Newcastle Grammar School

Flood Impact Assessment

R.T2030.001.03

March 2022

Revised Draft Report

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Synopsis

Flood Impact Assessment report documenting the existing flood risk profile at the Newcastle Grammar School site and the assessment of flood impacts both to and resulting from the proposed development. Appropriate measures for the management of the identified flood risks are also discussed.

Revision History

Revision	Description	Date
01	Draft	28/05/2021
02	Final	7/10/2021
03	Revised Draft (updated to incorporate Stage 2)	25/03/2022

Disclaimer

Executive Summary

Torrent Consulting was engaged by Hatch to undertake a Flood Impact Assessment to assist in the approval process for the proposed development at the Newcastle Grammar School – Park Campus, Cnr. Union Street and Parkway Avenue, Cooks Hill, NSW. This assessment has included development of a TUFLOW model for Cottage Creek and local catchment runoff and has simulated design flood conditions in accordance with the ARR 2019 guidelines, specifically the ensemble method for design flood hydrology.

Flood hazard mapping has been produced showing that the Site is of a low risk to property but a high risk to life, which requires appropriate management. The proposed Stage 1 building should conform to the requirements of the CoN DCP, with an appropriate FPL being 3.55 m AHD. The ground floor level is below the FPL but is principally designated as an open play area. However, there are proposed WC facilities, a lift and PE storage area on the ground floor that require appropriate flood-proofing to manage the risk to property and minimise damages in the event of a flood.

The bottom level of the Stage 2 building is proposed to be a basement car park at a level of around 1.1 m AHD. Control 3 of the Newcastle DCP Section 4.01.03 requires all potential water entry points to basement parking levels should be set at or above the PMF level, which is 4.8 m AHD. The only exception to this is for vehicle entry points, which can be set at the FPL. As determined for the Stage 1 building, an appropriate FPL for the Site is 3.55 m AHD. The vehicular access ramp to the proposed Stage 2 basement parking level will first need to rise by around 0.55 m above the existing ground surface level of 3.0 m AHD prior to descending.

To satisfy the management of risk to life requirements the proposed development needs certification by a Structural Engineer to withstand the hydraulic forces of the PMF conditions, i.e. a flood depth of 2.2 m, flood velocity of 1.8 m/s and velocity-depth product of 1.4 for the Stage 1 building and an external flood depth of 2.2 m, internal flood depth of 3.1 m, flood velocity of 2.0 m/s and velocity-depth product of 1.5 for the Stage 2 building.

Because the upper levels of the Stage 1 building (including the first floor) are all located above the PMF level, it inherently provides suitable flood-free refuge for the occupants. In practical terms, the same also applies to the upper floors of the Stage 2 building (with the first floor level being only 0.1 m below the PMF). It is understood that a Flood Emergency Response Plan is already in place for the Site. The Plan should be updated to include the upper floors of the Stage 1 and Stage 2 buildings as a flood refuge. Block A, Block B and the proposed Stage 1 and Stage 2 buildings should each provide their own individual flood refuge within the upper floors.

An assessment of potential future climate change has found the combined effects of increased rainfall intensity and sea level rise projections gives an expected increase in the 1% AEP flood level at the Site of around 0.15 m to 0.20 m for the 2050 planning horizon and 0.60 m to 0.75 m for the 2090-2100 planning horizon. These potential climate change impacts have implications for freeboard that the current FPL provides in the future. However, occupiable rooms of the proposed Stage 1 building are on the upper floors, which are significantly elevated above the FPL and potential future climate change impacts. As such, the proposed development would remain compatible with the flood hazard of the land, it is just the frequency of flood inundation that might be expected to increase.

The proposed development configuration has been modelled to assess potential flood impacts to neighbouring properties. The only location where the proposed development results in a tangible off-site impact is around the western corner of the Site within Union Street, with peak velocities of around 2 m/s. Whilst velocities of around 2 m/s are relatively high, they do not pose a significant risk to the existing building or road infrastructure or change the overall flood hazard or risk profile within the roadway and so are not considered to be adverse impacts. No specific measures are required to manage this impact.

Potential impacts to the social and economic costs to the community as a consequence of flooding from the proposed development are expected to be beneficial, as it is removing buildings that have an existing risk to property and offer no suitable flood refuge to manage the risk to life. These buildings are to be replaced by a new building that minimises the risk to property and provides substantial flood refuge opportunity.

Contents

List of Figures

List of Tables

1 Introduction

1.1 Project Background

Torrent Consulting was engaged by Hatch to undertake a Flood Impact Assessment to assist in the approval process for the proposed development at the Newcastle Grammar School – Park Campus, Cnr. Union Street and Parkway Avenue, Cooks Hill, NSW (the Site), as presented in Figure 1-1.

A previous Flood Impact Assessment was completed in October 2021 and only considered Stage 1 of the proposed development. The assessment has now been expanded to include the Stage 2 works.

The proposed Stage 1 development at the Site includes the construction of a new building and the removal of an existing three buildings and shed. It is being assessed for approval as a State Significant Development (SSD), for which the Secretary's Environmental Assessment Requirements (SEARs) include flooding.

Flood assessment requirements from NSW DPIE include:

- *Identify any flood risk on-site in consultation with Council and having regard to the most recent flood studies for the development area and the potential effects of climate change, sea level rise and an increase in rainfall intensity.*
- *Provide details regarding the preservation of the local flood storage area, of which the site is a part.*
- *Provide a detailed flood study and report prepared by a suitably qualified engineer.*
- *Provide details regarding the high flood risk affecting the site and the onsite flood refuge provisions.*
- *Provide an assessment of any detrimental increases in the potential flood affectation of other properties, assets and infrastructure.*
- *Assess the impacts of the development, including any changes to flood risk on-site or off-site, and detail design solutions to mitigate flood risk where required.*
- *Provide an updated flood emergency response plan reflecting the new building arrangement proposed as part of this development.*
- *Note: Council has advised that the June 2007 flood level was higher than the predicted 1% AEP flood level on the site.*

In addition to the above the City of Newcastle Council (CoN) stated that:

It is advised that flooding is a significant issue for this development. The site is part of the local flood storage area which will need to be preserved and has a high flood risk requiring onsite flood refuge. It is understood the school already have a flood emergency response plan which will need to be updated to reflect the new building arrangement. In this regard, the June 2007 flood level was higher than the predicted 1% AEP flood level on the site. The required flood assessment for the development is to be prepared by a suitably qualified engineer and address the relevant requirements of the Newcastle Development Control Plan 2012 and associated technical manual.

Documents detailing the proposed Stage 1 development were provided to Torrent Consulting, including:

- 4293.210505.Site Plan.dwg

- 4293.210505.Ground Floor.dwg
- 4293.210505.First Floor.dwg
- 4293.210505.Second Floor.dwg
- 4293.210505.Rooftop Play.dwg

Documents detailing the proposed Stage 2 development were provided to Torrent Consulting, including:

- 4293.2.13.Site Plan Proposed- Stage 02_RevA.dwg

1.2 Study Catchment

This Site is located within the Cottage Creek catchment, which totals some 7.6 km^2, around half of which is upstream of the Site, as presented in Figure 1-2.

The upper catchment is elevated at around 100 m AHD and includes the suburb of Merewether Heights and Scenic Drive, Merewether, draining to a low depression between Patrick Street and Glebe Road just south-east of The Junction. This catchment is drained via the concrete-lined channel past the Site and then through National Park.

The catchment area upstream of National Park is around 4.2 km^2. Around 0.5 km^2 of this catchment drains the Strzelecki ridge through Nesca Park to a local depression on Bruce Street, some 150 m or so to the east of and upslope of the Site. This local catchment is serviced by a 1200 mm diameter trunk drain discharging to the open channel upstream of the Site.

The low-lying catchment area around Hamilton South is also drained via a concrete-line channel into Cottage Creek at National Park, contributing an additional 1.6 km^2. A further 1 km^2 catchment draining Hamilton and Hamilton East drains to Cottage Creek via a smaller concrete-lined channel through the eastern side of National Park, under Smith Street.

Th total catchment area draining to Cottage Creek upstream of Parry Street, at the outlet from National Park is around 7.3 km^2. An additional 0.4 km^2 of local drainage discharges to Cottage Creek between National Park and the Throsby Basin of Newcastle Harbour.

The Site is subject to potential flooding from Cottage Creek and overland flow from the Nesca Park catchment. Flood modelling is required to establish the existing flood risk at the Site and to assess any potential flood impacts relating to the proposed development.

1.3 Previous Studies

The flood planning requirements in the Cottage Creek catchment are underpinned by the Newcastle City-wide Floodplain Risk Management Study and Plan (BMT WBM, 2012), which is based principally on modelling undertaken for the Throsby, Cottage and CBD Flood Study (BMT WBM, 2008).

More recent modelling of the Cottage Creek catchment has been completed, with CoN undertaking modelling assessments for Cottage Creek floodplain risk management options (Catchment Simulation Solutions, 2017) and the Honeysuckle Redevelopment Area Flood Study (BMT, 2018) was undertaken for the Hunter Central Coast Development Corporation (HCCDC).

Title:

Study Locality

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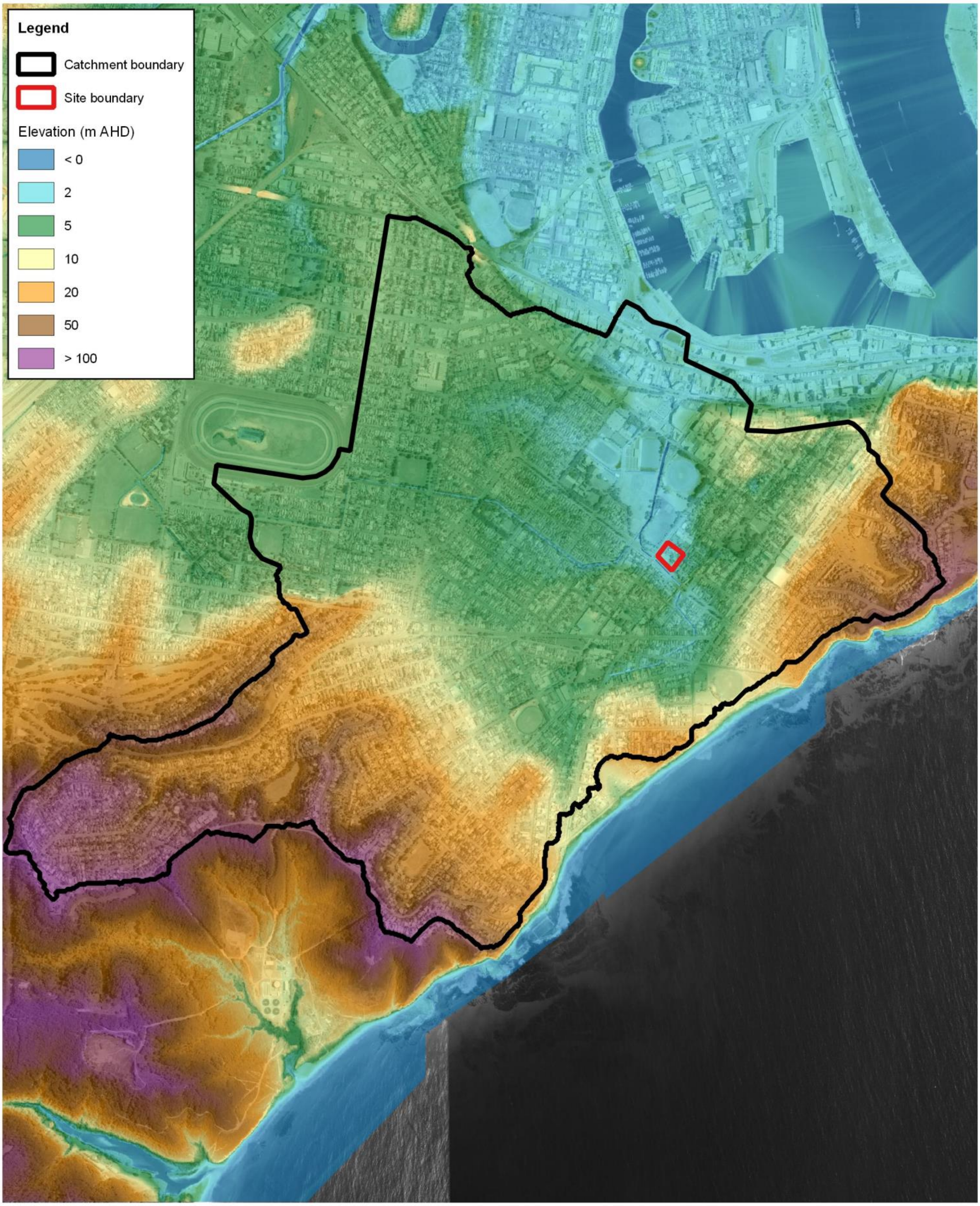

Title:

Cottage Creek Catchment Topography

0 0.7 1.4 km

approx. scale

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2 Model Development

2.1 Hydrologic Model

For this assessment, a TUFLOW model was developed covering the entire Cottage Creek catchment. The model utilised the NSW Spatial Services LiDAR data product, downloaded via the ELVIS Foundation Spatial Data portal to define the floodplain topography.

A 5 m horizontal grid resolution Digital Elevation Model (DEM) was developed using GIS-based terrain analysis techniques to remove sinks within the grid and create a hydrologically corrected DEM. This prevents the initial loss of catchment rainfall to artificial trapped storages. The TUFLOW model also adopted a 5 m grid cell resolution.

Land use coverage in the catchment was digitised using cadastral data and aerial imagery, to distinguish between urban and suburban lots, commercial areas, road reserves, grassed and vegetated open space. Depth-varying Manning's 'n' roughness coefficients were used to represent the different runoff mechanisms of sheet flow and channelised flow. Impervious percentages were also assigned to each land use type to simulate appropriate catchment infiltration losses, as presented in Table 2-1.

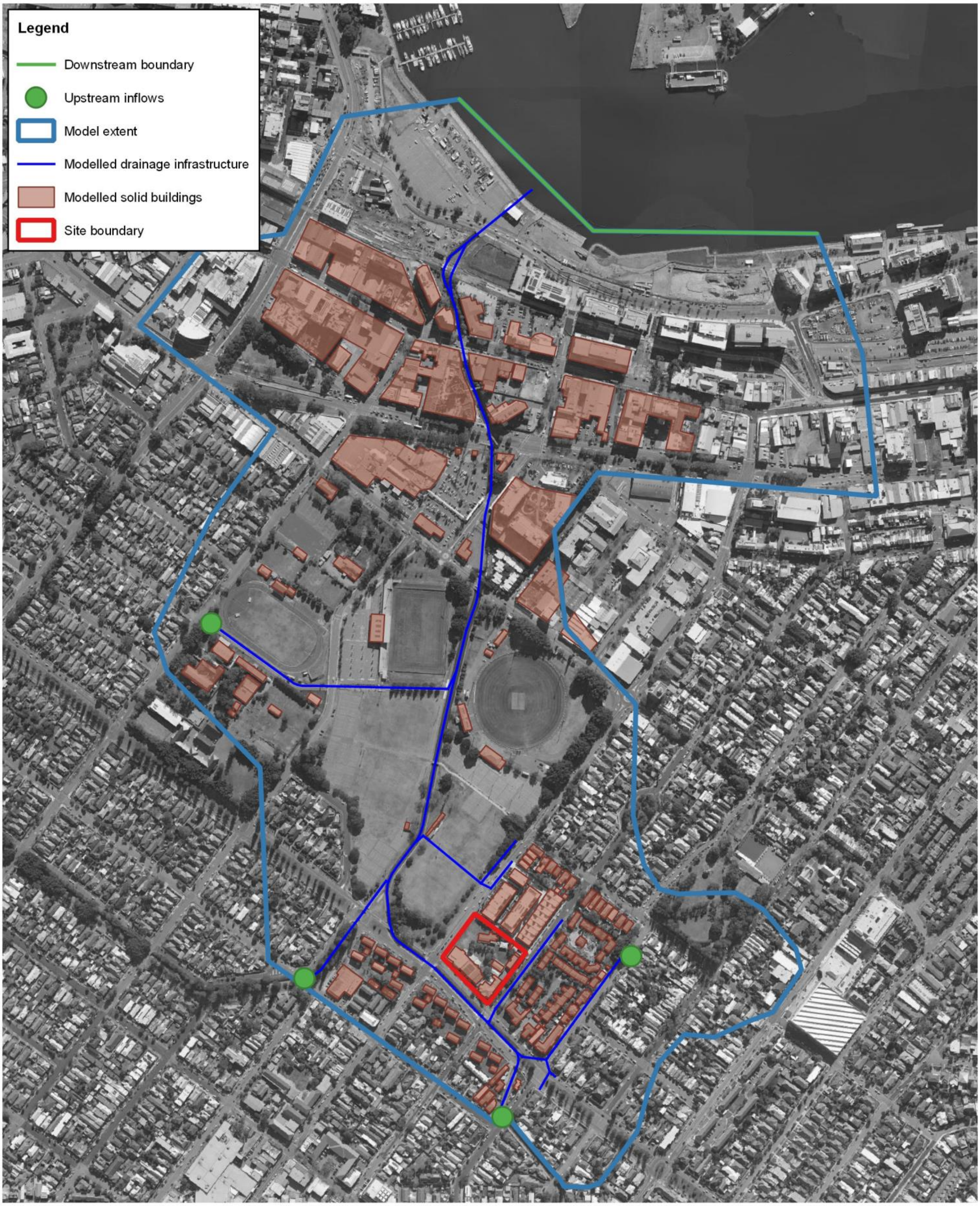
Table 2-1 Hydrological Model Land Use Parameters

Land Use	Sheet Flow 'n'	Channel Flow 'n'	Impervious Area
Urban lots	0.20	0.080	80%
Suburban lots	0.20	0.080	65%
Commercial areas	0.10	0.020	100%
Road reserve	0.15	0.025	75%
Grassed areas	0.30	0.050	0%
Vegetated areas	0.20	0.100	0%

The downstream boundary of the model was configured with a fixed tailwater condition of 0.6 m AHD, with flows freely discharging from the model above that level.

2.2 Hydraulic Model

A more detailed TUFLOW model was developed covering the Cottage Creek floodplain from Dawson Street, some 200 m upstream of the Site, downstream to the outlet at Newcastle Harbour, as presented in Figure 2-1. The model utilised the NSW Spatial Services LiDAR data product, downloaded via the ELVIS Foundation Spatial Data portal to define the floodplain topography. A 1 m horizontal grid resolution DEM was developed, with the TUFLOW model adopting a 2 m grid cell resolution.



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TUFLOW Model Components

0 200 400 m

approx. scale

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The concrete-lined open drainage channels were modelled as a 1D network, dynamically linked to the 2D floodplain. Channel dimensions were estimated using a combination of aerial imagery and LiDAR data, except for downstream of Hunter Street where survey was available. Hydraulic structures were represented as TUFLOW bridges, with appropriate pier losses and estimated bridge deck thicknesses. The modelled channel and bridge geometries were then validated through comparison with the 1% AEP flood condition in the Honeysuckle Redevelopment Area Flood Study.

Within the roads near the Site the subsurface drainage infrastructure was also modelled. Pipe alignments and sizes were based on the CoN GIS database. Most pipes are 300 mm, 375 mm or 450 mm diameter, the exception being the trunk drainage along Bruce Street, which is a 1200 mm diameter pipe. Invert levels were estimated using the LiDAR data and an assumed 600 mm minimum cover depth. Modelled stormwater pits dynamically link the 1D subsurface drainage network to the overland 2D floodplain.

Land use coverage in the catchment was digitised using cadastral data and aerial imagery, to distinguish between urban lots, areas of hardstand and grassed surfaces. The urban lots were assigned a Manning's 'n' roughness coefficient of 0.05, with areas of hardstand being assigned an 'n' value of 0.02 and grassed areas 0.035. The concrete-lined channels were modelled with a Manning's 'n' roughness coefficient of 0.015. Solid buildings and fences within the floodplain were digitised to block flow paths within the TUFLOW model.

Model inflow boundaries were extracted from the hydrological model and applied to the relative upstream inflow locations in the hydraulic model. Lateral inflows were also distributed along Cottage Creek between Parkway Avenue and Honeysuckle Drive. The downstream boundary of the model was configured as a dynamic tidal range, peaking at a level of 1.1 m AHD.

3 Design Flood Hydrology

3.1 Methodology

The TUFLOW hydrologic model (refer Section 2.1) was simulated (using the HPC solver) in accordance with the Australian Rainfall and Runoff (ARR) 2019 guidelines to derive design flood hydrology for the full range of design rainfall events between the 20% AEP and 0.2% AEP, with storm durations ranging from 15 minutes to six hours. The design rainfall depths were sourced from the BoM IFD (Intensity Frequency Duration) portal and are presented in Figure 3-1.

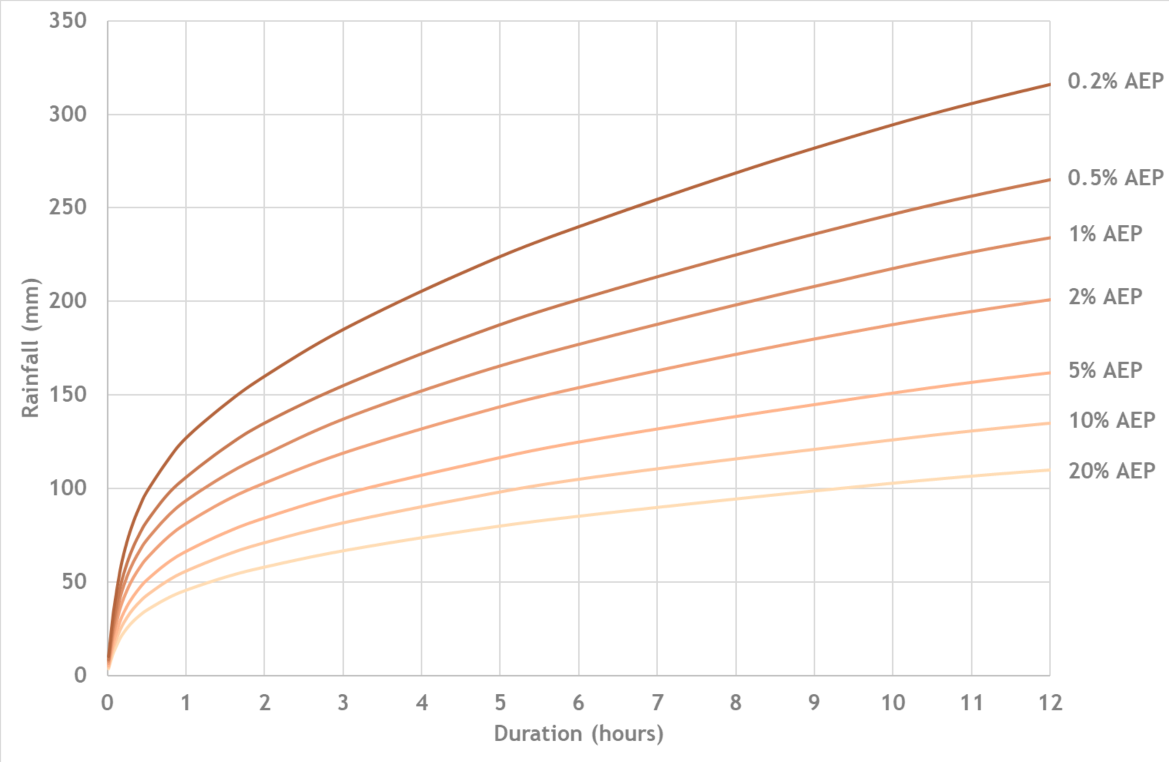


Figure 3-1 BoM Design Point Rainfall

There are three different catchments that need to be considered to derive critical flood conditions at the Site:

- The 0.5 km^2 catchment draining to Bruce Street
- The 3.6 km^2 catchment draining to Cottage Creek upstream of the Site
- The 7.3 km^2 catchment draining to Cottage Creek upstream of Parry Street.

The Bruce Street catchment provides the critical flood condition for overland flow through the Site from the Nesca Park catchment to the east. The Cottage Creek catchment upstream of the Site provides the critical condition for flood flows within the Cottage Creek channel adjacent the Site and the Cottage Creek catchment upstream of Parry Street provides the critical condition for backwater flooding from National Park.

An Areal Reduction Factor (ARF) was applied to the design point rainfall using these three contributing catchment areas. ARFs calculated in accordance with ARR 2019 for the South-east Coast ARF region range from around 0.95 at the 0.2% AEP 15-minute duration to around 1.00 at the 20% AEP six-hour duration for the Bruce Street catchment. For the Cottage Creek catchment upstream of the Site the ARFs range from around 0.88 at the 0.2% AEP 15-minute duration to around 0.99 at the 20% AEP six-hour duration. For the Cottage Creek catchment upstream of Parry Street the ARFs range from around 0.84 at the 0.2% AEP 15-minute duration to around 0.97 at the 20% AEP six-hour duration.

Design rainfall losses were represented using the Green-Ampt soil infiltration module in TUFLOW, with standard parameters for a Sandy Clay Loam. The rainfall-runoff process in urban areas is dominated by the short intense critical storm durations and the substantial area of impervious surfaces. The adopted loss conditions only have a minimal impact on the resultant catchment runoff because they represent such a small proportion of the total rainfall.

The ARR 2019 guidelines ensemble method to design flood hydrology involves the simulation of ten rainfall temporal patterns for each design event magnitude and duration, with the average condition of the ten being adopted for design purposes. The point rainfall temporal patterns provided for the East Coast South temporal rainfall region were adopted for the ensemble method accordingly.

The ensemble method will often produce a few event durations with similar peak flows. If only considering the absolute peak flow values when selecting the duration, a suite of temporal patterns exhibiting significant variance in simulated hydrograph shape may be adopted. This can result in the adoption of a hydrograph shape that is not best suited for design purposes, e.g. with a dual peak and/or too slow a response time. Typically, the shortest duration of those close to the overall critical peak flow condition provides the least variance in flows and most preferable simulated design flood hydrograph shape. It is also prudent to assess the critical duration across the full range of simulated design event magnitudes to maintain consistency where possible.

Once a suitable event duration has been selected for each design flood magnitude, a representative hydrograph must be selected from those simulated. Typically, the hydrograph that produced the peak flow closest to the calculated mean is selected. If multiple hydrographs provide suitable peak flows, then that with the most typical design hydrograph shape is preferable. However, if design flows at multiple locations are critical for the assessment then the hydrograph that provides a peak flow condition closest to the mean consistently at each location may be the most appropriate. The adoption of a suitable and consistent hydrograph shape is ultimately the key outcome of this process, as adopted hydrographs can always be scaled in magnitude to better match the calculated mean peak flow condition from the ensemble method if required.

For the simulation of the PMF (Probable Maximum Flood) condition the Generalised Short Duration Method (GSDM) published by the BoM was adopted. The critical duration of the PMF is typically shorter than that of the standard design flood events. The calculation of the Probable Maximum Precipitation (PMP) determined the rainfall depths for the 15-minute to three-hour duration events to be 180 mm to 740 mm for the Bruce Street catchment, 170 mm to 660 mm for the Cottage Creek catchment upstream of the Site and 160 mm to 620 mm for the Cottage Creek catchment upstream of Parry Street.

3.2 Model Results

The TUFLOW model simulations were analysed for the three key catchment outlets to identify the critical duration, i.e. that which produces the peak flood flows for each design event magnitude. This is undertaken by calculating the average peak flood flow and the peak flood flow variance of the ten simulated hydrographs for each design event duration and magnitude. This process can be better visualised by summarising the results for each design event magnitude as a box whisker plot containing the ensemble results for each of the simulated design event durations.

Figure 3-2 presents the ensemble method analysis for the 1% AEP event for the Bruce Street catchment. In this case the critical duration is identified as either the 30-minute or 45-minute event. With the critical duration tending to become longer for more frequent flood events, the 45-minute duration was adopted as being critical for the Bruce Street catchment.

Figure 3-3 presents the ensemble method analysis for the 1% AEP event for the Cottage Creek catchment upstream of the Site (Union Street). In this case the critical duration is readily identified as the 90-minute event.

Figure 3-4 presents the ensemble method analysis for the 1% AEP event for the Cottage Creek catchment upstream of the Parry Street. In this case the critical duration is identified as either the 2-hour or 3-hour event. With the critical duration tending to become longer for more frequent flood events, the 3-hour duration was adopted as being critical for the full Cottage Creek catchment.

3.3 Adopted Design Conditions

To best represent the critical flood conditions throughout the area covered by the hydraulic model, most of the upstream and lateral inflows were configured using the hydrographs that produced the critical flood condition for the Cottage Creek catchment upstream of Parry Street. It is this flood condition that is critical within National Park and the downstream environment of the Cottage Creek floodplain. The exceptions to this are the approach flows to the Site along Cottage Creek and excess runoff from the Bruce Street trunk drainage.

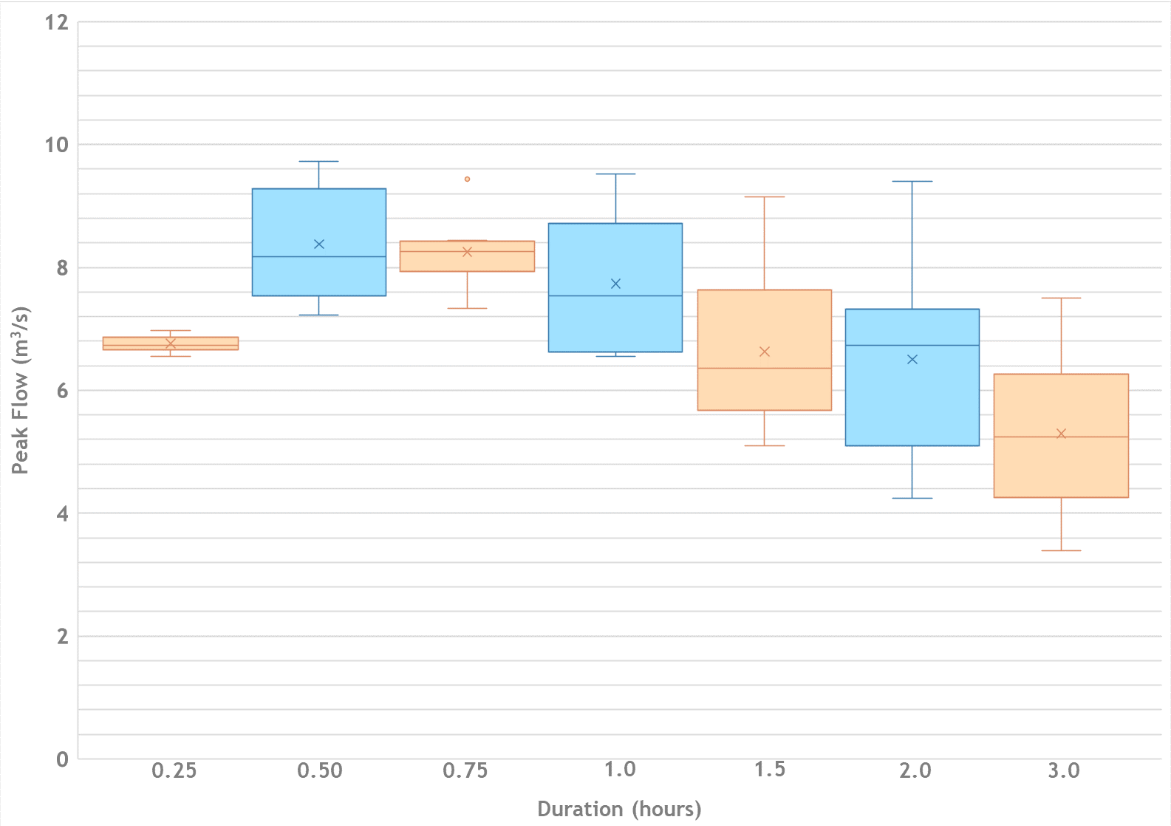


Figure 3-2 Bruce Street Ensemble Method Results for the 1% AEP Event

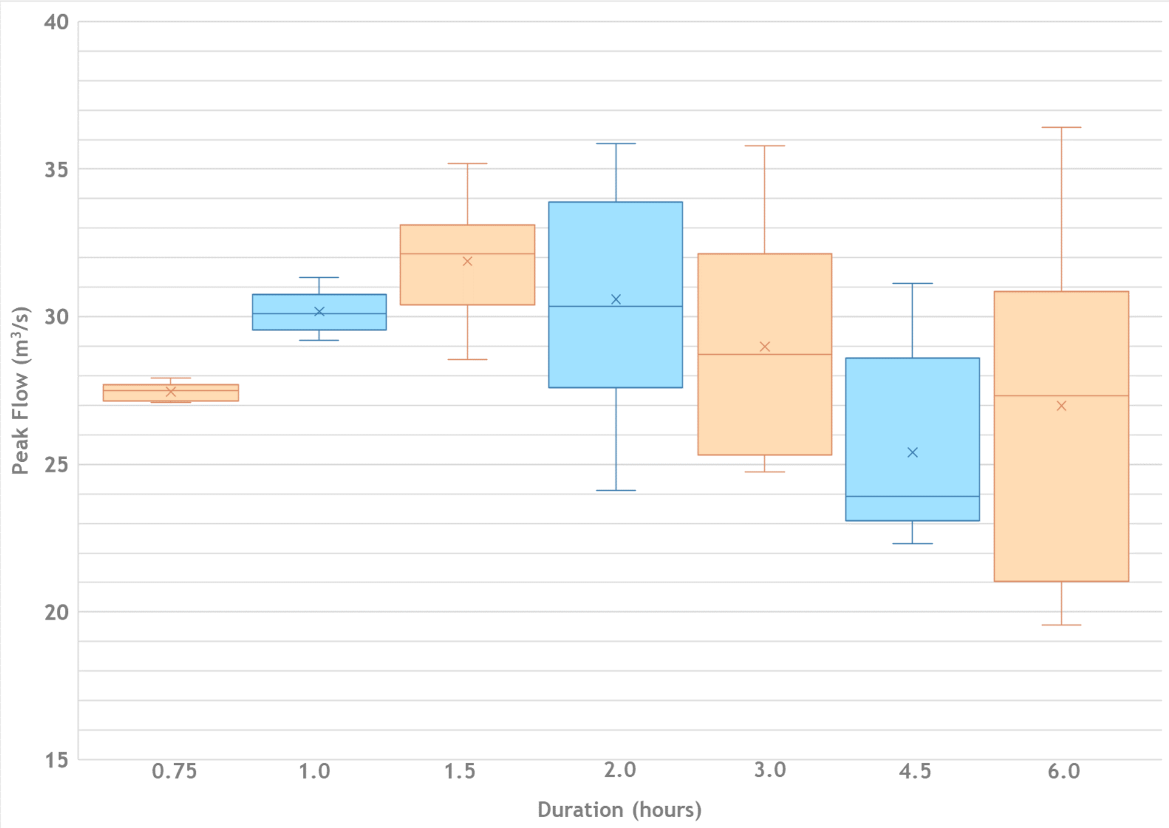


Figure 3-3 Union Street Ensemble Method Results for the 1% AEP Event

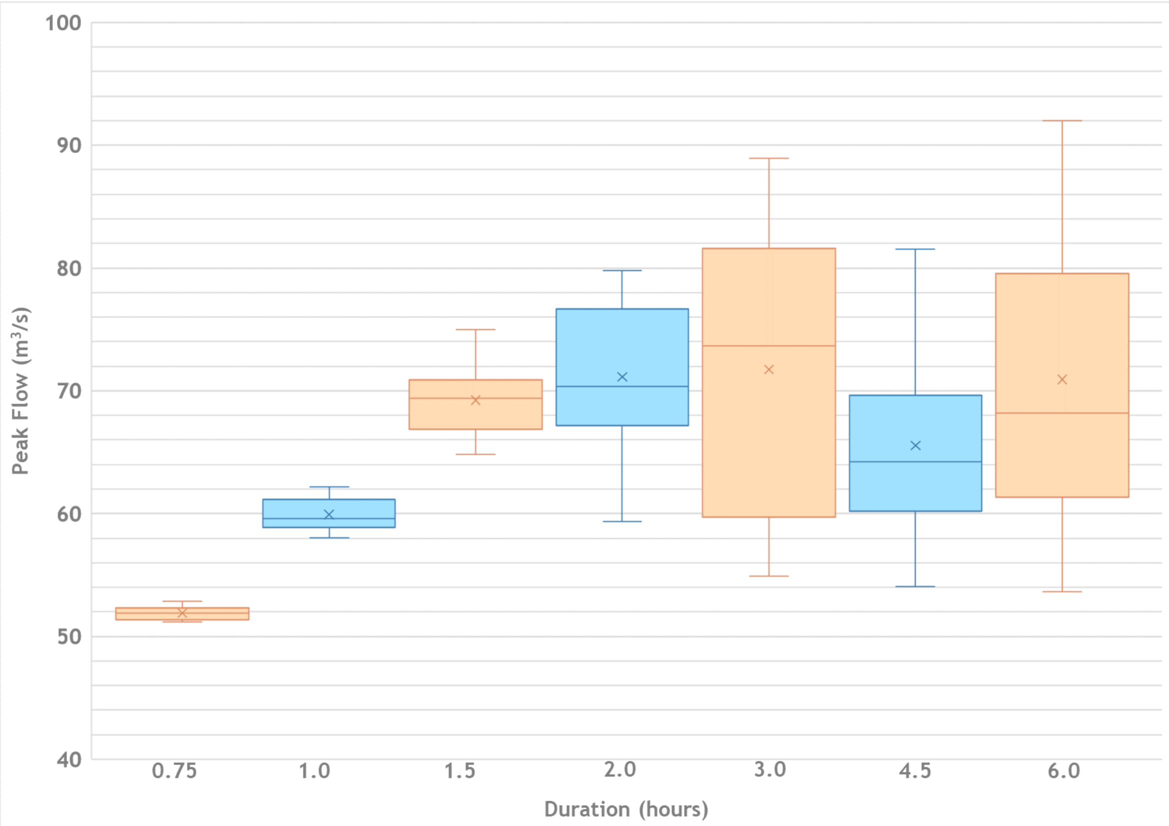


Figure 3-4 Parry Street Ensemble Method Results for the 1% AEP Event

The 45-minute critical duration hydrographs were input to the hydraulic model as inflows to the Bruce Street trunk drainage. For the Cottage Creek inflows upstream of the Site the 90-minute critical duration hydrographs were used, to provide the peak flow conditions that produce the critical flood levels local to the Site along Parkway Avenue. However, as this inflow point is a major contributor to the overall critical flood condition within National Park, the hydrographs were broadened by factoring up the time parameter to match the volume of the equivalent 3-hour critical duration hydrograph for the full Cottage Creek catchment. This ensures that the critical flood conditions within National Park and further downstream are maintained, as they are effectively a function of the flood volume being attenuated within National Park, with a controlled restricted outlet through the Cottage Creek channel reach downstream of King Street.

Models were also simulated using the 3-hour critical hydrographs throughout to confirm that the adopted design configuration was appropriate. This found that for the 2% AEP and rarer events the adopted design inflows resulted in negligible differences in peak flood levels in National Park. For the 5% AEP and more frequent events the peak flood levels in National Park resulted in less than a 0.1 m difference.

The simulation of the PMP durations found the 90-minute event to be the critical for the PMF condition for Cottage Creek upstream of the Site and at Parry Street. The 30-minute duration is the critical condition for the Bruce Street catchment.

4 Baseline Design Flood Conditions

4.1 Model Validation

The detailed TUFLOW hydraulic model (refer Section 2.2) was simulated (using the HPC solver) for the adopted design flood hydrology detailed in Section 3. The modelled 1% AEP peak flood level profile along Cottage Creek was extracted and compared to that modelled by BMT for the Honeysuckle Redevelopment Area Flood Study. The modelled channel and bridge geometries were reviewed and adjusted slightly where required, to improve the match between the two, as presented in Figure 4-1. The BMT profile does not show changes in flood levels upstream of Parkway Avenue, as this reach was only modelled as a backwater condition in the Flood Study.

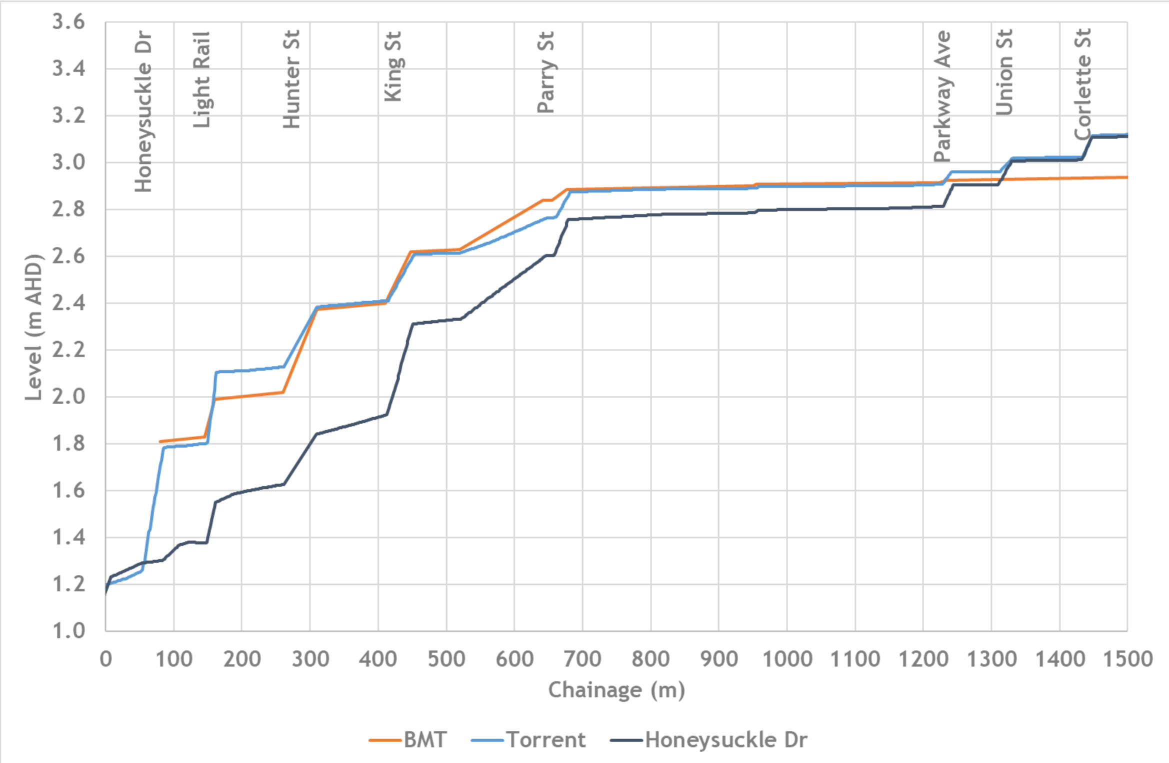
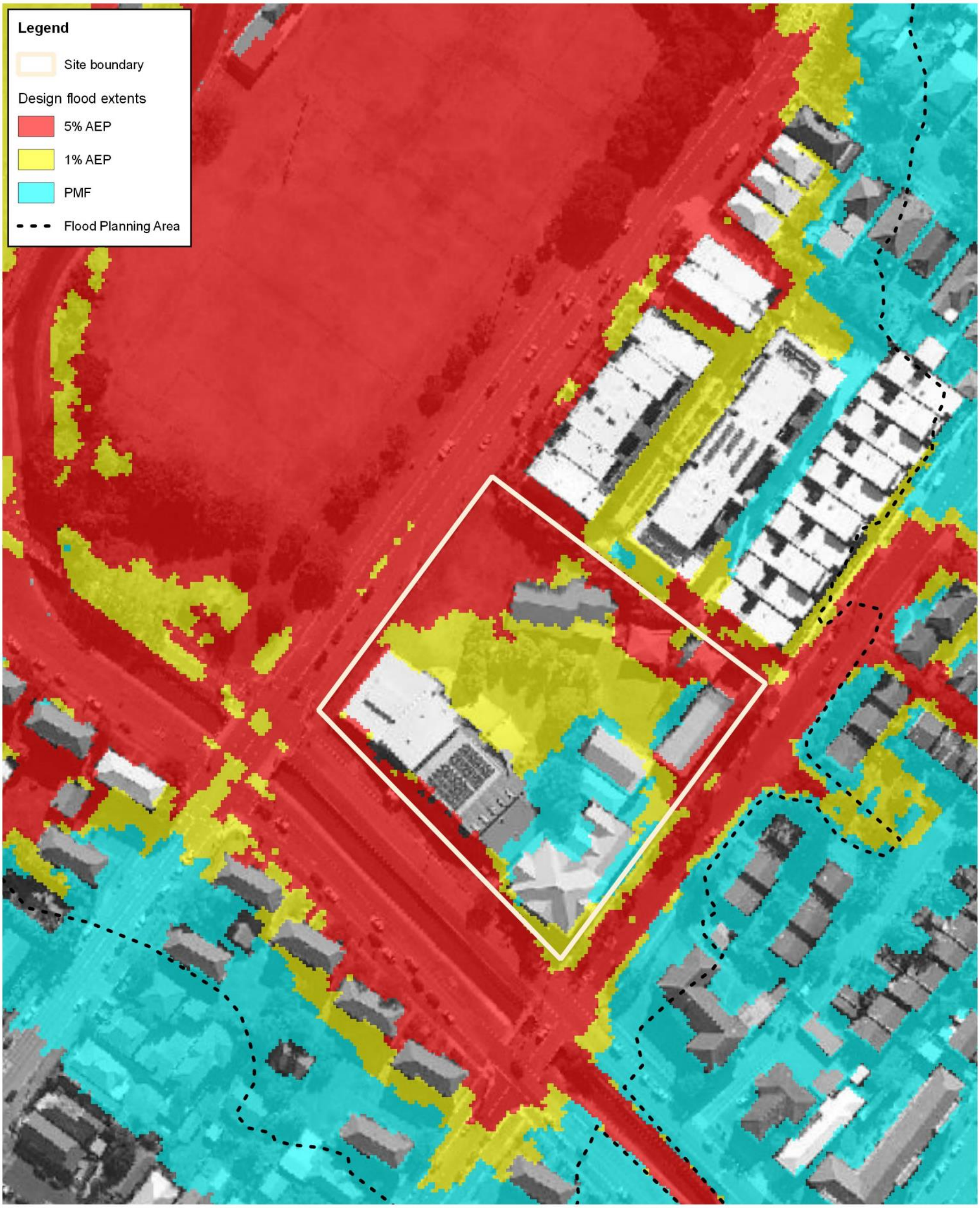


Figure 4-1 Modelled 1% AEP Flood Level Profile for Cottage Creek

The geometry of the new Honeysuckle Drive bridge has changed since the Honeysuckle Redevelopment Area Flood Study and so following model validation, the bridge details were updated, and the models re-simulated with the additional inclusion of the proposed adjacent creek channel naturalisation works. The impact of the new Honeysuckle Bridge has a significant impact on the upstream 1% AEP flood levels within Cottage Creek, reducing the peak by almost 0.6 m upstream of Hunter Street and by around 0.1 m through National Park.

4.2 Model Results

Figure 4-2 presents the modelled peak flood extents at the Site for the 5% AEP, 1% AEP and PMF events. The extent of the Flood Planning Area (FPA) has also been mapped, based on a Flood Planning Level (FPL) of 3.55 m AHD. Mapping of the modelled peak flood depths and levels for each of the simulated design events is presented in Appendix A.



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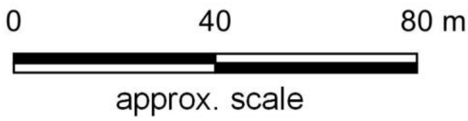
Modelled Flood Extents



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The modelled flood conditions show that the Site is not flood affected at the 20% AEP or 10% AEP events. At the 5% AEP event the capacity of Cottage Creek along Parkway Avenue is exceeded, resulting in out-of-bank inundation along Parkway Avenue encroaching within the Site, with peak flood levels reducing from around 2.8 m AHD to 2.7 m AHD from west to east. Along Union Street the flood levels reduce from 2.7 m AHD to 2.3 m AHD, which is the backwater flood level in National Park.

The trunk drainage capacity along Bruce Street is also exceeded at the 5% AEP event, which results in overland flow through to Corlette Street and then through the Site along the northern boundary. Peak flood levels along Corlette Street adjacent the Site reduce from around 3.4 m AHD to 2.8 m AHD from north to south. Through the Site the flood levels reduce from 3.4 m AHD to 2.3 m AHD, which is the backwater flood level in National Park.

Peak flood velocities within the Site at the 5% AEP event are around 0.1 m/s within the Cottage Creek floodplain along Parkway Avenue and Union Street. Along the overland flow path from Corlette Street along the northern boundary of the Site the modelled peak velocities are higher, at around 0.2 m/s to 0.4 m/s.

The flood behaviour at the 5% AEP event at the Site is exhibited for all rarer design flood events, albeit with increased flood levels, depths and velocities. The peak flood level conditions are summarised in Table 4-1. Peak flood levels are provided for four locations:

- Adjacent the north-east corner of the Site in Corlette Street
- Adjacent the south-east corner of the Site at cnr. Corlette St. & Parkway Ave.
- Adjacent the south-west corner of the Site at cnr. Parkway Ave. & Union St.
- Adjacent the north-west corner of the Site in Union St. i.e., the National Park backwater.

Table 4-1 Modelled Design Peak Flood Levels at the Site (m AHD)

Design Event	NE Corner	SE Corner	SW Corner	NW Corner
20% AEP	N/A	N/A	N/A	N/A
10% AEP	N/A	N/A	N/A	2.0
5% AEP	3.4	2.8	2.7	2.3
2% AEP	3.5	2.9	2.8	2.6
1% AEP	3.5	3.0	2.9	2.8
0.5% AEP	3.5	3.2	3.1	3.0
0.2% AEP	3.5	3.4	3.3	3.2
PMF	4.8	4.8	4.8	4.8

The modelled peak flood velocities within the Site along Parkway Avenue increase to around 0.2 m/s at the 2% AEP event, 0.3 m/s at the 1% AEP event, 0.4 m/s at the 0.5% AEP event, 0.5 m/s at the 0.2% AEP event and 0.6 m/s at the PMF event. Along the overland flow path through the northern part of the Site the modelled peak flood velocities increase locally to around 1.0 m/s at the 2% AEP event, 1.1 m/s at the 1% AEP event, 1.2 m/s at the 0.5% AEP event, 1.3 m/s at the 0.2% AEP event and 1.7 m/s at the PMF event.

4.3 Flood Hazard

The flood hazards have been determined in accordance with Guideline 7-3 of the Australian Disaster Resilience Handbook 7 Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia (AIDR, 2017). This produces a six-tier hazard classification, based on modelled flood depths, velocities and velocity-depth product. The hazard classes relate directly to the potential risk posed to people, vehicles and buildings, as presented in Figure 4-3.

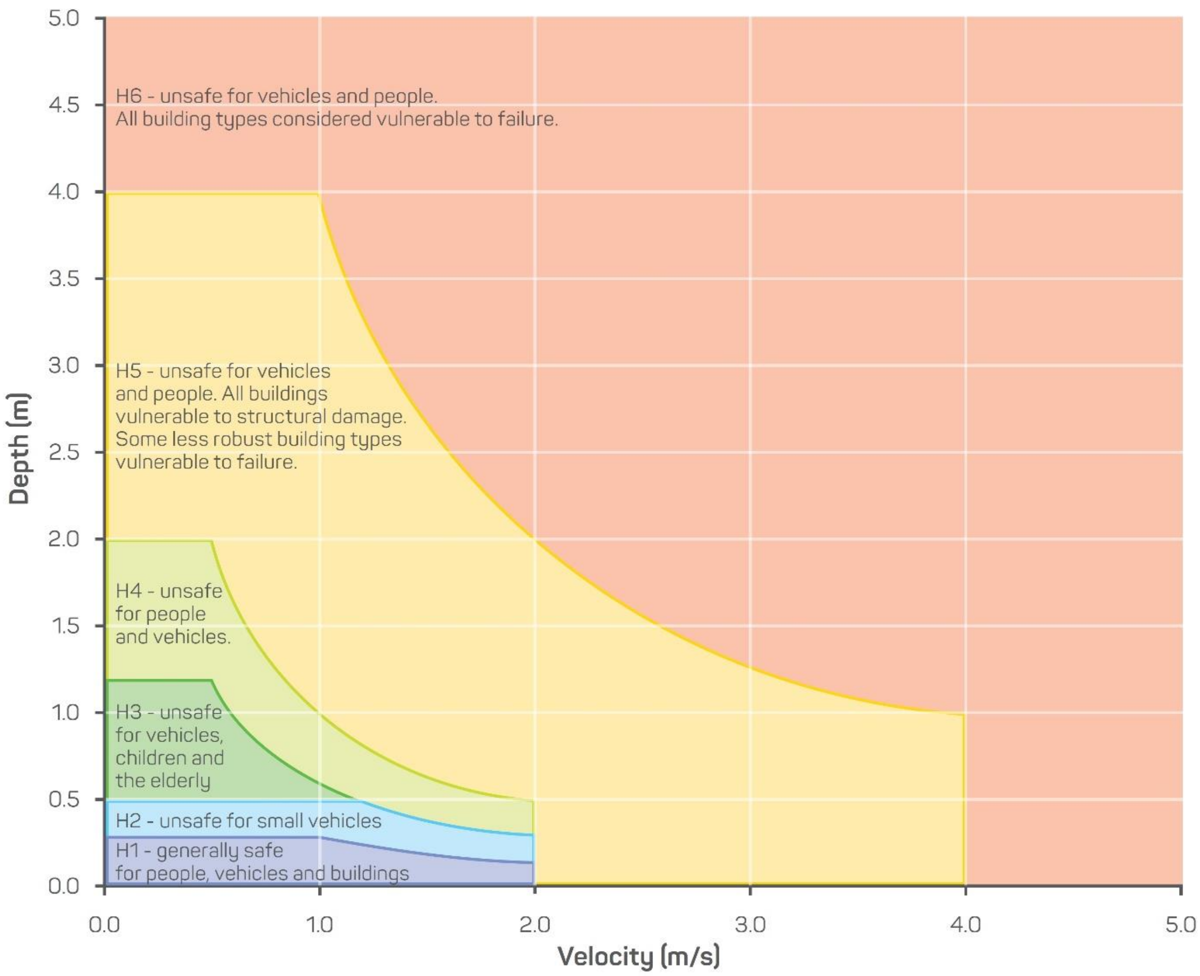


Figure 4-3 General Flood Hazard Vulnerability Curves (AIDR, 2017)

The flood hazard mapping is useful for providing context to the nature of the modelled flood risk and to identify potential constraints for the future development of the Site with regards to floodplain risk management. The principal consideration of good practice floodplain risk management is to ensure compatibility of the proposed development with the flood hazard of the land, including the risk to life and risk to property.

Mapping of the modelled peak flood hazard for each of the simulated design events is presented in Appendix B.

At the 5% AEP event the Site is effectively only impacted by a low hazard H1 flood environment, which presents a low risk to people, vehicles and buildings. At the 2% AEP event the increased flood depth creates some low hazard H2 flood conditions along Parkway Avenue and within the National Park backwater inundation in the north-west corner of the Site, which presents a potential risk for small vehicles.

As the peak flood depths increase with design event rarity, so does the resultant flood hazard. At the 1% AEP event the Site is exposed to some medium hazard H3 flooding in the north-west corner, which presents a potential risk for all vehicles, children and the elderly. The extent of H3 hazard within the Site increases at the 0.5% AEP event and includes the frontage along Parkway Avenue. At the 0.2% AEP event there are localised areas of medium hazard H4 in the north-west and south-west corners of the Site, which presents a potential risk to all vehicles and people.

There is no risk to buildings presented by the modelled flood hazards except at the PMF event when most of the Site is impacted by a high hazard H5 flood environment. This is principally a depth-driven hazard, as modelled peak velocities are effectively below 2.0 m/s. Whilst this presents a risk to less robust building structures, the heavy building constructions within the Site would not be expected to suffer significant damage.

5 Flood Impact Assessment

5.1 Management of Flood Risk

The principal consideration of good practice floodplain risk management is to ensure compatibility of the proposed development with the flood hazard of the land, including the risk to life and risk to property. Requirements within a Council's LEP (Local Environment Plan) and DCP (Development Control Plan) typically consider the management of flood risk. Section 4.01 of the CoN DCP 2012 covers Flood Management and stipulates four key sets of controls:

- Floodways
- Flood storage areas
- Management of risk to property
- Management of risk to life.

The Newcastle City-wide Floodplain Risk Management Study provides floodway and flood storage mapping in Map 4-A and Map 5-A of the Mapping Compendium for the 1% AEP and PMF events, respectively. These have been reproduced in Figure 5-1 and Figure 5-2. No parts of the Site are identified as being a floodway. At the 1% AEP event the north-west corner of the Site is identified as flood storage (comparable to the mapped H3 hazard area for the 1% AEP event presented in B-5). At the PMF event the entire Site is identified as being a flood storage area.

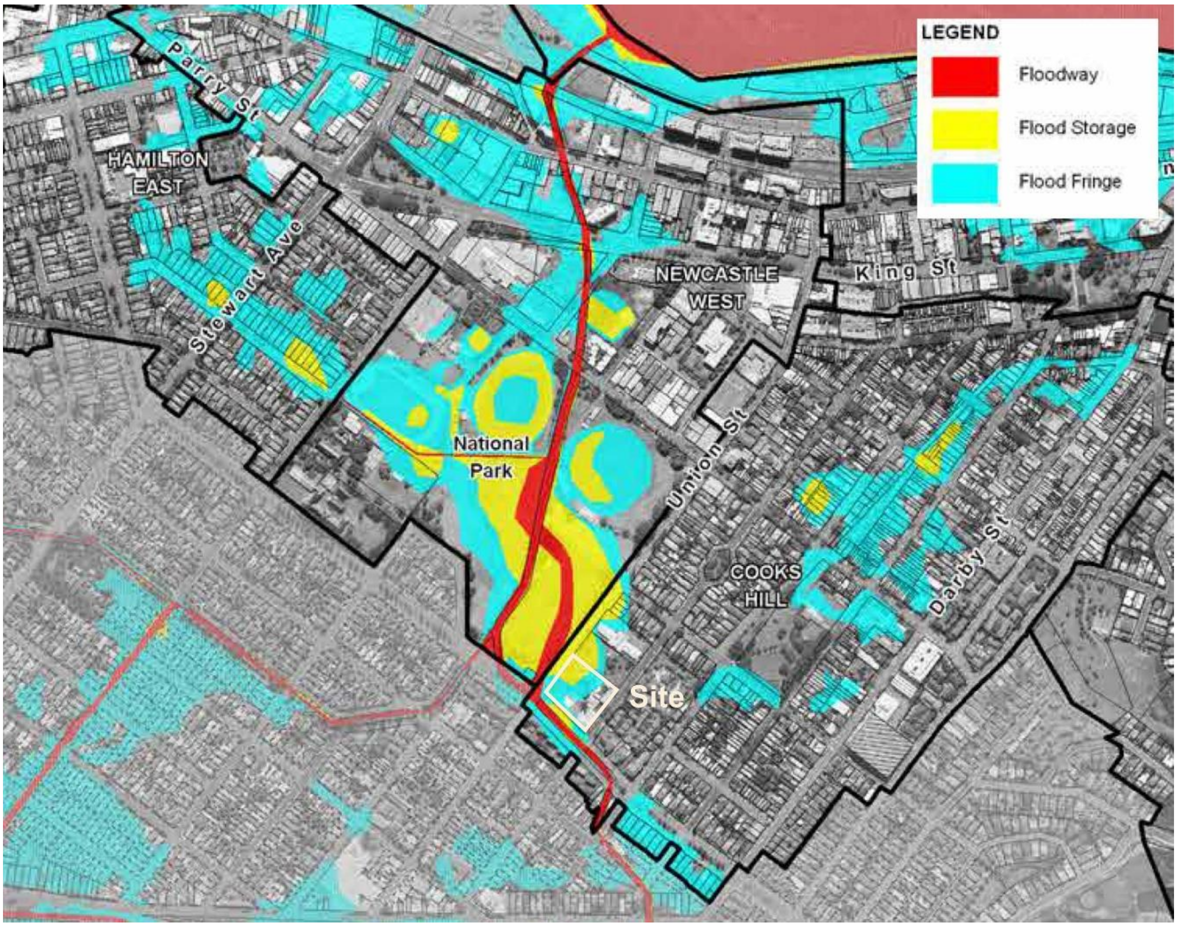


Figure 5-1 City of Newcastle 1% AEP Flood Function Map

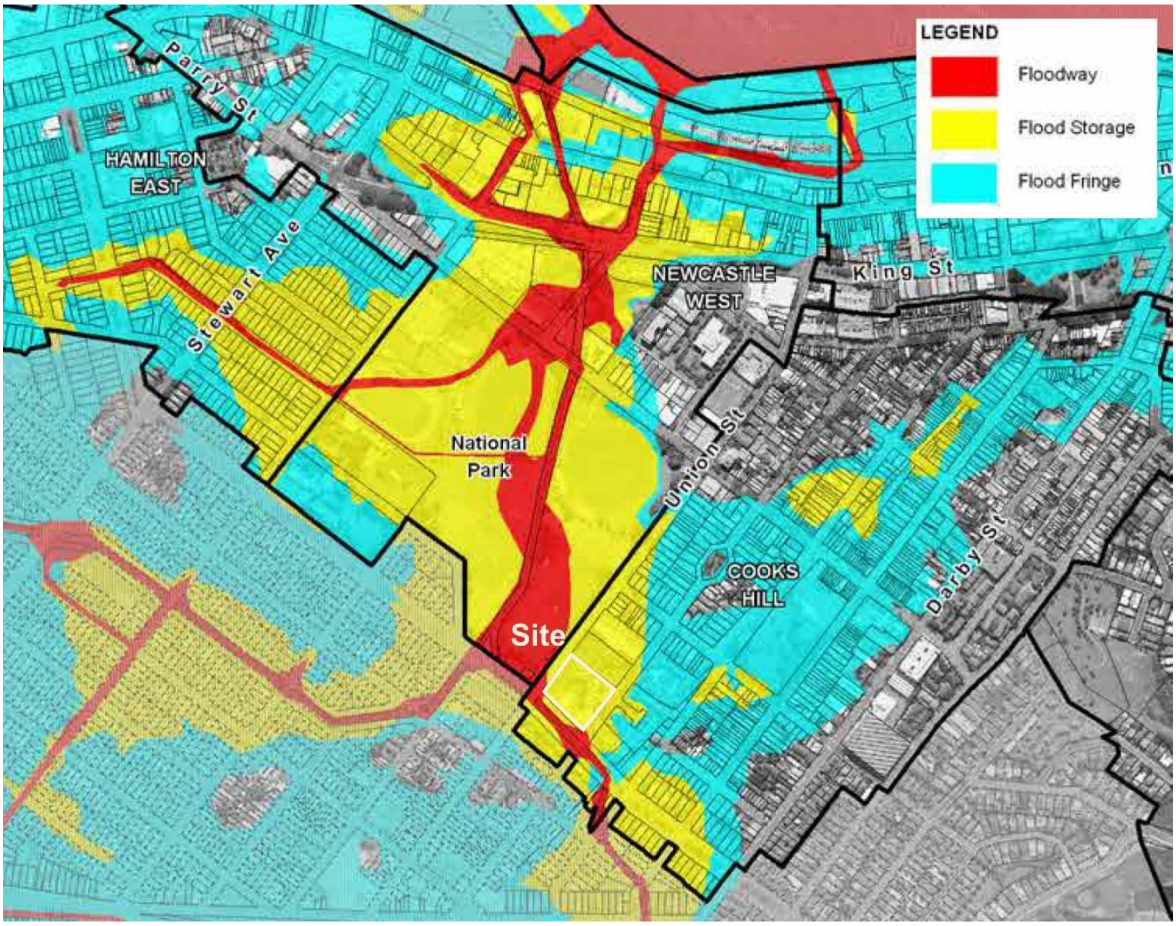


Figure 5-2 City of Newcastle PMF Flood Function Map

The principal control concerning flood storage areas is that no more than 20% of any development within the flood storage area is filled. Also, where fill is proposed, it should not result in adverse flood impacts to neighbouring properties.

As the proposed development does not include filling of the flood storage area, the requirements of the CoN DCP are readily satisfied.

5.1.1 Risk to Property

The management of risk to property controls include:

- Floor levels of all occupiable rooms are not set lower than the FPL, which is typically the 1% AEP flood level plus a 500 mm freeboard
- Garage levels are to be no lower than the 1% AEP flood level
- Electrical fixtures such as power points, light fittings and switches are sited above the FPL unless they are on a separate circuit (with earth leakage protection) to the rest of the building
- Where parts of the building are proposed below the flood planning level, they are constructed of water-resistant materials
- Areas where light vehicles are parked are not located in areas subject to a property hazard of P2 or higher, with parking for heavy vehicles not located in areas subject to a property hazard of P3 or higher

- Buildings of a light construction are generally unsuitable within areas subject to a property hazard of P4 or higher. Heavier building constructions may be suitable, but require certification by a Structural Engineer
- All building constructions are generally unsuitable within areas subject to a property hazard of P5 or higher. Where building in a P5 area is necessary, certification by a Structural Engineer is required.

The 1% AEP flood level at the Site is around 2.8 m AHD where the Stage 1 building construction is proposed and so the standard FPL requirement is that finished floor levels (FFLs) of occupiable rooms be set at or above a level of 3.3 m AHD. However, as the Cottage Creek catchment is susceptible to impacts of potential structure blockage, CoN adopted an FPL for the Honeysuckle Redevelopment Area of the 1% AEP flood event, inclusive of structure blockages, plus a 400 mm freeboard. Simulation of the 1% AEP event with a 20% blockage to all hydraulic structures results in a peak flood level at the Site of around 3.05 m AHD and a 50% blockage a flood level of around 3.25 m AHD. With the application of a 400 mm freeboard, this produces an FPL of around 3.55 m AHD.

The ground floor level of the proposed Stage 1 building is 2.57 m AHD. The principal use for the ground floor is as an open play space and so is not subject to application of an FPL. The occupiable rooms are located on the first floor and above, all of which have FFLs over 3 m above the FPL and almost 2 m above the PMF level. However, the ground floor does include WC facilities, a lift and a PE storage area.

Whilst the WCs cannot readily achieve the FFL requirements of the FPL, the principal intent of the management of risk to property can still be satisfied. This will require the WCs to be fit out using flood-compatible materials below the FPL of 3.55 m AHD. The electrical circuitry within the WCs (and throughout the ground floor level) should also be located above the FPL (or be on a separate circuit), as per the DCP requirements. The same applies to the lift, if possible, or alternatively the access to the lift can be raised to the FPL and ramp access provided. If possible, the PE storage area should include shelving to enable any high-value equipment to be stored at or above the FPL. These measures would minimise potential damages in the event of a flood inundating the ground floor level of the building.

The CoN DCP considers a P1-P5 risk to property classification, which directly correlates to an H1-H5 hydraulic hazard categories at the 1% AEP event. This was ahead of its time during the development of the City-wide Floodplain Risk Management Study. Whilst not identical to the current best practice AIDR guidelines, there are some similarities. Figure 5-3 is a reproduction of the AIDR classification in Figure 4-3, but has the CoN risk to property classes superimposed.

Risk to property classes P1 and P2 relate to the stability of light and heavy vehicles, respectively and can effectively be replaced with the H1 and H2 AIDR hazard classification. The proposed development includes seven parking bays beside the access road along the northern Site boundary. The parking bays are in an area of H2 to H3 hazard at the 1% AEP event and so do not satisfy the relevant requirements of the CoN DCP relating to vehicle parking. However, these parking bays are for a “kiss and drop” zone which will only be used around school opening or closing times, with vehicles being momentarily stopped rather than parked. Therefore, the overall intent of the management of risk to property is satisfied. The proposed Stage 1 building is only subject to an H3 hazard (P3 CoN risk to property) at the 1% AEP event and so readily satisfies the requirements relating to the management of risk to property.

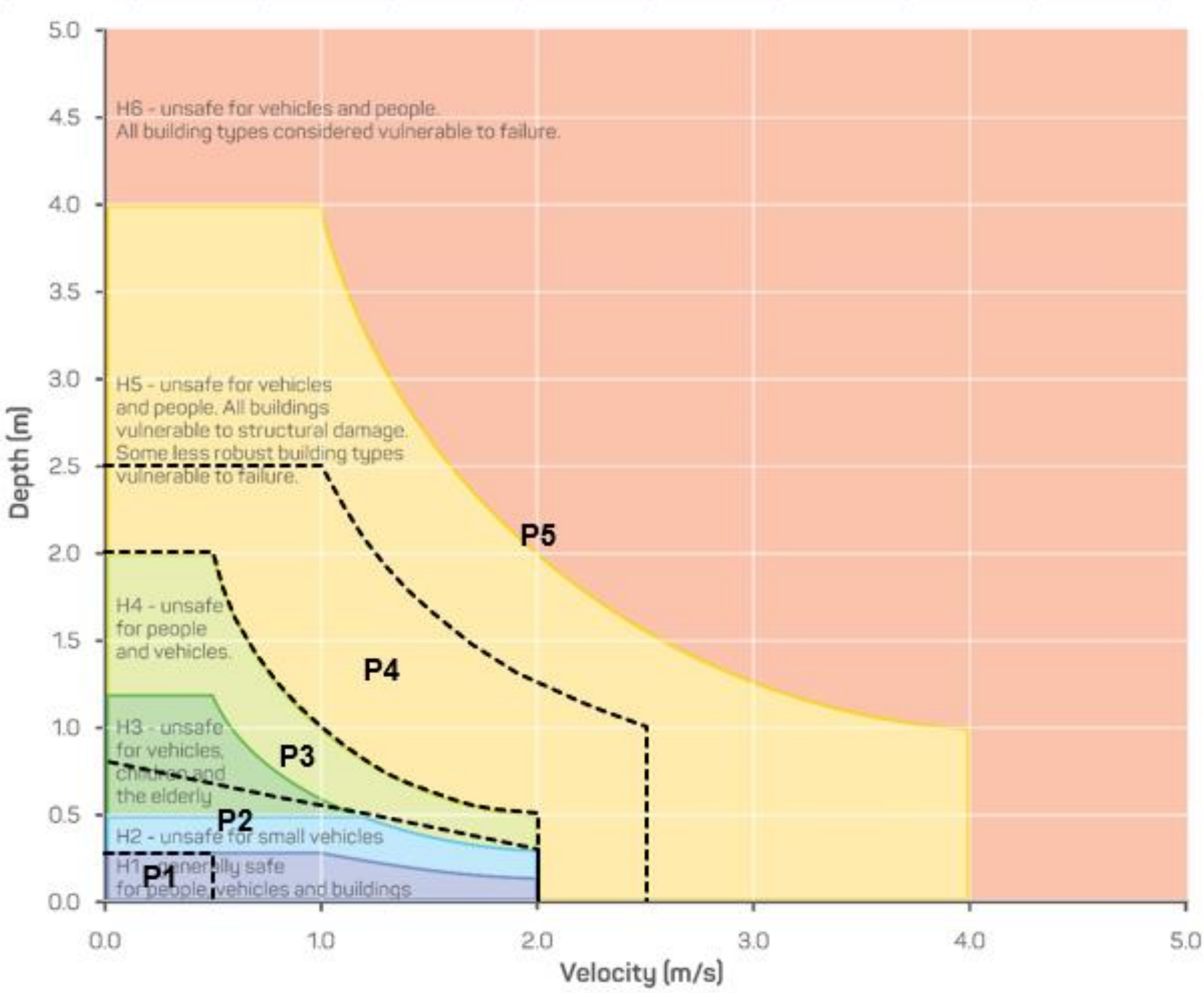


Figure 5-3 City of Newcastle DCP Risk to Property Classification (P1-P5)

The bottom level of the Stage 2 building is proposed to be a basement car park at a level of around 1.1 m AHD. Control 3 of the Newcastle DCP Section 4.01.03 requires all potential water entry points to basement parking levels should be set at or above the PMF level, which is 4.8 m AHD. The only exception to this is for vehicle entry points, which can be set at the FPL. As determined for the Stage 1 building, an appropriate FPL for the Site is 3.55 m AHD.

The vehicular access ramp to the proposed Stage 2 basement parking level will first need to rise by around 0.55 m above the existing ground surface level of 3.0 m AHD prior to descending to the intended parking level of 1.7 m AHD. All other potential water entry points such as pedestrian access and ventilation will need to be set at or above the PMF level of 4.8 m AHD prior to descent.

The proposed finished levels for floors in the Stage 2 building above the parking level are all significantly above the required FPL. Therefore, requirements for the management of risk to property from flooding are readily satisfied for the upper floors of the proposed Stage 2 development.

5.1.2 Risk to Life

The CoN DCP management of risk to life controls seek to ensure that the full potential risk to life can be managed up to the PMF event. This is guided by a risk to life classification of L1-L5 and is based on the H1-H5 hydraulic hazard categories at the PMF event. For the PMF event the entire Site is classified as H3 to H5 on the AIDR hazard classification, which becomes an L4 or L5 CoN risk to life classification. The differentiation between L4 and L5 are the thresholds of a 2.5 m flood depth, a 2.5 m/s flood velocity and velocity-depth product of 2.5.

The modelled peak flood velocities around the proposed Stage 1 building are generally lower than 1 m/s and peak locally at around 1.7 m/s. The modelled peak flood depth is around 2.2 m and the velocity-depth product is less than 1.4. Therefore, the proposed Stage 1 building is classified as an L4 risk to life hazard.

The modelled peak flood velocities around the proposed Stage 2 building are generally lower than 1.2 m/s and peak locally at around 2.0 m/s. The modelled peak flood depth is around 2.2 m and the velocity-depth product is less than 1.5. However, the internal flood depth within the building would be around 3.1 m. Therefore, the proposed Stage 2 building is classified as an L5 risk to life hazard.

Being located within a flash flood environment and being further than 40 m from flood-free land above the PMF, the proposed development is required to provide on-site flood-free refuge. The requirements for the onsite refuge are:

- The minimum FFL of the refuge is the level of the PMF, i.e., 4.8 m AHD and it is to be designed to cater for the number of people reasonably expected on the Site and are provided with emergency lighting
- To be of a construction type able to withstand the effects of flooding, requiring certification by a Structural Engineer that the building can withstand the hydraulic forces of the PMF conditions.

To satisfy the management of risk to life requirements the proposed development needs certification by a Structural Engineer to withstand the hydraulic forces of the PMF conditions, i.e. a flood depth of 2.2 m, flood velocity of 1.8 m/s and velocity-depth product of 1.4 for the Stage 1 building and an external flood depth of 2.2 m, internal flood depth of 3.1 m, flood velocity of 2.0 m/s and velocity-depth product of 1.5 for the Stage 2 building.

Because the upper levels of the Stage 1 building (including the first floor) are all located above the PMF level, it inherently provides suitable flood-free refuge for the occupants. For the Stage 2 building the first-floor level is 0.1 m below the PMF level and so only the second floor will provide a flood-free refuge. However, in practical terms the first floor could also be used as a flood refuge, with the PMF event only resulting in shallow low hazard inundation of 0.1 m depth to the first floor. In the extremely unlikely event that a PMF occurred whilst seeking refuge on the first floor, occupants could readily retreat to the second floor.

It is understood that a Flood Emergency Response Plan is already in place for the Site. The Plan should be updated to include the upper floors of the Stage 1 and Stage 2 buildings as a flood refuge. The buildings that are being demolished as part of the Stage 1 works had ground floor levels only and so their removal does not reduce the availability of existing flood refuge areas. Following the Stage 1 works, all buildings at the Site, i.e., Block A, Block B and the proposed Stage 1 and Stage 2 buildings should provide their own flood refuge within the upper floors.

5.1.3 Comparison to the June 2007 Flood Event

It was noted in the SEARs that the June 2007 flood level was higher than the predicted 1% AEP flood level. There are two main reasons for this. First, although the June 2007 flood has been considered comparable to a 1% AEP design event across the city, the local rainfall conditions in the upper Cottage Creek catchment suggest that Cottage Creek likely experienced a flood event of a 0.5% AEP to 0.2% AEP rarity. This was assessed in Section 3 of the Honeysuckle Redevelopment Area Flood Study. Also, the Cottage Creek channel experienced a significant structure blockage, including a shipping container within the railway bridge structure.

The available flood level survey data suggests that the peak flood level attained within National Park during the June 2007 event was around 3.4 m AHD. Table 5-1 presents the modelled peak flood levels in National Park for the 1% AEP, 0.5% AEP and 0.2% AEP events, considering modelled structure blockages of 0%, 20% and 50%. It shows that the peak flood levels experienced in the June 2007 event are comparable to a 0.5% AEP to 0.2% AEP design event with an impact of structure blockage. It should be noted that the model results presented in Table 5-1 include the new Honeysuckle Drive bridge and so for the June 2007 conditions these results would be expected to be around 0.1 m higher, as discussed in Section 4.1. The recommended FPL for the Site of 3.55 m AHD is 0.25 m higher than the standard 1% AEP plus 500 mm freeboard.

Table 5-1 Modelled Design Peak Flood Levels at National Park (m AHD)

Design Event	0% Blockage	20% Blockage	50% Blockage
1% AEP	2.80	3.05	3.25
0.5% AEP	3.00	3.15	3.35
0.2% AEP	3.25	3.35	3.50

5.1.4 Impacts of Potential Climate Change

In 2009 the NSW Government incorporated consideration of potential climate change impacts into relevant planning instruments. The NSW Sea Level Rise Policy Statement (DECCW, 2009) was prepared to support consistent adaptation to projected sea level rise impacts. The policy statement incorporated sea level rise planning benchmarks for use in assessing potential impacts of sea level rise in coastal areas, as well as in flood risk and coastal hazard assessments. The benchmarks were a projected rise in sea level, relative to the 1990 mean sea level, 0f 0.4 m by 2050 and 0.9 m by 2100.

The baseline design flood conditions in this assessment adopted a coincident 1.1 m AHD peak tide level at the downstream boundary, which is consistent with previous studies and similar to the highest astronomical tide. The 1% AEP event has been simulated with the downstream boundary increased by 0.4 m and 0.9 m to represent the potential sea level rise conditions for the 2050 and 2100 future planning horizons. This sensitivity test resulted in an increase to the 1% AEP flood level at the Site (from the National Park backwater inundation) of 0.05 m for the 2050 horizon and by 0.25 m for the 2100 horizon.

The Intergovernmental Panel on Climate Change (IPCC) is the leading body for the assessment of climate change globally. Since its establishment in 1988, the IPCC have released five climate change reports, the most recent of which is known as the “Fifth Assessment Report” (or AR5), which was released in four parts between September 2013 and November 2014. The AR5 provides a thorough discussion about climate change science, with the outcome of the study focused strongly on the documentation of the likely impacts of climate change in the global context.

The documented impacts were representative of broad geographical regions (i.e., polar and equatorial regions) and were based on a range of future greenhouse gas emissions and concentration scenarios (IPCC, 2013). These future scenarios are referred to as Representative Concentration Pathways (RCPs). They focus on the “concentrations” of greenhouse gases that lead directly to a changed climate and include a “pathway” – the trajectory of greenhouse gas concentrations over time, to reach a particular radiative forcing at 2100. The four RCPs cover a range of emission scenarios with and without climate mitigation policies. For example, RCP8.5 is based on minimal effort to reduce emissions. Focus is given to RCP4.5 (low emissions pathway) and RCP8.5 (high emissions pathway) and this has been reflected in the interim climate change factors presented in ARR 2019.

The ARR Data Hub provides projected increases in rainfall intensity at the Site of 6.4% to 9.0% at the 2050 horizon for the RCP4.5 and RCP8.5, respectively. At the 2090 horizon (the furthest provided) the projections in increased rainfall intensity are 9.5% to 19.7%. When taking the 0.5% AEP and 0.2% AEP flood events as surrogates for simulating increased rainfall intensities, the relative increases in rainfall intensity to the 1% AEP (3-hour duration) event are around 13% and 34%, respectively.

From this data we can determine that the expected increase in the 1% AEP flood level at the Site is around 0.10 m to 0.15 m for the 2050 planning horizon and 0.15 m to 0.30 m for the 2090 planning horizon. Combining these with the flood level increases of the sea level rise projections gives an expected increase in the 1% AEP flood level at the Site of around 0.15 m to 0.20 m for the 2050 planning horizon and 0.60 m to 0.75 m for the 2090-2100 planning horizon.

These potential climate change impacts have implications for freeboard that the current FPL provides in the future. However, occupiable rooms of the proposed Stage 1 building are on the upper floors, which are significantly elevated above the FPL and potential future climate change impacts. As such, the proposed development would remain compatible with the flood hazard of the land, it is just the frequency of flood inundation that might be expected to increase.

5.2 Management of Flood Impacts

In addition to the management of flood risk exposure of the proposed development, the potential for off-site flood impacts to the existing baseline flood conditions also need to be considered to avoid adverse impacts to neighbouring property and infrastructure. The details contained in the site plans were incorporated into the TUFLOW model to assess the potential flood impacts, including the construction of the Stage 1 building and the removal of the buildings that are to be demolished as part of the associated works.

The design flood events were then re-simulated, and the results compared to the baseline results to identify potential flood impacts.

The modelled post-development flood hazard classification is presented for the 1% AEP and PMF events in Figure 5-4 and Figure 5-5, respectively. The results of the flood impact assessment are presented in Appendix C and Appendix D for the modelled peak flood level and flood velocity impacts, respectively.

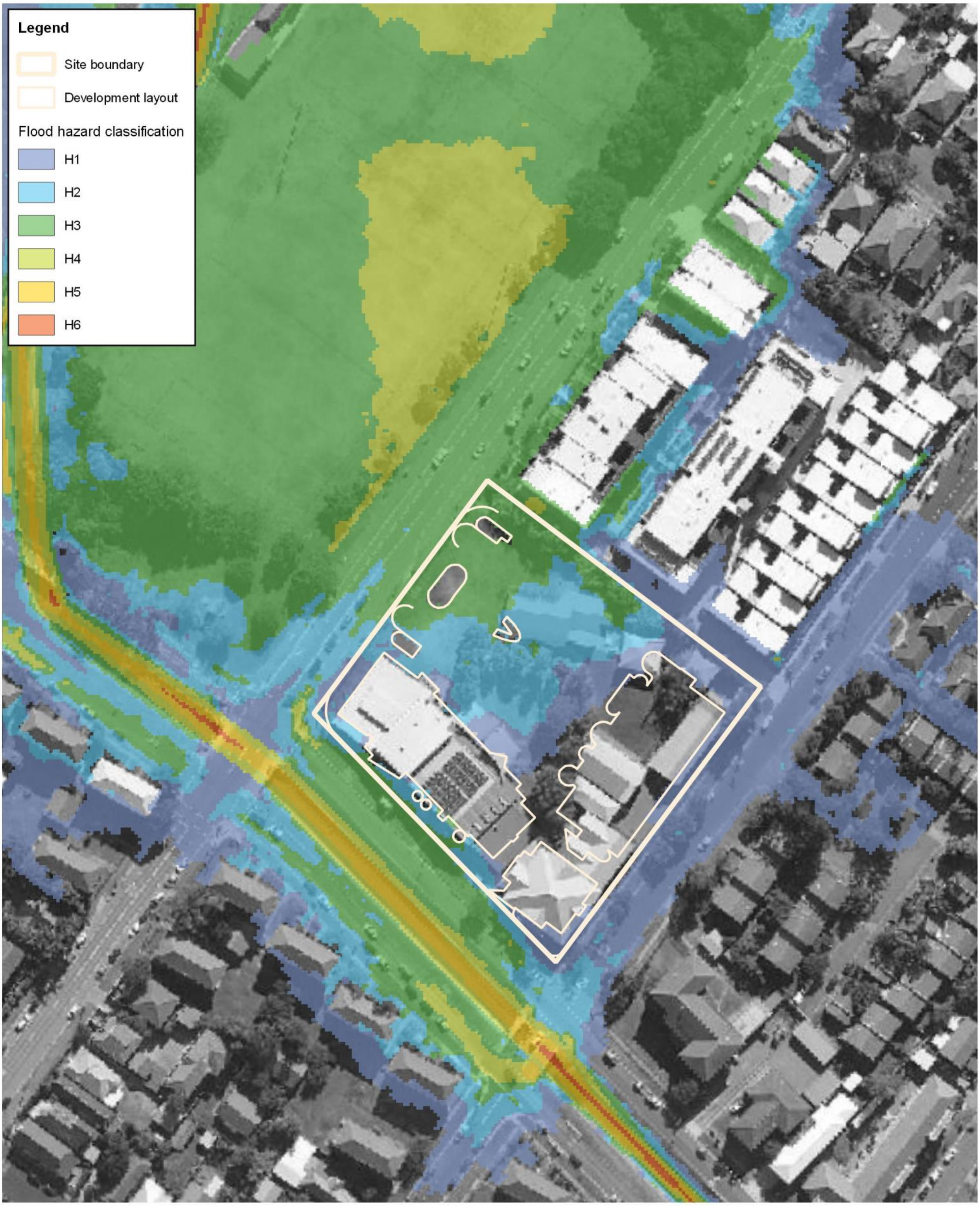
The assessment indicates that the flood impacts of the proposed development are largely contained within the Site boundary. There are no impacts at the 20% AEP or 10% AEP events as the Site is not inundated. The modelled changes to peak flood levels are largely negligible, with only a minor localised increase of 40 mm to 60 mm in the area between the Block A, Block B and Stage 2 buildings at the 0.2% AEP event.

At the 5% AEP event there is a localised increase to the modelled peak flood velocities along the northern side of the proposed Stage 2 building, where there is a redistribution of the local overland flow path through the northern part of the Site. However, the absolute modelled peak velocities along the flow path are similar in both the pre- and post-development conditions at around 0.4 m/s. The nature of this impact is consistent throughout the rarer design flood events, but overall peak velocities remain relatively low, being only around 1.0 m/s at the 0.2% AEP event.

For the 2% AEP and rarer events there is a localised increase to the modelled peak flood velocities near the western corner of the Block A building, around the proposed wall that will display the school crest. Between Block A and the proposed sign, the modelled peak flood velocities reach around 1.1 m/s at the 2% AEP event, 1.5 m/s at the 1% AEP event, 1.6 m/s at the 0.5% AEP event and 1.7 m/s at the 0.2% AEP event. Around the outside of the sign, within Union Street, the modelled peak flood velocities reach around 1.5 m/s at the 2% AEP event, 1.8 m/s at the 1% AEP event, 2.1 m/s at the 0.5% AEP event and 2.2 m/s at the 0.2% AEP event. Whilst velocities of around 2 m/s are relatively high, they do not pose a significant risk to the existing building or road infrastructure or change the overall flood hazard or risk profile within the roadway and so are not considered to be adverse impacts.

Because of the minor impacts of the proposed development and the surrounding urban environment there are negligible effects to beneficial floodplain inundation, erosion, siltation, riparian vegetation or the stability of river banks or watercourses.

Potential impacts to the social and economic costs to the community as a consequence of flooding from the proposed development are expected to be beneficial, as it is removing buildings that have an existing risk to property and offer no suitable flood refuge to manage the risk to life. These buildings are to be replaced by a new building that minimises the risk to property and provides substantial flood refuge opportunity.



Title:

1% AEP Flood Hazard Classification Post-development Conditions

0 40 80 m

approx. scale

Figure: **5-1**

Revision: **B**

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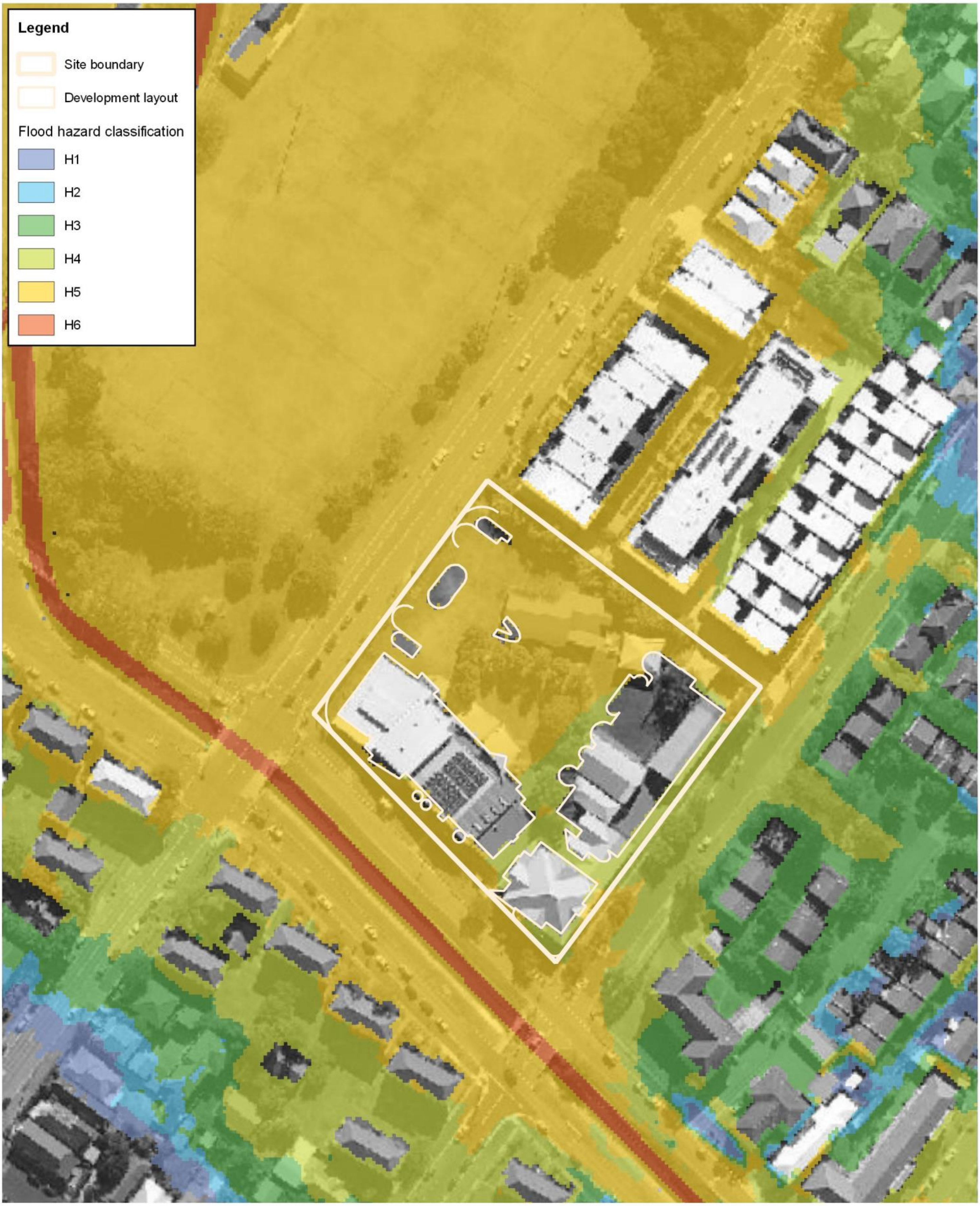

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PMF Flood Hazard Classification Post-development Conditions

0 40 80 m

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Figure: **5-2**

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6 Conclusion

Flood hazard mapping has been produced showing that the Site is of a low risk to property but a high risk to life, which requires appropriate management. The proposed Stage 1 building should conform to the requirements of the CoN DCP, with an appropriate FPL being 3.55 m AHD. The ground floor level is below the FPL but is principally designated as an open play area. However, there are proposed WC facilities, a lift and PE storage area on the ground floor that require appropriate flood-proofing to manage the risk to property and minimise damages in the event of a flood.

The bottom level of the Stage 2 building is proposed to be a basement car park at a level of around 1.1 m AHD. Control 3 of the Newcastle DCP Section 4.01.03 requires all potential water entry points to basement parking levels should be set at or above the PMF level, which is 4.8 m AHD. The only exception to this is for vehicle entry points, which can be set at the FPL. As determined for the Stage 1 building, an appropriate FPL for the Site is 3.55 m AHD. The vehicular access ramp to the proposed Stage 2 basement parking level will first need to rise by around 0.55 m above the existing ground surface level of 3.0 m AHD prior to descending.

To satisfy the management of risk to life requirements the proposed development needs certification by a Structural Engineer to withstand the hydraulic forces of the PMF conditions, i.e. a flood depth of 2.2 m, flood velocity of 1.8 m/s and velocity-depth product of 1.4 for the Stage 1 building and an external flood depth of 2.2 m, internal flood depth of 3.1 m, flood velocity of 2.0 m/s and velocity-depth product of 1.5 for the Stage 2 building.

Because the upper levels of the Stage 1 building (including the first floor) are all located above the PMF level, it inherently provides suitable flood-free refuge for the occupants. In practical terms, the same also applies to the upper floors of the Stage 2 building (with the first floor level being only 0.1 m below the PMF). It is understood that a Flood Emergency Response Plan is already in place for the Site. The Plan should be updated to include the upper floors of the Stage 1 and Stage 2 buildings as a flood refuge. Block A, Block B and the proposed Stage 1 and Stage 2 buildings should each provide their own individual flood refuge within the upper floors.

An assessment of potential future climate change has found the combined effects of increased rainfall intensity and sea level rise projections gives an expected increase in the 1% AEP flood level at the Site of around 0.15 m to 0.20 m for the 2050 planning horizon and 0.60 m to 0.75 m for the 2090-2100 planning horizon.

These potential climate change impacts have implications for freeboard that the current FPL provides in the future. However, occupiable rooms of the proposed Stage 1 building are on the upper floors, which are significantly elevated above the FPL and potential future climate change impacts. As such, the proposed development would remain compatible with the flood hazard of the land, it is just the frequency of flood inundation that might be expected to increase.

The proposed development configuration has been modelled to assess potential flood impacts to neighbouring properties. The only location where the proposed development results in a tangible off-site impact is around the western corner of the Site within Union Street, with peak velocities of around 2 m/s. Whilst velocities of around 2 m/s are relatively high, they do not pose a significant risk to the existing building or road infrastructure or change the overall flood hazard or risk profile within the roadway and so are not considered to be adverse impacts. No specific measures are required to manage this impact.

Potential impacts to the social and economic costs to the community as a consequence of flooding from the proposed development are expected to be beneficial, as it is removing buildings that have an existing risk to property and offer no suitable flood refuge to manage the risk to life. These buildings are to be replaced by a new building that minimises the risk to property and provides substantial flood refuge opportunity.

7 References

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Appendix A Baseline Design Flood Depth Mapping

Title:

Modelled 20% AEP Flood Depths and Levels

0 40 80 m

approx. scale

Figure:	A-1
Revision:	A

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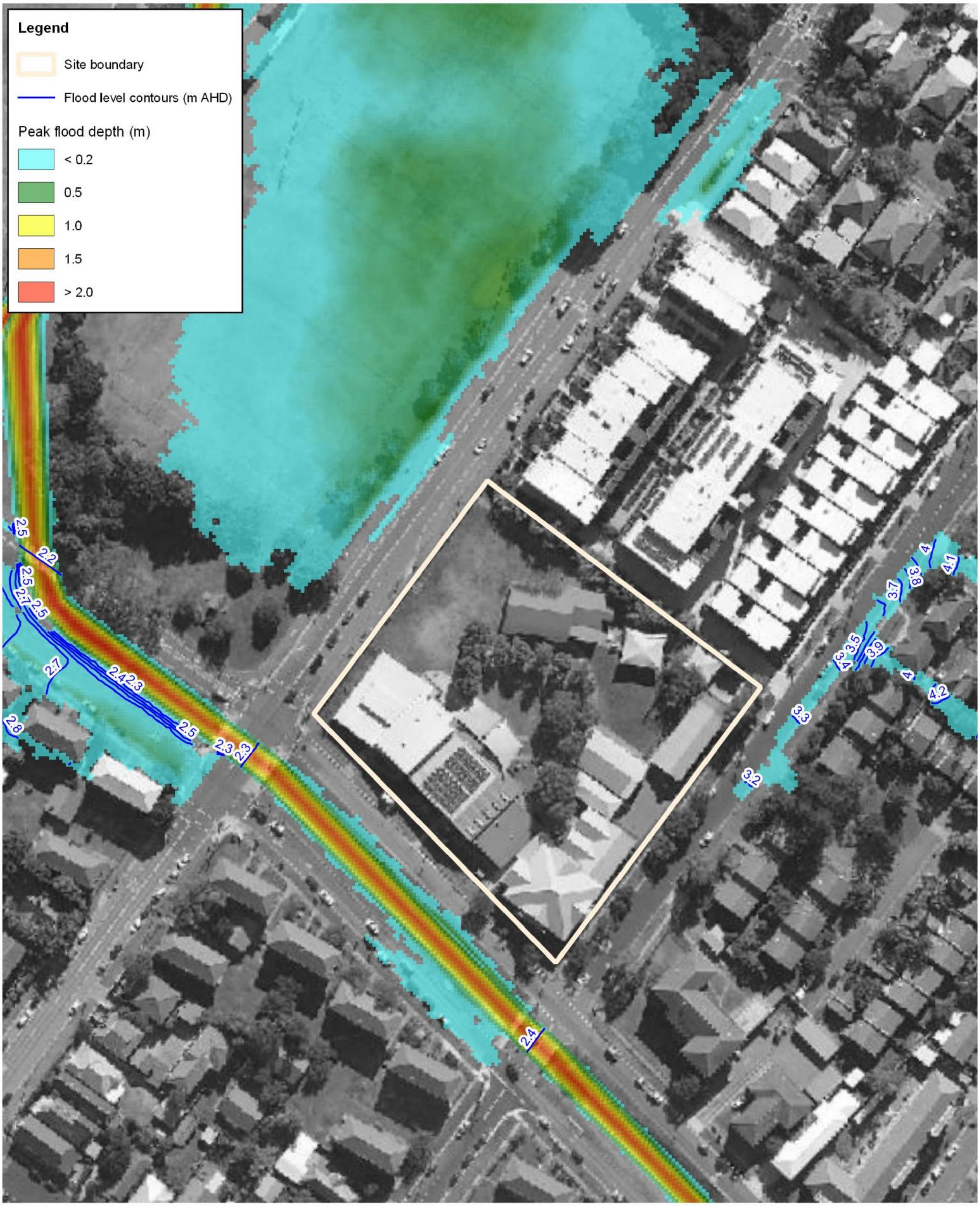

Title:

Modelled 10% AEP Flood Depths and Levels

0 40 80 m

approx. scale

Figure: **A-2**

Revision: **A**

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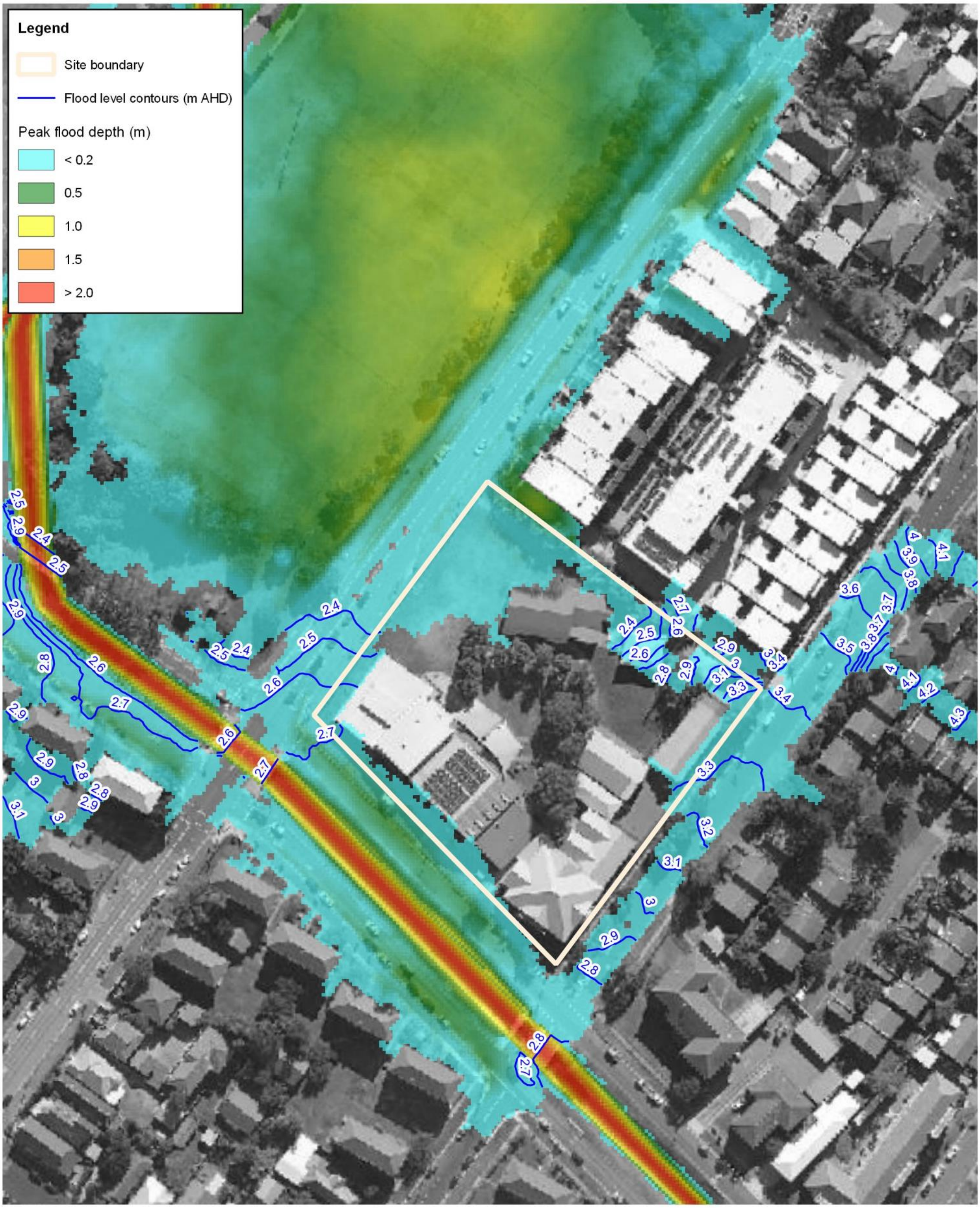

Title:

Modelled 5% AEP Flood Depths and Levels

0 40 80 m

approx. scale

Figure:	A-3
Revision:	A

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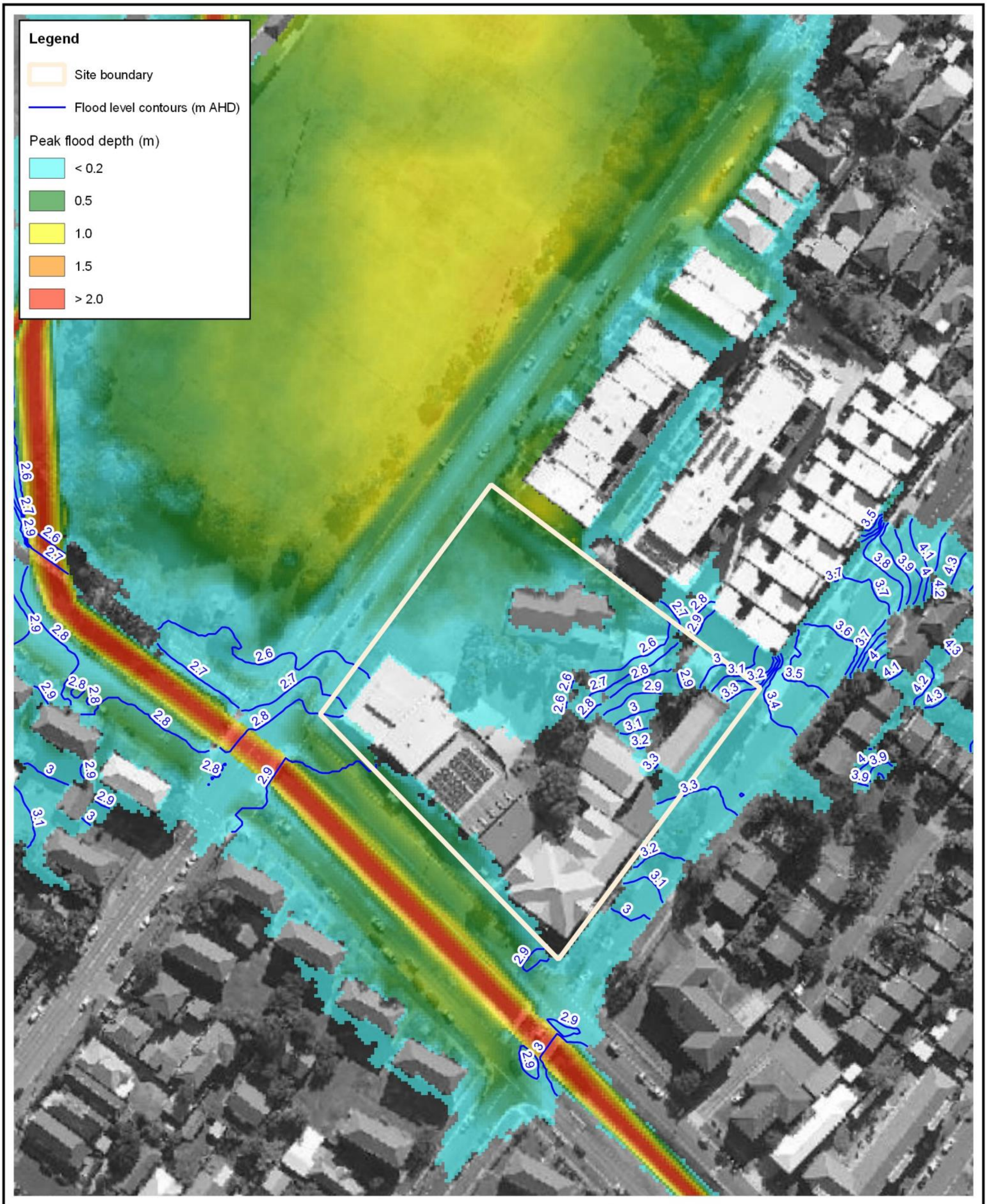

Title:

Modelled 2% AEP Flood Depths and Levels

Figure: **A-4**

Revision: **A**

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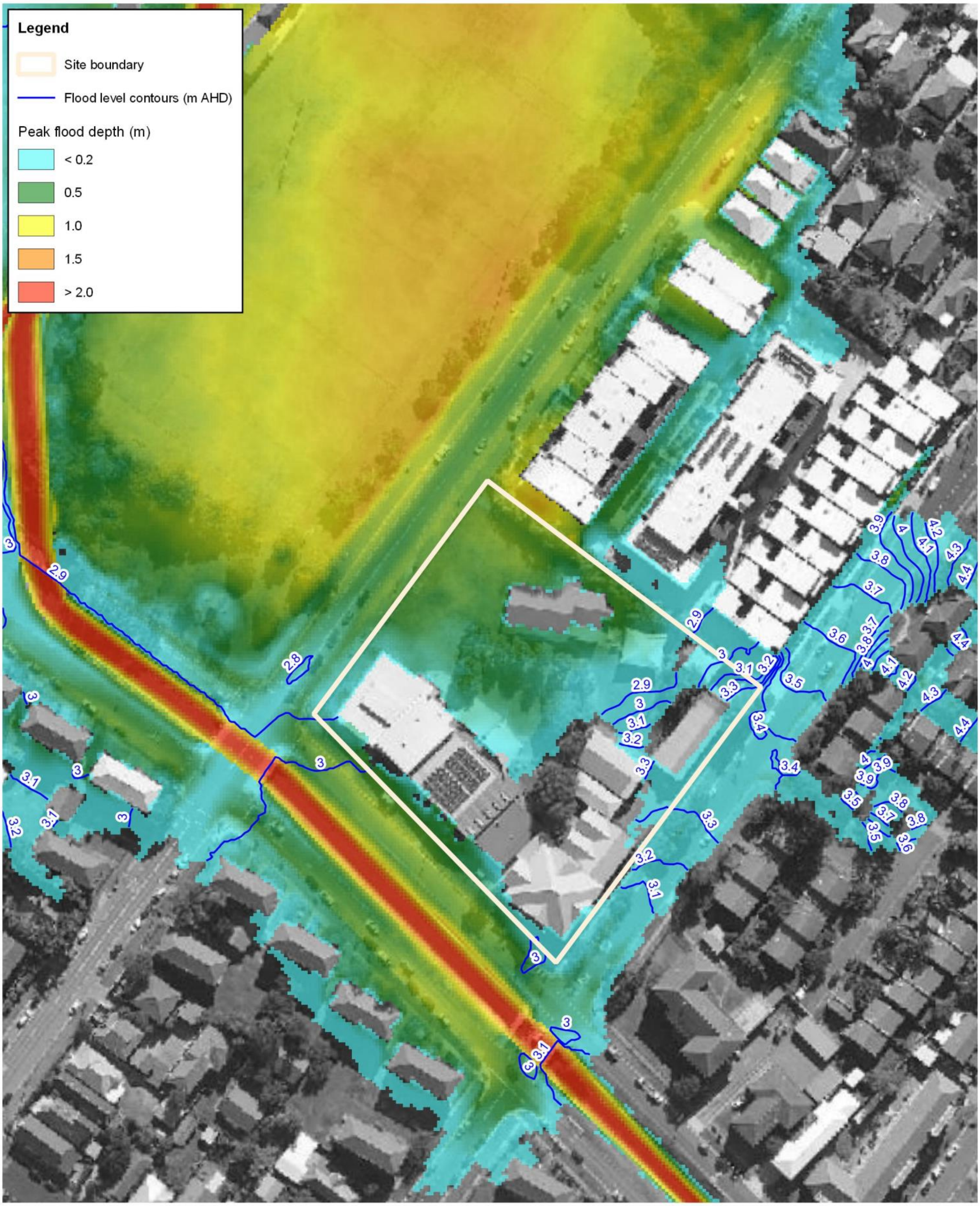

Title:

Modelled 1% AEP Flood Depths and Levels

0 40 80 m

approx. scale

Figure:	A-5
Revision:	A

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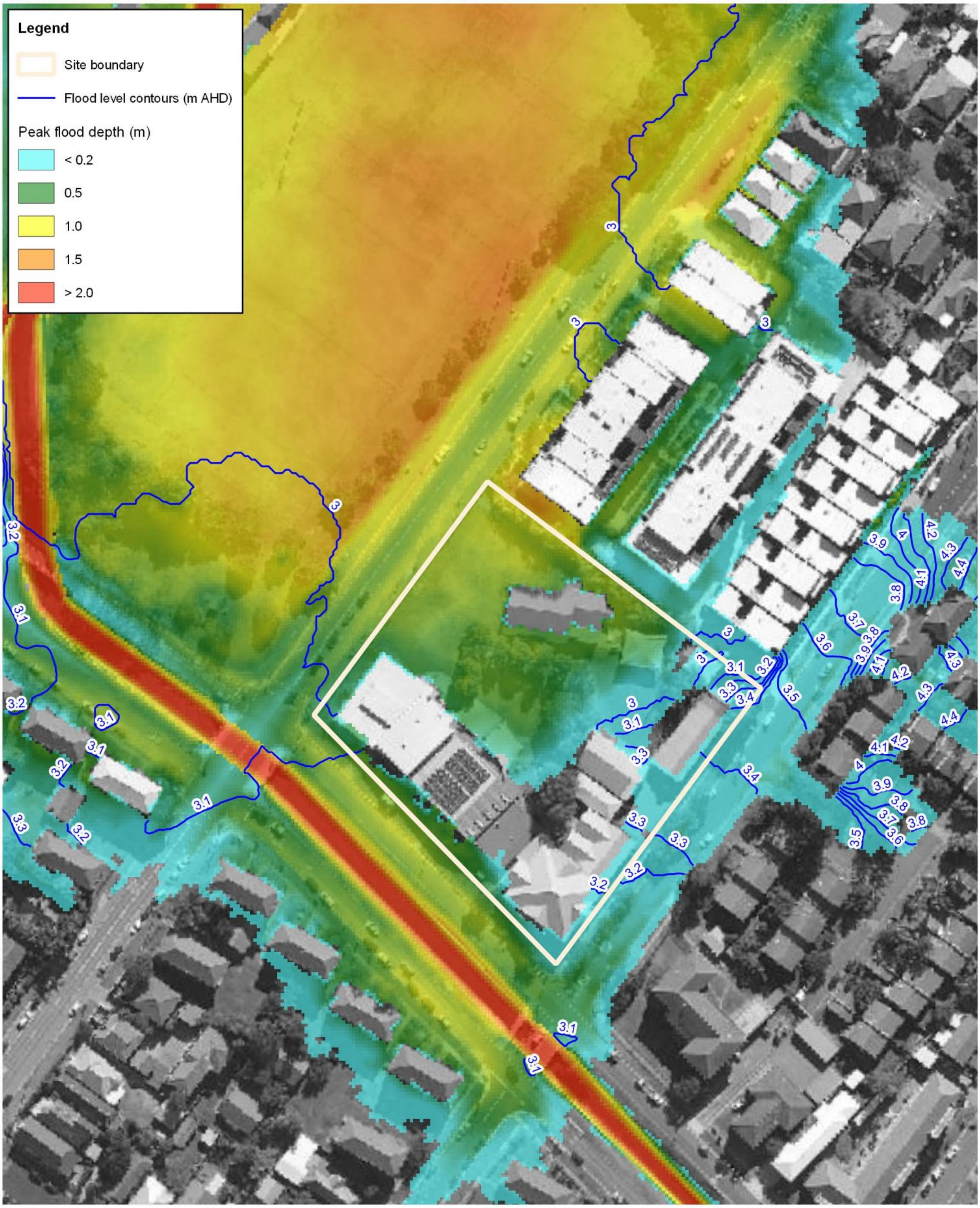

Title:

Modelled 0.5% AEP Flood Depths and Levels

0 40 80 m

approx. scale

Figure:	**A-6**
Revision:	**A**

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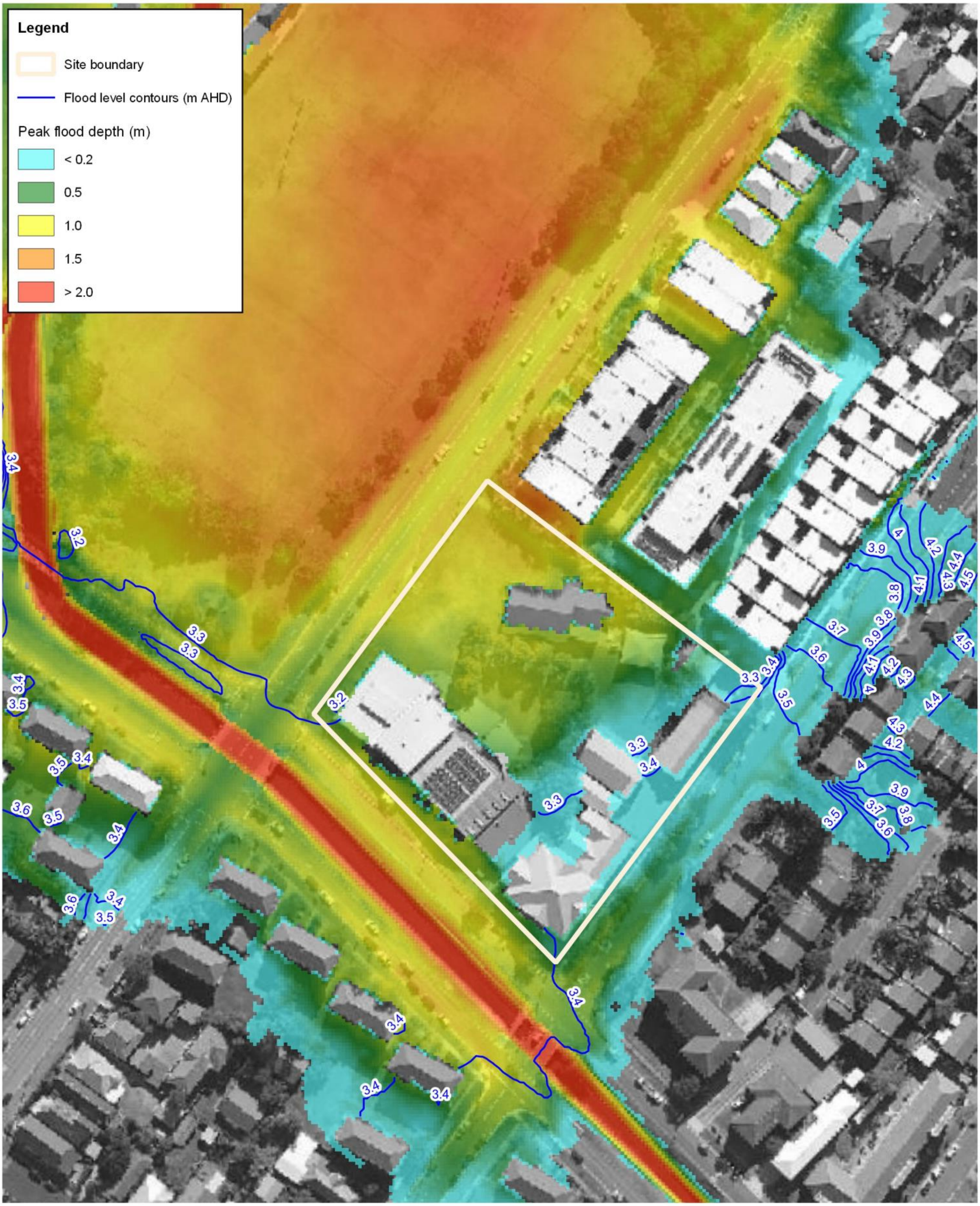

0 40 80 m

approx. scale

N

Title:

Modelled 0.2% AEP Flood Depths and Levels

Figure:	A-7
Revision:	A

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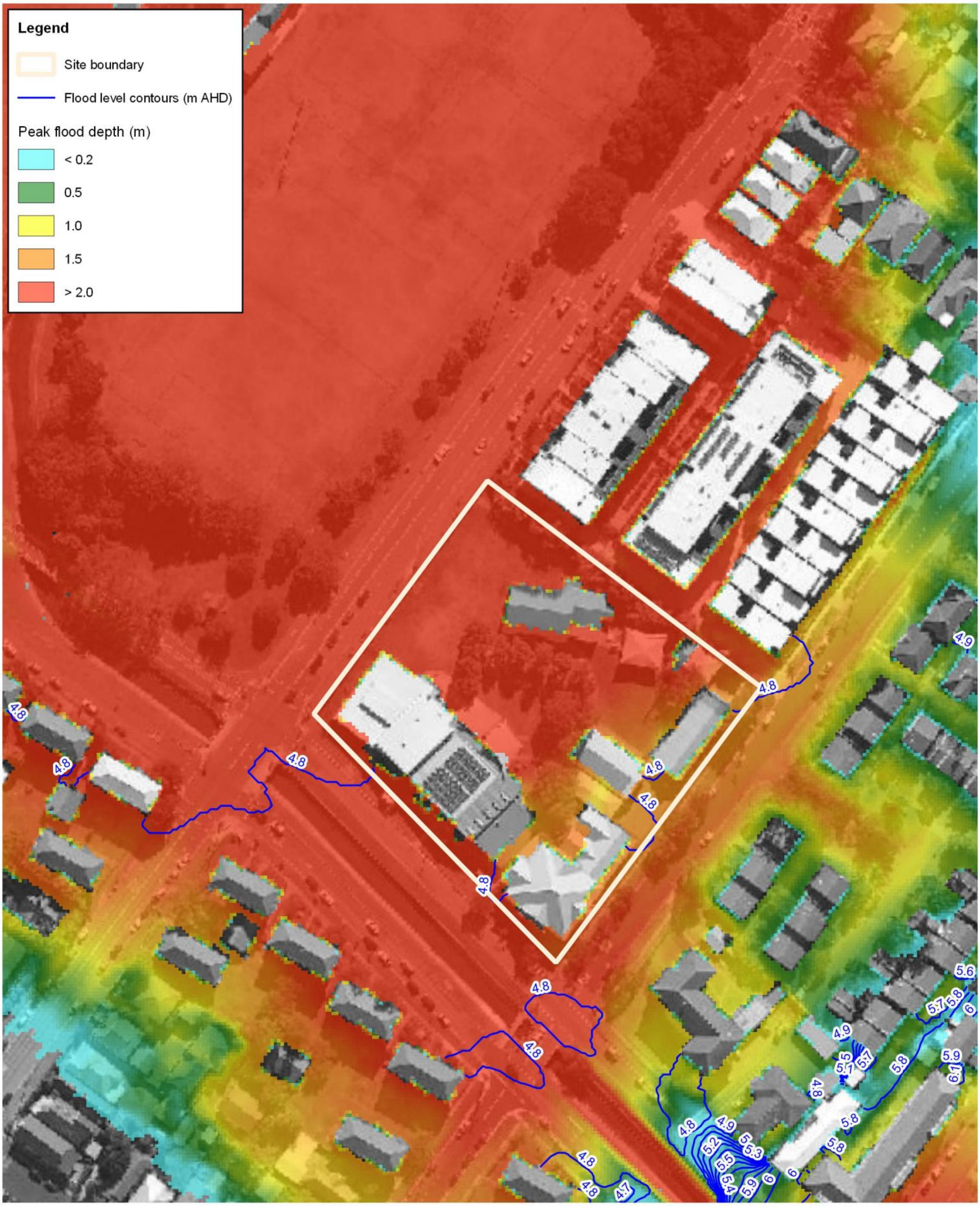

Title:

Modelled PMF Flood Depths and Levels

0 40 80 m

approx. scale

Figure: **A-8**

Revision: **A**

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Appendix B Baseline Design Flood Hazard Mapping

Title:

20% AEP Flood Hazard Classification

0 40 80 m
approx. scale

Figure: **B-1**

Revision: **A**

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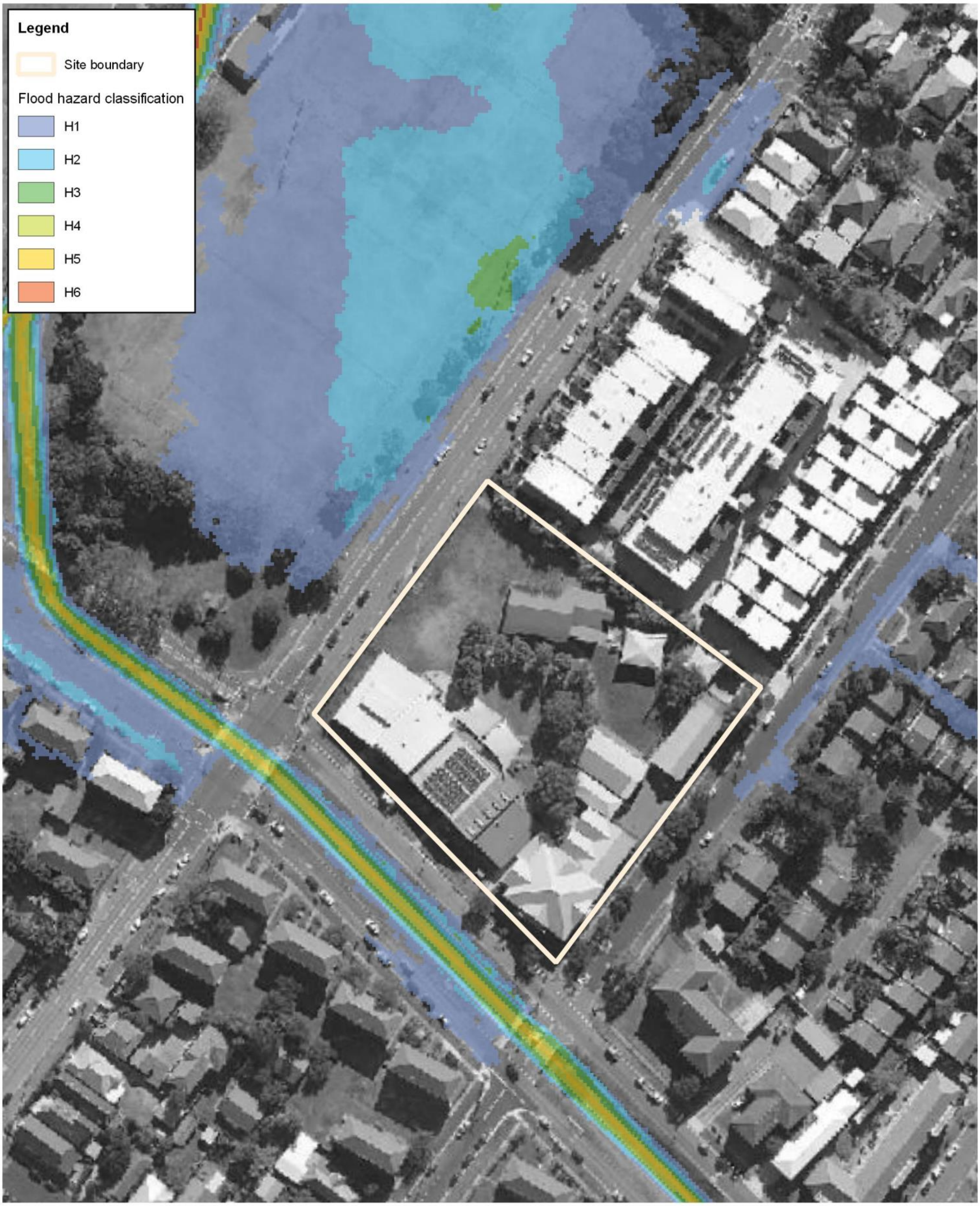

Title:

10% AEP Flood Hazard Classification

0 40 80 m

approx. scale

Figure: **B-2**

Revision: **A**

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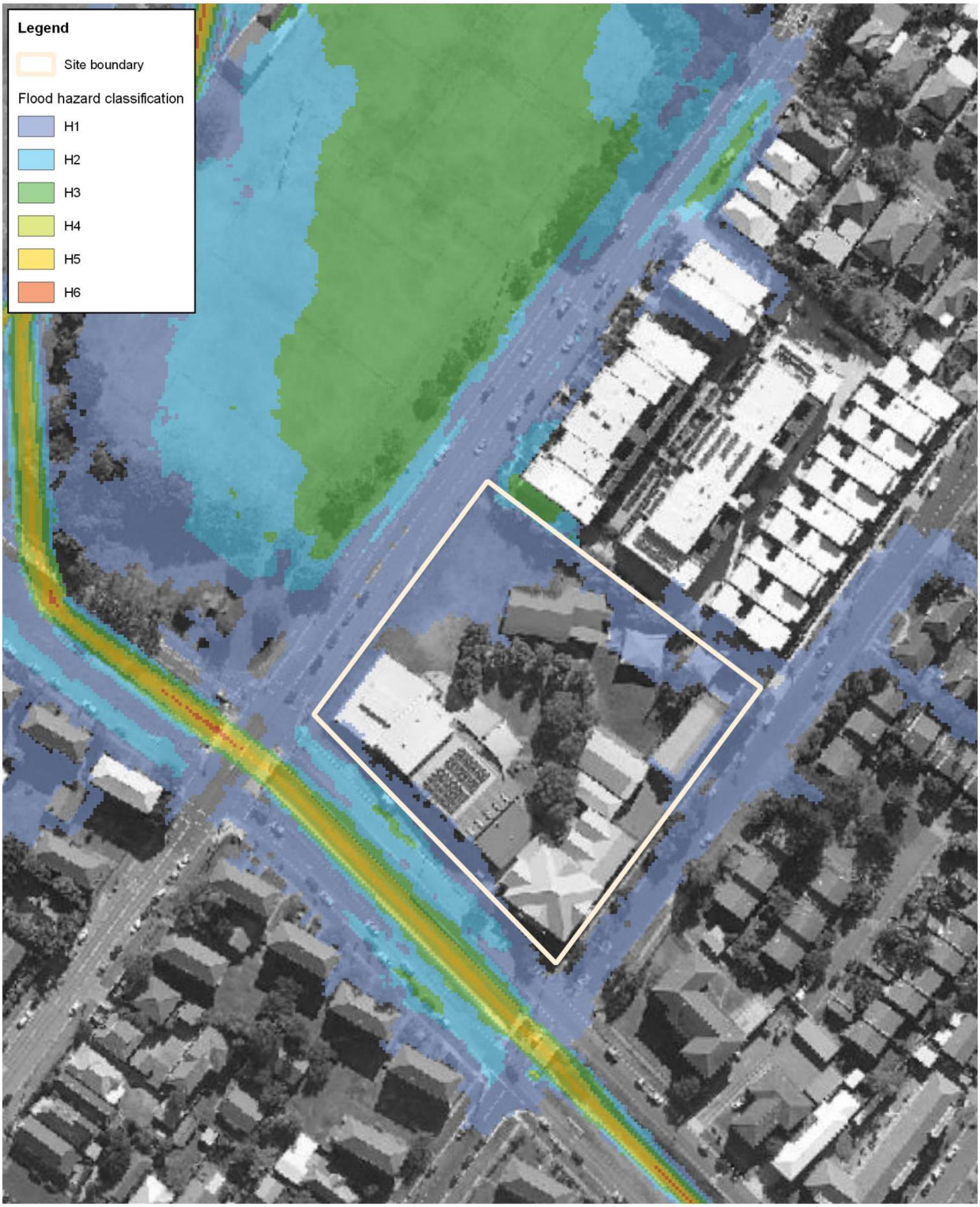

Title:

5% AEP Flood Hazard Classification

0 40 80 m

approx. scale

Figure: **B-3**

Revision: **A**

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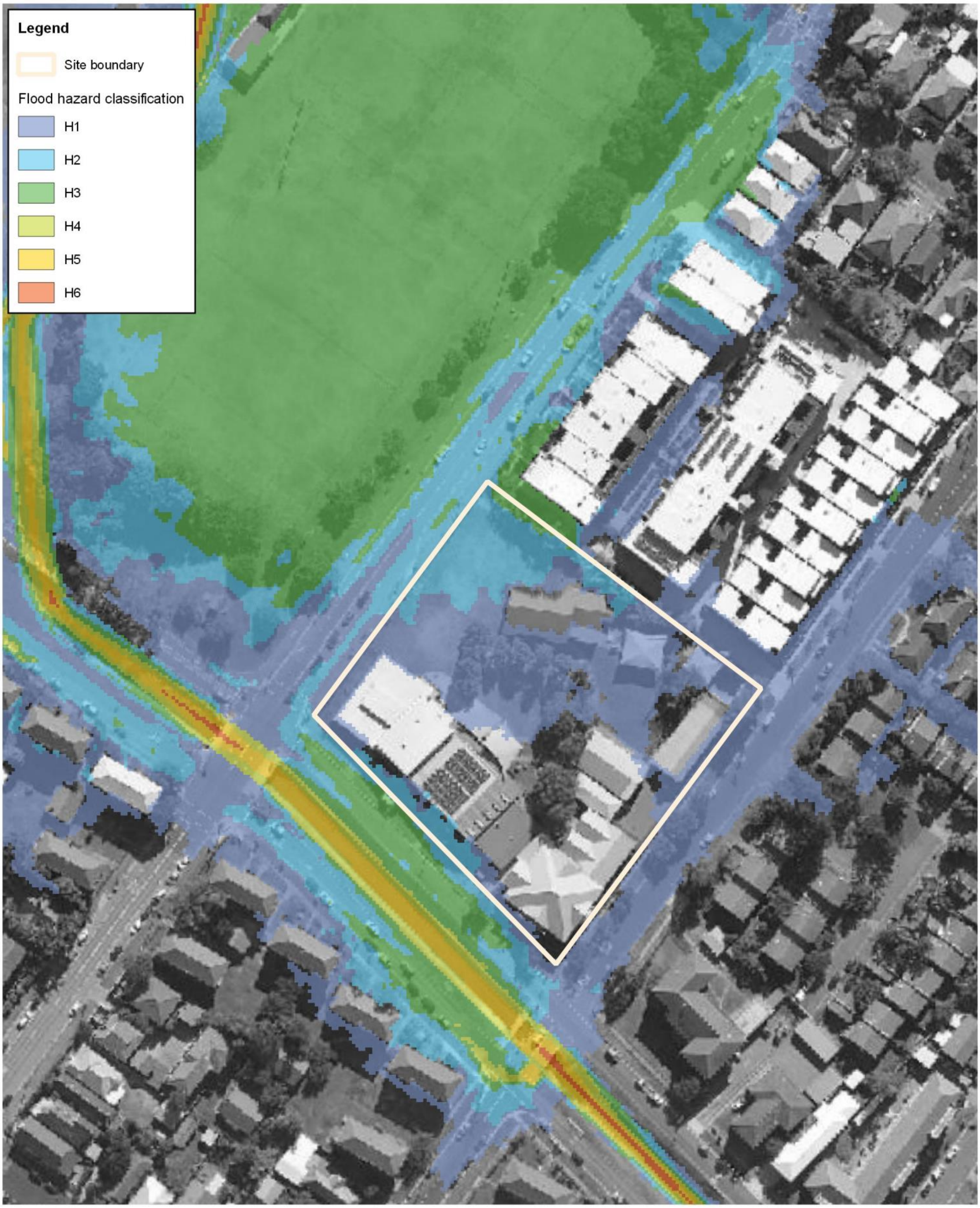

Title:

2% AEP Flood Hazard Classification

Figure: **B-4**

Revision: **A**

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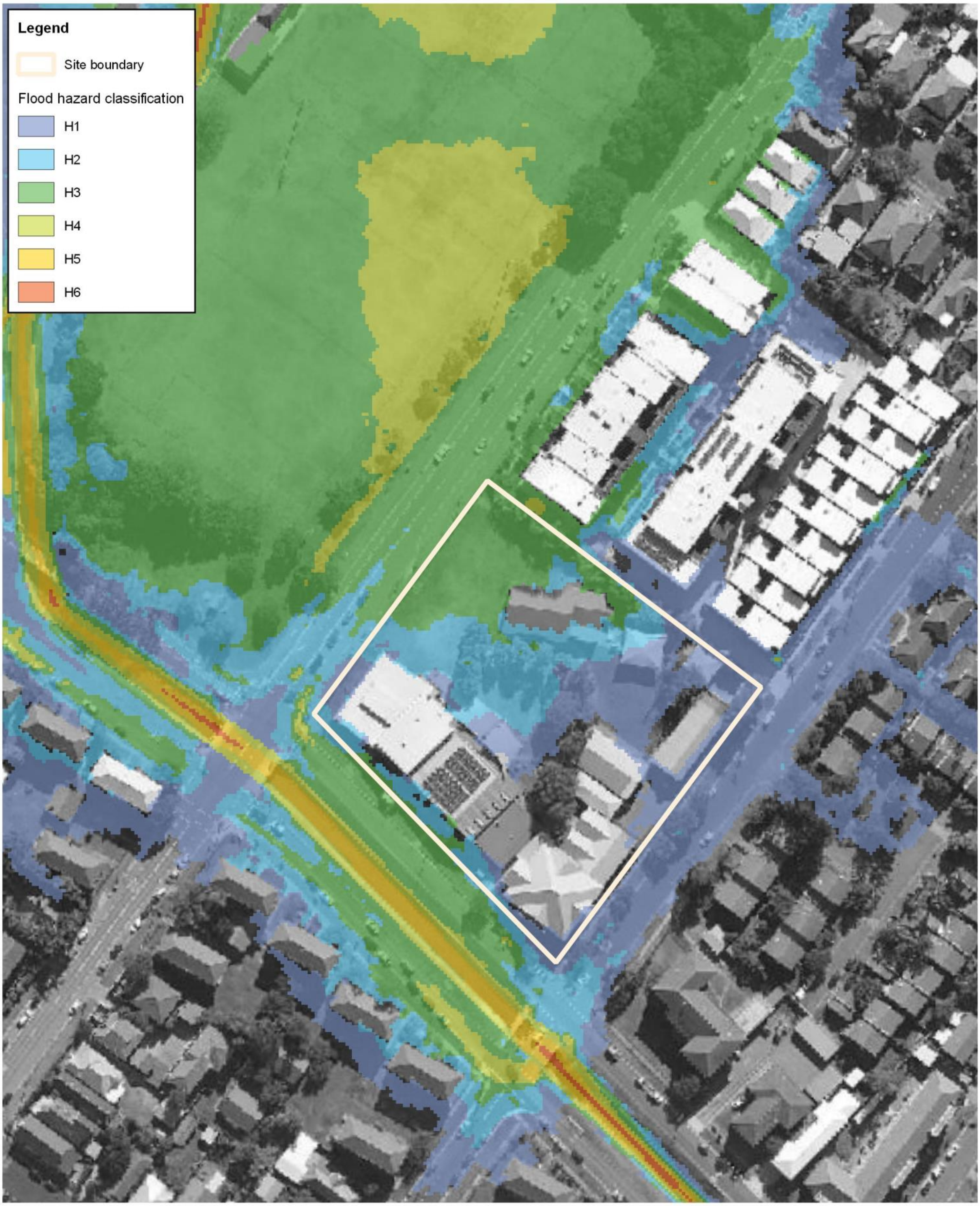

Title:

1% AEP Flood Hazard Classification

0 40 80 m

approx. scale

Figure: **B-5**

Revision: **A**

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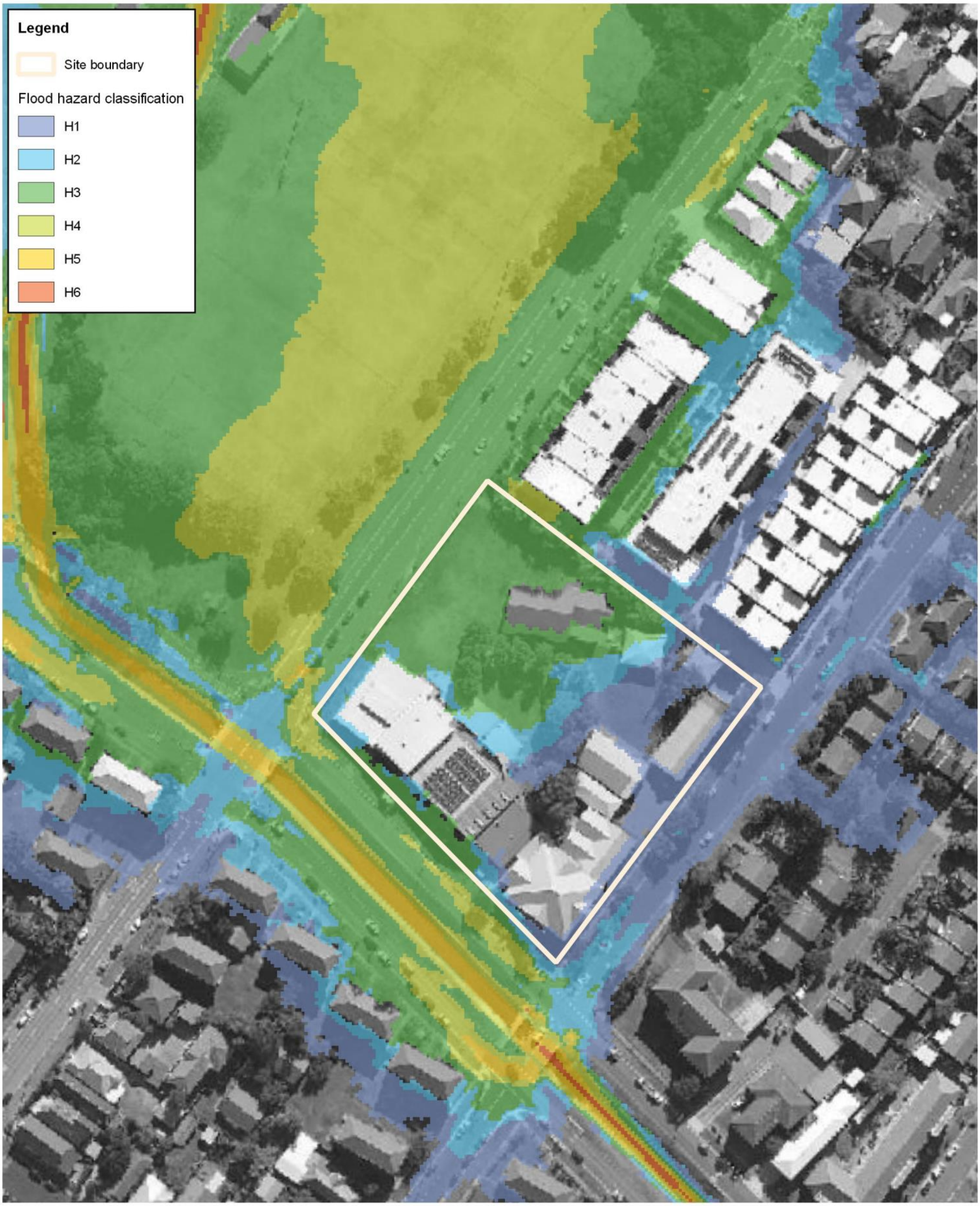

Title:

0.5% AEP Flood Hazard Classification

0 40 80 m

approx. scale

Figure: **B-6**

Revision: **A**

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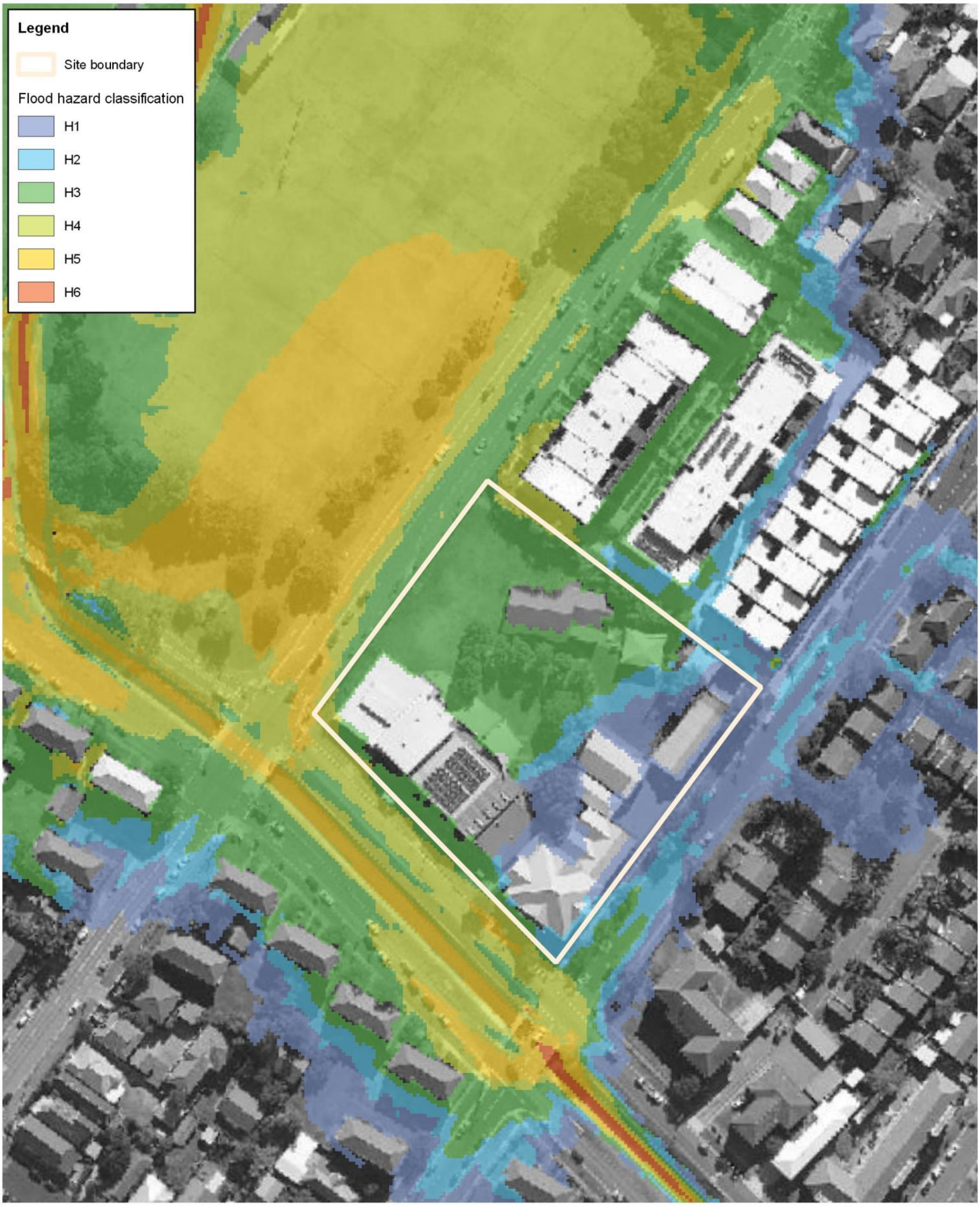

Title:

0.2% AEP Flood Hazard Classification

Figure: **B-7**

Revision: **A**

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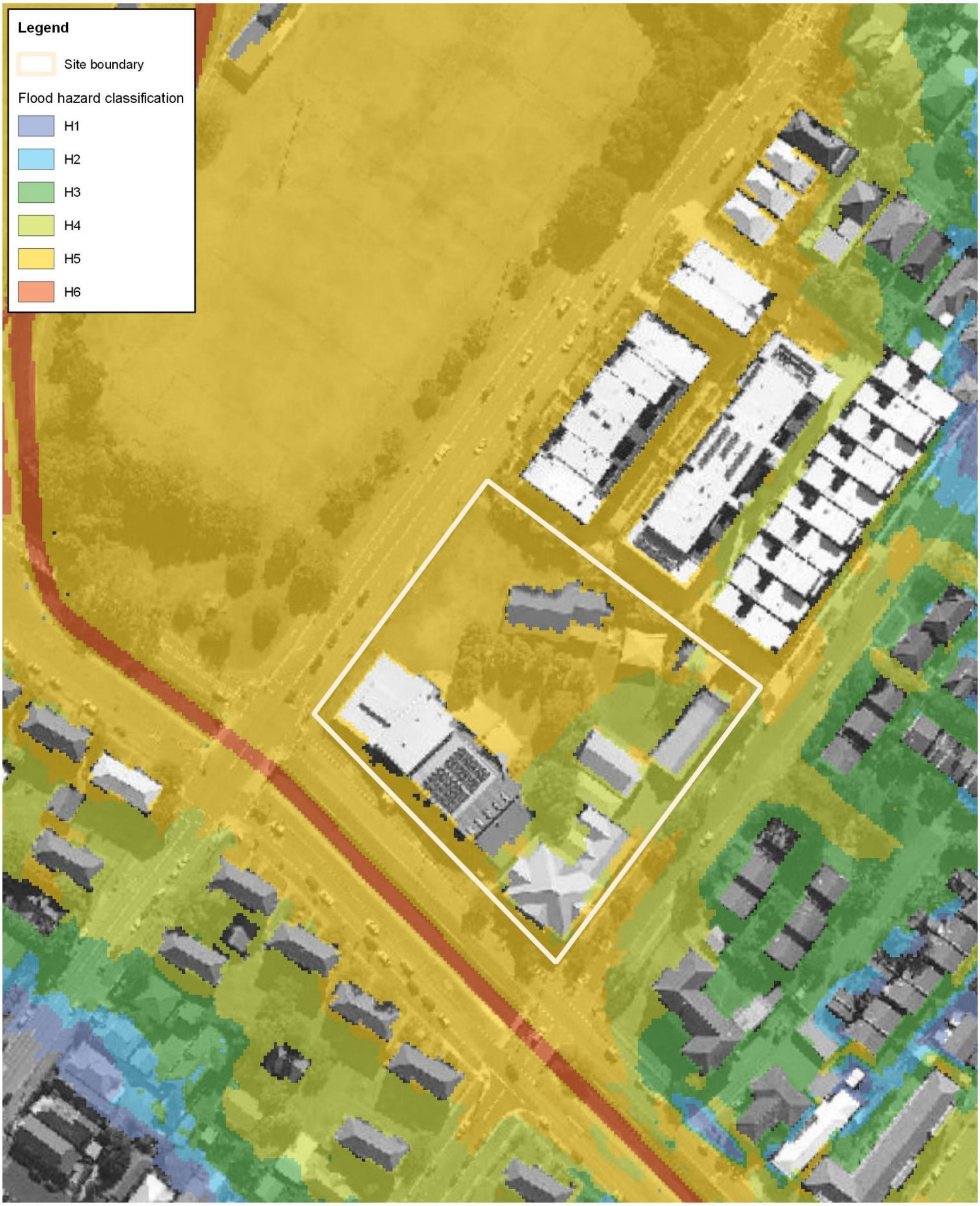

Title:

PMF Flood Hazard Classification

0 40 80 m

approx. scale

Figure:	B-8	*Information shown on this figure is compiled from numerous sources and may not be complete or accurate. Torrent Consulting cannot be held responsible for the misuse or misinterpretation of any information and offers no warranty guarentees or representations of any kind in connection to its accuracy or completeness. Torrent Consulting accepts no liability for any loss, damage or inconvenience caused as a result of reliance on the information.*
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Appendix C Flood Level Impact Mapping

Title:

Modelled 20% AEP Peak Flood Level Impact

0 40 80 m

approx. scale

Figure: **C-1**

Revision: **B**

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Filepath: \Projects\T2030_Newcastle_Grammar_FIA\GIS\T2030_301_210526_5y_h_impact.qgz

Title:

Modelled 10% AEP Peak Flood Level Impact

Figure: **C-2**

Revision: **B**

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Title:

Modelled 5% AEP Peak Flood Level Impact

Figure: **C-3**

Revision: **B**

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Title:

Modelled 2% AEP Peak Flood Level Impact

0 40 80 m

approx. scale

Figure:	**C-4**
Revision:	**B**

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Filepath: \Projects\T2030_Newcastle_Grammar_FIA\GIS\T2030_304_210526_50y_h_impact.qgz

Title:

Modelled 1% AEP Peak Flood Level Impact

Figure: **C-5**

Revision: **B**

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Title:

Modelled 0.5% AEP Peak Flood Level Impact

0 40 80 m

approx. scale

Figure: **C-6**

Revision: **B**

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Title:

Modelled 0.2% AEP Peak Flood Level Impact

0 40 80 m

approx. scale

Figure: **C-7**

Revision: **B**

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Appendix D Flood Velocity Impact Mapping

Title:

Modelled 20% AEP Peak Flood Velocity Impact

0 40 80 m

approx. scale

Figure: **D-1**

Revision: **B**

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Title:

Modelled 10% AEP Peak Flood Velocity Impact

Figure: **D-2**

Revision: **B**

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Title:

Modelled 5% AEP Peak Flood Velocity Impact

0 40 80 m

approx. scale

Figure: **D-3**

Revision: **B**

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Title:

Modelled 2% AEP Peak Flood Velocity Impact

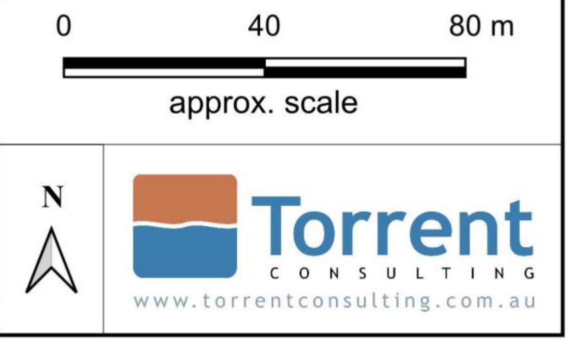

Figure: **D-4**

Revision: **B**

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Title:

Modelled 1% AEP Peak Flood Velocity Impact

Figure: **D-5**

Revision: **B**

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Title:

Modelled 0.5% AEP Peak Flood Velocity Impact

0 40 80 m

approx. scale

Figure:	**D-6**
Revision:	**B**

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Title:

Modelled 0.2% AEP Peak Flood Velocity Impact

Figure: **D-7**

Revision: **B**

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