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Arrow South Zone Change – Infrastructure Feasibility Report

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Arrow South Zone Change – Infrastructure Feasibility Report

Report prepared For: Arrowsouth Properties Limited

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1	Draft for comment	24th November 2025
2	Updated following review	26th November 2025
3	Updated following initial QLDC review	16th February 2026
4	Updated following review	25th February 2026

Executive Summary

Introduction

This Infrastructure Servicing Feasibility Report assesses the technical viability of servicing the proposed **Arrowtown South Plan Change** at McDonnell Road, Arrowtown. The proposal seeks to rezone approximately 4.7 hectares of land from its current Arrowtown South Special zoning to **Suburban Residential Zone**.

Development Scenario

To ensure a robust assessment of infrastructure capacity, a conservative maximum yield of **61 residential allotments** has been adopted for all demand calculations and modelling exercises. This is greater than the maximum yield that could be provided for by the proposed minimum lot size as shown on the concept subdivision plans and represents a conservative approach to assessing infrastructure capacity.

Key Findings by Discipline

- **Water Supply (Feasible):**
  Hydraulic modelling by Watershed Engineering confirms that the development can be serviced via connections to the existing mains on McDonnell Road and Brodie Avenue. The model demonstrates that the network maintains adequate residual pressures and can achieve the **FW2 Fire Fighting classification** (providing >25 L/s fire flow) required for residential development.

- **Wastewater Services (Feasible subject to upgrades):**
  Hydraulic modelling undertaken by HAL Consulting identifies that the existing McDonnell Road Pump Station and the immediate downstream network are currently operating at capacity. To service the proposed rezoning, specific upgrades are required to resolve both existing constraints and accommodate the new load. The required works include:
  - Upgrading the **McDonnell Road Pump Station** capacity from 14 L/s to 26 L/s.
  - Upsizing approximately 700m of the downstream gravity main from **150mm to 225mm diameter**.

  Subject to these upgrades, the disposal of wastewater is confirmed as feasible.

- **Stormwater Management (Feasible):**
  The change in land use will result in increased runoff. A site-specific Stormwater Feasibility Report by Civilised Ltd confirms that this can be managed via **on-site attenuation**. Two dry detention basins (with a combined storage volume of approximately 1,400 m^3) are proposed within the reserve areas. These basins will restrict post-development peak flows to pre-development levels for storm events up to the **1% AEP (100-year)** magnitude.

- **Power and Telecommunications (Feasible):**
  Aurora Energy has confirmed that the local electricity network has sufficient capacity to supply the development. Telecommunications services are available via extension of the adjacent fibre network or wireless alternatives.

Conclusion

The investigations conclude that the proposed rezoning is **fully serviceable** from an engineering perspective. While wastewater network upgrades are required, the solutions are clearly defined and technically viable. There are no impediments to infrastructure provision that would prevent the plan change from proceeding.

Table of Contents

1 Introduction

1.1 Background and Purpose

This Infrastructure Servicing Feasibility Report has been prepared by Civilised Ltd for **Arrowsouth Properties Ltd (APL)**. It supports the proposed plan change to rezone land at McDonnell Road, Arrowtown, from Arrowtown South Special zone to **Suburban Residential Zone (SRZ)**.

The purpose of this report is to assess the feasibility of servicing the proposed development that would be enabled by the plan change with Three Waters infrastructure (Potable Water, Wastewater, and Stormwater) and to demonstrate that the proposed rezoning can be adequately serviced without adversely affecting the existing Queenstown Lakes District Council **(QLDC)** network or the surrounding environment.

This report summarizes the findings of detailed modelling and investigative reports appended hereto, providing a consolidated overview of the infrastructure requirements.

1.2 Site Description

The subject site is located on the eastern side of McDonnell Road, Arrowtown, adjacent to the existing Arrowtown urban boundary and near the Arrowtown Retirement Village. The site comprises two parcels of land legally described as:

- Lot 102 DP 535793
- Lot 103 DP 535793

The total site area is approximately **4.7 hectares**. The land is currently undeveloped (greenfield) and is characterized by rolling topography with an existing watercourse traversing the site from northeast to southwest.

1.3 Proposed Development

The proposal seeks to rezone the land to enable residential development. While the final subdivision layout is subject to detailed design, two density scenarios were considered during the feasibility phase:

1. **Scenario A (Current PDP Density for the low density zoning being sought):** Minimum lot area of 450 m², yielding ~44 lots.
2. **Scenario B (Intensification Variation):** Minimum lot area of 300 m², yielding ~59 lots.

To ensure a robust assessment of infrastructure capacity, a yield of **61 residential allotments** has been adopted as the basis for all demand calculations in this report. This is greater than the maximum yield that could be provided for by the proposed minimum lot size as shown on the

concept subdivision plans and represents a conservative approach to assessing infrastructure capacity.

Conceptual subdivision plans showing potential lot configurations are included in **Appendix A**.

2 Design Standards and Level of Service

2.1 Regulatory Framework

All infrastructure investigation and design parameters referenced in this report have been developed in accordance with:

- **QLDC Land Development and Subdivision Code of Practice (2025) (CoP);**
- **NZS 4404:2010 Land Development and Subdivision Engineering;** and
- **The New Zealand Building Code.**

2.2 Water Supply Standards

Water demand has been calculated based on the QLDC CoP requirements for the Low Density Suburban Residential zone.

- **Daily Demand:** A demand of **2,100 litres per day per allotment** has been adopted (based on standard occupancy and per capita consumption rates).
- **Total Design Demand:** Based on the maximum yield of 61 lots, the total daily demand is calculated as **128.1 m³/day** (128,100 litres).
- **Fire Fighting:** The internal reticulation will be designed to meet **FW2** classification (residential) as per SNZ PAS 4509:2008, requiring a minimum flow of 12.5 L/s within 135m of any dwelling.

2.3 Wastewater Standards

Wastewater generation rates have been derived from the QLDC CoP standards for residential dwellings.

- **Daily Generation:** A generation rate of **750 litres per day per dwelling** has been adopted.
- **Total Design Load:** Based on 61 lots, the total average daily wastewater generation is calculated as **45.75 m³/day** (45,750 litres).
- **Peaking Factors:** For the purpose of hydraulic pipe sizing and network modelling, appropriate peaking factors **(PF)** for dry weather and wet weather flows **(PWWF)** have been applied by the QLDC modellers in accordance with the CoP.

2.4 Stormwater Management Standards

Stormwater management feasibility is based on the principle of **Hydraulic Neutrality**, ensuring that development does not increase flood risk downstream. This is consistent with the QLDC CoP (2025) and the stormwater reporting has been carried out in accordance with the latest version of the CoP.

- **Primary Design Standard:** Post-development peak discharge rates must not exceed pre-development levels for the **5% AEP (20-year)** and **1% AEP (100-year)** rainfall events.
- **Climate Change:** Rainfall data includes allowances for climate change (RCP 8.5 to 2100).
- **Quality:** Treatment of runoff via dry detention basins to facilitate sediment removal prior to discharge into the receiving watercourse.

3 Water Supply

3.1 Existing Infrastructure

The subject site is situated between two potential connection points to the existing QLDC water reticulation network.

- **McDonnell Road:** An existing **200mm diameter** water main runs along the southern berm of McDonnell Road, directly adjacent to the site's northern boundary.
- **Brodie Avenue:** An existing **125mm diameter** water main terminates at the current end of Brodie Avenue/Patton Place, adjacent to the site's eastern boundary.

3.2 Connection Strategy and Demand

It is proposed to service the development via a **ring-main configuration**. This involves creating a through-connection linking the existing 200mm main on McDonnell Road to the existing infrastructure at Brodie Avenue. This approach maximises network redundancy and ensures robust flow circulation.

As detailed in Section 2.2, the total theoretical peak daily demand for the 61 proposed allotments is **128.1 m³/day** (based on 2,100 L/Lot/day).

3.3 Hydraulic Modelling Assessment

A detailed hydraulic analysis was undertaken by **Watershed Engineering Ltd** (August 2025) to assess the capacity of the network to support the proposed rezoning. The full report is attached in **Appendix B**.

The model was updated to include the 61-lot scenario and validated against 2024/2025 SCADA data. The assessment yielded the following results regarding Level of Service (LoS):

Service Pressure

The Council's Code of Practice requires service pressures to be maintained between **200 kPa** (30m) and **900 kPa** (90m).

The modelling results confirm that the proposed development achieves high levels of service pressure.

- **Connection Point (Brodie Ave):** The residual pressure post-development is modelled at **61.73m (approx. 605 kPa)**.
- **Internal Performance:** Pressures within the subdivision are well within the allowable operating range.

Network Impact

The report notes that the wider Arrowtown network currently experiences high head losses during peak demand periods. While the addition of the Arrow South development results in a minor pressure drop (~2.4m) in the immediate vicinity, the system pressure remains significantly above the minimum 300kPa requirement and will not trigger a failure of service levels within the site or the immediate boundary.

3.4 Fire Fighting Supply

The proposed development requires a fire risk classification of **FW2** (Residential), which mandates a supply of **25 L/s** within 135m of any dwelling.

The modelling confirms that ample fire flow capacity is available.

- **Available Flow:** The model indicates an available fire flow of **70.46 L/s** at the Brodie Avenue hydrant (Hydrant ID: 328887). This is the nearest hydrant to the subject site.
- **Compliance:** This significantly exceeds the FW2 requirement of 25 L/s.

3.5 Conclusion on Water Supply

The hydraulic modelling confirms that the proposed rezoning to Lower Density Suburban Residential is **feasible** from a water supply perspective.

- The site can be serviced via standard connections to the existing McDonnell Road and Brodie Avenue mains.
- There is sufficient capacity to meet domestic demand and FW2 fire-fighting requirements.
- While the modeller noted existing constraints in the wider Arrowtown network (specifically head losses in the southern high-elevation zones), the proposed development does not trigger a failure of service levels within the site or the immediate boundary.

4 Wastewater Services

4.1 Existing Infrastructure

The site is in the catchment of the **McDonnell Road Wastewater Pump Station (WWPS)**, situated to the north-west of the subject site. This pump station conveys flows eastwards to the Norfolk Street Pump Station, which ultimately discharges to the Shotover Wastewater Treatment Plant.

The neighbouring Stage 1 of the Arrow South development is immediately adjacent to the existing McDonnell WWPS. The reticulation within the Stage 1 development consists of low pressure system due to topography and ground conditions. It is proposed that Stage 2 of Arrow South, which would be enabled by the proposed plan change, will connect directly to the pump station or the last manhole prior to the pump station (Manhole ID: 101471).

4.2 Projected Wastewater Load

Wastewater generation has been calculated in accordance with the QLDC Code of Practice (CoP) based on the maximum yield of 61 residential units.

- **Average Dry Weather Flow (ADWF):** 45.75 m^3/day (based on 750 L/dwelling/day).
- **Peak Wet Weather Flow (PWWF):** For hydraulic modelling purposes, a theoretical gravity peak flow of **2.65 L/s** was applied to the development.

4.3 Hydraulic Modelling Assessment

HAL Consulting was engaged to assess the capacity of the downstream network to accept the proposed flows. The assessment utilized the *Wakatipu Wastewater Calibrated Model (2024)*. The full report is attached in **Appendix C**.

The modelling identified a pre-existing capacity constraint within the network.

- **Existing Pump Station Capacity:** The McDonnell Road WWPS has a modelled capacity of **14 L/s**.
- **Existing Inflows:** The model indicates that *current* peak wet weather inflows (from Stage 1 of the Arrow South development, Lifestyle Village Stage 1, etc.) already reach approximately **18 L/s**, resulting in theoretical overflows during significant storm events.
- **Post-Development Inflows:** The addition of the Arrow South development increases the peak inflow to approximately **26 L/s**.

4.4 Proposed Connection Strategy

Due to known ground conditions in the area (high groundwater tables experienced during the adjacent Stage 1 Arrow South development) and assessed in the Geotechnical Report prepared by GeoSolve Limited in support of the plan change, APL proposes to service the internal subdivision via

a **Low Pressure Sewer (LPS)** system. This method minimizes the depth of excavations and reduces the risk of groundwater infiltration.

While QLDC generally prefers gravity reticulation, the HAL Consulting assessment utilized conservative gravity-based flow peaks to ensure the downstream analysis was robust. The modelling confirms that regardless of whether LPS or Gravity is used internally, the downstream constraints described below must be addressed.

4.5 Required Network Upgrades

The impact assessment concludes that the development is feasible **subject to specific upgrades** to the downstream infrastructure. The model demonstrates that the following upgrades resolve the capacity constraints for both the proposed development and the existing catchment:

1. **Pump Station Upgrade:** The McDonnell Road WWPS pumps require upgrading to increase capacity from **14 L/s to 26 L/s** (matching the post-development peak inflow).
2. **Gravity Main Upgrade:** To handle the increased pumping rate, the downstream gravity network requires upgrading to prevent surcharging. The model identifies approximately **700m of pipework** requiring upsizing from **150mm to 225mm diameter** (specifically between Manhole 101524 and Manhole 101561).

4.6 Conclusion on Wastewater

The disposal of wastewater from the proposed rezoning is **feasible**, provided the identified network upgrades are implemented.

- The modelling confirms that the proposed upgrades (Pump Station and Gravity Main) successfully mitigate the effects of the development and resolve existing pre-development capacity issues.
- The upgrades are technically viable and standard civil engineering works.
- Detailed design of these upgrades, and the funding mechanisms (e.g., Development Contributions), will be determined in consultation with QLDC during the subdivision consent phase.

5 Stormwater Management

5.1 Catchment Context

The site is traversed by an existing watercourse flowing from northeast to southwest. This channel conveys runoff from the upstream catchment, including parts of Feehly Hill, the Hills Golf Course, and properties along McDonnell Road.

Currently, the site is greenfield land with no formal drainage infrastructure. Runoff generally flows overland towards the watercourse. The soils on site have been identified as poorly drained and generally unsuitable for soakage/infiltration.

5.2 Impact of Development

The rezoning to Lower Density Suburban Residential will introduce impervious surfaces (roofs, roads, hardstanding) estimated at approximately **65% coverage**. Without mitigation, this would result in a significant increase in both runoff volume and peak discharge rates during rainfall events, potentially increasing flood risk to downstream properties.

5.3 Management Strategy

A Stormwater Feasibility Report has been prepared by **Civilised Ltd** to assess the management options (refer to **Appendix D**). The recommended strategy relies on the principle of **Hydraulic Neutrality**, where post-development peak flows are attenuated to match pre-development levels.

The proposed solution involves:

- **Dual-Network System:** Two separate drainage networks servicing the eastern and western portions of the development respectively.
- **Dry Detention Basins:** Construction of two attenuation basins located in the natural low points of the site (within proposed reserve areas). These basins are designed to remain dry during normal conditions and fill only during storm events.
- **Discharge Control:** Outflows from the basins will be restricted via multi-stage outlet structures (orifice plates and weirs) before discharging into the existing watercourse.

5.4 Feasibility Findings

Detailed hydrological modelling using *Autodesk Storm and Sanitary Analysis (SSA)* confirms that the proposed strategy is technically feasible. Key findings include:

- **Attenuated Performance:** The design successfully restricts post-development peak flows to pre-development levels for both the **5% AEP (20-year)** and **1% AEP (100-year)** rainfall events, including allowances for climate change.
- **Storage Requirements:** A total combined storage volume of approximately **1,400 m³** is required (split between the two basins).
- **Spatial Fit:** The site has sufficient area to accommodate the required basin footprints (approximately 400m² base area each).

Figure 1: Indicative pond size and location

5.5 Conclusion on Stormwater

The proposed rezoning can be serviced regarding stormwater management without adverse effects on the environment or downstream infrastructure. The proposed attenuation system is robust, complies with QLDC engineering standards, and fits spatially within the development concept.

6 Power

6.1 Existing Infrastructure

Existing electricity distribution infrastructure is located within the road reserve of McDonnell Road, directly adjacent to the subject site. The local network is owned and operated by **Aurora Energy**.

6.2 Availability of Supply

Confirmation has been sought from Aurora Energy regarding the capacity of the network to supply the proposed rezoning. Aurora has confirmed that the existing network has sufficient capacity to service the anticipated demand from the proposed 61 residential allotments.

Connection to the grid will typically be made via the existing reticulation on McDonnell Road or by extending the network from the adjacent subdivision at Brodie Avenue.

6.3 Conclusion on Power

There are no identified constraints to providing a reliable power supply to the development. Written confirmation from Aurora Energy supporting the feasibility of the connection is included in **Appendix E**.

7 Telecommunications

7.1 Existing Infrastructure

The subject site is located adjacent to the established urban area of Arrowtown and the recently completed residential stages at Brodie Avenue. These adjacent areas are serviced by reticulated Ultra-Fast Broadband (UFB) fibre infrastructure, principally provided by **Chorus**.

7.2 Proposed Servicing Strategy

The primary strategy for the development is to extend the existing fibre network from McDonnell Road or Brodie Avenue to service the new allotments. Given the proximity to existing infrastructure, it is anticipated that a standard extension of the Chorus network will be technically viable.

7.3 Alternative Connectivity

In the unlikely event that a physical fibre connection is constrained or unavailable at the time of development, the site is well-served by wireless telecommunications infrastructure.

- **Mobile Broadband:** The area has coverage from multiple mobile network operators (Spark, One NZ, 2degrees) offering 4G and 5G fixed-wireless broadband solutions suitable for residential use.
- **Satellite Services:** Low Earth Orbit (LEO) satellite providers (such as Starlink) offer high-speed internet options that operate independently of ground-based reticulation.

7.4 Conclusion on Telecommunications

Access to telecommunications services is not considered a constraint to the proposed rezoning. It is expected that the site will be serviced via a hardwired fibre extension, with robust wireless alternatives available to ensure connectivity is guaranteed for future residents.

8 Conclusions and Recommendations

8.1 Conclusion on Feasibility

This report has assessed the engineering feasibility of servicing the proposed Arrow South zone plan change at McDonnell Road, Arrowtown. Based on the projected demand from a maximum yield of **61 residential allotments**, the assessment concludes that the development is fully serviceable.

A summary of the infrastructure status and requirements is provided below:

Infrastructure	Feasibility Status	Key Requirements / Upgrades
Water Supply	**Feasible**	➢ Connection via ring-main to McDonnell Rd (200mm) and Brodie Ave (100mm). ➢ Modelling confirms adequate pressure and FW2 Fire Fighting flow.
Wastewater	**Feasible** *(Subject to Upgrades)*	➢ **Pump Station Upgrade:** McDonnell Rd PS capacity to be increased from 14 L/s to 26 L/s. ➢ **Network Upgrade:** Approximately 700m of downstream gravity main to be upsized from 150mm to 225mm.
Stormwater	**Feasible**	➢ **On-site Attenuation:** Construction of two dry detention basins (approx. 1,400m³ total volume) to achieve hydraulic neutrality for 1% AEP events.
Power	**Feasible**	➢ Connection to Aurora network on McDonnell Rd (Capacity confirmed).
Telecoms	**Feasible**	➢ Extension of Chorus fibre network or utilization of wireless/satellite alternatives

8.2 Recommendations

Based on the investigations, modelling, and preliminary design work undertaken, it is recommended that:

1. **Rezoning Approval:** The proposed rezoning to Lower Density Suburban Residential proceeds, as there are no engineering impediments that cannot be addressed through standard infrastructure upgrades and design solutions.
2. **Wastewater Upgrades:** APL acknowledges the capacity constraints identified in the McDonnell Road Pump Station and downstream network. It is recommended that the design and funding of these specific upgrades (Pump Station and Gravity Main) be agreed upon with QLDC as part of the subsequent subdivision consent process.
3. **Detailed Design:** The project proceeds to the detailed engineering design phase to finalize the geometric layout of the stormwater basins and the specific alignment of utility connections.

9 Limitations

This report has been written for the particular brief to Civilised Ltd from their client and no responsibility is accepted for the use of the report for any other purpose, or in any other context or by any third party without prior review and agreement.

In addition, this report contains information and recommendations based on information obtained from a variety of methods and sources including inspection, sampling or testing at specific times and locations with limited site coverage and by third parties as outlined in this report. This report does not purport to completely describe all site characteristics and properties, and it must be appreciated that the actual conditions encountered throughout the site may vary, particularly where ground conditions and continuity have been inferred between test locations. If conditions at the site are subsequently found to differ significantly from those described and/or anticipated in this report, Civilised Ltd must be notified to advise and provide further interpretation.

Appendix A

Drawings

- Concept Subdivision Plans - Coterra Ltd
- Site Location Map

DRAFT

NOTES:

1. Aerial imagery has been sourced from UAV survey completed by others on 5/04/2025.

2. Areas and dimensions are subject to survey and detailed design.

3. This lot layout is indicative only and does not comply with current QLDC Rules.

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Project:

ARROWSOUTH PROPERTIES LTD

8 BRODIE AVENUE, ARROWTOWN

Title:

ARROWSOUTH PLAN CHANGE

PROPOSED SUBDIVISION OF

LOTS 102 & 103 DP 535793

CONCEPT PLAN (450m2 MIN. LOTS)

Revision	Amendments	Date

Scale: 1:1500 (A3)

Job No:	Drawn By:	Reviewed By:	Datum:
5479	CRH		-

Drawing No:	Sheet No:	Revision:	Date Created:
5479-1	1 OF 2	A	FEB 2026

DRAFT

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Project:
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8 BRODIE AVENUE, ARROWTOWN

Title:
ARROWSOUTH PLAN CHANGE
PROPOSED SUBDIVISION OF
LOTS 102 & 103 DP 535793
CONCEPT PLAN (300m2 MIN. LOTS)

Revision	Amendments	Date

Scale:			
1:1500 (A3)			
Job No: 5479	Drawn By: CRH	Reviewed By:	Datum: -
Drawing No: 5479-1	Sheet No: 2 OF 2	Revision: A	Date Created: FEB 2026

Arrow South

A	21/11/2025	Initial issue	JFM
REV	DATE	DESCRIPTION	APPROVED

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JFM	21/11/2025
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JFM	21/11/2025
DRAWN	DATE
JFM	21/11/2025
CHECKED	DATE

CLIENT

ARROWSOUTH PROPERTIES LIMITED

PROJECT/LOCATION

ARROW SOUTH REZONING
MCDONNELL ROAD

CONTRACT NUMBER

SCALE (AT A3) 1:25000

TITLE

STORMWATER DRAINAGE
SITE LOCATION MAP

DRAWING NUMBER QV075-F-610

REVISION A

Appendix B

Water Supply Modelling Report

- By Watershed Engineering Ltd

WATERSHED

5 August 2025

Nisha Dahal
Queenstown Lakes District Council
10 Gorge Road
Queenstown

Dear Nisha,

MCDONNELL ROAD, ARROWTOWN - RESIDENTIAL DEVELOPMENT

As per your request, we have undertaken hydraulic modelling to assess the ability to supply water to the proposed 61 lot residential development adjacent to McDonnell Road, Arrowtown, with respect to achieving the levels of service required by Queenstown Lakes District Council. The site address is 8 Brodie Avenue, Arrowtown, Lot 102 DP 535793 and Lot 103 DP 535793 with a total land area 31,396 m^2 (shown in Figure 1).

Figure 1 Proposed Development Site

There is an existing NB200mm diameter water main in the verge of McDonnell Road as it runs adjacent to the development site. There is also an OD125mm diameter water main at the current end of Brodie Avenue adjacent to the site. It is proposed that the new, yet to be designed internal reticulation, will connect to these existing watermains.

Figures 2 shows the development site and a proposed water supply reticulation connection points.

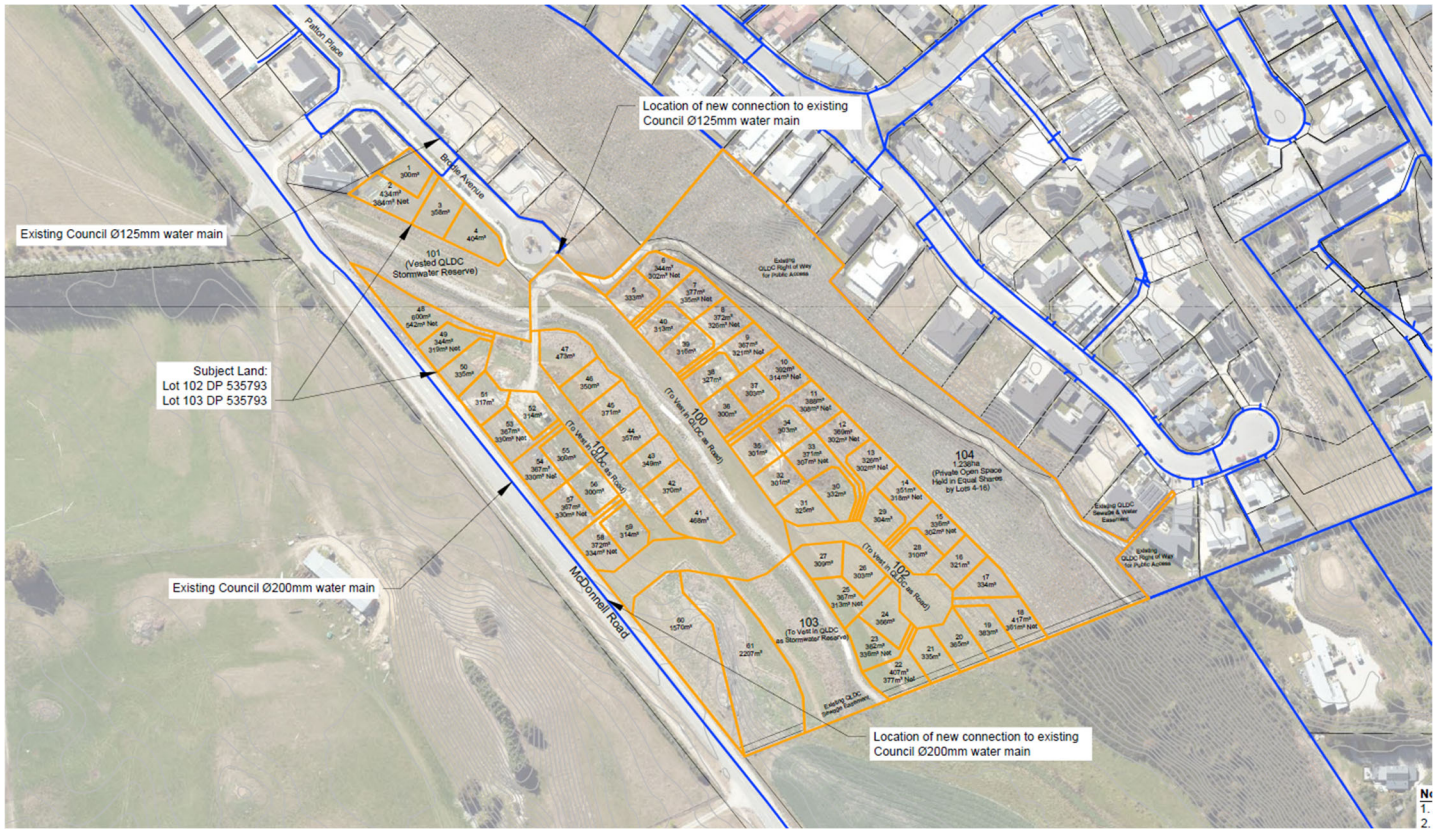

Figure 2 Development Proposed Water Supply Connection

1 EXISTING INFRASTRUCTURE AND SYSTEM PERFORMANCE

1.1 NETWORK DESCRIPTION

Arrowtown is located 20 km North-East of Queenstown. The network covers an area of approximately 480 Ha and services approximately 2,814 (Census 2018) inhabitants as well as the Millbrook Resort.

Arrowtown water supply network is supplied from two bores located in the Bush Creek Reserve. Water is conveyed to three hydraulically linked reservoirs via two rising mains. The reservoirs, maintained and operated by Veolia Water, are located at 23 Durham Street behind the cemetery. The total storage capacity is approximately 1,450m^3 at an elevation of 457.3m (TWL ~462.3m).

Under normal circumstances, water is supplied to Arrowtown and Millbrook by gravity. During peak summer demand, a pump station at the reservoirs operates to maintain sufficient pressure in the network to supply high elevation areas at the south end of the township.

1.2 HYDRAULIC MODEL

The hydraulic model of Arrowtown was built and calibrated in 2016 by Mott MacDonald and has not been updated since this time. The base model used for this analysis is the 2016 Peak Day model incorporating the Lifestyle Retirement Village on Lady Fayre Drive off McDonnell Road. The EPANet model scenario has been imported into Infoworks WS pro for ease of use, and validated to ensure results match the original EPANet model, however no further updates have been undertaken.

A review of the GIS suggests the following areas are not included in the existing model:

- Patton Place and Brodi Avenue
- Flynn Lane & O'Fee Way
- Extension of the Millbrook Scheme

Infrastructure for Patton Place, Brodi Avenue, and Flynn Lane and O'Fee Way have been updated in the model for this assessment. No infrastructure changes have been made to the Millbrook scheme.

The demand in the 2016 peak day base scenario is 61.61 L/s (or 5,323 m^3/day) which included the design demands for the retirement village on McDonnell Road as a proposed development.

SCADA for the 2024/2025 summer period has been reviewed to validate the current peak day. This data indicates highest demand day was 66.15L/s and the highest rolling 3-day average was 64.13L/s. The demand in the model has been increased by the additional properties in Patton Place, Brodie Avenue, Flynn Lane and O'Fee Way and scaling the base demands to meet the 3-day peak.

QLDC are likely to rebuild and recalibrate the model in the future to improve confidence in the outputs.

1.3 EXISTING SYSTEM PERFORMANCE

The performance of the existing network has been assessed with the intention of providing a baseline for the analysis of the proposed subdivision.

1.3.1 Supply and Treatment

As outlined in the QLDC Water Demand Management Plan (WDMP), 2025, Arrowtown's water supply experiences high per-connection consumption, with high irrigation demand, visitor-driven seasonal peaks, and increasing usage from Millbrook resort. Production is currently under strain.

The WDMP outlines peak-day pumping breaches sustainable intake capacity by 2027, and sustainable treatment limits (16-hour operation) are currently being breached during peak periods. The maximum flowrate for the UV system is 70L/s.

The hydraulic model indicates the bores are unable to supply the current peak day demands over a sustained period and the reservoirs slowly drain.

1.3.2 Reservoir Storage

There is a shortfall in reservoir storage which typical aims to meet 24 hours of average day demand or 12 hours of peak day demand. The existing storage capacity is approximately 1450m^3, compared with 12 hours peak day demand at 2,662m^3.

1.3.3 Reticulation Network

Figure 3 shows the minimum pressures and maximum pipe head losses for the scenario.

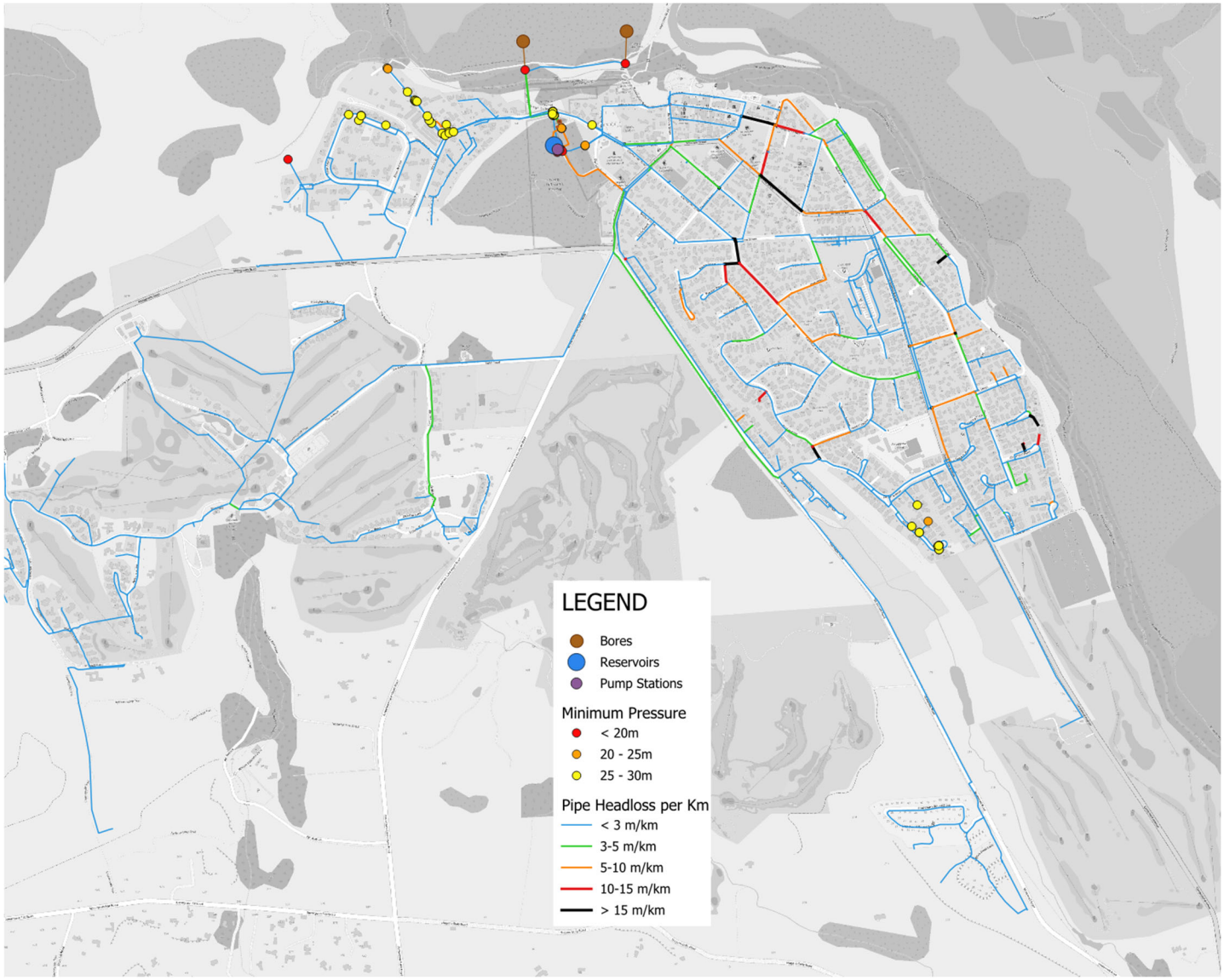

Figure 3 System Performance Pre-development (Minimum Pressure /Maximum Pipe Head Loss/km)

The existing Arrowtown system performance was run with the booster pumps on during peak demand. The network operates well with pressure generally within the QLDC levels of service. Lower pressures are shown at higher elevations in the southern area on Cotter Avenue and Advance Terrace, and in the northern area, on Bush Creek Road however pressure is maintained above 200kPa.

There is a closed valve in the model at the corner of Cotter Avenue and Centennial Avenue. It is assumed this valve has been closed to retain pressure in the Advance Terrace area. Opening the valve reduces pressures. However, this indicates pipe head losses in the network are impacting supply to the southern area.

High pipe head losses are shown in the watermains NB100mm diameter watermains on Bucking Street, Hertford Street and Sussex Street, and in the 150mm watermain on Adamson Drive.

In general, the network meets the residential FW2 fire risk classification of 25L/s, with the exception of the Butel Park area.

2 DEVELOPMENT DEMAND ASSESSMENT

The design parameters outlined in Queenstown Lakes District Council Land Development and Subdivision Code of Practice (2025) are as follows:

- Daily consumption of 700 L/person/day,
- Number of people per dwelling = 3,
- Peak hour factor of up to 4.0 (Queenstown), 6.6 (Rest of District),
- Firefighting demands as specified in SNZ PAS 4509,

The firefighting classification for residential development will be assessed as FW2 25L/s.

Table 1 shows the demand calculations for the development. A total peak hour factor of 6.6 has been used for residential demands, assuming an average to peak day factor of 2.30 and a peak hour factor of 2.87 based on the Arrowtown South diurnal demand profile used in the hydraulic model.

Table 1 McDonnell Road Development Demand Calculations

Development	Number of Residential Lots / Area Ha	Population	Average Demand (L/s)	Peak Demand (L/s)	Instantaneous Peak Demand (L/s)
McDonnell Road Development	61	183	1.48	3.41	9.79
Total	61	183	1.48	3.41	9.79

3 LEVELS OF SERVICE

The key design parameters outlined in Queenstown Lakes District Council Land Development and Subdivision Code of Practice (2025) are as follows:

- The design pressure shall be between 300 kPa and 750 kPa (30 m to 750 m).
- The head loss through pipe and fittings at the design flow rate shall be less than:
    - 5 m/km for DN ≤150;
    - 3 m/km for DN ≥200.

For the system performance assessment of the wider network using the hydraulic model, the levels of service previously agreed upon with QLDC are outlined below:

- The minimum service pressure is 200-300kpa
- The maximum service pressure is 750-900kpa

These levels of service along with the requirements of the Fire Fighting Water Supplies Code of Practice form the basis for the system performance analysis.

For firefighting, QLDC provides a minimum level of service of 25 L/s at 100 kPa within its water supply network. In commercial and industrial zones where there is currently 50 L/s at 100kPA, QLDC will maintain this higher level of service.

4 DEVELOPMENT ASSESSMENT

The existing system performance indicates the bore supply and treatment capacity are unable to keep up with peak demands over a sustained period, and there is insufficient reservoir storage to support the existing level of demand.

To undertake a hydraulic assessment of the reticulation network and assess the impacts of the proposed development, the reservoir has been model as a fixed head water source (unlimited source) at a hydraulic grade equivalent to 75% full (HGL 461.05m).

An assessment has been undertaken of the 61 Lot development on McDonnell Road, Arrowtown. The development proposes to connect to the existing NB200mm watermain on McDonnell Road linking through to the existing NB100mm watermain on Brodie Avenue.

The calculated demand has been applied to the current peak day scenario. The approved Tewa Banks residential development on Jopp Street has been included in the model based on a per capita demand of 1000L/connection, noting this development was previously assessed using full code of practice demands. No other proposed developments have been included.

Table 2 provides model result pre and post development.

Table 2 Results pre and post development

Demand Scenario	Pre-Development	Post-Development	Difference
Minimum Pressure (m)			
At hydrant 328887 (Brodie Avenue) RL. 391.59m	64.16 m	61.73 m	-2.43
At hydrant 113849 (Advance Terrace) RL 429.06 m	26.51 m	24.29 m	-2.22
At hydrant 113835 (Shanahan Lane / Devon Street) RL 404.60m	47.47 m	42.22 m	-5.25
Available Fire Flow (L/s)			
At hydrant 328887 (Brodie Avenue)	44.58 L/s	70.46 L/s	25.88
At hydrant 113849 (Advance Terrace)	22.19 L/s	21.63 L/s	-0.56
At hydrant 113835 (Shanahan Lane / Devon Street)	40.95 L/s	39.88 L/s	-1.07
Maximum Pipe Flow (L/s) / Head loss (HL/km)			
192484 (NB200mm on McDonnell Road)	23.3 L/s	30.7 L/s	7.4
	4.45 m/km	7.45 m/km	3
145812_1 (NB150mm on Adamson Drive)	31.2 L/s	36.34 L/s	5.14
	24.16 m/km (Velocity 1.95 m/s)	32.16 m/km (Velocity 2.27 m/s)	8

The assessment shows a drop in the minimum pressure of ~2.4m in the existing Brodie Avenue watermain resulting from the development. In the higher elevation area of Cotter Avenue and Advance Terrace, pressure is reduced by 2.2m. Given the existing lower pressures, this is potentially significant for these customers.

While the supply pressure for the subdivision remains above 30m (300kPa) additional pressure on the system increases the extent of the areas already below 30m.

Figure 4 shows the minimum pressures and maximum pipe head losses for the post development scenario. The area of low pressure on Advance Avenue between 20-25m (orange) is extended, high head loss watermains above 10m/km (red) and 15m/km (black) also increase.

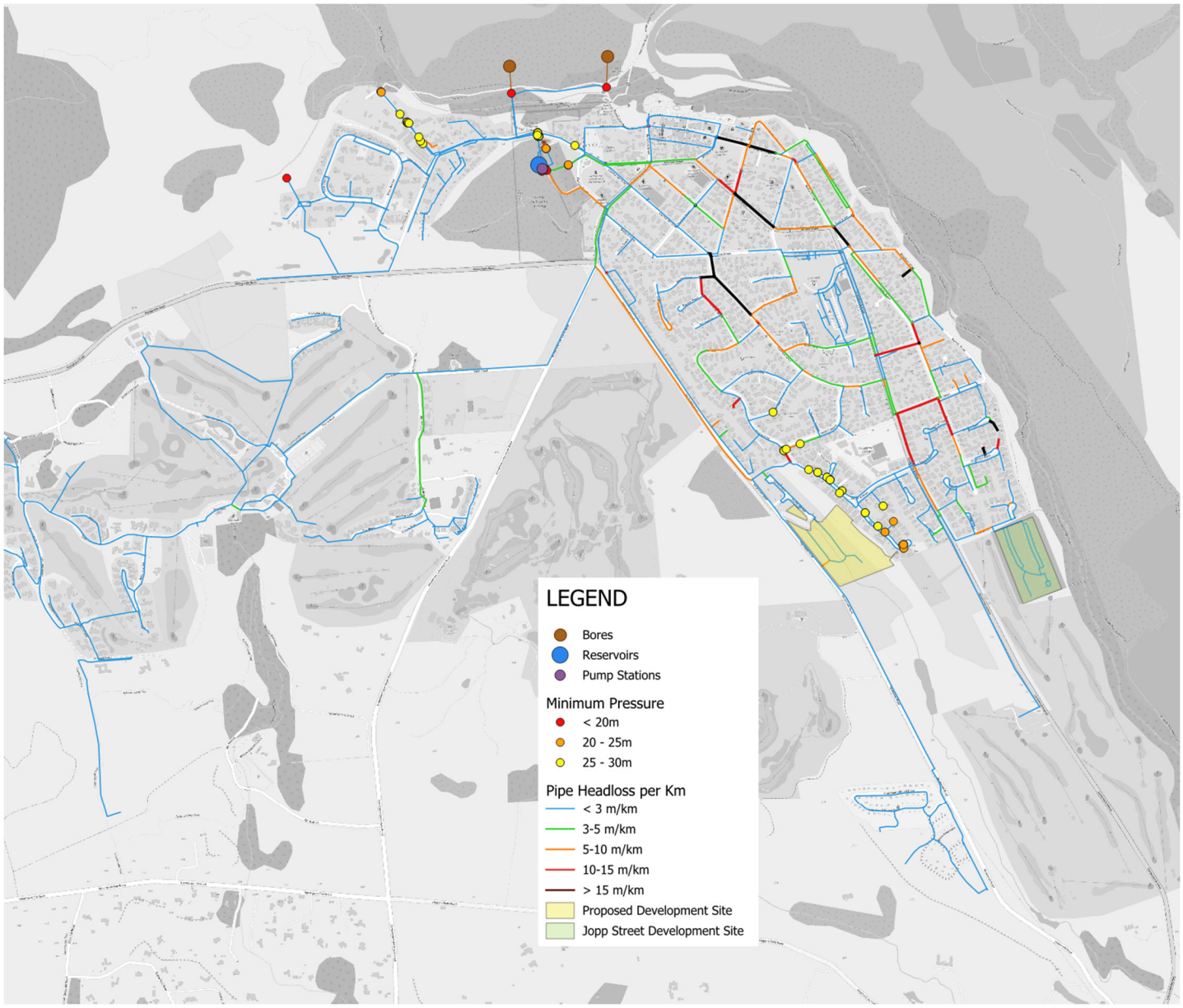

Figure 4 System Performance Post-development (Minimum Pressure /Maximum Pipe Head Loss/km)

The single hydrant available fire flow has also been assessed, and within the subdivision flows range from 55-75L/s, with flows improved to Brodie Avenue with the new connection. Flows in Advance Terrace are slightly reduced but should still achieve the FW2 residential classification of 25L/s from two hydrants.

5 SUMMARY

Hydraulic modelling has been undertaken to assess the ability to supply water to the proposed 61 lot residential development at 8 Brodie Avenue, adjacent to McDonnell Road.

The existing system performance indicates the bore supply and treatment capacity for Arrowtown are unable to keep up with peak demands over a sustained period, and there is insufficient reservoir storage to support the existing level of demand. It is important for QLDC to ensure the best outcomes for the community and future property owners, and to ensure the delivery of safe reliable drinking water. Issues relating to supply, treatment, and reservoir storage need to be addressed.

For the proposed development, pressures are within levels of service; however, pressures are impacted in the wider network and the extent of areas with lower pressures is increased. Lower pressures are a result of pipe head loss, and some pipes within the network have a head loss per unit distance greater than 20m/km which is excessive. While these issues are existing, additional development degrades the network performance.

It is noted, overall, pressures are within minimum level of service of 200kPa. The FW2 fire risk classification can be met within the development and in the higher elevation areas on Advance Terrace.

It is strongly recommended consideration is given to upgrades to improve supply to the southern areas of Arrowtown and reduce high head losses and velocities in the network under peak flow conditions. This recommendation was also made as part of the Tewa Banks residential development assessment on Jopp Street.

The hydraulic model is a representation of the physical water supply system and as such likely has some limitations. Watershed Engineering Limited were not involved in the original development of the Arrowtown hydraulic model. The base model scenario has been imported into Infoworks WS pro for ease of use with only the updates noted in this assessment.

As for all water supply network models, the demands and peaking factors used to assess the development are based on assumptions and the actual final water demands may vary.

We trust this report meets your requirements. Please contact us if you wish to discuss any aspects of this report further.

Kind regards,

Charlotte Broadbent

Charlotte Broadbent

Appendix C

Wastewater Modelling Report

- By HAL Consulting Ltd

QUEENSTOWN LAKES DISTRICT COUNCIL

MCDONNELL ROAD RESIDENTIAL DEVELOPMENT IMPACT ASSESSMENT

SEPTEMBER 2025

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Document Title	McDonnell Road Residential Development Impact Assessment
HAL Job Reference	J0821

Revision Schedule

Rev	Date	Description	Prepared by	Checked by	Reviewed by	Approved by
1	24/9/2025	Draft for client comment	RZ / BY	BDR	BDR	BDR

Contents

1 Introduction

1.1 Objective

The objective of this study is to use the existing hydraulic model (Wakatipu Wastewater Calibrated Model 2024) to assess the impact of the proposed McDonnell Road residential development on the Queenstown wastewater network. The current population (2021) scenario has been used.

1.2 Background

In 2024, Queenstown Lakes District Council (QLDC) engaged HAL Consulting to update and calibrate the existing Wakatipu wastewater model. The model was updated with the 2021 population and calibrated to flow survey data that was captured in 2020/21. The Wakatipu wastewater network consists of approximately ~231 km of wastewater pipe, 5106 manholes, and 39 pump stations. The network services approximately 18,200 rating units and conveys flows to the Shotover Wastewater Treatment Plant (WWTP) located towards the centre of the Wakatipu wastewater catchment.

The proposed McDonnell Road development is located towards the north-east of the Wakatipu catchment in Arrowtown. The proposal seeks to develop an existing greenfield site into 61 residential lots. The development is proposed to connect to the existing wastewater network either directly into the McDonnell Road Wastewater Pump Station located north-west of the site, or the last manhole prior to the pump station (COMPKEY 101471). From the McDonnell Road WWPS, flows are conveyed east to the Norfolk Street PS where flows are eventually conveyed to the Arrowtown-Lake Hayes PS that pumps to the Shotover WWTP.

2 Scope

The following tasks have been undertaken as part of this assessment to confirm if the network will have sufficient capacity in the existing network after the proposed development occurs:

Data Collection & Review

- A review of new development design flows based on QLDC CoP demand estimates.
- A review of the calculated new development design flows provided by the developer.

Modelling Assessment

- Assessment of modelled flows and overflows with and without proposed development (in conjunction with neighbouring developments which have been completed/approved).
- Assessment of cumulative impact of modelled flows with proposed development in conjunction with full build out of the neighbouring Arrowtown Lifestyle Villas development (understood to be approved)
- Comparison of modelled flows with theoretical (CoP) flows in the downstream network
- Review of reported overflows (as a result of wet weather) to corroborate modelled overflows, or to identify overflow locations not predicted by the model.
- Assessment of increased overflows with new development (if any).

- High Level Recommendations for future upgrades to mitigate increased overflow with new development (if overflows increased)

Reporting

- Concise Report to include:
    - Attached development plans as provided
    - Other known developments included within the assessment
    - Modelled Pipeline Long Sections with modelled maximum water level
    - Attached network screenshots
    - Pump station inflow/outflow/overflow graphs
    - Indicative network upgrades

Each of these tasks is discussed with more detail in the following sections.

3 Model Assumptions and Limitations

The model assumptions should be read in conjunction with the following reports.

- 'Wakatipu Wastewater Model Build & Calibration Report' (Beca, August 2016)
- 'Wakatipu Wastewater Network Future System Performance Report' (Beca, August 2017)
- 'Wakatipu Wastewater Model Review & Update – High & Medium Priority Fixes Memo' (HAL, 2018)
- 'Wakatipu Model Build and Calibration Memo' (HAL, July 2025).

The following limitations apply to the modelling undertaken as part of these studies:

- It should be noted that much of the asset level data in QLDC's GIS was anomalous and has been inferred as part of the recent model update (HAL, 2025) where necessary. Manhole and pipe level data has not been validated against as-builts or survey data as part of this assessment, or as part of the model update. Where potential network constraints are identified, it is recommended asset data in these areas is confirmed through manhole survey.
- The model was recalibrated against flows developed from a flow survey undertaken in 2020/21 and has resulted in a higher degree of confidence in the accuracy of the model than the previously calibrated model, but there are some areas of the network where some anomalies have been identified and further investigations recommended.
- The distribution of the modelled population is an approximation based on the 2021 Annex A population projections provided by QLDC, factored up for a high occupancy scenario based on peaking factors observed during the flow gauging period. It should be noted that the flow gauging took placed during periods affected by various stages of COVID lockdowns and travel restrictions, so may not necessarily reflect a normal peak holiday period. No allowance has been made for additional growth since 2021, other than known development areas.
- Pump station model parameters have been determined based on information provided by the QLDC planning team, SCADA data (where available) and pump station manuals, and the accuracy has not been validated as part of these studies (other than the pump station drawdown tests mentioned).
- This assessment focuses on the wastewater network downstream of the site, and does not consider sizing of infrastructure within the proposed site to service future development upstream of the site.
- It has been assumed that no existing overarching structure plan has been developed by QLDC for servicing this area.
- The impact of expected flows on the WWTP has not been considered as part of this assessment.

4 McDonnell Road Development Design Flows

4.1 Overview

The McDonnell Road proposal seeks to develop an existing greenfield site into 61 residential units. The development is proposed to connect to the existing wastewater network either directly into the McDonnell Road Wastewater Pump Station located north-west of the site, or the last manhole prior to the pump station (COMPKEY 101471) shown in Figure 1 below.

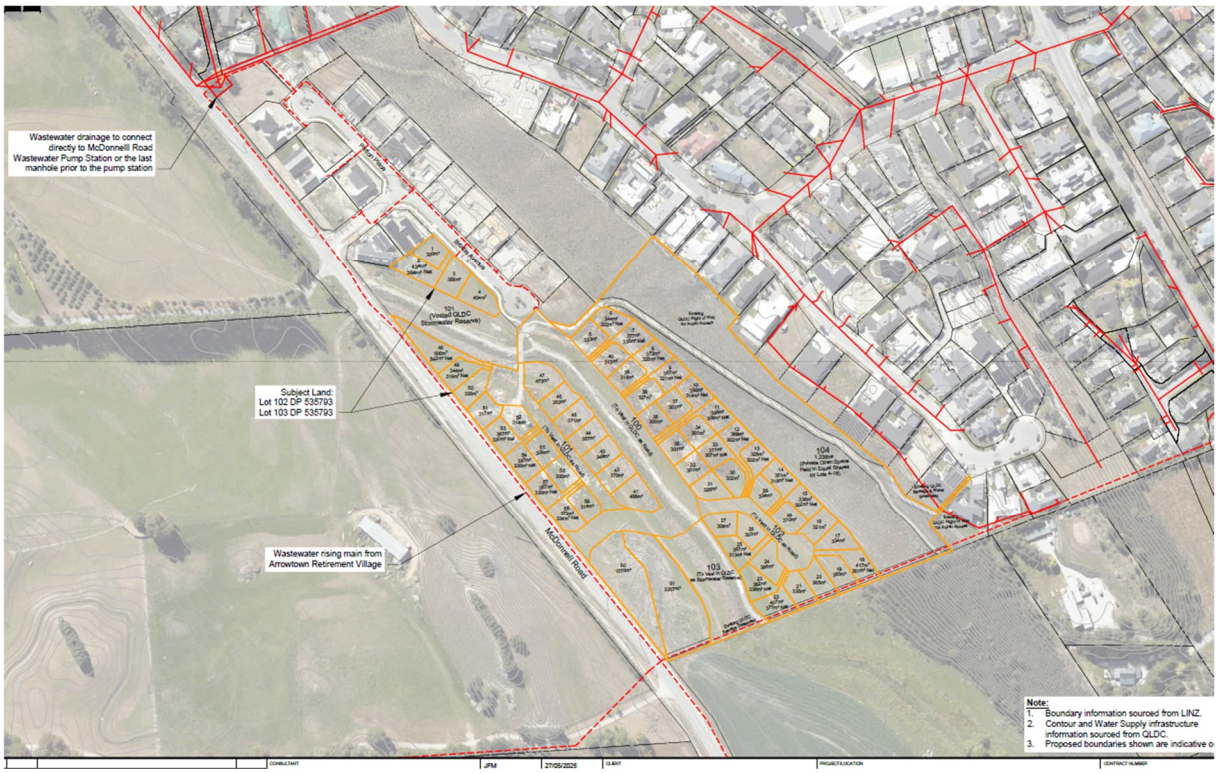

Figure 1: McDonnell Road development plans showing proposed wastewater network connections.

From the McDonnell Road WWPS (which has a capacity of 14 l/s based on a recent pump station drawdown test), flows are pumped to a gravity network to the east, where they are conveyed via gravity to the Norfolk Street PS. The Norfolk Street PS conveys flows south-west towards the Arrowtown-Lake Hayes PS, which eventually pumps to the Shotover WWTP. The development location is shown in Figure 2.

The McDonnell Road PS receives flows from three other known developments:

- The Arrowfield development located at Patten Place/Brodie Avenue. This is serviced by a low-pressure system (LPS) with an estimated flow of 1.4 l/s.
- The Arrowtown Lifestyle Village located south of the proposed development. This development site is currently under construction, with Stage 1 (~60 lots) complete at the time of writing this report. The total approved site includes 120 villas, 75 apartments, a 100-bed care home, a community centre, and associated facilities and amenities. The site is serviced by a pump station with a dual pump capacity of 8 l/s (based on a recent pump station drawdown test).
- 508A Arrow Lake Hayes Road, which is a recently consented residential development for 8 lots. (PWWF of ~ 0.4 l/s).

The Arrowfield development, Arrowtown Lifestyle Village (Stage 1), and 508A Lake Hayes Road development have been included in the base (pre-development) scenario for this assessment. A cumulative impact assessment of the final completed Arrowtown Lifestyle Village (~195 units plus a 100 bed care home and associated facilities) has also been included as a future (post-development scenario).

Figure 2: McDonnell Road development location and nearby developments

The McDonnell Road development is proposed to be internally serviced by a LPS due to anticipated poor ground conditions and problematic gravity drainage posing a high risk to existing infrastructure (as advised by the developer based on ground conditions for the neighbouring Arrowfield development). However, it is preferred by QLDC that a gravity system is used for the McDonnell Road development site unless proven unfeasible with sufficient supporting information.

4.2 Development Design Flows

The proposal at McDonnell Road seeks to develop 61 residential units. This results in a PWWF of 2.65 l/s for a traditional gravity system using the QLDC Land Development and Subdivision Code of Practice which assumes an occupancy of 3 people per dwelling, an ADWF of 250 l/p/day, a dry weather diurnal peaking factor of 2.5, and a wet weather dilution/infiltration factor of 2 (i.e. a WWF peaking factor of 2.5 x 2 = 5). The calculations are shown in Table 1 overleaf.

The development proposes to be serviced by a LPS due to potential problems anticipated servicing the development by a gravity system (as were experienced with the adjacent Patten Pl development), with a maximum instantaneous pump rate of 5.3 l/s based on the simultaneous operation of 7 grinder pumps. We note that this is significantly higher than the calculated theoretical flows for a conventional gravity

system as per the QLDC Code of Practice. Whilst it is preferred by QLDC that a gravity system is used unless proven unfeasible, QLDC have indicated that a LPS may be considered for this development due to the poor ground conditions identified in the vicinity of the McDonnel Rd pump station for the adjacent Patten Rd development. However the design flow of 5.3 l/s provided by the developer for the proposed LPS appears high for a LPS of this scale, which would normally be expected to be less than design flows for a traditional gravity system, so it is recommended this is reviewed. For this assessment, the theoretical PWWF of 2.65 l/s based on a gravity system will be used as the basis of this assessment.

Table 1: McDonnell Road development design flows

No of units	Type of units	Occupancy	Population	ADWF (l/p/day)	ADWF (l/s)	DWF Peaking Factor	PDWF (l/s)	WWF Peaking Factor	PWWF (l/s)
61	Residential Units	3	183	250	0.53	2.5	1.32	5	2.65
	Total Residential Flow		**183**						**2.65**

5 McDonnell Road Development Impact Assessment

The existing Wakatipu Wastewater Calibrated Model (2024) was used to determine the impact of the development on the current network. The following scenarios were assessed as part of the development impact assessment:

- A base pre-development scenario, which includes flows from the completed/committed Arrowfield development (20 lots), Arrowtown Lifestyle Village Stage 1 (60 lots), and 508A Lake Hayes Road development (8 lots).
- A post-development scenario (Scenario 1) – as above, with the addition of flows from the proposed McDonnell Road development (2.7 l/s).
- A cumulative post-development scenario (Scenario 2) – as above, with the addition of the full completed scale of the Arrowtown Lifestyle Village (a total of 120 villas, 75 apartments, 100-bed care home, a community centre, and associated facilities and amenities resulting in a PWWF of approximately 7 l/s).

For this assessment, due to the constraint identified at the McDonnel Rd WWPS, and the important role storage plays in mitigating overflows, flows from these developments have been represented dynamically (as opposed to constant flows as would normally be assumed) so as not to overestimate volumes, with parameters adopted to generate a peak flow in a 5 year design storm approximating the peak wet weather flows determined from the Code of Practice.

5.1 Pre-Development Scenario

The Wakatipu wastewater calibrated model (2024) was run for a 24-hour, 5-year ARI design storm to assess how the local network performs during storms of this magnitude, without the proposed development. The pre-development scenario includes flows from the nearby Arrowfield development, Stage 1 of the Arrowtown Lifestyle Village, and a recently consented development at 508A Lake Hayes Road.

Figure 3 shows a long section of the pipe preceding the McDonnell Road PS, downstream of the proposed McDonnell Road development (MH COMPKEY: 101471). The available storage at the PS is included in the long section, represented by the downstream conduit (Link ID: SPMR_ST01.1). The long section shows there is significant surcharge simulated in the pipe preceding the McDonnell Road PS. This appears to be a result of backwater from the pump station, as the modelled flow does not exceed the theoretical pipe full capacity. The predicted surcharge is less than 500mm below the lid level, so the risk of uncontrolled overflows is considered high in this scenario. It is noted there is an off line storage tank at this pump station, but the storage volume is relatively small (~9 m3) so the impact on buffering high peak flows is relatively minor.

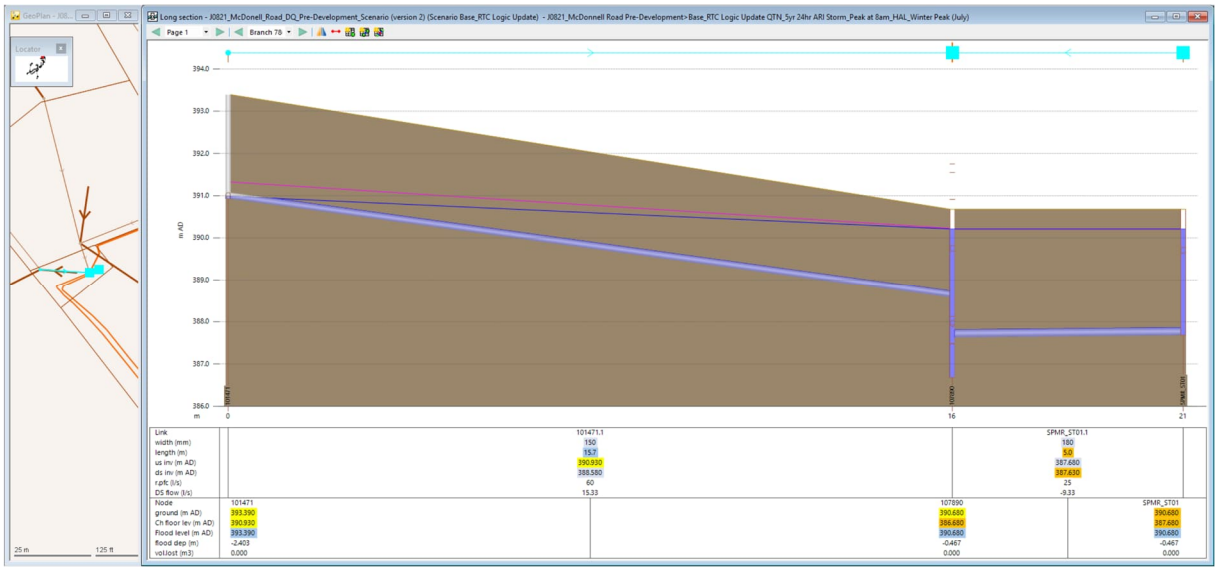

Figure 3: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Pre-development flows from MH COMPKEY: 107891 to McDonnell Road PS

5.2 Scenario 1: Post-development scenario with McDonnell Road development

The Wakatipu wastewater calibrated model (2024) was run for a 24-hour, 5-year ARI design storm to assess how the local network performs during storms of this magnitude in the post-development scenario, with the additional PWWF of ~2.7 l/s from the McDonnell Road development.

The McDonnell Road development is assumed to discharge at MH COMPKEY: 101471. A long section of the pipe downstream of the McDonnell Road development, preceding the McDonnell Road PS is shown in Figure 4 below. The available storage at the PS is included in the long section, represented by the downstream conduit (Link ID: SPMR_ST01.1).

The long section shows that the surcharge simulated in the pre-development scenario has been exacerbated in the post-development scenario. The modelled flows in the pipe still do not exceed the theoretical pipe full capacity, however an uncontrolled overflow of ~5 m^3 is predicted at the McDonnell Road PS (MH COMPKEY: 107890). Additionally, an overflow of ~4 m^3 is predicted at the McDonnell Road PS storage tank node, with the capacity of the storage tank exceeded for approximately 1 hour. This indicates the McDonnell Road PS (including the impact of storage) does not have sufficient capacity for the incoming flows and suggests an upgrade of the PS (or alternative solution) is required for the additional flows from the McDonnell Road development.

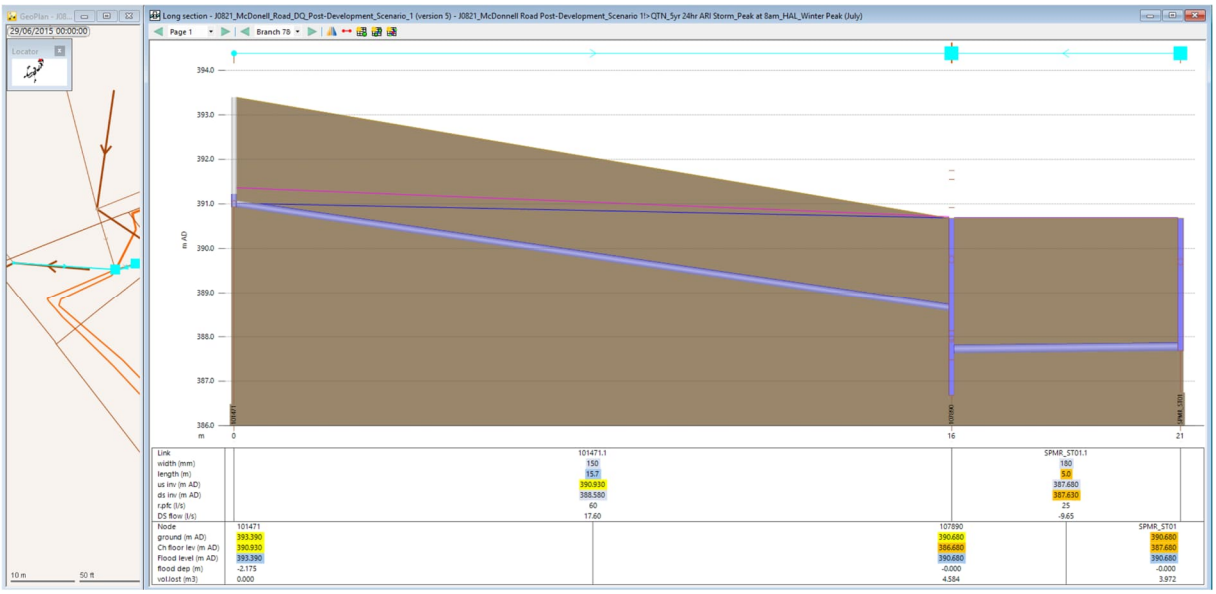

Figure 4: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Post-development flows from MH COMPKEY: 107891 to McDonnell Road PS – Scenario 1

5.3 Scenario 2: Post-development scenario with McDonnell Road development and the Arrowtown Lifestyle Village

The Wakatipu wastewater calibrated model (2024) was run for a 24-hour, 5-year ARI design storm to assess how the local network performs during storms of this magnitude in a cumulative post-development scenario, with the additional PWWF of ~2.7 l/s from the McDonnell Road development and the completed scale of the Arrowtown Lifestyle Village (PWWF of ~7 l/s). The Arrowtown Lifestyle Village is serviced by a pump station which connects directly into the McDonnell PS (MH COMPKEY: 107890).

Figure 5 shows a long section of the pipe preceding the McDonnell Road PS, downstream of the proposed McDonnell Road development (MH COMPKEY: 101471). The available storage at the PS is included in the long section, represented by the downstream conduit (Link ID: SPMR_ST01.1). The long section shows that the additional flows from the full completed scale of the Arrowtown Lifestyle Village exacerbates the predicted surcharge and overflows in the network. The volume of the predicted uncontrolled overflow at MH COMPKE7: 107890 increases from 5 m^3 to 18 m^3. Additionally, the predicted overflow at the McDonnell Road PS storage tank node increases from 4 m^3 to 15 m^3, with the capacity of the storage tank exceeded for approximately 2 hours.

As a result, upgrades to the McDonnell Road WWPS are required to accommodate the additional flows from the proposed McDonnell Road development and the fully completed scale of the Arrowtown Lifestyle Village.

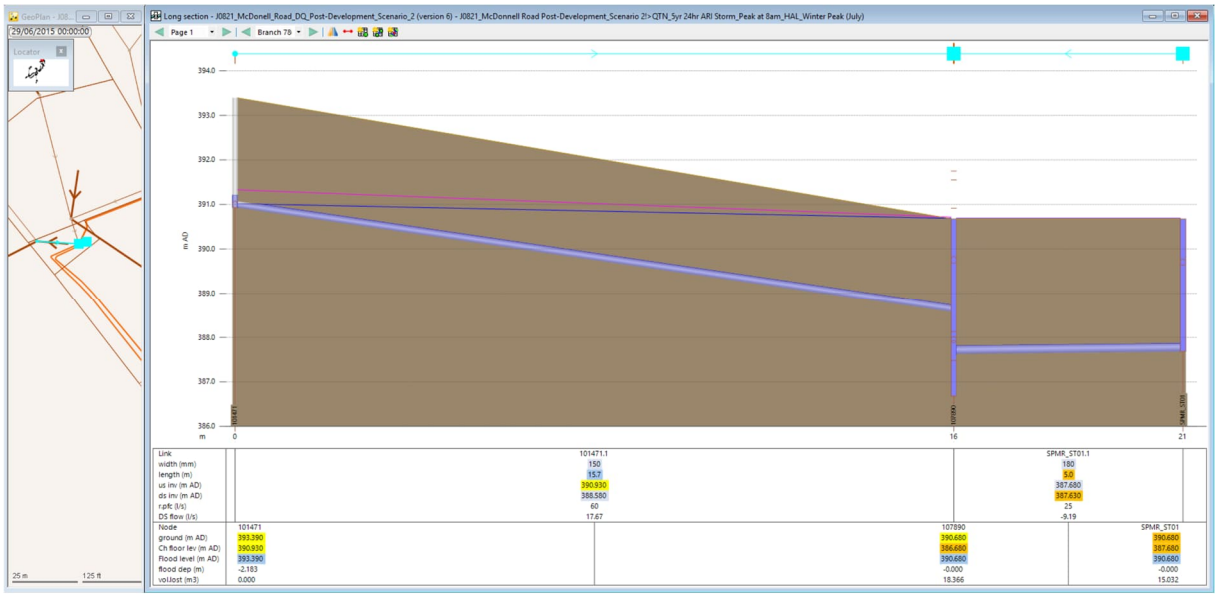

Figure 5: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Post-development flows from MH COMPKEY: 107891 to McDonnell Road PS – Scenario 2

5.4 Static Capacity Assessment

Based on the QLDC Land Development and Subdivision Code of Practice, a theoretical flow assessment was carried out for the catchment associated with the McDonnell Road WWPS. The theoretical flows were calculated using the 2021 modelled populations and an assumed residential land use based on the QLDC Proposed District Plan 2015. Based on the modelled data, the McDonnell Road WWPS has a combined maximum pump rate of 14 l/s, with 2 pumps operating as duty/assist (as determined through a pump station drawdown test).

Table 2 below compares the modelled and theoretical peak wet weather flows discharging to the McDonnell Road WWPS in the pre-development scenario, the post-development scenario including the McDonnell Road proposed development (Scenario 1), and the cumulative post-development scenario including the full-build of the Arrowtown Lifestyle Village (Scenario 2).

Table 2: Static capacity assessment of the McDonnell Road Pump Station

	Modelled Pump Rate (l/s)	Theoretical Incoming Peak Flow (l/s)	Modelled Incoming Peak Flow (l/s)
Pre-development Scenario	14	13.6	17.5
Post-development Scenario 1	14	16.2	25.9
Post-development Scenario 2	14	20.7	27.2

The comparison between the modelled and theoretical flows shows that:

- The McDonnell Road WWPS does not appear to have sufficient pass forward capacity for both the pre- and post-development peak modelled flows. The theoretical pre-development flows are predicted to come close to exceeding the modelled pump rate. The pre-development modelled flows are 1.25x the modelled pump rate, which explains the surcharging observed in the simulation prior to the inclusion of the post-development flows. The theoretical and modelled incoming peak flows exceed the pump station capacity in both post-development scenarios.
- The modelled flows (which are based on a calibrated model) are notably higher than the calculated theoretical flows, suggesting potential high inflow/infiltration in this catchment, however it should be noted that calibration was undertaken at a flow gauge some distance downstream, just upstream of the Norfolk St WWPS, with the same parameters assumed for the contributing catchment, which may not be realistic. As such, it is recommended QLDC undertake model validation against one or more significant wet weather events at the McDonnel Rd WWPS using SCADA data to improve the level of confidence in the modelled flows at this location.

5.5 Pump Station Assessment

An assessment of the McDonnell Road PS was completed to determine if there is sufficient capacity within the pump station for incoming flows in the pre- and post-development scenarios. Based on the modelled data, the McDonnell Road PS has a combined maximum pump rate of 14 l/s, with 2 pumps operating as duty/assist.

Figure 6 below shows the inflows into McDonnell Road PS in the pre- and post-development scenario, the pumped outflow in the post-development scenario, and the modelled pump capacity. As shown in Figure 6, the modelled pump capacity is exceeded in both the pre- and post-development scenarios. The pre-development scenario simulates a peak inflow rate of approximately 18 l/s in the 5-year ARI, 2-hr design storm, approximately 1.3x the modelled pump station capacity. In the post-development scenario 1, with the addition of the proposed McDonnell Road development flows (2.6 l/s), the peak inflow rate increases to 26 l/s, which is approximately 1.9x the modelled pump station capacity. In the post-development scenario 2, with the flows from the full build of the Arrowtown Lifestyle village (7 l/s), the peak inflow rate increases again to approximately 27 l/s.

Based on the modelled results, it is concluded that the McDonnell Road PS does not have sufficient capacity to accommodate the additional flows from the proposed development. It is recommended that the pumping capacity is upgraded to 26 l/s to match the expected flows in the post-development scenario with the McDonnell Road development (Scenario 1).

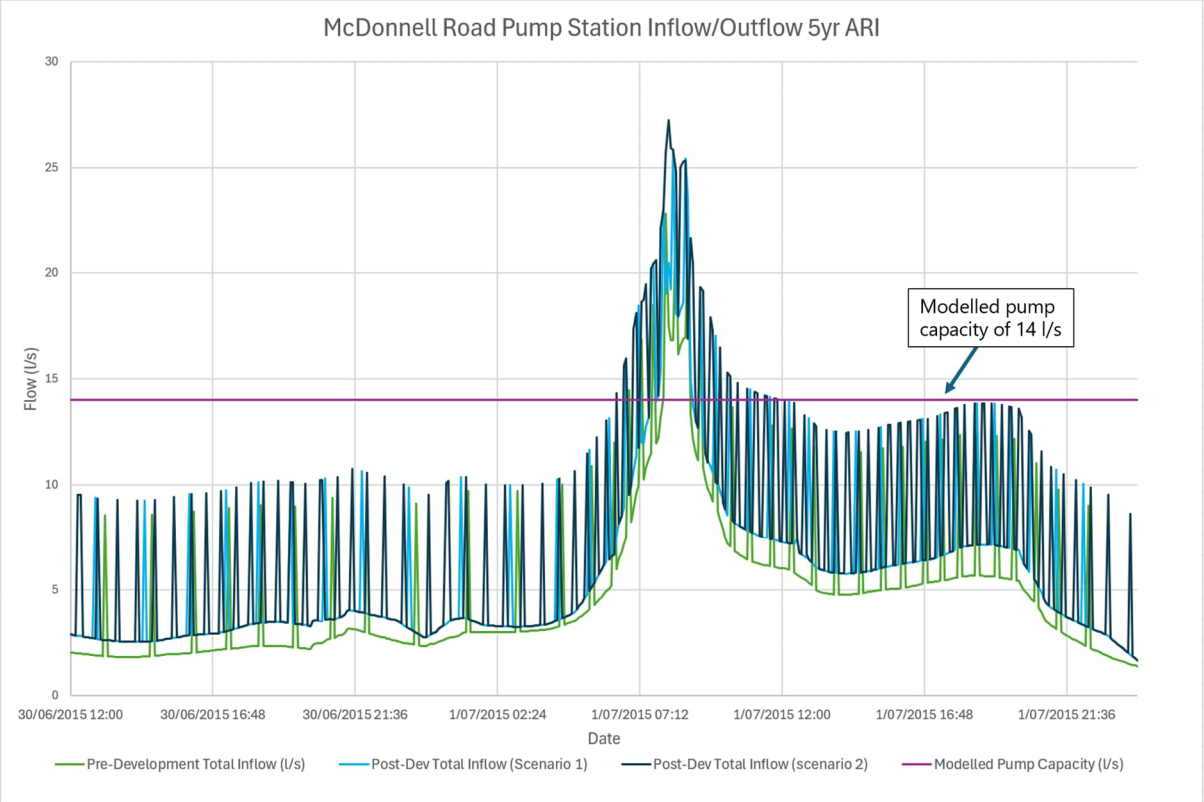

Figure 6: Pre- and post- development inflows at McDonnell Road PS

5.6 Proposed upgrades – McDonnell PS and downstream gravity network upgrade

An upgrade of the McDonnell PS from 14 l/s to 26 l/s was modelled to match the expected inflows in the post-development scenario with the additional flows from the McDonnell Road development. The Wakatipu wastewater model was run for a 24-hour, 5-year ARI design storm to assess how the local network performs in the cumulative post-development scenario incorporating the pump upgrades.

Figure 7 below shows the modelled flows from the assumed development discharge point (MH COMPKEY: 107891) to the McDonnell PS and storage. The upgrades of McDonnell PS address the issues of overflows at the PS storage which were previously modelled. However, Figure 8 below shows that the pump upgrades cause significant surcharge in the 150mm gravity network downstream of the McDonnell PS. The risk of uncontrolled overflows is considered high, with the water level within 500mm of the lid level at several manholes.

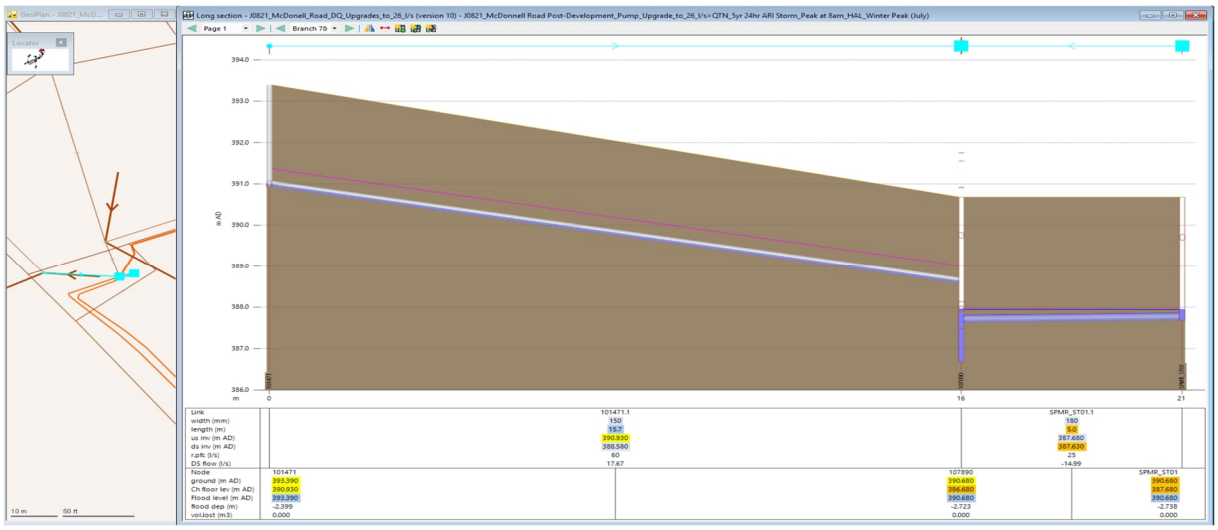

Figure 7: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Post-development flows from MH COMPKEY: 107891 to McDonnell Road PS – with proposed pump upgrades

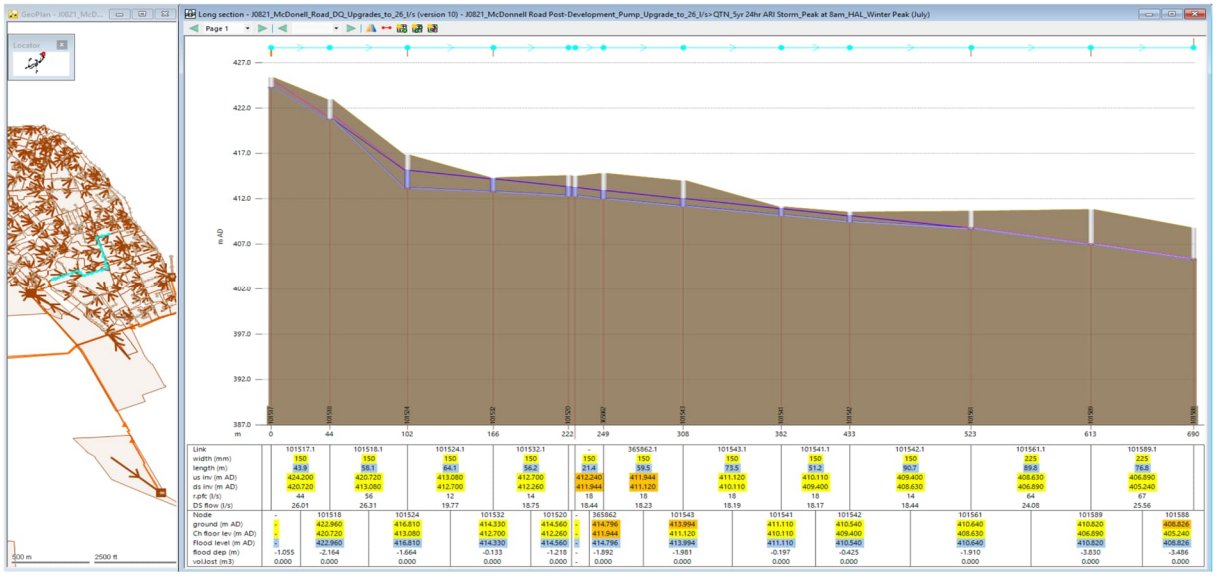

Figure 8: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Post-development flows from McDonnell Road PS to MH COMPKEY: 101588 – with proposed pump upgrades

Pipe upgrades are proposed in the downstream gravity network to address the predicted surcharge in the network as a result of an increase in pump capacity at McDonnell PS. An increase in diameter from 150mm to 225mm for the pipe network between MH COMPKEY: 101524 and MH COMPKEY: 101561 has been modelled (approximately 700m of pipe).

Figure 9 and Figure 10 shows a long section of the gravity network between the McDonnell PS and the next downstream pump station, Norfolk Street PS. The long section shows that the proposed pipe upgrades address the surcharge issues previously modelled, with no surcharge predicted in the downstream 150mm network. Further downstream, there is surcharge predicted in the 300mm network upstream of the Norfolk Street PS. However, this is not within predicted to reach within 500mm of modelled manhole lid levels, so the risk of uncontrolled overflows is considered moderate, but further investigation is recommended to better quantify this risk if these proposed upgrades are implemented.

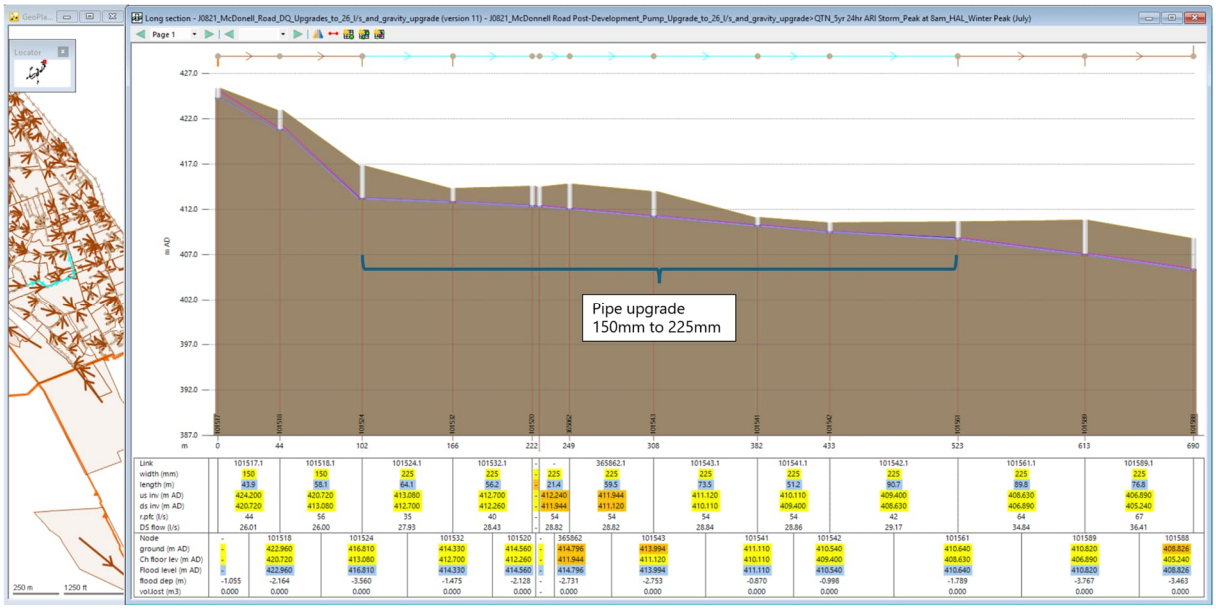

Figure 9: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Post-development flows with proposed pump upgrades and gravity network updates – Long Section 1

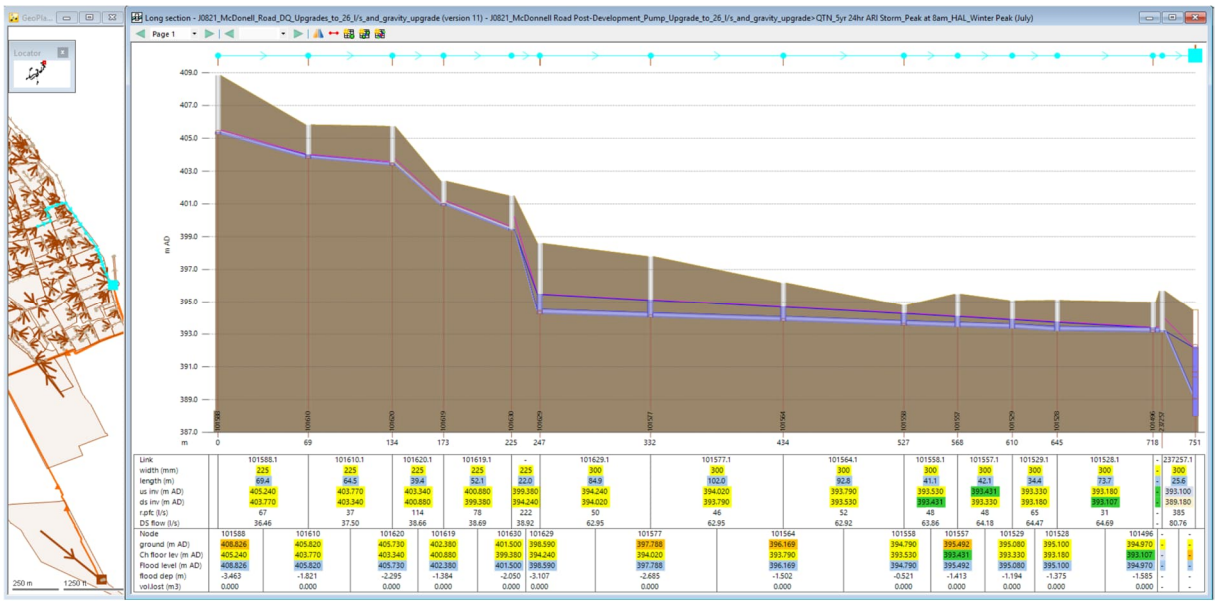

Figure 10: Wakatipu Wastewater Model (2021 population) 5-year ARI Design Storm - Post-development flows with

proposed pump upgrades and gravity network updates – Long Section 2

The modelled results show that the upgrade of pumping capacity at McDonnell PS appears to be feasible alongside an upgrade of the downstream gravity network from 150mm to 225mm (from MH COMPKEY: 101524 to MH COMPKEY: 101561). However, it is noted that this conveyance upgrade is the only option that has been considered as part of this development impact assessment and may not be the preferred option on a catchment-wide scale. It is recommended that QLDC undertake an options study in this area also considering additional future growth, including validation of existing flows against PS SCADA data at the McDonnell Road PS to improve confidence in modelled flows, and consider a wider range of potential options (such as buffer storage, PS operational controls, and potential diversions).

6 Conclusion

The objective of this study was to utilise the existing hydraulic model of the Wakatipu wastewater network (2024) to assess the impact of the proposed McDonnell Road development. The proposal seeks to develop an existing vacant site into a 61-unit residential development. A low-pressure system with a maximum total outflow of 5.3 l/s is proposed to service the development; however, it is understood that a low-pressure system is not the preferred solution by QLDC. Thus, the theoretical peak flow of 2.7 l/s from the proposed development if serviced by a gravity system has been used as the basis of this assessment.

The model was run for the existing (2021 population) scenario against a 24-hour, 5-year ARI design storm. The pre-development scenario includes the flows from the nearby Arrowfield development (approximately 1.4 l/s), Stage 1 of the Arrowtown Lifestyle Village (60 lots), and the recently consented development at 508A Arrow Lake Hayes Road. The pre-development scenario showed significant surcharge in the network upstream of the McDonnell Road PS. This appears to be a result of backwater from the pump station, as the modelled flow does not exceed the theoretical pipe full capacity.

The model was then run for the existing (2021 population) scenario with the additional PWWF of 2.7 l/s from the proposed McDonnell Road development. The surcharge modelled in the pre-development scenario was exacerbated, with overflows predicted at the McDonnell Road PS (overflow volume of 5 m^3) and storage tank (overflow volume of 4 m^3). Additionally, a cumulative post-development scenario was run which included the flows from the full completed scale of the Arrowtown Lifestyle Village. The cumulative post-development scenario showed the previously predicted surcharge, and overflows were exacerbated with the additional flows from the completed Arrowtown Lifestyle Village, with the predicted overflow volumes increasing to 18 m^3 at the McDonnell PS and 15.0 m^3 at the storage tank.

A pump station assessment of the McDonnell PS was completed to assess the performance of the pump station in the pre-development and post-development scenarios. Under the 24-hour, 5-year ARI design storm, the pump station capacity of 14 l/s was modelled to be exceeded in both scenarios, with the pre-development inflows modelled at approximately 18 l/s and the post-development inflows modelled at approximately 26 l/s and 27 l/s for the two post-development scenarios. It was concluded that the McDonnell PS does not have sufficient capacity for the additional flows of the McDonnell Road development and upgrades are required before the development can be approved.

A scenario was completed to assess the impact of a potential upgrade in the pumped capacity of McDonnell Road PS from 14 l/s to 26 l/s. The increase in pumped capacity was showed to resolve the surcharge and overflows modelled in the post-development scenarios. However, the increased pumped capacity of McDonnell Road PS triggered a need for upgrades in the downstream gravity network from 150mm to 225mm (from MH COMPKEY: 101524 to MH COMPKEY: 101561). With these upgrades, the downstream network appeared to have sufficient capacity for the additional flows. The impact of the increased flows from the McDonnell Road PS to the next downstream pump station (Norfolk Street PS) was considered. Minor surcharge was predicted upstream of the Norfolk Street PS, with the risk of uncontrolled overflows considered to be moderate. Further investigation is recommended to better quantify this risk if these upgrades are to be implemented.

Based on the modelled results, the upgrade of pumping capacity at McDonnell PS to 26 l/s appears to be feasible alongside an upgrade of the downstream gravity network from 150mm to 225mm (from MH COMPKEY: 101524 to MH COMPKEY: 101561). However, it is recommended that QLDC undertake additional model validation at the McDonnel Rd WWPS to improve confidence in modelled wet weather

flows, and undertake an options study in this area including future growth to consider a wider range of potential options (such as buffer storage, PS operational controls, and potential diversions) to determine the preferred option on a catchment-wide scale.

Appendix D

Stormwater Feasibility Report

- By Civilised Ltd

Issue 3
February 25, 2026

Arrow South Zone Change – Stormwater Feasibility Report

Prepared by: Civilised Ltd

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Arrow South Zone Change – Stormwater Feasibility Report

Report prepared For: Arrow South Ltd

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Report Reference: QS075
2026-02-25 Stormwater Feasibility.docx

Date: 25th February 2026

Issue	Details	Date
1	Draft for comment	21st November 2025
2	Updated following initial QLDC review	16th February 2026
3	Updated for the latest concept subdivision layout	25th February 2026

Executive Summary

Introduction: This report assesses the feasibility of providing stormwater attenuation for the proposed residential rezoning of the Arrow South Ltd site on McDonnell Road, Arrowtown.

Proposed Development: The rezoning proposal seeks to enable the creation of up to 59 new residential allotments on what is currently undeveloped land adjacent to the existing Arrowtown urban area. The area subject to the rezoning covers approximately 5.7 ha.

Key Challenge: The introduction of impervious surfaces associated with the development will result in increased runoff volumes and peak flow rates. This requires appropriate management to ensure that downstream effects on the receiving environment are no more than minor.

Proposed Solution: The proposed strategy utilizes **two dry attenuation basins** located adjacent to the internal watercourse. These basins will typically remain dry, holding water only during heavy or prolonged rainfall events. Outflow rates will be strictly controlled to remain below pre-development levels, effectively mitigating downstream risks.

Key Findings: The key findings of this report are:

- **Required Storage:** A total combined attenuation storage volume of approximately **1,400 m³** is required (split between two basins with volumes of 645 m³ and 753 m³ respectively).
- **Discharge Control:** The proposed design achieves hydraulic neutrality. Post-development peak flows are restricted to pre-development levels for both the **5% AEP (20-year)** and **1% AEP (100-year)** storm events.
- **Spatial Feasibility:** Confirmation that the site has sufficient area to accommodate the required basin footprints (approximately 400m² base area each).

Conclusion on Feasibility: The investigative and preliminary design work undertaken to date **shows** that it is technically and practically feasible to manage stormwater runoff from the site to prevent adverse downstream effects.

Primary Recommendations: Based on the findings of this assessment, it is recommended that:

1. The rezoning application proceeds, as the stormwater effects can be effectively managed within the site boundaries.
2. A site-specific **Geotechnical Investigation** be undertaken to confirm soil stability and groundwater levels for the basin embankments.
3. A detailed **Topographical Survey** of the watercourse and outfall locations be completed to finalize earthworks volumes.
4. The project proceeds to **Detailed Engineering Design** following a pre-application meeting with QLDC to confirm final design parameters.

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1 Introduction

1.1 Purpose of the Report

The purpose of this report is to evaluate the technical feasibility of constructing an on-site stormwater attenuation pond to manage runoff from the proposed residential development, ensuring post-development discharge rates do not exceed pre-development rates.

1.2 Project Background

It is proposed that the part of the area is rezoned to Suburban Residential Zone. The site is currently not zoned for development.

The number of allotments that the zoning will allow is unconfirmed at this stage due to the minimum lot size rules still being debated as part of the Queenstown Lakes District Council (QLDC) Proposed District Plan (PDP) implementation. Estimates for the number of allotments range from 44 to 59. For the evaluation of infrastructure, the highest estimated lot yield has been used for evaluating the stormwater runoff characteristics of the developed site.

1.3 Scope of Work

The tasks undertaken for this study include:

- Review of existing site conditions.
- Hydrological analysis for pre- and post-development scenarios.
- Conceptual sizing and siting of the attenuation pond.
- Assessment against regulatory requirements.
- Feasibility assessment and recommendations.

2 Site Description

2.1 Location and Legal Description

The site is located at McDonnell Road, Arrowtown. It is located east of McDonnell Road and is adjacent to a previous stage of development undertaken by the same developer in the area of Brodie Avenue and Patton Place.

The site is legally described as Lots 102 and 103 DP 535793.

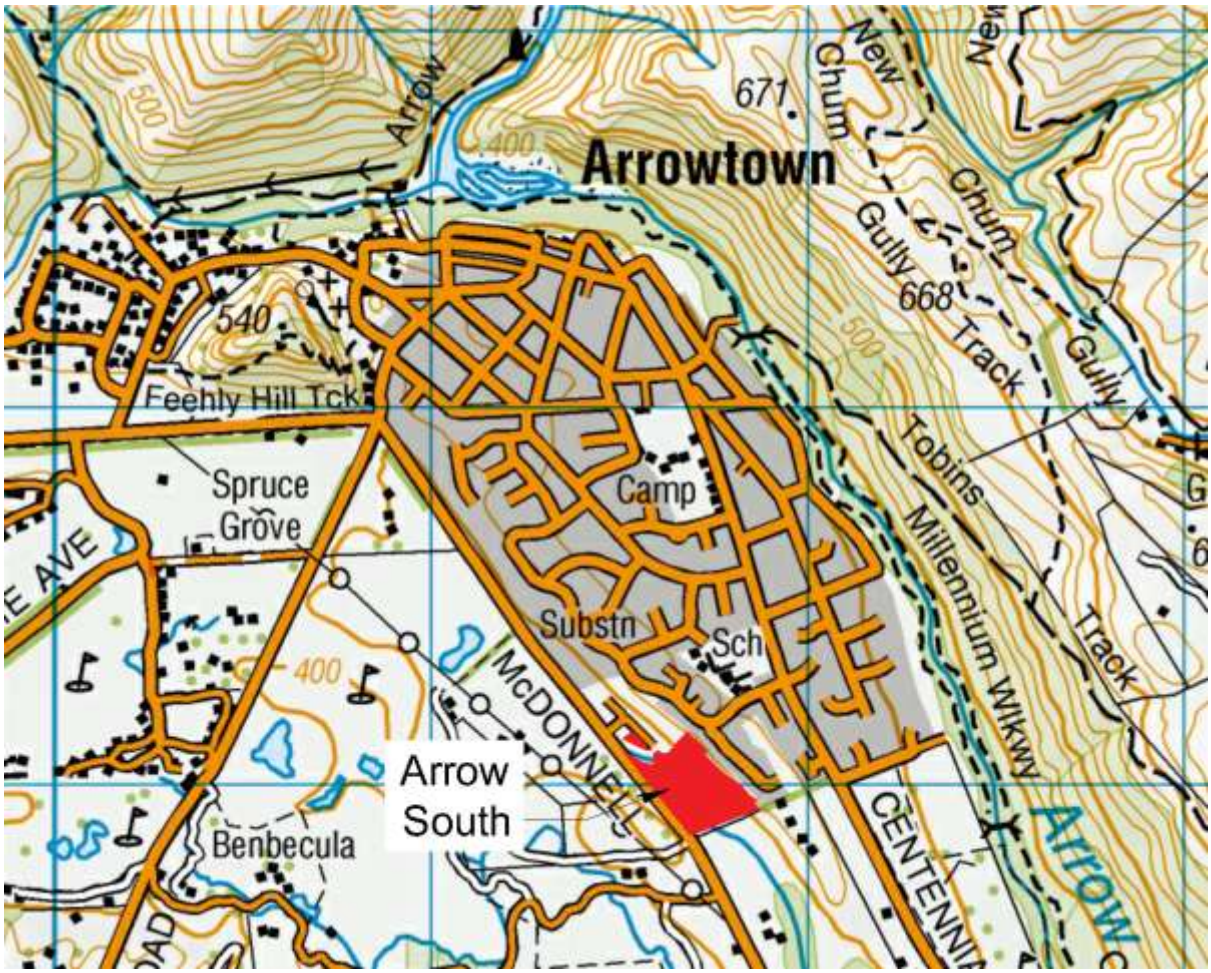

Figure 1 – Site Location Map

2.2 Existing Land Use and Topography

The land is currently vacant with some riparian and other plantings. There is an unnamed watercourse that runs through the land.

The land generally falls gently from the McDonnel Road frontage to the watercourse and then rises gently before gradually steepening towards Cotter Avenue and Advance Terrace properties that overlook the site.

Figure 2 – Overlooking the Site

2.3 Soil and Geological Conditions

The geotechnical evaluation for the site is based on a site-specific report by Geosolve Ltd and Landcare Research information from the S-MapOnline portal.

S-MapOnline indicates that soil units within the catchment of the creek running through the site are well-drained to moderately well-drained. However, the Geosolve Ltd report shows that soils within the site are actually not suited to soakage and possess poor drainage capacity.

Due to the presence of these poorly drained soils, runoff attenuation is the preferred management option. This ensures the development is undertaken sustainably without adversely affecting downstream properties.

The land is reasonably dry, with a median rainfall of 701–750 mm per annum.

2.4 Existing Drainage Patterns

Stormwater discharges from the site as sheet flow towards the internal watercourse, which flows from northeast to southwest.

The watercourse serves a catchment area that includes part of Feehly Hill, most of the McDonnell Road properties to the north, and significant areas to the west, including large parts of the Hills Golf Course. Flows from this catchment drain into the site via an incised channel.

There are upstream constraints that intermittently limit flow in the watercourse. These include various ponds within the golf course that buffer flows during periods of intense rainfall, and a culvert under McDonnell Road that restricts flow during heavy events. As noted previously, the first stage of the Arrow South development (near Brodie Avenue and Patton Place) also utilizes attenuation to reduce runoff entering the system.

There is no visible evidence of erosion caused by uncontrolled overland flows, nor are there any known flood records for the creek within the site.

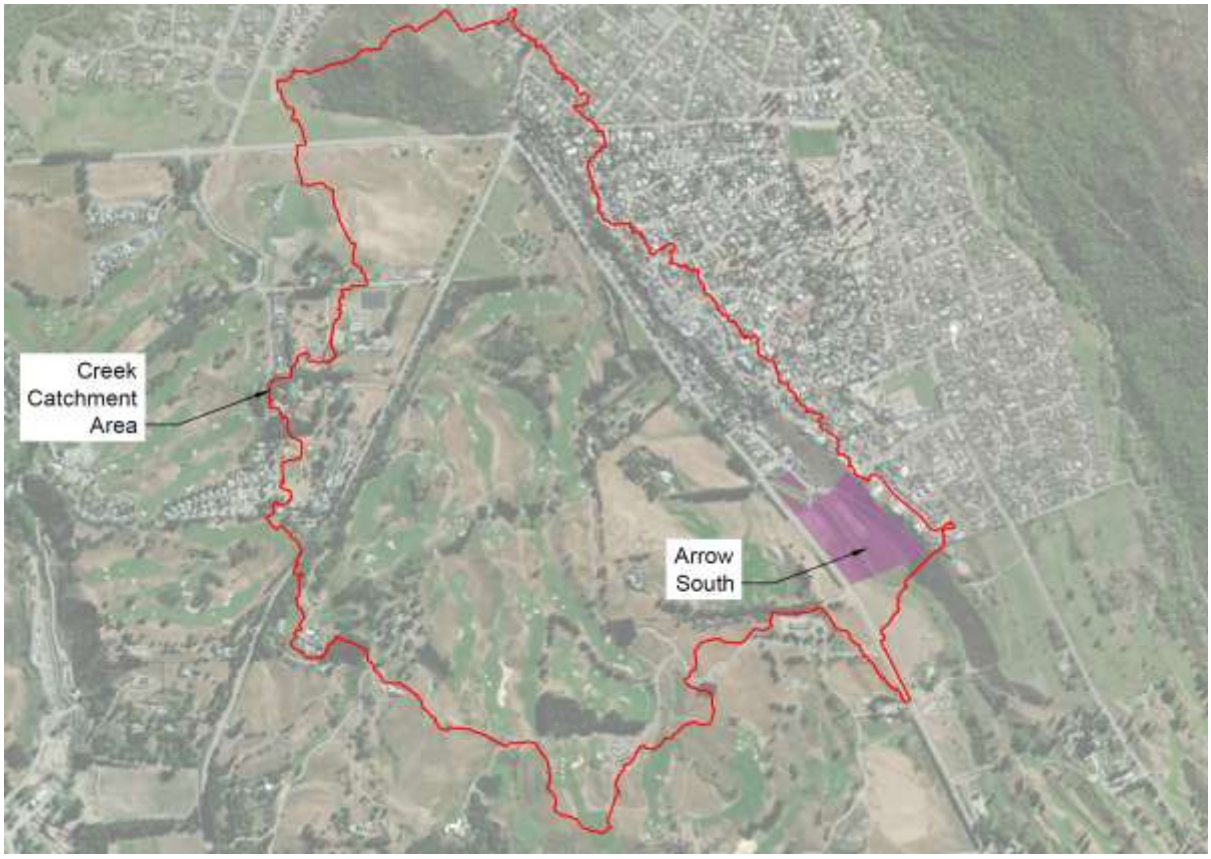

Figure 3 – Creek Catchment Aerial

3 Proposed Development

3.1 Rezoning and Development Concept

It is proposed to rezone the subject land as Suburban Residential ZoneC.

The final number of allotments is currently unconfirmed as minimum lot size rules are pending finalization under the QLDC PDP. Current estimates range from 44 to 59 lots. However, the highest

estimated yield (59 lots) has been used to evaluate stormwater runoff characteristics to ensure the infrastructure design is sufficient.

The development will include additional roading to provide access to future residential allotments. It is proposed that the steeper eastern slopes will become reserve areas, alongside other reserves intended to preserve existing walking tracks within the site.

A concept subdivision drawing showing a possible configuration has been prepared by Coterra Ltd and is included in Appendix A.

3.2 Proposed Impervious Areas

The site currently has no constructed impervious areas.

Development will introduce impervious surfaces including roading, dwellings, ancillary buildings, and footpaths, which will increase runoff from the developed portions of the site.

It is anticipated that the residential areas will have approximately 65% impervious coverage once fully developed. These figures will be refined as the project progresses, and lot sizes are finalized.

4 Stormwater Management Criteria

4.1 Regulatory Framework

The primary objectives for stormwater management are outlined in Policies 27.2.5.11 and 27.2.5.12 of the QLDC Proposed District Plan (PDP). These policies emphasize:

- 27.2.5.11: Ensuring appropriate design by considering viable alternatives, system capacity, and methods that maintain water quality and control peak flows.
- 27.2.5.12: Encouraging designs that integrate stormwater and flood management with open spaces, transport corridors, and ecological values.

This feasibility assessment has been prepared in accordance with the QLDC Land Development and Subdivision Code of Practice (2025) (CoP), NZS 4404:2010, and the New Zealand Building Code.

4.2 Attenuation Requirements

The specific hydraulic performance targets for the site are:

- **Discharge Control:** Post-development peak discharge rates must be attenuated to match pre-development levels for both the 5% AEP (20-year) and 1% AEP (100-year) storm events.

4.3 Setback and Environmental Constraints

Attenuation basins will be located outside the identified flood extent of the existing watercourse. The design will ensure a minimum freeboard of 500mm is maintained between the 1% AEP water level in the basins and any adjacent habitable floor levels.

Furthermore, basins will be located within vested reserve areas to facilitate efficient access for future operation and maintenance.

5 Hydrological & Hydraulic Analysis

5.1 Methodology

The hydrological and hydraulic analysis was undertaken using Autodesk Storm and Sanitary Analysis (SSA) software. The key modelling parameters include:

- Rainfall Data: Derived from HIRDS (Version 4) with allowances for climate change in accordance with the Code of Practice (CoP) (2025).
- Runoff Method: The SCS Curve Number (CN) method was utilized for catchment runoff generation.

5.2 Design Storm Data

Design storms were developed using the Nested Storm Profile methodology outlined in Appendix C of the Wellington Water Reference Guide for Design Storm Hydrology. This approach incorporates NIWA HIRDS V4 data and is an accepted methodology for hydrological modelling within the Queenstown Lakes District.

As required by the CoP, climate change effects have been incorporated into the design. The 1% AEP (100-year) rainfall depth was adjusted for Climate Change using Representative Concentration Pathway (RCP) 8.5 projected to the year 2100.

Full rainfall data used in the analysis is provided in Appendix B.

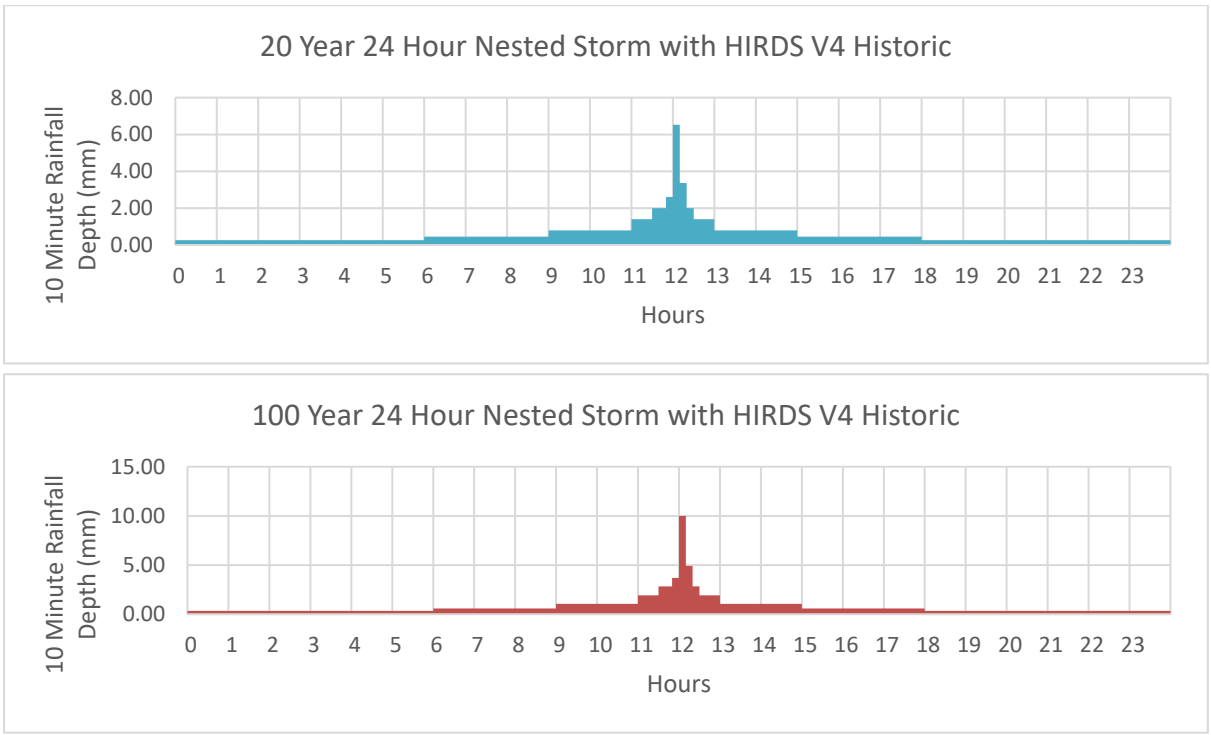

Figure 4: Storm Event Hyetographs – Historic Rainfall Data

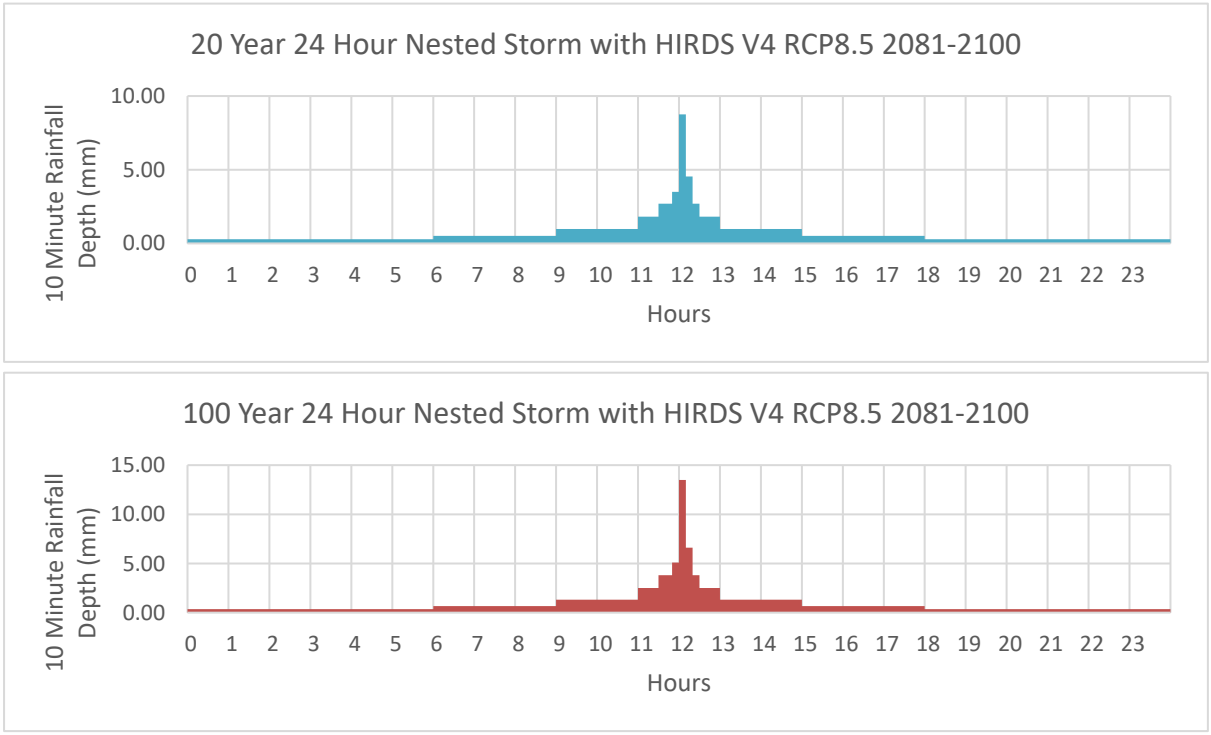

Figure 5: Storm Event Hyetographs – Projected Rainfall Data

5.3 Pre-Development (Existing) Conditions Analysis

Catchment boundaries were delineated based on the proposed development footprint (refer to Figure 6).

Figure 6: Site Catchments

The catchments are defined as follows:

- **Catchment 1**: Development area west of the watercourse.
- **Catchment 2**: Development area east of the watercourse.
- **Catchment 3**: Upper catchment area designated as reserve (unchanged by development).
- **Catchment 4**: Area adjacent to Stage 1. Runoff from this area is managed via the existing Stage 1 attenuation basin and is therefore excluded from this specific sizing analysis.

Since Catchment 3 remains unchanged and Catchment 4 is managed elsewhere, this analysis focuses on **Catchments 1 and 2**.

Parameters:

- **Curve Number (CN):** A CN of **61** was adopted, reflecting Soil Category B and open space with >75% grass cover.
- **Time of Concentration (Tc):** Calculated using the SCS method and rounded to the nearest 10-minute increment.
- **Rainfall:** Historic rainfall data was used for the pre-development scenario.

Catchment	Area (m²)	SCS Curve	Time of Concentration (min)	5% AEP Peak Runoff (L/s)	1% AEP Peak Runoff (L/s)
1	15,183	61	20	13.0	42.8
2	18,169	61	20	15.6	51.3

Table 1: Pre-Development Catchment Analysis

Refer to Appendix C for full hydrological analysis details.

5.4 Post-Development (Proposed) Conditions Analysis

Post-development catchments align with the pre-development boundaries but have been updated to reflect the proposed land use changes.

Parameters:

- **Curve Number (CN):** A CN of **85** was adopted, reflecting Soil Category B and a residential imperviousness of 65%.
- **Time of Concentration (Tc):** Reduced to 10 minutes to reflect channelised flow in a reticulated urban environment.
- **Rainfall:** The climate change scenario (RCP 8.5 to 2100) was applied to the post-development analysis.

Catchment	Area (m²)	SCS Curve	Time of Concentration (min)	5% AEP Peak Runoff (L/s)	1% AEP Peak Runoff (L/s)
1	15,183	85	10	145.8	249.8
2	18,169	85	10	174.7	299.0

Table 2: Post-Development Catchment Analysis

Refer to Appendix C for full hydrological analysis details.

6 Proposed Stormwater Attenuation System

6.1 Conceptual Pond Design

Given that the existing watercourse bisects the site, the stormwater strategy requires two independent drainage networks. Each network will service its respective catchment (East and West) and discharge into a dedicated attenuation basin.

Siting and Configuration

The basins will be located at the natural low points near the southern boundary to facilitate gravity drainage. A **Dry Detention Basin** design is proposed to minimize the permanent footprint and allow for integration into the reserve area.

For feasibility purposes, rectangular basins with the following indicative geometry have been modelled:

Pond	Base Length (m)	Base Width (m)	Base Area (m^2)	Side Slopes (H:V)	Volume at 1m deep (m^3)	Volume at 2m deep (m^3)
1	50	8	400	3:1	592	1,604
2	50	8	400	3:1	592	1,604

Table 3: Indicative Attenuation Pond Configuration

Note: Final basin geometry will be refined during detailed design to accommodate the required storage volumes identified in Section 6.2.

Inlet and Outlet Structures

Runoff will be conveyed to the basins via a combination of piped reticulation and overland flow paths. Scour protection will be installed at all inlet locations.

Discharge from the basins will be controlled via a **multi-stage outlet manhole** connected to the natural watercourse. The control hierarchy is as follows:

- **Primary Control (5% AEP):** An orifice plate located at the base of the manhole will restrict discharge to match the pre-development 5% AEP peak flow.
- **Secondary Control (1% AEP):** A rectangular weir (housed within the manhole) will activate during higher stage events. This weir is sized to ensure the combined discharge remains below the pre-development 1% AEP peak flow.
- **Emergency Overflow:** An overland spillway will be provided to safely convey flows in excess of the design capacity directly to the watercourse, preventing uncontrolled overtopping of the embankments.

6.2 Required Attenuation Volume

Hydraulic analysis was undertaken to determine the storage volume and outlet configuration required to meet the discharge criteria. The results are summarized below:

Pond	Max Depth in Pond (m)	Max Volume Stored in Pond (m^3)	Orifice Diameter (m)	Orifice Discharge (L/s)	Weir Height (m)	Weir Width (m)	Weir Flow (L/s)	Total Flow (L/s)
5% AEP Event								
1	0.83	466	0.08	12.2	-	-	-	12.2
2	0.92	532	0.09	15.2	-	-	-	15.2
1% AEP Event								
1	1.07	645	0.08	13.9	0.98	0.6	28.4	42.2
2	1.2	753	0.09	18.6	1.1	0.6	32.6	51.1

Table 4: Attenuation Storage Analysis Results

7 Feasibility Assessment

7.1 Technical Feasibility:

Spatial Fit

The conceptual design confirms that the proposed stormwater attenuation strategy can be accommodated within the site boundaries. The required footprints for the two dry detention basins (approximately 400m² base area each, plus embankments) are able to be compatible with a future development layout. Specifically, the basins will be sited within the lower-lying areas that will be designated as reserve, ensuring they do not encroach upon residential allotments while allowing for gravity drainage from the catchments. An indicative outline of the proposed ponds is shown on the figure below.

Figure 7: Indicative pond size and location

Hydraulic Performance

Hydraulic modelling using Autodesk SSA demonstrates that the proposed attenuation volumes and multi-stage outlet structures are sufficient to meet the regulatory targets. The design achieves hydraulic neutrality, ensuring that post-development peak discharge rates for both the 5% AEP and 1% AEP rainfall events do not exceed pre-development levels. The inclusion of emergency spillways ensures that extreme events beyond the 1% AEP magnitude can be conveyed safely to the watercourse without threatening the structural integrity of the basins or adjacent properties.

7.2 Environmental Considerations:

Water Quality and Erosion Control

Discharging concentrated flows into the existing watercourse presents a risk of localized scour. To mitigate this, the outlet design will include energy dissipation structures and rip-rap scour protection at the point of discharge. Furthermore, the attenuation of peak flows will protect the downstream channel morphology by reducing the frequency and intensity of high-velocity events that typically drive stream erosion.

While dry detention basins are primarily quantity control devices, they offer secondary water quality benefits by facilitating the settlement of suspended sediments. This aligns with QLDC PDP objectives to maintain downstream water quality.

Ecological Opportunities

The construction of the basins presents an opportunity for ecological enhancement. Rather than maintaining the basins solely as grassed depressions, there is potential to plant the bases and embankments with native wetland and riparian vegetation. This approach would:

- Enhance the amenity value of the reserve areas;
- Improve the filtration of runoff; and
- Create habitat continuity with the existing riparian corridor of the watercourse.

7.3 Maintenance and Operational Considerations:

Ownership and Access

It is anticipated that the attenuation assets, being located within vested reserve areas, will ultimately be vested in the QLDC for long-term ownership and operation. Alternatively, should the reserves remain private, a Residents' Society or Body Corporate would assume responsibility.

The design includes provision for maintenance access tracks (minimum 3.0m width) to allow service vehicles to reach the inlet and outlet structures.

Maintenance Activities

To ensure ongoing performance, a standard Operations and Maintenance (O&M) plan will be required. Typical activities include:

- **Regular Inspections:** Checking the orifice plates (80mm and 90mm diameter) for blockages, particularly after storm events.
- **Sediment Removal:** Periodic removal of accumulated sediment from the basin forebays to maintain storage capacity.
- **Vegetation Management:** Mowing of embankments or maintenance of wetland planting to ensure hydraulic roughness remains within design parameters.
- **Outlet Integrity:** Inspecting scour protection and spillways for stability.

7.4 Identified Risks and Mitigation:

Geotechnical Risks

As noted in Section 2.3, site soils are poorly drained and may present stability challenges for earthworks.

- *Mitigation:* A specific geotechnical investigation will be required during the detailed design phase to confirm embankment stability and foundation design for the outlet structures. If groundwater levels are high, the basin design may need to account for buoyancy or be lined to prevent groundwater ingress reducing available storage.

Regulatory Risks

There is a risk that Council Engineering Standards or PDP rules may evolve prior to Engineering Approval (EA), potentially altering discharge requirements.

- *Mitigation:* The feasibility design has adopted a conservative approach (using a maximum lot yield of 61 and identifying ample reserve space). A pre-application meeting with QLDC is recommended prior to detailed design to confirm final design parameters.

Construction Risks

Working near an active watercourse introduces risks regarding sediment discharge and erosion during earthworks.

- *Mitigation:* A comprehensive Erosion and Sediment Control Plan (ESCP) will be prepared in accordance with QLDC guidelines (or GD05 equivalent). This will include silt fences, decanting earth bunds, and staging works to minimize the duration of exposed soils near the waterway.

8 Conclusions and Recommendations

8.1 Conclusions:

Impact of Development

The proposed rezoning of the subject land to Suburban Residential Zone will transition the site from its current greenfield state to an urbanized environment. Without mitigation, the introduction of approximately 65% impervious coverage (roading, roofs, and hardstanding) would significantly increase both the volume and peak rate of stormwater runoff discharging to the local watercourse.

Mitigation Requirements

To manage this increased runoff and ensure hydraulic neutrality in accordance with QLDC and NZS 4404:2010 standards, on-site attenuation is required. Hydraulic modelling indicates that a combined total storage volume of approximately **1,400 m³** (split between two basins) is necessary to attenuate post-development flows to pre-development levels for the 1% AEP rainfall event.

Feasibility Statement

Based on the hydrological analysis and spatial evaluation undertaken for this report, the proposed stormwater management strategy is **technically feasible**. The required attenuation basins can be accommodated within the site without compromising environmental constraints. The conceptual design effectively mitigates downstream flood risks while adhering to the regulatory framework.

8.2 Recommendations:

Rezoning Support

It is recommended that the rezoning application proceed, on the basis that the proposed stormwater management concept provides a robust solution to manage the hydrological effects of the development.

Future Work

To progress the project from feasibility to construction, it is recommended that the following actions be undertaken prior to or during the detailed design phase:

1. **Geotechnical Investigation:** A site-specific investigation is required to confirm soil parameters for embankment stability and to determine standing groundwater levels, which may influence the depth and lining requirements of the basins.
2. **Topographical Survey:** A detailed survey of the proposed basin locations and the watercourse is essential to confirm gravity drainage gradients and finalize earthworks volumes.
3. **Environmental Impact Assessment (EIA):** An assessment focusing on the interaction between the basin outfalls and the watercourse should be completed to design appropriate scour protection and ecological enhancements.
4. **Regulatory Engagement:** A pre-application meeting with QLDC engineering officers is recommended to review the conceptual design parameters and confirm specific requirements prior to lodging Resource Consent.

Next Steps

Subject to the approval of the rezoning application, it is recommended that the project proceed to the **Detailed Engineering Design** phase to finalize the geometric layout and structural detailing of the stormwater management system.

9 Limitations

This report has been written for the particular brief to Civilised Ltd from their client and no responsibility is accepted for the use of the report for any other purpose, or in any other context or by any third party without prior review and agreement.

In addition, this report contains information and recommendations based on information obtained from a variety of methods and sources including inspection, sampling or testing at specific times and locations with limited site coverage and by third parties as outlined in this report. This report does

not purport to completely describe all site characteristics and properties, and it must be appreciated that the actual conditions encountered throughout the site may vary, particularly where ground conditions and continuity have been inferred between test locations. If conditions at the site are subsequently found to differ significantly from those described and/or anticipated in this report, Civilised Ltd must be notified to advise and provide further interpretation.

Appendix A

Drawings

- Concept Subdivision Plan - Coterra Ltd
- Site Location Map
- Wider Creek Catchment Aerial View
- Site Catchments
- Indicative Ponds

DRAFT

NOTES:

1. Aerial imagery has been sourced from UAV survey completed by others on 5/04/2025.
2. Areas and dimensions are subject to survey and detailed design.
3. This lot layout is indicative only and does not comply with current QLDC Rules.

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8 BRODIE AVENUE, ARROWTOWN

Title:
ARROWSOUTH PLAN CHANGE
PROPOSED SUBDIVISION OF
LOTS 102 & 103 DP 535793
CONCEPT PLAN (300m2 MIN. LOTS)

Revision	Amendments	Date

Scale: 1:1500 (A3)

Job No:	Drawn By:	Reviewed By:	Datum:
5479	CRH		-

Drawing No:	Sheet No:	Revision:	Date Created:
5479-1	2 OF 2	A	FEB 2026

Arrow South

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REV	DATE	DESCRIPTION	APPROVED

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CHECKED	DATE

CLIENT

ARROWSOUTH PROPERTIES LIMITED

PROJECT/LOCATION

ARROW SOUTH REZONING
MCDONNELL ROAD

CONTRACT NUMBER

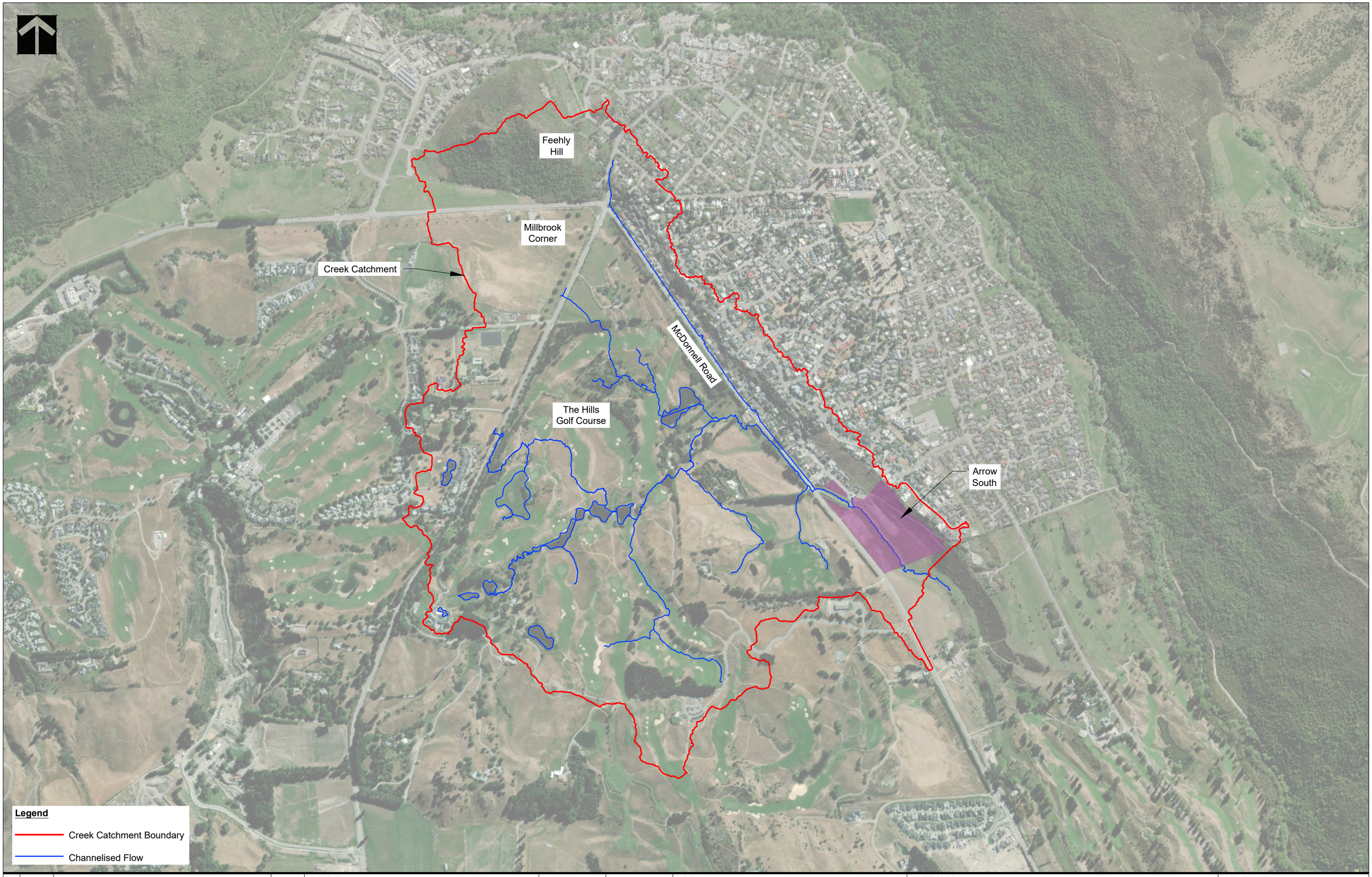
SCALE (AT A3) 1:25000

TITLE

STORMWATER DRAINAGE
SITE LOCATION MAP

DRAWING NUMBER QV075-F-610

REVISION A



REV	DATE	DESCRIPTION	APPROVED
A	21/11/2025	Initial issue	JFM

CONSULTANT

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CIVILISED LTD
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JFM	21/11/2025	
DESIGN	DATE	
JFM	21/11/2025	
DRAWN	DATE	
JFM	21/11/2025	
CHECKED	DATE	

CLIENT

ARROWSOUTH PROPERTIES LIMITED

PROJECT/LOCATION

ARROW SOUTH REZONING
MCDONNELL ROAD

CONTRACT NUMBER

SCALE (AT A3) 1:10000

TITLE

STORMWATER DRAINAGE
WIDER CREEK CATCHMENT AERIAL VIEW

DRAWING NUMBER QV075-F-620

REVISION A

REV	DATE	DESCRIPTION	APPROVED
A	21/11/2025	Initial issue	JFM

CONSULTANT

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JFM	21/11/2025
DESIGN	DATE
JFM	21/11/2025
DRAWN	DATE
JFM	21/11/2025
CHECKED	DATE

CLIENT

ARROWSOUTH PROPERTIES LIMITED

PROJECT/LOCATION

ARROW SOUTH REZONING
MCDONNELL ROAD

TITLE

STORMWATER DRAINAGE
SITE CATCHMENTS

CONTRACT NUMBER

SCALE (AT A3) 1:2000

DRAWING NUMBER QV075-F-630

REVISION A

Arrow South
Subject Land:
Lot 102 DP 535793
Lot 103 DP 535793

McDonnell Road

Pond 1: for drainage from the development area west of the watercourse

Pond 2: for drainage from the development area east of the watercourse

Creek

REV	DATE	DESCRIPTION	
A	16/02/2026	Initial issue	JFM
REV	DATE	DESCRIPTION	APPROVED

CONSULTANT

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JFM	16/02/2026
DESIGN	DATE
JFM	16/02/2026
DRAWN	DATE
JFM	16/02/2026
CHECKED	DATE

CLIENT

ARROWSOUTH PROPERTIES LIMITED

PROJECT/LOCATION

ARROW SOUTH REZONING
MCDONNELL ROAD

CONTRACT NUMBER

SCALE (AT A3) 1:2000

TITLE

STORMWATER DRAINAGE
INDICATIVE POND SIZES & LOCATIONS

DRAWING NUMBER

QV075-F-640

REVISION

A

Appendix B

Rainfall Data

- Rainfall data tables used for the analysis.

HIRDS V4 Intensity-Duration-Frequency Results

Based on QLDC CoP 2025

Rainfall intensities (mm/hr) :: Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h
1.58	63%	16.1	12.6	10.9	8.33	6.25	3.76	2.62	1.77
2	50%	18.1	14.2	12.2	9.27	6.92	4.14	2.88	1.93
5	20%	25.7	19.8	16.9	12.7	9.37	5.49	3.77	2.5
10	10%	32	24.4	20.8	15.5	11.3	6.54	4.45	2.93
20	5%	39.2	29.7	25.1	18.5	13.4	7.67	5.18	3.38
30	3.3%	43.8	33	27.8	20.5	14.8	8.38	5.63	3.65
40	2.5%	47.3	35.6	29.9	22	15.8	8.9	5.96	3.86
50	2%	50.2	37.7	31.6	23.2	16.6	9.32	6.23	4.02
60	1.7%	52.7	39.4	33	24.1	17.3	9.67	6.45	4.15
80	1.3%	56.7	42.3	35.4	25.8	18.4	10.2	6.8	4.36
100	1%	60	44.6	37.3	27.1	19.3	10.7	7.08	4.53
250	0.4%	74.8	55.1	45.8	32.9	23.2	12.6	8.29	5.25

Rainfall intensities (mm/hr) :: RCP8.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h
1.58	63%	21.1	16.5	14.2	10.9	8.05	4.67	3.17	2.09
2	50%	23.9	18.6	16	12.2	9.01	5.19	3.51	2.29
5	20%	34.2	26.3	22.5	16.9	12.3	6.97	4.66	3
10	10%	42.8	32.7	27.8	20.7	15	8.36	5.54	3.54
20	5%	52.6	39.8	33.7	24.9	17.9	9.87	6.48	4.09
30	3.3%	59	44.5	37.5	27.6	19.7	10.8	7.06	4.44
40	2.5%	63.7	47.9	40.3	29.6	21.1	11.5	7.49	4.69
50	2%	67.7	50.8	42.6	31.2	22.2	12	7.82	4.89
60	1.7%	71	53.1	44.6	32.6	23.1	12.5	8.11	5.06
80	1.3%	76.6	57.1	47.8	34.8	24.6	13.2	8.56	5.32
100	1%	81	60.3	50.4	36.6	25.8	13.9	8.93	5.54
250	0.4%	101	74.4	61.8	44.5	31	16.4	10.5	6.42

HIRDS V4 Depth-Duration-Frequency Results

Based on QLDC CoP 2025

Rainfall depths (mm) :: Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h
1.58	63%	2.69	4.21	5.44	8.33	12.5	22.6	31.5	42.5
2	50%	3.02	4.72	6.08	9.27	13.8	24.8	34.5	46.4
5	20%	4.28	6.6	8.45	12.7	18.7	32.9	45.2	59.9
10	10%	5.33	8.15	10.4	15.5	22.6	39.2	53.4	70.2
20	5%	6.53	9.89	12.5	18.5	26.9	46	62.2	81.1
30	3.3%	7.3	11	13.9	20.5	29.6	50.3	67.6	87.7
40	2.5%	7.89	11.9	15	22	31.6	53.4	71.6	92.6
50	2%	8.37	12.6	15.8	23.2	33.2	55.9	74.7	96.4
60	1.7%	8.78	13.1	16.5	24.1	34.6	58	77.4	99.6
80	1.3%	9.45	14.1	17.7	25.8	36.8	61.4	81.6	105
100	1%	9.99	14.9	18.6	27.1	38.6	64.1	85	109
250	0.4%	12.5	18.4	22.9	32.9	46.4	75.8	99.5	126

Rainfall depths (mm) :: RCP8.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h
1.58	63%	3.51	5.5	7.12	10.9	16.1	28	38	50.1
2	50%	3.98	6.2	8	12.2	18	31.1	42.1	55
5	20%	5.7	8.78	11.2	16.9	24.7	41.8	55.9	72
10	10%	7.14	10.9	13.9	20.7	30	50.2	66.5	84.9
20	5%	8.76	13.3	16.8	24.9	35.8	59.2	77.7	98.2
30	3.3%	9.83	14.8	18.7	27.6	39.4	64.8	84.7	106
40	2.5%	10.6	16	20.1	29.6	42.1	69	89.8	113
50	2%	11.3	16.9	21.3	31.2	44.4	72.2	93.8	117
60	1.7%	11.8	17.7	22.3	32.6	46.2	75.1	97.3	121
80	1.3%	12.8	19	23.9	34.8	49.2	79.5	103	128
100	1%	13.5	20.1	25.2	36.6	51.6	83.1	107	133
250	0.4%	16.9	24.8	30.9	44.5	62	98.3	125	154

Nested Storm Profile

Based on Appendix C, Wellington Water - Reference Guide for Design Storm Hydrology (9 April 2019),
and email from Beca dated 15/8/2023.

HIRDS V4 - Rainfall depths (mm) :: Historical Data

% AEP	10m	20m	30m	1h	2h	6h	12h	24h
5	6.53	9.89	12.5	18.5	26.9	46	62.2	81.1
1	9.99	14.9	18.6	27.1	38.6	64.1	85	109

HIRDS V4 - Rainfall depths (mm) :: RCP8.5 for the period 2081-2100

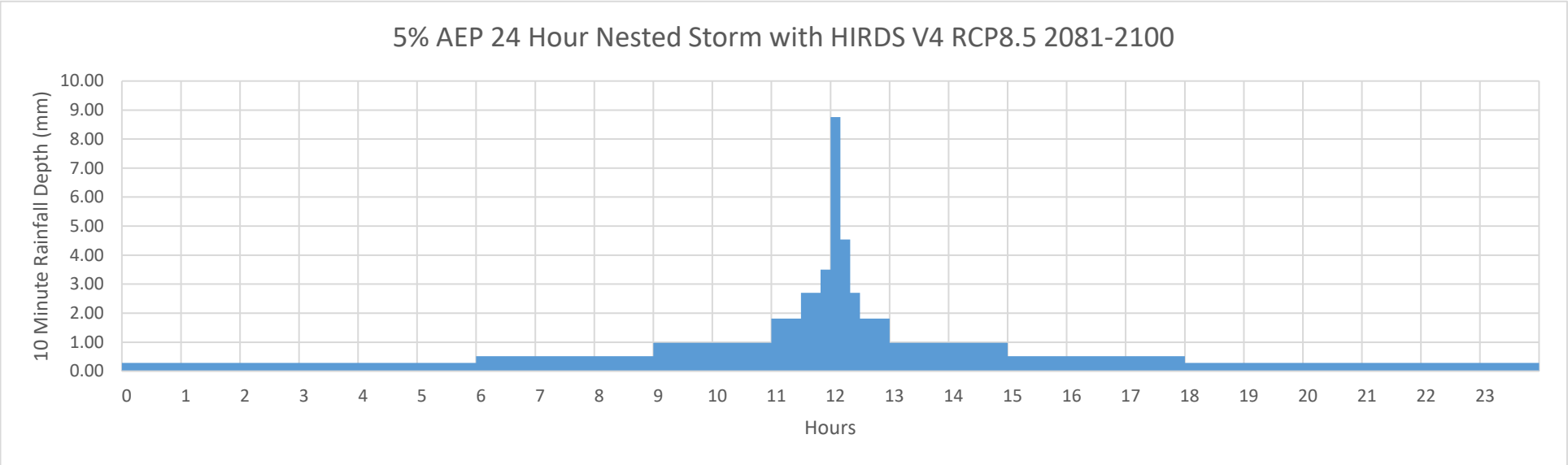
% AEP	10m	20m	30m	1h	2h	6h	12h	24h
5	8.76	13.3	16.8	24.9	35.8	59.2	77.7	98.2
1	13.5	20.1	25.2	36.6	51.6	83.1	107	133

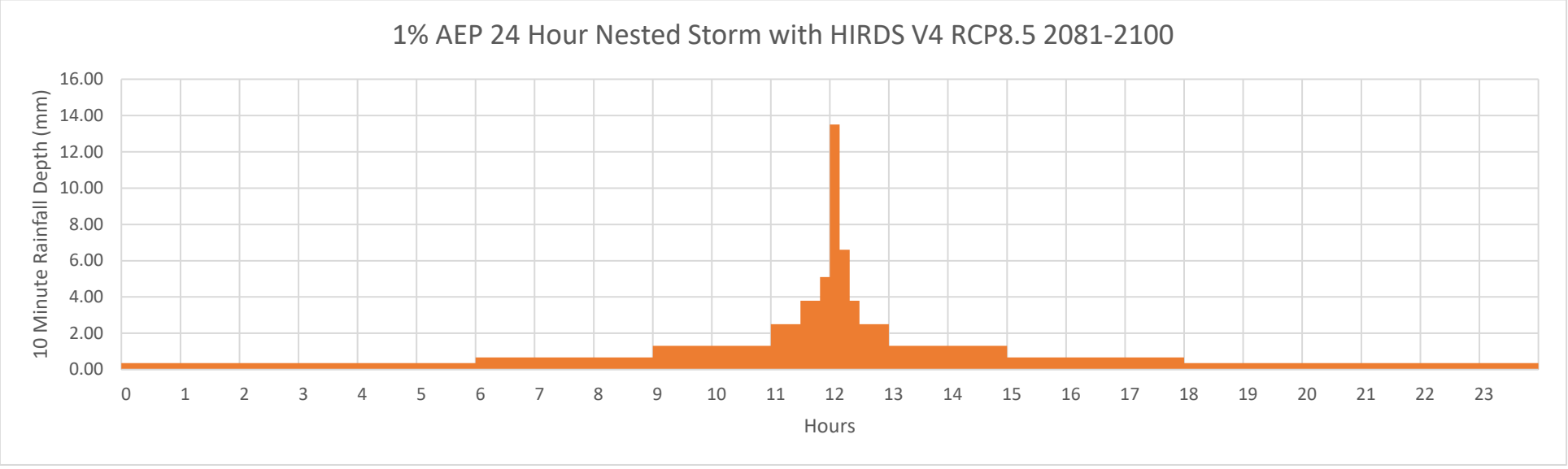
Start Time	End Time	Percentage of Rainfall Depth		Historic Data				RCP8..5 for the period 2081-2100			
				5% AEP (mm)	1% AEP (mm)	5% AEP (mm/hr)	1% AEP (mm/hr)	5% AEP (mm)	1% AEP (mm)	5% AEP (mm/hr)	1% AEP (mm/hr)
0:00	0:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
0:10	0:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
0:20	0:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
0:30	0:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
0:40	0:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
0:50	1:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
1:00	1:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
1:10	1:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
1:20	1:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
1:30	1:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
1:40	1:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
1:50	2:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
2:00	2:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
2:10	2:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
2:20	2:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
2:30	2:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
2:40	2:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
2:50	3:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
3:00	3:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
3:10	3:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
3:20	3:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
3:30	3:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
3:40	3:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
3:50	4:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
4:00	4:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
4:10	4:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
4:20	4:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
4:30	4:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
4:40	4:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
4:50	5:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
5:00	5:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
5:10	5:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
5:20	5:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
5:30	5:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
5:40	5:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
5:50	6:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
6:00	6:10	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
6:10	6:20	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
6:20	6:30	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
6:30	6:40	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
6:40	6:50	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
6:50	7:00	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
7:00	7:10	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
7:10	7:20	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
7:20	7:30	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
7:30	7:40	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
7:40	7:50	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
7:50	8:00	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
8:00	8:10	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
8:10	8:20	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
8:20	8:30	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
8:30	8:40	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
8:40	8:50	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
8:50	9:00	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
9:00	9:10	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
9:10	9:20	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
9:20	9:30	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
9:30	9:40	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
9:40	9:50	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
9:50	10:00	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
10:00	10:10	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88

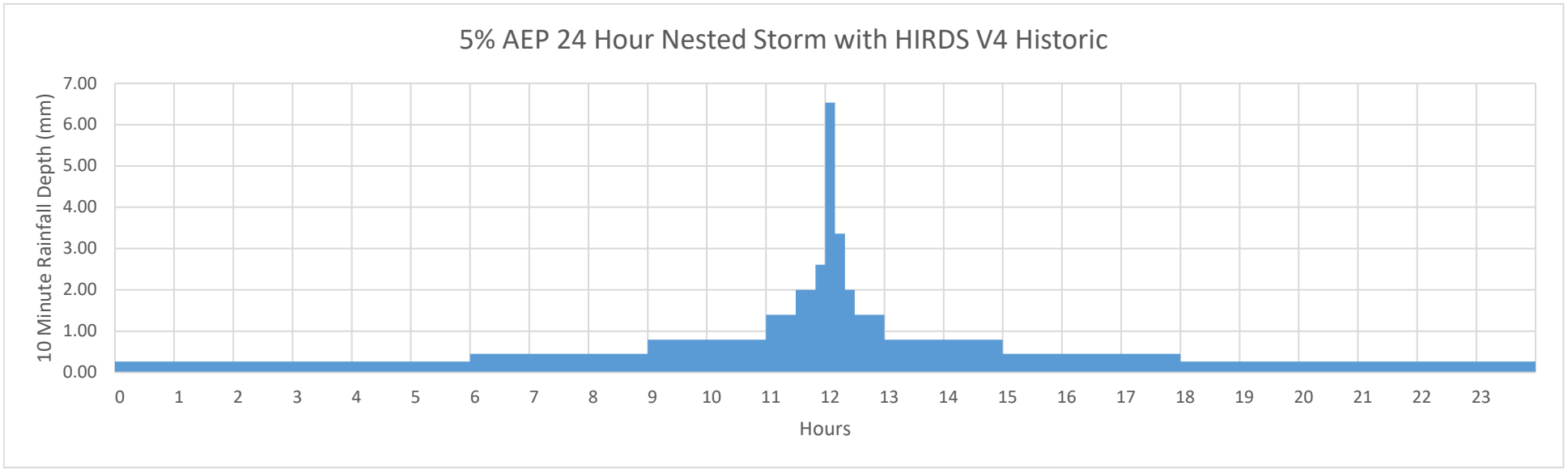
Time				5% AEP (mm)	1% AEP (mm)	5% AEP (mm/hr)	1% AEP (mm/hr)	5% AEP (mm)	1% AEP (mm)	5% AEP (mm/hr)	1% AEP (mm/hr)
10:10	10:20	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
10:20	10:30	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
10:30	10:40	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
10:40	10:50	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
10:50	11:00	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
11:00	11:10	16.67%	1/6 of 2 hour depth less 1 hour depth	1.40	1.92	8.40	11.50	1.82	2.50	10.90	15.00
11:10	11:20	16.67%	1/6 of 2 hour depth less 1 hour depth	1.40	1.92	8.40	11.50	1.82	2.50	10.90	15.00
11:20	11:30	16.67%	1/6 of 2 hour depth less 1 hour depth	1.40	1.92	8.40	11.50	1.82	2.50	10.90	15.00
11:30	11:40	33.33%	1/3 of 60 minute depth less 30 minute depth	2.00	2.83	12.00	17.00	2.70	3.80	16.20	22.80
11:40	11:50	33.33%	1/3 of 60 minute depth less 30 minute depth	2.00	2.83	12.00	17.00	2.70	3.80	16.20	22.80
11:50	12:00	100.00%	30 minute depth less 20 minute depth	2.61	3.70	15.66	22.20	3.50	5.10	21.00	30.60
12:00	12:10	100.00%	10 minute depth	6.53	9.99	39.18	59.94	8.76	13.50	52.56	81.00
12:10	12:20	100.00%	20 minute depth less 10 minute depth	3.36	4.91	20.16	29.46	4.54	6.60	27.24	39.60
12:20	12:30	33.33%	1/3 of 60 minute depth less 30 minute depth	2.00	2.83	12.00	17.00	2.70	3.80	16.20	22.80
12:30	12:40	16.67%	1/6 of 2 hour depth less 1 hour depth	1.40	1.92	8.40	11.50	1.82	2.50	10.90	15.00
12:40	12:50	16.67%	1/6 of 2 hour depth less 1 hour depth	1.40	1.92	8.40	11.50	1.82	2.50	10.90	15.00
12:50	13:00	16.67%	1/6 of 2 hour depth less 1 hour depth	1.40	1.92	8.40	11.50	1.82	2.50	10.90	15.00
13:00	13:10	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
13:10	13:20	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
13:20	13:30	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
13:30	13:40	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
13:40	13:50	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
13:50	14:00	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
14:00	14:10	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
14:10	14:20	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
14:20	14:30	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
14:30	14:40	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
14:40	14:50	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
14:50	15:00	4.17%	1/24 of 6 hour depth less 2 hour depth	0.80	1.06	4.78	6.38	0.98	1.31	5.85	7.88
15:00	15:10	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
15:10	15:20	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
15:20	15:30	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
15:30	15:40	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
15:40	15:50	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
15:50	16:00	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
16:00	16:10	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
16:10	16:20	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
16:20	16:30	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
16:30	16:40	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
16:40	16:50	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
16:50	17:00	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
17:00	17:10	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
17:10	17:20	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
17:20	17:30	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
17:30	17:40	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
17:40	17:50	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
17:50	18:00	2.78%	1/36 of 12 hour depth less 6 hour depth	0.45	0.58	2.70	3.48	0.51	0.66	3.08	3.98
18:00	18:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
18:10	18:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
18:20	18:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
18:30	18:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
18:40	18:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
18:50	19:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
19:00	19:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
19:10	19:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
19:20	19:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
19:30	19:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
19:40	19:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
19:50	20:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
20:00	20:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
20:10	20:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
20:20	20:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
20:30	20:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
20:40	20:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
20:50	21:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
21:00	21:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
21:10	21:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
21:20	21:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
21:30	21:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
21:40	21:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
21:50	22:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
22:00	22:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
22:10	22:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
22:20	22:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
22:30	22:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
22:40	22:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
22:50	23:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17

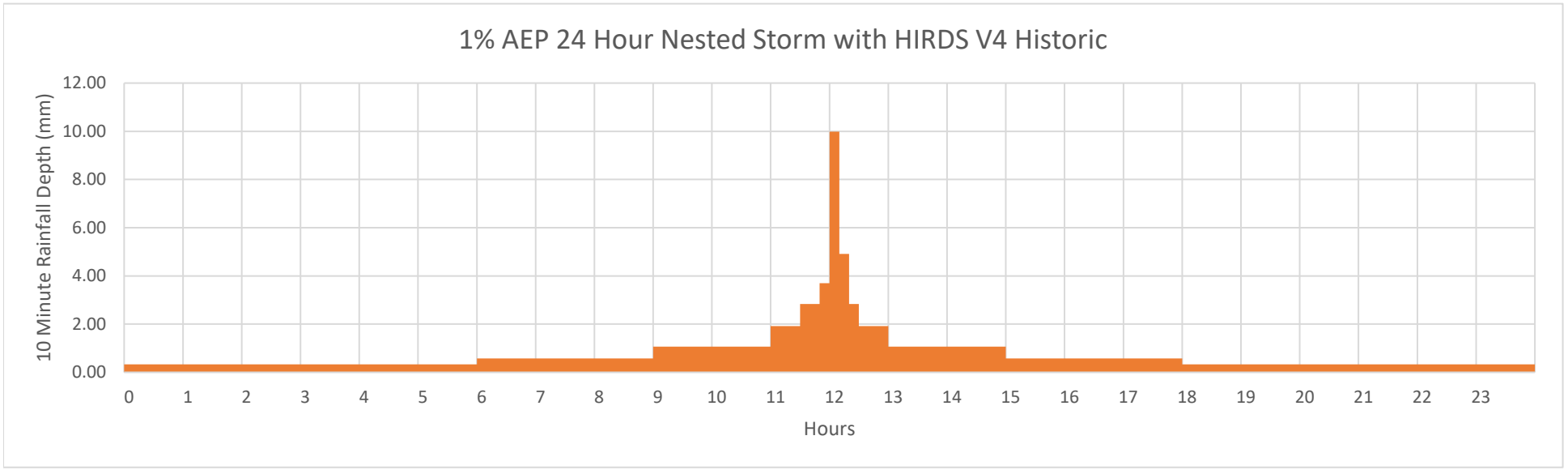
Time				5% AEP (mm)	1% AEP (mm)	5% AEP (mm/hr)	1% AEP (mm/hr)	5% AEP (mm)	1% AEP (mm)	5% AEP (mm/hr)	1% AEP (mm/hr)
23:00	23:10	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
23:10	23:20	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
23:20	23:30	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
23:30	23:40	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
23:40	23:50	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
23:50	24:00	1.39%	1/72 of 24 hour rainfall depth less 12 hour depth	0.26	0.33	1.58	2.00	0.28	0.36	1.71	2.17
			Totals:	**81**	**109**			**98**	**133**		

Nested Storm Hyetographs









Appendix C

Hydrological Calculations

- Catchment model input parameters
- Detailed model output for pre- and post-development scenarios
- Hydrographs for the pre- and post-development scenarios

Catchments

Catchment	Area (m^2)	Soil Group	Curve Number	Length of longest flow path (m)	Slope of longest flow path (m/m)	Sheet Flow					Shallow Concentrated					Channel Flow								TR55 Time of Concentration (minutes)	Adopted TOC (minutes)	Lag (min)	Comment
						Roughness	Flow Length (m)	Slope (m/m)	2yr, 24 hr rainfall (mm)	Sheet Flow Time (min)	Flow Length (m)	Slope (%)	Surface Type	Velocity (m/s)	Shallow Concentrated Time (min)	Mannings	Flow Length (m)	Channel Slope (m/m)	Wetted Area (m^2)	Wetted Perimeter (m)	Hydraulic radius (m)	Velocity (m/s)	Channel Flow Time (min)				
Pre Development Catchments																										
1	15,183	B	61	314	0.010	0.4	10	0.01	46.4	1.18	50	1.0	Grass	0.46	1.8	0.030	254	0.01	0.006	0.3	0.020	0.246	17.237	20.23	20.00	12.00	New Urban Area
2	18,169	B	61	303	0.010	0.4	10	0.01	46.4	1.18	50	1.0	Grass	0.46	1.8	0.030	243	0.01	0.006	0.3	0.020	0.246	16.490	19.48	20.00	12.00	New Urban Area
Post Development Catchments																										
1	15,183	B	85	314	0.010	0.4	10	0.01	46.4	1.18	50	1.0	Paved	0.62	1.3	0.011	254	0.01	0.006	0.3	0.020	0.670	6.320	8.84	10.00	6.00	New Urban Area
2	18,169	B	85	303	0.010	0.4	10	0.01	46.4	1.18	50	1.0	Paved	0.62	1.3	0.011	243	0.01	0.006	0.3	0.020	0.670	6.046	8.57	10.00	6.00	New Urban Area

Changes highlighted

Autodesk® Storm and Sanitary Analysis 2024 - Version 13.6.323 (Build 0)

****
Project Description
****
File Name ..... 2025-11-21 Pre-Development Analysis.SPF

****
Analysis Options
****
Flow Units ..... LPS
Subbasin Hydrograph Method. SCS TR-55
Time of Concentration..... User-Defined
Storage Node Exfiltration.. Constant rate, free surface area
Starting Date AUG-11-2025 00:00:00
Ending Date AUG-12-2025 00:00:00
Report Time Step 00:05:00

****
Element Count
****
Number of rain gages 1
Number of subbasins 2
Number of nodes 2
Number of links 0

****
Raingage Summary
****

Gage ID	Data Source	Data Type	Recording Interval (min)
Rain Gauge	20yr Historic, Nested Storm	INTENSITY	1.00

****
Subbasin Summary
****

Subbasin ID	Total Area (m²)	Peak Rate Factor
Pre-01	15183.00	484.00
Pre-02	18169.00	484.00

****
Node Summary
****

Node ID	Element Type	Invert Elevation (m)	Maximum Elev. (m)	Ponded Area (m²)	External Inflow
Out-Pre-01	OUTFALL	0.00	0.00	0.00	
Out-Pre-02	OUTFALL	0.00	0.00	0.00	

Runoff Quantity Continuity	Volume (hectare-m)	Depth (mm)
Total Precipitation	0.272	81.650
Surface Runoff	0.106	31.648
Continuity Error (%)	-0.001	

Flow Routing Continuity	Volume (hectare-m)	Volume (Mliters)
External Inflow	0.000	0.000
External Outflow	0.037	0.372
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.000	

****
Composite Curve Number Computations Report
****

Subbasin Pre-01

Soil/Surface Description	Area (m²)	Soil Group	CN
-	15183.00	-	61.00
Composite Area & Weighted CN	15183.00		61.00

Subbasin Pre-02

Soil/Surface Description	Area (m^2)	Soil Group	CN
-	18169.00	-	61.00
Composite Area & Weighted CN	18169.00		61.00

****
Subbasin Runoff Summary
****

Subbasin ID	Total Precip mm	Total Runoff mm	Peak Runoff LPS	Weighted Curve Number	Time of Concentration days	Time of Concentration hh:mm:ss
Pre-01	81.54	11.38	13.03	61.000	0	00:20:00
Pre-02	81.54	11.38	15.57	61.000	0	00:20:00

Analysis began on: Fri Nov 21 17:33:50 2025
Analysis ended on: Fri Nov 21 17:33:53 2025
Total elapsed time: 00:00:03

Autodesk® Storm and Sanitary Analysis 2024 - Version 13.6.323 (Build 0)

****
Project Description
****
File Name ..... 2025-11-21 Pre-Development Analysis.SPF

****
Analysis Options
****
Flow Units ..... LPS
Subbasin Hydrograph Method. SCS TR-55
Time of Concentration..... User-Defined
Storage Node Exfiltration.. Constant rate, free surface area
Starting Date AUG-11-2025 00:00:00
Ending Date AUG-12-2025 00:00:00
Report Time Step 00:05:00

****
Element Count
****
Number of rain gages 1
Number of subbasins 2
Number of nodes 2
Number of links 0

****
Raingage Summary
****

Gage ID	Data Source	Data Type	Recording Interval min
Rain Gauge	100yr Historic, Nested Storm	INTENSITY	1.00

****
Subbasin Summary
****

Subbasin ID	Total Area m²	Peak Rate Factor
Pre-01	15183.00	484.00
Pre-02	18169.00	484.00

****
Node Summary
****

Node ID	Element Type	Invert Elevation m	Maximum Elev. m	Ponded Area m²	External Inflow
Out-Pre-01	OUTFALL	0.00	0.00	0.00	
Out-Pre-02	OUTFALL	0.00	0.00	0.00	

Runoff Quantity Continuity	Volume hectare-m	Depth mm
Total Precipitation	0.365	109.367
Surface Runoff	0.229	68.780
Continuity Error (%)	-0.001	

Flow Routing Continuity	Volume hectare-m	Volume Mliters
External Inflow	0.000	0.000
External Outflow	0.081	0.809
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	0.000	

****
Composite Curve Number Computations Report
****

Subbasin Pre-01

Soil/Surface Description	Area (m²)	Soil Group	CN
-	15183.00	-	61.00
Composite Area & Weighted CN	15183.00		61.00

Subbasin Pre-02

Soil/Surface Description	Area (m²)	Soil Group	CN
-	18169.00	-	61.00
Composite Area & Weighted CN	18169.00		61.00

****
Subbasin Runoff Summary
****

Subbasin ID	Total Precip mm	Total Runoff mm	Peak Runoff LPS	Weighted Curve Number	Time of Concentration days	Time of Concentration hh:mm:ss
Pre-01	109.23	24.64	42.76	61.000	0	00:20:00
Pre-02	109.23	24.64	51.25	61.000	0	00:20:00

Analysis began on: Fri Nov 21 17:39:29 2025
Analysis ended on: Fri Nov 21 17:39:32 2025
Total elapsed time: 00:00:03

Autodesk® Storm and Sanitary Analysis 2024 - Version 13.6.323 (Build 0)

****
Project Description
****
File Name ..... 2025-11-21 Post-Development Analysis.SPF

****
Analysis Options
****
Flow Units ..... LPS
Subbasin Hydrograph Method. SCS TR-55
Time of Concentration..... User-Defined
Link Routing Method ..... Kinematic Wave
Storage Node Exfiltration.. Constant rate, free surface area
Starting Date AUG-11-2025 00:00:00
Ending Date AUG-12-2025 00:00:00
Report Time Step 00:05:00

****
Element Count
****
Number of rain gages 1
Number of subbasins 2
Number of nodes 4
Number of links 4

****
Raingage Summary
****

Gage ID	Data Source	Data Type	Recording Interval (min)
Rain Gauge	20yr RCP 8.5, Nested Storm	INTENSITY	1.00

****
Subbasin Summary
****

Subbasin ID	Total Area (m²)	Peak Rate Factor
Post-01	15183.00	484.00
Post-02	18169.00	484.00

****
Node Summary
****

Node ID	Element Type	Invert Elevation (m)	Maximum Elev. (m)	Ponded Area (m²)	External Inflow
Out-Post-01	OUTFALL	0.00	1.98	0.00	
Out-Post-02	OUTFALL	0.00	2.10	0.00	
Pond-Post-01	STORAGE	0.00	2.00	0.00	
Pond-Post-02	STORAGE	0.00	2.00	0.00	

****
Link Summary
****

Link ID	From Node	To Node	Element Type	Length (m)	Slope (%)	Manning's Roughness
OrificePlate01	Pond-Post-01	Out-Post-01	ORIFICE			
OrificePlate02	Pond-Post-02	Out-Post-02	ORIFICE			
Weir01	Pond-Post-01	Out-Post-01	WEIR			
Weir02	Pond-Post-02	Out-Post-02	WEIR			

****

Runoff Quantity Continuity	Volume (hectare-m)	Depth (mm)
Total Precipitation	0.329	98.542
Surface Runoff	0.560	168.028
Continuity Error (%)	-0.001	

****

Flow Routing Continuity	Volume (hectare-m)	Volume (Mliters)
External Inflow	0.000	0.000
External Outflow	0.127	1.273
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.070	0.705

Continuity Error (%) ..... -0.000

****
Composite Curve Number Computations Report
****

Subbasin Post-01

Soil/Surface Description	Area (m²)	Soil Group	CN
-	15183.00	-	85.00
Composite Area & Weighted CN	15183.00		85.00

Subbasin Post-02

Soil/Surface Description	Area (m²)	Soil Group	CN
-	18169.00	-	85.00
Composite Area & Weighted CN	18169.00		85.00

****
Subbasin Runoff Summary
****

Subbasin ID	Total Precip mm	Total Runoff mm	Peak Runoff LPS	Weighted Curve Number	Time of Concentration days	hh:mm:ss
Post-01	98.43	59.59	145.83	85.000	0	00:10:00
Post-02	98.43	59.59	174.72	85.000	0	00:10:00

****
Node Depth Summary
****

Node ID	Average Depth Attained m	Maximum Depth Attained m	Maximum HGL Attained m	Time of Max Occurrence days	hh:mm	Total Flooded Volume ha-mm	Total Time Flooded minutes	Retention Time hh:mm:ss
Out-Post-01	0.00	0.00	0.00	0	00:00	0	0	0:00:00
Out-Post-02	0.00	0.00	0.00	0	00:00	0	0	0:00:00
Pond-Post-01	0.40	0.83	0.83	0	15:19	0	0	0:00:00
Pond-Post-02	0.43	0.92	0.92	0	15:16	0	0	0:00:00

****
Node Flow Summary
****

Node ID	Element Type	Maximum Lateral Inflow LPS	Peak Inflow LPS	Time of Peak Inflow Occurrence days	hh:mm	Maximum Flooding Overflow LPS	Time of Peak Flooding Occurrence days	hh:mm
Out-Post-01	OUTFALL	0.00	12.18	0	15:19	0.00		
Out-Post-02	OUTFALL	0.00	16.21	0	15:16	0.00		
Pond-Post-01	STORAGE	138.10	138.10	0	12:20	0.00		
Pond-Post-02	STORAGE	165.89	165.89	0	12:20	0.00		

****
Storage Node Summary
****

Storage Node ID	Maximum Ponded Volume 1000 m³	Maximum Ponded Volume (%)	Time of Max Ponded Volume days hh:mm	Average Ponded Volume 1000 m³	Average Ponded Volume (%)	Maximum Storage Node Outflow LPS	Maximum Exfiltration Rate cmm	Time of Max. Exfiltration Rate hh:mm:ss	Total Exfiltrated Volume 1000 m³
Pond-Post-01	0.466	29	0 15:19	0.216	13	12.18	0.00	0:00:00	0.000
Pond-Post-02	0.532	33	0 15:15	0.238	15	16.21	0.00	0:00:00	0.000

****
Outfall Loading Summary
****

Outfall Node ID	Flow Frequency (%)	Average Flow LPS	Peak Inflow LPS
Out-Post-01	72.82	8.73	12.18
Out-Post-02	73.20	11.43	16.21
System	73.01	20.17	28.38

****
Link Flow Summary
****

Link ID	Element Type	Time of Peak Flow Occurrence days hh:mm	Maximum Velocity Attained m/sec	Length Factor	Peak Flow during Analysis LPS	Design Flow Capacity LPS	Ratio of Maximum /Design Flow	Ratio of Maximum Flow Depth	Total Time Reported Condition Surcharged minutes
OrificePlate01	ORIFICE	0 15:19			12.18			0.00	
OrificePlate02	ORIFICE	0 15:16			16.21			0.00	
Weir01	WEIR	0 00:00			0.00			0.00	
Weir02	WEIR	0 00:00			0.00			0.00	

****
Highest Flow Instability Indexes
****
All links are stable.

Analysis began on: Fri Nov 21 17:13:41 2025
Analysis ended on: Fri Nov 21 17:13:45 2025
Total elapsed time: 00:00:04

Autodesk® Storm and Sanitary Analysis 2024 - Version 13.6.323 (Build 0)

****
Project Description
****
File Name ..... 2025-11-21 Post-Development Analysis.SPF

****
Analysis Options
****
Flow Units ..... LPS
Subbasin Hydrograph Method. SCS TR-55
Time of Concentration..... User-Defined
Link Routing Method ..... Kinematic Wave
Storage Node Exfiltration.. Constant rate, free surface area
Starting Date AUG-11-2025 00:00:00
Ending Date AUG-12-2025 00:00:00
Report Time Step 00:05:00

****
Element Count
****
Number of rain gages 1
Number of subbasins 2
Number of nodes 4
Number of links 4

****
Raingage Summary
****

Gage ID	Data Source	Data Type	Recording Interval min
Rain Gauge	100yr RCP 8.5, Nested Storm	INTENSITY	1.00

****
Subbasin Summary
****

Subbasin ID	Total Area m²	Peak Rate Factor
Post-01	15183.00	484.00
Post-02	18169.00	484.00

****
Node Summary
****

Node ID	Element Type	Invert Elevation m	Maximum Elev. m	Ponded Area m²	External Inflow
Out-Post-01	OUTFALL	0.00	1.98	0.00	
Out-Post-02	OUTFALL	0.00	2.10	0.00	
Pond-Post-01	STORAGE	0.00	2.00	0.00	
Pond-Post-02	STORAGE	0.00	2.00	0.00	

****
Link Summary
****

Link ID	From Node	To Node	Element Type	Length m	Slope %	Manning's Roughness
OrificePlate01	Pond-Post-01	Out-Post-01	ORIFICE			
OrificePlate02	Pond-Post-02	Out-Post-02	ORIFICE			
Weir01	Pond-Post-01	Out-Post-01	WEIR			
Weir02	Pond-Post-02	Out-Post-02	WEIR			

Runoff Quantity Continuity	Volume hectare-m	Depth mm
Total Precipitation	0.446	133.783
Surface Runoff	0.862	258.472
Continuity Error (%)	-0.001	

Flow Routing Continuity	Volume hectare-m	Volume Mliters
External Inflow	0.000	0.000
External Outflow	0.201	2.006
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.104	1.036

Continuity Error (%) 0.000

****
Composite Curve Number Computations Report
****

Subbasin Post-01

Soil/Surface Description	Area (m²)	Soil Group	CN
-	15183.00	-	85.00
Composite Area & Weighted CN	15183.00		85.00

Subbasin Post-02

Soil/Surface Description	Area (m²)	Soil Group	CN
-	18169.00	-	85.00
Composite Area & Weighted CN	18169.00		85.00

****
Subbasin Runoff Summary
****

Subbasin ID	Total Precip mm	Total Runoff mm	Peak Runoff LPS	Weighted Curve Number	Time of Concentration days	hh:mm:ss
Post-01	133.64	91.69	249.76	85.000	0	00:10:00
Post-02	133.64	91.69	299.03	85.000	0	00:10:00

****
Node Depth Summary
****

Node ID	Average Depth Attained m	Maximum Depth Attained m	Maximum HGL Attained m	Time of Max Occurrence days	hh:mm	Total Flooded Volume ha-mm	Total Time Flooded minutes	Retention Time hh:mm:ss
Out-Post-01	0.00	0.00	0.00	0	00:00	0	0	0:00:00
Out-Post-02	0.00	0.00	0.00	0	00:00	0	0	0:00:00
Pond-Post-01	0.52	1.07	1.07	0	13:13	0	0	0:00:00
Pond-Post-02	0.58	1.20	1.20	0	13:13	0	0	0:00:00

****
Node Flow Summary
****

Node ID	Element Type	Maximum Lateral Inflow LPS	Peak Inflow LPS	Time of Peak Inflow Occurrence days	hh:mm	Maximum Flooding Overflow LPS	Time of Peak Flooding Occurrence days	hh:mm
Out-Post-01	OUTFALL	0.00	42.21	0	13:13	0.00		
Out-Post-02	OUTFALL	0.00	51.11	0	13:13	0.00		
Pond-Post-01	STORAGE	236.05	236.05	0	12:20	0.00		
Pond-Post-02	STORAGE	280.72	280.72	0	12:20	0.00		

****
Storage Node Summary
****

Storage Node ID	Maximum Ponded Volume 1000 m³	Maximum Ponded Volume (%)	Time of Max Ponded Volume days	hh:mm	Average Ponded Volume 1000 m³	Average Ponded Volume (%)	Maximum Storage Node Outflow LPS	Maximum Exfiltration Rate cmm	Time of Max. Exfiltration hh:mm:ss	Total Exfiltrated Volume 1000 m³
Pond-Post-01	0.646	40	0	13:12	0.302	19	42.21	0.00	0:00:00	0.000
Pond-Post-02	0.754	47	0	13:12	0.345	22	51.11	0.00	0:00:00	0.000

****
Outfall Loading Summary
****

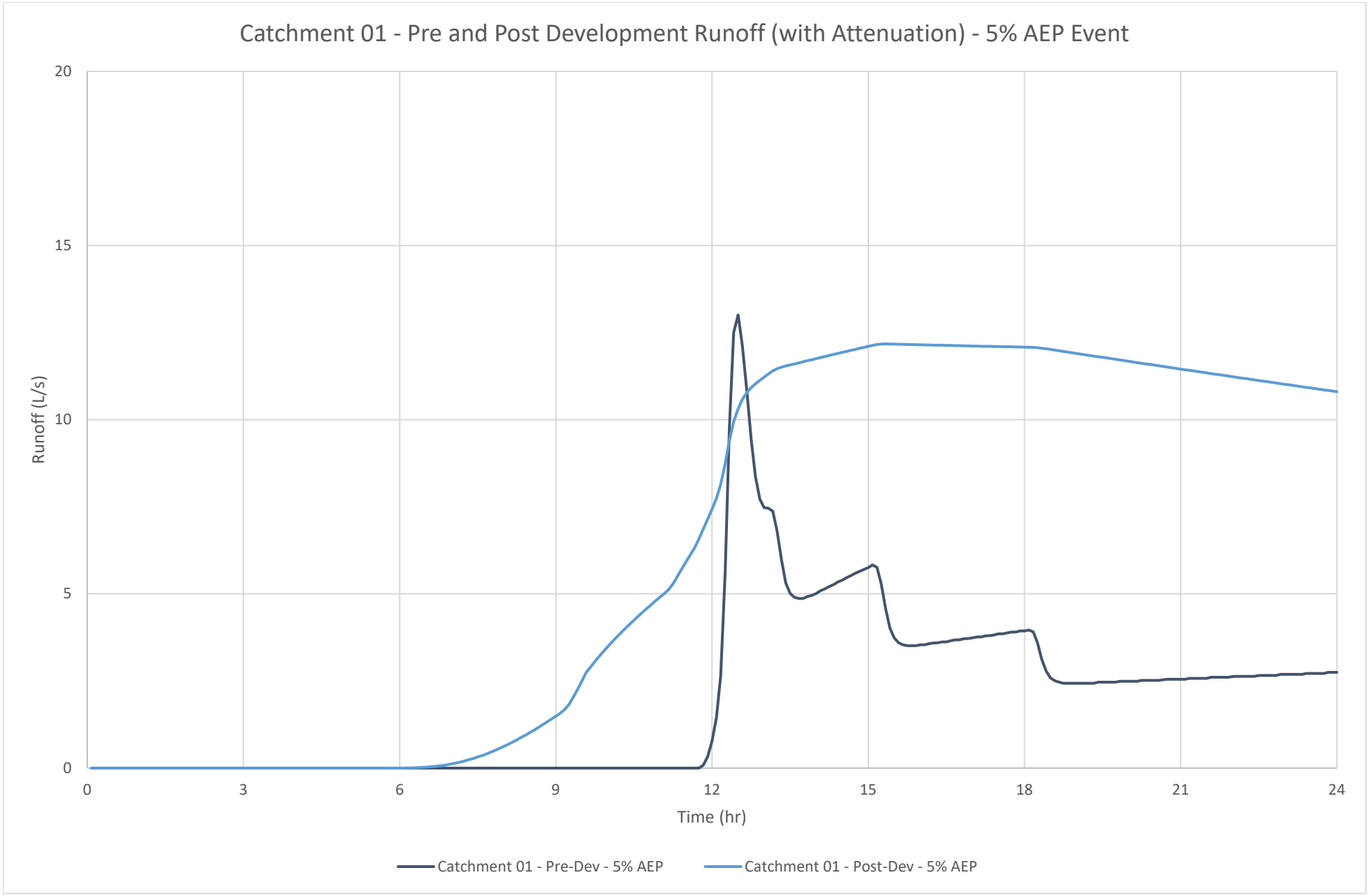
Outfall Node ID	Flow Frequency (%)	Average Flow LPS	Peak Inflow LPS
Out-Post-01	77.99	13.17	42.21
Out-Post-02	78.48	16.49	51.11
System	78.24	29.66	93.32

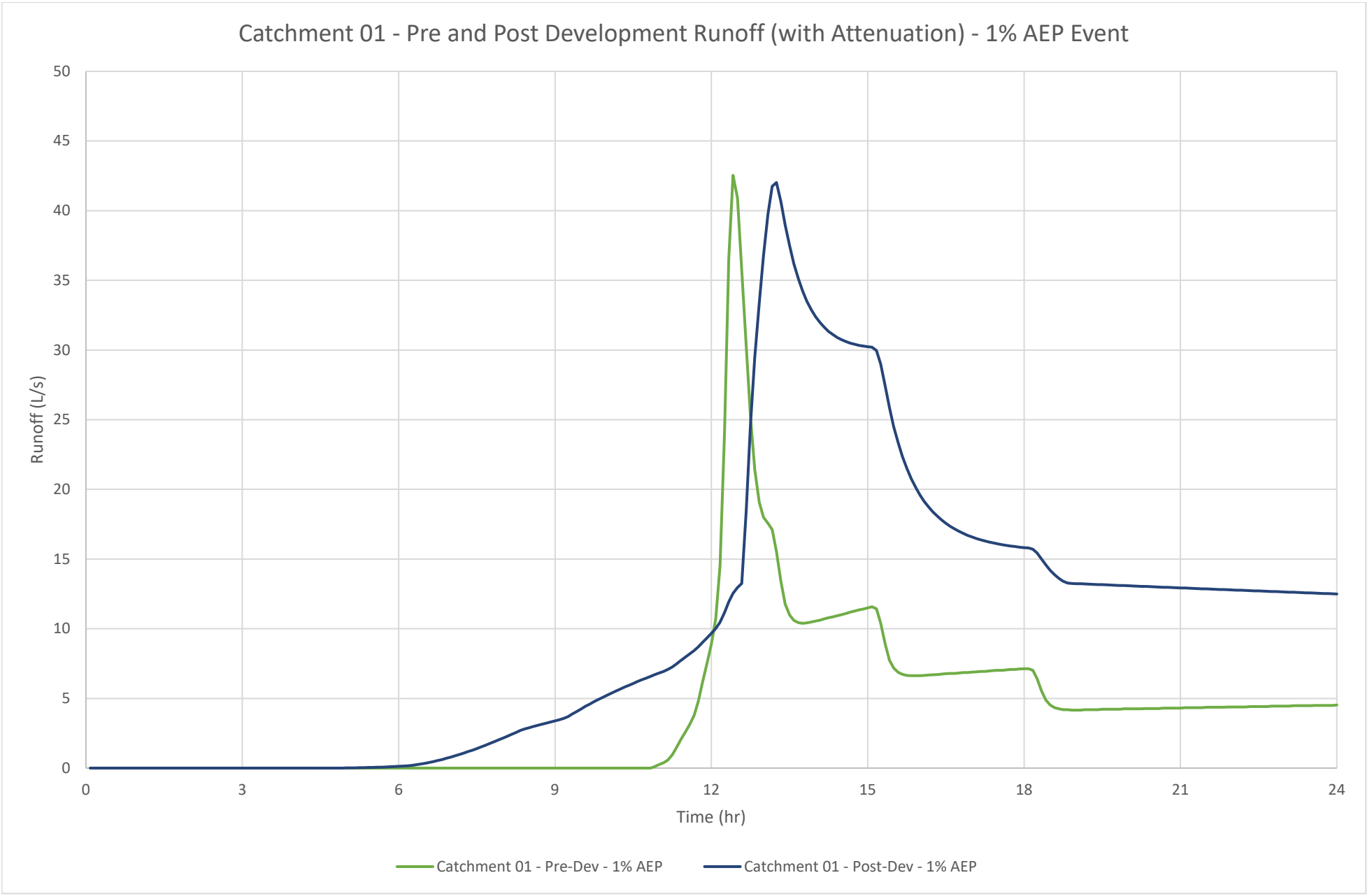
****
Link Flow Summary
****

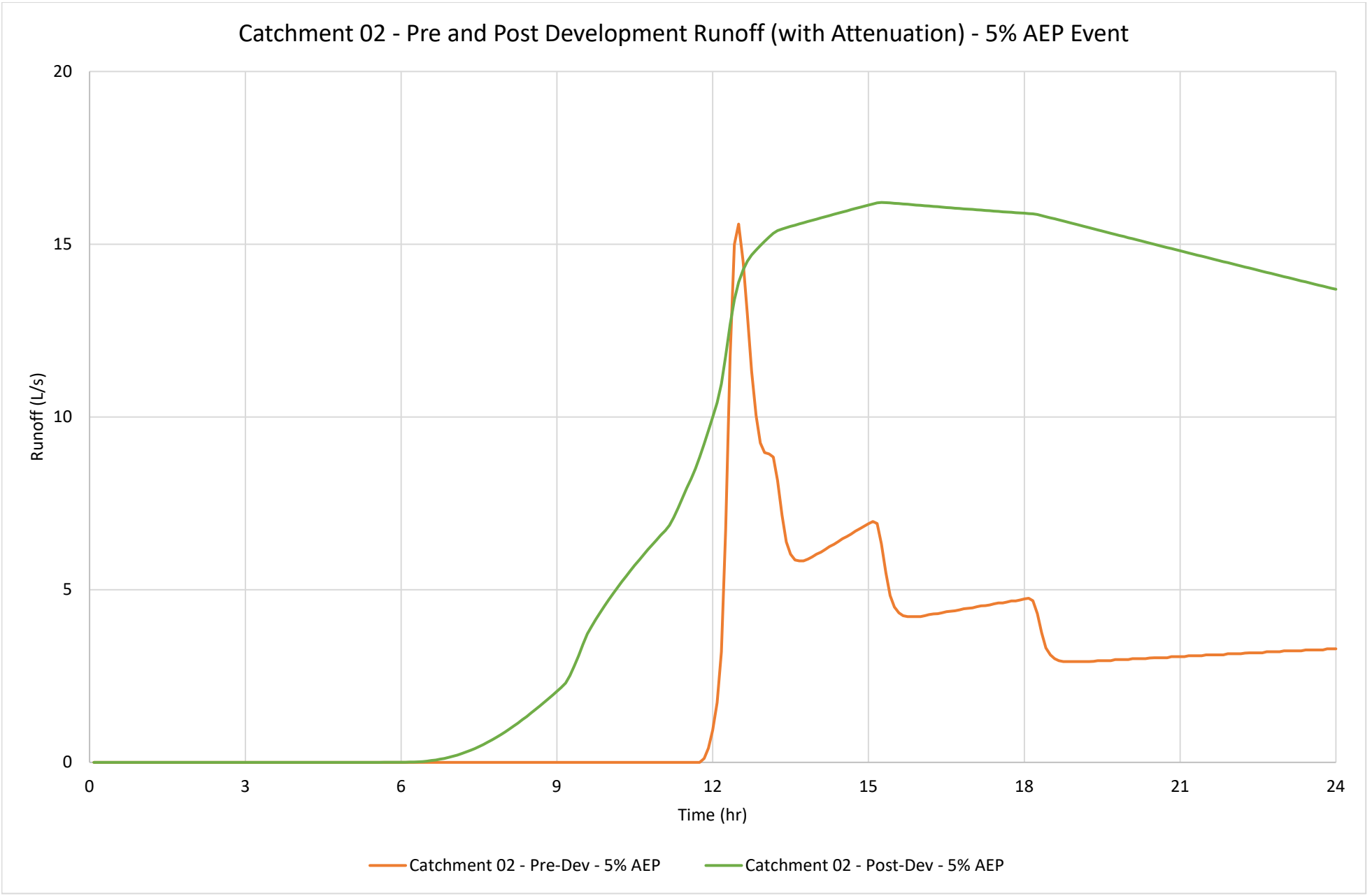
Link ID	Element Type	Time of Peak Flow Occurrence days hh:mm	Maximum Velocity Attained m/sec	Length Factor	Peak Flow during Analysis LPS	Design Flow Capacity LPS	Ratio of Maximum /Design Flow	Ratio of Maximum Flow Depth	Total Time Reported Condition Surcharged minutes
OrificePlate01	ORIFICE	0 13:13			13.86			0.00	
OrificePlate02	ORIFICE	0 13:13			18.56			0.00	
Weir01	WEIR	0 13:13			28.35			0.00	
Weir02	WEIR	0 13:13			32.55			0.00	

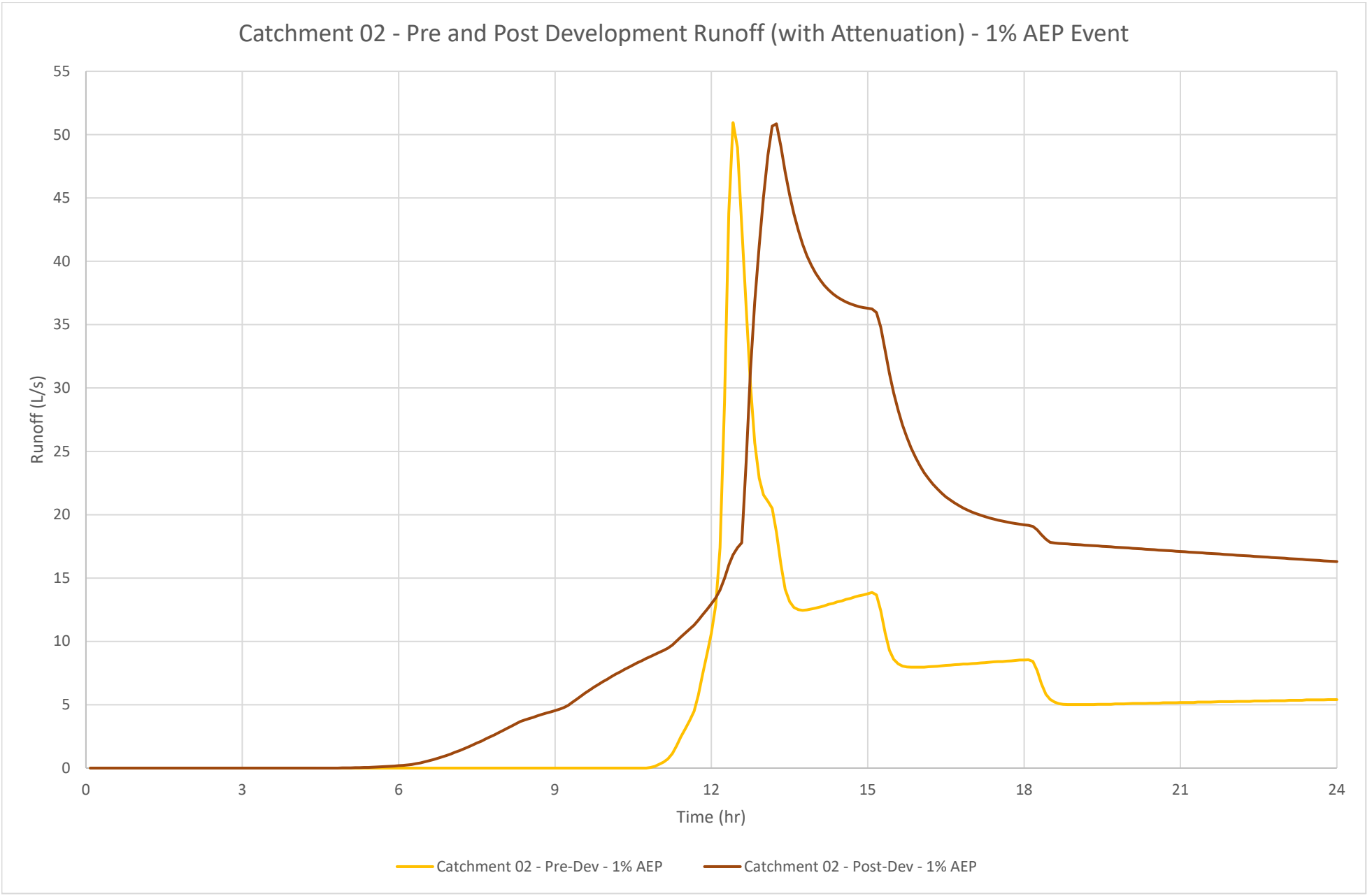
****
Highest Flow Instability Indexes
****
All links are stable.

Analysis began on: Fri Nov 21 17:04:11 2025
Analysis ended on: Fri Nov 21 17:04:14 2025
Total elapsed time: 00:00:03









Appendix E

Correspondence with Aurora Energy

- Email from Civilised Ltd to Aurora Energy
- Response from Aurora Energy

John McCartney <john@civilised.nz>

Proposed Zone Change - McDonnell Road, Arrowtown - Confirmation of Ability to Reticulate

1 message

John McCartney <john@civilised.nz> 22 November 2025 at 12:14
Reply-To: john@civilised.nz
To: GetConnected <getconnected@auroraenergy.nz>

Hi,

We represent Arrowsouth Properties Ltd who is currently seeking a zone change from QLDC for an area of land adjacent to Arrowtown on McDonnell Road. While this project is in its early stages, and the final allotment yield is unknown, a draft concept subdivision plan is attached that demonstrates up to 61 new residential allotments may result from the zone change.

The land is currently legally described as:

- Lot 102 DP 535793
- Lot 103 DP 535793

Could you please provide confirmation that Aurora can provide the appropriate power supply infrastructure to reticulate electricity to the site in the event that the zzone change is approved?

Please contact me if you require any further information at this stage.

Regards,

John McCartney
Civilised Ltd
Email: john@civilised.nz
Phone: 027 2233036

Appendix A - 1 Subdivision Concept Plan.r.pdf
677K

AURORA ENERGY LIMITED
PO Box 5140, Dunedin 9058
PH 0800 22 00 05
WEB www.auroraenergy.co.nz

24/11/2025 REF: 129480

John McCartney
Civilised Ltd
john@civilised.nz

Dear John,

ELECTRICITY SUPPLY AVAILABILITY FOR A PROPOSED 61 LOT SUBDIVISION MCDONNELL ROAD, ARROWTOWN 9371 LOT 102 & 103 DP 535793 .

Thank you for your inquiry outlining the above proposed development.
Subject to technical, legal and commercial requirements, Aurora Energy can make a Point of Supply[1] (PoS) available for this development.

Disclaimer

This letter confirms that a PoS **can** be made available. This letter **does not** imply that a PoS is available now, or that Aurora Energy will make a PoS available at its cost.

Next Steps

To arrange an electricity connection to the Aurora Energy network, a connection application will be required. General and technical requirements for electricity connections are contained in Aurora Energy's Network Connection Standard. Connection application forms and the Network Connection Standard are available from www.auroraenergy.co.nz.

Yours sincerely

Niel Frear
CUSTOMER INITIATED WORKS MANAGER

[1] Point of Supply is defined in section 2(3) of the Electricity Act 1993.