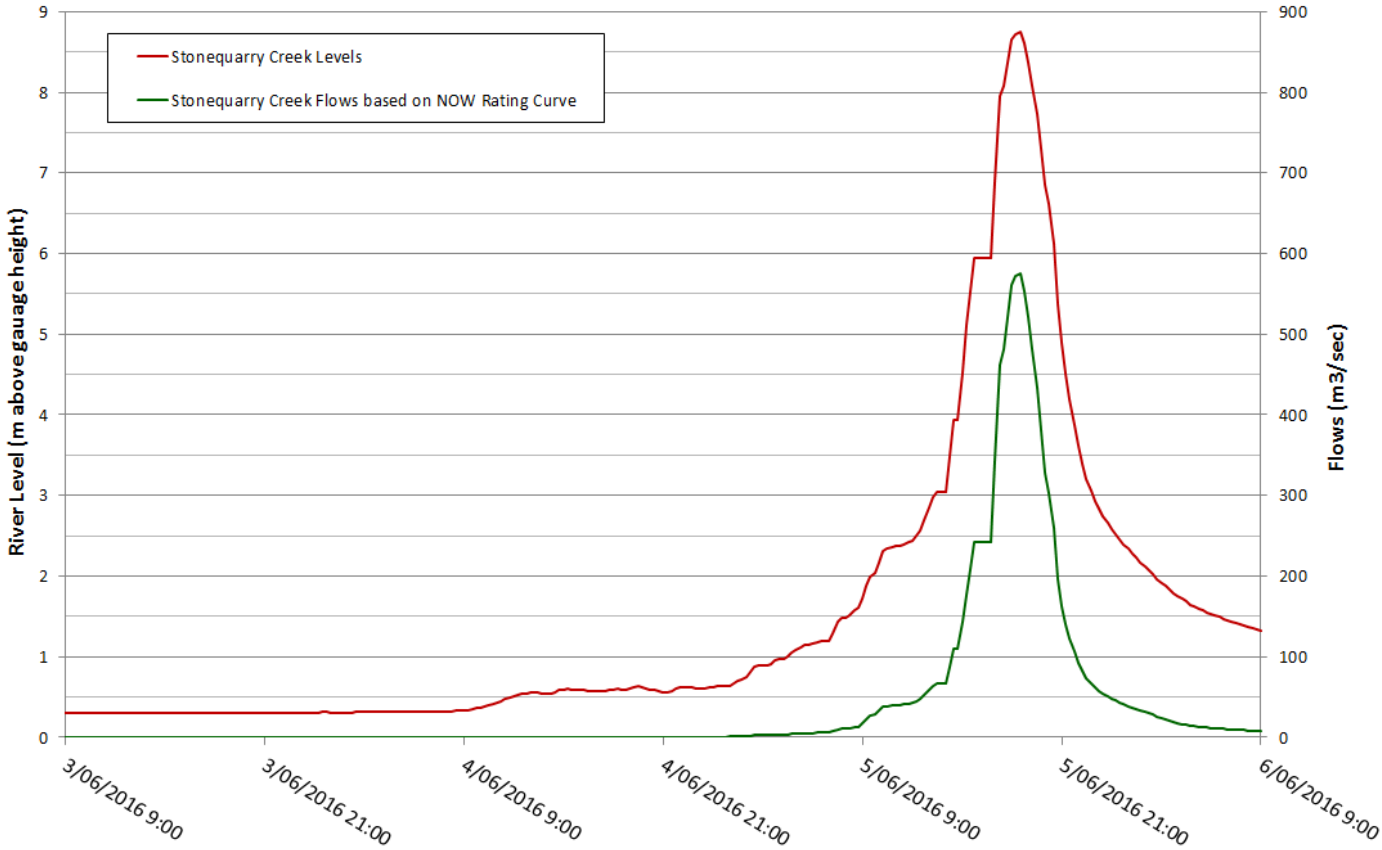


Plate 5 Recorded river levels and corresponding flows at Stonequarry Creek Gauge

Initial XP-RAFTS simulations of the June 2016 rainfall event were undertaken without any modification to the XP-RAFTS model that was developed for the Updated Flood Study. That is, all catchment and routing parameters such as roughness, slope and storage coefficients and the initial and continuing losses were left unchanged.

The flow hydrograph predicted by the base XP-RAFTS Flood Study model at the downstream limit of the model, which coincides with the Railway Crossing and the river level gauge (*NOW Gauge No. 212053*), is shown in **Plate 6**. The flow hydrograph determined by NOW is superimposed for comparison.

The base XP-RAFTS Flood Study model generated a peak flow at the gauge location that is within 20 m^3/s (*4%*) of the peak flow recorded during the flood event. The timing of the peak flow determined from the modelling is within 60 minutes of the time of the recorded peak.

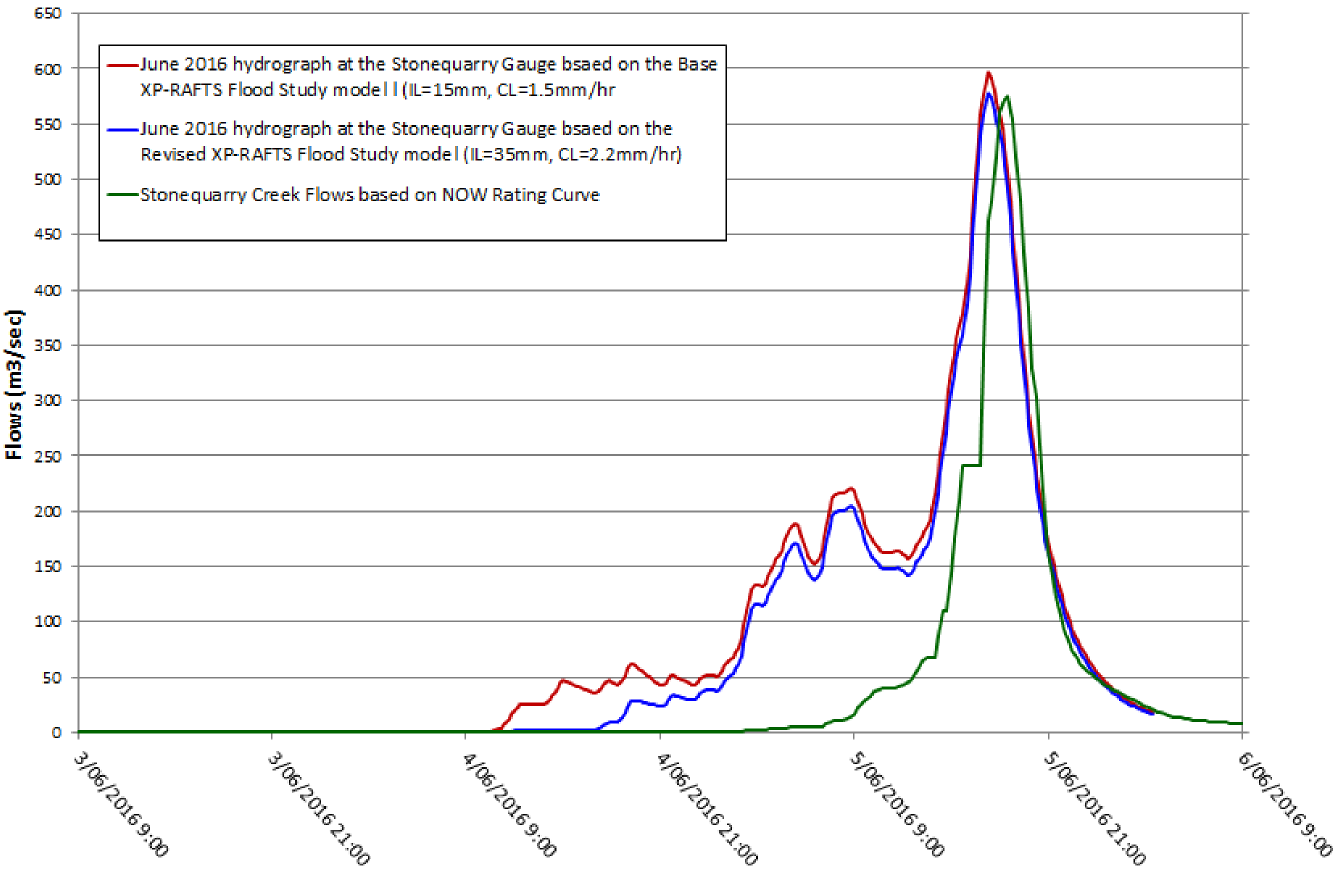


Plate 6 Comparison of XP-RAFTS hydrographs to the recorded/calculated flows at the Stonequarry Gauge

However, the flood hydrograph generated for the June 2016 event from the base XP-RAFTS Flood Study model does not reliably replicate the early stages of the flood as shown by the poor correlation evident in **Plate 6** between model generated and recorded flows over the duration of the 4th June and into the early part of 5th June. As shown in **Plate 6**, the base XP-RAFTS model predicts that flows would have risen along Stonequarry Creek much sooner and quicker indicating a faster response time for the catchment. Perhaps more importantly, the "fit" indicates that there is a poor correlation between simulated and recorded flood volume at this location. This suggests that while a reasonable 'fit' to peak discharge might have been achieved, the poor match to the shape of the hydrograph during the early stages of the flood suggests that the initial losses adopted in the XP-RAFTS Flood Study model are not representative of those that existed during the June 2016.

To try achieve a better '*fit*' between the simulated and recorded flows the XP-RAFTS Flood Study model was tested with varying values of initial and continuing rainfall losses for pervious catchments. These parameters are most commonly adjusted between historic events to better reflect antecedent rainfall conditions; for example, the weeks or months in the lead up to an event may have been dryer or wetter than the 'typical' catchment conditions adopted for design flood simulations, and therefore may not be replicated in a straight application of the model.

Review of **Plate 6** suggests that the initial and continuing losses adopted in the base XP-RAFTS Flood Study model are likely to be low and not representative of the June 2016 event. Increasing the initial (*mm*) and continuing loss rates (*mm/hr*) would act to slow the response time of the catchment while also reducing peak flow rates.

The initial and continuing losses adopted for the base XP-RAFTS Flood Study model are 15 mm and 1.5 mm/hr, respectively. Australian Rainfall & Runoff (*1987*) recommends loss rates ranging between 0 to 35 mm and 1 to 4 mm/hr for initial and continuing losses, respectively. Hence, the values adopted in the XP-RAFTS Flood Study model are in the lower bands of these ranges.

A sensitivity assessment was undertaken to test the impact of modified initial and continuing losses on the flow hydrograph at the Railway Viaduct. The assessment considered rainfall records for the three nearest gauges from preceding months which showed that there was below average rainfall over the 4 months prior to the event (*refer* **Appendix C**). The Council Depot gauge (*BOM Gauge No. 068052*) for example recorded 71 mm of rainfall over the preceding 4 months compared to the average rainfall for the period of 214 mm.

This lower than average rainfall would have resulted in particularly dry catchment conditions, suggesting greater than normal capacity for the catchment to 'absorb' a proportion of the early rainfall during the storm. This data supports the adoption of a higher than average initial and continuing loss rate for XP-RAFTS model simulations of the June 2016 event; i.e., values higher than currently adopted in the XP-RAFTS model for the modelling of design events.

The sensitivity analysis determined that increased initial and continuing loss values of 35 mm and 2.2 mm/hr, respectively, generate a simulate flood hydrograph that is a better 'fit' to the recorded flood hydrograph at the gauge. These revised loss values provided a closer match to the peak flow rate recorded at the Railway Viaduct. In that regard, the revised losses led to a predicted peak of 578 m^3/s compared to a recorded peak flow of 575 m^3/s.

The flow hydrograph determined using these revised parameters is superimposed on **Plate 6**.

The increased initial losses have acted to delay the rise in flows by approximately 8 hours. Although this has led to a closer match to the gauge, the rising limb of the two hydrographs are still not aligned, with the revised XP-RAFTS hydrograph still rising considerably sooner.

Although the simulated hydrograph could further be delayed by increasing the initial loss rates, sensitivity modelling showed initial losses would need to be increased to between 80 mm and 100 mm to achieve a reasonable match. This is considered to represent a very high estimate of initial losses, even with the below average rainfall preceding the event, which would be difficult to justify without detailed investigation. This suggests there may have been event-specific phenomenon unaccounted for, or potentially an error with the NOW Rating Curve for low gauge levels.

5 FLOOD MODELLING

The hydraulic modelling for this study is based on a previous RMA-2 two-dimensional flood model that was developed by Patterson Britton & Partners (*now WorleyParsons*) as part of an engagement to update the flood modelling that had been developed by DWR as part of the 1989 Flood Study. RMA-2 is a fully two-dimensional finite element modelling package developed by Resource Management Associates and Prof. Ian King from the University of New South Wales. It was chosen to replace the HEC-2 model over other hydrodynamic modelling software because it has the following attributes:

(i) RMA-2 is a fully two dimensional, dynamic, finite element model. Hence, it allows for overland flow and storage to be modelled within the floodplain and ensures that the interaction between mainstream and overbank flows is reliably simulated.

(ii) RMA-2 uses finite element methods to solve 2D depth averaged equations for turbulent energy losses, friction losses and horizontal momentum transfer. Therefore, it offers significant benefits over the more traditional finite difference techniques.

(iii) RMA-2 uses a variable grid geometry employing elements with irregular and curved boundaries which can be modified as required without the need for regeneration of the entire grid. This enables topographic features or hydraulic controls of any shape to be reliably represented within the model.

(iv) RMA-2 permits the simulation of floodplain elements that wet and dry during the analysis period.

A major advantage of using RMA-2 over traditional finite difference models is that the model resolution can be varied to cover regions of particular interest, or areas that have the potential to impact on flood behaviour; e.g., around urban areas.

RMA-2 also provides the flexibility to allow Council to investigate options that could be implemented to reduce flood damages and to assess future development scenarios. It is relatively simple to adjust the model network to incorporate structural flood mitigation works, such as channel modifications or levees. Hence, it is appropriately suited to being adapted to support any revisiting of the Floodplain Risk Management Study in accordance with the process outlined in **Section 2**.

The RMA-2 model for Stonequarry Creek was originally developed in 2005, after which it underwent further updates in 2009 to incorporate additional survey data that became available. The updates in 2009 involved extension to the RMA-2 model upstream beyond the upstream limits of the 1989 HEC-2 model.

As part of this study, the RMA-2 model of Stonequarry Creek and its tributaries has been further updated to incorporate the LiDAR survey that became available to Wollondilly Shire Council in 2012. In addition, the RMA-2 model and its parameters have been updated to be compatible with the latest Version of RMA-2 (*Version 85S*) developed by Prof. Ian King in 2013.

The update to the flood model involved the following:

- Preparation of a Digital Elevation Model (DEM) using the LiDAR data provided by Council.
- Refinement of the existing model mesh by picking-up the improved channel definition of Stonequarry Creek and its tributaries, followed by the refinement of floodplain areas, major roadways and building footprints.
- Validation of the flood model to historic floods and comparison with the 1989 Flood Study results.

5.1 DIGITAL ELEVATION MODEL (DEM)

Light Detection and Ranging (*LiDAR*) data is available for the entire study area. This LiDAR data is a very large data set that contain thousands of points that define existing ground surface elevations. The latest available data was collected by AAM Pty Ltd in August 2012 and has a nominal vertical accuracy of 0.15 metres across un-vegetated areas.

The LiDAR data set was processed to form a Digital Elevation Model (*DEM*) of the study area. The DEM is required as a base for development of the two-dimensional hydrodynamic flood model.

The extent of the available LiDAR data and the DEM that was created from it is shown in **Figure 4**.

5.2 MODEL NETWORK MESH

RMA-2 is a finite element model that represents topographic features via a network of geometric shapes (*i.e., triangles, squares and rectangles*). The geometric shapes are joined together to form a finite element mesh that covers the entire study area.

The existing RMA-2 model developed between 2005 and 2011 had been based on topographic data consisting largely of localised survey data sets, 2 metre surface contours and HEC-2 model cross-sections (*refer* **Section 3.3**). Because this data was relatively 'coarse' it followed that the RMA-2 model was developed with a relatively large network grid, capturing limited detail across some areas of the floodplain.

The RMA-2 model was updated to include the additional floodplain detail that had been captured by the more recently acquired LiDAR data. This required a detailed review of the floodplain features to identify where the RMA-2 network needed to be refined and/or modified to 'pick-up' additional detail.

The model network was also refined in order to incorporate the outlines of all existing buildings within the floodplain. This was completed using building outline polygons that had been collected by AAM Pty Ltd in 2012, in conjunction with a review of aerial photography. This allowed buildings to be '*blocked-out*' of the model to simulate the significant obstructions they impose on floodwaters.

At the conclusion of this exercise the updated RMA-2 model comprised a total of 27,500 nodes; compared to 4,200 in the original model. This substantial increase in model nodes reflects the level of additional topographic detail that has been incorporated into the RMA-2 model. A comparison of the original and updated RMA-2 model networks is provided in **Figure 10**.

As shown in **Figure 10**, the upstream limits of the model were not modified as part of the model updates. However, the downstream extent of the model was extended and updated to more reliably reflect the floodplain downstream of the Main Southern Railway Viaduct. This downstream extension had previously not been possible due to the lack of available topographic data.

The updated RMA-2 network is shown in greater detail in **Figure 11**. The elevations within the creek system and across the floodplain have been assigned based on the DEM developed for the study.

The channel and floodplain roughness parameter values were assigned to the RMA-2 model based on analysis of recent aerial photography and oblique photography of Stonequarry Creek and its channel and overbank vegetation. A review of the model network was undertaken as part of this process in order to identify locations where the network could be refined to better delineate significant differences in floodplain roughness.

The adopted hydrodynamic model roughness values are listed in **Table 8** for each element type. The material type distribution across the entire RMA-2 model network is shown in detail on **Figures 12** to **14**.

Table 8 ADOPTED RMA-2 ROUGHNESS VALUES

RMA-2 ELEMENT TYPE	DESCRIPTION	ROUGHNESS PARAMETER VALUE
1	Creek channel clear of vegetation	0.030
2	Creek channel with moderate vegetation	0.040
3	Heavily vegetated creek channel	0.060
4	Grassed floodplain	0.040
5	Floodplain with sparse trees	0.060
6	Floodplain with moderate coverage of trees	0.075
7	Floodplain with dense trees	0.090
8	Bridge crossings	0.100
9	Roadway	0.030
10	Industrial Development	0.065
11	Urban / Residential	0.040

Due to the limited availability of historic flood level, stream flow and/or rainfall data at the time of the development of the RMA-2 network it was not possible to calibrate the model to any historic floods. Notwithstanding, the adopted roughness values have been selected carefully and are all within acceptable ranges for the density and type of vegetation encountered within the Stonequarry Creek system.

The geometry of the major bridge crossings along Stonequarry Creek and its tributaries were defined in the model geometry according to the extents and elevations of key features such as embankments and approach and wingwall abutments. These bridge features were extracted from detailed design drawings and/or survey that had been made available at the study commencement. Where detailed information was not available bridge waterway openings were defined based on a combined analysis of the LiDAR data and available aerial photography.

Roughness parameters in the vicinity of the bridge undercroft and major culverts were increased to reflect the energy and friction losses that would be caused by the presence of bridge piers and the bridge deck (*for those cases where the bridge capacity was exceeded and the deck became submerged*).

This approach was adopted for all bridge crossings with the exception of the Main Southern Railway Viaduct, for which the outlines of the piers were included within the model network and blocked out individually. This approach was adopted for the Main Southern Railway Viaduct in recognition of the relatively large size of its piers and their locations/alignment within the Stonequarry Creek channel.

Comparisons of 2011 and 2014 RMA-2 topographic elevations along Stonequarry, Racecourse and Crawfords Creeks are presented in **Appendix D** as **Figures D1** to **D5**.

5.3 MODEL BOUNDARY CONDITIONS

5.3.1 Upstream Boundary Conditions

The upstream boundary conditions for the hydraulic model are provided by the discharge hydrographs generated from the XP-RAFTS hydrologic modelling of the upstream catchment.

The upstream boundary conditions correspond to the location of inflows into the creek system (*i.e., flows into Stonequarry, Racecourse, Crawfords and an unnamed creek*). The XP-RAFTS model nodes corresponding to these inflows are listed in **Table 9**. The locations of each of the XP-RAFTS model nodes are shown in **Figure 2**.

The locations of all upstream inflows into the RMA-2 model are shown on **Figure 11**.

Table 9 UPSTREAM BOUNDARY CONDITIONS FOR THE RMA-2 MODEL

TRIBUTARY	LOCATION (*refer* Figure 6)	RAFTS MODEL NODE (*refer* Figure 2)	1% AEP PEAK INFLOWS (m^3/s)	CRITICAL DURATION
Stonequarry Creek	300 metres upstream of Bakers Lodge Road	1.06	305	9 hours
Racecourse Creek	850 metres upstream of Confluence with Crawfords Creek	6.04	117	9 hours
Crawfords Creek	550 metres upstream of Confluence with Racecourse Creek	5.01	68	9 hours
Unnamed Creek	850 metres upstream of Evelyn Bridge crossing	4.02	60	9 hours

5.3.2 Downstream Boundary Conditions

Downstream boundary conditions must also be incorporated into the RMA-2 model. The downstream boundary condition is typically specified as a known time-varying water level or by a stage-discharge relationship.

The downstream boundary conditions for this study were determined based on consideration of those conditions previously adopted in the 1989 HEC-2 and 2011 RMA-2 models. In that regard, the HEC-2 model adopted a static water level at its downstream boundary corresponding to a level of 154.85 mAHD, while the RMA-2 model adopted a stage-discharge relationship determined using the 'normal' depth approach.

As discussed in the report titled, '*Stonequarry Creek – 2D Modelling and Climate Change Assessment*' (*2011*), the use of a stage-discharge relationship versus the static tailwater (*at 154.85 mAHD*) level resulted in a decrease in flood levels of between 0.3 to 0.6 metres as far upstream as the Stonequarry Bridge crossing. This comparison was conducted using the same HEC-2 model for each boundary condition scenario and therefore it was concluded that the static tailwater level adopted for the 1989 Flood Study was overly conservative.

The stage-discharge relationship determined using the 'normal' depth approach was adopted as the downstream boundary for the 2011 RMA-2 model. This relationship is shown in **Plate 7** and was applied to the most downstream HEC-2 cross-section location (*refer* **Figure 3**).

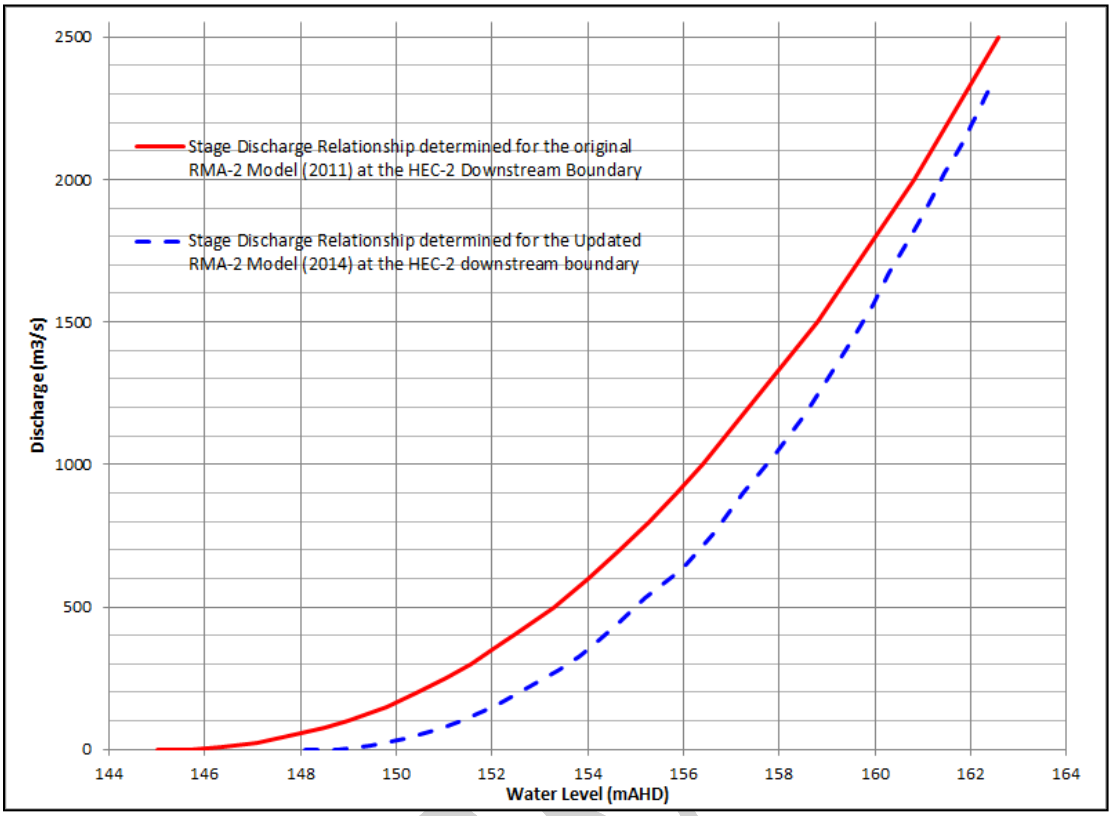


Plate 7 Comparison of Stage Discharge Relationships determined at the location of the 2011 Downstream Boundary

In recognition of the availability of the LiDAR data and its greater extent of coverage, the downstream model boundary was extended 700 metres downstream of the HEC-2 model boundary. This corresponds to a downstream boundary that is located approximately 400 metres downstream of the Prince Street Bridge crossing (*refer* **Figure 11**).

The stage-discharge relationship was revisited due to the change in boundary location. This involved the application of '*normal-depth*' calculations in using a channel slope extracted from the available LiDAR data. The revised stage-discharge relationship is provided in tabular form in **Table 10**.

For comparative purposes an updated relationship was extracted at the same location as the original stage-discharge relationship and is superimposed on **Plate 7**. The relationship has been extracted from the updated RMA-2 modelling results.

As shown in **Plate 7**, the updated RMA-2 model results have resulted in an upward shift in the stage-discharge relationship downstream of the Main Southern Railway Bridge. The increase is due to the change in topographic data combined with the downstream extension of the 2014 RMA-2 flood model.

Table 10 ADOPTED STAGE-DISCARGE RELATIONSHIP AT DOWNSTREAM BOUNDARY OF UPDATED RMA-2 MODEL (2014)

DISCHARGE (m^3/s)	WATER LEVEL (*mAHD*)
0	144.00
1	144.20
10	145.03
25	145.70
50	146.50
75	147.00
100	147.40
150	148.10
200	148.70
250	149.20
300	149.65
350	150.05
400	150.40
500	151.10
600	151.70
700	152.30
800	152.83
900	153.34
1,000	153.82
1,500	155.90
2,000	157.70
2,500	159.25
3,000	160.65
4,000	163.05

5.4 HYDRODYNAMIC MODEL VALIDATION – JUNE 2016 EVENT

As discussed, initial estimates of floodplain and river channel roughness parameters were based on aerial photograph analysis and field inspections. In order to validate the roughness parameters, it is ideal to calibrate the hydraulic model to historic flood events. Calibration involves the adjustment of model parameters within acceptable limits in order to match simulated flood levels with known historic flood levels.

The June 2016 flood event occurred after model development and the completion of all design event simulations. As a result, the model was not calibrated to the June 2016 event. The June 2016 event could however be used to validate the RMA-2 flood model by comparing peak flood levels predicted by the RMA-2 model to those recorded at the 76 available High Water Marks (*HWMs*) (*refer* **Figures 5** *to* **7** *discussed in* **Section 3.3.2**).

The following sections detail the findings of the RMA-2 validation against recorded flood levels for the June 2016 flood event. The adopted upstream and downstream boundary conditions are also discussed with reference to the input data used.

Further information detailing the severity of the June 2016 event is included in **Section 4.3.1**.

5.4.1 June 2016 Model Set-Up

In order to simulate the June 2016 flood using the RMA-2 model a reliable estimate is required for all upstream inflows and downstream flood levels. These inputs represent the upstream and downstream boundary conditions to the RMA-2 model.

Upstream Boundary Conditions

The inflow hydrographs for the June 2016 event at each model boundary location are shown in **Plate 8**.

Further discussion on the inflow hydrographs including the XP-RAFTS model input data (*rainfall*) used to generate them, and a comparison to recorded discharge data, is included in **Section 4.3**.

Downstream Boundary Condition

The downstream boundary condition for the RMA-2 model is based on a stage-discharge relationship determined using a 'normal depth' analysis (*refer* **Section 5.3.2**). The stage-discharge boundary allows water levels at the boundary to be updated within the model as the simulation progresses and flows increase and/or decrease.

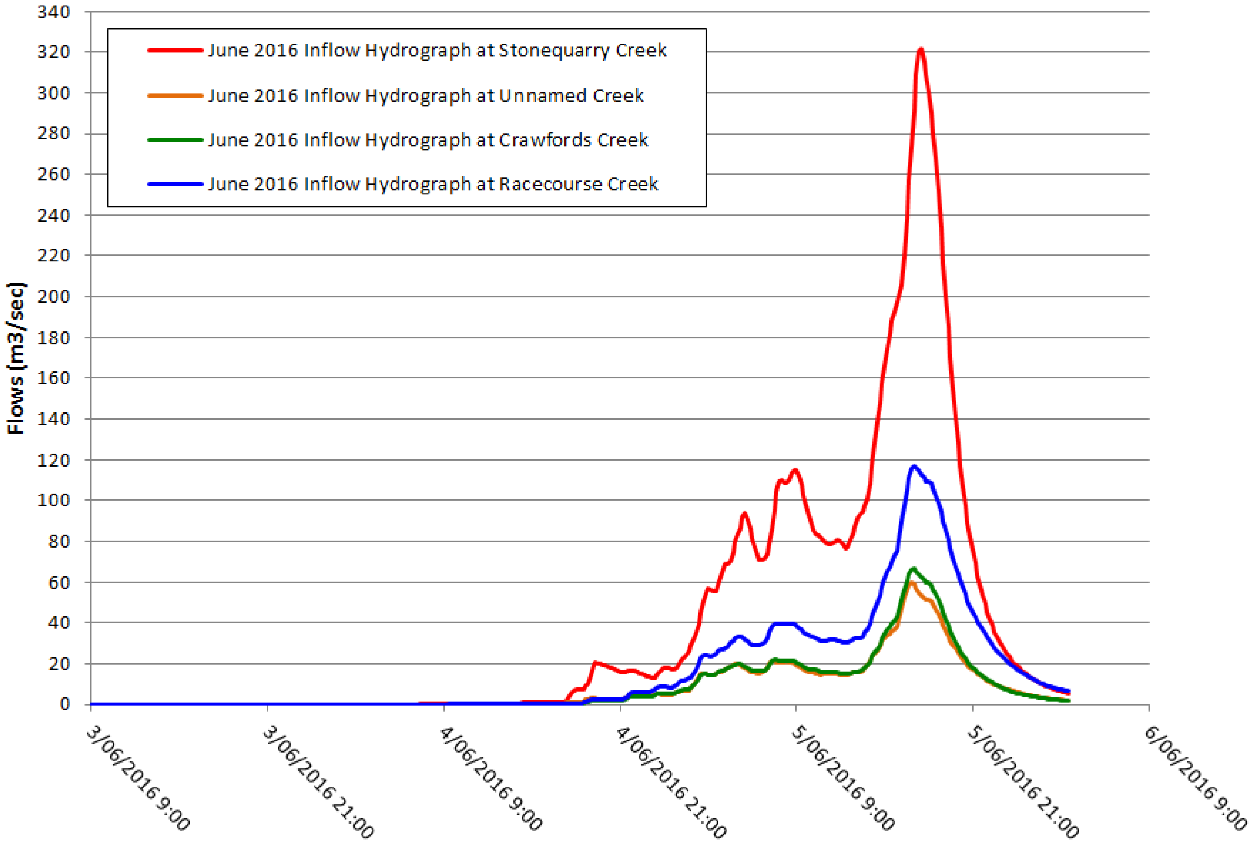


Plate 8 Adopted RMA-2 Inflow Hydrographs for the June 2016 Event

Model Network and Material Roughness

No changes were made to the RMA-2 model network or material roughness values and distribution as part of the June 2016 event validation. In that regard, the roughness values and distribution discussed in **Section 5.2** and presented in **Figures 12** to **14** were unchanged. Similarly, all buildings (*residential and commercial*) were completely 'blocked-out' of the model to simulate the significant obstructions they impose to floodwaters.

5.4.2 Comparison of Simulated and Recorded HWMs

The RMA-2 model was simulated with the boundary conditions discussed in **Section 5.4.1** and the inflow hydrographs shown in **Plate 8**.

In order to validate the model, the predicted flood level at the location of each HWM was extracted and recorded. This flood level was subsequently compared to the flood level recorded at the HWM and the difference noted.

The findings of this comparison are shown in **Figures 15**, **16** and **17**. The figures show the locations of each HWM and the calculated difference between modelled and recorded June 2016 flood levels.

Differences are shown to generally range between -0.05 and -0.20 metres, with the exception of scattered outliers.

Closer investigation of the outliers shows that in most cases there are inconsistencies between the levels for these outliers and the levels for recorded HWMs located immediately upstream or downstream. Other differences appear to be influenced by localised hydraulic effects, such as a loss of hydraulic efficiency due to debris build-up along fences or along the upstream side of bridges. These local and event specific occurrences are difficult to capture in hydraulic modelling unless event specific models and modelling parameters are adopted.

A statistical analysis of the flood level differences indicates that the RMA-2 model predicts flood levels to within an average of 0.18 metres and median of 0.145 metres when compared to all of the 76 recorded HWMs. This statistical assessment is broken down further in **Table 11**, providing the mean and median difference based on the HWMs included in each figure. This breakdown is beneficial as it provides an indication of the reliability of the model across the upper (upstream of the town), middle (*Picton Town Centre*) and lower (*downstream of the town*) model reaches.

Table 11 FINDINGS OF RMA-2 MODEL VALIDATION

	Figure 15 *Upstream Town*	**Figure 16** *Town Centre*	**Figure 17** *Downstream Town*	**All HWMs**
Number of HWMs	17	38	21	76
Mean Difference (m)	- 0.21	- 0.17	- 0.18	- 0.18
Median Difference (m)	- 0.19	- 0.15	- 0.13	- 0.145

Table 11 indicates that the simulated and recorded flood level differences are within acceptable ranges with the calculated mean and median differences only changing marginally between different sections of the Study Area. A mean difference of less than 0.2 metres for all figures is considered to represent a favourable validation. This indicates that the RMA-2 model generates peak flood levels across the Study Area that are in good agreement with the June 2016 HWM data and shows that the RMA-2 model is a reliable tool for the estimation of design flood characteristics. As a consequence, no event specific modifications were made to the adopted roughness values were considered to be warranted.

The mean and median differences shown in **Table 11** indicate a consistent trend that suggests the RMA-2 model may be under-predicting flood levels. This result was unexpected given the RMA-2 model had been found to predict flood levels that were already higher than those predicted in the 1989 Flood Study using the HEC-2 model. This is discussed further in **Section 6.3.3**.

6 DESIGN FLOOD ESTIMATION

6.1 GENERAL

Design floods are hypothetical floods that are commonly used for planning and floodplain risk management investigations. Design floods are based on statistical analysis of rainfall and flood records and are defined by their probability of occurrence. For example, a 1% AEP flood is the best estimate of a flood that will likely occur on average, once in every one hundred years.

It should be noted that there is no guarantee that the design 1% AEP flood will occur just once in a one hundred year period. It may occur more than once, or at no time at all in the one hundred year period. This is because design floods are based upon a statistical '*average*'.

The computer models described in **Sections 4** and **5** were used to derive design flood estimates for the 5%. 2%, 1%, 0.5% and 0.2% AEP floods as well as an Extreme Flood. The procedures employed in deriving these design floods are outlined in the following sections.

6.2 DESIGN FLOOD HYDROLOGY

6.2.1 Design Flood Simulations

The RAFTS hydrologic model described in **Section 4** was used to simulate runoff from the catchment for design storm conditions. The design storm conditions were based on rainfall intensities and temporal patterns for the study area, which were derived using standard procedures outlined in *'Australian Rainfall and Runoff – A Guide to Flood Estimation'* (*1987*) (*ARR 87*). The design storm rainfall data was generated by applying the principles of rainfall intensity estimation described in Chapter 2 of ARR 87.

A new edition of '*Australian Rainfall and Runoff*' was released in December 2016 and revised IFD Data was also made available at this time by the Bureau of Meteorology. As the flood study was essentially completed prior to the new release of ARR, including the hydrologic and hydraulic modelling runs based on ARR 87, Wollondilly Shire Council decided that it was not necessary to revisit the modelling for the flood study at this time. Considerations of the revised ARR 16 will need to be carried out in future studies.

For this study, the same IFD parameters were adopted as those determined and used as part of the modelling completed for the report titled, '*Stonequarry Creek – 2D Modelling and Climate Change Assessment*' (*2011*). These IFD parameters are provided in **Appendix B**.

A comparison of the IFD parameters adopted for this study, to those adopted as part of the '*Picton Flood Study*' (*1989*) shows little difference in values.

As discussed above, a critical storm duration of 9 hours was determined for Stonequarry, Racecourse and Crawfords Creeks, as this storm duration was found to generate the highest discharge in the area where peak flood levels are of most interest; that is, in the vicinity of the built up areas of Picton.

Discharge hydrographs were generated for locations throughout the catchment for a range of flood frequencies using the appropriate critical durations and the appropriate rainfall intensities and design temporal patterns. The design flood frequencies considered for this study include the 5%, 2%, 1%, 0.5% and 0.2% AEP flood events.

An estimate of the Probable Maximum Precipitation (*PMP*) for this study was adopted based on procedures outlined in the Bureau of Meteorology publication, '*The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method*' (*Bureau of Meteorology, 2003*). These procedures were applied to the Stonequarry Creek catchment to derive the PMP for rainfall contributing to flooding in the catchment.

A design temporal distribution was also determined in accordance with procedures outlined in the Bureau's publication. The temporal pattern was based on a standard mass curve which provided a distribution of total rainfall over 20 time intervals during each storm duration.

In simulating the Probable Maximum Flood (*PMF*), the three (*3*) hour PMP storm duration was found to be critical for the catchment as a whole. It should be noted that this duration is shorter than the critical storm duration determined for the other design flood events.

This adopted methodology for the estimation of discharges for design flood scenarios is consistent with the methodology employed for the 2011 investigations.

6.2.2 Hydrologic Modelling Results

Design discharge hydrographs determined using the RAFTS hydrologic model were used to define inflows into the RMA-2 hydrodynamic model.

A summary of the peak discharges for each tributary inflow is provided in **Table 12**. The peak discharges are referenced to the RAFTS model node numbers which are shown in **Figure 2**. For example, the peak discharge along Stonequarry Creek at the upstream extent of the RMA-2 model corresponds to the listed discharges in **Table 12** for XP-RAFTS model node number 1.06.

The design discharge hydrographs derived at the upstream extent of each of the tributaries are included within **Appendix D**.

6.2.3 Comparison of Design Flows with Previous Studies

As discussed in **Section 4.1.1**, flood modelling undertaken for the 1989 Flood Study was based on hydrology and peak flows predicted using a XP-RAFTS model developed specifically for that study. Although the same model has essentially been used for this study, some changes in IFD parameters, catchment roughness and percentage imperviousness values have been incorporated.

The adopted critical durations for design events have also been changed. As discussed in **Section 4.1.1**, investigations for this study determined that the critical duration for the catchment is 9 hours. Previous investigations in 1989 and 2011 adopted a critical storm duration of 6 hours.

A comparison of the peak discharges determined by the 1989 RAFTS hydrologic model and the updated XP-RAFTS hydrologic model is provided in **Section 4.2** (*refer* **Table 7**).

Table 12 PEAK DESIGN INFLOWS FOR THE RMA-2 FLOOD MODEL

TRIBUTARY	RAFTS MODEL NODE NUMBER[1]	STORM DURATION (hours)	PEAK DISCHARGE[2] (m³/s)					
			PMP	0.2% AEP	0.5% AEP	1% AEP	2% AEP	5% AEP
Stonequarry Creek	1.06	6		390	341	305	270	230
		3	1,624					
Racecourse Creek	6.04	6		154	132	117	102	85
		3	630					
Crawfords Creek	5.01	6		89	77	68	60	51
		3	387					
Unnamed Creek	4.02	6		78	67	60	52	44
		3	354					

1. *For node and catchment locations refer to* ***Figure 2****.*
2. *Peak discharges listed do not necessarily occur simultaneously.*

6.3 FLOOD HYDRAULICS

6.3.1 Design Flood Simulations

The updated RMA-2 hydrodynamic model was used to simulate flood behaviour across the floodplain of Stonequarry Creek and its tributaries. The model was used to simulate each of the design 5%, 2%, 1%, 0.5% and 0.2% AEP flood events, and the Probable Maximum Flood (*PMF*). The design simulations were based on a range of boundary condition data which is described in the following sections.

Boundary Conditions

Upstream boundary conditions were defined for each design flood based on the inflow hydrographs generated using the XP-RAFTS hydrologic model (*refer* **Table 12** *and* **Appendix D**). For example, design 1% AEP flood discharge hydrographs for creek inflows were extracted from the XP-RAFTS hydrologic model output and used to define the rate of flow into the area covered by the flood model.

A total of four (*4*) continuity line inflows were adopted to input flows into the upstream extents of the flood model along Stonequarry, Racecourse, Crawfords and an Unnamed Creek. The locations of all upstream boundary inflows are shown in **Figure 11**.

As discussed in **Section 5.3.2**, a stage-discharge relationship was adopted for this study as the downstream boundary condition.

6.3.2 Results and Discussion

Peak Flood levels

Peak flood level estimates were extracted from the hydrodynamic modelling results and were used to generate peak water surface profiles (WSPs) for each of the design events. The design flood surface profiles generated are presented in **Figures 18, 19** and **20**.

WSP Figures for each tributary are as follows:

- Stonequarry Creek Water Surface Profile - **Figure 18**;
- Racecourse Creek Water Surface Profile - **Figure 19**; and
- Crawfords Creek Water Surface Profile - **Figure 20**.

Extent of Inundation

The predicted extents of inundation across the floodplain for the 5% and 1% AEP floods and the Probable Maximum Flood were extracted from the modelling results and are presented in **Figures 21** to **29**. The study area has been split up into three (*3*) extents in order to provide sufficient detail at key locations.

Plate 9 on the following page provides an overview of the three (*3*) extents.

Figures 25 to **33** show that a substantial proportion of the study area is at risk of flooding during events up to and including the Probable Maximum Flood.

At the peak of the 1% AEP flood, the majority of overbank inundation occurs across undeveloped areas upstream of the Picton town centre and through the town centre itself. The extent of inundation within the Picton town centre is shown in greatest detail in **Figures 22**, **25** and **28** for the 5% and 1% AEP floods and for the Probable Maximum Flood, respectively.

As shown in **Figures 23**, **26** and **29**, significant inundation is also predicted to occur upstream of the Main Southern Railway Viaduct along the lower floodplain areas of Stonequarry Creek. As shown in **Figure 23**, significant inundation is predicted across Victoria Park during the 5% AEP flood. Unlike further upstream, inundation along these lower sections of the Study Area is largely influenced by the hydraulic control that is formed by the floodplain narrowing at the railway viaduct.

Floodwater Depths

Peak floodwater depths were extracted from the modelling results for the 5% and 1% AEP floods and are presented in **Figures 30** to **32**, and **Figures 33** to **35**, respectively.

These figures indicate that in major floods, floodwater depths of over 1 metre occur across large areas of the Picton town centre and across developed parts of the floodplain. Floodwater depth mapping was also extracted for the Probable Maximum Flood and is shown in **Figures 36** to **38**.

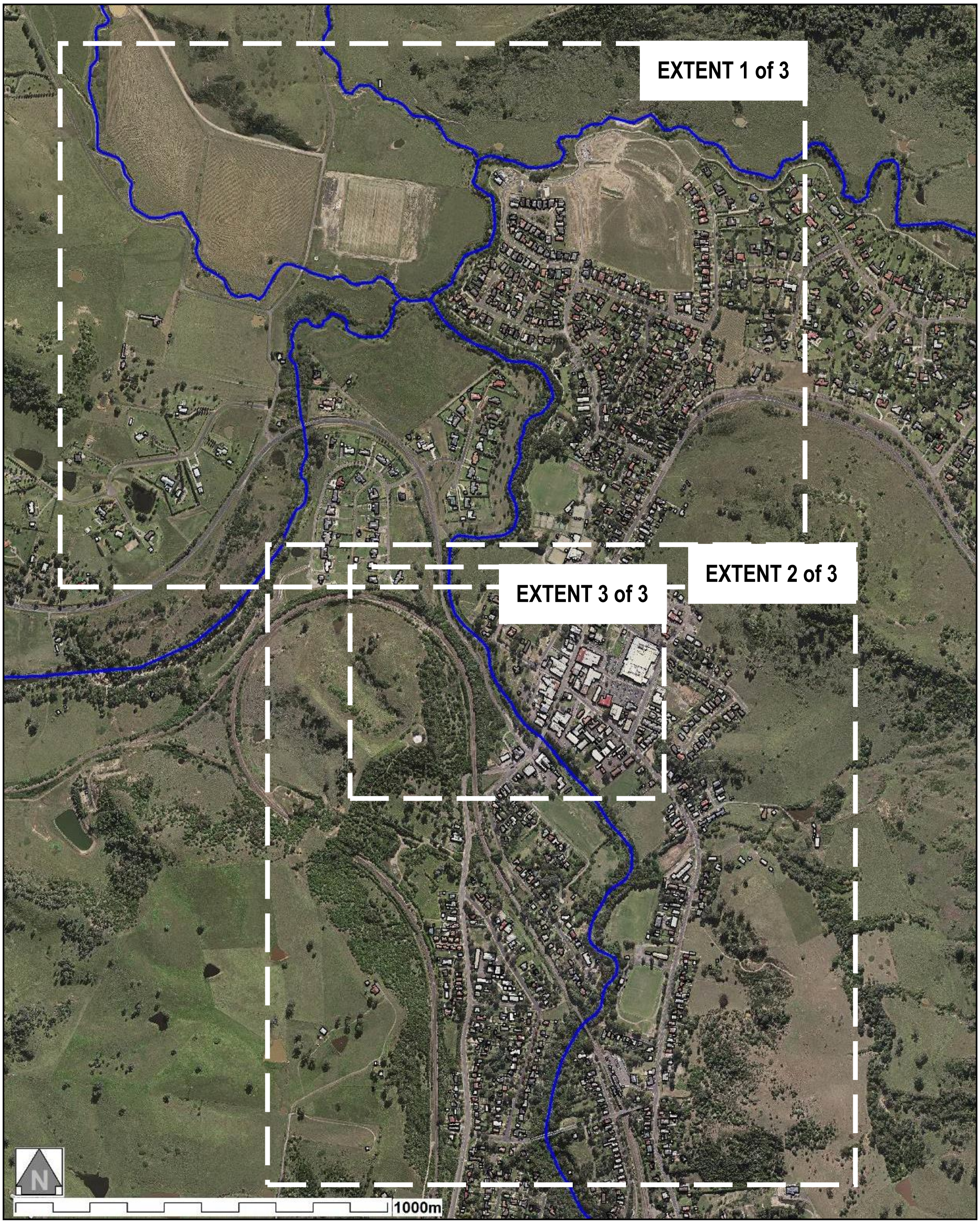


PLATE 9 OVERVIEW OF FIGURE EXTENTS FOR DESIGN MODELLING RESULTS

Flow Velocities

Peak flow velocities for the adopted design 5% and 1% AEP floods are superimposed over the floodwater depth plots shown in **Figures 39** to **44** as velocity vectors. The mapping indicates that the peak flow velocities are largest within the main channel of Stonequarry Creek and its tributaries.

In-channel velocities of between 1.8 and 3.5 m/s are typically observed along Stonequarry Creek at the peak of the 1% AEP flood (*refer* **Figures 42** *to* **44**). Adjacent to the town centre and upstream and downstream of Argyle Street, the in-channel velocities are generally higher, ranging between 2.6 and 3.2 m/s (*refer* **Figure 43**).

Across overbank areas velocities are considerably lower, rarely exceeding 1.0 m/s during the 1% AEP flood. Through the town centre, between Argyle Street and Elizabeth Street, velocities typically range between 0.4 and 0.8 m/s. As shown in **Figure 43**, velocities are greatest through the town along Argyle Street where they are effectively '*channelled*' towards Stonequarry Creek by the buildings. The modelling predicts localised peak velocities of up to 1.5 m/s along Argyle Street during the 1% AEP flood.

6.3.3 Comparison of Updated 1% AEP Flood Levels with 1989 HEC-2 Results

The updated modelling results for Stonequarry Creek show a reasonably good match with the water surface profiles generated by the 1989 HEC-2 model (*refer* **Table 13**).

The comparison between the 1% AEP modelling results generated from the RMA-2 and 1989 HEC-2 models shows that flood levels predicted by the RMA-2 model are on average higher than those predicted in 1989. As shown in **Figure 40**, the RMA-2 modelling results appear to follow quite closely the gradient of the HEC-2 model flood profile for both the natural and channel clearing scenarios. However, they are typically 200 to 300 mm higher compared to the natural channel scenario. At some locations this difference is much lower; for example, just upstream of the Main Southern Railway Viaduct and in the vicinity of the Argyle Street Bridge crossing.

The differences in flood levels are expected given the substantial variation in topographic data that has been adopted and the change in flood modelling approach; i.e., from one-dimensional to two-dimensional. The newly acquired topographic data (*LiDAR*) and two-dimensional modelling approach are considered more reliable than what was adopted/available in 1989 and as such, so too are the updated results.

The higher levels predicted by the RMA-2 model are also the result of the increase in peak discharges of between 20 to 30% throughout the study area as compared to the 1989 HEC-2 modelling. As discussed in **Section 4.2**, these increases have come as a result of further hydrologic assessment undertaken in 2011 which identified a longer critical duration for the study area.

Table 13 contains a comparison of peak 1% AEP flood levels at the locations of each of the HEC-2 model cross-sections (*refer* **Figure 3**). As shown, the RMA-2 model generated flood levels appear to match more closely the '*Natural Scenario*' than the '*Channel Clearing Scenario*'.

Table 13 COMPARISON BETWEEN 2017 RMA-2 AND 1989 HEC-2 FLOOD LEVELS

HEC-2 CROSS-SECTION No. (*Refer* Figure 3)	PREDICTED 1% AEP LEVEL (*mAHD*)				
	Updated RMA-2 Model (2017)	1989 HEC-2 (*Natural Scenario*)	DIFF (*m*)	1989 HEC-2 (Channel Clearing)	DIFF (*m*)
23 (*Upstream Limit*)	160.8	161.47	- 0.67	160.80	- 0.00
22	160.2	160.45	- 0.25	159.96	+ 0.24
21	160.1	159.8	+ 0.30	159.56	+ 0.54
20	159.9	159.56	+ 0.34	159.38	+ 0.52
19	159.8	159.27	+ 0.53	159.15	+ 0.65
18	159.1	158.76	+ 0.34	158.58	+ 0.52
17 (*Elizabeth Street*)	158.8	158.54	+ 0.26	158.36	+ 0.44
16	158.6	158.4	+ 0.20	158.20	+ 0.40
15	158.2	158.19	+ 0.01	157.91	+ 0.29
14 (*Upstream Argyle Street*)	158.0	158.11	- 0.11	157.87	+ 0.13
13 (*Downstream Argyle Street*)	157.9	157.99	- 0.09	157.85	+ 0.05
12	157.7	157.68	+ 0.02	157.11	+ 0.59
11	157.6	157.45	+ 0.15	157.00	+ 0.60
10	157.3	157.21	+ 0.09	156.84	+ 0.46
9 (*Baxters Lane*)	157.3	157.12	+ 0.18	156.82	+ 0.48
8	157.2	157.02	+ 0.18	156.76	+ 0.44
7	157.1	156.94	+ 0.16	156.69	+ 0.41
6	157.0	156.87	+ 0.13	156.58	+ 0.42
5	156.7	156.75	- 0.05	156.44	+ 0.26
4	156.3	156.39	- 0.09	156.08	+ 0.22
3 (*Upstream Railway Line*)	155.8	155.78	+ 0.02	155.47	+ 0.33
2 (*Downstream Railway Line*)	155.4	155.42	- 0.02	155.16	+ 0.24
1 (*Downstream Limit*)	155.3	154.85	+ 0.45	154.85	+ 0.45

Table 14 COMPARISON BETWEEN 2017 RMA-2 AND 2011 RMA-2 LEVELS

HEC-2 CROSS-SECTION No. (*Refer* **Figure 3**)	**PREDICTED 1% AEP LEVEL** (*mAHD*)		
	Original RMA-2 Model (2011)	**Updated RMA-2 Model (2017)**	**DIFF** (*m*)
23 (*Upstream Limit*)	160.8	160.8	+ 0.00
22	160.2	160.2	+ 0.00
21	160.2	160.1	- 0.10
20	159.8	159.9	+ 0.10
19	159.5	159.8	+ 0.30
18	159.0	159.1	+ 0.10
17 (*Elizabeth Street*)	158.8	158.8	+ 0.00
16	158.4	158.6	+ 0.20
15	157.9	158.2	+ 0.30
14 (*Upstream Argyle Street*)	157.8	158.0	+ 0.20
13 (*Downstream Argyle Street*)	157.6	157.9	+ 0.30
12	157.4	157.7	+ 0.30
11	157.2	157.6	+ 0.40
10	157.1	157.3	+ 0.20
9 (*Baxters Lane*)	157.1	157.3	+ 0.20
8	156.8	157.2	+ 0.40
7	156.7	157.1	+ 0.40
6	156.6	157.0	+ 0.40
5	156.1	156.7	+ 0.60
4	155.7	156.3	+ 0.60
3 (*Upstream Railway Line*)	155.1	155.8	+ 0.70
2 (*Downstream Railway Line*)	154.6	155.4	+ 0.80
1 (*Downstream Limit*)	154.6	155.3	+ 0.70

6.3.4 Comparison of Updated 5% and 1% AEP Flood Levels with 2011 RMA-2 Results

Stonequarry Creek

A comparison of the modelling results generated using the updated RMA-2 model to those generated using the original RMA-2 model (*2011*) was also undertaken. To ensure consistency with the comparison undertaken to the 1989 HEC-2 results, the HEC-2 cross-sections have again been used as the comparison locations.

Table 14 shows the results of the comparison of peak 1% AEP flood levels at each of the HEC-2 model cross-sections (*refer* **Figure 3**). Floodwater surface profiles for the 5% and 1% AEP floods are presented on **Figure 41** for Stonequarry Creek.

The comparison shows that the updated 2014 RMA-2 modelling results are generally within 200 mm of the 2011 RMA-2 modelling levels for areas upstream of Regreme Road (*refer* **Figure 41**). The external model appropriately accounts for the wider data set and the results are considered more reliable.

These relatively minor differences are considered to be directly related to the changes in topographic elevations adopted within the 2011 and updated 2017 models. These changes are due to the inclusion of more reliable topographic data in the 2017 model. The 2017 model was based on LiDAR data acquired in 2013, whereas the previous modelling was based on surface contours at 2 metre intervals provided by Council in 2005.

Downstream of Regreme Road the updated flood levels are typically higher than those predicted in 2011. As shown in **Figure 40** the updated levels are predicted to be higher on average by 200 mm to 500 mm, with maximum differences of up to 800 mm immediately upstream of the Main Southern Railway Viaduct.

Differences upstream of the viaduct are considered to be caused by a combination of the updated topographic data and also the change in adopted downstream boundary condition. As discussed in **Section 5.3**, the downstream boundary of the 2011 model was governed by the limited availability of topographic data which extended only a short distance beyond the railway viaduct. This limitation resulted in the under-prediction of flood levels in areas along the creek upstream from the viaduct.

A comparison of 2011 and 2017 RMA-2 topographic elevations along Stonequarry Creek is provided in **Figures D1** to **D3** of **Appendix D**.

Racecourse Creek

A comparison of 5% and 1% AEP flood levels along Racecourse Creek as predicted using the 2011 and 2017 RMA-2 models is shown in **Figure 41**. As shown, the floodwater gradients predicted by the 2011 and 2017 models are generally consistent with the exception of some localised variances and undulations.

The majority of these variances are located towards the upstream end of Racecourse Creek where the Creek is characterised by a series of sharp meanders.

A comparison of 2011 and 2017 topographic elevations highlighted that the LiDAR data "picked-up" a more incised channel than was previously simulated in the 2011 modelling. The 2011 model also adopted lower overbank elevations than those defined by the more recently acquired LiDAR data (*refer* **Figure D4** *in* **Appendix D**).

The 2017 modelling was also identified as more reliably representing the meandering channel which was picked-up in greater detail by the finer model network incorporated into the 2017 RMA-2 model. This has resulted in additional hydraulic losses when compared to the 2011 modelling which, when combined with the more incised channel, is responsible for the higher flood levels.

As shown in **Figure 41**, 2011 and 2017 flood levels are generally within 200 mm for areas downstream of the confluence with Crawfords Creek.

Crawfords Creek

Flood profiles comparing the predicted 5% and 1% AEP flood levels along Crawfords Creek are presented in **Figure 42**. As shown, peak flood levels derived from the 2011 and 2014 modelling are generally in good agreement, with the exception of the upstream extents of the study area where differences of up to 400 mm are predicted; i.e., along chainages 0 to 150 metres. These differences are attributed to the updated topographic data showing a general increase in topographic elevations across overbank areas (*refer* **Figure D5** *in* **Appendix D**).

Between chainages 150 metres to 550 metres on **Figure 42**, the differences in levels are much lower and generally less than 200 mm.

6.3.5 Comparison of Updated PMF Levels with 2011 RMA-2 Results

Stonequarry Creek

A comparison of floodwater surface profiles generated for the PMF was superimposed on **Figure 40** for Stonequarry Creek. As shown, the 2011 and 2014 PMF level profiles are generally in good agreement, with the 2014 levels shown to be approximately 100 mm higher along the entire reach of Stonequarry Creek. The significant differences downstream of the Main Southern Railway Viaduct are largely the result of the extension of the updated model and incorporation of the additional topographic data.

Racecourse Creek

A comparison of profiles was also completed for Racecourse Creek and is shown on **Figure 41**. As shown, the updated flood modelling has generally resulted in an increase in PMF levels of between 200 and 400 mm.

As shown in **Figure 41**, the differences are generally highest across the upstream reaches where the creek is generally more '*active*' with sharp meanders. A comparison of topographic elevations across these reaches indicates that the 2014 LiDAR data was generally higher across overbank areas, resulting in the channel that has been incorporated in the model being more incised.

Crawfords Creek

A comparison of PMF profiles was also completed for Crawfords Creek and is superimposed on **Figure 42**. As shown, the updated flood modelling has resulted in an increase in PMF levels with differences generally within 200 mm along the majority of Crawfords Creek. Notwithstanding, 2011 PMF levels are predicted to be up to 1 metre higher along the upper reaches (*between Chainages 0 and 200 metres*) (*refer* **Figure 42**).

A comparison of 2011 and 2014 topographic elevations indicates that the increase in levels was likely attributed to the 2011 RMA-2 model (*and the topography on which it was based*) showing much higher overbank elevations immediately downstream of the model inflow location. The 2011 and 2014 RMA-2 topographies across this area are shown in **Figure D5** of **Appendix D**. The increased topographic elevations in this area appear to cause a greater constriction in flow through this section of Crawfords Creek which has resulted in the increase in predicted PMF levels.

6.3.6 Comparison of Updated 1% AEP and June 2016 Flood Levels

Flood level difference mapping comparing flood levels predicted for the 1% AEP event and the June 2016 Flood are shown in **Figures 43**, **44** and **45**. The difference mapping indicates that peak flood levels for the June 2016 Flood were between 0.02 to 0.22 metres higher than corresponding design 1% AEP flood level estimates. The figures show that the flood level differences are generally highest downstream of the Town Centre where floodwaters are constricted by the railway crossing and the Stonequarry Creek gorge (*refer* **Figures 43** *to* **45**).

The difference between June 2016 and 1% AEP flood levels can further be broken-down into the following:

- 0.02 to 0.07 metres higher for areas upstream of the Town Centre (*refer* **Figure 43**),
- 0.07 to 0.16 metres higher for areas around the Town Centre (*refer* **Figure 44**), and,
- 0.160 to 0.22 metres higher for areas downstream of the Town (*refer* **Figure 45**).

The flood level comparison matches expectations based on the rainfall analysis for the June 2016 event (refer **Section 4.3**) showing rainfall records at the three nearest rainfall gauges all exceeded the rainfall depths required for a 1% AEP event over the critical catchment duration of 9 hours.

7 HAZARD AND HYDRAULIC CATEGORIES

7.1 GENERAL

The personal danger and physical property damage caused by a flood varies both in time and place across the floodplain. Accordingly, the variability of flood patterns across the floodplain over the full range of floods needs to be understood by flood prone landholders and by floodplain risk managers.

Representation of the variability of flood hazard across the floodplain provides floodplain risk managers with a tool to assess the existing flood risk and to determine the suitability of land use and future development. The hazard associated with a flood is represented by the static and dynamic energy of the flow, which is in essence, the depth and velocity of the floodwaters. Therefore, the flood hazard at a particular location within the floodplain, is a function of the velocity and depth of the floodwaters at that location.

The NSW Government's *'Floodplain Development Manual'* (*2005*), characterises hazards associated with flooding into a combination of three hydraulic categories and two hazard categories. Hazard categories are broken down into high and low hazard for each hydraulic category as follows:

- Low Hazard – Flood Fringe
- Low Hazard – Flood Storage
- Low Hazard – Floodway
- High Hazard – Flood Fringe
- High Hazard – Flood Storage
- High Hazard - Floodway

As a result, the manual effectively divides hazard into two categories, namely, high and low. An interpretation of the hazard at a particular site can be established from **Figure L1** and **L2** on the following page, which have been taken directly from the manual.

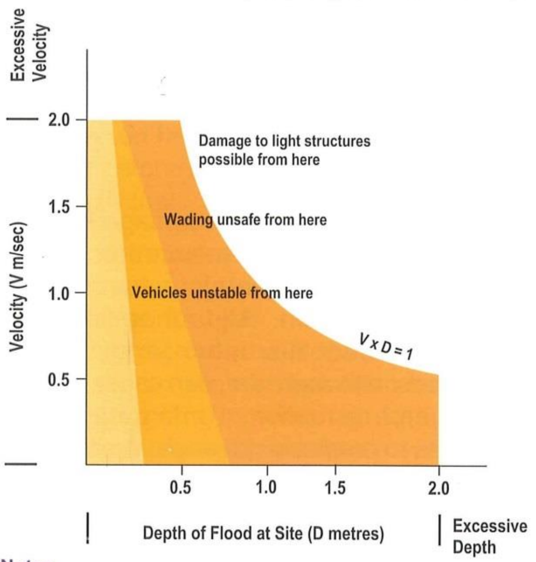
The first of these graphs shows approximate relationships between the depth and velocity of floodwaters and resulting hazard. This relationship has been used to define the provisional low and high hazard categories represented in the second of these plots.

7.2 PROVISIONAL FLOOD HAZARD

As shown in the **Figures L1** and **L2**, flood hazard is a measure of the degree of difficulty that pedestrians, cars and other vehicles will have in egressing flooded areas, and the likely damage to property and infrastructure.

Flood hazard is categorised according to a combination of the flow velocity and the depth of floodwater. The categories are defined by lower and upper bound values for the product of flow velocity and floodwater depth.

In order to provide greater discretisation of hazards across the floodplain, the '*high*' hazard categorisation shown in **Figure L2** has been further split up into '*High Hazard*', '*Very High Hazard*' and '*Extreme Hazard*'. A summary of the criteria adopted for each hazard category is listed in **Table 15** and is also presented in the coloured hazard chart shown as **Plate 3**.

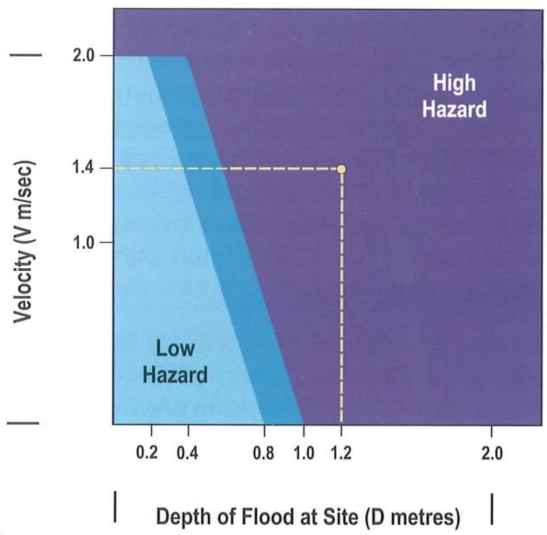


Notes

1. At velocities in excess of 2.0 m/s, the stability of foundations and poles can be affected by scour. Also, grass and earth surfaces begin to scour and can become rough and unstable
2. The velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only the average velocity is considered. For instance, the velocity of floodwaters in a model test has risen from an average of 1 m/sec to 3 m/sec between houses.
3. Vehicle instability is initially by buoyancy.
4. At floodwater depths in excess of 2.0 meters and even at low velocities, there can be damage to light-framed buildings from water pressure, flotation and debris impact.

Derived from laboratory testing and flood conditions which caused damage.

FIGURE L1 - Velocity & Depth Relationships



Notes

The degree of hazard may be either -

- reduced by establishment of an effective flood evacuation procedure.
- increased if evacuation difficulties exist.

In the transition zone highlight by the median colour, the degree of hazard is dependant on site conditions and the nature of the proposed development.

Example:

If the depth of flood water is **1.2 m**
and the velocity of floodwater is **1.4 m/sec**
then the provisional hazard is **high**

FIGURE L2 - Provisional Hydraulic Hazard Categories

Table 15 ADOPTED HAZARD CRITERIA

HAZARD CATEGORY	CRITERIA
Low (H1)	Depth (d) ≤ 0.8 m, velocity (v) ≤ 2.0 m/s, and $v \times d \leq 0.5$
Medium / Transition (H2)	exceeding Low criteria, and $d \leq 0.8$ m, $v \leq 2.0$ m/s, and $v \times d \leq 0.8$
High (H3)	exceeding Medium / Transitional criteria, and $d \leq 1.8$ m, $v \leq 3.0$ m/s, and $v \times d \leq 1.5$
Very High (H4)	exceeding High criteria, and 0.5 m/s < velocity < 4 m/s & $v \times d \leq 2.5$
Extreme (H5)	exceeding Very High criteria and v > 4 m/s

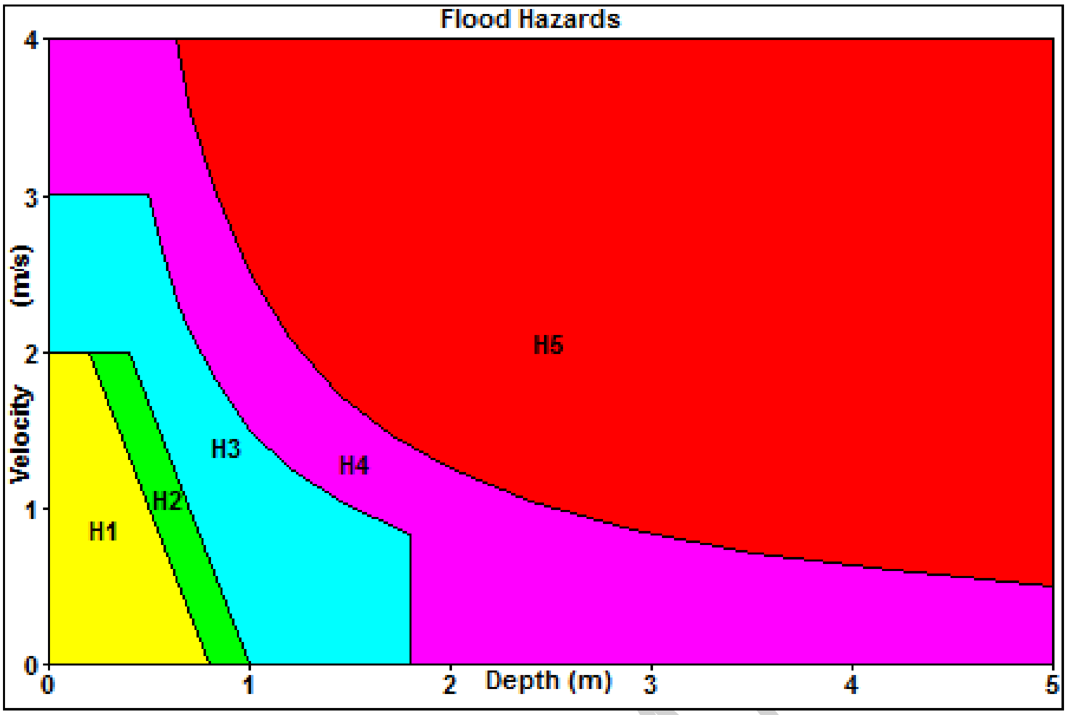


PLATE 10 PROVISIONAL FLOOD HAZARD CHART

Spatial and temporal distributions of flow, velocity and water level determined from the computer modelling undertaken as part of this study were used to determine the flood hazard along the floodplain of Stonequarry Creek and its tributaries. Flood hazard mapping for the 1% AEP flood is presented in **Figures 46** to **48** based on the hazard criteria shown in **Figure L2**.

Interpretation of the hazard mapping indicates that for large events like the 1% AEP flood, the majority of flooded land would fall within the high hazard category defined in the *'Floodplain Development Manual'* (*2005*). This is also the case at the town centre where floodplain areas in the vicinity of Argyle Street, Elizabeth Street and Cliffe Street are predicted to experience '*high hazard*' inundation.

It must be noted that the hazard represented in this mapping is provisional only. This is because it is based only on an interpretation of the flood hydraulics and does not reflect the effects of other factors that influence hazard (*see clause L6 to Appendix L of the Floodplain Development Manual*). For example, access to an otherwise low hazard area may be through a high hazard area and this may present an unacceptable risk to life and limb and as such the provisional low hazard area may be changed to high hazard.

Accordingly, modification of the hazard mapping presented in **Figures 46** to **48** will be required as part of investigations that will need to be undertaken in the future to develop / prepare an updated Floodplain Risk Management Plan for Stonequarry Creek and its tributaries.

7.3 HYDRAULIC CATEGORIES

7.3.1 Adopted Hydraulic Categorisation

The NSW Government's *'Floodplain Development Manual'* (*2005*) also characterises flood prone areas according to the hydraulic categories presented in **Table 16**. The hydraulic categories provide an indication of the potential for development across different sections of the floodplain to impact on existing flood behaviour.

Table 16 HYDRAULIC CATEGORY CRITERIA

HYDRAULIC CATEGORY	DESCRIPTION
FLOODWAY	• those areas where a significant volume of water flows during floods • often aligned with obvious natural channels • they are areas that, even if only partially blocked, would cause a significant increase in flood levels and/or a significant redistribution of flood flow, which may in turn adversely affect other areas • they are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.
FLOOD STORAGE	• those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood • If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. • Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.
FLOOD FRINGE	▪ the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. ▪ Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

Unlike for the hazard categorisation outlined on the previous page, the *'Floodplain Development Manual'* (*2005*) does not provide explicit quantitative criteria for defining hydraulic categories. This is because the extent of floodway, flood storage and flood fringe areas are largely dependent on the geomorphic characteristics of the floodplain in question.

Although there are no specific procedures for identifying or determining hydraulic categories, a rigorous methodology involving several stages of analytical analysis in conjunction with flood modelling has been developed by Thomas & Golaszewski (*2012*). This methodology has been applied with success to similar floodplains in NSW and has been shown to provide a robust procedure for defining floodway extent.

Most recently, this methodology was applied to the Lower Hastings River floodplain as part of investigations for the *'Hastings Floodplain Risk Management Study'* (*2012*), the Lower Camden Haven River floodplain as part of investigations for the *'Camden Haven*

Flood Study' (*2013*), as part of investigations for the '*Griffith Floodplain Risk Management Study*' (*2012*) and also as part of the '*South Creek Flood Study*' (*2015*).

The hydraulic category mapping that was prepared for Stonequarry Creek and its tributaries is presented in **Figures 49** to **51**.

The following sections describe the methodology that was employed to determine the hydraulic category mapping.

7.3.2 Adopted Methodology for Determination of Floodway Corridors

The adopted methodology for determination of hydraulic categories for the floodplain of Stonequarry Creek and its tributaries involved several stages of assessment that relied on rigorous analytical analysis of all available hydraulic, topographic, cadastral and geomorphic data-sets. The analysis also involved testing of hydraulic parameters and flood modelling to simulate the impact of encroachment on initial and revised estimates of floodway corridors.

Once the detailed investigations to determine the extents of floodway corridors were completed, an analytical assessment was also undertaken to determine the extent of flood storage and flood fringe areas. Each of these hydraulic categories was then combined to develop hydraulic category mapping for the study area which can be incorporated into future mapping layers linked to Council's Local Environmental Plan.

A detailed breakdown of the methodology applied to determine the hydraulic category mapping is outlined in the following sections.

Stage 1 – Determination of Preliminary Floodway Extent

A preliminary floodway extent was firstly determined based on an assessment of aerial photography, topographic data and existing 1% AEP flood modelling results. Determination of this extent or "line" considered the following:

- the location of flood storages that are readily identifiable from aerial photography;
- the location and potential impact of hydraulic controls and geomorphic features that could influence floodwater movement and flood characteristics (*e.g., velocity*);
- mapping of contours of 'velocity-depth' product (*V x D*); and,
- mapping of the variation in peak flow velocity.

Because of the complex nature of flooding along Stonequarry Creek and its tributaries and the varied floodplain types encountered across the study area, establishment of a standard set of criteria was <u>not</u> considered appropriate for the determination of all floodway extents. For example, definition of the floodway extent based on a single target value for velocity or velocity-depth product (*V x D*) would limit the reliability of the investigation findings.

Accordingly, to ensure the assessment of floodway extent was completed reliably, the study area was divided into numerous precincts to enable assessment on a '*local*' scale.

A set of interactive flood maps was produced for each of these precincts to show key hydraulic data including the variation in V x D, peak flow velocities and peak flood depths. The results of modeling of the design 1% AEP flood were used as the benchmark for the analysis.

The interactive flood maps were used to identify areas of the floodplain representing:

- high depth and high velocities; i.e., high V x D (*generally considered floodway*);
- high depth and low velocities (*generally considered flood storage*); and,
- low depth and low velocity (*generally considered flood fringe*).

In this regard, a typical "first pass" assessment of floodway extents was undertaken to identify areas where the velocity-depth product is greater than 1.0 m^2/s and where flow velocities are greater than 0.5 m/s. The alignment of significant flow paths across the floodplain (*i.e., potential flood runners*), as inferred by the velocity and V x D contour mapping, was also considered in determining the preliminary floodway extents.

The preliminary floodway extent was further verified by comparison with mapping of the width of the floodplain that would be required to convey 80% of the peak flow. Trial analyses for this project and similar floodplain risk management studies have shown a good correlation between the transitions in velocity-depth product contour mapping, geomorphic characteristics and the width of the floodplain that conveys about 80% of the flood flow. A discussion of this criteria and its appropriateness for defining floodway extent is provided in Thomas et al (*2012*).

The width occupied by 80% of the flow was readily determined for any location within the lower reaches of the floodplain using the *Flow Extraction* tool within waterRIDE™. This width was then used to verify and adjust the preliminary floodway extent.

Through mapping of the floodplain extent required to convey 80% of the flood flow it became evident that no one value of velocity-depth could be adopted for the entire study area. This was perhaps most evident when investigating the floodway extents along the tributaries where velocity-depth products where considerably higher than along much of Stonequarry Creek. Along the tributaries velocity-depth products of 2 to 3 m^2/s and 2 to 4 m^2/s and above were found to convey at least 80% of the flow and were representative of the floodway corridor along Crawfords Creek and Racecourse Creek, respectively.

Mapping showing the distribution of flows along a series of cross-sections relative to 1% AEP velocity-depth products is included in **Appendix F** as **Figures F1** to **F3** for Crawfords and Racecourse Creeks and the unnamed creek.

Along Stonequarry Creek appropriate velocity-depth products were found to be much lower and typically around 0.5 m^2/s to 1.5 m^2/s. At these values of velocity-depth product, a cross-sectional analysis found that at least 80% of the total 1% AEP flow would be 'captured'.

Due consideration was also given to the full range of design flood events; that is, the assessment was not solely reliant on hydraulic data for the 1% AEP event.

Particular attention was paid to identifying floodways that could emerge during flooding of the magnitude of the 0.5% AEP event and during a Probable Maximum Flood. This was of particular importance in the vicinity of the town centre where distinct flowpaths were more difficult to define and/or differentiate.

Stage 2 – Encroachment Testing of Adopted Preliminary Floodway Extent

The adopted preliminary floodway extent mapping was tested and verified across the entire reach of Stonequarry Creek and its tributaries.

The analyses involved flood modelling of 'encroachment' scenarios to test whether the 'Stage 1' floodway corridor was sufficiently sized to convey a significant proportion of total flood volume. A floodway corridor was considered sufficiently sized if the encroachment testing did not lead to increases in 1% AEP flood level of much greater than 100 mm.

Flood level difference mapping was prepared for each iteration of the modelling and the alignment of the preliminary floodway extent was adjusted where necessary; i.e., where flood level increases were found to be significant. Adjustment of the preliminary floodway extent was undertaken by re-applying the Stage 1 methodology. Areas that required the most attention were locations where the floodway boundary was not readily apparent from velocity or V x D contour mapping.

This iterative approach led to the development of a Refined Floodway Alignment which was adopted for this study and deemed to satisfy the adopted floodway criteria.

7.3.3 Adopted Methodology for Determining Flood Storage and Flood Fringe

Following determination of those areas of the floodplain categorised as floodway, investigations were focused towards identifying the remaining hydraulic categories, namely flood storage and flood fringe. As outlined in the NSW *'Floodplain Development Manual'* (*2005*), flood storage and flood fringe make up the remainder of the floodplain outside of the floodway corridor.

Flood storage areas are typically defined as those flood prone areas that afford significant temporary storage of floodwaters during a major flood. If filled or obstructed (*through the construction of levees or road embankments*) the reduction in storage would be expected to result in a commensurate increase in flood levels in nearby areas. The remaining flood prone areas not classified as floodway or flood storage are termed flood fringe.

In order to determine the boundary between flood storage and flood fringe, the variation in peak flood depths and velocities in areas outside of the floodway extent was mapped to identify areas inundated to depths of up to 0.3 metres and velocities of up to 1.0 m/s. A depth of 0.3 metres was selected as it is considered to be the transitional point up to which flood conditions become hazardous to people and vehicles and up to which any future development proposals would require substantial earthworks (*i.e., floodplain filling to elevate finished floor levels to meet Council requirements*).

In terms of the Stonequarry Creek floodplain and that of its tributaries, peak depths below 0.3 metres are also considered to correspond to areas where negligible flow is conveyed and represent a relatively small proportion of storage for floodwaters. This is further supported by an assessment of peak 1% AEP velocities, where concurrent mapping of both criteria showed velocities were less than 1.0 m/s at all locations where depths are predicted to be less than 0.3 metres.

In accordance with the Floodplain Development Manual (*2005*), this represents areas which are unlikely to have any significant impact on the pattern of floodwater distribution through a creek and floodplain system and associated flood levels.

Accordingly, the boundary between flood storage and flood fringe was defined by a peak 1% AEP flood depth of 0.3 metres and peak velocities of up to 1.0 m/s. Accordingly, the velocity-depth product for flood fringe areas is less than 0.3 m^2/s.

Flood storage and flood fringe mapping for the floodplains of Stonequarry Creek and its tributaries is presented as **Figures 49** to **51**.

8 CLIMATE CHANGE ASSESSMENT

The NSW Department of Environment and Climate Change's document entitled *"Practical Consideration of Climate Change"* (*2007*), discusses various methods for addressing the impacts of climate change in regard to flooding. The document suggests that climate change will increase the intensity of extreme rainfall events in NSW by up to 30% by the year 2070. In the Hawkesbury-Nepean region of NSW, extreme rainfall intensities are expected to vary between a reduction of 7% and an increase of 12% during this period (*DECC, 2007*).

To account for an increase in peak rainfall intensities, the document recommends that a sensitivity analysis be carried out by increasing rainfall intensities by 10%, 20% and 30%, respectively. To assess these impacts, the RAFTS hydrologic model for the catchment was re-run for the 1% AEP design rainfall with these increases imposed.

The resulting flow hydrographs at the four RMA-2 inflow locations are included in **Appendix E** as **Figures E5** to **E8**.

The updated RMA-2 model was used to simulate each of the climate change scenarios to determine the predicted impact of increased rainfall intensities (*10%, 20% and 30%*) on peak 1% AEP flood levels, depths and velocities.

The predicted impacts on peak 1% AEP flood levels are shown in **Figures 52** to **54**. Flood level difference mapping was also prepared for each climate change scenario and is shown in **Figure 55**, **Figure 56** and **Figure 57**; i.e., 1% AEP + 10% rainfall intensity increase, 1% AEP + 20% and 1% AEP + 30%.

The three climate change scenarios, show a significant increase in flood levels, which are greatest in the lower sections of the Study Area immediately upstream of the Main Southern Railway Viaduct. As shown in **Figures 55** to **57**, the 10%, 20% and 30% scenarios predict maximum flood level increases upstream of the railway viaduct of 0.48, 0.90 and 1.28 metres, respectively.

Flood level increases in areas further upstream are much less. For example, within the town the predicted flood level increases are approximately 0.32, 0.63 and 0.91 metres respectively for the 10% 20% and 30% rainfall increase scenarios (*refer* **Figures 55** *to* **57**).

9 REFERENCES

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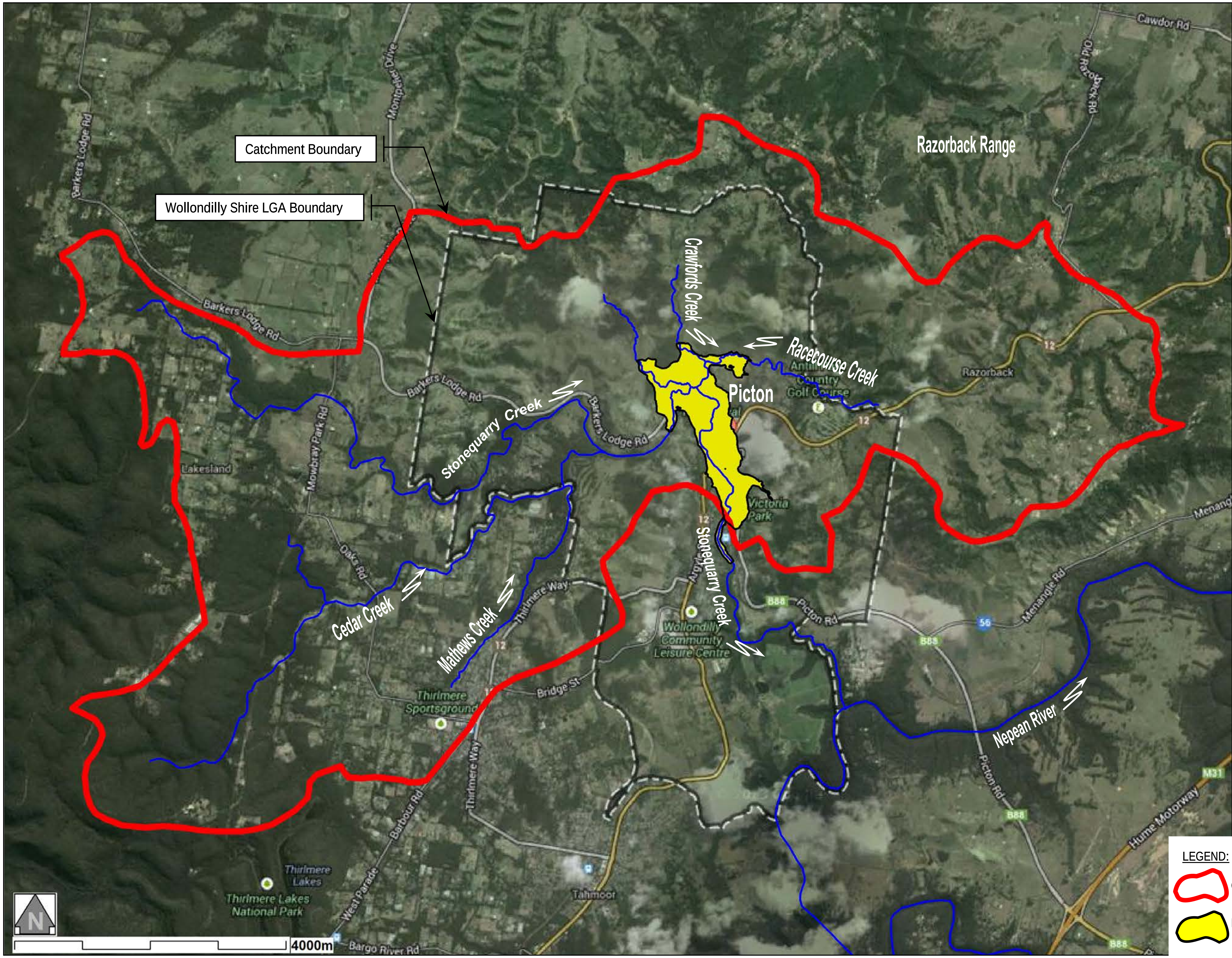
- Wollondilly Shire Council (2017), 'Picton / *Stonequarry Creek – Flood Study Peer Review'*; prepared by Manly Hydraulics Laboratory.

REPORT FIGURES

FIGURE 1

DRAFT



LOCATION OF STUDY AREA AND EXTENT OF STONEQUARRY CREEK CATCHMENT

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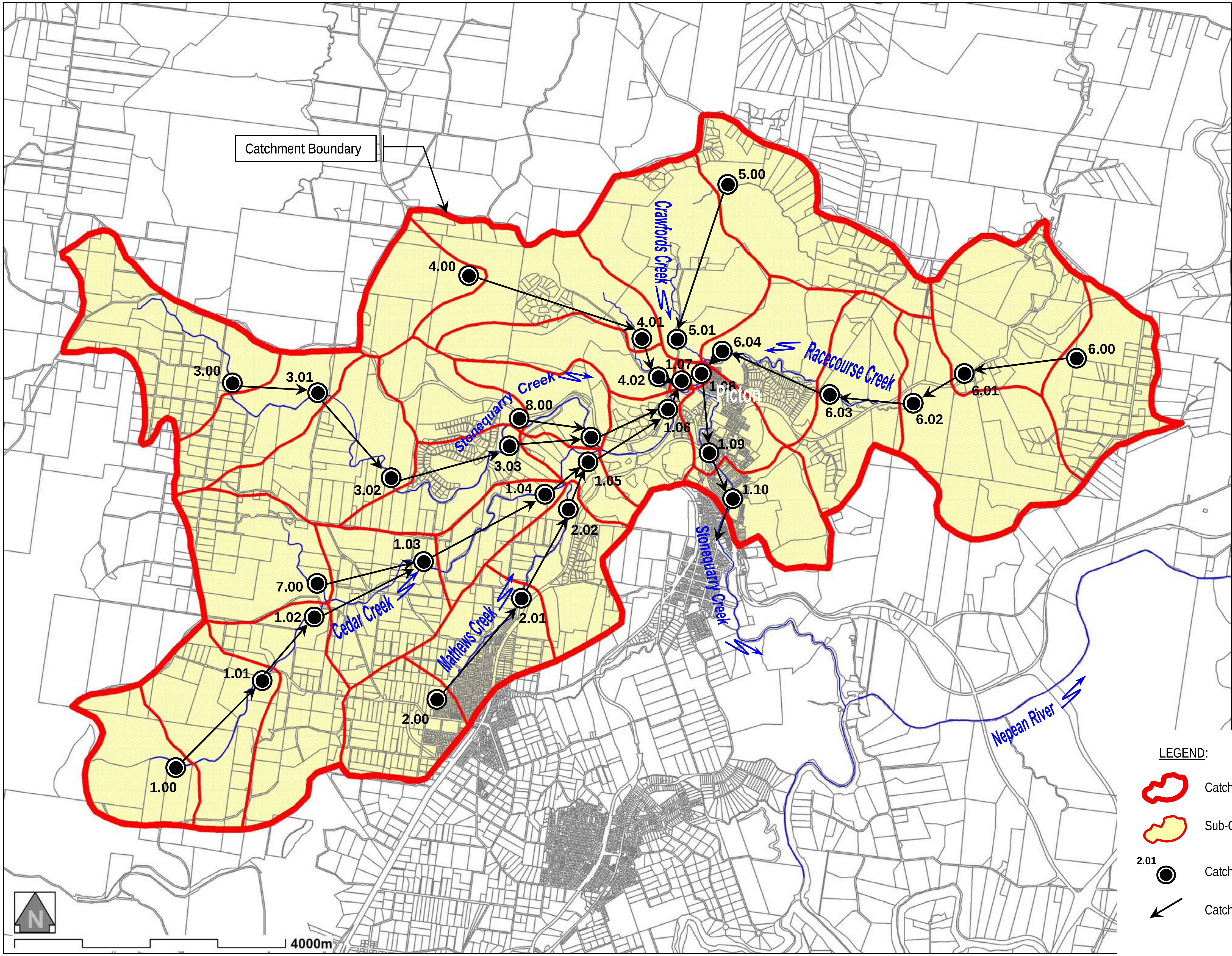
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140109_Fig1_Catchment Area.doc

FIGURE 2

DRAFT



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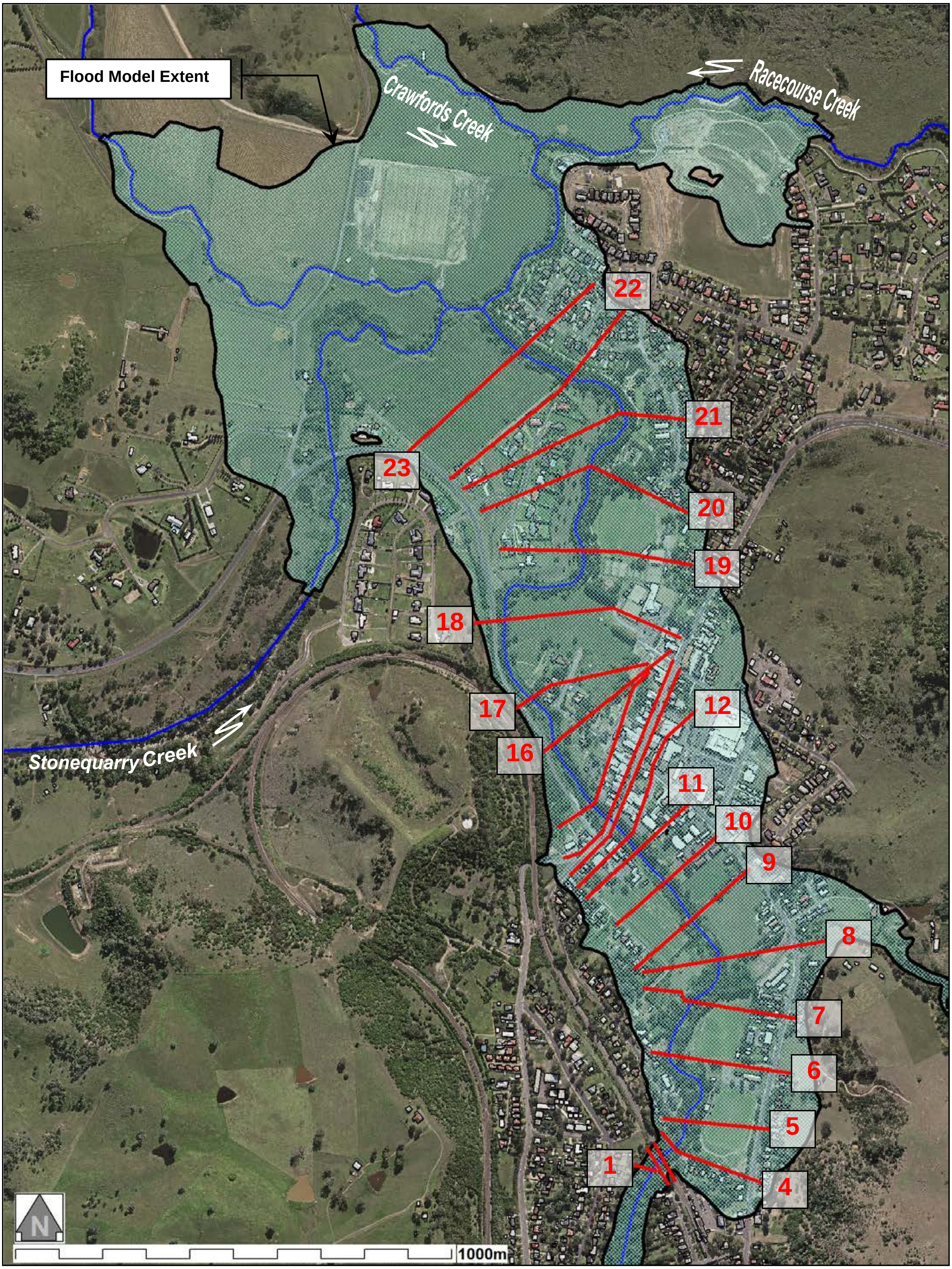
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140109_Fig2_RAFTS Layout.doc

XP-RAFTS HYDROLOGIC MODEL
SHOWING NODE AND LINK ARRANGEMENT



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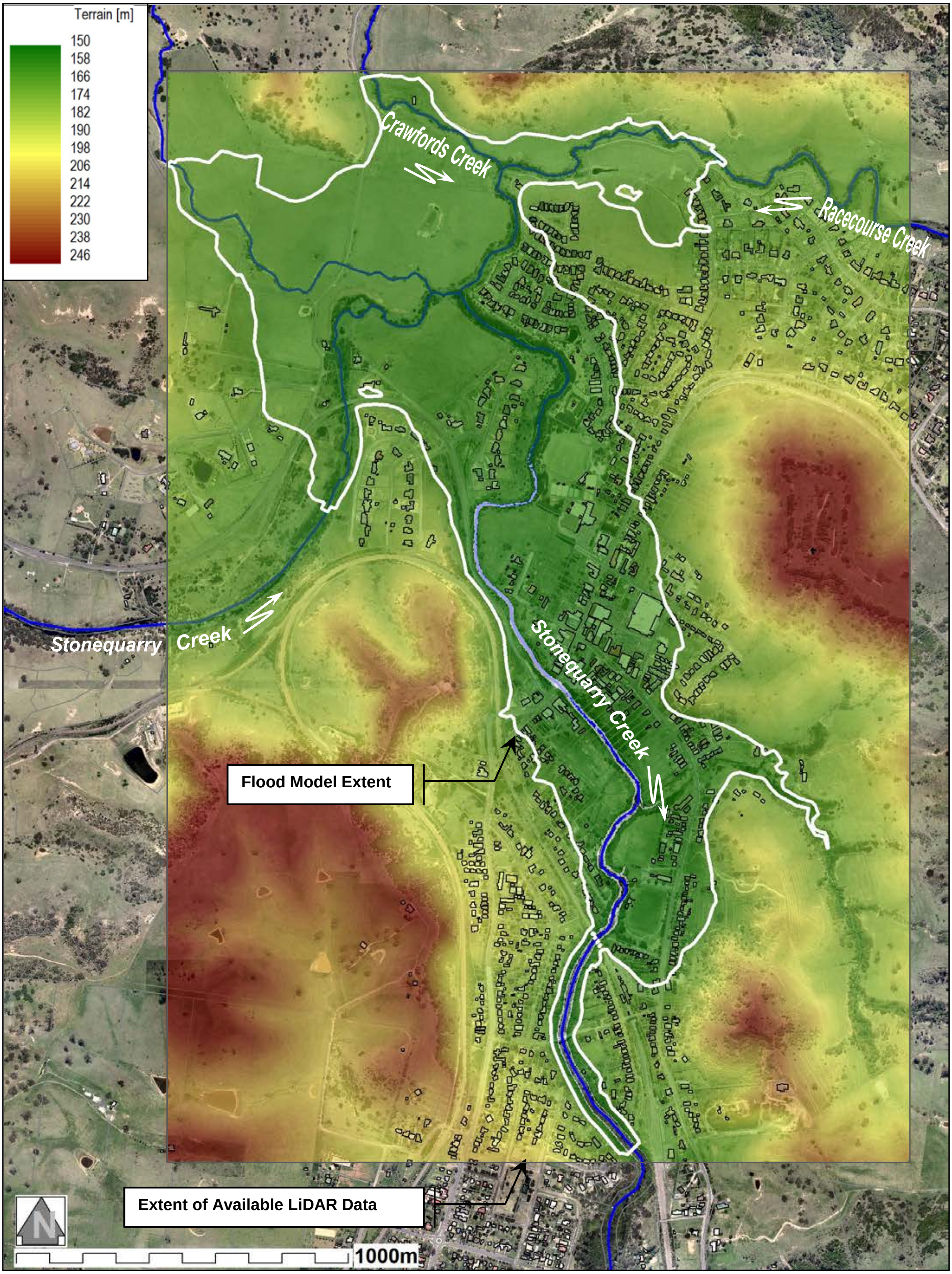
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140109_Fig3_1989 HEC-RAS Cross-Sections.doc

LOCATION OF 1989 HEC-2 MODEL CROSS-SECTIONS

FIGURE 4



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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140109_Fig4_Additional Data (2012).doc

ADDITIONAL FLOOD STUDY DATA DIGITAL ELEVATION MODEL

FIGURE 5

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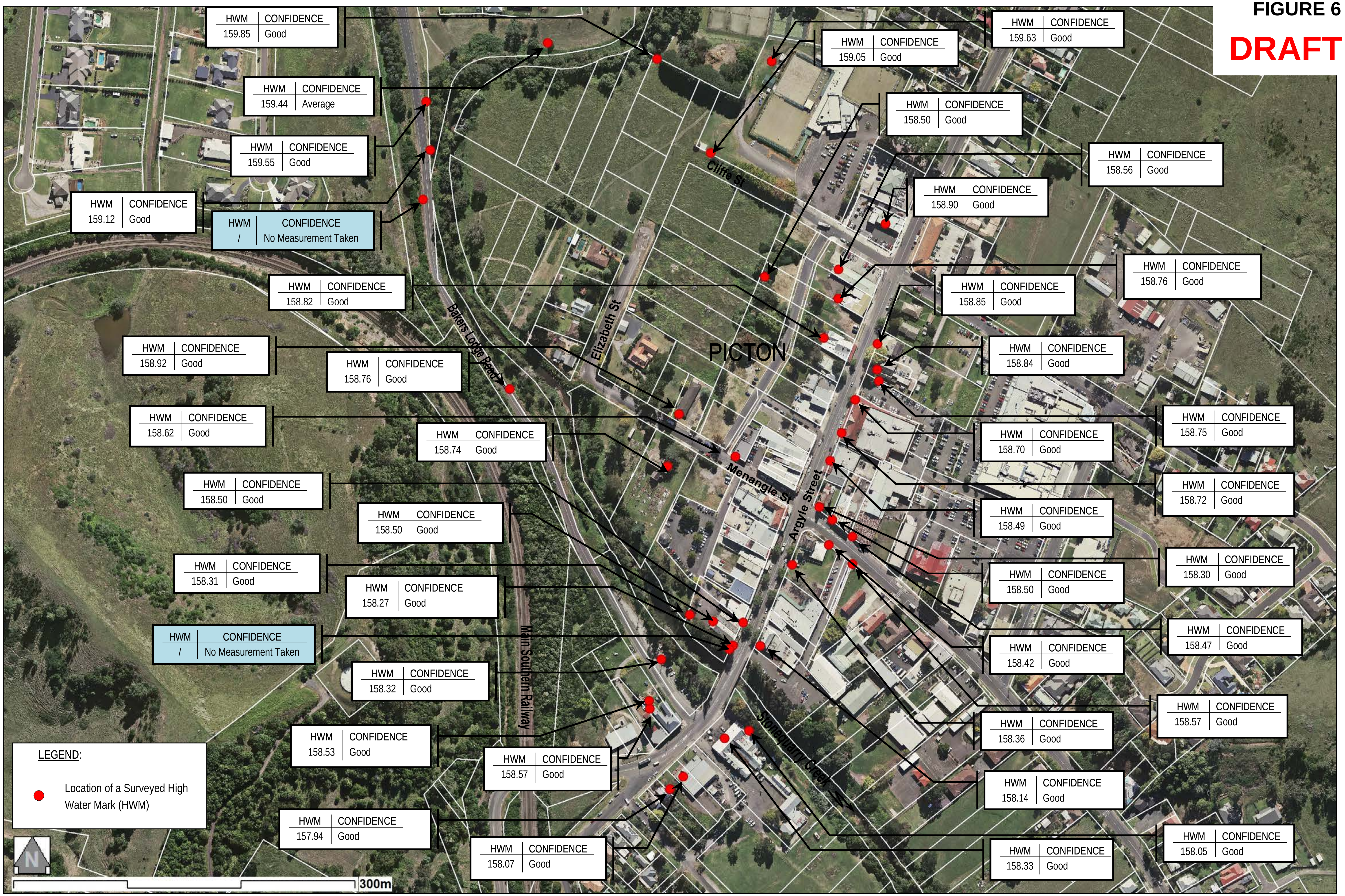
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig2_Predicted Flood levels (100yr_North).doc

JUNE 2016 HIGH WATER MARKS
[SHEET 1 OF 3]

FIGURE 6

DRAFT



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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig2_Predicted Flood levels (100yr_North).doc

JUNE 2016 HIGH WATER MARKS
[EXTENT 2 OF 3]

FIGURE 7

DRAFT

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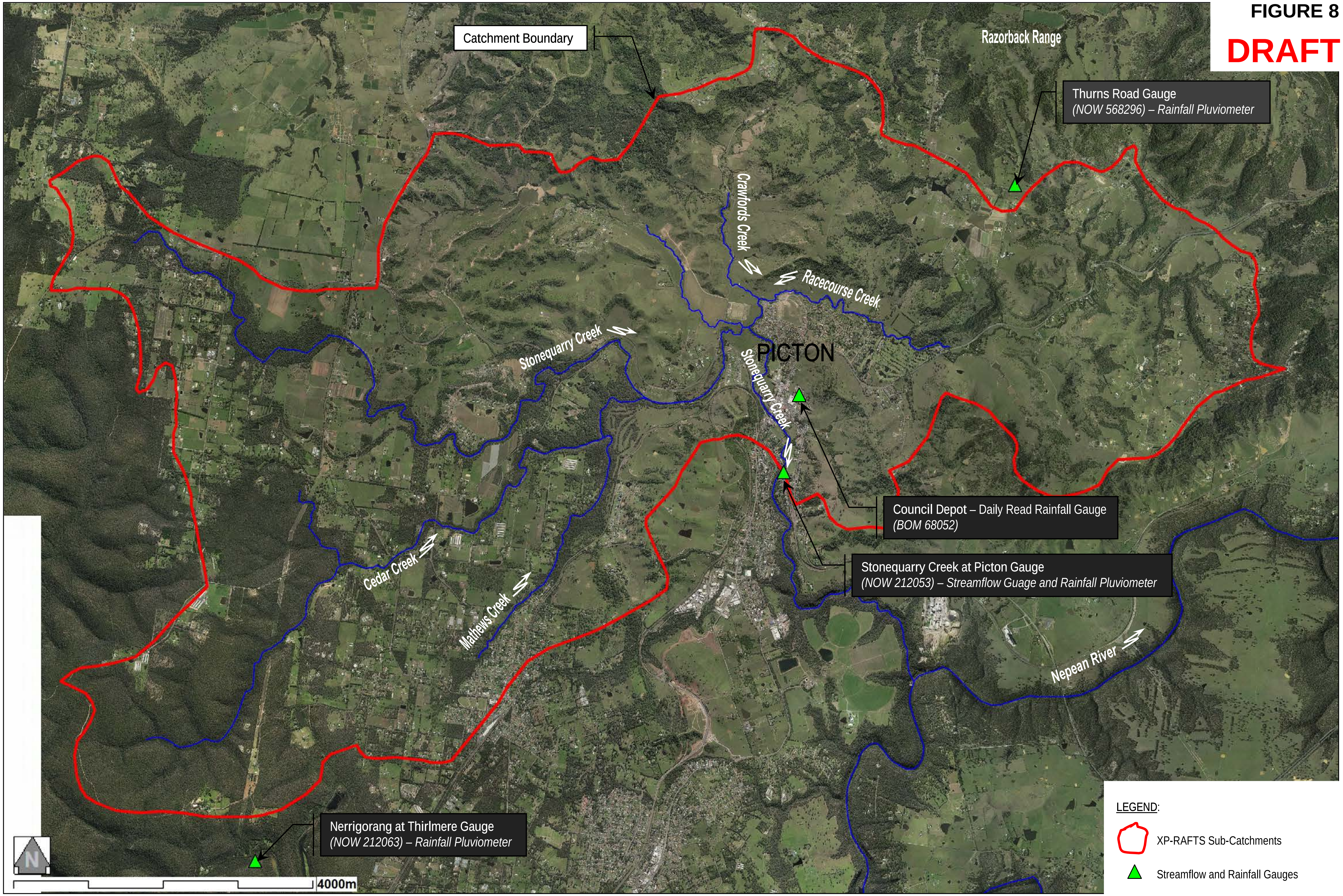
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig2_Predicted Flood levels (100yr_North).doc

JUNE 2016 HIGH WATER MARKS
[EXTENT 3 OF 3]

FIGURE 8

DRAFT



Advisian
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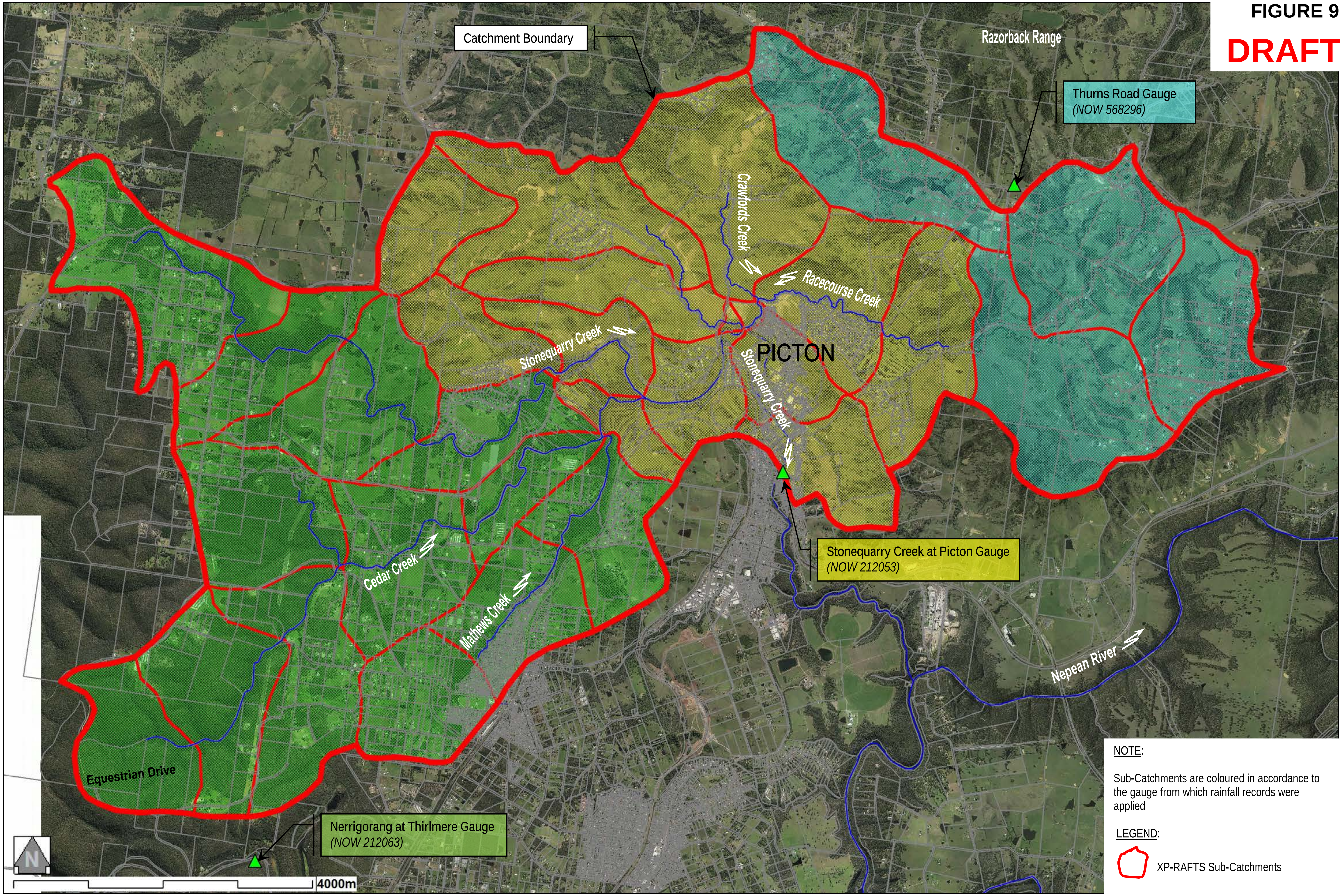
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg161013_Fig8_Streamflow and Rainfall Guages.doc

LOCATION OF NEARBY STREAMFLOW AND RAINFALL GAUGES



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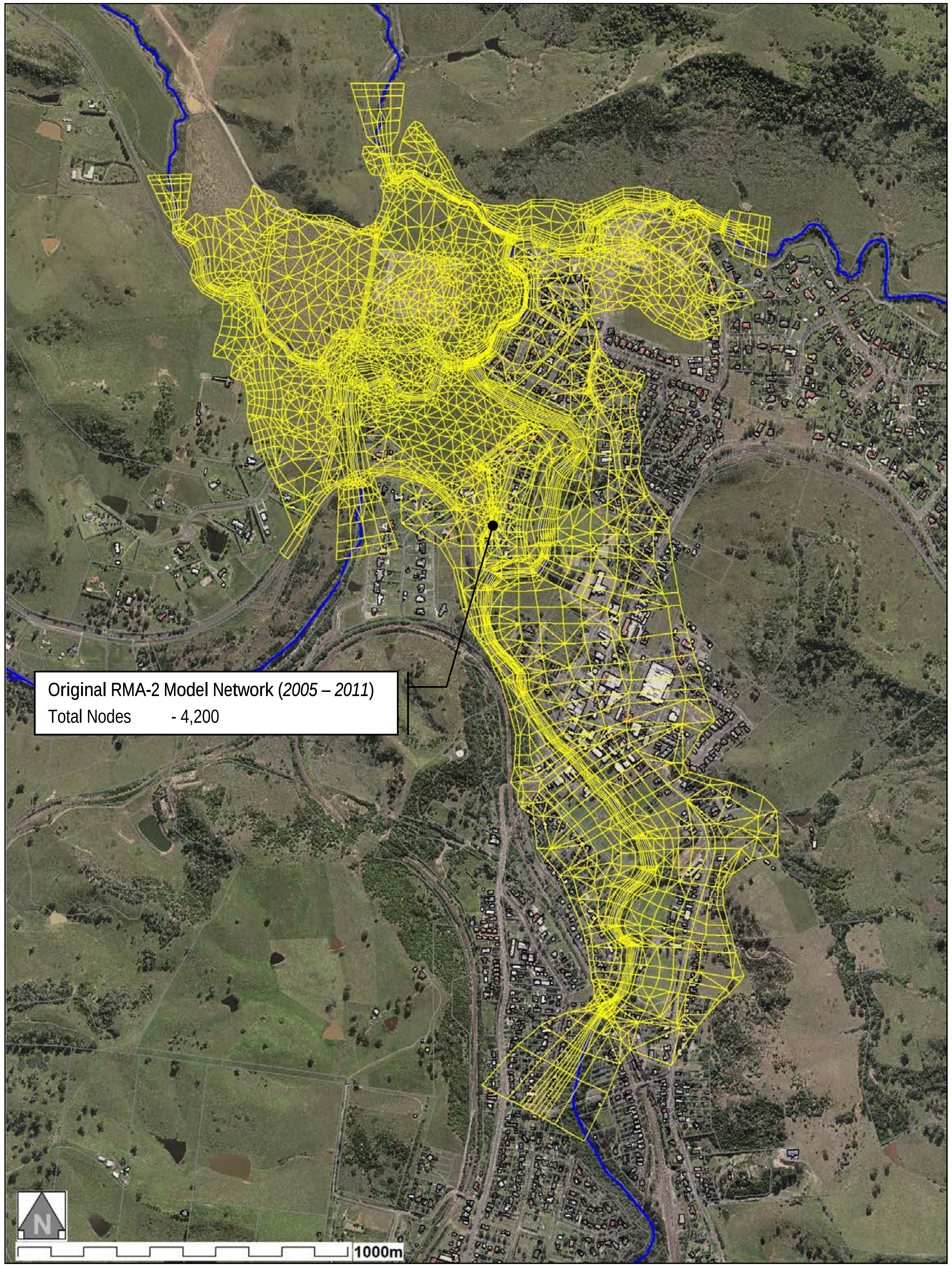
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg161013_Fig2_Adopted Gauge to Catchment Breaup.doc

JUNE 2016 GAUGE RAINFALL RECORDS ADOPTED FOR XP-RAFTS SUB-CATCHMENTS FOR VALIDATION



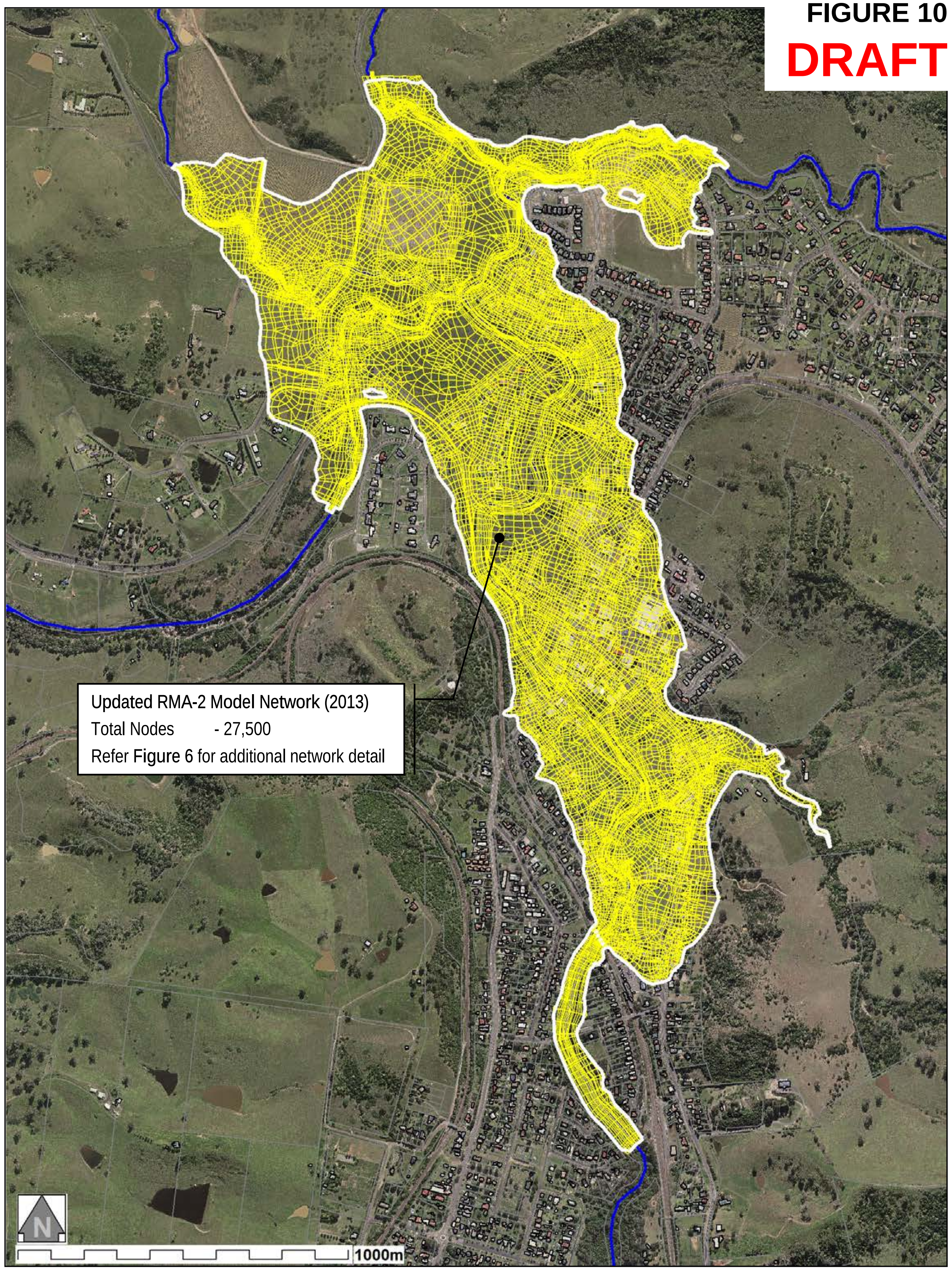


FIGURE 10

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COMPARISON OF PRE 2013 AND POST 2013 RMA-2 MODEL NETWORKS

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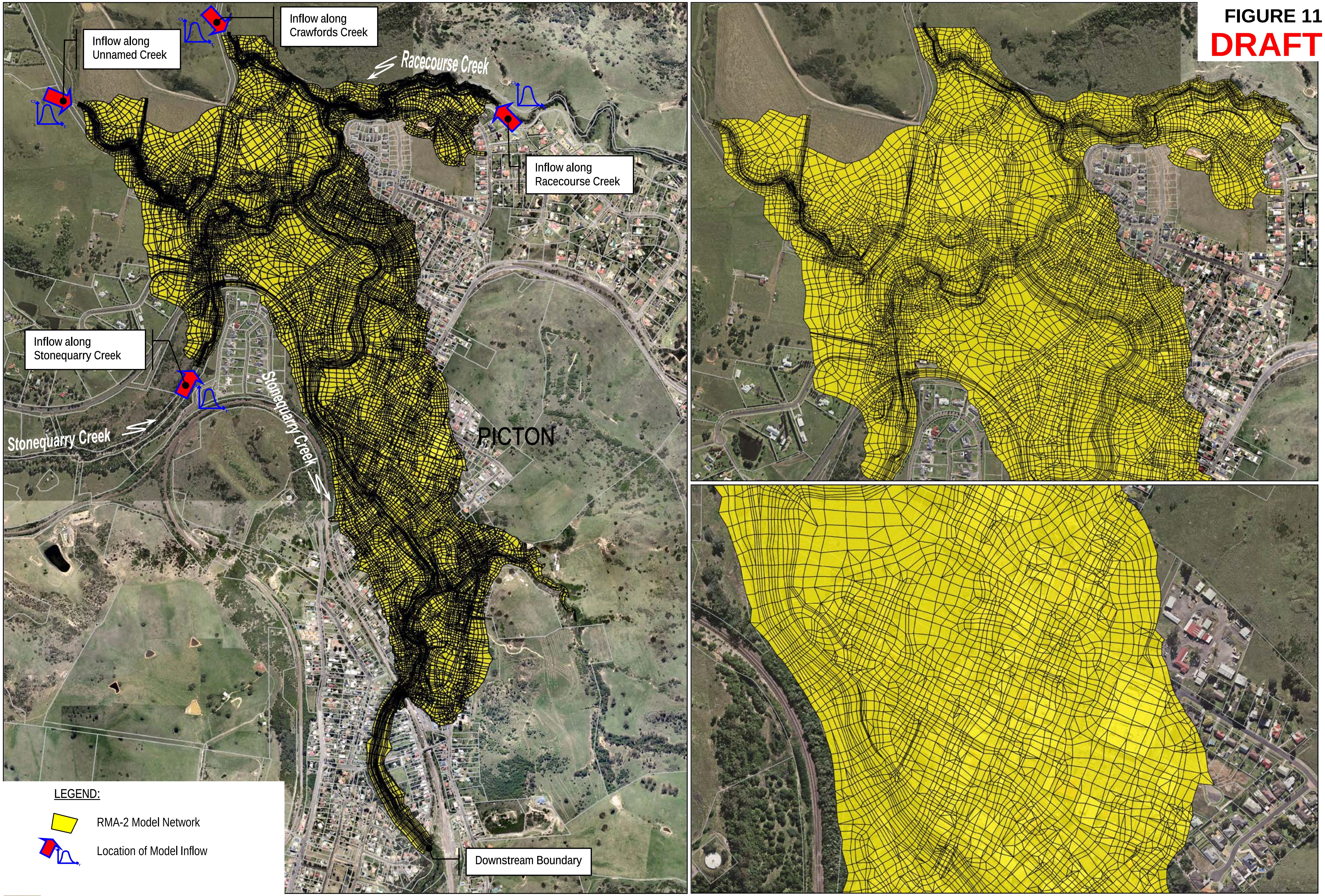
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140109_Fig10_Network Comparison (Old vs New).doc



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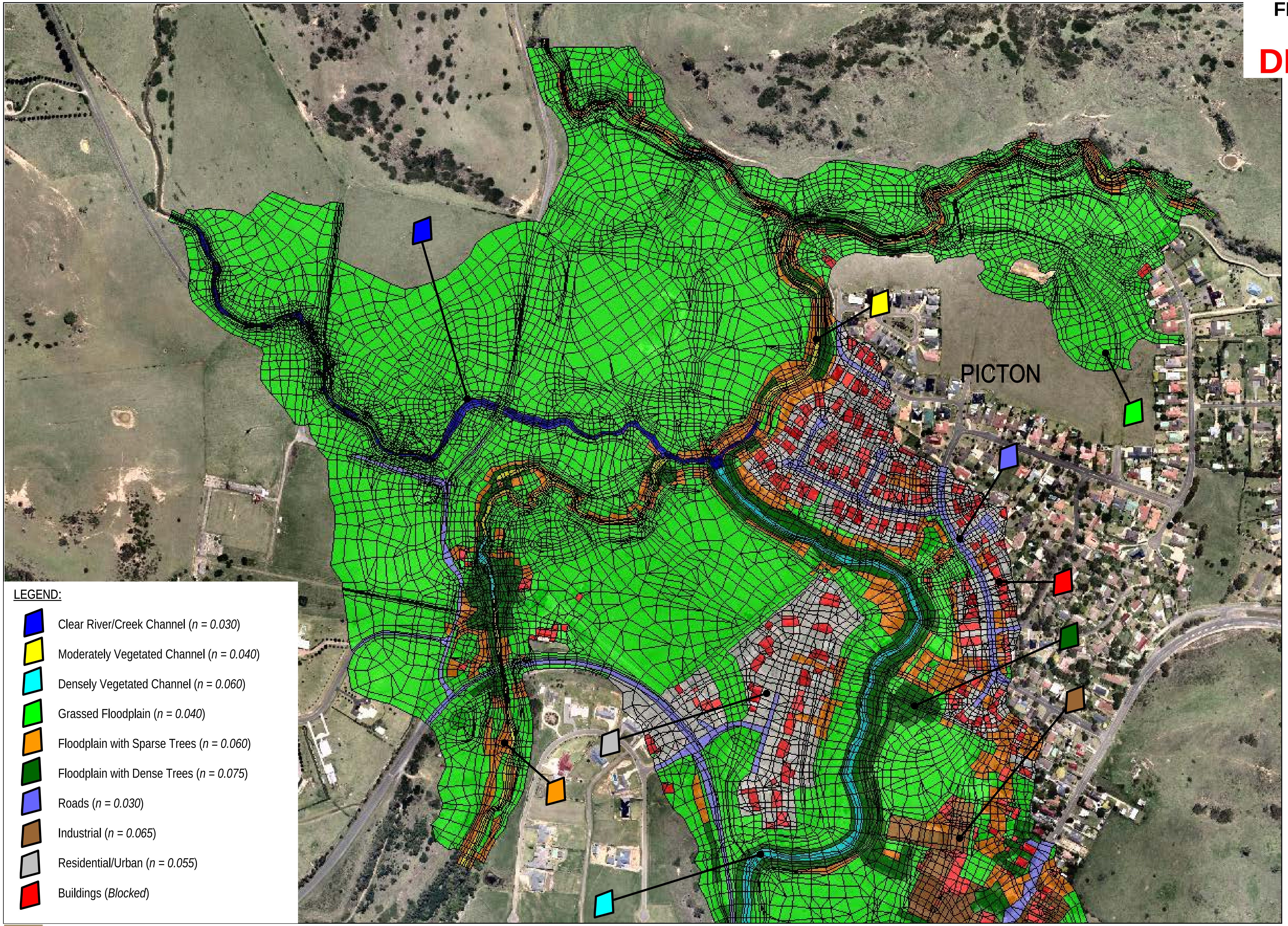
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig11_Updated RMA-2 Model Network.doc

EXTENT & LAYOUT OF THE UPDATED RMA-2 MODEL NETWORK

FIGURE 12

DRAFT



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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140106_Fig12_Element Type Distribution (North 1 of 3).doc

RMA-2 ELEMENT TYPES AND DISTRIBUTION
[NORTHERN SHEET 1 of 3]

FIGURE 13

DRAFT

RMA-2 ELEMENT TYPES AND DISTRIBUTION
[CENTRAL SHEET 2 of 3]

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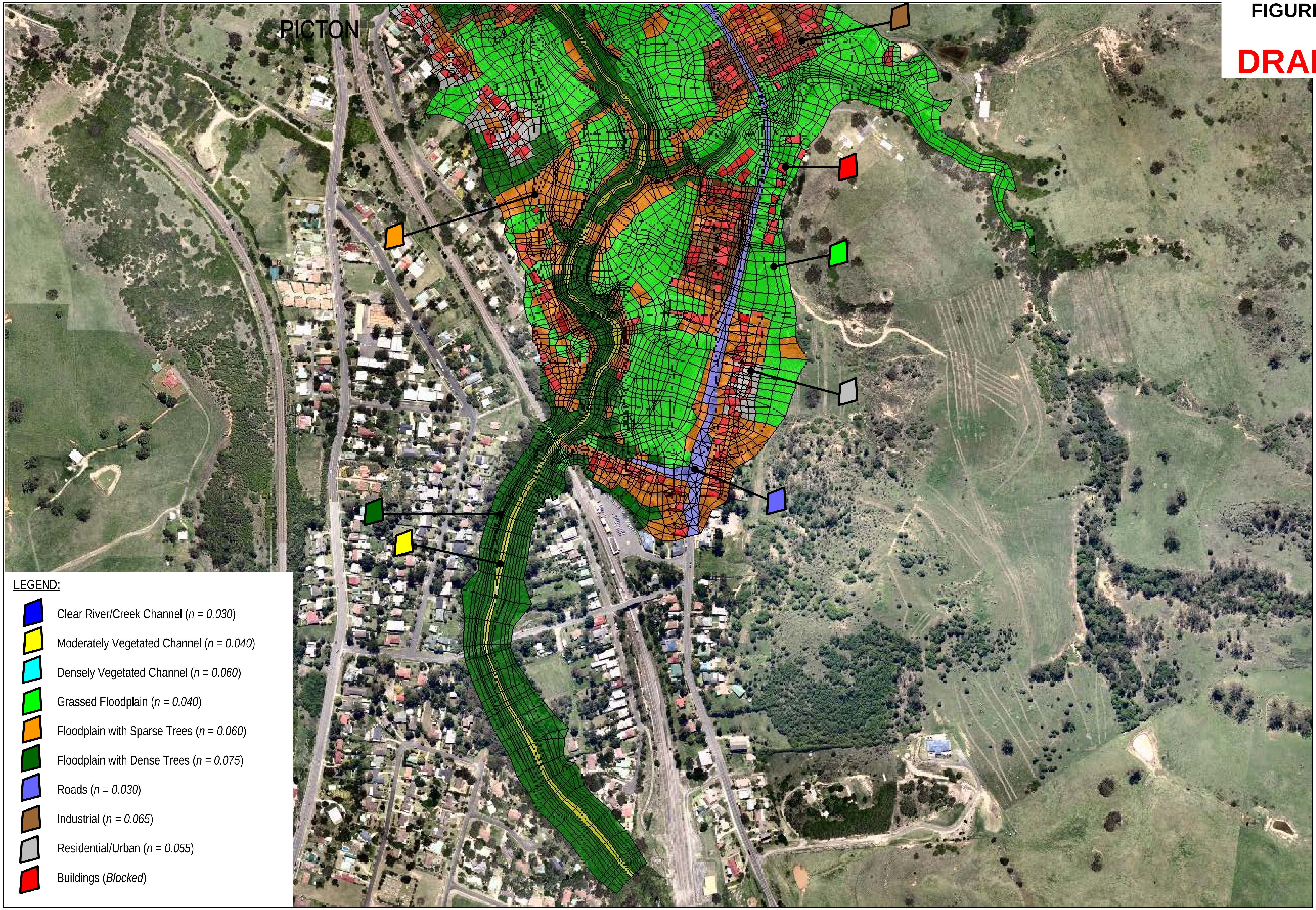
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140106_Fig13_Element Type Distribution (Centre 2 of 3).doc

FIGURE 14

DRAFT



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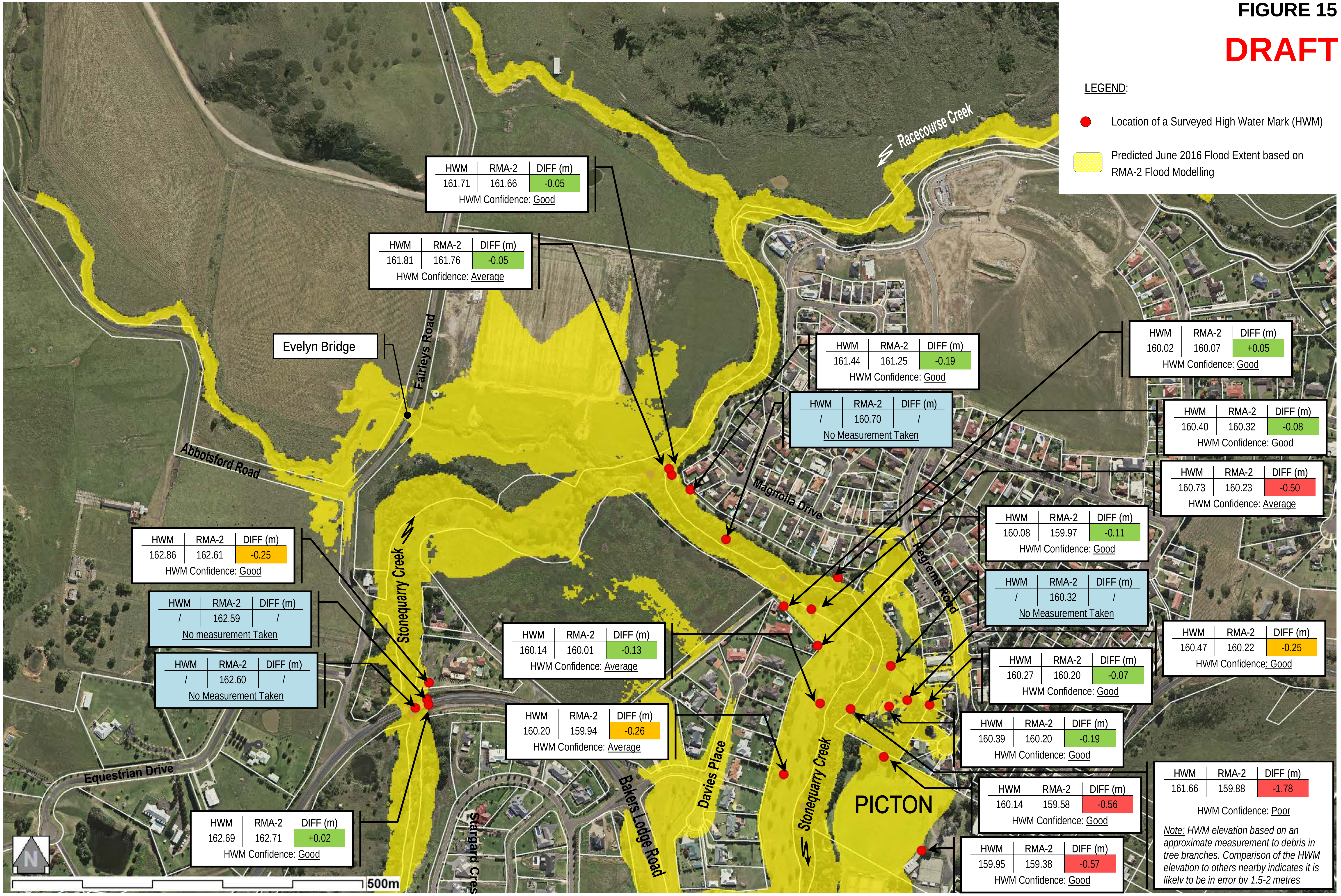
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140106_Fig14_Element Type Distribution (South 3 of 3).doc

RMA-2 ELEMENT TYPES AND DISTRIBUTION
[SOUTHERN SHEET 3 of 3]

FIGURE 15

DRAFT



Note: Shading representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Orange = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF RMA-2 MODELLED LEVELS FOR THE 'JUNE 2016' FLOOD EVENT TO SURVEYED HWMs
[EXTENT 1 OF 3]

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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig2_Predicted Flood levels (100yr_North).doc

FIGURE 16

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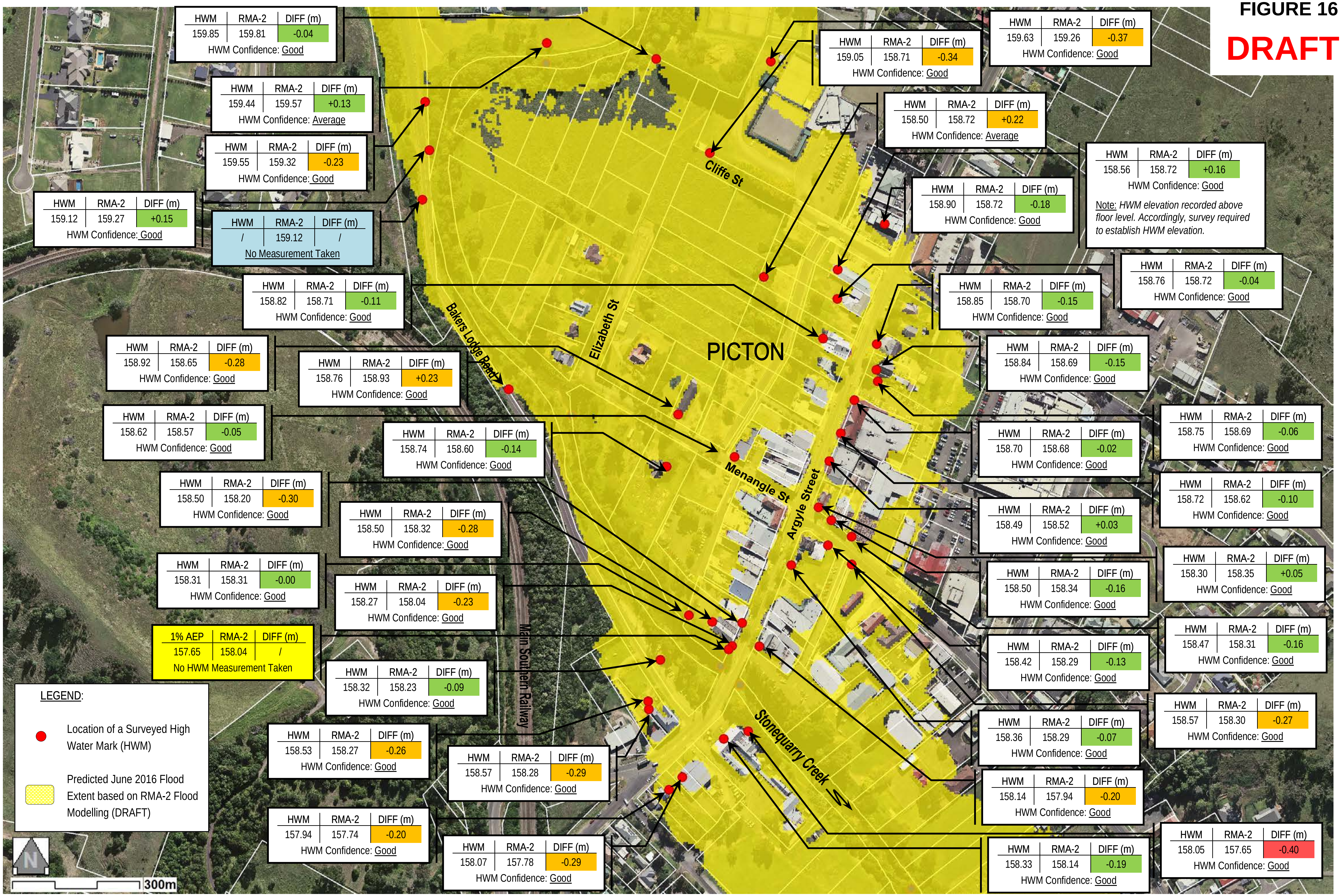

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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig2_Predicted Flood levels (100yr_North).doc

Note: Shading representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Orange = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF RMA-2 MODELLED LEVELS FOR THE 'JUNE 2016' FLOOD EVENT TO SURVEYED HWMs
[EXTENT 2 OF 3]

FIGURE 17

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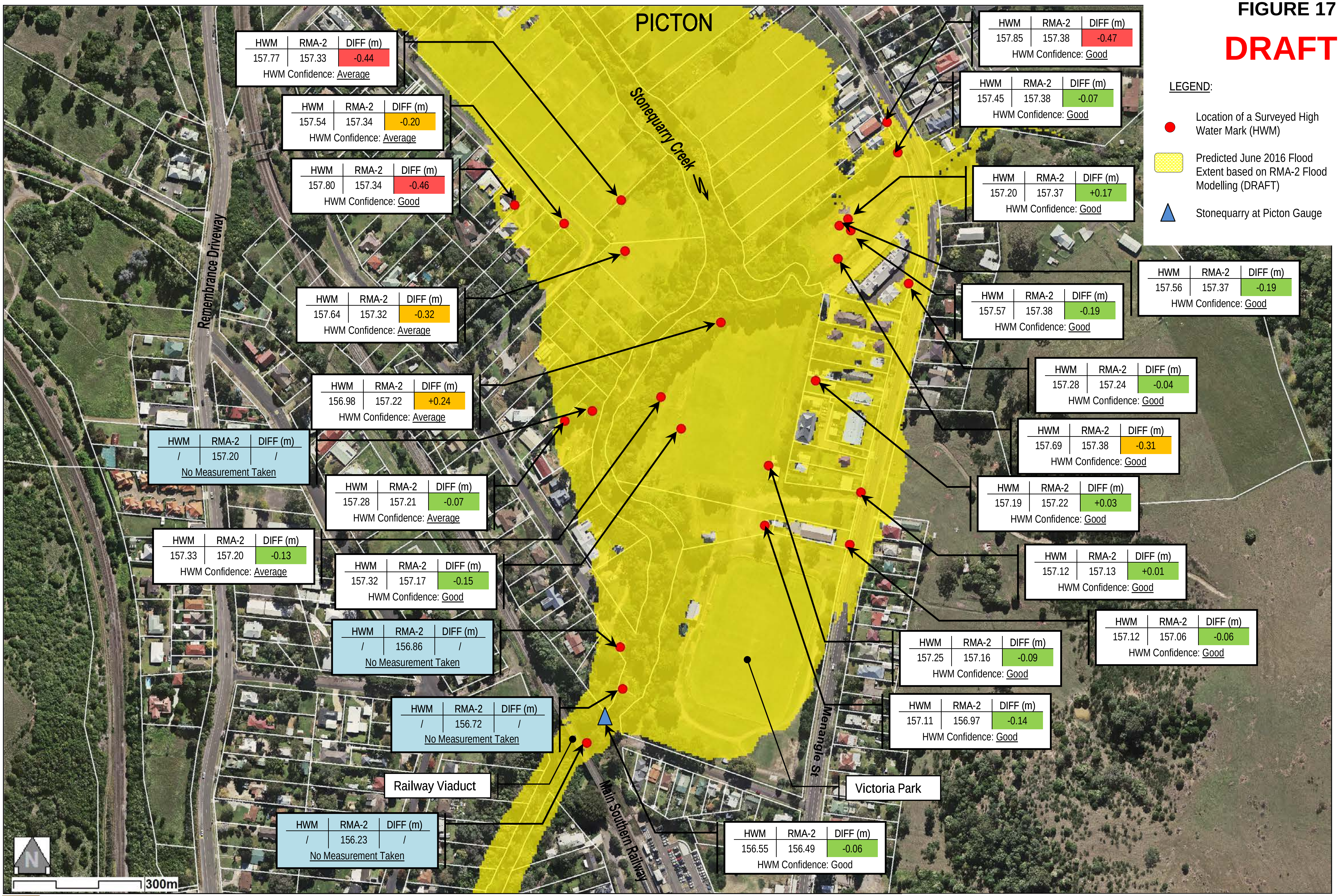

Note: Shading representative of magnitude of WL difference between observed and modelled flood levels.
Green = 0 – 200 mm difference, Orange = 200 – 400 mm difference, Red = 400+ mm

COMPARISON OF RMA-2 MODELLED LEVELS FOR THE 'JUNE 2016' FLOOD EVENT TO SURVEYED HWMs
[EXTENT 3 OF 3]

Advisian
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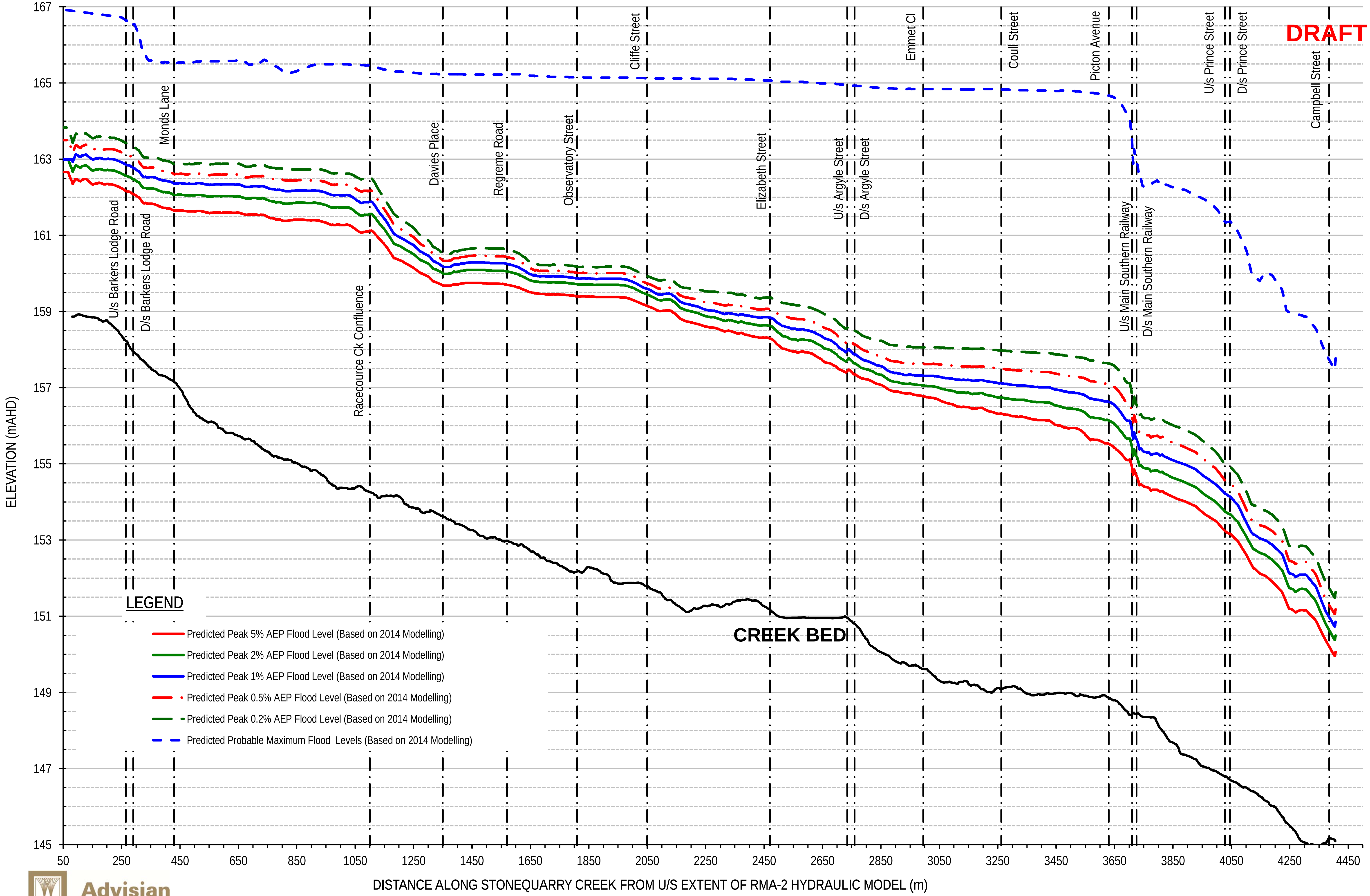
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig2_Predicted Flood levels (100yr_North).doc

FIGURE 18



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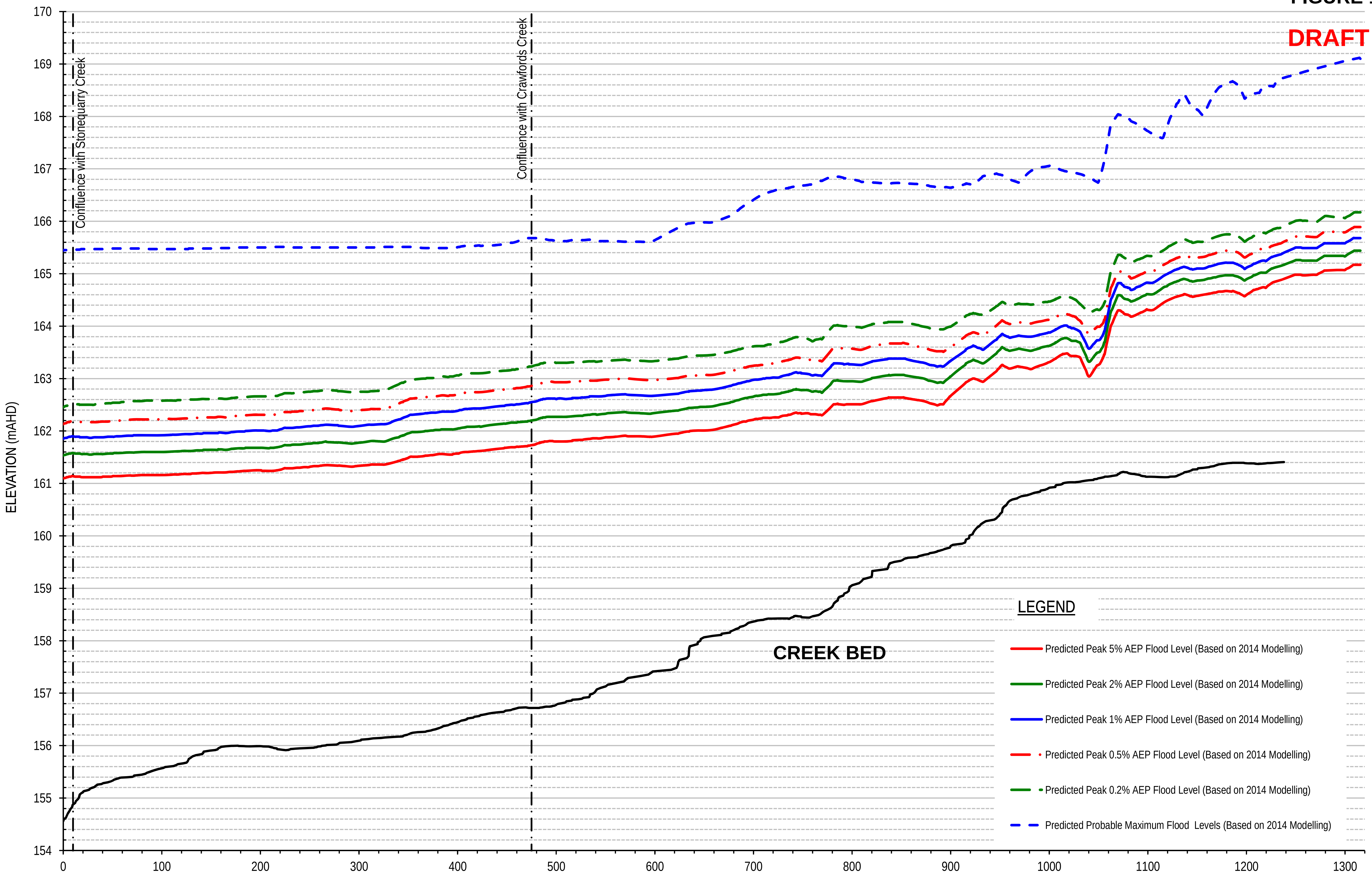
301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

PROFILES FOR 1% AEP SCENARIOS ALONG STONEQUARRY CREEK



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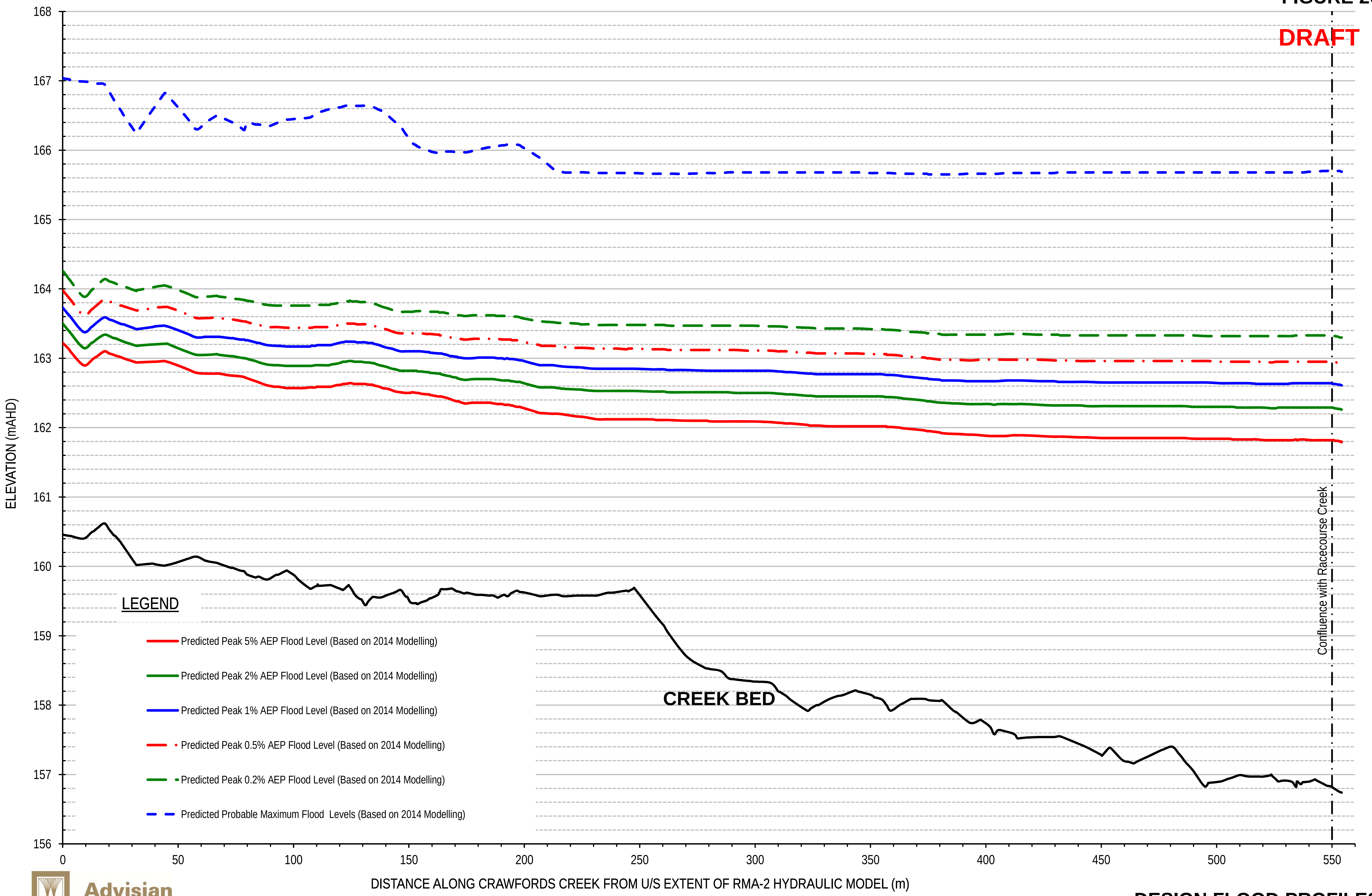
301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

DESIGN FLOOD PROFILES ALONG RACECOURSE CREEK

FIGURE 20



DESIGN FLOOD PROFILES ALONG CRAWFORDS CREEK

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301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

FIGURE 21

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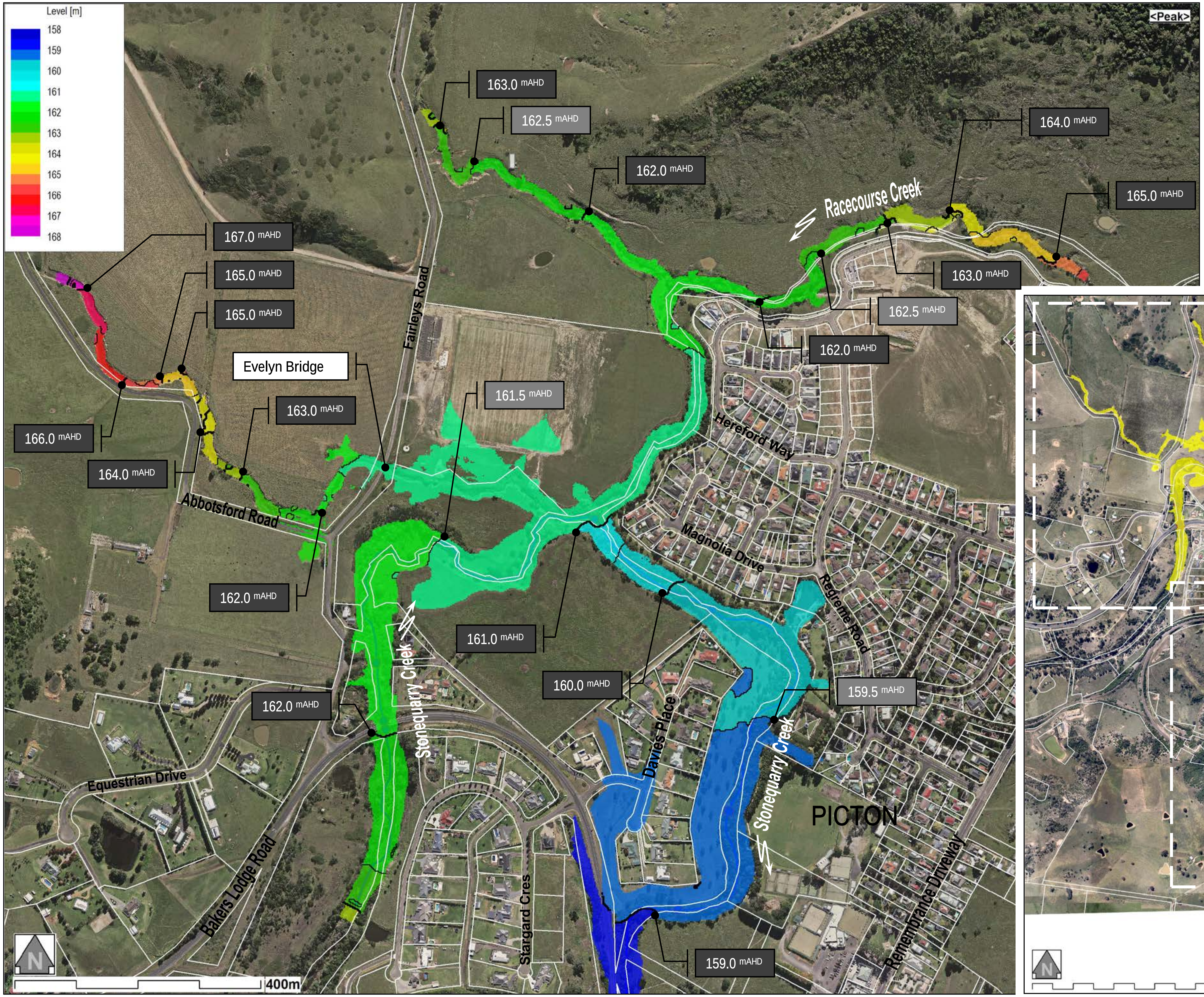
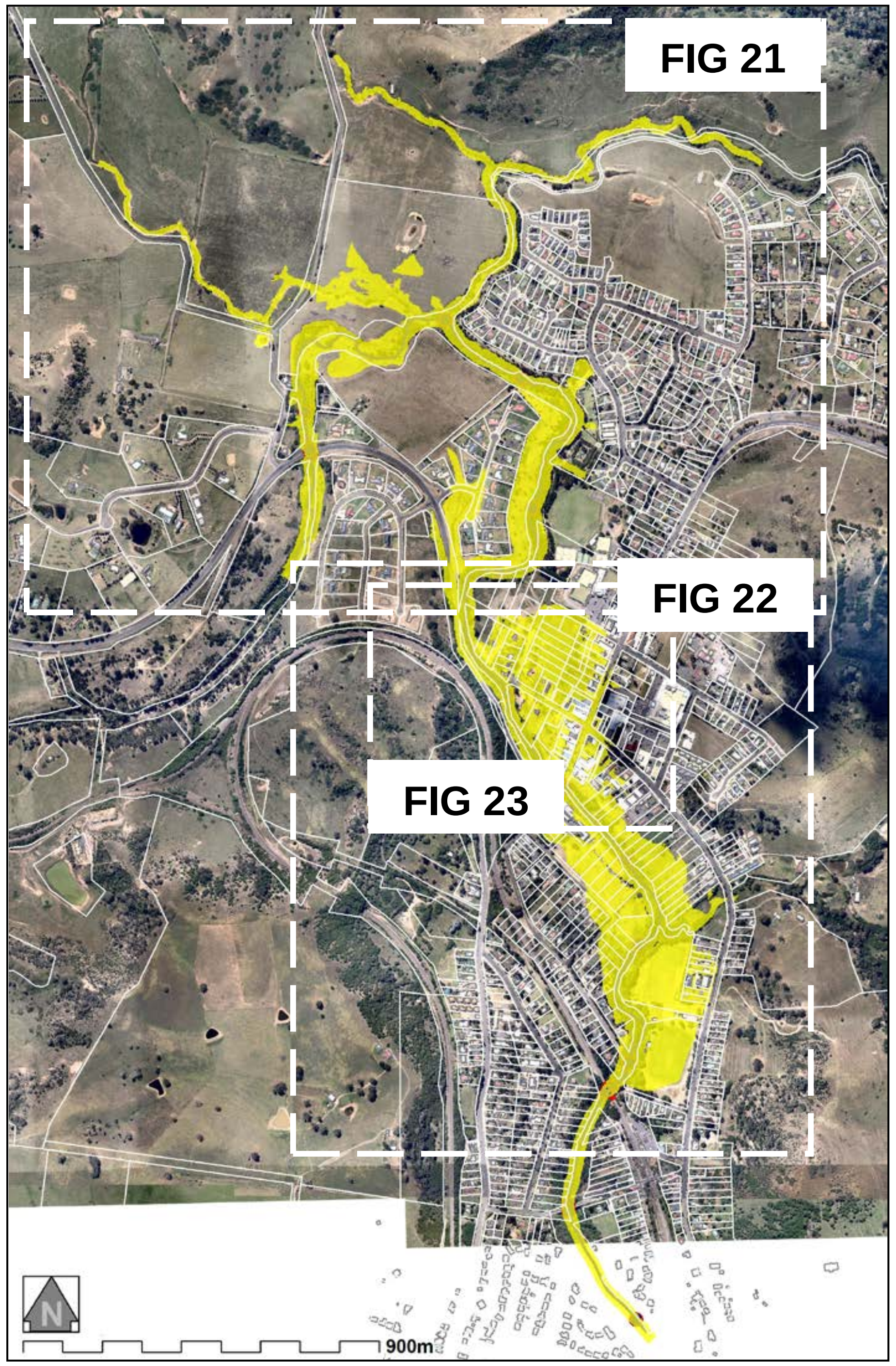


LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.5 metre Interval



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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig21_Predicted Flood levels (5% AEP_North).doc

PREDICTED FLOOD LEVELS AT THE PEAK OF THE 5% AEP FLOOD – SHEET 1 OF 3 [UPDATED RMA-2 FLOOD MODEL]

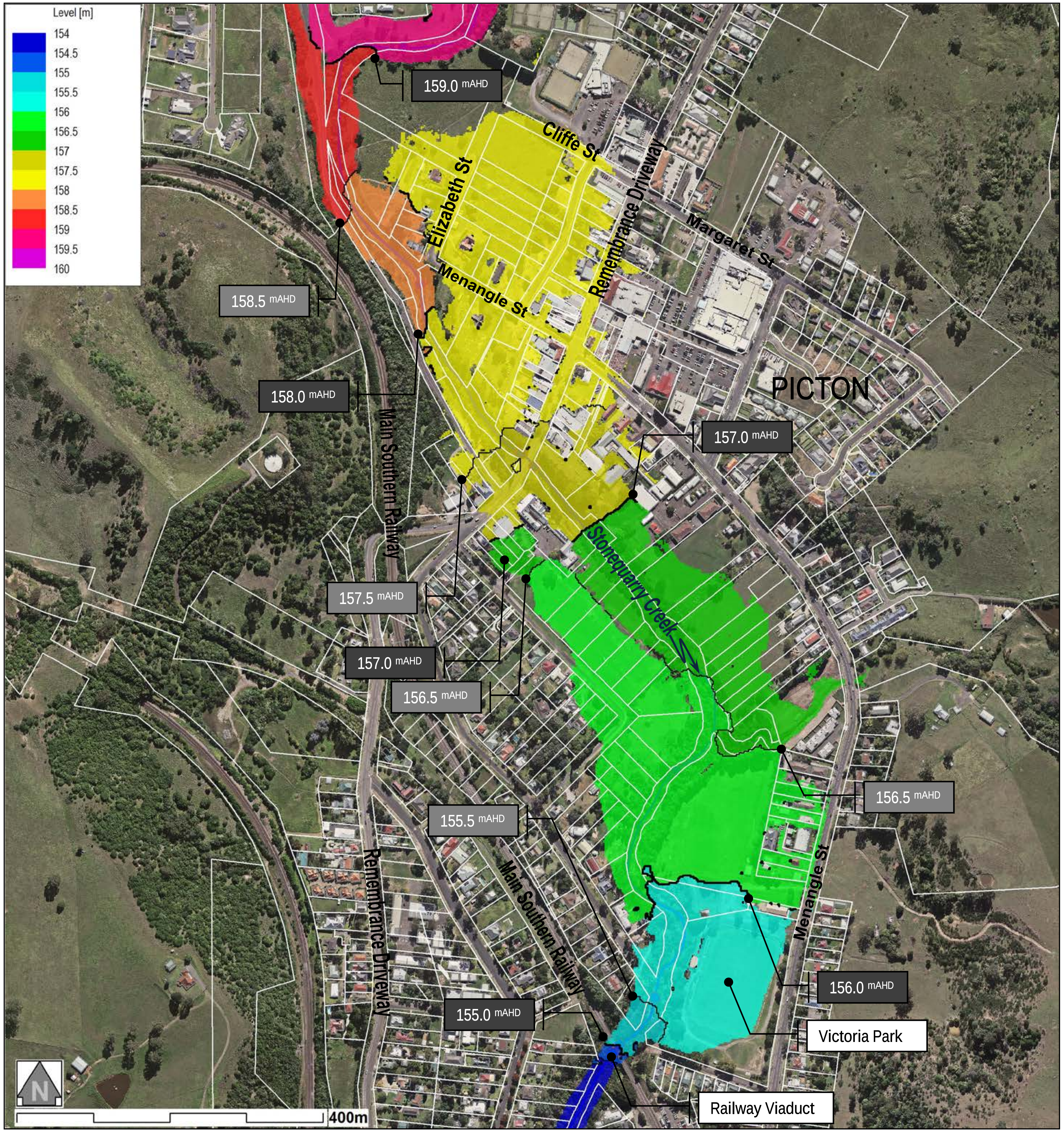
FIGURE 22

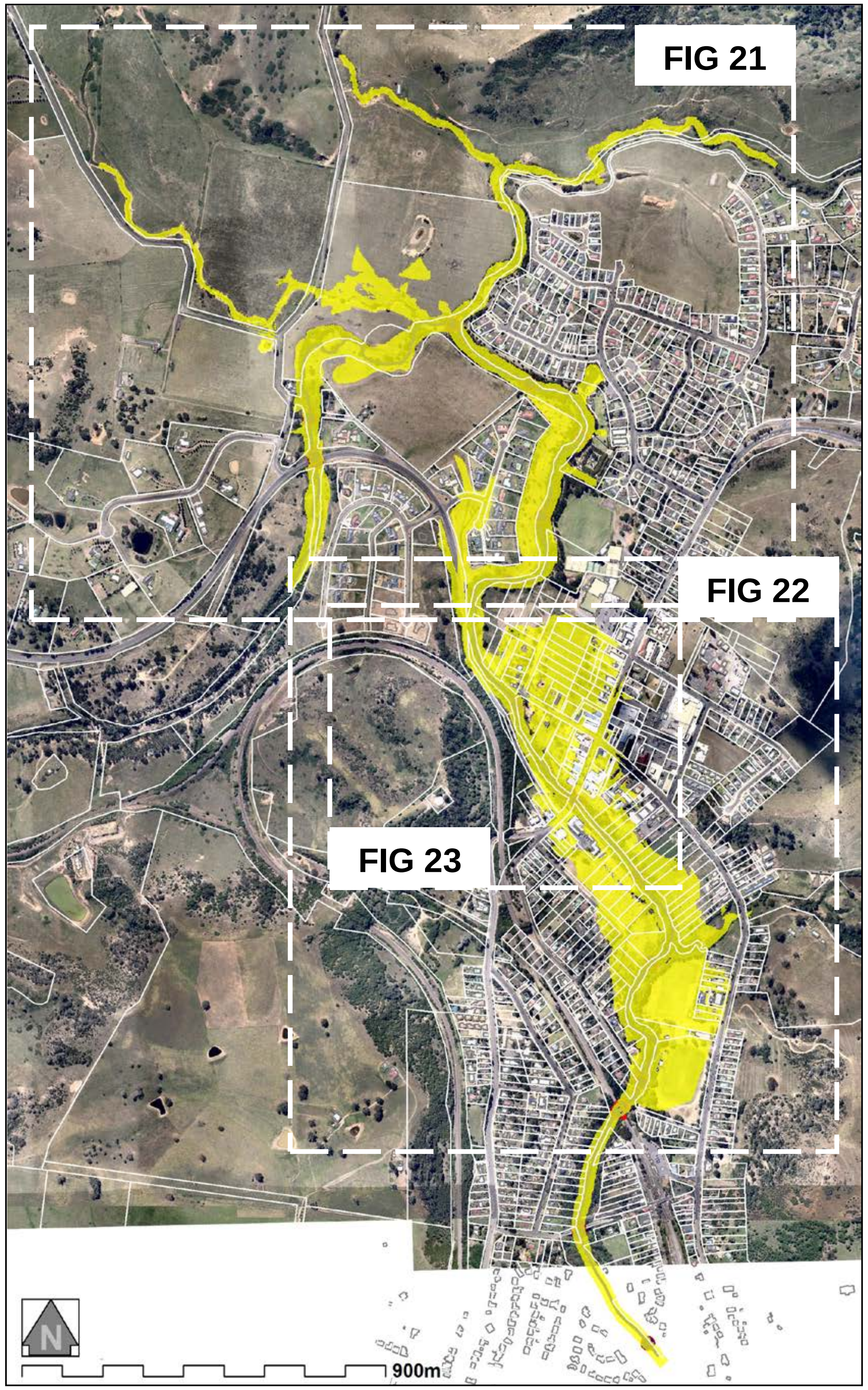
DRAFT

LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.5 metre Interval



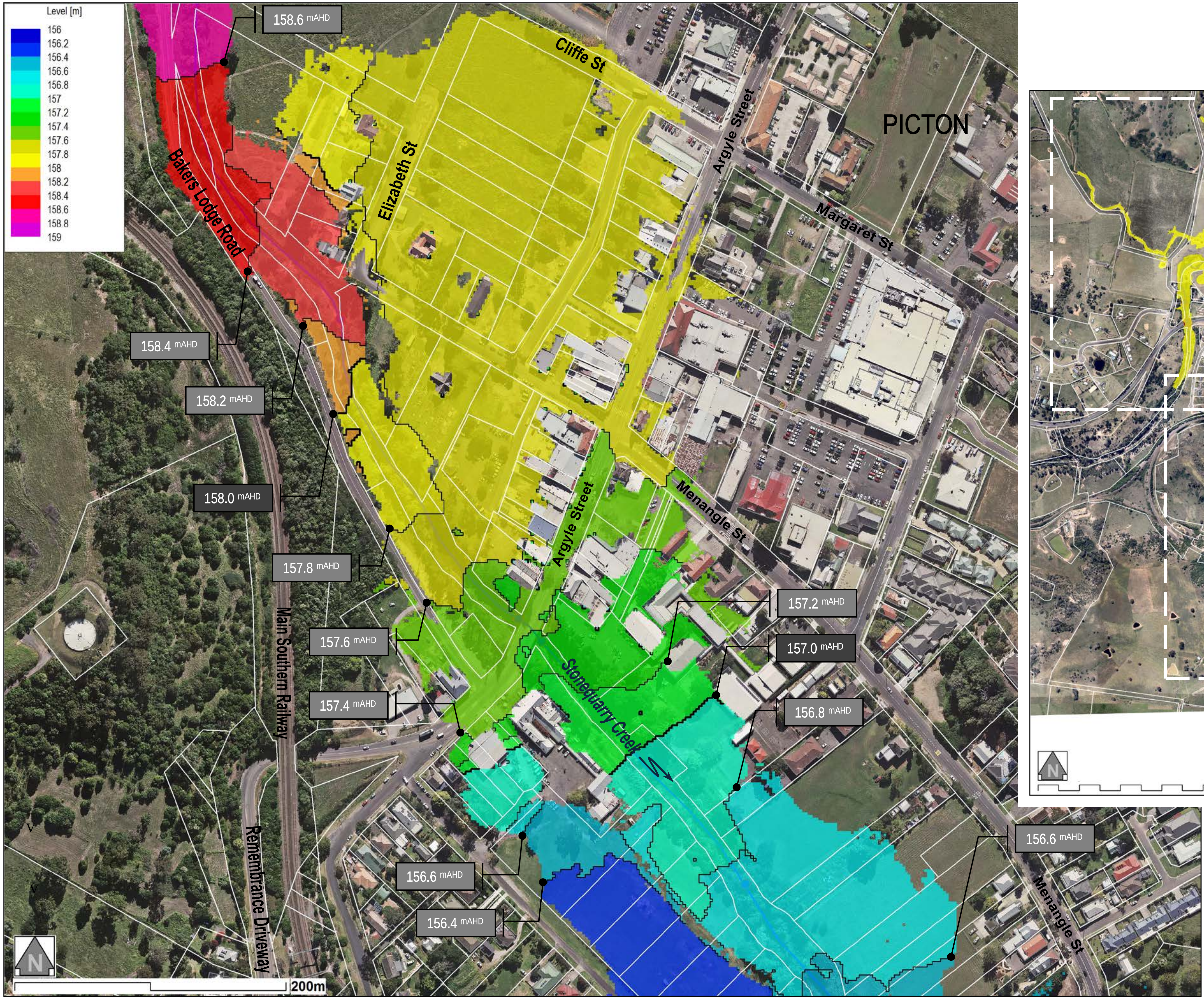


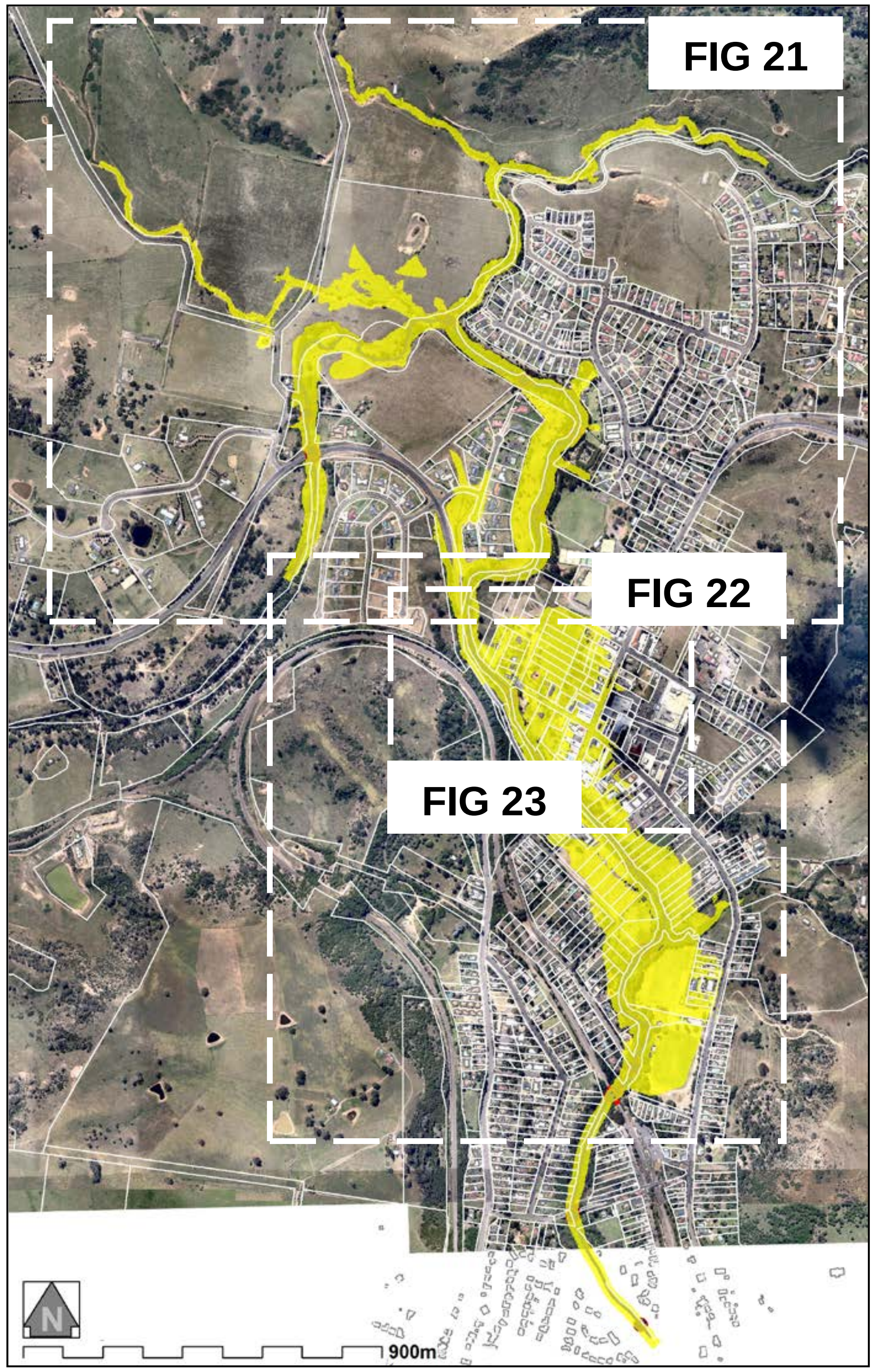
PREDICTED FLOOD LEVELS AT THE PEAK OF THE 5% AEP FLOOD – SHEET 2 OF 3 [UPDATED RMA-2 FLOOD MODEL]

Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig22_Predicted Flood levels (5% AEP_South).doc

FIGURE 23

DRAFT





LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.2 metre Interval

Advisian

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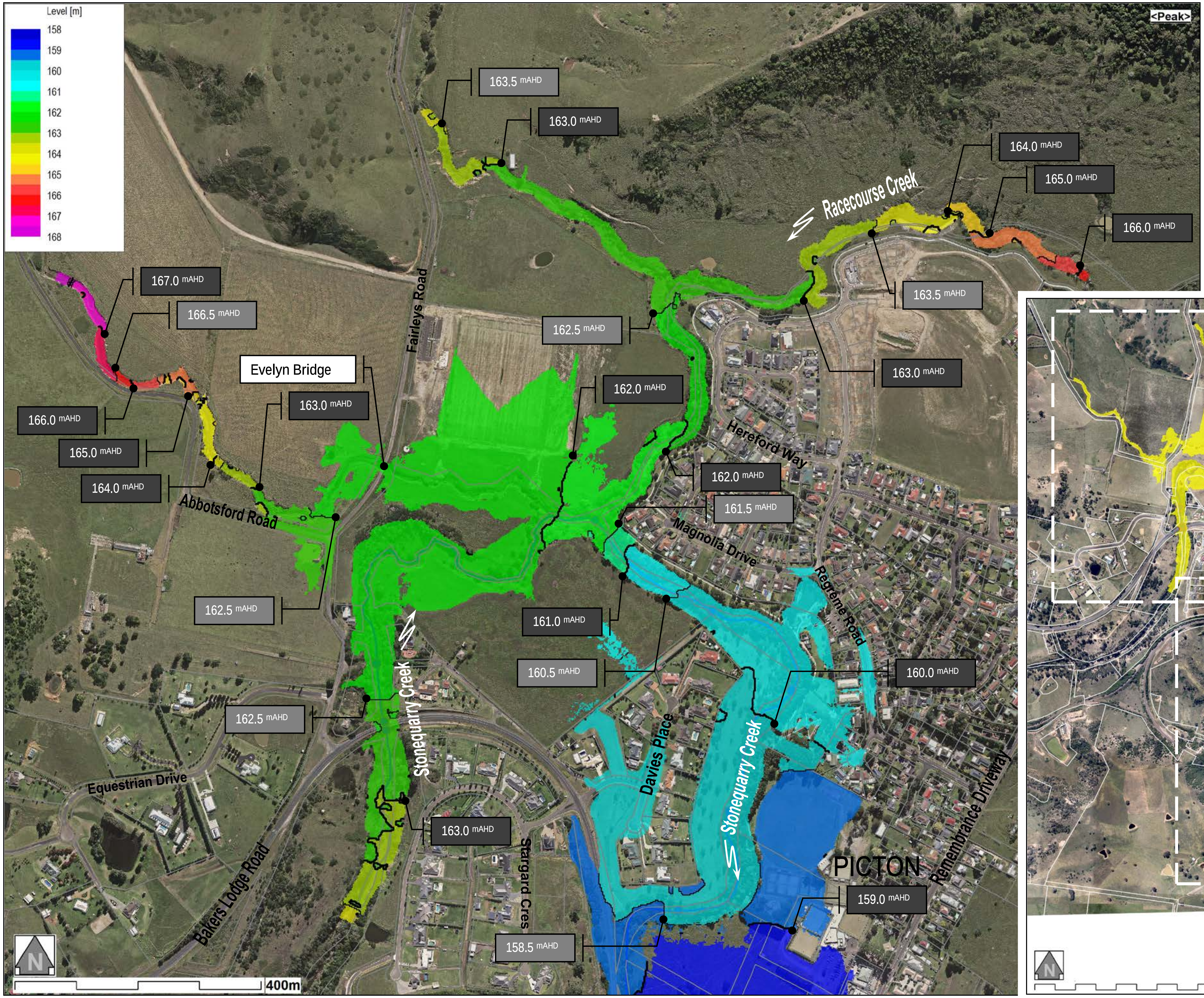
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig23_Predicted Flood levels (5% AEP_Town).doc

PREDICTED FLOOD LEVELS AT THE PEAK OF THE 5% AEP FLOOD – SHEET 3 OF 3 [UPDATED RMA-2 FLOOD MODEL]

FIGURE 24

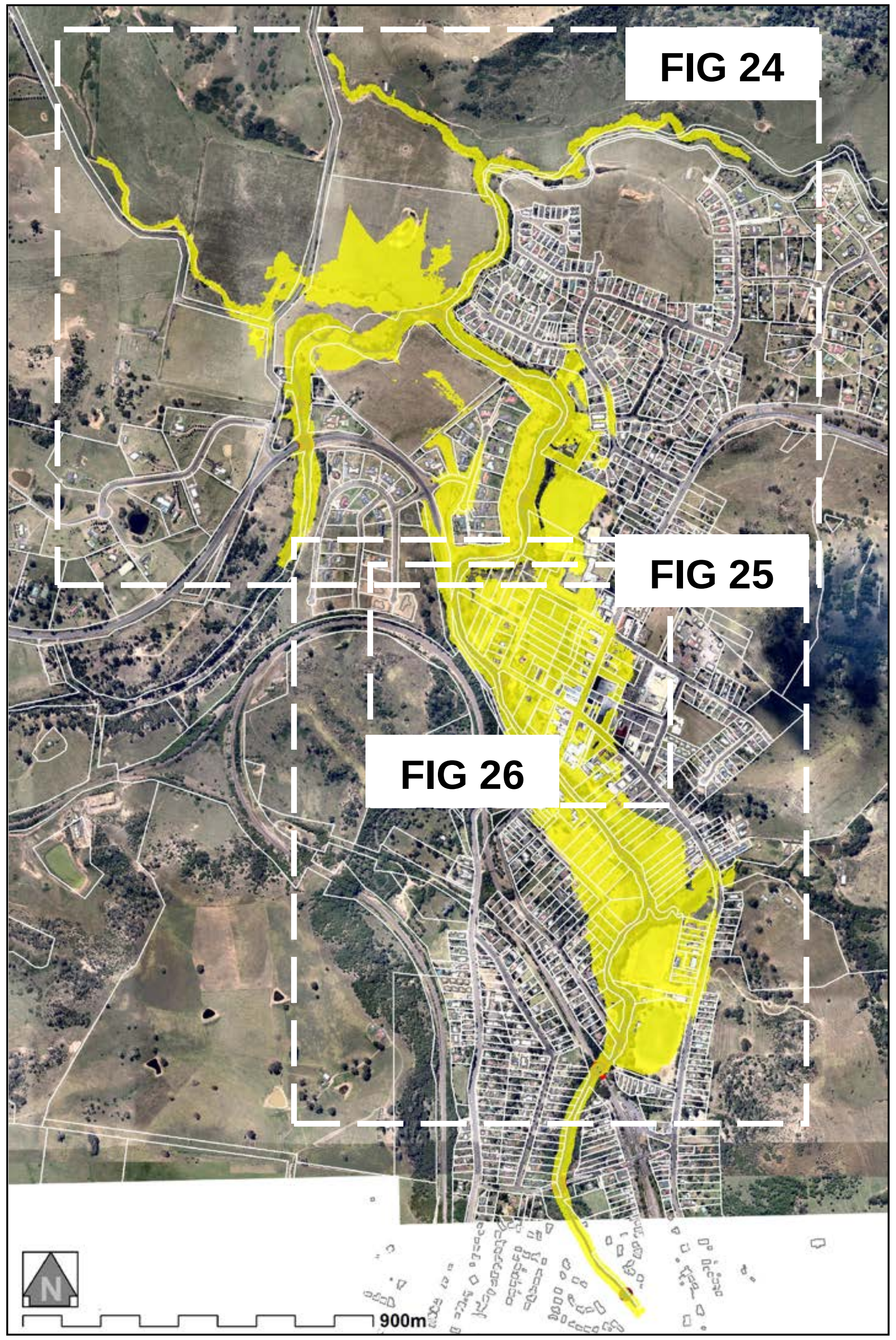
DRAFT



LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.5 metre Interval



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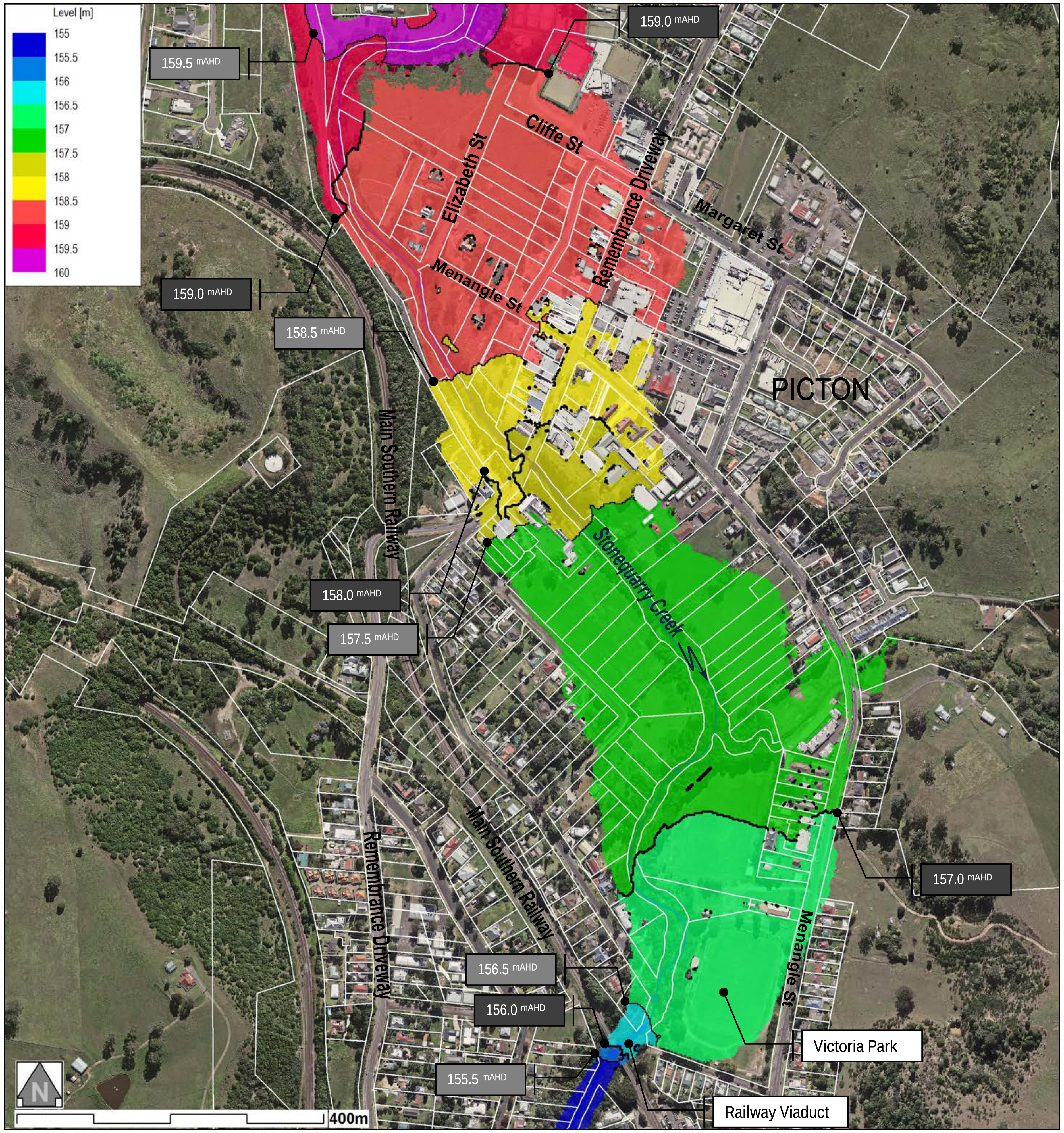
301015-03199-Stonequarry Ck Flood Modelling

fg301015-03199rg130417_Fig24_Predicted Flood levels (1% AEP_North).doc

PREDICTED FLOOD LEVELS AT THE PEAK OF THE 1% AEP ARI FLOOD – SHEET 1 OF 3 [UPDATED RMA-2 FLOOD MODEL]

FIGURE 25

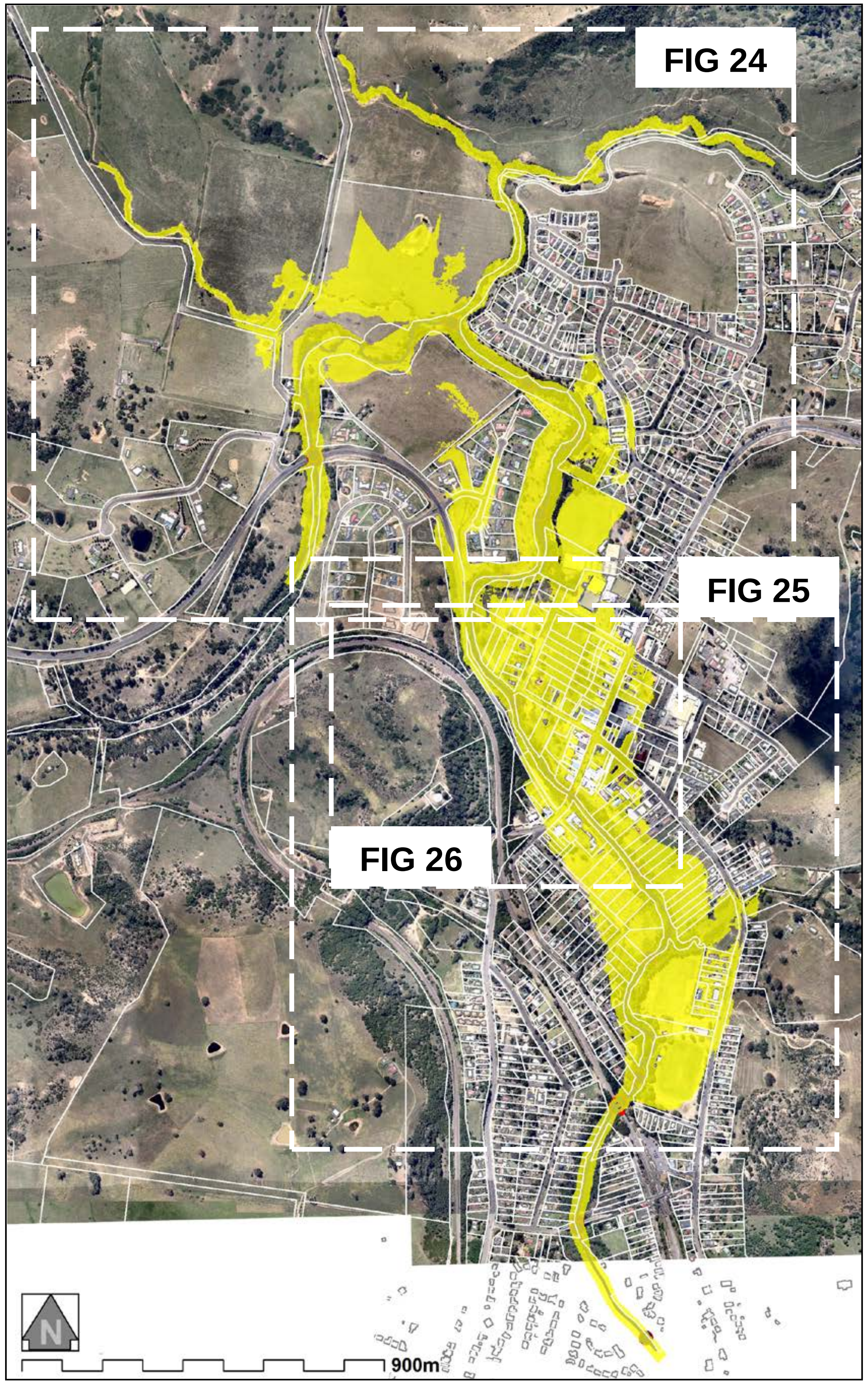
DRAFT



LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.5 metre Interval



PREDICTED FLOOD LEVELS AT THE PEAK OF THE 1% AEP FLOOD – SHEET 2 OF 3 [UPDATED RMA-2 FLOOD MODEL]

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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig25_Predicted Flood levels (1% AEP_South).doc

FIGURE 26

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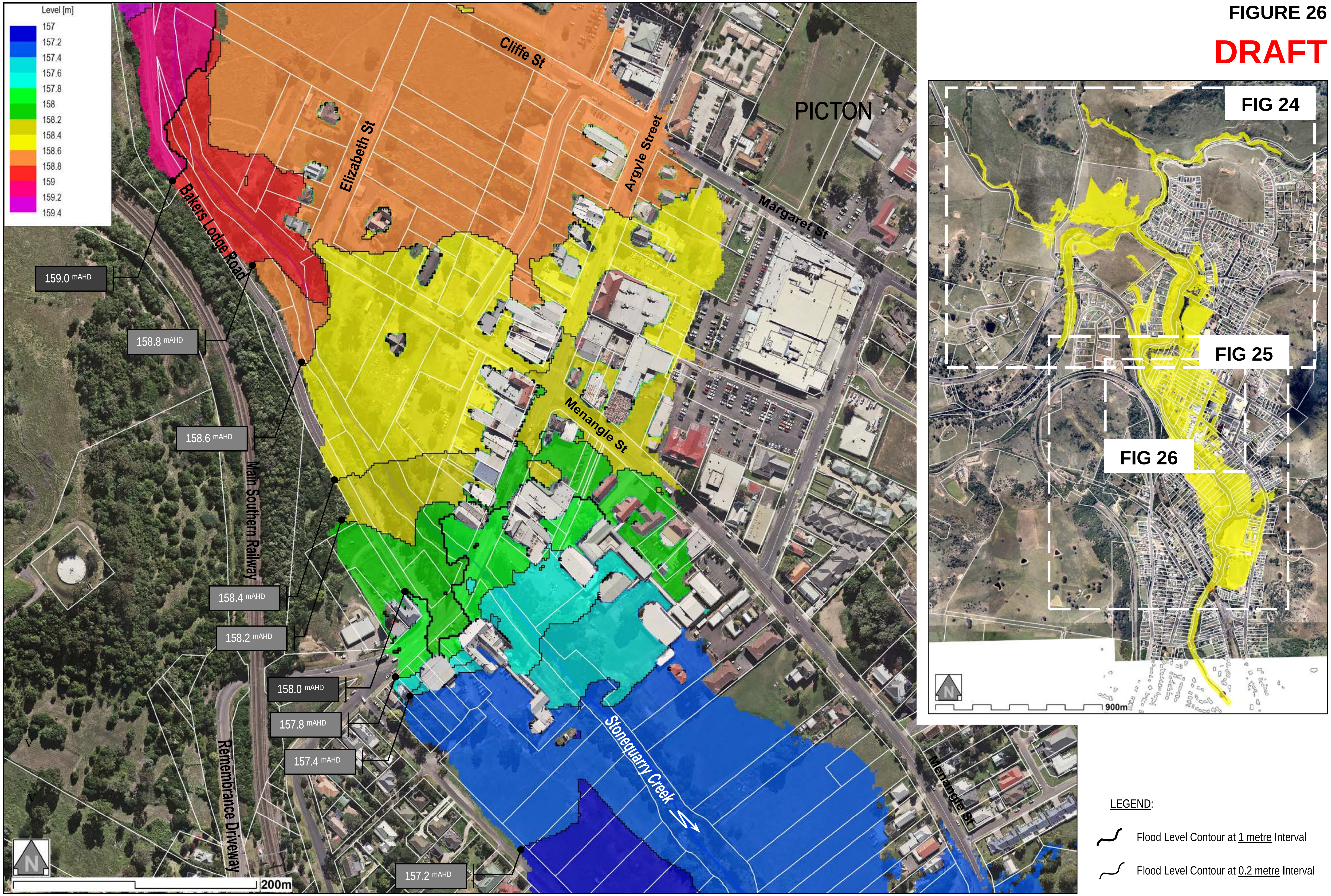

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301015-03199-Stonequarry Ck Flood Modelling

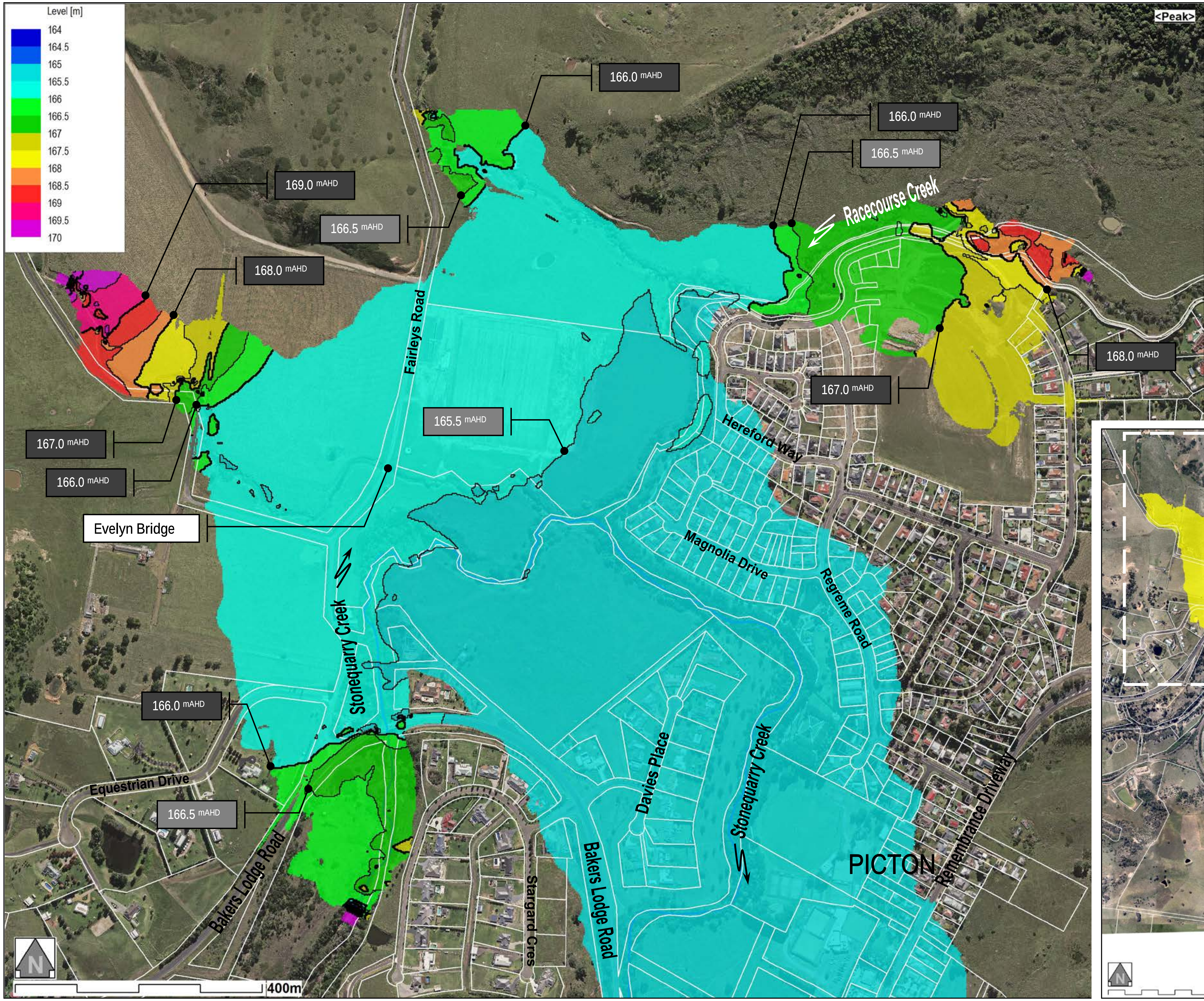
fg301015-03199rg130417_Fig26_Predicted Flood levels (1% AEP_Town).doc

PREDICTED FLOOD LEVELS AT THE PEAK OF THE 1% AEP FLOOD – SHEET 3 OF 3

[UPDATED RMA-2 FLOOD MODEL]

FIGURE 27

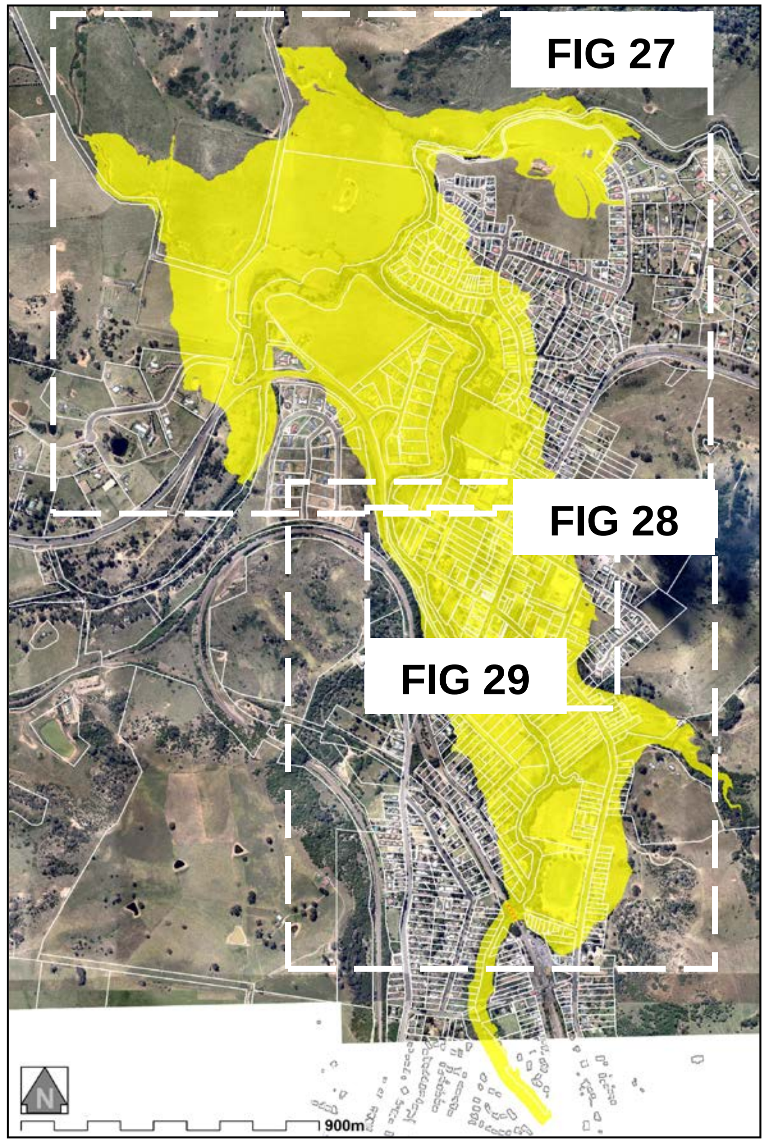
DRAFT



LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.5 metre Interval



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301015-03199-Stonequarry Ck Flood Modelling

fg301015-03199rg130417_Fig27_Predicted Flood levels (100yr_North).doc

PROBABLE MAXIMUM FLOOD LEVELS – SHEET 1 OF 3
[UPDATED RMA-2 FLOOD MODEL]

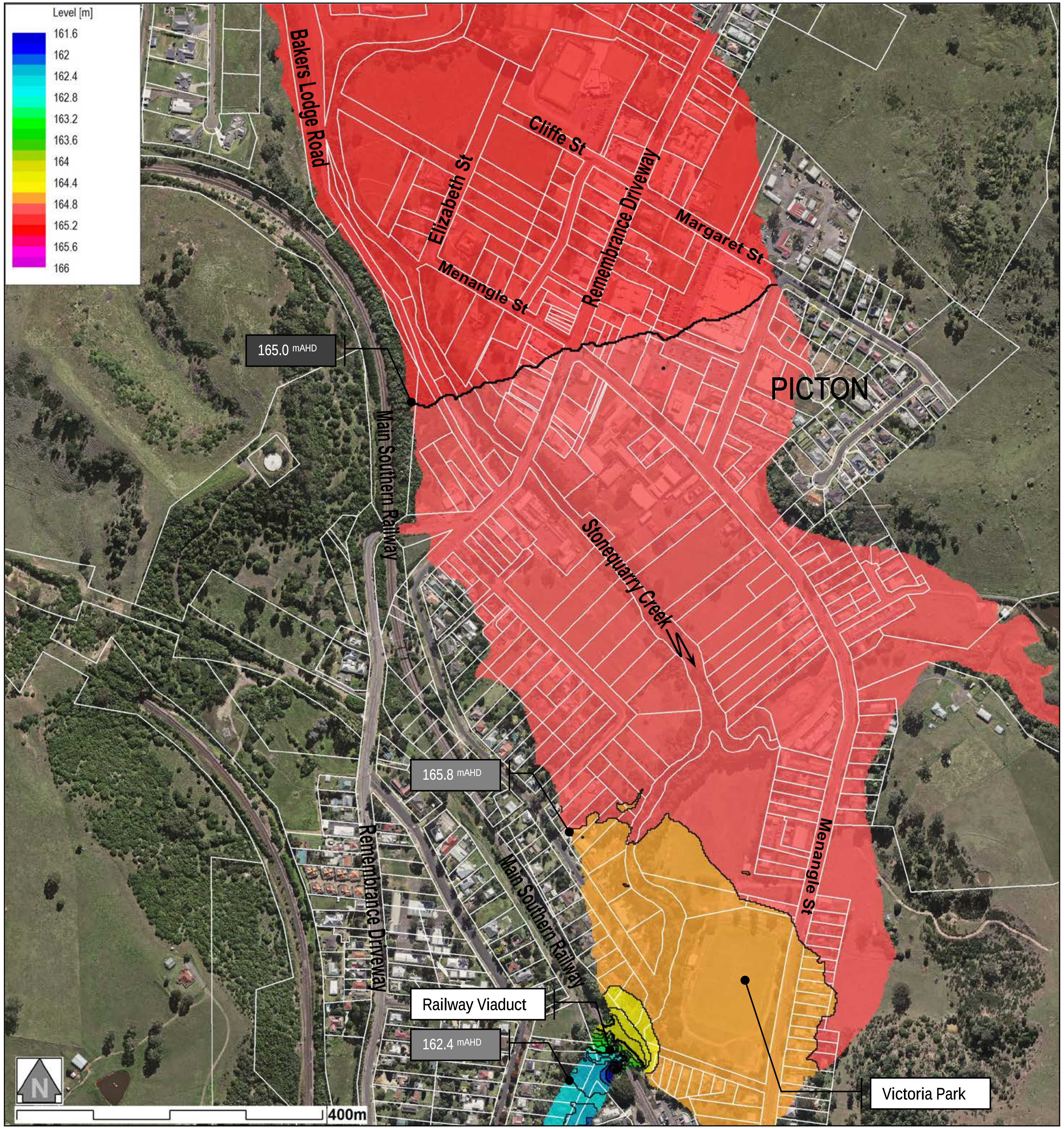
FIGURE 28

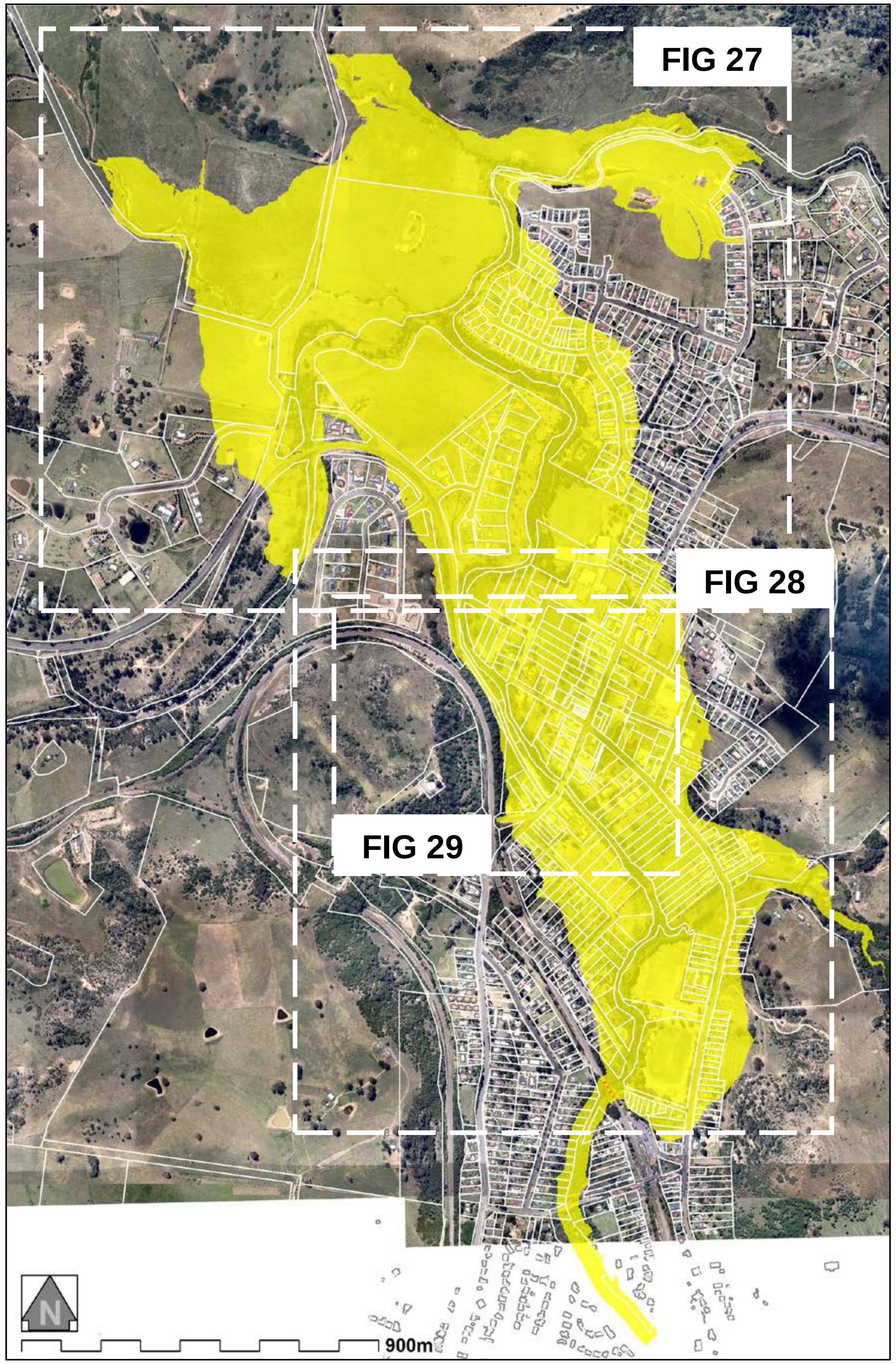
DRAFT

LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.2 metre Interval





PROBABLE MAXIMUM FLOOD LEVELS– SHEET 2 OF 3
[UPDATED RMA-2 FLOOD MODEL]

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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig28_Predicted Flood levels (PMF_South).doc

FIGURE 29

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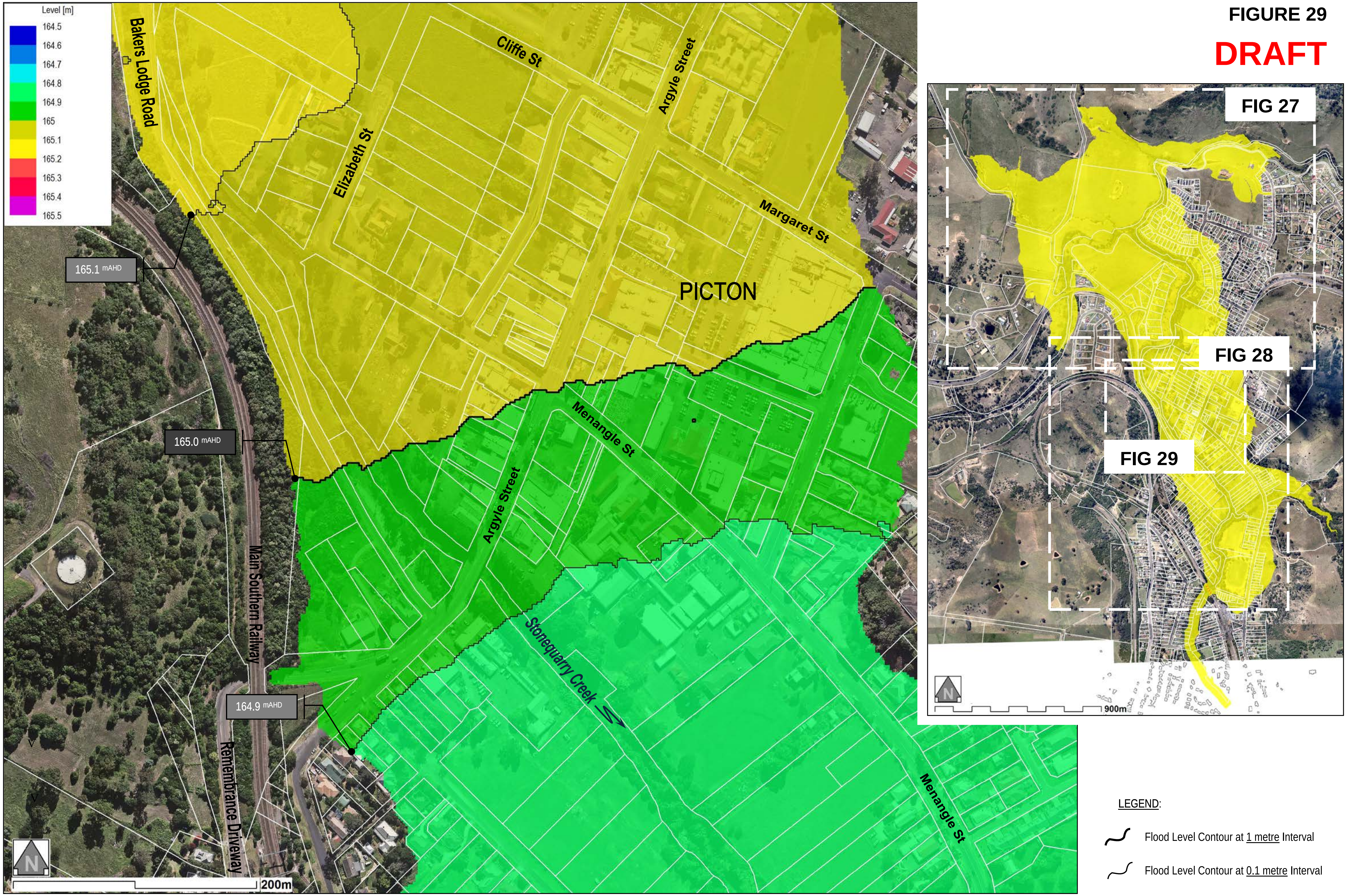

LEGEND:

Flood Level Contour at 1 metre Interval

Flood Level Contour at 0.1 metre Interval

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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig29_Predicted Flood levels (100yr_Town).doc

PROBABLE MAXIMUM FLOOD LEVELS – SHEET 3 OF 3
[UPDATED RMA-2 FLOOD MODEL]

FIGURE 30

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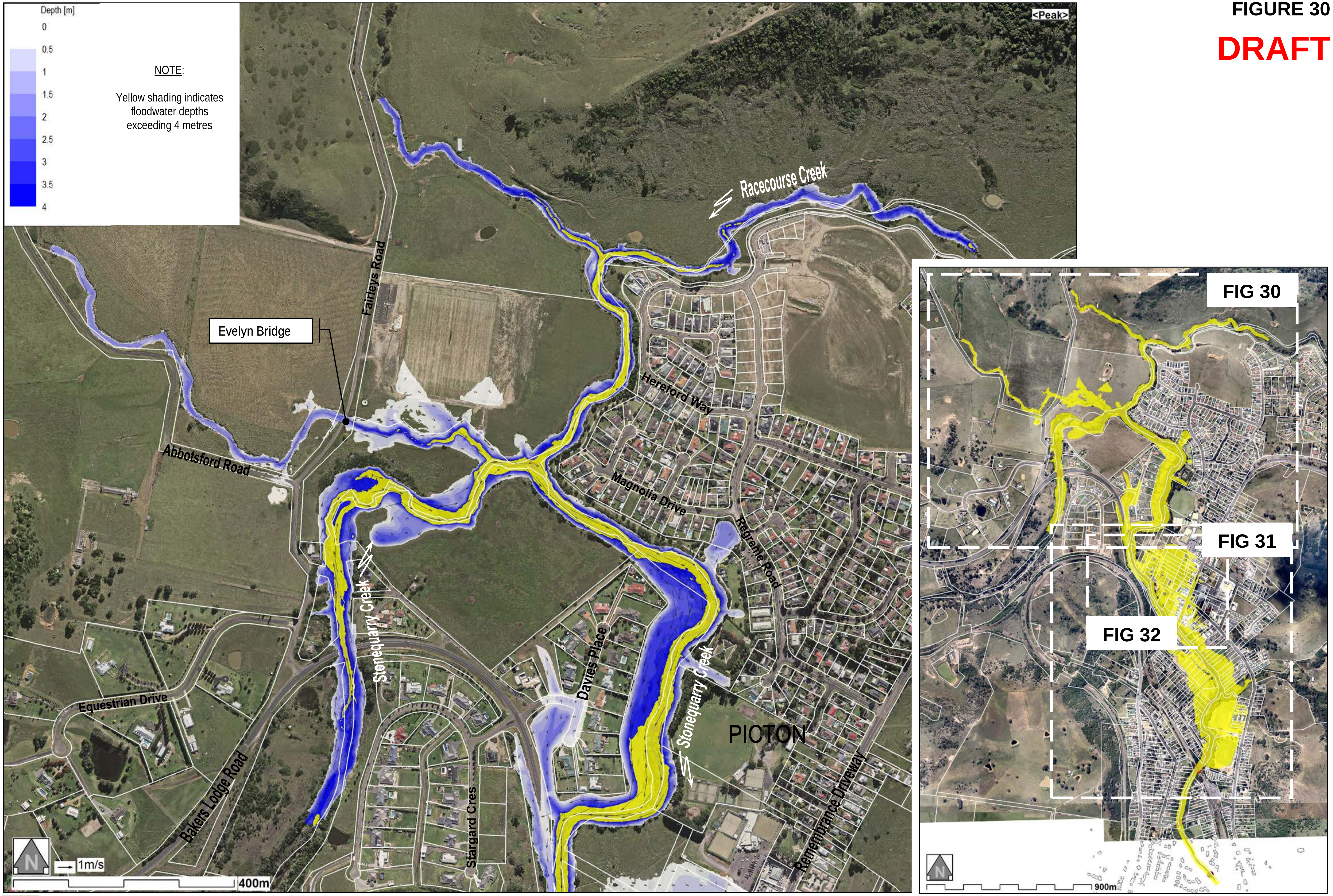

PREDICTED DEPTHS & VELOCITIES AT THE PEAK OF THE 5% AEP FLOOD – SHEET 1 OF 3 [UPDATED RMA-2 FLOOD MODEL]

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301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig30_Predicted Flood levels (5% AEP_North).doc

FIGURE 31

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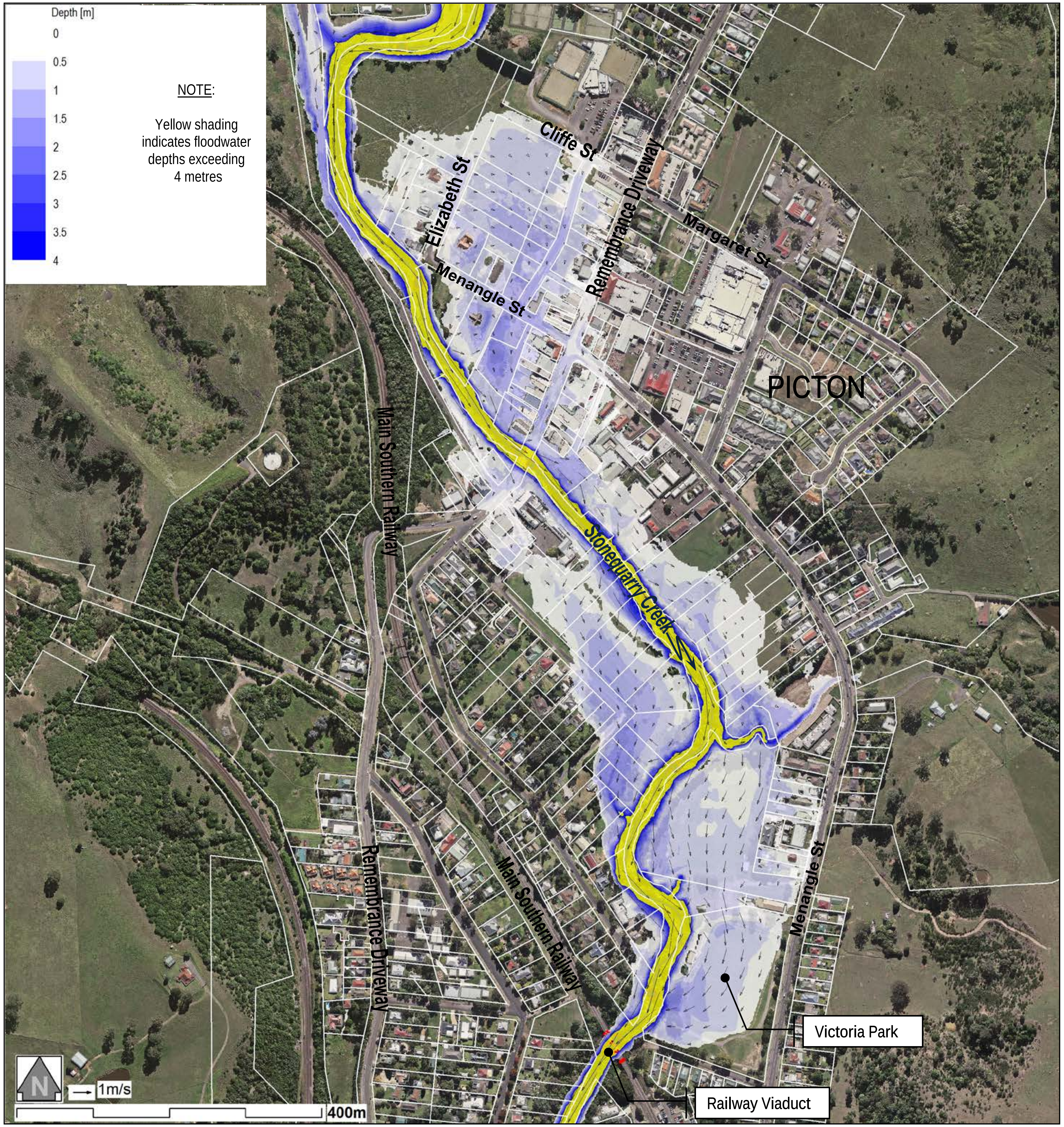

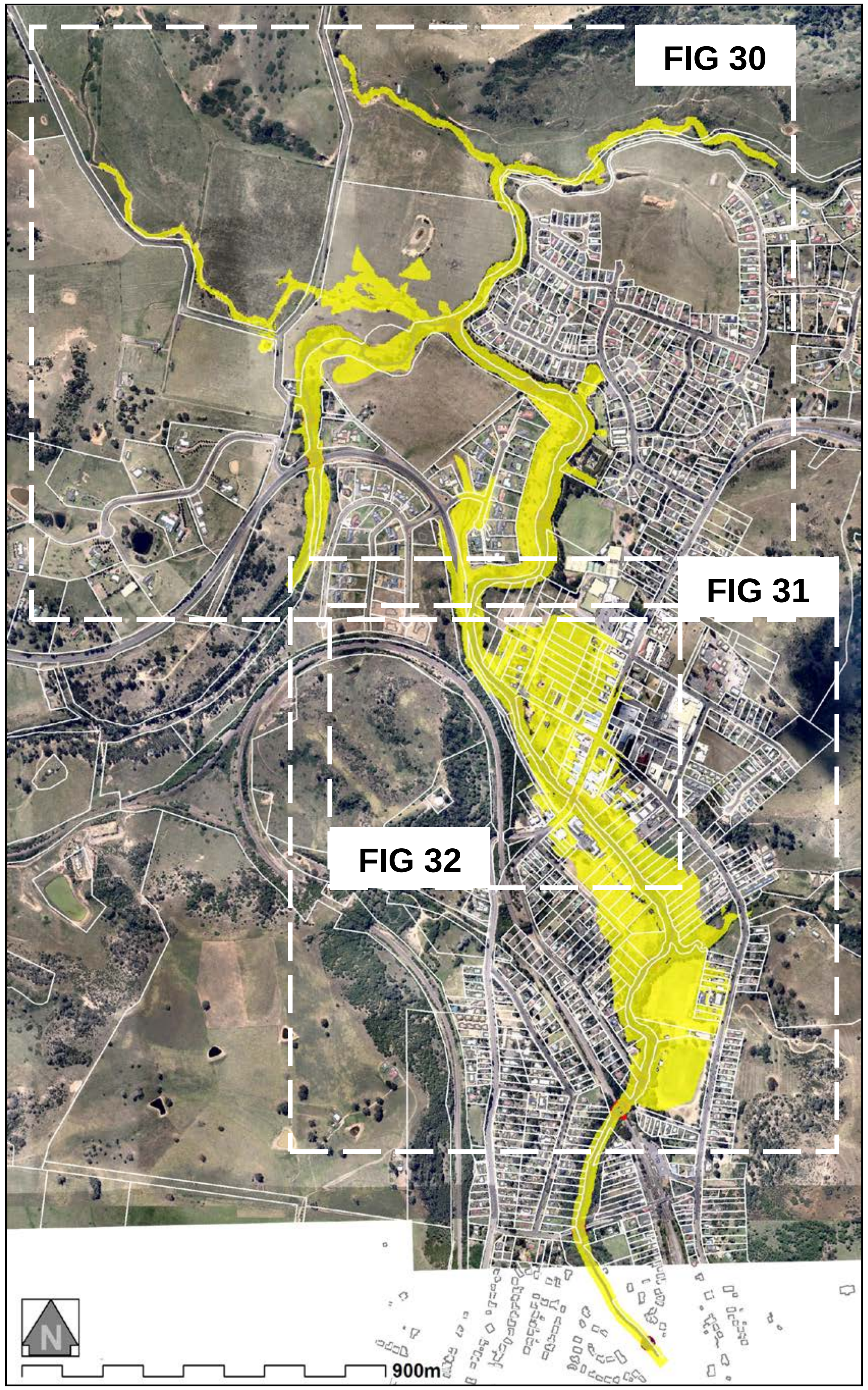

PREDICTED DEPTHS & VELOCITIES AT THE PEAK OF THE 5% AEP FLOOD – SHEET 2 OF 3 [UPDATED RMA-2 FLOOD MODEL]

Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig31_Predicted Flood levels (5% AEP_South).doc

FIGURE 32

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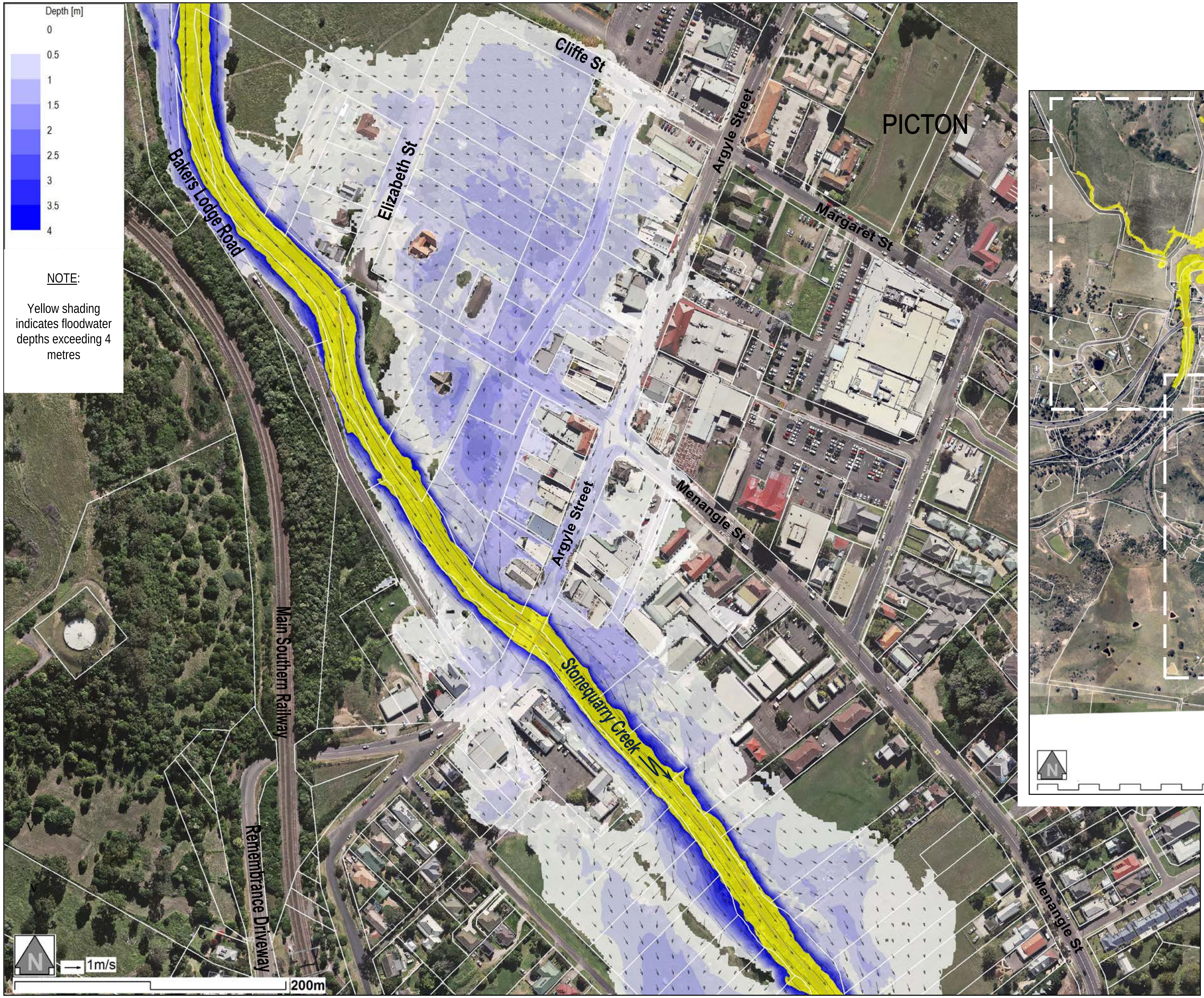
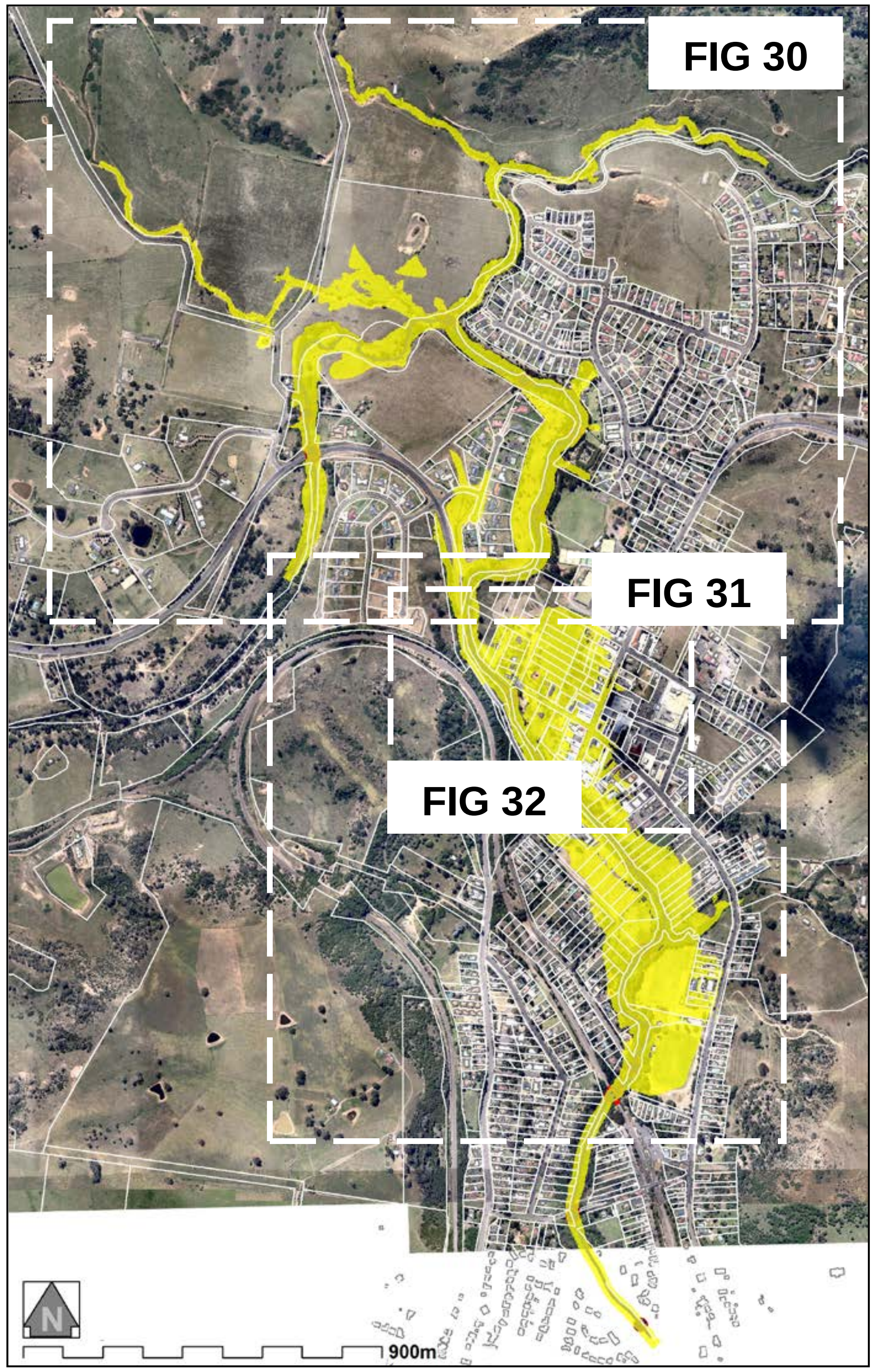




Advisian

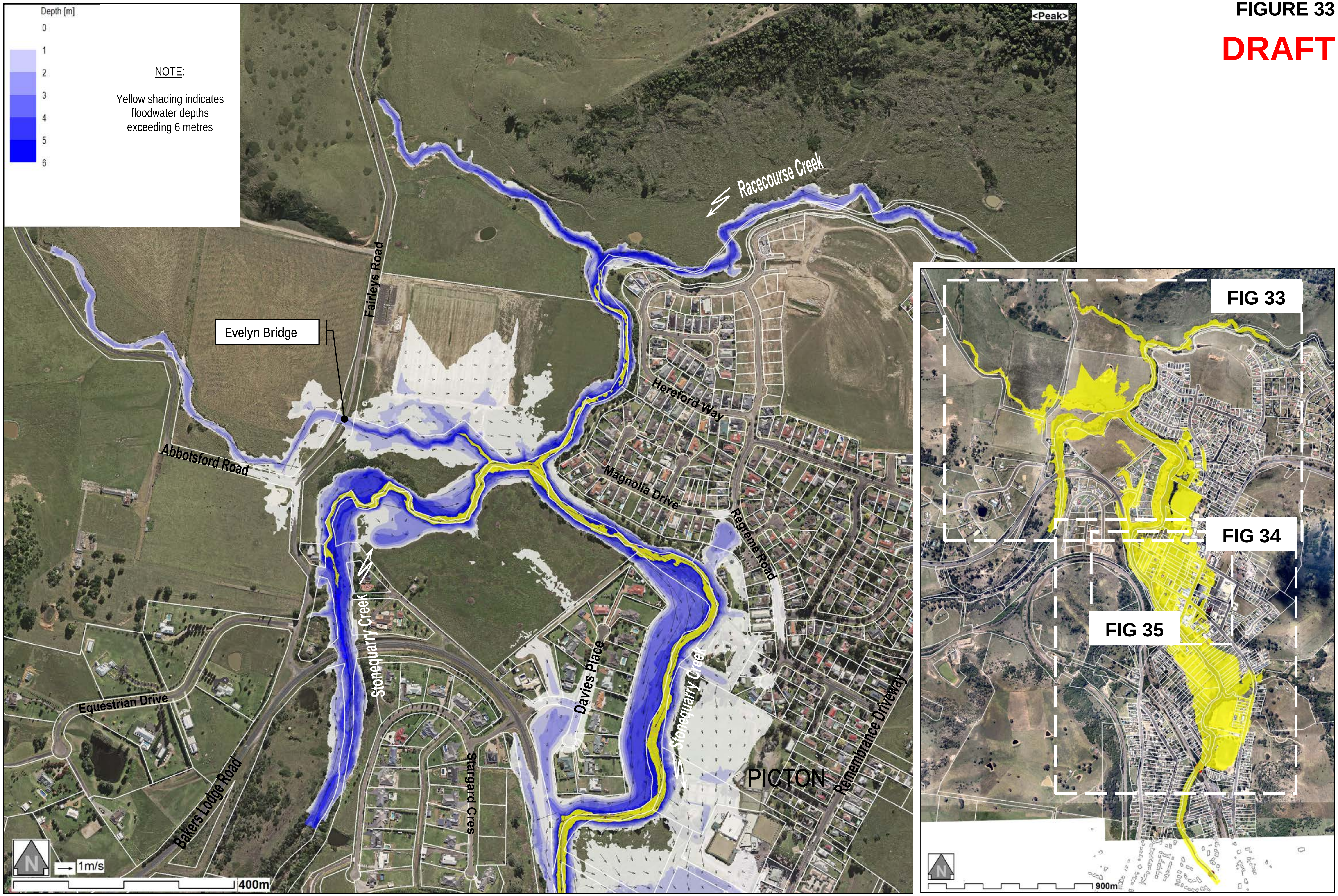
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig32_Predicted Flood levels (5% AEP_Town).doc

PREDICTED DEPTHS & VELOCITIES AT THE PEAK OF THE 5% AEP FLOOD – SHEET 3 OF 3
[UPDATED RMA-2 FLOOD MODEL]

FIGURE 33

DRAFT



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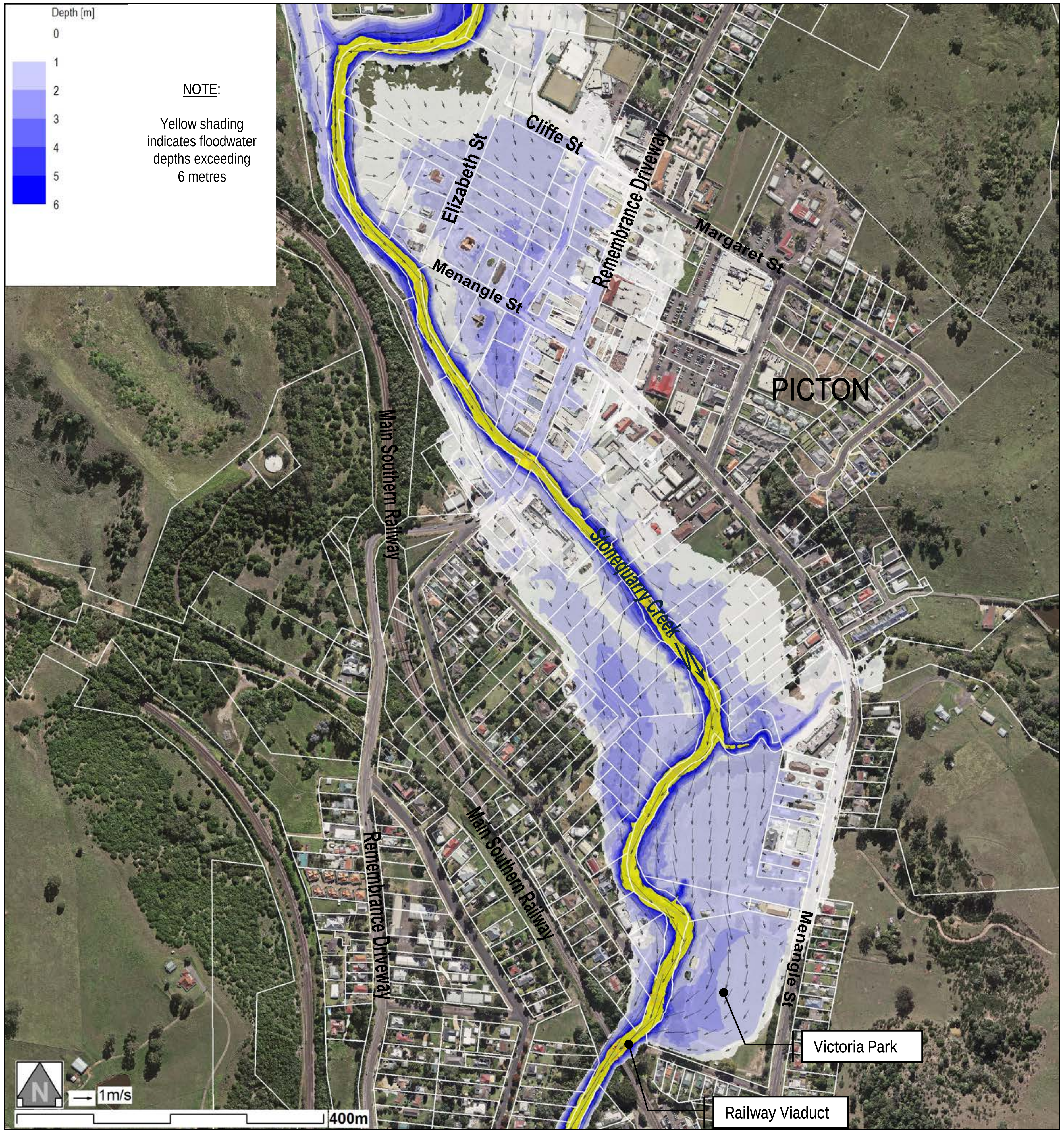
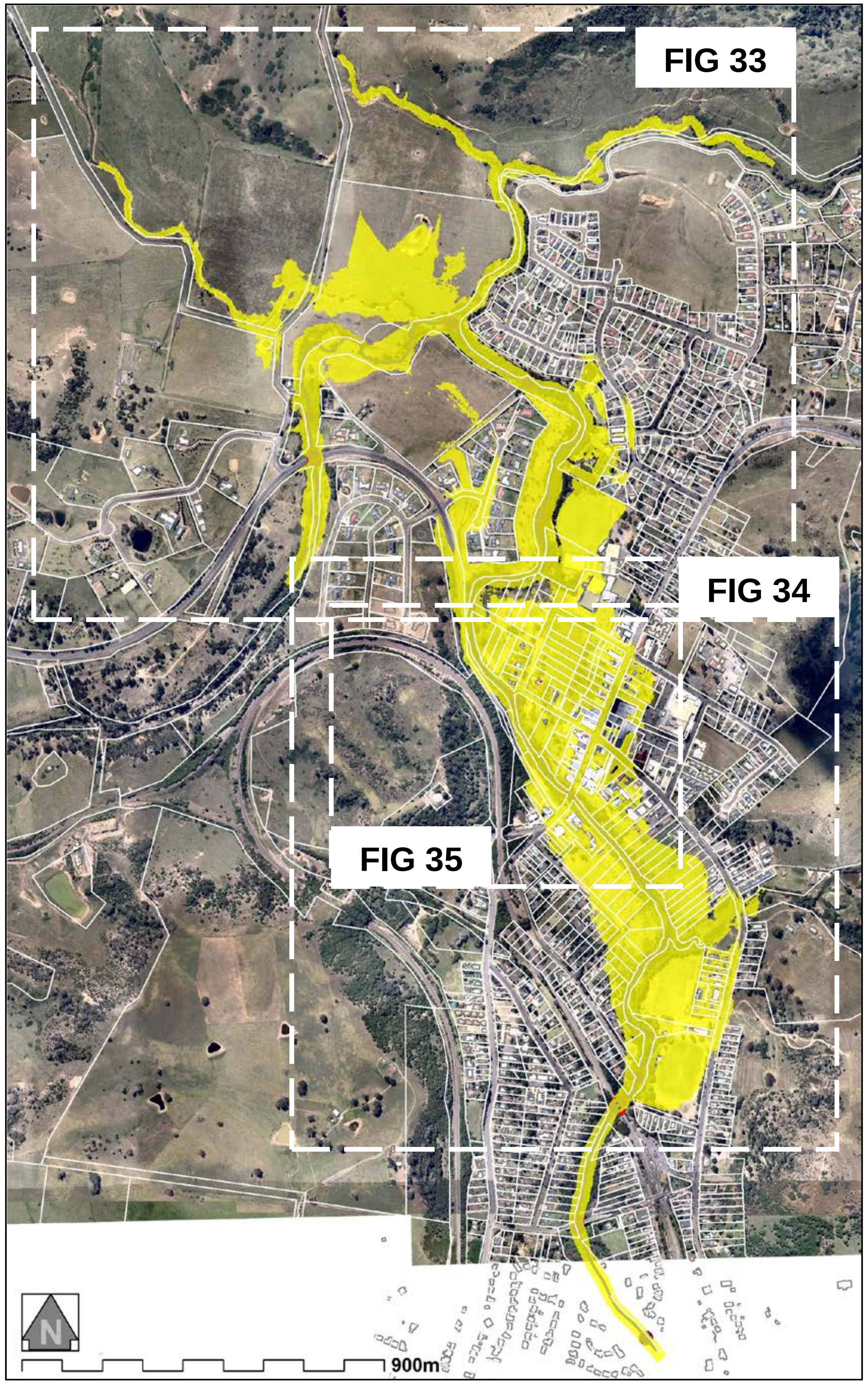
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig33_Predicted Flood levels (1% AEP_North).doc

PREDICTED DEPTHS & VELOCITIES AT THE PEAK OF THE 1% AEP FLOOD – SHEET 1 OF 3
[UPDATED RMA-2 FLOOD MODEL]

FIGURE 34

DRAFT



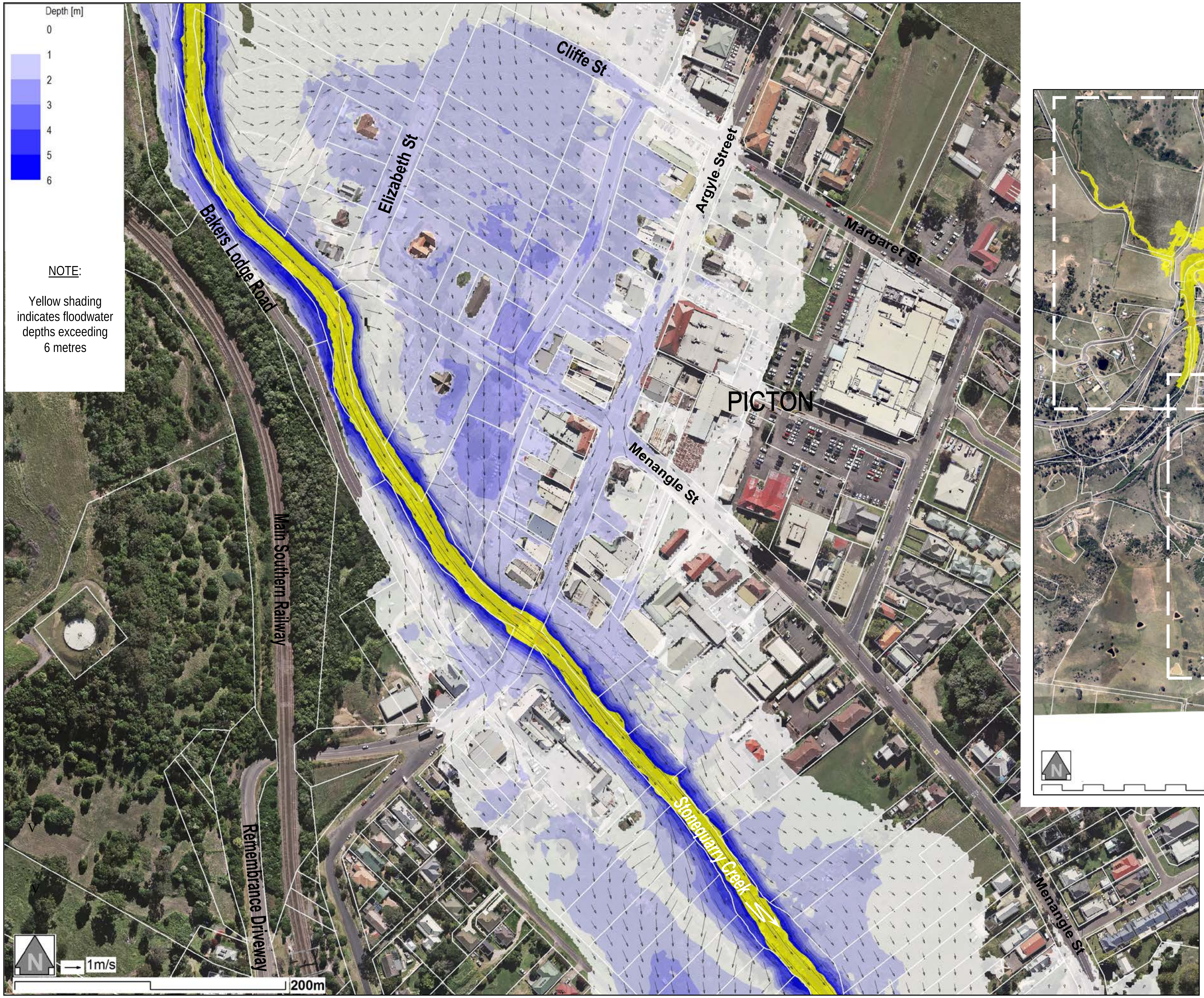


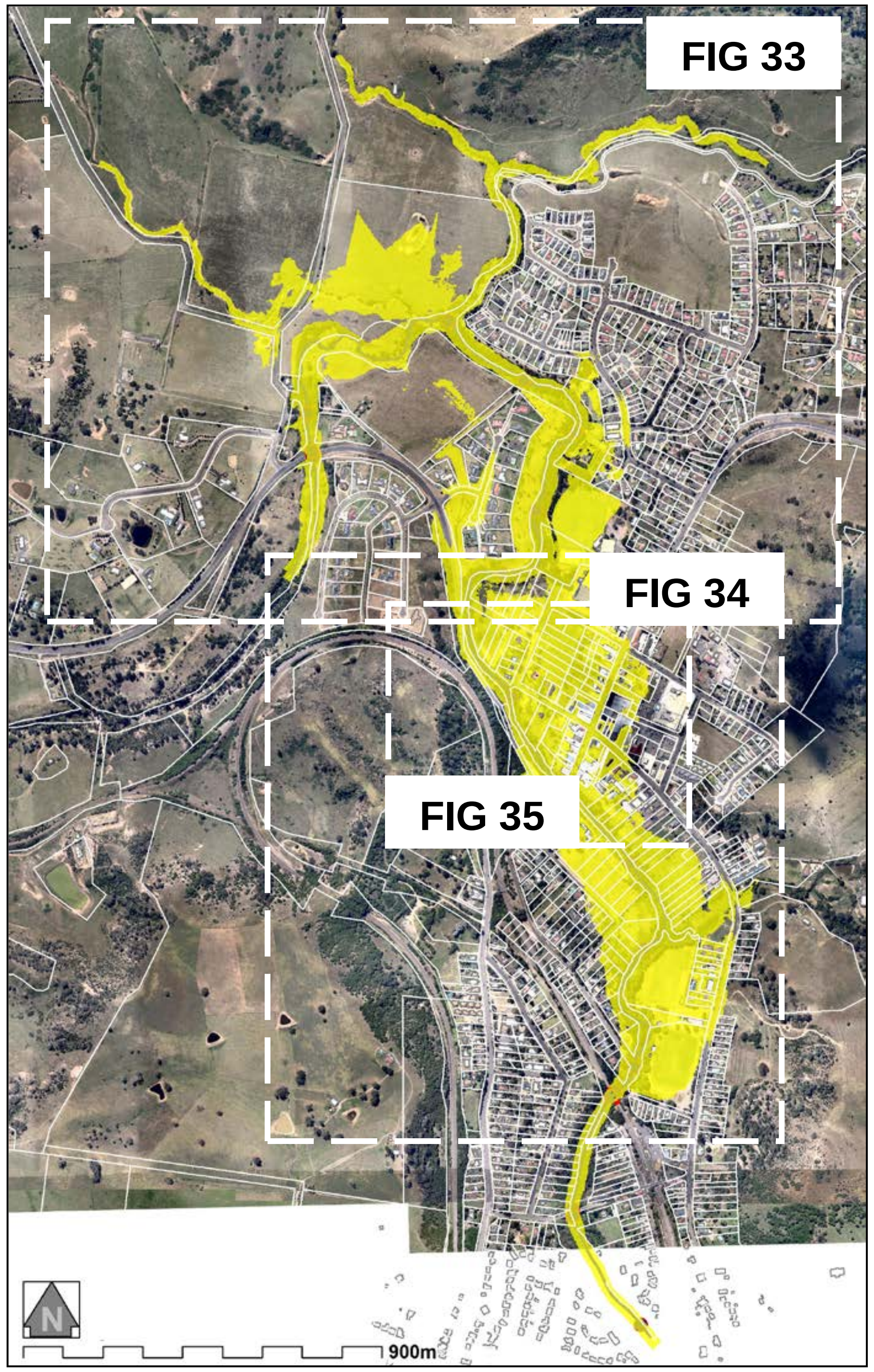
Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig34_Predicted Flood levels (1% AEP_South).doc

PREDICTED DEPTHS & VELOCITIES AT THE PEAK OF THE 1% AEP FLOOD – SHEET 2 OF 3
[UPDATED RMA-2 FLOOD MODEL]

FIGURE 35

DRAFT





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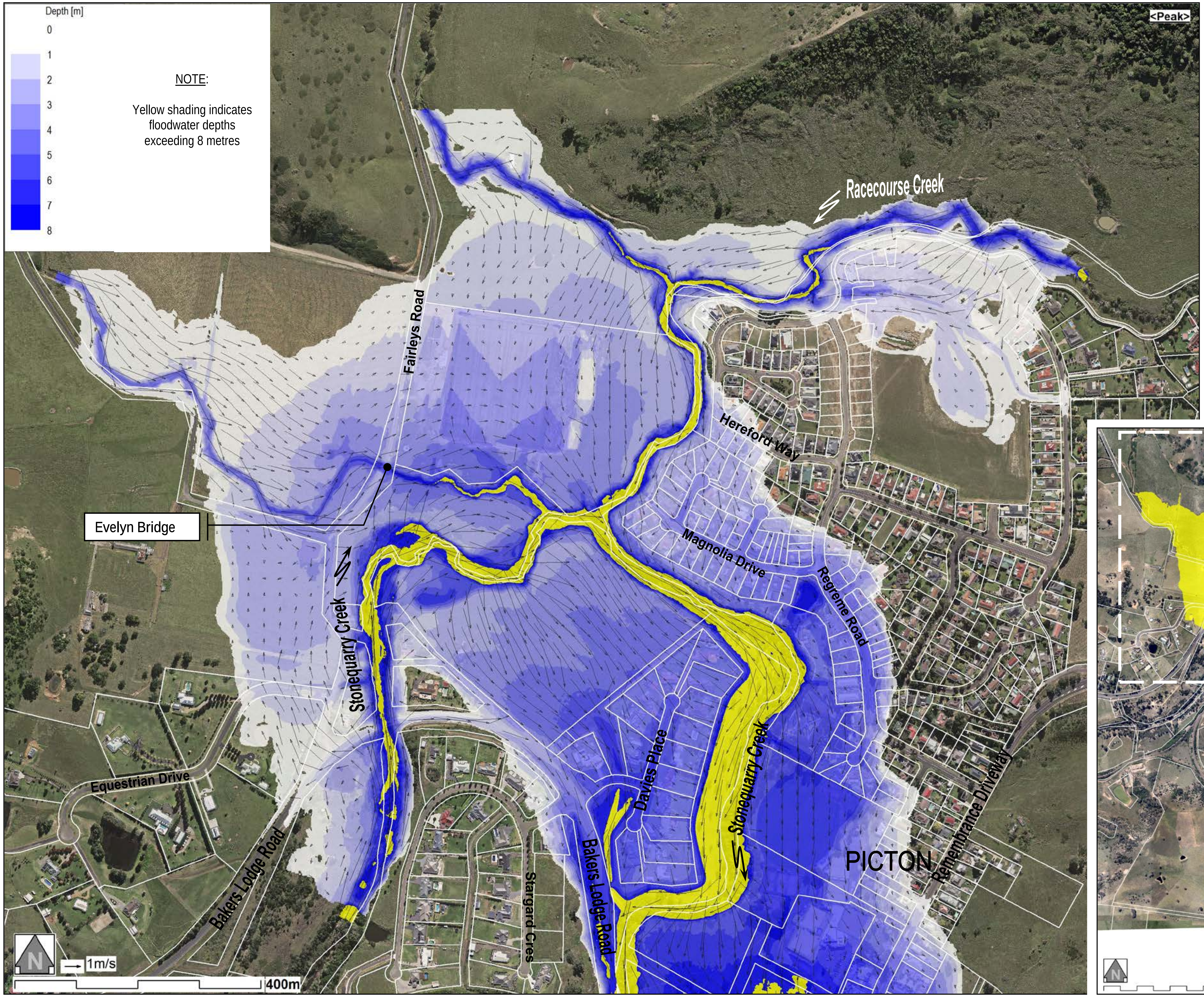
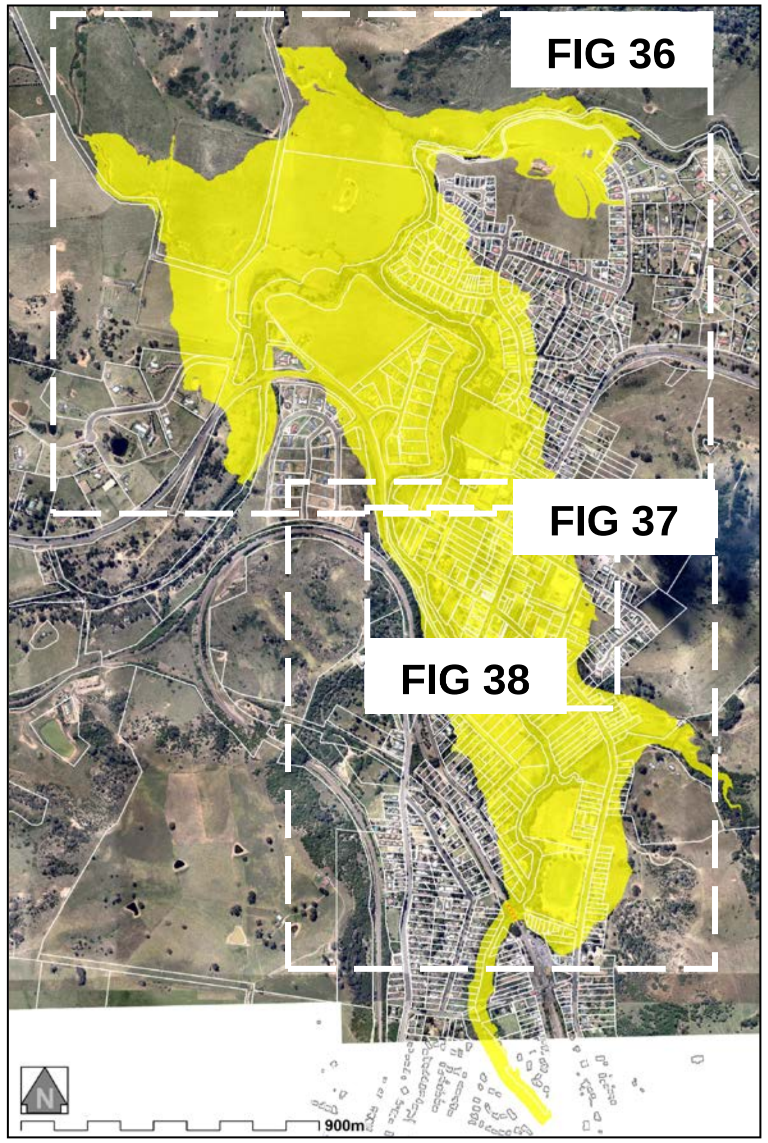
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig35_Predicted Flood levels (1% AEP_Town).doc

PREDICTED DEPTHS & VELOCITIES AT THE PEAK OF THE 1% AEP FLOOD – SHEET 3 OF 3 [UPDATED RMA-2 FLOOD MODEL]

FIGURE 36

DRAFT





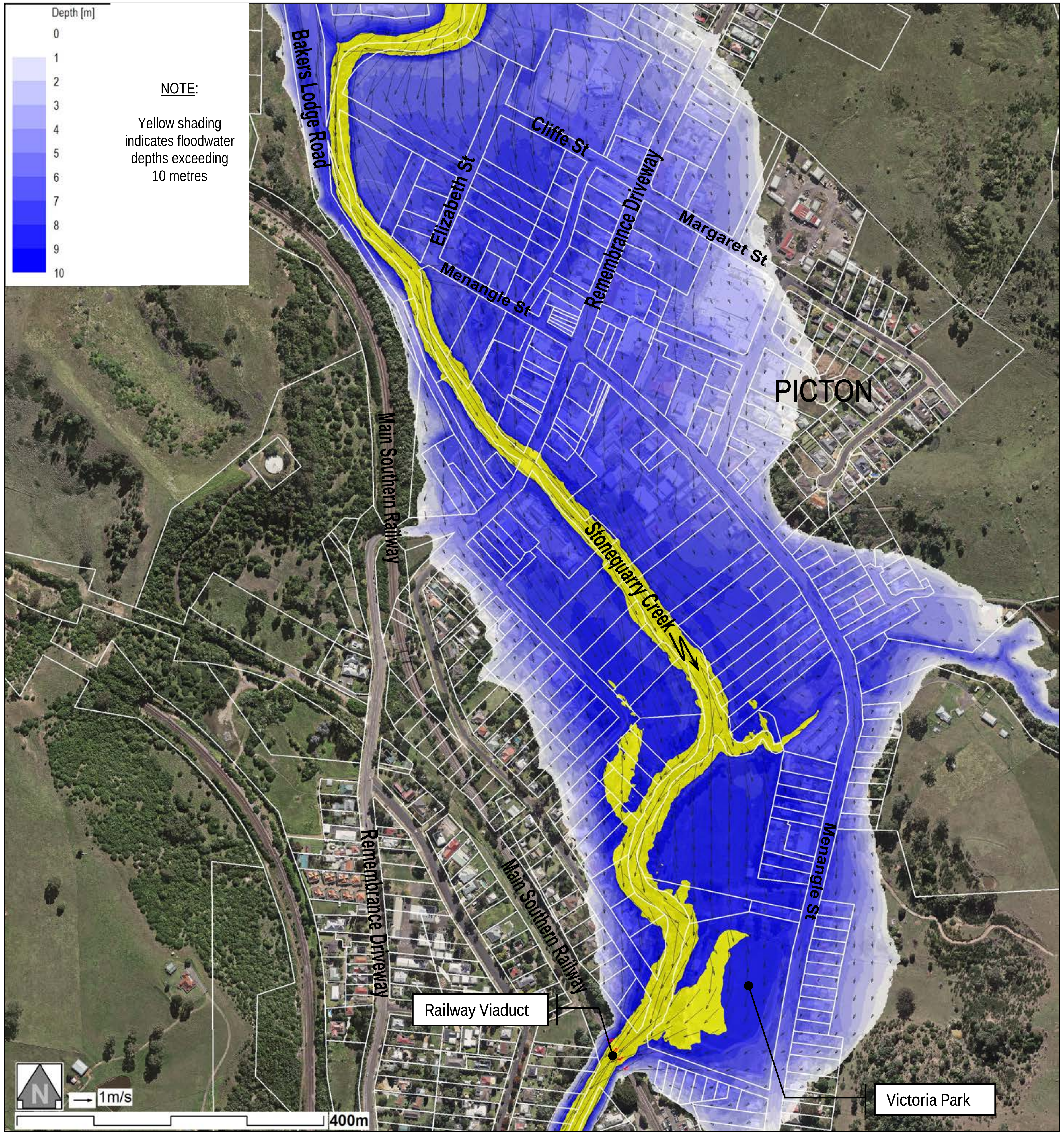
Advisian
WorleyParsons Group

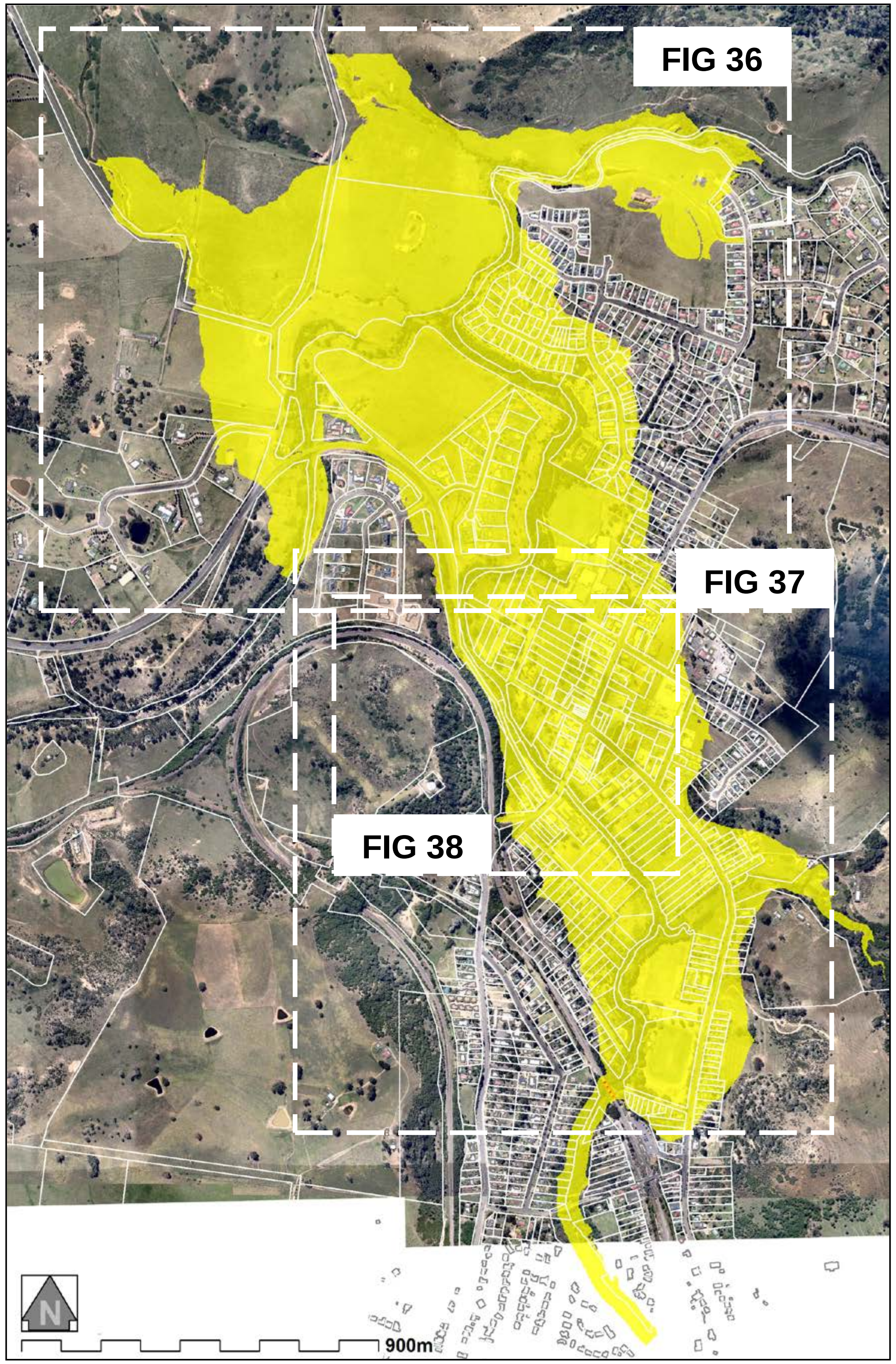
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig36_Predicted Flood levels (PMF_North).doc

PREDICTED DEPTHS & VELOCITIES FOR THE PROBABLE MAXIMUM FLOOD – SHEET 1 OF 3 [UPDATED RMA-2 FLOOD MODEL]

FIGURE 36

DRAFT



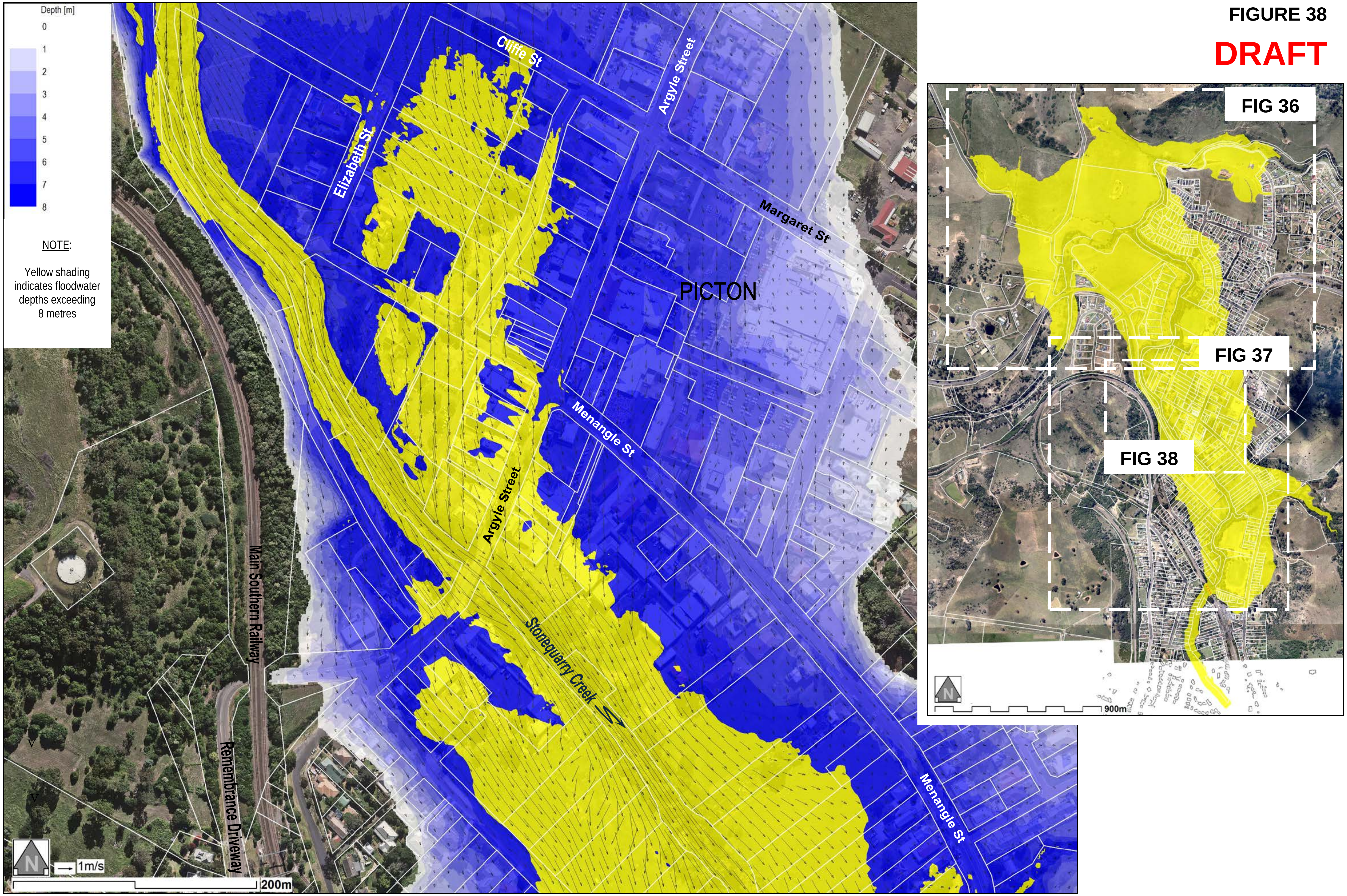


Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig37_Predicted Flood levels (PMF_South).doc

PREDICTED DEPTHS & VELOCITIES FOR THE PROBABLE MAXIMUM FLOOD – SHEET 2 OF 3
[UPDATED RMA-2 FLOOD MODEL]

FIGURE 38

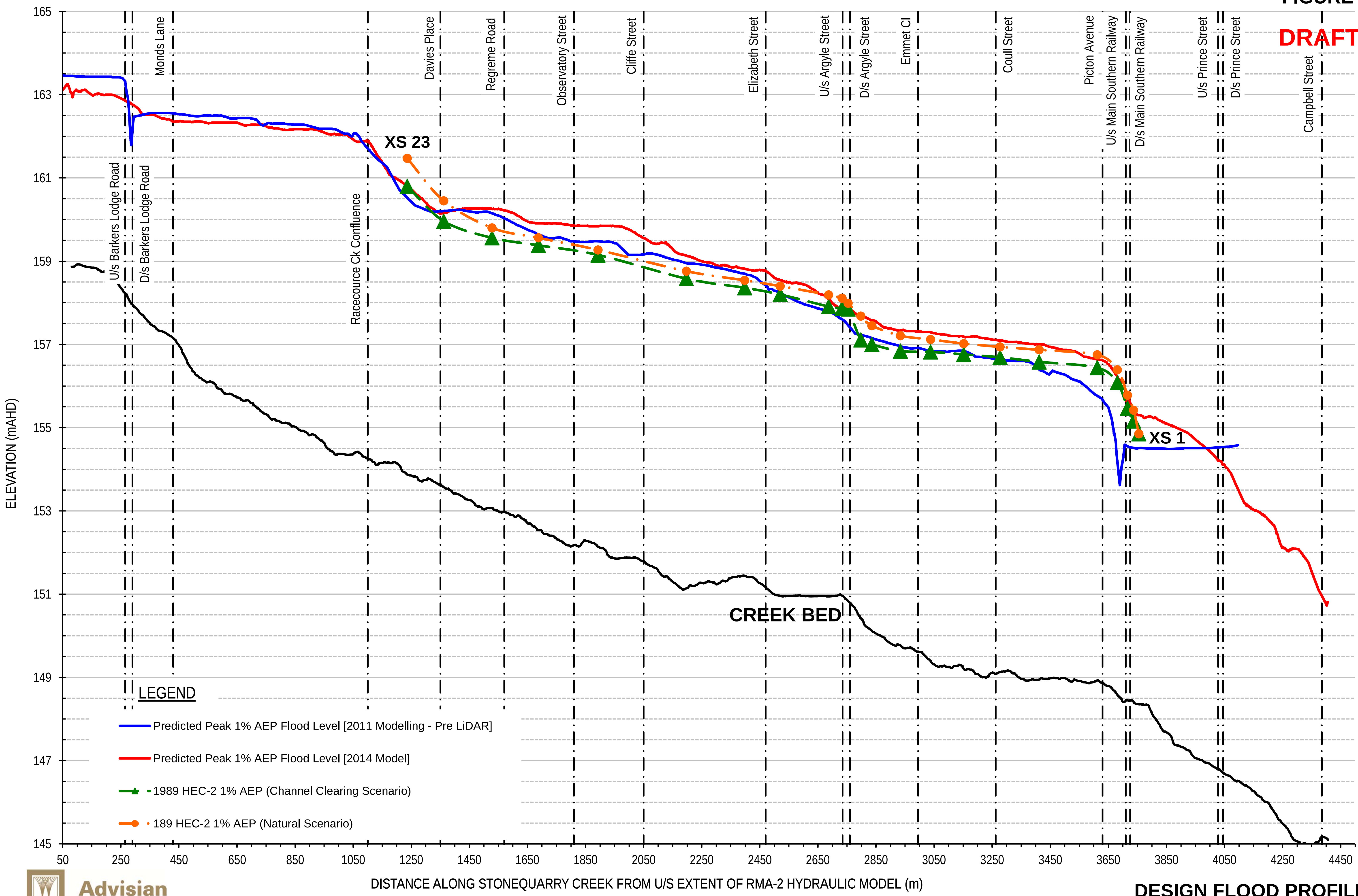
DRAFT



Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig38_Predicted Flood levels (PMF_Town).doc

PREDICTED DEPTHS & VELOCITIES FOR THE PROBABLE MAXIMUM FLOOD – SHEET 3 OF 3
[UPDATED RMA-2 FLOOD MODEL]

FIGURE 39

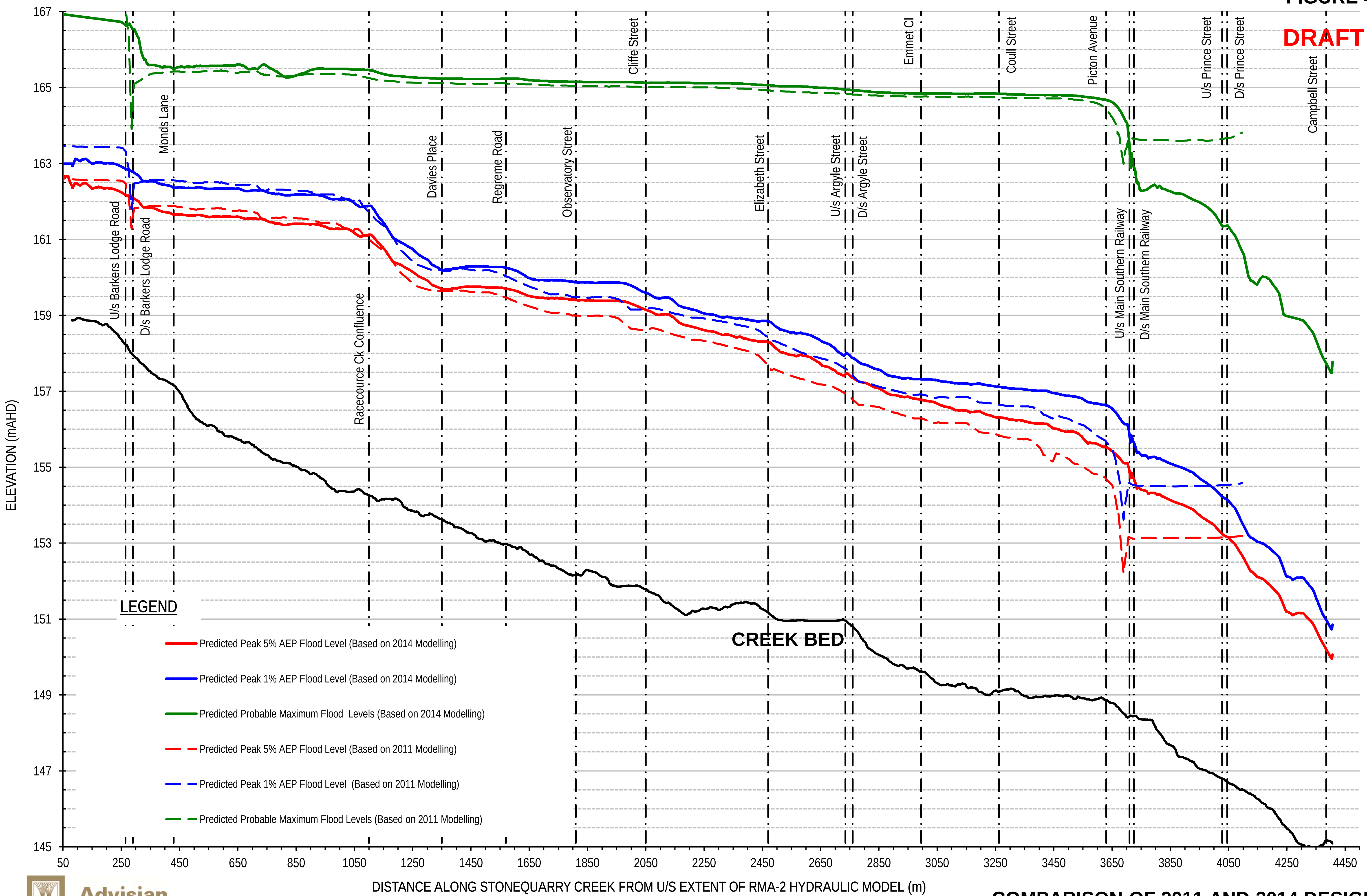


Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

DESIGN FLOOD PROFILES ALONG STONEQUARRY CREEK
[*COMPARISON OF PRE & POST 2014 LEVELS*]

FIGURE 40



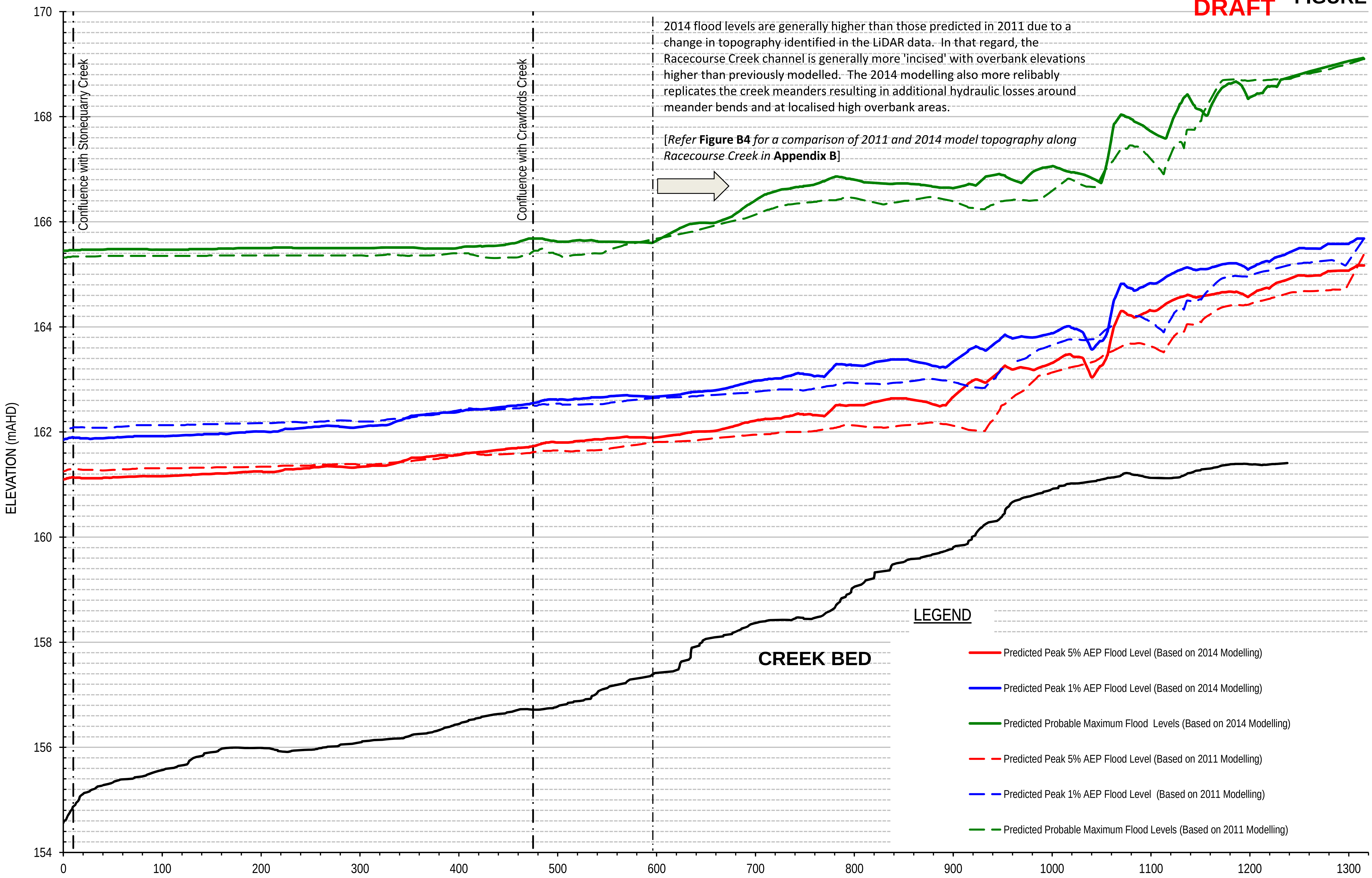
Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

COMPARISON OF 2011 AND 2014 DESIGN FLOOD PROFILES ALONG STONEQUARRY CREEK

FIGURE 41

DRAFT

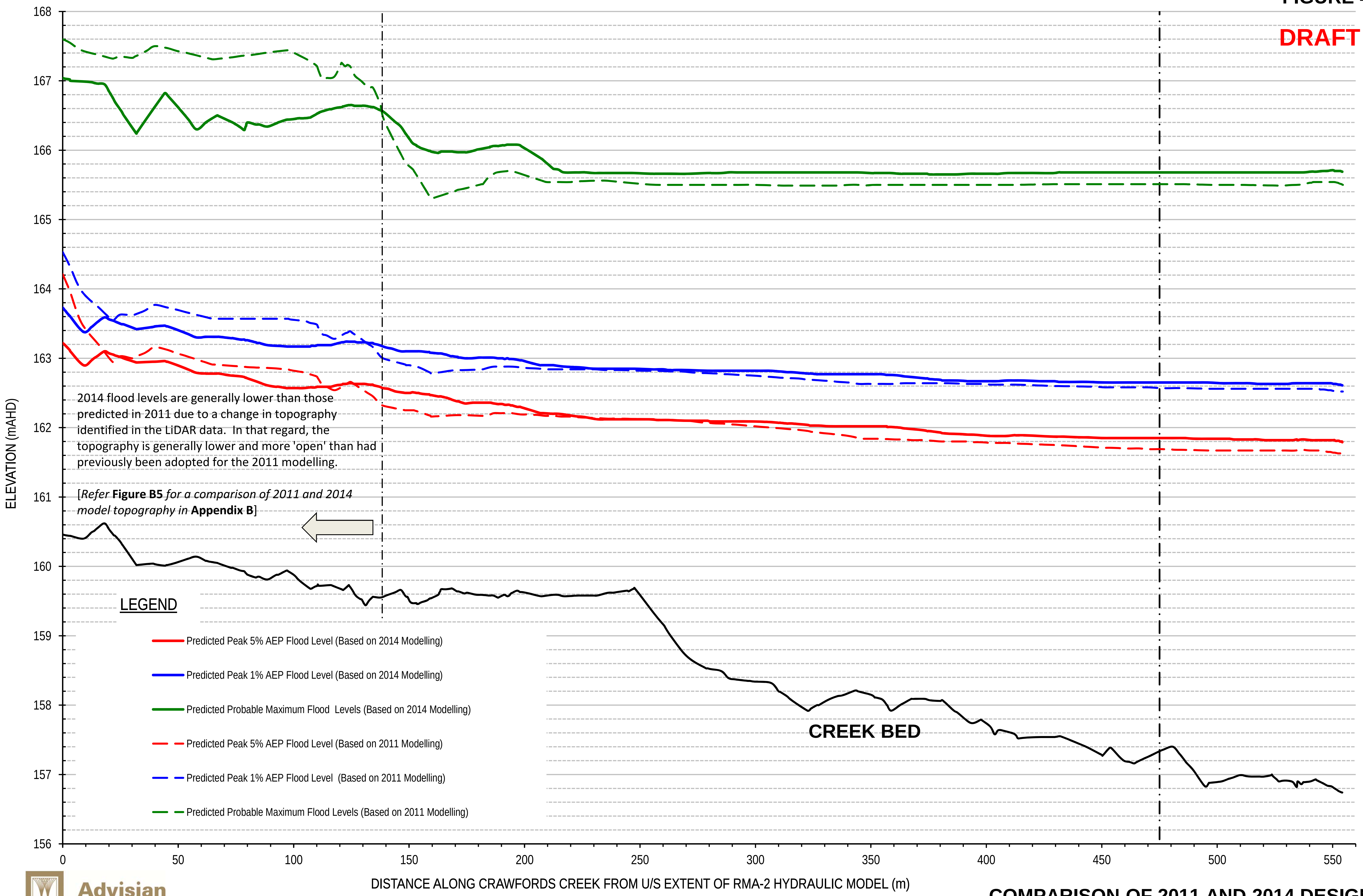


Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

COMPARISON OF 2011 AND 2014 DESIGN FLOOD PROFILES ALONG RACECOURSE CREEK

FIGURE 42



Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

COMPARISON OF 2011 AND 2014 DESIGN FLOOD PROFILES ALONG CRAWFORDS CREEK

FIGURE 43

DRAFT

Advisian

WorleyParsons Group

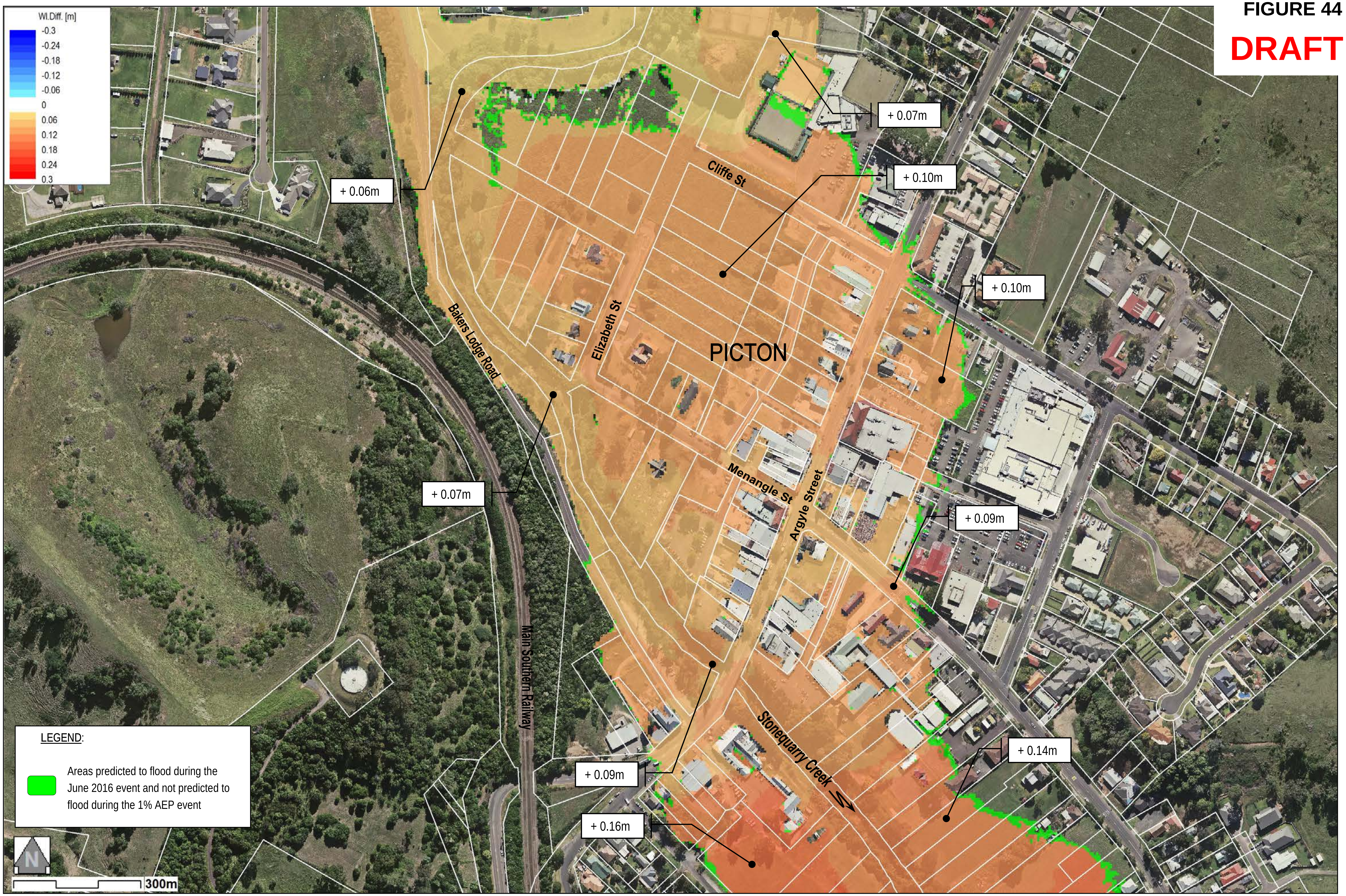
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg161013_Fig6_June 2016 less 1% AEP (North).doc

COMPARISON OF RMA-2 MODELLED LEVELS FOR THE 'JUNE 2016' FLOOD EVENT AND THE 1% AEP FLOOD

[June 2016 less 1% AEP - Extent 1 of 3]

FIGURE 44

DRAFT



Advisian

WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling

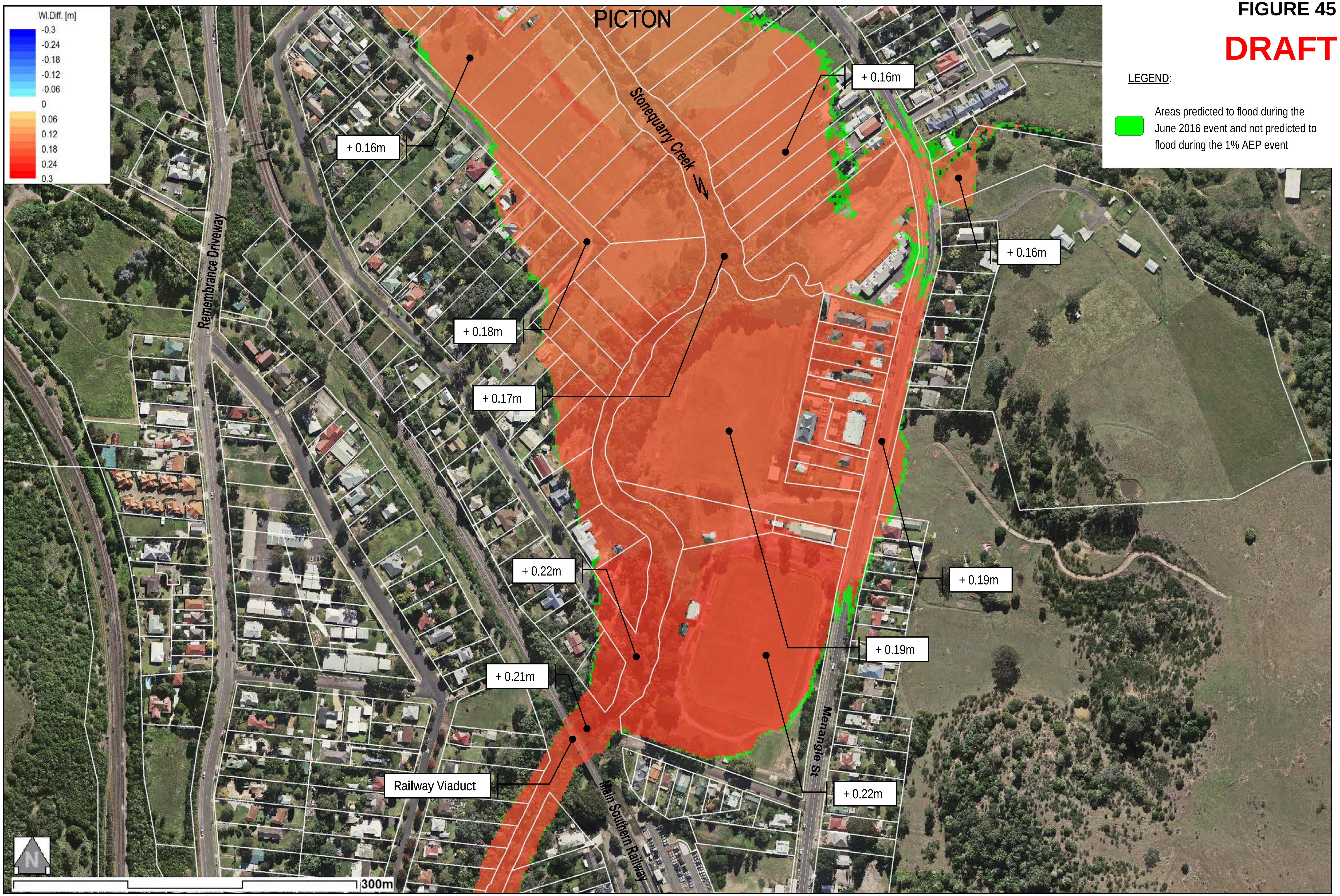
fg301015-03199rg161013_Fig7_June 2016 less 1% AEP (Town).doc

COMPARISON OF RMA-2 MODELLED LEVELS FOR THE 'JUNE 2016' FLOOD EVENT AND THE 1% AEP FLOOD

[June 2016 less 1% AEP - Extent 2 of 3]

FIGURE 45

DRAFT



LEGEND:

Areas predicted to flood during the June 2016 event and not predicted to flood during the 1% AEP event

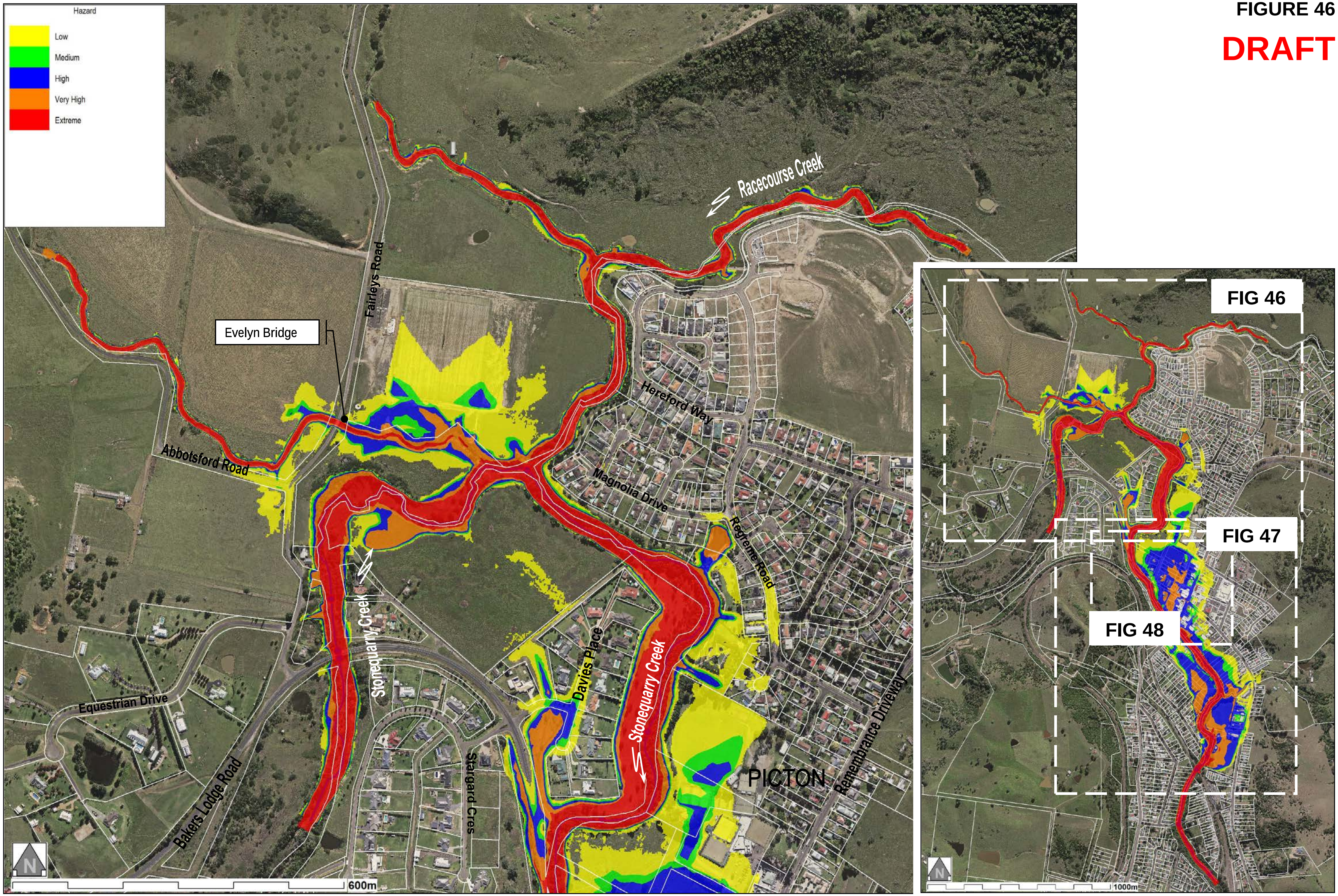
Advisian
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg161013_Fig8_June 2016 less 1% AEP (South).doc

COMPARISON OF RMA-2 MODELLED LEVELS FOR THE 'JUNE 2016' FLOOD EVENT AND THE 1% AEP FLOOD
[June 2016 less 1% AEP - Extent 3 of 3]

FIGURE 46

DRAFT



Advisian

WorleyParsons Group

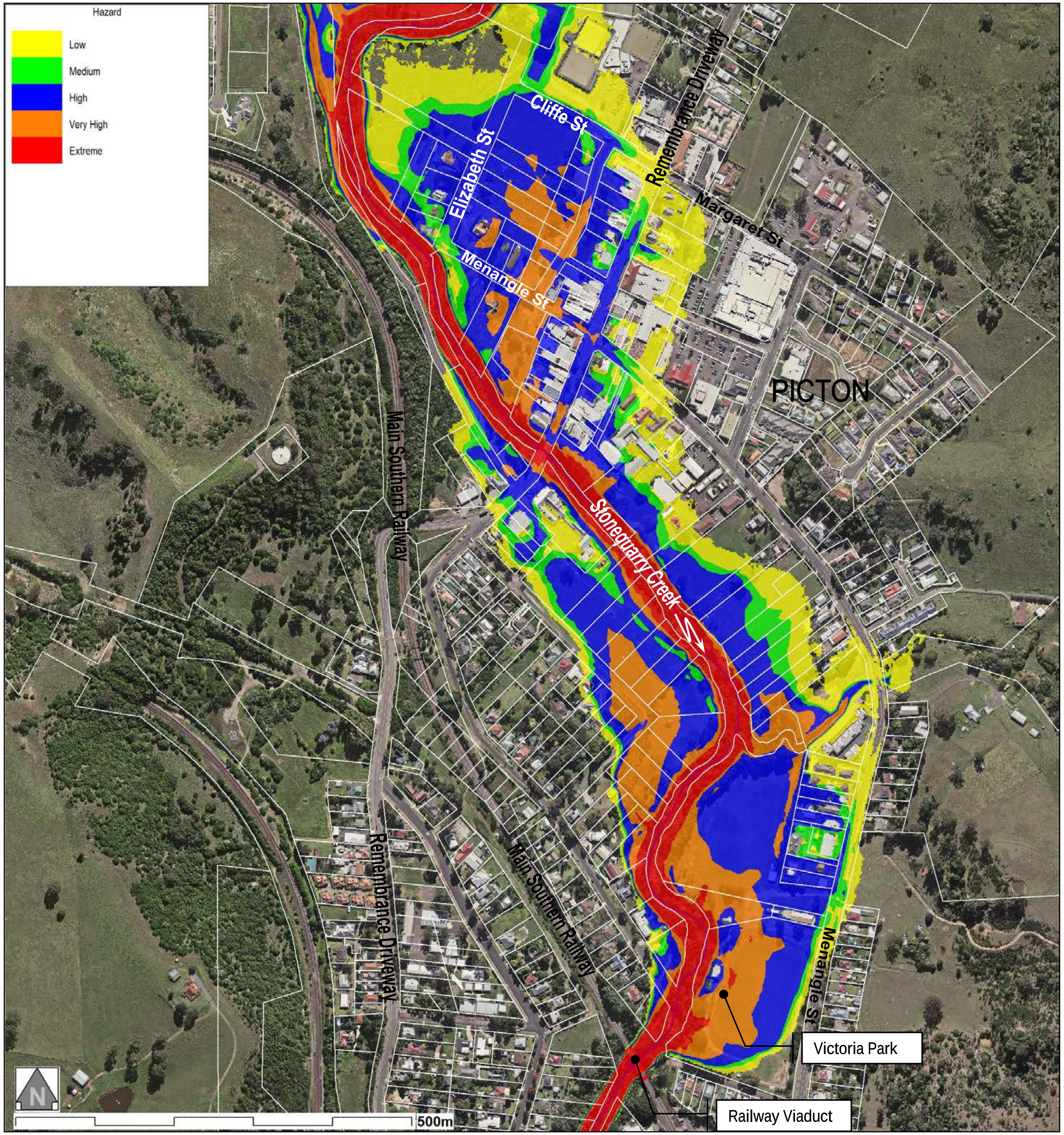
301015-03199-Stonequarry Ck Flood Modelling

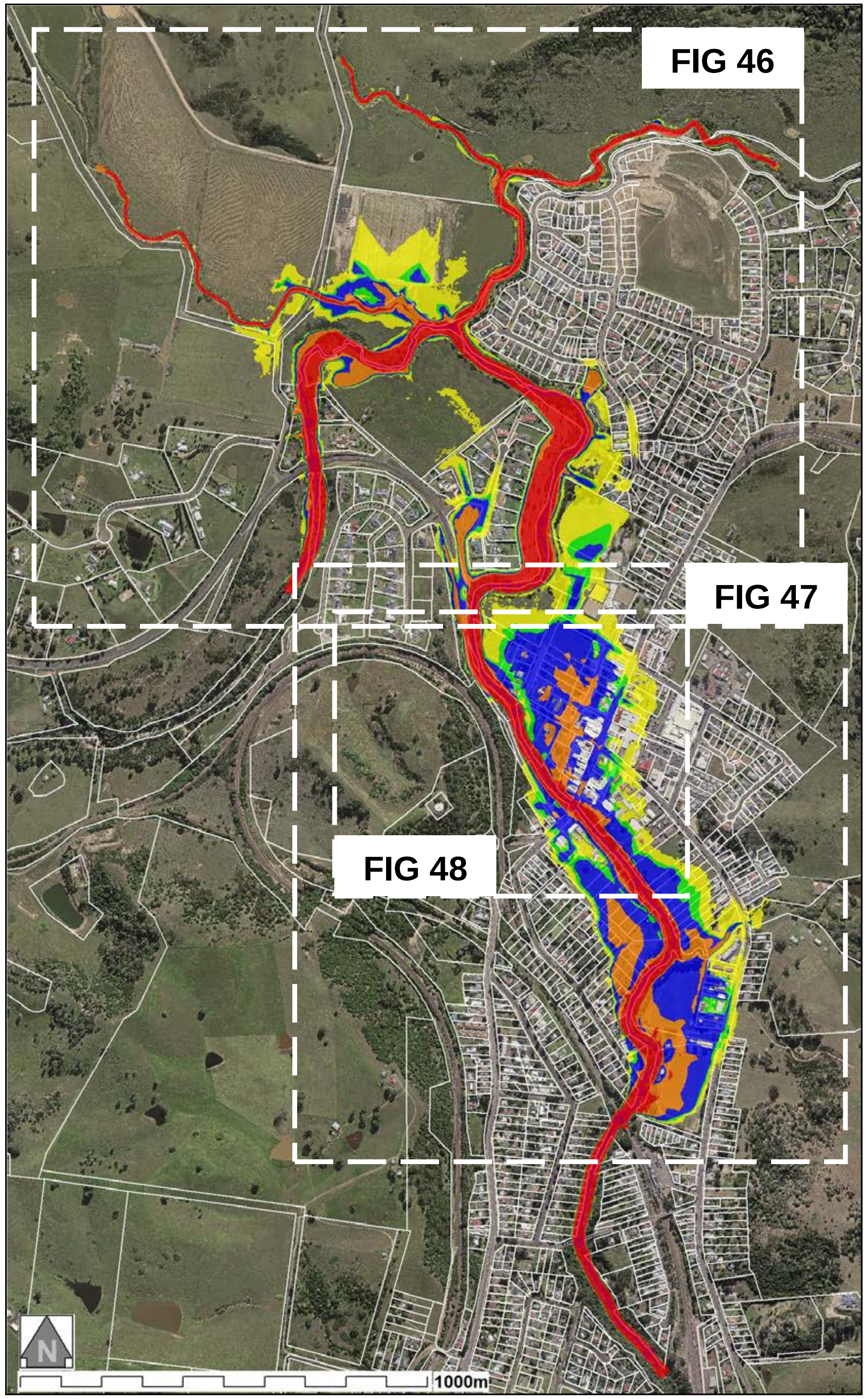
fg301015-03199rg130417_Fig46_Predicted Flood levels (1% AEP_North).doc

PROVISIONAL FLOOD HAZARD MAPPING
FOR THE 1% AEP FLOOD
[SHEET 1 OF 3]

FIGURE 47

DRAFT





Advisian

WorleyParsons Group

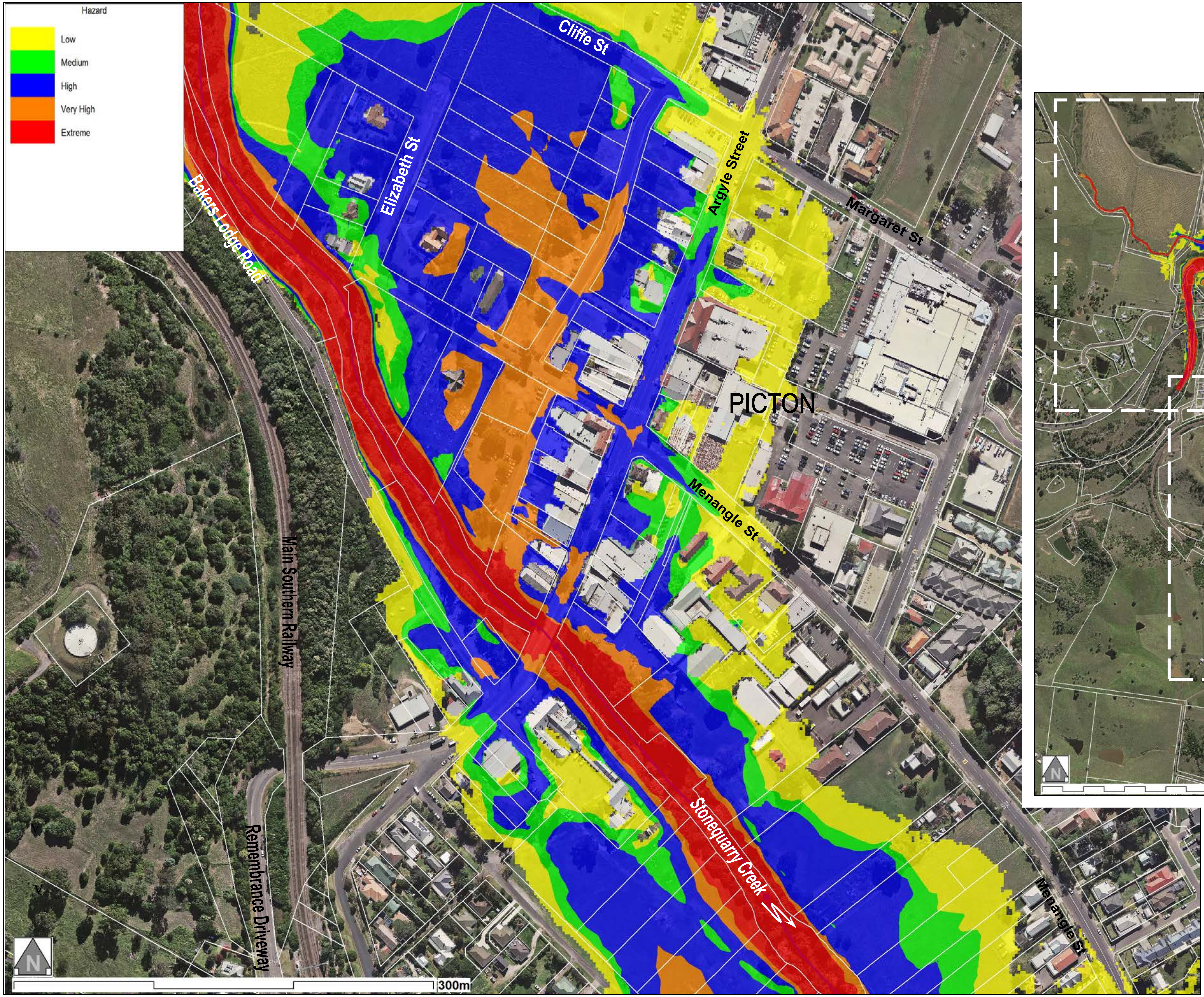
301015-03199-Stonequarry Ck Flood Modelling

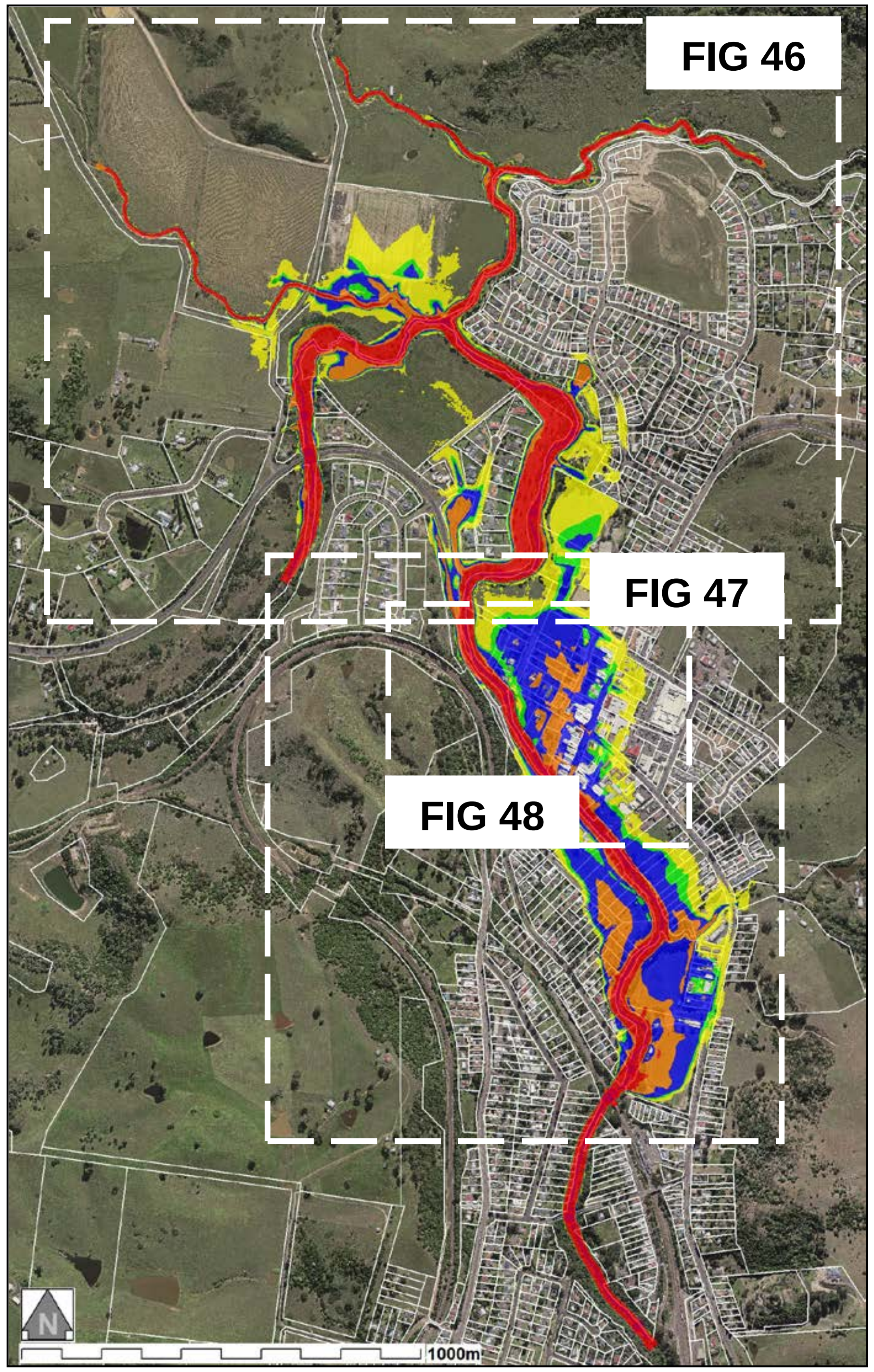
fg301015-03199rg130417_Fig47_Predicted Flood levels (1% AEP_South).doc

PROVISIONAL FLOOD HAZARD MAPPING
FOR THE 1% AEP FLOOD
[SHEET 2 OF 3]

FIGURE 48

DRAFT



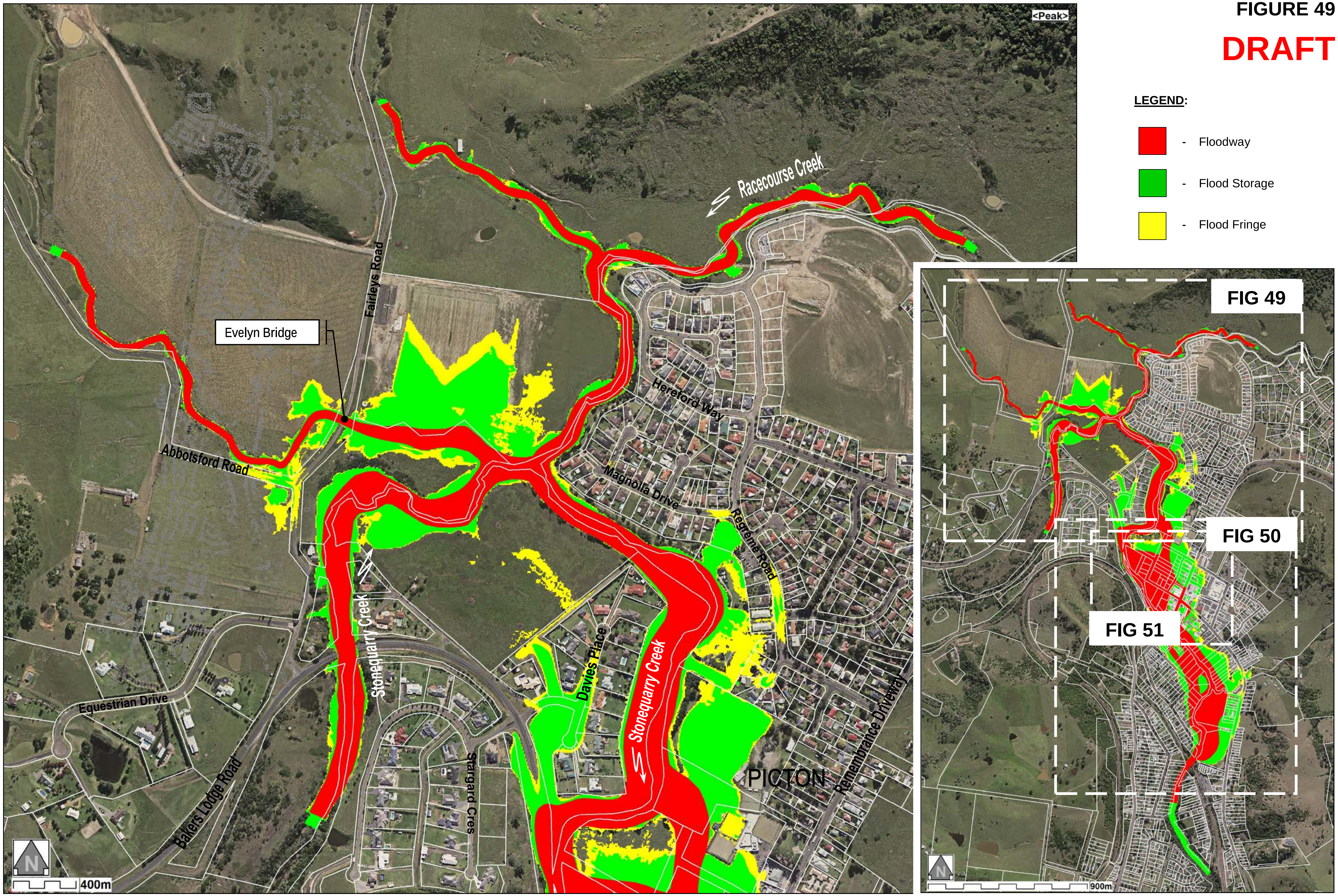


Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg130417_Fig48_Predicted Flood levels (1% AEP_Town).doc

PROVISIONAL FLOOD HAZARD MAPPING FOR THE 1% AEP FLOOD [SHEET 3 OF 3]

FIGURE 49

DRAFT

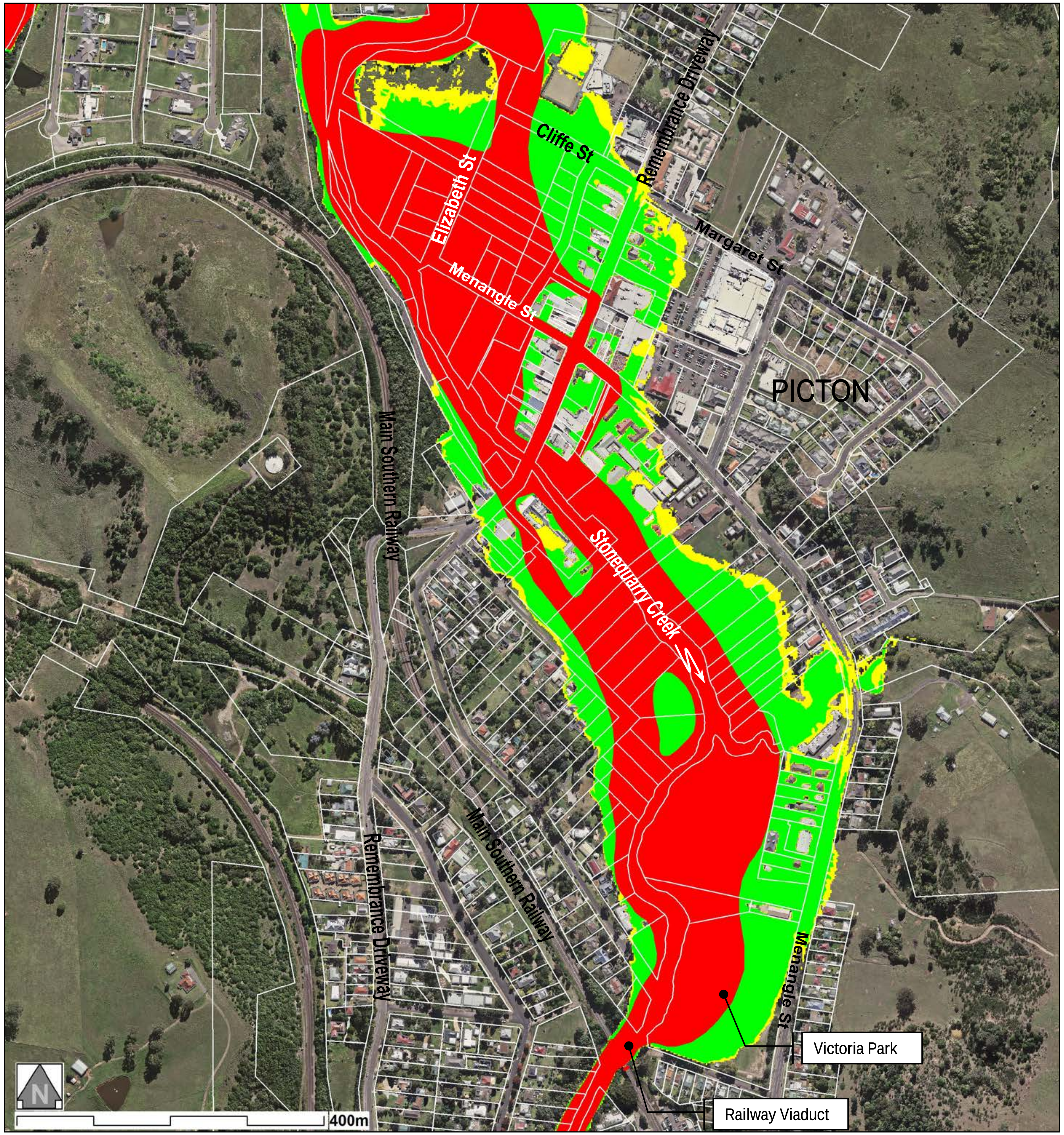


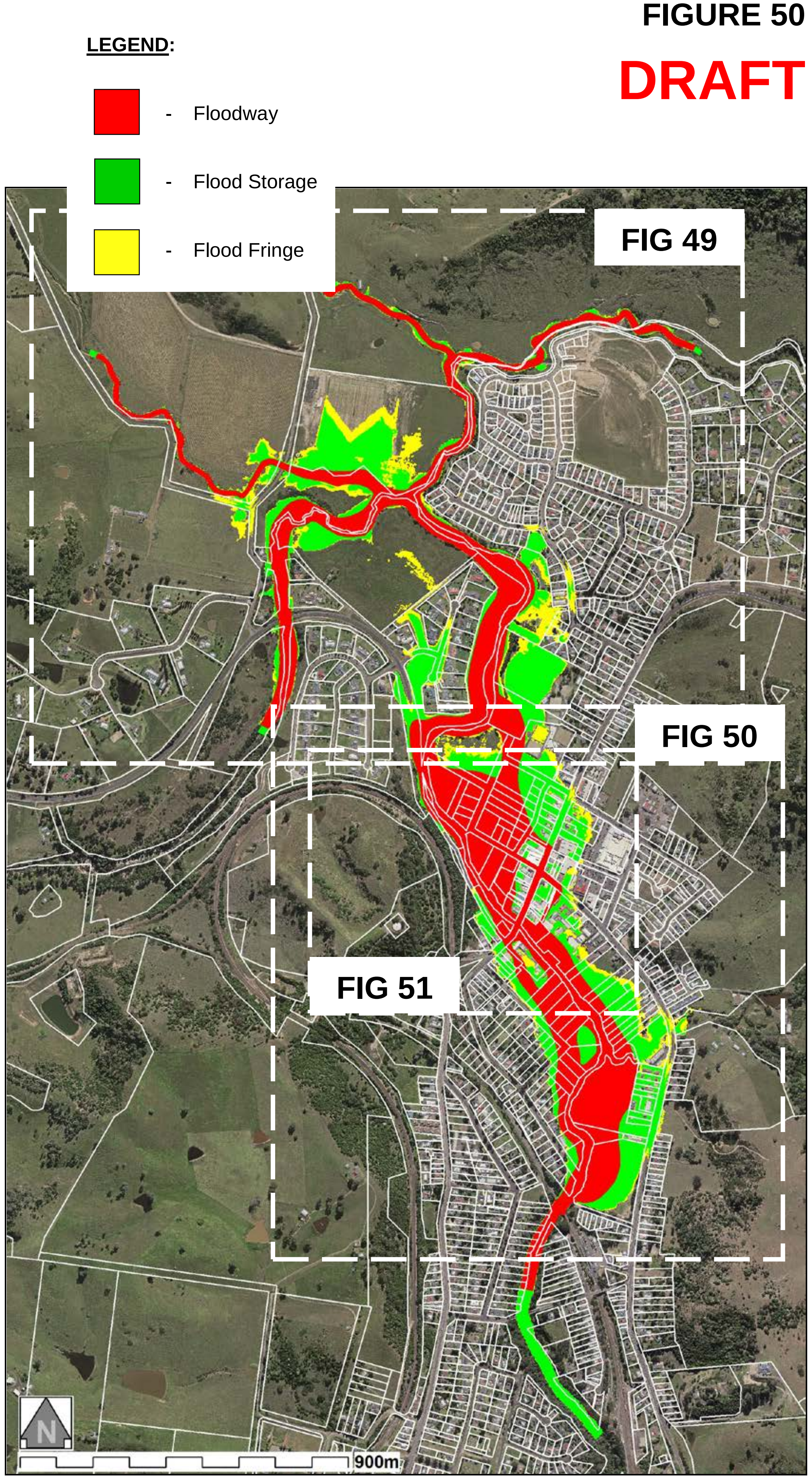
Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_Fig49_1% AEP Hydraulic Category Mapping (North).doc

HYDRAULIC CATEGORY MAPPING
[SHEET 1 OF 3]

FIGURE 50

DRAFT



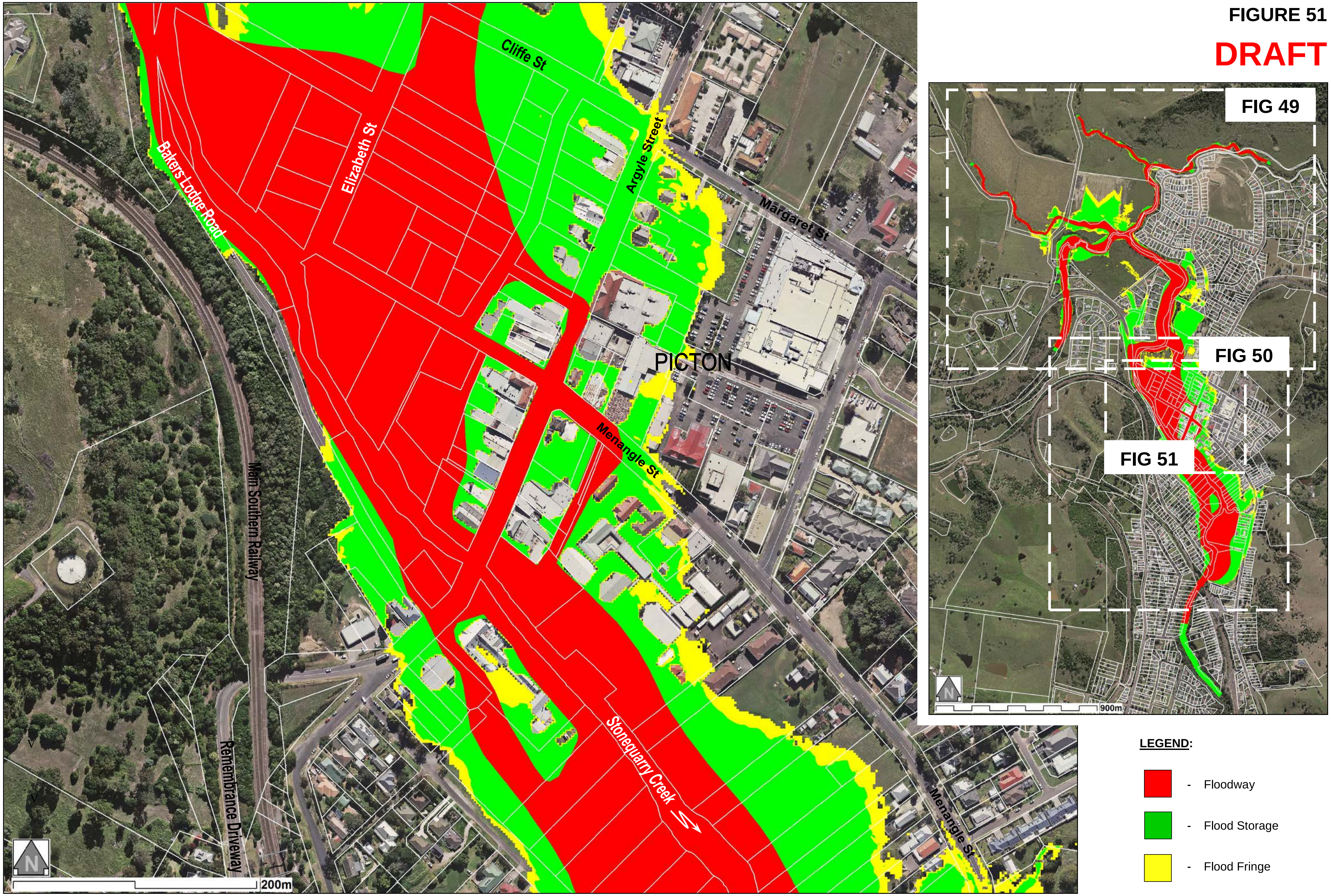


HYDRAULIC CATEGORY MAPPING
[SHEET 2 OF 3]

Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_Fig50_1% AEP Hydraulic Category Mapping (South).doc

FIGURE 51

DRAFT



HYDRAULIC CATEGORY MAPPING [SHEET 3 OF 3]

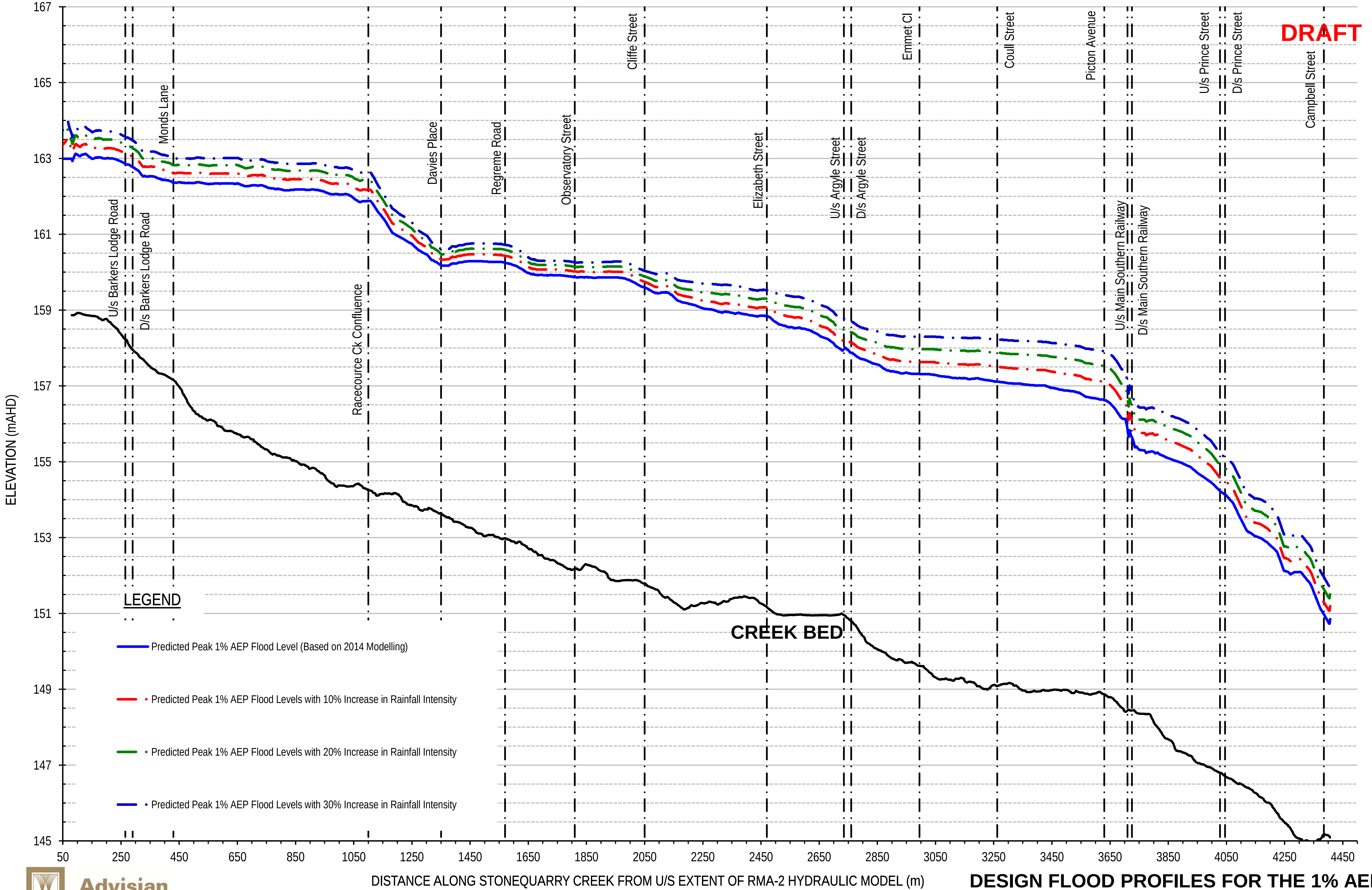
Advisian

WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling

fg301015-03199rg130417_Fig51_Predicted Flood levels (1% AEP_Town).doc

FIGURE 52



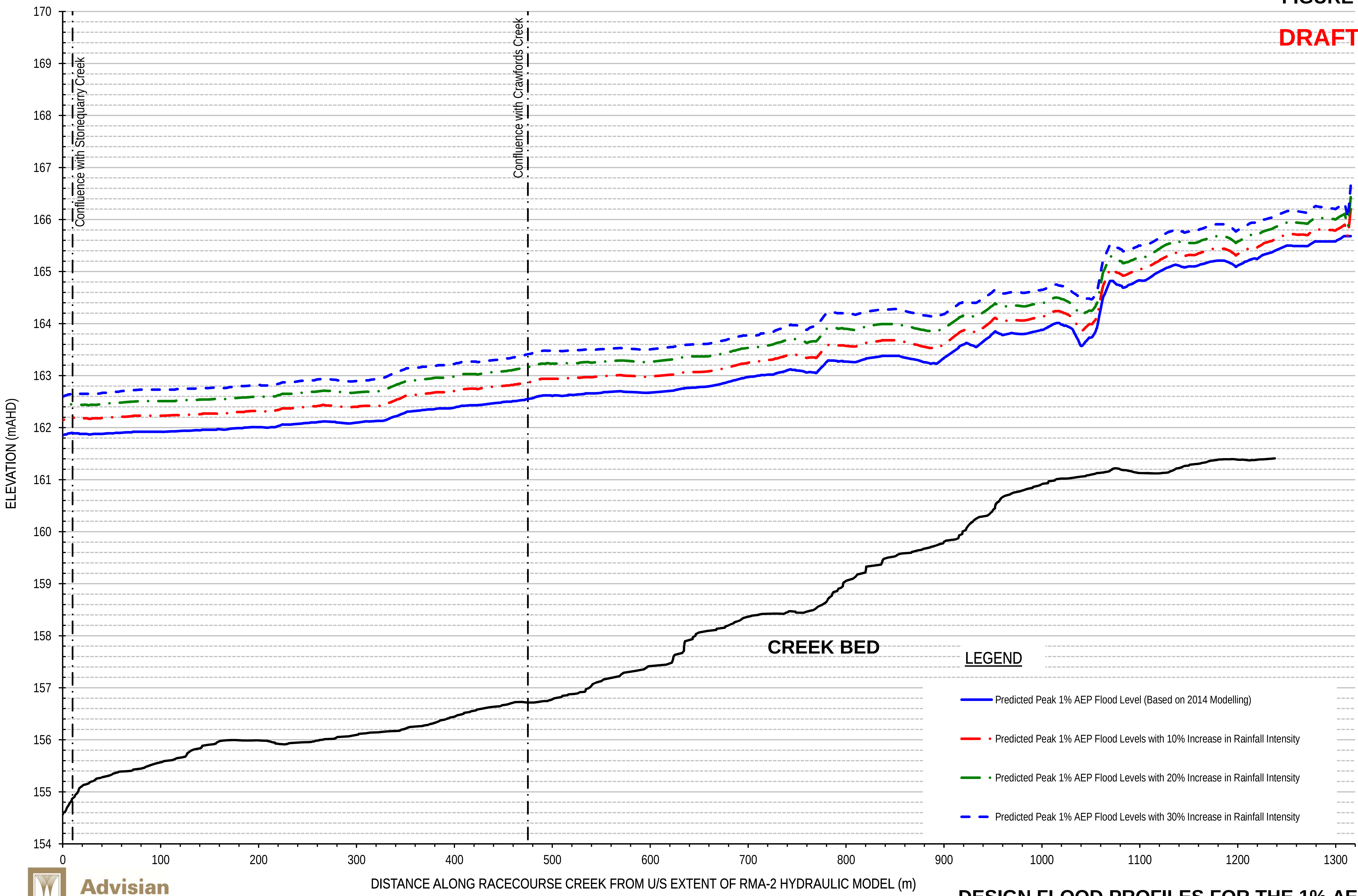
Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

DESIGN FLOOD PROFILES FOR THE 1% AEP CLIMATE CHANGE SCENARIOS ALONG STONEQUARRY CREEK

FIGURE 53

DRAFT

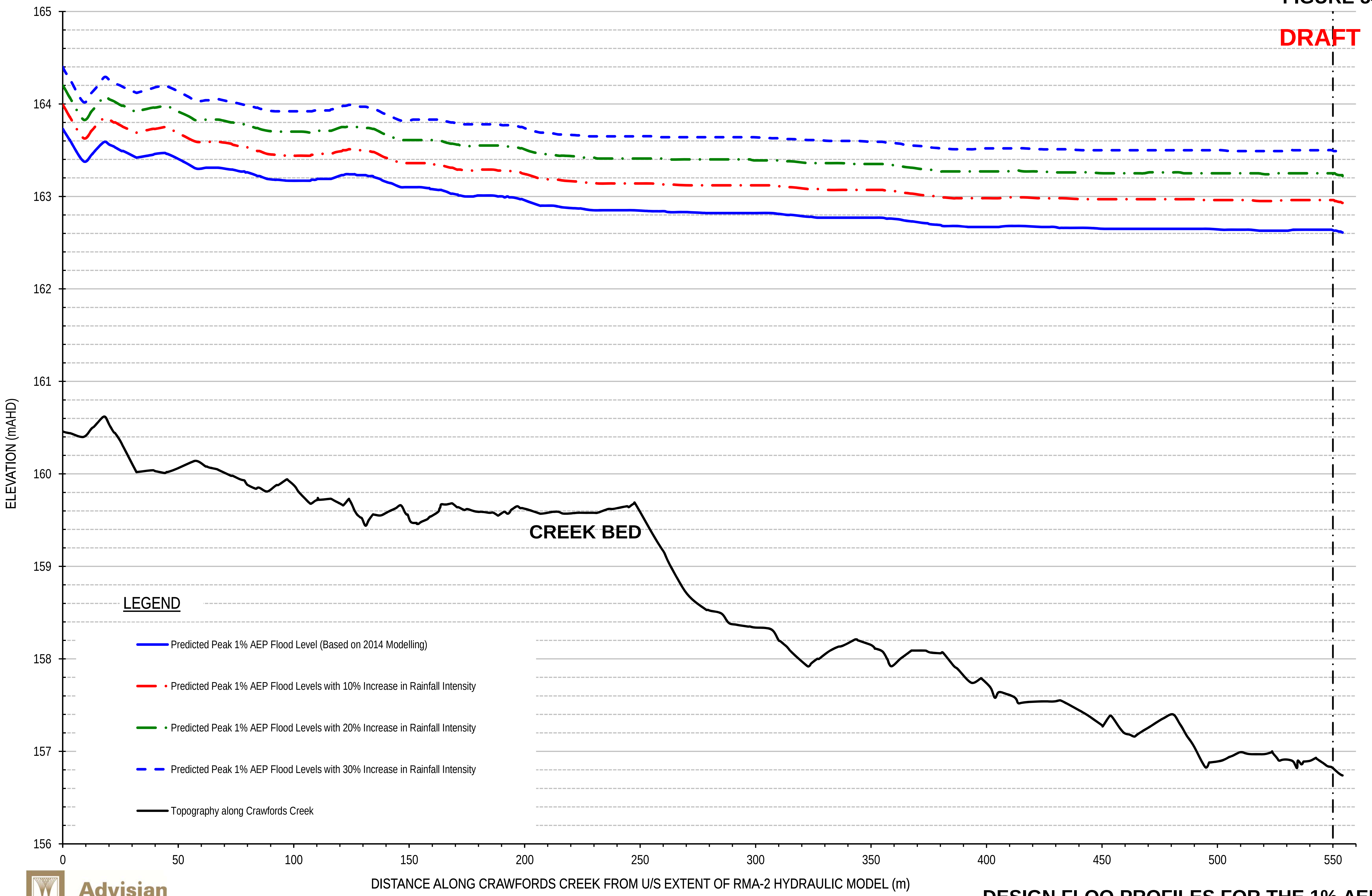


Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

DESIGN FLOOD PROFILES FOR THE 1% AEP CLIMATE CHANGE SCENARIOS ALONG RACECOURSE CREEK

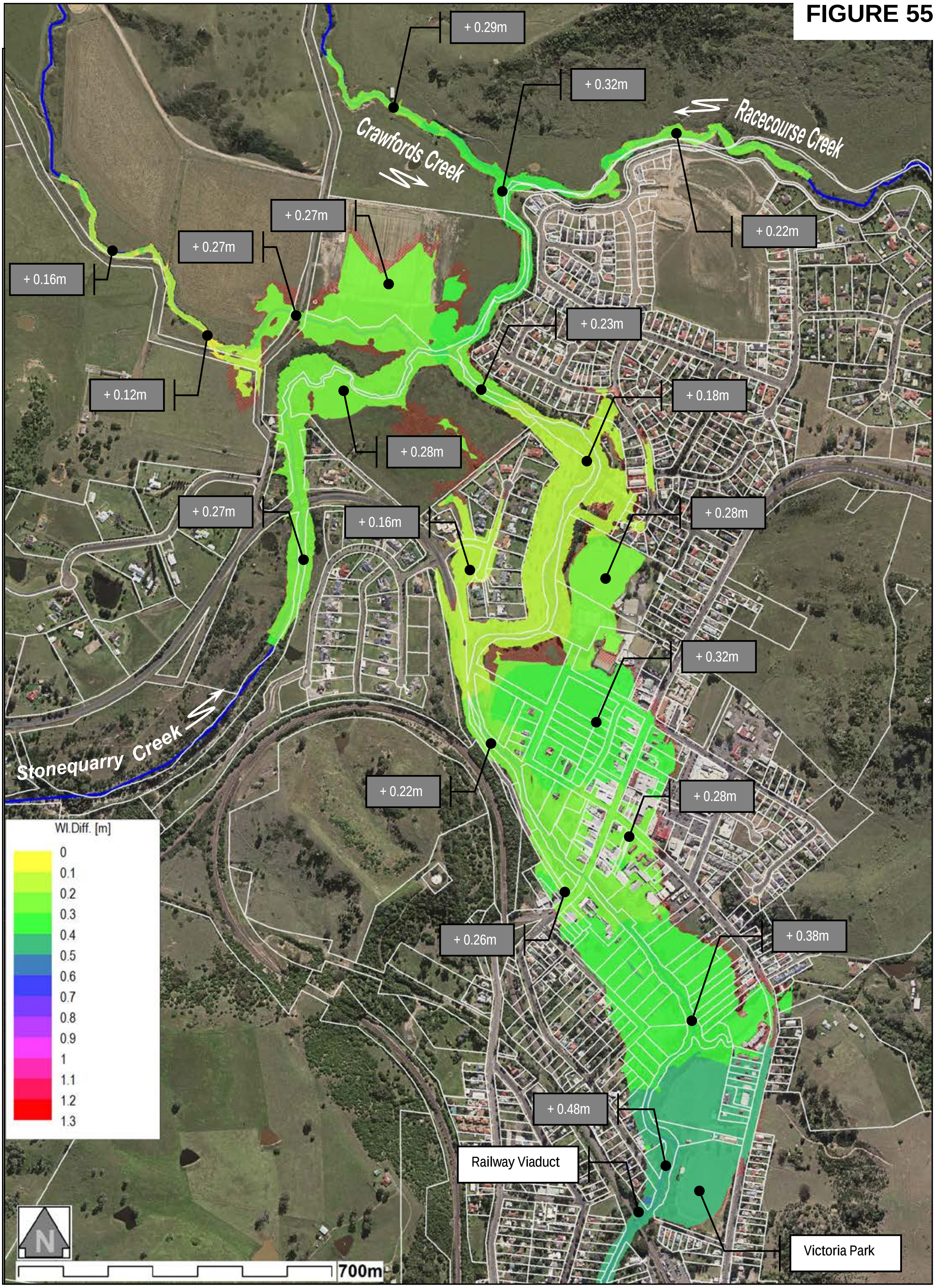
FIGURE 54



Advisian
WorleyParsons Group

301015-03199 - Stonequarry Ck FPRMS&P [Picton]
301015-03199rg130321_WSPs.xls

DESIGN FLOO PROFILES FOR THE 1% AEP CLIMATE CHANGE SCENARIOS ALONG CRAWFORDS CREEK

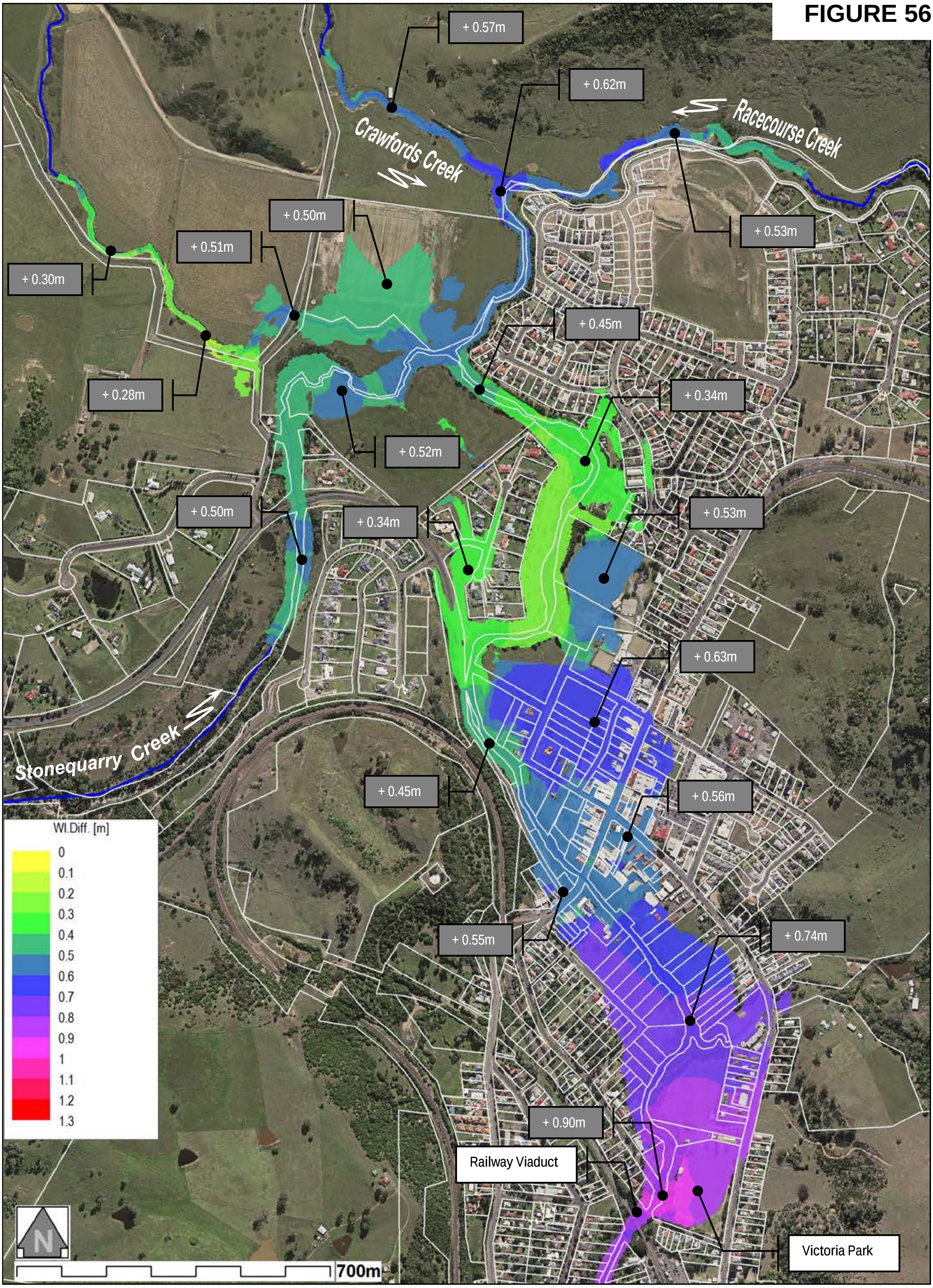


Advisian
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_Fig55_Level Increases (1% AEP + 10%).doc

PREDICTED INCREASES IN PEAK 1% AEP FLOOD LEVELS DUE TO 10% INCREASE IN RAINFALL INTENSITY

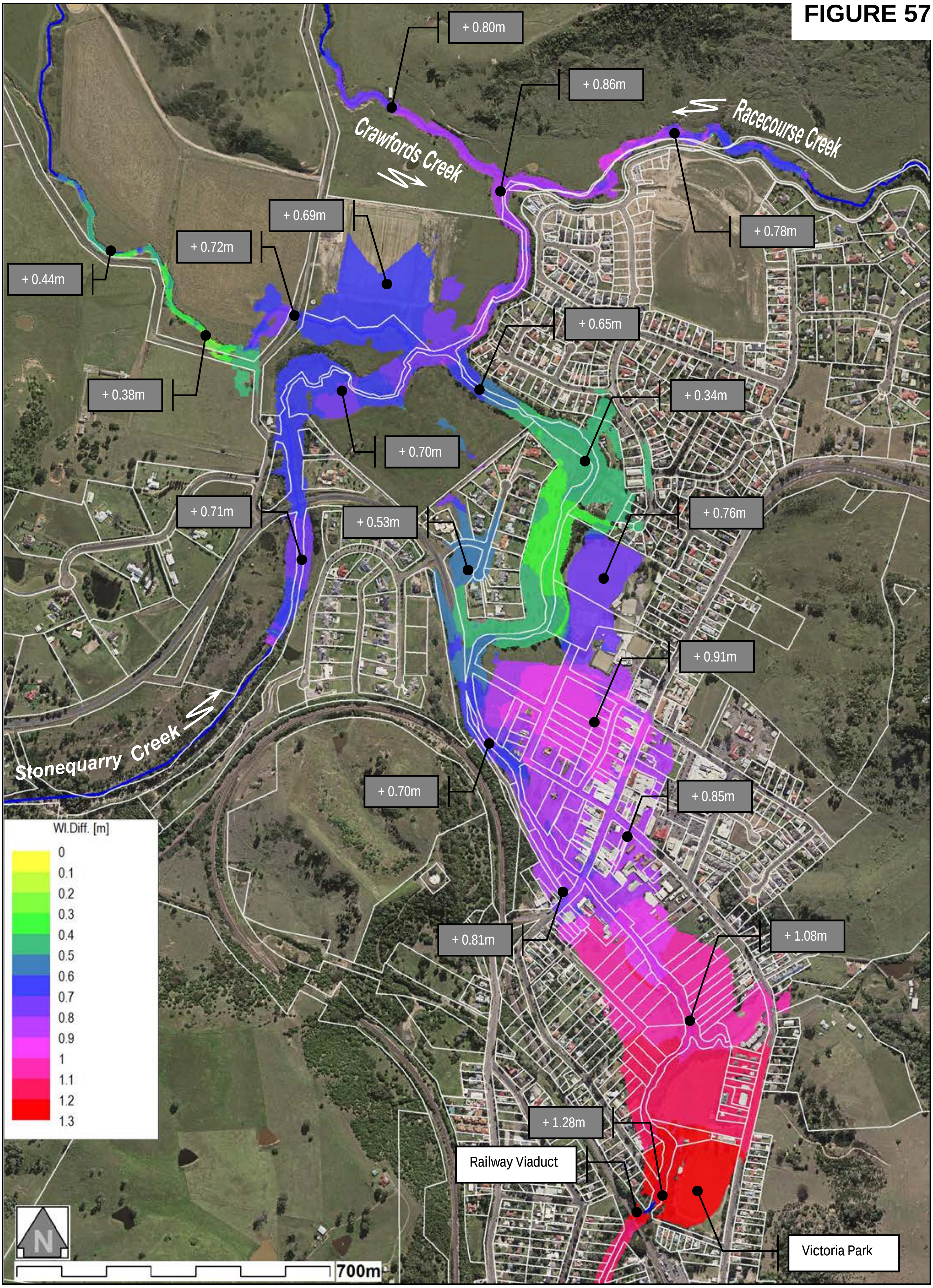
FIGURE 56



Advisian
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_Fig56_Level Increases (1% AEP + 20%).doc

PREDICTED INCREASES IN PEAK 1% AEP FLOOD LEVELS DUE TO 20% INCREASE IN RAINFALL INTENSITY



Advisian
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_Fig57_Level Increases (1% + 30%).doc

PREDICTED INCREASES IN PEAK 1% AEP FLOOD LEVELS DUE TO 30% INCREASE IN RAINFALL INTENSITY

Appendix A:
'Picton / Stonequarry Creek Flood Study Peer Review' (*Manly Hydraulics Laboratory, 2017*) with Advisian Response to Recommendations

APPENDIX A - FLOOD STUDY PEER REVIEW

A peer review of Issue 1 of the '*Picton/Stonequarry Creek Flood Study*' (*WorleyParsons, 2014*) was completed by Manly Hydraulics Laboratory (*MHL*) at the request of Wollondilly Shire Council. The objective of the peer review was to assess the key assumptions, procedures and conclusions made in the hydrologic and hydraulic modelling elements of the study and in the delineation of hazard and hydraulic categories.

The findings of the peer review are documented in the report titled '*Picton / Stonequarry Creek Flood Study – Flood Study Peer Review*' dated August 2017. This report is included at the end of **Appendix A**.

It is important to note that the peer review was undertaken based on a version of the Flood Study that pre-dated the June 2016 event. In that regard, the peer review does not include any comment on the hydrologic and hydraulic model validation that was completed with reference to data collected during the June 2016 event. The outcomes from the validation of the models to the data gathered from the June 2016 event are documented in this issue of the Flood Study (*Issue No 2*).

Section 6.2 of the peer review report provides a set of recommendations for consideration and inclusion in the Flood Study. As the peer review prepared following finalisation of the hydrologic and hydraulic models and all associated modelling, it was the decision of Council that not all recommendations would be addressed within the Flood Study; particularly those that would require updates to the models and modelling.

In lieu of the above, Council requested that Advisian provide comment on the recommendations made by MHL within the Flood Study. This commentary is provided in the following.

Recommendation 1: **The x year ARI terminology be amended to be in terms of AEP as discussed in (ARR, 2015).**

Response: Advisian agrees with this recommendation and has amended the Flood Study Report to include the AEP terminology.

Recommendation 2: **For future flood studies the catchment delineation should be revised with consideration of up-to-date topographic information.**

Response: Advisian agrees that this should be included as a matter of consideration in any future flood studies or any revisiting of the Floodplain Risk Management Study (*FRMS*). Although more recent topographic data could lead to some improvements in sub-catchment boundaries, we believe that the changes would be minimal and would therefore not have a significant impact on the peak flows at Picton.

Recommendation 3: **Amend Table 10 of the flood study report to show the critical duration as 9 hours and make clearer in the report that the critical duration for the PMF is 3 hours.**

Response: Advisian accepts this recommendation and has included the required changes in Issue 2 of the Flood Study.

Recommendation 4: Check if the mesh extent is limiting the flood extents. If so, increase the model extent accordingly.

Response: The RMA-2 mesh has been checked and is confirmed to be adequate for modelling of all events up to and including the Probable Maximum Flood (*PMF*). On this basis, no changes were made to the RMA-2 mesh between Issue 1 and Issue 2 of the Flood Study Report.

Recommendation 5: Review the DEM just north of the Stonequarry Creek and Racecourse Creek confluence (i.e., the larger orange and purple areas identified in Figure 4-1).

Response: The changes in topography identified by MHL are associated with surface re-grading undertaken in the construction of the sporting grounds located to the north-west of the confluence of Stonequarry and Racecourse Creeks. The flood modelling documented in this report indicates the area would be on the periphery of the floodplain for all events up to and including the 1% AEP flood. On this basis it is unlikely that changes in topography would have a significant impact on flood behaviour downstream of the confluence; i.e., around the Picton Town Centre.

Regardless of the above, any future Flood Studies or the FRMS should include a review of adopted topographic elevations against more up-to-date sources.

Recommendation 6: Revise Manning values adopted for roads, i.e., from n = 0.030 to n = 0.016 in accordance with (Chow, 1959).

Response: Advisian accepts this recommendation for inclusion in future versions of the RMA-2 model and iterations of the modelling. It is unlikely however that the revised Manning's or roughness value would cause any noticeable change in flood behaviour or flood characteristics.

Recommendation 7: Revised Manning values adopted within the creek channel as they appear to be on the low side.

Response: This recommendation is supported by the RMA-2 validation to the June 2016 event which shows that the RMA-2 model was generally under-predicting peak flood levels by between 0.13 and 0.21 metres. In that regard, increased roughness values for the creek channels would be expected to lead to an increase in peak flood levels which could improve the June 2016 validation.

Advisian agrees with the MHL recommendation for inclusion in future versions of the RMA-2 model and iterations of the modelling.

Recommendation 8: Add to the hydraulic model, hydrographs from sub-catchments 1.07, 1.08, 1.09 and 1.10.

Response: Advisian agrees with MHL that flows generated from the listed catchments should be included within the model. As the listed catchments cover parts of the floodplain within the extent of the RMA-2 model, the catchment flows could be included as '*local element inflows*'.

As the magnitude of flow generated from those local catchments is small compared to the total flow at those locations (*i.e., from all upstream catchments*) the change in levels if included is likely to be small. To test this, a sensitivity assessment was completed for the 1% AEP flood by including those local inflows. Peak 1% AEP flood levels generated from this simulation were compared to the 1% AEP results documented within this report. The comparison shows the local inflows would lead to an increase in 1% AEP flood levels within the Picton town centre of up to 50 mm and up to 150 mm immediately upstream of the Railway Viaduct.

It is recommended that all future iterations of the RMA-2 model and associated simulations include XP-RAFTS catchments 1.07, 1.08, 1.09 and 1.10 as '*local element inflows*'.

Recommendation 8: Future flood models should account for localised overland flooding.

Response: We do not consider this to be critical to the Flood Study given local overland flooding would not be a significant concern for most properties within the study area. It may be more appropriate to address overland flooding as part of a separate 'local overland flow' study or as part of the FRMS.

Recommendation 9: Undertake sensitivity of downstream tailwater levels to assess the affects.

Response: The downstream boundary for all simulations was based on a stage-discharge curve derived through '*normal-depth*' calculations completed at regular flow intervals. As the downstream boundary is located over 700 metres downstream of the Railway Viaduct and with a change in flood levels of approximately 4 metres, we do not believe levels upstream of the Viaduct will be sensitive to adjustments to the downstream boundary condition.

Recommendation 10: Validate / calibrate the model to recent or future flood data. This could be in the form of peak flood marks / levels observed by residents from recent flood events.

Response: The hydrologic (XP-RAFTS) and hydraulic (RMA-2) models have been validated against the June 2016 event subsequent to the above recommendation. The validation findings have been incorporated as part of updates made to produce Issue 2 of the Flood Study.

It is recommended that if any future modifications are made to the RMA-2 model (*such as those outlined in response to MHL Recommendations 5, 6 and 7*) than the June 2016 event be used as a calibration event to refine adopted Manning's roughness values. This will be particularly beneficial for the selection of appropriate channel roughness values which have already been identified as being on the 'low-side' (*refer MHL Recommendation 7*).

Recommendation 11: Amend the flood study to only use the standard flood hazard categories as defined in the NSW Floodplain Development Manual (2005), so as to align with Council's DCP.

Response: This recommendation was discussed with Council with the position reached that the flood hazard category mapping would not be updated. The adopted categories provide greater discretisation of hazards across the floodplain beyond the three standard categories of '*low*', '*transition*' and '*high*' provided in the Floodplain Development Manual (*FDM*) (*2005*).

Public Works
Manly Hydraulics Laboratory

Piction / Stonequarry Creek
Flood Study Peer Review

Report MHL2505
August 2017

Wollondilly Shire Council

A division of the Office of Finance and Services

Piction / Stonequarry Creek
Flood Study Peer Review

Report MHL2505
August 2017

Bronson McPherson
Principal Engineer Specialist Services
110b King Street
Manly Vale NSW 2093
T: 02 9949 0200
F: 02 9948 6185
E: bronson.mcpherson@finance.nsw.gov.au
W: www.mhl.nsw.gov.au

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Issue/ Revision	Author	Reviewer	Approved for Issue	
			Name	Date
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Contents

TABLES

FIGURES

1. Introduction

This report documents the Piction / Stonequarry Creek Flood Study Peer Review undertaken by NSW Public Works Manly Hydraulics Laboratory (MHL) for Wollondilly Shire Council (WSC). The latest version of the Picton / Stonequarry Creek Flood Study (WorleyParsons, 2014), was reviewed and is referred to in this report as the flood study.

The flood study was a result of numerous iterations, starting in 1989, due to changes within the catchment, modelling technology and the availability of detailed LiDAR survey data. The flood study utilised hydrological model XP-RAFTS (latest version as at 2005) and two-dimensional hydrodynamic model RMA-2. In addition, the 2014 flood study investigates the potential for climate change to impact peak 100 year ARI flood levels.

1.1 Scope and Objectives

The objective of the peer review was to assess key assumptions, procedures and conclusions. To achieve this, the following flood study attributes were reviewed:

- Hydrological
    - Hydrological Model
    - Sub-Catchments
    - Rainfall IFD
    - Losses
    - Hydrograph Volume Validation
    - Impervious Areas
    - Critical Durations
    - Climate Change
- Hydraulic
    - Hydraulic Model
    - Digital Elevation Model (DEM)
    - Model Network Mesh
    - 2D Materials / Roughness
    - Bridges
    - Model Flowpaths
    - Boundary conditions
    - Time Step
    - Mass Error
    - Calibration / Validation of Model
- Hazard and Hydraulic Categories

2. General

2.1 Correct AEP and ARI Terminology

Australian Rainfall and Runoff's (ARR) discussion paper on the preferred terminology for Average Reoccurrence Interval (ARI) and Annual Exceedance Probability (AEP) (ARR, 2015), states:

"The term "x year ARI" has caused confusion both within the industry, the community and other stakeholders. It has been interpreted by many to imply that the periods between exceedances of a given event magnitude.

The preferred new terminology is AEP and EY. Annual Exceedance Probability (AEP) where AEP expresses the probability of an event occurring or being exceeded in any year. Additionally, AEP are to be expressed as an exceedance probability using percentage probability; for example, the 1% AEP design flood discharge. Extreme flood probabilities associated with dam spillways are one example of a situation where percentage probability is not appropriate. In these cases, it is recommended that the probability be expressed as 1 in x AEP. Note that it is incorrect to express ARI as 1 in x year ARI or AEP as 1 in x year AEP.

For more frequent events an annualised exceedance probability is misleading and confusing. Furthermore, a recurrence interval approach also is misleading where strong seasonality is experienced. Consequently, events more frequent than 50% AEP should be expressed as x Exceedances per Year (EY). For example, 2 EY is equivalent to a design event with a 6 month recurrence interval when there is no seasonality in flood occurrence."

The flood study refers to flood events in terms of x year ARI. It is recommended that the terminology be amended to be in terms of AEP.

3. Hydrological

3.1 Hydrological Model

XP-RAFTS (*Version 7.0, 2008*) was used to undertake the hydrological modelling. XP-RAFTS is highly regarded hydrological modelling software typically used in flood studies throughout NSW and the world. Although the latest version of XP-RAFTS 2013 SP1 was not used, the 2008 version is considered suitable for the time of the modelling.

3.2 Sub-Catchments

A catchment plan is provided in Figure 2 of the flood study report. The catchment plan does not include topography and hence sub-catchment boundaries could not be confirmed. It is understood the sub-catchments were adopted for this study were originally defined for the 1989 flood study (Department of Water Resources, 1989). It is expected this to be suitable for the 2014 flood study, however it is recommended that for future flood studies the catchment delineation be revised with consideration of up to date topographic information.

3.3 Rainfall IFD

Intensity-Frequency-Duration (*IFD*) parameters were obtained from the Bureau of Meteorology (BoM). This would have been based in AR&R87. Since the study was undertaken BoM have released 2013 IFD design rainfalls.

BoM note that: (http://www.bom.gov.au/water/designRainfalls/ifd/transition-guidance.shtml)

> *"In most cases it would be prudent to use the AR&R87 design parameters and conduct sensitivity testing with revised AR&R design parameters (including the 2013 IFD design rainfalls) as they become available.*
>
> *The 2013 IFD design rainfalls should definitely **NOT** be used in conjunction with the following techniques:*
>
> - *Probabilistic Rational Method*
> - *Other regional flood techniques based on AR&R87 IFD design rainfalls."*

IFD from AR&R87 is considered suitable for this flood study. However, a recommendation for future improvements to the model would be to conduct sensitivity testing with revised AR&R design parameters (including the 2013 IFD design rainfalls).

A comparison of IFD values adopted in the Flood Study and values extracted by MHL for this review is provided in **Table 3-1**. The comparison found the Flood Study's IFD values are generally higher than values obtained from BoM's IFD tool for this review, but close enough to be insignificant.

Table 3-1 IFD data comparison – ARR87

Source	Flood Study (WP 2014)	Check with BoM's IFD tool (MHL 2016)			
Location	Picton	Picton township	Stonequarry Creek centre	Upper Western Catchment	Upper Eastern Catchment
Easting, Northing	NA	34°10'08", 150°36'42"	34°10'32", 150°33'33"	34°12'47", 150°30'47"	34°09'18", 150°39'28"
2yr 1hr	30.00	29.68	29.00	28.74	30.00
2yr 12hr	7.30	6.90	6.88	7.02	6.71
2yr 72hr	2.05	1.96	1.95	1.98	1.94
50yr 1hr	60.6	59.77	58.67	59.16	59.87
50yr 12hr	13.8	13.57	13.47	13.50	13.12
50yr 72hr	4.72	4.68	4.63	4.60	4.54
skew	0.00	0.02	0.02	0.02	0.01
F2	4.29	4.29	4.29	4.29	4.29
F50	15.76	15.77	15.76	15.76	15.78

3.4 Losses

The model adopted an initial loss of 15 mm and a continuing loss rate of 1.5 mm/hr and separate infiltration loss rates were incorporated for urban areas with initial loss of 2.5 mm and continuing losses of 0.5 mm/hr.

Book Two – Design Rainfall Considerations (ARR, 1987) recommends initial loss values of 10 to 35mm (varying with catchment size and mean annual rainfall) and a continual loss of 2.5mm/hr which references Cordery (1970a), Cordery and Webb (1974) and Avery (1983). In comparison the flood study values appear reasonable and although on the low side the losses would be considered conservative when defining design floods level and extents.

3.5 Hydrograph Volume Validation

Hydrographs were checked to see if they appear reasonable (refer to **Table 3-2**). The check involved comparing of the following:

- **Total hydrograph volumes** – sum of the hydrograph volumes input to the hydraulic model (refer to **Section 4.7**). This includes total hydrographs from sub-catchments 1.06, 6.04, 5.01 and 4.02, but excludes runoff from sub-catchments 1.07, 1.08, 1.09 and 1.10.
- **Total rainfall volumes** – volumes based on rainfall hyetographs, losses and catchment area, i.e. rainfall – losses x catchment area. The catchment area (79.13 km²) only accounts for area attributing runoff to sub-catchments 1.06, 6.04, 5.01 and 4.02

Factors not accounted for in these calculations include tailing out flows and impervious areas, however these were relatively minor. The assessment indicates that there does not appear to be any gross errors in the calculations of the hydrographs.

Table 3-2 Volume Comparison (kL)

ARI	Total hydrographs	Total rainfall hyetographs	% difference
5 year	5611	5546	-1.2%
20 year	7744	7712	-0.4%
50 year	9348	9345	0.0%
100 year	10551	10585	0.3%
200 year	11701	11837	1.1%
500 year	13359	13518	1.2%

3.6 Impervious Areas

Catchments with no urban areas were assigned 0% impervious and catchments with urban areas were assigned a % impervious accordingly. The flood study report notes that the model was updated for this flood study to account for increased urbanisation within the catchment. This was appropriate considering the catchment changes since the original model.

3.7 Critical Durations

The flood study report states that the critical duration was determined to be 9 hours. However, in Table 10 of the flood study report the peak design inflows for the RMA-2 model are the 6 hours critical duration for design events and the 3 hours for the PMF. This needs to be clarified in the flood study report.

3.8 Climate Change

Climate change was accounted for by applying 10%, 20% and 30% increases to the rainfall intensities for the 100 year ARI event which is standard practice in accordance with the guidelines, Practical Consideration of Climate Change (NSW Department of Environment and Climate Change, 2007).

4. Hydraulic

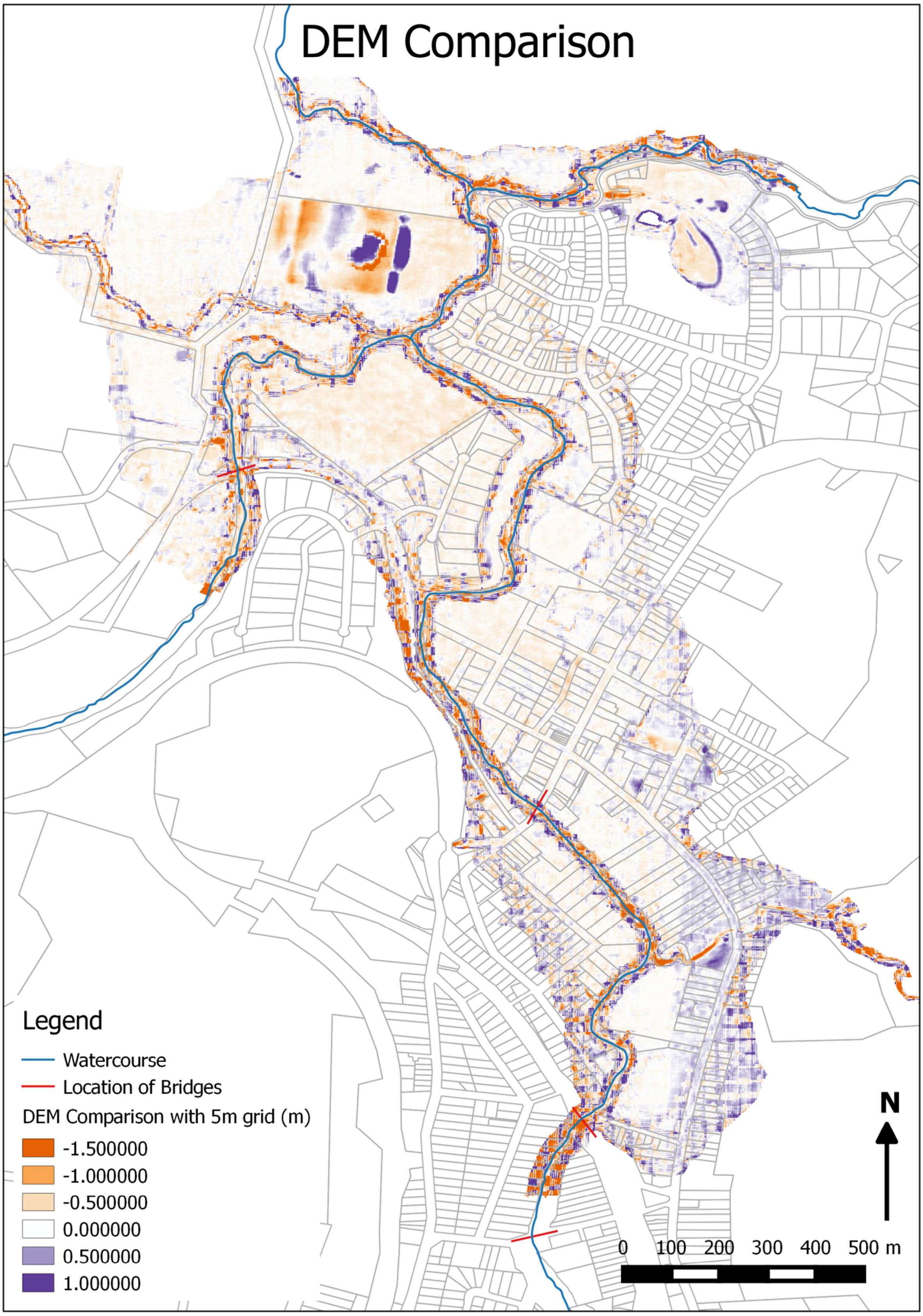
4.1 Hydraulic Model

2D hydraulic model RMA-2 was used to model the study area. RMA-2 is a widely renowned and utilised model for this type of application. The flood study model uses a reasonably current version of RMA-2.

4.2 Digital Elevation Model (DEM)

The DEM generated from the model mesh was compared with 5m grid surface data obtained from Geoscience Australia. The comparison is provided in **Figure 4-1**. Areas of level variations are noted along the creek embankments. However, these are likely a result of the relatively course 5m grid in steep areas. The main areas of concern are the larger orange and purple areas just north of the Stonequarry Creek and Racecourse Creek confluence. A check of the aerial imagery shows these areas to be fields and hence buildings are not the cause of the discrepancy. It is recommended that the DEM in this area be reviewed.

Figure 4-1 Comparison of model DEM and Geoscience DEM



4.3 Model Network Mesh

The model mesh is illustrated in Figure 5 of the flood study report where it demonstrates the model detail has increase from 4,200 nodes for the original RMA-2 model to 27,500 for the latest model. As stated in the flood study report, *this substantial increase in model nodes reflects the level of additional topographic detail that has been incorporated into the RMA-2 model*.

The mesh is of reasonable detail and follows the contours of the bed levels. The mesh was compared with the guidelines for typical 2D element resolution provided in Table 10-2 of Project 15: Two Dimensional Modelling in Urban and Rural Floodplains (ARR, 2012). The mesh elements are consistent with the guidelines and include:

- At least 5 mesh elements laterally across the channel
- 5 m to 10 m mesh elements in urban areas
- 5 m to 15 m mesh elements in rural areas

The Mesh extends approx. 300 m upstream of Bakers Lodge Road Bridge on Stonequarry Creek. This is a suitable distance considering the bridge will control the primary inflows to the model. The mesh also extends, on Racecource Creek, approx. 1000 m upstream of the confluence with Stonequarry Creek. This is a suitable distance for realistic flow patterns to form. Downstream of the railway bridge the model extends approx. 700 m and includes the Prince Road Bridge. This is a suitable distance for realistic tailwater levels to form at the Railway Bridge.

The model mesh was laid over the 100 year ARI and PMF flood extents as shown in **Figure 4-2** and **Figure 4-3**. The PMF extent bounds the majority of the mesh extent and the 100 year ARI extent bounds numerous sections of the mesh extent. It is possible the mesh was cropped based on the PMF extent but it is not clear if the mesh extent is limiting the flood extents.

It is understood the study area is defined as the centre of Picton and surrounding urban areas, upstream of the railway bridge, which are susceptible to flooding from Stonequarry Creek. This excludes areas downstream of the Railway Bridge, areas flooded by Racecourse Creek and areas flooded by overland flooding. Should the study area include these areas, the model extent would need to be revised.

Figure 4-2 Model Mesh and 100 year ARI Flood Extent

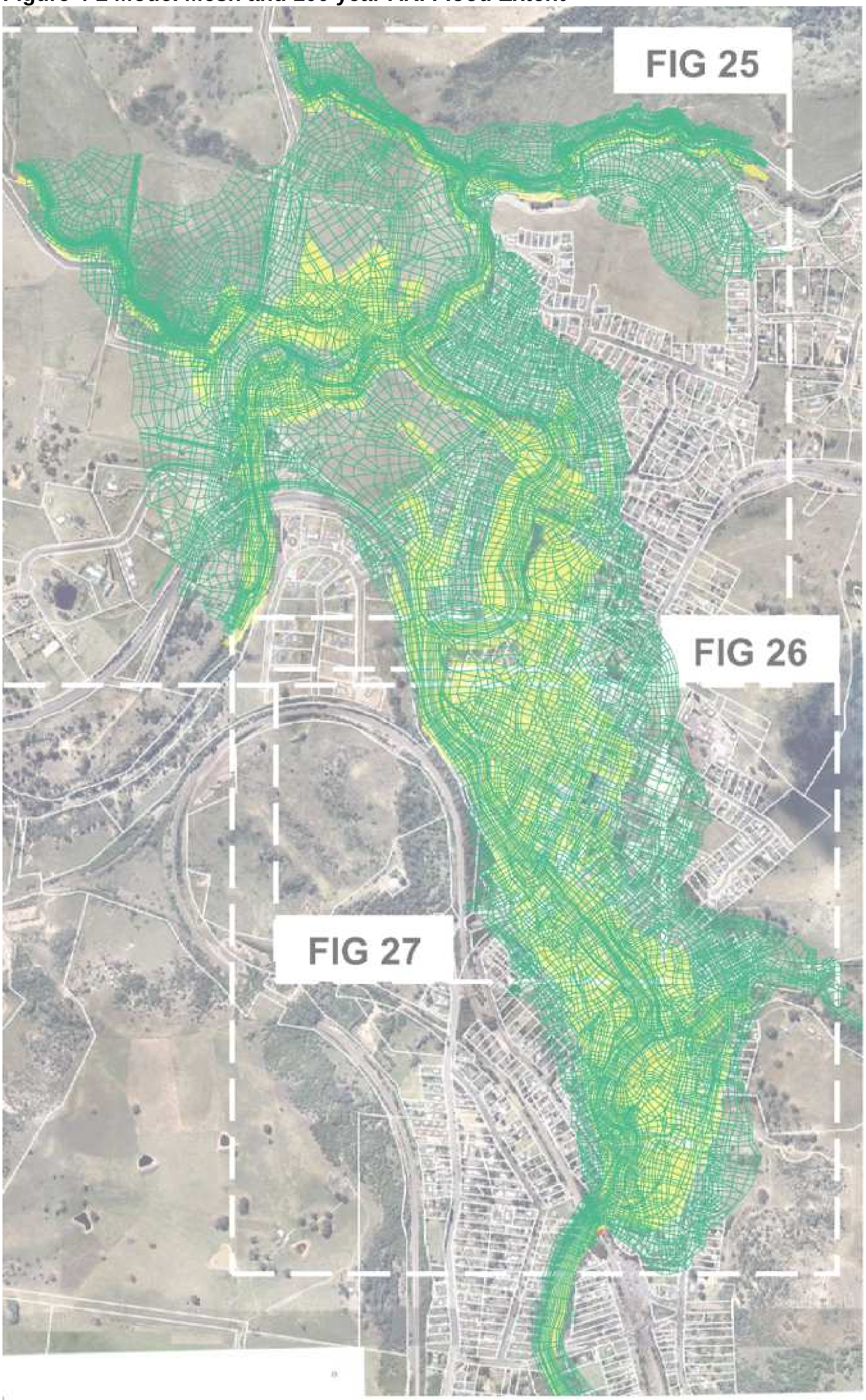
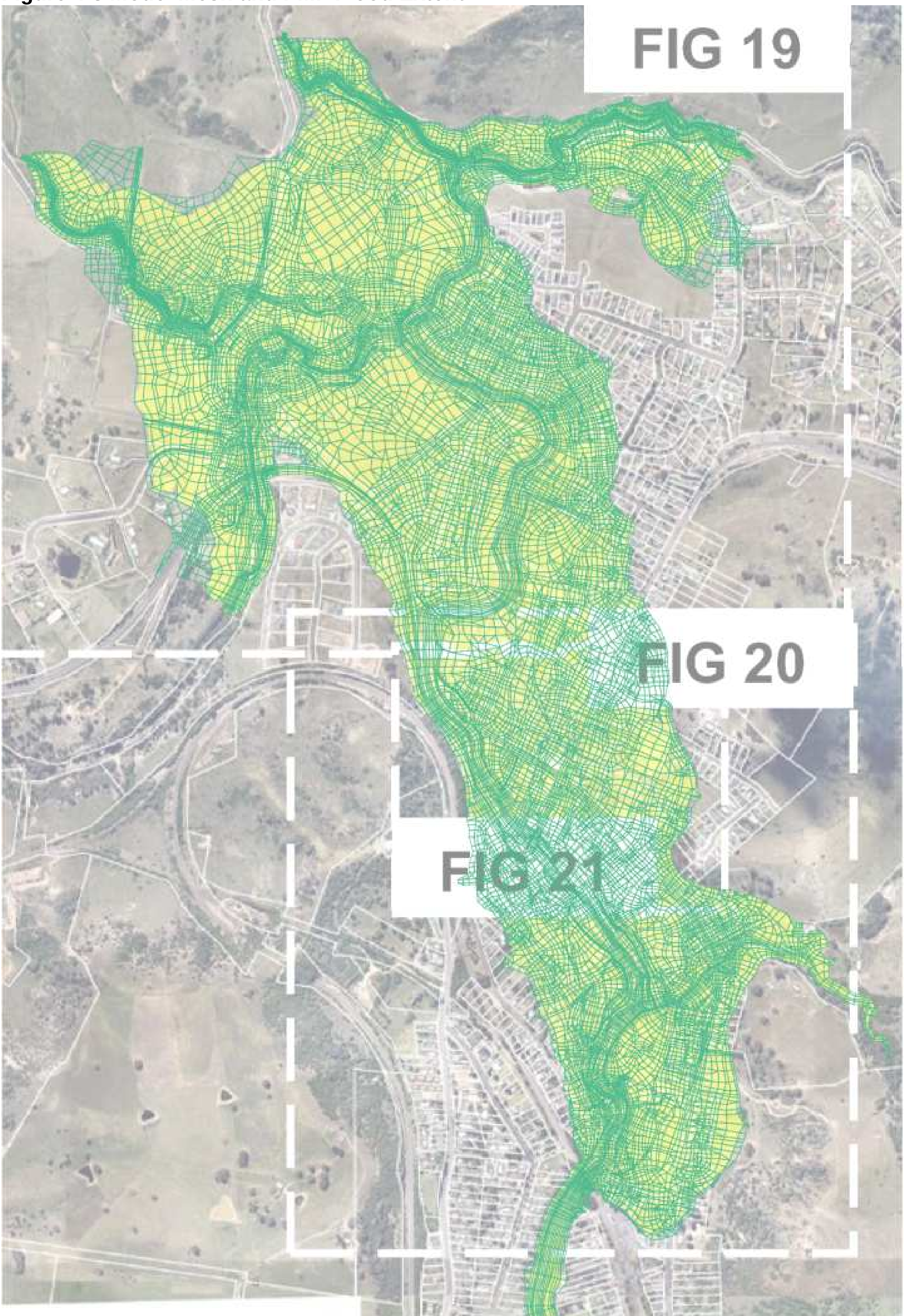


Figure 4-3 Model Mesh and PMF Flood Extent



4.4 2D Materials / Roughness

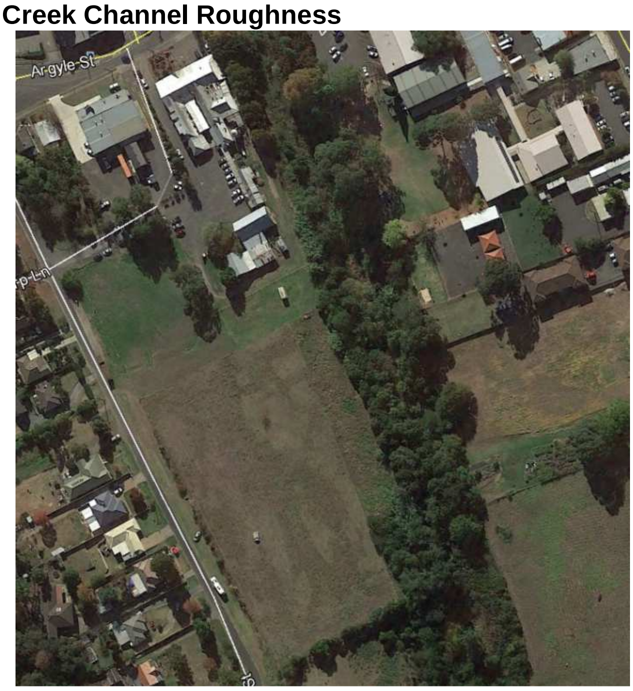
2D materials / roughness are illustrated in Figures 7, 8 and 9 of the flood study report and are provided in **Table 4-1**. Mannings roughness values were compared with typical values provided in Table 10-1 of Project 15: Two Dimensional Modelling in Urban and Rural Floodplains (ARR, 2012). Generally, the flood study values were within the typical ranges.

However, when compared with descriptions and values provided in Manning's n for Channels (Chow, 1959) and aerial images, the values appeared to be on the low side. For example, the section of creek shown in **Figure 4-4**, was assigned a Mannings n = 0.040, according to (Chow, 1959) this would be equivalent to *"clean, winding, some pools and shoals"*. This is one of many areas where the Manning's roughness value appears to be on the low side. Also according to (Chow, 1959) rough asphalt is n = 0.016 which suggests the value adopted in the flood study (n = 0.030) may be too high.

Table 4-1 2D materials/roughness

Material	Mannings n
Clear River/Creek Channel	0.030
Moderately Vegetated Channel	0.040
Densely Vegetated Channel	0.060
Grassed Floodplain	0.040
Floodplain with Sparse Trees	0.060
Floodplain with Dense Trees	0.075
Roads	0.030
Industrial	0.065
Residential/Urban	0.055
Buildings	Blocked

Figure 4-4 Example of Creek Channel Roughness



4.5 Bridges

Review of the DEM and flood study report found the bridges were modelled as follows:

- Embankments, approaches and wing wall abutments were based on design drawings and/or survey.
- Where design drawings were not available bridge waterways were defined based on a combined analysis of the LiDAR data and available aerial photography.
- Roughness parameters in the vicinity of the bridge under croft and major culverts were set to represent the energy and friction losses that would have been caused by the presence of bridge piers and the bridge deck.
- The Railway Viaduct piers were large enough to be picked-up within the DEM model network and were blocked out individually instead of using roughness parameters.
- The DEM shows that the invert of channels at the bridges continue linearly from the upstream to the downstream side of the bridge. The only notable differences to the channel profile were at the abutments.

This approach is appropriate given the available information.

4.6 Model Flowpaths

The DEM comparison provided in **Figure 4-1**, demonstrates there are no unexpected variances within the creek channels.

4.7 Boundary conditions

Boundary conditions are illustrated in Figure 6 of the flood study report. The upstream boundary conditions included inflow hydrographs corresponding to the location of inflows into the creek system (i.e. flows into Stonequarry, Racecourse, Crawfords and an unnamed creek). These hydrographs were obtained from the hydrological model discussed in **Section 3**.

Total outflow hydrographs from XP-RAFTS for sub-catchments 1.06, 6.04, 5.01 and 4.02 were adopted as upstream boundary conditions. However, sub-catchments downstream which contribute runoff to the study area do not appear to have been accounted for and include sub-catchments 1.07, 1.08, 1.09 and 1.10. This is 3.53 km² not accounted for out of the total catchment 84 km² which is approx. 4.2%.

The downstream boundary condition involved a stage-discharge relationship based on 'normal-depth' calculations in using a channel slope extracted from the available LiDAR data. This is an appropriate method given the available information and considering the boundary condition was set approx. 700 m downstream of the Railway Bridge and 400 m downstream of the Prince Street Bridge, as discussed in **Section 4.3**, which improves the likelihood that realistic tailwater levels would have formed at the Railway Bridge.

However, it is noted that sensitivity of the downstream tailwater levels was not undertaken and would be advised to assess the affects.

4.8 Time Step

The simulation runs are consistently setup and adopt very short time steps (0.005 seconds). The case examined (5 year ARI), shows reasonable flood development with few if any signs of oscillation.

4.9 Mass Error

The model run is stable and the resulting initial conditions appear reasonable. As a test, total inflow was compared with the computed flow leaving the system. The results are fully consistent.

4.10 Calibration / Validation of Model

The flood study report notes that the modelled 100 Year ARI flood levels were compared with those determined in the 1989 HEC-2 results. WSC made note that there is a fair degree of knowledge behind the 1989 model despite not being calibrated to real data.

The flood study report notes there is limited historic flood level, stream flow and/or rainfall data. However, since the study was undertaken in 2014 there was a significant flood event in which data would likely be available. Such data may be in the form of peak flood marks or levels noted by residence.

WSC noted that higher probability floods extend out of the creek more than expected given the absence of (observed) occurrences. Validating the model using recent or future flood data would help clarify this observation.

5. Hazard and Hydraulic Categories

The flood study includes additional flood hazard categories to the standard ones specified in the NSW Floodplain Development Manual (2005) and subsequently Council's DCP. It is recommended the flood study be changed to only use the standard flood hazard categories as defined in the NSW Floodplain Development Manual (2005) so as to align with Council's DCP.

"Flood Storage" is a new hydraulic category (specified in the NSW Floodplain Development Manual (2005)) not defined by previous flood studies. This does not impact Council's DCP definition for Flood Risk Precincts, because Council's DCP defines the precincts with regards to the hydraulic hazard category as specified in the NSW Floodplain Development Manual (2005), where the new hydraulic category "Flood Storage" is specified.

6. Conclusions and Recommendations

6.1 Conclusions

Following review of the Piction / Stonequarry Creek Flood Study it was concluded that:

- Although the latest version of XP-RAFTS 2013 SP1 (hydrological model) was not used the 2008 version is considered suitable for the time of the modelling.
- Catchment delineation was adopted from the 1989 flood study and is considered suitable for the latest flood study.
- The flood study's IFD values are generally higher than values obtained from BoM's IFD tool for this review, but are close enough to be insignificant.
- Rainfall losses are reasonable and although on the low side, they are considered conservative when defining design floods level and extents.
- The hydrograph volume validation assessment found there does not appear to be any gross errors in the calculations of the hydrographs.
- Designations of impervious areas were found to be appropriate.
- Climate change was assessed in accordance with standard practice.
- 2D hydraulic model RMA-2 was used to model the study area. RMA-2 is a widely renowned and utilised model for this type of application and is therefore considered suitable for this application.
- The mesh is of reasonable detail, follows the contours of the bed levels and is consistent with guidelines set out in (ARR, 2012).
- The PMF extent bounds the majority of the mesh extent and it is not clear if the mesh extent is limiting the flood extents.
- The hydraulic model only accounts for riverine flooding and does not account for localised overland flooding.
- Manning's roughness values are within typical ranges identified in (ARR, 2012). However, values appear on the low side when comparing with (Chow, 1959) and aerial images.
- The approach for modelling the bridges is appropriate given the available information.
- Flow paths are appropriate and there are no unexpected variances within the creek channels.
- Upstream boundary conditions do not account for runoff over the study area.
- Downstream boundary conditions, i.e. tailwater levels are appropriate but no sensitivity analysis was undertaken to assess the affects.
- The simulation runs are consistently setup and with the use of very short time steps (0.005 seconds).
- The model run is stable and the resulting initial conditions appear reasonable.
- The modelling has not (yet) been validated / calibrated, due to limited availability of data.
- The flood study includes additional flood hazard categories to the standard ones specified in the NSW Floodplain Development Manual (2005) and subsequently Council's DCP.

- “Flood Storage” is a new hydraulic category (specified in the NSW Floodplain Development Manual (2005)) and does not impact Council’s DCP definition for Flood Risk Precincts.

6.2 Recommendations

Following review of the Piction / Stonequarry Creek Flood Study it is recommended that:

- The x year ARI terminology be amended to be in terms of AEP as discussed in (ARR, 2015)
- For future flood studies the catchment delineation should be revised with consideration of up to date topographic information.
- Amend Table 10 of the flood study report to show the critical duration as 9 hours and make clearer in the report that the critical duration for the PMF is 3 hours.
- Check if the mesh extent is limiting the flood extents. If so, increase the model extent accordingly.
- Review the DEM just north of the Stonequarry Creek and Racecourse Creek confluence (i.e. the larger orange and purple areas identified in **Figure 4-1**).
- Revise Manning values adopted for roads, i.e. from n =0.030 to n = 0.016 in accordance with (Chow, 1959)
- Revise Manning values adopted within the creek channel as they appear to be on the low side.
- Future flood models should account for localised overland flooding.
- Add to the hydraulic model, hydrographs from sub-catchments 1.07, 1.08, 1.09 and 1.10.
- Undertake sensitivity of the downstream tailwater levels to assess the affects.
- Validate / calibrate the model to recent or future flood data. This could be in the form of peak flood marks / levels observed by residents from recent flood events.
- Amend the flood study to only use the standard flood hazard categories as defined in the NSW Floodplain Development Manual (2005), so as to align with Council’s DCP.

7. References

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Appendix B:
Adopted Parameters for XP-RAFTS Model

TABLE B1: IFD ANALYSIS BASED ON AUSTRALIAN RAINFALL & RUNOFF 1987

Site Location: Picton

Geographical factor for 6 min 2 yr storm = 4.29
Geographical factor for 6 min 50 yr storm = 15.76
Skewness = 0.00

2 Year ARI:
1 hour intensity = 30.0 mm/hr
12 hour intensity = 7.3 mm/hr
72 hour intensity = 2.05 mm/hr

50 Year ARI:
1 hour intensity = 60.6 mm/hr
12 hour intensity = 13.8 mm/hr
72 hour intensity = 4.72 mm/hr

IFD Table for Various ARIs and Duration

Duration	1 yr (mm/hr)	2 yr (mm/hr)	5 yr (mm/hr)	10 yr (mm/hr)	20 yr (mm/hr)	50 yr (mm/hr)	100 yr (mm/hr)	200 yr (mm/hr)	500 yr (mm/hr)
5 mins	75.53	97.72	127.32	144.69	167.42	197.3	220.13	243.35	274.73
6	70.8	91.59	119.34	135.63	156.93	184.94	206.33	228.1	257.52
10	57.85	74.84	97.51	110.82	128.23	151.12	168.6	186.39	210.43
15	48.28	62.46	81.38	92.49	107.03	126.13	140.73	155.57	175.64
20	42.04	54.39	70.87	80.54	93.2	109.83	122.54	135.47	152.95
30	34.12	44.14	57.52	65.37	75.65	89.15	99.47	109.96	124.15
45	27.32	35.35	46.06	52.35	60.58	71.4	79.66	88.07	99.43
1 hour	23.19	30	39.09	44.43	51.42	60.6	67.61	74.75	84.39
1.5	18.55	23.96	31.09	35.26	40.72	47.89	53.36	58.92	66.42
2	15.79	20.37	26.34	29.83	34.4	40.4	44.97	49.61	55.86
3	12.54	16.15	20.8	23.5	27.05	31.7	35.23	38.81	43.63
4.5	9.95	12.79	16.4	18.48	21.24	24.83	27.56	30.32	34.03
6	8.44	10.84	13.86	15.59	17.89	20.88	23.15	25.45	28.53
9	6.71	8.6	10.94	12.28	14.06	16.38	18.14	19.91	22.29
12	5.7	7.3	9.26	10.37	11.86	13.8	15.26	16.74	18.71
18	4.32	5.57	7.19	8.13	9.36	10.98	12.21	13.46	15.14
24	3.55	4.59	6	6.83	7.91	9.33	10.42	11.52	13.02
30	3.03	3.94	5.19	5.94	6.91	8.19	9.18	10.18	11.55
36	2.66	3.46	4.61	5.29	6.18	7.35	8.26	9.18	10.44
48	2.15	2.81	3.79	4.38	5.15	6.17	6.95	7.76	8.87
72	1.55	2.05	2.82	3.29	3.9	4.72	5.36	6.02	6.93

TABLE B2 – XP-RAFTS SUB-CATCHMENT PROPERTIES

Link	Catchment Area		Slope (%)		% Impervious		Pern		B		Link No.
###	**#1**	**#2**	**#1**	**#2**	**#1**	**#2**	**#1**	**#2**	**#1**	**#2**	**###**
1	248	0	5.63	0	0	0	0.025	0	0.38	0	1
1.01	332	0	1	0	0	0	0.025	0	1.06	0	1.01
1.02	347	0	0.77	0	5	0	0.025	0	1	0	1.02
7	318	0	2.58	0	5	0	0.025	0	0.52	0	2
1.03	325	0	1.43	0	5	0	0.025	0	0.7	0	1.03
1.04	149	0	2.5	0	0	0	0.025	0	0.44	0	1.04
2	237	0	2.94	0	15	0	0.025	0	0.34	0	3
2.01	245	0	2.11	0	15	0	0.025	0	0.42	0	3.01
2.02	259	0	2.63	0	15	0	0.025	0	0.38	0	3.02
1.05	97	0	1	0	0	0	0.025	0	0.56	0	1.05
3	444	0	2.86	0	0	0	0.025	0	0.74	0	4
3.01	250	0	1.58	0	0	0	0.025	0	0.74	0	4.01
3.02	357	0	1.6	0	0	0	0.025	0	0.88	0	4.02
3.03	244	0	1.88	0	5	0	0.025	0	0.54	0	4.03
8	192	0	4.8	0	0	0	0.025	0	0.36	0	5
3.04	170	0	1.05	0	0	0	0.025	0	0.74	0	4.04
1.06	177	0	0.87	0	0	0	0.025	0	0.82	0	1.06
4	188	0	5.22	0	0	0	0.025	0	0.34	0	6
4.01	380	0	1.25	0	0	0	0.025	0	1.02	0	6.01
4.02	215	0	0.77	0	0	0	0.025	0	1	0	6.02
1.07	0.01	0	0.8	0	5	0	0.025	0	0.0021	0	1.07
5	428	0	2.7	0	0	0	0.025	0	0.74	0	7
5.01	479	0	1.25	0	0	0	0.025	0	1.16	0	7.01
6	298	0	3.5	0	0	0	0.025	0	0.54	0	8
6.01	411	0	1.33	0	0	0	0.025	0	1.04	0	8.01
6.02	497	0	0.77	0	0	0	0.025	0	1.5	0	8.02
6.03	318	0	0.67	0	5	0	0.025	0	1.02	0	8.03
6.04	295	0	0.8	0	0	0	0.025	0	1.12	0	8.04
5.02	13	0	0.8	0	0	0	0.025	0	0.22	0	7.02
5.03	0.01	0	0.8	0	0	0	0.025	0	0.0026	0	7.03
1.08	0.01	0	0.8	0	5	0	0.025	0	0.0021	0	1.08
1.09	146	0	0.5	0	25	0	0.025	0	0.4	0	1.09
1.1	207	0	0.2	0	20	0	0.025	0	0.88	0	1.1

Appendix C:
Monthly Rainfall Data Preceding the June 2016 Event

Daily Rainfall (millimetres)

MENANGLE BRIDGE (NEPEAN RIVER)

Station Number: 068216 · State: NSW · Opened: 1963 · Status: Open · Latitude: 34.12°S · Longitude: 150.74°E · Elevation: Unknown m

2016	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	0	0	0	0	0	2.0	0	0	0			
2nd	0	2.0	0	0	0	1.0	0	0	7.0			
3rd	0	0	0	0	0	0	0	15.0	21.0			
4th	8.0	5.0	0	2.0	0	11.0	0	1.0	0			
5th	44.0	0	0	0	0	137.0	5.0	3.0	0			
6th	15.0	0	0	0	0	122.0	0	0	0			
7th	4.0	0	0	0	0	0	2.0	0	0			
8th	0	0	0	0	0	0	6.0	0	0			
9th	0	0	0	0	10.0	0	1.0	1.0	0			
10th	0	0	0	0	0	0	0	0	2.0			
11th	0	0	0	0	0	0	0	0	0			
12th	0	0	0	3.0	0	0	0	0	0			
13th	0	0	0	0	0	0	0	0	0			
14th	0	0	0	0	0	0	0	0				
15th	31.0	0	4.0	0	0	0	0	0				
16th	1.0	0	1.0	0	0	0	0	0				
17th	0	0	4.0	2.0	0	0	0	0				
18th	0	0	1.0	0	0	6.0	1.0	0				
19th	0	0	1.0	4.0	0	0	0	0				
20th	0	0	0	0	0	23.0	14.0	1.0				
21st	0	1.0	0	0	0	0	8.0	0				
22nd	10.0	0	0	0	0	0	1.0	0				
23rd	12.0	0	0	1.0	0	0	8.0	1.0				
24th	3.0	0	0	2.0	0	0	0	1.0				
25th	0	0	0	0	0	0	0	24.0				
26th	1.0	0	0	0	0	0	0	0				
27th	0	0	0	0	0	1.0	0	0				
28th	1.0	0	0	0	0	0	0	0				
29th	0	0	0	0	3.0	0	0	0				
30th	50.0		19.0	1.0	0	0	0	0				
31st	8.0		0		0		0	0				
Highest daily	50.0	5.0	19.0	4.0	10.0	137.0	14.0	24.0	21.0			
Monthly Total	188.0	8.0	30.0	15.0	13.0	303.0	46.0	47.0				

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Product code: IDCJAC0009 reference: 26100134

Australian Government
Bureau of Meteorology

Daily Rainfall (millimetres)

MENANGLE BRIDGE (NEPEAN RIVER)

Station Number: 068216 · State: NSW · Opened: 1963 · Status: Open · Latitude: 34.12°S · Longitude: 150.74°E · Elevation: Unknown m

Statistics for this station calculated over all years of data

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Highest daily	*100.0*	*79.0*	*84.0*	*68.0*	*38.0*	*137.0*	*56.0*	*61.0*	*32.0*	*90.0*	*35.0*	*53.0*
Date of highest daily	29th 2013	11th 2007	1st 2007	19th 2012	27th 2010	5th 2016	1st 2005	25th 2015	7th 2006	10th 2010	23rd 2013	11th 2004

1) Calculation of statistics

Summary statistics, other than the Highest and Lowest values, are only calculated if there are at least 20 years of data available.

2) Gaps and missing data

Gaps may be caused by a damaged instrument, a temporary change to the site operation, or due to the absence or illness of an observer.

3) Further information

http://www.bom.gov.au/climate/cdo/about/about-rain-data.shtml.

Product code: IDCJAC0009 reference: 26100134 Created on Tue 13 Sep 2016 14:29:25 PM EST

Daily Rainfall (millimetres)

OAKDALE (COOYONG PARK)

Station Number: 068125 · State: NSW · Opened: 1963 · Status: Open · Latitude: 34.09°S · Longitude: 150.51°E · Elevation: 440 m

2016	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	*0*	*0*	*0*	*0*	*0*	*9.4*	*0*	*0*				
2nd	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
3rd	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*20.2*				
4th	*6.6*	*0*	*0*	*1.0*	*0*	*12.0*	*0*	*0*				
5th	*39.4*	*0*	*0*	*0*	*0*	*100.0*	*4.4*	*2.0*				
6th	*26.0*	*0*	*0*	*0*	*0*	*149.0*	*1.4*	*0*				
7th	*0*	*0*	*0*	*0*	*0*	*0*	*0.8*	*1.8*				
8th	*0*	*8.0*	*0*	*0*	*0*	*0*	*0.8*	*0*				
9th	*0*	*0*	*0*	*0*	*9.2*	*0*	*0*	*0*				
10th	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
11th	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
12th	*0*	*0*	*0.4*	*0*	*0*	*0*	*2.4*	*0*				
13th	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
14th	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
15th	*33.0*	*0*	*2.0*	*0*	*0*	*0*	*0*	*0*				
16th	*1.4*	*0*	*4.0*	*0*	*0*	*0*	*0*	*0*				
17th	*0*	*0*	*3.2*	*6.6*	*0*	*0*	*0*	*0*				
18th	*0*	*0*	*0*	*0.6*	*0*	*6.4*	*1.4*	*0*				
19th	*0*	*0*	*0*	*6.0*	*0*	*0*	*0*	*0*				
20th	*0*	*0*	*0*	*0*	*0*	*32.4*	*12.4*	*0*				
21st	*0*	*26.2*	*0*	*0*	*0*	*0*	*10.0*	*0*				
22nd	*21.0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
23rd		*0*	*0*	*1.6*	*0*	*0*	*12.0*	*5.0*				
24th	*4.4*	*0*	*0*	*0.8*	*0*	*0*	*0*	*0*				
25th	*1.8*	*0*	*0*	*0*	*0*	*0*	*0*	*30.8*				
26th	*3.4*	*0*	*0*	*0*	*0*	*1.4*	*0*	*0*				
27th	*2.0*	*0.4*	*0*	*0*	*0*	*0*	*0*	*0*				
28th	*7.0*	*0*	*7.4*	*0*	*0*	*0*	*0*	*0*				
29th		*0*	*0*	*0*	*1.6*	*0*	*0*	*0*				
30th	*15.0*		*11.4*	*1.4*	*0*	*0*	*0*	*0*				
31st	*4.0*		*0*		*0*		*0*	*0*				
Highest daily	*39.4*	*26.2*	*11.4*	*6.6*	*9.2*	*149.0*	*12.4*	*30.8*				
Monthly Total		*34.6*	*28.4*	*18.0*	*10.8*	*310.6*	*45.6*	*59.8*				

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Product code: IDCJAC0009 reference: 26100117

Australian Government
Bureau of Meteorology

Daily Rainfall (millimetres)

OAKDALE (COOYONG PARK)

Station Number: 068125 · State: NSW · Opened: 1963 · Status: Open · Latitude: 34.09°S · Longitude: 150.51°E · Elevation: 440 m

Statistics for this station calculated over all years of data

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean	102.7	130.7	113.4	78.4	50.5	85.9	33.7	44.7	44.4	77.0	99.8	78.6
Median	76.3	109.8	85.3	68.6	37.1	49.5	19.4	25.7	42.5	56.6	74.2	78.2
Highest daily	131.0	173.6	125.0	162.1	76.2	208.0	52.0	203.2	80.6	106.6	195.8	84.6
Date of highest daily	29th 2013	11th 2007	22nd 1983	16th 1969	8th 1963	12th 1964	28th 1984	7th 1967	21st 1982	24th 1975	7th 1966	8th 1970

1) Calculation of statistics

Summary statistics, other than the Highest and Lowest values, are only calculated if there are at least 20 years of data available.

2) Gaps and missing data

Gaps may be caused by a damaged instrument, a temporary change to the site operation, or due to the absence or illness of an observer.

3) Further information

http://www.bom.gov.au/climate/cdo/about/about-rain-data.shtml.

Product code: IDCJAC0009 reference: 26100117 Created on Tue 13 Sep 2016 14:27:56 PM EST

Australian Government
Bureau of Meteorology

Daily Rainfall (millimetres)

PICTON COUNCIL DEPOT

Station Number: 068052 · State: NSW · Opened: 1880 · Status: Open · Latitude: 34.17°S · Longitude: 150.61°E · Elevation: 165 m

2016	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	*0*	*0*	*0*	*0*	*0*	*1.2*	*0*	*0*	*0*			
2nd	*0*	*1.2*	*0*	↓	*0*	*1.2*	*0*	*0*	*3.4*			
3rd	*0*	*0*	*0*	↓	*2.0*	*0*	*0*	*17.0*	↓			
4th	*0.6*	*3.0*	*0*	*0.2*	*0*	↓	*0*	*0.2*	↓			
5th	*47.6*	*0*	*0*	*0*	*0*	↓	*1.2*	*1.6*	*25.2*			
6th	*16.8*	↓	*0*	*0*	↓	*245.0*	*3.2*	↓	*0*			
7th	*5.6*	↓	*0*	*0*	↓	*0*	*0.4*	↓	*0*			
8th	*0.2*	*0.4*	*0*	*0*	↓	*0*	*2.0*	*1.0*	*0*			
9th	*0*	*0*	*0.2*	*0*	↓	*0.4*	↓	*0.2*	*0*			
10th	*0*	*0*	*0*	*0*	*13.6*	*0*	↓	*0*	↓			
11th	*0*	*0*	*0*	*0*	*0*	*0*	*6.4*	*0*	↓			
12th	*0*	*0*	*0*	*1.8*	*0*	*0*	*0*	*0*	*2.6*			
13th	*0*	*0*	*0*	*0*	*0*	*0*	*0.6*	*0*	*0*			
14th	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*				
15th	*32.0*	*0*	*7.0*	*0*	*0*	*0*	*0*	*0*				
16th	*0*	*0*	*0*	↓	*0*	*0*	↓	*0*				
17th	*0*	*0*	*2.0*	↓	*0*	*0*	↓	*0*				
18th	*0*	*0*	*0*	*2.0*	*0*	↓	*2.0*	*0*				
19th	*0*	*0*	↓	*2.5*	*0*	↓	*0*	*0*				
20th	*0*	↓	↓	*0*	*0*	*30.0*	*7.0*	↓				
21st	*0*	↓	*2.6*	*0*	*0*	*0.2*	*15.2*	↓				
22nd	↓	*6.4*	*0*	*0*	*0*	*0*	*1.0*	*1.2*				
23rd	↓	*0*	*0*	*0*	*0*	*0*	↓	*2.8*				
24th	*37.0*	*0*	*0*	↓	*0*	*0*	↓	*0.2*				
25th	*0*	*0*	*0*	↓	*0*	↓	*11.6*	*24.2*				
26th	↓	*0*	*0*	*2.0*	*0.2*	↓	*0*	*0.2*				
27th	*11.6*	*0*	*0*	*0.2*	*0*	*0.6*	*0*	*0*				
28th	*7.2*	*0*	*0*	*0*	↓	*0.8*	*0*	*0*				
29th	↓	*0*	*5.8*	*0*	↓	*0*	*0*	*0*				
30th	↓		*14.0*	*0*	*4.2*	*0*	*0*	*0*				
31st	*15.6*		*0*		*0*		*0*	*0*				
Highest daily	*47.6*	*3.0*	*14.0*	*2.5*	*2.0*	*1.2*	*15.2*	*24.2*	*3.4*			
Monthly Total	*174.2*	*11.0*	*31.6*	*8.7*	*20.0*	*279.4*	*50.6*	*48.6*				

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Product code: IDCJAC0009 reference: 26099849

Australian Government
Bureau of Meteorology

Daily Rainfall (millimetres)

PICTON COUNCIL DEPOT

Station Number: 068052 · State: NSW · Opened: 1880 · Status: Open · Latitude: 34.17°S · Longitude: 150.61°E · Elevation: 165 m

Statistics for this station calculated over all years of data

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean	87.4	90.0	87.8	69.9	56.1	67.7	49.7	44.9	44.0	63.7	72.2	70.3
Median	67.1	66.4	67.2	49.3	31.2	43.3	26.1	25.2	37.5	49.5	55.5	54.4
Highest daily	211.6	216.7	132.6	156.0	132.1	201.9	124.5	118.4	77.5	141.5	245.9	104.1
Date of highest daily	23rd 1933	10th 1956	25th 1890	16th 1969	21st 1949	12th 1964	10th 1904	30th 1963	11th 1929	5th 1916	9th 1966	13th 1910

1) Calculation of statistics

Summary statistics, other than the Highest and Lowest values, are only calculated if there are at least 20 years of data available.

2) Gaps and missing data

Gaps may be caused by a damaged instrument, a temporary change to the site operation, or due to the absence or illness of an observer.

3) Further information

http://www.bom.gov.au/climate/cdo/about/about-rain-data.shtml.

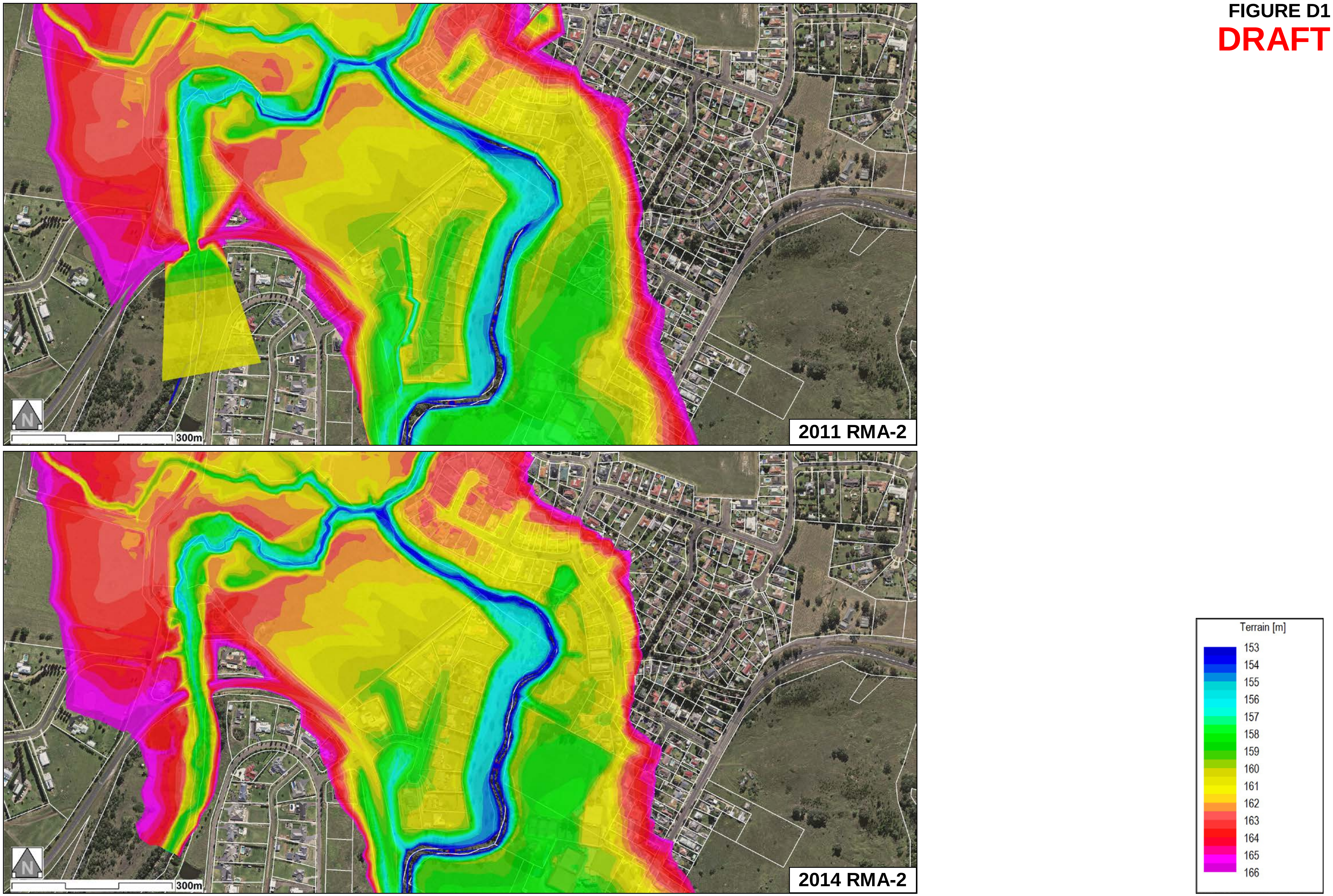
Product code: IDCJAC0009 reference: 26099849 Created on Tue 13 Sep 2016 14:18:48 PM EST

Australian Government
Bureau of Meteorology

Appendix D:
2011 & 2017 Topography Comparison

FIGURE D1

DRAFT

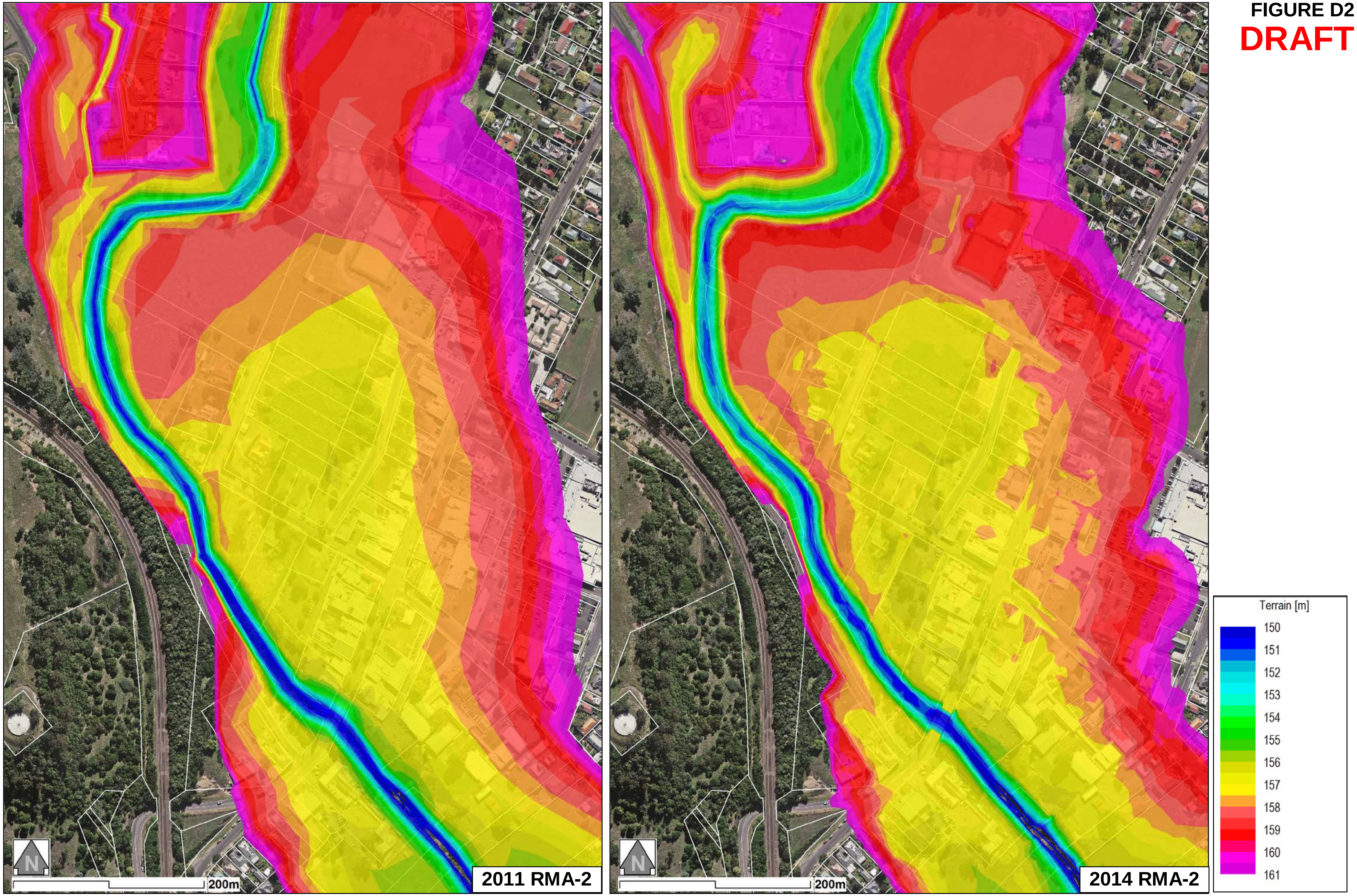


COMPARISON OF 2011 AND 2014 RMA-2 TOPOGRAPHY ALONG STONEQUARRY CREEK (*1 of 3*)

Advisian
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_FigC1_2011 & 2014 Topography (Stonequarry Ck).doc

FIGURE D2
DRAFT

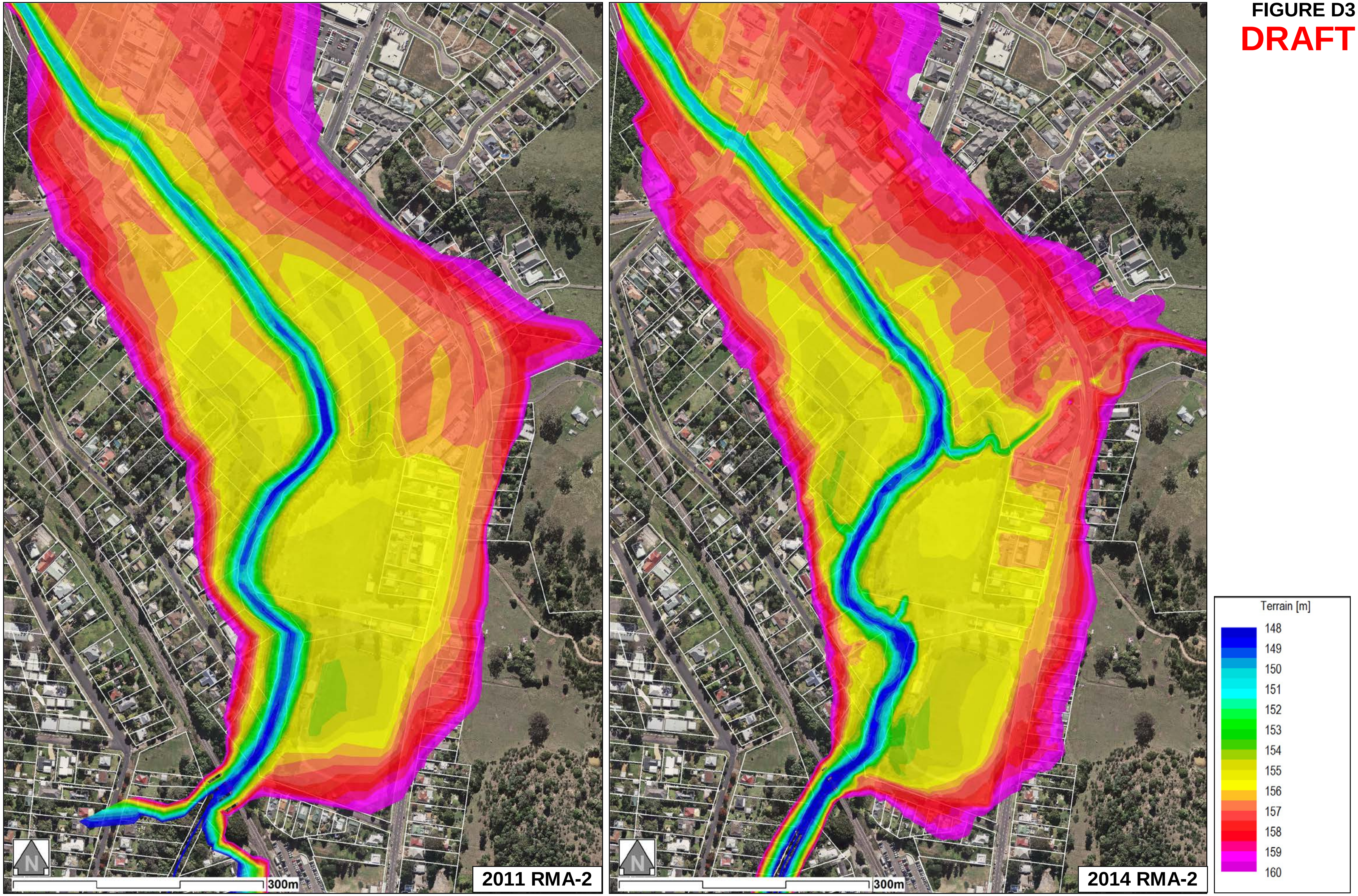


COMPARISON OF 2011 AND 2014 RMA-2 TOPOGRAPHY ALONG STONEQUARRY CREEK (*2of 3*)

Advisian
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_FigC2_2011 & 2014 Topography (Stonequarry Ck).doc

FIGURE D3
DRAFT

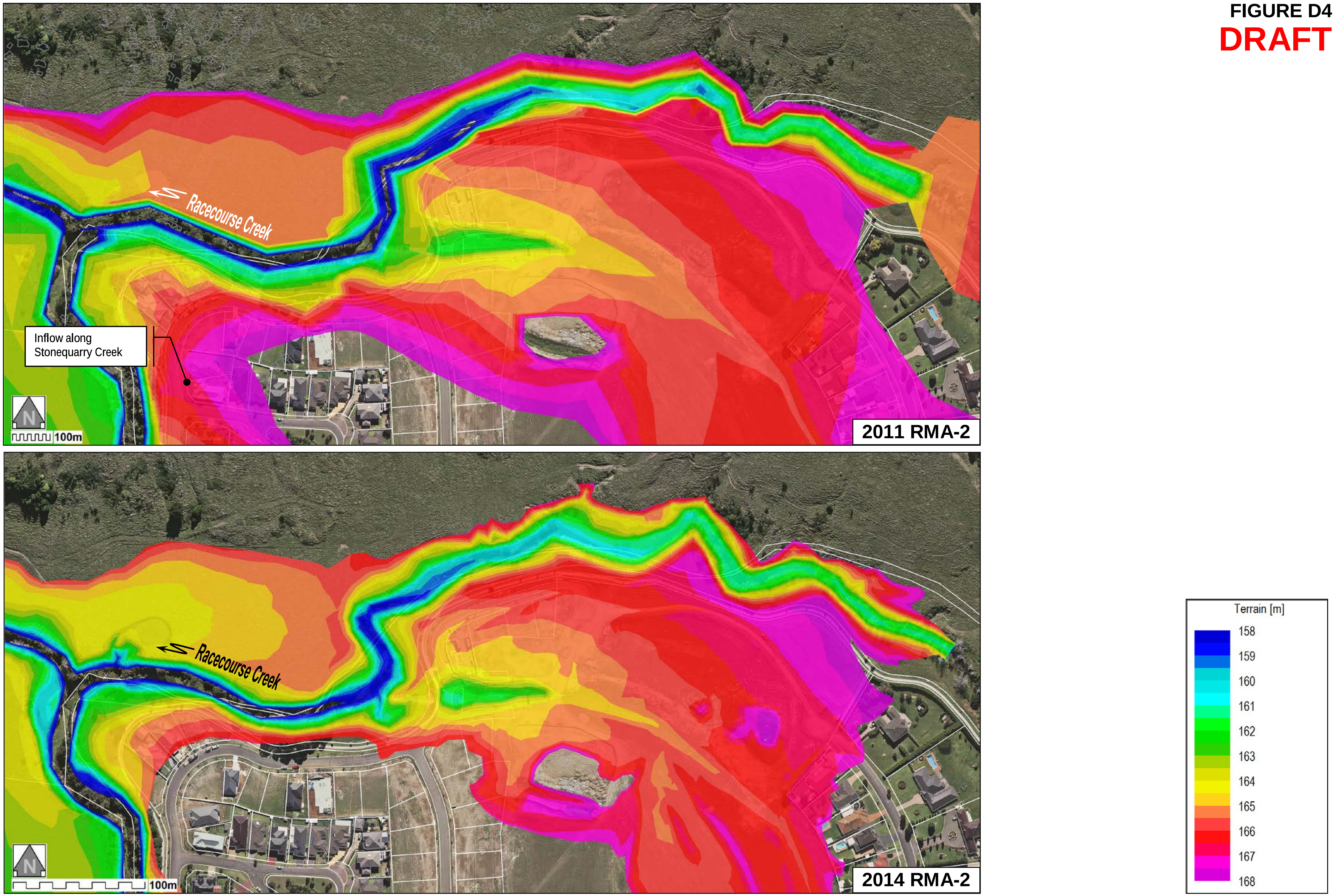


Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_FigC2_2011 & 2014 Topography (Stonequarry Ck).doc

COMPARISON OF 2011 AND 2014 RMA-2 TOPOGRAPHY ALONG STONEQUARRY CREEK (*3 of 3*)

FIGURE D4

DRAFT

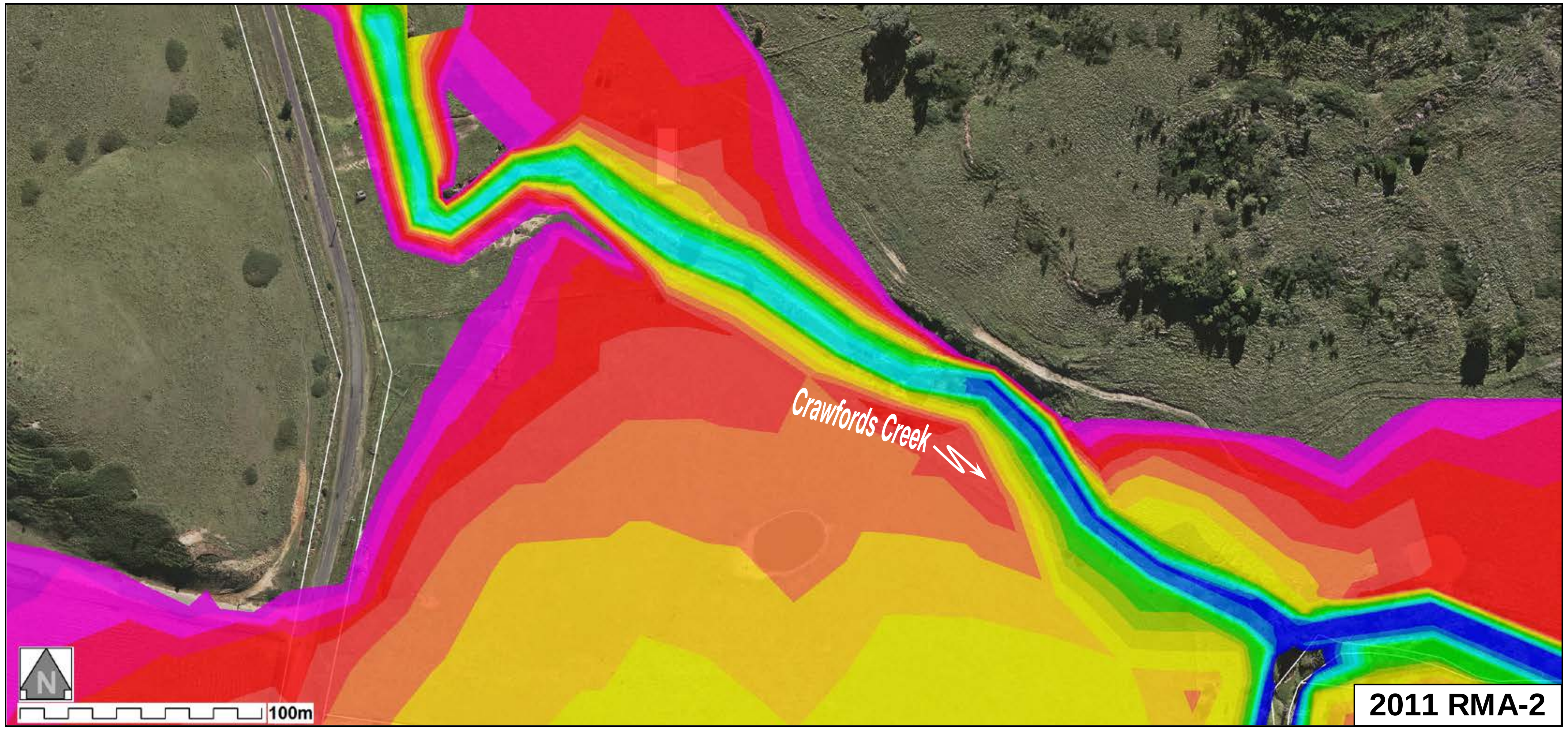


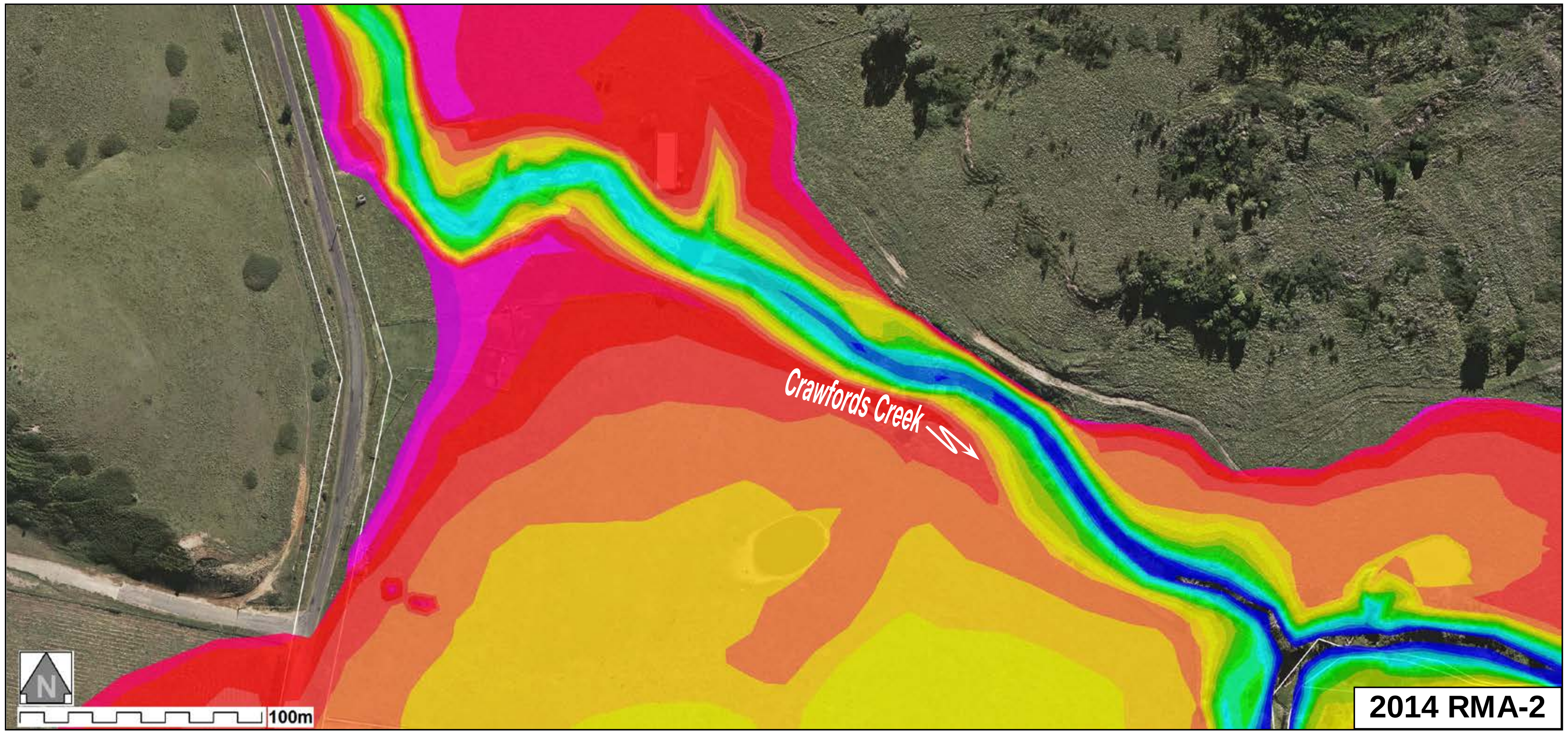
COMPARISON OF 2011 AND 2014 RMA-2 TOPOGRAPHY ALONG RACECOURSE CREEK

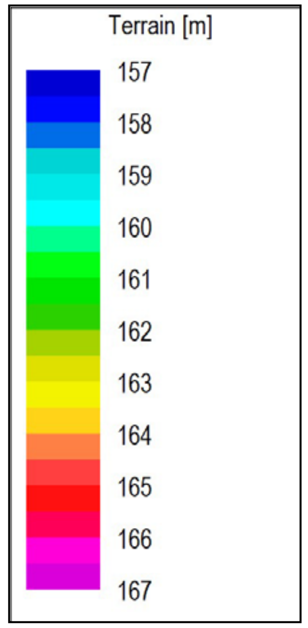
Advisian
WorleyParsons Group
301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_FigC2_2011 & 2014 Topography (Racecourse Ck).doc

FIGURE D5

DRAFT







COMPARISON OF 2011 AND 2014 RMA-2 TOPOGRAPHY ALONG CRAWFORDS CREEK

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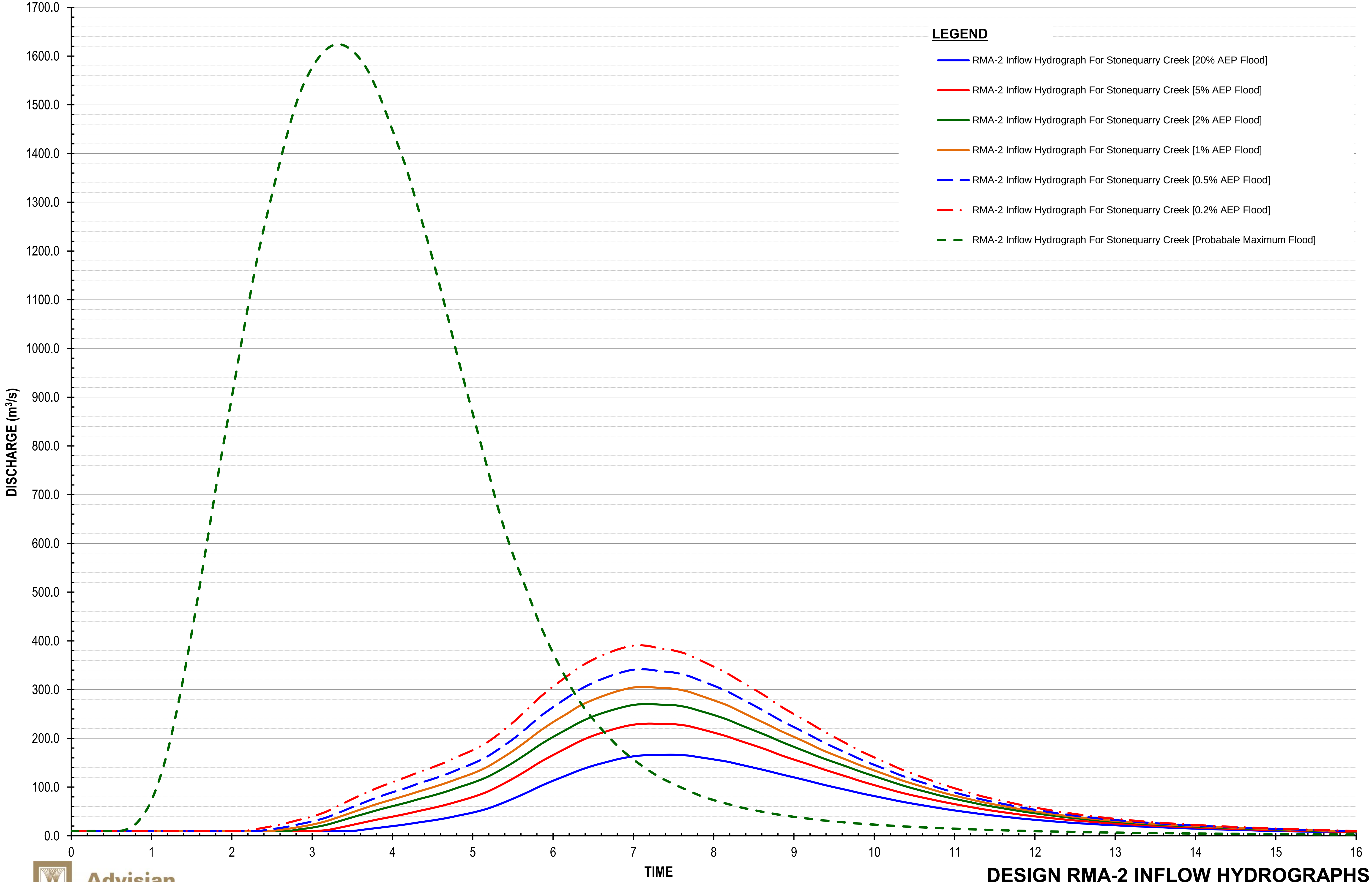
WorleyParsons Group

301015-03199-Stonequarry Ck Flood Modelling
fg301015-03199rg140703_FigC2_2011 & 2014 Topography (Racecourse Ck).doc

Appendix E:
Discharge Hydrographs at Model Inflow Locations

FIGURE E1



DESIGN RMA-2 INFLOW HYDROGRAPHS FOR STONEQUARRY CREEK AT THE UPSTREAM EXTENT OF THE RMA-2 MODEL

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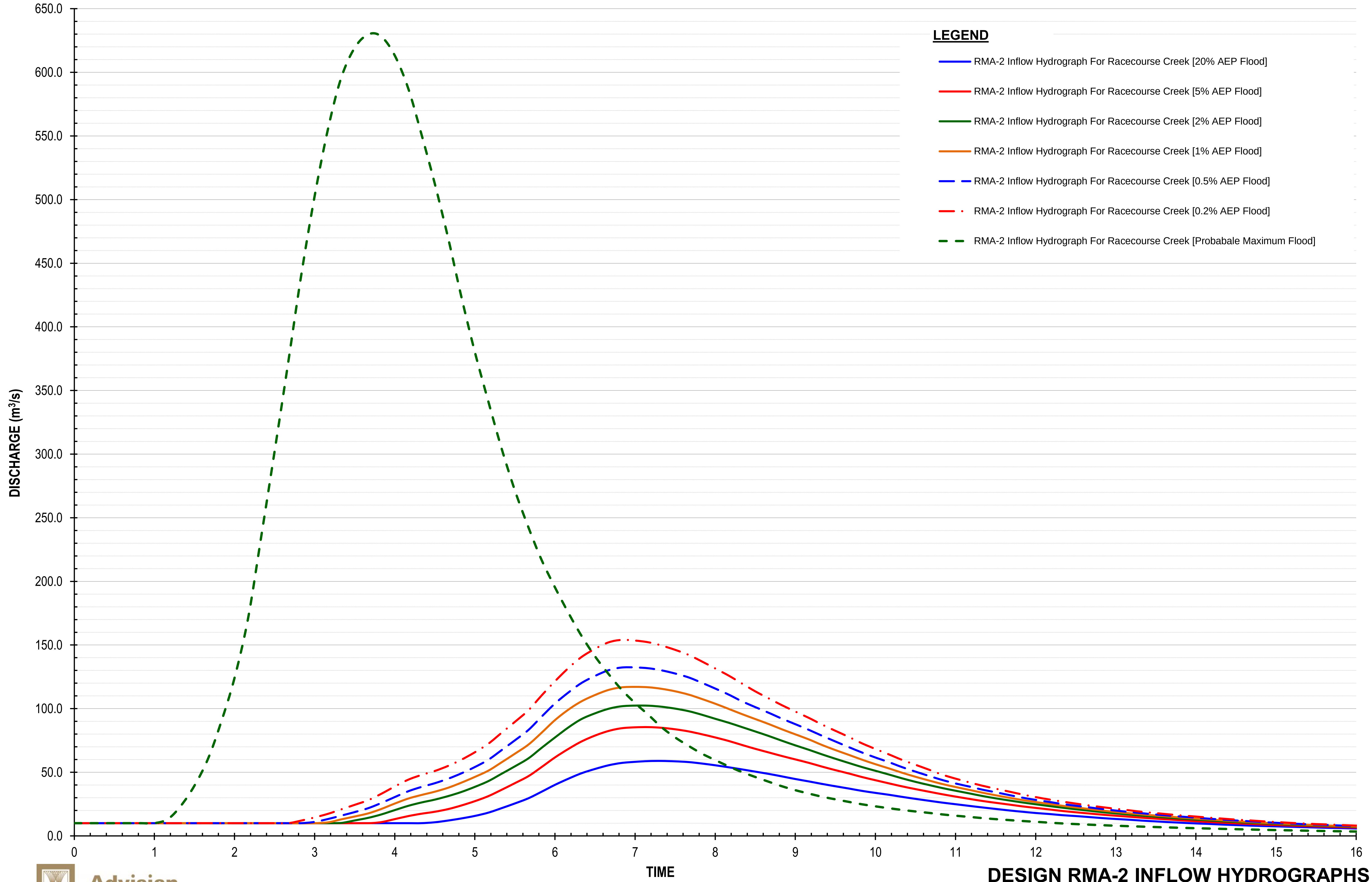
301015-03199 - Stonequarry Creek FS
fg301015-03199rg140707-APPENDIX D.xls

FIGURE E2



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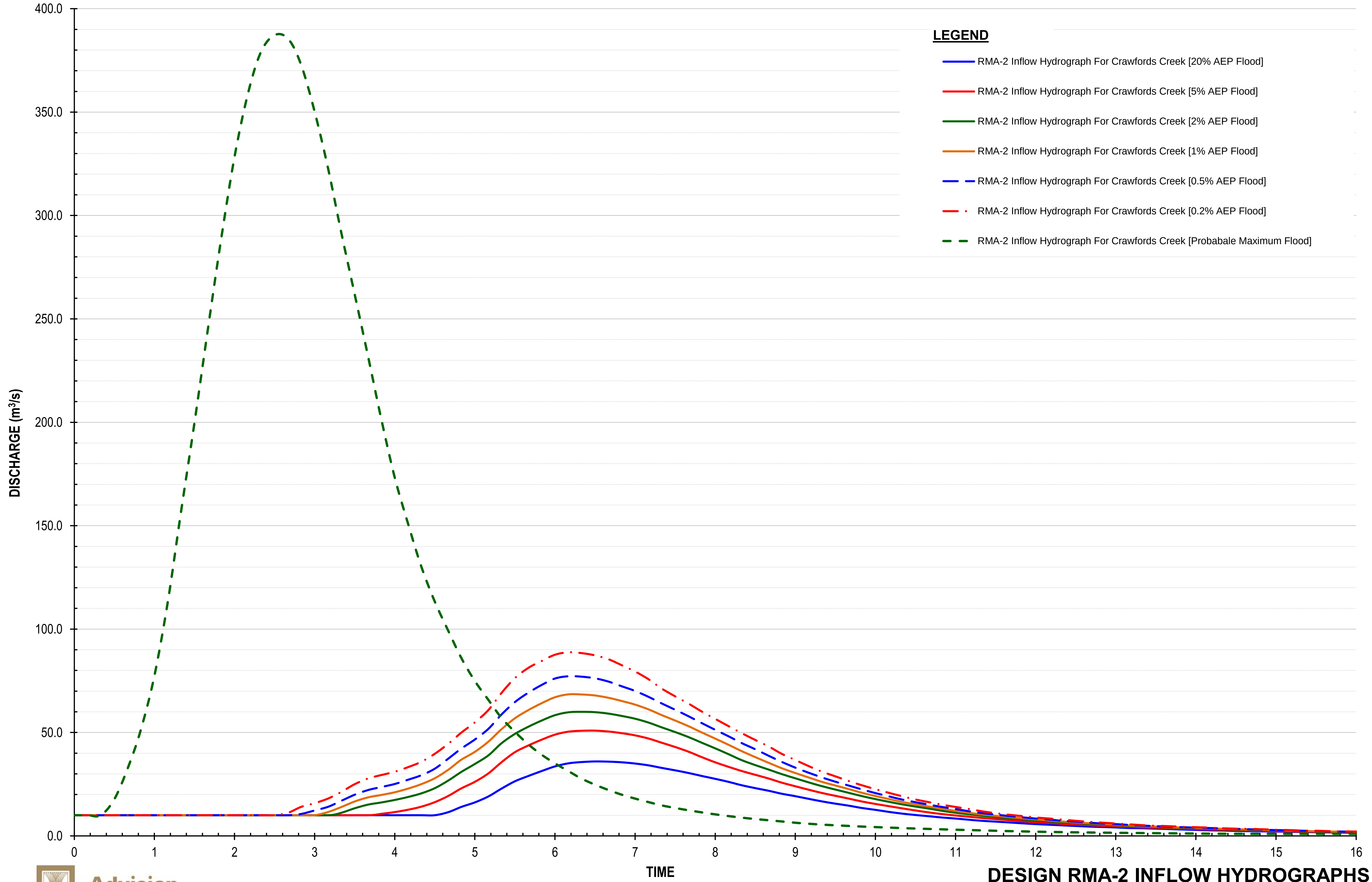
DESIGN RMA-2 INFLOW HYDROGRAPHS FOR RACECOURSE CREEK AT THE UPSTREAM EXTENT OF THE RMA-2 MODEL

FIGURE E3



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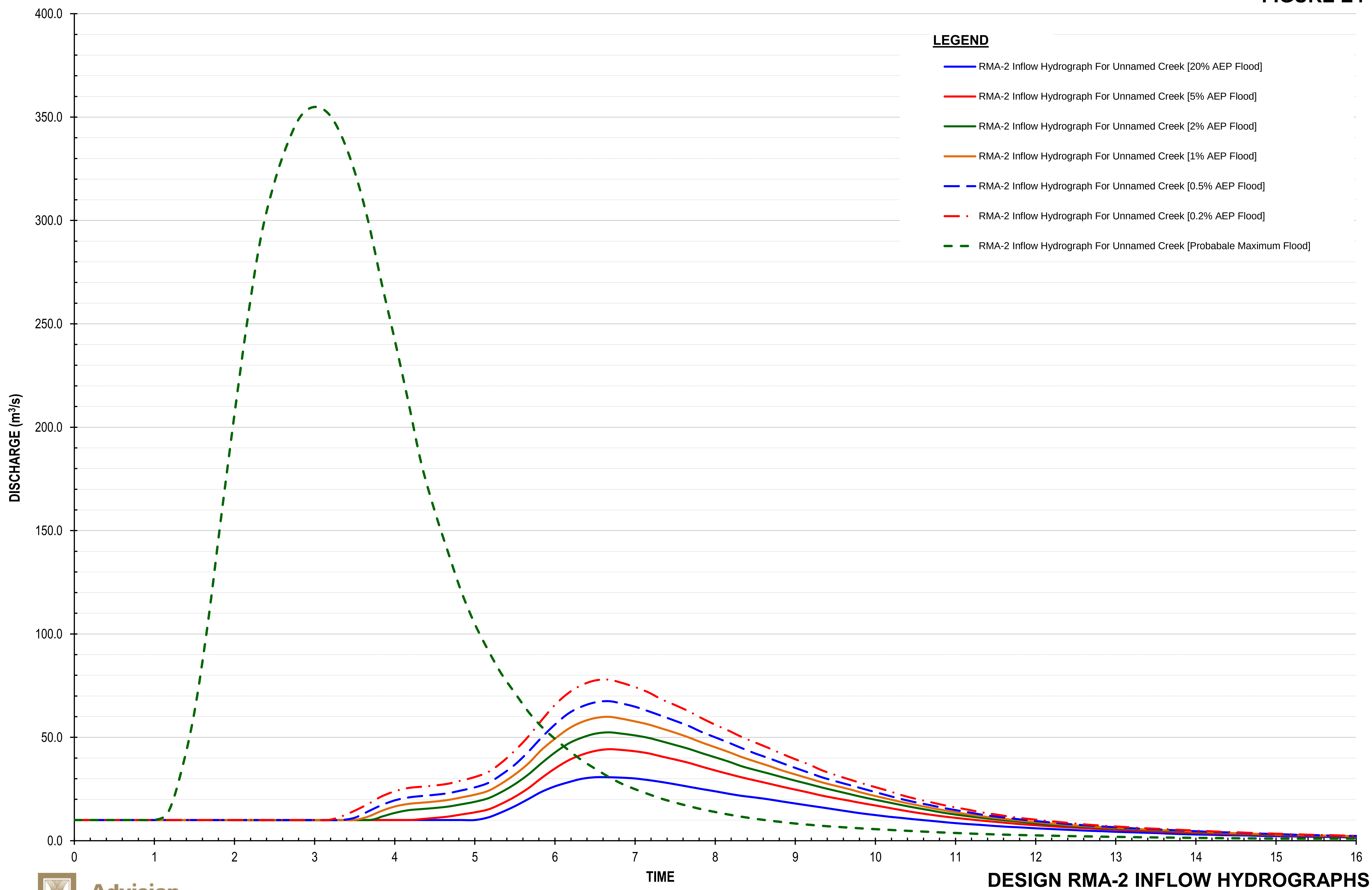
fg301015-03199rg140707-APPENDIX D.xls

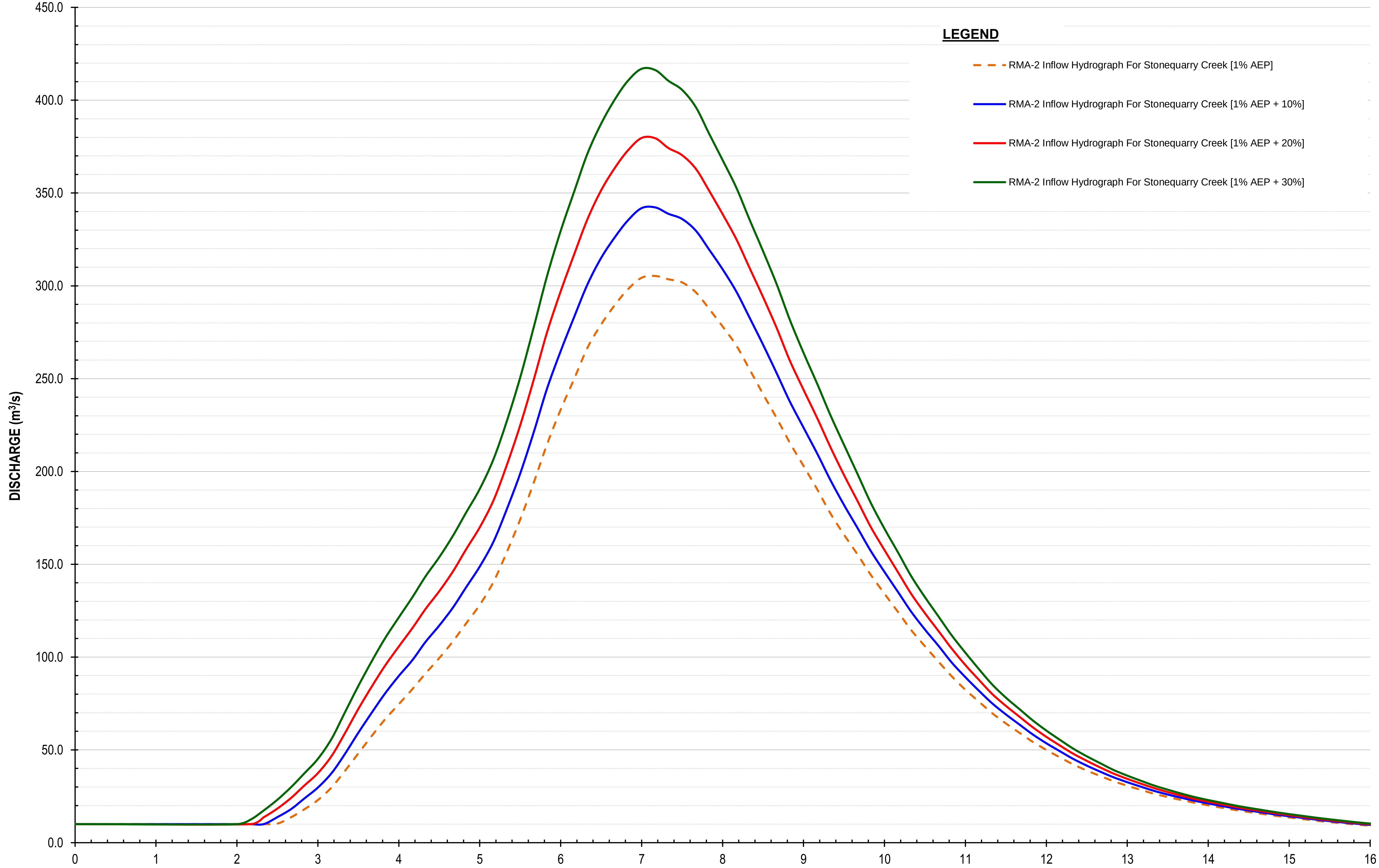
DESIGN RMA-2 INFLOW HYDROGRAPHS FOR CRAWFORDS CREEK AT THE UPSTREAM EXTENT OF THE RMA-2 MODEL

FIGURE E4



DESIGN RMA-2 INFLOW HYDROGRAPHS FOR UNNAMED CREEK AT THE UPSTREAM EXTENT OF THE RMA-2 MODEL

FIGURE E5



DESIGN RMA-2 INFLOW HYDROGRAPHS FOR STONEQUARRY CREEK FOR THE 10%, 20% & 30% INCREASED RAINFALL CLIMATE CHANGE SCENARIOS

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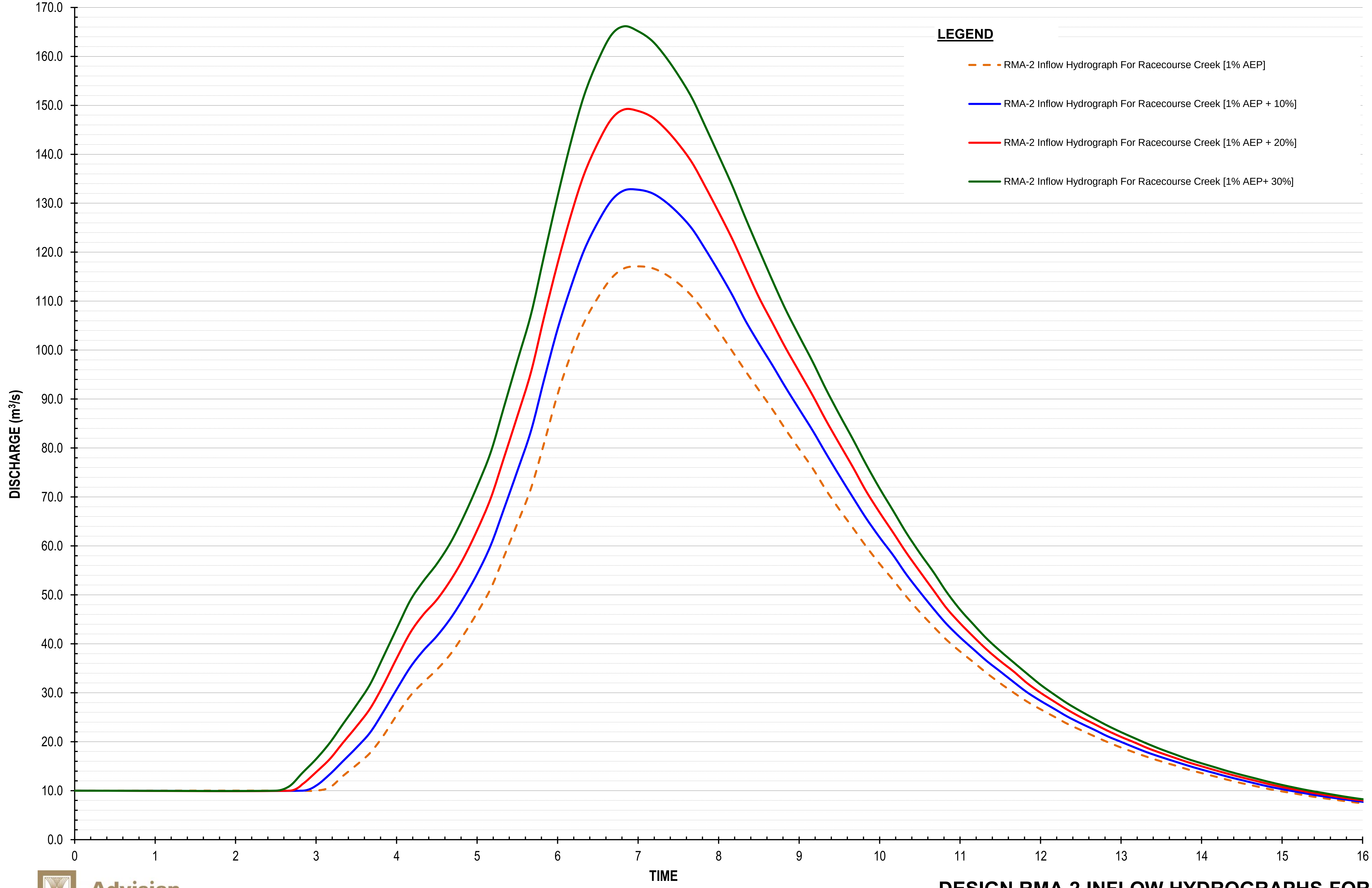
fg301015-03199rg140707-APPENDIX D.xls

FIGURE E6



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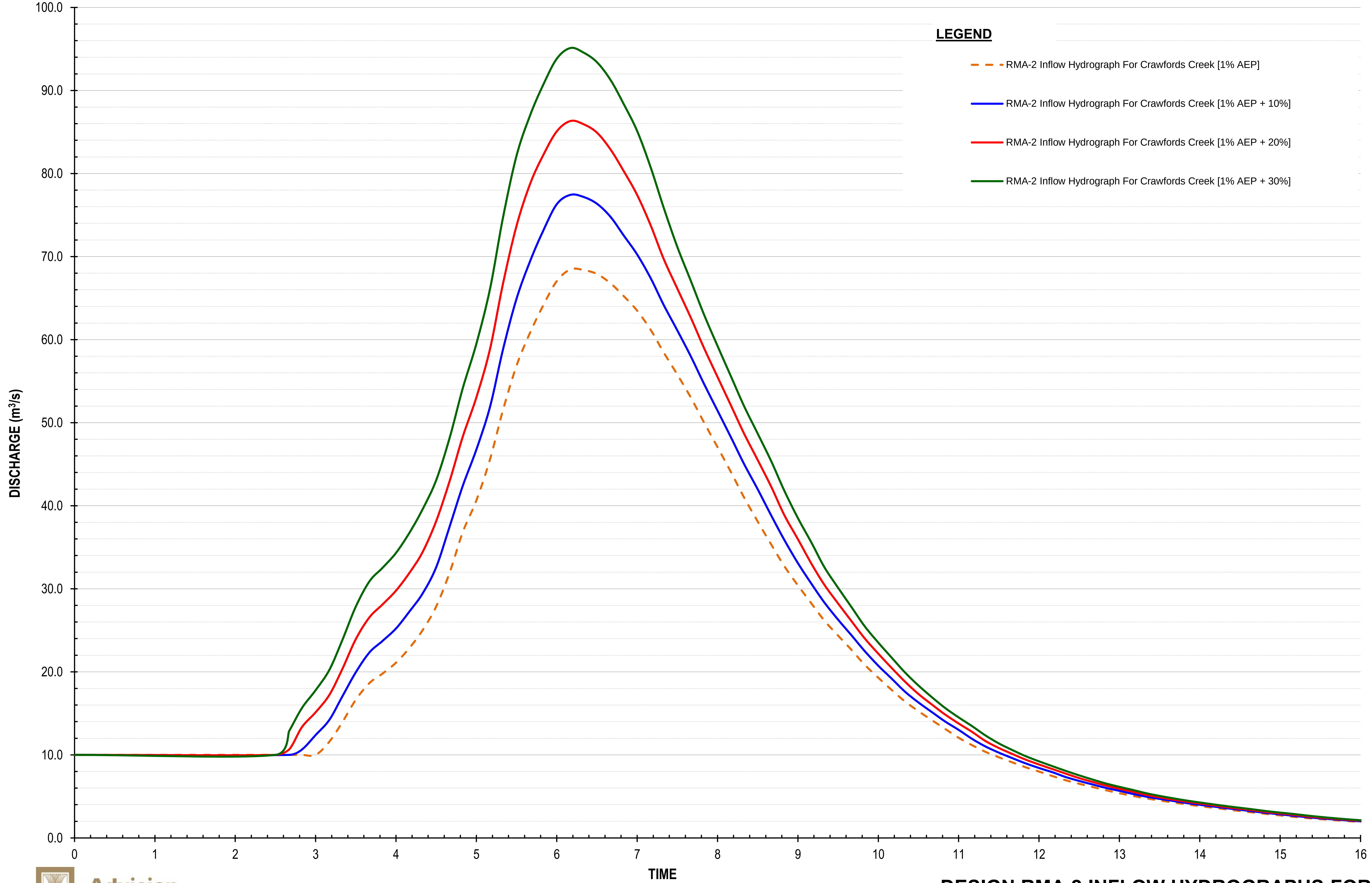
WorleyParsons Group

301015-03199 - Stonequarry Creek FS
fg301015-03199rg140707-APPENDIX D.xls

DESIGN RMA-2 INFLOW HYDROGRAPHS FOR STONEQUARRY CREEK FOR THE 10%, 20% & 30% INCREASED RAINFALL CLIMATE CHANGE SCENARIOS

FIGURE E7

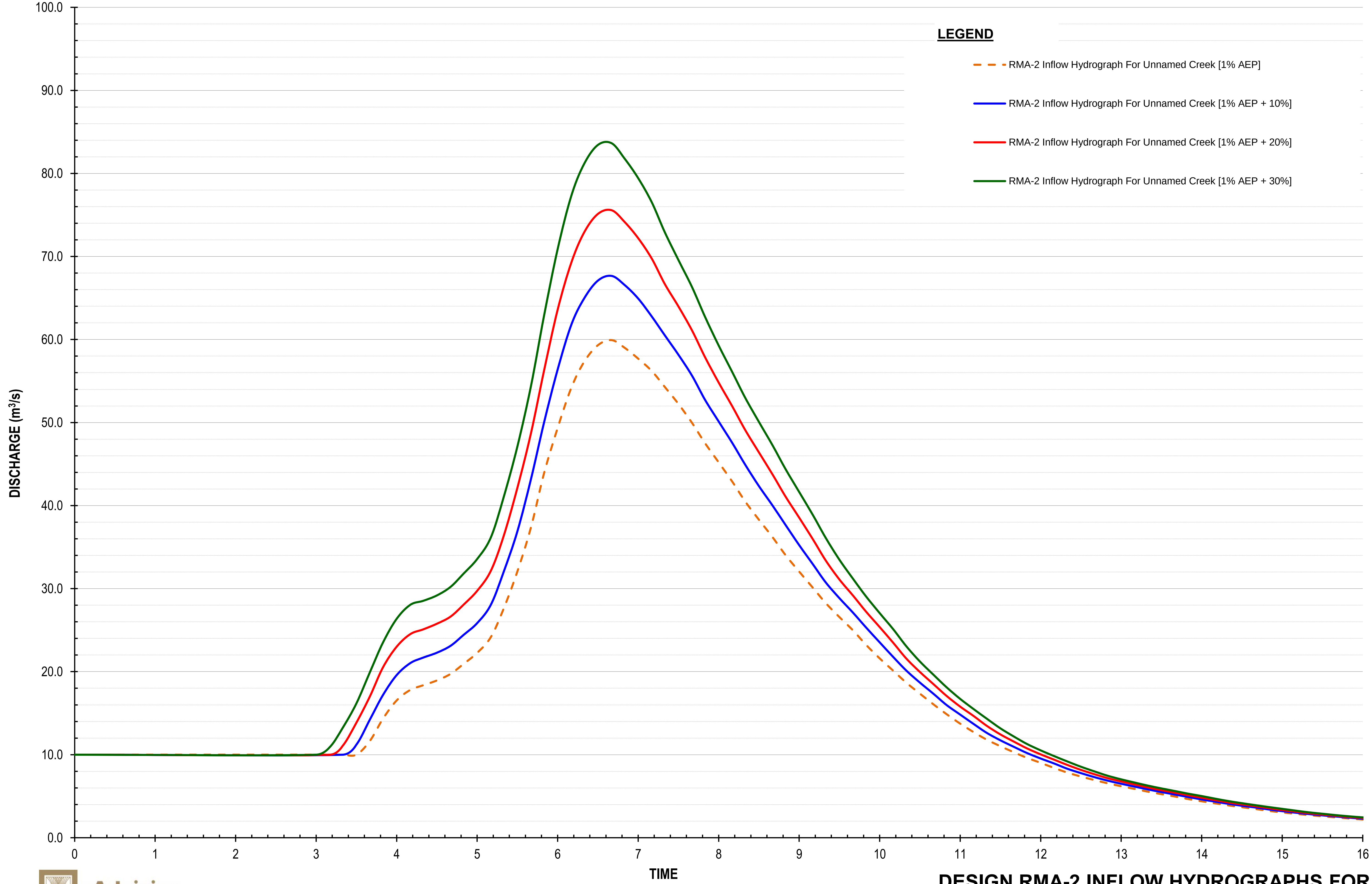


Advisian
WorleyParsons Group

301015-03199 - Stonequarry Creek FS
fg301015-03199rg140707-APPENDIX D.xls

DESIGN RMA-2 INFLOW HYDROGRAPHS FOR STONEQUARRY CREEK FOR THE 10%, 20% & 30% INCREASED RAINFALL CLIMATE CHANGE SCENARIOS

FIGURE E8



Advisian
WorleyParsons Group

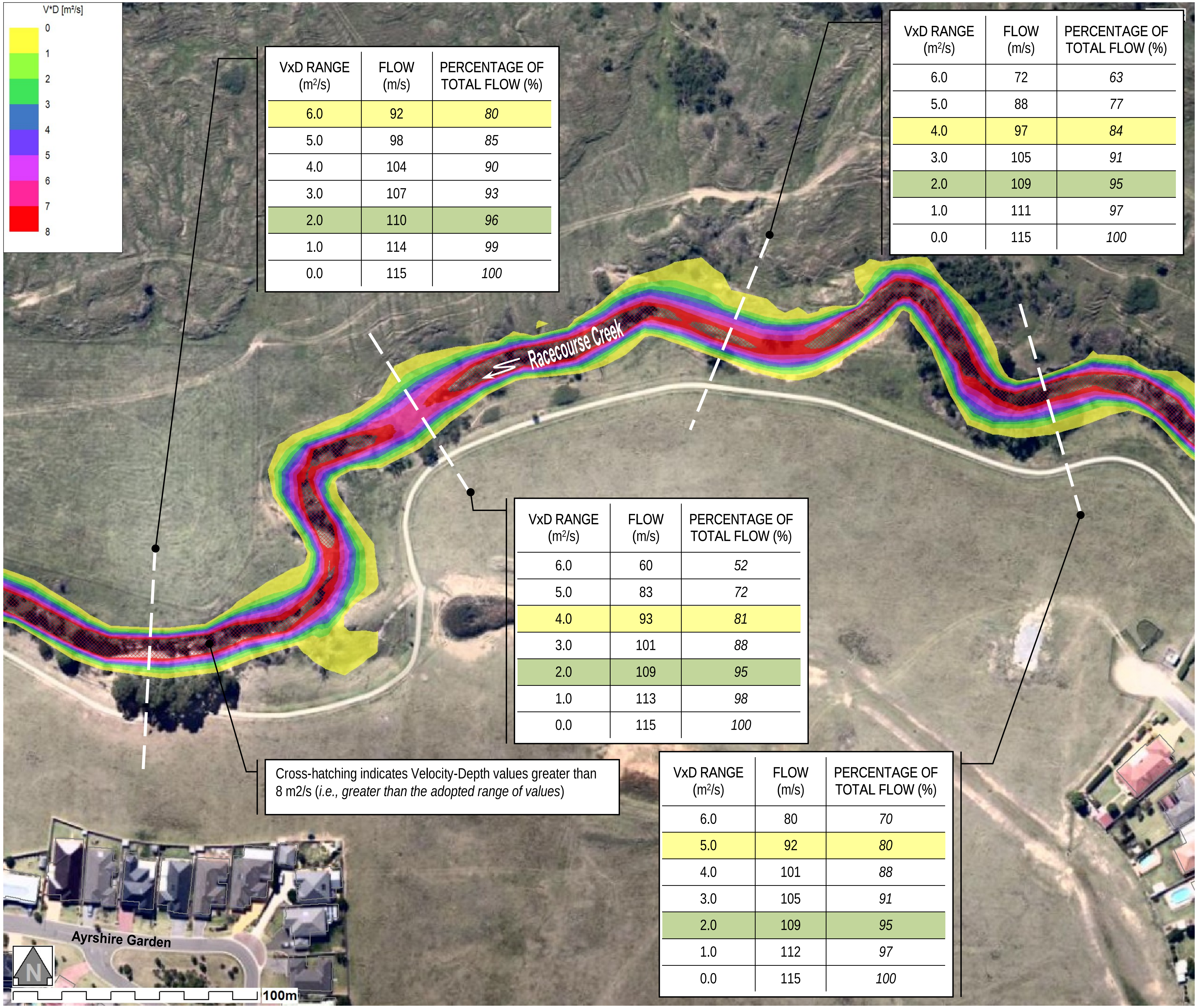
301015-03199 - Stonequarry Creek FS
fg301015-03199rg140707-APPENDIX D.xls

DESIGN RMA-2 INFLOW HYDROGRAPHS FOR STONEQUARRY CREEK FOR THE 10%, 20% & 30% INCREASED RAINFALL CLIMATE CHANGE SCENARIOS

Appendix F:
Distribution of Flows Relative to VxD Product Along Tributaries

FIGURE F1

DRAFT



VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	92	*80*
5.0	98	*85*
4.0	104	*90*
3.0	107	*93*
2.0	110	*96*
1.0	114	*99*
0.0	115	*100*

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	72	*63*
5.0	88	*77*
4.0	97	*84*
3.0	105	*91*
2.0	109	*95*
1.0	111	*97*
0.0	115	*100*

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	60	*52*
5.0	83	*72*
4.0	93	*81*
3.0	101	*88*
2.0	109	*95*
1.0	113	*98*
0.0	115	*100*

Cross-hatching indicates Velocity-Depth values greater than 8 m2/s (*i.e., greater than the adopted range of values*)

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	80	*70*
5.0	92	*80*
4.0	101	*88*
3.0	105	*91*
2.0	109	*95*
1.0	112	*97*
0.0	115	*100*

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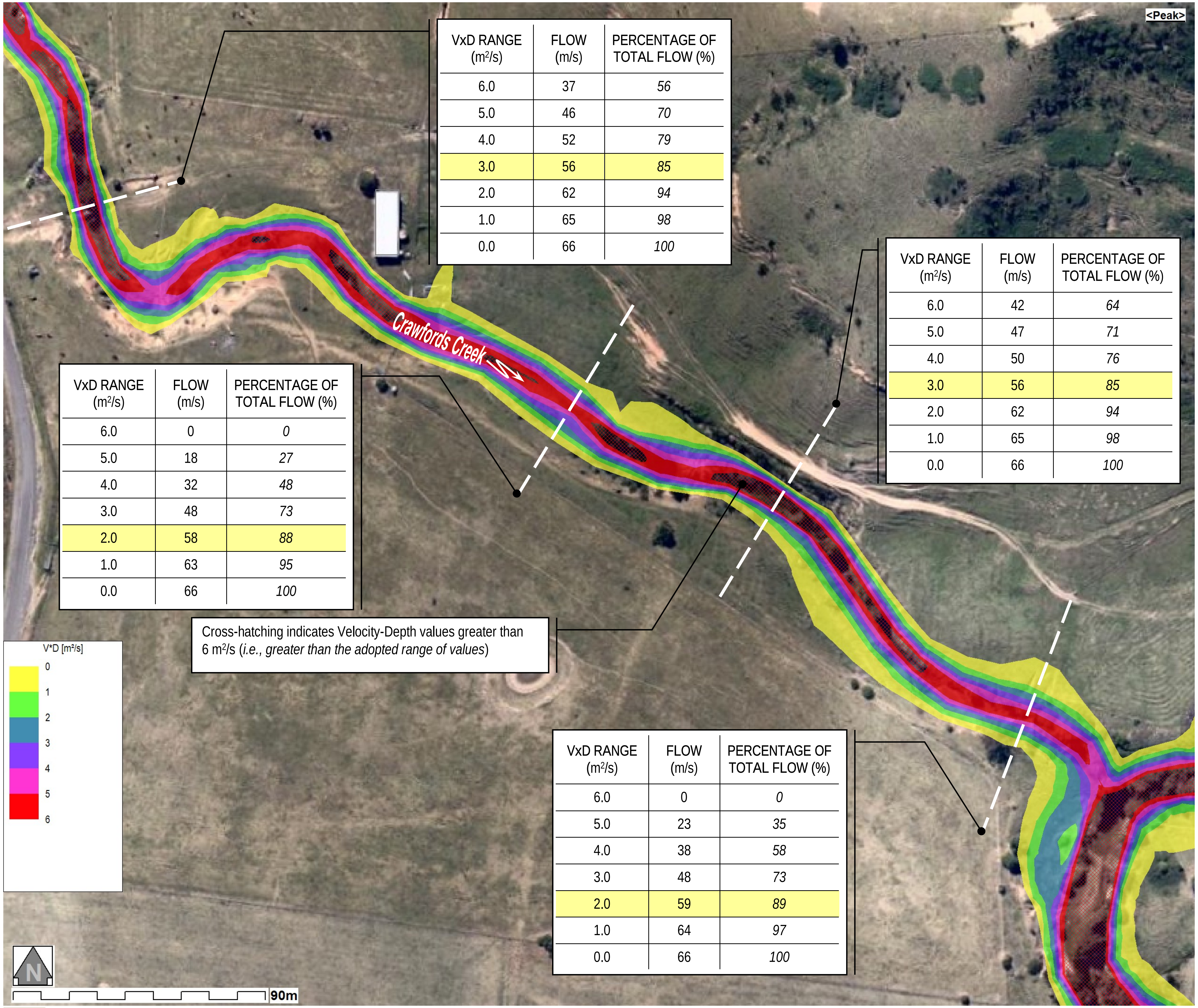
DISTRIBUTION OF CROSS-SECTIONAL FLOW RELATIVE TO VxD PRODUCT ALONG RACECOURSE CREEK FOR THE 1% AEP FLOOD (*UPSTREAM OF CRAWFORDS CK*)

FIGURE F2

DRAFT



VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	37	*56*
5.0	46	*70*
4.0	52	*79*
3.0	56	*85*
2.0	62	*94*
1.0	65	*98*
0.0	66	*100*

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	42	*64*
5.0	47	*71*
4.0	50	*76*
3.0	56	*85*
2.0	62	*94*
1.0	65	*98*
0.0	66	*100*

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	0	*0*
5.0	18	*27*
4.0	32	*48*
3.0	48	*73*
2.0	58	*88*
1.0	63	*95*
0.0	66	*100*

Cross-hatching indicates Velocity-Depth values greater than 6 m^2/s (*i.e., greater than the adopted range of values*)

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
6.0	0	*0*
5.0	23	*35*
4.0	38	*58*
3.0	48	*73*
2.0	59	*89*
1.0	64	*97*
0.0	66	*100*

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DISTRIBUTION OF CROSS-SECTIONAL FLOW RELATIVE TO VxD PRODUCT ALONG CRAWFORDS CREEK FOR THE 1% AEP FLOOD

FIGURE F3

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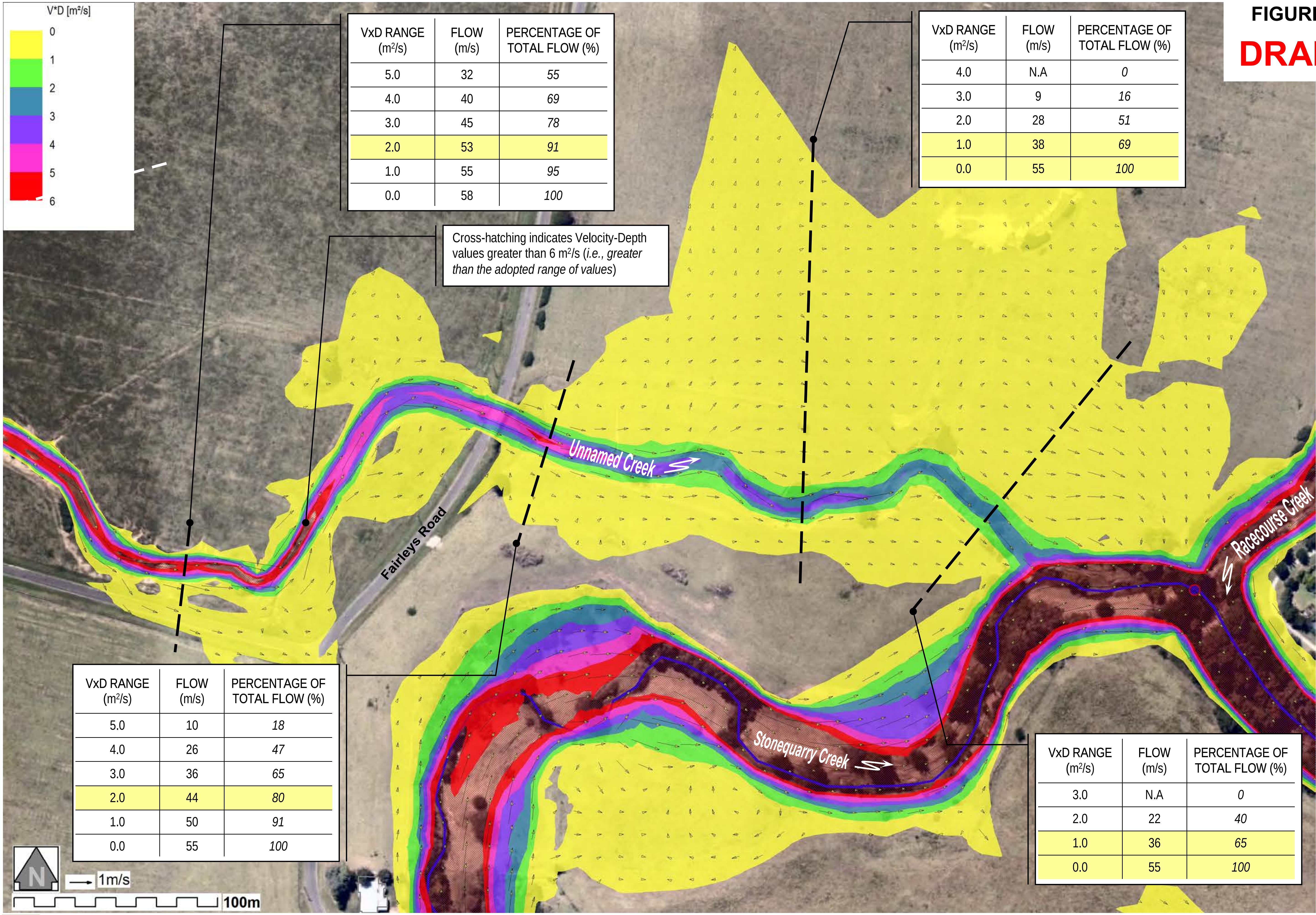

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
5.0	32	*55*
4.0	40	*69*
3.0	45	*78*
2.0	53	*91*
1.0	55	*95*
0.0	58	*100*

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
4.0	N.A	*0*
3.0	9	*16*
2.0	28	*51*
1.0	38	*69*
0.0	55	*100*

Cross-hatching indicates Velocity-Depth values greater than 6 m^2/s (*i.e., greater than the adopted range of values*)

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
5.0	10	*18*
4.0	26	*47*
3.0	36	*65*
2.0	44	*80*
1.0	50	*91*
0.0	55	*100*

VxD RANGE (m^2/s)	FLOW (m/s)	PERCENTAGE OF TOTAL FLOW (%)
3.0	N.A	*0*
2.0	22	*40*
1.0	36	*65*
0.0	55	*100*

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DISTRIBUTION OF CROSS-SECTIONAL FLOW RELATIVE TO VxD PRODUCT ALONG UNNAMED CREEK FOR THE 1% AEP FLOOD