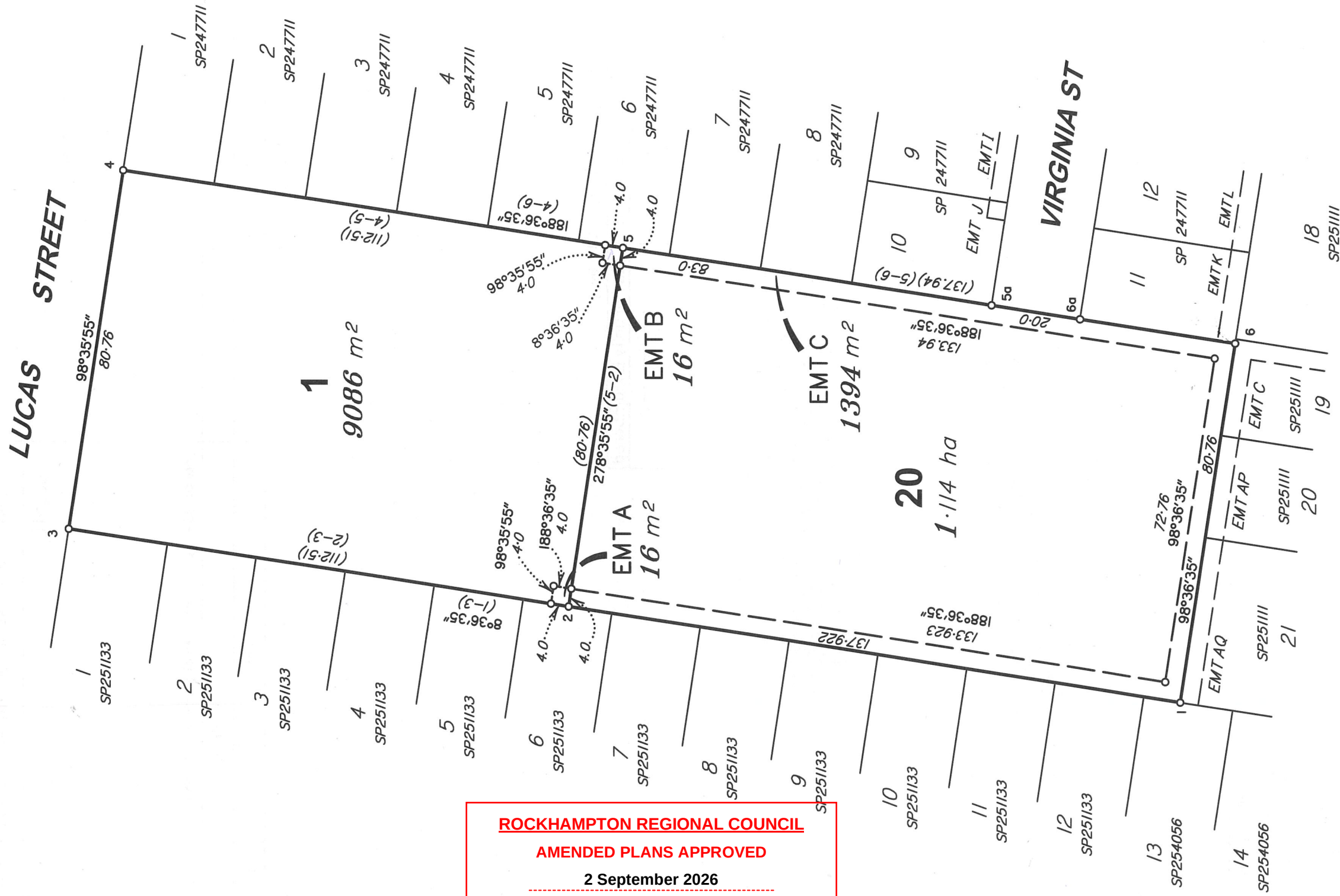




| REFERENCE MARKS |                 |          |            |       |
|-----------------|-----------------|----------|------------|-------|
| STN             | TO              | ORIGIN   | BEARING    | DIST  |
| 1               | OIP             | SP251/33 | 188°36'45" | 0.725 |
| 3               | O. Plug in Kerb | SP251/33 | 356°14'    | 4.25  |

Sunrise Surveying Pty Ltd (ACN 162 352 909) hereby certify that the land comprised in this plan was surveyed by the corporation, by Max William KIRBY, surveying graduate, for whose work the corporation accepts responsibility, under the supervision of Leigh Robert James TINGLE, cadastral surveyor and that the plan is accurate, that the said survey was performed in accordance with the Survey and Mapping Infrastructure Act 2003 and Surveyors Act 2003 and associated Regulations and Standards and that the said survey was completed on ...../...../2026.

Cadastral Surveyor

Date.

***Plan of Lots 1 & 20***  
***and EMTs A & B in Lot 1 and EMT C in Lot 20***

*Cancelling Lot 1 on RP615378*

**Stage 1** AS amended by council 11/8/2026

ROCKHAMPTON LOCALITY: GRACEMERE  
NT: REGIONAL COUNCIL

*Cancelling Lot 1 on RP615378*

# Stage 1

LOCALITY: GRACEMERE

Meridian: SP247711

Survey  
Records:

SP334503

State copyright reserved.

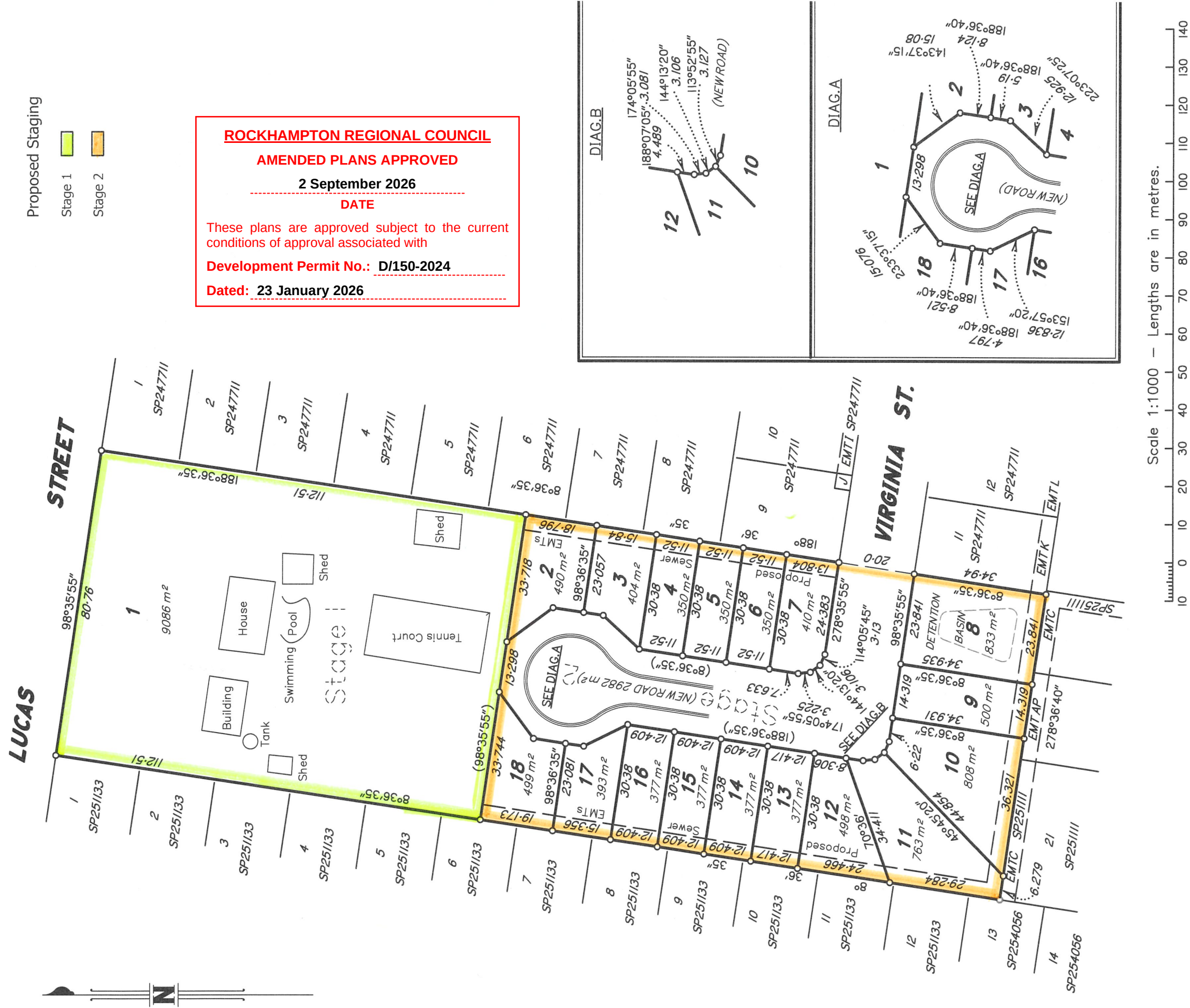
**1:1000**

## STANDARD

Scale:

Format:

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Proposed Staging

- Stage 1
- Stage 2

ROCKHAMPTON REGIONAL COUNCIL

AMENDED PLANS APPROVED

2 September 2026

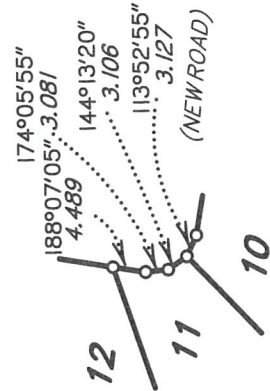
DATE

These plans are approved subject to the current conditions of approval associated with

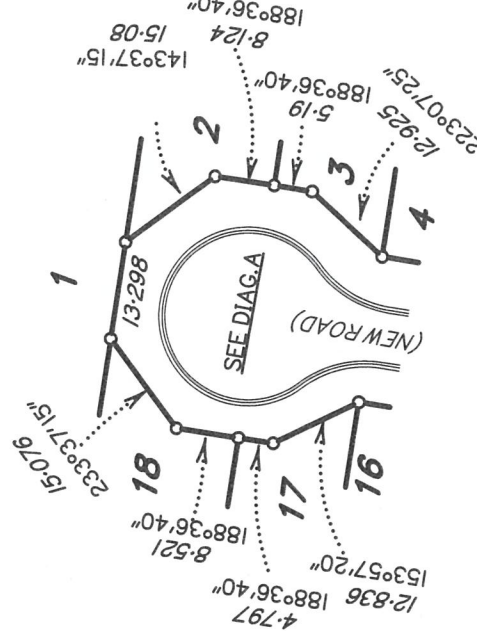
Development Permit No.: D/150-2024

Dated: 23 January 2026

DIAG.B



DIAG.A



Scale 1:1000 – Lengths are in metres.

## Staging Plan of Proposed Lots 1 to 18 cancelling Lot 1 on RP615378 Stage 2 As amended by council 11/8/2026

This plan has been prepared for a reconfiguration of a Lot Application to Rockhampton Regional Council and should not be used for any other purposes, especially for any financial dealings. Areas and dimensions are approximate only and subject to final Survey.

GSPC

(Gracemere Surveying and Planning Consultants Pty Ltd)  
ABN: 40 124 780 445  
PO Box 379 Gracemere QLD 4702  
Rockhampton & Toowoomba  
PH: (07) 4922 7033 email: admin@gspc.com.au FAX: (07) 4922 7044

|                  |             |
|------------------|-------------|
| LOCALITY         | Gracemere   |
| PARISH           | ---         |
| LOCAL GOVERNMENT | Rockhampton |
| HORIZONTAL DATUM | MGA         |
| MERIDIAN         | SP247711    |
| VERTICAL DATUM   | ---         |
| DERIVED vide     | ---         |
| MAP REF          | 8951-22242  |

|               |          |
|---------------|----------|
| PLAN SCALE    | 1:1000   |
| AUTOCAD SCALE | 1:1000   |
| DATE          | 3-2-2026 |
| DRAWN         | RUPEH    |

|         |           |
|---------|-----------|
| SHEET 1 | OF 1      |
| REF.    | 241349-10 |

2025



**ROCKHAMPTON REGIONAL COUNCIL**

**APPROVED PLANS**

These plans are approved subject to the current conditions of approval associated with

**Development Permit No.:** D/150-2024

**Dated:** 23 January 2026

**PROPOSED RESIDENTIAL SUBDIVISION  
82 LUCAS STREET, GRACEMERE**

## **STORMWATER MANAGEMENT PLAN**

**FOR J STAGG**

D24.363-RP01

**J STAGG**

# STORMWATER MANAGEMENT PLAN

## PROPOSED RESIDENTIAL SUBDIVISION 82 LUCAS STREET, GRACEMERE

### 1. Document History & Status

| ISSUE | DATE       | ISSUED TO | DESCRIPTION                 | BY | APPROVED |
|-------|------------|-----------|-----------------------------|----|----------|
| A     | 22/01/2025 | S Govind  | For response to council RFI | AL | TL       |
| B     | 14/03/2025 | S Govind  | For response to council RFI | AL | TL       |
| C     | 01/08/2025 | S Govind  | For response to council RFI | AL | TL       |
| D     | 27.11.2025 | S Govind  | Lot yield altered           | AL | TL       |

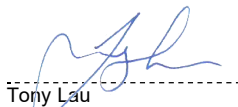
Prepared By



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Australia

Reviewed By



Tony Lau  
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RPEQ 19272

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Email: reception@dieligh.com.au  
Date: 21/11/2025  
Reference: D24.363-RP01

## 2. Introduction

This report was prepared for J Stagg in support of a proposed development to the subject site at 82 Lucas Street, Gracemere. This report should be read in conjunction with the overall application relating to this project. The proponent is seeking approval to develop the site with a proposed 18-lot residential subdivision (17 new lots plus 1 lot for an existing dwelling).

The land subject to this application is described as 1 RP615378 which has a total area of 20,230m<sup>2</sup>.

## 3. Analysis Methodology

Hydrological modelling has been undertaken utilising an Initial Loss – Continuing Loss (IL-CL) model in Watercom DRAINS, as per Australian Rainfall and Runoff (ARR) guidelines for both pre-development and post-development catchments to size the volume of detention required to ensure post-development peak discharges for all Annual Exceedance Probabilities (AEPs) did not exceed pre-development peak discharges up to and including a 1% AEP flood event.

Rainfall data and temporal patterns were obtained from Bureau of Meteorology (BOM) Design Rainfall Data System (2016) and ARR Data Hub respectively.

The following losses, obtained from the ARR Data Hub, were adopted:

- Storm Initial Losses = 34 mm
- Storm Continuing Losses = 1.8 mm/h

The 'lite' hydraulic model was utilized for this report. This model analyses overflow routes (overland flow paths, such as kerbs and swales) using simple normal depth calculations. This means that storage effects in overflow routes are ignored in the Lite hydraulic model and therefore peak flows, depths and velocities in overflow routes may be overestimated. It was considered appropriate to adopt this model in this case due to the more conservative overland flow outcomes compared to the Full Unsteady hydraulic model.

## 4. Existing Stormwater Conditions

### 4.1 Site Runoff

The subject site is currently developed with a residential dwelling, a secondary building, three sheds, sealed access, swimming pool and a tennis court. These structures are all located in proximity to the Lucas Street frontage. The rear of the site is undeveloped, with moderate grass cover and scattered trees observed.

Based on aerial imagery and information provided by Gracemere Surveying and Planning Consultants (GSPC), fraction impervious (Fi) of the existing site is approximately 0.068. For the pre-development catchment, this corresponds to the following area breakdown:

- 1.8% Effective Impervious Area (EIA)
- 5% Remaining Impervious Area (RIA)
- 93.2% Pervious Area (PA)

The site generally falls to the south with an average slope of 4%.

Runoff from the site is captured by a downstream stormwater channel, located in easements AQSP251111, APSP251111 and CSP251111. Based on aerial imagery and a site visit, the channel has the following approximated features:

- 1.5m wide concrete base with a smooth spoon drain formation
- 0.4m high retaining wall on downstream side
- 1.5m grassed batter on upstream side

Grated field inlets were observed at various points along the channel. Rockhampton Regional Council (RRC) confirmed that the outlet pipe from the downstream pit has a diameter of 150mm.

Based on the average flowpath slope and existing surfacing conditions, an overall time of concentration (Tc) of 23 minutes has been adopted in accordance with QUDM Figure 4.4 with a C<sub>10</sub> value of 0.790 in accordance with QUDM Table 4.5.4.

| Friends Equation (Eq 4.5) - Shallow overland sheet flow |                 |           |   |         |
|---------------------------------------------------------|-----------------|-----------|---|---------|
| L                                                       | Surface         | n         | S | Tc      |
| m                                                       |                 | Manning's | % | minutes |
| 250                                                     | Average Grassed | 0.045     | 4 | 23      |

The following table summarizes the run results for a range of events for a time of concentration of 23 minutes.

| PRE-DEVELOPMENT CASE |       |                |                |                                                                                               |            |
|----------------------|-------|----------------|----------------|-----------------------------------------------------------------------------------------------|------------|
| Event AEP            | Q     | Critical Storm | Burst Duration | Fi                                                                                            | 0.0        |
| %                    | m3/s  |                |                | Tc                                                                                            | 23 minutes |
| 50                   | 0.168 | Storm 6        | 2 hours        | IL-CL model in Watercom DRAINS<br>with lite hydraulic analysis in<br>accordance with ARR 2019 |            |
| 20                   | 0.296 | Storm 4        | 1 hour         |                                                                                               |            |
| 10                   | 0.355 | Storm 6        | 45 minutes     |                                                                                               |            |
| 5                    | 0.474 | Storm 7        | 45 minutes     |                                                                                               |            |
| 2                    | 0.587 | Storm 6        | 45 minutes     |                                                                                               |            |
| 1                    | 0.684 | Storm 6        | 45 minutes     |                                                                                               |            |

## 4.2 Downstream Stormwater Infrastructure Capacity

### 4.2.1 Stormwater Network and Roads

Approved Graham Scott & Associates Pty Ltd (GSA) drawings 100017/1-41 for the external "Graceview" subdivision Stages 1A and 1B (Development Permit D/302-2010) were reviewed as part of this report in order to assess road flow and network capacity between the site and the existing detention basin.

The following constraints with the approved design are noted:

- Catchment AE, as per drawing 100017/38, have major and minor coefficients that appear to correspond to a Fi of 0.0. It is not possible to perfectly replicate the coefficients using current design rainfall data due to BOM updates since the drawings were approved.
- Catchment AE was proposed to partially discharge to stormwater line 5 (Marc Crescent) via an inter-allotment roof water line.

- External stormwater lines 5 (in Marc Crescent), 2 (in Virginia Crescent) and 1 (in Benjamin Drive) do not appear to have any additional capacity to cater for additional flows from the subject site, based on longitudinal sections on drawings 100017/18 to 100017/20 and calculation tables on drawings 100017/39 to 100017/41. The hydraulic grade line is close to the design surface, meaning that additional inflows to these lines would result in surcharge from stormwater pits.

Izzard's equation was applied to Virginia Crescent and Benjamin Drive, based on the Road 2 & 3 typical cross sections and approximated longitudinal grades denoted on GSA's design drawings. It is noted that overland/road capacities were not included in the stormwater design tables on GSA's drawings, and thus road flows cannot be determined without replicating the network model.

Izzard's equation indicated overland capacities of approximately 0.423m<sup>3</sup>/s and 0.656m<sup>3</sup>/s for Virginia Crescent and Benjamin Drive respectively. In general, the gap flow between total flow and pipe flow denoted in the stormwater design tables in GSA's plans indicate the road formation is at capacity.

There appears to be no additional capacity in the existing underground network or overland flow paths (kerb and channel) to receive additional flows.

It will be necessary to provide additional or upgraded infrastructure to cater for the proposed development. Based on preliminary modelling, this would require:

- On-site detention within the proposed development area, or;
- An additional 375mm diameter pipe, installed in the footpath behind the existing stormwater network, from the development site to the stormwater reserve in easements AQSP251111, APSP251111 and CSP251111. This would involve installation of approximately 380m of pipe and requires a minimum of two road crossings.
- Upgrade of the existing pipe network, from the development site to the existing outlet. New pipe sizes required to properly cater for the proposed development cannot be properly determined without additional stormwater network modelling. Sufficient information to do so was not available at the time of this report.

The proponent's preferred option at this stage is on-site detention.

#### 4.2.2 Interallotment Drainage Features

GSA Drawings 100348/217 and 100348/222 for the external "Graceview" subdivision Stage 4 (Development Permit D/92-2011) were reviewed as part of this report in order to assess impacts on allotments located immediately downstream.

It is noted that the interallotment pipes have the following peak capacities, as per QUDM Chart A1-1 and validated by review of GDA design longitudinal sections:

- 1/RW6 – 2/RW6 (from Lot 21 SP251111 to Lot 20 SP251111) – 0.09m<sup>3</sup>/s
- 2/RW6 – 3/RW6 (from Lot 20 SP251111 to Lot 19 SP251111) – 0.16m<sup>3</sup>/s
- 3/RW6 – 2/5 (from Lot 19 SP251111 to Marc Crescent) – 0.26m<sup>3</sup>/s

The capacity of Swale 1, which is constructed over the top of the interallotment drainage line, comprises of a concrete base, short retaining wall and rock-lined batter. The capacity of Swale 1 was reviewed utilising Autodesk Hydraflow Express. For full-depth flows (0.5m deep) and a

minimum longitudinal grade as noted on approved GSA plans, the peak capacity of Swale 1 is 1.482m<sup>3</sup>/s. The analysis report has been provided in **Appendix A**.

## 5. Post Developed Site Flows and Management

### 5.1 Post Developed Runoff Conditions

The proposed development will increase the impervious area of the site and thus increase the volume of stormwater runoff. Based on Capricorn Municipal Development Guidelines (CMDG) Table D05.06.1, a  $F_i$  for the development area of 0.80 was adopted based on catchments being inclusive of roads, and an average lot size of 450m<sup>2</sup> – 600m<sup>2</sup>. A  $F_i$  of 0.15 for Lot 1 was adopted based on the existing impervious areas identified in Section 4 for the reduced lot area nominated on the proposal plans. For Lot 1, this corresponds to the following area breakdown:

- 4% EIA
- 11% RIA
- 85% PA

The Lot 1 catchment has been included in inflow catchments for the proposed stormwater network, with inlets and pipes sized accordingly.

For the development area, this corresponds to the following area breakdown:

- 80% EIA
- 0% RIA
- 20% PA

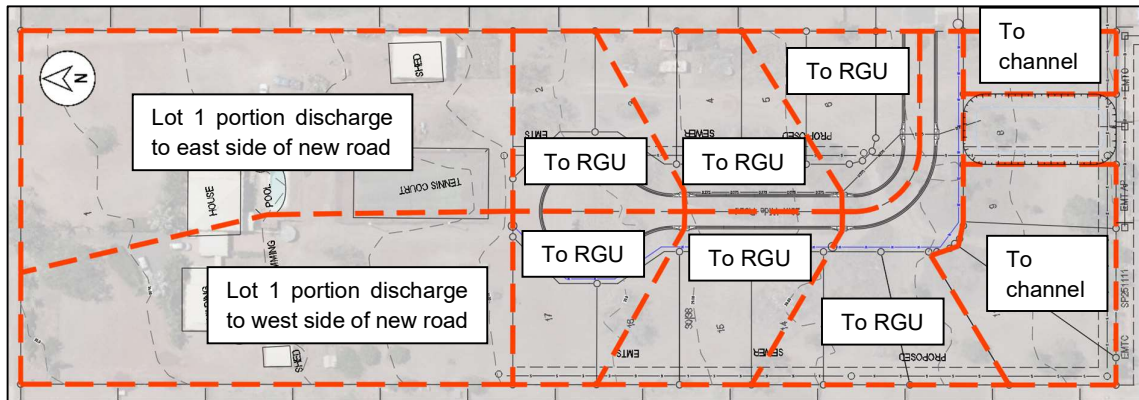
Proposed bulk earthworks is not known at this time. However, based on the existing site topography and current lot layout proposal, the following post-development time of concentration(s) have been estimated:

- 5 minutes for EIA
- 15 for PA, assuming 'typical' residential allotment grading consistent with the surrounding area i.e. generally around 3% maximum fall

Additionally, it is assumed that:

- Lots 2-6 and 12-17 will fall to the proposed road, with runoff to be captured by road gully units (RGUs)
- Lots 7-11 will fall to the south boundary and can be diverted, as required, with a swale formation in the proposed easement
- A sag will be provided in the proposed road grading, nominally at the tangent point of the curve in the road, to ensure all stormwater runoff is kept within the site prior to being released in a controlled manner

The assumed catchments to each RGU or channel are shown below. Final catchments shall be determined at detailed design, based on the bulk earthworks design.



A preliminary stormwater layout has been provided on the drawings attached in **Appendix B**. The preliminary DRAINS model indicates that the pipe diameters, kerb inlet dimensions and quantity of inlets may be sufficient. Further analysis must be undertaken at detailed design to rationalise the stormwater network design.

Due to insufficient capacity in the downstream stormwater network, it is considered necessary to provide detention within the proposed development to ensure no worsening and no nuisance to downstream allotments.

## 5.2 Quantity Mitigation Strategy

### 5.2.1 On-Site Detention

It is proposed to provide a 1.0m deep detention basin in Lot 8 with a storage volume of approximately 417m<sup>3</sup>, formed with 1 in 4 batter slopes. The proposed diversion swale at the rear of Lots 7 and 9 – 11 will discharge to the basin via suitably sized headwalls and pipes, noted indicatively on the plans attached in **Appendix B**. If deemed practicable at detailed design, it is preferred to discharge the diversion drains to the swales as channelised flow without pipes.

The basin depth and shape shall be validated at detailed design stage when detailed survey of the site and downstream stormwater infrastructure is available. A minimum buffer of 0.5m clear space should be provided between the top of the batters and any adjacent boundary fences to avoid conflicts.

It is proposed to provide an outlet with the following configuration:

- 450mm diameter outlet pipe, installed in an approximately 800mm deep 900x900 drop-structure, discharging directly into existing interallotment pit 2/RW6
- 4m wide broad crested weir 850mm above the invert of the basin. Rock protection shall be provided at the weir, which only conveys outflows in 5% AEP storm events and greater

Provision of the basin provides a reduction in site runoff in all events, excluding the peak design rainfall (1% AEP or 1 in 100 year ARI.) However, it is noted that both the proportional increase (being 3% more than existing) and the magnitude of the increase (an additional 22 L/s) are minor, so it is expected that the increase will not result in a nuisance to downstream properties.

Untreated site runoff, based on the post-developed site conditions including all stormwater infrastructure such as kerb and channel, pipes and a diversion channel but excluding the detention basin has been provided to demonstrate the quantitative increase in runoff as a result of the proposed development without provision of on-site detention.

| Event AEP % | Pre-Development Flow m <sup>3</sup> /s | Post-Development Flow Untreated m <sup>3</sup> /s | Change % | Post-Development Flow with Basin m <sup>3</sup> /s | Change % |
|-------------|----------------------------------------|---------------------------------------------------|----------|----------------------------------------------------|----------|
| 50          | 0.168                                  | 0.320                                             | 90%      | 0.160                                              | -5%      |
| 20          | 0.296                                  | 0.432                                             | 46%      | 0.257                                              | -13%     |
| 10          | 0.355                                  | 0.491                                             | 38%      | 0.297                                              | -16%     |
| 5           | 0.474                                  | 0.612                                             | 29%      | 0.364                                              | -23%     |
| 2           | 0.587                                  | 0.806                                             | 37%      | 0.518                                              | -12%     |
| 1           | 0.684                                  | 0.965                                             | 41%      | 0.707                                              | +3%      |

Example hydrographs for the detention basin have been provided in **Appendix C**. A copy of the preliminary Watercom DRAINS model is attached to this report. A summary table for the basin outlet hydraulics for each storm frequency, extracted from the DRAINS model, is provided in **Appendix D**.

### 5.2.2 No Nuisance Flooding to Downstream Properties

It is important to consider the impact of the altered stormwater flows from the post-developed site. While total flow volumes remain similar to pre-development conditions, they shall be concentrated in nature rather than as distributed flows across the south boundary. Additionally, concentrated flows shall be released perpendicular to the existing interallotment channel, which introduces a risk of flows overtopping the Swale 1 retaining wall and impacting the downstream residence.

In order to ensure no risk of nuisance or flooding to the existing property adjacent to the basin outlet, being 20 SP251111, the flow components were considered:

- Volume of surcharge from 2/RW6, noting that pipe outflow capacity (375mm diameter interallotment pipe) shall be less than pipe inflow capacity (450mm diameter outlet pipe from the basin) in peak storm events
- Velocity of weir flows in peak storm events
- Depth of flows in Swale 1 when subject to both weir flow and surcharging flows from the pipe network

The preliminary DRAINS model indicates a pipe outflow of 0.362m<sup>3</sup>/s from the basin during a 1% AEP storm. Noting the interallotment pipe capacities noted in section 4.2.2 and that construction of the proposed diversion swale shall result in little to no upstream interallotment channel flow, it is anticipated that 0.202m<sup>3</sup>/s shall surcharge from pit 2/RW6. The preliminary DRAINS model indicates a weir outflow of 0.345m<sup>3</sup>/s from the basin during a 1% AEP storm.

The resulting total flows released to Swale 1 are 0.547m<sup>3</sup>/s. This is well below the maximum capacity of Swale 1 as indicated in section 4.2.2. Therefore, it is expected that acceptable flow depths shall be observed in Swale 1 with no risk of overloading downstream overland stormwater features.

To assess potential risks under severe storm conditions, it was deemed appropriate to consider the full-depth weir outflow conditions rather than the 1% AEP outflows. The maximum weir capacity of 0.774m<sup>3</sup>/s and associated flow velocity of 1.289m/s were estimated with Autodesk Hydraflow Express. An output report is attached in **Appendix E**. Given that the surcharge volume from Pit 2/RW6 during rarer storm events is expected to resemble

conditions associated with a 1% AEP storm, total maximum outflows from the basin to Swale 1 is expected to be in the order of 0.976 m<sup>3</sup>/s. The capacity of Swale 1 is sufficient to cater for these flows.

Given the maximum weir flow velocity of 1.289m/s – which shall be further mitigated by the proposed rock outlet protection before reaching the property boundary – perpendicular surface inflows to Swale 1 shall be gentle and are not expected to ‘jet’ over the existing retaining wall or cause nuisance to the existing residence at 20 SP251111.

### **5.2.3 Erosion Control**

Noting the weir width and outflow rate indicated in sections 5.2.1 and 5.2.2 respectively, unit weir flows shall be 0.193m<sup>2</sup>/s. Based on recommendations in various Catchments and Creeks documents, a nominal rock chute with a length of 2.5m and rock sizing of d<sub>50</sub> = 200mm was adopted. The requirements for rock lining should be reviewed at detailed design to suit the final basin configuration.

### **5.2.4 Future Considerations**

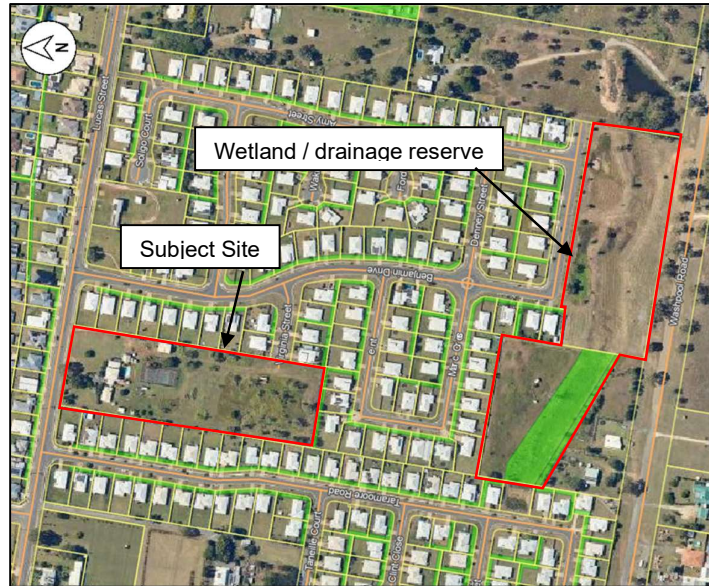
There is opportunity to regain a lot by extending the development further into the south-west corner of Lot 1. The on-site detention strategy would need to be revisited since provision of an additional lot or lots would further increase the total post-development runoff, so additional storage volume would likely be required.

## **5.3 Quality Mitigation Strategy**

Due to the size of the development (>2500m<sup>2</sup>) and the number of new lots created (>6), State Planning Policy Healthy Waters has been triggered.

An existing ‘wetland’ / stormwater easement was observed at the bottom of the catchment which captures runoff from the subject site and surrounding developed area. Based on discussion with GSPC, it is believed that this easement was designed to cater for flows from the entire catchment on the basis that all upstream land was fully developed. RRC does not have any record of a Stormwater Management Plan associated with the Graceview development, so the methodology to size the drainage easement cannot be reviewed. However, based on a site inspection and review of imagery, the sizing appears reasonable for the extent of upstream development.

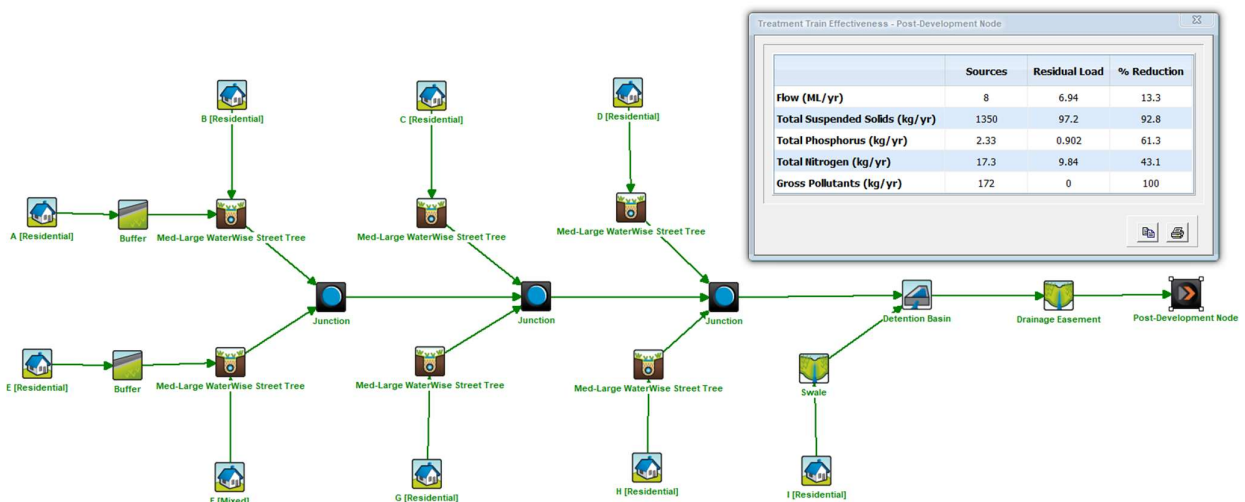
It is noted that approvals for the surrounding urban area were stamped prior to implementation of SPP water quality guidelines, so it is likely that water quality was not considered. However, this drainage reserve is likely to provide some degree of improvement to water quality prior to runoff from the development site reaching receiving waters.



It is necessary to implement additional stormwater quality improvement devices (SQIDs) to demonstrate a compliant quantitative reduction in pollutants from the development site.

It is proposed to provide a WaterWise Street Tree at or immediately upstream of each stormwater inlet, corresponding to a total of twelve (12) medium to large Street Trees for the whole development. The Street Trees were initially modelled as six (6) trees in accordance with Water by Design guidelines assume a ~6% filter-to-catchment ratio, corresponding to 70m<sup>2</sup> of filter media per tree. However, following discussion with council, it is noted this would require full-width utilisation of verges and council will not accept services being installed through the filter area. Therefore, it is proposed to provide the same total filter area, equating to 35m<sup>2</sup> per tree.

Given the ratio of filter media area to catchment area is the same, remodelling should not be required and the below MUSIC model treatment train is representative of the proposed treatment. The drainage reserve was incorporated into the model as a 'swale' since it does not appear to permanently hold water, with dimensions approximated from information available on Queensland Globe.



It is acknowledged that the Total Nitrogen reduction is slightly below SPP requirements (45%) for the Central Queensland (north) climatic zone. However, it is believed that the outcome is consistent with the intent of the Healthy Waters guidelines and Dileigh request a slight relaxation of the water quality targets. All other reduction targets are met or exceeded.

To further prevent release of gross pollutants from the site to the downstream network, it is proposed to provide a gross pollutant trap (GPT) to the basin drop-structure outlet in the form of a trash rack or similar.

The quantity of Watewise Street Trees could be reduced if some bio-retention filter media and underdrainage is provided in the detention basin, allowing it to function as a combined attenuation and treatment device. Tentatively, 300m<sup>2</sup> of filter media could practically be constructed in the basin, allowing up to ten (10) Watewise Street Trees to be omitted from the final treatment train at the developer's discretion and as per council preference.

## 6. Conclusion

The proposed development will increase the impervious area of the site. It is proposed to mitigate the increase in stormwater runoff by providing a detention basin with a nominal volume of 417m<sup>3</sup> in Lot 8. The basin shall discharge to the existing stormwater infrastructure in the existing easements (AQSP251111, APSP251111 and CSP251111) downstream via a low-flow drop structure with a pipe outlet and high-flow weir. It is proposed to provide Watewise Street Trees along the proposed road, with a minimum requirement of twelve (12) trees. Alternatively, a combination of Watewise Street Trees and bioretention in the basin could be provided. A GPT is to be provided in the detention basin drop structure to capture large debris.

Ashleigh Lucas  
For and On Behalf of  
Dileigh Consulting Engineers Pty Ltd

## Appendix A – Existing ‘Swale 1’ Hydraflow Express Report

# Channel Report

## Existing Swale A - Peak Capacity Check

### User-defined

Invert Elev (m) = 1.0000  
Slope (%) = 0.3000  
N-Value = 0.015

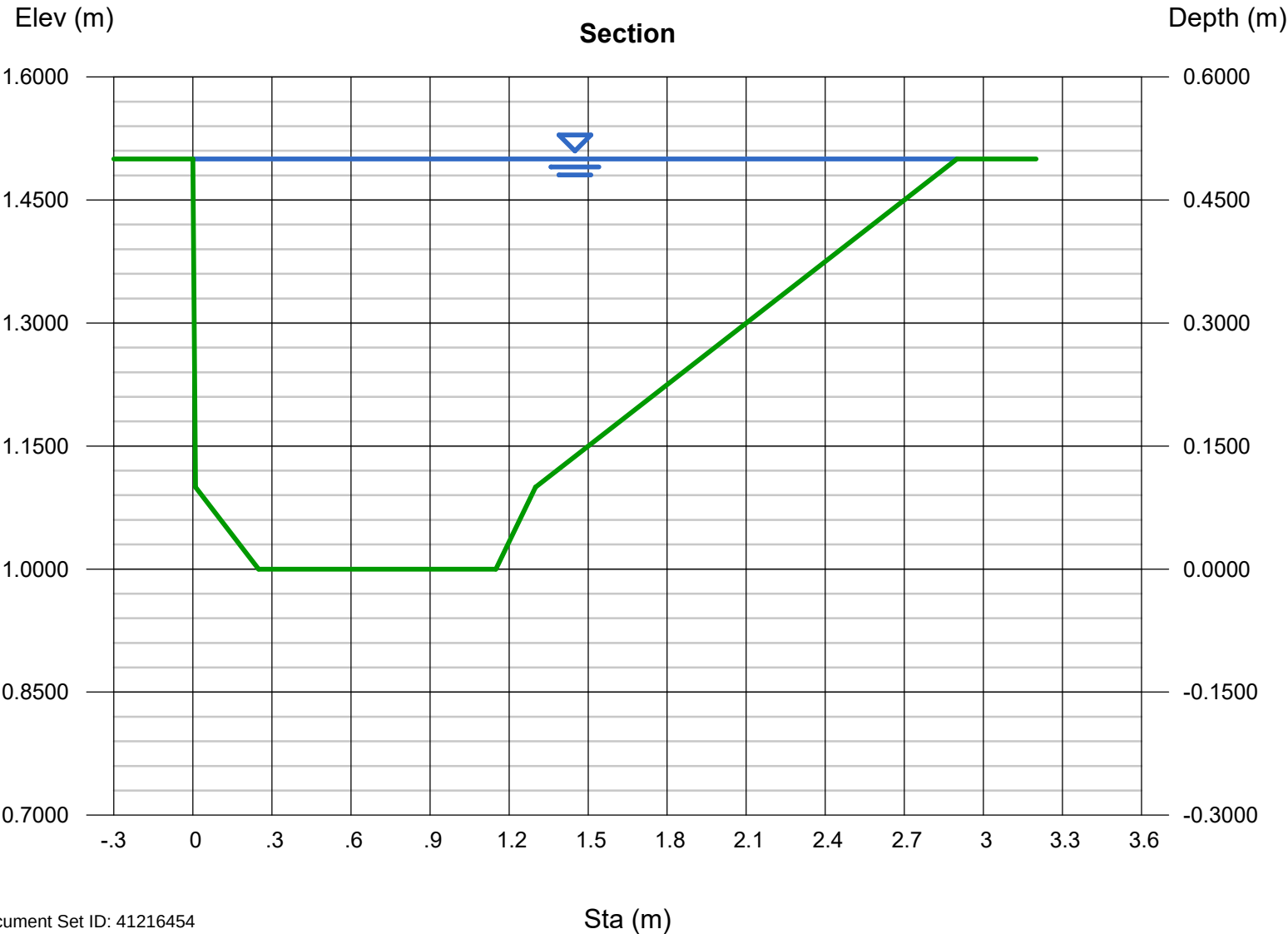
### Calculations

Compute by: Known Depth  
Known Depth (m) = 0.5000

### Highlighted

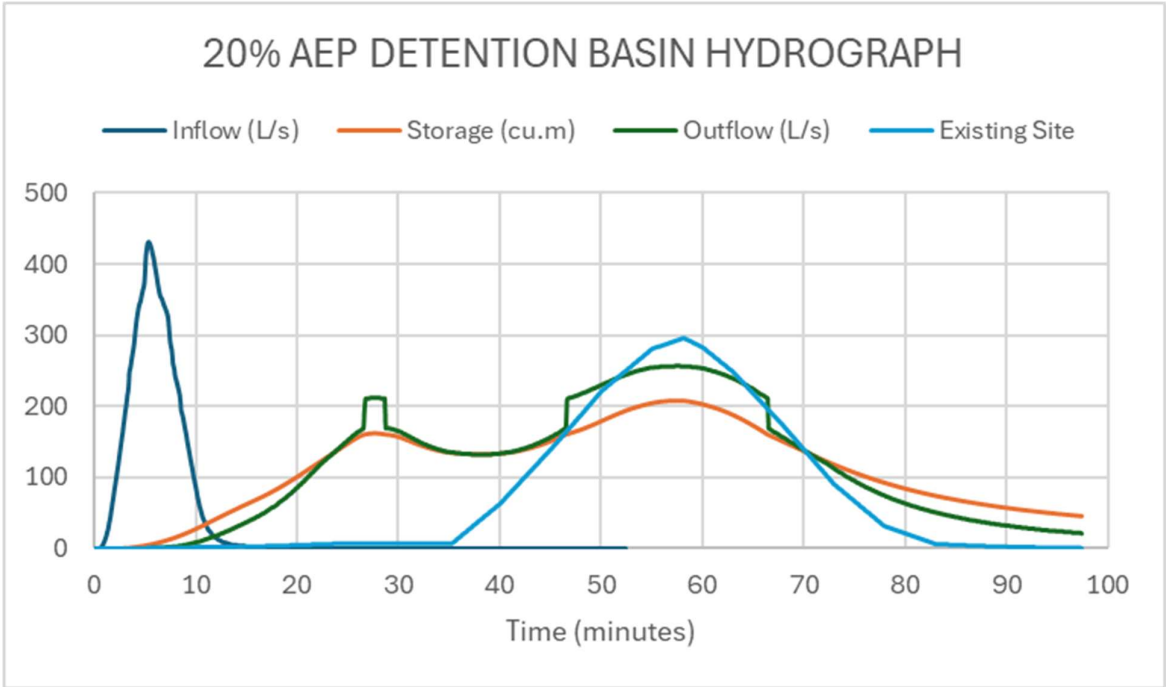
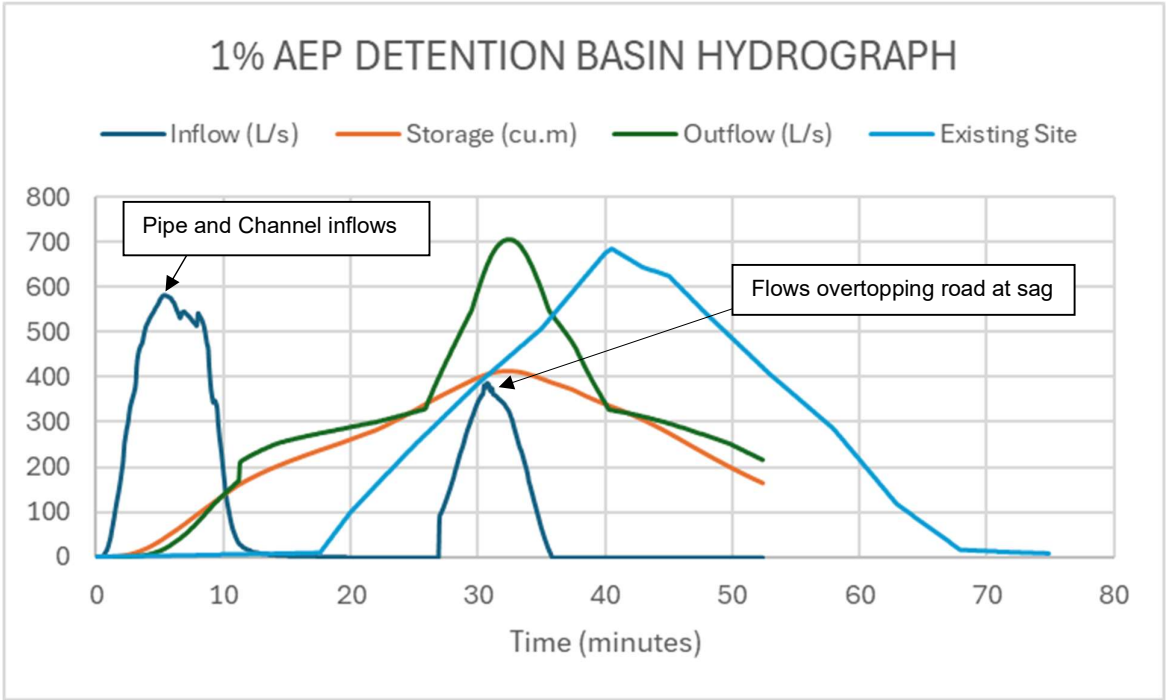
Depth (m) = 0.5000  
Q (cms) = 1.4823  
Area (sqm) = 0.9475  
Velocity (m/s) = 1.5644  
Wetted Perim (m) = 3.3896  
Crit Depth, Yc (m) = 0.4694  
Top Width (m) = 2.9000  
EGL (m) = 0.6248

(Sta, El, n)-(Sta, El, n)...  
( 0.0000, 1.5000)-(0.0100, 1.1000, 0.015)-(0.2500, 1.0000, 0.015)-(1.1500, 1.0000, 0.015)-(1.3000, 1.1000, 0.015)-(2.9000, 1.5000, 0.015)



## Appendix B – Stormwater Management Strategy Drawings

Appendix C – Detention Basin Hydrographs



## Appendix D – Basin Outlet Hydraulics Summary

|                    | Event AEP | Low-Flow Pipe            |                | High-Flow Weir           |                |           |
|--------------------|-----------|--------------------------|----------------|--------------------------|----------------|-----------|
|                    |           | Flow (m <sup>3</sup> /s) | Velocity (m/s) | Flow (m <sup>3</sup> /s) | Velocity (m/s) | Depth (m) |
| Pipe flow only {   | 50        | 0.160                    | 1.5            | 0                        | 0              | 0         |
|                    | 20        | 0.257                    | 1.9            | 0                        | 0              | 0         |
|                    | 10        | 0.297                    | 2.1            | 0                        | 0              | 0         |
| Pipe & weir flow { | 5         | 0.332                    | 2.2            | 0.032                    | 0.4            | 0.037     |
|                    | 2         | 0.349                    | 2.3            | 0.169                    | 0.7            | 0.074     |
|                    | 1         | 0.362                    | 2.4            | 0.345                    | 0.9            | 0.107     |

*All results are extracted from the preliminary Watercom DRAINS model prepared for this report.*

## Appendix E – Proposed Basin Weir Hydraflow Express Report

# Weir Report

## Maximum Weir Outflow for Prelim Basin

### Rectangular Weir

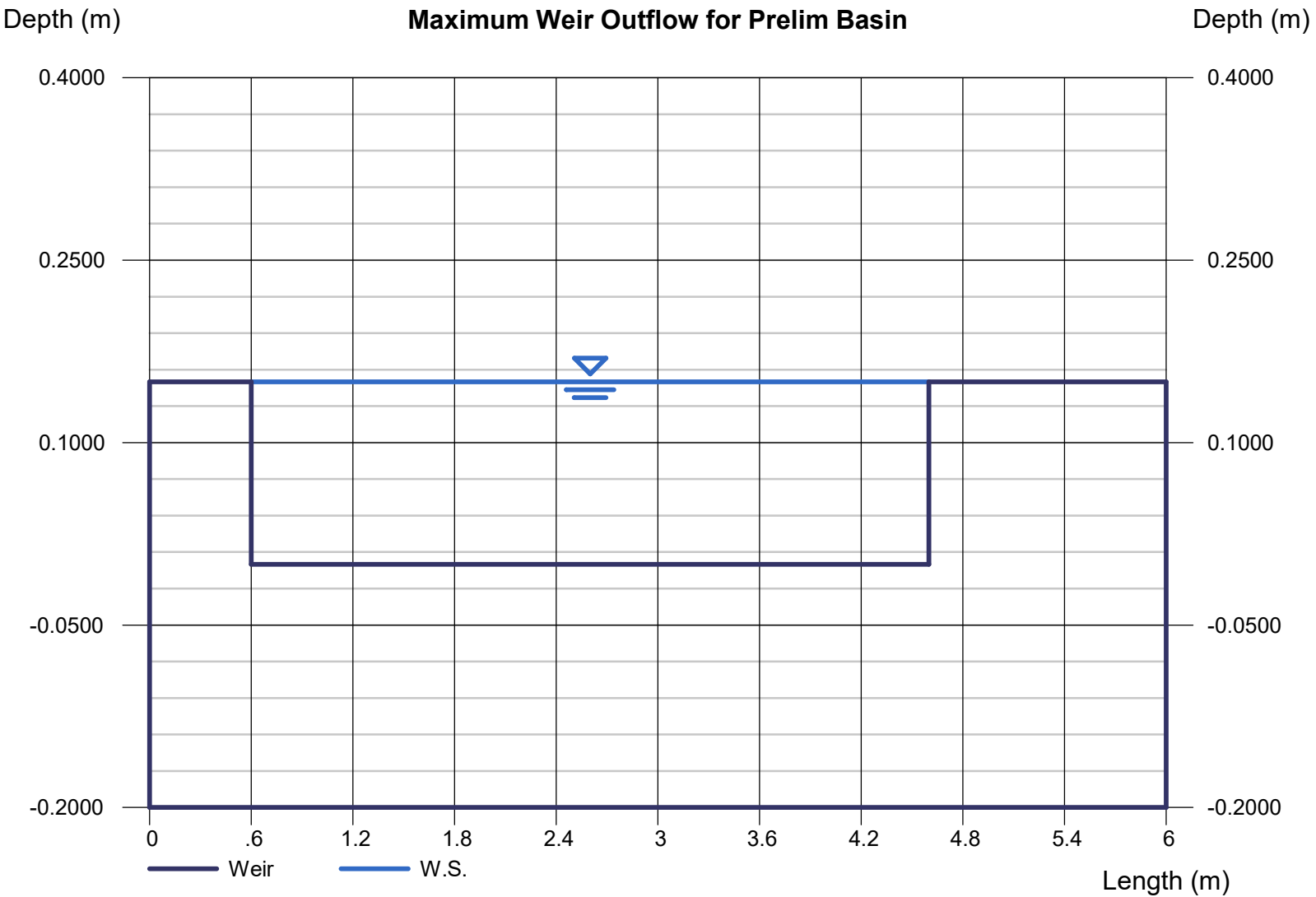
Crest = Sharp  
Bottom Length (m) = 4.0000  
Total Depth (m) = 0.1500

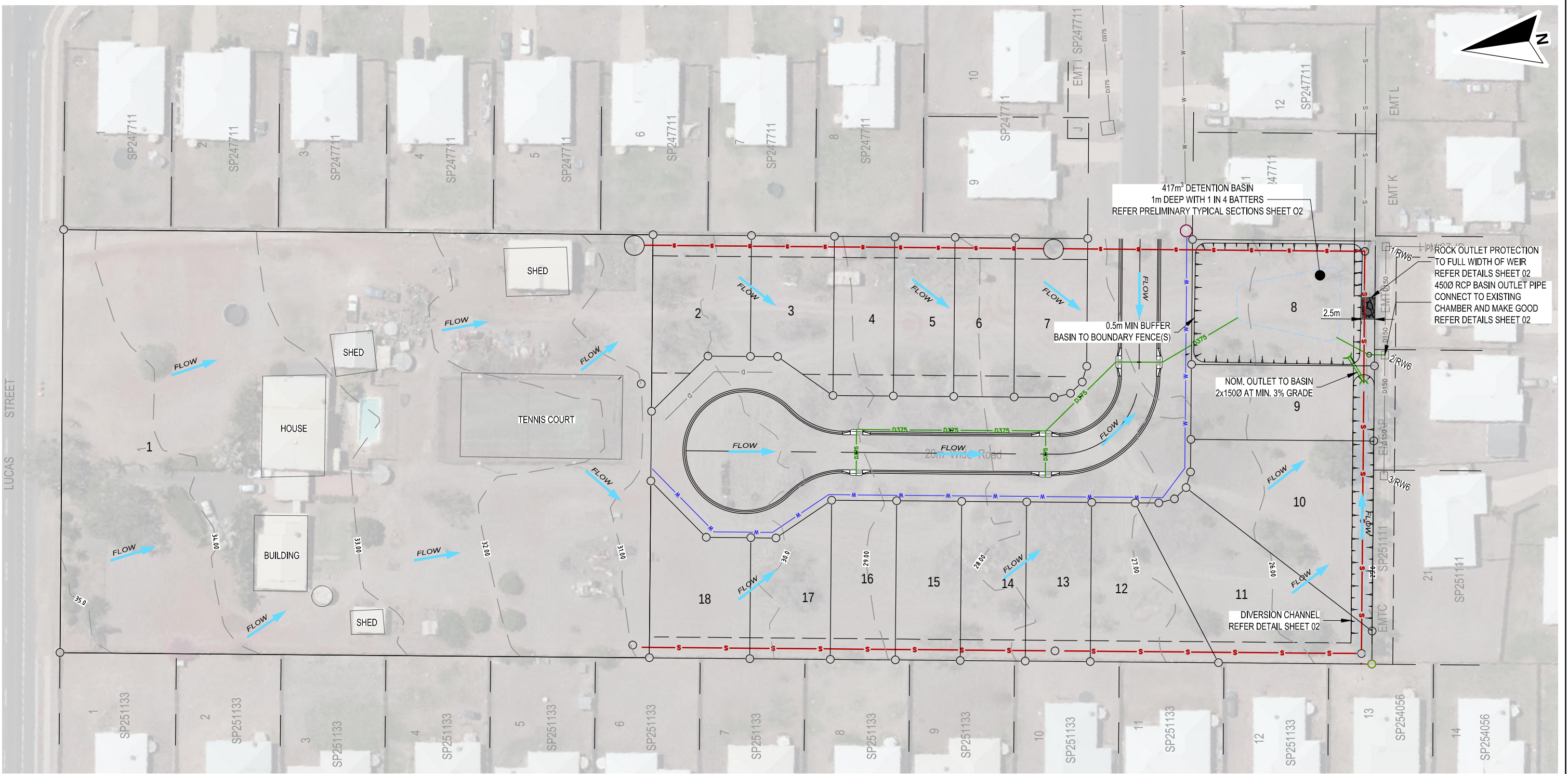
### Highlighted

Depth (m) = 0.1500  
Q (cms) = 0.7738  
Area (sqm) = 0.6000  
Velocity (m/s) = 1.2897  
Top Width (m) = 4.0000

### Calculations

Weir Coeff. Cw = 3.3300  
Compute by: Known Depth  
Known Depth (m) = 0.1500





LEGEND

- W EXISTING 1500 WATER MAIN (FROM RECORDS)
- S EXISTING 1500 SEWER MAIN (FROM RECORDS)
- D EXISTING STORMWATER (FROM RECORDS, DIAMETER AS INDICATED)
- 30.00 EXISTING SURFACE CONTOUR (LIDAR)
- W PROPOSED WATER MAIN
- S PROPOSED SEWER MAIN
- D PROPOSED DRAINAGE AND 2400 RGU (DIAMETER AS INDICATED)
- PROPOSED MOUNTABLE KERB AND CHANNEL
- PROPOSED TOP OF BATTER
- PROPOSED TOE OF BATTER
- FLOW ASSUMED POST-DEVELOPMENT OVERLAND FLOW DIRECTION

EXISTING LEVELS AND SERVICES

- THE CONTRACTOR SHALL VERIFY THE LOCATIONS AND LEVELS OF ALL EXISTING SERVICES WITH THE RELEVANT AUTHORITIES INCLUDING "DIAL BEFORE YOU DIG" PRIOR TO COMMENCING CONSTRUCTION.
- ANY COSTS ASSOCIATED WITH REPAIRING DAMAGE TO EXISTING SERVICES SHALL BE PAID FOR BY THE CONTRACTOR.
- THE CONTRACTOR SHALL VERIFY THAT THE EXISTING LEVELS ARE AS PER THIS DESIGN WHERE CONNECTIONS TO EXISTING INFRASTRUCTURE ARE REQUIRED. ANY DIFFERENCES TO BE NOTIFIED TO THE ENGINEER PRIOR TO ORDERING MATERIALS OR COMMENCING ANY WORKS.
- PRIOR TO COMMENCING WORKS THE CONTRACTOR SHALL VERIFY THAT THERE ARE NO CLASHES BETWEEN ANY CROSSING SERVICE OR PIPELINE. ANY CLASHES TO BE NOTIFIED TO THE ENGINEER PRIOR TO WORKS COMMENCING.
- PRIOR TO COMMENCING WORKS THE CONTRACTOR SHALL VERIFY LOCATION AND DETAILS OF ALL EXISTING SERVICE CONNECTIONS TO NEW ALLOTMENTS PREVIOUSLY INSTALLED

STORMWATER MANAGEMENT NOTES:

- ALL CALCULATIONS CARRIED OUT IN ACCORDANCE WITH THE QUEENSLAND URBAN DRAINAGE MANUAL AND MODELLED USING THE IL-CL METHOD IN WATERCOM DRAINS
- SITE CATCHMENTS TAKEN FROM ACTUAL SITE AREAS USING ACTUAL FRACTION IMPERVIOUS AND 80% FRACTION IMPERVIOUS FOR PROPOSED DEVELOPMENT AREAS IN ACCORDANCE WITH CMDG TABLE D05.06.1
- RAINFALL INTENSITIES TAKEN FROM BUREAU OF METEOROLOGY RAINFALL IFD DATA SYSTEM
- RUNOFF FROM LOTS 7-11 TO BE DIVERTED TO DETENTION BASIN WITH A CHANNEL
- RUNOFF FROM ALL OTHER LOTS TO BE CAPTURED IN PITS AND PIPES IN THE PROPOSED ROAD WITH FINAL LOCATIONS, QUANTITIES AND SIZES TO BE DETERMINED AT DETAILED DESIGN
- ALL PIPES MODELLED WITH PRELIMINARY DESIGN GRADES OF BETWEEN 0.5% AND 3.0%

ROCKHAMPTON REGIONAL COUNCIL

APPROVED PLANS

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Development Permit No.: D/150-2024

Dated: 23 January 2026

DWG No.

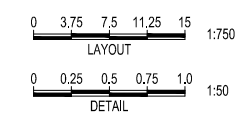
D24.363 SW01

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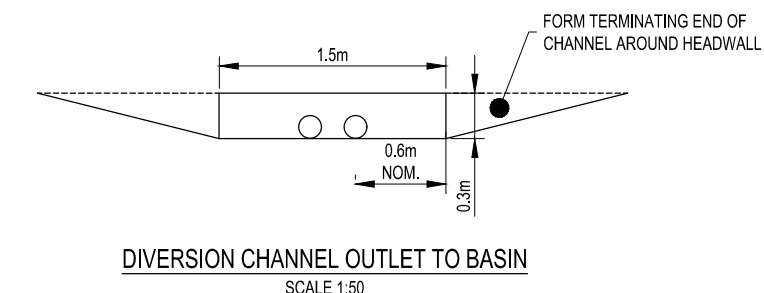
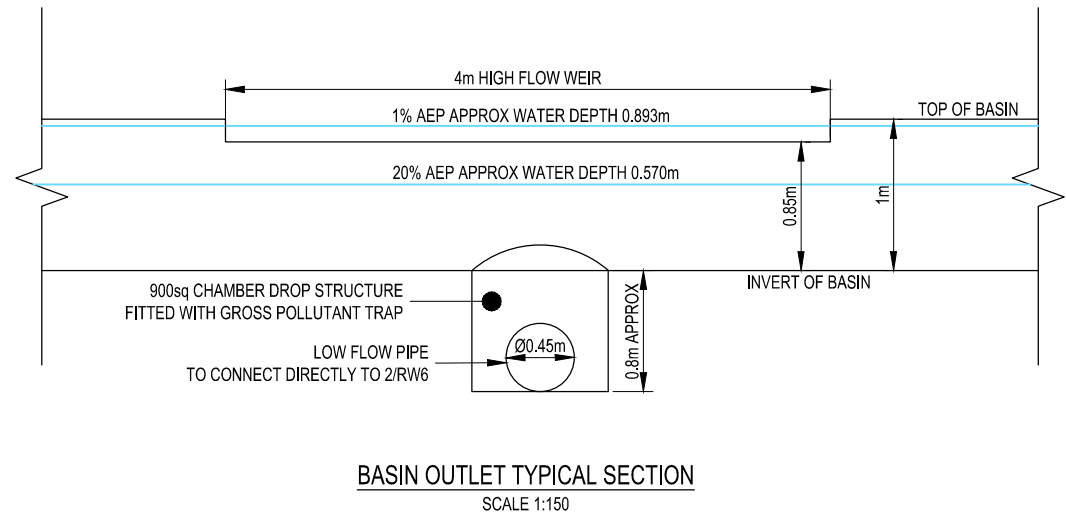
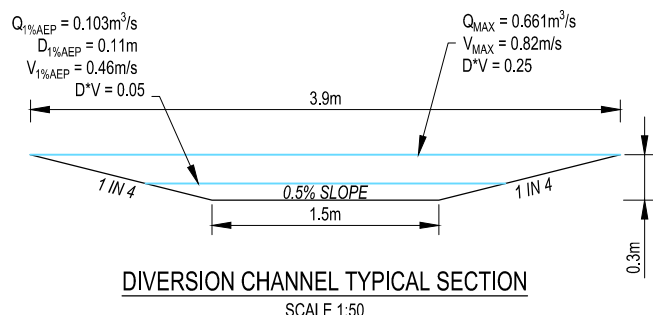
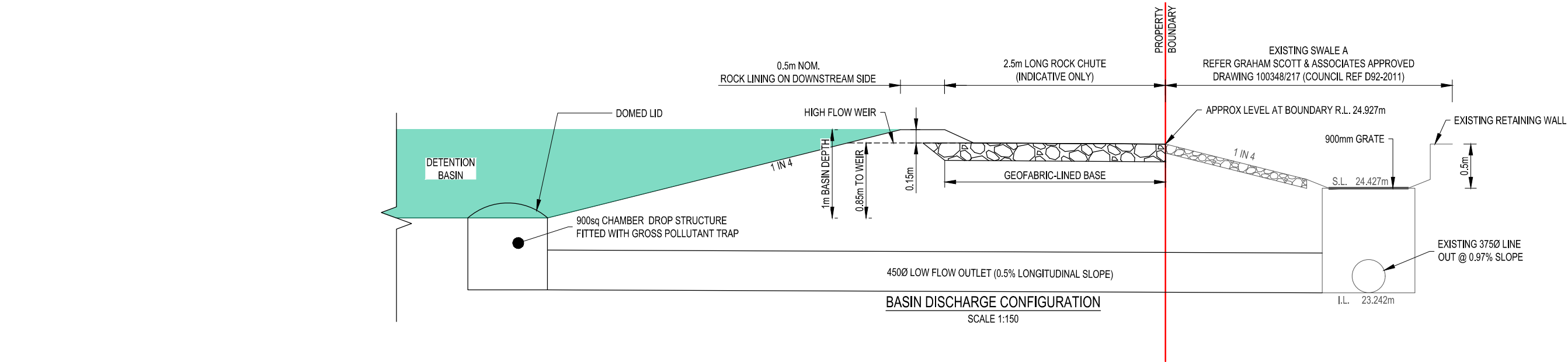
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|-----|------------------------------------------------|------------|
| A   | RESPONSE TO COUNCIL RFI                        | 23/01/2025 |
| B   | RESPONSE TO COUNCIL RFI                        | 14/03/2025 |
| C   | RESPONSE TO COUNCIL FURTHER ADVICE             | 01/08/2025 |
| D   | LOT YIELD AND STORMWATER CONFIGURATION ALTERED |            |



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| DRAFTED    | AML   |
| DESIGNED   | AML   |
| CHECKED    | T LAU |
| APPROVED   | T LAU |
| RPEQ 19272 | SIGN  |
| 27.11.2025 |       |

J STAGG  
PROPOSED RESIDENTIAL SUBDIVISION  
82 LUCAS STREET, GRACEMERE  
STORMWATER CONCEPT & MANAGEMENT PLAN  
PRELIMINARY STORMWATER LAYOUT



**ROCKHAMPTON REGIONAL COUNCIL**  
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**Dated: 23 January 2026**

DATUM: HORIZ. GDA 94 VERT. AHD

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FULL SIZE A3

Document Set ID: 41216454

Version: 1, Version Date: 01/12/2025

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|     |                                                |            |
|     |                                                |            |
|     |                                                |            |
|     |                                                |            |
|     |                                                |            |

DILEIGH

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PROPOSED RESIDENTIAL SUBDIVISION

82 LUCAS STREET, GRACEMERE

STORMWATER CONCEPT & MANAGEMENT PLAN

PRELIMINARY STORMWATER LAYOUT

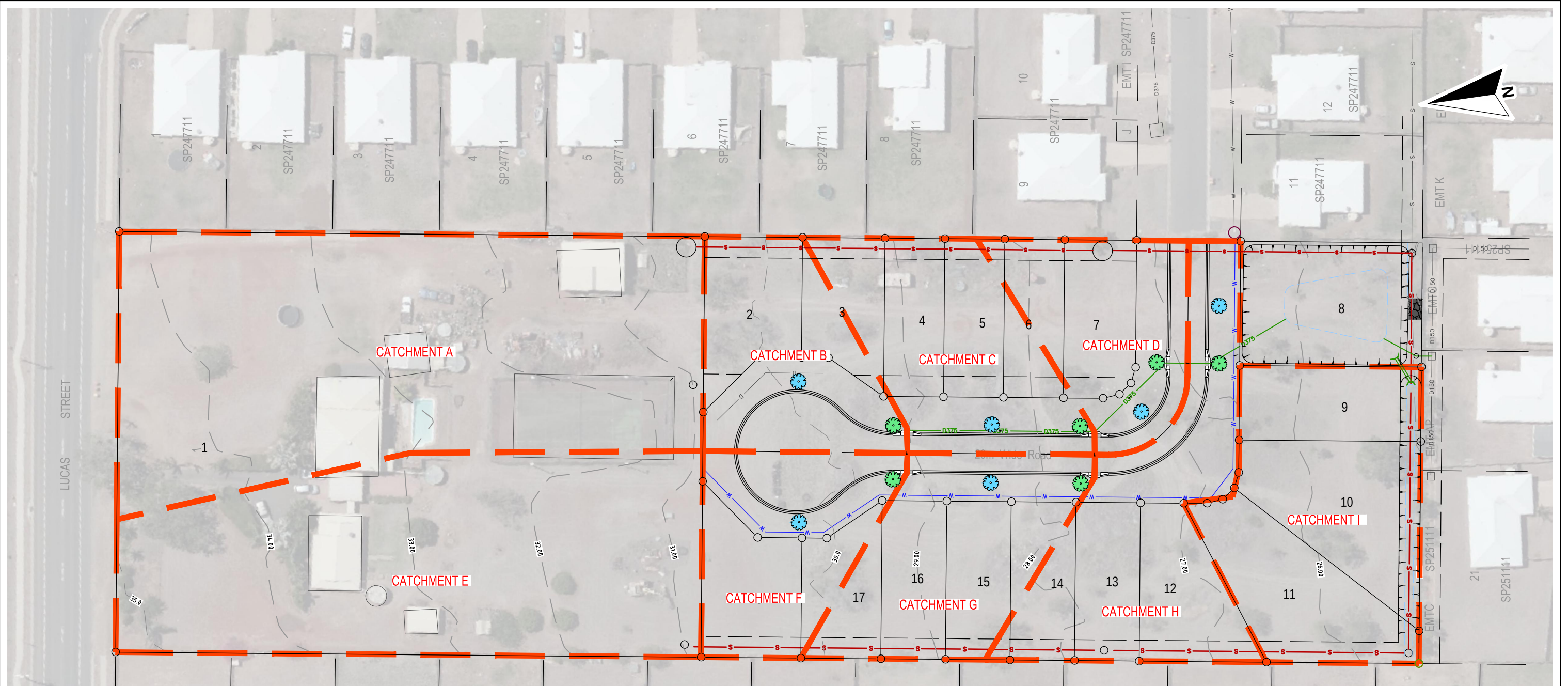
DWG No.

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LEGEND

- W EXISTING 1500 WATER MAIN (FROM RECORDS)
- S EXISTING 1500 SEWER MAIN (FROM RECORDS)
- D EXISTING STORMWATER (FROM RECORDS, DIAMETER AS INDICATED)
- 30.00 EXISTING SURFACE CONTOUR (LIDAR)
- W PROPOSED WATER MAIN
- S PROPOSED SEWER MAIN
- D PROPOSED DRAINAGE AND 2400 RGU (DIAMETER AS INDICATED)
- PROPOSED MOUNTABLE KERB AND CHANNEL
- PROPOSED TOP OF BATTER
- PROPOSED TOE OF BATTER
- ASSUMED CATCHMENT AREA
- MEDIUM - LARGE WATERWISE STREET TREE WITHIN TURF (IN ACCORDANCE WITH WATER BY DESIGN GUIDELINES AND BCC STD DWG VSD-9034)
- MEDIUM - LARGE WATERWISE STREET TREE WITH WSUD KERB INLET (IN ACCORDANCE WITH WATER BY DESIGN GUIDELINES AND BCC STD DWG UMS 570)

ROCKHAMPTON REGIONAL COUNCIL

AMENDED PLANS APPROVED

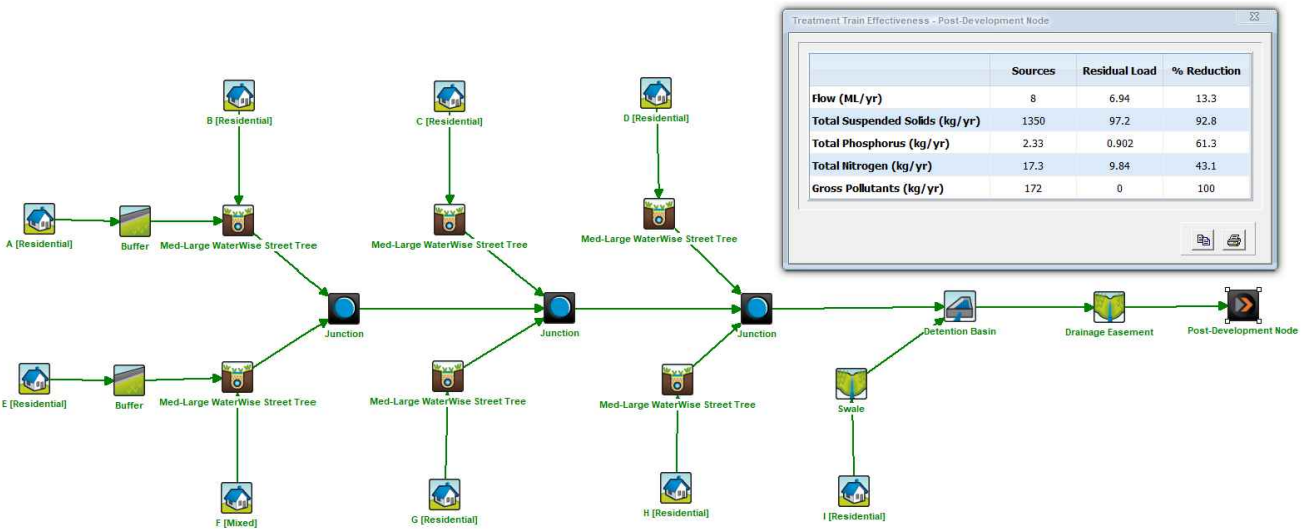
2 September 2026

DATE

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Dated: 23 January 2026



DATUM: HORIZ. GDA 94 VERT. AHD

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PRELIMINARY QUALITY TREATMENT TRAIN

DWG No. D24.363-SW03

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