

Farrar West

Stormwater Management Plan Part 1

Project No: 17-182



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Revision	Description	Author	Date
0	Issued for Development Application	JC	25 September 2026

This memo provides an assessment of stormwater to support the development application, focusing on on-site drainage conditions and their integration with future infrastructure development.

1 Introduction

1.1 Background

This Part 1 Stormwater Management Plan summarises the stormwater analysis and design for the Farrar West Project, located in Palmerston, Northern Territory. The project proposes to subdivide the existing gazetted and undeveloped lot into approximately 250 individual lots.

The development proposes to include stormwater components, such as pits and pipe networks, swales, a detention basin, culverts to cross the access roads, and rock protection sized to accommodate the designed discharge and velocities. The design is intended to interface with existing drainage infrastructure both upstream and downstream of the Farrar West Local Catchment.

Colliers Engineering and Design has been engaged to undertake Civil Engineering design for the Farrar West development. In conjunction with this, WRM water and environment are undertaking the flood assessment and hydrological modelling for the site. Colliers has coordinated design aspects closely with WRM environment, such that results generally converge, and the required flood modelling outputs are achieved. It is noted that although the civil design considers the major event for the local design, reference should be made to the flood impact assessment and hydrological model with respect to the performance of the major event in the context of the broader catchment.

The proposed site is shown in Figure 1.



Figure 1: Site Subject Locality

1.2 References

- Australian Rainfall and Runoff (AR&R), 2024
- Australian Rainfall and Runoff (AR&R), Data Hub
- Austroads Guide to Road Design (AGRD)
 - Part 5: Drainage – General and Hydrology Considerations (2021)
 - Part 5A: Drainage – Road Surface, Networks, Basins and Subsurface (2021)
 - Part 5B: Drainage – Open Channels, Culverts and Floodways (2013)
- Bureau of Meteorology Hydrological data
- DLI Standard Drawings
- NTG Subdivision Development Guidelines, 2025
- NTG Subdivision Development Guidelines, City of Palmerston Variation, 2025
- FHWA Hydraulic Design Series No. 5 (HDS-5) – Hydraulic Design of Highway Culverts, as implemented in HY-8.
- Queensland Urban Drainage QUDM 2017.

2 Design Criteria

The design criteria applied to the stormwater analysis and design are summarised below.

Table 1: Design criteria

Component	Adopted Criteria	Source	Comments
Pits and Pipe Networks			
Minor storm event	50% AEP	NTG SDG, 2025	Underground pit and pipe network
Major storm event	1% AEP	NTG SDG, 2025	Overland flow paths, swales, detention and building immunity
Rainfall data	IFD data	BoM website	
Hydrologic model	IL-CL method	AR&R 2019	
Initial Loss	25.0 mm Pervious 1.0mm Impervious	ARR Data Hub	
Continuing Loss	4.1 mm/hr Pervious 0.0mm Impervious	ARR Data Hub	
Sag pit blockage factor	100%	NTG SDG, 2025, CoP Variation, 2025	
Tailwater condition	Assume tailwater levels at obvert.		
Culverts and Crossings			
Culvert design	No overtopping the road in major storm event 200mm freeboard to edge of street shoulder	NTG SDG, 2025	Inlet and outlet control assessed in HY-8.
Minor storm event	5% AEP	NTG SDG, 2025	200mm freeboard to road edge

Component	Adopted Criteria	Source	Comments
Major storm event	1% AEP	NTG SDG, 2025	
Swale			
Geometry	2.0m base width minimum, batters no steeper than 1V:6H	NTG SDG, 2025	
Surface	Typical surface finish shall be grass lined channels Typical Manning's n = 0.04	ARR Book 6	
Flow depth	Maximum flow depth = 750mm	NTG subdivision development guidelines	For open drains. Where flow depth exceeds this value, this will be highlighted in this report for approval by the relevant authority.
Depth x Velocity	Maximum of 0.4 m ² /s (minor event) Maximum of 0.6 m ² /s (major event)	NTG subdivision development guidelines	For open drains. Where DV exceeds this value, this will be highlighted in this report for approval by the relevant authority.
Freeboard	Minimum freeboard to be a maximum of the following: 300mm 20% of channel depth	NTG subdivision development guidelines	
Grade	Desirable Minimum 0.5% Absolute Minimum 0.3%	NTG subdivision development guidelines	
Detention Basin			
Discharge control	Post-development peak discharge ≤ pre-development peak discharge at each existing discharge point		
Scour Protection			
Outlet scour protection	Rock protection sized for the outlet velocity in the major event		

3 Existing Conditions

3.1 Flood Assessment

Flood assessment for the catchment has been completed by WRM. Refer WRM flood modelling report, which is included for reference in Appendix One. Modelling indicates significant flood extents, depths and velocities on the eastern side of Lots 13256 and 13257.

3.2 Existing Site and Catchment

The site generally falls from the north to the south, with the Farrar West catchment being located at the top of the stormwater system that discharges into Mitchell Creek to the South.

A number of studies have been conducted on the existing catchments to inform the Roystonea Avenue duplication project and for the subdivision itself. The wider area enlarged catchment plan completed by Aurecon was completed for the Roystonea Avenue duplication project. The project specific pre-development catchment plan completed for the Farrar West is consistent with the Aurecon studies for overall catchment areas, with the high level plan show below in Figure 2.

The existing greater catchment at Farrar West commences from the Yarrawonga commercial area to the west and continues to the Mitchell Creek Estate residential area to the east. The catchment also includes the runoff crossing Temple Terrace, flows along Farrar Boulevard, runoff from the Palmerston Health Precinct, areas north of the median on Roystonea Avenue and the residential area of Farrar to the east.

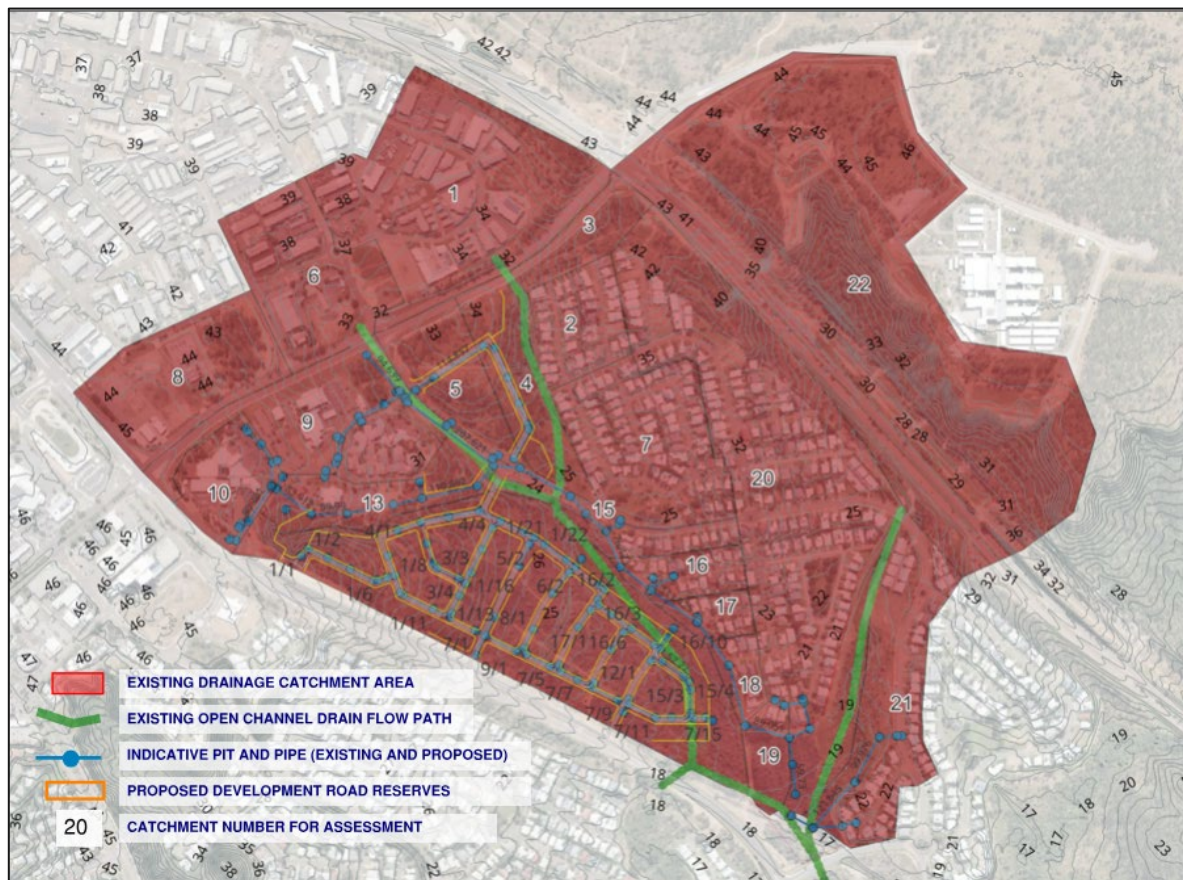


Figure 2: Existing Catchment Plan

A pre-development catchment and drainage plan has been prepared and is included within Appendix Two, which provides further details.

3.3 Existing Catchments – Hydrology

Assessment of existing conditions and proposed design has been undertaken in DRAINS hydrological/hydraulic design software, using parameters described in section 2, and described further in section 3.5. A summary for the runoff for respective catchments for both the minor and major events is shown below in Table 1, corresponding with the catchments as indicated in Figure 3

Table 1: Existing catchment stormwater runoff estimation

Catchment No.	Area (Ha)	Minor Event (50% AEP) m ³ /s	Major Event (1% AEP) m ³ /s
1	16.7	2.9	7.3
2	3.8	0.8	2.0
3	2.5	0.3	0.8
4	4.3	0.5	1.2
5	4.2	0.6	1.4
6	7.4	1.5	3.8
7	8.5	1.8	4.5
8	7.4	1.4	3.4
9	5.7	1.0	2.6
10	3.4	0.7	1.8
11	13.2	1.5	3.6
12	2.6	0.5	1.4
13	4.3	0.5	1.2
14	10.2	1.1	2.8
15	2.8	0.6	1.5
16	2.1	0.4	1.1
17	1.3	0.3	0.7
18	1.3	0.3	0.7
19	5.3	1.1	2.8
20	25.6	4.3	10.8
21	8.5	1.4	3.6
22	40.1	4.4	11

There are small detention basins located at the outlets of the Farrar residential areas from the north-east. The Yarrawonga and Palmerston Health Precinct areas north-west of the site do not have any detention basins.

Currently stormwater from pre-development catchment areas running through the Farrar West site discharges across Roystonea Avenue through 3 1200 x 900 RCBCs. There is an overflow open drain connecting to the Roystonea culverts that has a crest level of approximately 18.40m. For larger storm events which exceed the capacity of the Roystonea culverts, this flow will surcharge with overflow discharging to the south overflowing the drain crest towards Inverway Circuit, where there are noted to be 4 x 4650mm x 900mm Box Culverts under Inverway Circuit heading east.

It is noted that the Farrar site occurs within the upper sections of the extensive Mitchell Creek catchment, recognised as a sensitive catchment that has had much assessment and effort to retain its natural function. The main features of the Farrar West site include open drainage lines/tributaries following the natural low flow paths through the vegetated site towards Roystonea Avenue and Inverway Circuit.

The existing undeveloped site consists of medium or dense bushland, wholly pervious of assumed medium permeability. The site currently consists of two main drainage flow paths, with flows from Yarrawonga, Health Precinct and northern areas via Temple Terrace through a significant open drain along the eastern side of lot 13257 (north of Farrar Blvd), which crosses under Farrar through a series of culverts to then follow the main drain flow path through the proposed development site as shown in Figure 3. This is generally referred to as the western flow path through this document. There is a second flow path originating north of Stuart Hwy that flows predominantly through a grassed open drain along the north-eastern section of the existing Farrar stage A and B Development, where it crosses under a bridge and opens into natural drainage flow path through woodland heading towards the culverts under Inverway Circuit, shown as catchments 22, 21 and 20 in Figure 3. This is referred to as the eastern flow path. The high flow channel through the site discharging into the Roystonea Avenue culverts is overgrown with existing vegetation, which will restrict the inflow to the culverts compared to a post development channel of comparable cross section.

There is a third flow path from the north-west from the Health Precinct area entering the site via pipe outlets from Gurd St, which run along the northern side of the lot 13256 at the back of Brennan Court. This flow path is considered within the proposed design, however has minor flows compared to the other flow paths, where any surcharge or overland flow discharges to the western flow path and main drainage channel through the site, and therefore is considered within the western flow path.

Existing catchments 20, 21 and 22 discharge directly into the Inverway Circuit culverts, and have been included in the assessment as the catchments and discharge points combine in both minor and major events when the Roystonea Avenue culvert reaches capacity. The proposed development design also plans to implement a detention basin to converge these flow paths and assist flow control and sediment management prior to discharge to the downstream culverts.



Figure 3: Existing Channel



Figure 4: Roystonea Ave Channel

3.4 Culverts

There are 4 major culverts within the catchment area which have been inspected, reviewed and assessed for hydraulic capacity. Two of the culverts have been included in the Aurecon report for the duplication of Roystonea Avenue, T20-2421, culverts 2 and 3, which function as the points of discharge for the proposed subdivision.

Culvert capacity assessment has been undertaken using software HY-8, with inflow rates as determined by catchment assessment described above.

Culvert Ref.	Roadway / location	Culvert Size / Type	Proposed works
Cul 1	Stuart Highway – 760 m east of Temple Terrace	2 No. 1650 RCP	No change. Has adequate capacity
Cul 2	Roystonea Avenue – 310 m west of Inverway Cct	3 No. 1200 x 900 RCBC	No change. Cannot be increased in size otherwise flooding of Buckingham Street would occur
Cul 3	Inverway Circuit – 120 m north of Roystonea Av.	4 No. 3750 x 900 RCBC	No change. Has adequate capacity
Cul 4	Buckingham Street	6 No. 1800 x 600 RCBC	No change. Has adequate capacity
Cul 5	Lakeview Boulevard	3 No. 2400 x 600 RCBC & 2 No. 2700 x 750 Links	No change. Has adequate capacity
Cul 6	Roystonea Avenue – 180 m north of Buscall Av.	7 No. 1200 x 1200 RCBCs & 4 No. 1200 x 1300 Links	No change. Culvert length is adequate to accommodate new carriageway
Cul 7	Roystonea Avenue – 460 m north of Lambrick Av.	3 No. 2700 x 1500 RCBCs & 1 No 2700 x 1650 Link	No change. Culvert length is adequate to accommodate new carriageway

Figure 5: Existing Culverts (Aurecon T20-2421, 2023)

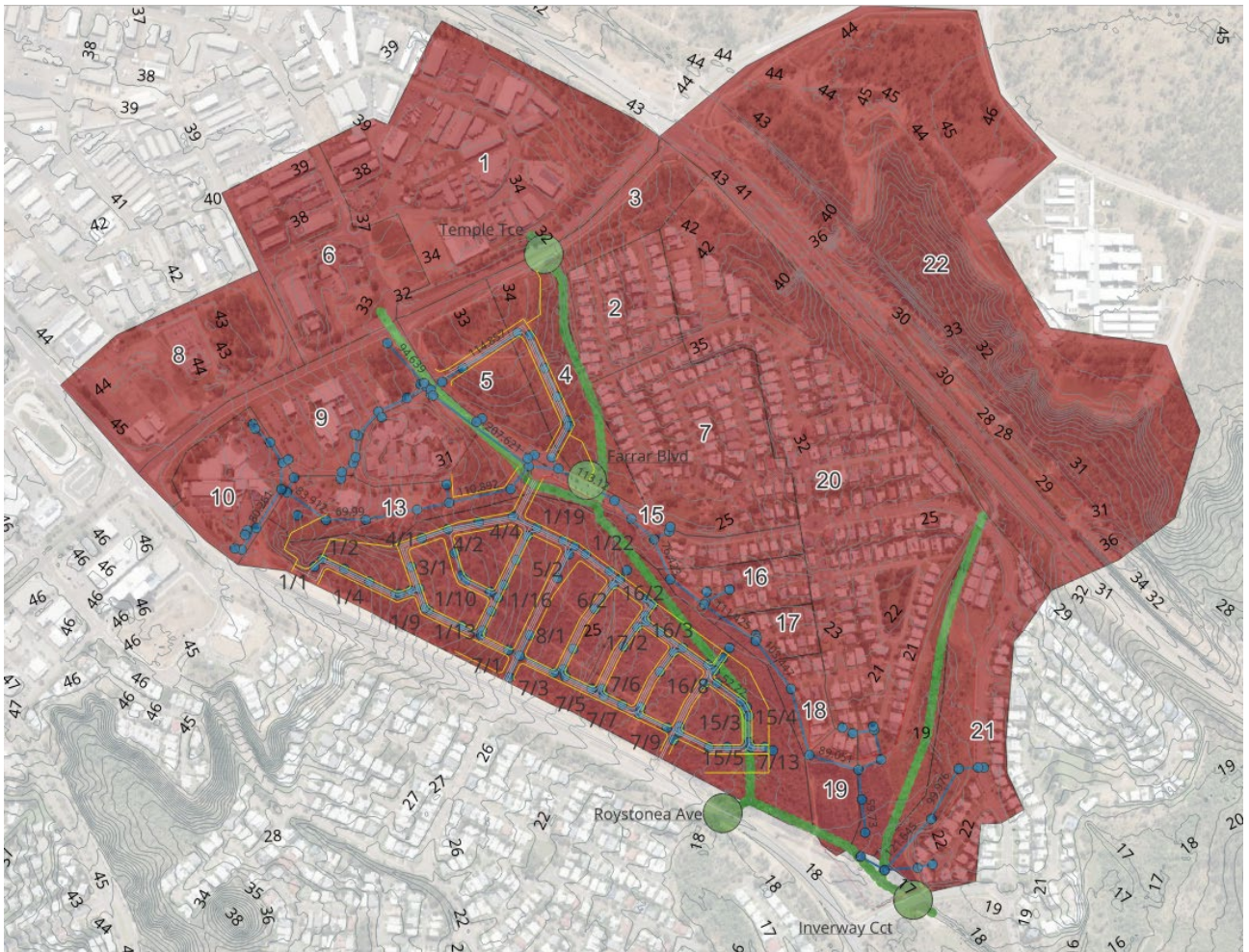


Figure 5: Culvert Locations

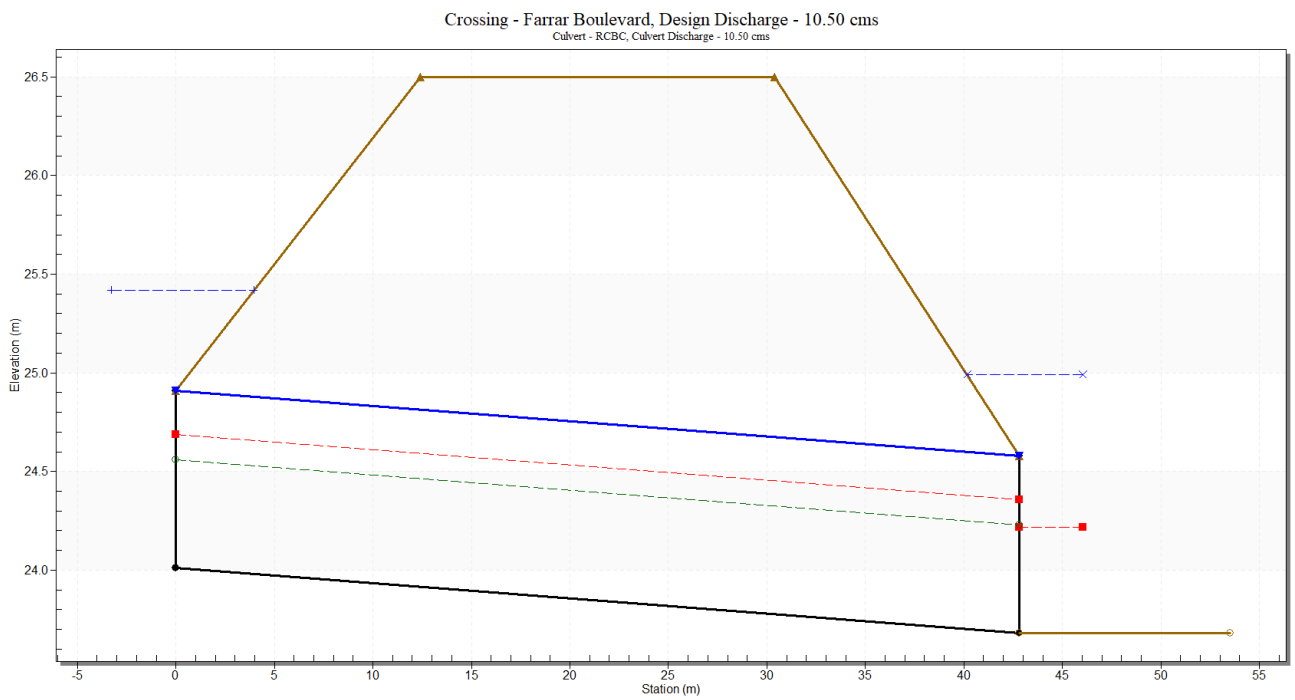
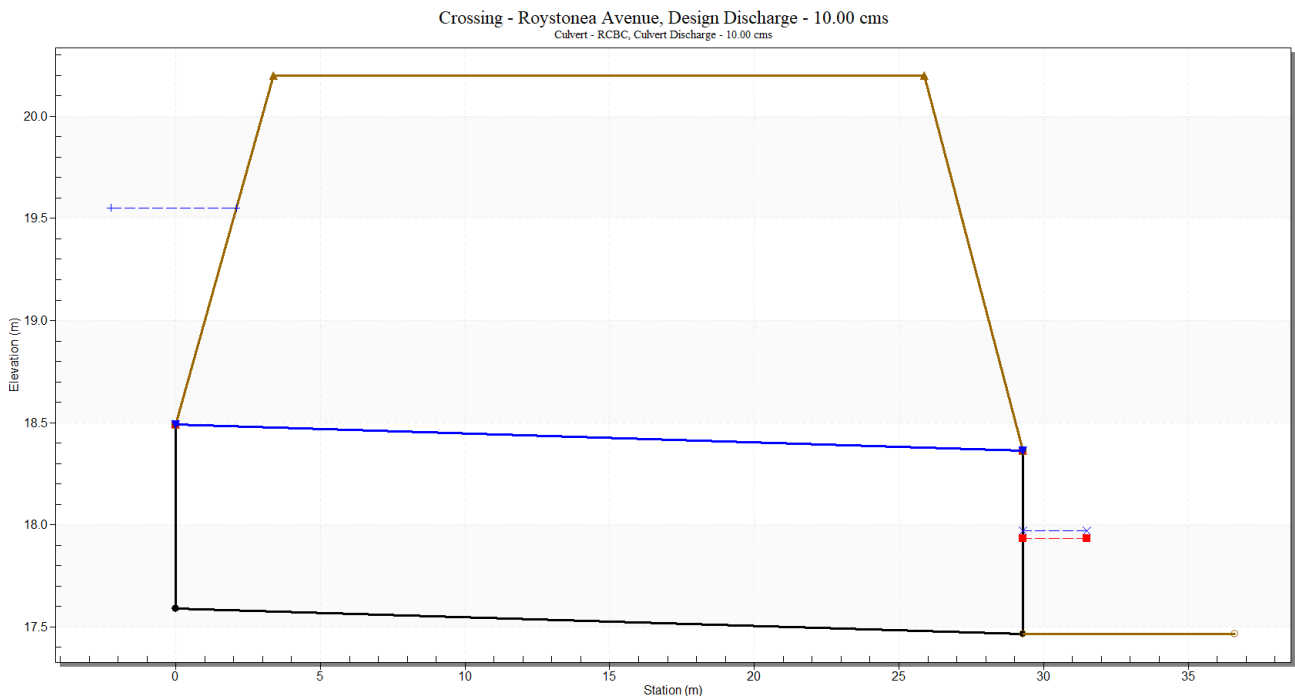


Figure 8: Roystonea Ave Culverts

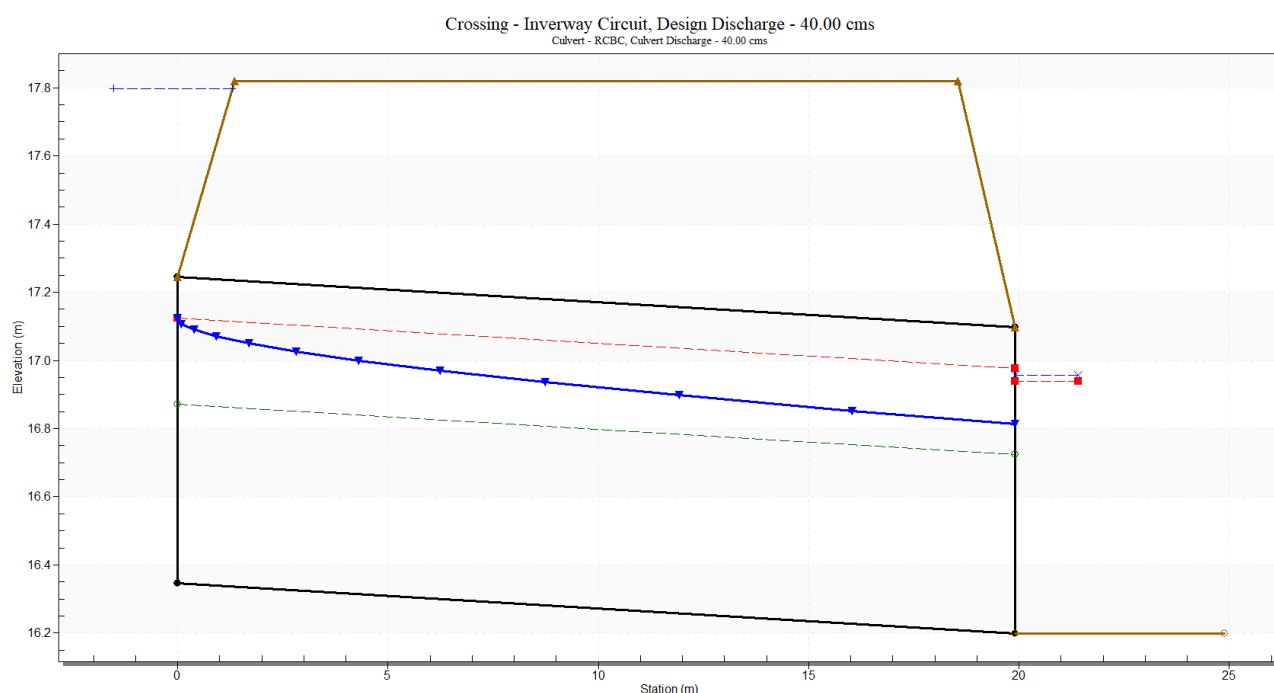


Figure 9: Inverway Circuit Culverts

Culvert	Arrangement	Existing Capacity (m ³ /s)	Overtopping Level (m AHD)	1% AEP Pre-Development Flows (m ³ /s)
Temple Tce	1 x 1350 RCP 1 x 450 RCP	6.7	33.8	4.9
Farrar Blvd	5 x 1200 x 900 RCBC	16.4	26.5	10.3
Roystonea Ave	3 x 1200 x 900 RCBC	10.3	20.2 (18.4 spillway*)	10.3**
Inverway Cct	4 x 4650 x 900 RCBC	40.6	17.8	37.

*Per Aurecon design report T20-2421

**Capacity Constrained. Upgrade may result in downstream flooding.

The existing culvert on Roystonea Avenue is currently capacity constrained, and any upgrade is likely to increase the likelihood and severity of downstream flooding on Buckingham Street. Per Aurecon design report T20-2421, this culvert is suitable for a flow equivalent to the 0.5EY (1 in 2-year ARI) flow. Larger events overtop the spillway to the south and flows through the Inverway Cct culverts.

The existing Temple Tce and Farrar Blvd culverts appear to have adequate capacity. Further reference shall be made to the flood and hydrology report where there may be alternative outputs due to broader considerations.

In order to achieve the proposed development layout and establishment of the main Public Open Space (POS) area proposed along Farrar Boulevard, it is required to rearrange the existing drainage infrastructure in the vicinity of Farrar Blvd including the main 1650mm drainage pipe running east along Farrar Boulevard, along with the 5 x 1200 x 900 culverts crossing Farrar. Realignment of the existing culverts on Farrar Blvd are currently being considered within the headworks scope being delivered by NTG, however, is subject to further coordination and confirmation..

3.5 Hydrology and Hydraulic Modelling

3.5.1 Hydrology

An IL-CL hydrological model has been prepared to estimate design discharges at key locations for the adopted minor and major storm events. The model uses the site IFD data and sub-catchment parameters to generate peak flows, HGL, and velocities for the pit and pipe network, swales, culvert crossings and detention basin assessment.

Procured date	Northing	Easting
17 July 2026 (BoM)	12.4875 (S)	130.9875 (E)

Culverts and Crossings		
Event	Major (1% AEP), mm/hr	Minor (50% AEP), mm/hr
5min (mm/hr)	270	154
10min (mm/hr)	231	128
1hr (mm/hr)	99.5	55.8
2hr (mm/hr)	65.4	34.9
24hr (mm/hr)	16.4	5.65
72hr (mm/hr)	8.72	3.07
IL-CL Model Parameters	Initial Loss	Continuing Loss
Impervious Area	1mm	0mm
Pervious Area	25mm/hr	4.1mm/hr

3.5.2 Hydraulic

Existing information at the following areas had been provided by the relevant authorities to inform the pre-development drainage model:

- Roystonea Road Duplication Design (2022)
- Yarrawonga Stage 7 (2001)
- Palmerston GP Superclinic & Early Childhood Centre (2009)
- Palmerston Health Precinct (2000)
- Site Survey (2026)
- Roystonea Avenue and Temple Terrace Intersection Redevelopment Stage 2 (2017, partial)

Modelling philosophy:

- The pit and pipe network input into DRAINS had been taken from the available information outlined above. Where information is insufficient to fully capture existing conditions, assumptions were made from first principles or inferred from other existing information packages.
- The existing network was analysed in DRAINS, including inlet capacity, pipe capacity, hydraulic grade lines, ponding and overflow routes.
- The four culvert crossings have been checked in HY-8 using peak flows from the DRAINS hydrology model and cross checked against the flood model.

Predevelopment site conditions for the proposed development site were observed to be open space, with medium density bushland. Medium soil permeability has been assumed. The existing pit and pipe data information received from the City of Palmerston and DLI has been incorporated into the model in order to assess flow contributions into the site and understand existing outlet pre-development values. The key nodes assessed that input to stormwater flow through the site as part of the hydrological model are shown below in



Figure 11: Outflow Nodes

Table 2 below. The corresponding flow hydrographs at the major storm event for each of these nodes is included below in Figure 11 to Figure 15, depicting flow characteristics and a typical time of concentration for the respective nodes, dependent on contributing catchments.



Figure 11: Outflow Nodes

Table 2: Pre-development peak flows at key nodes for minor and major events

Outlet	Minor Event (m ³ /s)	Major Event (m ³ /s)
Temple Terrace	2.7	4.9
Farrar Boulevard	4.9	10.3
Roystonea Avenue	7.8	9.5
Inverway Court	17.2	37.9
Low Flow Inverway Court	1.3	1.7

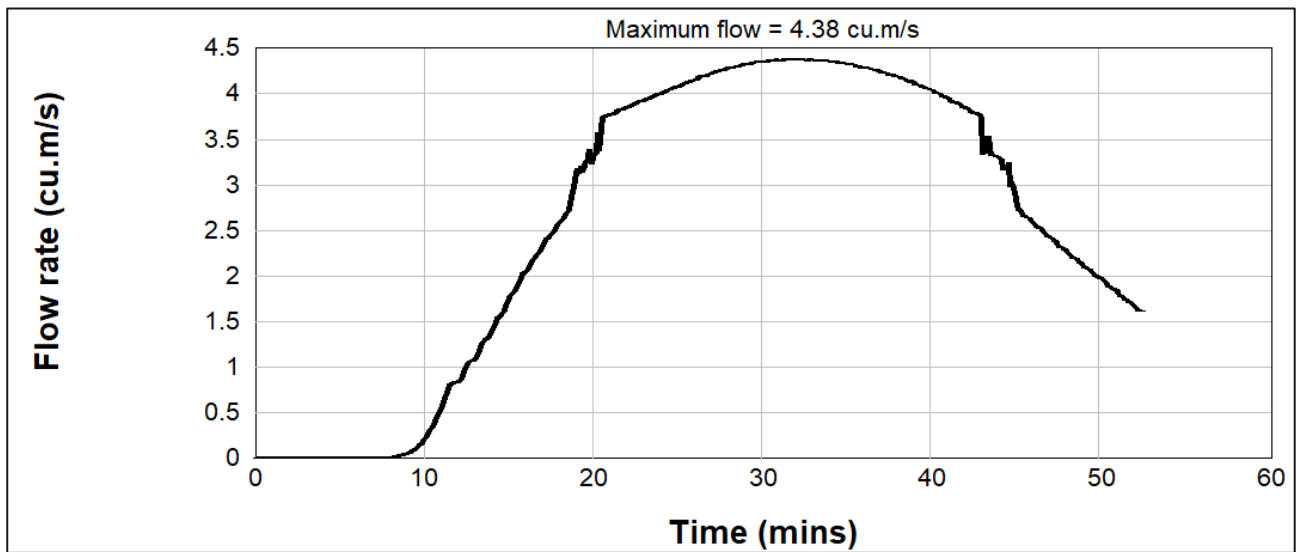


Figure 12: Temple Tce

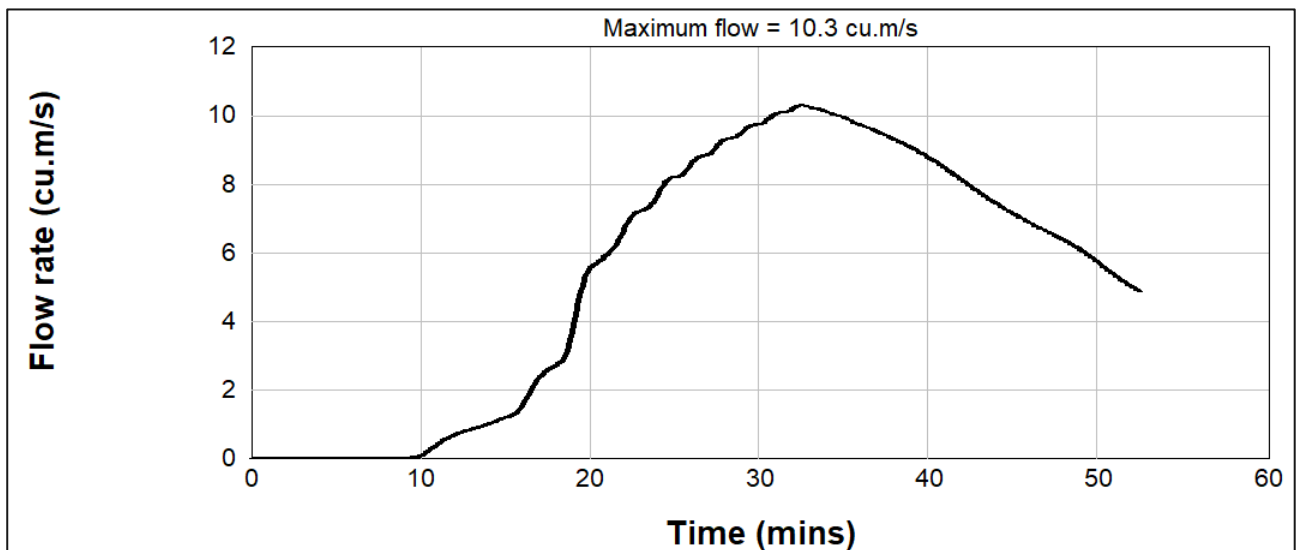


Figure 13: Farrar Blvd

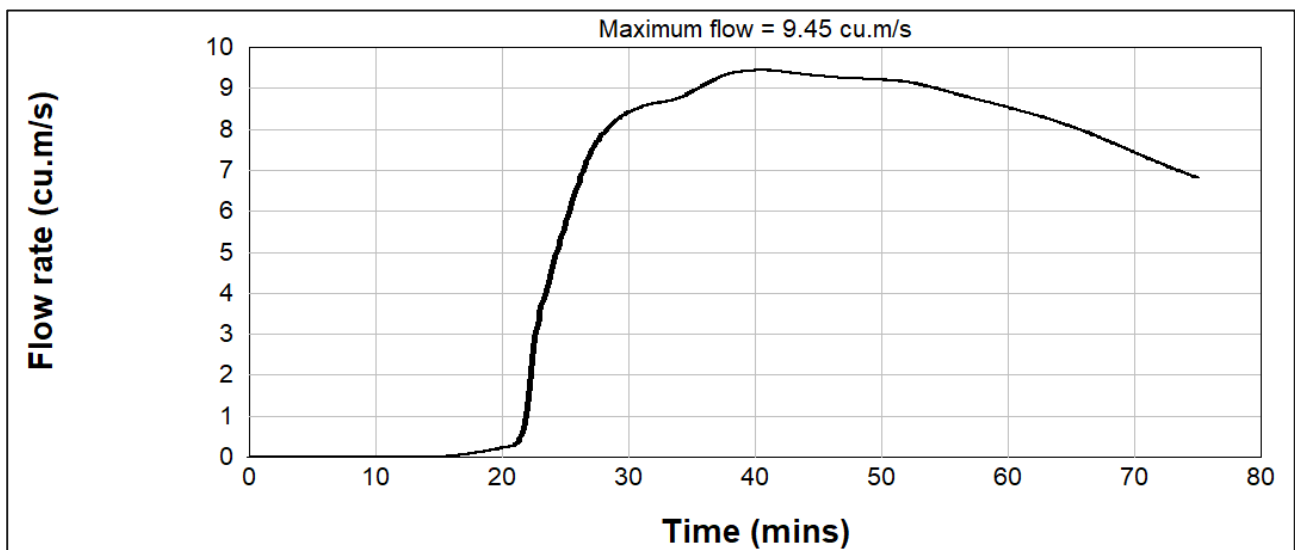


Figure 14: Roystonea Ave

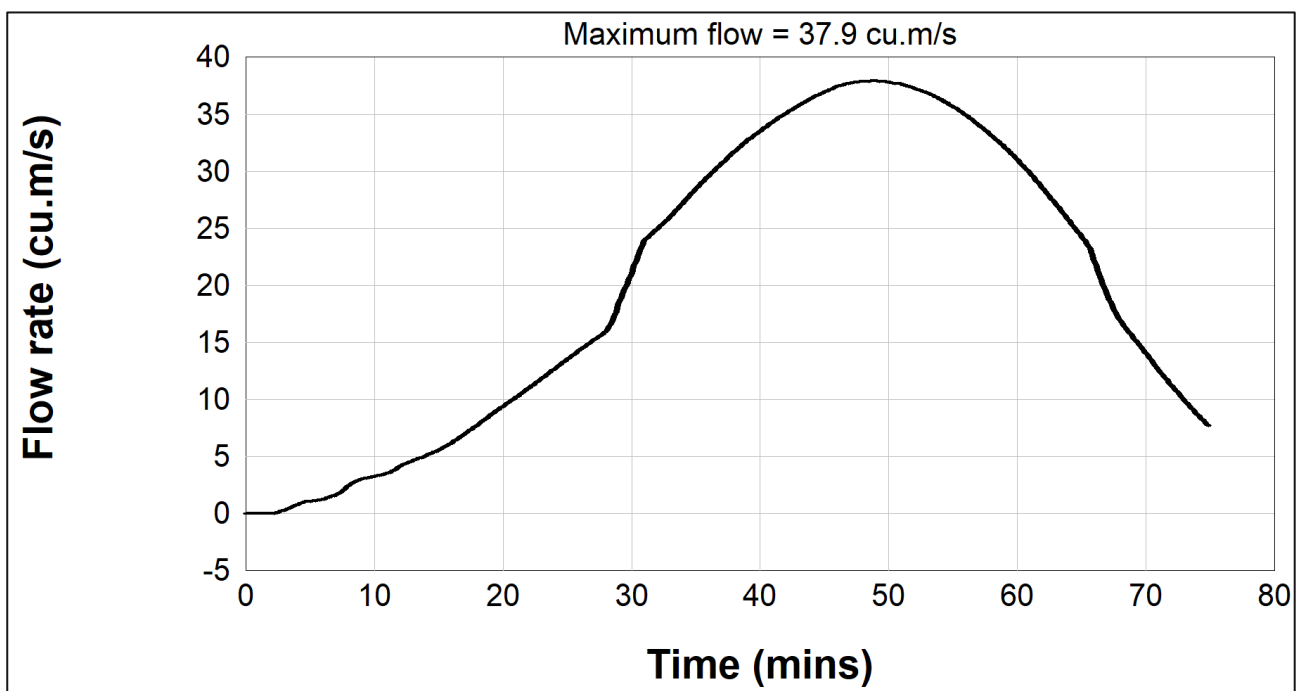


Figure 15: Inverway Cct

4 Post Development Conditions

4.1 Catchments

The proposed development includes roadways, and developed residential lots, which will increase the degree of imperviousness across the site and therefore increase stormwater runoff heading towards the ultimate discharge points of Roystonea Ave and Inverway Circuit. A post development catchment plan and proposed overall drainage layout has been prepared for the site and included in Appendix Three and Four. These have been carried out in coordination with DRAINS model iterations, with input catchments, pit and pipe layouts, outlets, open channels and overflow routes in order to assess discharge conditions and ensure that post-development outflow is not increased from pre-development conditions.

Typical sub-catchment sizes are determined by capacity and distance between inlet pits. Catchments are graded towards the frontages of the lots and discharge onto either the kerb and gutter or directly into the underground system.

There is a general increase in impermeability and corresponding decrease in the time of concentration and estimated discharge. The increase in runoff volume will be accommodated through the implementation of a new detention basin downstream to constrain overall flows at the nominated discharge points to match or improve on the existing conditions. As shown on development layout plans, the detention basin now intersects both flow paths from the west and east sides of the development, which will effectively slow flows prior to reaching the outfall culverts, and assist with sediment and nutrient control.

The detailed proposed site catchments plan is shown in Appendix Three.

4.2 Stormwater Strategy

The proposed development includes building pads, access roads, and a car park, which will change the site's imperviousness, runoff coefficient, flow paths and time of concentration. The stormwater in the developed area will be managed by a system of pit and pipe networks, swales, culverts, and outlet protection. The additional peak flow resulting from a change in surface runoff will be controlled by a detention basin. The developed lots have been split into new sub-catchments for use in the DRAINS model to track and manage stormwater.

The stormwater strategy is summarised as follows:

- Minor storm event
 - The pit and pipe network captures surface runoff from the proposed subdivision, with hydraulic grade lines to remain below surface levels with adequate freeboard. This shall be reviewed and refined during subsequent design and as part of the staged design and accurate levels refinement.
 - Public Open Space areas shall remain free from surface flow paths with adequate freeboard.
 - Collected flows discharge safely to nominated flow paths, underground networks or drainage swales.
 - Culverts are checked for minor event headwater, outlet velocity and road overtopping risk.
 - Outlet rock protection is provided where pipe and culvert discharges enter the swales, dimensions to suit modelled velocities.
 - Buildings and lots are intended to grade towards the street network and generally will include lot connection pits and pipework into the street drainage system. These details will be confirmed during detailed design and development.
- Major storm event
 - Blockage factors for all pits set to 1.0 (fully blocked, as per CoP addendum to NTG SDG).
 - Pit overflows and overland flow paths are directed away from buildings and toward the swale network.
 - Habitable buildings maintain 300mm freeboard in the major event.
 - All major flow paths maintain 300mm/20% channel depth freeboard in the major event.
 - The swales are to convey the 1% AEP flow with adequate freeboard.

- Culverts are checked for major event headwater, outlet velocity and road overtopping risk.

The detention basin has been sized using Austroads methods and first-pass hydrology and subsequently in DRAINS and flood modelling. Final sizing shall be confirmed during detailed design for the Part 2 Stormwater Management Plan in conjunction with the civil topographic design. Preliminary earthworks modelling has been completed for the site, and the basin modelled accurately at this point. This surface has been incorporated into the flood model completed by WRM and the outputs checked for consistency, with both the local hydraulic model and flood model outputs showing compliance with no worsening post-development compared to pre-development conditions.

At the time of flood modelling, feature survey information was not completed for Farrar Boulevard, and Lidar information has been used in this area. This will be updated with accurate survey information in subsequent design.

Outlet scour protection shall be sized using the Catchments and Creeks rock sizing guidance based on the adopted outlet discharges and velocities in the Part 2 Stormwater Management Plan.

Where available, all stormwater infrastructure including pits, pipes and subsoil drainage shall be as per DLI or NTG SDG Standard Drawings and Specifications, typically minimum DN375 RCP pipes at 0.5% minimum grade and standard DLI side entry and grated inlet pits. Kerb profiles shall be as per DLI standard drawings, with kerb types to be confirmed in subsequent design and as required by the City of Palmerston.

The major drainage channel on the eastern side of the subdivision has been sized at 20m base width, and 1:6 batters. Overall typical physical depth of the channel is 2.0m, with an additional shoulder at the top separating the flow path and property boundary. Typical flow depths in the channel for the major event are approximately 600mm. A typical output of the main drainage flow is included below in Figure 16 below.

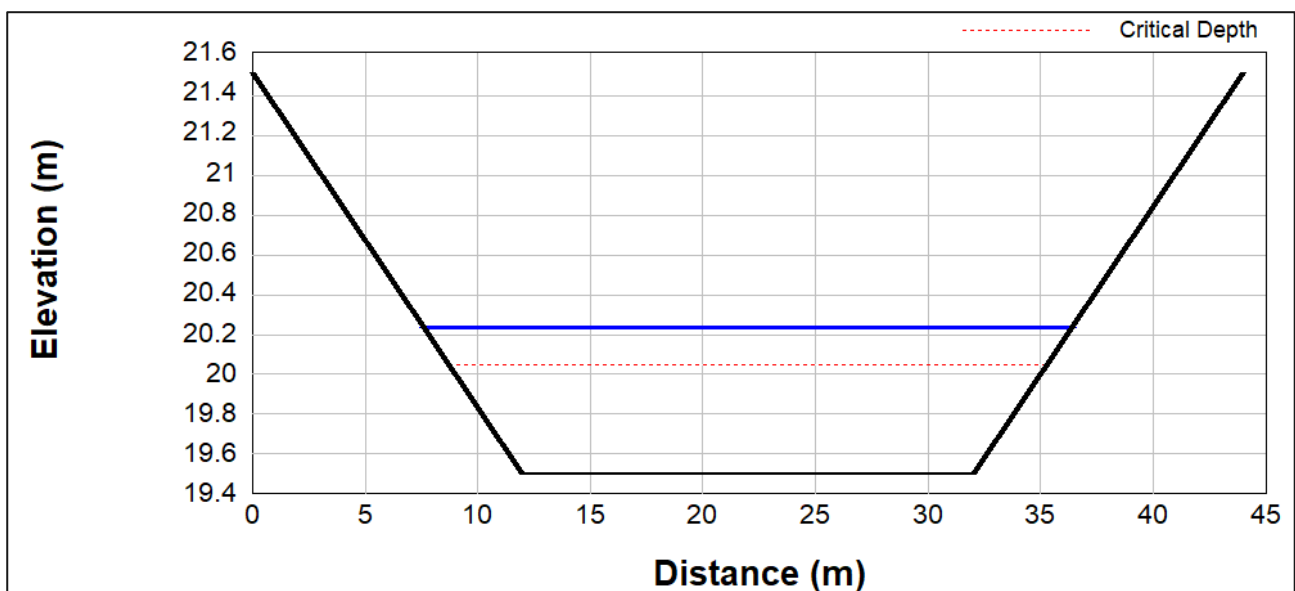


Figure 16: Typical Channel Cross Section

4.3 Culverts

Existing culverts are generally retained in the design. The existing culverts on Temple Tce, Roystonea Ave and Inverway Cct shall be retained in their current configurations. A capacity check with DRAINS and HY-8 was completed on the existing culverts to confirm sufficient capacity is available in the system.

The culvert on Roystonea Avenue is undersized and cannot accommodate the total discharge coming from the upstream catchment in either pre or post development conditions. Once the hydraulic capacity of the culverts have been reached at Roystonea Avenue, the flow paths overtop a local crest and diverts to the Inverway Circuit culverts. Upgrade of the Roystonea Avenue culverts have not been proposed, as it may increase downstream flooding on Buckingham Street per Aurecon's Roystonea Road Duplication Design Report.

The subdivision proposes a large detention basin which converges the west and east flow paths and attenuates flows at Roystonea Avenue and Inverway Circuit to limit the overall flow rate through the culverts to pre-development levels.

The existing culvert on Farrar Boulevard has been proposed for realignment and upgrade. The capacity of this system is sufficient for pre and post development discharge, with the upgrade intended to facilitate the installation of a larger public open space for a park facility and move the outlet to a safer location.

A new minor culvert will be required at the southern side of Brennan Court to allow vehicle access during construction and for stages 1 and 2 access prior to the major intersection upgrades at Farrar Boulevard and Roystonea Avenue. This culvert has been sized to be approximately 1x1,500mmx900mm to suit up to 1.5 m³/s from the upstream catchment. This will also be required to facilitate permanent path connections onto Brennan Court.

Table 3: Post Development Culvert Analysis Results¹

Culvert	Configuration	Design Discharge	Headwater Elevation	Outlet Velocity	Invert Levels
Temple Tce	1x1350 RCP 1x450 RCP	5.0 m ³ /s	32.6 mAHD	3.6m/s	Inlet IL = 30.3m Outlet IL = 30.1m
Farrar Blvd	5x1200x900	10.5 m ³ /s	25.4 mAHD	1.9m/s	Inlet IL = 24.0m Outlet IL = 23.7m
Brennan Ct (new culvert)	TBC at Part 2 Stormwater Management Plan				
Roystonea Ave	3x1200x900	10.0 m ³ /s	19.6 mAHD	3.1m/s	Inlet IL = 17.6m Outlet IL = 17.5m
Inverway Cct	4x4650x900	40.0 m ³ /s	17.8 mAHD	3.5m/s	Inlet IL = 16.3m Outlet IL = 16.2m

¹ Subject to change in final design

Culvert positioning, sizes and levels shall be confirmed as part of Detailed Design, where appropriate scour protection will be selected at this time.

4.4 Farrar Boulevard Drainage Rearrangement

A significant element of the proposed design involves the rearrangement of existing drainage infrastructure near the existing Farrar Boulevard Roundabout. Currently a 1650mm drainage pipe travels south along the west side of Farrar Boulevard, ending at a surcharge pit near the Roundabout. In addition, a 750mm pipe from the Health Precinct and Gurd St Catchment discharge at this location, which then initiates the main open drainage line that cuts through the Farrar West site. To facilitate a larger proposed main POS area and suit the proposed development layout, the main drainage channel is shown to be relocated to the east of the site adjacent existing development. Also the 1650mm, 750mm drainage line and Farrar Boulevard culvert crossings are proposed to be relocated further east, such that these flows do not sever the site and enable establishment of usable POS, as shown in Figure 17 below, as excerpted from the Drainage Layout plan.

These works are intended to be largely contained within external Road Reserves, and considered as external headworks. The scope of this is being discussed with DLPE and is subject to further coordination and confirmation as part of detailed design.

The extension and rearrangement of drainage lines and relocation of culverts under Farrar Boulevard has been included in the Part 1 Stormwater Management Plan and subsequent detailed design stages.

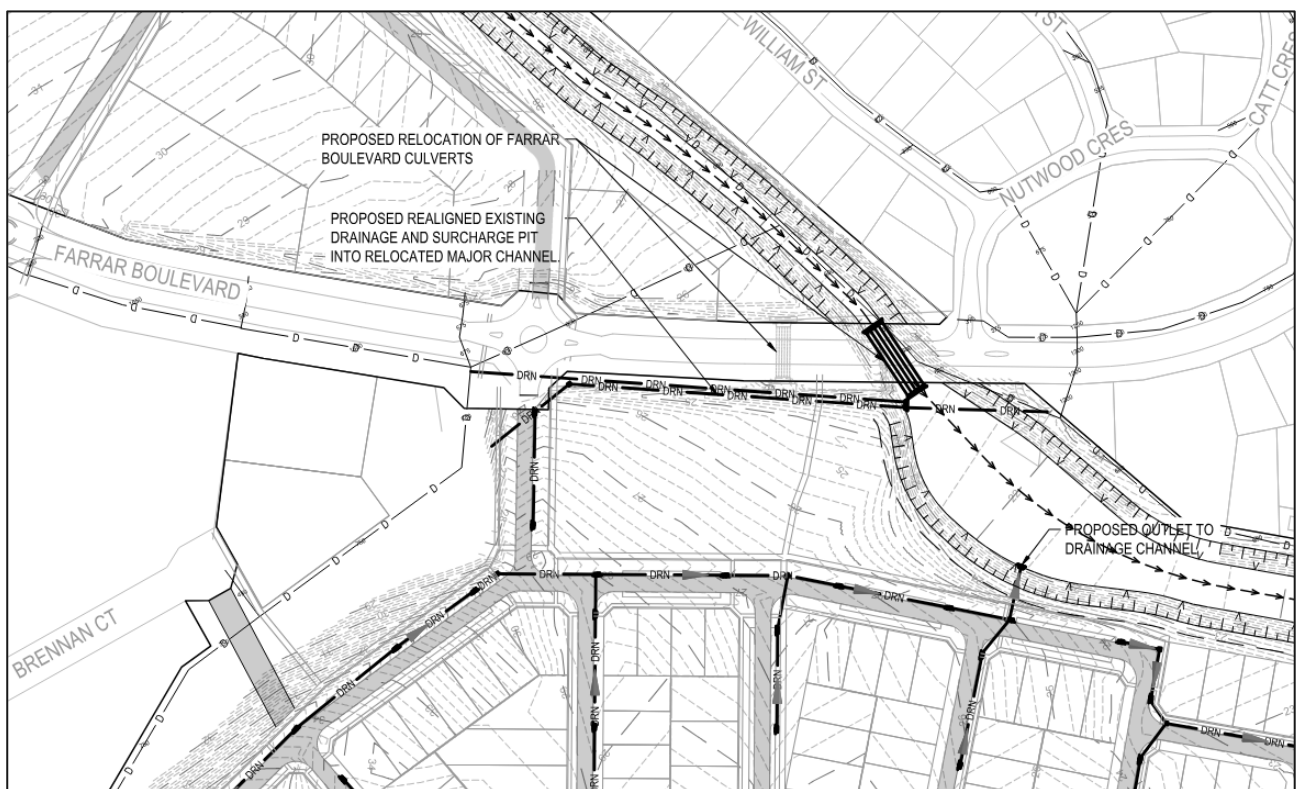


Figure 17: Farrar Boulevard Drainage rearrangement headworks

4.5 Detention Basin

A large detention basin has been incorporated into the subdivision design, which is divided into two main components, being the larger eastern section grading into the Inverway culvert outlets, and smaller western section grading towards the Roystonea culverts which aligns with the main upstream drainage channel, separated by a low ridge spillway approximately 500mm high. The eastern section incorporates the existing channel from the upstream catchments of the existing Farrar subdivision which discharges into the Inverway Circuit culverts.

The smaller western basin incorporates the existing channel from the Farrar West subdivision, with the channel directing discharge into the Roystonea Avenue culverts. In larger flows, stormwater will overtop the interconnecting spillway and fill the overall basin. The overall volume of the basin to the main overflow Inverway spillway level of 19.0m is approximately 24,500m³. Flood modelling indicates that the peak water level within the basin reaches approximately RL19.8 for the 1% AEP event, resulting in approximately 500mm freeboard to adjacent development lots and at least 600mm freeboard to existing lots within the Farrar area.

The detention basins are proposed to be outlet controlled through box culverts for low flow discharge volumes, with a larger spillway for the major events. As such they are intended to be “dry” detention basins, with the intent to drain away within at least 72 hours. In addition, it is proposed to install a surface pit (not shown in plans at this stage) and the corresponding pipe and subsoil drainage systems connected to the existing low-flow drainage system to further assist with basin emptying. The high flow spillway is shown as approximately 20m wide above the low-flow Inverway culvert outlets at a level of 19.0m, which will overtop towards the Inverway flow path during major storms. These details will need to be confirmed during detailed design. Water levels begin to breach the main spillway towards Inverway Circuit at approximately the 20% AEP critical event.

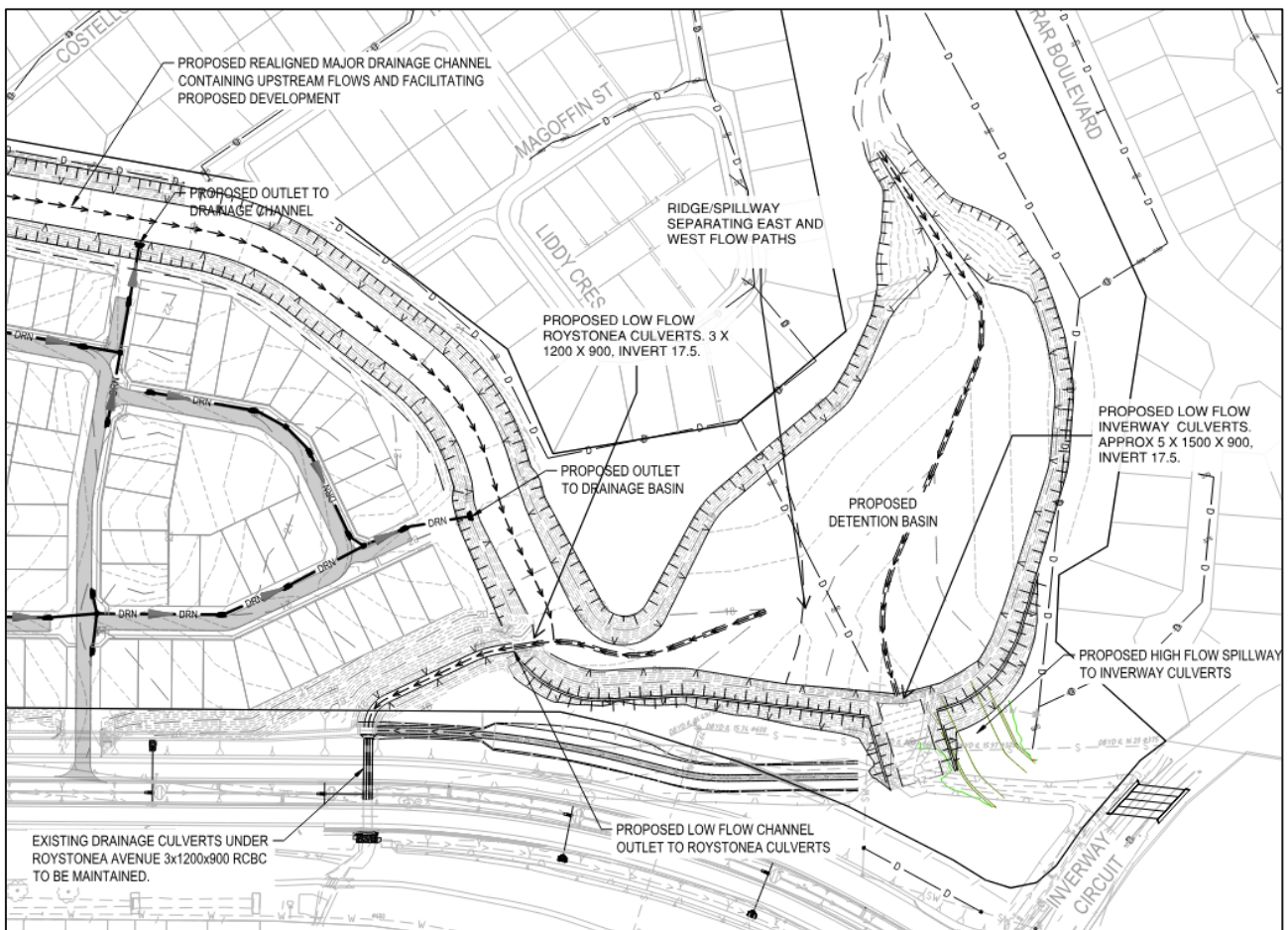


Figure 18: Proposed drainage basin and flow paths



Figure 19: Proposed Detention Basin Earthworks

Table 4: Detention Basin Layout

Inlet	Inflow Rate	Inlet Level
Farrar West channel (western section of basin)	29.6 m ³ /s (at 1%AEP)	18.0m AHD
Farrar East channel (Eastern section of basin)	22.6 m ³ /s (at 1%AEP)	19.0m AHD

Table 4: Proposed Detention Basin Outlets

Detention Components	Outflow Rate	Invert/Spillway Level	Description
Low level outlet (subsoil pipes connecting to existing low flow drainage system)	Nominal flows only	16.23m AHD	DN300 PVC at 0.5%
Roystonea Avenue	8.3 m ³ /s (at 1%AEP)	17.5m AHD	3x1200x900 RCBC at 0.3%

Detention Components	Outflow Rate	Invert/Spillway Level	Description
Interconnecting spillway (between west and east sections)	22.0 m ³ /s (at 1%AEP)	18.0m AHD	Approximately 30m long crest at 1.0%
Inverway Circuit low flow culvert	20.2m ³ /s (at 1%AEP)	17.5m AHD	5x1500x900 RCBC at 0.5%
Inverway major Spillway	11.3 m ³ /s (at 1%AEP)	19.0m AHD	20m swale at 3.0%

Table 5: Detention Basin Physical Parameters

Detention Components	Comments
Basin floor level	17.5 m AHD
High water level (1% AEP)	19.8
Stored volume (1% AEP 19.8m AHD))	54,500m ³
Batter slopes	1:6
Floor slope	Min 1%
Maximum storage capacity ² (to 20.1m AHD)	68,000 m ³
Storage capacity to spillway level (19.0m AHD)	24,500 m ³

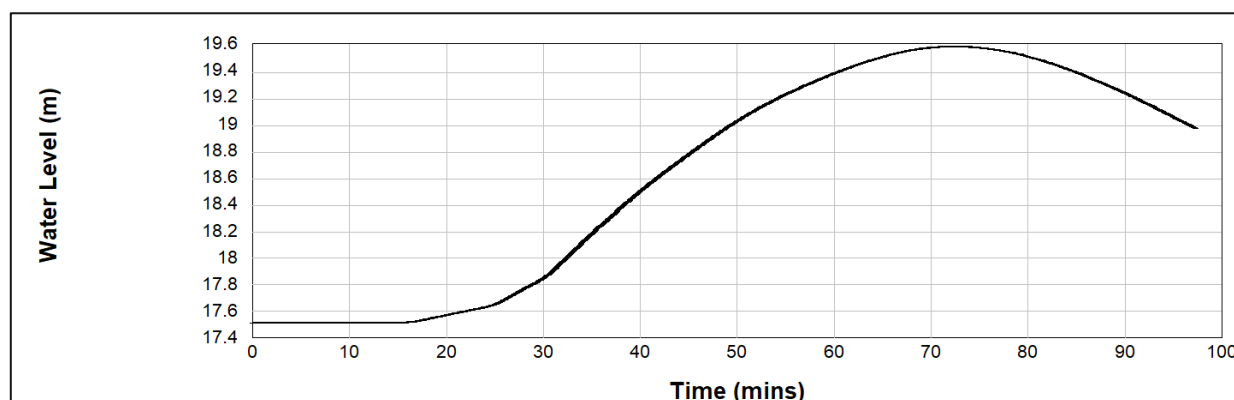


Figure 20: 1% AEP Water Level Graph

² At 20.0m AHD



Figure 21: Site Discharge Points

Site discharge of the post development conditions is noted at 3 nominated locations, at the low flow piped system on Inverway Circuit, at the Roystonea Avenue culverts and at the Inverway Circuit culverts. For the purposes of overall stormwater assessment and drainage performance, the existing low flow pipe is largely ignored as it continues to operate in its existing state and would be proposed to be used largely for subsoil connection and final drying of the basin.

As a result, assessment of post-development outflow compared to pre-development is made at the known points of discharge being the Roystonea culverts and Inverway culverts.

For the Roystonea Ave/Western flow path, the design proposes to implement equivalent low flow culvert outlets within the basin to constrain the post-development outlet at this point so it cannot surpass existing. The detention basin is then sized to ensure flows are sufficiently attenuated to limit total outflow towards the Inverway flow path to maximum pre-development conditions. The design has comfortably achieved this, as shown in Table 6 below.

Table 6: Major Event Discharge

Detention Components	Discharge Rate (Pre-Development)	Discharge Rate (Post-Development)	Change in peak discharge (1% AEP)
Low level outlet	1.7 m ³ /s	1.7 m ³ /s	0%
Roystonea Avenue	9.3 m ³ /s	8.7 m ³ /s	-6%
Inverway Circuit	36.5 m ³ /s	30.8 m ³ /s	-16%
Total	47.5 m ³ /s	41.2 m ³ /s	-13%

4.6 Stormwater Quality

Stormwater quality will be modelled on MUSIC and designed to match pre-development condition, to be detailed in Part 2 Stormwater Management Plan and coordinated with WRM water and environmental consultants.

It is noted that the vegetated drainage channel will be effective in nutrient and sediment management, however, during detailed design there may be further consideration of additional measures such as low check weirs or drop structures to further improve water quality performance. Landscape design of the channel and any planting is also intended to be further investigated and may provide additional water quality improvement, however all measures will need to consider maintenance and City of Palmerston requirements. Devices and treatment methodology with low maintenance skill and frequency requirements will be prioritised for implementation to reduce maintenance operator commitments.

It is also intended that all drainage lines prior to discharge to the main drainage channel will have Gross Pollutant Traps in accessible locations prior to all outlets. These may be sufficient to address all water quality objectives and shall be further assessed and modelled for outcomes during detailed design.

5 Conclusion and Recommendations

5.1 Conclusion

- The proposed stormwater system comprising pits, pipes, swales, culverts and detention infrastructure has been developed to manage runoff from the proposed Farrar West subdivision.
- Part 1 hydrologic and hydraulic modelling indicates that the proposed drainage system can convey the adopted minor and major design events in accordance with the nominated design criteria.
- A large detention basin has been incorporated into the preliminary subdivision design to mitigate the increased post-development runoff and limit discharge rates to existing conditions at both Roystonea Street and Inverway Circuit
- Culvert crossings and outlet protection measures shall be incorporated to maintain cross-drainage and minimise erosion risk.
- Water quality objectives and assessment is proposed to be further assessed as part of Part 2 of the Stormwater Management Plan, however, the preliminary design has been established to allow these requirements to be achieved.

5.2 Part 2 Stormwater Management Plan

- Confirm detention basin sizing and outlet configuration following completion of the final civil grading design and updated DRAINS modelling and verify this with the flood model from WRM hydrologists
- Review the stormwater design if there are significant changes to site layout, finished surface levels or impervious areas.
- Schematic stormwater layout, detailed pit and pipe location and geometry
- Detailed flow direction, management controls and infrastructure sizing
- Confirmation of post development flow rate at nominated discharge points on Roystonea Avenue and Inverway Circuit
- Risk assessment of the drainage system as a part of the project Safety in Design documentation
- Further development of any stormwater quality improvement devices and strategies.

Appendices

Appendices

Appendix One:	Draft Flood Impact Assessment by WRM Water and Environment
Appendix Two:	Existing Catchment Plan
Appendix Three:	Proposed Development and Catchments
Appendix Four:	Pit and Pipe Network Layout

Appendix One:

**Draft Flood Impact Assessment by WRM
Water and Environment**



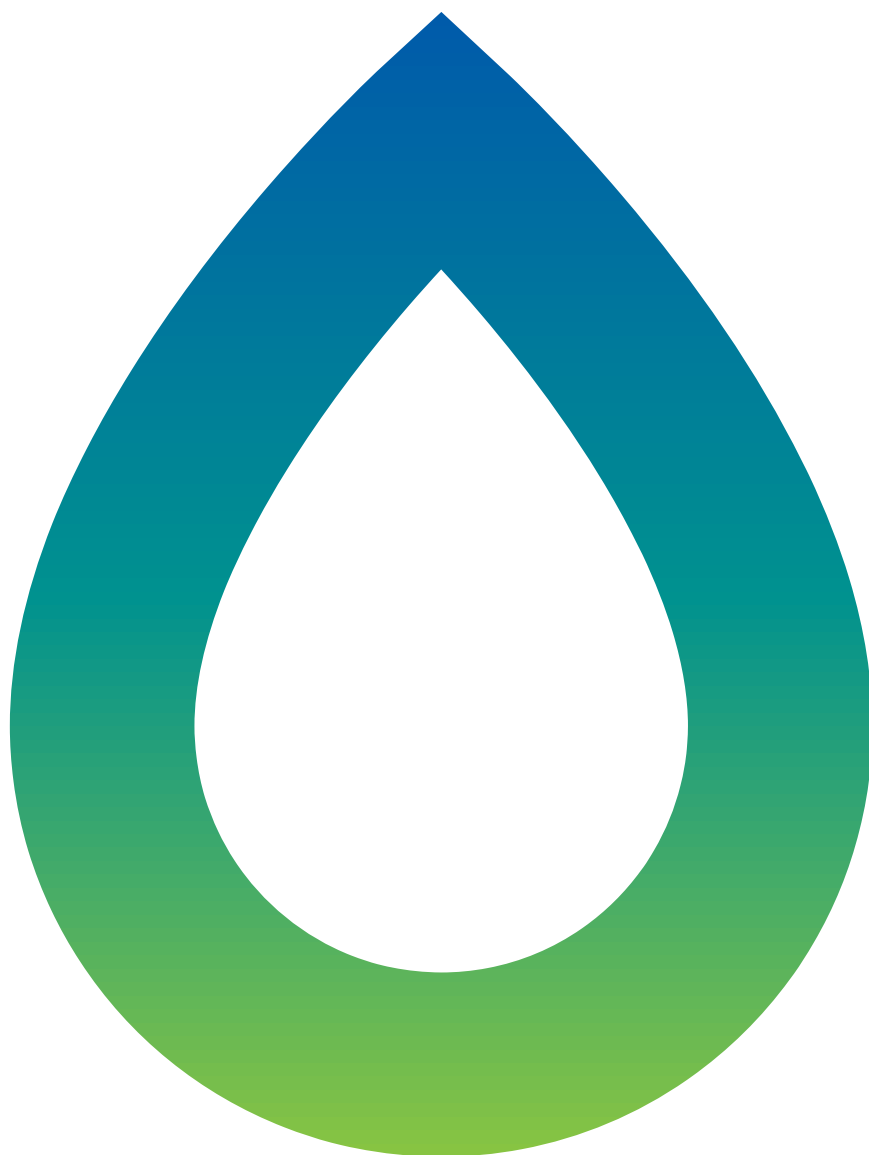
FARRAR WEST DEVELOPMENT

Flood Impact Assessment

Larrakia Development Corporation (LDC)

28 September 2026

2134-02-B1_DRAFT



DETAILS

Report Title	Farrar West Development, Flood Impact Assessment
Client	Larrakia Development Corporation (LDC)

THIS REVISION

Report Number	2134-02-B1_DRAFT
Date	28 September 2026
Author	TN, TY
Reviewer	JO/CB

DRAFT

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1 INTRODUCTION

1.1 BACKGROUND

Cuttone Property Group (referred to as Cuttone) has acquired a Development Agreement with the Northern Territory Government for the Farrar West development site (referred to as Farrar West or the Development Site). Figure 1.1 shows the location of the Development Site, which is bounded by Temple Terrace to the north and Roystonea Avenue to the southwest. The Development Site includes the following two (2) locations (see Figure 1.1)

- The North Site, which is bounded by Farrar Boulevard to the southwest and Temple Terrace to the north; and
- The South Site, which is bounded by Farrar Boulevard to the northeast and Roystonea Avenue to the southwest.

Cuttone, on behalf of LDC, requested WRM Water & Environment Pty Ltd (WRM) to undertake a flood impact assessment for the trunk stormwater management infrastructure proposed for the Development Site.

Hydrologic (URBS) and hydraulic (TUFLOW) models of Mitchell Creek and its tributaries were developed by WRM to assess the potential flood impacts of the Development Site trunk drainage design, provided by Colliers.

This report presents the methodology and results of the hydrologic and hydraulic modelling.

1.2 REPORT STRUCTURE

The report is structured as follows:

- Section 2 describes the drainage characteristics of the study area under existing and proposed developed conditions;
- Section 3 is an overview of the data available for this study;
- Section 4 describes the development, validation of the existing conditions hydrologic model and design discharge estimates;
- Section 5 presents the developed conditions hydrologic model;
- Section 6 presents the development of the existing and developed conditions hydraulic model and the flood impact assessment analysis;
- Section 7 presents the conclusion of the study;
- Section 8 is a list of references;
- Appendix A presents proposed Farrar West development;
- Appendix B presents details of the hydrologic model configuration;
- Appendix C presents the Rational Method calculation details;
- Appendix D presents the hydraulic structure details of the TUFLOW model; and
- Appendix E presents the flood maps for 50% and 10% AEPs.



Figure 1.1 Farrar West locality and regional drainage features

2 CATCHMENT AND DRAINAGE CHARACTERISTICS

2.1 EXISTING CONDITIONS

2.1.1 Regional drainage characteristics

Figure 1.1 shows the regional drainage features in the vicinity of the Development Site. The Development Site is located within the Mitchell Creek catchment. Mitchell Creek is a tributary of the Elizabeth River, with a catchment area to the junction with Brooking Creek of about 18.1 km² (see Figure 1.1)

Mitchell Creek rises on the northeastern side of the Stuart Highway, about 1.3 km southeast of the Palmerston Hospital. The eastern arm of Mitchell Creek flows west to the Stuart Highway, before turning to flow south toward the Elizabeth River. The western arm of Mitchell Creek flows southeast to pass under Temple Terrace, Farrar Boulevard and Lambrick Avenue, before joining the eastern arm and flowing south toward the Elizabeth River.

2.1.2 Site drainage characteristics

Based on available topographic data and as constructed drawings, key local drainage features of the Development Site include Mitchell Creek tributaries and several local flowpaths. Figure 2.2 shows the local drainage features in the vicinity of the Development Site. The following is of note:

- There are three (3) tributaries of Mitchell Creek (western arm):
 - Tributary 1 (TB1) originates north of Temple Terrace and flows along the west side of Farrar Boulevard and northeast of the South Site. TB1 flows in a south-easterly direction, to the southwest of Farrar Boulevard, before turning south-southwest and crossing Roystonea Avenue. It then continues southeast to join Mitchell Creek approximately 230 m east of Roystonea Avenue. It is understood that an existing drain called the Inverway Drain was recently constructed on the northeast side of Roystonea Avenue and crosses Inverway Cct. It is understood that this drain was constructed to mitigate flooding in the area.
 - Tributary 2 (TB2) originates north of Temple Terrace and flows along the northern boundary of the North Site in a south-easterly direction. After passing under Farrar Boulevard, it turns westward and discharges into TB1.
 - Tributary 3 (TB3) originates southwest of Stuart Highway and flows along eastern boundary of the South Site in a south-westerly direction to the centre of Inverway Drain.
- Nine (9) primary overland flowpaths transport flow from the Site to the tributaries including:
 - FP1 (9.39 ha) is located along the north boundary of the South Site catchment, including two flowpaths FP1a and FP1b which convey discharges to FP1, and drains in a north easterly direction to convey discharge to the TB1;
 - FP2 (3.66 ha) is situated in the northwest portion of the South Site catchment, and drains in a northerly direction to convey discharges from the South Site to the TB1;
 - FP3 (7.64 ha) is in the middle of the South Site catchment, and drains in a northerly direction to convey discharges from the South Site to the TB1;
 - FP4 (3.54 ha) and FP5 (2.53 ha) are located in the East of the South Site catchment, and drains in an easterly direction to convey discharges from South Site to the TB1;
 - FP6 and FP7 (1.31 ha and 2.64 ha) are located in the west portion of the North Site catchment, and drains in a south westerly direction from North Site to the TB2;
 - FP8 and FP9 (0.96 ha and 0.4 ha) are located in the east portion of the North Site catchment, and drains in an easterly direction from North Site to the TB2;

2.2 PROPOSED DEVELOPMENT CONDITIONS

2.2.1 Proposed trunk drainage works

The proposed Farrar West development concept provided by Hatch and the stormwater layout provided by Colliers are shown in Appendix A. The Development Site and associated impact mitigation measures comprise the following:

- Urbanisation of the Site, encompassing areas located to the northeast and southwest of the Farrar Boulevard; and
- A proposed detention basin, situated at the downstream end of TB1, between the Development Site and the existing Farrar subdivision;
- One proposed open drain designed to convey runoff from the Development Site as well as external upstream catchments to the proposed detention basin; and
- Three proposed pipes designed along Farrar Boulevard to convey discharge to the open drain.

2.2.2 Detention basin

The proposed detention basin will mitigate the potential peak discharge impacts of the proposed Farrar West development, for which the conceptual plan is shown in Figure 2.3. The adopted detention basin details are as followings:

- Detention basin spillway level: 19 mAHD.
- Width of spillway crest: 20 m.
- Basin embankment crest level: 20.1 mAHD (provided by Colliers) and increased to 21 mAHD in TUFLOW model (refer to Section 6).
- Two low flow culverts through the basin bank, with details as shown in Table 2.1.
- Upstream invert level of the drain connecting to the culvert DB_002: approximately 17.57 mAHD.
- Downstream invert level of the drain connecting to the culvert DB_002: approximately 17.4 mAHD.

Table 2.1 Details of low flow culverts through the basin bank

Culvert ID	Types	US Invert (mAHD)	DS Invert (mAHD)	Diameter/ Width (m)	Height (m)	No. of barrels
DB_001	RCBC	18.5	18.0	0.9	0.6	3
DB_002	RCBC	17.65	17.57	0.9	0.6	3

The low flow culverts in the flood model may differ from the civil design and the culvert details will be reviewed and updated during detailed design phase.

2.2.3 Open drain

Figure 2.4 shows the open drain and road crossing culvert proposed to convey runoff from the upstream catchment draining into the proposed detention basin. The proposed drain provided by Colliers includes the following:

- The drain will be located along the northeastern boundaries of the North Site, following TB2. On the downstream side of Farrar Boulevard, it follows the boundary of the South Site, following TB1. The drain will commence immediately downstream of the culvert Cop_001 under Farrar Boulevard and terminate at the detention basin inlet. The details are as followings:
 - Length: 1,000 m;
 - Bed width: 8 m upstream of the culvert Cop_001 and 20 m downstream of the culvert;

- Top width: 25 m upstream of the culvert Cop_001 and 38 m downstream of the culvert (varies);
- Slope: 1.6% upstream of the culvert Cop_001 and 0.7% downstream of the culvert Cop_001;
- A road crossing will be located where the drain intersects Farrar Boulevard and will comprise seven 1.2 m wide × 0.6 m high RCBCs.

The longitudinal profile of the proposed drain is shown in the Figure 2.1. Minor discrepancies have been identified between the developed conditions model and the civil design. The culvert configuration and dimensions will be confirmed in the next design phase.

2.2.4 Pipes

The existing low-flow drainage pipes crossing Farrar Boulevard and connecting the pipe along the open drain have been rearranged as part of the proposed design. Instead, three proposed RCPs along Farrar Boulevard were designed to convey flows from the upstream pipe network along Farrar Boulevard and the western portion of the South Site to the open drain.

Details of the proposed pipes are provided in Table 2.2 with Figure 2.5 showing the locations of these pipes. There are minor discrepancies with the civil design and the pipe details are to be confirmed during detailed design phase.

Table 2.2 Proposed pipes details

ID	US Invert (mAHD)	DS Invert (mAHD)	Diameter (m)	No. of barrels
MCL_0058	23.53	23.37	1.5	2
MCL_0059	23.53	23.37	1.5	2
MCL_0060	23.36	23.3	1.8	1

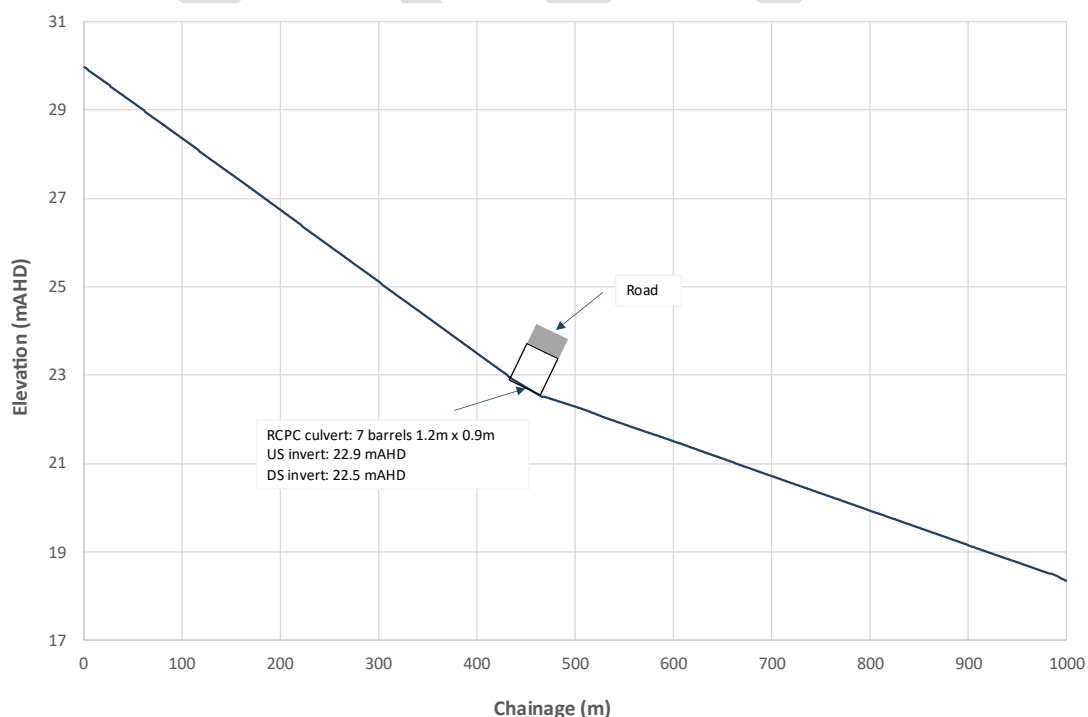


Figure 2.1 Longitudinal profile of the proposed drain



Figure 2.2 Local drainage characteristic – Existing conditions

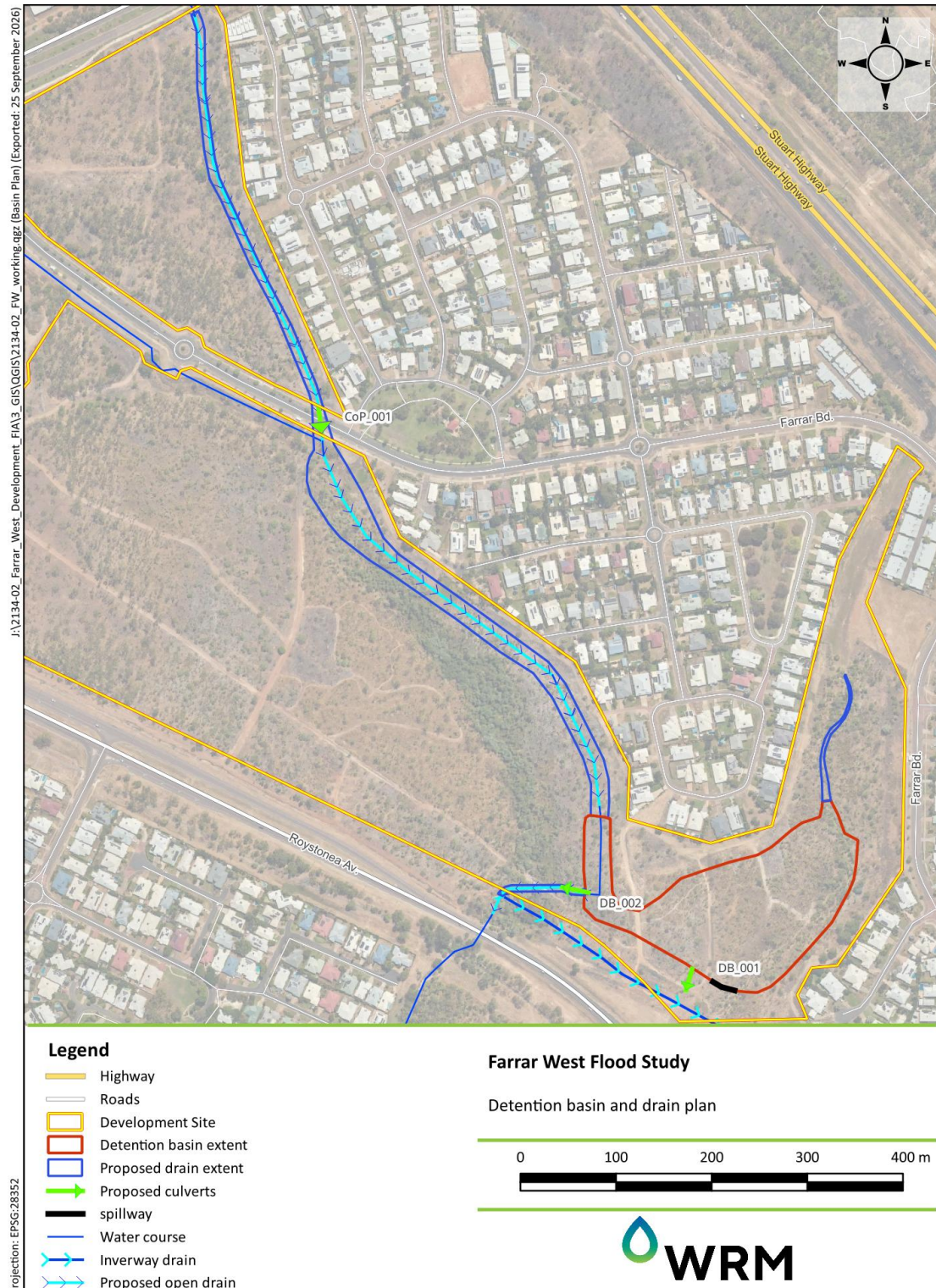


Figure 2.3 Detention basin and open drain plan

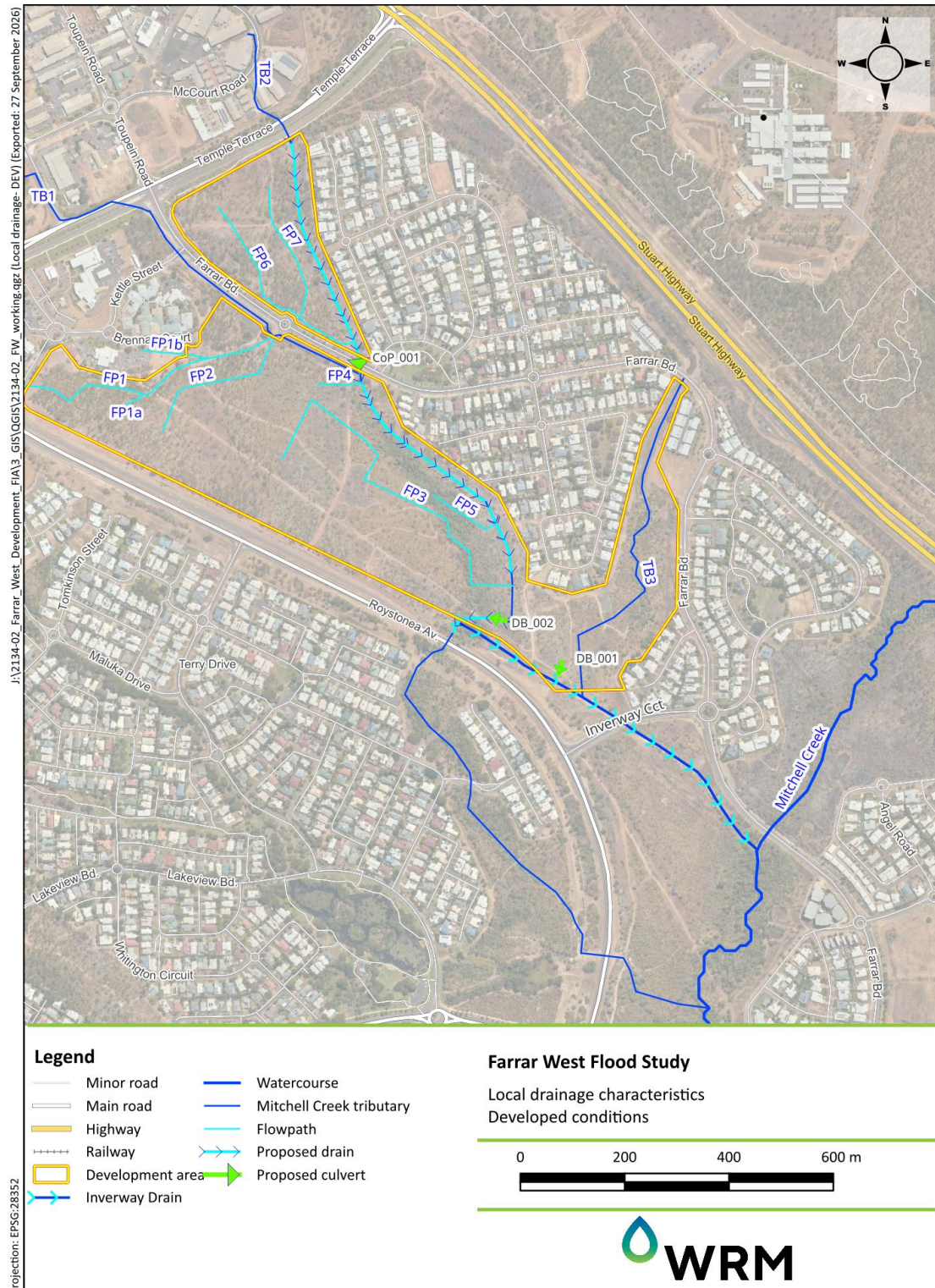


Figure 2.4 Local drainage characteristic – Developed conditions



Figure 2.5 Proposed pipe configuration

3 AVAILABLE DATA

Available data for the purpose of this study consisted of following:

- Design rainfall depths obtained from Bureau of Meteorology (BoM);
- Australia Rainfall and Runoff (ARR 2019) hydrologic design parameters;
- A 1m resolution LiDAR acquired in May 2018 provided by Department of Infrastructure, Planning and Logistics (DIPL);
- 1m design resolution surface to incorporate the Inverway Drain in June 2025;
- An existing ground LiDAR provided by Colliers on 31 July 2026;
- Roystonea Road survey provided by Earl James & Associates (EJA) on 4 August 2026;
- A current ground survey provided by EJA on 4 September 2026;
- Site layout including stormwater plan provided by Colliers on 28 July 2026 and 24 August 2026;
- A concept layout plan provided by Hatch on 2 September 2026;
- A proposed surface generated for detention basin and drains tins named OUT 26.09.23 - PROPOSED SURFACE TIN.12daz provided by Colliers on 23 September 2026;
- Details of hydraulic structures were obtained from a previous study (WRM, 2023) and drawing documents in pdf format provided by Colliers and EJA in July and September 2026 separately.

4 EXISTING CONDITIONS HYDROLOGIC MODELLING

4.1 OVERVIEW

Design discharges for the Site were estimated using the Unified River Basin Simulator (URBS) rainfall runoff routing model (Caroll, 2021). Version 6.61 of the URBS software (updated and released in 2021) was employed for this study.

The URBS hydrologic model was used to estimate design discharge hydrographs for three design events including the 50 % annual exceedance probability (AEP), 10% AEP and 1% AEP. The peak discharges estimated by URBS model were validated against the Rational Method (RM) discharge estimates.

Estimated design discharges from the validated URBS model were input into the hydraulic model. The design rainfall and temporal pattern data for the validated model uses the latest available information from the ARR Data Hub¹.

4.2 MODEL CONFIGURATION

4.2.1 Model configuration

Table 4.1 shows the configuration of the Mitchell Creek URBS model developed for this study. The sub-catchment area, links, and catchments slope were derived from the available topography data. A total of 61 sub-catchments were delineated with areas ranging from 0.4 ha to 183 ha. The sub-catchment characteristics are presented in the Table B.1, Appendix B.

Table 4.1 presents the URBS parameters adopted for the hydrologic model. The parameters were derived by validating the URBS discharges against Rational Method (see Section 4.3).

Figure 4.2 shows the URBS model land use under existing conditions.

Table 4.1 Mitchell Creek URBS model sub-catchment area

Parameter	Adopted value
α (Channel lag parameter)	0.21
β (Catchment lag parameter)	1.0
m (Catchment non-linearity parameter)	0.715

4.2.2 Design rainfall and losses

Design rainfall intensities for a wide range of design events and durations from 2 minutes to 18 hours were obtained from the Commonwealth Bureau of Meteorology (BOM) Design Rainfall Data System (<http://www.bom.gov.au/water/designRainfalls/revised-ird/>). Rainfall depths were obtained at the centroid of each URBS model sub-catchment. The adopted design rainfall intensities were uplifted from BoM obtained design rainfall intensities based on the AR&R guideline (Ball et. al, 2019b). Table 4.2 shows the adopted design rainfall intensities.

The initial and continuing storm losses were adopted for the hydrologic URBS model by validating against Rational Method. Different losses were adopted for each event to reflect the antecedent catchment condition. Table 4.3 shows the loss values for range of design events simulation.

¹ <https://data.arr-software.org/>

Table 4.2 Adopted design rainfall intensities

Duration (minutes)	Design Rainfall intensities (mm/hr)		
	50%	10%	1%
2	241.0	296.2	362.2
5	203.9	253.0	323.8
10	170.3	211.1	277.0
15	148.7	183.5	241.0
20	131.9	164.3	213.5
25	119.8	148.7	191.9
30	109.7	135.5	175.1
45	88.3	109.7	141.5
60	74.2	92.9	119.3
90	56.1	71.1	92.2
120	45.3	58.2	76.5
180	33.1	43.7	59.2
270	24.0	32.6	46.0
360	19.1	26.5	38.8
540	13.8	20.0	30.7
720	11.2	16.4	26.3
1080	8.4	12.8	21.1

Table 4.3 Adopted initial and continuing losses for design events simulation

Events (%AEP)	Initial loss (mm)	Continuing loss (mm/h)
50	50	3.0
10	45	2.6
1	37	1.2

4.2.3 Temporal Pattern

The ensemble event approach described in ARR2019 was used for this analysis. This approach uses an ‘ensemble’ of 10 temporal patterns for each storm duration to derive a wide range of estimated flood peaks for each AEP event.

The critical events adopted were based on the URBS peak discharge. The duration producing the highest mean peak discharges (estimated from 10 temporal patterns) at selected locations was selected as the critical duration. The temporal patterns producing the peak discharges closest and higher than the median peak for the critical duration was selected as the design temporal pattern. The selected locations are at the proposed works on the study area.

4.3 MODEL VALIDATION

In an absence of reliable flow data within the study area to calibrate the URBS model, the model was validated against the Rational Method (RM) as an alternative estimate of peak design discharges. Rational Method calculations were undertaken in accordance with QUDM (DEWS, 2013). Adopted RM parameters and calculation details are provided in Appendix C.

The following locations were used for validation:

- Total flow of catchment 270413 (catchment area of 7.3 ha);
- Total flow to catchment 270416 (catchment area of 5.9 ha);

Figure 4.1 shows the URBS model validation locations.

Table 4.4 shows the comparison between the URBS model and RM peak discharges estimates for the 1% AEP design event. Model results showed very good agreement with the RM estimates at the two locations with URBS model discharges within 4% of RM discharges estimates.

Table 4.4 URBS model validation results – 1% AEP event

Catchment	Rational Method (m ³ /s)	URBS (m ³ /s)	Difference (%)
270413	4.20	4.36	4
270416	2.40	2.38	-1

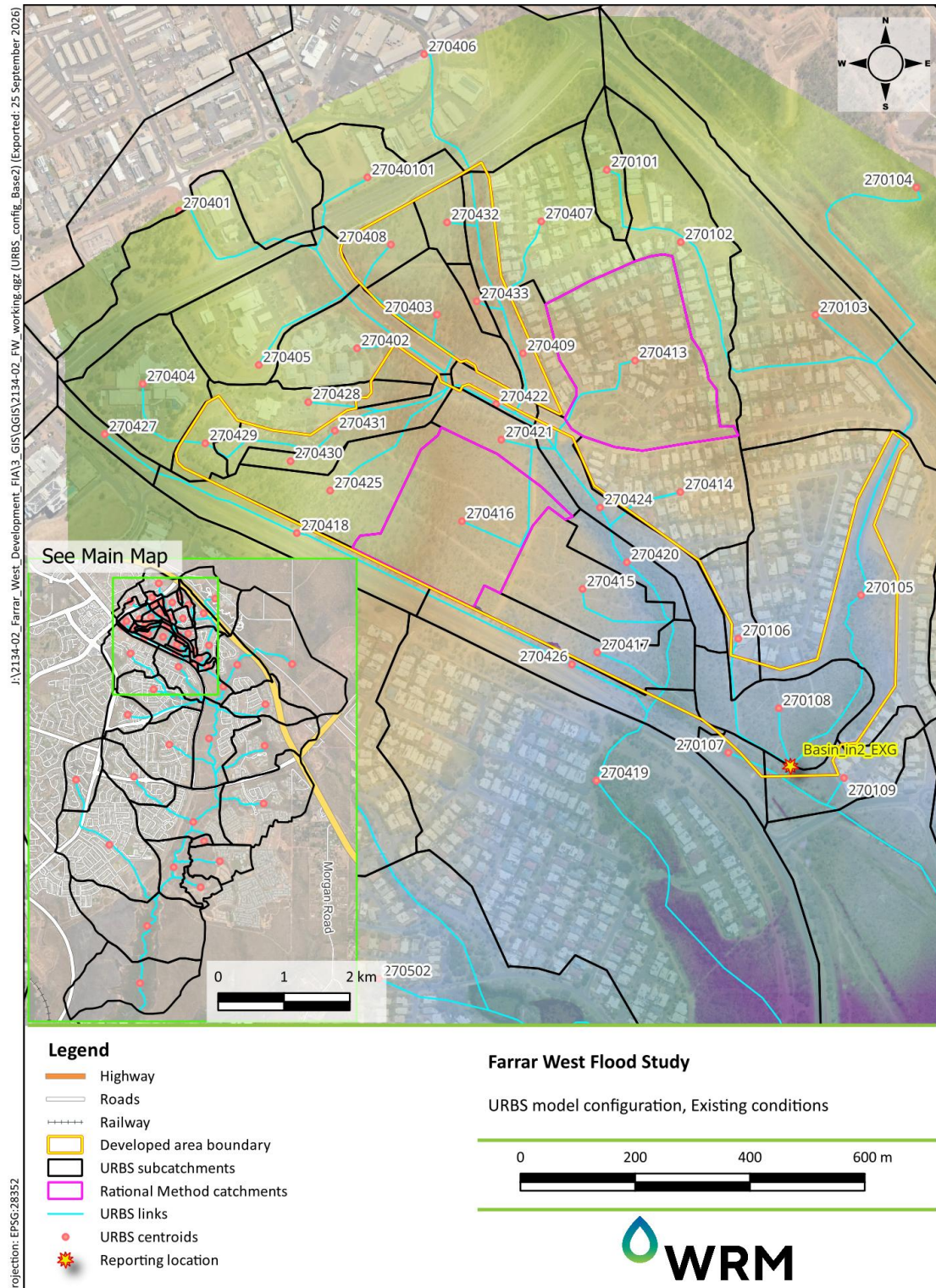


Figure 4.1 Mitchell Creek URBS configuration – Existing conditions

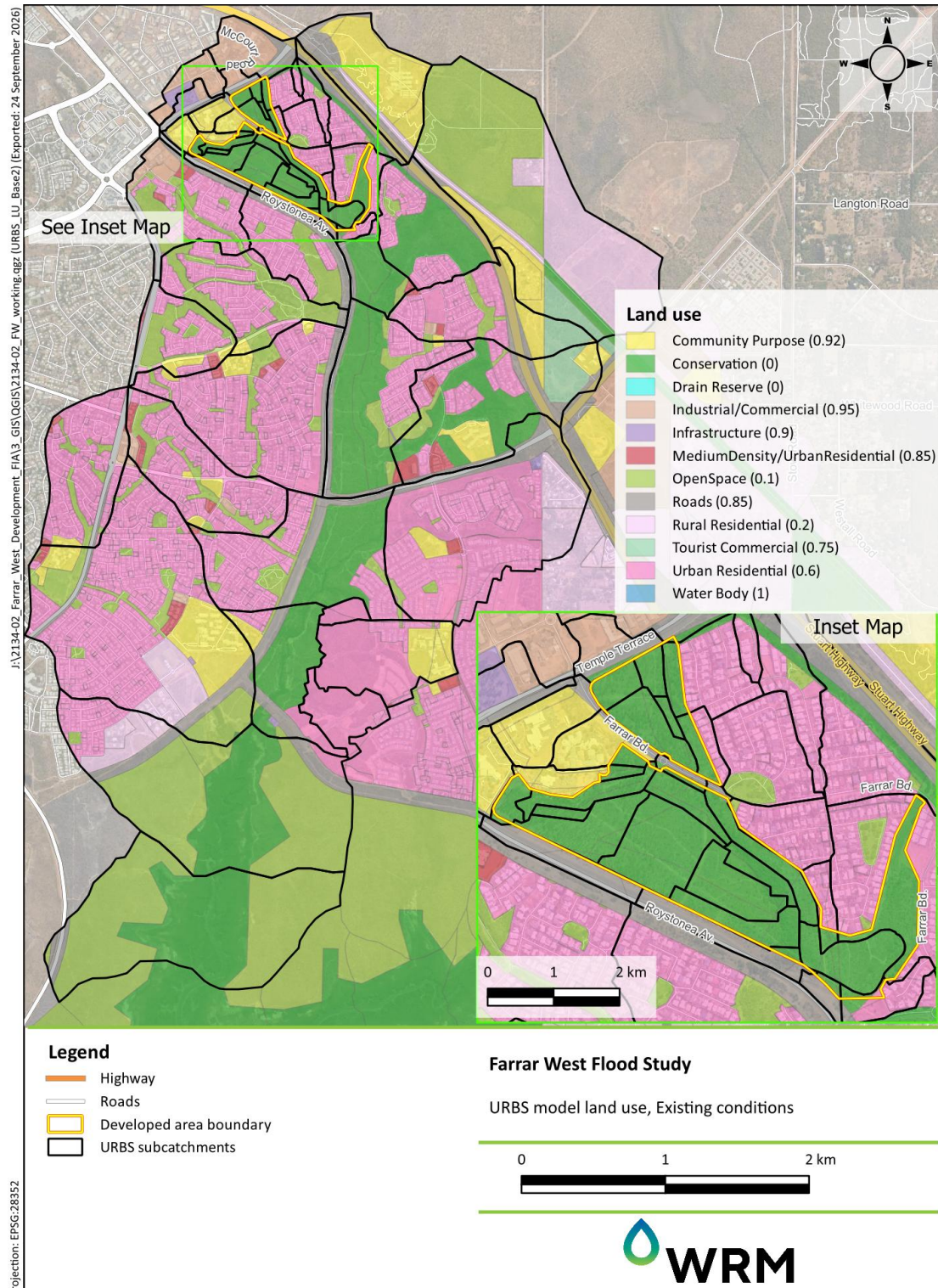


Figure 4.2 Mitchell Creek URBS land use map – Existing conditions

5 DEVELOPED CONDITIONS HYDROLOGIC MODELLING

5.1 MODEL CONFIGURATION CHANGES

The existing conditions URBS model configuration was updated to incorporate the proposed works associated with the Development Site. Figure 5.1 presents the Mitchell Creek URBS model configuration for developed conditions and Figure 5.2 presents the land use for developed conditions.

The imperviousness and catchment slopes at the Development Site was modified to reflect the proposed land use changes. The flow paths in and around the Development Site were redirected due to the change of design surface and drainage structures (e.g. culverts).

The sub-catchment characteristics for developed mitigated conditions model are presented in Table B.2, Appendix B.

5.2 SUB-CATCHMENT FLOWS

The developed mitigated conditions URBS model was used to estimate design discharges for the study area based on the ARR 2019 guidelines (Ball et al., 2019). Local flows from each URBS sub-catchment were input as inflows to the TUFLOW model (Section 6.2), which was used to resolve flow behaviour through and around the Development Site.

Table 5.1 shows the representative storm duration temporal pattern hydrographs from URBS which were selected for use in TUFLOW hydraulic model. The representative temporal patterns were selected based on an assessment of the URBS critical design storms at selected key reporting locations (see Figure 4.1 and Figure 5.1) for an ensemble of 10 temporal patterns. Inflow hydrographs for storm durations from 15 minutes to 6 hours were considered to derive the critical temporal patterns.

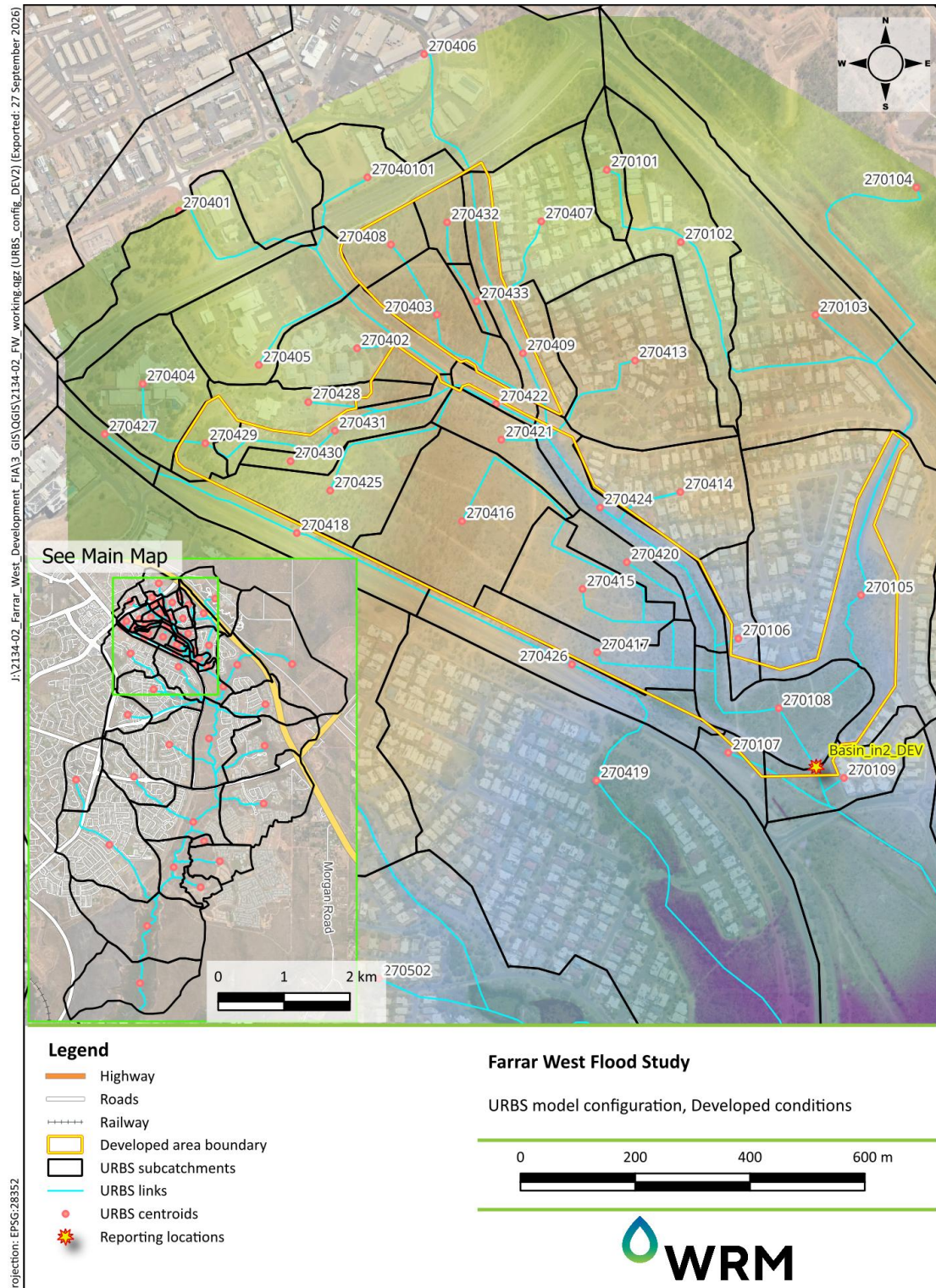


Figure 5.1 Mitchell Creek URBS configuration – Developed conditions

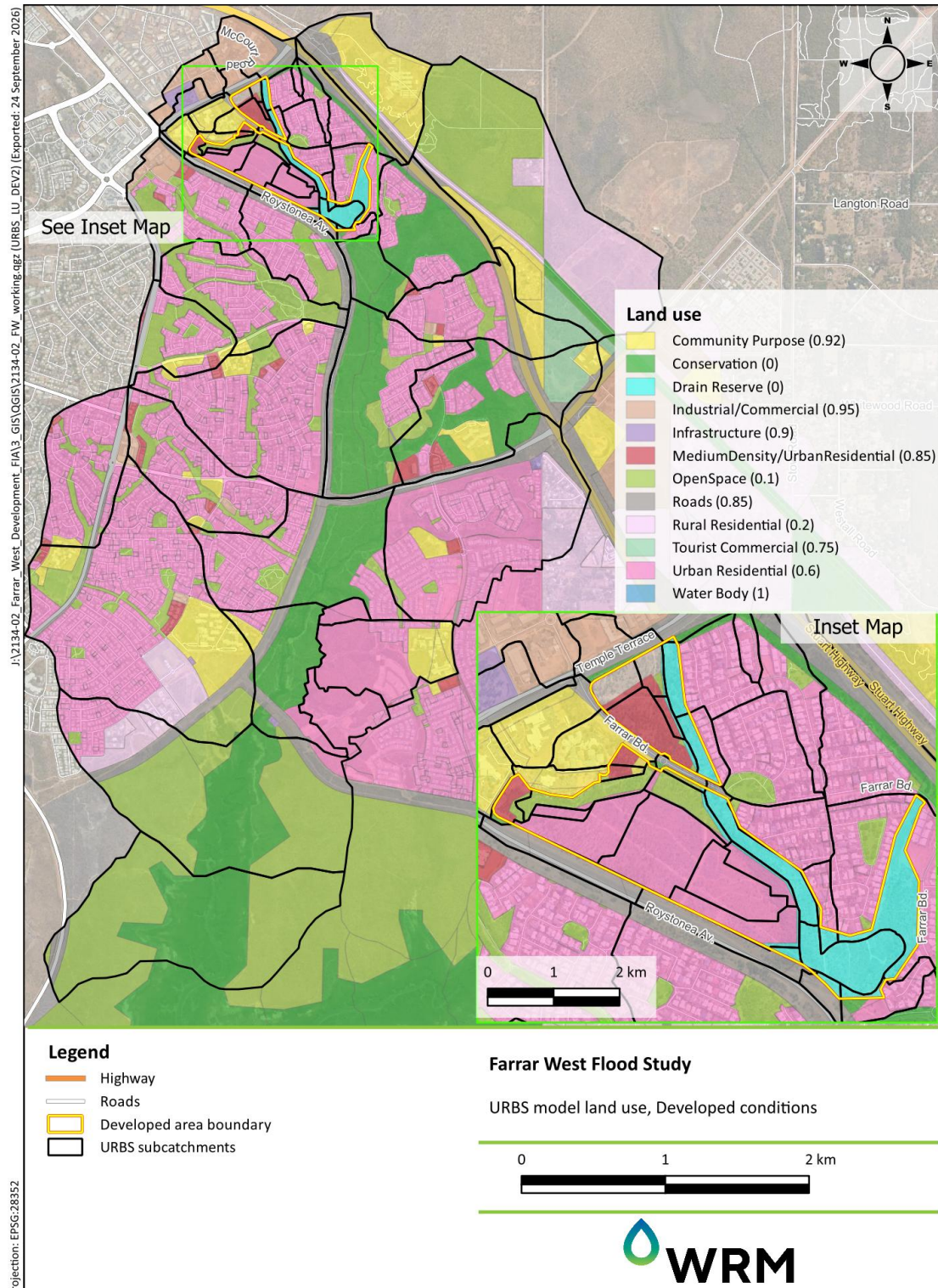


Figure 5.2 Mitchell Creek URBS land use – Developed conditions

Table 5.1 URBS representative temporal patterns at reporting locations

AEP (%)	Critical durations (mins)	Temporal patterns	
		Basin_in2_EXG2	Basin_in2_DEV2
50	10	-	9
	15	7	-
	20	9	9
	25	4	5
	60	3	3
	90	4	4
	120	1	1
	180	7	7
10	10	8,10	8,10
	20	7,8	7,8
	30	6	-
	60	-	7
	90	5	5
	120	8	8
	180	2	2
1	10	9	9
	15	2	8
	20	3	3
	25	2	10
	30	-	3
	60	-	5
	90	7	7
	120	5	5
	180	1	1

6 FLOOD IMPACT ASSESSMENT AND FLOOD MAPPING

6.1 OVERVIEW

The two-dimensional TUFLOW hydrodynamic model (BMT, 2023) was used to simulate the flow behaviour of Mitchell Creek and its tributaries and determine the flood extents, levels, depths and velocities for the 50% AEP, 10% AEP, and 1% AEP events in the vicinity of the Development Site. The Farrar West TUFLOW hydraulic model was used to assess the following two conditions to assess the potential flood impacts of the Development:

- Existing conditions – based on 2018 LiDAR topographic data, incorporating the current terrain and Inverway Drain; and
- Developed conditions – based on the 2018 LiDAR plus the proposed development, design drains, detention basin and new culverts.

6.2 EXISTING CONDITIONS MODELLING

6.2.1 Model extent and resolution

Figure 6.1 shows the overall extent of the TUFLOW model configuration developed for existing conditions for this study and Figure 6.2 provides a more detailed view of the Development Site and its immediate surroundings. The model covers an area of approximately 7.8 km². A 2 m grid cell size was adopted for the TUFLOW model configuration.

6.2.2 Topography data

The available topographic data was used to represent ground elevations for the existing conditions across the study area in the hydraulic model domain. The topographic datasets listed in 3 are incorporated into the hydraulic model in the order of priority outlined below:

1. Roystonea Road survey provided by EJA on 4 August 2026;
2. Current ground survey provided by Colliers on 4 September 2026;
3. 1m design resolution surface to incorporate the Inverway Drain;
4. Existing ground LiDAR provided by Colliers on 31 July 2026;
5. 1m resolution 2018 LiDAR.

6.2.3 Inflow and outflow boundaries

Figure 6.1 shows the locations of the inflow boundaries used in the TUFLOW model. Discharge hydrographs at the inflow boundaries were obtained from the URBS model (with names corresponding to the URBS model sub-catchment – see Figure 4.1). The boundaries were configured as following:

- Fifty (50) inflow polygons (2d_sa) were utilised to represent sub-catchment inflows within the TUFLOW model extent; and
- One (1) outflow boundary (QT) with a normal depth slope of 0.1% was applied at the downstream boundary of the model.

6.2.4 Adopted Manning's 'n' values

The TUFLOW model uses Manning's 'n' values to represent hydraulic resistance (notionally channel or floodplain roughness). Discrete regions of continuous vegetation types and land uses were mapped, and appropriate roughness values assigned to each region. Vegetation and land use mapping was undertaken using an aerial photograph. Figure 6.3 presents land use type and their corresponding surface roughness values. Table 6.1 shows the adopted Manning's 'n' values for the TUFLOW model.

Table 6.1 Adopted hydraulic roughness coefficients

Landuse	Adopted Manning's 'n'
Road Reserve	0.025
Industrial area	0.300
Medium Residential	0.250
Rural Residential	0.055
Open space	0.040
Community Purpose	0.150
Public Recreational	0.055
Water body	0.020
Dense vegetation	0.090
Drain	0.045

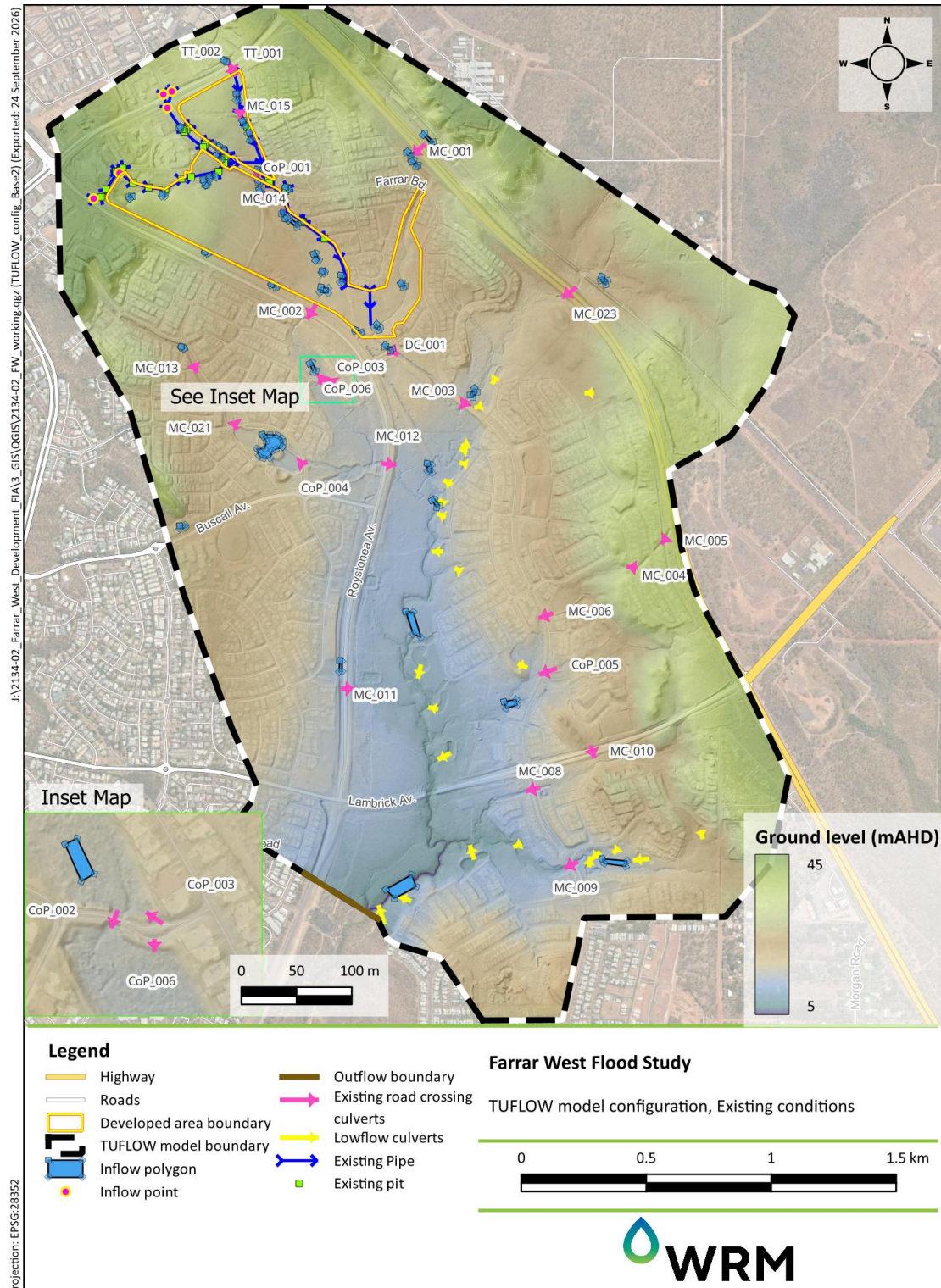


Figure 6.1 TUFLOW configuration, existing conditions

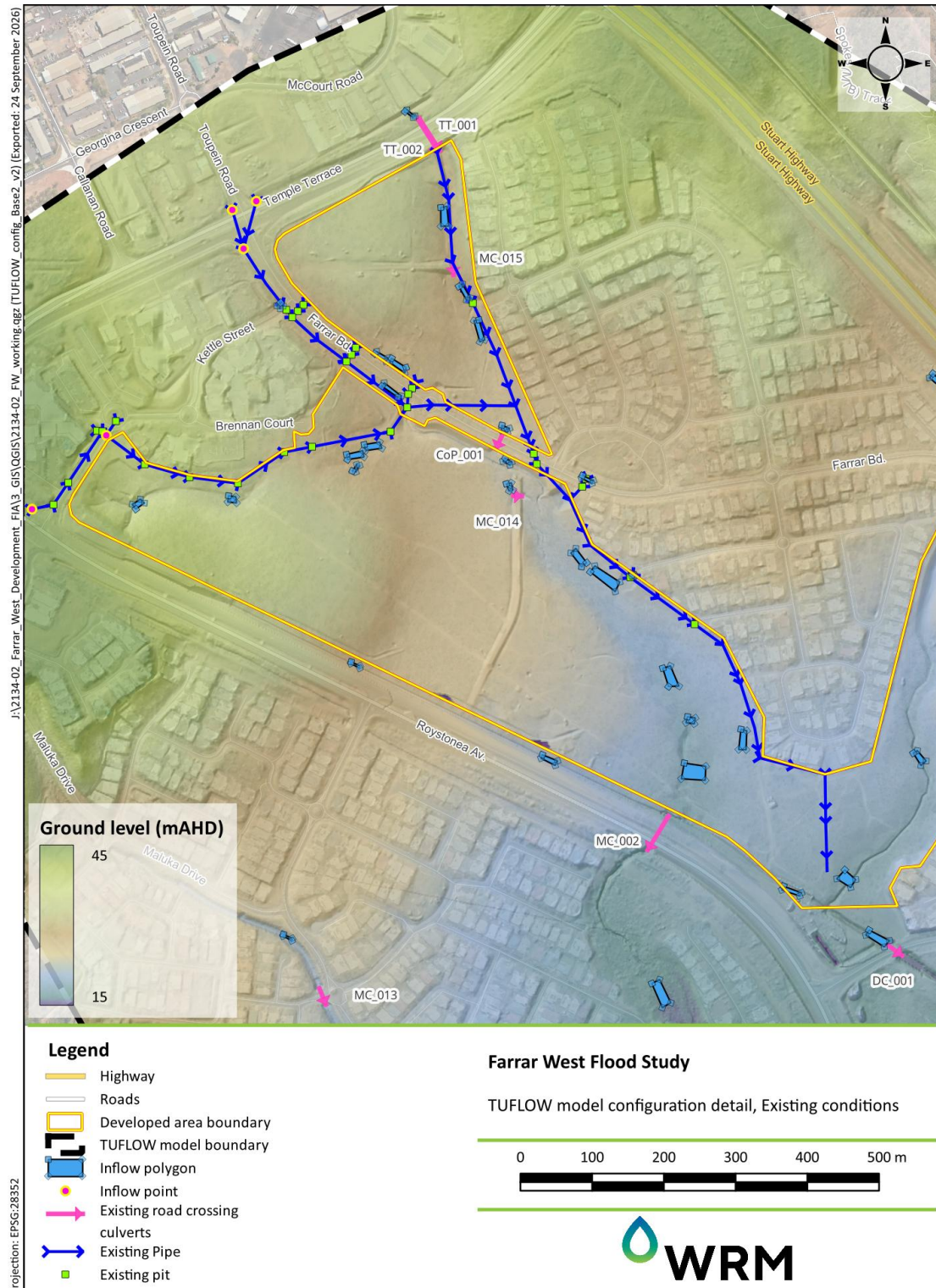


Figure 6.2 TUFLOW configuration, existing conditions – Zoom

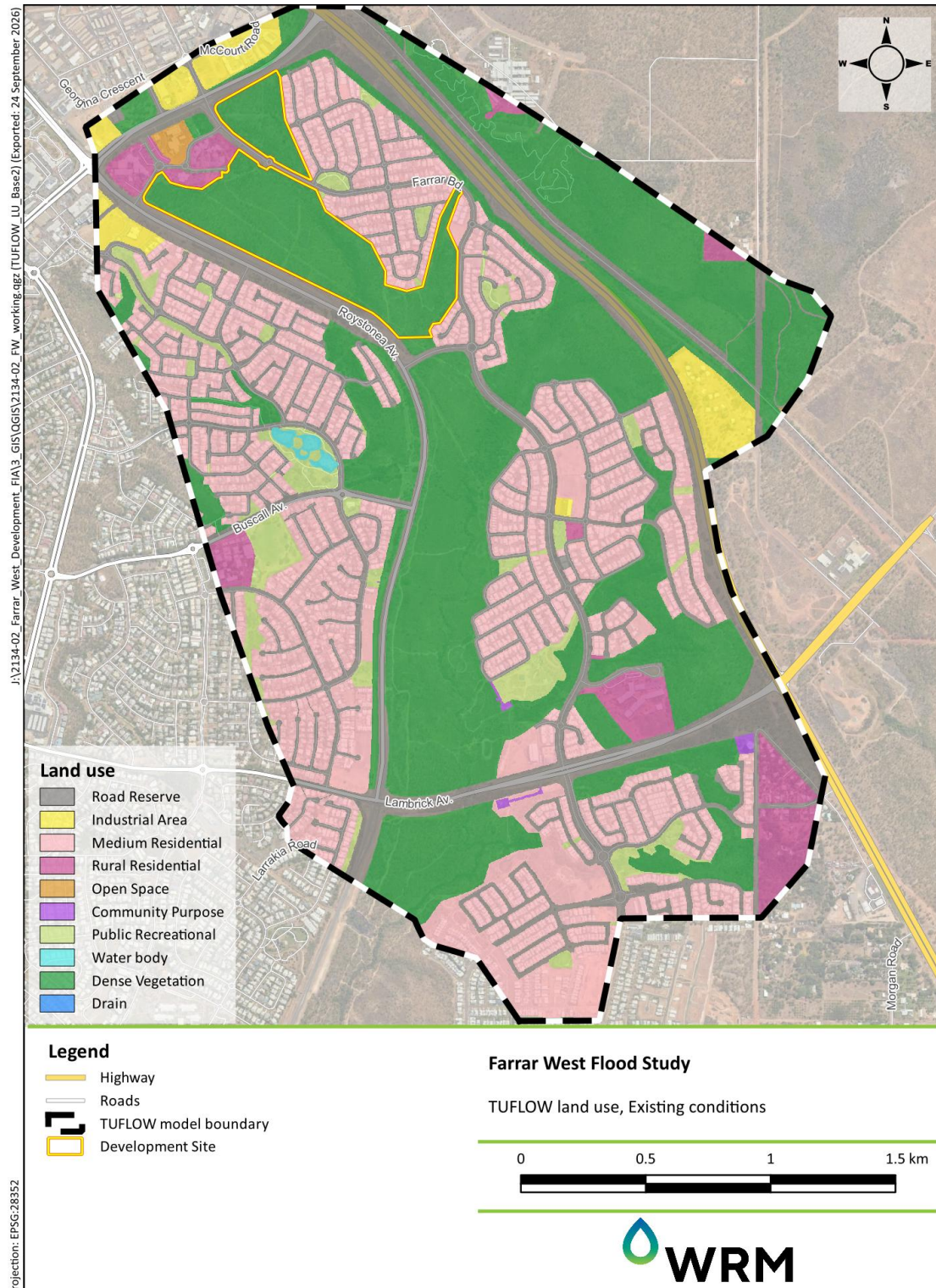


Figure 6.3 TUFLOW model hydraulic roughness mapping, existing condition

6.2.5 Hydraulic structures

Figure 6.1 shows the location of the hydraulic structures represented in the TUFLOW model. The details of the hydraulic structures (e.g. culverts, pipes and pits) were obtained from the survey provided by EJA. Appendix D presents the details of hydraulic structures included in the hydraulic model, which included the following:

- 25 road crossings culverts within the 2D domain were modelled. The details of road crossing culverts are presented in Table D.1 (Appendix D).
- 24 low flow pipes were included in the model configuration. The details of these pipes are presented in Table D.2 (Appendix D).
- 49 stormwater network pipes were embedded within the 2D domain of the TUFLOW model. Table D.3 in Appendix D shows the hydraulic characteristics adopted for each stormwater pipe.

6.2.6 Existing conditions flooding results

Figure 6.4 and Figure 6.5 show the predicted peak flood extents, levels, depths, and velocities under existing conditions for the 1% AEP event. The results show that significant proportions of the North Area and South Area are generally flood free except for within approximately 50 m of the development perimeter boundary as well as the flowpath through the centre of the South Area from TB1 to Roystonea Road and the flowpath along TB3 to Roystonea Road.

Figure E.1 to Figure E.4 in Appendix E show the predicted peak flood extents, levels, depths, and velocities under existing conditions for the 50% and 10% AEP events.

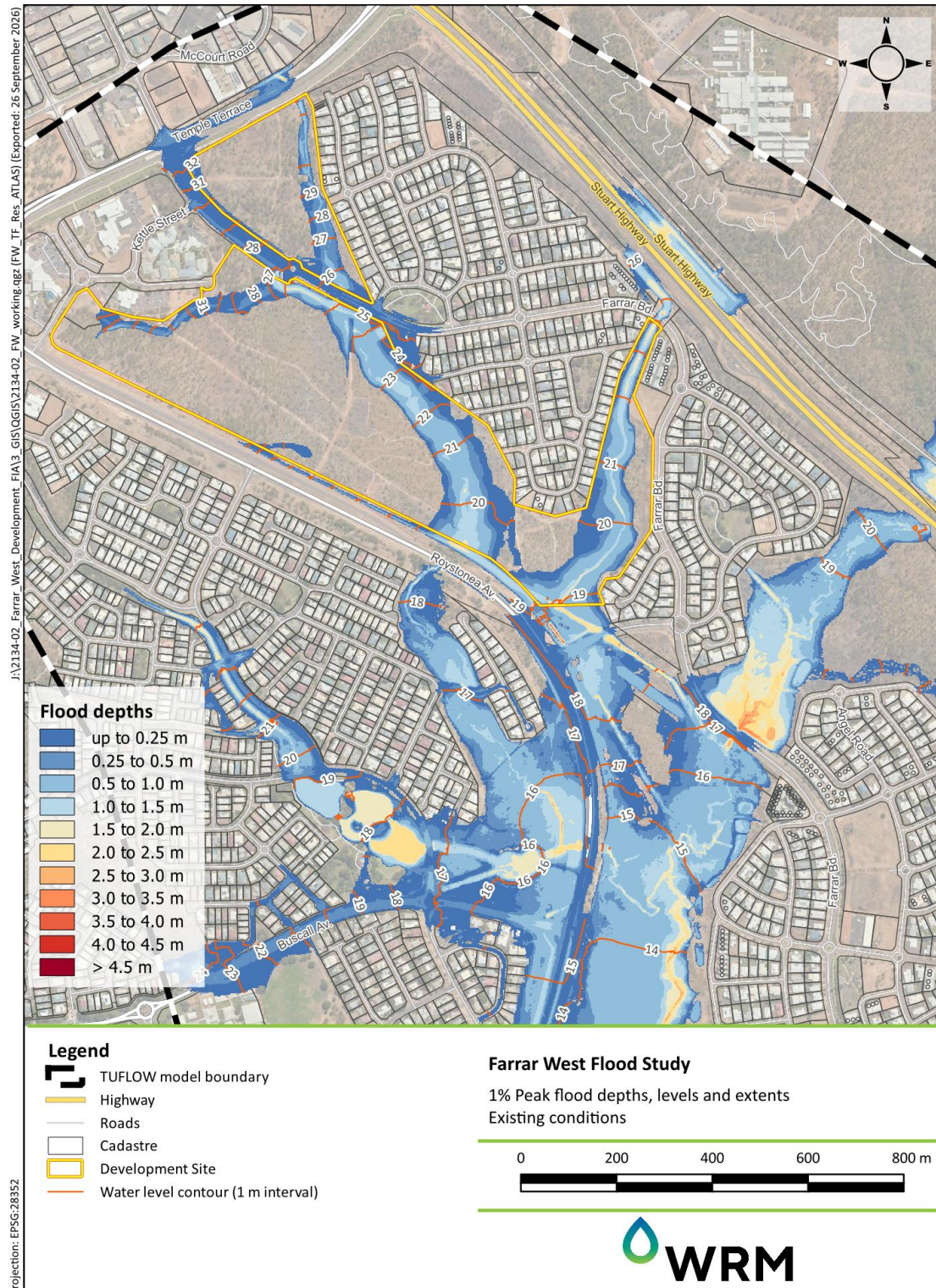


Figure 6.4 Existing conditions 1% AEP flood depths, levels and extents map

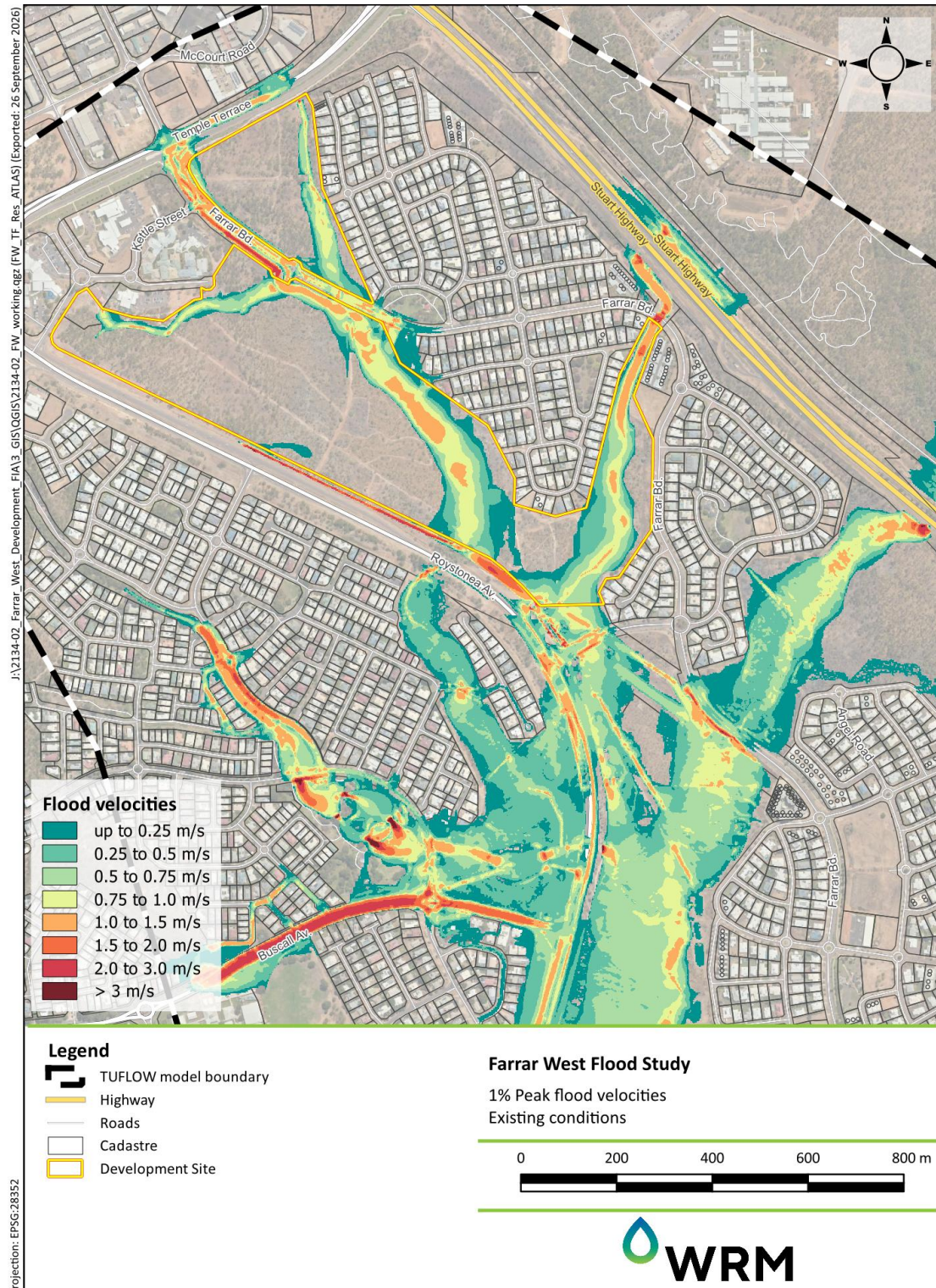


Figure 6.5 Existing conditions 1% AEP flood velocity map

6.3 ASSESSMENT OF POTENTIAL DEVELOPED CONDITIONS IMPACT

6.3.1 Developed conditions flood model changes

The existing conditions TUFLOW model was updated to include the developed conditions for the proposed Farrar West development. The developed conditions TUFLOW model configuration is presented in Figure 6.6.

The model changes were based on the concept designs provided by Colliers.

The following changes were made to the existing conditions hydraulic model configuration:

- The changes to the existing conditions topography includes:
 - A 1 m design surface of the developed residential area, the proposed detention basin and the drains connecting to the basin within the Development Site provided by Colliers on 23 September 2026;
 - TUFLOW 2D elevation modifiers (Z-shapes) were used at the following locations:
 - The basin crest which was increased to 21 mAHD;
 - The Inverway drain at downstream of the basin outlet; and
 - The basin bank at the drain towards the Roystonea Road.
- New proposed culverts were modelled as 1 Dimensional (1D) network structures based on the latest site layout provided by Colliers on 28 July 2026 and 24 August 2026. The modelled culvert details are shown in Table D.4 in Appendix D. The adopted culvert invert levels were based on the latest site layout and provided design ground surface;
- The existing culverts Cop_001, MC-014 and MC_015 were removed due to the proposed drain;
- Existing pipes MCL_0059 and MCL_0058, which cross the Farrar Boulevard and drain to the pipe MCL_0056, were removed. The proposed pipes (MCL_0058, MCL_0059 and MCL_0060) were modelled as 1 Dimensional (1D) network structures, conveying discharge to the drain, based on the latest layout design provided by Colliers on 24 August 2026. Table D.5 shows the details of proposed pipes;
- Land use mapping within the Development Site was updated and is shown in Figure 6.7, based on the proposed design provided by Hatch on 2 September 2026; and
- Updated discharge hydrographs produced by the developed conditions URBS model were applied to the developed conditions TUFLOW model as inflow hydrographs for the catchments within the Development Site, using the 'Surface Area' (SA) inflow approach.

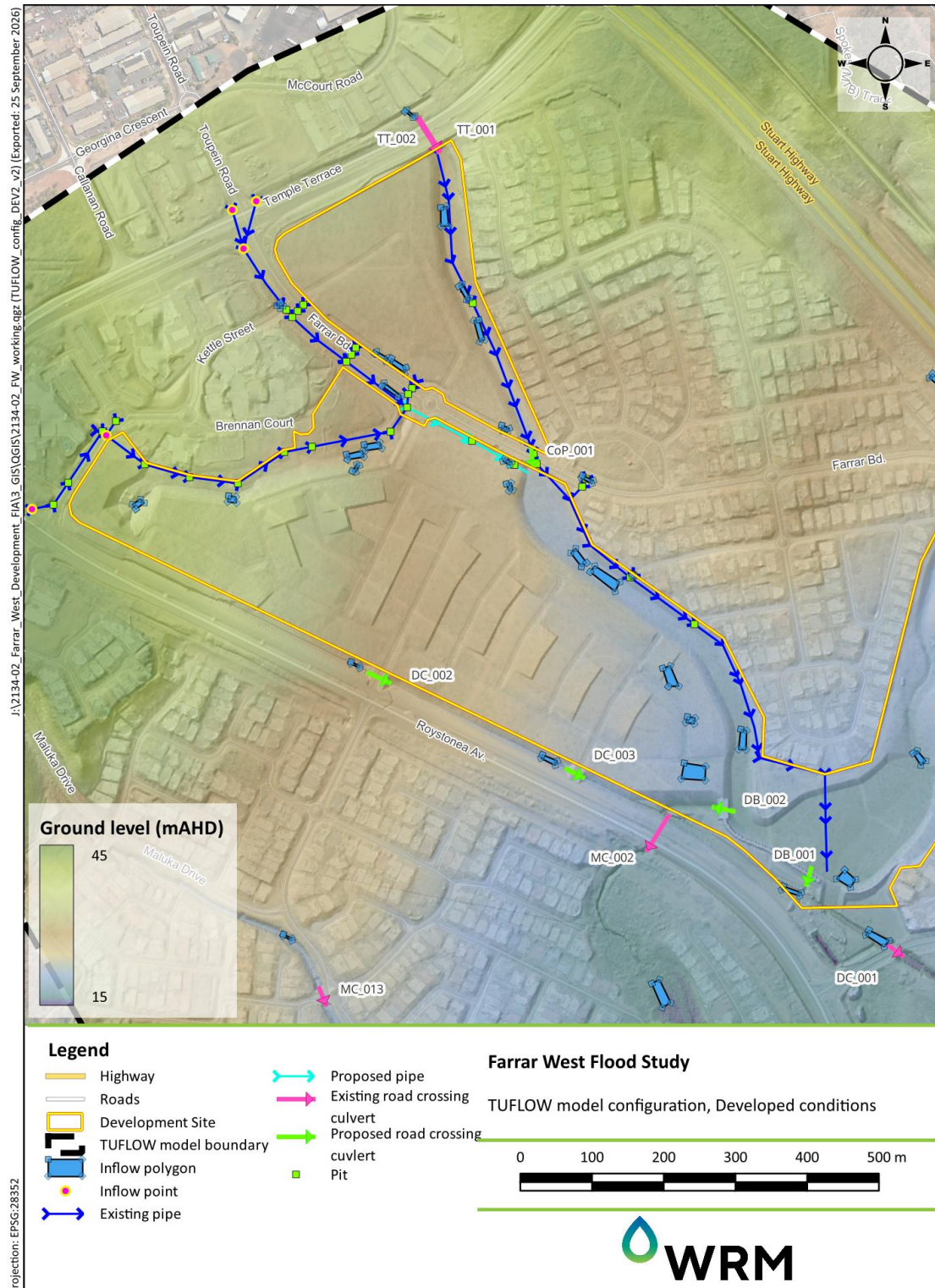


Figure 6.6 TUFLOW configuration, developed conditions

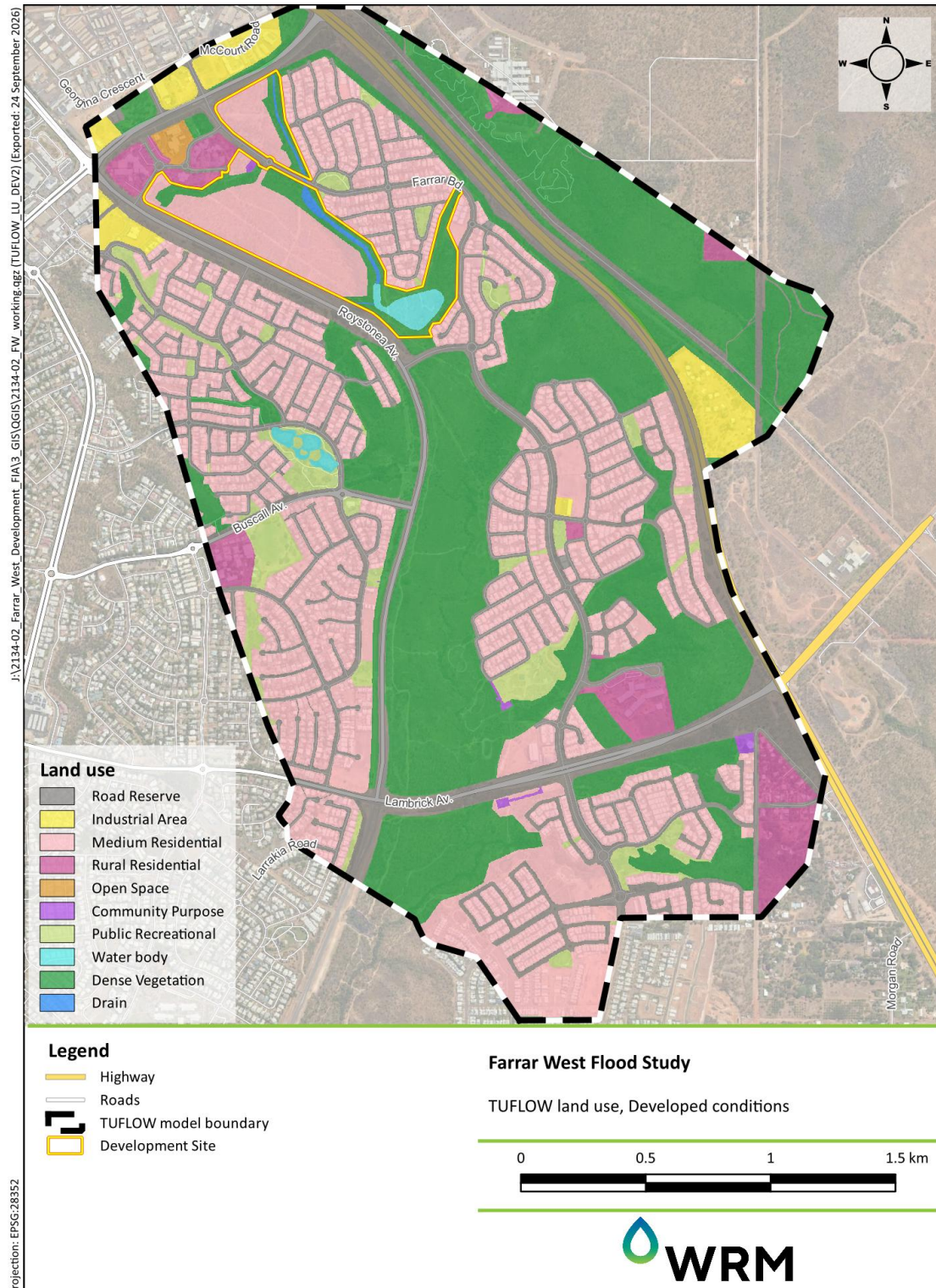


Figure 6.7 TUFLOW model land use mapping, developed conditions

6.3.2 Developed conditions flooding results

Figure 6.8 and Figure 6.9 show the predicted peak flood extents, levels, depths, and velocities during developed conditions for 1% AEP event.

Figure E.5 to Figure E.8 in Appendix E show the predicted peak flood extents, levels, depths and velocities during developed conditions for 50% and 10% AEP events.

6.3.3 Flood impact assessment

Figure 6.10 and Figure 6.11 present the changes in flood extents, levels, depths, and flood velocities due to the impact of the development including the detention basin for 1% AEP event.

Results from the hydraulic modelling indicate that the developed conditions would result in some changes in the 1% AEP flood behaviour in the vicinity of the Development Site as follows:

- **Increased adjacent flood levels and extent:** Flood levels are predicted to increase mainly along the Farrar Boulevard within the South Site and at the proposed detention basin. Localised increases also occur along the western boundary of the South Site. Areas shown as previously dry and wet under developed conditions occur locally at the northwest corner of the North Site and in the proposed open drain and detention basin within the Development Site.
- **Reduced adjacent flood levels and extent:** Flood levels are predicted to decrease along the open drains which are upstream of the basin. Decreases also occur over a significant area downstream area of the Development Site. Areas shown as wet under existing conditions and dry under developed conditions occur locally along the former flowpaths and around the proposed drainage and detention basin works.
- **Change in flow velocities:** Flood velocities generally increase within the proposed drains by up to 2.0 m/s, while velocities downstream of the detention basin are reduced by up to 1 m/s. It is recommended that suitable erosion protection is provided along the drains (particularly at drop structures) and the detention basin inlet and outlet structures.

Figure E.9 to Figure E.12 in Appendix E show the changes in flood extents, levels and flood velocities due to the impact of the development including the detention basin for 50% and 10% AEP events.

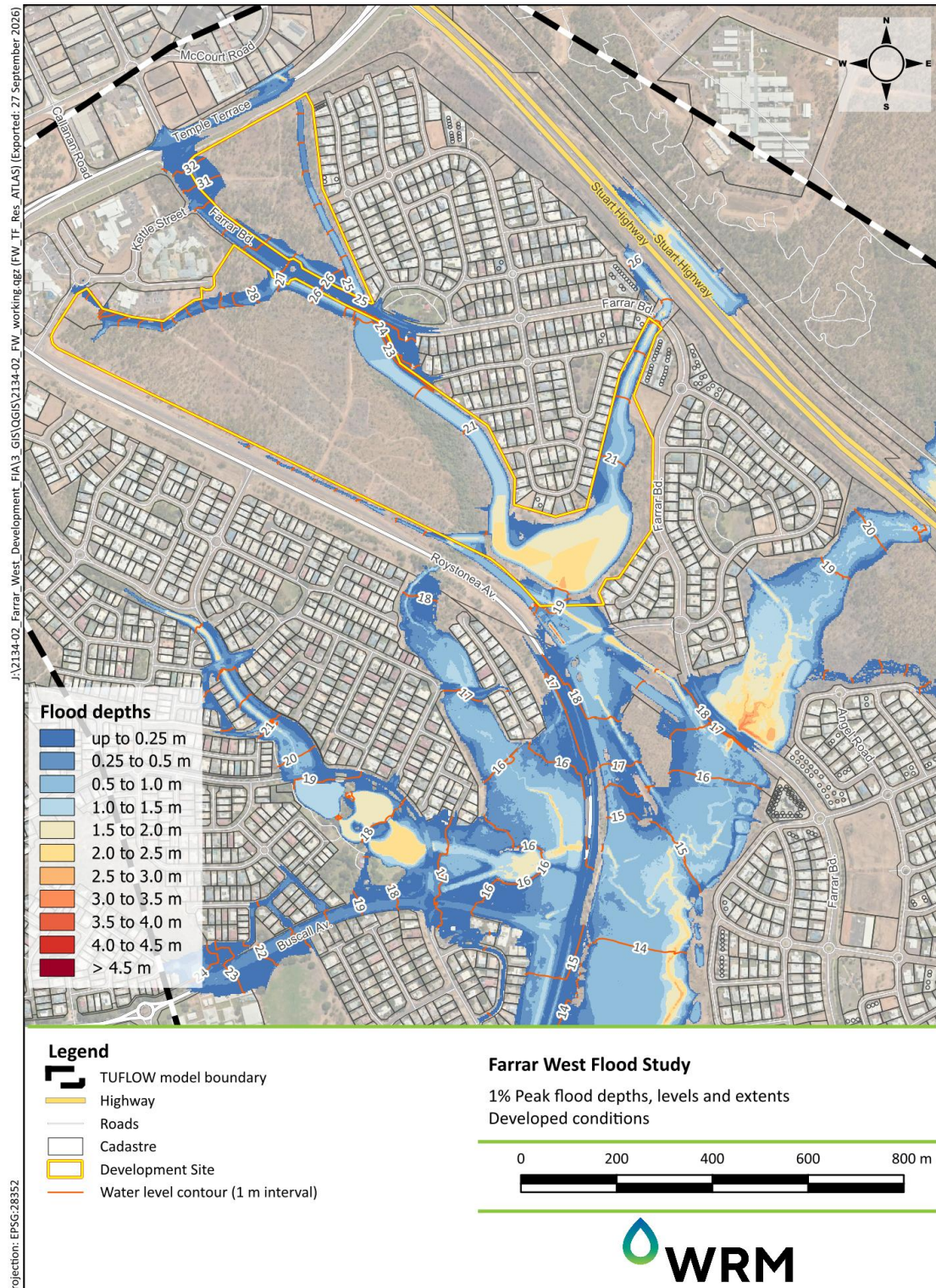


Figure 6.8 Developed conditions 1% AEP flood depth map

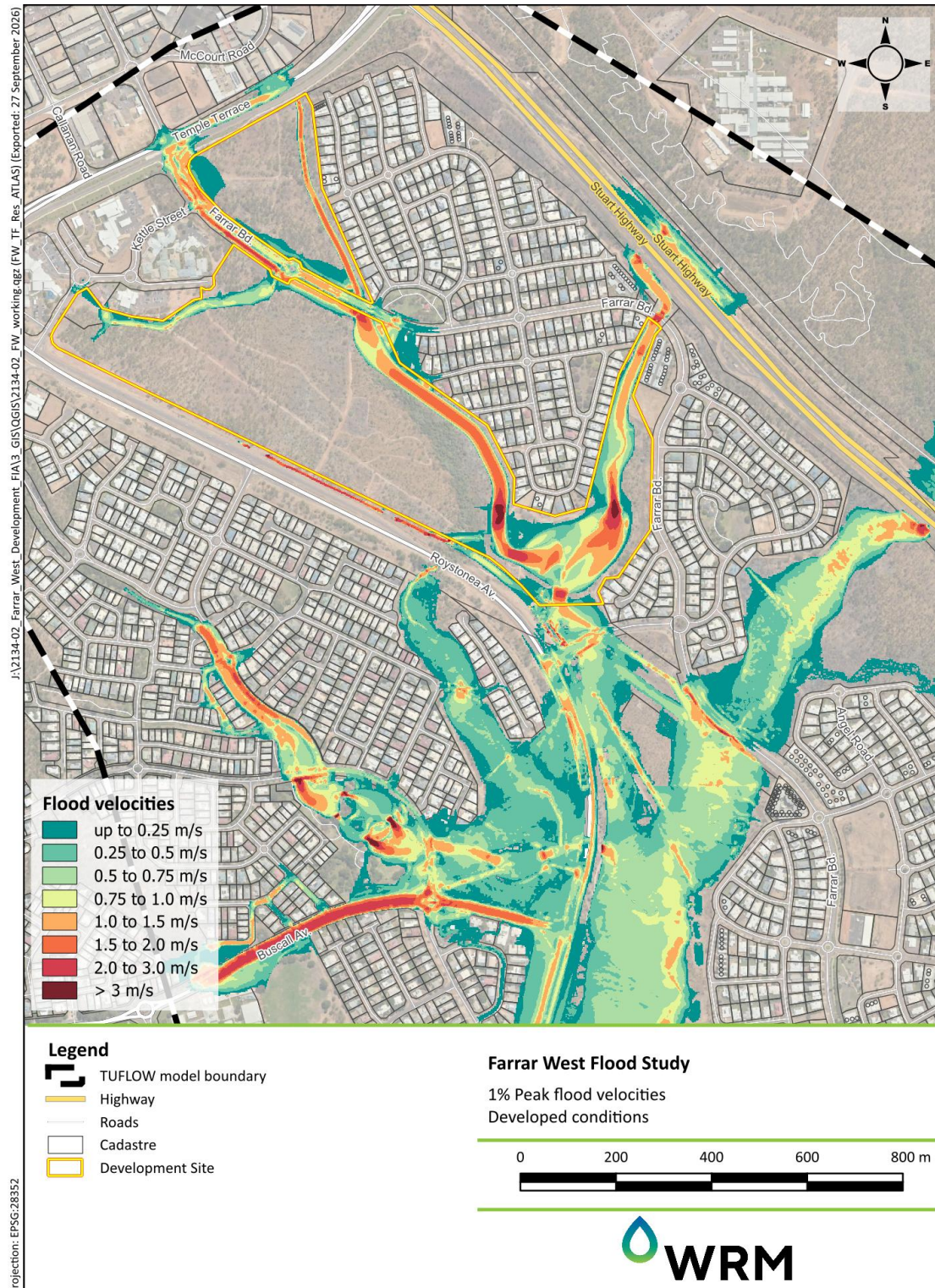


Figure 6.9 Developed conditions 1% AEP flood velocity map

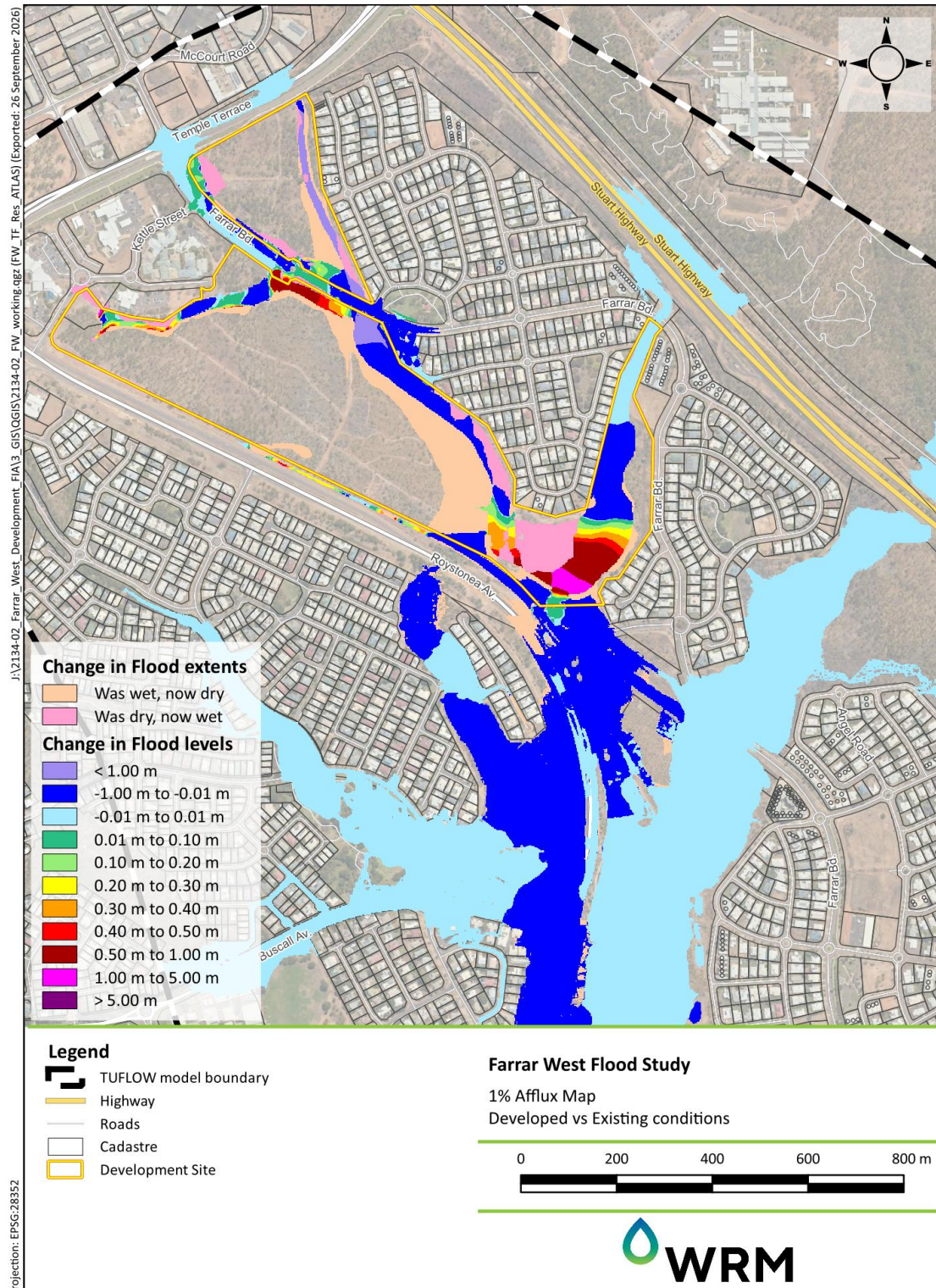


Figure 6.10 Change in flood levels map for 1% AEP event

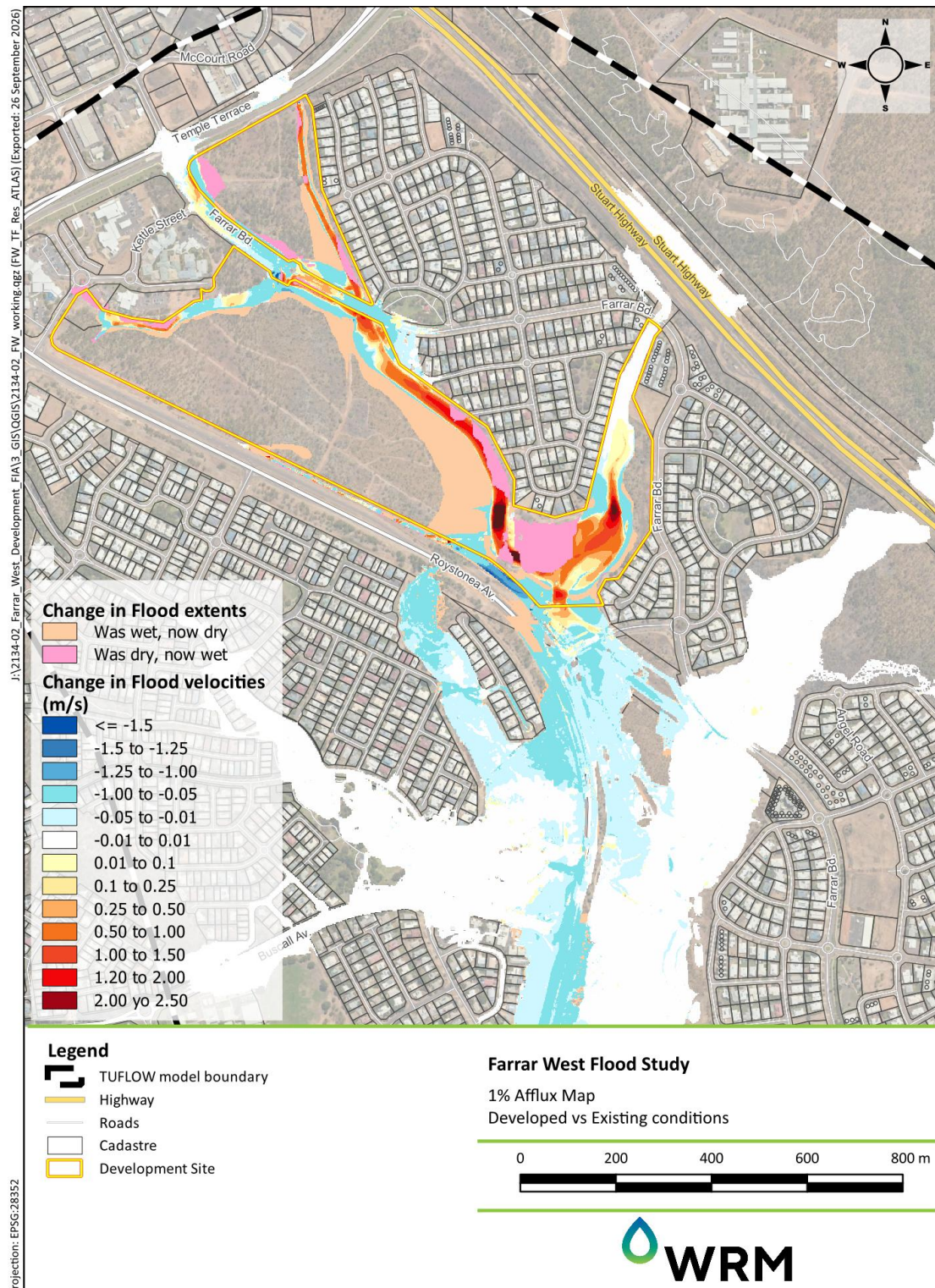


Figure 6.11 Change in flood velocities map for 1% AEP event

7 CONCLUSIONS

WRM was requested by Cuttone, on behalf of LDC, to undertake a flood impact assessment of Farrar West development under existing and developed conditions for the 50% AEP, 10% AEP and 1% AEP events.

Mitchell Creek URBS hydrologic and TUFLOW hydraulic models were developed for existing and developed conditions. The developed conditions models assessed the Development Site layout including concept designs of the proposed detention basin and drains provided by Colliers.

The URBS hydrologic models were validated by comparing the URBS design discharges with Rational Method discharge estimates. The validated URBS model was then used to produce estimated design discharge hydrographs for a range of events from 50% AEP to 1% AEP events.

The TUFLOW hydraulic models were used to predict peak flood extents, depths, levels, and flood velocities for existing and developed conditions for the three design events. Flood maps were produced for all design events modelled.

The flood modelling results show that the conceptual detention basin configuration is effective in mitigating the impact of the Farrar West development on downstream flooding. The detention basin results in a reduction in the 1% AEP peak discharges off the Development Site, which produces a reduction in downstream flood levels. Some increases in 1% AEP peak flood levels were predicted, however these impacts were primarily limited to the Development Site and did not extend into any private property.

It is acknowledged that the structures represented in the flood model may differ from the civil design at this stage of the design. It is expected that the flood model will be reviewed and updated in future stages as the design progresses and the final civil design layout is confirmed. Additionally, detailed survey for Farrar Boulevard was unavailable at the time of this assessment and 2018 LiDAR data was used to represent the existing ground surface in this location. Modelling will be updated to include feature survey once the detailed survey is available.

8 REFERENCES

Ball et al., 2019	'Australian Rainfall and Runoff: A Guide to Flood Estimation', Ball J, Banister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors), © Commonwealth of Australia (Geoscience Australia), 2019.
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DIPL, 2025	'Northern Territory Subdivision Development Guidelines', Department of Infrastructure, Planning and Logistics, Darwin NT, 2025.
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APPENDIX A PROPOSED FARRAR WEST DEVELOPMENT

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Figure A.1 Conceptual Plan

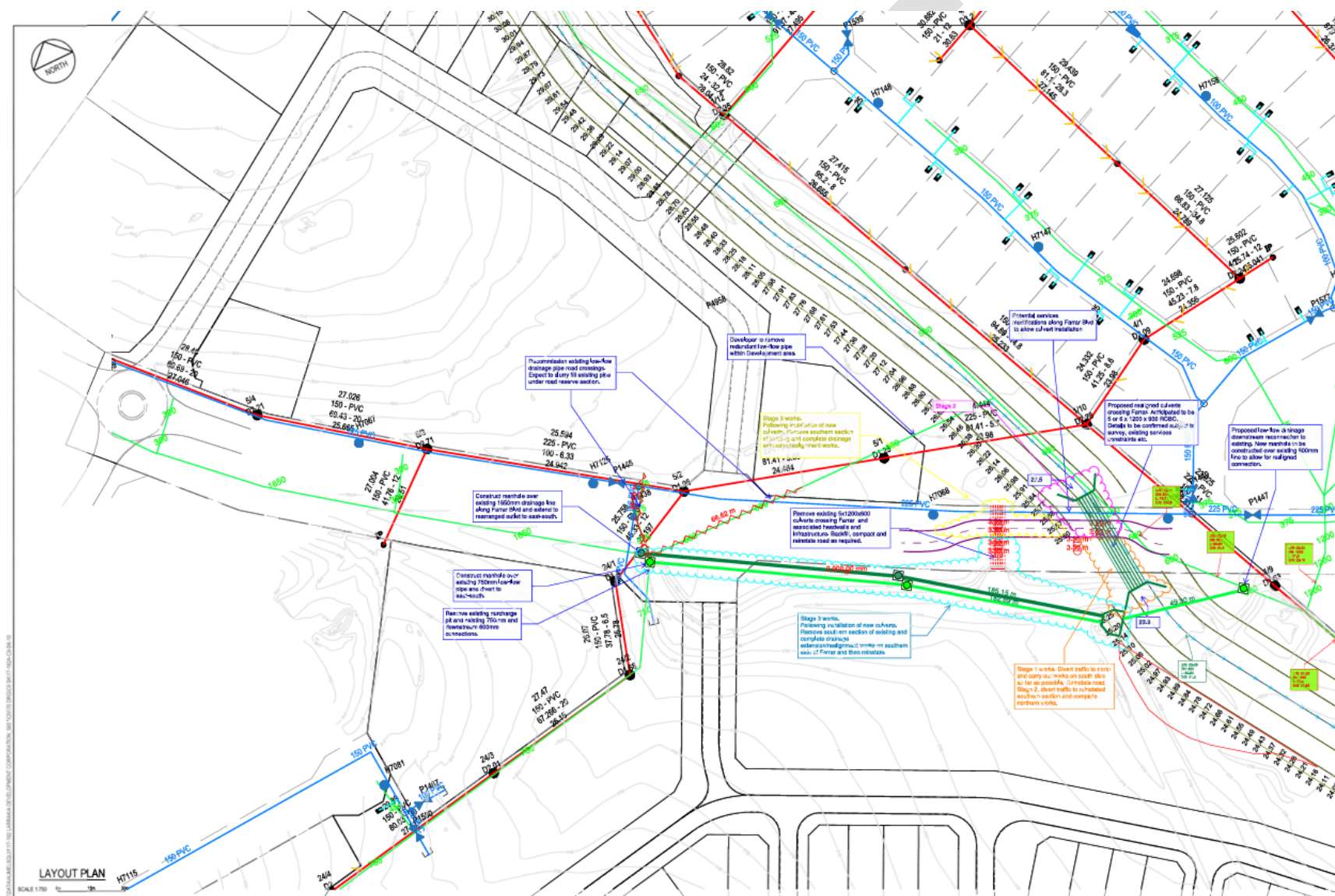


Figure A.2 Farrar West concept stormwater drainage design

APPENDIX B HYDROLOGICAL MODEL CONFIGURATION

DRAFT

Table B.1 Hydrologic sub-catchment characteristics for existing conditions model

Sub-catchment	Area (km2)	Impervious	CS (m/m)
2514	0.345	0.2	0.047
2601	2.461	0.469	0.056
2602	1.427	0.187	0.054
2603	1.013	0.287	0.049
2604	1.36	0.307	0.051
2605	0.414	0.2	0.042
2606	0.594	0.201	0.079
2607	0.501	0.2	0.05
2609	0.377	0.2	0.037
2610	0.249	0.28	0.04
2611	0.498	0.277	0.037
2613	0.738	0.14	0.029
2614	0.603	0.066	0.022
2615	0.185	0.272	0.039
2616	0.282	0.227	0.043
2702	1.635	0.235	0.045
2707	0.829	0.492	0.029
2709	1.264	0.566	0.037
2710	0.574	0.547	0.04
2712	0.871	0.368	0.037
2714	0.51	0.592	0.049
2715	1.834	0.554	0.054
2718	1.643	0.179	0.056
2825	1.306	0.106	0.028
2826	1.571	0.062	0.03
2827	1.353	0.083	0.047
2860	1.06	0.056	0.014
2861	1.154	0.116	0.018
260801	0.49	0.6	0.055
260802	0.254	0.601	0.043
260803	0.113	0.6	0.054
260804	0.995	0.228	0.036
261201	0.483	0.598	0.044
261202	0.997	0.176	0.029
270101	0.013	0.527	0.051
270102	0.052	0.522	0.06

Sub-catchment	Area (km2)	Impervious	CS (m/m)
270103	0.144	0.625	0.077
270104	0.469	0.855	0.062
270105	0.148	0.408	0.049
270106	0.015	0.487	0.039
270107	0.01	0.669	0.118
270108	0.038	0	0.036
270109	0.026	0.506	0.087
270301	0.475	0.319	0.048
270302	0.499	0.658	0.061
270303	0.172	0.129	0.041
270401	0.096	0.941	0.04
270402	0.035	0.909	0.07
270403	0.026	0.003	0.045
270404	0.038	0.914	0.048
270405	0.044	0.913	0.075
270406	0.173	0.907	0.055
270407	0.058	0.46	0.084
270408	0.013	0.001	0.029
270409	0.019	0.312	0.057
270413	0.073	0.522	0.054
270414	0.043	0.575	0.053
270415	0.035	0	0.029
270416	0.059	0.002	0.037
270417	0.025	0.038	0.036
270418	0.03	0.828	0.088
270419	0.344	0.521	0.054
270420	0.013	0	0.032
270421	0.017	0	0.058
270422	0.007	0.426	0.069
270424	0.026	0.051	0.042
270425	0.037	0	0.044
270426	0.031	0.781	0.092
270427	0.007	0.85	0.035
270428	0.021	0.825	0.061
270429	0.01	0.246	0.091
270430	0.005	0	0.089
270431	0.02	0.02	0.071

Sub-catchment	Area (km2)	Impervious	CS (m/m)
270432	0.01	0	0.045
270433	0.004	0	0.056
270501	0.456	0.541	0.058
270502	0.509	0.389	0.047
270503	0.312	0.483	0.043
270601	0.959	0.591	0.039
270602	0.736	0.243	0.022
271301	0.427	0.307	0.05
271302	0.186	0.6	0.079
271303	0.453	0.678	0.042
271304	0.045	0.6	0.051
27040101	0.031	0.823	0.046
27040102	0.049	0.894	0.057

Table B.2 Hydrologic sub-catchment characteristics for developed conditions model

Sub-catchment	Area (km2)	Impervious	CS (m/m)
2514	0.345	0.2	0.047
2601	2.461	0.469	0.056
2602	1.427	0.187	0.054
2603	1.013	0.287	0.049
2604	1.36	0.307	0.051
2605	0.414	0.2	0.042
2606	0.594	0.201	0.079
2607	0.501	0.2	0.05
2609	0.377	0.2	0.037
2610	0.249	0.28	0.04
2611	0.498	0.277	0.037
2613	0.738	0.14	0.029
2614	0.603	0.066	0.022
2615	0.185	0.272	0.039
2616	0.282	0.227	0.043
2702	1.635	0.235	0.045
2707	0.829	0.492	0.029
2709	1.264	0.566	0.037
2710	0.574	0.547	0.04
2712	0.871	0.368	0.037
2714	0.51	0.592	0.049
2715	1.834	0.554	0.054
2718	1.643	0.179	0.056
2825	1.306	0.106	0.028
2826	1.571	0.062	0.03
2827	1.353	0.083	0.047
2860	1.06	0.056	0.014
2861	1.154	0.116	0.018
260801	0.49	0.6	0.055
260802	0.254	0.601	0.043
260803	0.113	0.6	0.054
260804	0.995	0.228	0.036
261201	0.483	0.598	0.044
261202	0.997	0.176	0.029
270101	0.013	0.527	0.051
270102	0.052	0.522	0.06

Sub-catchment	Area (km2)	Impervious	CS (m/m)
270103	0.144	0.625	0.077
270104	0.469	0.855	0.062
270105	0.148	0.408	0.049
270106	0.015	0.444	0.039
270107	0.01	0.669	0.061
270108	0.038	0	0.049
270109	0.026	0.491	0.08
270301	0.475	0.319	0.048
270302	0.499	0.658	0.061
270303	0.172	0.129	0.041
270401	0.096	0.941	0.04
270402	0.035	0.903	0.07
270403	0.026	0.798	0.033
270404	0.038	0.914	0.048
270405	0.044	0.913	0.075
270406	0.173	0.907	0.055
270407	0.058	0.46	0.082
270408	0.013	0.888	0.029
270409	0.019	0.312	0.127
270413	0.073	0.522	0.054
270414	0.043	0.575	0.053
270415	0.035	0.6	0.033
270416	0.059	0.6	0.038
270417	0.025	0.556	0.032
270418	0.03	0.829	0.098
270419	0.344	0.521	0.054
270420	0.013	0.507	0.023
270421	0.017	0.169	0.055
270422	0.007	0.451	0.06
270424	0.026	0.043	0.082
270425	0.037	0.6	0.054
270426	0.031	0.78	0.086
270427	0.007	0.85	0.035
270428	0.021	0.884	0.059
270429	0.01	0.647	0.074
270430	0.005	0.589	0.03
270431	0.02	0.157	0.155

Sub-catchment	Area (km2)	Impervious	CS (m/m)
270432	0.01	0.707	0.03
270433	0.004	0.61	0.038
270501	0.456	0.541	0.058
270502	0.509	0.389	0.047
270503	0.312	0.483	0.043
270601	0.959	0.591	0.039
270602	0.736	0.243	0.022
271301	0.427	0.307	0.05
271302	0.186	0.6	0.079
271303	0.453	0.678	0.042
271304	0.045	0.6	0.051
27040101	0.031	0.823	0.046
27040102	0.049	0.894	0.057

APPENDIX C RATIONAL METHOD CALCULATIONS

Catchment:		270413				
Catchment area and coefficient of runoff						
Catchment Area (ha)		7.3				
C ₁₀		0.8				
Time of concentration						
Standard inlet time (mins)		13				
Pipe length (m)		165	150	0		
Pipe velocity (m/s)		0.95	0.7	1		
Pipe travel time (mins)		2.89	3.57	0.00		
t _c (mins)		19.5				
Design Discharges						
ARI (years)	AEP (%)	Frequency Factor F _y	C _y	t _c (mins)	Rainfall Intensity (mm/h)	Peak Discharge (m ³ /s)
1	63	0.80	0.64	19.5	111.0	1.44
1.44	50	0.85	0.68	19.5	120.3	1.66
4.48	20	0.95	0.76	19.5	147.9	2.28
10	10	1.00	0.80	19.5	166.0	2.69
20	5	1.05	0.84	19.5	181.8	3.10
50	2	1.15	0.92	19.5	201.3	3.76
100	1	1.20	0.96	19.5	215.9	4.20

Figure C.1 Rational Method calculations at catchment 270413

Catchment:		Catchment 270416							
Catchment area and coefficient of runoff									
Catchment Area (ha)		5.90							
C ₁₀		0.70							
Overland flow travel time (Friend's Equ)									
Horton's 'n'		0.065							
Length (m)		100							
Slope (m/m)		0.035							
Overland flow travel time (mins)		25.1							
Channel characteristics									
Channel length (m)		270							
Channel slope (m/m)		0.032							
Manning's 'n'		0.065							
Channel bottom width (m)		5.00							
Channel side slope (m/m)		0.050							
Design Discharges									
ARI	AEP	Frequency Factor	C _y	Channel Velocity ^a	Channel Travel Time	t _c ^b	Rainfall Intensity	Peak Discharge	
(years)	(%)	F _y		(m/s)	(mins)	(mins)	(mm/h)	(m ³ /s)	
1	63	0.80	0.56	0.64	7.03	32.1	87.6	0.80	
1.44	50	0.85	0.60	0.67	6.74	31.8	95.7	0.93	
4.48	20	0.95	0.67	0.73	6.14	31.2	119.7	1.30	
10	10	1.00	0.70	0.77	5.88	31.0	133.3	1.53	
20	5	1.05	0.74	0.80	5.65	30.7	146.8	1.77	
50	2	1.15	0.81	0.84	5.36	30.4	163.0	2.15	
100	1	1.20	0.84	0.87	5.20	30.3	174.2	2.40	
a - Channel velocity calculated using Mannings's equation									
b - Time of Concentration (t _c) = Overland Flow Travel Time + Channel Travel Time									

Figure C.2 Rational Method calculations at catchment 270416

APPENDIX D TUFLOW MODEL HYDRAULIC STRUCTURES

D.1 EXSITING CONDITIONS TUFLOW MODEL STRUCTURES

DRAFT

Table D.1 Adopted existing road crossing culverts details

Culvert ID	Type	US Invert (mAHD)	DS Invert (mAHD)	Diameter/Width (m)	Height (m)	No. of barrels
MC_001	RCP	27.13	24.9	1.6	-	2
MC_002	RCBC	17.46	17.315	1.2	0.9	3
MC_003	RCBC	15.57	15.305	2.1	2.1	2
MC_004	RCP	34.94	34.59	1.2	-	2
MC_005	RCP	37.05	37.02	0.75	-	2
MC_006	RCBC	17.91	17.69	1.5	0.9	2
MC_008	RCBC	12.72	12.69	1.2	0.6	5
MC_009	RCBC	11.02	10.57	1.2	0.6	7
MC_010	RCP	18.61	18.28	0.75	-	1
MC_011	RCBC	11.56	11.23	2.7	1.5	4
MC_012	RCBC	13.78	13.53	1.2	1.2	11
MC_013	RCBC	21.1	20.95	1.2	0.9	5
MC_021	RCBC	18.8	17.405	1.8	0.9	4
CoP_001	RCBC	24.68	23.69	1.2	0.9	5
CoP_002	RCBC	15.76	15.66	1.8	0.6	6
CoP_004	RCBC	15.61	15.52	2.1	0.6	5
CoP_003	RCBC	16.5	16.08	0.9	0.3	1
CoP_006	RCBC	16.52	16.5	0.9	0.3	1
CoP_005	RCBC	14.7	14.41	1.5	0.9	2
MC_023	RCBC	20.04	19.84	2.7	1.2	5
TT_001	RCP	30.84	30.064	1.35	-	1
TT_002	RCP	31.27	31.131	0.45	-	1
MC_014	RCBC	23.05	22.95	1.2	0.9	4
MC_015	RCBC	28.75	28.7	1.2	0.625	3
DC_001	RCBC	16.34	16.2	4.65	0.9	4

Table D.2 Adopted model RCP low flow pipes details

ID	US Invert (mAHD)	DS Invert (mAHD)	Diameter (m)	No. of barrels
MCLF_001	7.51	7.25	0.525	1
MCLF_002	25.52	25.17	0.525	1
MCLF_003	17.18	17	0.525	1
MCLF_004	15.5	15.46	0.525	1
MCLF_005	14.4	14.38	0.525	1
MCLF_006	14.11	14.1	0.525	1
MCLF_007	14.06	13.96	0.525	1
MCLF_008	14.03	13.97	0.525	1
MCLF_009	13.58	13.24	0.525	1
MCLF_010	13.5	13.08	0.525	1
MCLF_011	12.65	11.96	0.525	1
MCLF_012	14.1	14.05	0.525	1
MCLF_013	9.5	9.03	0.525	1
MCLF_014	9	8.77	0.525	1
MCLF_015	13.95	13.89	0.525	1
MCLF_016	22.7	22.65	0.525	1
MCLF_017	16	15.77	0.525	1
MCLF_018	14.7	14.65	0.525	1
MCLF_019	12.2	12.04	0.525	1
MCLF_020	12.2	12.07	0.525	1
MCLF_021	8.65	8.51	0.525	1
MCLF_022	8	7.59	0.525	1
MCLF_023	9.535	9.2	0.525	1
MCLF_024	8	4.7	0.525	1

Table D.3 Adopted TUFLOW model existing RCP pipes details

ID	US Invert (mAHD)	DS Invert (mAHD)	Diameter (m)	No. of barrels
MCL_0001	27.4	27.2	0.45	1
MCL_0002	27.6	27.2	1.35	1
MCL_0009	21.43	21.35	0.75	1
MCL_0010	20.8	20.45	0.6	1
MCL_0011	21.25	20.86	0.6	1
MCL_0013	22.2	21.3	0.6	1
MCL_0014	22.3	22.27	1.2	1
MCL_0015	22.34	22.26	0.6	1
MCL_0017	20.4	19.8	0.6	1
MCL_0018	19.75	19.22	0.6	1
MCL_0019	19.1	18.64	0.6	1
MCL_0020	18.59	18.13	0.6	1
MCL_0021	16.96	16.62	0.6	1
MCL_0022	16.57	16.23	0.6	1
MCL_0056	22.77	22.39	0.6	1
MCL_0057	25.65	24.5	0.6	1
MCL_0058	23.3	22.82	0.6	1
MCL_0059	23.53	23.35	0.6	1
MCL_0060	23.3	22.82	0.6	1
MCL_0061	22.77	22.39	0.6	1
MCL_0062	22.2	21.3	0.6	1
MCL_0063	22.25	22.1	1.2	1
MCL_0064	24.58	24.47	0.675	1
MCL_0065	25.47	24.62	0.675	1
MCL_0066	26.12	25.52	0.675	1
MCL_0067	24.86	23.53	1.65	1
MCL_0068	26.7	26.6	0.3	1
MCL_0069	26.55	26.21	0.3	1
MCL_0070	26.1	24.9	1.65	1
MCL_0071	27.18	26.22	1.65	1
MCL_0072	27.84	27.52	0.3	1
MCL_0073	27.99	27.89	0.3	1
MCL_0074	26.85	25.7	0.6	1
MCL_0080	28.3	26.9	0.6	1
MCL_0081	28.8	28.35	0.6	1

ID	US Invert (mAHD)	DS Invert (mAHD)	Diameter (m)	No. of barrels
MCL_0082	29.25	28.85	0.6	1
SW001	39.2	38.65	0.375	1
SW002	37.45	35.5	1.2	1
SW003	35.37	33.53	0.6	1
SW004	33.42	31.35	0.6	1
SW005	31.29	29.76	0.75	1
SW006	29.71	28.78	0.75	1
SW007	28.72	25.57	0.75	1
SW008	25.39	23.64	0.75	1
SW012	37.75	37.5	1.2	1
SW013	38.37	38.16	1.2	1
SW014	42.05	41.47	0.6	1
SW015	41.414	40.49	0.6	1
SW016	40.22	38.104	0.75	1

D.2 DEVELOPED CONDITIONS TUFLOW MODEL STRUCTURES

Table D.4 Proposed TUFLOW model road crossing culverts details

Culvert ID	Type	US Invert (mAHD)	DS Invert (mAHD)	Diameter/Width (m)	Height (m)	No. of barrels
CoP_001	RCBC	22.9	22.5	1.2	0.6	7
DB_001	RCBC	18.5	18.0	0.9	0.6	3
DB_002	RCBC	17.65	17.57	0.9	0.6	3
DC_002	RCBC	26.878	26.15	1.2	0.6	2
DC_003	RCBC	20.35	20.3	1.2	0.6	4

Table D.5 Proposed TUFLOW model RCP pipes details

ID	US Invert (mAHD)	DS Invert (mAHD)	Diameter (m)	No. of barrels
MCL_0058	23.53	23.37	1.5	2
MCL_0059	23.53	23.37	1.5	2
MCL_0060	23.36	23.3	1.8	1

APPENDIX E FLOOD MAPS

E.1 EXISTING CONDITIONS FLOOD MAPS

DRAFT

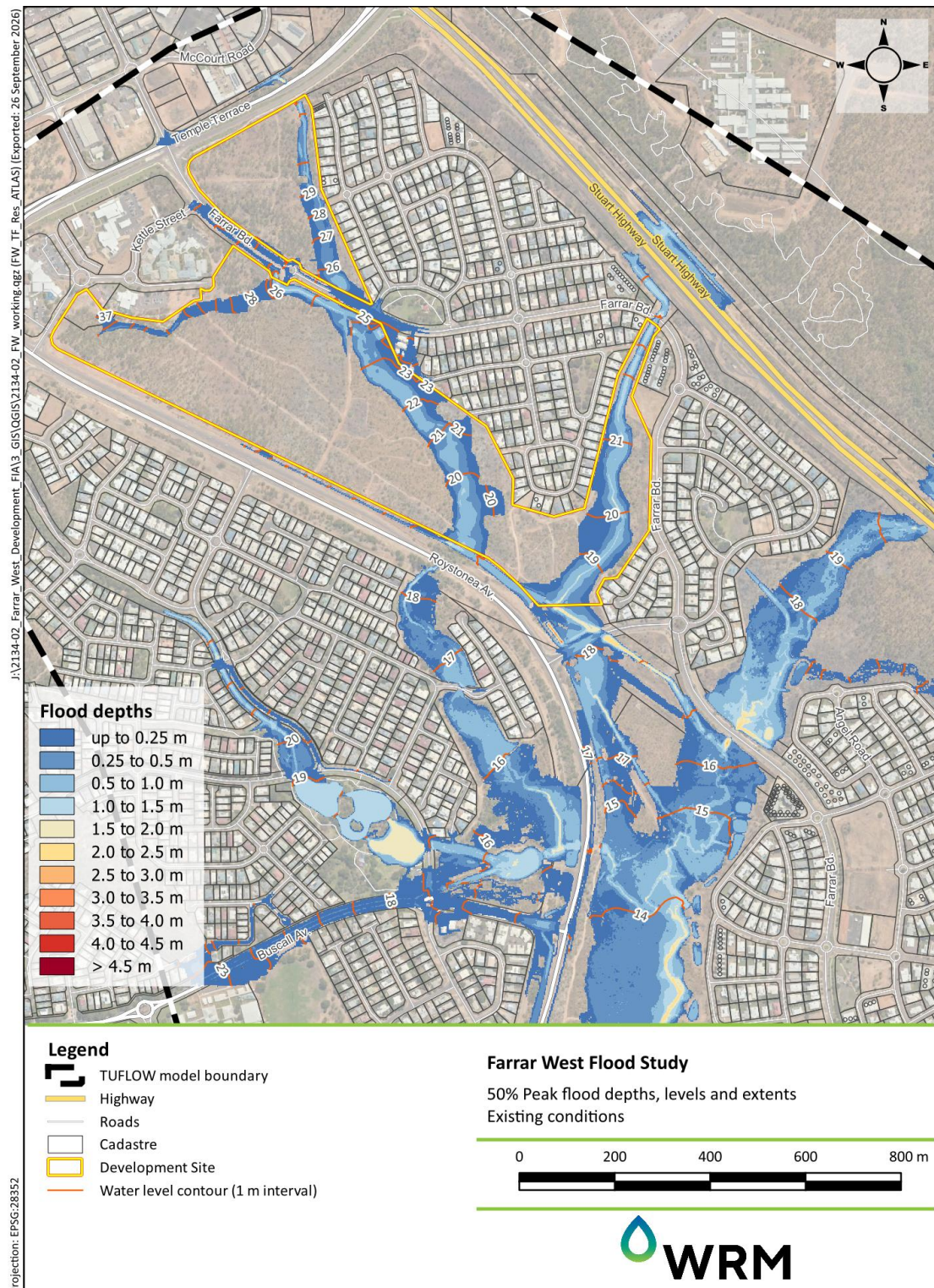


Figure E.1 Existing conditions 50% AEP flood depths, levels and extents map

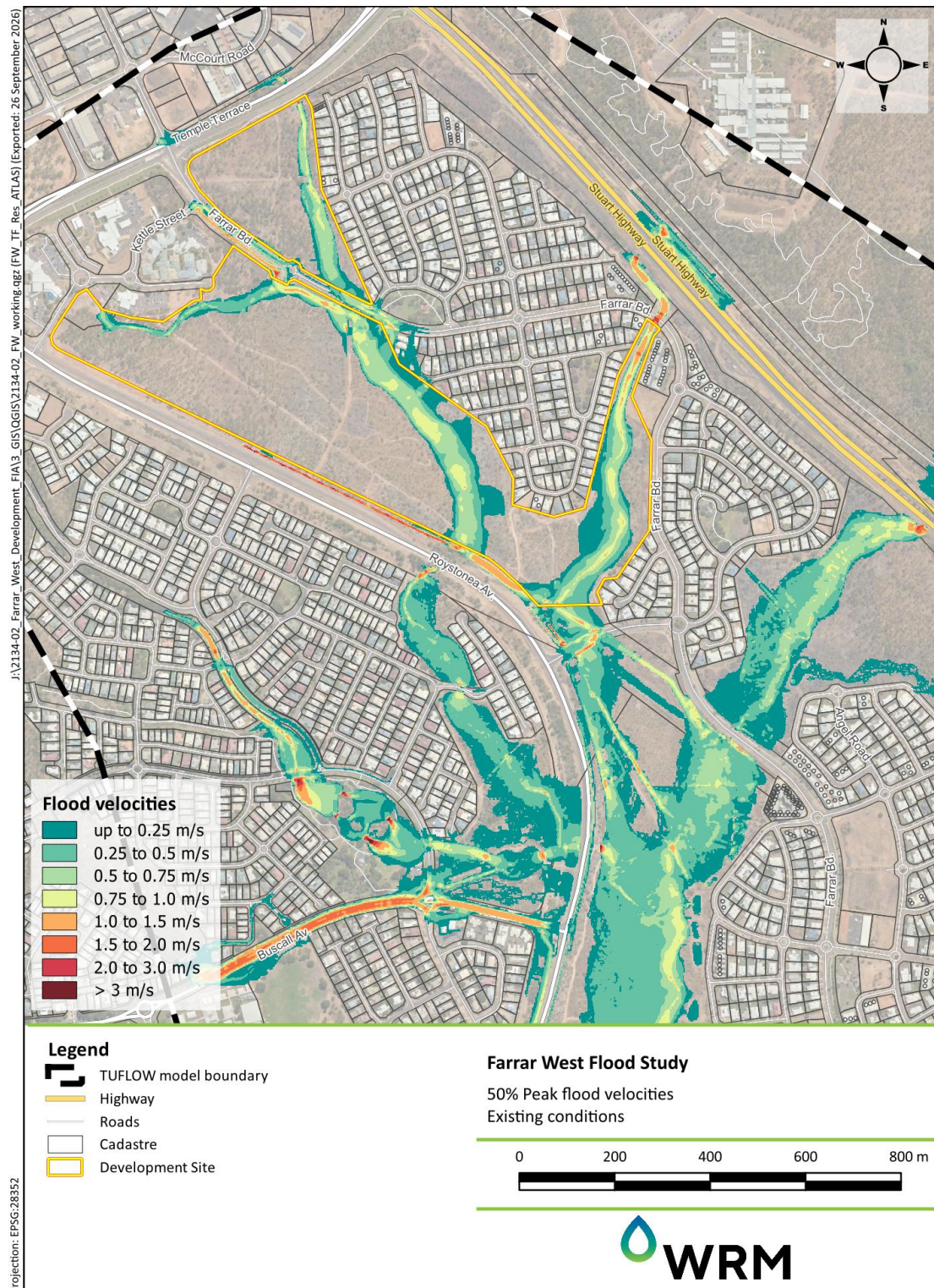


Figure E.2 Existing conditions 50% AEP flood velocities map

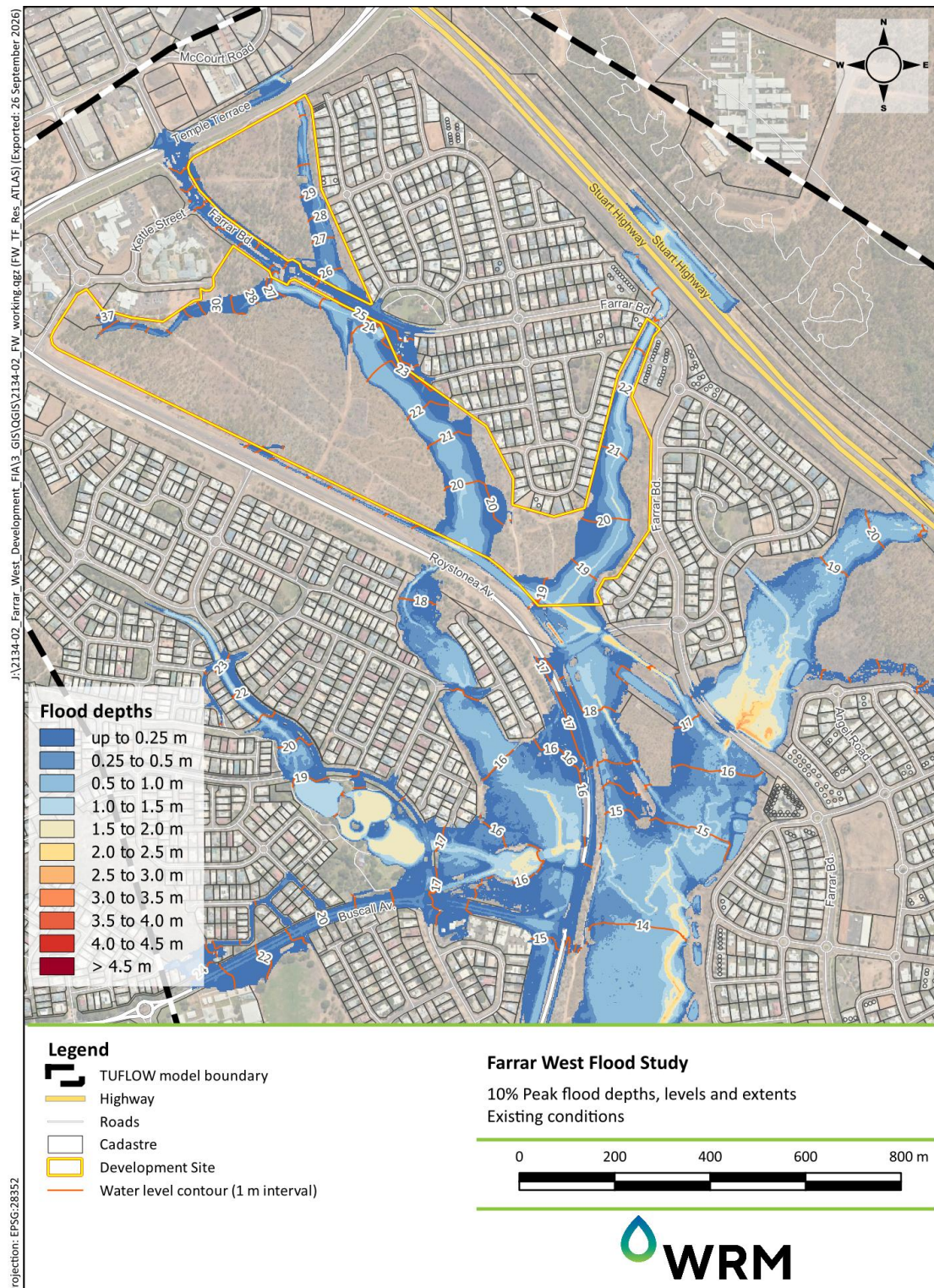


Figure E.3 Existing conditions 10% AEP flood depths, levels and extents map

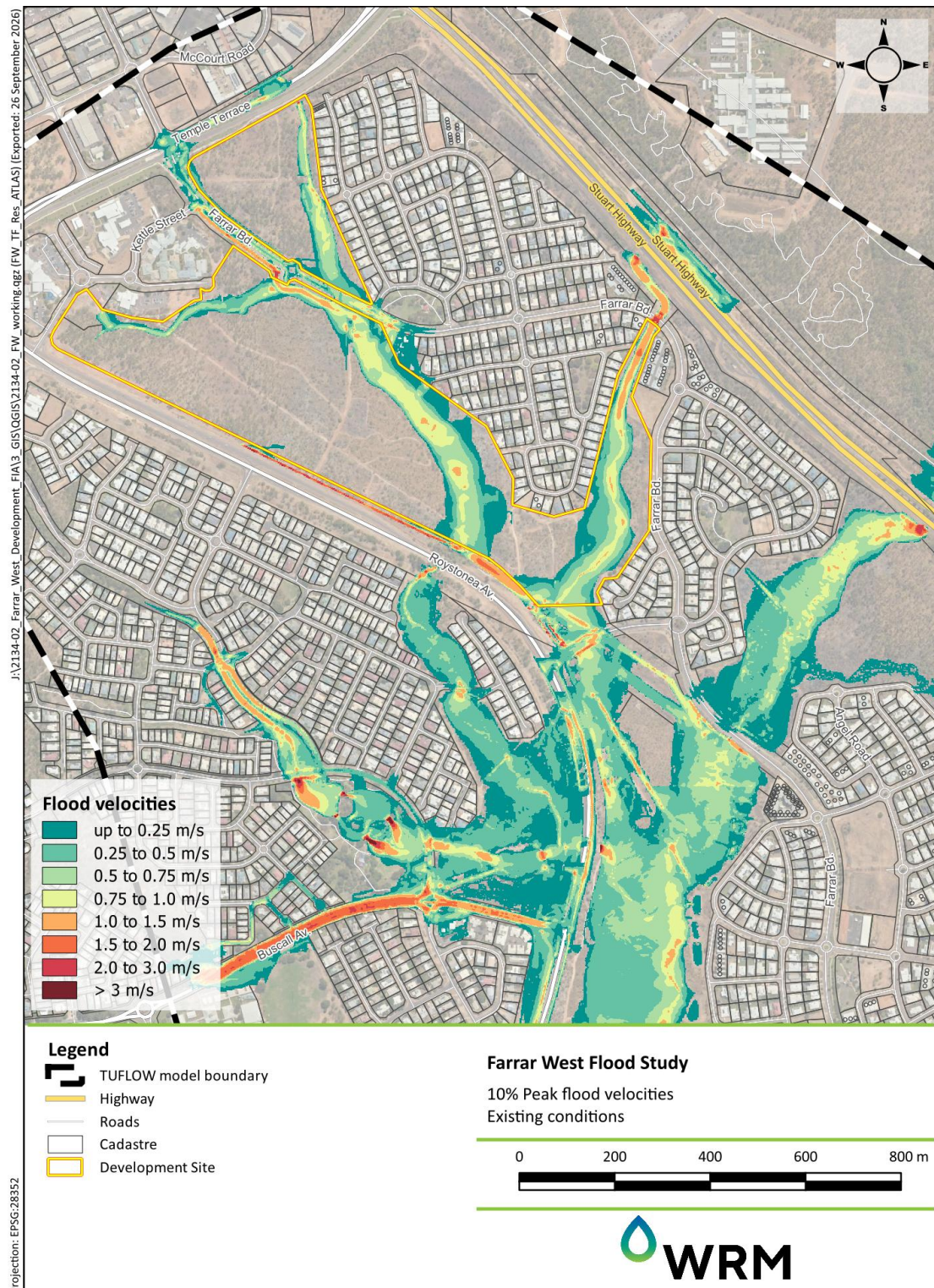


Figure E.4 Existing conditions 10% AEP flood velocities map

E.2 DEVELOPED CONDITIONS FLOOD MAPS

DRAFT

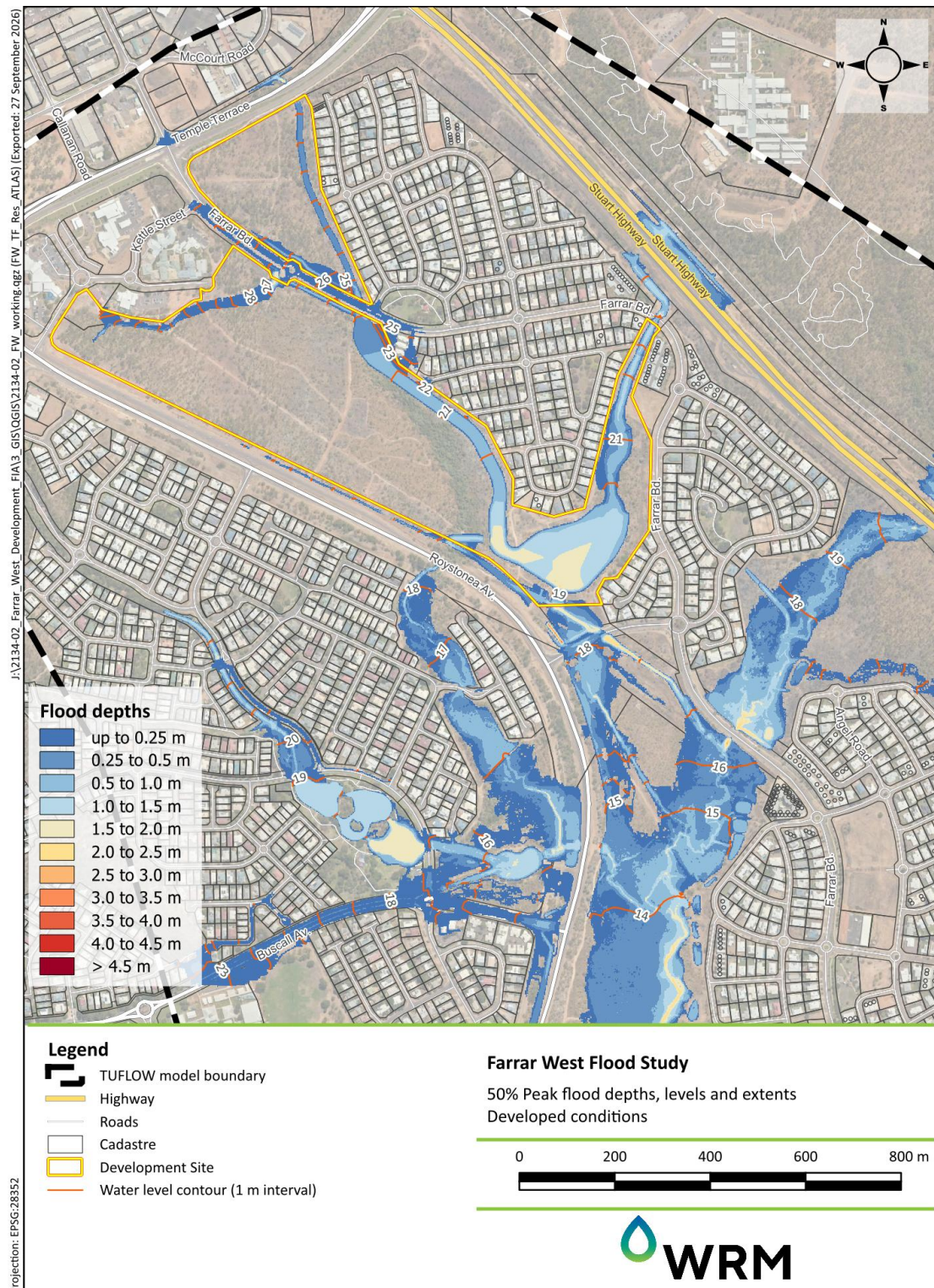


Figure E.5 Developed conditions 50% AEP flood depths, levels and extents map

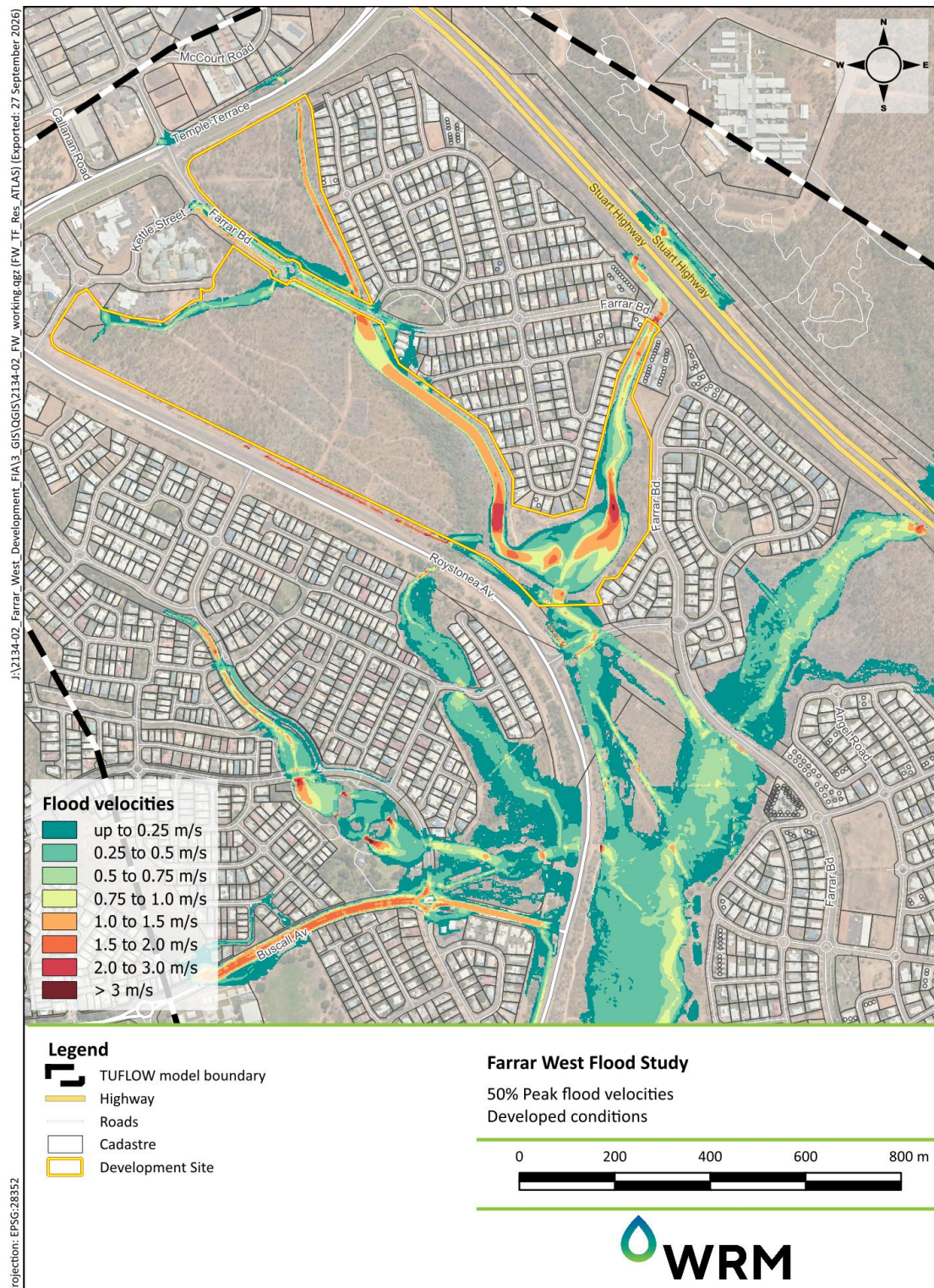


Figure E.6 Developed conditions 50% AEP flood velocities map

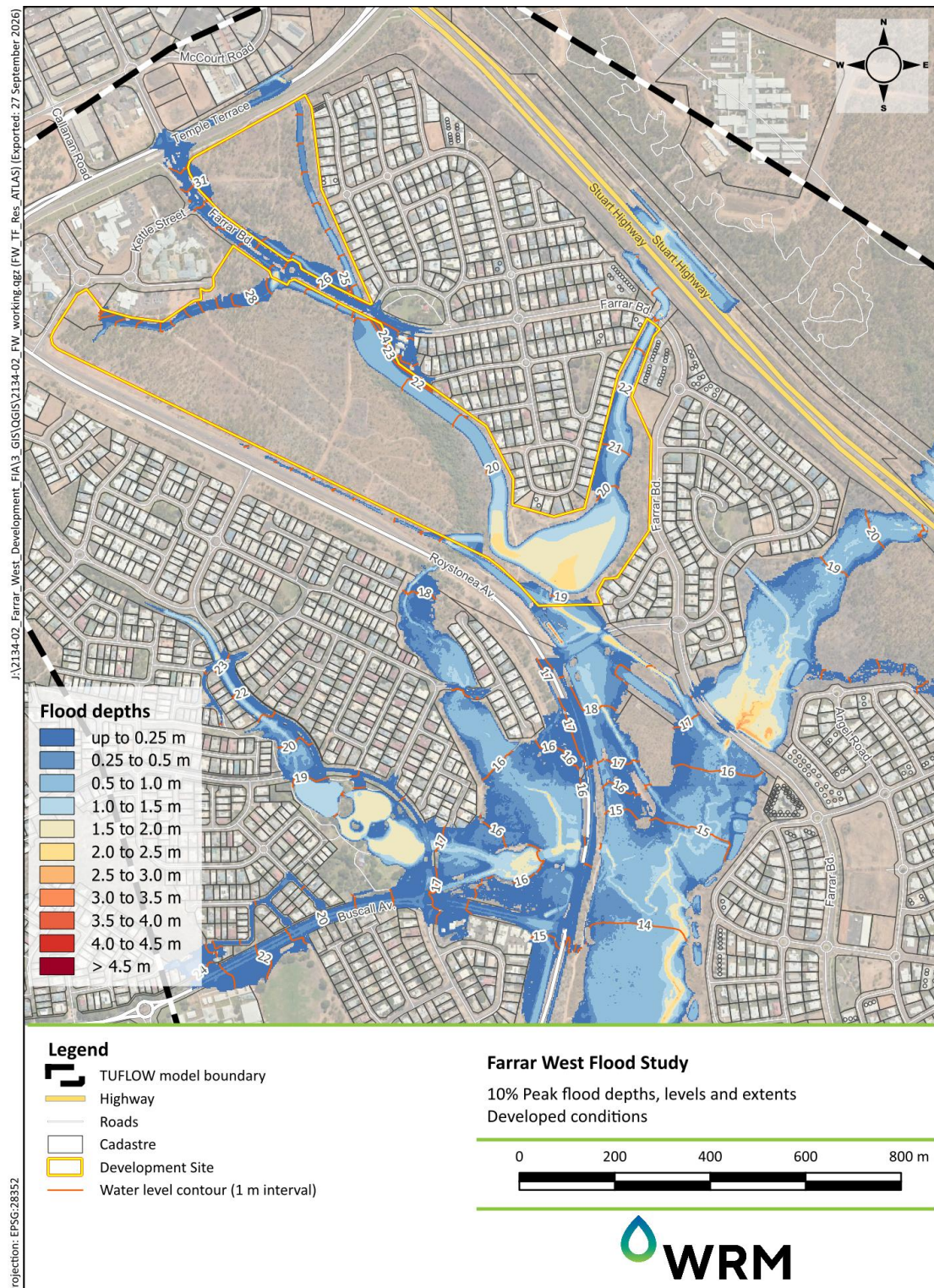


Figure E.7 Developed conditions 10% AEP flood depths, levels and extents map

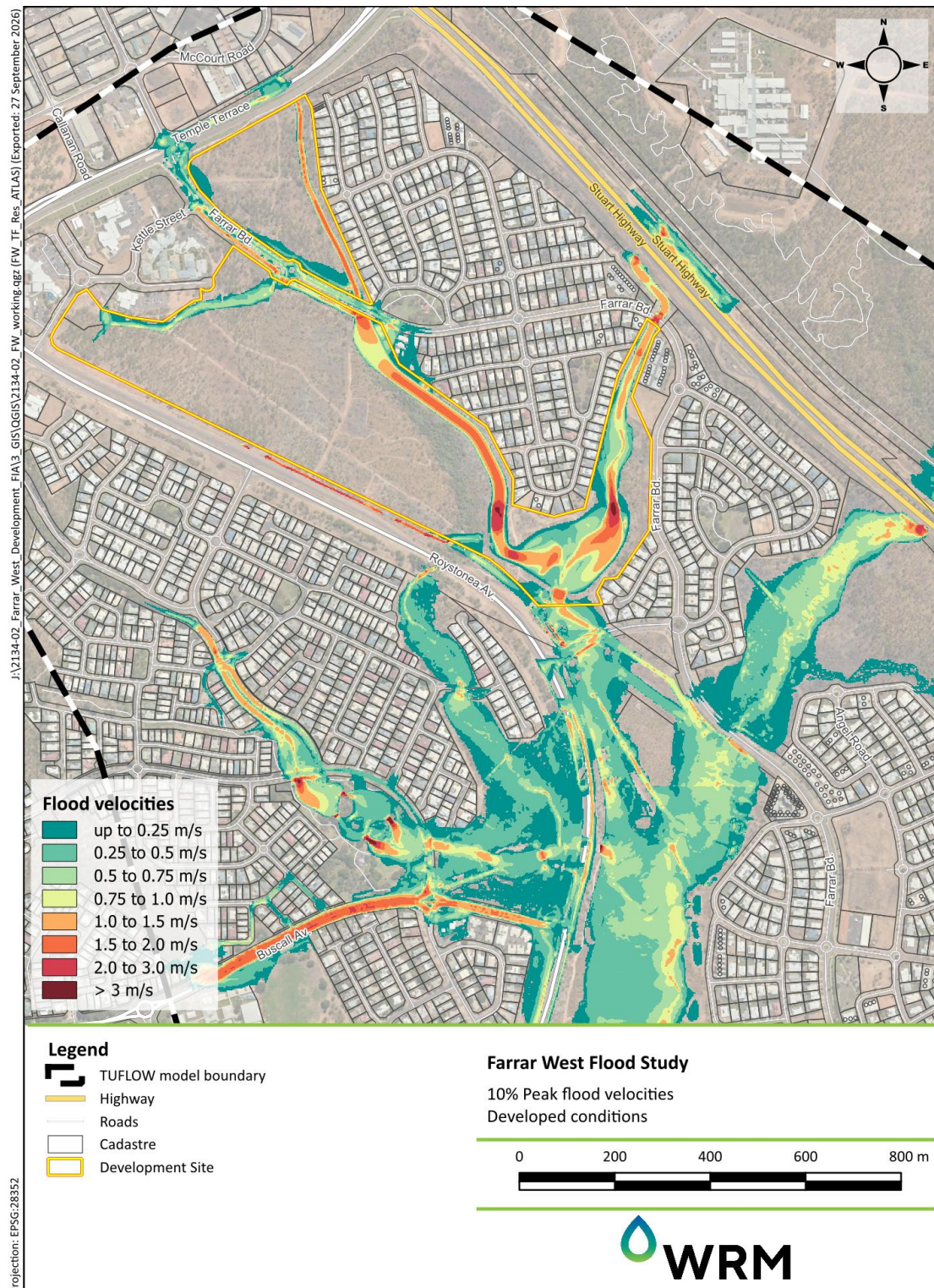


Figure E.8 Developed conditions 10% AEP flood velocities map

E.3 IMPACT MAPS

DRAFT

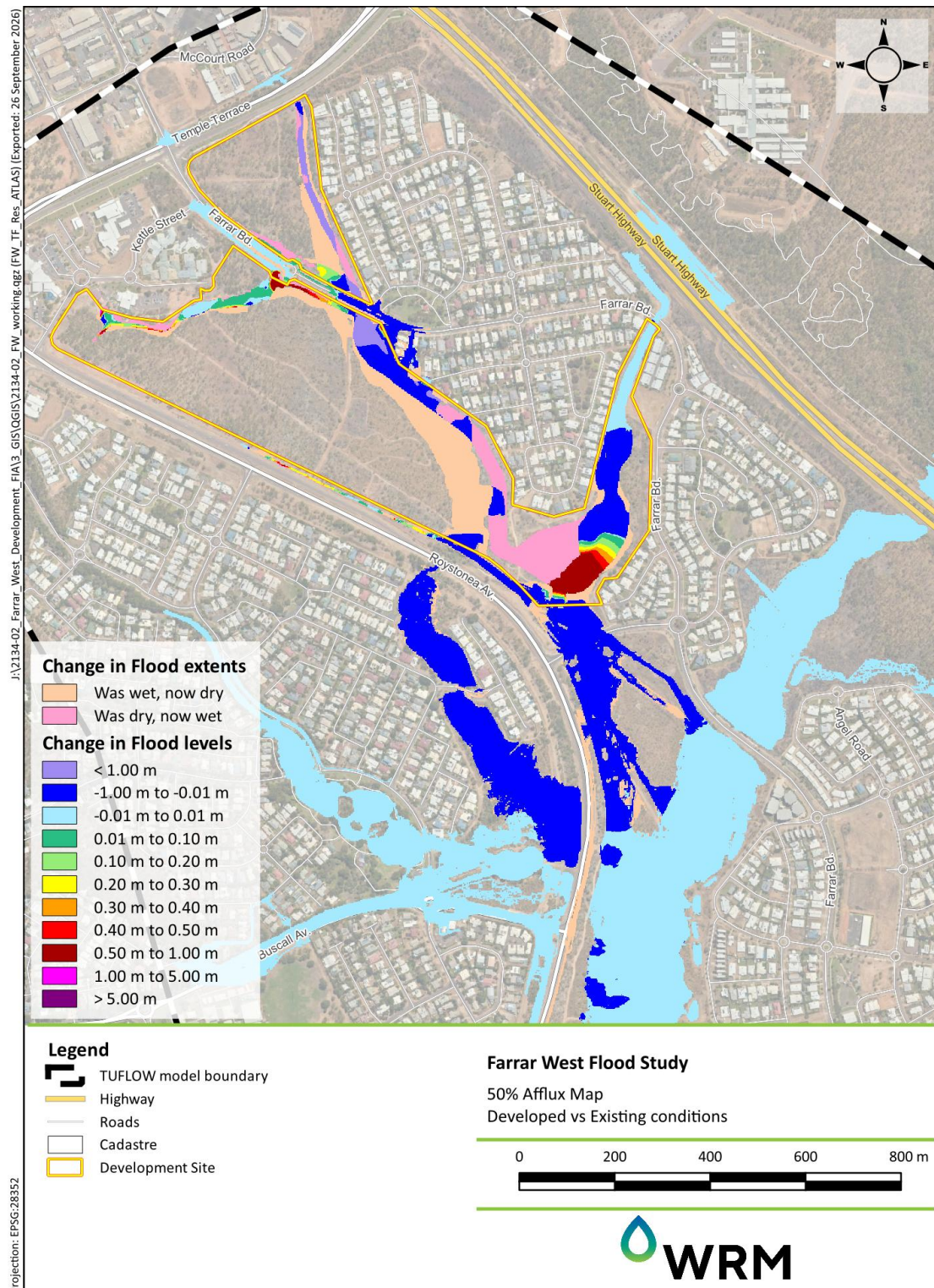


Figure E.9 Change in flood levels map for 50% AEP event

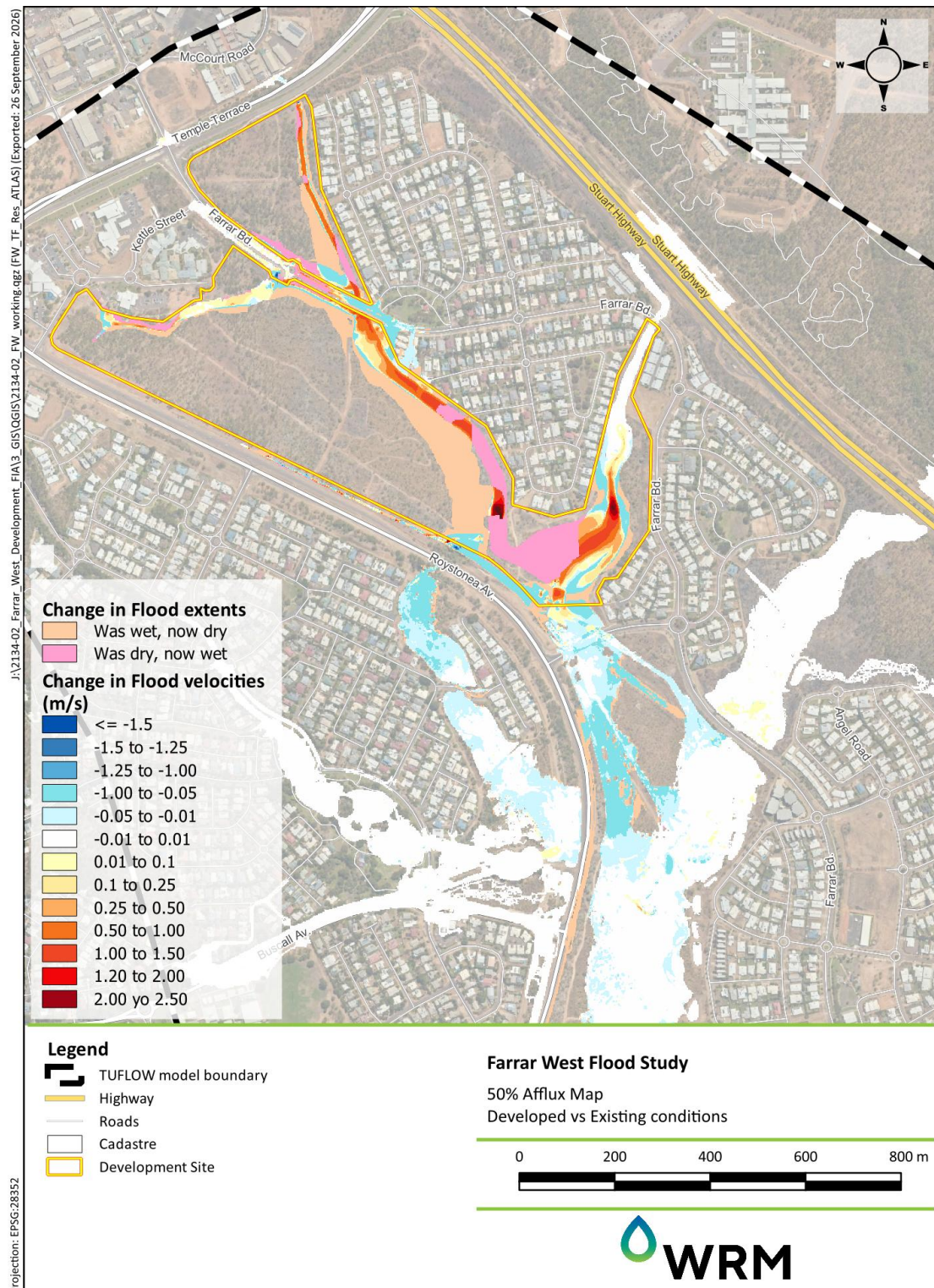


Figure E.10 Change in flood velocities map for 50% AEP event

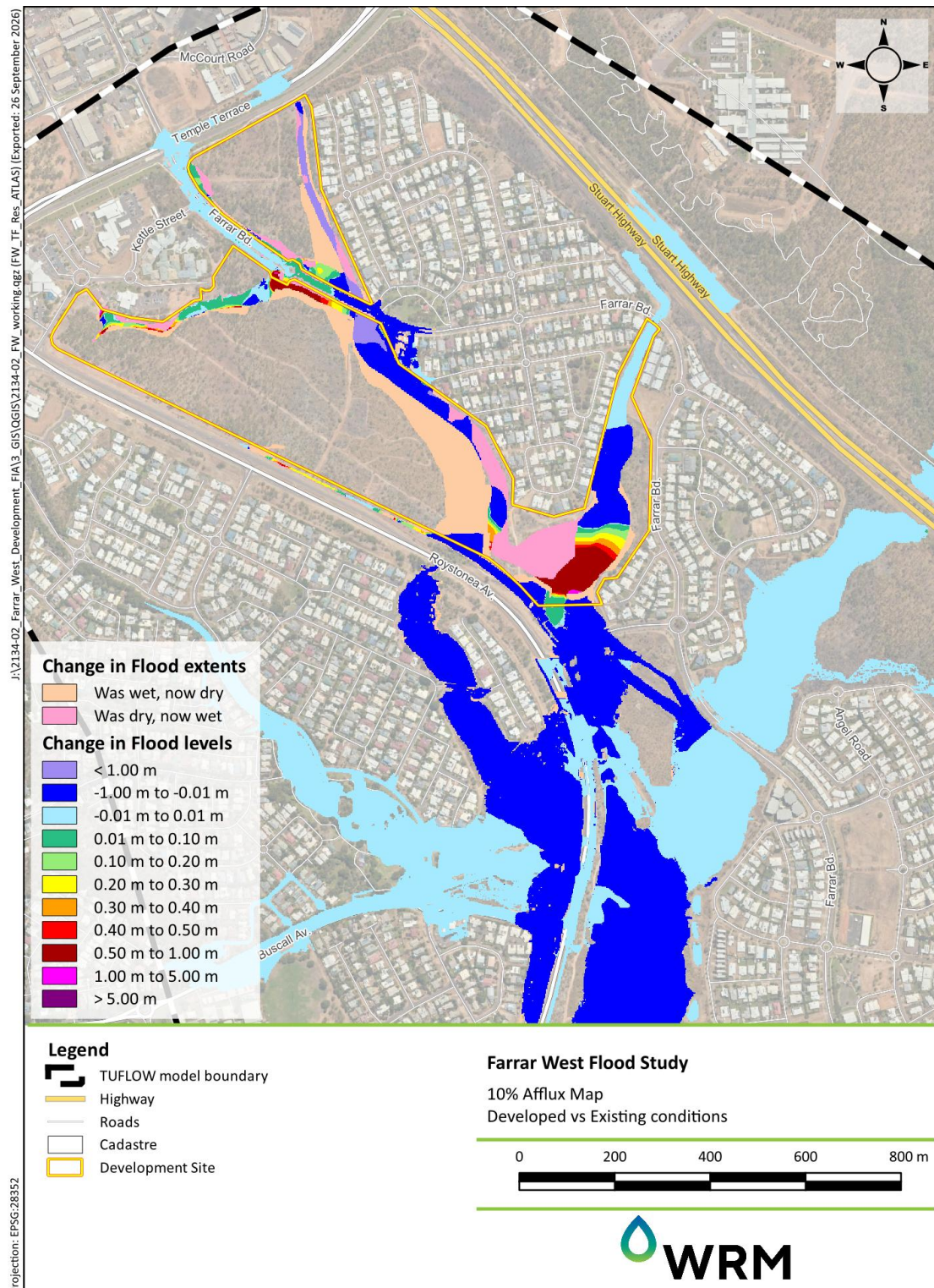


Figure E.11 Change in flood levels map for 10% AEP event

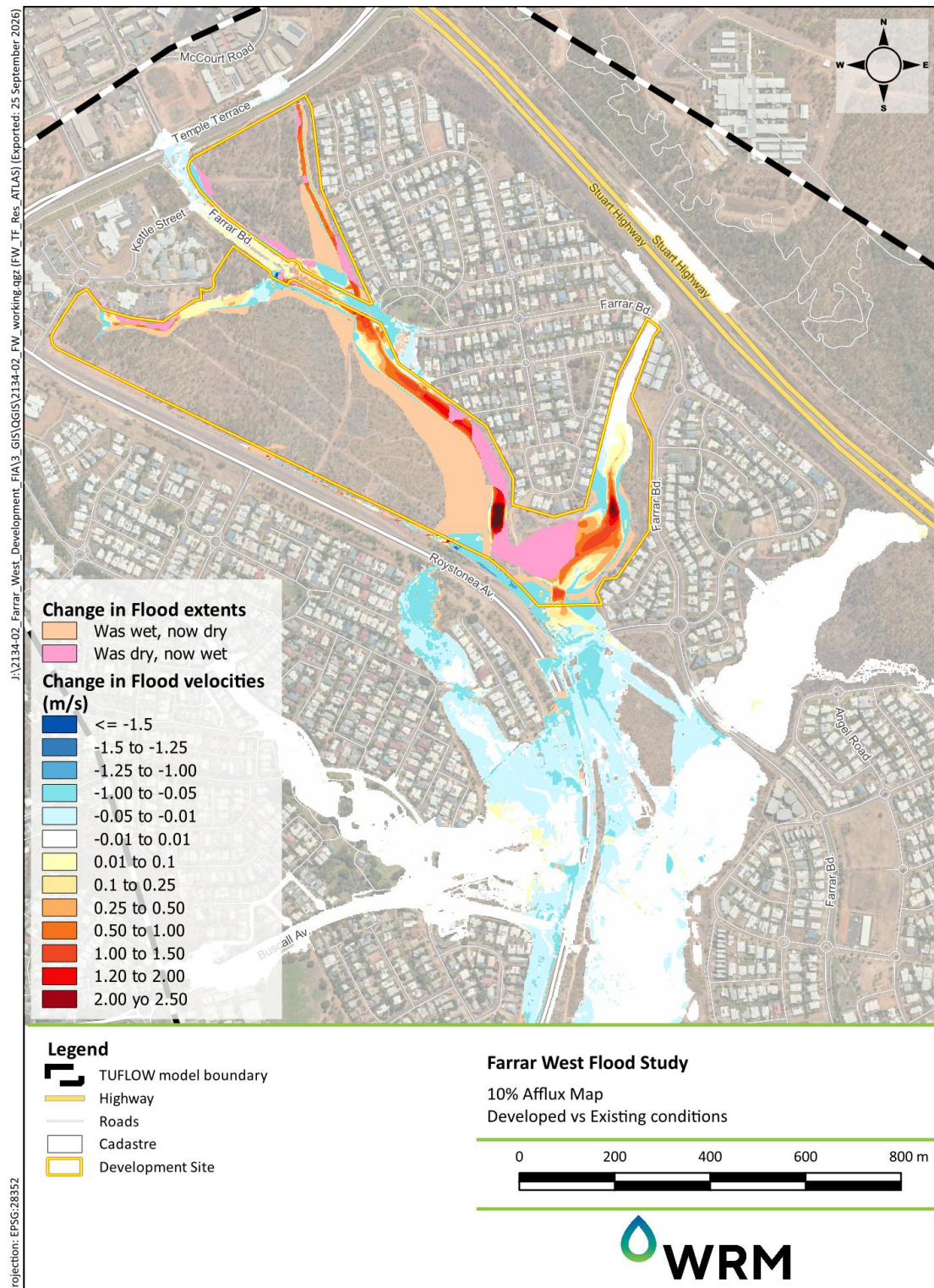


Figure E.12 Change in flood velocities map for 10% AEP event



Level 1, 369 Ann Street Brisbane
PO Box 10703 Brisbane Adelaide Street Qld 4000
07 3225 0200

Level 5, 93 Mitchell Street Darwin
GPO Box 43348 Casuarina NT 0811
08 8911 0060

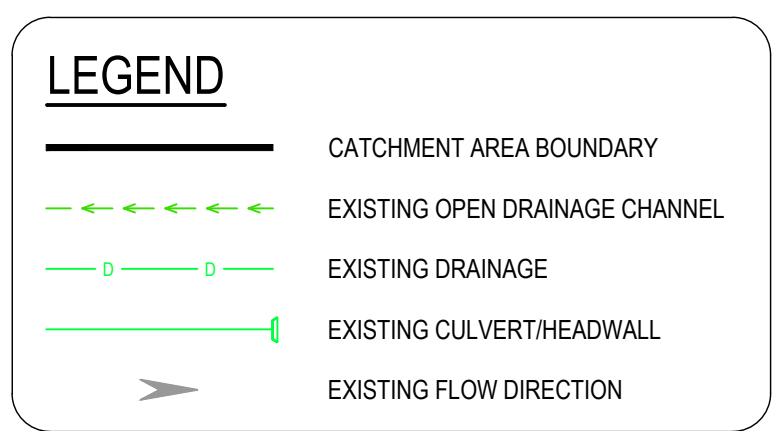
wrm@wrmwater.com.au
wrmwater.com.au

ABN 96 107 404 544

Appendix Two:

Existing Catchment Plan

1. DRAWING TO BE READ IN CONJUNCTION WITH STORMWATER MANAGEMENT PLAN - PART 1
2. CATCHMENTS ESTIMATED FROM LIDAR INFORMATION AND PREVIOUS REPORTING PROVIDED BY DLI.

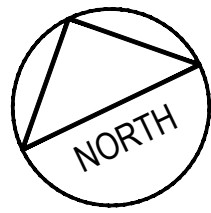


DRAWING STATUS				CLIENT & JOB	
DEVELOPMENT APPLICATION				LARRAKIA DEVELOPMENT CORPORATION FARRAR WEST DEVELOPMENT	
DATE		GRD	DATUM	TITLE	
SEPTEMBER 2026		MG494-Z 52	AHD	STORMWATER CATCHMENT PLAN PRE-DEVELOPMENT	
DESIGNED	DRAWN	CHECKED	APPROVED		
PO		JC	FO	 / ...	
W.A.P.C.		SCALE U.N.D.		SHEET SIZE	ORIG. NO.
-		1:2500 @		A1	17182-DA-C3-DC-01
					REVISION
					A

Appendix Three: Proposed Development and Catchments

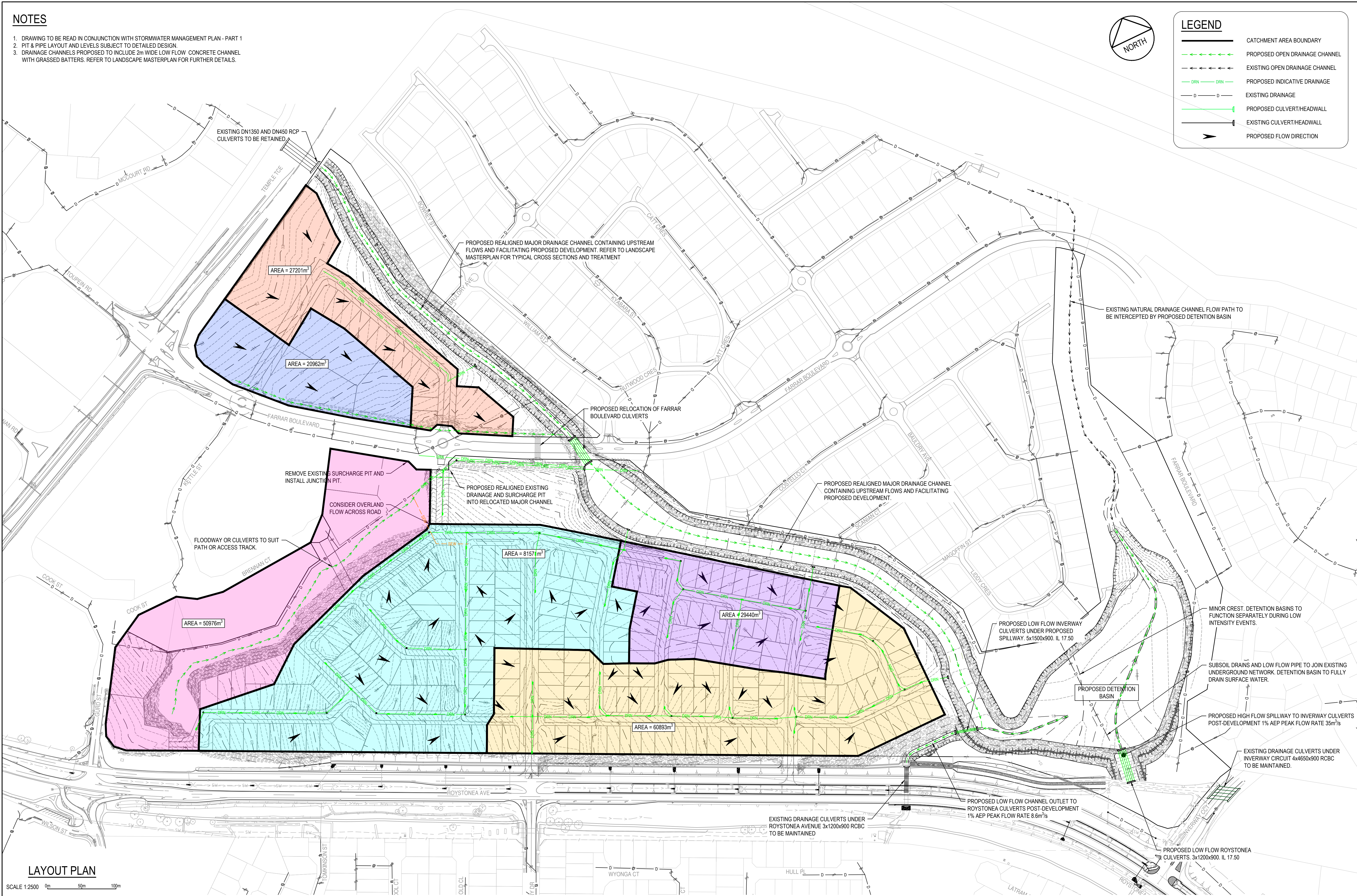
NOTES

1. DRAWING TO BE READ IN CONJUNCTION WITH STORMWATER MANAGEMENT PLAN - PART 1
2. PIT & PIPE LAYOUT AND LEVELS SUBJECT TO DETAILED DESIGN.
3. DRAINAGE CHANNELS PROPOSED TO INCLUDE 2m WIDE LOW FLOW CONCRETE CHANNEL WITH GRASSED BATTERS. REFER TO LANDSCAPE MASTERPLAN FOR FURTHER DETAILS.



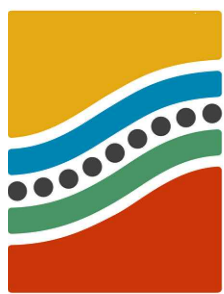
LEGEND

- CATCHMENT AREA BOUNDARY
- PROPOSED OPEN DRAINAGE CHANNEL
- EXISTING OPEN DRAINAGE CHANNEL
- PROPOSED INDICATIVE DRAINAGE
- EXISTING DRAINAGE
- PROPOSED CULVERT/HEADWALL
- EXISTING CULVERT/HEADWALL
- PROPOSED FLOW DIRECTION



LAYOUT PLAN

SCALE 1:2500 0m 50m 100m



Larrakia
Development
Corporation



DEVELOPMENT APPLICATION

DATE	SEPTEMBER 2026	GRD	MG94-Z-52	DATUM	AHD
DESIGNED	PO	DRAWN	JC	CHECKED	FO
W.A.P.C.	-	SCALE	1:2500	@	A1

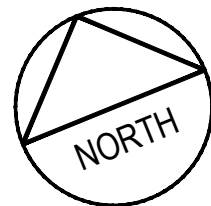
LARRAKIA DEVELOPMENT CORPORATION
FARRAR WEST DEVELOPMENT

STORMWATER CATCHMENT PLAN
POST-DEVELOPMENT

17182-DA-C3-DC-02

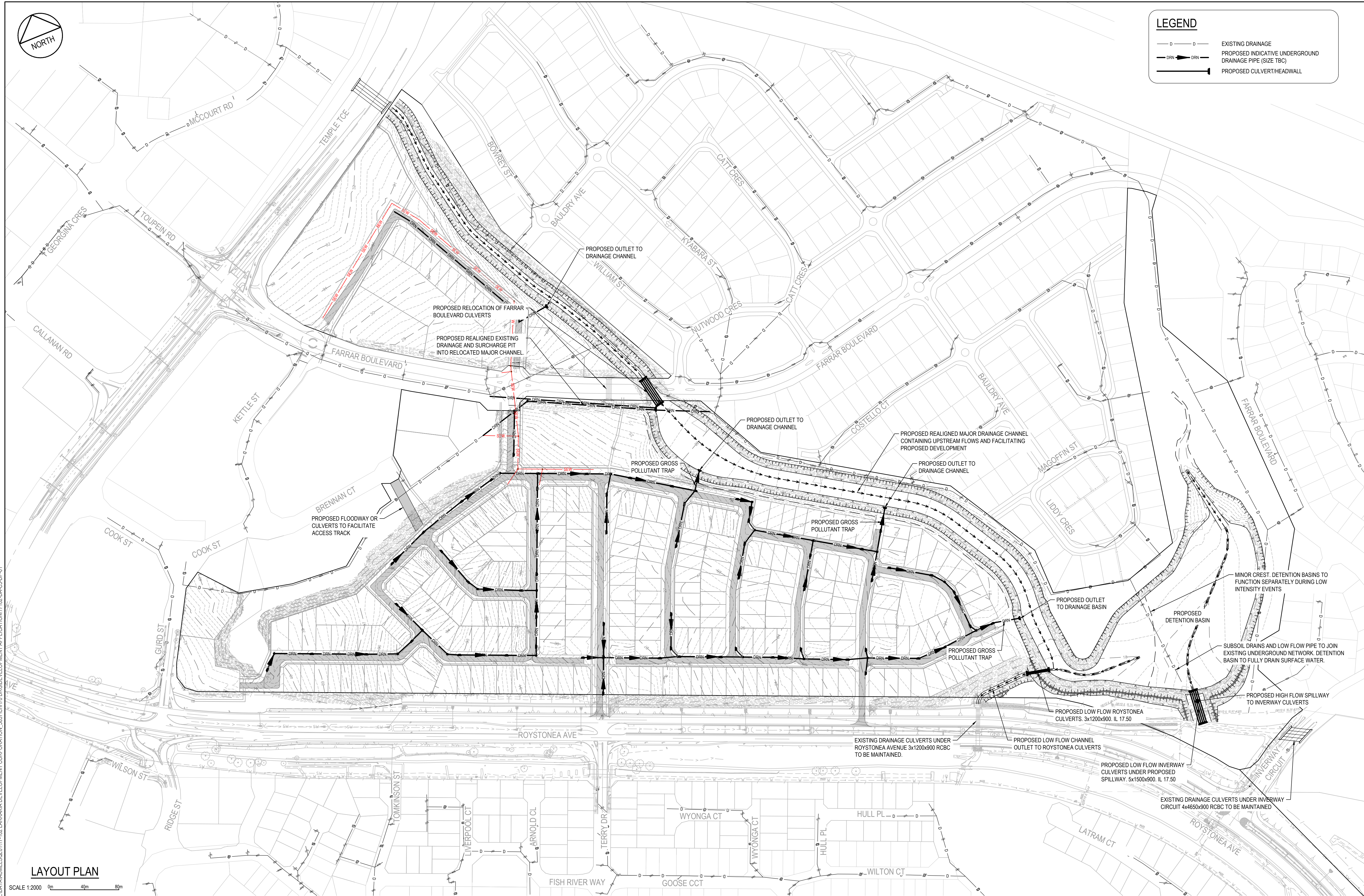
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Appendix Four: Pit and Pipe Network Layout



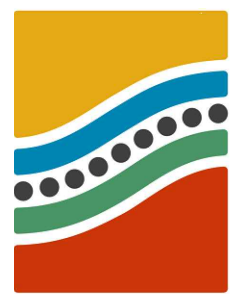
LEGEND

- EXISTING DRAINAGE
- PROPOSED INDICATIVE UNDERGROUND DRAINAGE PIPE (SIZE TBC)
- PROPOSED CULVERT/HEADWALL



LAYOUT PLAN

SCALE 1:2000 0m 40m 80m



Larrakia Development Corporation



DEVELOPMENT APPLICATION

DATE	SEPTEMBER 2026	GRD	MG94-Z-52	DATUM	AHD
DESIGNED	PO	DRAWN	JC	CHECKED	FO
NT DCA	-	SCALE	1:2000	@	A1

LARRAKIA DEVELOPMENT CORPORATION
FARRAR WEST DEVELOPMENT
STORMWATER DRAINAGE
LAYOUT PLAN

17182-DA-C3-DP-01

A

FILE PATH: C:\1205 WORKSPACE\DATA\17182-DA-C3-DP-01\17182-DA-C3-DP-01.DWG