APPENDIX M

FLOOD IMPACT ASSESSMENT

REPORT

Dalswinton Quarry Flood Investigation

Dalswinton Quarry Flood Investigation and Impact Assessment

Client: Rosebrook Sand & Gravel / HDB Town Planning and Design

Reference: PA1833R001D0.1_v3a

Revision: 0.1/Draft

Date: 21 October 2020

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Document title:	Dalswinton Quarry Flood Investigation
Document short title:	
Reference:	PA1833R001D0.1_v3a
Revision:	Final
Date:	21 October 2020
Project name:	
Project number:	PA1833
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Classification
Project related

MANAGEMENT SYSTEM CERTIFICATION
DNV·GL
ISO 9001 = ISO 14001
OHSAS 18001

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1 Introduction

1.1 Background

Rosebrook Sand and Gravel (RSG) are preparing a new development application that will expand the current quarrying operations that are being undertaken at their Dalswinton Quarry located at Lot 72 DP1199484, 511 Dalswinton Road, Dalswinton (refer **Figure 1-1**). The quarry is located 100 km north-west of Newcastle on the Hunter River floodplain

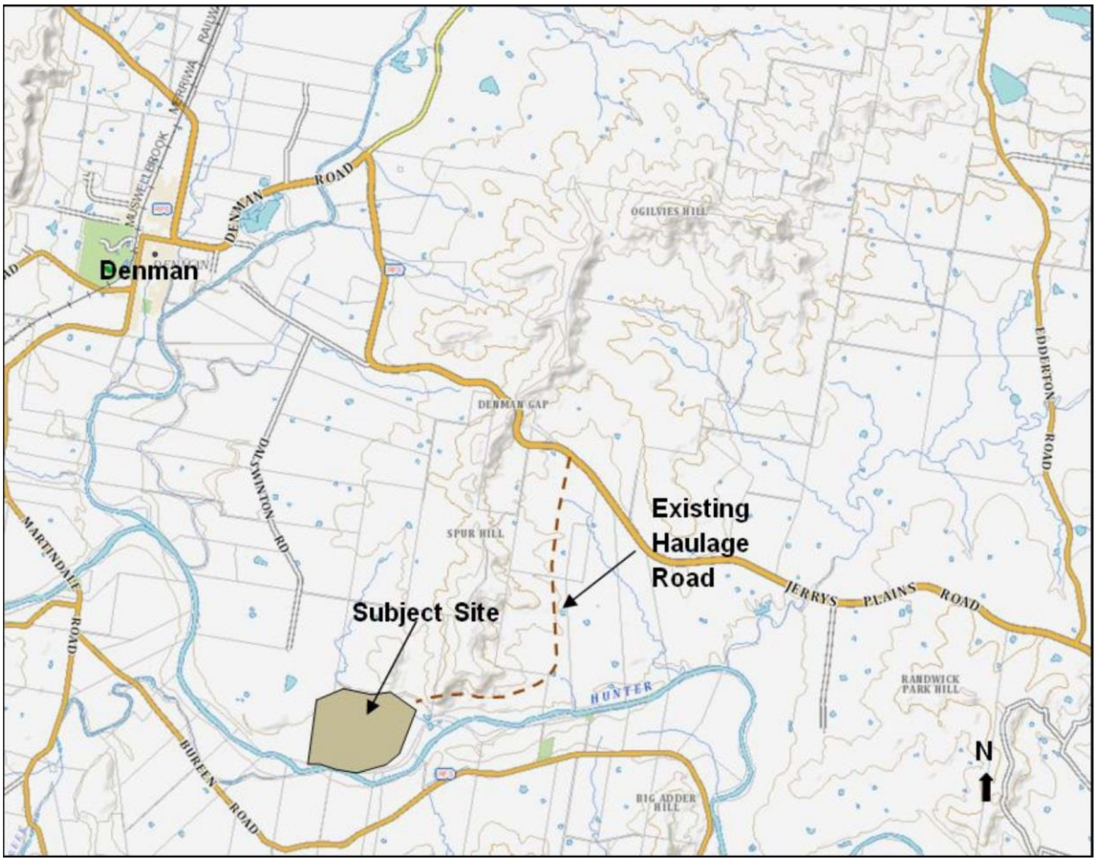

Figure 1-1: *Proposed Development Site: Lot 72 DP1199484, 511 Dalswinton Road, Dalswinton*

The new development application will seek to vary the footprint (refer **Figure 1-2 -** Proposed Work Area 2) and continue the extraction operation post 2022 (which at present is limited to 13 November 2022 under the current Development Application 410/1995).

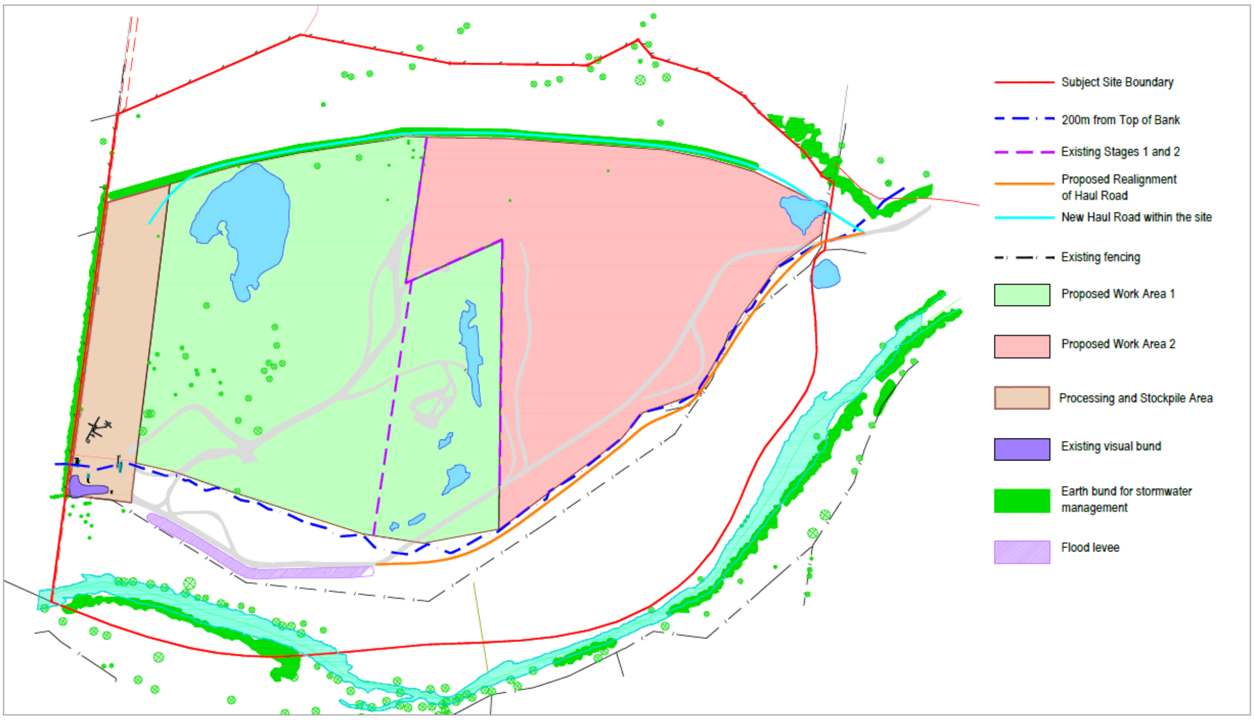

Figure 1-2: *Proposed Expansion of Dalswinton Quarry (Source: HDB)*

1.2 Recent Related Studies

As part of the Hunter River (Muswellbrook to Denman) Floodplain Risk Management Study and Plan (FRMS&P), Royal HaskoningDHV (RHDHV) completed an update and upgrade to the WorleyParsons (2014) flood model based on a comprehensive review of flood information for the Hunter River, stream gauge review and analysis, revision of the hydraulic roughness parameters, flood frequency analysis using data from the Muswellbrook stream gauge, model re-calibration and revised design event models for a range of design flood events using the Australian Rainfall and Runoff (ARR) 2016 (Commonwealth of Australia) guidelines.

A key finding of that work was that the changes to the stream gauge rating curve has led to the rated channel capacity at the Muswellbrook stream gauge (no. 210002) being reduced to approximately 45% of its assessed 1990 capacity. The current estimate is that flood levels have been reduced by up to 350 mm for both the 1% and 5% Annual Exceedance Probability (AEP) flood events, with reductions of up to 45% for peak flows for corresponding events.

1.3 Project Objectives and Scope

RHDHV was engaged by HDB Town Planning & Design (HDB) (on behalf of Rosebrook Sand and Gravel (RSG)) to provide an up-to-date robust and defensible flood model that can be used to quantify the existing flood risk and also the flood impact of the proposed development.

In relation to this flood investigation, the following key requirements have been specified as per the revised Secretary's Environmental Assessment Requirements (SEARs) issued on 14 August 2018:

- Impact of flooding on the site (water quality - among others);
- Predicted flood heights;
- The effect of the quarry infrastructure and stockpiles on flood flow;
- The risk of erosion in the quarry due to flooding;

- The risk of the river diverting its current course should the quarry be subject to flooding and erosion;
- The risk of quarry equipment being washed away and polluting the downstream environment during floods; and
- Demonstrate that the development will not increase the flood heights either upstream or downstream of the development.

This report documents the methodologies, assumptions and results from the above assessment

1.4 Report Structure

This report documents the flood impact assessment and is structured as follows:

- **Section 2** reviews information that was used in this report.
- **Section 3** describes the flood model setup.
- **Section 4** documents the adopted model hydrology including a comparison of design flow to flood frequency analysis that has been undertaken at a number of relevant stream gauges.
- **Section 5** provides the results of the existing flood conditions.
- **Section 6** provides the results of the proposed developed (final landform) flood conditions and includes an assessment of potential morphologic change.
- **Section 7** provides a summary of the study.

2 Review of Available Information

This section reviews information that was used to inform the flood impact assessment.

2.1 Relevant Reports

The following reports were reviewed as part of the flood impact assessment process.

Hunter River Flood Study (Muswellbrook to Denman): (WorleyParsons, 2014)

The Hunter River Flood Study (Muswellbrook to Denman) was produced by WorleyParsons in 2014 as part of the NSW Government's Floodplain Management Program. The study is informed by an integrated hydrologic and hydraulic model of the Upper Hunter River Floodplain Catchment. The model encompasses the entire extent of the Hunter River Floodplain that is located within the Muswellbrook Council Local Government Area (LGA). The upstream portion of the model (from the upstream LGA boundary to the Goulburn River) was developed in TUFLOW as a two-dimensional (2D) hydraulic model, while the lower portion of the model (from the Goulburn River to the downstream LGA boundary) was developed in TUFLOW as a one-dimensional (1D) hydraulic model dynamically linked to the upstream 2D model.

Surface elevations within the hydraulic model are informed by Light Detection and Ranging (LiDAR) data that was acquired by State Water in 2010. The integrated hydrologic and hydraulic models were calibrated using available information from flood events that occurred in 1998, 2000 and 2007. The study did not attempt to use available information from the 1955 or 1971 events or the extensive Muswellbrook stream gauge record to verify the model results.

The hydrologic and hydraulic models developed as part of that study were provided to RHDHV for use in the FRMS. RHDHV have modified some aspects of the models. All modifications are noted in **Section 3** of this report.

Hunter River (Muswellbrook to Denman) FRMS&P – Model Revision Report (RHDHV, 2017)

The Flood Study Revision (RHDHV, 2017) was required to produce an up-to-date flood study to provide appropriate information regarding flood risk to form the basis of the FRMS&P. The study included model re-calibration and validation of the models initially developed in the WorleyParsons (2014) flood study as well as updating the hydrology to use the latest ARR 2016 guidelines and techniques. The following scope for the model revision process was established by RHDHV in consultation with DPIE (then OEH) and Council:

- Review and analyse recent changes to stream gauge rating curves.
- Modify the Hunter River hydraulic model to more reliably represent the current floodplain characteristics.
- Recalibration of the Hunter River hydrologic and hydraulic models using data from flood events that occurred in 1998 and 2000.
- Undertake flood frequency analysis using data from the Muswellbrook stream gauge.
- Apply the outcomes from the model calibration and verification process and the Australian Rainfall and Runoff 2016 methods to establish revised design event conditions for a full range of Annual Exceedance Probability (AEP) flood events.
- Verify the revised design model outcomes using available data from the 1955 and 1971 events.

Hunter River (Muswellbrook to Denman) Floodplain Risk Management Study and Plan (RHDHV, 2019)

As part of the Muswellbrook Floodplain Risk Management Study and Plan (FRMS&P), Royal HaskoningDHV (RHDHV) completed an update and upgrade to the WorleyParsons (2014) flood model based on a comprehensive review of flood information for the Hunter River, stream gauge review and analysis, revision of the hydraulic roughness parameters, flood frequency analysis using data from the Muswellbrook stream gauge, model re-calibration and revised design event models for a range of design flood events using the Australian Rainfall and Runoff (ARR) 2016 (Commonwealth of Australia) guidelines.

A key finding of that work was that the changes to the stream gauge rating curve has led to the rated channel capacity at the Muswellbrook stream gauge (no. 210002) being reduced to approximately 45% of its assessed 1990 capacity. The current estimate is that flood levels have been reduced by up to 350 mm for both the 1% and 5% Annual Exceedance Probability (AEP) flood events, with reductions of up to 45% for peak flows for corresponding events.

The Muswellbrook FRMS&P provides design levels used for setting of flood planning levels (FPL) used to assess development applications.

2.2 Model and GIS Files

The hydrologic and hydraulic models and GIS files that were developed as part of the Muswellbrook FRMS&P (RHDHV, 2017) were provided to RHDHV for use in the flood investigation. The files were provided on the basis that any updates or improvements made during the study would be provided back to Council and DPIE.

RHDHV have modified some aspects of the models. All modifications are noted in **Section 3** of this report.

2.2.1 Description and Review of Existing Hydrologic Model

An XP-RAFTS hydrologic model capable of estimating design inflows to the Hunter River and Goulburn River was produced as part of the Hunter River Flood Study (Muswellbrook to Denman) (WorleyParsons, 2014). The sub-catchment breakdown used in the XP-RAFTS model is presented in **Figure 2-1**. As discussed in Section 2.1, the hydrologic model was recalibrated and update to ARR2016 during the Hunter River (Muswellbrook to Denman) FRMS&P study (RHDHV, 2017). Checks on the adopted hydrology are presented in Section 4.

Details of the catchment areas investigated as part of this study are detailed in **Table 2-1.**

***Table 2-1**: Details of Study Area Catchments*

Source	Catchment Size
Hunter River (above Denman))	4,510 km^2
Goulburn River	7,800 km^2

2.2.2 Description and Review of Existing Hydraulic Model

A hydraulic model of the study area was developed using TUFLOW as part of the Hunter River Flood Study (Muswellbrook to Denman) (WorleyParsons, 2014). The model comprised a 2D model domain over a 35km reach of the Hunter River floodplain (from ~5km upstream of Muswellbrook to the confluence of the Goulburn River) and a 1D representation of the ~30km reach of the Hunter River floodplain from the confluence of the Goulburn River to near Jerrys Plains. Model extents and features are presented in **Figure 2-2**.

During the Flood Study Revision (RHDHV, 2017) undertaken as part of the FRMS&P, significant model updates and improvements (mainly improved representation of in channel vegetation changes and also depths of the weir pools) were made to ensure the model was able to represent the current conditions of the main channel and floodplain. This meant that the model was now able to more closely match the observed rating curves at the Muswellbrook and Denman gauges. The model updates mean that a good model calibration to the 1998 and 2000 flood events was possible and further verification to large events in 1971 and 1955 allowed a high level of confidence to be associated with model results. The updates to the hydraulic model mean that despite significant reduction in design hydrology, average reductions in design flood levels are 0.2m for the 1% AEP (100yr ARI).

Predicted design depths for the 1% AEP (100yr ARI) event produced during the FRMS&P (RHDHV, 2018) are provided in **Figure 2-3**. It should be noted that the model is only 1D in the lower part of the model. On closer examination of the model, it shows that the 1D part of the model was overestimating conveyance as (due to topographical features of the floodplain) parallel high flow channels were being incorrectly included in conveyance calculations. The 1D part of the model would also be unable to resolve complex flow features in the vicinity of the quarry. This meant that converting the model to be fully 2D in the downstream section of the model was required in this study. Updates to the model are described in Section 3.

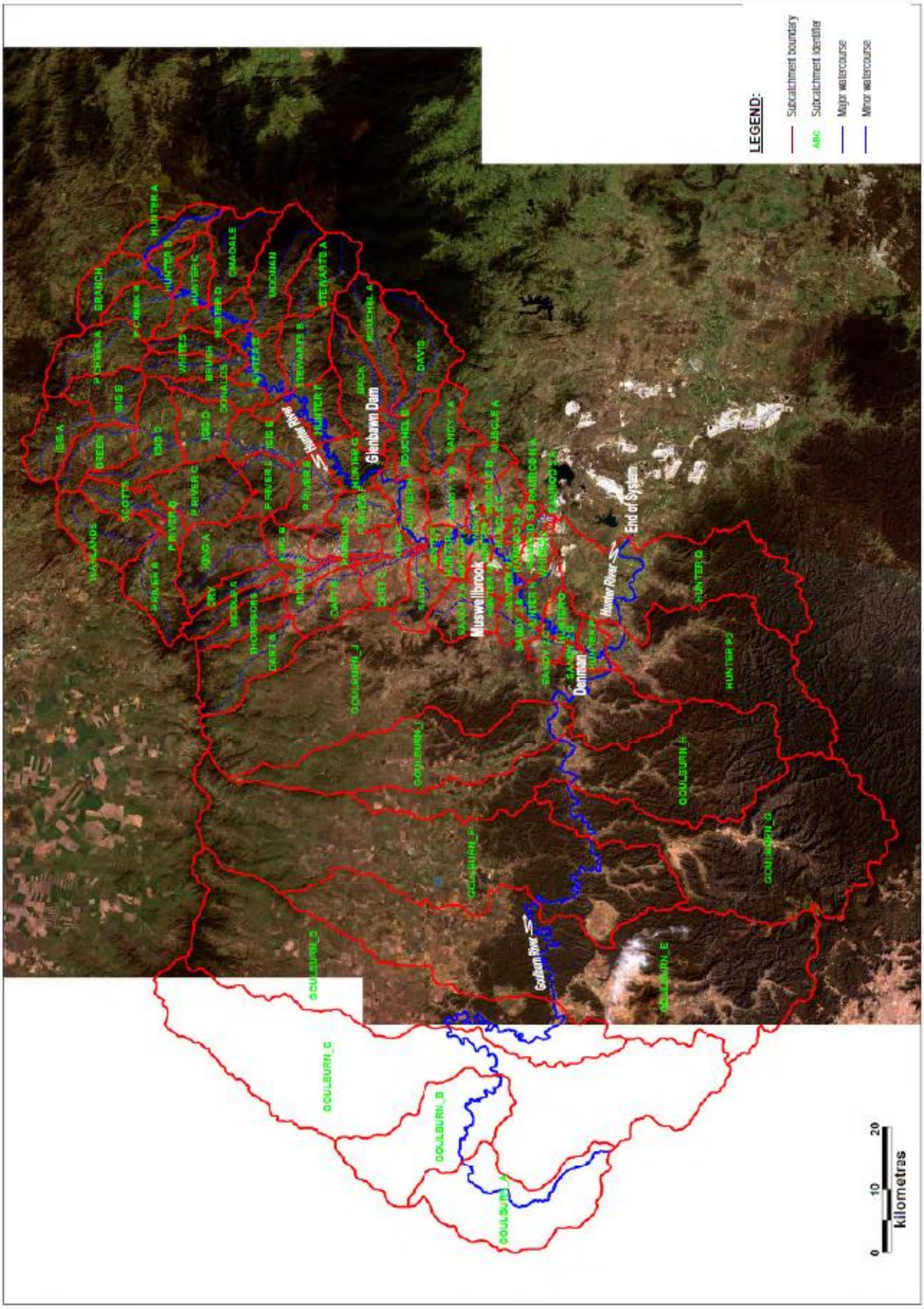

Figure 2-1: *XP-RAFTS Hunter and Goulburn River Subcatchments (Worley Parson, 2014)*

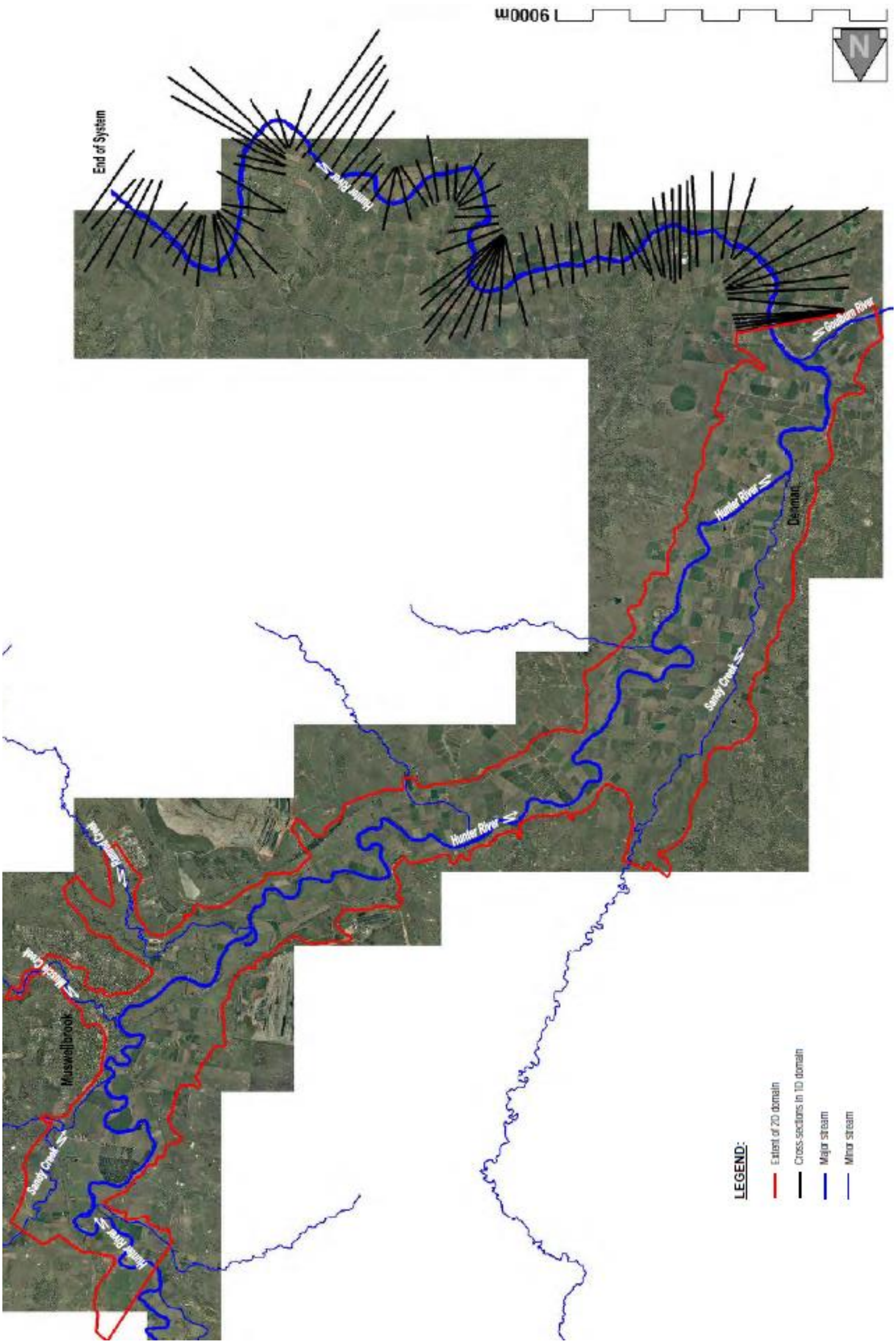

***Figure 2-2**: Hunter River Flood Study Model (Worley Parson, 2014)*

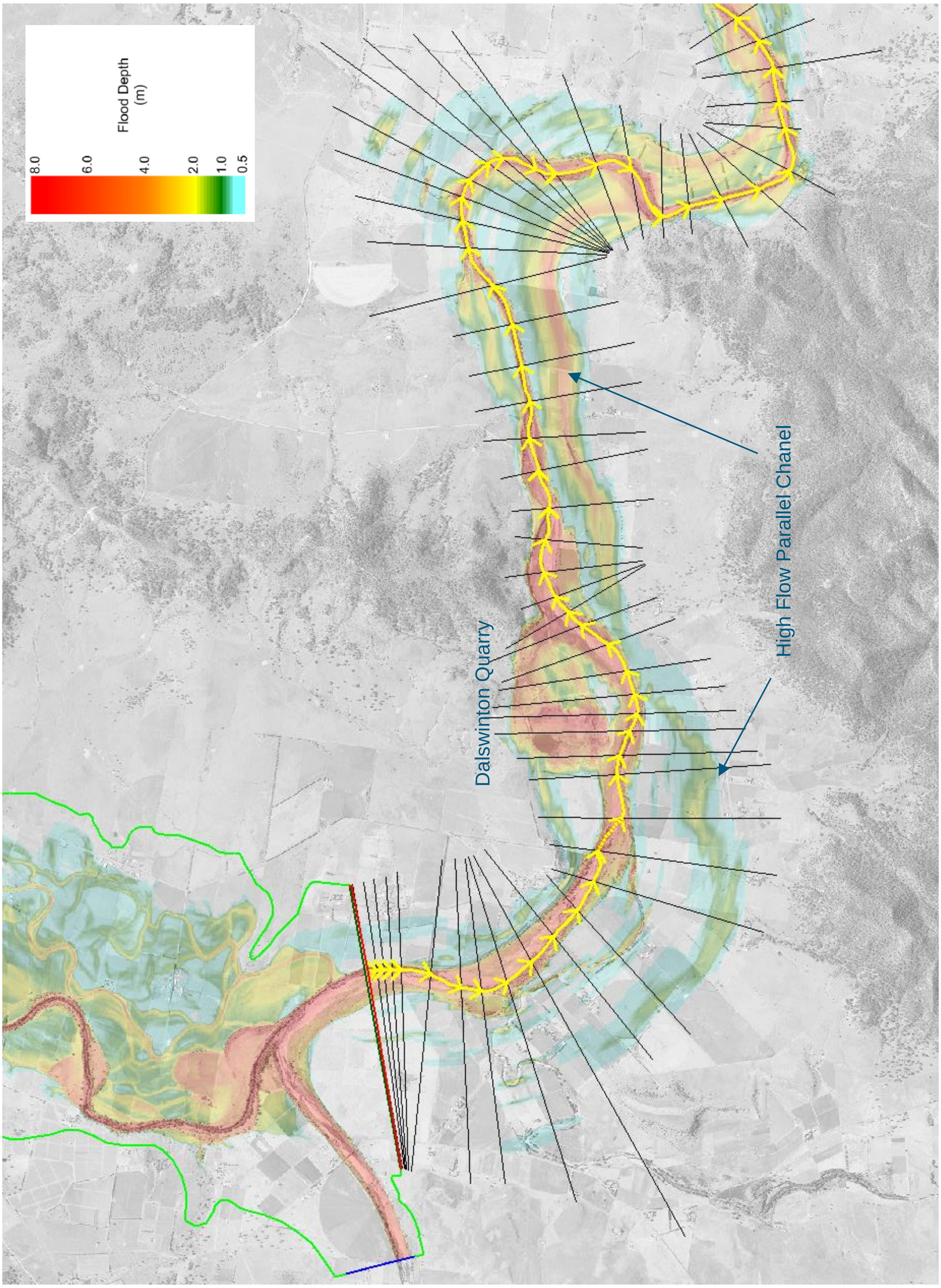

***Figure 2-3**: 100 year ARI Flood Depth (Hunter River FRMS&P (RHDHV, 2017)) and Model Features*

3 Model Setup and Description

A hydraulic model of the study area was developed using TUFLOW. The model was based on that developed as part of the Hunter River Flood Study (Muswellbrook to Denman) (WorleyParsons, 2014) and which was further revised and improved as part of the FRMS&P (RHDHV, 2017 & 2018). A review of the existing model was presented in Section 2.2.2 and found that the downstream 1D reach of the model was not suitable for this study. The following sections described the conversion of the 1D reach of the model to a full 2D domain.

3.1 Elevation data and 2D Extent

Updating the model from 1D to 2D required the availability of suitable elevation data. Elevation data was sourced from the NSW Foundation Spatial Data Framework (FSDF) (http://elevation.fsdf.org.au). The LiDAR data was flow on the 2nd November 2017. The data set contains a ground surface model in grid format derived from Spatial Services Category 2 (Classification Level 3) LiDAR (Light Detection and Ranging) from an ALS80 (SN8250). The data used to create this DEM has an accuracy of 0.3m (95% Confidence Interval) vertical and 0.8m (95% Confidence Interval) horizontal. The data is presented in **Figure 3-2.**

The adopted 2D model extent is also presented in **Figure 3-2.** It covers a similar area to the 1D domain and is large enough to appropriately model the PMF event.

Improved Definition of Channel Invert

The hydraulic model's DEM represents the surface levels of the floodplain and channel. The DEM is based on the LiDAR survey data. The LiDAR survey measured the level of standing water in the channel at the time of survey, rather than the channel invert. This has resulted in the cross-sectional area of the channel being understated. There is no survey information available that reliably defines the channel bathymetry. In the absence of any definitive data, it was decided to lower the channel invert of pool zones by 2 m to improve the channel conveyance.

Figure 3-1 shows a channel section at the Muswellbrook Gauge. The LiDAR levels and the adopted channel deepening approach are shown diagrammatically. The locations of the deepened channel pool areas is presented in **Figure 3-4**.

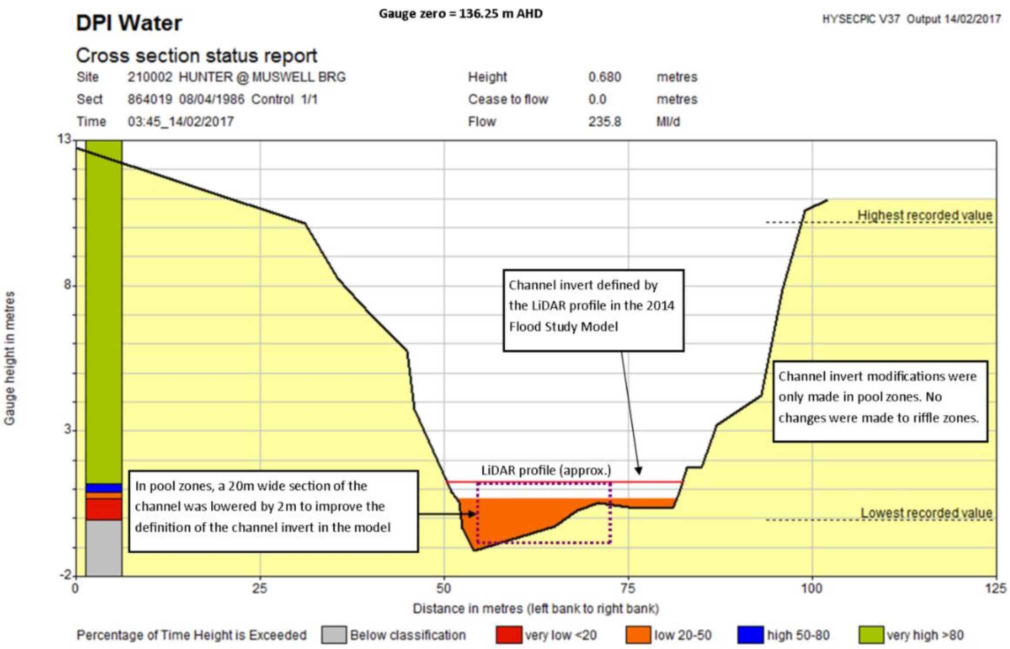

***Figure 3-1**: Channel deepening approach*

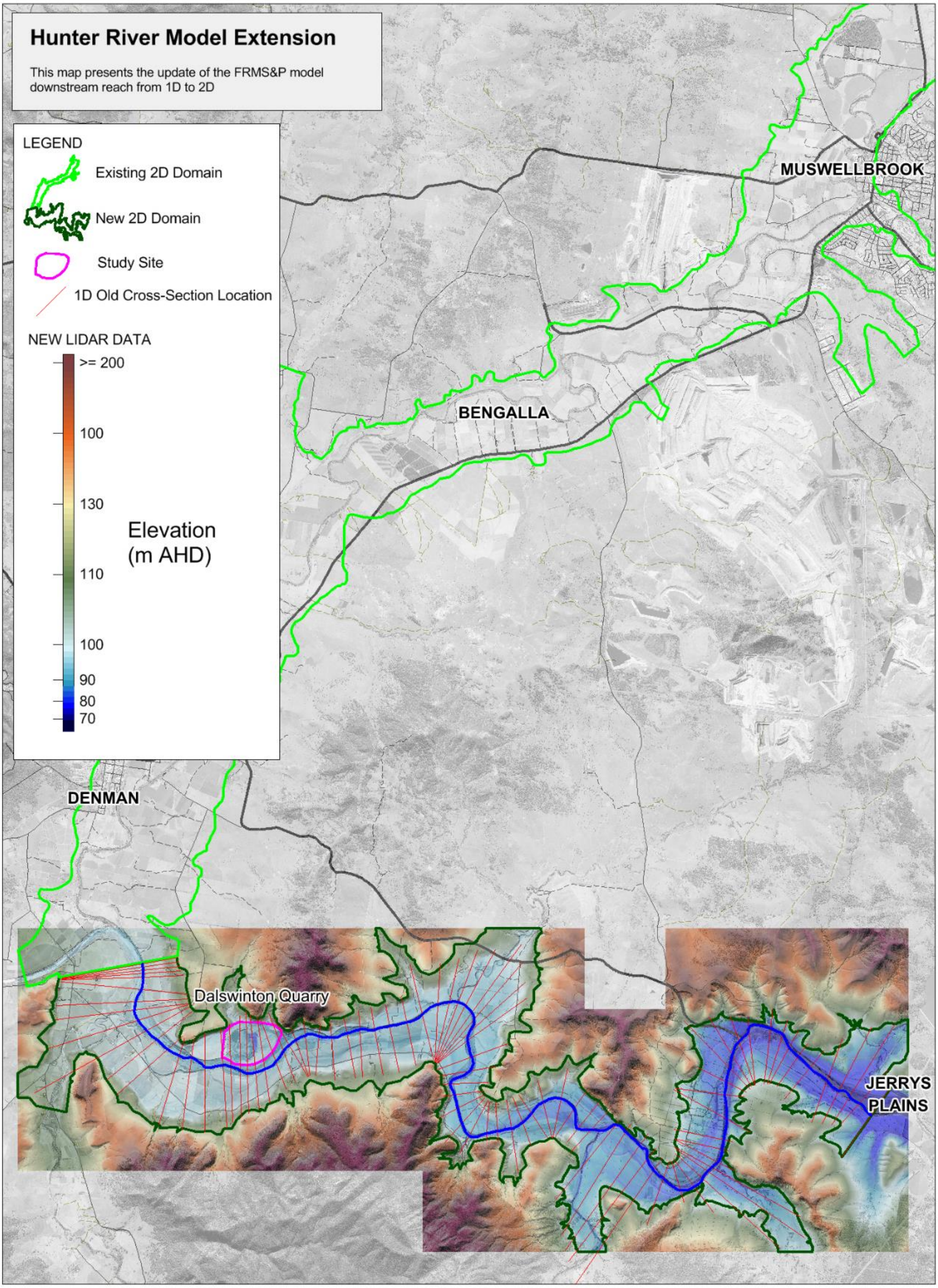

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FIGURE 3-1

HYDRAULIC MODEL EXTENT AND ELEVATION DATA

NORTH

0 1 2km
Approx. Scale

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3.2 Roughness Schematisation

Hydraulic roughness is a key parameter in any hydraulic model. Typically, a floodplain is divided into categories of roughness based on land use and the presence / absence of vegetation and other blockages or resistance to flow. A review of the definition of hydraulic roughness categories in the TUFLOW model developed for the Muswellbrook Flood Study (Worley Parsons, 2014) concluded that:

- The channel zone was generally defined as the base of the channel only, with the channel banks assumed to be low roughness floodplain category.
- Areas of dense floodplain vegetation were not defined.
- The default (floodplain) roughness (Manning's *n*) value assigned was near the lower limit expected for the combined surface types present in the broader floodplain area.
- Hydraulic roughness definition was revised in the entire 2D model domain. Key changes included:
    - More reliable definition of the channel bank / floodplain interface.
    - The channel zone was divided into a vegetated and un-vegetated category.
    - Areas of dense floodplain vegetation (i.e. olive groves, remnant vegetation) were included.
    - The floodplain roughness value was increased to reflect conditions between densely vegetated areas and clean straight channels.

Figure 3-3 shows an example of the changes made to the roughness category definition around Muswellbrook, a key area of the hydraulic model. The locations of in channel vegetation, a part of defining model roughness is presented in **Figure 3-4**.

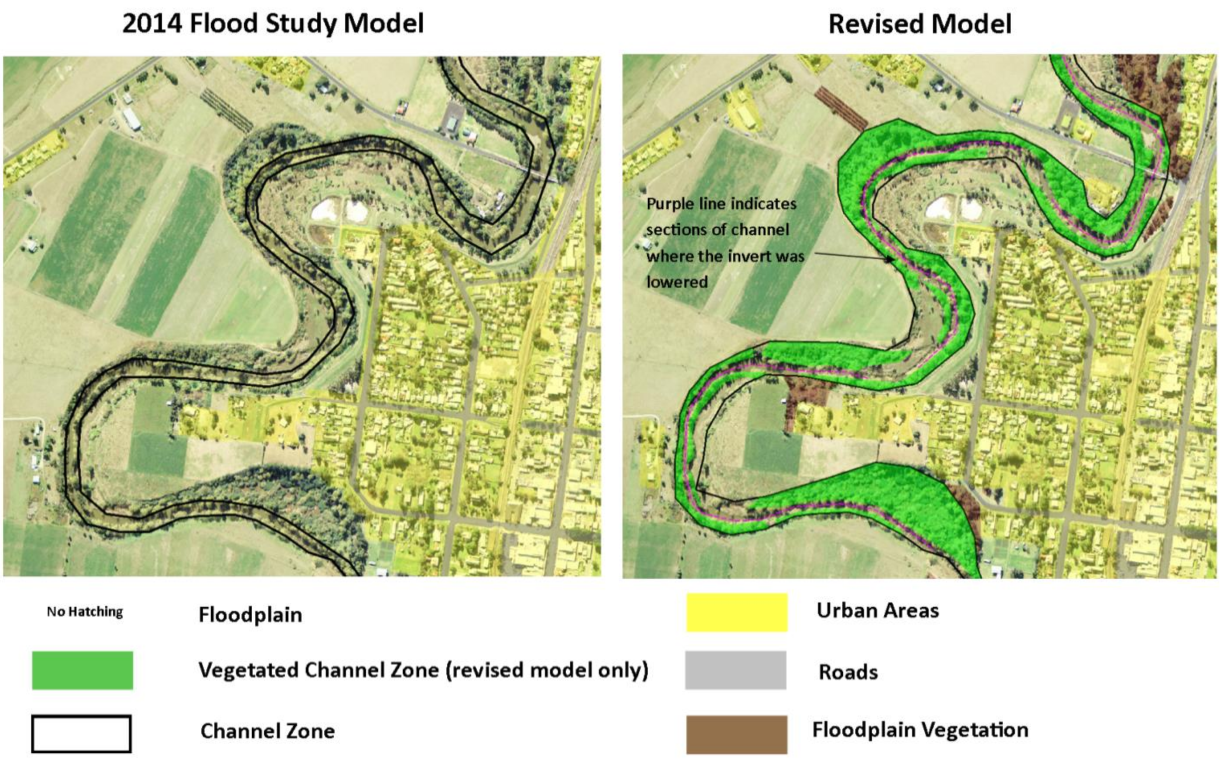

***Figure 3-3**: Changes to roughness categories*

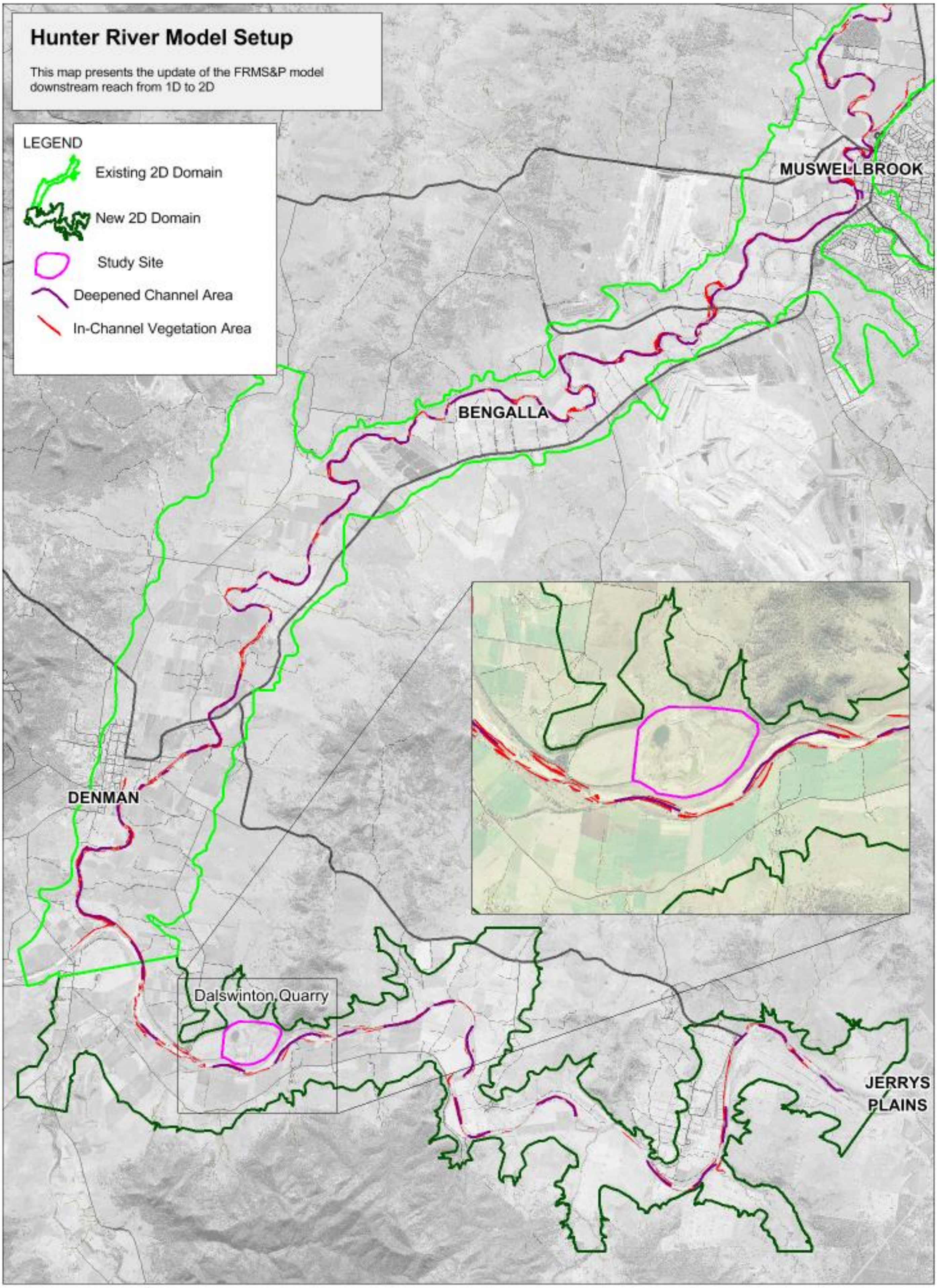

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FIGURE 3-4 REPRESENTATION OF IN-CHANNEL VEGETATION AND DEEPER CHANNEL POOL AREAS

NORTH

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Disclaimer: While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, this map contains information from a number of sources - no warranty is given that the information contained on this map is free from error or omission. Any reliance placed on such information shall be the sole risk of the user. Please verify the accuracy of all information prior to using it. This map is not a design document.

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4 Adopted Hydrology, Flood Frequency Analysis and Checks

4.1 Outcomes of Previous Investigations by RHDHV (2017)

4.1.1 Flood Frequency Analysis

Flood Frequency Analysis (FFA) applies observed annual peak discharge data to calculate the AEP of a given design discharge. This analysis assumes that previous floods will occur at the same frequency in the future and that the flood record is an accurate representation of the catchment's flood behaviour.

A comprehensive FFA was undertaken as part of the Muswellbrook Floodplain Risk Management Study and Plan (FRMS&P) (RHDHV (2017)) to estimate design flows at the Muswellbrook stream gauge. The following six step process was applied to complete the FFA:

- **Step 1** – Assess the hypothesis from the 1986 Flood Study that the Post-Glenbawn Dam and Post-Glenbawn Dam Upgrade series are homogenous
- **Step 2** – Undertake FFA on the Post Glenbawn Dam data set (1956 to 2016)
- **Step 3** – Undertake FFA on the Pre-Glenbawn Dam data set (1907 to 1955)
- **Step 4** – Compare Pre and Post Glenbawn Dam FFA results
- **Step 5** – Apply Bayesian Methods to incorporate Pre-Glenbawn Dam data and historical flood events into the post dam FFA
- **Step 6** – Undertake the final FFA.

The results of statistical analysis (t-test and the Mann-Whitney U-test) undertaken by RHDHV (2017) showed that the impact of the Glenbawn Dam upgrade on the two data sets is not statistically significant ($p>0.05$). This analysis verified that the Post Glenbawn Dam and Post Glenbawn Dam Upgrades were statistically similar and could be merged together to extend the annual maxima dataset.

The FFA analysis also revealed that the attenuation of peak discharges provided by Glenbawn Dam result in lower peak discharge estimates for the Post-Glenbawn Dam complete series (as expected). The FFA on the post-Glenbawn Dam and Pre-Glenbawn Dam datasets indicate that the 1955 flood event would have exceeded the peak discharge in the 1971 flood event had Glenbawn Dam been constructed at the time of the event.

Full details of the FFA undertaken are presented in RHDHV (2017a) while the final FFA is presented in **Figure 4-1** and summarised in **Table 4-5**.

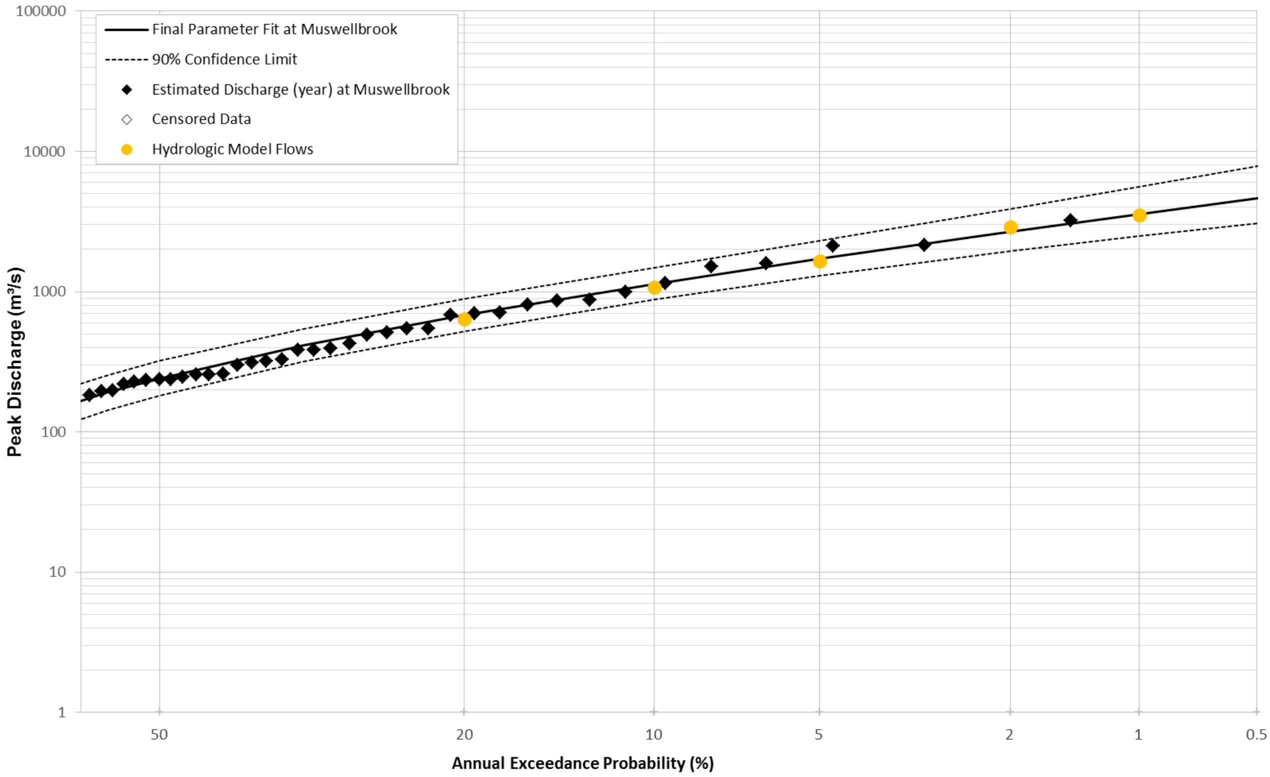

***Figure 4-1**: Final Muswellbrook Flood Frequency Analysis with Hydrologic Model Flows*

4.2 Hydrologic Modelling of ARR16 Design Events

Hydrologic modelling was undertaken as of the Muswellbrook Floodplain Risk Management Study and Plan (FRMS&P). The design hydrology was reviewed and found to be suitable for the current study. A description of the hydrological modelling as originally provided in RHDHV (2017) is presented below.

Hydrologic modelling has been undertaken in accordance with ARR 2016 using the XP-RAFTS model of the Hunter River catchment which was revised during the model calibration process that is described in **Section 4** of RHDHV (2017). A range of design events between the 20% and 0.2% AEP were simulated. This section describes the methodologies and assumptions applied to simulating the design events. Results are also discussed.

4.2.1 ARR 2016 - Design Rainfall

The Bureau of Meteorology (BoM) revised Intensity-Frequency Duration (IFD) rainfall depths as part of the ARR 2016 program. Design rainfall data provided by the BoM is an important input into a hydrologic model to determine flows for design storm events. This data was obtained for a range of storm events (both AEP and duration) in gridded format.

In large catchments with great changes in elevation, such as in the Hunter River Catchment, it is common for IFD depths to vary significantly across the catchment. As such, the average design rainfall depth for each sub-catchment was extracted from the gridded data provided by the BoM and input into the hydrologic model on a sub-catchment by sub-catchment basis.

4.2.2 Design Temporal Patterns

ARR 2016 recommends undertaking hydrologic modelling using an “ensemble” of ten storm temporal patterns. These ensembles account for the variability of temporal patterns that can occur in events of similar magnitudes. In the analysis of the resulting flows, ARR 2016 recommends selecting the temporal pattern that produces the peak flow just above the mean peak flow (i.e. the 6th highest peak flow). For the Hunter River catchment, an ensemble of “East Coast South” areal temporal patterns were applied to all design rainfall simulations.

4.2.3 Design Loss Parameters

ARR 2016 recommends using catchment specific loss parameters from calibrated hydrologic models if they are available. Otherwise, ARR 2016 provides recommended initial, continuing and pre-burst losses for ungauged catchments. For the Hunter River Catchment, ARR 2016 recommends an initial loss of 44 mm and continuing loss of 3.1 mm/hr.

The current study selected design continuing loss parameters based on the model calibration process. As discussed in **Section 4**, the calibration process applied a continuing loss of 1.5 mm/hr for both the 1998 and 2000 event simulations.

The initial losses were determined based on the design flow estimates from the FFA and the recommended initial and pre-burst losses from ARR 2016. For frequent flood events, it was found that an initial loss of 55 mm produces a hydrologic model flow that matches the flows derived in the FFA. For more rare events, the initial and pre-burst losses recommended in ARR 2016 were found to match the FFA flows.

Table 4-1 summarises the loss parameters adopted in the hydrologic model.

Table 4-1*: Hydrologic Model Losses*

Event (AEP)	Continuing Loss (mm/hr)	Pre-Burst Loss (mm)	Initial Loss adopted (mm)
20%	1.5	-	55
10%	1.5	-	55
5%	1.5	-	55
2%	1.5	*7.8*	36.2*
1%	1.5	*10.6*	33.4*
0.2%	1.5	*10.6^*	33.4*
0.5%	1.5	*10.6^*	33.4*

* Note: ARR 2016 Initial loss equals recommended initial loss (44 mm) minus pre-burst loss
^ Note: Preburst losses are not provided for events greater than the 1% AEP

4.2.4 Areal Reduction Factors

Areal Reduction Factors (ARF) are used to account for the spatial variation of design rainfall data which relates to a specific point in a catchment rather than to the entire catchment area. ARR 2016 recommends using the following equations for the South East Coast Region, where the Hunter River catchment is located.

Equation 1: Short duration ARF equation (less than and equal to 12 hours)

$$ARF = Min[1, 1 - 0.287(Area^{0.265} - 0.439log_{10}(Duration)).Duration^{-0.36} + 2.26 \times 10^{-3} \times Area^{0.226}.Duration^{0.125}(0.3 + log_{10}(AEP)) + 0.0141 \times Area^{0.213} \times 10^{-0.021\frac{(Duration-180)^2}{1140}}(0.3 + log_{10}(AEP))]$$

Equation 2: Equation for durations between 12 hours and 24 hours

$$ARF = ARF_{12\,hour} + (ARF_{24\,hour} - ARF_{12\,hour})\frac{(Duration - 720)}{720}$$

Equation 3: Long duration ARF equation (greater than 24 hours to 168 hours)

$$ARF = Min\{1, [1 - a(Area^b - clog_{10}Duration)Duration^{-d} + eArea^f Duration^g(0.3 + log_{10}AEP) + h10^{iArea\frac{Duration}{1440}}(0.3 + log_{10}AEP)]\}$$

Where:
Duration = storm duration (minutes)
Area = area of interest (km²)
AEP = Annual exceedance probability as a fraction (between 0.5 and 0.0005).

***Table 4-2**: Parameters for ARF long duration equation (Equation 3)*

Region	a	b	c	d	e	f	g	h	i
South – East Coast	0.06	0.361	0	0.317	8.11E-05	0.651	0	0	0

The equations above were used to calculate the ARF for the hydrologic modelling undertaken in the current study. By way of example, an ARF of 0.85 was calculated for the 1% AEP design event.

4.2.5 Critical Duration Assessment

For all AEP event simulations, a critical duration assessment was carried out for flows at the Muswellbrook Gauge to determine which storm duration produces the highest flows in the Muswellbrook area. The flow hydrographs for the 1% AEP event of varying durations at the Muswellbrook gauge are shown in **Figure 4-2**. The 24 hour duration event was found to be critical along the Hunter River at the Muswellbrook Gauge.

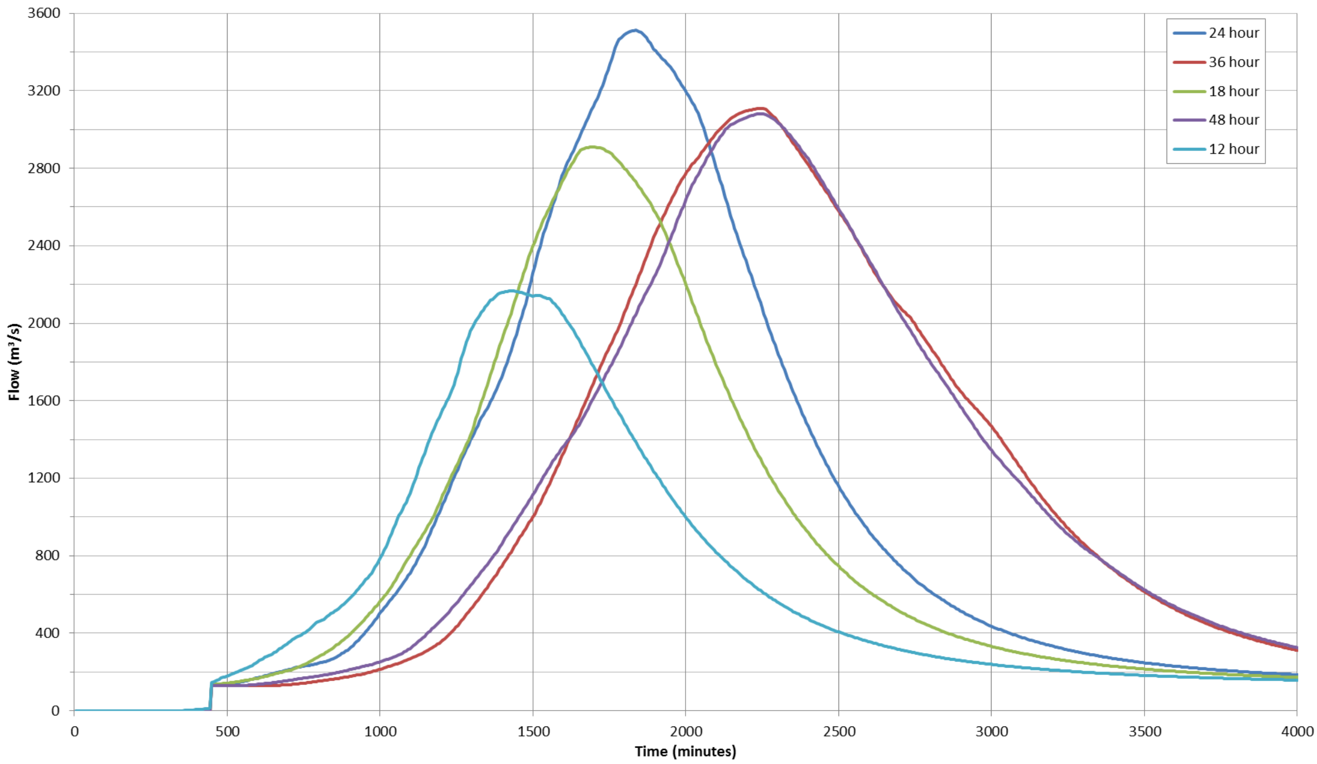

Figure 4-2: *Muswellbrook Gauge, Hunter River – Critical Duration -1% AEP Flow Hydrographs*

In contrast, the 2014 Flood Study found that the 48 hour and 36 hour durations were critical using the techniques recommended in ARR 1987.

4.2.6 Ensemble Storm Analysis

ARR 2016 recommends undertaking hydrologic modelling using an ensemble of ten storm temporal patterns. These ensembles account for the variability of temporal patterns that can occur in events of similar magnitudes. In the analysis of the resulting flows, ARR 2016 recommends selecting the temporal pattern that produces the peak flow just above the mean peak flow (i.e. the 6th highest peak flow). **Figure 4-3** shows the ten ensemble storm hydrographs at the Hunter River inflow boundary. It is noted that Hunter River inflow boundary is located upstream of the Muswellbrook Gauge. Hence, the peak flows are slightly lower than results reported at the Muswellbrook Gauge location. Storm 9 produced the 6th highest flow and was adopted for design event simulations.

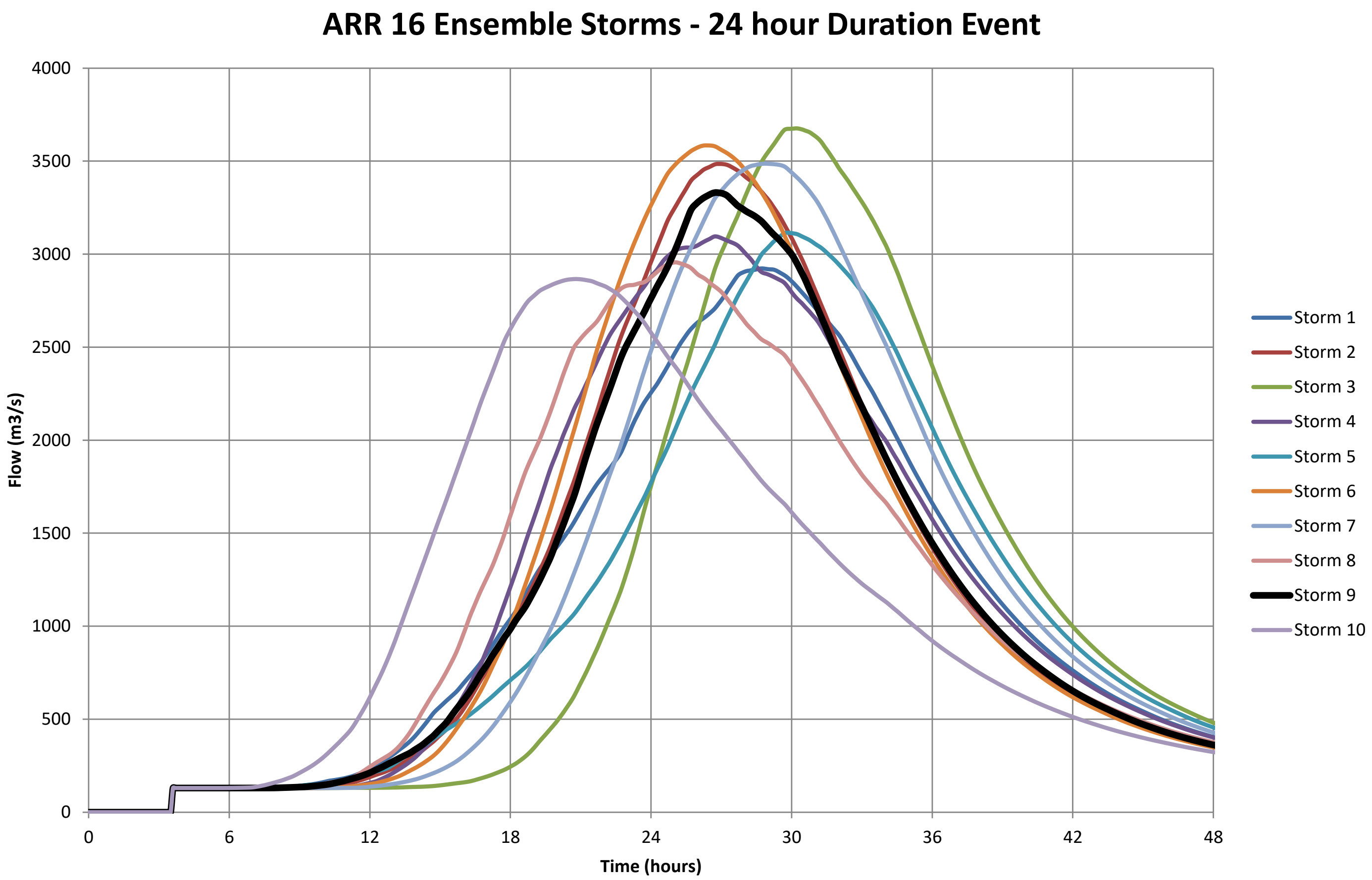

Figure 4-3: *1% AEP: Ensemble Storm hydrographs (Hunter River Inflow Boundary)*

4.2.7 Hydrologic Model Design Flow Results

Design flow results derived in the hydrologic model are presented in **Table 4-3** below. The 1971 and 1976 events are also presented to provide historical context to these revised design flow results. It is noted that the revised design peak flows are similar to the flows calculated using the FFA that are documented in **Table 4-5**.

Table 4-3*: Design and Historic Event Flows (Muswellbrook Gauge)*

	Post Glenbawn Dam Construction								
Event (AEP)	**20%**	**10%**	**5%**	**1976**	**2%**	**1971**	**1%**	**0.5%**	**0.2%**
Flow at Muswellbrook Gauge (m^3/s)	637	1076	1653	**2109**	2895	**3207**	3512	4072	4857

4.3 ARR 1987 vs ARR 2016

The revised hydrologic model (as described in **Section 4** of RHDHV (2017) was applied to simulate the governing duration 1% AEP event applying both the ARR 1987 and ARR 2016 methods. For the ARR 1987 method simulation, the initial and continue losses adopted in the Flood Study (Worley Parsons, 2014) were applied. For the ARR 2016 method, the revised loss assumptions that are documented in **Section 4.2** were applied. The resulting hydrographs are provided in **Figure 4-4**.

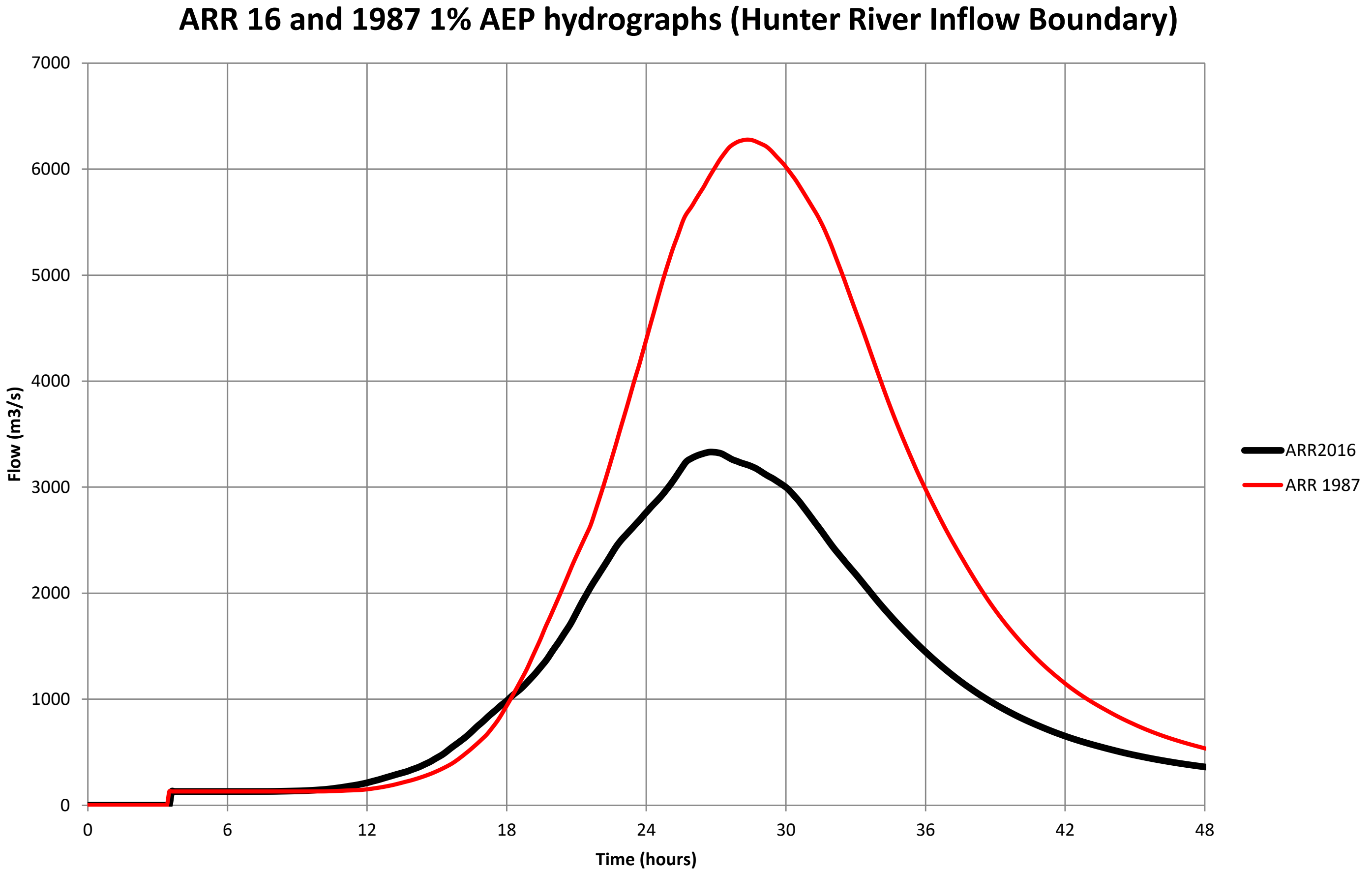

Figure 4-4: *1% AEP Hunter River Inflow Hydrographs: ARR1987 and ARR 2016 Methods*

The hydrographs provided in **Figure 4-4** show that the ARR 2016 method produces a peak flow that is substantially lower than the flow calculated using the ARR 1987 method. This is due to the ARR 2016 method producing substantially lower rainfall excess (i.e. the portion of the IFD rainfall that is converted to runoff in the model). **Table 4-4** provides a break-down of some of the key contributing factors at two locations within the simulated catchment area.

Table 4-4: *Comparison of ARR 1987 and ARR 2016 Methods*

Catchment	**Hunter River Catchment @ Muswellbrook (HUNTER I)**						
1% AEP	Critical Duration	IFD Rainfall (mm)	Areal Reduction Factor	Initial Loss (mm)	Continuing Loss (mm/hr)	Rainfall Excess (mm)	Resulting Flow (m^3/s)
ARR 1987	36 hour	206	0.92	20.0	2.5	117	6,280
ARR 2016	24 hour	155	0.85	33.4	1.5	80	3,330
Catchment	**Pages River Headwater Catchment (P RIVER F)**						
ARR 1987	36 hour	207	0.92	20.0	2.0	132	3,140
ARR 2016	24 hour	152	0.85	33.4	1.5	82	1290

The information in **Table 4-4** indicates that the rainfall excess calculated using the ARR 1987 method is approximately 50% higher than the depth calculated using the ARR 2016 method. The key contributing factors to this are:

- Lower IFD depths (reduced from 206 mm to 155 mm). This is partially due to the ARR 1987 method having a longer critical duration event; and
- Lower ARF (reduced from 0.92 to 0.85).

It is noted that the higher initial losses applied to the ARR 2016 method simulation is approximately offset by lower continuing losses.

4.3.1 Comparison of Hunter River Hydrologic Assessments

A comparison of adopted hydrologic inflows to the comprehensive flood frequency analysis (FFA) of flow gauge data at Muswellbrook (RHDHV, 2017) are presented in **Table 4-5**. The data shows that the adopted hydrologic inflows (from an XP-RAFTS model) are within 2 to 10% of those derived from a comprehensive flood frequency analysis (FFA) of flow gauge data at Muswellbrook for all events up to the 1% AEP. The close agreement between the FFA and the design hydrologic estimate using ARR2016 adopted in this study allow a good degree of certainty to be associated with the estimates of flood levels calculated in this study.

Hydrologic inflows presented in RHDHV (2017) are also compared to the hydrologic inflows estimated in WorleyParsons (2014) as presented in **Table 4-5.** In general the adopted hydrologic flow used in this study is typically 30% lower than those calculated in Worley Parsons (2014). The adoption of ARR2016 procedures and in particular updated IFD data is responsible for the majority of the differences in design hydrology as discussed in RHDHV (2017).

Table 4-5: *Flood Frequency Analysis & Design Flow Comparison at the Muswellbrook Gauge*

Event (AEP)	FFA Flow (m^3/s)	Adopted Hydrologic Model Flows (m^3/s)	Previous Flood Study[2] Hydrologic Model Flows (m^3/s)
0.2 EY	**680**	**640***	**1125***
10%	**1137**	**1080**	**2430**
5%	**1714**	**1650**	**3107**
2%	**2682**	**2900**	**3973**
1%	**3583**	**3510**	**4857**
0.5%	**4643**	**4070**	**5800**
0.2%	**6308**	**4860**	**7199**

*Note: 0.2 EY has a slightly different probability of occurrence to the 20% AEP, equivalent to 18.13% AEP

[2] Note: From Table 6.2 Worley Parsons (2014). Also flows are from upstream of Muswellbrook Gauge so are slightly lower than if a comparison at the actual gauge was available.

4.4 Check on Downstream Design Discharge Estimates

As the FFA and Design hydrology undertaken for the Muswellbrook FRMS&P focussed on the Muswellbrook gauge, no significant checks on design discharge estimates downstream of the Goulburn River confluence were required. However, as the current study is significantly influenced by Goulburn River inflows, additional checks on total system hydrology were required. The checks involved:

- Ensuring that total design discharges on the Hunter River downstream of the Goulburn River confluence match available FFA for any suitable gauges (this Section).
- Checking that design storms for the Goulburn River alone are not of a greater magnitude than combined system discharges (**Section 4.5**).

4.4.1 Suitability of Gauge and AMAX DATA

The Liddell Gauge (210083) is less than 5 km downstream from the end of the TUFLOW flood model so can be directly compared to modelled discharges. There are 48 years of available data which means the gauge will produce a good estimate for design flows up to the 50 yr ARI and a reasonable estimate of the 100 yr ARI.

Annual Maxima (AMAX) Series Data

Flow data is available for the Hunter River at Liddell for the period 1969 to 2017. **Table 4-6** presents annual maximum series of peak flood flows for the gauge.

Table 4-6: *Liddell Gauge Annual Series Data Set*

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1969	185.5	**1986**	38.8	**2003**	57.7
1970	209.2	**1987**	150.8	**2004**	64.2
1971	4365.9	**1988**	274.3	**2005**	45.9
1972	916.9	**1989**	1041.6	**2006**	6.0
1973	542.1	**1990**	1504.4	**2007**	2586.6
1974	855.0	**1991**	67.9	**2008**	436.4
1975	109.7	**1992**	2780.3	**2009**	94.8
1976	3044.3	**1993**	101.6	**2010**	942.9
1977	3545.9	**1994**	22.2	**2011**	822.5
1978	914.0	**1995**	302.2	**2012**	738.4
1979	601.9	**1996**	615.0	**2013**	1028.9
1980	56.4	**1997**	201.4	**2014**	46.8
1981	126.0	**1998**	2289.8	**2015**	180.3
1982	697.9	**1999**	97.4	**2016**	328.7
1983	154.7	**2000**	2000.2	**2017**	39.9
1984	1513.0	**2001**	239.7		
1985	277.8	**2002**	58.2		

4.4.2 Flood Frequency Analysis Methodology

The FFA was undertaken using the Flike software package (version 5.0.251.0), using the annual maximum method. This method applies the highest recorded discharge for each year of record to the FFA. This method prevents the inclusion of successive dependent peaks. A Bayesian maximum likelihood approach was used to fit a specified probability distribution for each data set. This analysis used a Log-Pearson III (LP3) distribution.

FFA was undertaken for both:

- Bayesian with no prior information, and
- Bayesian with Gaussian prior distribution from Regional Flood Frequency Estimation (RFFE) analysis.

Bayesian with Gaussian prior distribution from Regional Flood Frequency Estimation (RFFE) analysis tends to produce a better fit as it uses distribution fitting parameters taken from a much larger pool of data. The larger pool of data is taken from gauges with similar geographical characteristics using the RFFE website (https://rffe.arr-software.org/) which is a tool provided as part of the new ARR2016 guideline.

4.4.3 Results

The design FFA plot at Liddell for the Bayesian with no prior information is displayed in **Figure 4-5** while the FFA plot at Liddell for the Bayesian with Gaussian prior distribution from RFFE analysis is presented in **Figure 4-6**. The graphs show that the fit for the FFA using Bayesian with Gaussian prior distribution from RFFE provides a better match to the available data (especially for the more extreme events) and also has much tighter 90% confidence limits. The improved FFA fit also matches more closely with the hydrologic estimates of design discharge. FFA and hydrologic design flows are tabulated in **Table 4-7** and show that events from the 10% AEP (10yr ARI) to the 1% AEP (100yr ARI) are with 5-10% of each other given good confidence in the adopted hydrology.

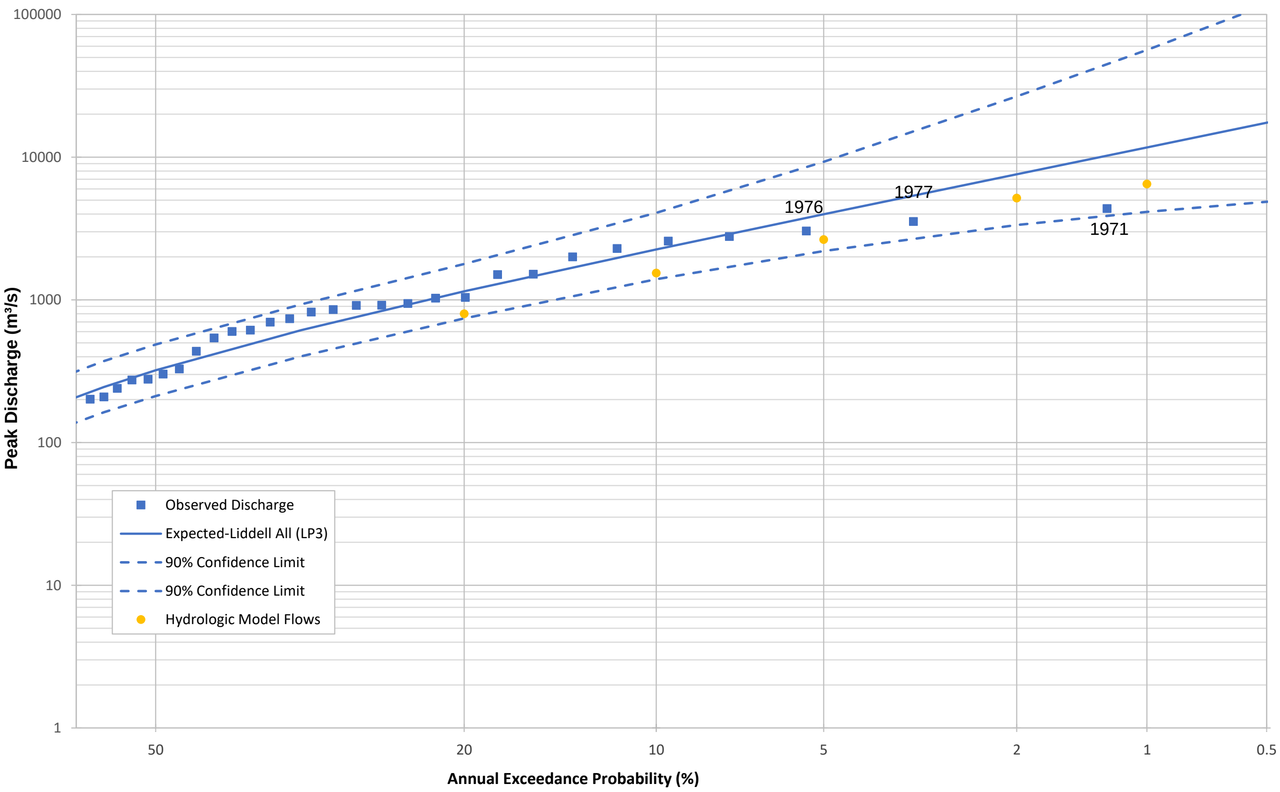

***Figure 4-5**: Liddell Flood Frequency Analysis (Bayesian with no prior information) and Hydrologic Model Flows*

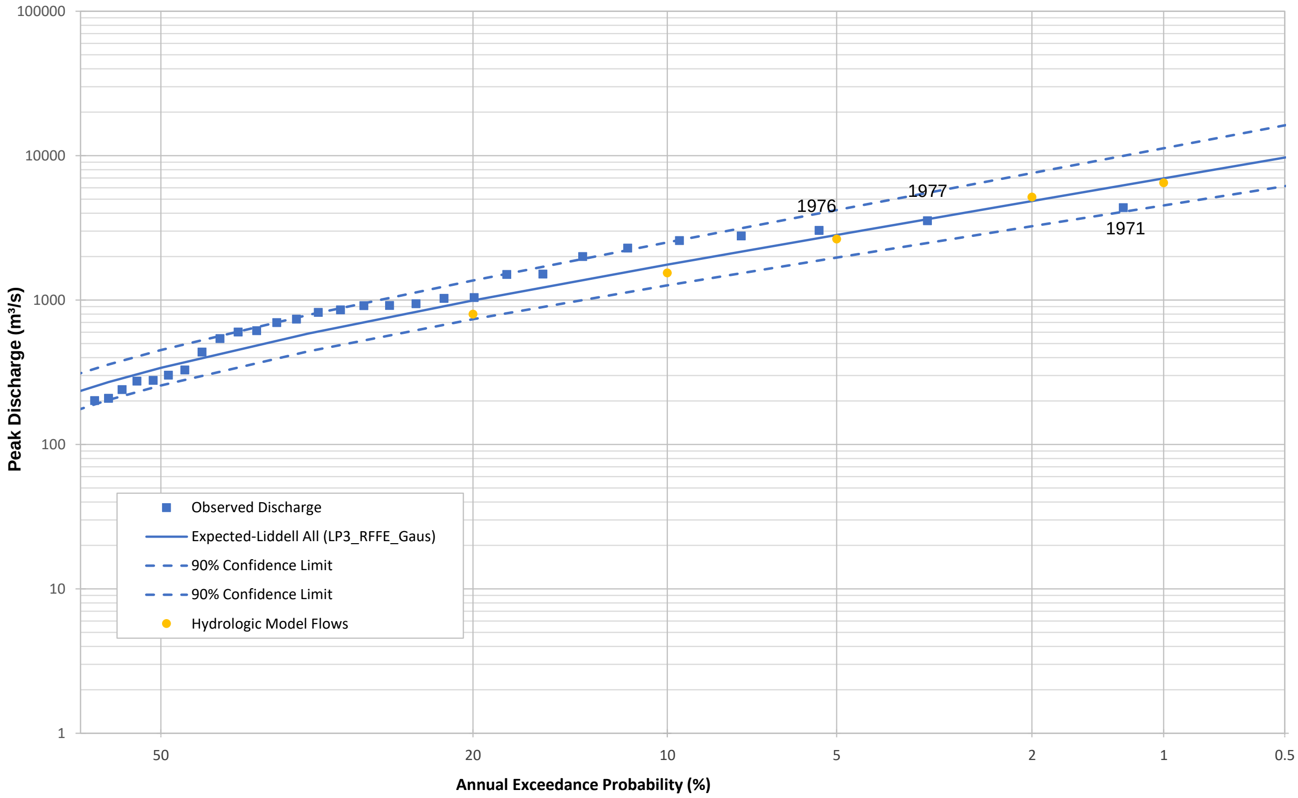

Figure 4-6*: Liddell Flood Frequency Analysis (Bayesian with prior RFFE information) and Hydrologic Model Flows*

Table 4-7*: Flood Frequency Analysis and Hydrologic Design Flows at the Liddell Gauge*

Event (AEP)	Flow (m^3/s)	90% Confidence Limits		Hydrologic Model Flows (m^3/s)
		Lower Flow (m^3/s)	Upper Flow (m^3/s)	
10%	**1758**	1265	2512	**1542**
5%	**2826**	1971	4206	**2648**
2%	**4844**	3247	7575	**5173**
1%	**6954**	4531	11260	**6489**

4.5 Check on Goulburn River Design Discharge Estimates

As the FFA and Design hydrology undertaken for the Muswellbrook FRMS&P focussed on the Muswellbrook gauge, no significant checks on design discharge estimates of the Goulburn River confluence were required. While the above check (**Section 4.4**) shows that the combined design hydrology is appropriate a check of whether the Goulburn River alone could produce higher design discharges than the combined system was made.

4.5.1 Suitability of Gauge and AMAX DATA

The Sandy Hollow Gauge (210031) is less than 20 km from the confluence of the Hunter River and is located in a good position to measure Goulburn River flows at the end of the catchment. There are 63 years of available data which means the gauge will produce a good estimate for design flows up to the 50 yr ARI and a reasonable estimate of the 100 yr ARI.

Annual Maxima (AMAX) Series Data

Flow data is available for the Hunter River at Sandy Hollow for the period 1954 to 2017. However, to reduce the influence of non-flood years a 20 m^3/s threshold was adopted reducing the record from 63 to 53 years. **Table 4-8** presents annual maximum series of peak flood flows for the gauge.

Table 4-8: Sandy Hollow Gauge Ranked Annual Series Data Set

Year	Flow (m^3/s)	Year	Flow (m^3/s)	Year	Flow (m^3/s)
1955	6591.2	**1984**	436.3	**2017**	84.3
1971	3191.6	**1973**	352.4	**1968**	84.1
1977	2800.7	**1963**	340.1	**1962**	77.2
2007	2747.8	**1974**	323.2	**1969**	73.6
1992	1664.6	**2008**	299.4	**1978**	72.9
1972	1291.8	**2016**	225.6	**1970**	64.4
1956	1138.7	**1958**	208.1	**2014**	61.7
2013	1072.3	**1988**	194.4	**1967**	54.1
2010	1067.2	**1979**	191.1	**2005**	51.4
1976	889.5	**1996**	130.8	**1986**	46.8
1989	876.2	**2015**	118.9	**1966**	41.6
1990	853.3	**1983**	114.1	**1997**	35.2
1998	846.1	**1981**	104.4	**2003**	35.0
2012	764.6	**1959**	100.7	**1991**	34.7
2011	605.3	**1961**	96.3	**1975**	30.6
1982	594.9	**1985**	93.7	**2009**	26.0
2000	524.9	**1987**	93.7	**1960**	23.5
1964	503.9	**1995**	90.6		

4.5.2 Flood Frequency Analysis Methodology

The FFA was undertaken using the Flike software package (version 5.0.251.0), using the annual maximum method. This method applies the highest recorded discharge for each year of record to the FFA. This method prevents the inclusion of successive dependent peaks. A Bayesian maximum likelihood approach was used to fit a specified probability distribution for each data set. This analysis used a Log-Pearson III (LP3) distribution.

FFA was undertaken for Bayesian with Gaussian prior distribution from Regional Flood Frequency Estimation (RFFE) analysis. Bayesian with Gaussian prior distribution from Regional Flood Frequency Estimation (RFFE) analysis tends to produce a better fit as it uses distribution fitting parameters taken from a much larger pool of data. The larger pool of data is taken from gauges with similar geographical characteristics using the RFFE website (https://rffe.arr-software.org/) which is a tool provided as part of the new ARR2016 guideline.

4.5.3 Results

The design FFA plot at Sandy Hollow is presented in **Figure 4-7**. FFA and hydrologic design flows are tabulated in **Table 4-9**. The results show that the adopted hydrologic design estimates under predicts the FFA derived estimates of design hydrology by 50-10%. This is partly due to the adoption of ARR2016 initial losses but is likely to be due to the adoption of a 24 hour storm duration which is critical for the Hunter River but is not critical for the Goulburn River which is likely to be closer to 48 hours. Also when looking at flows from both catchments a larger aerial reduction factor (ARF) is required. The estimates of total flow also need to consider the likelihood of peaks from both the Goulburn River and the Hunter River coinciding which is why the design estimates of the combined flows are not the simple addition of the two catchments.

While there is a difference between the hydrologic estimates of Goulburn River inflows and FFA estimates, because the check on combined design flows at the Liddell Gauge (**Section 4.4**) show that there is good total agreement between hydrogical and FFA design flows, the adopted design hydrology is appropriate for this study. A comparison of FFA design estimates of Sandy Hollow flows (**Table 4-9**) to Liddell flows (**Table 4-7**) of Goulburn River to combine flows shows that combined flows are always greater than Goulburn River only flows and there is no risk we have underestimated the design hydrology.

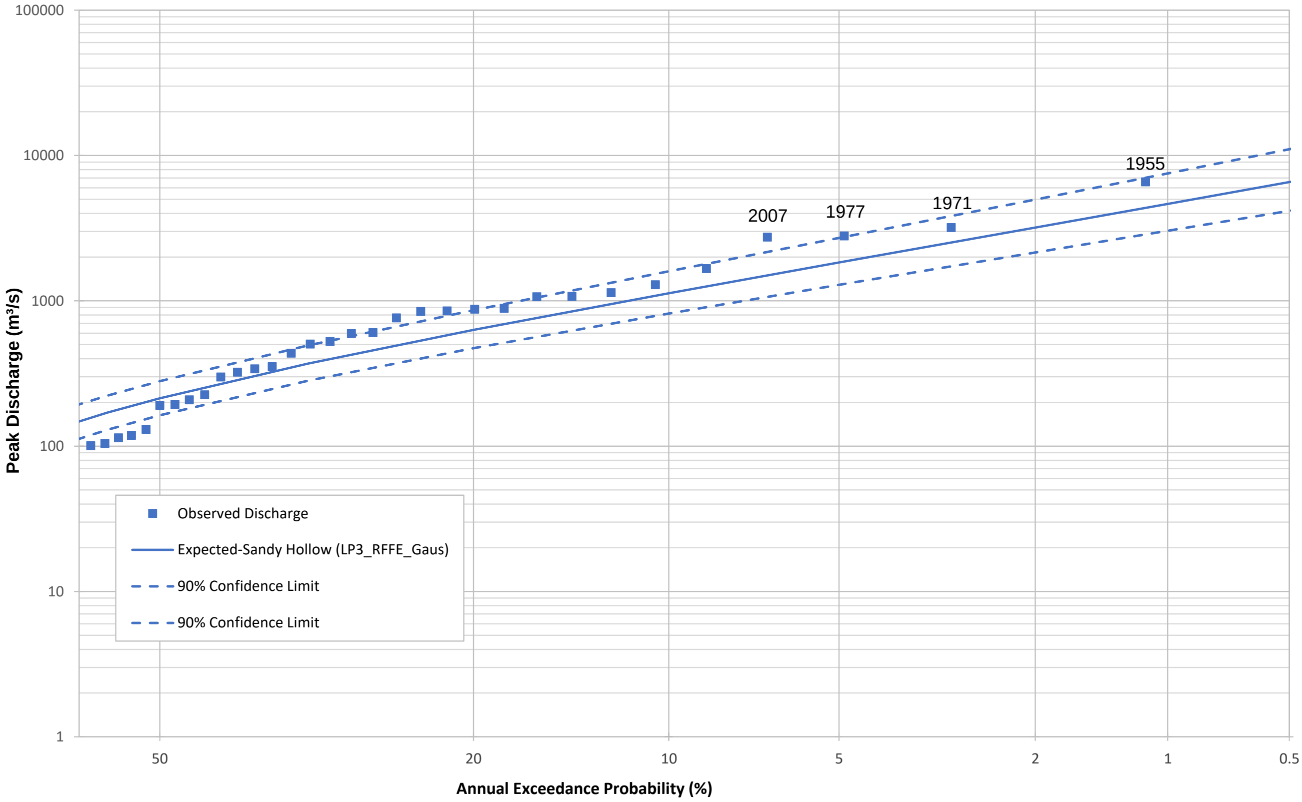

Figure 4-7*: Sandy Hollow Flood Frequency Analysis (Bayesian with prior RFFE information) and Hydrologic Model Flows*

***Table 4-9**: Flood Frequency Analysis and Hydrologic Design Flows at the Sandy Hollow Gauge*

Event (AEP)	Flow (m^3/s)	90% Confidence Limits		Hydrologic Model Flows (m^3/s)
		Lower Flow (m^3/s)	Upper Flow (m^3/s)	
10%	**1129**	818	1598	**581**
5%	**1836**	1290	2707	**1060**
2%	**3198**	2149	4980	**2125**
1%	**4651**	3031	7534	**2743**

5 Existing Conditions Design Event Results

Existing condition design flood events were simulated for the 10% AEP (1 in 10 yr ARI) 1% AEP (1 in 100 yr ARI) magnitude design floods using the hydraulic (TUFLOW) model described in Section 3 and the hydrology described in Section 4.

Maps of existing condition flood level and depth are presented in **Figure 5-1** for the 10% AEP event and **Figure 5-2** for the 1% AEP event.

Maps of existing condition flood velocities are presented in **Figure 5-3** for the 10% AEP event and **Figure 5-4** for the 1% AEP event.

Maps of existing condition flood hazard are presented in **Figure 5-5** for the 10% AEP event and **Figure 5-6** for the 1% AEP event.

A map of the floodplain function / hydraulic classification in the 1% AEP event is provided in **Figure 5-7**. The map was produced using the same method used in the Muswellbrook FRMS&P (RHDHV, 2019).

From the figures we can see:

- in the 10% AEP (1 in 10 yr ARI) design event, the quarry is flooded to a level of 95.68 m AHD from tailwater flooding. In the 10% AEP event, flooding is largely within the Hunter River banks.
- in the 1% AEP (1 in 100 yr ARI) design event, the quarry is flooded to a depth of 98.87 m AHD largely from tailwater flooding, though a flood-runner on the northern section of the Hunter River floodplain discharges directly into the north-western bound of the quarry and the flood bund to the south of the quarry is also overtopped. A section of the western extent of the quarry is flood free, though becomes an island with no evacuation routes and may be inundated in larger events.
- In both events, flood velocity is generally 0.5 m/s or less though some regions experience higher flood velocity above 2 m/s.
- In both the 10 and 1% AEP flood hazard would be > H5 or H6 so would be unsafe for people and vehicles and structures would require specialist engineering.
- The hydraulic classification for 1% AEP event presented in **Figure 5-7** shows the area is mostly classified as flood storage, though in areas where high velocities flows exist it may be considered a floodway.

5.1 Flood Considerations for Existing Quarry Operations

Because the quarry can be flooded in relatively minor events such as the 10% AEP, it is recommended that quarry operations are only undertaken during flood free conditions and that equipment is moved to higher ground if flood warnings are given for this stretch of the Hunter River. However, while flood warnings are provided at Muswellbrook and Denman, no flood warning appears to be available for the Goulburn River (Sandy Hollow gauge), so any increases in flood levels at this gauge should be treated with caution.

The movement of plant equipment (including any fuel stores) to higher ground in the event of flood should help minimise any potential negative impact on water quality of the quarry operation.

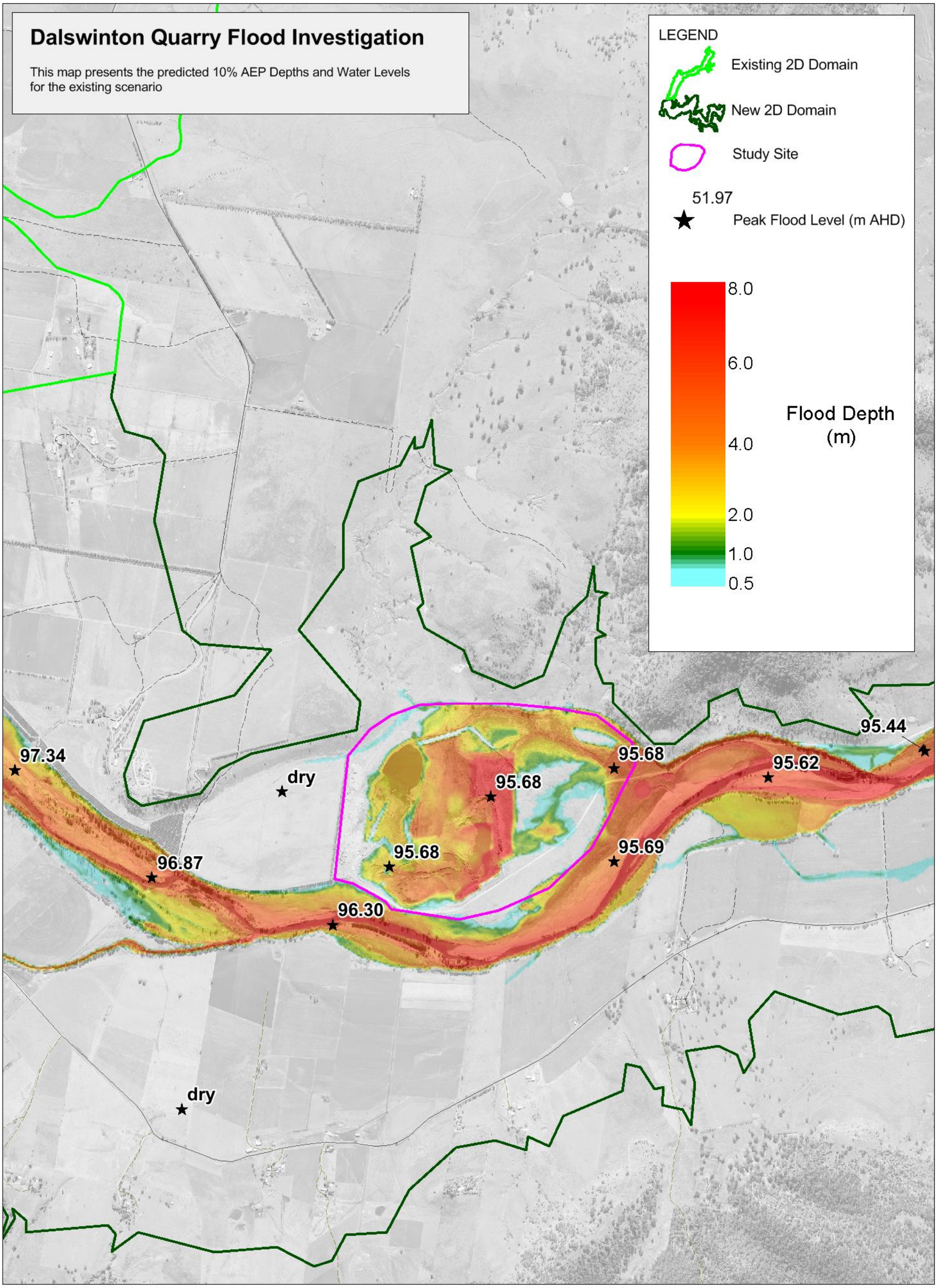

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FIGURE 5-1

10% AEP PEAK FLOOD DEPTH
EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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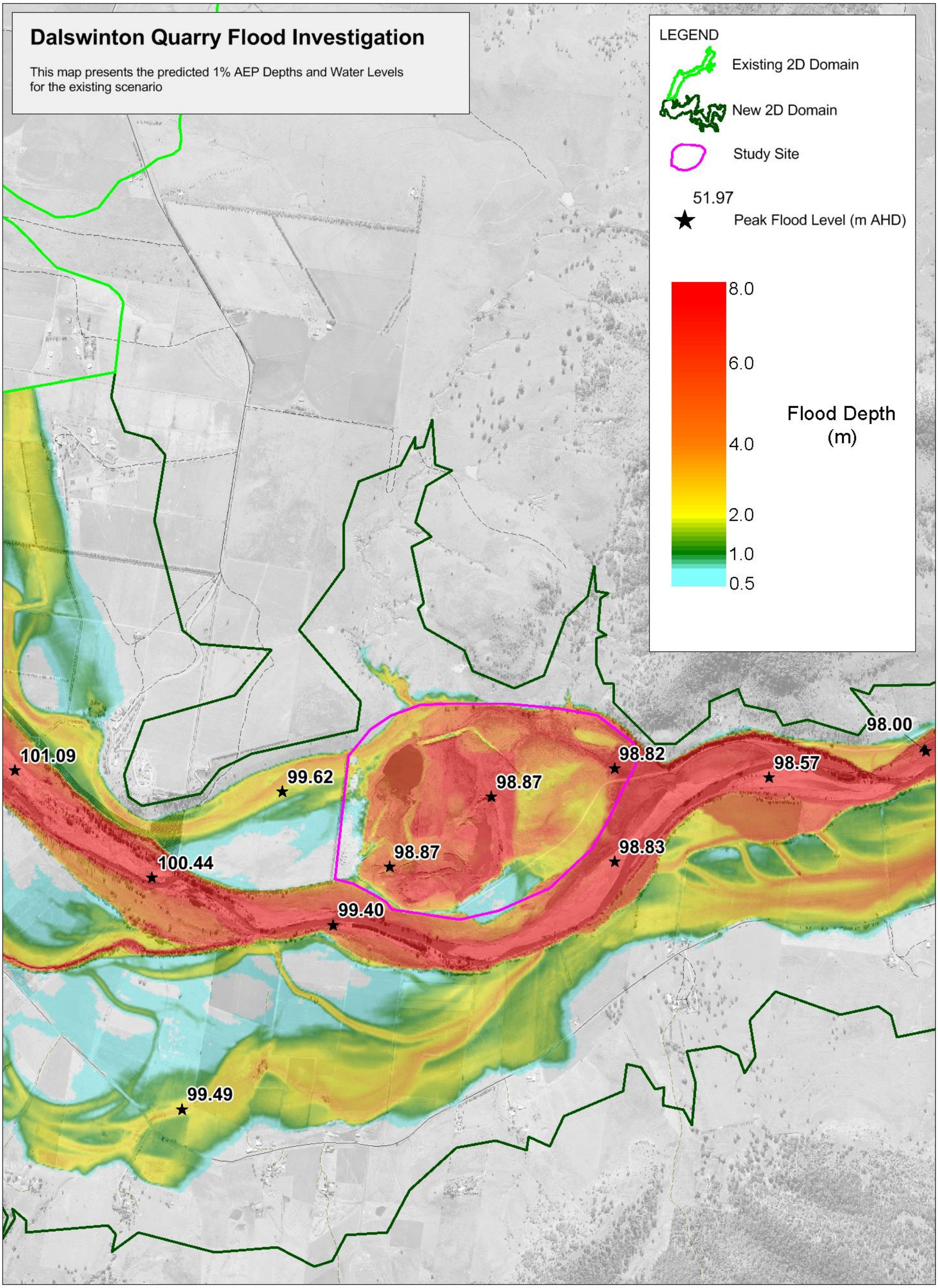

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FIGURE 5-2

1% AEP PEAK FLOOD DEPTH EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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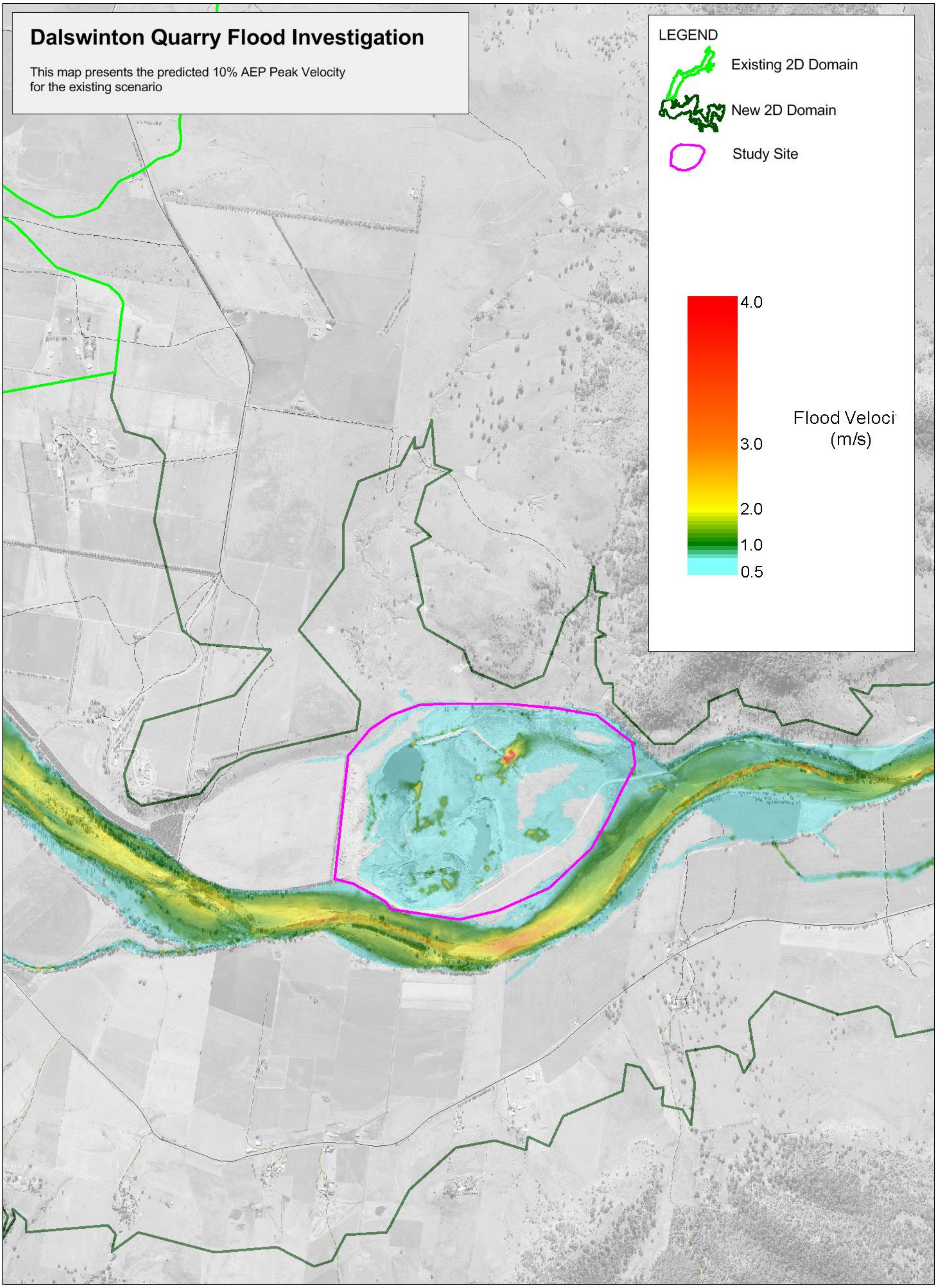

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FIGURE 5-3

10% AEP PEAK FLOOD VELOCITY
EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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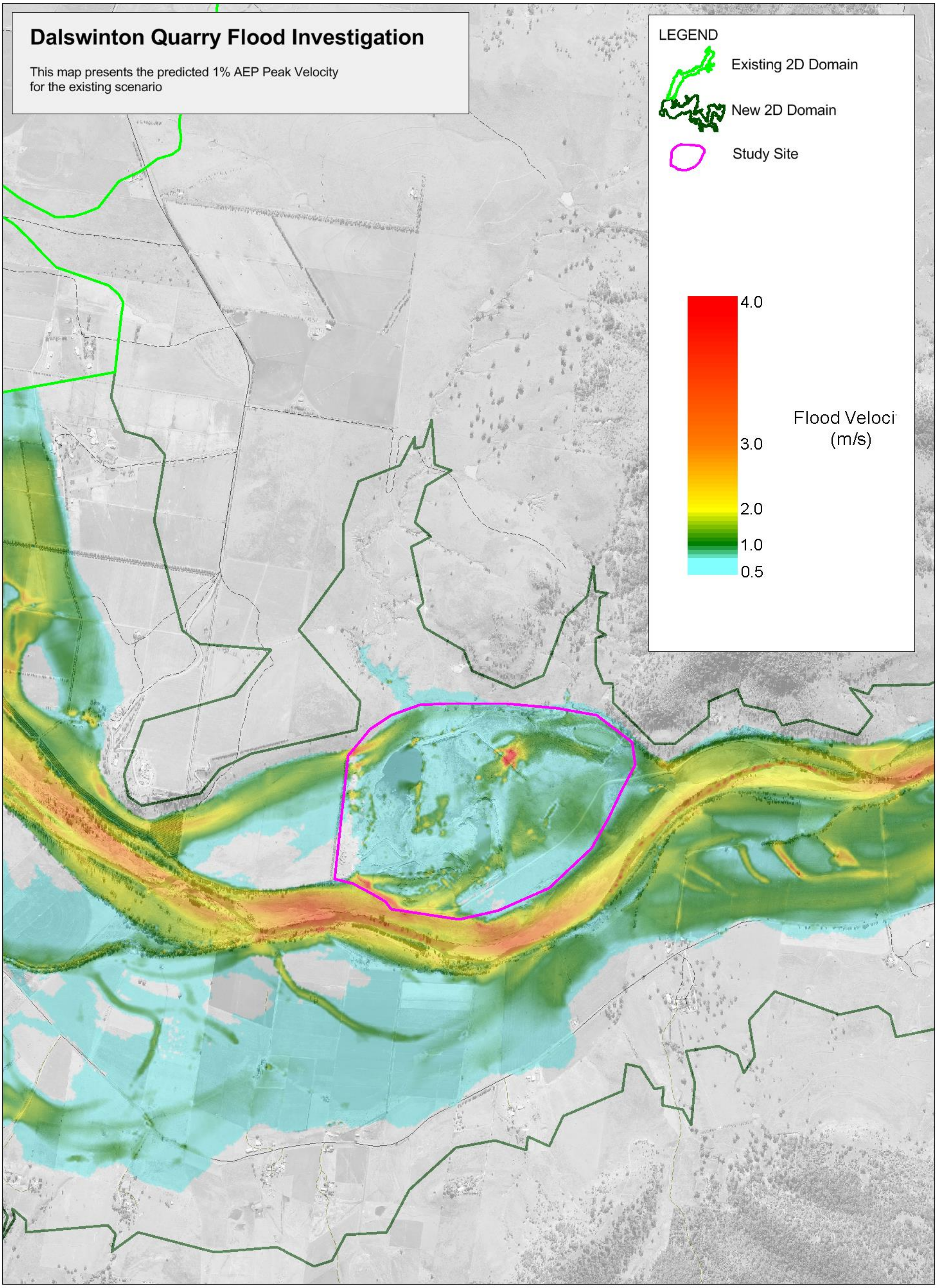

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FIGURE 5-4

1% AEP PEAK FLOOD VELOCITY EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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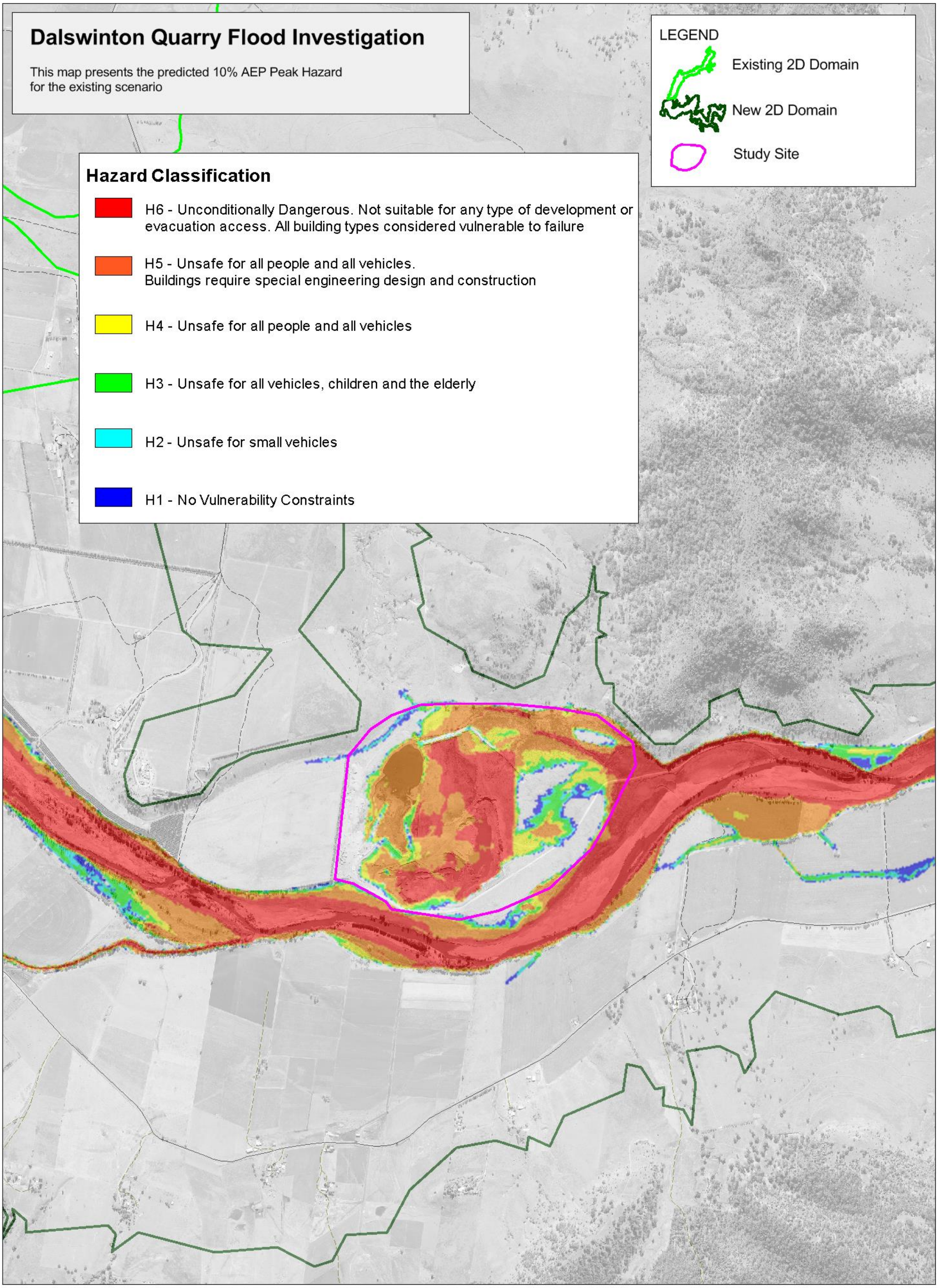

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FIGURE 5-5

10% AEP PEAK FLOOD HAZARD
EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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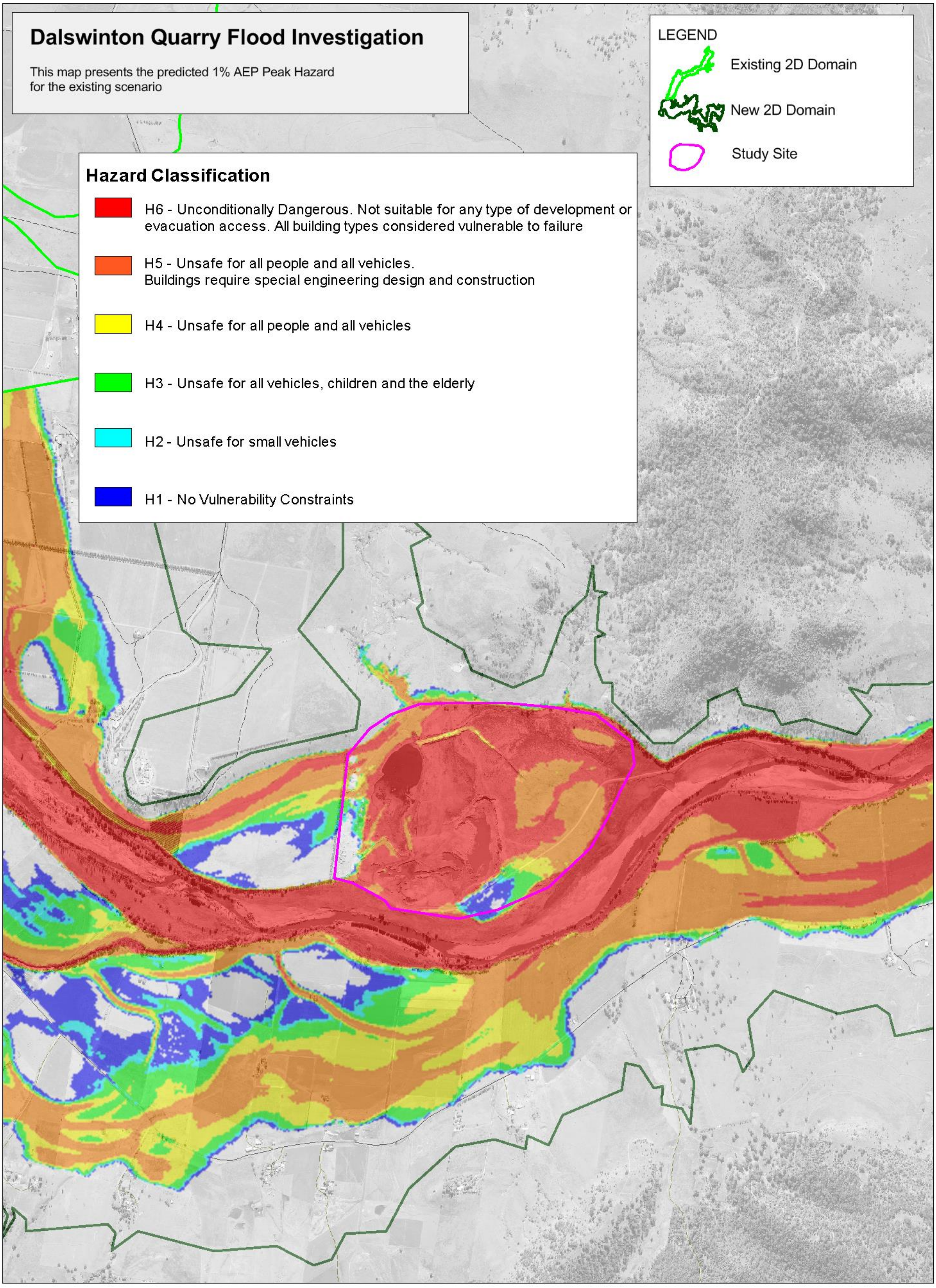

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FIGURE 5-6

1% AEP PEAK FLOOD HAZARD
EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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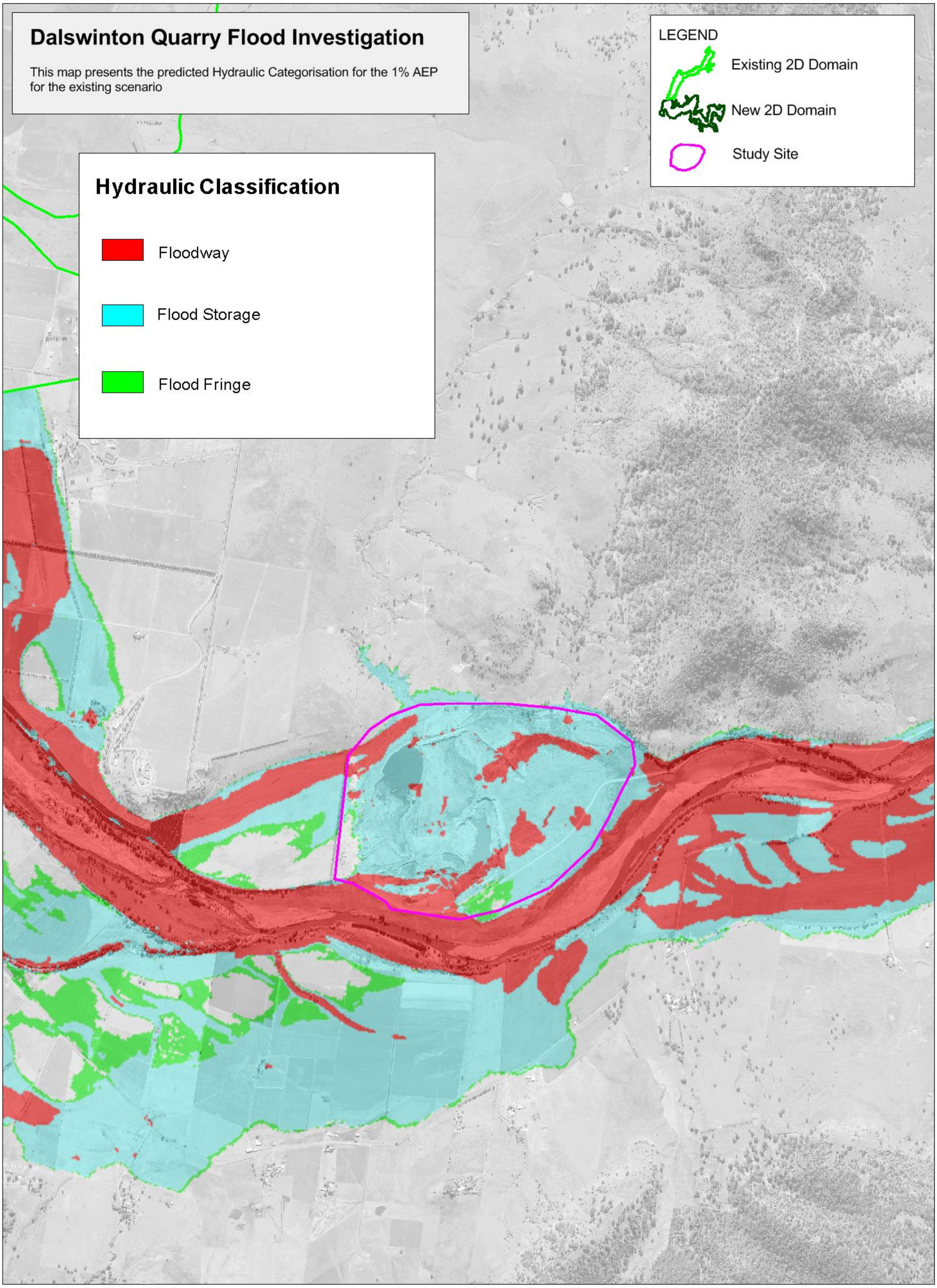

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FIGURE 5-7

1% AEP HYDRAULIC CLASSIFICATION EXISTING CONDITIONS

NORTH

0 250 500m
Approx. Scale

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6 Proposed Conditions and Flood Impact

6.1 Change in Ground Elevations and Storage Volume

Flood conditions for the proposed (developed) scenario are based on updating the flood model to include ground elevation data representative of the final quarry landform which is presented in **Figure 6-1**. The proposed final landform includes a gently sloping floor to the quarry that ranges from 92.5 m AHD down to 89.0 m AHD, though some lower areas of 88.0 m AHD are present to the east of the site.

The proposed changes to ground surface elevation can be observed by comparing **Figure 6-1** to **Figure 6-2** which shows the ground levels of the existing quarry (based on 2017 LiDAR data). The changes in ground elevation are further detailed in **Figure 6-3** (Section A-A) and **Figure 6-4** (Section B-B) which provide long section plots showing existing (red line) and developed (green line) ground surface elevations. Long Section A-A (**Figure 6-3**) shows the removal of a stockpile at chainage 100m (and refer **Figure 6-2** for location).

GIS analysis was used to determine the change in landform volume, which shows that the final landform results in a net export of 2.1 million m^3 of material (comprising a fill of 460,482 m^3 and cut of -2,567,384 m^3). This is equivalent to an area weighted reduction in ground surface elevation of 1.7m, which is now potentially available as flood storage.

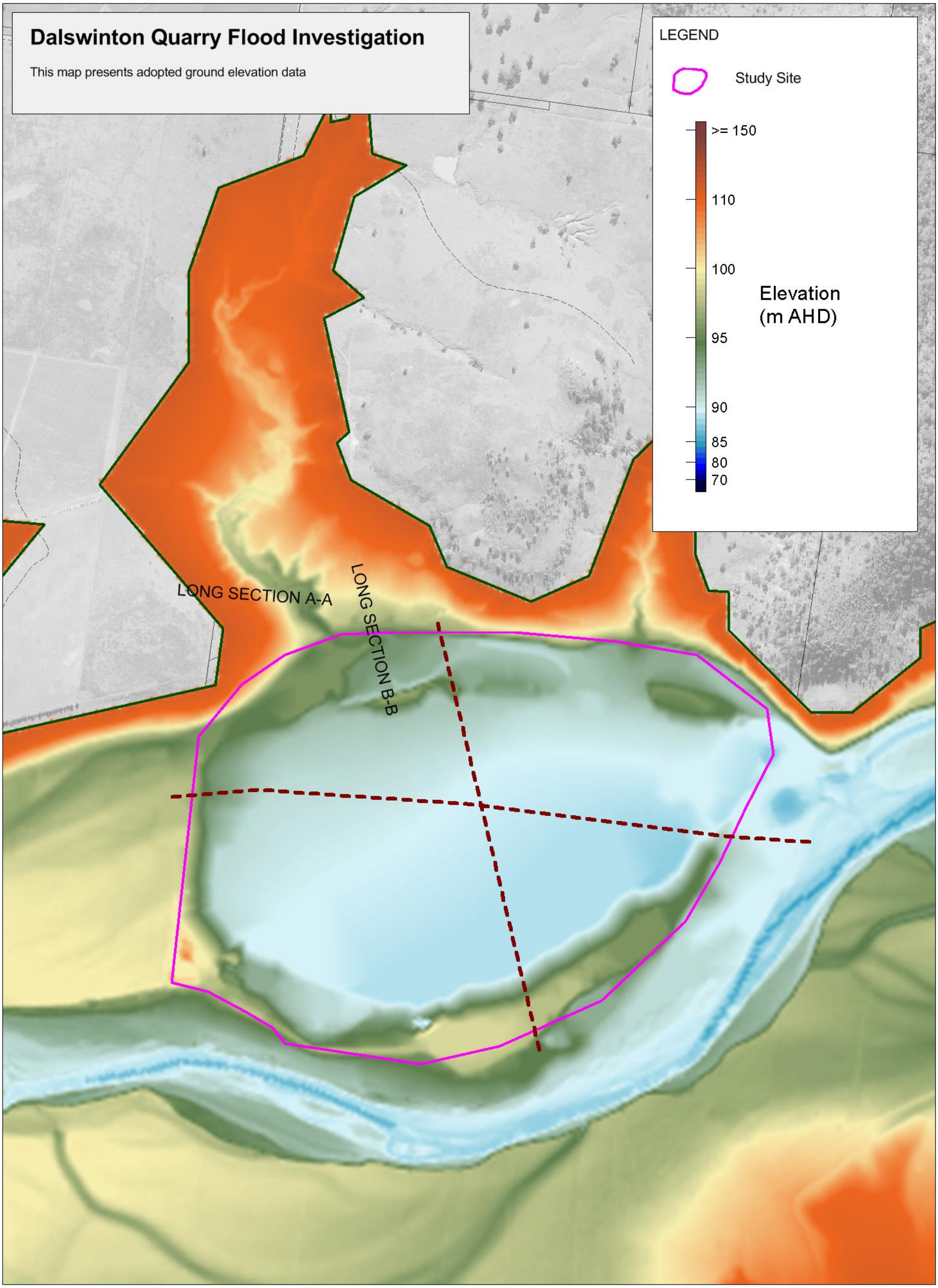

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FIGURE 6-1

GROUND ELEVATION DATA
DEVELOPED CONDITIONS

NORTH

0 125 250m
Approx. Scale

SCALE 1:12,500

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Dalswinton Quarry Flood Investigation

This map presents adopted ground elevation data

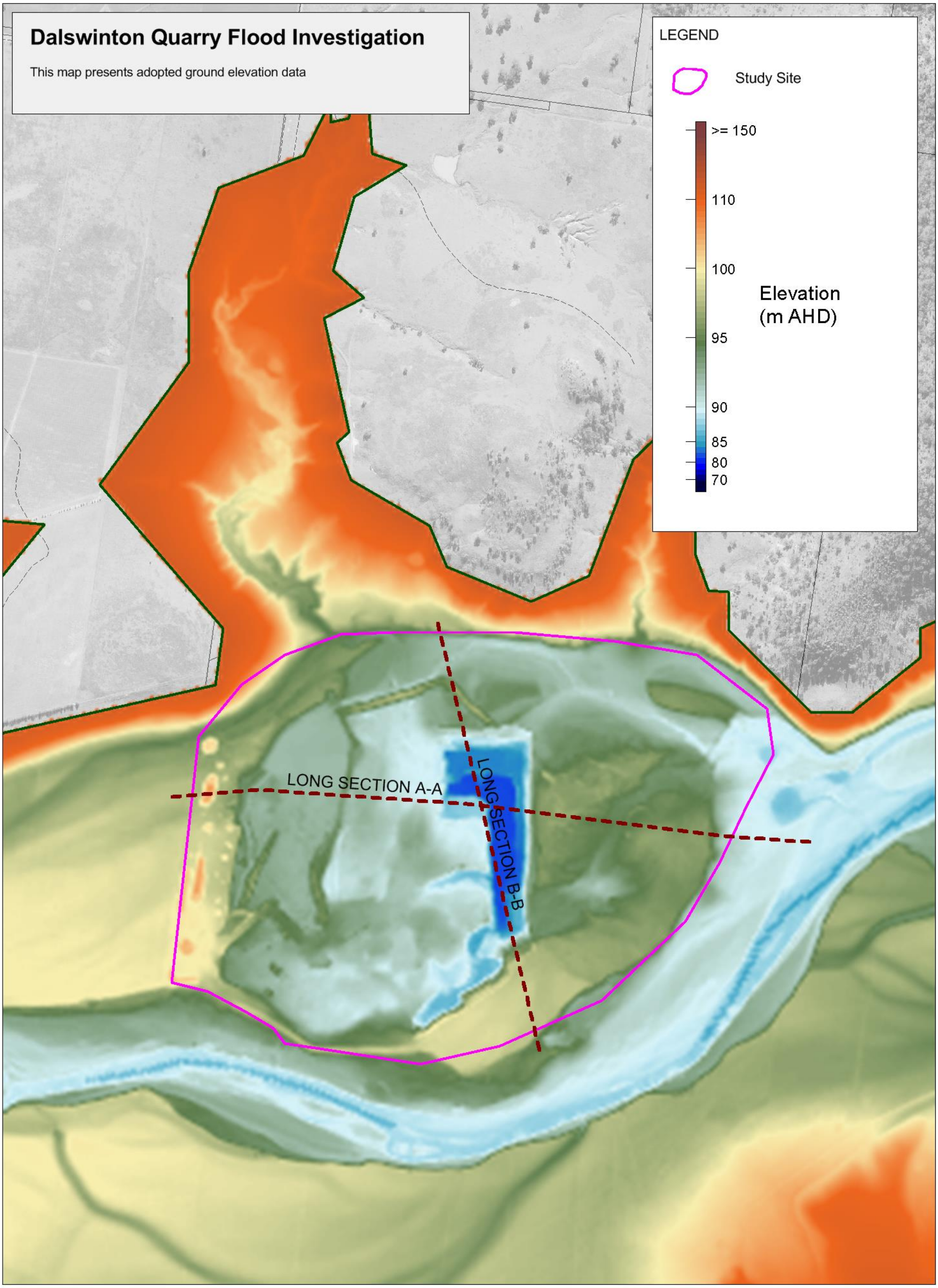

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FIGURE 6-2

GROUND ELEVATION DATA
EXISTING CONDITIONS

SCALE 1:12,500

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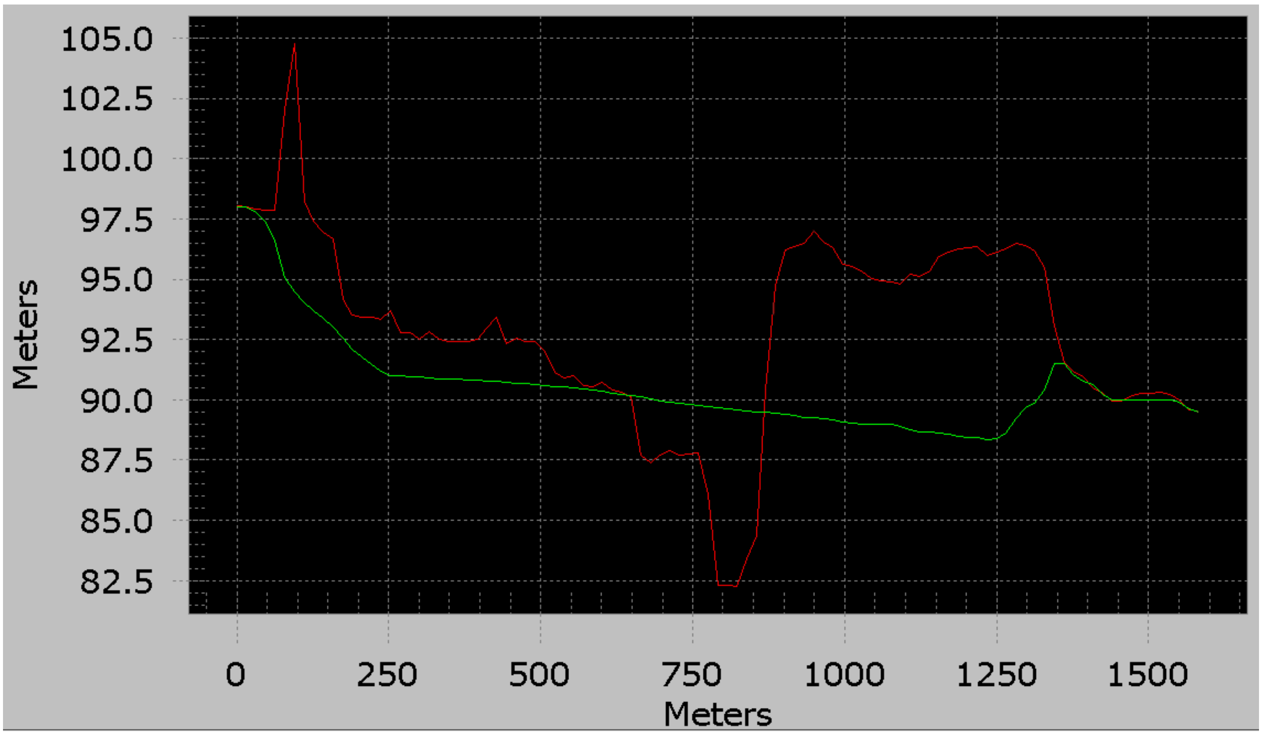

***Figure 6-3**: Long Section A-A (Existing (Red) & Developed (Green) Ground Elevations)*

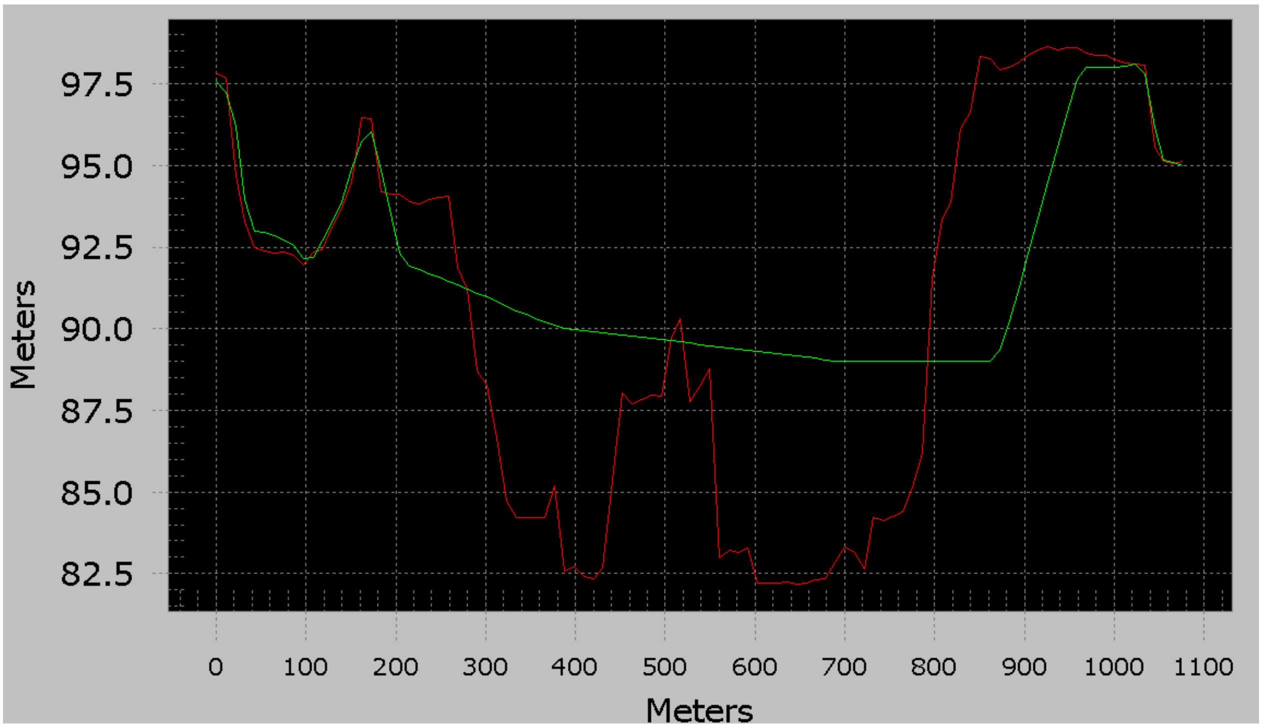

***Figure 6-4**: Long Section B-B (Existing (Red) & Developed (Green) Ground Elevations)*

6.2 Developed Conditions Flood Behaviour

Developed (final landform) condition design flood events were simulated for the 10% AEP (1 in 10 yr ARI) 1% AEP (1 in 100 yr ARI) magnitude design floods using the hydraulic (TUFLOW) model described in Section 3 and the hydrology described in Section 4.

Maps of developed condition flood level and depth are presented in **Figure 6-5** for the 10% AEP event and **Figure 6-6** for the 1% AEP event.

Figure 6-7 presents the predicted flood impact (i.e. change in peak water level) associated with the final quarry landform.

Maps of developed condition flood velocities are presented in **Figure 6-8** for the 10% AEP event and **Figure 6-9** for the 1% AEP event.

Maps of developed condition flood hazard are presented in **Figure 6-10** for the 10% AEP event and **Figure 6-11** for the 1% AEP event.

From the figures we can see:

- in the 10% AEP (1 in 10 yr ARI) design event, the quarry is flooded to a level of 95.68 m AHD from tailwater flooding. In the 10% AEP event, flooding is largely within the Hunter River banks. While flood levels are the same within the quarry, the additional flood storage has dropped peak flood levels at a number of locations along the river by between 1 and 3cm.
- in the 1% AEP (1 in 100 yr ARI) design event, the quarry is flooded to a depth of 98.83 m AHD largely from tailwater flooding, though a flood-runner on the northern section of the Hunter River floodplain discharges directly into the north-western bound of the quarry and the flood bund to the south of the quarry is also overtopped. The removal of a number of stockpiles has increased the peak flow of the northern flood runner.
- In both events, flood velocities are generally 0.5 m/s or less though some regions experience higher velocities above 2 m/s. The flattening of the floor of the quarry has changed the velocity distribution compared to the existing case.
- In both the 10 and 1% AEP flood hazard would be > H5 or H6 so would be unsafe for people and vehicles and structures would require specialist engineering.

6.3 Change in Peak Flood Levels due to Final Quarry Form

Figure 6-7 presents the predicted flood impact (i.e. change in peak water level) associated with the final quarry landform and shows that there is a general reduction in peak flood levels of between 4 and 9cm upstream of the quarry. The reduced flood levels are a result of the net export of 2.1 million m^3 of quarry material which is now available as flood storage. The removal of the quarry stockpile at the western quarry boundary has also enhanced available flood conveyance for the northern flood runner which produces a local reduction in peak flood level of above 0.4m. While there are a few small areas within the quarry where peak flood levels may increase by 1-2cm, downstream changes in flood levels are less than 1cm.

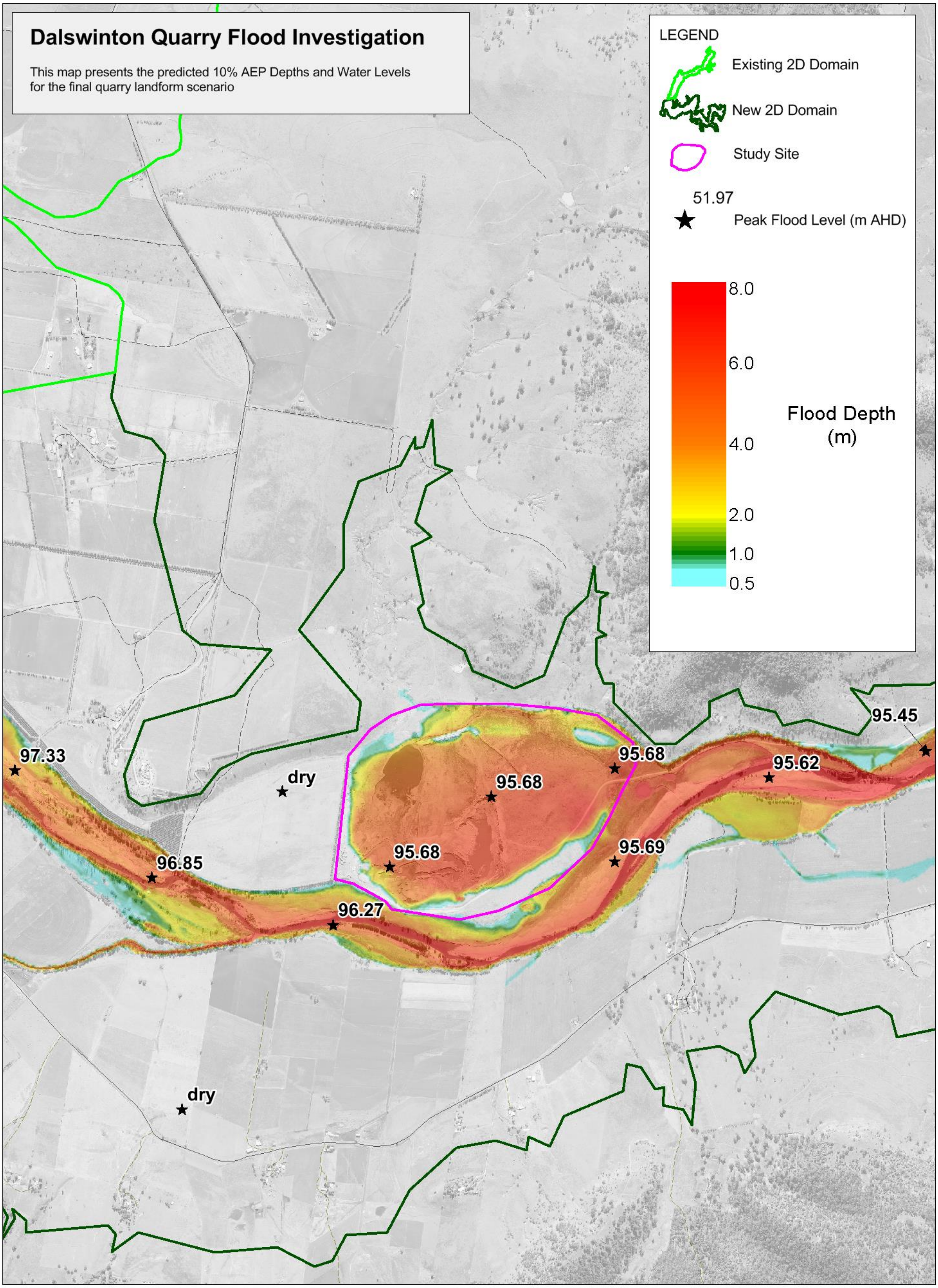

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FIGURE 6-5

10% AEP PEAK FLOOD DEPTH
DEVELOPED CONDITIONS

NORTH
0 250 500m
Approx. Scale

SCALE 1:25,000
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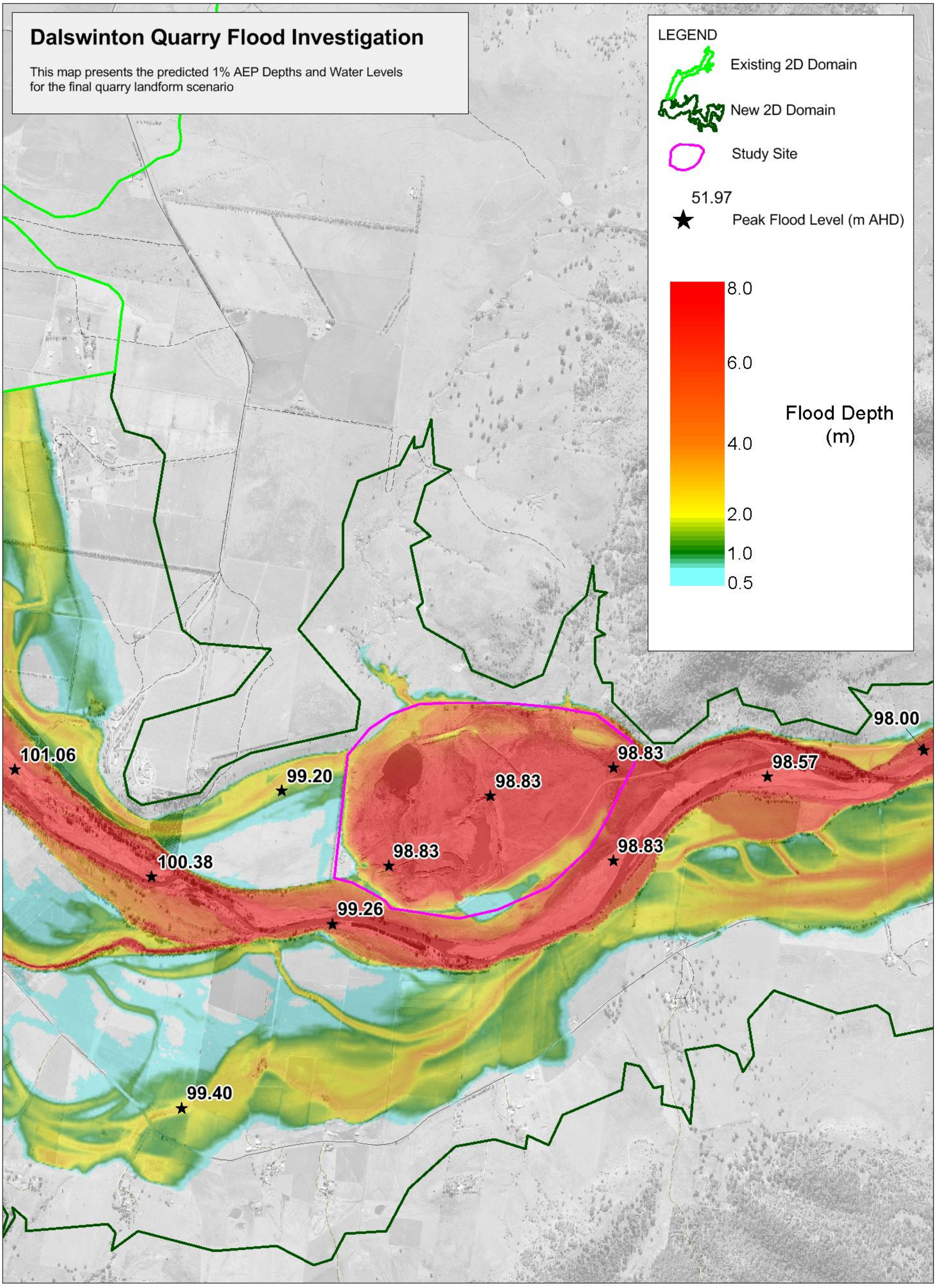

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FIGURE 6-6

1% AEP PEAK FLOOD DEPTH
DEVELOPED CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

PAGE SIZE A4

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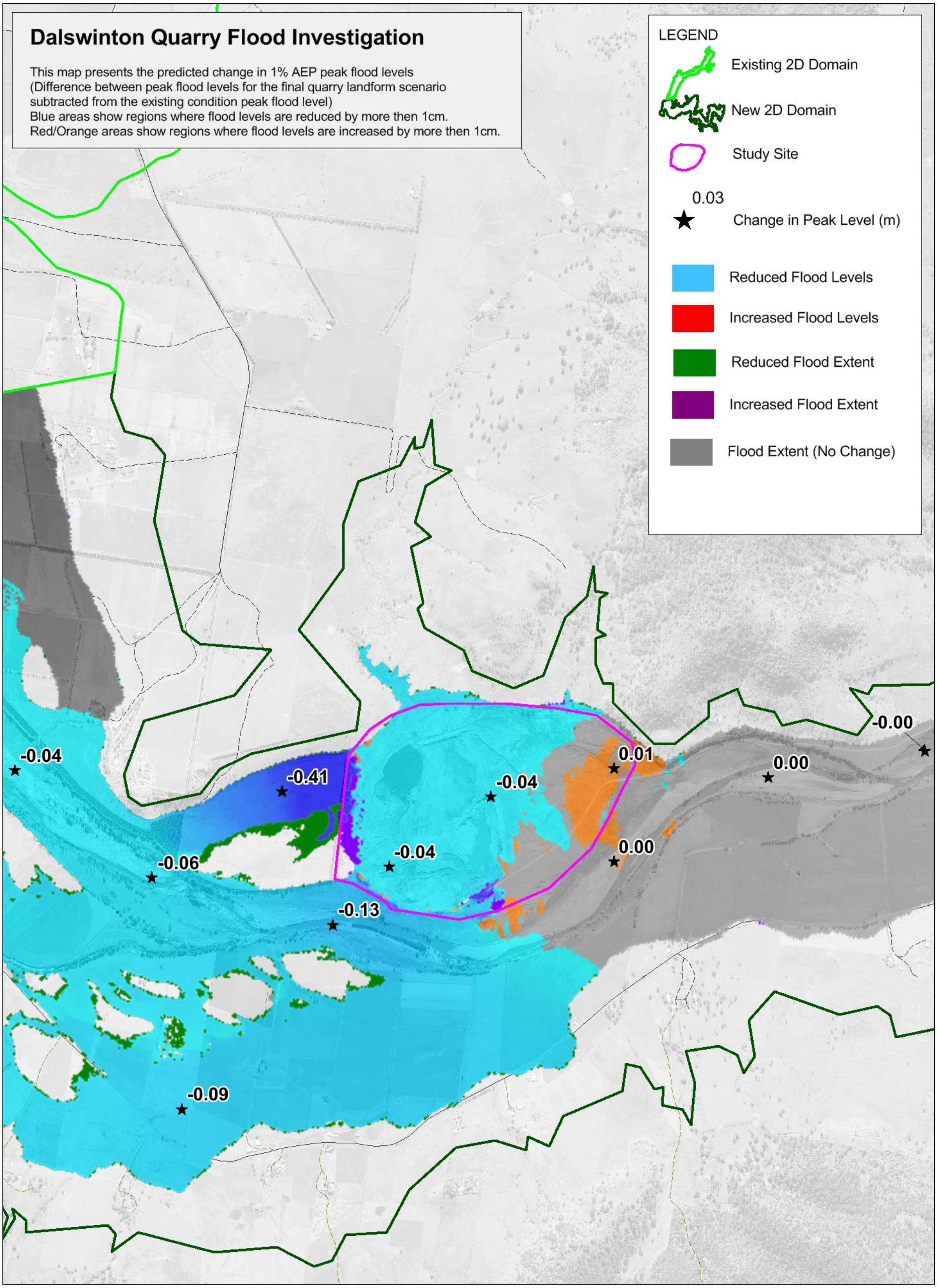

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FIGURE 6-7

1% AEP FLOOD IMPACT (FLOOD LEVEL DIFFERENCE)

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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Dalswinton Quarry Flood Investigation

This map presents the predicted 10% AEP Peak Velocity
for the final quarry lanform scenario

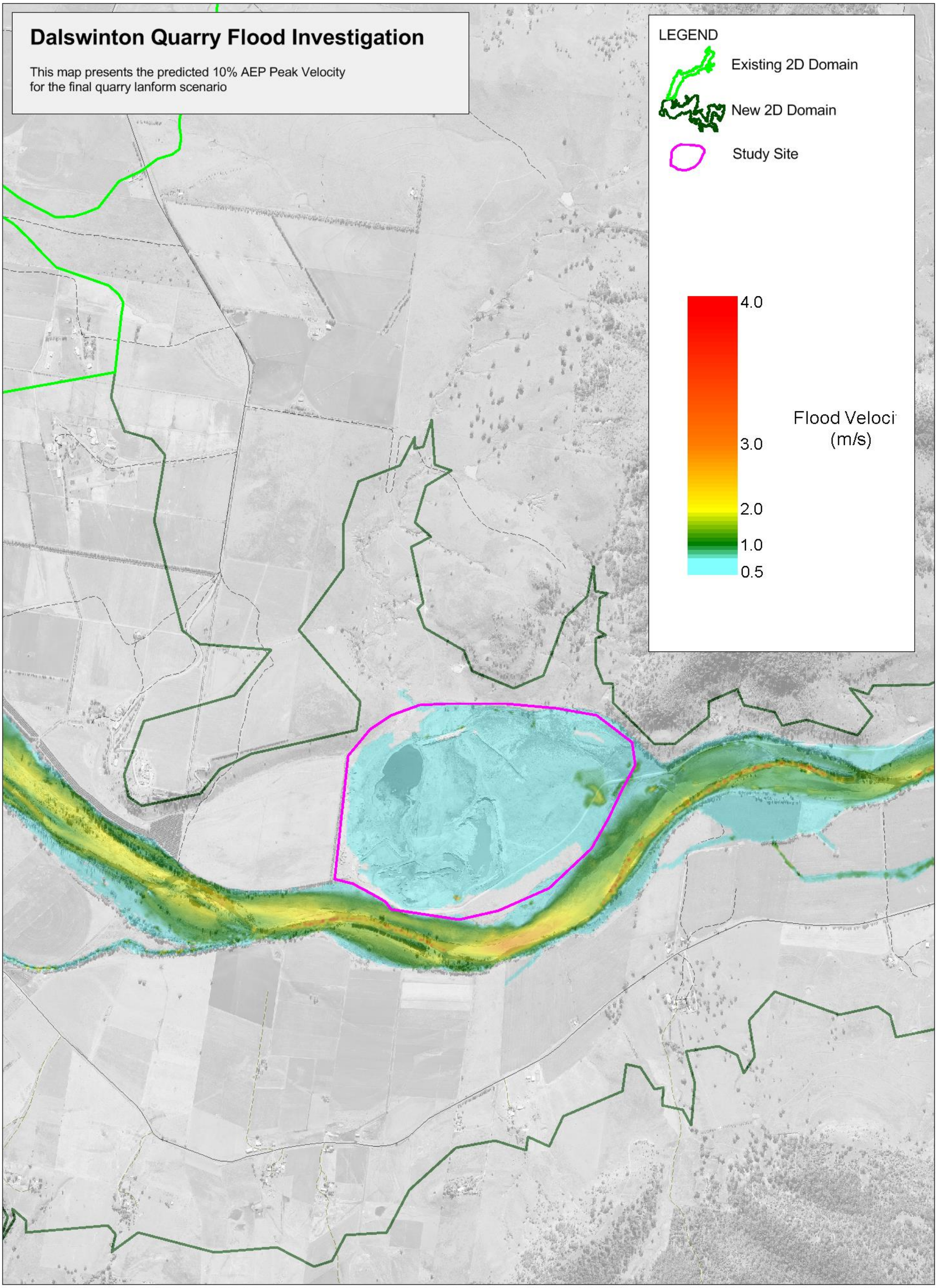

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FIGURE 6-8

10% AEP PEAK FLOOD VELOCITY
DEVELOPED CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

PAGE SIZE A4

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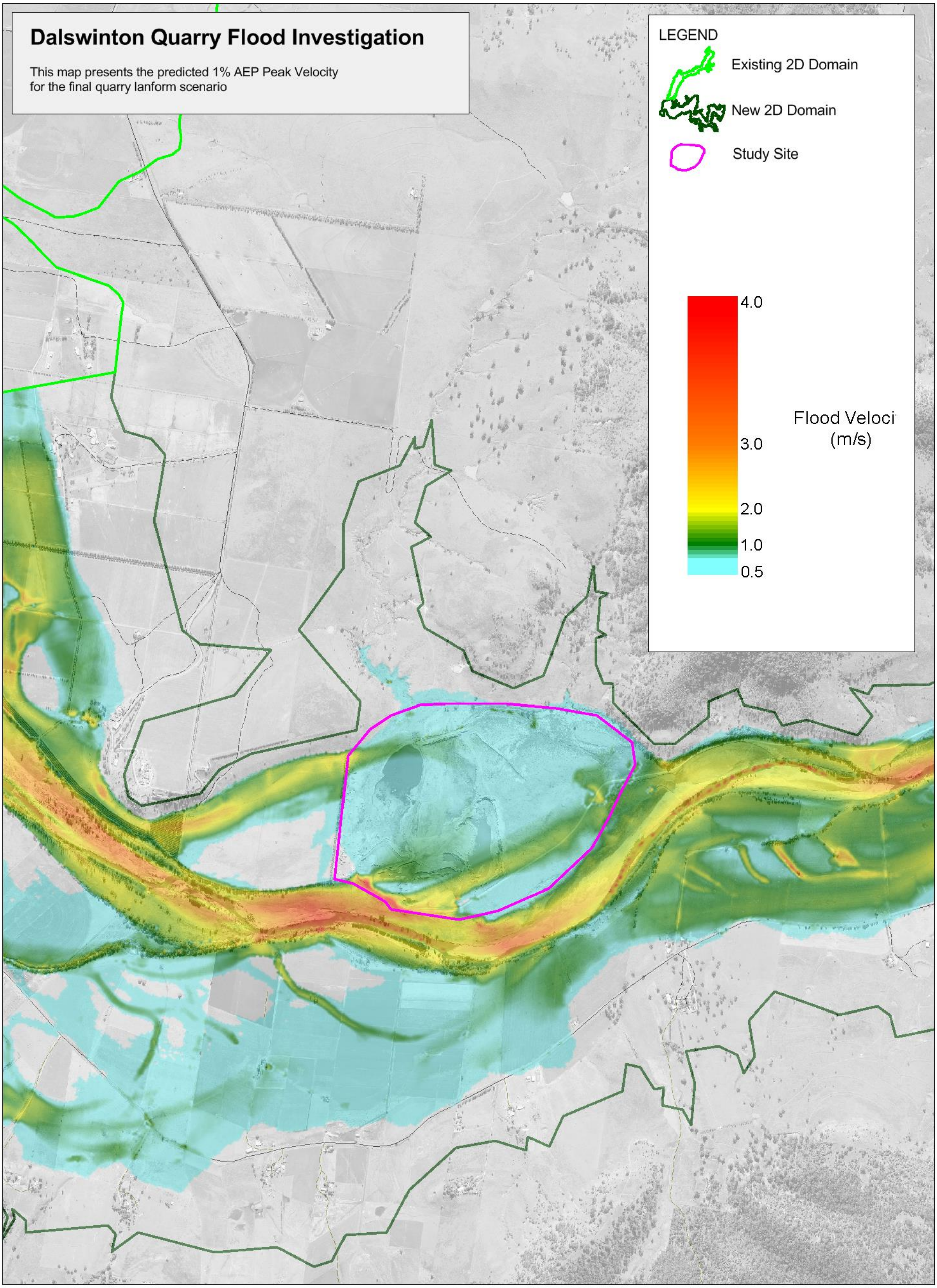

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FIGURE 6-9

1% AEP PEAK FLOOD VELOCITY DEVELOPED CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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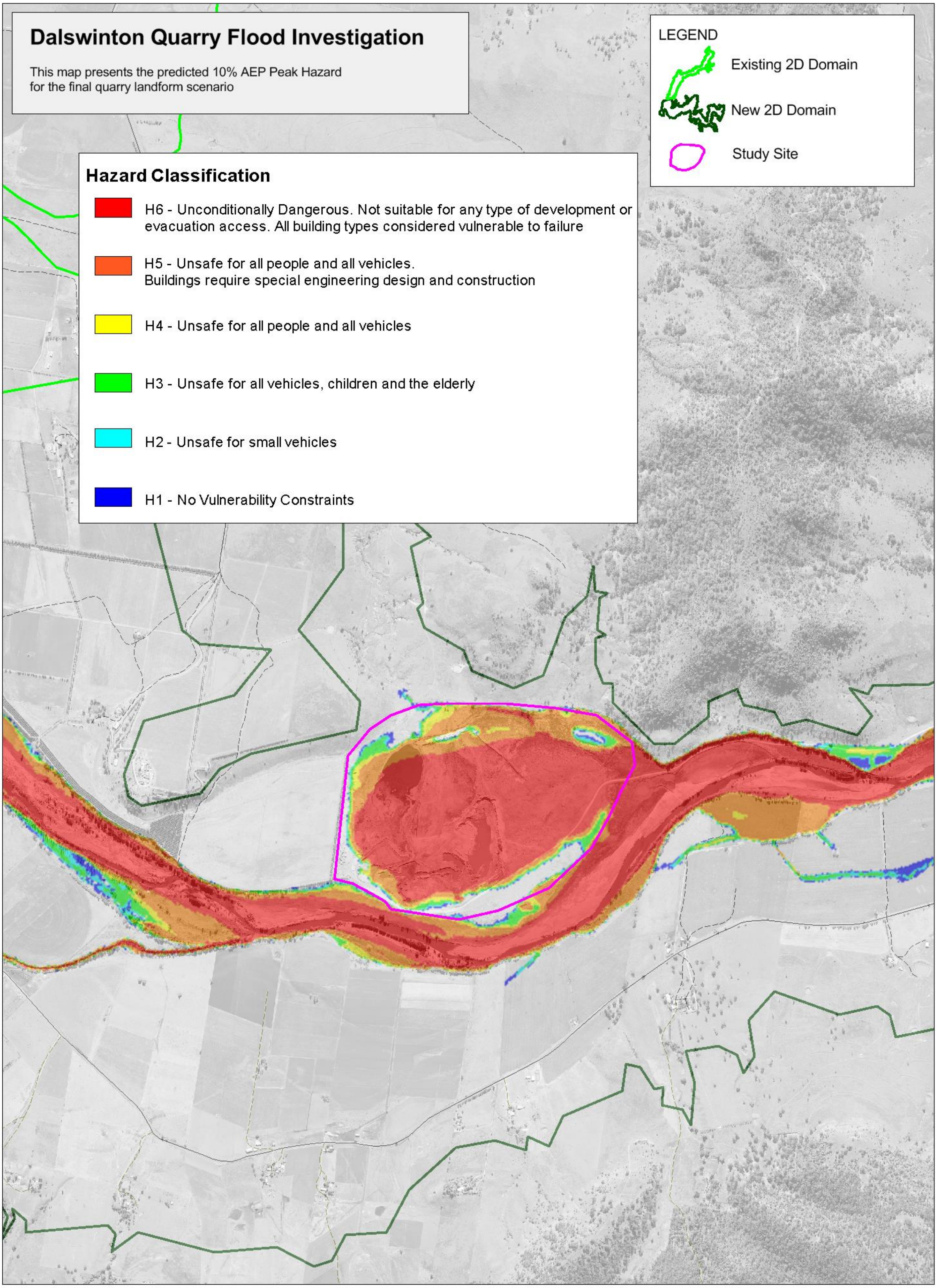

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FIGURE 6-10

10% AEP PEAK FLOOD HAZARD
DEVELOPED CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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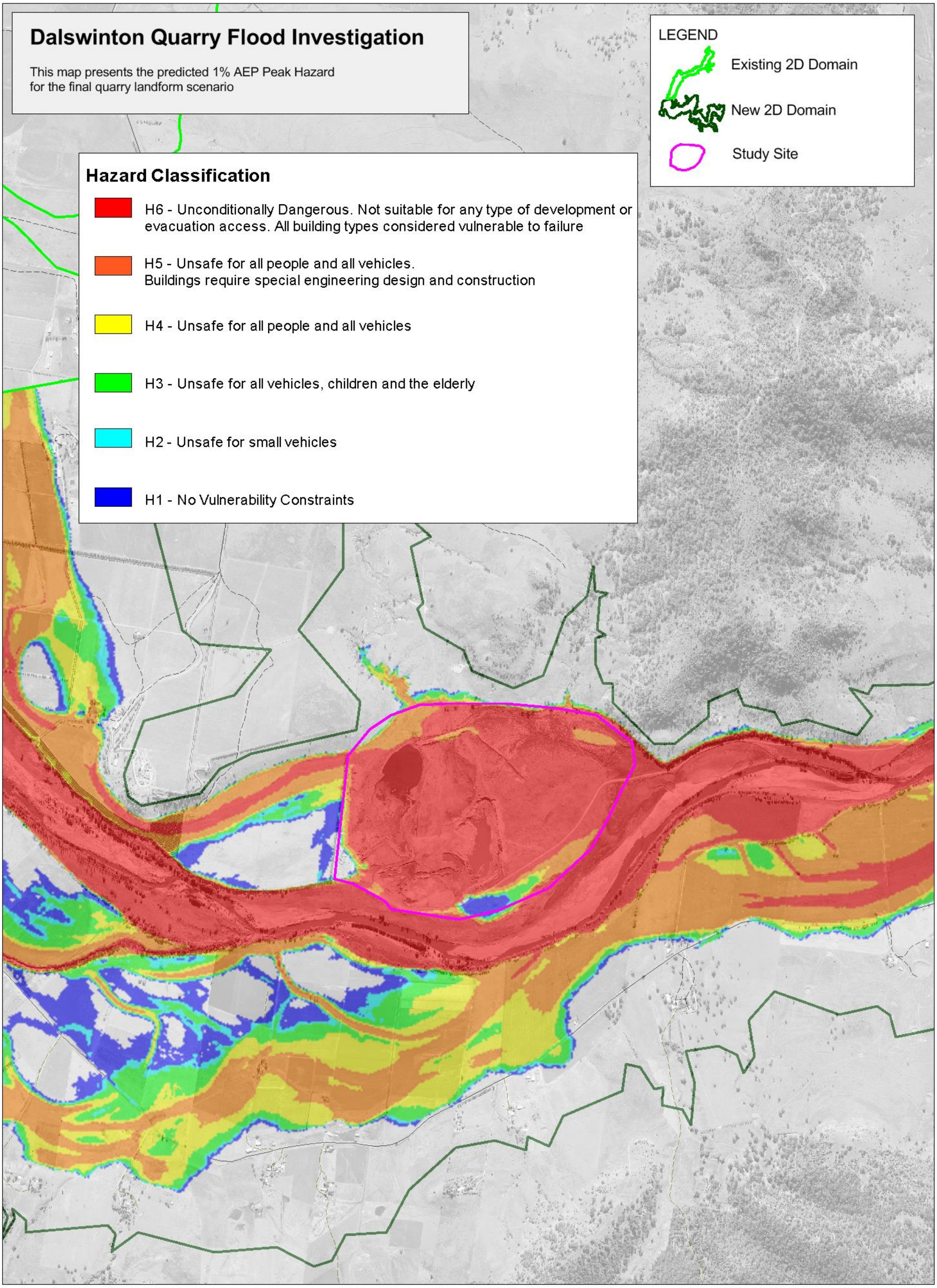

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FIGURE 6-11

1% AEP PEAK FLOOD HAZARD
DEVELOPED CONDITIONS

NORTH

0 250 500m
Approx. Scale

SCALE 1:25,000

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6.4 Erosion Risk and Potential Morphologic Change

During events significantly larger than the 10% AEP (1 in 10 year ARI) it would be possible for flood waters to overtop the narrow embankment that runs for approximately 500m along the south-western boundary of the quarry adjacent to the Hunter River (as presented in **Figure 6-12**). The crest of the embankment is typically between 97 and 98 m AHD, with a crest width of approximately 10m.

Given that peak 1% AEP velocities overtopping the bund are above 3 m/s, it is likely that, unless the bund is protected by adequately sized armour rock, (assuming the material is unconsolidated alluvial sand and gravel) it could be eroded during a large flood event.

During the 1% AEP events the peak upstream flood level is 99.4 m AHD and the downstream flood level is 98.87 m AHD. Because there is a relatively small water level difference across the bund, even if a sudden bund failure this is unlikely to be significantly more hazardous than the predicted existing conditions.

If the bund is breached, given the embankment material is likely to be largely unconsolidated, it could potentially scour to a width of 50 to 200m and may scour down to a depth of 92m AHD. If such a breach is considered unacceptable it would be necessary to either:

- armour the entire crest of the embankment; or
- increase the crest elevation of the bund to between 100.5 and 99.5 m AHD (to protect it in the 1% AEP event). Though the flood impact of such an increase would need to be investigated and the breach could still occur in larger events.

It should also be noted that the chance of bund failure is the same for the existing and developed conditions. If a breach was to occur, quarry plant could be used to reform the eroded landform back to the current condition.

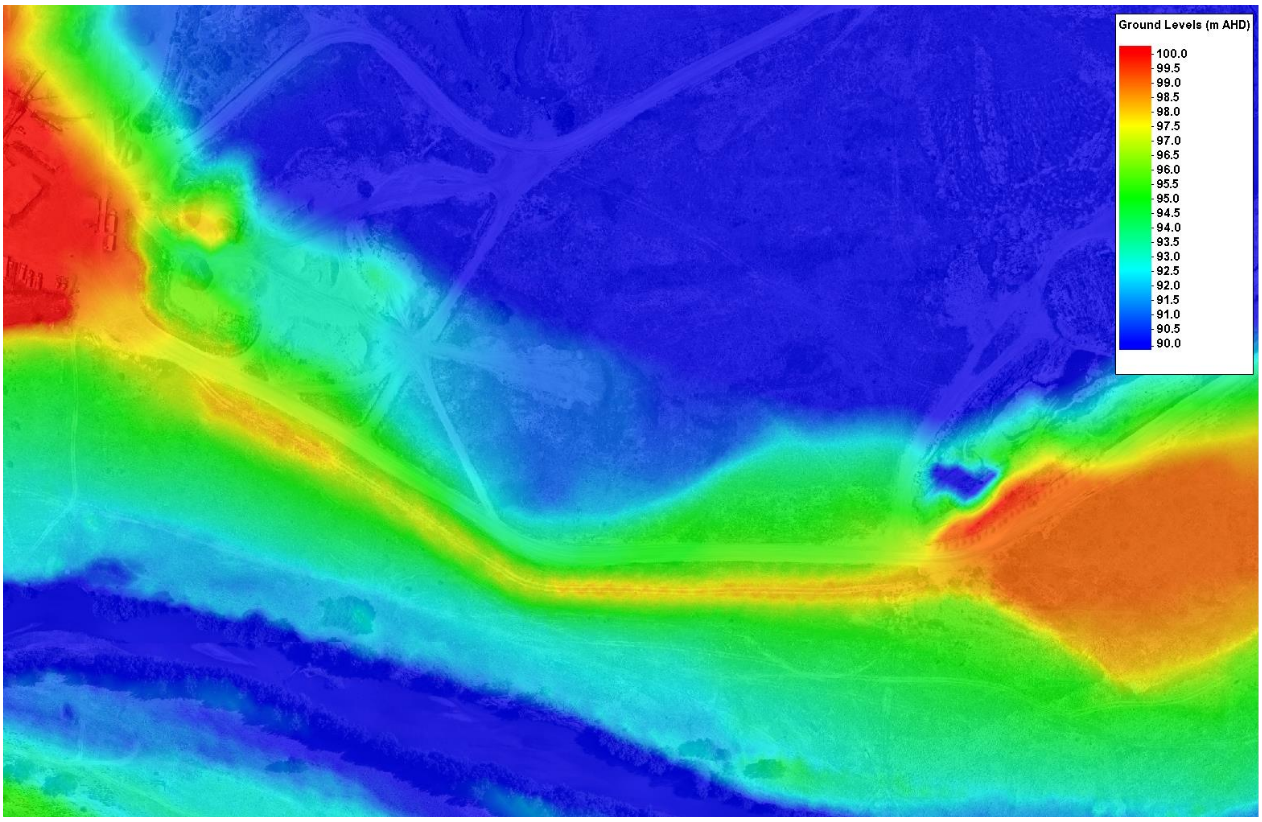

***Figure 6-12**: Detail of Raised Bund between Hunter River and Quarry (Final Landform)*

6.5 Flood Considerations for Future Quarry Operations

Flood considerations for quarry operation for the future development are largely the same as that for the existing quarry operation as presented in Section 5.1.

It is recommended that stockpiles are not placed in any floodways (refer **Figure 5-7**) and that all plant material (especially fuel stores) are stored above the 1% AEP flood level. A quarry operations management plan should also consider flood conditions and include a flood evacuation plan that defines suitable water level triggers on when the quarry should be evacuated.

7 Summary

This report presents a flood impact assessment that quantifies the flood impact associated with the proposed expansion of the Dalswinton Quarry currently operated by Rosebrook Sand and Gravel.

A suitable flood model based on an extension of the Hunter River (Muswellbrook to Denman) FRMS&P flood model was developed. Checks of hydrology were also undertaken including a comparison of design event flows to flood frequency analysis (FFA) for gauges at Sandy Hollow (Goulburn River) and Liddell (Hunter River).

The developed condition is based on the final quarry landform which included a net export of 2.1 million m^3 of quarry material which is now potentially available as flood storage.

The model was used to quantify the existing and developed flood conditions for the 10% AEP (1 in 10yr ARI) and 1% AEP (1 in 100yr ARI) design flood events.

The predicted flood impact (i.e. change in peak water level) associated with the final quarry landform is presented in **Figure 6-7** and shows that there is a general reduction in peak flood levels of between 4 and 9cm upstream of the quarry. While there are a few small areas within the quarry where peak flood levels may increase by 1-2cm, downstream changes in flood levels are less than 1cm.

During events significantly larger than the 10% AEP (1 in 10 year ARI) it would be possible for flood waters to overtop the narrow embankment that runs for approximately 500m along the south-western boundary of the quarry adjacent to the Hunter River. If the bund is breached it could potentially scour to a width of 50 to 200m and may scour down to a depth of 92m AHD. If such a breach is considered unacceptable it would be necessary to either:

- armour the entire crest of the embankment; or
- increase the crest elevation of the bund to between 100.5 and 99.5 m AHD (to protect it in the 1% AEP event). Though the flood impact of such an increase would need to be investigated and the breach could still occur in larger events.

It should also be noted that the chance of bund failure is the same for the existing and developed conditions. If a breach was to occur, quarry plant could be used to reform the eroded landform back to the current condition.

8 References

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