Water Cycle Management Report

Rosalind Park
Planning Proposal

Prepared for: Leda Holdings Pty Ltd
Location: 11 Leda House, 5 Hunter Street, Sydney, NSW, 2000
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Our Reference: 467-21

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This report has been prepared by Craig & Rhodes Pty Ltd and has undergone a quality assurance review. The signatures below confirm review completion.

1. Introduction

A Water Cycle Management Plan (WCMP) is required to support the Rosalind Park Planning Proposal at 33 Medhurst Road, 101 and 111 Menangle Road, Menangle Park. The proposal includes a residential estate development of between 1300 and 1650 Lots including retail, and community uses over an area of approximately 264 Ha. The Water Cycle Management Plan has been developed in accordance with the Campbelltown City Council Development Requirements and Engineering Design Guide for Development. In addition, particular attention has been paid to incorporating the latest ARR2019 Guidelines for A Guide to Flood Estimation.

1.1 Scope

The scope of works as outlined in the Proposal dated 1st November 2021 consists of the following tasks and deliverables:

1. Prepare a detailed WCMP that assesses whether the Site is subject to flooding as defined in the Campbelltown City Council Engineering Design Guide.
2. Define the planning flood design requirements for development and the anticipated urban built form.
3. Precinct concept drainage design for stormwater quality and quantity measures which will include consideration of minor and major flow management.
4. Prepare the required flood modelling and issue modelling files and mapping results to the Campbelltown City Council for their assessment.
5. Liaise with and attend meetings with Campbelltown City Council and respond to questions during the rezoning assessment phase.

1.2 Objectives

The objective of this report is to outline the Flood Modelling undertaken, the methodology and assumptions and the Stormwater Quality analysis. The report presents the results of the analysis and recommendations.

2. Flood Assessment

2.1 Data Collation and Review

The following data was provided by Leda Holdings and has been reviewed as part of the flooding assessment:

- Menangle Park Planning Proposal, Dahua Group Aust Pty Ltd, APP Corporation Pty Ltd (November 2018)
- Masterplan Water Cycle Management Report – Menangle Park, Prepared by SMEC (November 2018)
- Menangle Park LES, Local Flooding and Stormwater Quantity Management (Detention), (May 2010)
- Menangle Park Hand Draw Subdivision Plan, Design + Planning, (August 2021)
- Menangle Park Stormwater Quantity Management Strategy, Appendix E, GHD, (May 2010)
- LEDME-1-026 Concept Master Plan_Rev F

In addition to the above, the following data was obtained by Craig & Rhodes to facilitate the flood modelling:

- Campbelltown (Sustainable City) Development Control Plan 2009 Volume 2 Engineering Design for Development (June 2009)
- Nepean River Flood Study, Prepared by Worley Parsons for Camden Council (April 2015)
- LiDAR – Wollongong201101-LID1-AHD_2926216 – 2966224_0002_0002_1m. The 1m Digital Elevation Model is produced using Triangular Irregular Network (TIN) methods, averaging ground heights to formulate a regular grid. The data has a vertical accuracy of 0.3m and horizontal accuracy of 0.8m.
- Field Survey was undertaken by Craig & Rhodes for the existing drainage structures present within the site as well as cross sections of the tributaries that traverse the site to allow greater definition of the channels in the flood model.

The data outlined above was reviewed for data gaps only, a quality assessment was not undertaken, and it assumed that all provided data is suitable for use.

2.1.1 Data Gaps

The following data remains outstanding for the Rosalind Park Planning Proposal:

- Tailwater Conditions at Nepean River – the hydrological models or hydrographs for the section of the Nepean River upstream of Hume Motorway
- Detailed feature and levels survey of the site will be required for detailed design

2.2 Study Area

Rosalind Park consists of approximately 264 Ha of predominantly rural land. The site is bordered by Menangle Creek to the east and south and several tributaries of Menangle Creek traverse the site from north to south-east and north to south. From aerial imagery and site photography (undertaken by the Craig and Rhodes surveyors), the creeks appear to be heavily vegetated. Surveyed cross sections have been obtained for the tributaries to further define the channels. The creeks are unnamed and for the purposes of communication within this report a naming convention has been adopted. The naming convention adopted is illustrated within Figure 2-1.

At the downstream south-west corner of the site, Menangle Creek confluences with Woodhouse Creek and together these both outfall to the Nepean River only 700 metres downstream of the confluence and only 120 metres downstream of the site boundary.

To the south-east of the site is the Mount Gilead Residential Estate, and to the west the site is bounded by the Hume Motorway. A similar residential estate is currently being developed to the west of the Hume Motorway – Menangle Park Estate.

Rosalind Park includes a sandstone quarry located in the southern end of the site that is over 60m deep, although, due to raised bunds around most of the quarry, it is not a significant consideration for flooding.

An image of the study area is provided in Figure 2-1.

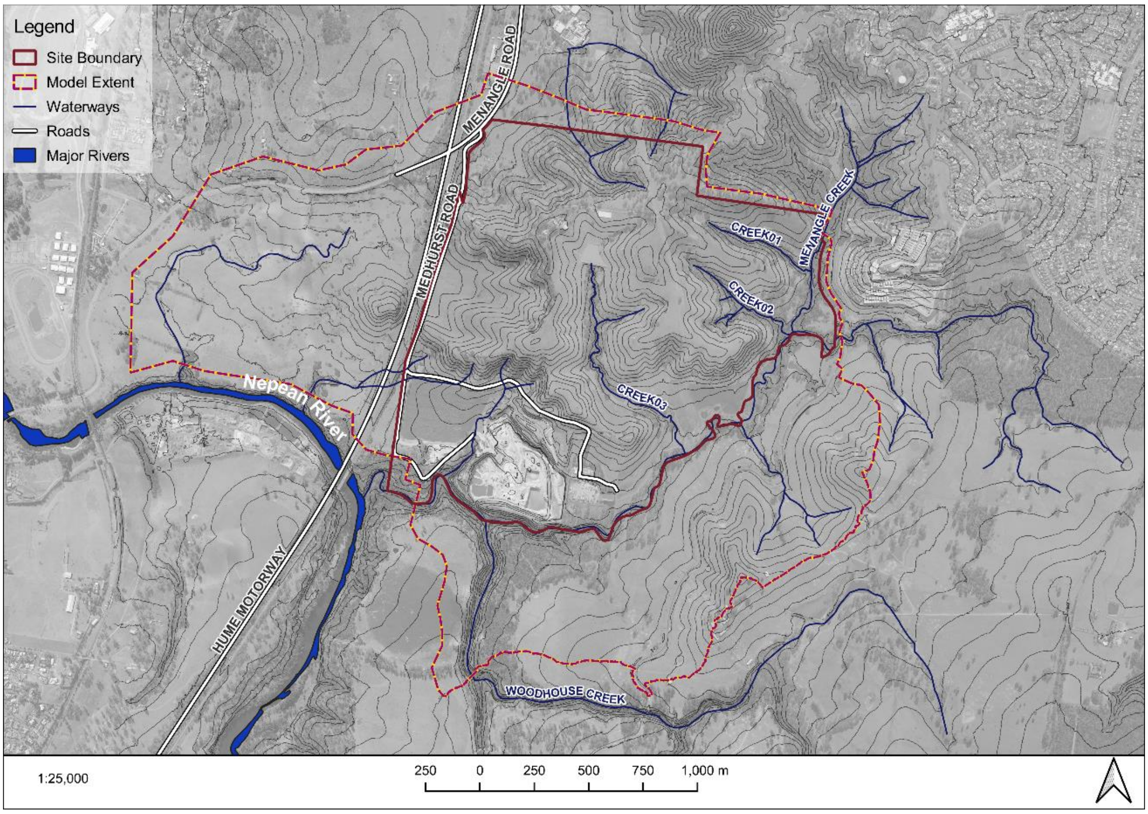

Figure 2-1 Study Area

2.3 Hydrologic Model Development

A new hydrologic model was developed for the Menangle Creek and Woodhouse Creek catchments from the outlet at the Nepean River to the upper reaches of the catchments. In addition, several smaller catchments that flow west under the Hume Motorway were modelled as they traverse the site.

The modelling approach followed the latest NSW jurisdictional advice provided on the ARR2016 Data Hub, informed by a study undertaken for the NSW Office of Environment and Heritage by WMA Water (Podger, 2019).

The hydrologic model was developed in the industry standard software, RORB. RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall-excess and routes this through catchment storage to produce runoff hydrographs at any location (Laurenson, E. et.al., 2010).

RORB is an ideal choice to model the Rosalind Park Proposal as it can also be used to design retarding basins quickly and efficiently if required.

2.3.1 RORB Model

RORB estimates the flood hydrographs based on a catchment and sub catchment delineation where the rainfall excess from a sub area is assumed to enter the channel network at a point on the main-stream and provides for flow entering the channel system to be routed through a series of storages to the catchment outlet, this accounts for the attenuation effects within a reach. The calculation of the attenuation effects requires the routing parameters k_c and m in the equation:

$$S=3600kQ^m$$

Equation 2-1 RORB Storage-Discharge Relation

where S is the storage in (m^3), m is a dimensionless exponent and k is a dimensional empirical coefficient that is dependent upon k_c (the main user entered parameter) and the relative delay time.

The channel attenuation is dependent upon the relative delay time within the reach and this is calculated by input of the reach lengths and slopes, and the reach slope calculation depends upon the Reach Type adopted by the modeler. The RORB Reach Types available are as follows:

1. Natural
2. Excavated but unlined
3. Lined or piped
4. Drowned (for example by a reservoir or lake) (Laurenson, E. et.al., 2010)

The final parameters chosen for the Medhurst RORB hydrologic modelling are outlined in Sections 2.3.2 to 2.3.4 below.

2.3.2 Catchment Delineation

Delineation of the Menangle Creek and Woodhouse Creek catchment boundaries was undertaken utilising a QGIS plugin, GRASS v2.12. Using the Wollongong (2011) LiDAR discussed in 2.1, (DEM), GRASS automated the delineation of the catchment boundary and streams for the catchment. The catchments were further refined in GRASS to create sub catchments, reaches and nodes for input into the RORB modelling software. A schematic of the catchment delineation is provided in Appendix A.

2.3.3 Model Parameters

To undertake the hydrological modelling, parameters are required for the imperviousness of the catchments, the reach types (as discussed in 2.3.1), the routing parameters and the losses. The parameters adopted for the hydrologic modelling are outlined in Table 2-1.

Table 2-1 RORB Parameters adopted

Element	Parameter Adopted	Justification
Catchment Impervious Values	See Appendix B	Aerially Weighted Values based on the Campbelltown DCP Table 4.2 and other values as per industry standard.
Reach Types	The majority of reaches throughout the catchment were natural Reaches. Where reach slopes are greater than 3%,	

	Reach Type 2 was adopted. Reach Type 3 was adopted for flow over roads.[1]	
Routing Parameters	k_c = 4.43	ARR2016 recommends for NSW catchments east and west of the Great Dividing Range k_{c-}= 1.18$A^{0.46}$ (Ladson, n.d.)
	m = 0.8	Recommended by RORB manual
Losses	Initial Loss	45mm x probability neutral burst initial loss multipliers for each duration as per the NSW jurisdiction advice, obtained from the Data Hub.
	Continuing Loss = 0.96	2.4 mm/hour x 0.4 as recommended by the NSW jurisdiction specific requirements, obtained from the Data Hub.

The only change to the RORB model between existing and developed conditions, at this stage is the fraction impervious used. As the design develops, some reach types and directions may need to be updated where there is flow over roads or piped flow.

The RORB schematic and further parameters are outlined in Appendix A and Appendix B.

2.3.4 Hydrological Model Results

*The hydrologic modelling was analysed to determine the "1-up" median temporal pattern and the maximum (of the medians) duration. This analysis determines the critical duration of the catchment, and the resultant hydrographs will be utilised in the TUFLOW hydraulic modelling. This process is outlined in **Error! Reference source not found.**.*

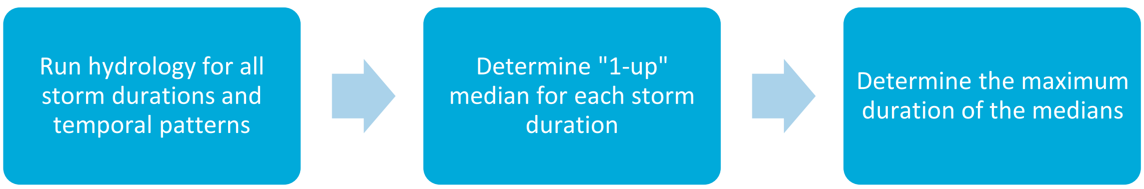

Figure 2-2 - Hydrology results analysis methodology

[1] Reach Type 1 is a Natural Reach, which assumes there is storage available and does not consider the reach slope. Reach Type 2 considers the slope and for steeper slope reduces the storage available within the reach. Reach Type 3 is for lined or piped and can be used for overland flow over roads.

EXISTING CONDITIONS

The results of the existing conditions hydrologic modelling at key locations throughout the catchment are outlined in Table 2-2.

Table 2-2 Existing Conditions RORB results

Location	Flow (m^3/s)	Duration	Median Temporal Pattern
Inflow to northeast corner of site from upstream catchments	14.5	45 mins	TP26
Inflow to TUFLOW from Woodhouse Creek Catchments	59.4	6 Hour	TP24
Inflow to TUFLOW from tributaries east of site	15.3	2 Hour	TP24
Outlet at Nepean River	92.7	6 Hour	TP21
The catchment on the west of the site that outfalls to the Hume Motorway	16.9	25 minutes	TP23

DEVELOPED CONDITIONS

The results of the developed conditions hydrologic modelling at key locations throughout the catchment are outlined in Table 2-3.

Table 2-3 Developed conditions RORB results

Location	Flow (m^3/s)	Duration	Median Temporal Pattern
Inflow to northeast corner of site from upstream catchments	13.8	45 minutes	TP27
Inflow to TUFLOW from Woodhouse Creek Catchments	59.4	6 Hours	TP24
Inflow to TUFLOW from tributaries east of site	15.3	2 Hours	TP24
Outlet at Nepean River	92.9	12 Hour	TP30
The catchment on the west of the site that outfalls to the Hume Motorway	17.0	25 minutes	TP29

2.4 Hydraulic Model Development

A new hydraulic model was built in the industry standard software, TUFLOW. TUFLOW is a suite of urban drainage, catchment flooding and coastal simulation software. It can model both 1D and 2D environments for underground drainage and surface water modelling respectively. In the case of the Medhurst model, the TUFLOW HPC (Heavily Parallelised Compute) explicit solver was used so that the latest terrain integrations could be incorporated into the model and for simulation efficiency.

Development of the hydraulic model was informed by aerial imagery, cadastral boundaries, survey and LiDAR. The existing conditions model is built within 1D and 2D utilising the inflow hydrographs from RORB. The purpose of the model is to understand the catchment conditions and existing flood regime in order to understand the high-level impacts of the proposed development, if any.

2.4.1 Digital Elevation Model

The Digital Elevation Model (DEM) was developed from the freely available Wollongong 2011 LiDAR in conjunction with cross sectional survey of the open channels. The LiDAR is produced using a Triangular Irregular Network (TIN) methodology and has a vertical accuracy of 0.3m and a horizontal accuracy of 0.8m. The LiDAR was available in GDA94, and this project is being undertaken in GDA2020, so the LiDAR was reprojected into GDA2020 MGA Zone 56. The LiDAR was then processed to "pit-fill" the terrain. This process removes terrain artefacts and the processing tool used takes a conservative approach so that actual depressions in the terrain remain untouched. The terrain was "pit-filled" because a rainfall excess approach was adopted for the modelling, and it was important to ensure that water does not pool in the model where it should be flowing. The DEM was further refined utilising cross-sectional survey of the channels.

The DEM is illustrated in Figure 2-3.

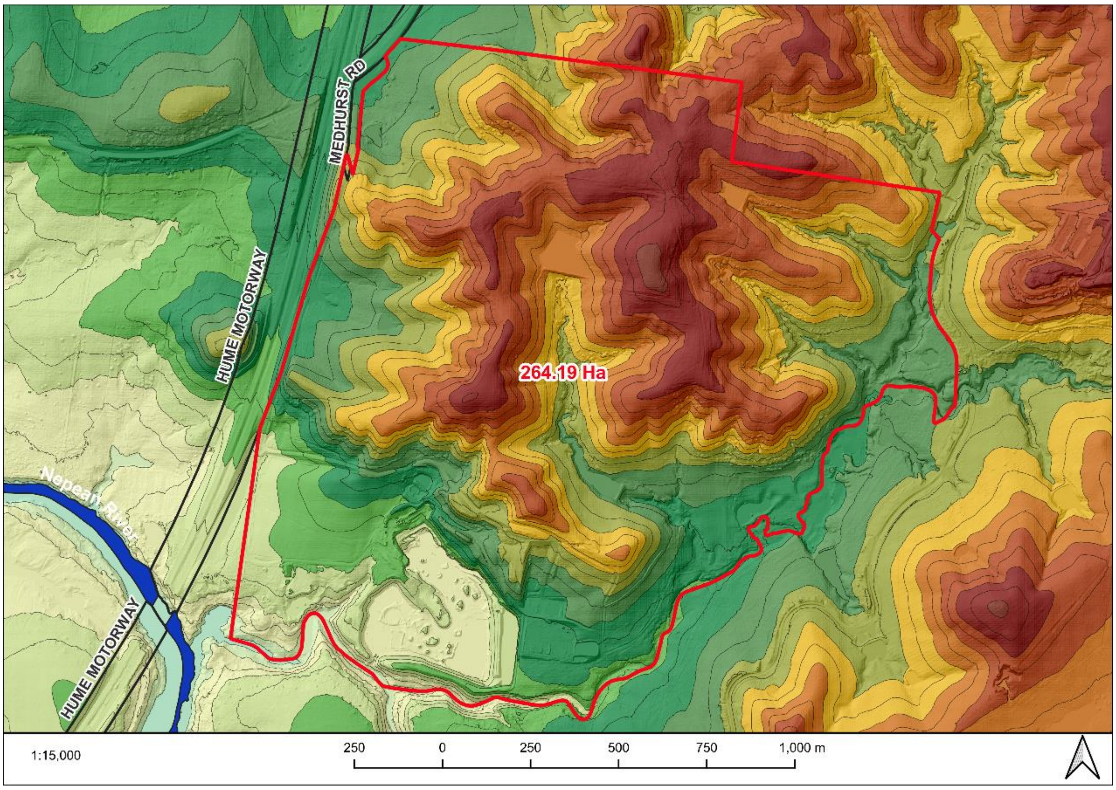

Figure 2-3 Digital Elevation Model

2.4.2 Boundary Conditions

Outflow Boundaries

The Medhurst TUFLOW model used automatically generated head-flow (HQ) boundary conditions based on the resultant water surface elevation gradient, as per the TUFLOW Manual.

Inflow Boundaries

The inflow to the Medhurst model adopted a combination of inflow hydrographs and a rainfall excess methodology. The inflow hydrographs captured the total flows from the upstream Menangle Creek and Woodhouse Creek catchments, whilst the rainfall excess hyetographs represented the rainfall that falls directly on the development.

An image of the model extents and boundary conditions is provided in Figure 2-4.

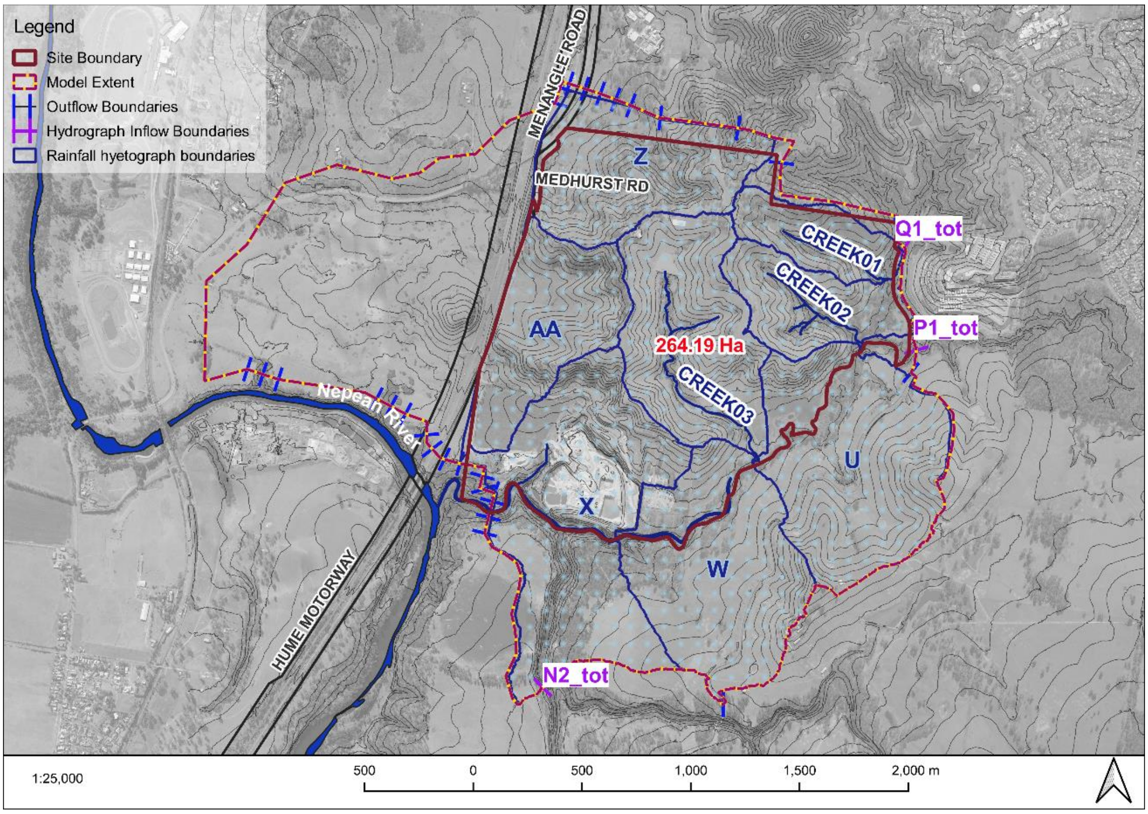

Figure 2-4 Model extents and boundary conditions

STORM EVENTS

At this stage of the analysis the 1% AEP flood event has been modelled and the PMF and lower AEP events, including the 20%, 10% and 5% AEP will be undertaken for the Development Application.

The storm events adopted for the hydraulic modelling was determined as part of the hydrologic modelling as discussed in Section 2.3.4. The storm events extracted from RORB and modelled in TUFLOW are outlined in Table 2-4.

Table 2-4 Storm Events Modelled

Storm Event	Duration	Temporal Patterns
1% AEP	25 minute	TP03
	45 minute	TP06
	90 minute	TP07
	120 minute	TP04
	360 minute	TP04
	720 minute	TP02

2.4.3 Manning's Roughness

Manning's 'n' values or Manning's roughness values were selected according to the latest ARR2019 Guidelines, Table 6.2.2 *Valid Manning 'n' Ranges for Different Land Use Types (Lambert et. al., 2019).* In some cases, variable roughness values were used due to the rainfall excess approach adopted. The Manning's 'n' values used within the Medhurst model are outlined in Table 2-5 and illustrated in Figure 2-5.

Table 2-5 Manning's 'n' Roughness Values

Material ID	Manning's n	Description Materials values consistent with ARR2019, Book 6, Chapter 2
3	0.3	RESIDENTIAL: URBAN (HIGHER DENSITY)
4	0.15	RESIDENTIAL: RURAL (LOWER DENSITY)
7	0.03, 0.02, 0.1, 3.00**	RESIDENTIAL: RURAL (LOWER DENSITY) Building footprints
9	0.5	INDUSTRIAL/COMMERCIAL (Or large/significant buildings on site)
11	0.03	OPEN PERVIOUS AREAS Minimal vegetation (grassed)
12	0.06	OPEN PERVIOUS AREAS Moderate vegetation (shrubs)
13	0.1	OPEN PERVIOUS AREAS Heavy vegetation
16	0.08	VEGETATED WATERWAY / CHANNEL
17	0.03, 0.08, 0.1, 0.02	CAR PARK / PAVEMENT / DRIVEWAY / ROAD
19	0.018	CONCRETE LINED CHANNELS
20	0.025	WATERBODIES / LAKES – No emergent vegetation
21	0.065	WETLANDS with emergent vegetation
22	0.045	OVERARCHING MODEL n VALUE Open space pervious areas, generally coarsely grassed, some vegetation
23	0.2	FORRESTED – Very heavy vegetation
24	0.05, 0.1, 0.1, 0.025	Limestone Quarry From aerial photography, quarry is significantly dug out, appears to be limestone and has several water-filled pits
25	0.03, 0.1, 0.1, 0.035	Limestone Quarry - Quarry surrounding roads and dirt

** Note that where four values are stated, TUFLOW is reading two pairs of depth values as y_1, n_1, y_2, n_2. For example, in Material ID 7, for depths <= 0.03m, TUFLOW is assigning an n value of 0.02 and when the depth is >= 0.1m, TUFLOW is assigning an n value of 3.00. In between the depths of 0.03m and 0.1m, TUFLOW linearly interpolates the n value. This approach is often adopted for rain-on-grid or rainfall excess models to account for shallow sheet flows.

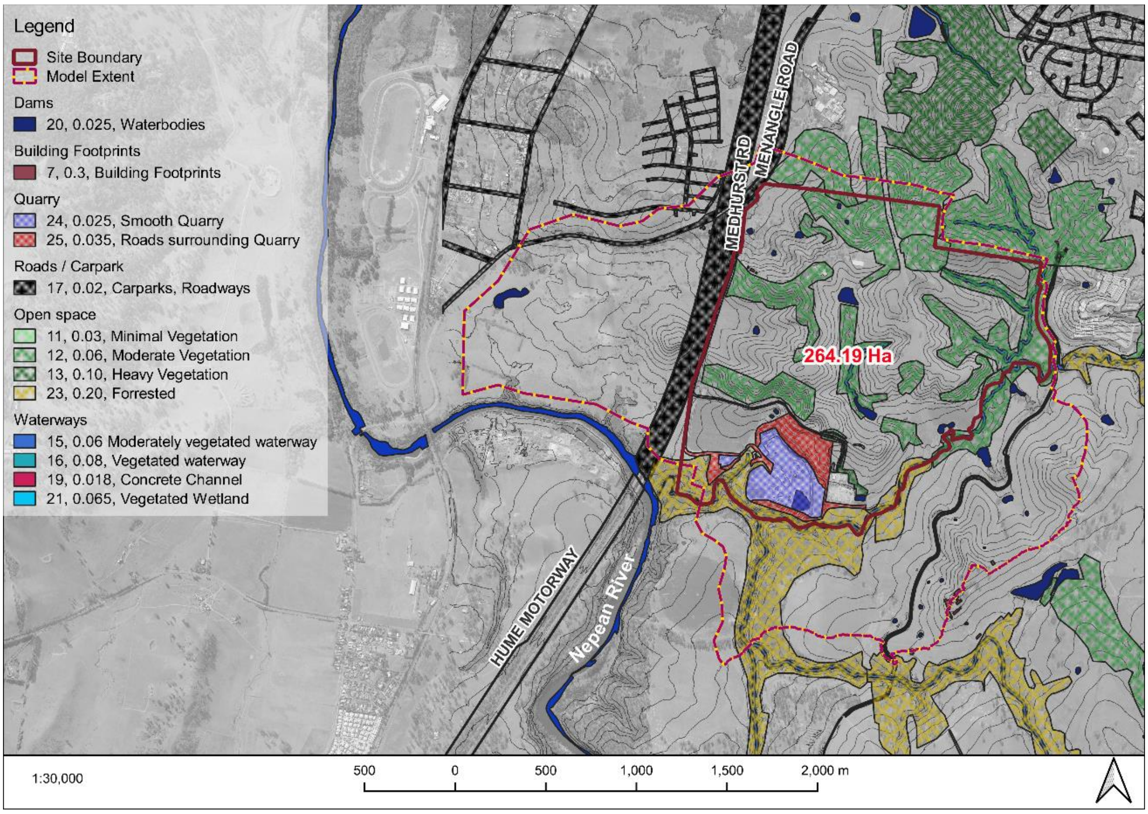

Figure 2-5 Manning's 'n' Roughness Map

2.4.4 Dams

The catchment has several farm dams and reservoirs scattered throughout. The modelling has taken a conservative approach and assumed that the dams are full at the beginning of the simulation. This approach assumes that the dams do not play a role in flood detention or storage and that most flood waters travel downstream to the site.

2.4.5 Structures

A site survey was undertaken to determine the size, shape and number of culverts and/or bridges present within the vicinity of the subject site.

The structures included within the model are detailed in Table 2-6.

Table 2-6 Existing Conditions Structures

ID	Type	Size	Number
MENRD01_CU	C	1.5 m diameter	2
MEDRD03_CU	C	0.9m diameter	1
QUARRY01_CU	C	0.45m diameter	1
MEDRD02_PI	C	1.5m diameter	1
MEDRD03_PI	C	1.5m diameter	1
HUME01_PI	C	0.9m diameter	1
HUME02_PI	C	0.9m diameter	1

2.4.6 Developed Conditions Model

The TUFLOW hydraulic model was updated to include proposed development changes as follows:

1. A design tin was included to model the subdivision grading;
2. The dams were removed at the locations of the design tin;
3. The impervious percentages were updated in the hydrology and the hydrology re-run to produce developed runoff hydrographs;
4. The Mannings 'n' roughness layer was updated to incorporate the subdivision; and
5. A culvert was added at the road crossing within the Riparian Corridor.

The additional structures included for Developed conditions at the Riparian Corridor are outlined in Table 2-7.

Table 2-7 - Developed conditions structures

ID	Type	Size	Number
CK03_DES01	R	3.0mW x 2.4m H	4

2.5 Calibration / Validation

The Menangle Creek and Woodhouse Creek catchments are both ungauged catchments and no calibration data is currently available. In addition, data is not available for the downstream Nepean River where Menangle Creek outfalls to the Nepean River. As such, design flows have been adopted for this study.

The resultant flows generated by the hydrologic modelling was validated against the ARR2019 Regional Flood Frequency Estimation Model (RFFE) and the NSW rural rational method as outlined in the Campbelltown DCP and the validation results are outlined below.

Due to a lack of flood level data, the hydraulic model will not be calibrated/validated.

2.5.1 Regional Flood Frequency Estimation Model

Regional Flood Frequency Estimation (RFFE) is a data-driven approach, which attempts to transfer flood characteristics from a group of gauged catchments to ungauged locations of interest (where design floods need to be estimated). In this case, the design flood needs to be estimated at the outlet of the Menangle / Woodhouse Creek catchments for the purposes of validation against the RORB model. Figure 2-6, below, obtained from the indicates that the 1% AEP runoff from the catchment, which is 17.73km^2 would be approximately 223m^3/s, with confidence limits between 81.8m^3/s and 615m^3/s.

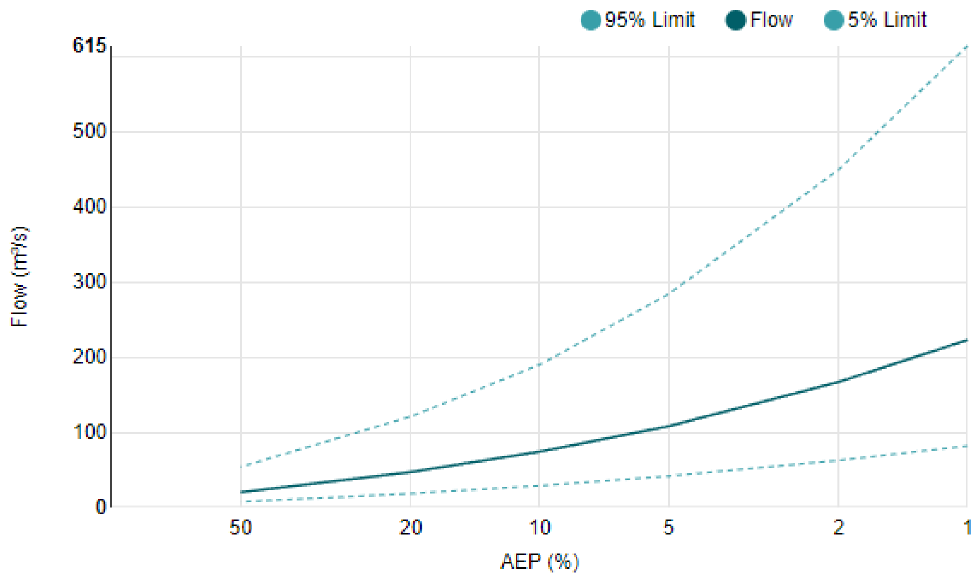

Figure 2-6 RFFE Flow (m^3/s) for the Menangle / Woodhouse Creek Catchments

According to Figure 2-7, however, the Flow generated by catchments of similar sizes would be typically lower, or possibly much lower. As such, the 223m^3/s suggested by the tool may potentially be higher than expected. Further evidence of this is provided in Section 2.5.2.

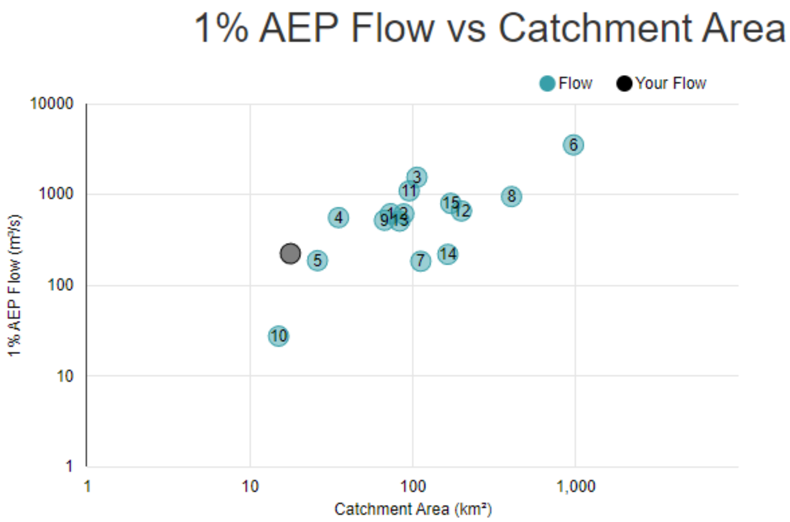

Figure 2-7 - RFFE 1% AEP Flow vs Catchment Area for Menangle / Woodhouse Creek Catchments

It should also be noted that the RFFE technique, whilst being the best available database of gauged catchments, is still only a small number of catchments in comparison to the wide range of conditions experienced across Australia (Ataur, R. et.al., 2019).

2.5.2 NSW Flood Frequency Analysis Reconciled Losses

In the Data Hub NSW jurisdiction specific advice, a catchment map is provided for deriving losses according to Flood Frequency Analysis Reconciled Losses. There are no catchments close enough to the Menangle and Woodhouse Creek catchments to derive losses for the hydrology. However, it is noted that the closest two gauged catchments – Wedderburn and Mulgoa Road show significantly lower flows than what is produced by the RFFE.

2.5.3 NSW Rational Method

The Campbelltown (Sustainable City) Development Control Plan (DCP) (2009) recommends the Rural Rational Method as an acceptable method for calculating flows. Whilst the Rational Method is no longer consider best practice under the latest ARR2019 Guidelines, the Rational Method is still a valuable tool for comparison of flows from a computer-generated hydrologic model. The method as outlined in the DCP is illustrated in Figure 2-8.

The Rational Method will be acceptable for all sized rural catchments.

$$Q_Y = 0.278 \times C_Y \times I_{tc, y} \times A$$

Where:

- Q_Y = peak flow rate (m^3/s) of average recurrence interval Y years
- C_Y = runoff coefficient (dimensionless) for ARI of Y years
- $I_{tc, y}$ = average rainfall intensity (mm/hr) for design duration of t_c hours and ARI of Y years found in the tables in Appendix B
- A = area of catchment (km^2)

The Probabilistic Rational Method discussed in AR&R is to be used to determine the time of concentration, t_c for rural catchments.

$$t_c = 0.76A^{0.38}$$

Where: A = area of catchment (km^2)

Further information can be found in Book 5 of AR&R.

Figure 2-8 - Rural Rational Method Outlined in Campbelltown DCP (2009)

The Rural Rational methodology was used to calculated runoff from the Menangle and Woodhouse Creek catchments, assuming a runoff coefficient of 0.31, which utilised an effective area approach with a majority rural catchment. All validation results are outlined in Table 2-8.

Table 2-8 - Validation Results – Existing Conditions

Location	RORB (m^3/s)	RFFE (m^3/s)	Rational (m^3/s)	TUFLOW (m^3/s)
Catchment Outlet at Nepean River	92.73	223 5% CL = 81.8 95% CL = 615	106.7[2]	105.0

As can be seen from the above table, the RORB, Rational and TUFLOW results are all within +/- 12% of each other and the final RORB result falls quite close to the 5% confidence limit of the RFFE, whereas the RFFE suggested result produces a significantly higher flow. Given the discussion in 2.5.3, it is suggested that the RFFE method be ruled out for validation of the results.

[2] Note that this value has used the 2016 IFD values, where the Rational method would typically use the 1987 values, however, since all other validation methods use the 2016 IFD values, it was more appropriate to also use the 2016 IFD values for this Rational Method calculation.

2.6 Existing Conditions Flood Results

2.6.1 1% AEP

The critical duration for the majority of the Menangle Creek and tributaries is typically 120-minute, with most of the upstream rainfall being 90 minutes. The inflow from the Woodhouse Creek catchments are 360minutes. This analysis indicates the site is subject to short duration flash flooding rather than long duration riverine flooding and it is unlikely that there will be warning time for at-site flooding.

The flooding has been filtered to illustrate only flooding that is greater than 50mm depth as it is assumed that flooding less than 50mm is shallow overland flows that can be managed via the drainage strategy.

There are three main tributaries of Menangle Creek that traverse the site, as discussed earlier, these have been named Creek01, Creek02 and Creek03 for ease of discussion. Typically, the flooding is contained within the channels and surrounding vegetated reserve areas. Creek01 conveys flows between 1.8 m^3/s to around 6 m^3/s north-west to south-east. Creek02 conveys flows between 3.5 m^3/s and 8 m^3/s and the Creek03, the main channel that traverses the property from north to south and will be retained as a natural channel within the subdivision design, conveys flows between 5 and 15 m^3/s in existing conditions. There is a small channel to the west of the quarry that conveys approximately 5 m^3/s.

Outside of these channels the flooding is mainly shallow sheet overland flows of depths up to 200-300mm and this flooding can be easily managed by the future stormwater strategy. The flooding within the quarry itself is quite deep, but that is due only to rainfall caught by the deep terrain. The overland flows will need to be managed once the quarry is filled and developed conditions runoff determined.

There is approximately 11 m^3/s flowing off the central western sub-catchment of the site crossing under the eastern lane of the Hume Highway into the swale between the two lanes. This flow then travels in a southerly direction towards the Nepean River.[3]

Finally, depths within Menangle Creek range between 1 and 4 metres.

The 1% AEP existing conditions flood depth is illustrated in Figure 2-9 below.

[3] It should be noted that details of culverts under the Hume Highway are currently unavailable and as such, flow is shown overtopping the Hume, where it is more likely that there are culverts present at this location.

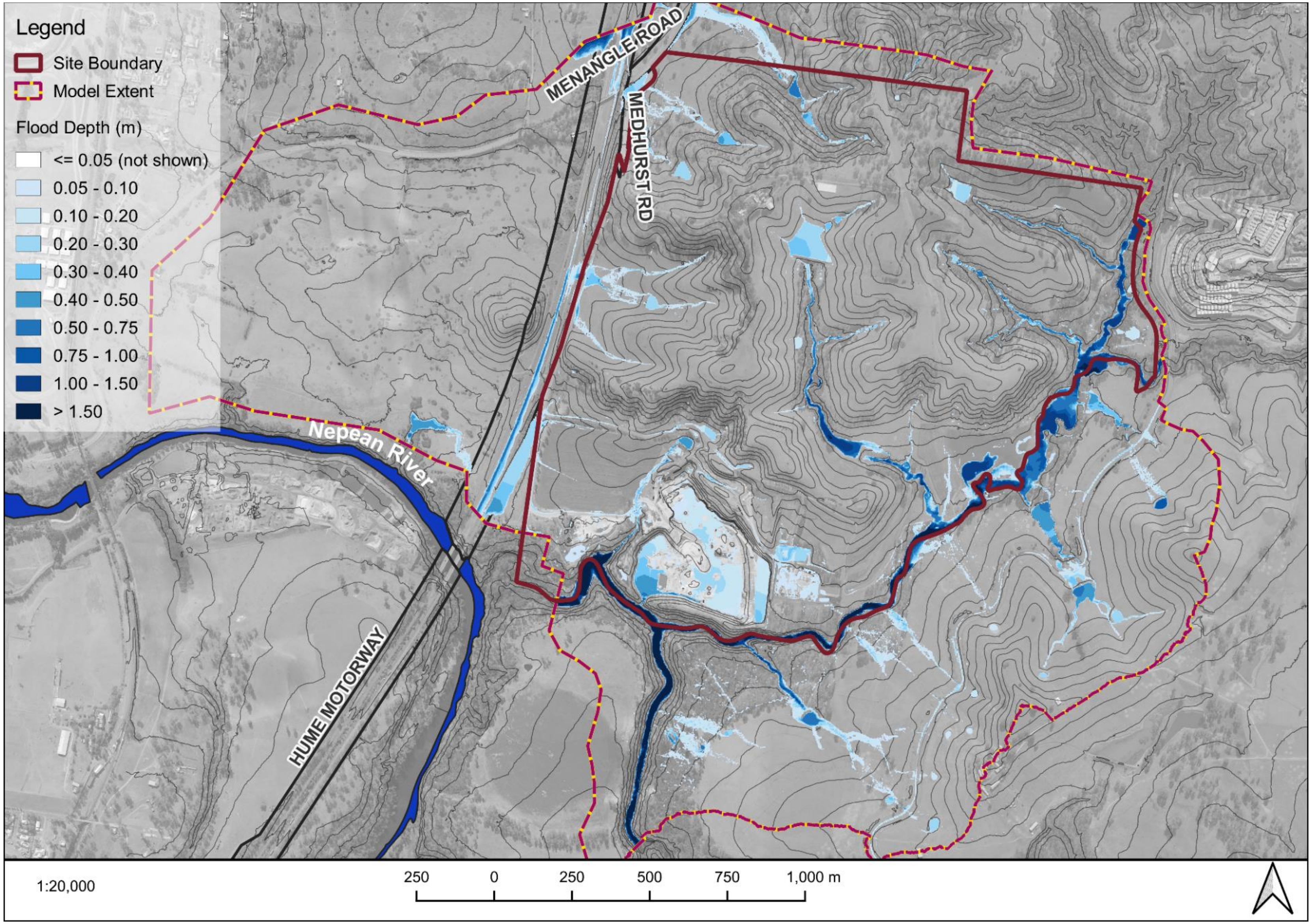

Figure 2-9 1% AEP Existing conditions flood depth (m)

2.6.2 Other Flood Events

The existing conditions modelling for the 20%, 10% and 5% flood events and the PMF will be undertaken at the next stage of the Project once Council feedback has been received on the modelling undertaken to date.

2.7 Developed Conditions Flood Results

2.7.1 1% AEP

Like the Existing Conditions results, the critical duration for the majority of Menangle Creek and the tributaries is 2 hours. However, within the developed conditions, the impervious percentage across the site has increased and therefore the runoff is slightly faster. It is observed that a larger portion of the flooding has a critical duration of 90-minutes across the site area than in the existing conditions. The runoff from the small catchment in the central west of the site was previously 90minutes, but has increased to 120minutes in developed conditions, which means the flow along the Hume Highway swale has increased to 120minutes.

In the developed case, Creek01 will ultimately be piped, however is showing overtopping in the current design. Creek02 conveys flows between approximately 3 m^3/s to 8 m^3/s and Creek03, which is to be retained as a riparian corridor conveys flows between 5 and 15 m^3/s in developed conditions. There is a small channel to the west of the quarry that conveys approximately 5 m^3/s. The small channel to the west of the corridor is no longer functioning in the same manner as in existing conditions.

Outside of the Creek corridors, the flow is typically shallow, in the order of 150mm to 300mm and is conveyed by the internal road network.

There is approximately 11 m^3/s flowing off the central western sub-catchment of the site across the eastern lane of the Hume Highway into the swale between the two lanes. This flow then travels in a southerly direction towards the Nepean River.

The flood depth in Menangle Creek is similar to that of the existing conditions flooding.

The 1% AEP developed conditions flood depth is illustrated in Figure 2-10 below.

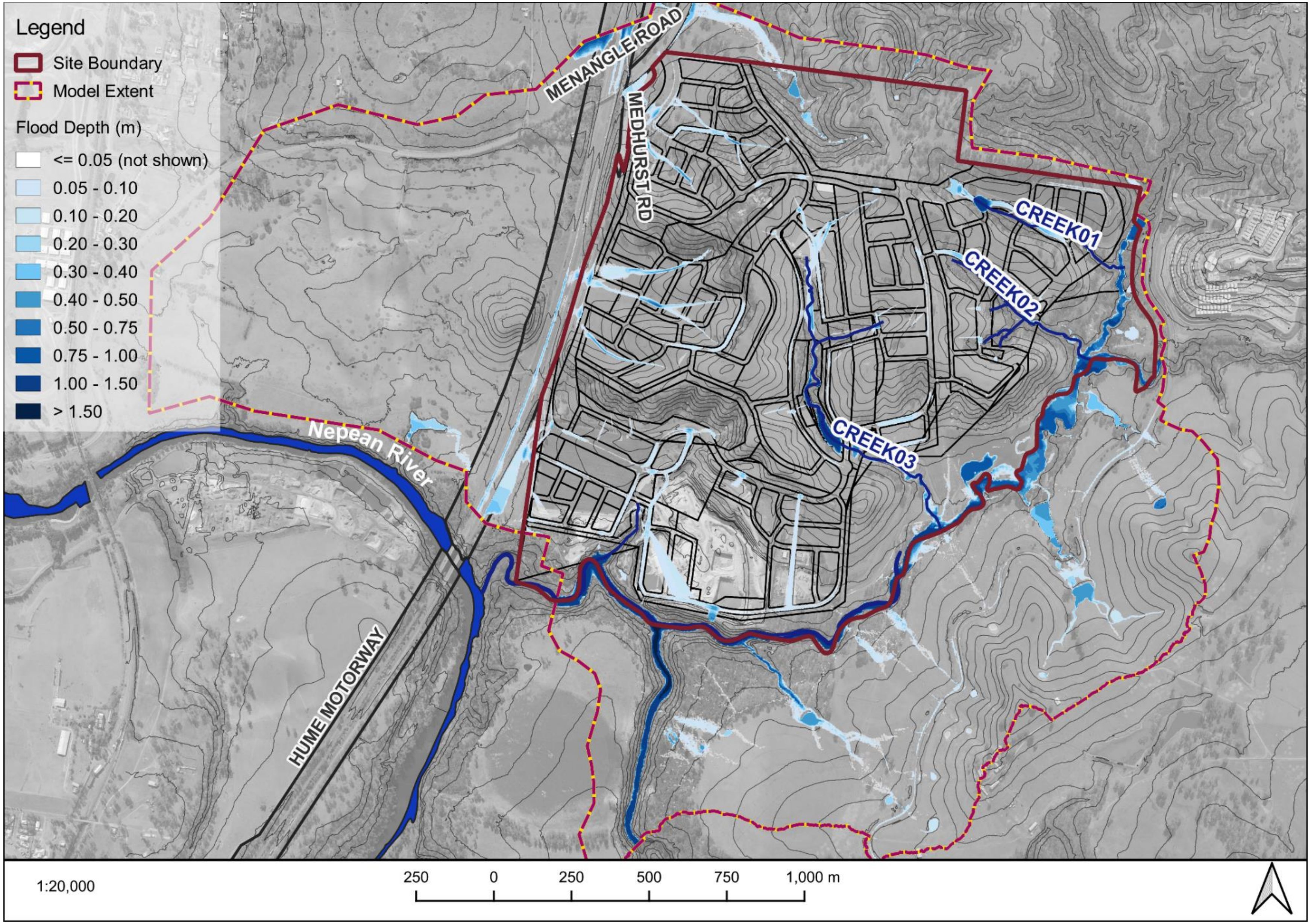

Figure 2-10 1% AEP Developed conditions flood depth (m)

2.7.2 Other Flood Events

The developed conditions modelling for the 20%, 10% and 5% flood events and the PMF will be undertaken at the Development Application stage of the Project once Council feedback has been received on the modelling undertaken to date.

2.8 Change in Flood Levels

To determine the impacts of the proposed development concept plans, an afflux map was produced. An afflux map is the difference between the water levels in developed conditions and the water levels in existing conditions. The afflux map indicates that there are significant changes in flood level across the site itself as the site levels have significantly changed, however there are no off-site impacts on the downstream proposed Menangle Park development location.

Key points to note from the Afflux Map are as follows:

- There is an appearance of over 25m afflux in the vicinity of the quarry, however, this is only because the quarry is being filled and represents a change in terrain level, not a change in flood level
- There is an increase in flood levels towards the top northeast corner of the site, where a new road is to be constructed
- There is a minor increase in flood levels (less than 100mm) at the northwest corner of the site on Menangle Road
- The flood levels are typically decreased on Medhurst Road, except where the levels of the new Medhurst Road are planned to be raised
- The flows running off the central west sub-catchment of the site, crossing under the Hume Motorway and south towards the Nepean River are decreased by approximately 200mm
- Flows are typically decreased within Menangle Creek. However, this needs to be confirmed with further modelling of the proposed road crossing of the Riparian Corridor at Creek03. Currently Creek03 is showing significant afflux within the Riparian Corridor. It is believed that incorporation of a culvert at this location is not the appropriate crossing and that a larger clear span crossing would better serve conveyance of flows at this location

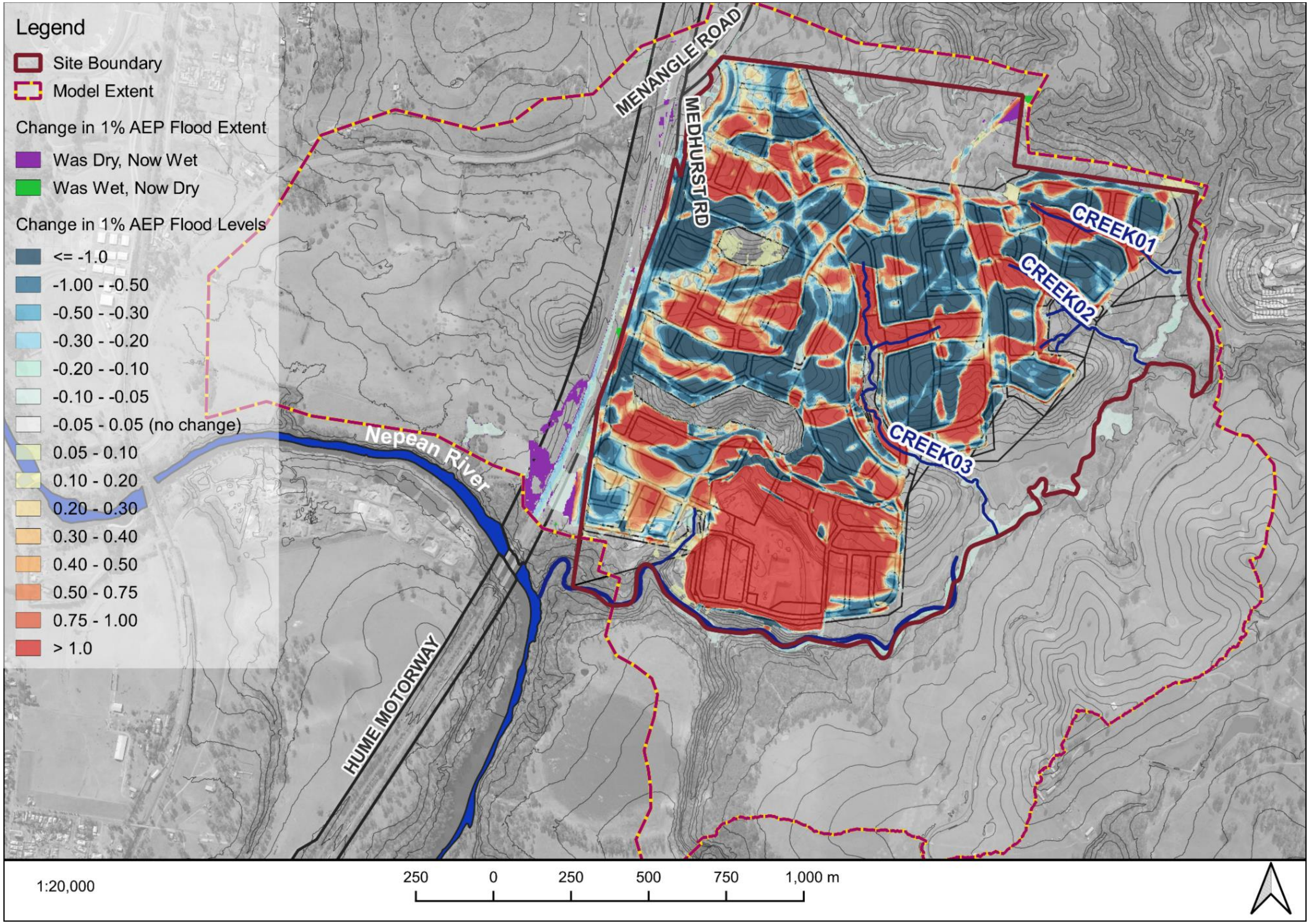

Figure 2-11 1% AEP Flood Afflux (Difference between developed conditions and existing conditions water levels)

3. Water Quality Management Strategy

3.1 Pollutant Reduction Targets

Although there is no relevant site specific DCP, the neighbouring Menangle Park Precinct development does have a site specific DCP which specifies the pollutant retention targets as shown in Table 3-1. These stretch targets are generally more stringent than those that are typically adopted across Sydney and Australia, and thus they have been adopted for the purposes of this study. Refer to the Menangle Park WSUD Strategy report (AECOM, 2010) for further detail about the selection of these water quality stretch targets.

Table 3-1 Stormwater Quality PerformanceTargets

Pollutant	Typical % Reduction	Stretch Target % Reduction
Total Suspended Solids (TSS)	85%	85%
Total Phosphorus (TP)	65%	70%
Total Nitrogen (TN)	45%	55%
Gross Pollutants	90%	90%
Stream Erosion Index (SEI)	1.0 – 3.5	1.0 – 2.0

3.2 Water Quality Management Strategy

The adopted stormwater quality management strategy includes a provision of a treatment train to treat surface runoff with reflection to the drainage network for the ultimate condition. These treatment systems can be integrated within the landscape and open space areas and distributed throughout the catchment or concentrated in centralised locations.

The following water quality control assets are proposed for implementation:

a. Rainwater tanks – for collection of runoff from roofs and re-use of water for irrigation and household use.
b. Proprietary Gross Pollutant Traps (GPTs) - for removal of coarse sediment and large debris, reducing maintenance obligations and pollutant loads on the tertiary treatment controls. Sized generally for the 3–6-month flow.
c. Bioretention systems - for capture of finer sediments and treatment of nutrients. This may come in various forms such as basins, swales or vegetated channels.

3.3 Methodology

The stormwater quality management modelling has been prepared using MUSIC (Model for Urban Stormwater Improvement Conceptualisation) Version 6.3. As Campbelltown Council does not have MUSIC-link data, modelling was completed by adopting Blacktown City Council's MUSIC-link Data Version 6.34 (Blacktown Development.mlb).

3.4 Hydrologic Data Inputs

Blacktown's MUSIC-link mlb. file uses 6 mins rainfall and monthly PET evaporation data from the Rainfall Station -67035 LIVERPOOL(WHITLAM) records with the times series 01/01/1967 to 31/12/1976 used. Rainfall and PET for the period are presented in Figure 3-1 below.

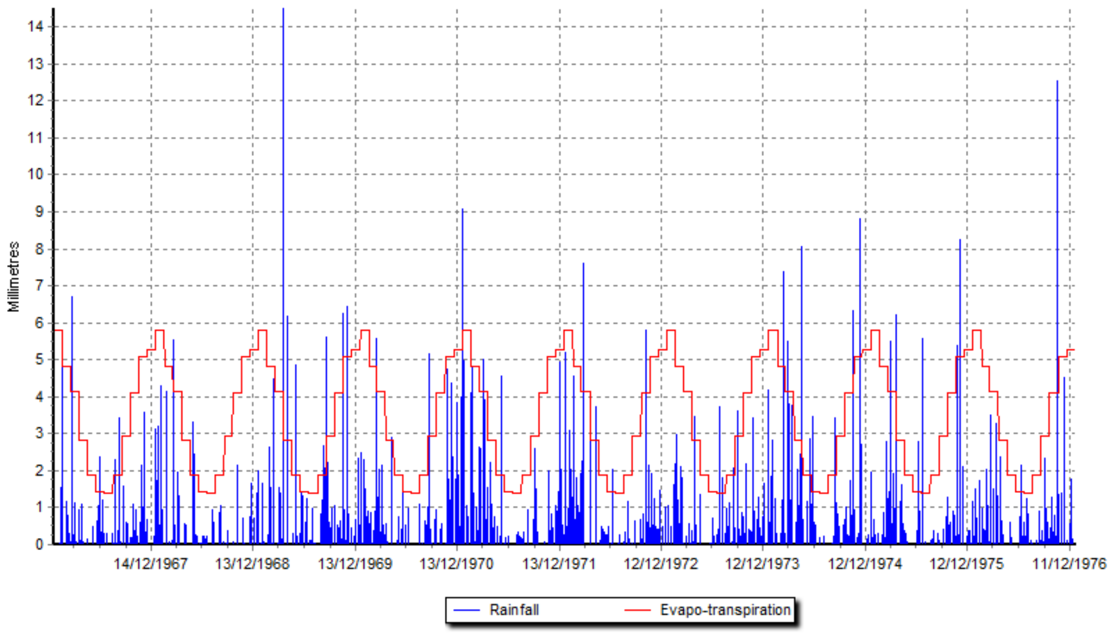

Figure 3-1 Rainfall and PET graph (MUSIC)

3.5 Source Node Data Inputs

Source Node parameters were adopted from Blacktown City Council's MUSIC-link Data. The following table summarises the source node inputs used within the MUSIC model.

Table 3-2 Stormwater Quality parameters Source Nodes

Landuse Category		Total Suspended Solids (mg/L Log_{10})		Total Phosphorus (mg/L Log_{10})		Total Nitrogen (mg/L Log_{10})	
		Storm Flow	Base Flow	Storm Flow	Base Flow	Storm Flow	Base Flow
Urban Areas	Mean Std Dev	2.15 0.32	1.20 0.17	-0.60 0.25	-0.85 0.19	0.30 0.19	0.11 0.12
Roof Areas	Mean Std Dev	1.30 0.32	1.10 0.17	-0.89 0.25	-0.82 0.19	0.30 0.19	0.32 0.12
Pervious Areas	Mean Std Dev	2.15 0.32	1.20 0.17	-0.60 0.25	-0.85 0.19	0.30 0.19	0.11 0.12

3.6 Rainfall-Runoff Parameters

MUSIC rainfall-runoff parameters were adopted from Blacktown City Council's MUSIC-link Data. The following table summarises the parameters used within the MUSIC model.

Table 3-3 Rainfall-Runoff parameter table

Parameter	Roof / Mixed Node Type	Residential Node Type
Rainfall threshold (mm/day)	1.4	1.0
Soil storage capacity (mm)	170	120
Initial Storage (%)	30	25
Field capacity (mm)	70	80
Infiltration capacity coefficient a	210	200
Infiltration capacity exponent b	4.7	1.0
Initial depth (mm)	10	10
Daily recharge rate (%)	50	25
Daily baseflow rate (%)	4	5
Deep seepage (%)	0	0

3.7 Catchment Details

The proposed development site has been divided into several sub-catchments based on the RORB model developed for the existing conditions, proposed grading and the land use. The site is divided into 4 categories:

- Low density residential area;
- Medium density residential area
- Roof area draining to rainwater tanks
- Parkland / open space area

The road areas were included within the residential area catchments, whilst the parkland and open space areas were modelled as a separate source node. Areas of bushland and riparian corridor that are undisturbed from existing conditions have been excluded from the model.

The total area of the roof source nodes has been calculated by estimating the number of dwellings in each catchment and multiplying the value by an average roof area of 200 m^2. These roof nodes are modelled to be 100% impervious. The residential areas have been modelled with source nodes as either low or medium density residential and includes the remainder of the catchment area.

The effective impervious area (EIA) of the catchment has been calculated based on both the percentage impervious values for various land uses recommended in the Campbelltown Council DCP and the NSW MUSIC Modelling Guidelines (BMT WBM, 2015) and is summarised in Table 3-4**.** The EIA for the residential source nodes has been further reduced to account for the separation of the 100% impervious roof area

Table 3-4 Catchment Landuse Characteristics

	Total Impervious Area (%)	EIA Factor	Adopted EIA (%)
Low Density Residential	70	0.60	36 [1]
Medium Density Residential	80	0.60	42 [1]
Roof	100	1.00	100
Parkland	30	0.05	10 [2]

[1] The adopted impervious area accounts for the separation of the 100% impervious roof areas from the residential areas. The sum of the impervious roof area and the impervious area for the residential areas is equal to the total impervious area of the urban catchment.

[2] 10% is adopted to assume potential future amenity buildings, footpaths and hard surfaces.

3.8 Treatment Train

The stormwater design for the development will use a combination of at source and conveyance controls to treat the stormwater runoff from the site. The following are the treatment trains proposed for this development.

Rainwater Tanks

Rainwater tanks are proposed for each dwelling as part of the treatment train and BASIX requirements in accordance with Council guidelines as shown below in Table 3-5. A 3 kL tank has been adopted in this model.

The internal daily re-use rate has been adopted as 0.115 kL/dwelling/day with an outdoor daily re-use rate of 0.151 kL/dwelling/day, which sums up to a total re-use rate of 0.266 kL/dwelling/day in accordance with the NSW MUSIC Modelling Guidelines (BMT WBM, 2015). The tanks have been modelled as half full at the start of the storm event, with a 25% reduction in the number of tanks to account for owner non-compliance.

Table 3-5 Rainwater tanks recommended in Council DCP

Table 2.4.1 Rainwater Tank Capacity

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Roof Area	Capacity of Rainwater Tank
101 m^2 to 200 m^2	3,000L
201 m^2 to 1,000 m^2	5,000L
1,001 m^2 to 5,000 m^2	10,000L
5,001 m^2 to 10,000 m^2	20,000L
10,001 m^2 to 20,000 m^2	50,000L
above 20,000 m^2	100,000L

Gross Pollutant Traps

Gross Pollutant Traps (GPTs) are proposed upstream of the bioretention systems. The performance criteria of the GPTs is presented in Table 3-6.

Table 3-6 Gross Pollutant Trap capture efficiency table

Pollutant	Capture Efficiency
Gross Pollutant (>2000µm)	98%
Total Suspended Solids (TSS) (20 - 2000µm)	75%
Total Phosphorous (TP)	30%
Total Nitrogen (TN)	0%
Total Petroleum/Hydrocarbon/oils	98%

Bioretention Basins

Bioretention systems are proposed for all impervious catchments. The basins will have a high flow bypass to help safely convey the 100-year flow and to treat low flows before they are discharged downstream. Figure 3-2 shows a typical section of the bioretention basin.

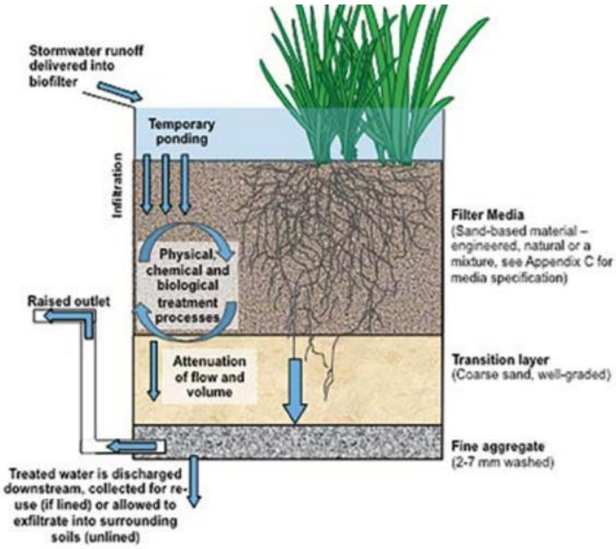


Figure 3-2 Bioretention system schematic

3.9 Water Quality Treatment Bioretention Basins

The design parameters adopted for the bio-retention systems are shown in Table 3-7. Filter media depths are proposed to be 0.5m. Extended detention depth of all bio-retention basins has been modelled as 0.3m. For bio-retention basins co-located within detention basins, a weir should be constructed to be 0.3m above the base of the bio-retention basin, ensuring at least an extended detention depth of 0.3 m.

Within the MUSIC model, the basin surface area (the surface area at the extended detention depth) has been set equal to the filter media area (basin invert area). This is considered a conservative approach as in reality all basins are likely to have side slopes of at least 1V:4H meaning the surface area will be greater than the filter media area.

However, this simplified approach is considered appropriate at this stage as it allows for optimisation of bio-retention design in later detailed design stages.

The water quality treatment devices proposed across the development are shown indicatively Figure 3-3, with the MUSIC model layout provided in Figure 3-4. The filter media areas provided are summarised in Table 3-8.

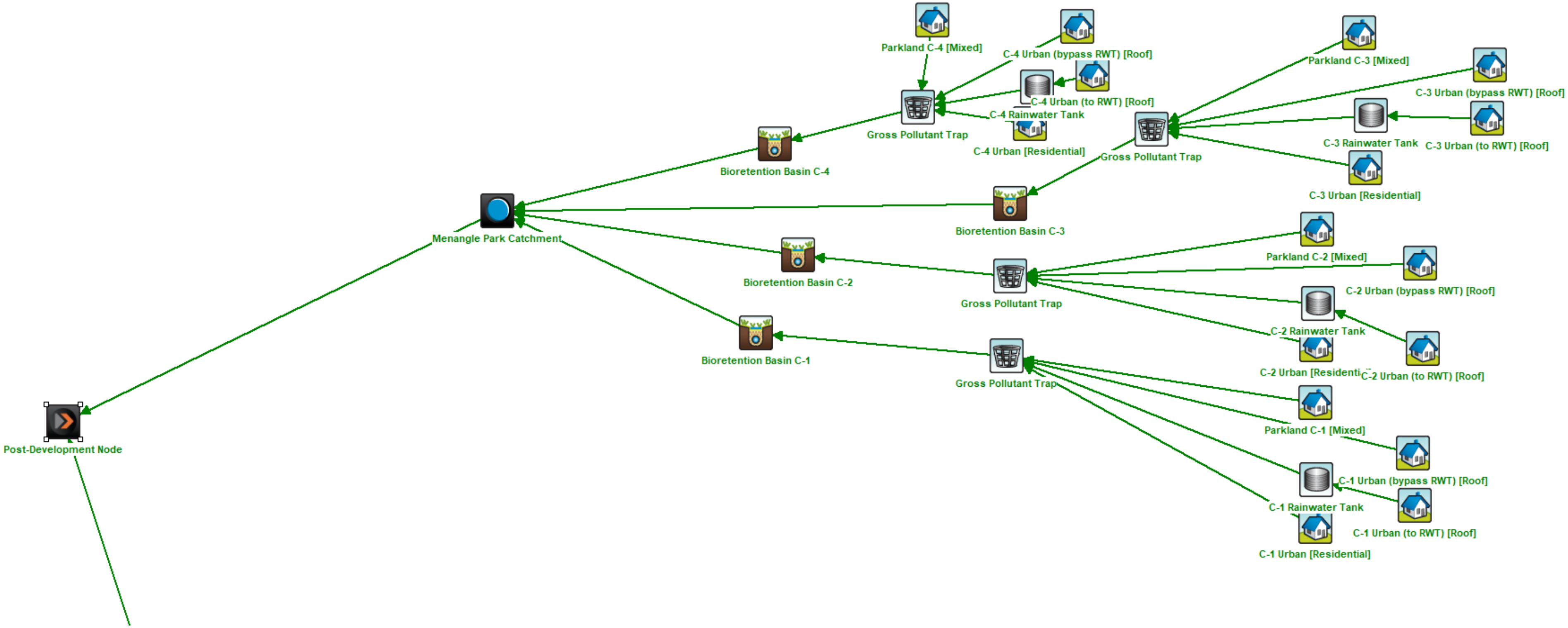
Table 3-7 Bioretention Filter Media Areas Provided

Parameters	Value
Saturated Hydraulic Conductivity (mm/hr)	100
Filter Depth (m)	0.5
Extended Detention (m) 0.3	0.3
TN Content (mg/kg) 400	750
Orthophosphate Content (mg/kg) 40	40
Exfiltration Rate (mm/hr) 0.0	0.0
Base Lined	YES

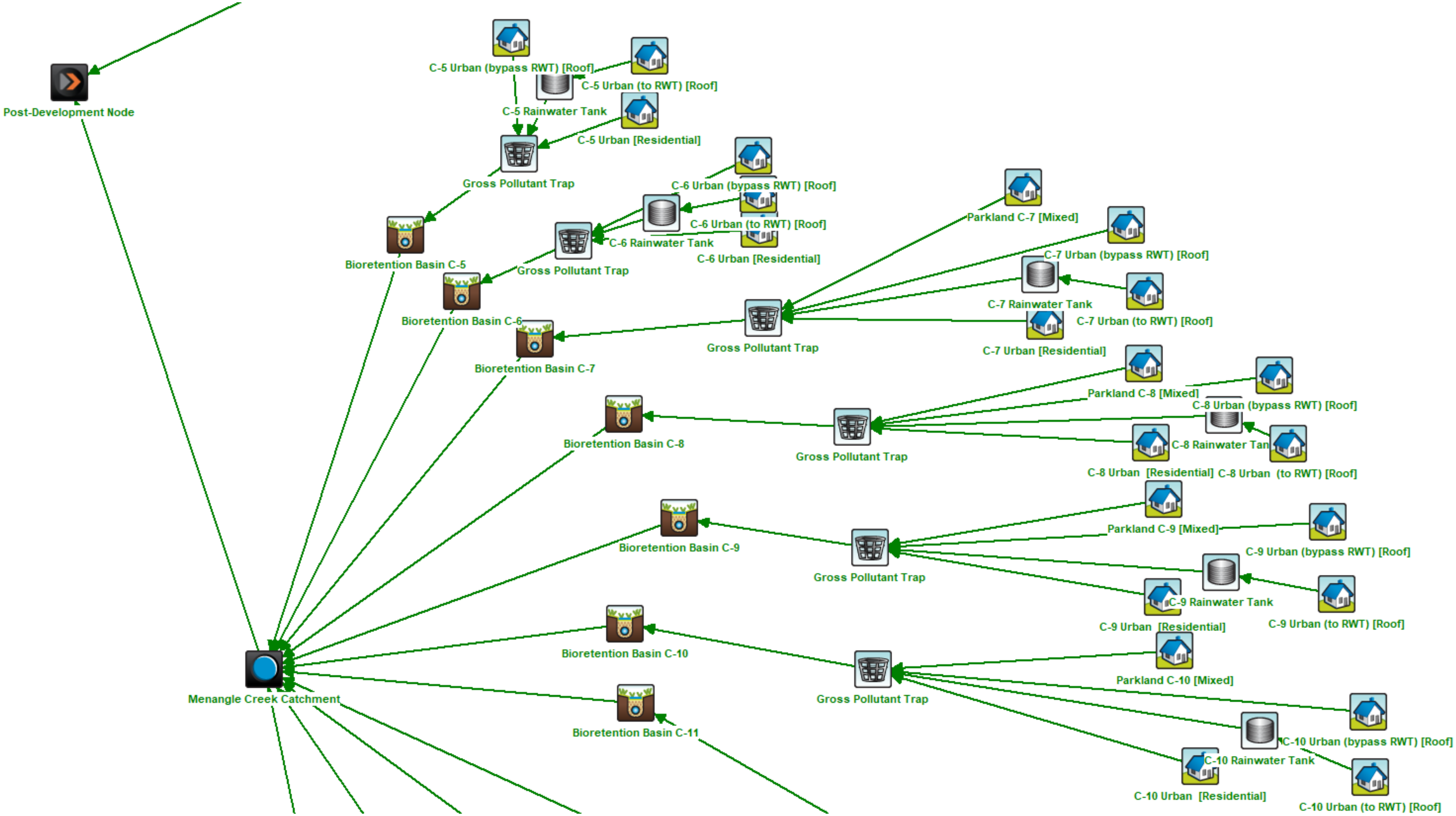
Table 3-8 Bioretention Filter Media Areas Provided

Bioretention Basin Areas	
Basin Name	**Filter Media Areas (m^2)**
C-1	2300
C-2	2000
C-3	900
C-4	1200
C-5	200
C-6	200
C-7	1200
C-8	2700
C-9	500
C-10	2900
C-11	1700
C-12	500
C-13	800
C-14	200
C-15	1200

Figure 3-3 Indicative Bioretention Basin Locations and Filter Area Footprint



Parkland C-4 [Mixed]
C-4 Urban (bypass RWT) [Roof]
C-4 Urban (to RWT) [Roof]
C-4 Rainwater Tank
C-4 Urban [Residential]
Gross Pollutant Trap
Bioretention Basin C-4
Parkland C-3 [Mixed]
C-3 Urban (bypass RWT) [Roof]
C-3 Rainwater Tank
C-3 Urban (to RWT) [Roof]
Gross Pollutant Trap
C-3 Urban [Residential]
Bioretention Basin C-3
Menangle Park Catchment
Parkland C-2 [Mixed]
C-2 Urban (bypass RWT) [Roof]
Bioretention Basin C-2
Gross Pollutant Trap
C-2 Rainwater Tank
C-2 Urban [Residenti
C-2 Urban (to RWT) [Roof]
Bioretention Basin C-1
Gross Pollutant Trap
Parkland C-1 [Mixed]
C-1 Urban (bypass RWT) [Roof]
C-1 Rainwater Tank
C-1 Urban (to RWT) [Roof]
C-1 Urban [Residential]
Post-Development Node


Post-Development Node
C-5 Urban (bypass RWT) [Roof]
C-5 Urban (to RWT) [Roof]
C-5 Rainwater Tank
C-5 Urban [Residential]
Gross Pollutant Trap
Bioretention Basin C-5
C-6 Urban (bypass RWT) [Roof]
C-6 Urban (to RWT) [Roof]
C-6 Rainwater Tank
C-6 Urban [Residential]
Gross Pollutant Trap
Bioretention Basin C-6
Parkland C-7 [Mixed]
C-7 Urban (bypass RWT) [Roof]
C-7 Rainwater Tank
C-7 Urban (to RWT) [Roof]
C-7 Urban [Residential]
Gross Pollutant Trap
Bioretention Basin C-7
Parkland C-8 [Mixed]
C-8 Urban (bypass RWT) [Roof]
C-8 Rainwater Tan
C-8 Urban [Residential]
C-8 Urban (to RWT) [Roof]
Gross Pollutant Trap
Bioretention Basin C-8
Parkland C-9 [Mixed]
C-9 Urban (bypass RWT) [Roof]
C-9 Rainwater Tank
C-9 Urban [Residential]
C-9 Urban (to RWT) [Roof]
Gross Pollutant Trap
Bioretention Basin C-9
Parkland C-10 [Mixed]
C-10 Urban (bypass RWT) [Roof]
C-10 Rainwater Tank
C-10 Urban [Residential]
C-10 Urban (to RWT) [Roof]
Gross Pollutant Trap
Bioretention Basin C-10
Bioretention Basin C-11
Menangle Creek Catchment

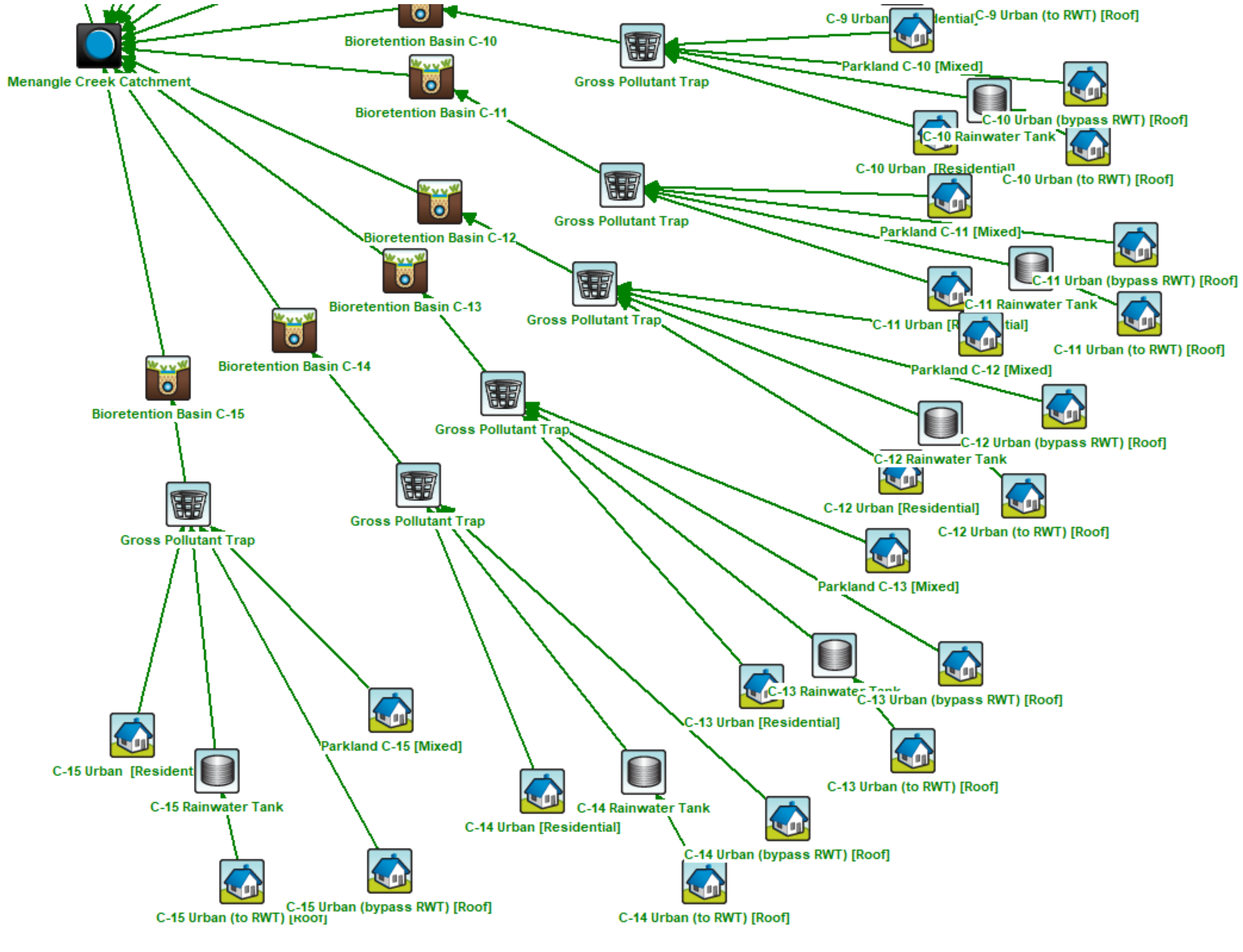


Figure 3-4 Post Development MUSIC MODEL (REF 467-21_PP_WSUD-003_POST DEV)

3.10 Water Quality Modelling Results

The modelling results analysis in MUSIC indicates that the proposed treatment train provides adequate treatment, which exceeds the water quality treatment stretch targets. The targets are exceeded for the Menangle Park, Menangle Creek and combined post-development catchments. It is noted that the majority of the development drains towards Menangle Creek, however a portion of the site drains west towards the Hume Highway / Menangle Park catchment, hence the catchments have been split as such in the MUSIC model. The Modelled Results are presented below in Table 3-9, Table 3-10 and Table 3-11 respectively.

Table 3-9 Performance Results Menangle Park (MUSIC Modelling)- PP (REF 467-21_PP_WSUD-003_POST_DEV)

Pollutant	Post-Development without Treatment	Post-Development with Treatment	Overall Reduction	Meets Performance Objectives
Total Suspended Solids (kg/yr)	30600	1870	93.9	Yes
Total Phosphorus (kg/yr)	63.2	17.6	72.2	Yes
Total Nitrogen (kg/yr)	576	211	63.4	Yes
Gross Pollutants (kg/yr)	7800	0	100	Yes

Table 3-10 Performance Results Menangle Creek (MUSIC Modelling)- PP (REF 467-21_PP_WSUD-003_POST_DEV)

Pollutant	Post-Development without Treatment	Post-Development with Treatment	Overall Reduction	Meets Performance Objectives
Total Suspended Solids (kg/yr)	51900	3210	93.8	Yes
Total Phosphorus (kg/yr)	111	30	72.9	Yes
Total Nitrogen (kg/yr)	1010	355	64.7	Yes
Gross Pollutants (kg/yr)	12800	0	100	Yes

Table 3-11 Performance Results Post Development Node (Combined) (MUSIC Modelling)- PP (REF 467-21_PP_WSUD-003_POST_DEV)

Pollutant	Post-Development without Treatment	Post-Development with Treatment	Overall Reduction	Meets Performance Objectives
Total Suspended Solids (kg/yr)	82500	5080	93.8	Yes
Total Phosphorus (kg/yr)	174	47.6	72.6	Yes
Total Nitrogen (kg/yr)	1580	565	64.3	Yes
Gross Pollutants (kg/yr)	20600	0	100	Yes

3.11 Stream Erosion Index

The treatment of water through WSUD devices is expected to limit the Stream Erosion Index (SEI) to between 1 and 2. SEI calculations will need to be shown during detailed design at a later stage of the Development Application.

4. Detention Basin Strategy

4.1 Menangle Creek

Rosalind Park is located at the downstream end of the catchment, close to the outfall of Menangle Creek into the Nepean River. It is proposed to avoid incorporating a detention basin strategy for the proposed development outfall in the direction of Menangle Creek as an analysis of the hydrology undertaken for the Menangle and Woodhouse Creek catchments demonstrates that there is a risk of coinciding the peaks of the flows from the site, with the Menangle Creek peaks.

Figure 4-1 demonstrates that the peak of the outfall hydrograph from Creek03 occurs at exactly 2.0 hours, with a peak flow of approximately 6.4m^3/s, whilst the peak flow of Menangle Creek, at the location of the confluence with Creek03, occurs at exactly 3.0 hours, with a peak flow of approximately 30m^3/s.

If the flow from Creek03 were to be retarded, it is very likely that the peak from Creek03 would occur at close to 3.0 hours, coinciding with the peak of Menangle Creek and thus increasing the peak flows within Menangle Creek.

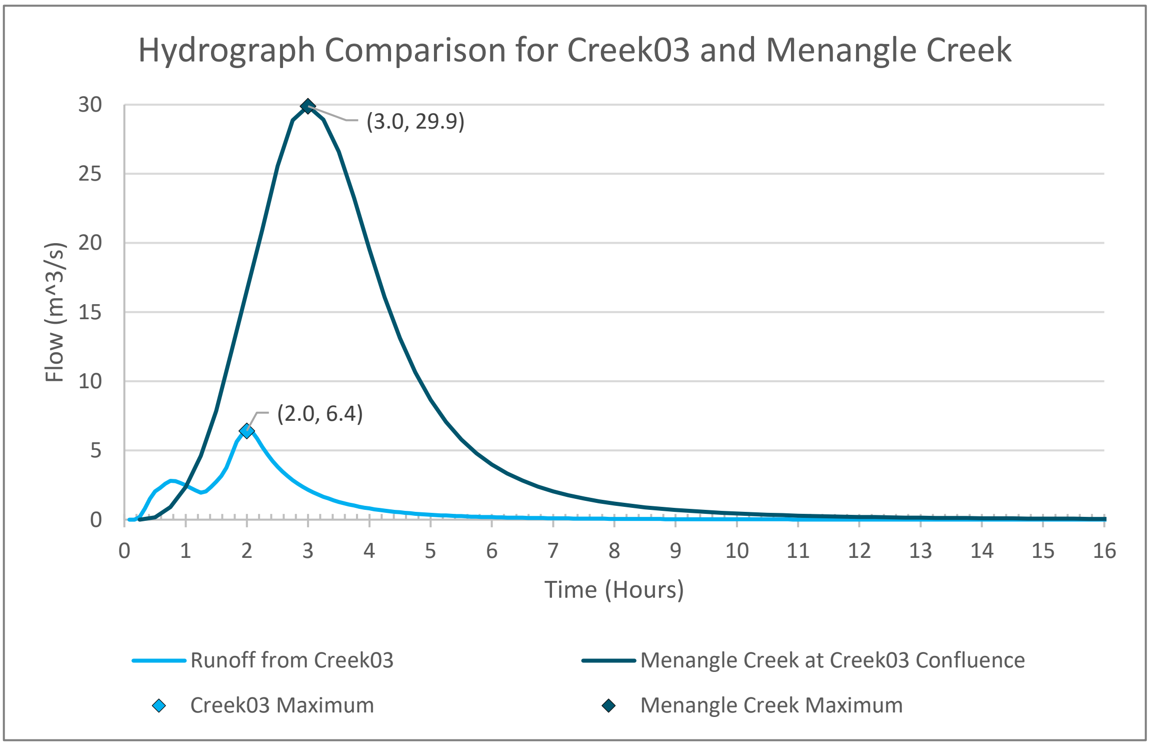


Figure 4-1 - Hydrograph of Creek03 vs Menangle Creek

Similarly, for Creek01 and Creek02, where the critical duration is 90minutes and the peak Menangle Creek flow at these confluences is closer to 2 hours, there is a risk of coinciding the peak flows if the flows were to be retarded.

4.2 Outfall to Nepean River

The analysis undertaken above applies to the Nepean River Catchments, however, the hydrology for the Nepean River is not currently available and therefore it is not possible to undertake a comparison of the hydrographs at the location of the Menangle Creek outfall. A time of concentration analysis has been undertaken and the results are included in Table 4-1, however, it is important to note that the Nepean River catchment is very complex and includes numerous tributaries, reservoirs and dams and therefore a simple Time of Concentration methodology is not likely to be accurate. Due to the likely inaccuracies of this approach, three time of concentration equations were compared to determine an average time of concentration for both the Nepean River and the catchment upstream of the Development Site.

Table 4-1 - Comparison of Time of Concentration for Nepean River and Development Site

Time of Concentration Methodology	Nepean River (Hours)	Development Catchment (Hours)
Campbelltown DCP (ARR1987 Adam's Method)	12.0	2.0
Bransby Williams Method (For Rural Catchments)	29 Hours	3 Hours
Tc = L/V (assuming velocity = 1.5 m/s)	17 Hours	1 Hour
Average Time of Concentration	19 Hours	2 Hours

Table 4-1 illustrates, that the runoff from the development site is unlikely to coincide with the peak flows from the Nepean River catchment. Nevertheless, it would be more conservative to avoid detaining flows on site to ensure that flows are released into the Nepean River prior to the peak flow arriving at the site.

4.3 Western Catchments

Since there are no offsite impacts from the development site in a westerly direction and flows into the swale between the Hume Motorway lanes is reduced, there is not a need to detain flows in this direction.

5. Risks and Opportunities

The following risks and opportunities have been identified through undertaking preliminary high-level flood modelling.

Table 5-1 Table of Risks

Item	Risk	Action
Tailwater Conditions	Without knowledge of the downstream flooding at the Nepean River, there is a risk that the flooding at the downstream boundary of the site will be underestimated, and that the Nepean River flooding may be backwatering onto the site.	Obtain the Flood Report and associated models for the Upper Nepean River from Camden Council
Calibration	Flood modelling without calibration runs the risk that the modelling will not adequately capture the true flow regime.	Obtain the Flood Report and associated models from Campbelltown Council and calibrate the model to the Nepean River or other adjacent catchments flooding where possible.
Modelling Risks	The model has been prepared with the available data and within the available time frame. There are minor changes required, but these are not significant at Planning Proposal Stage.	Model boundary conditions require minor updates. The riparian corridor structure needs to be updated from a culvert to a wider opening. Detailed site survey will be required for Design. Nepean River models are required for calibration.
Detention Risks	The incorporation of detention basins on the eastern side of the site would risk coinciding the peak flows from the development site with the peak flows in Menangle Creek.	Avoid the use of detention basins on site.

Table 5-2 Table of Opportunities

Item	Opportunity	
Design structure analysis	There is an opportunity to change the structure at the location of the Riparian crossing from a culvert to a wider opening.	This will allow conveyance of flows downstream, will reduce the impacts in the Riparian Corridor and maintain the flow regime at Menangle Creek.
Drainage Design	There is an opportunity to incorporate a drainage design into the TUFLOW model.	Once the drainage design has been incorporated, the shallow sheet overland flows present on the current flood maps will be significantly reduced.

6. Next Steps – Development Application (DA)

The next steps for the Rosalind Park Water Cycle Management Plan are as follows:

Flood Modelling

1. Calibrate the model to the Upper Nepean Flood Study and incorporate the Nepean hydrographs where possible;
2. Incorporate feedback from Council and then run the model for the 20%, 10% and PMF flood events;
3. Model any mitigation options as required

Civil Design

1. Earthworks for master planning to ensure the design can minimise the cutting and filling of areas
2. Investigate the constructability of the basin
3. Optimise Road layout and grades to match closely to the existing levels
4. Develop DA civil design across the whole site including detailed lot grading, basin and WSUD grading, tie in grading across the whole site including any required retaining wall extents and detailed stormwater layout and internal catchment extents,

7. Confirmation from Council

A critical component to the success of this planning proposal and to facilitate a future thriving community in this area is for Council and Department of Planning to approve the assumptions and findings detailed in this report.

8. Conclusion and Recommendations

A new hydrologic model was developed in RORB for the subject site, and a new hydraulic model developed in TUFLOW to model the existing flooding conditions across the proposed development site. Results of this analysis show that the flooding within the three tributaries that traverse the site is well contained and does not overtop the banks. There is a presence of shallow sheet flooding throughout the site to depths up to 150mm, which can easily be managed within a drainage strategy.

The model was updated to incorporate the changes due to the proposed development and the results compared to the existing conditions. The afflux map shows that whilst there are significant changes within the site due to the changed levels, there are no flood impacts off site, particular on the downstream properties.

A detention basin strategy has not been developed as analysis of the hydrographs shows that due to the site being at the downstream of the catchment there is a risk that site runoff peaks will coincide with peaks in Menangle Creek. It is a more conservative approach to avoid this scenario, preventing the peaks occurring together and increasing peak flow to the Nepean River.

The water quality management strategy for the proposed development has been developed and modelled in MUSIC. It is demonstrated by the model that the proposed treatment devices are able to achieve the same stretch targets adopted in the neighbouring Menangle Park development.

It is recommended that the flood modelling, water quality management strategy and basin strategy submitted for the purposes of the Planning Proposal be accepted with the lower AEP storm events and the PMF storm event to be submitted at a later stage.

It is recommended that a future drainage strategy be developed to deal with the shallow sheet overland flows. It is not recommended to include detention basins for this site, for the reasons discussed within the body of the report.

9. References

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Appendix A
RORB Delineation

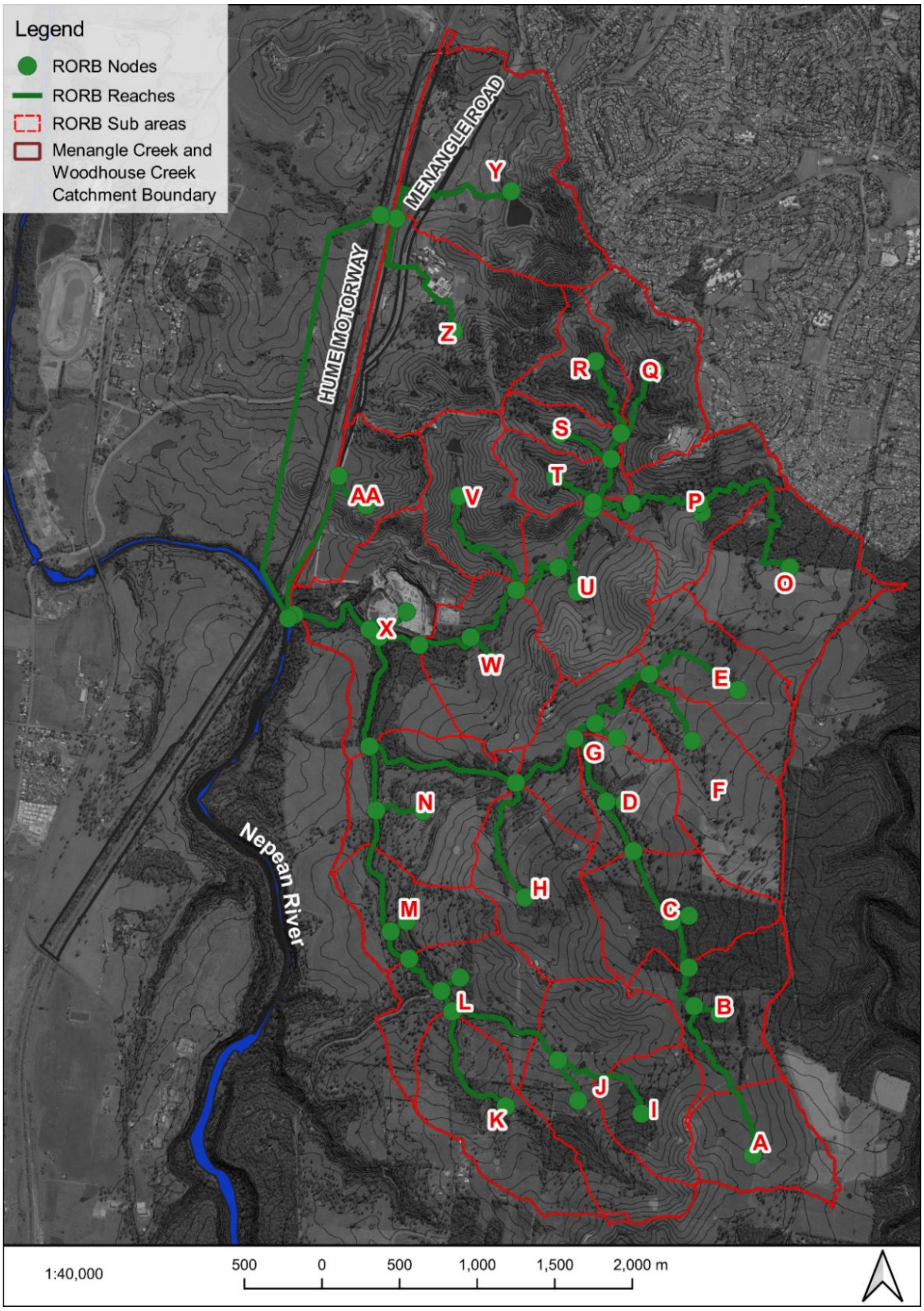


Figure 9-1 RORB Catchment Delineation

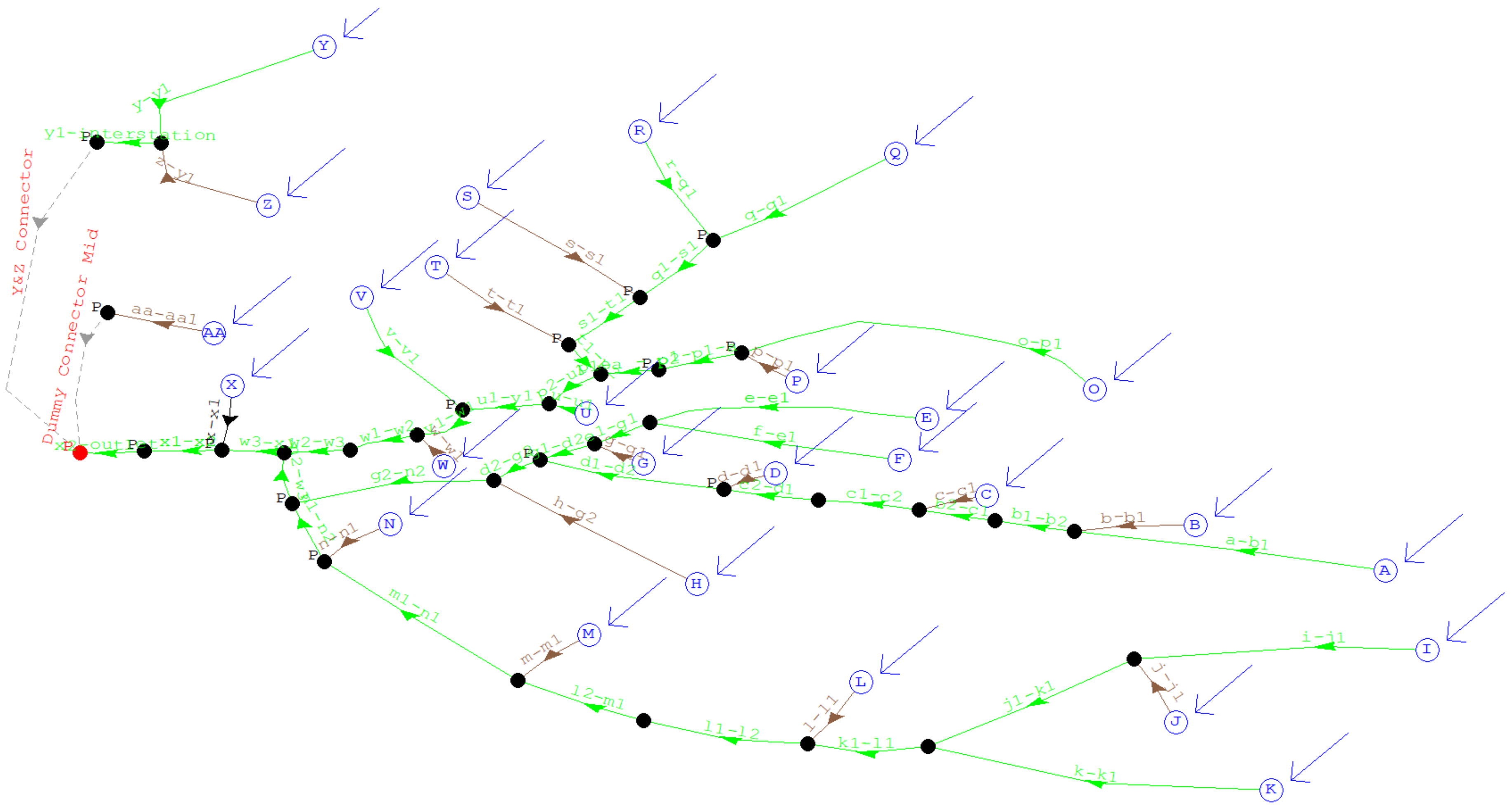


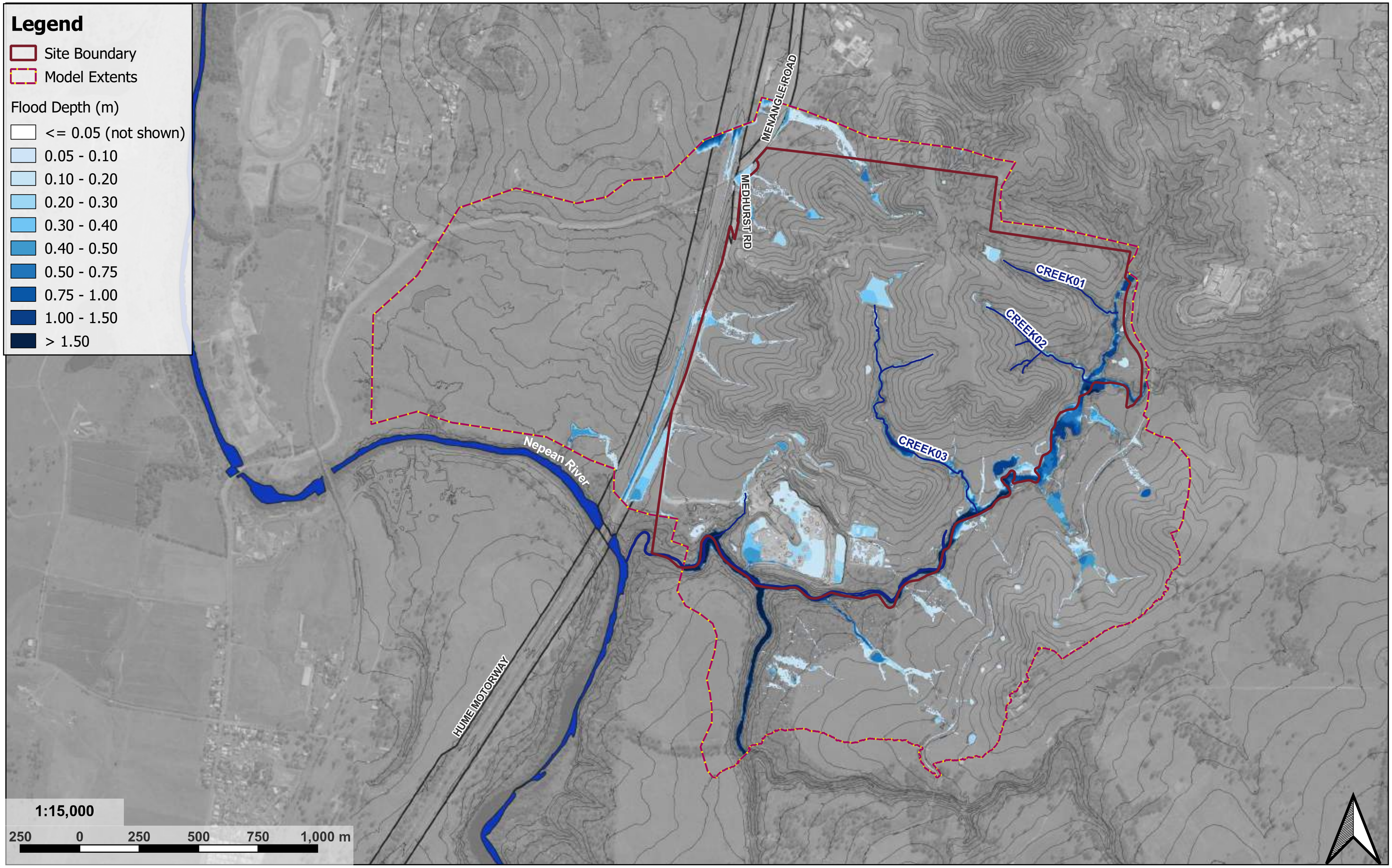
Figure 9-2 RORB Schematic

Appendix B
RORB Data Tables

Sub Area	Area (km^2)	Existing Conditions Fraction Impervious	Design Conditions Fraction Impervious
A	0.530	0.100	0.100
B	0.640	0.100	0.100
C	0.660	0.100	0.100
D	0.420	0.100	0.100
G	0.520	0.100	0.100
F	0.720	0.100	0.100
E	0.580	0.100	0.100
H	0.710	0.100	0.100
N	0.810	0.100	0.100
M	0.460	0.100	0.100
L	0.630	0.100	0.100
J	0.900	0.100	0.100
I	0.420	0.100	0.100
K	0.760	0.100	0.100
W	0.690	0.100	0.410
X	0.970	0.126	0.490
U	0.740	0.100	0.490
V	0.570	0.100	0.620
P	0.650	0.240	0.240
O	0.910	0.100	0.100
T	0.280	0.100	0.550
S	0.200	0.100	0.510
Q	0.590	0.310	0.310
R	0.340	0.100	0.400
AA	0.450	0.100	0.710
Z	1.240	0.224	0.530
Y	1.360	0.173	0.173

Appendix C

Flood Mapping



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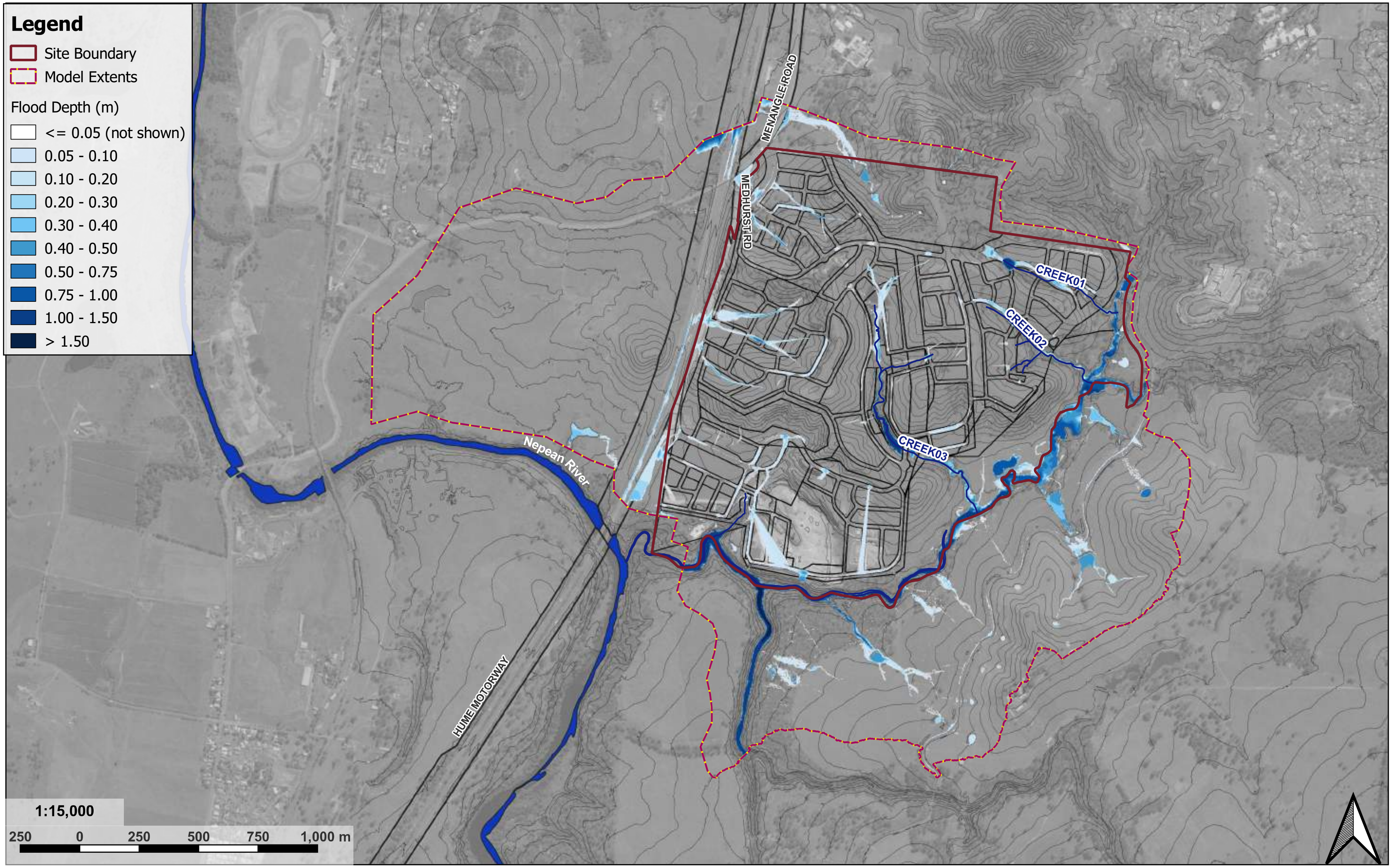
Appendix C
1% AEP Existing Case Flood Depth
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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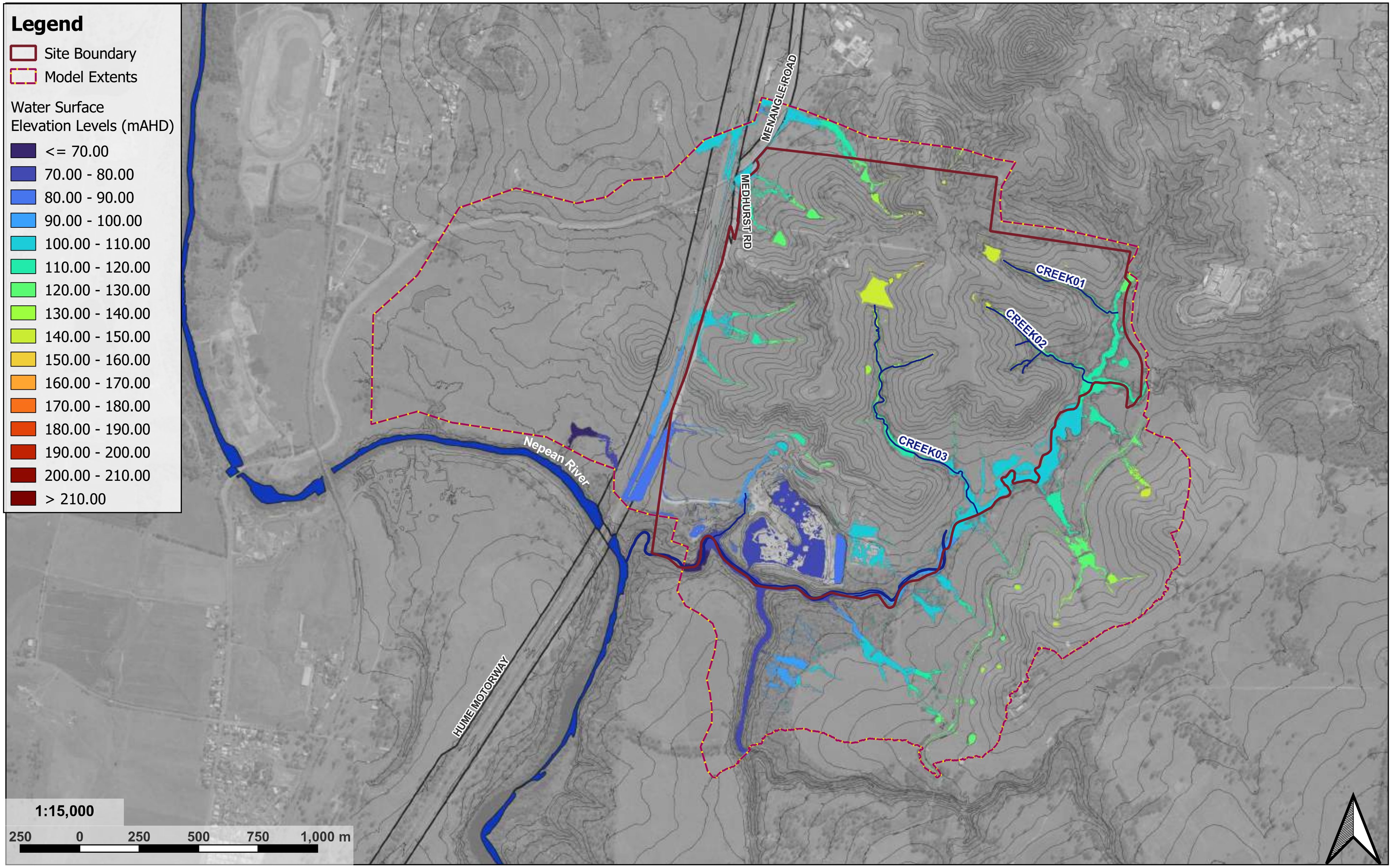
Appendix C
1% AEP Developed Case Flood Depth
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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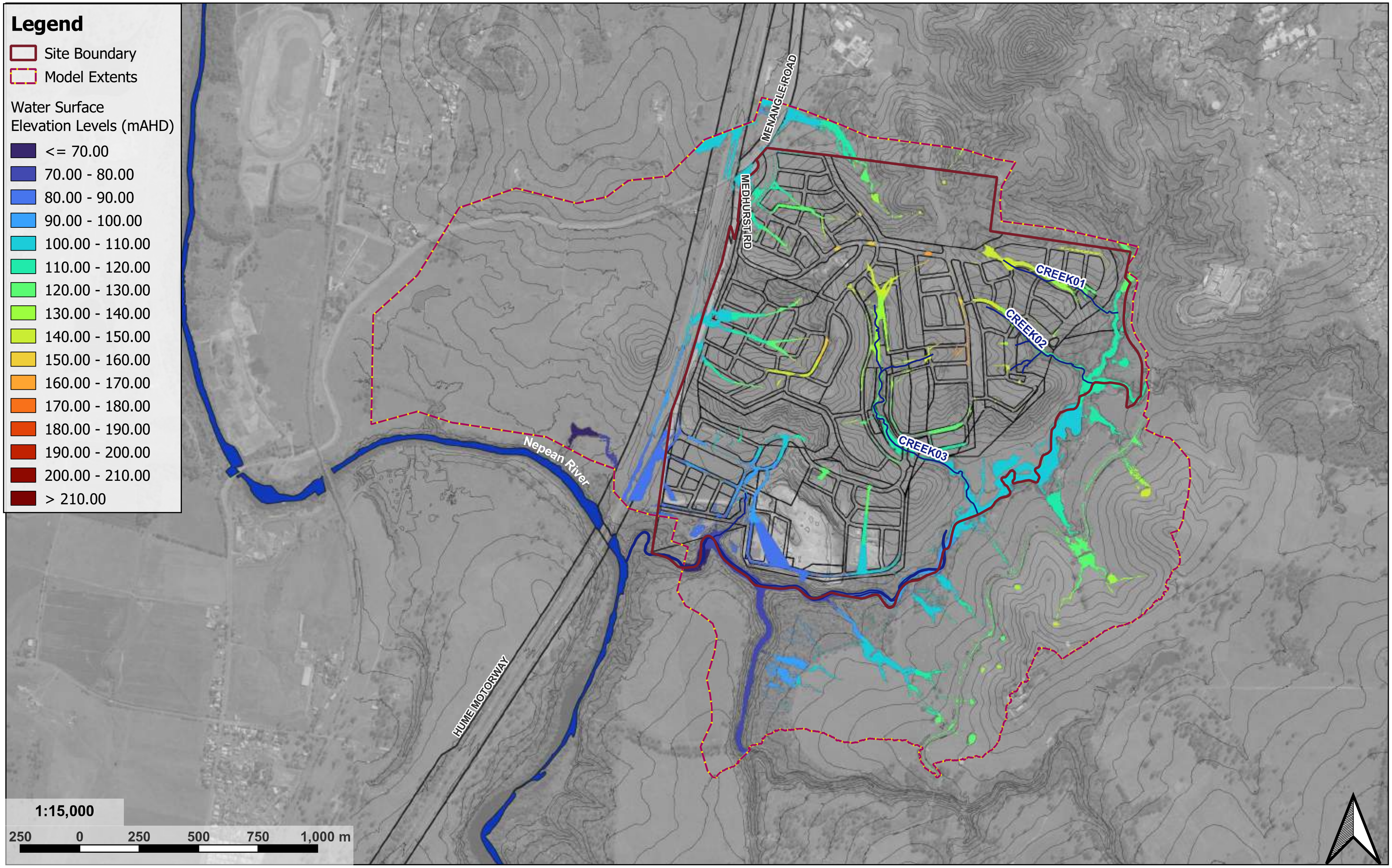
Appendix C
1% AEP Existing Case Water Surface Elevation Levels
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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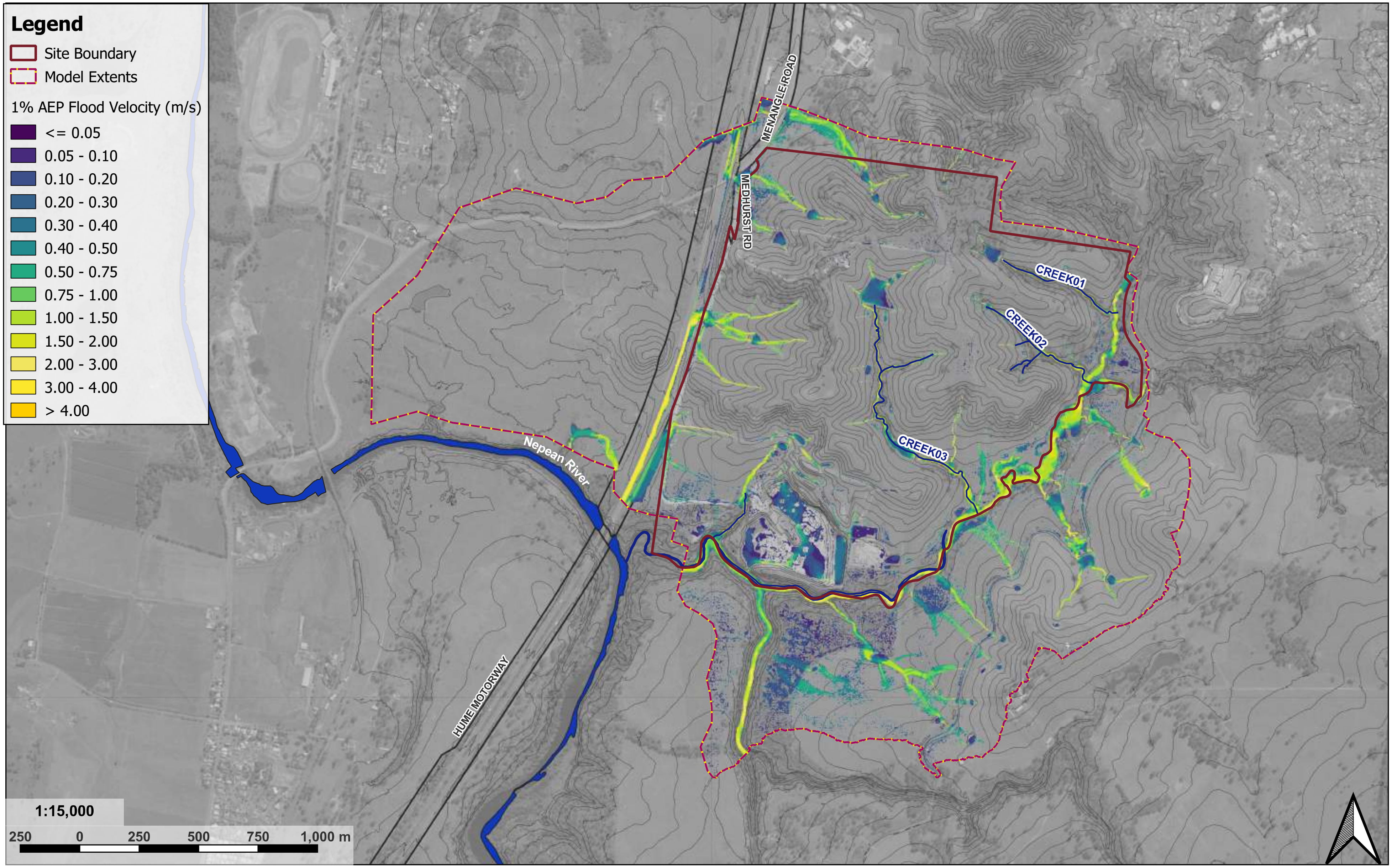
Appendix C
1% AEP Developed Case Water Surface Elevation Levels
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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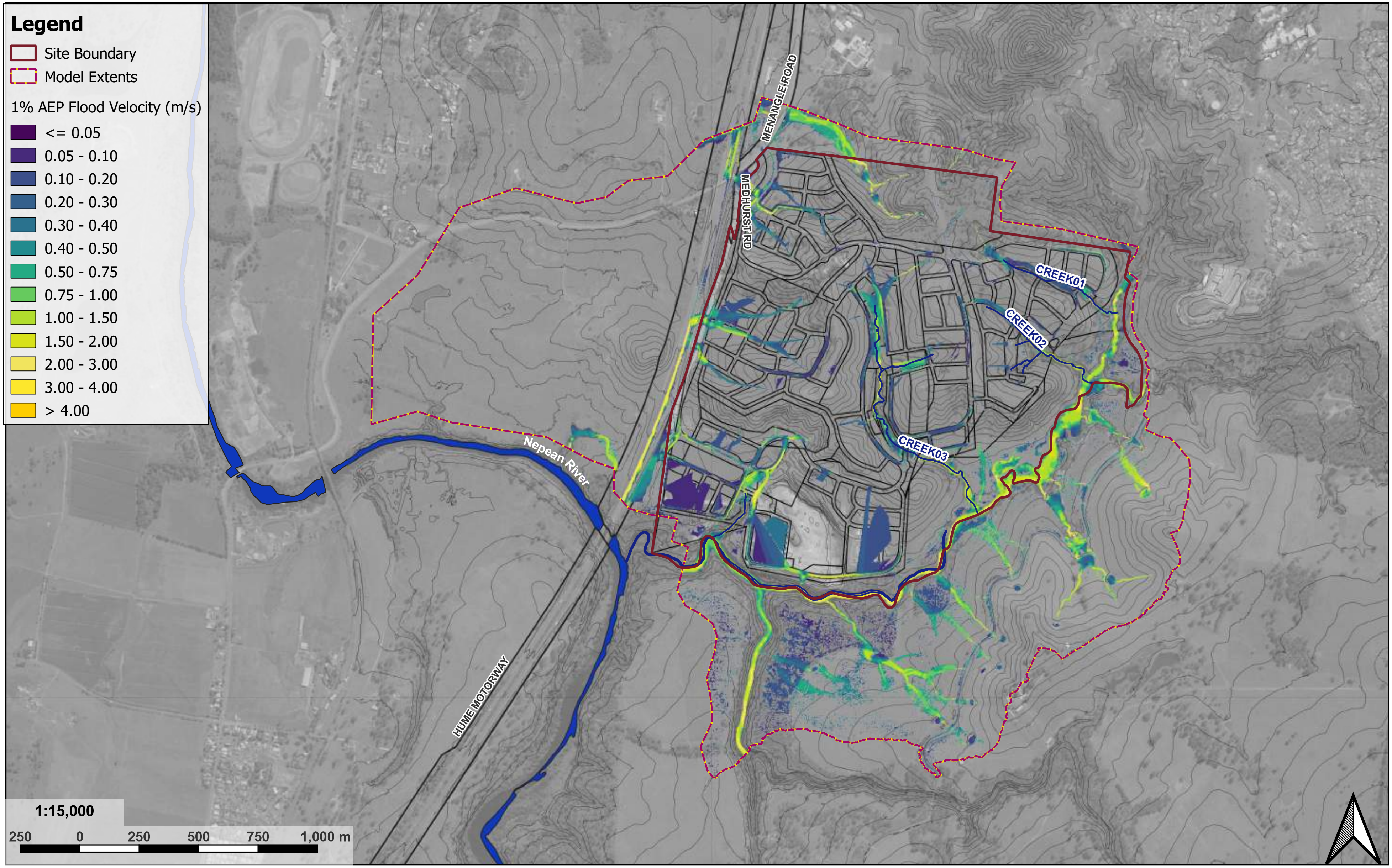
Appendix C
1% AEP Existing Case Flood Velocity
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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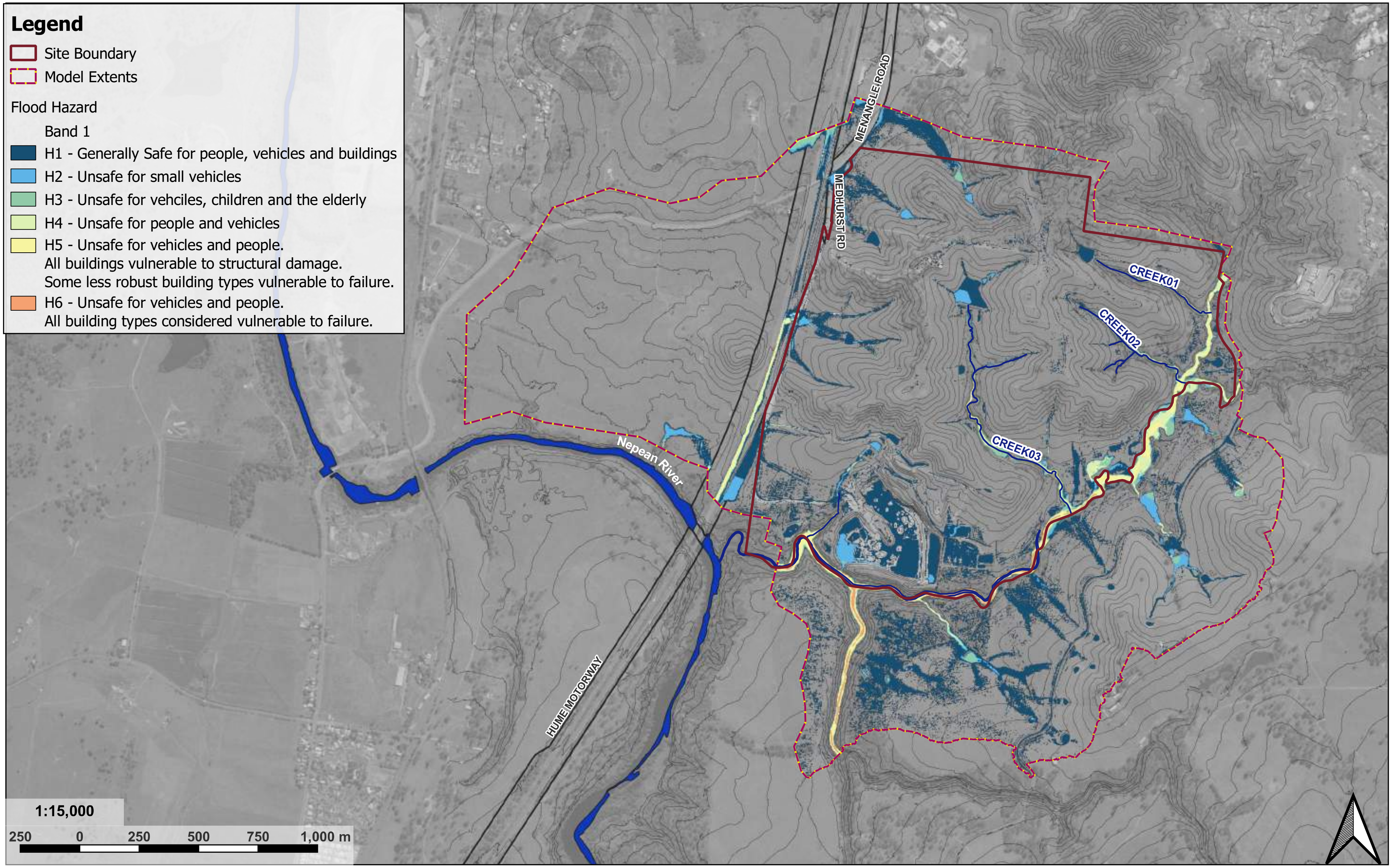
Appendix C
1% AEP Developed Case Flood Velocity
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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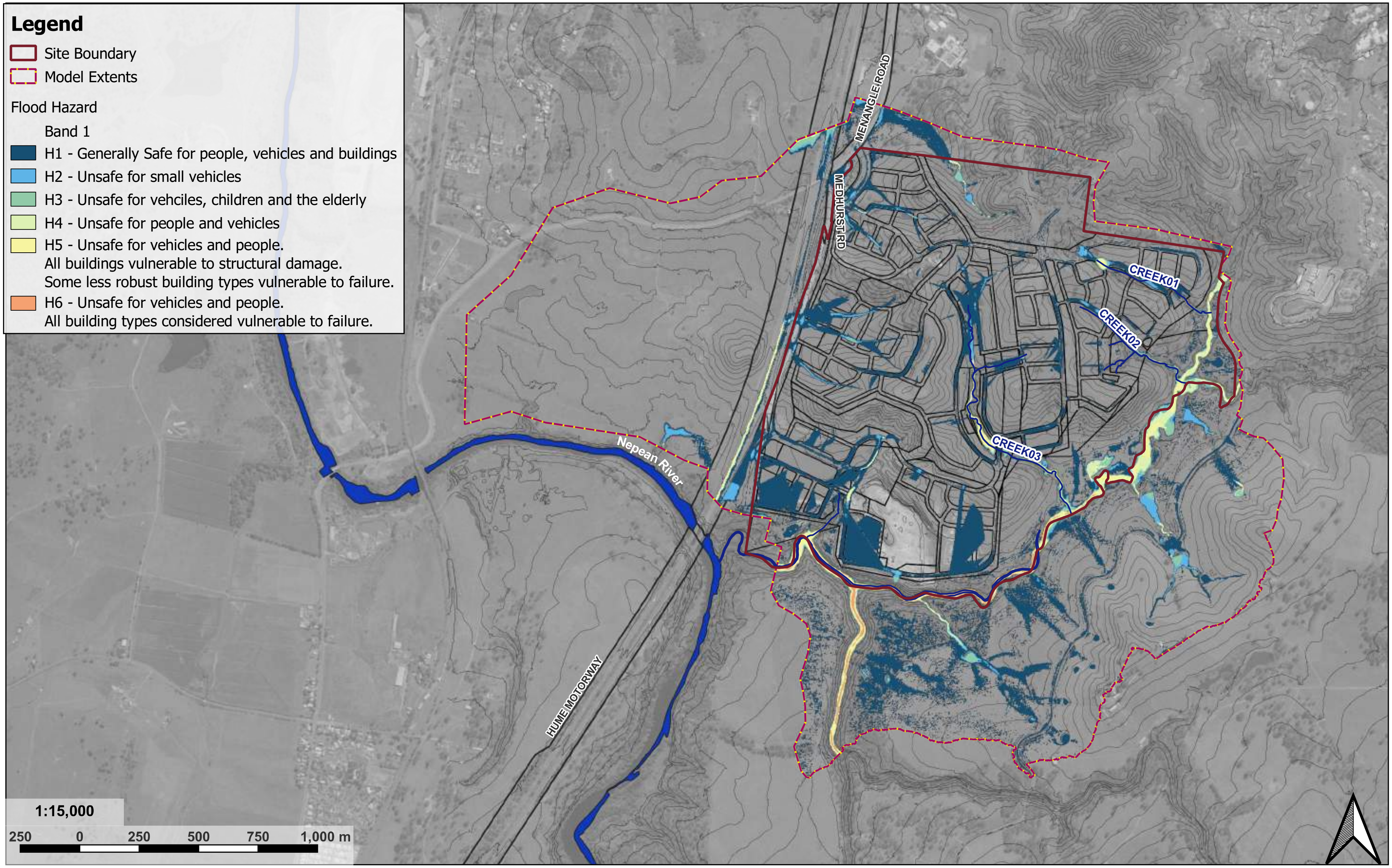
Appendix C
1% AEP Existing Case Flood Hazard
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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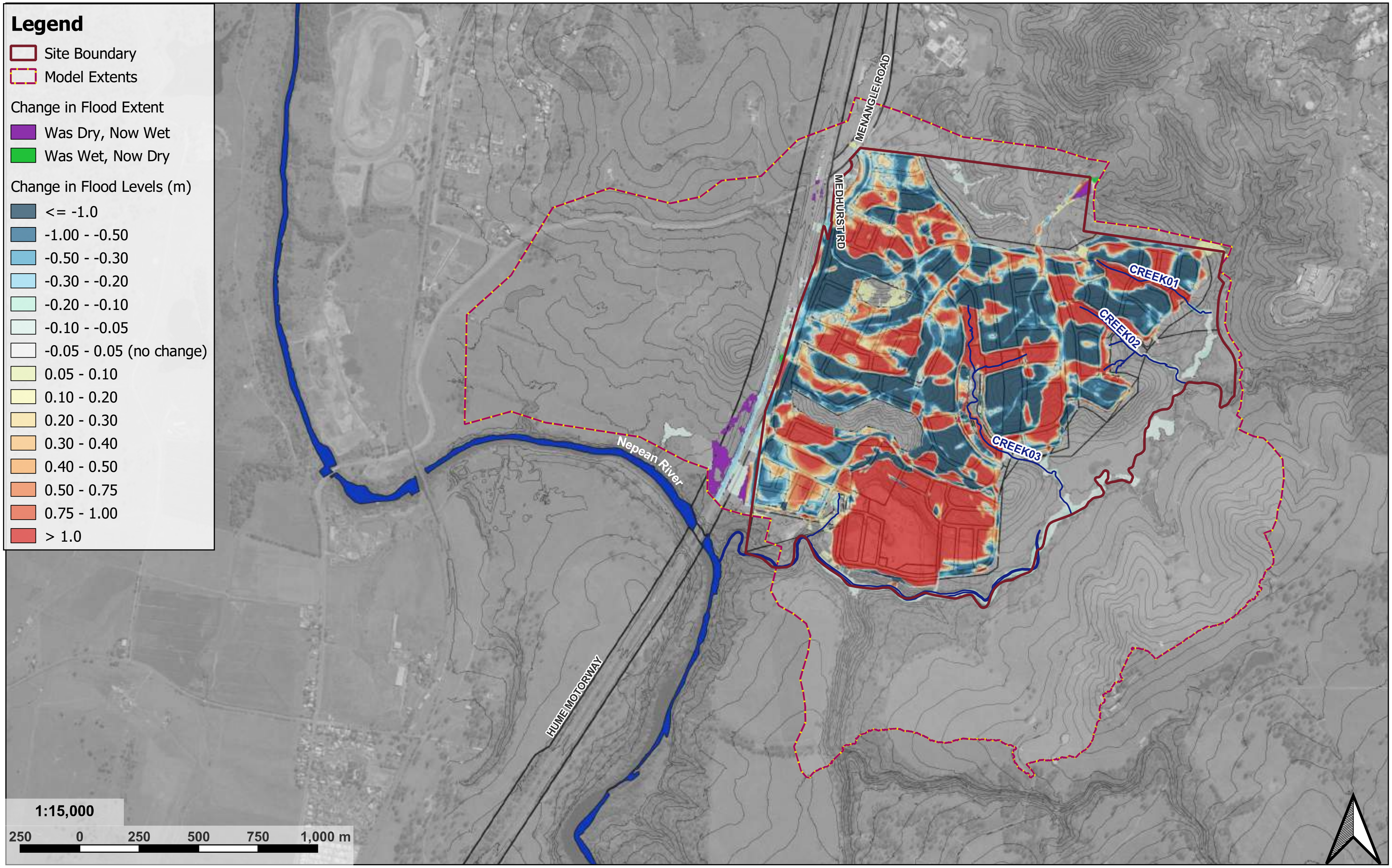
Appendix C
1% AEP Developed Case Flood Hazard
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd



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Appendix C
Change in Flood Levels / Extent between Existing and Developed
Project: Medhurst Feasibility Study
Project Number: 467-21
Client: Leda Design & Construction Pty Ltd