Appendix L

Integrated Water Management Plan

Integrated Water Management Plan

Scenic Rim Agricultural Industrial Precinct

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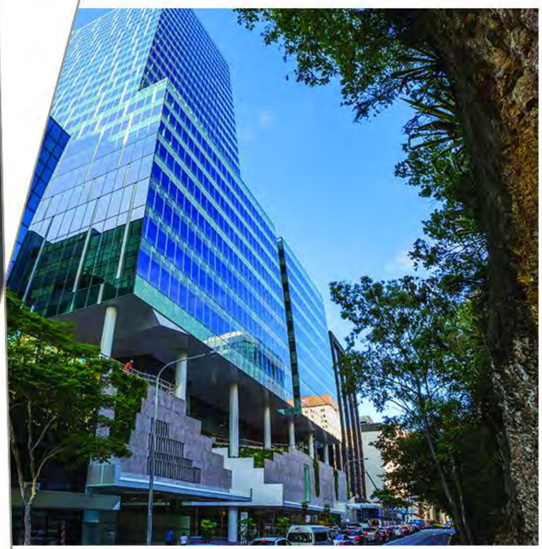

Prepared for

Kalfresh Pty Ltd

18/02/2020

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Document Information

Prepared for	Kalfresh Pty Ltd
Project Name	Scenic Rim Agricultural Industrial Precinct
File Reference	510357_R001_V3_Kalfresh.docx
Job Reference	510357
Date	18/02/2020
Version Number	3

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Effective Date 18/02/2020

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Date Approved 18/02/2020

Document History

Version	Effective Date	Description of Revision	Prepared by	Reviewed by
V001	13/12/2019	Draft	ZM	AP
V002	19/12/2019	Final	ZM	SM
V003	18/02/2020	Updated Layout/Quantity Analysis	ZM	SM

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Appendices

Tables

Figures

1 Introduction

Cardno has been engaged by Kalfresh Pty Ltd to prepare an Integrated Water Management Report to support the Development Application (DA) for the proposed Scenic Rim Agricultural Industrial Precinct. The site is located within the Scenic Rim Regional Council local government area. As described in the *Initial Advice Statement, April 2019*, the development is to occur on the following lots:

- Lot 2 on SP192221;
- Lot 3 on SP192221;
- Lot 4 on SP192221;
- Lot 2 on RP20974;
- Lot 2 on RP44024
- Lot 1 on RP216694; and
- Lot 2 on RP44024.

The proposed site is shown in Figure 1-1.

This report provides engineering advice to address issues relating to stormwater quantity and quality, as well as flooding of the subject site.

Figure 1-1 Site Location

2 Site Description and Proposed Development

2.1 Site Description

The site is located at Kalbar in the Scenic Rim Region alongside the Cunningham Highway approximately 5km North East of Aratula. Generally the site is flat with a slope of approximately 0.5 % to the North West corner. Currently a drainage path cuts through the development area from South to North as shown in Figure 2-1.

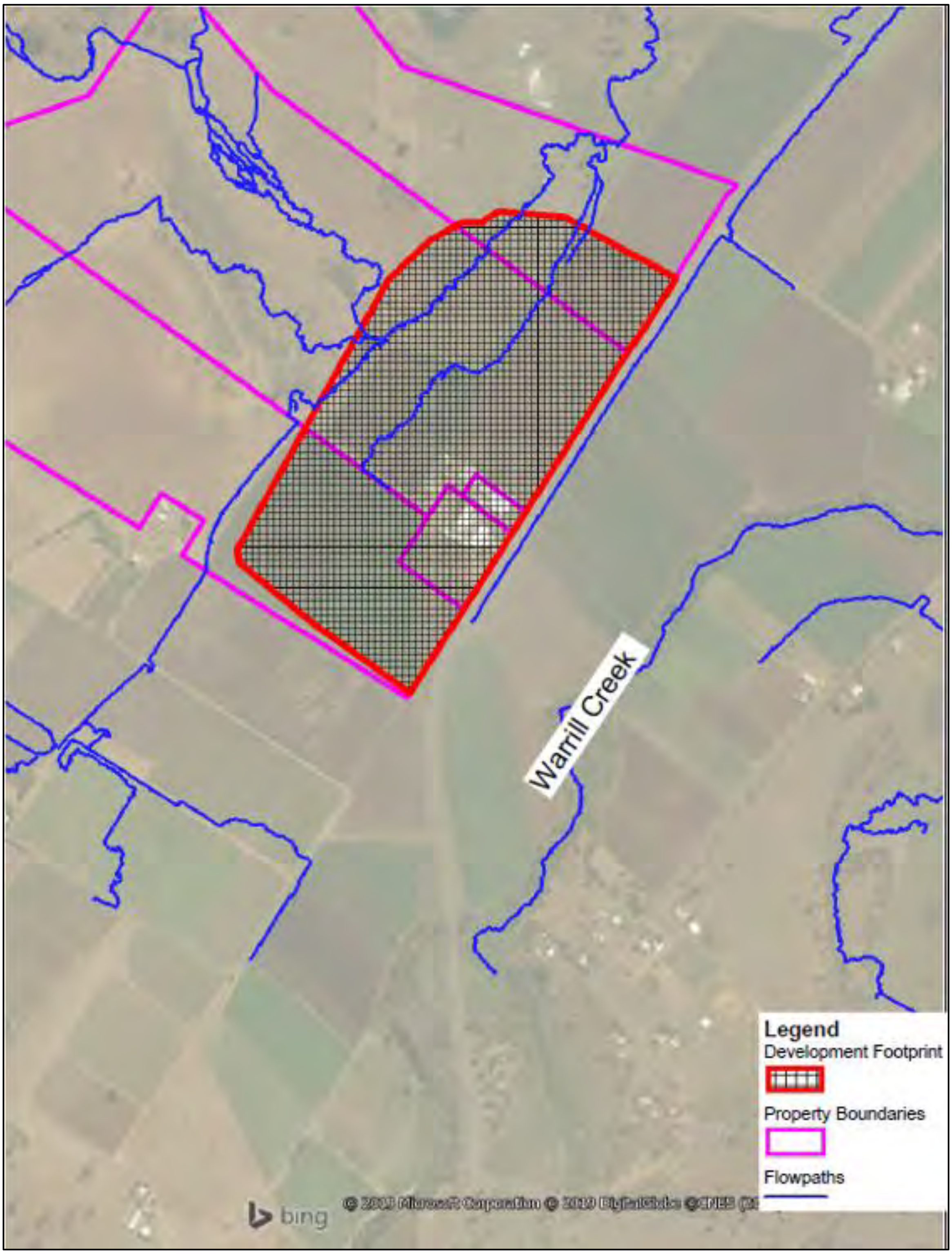


Figure 2-1 Existing flowpaths

The site is located on a floodplain that is inundated by Warrill Creek. Additional to the Warrill Creek flooding there are two catchments located to the West of the site, draining along the western boundary of the development area.

2.2 Land Encumbrances

The site is impacted by the 1% AEP flood inundation mapping, part of the SRRC Flood and Inundation Hazards Overlay.

2.3 Proposed Development

It is proposed to develop the site into a Rural Enterprise Precinct, enabling local food businesses to base themselves where the raw ingredients are grown. A bio energy facility is also proposed as part of the development.

2.4 Lawful Points of Discharge

The Lawful Point of Discharge for the site is the current flow path that exits the site to the North. It is intended that onsite detention will return peak flows to equal or less than existing.

3 Hydrology

3.1 Background

Regional inflows were sourced from the existing Warrill Creek Flood study. The Warrill Creek Flood Study was completed in 22 May 2018 by Aurecon and includes the adjacent Reynolds Creek. These two creeks have a combined contributing catchment of approximately 42,000 ha with Warrill Creek catchment approximately 17,000 ha. Major flows (Warrill and Reynolds Creeks) were extracted from the Warrill Creek Flood Study (2018), local catchment flows were sourced directly from the Warrill Creek Flood Study input layers.

To assess the development sites detention requirements a XPRAFTS model of the site was also developed.

3.2 Model Setup

3.2.1 Catchment delineation

Existing site catchments were determined from the design digital elevation model (DEM) for use in the quantity analysis.

Catchments to the west of the site were calculated using the QGIS watershed function on the DEM supplied with the IAS data. Local catchment flows sourced from the Warrill Creek Flood Study were proportioned based on the catchment areas determined above and input to the west of the site area.

3.2.2 Rainfall Parameters

Rainfall parameters input to RAFTS were sourced using ARR 2019 BOM IFD values.

3.2.3 RAFTS Parameters

Catchment characteristics are shown in Table 3-1. Land uses were defined as per aerial imagery. Catchment slopes were determined using drainage lines for each catchment.

A critical aspect of hydrological models is the choice of loss rates (e.g. initial and continuing losses). Section 4.2.3 of QUDM recommends that the latest version of *Australian Rainfall and Runoff* should be used when selecting loss rates. As such, loss rates were set based on Book 4, Catchment Simulation for Design Flood Estimation. Catchments were split into impervious/ pervious areas with each sub-area having a different loss rate applied. Initial and continuing loss values were sources using the AR&R 2019 data hub with values shown in Table 3-2.

Table 3-1 Existing RAFTS Catchments

Catchment ID	Total Area (ha)	Percentage Impervious (%)	Vectored Slope (%)	Catchment Manning's 'n'
C2	2	0	0.5	0.045
C3	2.2	0	0.5	0.045
C4	2.1	0	0.5	0.045
C5	5	0	0.5	0.045
C6	4	0	0.5	0.045
C7	1.45	0	0.5	0.045
C8	1.4	0	0.5	0.045
C9	1.5	0	0.5	0.045
C10	2	0	0.5	0.045
C11	11.5	0	0.5	0.045
C12	1.6	0	0.5	0.045
C13	2.1	0	0.5	0.045

Table 3-2 Initial and Continuing Losses for RAFTS Modelling

Land Use	Initial and Continuing Losses
Rural Areas	21mm IL/ 3mm/h CL
Industrial	2mm IL/2mm/h CL

3.2.4 Rational Method Verification

To validate the peak design flows derived from the hydrological model, AR&R 2019 recommends that the Rational Method be used where there is no real data present to validate flows. The C_{10} coefficient for use within the Rational Method was sourced from Chapter 4 of QUDM. The Bransby Williams Method was used for estimating the time of concentration for each selected catchment. Catchments C2 and C11 were chosen for this comparison. C10 values were sourced using Table 4.5.3 and 4.5.4 of the *Queensland Urban Drainage Manual 2017*. The results of this verification are shown in Table 3-3 below for the 1% AEP event. It was found that the RAFTS model produced higher peak flows than the Rational Method for Catchment C11 and lower flows for C2. The RAFTS flows calculated were of a similar magnitude, therefore they were considered acceptable to use in the quantity analysis.

Table 3-3 Rational Method Verification

Parameter	Value	
	C2	C11
C10	0.4	0.4
t_c (min)	10	36
I (100yr,Xmin)	234	107
A (ha)	1.53	11.5
Rational Method 1% Flow (m^3/s)	0.56	1.76
RAFTS Peak 1% AEP Flow (m^3/s)	0.34	1.89
Difference (m^3/s)	0.22	-0.13

3.2.5 Modelled Events

As part of AR&R 2019, an ensemble approach to design flood estimation is recommended which, alongside each defined AEP event and duration, are 10 temporal patterns. As part of this hydrology model 110 design storms were run for each AEP event to determine the appropriate design flow for use within the hydraulic model. The modelled AEP events are shown in Table 3-4.

Table 3-4 Modelled Events

AEP	Duration (hrs)	Temporal Pattern
1%	0.16 to 4.5	1 to 10
2%	0.16 to 4.5	1 to 10
5%	0.16 to 4.5	1 to 10
10%	0.16 to 4.5	1 to 10
20%	0.16 to 4.5	1 to 10
50%	0.16 to 4.5	1 to 10

3.2.6 Critical Durations/ Ensemble Selection

A critical aspect of hydrological and hydraulic models is the choice of the critical storm. AR&R 2019 recommends that for most hydraulic models, for each AEP event, a representative critical duration and temporal pattern be used (Book 2, Chapter 5). This is to reduce run times while providing the most accurate flood estimation.

The critical storm was selected for each AEP by calculating the highest average flow across all durations and temporal patterns for each AEP. The duration and temporal pattern that produced the flow closest to the calculated average was then used as the representative ensemble for use within the hydraulic model. This approach limits the amount of runs in the hydraulic model from approximately 660 to 6. The selected ensembles and associated critical durations are shown in Table 3-5.

Table 3-5 Critical Events

Event (AEP)	Critical Duration (hrs)	Temporal Pattern
1%	1	10
2%	1	6
5%	2	9
10%	2	9
20%	2	6
50%	3	1

3.3 Results

Peak flows for each of the sub catchments are listed in Table 3-6 for the 1% AEP event. The critical storm duration for the 1% and 2% AEP events was determined to be 1 hour. For the 5%, 10% and 20% AEP events, the critical storm duration was 2 hours. For the 50% AEP event this increased to 3 hours. This difference in critical storm duration is considered to be typical for models run with ARR 2019 rainfall patterns. In addition to the peak flows coming from each catchment, the existing peak total flow from the proposed development site was found to be 7.8 m^3/s in the 1% AEP event.

Table 3-6 Existing 1% AEP Peak Flows (m^3/s)

Catchment ID	Peak Flow (m^3/s)
C2	0.34
C3	0.37
C4	0.35
C5	0.67
C6	0.57
C7	0.26
C8	0.26
C9	0.27
C10	0.34
C11	1.89
C12	0.29
C13	0.35

4 Stormwater Quantity Management

4.1 Background

Using XP-RAFTS, a hydrological model was developed for the site to represent the proposed development as outlined in Section 3. The model was then modified to determine the effects that the proposed development would have on the stormwater discharge (peak flow rates) coming from the subject site. It was anticipated that, as a result of the development, peak flow rates will increase. As such, recommendations are made to ensure peak flows from the site are equal to or less than those for existing conditions where possible. Detention storage on site will be achieved through a proposed detention basin system.

4.2 Model Setup

4.2.1 Catchment Parameters

Catchment parameters for the existing and developed local catchment hydrology are shown in Table 4-1. The existing scenario is as per the hydrological model described above. The changes made for the developed scenario involved updating the percentage impervious values to represent the proposed development. These changes are shown in the table below.

Table 4-1 Development Site Catchment Parameters

Catchment Parameters Existing vs Developed				
Catchment ID	Area (ha)	Percentage Impervious (Existing) (%)	Percentage Impervious (Developed) (%)	Catchment Manning's 'n' Perv/Imperv
C2	2	0	90	0.045/0.025
C3	2.2	0	90	0.045/0.025
C4	2.1	0	90	0.045/0.025
C5	5	0	90	0.045/0.025
C6	4	0	90	0.045/0.025
C7	1.45	0	90	0.045/0.025
C8	1.4	0	90	0.045/0.025
C9	1.5	0	90	0.045/0.025
C10	2	0	90	0.045/0.025
C11	11.5	0	90	0.045/0.025
C12	1.6	0	90	0.045/0.025
C13	2.1	0	90	0.045/0.025

4.2.2 RAFTS Parameters

Catchment information was input to the RAFTS model and analysed. Only the runoff from the site itself was considered for the assessment of non-worsening compared to existing conditions. As expected, the proposed development results in an increase in peak runoff from the site, with a peak flow of 13.2 m^3/s discharging from the site for the 1% AEP event without any mitigation measures. This is an increase of approximately 7.6 m^3/s when compared to the existing scenario.

4.2.3 Stormwater Detention

Increases in impervious area within the proposed development have resulted in an increase in local catchment stormwater discharges. A number of detention basins are therefore proposed to maintain peak stormwater discharges to values equal to or less than existing conditions. The design characteristics for the proposed basins are shown in Table 4-3. These values may be altered subject to detailed design to ensure satisfactory performance.

4.3 Quantity Modelling Results

The proposed detention system was incorporated in the XP-RAFTS model which was run for the same events as discussed in Section 4.2.2. The peak discharges from the site for pre-development conditions and for post-development conditions, with the detention storage included are shown in Table 4-2.

Table 4-2 Peak XP-RAFTS Discharge

Peak Flow Rate			
Event (AEP)	Pre-Development Peak Discharge (m^3/s) (Site Only)	Post-Developed (Mitigated) Peak Discharge (m^3/s) (Site Only)	Difference %
1%	5.9	4.9	-17
2%	4.9	3.8	-22
5%	3.8	2.6	-32
10%	3	2	-33
20%	2.2	1.2	-45
50%	0.86	0.8	-7

As shown above, the proposed basins result in a decrease in peak discharges from the site for all AEP events modelled in the mitigated developed scenario.

The final number of detention basins and sizing is expected to be refined during the detailed design stage. Table 4-3 shows the basin parameters and maximum storage volume recorded for each modelled event across all basins.

Table 4-3 Detention Basin Data

Basin ID	Discharge Pipe Diameter (m)	Peak Basin Stage (m)	Spillway Level (m)	Peak basin storage (m^3)	Spillway Width (m)	Approximate Surface Area (m^2)
B2	0.225	1.41	1.4	835	5	873
B3	0.225	1.43	1.4	852	5	880
B4	0.225	1.35	1.3	776	5	824
B5	0.225	1.44	1.3	862	5	889
B6	0.225	1.90	1.8	1343	5	1139
B7	0.225	1.21	1.2	663	5	776
B8	0.225	1.15	1.1	617	5	776
B9	0.225	1.19	1.2	646	5	776
B10	0.225	1.06	0.9	545	5	684
B11	0.225	1.52	1.4	6759	7	5,184
B12	0.225	1.49	1.4	903	5	924
B13	0.225	1.54	1.2	953	5	940

5 Stormwater Quality Management

5.1 Stormwater Quality Objectives

5.1.1 Operational Phase

The stormwater quality design objectives applicable to the site, as outlined in Table B of the *State Planning Policy (SPP)* (Department of Infrastructure, Local Government and Planning, 2017) are:

> Total Suspended Solids (TSS) 80% removal of mean annual load from unmitigated development.

> Total Phosphorous (TP) 60% removal of mean annual load from unmitigated development.

> Total Nitrogen (TN) 45% removal of mean annual load from unmitigated development.

> Gross Pollutants > 5mm 90% removal of mean annual load from unmitigated development.

5.1.2 Construction Stage

The construction phase stormwater quality design objectives applicable to the site are outlined in Table A of the DILGP *State Planning Policy* (Department of Infrastructure, Local Government and Planning, 2017).

The release values for stormwater captured in a sediment basin are not to exceed the following limits:

> Total Suspended Solids (TSS): 50 mg/L

> pH: 6.5 – 8.0

Appropriate erosion and sediment control measures will be required to be designed, constructed and operated in accordance with the SPP 2017 guidelines during the construction phase of the development.

5.2 Proposed Stormwater Quality Treatment

5.2.1 Summary

The proposed stormwater treatment train is to treat the site roadways only with the catchment shown in Figure 5-1. It has been assumed that individual lots will install their own water quality devices as part of the development approval process for each lot.

The treatment train consists of an end of line gross pollution trap (GPT) and a bio-basin. The bio-basin will be contained within the proposed main detention basin. Currently the GPT has been modelled as one unit. This system may be modified during the detailed design stage if a single GPT proves difficult to incorporate.

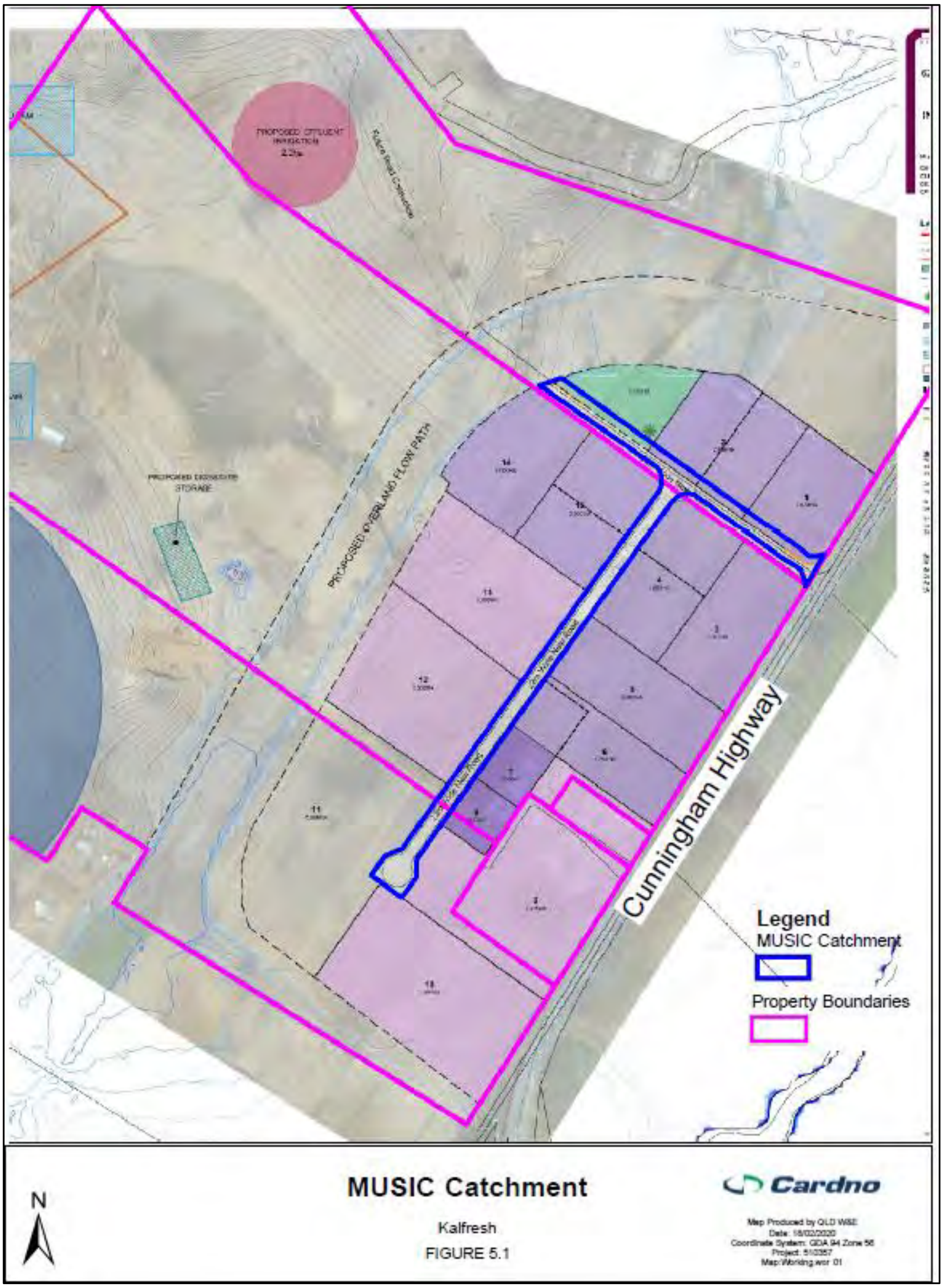


Figure 5-1 MUSIC Catchment

5.2.2 MUSIC Model

A MUSIC (Ver. 6.3) model was set up to determine the efficiency of the stormwater quality treatment train proposed for the development. The meteorological data adopted for the site was from the Harrisville Post Office weather station (40094) as recommended in the *MUSIC Modelling Guidelines – Version 1.0 2010* (Water By Design, 2010), for the 11 year period from 01/01/1997 through to 31/12/2006.

The MUSIC source node characteristics were based on the values for industrial developments as outlined in the *MUSIC Modelling Water by Design Guidelines* (Healthy Land and Water (2018) MUSIC Modelling Guidelines), Table 3.9. The adopted source node characteristics are summarised in Table 5-1 and Table 5-2. The MUSIC model layout is shown in Figure 5-2.

Table 5-1 Rainfall Runoff Parameters for Industrial Source Nodes

Parameter	Value
Rainfall threshold (mm)	1
Soil storage capacity (mm)	18
Initial storage (% capacity)	10
Field capacity (mm)	80
Infiltration capacity coefficient a	243
Infiltration capacity exponent b	0.6
Initial depth (mm)	50
Daily recharge rate (%)	0
Daily baseflow rate (%)	31
Daily deep seepage rate (%)	0

Table 5-2 Pollutant Export Parameters for Industrial Source Nodes

Parameter	Surface Type	TSS mg/L	TP mg/L	TN mg/L
Base Flow	Roads	0.78	-1.11	0.14
Storm Flow	Roads	2.43	-0.30	0.25

Figure 5-2 MUSIC Model Layout

The catchment area shown in Figure 5-2 was determined via design drawings. Catchment area parameters are shown in Table 5-3.

Table 5-3 Catchment Area and Proposed Stormwater Treatment

	Area (ha)	% Impervious	MUSIC Node	Proposed Treatment
Catchment 1				
Road	2.57	95	Industrial	GPT, Bio Basin

The Bio Basin parameters are shown in Table 5-4.

Table 5-4 Bio-Basin 1 Details

Parameter	Bio-retention Basin
Extended Detention Depth (m)	0.5
Saturated Hydraulic Conductivity (mm/hr)	200
Filter Depth (m)	0.4
Filter Area (m^2)	80
TN Content of Filter Media (mg/kg)	400
Orthophosphate Content of Filter Media (mg/kg)	30

GPT capture rate details shown in Table 5-5 are as provided by the manufacturer.

Table 5-5 GPT Pollutant Capture Rates

Gross Pollutants (kg/ML)		Total Suspended Solids (mg/L)		Total Phosphorous (mg/L)		Total Nitrogen (mg/L)	
Input	**Output**	**Input**	**Output**	**Input**	**Output**	**Input**	**Output**
0	0	0	0	0	0	0	0
14.9664	2.2499	500	295	5.0	3.3	5.0	3.8

5.3 MUSIC Model Results

The results from the MUSIC modelling are presented in Table 5-6. As demonstrated by this table, the water quality objectives have been satisfied for all pollutants.

Table 5-6 MUSIC Model Results

Parameter	Mean Annual Load (kg/yr)		% Removal	Target (%)
	Sources	Residual		
Total Suspended Solids	5490	1080	80.3	80
Total Phosphorus	8.97	2.53	71.8	60
Total Nitrogen	29.6	15.7	47.1	45
Gross Pollutants	379	0	100	99.9

5.4 Construction Phase Water Quality Management

The construction phase of the development requires erosion and sediment control measures to be put in place to ensure that there is not a significant increase in the pollutant loads discharging from the site. During the construction phase an erosion and sediment control plan will be developed ensuring that the construction stage stormwater quality objectives are met. When appropriate, implemented erosion and sediment controls will be removed, generally as a minimum when 80% site cover has been achieved.

Further erosion and sediment control measures will be implemented in accordance with Council requirements and in consideration of the *Best practice guidelines for the control of stormwater pollution from building sites* (Healthy Waterways, 2002).

5.5 Maintenance

Maintenance of the stormwater quality devices is to be carried out by Kalfresh Pty Ltd or a nominated contractor. During the construction phase, erosion and sediment control devices will be maintained by the contractor/developer.

The maintenance requirements of any treatment devices installed are to be carried out in accordance with water sensitive urban design (WSUD) and manufacturer’s guidelines.

6 Hydraulic Analysis - Regional Flooding

6.1 Background

The 1D/2D modelling program TUFLOW, was used to compute the channel and overland flow components of the subject site and surrounding area. TUFLOW is a suite of advanced numerical engines and supporting tools used for simulating free-surface water flow for urban waterways, rivers, floodplains, estuaries and coastlines.

The site is subject to both local and regional flooding. Local flooding is caused by catchments west of the site draining through the North West portion on the proposed development site. Regional flooding from the Warrill Creek catchment impacts the site via overflow from Warrill Creek located East of the development area

6.2 Model Setup

To determine the effects of the proposed development on surrounding properties, a TUFLOW model was developed. The inputs into the model are outlined below;

> Hydrological (flow) information;
> Topography (LiDAR);
> Key drainage infrastructure (1D);
> Land uses;
> Key drainage infrastructure; and
> Model boundary conditions.

Flow information for the TUFLOW model was obtained from the hydrological modelling described earlier in this report (Section 3).

6.2.1 Grid Size

Modelling was performed with a cell size of 10 metres. This size was deemed to provide sufficient resolution while ensuring reasonable run times and stability.

6.2.2 Topography

LIDAR topographical data was supplied with the Warrill Creek Flood Model extraction. This was used as the key input to represent the existing scenario topography. This data was obtained on a 1 metre grid and was deemed suitable for use within the model. The existing topography is shown in Appendix A-1.

In order to assess for impacts caused by the proposed development a design DEM was input into the model representing the development. The design DEM was overlayed on top of the existing topography. This topography is shown in Appendix A-2 with the differences between the topography shown in Appendix A-3.

6.2.3 Boundary Conditions

Flow inputs for the TUFLOW model were obtained from the hydrological modelling described earlier in this report (Section 3).

The boundary conditions for the model consisted of ‘local’ inflows from each of the western catchments, as described above and Reynolds Creek. Inflows extracted from the Warrill Creek model were input into the model at Warrill Creek, upstream of the development site. The downstream boundary of the model adopted a head-time boundary (HT) extracted from the Warrill Creek model. The boundary conditions of the model are shown in Appendix A-4.

6.2.4 Hydraulic Roughness

Manning’s n roughness values were determined using a combination of aerial imagery and site photographs. These values are shown in Table 6-1. The existing and developed hydraulic roughness values are shown in Appendix A-5 and A-6 respectively.

Table 6-1 Manning's 'n' Values

Type	Manning's n
Roads	0.022
Residential	0.100
Open Space	0.035
Cropping	0.050
Commercial/Industrial	0.500
Vegetated Bush – Medium Density	0.080
Vegetated Bush – High Density	0.120
Watercourse	0.050
Watercourse (Dense)	0.070
Railway	0.030
Buildings	1.000

6.2.5 Stormwater Structures

Existing stormwater infrastructure details were sourced from the Warrill Creek Flood model and used as supplied.

6.3 Modelled Events

As discussed in Section 3 flood events were sourced from the existing Warrill Creek Flood Study. This limited the number of events to those previously modelled.

As part of the existing and developed scenarios the events run within the hydraulic model are shown in Table 6-2.

Table 6-2 Hydraulic Modelling Events

Event (AEP)	Critical Durations
10%	6/12hr
5%	6/12hr
2%	6/12hr
1%	6/12hr
1% CC	6/12hr

6.4 Sensitivity Testing

As this model was developed from the approved Warrill Creek flood model, no sensitivity analysis was performed.

6.5 Hydraulic Modelling Results

6.5.1 Pre- Development Scenario

Maps of the peak flood levels for the pre-development scenario are shown in Appendix B for the 10% to 1% CC AEP events with peak flood depths shown in Appendix C.

The existing case results show that approximately half of the proposed development area is inundated in all AEP events. Depths of up to 1 metre on the Northern portion of the site are shown in Appendix C. As shown in Appendix B the majority of the Eastern portion of the site remains flood free during smaller events (10, 5% AEP) with extensive inundation during larger events. During the 1%CC event the site is completely covered. This flooding is caused by overland flow from Warrill Creek and flows from the western catchments. During flood events the water flows from the South to the North via the Western areas of the site, exiting into the existing “creek” line. In events greater than the 5% AEP floodwaters also cross the highway from East to West, onto the development site.

6.5.2 Post- Development Scenario

Maps of the peak flood levels for the post-development scenario are shown in Appendix B for the Q10 to Q100CC events with peak flood depths shown in Appendix C. Appendix D confirms that topography modifications have resulted in minor changes to water surface levels.

As a result of filling on the development site, flood waters no longer encroach onto the proposed development area. Flows that previously covered the Western portion of the site are now diverted along the western boundary via a drainage channel. Flows from Warrill Creek enter this drain at the South West corner of the site, discharging to the North West. Flows from the western catchment discharge into the drainage channel to be conveyed North, exiting the site as per the existing case.

6.5.3 Impacts of Development

Maps of the flooding post development are shown in Appendix D for the 10%, 5%, 2%, 1% and 1% CC AEP events. Due to the fill encroaching on the flood extents, water level increases have been introduced in some areas. However not all of these increases are deemed a result of the proposed development.

During the developed case 1% AEP, maximum water surfaces downstream of the site are 81.3 mAHD (Appendix B-4), occurring at a location on the North West boundary where water surfaces are 81.14 mAHD during the existing case 1% AEP event. The developed case water level is approximately 4 metres below the nearest structure located at 85.2 mAHD and 5.5 metres below the nearest residence located at 86.8 mAHD. No water surface level increases are located on residence accesses. Therefore, increases noted are deemed inconsequential with no actionable nuisance at this location.

Increases shown on the East side of the highway during the 1% CC AEP are a result of flows across the highway being restricted in the developed case. In the existing case floodwater in events greater than the 5% AEP flow from East to West across the highway. This movement is restricted in the developed case as a result of lot filling. Due to the coarse model definition, swale drains alongside the highway adjacent to the lack detail which combined with the lot filling, contributes to the water surface level increases shown.

During the 1% CC AEP event, peak increases shown on the Eastern side of the highway are 30 mm located in one section of the Eastern swale drain. Water depths at this location are 700 mm deep during the existing case event with extensive flooded areas surrounding it. No changes to flood extents are noted as a result of the increases shown.

During the 1% CC and the 2% AEP event, a number of areas to the East of the highway are showing minor water surface level increases. While some of these increases are a product of the items noted above, other areas further East of Warrill Creek show increases which cannot reasonable be attributed to the proposed development.

This is particularly the case for increases adjacent to and on the Eastern side of Warrill Creek (Appendix D-3). At these locations the existing water surface level (83.7 mAHD) is approximately 2 metres above the water surface level adjacent to the proposed development (81.8 mAHD). Therefore, it is implausible that changes to the development area topography approximately 500 metres to the West, on the opposite side of the highway have induced these impacts.

These impacts have been deemed a result of minor variations in flood levels within Warrill Creek between cases caused by the topography definition. This is also the case for an isolated area of depth increase shown to the south of the development area in the 5% AEP event. It should be noted that even with the increases shown, the developed 5% AEP water surface levels are 1 metre below the existing 1% AEP levels.

Generally, all water surface level increases that can be attributed to the proposed development are located in areas that have flood depths greater than 900 mm during the existing case with no changes to the flooding extent. None of the increases reported pose a risk to persons or infrastructure. The majority of offsite increase are less than 50 mm. These increases dissipate quickly moving downstream away from the development.

Near the neighbouring property to the South West, the water surface level decreases from 84.3 mAHD pre-development to 83.4 mAHD post development, a 900 mm decrease for the 1% AEP event. This is a result of improvements in flow paths at this location.

7 Conclusions

This report has investigated the stormwater issues for both the existing and developed cases for the property located at Kalbar on the Cunningham Highway.

Water quality modelling has demonstrated that the development can meet the required State Planning Policy requirements regarding water quality discharge limits. The proposed bio-retention basin will be incorporated into the quantity management system. It should be noted that the water quality measures discussed in this report are only intended to treat the development road reserve area. It has been assumed that individual lots will be providing their own treatment systems located on each lot. It is assumed that these treatment areas will be located in the detention basins proposed.

Stormwater quantity management has demonstrated that post-development flows can be reduced to less than those present in the existing case for all cases except the 50% AEP event which shows a 1% increase. This mitigation has been achieved via the implementation of detention basins located throughout the development. It is expected that the detention basin detailing will be further refined during the detailed design stage.

Hydraulic modelling has demonstrated that the proposed development will not have significant adverse effects on the surrounding properties with no actionable nuisance. While some areas are showing water level increases these are not considered to be significant due to these areas being currently inundated by depths up to 1.6 metres deep in the existing case modelling. No significant changes to flooding extents are evident as demonstrated in Appendix D with no changes to the current flood hazard categories.

8 Qualifications

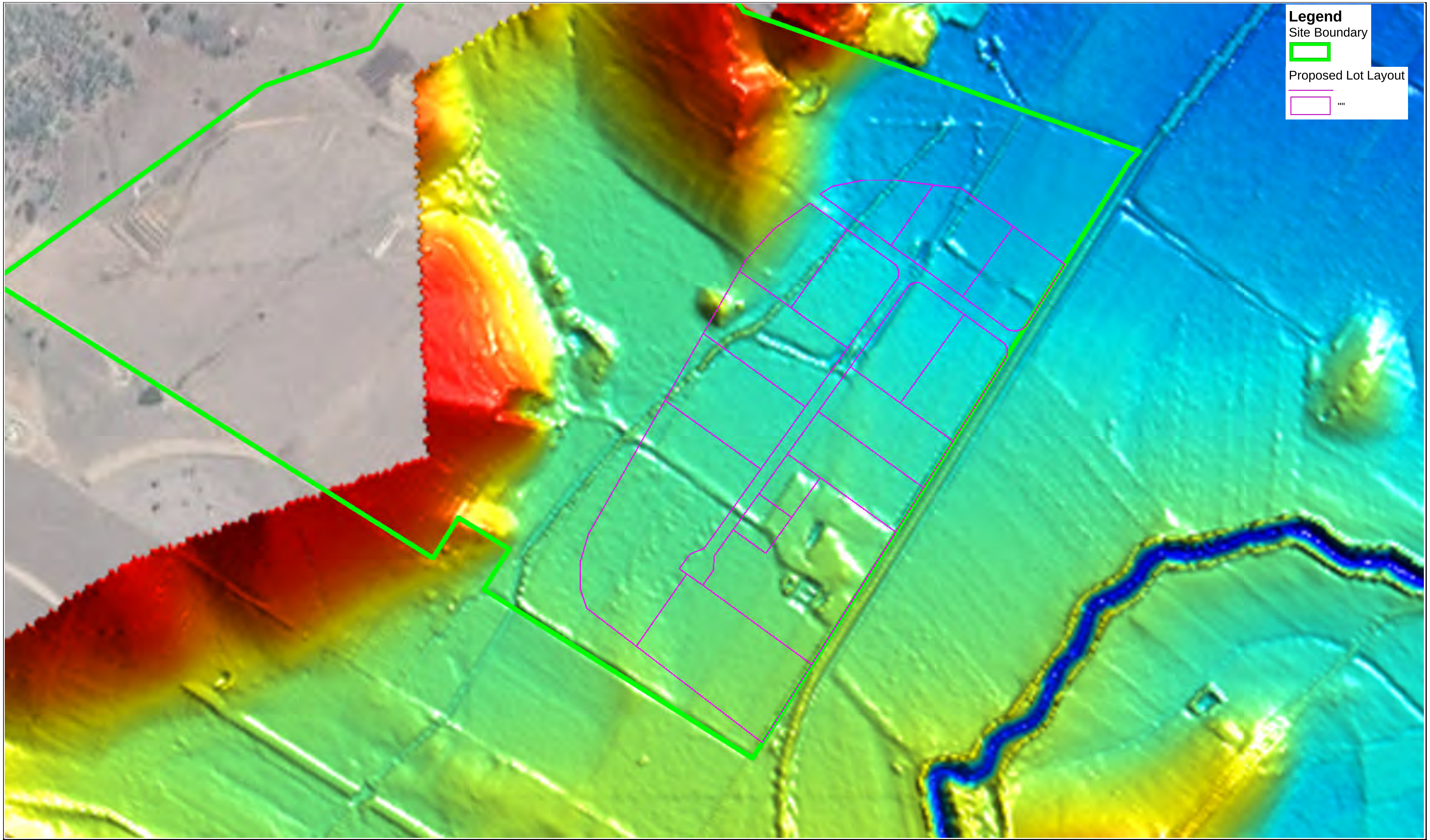
This technical memo has been prepared by Cardno (Qld) Pty Ltd for Kalfresh Pty Ltd. Specifically, this memo provides information relating to existing flooding within the development area. Our analysis and overall approach have been catered to the specific requirements of Kalfresh, and may not be applicable beyond this scope. For this reason, any other third parties other than Kalfresh are not authorized to utilise this memo without further input and advice from Cardno (Qld) Pty Ltd.

Whilst this report accurately assesses catchment hydrologic and hydraulic characteristics based on design storms using industry standard modelling techniques and engineering practices, the actual future observed flows, levels and extent of inundation may vary from those predicted.

APPENDIX

A

TUFLOW MODEL SETUP

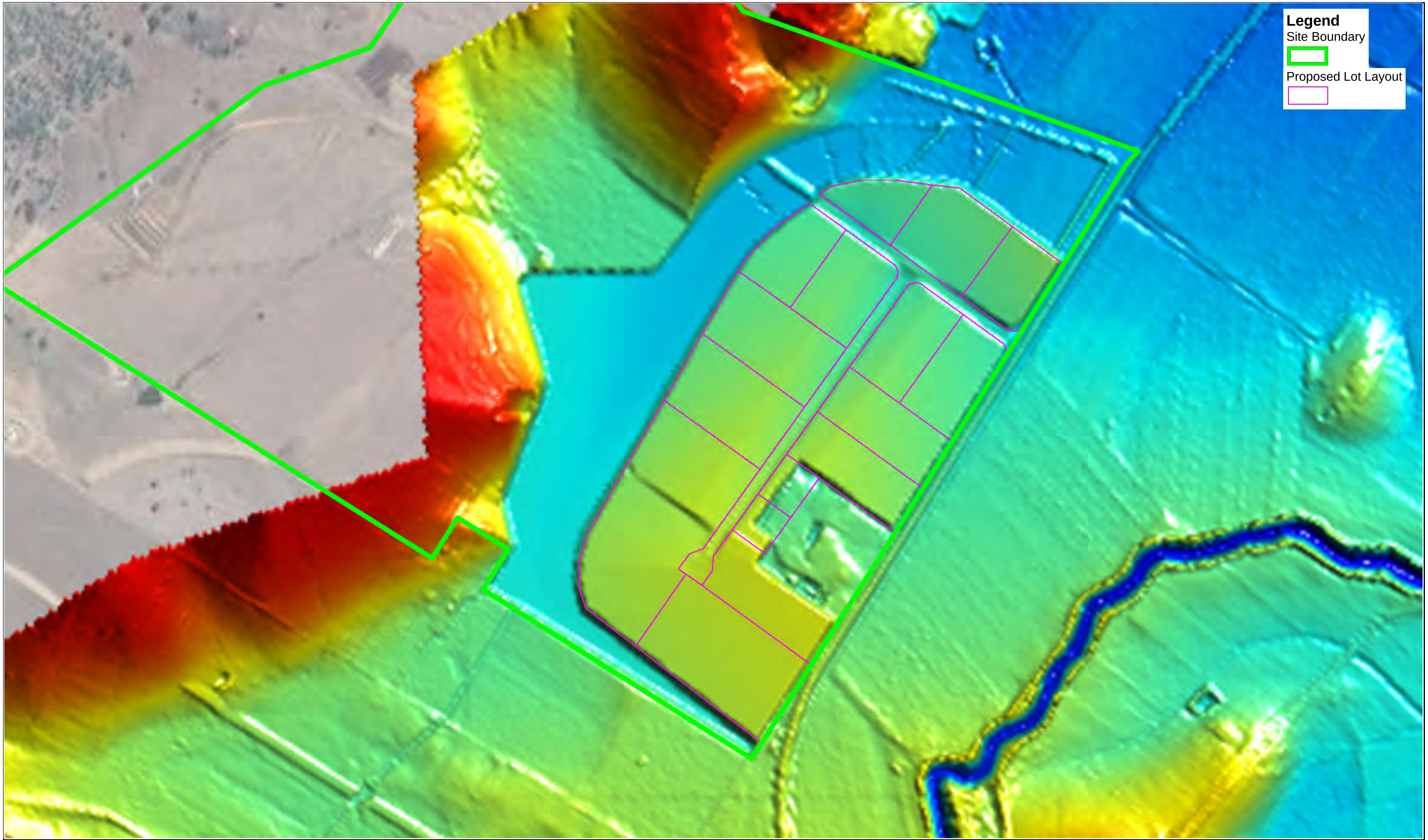


Existing Topography

Appendix A-1

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



Developed Topography

N

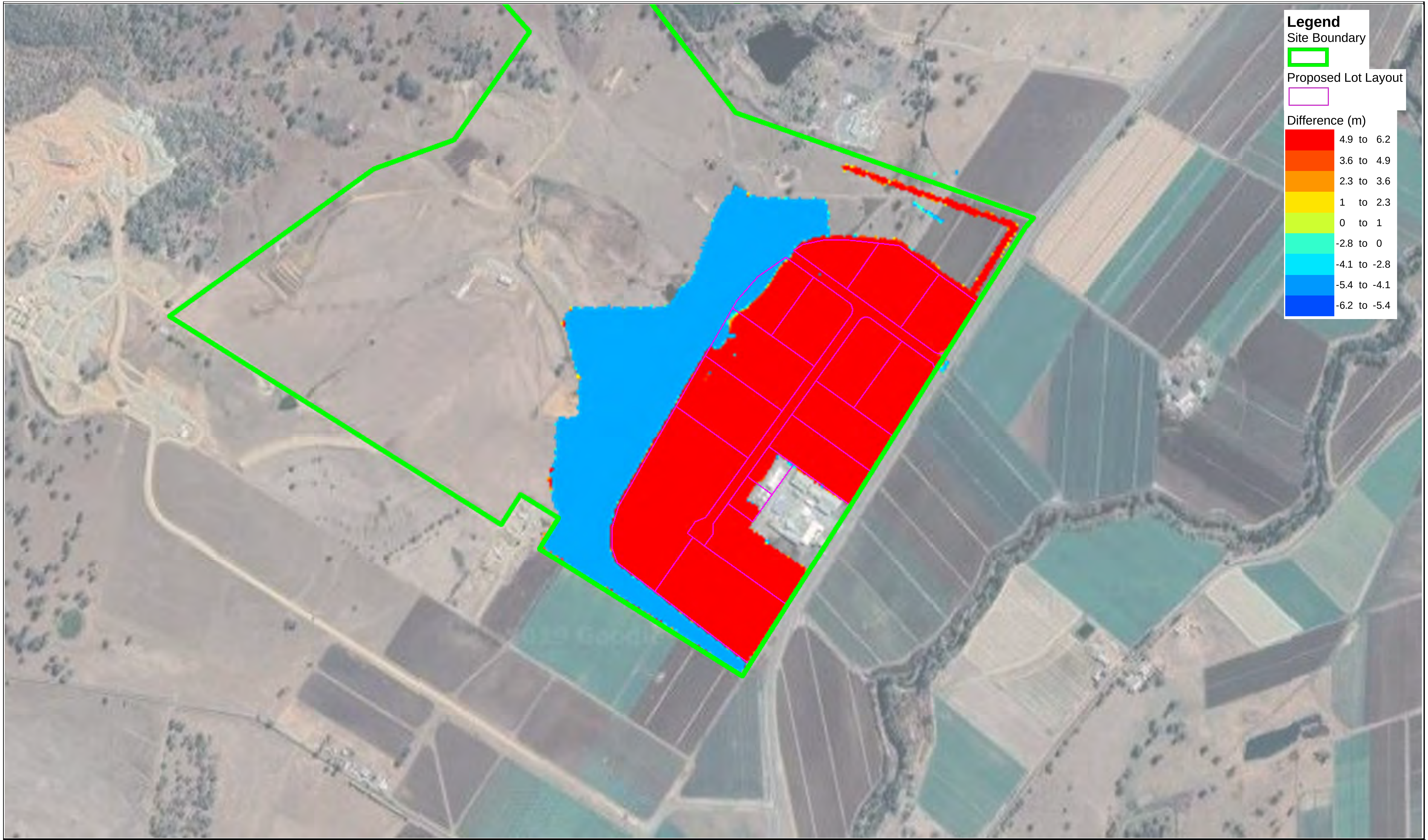
0 300

metres

Appendix A-2

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



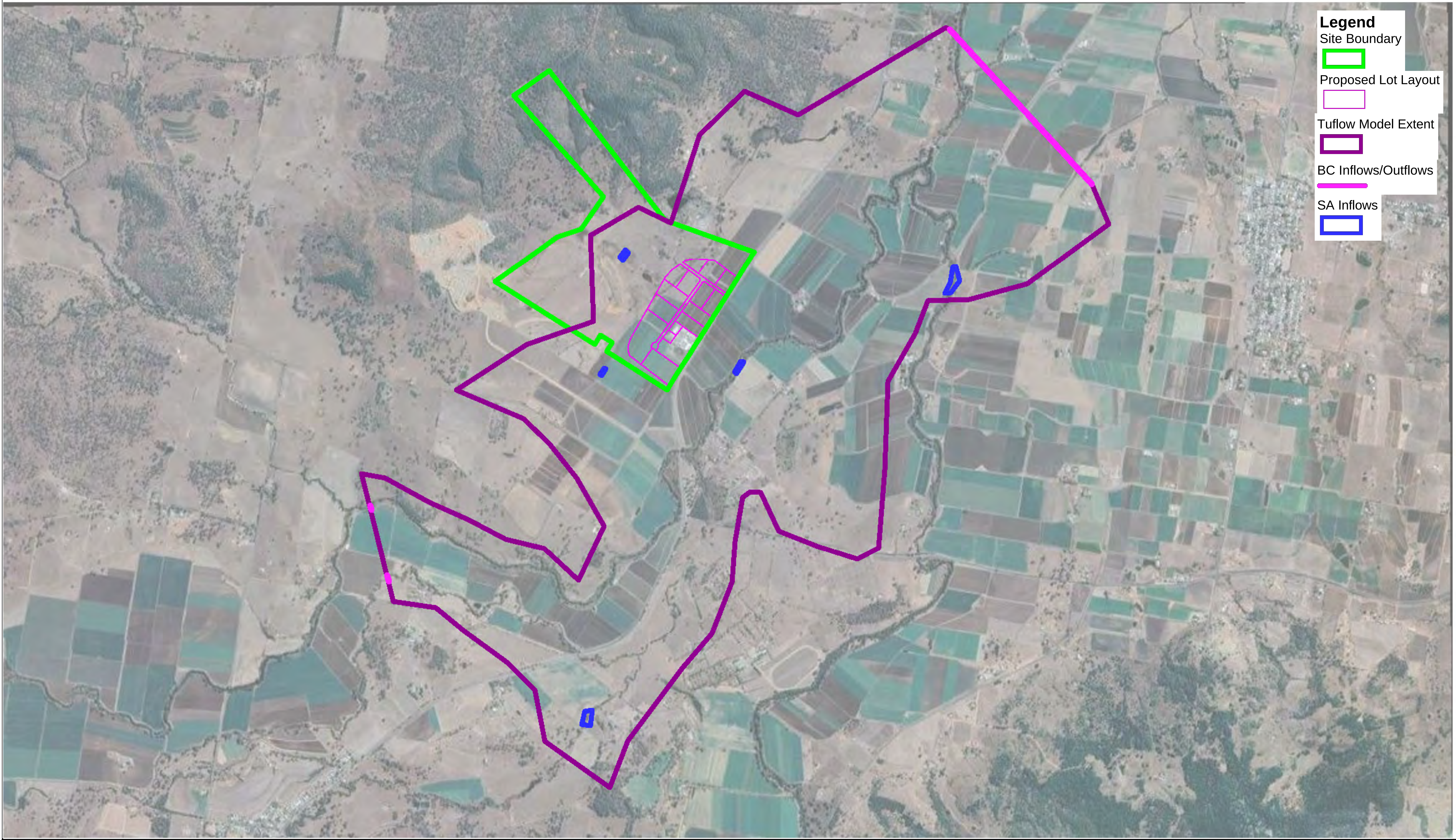
Topography Difference

Appendix A-3

N
0 300
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



TUFLOW Boundary Conditions

Appendix A-4

N

0 1,366

metres

Cardno

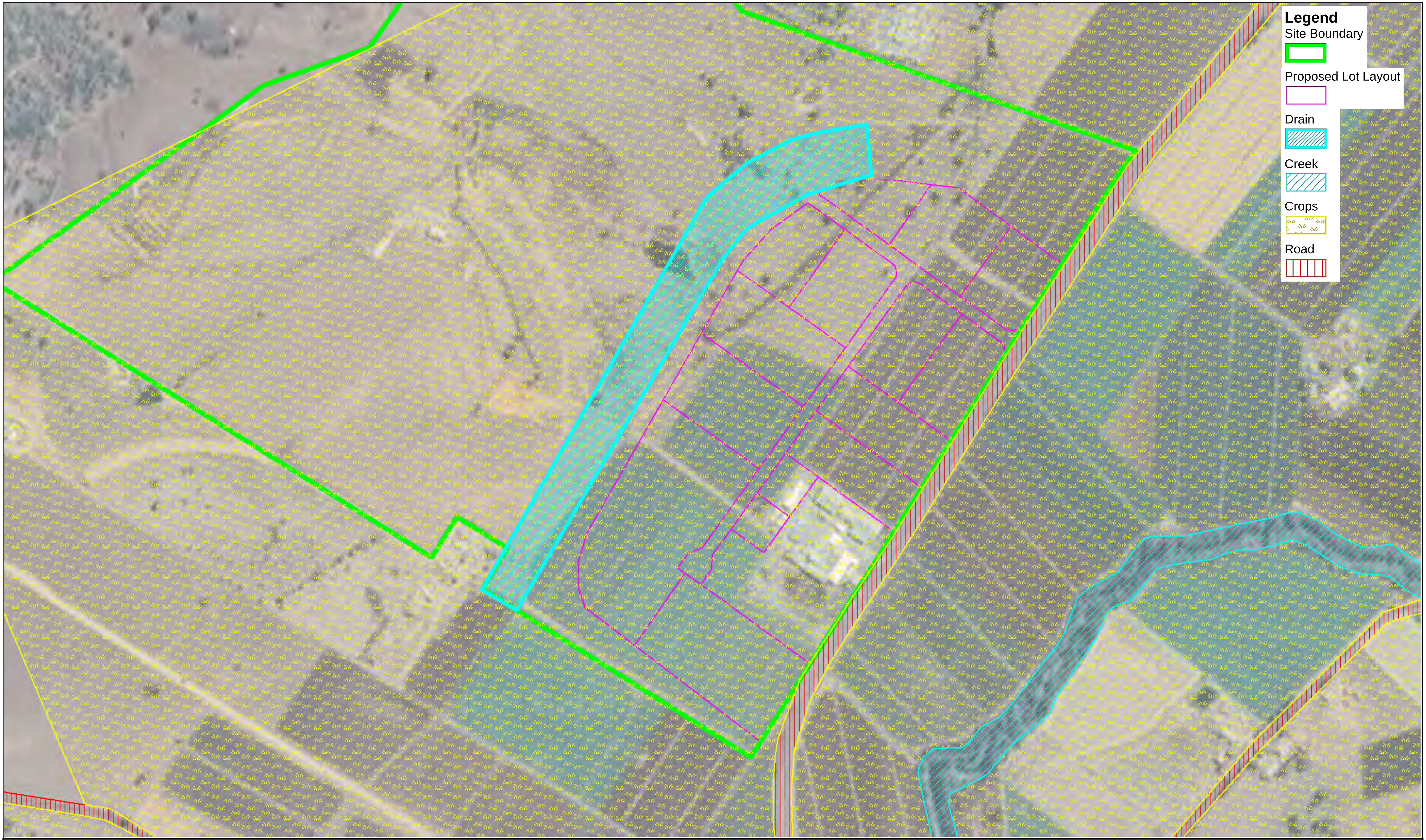
Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

Existing Mannings Values

Appendix A-5

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



Developed Mannings Values

Appendix A-6

N

0 300

metres

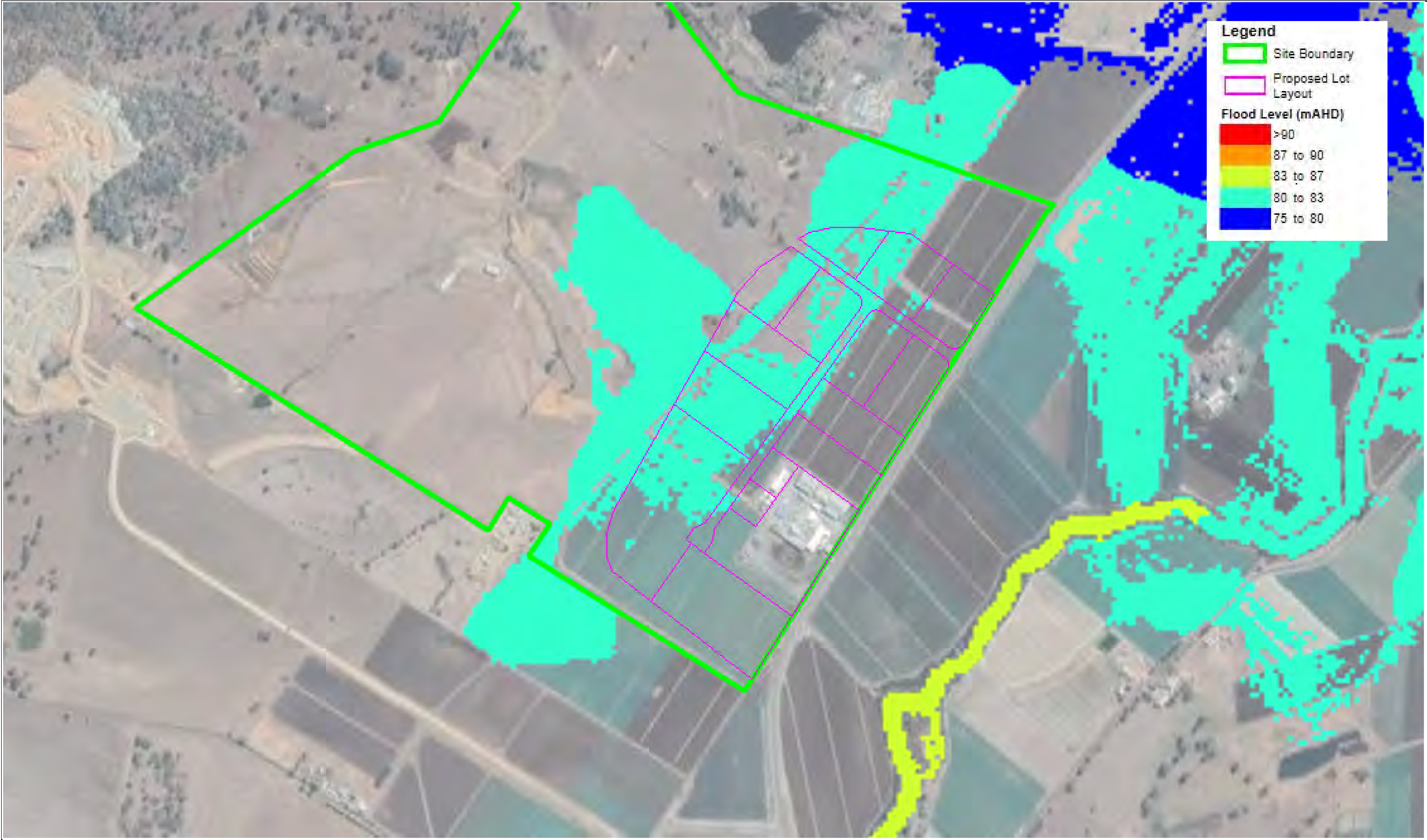
Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

APPENDIX

B

WATER SURFACE LEVELS



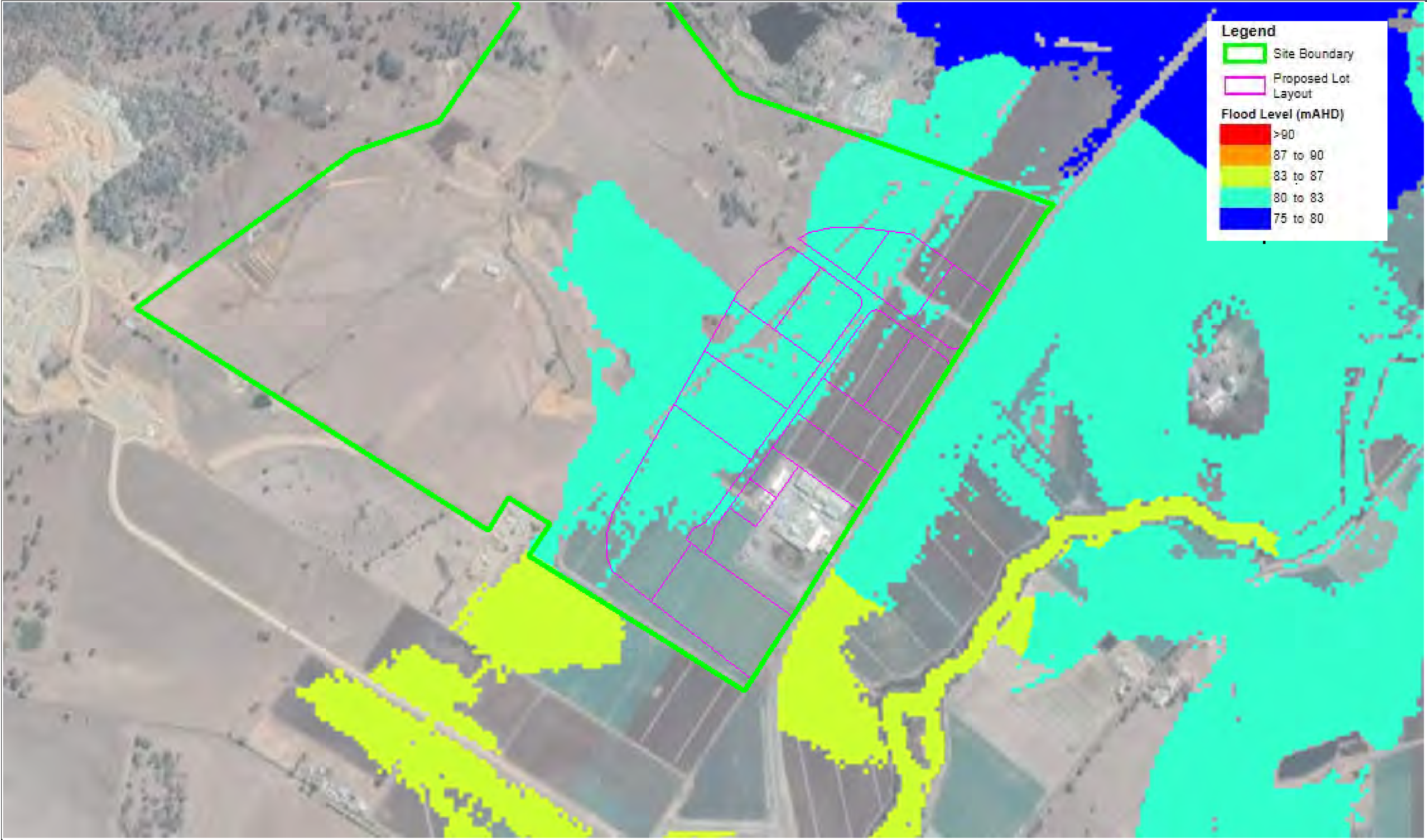
Existing 10% AEP

Appendix B-1

N
0 375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



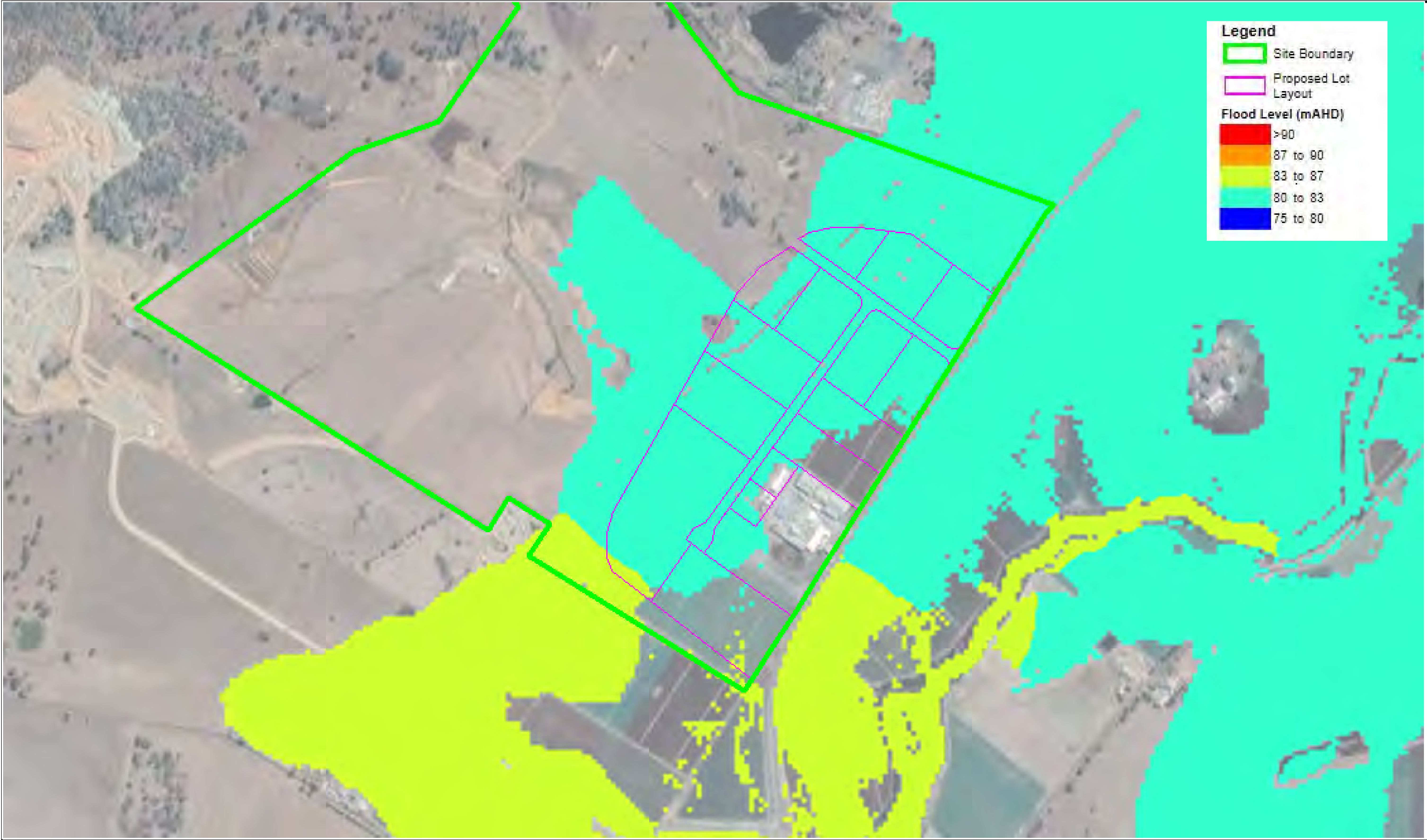
N
0
375
metres

Existing 5% AEP

Appendix B-2

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



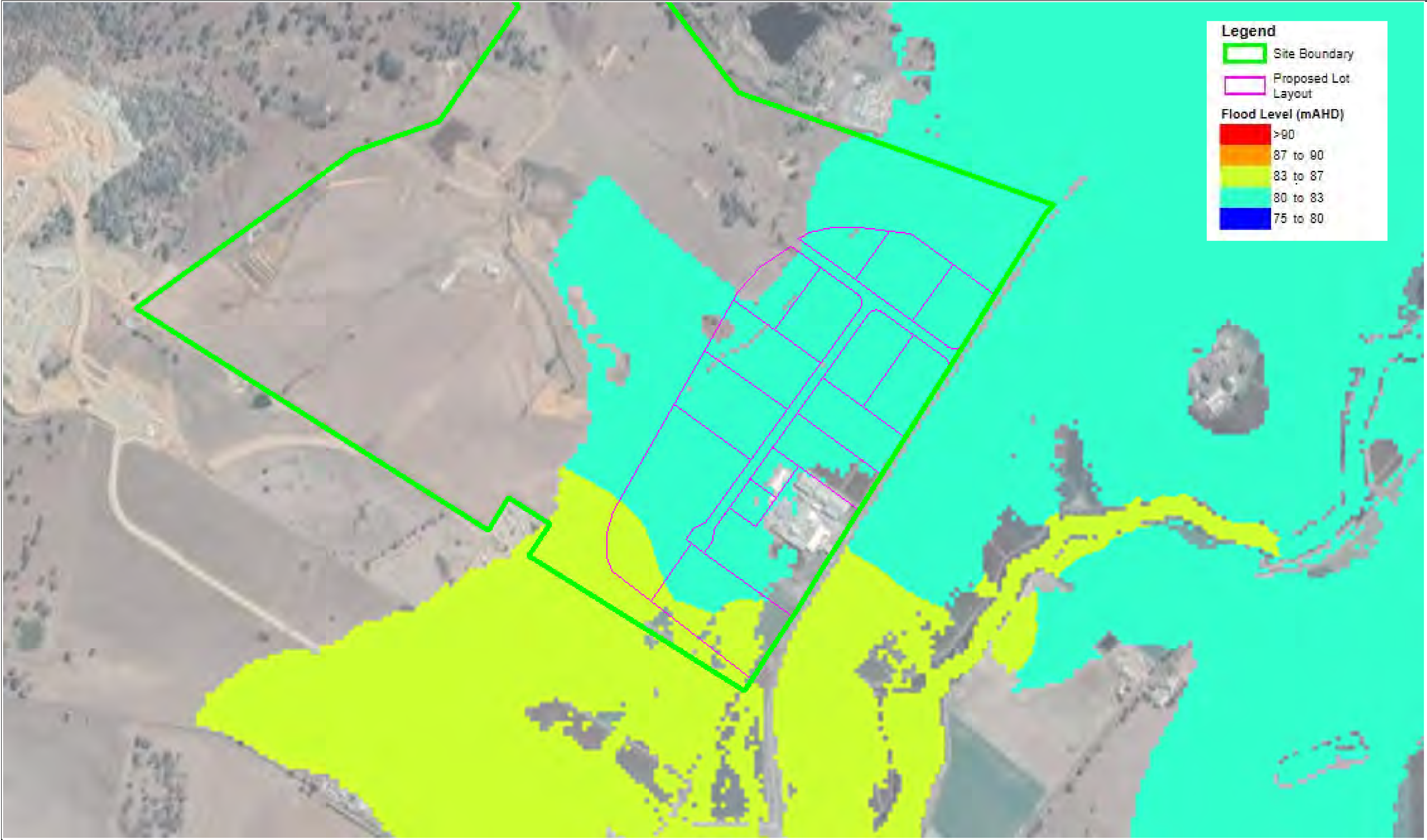
Existing 2% AEP

Appendix B-3

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



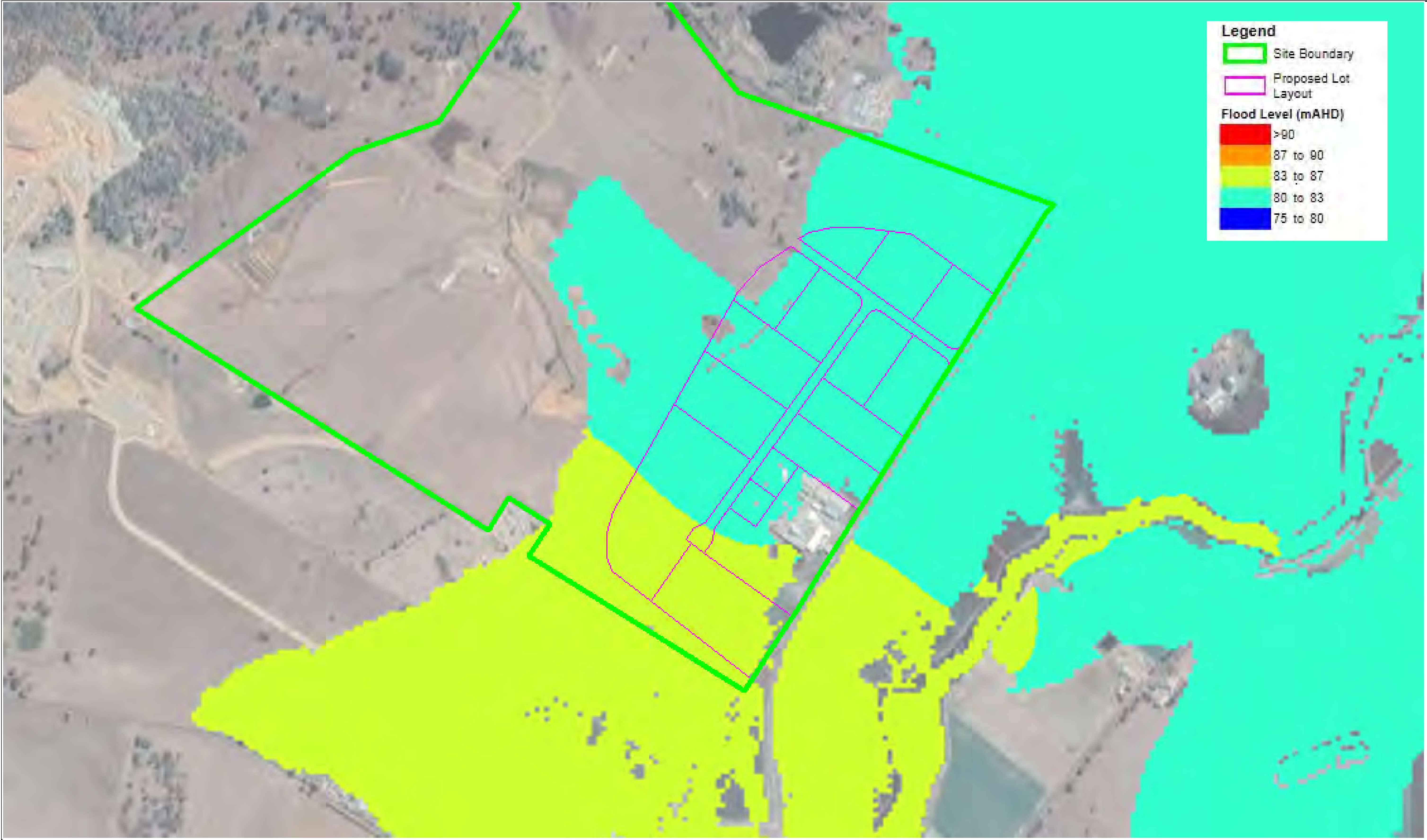
Existing 1% AEP

Appendix B-4

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



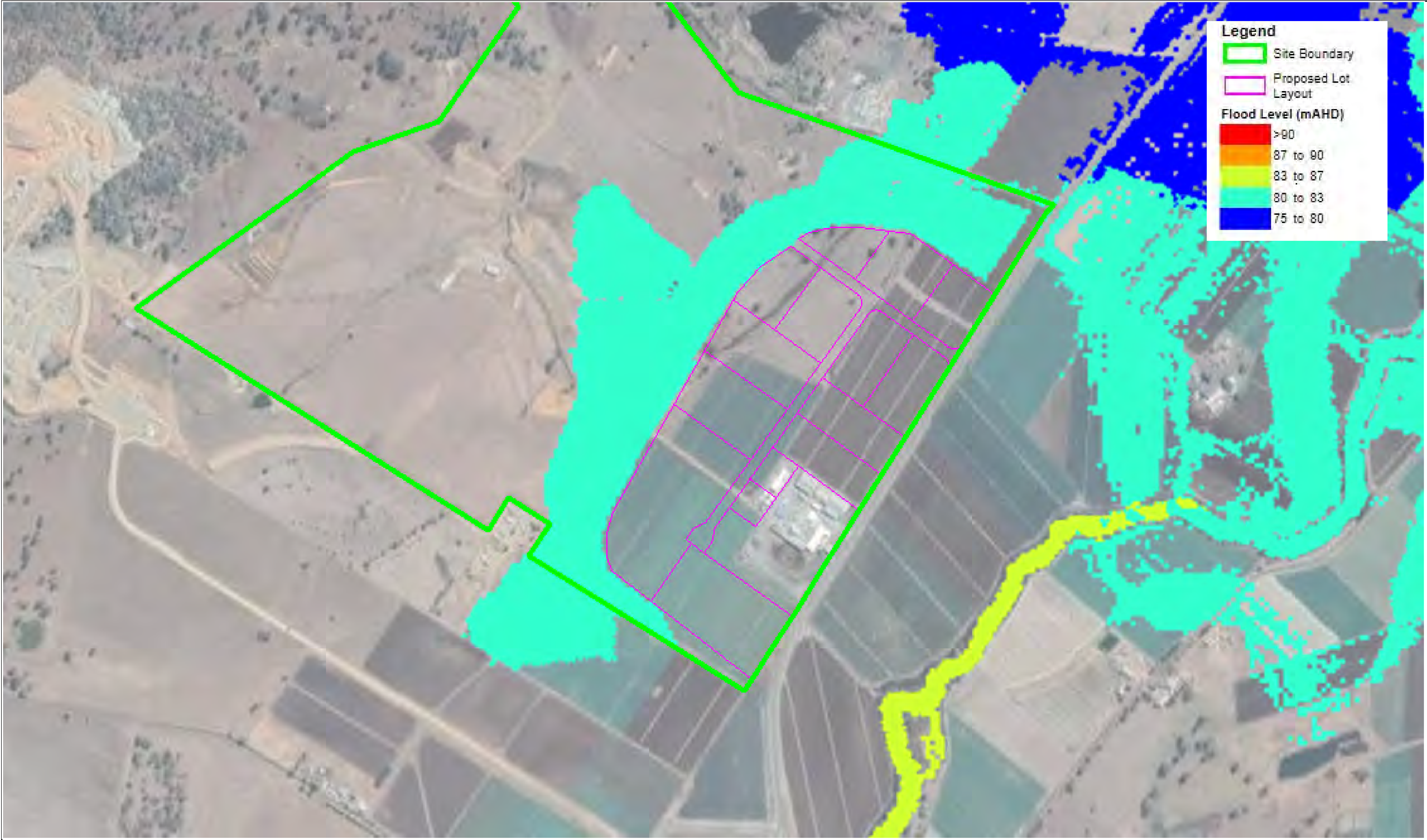
Existing 1% AEP CC

Appendix B-5

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



Developed 10% AEP

Appendix B-6

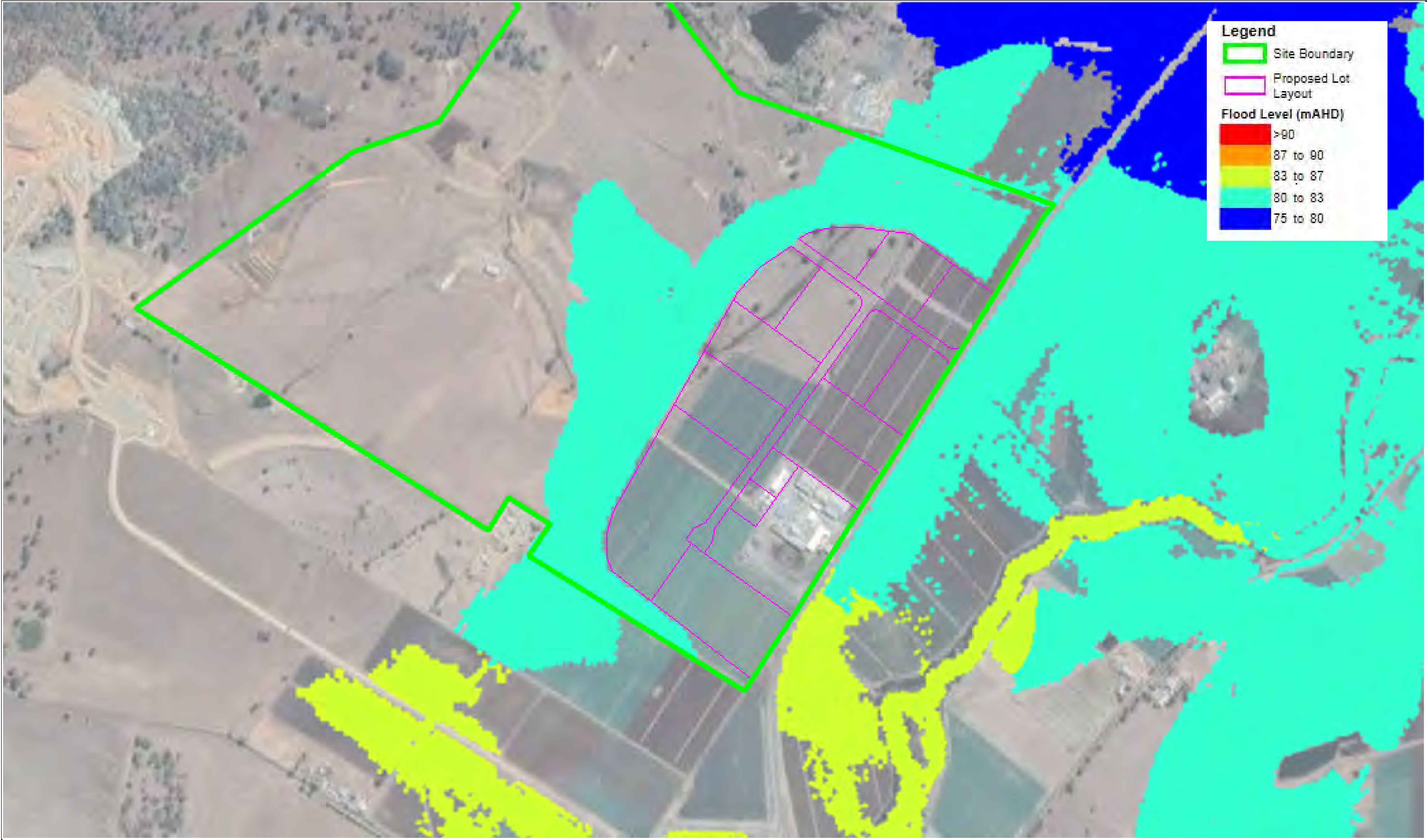
N

0 375

metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



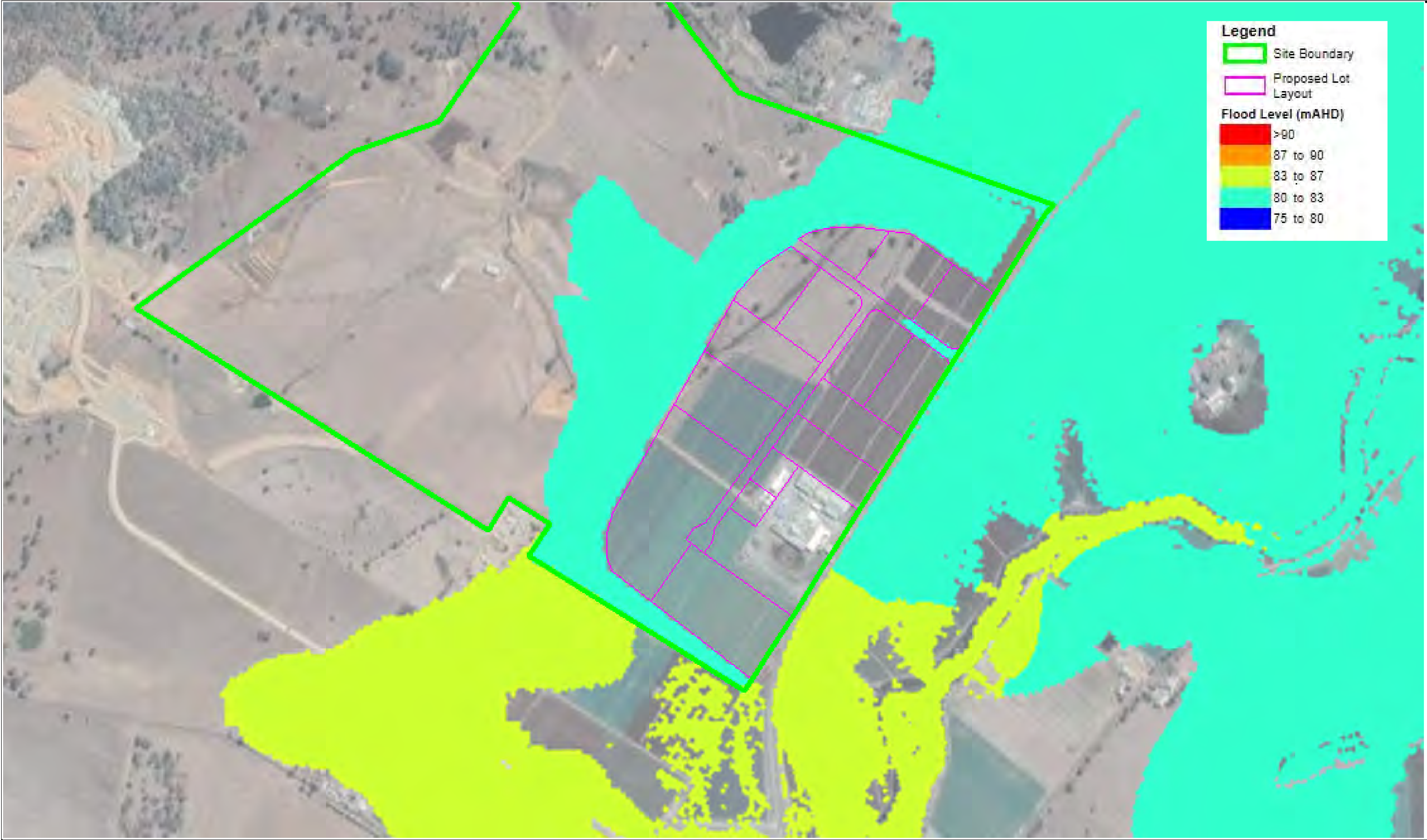
Developed 5% AEP

Appendix B-7

N
0 375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



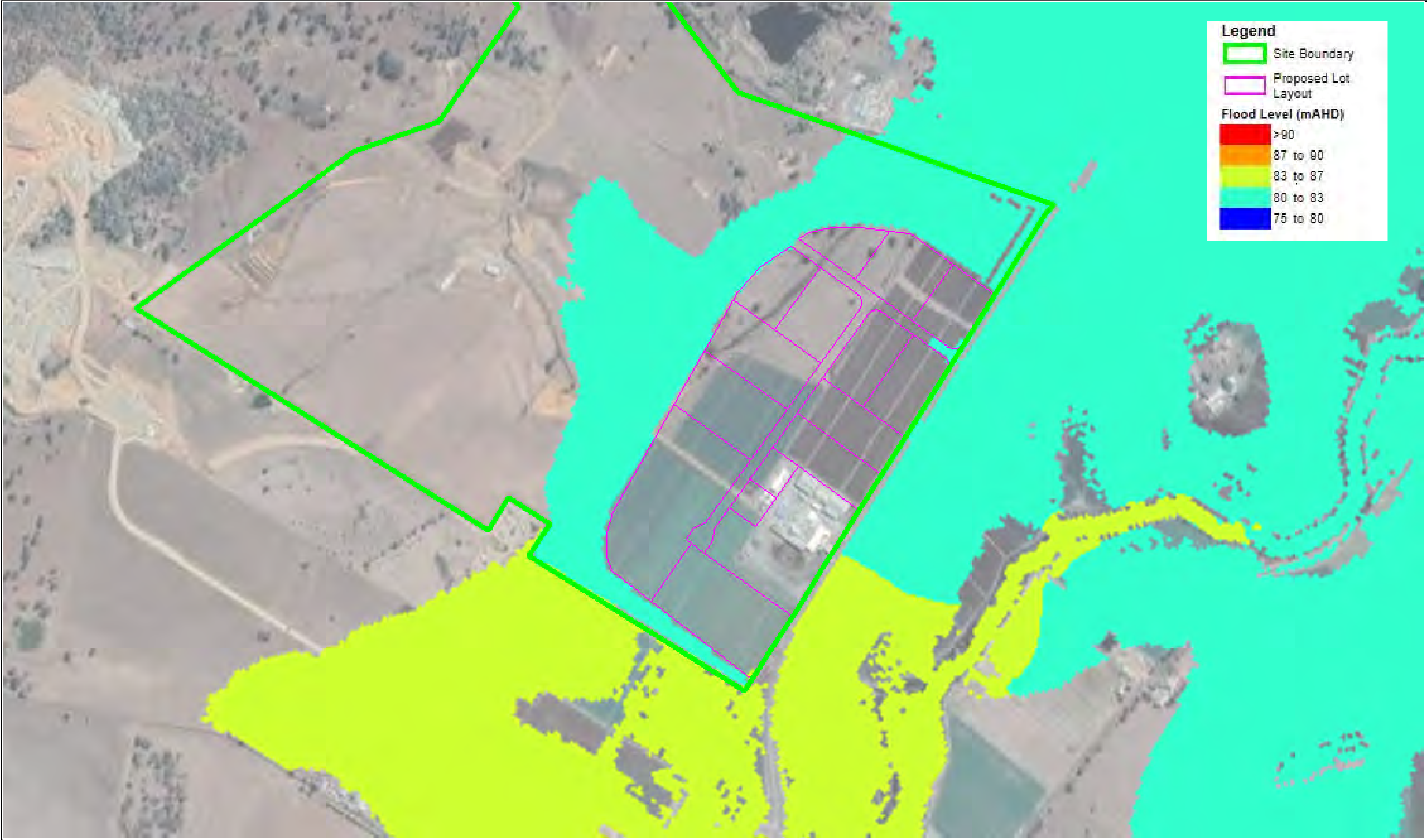
Developed 2% AEP

Appendix B-8

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



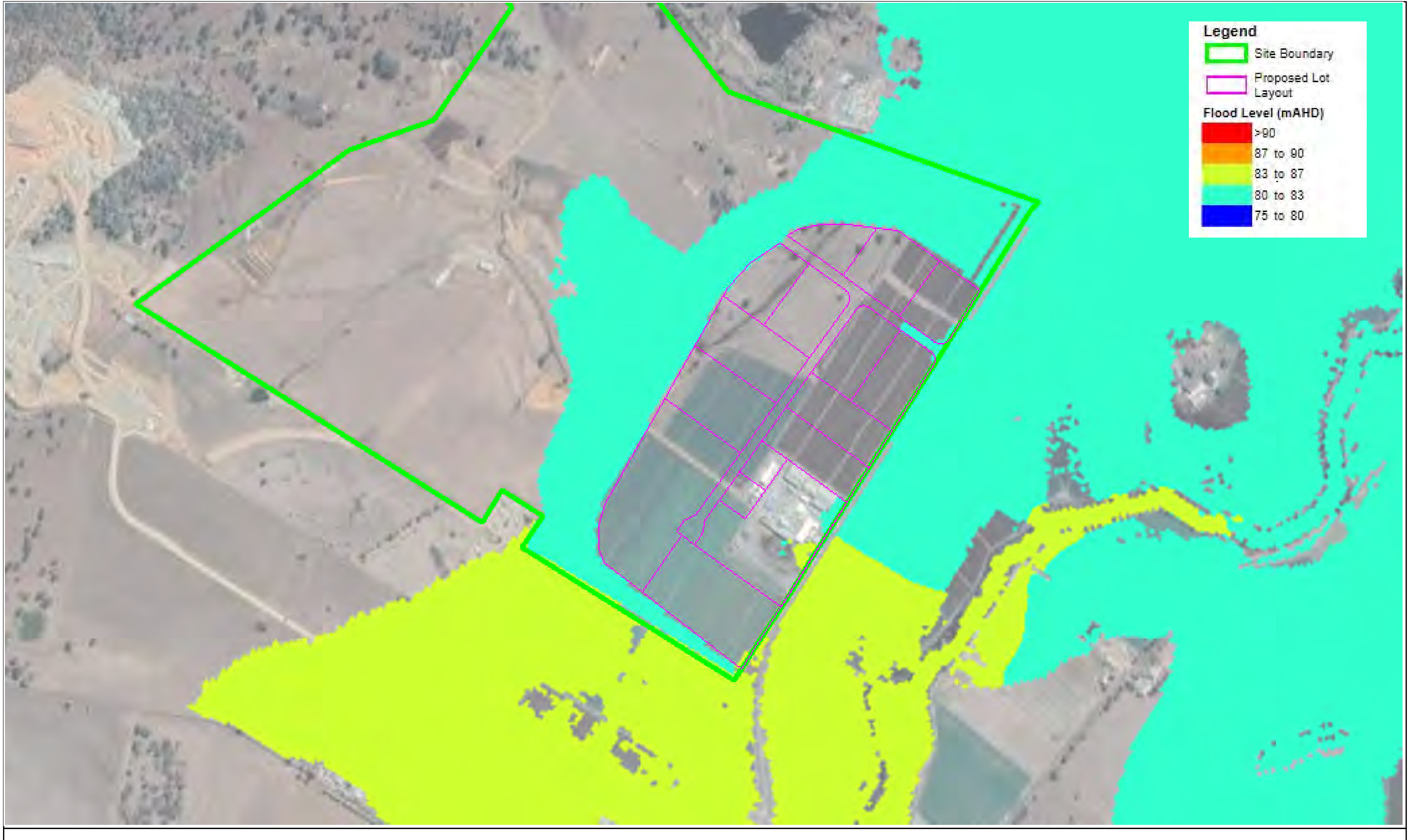
Developed 1% AEP

Appendix B-9

N
0 375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



Developed 1% AEP CC

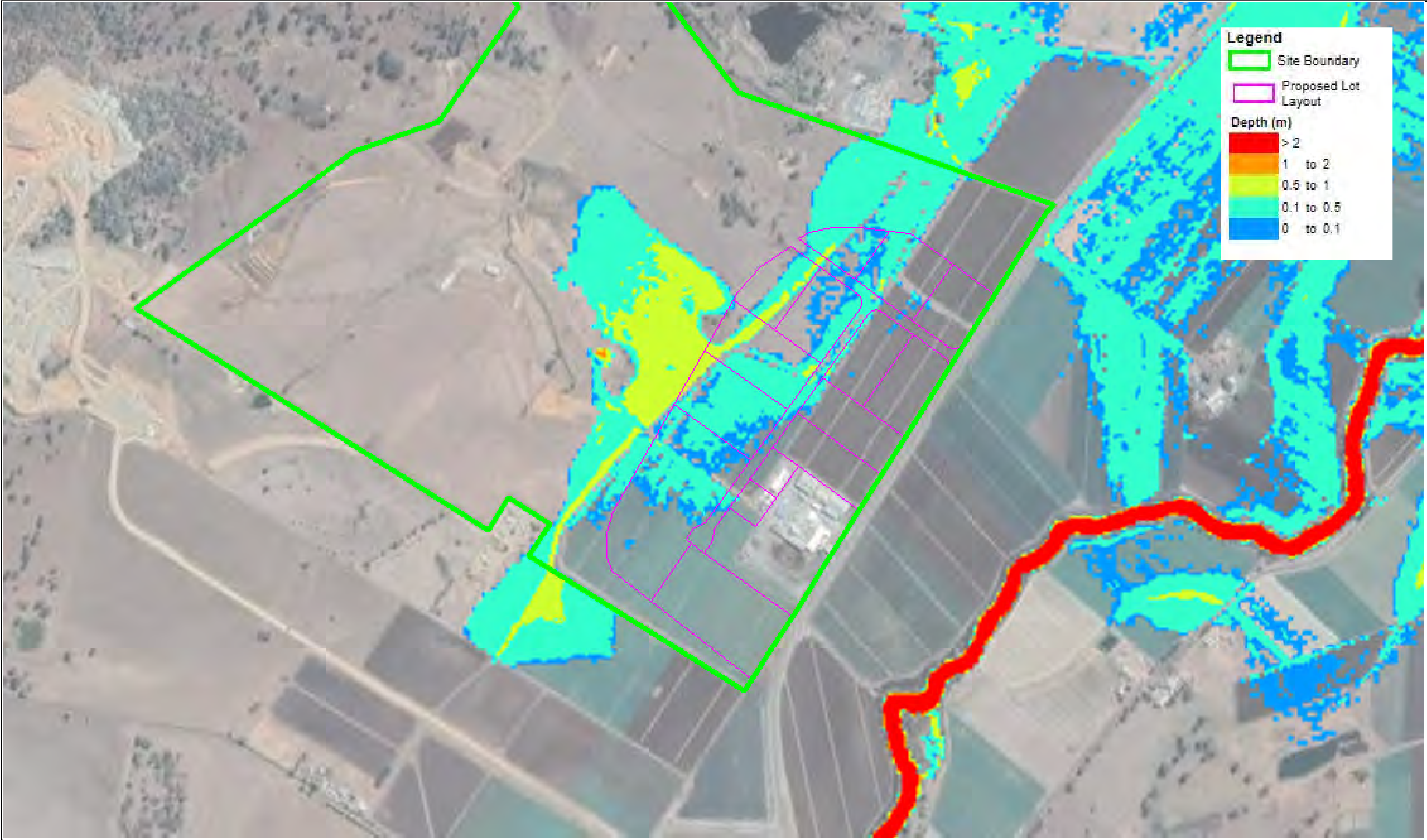
Appendix B-10

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

APPENDIX

C

WATER DEPTHS



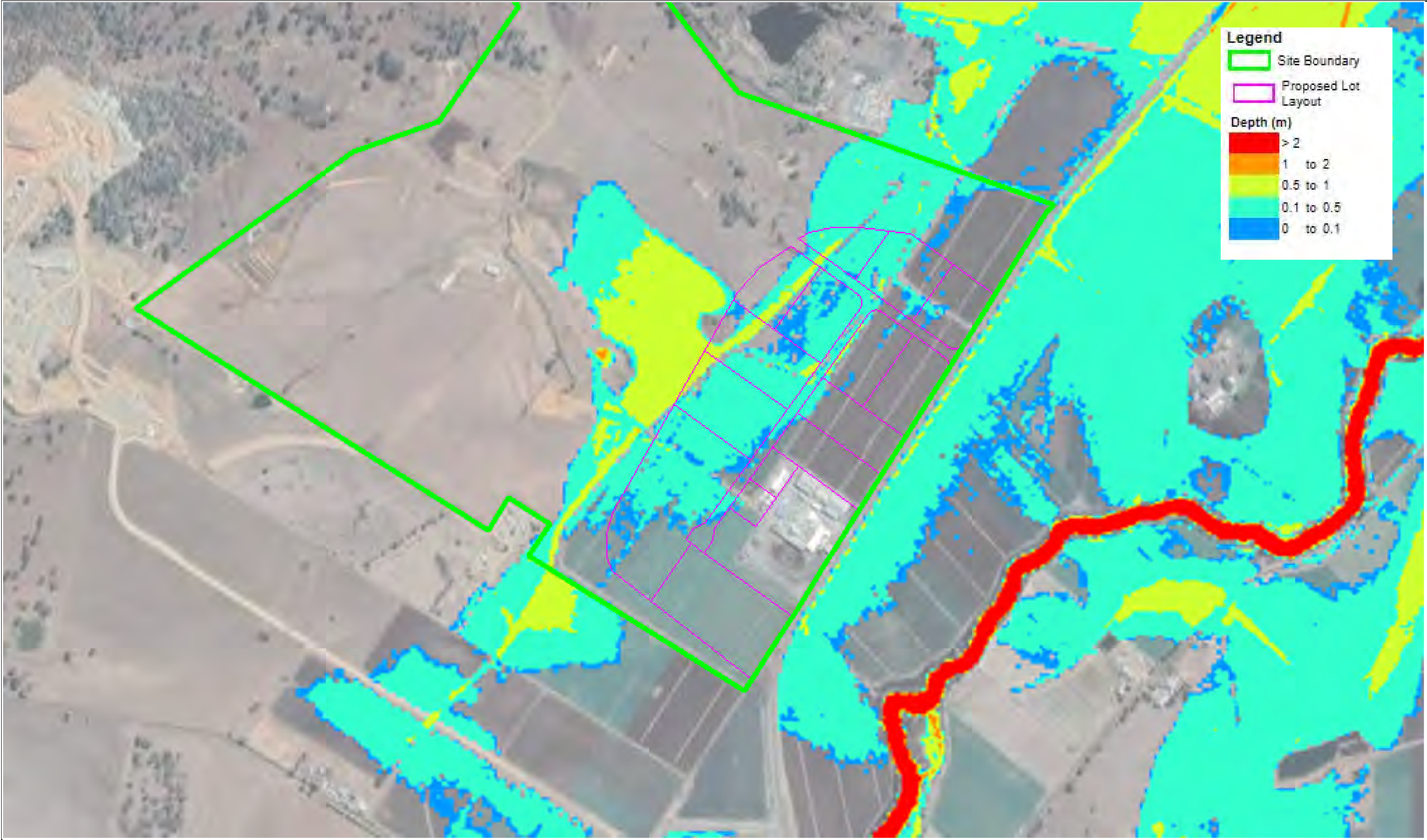
Existing 10% AEP

Appendix C-1

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



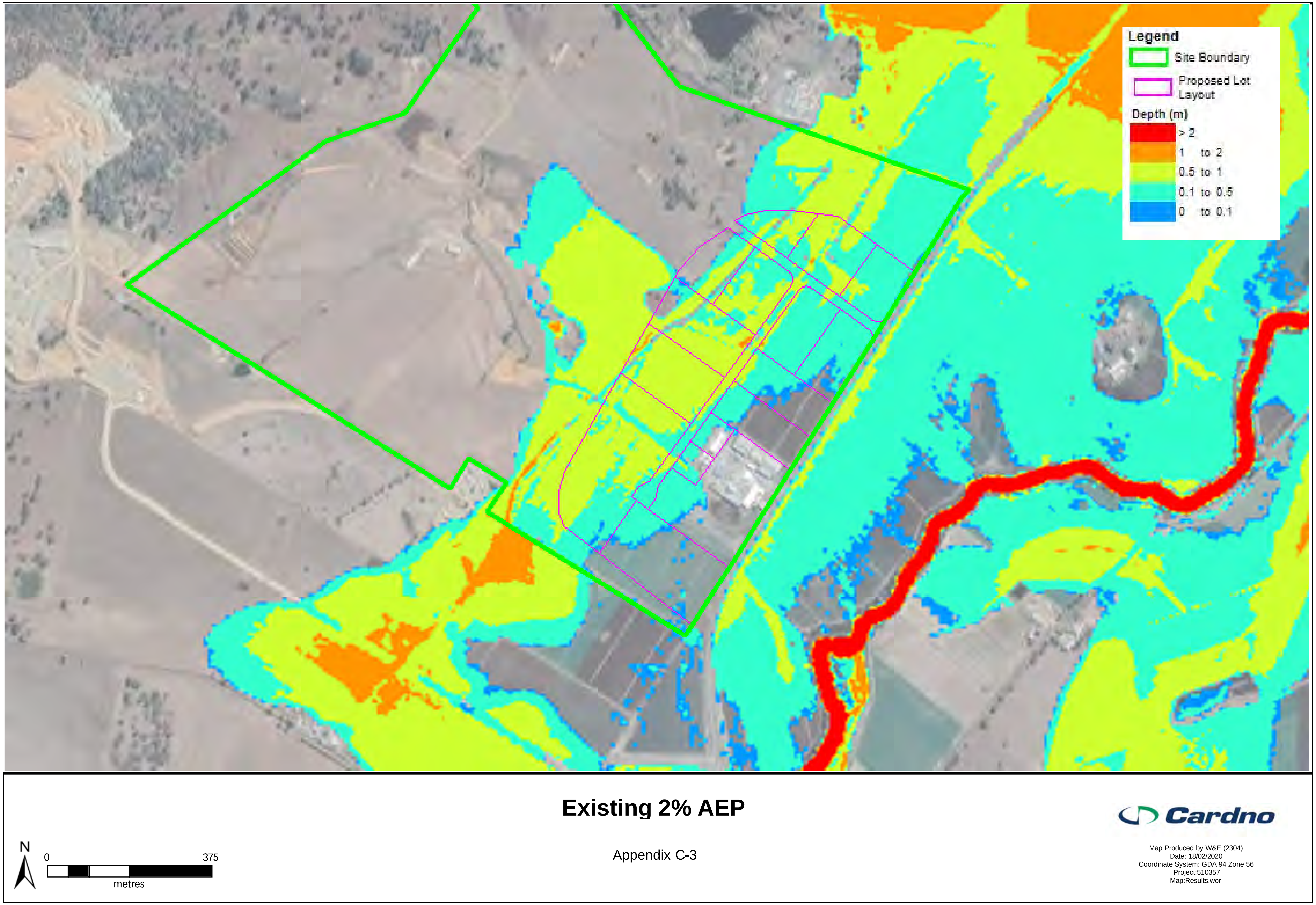
Existing 5% AEP

Appendix C-2

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

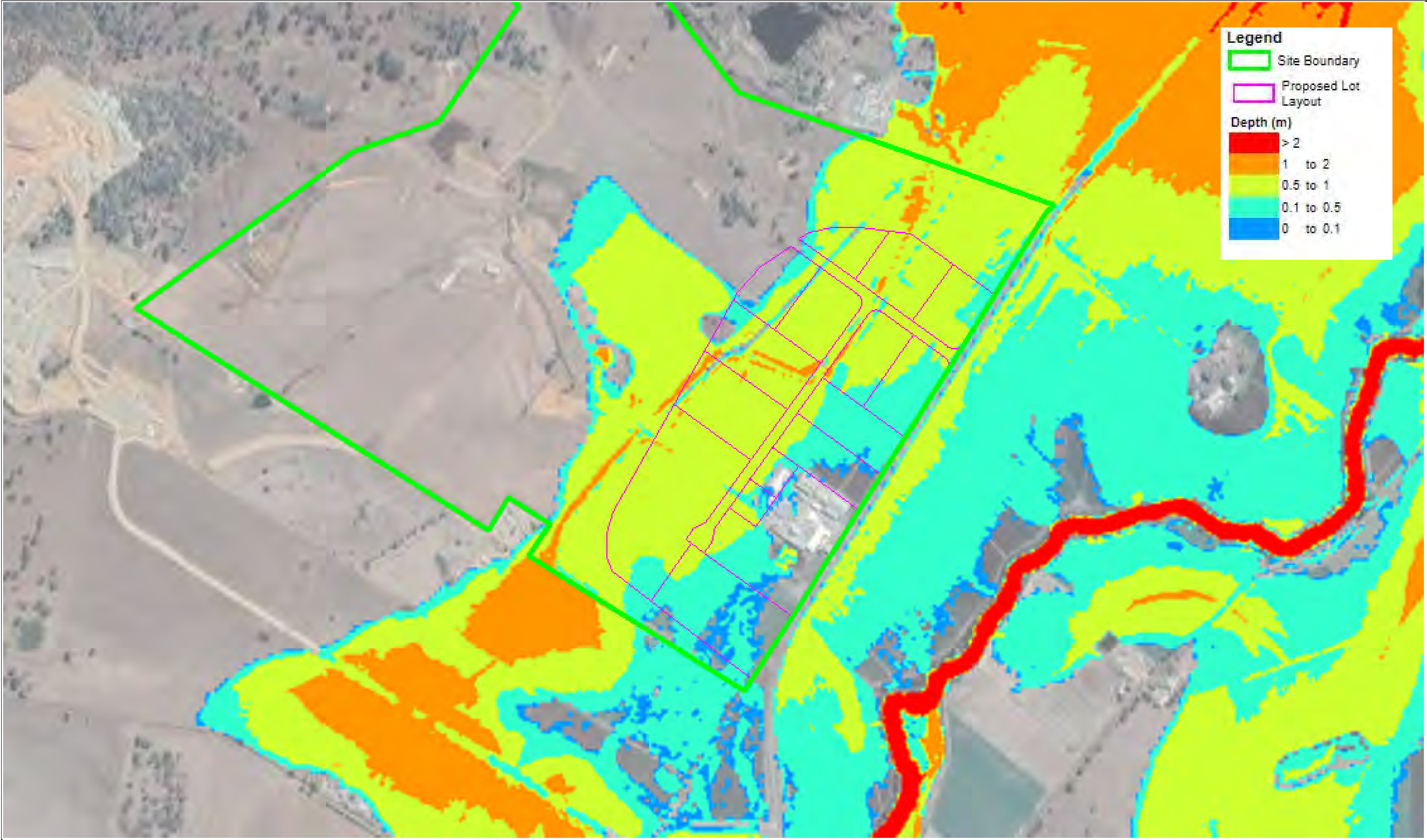


Existing 2% AEP

Appendix C-3

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



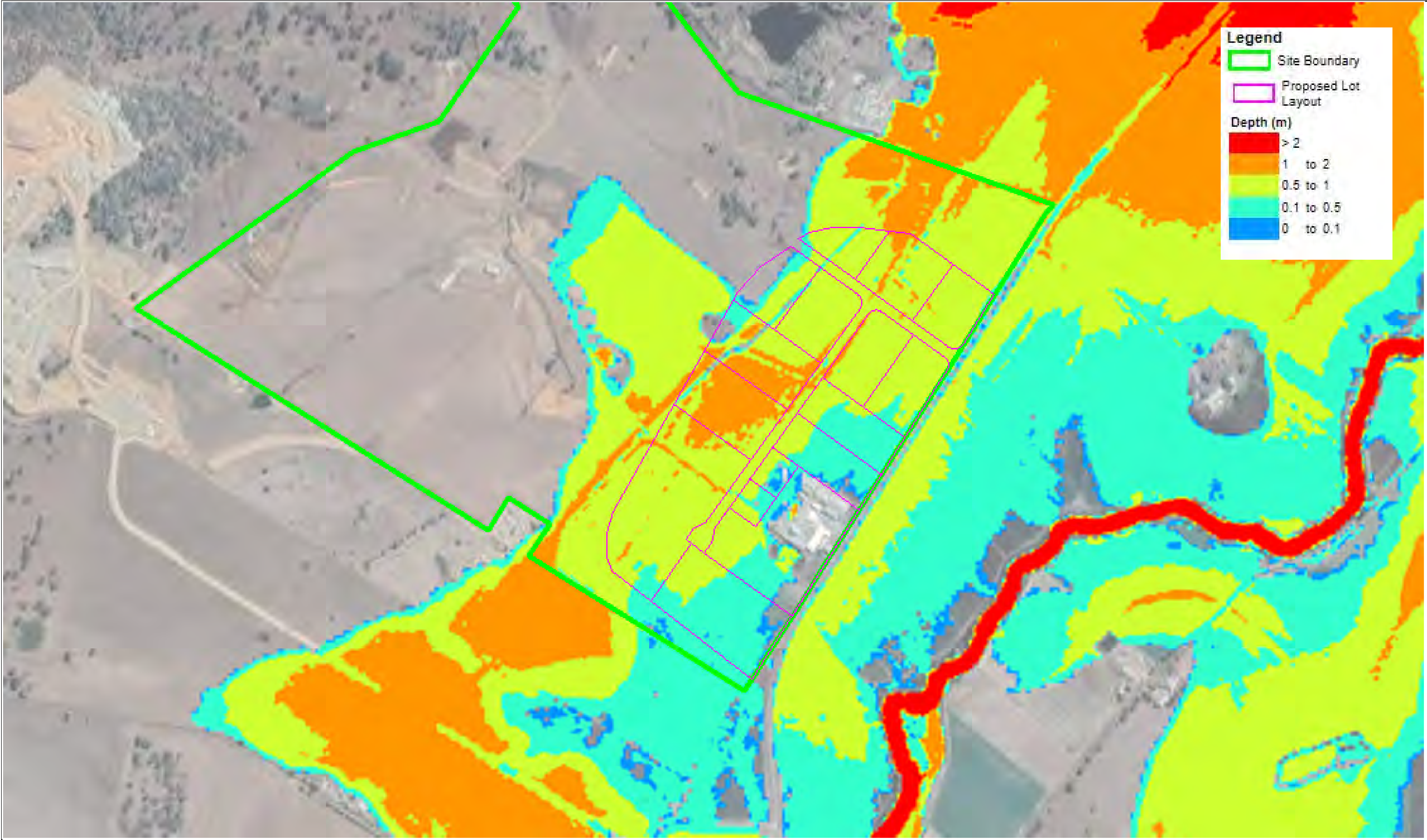
Existing 1% AEP

Appendix C-4

N
0
375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



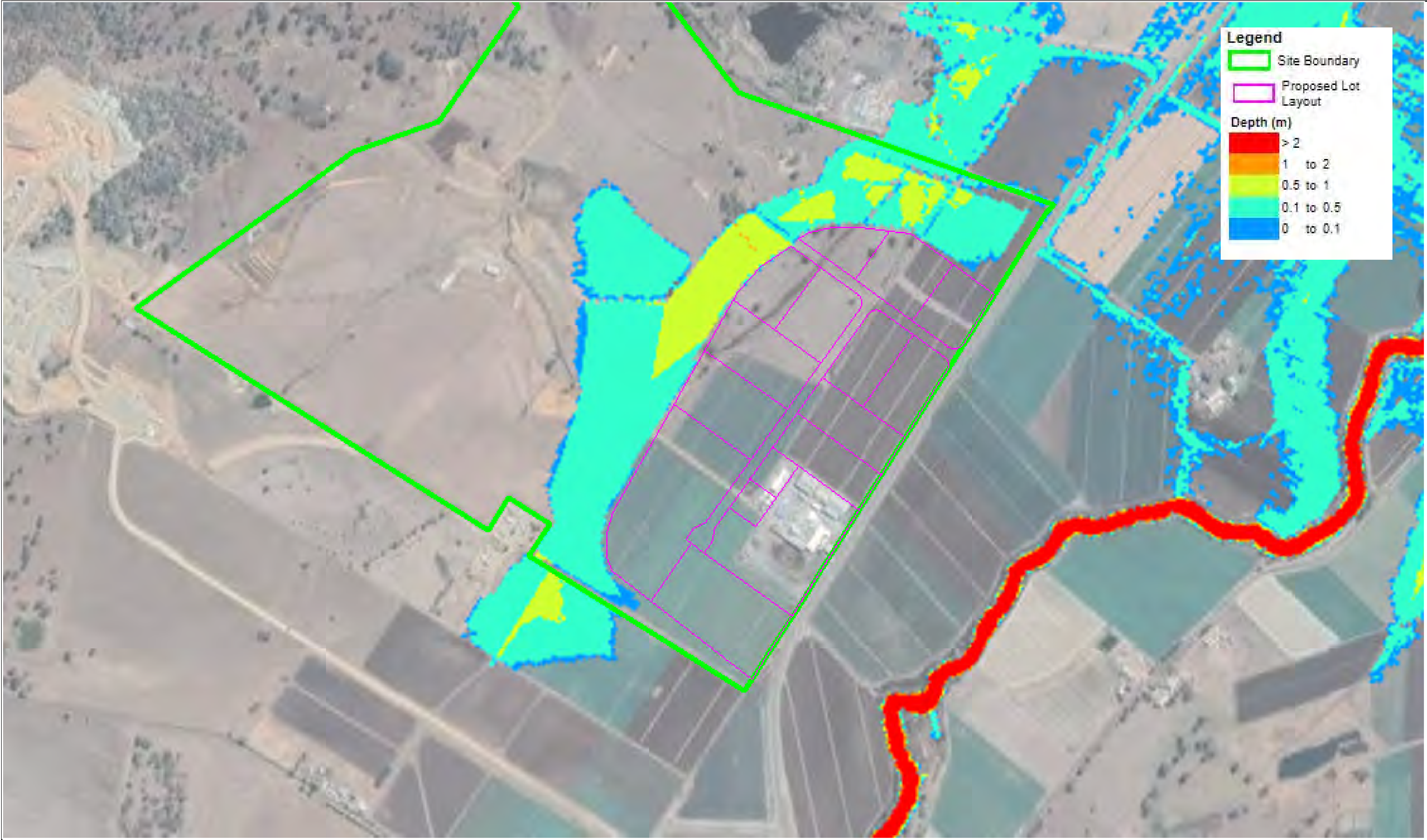
Existing 1% AEP CC

N
0 375
metres

Appendix C-5

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



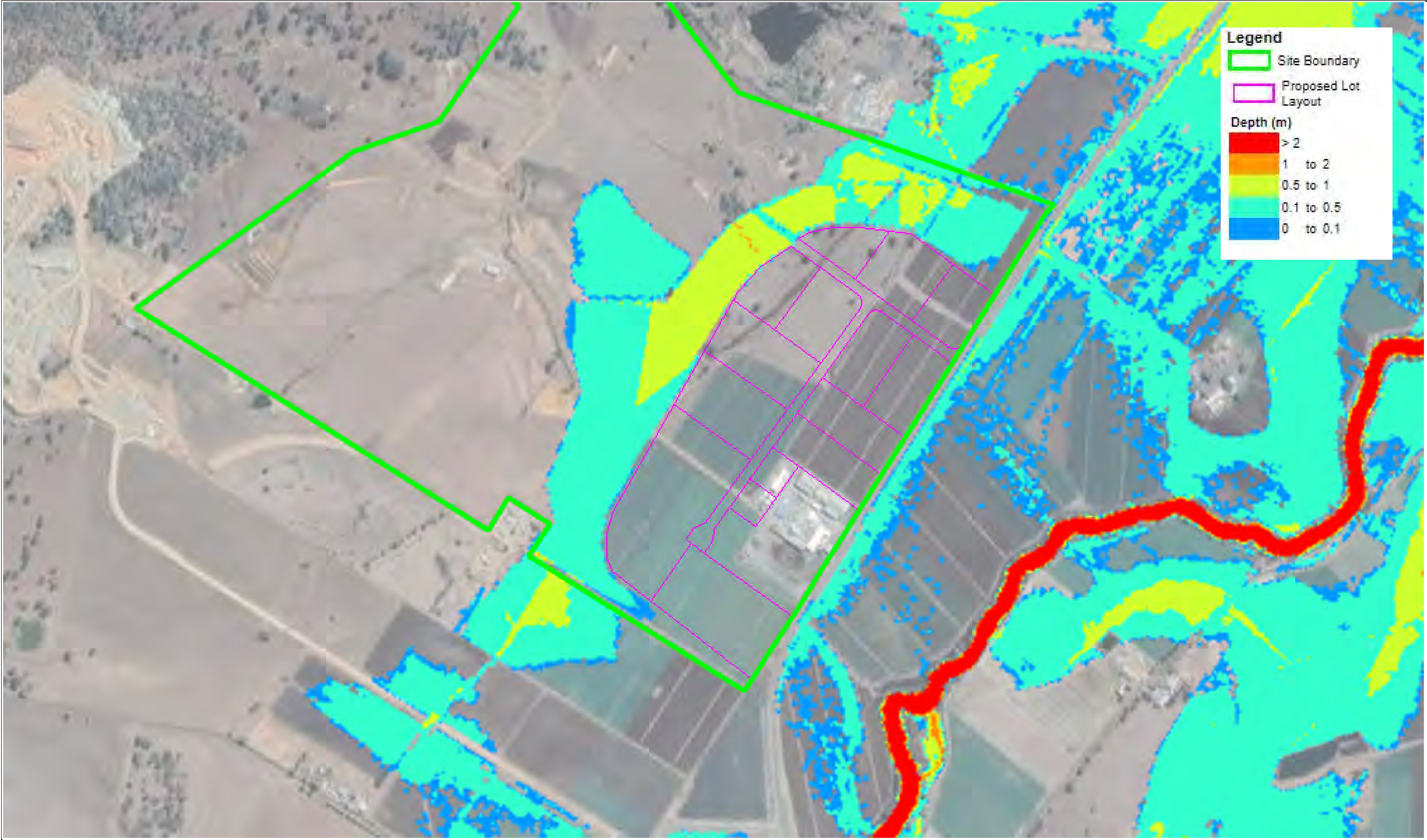
Developed 10Yr

Appendix C-6

N
0 375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



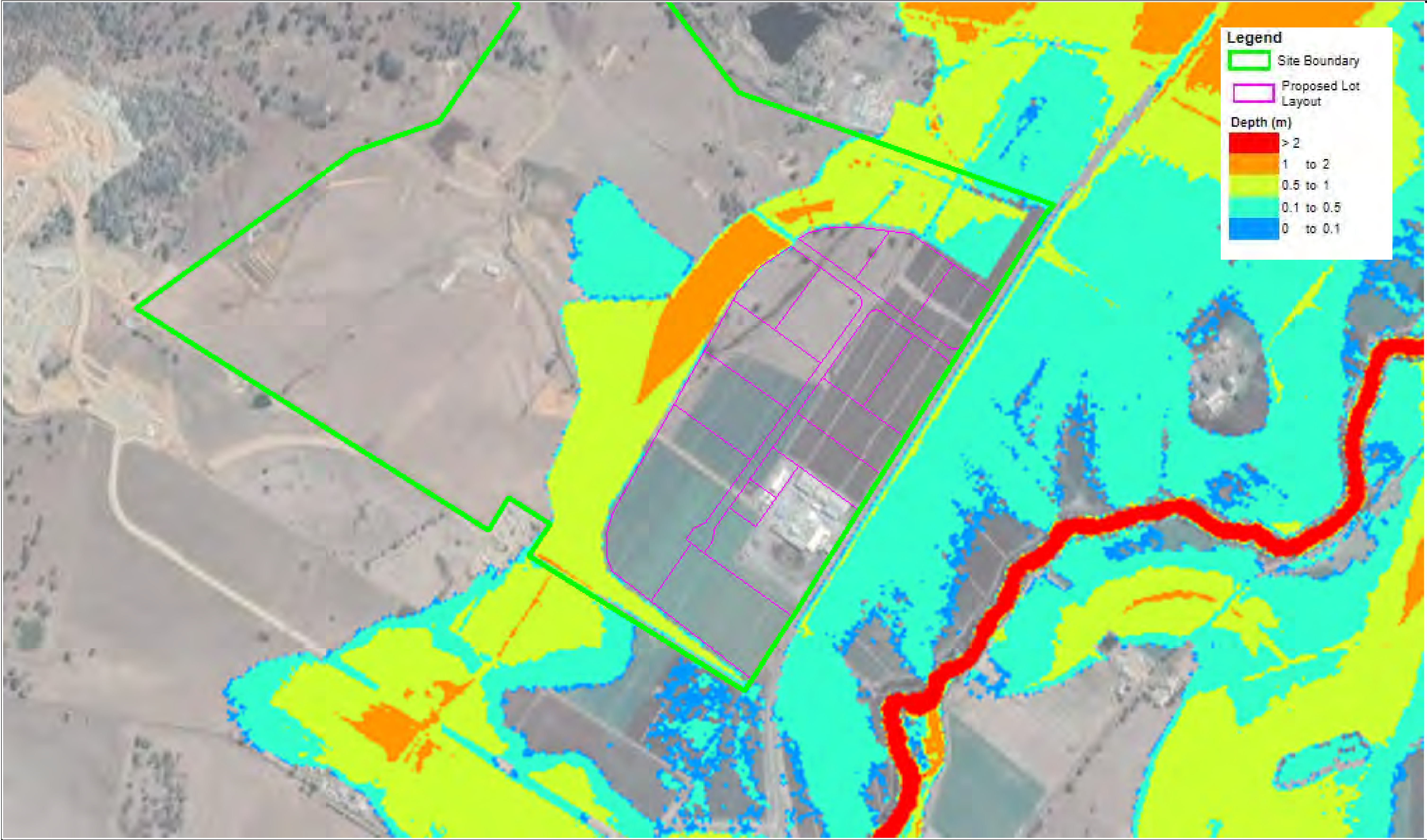
Developed 20yr

Appendix C-7

N
0 375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



Developed 50Yr

Appendix C-8

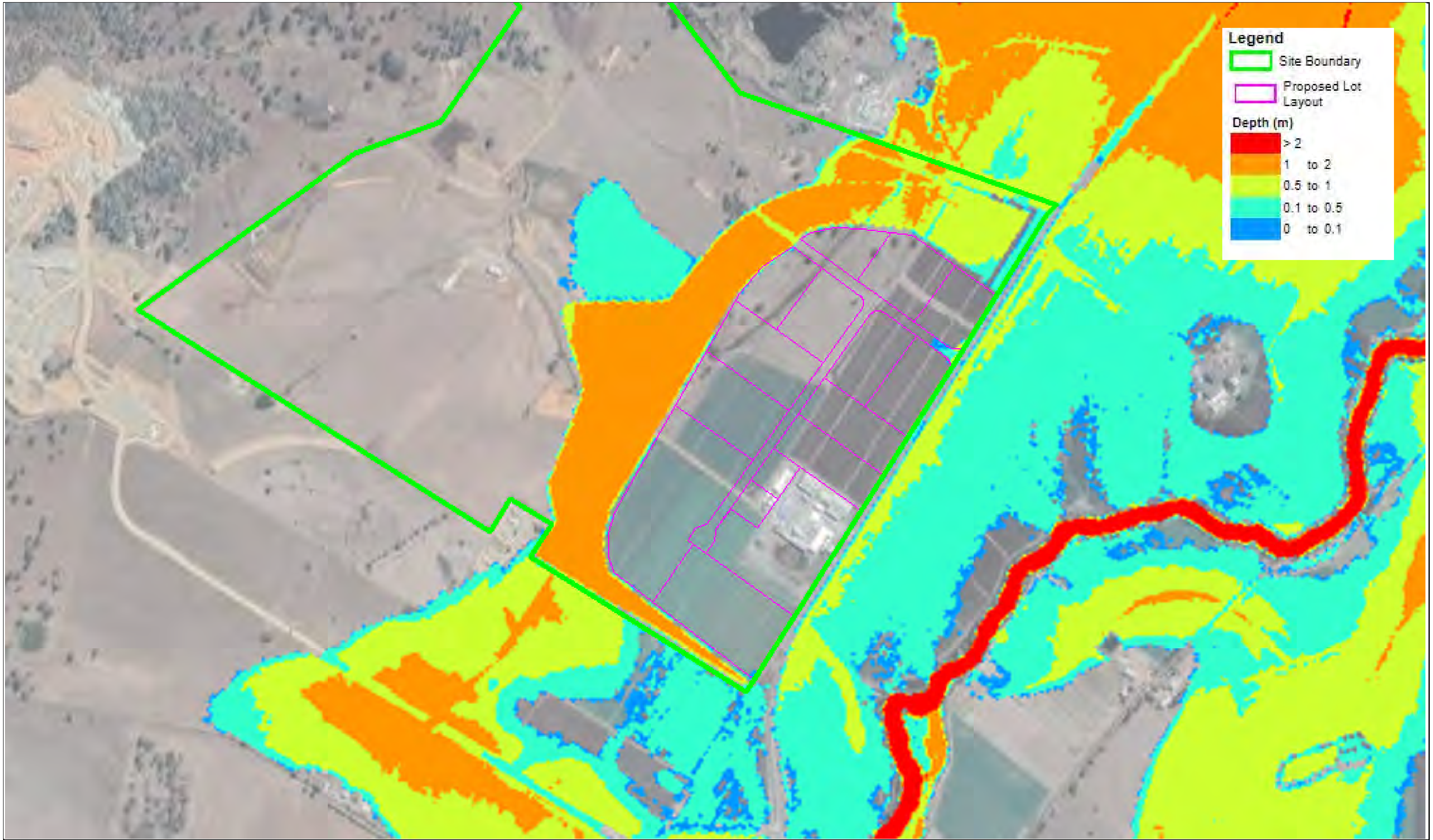
N

0 397.1

metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



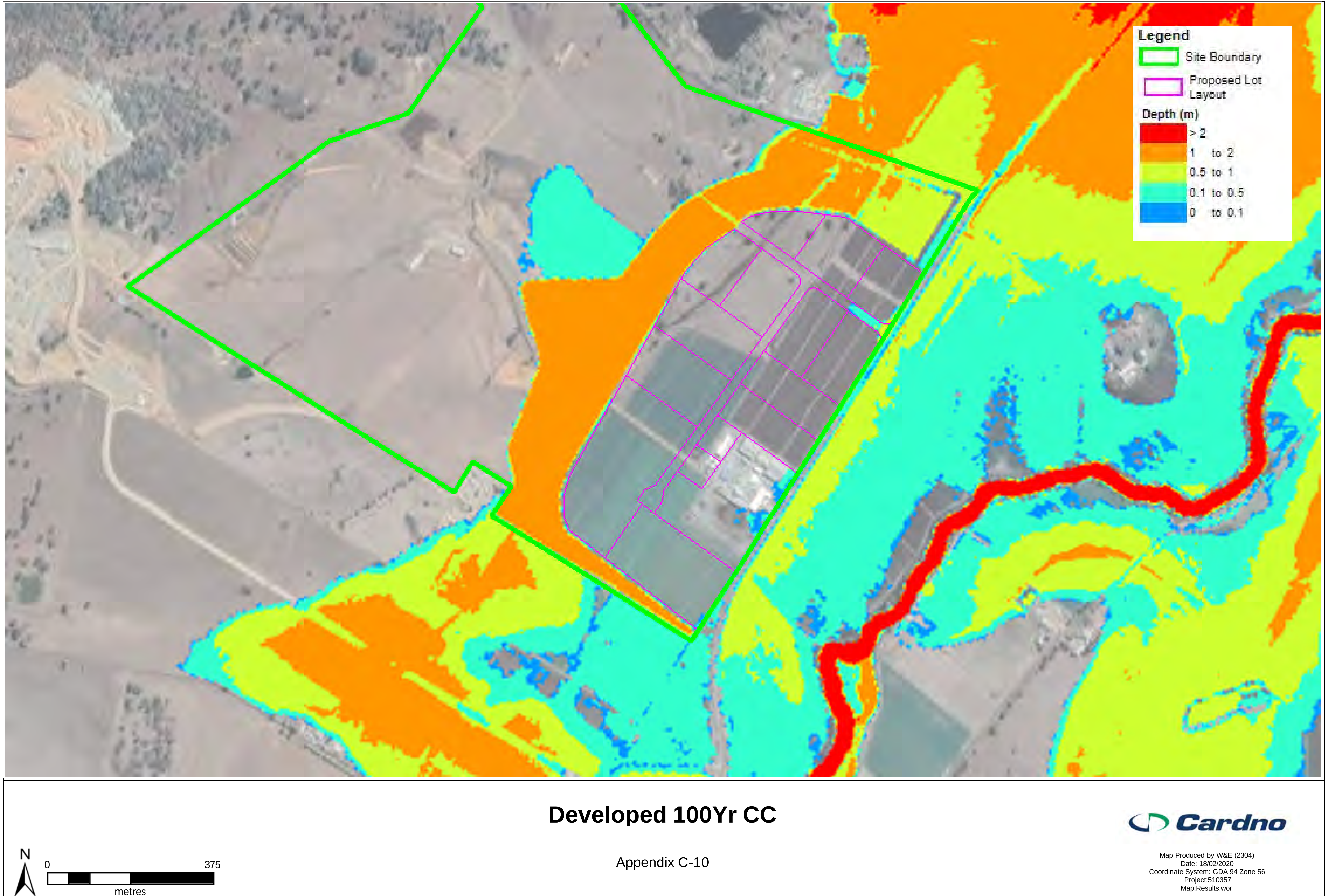
Developed 100Yr

Appendix C-9

N
0 375
metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



Developed 100Yr CC

Appendix C-10

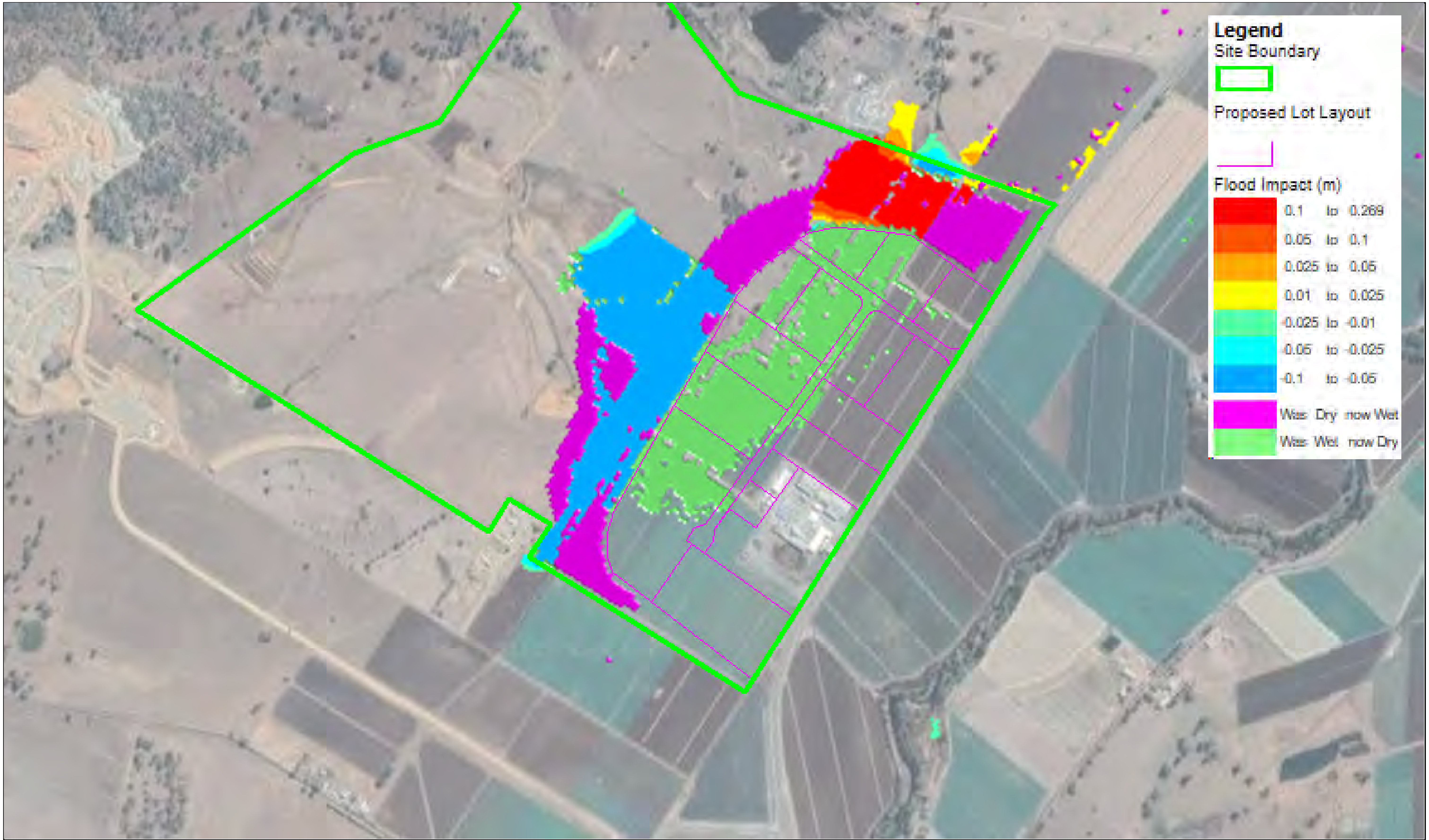
Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

APPENDIX

D

IMPACTS

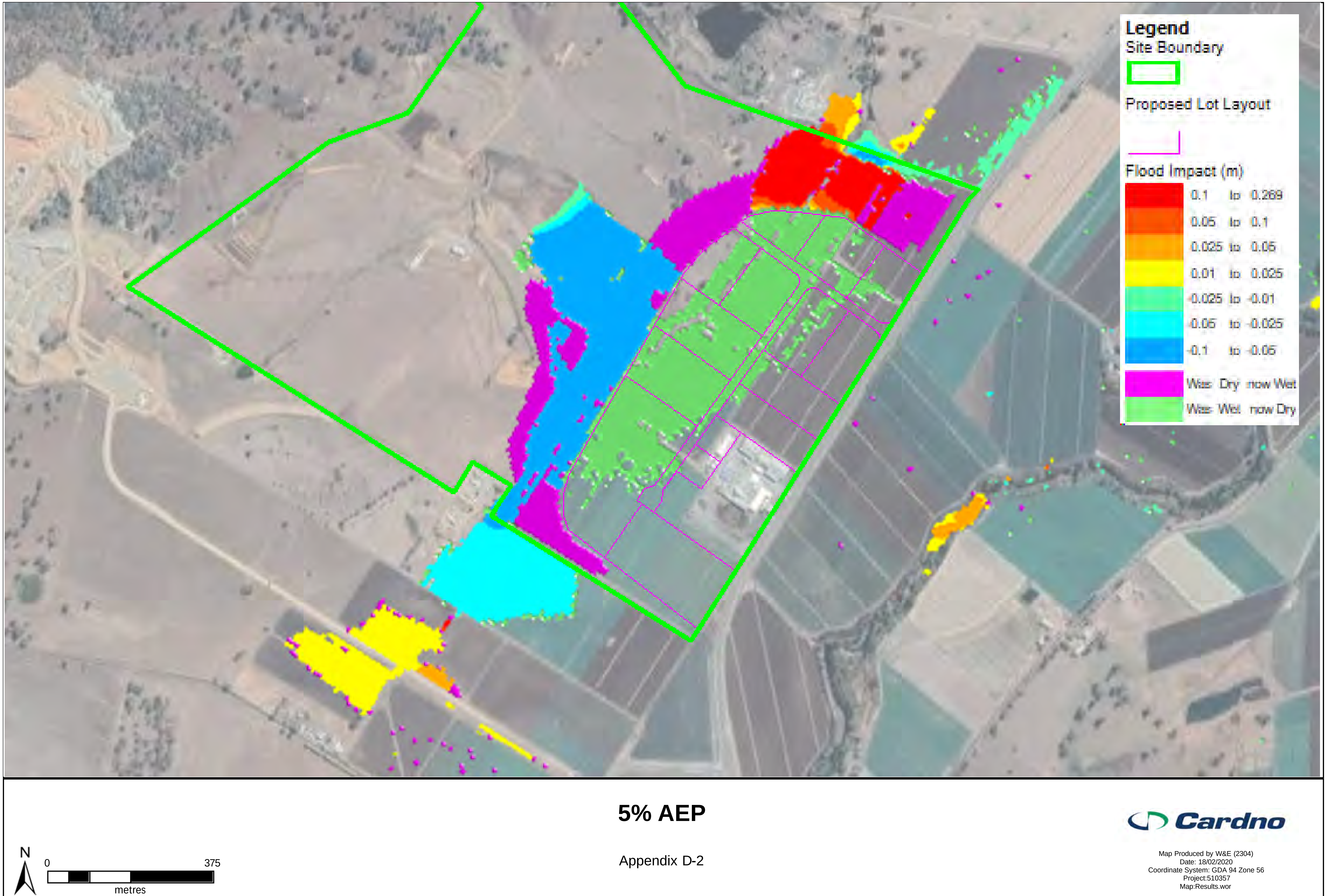


10% AEP

Appendix D-1

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



5% AEP

Appendix D-2

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

2% AEP

Appendix D-3

N

0 375

metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor

1% AEP

Appendix D-4

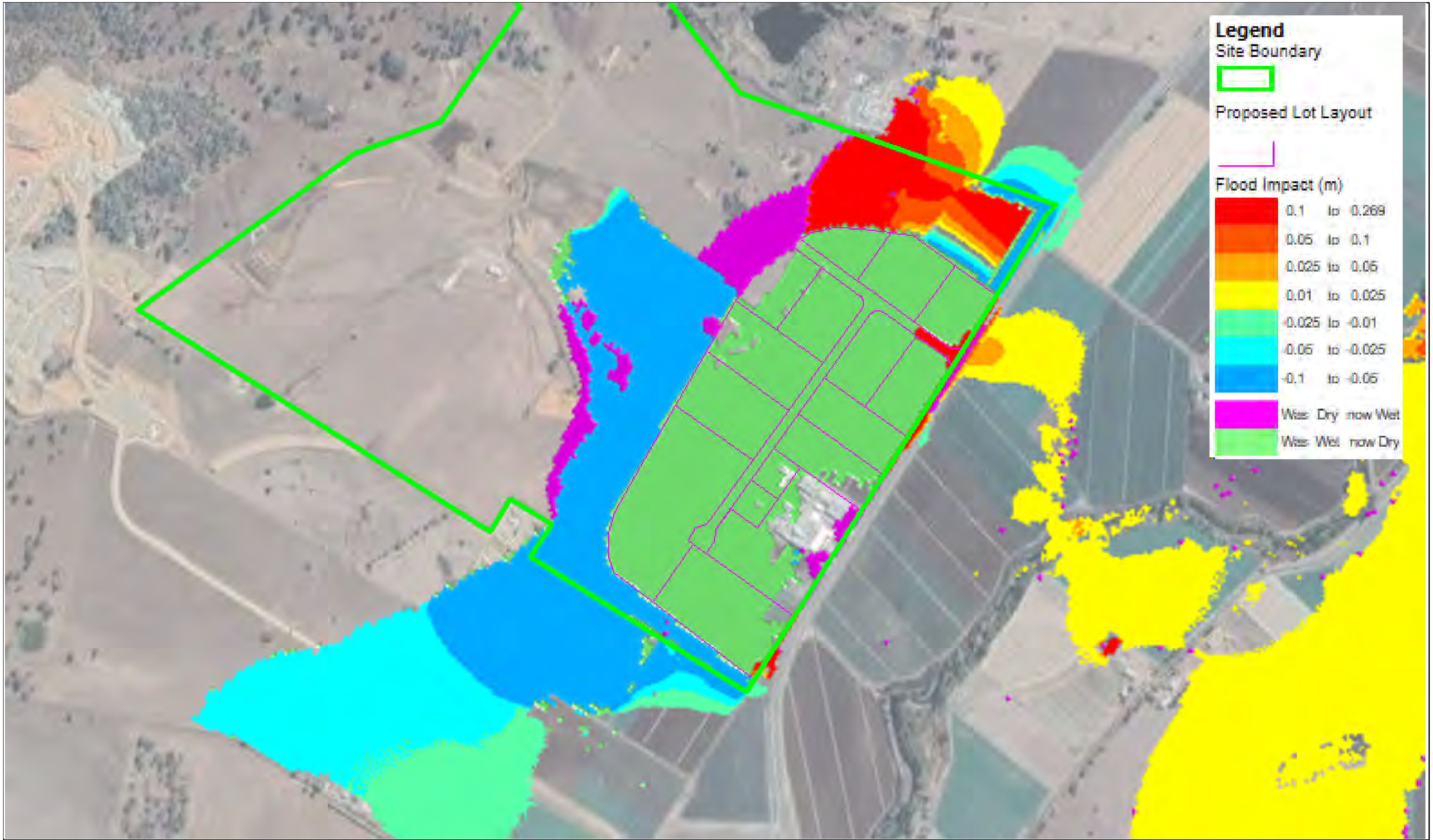
N

0 375

metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor



1% AEP CC

Appendix D-5

N

0 375

metres

Cardno

Map Produced by W&E (2304)
Date: 18/02/2020
Coordinate System: GDA 94 Zone 56
Project:510357
Map:Results.wor