Marri Wind Farm

Marri Windfarm Flood Study

Alinta Energy

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Abbreviations

AEP	Annual Exceedance Probability
AGBT	Austroads Guide to Bridge Technology
AHD	Above Height Datum
ARF	Aerial Reduction Factor
ARI	Annual Recurrence Interval
ARR19	Australian Rainfall and Runoff 2019
ARR87	Australian Rainfall and Runoff 1987
BOM	Bureau of Meteorology
DEM	Digital Elevation Model
DTM	Digital Terrain Model
FABDEM	Forest And Buildings removed Copernicus DEM
FFA	Flood Frequency Analysis
GPU	Graphics Processing Unit
HPC	Heavily Parallelised Compute
IFD	Intensity-Frequency-Duration
LiDAR	Light Detection and Ranging
MRWA	Main Roads Western Australia
PRM	Probabilistic Rational Method
QUDM	Queensland Urban Drainage Manual
RFFE	Regional Flood Frequency Estimation
SLK	Straight Line Kilometre
SLS	Serviceability Limit States
SRTM	Shuttle Radar Topography Mission
ULS	Ultimate Limit States
WBK	Width Between Kerbs
WUC	Works Under Contract

1 Project Background

1.1 Introduction

Alinta Energy has engaged Aurecon to undertake a Hydrological and Hydraulic Desktop Study for the proposed Marri Wind Farm (the Project) in the Shire of Dandaragan, approximately 110 kilometres (km) north of Perth, Western Australia.

The primary objective of this study is to assess the existing pre-development surface water regime within the proposed project site and evaluate potential flooding risks to the proposed infrastructure during both construction and operational phases. The modelling does not include changes to topography or runoff that may result from the proposed project. It also provides additional context to the water resource impact assessment study.

1.2 Flood Study Scope

The scope of this study is as follows:

- Flood modelling to assess overland flow paths across the existing topography and interaction with the proposed structures
- Local surface water assessment through and over the project site from Caren Caren Brook and Moore River.
- Overlay of proposed infrastructure layout, including access tracks, laydown areas, temporary accommodation camps, turbine locations, cable trenches, and electrical infrastructure - provided by Alinta.
- Hydrological and hydraulic analysis for 50%, 20%, 10%, 5%, 1%, 0.5% and 0.2% Annual Exceedance Probability (AEP) storm events to determine flood extents, flood depths, flood levels, flood velocities, flood hazards and bed shear stress.
- Flood modelling outputs for Depth (d), Flood Level (h), Velocity (V), Flood Hazard (ZAEM1) and Bed Shear Stress (BSS).
- Development of flood risk mapping, delivered in both PDF and GIS formats for the nominated design AEP events, to inform infrastructure design and layout refinement.
- Preparation of a Technical Memorandum (this report), outlining the methodology, assumptions, results and recommendations to guide future design development and surface water management strategies.

1.3 Design Criteria

The flood risk assessment will inform the design in relation to flood risk. Typical flood probabilities presented in Table 1-1 have been adopted for this assessment.

Table 1-1 Summary of the simulation criteria for design elements

Design Element	Design Event
Construction flood assessments	50% AEP (1 in 2 year ARI)
Access track serviceability	Typically 10% AEP (1 in 10 year ARI); Varies between 5 - 20% AEP pending optimisation per site
Power Substations	1% AEP (1 in 100 year ARI) or 0.5% AEP (1 in 200 year ARI) or 0.2% AEP (1 in 500 year ARI)
Turbine footings	0.5% AEP (1 in 200 year ARI) or 0.2% AEP (1 in 500 year ARI)

1.4 Standards, Guidelines and Regulations

The design shall comply with, but is not limited to:

- Code of Practice – Urban and Peri-Urban Drainage Modelling
- Austroads: Guide to Bridge Technology Part 8: Hydraulic Design of Waterway Structures
- MRWA supplement to Austroads: Guide to Road Design – Part 5 – Drainage: General and Hydrology Considerations
- Austroads: Guide to Road Design – Part 5 – Drainage: General and Hydrology Considerations
- Australian Rainfall and Runoff 2019

In the event of conflicting guidance, precedence shall be given to the document listed higher.

1.5 Location

The project site is located approximately 110 kilometres north of Perth, within the Shire of Dandaragan in Western Australia. The wind farm is proposed to span over ~12,550 hectares of predominantly agricultural land.

The site is strategically positioned to connect to the 330 kV Western Power transmission network and benefit from the Clean Energy Link North infrastructure upgrade. The area features gently undulating terrain typical of the region, with various drainage pathways influencing infrastructure layout and constructability.

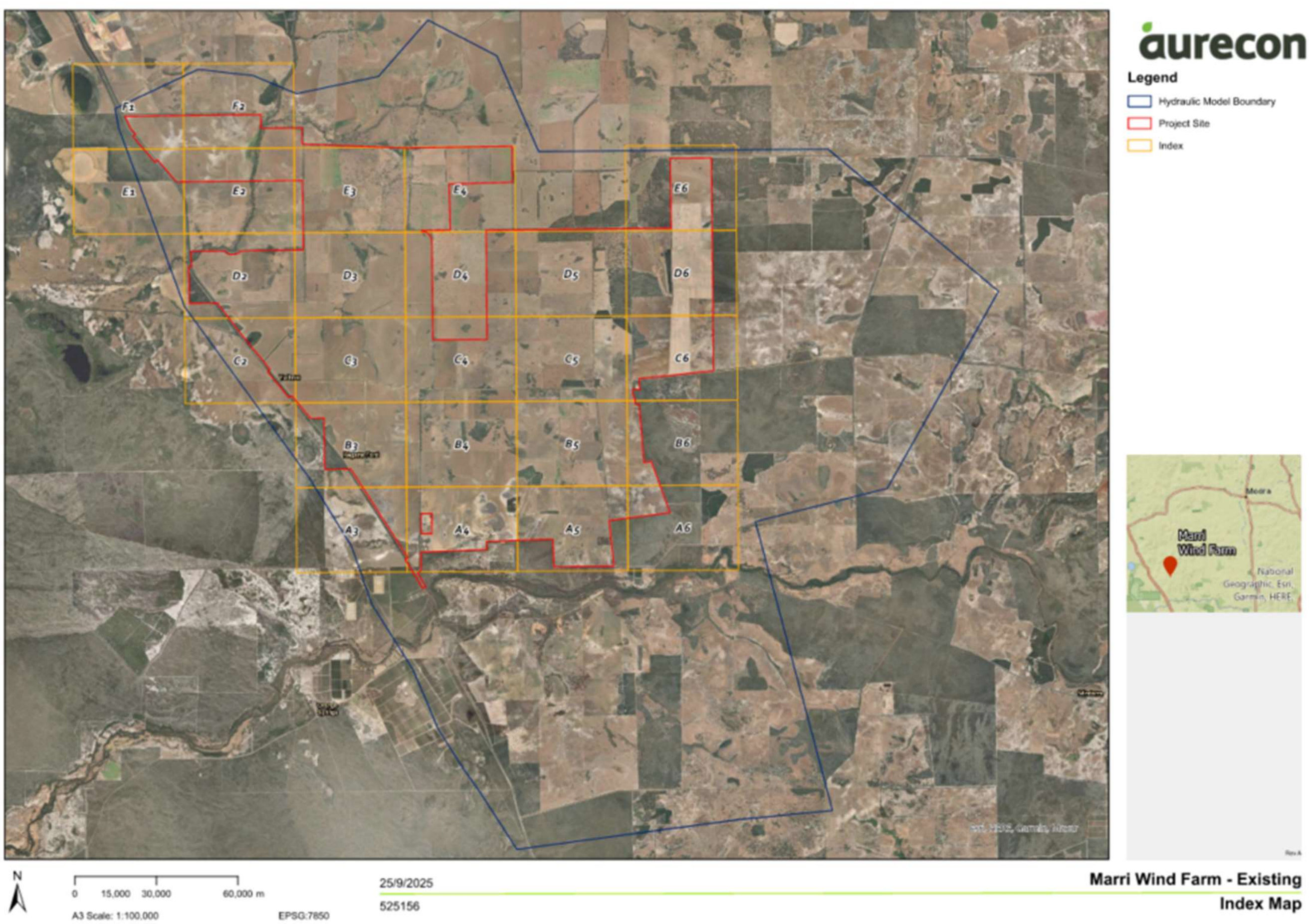

Figure 1-1 Proposed Project Development Envelope Location

2 Input Data

2.1 Previous Studies

Previous studies have been completed which provided additional data and information about the region, catchment and site. These studies included:

- Moora flood management study, September 2000, Water & Rivers Commission, Western Australia

2.2 Terrain Data

A 0.5 metre (m) LiDAR DTM dataset covering the full project footprint was supplied for this study. To supplement this, 5 m LiDAR DTM topographic data was extracted from the publicly available Geoscience Australia Elevation Information System (ELVIS) for areas outside the extent of the 0.5 m LiDAR DTM and beyond the project boundary. This combined dataset was deemed sufficient for the hydrological and hydraulic analysis (refer Figure 2-1).

While formal accuracy documentation for the supplied DTM was not provided, cross-checks against the 5 m LiDAR DTM data indicated general consistency in key landform features.

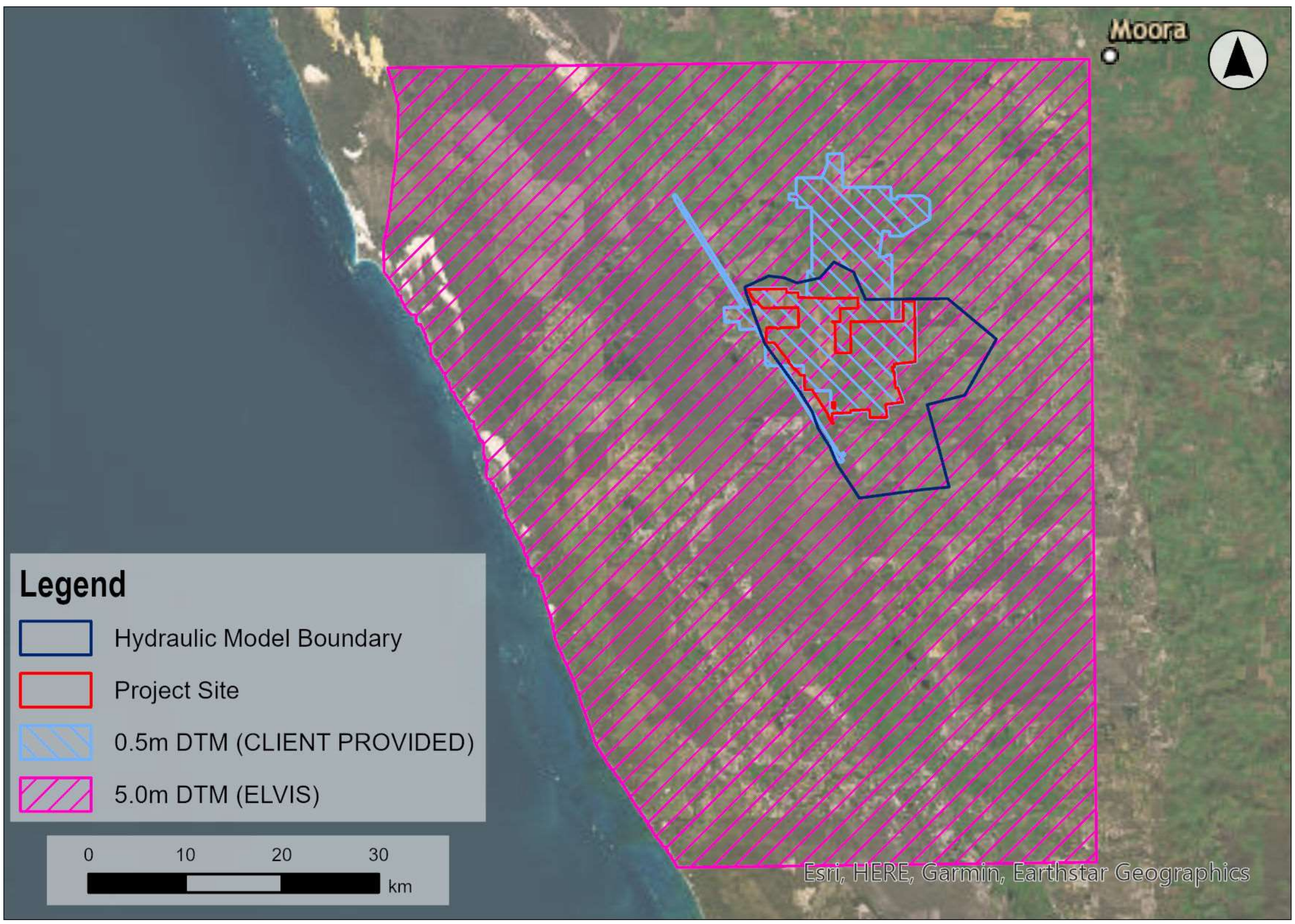

Figure 2-1 Digital Terrain Model (DTM) Coverage

2.3 Moore River Streamflow Data

Streamflow data for the Moore River – Quinns Ford catchment was obtained from the Department of Water and Environmental Regulation (DWER) gauging station 617001. The dataset provides a continuous 56-year record of daily maximum discharge values (in m³/s) from 6 May 1969 to 11 February 2025, comprising 19,947 records (refer Appendix I). Recorded discharge ranged from 0.005 m³/s to a peak of 446.453 m³/s, with an average daily discharge of 2.255 m³/s. The 95th percentile value was 9.481 m³/s, indicating the upper range of high-flow conditions. Flow patterns exhibit strong seasonal variation, with elevated discharges typically occurring during the winter months. The largest historical flood events are summarised in Table 2-1, which align closely with the modelling outputs and confirm that the channel exhibits flood immunity.

Table 2-1 Historical peak flood events at Quinns gauging station (617001)

Date	Peak Discharge (m³/s)	Notes
26 July 1983	270	Significant winter flood event
12 July 1995	318	High winter peak
22 March 1999	446	Largest recorded event

The data was used to assess historical flow behaviour, calibrate hydrological estimates, and understand catchment response during significant rainfall events. The distribution of peak annual flows is shown in Figure 2-2, adapted from the *Moora Flood Management Study* (Water & Rivers Commission, 2000). The recorded annual maximum discharges are compared with fitted Log-Pearson III and Log-Normal distributions, together with the 5% and 95% confidence limits. The close alignment of the statistical fits with the observed record confirms the suitability of the gauging dataset for calibration of design flood estimates. Adopted design discharges for this analysis from the Moore River at Quinn's Ford are summarised in Table 3-2.

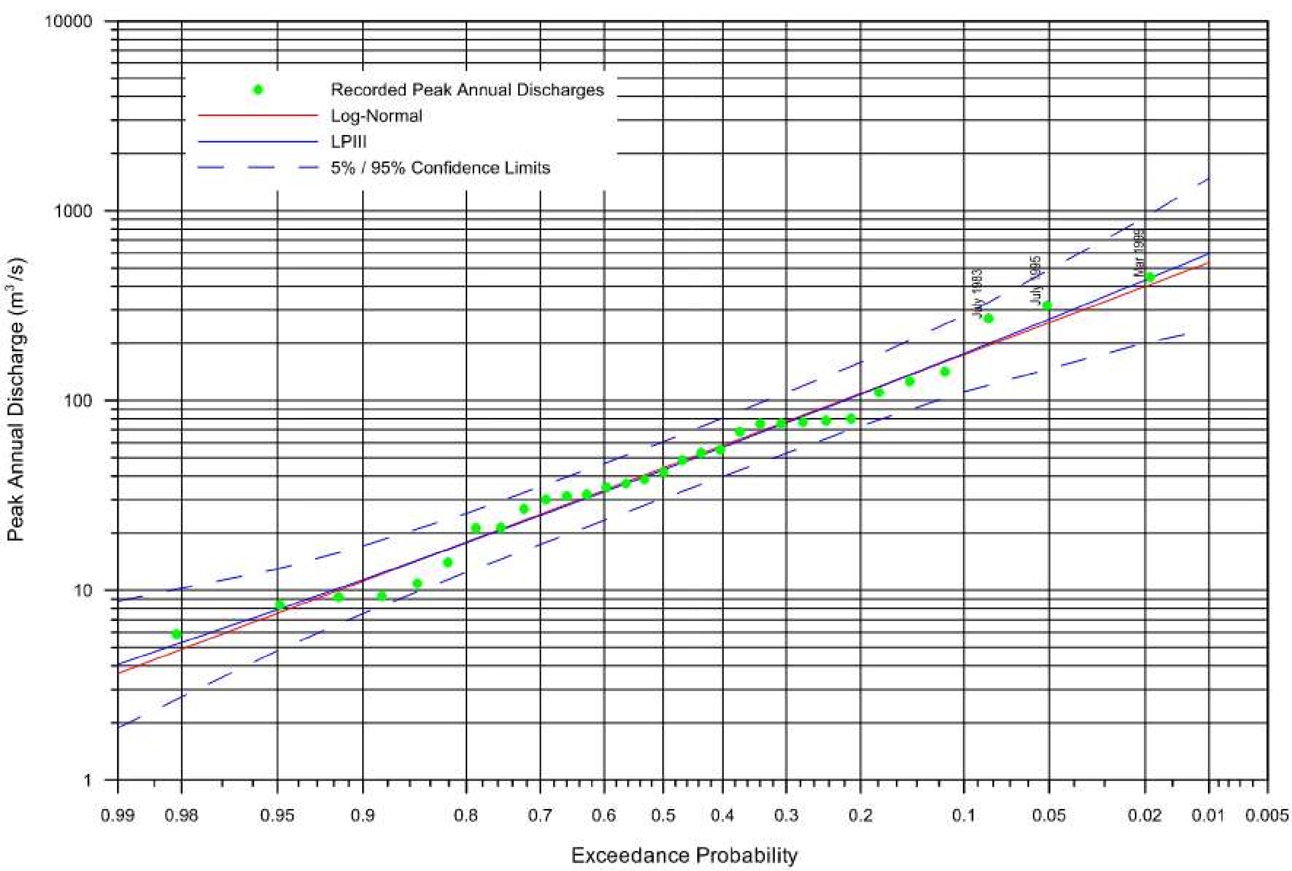

Figure 2-2 Distribution of Peak Annual Flows, Moore River at Quinn's Ford

3 Hydrology

This section outlines the hydrological inputs adopted for the study, comprising local catchment runoff generated within the hydraulic model boundary and external upstream inflows from Caren Caren Brook and the Moore River. The local catchment hydrology within the hydraulic model boundary is described in Section 3.1, while the derivation of external inflows is presented in Section 3.2. These external inflows were converted into hydrographs (QT – flow versus time) and applied to the hydraulic model as 2D boundary conditions.

3.1 Local Catchment Hydrology

The ARR to TUFLOW plugin was used to automatically incorporate ARR 2019 input parameters and Bureau of Meteorology (BOM) design rainfall data into the local catchment hydraulic model. The ARR 2019 hydrological methodology applies the ensemble approach, in which predefined areal temporal patterns are combined with Intensity Frequency Duration (IFD) rainfall data to generate ten storm patterns for each duration. All nominated design events were assessed over 10 minutes to 30-hours (1800 minutes) (refer Section 4.1).

As the catchment area exceeds 20 km², spatial variability of design rainfall was assessed in line with ARR 2019. Across the catchment, rainfall depths showed only minor variation (±5%) for the assessed durations and AEP events. Figure 3-1 shows the 720 minutes, 1% AEP rainfall depths across the hydraulic model domain. As this variation was not considered significant, point rainfall with no spatial pattern was adopted for the hydraulic modelling, The rainfall depths and temporal patterns extracted by the plugin were based on the catchment centroid at coordinates 30.9337°S, 115.745°E.

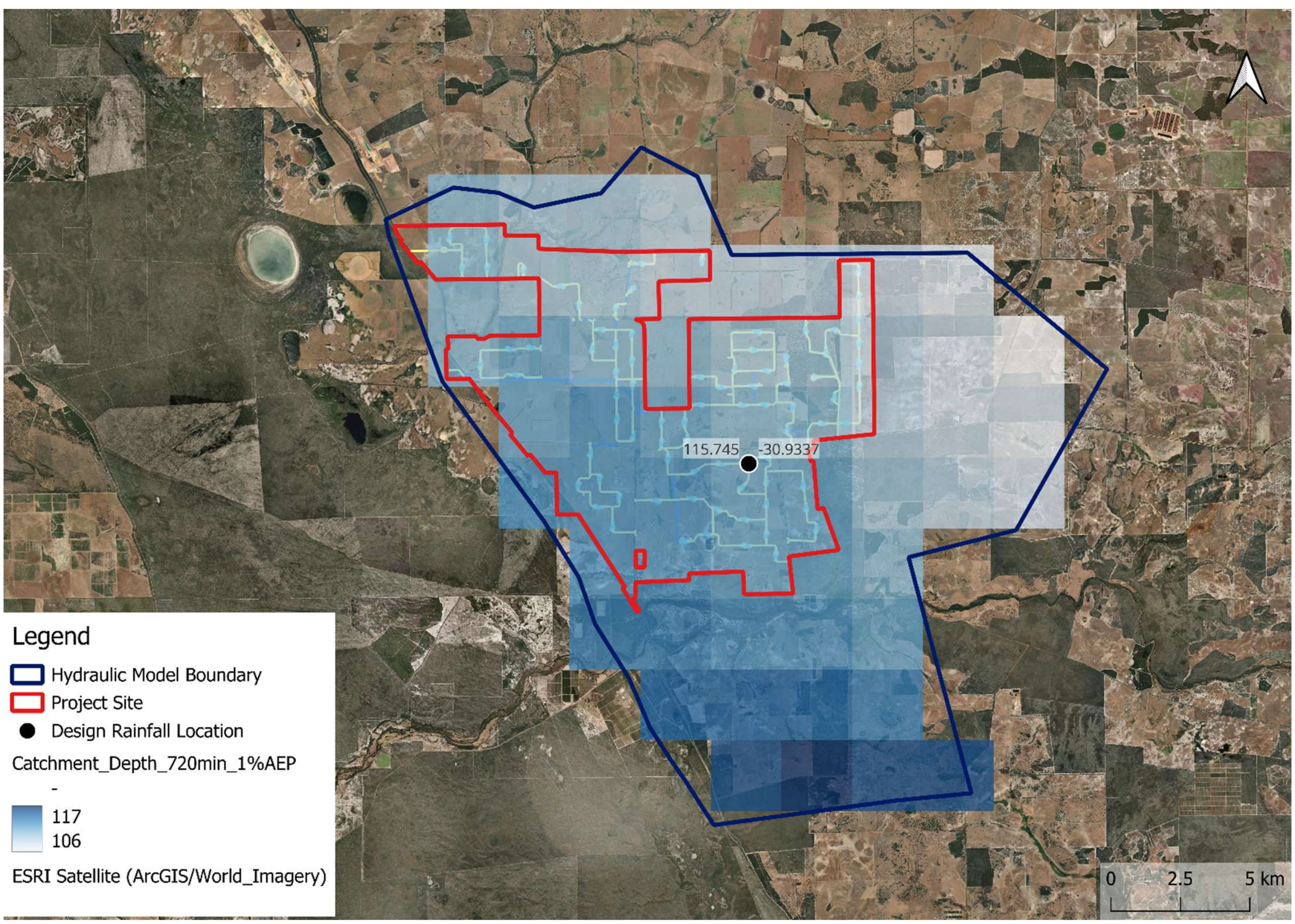

Figure 3-1 Spatial variation in BOM rainfall dept for 1% AEP 720 minutes

3.1.1 Rainfall Data

A rain on grid (ROG) approach is adopted in the hydraulic model to represent local catchment hydrology. In ROG modelling, rainfall is applied directly to each active 2D cell, so runoff forms on the ground surface in response to topography, land use/roughness and losses without a separate hydrology model.

This methodology aligns with ARR 2019 guidelines and is considered suitable for providing flood risk information within the site. Design rainfall depths for a range of event durations and probabilities extracted via ARR to TUFLOW plugin at the catchment centroid are presented in Figure 3-2.

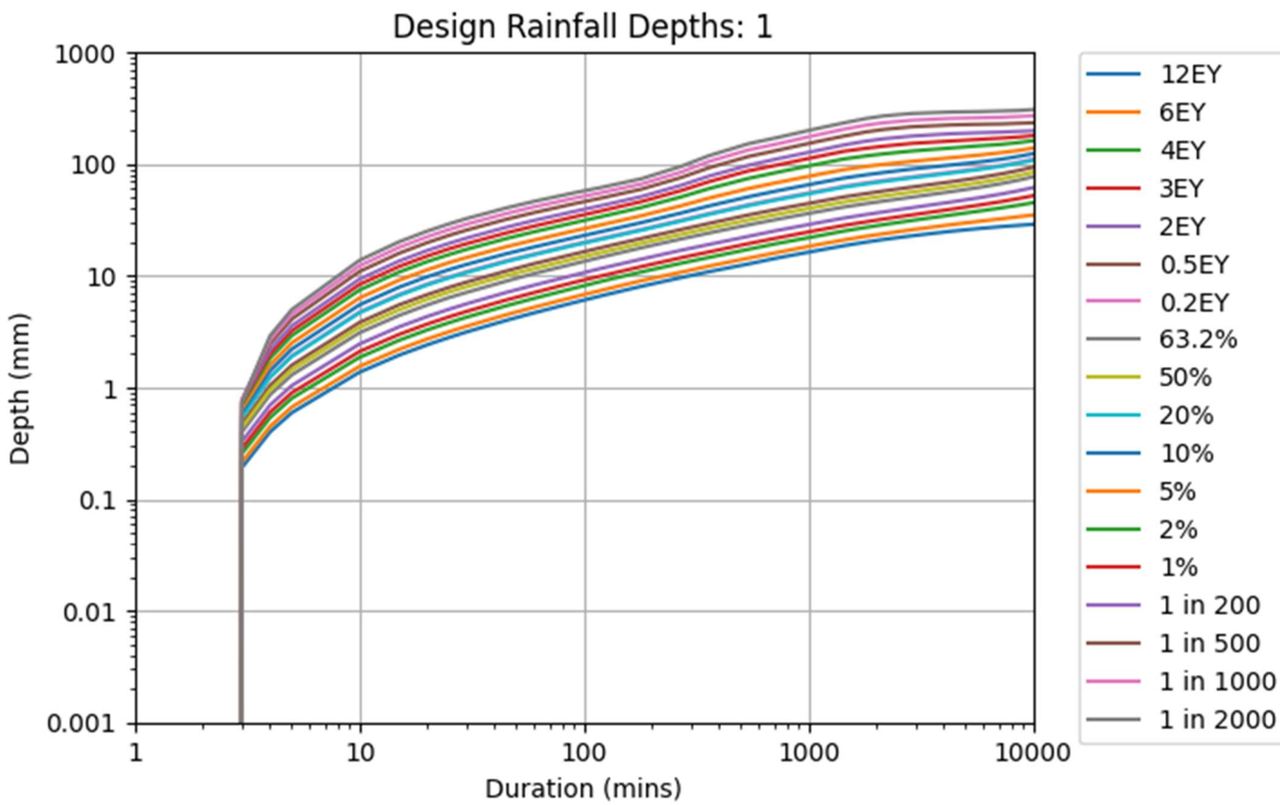

Figure 3-2 ARR Data Hub – Design Rainfall Depths

3.1.2 Rainfall Losses

In accordance with ARR 2019, rainfall losses and pre-burst adjustments were applied to ensure realistic hydrologic representation of the local catchment hydraulic model. Losses account for infiltration and temporary surface storage, with runoff estimated using the Initial Loss (IL) to represent rainfall absorbed before runoff commences, and Continuous Loss (CL) to capture ongoing infiltration and subsurface movement during storm events. These values reflect the sandy soils and variable antecedent moisture conditions typical of the project site. In addition, the median pre-burst depth was removed from rainfall inputs to isolate the main burst of rainfall that drives runoff, avoiding overestimation from preceding low-intensity rainfall. This approach ensures that the effective rainfall applied in the TUFLOW ROG modelling reflects realistic catchment responses across the full range of design events. Median Pre-burst depths and design initial losses extracted via ARR to TUFLOW plugin are shown in Figure 3-3 and Figure 3-4 respectively.

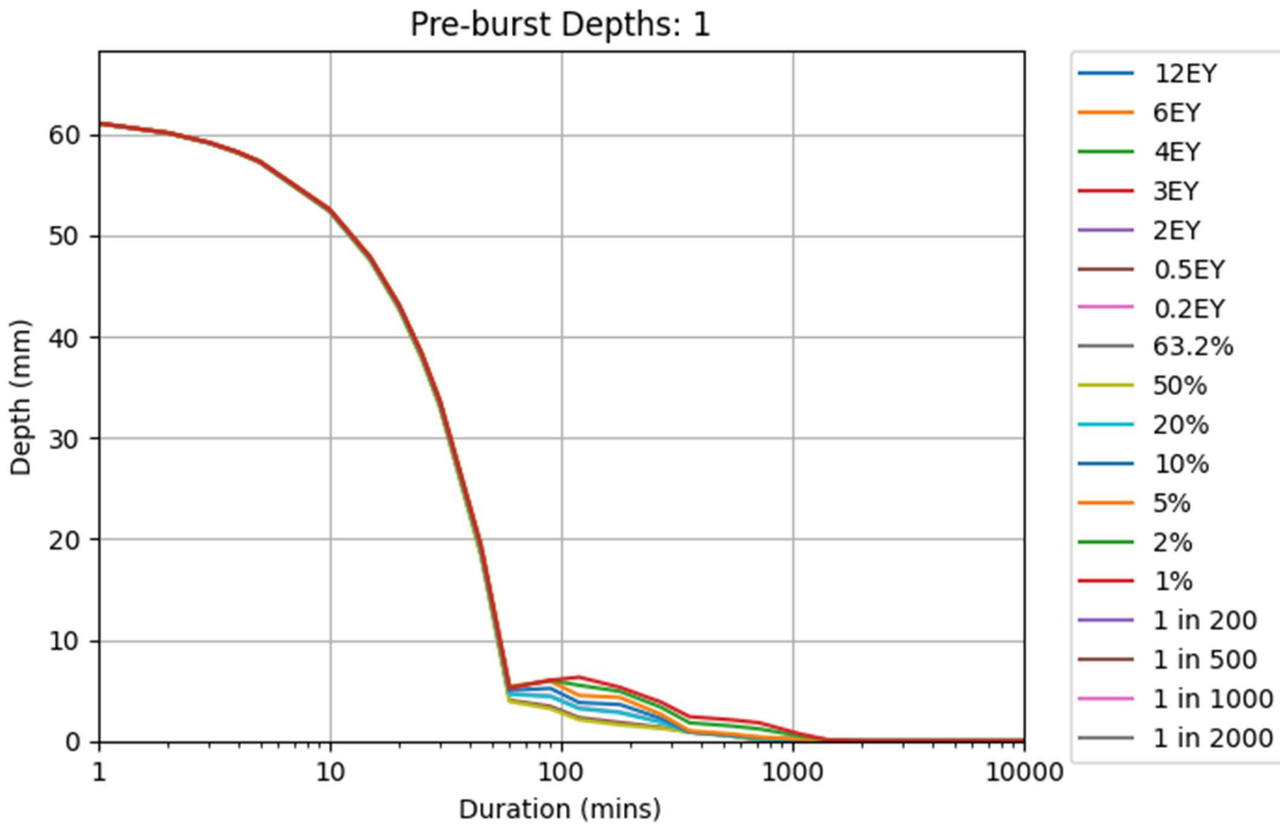

Figure 3-3 ARR Data Hub – Pre-Burst Depths

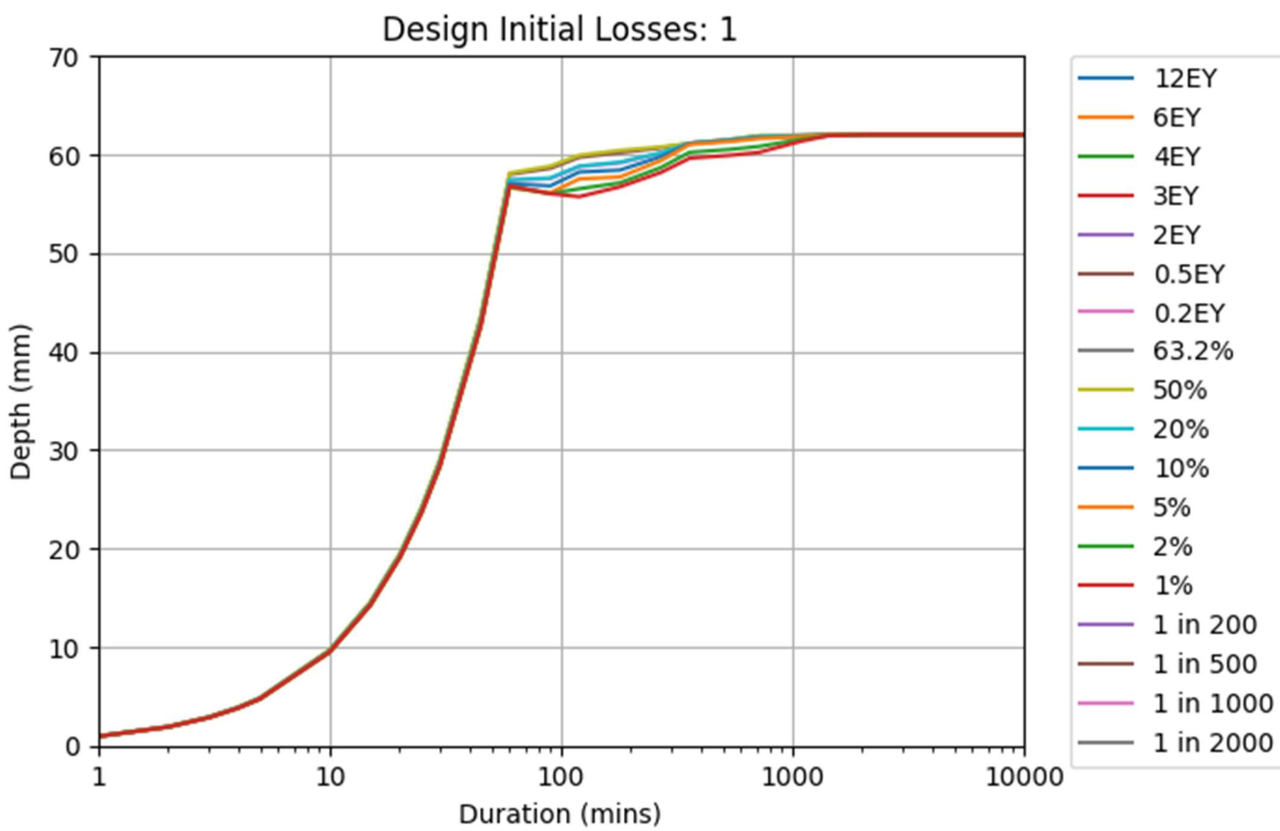

Figure 3-4 ARR Data Hub – Design Initial Losses

3.1.3 Land Use

Land use data for the hydraulic model was sourced from the Digital Earth Australia (DEA) Land Cover dataset (Landsat), refer Figure 3-5. The dominant land use types within the catchments are Cultivated Terrestrial Vegetation (CTV) and (Semi-) Natural Terrestrial Vegetation (NTV). Manning's roughness values adopted for each land use type are provided in Table 4-1.

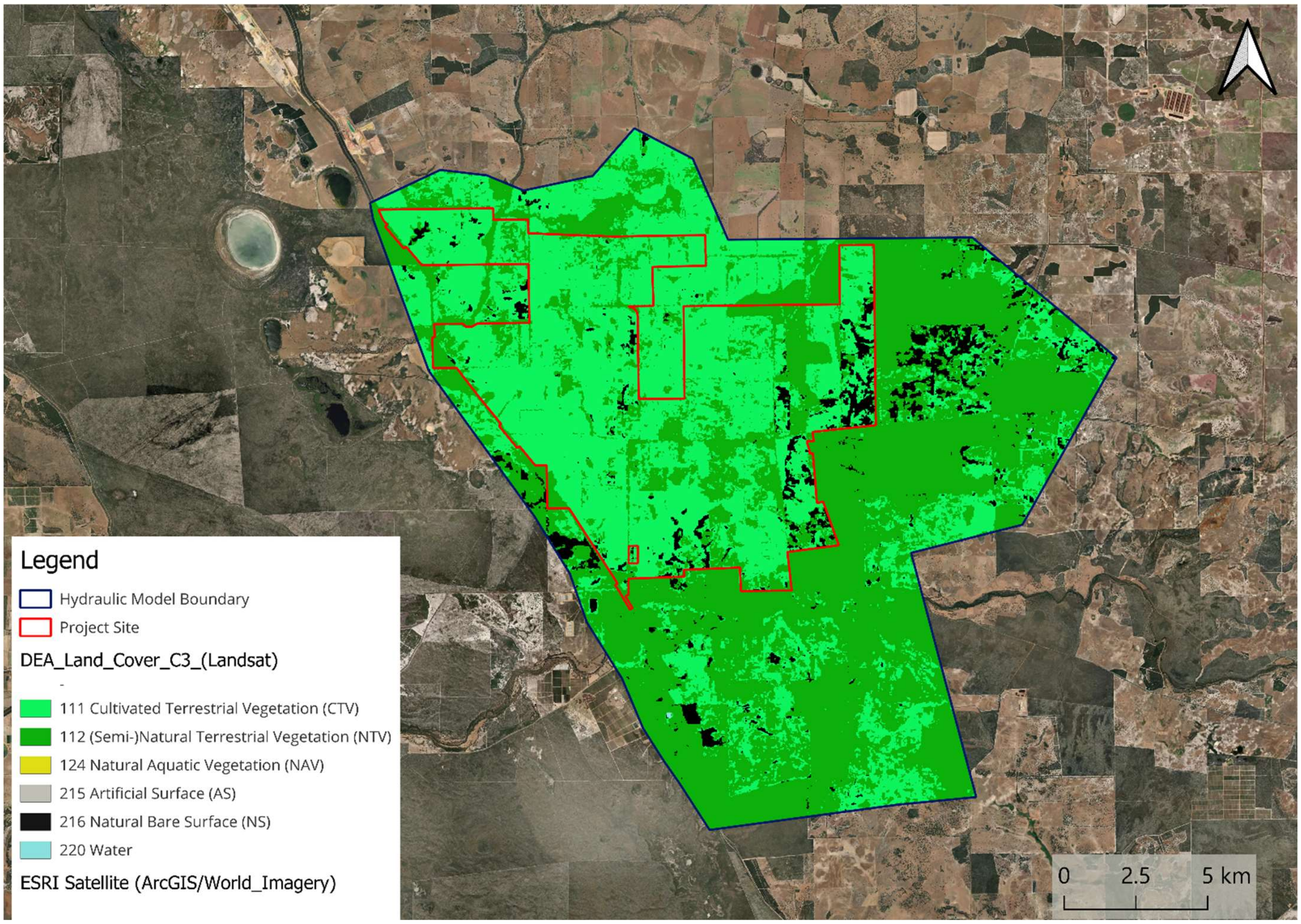

Figure 3-5 DEA Land Cover dataset (Landsat)

3.2 External Catchments

The project site is impacted by two external catchments at the upstream - Caren Caren Brook and Moore River. This section discusses the hydrology and inflow hydrographs generated from these catchments which were applied to the hydraulic model as 2D boundary conditions.

3.2.1 Caren Caren Brook Inflow Hydrographs

Caren Caren Brook traverses the northern side of the project site. Catchments, sub-catchments and flow paths were delineated from FABDEM terrain model using CatchmentSIM; the contributing area at the upstream hydraulic model boundary is approximately 314 km² (refer Figure 3-6).

Design inflow hydrographs were derived with RORB hydrological model, which applies rainfall, subtracts losses to obtain rainfall-excess, and routes this through catchment storages. Storm Injector was used to manage batch runs, plot results and identify critical durations, peak flows and hydrographs for input to the hydraulic model. In the absence of nearby gauging, the RFFE method (Appendix J) was used to calibrate the RORB kc routing parameter for each design events.

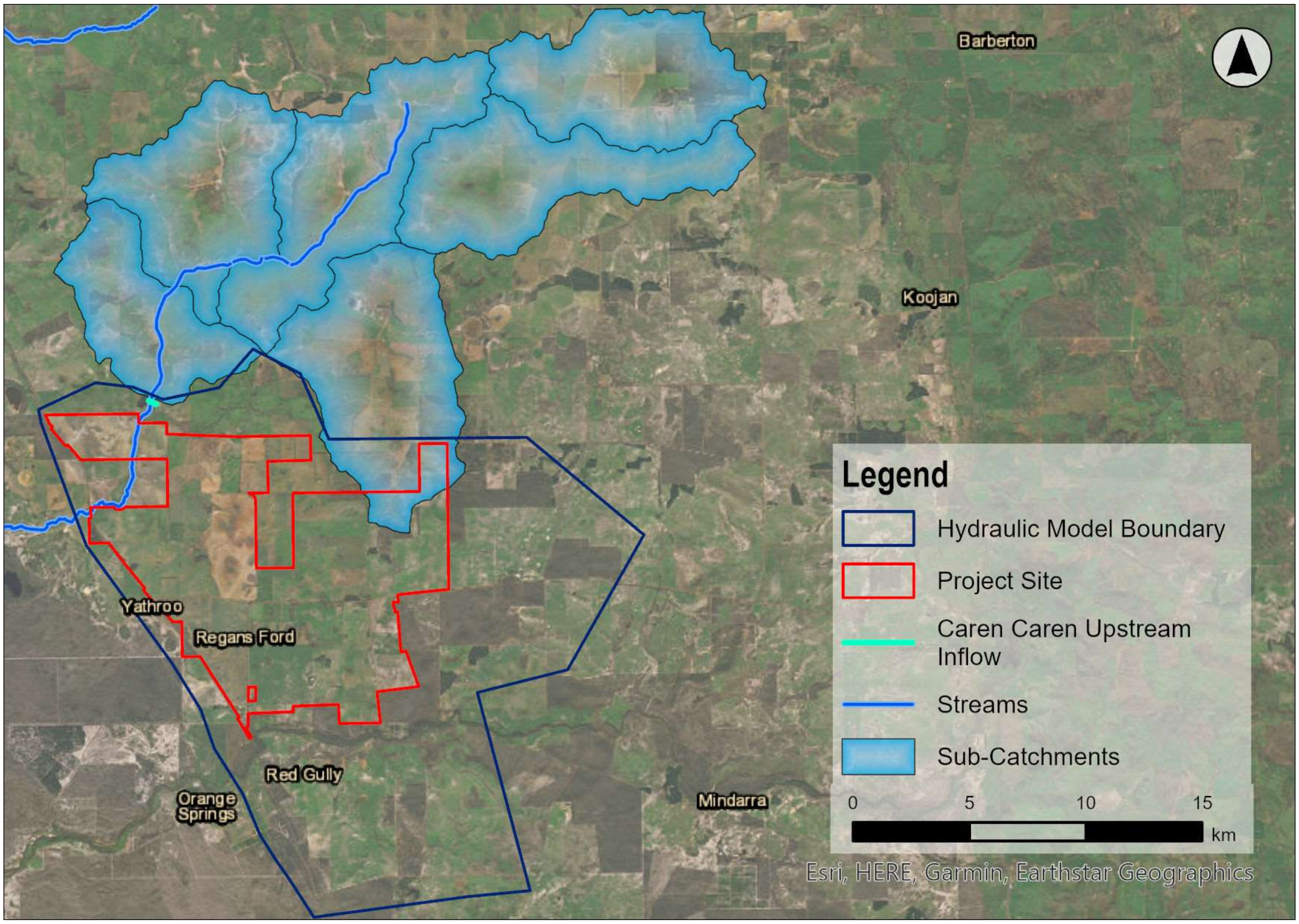

Figure 3-6 Catchment delineation of Caren Caren Brook using CatchmentSIM

Durations from 10 minutes to 144 hours (8640 minutes) were assessed; the critical duration for all design AEPs was 24 hours (1440 minutes), which lies within the hydraulic model duration window (10 minutes to 30 hours / 1,800 minutes). The boundary inflow at the hydraulic model entry was generated using the median temporal pattern for the duration range and applied uniformly across the ten ROG temporal-patterns. The 24 hours inflow hydrographs by AEP are shown in Figure 3-7, and peak flows at the 1440 minutes critical duration are summarised in Table 3-1.

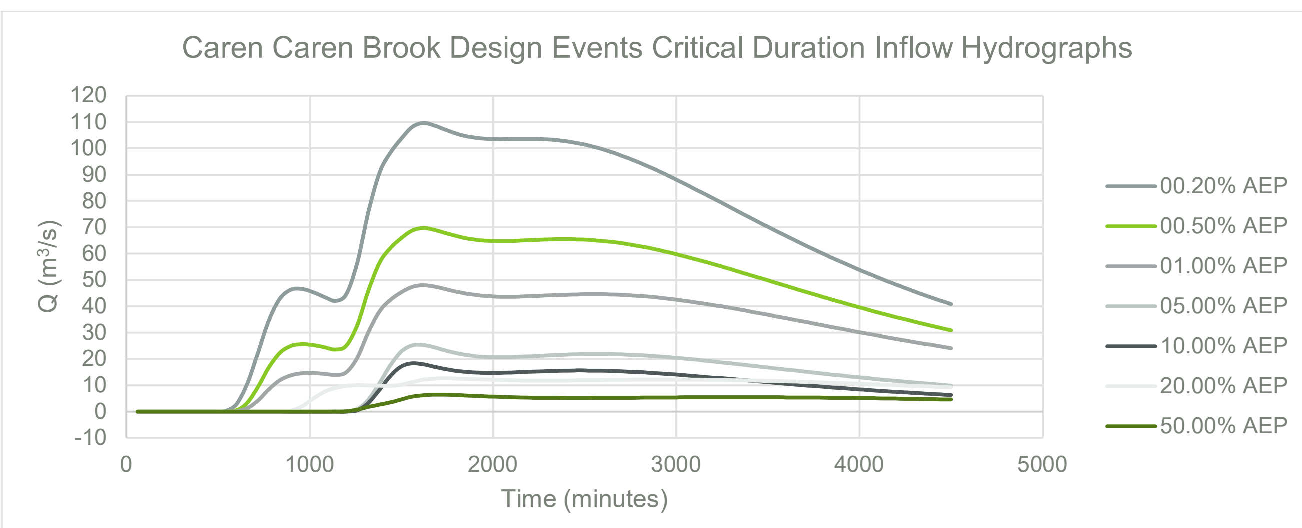

Figure 3-7 Caren Caren Brook Design Events Critical Duration Inflow Hydrographs

Table 3-1 Peak Flows by AEP from Caren Caren Brook Upstream

Design Event (AEP)	Peak Flow (m³/s)	Critical Duration (minutes)
50% AEP	6.41	1440
20% AEP	12.6	1440
10% AEP	18.3	1440
5% AEP	25.31	1440
1% AEP	48	1440
0.5% AEP	69.72	1440
0.2% AEP	109.6	1440

3.2.2 Moore River

Moore River traverses east-west south of the project site and does not cross the site. However, given its proximity, the river's flooding behaviour is relevant to the assessment. A stream gauging station with calibrated peak flows for design events are available upstream of the project site (refer section 2.3), and is suitable for this study's hydraulic inputs. The contributing area at the upstream hydraulic model boundary is approximately 11,400 km² (refer Figure 3-8).

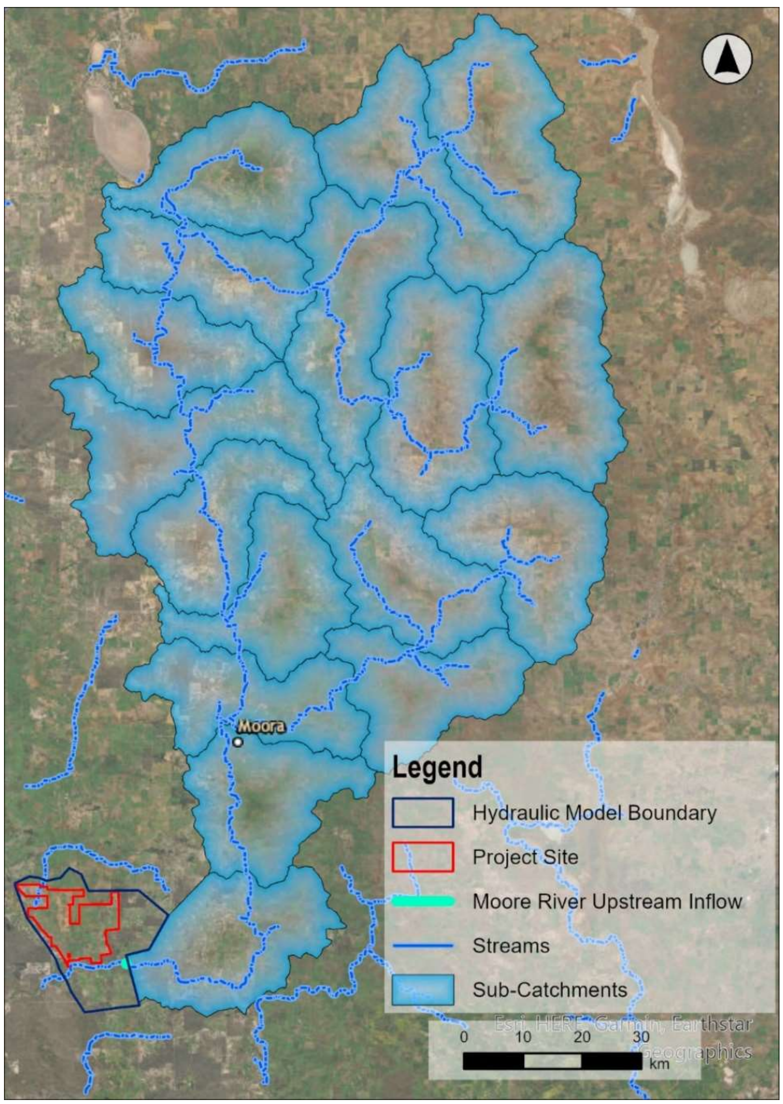

Figure 3-8 Catchment delineation of Moore River

The adopted peak flows are summarised in Table 3-2, adapted from the *Moora Flood Management Study* (Water & Rivers Commission, 2000). For conservatism, a constant (steady) hydrograph equal to the peak flow was applied at the Moore River inflow boundary across the hydraulic model's design duration range.

Table 3-2 Peak Flows for Moore River at Quinn's Ford Ford

Design Event (AEP)	Peak Flow (m^3/s)	Duration (minutes)
50% AEP	57	1440
20% AEP	145	1440
10% AEP	229	1440
5% AEP	331	1440
1% AEP	584	1440
0.5% AEP	653.105	1440
0.2% AEP	776.804	1440

3.3 Climate Change Considerations

This study uses baseline ARR 2019 design rainfall intensities only; climate change adjustments have not been applied as the objective is to describe pre-development flood behaviour.

According to ARR (2019; Book 1, v4.2), rainfall intensities are projected to increase under climate change. As an illustrative benchmark (for the 1EY event, ~63.2% AEP), indicative scaling factors are ~15% per °C for 1-hour and ~8% per °C for 24-hour durations. In practice, project site specific factors are obtained from the ARR Data Hub to produce AEP-specific climate-adjusted IFDs for design applications.

For a high emissions scenario leading to ~2 °C of warming by 2100, short-duration design rainfall could increase by up to ~30%, while long-duration design rainfall could increase by up to ~15%. These changes would likely result in higher peak discharges, expanded flood extents, and elevated hazard categories across the project site.

To inform sensitivity for later design phases, Figure 3-9 summarises ARR Data Hub climate-change rainfall factors for the project site under SSP2-4.5 and SSP3-7.0 (SSP = Shared Socioeconomic Pathway; coupled socio-economic and emissions trajectories used in CMIP6 climate projections; the "4.5" and "7.0" indicate approximate radiative forcing levels in 2100, in W/m^2) across Medium-term and Long-term horizons. These factors scale baseline IFDs by duration. Consistent with ARR guidance, short-duration storms show the largest uplifts, and higher-emissions/longer-term scenarios produce larger increases.

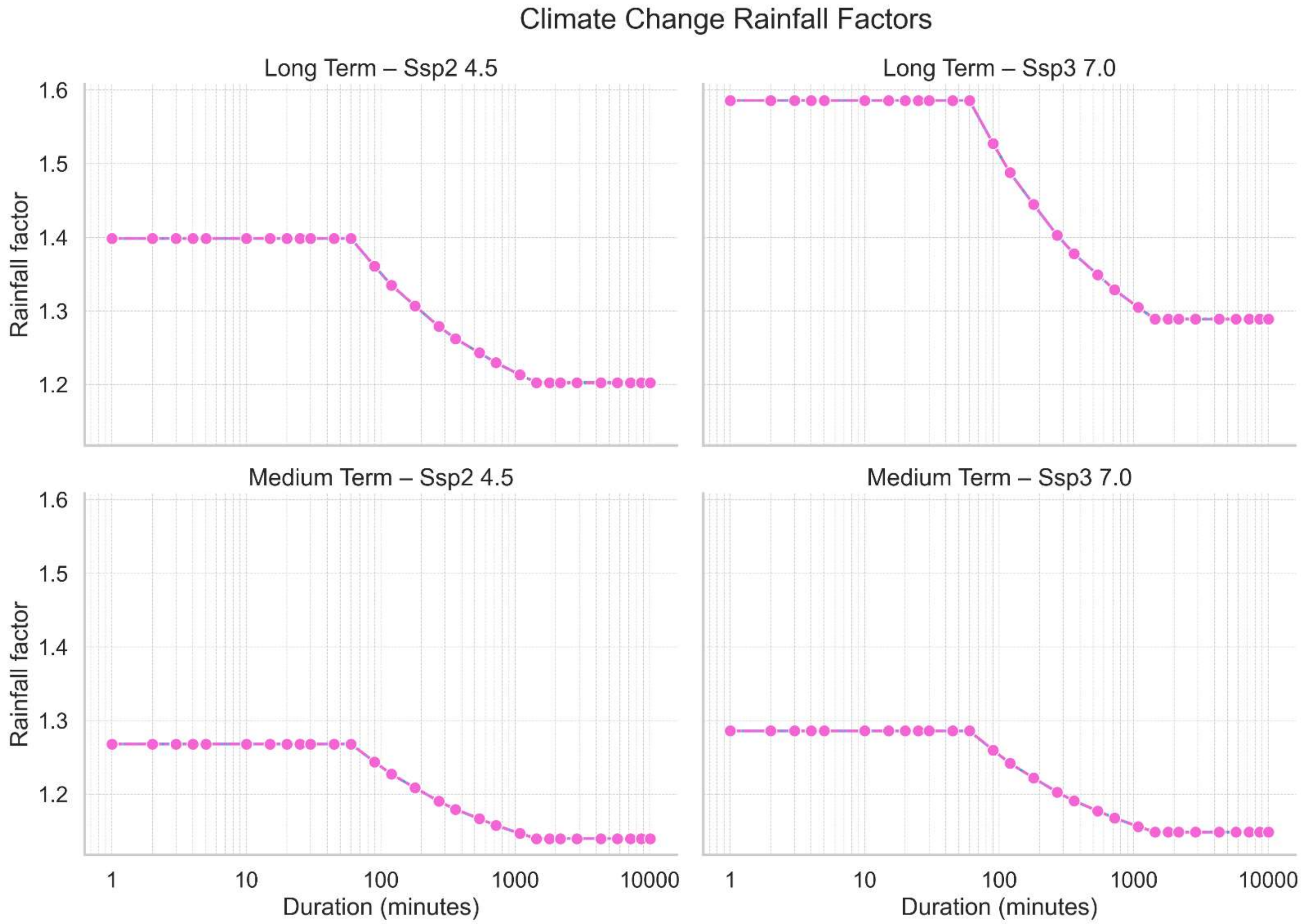

Figure 3-9 ARR Data Hub - Climate change rainfall factors by duration (SSP2-4.5 & SSP3-7.0; Medium- and Long-term)

Figure 3-10 and Figure 3-11 translate those uplifts into an indicative "equivalent AEP" view that expresses how the frequency of a fixed present-day design depth would change under climate-adjusted IFDs. In these plots, for a depth equal to today's 10% AEP, the equivalent AEP generally increases, with the largest shifts around 30–90 minutes; for a depth equal to today's 1% AEP, the equivalent AEP also increases, with Long-term SSP3-7.0 showing the greatest change—again most pronounced at short durations.

While no climate change was applied in this assessment, subsequent phases should consider

- scaling IFDs using ARR Data Hub factors for the selected scenario/horizon, or
- adopting a rarer baseline AEP to maintain an equivalent risk level under climate-adjusted conditions.

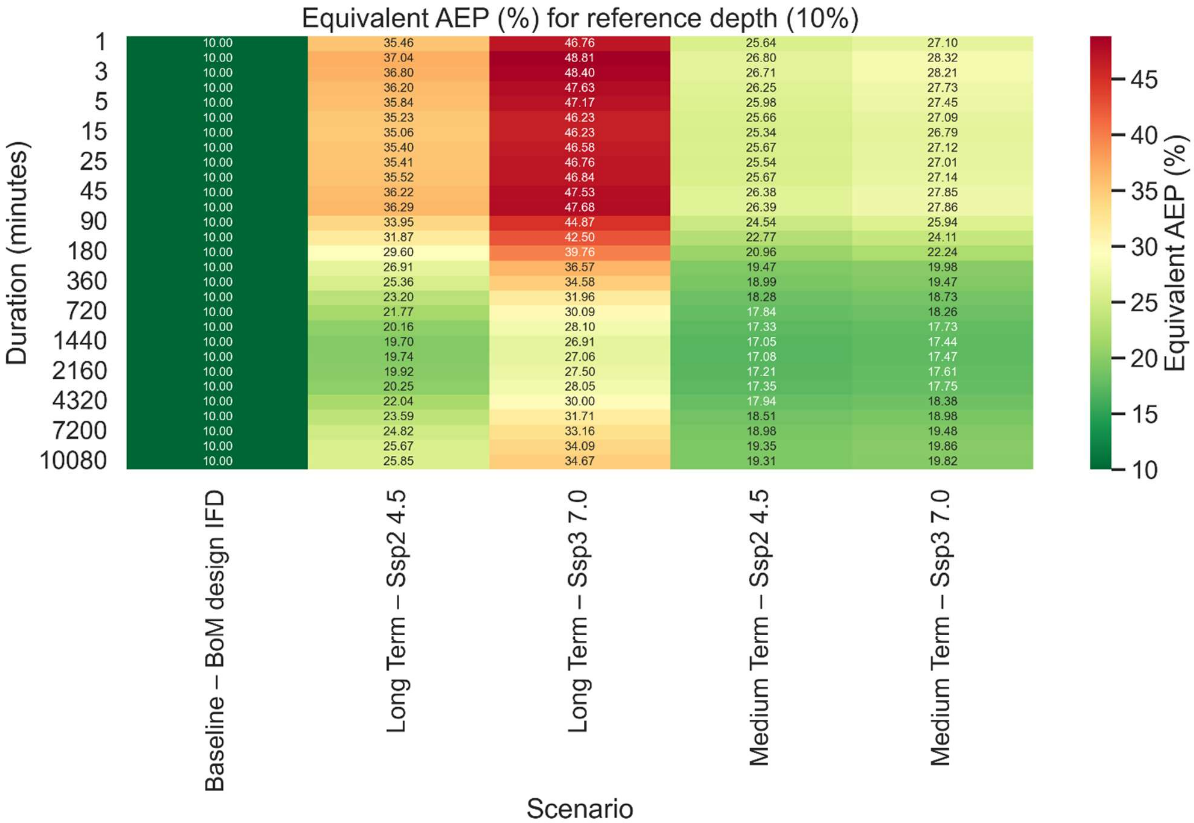

Figure 3-10 Equivalent AEP (%) for today's 10% AEP depth—indicative translation using ARR-adjusted IFDs

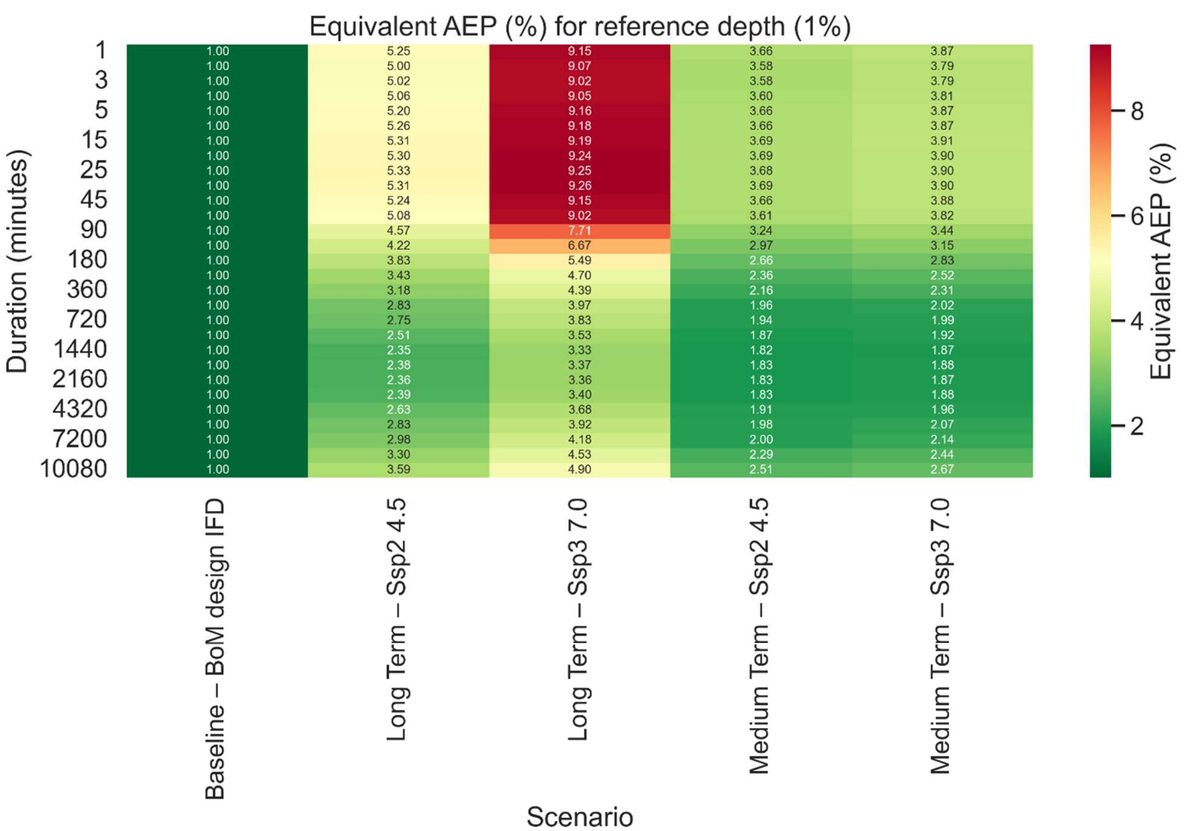

Figure 3-11 Equivalent AEP (%) for today's 1% AEP depth—indicative translation using ARR-adjusted IFDs

4 Hydraulics

Rainfall–runoff within the model extent (refer Figure 4-1) was simulated using the ROG method described in Section 3.1.1. External inflows from Caren Caren Brook and the Moore River were applied as 2D boundary conditions (refer Section 3.2).

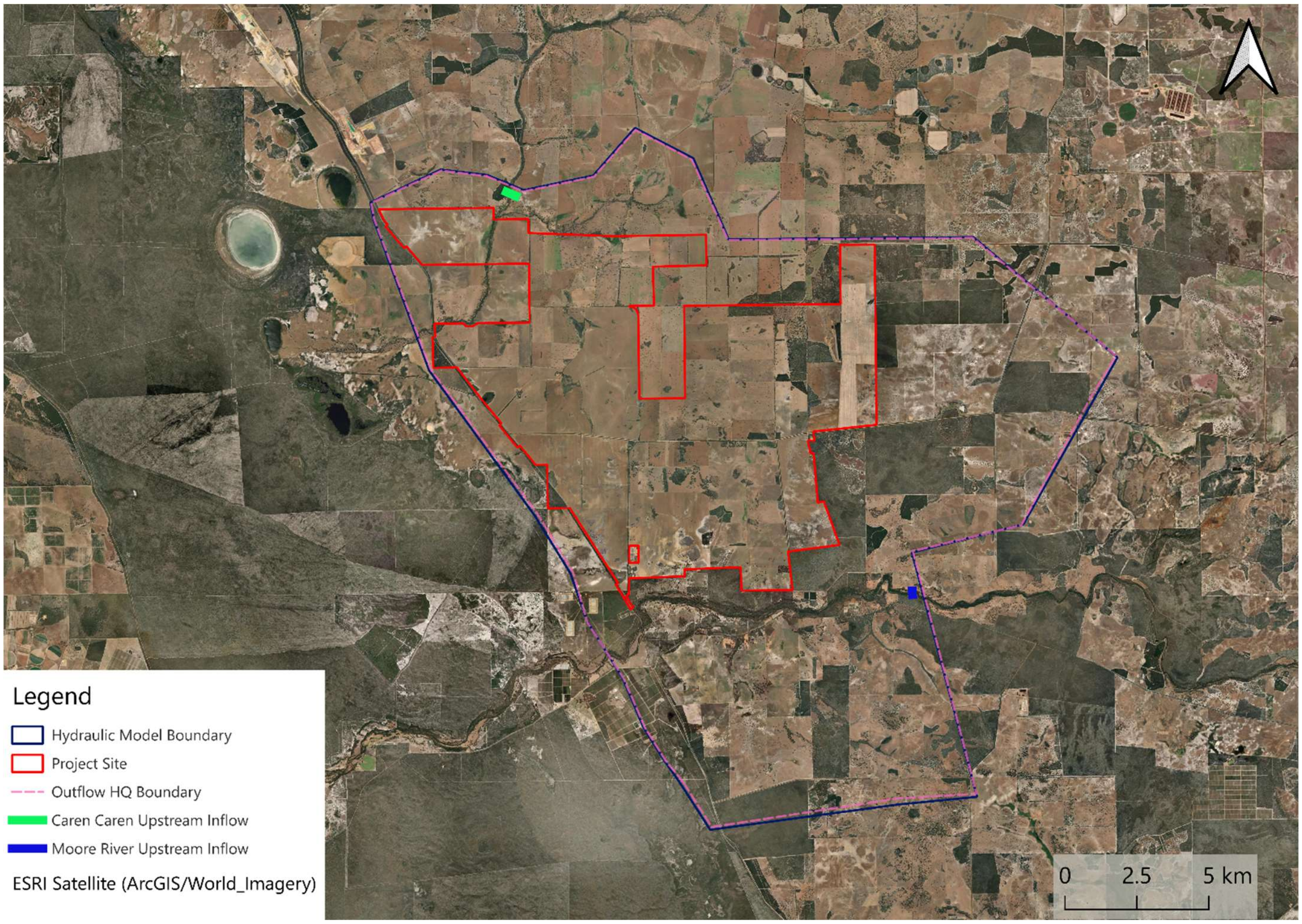

Figure 4-1 TUFLOW Model Extent – Hydraulic Model Boundary

4.1 TUFLOW Model Setup

A 1D/2D hydraulic model was developed in TUFLOW using version 2023-03-AF with the HPC GPU solver. TUFLOW is a computer program for simulating depth-averaged, one and two-dimensional free-surface flows such as occurs from floods and tides, with the 2D solution occurring over a regular grid of square elements.

The modelling represents existing ground conditions only and does not include any changes to topography, drainage, or runoff that may result from the proposed project.

The model incorporates a combination of the client-supplied 0.5 m LiDAR DTM and 5 m LiDAR DTM sourced from ELVIS. After a sensitivity analysis, the model adopts a 30 m grid with sub-grid scale sampling at 7.5 m sample distance. At this resolution, channel definition and other features are well represented in the model for the purposes of flood risk mapping.

Material roughness was applied by digitising the areas based on the DEA Land Cover (Landsat) data set and visual inspection of aerial imagery.

This section discusses the parameters and information used to develop the hydraulic model. The key model parameters are summarised in Table 4-1 below:

Table 4-1 TUFLOW Model Parameters Summary

MODEL PARAMETERS	VALUE	COMMENTS
Model Topography	0.5 m LiDAR DTM and 5 m LiDAR DTM for the area outside 0.5 m DTM	0.5m DTM provided by the client. 5m DTM sourced from ELVIS
Model Solver	HPC adaptive timestep using GPU	
Grid Size and Sub Grid Sampling (SGS)	30m	SGS Sample Frequency = 5
Inflow Boundaries	2d_rf boundary 2d_bc QT boundary	The 2d_rf boundary applies rainfall across the site. External catchment inflows from Caren Caren Brook and Moore River.
Outflow Boundaries	2d_bc HQ boundary.	The distances are sufficiently far from the areas of interest to avoid boundary impacts.
1-d network	1d_nwk for culverts along Brand Highway was utilised.	Refer to Section 4.3
2d_bg	2d_bg was used to include the two bridges on Brand Highway.	Refer to Section 4.4
Manning's Roughness Assumption	Pasture: 0.06 Roads: 0.025 Buildings: 0.1 Ponds and other water: 0.03 Vegetated creek: 0.08 Cultivated Terrestrial Vegetation (CTV): 0.06 (Semi-)Natural Terrestrial Vegetation (NTV): 0.065 Natural Aquatic Vegetation (NAV): 0.05 Artificial Surface (AS): 0.025 Natural Bare Surface (NS):0.03 Water: 0.01	
Durations and Temporal Patterns	Various durations from 10 minutes to 30 hours (10m, 20m, 25m, 30m, 45m, 1h, 1.5h, 2h, 3h,4.5h, 6h, 9h, 12h, 18h, 24h and 30h) for the modelled AEP design rainfall events.	The model was run for all standard Point Temporal Patterns for all durations.

4.3 Culverts

The downstream culverts under Brand Highway were modelled as part of the 1D hydraulic network. As these culverts were not field surveyed, their configuration was based on reasonable assumptions regarding invert levels, sizes, and grades. These assumptions were informed through a combination of aerial imagery and analysis of the Digital Terrain Model (DTM). The approximate locations of the culverts were identified using spatial datasets from the Main Roads WA Open Data portal. Refer to Figure 4-2, where the culvert locations are represented as red dots along the western boundary of the site under Brand Highway. The two bridges on Brand Highway are shown as pink stars.

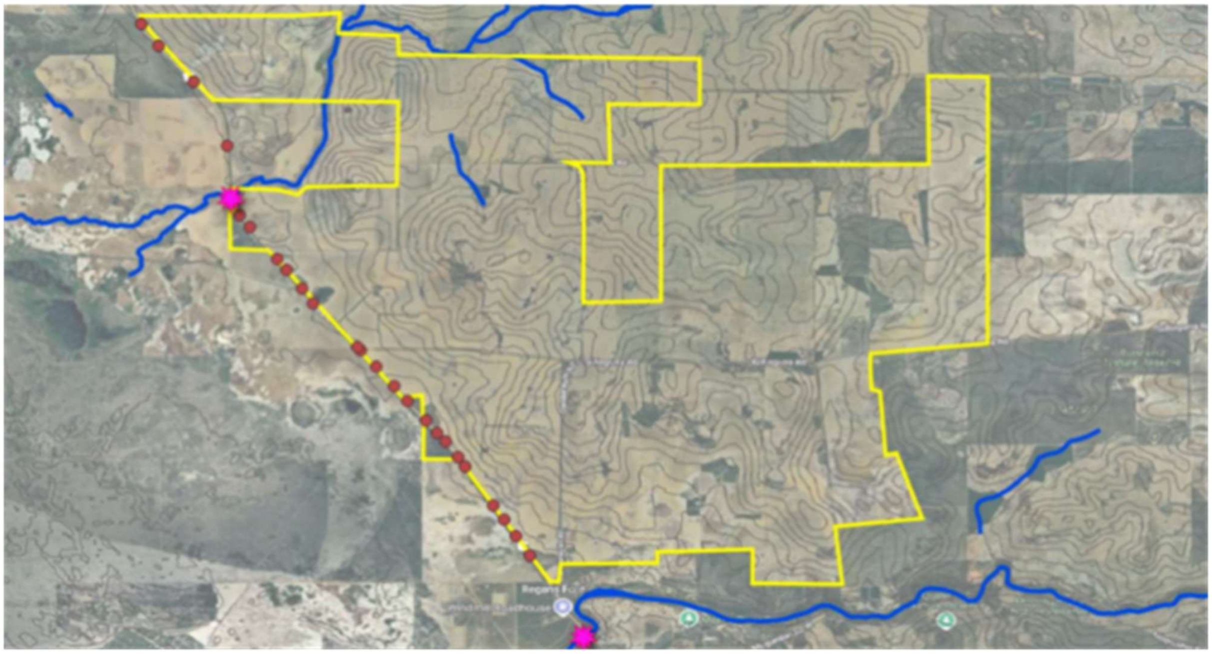

Figure 4-2 Culverts and Bridges on Brand Highway

4.4 Bridge Hydraulics

The two bridge structures were represented as 2D elements within the TUFLOW model using the 2d_bg layer. As with the culverts, no field survey data was available for the bridges. Therefore, their geometry and configuration were based on reasonable assumptions informed by aerial imagery and DTM analysis.

4.5 Existing Flood Regime

The Marri Wind Farm site lies within a rural catchment characterised by undulating terrain, broad overland flow paths, and intermittent natural drainage features. The project site is traversed by two major watercourses — Caren Caren Brook and the Moore River — which represent the primary surface water conveyance channels across the project site (refer section 3.2).

Under existing conditions, surface runoff generated across the project site is generally unconfined and drains toward the Brand Highway, which forms the western boundary of the study area. Drainage beneath the highway is facilitated via a network of MRWA culverts and two major bridge structures (refer section 4.3 and 4.4) that cross Caren-Caren Brook and the Moore River. These structures represent key hydraulic control points influencing downstream flow conditions and flood behaviour.

5 Modelling Results

The results for the 1% AEP flood event are discussed in this section. The flood mapping outputs for the 50%, 20%, 10%, 5%, 1%, 0.5% and 0.2% AEPs are included in Appendix A to Appendix H.

As noted in Table 4-1, the model was run for a range of durations and temporal patterns. The results were processed to create maximum median flood maps, which display the maximum results from the durations for the median values from the temporal patterns. The results in the flood maps show the critical design flood parameters at all locations across the modelled area.

The peak flood depth for the 1% AEP flood event is shown in Figure 5-1 below. This figure shows the access roads and hardstands which will be impacted by flood water.

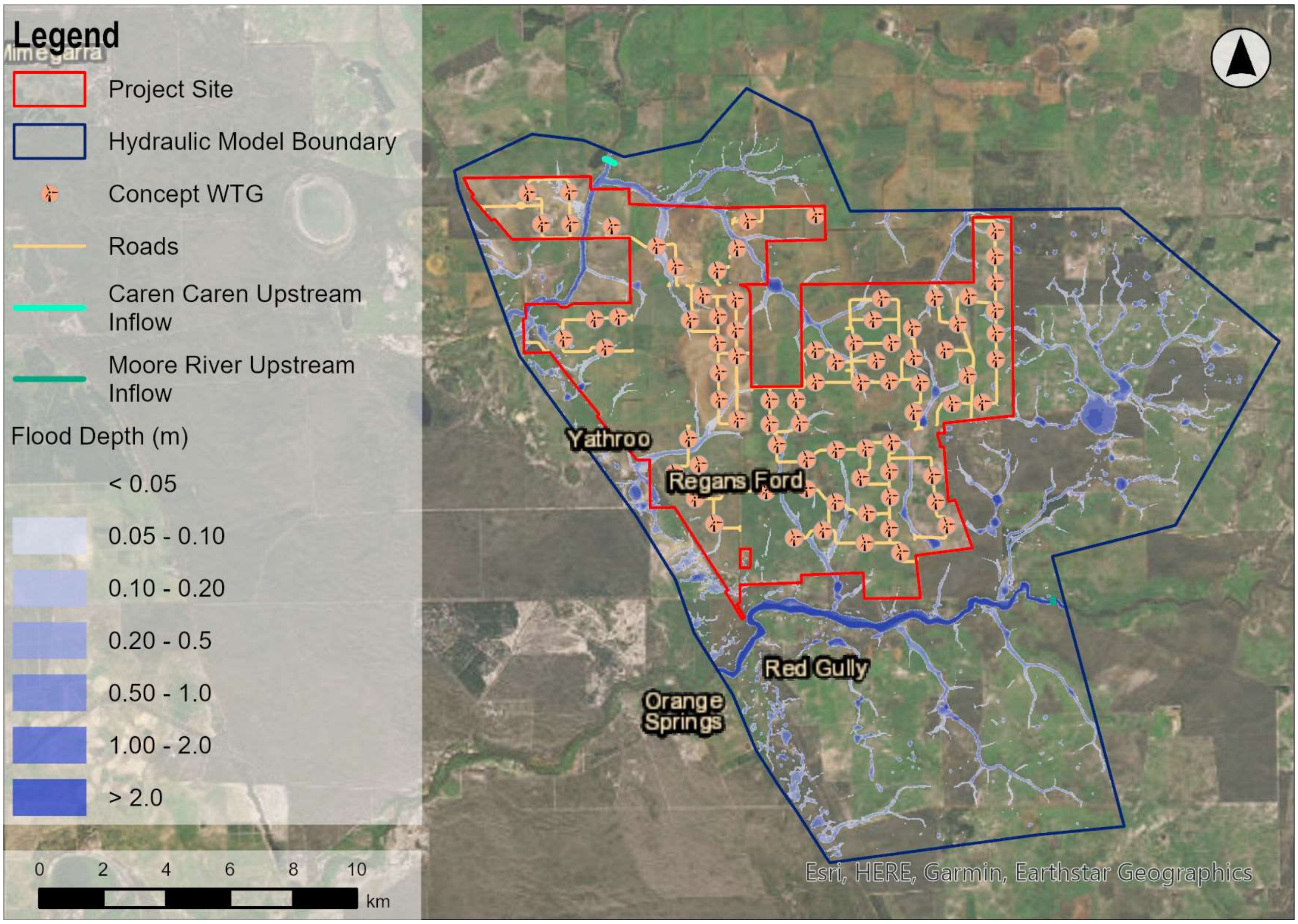

Figure 5-1: Peak Flood Depth for 1% AEP for the Existing Condition

The Peak flood level, velocity, hazard category and Bed Shear Stress (BSS) for the 1% AEP are also shown in Figure 5-2 to Figure 5-5 below.

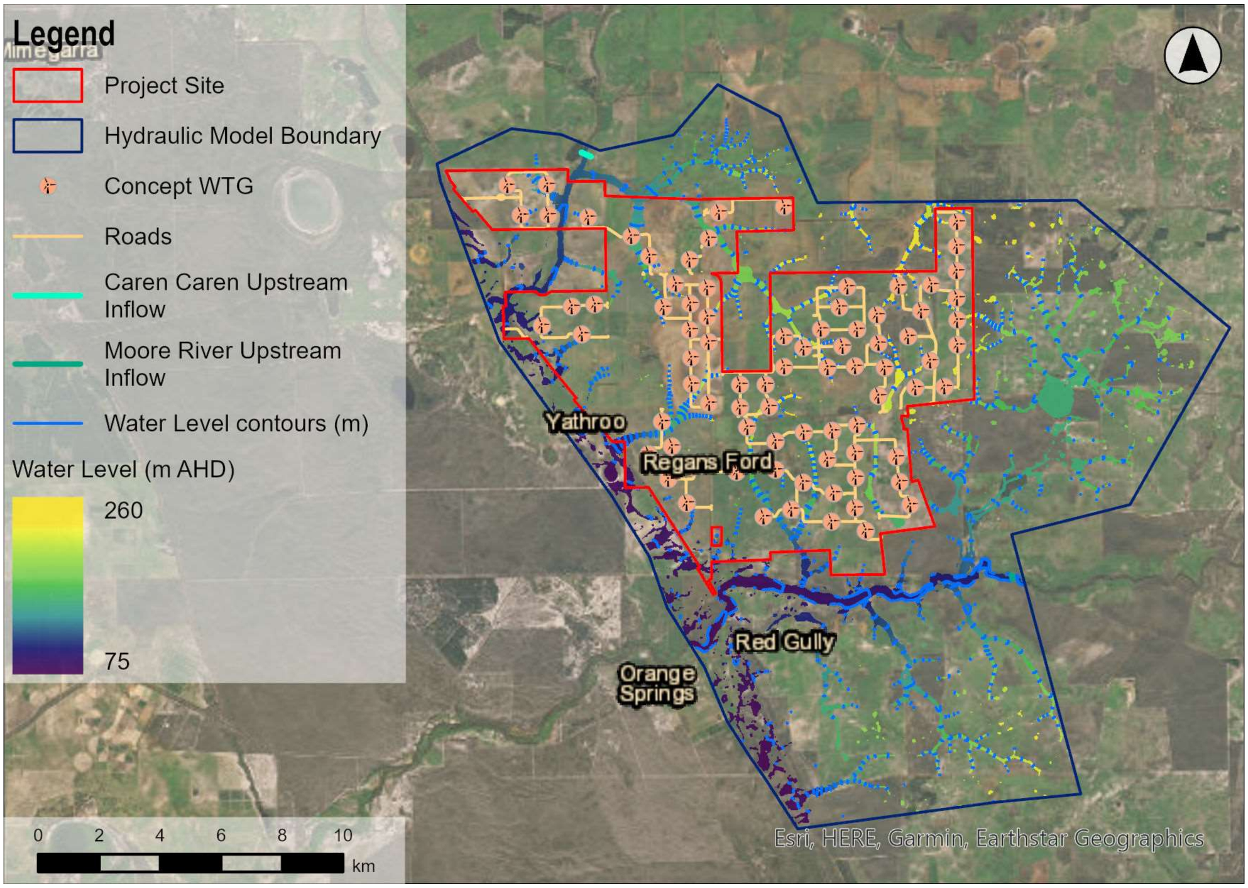

Figure 5-2: Peak Flood Level for 1% AEP for the Existing Condition

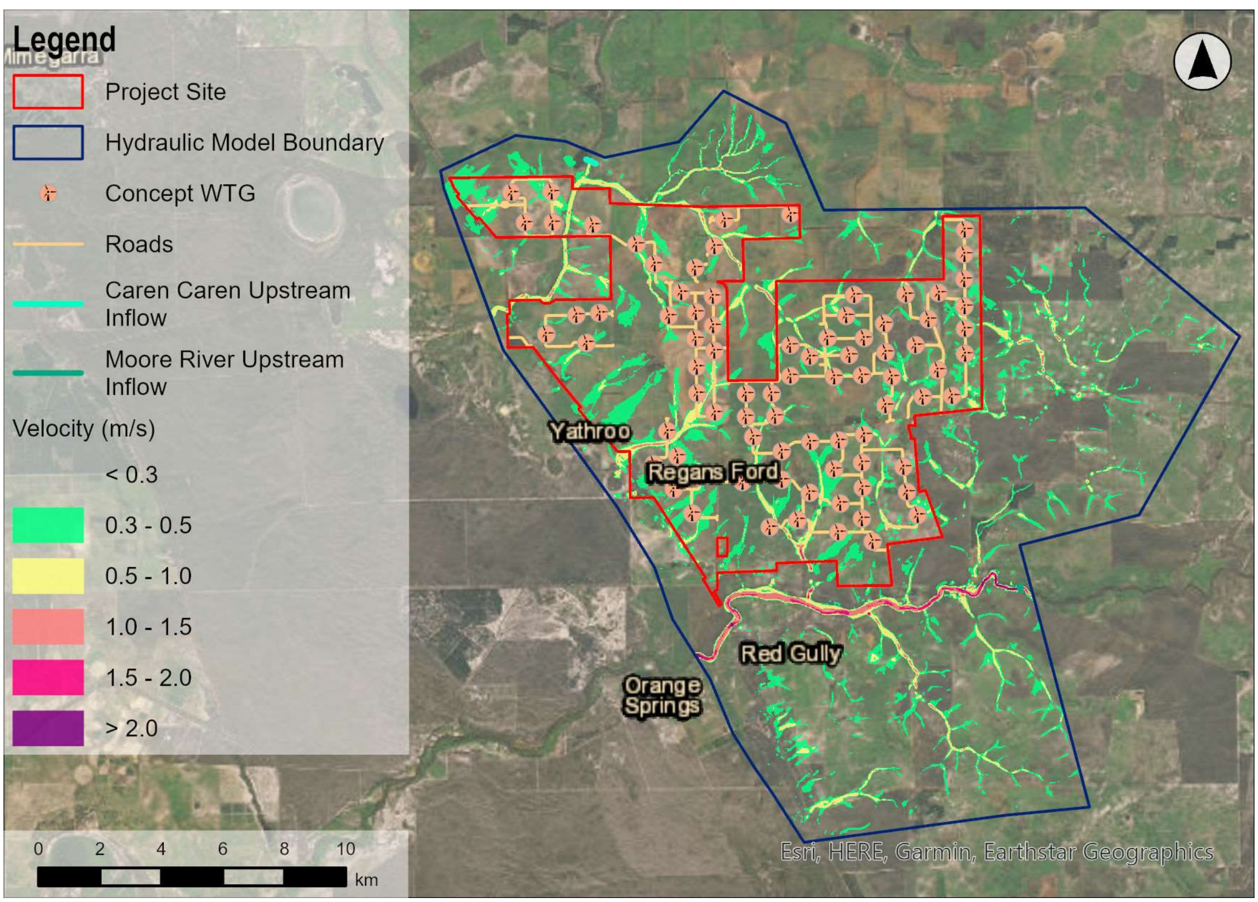

Figure 5-3 Peak Flood Velocity for 1% AEP for the Existing Condition

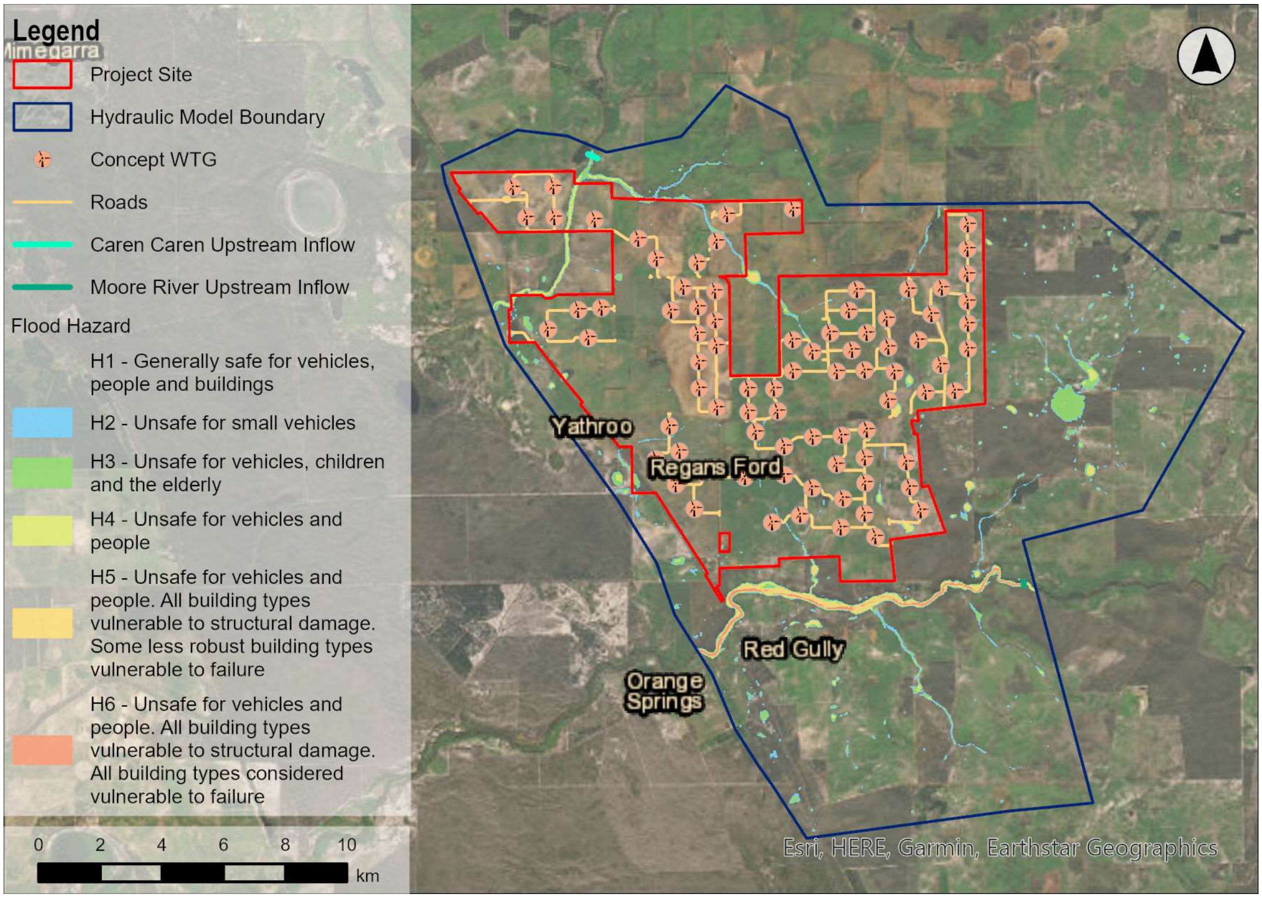

Figure 5-4 Peak Flood Hazard Category for 1% AEP for the Existing Condition

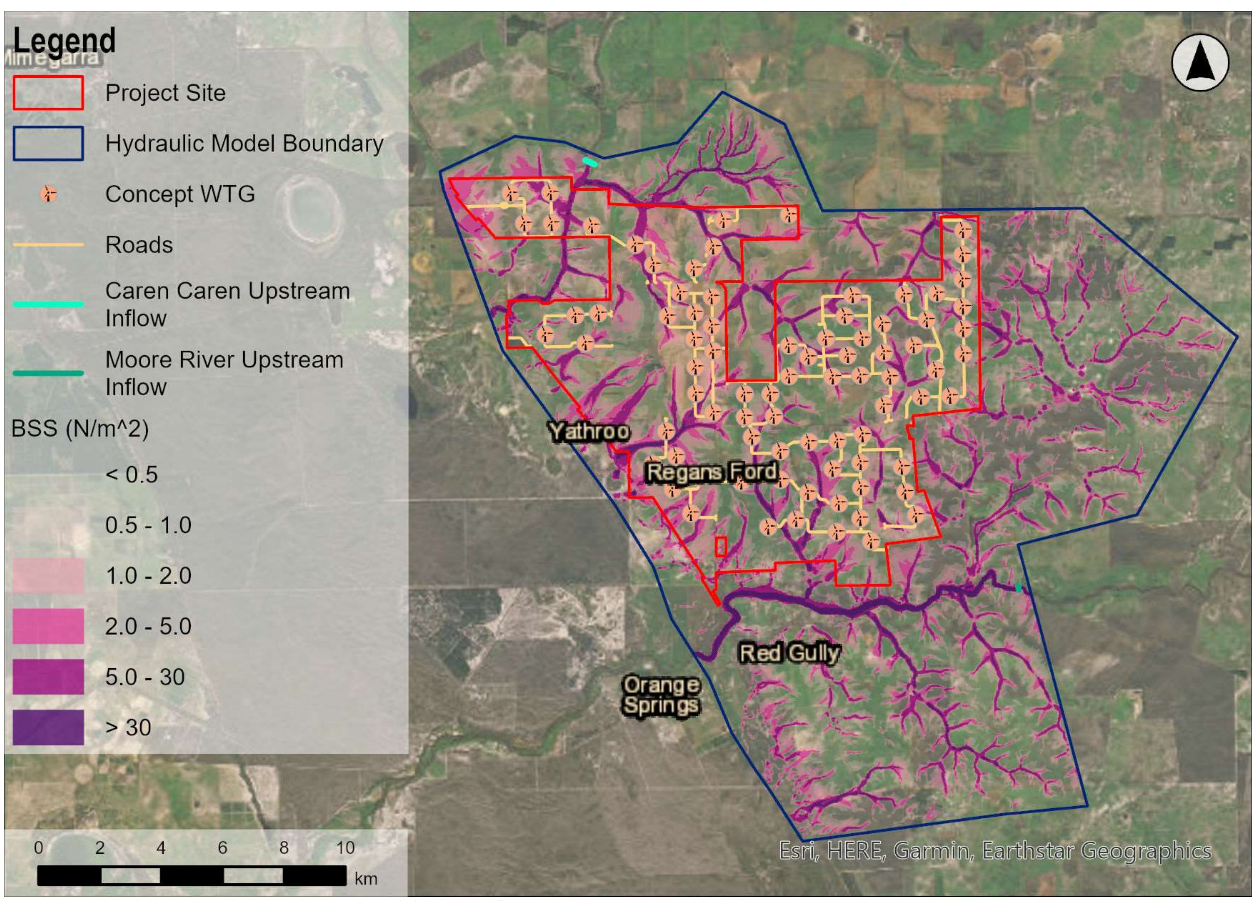

Figure 5-5 Bed Shear Stress (BSS) for 1% AEP for the Existing Condition

5.1 Caren Caren Brook Crossing

No road crossing is proposed over Caren Caren Brook in the current layout. Temporary construction access to the four NW turbines will be taken from Brand Hwy. The cable corridor will traverse the brook via trenchless horizontal directional drilling (HDD) beneath the channel, avoiding in-stream works. This approach was selected because the flood study indicates a road crossing would require major bridge/culvert works, which are not acceptable given heritage/PNT constraints and would increase flood risk. Figure 5-6 shows the cable alignment over the 1% AEP flood-depth map; Figure 5-7 presents the design-event peak flows at the proposed crossing.

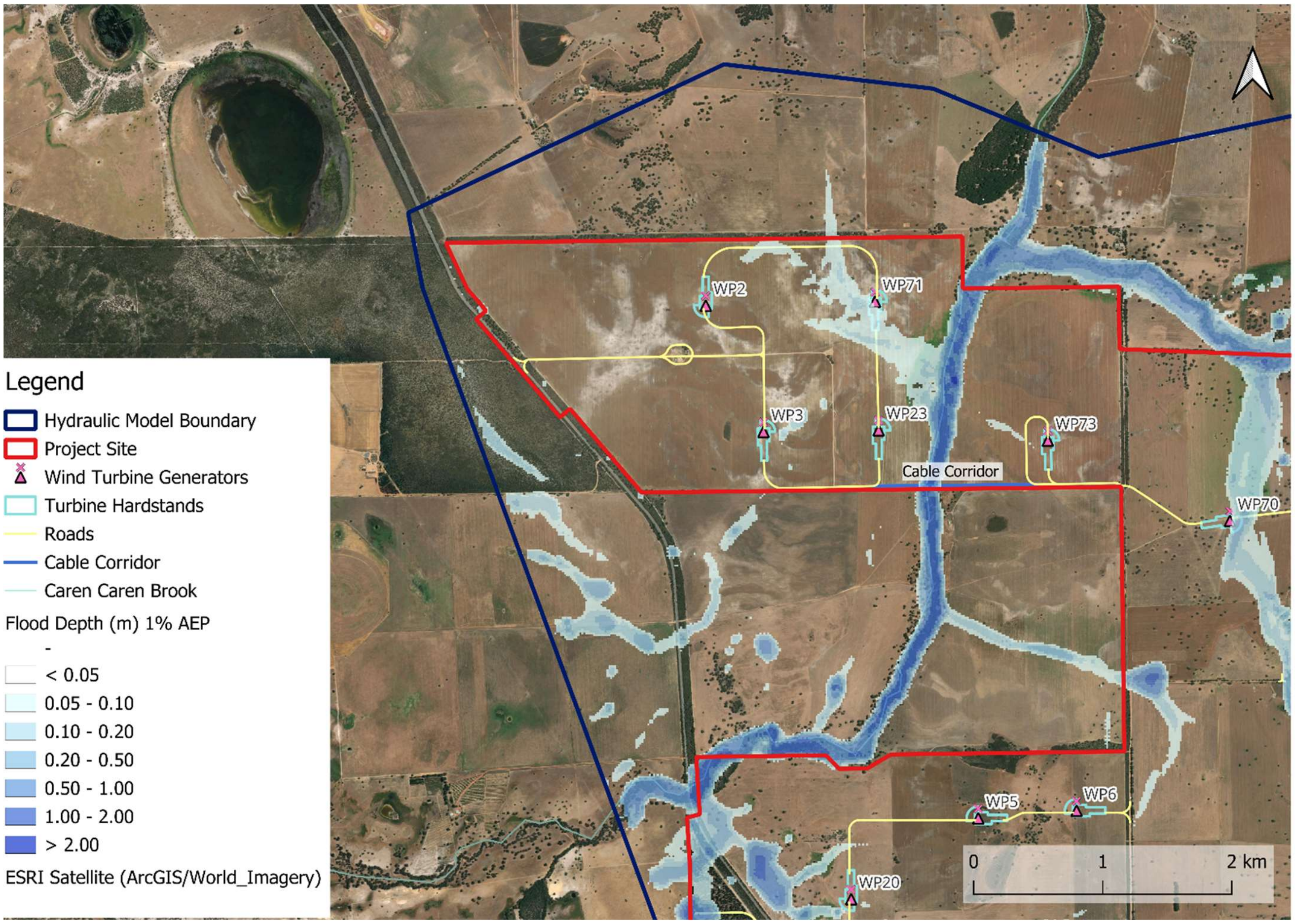

Figure 5-6 Caren Caren Brook cable corridor overlaid on 1% AEP peak flood depths

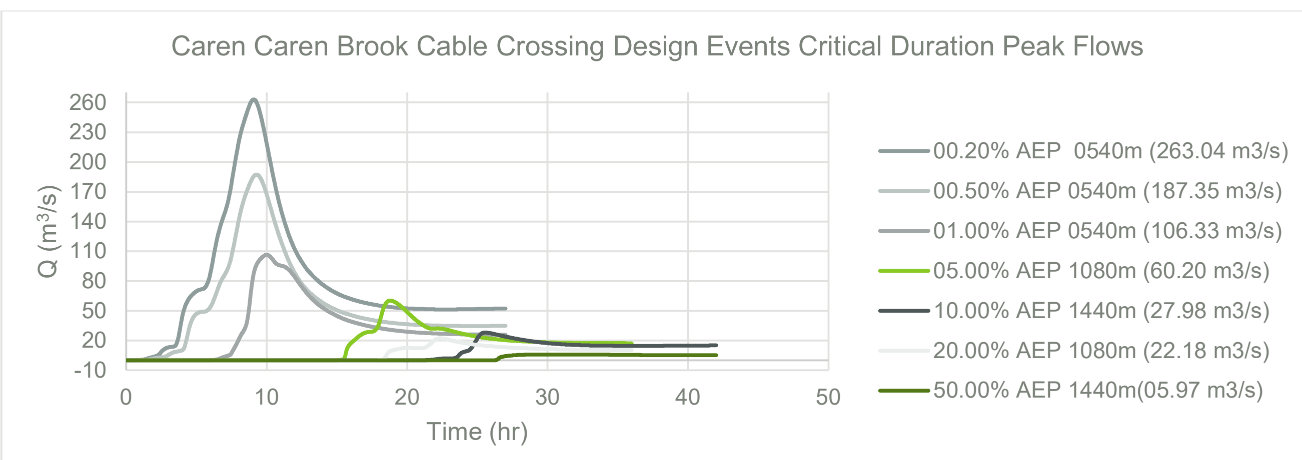

Figure 5-7 Caren Caren Brook design events-critical duration-peak flows at the cable corridor crossing

5.2 Sedimentation and Erosion

This section provides a high level erosion screening method using bed shear stress (BSS) rasters from TUFLOW model and soil-specific critical shear stress thresholds. Soil mapping was sourced from Soil Landscape Mapping – WA attributed by WA Soil Group (DPIRD-076) (refer Figure 5-8). The adopted critical shear stress thresholds for each WA Soil Group are listed in Table 5-1. Detailed BSS maps by event are provided in Appendix A to Appendix H.

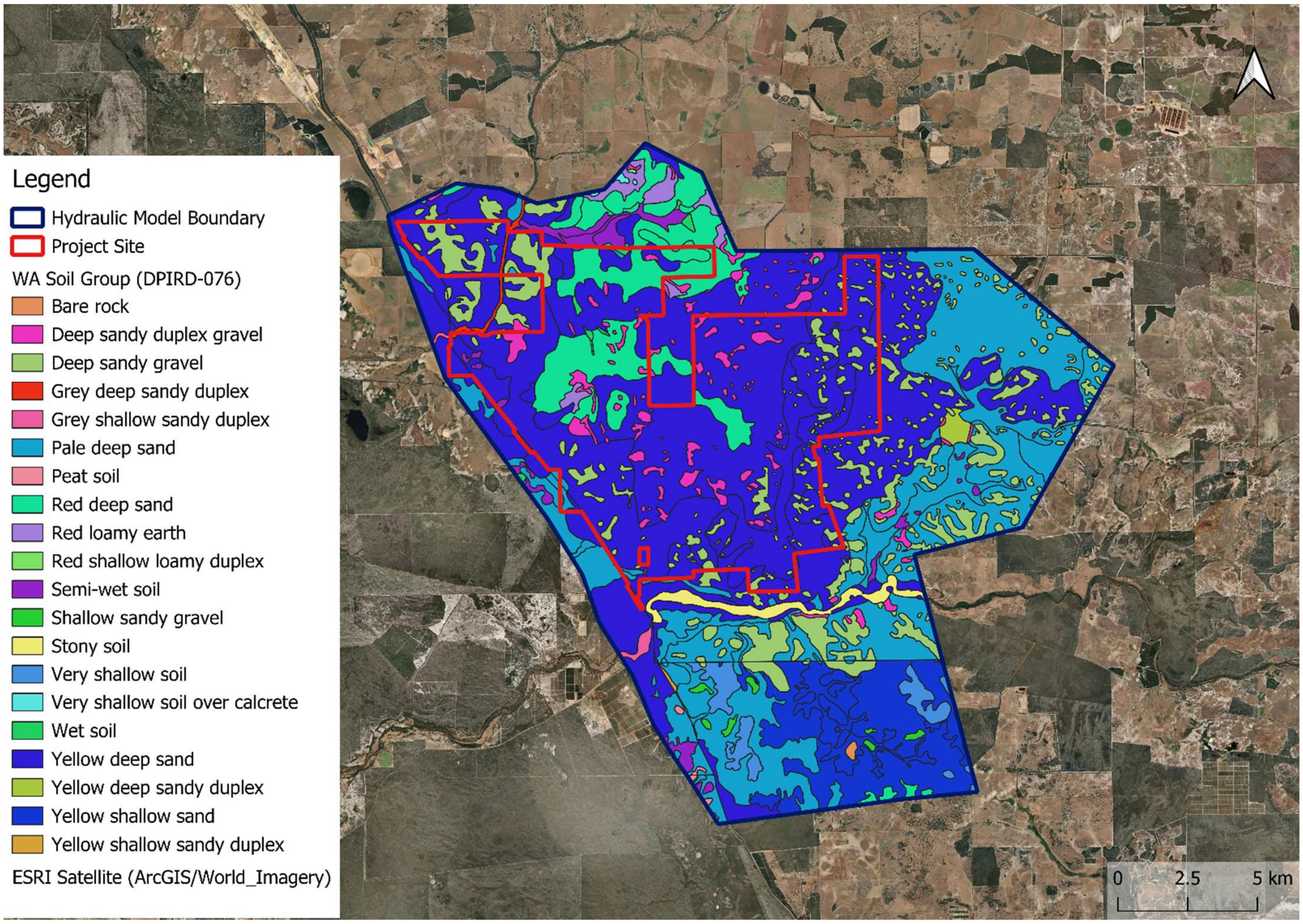

Figure 5-8 WA Soil Groups (DPIRD-076)

Table 5-1 Critical bed shear stress thresholds by WA Soil Group used for erosion screening

WA Soil Group	Critical Shear Stress range (N/m^2)	Adopted Critical Shear Stress (N/m^2)	Notes
Bare rock	10–30	15	Practically immune to initiation; confirm local rock at structures.
Deep sandy duplex gravel	4–8	5	Sand over clay with gravel lag; use higher end if visibly coarse.
Deep sandy gravel	4–8	5	—
Grey deep sandy duplex	1.5–3	2	Sandy A over clay; treat as sand for initiation.
Grey shallow sandy duplex	1.5–3	2	—
Pale deep sand	0.8–2.5	1.5	Non-cohesive.
Peat soil	0.3–0.8	0.5	Highly variable; very erodible if unvegetated.
Red deep sand	0.8–2.5	1.5	—

Red loamy earth	1.5–3	2.5	Moderate cohesion.
Red shallow loamy duplex	2–4	3	Loam over clay; slightly higher than sand.
Semi-wet soil	1.5–3	2	Hydromorphic; conservative.
Shallow sandy gravel	4–8	5	—
Stony soil	8–20	12	Stone armouring; confirm in field.
Very shallow soil	4–8	5	Often lithic/stony; adjust if clearly sandy.
Very shallow soil over calcrete	8–20	12	Thin cover on calcrete; armoured response.
Wet soil	3–6	4	Dispersion can reduce shear stress
Yellow deep sand	0.8–2.5	1.5	—
Yellow deep sandy duplex	1.5–3	2	—
Yellow shallow sand	1.0–2.0	1.5	—
Yellow shallow sandy duplex	1.5–3	2	—

Existing pre-development land cover within the model domain (refer Section 3.1.3) influences erosion susceptibility; the adopted cover factors to be used to adjust soil critical shear stress are summarised in Table 5-2.

Table 5-2 Cover factors by DEA Land Cover class

DEA code	Class name	Typical range	Cover factor, Cf	Notes / when to adjust
216	Natural Bare Surface (NS)	—	1	Baseline (no additional protection). Use where surface is largely bare/armoured.
111	Cultivated Terrestrial Vegetation (CTV)	1.2–1.6	1.5	Crops/pasture/stubble. Use lower end if recently cultivated or sparse; higher if dense stubble/cover crop.
112	(Semi-)Natural Terrestrial Vegetation (NTV)	1.8–2.2	2	Grassland/shrub/woodland. Roots/roughness add resistance; reduce if cover is sparse or recently burnt.
124	Natural Aquatic Vegetation (NAV)	2.0–2.8	2.5	Emergent reeds/rushes/macrophytes; strong reinforcement. Adjust by stand density.
215	Artificial Surface (AS)	—	-	Pavement/compacted pads. Exclude from soil-erosion screening.
220	Water	—	1	Open water (no vegetation). If bed macrophytes present, apply NAV factor instead.

5.3 Flood Model Results Disclaimer

5.3.1 Model Output Interpretation

This flood model presents maximum envelope results rather than temporal snapshots. The flood maps display the maximum depth, height, velocity, ZAEM1 and BSS values that could potentially occur at each geographic point, regardless of timing. These results do not represent conditions at any single moment in time.

5.3.2 Catchment Analysis Limitations

Joint probability assessment was not conducted for the following catchments:

- Caren Brook
- Moore River
- Wind farm runoff

The model assumes simultaneous maximum flooding from the Moore River and Wind farm runoff catchments, which represents an extremely conservative scenario. These flood events are highly unlikely to occur simultaneously at their peak intensities.

5.3.3 Statistical Independence

The three catchments analysed are almost entirely statistically independent. This means that flooding in one catchment does not significantly influence the probability or magnitude of flooding in the others.

5.3.4 Intended Use

These results are designed for planning purposes and provide a worst-case scenario envelope for risk assessment and infrastructure planning. The simultaneous maximum approach offers a conservative foundation for:

- Emergency response planning
- Infrastructure design considerations
- Land use planning decisions
- Risk assessment frameworks

5.3.5 Important Considerations

Users should understand that:

- Actual flood events will likely be significantly less severe than these maximum envelope results
- The probability of experiencing the maximum values shown is extremely low
- Local drainage conditions and real-time meteorological factors will influence actual flood behaviour
- These results are based on existing (pre-development) conditions. The modelling has not included the proposed scheme design and therefore does not account for the potential impacts of new roads, drainage infrastructure, or other site modifications.
- For questions regarding methodology or specific applications of these results, please consult with qualified flood risk professionals.

6 Conclusion

This flood modelling study has been undertaken for the Marri Wind Farm under existing pre-development conditions. A hydraulic model was developed in TUFLOW, in-line with the principles and methodologies outlined in the Australian Rainfall and Runoff 2019 (ARR 2019) guidelines. The model was simulated for the 0.2% 0.5%, 1%, 5%, 10%, 20% and 50% Annual Exceedance Probability (AEP) design rainfall events across a wide range of durations to capture critical flood behaviour across the project site.

The outcomes of the study have been presented as flood maps (Appendix A to Appendix H), providing a high-level understanding of flood risk within the project site. These results are intended to support the design development of the proposed wind farm infrastructure by highlighting areas subject to potential flooding and guiding future design considerations. Once the proposed layout is designed should be modelled to measure flood impact.

6.1 Recommendations

- This assessment provides a high-level understanding of flood risk based on data available at the time of the study. A review of the analysis should be undertaken should design level or flood impacts be required.
- The analysis considers flood risk at a broad project scale. Localised drainage issues are not captured and would require refined localised modelling to inform any site stormwater management strategies and designs.
- Any proposed filling of land or excavation and levelling may alter the ground levels locally, and further assessment will be required to assess the influence on flood behaviour.
- To improve flood modelling accuracy, the downstream bridges and drainage culverts under Brand Highway should be surveyed and updated in the model, as their dimensions and inverts are currently based on DTM and imagery. These culverts represent a key downstream control for the site and may significantly affect post development model performance.
- Post-development flood modelling has not been undertaken, as these adjustments to the site layout based on this investigation’s findings may require adjustments that should be assessed in further design stages.
- Rainfall runoff estimate has been undertaken on historical rainfall probabilities and no assessment on increased flood risk due to climate change has been made.

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Appendix A – Site Location

Appendix B – 50% AEP Flood Maps

Appendix C – 20% AEP Flood Maps

Appendix D – 10% AEP Flood Maps

Appendix E – 5.0% AEP Flood Maps

Appendix F – 1.0% AEP Flood Maps

Appendix G – 0.5% AEP Flood Maps

Appendix H – 0.2% AEP Flood Maps

Appendix I – Quinns Catchment Gauging Station 617001- Streamflow Data

Appendix J – Regional Flood Frequency Estimation (RFFE) Model for Caren Caren Brook Catchment

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