TECHNICAL PAPER 04

Hydrology and Flooding Impact Assessment

Appendix A Flood model development

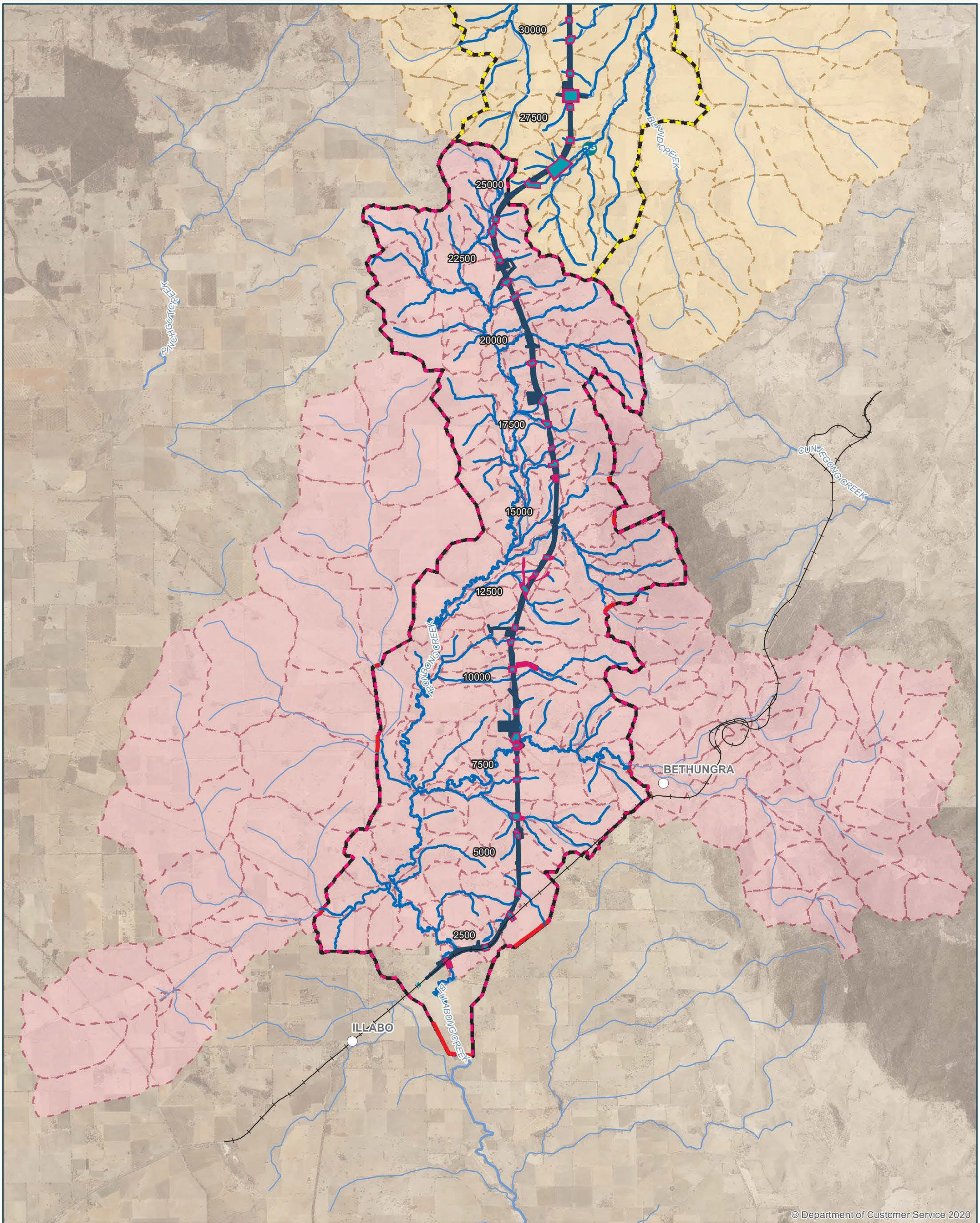
ILLABO TO STOCKINBINGAL ENVIRONMENTAL IMPACT STATEMENT

A.1 Modelling approach

Hydrologic and hydraulic models were developed for the proposal to provide an understanding of existing flood conditions along the proposal alignment and to enable flood impacts of the proposal to be assessed. The following process was used in development of proposal flood modelling:

- Hydrologic (XP-RAFTS) models were developed primarily to relate rainfall falling over the catchment areas upstream of the proposal into runoff generated from each sub-catchment. For larger catchment areas, flow routing through the catchments has also been represented within the hydrologic models to enable larger inflow hydrographs to be generated and extracted for use in the hydraulic models. For the smaller catchments, closer to the proposal, catchment flow hydrographs were applied directly to the locations generated within the hydraulic model.
- Hydraulic (TUFLOW) models were built to represent how flow travelled through the catchments/ watercourse/ floodplain areas. The hydraulic models were used to establish an understanding of existing flood extents, depths, velocity and flow distribution. The models were also used to represent the proposal conditions to inform proposal design and so that any proposal flood impacts could be identified and understood.

Due to the modelling approach adopted for the proposal, with TUFLOW being the primary tool used to represent routing of flow through the catchment areas, calibration of hydrologic model parameters also required use of the hydraulic models. Further details of each model and the calibration adopted are provided below. Figure A.1 shows the models extents and layouts and Figure A.2 shows the grid extents as discussed below.



ILLABO TO STOCKINBINGAL Model Extents

0 0.5 1 1.5 2 Kilometers

Coordinate System: GDA 1994 MGA Zone 55

Date: 8/5/2022
Paper: A3
Author: IRDJV
Scale: 1:100,000
Data Sources: IRDJV, ARTC, LPI

- Proposal Site
- Inflows
- Boundaries
- Major Watercourse
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- Existing Rail
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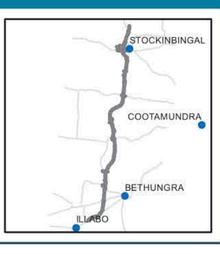

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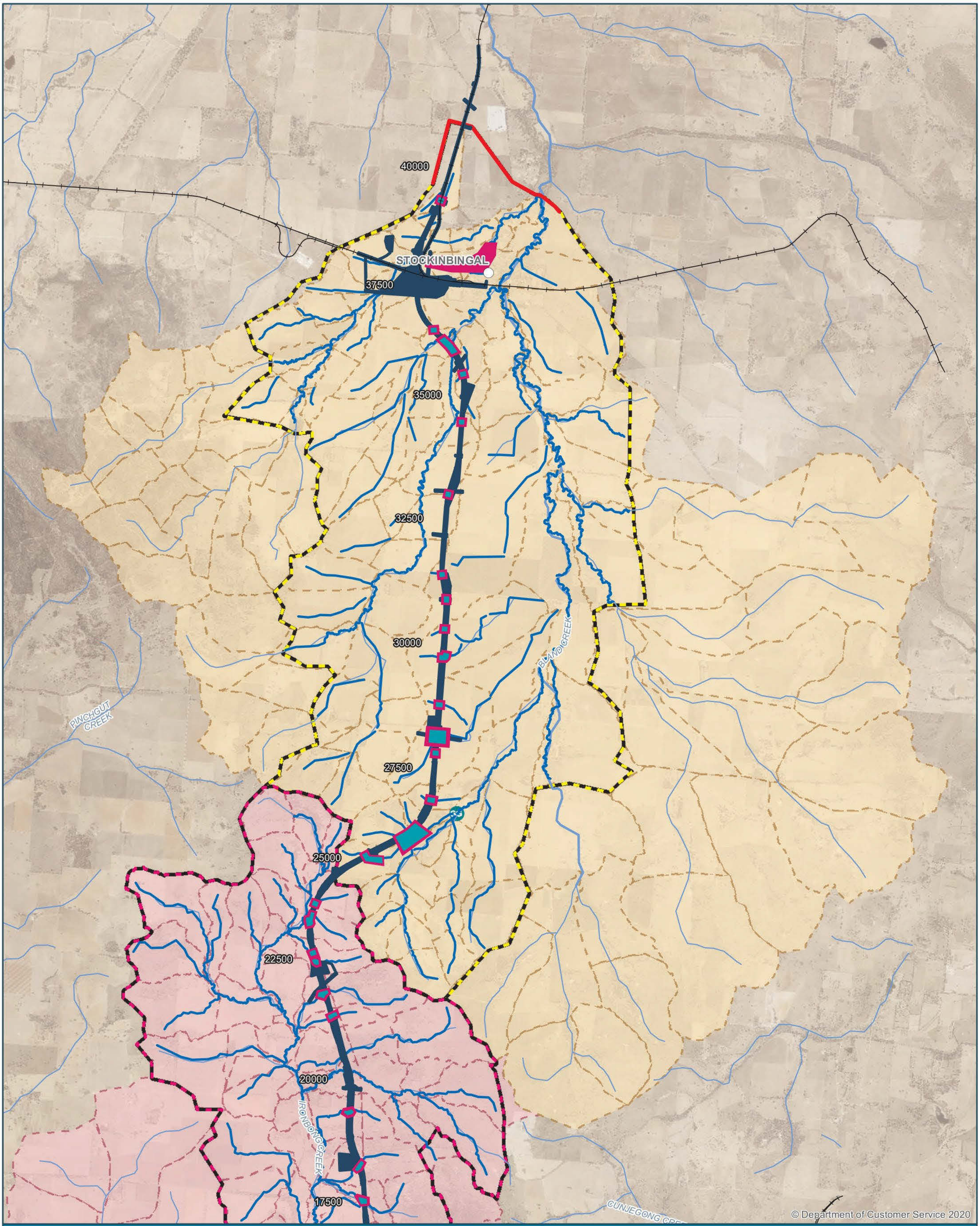

ILLABO TO STOCKINBINGAL Model Extents

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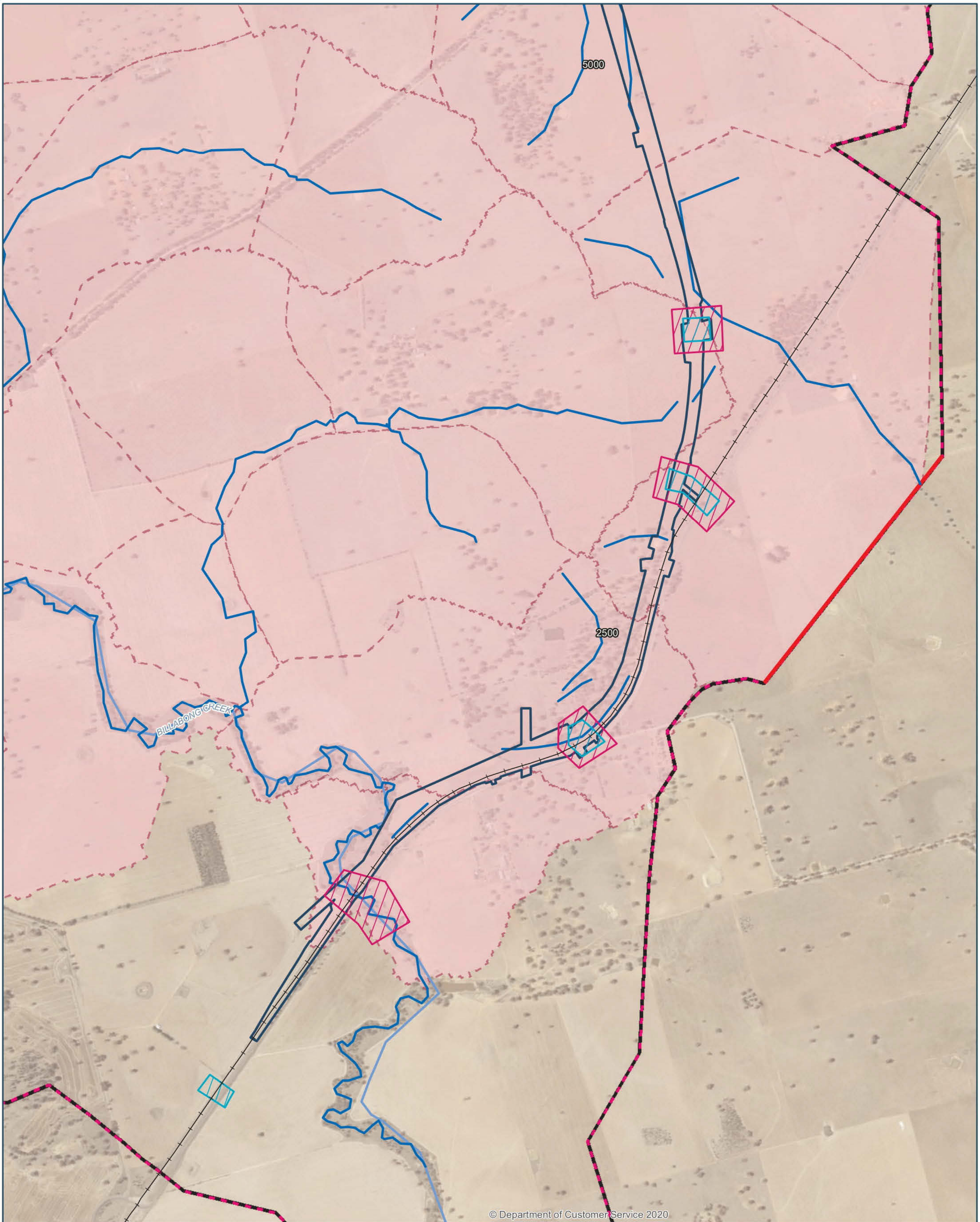

ILLABO TO STOCKINBINGAL Grid Extents

0 150 300 450 600 Meters

Coordinate System: GDA 1994 MGA Zone 55

Date: 8/18/2022 Paper: A3
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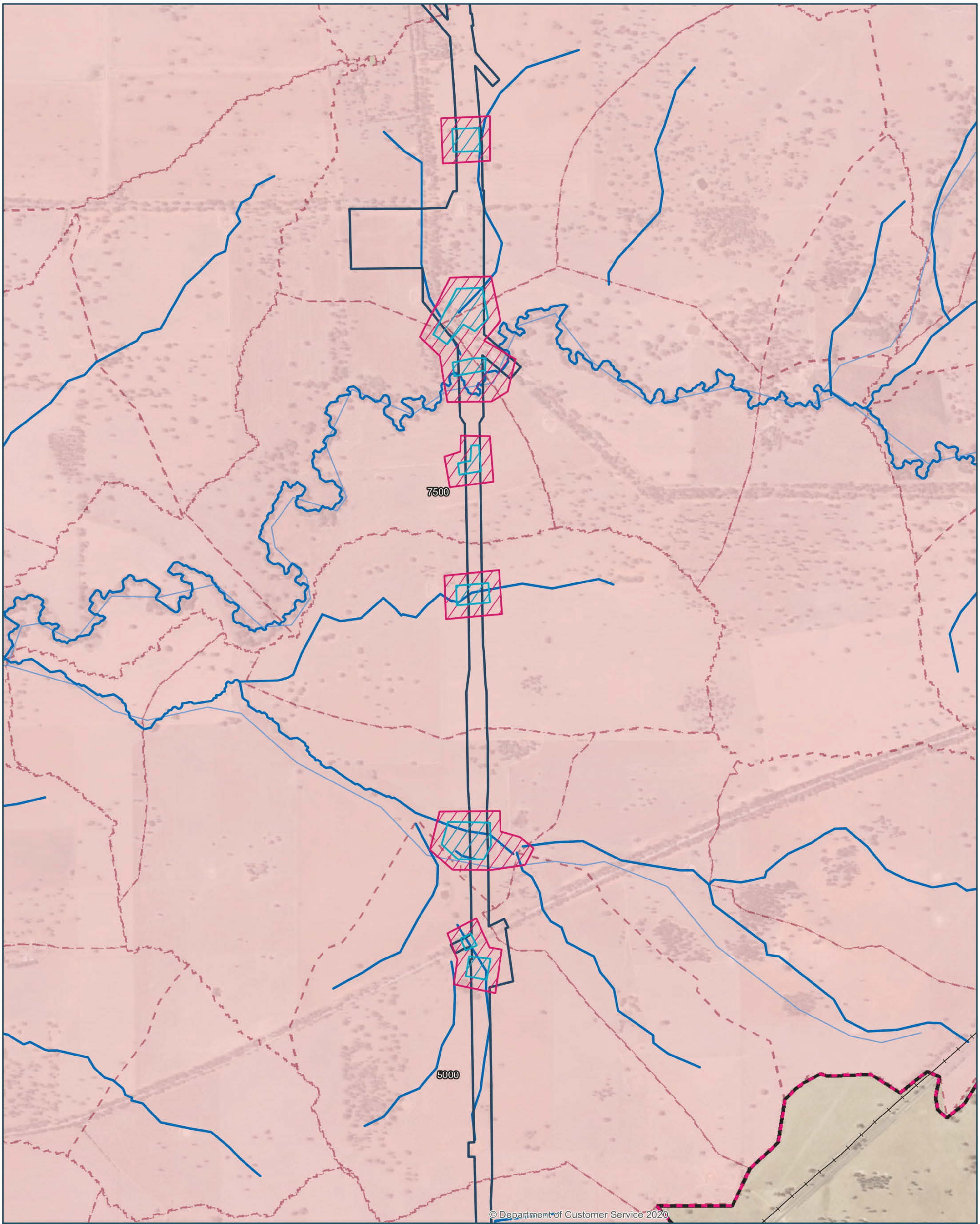

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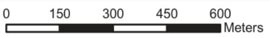

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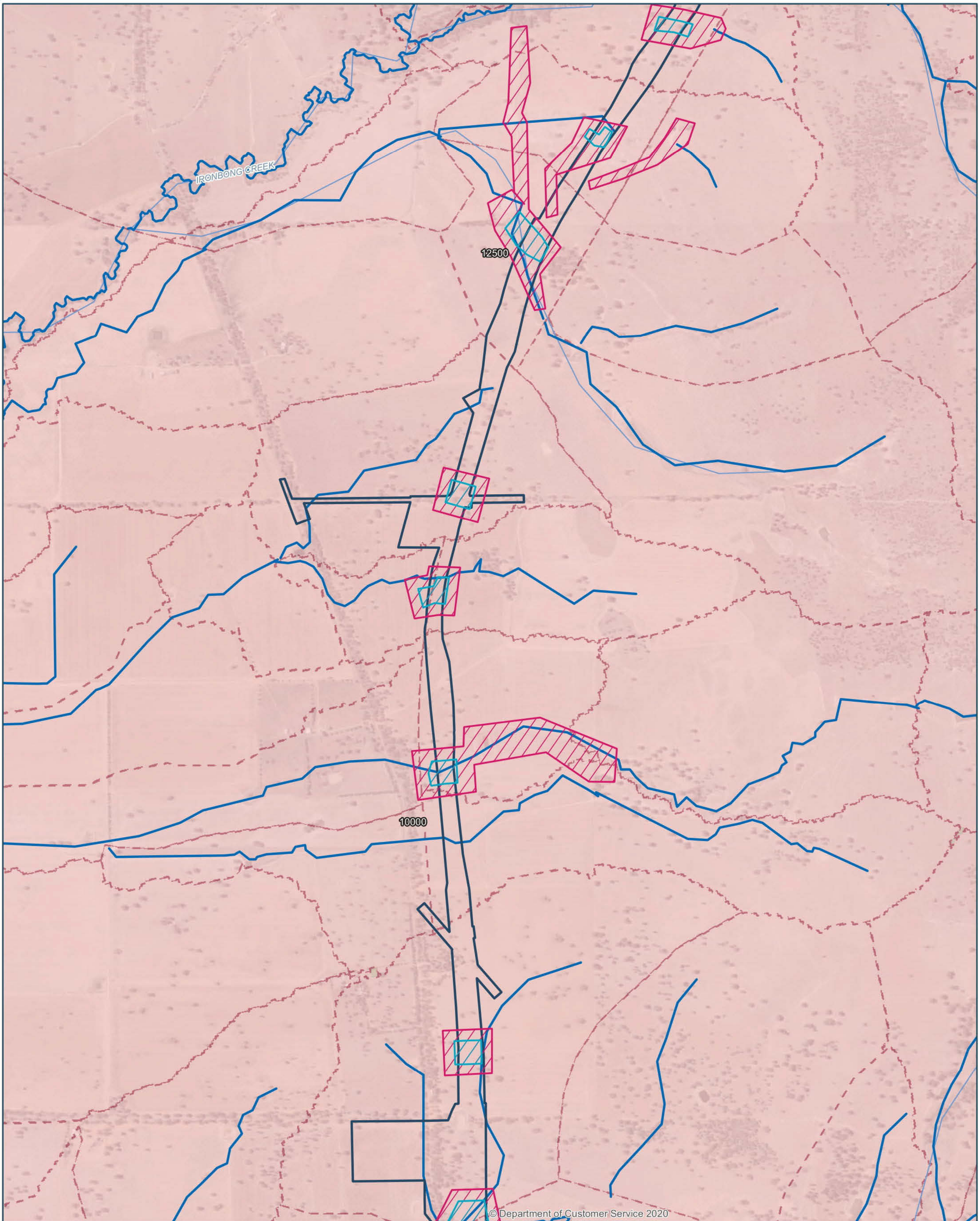

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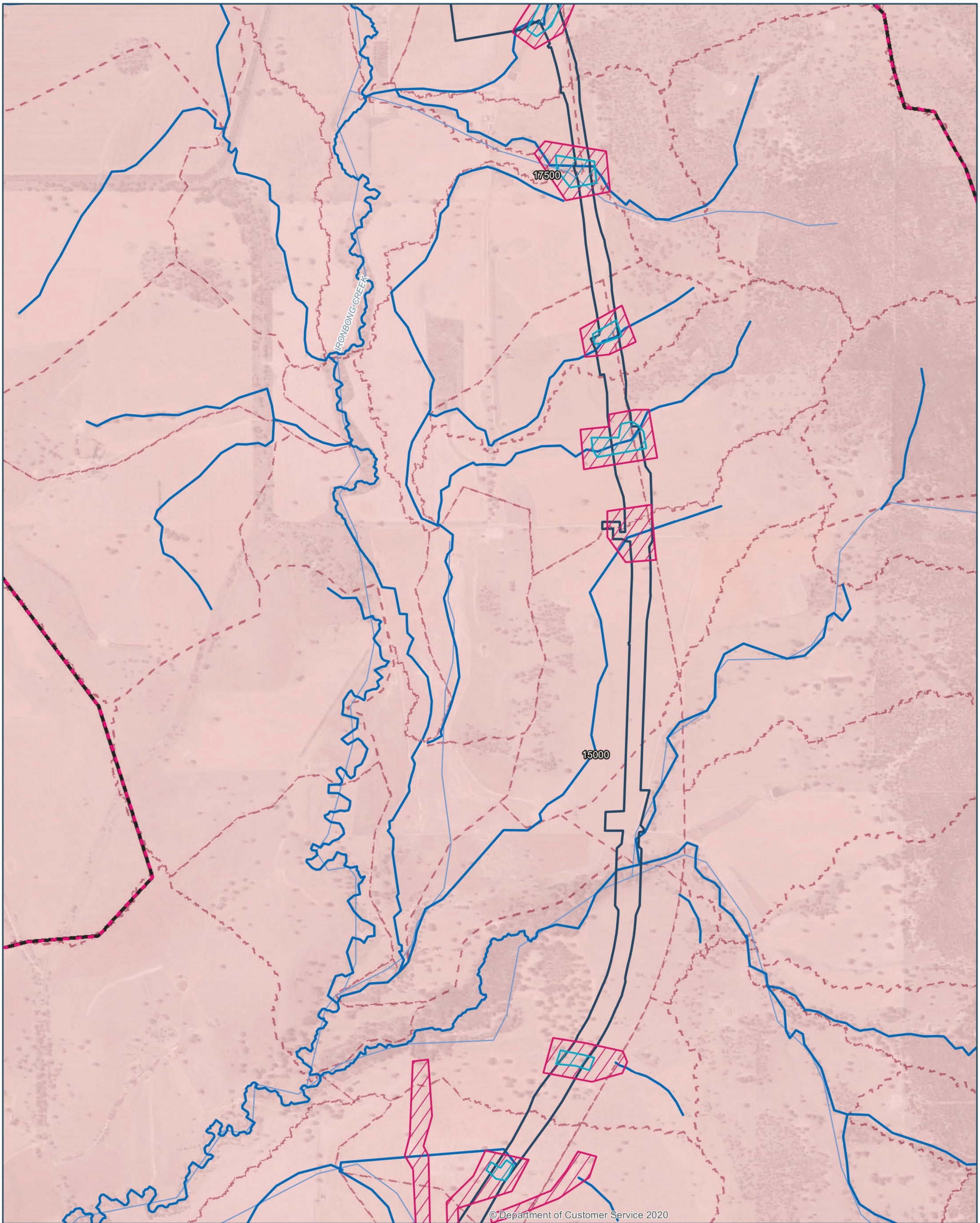

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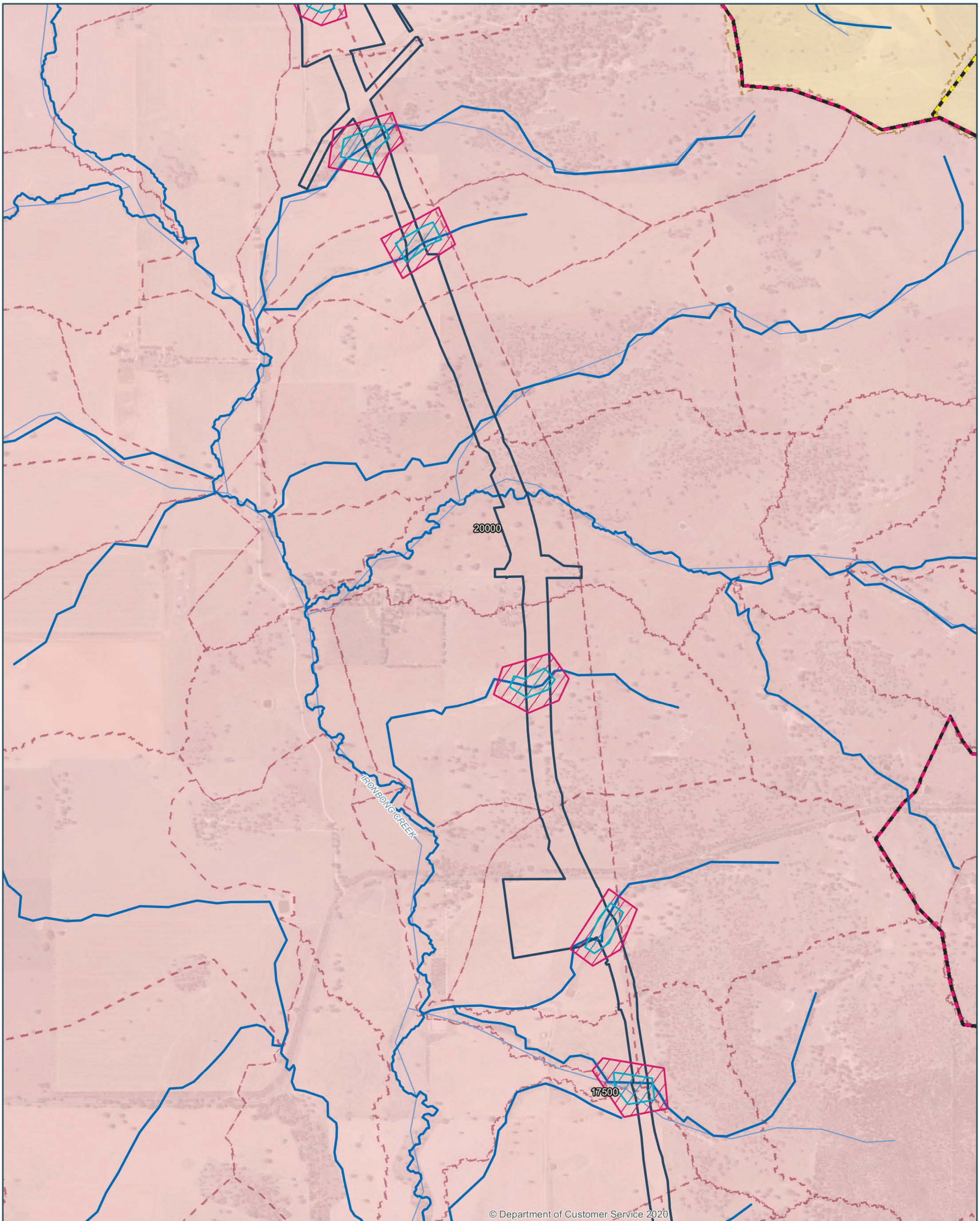

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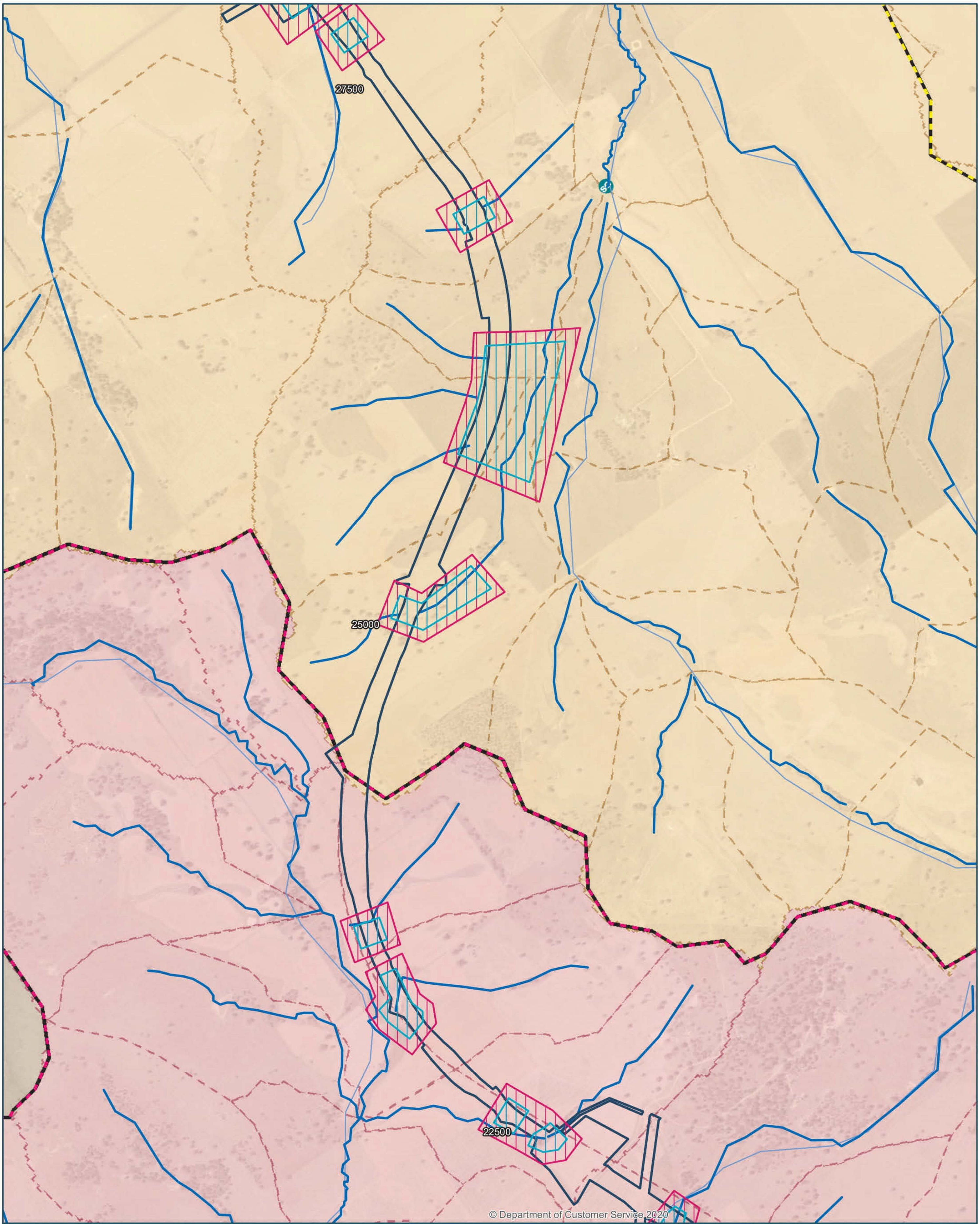

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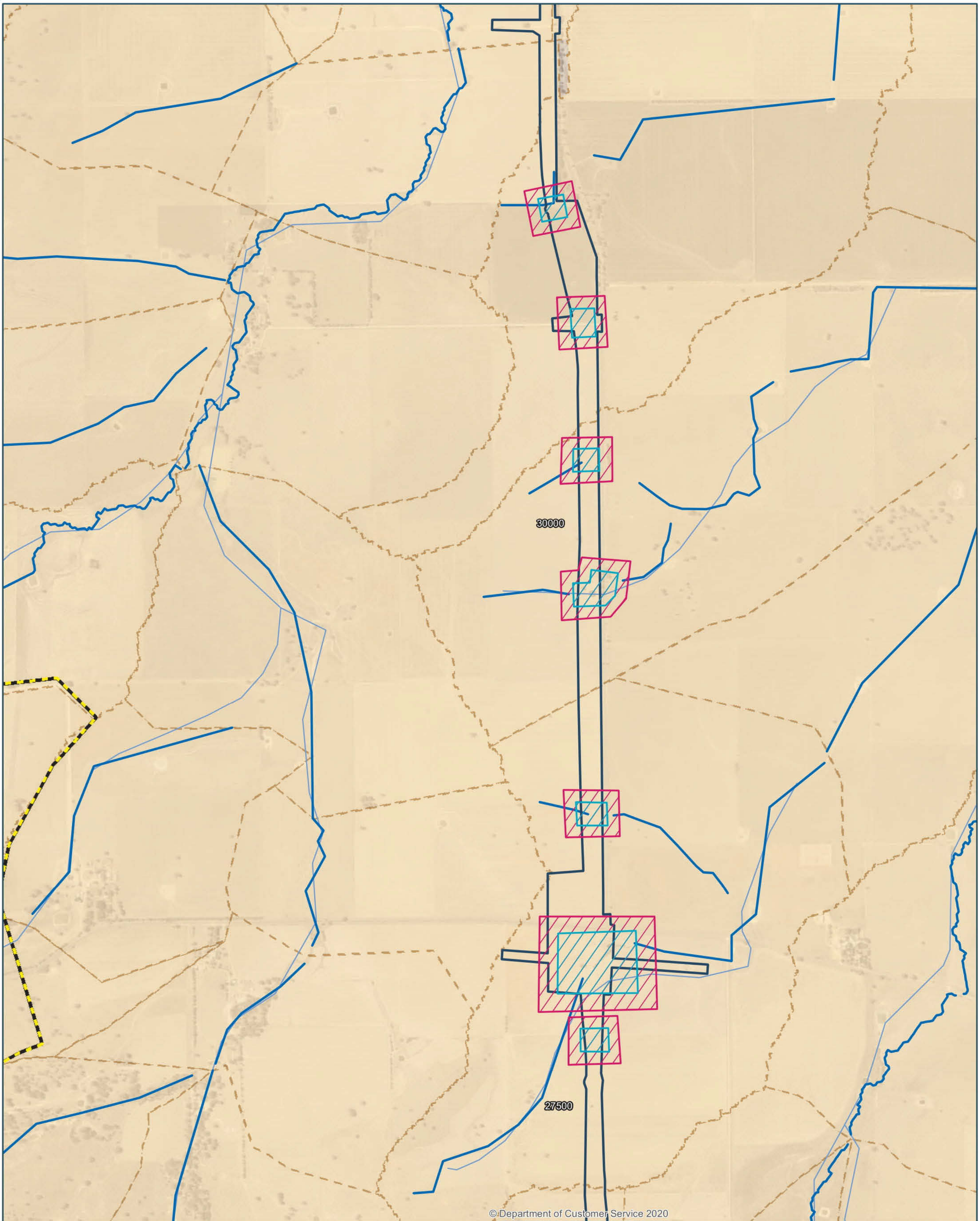

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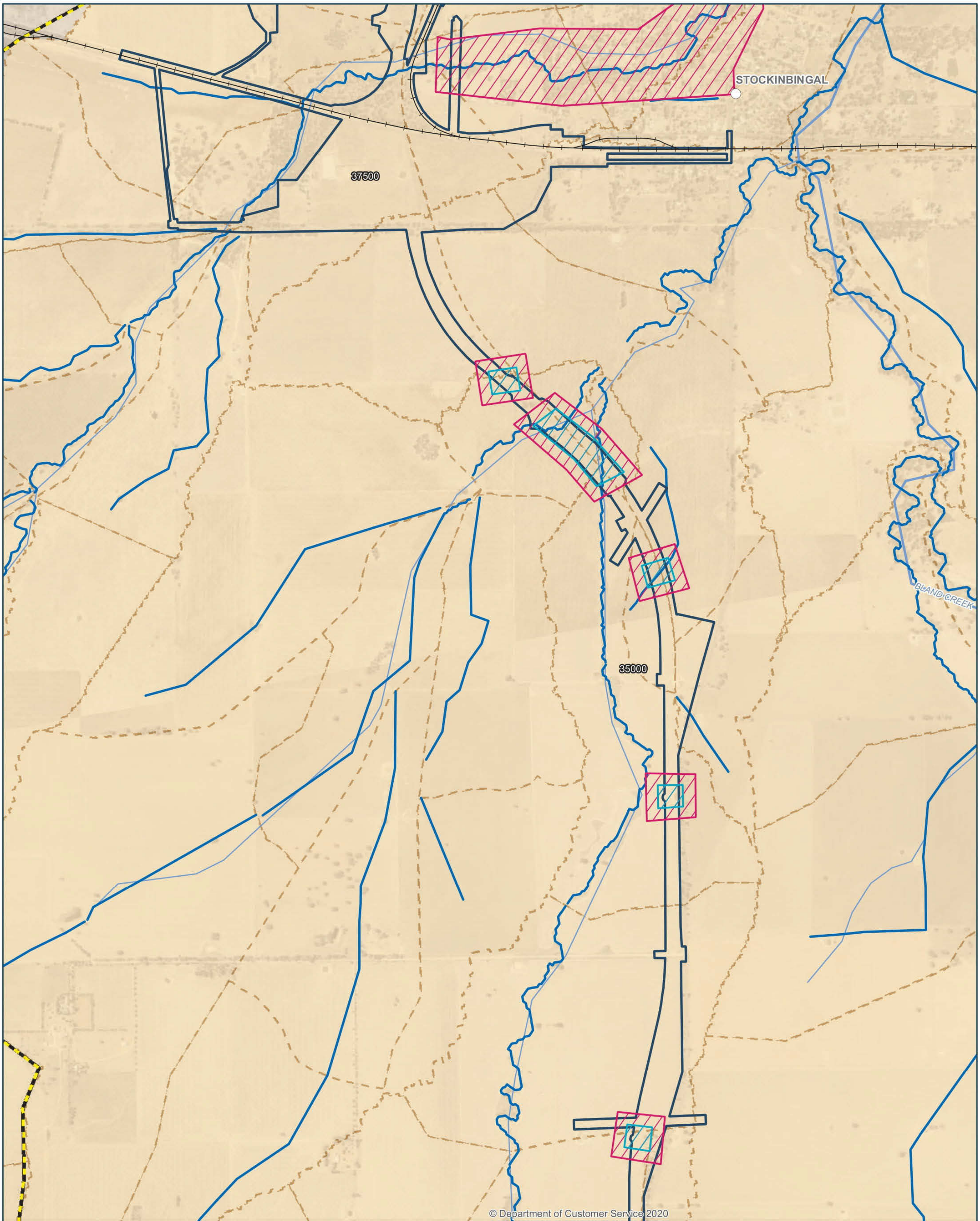

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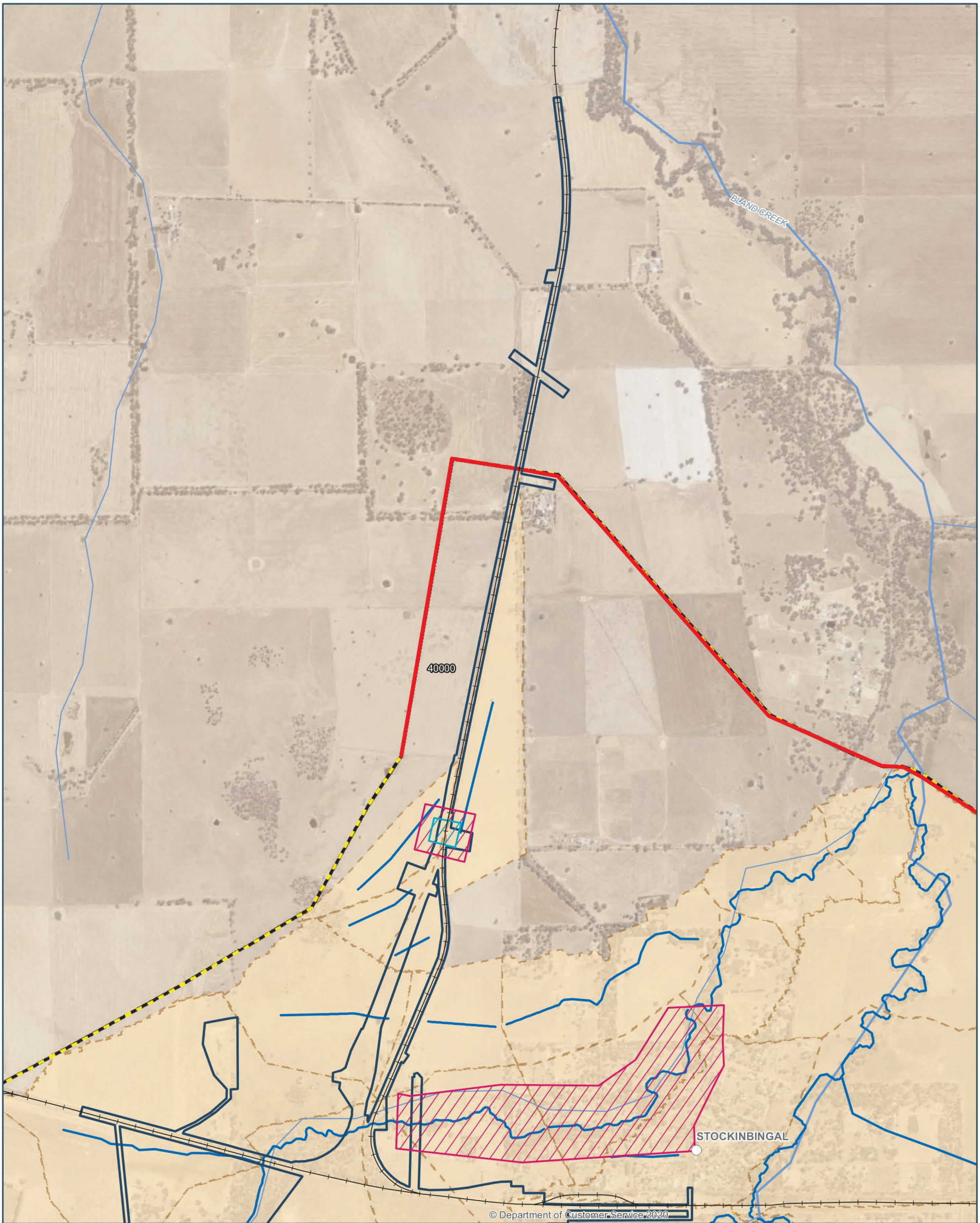

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A.2 Hydrological model

To inform the understanding of the existing flood behaviour across the proposal site and on land upstream and downstream hydrologic models have been developed. The details of these models are described below.

A.2.1 Model construction

The XP-Rafts software has been utilised for the development of the hydrology for the proposal. XP-RAFTS uses the Laurenson non-linear runoff routing procedure to develop a stormwater runoff hydrograph from either historic rainfall data or design rainfall data. The 2018.1 version of the software is able to utilise the latest ARR Data Hub design rainfall information.

A.2.1.1 Rainfall depths and temporal patterns

The design rainfall was specified as per the ARR 2019 design guidelines (Chapter 3, Book 2, ARR 2019). Rainfall depths for the range of design storms were generated from the Bureau of Meteorology 2016 Intensity-Frequency-Duration (IFD) dataset and applied to temporal patterns sourced from the ARR 2016 datahub. The data was extracted for each of the two hydrological models separately, giving area specific rainfall parameters for each of the section.

Pre-burst rainfall was generated from the ARR 2019 datahub plugin within XP-RAFTS for each section and applied to the hydrological models.

A.2.1.2 Catchment loss

Initial catchment rainfall losses were generated from the ARR 2019 datahub website for the sections of the proposal area. These provided a starting estimate but were later refined following calibration and verification of the hydrologic model.

A.2.1.3 Areal Reduction Factor

An Areal Reduction Factor (ARF) is a reduction factor applied to rainfall depth in larger catchments to allow for the fact that larger catchments are less likely to experience the high intensity rainfall depth estimated at a point location simultaneously across the entire area as per ARR 2016 design guidelines (Chapter 4, Book 2, ARR 2016).

The ARR 2016 guideline estimates the ARF factor to the point of interest (e.g. to an individual cross drainage structure) with the factor varying based on AEP, storm duration and catchment area. ARR 2016 also states that "*There has been limited research on ARF applicable to catchments that are less than 10km^2. The recommended procedure is to adopt an ARF of unity for catchments that are less than 1km^2, with an interpolation to the empirically derived equations for catchments that are between 1 and 10km^2*".

The XP-RAFTS ARR Storm Generator reads the proposal specific data from the ARR Data Hub, including the Text File, ARR Temporal Patterns Increments File, and BOM Design Rainfall. The generator then produces an XPX file with all the rainfall event of selected Annual Exceedance Probability (AEP) and all of the durations for the given location. The software allows for an ARF to be applied if required. For the proposal, the catchments that have an area greater than 10 km^2 had an ARF applied. The ARF was calculated by the Storm Generator based on the catchment area and duration in accordance with equations 2.4.1 (Chapter 4, Book 2, ARR 2016) for durations less than 12 hr and equation 2.4.2 (Chapter 4, Book 2, ARR 2016) for durations between 12 and 24 hours.

The table below demonstrates the range of catchment areas in the proposal area and a summary of where ARF have been applied. The values for key catchments are included in the table below and vary between 0.85 and 0.98.

Table A.1 Summary of ARF values across the proposal site

	Area (km^2)	0.2EY	10AEP	5AEP	2AEP	1AEP	0.00005AEP
Dudauman Creek	27.37	0.9643	0.9615	0.9582	0.9539	0.9506	0.9365
Powder Horn Creek	6.97	0.9848	0.9831	0.981	0.9784	0.9763	0.9676
Powder Horn Creek Tributary	24.29	0.9724	0.9637	0.9605	0.9563	0.9531	0.9394
Run Boundary Creek	13.56	0.976	0.9736	0.9708	0.9671	0.9643	0.9523
Ulandra Creek	71.81	0.9443	0.9408	0.9368	0.9314	0.9273	0.9097
Billabong Creek	318.82	0.9016	0.8967	0.891	0.8834	0.8777	0.8531

A.2.2 Verification

A.2.2.1 Verification approach

Calibration is where the model input parameters are manipulated to replicate a historic event. Calibration requires gauged data such as rainfall and stream flow or water heights. Calibration uses one or more historic events to establish model input parameters and validation is where the input parameters are tested against one or more different historic event data. Due to the modelling approach adopted for the proposal, with TUFLOW being the primary tool used to represent routing of flow through the catchment areas, calibration of hydrologic model parameters also required use of the hydraulic models.

An extensive data review and calibration process has been attempted using data from the Wattle Creek gauge (see section A.4 below). Unfortunately, this analysis concluded that the Wattle Creek stage discharge curve is underestimating flows for larger flood events and ultimately could not be used in model calibration.

Model calibration relied on the Regional Flood Frequency Estimation (RFFE) technique; specifically, modelled catchment losses were adjusted in the RAFTS models and adopted based on a review of RFFE against Billabong Creek catchment flows. Billabong Creek is the largest catchment in the proposal area at 318km^2 and incorporates 25km of the 37km new rail track as well as four of the waterway bridges required for the proposal.

A.2.2.2 Regional Flood Frequency Estimation

Regional Flood Frequency Estimation (RFFE) is a technique that uses readily available flow data from all nearby gauges to develop flood frequency information for the location of interest. Due to lack of gauge and historic data across the larger catchments intersecting the proposal the RFFE technique was applied to the other catchments intersecting the proposal in both the Lachlan and Billabong Creek catchments.

The ARR Regional Flood Frequency Estimation Model for the 4th edition of Australian Rainfall and Runoff was utilised to provide estimated flood quartiles. It is noted that this used gauges up to 150km away from the proposal catchment. The gauge nearest to the proposal area is location 21km to the east of the Wattle Creek at Dudauman Gauge (412134) and has 36 years of record. A total of 15 gauges were utilised for the estimate with an average record length of 36 years. The gauge with the longest period of record (64 years) is located approximately 84km away from the Wattle Creek gauge. The results are presented in the table below.

Table A.2 RFFE for catchments intersecting the proposal

Catchment	Area	RFFE 1% AEP	RFFE 1% AEP 5% limit	RFFE 1% AEP 95% limit
Isobel Creek	6.7	12.4	4.7	33
Powder Horn Tributary	7.0	10.7	4.12	28.1
Run Boundary Creek	18.6	34.4	12.9	92.4
Powder Horn Creek	24.3	17	6.48	44.8
Dudauman Creek	27.4	22.8	8.71	60.3
Ulandra Creek	71.8	67.8	24.9	187
Billabong Creek	320.3	291	106	808

With Billabong Creek being the largest catchment within the proposal area the calibration was focussed on the Billabong Creek catchment to attempt to find a set of parameters and losses that best match the RFFE estimates. Results from the testing of losses at the Billabong Creek modelled location against RFFE estimates are presented in the table below. The adopted losses that provided the best fit were an initial loss of 30mm and a continuing loss of 1.72mm/hr.

Table A.3 Billabong Creek calibration adopted losses results

AEP	Adopted loses (TUFLOW)	Adopted losses XP-RAFTS	Expected quantile	RFFE 2014		RFFE 2021#
				5% limit	95% limit	
10%	83	123	112	41.4	305	110
5%	160	218	156	57.5	423	166.3
2%	235	289	226	83.1	621	243.3
1%	274	350	291	106	808	298.8

Proposed Alternative RFFE Technique (Email WMA Water, 21/4/2021)

For the Billabong Creek catchment, Test 4 provides a close correlation to RFFE for all four AEPs tested. Figure A.3 below presents the data from the tests graphically.

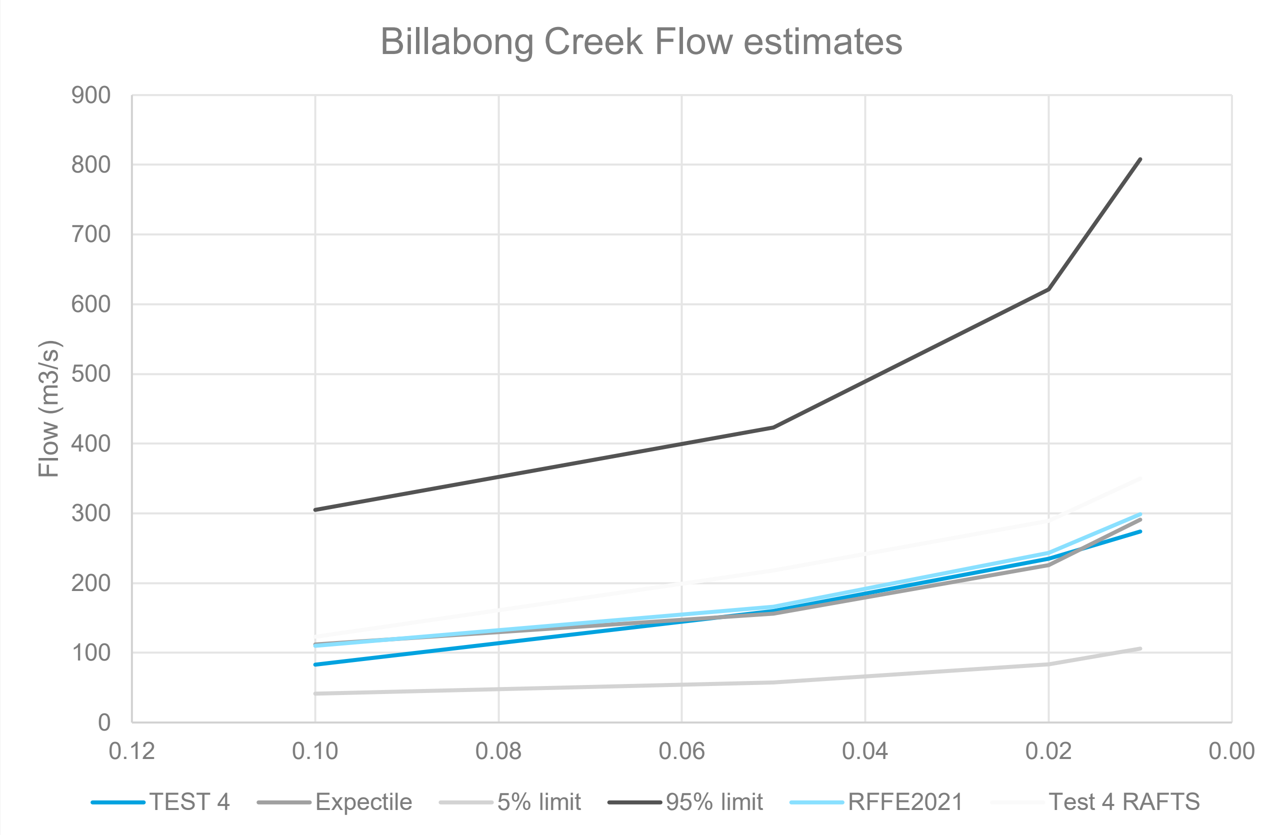


Figure A.3 Billabong Creek flow estimates

The results at the smaller catchments considered for these sensitivity tests are tabulated below. These results indicate that the adopted losses produced flows that were close to the RFFE.

Table A.4 Peak flows for Dudauman and Ulandra Creek sensitivity tests (1% AEP)

Catchment/model		Adopted losses flow (m³/s)	RFFE expected quantile (m³/s)
Dudauman Creek	RAFTS	45.7	22.8
	TUFLOW	64.1	
Ulandra Creek	RAFTS	153	67.8
	TUFLOW	136	

A.2.3 Adopted parameters for XP-Rafts

Calibration and verification of the XP-RAFTS hydrologic parameters involved both the XP-RAFTS models and the TUFLOW hydraulic models. The adopted parameters considered both the Lachlan and Billabong catchments and the fitting of the peak flow against RFFE and a review of peak water level against historic data.

The adopted parameters for XP-RAFTS are as follows:

- mannings ‘n’ roughness 0.07
- lag – stream length by average velocity extracted from the hydraulic model
- initial loss – 30mm
- continuing loss – 1.72mm/hr.

A.2.4 Design event modelling

Using the model parameters determined from the calibration the design rainfall event modelling has been in accordance with the latest advice in ARR 2016. The design storms were simulated for durations of 30 minutes through to 24 hrs for all the required AEPs as listed in the table below.

Table A.5 Hydrological design events

Design event	Approximate equivalent Average Recurrence Interval (ARI)	Purpose of event analysis
0.2EY	5 year ARI	Low order event for impact assessment.
10% AEP	10 year ARI	Medium event for flood impact assessment and potential lower standard adopted for hydraulic design.
5% AEP	20 year ARI	Medium event for flood impact assessment and potential lower standard adopted for hydraulic design.
2% AEP	50 year ARI	Medium event for flood impact assessment and potential lower standard adopted for hydraulic design.
1% AEP	100 year ARI	Typical standard for hydraulic design.
1% AEP+ CC	100y ARI and climate change	Climate change factor of 1.12 applied to 1% AEP rainfall depths to assess the impact of predicted future climate conditions.
0.05% AEP	2000 year ARI	Extreme event to inform loading for structural stability assessments for bridges (if required).
PMF	NA	Extreme event to understand full extent of flood extent.

The hydrological modelling has been undertaken using the ensemble method of flow estimation as detailed within the ARR 2019 design guidelines (Chapter 3, Book 4, ARR 2016). Each flood event (AEP) was run for a range of standard durations and for an ensemble of 10 temporal patterns within each duration. Results were extracted for the critical flow at each watercourse separately and the median of these flows was selected as the design flow for each AEP event. The point of interest for each watercourse was defined by the proposal site intersecting with the watercourse. Catchment delineation was defined to this intersection when the proposal site had been determined.

A.3 Hydraulic modelling

Hydraulic models have been used to simulate the interaction between runoff hydrographs generated by the hydrological models, site topography and hydraulic structures along the proposal site and across land upstream and downstream of the proposal site.

Two dimensional (2D) coupled with 1 dimensional hydraulic models have been developed using the TUFLOW hydraulic modelling software program. The models have been built using the 2020-10-AD version of TUFLOW GPU (Graphical Processing Unit) and adopt the HPC (Heavily Parallelised Computations) solver.

TUFLOW was selected due to its adaptability, fast run times and ability to input data from and output data to civil design, GIS and BIM programs.

The TUFLOW models were used to simulate the following events for existing conditions PMF, 0.05%, 1% AEP+CC, 1% AEP, 2% AEP, 5% AEP, 10% AEP and 0.2EY.

A.3.1 Model construction

The models are largely 2 dimensional (2D) models with some features modelled in 1 dimension and then dynamically linked to the 2D model domain. Elements that have been modelled in 1D include:

- culverts
- bridges.

A Quad-tree modelling approach was adopted as it achieved a balance between sufficient resolution across the proposal study area, whilst providing increased resolution at locations of interest. The quad-tree approach allows for smaller grid sizes to be used to define the topography of the floodplain at points of interest and where the ground levels vary over short distances. A 10m grid was adopted as the first grid level, with a second level of 5m and a final third level of 2.5m. The extent of the 2.5m grid is approximately a distance of 50m upstream and downstream of design culverts/bridges with the 5m grid roughly 100m upstream and downstream and laterally covering the flood extent for the PMF event. A 5m grid was adopted to appropriately represent the Stockinbingal levees, with top of bank data used to define the overtopping levels.

The TUFLOW HPC modelling solution adopted for this proposal implemented an adaptive time step solution that allows the solution to vary the timestep and repeat timesteps as required to maintain stability when resolving the equation.

Two models have been built, one covering the Lachlan catchment area and one covering the Murrumbidgee catchment area, similarly to the hydrologic model breakdown. The extents of the models have been based on the available LiDAR dataset and the proposal study area.

A.3.1.1 Topography and survey data

LiDAR datasets as provided by ARTC and were used to build surface elevation models for the proposal study area between Illabo and Stockinbingal. The available LIDAR is suitable for building surface elevations models to assist with understanding surface runoff behaviour across broad areas. The LIDAR does not however have sufficient detail to determine creek bathymetry and waterway structure sizes.

A.3.1.2 Existing structures

There are several existing structures within the study area that will influence flood behaviour in both the Lachlan and Murrumbidgee TUFLOW models. It is noted that detailed survey of these structures and the watercourse are required to adequately inform the hydraulic models. The location, model and information source are detailed in the table below.

Table A.6 Existing structures

Structure name	Model	Details	Comments
Dudauman Creek Burley Griffin Way (left) Crossing	LAC	Site visit, structure inspection report.	Detailed survey not part of current scope but should be considered for future design development.
Dudauman Creek Burley Griffin Way (right) Crossing	LAC	Site visit, structure inspection report.	Detailed survey not part of current scope but should be considered for future design development.
Dudauman Creek Lake Cargelligo Rail (left) crossing	LAC	Site visit, structure inspection report.	Detailed site survey incorporated into model.
Dudauman Creek Lake Cargelligo Rail (right) crossing	LAC	Site visit, structure inspection report.	Detailed site survey incorporated into model.
Dudauman Creek S2P crossing	LAC	Site visit, structure inspection report, detailed survey.	Detailed site survey incorporated into model.

Structure name	Model	Details	Comments
Dudauman Creek Levees	LAC	Detailed survey	Detailed site survey incorporated into model.
Ulandra Creek Ironbong Road Crossing	MUR	Site visit, detailed survey.	Detailed site survey incorporated into model.
Billabong Creek Main south line crossing	MUR	Site visit, detailed survey of structure and creek channel upstream of structure.	Detailed site survey incorporated into model.
Billabong Creek Olympic Highway crossing	MUR	Site visit.	Detailed survey not part of current scope.

A.3.1.3 Burley Griffin Way representation

Burley Griffin Way a state classified road and is generally perpendicular to the rail alignment between south of Stockinbingal and Dudauman Creek. Representation of the major road was included within the LAC model. The elevation of the Burley Griffin Way has been represented based on LiDAR survey used for the wider flood modelling. The ridge of the road was set using a TUFLOW '*2d_zline*' to ensure the high points on the highway are represented.

Road culverts and bridges were represented in the models based on survey data received from RMS. This data did not contain full details of the structures (e.g. no culvert invert data was available) so estimations were made where necessary. Further survey data will be sought in key locations to improve representation for future design stages.

A.3.1.4 Bridge representations

Bridge structures have been represented in the hydraulic model using a 'layered flow constriction' type TUFLOW input. This representation of the bridge structure allows a depth varied form loss coefficient to be applied to represent the different elements of the bridge structure.

The representation of the existing rail embankment and bridge abutments are included in the 2D TUFLOW model grid and this representation inherently simulates the contraction and expansion losses as flow passes through the bridge structure. The form losses are applied uniformly across the width of the bridge structure opening to represent the additional losses due to piers, which are not represented in the TUFLOW model grid. At bridges that surcharge (i.e. flows that exceed the soffit level), the layered flow constriction file allows the level of the soffit to be set with an additional loss factor and blockage induced when this level is exceeded to represent surcharging of the bridge. The FLC values adopted for layer one represent hydraulic losses associated with the bridge piers and are derived using the process outlined in Section 5.4 of Austroads (1994) based on the approach from Bradley (1978).

The bridge structure is generally represented with layers representing the following:

- Layer 1 – FLC value representing the bridge piers with blockage factor where required to represent reduced waterway opening. FLC value varies depending on bridge design
- Layer 2 – FLC value (1.56) representing the bridge deck and parapet with 100% blockage factor
- Layer 3 – FLC value (0.50) representing bridge safety barriers/railings with 50% blockage factor
- Layer 4 – Flow over the top of railings – assumed to be unimpeded.

Bridge representations in the model have been derived from survey provided, or site images in lieu of detailed survey and structural design drawings for proposed structures.

A.3.1.5 Culverts

Culverts have been represented in the model as one dimensional (1d) elements. These 1d elements are then connected to the 2d grid to allow the transfer of flow from the 2d to the 1d. The existing culvert details have been sourced from detailed survey at key proposal locations or have been estimated based on field investigations. Refer to section 4.3.6 on Model limitations.

For all culverts, a mannings n of 0.013 s/m$^{1/3}$ has been adopted, inlet and outlet losses of 0.5 and 1.0 respectively and a blockage factor of 15% have been adopted for both existing and design conditions.

A.3.1.6 Boundary conditions

Hydrographs for incoming flows were imported from the hydrological model for the 18%, 10%, 5%, 2%, 1% and 0.05% AEP storm events, for the existing and design cases. Incoming flows were applied on a sub-catchment scale using a *'2d_sa'* TUFLOW boundary for local catchment flows and using a *'2d_bc'* flow versus time (QT) boundary for concentrated upstream overland flow in rivers and creeks.

A water level versus flow (HQ) boundary condition with a slope matching the channel bed has been used as the downstream boundary of the TUFLOW model. Preliminary sensitivity testing of the adopted slope indicated that the model boundary is of sufficient distance from the proposal to not influence the results. For the Lachlan model, the downstream boundary has been set a sufficient distance from the model to ensure it does not impact flood levels or behaviour in the vicinity of the proposal.

For the Murrumbidgee model, the downstream boundary has been extended to the limit of the LIDAR along the main Billabong Creek channel and to the west beyond the proposal study area. The adopted boundary conditions do not appear to impact flood levels upstream of the existing Main Southern Rail alignment but no information is available on the existing Olympic Highway crossing of Billabong Creek and creek definition is limited to LIDAR. It is noted the detailed survey of Billabong Creek is available upstream of the rail but not downstream which is likely to limit flood behaviour accuracy downstream of the Olympic Highway.

A.3.1.7 Initial water level – farm dams

As a conservative approach, the initial water level in the farm dams across the proposal area has been set to the top of dam embankment heights. These heights have been determined from the LIDAR and set for each farm dam identified in the aerial photographs. All the farm dams upstream and within the proposal corridor have been included. This approach is conservative as it assumes that any overland flow or stream flow reaching the dam results in overflows and no storage within the dam.

A.3.1.8 Manning's 'n' values for floodplain areas

The Manning's 'n' values used in the hydraulic models for floodplain areas are consistent with ARR 2016 guidance and were estimated from land use mapping and aerial photography. The Manning's 'n' values adopted are the same as those adopted on other sections of the Inland Rail proposal.

The land use mapping did not adequately identify the creeks intersected by the proposal. So, the creeks identified from the aerial photograph with heavy vegetation have been manually digitised within the proposal boundary (as at 70% Reference design) to have a roughness of "0.08". The LiDAR does not provide adequate detail to define top of bank to set the extent of the roughness, so the aerial photography has been relied upon. The watercourses with a digitised section include, Dudauman, Run Boundary, Ulandra and Billabong Creeks.

The Manning's 'n' values adopted for the floodplain areas are provided in the table below.

Table A.7 Manning's 'n' values adopted for floodplain areas

Land use	Manning's 'n' value (s/m$^{1/3}$)
Pasture	0.06
Roads/Rail	0.02
Buildings	3
Ponds and other water	0.03
Urbanised Areas	0.1
Industrial Areas	0.1
Low Density Urbanised Areas	0.08
Heavily Vegetated Creek	0.08
Maintained Grass	0.04

A.3.1.9 Blockage

Blockage of hydraulic structures in both existing and design scenarios has been assessed at 100% Reference design stage. ARR 2019 Book 6, Chapter 6 (Ball et al, 2019) outlines the most up to date guidance for the estimation of the risk of blockage of culvert structures. The approach considers a range of site specific factors to understand the risk and then calculate the estimated percentage blocked for a structure.

ARR2019 suggests that there are several factors that most influence the likely blockage of bridge and culvert structures. These factors are as follows:

- debris type and dimensions
- debris availability
- debris mobility
- debris transportability
- structure interaction
- random chance.

The ARR2019 risk assessment for blockage determined values from 0% blockage to 25% blockage. Since the design is still being developed a uniform blockage of 15% was adopted for all new culverts rather than reassessing for each design iteration.

For existing culverts the same assessment was applied except for the Stockinbingal to Parkes rail line and Lake Cargelligo line culverts for which maintenance records and visual inspections during site visits indicated 50% blockage.

A.3.2 Reference design model build

The Reference Design is based on the selected alignment from previous phases (refer to Alignment report/MCA) and has undergone several iterations in terms of vertical and horizontal alignment. For the drainage related infrastructure, the Reference design has been governed by minimising impacts to existing flood behaviour as outlined in the HHIP.

The reference design has been developed in conjunction with the earthworks, structures, roads and environmental considerations. Each new design element which has been included in the post upgrade flood model is documented below.

A.3.2.1 Rail embankment

The rail embankment consists of 37km of new track with a passing loop (chainage) and rail siding (chainage) that ties in near Illabo and north of Stockinbingal. The embankment starts in fill at Illabo and enters cuttings and fill sections throughout the 37km. The highest fill embankment is approximately 15m high near Isobel Creek. The rail embankment has been included in the model as an adjusted surface morphology (Triangular irregular networks (TIN)) so that the full horizontal extent of the embankment can be included.

The top of rail is included in the model as a “Z_line” to captures the top of the rail. This approach informs the flood immunity criteria which is to should of formation and the overtopping of rail risk assessment.

A.3.2.2 Road embankment

Burley Griffin Way (BGW) is to be re-aligned from its current alignment to provide a safer crossing of the proposed Inland Rail and Lake Cargelligo Lines. The BGW will be raised to go over the IR alignment which will result in an embankment and rail underbridge. The BGW embankment has also been included in the model as an updated surface TIN.

A.3.2.3 Bridges

Bridge structures have been represented in the hydraulic model using a ‘layered flow constriction’ type TUFLOW input. This representation of the bridge structure allows a depth varied form loss coefficient to be applied to represent the different elements of the bridge structure.

Thirteen (13) new bridges in the proposal, consisting of:

- a road overpass as Burley Griffin Way (BGW)
- three local road crossings, Ironbong Road, Old Cootamundra Road and Dirnaseer Road
- 9 new waterway crossings.

A.3.2.4 Culverts

Culverts have been represented in the model as one dimensional (1D) elements. These 1D elements are then connected to the 2D grid to allow the transfer of flow from the 2D to the 1D. The existing culvert details have been sourced from detailed survey at key proposal locations or have been estimated based on field investigations.

For all culverts, a mannings n of 0.013 s/m$^{1/3}$ has been adopted, inlet and outlet losses of 0.5 and 1.0 respectively and a blockage factor of 15% have been adopted for both existing and design conditions.

A total of 59 new banks of culverts throughout the proposal are utilised within the flood model. These typically cross the rail or level crossings in the same location as the concentrated flow path in the existing conditions. Additional culverts are provided around BGW that have the purpose of balancing flow around the proposed overpass.

A.3.2.5 Diversion drains

There are a number of diversion drains included in the proposal design. These are designed to divert flow around the rail embankment and back to its original flow path. Diversion drains have typically been used in the following situations:

- around BGW to divert flow around the road embankment
- in cut locations where flow approaches the top of the rail cutting
- in fill locations to divert flow along the rail embankment.

A.3.2.6 Level crossings

Up to 11 level crossings are proposed to allow access across the rail. These level crossings have been included in the flood model and preliminary drainage sized to minimise impacts to overland flows.

A.3.2.7 Washout calculations

Methodology

To assess the impact of washout on the rail embankment the potential for breach was assessed. The Froehlich (2008) approach was used to estimate the appropriate widths at which the embankment would scour. From this calculation a width of 20m and 40m was used. The Froehlich approach requires the volume of the water against the embankment, for this assessment the volume of water at the peak of the PMF event between the existing rail embankment heading west and the new alignment was used. This area acts similar to a basin, allowing water to build up and only escape through the Dudauman Creek bridge.

In order to calculate the breach widths a height of breach of 3.5m and a volume of 305600m^3 was taken (shown in Figure A.4 below) which was roughly the typical height along the embankment in this area. Using the Froehlich equation below

$$B = 0.27 * k * Vw^{0.32} * Hb^{0.04}$$

k = 1.3 (for overtopping failures)

Vw = Volume upstream that will flow from the reservoir

Hb = Height of breach

Using this calculation an average breach width of 21m was assessed. In order to provide sensitivity around this value 2 widths were used 20m and 40m.

In order to calculate the breach formation time the following formula below was used.

$$t = 63.2 * (Vw/(g * Hb^2))^{0.5}$$

This gave an estimation that the time to breach would be around 0.8 hours. Therefore, two breach times were assumed in order to provide a conservative overview of the washout.

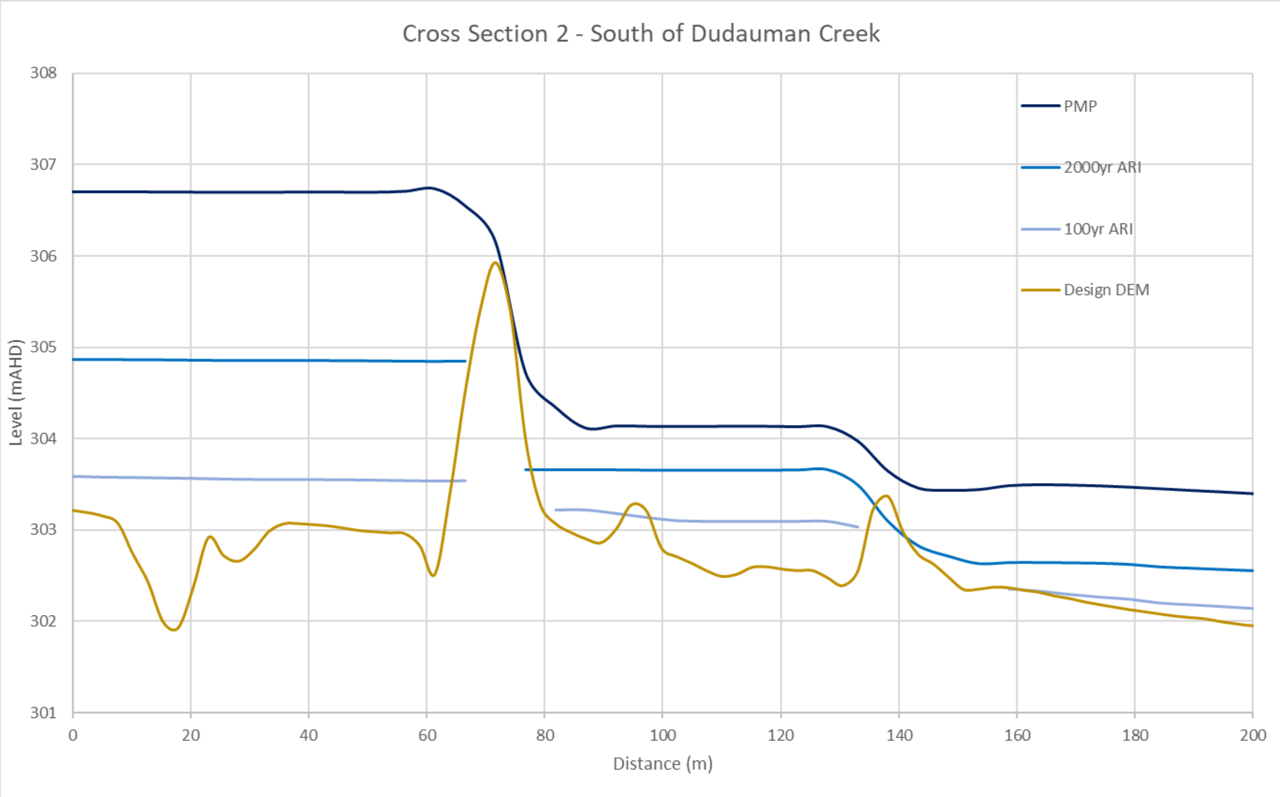


Figure A.4 Proposed embankment cross section

Within Tuflow Hydraulic modelling a variable Z shape layer can be used to provide the change in topography of a specified area over time. The breach can be triggered using different methods including:

- a specific time
- when the water level reaches a specific level at a trigger location
- when a specified water level difference is exceeded between two trigger locations.

For this assessment a trigger location for a specific level was used. For the level assumed to trigger the breach scenario the peak water level in Dudauman Creek for the proposed design PMF event was used. The location of the trigger location and breaching locations is shown in Figure A.5 below. The breach is defined by a polygon determining the extent of the breach, lines defining the breach width and points defining the level to which the topography should lower to. For all scenarios the existing ground levels within the topography were used, providing a complete wash out of the specified section of the embankment. Once these layers are specified the PMF 6 hour event only was run for all four scenarios. The 6hr PMF event was previously determined to be the only critical duration in the Lachlan Catchment model.

The scenarios undertaken include:

Table A.8 Breach scenarios

Scenario	Average breach width (m)	Breach formation time (hrs)
1	20	0.5
2	20	1.0
3	40	0.5
4	40	1.0

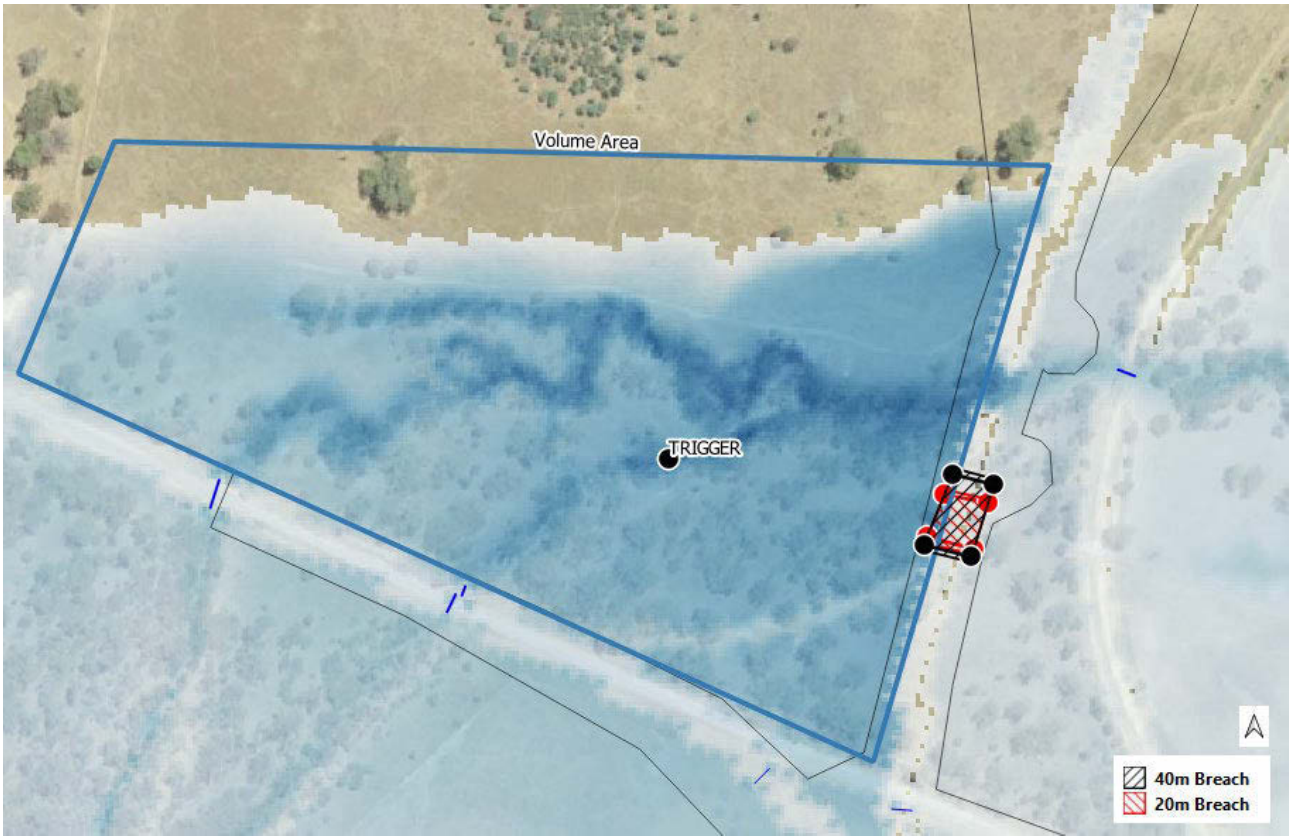


Figure A.5 Breach modelling inputs

Assumptions

The key assumptions include:

- estimated width of breach based on volume calculation
- estimated depth of breach – assumed that the embankment completely scours to existing level
- volume upstream of the breach has been kept the same for all scenario calculations
- timing of breach – based on formula for Basins and Dams
- instantaneous embankment failure
- assumed breach would occur at peak water level – depending on soil composition etc could happen earlier and later although peak is most conservative.

Results

Due to the town of Stockinbingal being located downstream from the proposal the results were assessed to understand the change in hydraulic hazard. For the 4 scenarios modelled, the largest impacts occur in Scenario 2, with a breach width of 20m over a 1-hour duration. In this scenario, 131 buildings have increased flood Hazard (Velocity x Depth), including 121 houses, 7 sheds and the pub, club, and bowling green. Of these affected buildings, 11 houses only have an increased level of hazard in the PMP Event compared to no washout design results. These affected buildings are shown in Table A.9 below.

Table A.9 Washout results for scenario 2

FID	Property type	Existing VxD classification	Scenario 2 VxD classification	Design velocity (m/s)	Previous classification	New classification
106	House	0.55	0.61	0.68	<=0.6	<=1.0
108	House	0.27	0.33	0.72	<=0.3	<=0.6
113	House	0.56	0.64	0.98	<=0.6	<=1.0
115	House	0.53	0.63	0.76	<=0.6	<=1.0
116	House	0.90	1.03	0.83	<=1.0	<=4.0
141	House	0.58	0.61	0.80	<=0.6	<=1.0
145	House	0.58	0.61	0.92	<=0.6	<=1.0
150	House	0.60	0.62	0.80	<=0.6	<=1.0
171	House	0.29	0.31	0.61	<=0.3	<=0.6
229	House	0.58	0.60	1.06	<=0.6	<=1.0
239	House	0.59	0.61	1.41	<=0.6	<=1.0

A.3.2.8 Model limitations

The most up to date data has been used to build the hydraulic models. However the following limitations should be noted in regards to the modelling:

- The hydraulic model has not been established to understand flood behaviour through the urban areas of Stockinbingal. The intent of the hydraulic model is to establish and understand flood behaviour in the rural areas of the proposal. Further model definition with detailed survey of Stockinbingal would be required to adequately model flood behaviour through the urban/built-up areas. The model results for Stockinbingal are therefore limited and will only be used for relative assessment. The models developed for this proposal should not be used to assess the effectiveness of current or proposed flood management measures for Stockinbingal unless updated with detailed information for the urban areas of Stockinbingal.
- It is noted that there are a considerable number of farm dams across the proposal area. The influence of these on flood model results has not been tested but the conservative approach of starting full is considered suitable for Reference design.
- No information is available on the existing Olympic Highway crossing of Billabong Creek and creek definition is limited to LIDAR downstream of the highway. It is noted the detailed survey of Billabong Creek is available upstream of the rail but not downstream which is likely to limit flood behaviour accuracy downstream of the Olympic Highway.

A.4 Wattle Creek Gauge 412134

A.4.1 Gauge data

The Wattle Creek gauge (412134) site was installed as part of the flood warning network for Cootamundra Shire Council (WaterNSW Site Summary Report, 9/6/2018). The gauge is a low profile crump weir with a cease to flow level of 0.5m. The following two photographs show the low profile concrete crump weir used as the control for the flow gauging and the rainfall gauge.

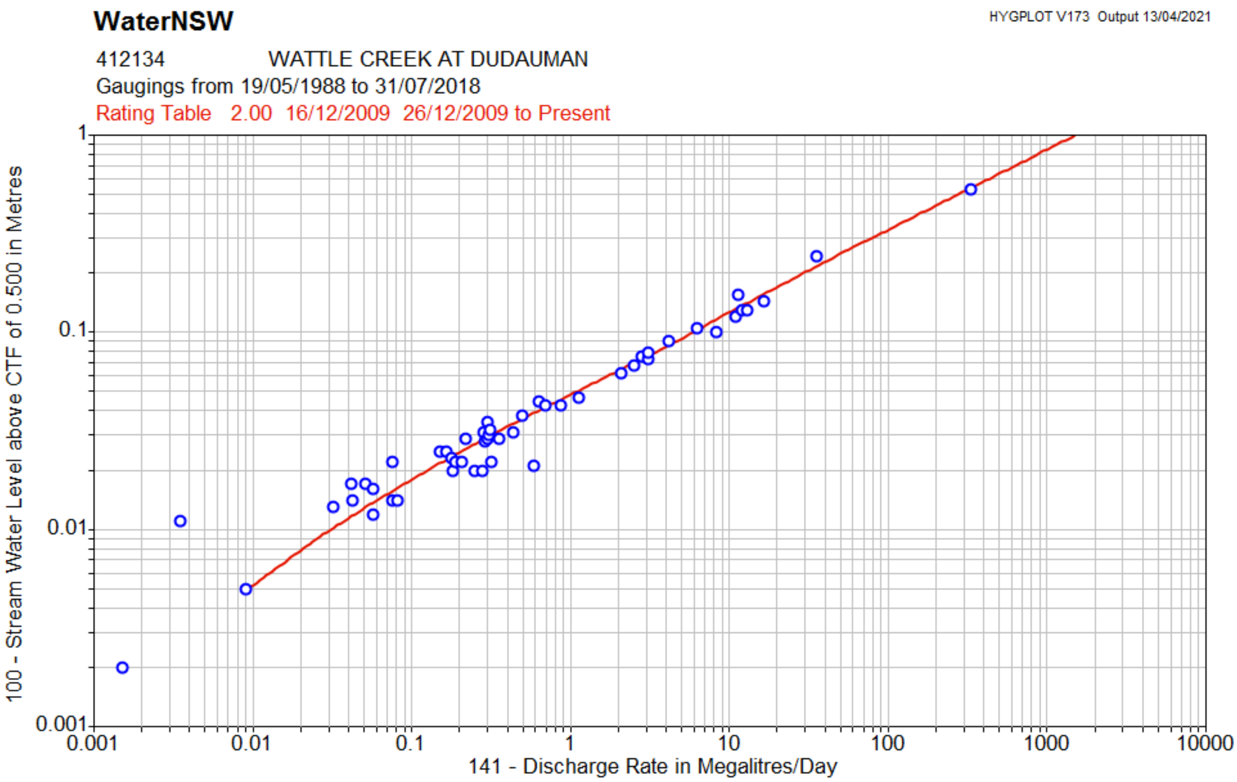
Photo A.1 Gauge 412134 Wattle Creek at Dudauman Control Weir

Photo A.2 Gauge 412134 Wattle Creek at Dudauman rainfall capture

A total of 75 gaugings have been taken between 19/05/1988 and 12/01/2021 and presented in below which shows the relationship between the recorded water level and the flow (WaterNSW Site Summary Report, accessed 12/02/2021). The gaugings have been used to develop a rating table for the site. All the water level data measured has been converted to a discharge based on the rating table.

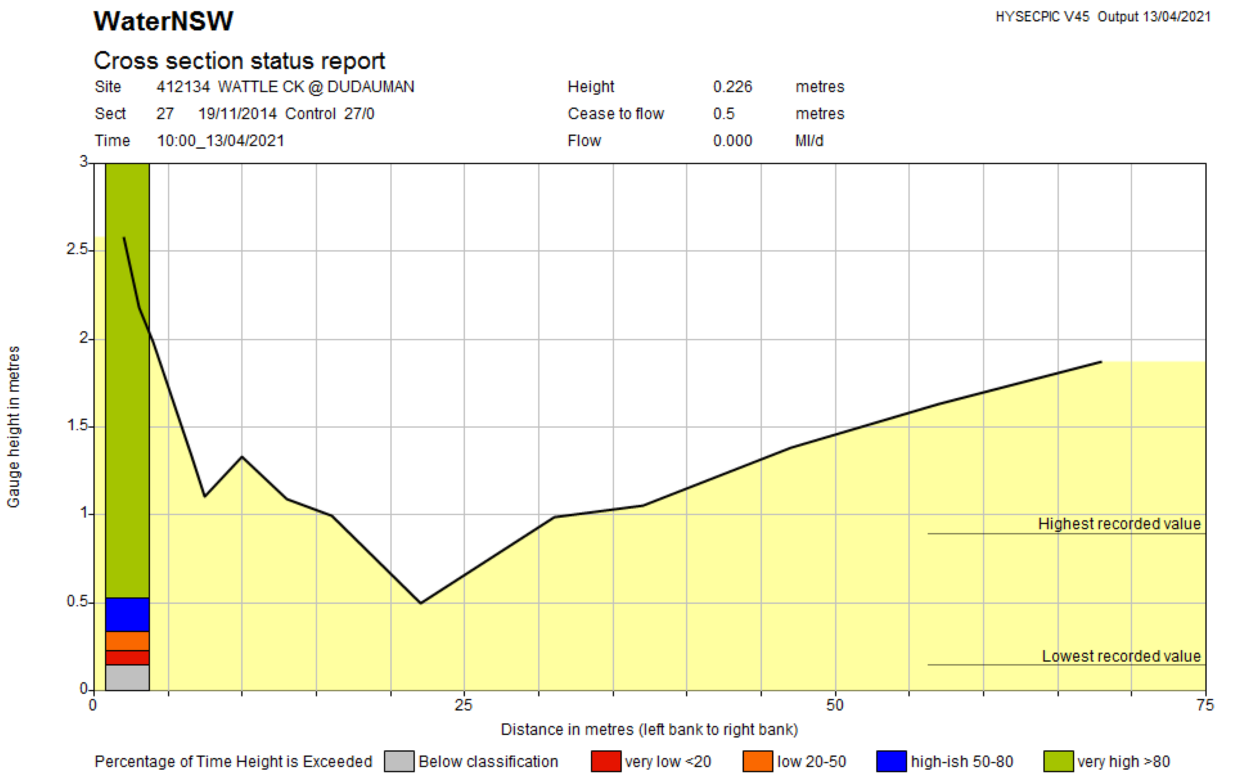
The available point data for the entire period of record (1988 to 2018) has been downloaded from the WaterNSW Real Time data website. This data included rainfall, water level (relative to a local datum) and discharge (ML/d).

The rating curve as presented below shows that only two flow event have been recorded above a flow of 20ML/d which equates to a flow of 0.23m^3/s. The two data points are likely to be beyond the concrete section of the gauge and therefore their accuracy is questionable. Despite the gauge records indicating 60% of the recordings to be good quality the conversion from height to flow may not be accurate for the high flow events.



Source: *https://realtimedata.waternsw.com.au/*

Figure A.6 Graph of recorded water level and rated flow



Source: *https://realtimedata.waternsw.com.au/*

Figure A.7 Cross-section at Wattle Creek at Dudauman Gauge 412134

A review of the Wattle Creek at Dudauman (412134) gauge indicates the highest water levels recorded are presented in the table below with four events chosen for purposes of calibration and verification events shown in bold and presented graphically below.

Table A.10 Wattle Creek at Dudauman 412134 maximum recorded rainfall, water level and discharge

Date	Rainfall (mm) (daily total)	Level (m)	Discharge (ml/d)	Discharge (m^3/s)
9/12/2010	46.5	1.21	477	5.5
3/3/2012	44.5	1.135	367	4.3
21/09/2016*	46	1.043	252	2.9
4/03/2012#	18	1.021	229	2.7
1/10/2016*	4.5	0.994	202	2.3
13/05/1995	34.13	0.983	191	2.2

The rainfall on 3 March 2012 would have created a wet catchment which would have contributed to the high flood level on 4 March 2012. This event is therefore not likely to be independent of the 3 March 2012 event.

* September 2016 was wettest month on record and the wet catchment contributed to a high flow in October despite low rainfall.

The response time for the catchment from peak rainfall depth to peak flood level is about 2 hours as can be determined from the rainfall and flow data.

Historic daily rainfall for up to 15 Bureau of Meteorology (BOM) rain gauges around the proposal were also reviewed to provide an understanding of the behaviour of rainfall across the proposal and understand catchment conditions before each peak flow event.

The December 2010 event appeared to be more significant for the Lachlan Valley but the remaining three events (March 2012, September 2016 and May 1995) all has similar daily rainfall depths across the proposal area. The range of recorded rainfall depths for each of the events across the nearest gauges was considered insignificant and subsequently a uniform depth was applied across the small Wattle Creek catchment.

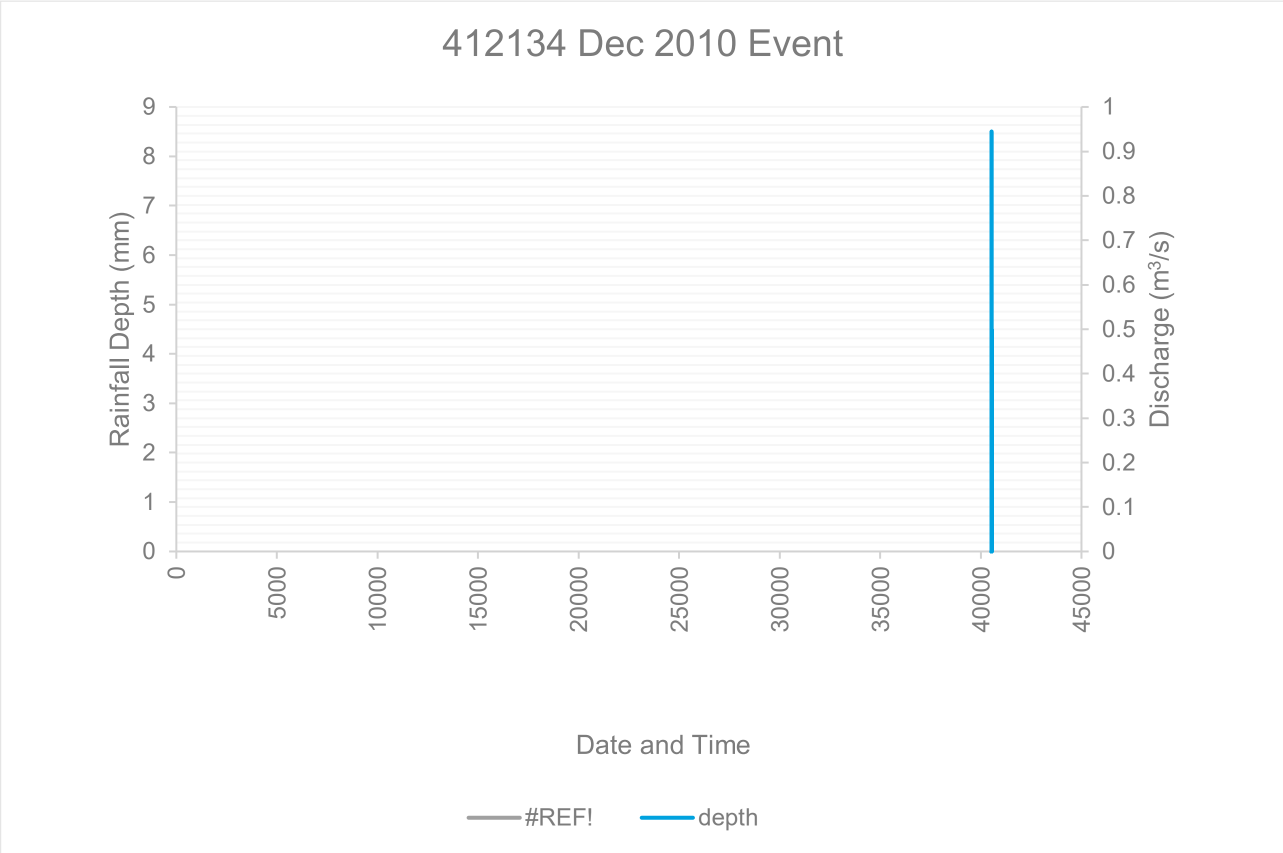


Figure A.8 December 2010 flow and rainfall data

The historic rainfall data indicates that there was no rainfall over most of the region for the 3 days before the 9 December rainfall.

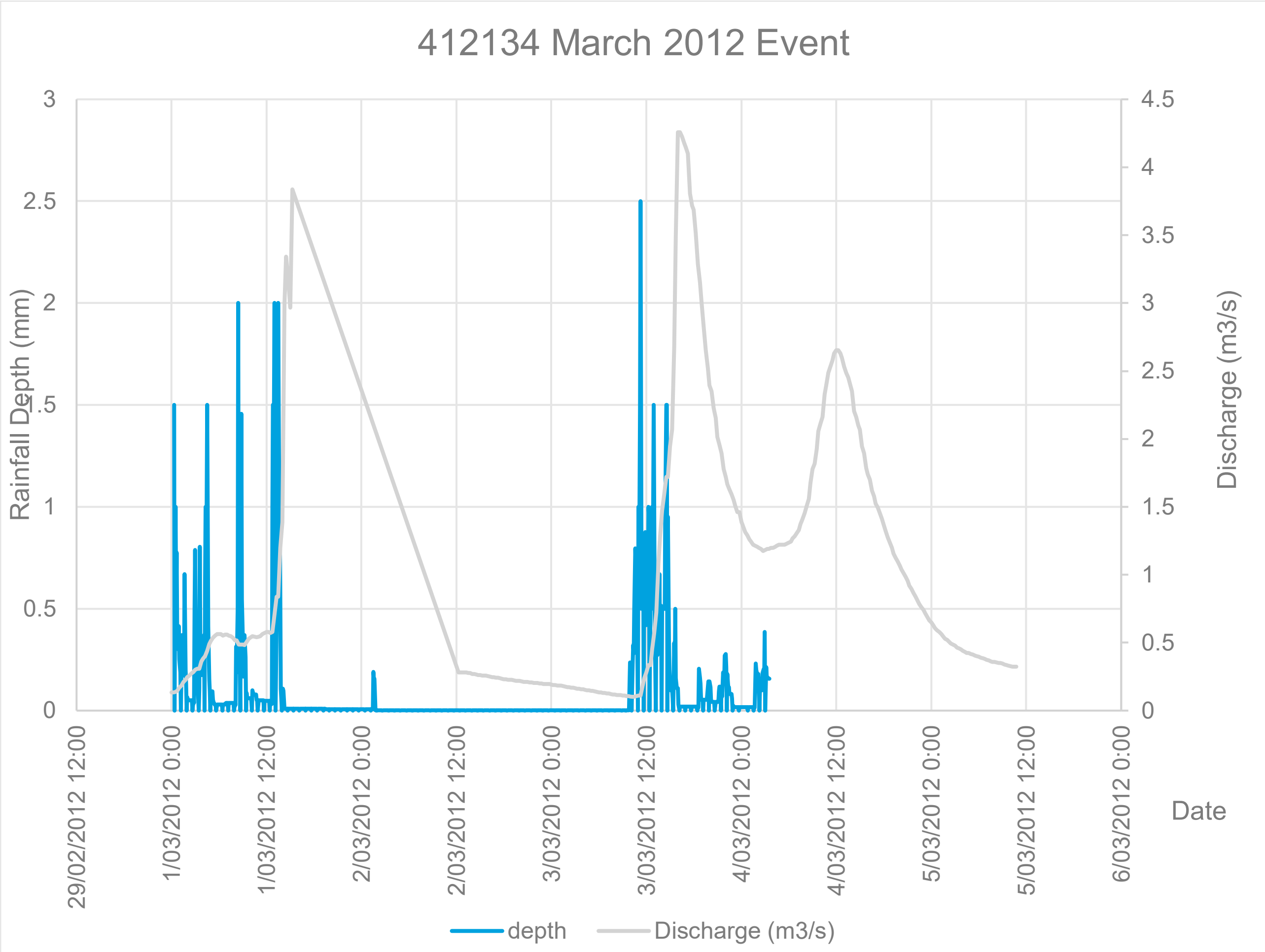


Figure A.9 1–3 March 2012 flow and rainfall

Close inspection of the gauge data indicates that the no data was recorded at the gauge between 3pm on 1 March 2012 and 12pm on 2 March 2012. The gauge records indicate no data available (error code 201). The 3 to 4 March event was therefore considered in isolation.

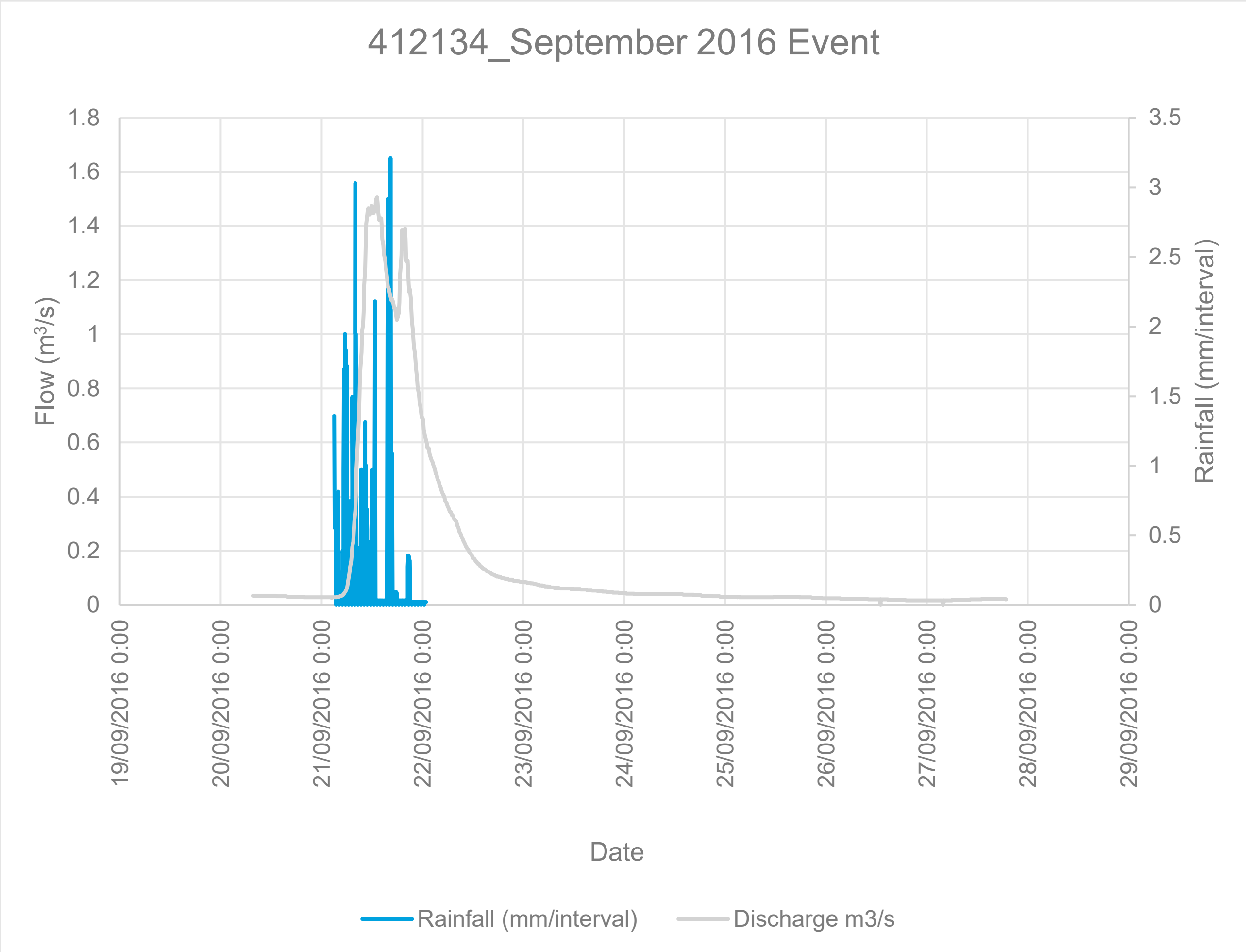


Figure A.10 September 2016 flow and rainfall

The BOM describe September 2016 is the wettest September on record for NSW. (Monthly Climate Summary for NSW, Product code IDCKGC5R0, 4/10/2016). For the region (southwest slopes), Cootamundra Airport and Young Airport recorded their highest September rainfall total on record, 204.6mm and 216.2mm on record and Temora Airport recorded a very high record with 204.8mm for the month. (Monthly Climate Summary for NSW, Product code IDCKGC5R0, 4/10/2016).

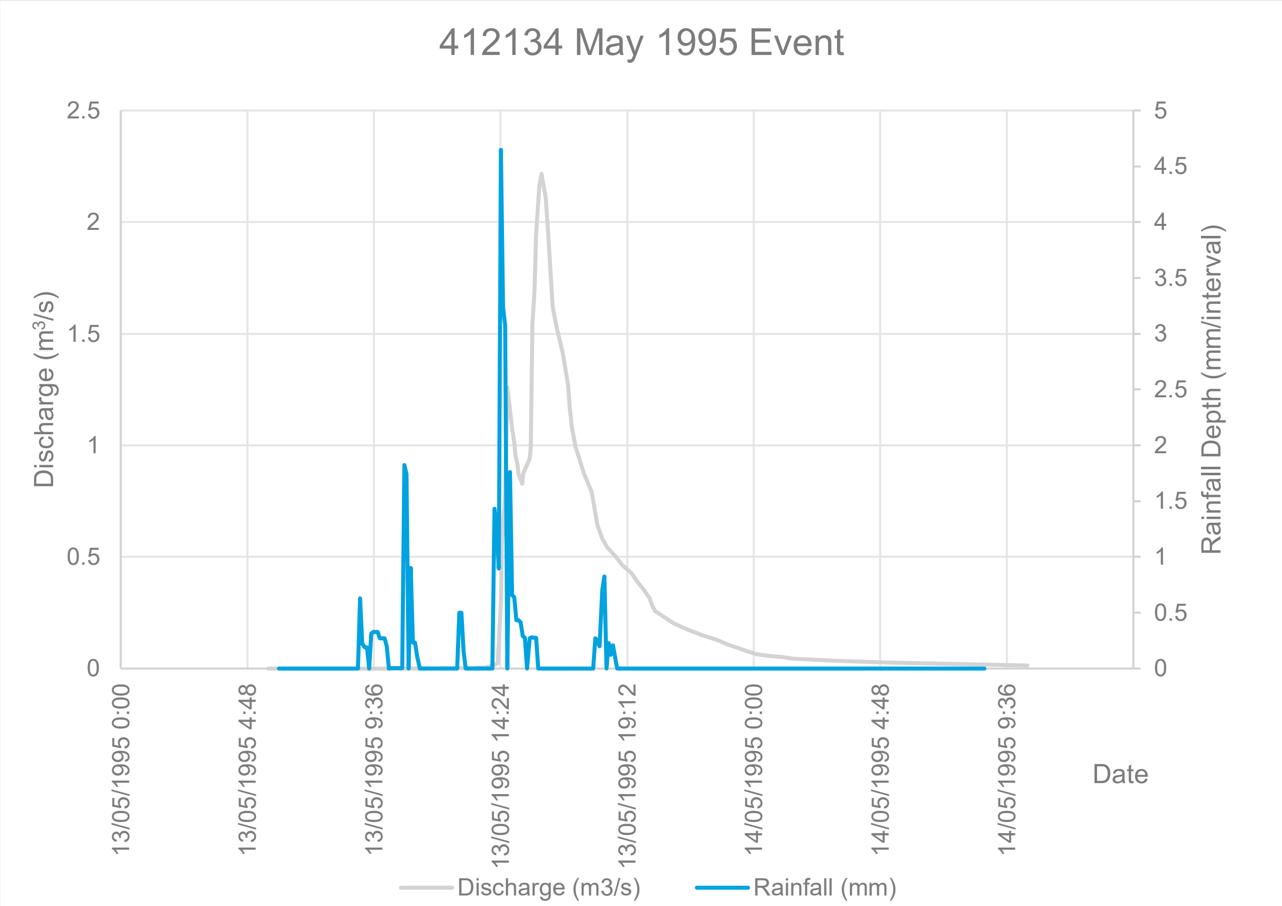


Figure A.11 May 1995 recorded flow and rainfall

The May 1995 flow and rainfall data indicated that the response time was 1.5 hours for the peak rainfall to peak flow.

A.4.2 Flood frequency analysis

The historic data was sorted into annual maximum values to enable a flood frequency analysis to be completed. Flood Frequency Analysis (FFA) refers to procedures that use recorded and related flood data to identify underlying probability model of flood peaks, at a particular location in the catchment. The information from an FFA can identify the probability of historic flood events and select the range for input parameters input into hydrologic and hydraulic models.

The TUFLOW FLIKE software package has been utilised to analyse the annual maximum values from the Wattle Creek 412134 Gauge for the period of 1988 to 2017. The data available includes instantaneous maximum recorded values for each calendar year of record and is presented in the table below. A close review of this data indicates that there were many years where there are significant data gaps in the records, but the records did indicate that many of the annual maximums occurred in September.

Table A.11 Wattle Creek at Dudauman 413134 gauge annual maximum discharge

Year	Discharge (m^3/s)
1988	0.3
1989	0.6
1990	1.4
1991	0.2

Year	Discharge (m^3/s)
1992	0.8
1993	1.1
1994	1.1
1995	2.2
1996	0.5
1997	0.2
1998	0.5
1999	0.1
2000	0.3
2001	0.0
2002	1.4
2003	0.0
2004	0.0
2005	0.5
2006	0.0
2007	1.1
2008	0
2009	0.2
2010	5.5
2011	1.1
2012	4.3
2013	0.1
2014	0.1
2015	0.8
2016	2.9
2017	0.2
2018	0.1
2019	0.00

The Bayesian Log Pearson III fit method appeared to provide the best fit for the data. The records include a zero-flow record and several low flow records. The Grubbs Beck test was then used to identify potential influential low flow (PILF) discharges. The FLIKE software is able to perform the test. From the records it identified 8 PILFs below the censor value of 1.25m^3/s. The censored data also fitted well with the Bayesian LPIII distribution. The largest flow of 5.52m^3/s was estimated to have an AEP of 1.9%. The FFA results are presented in the table below and show that the censored data produced higher design flow estimates than when all data (including zeros) were included. The plots below show a good correlation between the expected estimate and the recorded data between 2% and 5% AEPs. This is expected since the period of record is 24-30 years of data.

Table A.12 Flood frequency analysis

AEP (%)	Non-censored data discharge m^3/s	Grubbs Beck censored data discharge m^3/s
50	0.4	0.33
20	1.51	1.35
10	2.7	2.6
5	4.15	4.3
2	6.4	7.25
1	8.3	10.01

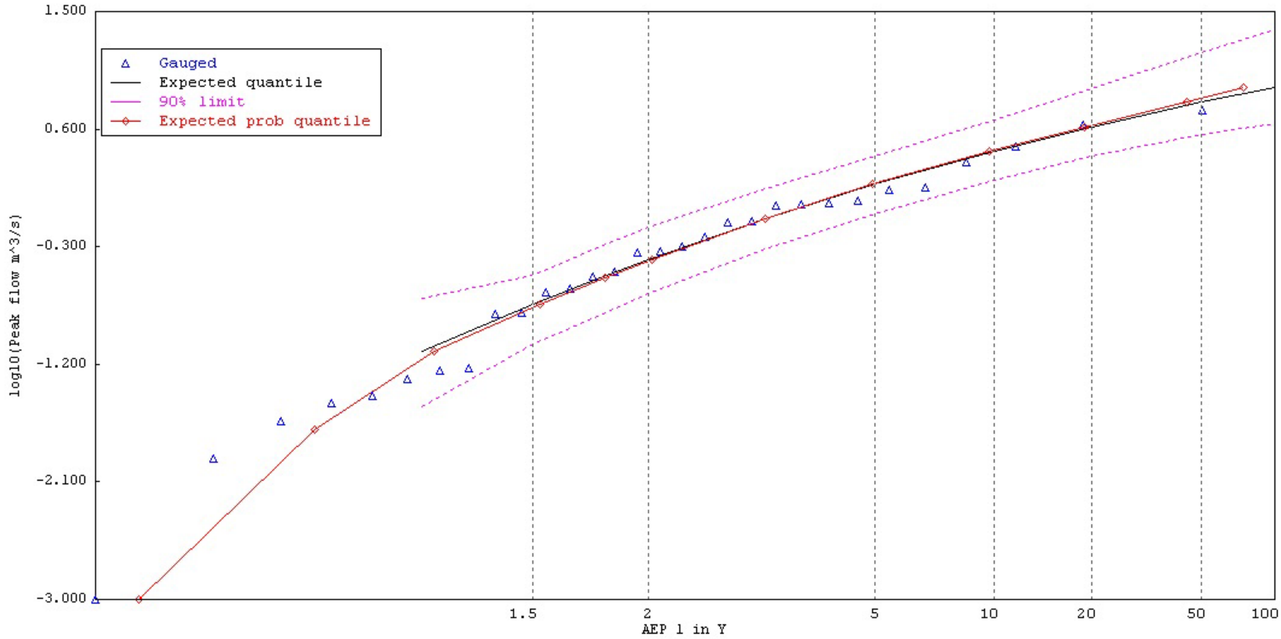


Figure A.12 Gauge 412134 Non-Censored Data Bayesian LPIII plot

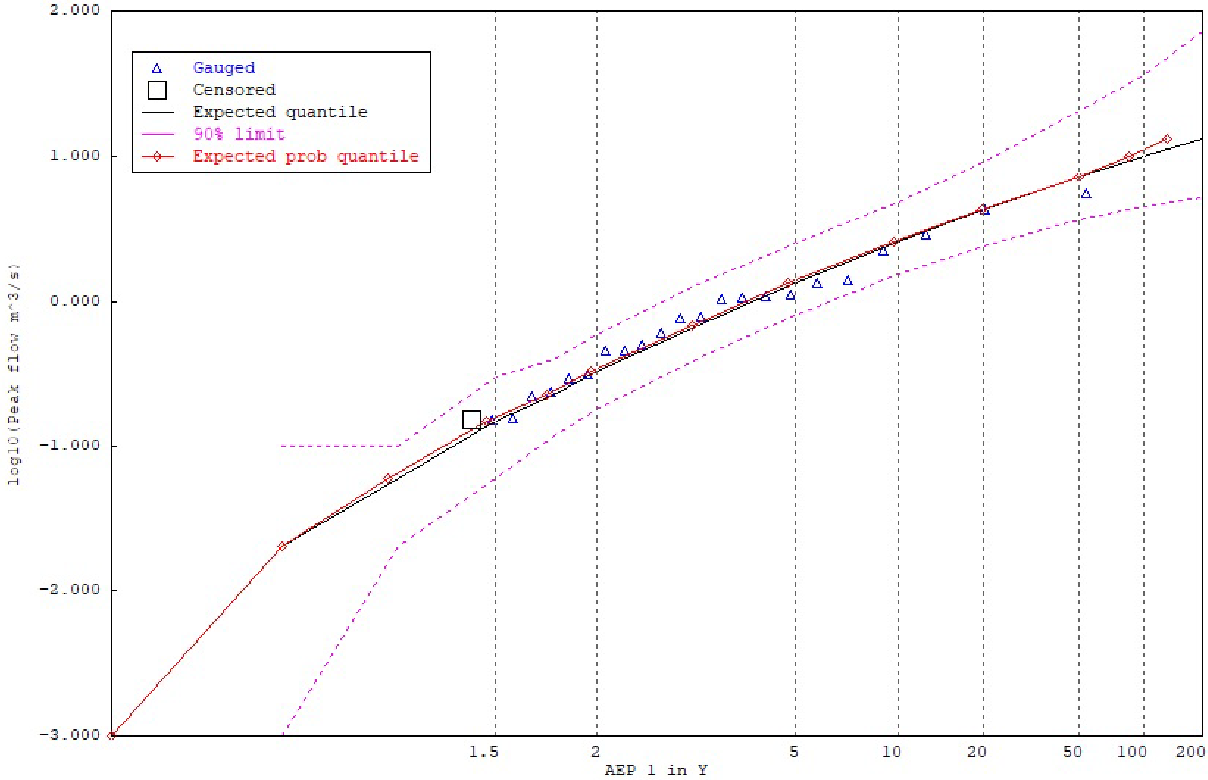


Figure A.13 Gauge 412134 Censored Data Bayesian LPIII Plot

The FFA was then used to estimate the return period for the highest four flows events. The table below presents the data for the estimated AEP based on the censored data FFA. For comparison, the December 2010 event was estimated to be a 1.9% AEP when the data was not censored.

Table A.13 Estimate AEP of historic events

Historic event	Estimated AEP %(censored data)
December 2010	1.9%
March 2012	4.8%
September 2016	8%
May 1995	11%

Table A.14 Preliminary design flows at Wattle Creek at Dudauman gauge 412134

AEP	Modelled median flow (m^3/s)	FFA expected quantile (censored) (m^3/s)	RFFE expected quantile (m^3/s)
5	4.68	4.73	4.93
2	10.26	7.54	7.27
1	11.39	10.2	9.44