STATE EMERGENCY SERVICE

TASMANIAN STRATEGIC FLOOD MAP TASMAN STUDY AREA DESIGN FLOOD MODELLING

ADDENDUM TO CALIBRATION REPORT

MARCH 2023

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Project Tasmanian Strategic Flood Map Tasman Study Area Design Flood Modelling	**Project Number** **120038**
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Revision History

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LIST OF ACRONYMS

AEP	Annual Exceedance Probability
AMS	Annual Maximum Series
ARF	Areal Reduction Factor
ARR	Australian Rainfall and Runoff
ATP	Areal Temporal Patterns
Bureau/BoM	Bureau of Meteorology
CC	Climate Change
CFEV	Conservation of Freshwater Ecosystem Values (DPIPWE/DNRE)
CL	Continuing Loss
DEM	Digital Elevation Model
DNRE	Department of Natural Resources and Environment Tasmania (formerly DPIPWE)
DPIPWE	Department of Primary Industries, Water and Environment
DRM	Direct Rainfall Method
DTM	Digital Terrain Model
FFA	Flood Frequency Analysis
FLIKE	Software for flood frequency analysis
FSL	Full Supply Level
GIS	Geographic Information System
GEV	Generalised Extreme Value distribution
HAT	Highest Astronomical Tide
has	Human Settlement Area
ICM	Infoworks ICM software (Innovyze)
IL	Initial Loss
IFD	Intensity, Frequency and Duration (Rainfall)
ISIS	ISIS 2D modelling software
LiDAR	Light Detection and Ranging
mAHD	meters above Australian Height Datum
NTC	National Tide Centre
PERN	Catchment routing parameter in RAFTS
Pluvi	Pluviograph – Rain gauge with ability to record rain in real time
PTP	Point Temporal Patterns
R	Channel routing param in WMAWater RAFTS WBNM hybrid model
RAF	RAFTS Adjustment Factor
RAFTS	hydrologic model
RCP	Representative Concentration Pathways (RCPs) (CC scenarios)
RORB	RORB hydrological modelling software
SES	State Emergency Service
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydrodynamic model)
TP	Rainfall Temporal Patterns

1. INTRODUCTION

This report is an addendum to the Tasmanian Strategic Flood Map Tasman Study Area Calibration Report (WMAwater, 2023). The study area, available data, model calibration, limitations and uncertainty statements are provided in the calibration report.

This report outlines the data, methodology and the results of modelling the design flood events for the Tasman Study Area.

2. DATA

2.1. Previous Flood Studies

Previous flood studies in the study area were provided to WMAwater as part of the project data library. The studies that include modelling of the 1% AEP event are listed in Table 1.

Table 1: Previous flood studies

Flood study name	Study year	Study area	Available data
Richmond Flood Plain Study	1995	Coal River (for the township of Richmond)	Some design flows and levels contained in the report. Flood mapping not available.
Sorell, Rivulet Flood Study	2006	Sorell Rivulet (for the township of Sorell)	Some design flows and levels contained in the report. Flood mapping available (PDF format only).

2.2. Flow Data

Flood Frequency Analysis (FFA) was performed on annual maximum series (AMS) from flow gauges within the catchment. The gauges used for FFA are shown in Table 2. The other gauges in the study area were not included in the FFA due to insufficient record length, inconsistent datasets, unreliable rating curves and/or being impacted by dams. More detail on the quality of the gauge data is provided in the calibration report (WMAwater, 2023).

Table 2: Flow gauges used for FFA

Gauge number	Gauge name	River	Period of record	Number of points in AMS
3203-1	Coal River at Baden	Coal	13/07/1971 – present	44
3208-1	Coal River at Richmond	Coal	25/01/1995 – present	21

2.3. Design Inputs

The design inputs used in the study (Intensity Frequency Duration (IFD) depths, losses, pre-burst rainfalls, Areal Reduction Factors (ARFs) and temporal patterns) were obtained through the ARR Data Hub (Babister et al, 2016) and the Bureau of Meteorology website (Bureau of Meteorology, 2019).

2.3.1. Design Rainfall Depths and Spatial Pattern

Intensity Frequency Duration (IFD) information was sourced from the Bureau of Meteorology website (Bureau of Meteorology, 2019). IFD information was sourced for each individual sub-catchment to give a spatial pattern across the study area. Examples of sub-catchment rainfalls are shown in Figure A 1 to Figure A 3.

2.3.2. Temporal Patterns

ARR 2016 Book 2 Chapter 5 (Ball et. al., 2019) recommends the use of areal temporal patterns for catchments greater than 75 km^2. Therefore, for the flood frequency analysis, the areal temporal patterns relevant to this location were downloaded from the ARR Data Hub. An example of the temporal patterns downloaded from the Data Hub is shown in Figure A 4.

For selection of the final design runs applicable to the entire study area, areal and point temporal patterns were downloaded from the ARR Data Hub. Temporal patterns were filtered for embedded bursts and in some cases patterns with large, embedded bursts causing significant outliers were removed. When assessing the reference critical flow for each sub-catchment (as described in the Hydrology Methods Report (WMAwater, 2021a)), point temporal patterns were used for sub-catchments with an upstream area of less than 75 km^2 or used to assess shorter storms if the critical duration on a larger catchment was identified as 12 hours (the shortest duration available with areal temporal patterns).

2.3.3. Pre-burst

Pre-burst rainfall depths were taken from the ARR Data Hub as a ratio of the IFD depths. As ILs calibrated to the FFA were greater than 0 there was no need to include sensitivity to adding a pre-burst temporal pattern for this study area, as the pre-burst has effectively been removed from the IL with some IL depth remaining.

2.3.4. Losses

Initial values for sub-catchment initial loss (IL) and continuing loss (CL) were derived from the unpublished Hydrologic Soil Groups of Tasmania data that was provided for use in this project (DPIPWE, 2019).

2.3.5. Baseflow

In line with ARR 2016 Book 5 Chapter 4 (Ball et. al., 2019), where baseflows of less than 5% are considered a small component compared to runoff, a simplified approach to baseflow calculations was undertaken. One of the main two calibration events (2009) did have a smaller event in the week prior to the main event, so flows were still higher than 5% of the event peak at the beginning of this event. However, the top 10 AMS events at Richmond were reviewed on DNRE's data portal (DNRE 2022) and all the others had baseflow less than 5% of the event peak, in most cases less than 1%, so it was considered appropriate to assume typical baseflow of less than 5% of event peaks. So therefore baseflows will be a small component of the hydrograph for the AEPs of interest (2%, 1% and 0.5%) and therefore baseflow was not included in the design events.

2.3.6. Direct Rainfall

Two hour direct rainfall storms were created using each sub-catchment's IFD depths using the method described in the Hydrodynamic Methods Report (WMAwater, 2021b).

2.3.7. Climate Change

2.3.7.1. Rainfall Factors

Climate change factors for the study area were downloaded from the ARR Data Hub. ARR recommends the use of the RCP4.5 and RCP8.5 values, however the Tasmanian Interim Planning Scheme recommends the use of RCP8.5 and this has been adopted for this project. Using RCP8.5 results for the year 2090, gives a rainfall scaling factor of 16.3% to the IFDs.

2.3.7.2. Boundary Conditions

Sea level rise was included in the climate change scenario and was applied at the downstream boundary of the hydrodynamic model. The rise in water level was taken from the Tasmanian Local Council Sea Level Rise Planning Allowances, which uses sea level rise projections based on RCP 8.5 for 2100. This gave a rise in sea level of 0.86 m for the Tasman Council area.

The levels from this document were deemed most appropriate to be consistent with best practise planning around Tasmanian Councils.

3. OVERVIEW OF METHODOLOGY

The hydrological and hydrodynamic design modelling methodology has been outlined in the Hydrology Methods Report (WMAwater, 2021a) and the Hydrodynamic Methods Report (WMAwater, 2021b). Details on the methods are only included in this report where they deviate from the methods described in these reports or are specific for this catchment.

The modelling method for the design events includes the following steps.

- Data preparation
    - Fitting FFA to suitable flow records
    - Extraction of design data – IFDs, temporal patterns, pre-burst rainfalls from ARR DataHub (automated in the modelling process), derivation of direct rainfall storms
- Hydrologic modelling
    - Identification of flow gauge locations
    - Identification of dam and diversion locations
    - Sub-catchment delineation
    - Include dam storage and spillway ratings where required
    - Event calibration for PERN parameter and event losses, using automated WMAwater RAFTS modelling tool, IDW rainfall surfaces and available flow data.
    - Output event sub-catchment rainfalls, routing parameters and event losses for input to hydraulic model
    - Calibration of design losses to FFA using automated WMAwater RAFTS model
    - Run design events in WMAwater RAFTS modelling tool, with design data, calibrated routing parameters and design losses. Outputs design sub-catchment rainfalls for input to hydrodynamic model.
- Hydrodynamic modelling
    - Run design events and direct rainfall through the calibrated hydrodynamic model with the applicable downstream boundary levels and dam initial conditions.
    - Output design event and direct rainfall results for processing.
- Mapping
    - Convert design event and direct rainfall results to a grid format with a grid resolution of at least 10 m.
    - Envelope design event results to produce the maximum envelope of the inputs.
    - Filter direct rainfall results using a peak flood depth filter of 0.1 m. Clip direct rainfall results to the design event envelope.
    - Map the design event envelope and filtered direct rainfall results.

4. CALIBRATION OF DESIGN LOSSES

FFA was undertaken at the gauges identified in Table 2. The results of the FFA are shown in Figure 1 and Figure 2. The fitting method and distribution that provided the best fit to the data at each site is shown in Table 3.

Table 3: Fitting method and distribution used for FFA

Gauge number	Gauge name	Fitting method	Distribution
3203-1	Coal River at Baden	Bayesian	Log Pearson III
3208-1	Coal River at Richmond	Bayesian	Log Pearson III

The calibrated external hydrologic model for each study area was run through the solver and the initial and continuing losses that best matched the curve were estimated. As the events of relevance to this study are of 2% AEP or larger, the results were weighted to this end of the FFA curve. The catchment-average continuing loss was distributed across the study area using the hydrological soil group final infiltration rates. The gauge at Richmond covers a much larger proportion of the catchment area than the gauge at Baden, is at a major town, and is more representative of most developed areas in the study area. Therefore, the calibration was focused on fitting flows at Richmond despite the much shorter record length available at this gauge.

The percentage differences between the FFA and the modelled peak flow for the 2% and 1% AEP events are shown in Table 4. The modelled data provided a good fit to the FFA 1% and 2% AEP peak flows at Richmond. Use of the same losses over the catchment results in an underestimation of flows at Coal River at Baden, but the modelled flows are within the confidence intervals of the FFA.

Table 4: FFA and modelled peak flows

Gauge name	FFA 2% AEP peak flow (m^3/s)	Modelled 2% AEP peak flow (m^3/s)	2% AEP percent difference	FFA 1% AEP peak flow (m^3/s)	Modelled 1% AEP peak flow (m^3/s)	1% AEP percent difference
Coal River at Baden	78	60	-24%	92	71	-22%
Coal River at Richmond	325	329	1%	399	406	2%

The adopted loss values are shown in Table 5, and comparisons to site FFAs are shown in Figure 1 and Figure 2.

Table 5: Adopted losses

Initial Loss (mm)	Continuing Loss (mm/h)			
	Soil Type A	Soil Type B	Soil Type C	Soil Type D
30	5	2.6	1.2	0.6

5. DESIGN EVENT MODELLING

5.1. Design Event Selection

Design inputs were run through the hydrological model across the entire study area with a range of ARFs to select representative ARFs, storm durations and temporal patterns to be run through the hydrodynamic model. The selected storms and the number of sub-catchments best represented by each are shown in Table 6. The temporal patterns for each selected run are shown Figure 3 and Figure A 4.

Table 6: Selected storms for each AEP with the number of sub-catchments best represented by each set

AEP	Storm duration (min)	ARF bin	# sub-catchments
2%	720	45	26
2%	1080	120	9
2%	1080	250	35
2%	1440	250	22
1%	720	45	46
1%	1080	120	4
1%	1080	250	21
1%	1440	250	21
0.5%	720	45	61
0.5%	1080	120	9
0.5%	1080	250	9
0.5%	1440	250	13

Diagram 1 shows the ARF-duration-TP set used to give representative flows for each sub-catchment for the 1% AEP event. Headwater sub-catchments where only direct rainfall is applied are also shown. In the headwater catchments, direct rainfall was defined as the dominating event, with the rainfall intensities factored to account for losses via a runoff coefficient. For this study area, a runoff coefficient of 50% was adopted. Although direct rainfall is applied to all sub-catchments, the mapping process detailed in Section 3 ensures that primary flow paths are not defined by this event.

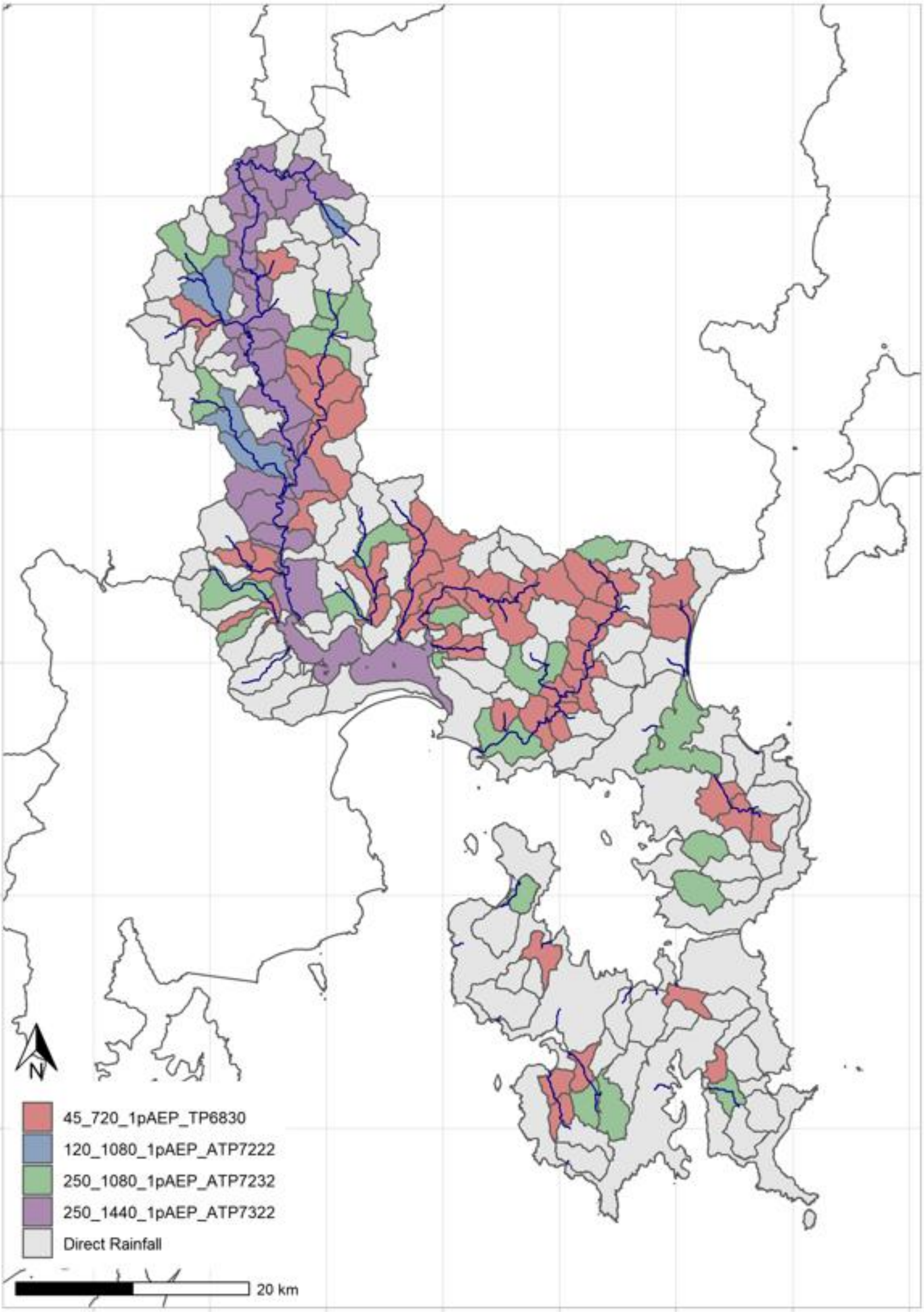

Diagram 1: ARF set relevant for each sub-catchment for the 1% AEP event

The selection of four ARF-duration-TP sets per AEP does introduce errors when compared to running the ideal ARF-duration-TP set through the hydrodynamic model for each sub-catchment, however running thousands of runs of the hydrodynamic model is not computationally feasible.

The percentage errors for each sub-catchment are shown in Figure B 1 to Figure B 3 and a summary of the magnitude of the errors is shown in Table 7. Each sub-catchment's absolute percentage error is calculated using the following equation:

$SC\_Q\_Peak_{ref}$ = Sub-catchment peak flow run with ARF from that sub-catchment's ARF bin, with critical duration calculated at this gauge, and TP above the mean selected.

$SC\_Q\_Peak_{sel}$ = Sub-catchment peak flow run with ARF, storm duration and TP from the selected pattern as shown in Diagram 1

$$\text{Absolute subcatchment percentage error} = \left| \frac{(\mathrm{SC\_Q\_Peak}_{sel} - \mathrm{SC\_Q\_Peak}_{ref})}{\mathrm{SC\_Q\_Peak}_{ref}} \right| \times 100$$

Table 7: Sub-catchment errors using the ARF-TP-duration sets shown in Table 6 for each AEP

AEP	Absolute sub-catchment error		
	Mean across sub-catchments	90^{th} %ile across sub-catchments	Max of all sub-catchments
2%	3.6	7.5	10.5
1%	3.4	7.1	8.6
0.5%	3.3	7.1	12.7

The selected storms and direct rainfall were then run through the calibrated hydrodynamic model as documented in the calibration report (WMAwater, 2023). For the design event modelling, a static tailwater level set to the highest astronomical tide was adopted for the downstream boundary. This data was provided by the National Tide Centre (NTC) in 5 km² grid cells, and the mean value of these grid cells within the study area was used.

Table 8 below summarises the downstream boundary levels and dam initial conditions for each design event.

Table 8. Downstream boundary levels and dam initial conditions for each AEP

AEP	Downstream boundary	Craigbourne Dam	Duckhole Dam
2%	HAT (0.78 mAHD)	FSL (166.2 mAHD)	FSL (136.35 mAHD)
1%			
0.5%			
1% CC	HAT + sea level rise (1.64 mAHD)		

The following issues were raised in the calibration report as items that may require further attention in the design modelling (data permitting):

- For Craigbourne Dam, the DEM of the dam was artificially lowered in the absence of bathymetry to enable the assessment of the dam below the FSL (Section 5.1 of the calibration report). The dam is started at FSL in the modelled design events and therefore, this item should not impact the outcomes of the design modelling.
- For Craigbourne Dam, the secondary spillway was modelled in the hydrodynamic model using a rating that was approximated from a combined rating of the primary and secondary spillway in the 1995 Richmond Flood Study (Section 5.5 of the calibration report). The secondary spillway is not activated in the modelled design events and therefore, this item should not impact the outcomes of the design modelling.
- For Richmond Bridge, an arch-shaped cross-section was approximated using an obvert level in the 1995 Richmond Flood Study and available photography (Section 5.4 of the Calibration report). Additional data was not available to enable the further refinement of this cross-section.
- For Coal River, it was noted that the absence of river channel bathymetry was affecting the match to the recorded water levels at the Coal River at Richmond gauge (Section 6.3.5 of the calibration report) and may be resulting in higher-than-expected water levels through Richmond (Section 6.4.1 of the calibration report). Additional data was not available to enable the further refinement of the river channel.

Bathymetry of the Orielton Lagoon, Pitt Water, and Blackman Bay areas were not available in the supplied DEM (which adopts a value of -10 mAHD through these areas). It is recommended that the design mapping in this area is disregarded, until such time that updated topographic data is available and further modelling is undertaken.

5.2. Design Event Results

The results of the design event modelling are shown in Figure 4 to Figure 19 in terms of peak flood level, depth, velocity, and hydraulic hazard for the 2%, 1%, 1% CC, and 0.5% AEP design events. The results shown are of the design event envelope and filtered direct rainfall results, as detailed in Section 3. A critical event plot for the 1% AEP design event is provided in Figure 20.

For direct rainfall only, in some areas the peak flow for headwater catchments was found to be higher in the hydrodynamic model than in the external hydrologic model. To ensure that the overestimation of these peak flows in the headwater catchments would not impact the design results, the direct rainfall results were clipped to the design event envelope.

The outcomes of the design event modelling have been reviewed against the gauge FFA and previous flood studies.

5.2.1. Review of Results at Coal River at Baden

A review of the design flows produced from the hydrodynamic model at Coal River at Baden was undertaken, by comparing to the flows derived from the FFA. The modelled peak flows show a poor match to the FFA peak flows at this location (Table 9), however, this is consistent with the discussion presented in Section 4.

Table 9: Design flows Coal River at Baden

Parameter	2% AEP	1% AEP	1% AEP CC	0.5% AEP
Modelled peak flow (m^3/s)	63	75	95	93
FFA peak flow (m^3/s)	78	92	n/a	104
Peak flow difference (%)	-19%	-18%	n/a	-11%

5.2.2. Review of Results at Coal River at Richmond

A review of the design flows produced from the hydrodynamic model at Coal River at Richmond was undertaken, by comparing to the flows derived from the FFA. The modelled peak flows show a fair match to the FFA peak flows at this location (Table 10). The variances between the final modelled flows and the calibrated flows shown in Section 4 are largely due to the binning of ARFs and TPs approach taken for this study. As described in the Hydrology Methods Report (WMAwater, 2021a), this is set up to be slightly conservative so at times may cause an increase in flows.

Table 10: Design flows at Coal River at Richmond

Parameter	2% AEP	1% AEP	1% AEP CC	0.5% AEP
Modelled peak flow (m^3/s)	360	439	563	547
FFA peak flow (m^3/s)	325	399	n/a	470
Peak flow difference (%)	+11%	+10%	n/a	+16%

5.2.3. Comparison to Previous Flood Studies

5.2.3.1. Coal River

A flood study was undertaken by Hydro-Electric Commission (HEC) in June 1995 (HEC, 1995) of the lower reaches of Coal River in Richmond. As noted in Table 1, the flood study report contained some references to the estimated design flows and levels. Flood mapping was not available.

The 1995 study involved the estimation of the 5%, 2%, and 1% AEP design flows and levels. The design flows were derived from a RORB hydrologic model and design levels were derived from a MIKE-11 hydraulic model. It is noted that the design flows presented in the 1995 study are greater than that of the present study (1% AEP design flow of 700 m^3/s (HEC) and 440 m^3/s (present study), at Coal River at Richmond).

Further investigation into the flows noted that the difference in the 1% AEP design flow is largely due to limitations of the 1995 study in calibrating their design estimates to their FFA estimates. At the time of the 1995 study, there was only 5 years of record at Coal River at Richmond and the FFA at the site was therefore undertaken using these records, supplemented with flows derived by transformation of the pre-dam records at Richmond and transposition of Coal River u/s White Kangaroo flows. The exact process for accounting for the role of the dam in the transformation is unclear, however it is noted that the dam has a significant impact on even large flow events at Richmond. This provided 23 additional years (pre-dam) and 2 additional years (post dam).

The 1995 study presented a 1% AEP FFA estimate of 530 m^3/s at Richmond however, ultimately adopted a 1% AEP design flow of 700 m^3/s. As detailed in Section 4, the present study uses a 1% AEP FFA flow of 400 m^3/s at Richmond, derived using the record now available at this gauge post dam construction.

Given the 21 additional years of record at Coal River at Richmond since the 1995 study, it is recommended that, if a detailed flood study is undertaken at this site in future, the decisions of the 1995 study are revisited to better incorporate the FFA estimate at the site, which would negate the need to use complicated assumptions to transform pre-dam data. This is likely to decrease the 1% AEP design flows through Richmond.

The estimated design levels from the 1995 study and the present study for the 2% and 1% AEP design events are compared in Table 11. The location of the comparison points and the modelled flood extent for the 1% AEP design event in the present study are shown in Diagram 2.

Table 11. Design levels along Coal River (mAHD)

Comparison Point	2% AEP			1% AEP		
	1995 Study	Present Study	Difference (m)	1995 Study	Present Study	Difference (m)
A	9.6	10.3	+0.7	10.4	10.7	+0.3
B	8.8	9.8	+1.0	9.7	10.4	+0.7
C (u/s of bridge)	7.6	8.2	+0.6	8.5	8.8	+0.3
D (d/s of bridge)	7.4	6.9	-0.5	8.1	7.3	-0.8
E (u/s of weir)	7.1	6.7	-0.4	7.7	7.0	-0.7
F (d/s of weir)	6.2	6.7	+0.5	6.8	7.0	+0.2
G	4.7	5.7	+1.0	5.3	6.1	+0.8

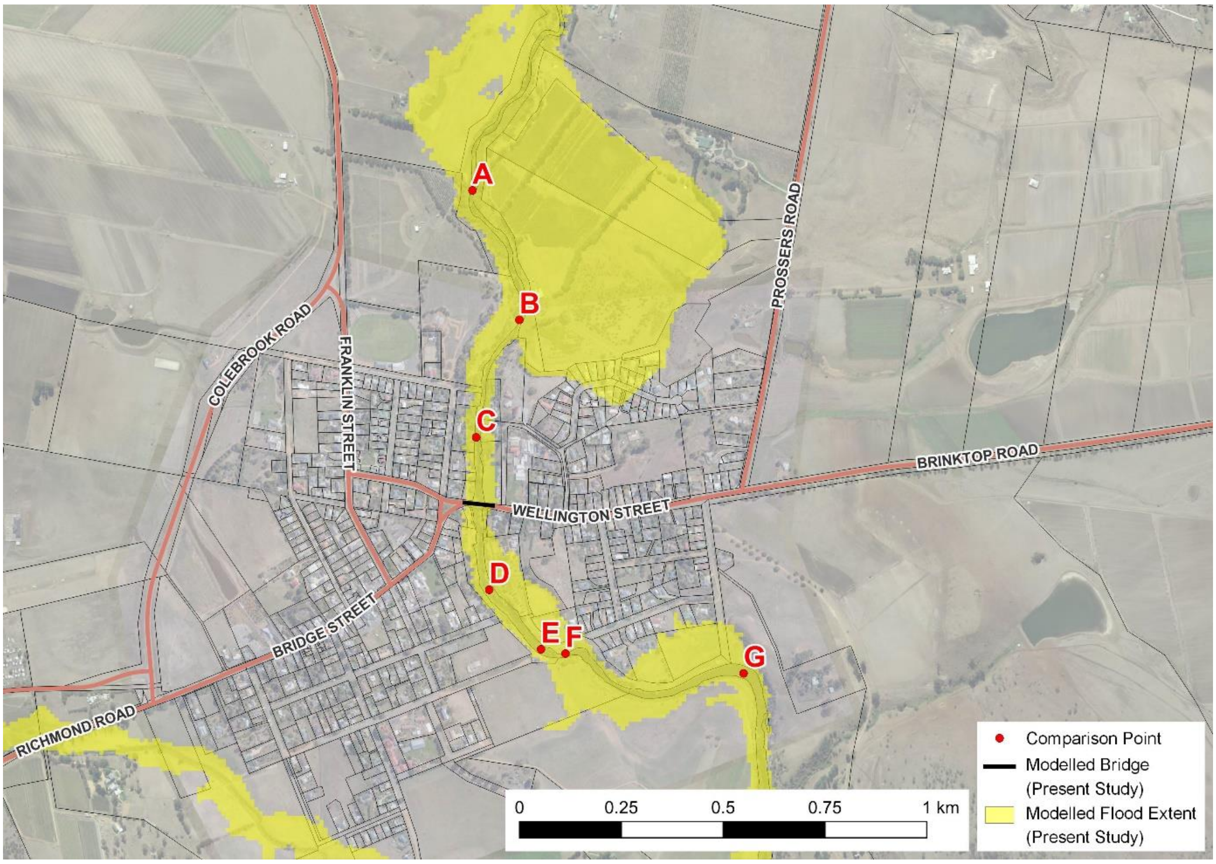

Diagram 2. Modelled flood extent for the 1% AEP design event at Richmond

The 1% AEP design levels in the present study are generally higher than those of the 1995 study. This is unexpected, given the 1% AEP design flow in the present study is less than that of the previous study.

Further investigation into the modelled levels suggests that the present study may be underestimating the conveyance through the narrowing of the waterway (between points B and

C) and the conveyance through Richmond Bridge (between points C and D). For example, the difference in the 1% AEP design level between points C and D is 1.5 m in the present study (compared to 0.4 m in the previous study). The lack of river bathymetry through this area may also be contributing to the elevated water levels. As noted in Section 5.1, additional data was not available to enable the further refinement of Richmond Bridge or river bathymetry, and the method used follows the agreed methods outlined in the Hydrodynamic Methods Report (WMAwater, 2021b).

5.2.3.2. Sorell Rivulet

A flood study was undertaken by Hydro Tasmania Consulting (HTC) in 2006 (HTC, 2006) of the lower reaches of Sorell Rivulet in Sorell. As noted in Table 1, the flood study report contained some references to the estimated design flows and levels. The flood study report also contained some flood mapping, which was georeferenced to enable a visual comparison to the estimated design extent.

The 2006 study involved the estimation of the 20%, 10%, 5%, and 1% AEP design flows and levels. The design flows were derived from a Hydstra hydrologic model and design levels were derived from a MIKE-11 hydraulic model. It is noted that the design flows presented in the 2006 study are greater than that of the present study (1% AEP design flow of 115 m^3/s (HTC) and 70 m^3/s (present study), approximately 1km upstream of Sorell Rivulet at Arthur Highway).

Further investigation into the flows noted that the design flows in the 2006 study were calibrated to the Iron Creek catchment (factored by catchment and rainfall). As detailed in Section 4, the design flows in the present study were derived from the Coal River catchment only. The Iron Creek catchment was not used in calibration in this study as the project calibration events were not significant at this site.

The estimated design levels between the 2006 study and the present study for the 1% AEP design events are compared in Table 12. The location of the comparison points and a comparison of the modelled flood extent for the 1% AEP design event between the 2006 study and the present study are shown in Diagram 3.

Table 12. Design levels along Sorell Rivulet (mAHD)

Comparison Point	1% AEP		
	2006 Study	Present Study	Difference (m)
H	18.86	18.63	-0.23
I	17.26	17.14	-0.12
J	16.23	16.08	-0.15
K	15.41	15.38	-0.03
L	14.45	14.71	+0.26

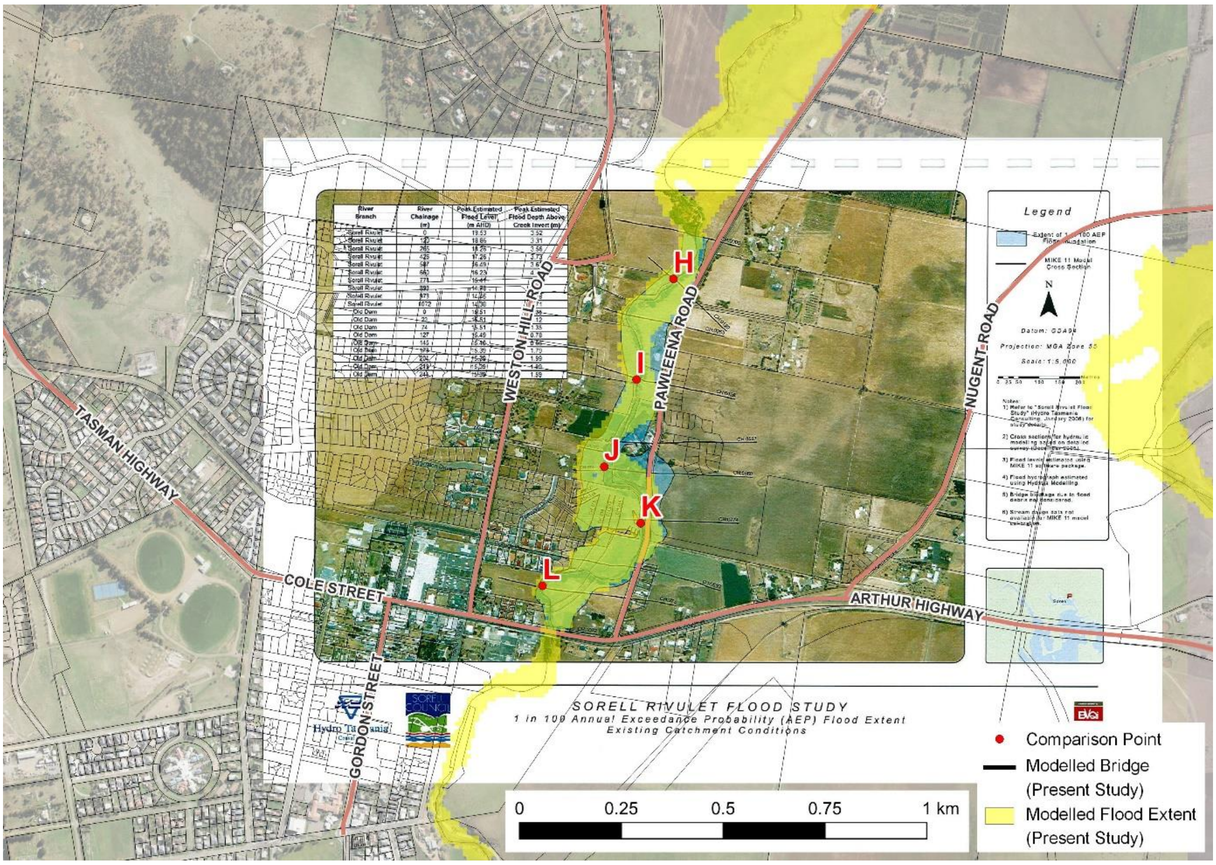

Diagram 3. Modelled flood extent for the 1% AEP design event at Sorell (2006 study shown in the background courtesy of HTC, 2006)

The 1% AEP design levels in the present study are generally lower than those of the 2006 study. This is expected, given the 1% AEP design flow in the present study is less than that of the previous study.

It is noted that the 1% AEP design level in the present study is greater than that of the 2006 study at Cole Street Bridge. This suggests that the present study may be underestimating the conveyance of the bridge in comparison to the 2006 study.

6. LIMITATIONS

A detailed uncertainty assessment of the data, hydrological calibration and hydrodynamic model is contained in the Tasman Calibration Report (WMAwater, 2023)

The selection of limited duration-TP-ARF sets introduces some errors across the catchment as described in Section 5.1. This is appropriate for a regional method, however site-specific ARFs, critical durations and TP selection should be used for detailed design modelling at specific locations.

As noted in Section 5.2, it is recommended that the design mapping in the Orielton Lagoon, Pitt Water, and Blackman Bay areas are disregarded until such that time that updated topographic data is available and further modelling is undertaken. It was also noted that there is some uncertainty introduced by the direct rainfall application on the headwater catchments. While the method used is appropriate for broad scale mapping, a full design event assessment should be undertaken for any future focussed studies in this area.

The comparison to previous detailed flood studies at Richmond and Sorrell showed that the modelling of conveyance at bridges in this regional hydrodynamic model is limited by the simplified structure data and lack of information on channel bathymetry. This should be revised in any future detailed studies for these areas that build on this work.

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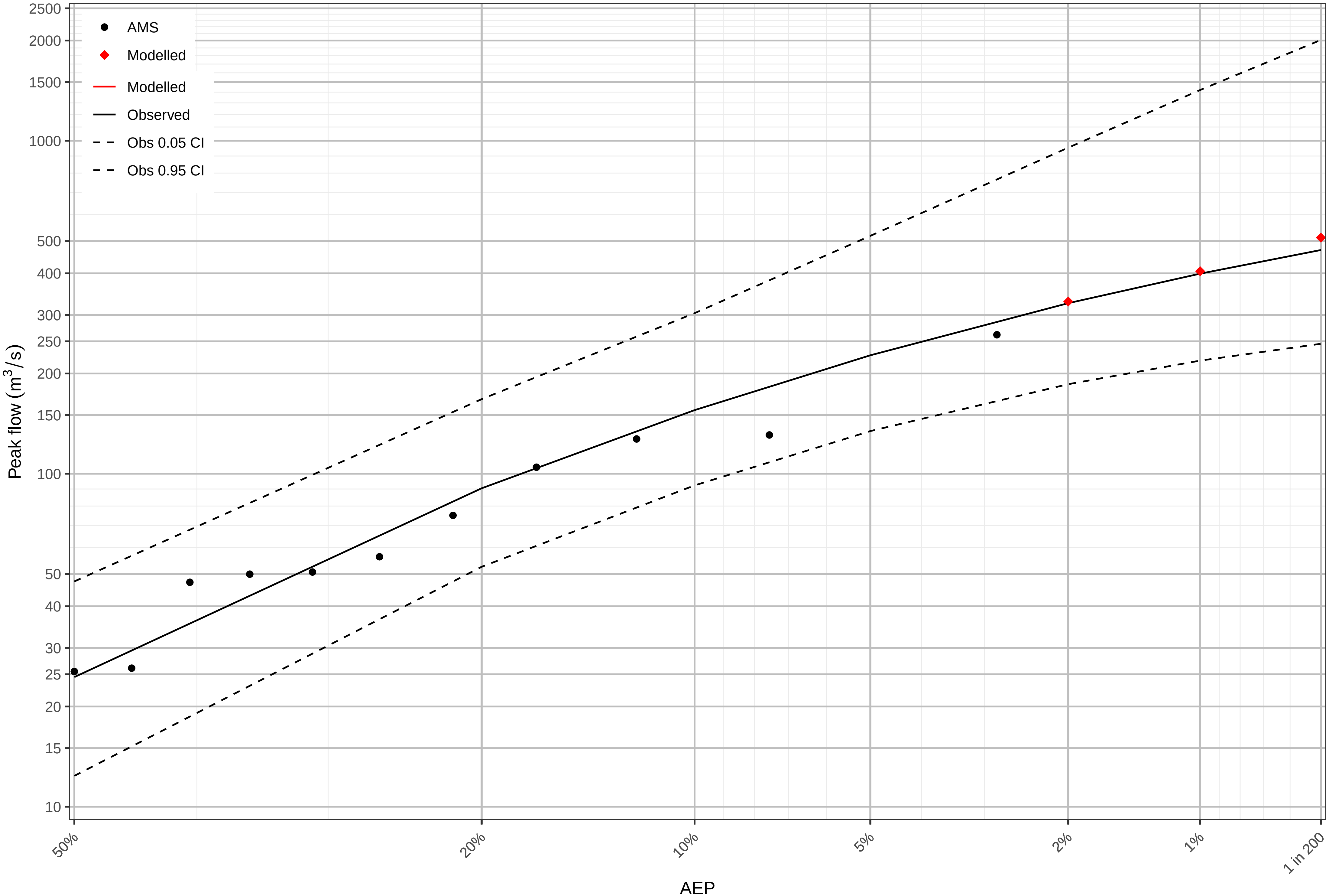
WMAwater (2023): Tasmanian Strategic Flood Map Tasman Catchment Model Calibration Report, March 2023. Report for State Emergency Service, Tasmania.

Figures

FIGURE 1

Coal River at Richmond



//skyfall.wmawater.com.au/jobs/Jobs/120038/Hydrology/Statewide/FFA_Calibration/Tasman/Coal_Rchm/Run_Creation/run_output/20220623_0904-calibv1_ATP/outputs/plots/FFA/FFA_Tsm135_lognormal_ModAsPts.png

//skyfall.wmawater.com.au/jobs/Jobs/120038/Hydrology/Statewide/FFA_Calibration/Tasman/Coal_Bden/Run_Creation/run_output/20220630_1437-PTP_Richcalib/outputs/plots/FFA/FFA_Tsm107_lognormal_ModAsPts.png

FIGURE 2

Coal River at Baden

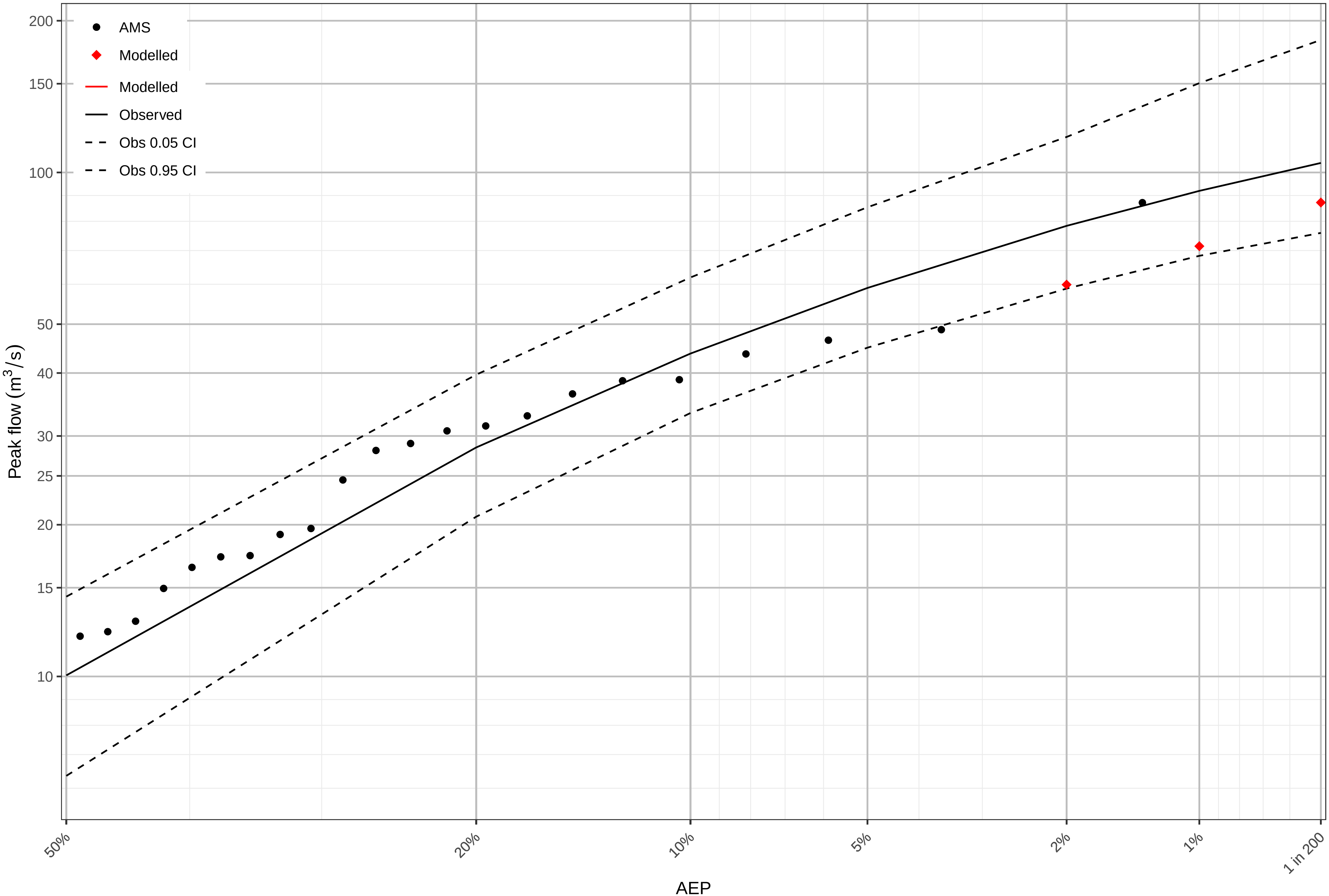
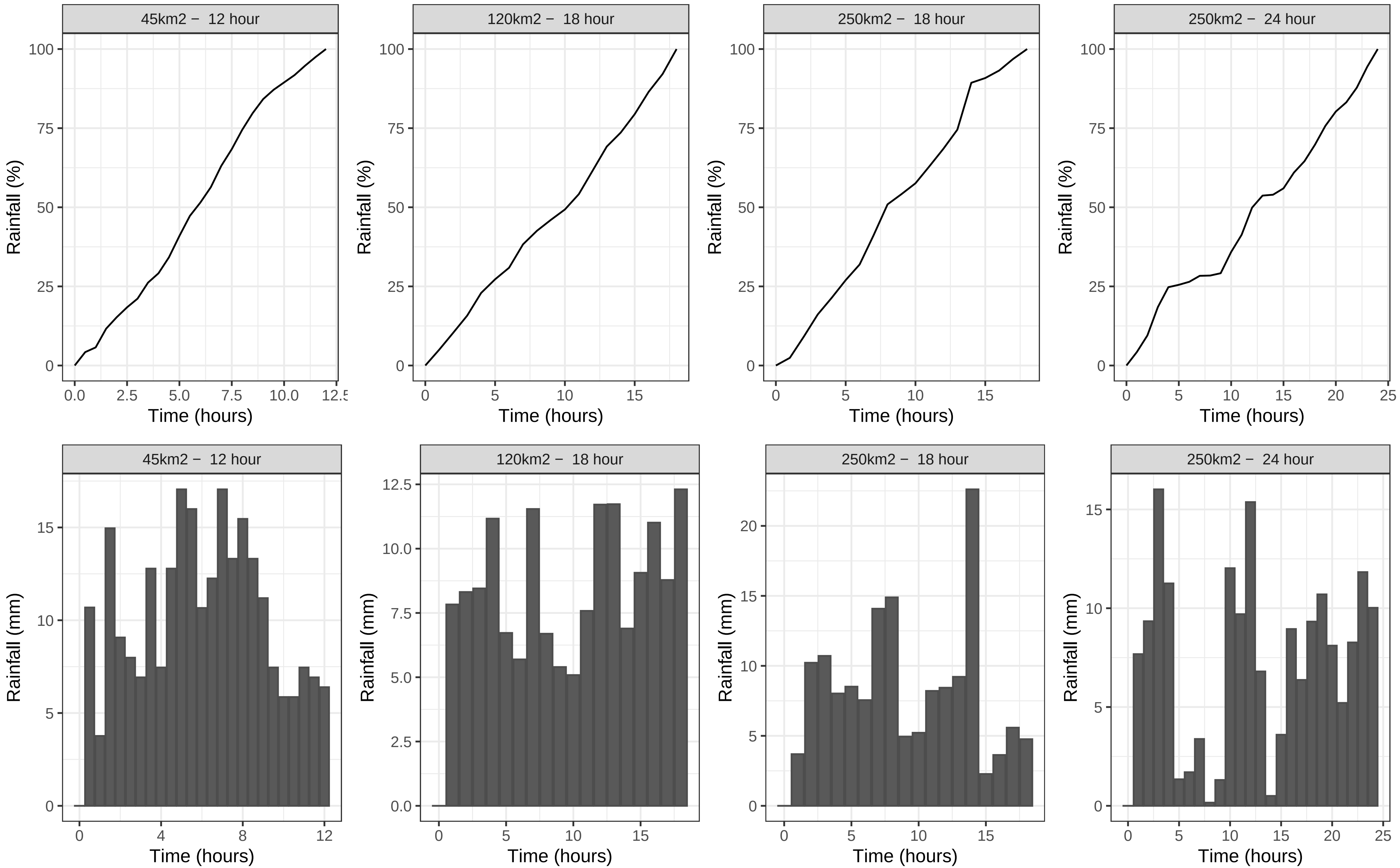


FIGURE 3

SELECTED DESIGN TEMPORAL PATTERNS ALL AEPS BY STORM DURATION AND ARF AREA



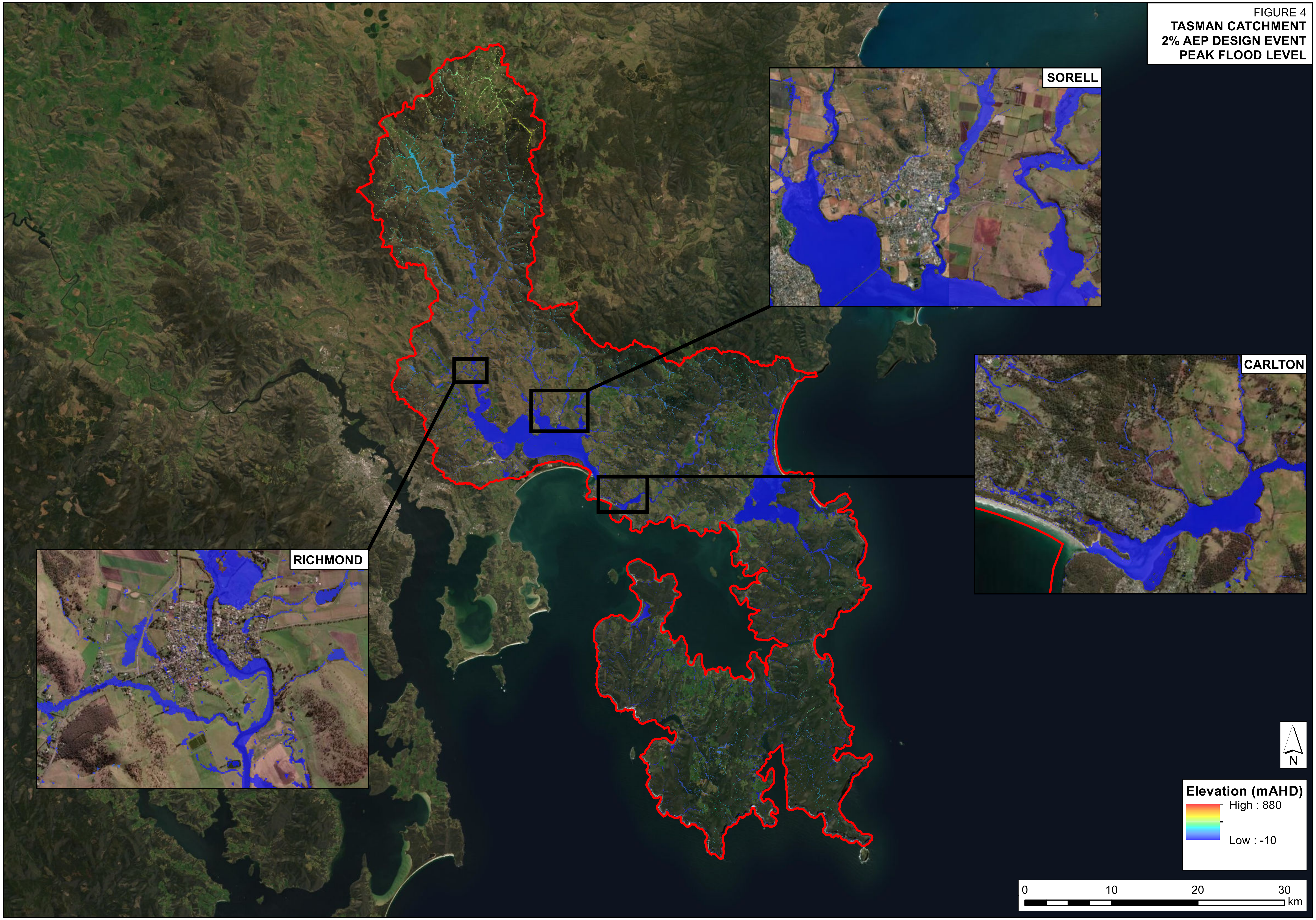


FIGURE 4
TASMAN CATCHMENT
2% AEP DESIGN EVENT
PEAK FLOOD LEVEL
SORELL
CARLTON
RICHMOND
N
Elevation (mAHD)
High : 880
Low : -10
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure04_2pAEP_PeakFloodLevel.mxd

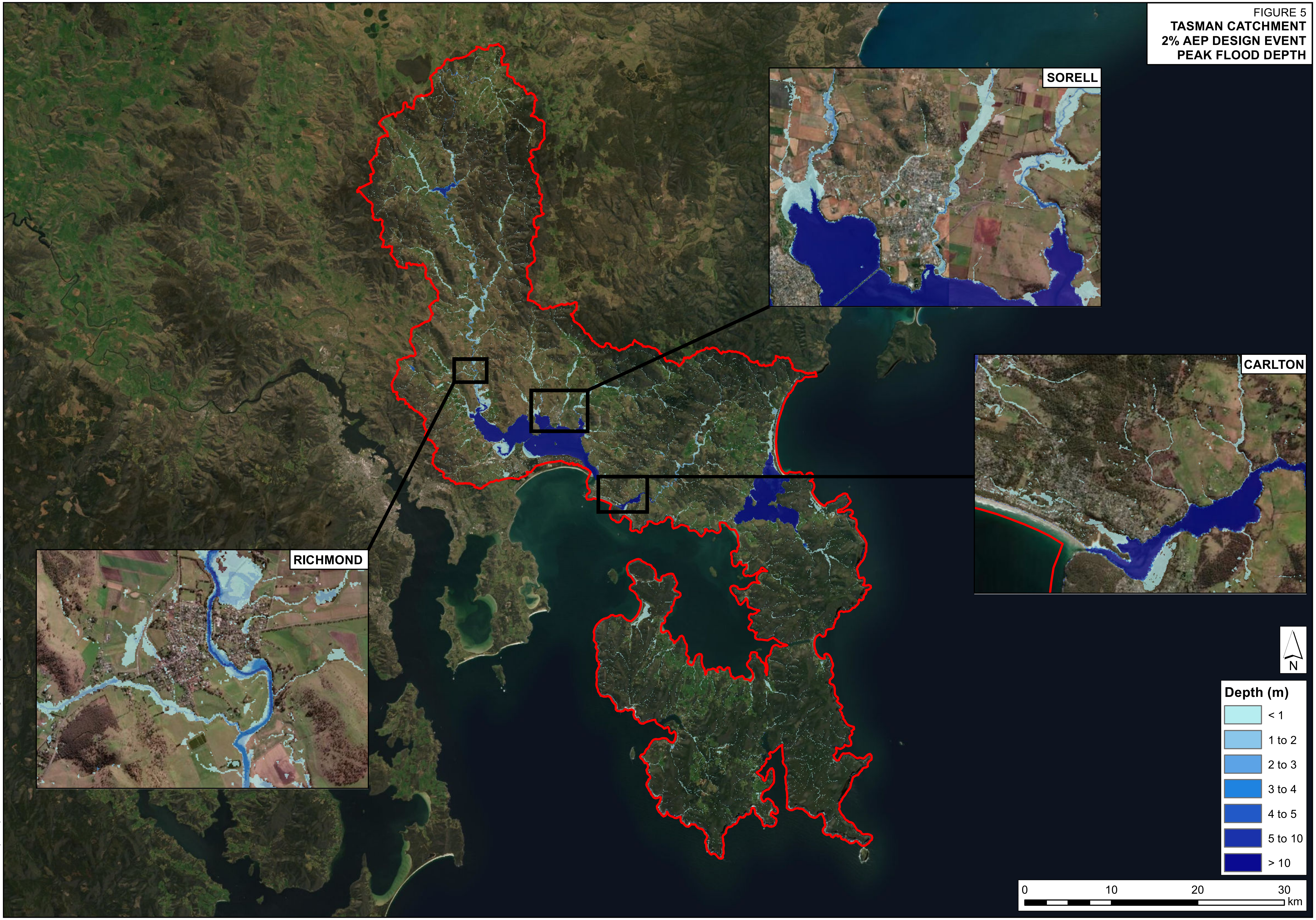
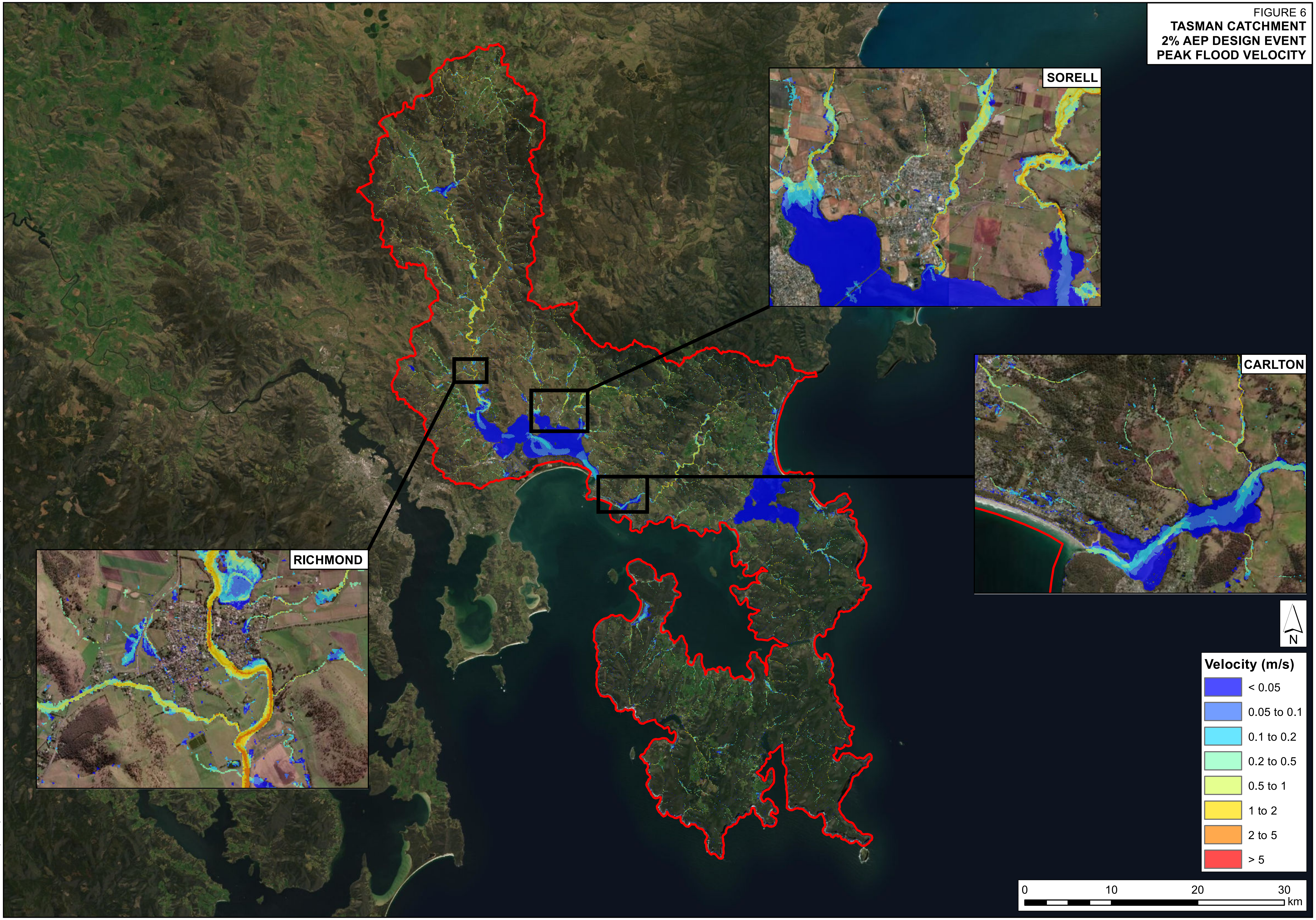
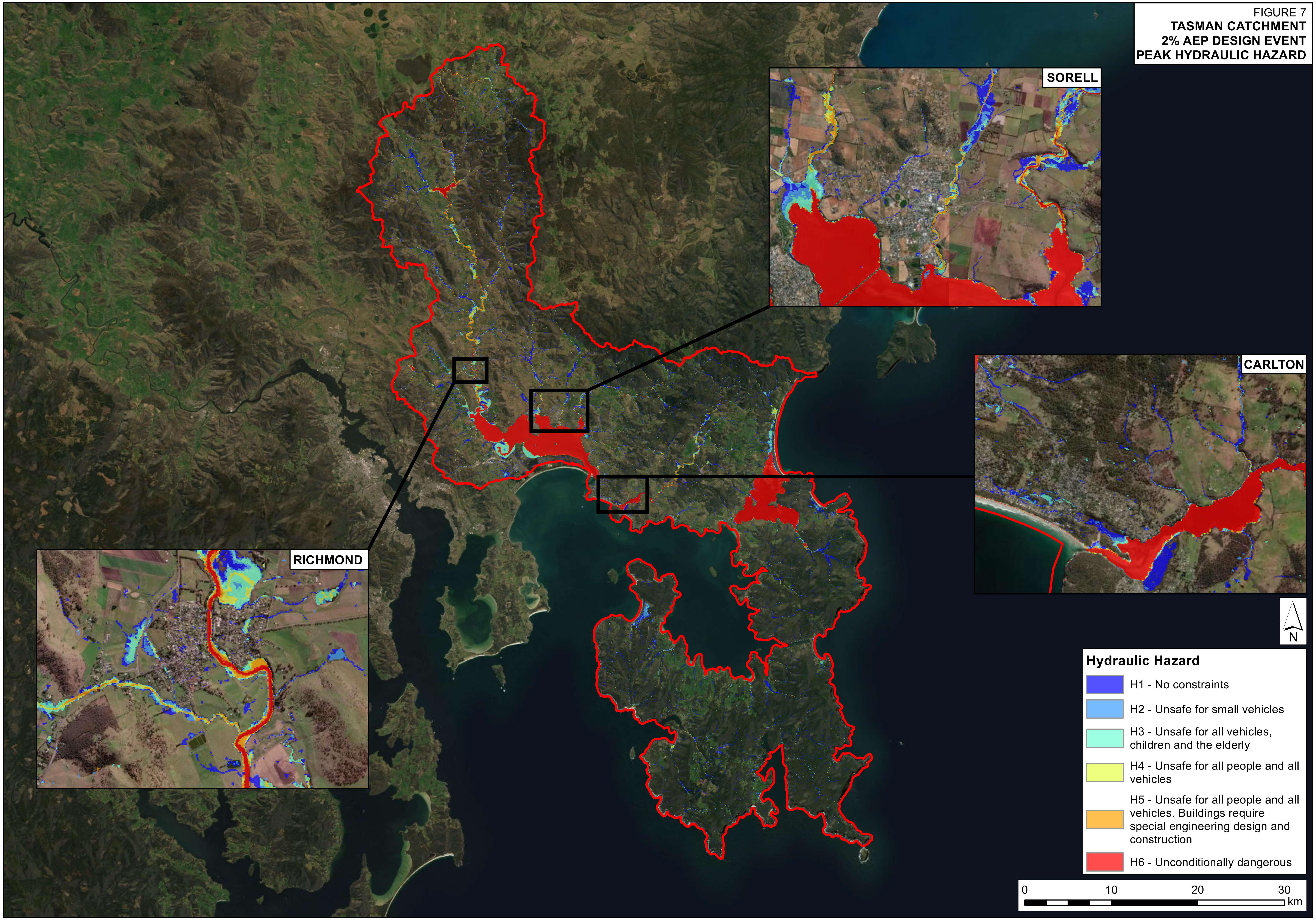


FIGURE 5
TASMAN CATCHMENT
2% AEP DESIGN EVENT
PEAK FLOOD DEPTH

J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure05_2pAEP_PeakFloodDepth.mxd



J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure06_2pAEP_PeakFloodVelocity.mxd



J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure07_2pAEP_PeakHydraulicHazard.mxd

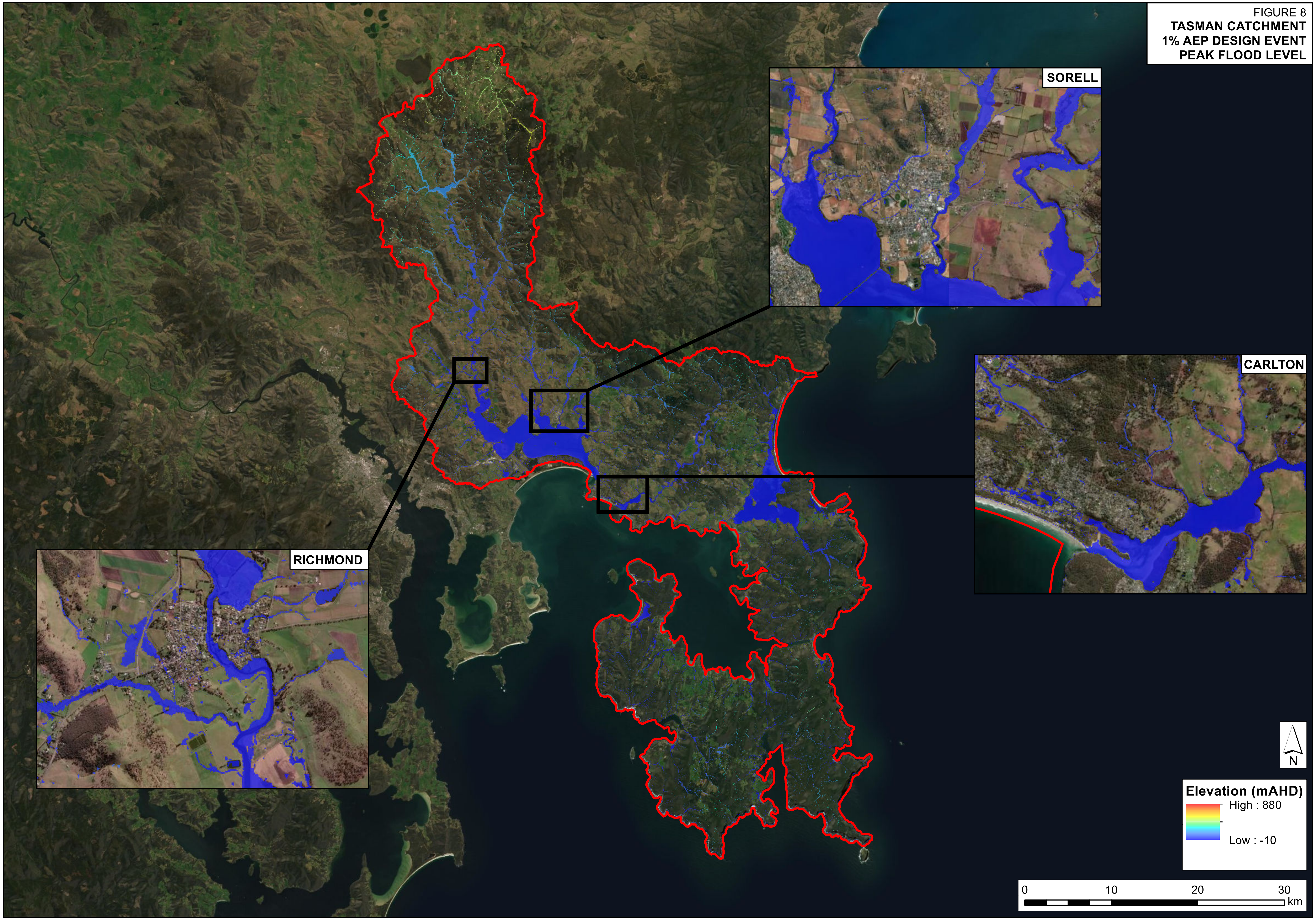
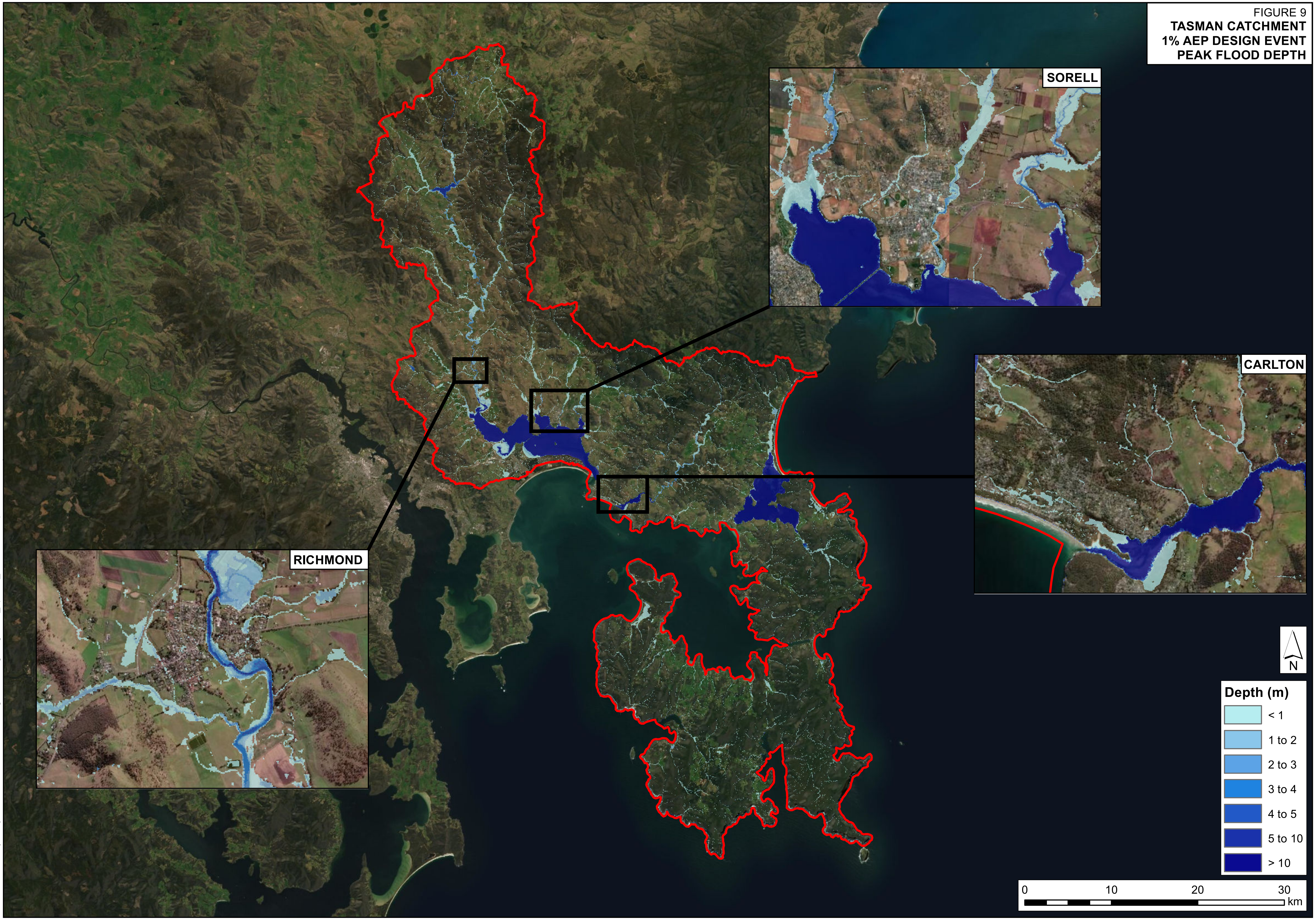


FIGURE 8
TASMAN CATCHMENT
1% AEP DESIGN EVENT
PEAK FLOOD LEVEL
SORELL
CARLTON
RICHMOND
N
Elevation (mAHD)
High : 880
Low : -10
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure08_1pAEP_PeakFloodLevel.mxd



J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure09_1pAEP_PeakFloodDepth.mxd

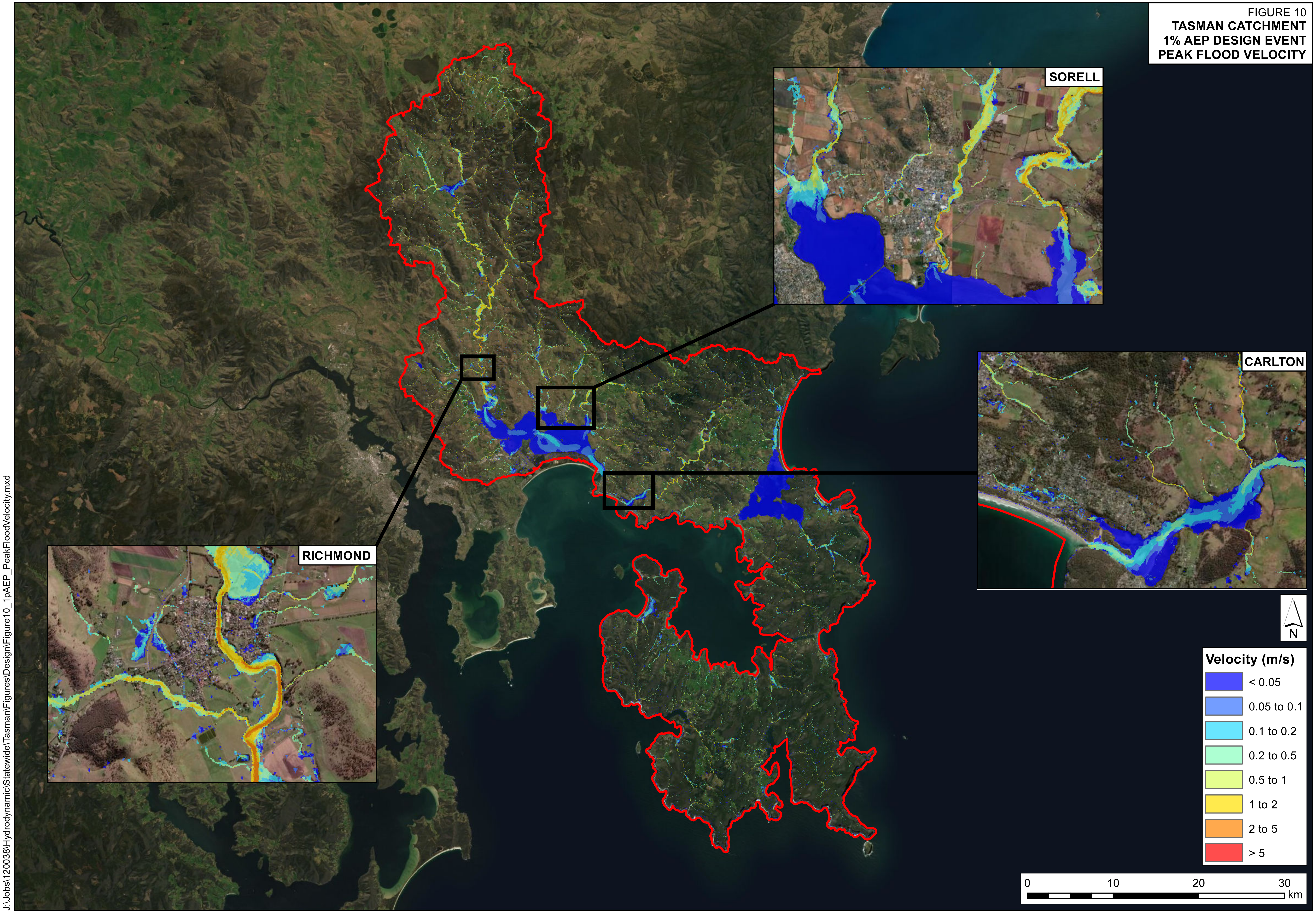


FIGURE 10
TASMAN CATCHMENT
1% AEP DESIGN EVENT
PEAK FLOOD VELOCITY
SORELL
CARLTON
RICHMOND
N
Velocity (m/s)
< 0.05
0.05 to 0.1
0.1 to 0.2
0.2 to 0.5
0.5 to 1
1 to 2
2 to 5
> 5
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure10_1pAEP_PeakFloodVelocity.mxd

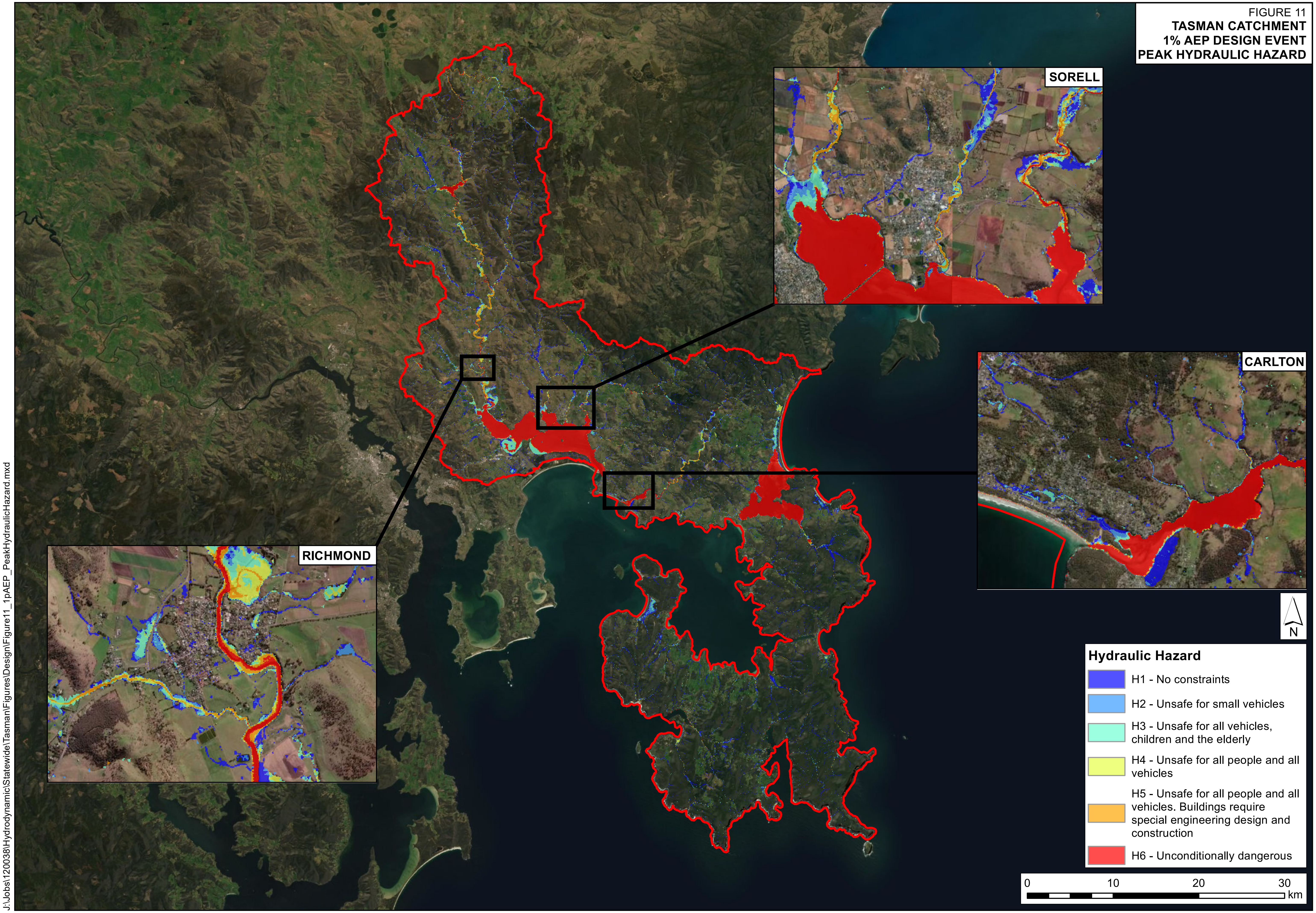


FIGURE 11
TASMAN CATCHMENT
1% AEP DESIGN EVENT
PEAK HYDRAULIC HAZARD
SORELL
CARLTON
RICHMOND
N
Hydraulic Hazard
H1 - No constraints
H2 - Unsafe for small vehicles
H3 - Unsafe for all vehicles, children and the elderly
H4 - Unsafe for all people and all vehicles
H5 - Unsafe for all people and all vehicles. Buildings require special engineering design and construction
H6 - Unconditionally dangerous
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure11_1pAEP_PeakHydraulicHazard.mxd

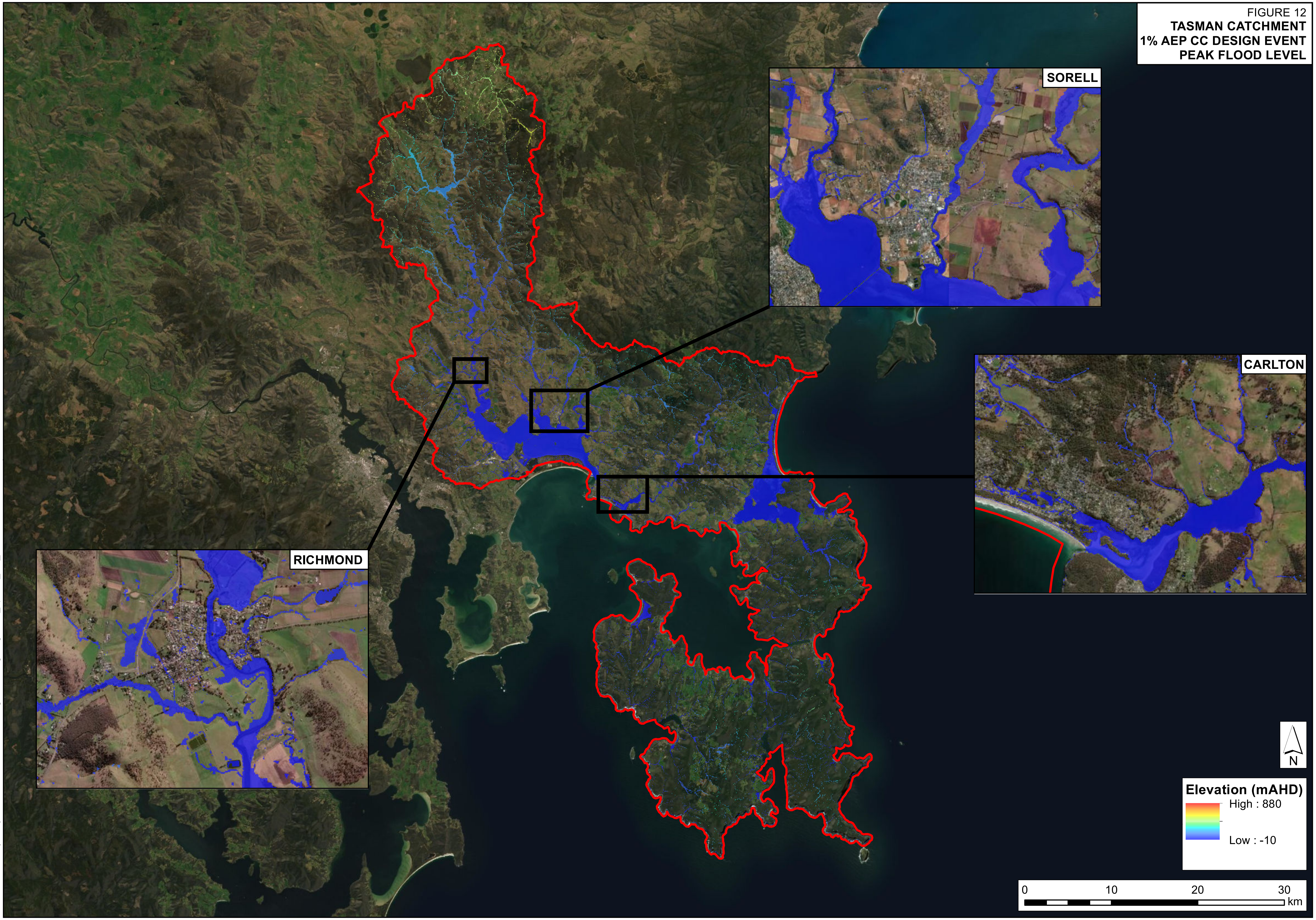


FIGURE 12
TASMAN CATCHMENT
1% AEP CC DESIGN EVENT
PEAK FLOOD LEVEL

J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure12_1pAEP_CC_PeakFloodLevel.mxd

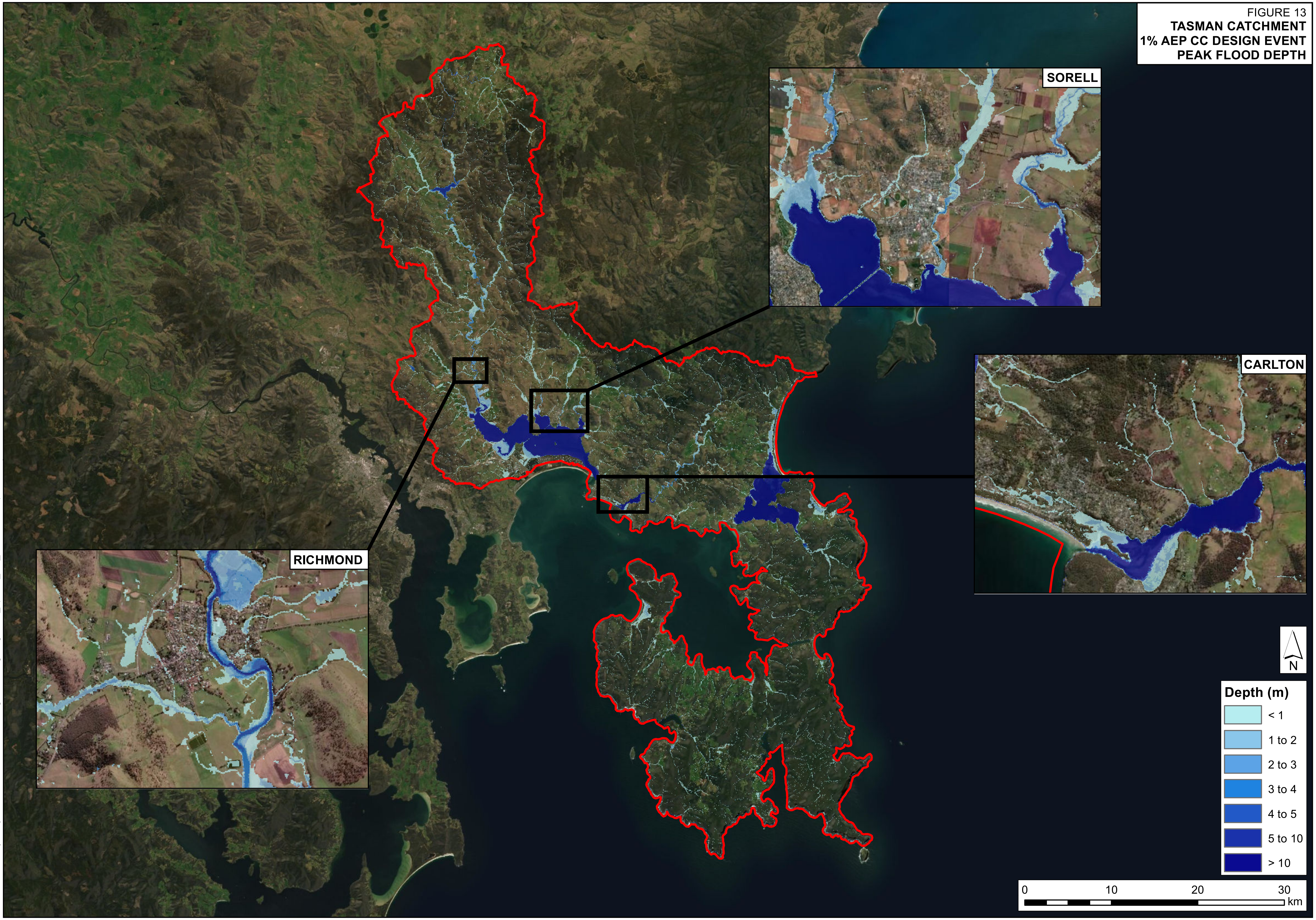


FIGURE 13
TASMAN CATCHMENT
1% AEP CC DESIGN EVENT
PEAK FLOOD DEPTH
SORELL
CARLTON
RICHMOND
N
Depth (m)
< 1
1 to 2
2 to 3
3 to 4
4 to 5
5 to 10
> 10
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure13_1pAEP_CC_PeakFloodDepth.mxd

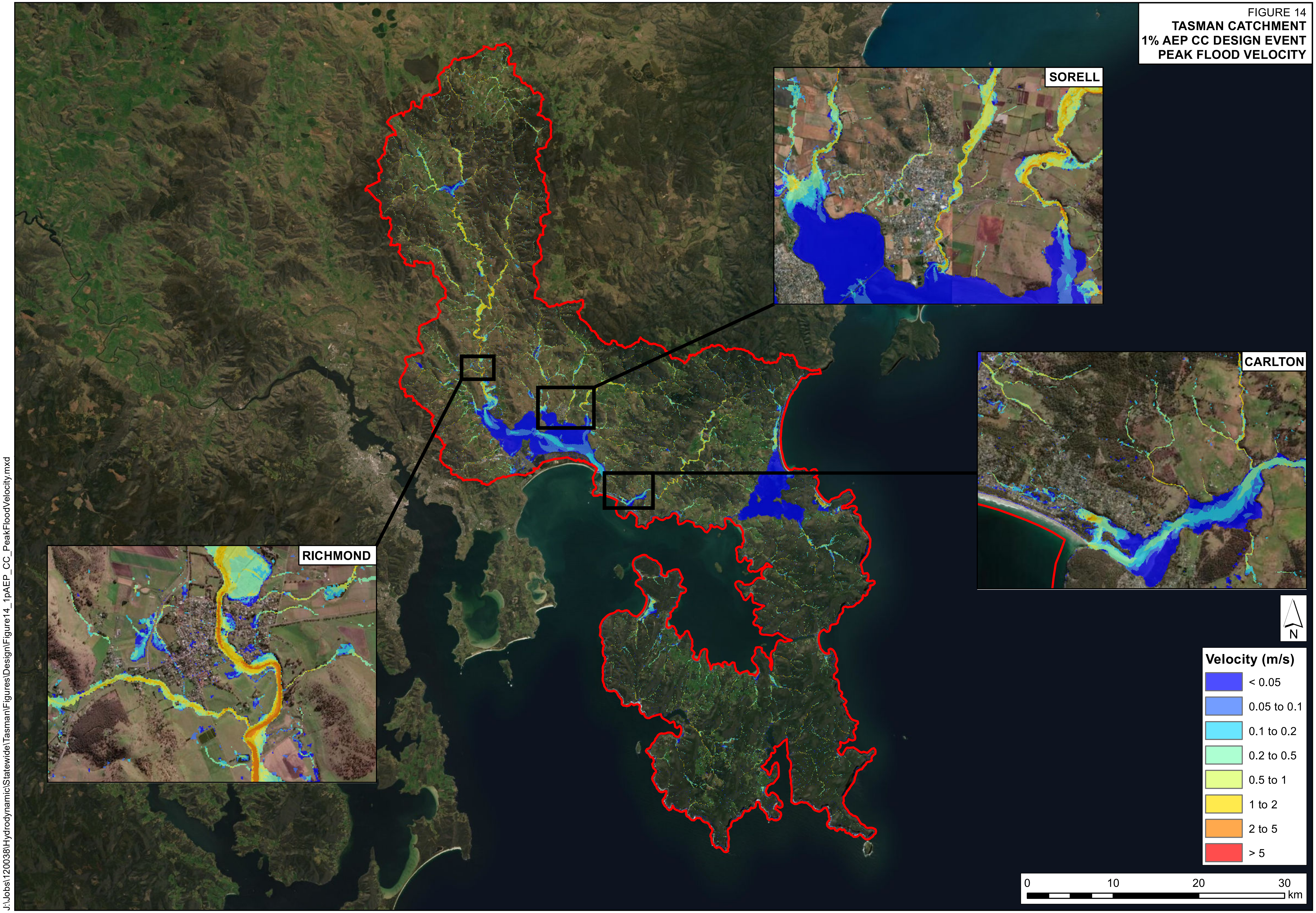
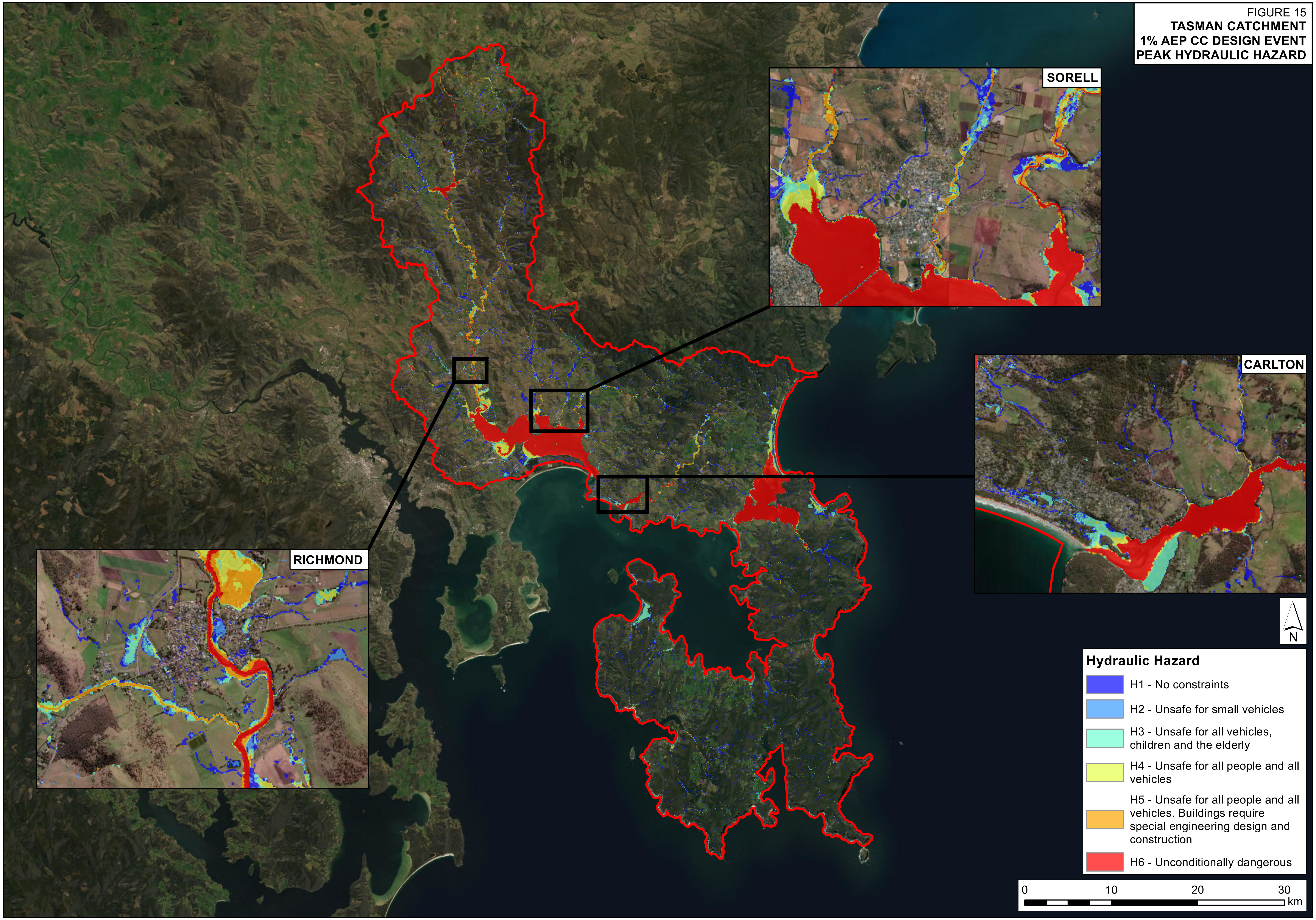
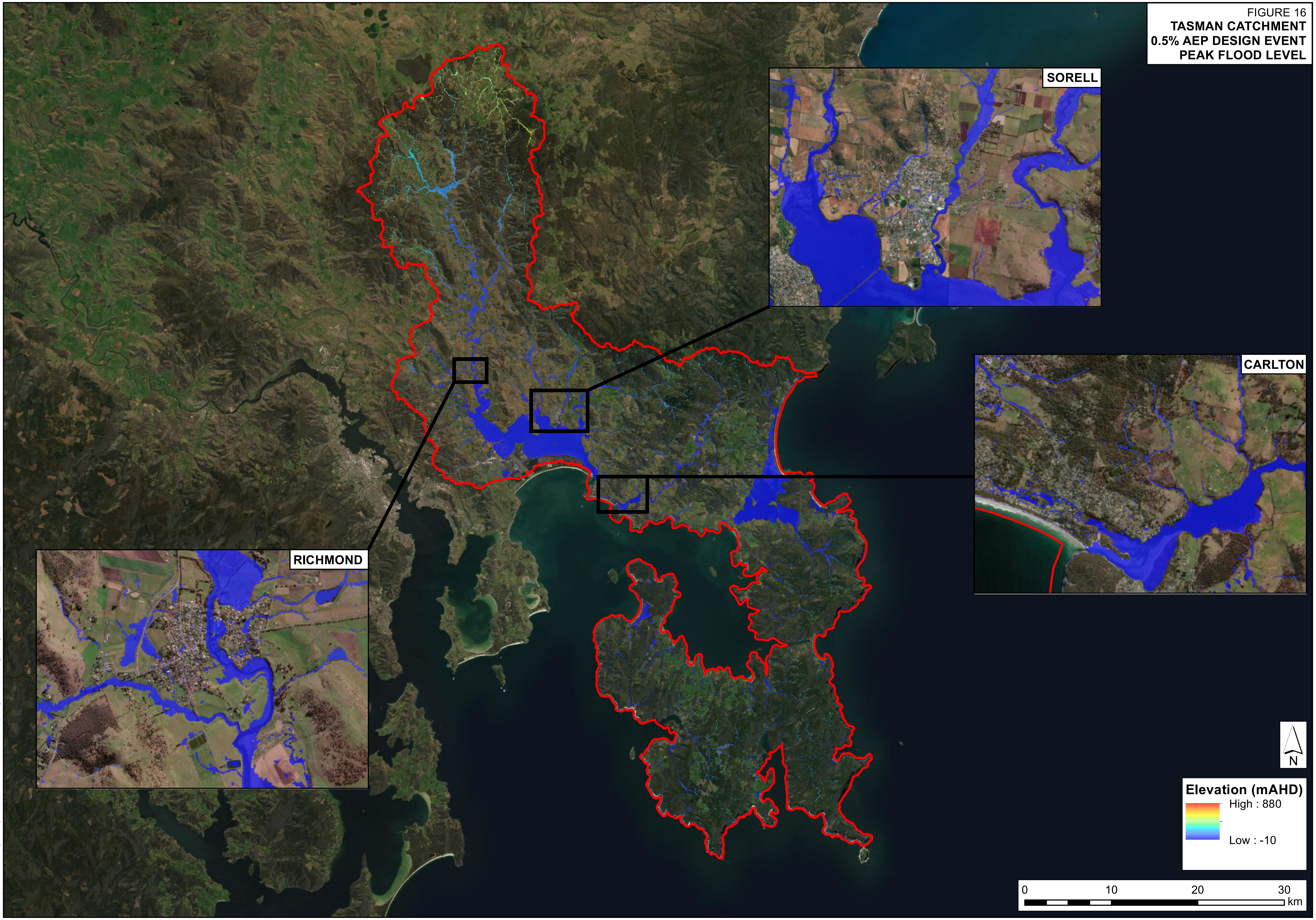


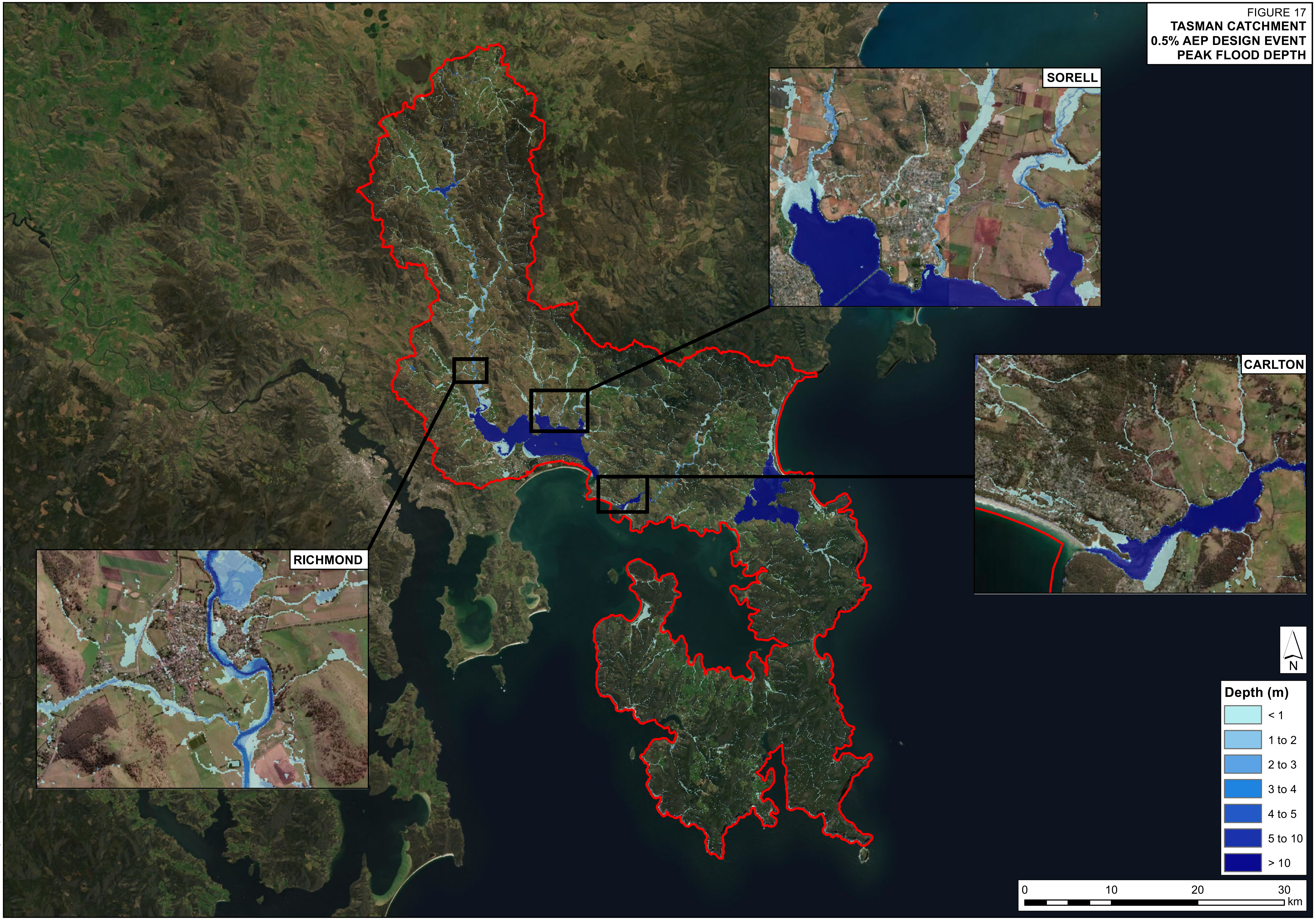
FIGURE 14
TASMAN CATCHMENT
1% AEP CC DESIGN EVENT
PEAK FLOOD VELOCITY
SORELL
CARLTON
RICHMOND
N
Velocity (m/s)
< 0.05
0.05 to 0.1
0.1 to 0.2
0.2 to 0.5
0.5 to 1
1 to 2
2 to 5
> 5
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure14_1pAEP_CC_PeakFloodVelocity.mxd



J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure15_1pAEP_CC_PeakHydraulicHazard.mxd



J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure16_0.5pAEP_PeakFloodLevel.mxd



J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure17_0.5pAEP_PeakFloodDepth.mxd

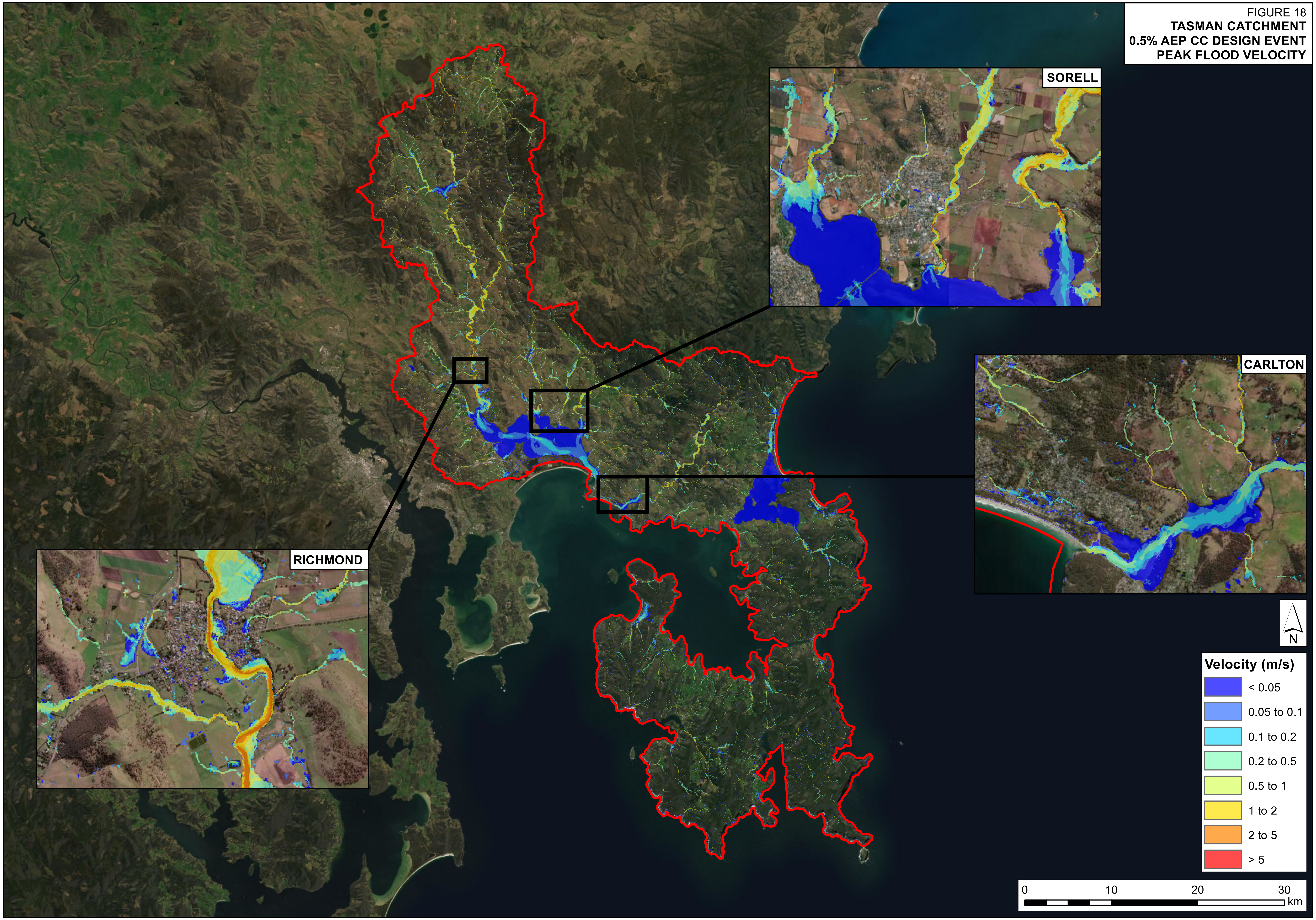


FIGURE 18
TASMAN CATCHMENT
0.5% AEP CC DESIGN EVENT
PEAK FLOOD VELOCITY
SORELL
CARLTON
RICHMOND
N
Velocity (m/s)
< 0.05
0.05 to 0.1
0.1 to 0.2
0.2 to 0.5
0.5 to 1
1 to 2
2 to 5
> 5
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure18_0.5pAEP_PeakFloodVelocity.mxd

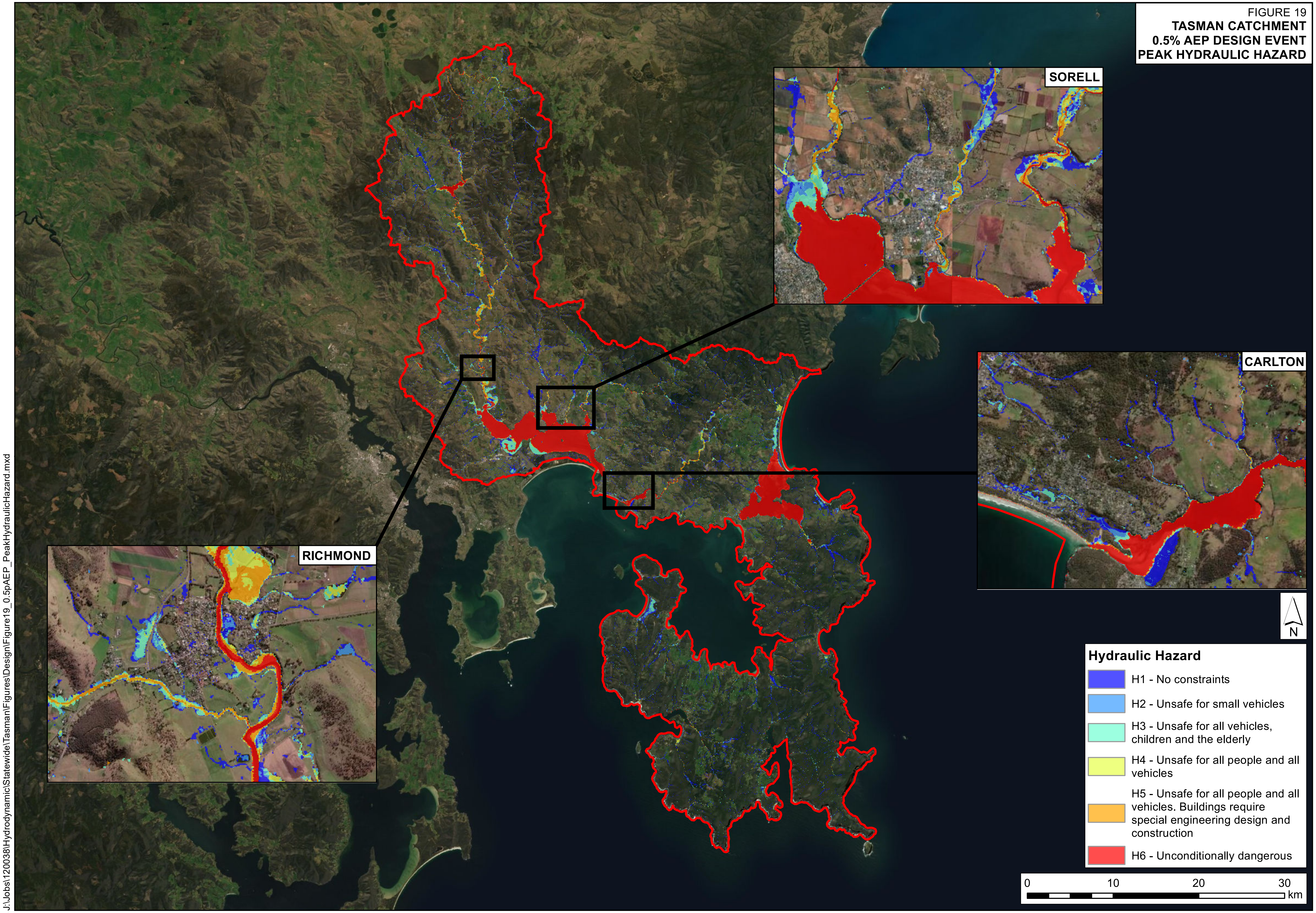


FIGURE 19
TASMAN CATCHMENT
0.5% AEP DESIGN EVENT
PEAK HYDRAULIC HAZARD
SORELL
CARLTON
RICHMOND
N
Hydraulic Hazard
H1 - No constraints
H2 - Unsafe for small vehicles
H3 - Unsafe for all vehicles, children and the elderly
H4 - Unsafe for all people and all vehicles
H5 - Unsafe for all people and all vehicles. Buildings require special engineering design and construction
H6 - Unconditionally dangerous
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure19_0.5pAEP_PeakHydraulicHazard.mxd

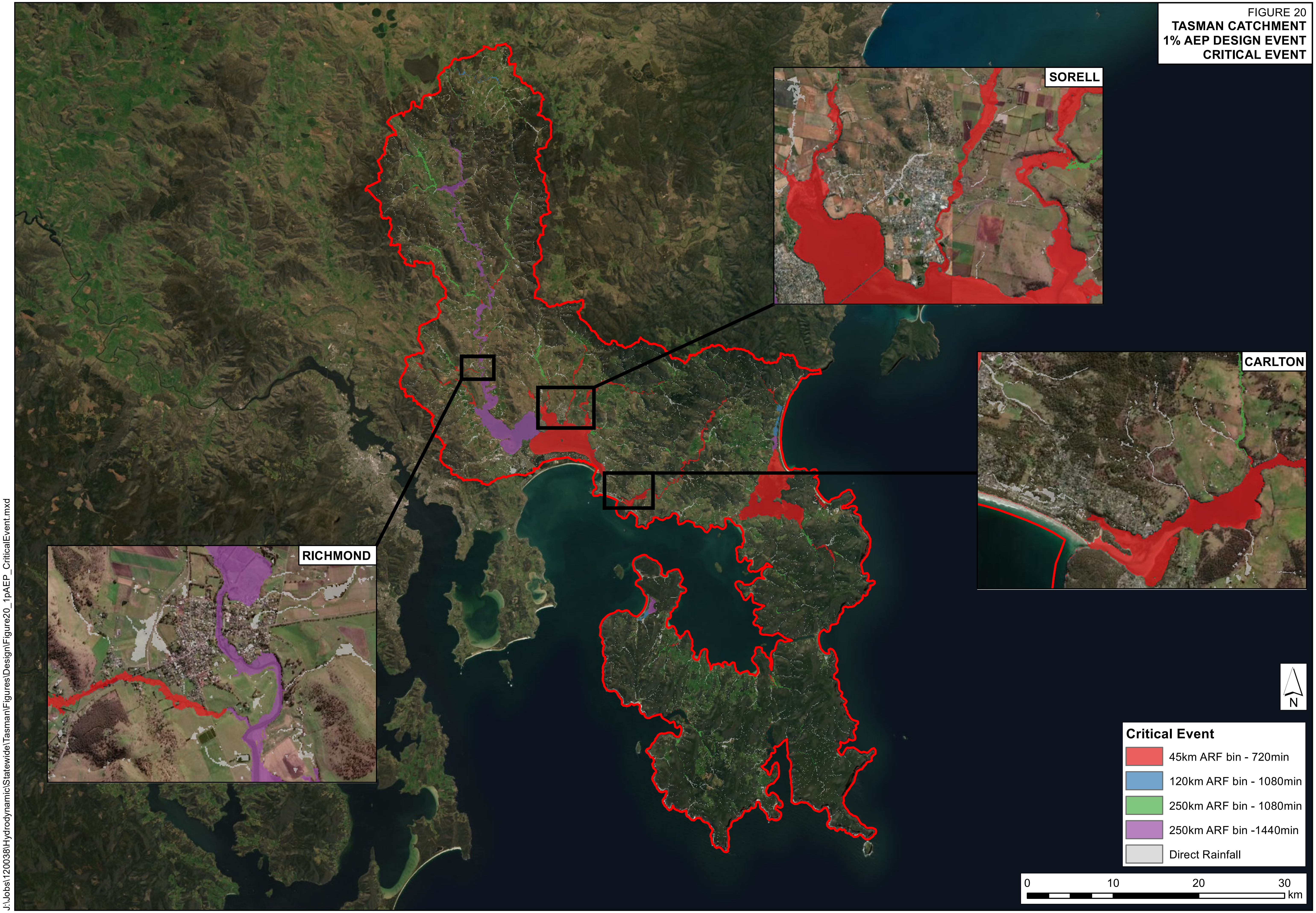


FIGURE 20
TASMAN CATCHMENT
1% AEP DESIGN EVENT
CRITICAL EVENT
SORELL
CARLTON
RICHMOND
Critical Event
45km ARF bin - 720min
120km ARF bin - 1080min
250km ARF bin - 1080min
250km ARF bin -1440min
Direct Rainfall
N
0
10
20
30
km
J:\Jobs\120038\Hydrodynamic\Statewide\Tasman\Figures\Design\Figure20_1pAEP_CriticalEvent.mxd

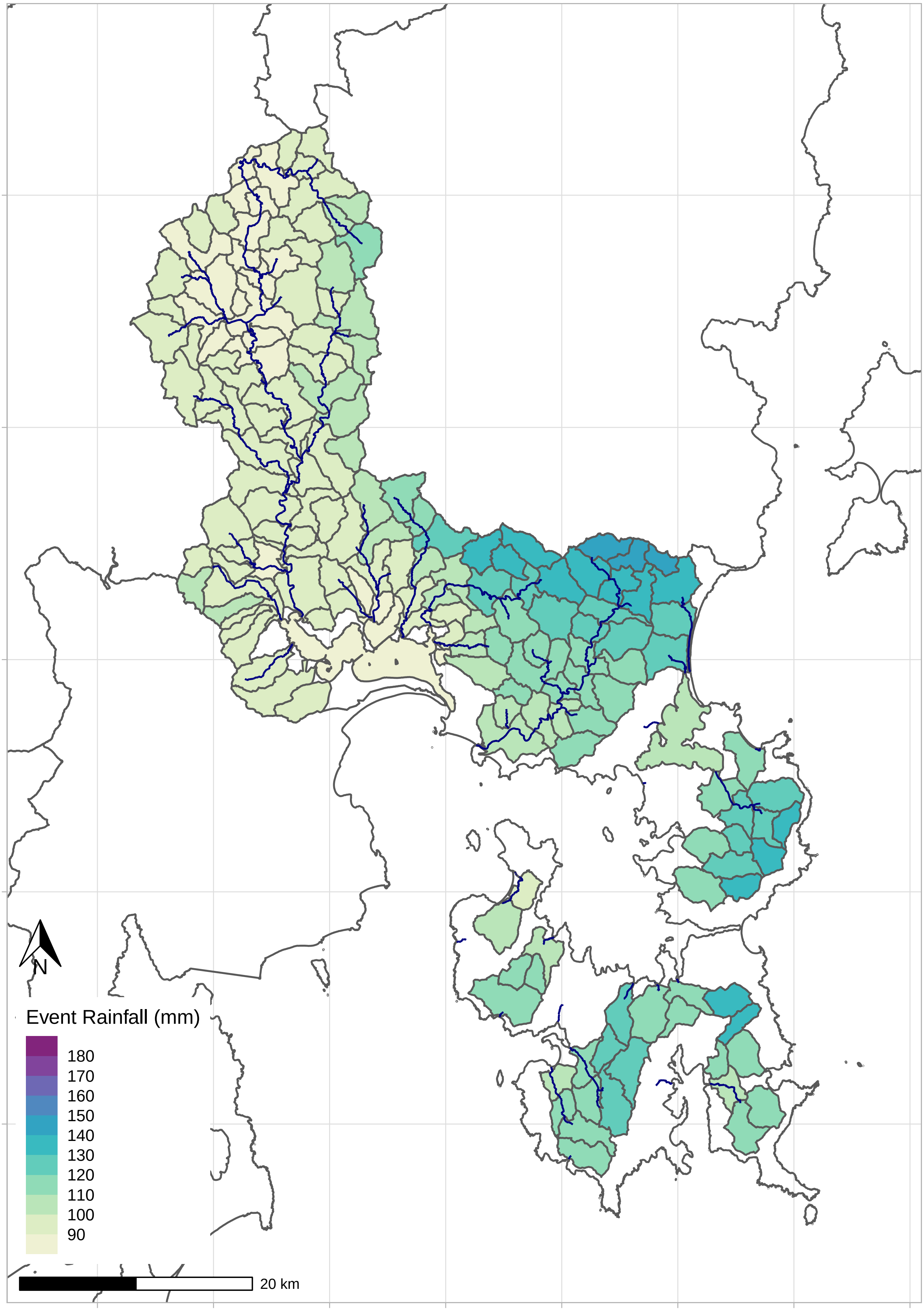
Appendix A

APPENDIX A. DESIGN EVENT DATA

FIGURE A1

DESIGN RAINFALL DEPTHS

1080MIN 2%AEP



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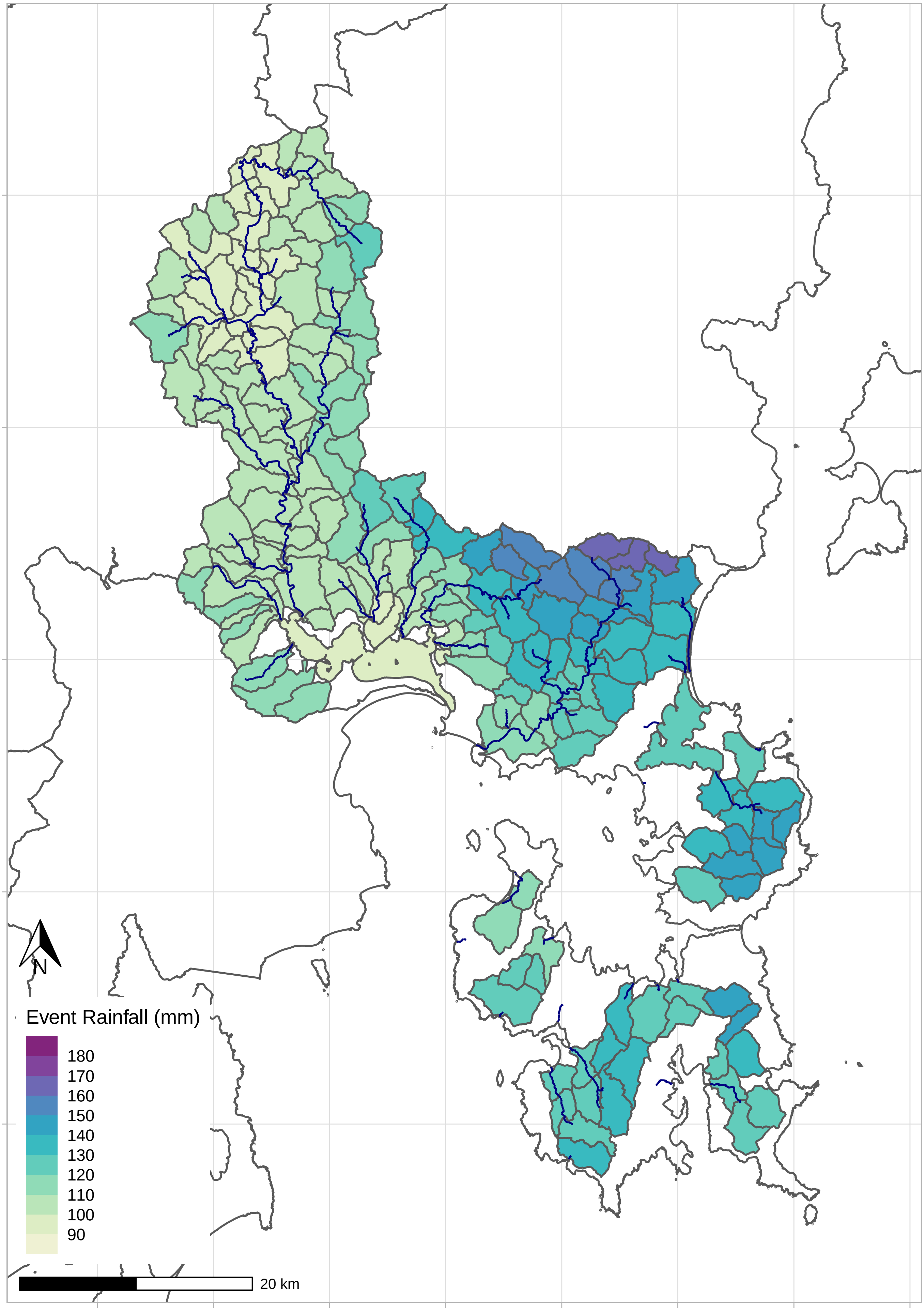
J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/Tasmanv1/Report_Figures/FigureA1_1080_2pAEP_same_scale.pdf

FIGURE A2

DESIGN RAINFALL DEPTHS

1080MIN 1%AEP

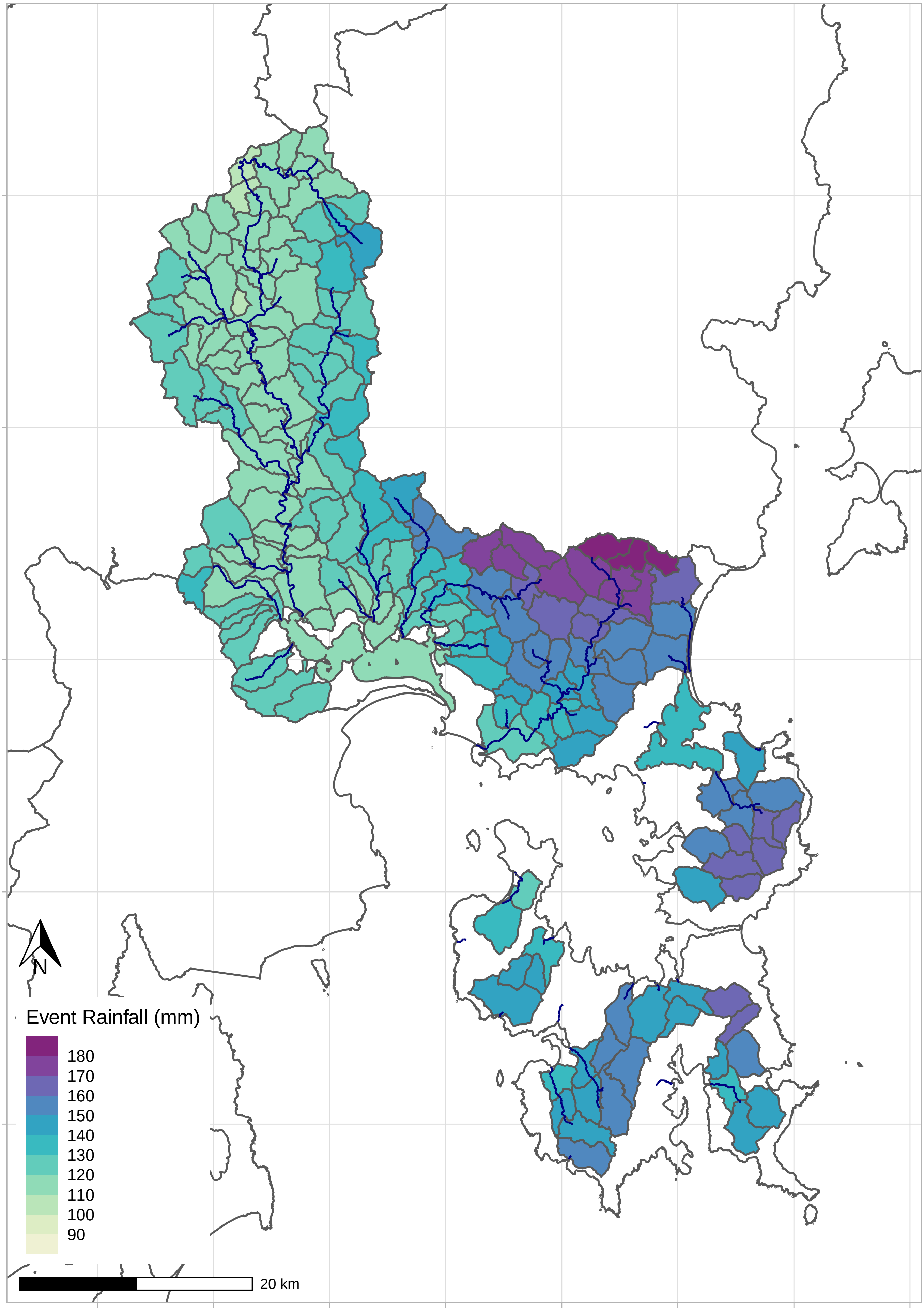


created by J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/7_Tasman_Design_Report_Plots.R
J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/Tasmanv1/Report_Figures/FigureA2_1080_1pAEP_same_scale.pdf

FIGURE A3

DESIGN RAINFALL DEPTHS

1080MIN 0.5%AEP



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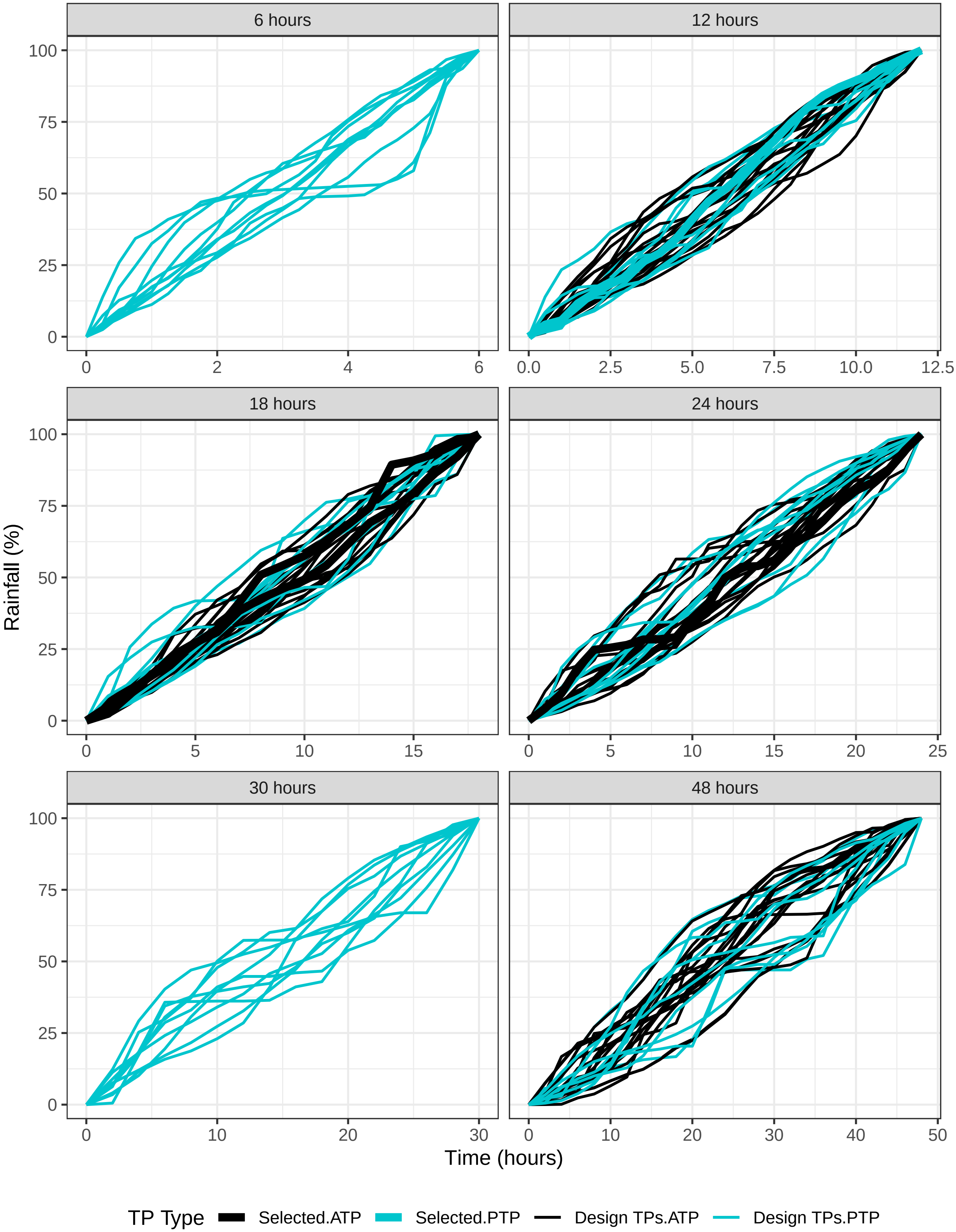
J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/Tasmanv1/Report_Figures/FigureA3_1080_1in200AEP_same_scale.pdf

FIGURE A4
DESIGN AREAL TEMPORAL PATTERNS
DURATIONS FROM 2 TO 48 HOURS



created by J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/7_Tasman_Design_Report_Plots.R

Appendix B

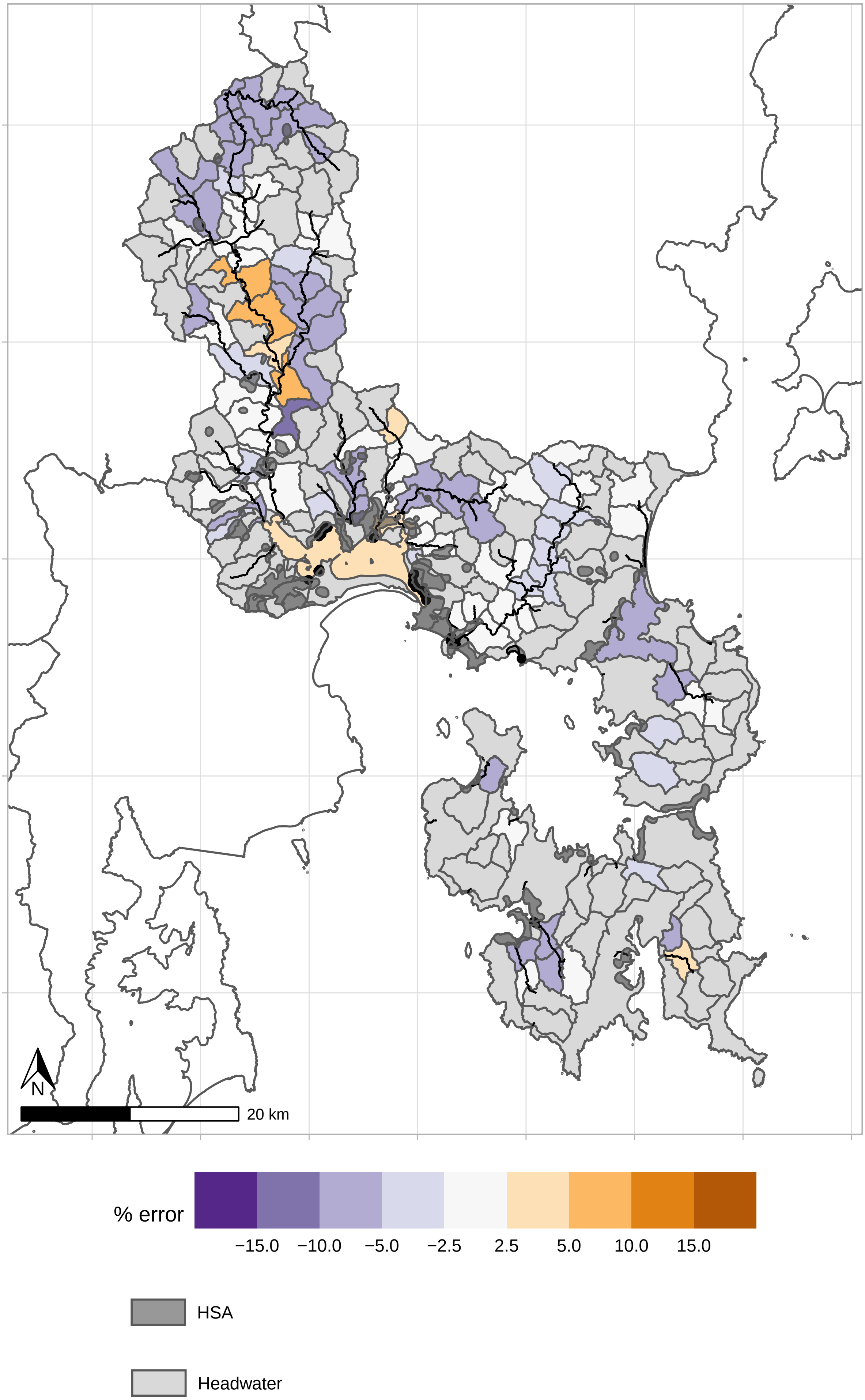
APPENDIX B. DESIGN PEAK ERRORS

Figure B1

Tasman Catchment

Percentage error in peak flows using selected runs

2%AEP



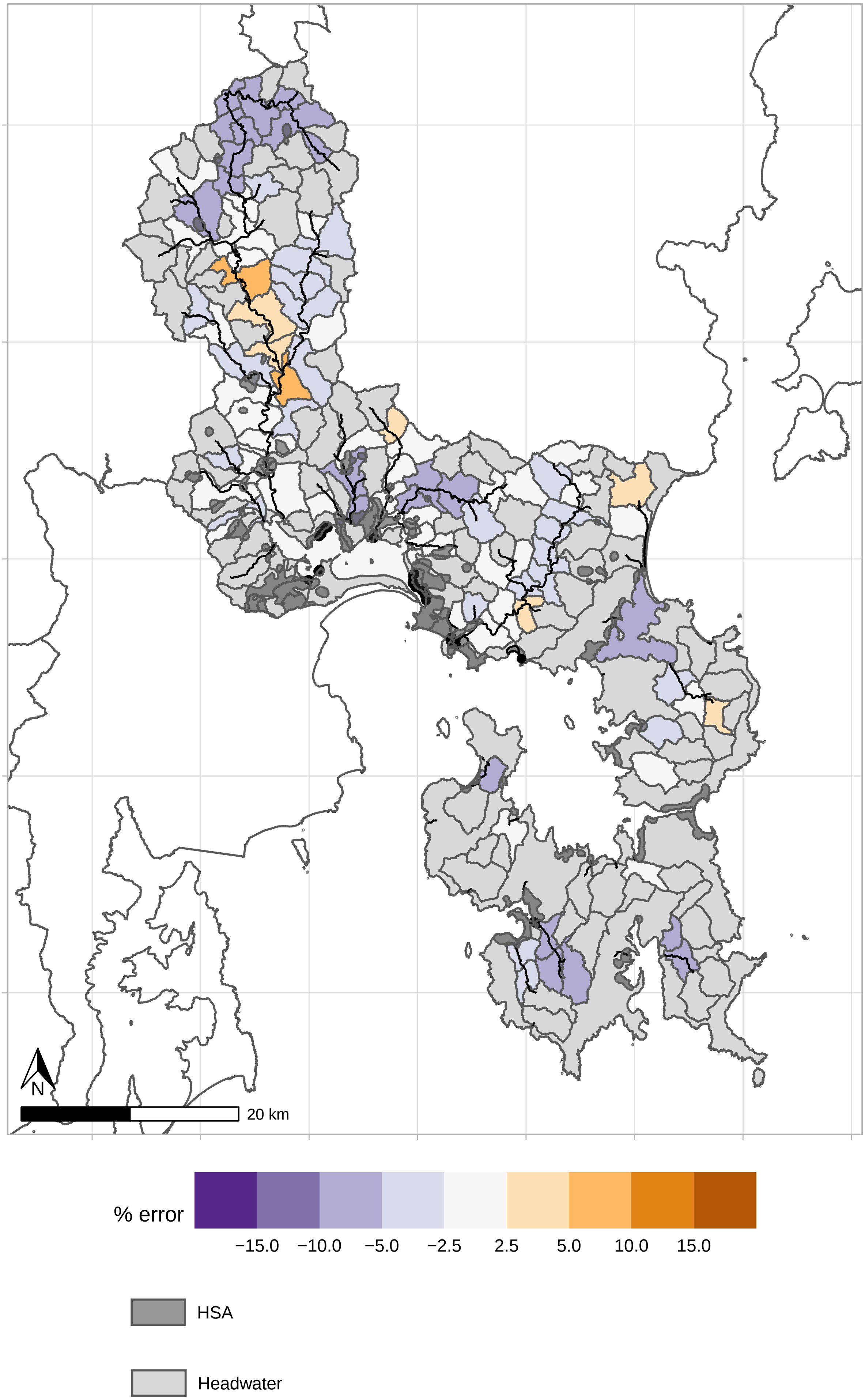
created by J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/7_Tasman_Design_Report_Plots.R

Figure B2

Tasman Catchment

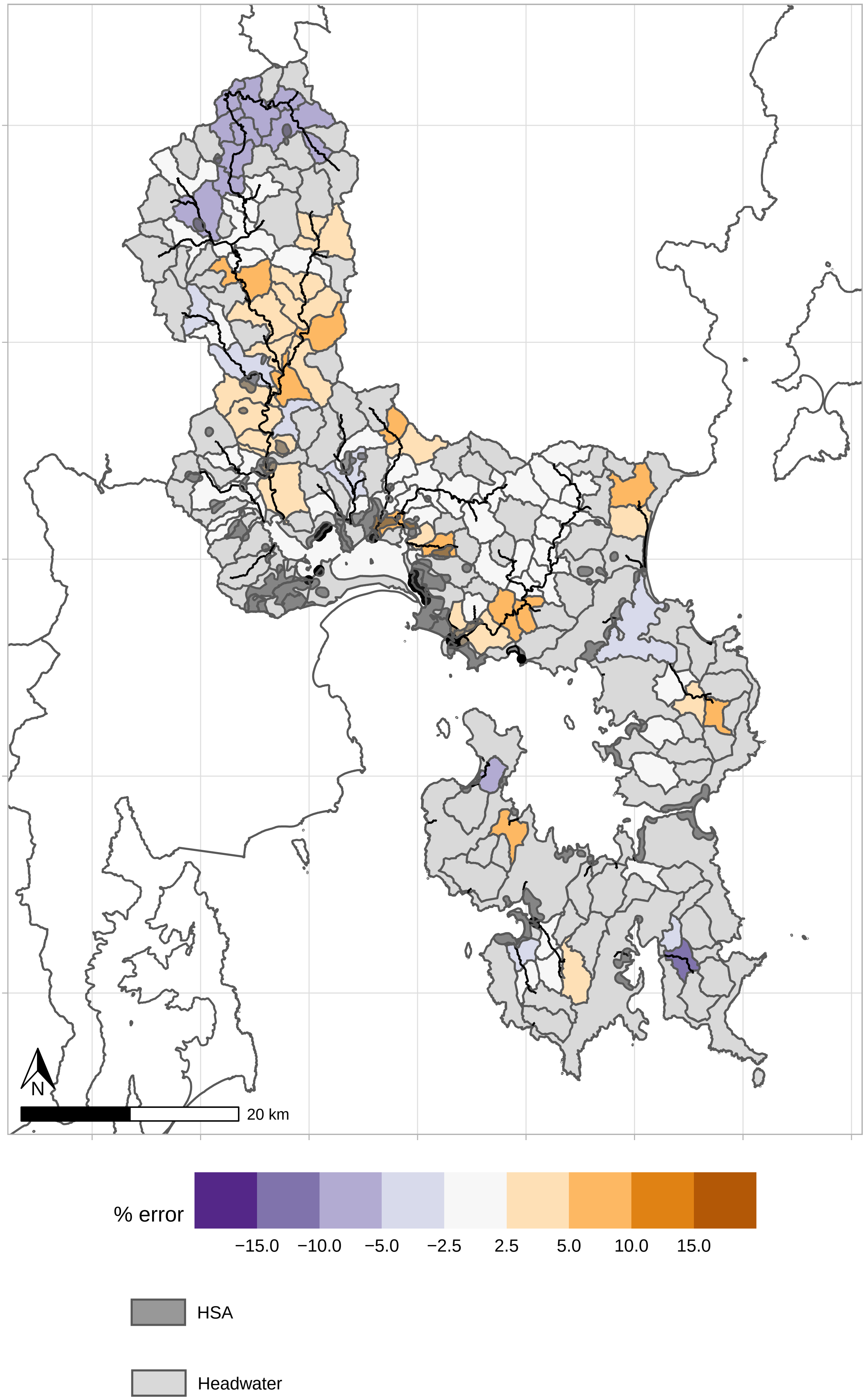
Percentage error in peak flows using selected runs

1%AEP



created by J:/Jobs/120038/Hydrology/Statewide/Design_Events/Tasman/7_Tasman_Design_Report_Plots.R

Figure B3
Tasman Catchment
Percentage error in peak flows using selected runs
0.5%AEP



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