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Report

East Pilbara Generation Hub - Post Development Hydrology Study

Pilbara Decarbonisation Wind Generation Project

30 September 2024

AUSS0003-0000-HG-REP-0002

Rev: 0

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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 East Pilbara Generation Hub (EPGH) Wind Project. The assessment is intended to support detailed design of infrastructure and environmental approvals submissions.

The EPGH site is located approximately 30 km southeast of the town of Marble Bar in the Pilbara region of Western Australia. Two wind turbine groups (Group 3 and Group 4) are proposed, with Group 3 located in the headwaters of the Camel Creek catchment and Group 4 within the Yandicoogina Creek catchment, both of which contribute to the regional Coongan River system.

A baseline hydrological assessment was completed in 2022 to assess the existing surface water flow behaviour within and surrounding the EPGH development area. TUFLOW hydraulic models, developed as part of the baseline study, were used and 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 in proximity to the turbine access track/waterway crossings. Disruptions to regional surface water flows, downstream of the development area, are considered negligible (< 1 % change in flow volume, flood depths and velocities) for all modelled events.

Two different types of water crossings (low-level floodway and floodway crossing with culverts) were proposed and assessed as part of this assessment. Model results suggest that impacts of the low-level floodway crossings are low. Waterway crossings with culverts, on the other hand, 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. Changes to flow velocities in vicinity of the culvert crossings may influence the existing geomorphological conditions of the creek systems. Higher rate of sediment deposition may occur upstream of the road embankment due to backwater development. Significant increase in soil erosion, however, is not expected as velocity increases caused by the crossings are relatively minor.

Based on the findings of the assessment, the overall impacts of the development on existing 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 the floodway batters and driving surface to minimise erosion and sedimentation downstream.

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1 INTRODUCTION

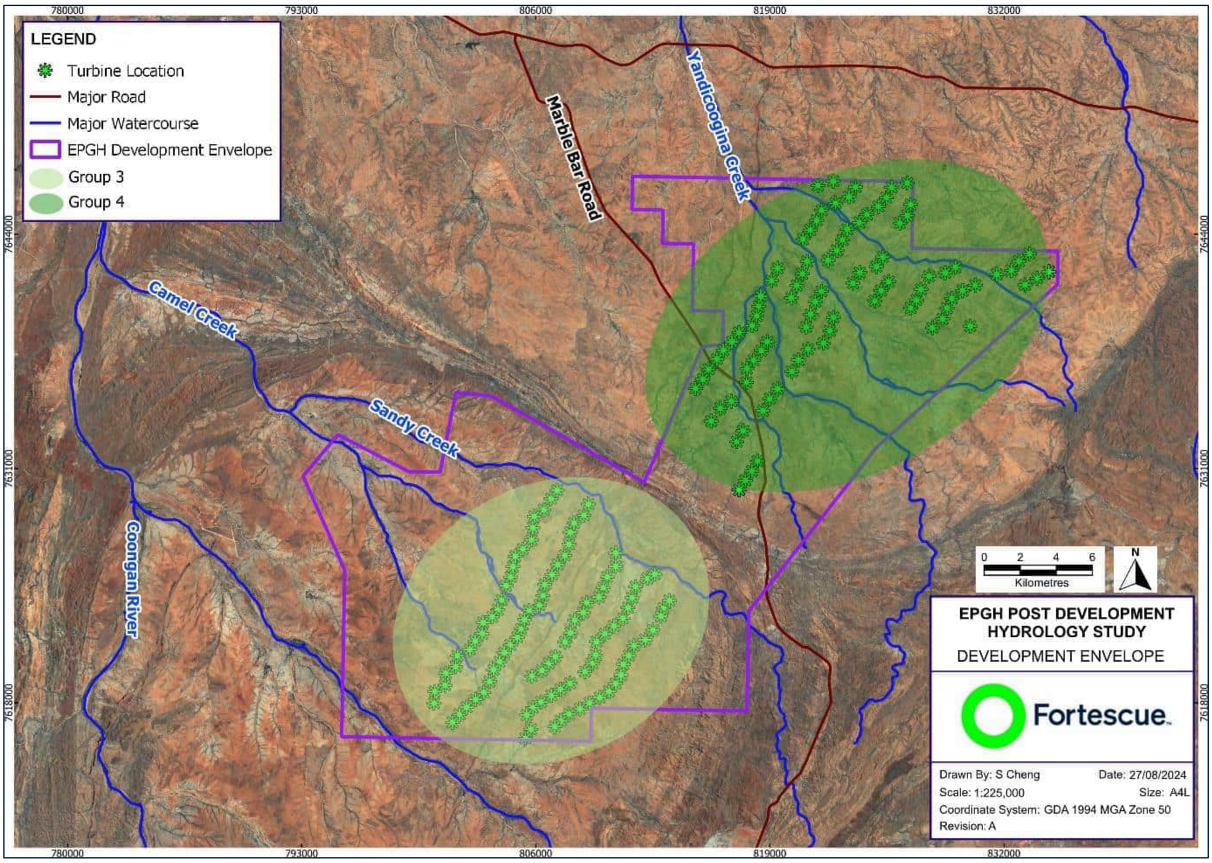
1.1 Background

Fortescue is seeking to achieve 100 % decarbonisation of its Pilbara operations by 2030. In achieving 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 is currently being undertaken by Fortescue to develop two specific wind turbine groups (Group 3 and Group 4) within the East Pilbara Generation Hub (EPGH) site. The turbine groups, each with a target of 100 turbines, 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 Energy Company (PEC) transmission network, a privately owned electrical network which connects to Fortescue's Pilbara based mine sites.

The EPGH Development Envelope covers an area of 62,926 ha and is located approximately 30 km southeast of the town of Marble Bar in the Pilbara region of Western Australia. The EPGH Development Envelope and the locations of the two turbine groups are shown in Figure 1.

Figure 1: East Pilbara Generation Hub Development Envelope



A baseline hydrological assessment was completed in 2022 to assess the existing surface water flow behaviour within and surrounding the EPGH 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 EPGH Baseline Hydrology Study Report (AUS0311-0000-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.

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 work.
- 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
EPGH Baseline Hydrology Study	AUS0311-0000-HG-REP-0001	Baseline hydrological assessment to develop an understanding of surface water flow behaviour within and surrounding the EPGH development area. The assessment is intended to support PFS layout/design of infrastructure and regulatory approval submissions.

Document	Document Number	Description
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.
East Pilbara Creek Crossing Design Standard Review	AUSS0003-0000-PM-KDN-0001	A Key Decision Note summarising the assessment undertaken to define an acceptable design standard for waterway crossings along turbine access tracks within EPGH.
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 estimation and to support the environmental approvals process.

2 CATCHMENT CHARACTERISTICS

This section provides a summary of the catchment characteristics for the EPGH 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 (AUS0311-0000-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 system[1], the EPGH development area is described as desert: hot (persistently dry).

Rainfall records are available from BoM weather station at Marble Bar (BoM Ref: 004016), the closest operating weather station with sufficient long-term data record for analysis. The average annual rainfall estimated for the period from 1896 to 2023 is 363 mm (based on October to September water year). Rainfall is highly variable between years with the annual recorded rainfall for the area varying from 60 mm to 920 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 Marble Bar is presented in Figure 3.

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

The mean annual Class A pan evaporation estimated for the area (from BoM gridded data, 1975 to 2000) is approximately 3,365 mm, which exceeds the mean annual rainfall keeping the landscape typically dry. Monthly evaporation at Marble Bar is provided in Figure 3.

Figure 2: Annual Rainfall at Marble Bar (BoM Ref: 004016)

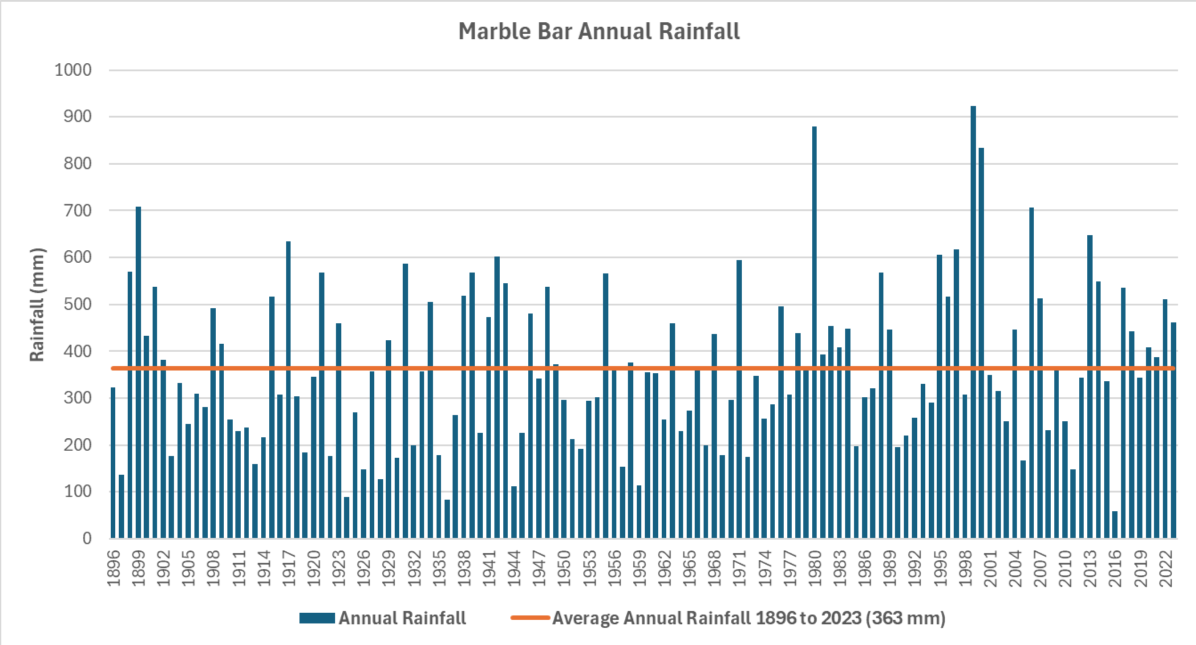
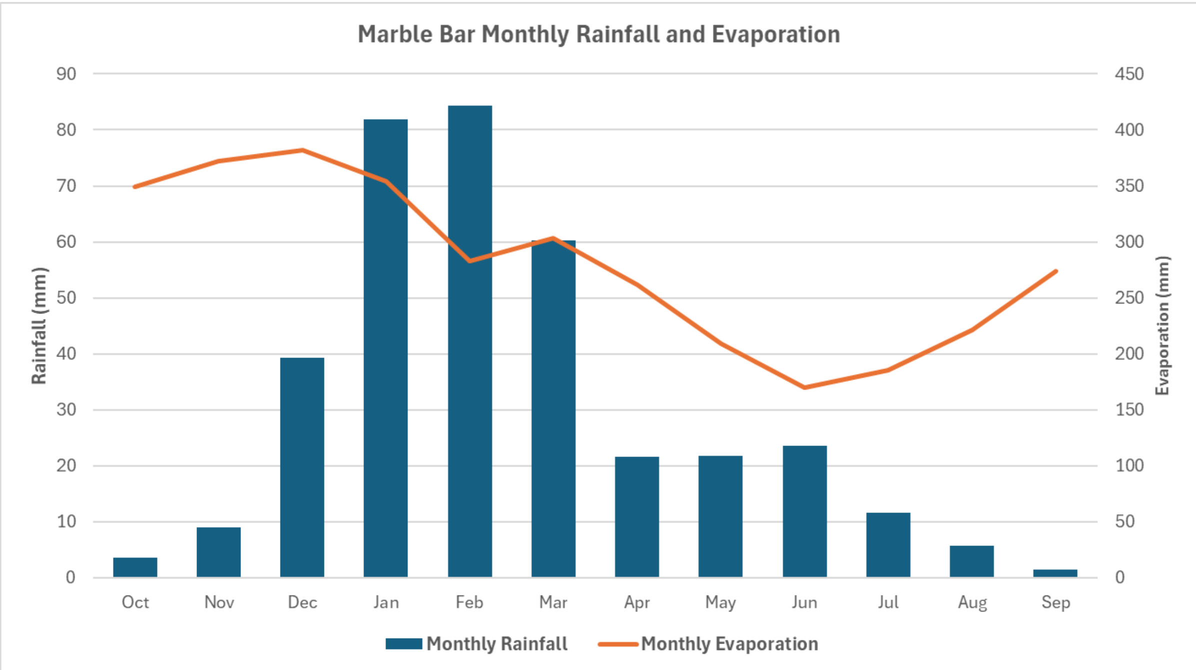


Figure 3: Monthly Rainfall and Evaporation at Marble Bar (BoM Ref: 004016)



2.2 Catchments

The EPGH development area is located within the Coongan River catchment, which has a total area of approximately 7,090 km^2. The headwaters of the Coongan River rise from the Chichester Range. The river flows in a northerly direction past Marble Bar then through the Gorge Range before discharging into the De Grey River, approximately 115 km downstream of EPGH. Major tributaries of the Coongan River include Camel Creek, Talga River and Emu Creek.

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 in operation since 1966 with a total of 57 years of data record. Detailed analysis of observed streamflow records at the gauging station was carried out, as part of the baseline assessment, to inform the hydrologic and hydraulic modelling for the development area. Table 2 illustrates the peak flow estimates for the Coongan River catchment based on Flood Frequency Analysis (FFA) of streamflow records at Marble Bar.

Table 2: Peak Flow Estimates for Coongan River Catchment

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

Note: Annual Exceedance Probability

The main creek systems of the development area are presented in Figure 4. The contributing catchments of the development area (Camel Creek catchment for Group 3 and Yandicoogina Creek catchment for Group 4) generally drain in a north-westerly direction towards the Coongan/Talga Rivers. The general catchment characteristics for the main creek systems contributing to the development area are presented in Table 3.

Figure 4: EPGH Development Area Main Creek Systems

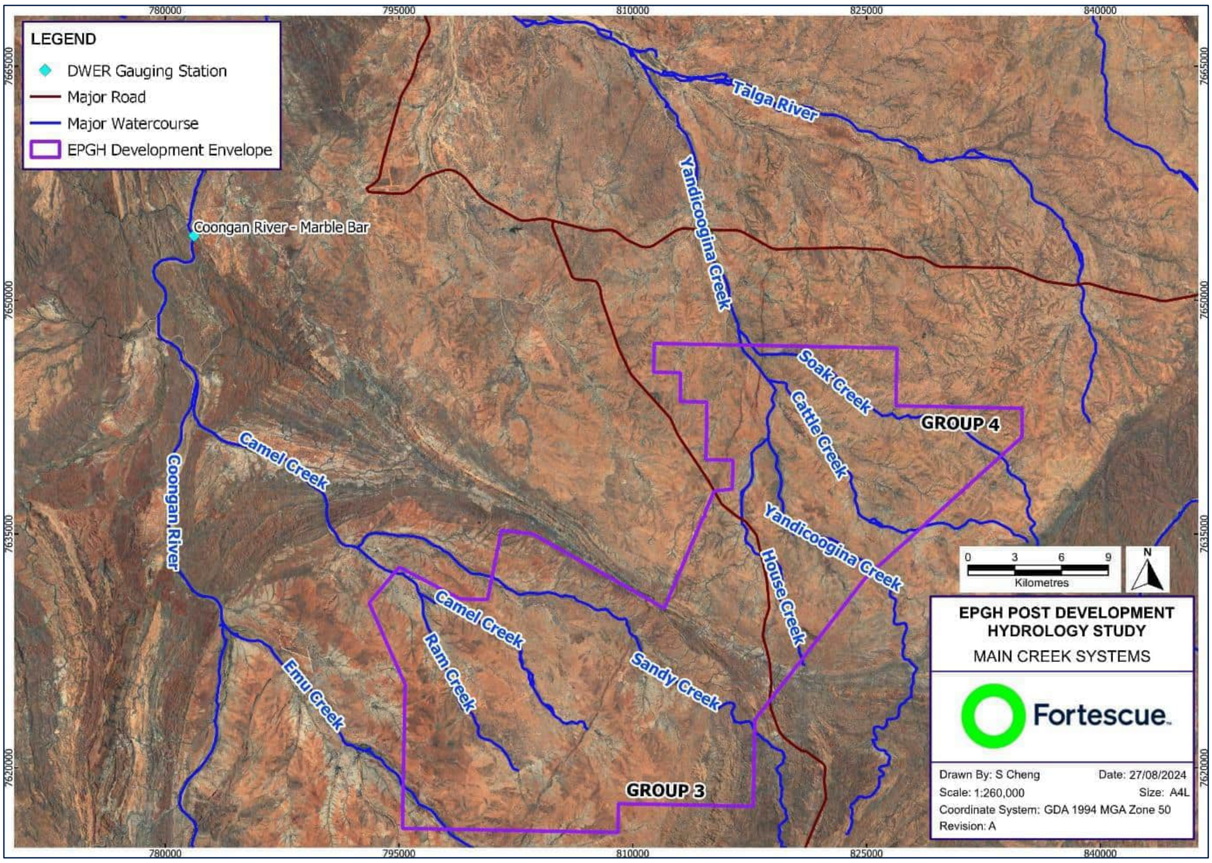


Table 3: Catchment Characteristics for Main Creek Systems within EPGH Development Area

Creek System	Catchment Area (km^2)	Mainstream Length (km)	Equal Area Slope (m/km)
Emu Creek	599	53.1	3.23
Ram Creek	73.0	18.5	3.43
Camel Creek	568	63.7	3.09
Sandy Creek	262	47.5	3.62
House Creek	109	24.0	4.31
Yandicoogina Creek	680	63.4	3.57
Cattle Creek	91.6	23.2	3.45
Soak Creek	94.7	26.9	3.04

3 PFS ENGINEERING DESIGN

3.1 Basis of Design

The PFS engineering assessment for the EPGH project includes the array design of wind turbine Group 3 and Group 4, electrical, control and telecommunications system design for the turbine groups, and civil design of turbine hardstand areas, access tracks and associated waterway crossings with the development. Design criteria, key assumptions and standards used to inform engineering design are provided in the basis of design documents as 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 crossings of major watercourses and flow paths. However, in some instances this is unavoidable to provide connectivity to turbines within the development. The following assumptions are considered to minimise social/environmental impacts:

- Track alignment and crossing design to utilise existing cleared tracks 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 EPGH is outlined in Key Decision Note AUSS0003-0000-PM-KDN-0001. The crossings include a combination of floodways and culverts designed to be trafficable for storm events up to the 50 % (1 in 2) AEP.

3.2 Waterway Crossing Design

The PFS civil design, including waterway crossing design, for Group 3 and Group 4 was completed and detailed in the civil engineering assessment report AUSS0003-0000-CI-REP-0001. The design footprints and crossing locations for Group 3 and 4 are presented in Figure 5 and Figure 6, respectively.

Total 28 crossing locations were identified for EPGH with 14 located in Group 3 and 14 in Group 4. 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 and design details of the crossings are provided in Table 5 and Table 6.

Six southernmost turbines located west of House Creek in Group 4 (indicated in grey in Figure 6), along with their associated footprint and crossings (G4_C11 and G4_C12), were included in the modelling but will not proceed to development.

Figure 5: PFS Design Footprint for EPGH Group 3

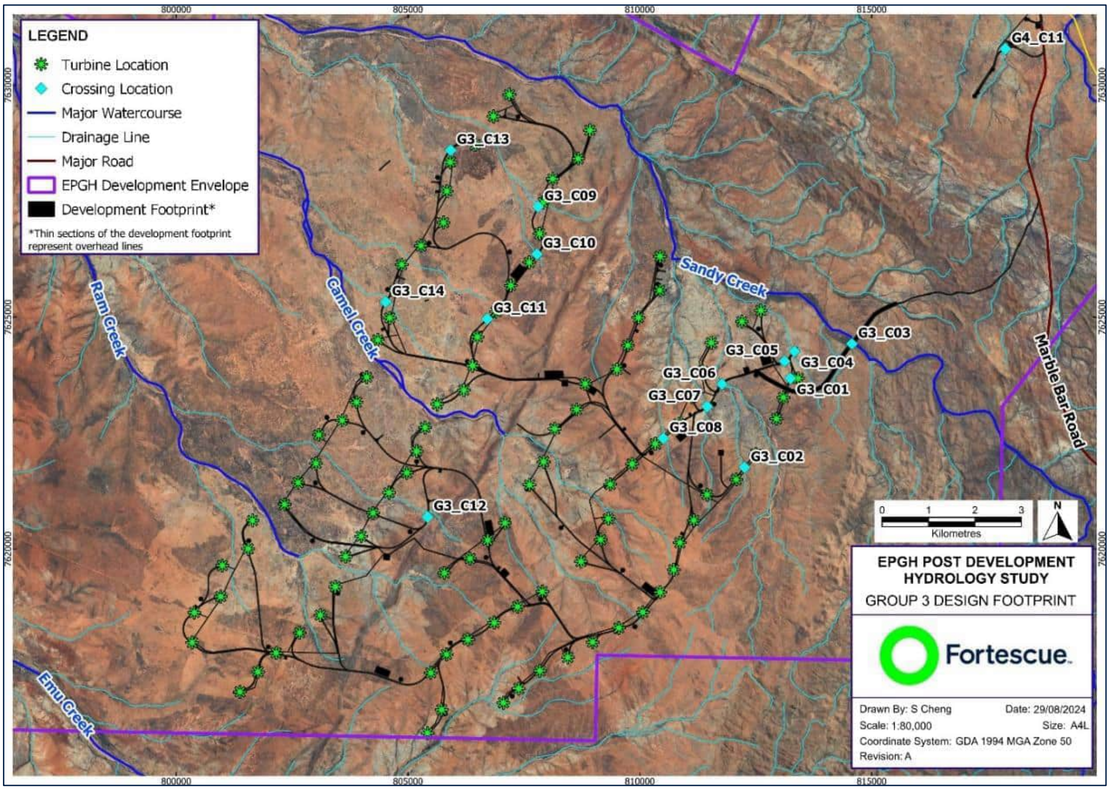


Figure 6: PFS Design Footprint for EPGH Group 4

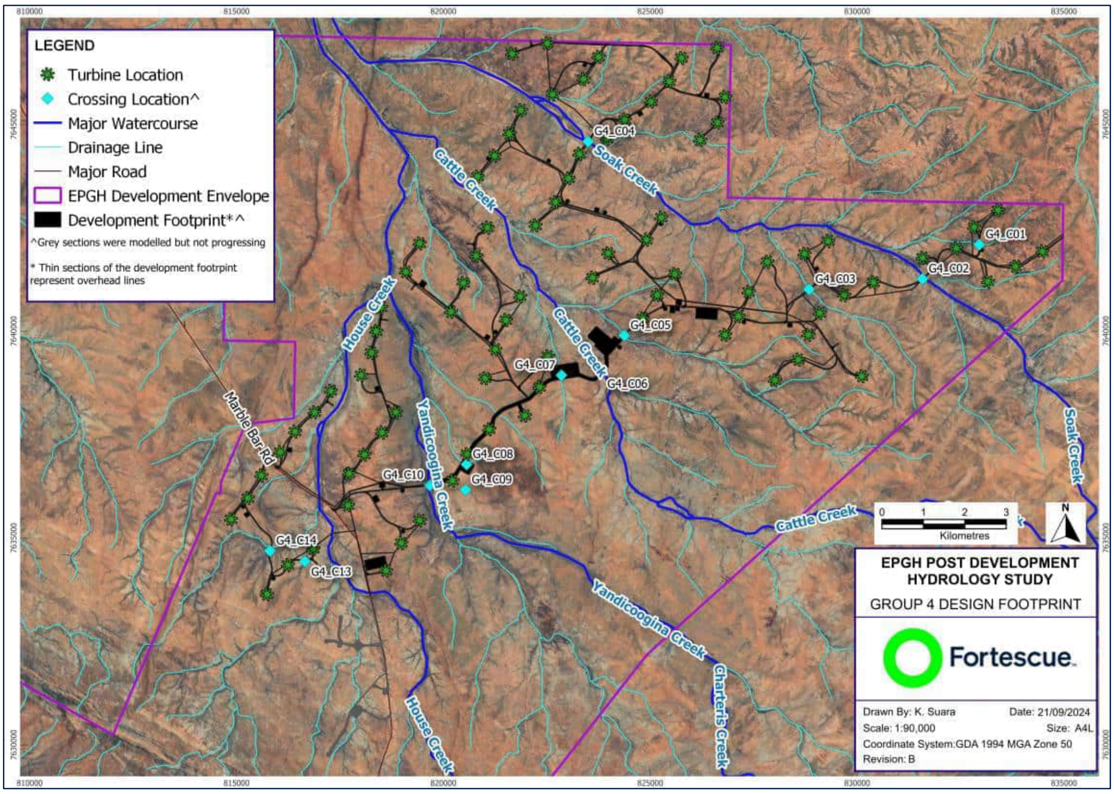


Table 4: Key Design Inputs/Parameters for Culverts and Floodway Assessment

Input/Parameter	Value	Comments/Reference
Design case	50 % AEP trafficable	East Pilbara Creek Crossing Design Standard Review (AUSS0003-0000-PM-KDN-0001) One of the following two options is adopted for design based on the size of the contributing catchment and magnitude of the design peak flow rate: 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.
Minimum culvert size	450 mm	Fortescue Civil Engineering Requirements (FFI-0000-CI-SOR-0001)
Minimum culvert cover	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: Waterway Crossing Design for EPGH Group 3

Parameter	C01	C02	C03_a(1)	C03_b(1)	C04	C05	C06	C07	C08	C09	C10	C11	C12	C13	C14
Lat	-21.46	-21.48	-21.46	-21.46	-21.46	-21.46	-21.46	-21.47	-21.47	-21.43	-21.44	-21.45	-21.49	-21.42	-21.45
Long	120.02	120.01	120.03	120.04	120.02	120.02	120.01	120.01	120.00	119.97	119.97	119.96	119.95	119.95	119.94
Road design level (m AHD)	316.16	320.86	316.99		312.75	313.74	312.25	315.42	318.54	285.92	290.95	296.15	307.05	278.87	285.54
Design case	50 % AEP														
Catchment area (km^2)	0.65	0.96	101.00		0.57	0.92	14.53	1.77	1.61	2.07	2.15	1.88	1.73	2.41	1.76
Peak flow (m^3/s)	3.48	3.94	81.69		2.97	4.27	21.52	5.68	5.93	6.60	7.68	6.72	5.58	7.34	5.19
Crossing type	Low-level floodway		Floodway with culverts		Low-level Floodway		Floodway with culverts			Low-level floodway					
Inlet elevation (m AHD)	-	-	314.52	314.49	-	-	310.60	314.39	317.48	-	-	-	-	-	-
Outlet elevation (m AHD)	-	-	314.46	314.30	-	-	310.55	314.27	317.26	-	-	-	-	-	-
Culvert type	-	-	CSP	CSP	-	-	CSP	CSP	CSP	-	-	-	-	-	-
Culvert size (mm)	-	-	1800	1800	-	-	1050	450	450	-	-	-	-	-	-
No. of barrels	-	-	5	5	-	-	4	2	3	-	-	-	-	-	-
Pipe length (m)	-	-	26.20	26.86	-	-	20.80	15.06	22.09	-	-	-	-	-	-

Note:

(1) C03 represents the crossing location at Sandy Creek. This location is recommended by the Nyamal people and is located along a braided section of Sandy Creek. Two sets of culverts are required to manage multiple flow paths.

Table 6: Waterway Crossing Design for EPGH Group 4

Parameter	C01	C02	C03_a(1)	C03_b(1)	C04	C05	C06	C07	C08	C09	C10_a(2)	C10_b(2)	C11	C12	C13	C14
Lat	-21.29	-21.30	-21.30	-21.30	-21.27	-21.31	-21.32	-21.33	-21.34	-21.35	-21.35	-21.35	-21.40	-21.38	-21.36	-21.36
Long	120.21	120.20	120.17	120.17	120.12	120.13	120.12	120.11	120.09	120.09	120.08	120.08	120.07	120.07	120.05	120.05
Road design level (m AHD)	293.80	288.56	281.46	281.46	256.22	269.16	268.20	268.65	272.07	271.50	267.99	267.99	292.41	282.34	275.70	277.28
Design case	50 % AEP															
Catchment area (km²)	2.10	21.17	6.83		71.66	15.00	54.20	1.43	0.83	0.78	173.06		2.17	11.57	1.01	5.98
Peak flow (m³/s)	7.98	39.68	14.82		59.82	19.30	47.51	5.36	3.66	3.66	79.03		7.52	17.31	4.50	10.62
Crossing type	Low-level floodway	Floodway with culverts						Low-level floodway			Floodway with culverts				Low-level floodway	Floodway with culverts
Inlet elevation (m AHD)	-	286.47	280.26	280.17	253.99	267.53	266.40	-	-	-	265.55	266.15	291.20	280.69	-	276.10
Outlet elevation (m AHD)	-	286.29	280.08	280.08	253.93	267.44	266.35	-	-	-	265.48	266.09	291.07	280.64	-	276.05
Culvert type	-	CSP	CSP	CSP	CSP	CSP	CSP	-	-	-	CSP	CSP	CSP	CSP	-	CSP
Culvert size (mm)	-	1500	600	600	1650	1050	1200	-	-	-	1800	1200	600	1050	-	600
No. of barrels	-	4	3	2	8	3	12	-	-	-	6	4	2	3	-	4
Pipe length (m)	-	24.22	16.89	17.44	24.35	19.70	21.05	-	-	-	27.26	21.69	15.69	19.73	-	19.28

Note:

(1) C03 is located along a braided section of a minor watercourse. Hence two sets of culverts are required to manage multiple flow paths.

(2) C10 represents the crossing location at Yandicoogina 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 EPGH 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 for the respective Group 3 and Group 4 areas 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 (AUS0311-0000-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 Group 3 and Group 4 areas, with associated temporal pattern ensembles for the Rangelands region (where EPGH 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 km^2, 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 Group 3 and Group 4 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 7 and Table 8.

Table 7: EPGH Group 3 Point IFD

Duration (hour)	Rainfall Depth (mm)					
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP
1	28.1	39.4	47.0	54.5	64.4	71.9
2	34.5	48.9	58.8	68.7	82.1	92.5
3	38.4	55.2	67.0	78.9	95.1	108
4.5	42.7	62.4	76.5	90.9	111	127
6	46.1	68.2	84.3	101	124	142
9	51.3	77.4	96.6	117	145	167
12	55.4	84.7	106	129	161	187

Duration (hour)	Rainfall Depth (mm)					
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP
18	28.1	39.4	47.0	54.5	64.4	71.9

Note: Location where IFD data were extracted: Lat -21.47; Long 119.98

Table 8: EPGH Group 4 Point IFD

Duration (hour)	Rainfall Depth (mm)					
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP
1	28.3	39.7	47.4	55.1	65.2	72.9
2	34.2	48.5	58.4	68.3	81.7	92.1
3	37.8	54.3	65.9	77.6	93.6	106
4.5	41.7	60.9	74.6	88.6	108	124
6	44.8	66.3	81.8	97.8	120	138
9	49.7	74.9	93.4	113	140	162
12	53.6	81.9	103	125	155	180
18	28.3	39.7	47.4	55.1	65.2	72.9

Note: Location where IFD data were extracted: Lat -21.47; Long 119.98

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 9.

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 9 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 9: Design Events Assessed in TUFLOW

Storm Detail	Events Assessed
Annual Exceedance Probabilities (AEPs)	1%, 2%, 5%, 10%, 20% and 50%
Design Storm Durations (hour)	1, 2, 3, 4.5, 6, 9, 12 and 18
Temporal Patterns (TPs)	10 TPs for each storm duration

4.3 Model Development

Two-dimensional (2D) TUFLOW models were used to assess the surface water flow characteristics for the Group 3 and Group 4 areas under baseline and post development conditions[2]. 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 10.

[2] The baseline models were developed using an older version (2020-10-AC) of TUFLOW and the post development utilised a more up-to-date version (2023-03-AE). For this assessment the baseline models were re-run using the newer version of TUFLOW to ensure accurate comparison of pre- and post-development model results.

Figure 7: Baseline Model Configuration

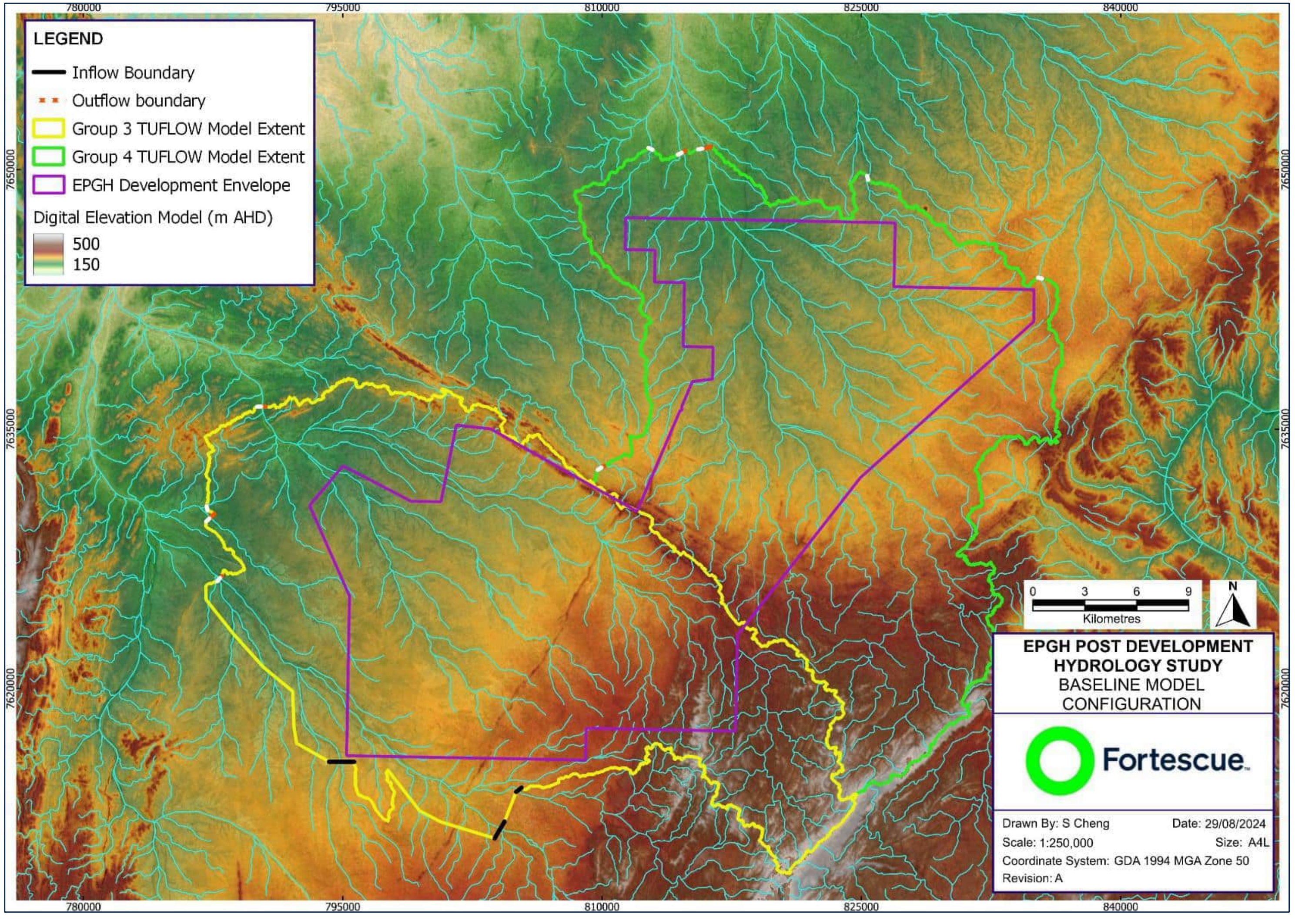


Figure 8: Post Development Model Configuration

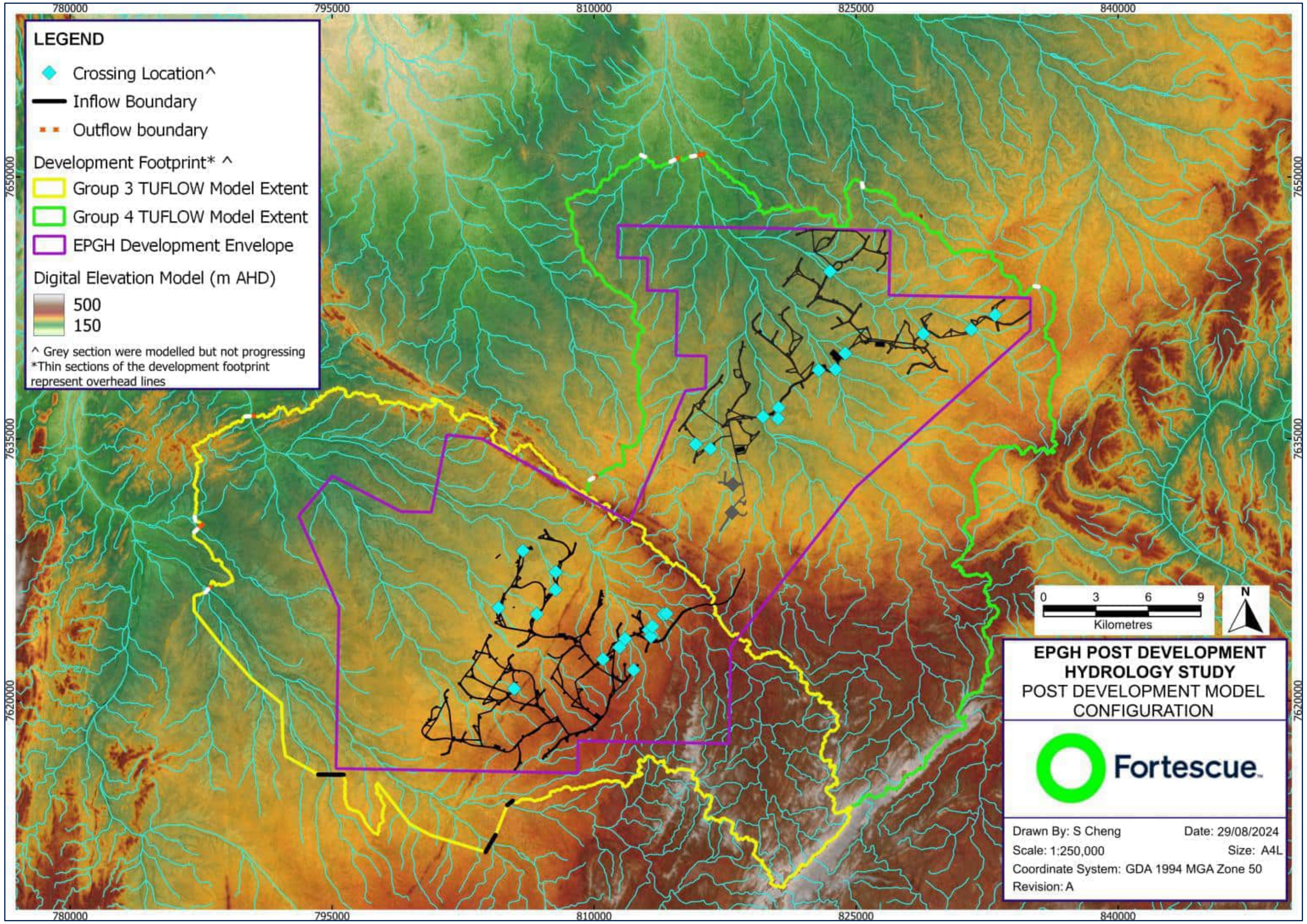


Table 10: TUFLOW Model Input Data and Assumptions

<table>
<tr><th rowspan="2">Model Input/Parameter</th><th colspan="2">Baseline</th><th colspan="2">Post Development</th></tr>
<tr><th>Group 3</th><th>Group 4</th><th>Group 3</th><th>Group 4</th></tr>
<tr><td colspan="5">Rainfall</td></tr>
<tr><td>Rainfall depths</td><td colspan="2">Extracted from BoM Design Rainfall Data System (BoM, 2016) (see Section 4.2).</td><td colspan="2">No change from baseline</td></tr>
<tr><td>Rainfall losses</td><td colspan="2">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 surrounding gauged watercourses, i.e., Coongan River at Marble Bar (DWER Ref: 710204) and Nullagine River at Nullagine (DWER Ref: 710004). RORB rainfall-runoff models were developed for the gauged catchments 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 (AUS0311-0000-HG-REP-0001).

The final rainfall loss parameters adopted for modelling are:
ILs = 20 mm
CL = 4.3 mm/hr</td><td colspan="2">No change from baseline for natural undisturbed area:
ILs = 20 mm
CL = 4.3 mm/hr

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 = 1.3 mm/hr</td></tr>
<tr><td>Pre-burst</td><td colspan="2">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 (ILB). Where pre-burst exceeded ILs, an ILB of zero was adopted for simulation. The pre-burst depths used in the modelling are provided in Table 11 and Table 12.</td><td colspan="2">No change from baseline</td></tr>
<tr><td>Pre-wetting catchment</td><td colspan="2">10 mm of rainfall (no losses adopted) simulated over a long simulation period with the final timestep read in as an initial water level raster for the design event runs. The purpose of this is to fill the observed micro storages in the Digital Elevation Model (DEM) that were considered to be a result of photogrammetry noise.</td><td colspan="2">No change from baseline</td></tr>
</table>

Model Input/Parameter	Baseline		Post Development	
	Group 3	Group 4	Group 3	Group 4
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	
Areal Reduction Factor (ARF)	Determined using parameters extracted from the ARR Datahub (Babister et al., 2016) for the Northern Coastal region and based on catchment areas within model domains.		No change from baseline	
Terrain				
Model terrain	This forms the basis of the hydraulic model and was developed based on 1 m photogrammetrically derived DEM for the development area, dated February 2022.		The post development model terrain was developed based on the following datasets: • 1 m photogrammetrically derived DEM for the development area, dated February 2022 • 3D civil design for Group 3 and Group 4 incorporated in the models as 1 m resolution DEM • 3D breaklines to reinforce the road embankments at the crossing locations.	
Total model area	621 km^2	615 km^2	No change from baseline	
Grid size (Sub-grid sampling distance)	20 m (1 m) Due to the large model domain sizes, the models were simulated with a relatively coarse grid size of 20 m to optimise computational demand and run times. However, sub-grid sampling (SGS) functionality was 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	

<table>
<thead>
<tr><th rowspan="2">Model Input/Parameter</th><th colspan="2">Baseline</th><th colspan="2">Post Development</th></tr>
<tr><th>Group 3</th><th>Group 4</th><th>Group 3</th><th>Group 4</th></tr>
</thead>
<tbody>
<tr><td>Manning’s n roughness</td><td colspan="2">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:
• Typical Pilbara grasslands/main catchment areas (0.1, 0.1, 0.2, 0.04) – a depth-varying approach was adopted for these areas:
    • For flood depth ≤ 0.1m, 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 ≥ 0.2 m, a roughness of 0.04 was used.
• Medium density riparian vegetation (0.06)
• Higher density riparian vegetation (0.08)</td><td colspan="2">No change from baseline for natural undisturbed areas.
For the proposed disturbed areas, a manning’s n roughness of 0.025 was adopted.</td></tr>
<tr><td colspan="5">Boundary Conditions</td></tr>
<tr><td>Inflow boundary</td><td>The detailed topographic data coverage did not extend to the top of the Emu Creek catchment, which flows to the far southwestern corner of the Group 3 area. Hence a local RORB rainfall-runoff model was developed for the Emu Creek catchment to derive inflow boundaries for the TUFLOW model.
Hydrographs (QT) and sub-area rainfall excess (SA) were extracted from the Emu Creek RORB model and applied in TUFLOW.</td><td>N/A</td><td colspan="2">No change from baseline</td></tr>
<tr><td>Direct rainfall</td><td colspan="2">Design rainfall hyetographs, with consideration of the estimated ARFs, applied as direct rainfall over the entire 2D model domain.</td><td colspan="2">No change from baseline</td></tr>
</tbody>
</table>

Model Input/Parameter	Baseline		Post Development	
	Group 3	Group 4	Group 3	Group 4
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				
Culvert	N/A		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 Table 5 and Table 6. Parameters adopted across the structures are as follows: • Manning's n roughness (0.024) • Adopted inlet loss (0.9) • Adopted outlet loss (1)	

Table 11: EPGH Group 3 Median Pre-Burst

Duration (hours)	Rainfall Depth (mm)					
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP
1	0.7	1.1	1.4	1.7	2.3	2.8
2	0	0.2	0.3	0.4	2.2	3.6
3	0	1.0	1.7	2.4	9.3	14.5
6	0	1.5	2.5	3.4	20.7	33.6
12	0	1.0	1.7	2.4	9.0	14.0

Table 12: EPGH Group 4 Median Pre-Burst

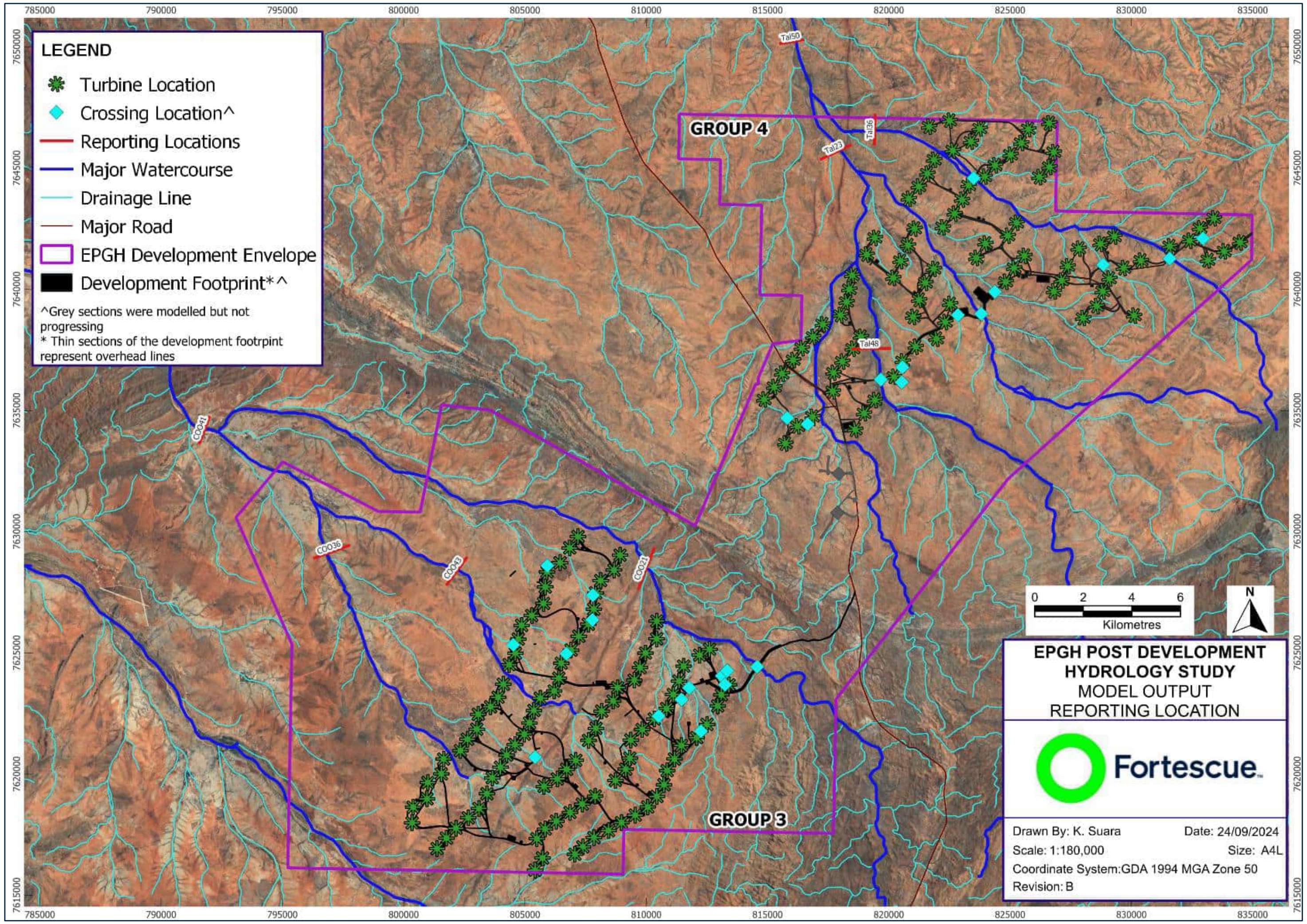
Duration (hours)	Rainfall Depth (mm)					
	50 % AEP	20 % AEP	10 % AEP	5 % AEP	2 % AEP	1% AEP
1	0.6	1.0	1.2	1.5	2.2	2.8
2	0	0.2	0.3	0.4	2.2	3.6
3	0	1.0	1.6	2.2	8.6	13.4
6	0	1.5	2.5	3.4	22.5	36.7
12	0	1.3	2.2	3.0	10.3	15.7

5 MODEL RESULTS AND DISCUSSION

Baseline and post development surface water flow conditions for the 50 % to 1 % 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 the Group 3 and Group 4 areas. It should be noted that the 50 % and 1 % AEP storms represent the design standard for the waterway crossings at EPGH and flood immunity requirement for critical infrastructure, including power generation area, for Fortescue projects, respectively (Section 3).

Both the baseline and post development model results showed that the 12-hour duration storm is the critical storm (i.e., storm duration resulting in the highest peak flow) for major watercourses, including Sandy Creek, Camel Creek and Yandicoogina Creek, that drain through EPGH. Hence model results for the 12-hour duration storm are presented in this report. Reporting locations for both Group 3 and Group 4 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 (AUS0311-0000-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 Group 3

The estimated flood peaks, total flow volume, maximum flood depth and velocity for Group 3 at COO21 (Sandy Creek), COO43 (Camel Creek), COO36 (Ram Creek) and COO41 (Camel Creek – model outlet), as indicated in Figure 9, are presented in Table 13.

Table 13: Baseline Model Results for Group 3

Reporting Location	Results	50 % AEP	10 % AEP	1 % AEP
COO21 (Sandy Creek)	Peak flow (m^3/s)	61	368	976
	Flow volume (ML)	1,235	6,879	18,950
	Maximum flood depth (m)	0.92	2.33	3.60
	Maximum velocity (m/s)	1.31	2.62	3.28
COO43 (Camel Creek)	Peak flow (m^3/s)	29	53	194
	Flow volume (ML)	614	1,770	5,182
	Maximum flood depth (m)	0.72	1.13	1.49
	Maximum velocity (m/s)	0.79	1.25	1.48
COO36 (Ram Creek)	Peak flow (m^3/s)	37	81	298
	Flow volume (ML)	969	2,849	8,397
	Maximum flood depth (m)	0.72	1.06	1.56
	Maximum velocity (m/s)	0.85	1.24	1.55
COO41 (Camel Creek – model outlet)	Peak flow (m^3/s)	133	479	1,540
	Flow volume (ML)	3,878	17,076	48,934
	Maximum flood depth (m)	1.41	2.62	3.63
	Maximum velocity (m/s)	1.24	2.21	2.78

Sandy Creek, which runs along the northern extent of Group 3, is the largest watercourse that drains through the area and is therefore predicted to have the highest hydraulic intensities within the Group 3 development area. Estimated flood peaks for the creek at COO21 approach 61 m^3/s, 368 m^3/s and 976 m^3/s for the 50 %, 10 % and 1 % AEP events, respectively. Maximum flood depths and velocities for Sandy Creek were estimated to reach 0.9 m and 1.3 m/s for the 50 % AEP event and increasing to 3.6 m and 3.3 m/s for the 1 % AEP event at the same location.

Sandy Creek is a major tributary of Camel Creek, which itself is a tributary of the Coongan River. Surface water flow conditions along Camel Creek near the model outlet, i.e., downstream of the development area, are represented by reporting location COO41 as shown in Table 13. Estimated flood peaks are in the order of 130 m^3/s, 480 m^3/s and 1,540 m^3/s for the 50 %, 10 % and 1 % AEP events, with the maximum flood depths ranging from 1.4 m to 3.6 m and flow velocities from 1.2 m/s to 2.8 m/s.

Ram Creek and the upper reaches of Camel Creek through the centre of Group 3 development area were estimated to have less intense hydraulic behaviours (typically maximum flood depths are below 2 m and flow velocities below 2 m/s) due to the smaller contributing catchment areas and hence lower peak flows.

5.1.2 Group 4

The estimated flood peaks, total flow volume, maximum flood depth and velocity for Group 4 at TAL36 (Soak Creek), TAL48 (Yandicoogina Creek), TAL23 (Yandicoogina Creek including contribution from Cattle Creek and House Creek) and TAL50 (Yandicoogina Creek – model outlet), as indicated in Figure 9, are presented in Table 14.

Table 14: Baseline Model Results for Group 4

Reporting Location	Results	50 % AEP	10 % AEP	1 % AEP
TAL36 (Soak Creek)	Peak flow (m^3/s)	40	154	392
	Flow volume (ML)	1,067	4,179	10,410
	Maximum flood depth (m)	0.77	1.36	1.83
	Maximum velocity (m/s)	0.97	1.43	1.74
TAL48 (Yandicoogina Creek)	Peak flow (m^3/s)	141	435	979
	Flow volume (ML)	2,319	8,163	20,655
	Maximum flood depth (m)	1.82	2.47	3.20
	Maximum velocity (m/s)	1.07	1.58	1.90
TAL23 (Yandicoogina Creek including contribution from Cattle Creek and House Creek)	Peak flow (m^3/s)	164	638	1,825
	Flow volume (ML)	4,822	18,514	46,885
	Maximum flood depth (m)	1.55	2.71	3.97
	Maximum velocity (m/s)	1.07	1.78	2.38
TAL50 (Yandicoogina Creek – model outlet)	Peak flow (m^3/s)	187	733	2,233
	Flow volume (ML)	6,266	24,709	62,727
	Maximum flood depth (m)	1.54	2.54	3.66
	Maximum velocity (m/s)	1.11	1.97	2.92

Yandicoogina Creek is the largest watercourse that drains through the Group 4 development area. Other smaller creeks that flow through the site, including Soak Creek, Cattle Creek and House Creek, are all tributaries of Yandicoogina Creek, which itself is a tributary of the Talga River.

Peak flows for Yandicoogina Creek at TAL48 were estimated to reach 141 m^3/s, 435 m^3/s and 979 m^3/s for the 50 %, 10 % and 1 % AEP events, with the maximum flood depths and velocities approaching 1.8 m and 1.1 m/s for the 50 % AEP event and increasing to 3.2 m and 1.9 m/s for the 1 % AEP event. Near the model outlet at TAL50, i.e., downstream of the development area, flow rates and volumes for Yandicoogina Creek increase significantly due to the larger contributing catchment area. Peak flows are shown to increase to 187 m^3/s, 733 m^3/s and 2,233 m^3/s for the 50 %, 10 % and 1 % AEP events, but this only resulted in minor increases in flood depths, and a decrease for the 50 % AEP event. This is likely due to changes in the channel geometry where the creek becomes less incised as it travels downstream. Changes in flow velocities are more noticeable, in particular for the 1 % AEP event where the maximum velocity has increased from 1.9 m/s to 2.9 m/s.

Due to their smaller catchment size and hence runoff generating potential, the smaller creeks that drain through the site were estimated to have lower hydraulic energies with the maximum 1 % AEP flood depths and velocities typically under 2.5 m and 2 m/s.

5.2 Post Development Conditions

This section provides a detailed discussion of the post development surface water flow conditions for Group 3 and Group 4.

Maximum flood depth, velocity and afflux mapping for the 50 %, 10 % and 1 % AEP events are presented in Appendix A. 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.

Although the afflux maps show localised impacts, the flood results for the area in the southernmost section of Group 4, which will not move forward in development, are expected to remain consistent with baseline conditions. Furthermore, the inclusion of this area has no influence on the post-development model results outside the immediate vicinity of the associated crossings.

The presented data in the maps has undergone a filtering process to aid in improved visualisation of key drainage features across the sites (depths of less than 20 mm not shown).

5.2.1 Group 3

A comparison of the estimated flood peaks, total flow volume, maximum flood depth and velocity for Group 3 at the identified reporting locations are presented in Table 15.

Table 15: Comparison of Baseline and Post Development Model Results for Group 3

Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
COO21 (Sandy Creek)	Peak flow (m^3/s)	60.5	62.0	2.4%	368.1	368.5	0.1%	976.2	976.8	0.1%
	Flow volume (ML)	1,235	1,245	0.8%	6,879	6,893	0.2%	18,950	18,964	0.1%

Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
	Maximum flood depth (m)	0.92	0.93	1.3%	2.33	2.33	0.0%	3.60	3.60	0.0%
	Maximum velocity (m/s)	1.31	1.33	1.2%	2.62	2.62	0.1%	3.28	3.28	0.0%
COO43 (Camel Creek)	Peak flow (m^3/s)	29.1	29.4	1.1%	52.9	53.7	1.5%	194.2	195.0	0.4%
	Flow volume (ML)	614	619	0.8%	1,770	1,784	0.8%	5,182	5,174	-0.1%
	Maximum flood depth (m)	0.72	0.73	0.4%	1.13	1.13	0.3%	1.49	1.49	0.0%
	Maximum velocity (m/s)	0.79	0.79	0.5%	1.25	1.25	0.2%	1.48	1.48	-0.1%
COO36 (Ram Creek)	Peak flow (m^3/s)	37	37.7	1.2%	81.5	83.0	1.8%	297.7	303.3	1.9%
	Flow volume (ML)	969	975	0.6%	2,849	2,877	1.0%	8,397	8,447	0.6%
	Maximum flood depth (m)	0.72	0.73	0.6%	1.06	1.07	0.6%	1.56	1.56	0.5%
	Maximum velocity (m/s)	0.85	0.85	0.5%	1.24	1.24	0.3%	1.55	1.55	0.3%
COO41 (Camel Creek – model outlet)	Peak flow (m^3/s)	133	134.8	1.6%	479.1	477.8	-0.3%	1,540	1,540	0.0%
	Flow volume (ML)	3,878	3,899	0.6%	17,076	17,139	0.4%	48,934	48,995	0.1%
	Maximum flood depth (m)	1.41	1.41	0.4%	2.62	2.63	0.4%	3.63	3.63	0.0%
	Maximum velocity (m/s)	1.24	1.24	0.2%	2.21	2.22	0.4%	2.78	2.78	0.0%

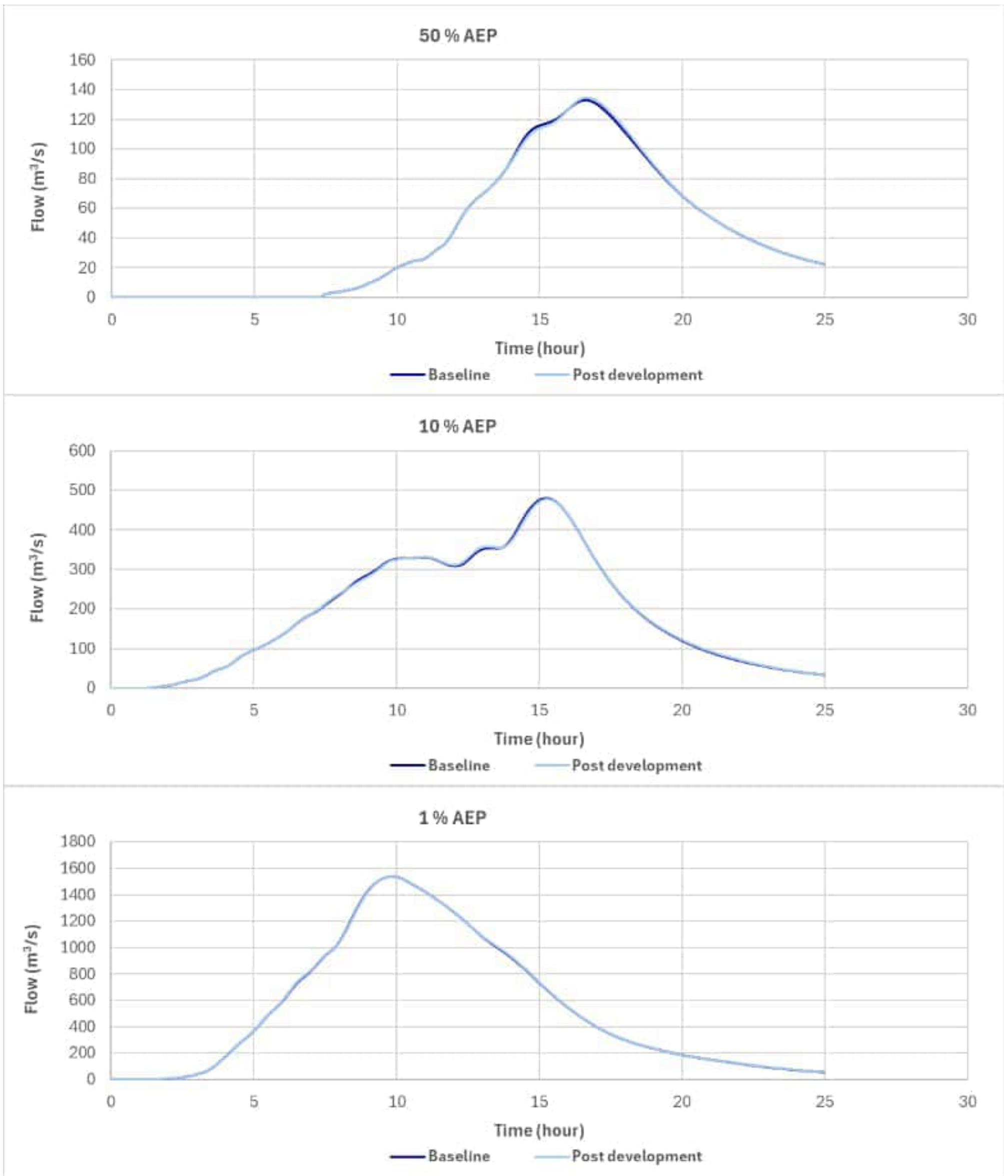
In Group 3, the proposed wind turbines and associated hardstand areas/supporting infrastructure are located outside of the estimated 1 % AEP flood extent. Hence, disruption to natural surface water flows within the catchment is expected to be negligible. This is reflected in the model results as shown Table 15 where the predicted changes in flow conditions between baseline and post development conditions are generally below 2 % for the modelled storms. The most notable change is observed at reporting location COO21 (Sandy Creek) for the 50 % AEP event where the estimated flood peak was increased by 2.4 % (i.e., 1.5 m^3/s) under post development conditions. This is translated to a 0.8 % increase in total flow volume and a 1.3 % and 1.2 % increase in maximum flood depth and velocity. However, the predicted flow regime change is shown to decrease progressively with AEP, with the 1 % AEP event showing almost no change between baseline and post development conditions.

In general, the modelling results indicate that the volume of runoff contributing to the main creek systems would increase under post development conditions. This is due to the increase in impervious areas introduced by the proposed paved/compacted areas of the development, which reduce the volume of rainfall infiltrating into the ground. However, as shown in Table 15, the predicted increase in runoff volume is small (< 1 %). This is to be expected as the project disturbance footprint (~4.89 km^2) is less than 1 % of the total Camel Creek catchment (568 km^2), and less than 0.1 % of the regional Coongan River catchment (7,090 km^2).

Given the minor changes in flow volume, the predicted differences in flood depths and velocities between baseline and post development conditions are also small (generally within 1 % as shown in Table 15). This is illustrated in the afflux mapping where the proposed changes in maximum flood depths and velocities are generally less than 5 cm and 0.1 m/s along the main creeks within the development area. Exceptions are limited to areas adjacent to the waterway crossings where more noticeable changes in depths and velocities are observed (more detailed discussion of local impacts surrounding the waterway crossings are provided below). 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.

A comparison of the flow hydrographs at reporting location COO41 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 catchment. 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 Group 3 design and layout are unlikely to modify the surface water flow quantity and quality of the main creeks contributing to the Coongan River system.

Figure 10: Comparison of Baseline and Post Development Flow Hydrographs at COO41



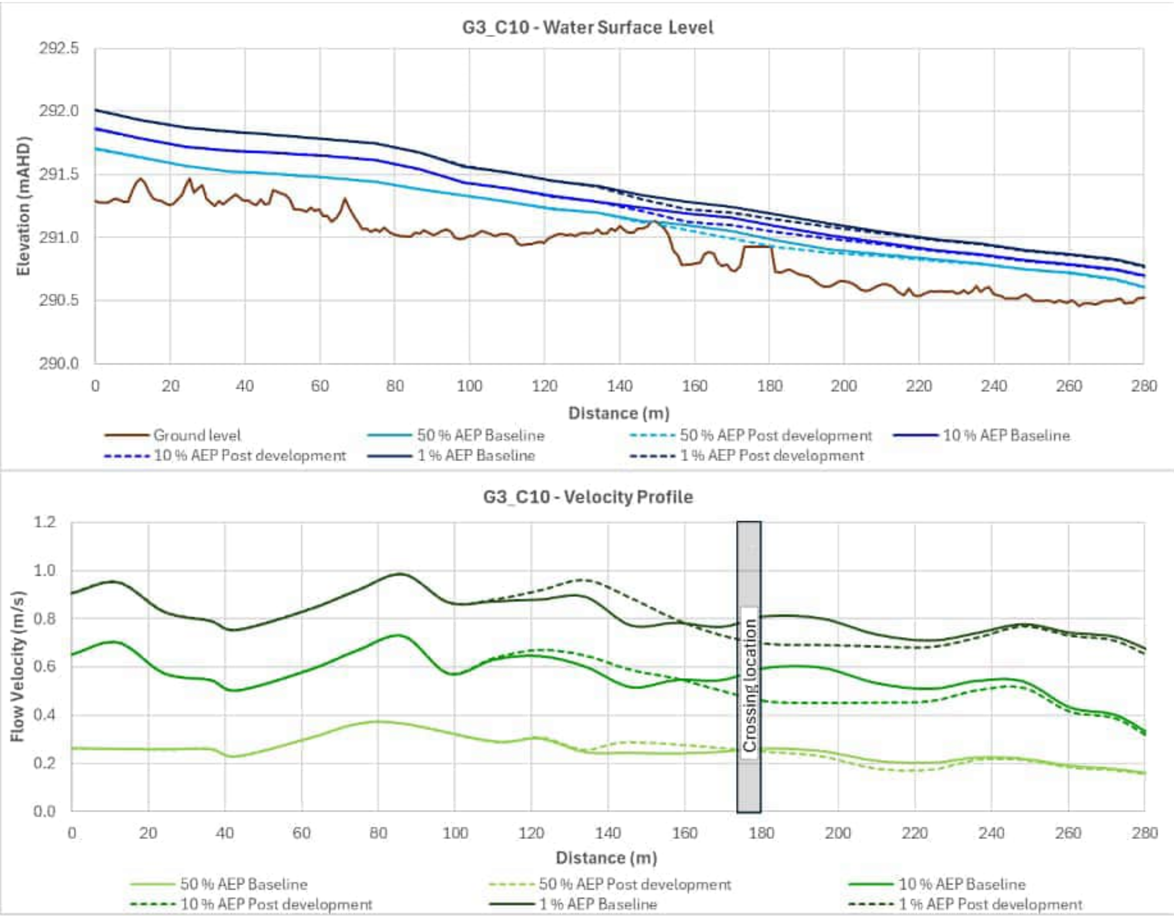
5.2.1.1 Local Impacts – Waterway Crossings

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 previously do 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 only restricted to headwater areas and are localised in regions immediately upstream and downstream of the access track.

Majority of the waterway crossings in Group 3 are minor with small contributing catchment areas (< 3 km^2) and low design peak flow rates of less than 10 m^3/s for the 50 % AEP event. All of these crossings adopt the low-level floodway design where the road elevation is set at or very close to the natural creek level, hence impacts to surface water flows are minimal.

One such example is crossing G3_C10, and a comparison of the baseline and post development water surface levels and velocity profiles at the crossing is illustrated in Figure 11. Water levels are shown to decrease slightly (~ 0.15 m) over the crossing, but these changes do not propagate more than 20 m upstream and downstream of the structure for all the modelled events. Flow velocities are shown to increase upstream but decrease downstream of the structure when compared to baseline conditions. These changes, however, are marginal (< 0.15 m/s for all the modelled events) and are restricted to areas close to the crossing (within 100 m upstream and downstream of the structure). In summary, the modelling results show that the impacts of the low-level floodway crossings are minor, with the area of influence confined to within 100 m upstream and downstream of the structures.

Figure 11: Maximum Surface Water Level and Velocity Profile at Waterway Crossing G3_C10



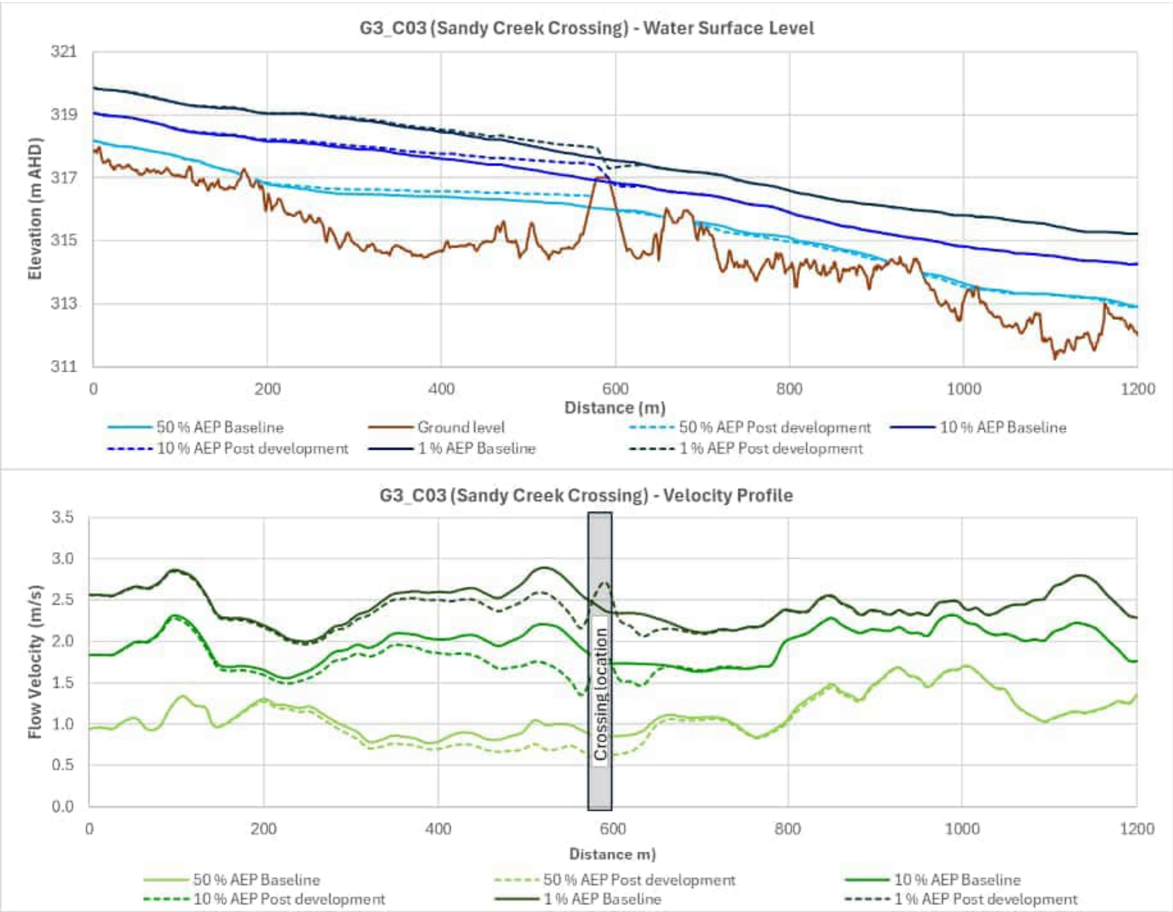
There are four waterway crossings in Group 3 that require the installation of culverts to meet design requirement and to maintain flows. The most significant is the Sandy Creek crossing (G3_C03), which connects Marble Bar Road to the development. To accommodate the culvert structures, raising of the road above the natural creek level is required, which can potentially cause water level to increase upstream (i.e., backwater effects) and decrease downstream of the crossing.

Figure 12 presents the maximum water surface levels and velocity profiles at the Sandy Creek crossing under baseline and post development conditions. Flood levels are shown to increase upstream with the build-up reaches a maximum of 0.5 m for the modelled events.

However, as shown in the plot, the effects of backwater do not propagate more than 400 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. With the increase in flood levels upstream, the accompanying flow velocities would decrease as demonstrated by the velocity plot in Figure 12. Maximum flow velocities are shown to drop by 0.3 m/s to 0.5 m/s for the modelled events with the area of influence restricted to within 400 m upstream of the crossing. Modelling results show that maximum velocities would increase as water flows over the crossing and decrease again on the downstream side of the structure before returning to baseline level approximately 100 m downstream.

This type of crossing is likely to create greater flow regime change when compared to the low-level floodway design as discussed above. However, despite the greater impacts, the effects are still 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 G3_C03 (Sandy Creek Crossing)



5.2.2 Group 4

A comparison of the estimated flood peaks, total flow volume, maximum flood depth and velocity for Group 4 at the identified reporting locations are presented in Table 16.

Table 16: Comparison of Baseline and Post Development Model Results for Group 4

Reporting Location	Results	50 % AEP			10 % AEP			1 % AEP		
		Base	Post	% diff	Base	Post	% diff	Base	Post	% diff
TAL36 (Soak Creek)	Peak flow (m^3/s)	40	36	-11.2%	154	146	-5.1%	392	386	-1.7%
	Flow volume (ML)	1,067	1,075	0.8%	4,179	4,195	0.4%	10,410	10,429	0.2%
	Maximum flood depth (m)	0.77	0.73	-4.6%	1.36	1.33	-1.7%	1.83	1.82	-0.7%
	Maximum velocity (m/s)	0.97	0.93	-3.2%	1.43	1.42	-0.9%	1.74	1.74	-0.3%
TAL48 (Yandicoogina Creek)	Peak flow (m^3/s)	141	141	0.0%	435	435	0.0%	979	979	0.0%
	Flow volume (ML)	2,319	2,322	0.1%	8,163	8,169	0.1%	20,655	20,664	0.0%
	Maximum flood depth (m)	1.82	1.82	0.0%	2.47	2.47	0.0%	3.20	3.20	0.0%
	Maximum velocity (m/s)	1.07	1.08	0.8%	1.58	1.58	0.0%	1.90	1.90	-0.1%
TAL23 (Yandicoogina Creek including contribution from Cattle Creek and House Creek)	Peak flow (m^3/s)	164	166	1.1%	638	640	0.3%	1,825	1,824	-0.1%
	Flow volume (ML)	4,822	4,833	0.2%	18,514	18,537	0.1%	46,885	46,903	0.0%
	Maximum flood depth (m)	1.55	1.56	0.4%	2.71	2.71	0.0%	3.97	3.97	0.0%
	Maximum velocity (m/s)	1.07	1.07	0.4%	1.78	1.78	0.1%	2.38	2.38	0.0%
TAL50 (Yandicoogina Creek – model outlet)	Peak flow (m^3/s)	187	180	-3.6%	733	737	0.5%	2,233	2,233	0.0%
	Flow volume (ML)	6,266	6,280	0.2%	24,709	24,744	0.1%	62,727	62,747	0.0%
	Maximum flood depth (m)	1.54	1.53	-0.8%	2.54	2.54	0.1%	3.66	3.66	0.0%
	Maximum velocity (m/s)	1.11	1.11	-0.4%	1.97	1.97	0.1%	2.92	2.92	0.0%

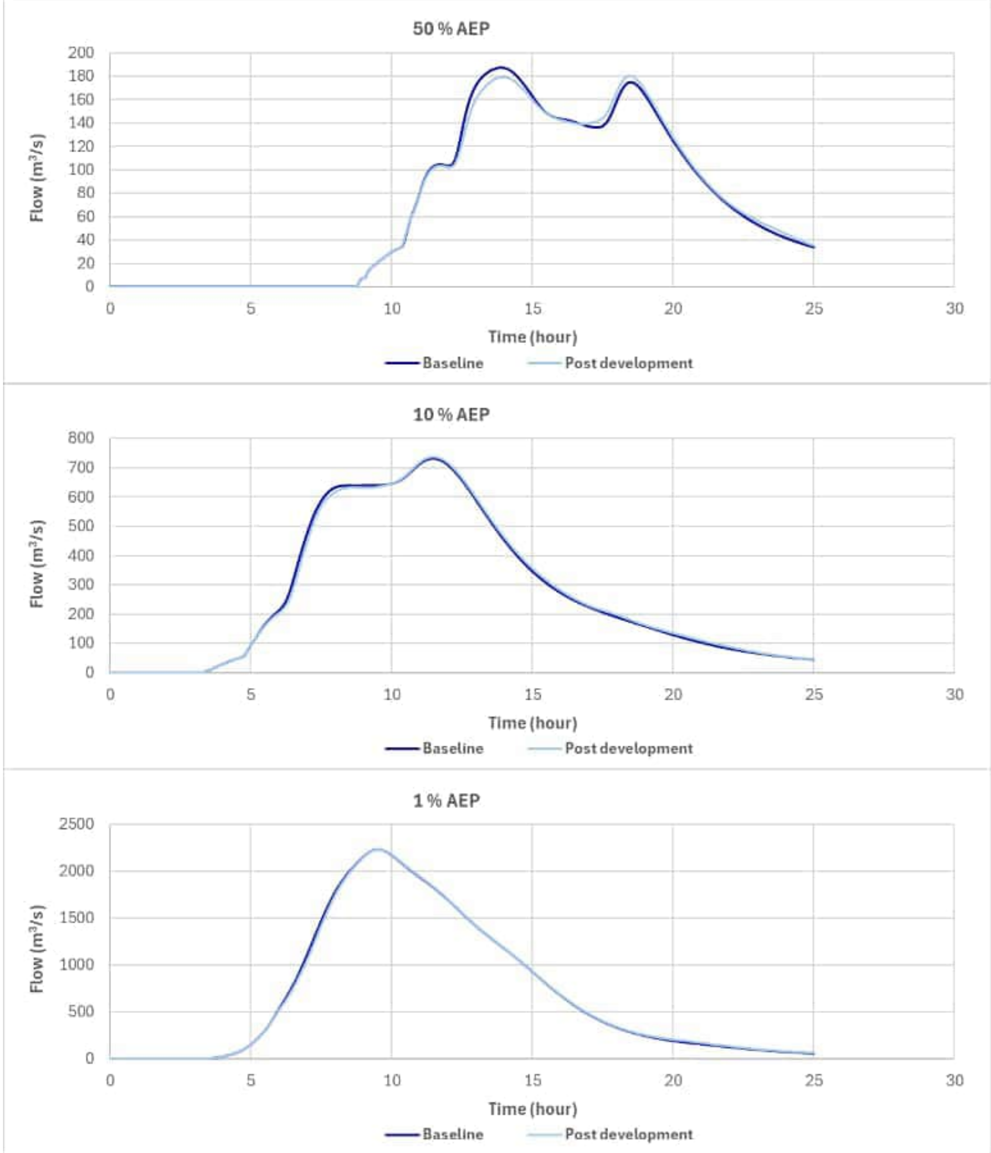
For Group 4, the development area is located adjacent to main creek systems hence the design/positioning of the wind turbines (all wind turbines and associated hardstand areas are located outside of the estimated 1 % AEP flood extents for the main creeks), access track and waterway crossings were carried out to ensure minimal disruption to surface water flows within the catchment.

In general, the modelling results show that the impacts to surface water flows are low, for the 10 % and 1 % AEP storms. For the 50 % AEP event, the predicted changes are slightly higher, most notably in Soak Creek at reporting location TAL36 mainly due to the presence of several waterway crossings with culverts in the upstream catchment. This is unavoidable to provide connection to the turbines in the eastern and north-eastern end of the development area. These crossings, due to the raised road section, have the potential to cause backwater effects, which can subsequently dampen flow peaks. This is shown at TAL36, where the peak flow rates were reduced under post development conditions. However, the effects would decrease progressively with AEP, where less than 2 % change in flood peak is observed for the 1 % AEP storm. It is also important to note that despite the reduction in flow peaks, the overall effects on the total flow volume are minimal (< 1 % change). This suggests the volume of flow travelling downstream would be unaffected by the placement of crossings along the creek.

Similar to Group 3, the modelling results indicate that the volume of runoff contributing to the main creek systems would increase under post development conditions due to reduction in rainfall infiltration caused by the proposed paved/compacted areas. However, the predicted increase is minor (< 1 %) due to the small size of the development (~4.76 km^2) compared to the catchment of Yandicoogina Creek (680 km^2).

As shown in Table 16 and 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 main creeks. The only exceptions, again, are at the waterway crossings where localised modifications in depths and velocities are observed. Downstream of the development at TAL50, the proposed flow regime changes are minimal (Figure 13, Table 16 and afflux mapping in Appendix A), 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 TAL50

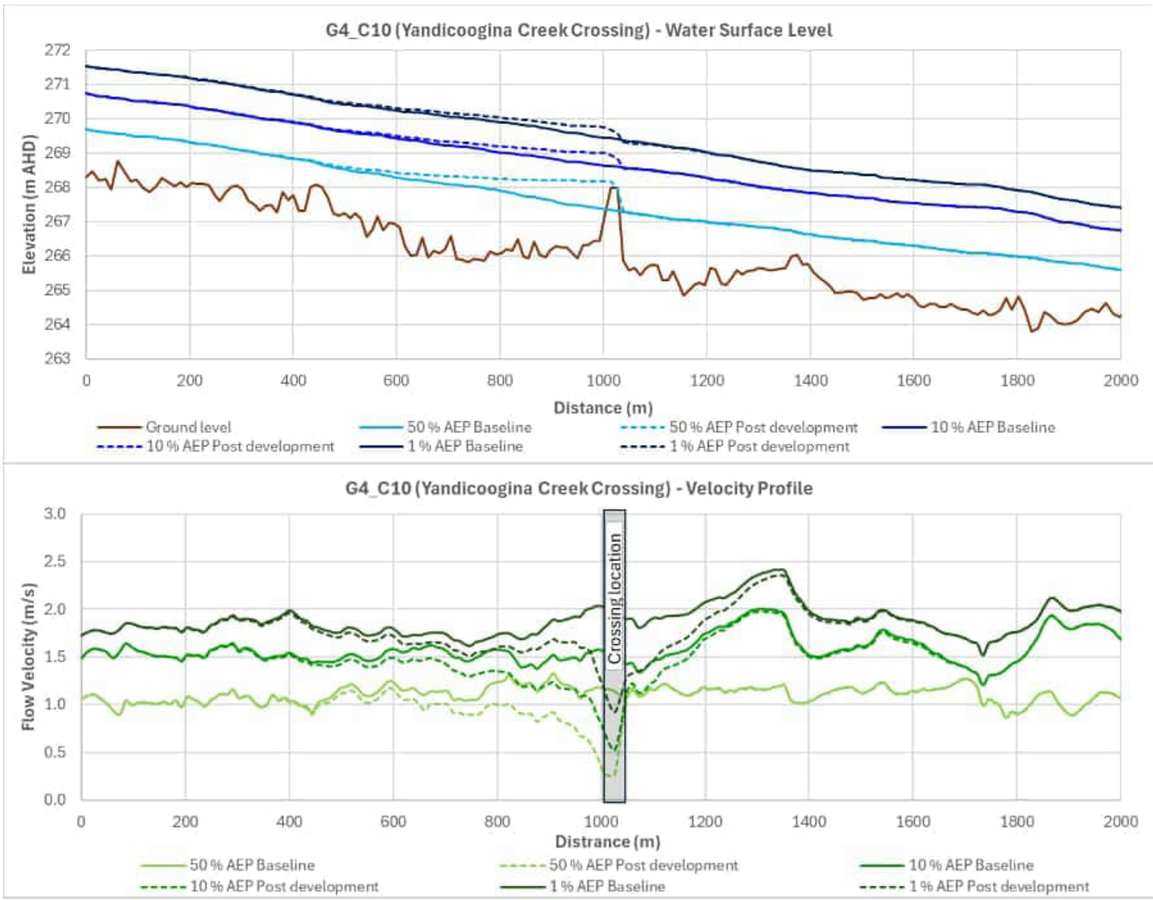


5.2.2.1 Local Impacts – Waterway Crossings

Similar to the findings for Group 3, the changes in wet/dry areas in Group 4 are localised and limited to areas surrounding the turbine access track. Hence significant impacts to major flow paths downstream of the development are not expected. However, unlike Group 3, majority of the waterway crossings in Group 4 require the installation of culverts to meet the design standard. The reason for this lies in the development layout and the positioning of the infrastructure within the catchment, which necessitate the crossing of larger creek systems with bigger contributing catchments (generally > 5 km^2) and larger design peak flows (> 10 m^3/s) for the 50 % AEP event.

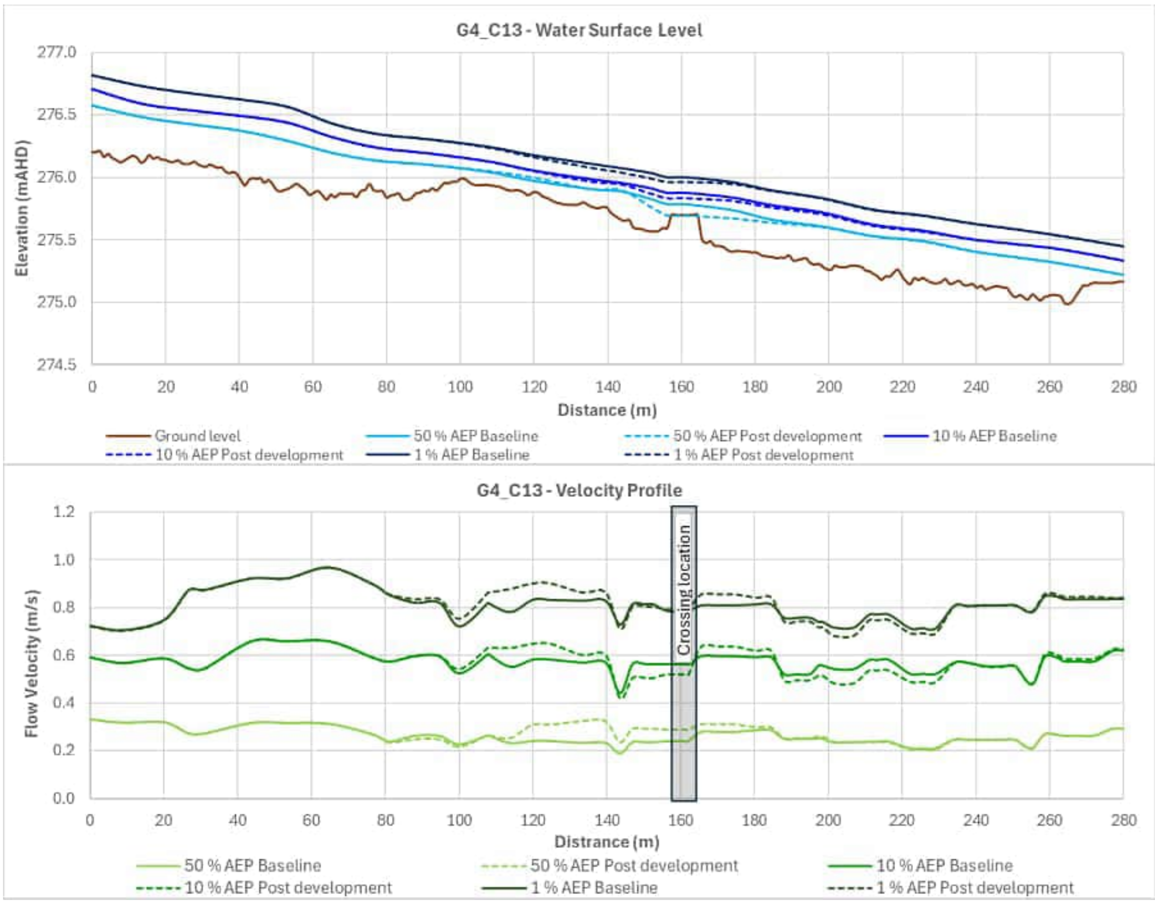
The largest waterway crossing in Group 4 is the Yandicoogina Creek crossing at G4_C10. Figure 14 provides a comparison of the maximum water surface levels and velocity profiles at the Yandicoogina Creek crossing under baseline and post development conditions. The estimated water level changes upstream of the crossing are in the order of 0.3 m to 0.8 m from the baseline, with the backwater effects extending for approximately 600 m upstream for all the modelled events. Water level changes are minimal downstream of the structure, which suggests impacts to downstream flow regime are likely to be negligible. Maximum velocities are shown to decrease by up to 1 m/s immediately upstream of the crossing, with the area of influence extending for approximately 600 m upgradient 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 400 m downstream of the structure.

Figure 14: Maximum Surface Water Level and Velocity Profile at Waterway Crossing G4_C10 (Yandicoogina Creek Crossing)



There are five minor waterway crossings in Group 4, all of which adopted the low-level floodway design. One such example is crossing G4_C13, and a comparison of the baseline and post development water surface levels and velocity profiles at the crossing is provided in Figure 15. Water levels are shown to decline by up to 0.1 m over the crossing with the zone of influence confined to within 40 m upstream and downstream of the structure. Velocity changes around the crossing are minor (within ± 0.1 m/s from baseline) with the effects extending no more than 60 m upstream/downstream of the structure.

Figure 15: Maximum Surface Water Level and Velocity Profile at Waterway Crossing G4_C13



In summary, the modelling results show that the impacts of waterway crossings in Group 4 are larger than that in Group 3 due to the presence of more significant structures to facilitate the project layout and design. However, the effects are predominately local and do not extend beyond the proximity of the crossings to impact on downstream flow regime.

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. Comparison of modelling of baseline and proposed 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 change are confined to isolated areas along the turbine access track/waterway crossings, while disruptions to regional surface water flows, downstream of the development area, are negligible.

Changes in total flow volume near the model outlet (for both Group 3 and Group 4) are less than 1 %, indicating the quantity of flow contributing downstream to the regional Coongan River system is unlikely to be affected by the development. There is a slight dampening of flood peaks in Group 4 due to the presence of more significant crossing structures causing backwater effects, but these reductions in peak flow rates do not translate to reductions in total flow volume. Proposed changes in maximum flood depths downstream of the project disturbance footprint are marginal (< 1 % for both Group 3 and Group 4) and the flood extents for the modelled storms remain unaffected by the development. The minor changes in flow velocities near the model outlet (also < 1 % for both Group 3 and Group 4) are not expected to alter the existing geomorphic regime of the receiving watercourses, including those contributing downstream to the regional Coongan River system.

Turbine access tracks and waterway crossings have the potential to cause local level impact on creek hydrology and morphology. Two different types of waterway crossings (low-level floodway and floodway crossing with culverts) were proposed and assessed as part of this assessment. Modelling results show that the impacts of the low-level floodway crossings are minimal (flood depth and velocity changes are within ± 0.15 m and ± 0.15 m/s over the crossings), with the area of influence generally confined to within 100 m upstream and downstream of the structures.

Waterway crossings with culverts have the potential to cause greater flow regime changes, due to flow constrictions and obstructions caused by the culvert structures and road embankment. Backwater development is expected, but the effects are unlikely to extend more than 600 m upstream of the structures. Modifications to flow velocities are also expected as flows accelerate over the floodway and attenuate when ponding occurs upstream of the road. However, these changes are localised and unlikely to extend beyond the proximity of the crossings to impact on regional hydrologic regime.

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

6.2 Geomorphology

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 17.

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 17, 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 creek.

It was demonstrated by the modelling results that velocity changes (± 0.15 m/s) caused by the low-level floodway crossings are too small to alter the existing sediment mobilisation regime, indicating this type of crossing is unlikely to impact on the natural geomorphological characteristics of the creek systems. Velocity changes associated with the culvert crossings are more noticeable, particular on the upstream side of the structures where velocities could decrease by up to 1 m/s due to backwater development. This may encourage sediment deposition upstream of the structures. Velocity increases over and immediately downstream of the crossings are relatively minor (within + 0.5 m/s), hence significant increase in erosion or scouring is not expected as a result of the waterway crossings.

Table 17: 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 EPGH project on existing surface water flow regime. The assessment was built upon the baseline assessment works undertaken in 2022 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 development are localised and limited to areas in proximity to the turbine access track/waterway crossings. Disruptions to regional surface water flows, downstream of the development area, are considered negligible for all the modelled events.
- The volume of runoff contributing to the main creek systems would increase under post development conditions due to changes in land use which reduces the volume of rainfall infiltrating into the ground. The magnitude of change is small (< 1%) due to the small size of the development area compared to the contributing catchments of the creek systems.
- There is a slight dampening of flood peaks caused by the waterway crossings in Group 4 but the reductions in peak flow rates do not translate to reductions in total flow volume.

- Proposed changes in maximum flood depths and velocities downstream of the project disturbance footprint are marginal (< 1 % for both Group 3 and Group 4) indicating the development is unlikely to modify the hydrology and morphology of the downstream watercourses, including the Coongan River.
- Impacts of the low-level floodway crossings on creek hydrology and morphology are low. Waterway crossings with culverts 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. Significant increase in erosion, however, is not expected as velocity increases caused by the crossings are relatively minor.
- Overall, the impacts of the project design and layout on the existing surface water flow regime are 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 creeks and rivers within the development area.

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., & Testioni, I. (2016). Australian Rainfall and Runoff: A Guide to Flood Estimation. Commonweath of Australia (Geoscience Australia).

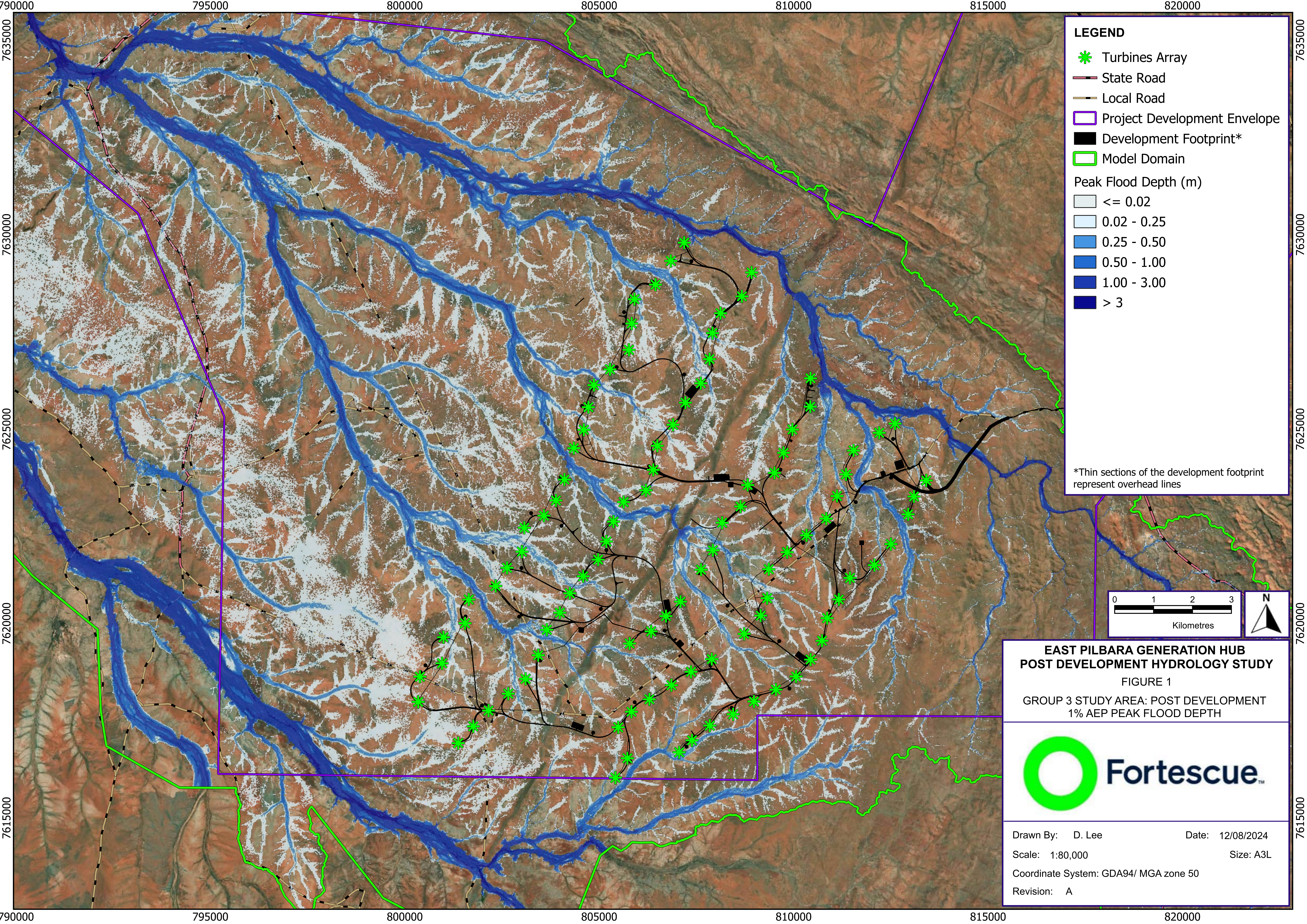
[3] B oM. (2016). Design Rainfall Data System. Retrieved from http://www.bom.gov.au/water/designRainfalls/revised-ifd/.

[4] Subramanya, K. (2009). Flow in Open Channels. New Delhi: Tata McGraw-Hill.

DOCUMENT CONTROL

East Pilbara Generation Hub - Post Development Hydrology Study		
Status	IFU - Issued for Use	30-Sep-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 FLOOD MAPPING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State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Peak Flood Depth (m)
<= 0.02
0.02 - 0.25
0.25 - 0.50
0.50 - 1.00
1.00 - 3.00
> 3
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
790000
795000
800000
805000
810000
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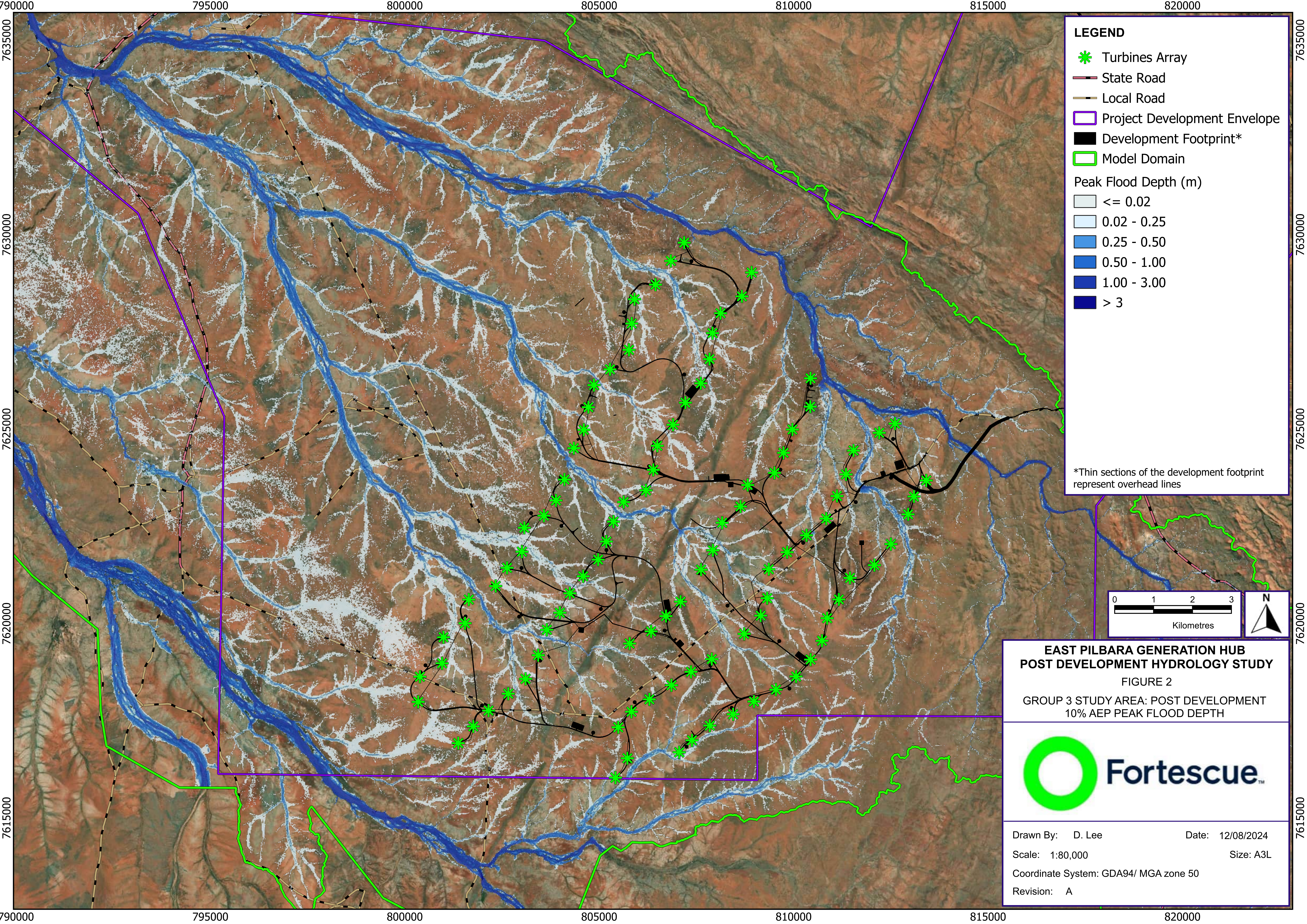
7635000
7630000
7625000
7620000
7615000
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 1
GROUP 3 STUDY AREA: POST DEVELOPMENT
1% AEP PEAK FLOOD DEPTH
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A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LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Peak Flood Depth (m)
<= 0.02
0.02 - 0.25
0.25 - 0.50
0.50 - 1.00
1.00 - 3.00
> 3
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 2
GROUP 3 STUDY AREA: POST DEVELOPMENT
10% AEP PEAK FLOOD DEPTH
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
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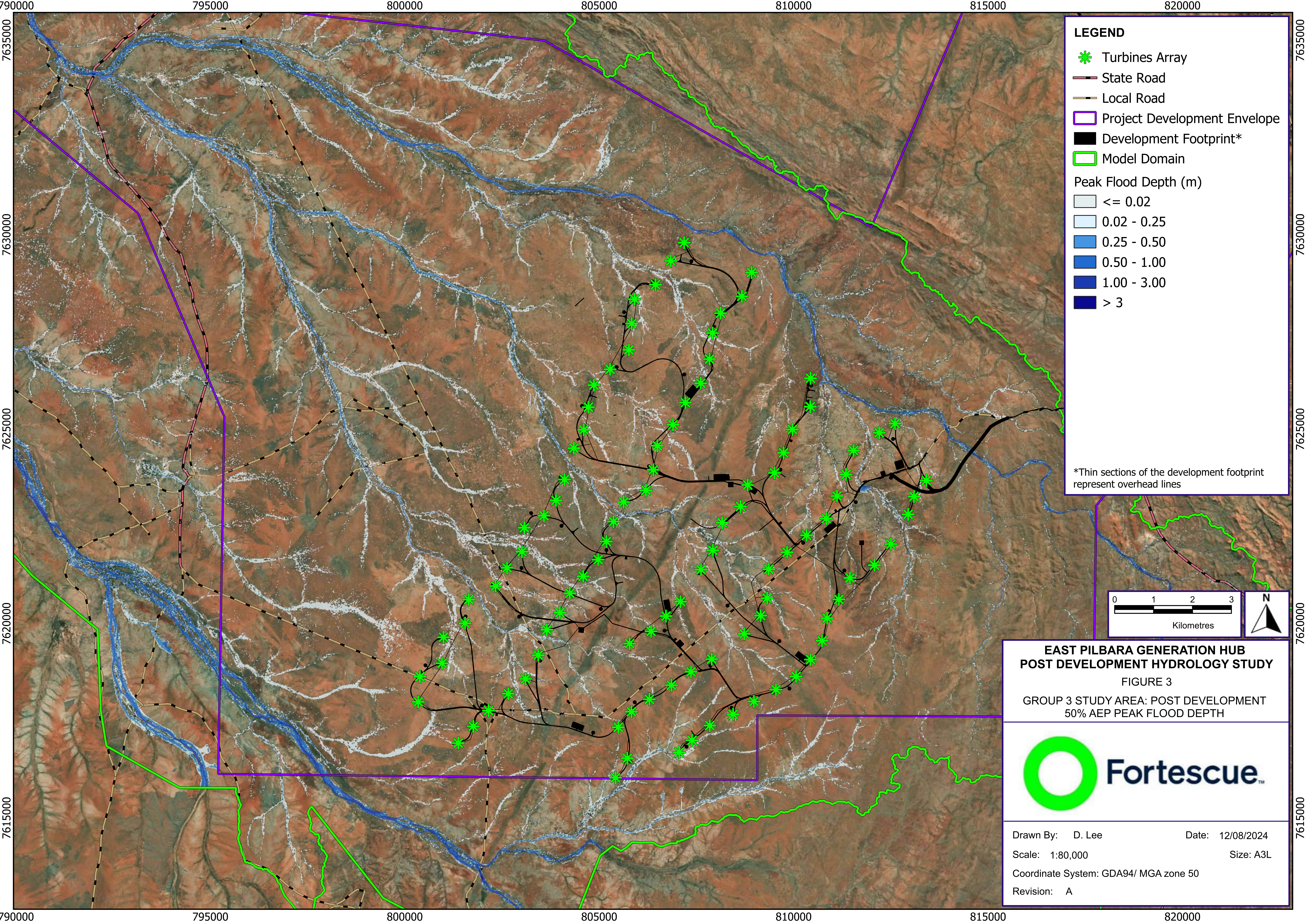
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Local Road
Project Development Envelope
Development Footprint*
Model Domain
Peak Flood Depth (m)
<= 0.02
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1.00 - 3.00
> 3
*Thin sections of the development footprint represent overhead lines
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Kilometres
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EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 3
GROUP 3 STUDY AREA: POST DEVELOPMENT
50% AEP PEAK FLOOD DEPTH
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
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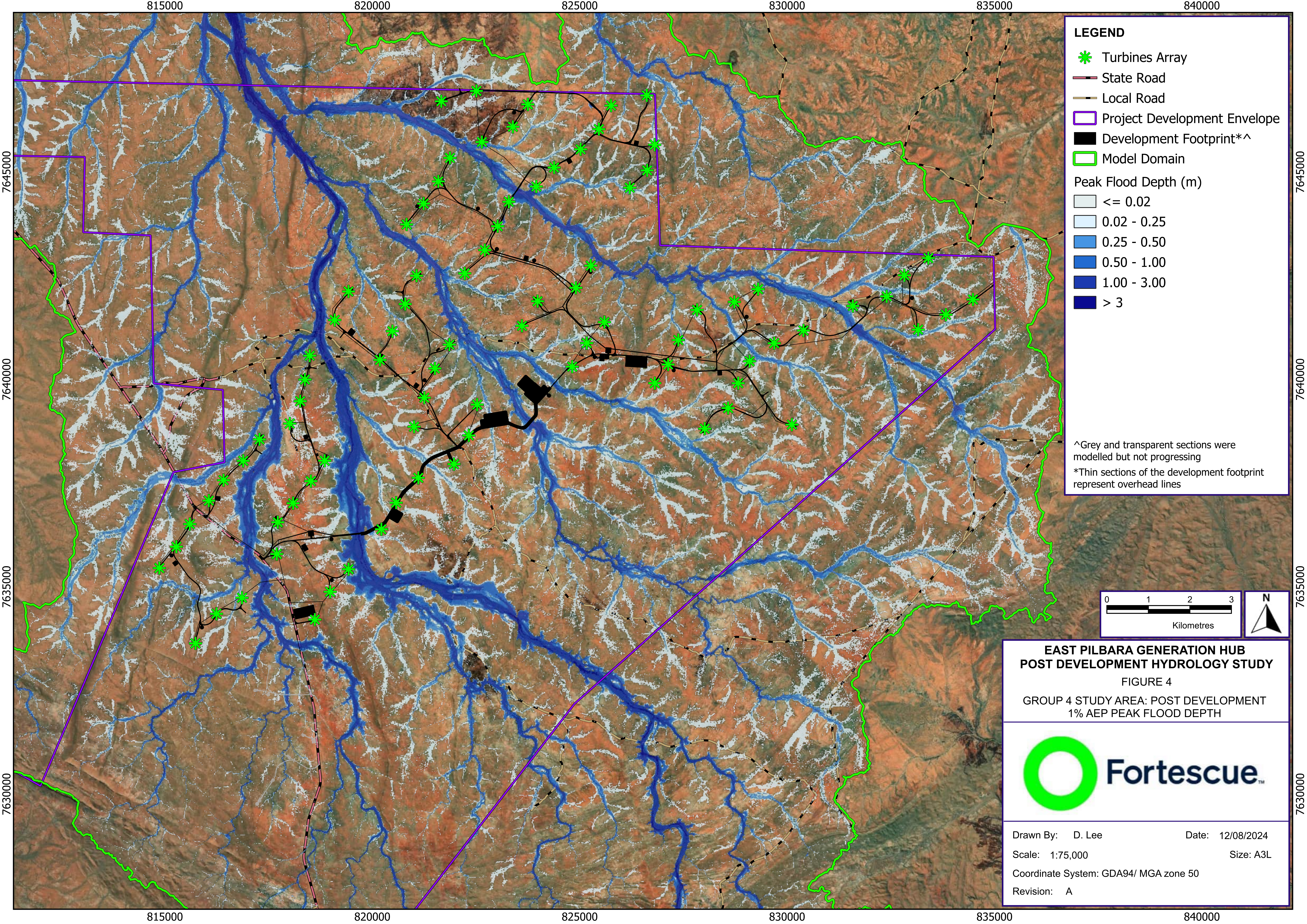
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LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
Peak Flood Depth (m)
<= 0.02
0.02 - 0.25
0.25 - 0.50
0.50 - 1.00
1.00 - 3.00
> 3
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 4
GROUP 4 STUDY AREA: POST DEVELOPMENT
1% AEP PEAK FLOOD DEPTH
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
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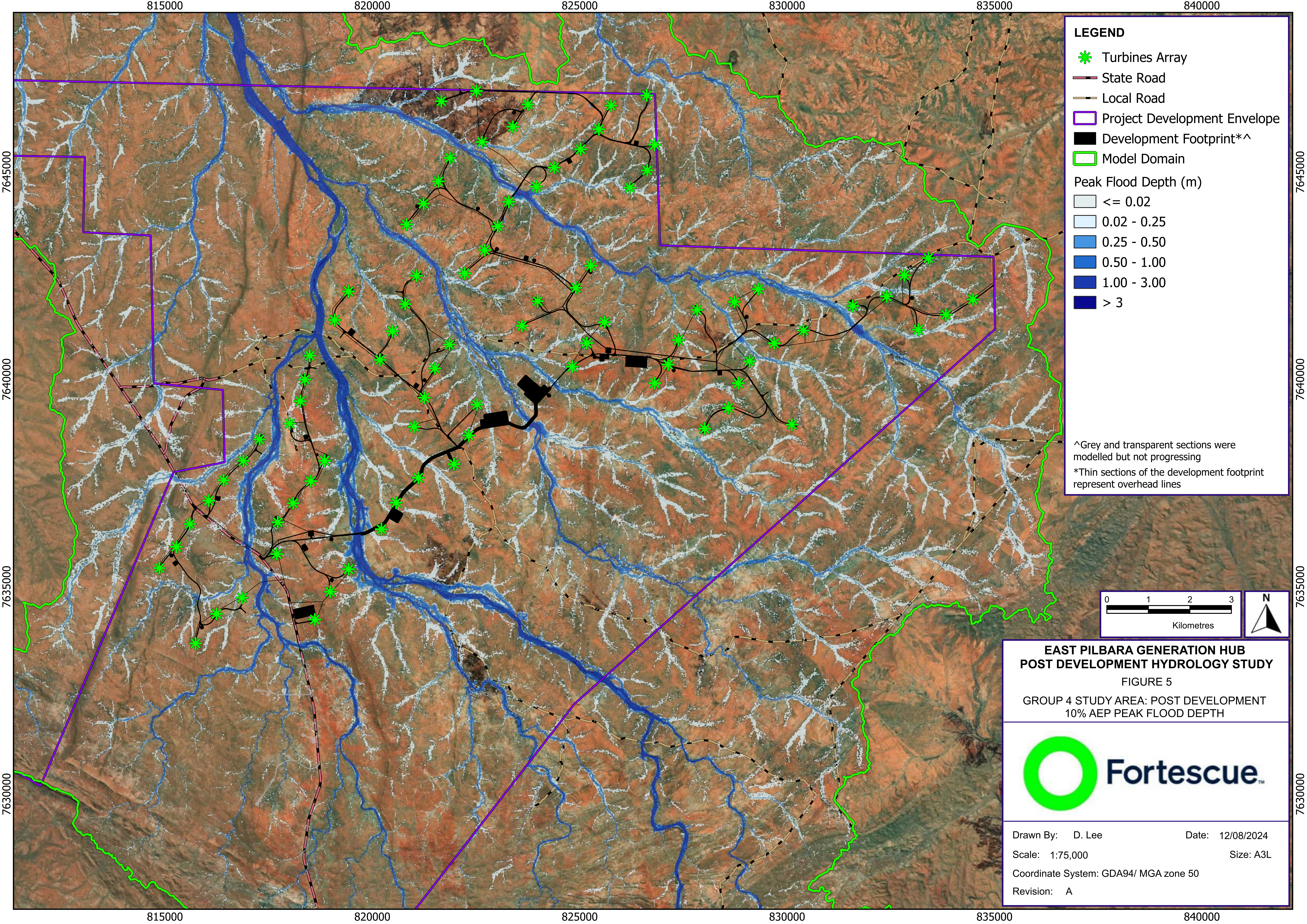
Revision: A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Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
Peak Flood Depth (m)
<= 0.02
0.02 - 0.25
0.25 - 0.50
0.50 - 1.00
1.00 - 3.00
> 3
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
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EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 5
GROUP 4 STUDY AREA: POST DEVELOPMENT
10% AEP PEAK FLOOD DEPTH
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
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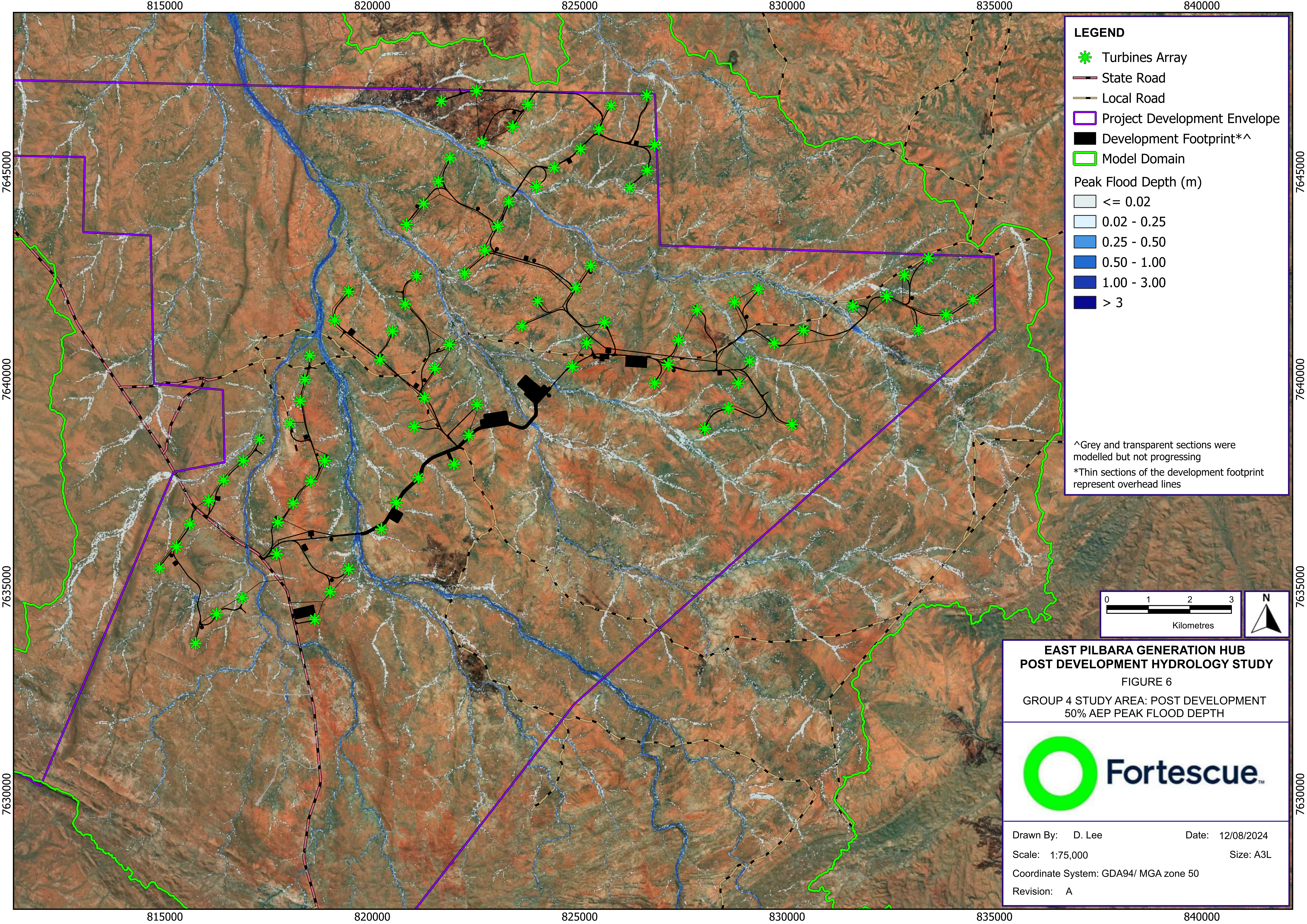

EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

FIGURE 6

GROUP 4 STUDY AREA: POST DEVELOPMENT
50% AEP PEAK FLOOD DEPTH

Fortescue

Drawn By:	D. Lee	Date:	12/08/2024
Scale:	1:75,000	Size:	A3L
Coordinate System:	GDA94/ MGA zone 50		
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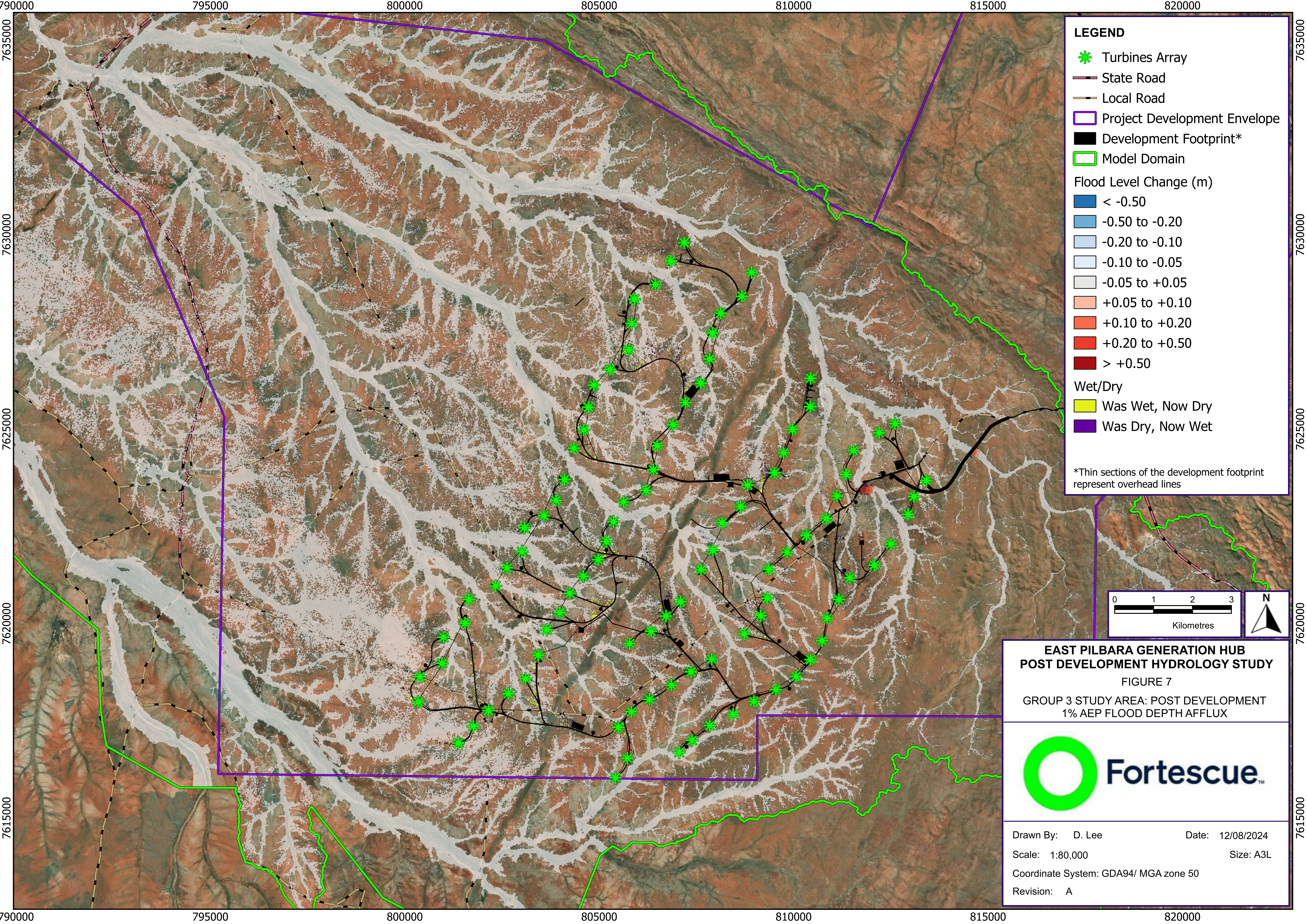
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Flood Level Change (m)
< -0.50
-0.50 to -0.20
-0.20 to -0.10
-0.10 to -0.05
-0.05 to +0.05
+0.05 to +0.10
+0.10 to +0.20
+0.20 to +0.50
> +0.50
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 7
GROUP 3 STUDY AREA: POST DEVELOPMENT
1% AEP FLOOD DEPTH AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
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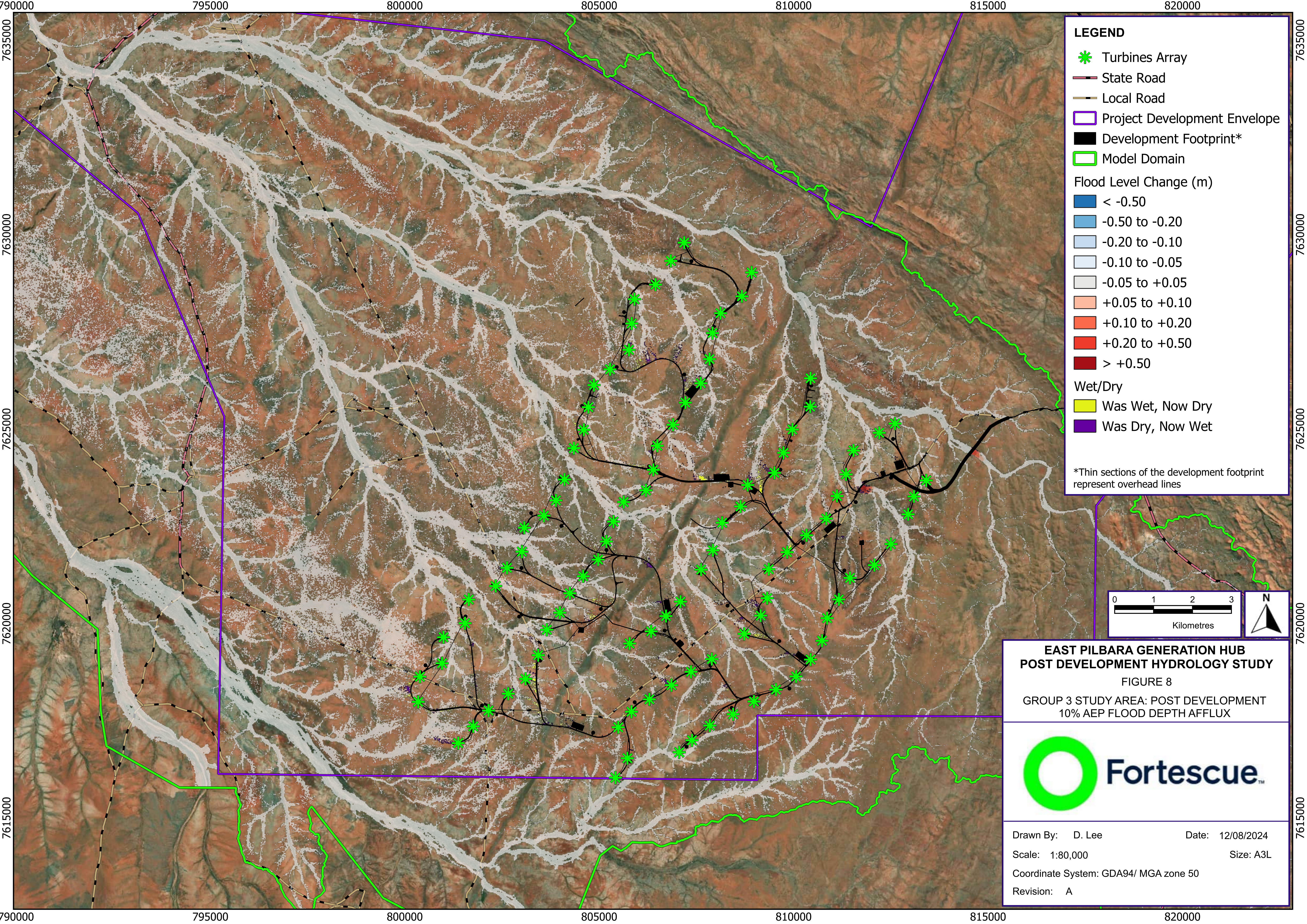

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820000
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7630000
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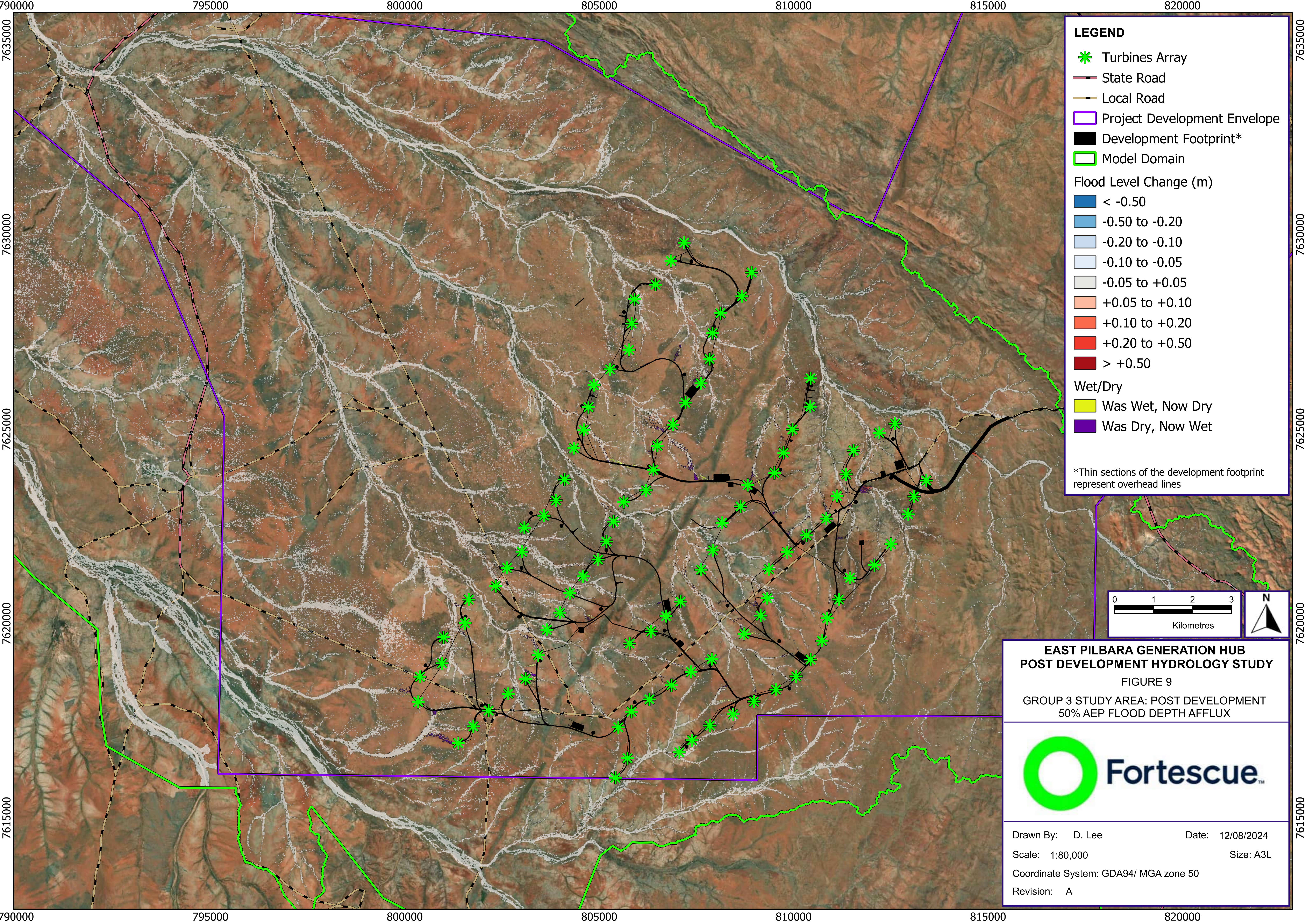
7620000
7615000
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Flood Level Change (m)
< -0.50
-0.50 to -0.20
-0.20 to -0.10
-0.10 to -0.05
-0.05 to +0.05
+0.05 to +0.10
+0.10 to +0.20
+0.20 to +0.50
> +0.50
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 8
GROUP 3 STUDY AREA: POST DEVELOPMENT
10% AEP FLOOD DEPTH AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



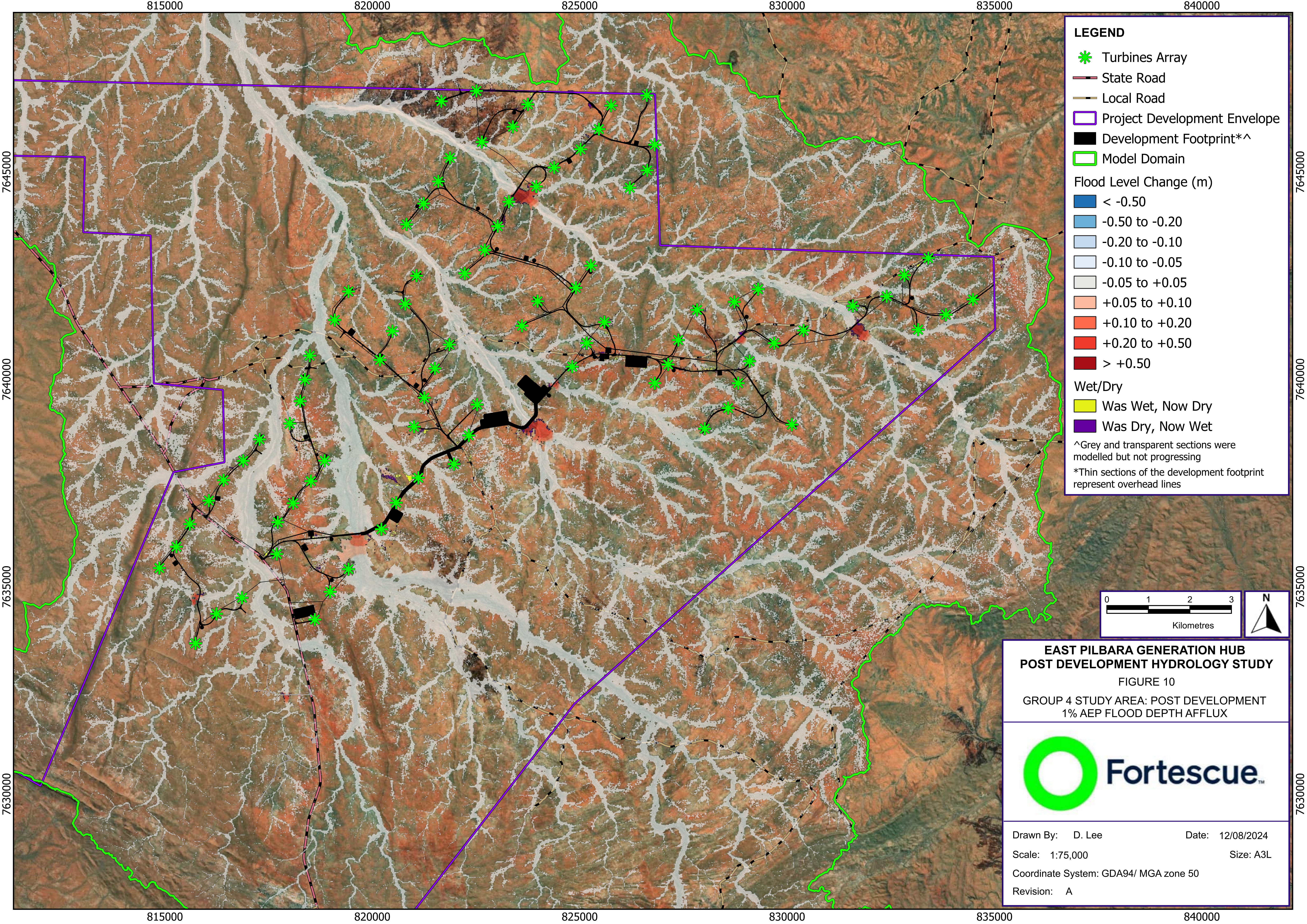
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

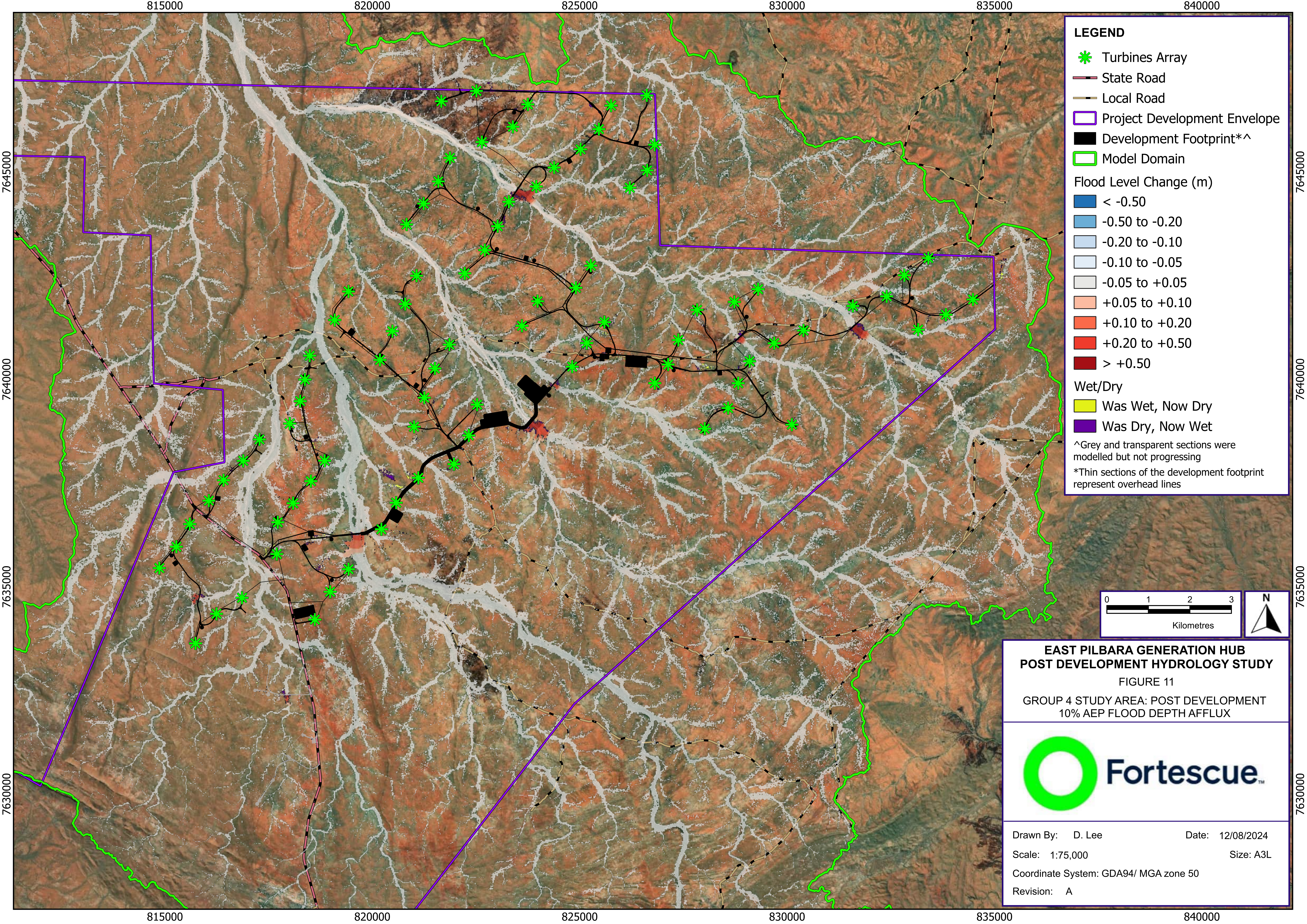
FIGURE 9

GROUP 3 STUDY AREA: POST DEVELOPMENT
50% AEP FLOOD DEPTH AFFLUX

Fortescue

Drawn By: D. Lee	Date: 12/08/2024
Scale: 1:80,000	Size: A3L
Coordinate System: GDA94/ MGA zone 50	
Revision: A	


LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
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
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
815000
820000
825000
830000
835000
840000
7645000
7640000
7635000
7630000
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 10
GROUP 4 STUDY AREA: POST DEVELOPMENT
1% AEP FLOOD DEPTH AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



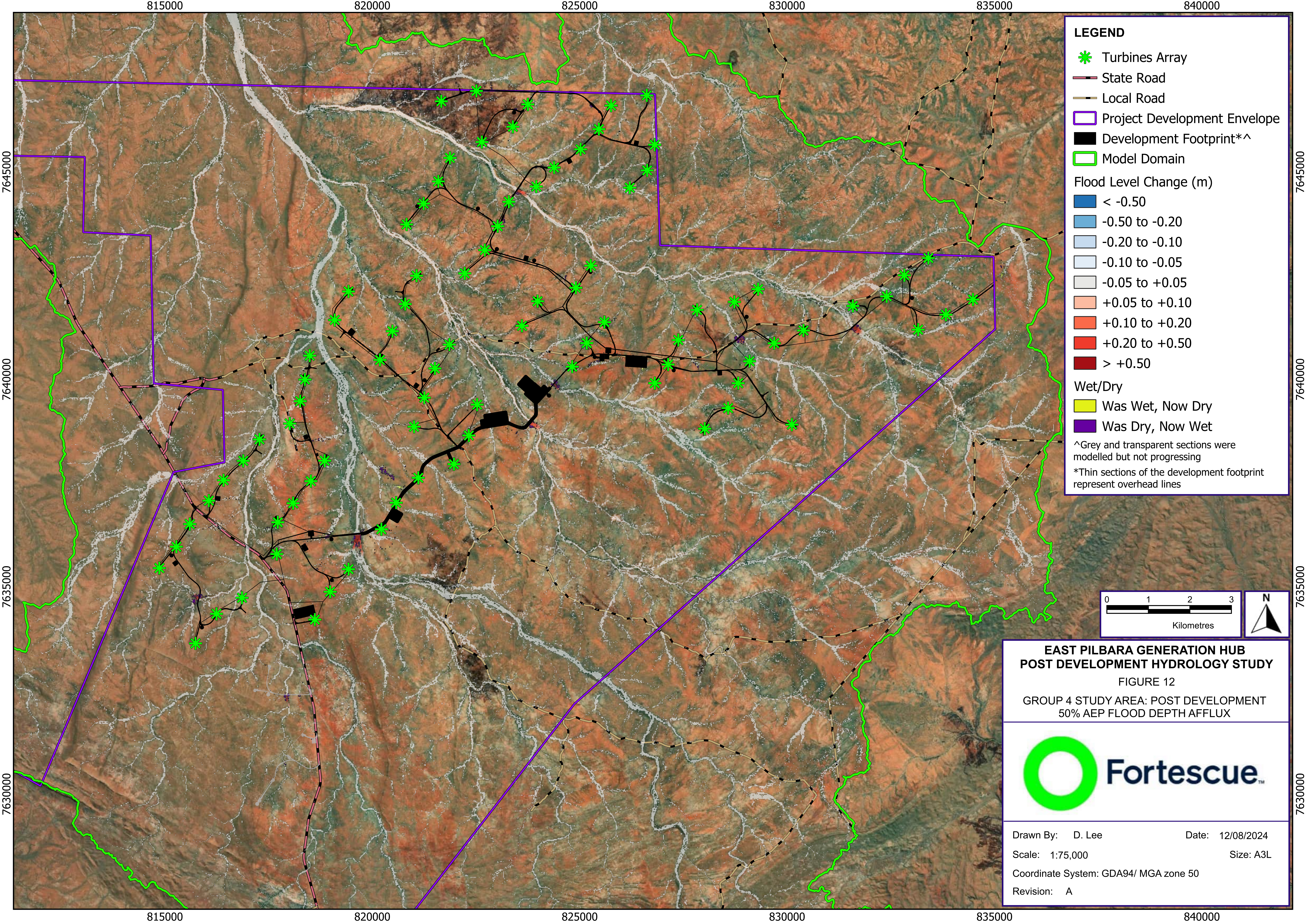
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

FIGURE 11

GROUP 4 STUDY AREA: POST DEVELOPMENT
10% AEP FLOOD DEPTH AFFLUX

Fortescue

Drawn By: D. Lee	Date: 12/08/2024
Scale: 1:75,000	Size: A3L
Coordinate System: GDA94/ MGA zone 50	
Revision: A	



EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

FIGURE 12

GROUP 4 STUDY AREA: POST DEVELOPMENT
50% AEP FLOOD DEPTH AFFLUX

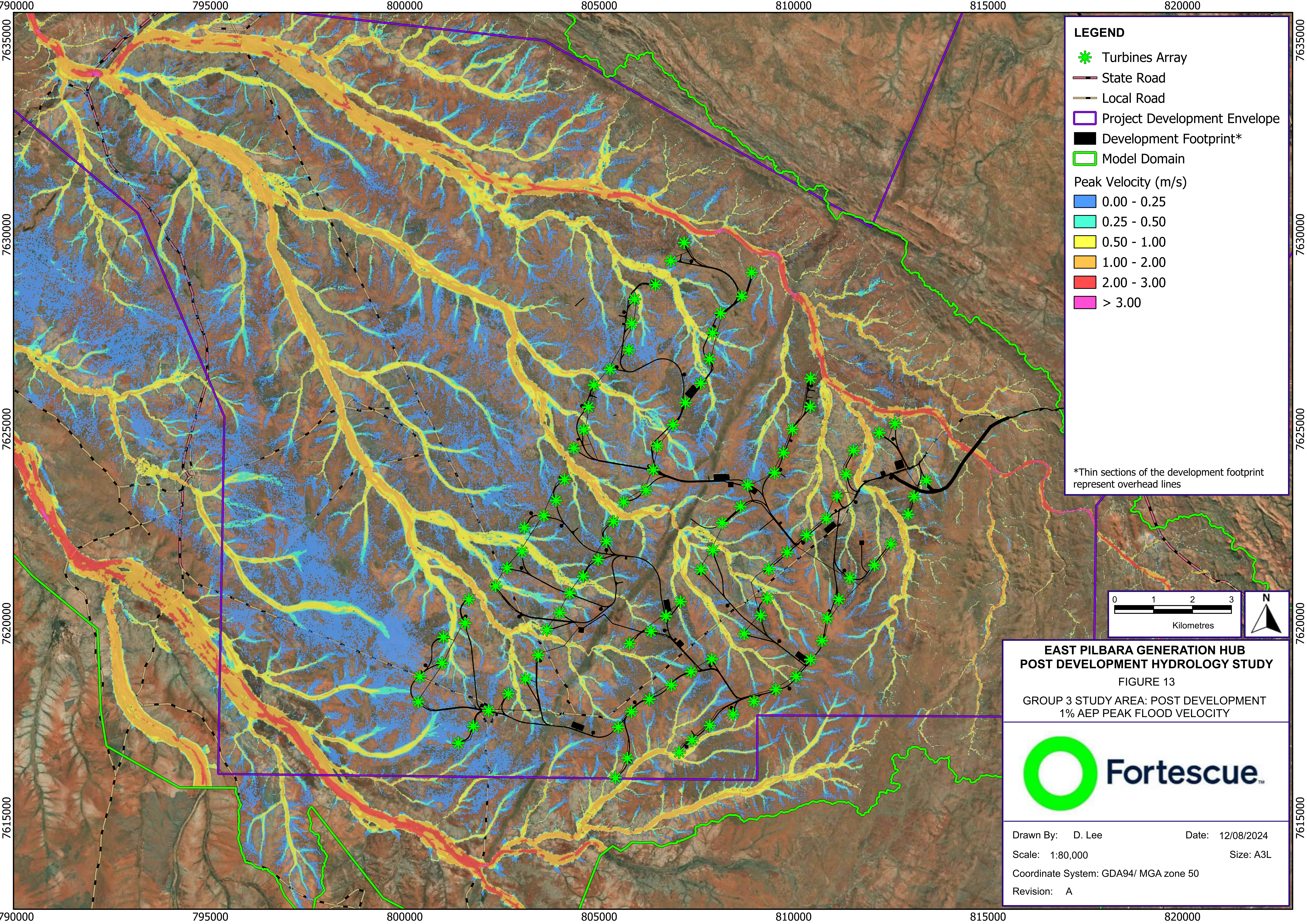
Fortescue

Drawn By: D. Lee | Date: 12/08/2024

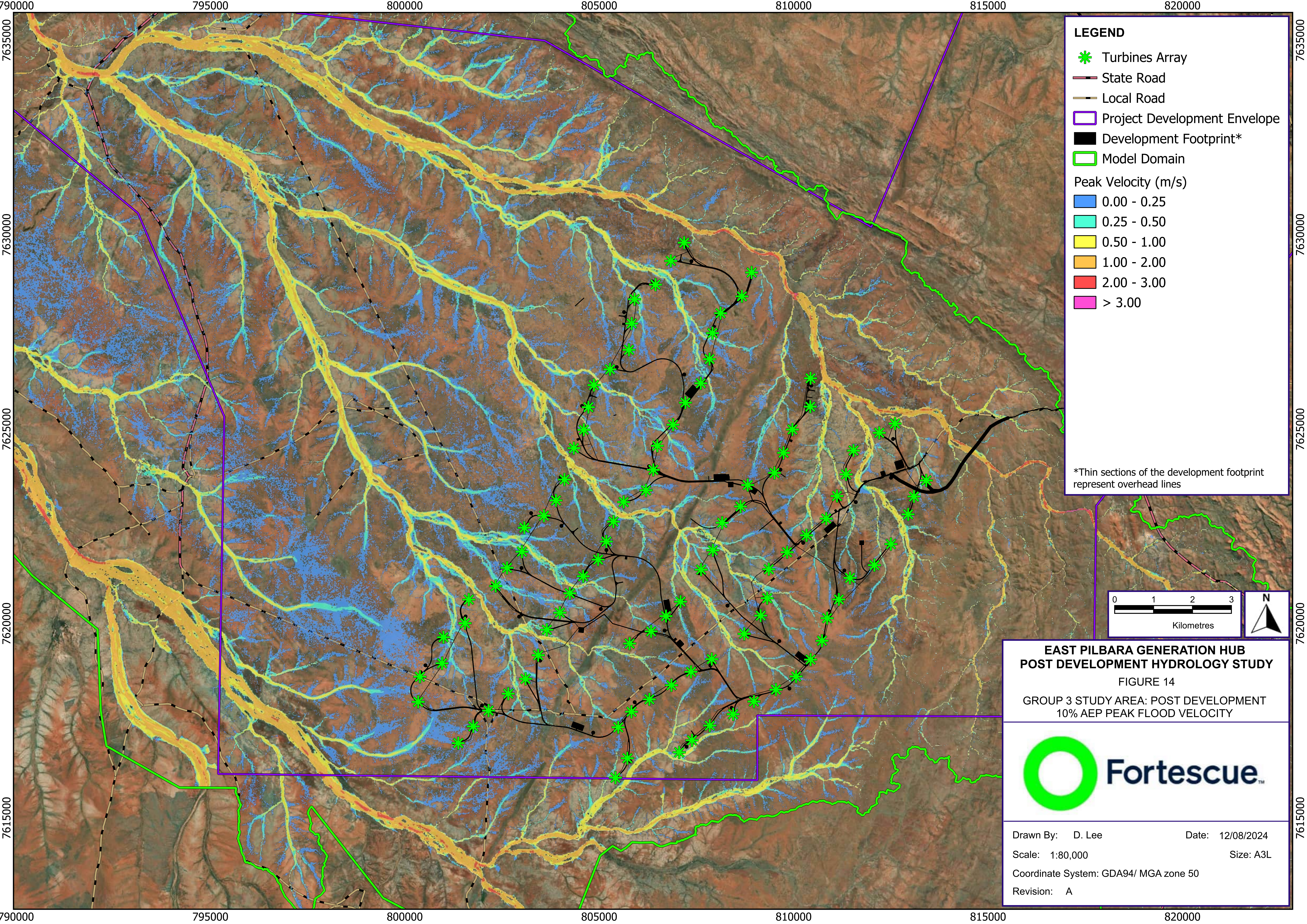
Scale: 1:75,000 | Size: A3L

Coordinate System: GDA94/ MGA zone 50

Revision: A



LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
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
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
790000
795000
800000
805000
810000
815000
820000
7635000
7630000
7625000
7620000
7615000
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 13
GROUP 3 STUDY AREA: POST DEVELOPMENT
1% AEP PEAK FLOOD VELOCITY
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



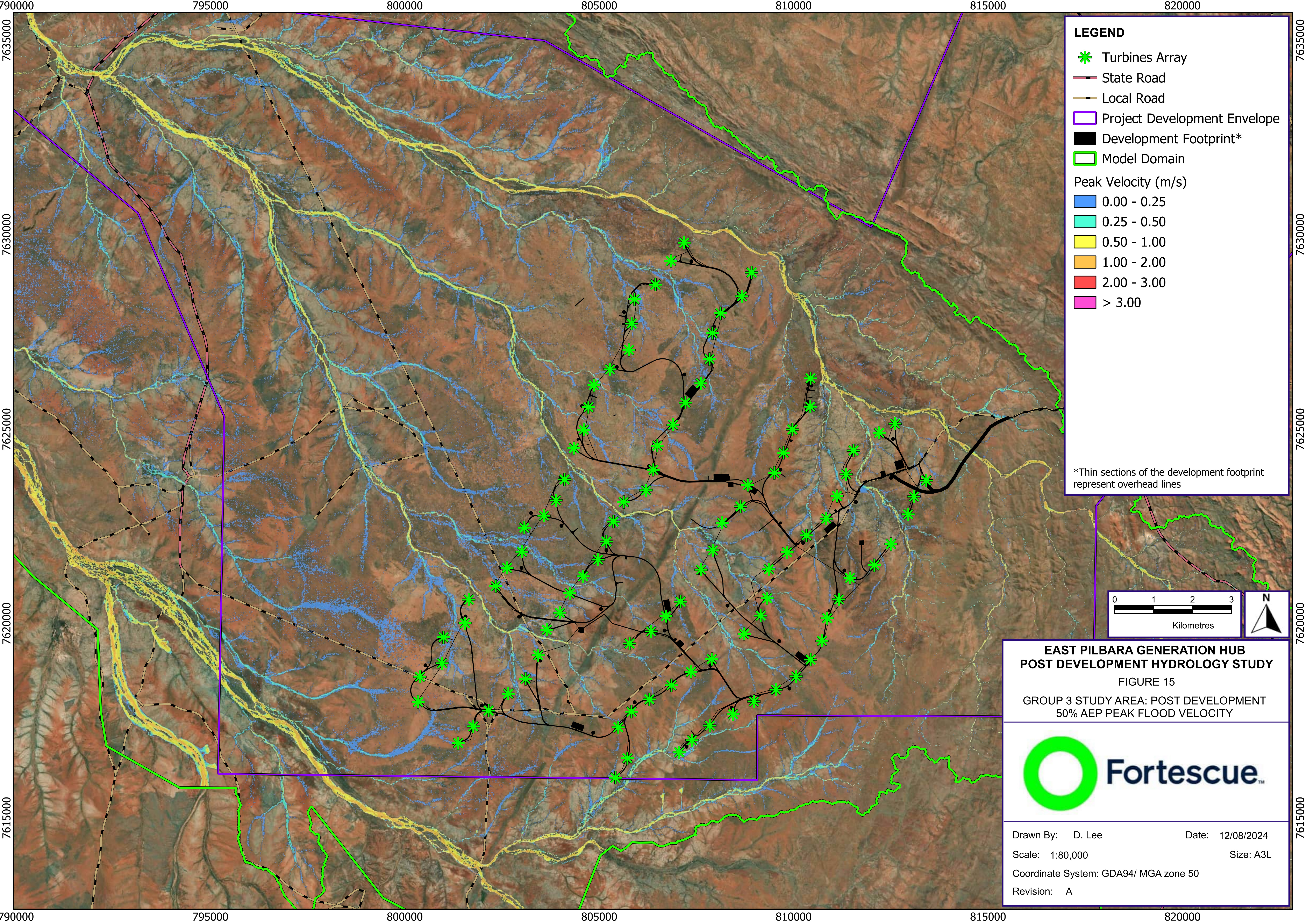
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

FIGURE 14

GROUP 3 STUDY AREA: POST DEVELOPMENT
10% AEP PEAK FLOOD VELOCITY

Fortescue

Drawn By: D. Lee | Date: 12/08/2024
Scale: 1:80,000 | Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



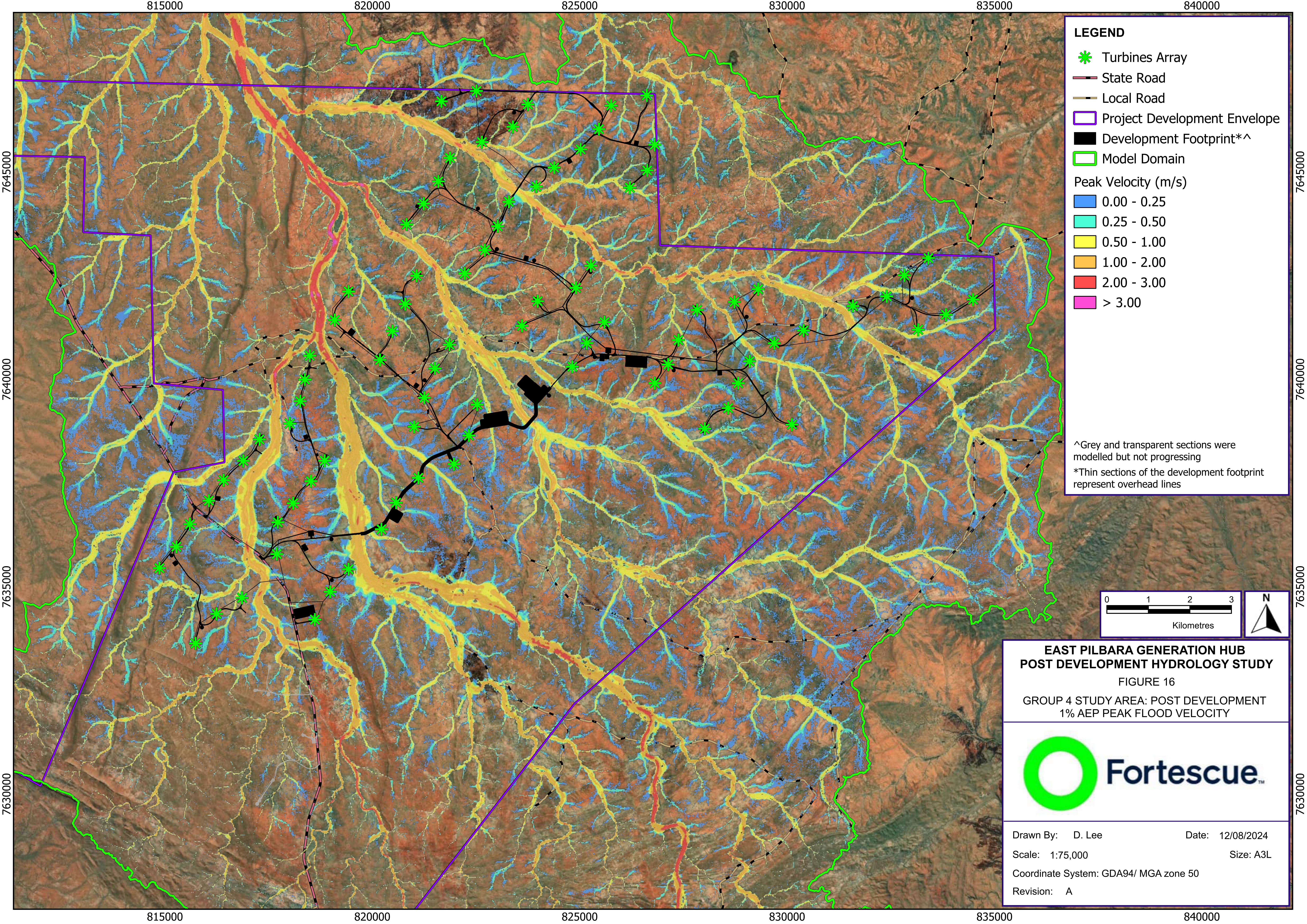
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

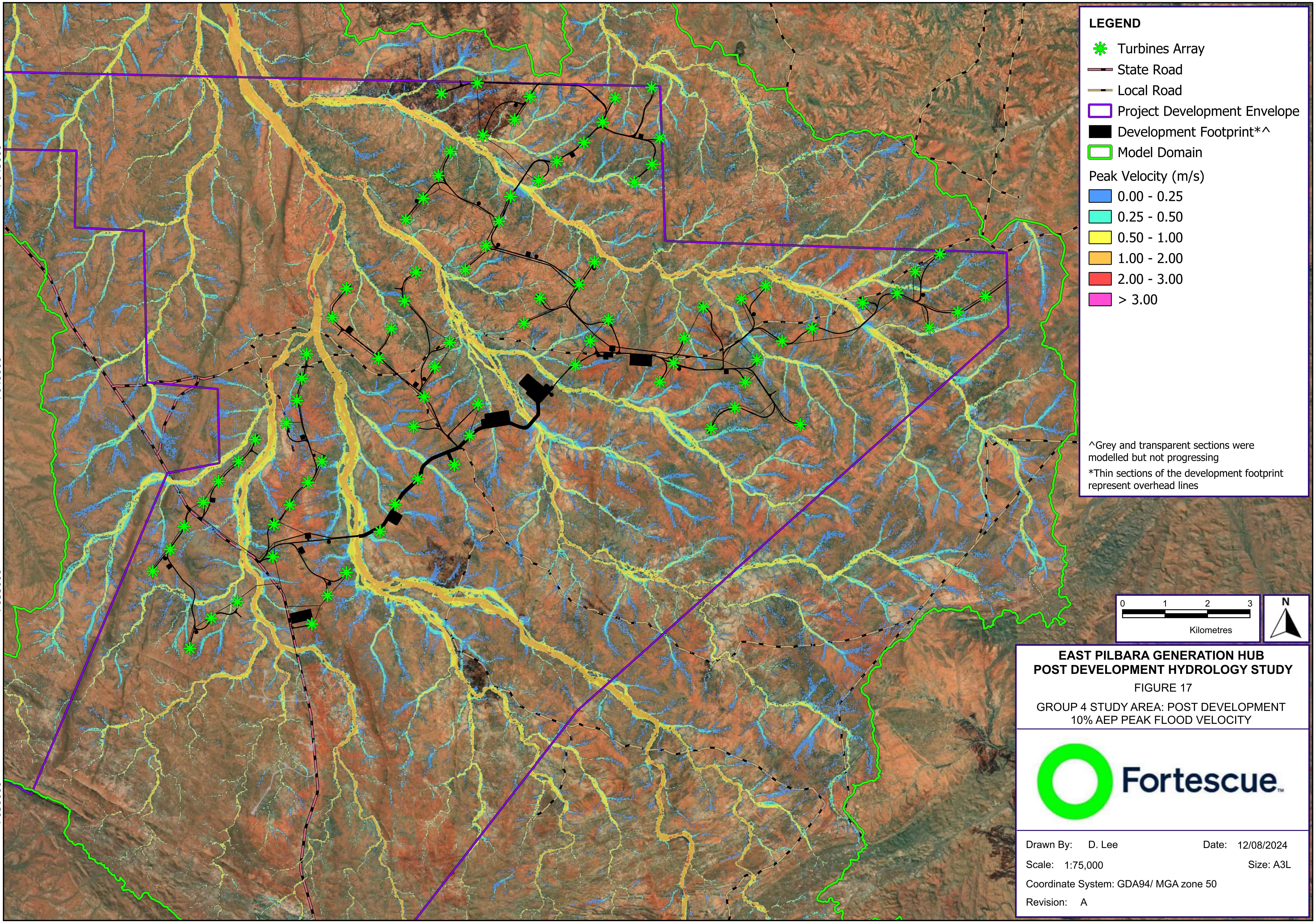
FIGURE 15

GROUP 3 STUDY AREA: POST DEVELOPMENT
50% AEP PEAK FLOOD VELOCITY

Fortescue

Drawn By: D. Lee	Date: 12/08/2024
Scale: 1:80,000	Size: A3L
Coordinate System: GDA94/ MGA zone 50	
Revision: A	


LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
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
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
815000
820000
825000
830000
835000
840000
7645000
7640000
7635000
7630000
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 16
GROUP 4 STUDY AREA: POST DEVELOPMENT
1% AEP PEAK FLOOD VELOCITY
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY

FIGURE 17

GROUP 4 STUDY AREA: POST DEVELOPMENT
10% AEP PEAK FLOOD VELOCITY

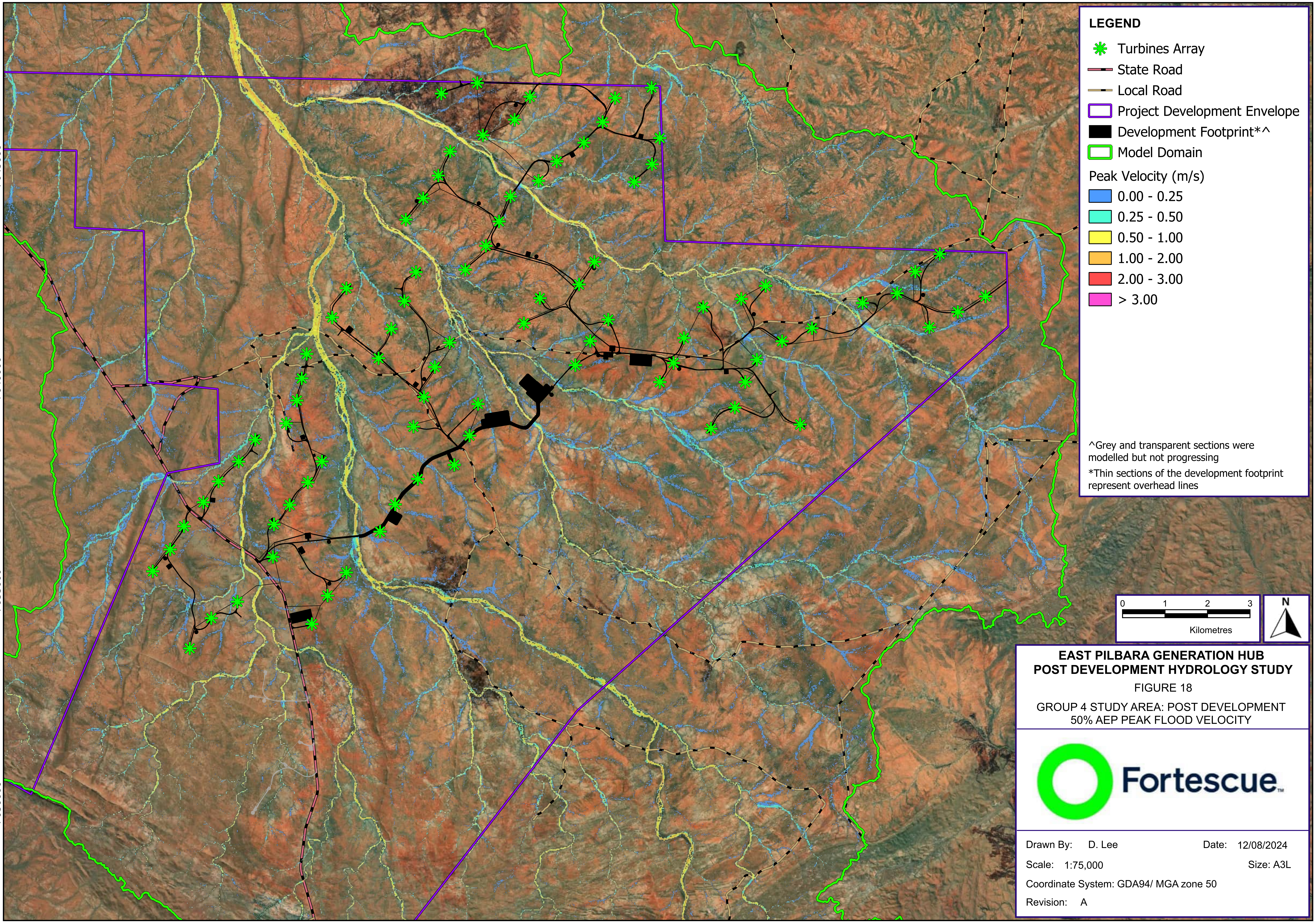
Fortescue

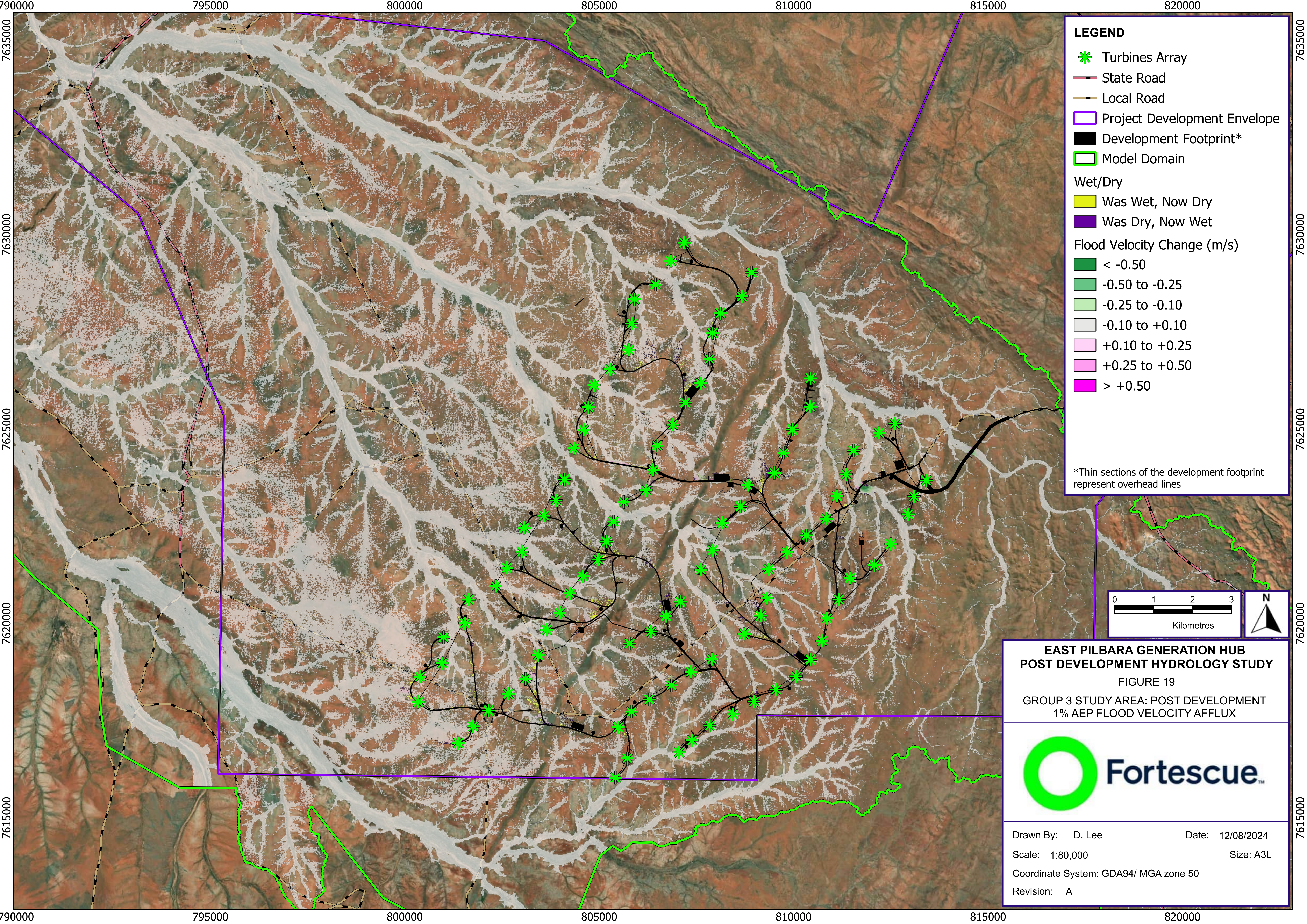
Drawn By: D. Lee Date: 12/08/2024

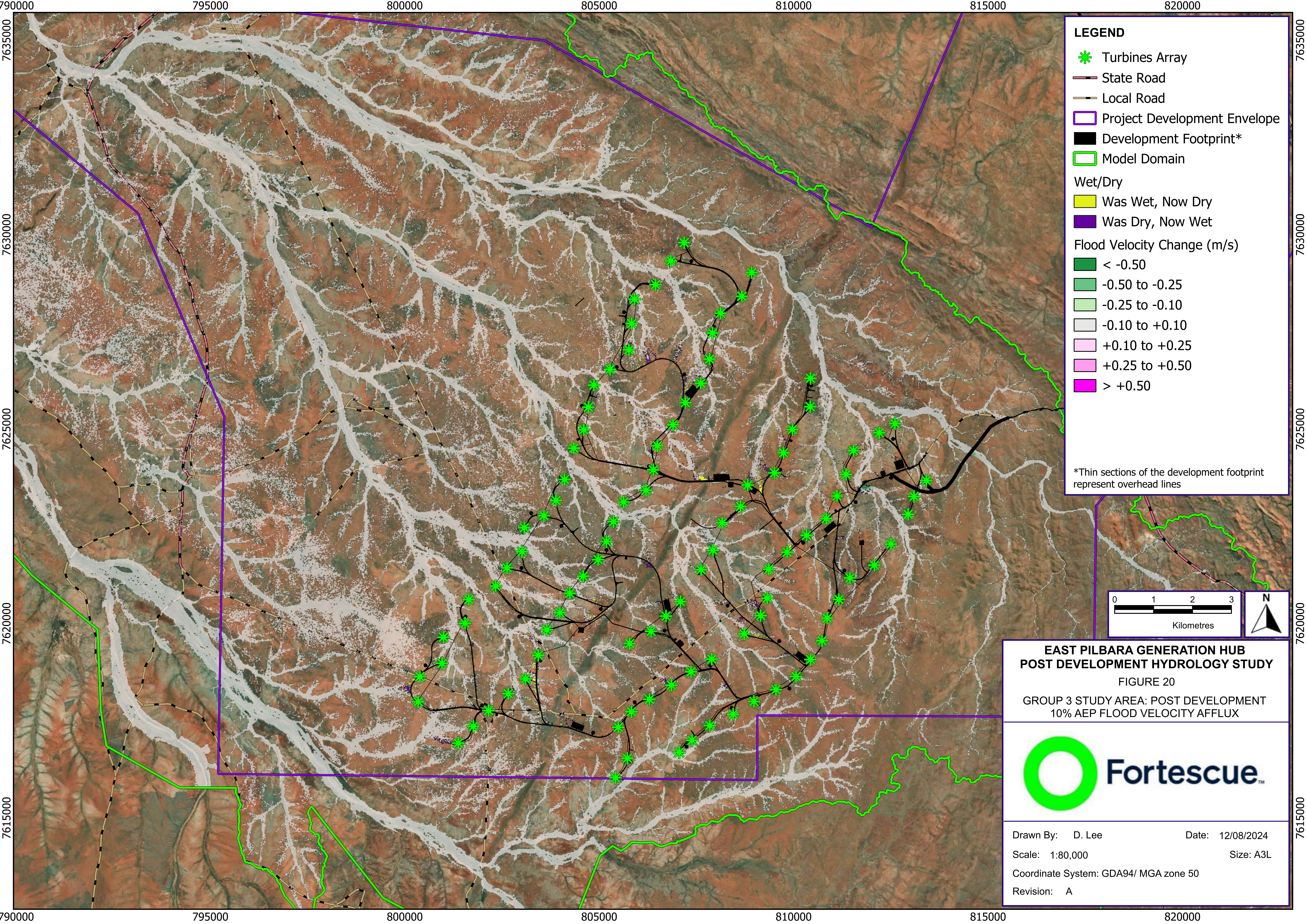
Scale: 1:75,000 Size: A3L

Coordinate System: GDA94/ MGA zone 50

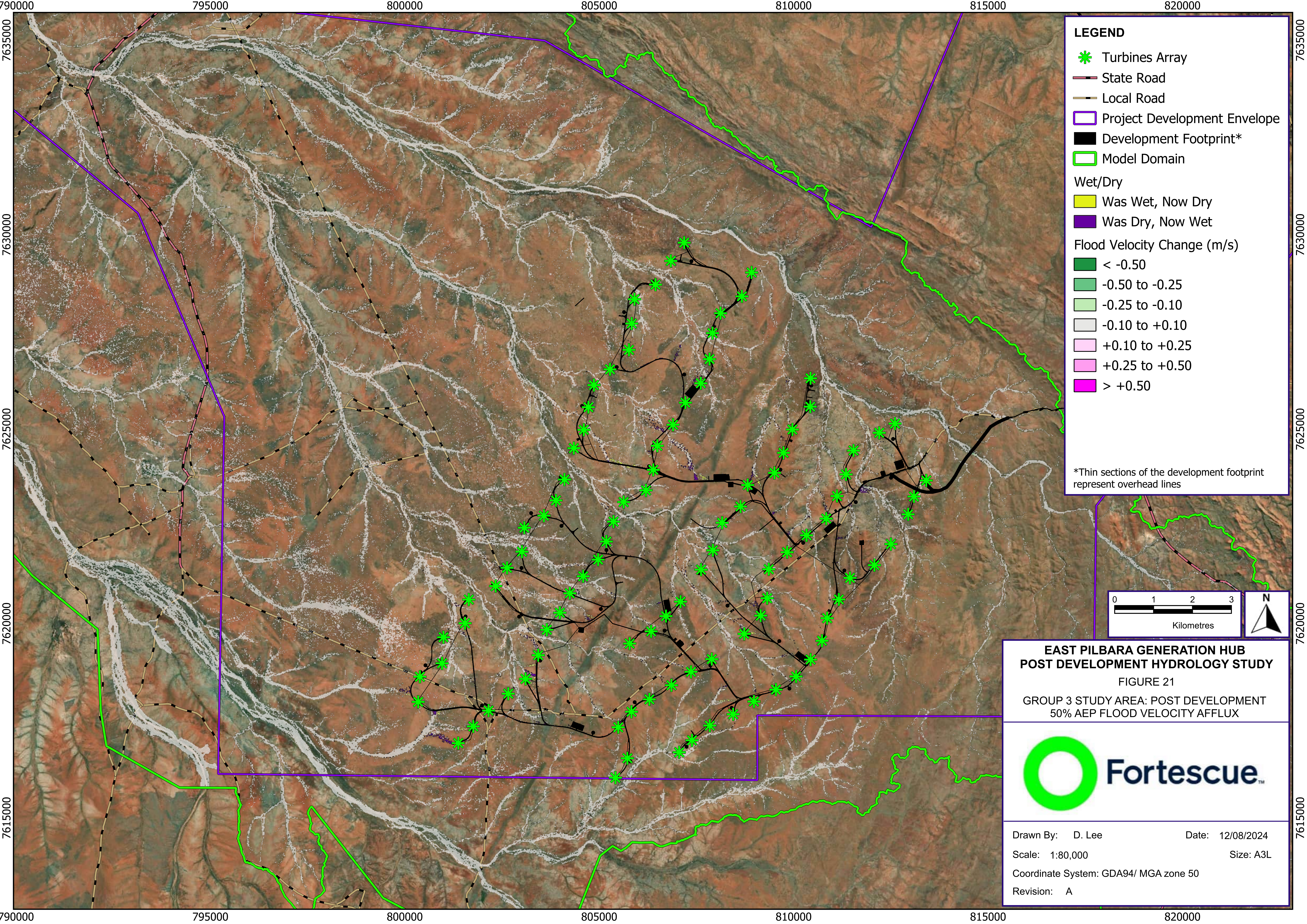
Revision: A


815000
820000
825000
830000
835000
840000
7645000
7640000
7635000
7630000
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
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
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 18
GROUP 4 STUDY AREA: POST DEVELOPMENT
50% AEP PEAK FLOOD VELOCITY
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

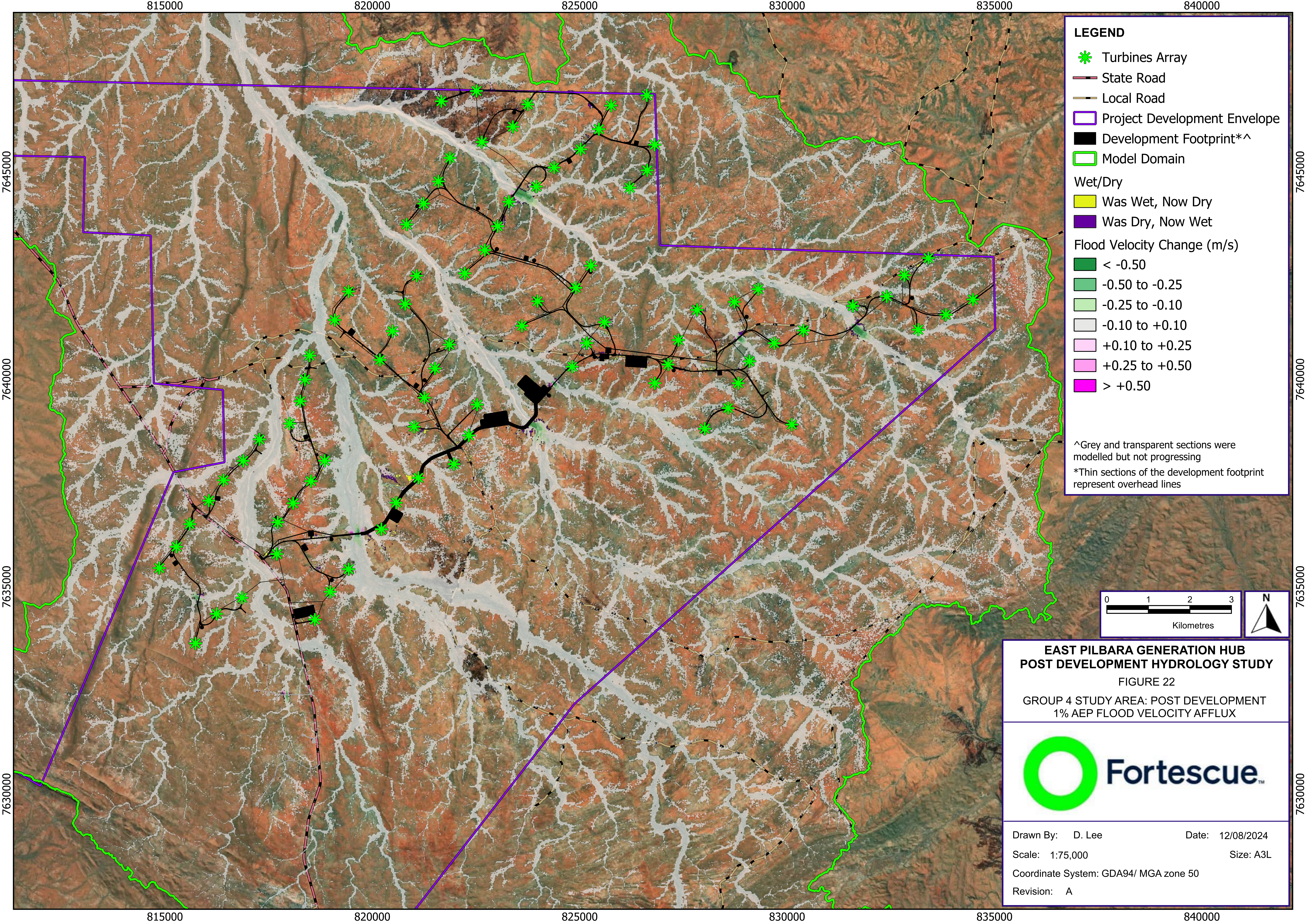

LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
Flood Velocity Change (m/s)
< -0.50
-0.50 to -0.25
-0.25 to -0.10
-0.10 to +0.10
+0.10 to +0.25
+0.25 to +0.50
> +0.50
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 19
GROUP 3 STUDY AREA: POST DEVELOPMENT
1% AEP FLOOD VELOCITY AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A
790000
795000
800000
805000
810000
815000
820000
7635000
7630000
7625000
7620000
7615000



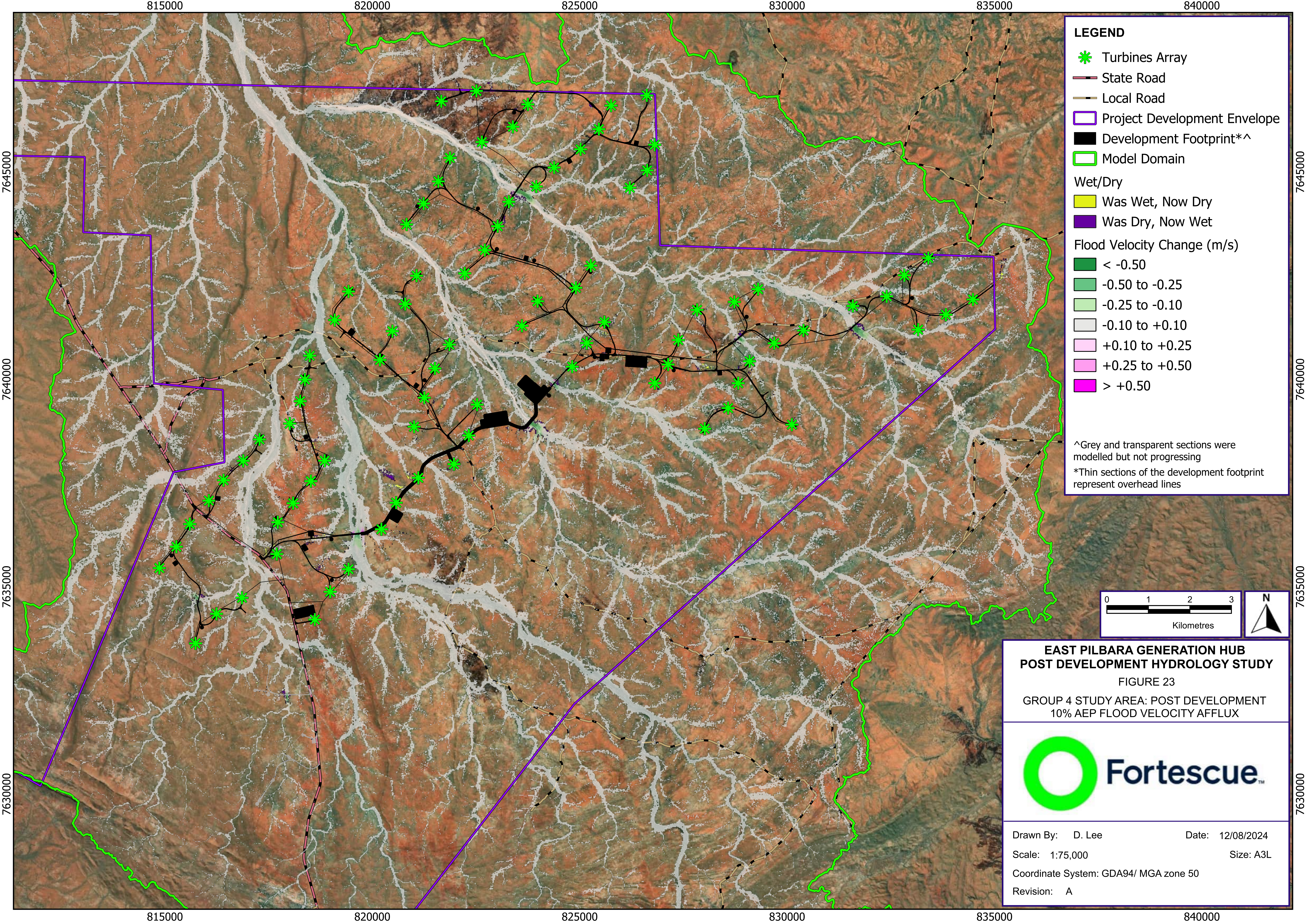
790000
795000
800000
805000
810000
815000
820000
7635000
7630000
7625000
7620000
7615000
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
Flood Velocity Change (m/s)
< -0.50
-0.50 to -0.25
-0.25 to -0.10
-0.10 to +0.10
+0.10 to +0.25
+0.25 to +0.50
> +0.50
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 20
GROUP 3 STUDY AREA: POST DEVELOPMENT
10% AEP FLOOD VELOCITY AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A

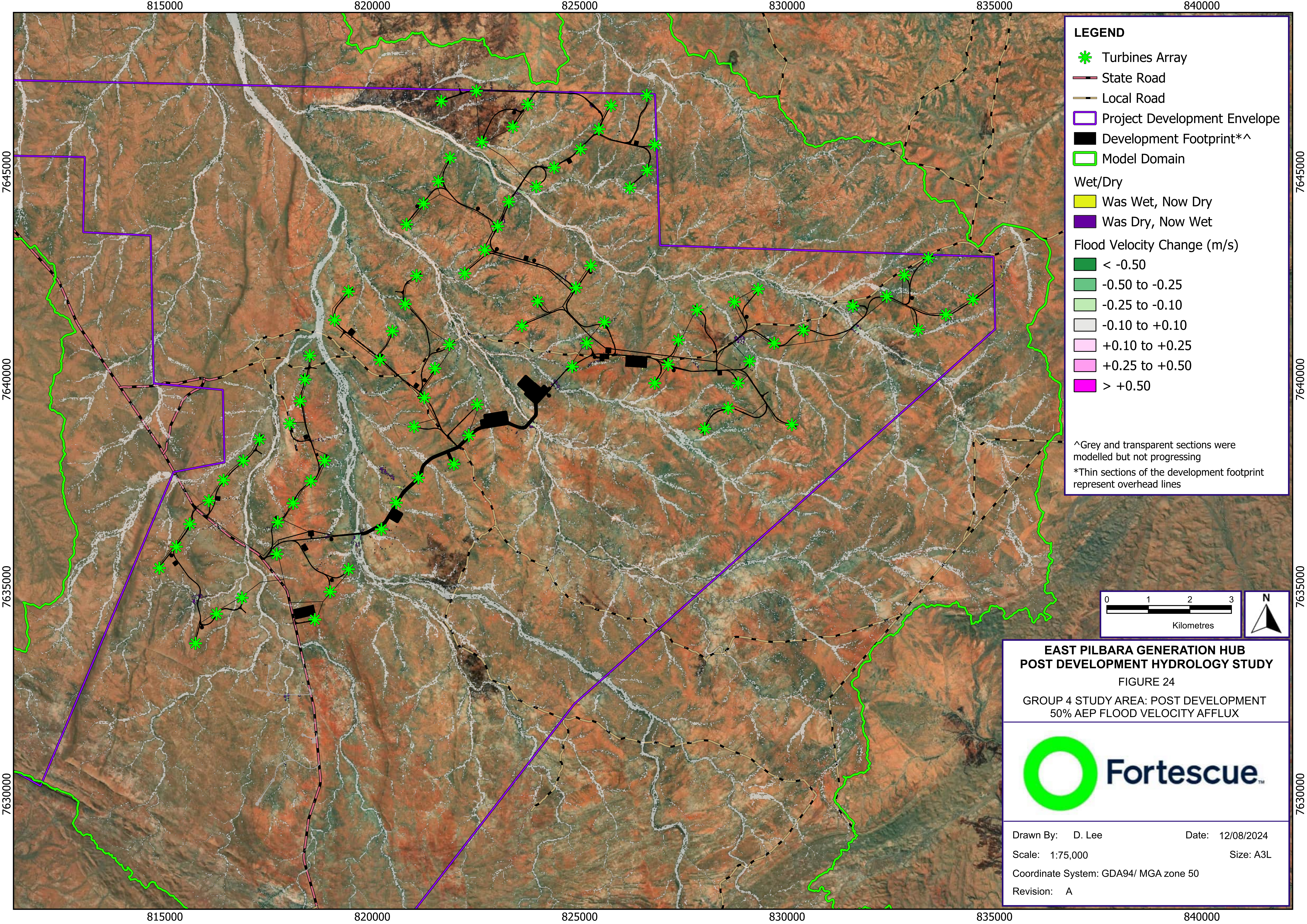


790000
795000
800000
805000
810000
815000
820000
7635000
7630000
7625000
7620000
7615000
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*
Model Domain
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
Flood Velocity Change (m/s)
< -0.50
-0.50 to -0.25
-0.25 to -0.10
-0.10 to +0.10
+0.10 to +0.25
+0.25 to +0.50
> +0.50
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 21
GROUP 3 STUDY AREA: POST DEVELOPMENT
50% AEP FLOOD VELOCITY AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:80,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
Flood Velocity Change (m/s)
< -0.50
-0.50 to -0.25
-0.25 to -0.10
-0.10 to +0.10
+0.10 to +0.25
+0.25 to +0.50
> +0.50
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
815000
820000
825000
830000
835000
840000
7645000
7640000
7635000
7630000
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 22
GROUP 4 STUDY AREA: POST DEVELOPMENT
1% AEP FLOOD VELOCITY AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A


815000
820000
825000
830000
835000
840000
7645000
7640000
7635000
7630000
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
Flood Velocity Change (m/s)
< -0.50
-0.50 to -0.25
-0.25 to -0.10
-0.10 to +0.10
+0.10 to +0.25
+0.25 to +0.50
> +0.50
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 23
GROUP 4 STUDY AREA: POST DEVELOPMENT
10% AEP FLOOD VELOCITY AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A



815000
820000
825000
830000
835000
840000
7645000
7640000
7635000
7630000
LEGEND
Turbines Array
State Road
Local Road
Project Development Envelope
Development Footprint*^
Model Domain
Wet/Dry
Was Wet, Now Dry
Was Dry, Now Wet
Flood Velocity Change (m/s)
< -0.50
-0.50 to -0.25
-0.25 to -0.10
-0.10 to +0.10
+0.10 to +0.25
+0.25 to +0.50
> +0.50
^Grey and transparent sections were modelled but not progressing
*Thin sections of the development footprint represent overhead lines
0 1 2 3
Kilometres
N
EAST PILBARA GENERATION HUB
POST DEVELOPMENT HYDROLOGY STUDY
FIGURE 24
GROUP 4 STUDY AREA: POST DEVELOPMENT
50% AEP FLOOD VELOCITY AFFLUX
Fortescue
Drawn By: D. Lee
Date: 12/08/2024
Scale: 1:75,000
Size: A3L
Coordinate System: GDA94/ MGA zone 50
Revision: A