Wallerawang Battery Energy Storage System (BESS)

Flooding Assessment

Version 3

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Copyright and Limitation

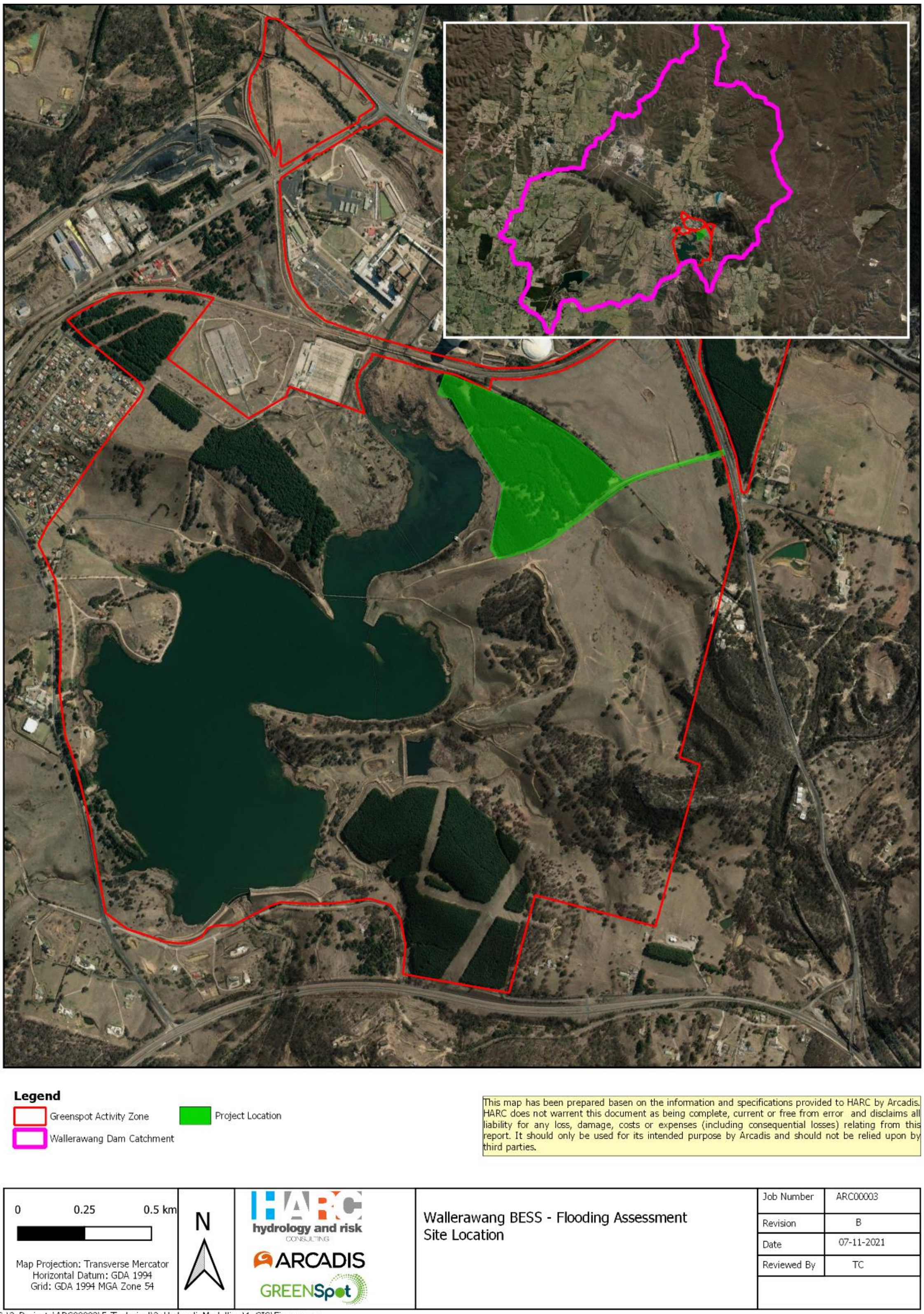
Contents

1. Introduction

Greenspot, the owners of the former Wallerawang power station, are proposing to repurpose the site. ARCADIS are assisting Greenspot with the Environmental Impact Statement (EIS). One of the proposed developments is the installation of a Battery Energy Storage System (BESS). This report details the impact of the proposed BESS works on flood extents. The results in this report are an input to the EIS and do not provide commentary on if the proposed works are compliant or otherwise with the terms of the EIS. Figure 1-1 shows the locality of the proposed works. The proposed development site is located approximately 2 km north east of Wallerawang Dam.

The former Wallerawang power station is located near the town of Wallerawang which is located in the Cox's River basin. Wallerawang is located approximately 10 km north west of Lithgow.

In 2019, SGM Consulting along with HARC completed a hydrologic and dambreak assessment for Wallerawang Dam for Energy Australia. As part of the 2019 assessment a hydrologic model (RORB) was set up and calibrated, which included Wallerawang Dam. Also, as part of the 2019 assessment a hydraulic model (TUFLOW) was set up to model the impact of extreme floods. These models have been adapted for use in this investigation.



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■ **Figure 1-1: Locality plan**

1.1 Scope and limitations

This report has been prepared by HARC for ARCADIS and Greenspot. The services undertaken by HARC in connection with preparing this report were limited to those specifically detailed in this report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. HARC has prepared this report on the basis of existing available information provided by ARCADIS (drawings of the proposed design) and Energy Australia (LiDAR), as well as published methodologies (e.g. Australian Rainfall and Runoff), which HARC has not independently verified or checked beyond the agreed scope of work.

The opinions, conclusions and any recommendations in this report are based on certain assumptions made by HARC using the existing available information and methodologies mentioned above and as described in this report. HARC does not accept liability in connection with such unverified information, including liability arising from incorrect assumptions and errors and omissions in the report which were caused by errors or omissions in that information.

2. Hydrological Assessment

As mentioned in Section 1, HARC undertook a hydrological assessment on the Cox's River which included Wallerawang Dam. The hydrological assessment was undertaken using a rainfall runoff model (RORB). The following provides a summary of the establishment, calibration and verification of the RORB model as relevant for this investigation.

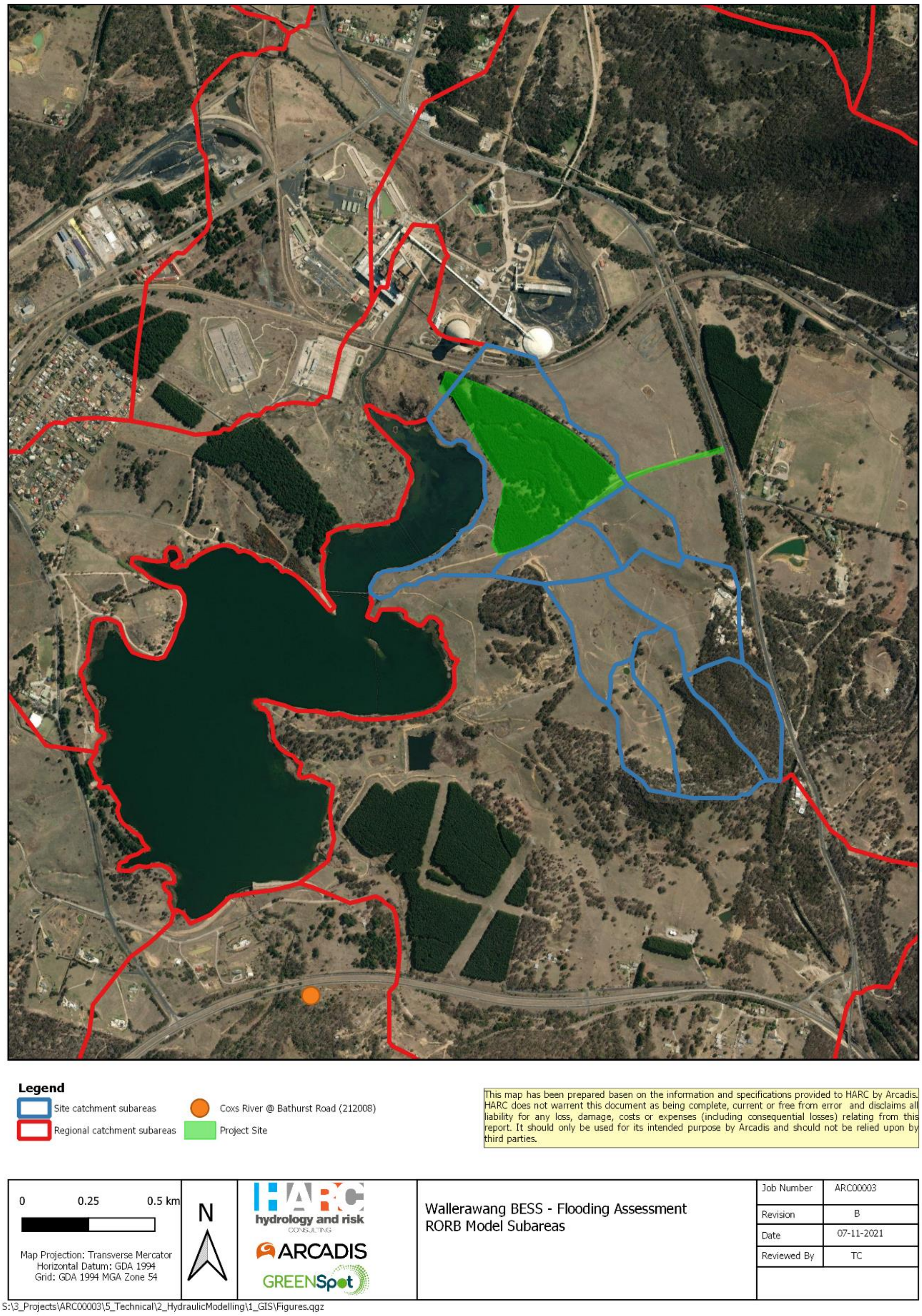
RORB (Laurenson, Mein and Nathan, 2010) is a general runoff and streamflow routing program that is used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to determine rainfall excess and routes this through catchment storages to produce streamflow hydrographs at points of interest. The model is spatially distributed, non-linear, and applicable to both rural and urban catchments. It makes provision for both temporal and areal spatial distribution of rainfall as well as losses, and can model flows at any number of points throughout a catchment (including upstream and downstream of reservoirs). RORB also has the capacity to use a Monte-Carlo approach to produce design flood estimates that incorporate the joint probability of several factors that influence flood characteristics.

In general terms, development of a RORB model entails sub-dividing the catchment into a series of subareas to suit the catchment topography and other features such as the location of gauging stations and storage locations.

The RORB model layout of this site is shown in Figure 2-1. This figure includes the broader subareas (red polygons) as well as the site subareas (blue polygons). Additional subdivision of the catchment upstream of the proposed project site was undertaken for this project to represent the routing of this smaller local catchment.

Four different types of reaches can be defined in RORB, each having different properties and different relative delay times. The reach types are identified as natural, excavated but unlined, lined channel or pipe and drowned reaches. Drowned reaches were used within reservoir water bodies; natural reaches were used for all other reaches. Natural reaches were mainly used for this model and drowned reaches were used in the reservoir.

Impervious fractions are required for each sub-area. For rural areas the impervious fraction was assumed to be zero. For the town areas, the NSW Land Use layer has been used to assign impervious fractions. These have been applied as an effective impervious fraction (EI).



▪ Figure 2-1 RORB model layout

RORB models are based on catchment geometry and topographic data, and the two principal routing parameters are k_c and m. The parameter m describes the degree of non-linearity of the catchment's response to rainfall excess, while the parameter k_c describes the delay in the catchment's response to rainfall excess.

A value of 0.8 was adopted for the non-linearity parameter, m, for this study, which is recommended by Laurenson et al. (2010) as well as Book 8 of Australian Rainfall and Runoff (Nathan and Weinmann, 2016) for modelling very large and extreme flood events. The value of the routing parameter, k_c, was selected from the calibration of the RORB model undertaken for the work for Energy Australia (SGM and HARC, 2019).

The Cox's River catchment was calibrated to several historic events and at several locations throughout the catchment. The closest gauge to the proposed development site is the Bathurst Road gauge (212008) which is located immediately downstream of Wallerawang Dam. The streamflow gauge is shown on Figure 2-1. The k_c value at this site was based on calibration to the following events:

- August 1990
- August 1998; and
- July 1988

A summary of the calibration results used for the choice of k_c at the Bathurst Road gauge (212008) are shown in Figure 2-2. Table 2-1 summarises the calibrated k_c values adopted for the Bathurst Road gauge from the calibration process.

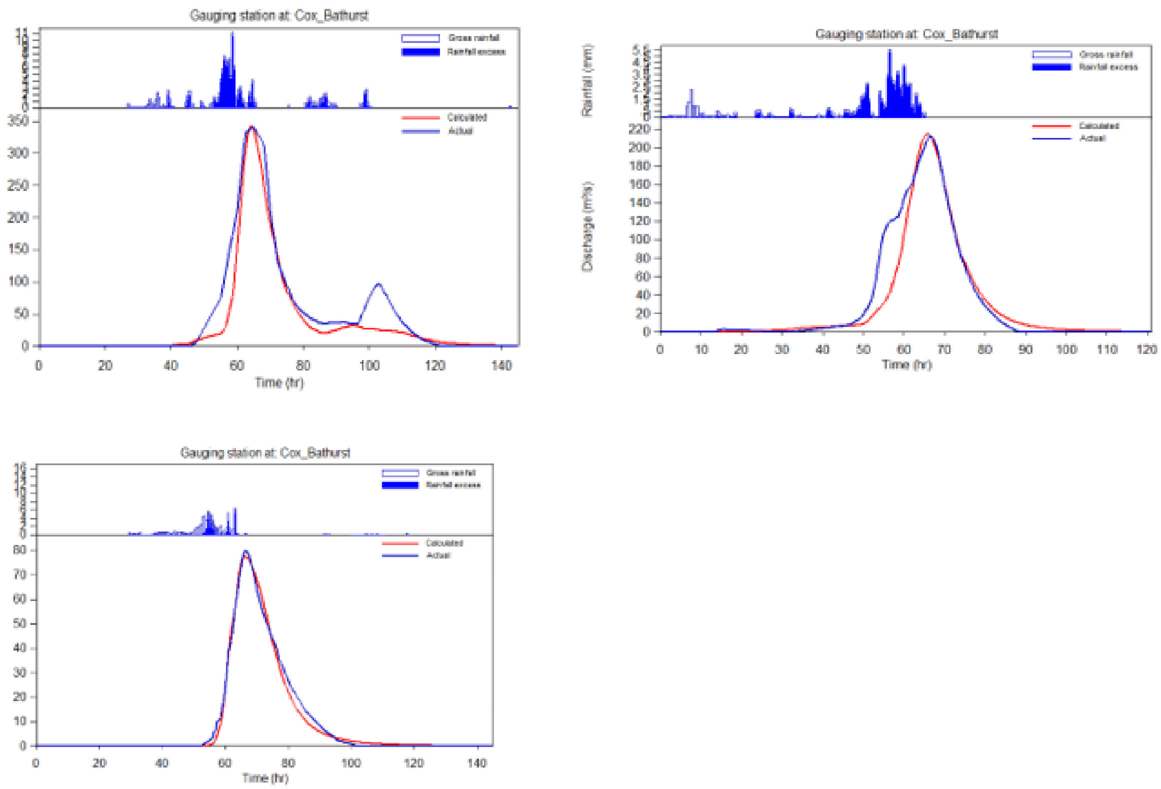


Figure 2-2 Calibration results used to assign k_c value Left top: August 1990, Right top: August 1998 and Bottom Left: July 1988

Table 2-1 Summary of adopted routing parameter values from SGM and HARC, 2019

Gauge	Name	k_c value			Adopted k_c	d_{av}*	k_c/d_{av}
		1990	1998	1988			
212008	Coxs River at Bathurst Rd	20	20	20	20	13.3	1.5

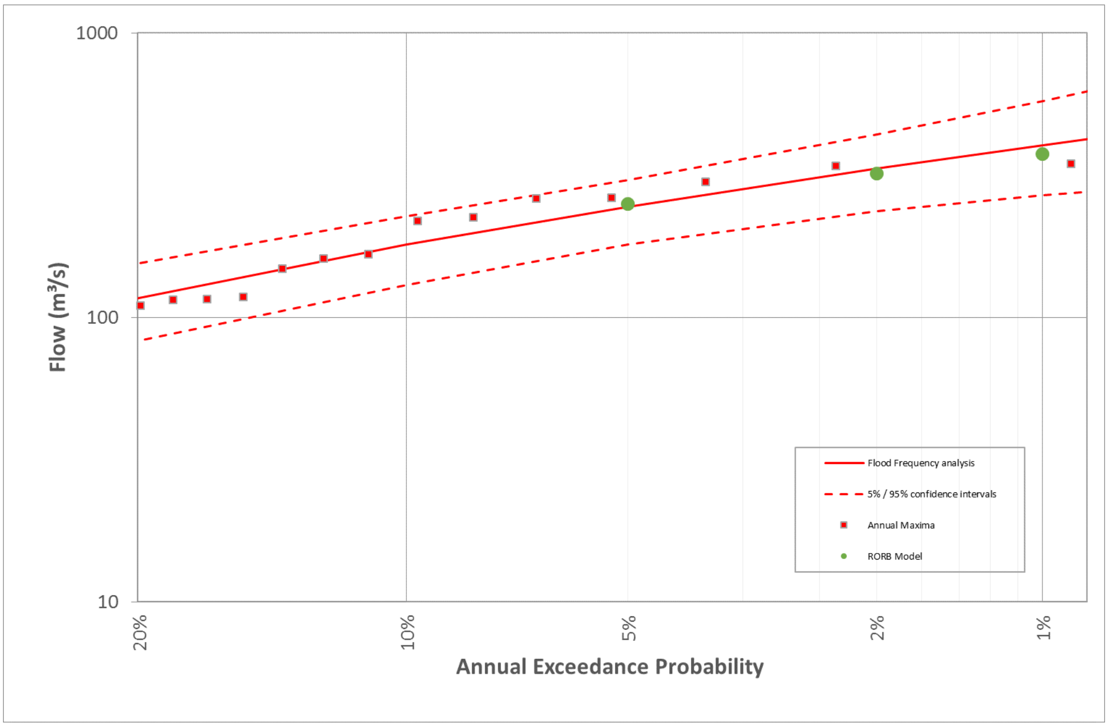
* d_{av} is the weighted average flow distance to the catchment outlet (this is calculated automatically in the RORB model)

As shown in Figure 2-2 in general, a good calibration was achieved. As with all hydrological modelling, the observed variations between the recorded and modelled hydrographs can be the result of a number of uncertainties, including factors such as historic changes in catchment conditions, recorded rainfall and streamflow data errors, baseflow separation uncertainties, and the lack of adequate rainfall gauges to represent the temporal and spatial variability of the storms across the catchment. It must be noted that RORB (and all hydrologic models) are only a representation of a variable and complex rainfall runoff process. Notwithstanding this, at a high level the quality of the calibrations obtained across the catchment are considered more than sufficient to warrant use of this model for use in this flood impact assessment.

The other parameters required in RORB represent rainfall losses, using either an initial loss/continuing loss model, or an initial loss/proportional loss (i.e. runoff coefficient) model. An initial loss/continuing loss model was adopted for this study because it is more appropriate for modelling large floods.

Current practice in design flood estimation includes verification of the results from rainfall runoff modelling (such as RORB) against at-site flood frequency analysis from observed streamflows. Selection of the loss values used for design were determined from the verification process. The following summarises the verification process undertaken in the 2019 investigation (SGM and HARC, 2019). Suitable median initial loss and continuing loss values for use in design were estimated using this process, which involved running the model in design mode and varying the losses until there was an acceptable match between the RORB flood frequency quantiles and the gauged flood frequency curves.

The RORB model was then run in Monte Carlo simulation mode using standard design inputs from Australia Rainfall and Runoff (2019) to estimate flood frequency quantiles for flood events with AEPs of 10%, 5%, 2% and 1%. Figure 2-3 shows the GEV distribution (quantile) for the gauge, Coxs @ Bathurst Road and the estimates from the RORB model. Figure 2-3 demonstrates that the RORB model provides a good match to the distribution fitted to gauged annual maxima for the 5%, 2% and 1% AEP flood quantiles with a k_c of 20, median IL of 10 mm and a fixed CL of 1.5 mm/h.



- Figure 2-3 - Flood frequency curve and verification for Cox @ Bathurst Rd with RORB model Monte-Carlo run outputs for flood quantiles between 10% and 1% AEP (for k_c of 20, median IL of 10 mm and CL of 1.5 mm/h)

Table 2-2 summarises the adopted routing (from calibration) and loss (from verification) model parameter values adopted for this project.

- Table 2-2 Summary of adopted RORB model parameter values

Gauge No.	Gauge Name	k_c	m	IL (mm)	CL (mm/hr)
212008	Coxs River @ Bathurst	20	0.8	10	1.5

2.1 Design Hydrology

Using the design parameters discussed in Section 2, the RORB model was run with a local thunder storm where the event is centred of the proposed development site catchment (~1km^2) and a regional storm where the storm is centred over of the catchment upstream of the Wallerawang dam (~150 km^2). It is important to consider both of these impacts to determine which would have the highest flow across the project site. As required by the brief three AEP events were considered, 5% AEP (1 in 20 AEP), 1% AEP (1 in 100 AEP) and Probable Maximum Flood (PMF).

For this investigation a deterministic Probable Maximum Flood (PMF) was derived. The deterministic PMF was estimated using an approach consistent with Section 4 of ARR Book 8 (Nathan and Weinmann, 2016):

- initial loss of 0 mm;
- continuing loss rate of 1 mm/h;

- most conservative peak outflow flood derived from the ten temporal patterns in the Monte-Carlo sample that was used for estimation of design floods; and
- Reservoirs start at FSL prior to commencement of the flood event.

The PMF results should be considered as an initial assessment which are likely to be conservative. In the situation that the PMF is to be adopted as the design flood, then it is recommended that additional analyses are undertaken to check the reasonableness of the estimates.

Table 2-3 shows the peak design inflows into the project site. For all AEPs considered for flood modelling, the local storm has a higher peak flow and will be used in the hydraulic modelling.

Table 2-3 Design flows to the proposed site

AEP	Coxs River Flow (m^3/s)		Site Catchment Flow (m^3/s)	
	Local Storm	Regional Storm	Local Storm	Regional Storm
5% (1 in 20)	225	195	3.6	2.5
1% (1in 100)	338	290	5.5	3.6
PMF	4740 (3hr)	4830 (6hr)	56.1	47.5

It should be noted that for all results except for the regional PMF, the local storm is larger than the regional storm. This is due to the higher ARF that has been applied to the regional storm compared to the local storm. For the purposes of flood modelling the higher local storm flows have been adopted. These results are discussed in Section 4.

3. Hydraulic Modelling

3.1 Approach

The hydraulic assessment was undertaken using a two dimensional model (TUFLOW) which was established for the Cox's River dams consequence assessment (SGM and HARC, 2019). The existing TUFLOW model was truncated to terminate at Bathurst Road.

The TUFLOW model was used to model the existing and proposed conditions.

3.2 TUFLOW model

The key inputs to hydraulic models are:

- Topographical information
- Cell size
- Roughness values
- Hydraulic structures
- Boundary conditions

Model runs were performed with the latest version (at time of assessment) build of TUFLOW HPC, specifically, 2020-10-AA-iSP-w64.

The geometry of the 2D floodplain and watercourses were established by reading in a uniform grid of square elements from the Digital Elevation Model (DEM). The topographic data (LIDAR) for the whole model extent was commissioned specifically for the dambreak assessment (SGM and HARC, 2019). The LiDAR survey was undertaken by AAM Group. The LIDAR was reported to have a ± 10cm vertical accuracy and from the LIDAR point cloud a 1 metre interval Digital Elevation Model (DEM) was derived.

For Lake Wallace (Wallerang Reservoir) and Thompsons Creek reservoir – upstream of Wallerang Reservoir – a bathymetric survey was undertaken also as part of the dambreak assessment (SGM and HARC, 2019). The bathymetric survey was undertaken by Hydrographic & Cadastral Survey Pty Ltd. The bathymetry was reported to have a total horizontal uncertainty of ±1.0m and a total vertical uncertainty ± 0.08m.

Manning's n roughness values for the model were determined primarily using freely available satellite imagery and the NSW Landuse 2017 datasets (https://data.nsw.gov.au/data/dataset/nsw-landuse-2017). Using Manning's n values listed in Table 3-1 each NSW Landuse layer was assigned a Manning's n value and the surface roughness layer is shown in Figure 3-1. The number adopted for Manning's n categories were selected to be in line with the values provided by ARR2019.

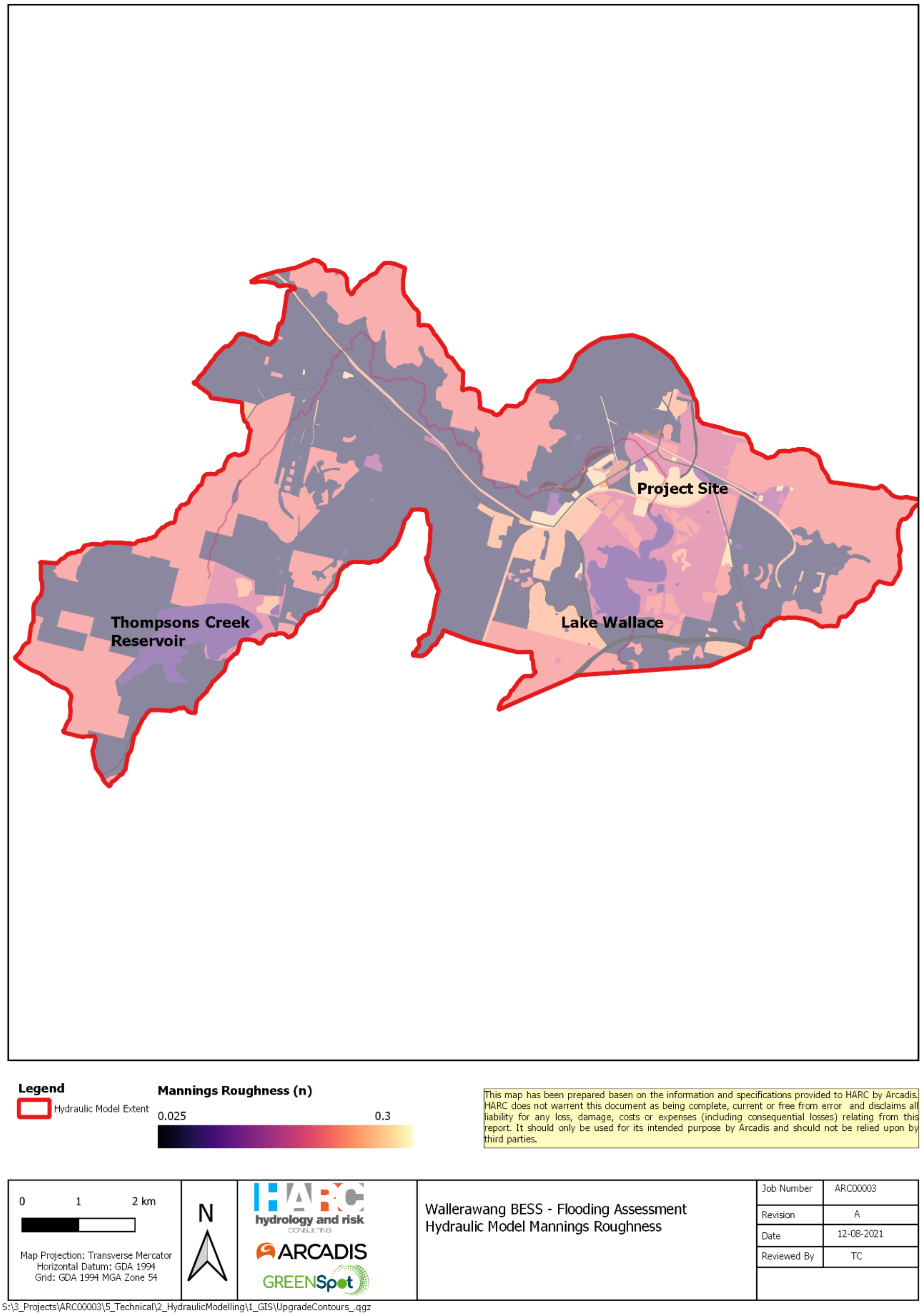
Large hydraulic structures for this study consist of the bridges over the various waterways. It is important to ensure that these impediments and constrictions to flow are accurately represented. The bridge information was taken from the data supplied for the dambreak assessment (SGM and HARC, 2019).

The inflows to the hydraulic model for the design runs were taken from the RORB model (as described in Section 2). The outflow boundary of the model was set as normal depth and is sufficiently far enough downstream from the site as to not influence the results.

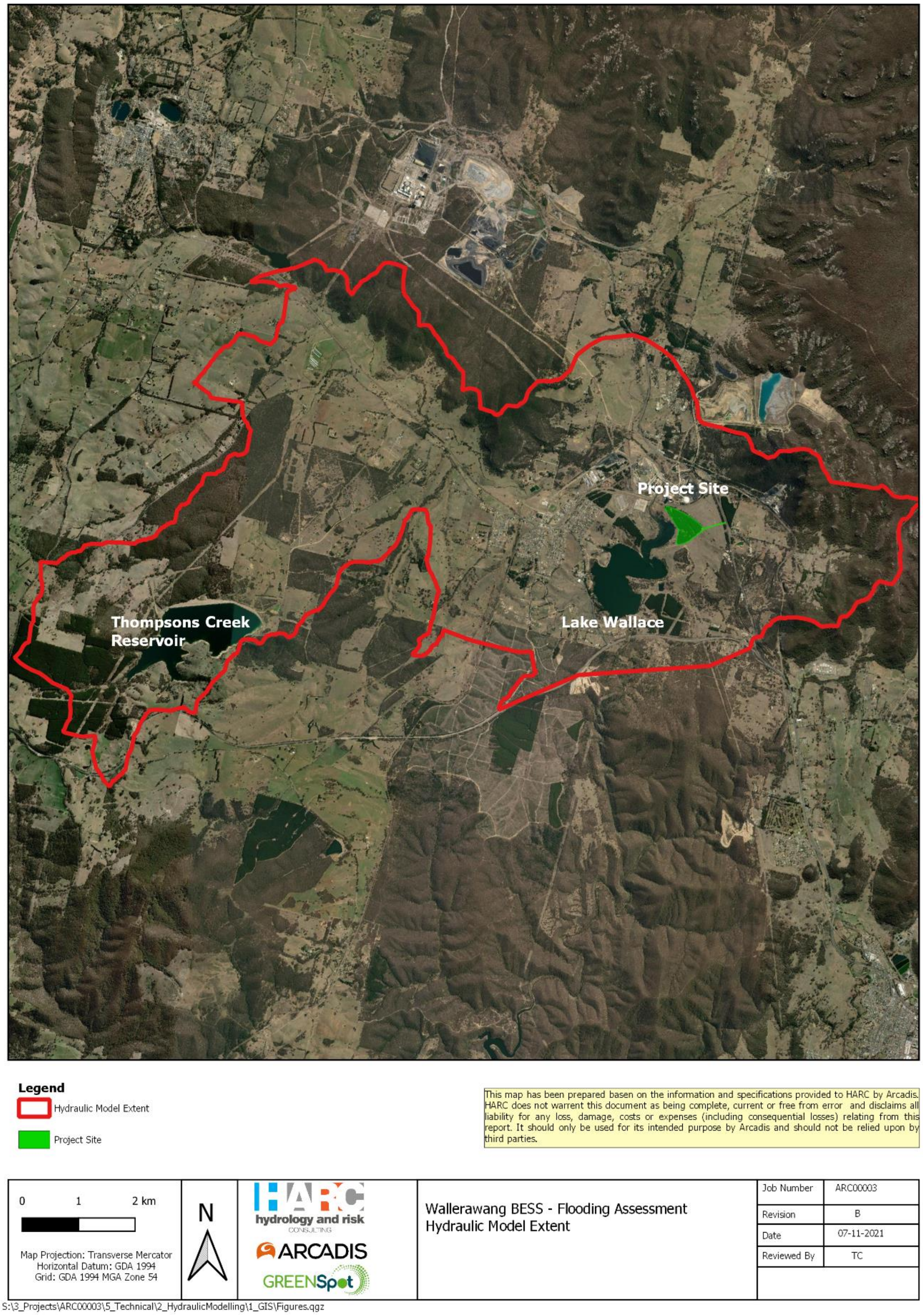
The hydraulic model extent is shown in Figure 3-2.

Table 3-1 Manning's n values for different land use types

Land Use Type	Manning's n adopted
Residential areas – high density	0.35
Residential areas – low density	0.15
Industrial/commercial – low density	0.30
Open space or waterway – minimal vegetation	0.03
Open space or waterway – moderate vegetation	0.06
Open space or waterway – heavy vegetation	0.095
Paved roads/car park/driveways	0.025
Railway line	0.05
Grass reserves/floodway (regularly mowed)	0.035
Rural floodplains in clear paddocks	0.03 - 0.06
Forested (heavy stand of timber)	0.12
Dam/Reservoir body of water	0.035



- Figure 3-1 Surface roughness distribution (Manning's n)



■ Figure 3-2 TUFLOW model extent

3.3 Existing conditions

Using the TUFLOW model the 5%, 1% and PMF design events were placed into the model to establish the existing flood extents.

3.4 Proposed conditions

The proposed conditions were supplied by Arcadis as a 3D CAD model. The project works are to provide a pad for the proposed battery which involves filling in the natural creek through the centre of the site by moving fill from the south and eastern sides of the site. The cut and fil locations for the project site are shown in Figure 3-3. Th

A 1.2 m diameter pipe was placed in the location of the natural creek. The pipe was sized to handle the 1 in 100 AEP flow (i.e. 5.5 m^3/s)

- Figure 3-3 Cut and Fill locations for the project site

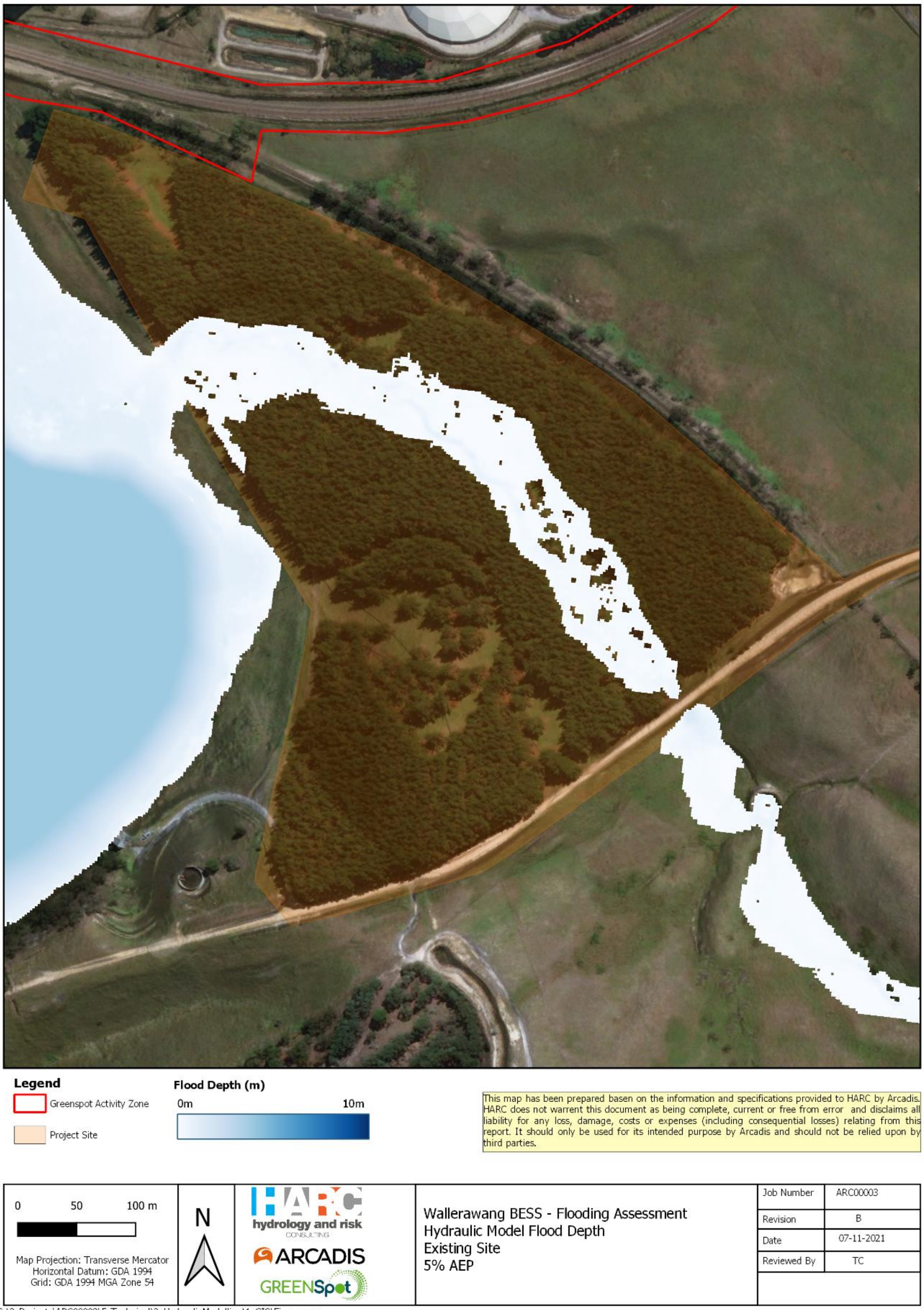
4. Results

This section shows the results from the hydraulic modelling.

4.1 Flood Extents and Levels (m AHD)

The flood extents and depths (m) for the existing and proposed conditions for the design events are show in Figure 4-1 to Figure 4-6. Although this report is focused on the proposed upgrade conditions, the existing conditions are shows to provide comparison to the proposed conditions, to ensure that there isn't a significant change in flood impact between the two scenarios, particularly outside of the project site.

Figure 4-6 which shows the upgraded PMF flood depth shows 'sheet flow' in the north-east side of the project side. That is depths less than 100mm that slowly drain away as the area has been levelled having no gradient.



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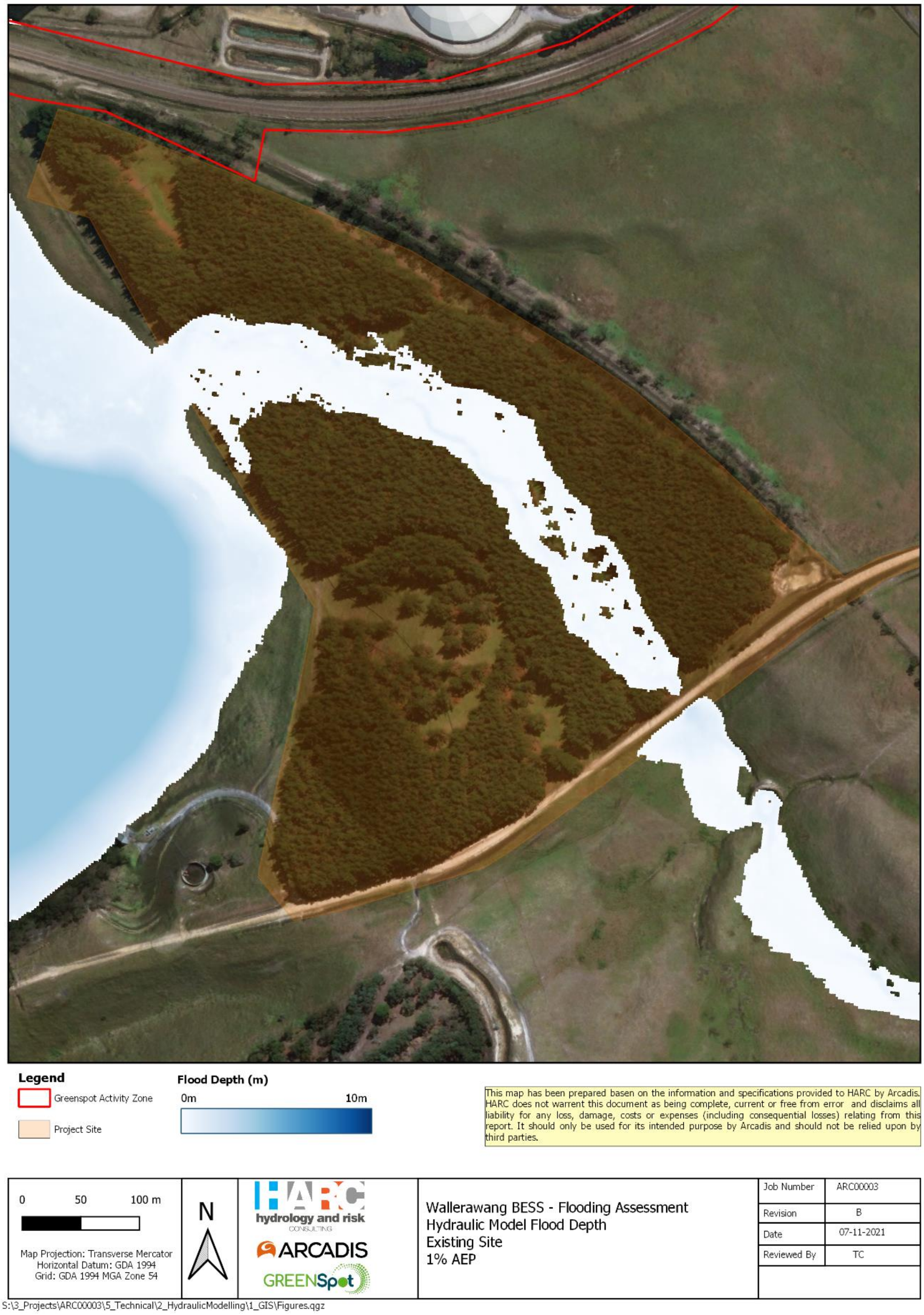
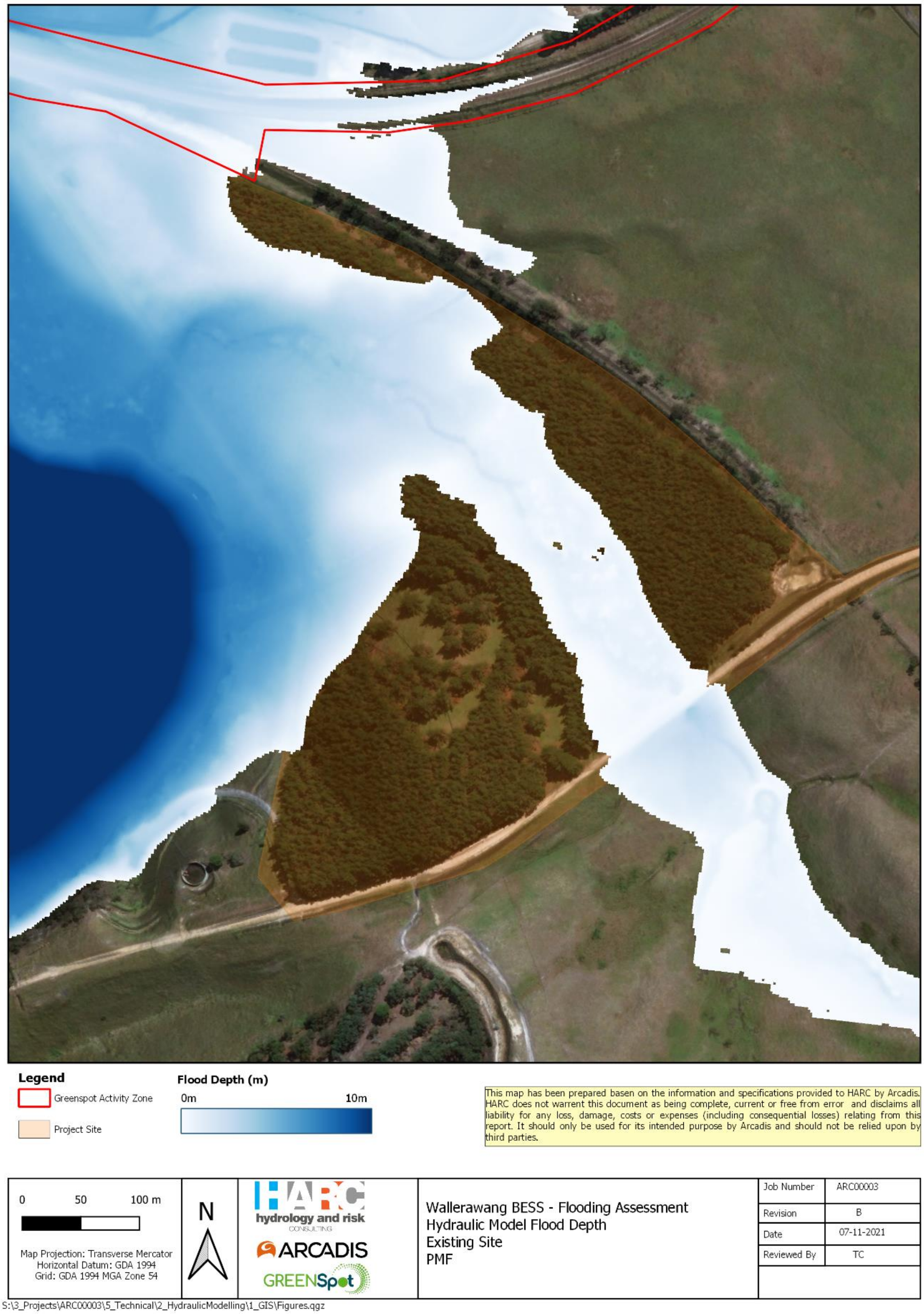
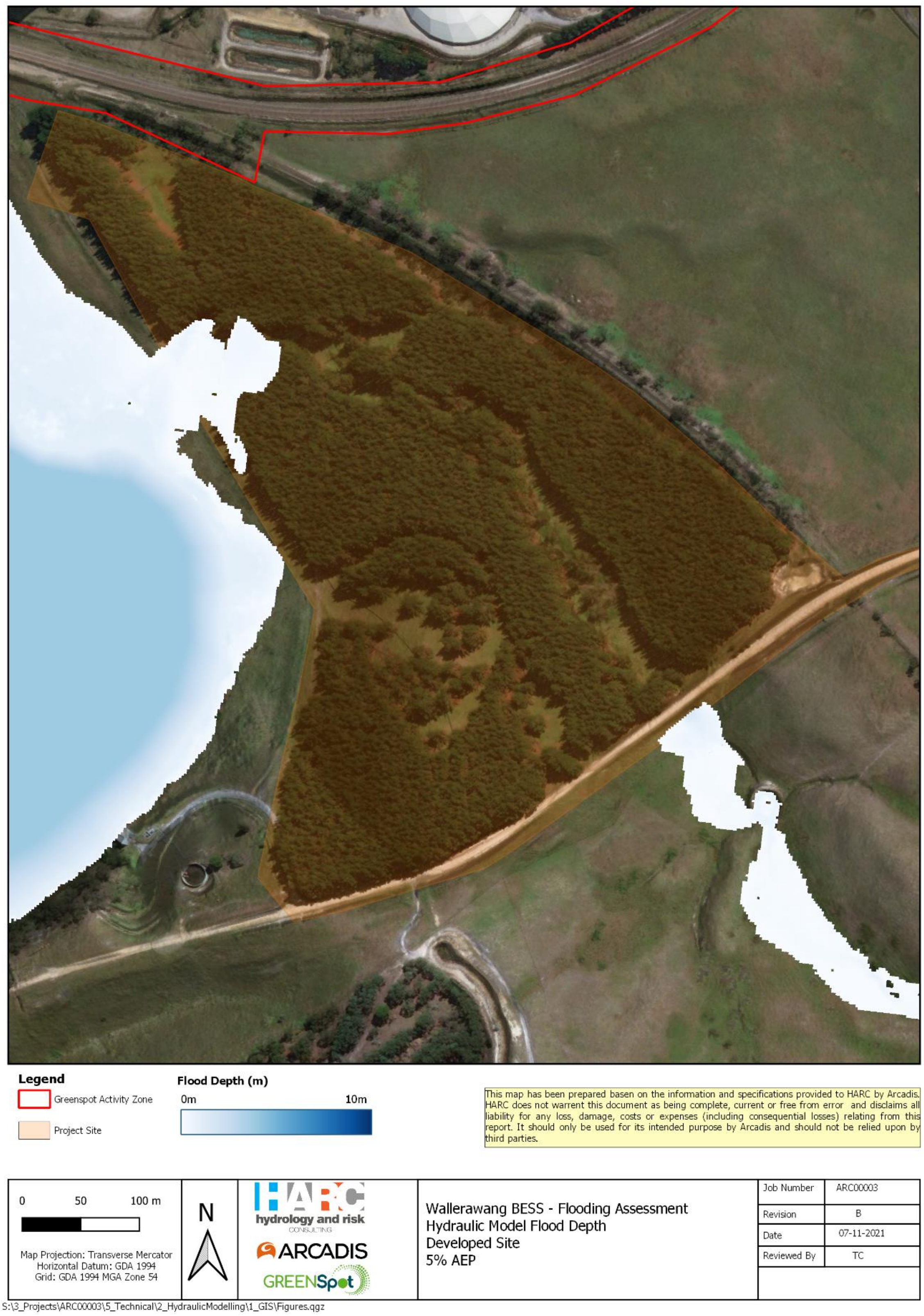
- Figure 4-1 5% AEP Flood Extents and Depths - Existing Conditions



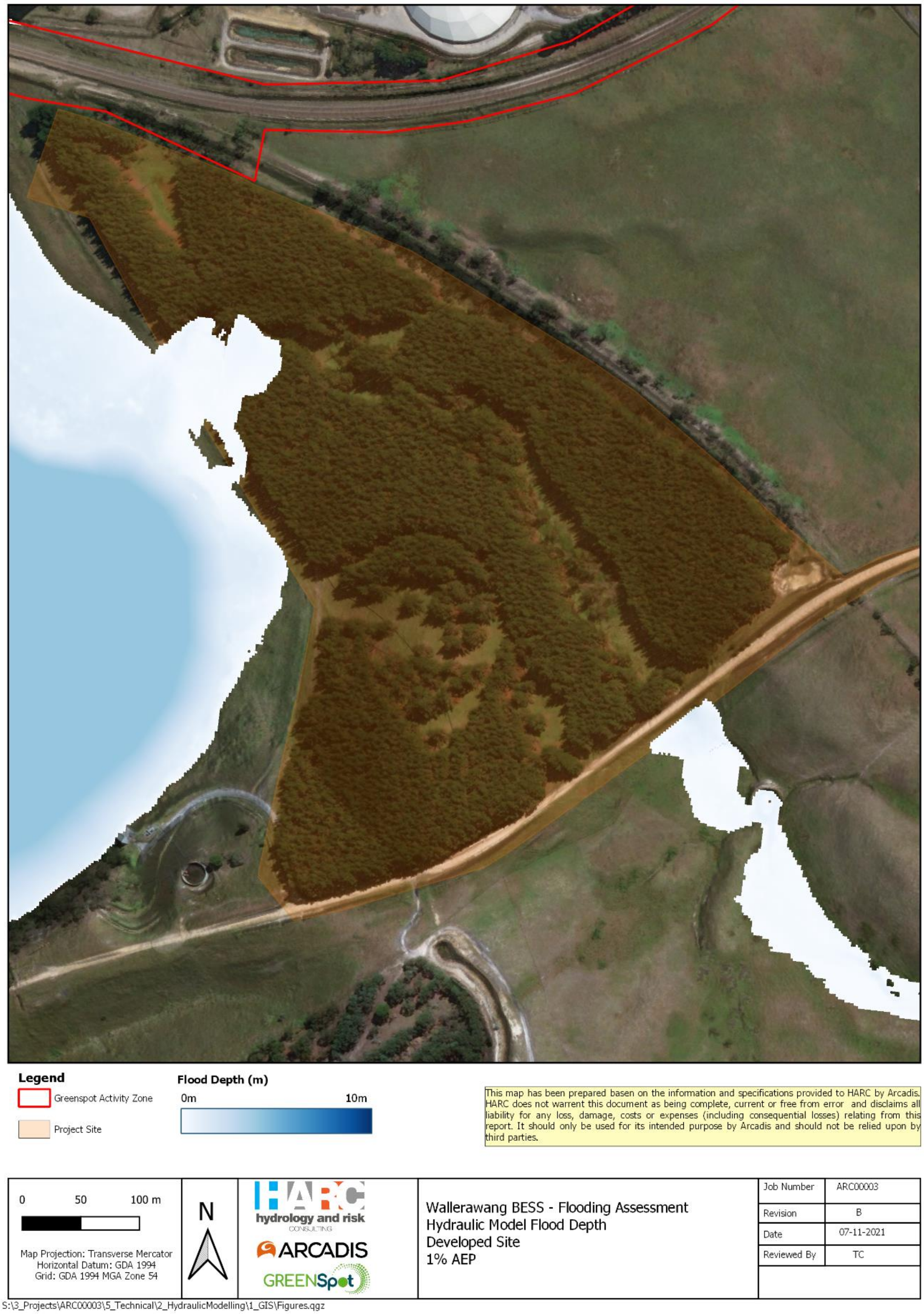
- Figure 4-2 1% AEP Flood Extents and Depths - Existing Conditions



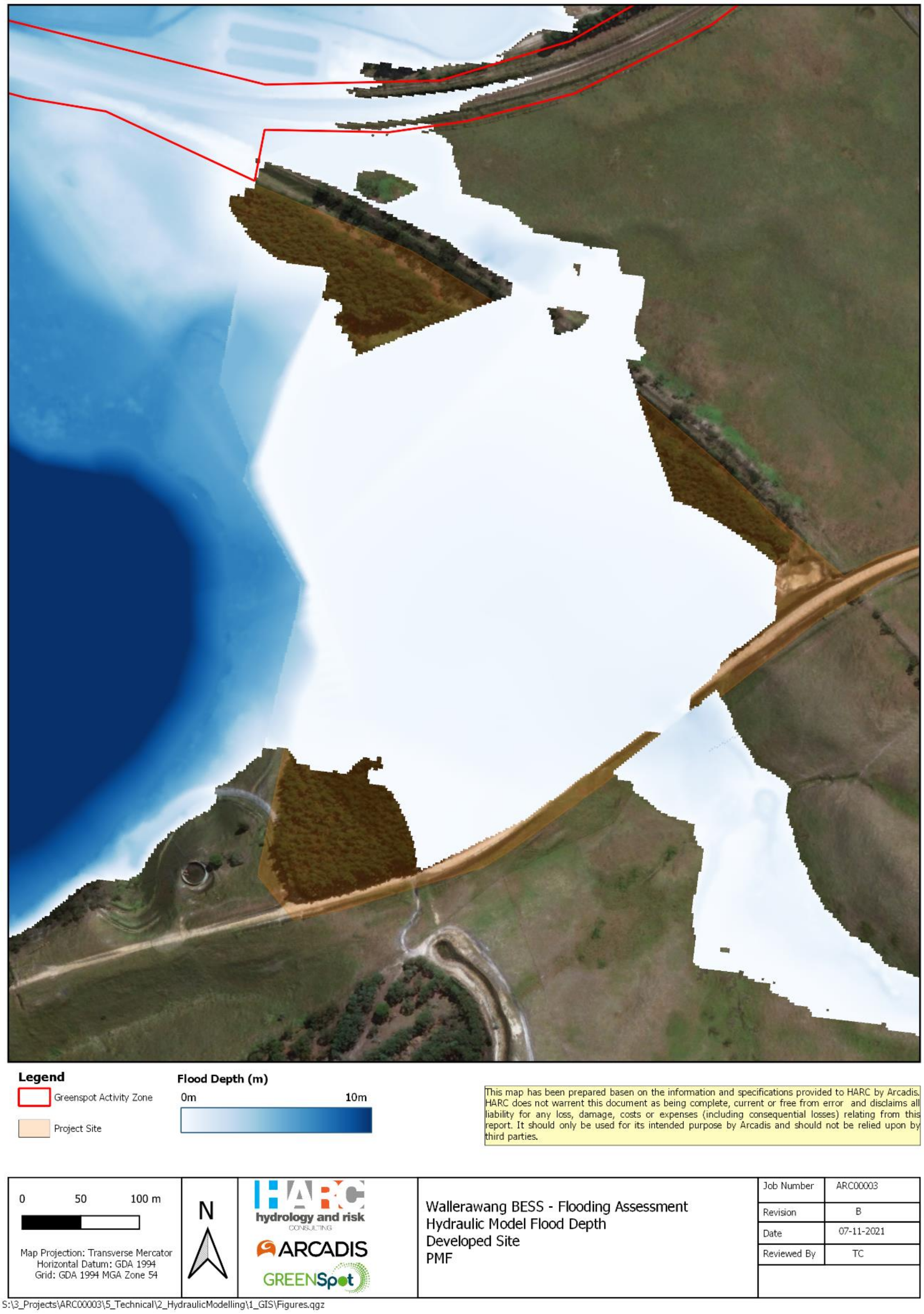
- Figure 4-3 PMF Flood Extents and Depths - Existing Conditions



- Figure 4-4 5% AEP Flood Extents and Depths – Proposed Conditions



■ Figure 4-5 1% AEP Flood Extents and Depths – Proposed Conditions

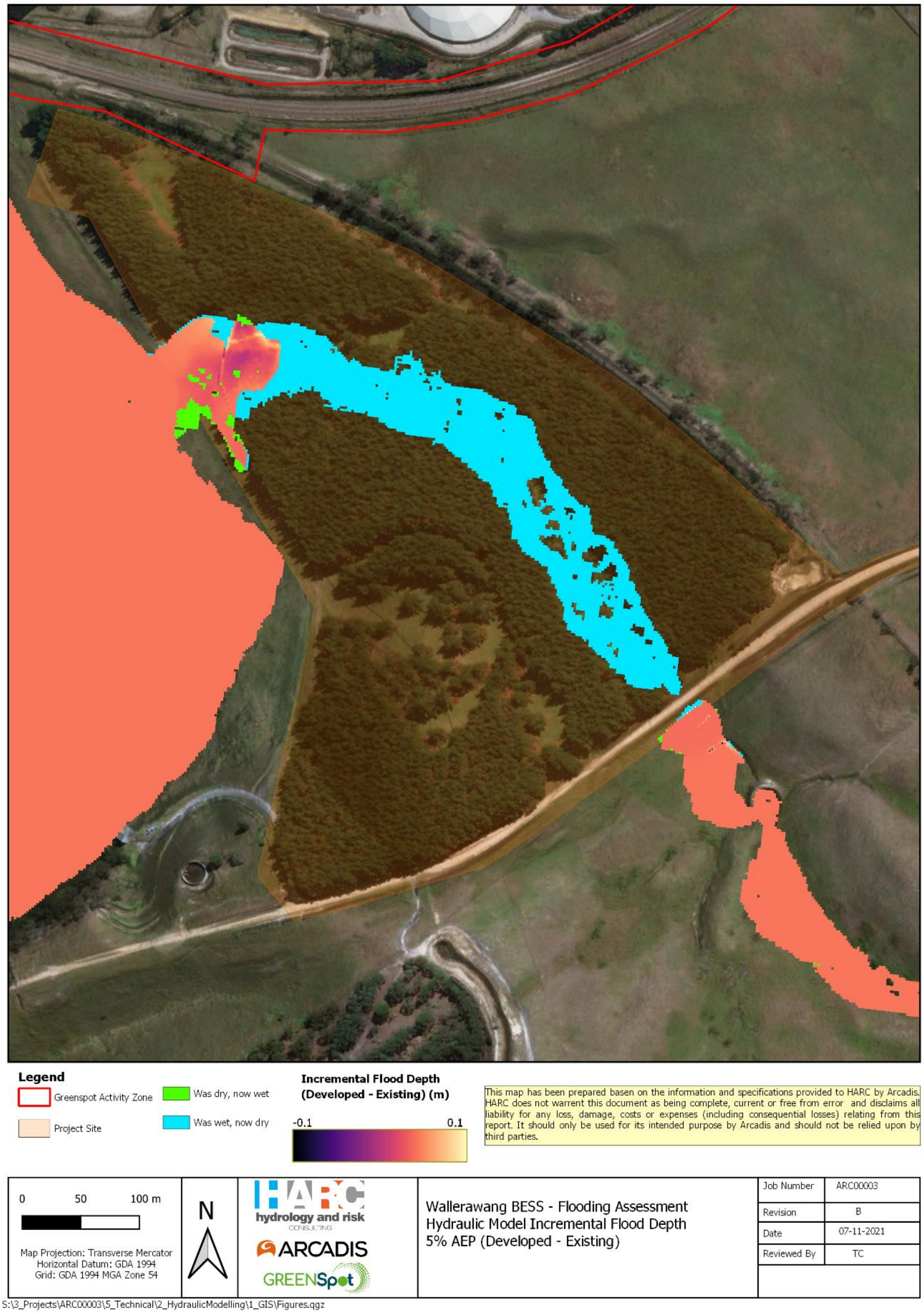


■ Figure 4-6 PMF Flood Extents and Depths - Proposed Conditions

4.2 Flood Level Afflux

The flood level afflux results for the proposed conditions for each of the AEP events modelled are shown in Figure 4-7 to Figure 4-9. The afflux was calculated by subtracting the existing conditions modelled flood levels from the proposed conditions modelled flood levels. A positive number means that the flood levels have gone up compared to the existing flood level and a negative number means the flood level have gone down.

All afflux figures show “was wet, now dry” and “was dry, now wet” zones. These are areas where only one of either existing or proposed scenarios show flooding. “Was wet, now dry” is when existing shows flooding but not the proposed scenario, whereas “was dry, now wet” is the opposite.



- Figure 4-7 5% AEP Incremental flood depth (Development minus Existing)

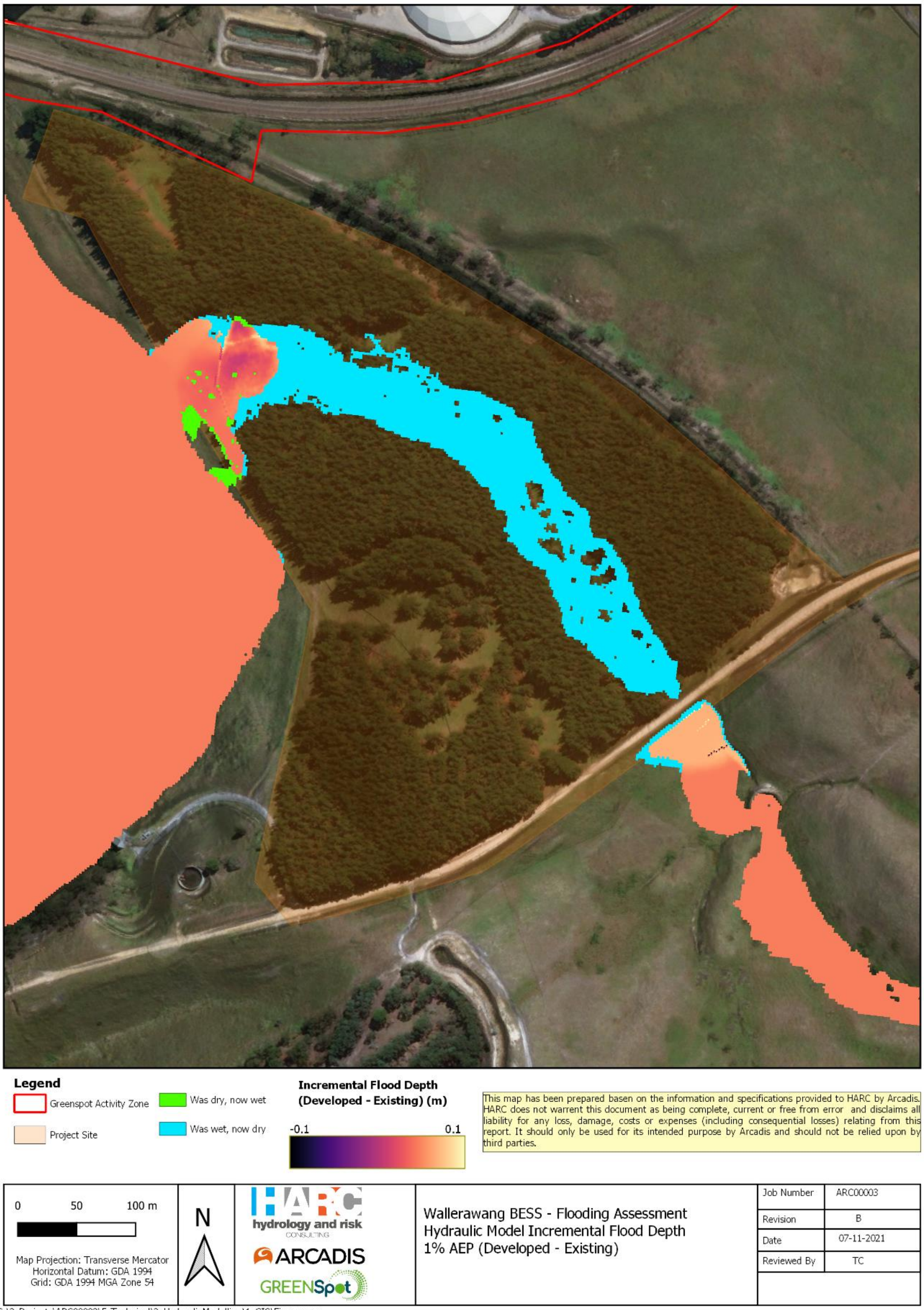
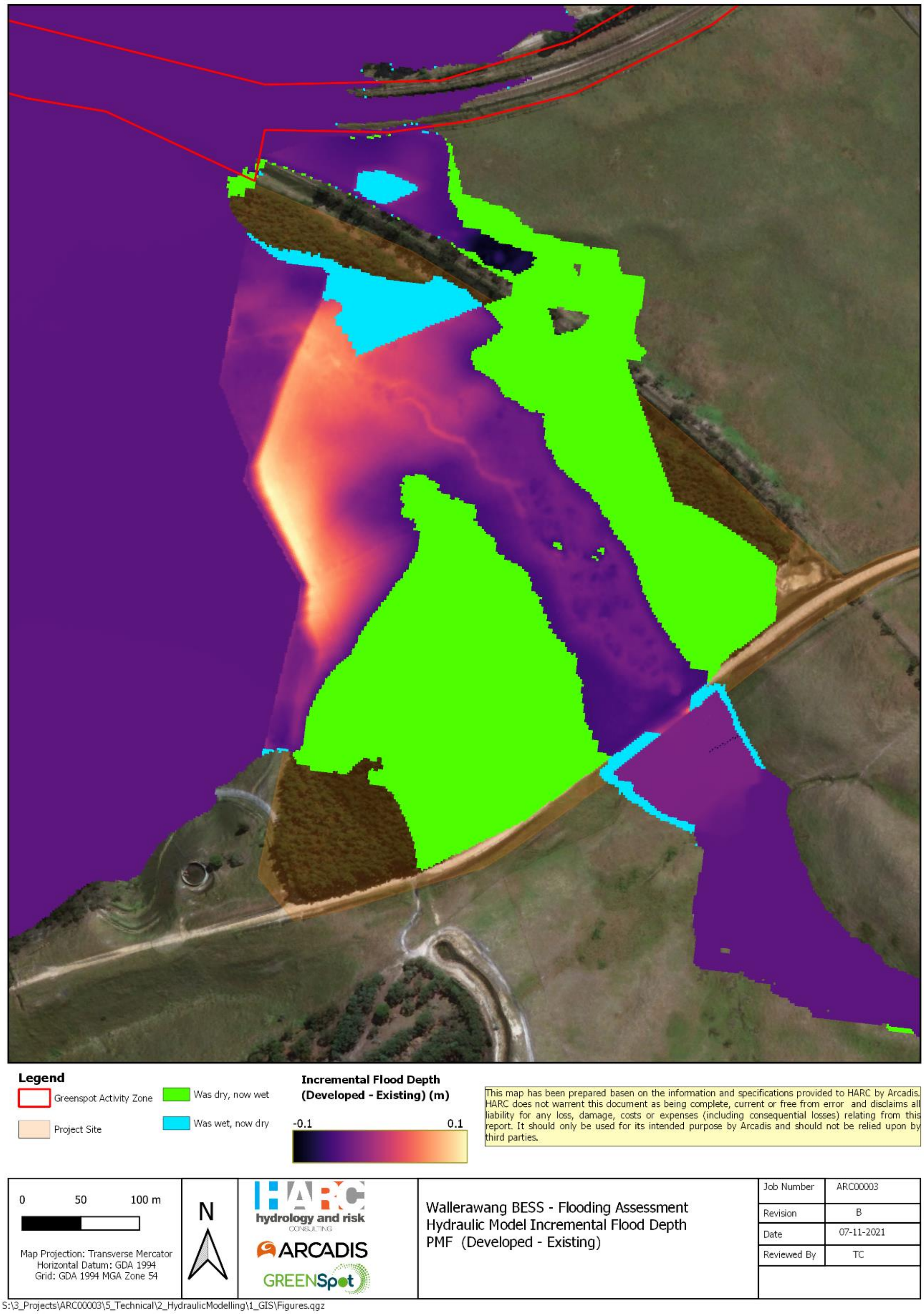


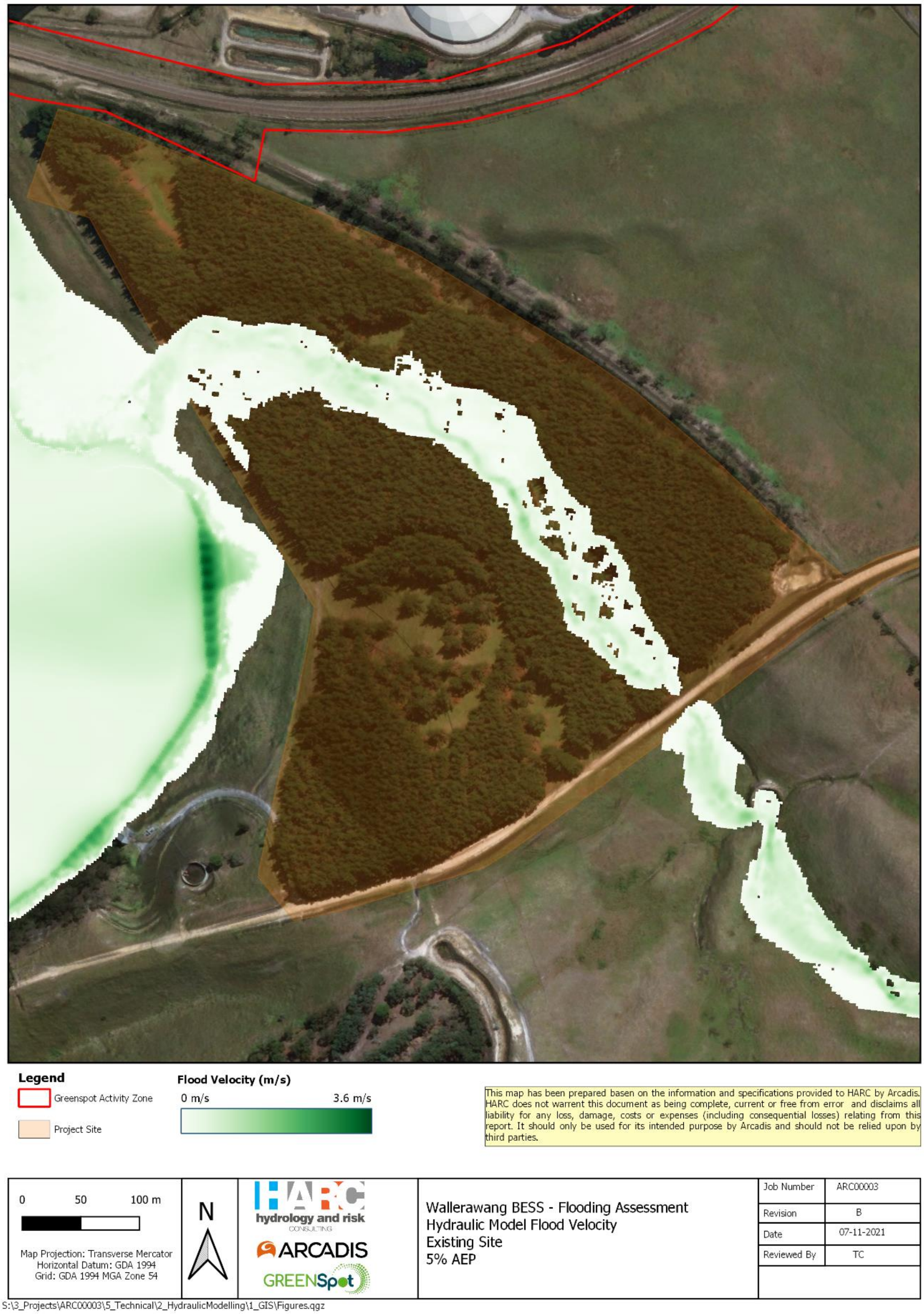
Figure 4-8 1% AEP Incremental flood depth (Development minus Existing)



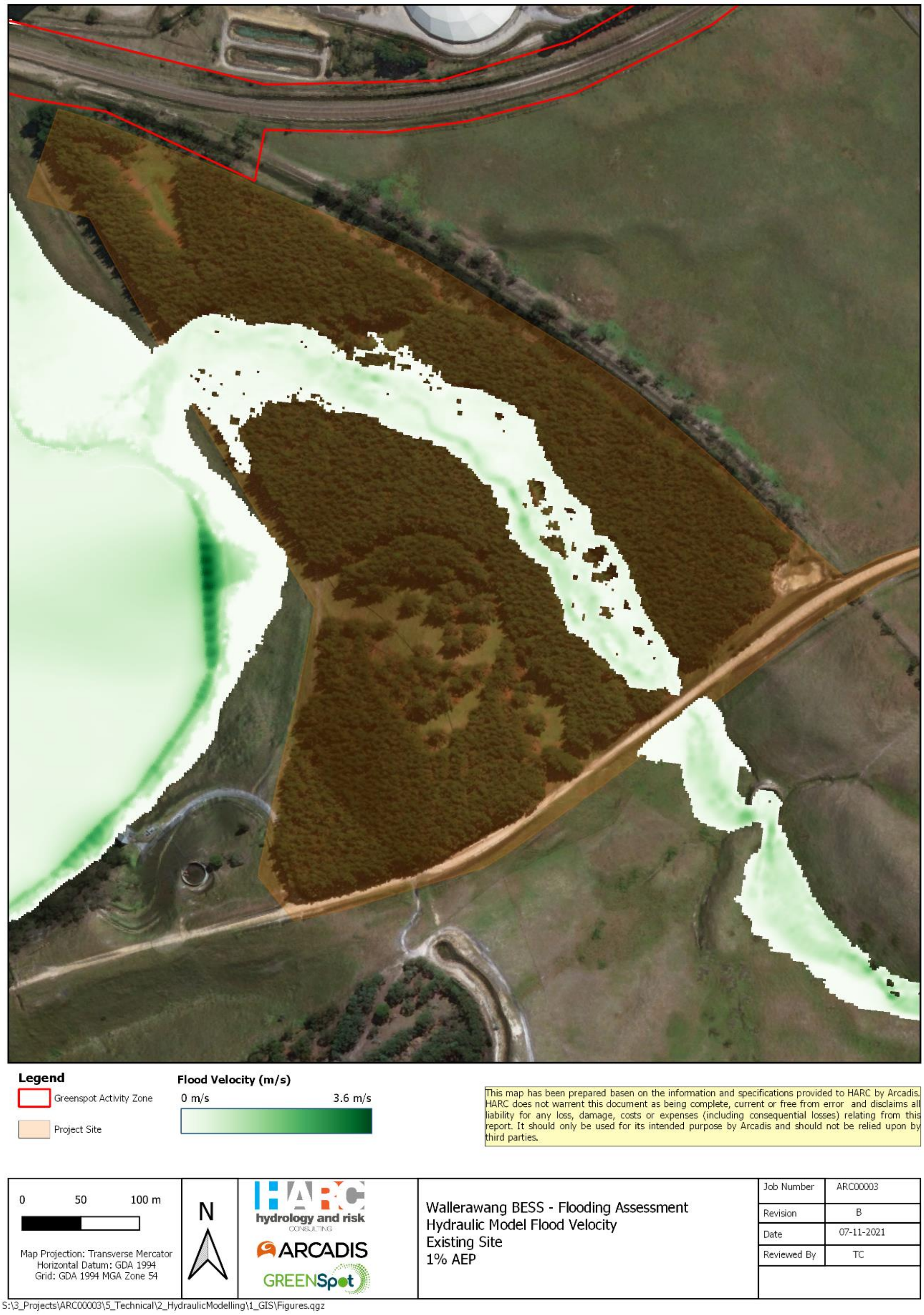
- Figure 4-9 PMF Incremental flood depth (Development minus Existing)

4.3 Velocities

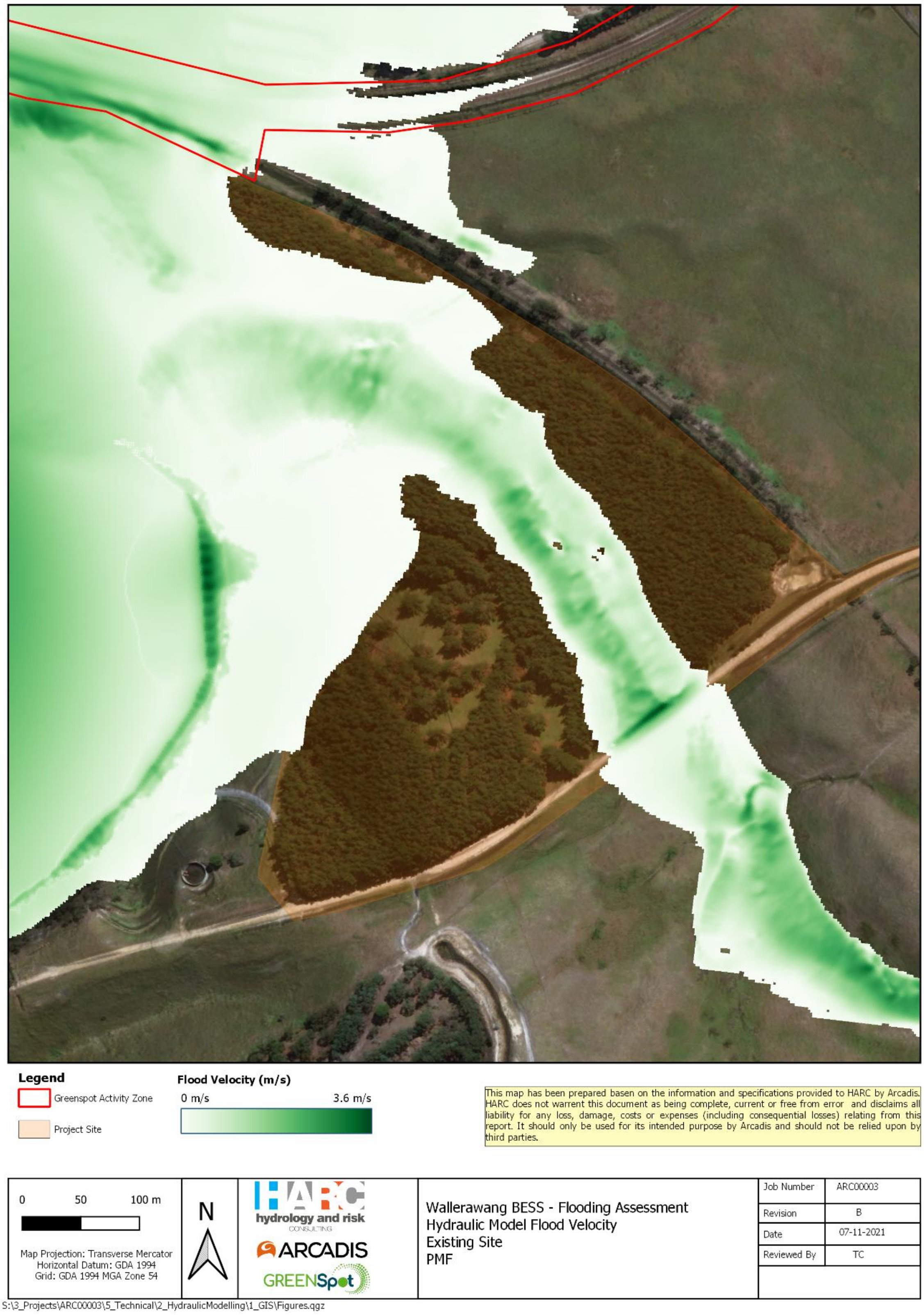
The velocity results for the existing and proposed developed conditions for each of the AEP events modelled are shown in Figure 4-11 and Figure 4-10.



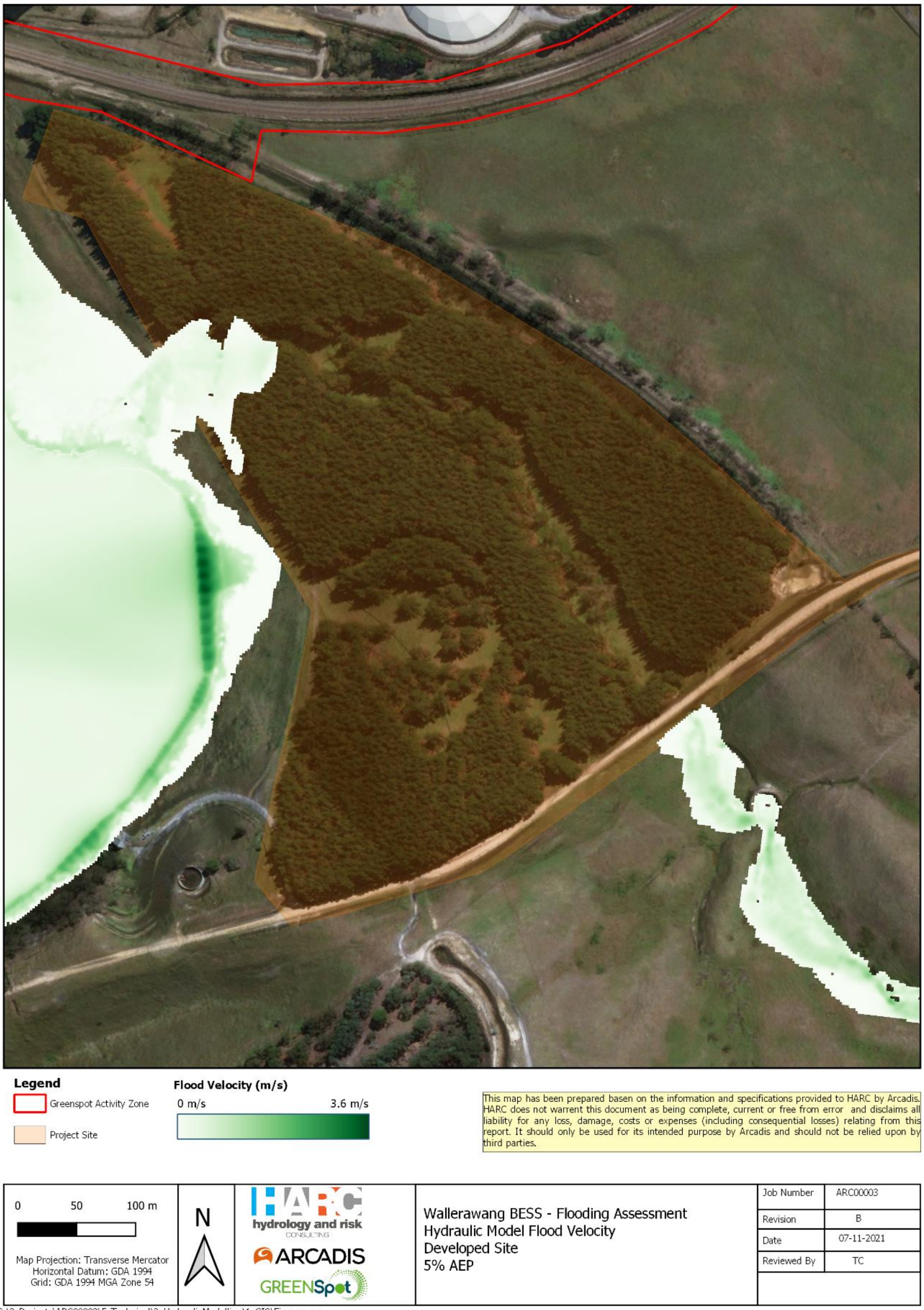
- Figure 4-10 5% AEP Velocity – Existing



■ Figure 4-11 1% AEP Velocity - Existing



■ Figure 4-12 PMF Velocity – Existing



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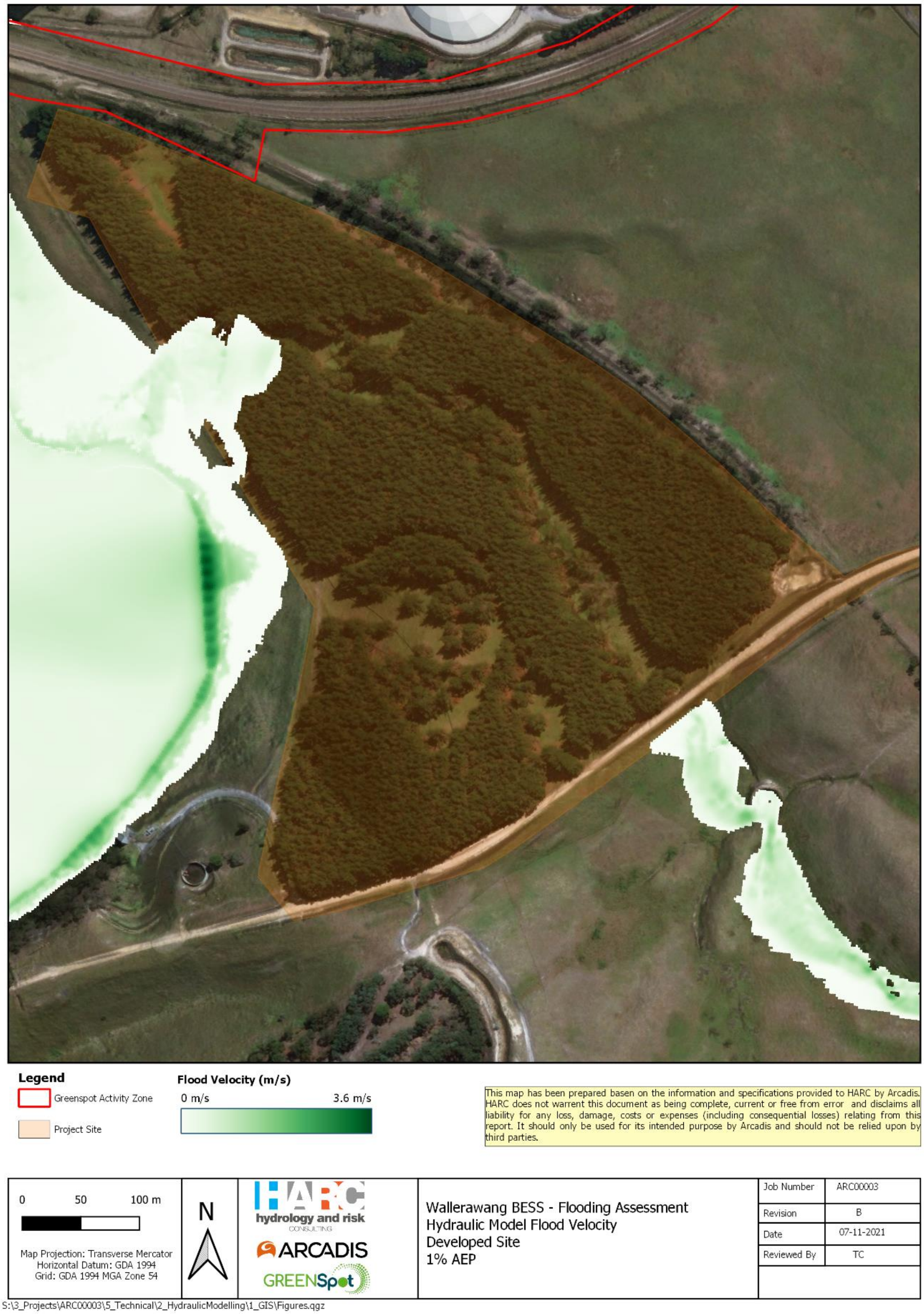
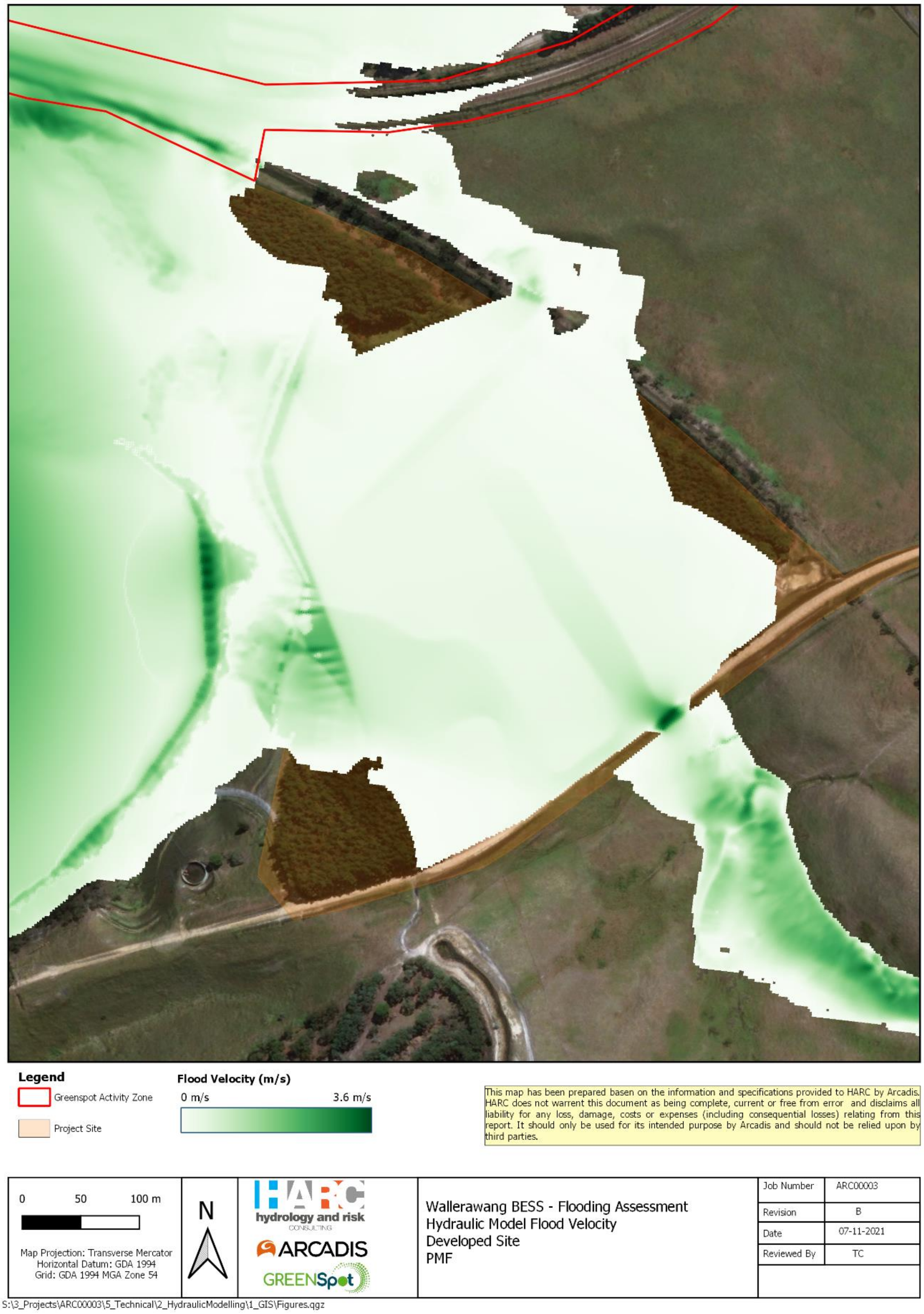
- Figure 4-13 5% AEP Velocity – Proposed Conditions



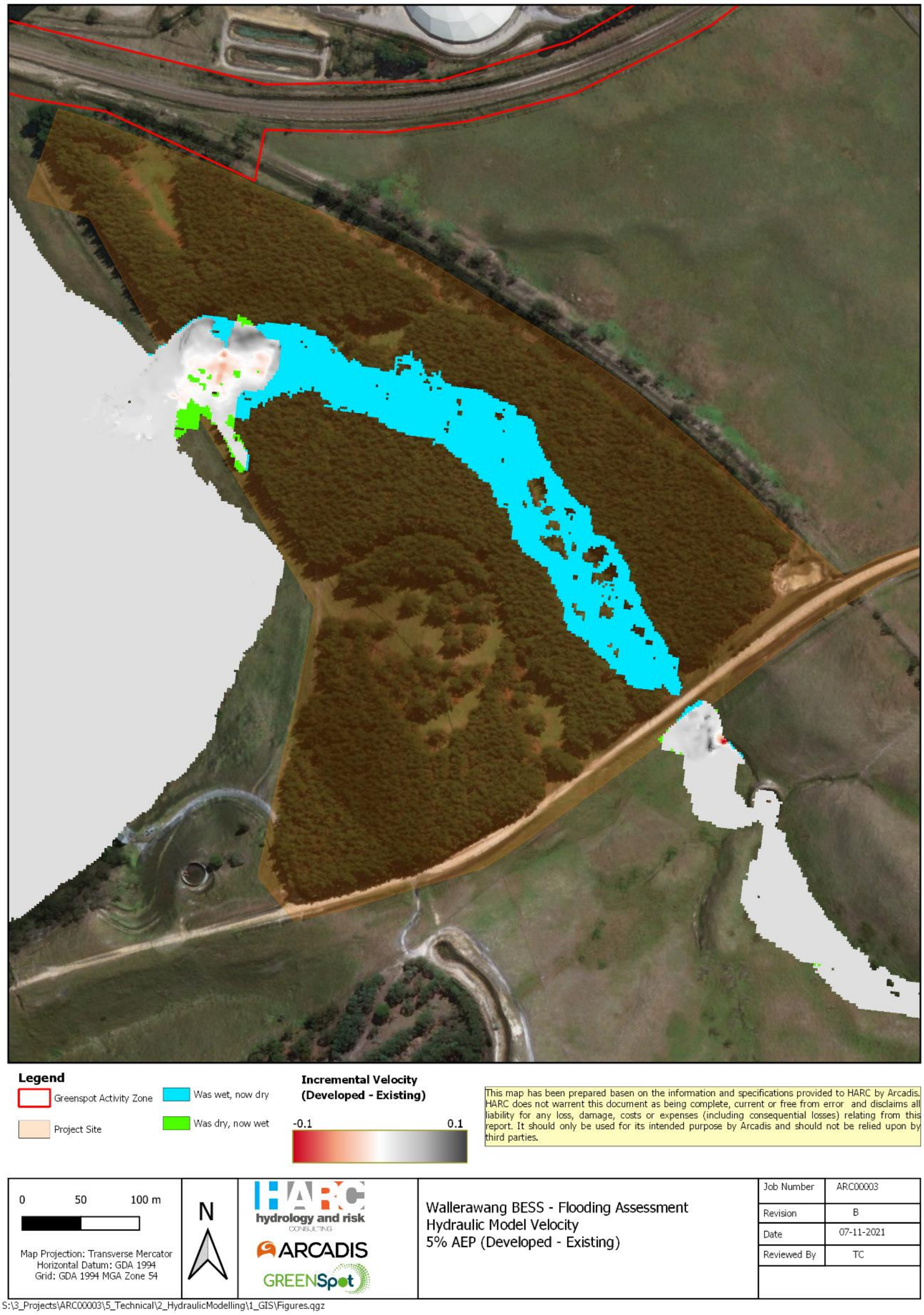
- Figure 4-14 1% AEP Velocity – Proposed Conditions



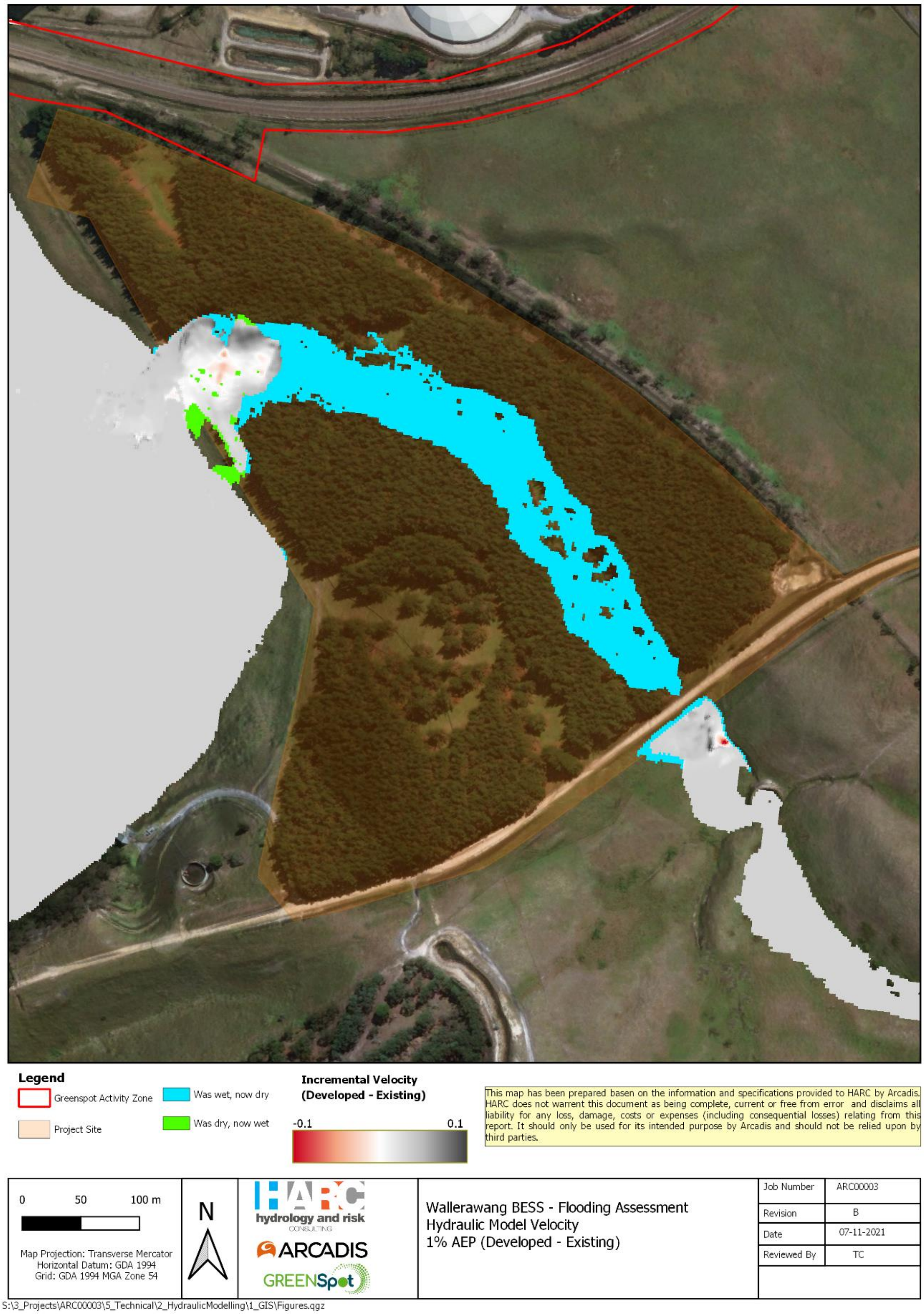
- Figure 4-15 PMF Velocity – Proposed Conditions

4.4 Velocity Afflux

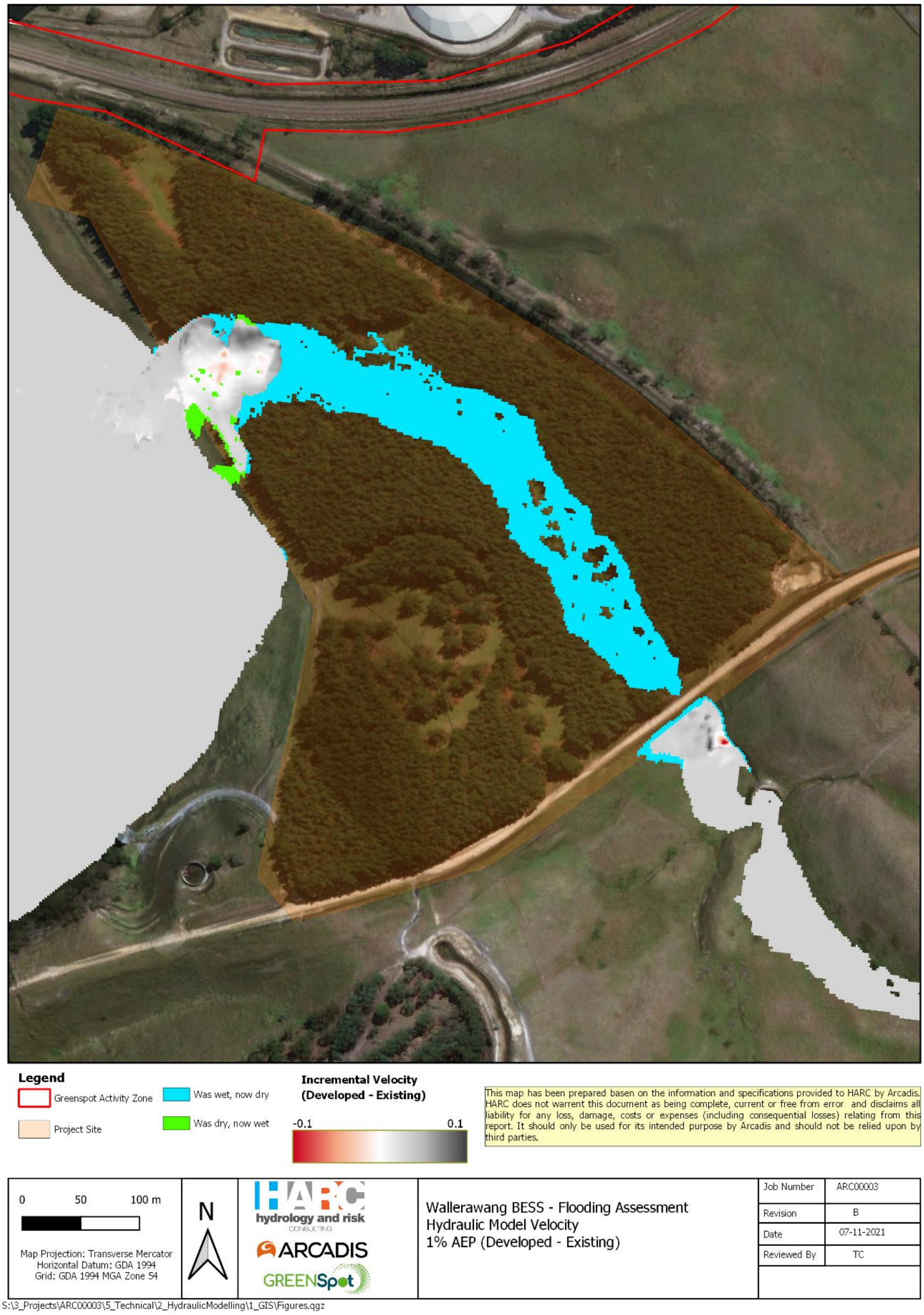
The velocity afflux results for the proposed conditions for each of the AEP events modelled are shown in Figure 4-17 and Figure 4-18. The velocity afflux was calculated by subtracting the proposed conditions modelled velocities from the existing conditions modelled velocities. A positive number means that the velocities have gone up compared to the existing velocities and a negative number means the velocities have gone down.



- Figure 4-16 5% AEP Afflux (Velocity)



▪ Figure 4-17 1% AEP Afflux (Velocity)



- Figure 4-18 PMF Afflux (Velocity)

5. Discussion

New amendments to the 'Flood Planning' provisions of the Standard Instrument local environmental plans Schedule 4 of the EPA Regulation 2000 took effect on 14 July 2021. The new provisions have been introduced in connection with the NSW Government's new 'flood-prone land package' which aims to improve the management of flood risk in light of recent flooding events that have caused significant risk to life and damage to property, including up to and beyond the 1% annual exceedance probability (AEP) flood level. To achieve this, consent authorities need to consider the full range of flood behaviour, such as the upper bound of possible flood impacts (the probable maximum flood (PMF)) as well a more frequent event (e.g. 5% AEP).

From the modelling it shows that for both the 1% and 5% AEP events, there is very little impact on flood levels (up to 100 mm) and these impacts are very localised immediately downstream of where the proposed pipe discharges back onto the floodplain. All impacts are within the property owned by Greenspot. For both depth and velocity there is very little incremental impact between the existing and development scenarios for all AEPs.

For the PMF scenario, there is additional flooding across the site. This can be seen in Figure 4-6 and Figure 4-9 showing additional sheet flow (which averages 100mm of depth) across the proposed site.

6. Conclusion

Based on the results of the hydraulic analysis presented in this report for the development of a Battery Energy Storage System, the following conclusions have been drawn:

- In general, the works proposed continue to maintain the free passage of flood waters up to and including the 1% AEP event.
- There is a slight (up to 100 mm) increase in flood levels immediately downstream downstream of where the proposed pipe discharges back onto the floodplain. All impacts are within the property owned by Greenspot.
- There is negligible increase in velocity for the 5% AEP and 1% AEP scenarios.
- For the PMF scenario, there is additional sheet flooding (which is approximately 100 mm deep) across the proposed site.

7. References

Laurenson, E.M., Mein, R.G. & Nathan, R.J. (2010), RORB Version 6 Runoff Routing Program User Manual, January 2010

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