



Report

Bonney Downs - Post Development Hydrology Study

Pilbara Decarbonisation Wind Generation Project

19 December 2024

AUSS0003-0000-HG-REP-0004

Rev: A



EXECUTIVE SUMMARY

A post development hydrological assessment was carried out to evaluate the potential changes to existing surface water flow regime as a result of the development of the Bonney Downs Wind Project. The assessment is intended to support detailed design of infrastructure and environmental approvals submissions.

The Bonney Downs site is located approximately 25 km southwest of the town of Nullagine in the Pilbara region of Western Australia. The development area is located in the headwaters of four river catchments, including the Nullagine River, Coongan River, Shaw River and the Fortescue River. Main watercourses that drain through the site include the Nullagine River to the east, Bonnie Creek to the north and Coongan River to the west.

A baseline hydrological assessment was completed in 2023 to assess the existing surface water flow behaviour within and surrounding the Bonney Downs development area. TUFLOW hydraulic models, developed as part of the baseline study, were updated by incorporating the project design and layout to evaluate the post development surface water flow characteristics in this assessment. The degree and extent of change to the flow regime, and the associated impact on creek hydrology/morphology were determined by comparing the flood elevations and velocities for various events under baseline and post development conditions.

Results from the model simulations indicate that changes to surface water flow regime are localised and limited to areas near the turbine access track/waterway crossings. Disruptions to regional surface water flows are considered negligible (< 1 % change in flow volume and < 2 % change in flood depths and velocities) for all modelled events.

Impacts of the minor crossings (with catchment areas < 5 km²) on creek hydrology and morphology are low. Major waterway crossings (with catchment areas > 5 km²) have the potential to cause greater changes to the flow regime, but these changes are localised and unlikely to propagate beyond the proximity of the structures to impact on regional surface water flows. This include the Bonnie Creek crossing, which effects are not expected to impact on Bonnie Pool, a sensitive receptor located approximately 7 km downstream of the crossing.

Changes to flow velocities in vicinity of the waterway crossings may influence the existing geomorphological conditions of the creek systems. Higher rates of sediment deposition may occur upstream of the road embankment due to backwater development. Local erosion/scour may occur at the outlet of the culvert structures due to increases in flow velocities. Erosion control measures are therefore proposed to mitigate this risk.

Based on the findings of the assessment, the overall impacts of the development on regional surface water flow regime are considered to be very low.

It is recommended that appropriate management and mitigation measures be implemented to further minimise the impacts on creek hydrology and morphology within the development area. These include the development and implementation of a construction surface water management plan, installation of necessary erosion control measures at the inlet/outlet of culvert structures and rock armouring of the floodway batters and driving surface to minimise erosion and sedimentation downstream.



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1 INTRODUCTION

1.1 Background

Fortescue is seeking to achieve 100 % decarbonisation of its Pilbara operations by 2030. To achieve this, a series of power generation and transmission projects are planned.

Preliminary modelling has shown that power output totalling 900 MW of wind generation and 1.2 GW of solar PV generation is required to achieve the 100 % decarbonisation target.

A Pre-Feasibility (PFS) study has been undertaken by Fortescue to develop 201 wind turbines within the Bonney Downs site. The turbine locations have been selected based on land access and power generation potential. The energy produced by the selected turbines is to be fed into the Pilbara Transmission Project (PTP) Stage 8 transmission line, a 56 km 220 kV overhead transmission line, that connects Bonney Downs to Christmas Creek mine in the south.

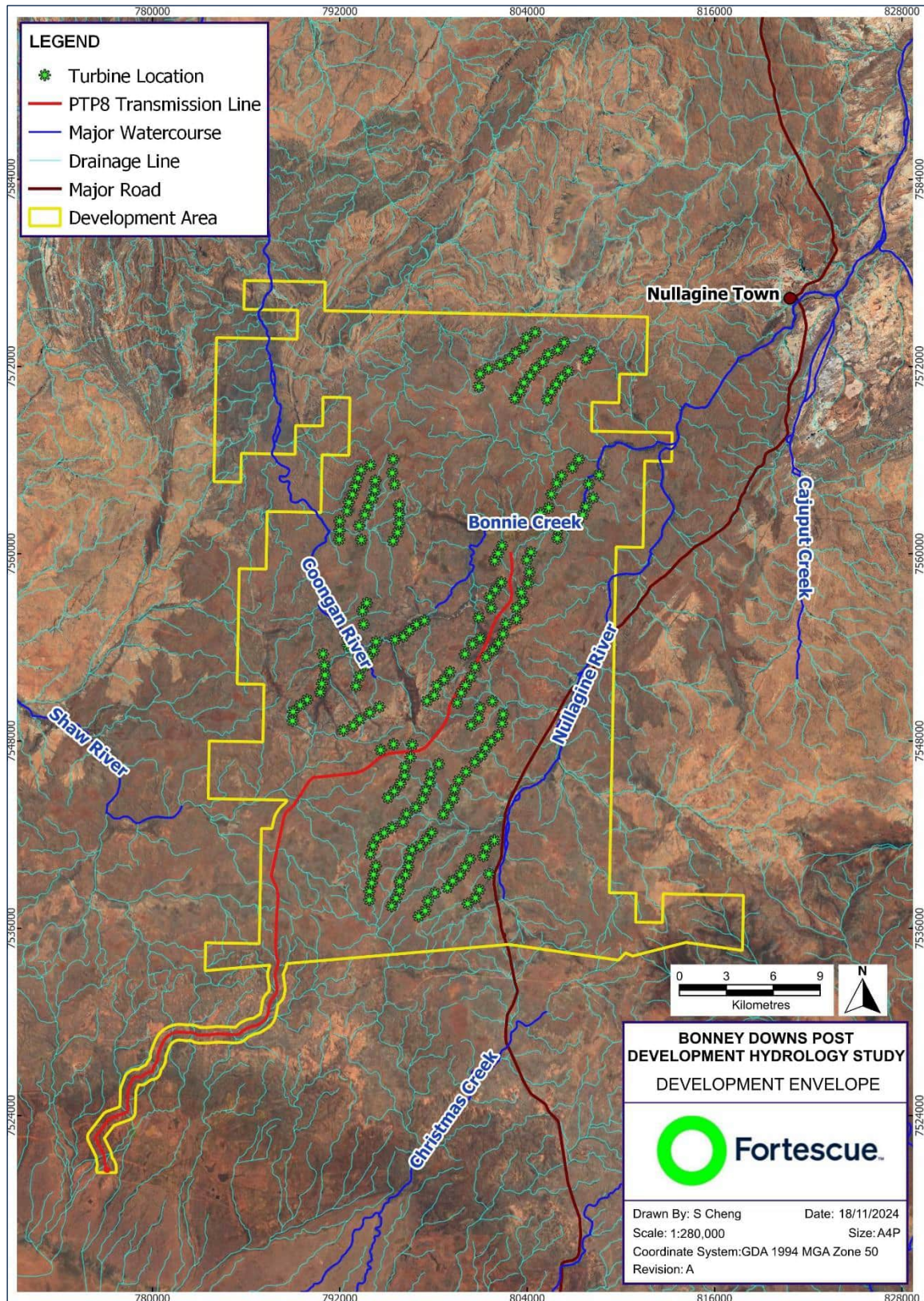
The Bonney Downs Development Envelope covers an area of 103,000 ha and is located approximately 25 km southwest of the town of Nullagine in the Pilbara region of Western Australia. The Bonney Downs Development Envelope and the proposed wind turbine locations are shown in Figure 1.

A baseline hydrological assessment was completed in 2023 to assess the existing surface water flow behaviour within and surrounding the Bonney Downs development area in support of the PFS layout/design of wind turbines, associated infrastructure and environmental approvals process. Details of the baseline assessment works, including site characteristics, design event hydrology and flood modelling results, are provided in the Bonney Downs Baseline Hydrology Study Report (AUSS0003-0000-HG-REP-0001).

To support the environmental approvals process, a post development hydrological assessment is also required to evaluate the potential impacts of the project design and layout on the existing surface water flow regime. This report summarises the post development assessment, including proposed design, post development hydrology and flood modelling results, and findings of the surface water impact assessment, for the wind power generation area. A standalone report has been produced for the assessment of the PTP Stage 8 transmission line corridor and is provided in 540PT-5632-AS-HY-0001.



Figure 1: Bonney Downs Development Envelope





1.2 Objectives

The objectives of the post development hydrological assessment are summarised as follows:

- Undertake hydraulic modelling to estimate the surface water flow characteristics, including flow volumes, flood depths and velocities, under post development conditions. The modelling considers the project design and layout including the turbine hardstands, access tracks and waterway crossings within the development and focuses on:
 - Evaluating the potential impacts of major creek crossings (crossings along named rivers/creeks/tributaries) on existing flood levels, velocities and subsequently the erosion/scouring potential of the river/creek systems.
 - Evaluating potential changes to natural surface water flows and quality due to land use/cover changes as a result of the development.
- Conduct impact assessment and provide recommendations for mitigations based on findings of the modelling.
- Undertake post development inundation mapping, including afflux mapping (i.e., difference mapping) to illustrate potential changes to the natural flow regime as a result of the development.
- Develop a post development hydrological assessment report documenting the methodology, findings of the modelling work and impact assessment, and mitigation recommendations suitable for environmental approvals submission (this document).

1.3 Reference Documentation

Below is a list of reference documentation to be read in conjunction with this report.

Table 1: Reference Documentation

Document	Document Number	Description
Bonney Downs Baseline Hydrology Study	AUSS0003-0000-HG-REP-0001	Baseline hydrological assessment to develop an understanding of surface water flow behaviour within and surrounding the Bonney Downs development area. The assessment is intended to support PFS layout/design of infrastructure and regulatory approval submissions.
Bonney Downs to Christmas Creek Transmission Corridor (PTP Stage 8) - Baseline Hydrology and Qualitative Impact Assessment Report	540PT-5632-AS-HY-0001	Baseline hydrology study and qualitative impact assessment conducted for the PTP Stage 8 transmission line corridor to support regulatory approval submissions and placement of structure systems.
PFS Civil Engineering Basis of Design	AUSS0003-0000-CI-BOD-0001	Basis of design document outlining design criteria, key assumptions and standards which inform the civil engineering PFS design.
PFS Wind Engineering Basis of Design	AUSS0003-0000-GR-BOD-0001	Basis of design document outlining design criteria, key assumptions and standards which inform the PFS wind turbine layout.
Electrical, Control and Telecommunications Basis of Design	AUSS0003-0000-EL-BOD-0001	Basis of design document outlining design criteria, key assumptions and standards which inform the PFS electrical design.



Document	Document Number	Description
Bonney Downs Creek Crossing Design Standard Review	AUSS0003-0000-PM-KDN-0002	A Key Decision Note summarising the assessment undertaken to define an acceptable design standard for waterway crossings along turbine access tracks within Bonney Downs.
PFS Civil Engineering Assessment	AUSS0003-0000-CI-REP-0001	Civil engineering assessment report detailing the PFS design of the proposed wind farms to inform a Class 4 CAPEX estimate and to support the environmental approvals process.

2 CATCHMENT CHARACTERISTICS

This section provides a summary of the catchment characteristics for the Bonney Downs development area. Detailed analyses of the climatic and hydrologic behaviours of the contributing catchments to the site are provided in the Baseline Hydrology Study Report (AUSS0003-0000-HG-REP-0001).

2.1 Climate

The Pilbara region is a semi-arid to arid environment characterised by hot summers and warm winters. The region experiences climate extremes, where severe droughts and major floods can follow in close succession. Using the Bureau of Meteorology (BoM) Köppen climate classification system¹, the Bonney Downs development area is described as desert: hot (persistently dry).

Rainfall records are available from BoM weather station at Bonney Downs (BoM Ref: 004006), the closest operating weather station with sufficient long-term data record for analysis. The average annual rainfall estimated for the period from 1907 to 2024 is 325 mm (based on October to September water year). Rainfall is highly variable between years with the annual recorded rainfall for the area varying from 40 mm to 910 mm (Figure 2).

Rainfall is also highly seasonal with approximately 70 % of the annual total occurring between December and March. It is typically associated with tropical low-pressure systems and thunderstorm activities from the monsoonal trough that develops over northern Australia during summer. Winters are typically dry and mild though winter rain events can occur in June and July as a result of tropical cloud bands that intermittently affect the area. The mean monthly rainfall at Bonney Downs is presented in Figure 3.

The mean annual Class A pan evaporation estimated for the area (from BoM gridded data, 1975 to 2000) is approximately 3,449 mm, which exceeds the mean annual rainfall keeping the landscape typically dry. Monthly evaporation at Bonney Downs is provided in Figure 3.

¹ Climate classification maps, Bureau of Meteorology (bom.gov.au):
<http://www.bom.gov.au/climate/maps/averages/climate-classification/?mctype=kpn>



Figure 2: Annual Rainfall at Bonney Downs (BoM Ref: 004006)

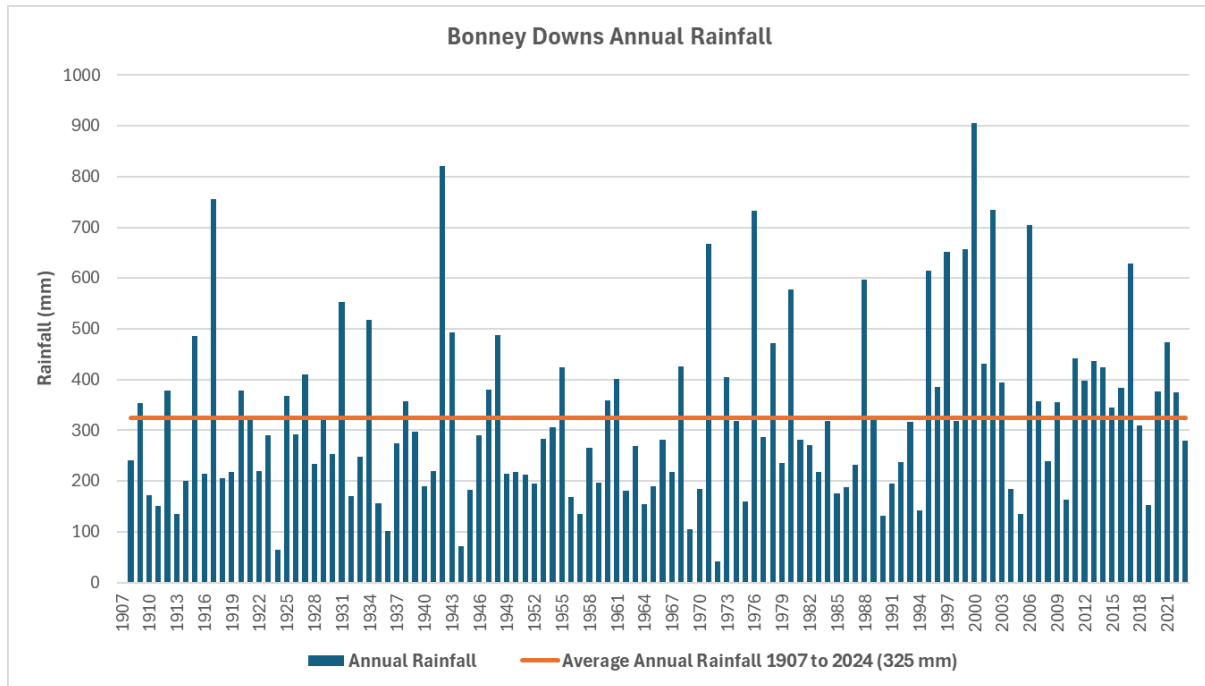
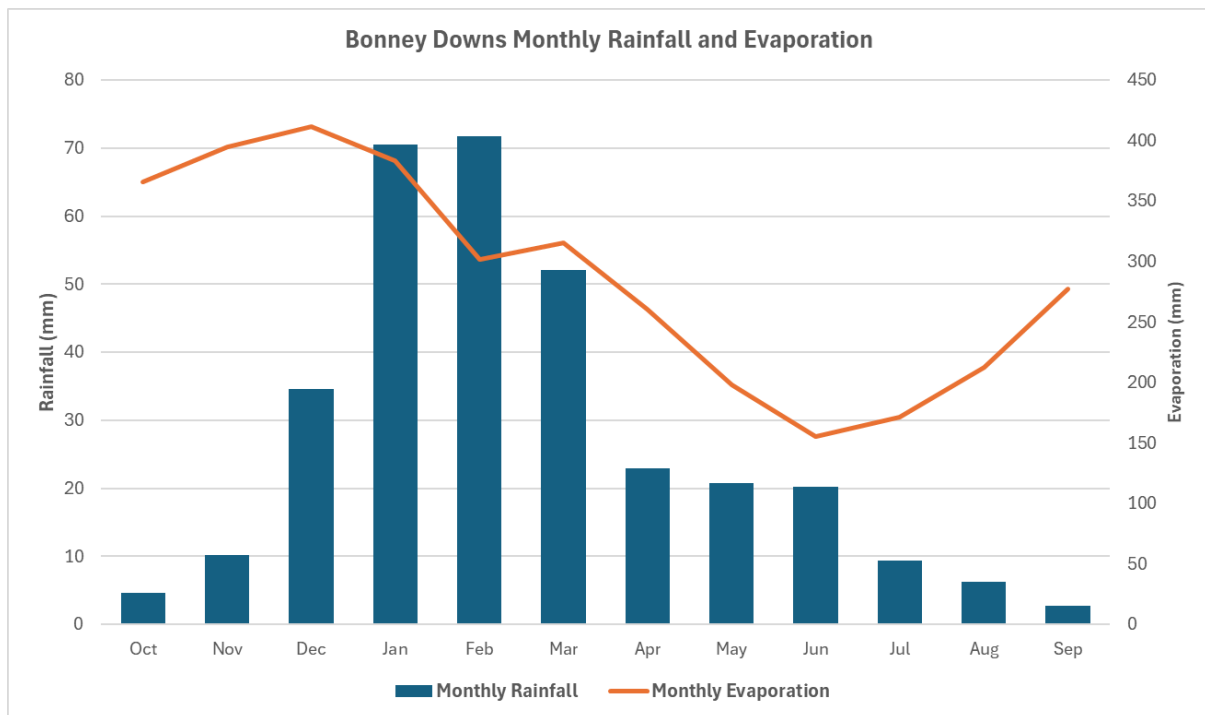


Figure 3: Monthly Rainfall and Evaporation at Bonney Downs (BoM Ref: 004006)





2.2 Catchments

The Bonney Downs development area is located in the headwater areas of four river systems, including the Nullagine River, Coongan River, Shaw River and the Fortescue River. The Nullagine, Coongan and Shaw Rivers flow in a general northerly direction towards the De Grey River, approximately 200 km downstream of Bonney Downs. Surface water flows along the southern boundary of the development area contribute to the Fortescue River which flows from east to west towards the sea. The regional catchments and flow direction around the Bonney Downs development area are illustrated in Figure 4.

The main watercourses that drain through the development area are presented in Figure 5 and include the Nullagine River to the east, Bonnie Creek (a tributary of the Nullagine River) to the north and Coongan River to the west. A number of ephemeral pools, including Bonnie Pool, are found along Bonnie Creek. Bonnie Pool was identified as an ethnographic site and has high indigenous heritage values for the Palyku people, the Native Title Group in the area. The Pool appears to be fed by both surface and groundwater flows and would be flushed/scoured during wet season flows (Strategen, 2013).

The general catchment characteristics for the main watercourses contributing to the development area are provided in Table 2.

Figure 4: Regional Catchments and Flow Direction around Bonney Downs Development Area

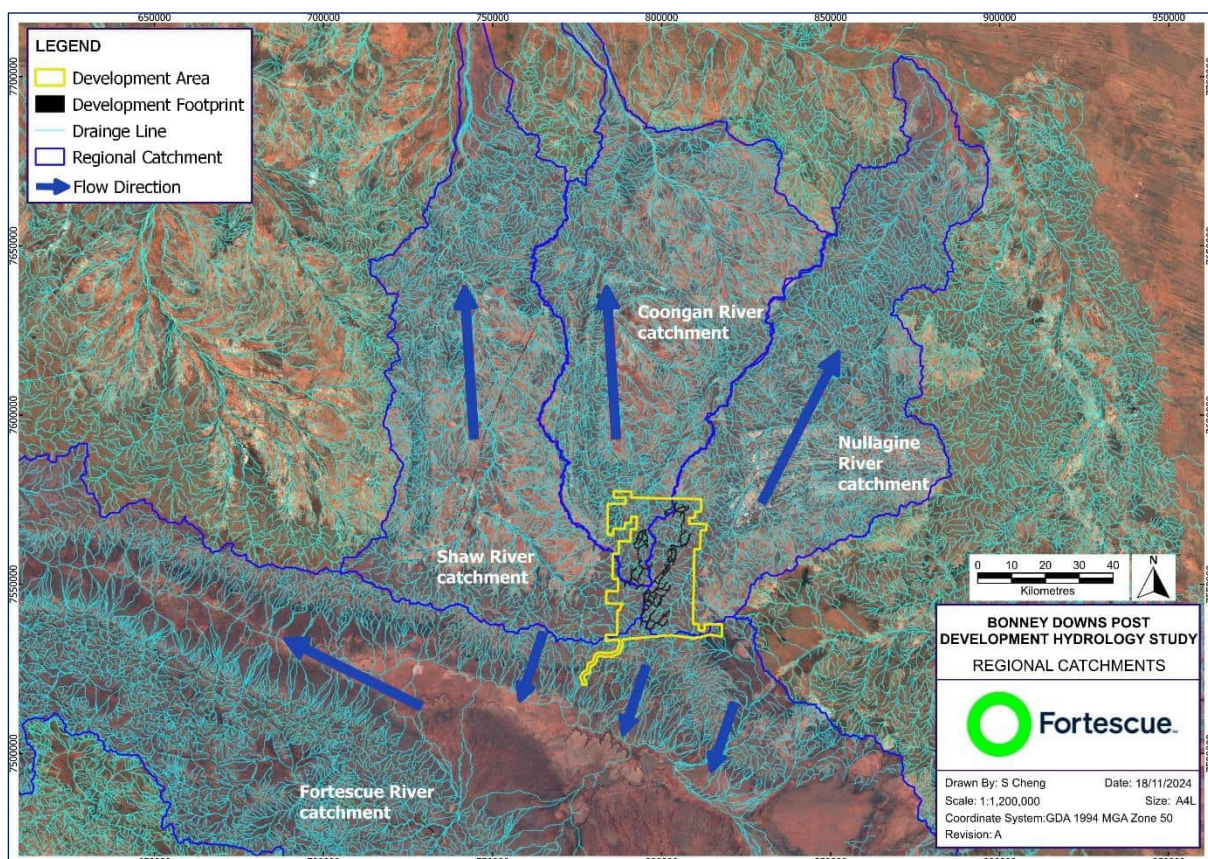




Figure 5: Bonney Downs Development Area Main Watercourses

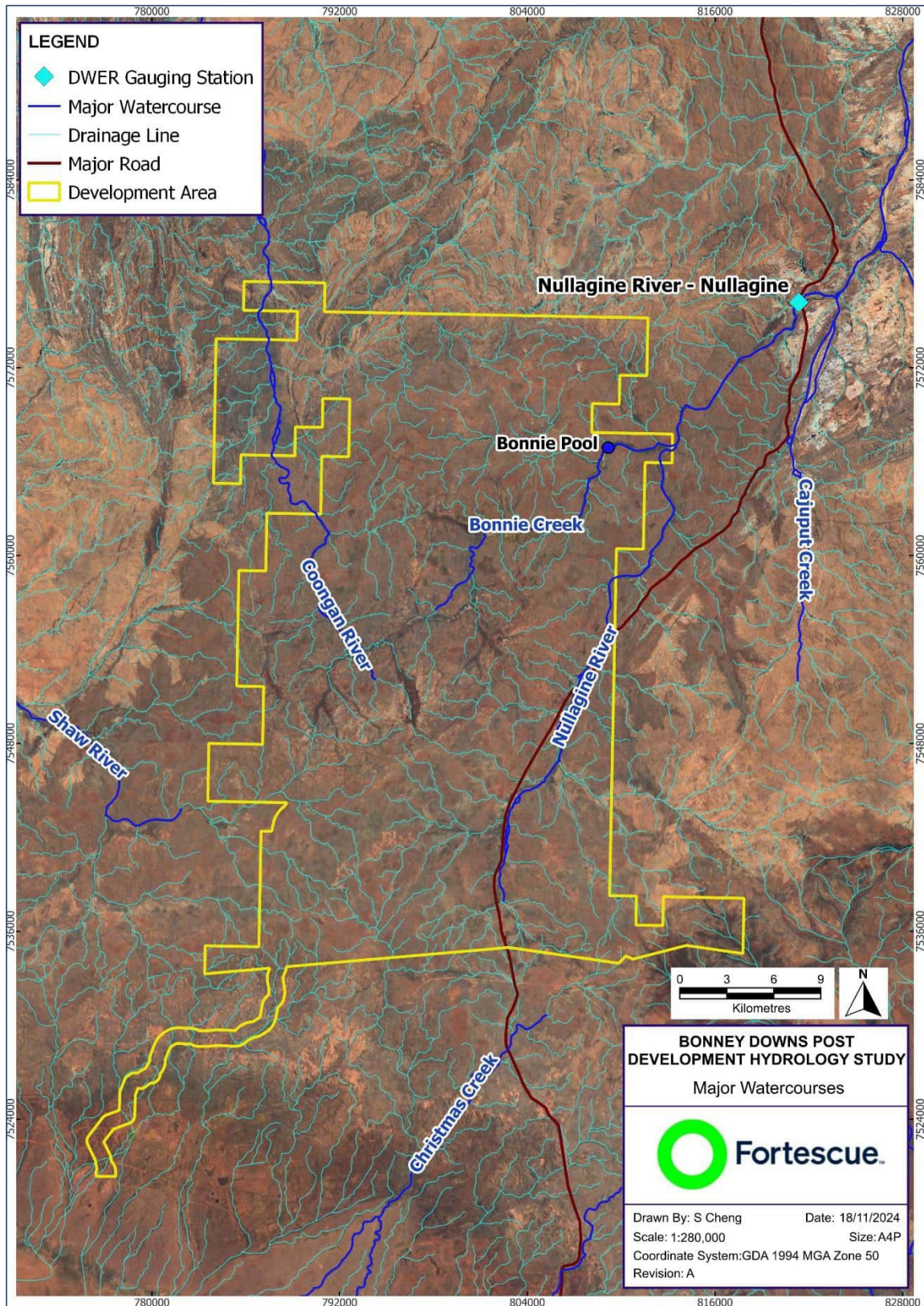




Table 2: Catchment Characteristics for Bonney Downs Main Watercourses

Creek System	Total Catchment Area (km ²)	Mainstream Length (km)	Equal Area Slope (m/km)
Nullagine River	7,219	261	1.29
Bonnie Creek	254	32.2	2.94
Coongan River	7,091	209	1.95

The Coongan River catchment contains one Department of Water and Environment Regulation (DWER) gauging station at Marble Bar (DWER ref: 710204). The gauging station is located approximately 100 km north of Bonney Downs and is in operation since 1966 with a total of 57 years of data record. The Nullagine River is gauged at two locations, Nullagine (DWER ref: 710004) and Tumbinna Pool (DWER ref: 710008) located approximately 25 km and 110 km northeast of the development area. The gauging stations are in operation since 1997 for Nullagine and 2002 for Tumbinna Pool, with a total of 27 and 21 years of data record.

Detailed analysis of observed streamflow records at Marble Bar and Nullagine gauging stations was carried out to inform the hydrologic and hydraulic modelling for the development area. Table 3 illustrates the peak flow estimates for the Coongan and Nullagine River catchments based on Flood Frequency Analysis (FFA) of streamflow records at Marble Bar/Nullagine.

Table 3: Peak Flow Estimates for Coongan and Nullagine River Catchments

DWER Stream Gauge	Catchment Area to Gauge (km ²)	Peak Flow (m ³ /s)					
		50 % AEP*	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1 % AEP
Coongan River at Marble Bar (710204)	3,736	508	1,148	1,736	2,463	3,721	4,972
Nullagine River at Nullagine (710004)	873	232	738	1,198	1,698	2,376	2,895

Note: Annual Exceedance Probability

2.3 Geomorphology

The geomorphology of the area is a reflection of the dissection of an ancient peneplain which is commonly referred to as the “Hamersley Surface” which is preserved as an extensive and incised plateaux with prominent erosion scarps.

Land surface types across the Pilbara have been defined based upon whether they are primarily erosional or depositional and secondly on genesis, soil and drainage features. In the Bonney Downs area the Bonney land surface type (erosional) predominates and is characterised by gently undulating stoney plains, rises and low hills of basalt, tributary drainage patterns and alkaline soils with relief typically up to 30 m.

The Robe land surface type is also found within the Bonney Downs project area and gives rise to conspicuous chains of flat-topped limonite mesas and buttes with steep breakaway faces. These are a source of iron ore in the form of pisolitic limonite with relief up to 50 m.



Minor alluvial plains are also found within the project area and exhibit as level to gently undulating plains commonly found between erosional surfaces and creek lines. These areas typically have mantles of ironstone grit and pebbles. They are subject to sheet water flow after rainfall.

Minor calcreted drainage plains are also occasionally present as level or extremely low relief (<9 m) within valley fill sediments within palaeovalleys and drainage systems. Undissected bodies may form mounds on valley floors, separated by alluvial channels.

Level to gently undulating basalt derived stoney plains and gilgai plains with clay soils can also be found occurring in the project area associated with drainage tracts.

3 PFS ENGINEERING DESIGN AND DEVELOPMENT FOOTPRINT

3.1 Basis of Design

The PFS engineering assessment for the Bonney Downs project includes the array design of wind turbines, electrical, control and telecommunications system design for the turbine groups, and civil design of turbine hardstand areas, access tracks and associated waterway crossings within the development. Design criteria, key assumptions and standards used to inform engineering design are provided in the basis of design documents listed in Table 1. Design criteria that are relevant to surface water management are summarised in the subsections below.

3.1.1 Turbine Layout, Hardstand Areas and Supporting Infrastructure

The following design criteria are adopted to minimise flooding impacts and disruption to natural surface water flows:

- Turbine locations to avoid major watercourses and associated 1 % Annual Exceedance Probability (AEP) floodplain area.
- The turbine hardstand areas to be positioned outside of noted flood prone areas and set at an elevation to achieve 300 mm freeboard to any surface water ponding levels up to the 1 % AEP storm event.
- Supporting infrastructure, including accommodation camp, substation compound, operations building, etc., to be located outside of flood prone areas and set at an elevation to achieve 500 mm freeboard to any surface water ponding levels up to the 1 % AEP storm event.

3.1.2 Turbine Access Track and Waterway Crossing

The civil design of turbine access track aims to minimise crossing major watercourses and flow paths. Where this is unavoidable due to the need to access the turbines, the following design principles are considered to minimise social/environmental impacts:

- Track alignment and crossing design to utilise existing cleared tracks where possible.
- Each watercourse is crossed only once within the development area.
- Crossing of major watercourses (named rivers/creeks) is minimised where possible.



- Waterway crossings to be positioned perpendicular to the flow direction where possible to reduce the effects of streamflow energy on the structure as well as impacts resulting from the redirection of flows against channel banks.
- Crossing design to minimise impacts to heritage sites/features located adjacent to watercourses.
- Crossing design to maintain flow continuity and minimise impacts to volume and flow rates of watercourses.
- Crossing design to avoid pools (permanent or semi-permanent), which are likely to be habitats for aquatic flora and fauna.
- Crossing to be designed to minimise disturbance footprint in watercourse.

The design approach to waterway crossings at Bonney Downs is outlined in Key Decision Note AUSS0003-0000-PM-KDN-0002. The crossings include a combination of floodways and culverts designed to maintain trafficability during storm events up to the 10 % (1 in 10) AEP, except for eight crossing locations (Figure 6) where the design standard is the 1 % (1 in 100) AEP event.

3.2 Waterway Crossing Design

The PFS civil design, including waterway crossing design, for Bonney Downs was completed and detailed in the civil engineering assessment report AUSS0003-0000-CI-REP-0001. The design footprints and crossing locations for Bonney Downs are presented in Figure 6.

Total 50 crossing locations were identified within the development area. Culvert and floodway assessment, including estimation of culvert size, number and roadway hydraulics, was carried out for the crossing locations. Key design inputs/parameters considered in the assessment are summarised in Table 4. Findings of the culvert and floodway assessment are provided in Appendix A and design details of the major waterway crossings, i.e., those that intersect major watercourses (named rivers/creeks) and/or with contributing catchment areas exceeding 5 km², are presented in Table 5.



Figure 6: PFS Design Footprint for Bonney Downs

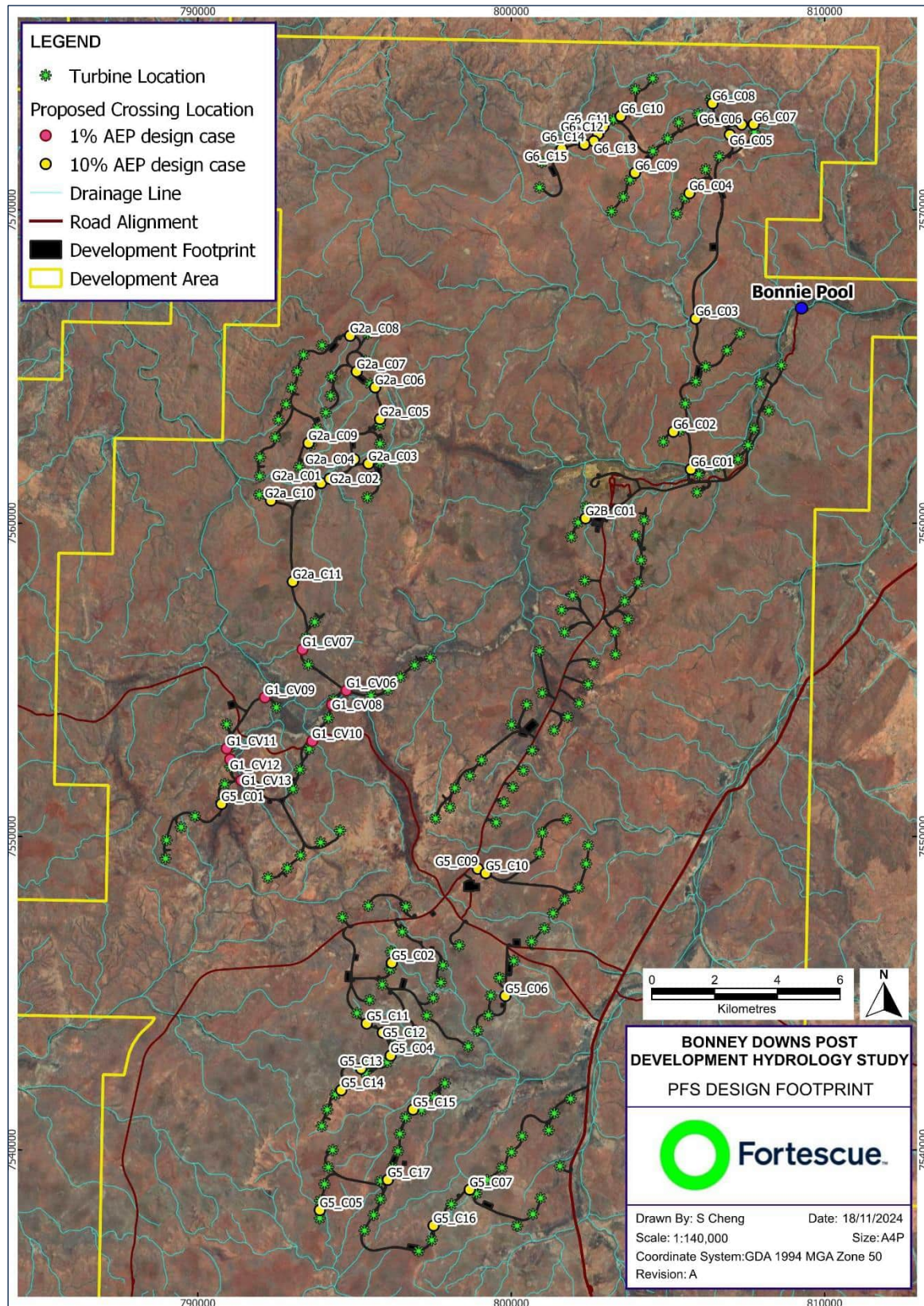




Table 4: Key Design Inputs/Parameters for Culverts and Floodway Assessment

Input/Parameter	Value	Comments/Reference
Design case	10 % AEP trafficable 1 % AEP for eight crossing locations as shown in Figure 6	Bonney Downs Creek Crossing Design Standard Review (AUSS0003-0000-PM-KDN-0002) One of the following options was adopted for design based on the size of the contributing catchment, magnitude of the design peak flow rate and design case: <ul style="list-style-type: none"> • Floodway with culverts – pipes to be placed underneath the road to maintain continuous flow. Creek flows for larger storms will overtop the road. • Low-level floodway– Road crossing at or very close to the natural creek level. Creek flows will overtop the road for all storm events. • Culvert crossings – pipes to be placed underneath the road designed to accommodate up to the 1 % AEP event. This design option was adopted for crossings G1_CV06, G1_CV07, G1_CV08, G1_CV09, G1_CV10, G1_CV11, G1_CV12, G1_CV13.
Minimum culvert diameter	450 mm	Fortescue Civil Engineering Requirements (FFI-0000-CI-SOR-0001)
Minimum culvert cover	0.1 m for reinforced concrete box culvert (RCBC) 0.6 m for corrugated steel pipe (CSP)	(Austroads, 2023)
Minimum culvert slope	0.25 %	(Austroads, 2023)
Maximum flood depth over road	0.2 m	Maximum passable depth for conventional cars (MRWA, 2006)
Maximum flow velocity over road	2.0 m/s	Limiting velocity based on the vulnerability thresholds defined by Smith et al., 2014 (ARR, 2019)
Road flood hazard classification (depth x velocity)	0.3 m ² /s	Flood hazard classification limits defined by Smith et al., 2014 (ARR, 2019)



Table 5: Findings of Culverts / Floodway Assessment for Major Waterway Crossings in Bonney Downs

Parameter	G1_CV08	G1_CV10	G2a_C01	G2a_C02	G2a_C04 ⁽¹⁾		G6_C01 ⁽²⁾		G6_C03
Lat	-22.09	-22.10	-22.03	-22.03	-22.02		-22.02		-21.98
Long	119.85	119.85	119.85	119.85	119.86		119.96		119.96
Road design level (m AHD)	481.48	482.69	459.46	460.50	465.91		446.77		443.16
Design case	1 % AEP			10 % AEP					
Catchment area (km ²)	5.52	11.2	7.62	7.91	9.22		86.4		58.2
Peak flow (m ³ /s)	75.8	115.0	39.7	41.6	51.3		198.7		167.5
Crossing type	Culvert crossing			Floodway with culverts					
Inlet elevation (m AHD)	479.52	478.65	457.227	458.11	463.42	464.58	443.66	444.22	439.57
Outlet elevation (m AHD)	479.26	478.42	456.824	458.04	463.18	464.28	443.59	444.15	439.37
Culvert type	RCBC			CSP					
Culvert size (mm)	1800 H x 2100 W	2100 H x 2100 W	1650	1800	1800	1200	2400	1800	3000
No. of barrels	10	9	6	6	6	3	10	6	8
Culvert length (m)	26.7	38.3	25.6	29.0	27.3	22.8	31.1	28.9	34.5

Note:

- (1) G2a_C04 is located along a braided section of a watercourse. Hence two sets of culverts are required to manage multiple flow paths.
- (2) G6_C01 represents the crossing location at Bonnie Creek and is located along a braided section of the watercourse. Hence two sets of culverts are required to manage multiple flow paths.



4 MODELLING APPROACH

4.1 Overview

Hydrologic and hydraulic modelling of the local catchments in the Bonney Downs development area was undertaken using TUFLOW. TUFLOW is a linked 1D/2D hydrodynamic computational engine for simulating free-surface long wave propagation process (tides, floods, tsunamis, dam breaks) by solving the full one- and two-dimensional versions of the Navier-Stokes equations incorporating all physical terms including inertia (1D and 2D) and sub-grid turbulence (2D) (BMT, 2018).

Two separate TUFLOW models (BD1 and BD2) were developed as part of the baseline assessment to assess the existing surface water flow characteristics, i.e., flood peaks, flow depths and velocities, for the development area. Detailed discussion of the methodology, model inputs/assumptions, model calibration/validation process, sensitivity analysis and modelling results are provided in the baseline hydrology report (AUSS0003-0000-HG-REP-0001).

The baseline models were used and updated by incorporating the project design and layout (as discussed in Section 3) to evaluate the post development surface water flow characteristics in this assessment.

4.2 Design Rainfall

Design rainfall depths, i.e., Intensity-Frequency-Duration (IFD) data, were sourced from the BoM Design Rainfall Data System (BoM, 2016) for the BD1 and BD2 model areas, with associated temporal pattern ensembles for the Rangelands region (where Bonney Downs is situated) extracted from the Australian Rainfall and Runoff (ARR) Datahub (Babister et al., 2016).

As the catchments contributing to the development area are larger than 20 km², consideration of spatial variability of rainfall was undertaken. Regional gridded IFD data was extracted to assess spatial variability of the design rainfall estimates in the model. The mean gridded IFD values were checked against the point IFD values obtained for the centroid of the BD1 and BD2 model areas. The review showed that the point values closely replicate the mean gridded values, hence the point IFD data were considered appropriate for use in the modelling. The resultant IFD values adopted in this assessment are presented in Table 6 and Table 7.

Table 6: BD1 Point IFD

Duration (hour)	Rainfall Depth (mm)						
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1 % AEP	0.5 % AEP
0.5	20.6	29.0	34.5	39.9	46.7	51.9	59.6
1	26.5	37.3	44.5	51.5	60.6	67.5	77.4
2	32.6	46.3	55.8	65.1	77.5	87.2	100
3	36.5	52.5	63.7	75.0	90.2	102	117
6	44.3	65.6	80.9	96.7	119	137	157
12	54.1	82.5	103	125	157	182	210



Duration (hour)	Rainfall Depth (mm)						
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1 % AEP	0.5 % AEP
18	60.9	94.0	119	144	181	211	243

Note: Location where IFD data were extracted: Lat -22.02; Long 119.81

Table 7: BD2 Point IFD

Duration (hour)	Rainfall Depth (mm)						
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1 % AEP	0.5 % AEP
0.5	20.1	28.4	33.9	39.2	46.0	51.1	58.8
1	25.7	36.3	43.4	50.3	59.2	65.9	75.8
2	31.7	45.1	54.3	63.4	75.4	84.8	97.5
3	35.6	51.3	62.2	73.1	87.9	99.5	115
6	43.8	64.6	79.6	95.0	117	134	155
12	54.3	82.3	103	124	155	180	208
18	61.5	94.4	119	144	180	210	242

Note: Location where IFD data were extracted: Lat -22.08; Long 120.01

The assessment of the catchment response to rainfall was undertaken using an ensemble simulation approach whereby design rainfall and temporal patterns were combined and applied as a global uniform rainfall boundary within TUFLOW. The storm events and durations included in the ensemble are provided in Table 8.

Flood magnitudes are generally very sensitive to temporal patterns and thus the ensemble approach provides a straightforward means of avoiding the introduction of bias due to this source of variability (Ball, et al., 2016). The storm events and durations as listed in Table 8 were simulated in the TUFLOW models to determine the critical duration (i.e., the storm duration resulting in the highest peak flow) at various locations in the catchments within the development area.

Table 8: Design Events Assessed in TUFLOW

Storm Detail	Events Assessed
Annual Exceedance Probabilities (AEPs)	0.5% 1%, 2%, 5%, 10%, 20% and 50%
Design Storm Durations (hour)	0.5, 1, 2, 3, 6, 12 and 18
Temporal Patterns (TPs)	10 TPs for each storm duration

4.3 Model Development

Two-dimensional (2D) TUFLOW models (version 2023-03-AA) were used to assess the surface water flow characteristics for the Bonney Downs development area under baseline and post development conditions. The model extent, configuration and boundary conditions of the baseline and post development models are provided in Figure 7 and Figure 8. A summary of the input data and assumptions applied in the models are provided in Table 9.



Figure 7: Baseline Model Configuration

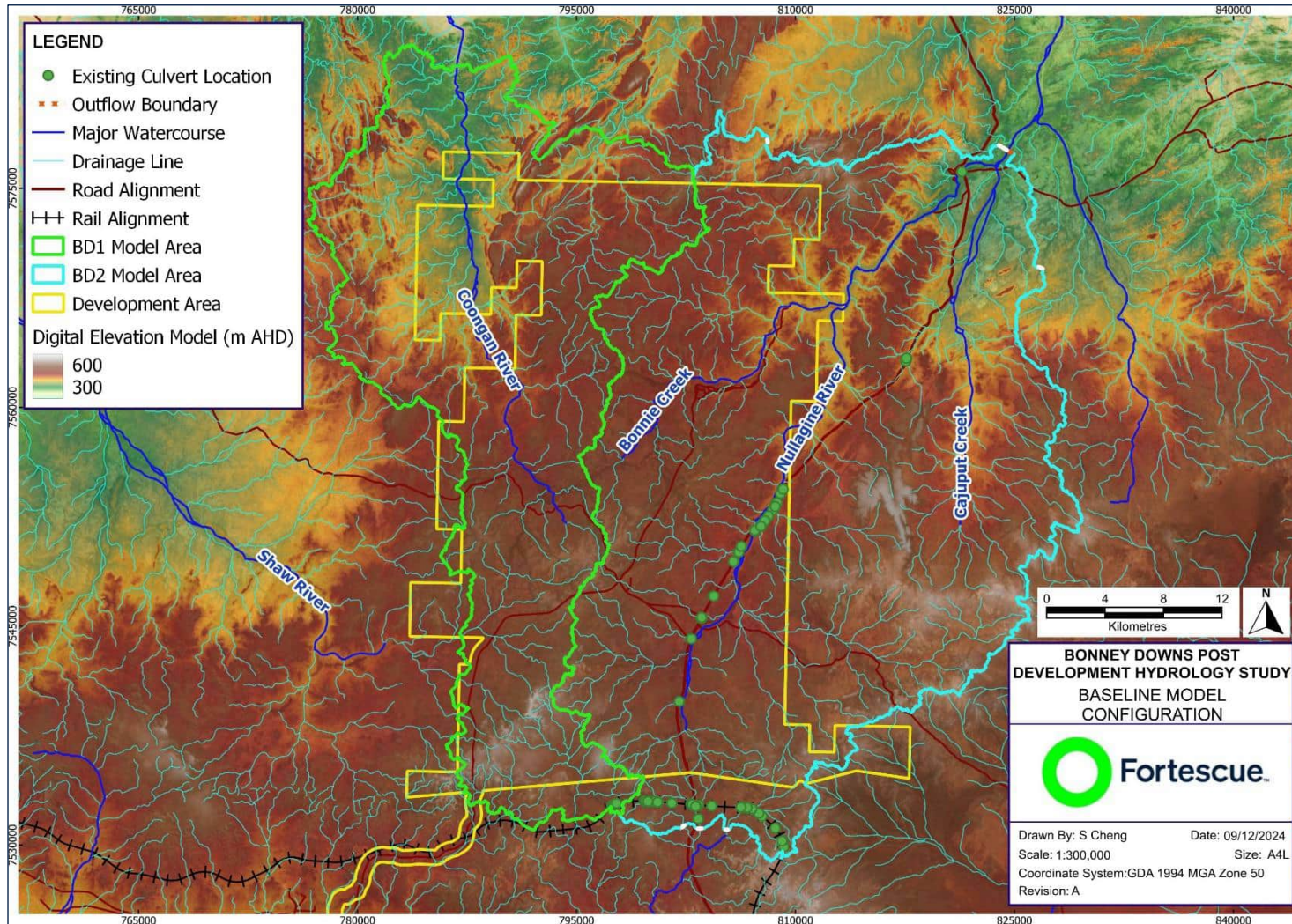




Figure 8: Post Development Model Configuration

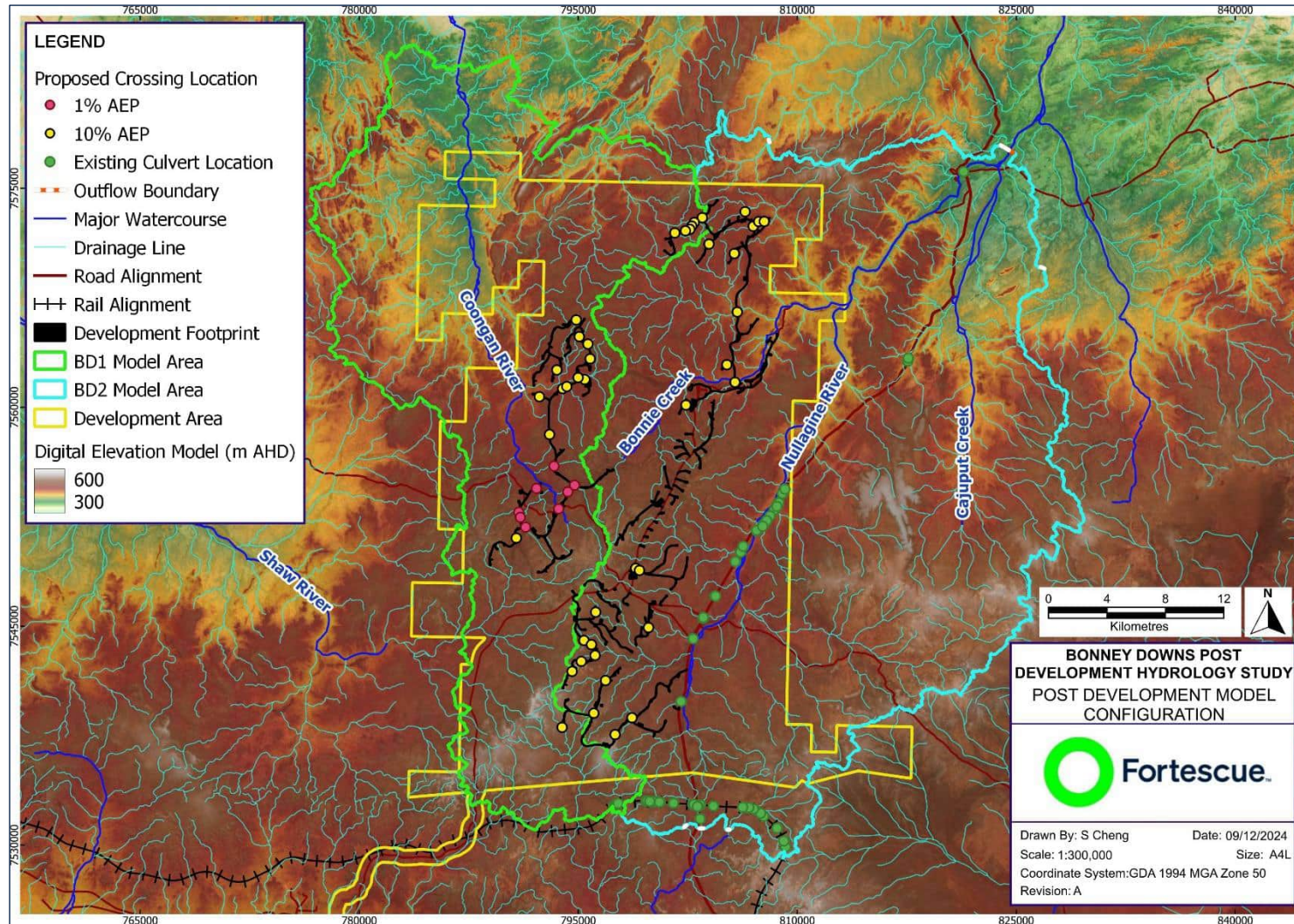




Table 9: TUFLOW Model Input Data and Assumptions

Model Input/Parameter	Baseline		Post Development	
	BD1	BD2	BD1	BD2
Rainfall				
Rainfall depths	Extracted from BoM Design Rainfall Data System (BoM, 2016) (see Section 4.2).		No change from baseline	
Rainfall losses	<p>Storm initial and continuing losses (ILs and CL) were applied to the design rainfall to generate rainfall excess and runoff within the TUFLOW models. The loss values adopted for modelling were determined based on a loss validation exercise to characterise regional rainfall-runoff relationship using gauged data for the Nullagine River at Nullagine (DWER Ref: 710004). A RORB rainfall-runoff model was developed for the gauged Nullagine River catchment and the results were compared against the FFA flood quantiles to determine a set of ILs and CL that produced the best fit across the AEP range of interest. Details regarding the loss validation assessment are provided in the Baseline Hydrology Study (AUSS0003-0000-HG-REP-0001).</p> <p>The final rainfall loss parameters adopted for modelling are: ILs = 20 mm CL = 2.6 mm/hr</p>		<p>No change from baseline for natural undisturbed area: ILs = 20 mm CL = 2.6 mm/hr</p> <p>For the proposed disturbed areas (i.e., project disturbance footprint), rainfall losses were reduced by 70 % to account for the reduction in infiltration due to land use change (increase in imperviousness) over areas of the turbine hardstands, supporting infrastructure and access track. The adopted rainfall loss parameters are: ILs = 6 mm CL = 0.8 mm/hr</p>	
Pre-burst	Median pre-burst rainfall depths were extracted from ARR Datahub (Babister et al., 2016). Pre-burst rainfall was removed from the Storm IL (ILs) prior to simulation in the model to represent the Burst IL (IL _B). Where pre-burst exceeded ILs, an IL _B of zero was adopted for simulation. The pre-burst depths used in the modelling are provided in Table 10 and Table 11.		No change from baseline	
Pre-wetting catchment	No pre-wetting was used due to the well-defined nature of the terrain within the development area and minimal observed micro-storage.		No change from baseline	
Temporal patterns	Areal TPs adopted for catchments within model domains with area greater than 75 km ² and point TPs for catchments less than 75 km ² .		No change from baseline	



Model Input/Parameter	Baseline		Post Development	
	BD1	BD2	BD1	BD2
Areal Reduction Factor (ARF)	Determined using parameters extracted from the ARR Datahub (Babister et al., 2016) for the Northern Coastal region and based on storm durations and catchment areas within the model domains (further details refer to the Baseline Hydrology Study, AUSS0003-0000-HG-REP-0001).		No change from baseline	
Terrain				
Model terrain	This forms the basis of the hydraulic model and was developed based on the following datasets: <ul style="list-style-type: none"> • 1 m photogrammetrically derived Digital Elevation Model (DEM) for Bonney Downs, dated February 2023 • 1 m LiDAR derived DEM for Kutayi, dated June 2020 • 1 m LiDAR derived DEM for Kutayi, dated June 2019 		The post development model terrain was developed based on the following datasets: <ul style="list-style-type: none"> • 1 m photogrammetrically derived DEM for Bonney Downs, dated February 2023 • 1 m LiDAR derived DEM for Kutayi, dated June 2020 • 1 m LiDAR derived DEM for Kutayi, dated June 2019 • 3D civil design for Bonney Downs incorporated in the models as 1 m resolution DEM • 3D breaklines to reinforce the road embankments at the crossing locations. 	
Total model area	690 km ²	1220 km ²	No change from baseline	
Base grid size (Sub-grid sampling distance) Quadtree grid sizes (Sub-grid sampling distance)	40 m (1m) – General development area 20 m (1 m) – Proposed infrastructure areas, roads and rail Due to the large model domain sizes, the models were simulated with a relatively coarse grid size of 40 m within the general development area, with discrete areas of higher resolution (20 m) over the proposed turbine hardstands and linear infrastructure such as road and railways. Sub-grid sampling (SGS) functionality was also used to sample the topographic data at 1 m resolution to maximum hydrologic routing accuracy and hydrologic estimate convergence between cell sizes.		No change from baseline	



Model Input/Parameter	Baseline		Post Development	
	BD1	BD2	BD1	BD2
Manning's n roughness	Sentinel-2 satellite data and aerial imagery were used to define classification bands of similar vegetation density and associated Manning's n roughness value. The following Manning's n roughness values were used in the models: <ul style="list-style-type: none"> • Cleared or barren land (0.03) • Typical Pilbara grasslands/main catchment areas (0.1, 0.1, 0.2, 0.04) – a depth-varying approach was adopted for these areas: <ul style="list-style-type: none"> • For flood depth $\leq 0.1\text{m}$, a roughness of 0.1 was used • For depth between 0.1 m and 0.2 m, roughness was interpolated between 0.1 and 0.04 • For depth $\geq 0.2\text{ m}$, a roughness of 0.04 was used. • Medium density riparian vegetation (0.06) • Higher density riparian vegetation (0.08) 		No change from baseline for natural undisturbed areas. For the proposed disturbed areas, a manning's n roughness of 0.025 was adopted.	
Boundary Conditions				
Inflow boundary	N/A		No change from baseline	
Direct rainfall	Design rainfall hyetographs, with consideration of the estimated ARFs, applied as direct rainfall over the entire 2D model domain.		No change from baseline	
Outflow boundary	Automated stage-discharge curve (HQ) with stream bed slope used as a proxy for water surface slope.		No change from baseline	
Hydraulic Structures				



Model Input/Parameter	Baseline		Post Development																									
	BD1	BD2	BD1	BD2																								
Culvert	Existing culvert structures along Marble Bar Road (details obtained from Main Roads Western Australia) and the Roy Hill railway line (details obtained from high resolution aerial imagery and LiDAR data) were included in the models as 1D (ESTRY) inserts within TUFLOW. Parameters adopted across the structures are as follows:		No change from baseline for culvert structures along Marble Bar Road and Roy Hill railway line.																									
	<table border="1"> <thead> <tr> <th>Culvert Type</th> <th>Manning's n</th> <th>Inlet Loss</th> <th>Outlet Loss</th> </tr> </thead> <tbody> <tr> <td>CSP / Protruding (no headwall)</td> <td>0.024</td> <td>0.9</td> <td rowspan="3">1</td> </tr> <tr> <td>Reinforced concrete pipe (RCP) / Concrete headwall with wingwalls</td> <td rowspan="2">0.013</td> <td>0.5</td> </tr> <tr> <td>RCBC / Concrete headwall with wingwalls</td> <td>0.4</td> </tr> </tbody> </table>		Culvert Type	Manning's n	Inlet Loss	Outlet Loss	CSP / Protruding (no headwall)	0.024	0.9	1	Reinforced concrete pipe (RCP) / Concrete headwall with wingwalls	0.013	0.5	RCBC / Concrete headwall with wingwalls	0.4	<p>Culvert features at the proposed crossings were included in the models as 1D (ESTRY) inserts within TUFLOW. Details of the proposed culverts, including type/material, invert level, dimension, number and pipe length, are provided in Appendix A. Parameters adopted across the structures are as follows:</p> <table border="1"> <thead> <tr> <th>Culvert Type</th> <th>Manning's n</th> <th>Inlet Loss</th> <th>Outlet Loss</th> </tr> </thead> <tbody> <tr> <td>CSP</td> <td>0.024</td> <td>0.9</td> <td>1</td> </tr> <tr> <td>RCBC</td> <td>0.012</td> <td>0.4</td> <td>1</td> </tr> </tbody> </table>		Culvert Type	Manning's n	Inlet Loss	Outlet Loss	CSP	0.024	0.9	1	RCBC	0.012	0.4
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CSP	0.024	0.9	1																									
RCBC	0.012	0.4	1																									



Table 10: BD1 Median Pre-Burst

Duration (hours)	Rainfall Depth (mm)						
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP	0.5% AEP
0.5	0.5	0.8	1.1	1.3	1.7	2.1	2.4
1	0.6	1.1	1.4	1.7	2.3	2.7	3.1
2	0.3	0.7	0.9	1.1	2.4	3.3	3.8
3	0.0	0.7	1.1	1.6	5.7	8.8	10.1
6	0.0	1.3	2.2	3.0	20	32.7	37.5
12	0.0	0.8	1.4	1.9	7.3	11.4	13.2
18	0.0	0.2	0.3	0.4	5.6	9.5	10.9

Table 11: BD2 Median Pre-Burst

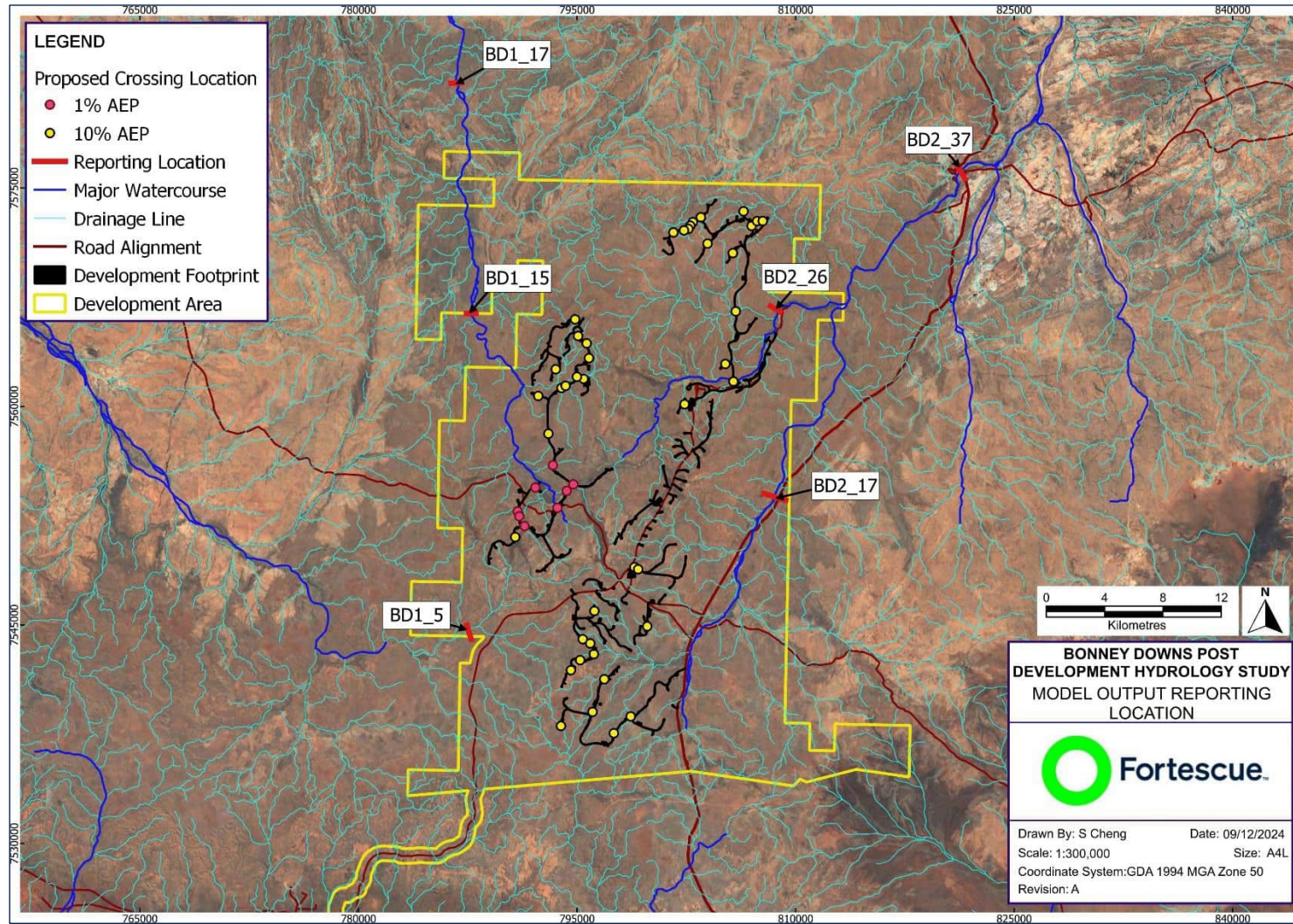
Duration (hours)	Rainfall Depth (mm)						
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP	0.5% AEP
0.5	0.8	1.1	1.3	1.5	1.8	2.0	2.3
1	1.0	1.4	1.7	1.9	2.3	2.6	3.0
2	0.5	0.8	0.9	1.1	2.1	2.8	3.2
3	0.3	1.3	2.0	2.7	5.4	7.5	8.6
6	0.0	1.4	2.3	3.2	18.3	29.7	34.4
12	0.0	0.7	1.1	1.5	8.6	13.9	16.0
18	0.0	0.3	0.4	0.6	7.0	11.8	13.6

5 MODEL RESULTS AND DISCUSSION

Baseline and post development surface water flow conditions for the 50 % to 0.5 % AEP design events were simulated and model results for the 50 %, 10 % and 1 % AEP storms are presented in this report to illustrate the low, medium and high flow situations in Bonney Downs development area. The 10 % and 1 % AEP storms represent the design standard for the waterway crossings at Bonney Downs and flood immunity requirement for critical infrastructure, including power generation area, for Fortescue projects (Section 3). Reporting locations for both BD1 and BD2 model areas were selected and shown in Figure 9.



Figure 9: Model Results Reporting Location





5.1 Baseline Conditions

Detailed discussion of the baseline surface water flow conditions, including maximum flood depth and velocity mapping for the modelled events, are presented in the Baseline Hydrology Report (AUSS0003-0000-HG-REP-0001). This section provides an overview of the major findings for the purpose of comparison against the post development conditions as detailed in Section 5.2.

5.1.1 BD1 Model Area

The estimated flood peaks, total flow volume, maximum flood depth and velocity for BD1 model area at BD1_15 (Coongan River), BD1_5 (Upper reach of Shaw River) and BD1_17 (Coongan River – model outlet), as indicated in Figure 9, are presented in Table 12.

Table 12: Baseline Model Results for BD1 Model Area

Reporting Location	Results	50 % AEP	10 % AEP	1 % AEP
BD1_15 (Coongan River)	Critical duration (hours)	18	12	12
	Peak flow (m ³ /s)	91.5	420	942
	Flow volume (ML)	1,882	7,425	18,238
	Maximum flood depth (m)	3.24	4.69	5.96
	Maximum velocity (m/s)	0.66	1.77	2.89
BD1_5 (Upper reach of Shaw River)	Critical duration (hours)	12	12	3
	Peak flow (m ³ /s)	56.1	218	505
	Flow volume (ML)	543	2,791	3,683
	Maximum flood depth (m)	1.71	2.55	3.29
	Maximum velocity (m/s)	0.83	1.35	1.77
BD1_17 (Coongan River – model outlet)	Critical duration (hours)	18	12	12
	Peak flow (m ³ /s)	176	926	2,168
	Flow volume (ML)	4,947	18,927	48,064
	Maximum flood depth (m)	3.51	5.75	7.49
	Maximum velocity (m/s)	1.67	2.97	4.00

The BD1 model area covers the western portion of the development area, which includes the headwater areas of the Coongan River to the north and headwater areas of the Shaw/Fortescue Rivers to the south. The Coongan River is the largest watercourse that drains through the area and is therefore predicted to have the highest hydraulic intensities within the BD1 model domain. Estimated flood peaks for the River at BD1_15 approach 91 m³/s, 420 m³/s and 942 m³/s for the 50 %, 10 % and 1 % AEP events, respectively. Maximum flood depths and velocities for the Coongan River were estimated to reach 3.2 m and 0.7 m/s for the 50 % AEP event and increasing to 6.0 m and 2.9 m/s for the 1 % AEP event at the same location.

Near the model outlet at BD1_17, i.e., downstream of the development area, peak flows along the Coongan River are shown to increase to 176 m³/s, 926 m³/s and 2,168 m³/s for the 50 %, 10 % and 1 % AEP events, with the maximum flood depths and velocities approaching 3.5 m to 7.5 m and 1.7 m/s to 4.0 m/s for the modelled events.



BD1_5 represents the upper reach of the Shaw River. Due to the smaller catchment size and hence runoff generating potential, this system was estimated to have lower hydraulic energies than the Coongan River with the maximum 1 % AEP flood depths and velocities typically under 3.5 m and 2 m/s.

The development area overlaps a significantly small portion of the Fortescue River catchment with limited disturbance proposed within the area. Hence surface water flows contributing to the Fortescue River are not expected to be affected by the wind power generation area. Discussion of the potential effects of the PTP Stage 8 transmission line on surface water flows in the catchment is provided in 540PT-5632-AS-HY-0001.

5.1.2 BD2 Model Area

The estimated flood peaks, total flow volume, maximum flood depth and velocity for BD2 model area at BD2_17 (Nullagine River), BD2_26 (Bonnie Creek at Bonnie Pool) and BD2_37 (Nullagine River at Nullagine gauging station – model outlet), as indicated in Figure 9, are presented in Table 13.

Table 13: Baseline Model Results for BD2 Model Area

Reporting Location	Results	50 % AEP	10 % AEP	1 % AEP
BD2_17 (Nullagine River)	Critical duration (hours)	6	6	6
	Peak flow (m ³ /s)	221	935	2,121
	Flow volume (ML)	4,447	15,588	41,462
	Maximum flood depth (m)	1.94	3.06	4.20
	Maximum velocity (m/s)	1.33	1.99	2.43
BD2_26 (Bonnie Creek at Bonnie Pool)	Critical duration (hours)	6	18	6
	Peak flow (m ³ /s)	63.6	347	939
	Flow volume (ML)	1,405	8,398	16,733
	Maximum flood depth (m)	2.69	4.20	5.93
	Maximum velocity (m/s)	0.54	1.50	2.22
BD2_37 (Nullagine River at Nullagine gauging station – model outlet)	Critical duration (hours)	18	18	12
	Peak flow (m ³ /s)	239	1,272	3,271
	Flow volume (ML)	7,994	38,967	99,488
	Maximum flood depth (m)	4.61	6.30	7.79
	Maximum velocity (m/s)	1.07	1.87	2.14

The BD2 model area covers the eastern portion of the development area, which is located in the headwaters of the Nullagine River catchment. Nullagine River is the largest watercourse that drains through the area. Other smaller creeks that flow through the site, including Bonnie Creek, are all tributaries of the Nullagine River, which itself is a tributary of the De Grey River.



Peak flows for the Nullagine River at BD2_17 were estimated to reach 221 m³/s, 935 m³/s and 2,121 m³/s for the 50 %, 10 % and 1 % AEP events, with the maximum flood depths and velocities increasing from 1.9 m and 1.3 m/s for the 50 % AEP event to 4.2 m and 2.4 m/s for the 1 % AEP event. Near the model outlet at BD2_37, i.e., downstream of the development area at the Nullagine gauging station, flow rates and volumes for the Nullagine River increase significantly due to the larger contributing area. Peak flows are shown to increase to 239 m³/s, 1,272 m³/s and 3,271 m³/s for the 50 %, 10 % and 1 % AEP events, with the maximum flood depths approaching 4.6 m to 7.8 m.

The Bonnie Creek catchment drains the north-eastern portion of the development area. Estimated flood peaks at Bonnie Pool are in the order of 68 m³/s, 355 m³/s and 951 m³/s for the 50 %, 10 % and 1 % AEP events, with the maximum flood depths ranging from 2.7 m to 5.9 m and flow velocities from 0.5 m/s to 2.2 m/s.

Due to the steep and incised nature of the landforms in the BD2 model domain, flood flows and associated floodplain are predominantly confined to the defined watercourses with limited overbank or overland flows. This results in high flood depths particularly along the major watercourses with the 50 % AEP flood level reaching and/or exceeding 2 m and the 1 % AEP flood level exceeding 4 m.

5.2 Post Development Conditions

This section provides a detailed discussion of the post development surface water flow conditions for Bonney Downs (both BD1 and BD2 model areas).

Maximum flood depth, velocity and afflux mapping for the 50 %, 10 % and 1 % AEP events are presented in Appendix B. The afflux (difference) maps present changes in maximum flood depths and velocities between baseline and post development conditions and provide a means to identify potential impacts to existing surface water flow regime as a result of the development.

The presented data was draped over the 1 m DEM data to aid in improved visualisation of key drainage features across the development area.

5.2.1 BD1 Model Area

A comparison of the estimated flood peaks, total flow volume, maximum flood depth and velocity for BD1 model area at the identified reporting locations are presented in Table 14.

Table 14: Comparison of Baseline and Post Development Model Results for BD1 Model Area

Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
BD1_15 (Coongan River)	Critical duration (hours)	18	18	-	12	12	-	12	12	-
	Peak flow (m ³ /s)	91.5	92.0	0.5 %	420	416.6	-0.7%	942	933	-0.9 %
	Flow volume (ML)	1,882	1,897	0.8 %	7,425	7,425	0.0%	18,238	18,217	-0.1 %
	Maximum flood depth (m)	3.24	3.24	0.1 %	4.69	4.68	-0.1%	5.96	5.94	-0.3%



Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
	Maximum velocity (m/s)	0.66	0.66	0.2%	1.77	1.76	-0.3%	2.89	2.88	-0.5%
BD1_5 (Upper reach of Shaw River)	Critical duration (hours)	12	12	-	12	12	-	3	3	-
	Peak flow (m ³ /s)	56.1	56.3	0.3%	218	219	0.1%	505	504	0.0%
	Flow volume (ML)	543	545	0.5%	2,791	2,794	0.1%	3,683	3,683	0.0%
	Maximum flood depth (m)	1.71	1.72	0.2%	2.55	2.56	0.0%	3.29	3.29	0.0%
	Maximum velocity (m/s)	0.83	0.83	0.2%	1.35	1.36	0.1%	1.77	1.77	-0.1%
BD1_17 (Coongan River – model outlet)	Critical duration (hours)	18	18	-	12	12	-	12	12	-
	Peak flow (m ³ /s)	176	178	1.3%	926	925	-0.1%	2,168	2,161	-0.3%
	Flow volume (ML)	4,947	4,962	0.3%	18,927	18,916	-0.1%	48,064	48,034	-0.1%
	Maximum flood depth (m)	3.51	3.53	0.4%	5.75	5.75	0.0%	7.49	7.50	0.1%
	Maximum velocity (m/s)	1.67	1.68	0.5%	2.97	2.97	0.0%	4.00	4.00	0.0%

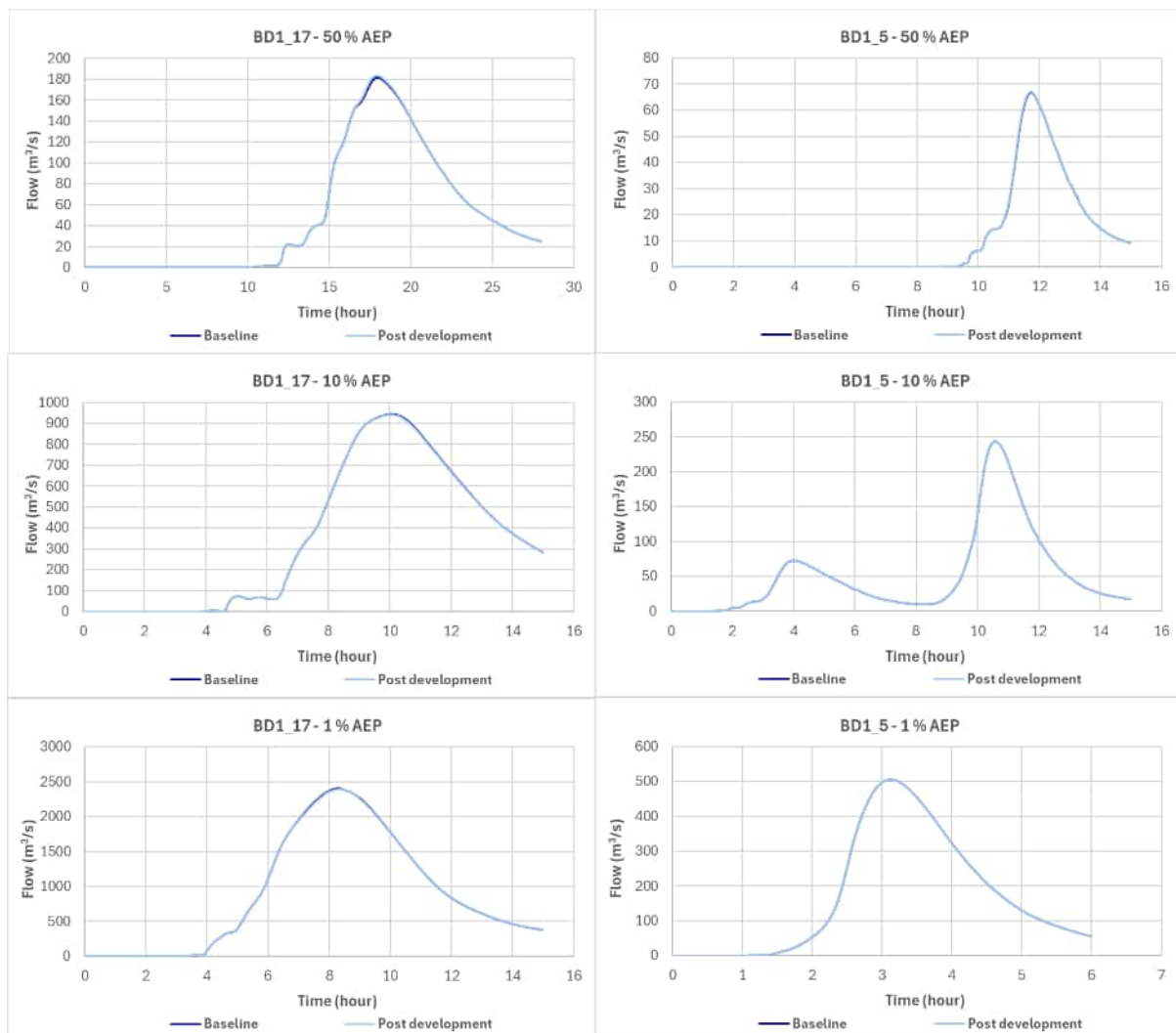
Within BD1 model area, the proposed wind turbines and associated hardstand areas/supporting infrastructure are located outside of the estimated 1 % AEP flood extent. Hence, disruption to natural surface water flows is expected to be negligible. This is reflected in the model results as shown in Table 14 where the predicted changes in flow conditions between baseline and post development conditions are less than 1 % for the modelled storms.

Proposed changes in flood peaks and runoff volume contributing to the main watercourses are small under post development conditions. This is to be expected as the project disturbance footprint within BD1 (~1.49 km²) is less than 1 % of the total model area (691 km²), and less than 0.1 % of the regional Coongan and Shaw River catchments. This subsequently resulted in minor modifications in flood depths and velocities post development (< 1 % as shown in Table 14), as illustrated in the afflux mapping where the proposed changes in maximum flood depths and velocities are generally less than 0.05 m and 0.1 m/s along the main creeks/watercourses within the model area. Exceptions are limited to areas adjacent to the waterway crossings where more noticeable changes in depths and velocities are observed. These changes in maximum flood depths did not alter the inundation (i.e., flood) extents for the modelled storms. Similarly, the minor changes in maximum velocities are not expected to alter the erosional and depositional regime of the creek systems. More detailed discussion of local impacts surrounding the waterway crossings are provided below.



A comparison of the flow hydrographs at reporting locations BD1_17 (Coongan River – model outlet) and BD1_5 (upper reach of Shaw River) is provided in Figure 10. The proposed changes in flow regime downstream of the development area are minimal, which suggests the proposed development is not expected to alter the surface water flow regime of the downstream catchments. This is supported by the afflux mapping, which shows the proposed changes in maximum flood depths and velocities are negligible for areas downstream of the development. Based on this, it can be concluded that the project design and layout are unlikely to modify the surface water flow quantity and quality of the regional Coongan and Shaw River systems.

Figure 10: Comparison of Baseline and Post Development Flow Hydrographs at BD1_17 and BD1_5





5.2.1.1 Local Impacts – Waterway Crossings

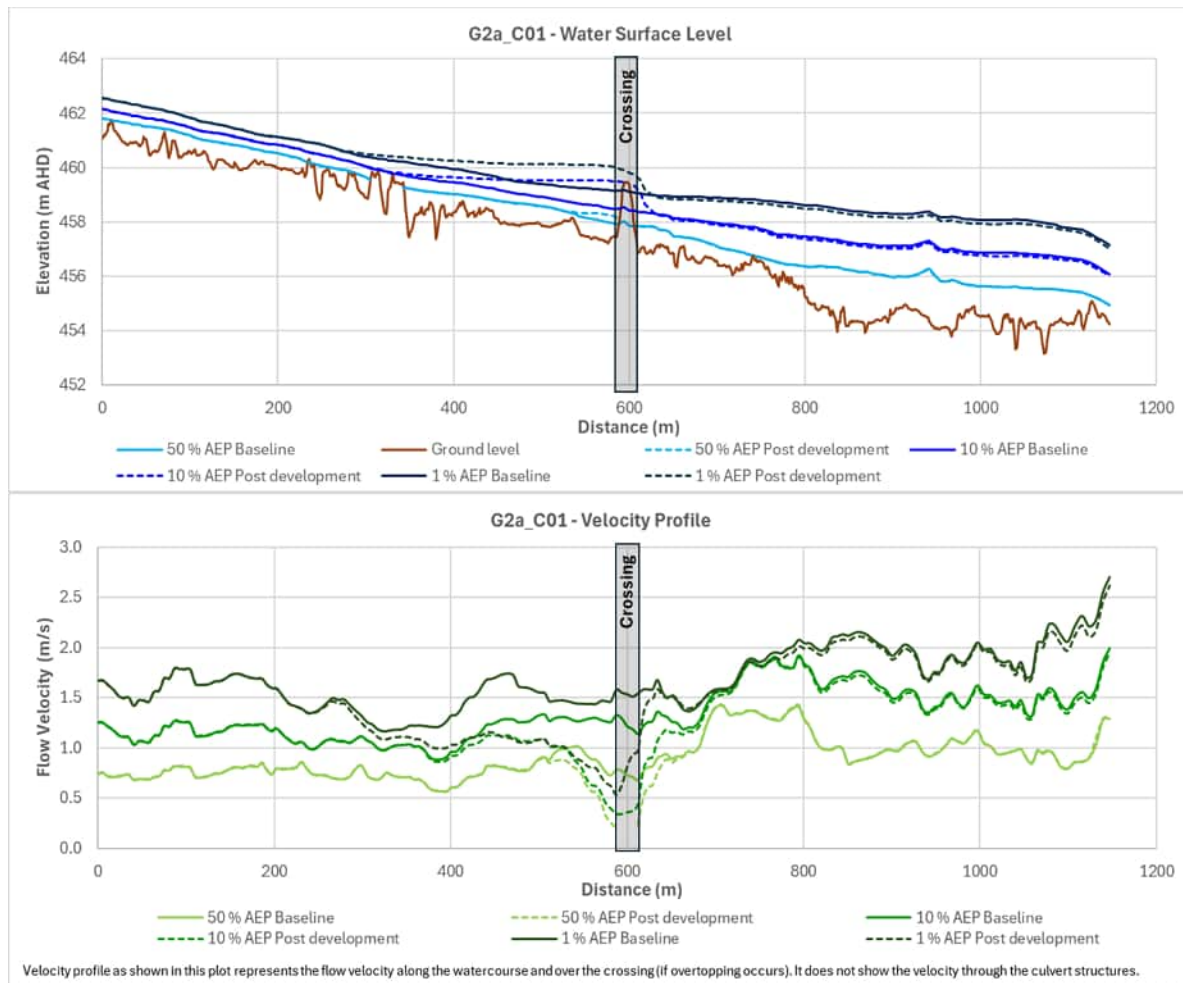
Linear infrastructure, such as turbine access track, has the potential to alter surface water flow paths by obstructing and changing flow direction, and potentially disrupting existing flow paths or creating new ones that did not exist under baseline conditions. These changes are demonstrated by the “Was Wet, Now Dry” and “Was Dry, Now Wet” layers in the afflux mapping. The access track layout has been developed in consideration of the site hydrology and waterway crossings have been designed to maintain continuous flow of major flow paths. Hence, the changes in wet/dry areas are confined to headwater areas and are localised in regions immediately upstream and downstream of the access track.

Majority of the waterway crossings in BD1 model area are minor with small contributing catchment areas of less than 5 km². There are five major waterway crossings in BD1 where the contributing catchment areas exceed 5 km². One such example is crossing G2a_C01, which adopted the 10 % AEP design standard (i.e., the road would remain trafficable for events up to the 10 % AEP). This crossing requires the installation of culverts to meet the trafficability design requirement and maintain flows. This consequently requires raising of the road above the natural creek level, which can potentially cause water level to increase upstream (i.e., backwater effects) and decrease downstream of the crossing.

A comparison of the baseline and post development water surface levels and velocity profiles at the crossing is illustrated in Figure 11. The estimated water level and velocity changes are minor for the 50 % AEP event. This suggests impacts to flows for the smaller storms are limited as the capacity of the culverts are designed to accommodate flows for the lower order events (smaller than the 10 % AEP) with minimal obstruction. For the 10 % and 1 % AEP events, flood levels are shown to increase upstream with the build-up reaching a maximum of 1 m for both events. However, as shown in the plot, the effects of backwater do not propagate more than 300 m upstream of the crossing. The estimated water level changes are minimal downstream of the structure, which suggests impacts to downstream flow regime are likely to be negligible with the implementation of the crossing. Maximum flow velocities are shown to decrease by 1 m/s for the 10 % and 1 % AEP events with the area of influence restricted to within 300 m upstream of the crossing. Modelling results show that maximum velocities would increase as water flows over the crossing and return to the baseline level approximately 100 m downstream.



Figure 11: Maximum Surface Water Level and Velocity Profile at Waterway Crossing G2a_C01



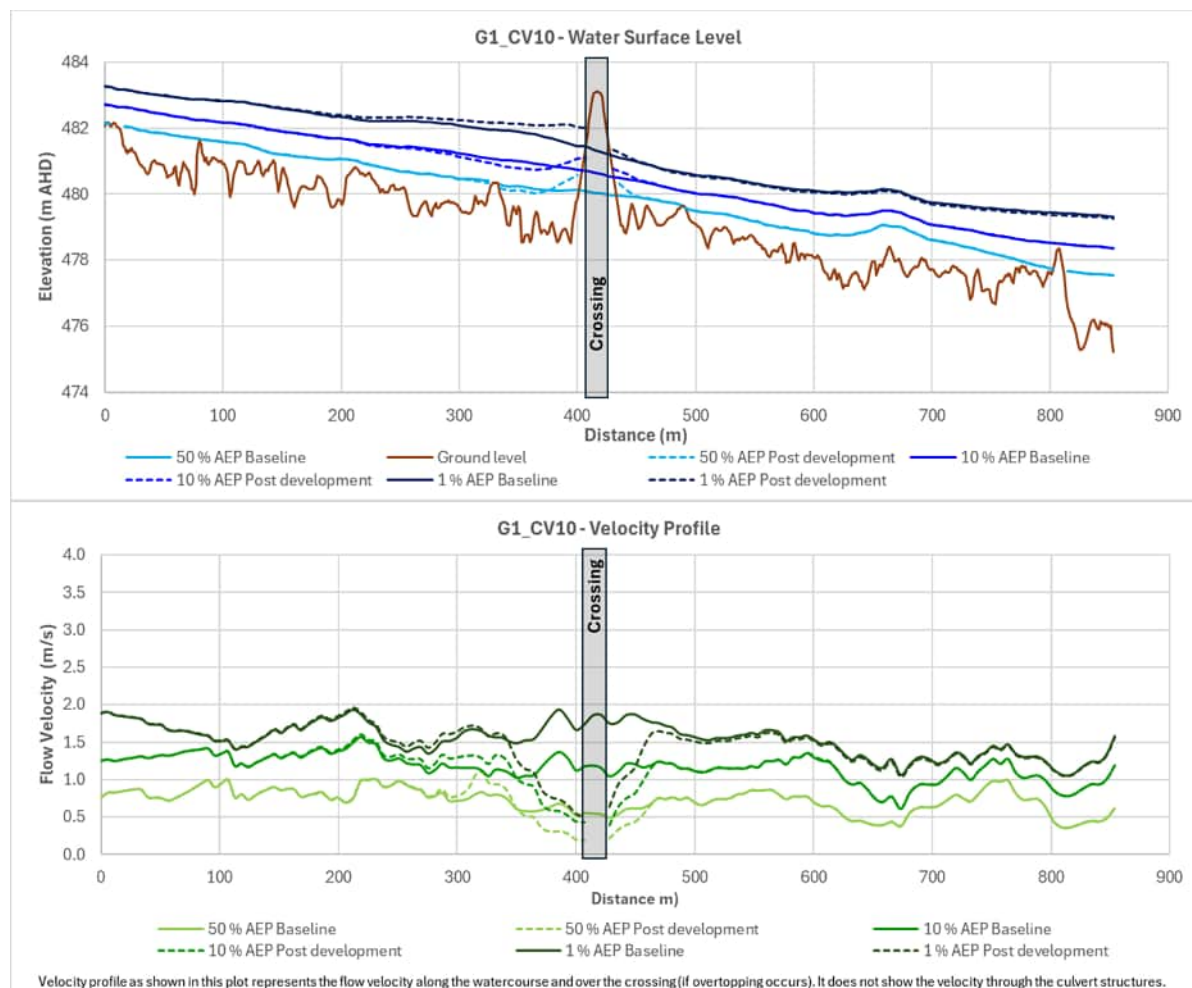
Among the five major waterway crossings in BD1, two (G1_CV08 and G1_CV10) were designed to accommodate the 1 % AEP event (i.e., the road would remain trafficable for events up to the 1 % AEP). Figure 12 presents the maximum water surface levels and velocity profiles at G1_CV10 under baseline and post development conditions.

The estimated water level changes upstream of the crossing are in the order of 0.2 m to 0.5 m from the baseline, with the backwater effects extending for approximately 100 m to 200 m upstream for the modelled events. Water level changes are minimal downstream of the structure, which suggests impacts to downstream flow regime are likely to be negligible. Estimated velocity changes are in the order of 0.5 m/s to 1.2 m/s upstream of the crossing, with the area of influence extending for approximately 150 m upgradient of the structure. As shown in Figure 12, flow velocities would return to the baseline level within 100 m downstream of the crossing.

The modelling results show that G1_CV10 (with a higher design standard) has a lower potential to cause backwater effects when compared to G2a_C01 as the capacity of the culverts can accommodate larger flows, which reduce the build-up of water and extent of ponding upstream of the road embankment. However, despite the greater impacts, the changes caused by crossing G2a_C01 are limited to areas in vicinity of the crossing and do not extend far to impact on downstream flow conditions.



Figure 12: Maximum Surface Water Level and Velocity Profile at Waterway Crossing G1_CV10



5.2.2 BD2 Model Area

A comparison of the estimated flood peaks, total flow volume, maximum flood depth and velocity for BD2 model area at the identified reporting locations are presented in Table 15.

Table 15: Comparison of Baseline and Post Development Model Results for BD2 Model Area

Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
BD2_17 (Nullagine River)	Critical duration (hours)	6	6	-	6	6	-	6	6	-
	Peak flow (m ³ /s)	221	222	0.5%	935	934	-0.1%	2,121	2,119	-0.1%
	Flow volume (ML)	4,447	4,466	0.4%	15,588	15,604	0.1%	41,462	41,438	-0.1%
	Maximum flood depth (m)	1.94	1.95	0.2%	3.06	3.06	-0.1%	4.20	4.20	0.0%
	Maximum velocity (m/s)	1.33	1.33	0.2%	1.99	1.99	-0.1%	2.43	2.43	0.0%



Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
BD2_26 (Bonnie Creek at Bonnie Pool)	Critical duration (hours)	6	6	-	18	18	-	6	6	-
	Peak flow (m ³ /s)	63.6	62.0	-2.4	347	343	-1.2%	939	928	-1.1
	Flow volume (ML)	1,405	1,391	-1.0%	8,398	8,380	-0.2%	16,733	16,660	-0.4%
	Maximum flood depth (m)	2.69	2.68	-0.4%	4.20	4.18	-0.4%	5.93	5.91	-0.4%
	Maximum velocity (m/s)	0.54	0.53	-1.5%	1.50	1.49	-0.4%	2.22	2.20	-1.2%
BD2_37 (Nullagine River at Nullagine gauging station – model outlet)	Critical duration (hours)	18	18	-	18	18	-	12	12	-
	Peak flow (m ³ /s)	239	240	0.5%	1,272	1,273	0.0%	3,271	3,268	-0.1%
	Flow volume (ML)	7,994	8,011	0.2%	38,967	38,983	0.0%	99,488	99,448	0.0%
	Maximum flood depth (m)	4.61	4.62	0.0%	6.30	6.30	0.0%	7.79	7.78	0.0%
	Maximum velocity (m/s)	1.07	1.07	0.1%	1.87	1.87	0.0%	2.14	2.14	0.0%

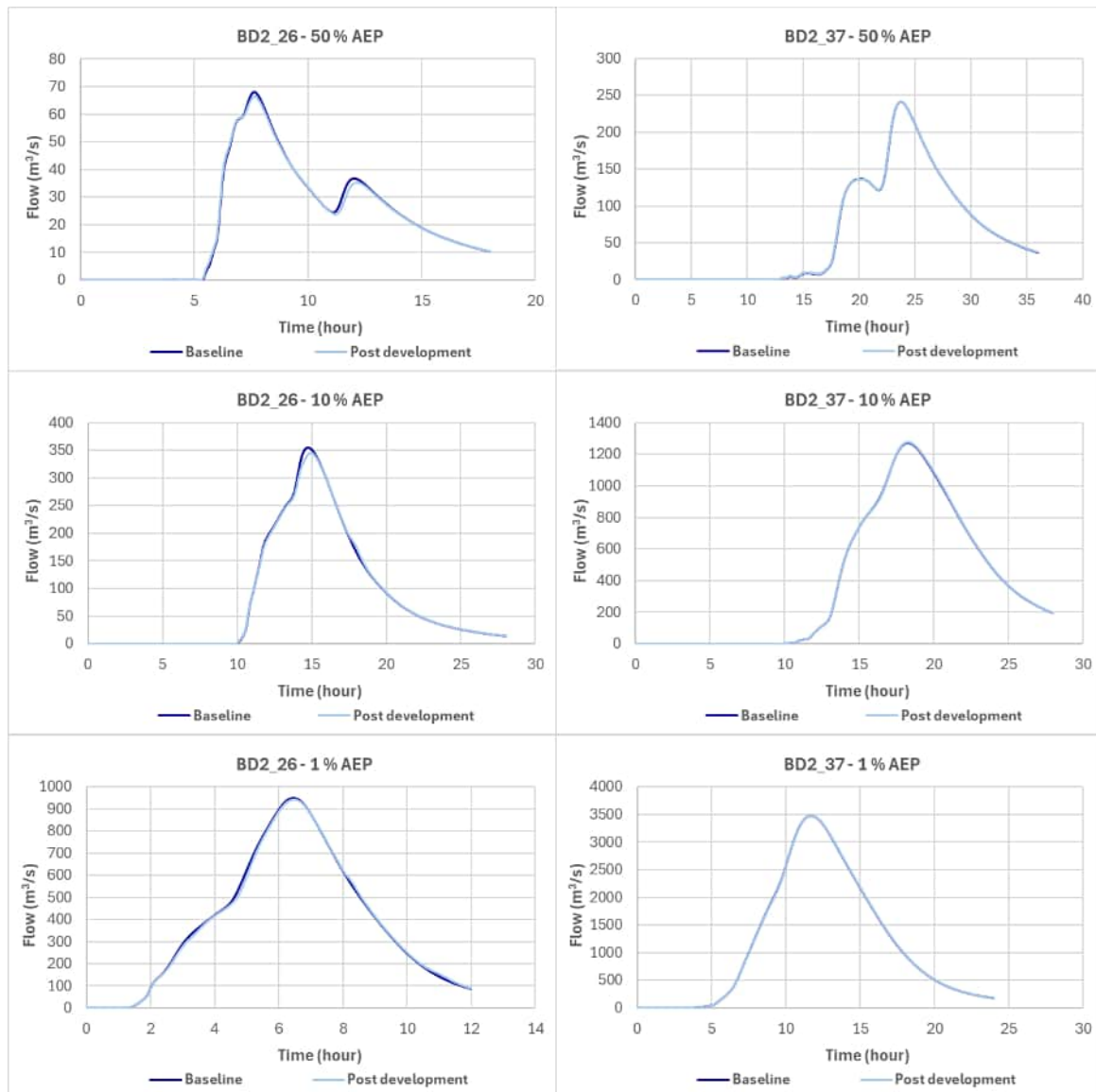
Similar to BD1, the proposed wind turbines and hardstand areas within BD2 model area were designed to avoid major watercourses and associated 1 % AEP floodplain area. Hence, significant disruption to surface water flows is not expected.

In general, the modelling results show that the impacts to surface water flows are low for all the modelled storms. Estimated changes in flood peaks and runoff volume contributing to the main watercourses, including the Nullagine River and Bonnie Creek, are small under post development conditions. As shown in Table 15, the overall effects on the total flow volume contributing to Bonnie Creek and subsequently to Bonnie Pool at BD2_26 are minimal (within 1 % change). This translated to a small/immaterial change in maximum flood depths and velocities (within ± 0.02 m and ± 0.02 m/s) at the Pool. Based on this, it can be concluded that significant modifications to the existing surface water flow regime along Bonnie Creek and Bonnie Pool are not expected under post development conditions (Figure 13).

Similarly for the Nullagine River, the predicted changes in flow peaks/volume are minor (< 1 %) due to the small size of the project footprint (~4.63 km² within BD2) compared to the catchment of the River (873 km² at the Nullagine gauging station). As shown in the afflux mapping, the proposed changes in maximum flood depths and velocities under post development conditions are minimal, generally not exceeding 5 cm and 0.1 m/s along the River and main creeks/watercourses within the development area. The only exceptions are at the waterway crossings and areas immediately upstream/downstream of the access track where localised modifications in depths and velocities are observed. Downstream of the development at BD2_37, the proposed flow regime changes are minimal (Figure 13 and Table 15), which indicates the downstream surface water flow conditions are unlikely to be impacted by the development of the wind farm.



Figure 13: Comparison of Baseline and Post Development Flow Hydrographs at BD2_37 and BD2_26



5.2.2.1 Local Impacts – Waterway Crossings

Similar to BD1, majority of the waterway crossings in BD2 model area are minor with small contributing catchment areas of less than 5 km². These crossings either adopt the low-level floodway design where the road elevation is set at or very close to the natural creek level or design with relief culverts to maintain continuous flow. Modelling results show that impacts of these crossings are small and localised (area of influence generally confined to within 200 m upstream and downstream of the structures). Hence, significant modifications to major flow paths downstream of the development are not expected.

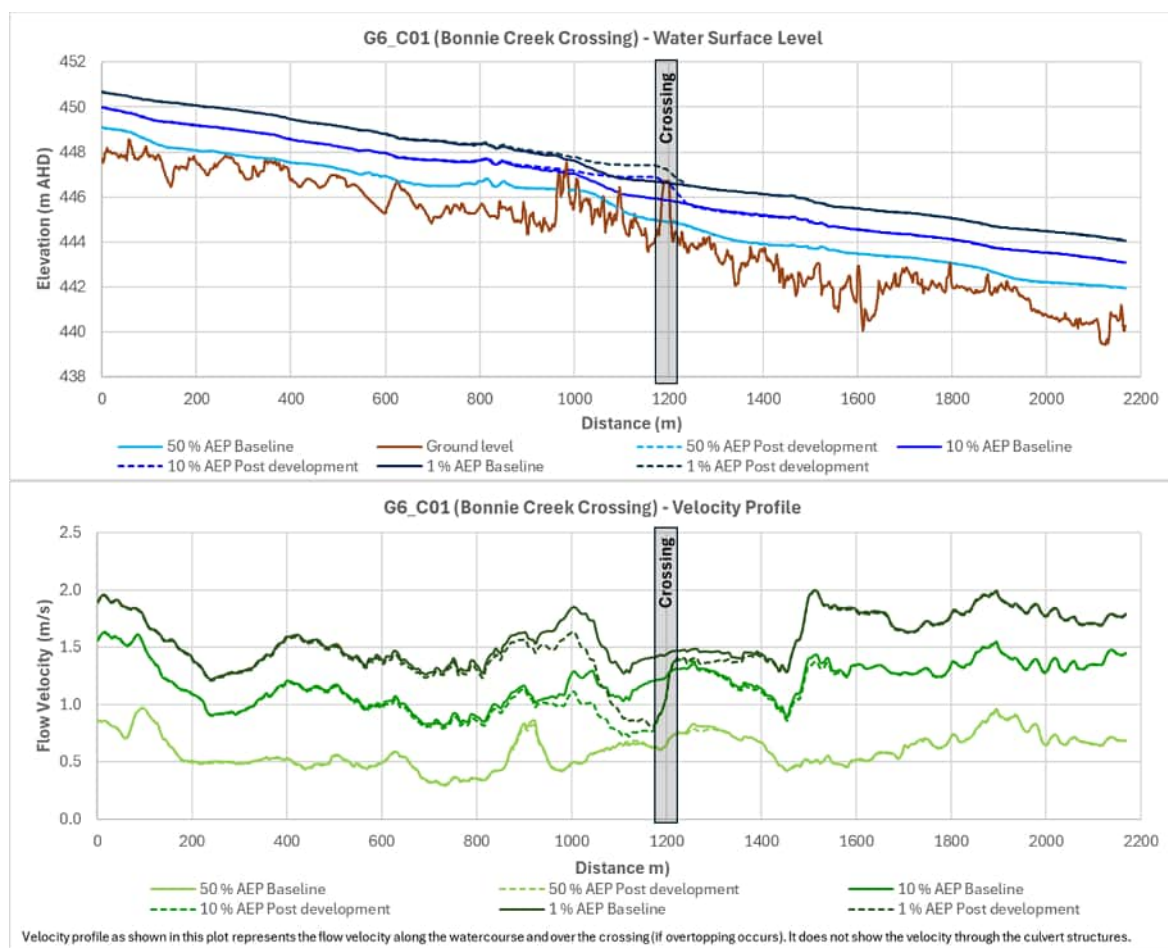
There are two major waterway crossings (contributing catchment area > 5 km²) in BD2, and the most significant is the Bonnie Creek crossing at G6_C01 (the proposed crossing is located approximately 7 km upstream of Bonnie Pool). Figure 14 provides a comparison of the maximum water surface levels and velocity profiles at the crossing under baseline and post development conditions. Flows for the 50 % AEP storm are generally unaffected by the crossing indicating minimal obstruction to surface water flows for the frequent events (events



smaller than the 10% AEP, which is the design standard of the crossing). For the 10 % and 1 % AEP event, flood levels are shown to increase by up to 1 m upstream, with the backwater effects extending for approximately 400 m upgradient of the structure. Water level changes are minimal downstream, indicating impacts to downstream flow regime are likely to be negligible. Maximum flow velocities are shown to decrease by up to 0.5 m/s upstream for both the 10 % and 1 % AEP events, with the effects extending no more than 400 m upstream of the structure. As shown in Figure 14, flow velocities would increase as water flows over the road and continue to increase until returning to the baseline level within 300 m downstream of the structure.

In summary the modelling results show that the effects of the Bonnie Creek crossing are predominately local and do not extend beyond the proximity of the structure to impact on the downstream hydrology/morphology of Bonnie Creek and Bonnie Pool.

Figure 14: Maximum Surface Water Level and Velocity Profile at Waterway Crossing G6_C01 (Bonnie Creek Crossing)



6 DISCUSSION OF POTENTIAL IMPACTS

Turbine hardstand areas, access tracks and waterway crossings have the potential to cause local level impact on creek hydrology and morphology. By comparing modelling results of baseline and post development scenarios, areas affected by the project have been identified to assess potential impacts.



6.1 Regional Hydrologic Regime

Model results indicate that areas with noticeable flow regime changes are confined to isolated areas along the turbine access track/waterway crossings, while disruptions to regional surface water flows are negligible. In summary, the assessment shows that:

- Changes in total flow volume near the model outlets (for both BD1 and BD2 model areas) are less than 1 %, indicating the quantity of flow contributing to the regional Coongan, Shaw and Nullagine River systems is unlikely to be affected by the development.
- Estimated changes in runoff volume contributing to Bonnie Creek and Bonnie Pool are within 1 %, hence significant modifications to the quantity of flow contributing to these surface water features are not expected post development.
- Changes in maximum flood depths along the main watercourses (including Coongan River, Nullagine River, Bonnie Creek and upper reach of the Shaw River) are marginal (within ± 0.02 m) and the flood extents for the modelled storms remain unaffected by the development.

Turbine access tracks and waterway crossings have the potential to cause local level impact on creek hydrology and morphology. Majority of the waterway crossings within the Bonney Downs development area are minor with small contributing catchment areas of less than 5 km². Modelling results show that impacts of these crossings are small, with the area of influence generally confined to within 200 m upstream and downstream of the structures.

Major waterway crossings within the development area have the potential to cause greater flow regime changes, due to flow constrictions and obstructions caused by the culvert structures and road embankment (generally higher to accommodate the larger culverts). Greater increase in water level and backwater development are expected when compared to the minor crossings, but the effects are unlikely to extend more than 400 m upstream of the structures (e.g. See Figure 14). Modifications to flow velocities are also expected to be larger, but these changes are localised and unlikely to extend beyond the proximity of the crossings to impact on regional hydrologic regime.

Modelling results show that the effects of the Bonnie Creek crossing on low flow conditions, i.e., flows for events smaller than the 10 % AEP, are negligible, but greater flow regime change is predicted for the larger events. However, the effects are predominately local and unlikely to extend more than 300 m downstream to impact on the regional flow conditions of Bonnie Creek and the sensitive receptor Bonnie Pool, which is located approximately 7 km downstream of the crossing.

Based on the findings of the assessment, the impacts of the project design and layout on the regional hydrologic regime are considered to be very low.

6.2 Geomorphology

Changes in maximum flow velocities along the main watercourses (including Coongan River, Nullagine River, Bonnie Creek and upper reach of the Shaw River) are marginal (within ± 0.02 m/s) are not expected to alter the existing geomorphic regime of the main watercourses within and downstream of the development area.

Hydraulic loadings along the creeks/watercourses within the development area are episodic, occurring during larger events, and will vary longitudinally and laterally with the creek



channel, reflecting variations in hydraulic conditions. Areas with high erosional potential are areas characterised by high velocity, while deposition will occur in areas when velocity drops. Permissible velocity, which is the velocity at which particles of a specific mean size begin to be mobilised, can be used as an indication of when channel erosion would occur. The categories of permissible velocity, established based on Subramanya (2009), is provided in Table 16.

Modifications to flow velocities as a result of flow constrictions/obstructions caused by the waterway crossings may affect the existing geomorphological regime by increasing the erosion potential of creek sections where velocity increases or encouraging sediment deposition where ponding occurs and velocity drops. Based on the permissible velocity categories as shown in Table 16, if velocity changes are within 0.5 m/s, there are unlikely to be significant changes to the mobilisation of sediments specific to that velocity range. However, if changes are more than ± 0.5 m/s, there is the potential for an increase or decrease in sediment mobilisation, which signifies a more noticeable change in geomorphological characteristics of the watercourse.

Based on the modelling results, velocity changes caused by the minor crossings are generally within ± 0.5 m/s and therefore unlikely to significantly alter the existing sediment mobilisation regime and geomorphological characteristics of the watercourses. Velocity changes associated with the major crossings are more noticeable, particularly on the upstream side of the structures where velocities could decrease by up to 1.5 m/s due to backwater development. This may encourage sediment deposition upstream of the structures. Velocity increases at the culvert outlets are in the order of + 1 m/s, suggesting local erosion/scour may occur along the creek bed and banks immediately downstream of the crossings. To mitigate this risk, erosion control measures at the inlet and outlet of the culvert structures are recommended.

Table 16: Categories of Permissible Velocity and Thresholds for Sediment Mobilisation (Subramanya, 2009)

Velocity (m/s)	Category
0.0-0.5	OK for most areas
0.5-1.0	Threshold for silts, fine sands and fine gravels
1.0-1.5	Threshold for clays and gravels up to 25 mm
1.5-2.0	Threshold for gravels up to 50 mm
2.0-2.5	Threshold for gravels up to 150 mm
2.5 -3.5	Thresholds for riprap with d50 = 150-225 mm
3.5-4.0	Thresholds for gravels and riprap with d50 = 300 mm
4.0-4.5	Thresholds for gravels and riprap with d50 = 450 mm
4.5-5.0	Thresholds for gravels and riprap with d50 = 450 mm
5.0-5.5	Thresholds for riprap with d50 = 600 mm, gabions and concrete
5.5-6.0	Thresholds for riprap with d50 = 600 mm, gabions and concrete
>6.0	High velocity for most areas



7 CONCLUSIONS

A post development hydrological assessment was carried out to evaluate the potential impacts of the Bonney Downs project on existing surface water flow regime. The assessment was built upon the baseline assessment works undertaken in 2023 and utilised the existing TUFLOW hydraulic models by incorporating the project design and layout to estimate the surface water flow characteristics under post development conditions.

It can be concluded from the assessment that:

- Changes in surface water flow regime as a result of the proposed development are localised and limited to areas in proximity to the turbine access track/waterway crossings. Disruptions to regional surface water flows are considered negligible for all the modelled events.
- Changes in total flow volume contributing to the main watercourses (including Coongan River, Shaw River, Nullagine River and Bonnie Creek) are marginal (< 1 %) due to the small size of the development footprint compared to the contributing catchments of these watercourses.
- Proposed changes in maximum flood depths and velocities along the main watercourses are minimal (within ± 0.02 m and ± 0.02 m/s) indicating the development is unlikely to modify the hydrology and morphology of these watercourses both within and downstream of the development area.
- Impacts of the minor crossings on creek hydrology and morphology are low. Major waterway crossings have the potential to cause greater changes to existing flow regime, but these changes are localised and unlikely to propagate beyond the proximity of the structures.
- Changes to flow velocities in vicinity of the crossings may influence the existing geomorphological conditions of the creek systems. Higher rate of sediment deposition may take place upstream of the road embankment due to backwater development. Local erosion/scour may occur at the outlet of the culvert structures due to increases in flow velocities. Erosion control measures are therefore recommended to mitigate this risk.
- The effects of the Bonnie Creek crossing are not expected to propagate beyond the proximity of the structure to impact on Bonnie Pool, a sensitive receptor located approximately 7 km downstream of the crossing.
- Overall, the impacts of the project design and layout on the regional surface water flow regime are considered to be very low.

8 RECOMMENDATIONS

Considering the technical assessment and conclusions of this study, the following mitigation strategy is recommended to support project planning and design to further minimise impacts to natural surface water flow regime:

- Surface water management plan to be developed and implemented to manage flood risk and minimise soil erosion and the potential for the transport of sediment to downstream waters during the construction phase.



- Construction and maintenance of the waterway crossings to be scheduled outside of the wet season where possible, or for a time period when rainfall and runoff are unlikely.
- Install necessary erosion control measures at the inlet and outlet of the culvert structures to minimise the risk of bed and bank erosion and local scour, and to prevent undermining of the structures.
- Armouring the floodway batters and driving surface to minimise erosion and scour as water flows over the road.
- Identify monitoring requirements and undertake baseline monitoring of Bonnie Pool if/as required.

9 REFERENCES

This report and all internal supporting documents will be managed as per Fortescue Document Governance Standards. These may be read in conjunction with this report.

- [1] Babister, M., Trim, A., Testoni, I., & Retallick, M. (2016). The Australian Rainfall and Runoff Datahub. Retrieved from <https://data.arr-software.org/>.
- [2] Ball, J., Babister, M., Nathan, R., Weeks, W., Weinmann, E., Retallick, M., & Testoni, I. (2016). Australian Rainfall and Runoff: A Guide to Flood Estimation. Commonwealth of Australia (Geoscience Australia).
- [3] BoM. (2016). Design Rainfall Data System. Retrieved from <http://www.bom.gov.au/water/designRainfalls/revised-ifd/>.
- [4] Straten. (2013). Nullagine Iron Ore Extension Project Section 38 Referral Supporting Document.
- [5] Subramanya, K. (2009). Flow in Open Channels. New Delhi: Tata McGraw-Hill.



DOCUMENT CONTROL

Bonney Downs - Post Development Hydrology Study		
Status	IFR - Issued for Review	20-Dec-24
Summary of Changes		
Author	Sheena Cheng	_____ Signature
Checked or Squad Review# (if applicable)	Squad Check	_____ Signature
Approved	Whelan Naidoo	_____ Signature
Next Review Date (if applicable)	Enter a date	



APPENDIX A FINDINGS OF CULVERTS AND FLOODWAY ASSESSMENT



Table A1: Findings of Culverts / Floodway Assessment for Waterway Crossings in Bonney Downs

Parameter	G1_CV06	G1_CV07	G1_CV08	G1_CV09	G1_CV10	G1_CV11	G1_CV12	G1_CV13	G2B_C01
Lat	-22.09	-22.08	-22.09	-22.09	-22.10	-22.11	-22.11	-22.12	-22.04
Long	119.86	119.84	119.85	119.83	119.85	119.82	119.82	119.82	119.93
Road design level (m AHD)	487.44	487.44	481.48	472.61	482.69	491.45	492.10	494.85	470.19
Design case	1 % AEP								10 % AEP
Catchment area (km ²)	0.52	3.13	5.52	0.90	11.2	3.89	1.01	5.00	2.69
Peak flow (m ³ /s)	15.9	47.9	75.8	22.3	115.0	56.5	22.5	70.0	10.1
Crossing type	Culvert crossing								Floodway with culverts
Inlet elevation (m AHD)	485.94	474.14	479.52	471.23	478.65	488.47	489.62	492.11	467.87
Outlet elevation (m AHD)	485.77	474.08	479.26	471.14	478.42	488.32	489.48	491.97	467.79
Culvert type	RCBC	CSP	RCBC			CSP			CSP
Culvert size (mm)	900 H x 1200 W	1500	1800 H x 2100 W	900 H x 1200 W	2100 H x 2100 W	1500	1350	1800	1050
No. of barrels	6	9	10	9	9	12	7	10	3
Culvert length (m)	34.4	22.7	26.7	18.8	38.3	46.3	28.3	28.2	30.2



Parameter	G2a_C01	G2a_C02	G2a_C03	G2a_C04 ⁽¹⁾	G2a_C05	G2a_C06	G2a_C07	G2a_C08	G2a_C09	G2a_C10	G2a_C11	
Lat	-22.03	-22.03	-22.02	-22.02	-22.01	-22.01	-22.00	-21.99	-22.02	-22.03	-22.06	
Long	119.85	119.85	119.86	119.86	119.87	119.86	119.86	119.86	119.84	119.83	119.84	
Road design level (m AHD)	459.46	460.50	469.50	465.91	490.47	487.01	486.67	489.39	464.79	459.23	480.30	
Design case	10 % AEP											
Catchment area (km ²)	7.62	7.91	0.21	9.22	0.05	0.88	1.90	0.32	0.60	0.13	0.67	
Peak flow (m ³ /s)	39.7	41.6	1.84	51.3	1.19	4.63	6.02	3.69	4.55	2.13	8.38	
Crossing type	Floodway with culverts											
Inlet elevation (m AHD)	457.227	458.11	468.17	463.42	464.58	488.91	485.54	485.07	487.87	463.27	458.04	478.66
Outlet elevation (m AHD)	456.824	458.04	467.93	463.18	464.28	488.49	485.79	484.88	487.82	462.87	457.80	478.53
Culvert type	CSP											
Culvert size (mm)	1650	1800	900	1800	1200	900	900	1050	900	900	600	1050
No. of barrels	6	6	1	6	3	1	3	3	3	3	3	4
Culvert length (m)	25.6	29.0	23.4	27.3	22.8	21.3	21.0	20.4	18.7	20.0	17.0	21.0



Parameter	G5_C01 ⁽¹⁾	G5_C02	G5_C04	G5_C05	G5_C06	G5_C07	G5_C09	G5_C10	G5_C11	G5_C12	G5_C13	
Lat	-22.12	-22.17	-22.19	-22.24	-22.18	-22.23	-22.14	-22.14	-22.18	-22.19	-22.18	
Long	119.82	119.87	119.87	119.85	119.91	119.90	119.90	119.90	119.87	119.87	119.87	
Road design level (m AHD)	505.26	507.27	499.35	522.39	491.14	497.58	501.56	503.62	508.32	503.28	512.98	
Design case	10 % AEP											
Catchment area (km ²)	0.85	1.60	1.79	3.30	1.15	2.03	0.39	0.16	0.21	0.19	0.14	
Peak flow (m ³ /s)	5.28	7.66	9.41	9.93	6.57	8.94	3.45		1.85	2.15		
Crossing type	Floodway with culverts								Low-level floodway	Floodway with culverts		
Inlet elevation (m AHD)	503.69	503.63	505.06	497.66	520.60	489.42	495.88	500.10	501.88	-	501.80	511.33
Outlet elevation (m AHD)	503.61	503.48	504.99	497.47	520.44	489.19	495.83	499.87	504.45	-	501.61	510.68
Culvert type	CSP								-	CSP		
Culvert size (mm)	900	900	1050	1050	1200	1050	1050	900	900	-	900	900
No. of barrels	1	2	3	4	3	3	4	2	1	-	2	1
Culvert length (m)	19.1	26.0	23.7	19.7	22.2	22.9	20.5	16.6	20.0	-	19.3	25.6



Parameter	G5_C14	G5_C15	G5_C16	G5_C17	G6_C01 ⁽¹⁾	G6_C02	G6_C03	G6_C04	G6_C05	G6_C06	
Lat	-22.20	-22.21	-22.24	-22.23	-22.02	-22.01	-21.98	-21.94	-21.93	-21.92	
Long	119.86	119.88	119.89	119.87	119.96	119.96	119.96	119.96	119.97	119.98	
Road design level (m AHD)	525.54	502.46	517.38	520.10	446.77	458.61	443.16	471.01	455.34	454.24	
Design case	10 % AEP										
Catchment area (km ²)	0.05	0.04	0.22	0.06	86.4	0.74	58.2	0.05	2.76	0.16	
Peak flow (m ³ /s)	0.61	0.74	2.57	1.62	198.7	4.97	167.5	1.32	12.4	1.91	
Crossing type	Floodway with culverts	Low-level floodway	Floodway with culverts								
Inlet elevation (m AHD)	524.44	-	515.83	518.51	443.66	444.22	456.98	439.57	469.86	453.29	452.73
Outlet elevation (m AHD)	524.19	-	515.55	518.06	443.59	444.15	456.90	439.37	469.47	453.23	452.42
Culvert type	CSP	-	CSP								
Culvert size (mm)	600	-	900	900	2400	1800	900	3000	900	1500	900
No. of barrels	1	-	2	1	10	6	3	8	1	3	2
Culvert length (m)	23.2	-	20.5	18.6	31.1	28.9	23.9	34.5	26.3	23.4	32.7



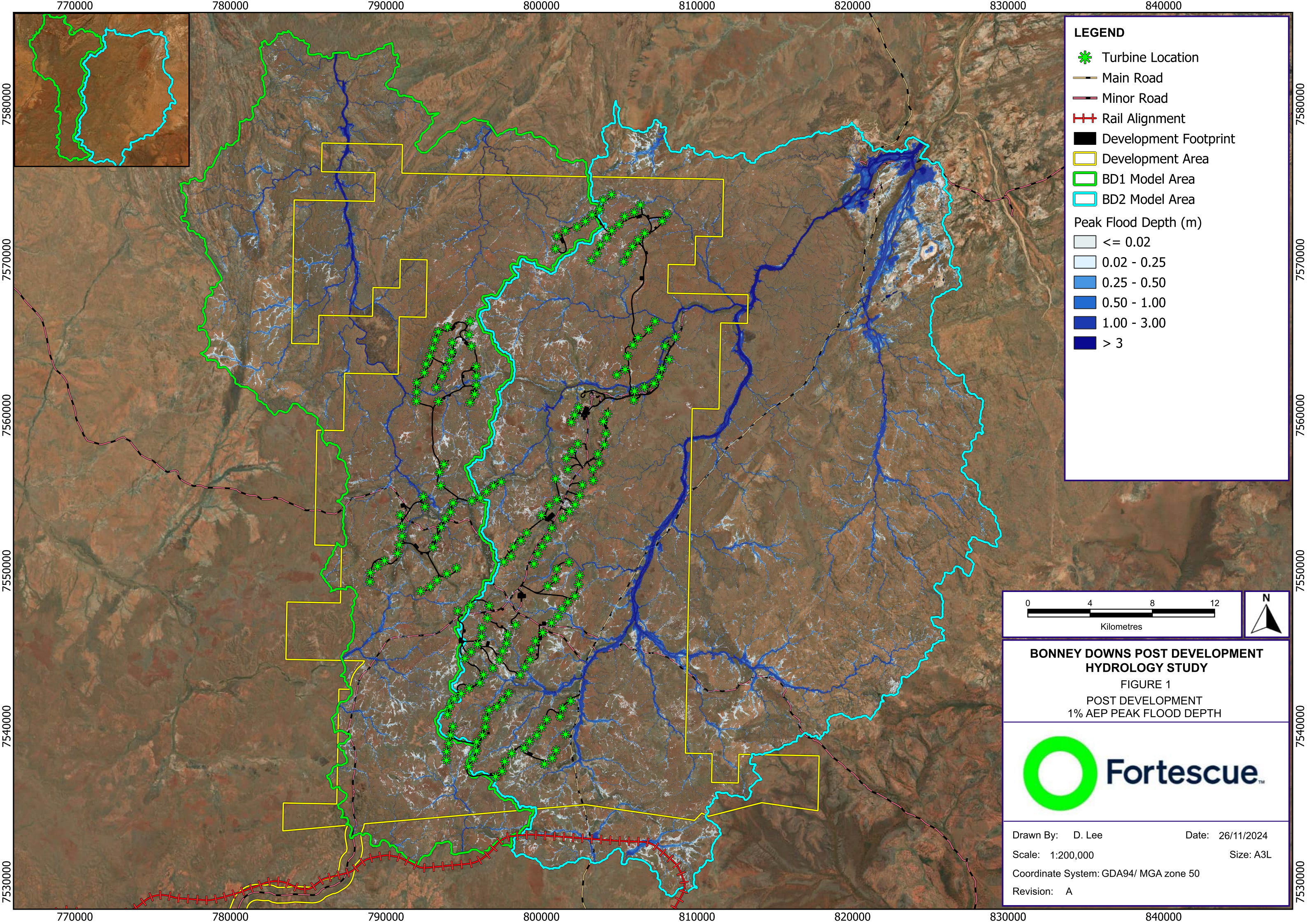
Parameter	G6_C07	G6_C08	G6_C09	G6_C10	G6_C11	G6_C12	G6_C13	G6_C14	G6_C15
Lat	-21.92	-21.92	-21.94	-21.92	-21.92	-21.93	-21.93	-21.93	-21.93
Long	119.98	119.97	119.94	119.94	119.93	119.93	119.93	119.93	119.92
Road design level (m AHD)	451.22	456.06	481.49	478.47	469.60	473.15	473.46	471.98	466.28
Design case	10 % AEP								
Catchment area (km ²)	0.11	1.92	0.83	0.05	0.84	0.02	0.21	0.04	1.98
Peak flow (m ³ /s)	1.70	14.6	4.24	0.77	5.18	0.60	2.37	0.94	12.8
Crossing type	Floodway with culverts								
Inlet elevation (m AHD)	449.63	454.13	480.02	477.37	467.98	471.94	472.01	470.78	464.43
Outlet elevation (m AHD)	449.33	453.87	479.84	476.94	467.86	471.51	471.70	470.46	464.33
Culvert type	CSP								
Culvert size (mm)	900	1500	900	600	900	600	900	600	1200
No. of barrels	1	3	3	2	3	1	2	2	4
Culvert length (m)	20.4	24.6	19.4	28.6	19.5	28.5	20.5	20.9	23.7

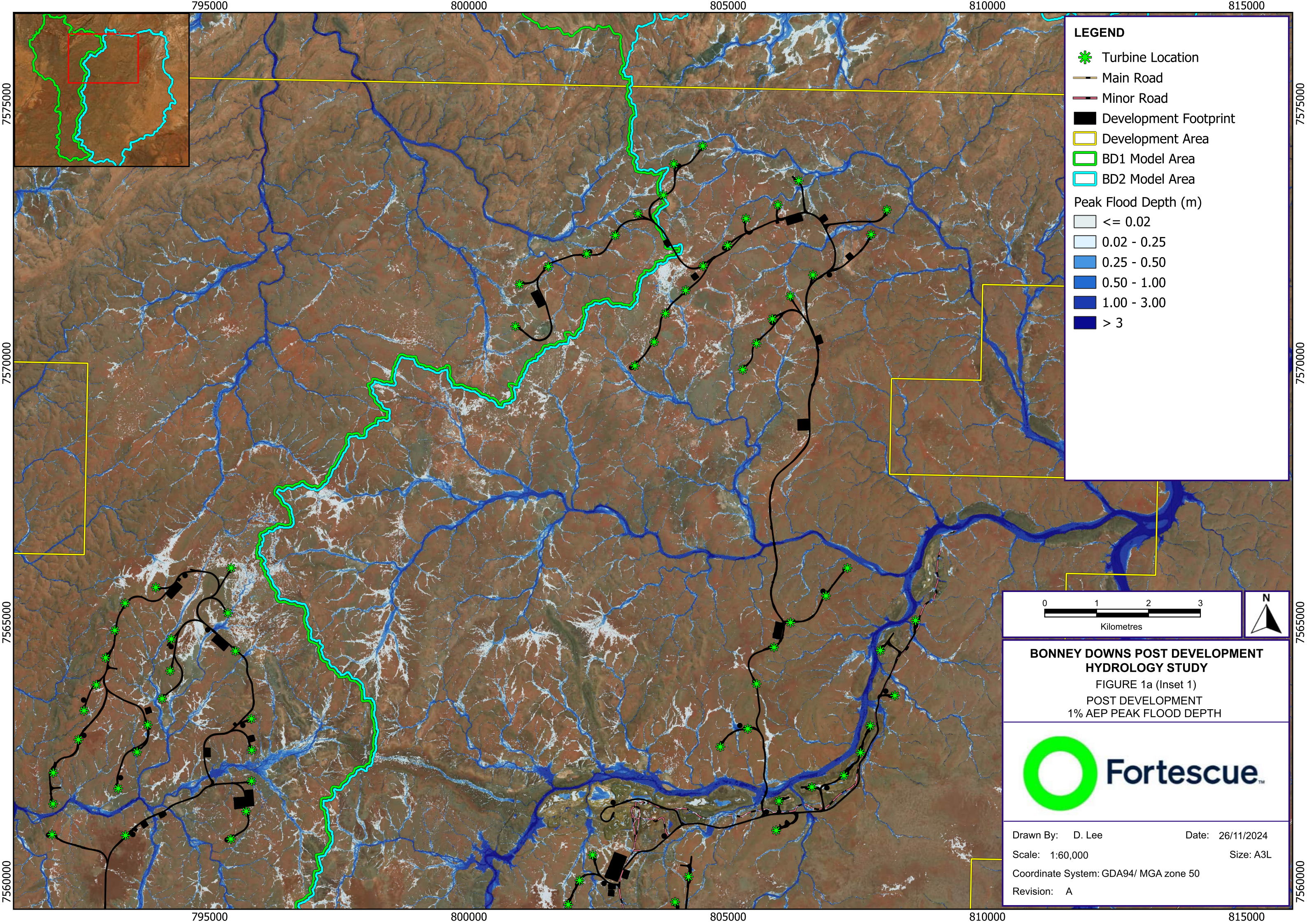
Notes:

- (1) Crossing is located along a braided section of a watercourse. Hence two sets of culverts are required to manage multiple flow paths.



APPENDIX B FLOOD MAPPING



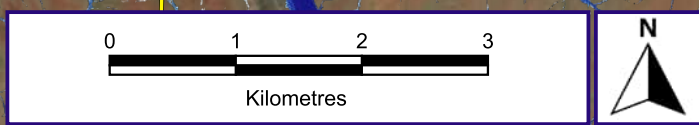


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

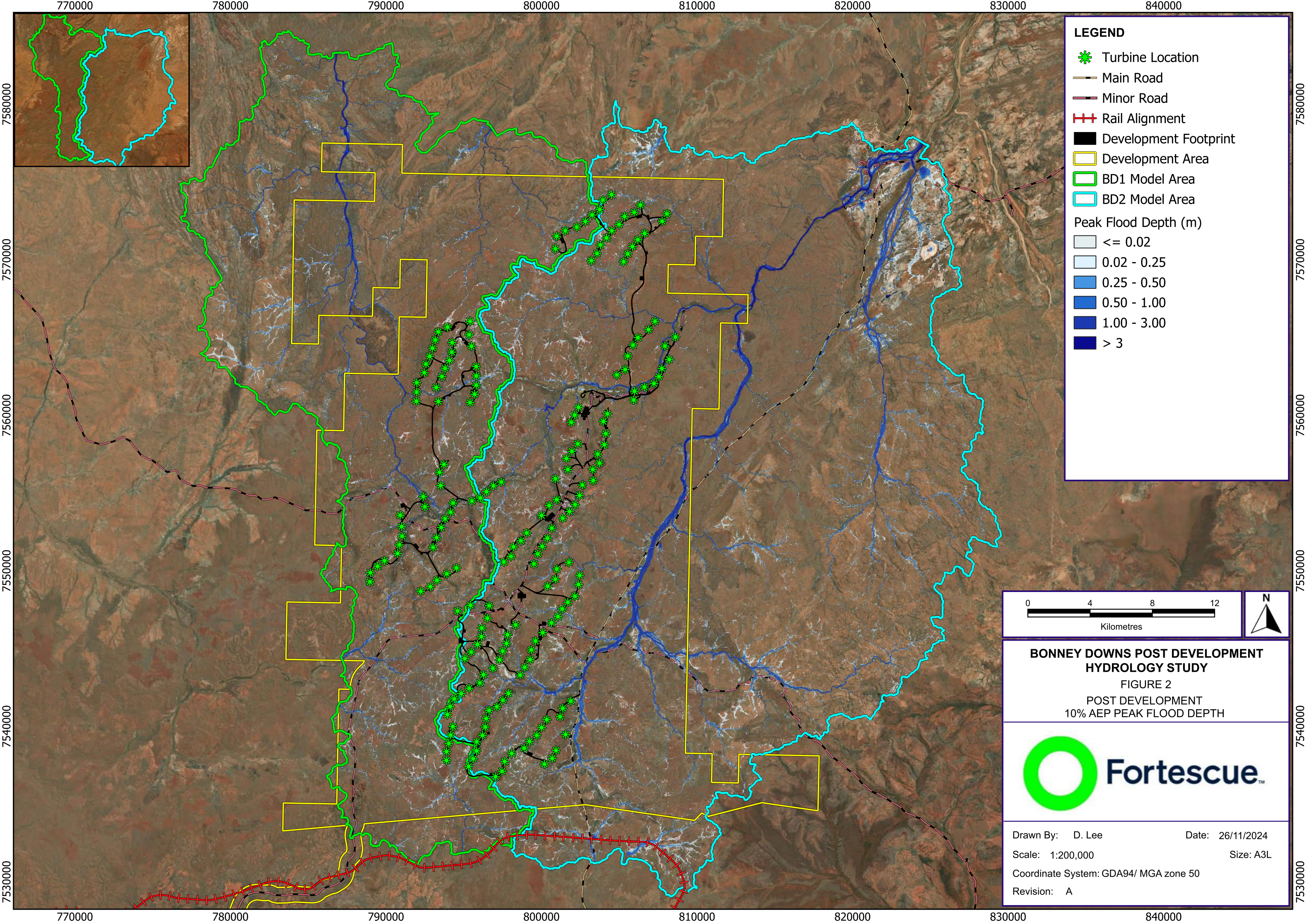
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 1a (Inset 1)
POST DEVELOPMENT
1% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

- Turbine Location
- Main Road
- Minor Road
- Rail Alignment
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

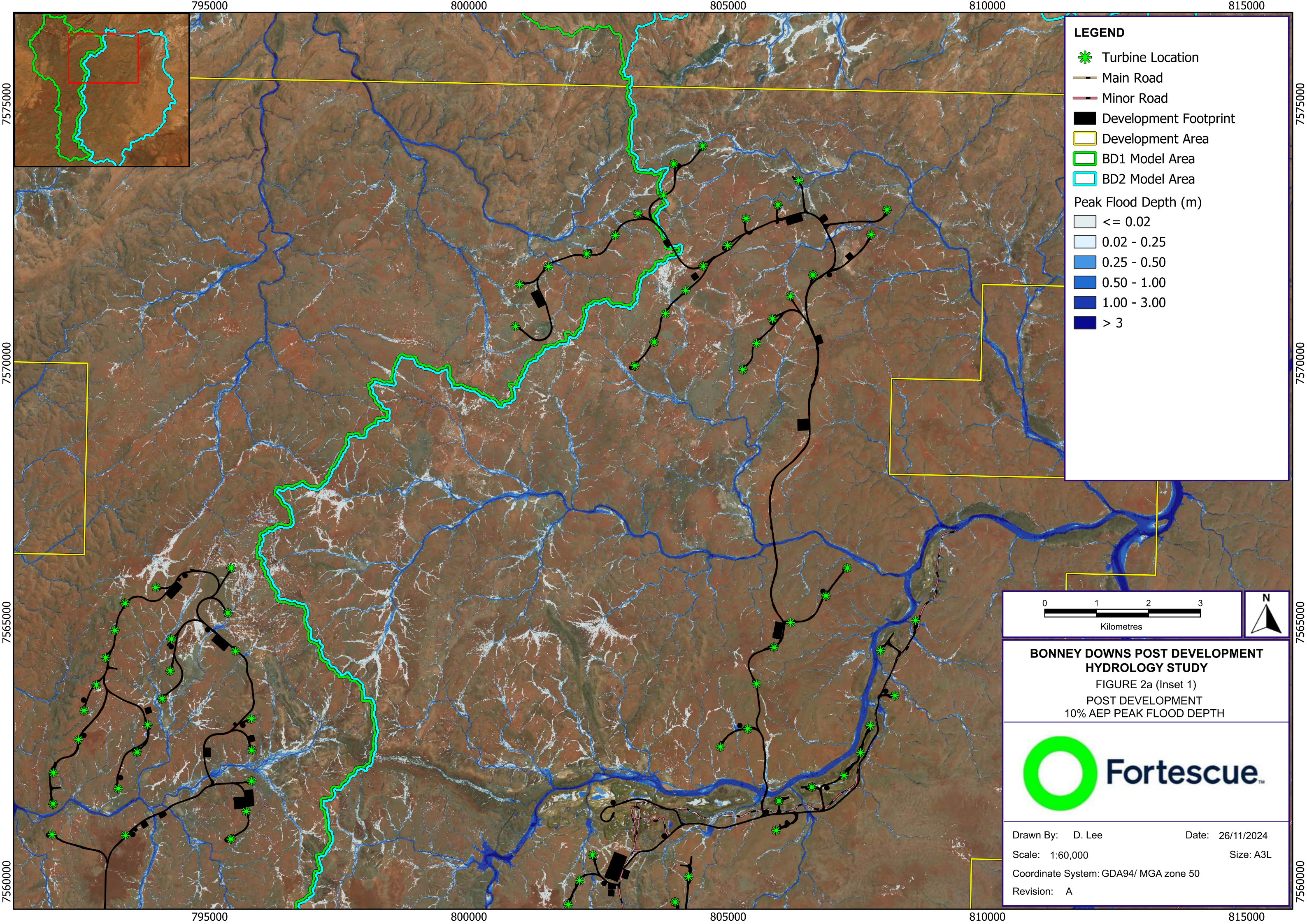
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3

0 4 8 12
Kilometres

**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 2
POST DEVELOPMENT
10% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:200,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

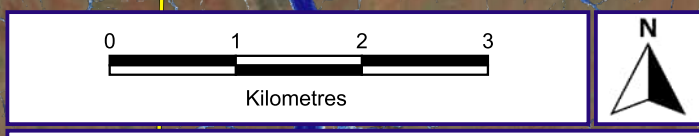


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

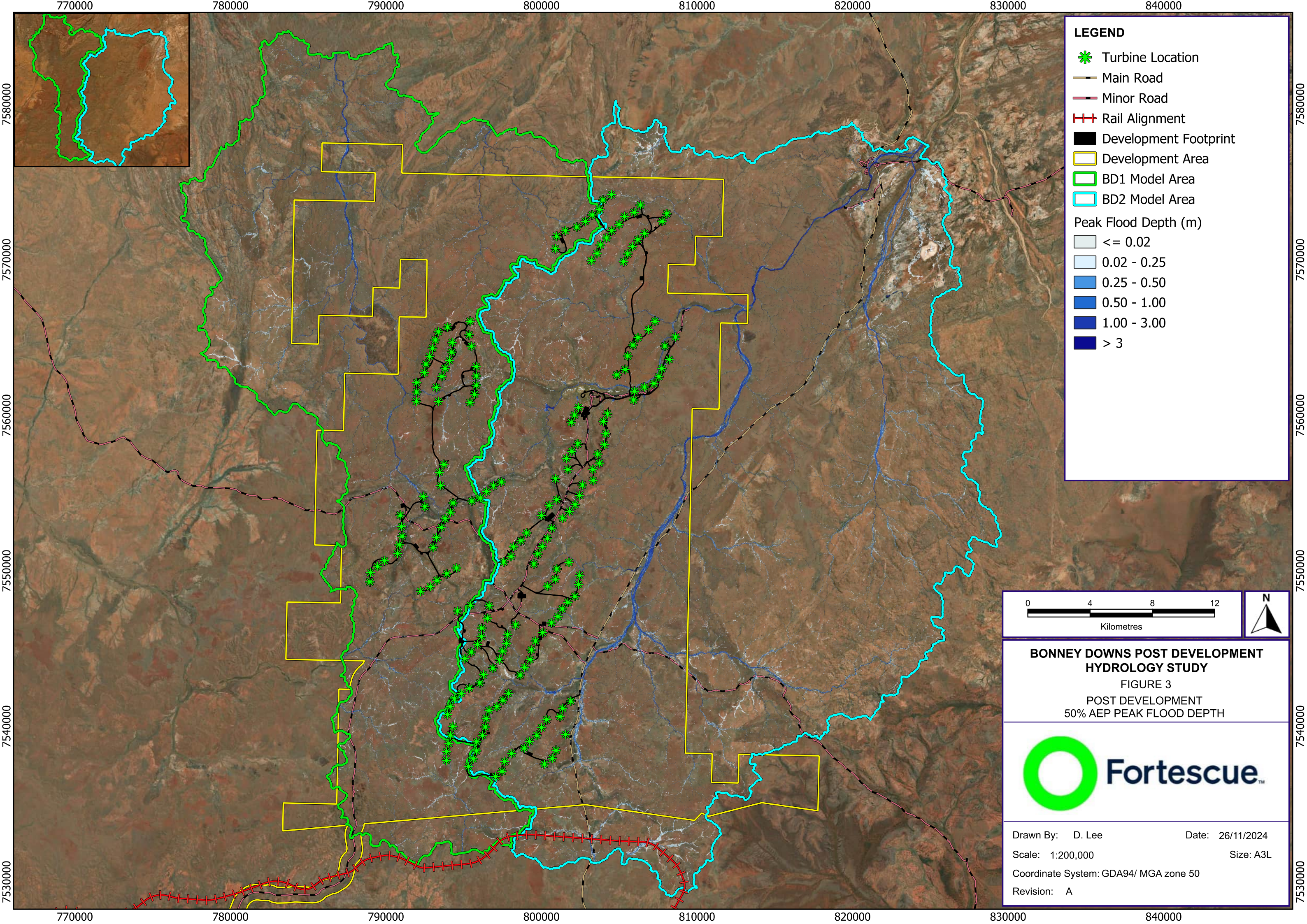
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 2a (Inset 1)
POST DEVELOPMENT
10% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

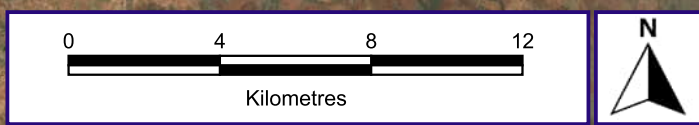


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Rail Alignment
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

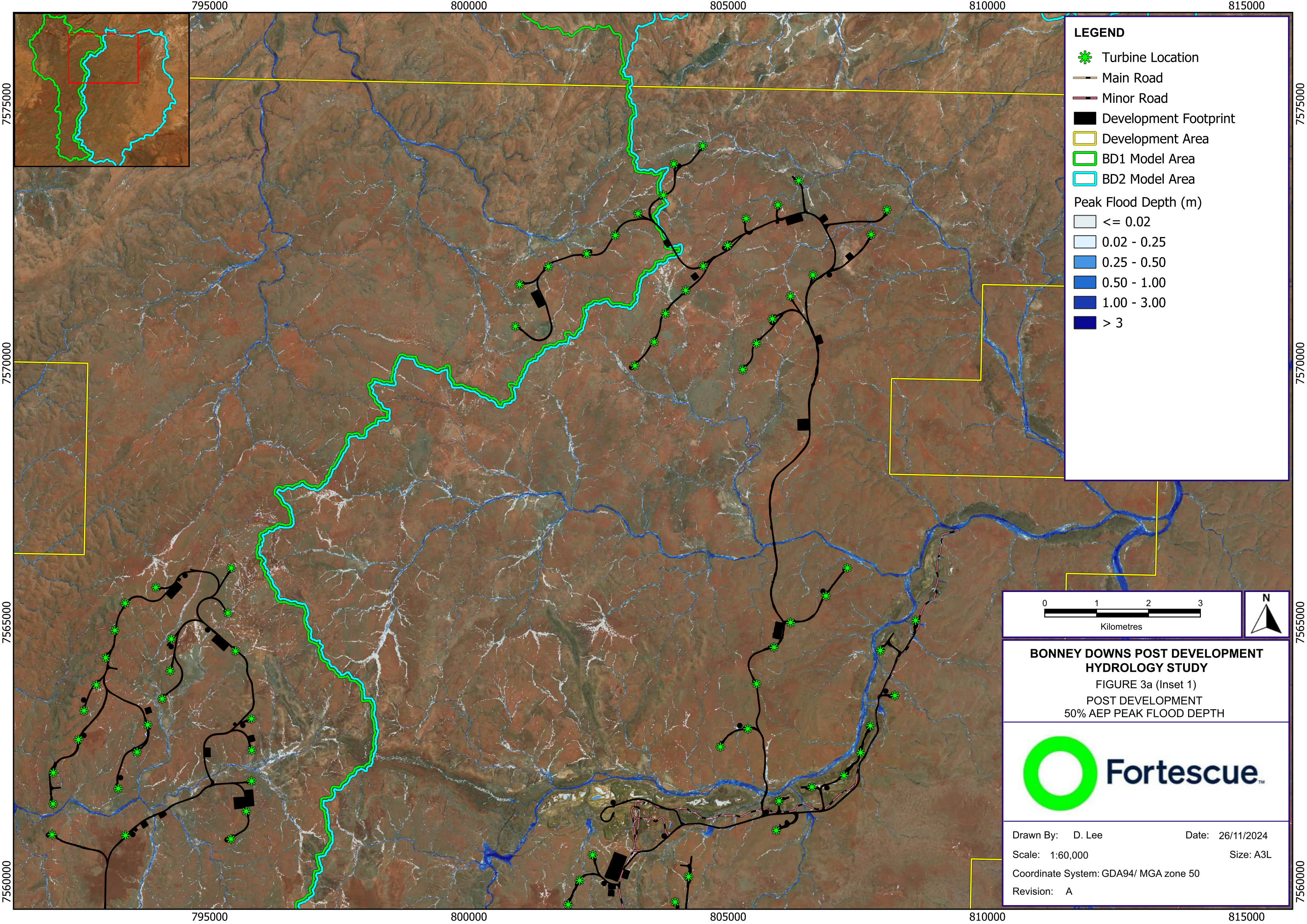
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
 FIGURE 3
 POST DEVELOPMENT
 50% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
 Scale: 1:200,000 Size: A3L
 Coordinate System: GDA94/ MGA zone 50
 Revision: A

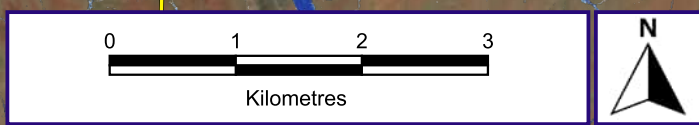


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

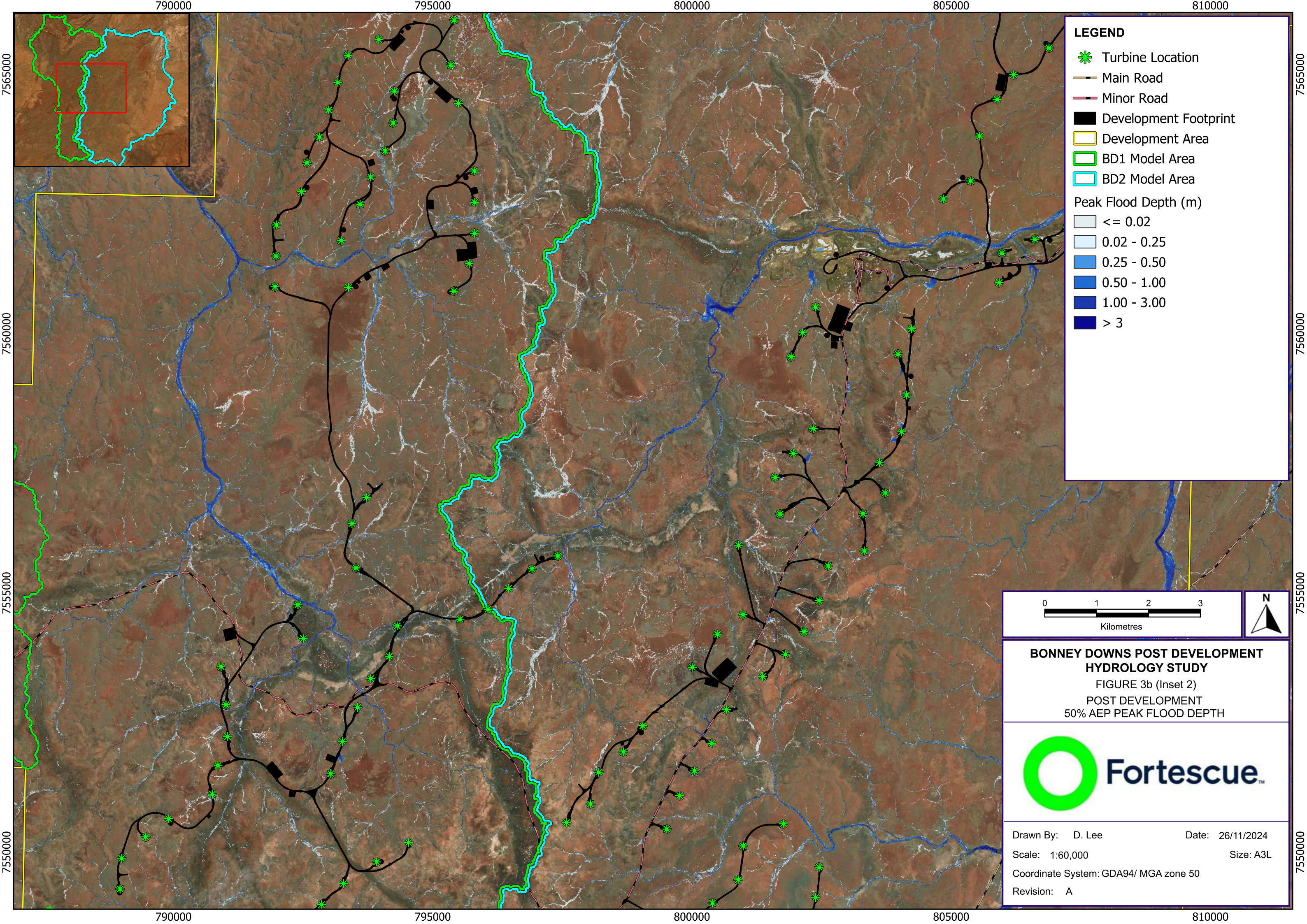
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 3a (Inset 1)
POST DEVELOPMENT
50% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

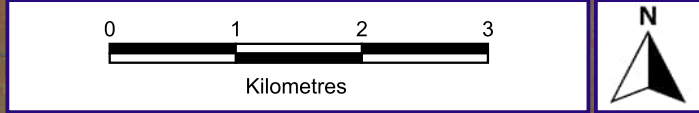


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

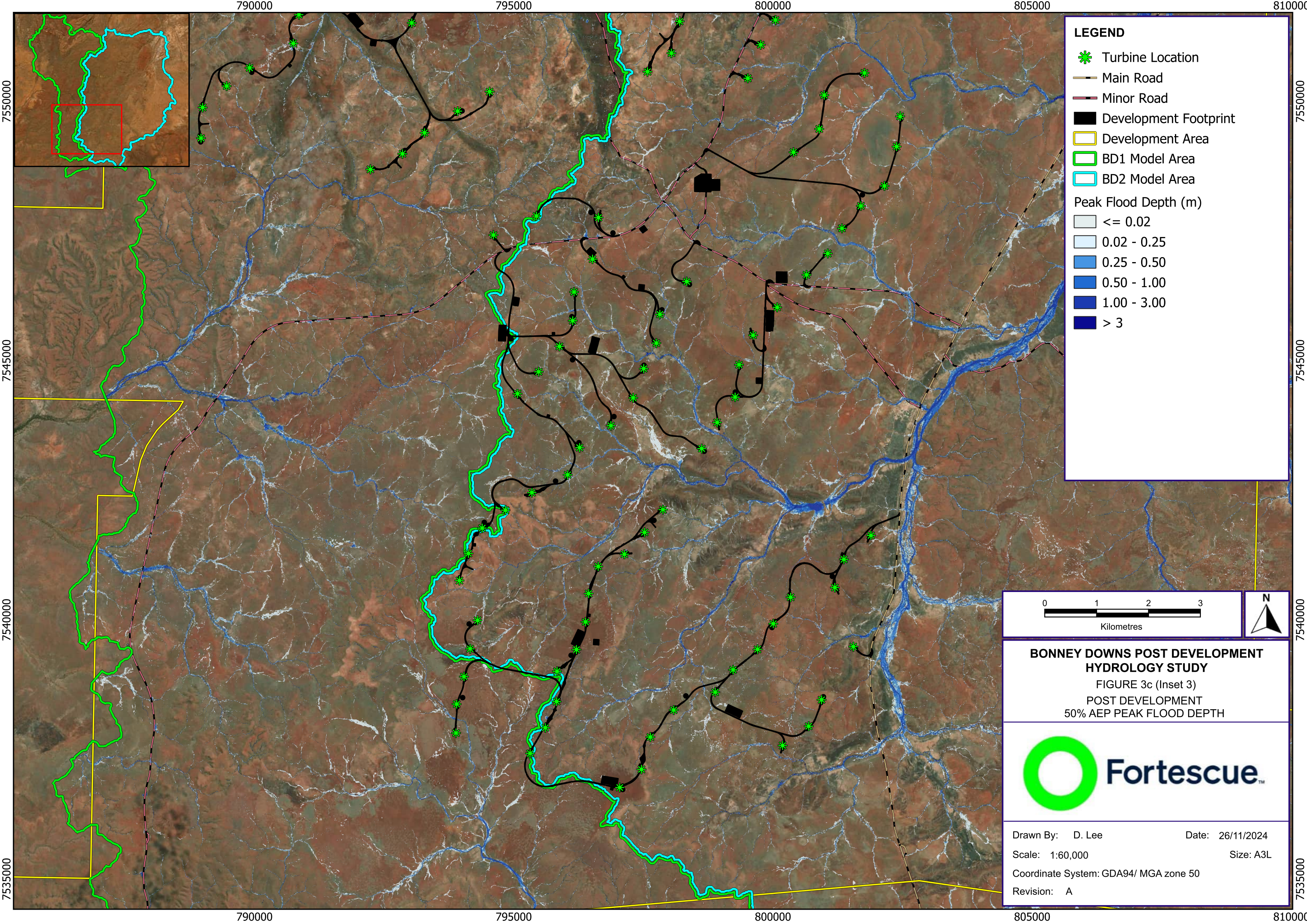
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 3b (Inset 2)
POST DEVELOPMENT
50% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Flood Depth (m)

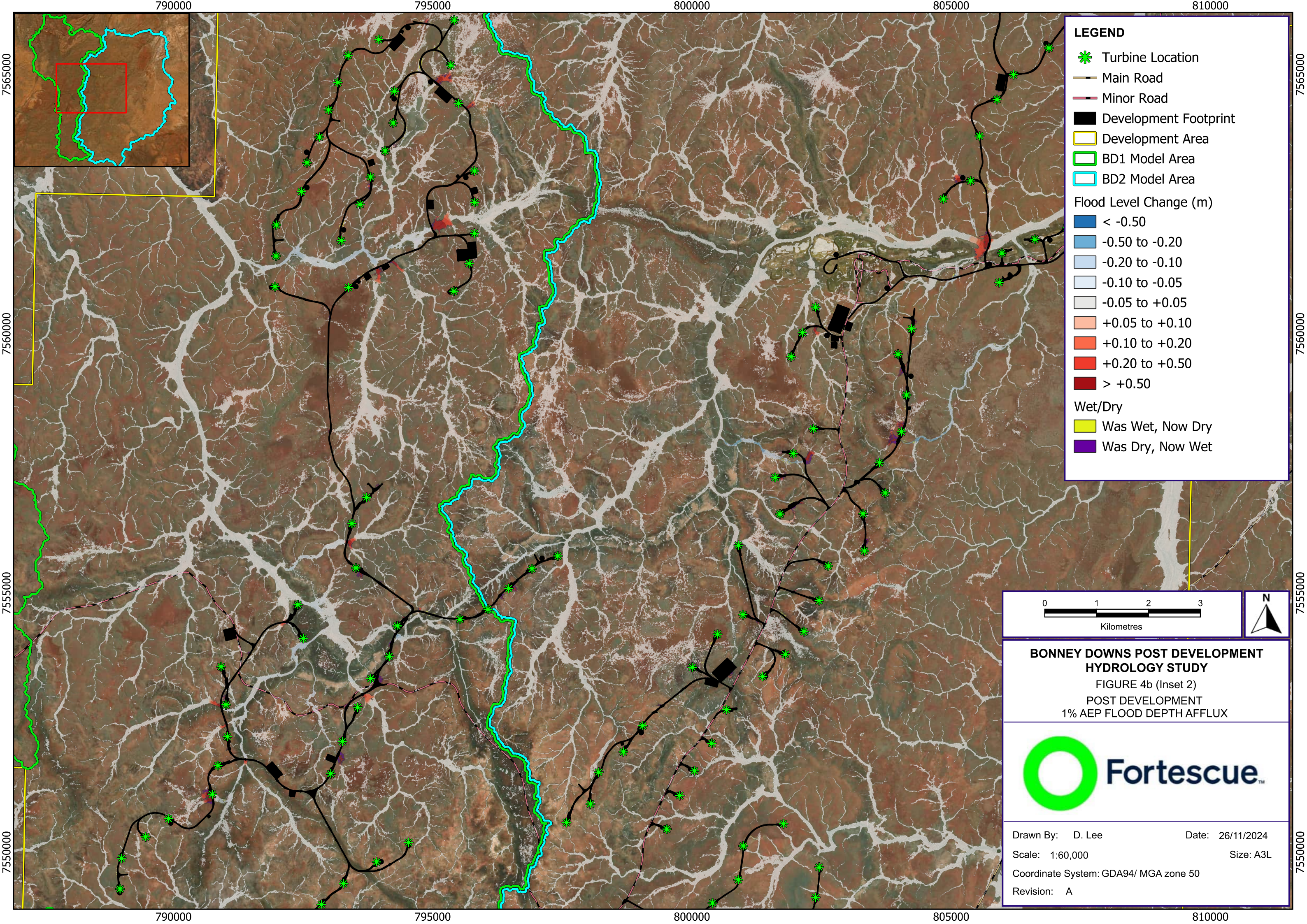
- <= 0.02
- 0.02 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 3.00
- > 3

0 1 2 3
Kilometres

**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 3c (Inset 3)
POST DEVELOPMENT
50% AEP PEAK FLOOD DEPTH



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

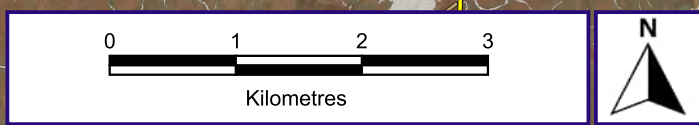
- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

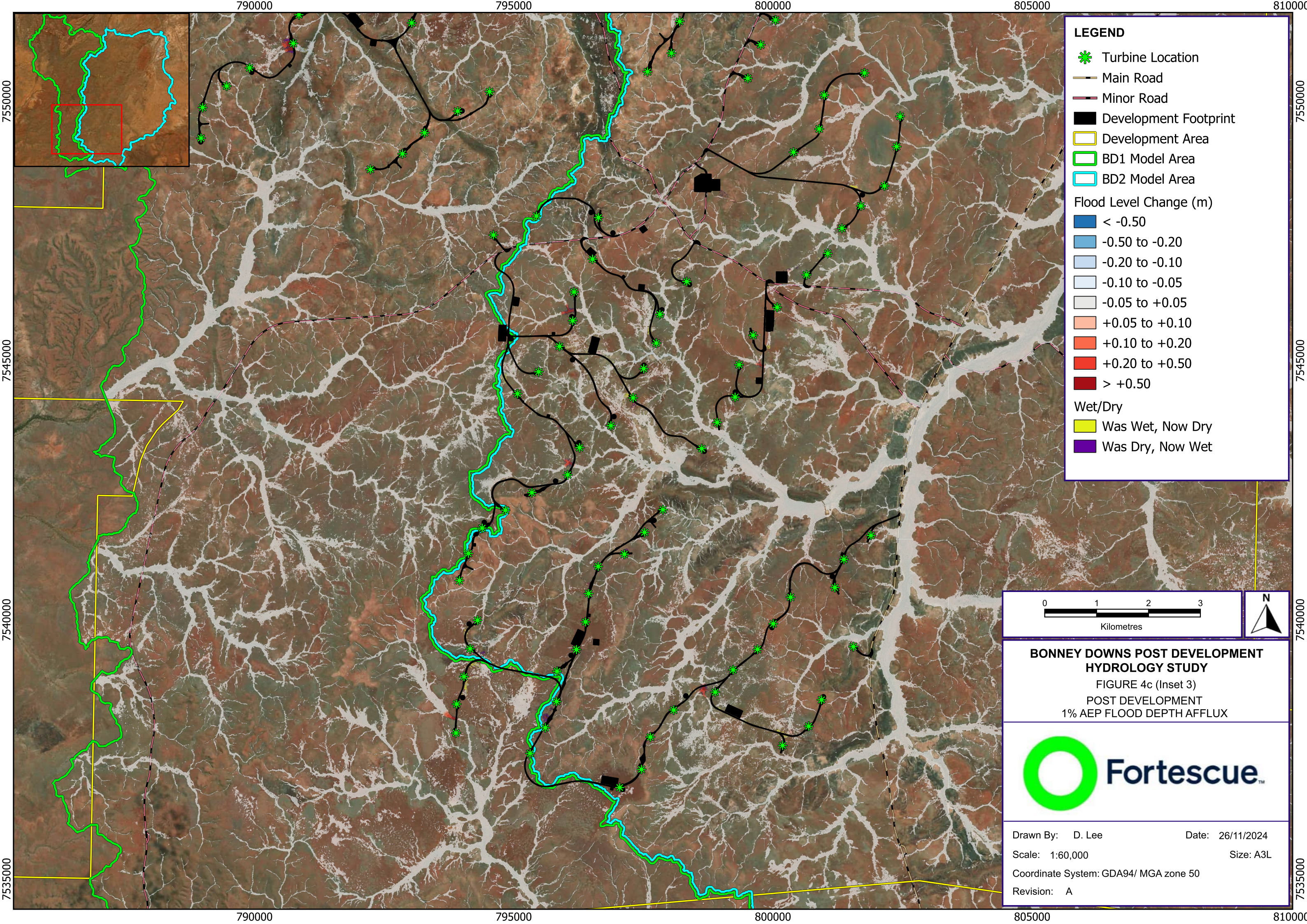
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 4b (Inset 2)
POST DEVELOPMENT
1% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

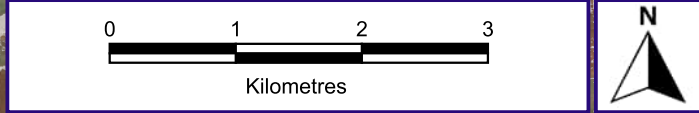
- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

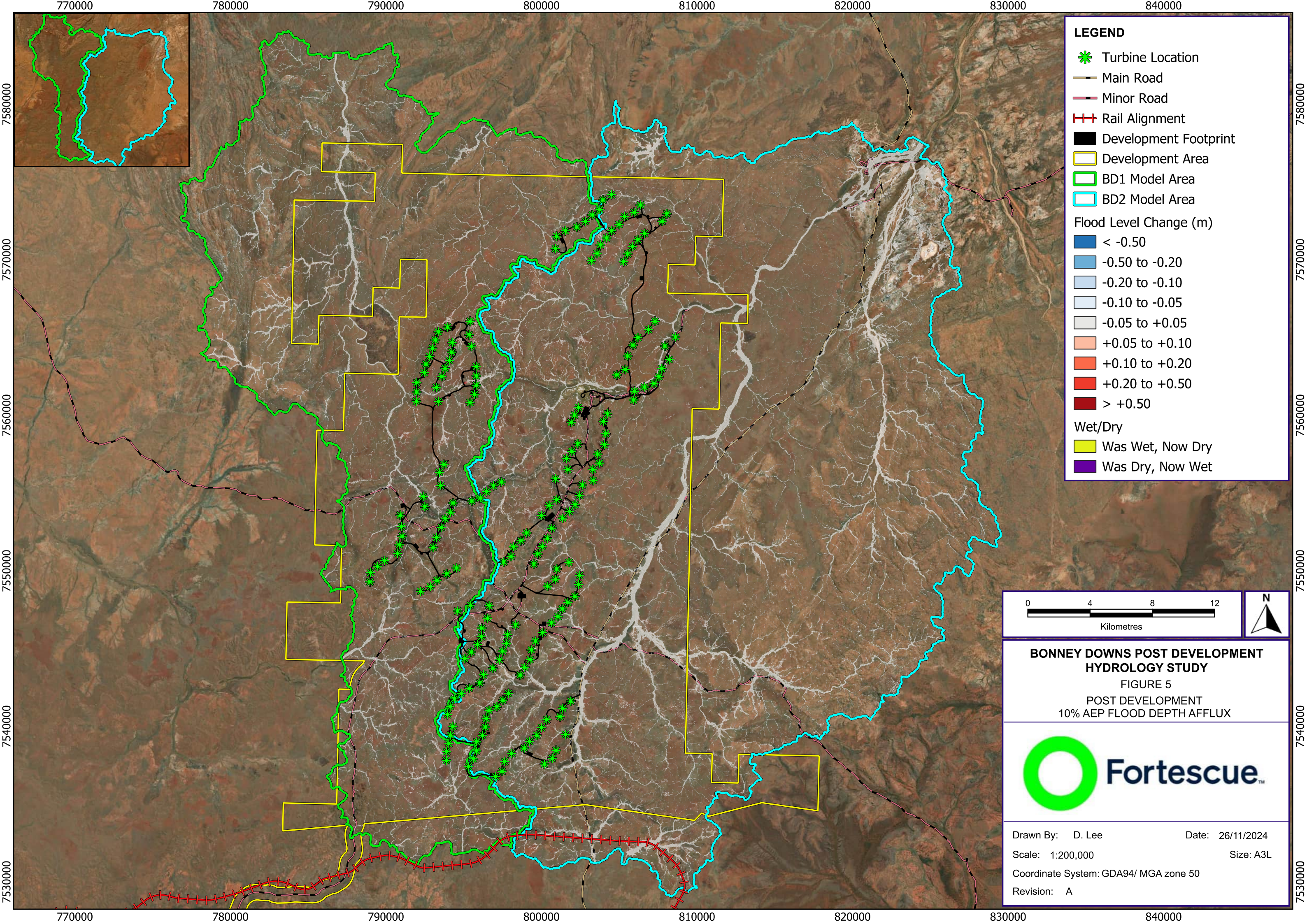
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 4c (Inset 3)
POST DEVELOPMENT
1% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

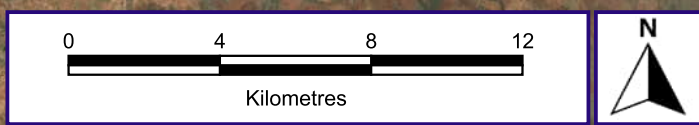
- Turbine Location
- Main Road
- Minor Road
- Rail Alignment
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

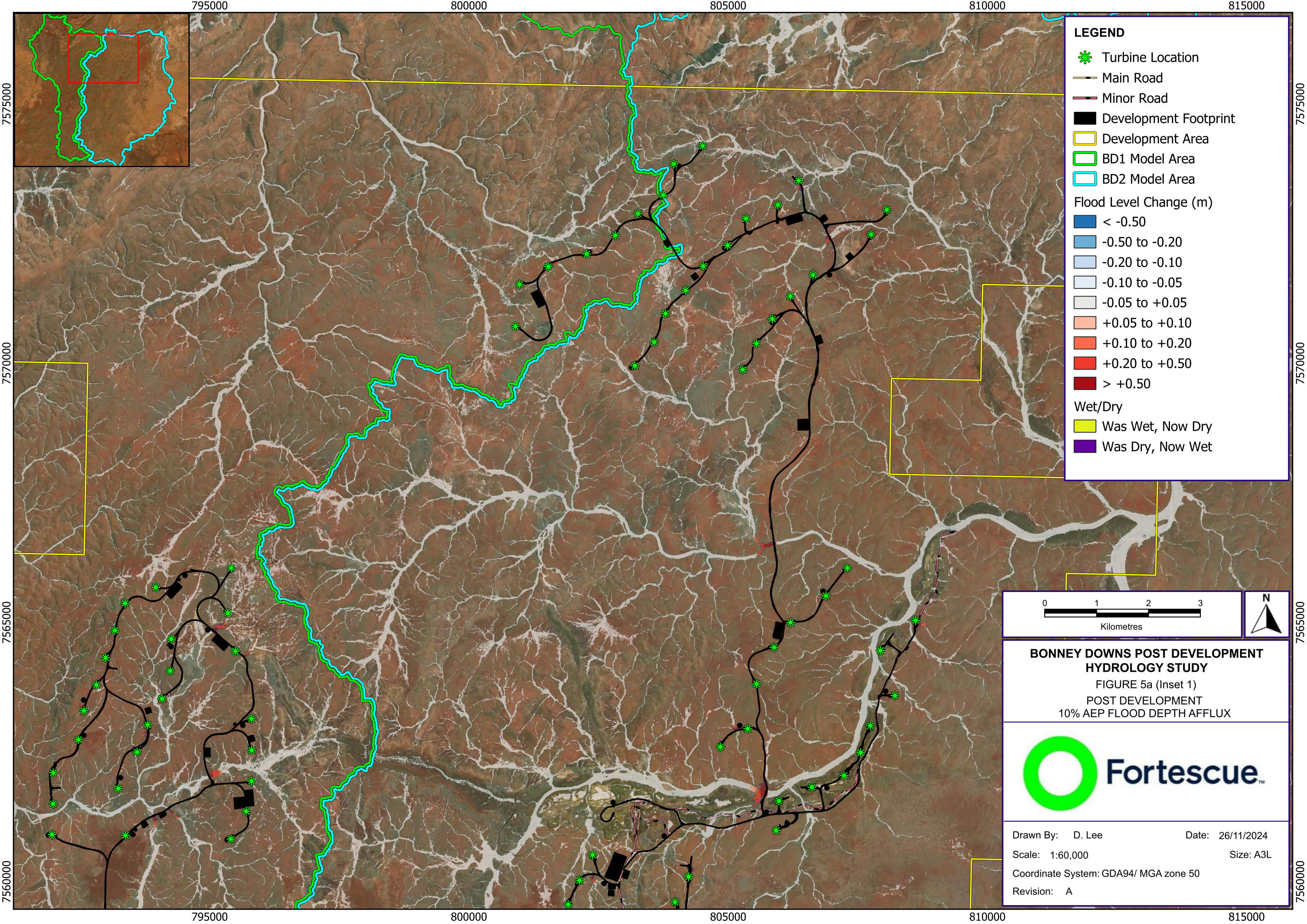
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
 FIGURE 5
 POST DEVELOPMENT
 10% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
 Scale: 1:200,000 Size: A3L
 Coordinate System: GDA94/ MGA zone 50
 Revision: A



LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

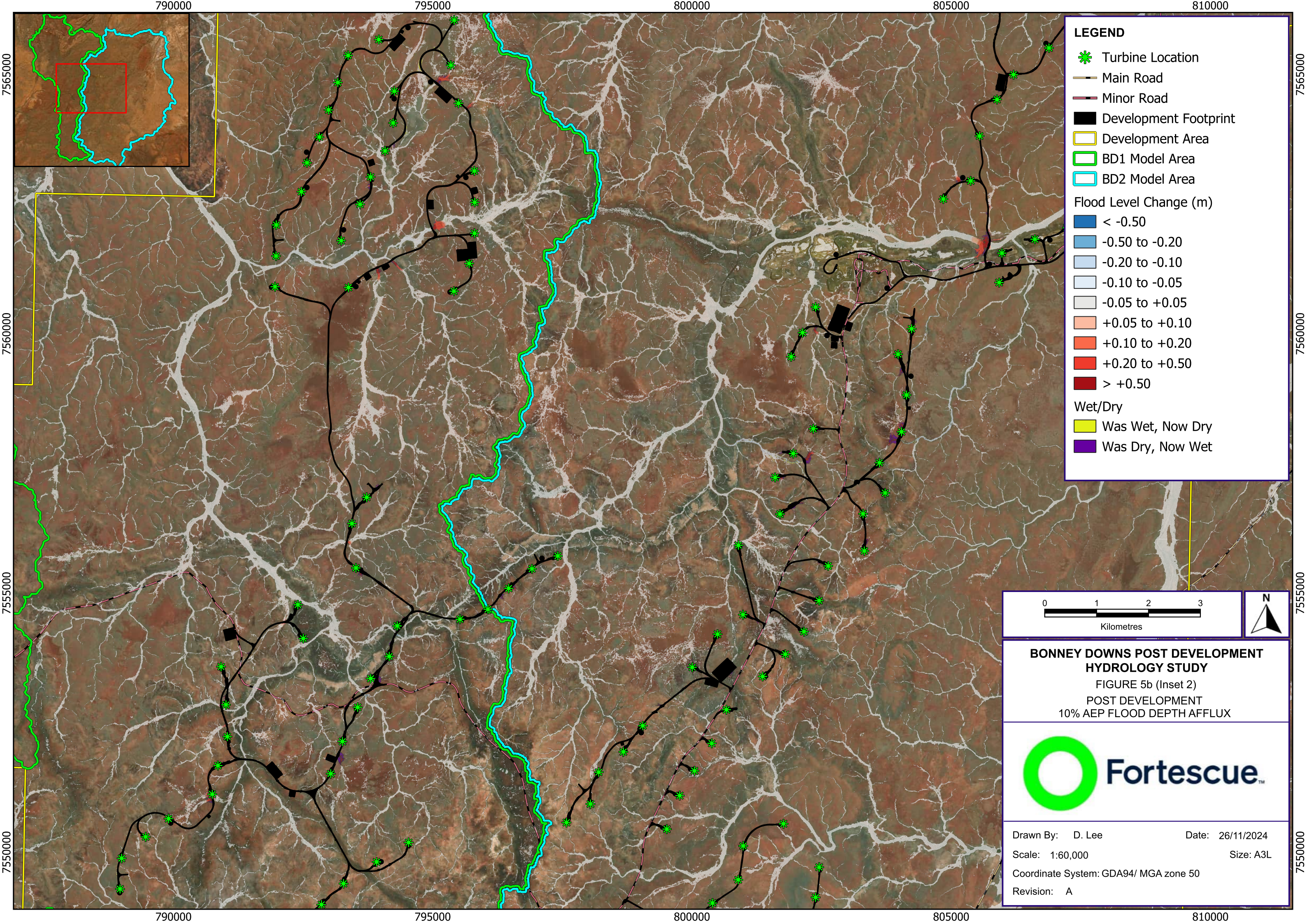
- Was Wet, Now Dry
- Was Dry, Now Wet

0 1 2 3
Kilometres

**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 5a (Inset 1)
POST DEVELOPMENT
10% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

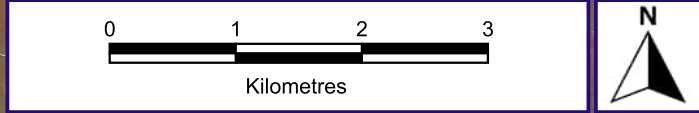
- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

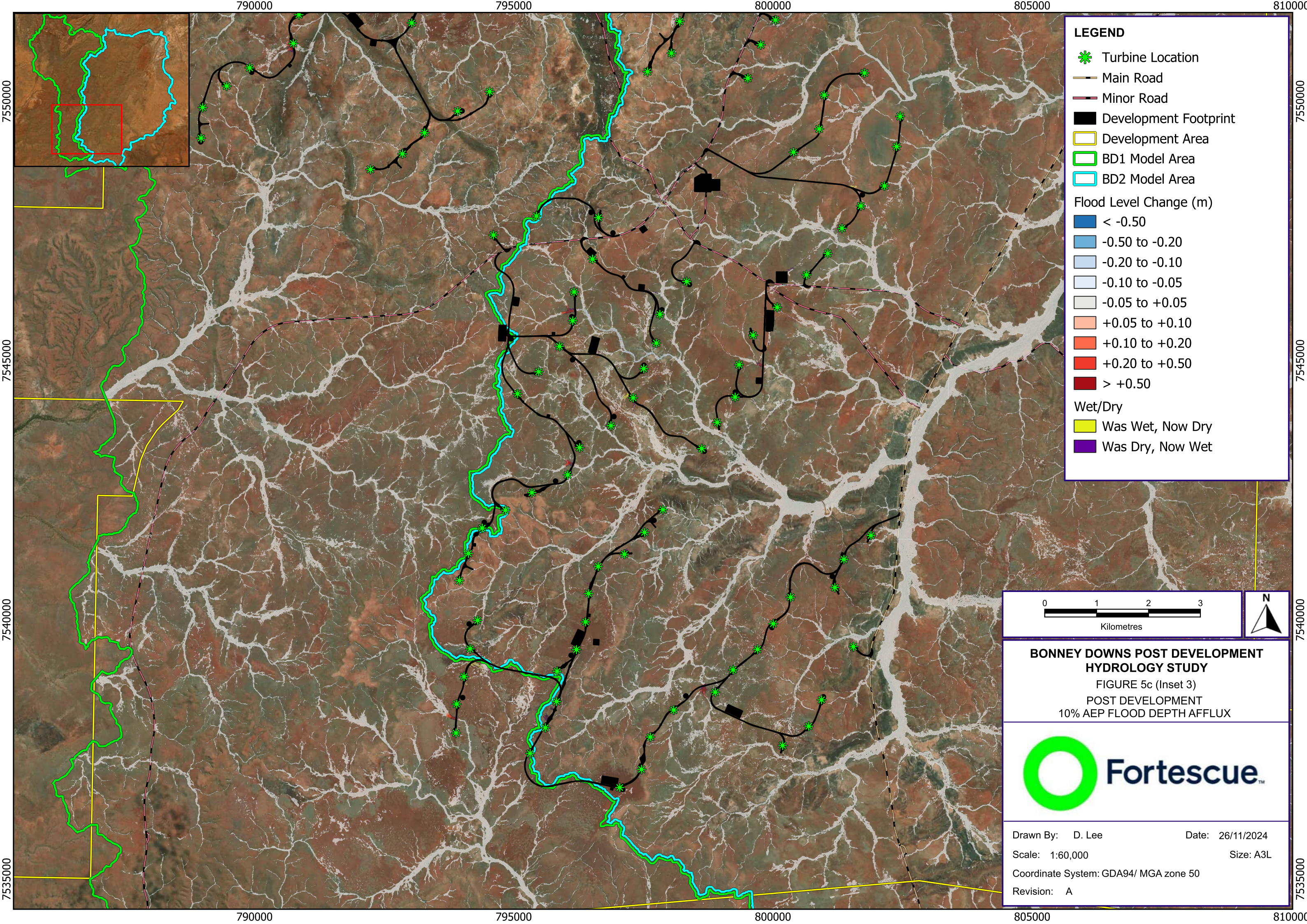
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 5b (Inset 2)
POST DEVELOPMENT
10% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

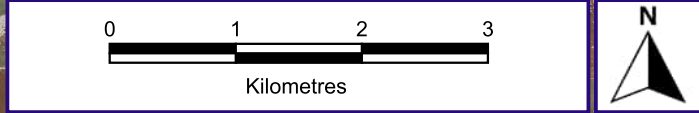
- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

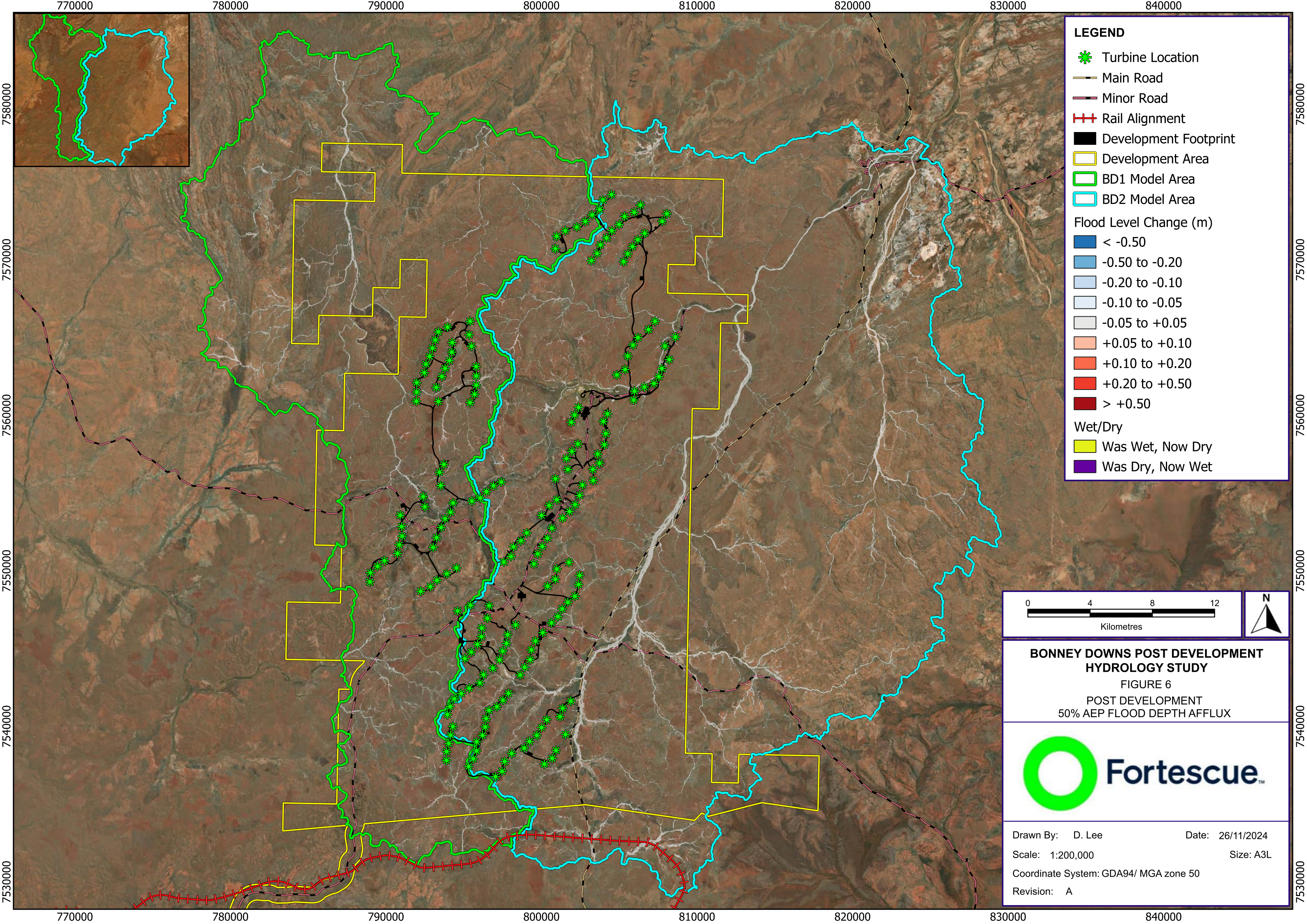
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 5c (Inset 3)
POST DEVELOPMENT
10% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

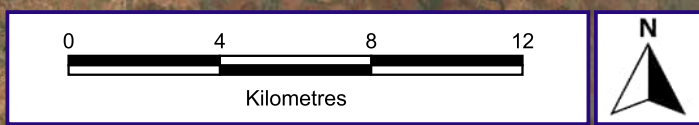
- Turbine Location
- Main Road
- Minor Road
- Rail Alignment
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

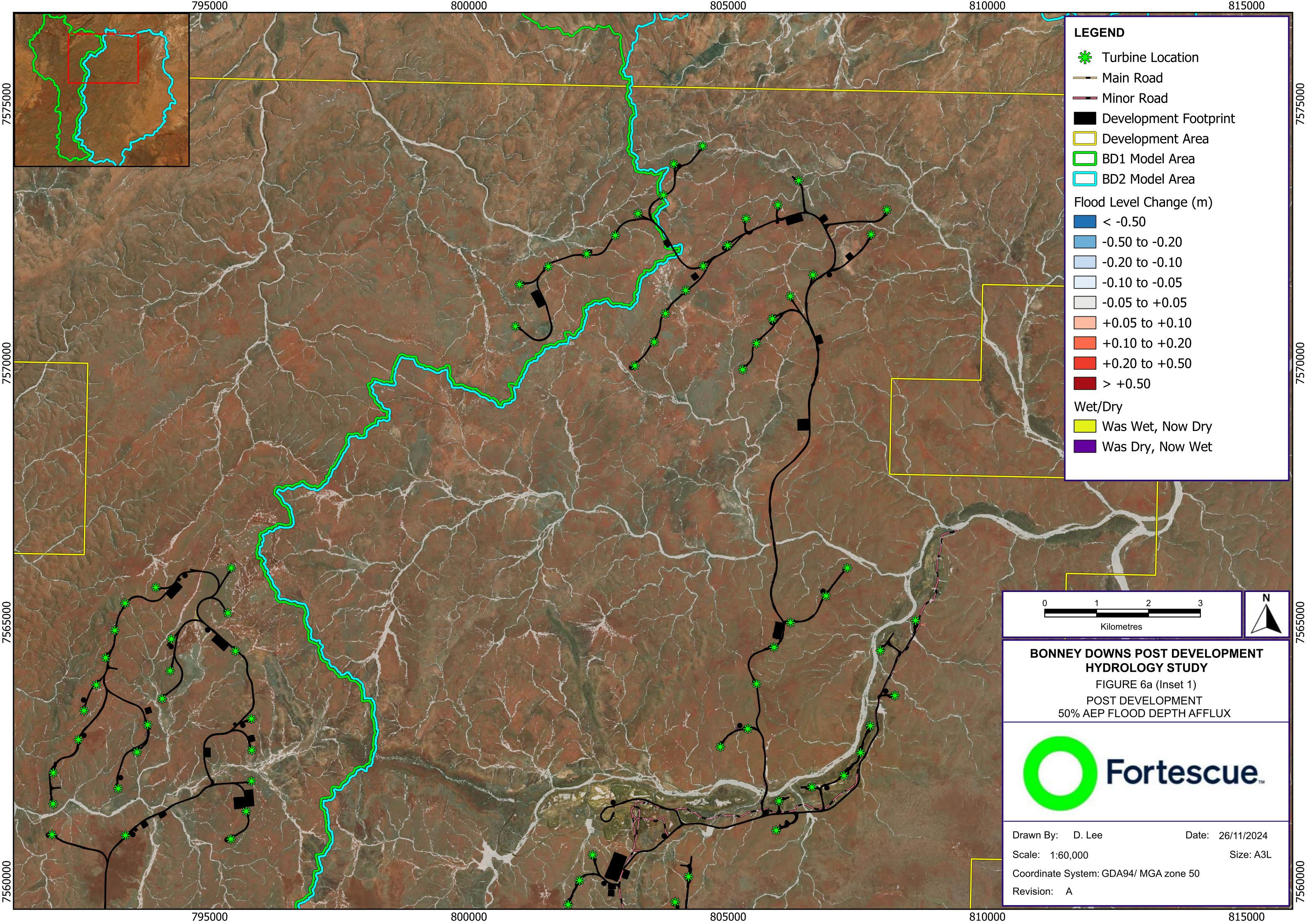
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 6
POST DEVELOPMENT
50% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:200,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

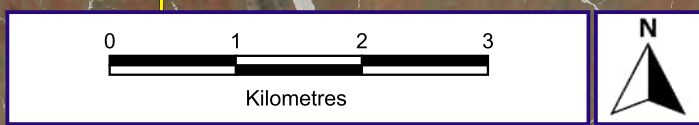
- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

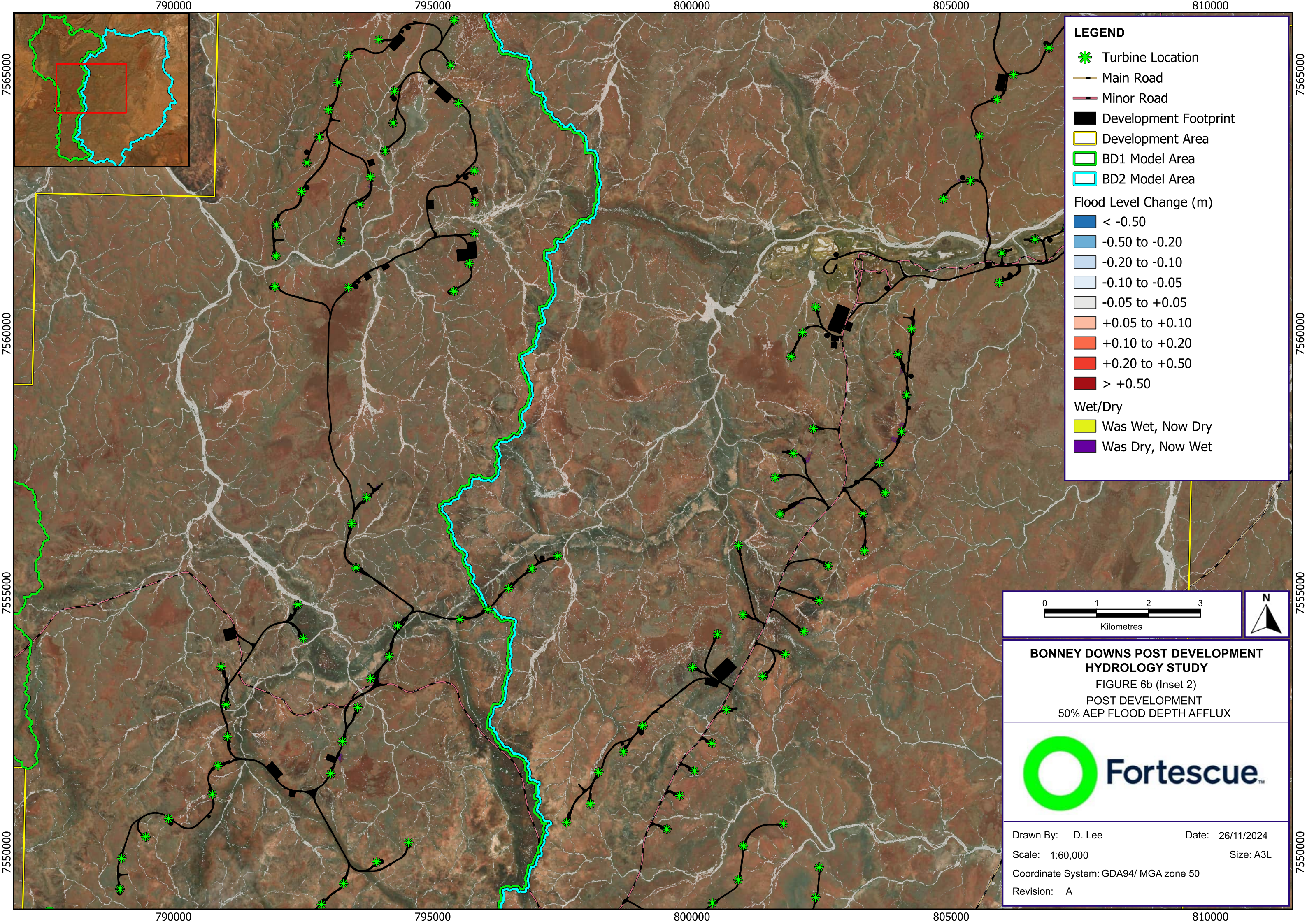
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 6a (Inset 1)
POST DEVELOPMENT
50% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND

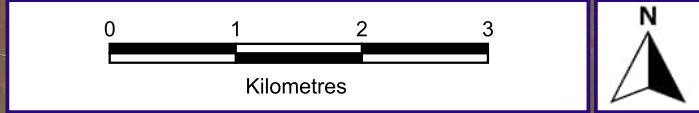
- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

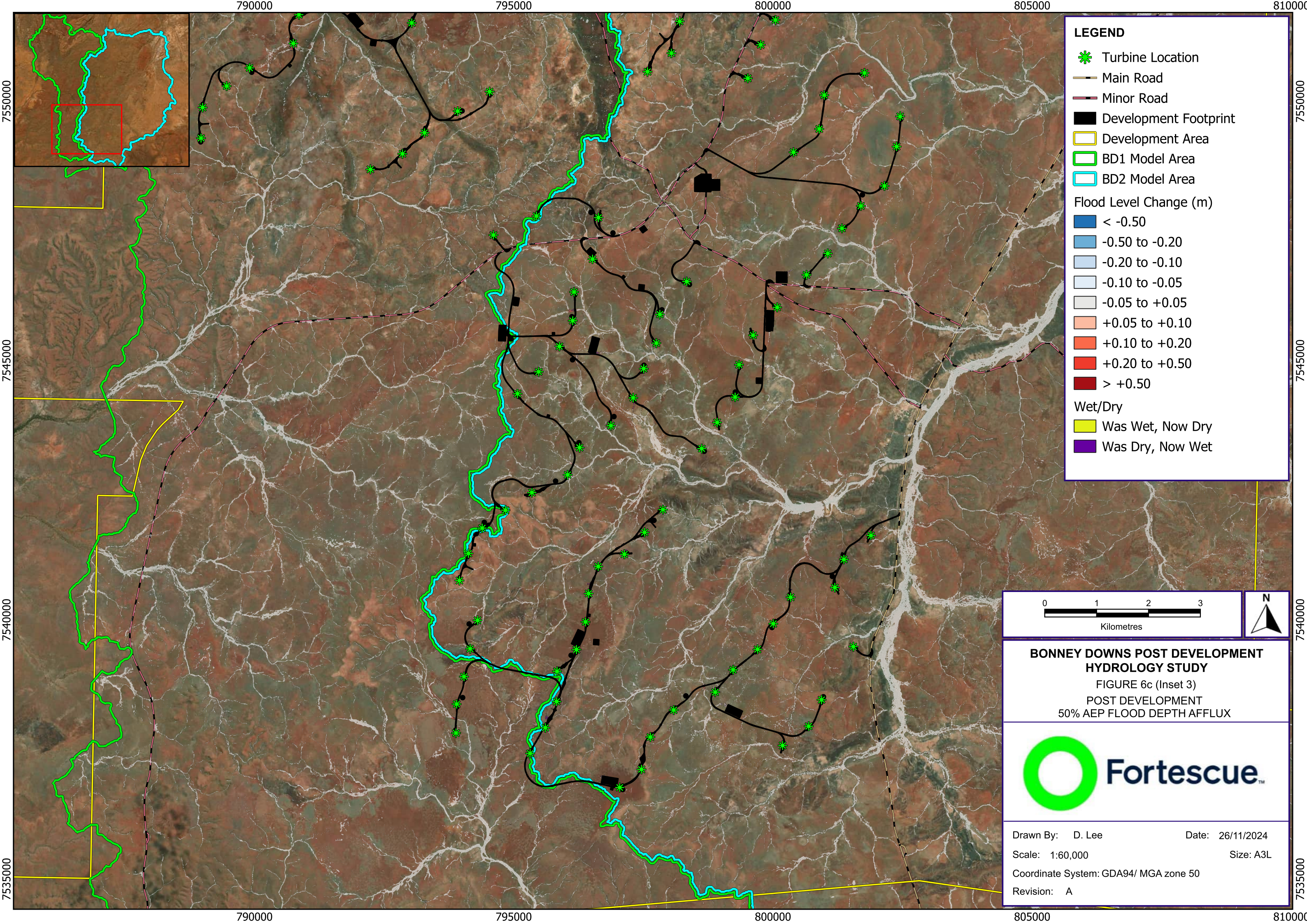
- Was Wet, Now Dry
- Was Dry, Now Wet



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
 FIGURE 6b (Inset 2)
 POST DEVELOPMENT
 50% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
 Scale: 1:60,000 Size: A3L
 Coordinate System: GDA94/ MGA zone 50
 Revision: A



LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Flood Level Change (m)

- < -0.50
- 0.50 to -0.20
- 0.20 to -0.10
- 0.10 to -0.05
- 0.05 to +0.05
- +0.05 to +0.10
- +0.10 to +0.20
- +0.20 to +0.50
- > +0.50

Wet/Dry

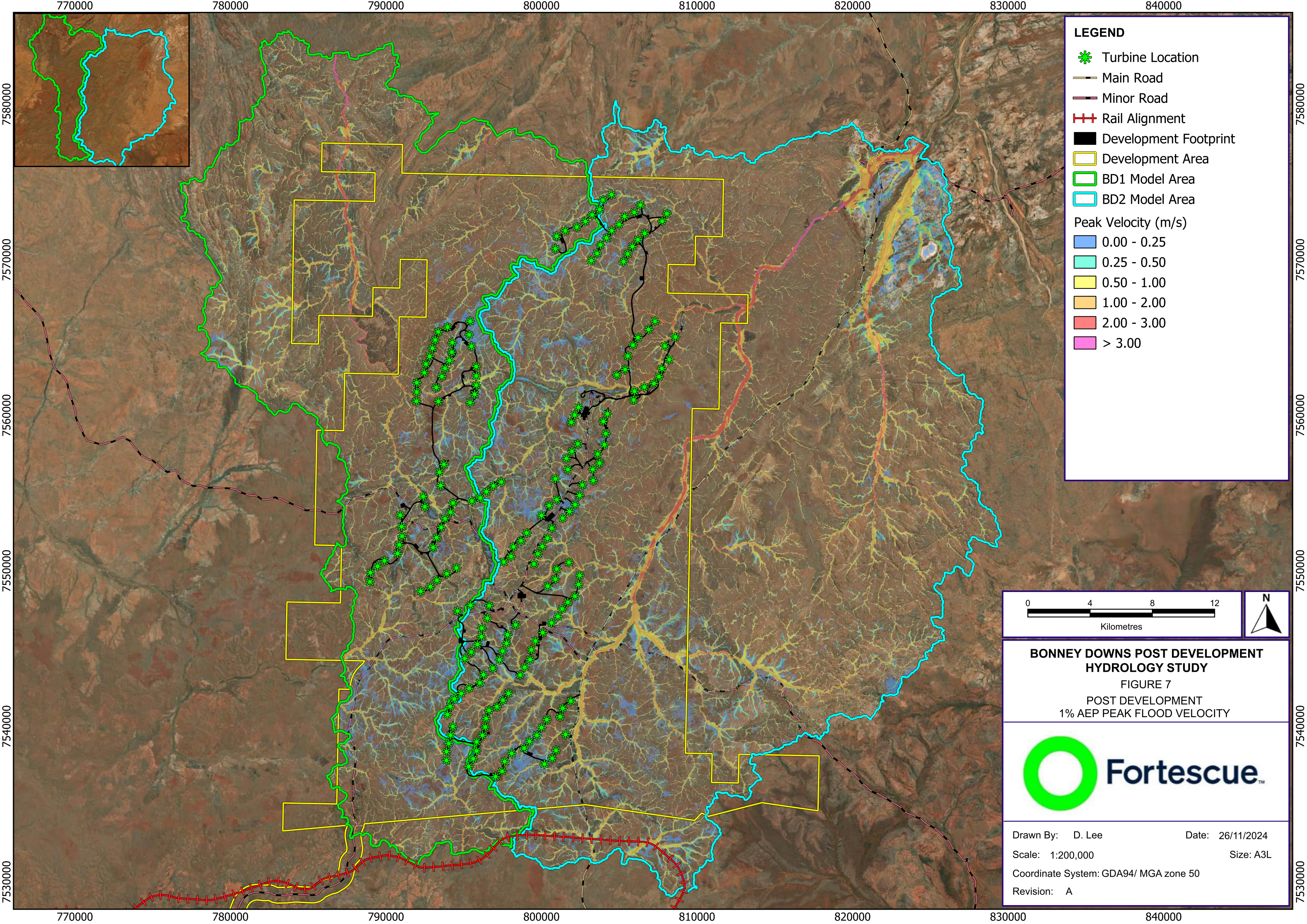
- Was Wet, Now Dry
- Was Dry, Now Wet

0 1 2 3
Kilometres

**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 6c (Inset 3)
POST DEVELOPMENT
50% AEP FLOOD DEPTH AFFLUX



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

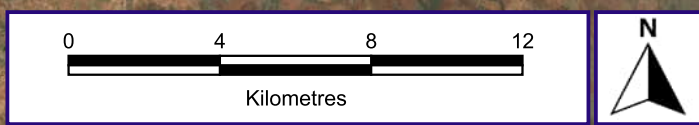


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Rail Alignment
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Velocity (m/s)

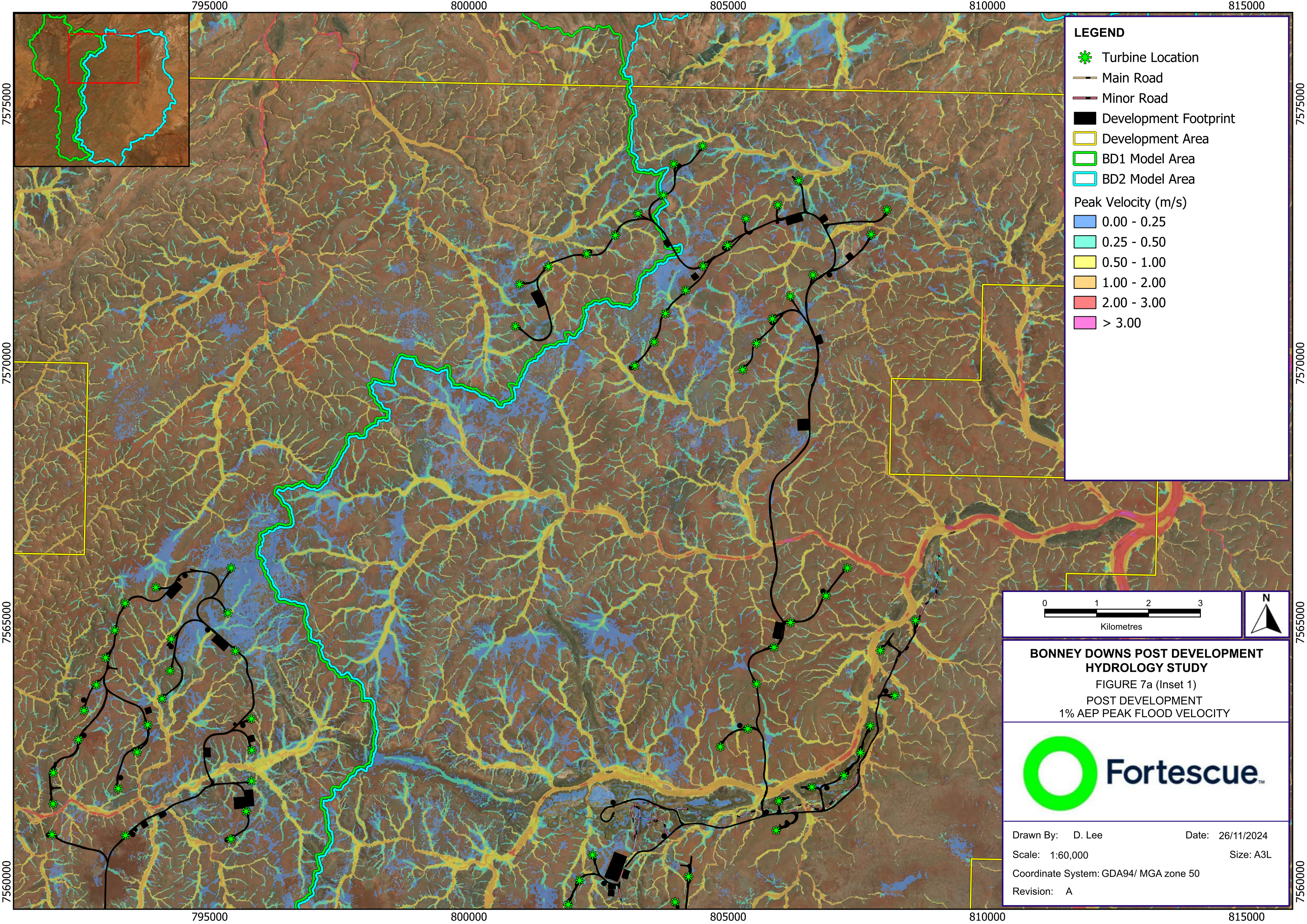
- 0.00 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 2.00
- 2.00 - 3.00
- > 3.00



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 7
POST DEVELOPMENT
1% AEP PEAK FLOOD VELOCITY



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:200,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

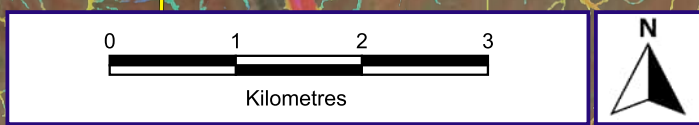


LEGEND

- Turbine Location
- Main Road
- Minor Road
- Development Footprint
- Development Area
- BD1 Model Area
- BD2 Model Area

Peak Velocity (m/s)

- 0.00 - 0.25
- 0.25 - 0.50
- 0.50 - 1.00
- 1.00 - 2.00
- 2.00 - 3.00
- > 3.00



**BONNEY DOWNS POST DEVELOPMENT
HYDROLOGY STUDY**
FIGURE 7a (Inset 1)
POST DEVELOPMENT
1% AEP PEAK FLOOD VELOCITY



Drawn By: D. Lee Date: 26/11/2024
Scale: 1:60,000 Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A