# Moss Vale Plastics Recycling and Reprocessing Facility

## Technical Report 11 – Water and Wastewater Modelling

Plasrefine Recycling Pty Ltd

17 January 2022

→ **The Power of Commitment**

**ABN 39 008 488 373 ABN 39 008 488 373**
6/20 Smith Street
Parramatta, NSW 2150, Australia
**T** (02) 8898 8800 | **E** sydmail@ghd.comsydmail@ghd.com | **ghd.com**

| | |
|---|---|
| **File name** | https://projectsportal.ghd.com/sites/pp15_05/recyclingplantfeasib/_layouts/15/DocIdRedir.aspx?ID=12524108-27927-534 |
| **Author** | James Tesoriero |
| **Project manager** | Anna Montgomery |
| **Client name** | Plasrefine Recycling Pty Ltd |
| **Project name** | Moss Vale Plastics Recycling and Reprocessing Facility |
| **Document title** | Moss Vale Plastics Recycling and Reprocessing Facility \| Technical Report 11 – Water and Wastewater Modelling |
| **Revision version** | Rev 2 |
| **Project number** | 12524108 |

**Document status**

| Status Code | Revision | Author | Reviewer | | Approved for issue | | |
|---|---|---|---|---|---|---|---|
| | | | Name | Signature | Name | Signature | Date |
| S4 | 0 | J. Tesoriero | R. Fraser | | D Gamble | | 3/11/21 |
| S4 | 1 | J. Tesoriero | R. Fraser | | D Gamble | | 21/21/21 |
| S4 | 2 | J. Tesoriero | R. Fraser | | D Gamble | | 14/1/22 |
| | | | | | | | |

# Executive summary

## Introduction

Plasrefine Recycling Pty Ltd (Plasrefine Recycling) ('the proponent') is seeking approval to construct and operate a plastics recycling and reprocessing facility in Moss Vale, NSW ('the facility'). The facility involves constructing and operating a plastics recycling and reprocessing facility with capacity to receive up to 120,000 tonnes per year of mixed plastics.

GHD Pty Ltd (GHD) has been engaged by the proponent to provide design and planning support for the establishment of the facility at Moss Vale.

Wingecarribee Shire Council (Council) requires water and wastewater modelling be conducted to ensure the proposed development does not detrimentally impact on the integrity of the Council's existing water and wastewater system. After discussion with the Council, it has been agreed that GHD will undertake the modelling, on behalf of Council.

## This report

This Water and Wastewater Modelling Report outlines the hydraulic modelling undertaken to assess the impact of the proposed facility on the existing systems. The proposed facility must comply with Council's water and wastewater design standards. The outcomes of this study will assist Council to review/revise/approve the proposed connection of the facility to the water and wastewater system.

## Assessment summary

#### Wastewater

The current sewer network can accommodate the additional wastewater discharges from the proposed facility. The following outcomes were determined from the wastewater modelling, using the 2021 wastewater flows:

– The addition of the proposed facility's wastewater flows is not expected to cause any dry or wet weather overflows.
– The existing SPS-MV15 has over 8 hours emergency storage with the addition of the proposed facility's flows.
– A new DN225 550 m long gravity main is required to connect the proposed facility to the existing wastewater network.
– The SPS-MV15 has capacity to accommodate the additional wastewater flows from the proposed facility.

Overall, the modelling indicates that the inclusion of the proposed facility's flows to the existing 2021 flows will not adversely impact the Council's wastewater network.

#### Capacity at the Moss Vale Wastewater Treatment Plant (WWTP)

The current capacity of the WWTP is 9,000 EP. From Council advice the WWTP is nearing capacity in 2021. There are plans to upgrade the WWTP to 18,000 EP in 2025/2026. The wastewater flows from the proposed facility is equivalent to 20 houses. Therefore, the additional load from the proposed facility is going have an insignificant impact on the WWTP.

If required, wastewater flows from the proposed facility can be stored onsite for up to 24 hours. The wastewater can be discharged overnight when there are lower flows in the wastewater system.

#### Water

The current water network can accommodate the additional water demands of the proposed facility. The following outcomes were determined from the wastewater modelling, using the 2021 demands:

– The minimum pressure for all customer points is always above the required 12 m of pressure during the peak day demand.

- Maximum head losses within the network greater are than 5 m/km for reticulation mains and 3 m/km for trunk mains. However, these head losses do not cause any customer points to fall under the minimum pressure criteria of 12 m (which is the greater concern).
- The Hill Road Reservoir passes both the static and dynamic performance requirements.
- It is planned that the proposed facility will store 1,200 kL of firefighting water on-site. Under the Australian Standards (AS 2419.1-2005) a total volume of 432 kL is required for firefighting purposes. Therefore, the proposed facility is not reliant on the existing water network for firefighting flows as it has more than 432 kL of water stored onsite for firefighting purposes.
- The sensitivity analysis for the 2026 horizon indicates that the system will be overloaded, with and without the demands from the proposed facility. By 2026, Council plans to complete several major water projects to increase the water supply to Moss Vale area.

The modelling indicates that the inclusion of the proposed facility's flows to the 2021 horizon will not adversely impact the Council's water network.

**Proposed Utility Connections**

Figure 0.1.1 shows the proposed facility with the potential connection point to the existing wastewater system. To connect the proposed facility to the existing wastewater network a new DN225 gravity main approximately 550 m long will need to be constructed. The proponent will pay for the construction of the proposed gravity main. There will be capacity within this main for additional flows from future customers (such as adjoining properties) joining the system. This new main will cross Douglas Road and the existing rising main.

Figure 0.1.2 shows the potential connection of the proposed facility to the water network via the existing gravity fed DN150 Asbestos Concrete (AC) main. A short pipe DN150 pipe (likely less than 100 m) will need to be constructed to connect the proposed facility to the existing water network. The proponent will pay for the construction of this pipe.

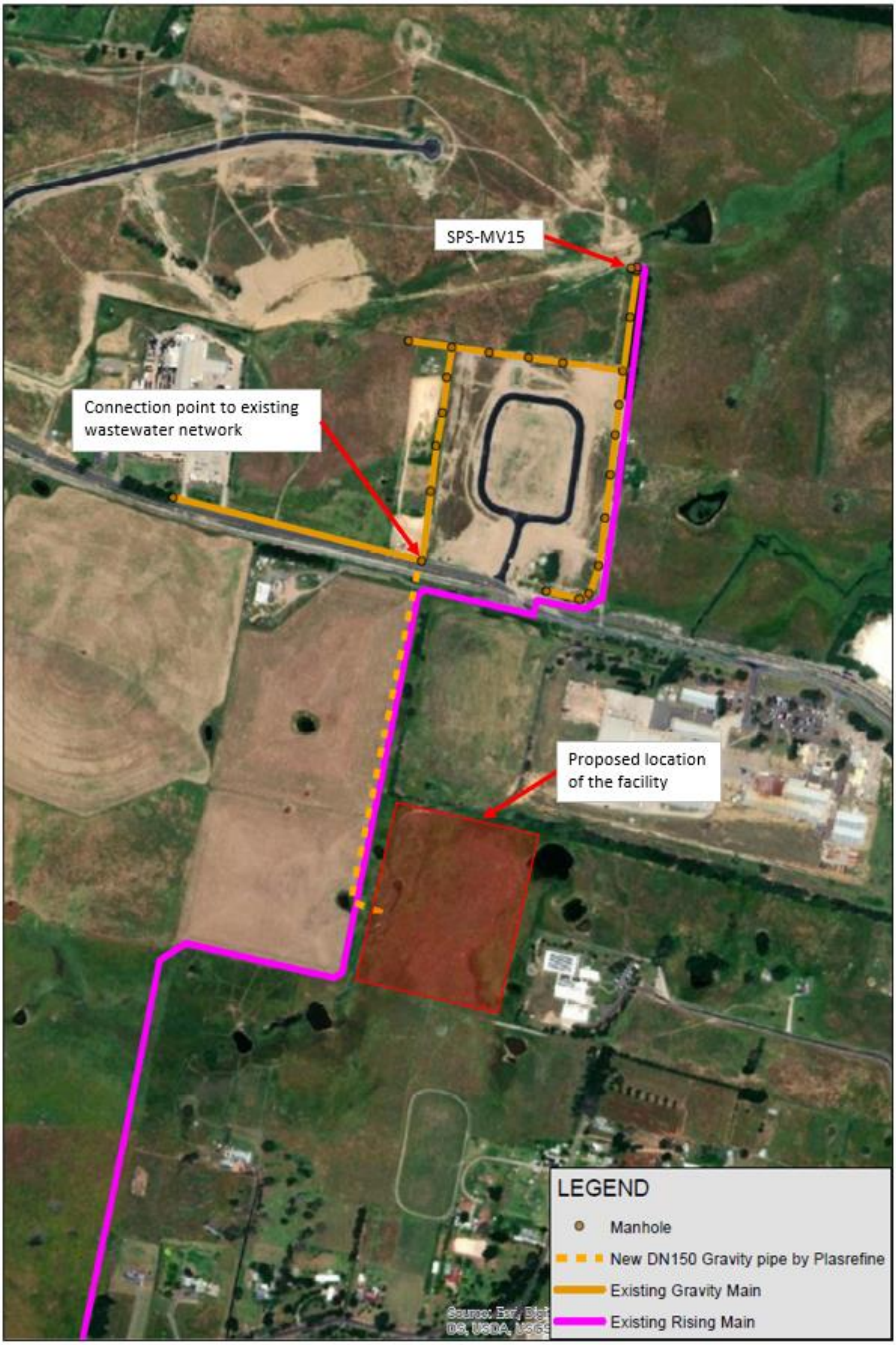



*Figure 0.1.1 Connection point to existing wastewater network*


*Figure 0.1.2 Connection point to existing water network*

# Contents



## Table index

# Figure index

# 1. Introduction

## 1.1 Background

Plasrefine Recycling Pty Ltd (Plasrefine Recycling) ('the proponent') is seeking approval to construct and operate a plastics recycling and reprocessing facility in Moss Vale, NSW ('the proposed facility').

The facility involves constructing and operating a plastics recycling and reprocessing facility with capacity to receive up to 120,000 tonnes per year of mixed plastics.

The proposed facility would sort the plastics into different types and convert the various plastics to flakes and pellets. The facility would also be able to produce more advanced products. The outputs of the proposed facility would help fill the gap in local processing capacity for mixed plastics.

GHD Pty Ltd (GHD) has been engaged by the proponent to provide design and planning support for the establishment of the proposed facility at Moss Vale.

Wingecarribee Shire Council (Council) requires water and wastewater modelling to be conducted to ensure the proposed facility does not detrimentally impact the integrity of the Council's existing water and wastewater system. After discussion with the Council, it has been agreed that GHD will undertake the modelling, on behalf of Council.

## 1.2 Purpose of this report

The purpose of the report is to indicate whether the water and wastewater systems at Moss Vale has the capacity to accept the new water demands and wastewater flows. This report documents the hydraulic modelling undertaken and to assess the impact of the proposed facility on the existing systems. The proposed facility must comply with Council's water and wastewater design standards. The outcomes of this study will assist Council to review/revise/approve the proposed connection of the facility to the water and wastewater system.

## 1.3 Scope and limitations

The scope of the modelling assessment was to:

- Assess the existing water infrastructure performance without the proposed facility for the 2021 water demand.
- Assess the impact of the proposed facility demands/flows on the existing water and wastewater systems.
- Conclude whether the water and the wastewater systems have sufficient capacity to accept the proposed water demands/wastewater flows.
- Conclude whether the WWTP has the capacity to accept the proposed wastewater flows.
- Propose suitable connection points.

This report: has been prepared by GHD for Plasrefine Recycling Pty Ltd and may only be used and relied on by Plasrefine Recycling Pty Ltd and Wingecarribee Shire Council for the purpose agreed between GHD and Plasrefine Recycling Pty Ltd as set out in Section 1.2 of this report.

GHD otherwise disclaims responsibility to any person other than Plasrefine Recycling Pty Ltd arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer section 1.4 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

## 1.4 Assumptions

The following assumption were included in this analysis:

- The 2021 water demands have been used for the water modelling.
- The 2021 wastewater flows have been used for the wastewater modelling.
- Flow per equivalent population (EP) is 230 L/EP/day (from WSC standards).
- EP per tenement is 3.5 (from WSC standards).
- The proposed facility will be operating 24-hours per day.
- 140 employees will be working at the proposed facility per day, spread evenly over three 8-hour shifts.
- Admin flows incorporates toilet, sink and shower flows.
- The total admin shower flows were calculated as 3,300 L/day with a peak flow of 0.52 L/s.
- The following values were used to calculate the shower admin flows:
  - Two thirds of the employees (93) will have end of shift showers (31 per shift).
  - Average shower head flows of 6.2 L/min (0.10 L/s)[1].
  - Total of 5 shower cubicles at the facility[2].
  - Average shower length 5 minutes.
- The admin shower flows has been represented as the three 35-minute peaks to occur at the end of each shift.
- The total for admin toilet and flush flows was calculated as 2,500 L/day.
- The following values were used to calculate the toilet and sink admin flows:
  - Average toilet flush of 6 L/flush.
  - Average of 2 toilet flushes per employee per day.
  - Average sink flow of 6 L/employee/day.
- The admin toilet and sink flows were represent as constant flow of 0.03 L/s across the entire day.
- The trade waste is up to 10,000 L/day (provided by the proponent) which was modelled as a constant flow of 0.12 L/s across the entire day.
- The total process water demand for the proposed facility is 40,500 L/day (provided by the proponent) and modelled as a constant flow of 0.47 L/s across the entire day.
- Maximum head losses greater than the specified criteria (Table 4.2) are permissible if they don't cause customer points to fall below the minimum pressure requirement of 12 m.
- The PDD/ADD ratio for the proposed facility's demand is 1.2 to be conservative.

## 1.5 Location

The proposed facility will be located about 140 kilometres southwest of the Sydney central business district and approximately 2.8 kilometres northwest of the Moss Vale town centre at 74-76 Beaconsfield Road. It will be located on the western side of Beaconsfield Road and on the southern side of Douglas Road, within the Wingecarribee local government area.

The proposed plastics recycling and reprocessing facility and ancillary infrastructure would be located on the northern parcel of land in Lot 11 DP 1084421. This parcel of land has a total site area of about 7.7 hectares.

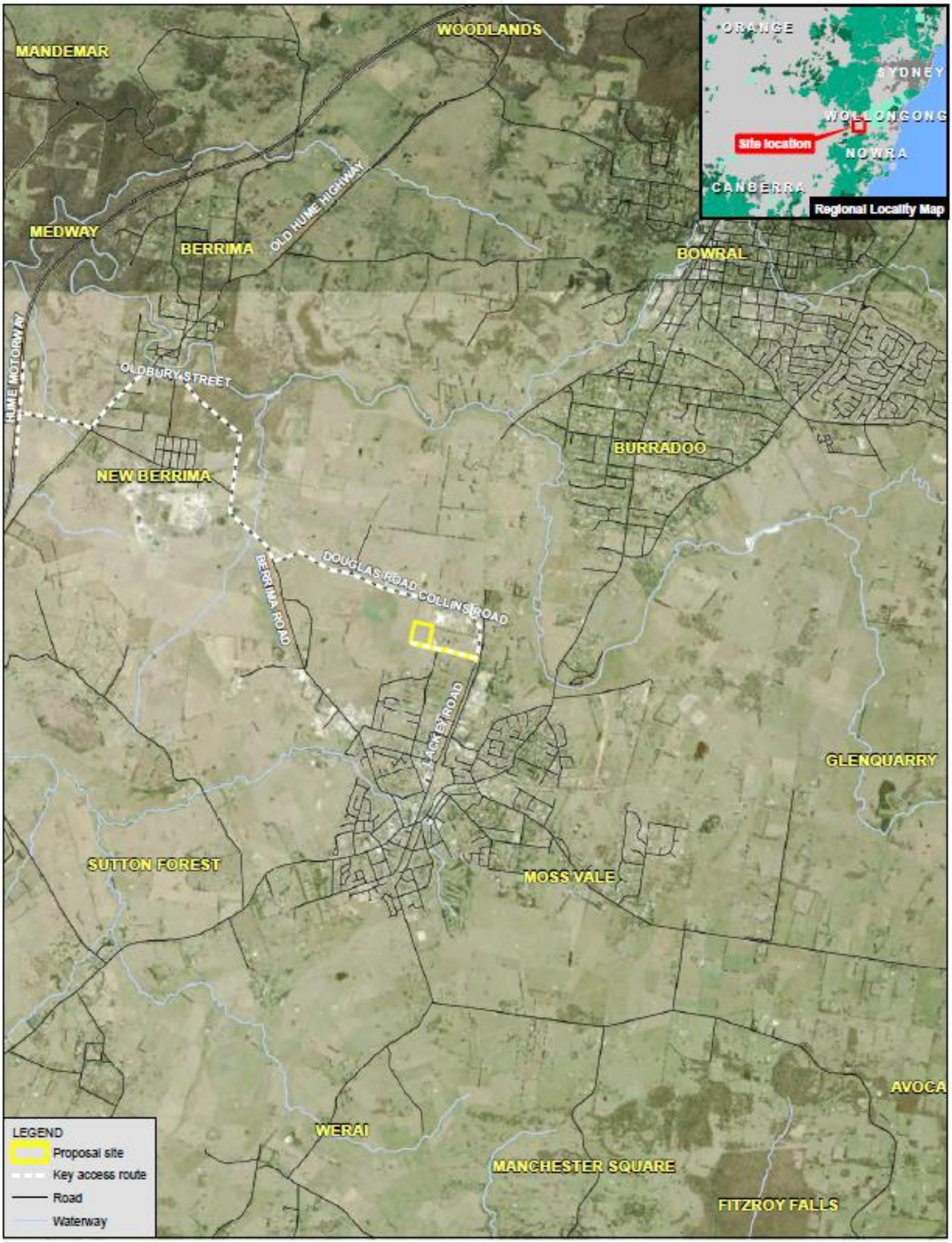
The proposed facility would be accessed via Braddon Road (currently unformed) and a new connection to Lackey Road via Lot 4 DP 26490 and Lot 10 DP1084421 (the Braddon Road east extension).

Neighbouring suburbs include Sutton Forest, New Berrima, Berrima and Bowral. The nearest residences are to the southeast of the proposed plastics recycling and reprocessing facility site, on Beaconsfield Road, Moss Vale.

[1] Measure flow rate from James Tesoriero's shower.
[2] Section 3.8 of the Code of Practice: Managing the Work Environment and Facilities.

The proposed facility would be located within the Moss Vale Enterprise Corridor (MVEC) catchment. The MVEC is a significant area of land between Moss Vale and New Berrima set aside for employment generating development under the Wingecarribee Shire Local Environmental Plan 2010.

An overview of the development is provided in Figure 1.1 and Figure 1.2, overleaf.




Paper Size ISO A4

0 1 2

Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

N

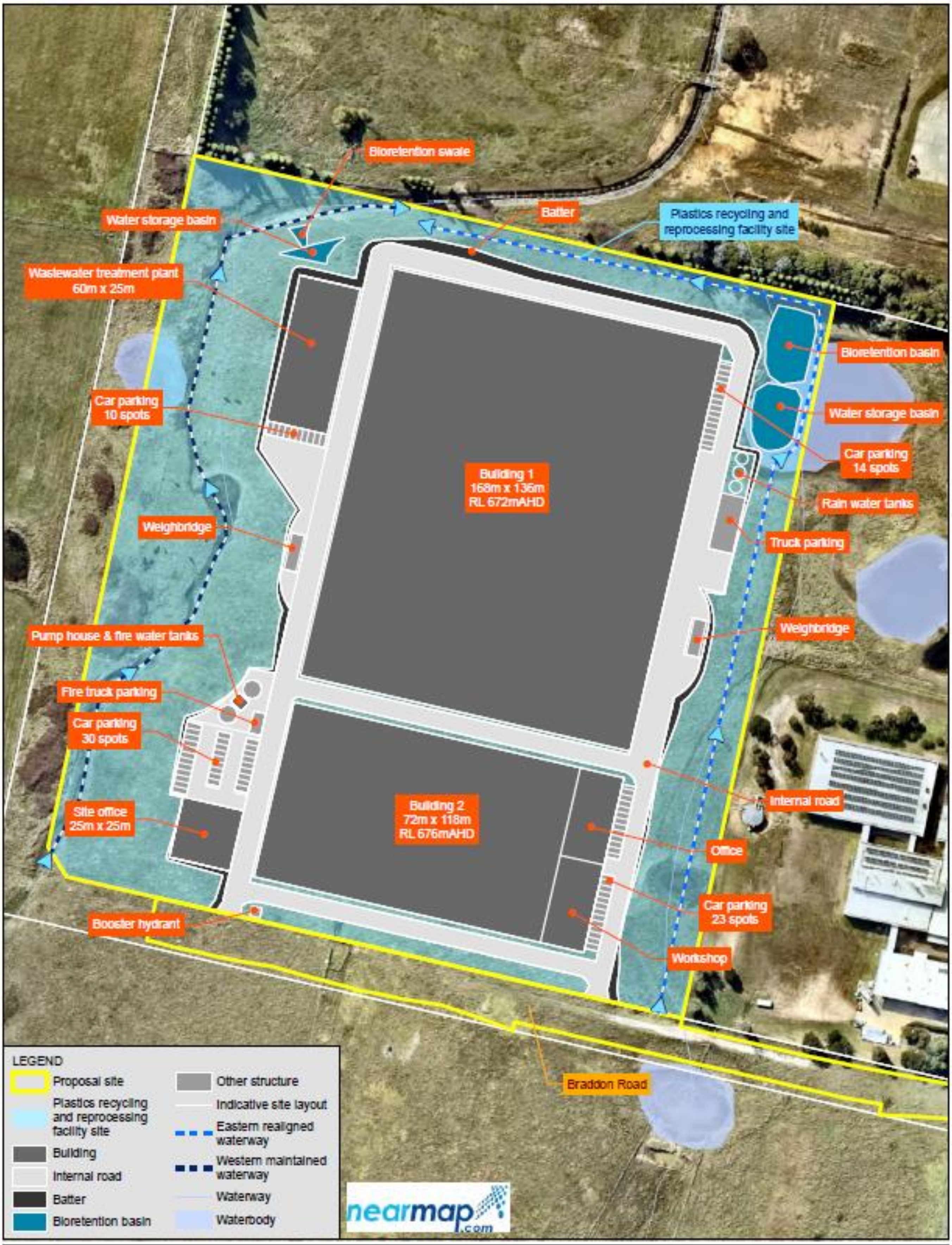
GHD

Plasrefine Recycling Pty Ltd
Moss Vale Plastics Recycling and Reprocessing Facility

Project No. 12524108
Revision No. A
Date 17 Sep 2021

Proposal site location

FIGURE 1.1




Paper Size ISO A4

0 20 40

Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

N

GHD

Plasrefine Recycling Pty Ltd
Moss Vale Plastics Recycling and Reprocessing Facility

Project No. 12524108
Revision No. A
Date 06 Oct 2021

**Proposed site layout**

**FIGURE 1.2**

# 2. Water balance

Figure 2.1 shows the water balance for the proposed facility. The values in Figure 2.1 have been confirmed by the proponent. The total source flow is 46.3 kL/day and consists of the following:

– Toilets, sink and shower flows – 5.8 kL/day
– Process flows – 40.5 kL/day

Section 1.4 outlines how the toilet, sink and shower flows were calculated. The process flows were provided to GHD by the proponent. For the water analysis, the process flows are assumed to be sourced entirely from the water network, however it is understood that significant potential exists for rainwater harvesting, and rainwater storage capacity has been allowed for on site.

Section 1.4 outlines how the toilets, sink and shower flows were calculated. The evaporation, process flows to the sewer and dewatered sediment values were provided to GHD by the proponent.

The discharge flow consists of the following components:

– Toilets, sink and shower flows – 5.8 kL/day
– Process flows to sewer – up to 10 kL/day
– Dewatered sediment – 0.5 kL/day

In addition, approximately 30 kL/day is estimated by the proponent to be lost through evaporation, however this may be an overestimate.

Based on this, the total discharge flow equals to 46.3 kL/day which matches the total source flow. The toilets, sink and shower flows match the equivalent source category. If the values change in Figure 2.1 then additional modelling will be required to confirm that the water and wastewater systems at Moss Vale have the capacity to accept the new water demands and wastewater flows.

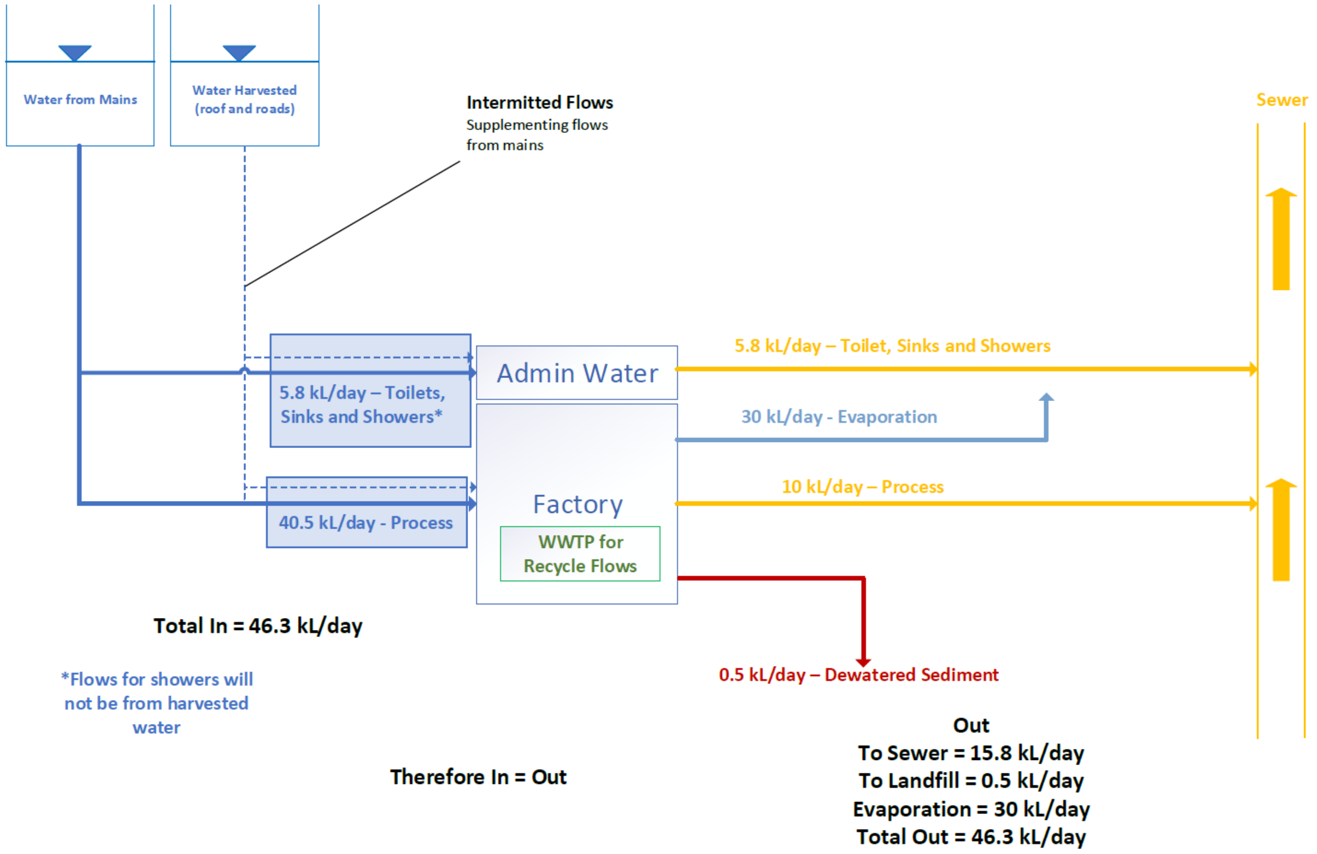



*Figure 2.1 Water balance diagram for the proposed facility*

# 3. Wastewater system

## 3.1 Introduction

This section will detail the following:

– The existing wastewater system
– Model configurations to incorporate the proposed facility (including proposed connection point)
– WSC design standards
– Forecasted flows
– System performance with the inclusion of the proposed facility

## 3.2 Existing system with proposed connection

Figure 3.1 shows an overview of the existing wastewater system downstream of the proposed facility with the potential connection point. The location of the proposed facility is also shown in the figure. To connect the proposed facility to the existing wastewater network a new DN225 gravity main approximately 550 m long will need to be constructed. The proponent will pay for the construction of the proposed gravity main.

Table 5.6 from WSA 02-2014-3.1 was used to size the proposed main. The estimated EP for the proposed facility is 69. The current grade of the current land surface is approximately 1 in 150. From Table 5.6 in the WSA code the maximum number of EPs for a DN150 pipe in Sydney[3] with a 1 in 150 grade is 95. Council requires all industrial connection to be DN225. Therefore, a DN225 is suitable to service the proposed facility. There will be capacity within this main for additional flows from future customers (such as adjoining properties) joining the system.

This proposed main is represented as section A-B in Figure 3.1. This new main will cross Douglas Road and the existing rising main RM00205.

From point B the flows run along an existing DN150 UPVC gravity main to the sewage pump station (SPS) SPS-MV15. Wastewater is pumped via the DN150 HDPE rising main (RM00205) for 2,980 m discharging into a gravity network that eventually discharges to the Moss Vale Wastewater Treatment Plant (WWTP). Table 3.1 outlines the size, length, and material type of each of the sections shown in Figure 3.1.

The current capacity of the WWTP in 2021 is 9,000 EP. There are currently plans to upgrade the WWTP to 18,000 EP by 2025/2026.

The total existing dry weather flows upstream of SPS-MV15 (without the proposed facility) is 3,730 L/day. With the inclusion of the proposed facility the dry weather flows will increase to 19,530 L/day.

*Table 3.1 Pipe sizes from overview*

| Section | Size | Material | Length (m) | State of section |
|---|---|---|---|---|
| A-B | DN225 | UPVC | 550 | Proposed |
| B-C | DN150 | UPVC | 760 | Existing |
| C-D | DN150 | HDPE | 2980 | Existing |
| D-E | DN150 | UPVC | 278 | Existing |
| E-F | DN300 | AC | 903 | Existing |
| F-G | DN750 | AC | 395 | Existing |
| G-H | DN750 | AC | 458 | Existing |

[3]The proposed facility will be located 140 kilometres southwest of the Sydney central business district.

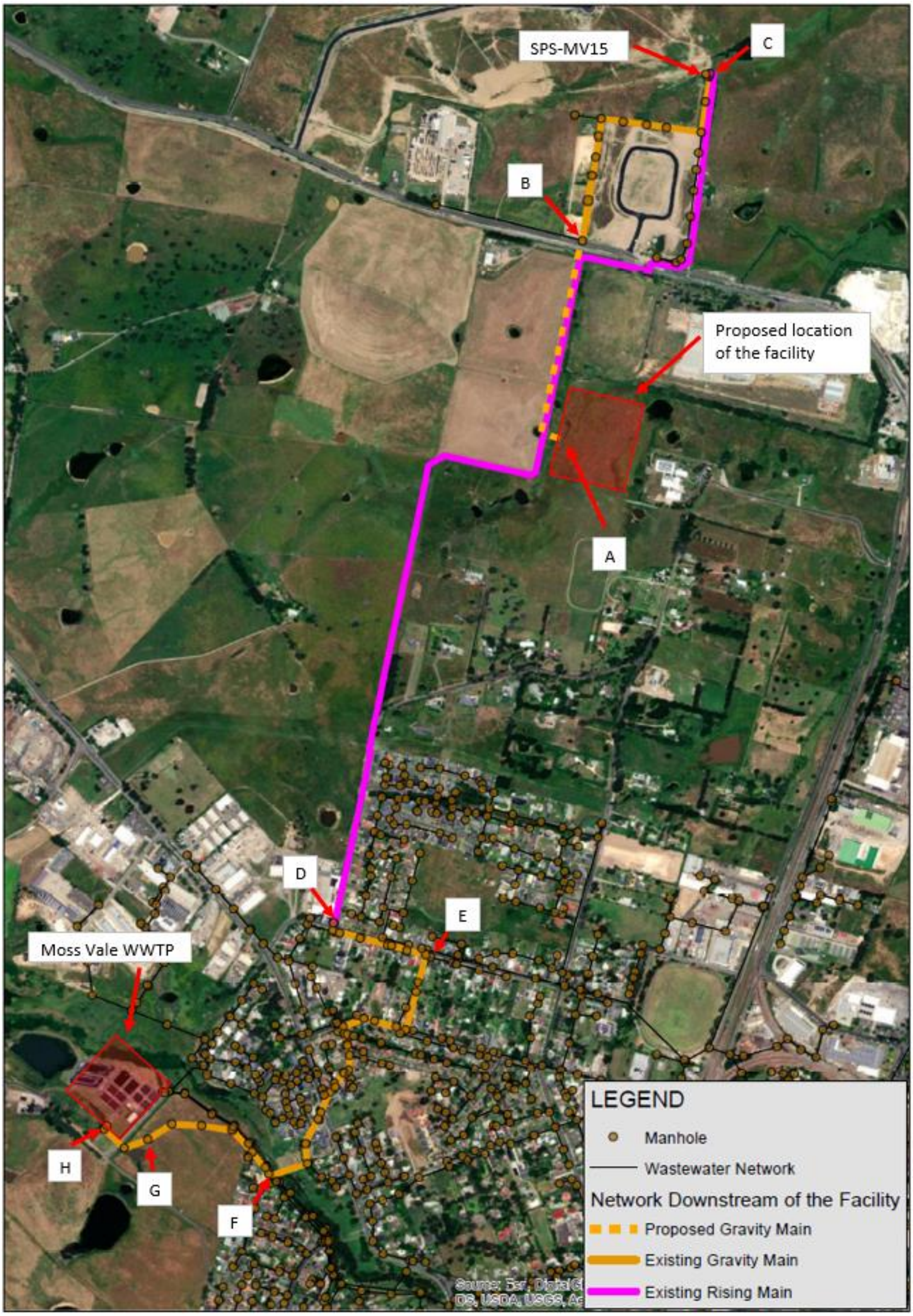



*Figure 3.1 Overview of existing wastewater system with the proposed gravity and facility*

## 3.3 WSC design standards

WSC design standards are as follows:

– There should be no dry weather overflow from the system.
– There should be no dry or wet weather overflow from a pumping station.
– Wet weather allowance – inflow/infiltration (I/I): provide 2% of total area as notional wet weather contribution to the wastewater system.
– Pumping station requirements: emergency storage is 8 hrs average dry weather flow.

## 3.4 Wastewater flow forecast for the proposed facility

The break down for the admin flow and trade waste is provided in Table 3.2. The values in Table 3.2Table 4.4 are from Figure 2.1 in section 2. These values have been confirmed by the proponent. Admin flow incorporates toilet, sink and shower flows from the employees of the facility. The calculation of the toilet, sink and shower flows is outlined in Section 1.4.

*Table 3.2 Estimated admin wastewater flows from the proposed facility [1]*

| Development Type | Average Flow per day (L/day)[4] | Average Flow per shift (L/8-hour shift) | Duration of flow per 8-hour shift | Peak Flow (L/s) |
|---|---|---|---|---|
| Admin Water – Shower Flows | 3,300 | 1,100 | 35 minutes | 0.52 |
| Admin Water – Toilet and Sink Flows | 2,500 | 833 | 8 hours | 0.03 |
| Trade Waste (Process) | 10,000 | 3,333 | 8 hours | 0.12 |
| **Total** | **15,800** | **5,266** | | **0.67** |

[1] These calculations assume a 24-hour a day operation.

The trade waste, toilet and sink flows is represented as a constant flow across the entire day, while the shower flows occur over 35-minute peaks periods. This is because the shower flows will occur when the employees are showering at the end of their shifts.

## 3.5 Wastewater flow profile for the proposed facility

Figure 3.2 shows the flow profile for the proposed facility. In the figure the trade waste, toilet and sink flows are represented as constant flows across the entire day, while the end-of-shift shower flows have been included as three 35-minute peaks, to account for when the employees are having their end of shift showers.

[4]Value from Figure 2.1

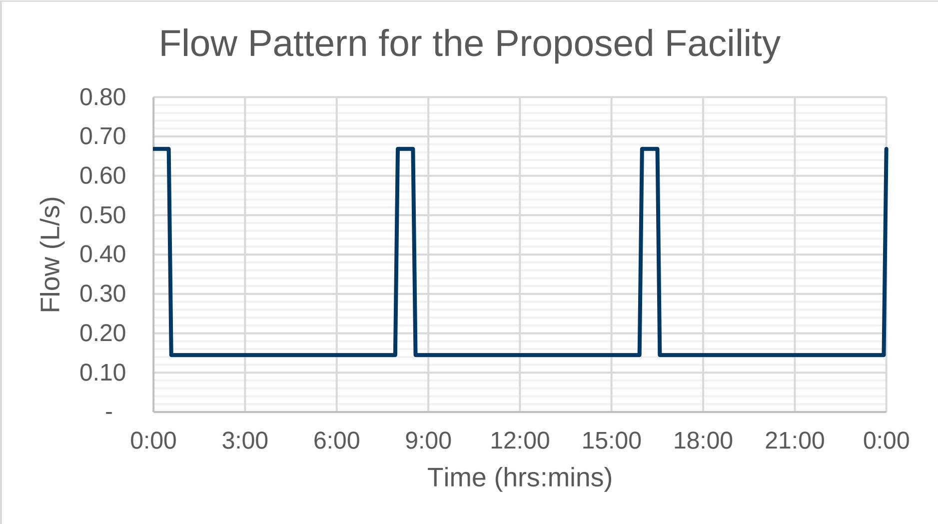



***Figure 3.2 Wastewater profile for the proposed facility***

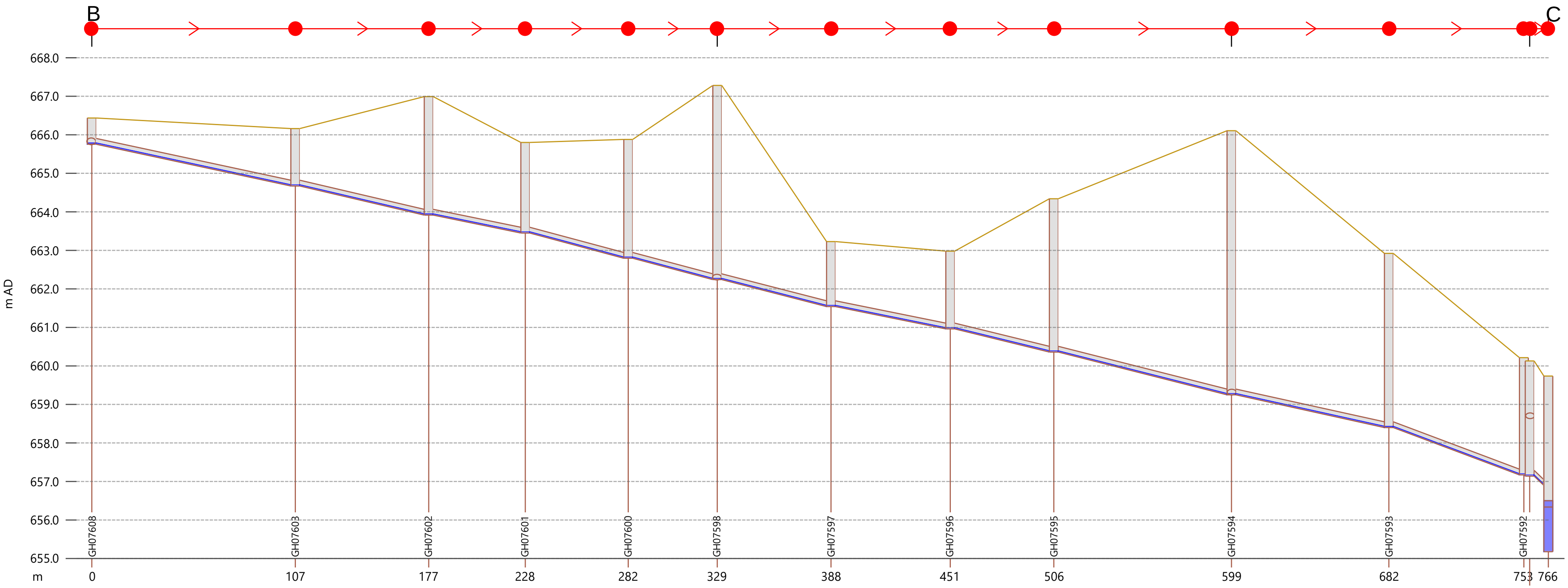
# 3.6 Wastewater System performance with the proposed facility

The following sections outlines the performance of the wastewater network with the existing flows and the flows from proposed facility in 2021.

## 3.6.1 Dry weather performance

### Dry weather overflows

Figure 3.3 and Figure 3.4 show the maximum depth of water within the pipes during dry weather flows. Figure 3.3 shows the long section from location B to SPS-MV15 (section B-C, Figure 3.1). Figure 3.4 shows the long section from the rising main discharge point to Moss Vale WWTP (section D-H, Figure 3.1). The sections labelled Figure 3.1 are shown in Figure 3.3 and Figure 3.4. The two long sections indicate that the pipes are close to being full for short periods (while SPS-MV15 is operating) during dry weather flows. However, the water level does not rise-up into the manholes. Therefore, no dry weather overflows were predicted from the modelling.




| Link | height (mm) | us inv (m AD) | ds inv (m AD) | US depth (m) | US flow (l/s) |
|---|---|---|---|---|---|
| GH07608.1 | 150 | 665.755 | 664.670 | 0.028 | 0.70 |
| GH07603.1 | 150 | 664.670 | 663.915 | 0.028 | 0.70 |
| GH07602.1 | 150 | 663.915 | 663.450 | 0.029 | 0.70 |
| GH07601.1 | 150 | 663.450 | 662.795 | 0.028 | 0.70 |
| GH07600.1 | 150 | 662.795 | 662.240 | 0.028 | 0.70 |
| GH07598.1 | 150 | 662.240 | 661.540 | 0.028 | 0.70 |
| GH07597.1 | 150 | 661.540 | 660.960 | 0.029 | 0.70 |
| GH07596.1 | 150 | 660.960 | 660.360 | 0.028 | 0.70 |
| GH07595.1 | 150 | 660.360 | 659.250 | 0.028 | 0.70 |
| GH07594.1 | 150 | 659.250 | 658.400 | 0.028 | 0.73 |
| GH07593.1 | 150 | 658.400 | 657.170 | 0.027 | 0.73 |
| - | - | - | - | - | - |

| Node | ground (m AD) |
|---|---|
| GH07608 | 666.435 |
| GH07603 | 666.160 |
| GH07602 | 666.990 |
| GH07601 | 665.801 |
| GH07600 | 665.879 |
| GH07598 | 667.281 |
| GH07597 | 663.228 |
| GH07596 | 662.980 |
| GH07595 | 664.340 |
| GH07594 | 666.105 |
| GH07593 | 662.921 |
| GH07592 | 660.214 |

***Figure 3.3 Dry weather long section, Section B to C***

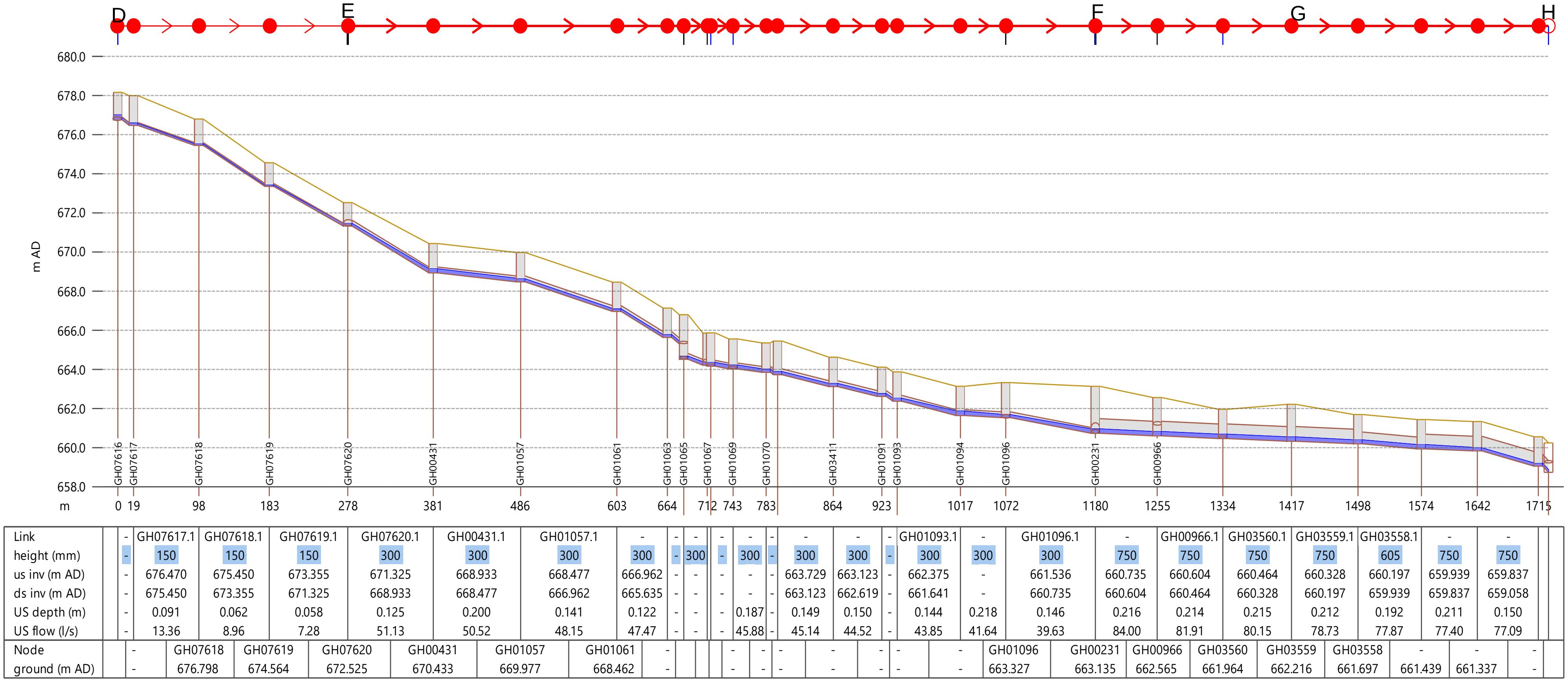



*Figure 3.4* *Dry weather long section, Section D to H*

### SPS-MV15 pump and wet well operation in dry weather

Figure 3.5 shows the pump at SPS-MV15 typically operates between 19 to 20 L/s during dry weather flows. The figure also shows that maximum pump starts per hours is 1.3. This is acceptable as it is less than the generally recommended eight starts per hour. Figure 3.6 shows the level in wet well at SPS-MV15 operates between 656.3 m AHD and 656.5 m AHD.

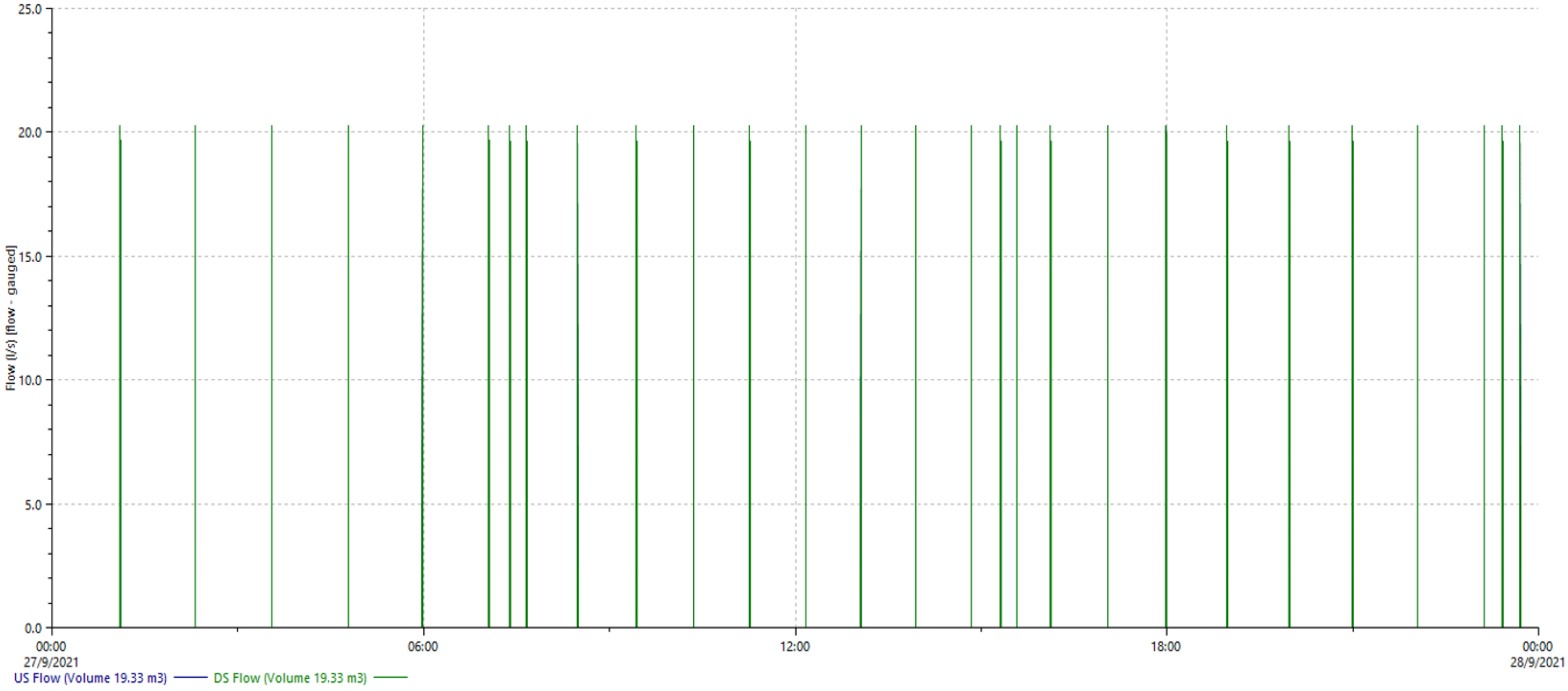



***Figure 3.5 SPS-MV15 operation, during dry weather flows***

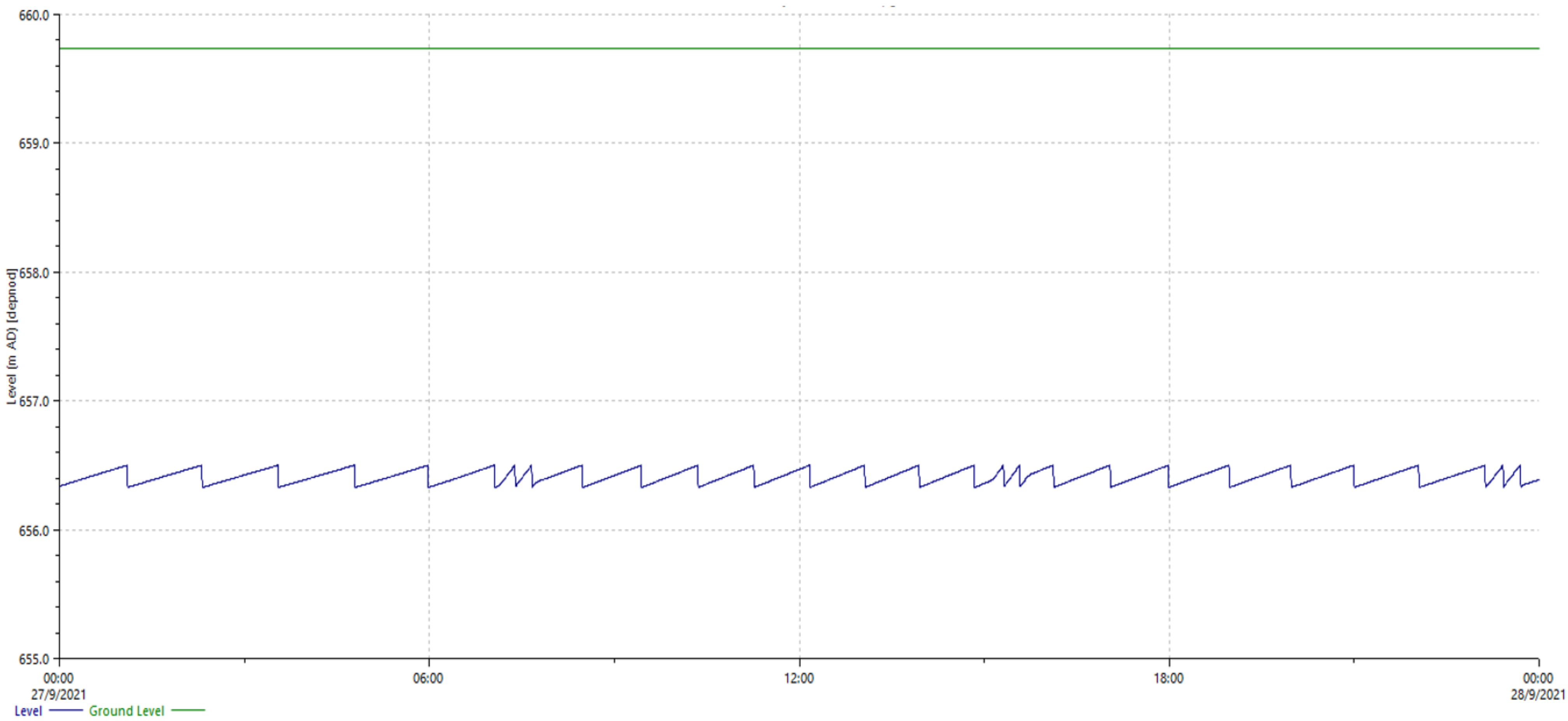



***Figure 3.6*** ***SPS-MV15 wet well operation, dry weather flows***

### Emergency storage at SPS-MV15 wet well during dry weather flows

It is a requirement that all wet wells within the wastewater network can store a minimum of 8-hours of dry weather flows above the pump switch on level (656.5 m AHD). This requirement was tested on the wet well at SPS-MV15 with the inclusion of the flows from the proposed facility. To test the storage at SPS-MV15, the pump was turned off to allow the wet well to fill up with dry weather flows. For this test, the pump operation was switched off at 7:00 AM.

Figure 3.7 shows the change in level (m AHD) within the wet well as it begins to fill up with dry weather flows. The figure shows that the wet well reaches its capacity at 9:00 AM the next day, therefore the pump has over 24 hours of dry weather storage with the inclusion of the proposed facility's flows.

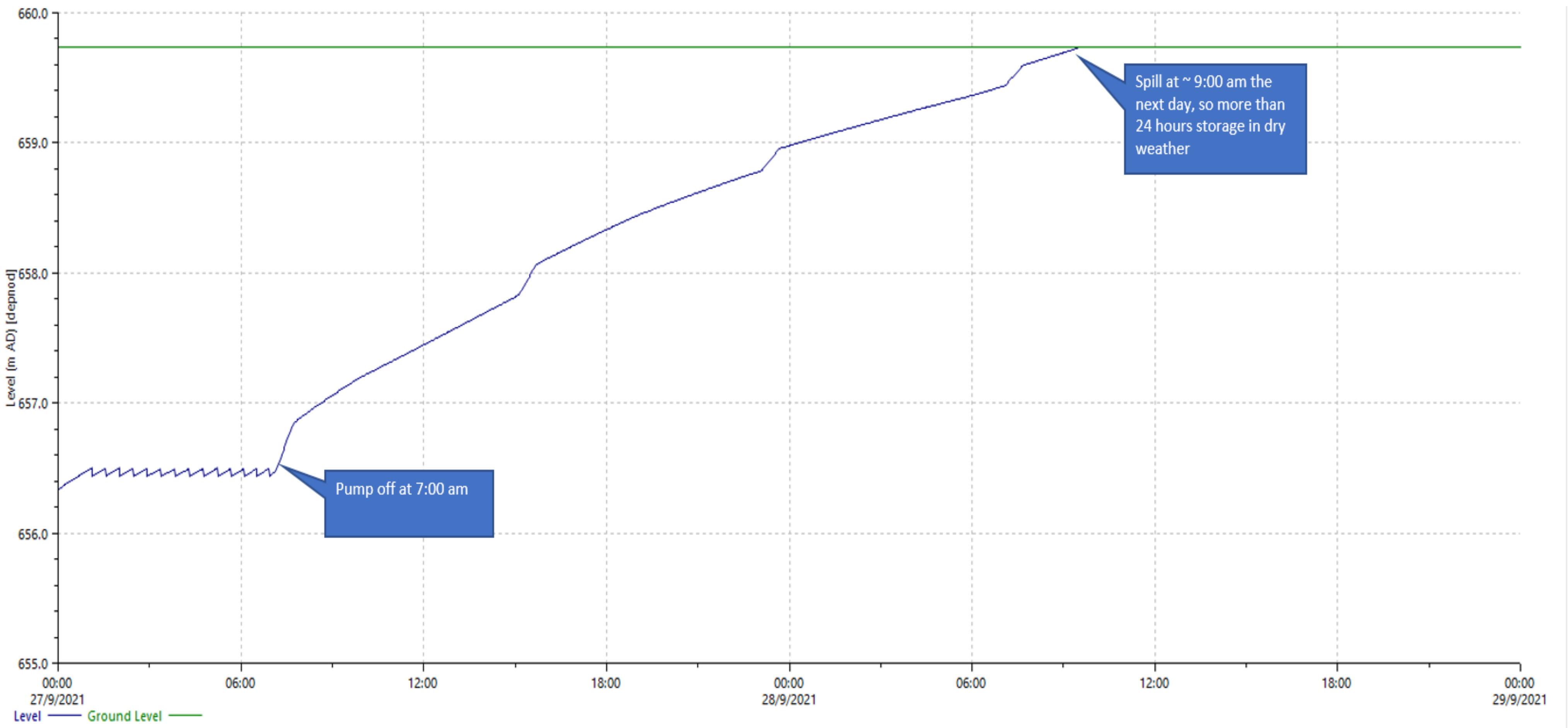



*Figure 3.7* *Volume within wet well SPS-MV15*

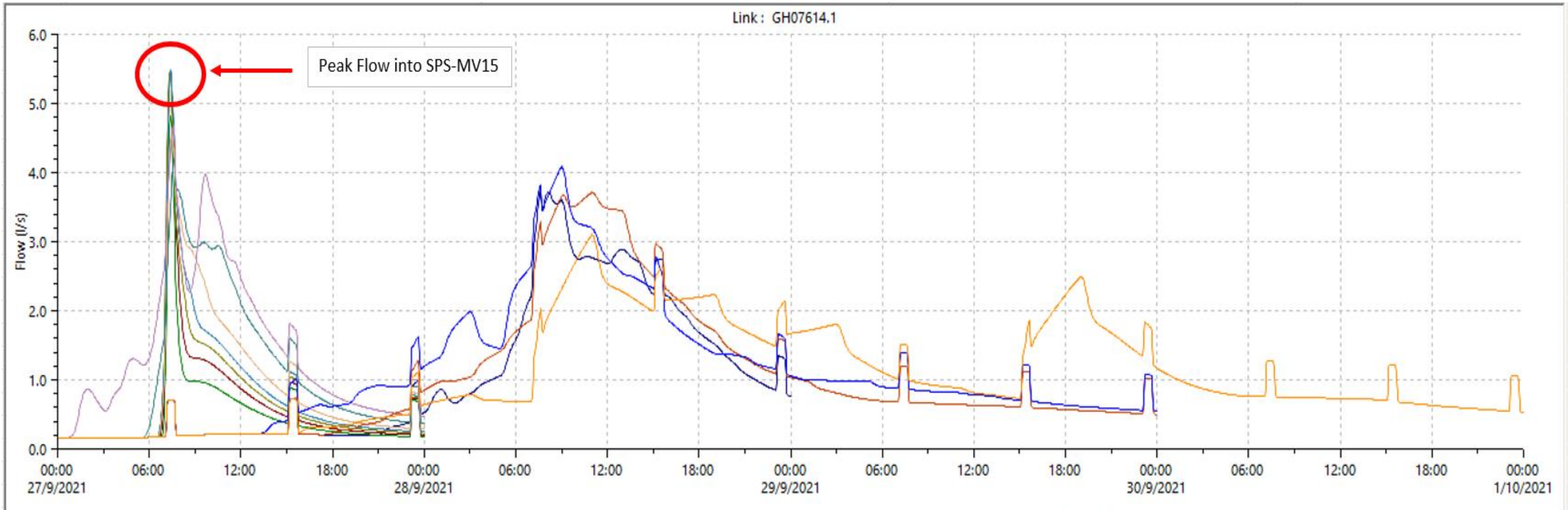
### 3.6.2 Wet weather performance

**SPS-MV15 pump and wet well capacity during wet weather events**

Figure 3.8 shows the predicted inflows into SPS-MV15 during the wet weather events with the addition of the proposed facility's flows. The flows in Figure 3.8 include the wet weather allowance for inflow/infiltration in the proposed DN225 main (section A-B, Figure 3.1).

The peak discharge into SPS-MV15 is 5.48 L/s for the 1 in 2-year 2-hour storm (highlighted in Figure 3.8). Figure 3.9 shows the pump operation at SPS-MV15 during the peak flow from the 1 in 2-year 2-hour storm. From the figure the pump is operating around 20 L/s (within its normal operation range). Hence, the pump can accommodate the peak wet weather flow of 5.48 L/s.

Figure 3.10 shows the level in the SPS-MV15 wet-well during the critical 1 in 2-year 2-hour storm. From the figure it can been seen that the wet well remains within its normal range between 656.3 m AHD and 656.5 m AHD. Hence, the wet-well can accommodate the peak wet weather flow of 5.48 L/s.




| | Flow (l/s) | | |
|---|---|---|---|
| | Min | Max | Volume (m3) |
| ...Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-24.0 | 0.154 | 3.775 | 181.841 |
| ...>Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-0.5 | 0.154 | 4.827 | 43.130 |
| ...>Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-1.0 | 0.154 | 5.426 | 53.914 |
| ...>Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-1.5 | 0.154 | 5.449 | 61.815 |
| ...>Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-2.0 | 0.154 | 5.481 | 68.102 |
| ...>Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-3.0 | 0.154 | 4.559 | 77.999 |
| ...>Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-6.0 | 0.154 | 4.199 | 98.610 |
| ...Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2yr_M2-12.0 | 0.163 | 4.682 | 126.452 |
| ...Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2 Yr_M2-36.0 | 0.154 | 3.716 | 266.320 |
| ...Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2 Yr_M2-48.0 | 0.154 | 4.097 | 292.317 |
| ...Simulations>WWF 2025 No Upgrades>2025 - No Upgrades 2 Yr M2-72.0 | 0.154 | 3.109 | 362.054 |

***Figure 3.8*** ***Wet weather inflows into SPS-MV15***

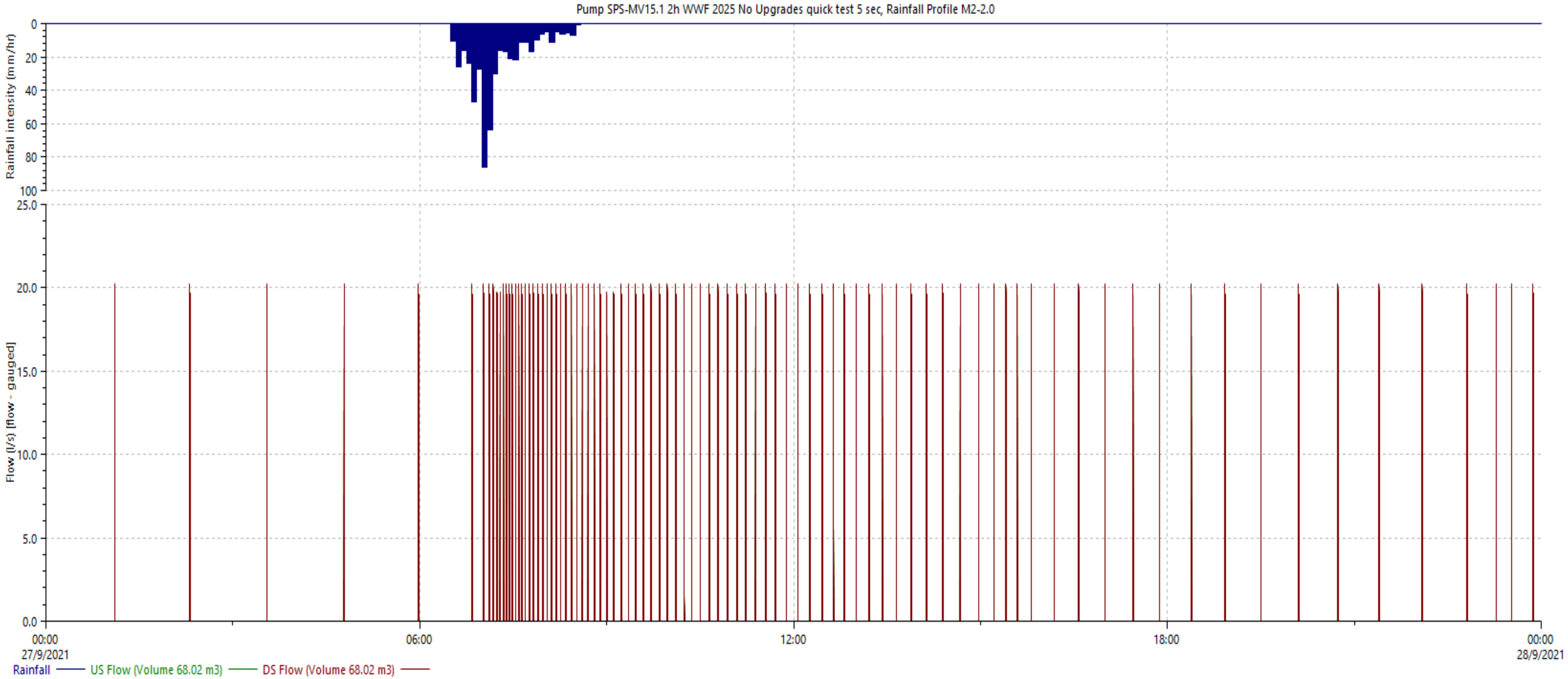



*Figure 3.9* *SPS-MV15 pump operation during the 1 in 2-year 2-hour storm*

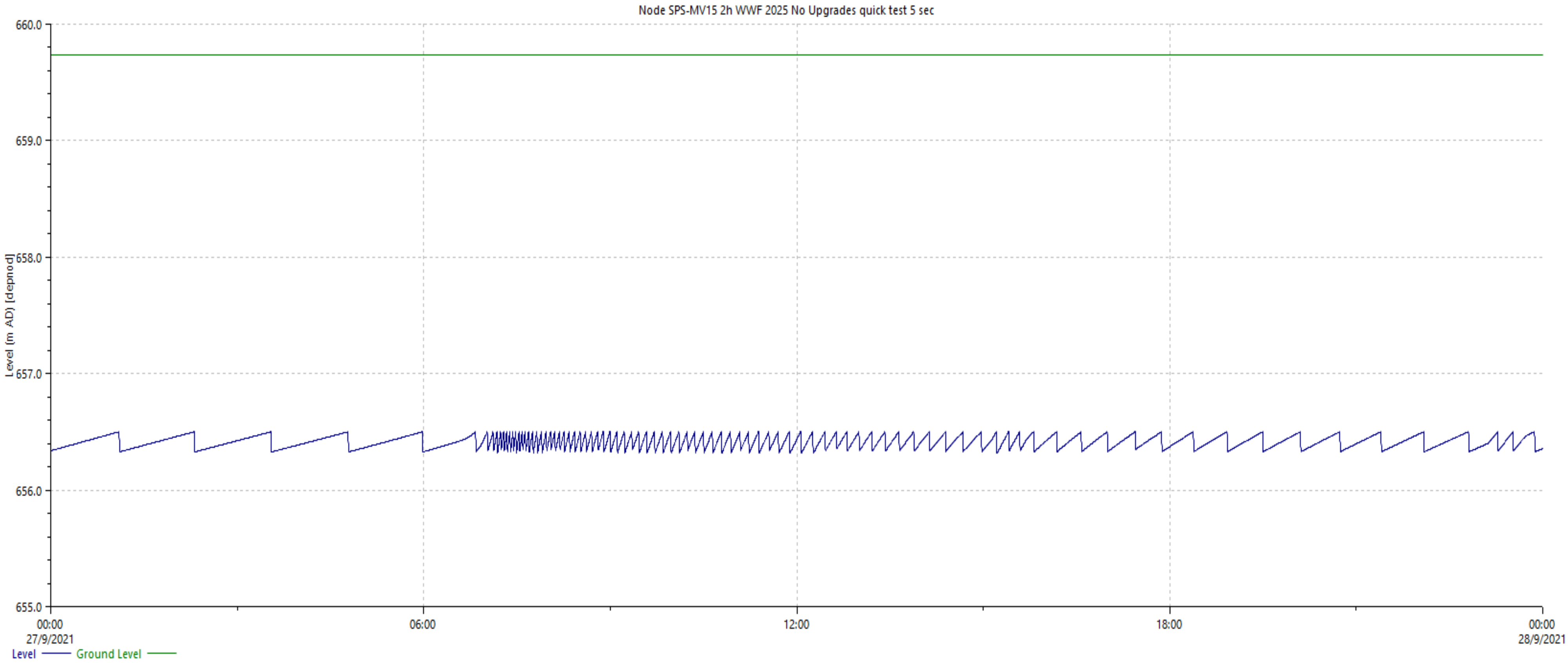
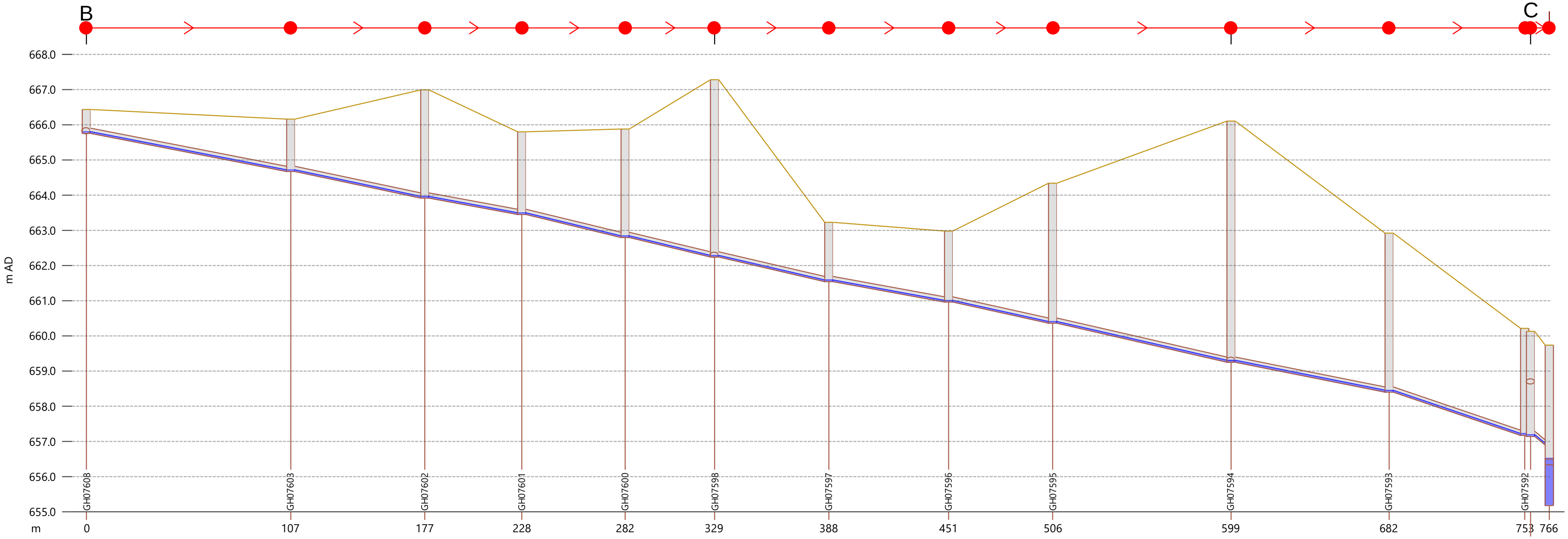



*Figure 3.10 SPS-MV15 wet well operation during the 1 in 2-year 2-hour storm*

### Wet weather overflows

Figure 3.11, Figure 3.12 and Figure 3.13 show the maximum water depth within the pipes for the critical storm event. For the section B-C in Figure 3.1, the critical storm event is the 1 in 2-year 2-hour storm. This long section is present in Figure 3.11. For section D-H (Figure 3.1) the critical storms are the 1 in 2-year 2-hour storm and 1 in 2-year 48-hour storm. Figure 3.12 shows the long section for the 1 in 2-year 2-hour storm, while Figure 3.13 shows the long section for the 1 in 2-year 48-hour storm.

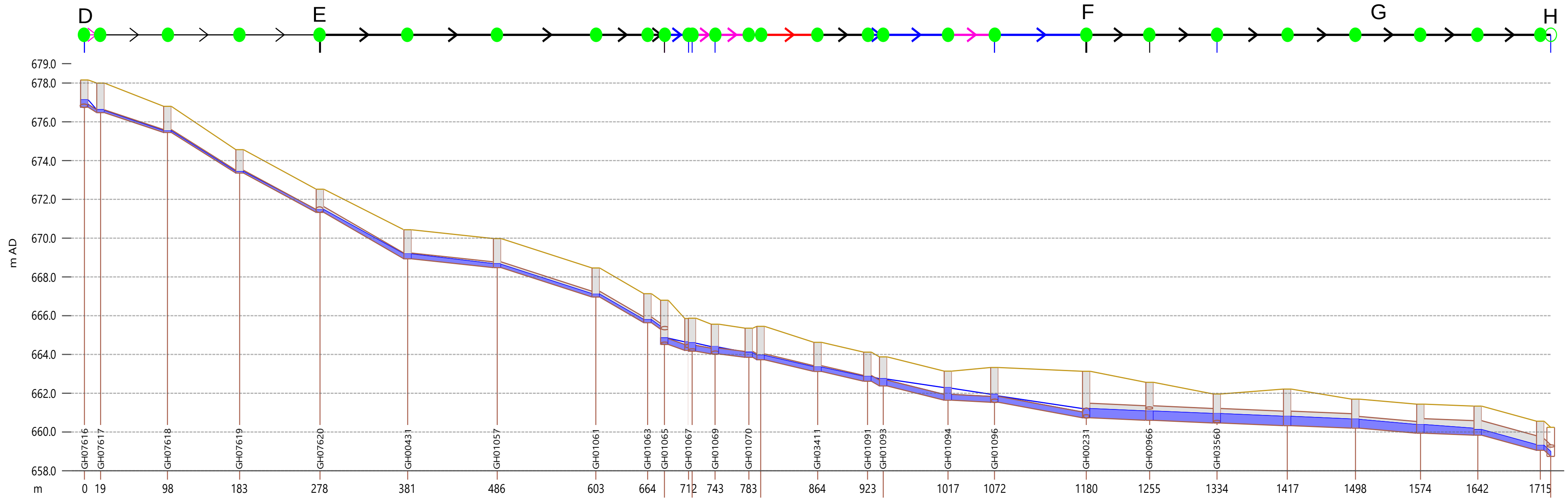
The sections labelled in Figure 3.1 are shown in Figure 3.11, Figure 3.12 and Figure 3.13. The figures show that there are no wet weather overflows for the manholes between section B-H (Figure 3.1) for all storm events with the inclusion of the proposed facility's flows on the existing system.




| Link | GH07608.1 | GH07603.1 | GH07602.1 | GH07601.1 | GH07600.1 | GH07598.1 | GH07597.1 | GH07596.1 | GH07595.1 | GH07594.1 | GH07593.1 | - |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| height (mm) | 150 | 150 | 150 | 150 | 150 | 150 | 150 | 150 | 150 | 150 | 150 | - |
| us inv (m AD) | 665.755 | 664.670 | 663.915 | 663.450 | 662.795 | 662.240 | 661.540 | 660.960 | 660.360 | 659.250 | 658.400 | - |
| ds inv (m AD) | 664.670 | 663.915 | 663.450 | 662.795 | 662.240 | 661.540 | 660.960 | 660.360 | 659.250 | 658.400 | 657.170 | - |
| US depth (m) | 0.046 | 0.046 | 0.047 | 0.044 | 0.045 | 0.045 | 0.047 | 0.045 | 0.045 | 0.054 | 0.048 | - |
| US flow (l/s) | 3.02 | 3.01 | 3.01 | 3.00 | 3.00 | 3.00 | 3.00 | 2.99 | 2.99 | 4.31 | 4.30 | - |

| Node | GH07608 | GH07603 | GH07602 | GH07601 | GH07600 | GH07598 | GH07597 | GH07596 | GH07595 | GH07594 | GH07593 | GH07592 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ground (m AD) | 666.435 | 666.160 | 666.990 | 665.801 | 665.879 | 667.281 | 663.228 | 662.980 | 664.340 | 666.105 | 662.921 | 660.214 |

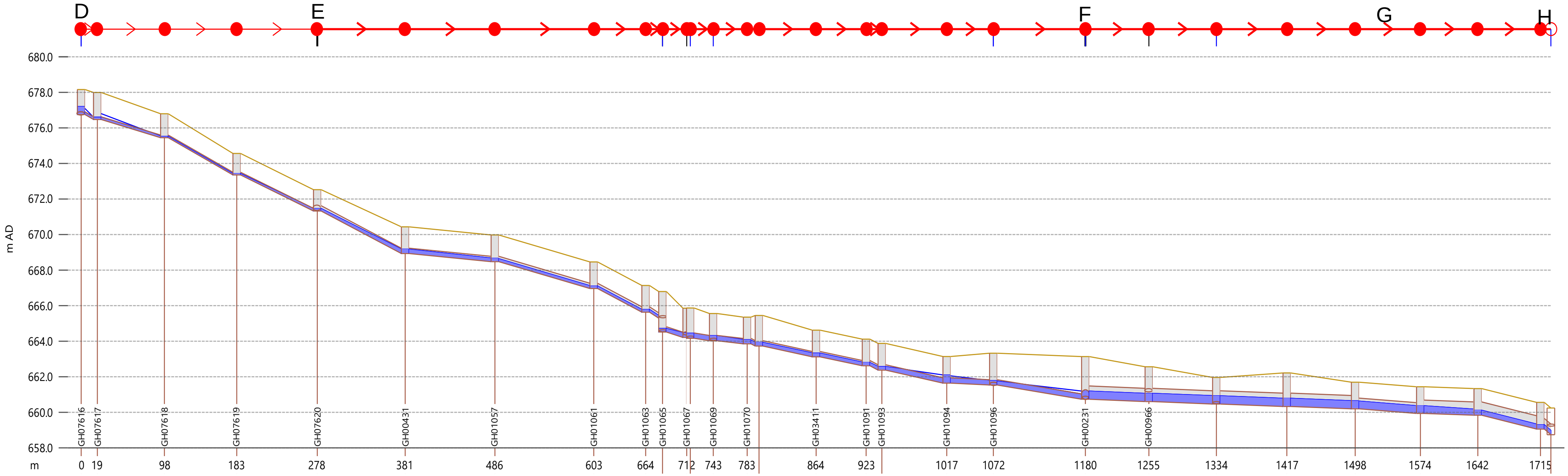
*Figure 3.11* *B-C long section for 1 in 2-year 2-hour storm*




| Link | | - | GH07617.1 | GH07618.1 | GH07619.1 | GH07620.1 | GH00431.1 | GH01057.1 | - | - | - | - | - | - | GH01071.1 | - | - | GH01093.1 | - | GH01096.1 | GH00231.1 | GH00966.1 | GH03560.1 | GH03559.1 | GH03558.1 | GH03557.1 | GH03555.1 | - |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| height (mm) | | - | 150 | 150 | 150 | 300 | 300 | 300 | 300 | - | 300 | 300 | 300 | - | 300 | 300 | - | 300 | 300 | 300 | 750 | 750 | 750 | 750 | 605 | 750 | 750 | - |
| us inv (m AD) | | - | 676.470 | 675.450 | 673.355 | 671.325 | 668.933 | 668.477 | 666.962 | - | - | - | - | - | 663.729 | 663.123 | - | 662.375 | 661.641 | 661.536 | 660.735 | 660.604 | 660.464 | 660.328 | 660.197 | 659.939 | 659.837 | - |
| ds inv (m AD) | | - | 675.450 | 673.355 | 671.325 | 668.933 | 668.477 | 666.962 | 665.635 | - | - | - | - | - | 663.123 | 662.619 | - | 661.641 | 661.536 | 660.735 | 660.604 | 660.464 | 660.328 | 660.197 | 659.939 | 659.837 | 659.058 | - |
| US depth (m) | | - | 0.100 | 0.074 | 0.073 | 0.141 | 0.244 | 0.164 | 0.141 | - | - | - | 0.348 | - | 0.214 | 0.217 | - | 0.356 | 0.615 | 0.328 | 0.469 | 0.472 | 0.475 | 0.472 | 0.445 | 0.435 | 0.283 | - |
| US flow (l/s) | | - | 16.82 | 12.68 | 11.57 | 64.34 | 63.83 | 62.70 | 62.46 | - | - | - | 78.78 | - | 78.60 | 78.45 | - | 75.82 | 74.26 | 75.40 | 324.39 | 323.57 | 323.12 | 322.34 | 321.77 | 321.59 | 321.50 | - |

| Node | | - | GH07618 | GH07619 | GH07620 | GH00431 | GH01057 | GH01061 | - | - | | | - | - | - | - | - | - | GH01094 | GH01096 | GH00231 | GH00966 | GH03560 | GH03559 | GH03558 | GH03557 | GH03555 | - |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ground (m AD) | | - | 676.798 | 674.564 | 672.525 | 670.433 | 669.977 | 668.462 | - | - | | | - | - | - | 664.623 | - | - | 663.141 | 663.327 | 663.135 | 662.565 | 661.964 | 662.216 | 661.697 | 661.439 | 661.337 | - |

***Figure 3.12 D-H long section for 1 in 2-year 2-hour storm***




| Link | height (mm) | us inv (m AD) | ds inv (m AD) | US depth (m) | US flow (l/s) |
|---|---|---|---|---|---|
| - | - | - | - | - | - |
| GH07617.1 | 150 | 676.470 | 675.450 | 0.333 | 22.37 |
| GH07618.1 | 150 | 675.450 | 673.355 | 0.071 | 11.66 |
| GH07619.1 | 150 | 673.355 | 671.325 | 0.070 | 10.58 |
| GH07620.1 | 300 | 671.325 | 668.933 | 0.141 | 63.79 |
| GH00431.1 | 300 | 668.933 | 668.477 | 0.240 | 63.06 |
| GH01057.1 | 300 | 668.477 | 666.962 | 0.162 | 61.24 |
| - | 300 | 666.962 | 665.635 | 0.139 | 60.77 |
| - | - | - | - | - | - |
| - | 300 | - | - | - | - |
| - | 300 | - | - | - | - |
| - | 300 | - | - | 0.274 | 72.17 |
| - | - | - | - | - | - |
| GH01071.1 | 300 | 663.729 | 663.123 | 0.199 | 71.61 |
| - | 300 | 663.123 | 662.619 | 0.202 | 71.32 |
| - | - | - | - | - | - |
| GH01093.1 | 300 | 662.375 | 661.641 | 0.194 | 71.04 |
| - | 300 | 661.641 | 661.536 | 0.414 | 67.74 |
| GH01096.1 | 300 | 661.536 | 660.735 | 0.215 | 69.43 |
| GH00231.1 | 750 | 660.735 | 660.604 | 0.455 | 310.80 |
| GH00966.1 | 750 | 660.604 | 660.464 | 0.457 | 309.98 |
| GH03560.1 | 750 | 660.464 | 660.328 | 0.461 | 309.35 |
| GH03559.1 | 750 | 660.328 | 660.197 | 0.456 | 308.32 |
| GH03558.1 | 605 | 660.197 | 659.939 | 0.430 | 307.50 |
| GH03557.1 | 750 | 659.939 | 659.837 | 0.424 | 307.22 |
| GH03555.1 | 750 | 659.837 | 659.058 | 0.277 | 307.09 |
| - | - | - | - | - | - |

| Node | ground (m AD) |
|---|---|
| - | - |
| GH07618 | 676.798 |
| GH07619 | 674.564 |
| GH07620 | 672.525 |
| GH00431 | 670.433 |
| GH01057 | 669.977 |
| GH01061 | 668.462 |
| - | - |
| - | - |
| - | - |
| - | - |
| - | - |
| - | - |
| - | 664.623 |
| - | - |
| - | - |
| GH01094 | 663.141 |
| GH01096 | 663.327 |
| GH00231 | 663.135 |
| GH00966 | 662.565 |
| GH03560 | 661.964 |
| GH03559 | 662.216 |
| GH03558 | 661.697 |
| GH03557 | 661.439 |
| GH03555 | 661.337 |
| - | - |

***Figure 3.13*** ***D-H long section for 1 in 2-year 48-hour storm***

The existing system performance results showed no surcharging of the gravity system downstream of the proposed facility site. Manhole (MH) overflows are predicted in parts of the Moss Vale catchment however these are hydraulically independent from the network downstream from the development connection point. The 'ID' within the rectangular labels in Figure 3.14 indicate the locality of the spills. The flow path from the facility to the WWTP is shown as the red line in Figure 3.14. MH GH03605 (highlighted in Figure 3.14) has the largest spill volume over all the wet weather events.

Table 3.3 details the predicted MH overflow volumes for all design storm durations. The 2-hour duration has been deemed the critical event due to the largest MH overflow volume and highest peak flows in the system.

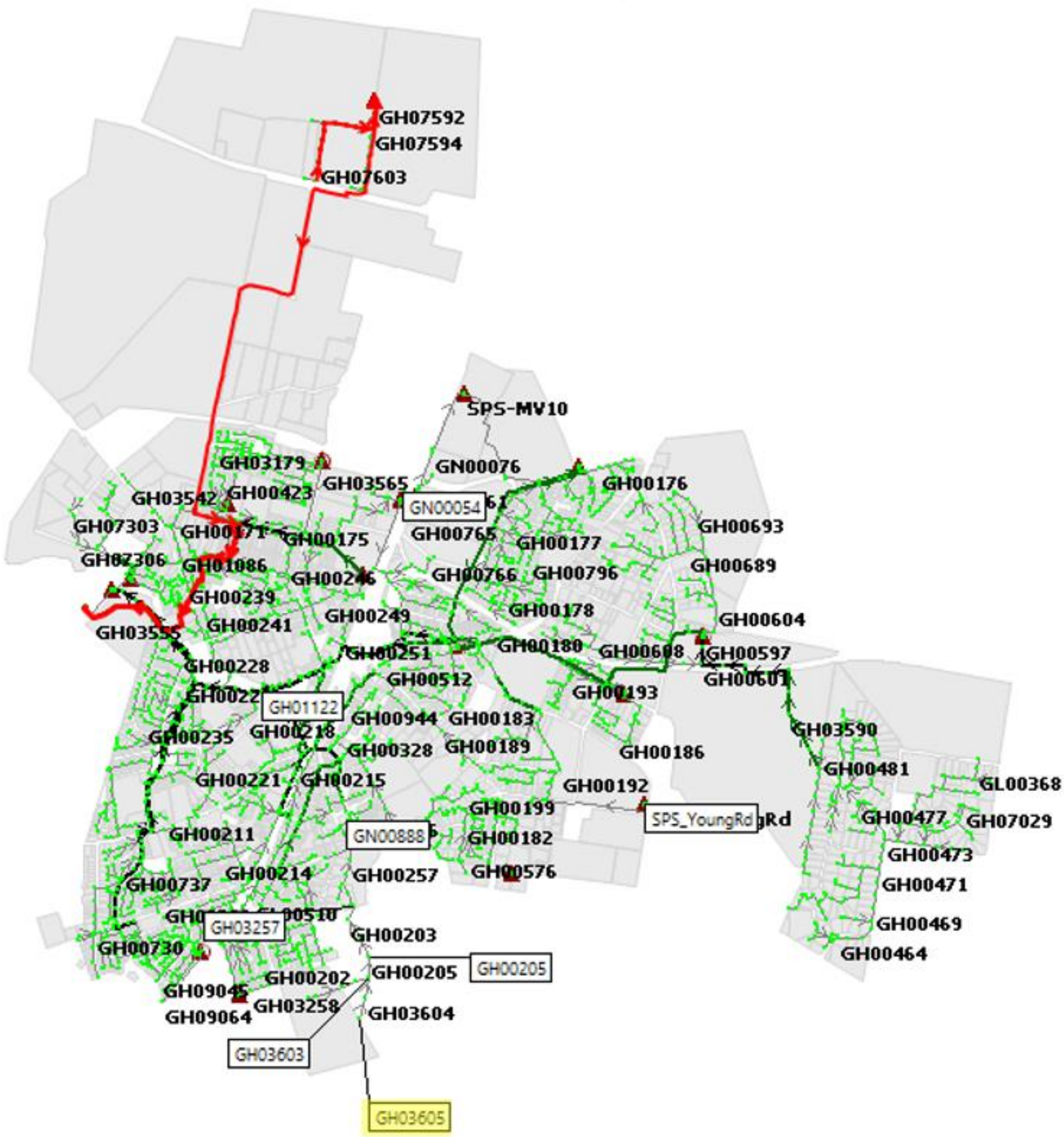



***Figure 3.14 Location of wet weather overflows***

*Table 3.3 MH overflows for all design storms*

| Node ID | 2yr - 24.0 hr (m3) | 2yr - 0.5 hr (m3) | 2yr - 1.0 hr (m3) | 2yr - 1.5 hr (m3) | 2yr - 2.0 hr (m3) | 2yr - 3.0 hr (m3) | 2yr - 6.0 hr (m3) | 2yr - 12.0 hr (m3) | 2yr - 36.0 hr (m3) | 2yr - 48.0 hr (m3) |
|---|---|---|---|---|---|---|---|---|---|---|
| GH00205 | 21.8 | 3.6 | 5.4 | 6.7 | 7.9 | 9.8 | 13.5 | 16 | 24.4 | 24.9 |
| GH01122 | 0 | 0 | 4.4 | 17.1 | 35.9 | 13.6 | 21 | 0 | 0 | 0 |
| GH03257 | 115.2 | 23.1 | 28.5 | 32.2 | 35.6 | 41 | 56.1 | 72.5 | 126.6 | 184.4 |
| GH03603 | 0 | 0 | 0.2 | 0.3 | 0.3 | 0 | 0 | 0 | 0 | 0 |
| GH03605 | 829.9 | 201.8 | 293.3 | 354.8 | 401.7 | 473.2 | 609.6 | 668.7 | 920.8 | 837.8 |
| GN00054 | 0.3 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 12 | 7.1 |
| GN00888 | 10.4 | 1.3 | 3.6 | 5.1 | 6.5 | 8.1 | 11.5 | 12.3 | 15.1 | 12.4 |
| SPS_YoungRd | 67.2 | 0 | 0 | 0 | 0 | 0 | 0 | 5.4 | 100.5 | 89 |
| **Total** | **1044.8** | **229.8** | **335.4** | **416.2** | **487.9** | **545.7** | **711.7** | **774.9** | **1199.4** | **1155.6** |

### 3.6.3 Capacity at WWTP

The current capacity of the WWTP is 9,000 EP. From Council advice the WWTP is nearing capacity in 2021. There are plans to upgrade the WWTP to 18,000 EP in 2025/2026. The wastewater flows from the proposed facility is equivalent to 69 EP. This represents 0.8% of the current capacity of the WWTP. Therefore, the additional load from the proposed facility is going have an insignificant impact on the WWTP.

# 4. Water supply system

## 4.1 Introduction

This section will detail the water modelling undertaken and will include the following:

- WSC design standards
- Model configurations to incorporate the new facility
- System performance with and without the inclusion of the facility

## 4.2 WSC water network design standards

Table 4.1, Table 4.2 and [1] Velocities in the reticulation network < 2 m/s. Velocities exceeding this value should be approved by Council. For firefighting, velocities up to 4.0 m/s are acceptable. The modelling found no velocities greater than 2 m/s.

[2] These are target values and can be exceeded in certain circumstances in consultation with Council.

Table 4.3, outline the Council's water design standards.

*Table 4.1 Pressure Design Standards*

| Design Standard | Value | Comments |
|---|---|---|
| Minimum pressure | 12 m | |
| Max pressure (less than) | 90 m | |
| Fire flow | 10 L/s | at 15 m residual pressure in the water main |

*Table 4.2 Velocity and Head loss Design Standards*

| Design Standard | Value | Comments |
|---|---|---|
| Maximum velocity in mains[1] | 2 m/s | |
| Target maximum head loss in mains [2] | 5 m/km | for reticulation mains |
| Target maximum head loss in mains [2] | 3 m/km | for trunk mains |

[1] Velocities in the reticulation network < 2 m/s. Velocities exceeding this value should be approved by Council. For firefighting, velocities up to 4.0 m/s are acceptable. The modelling found no velocities greater than 2 m/s.

[2] These are target values and can be exceeded in certain circumstances in consultation with Council.

*Table 4.3 Reservoir Design Standards*

| Design Standard | Value | Comments |
|---|---|---|
| Total storage | 24 hr PDD | Static criteria |
| Reserve storage at the lowest operating range | 12 hr PDD | Dynamic criteria |

## 4.3 Water demand estimates

Table 4.4 shows the water demand estimates for the proposed facility's and includes average day demand (ADD) and peak day demand (PDD). The values in Table 4.4 are from Figure 2.1 in section 2. These values have been confirmed by the proponent.

*Table 4.4 Water demand estimates for the facility [1]*

| Water Design Demands | Average Flow per day (L/day) | Average Flow per shift (L/8-hour shift) | Duration of flow per 8-hour shift | Peak Flow (L/s) |
|---|---|---|---|---|
| Admin Water ADD – Shower Flows | 3,300 | 1,100 | 35 minutes | 0.52 |
| Admin Water ADD – Sink and Toilet Flows | 2,500 | 833 | 8 hours | 0.03 |
| Process ADD | 40,500 | 13,500 | 8 hours | 0.47 |
| Total ADD | 46,300 | 15,433 | - | 1.01 |
| **Total PDD [2]** | **55,560** | **18,520** | **-** | **1.21** |

[1] These calculations assume a 24-hour a day operation.

[2] PDD/ADD ration for the proposed facility's demand is 1.2.

A PDD/ADD ratio has been included for the proposed facility's demand. Generally, industrial, and commercial demands are not given peaking factors because their demands do not increase during peak days. However, a factor has been applied to the facility to be conservative.

## 4.4 Water demand profile for the proposed facility

Figure 4.1 shows the water demand profile for the proposed facility. In the figure the process, toilet and sink demands have been represented as constant flows across the entire day. While the shower demand has been included as three 35-minute peaks. This is because the shower flows will occur when the employees are showering at the end of their shifts. Refer to section 1.4 and section 2 for further information on the admin and process flows.

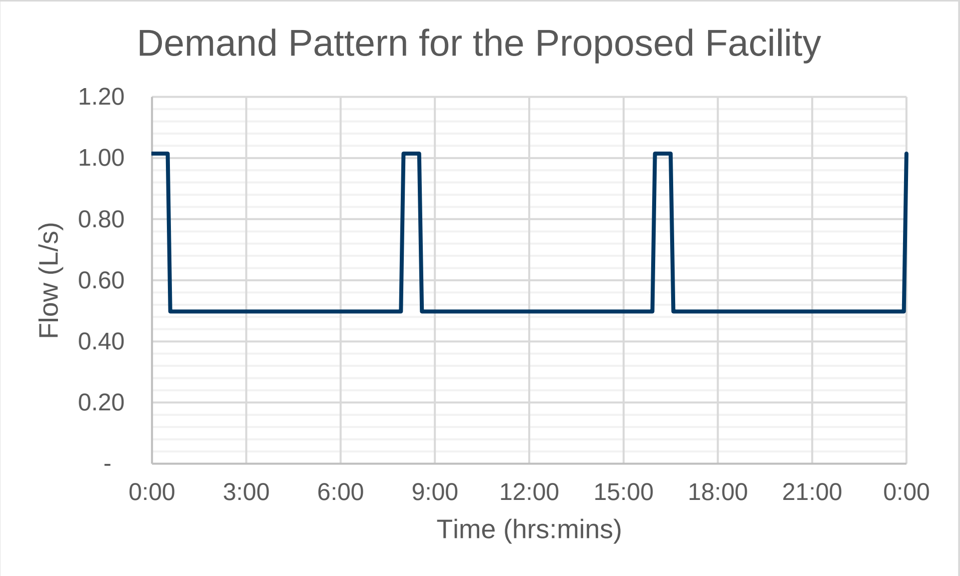



*Figure 4.1 Demand profile for the proposed facility*

## 4.5 Model configuration

It is suggested to connect the proposed facility to the water network via the existing gravity fed DN150 Asbestos Concrete (AC) main. The proposed facility will be serviced by the Hill Road Reservoir which has a total capacity of 10 ML. However, the Hill Road Reservoir can only fill to 60 percent capacity due to hydraulic restrictions in the existing network.

Figure 4.2 shows an overview of the water network as well as the flow path from the Hill Road Reservoir to the proposed facility. Figure 4.3 shows the inclusion of the proposed facility as a customer point into the model. The figure also shows the customer point demand assignment to the existing adjacent DN150 AC pipe for the 2021 network.

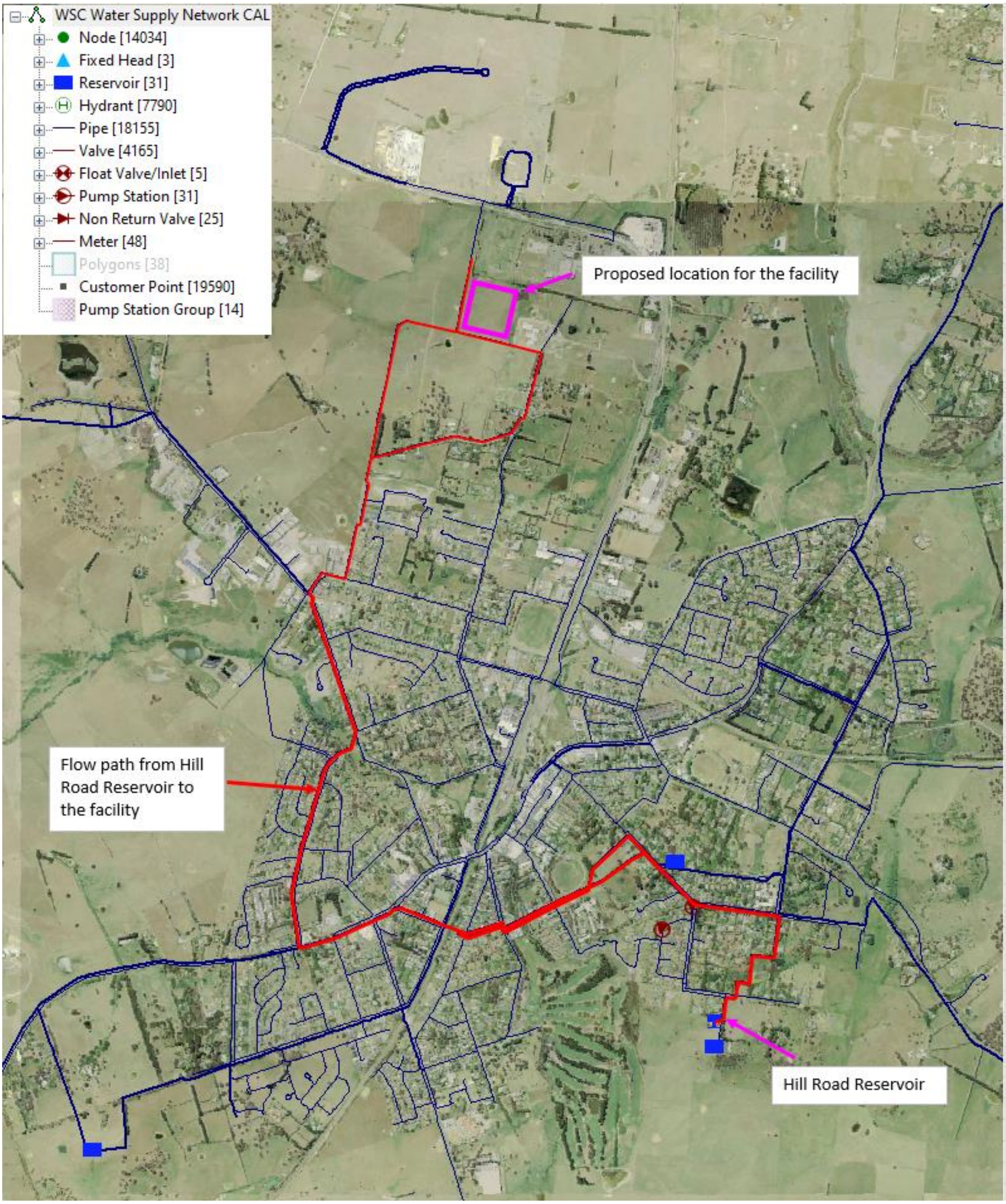



*Figure 4.2* *Overview of water network*

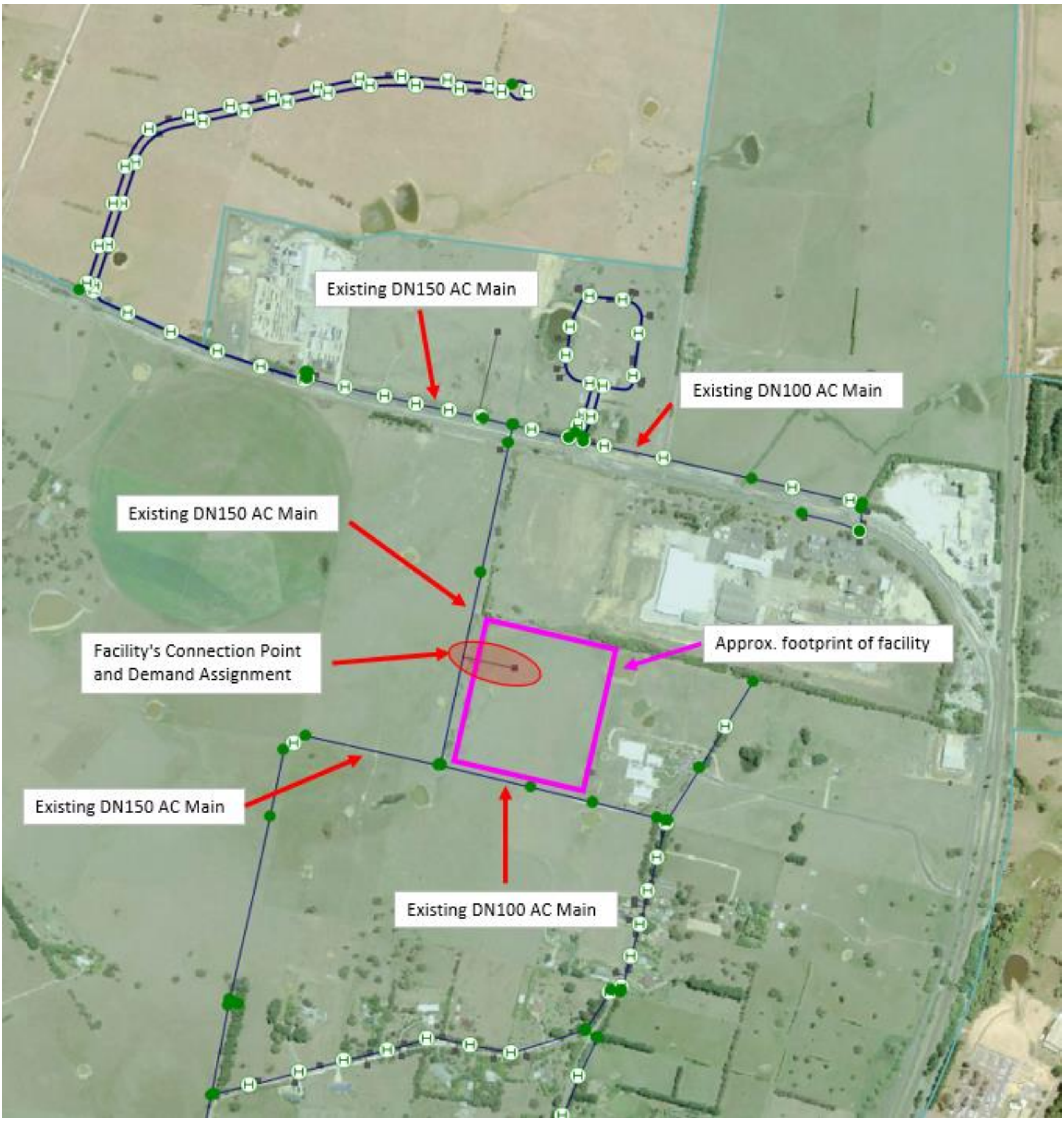



***Figure 4.3 Proposed connection of the proposed facility to existing network***

## 4.6 System performance results

### 4.6.1 Minimum pressure

Figure 4.4 shows the minimum pressure for the PDD 2021 horizon, without the proposed facility. While Figure 4.5 shows the minimum pressure PDD 2021 horizon, with the proposed facility. The figures show the minimum pressures flows during peak flows for each point. Both figures show the location of the proposed facility. The change of minimum pressure between Figure 4.4 and Figure 4.5, is shown in Figure 4.6.

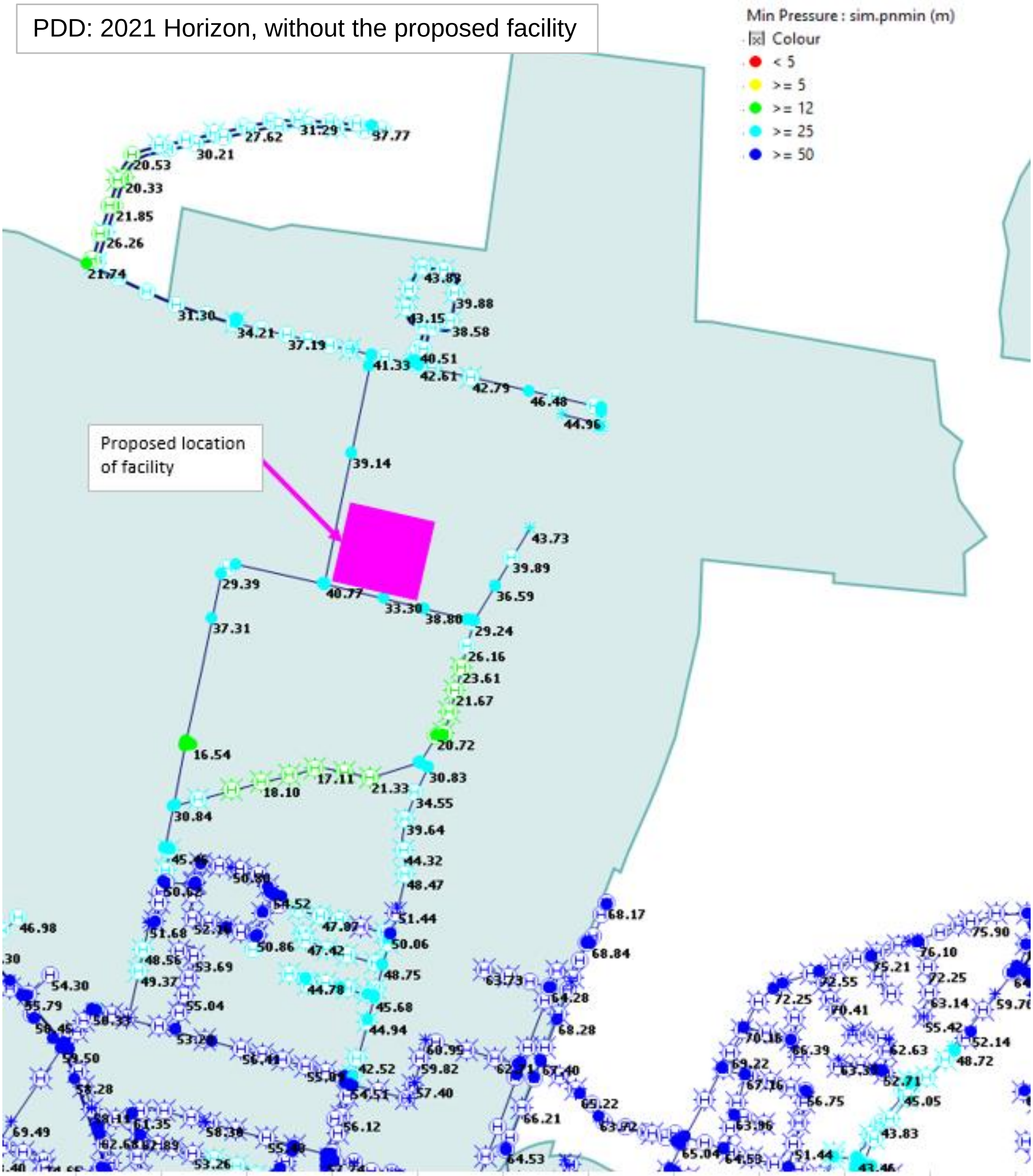



*Figure 4.4 Existing system performance without the proposed facility, minimum pressures (PDD: 2021 horizon)*

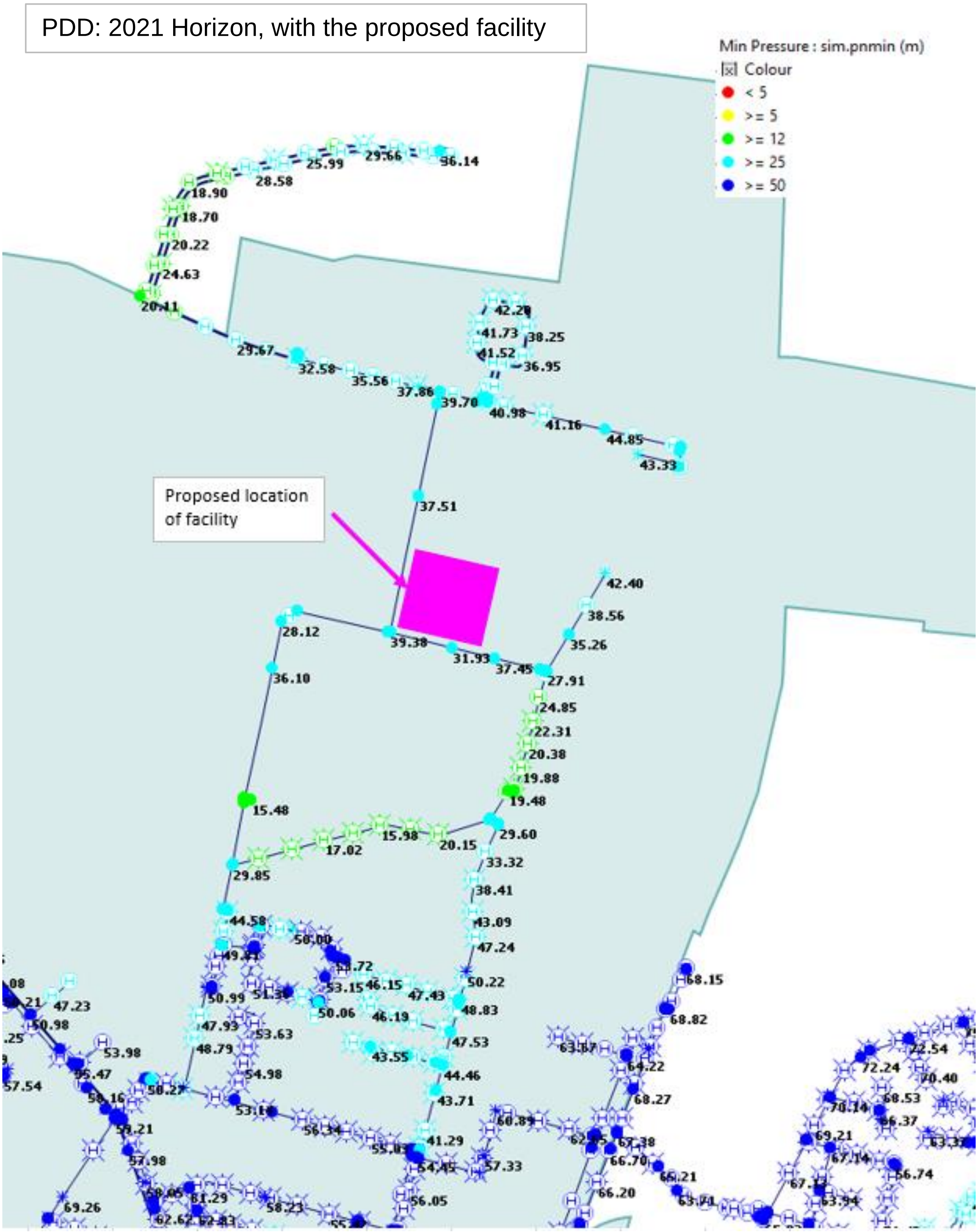



*Figure 4.5 System performance with the proposed facility, minimum pressures (PDD: 2021 horizon)*

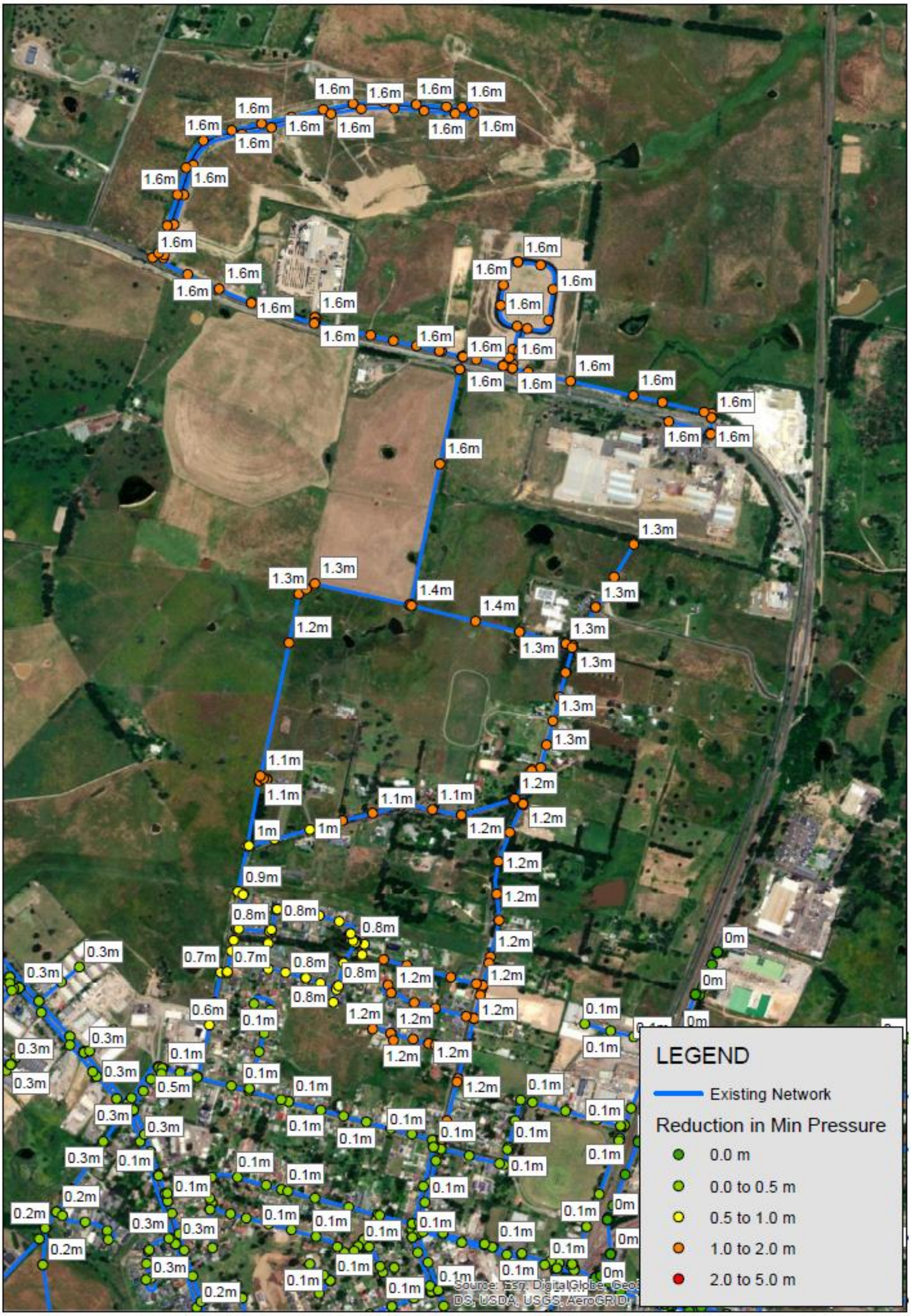



***Figure 4.6 Reduction in minimum pressure***

Figure 4.5 shows a reduction in minimum pressure of between 0.1 to 1.6 m with the inclusion of the proposed facility's demands within the network. However, no nodes fall under the minimum allowable minimum pressure of 12 m with the inclusion of the proposed facility's demand, as shown in Figure 4.5.

## 4.6.2 Maximum head losses

Figure 4.7 shows the maximum head losses for the PDD 2021 horizon, without the proposed facility. While Figure 4.8 shows the maximum head losses with for the PDD 2021 horizon, with the proposed facility. The figures show the maximum head losses during peak flows for each pipe. Both figures show the location of the proposed facility.

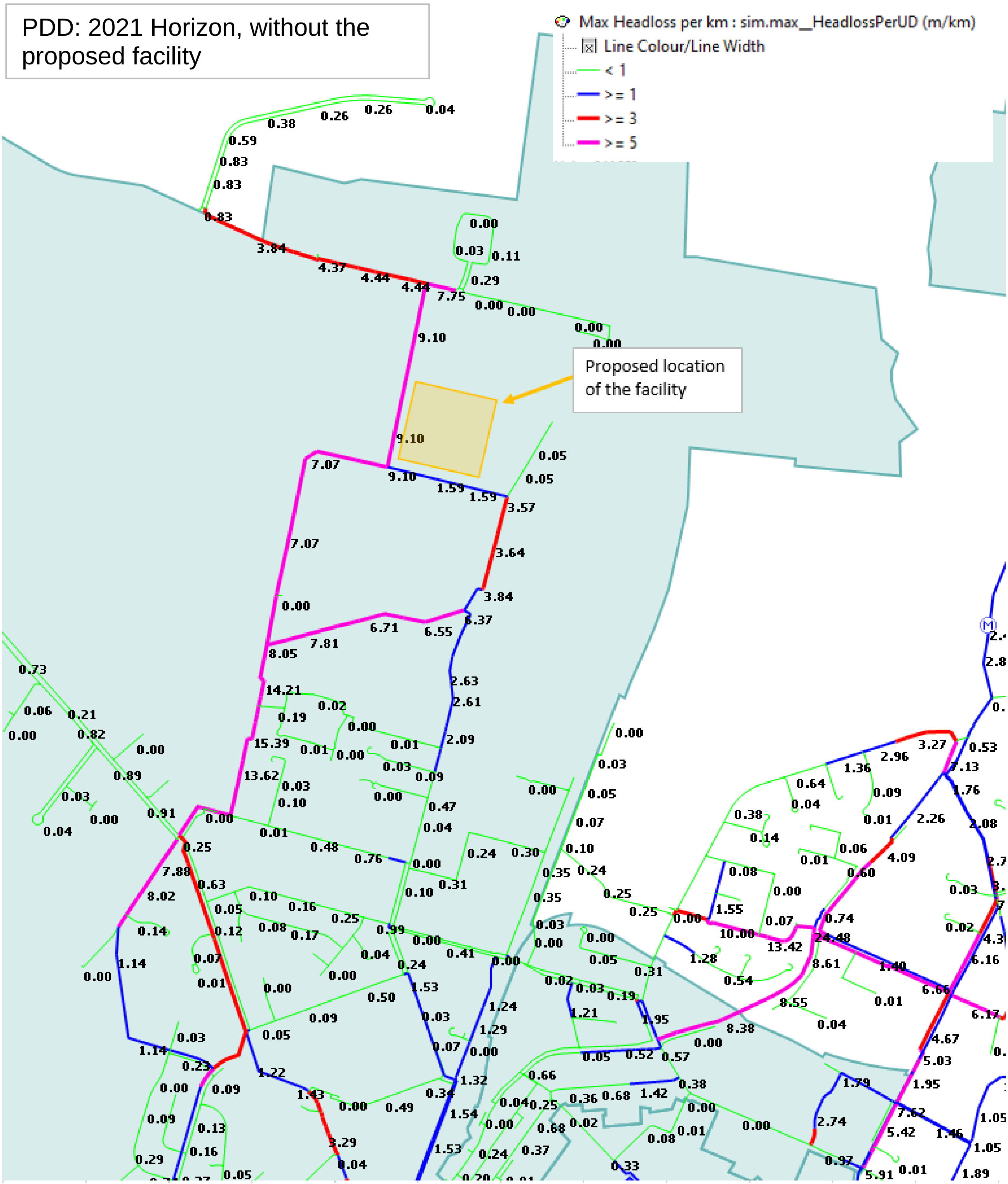



*Figure 4.7 Existing system performance without the proposed facility, maximum head losses (PDD: 2021 horizon)*

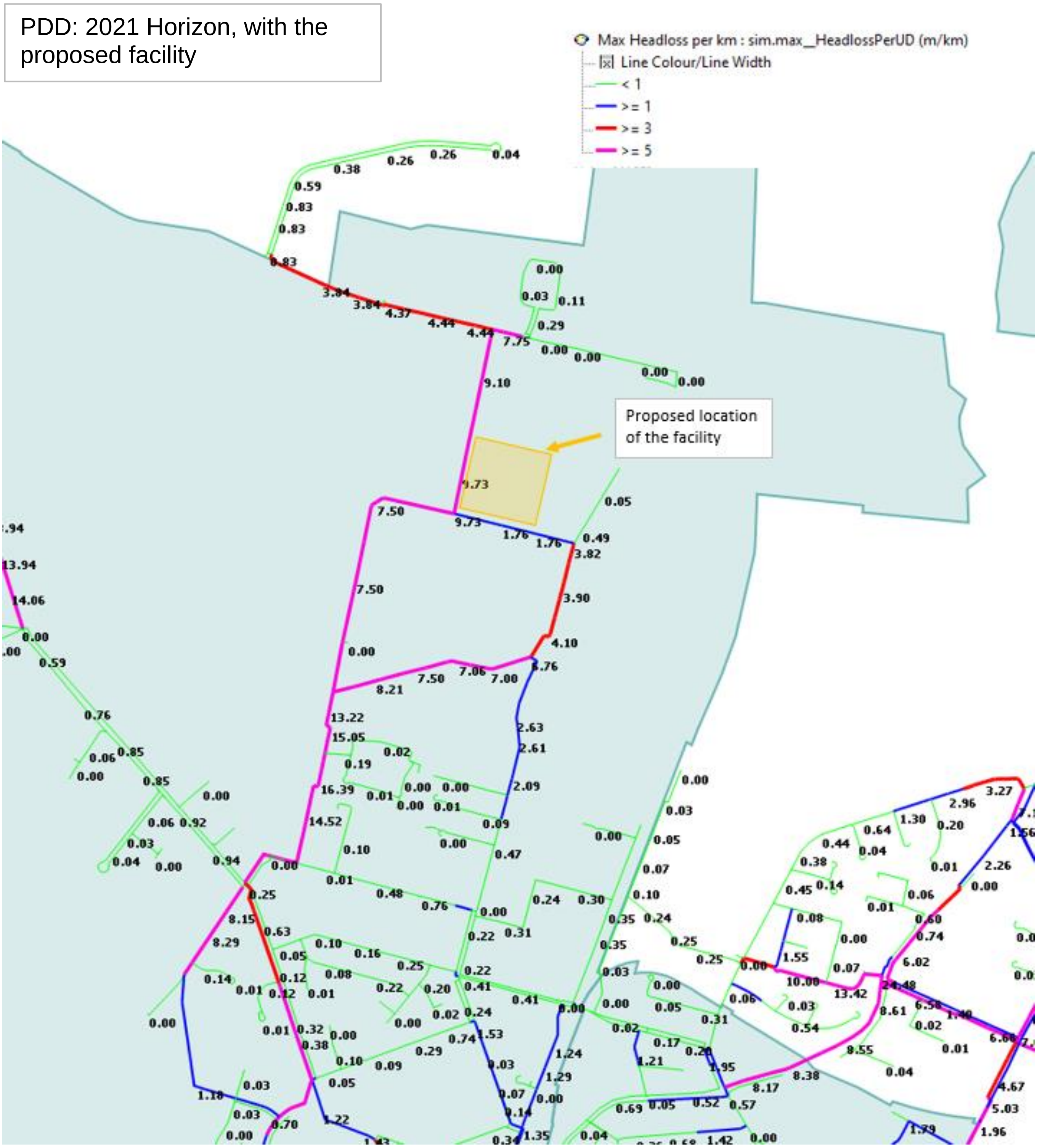



*Figure 4.8* ***System performance with the facility, maximum head losses (PDD: 2021 horizon)***

Figure 4.7 shows that there are already maximum head losses within the existing system greater than the criteria set out in Table 4.2. With the addition of the proposed facility's demand (Figure 4.8), the maximum head losses increase. To relieve these maximum head losses (greater than the criteria) the mains either need to be upsized or duplicated. However, these head loss are not great enough to cause the minimum pressures to fall below 12 m in Figure 4.5, which is the greater concern. Therefore, immediate action is not required.

## 4.6.3 Peak velocity

Figure 4.9 shows the peak velocities for the PDD 2021 horizon, without the proposed facility. While Figure 4.10 shows the peak velocities for the PDD 2021 horizon, with the proposed facility. The figures show the maximum velocities during peak flows for each pipe. Both figures show the location of the proposed facility. From reviewing both figures, peak velocities of greater than 2 m/s are not expected to occur.

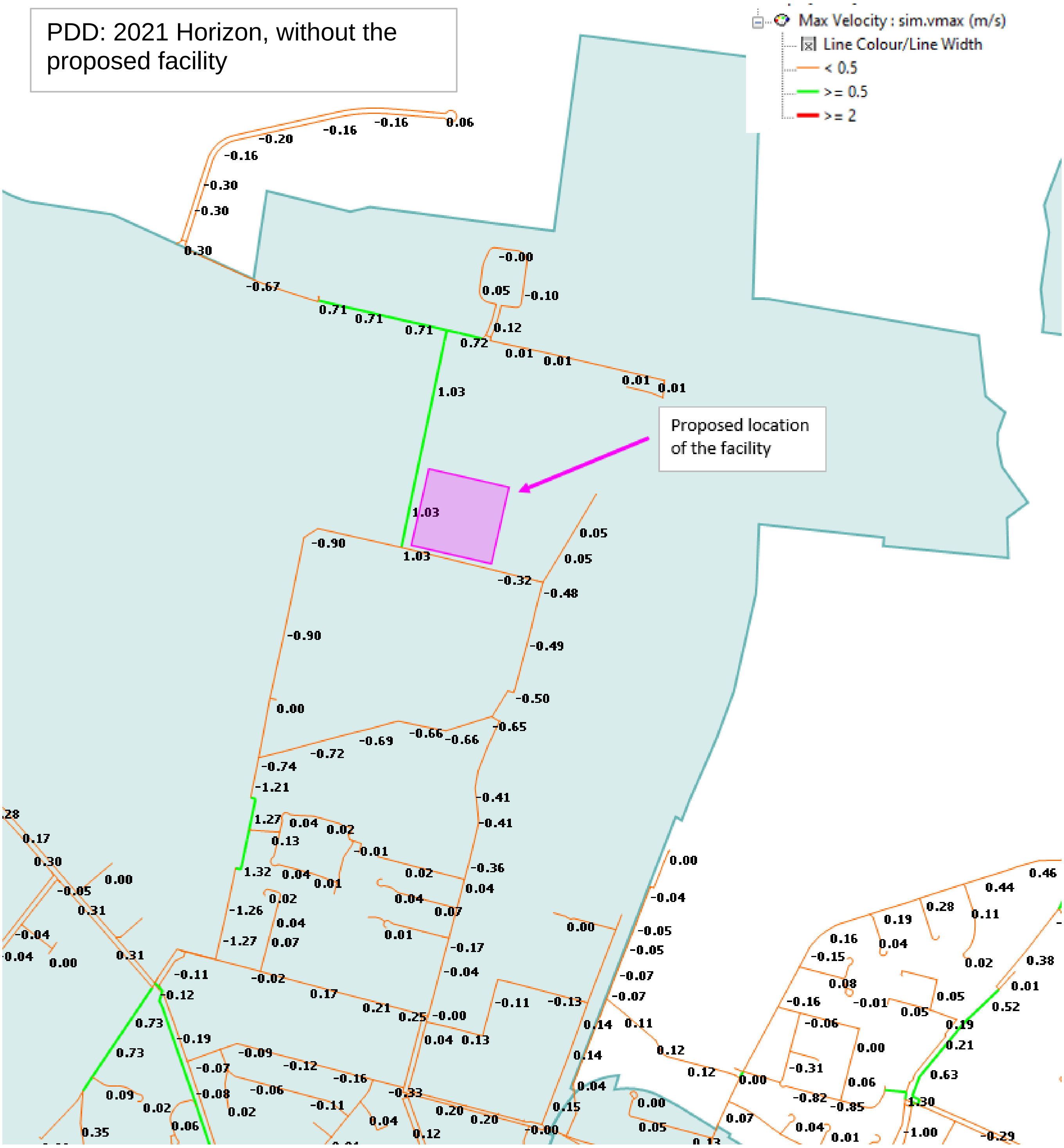



*Figure 4.9 Existing system performance without the proposed facility, peak velocities (PDD: 2021 horizon)*

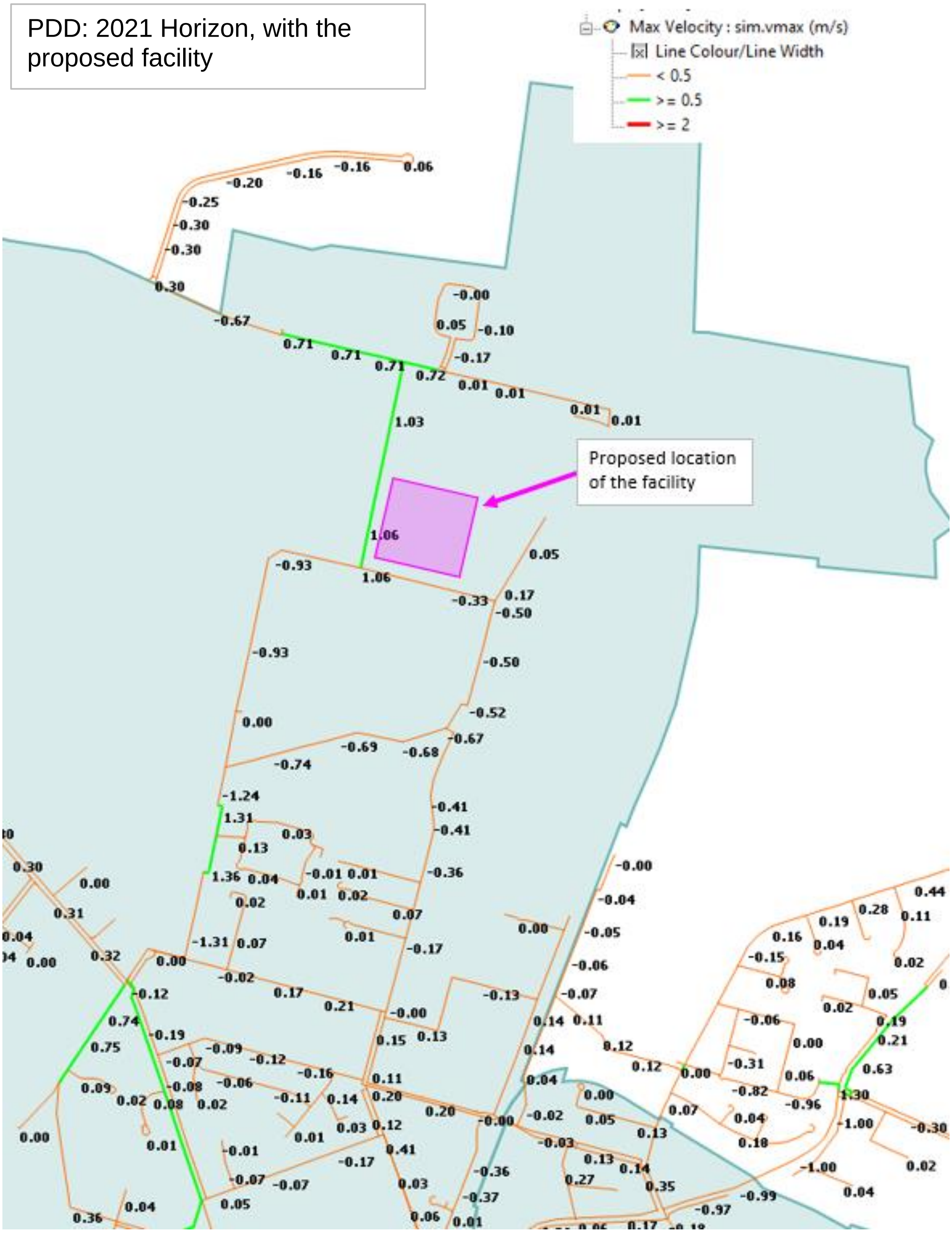



*Figure 4.10 System performance with the proposed facility, peak velocities (PDD: 2021 horizon)*

## 4.6.4 Reservoir storage

There are two storage criteria that a reservoir needs to meet. The first is the static volume criterion. For this criterion the total volume of the reservoir needs to be greater than the total PDD volume over a 24-hour period. The second criterion is the dynamic volume. For this criterion the volume of the reservoir at its lowest operating range must be greater than the PDD over a 12-hour period.

The facility will be serviced by the Hill Road Reservoir, which has a total storage volume of 10,000 $m^3$. However due to hydraulic restrictions in the existing network the reservoir can only fill to 60 percent of its total volume. Hence, for this assessment the static volume is 6,000 $m^3$. At its lowest operating range, the reservoir has a storage of 3,710 $m^3$, which is equates to 37 percent. Table 4.5 and Table 4.6 show the performance of the reservoir for the 2021 PDD flows with and without the demands from the proposed facility.

***Table 4.5 Static volume analysis (PDD: 2021 horizon)***

| Condition | 24 hr PDD ($m^3$) | Static Volume of Reservoir ($m^3$) | Pass/Fail |
|---|---|---|---|
| Without the proposed facility | 4,598 | 6,000 | Pass |
| With the proposed facility | 4,653 | 6,000 | Pass |

***Table 4.6 Dynamic volume analysis (PDD: 2021 horizon)***

| Condition | 12 hr PDD ($m^3$) | Dynamic Volume of Reservoir ($m^3$) | Pass/Fail |
|---|---|---|---|
| Without the proposed facility | 2,299 | 3,710 | Pass |
| With the proposed facility | 2,327 | 3,710 | Pass |

As show in the tables above, the reservoir passes both storage criteria with and without the demand from the proposed facility.

## 4.6.5 Fire flows

AS 2419.1-2005 requirements outline the required firefighting flows for the proposed facility (see Section 2.4 in Technical Report 5 – Fire and Incident Management Review for more information):

– The fire hydrant system will be designed for at least 3 fire hydrants, (Table 2.1, AS 2419.1-2005).
– The minimum flow of each hydrant is 10 L/s, (Table 2.2, AS 2419.1-2005).
– The fire hydrant system will provide the above simultaneous flow rate for at least 4 hours minimum duration (Clause 4.2, AS 2419.1-2005).
– An automatic fire sprinkler system would be required for Building 1. It is anticipated that Building 2 will also require an automatic fire sprinkler system. The sprinkler water supply time would be at least 2 hours. The estimated total flow required for the sprinkler system is 52 L/s
– Total 1,200 kL fire water onsite storage

From the above, the three hydrants simultaneously flowing 10 L/s each for four hours will require a total volume of 432 kL for firefighting purposes. While the sprinklers running for 2 hours with a 52 L/s flow will require a total volume of 374 kL. Therefore, the total fire water storage required is 806 kL. It is planned that the proposed facility will have two fire water tanks (total volume of up to 1,200 kL, depending upon requirements). Therefore, the proposed facility is not reliant on the existing water network for firefighting flows as it has more than 806 kL of water stored onsite for fighting purposes.

The tanks will be refilled through an infill connection from the existing water network. When these tanks are filling the inflow will be limited to 2.5 L/s. This can be increased to 11 L/s between 8.30 pm to 5.30 am (off-peak demand). These flows have been modelled against 2021 PDD and no other demands are coming from the proposed facility. With these flows the minimum service pressure requirements are met in the system. While the fire tanks are filling the proposed facility will not be in operation, until sufficient water is stored on site and available for firefighting

## 4.7 Water sensitivity analysis for 2026 PDD horizon

Council has requested that due to the high water demands for the proposed facility, a sensitivity analysis is to be performed. This analysis involved running the water model for the 2026 horizon with and without the proposed facility's water demands to determine the impact on the existing network. By 2026 the Council plans to complete several major water projects to increase the water supply to Moss Vale area.

Figure 4.11 shows the minimum pressure for the PDD 2026 horizon without the proposed facility. While Figure 4.12 shows the minimum pressure for the PDD 2026 horizon with the proposed facility. The figures show the minimum pressures flows during peak flows for each point.

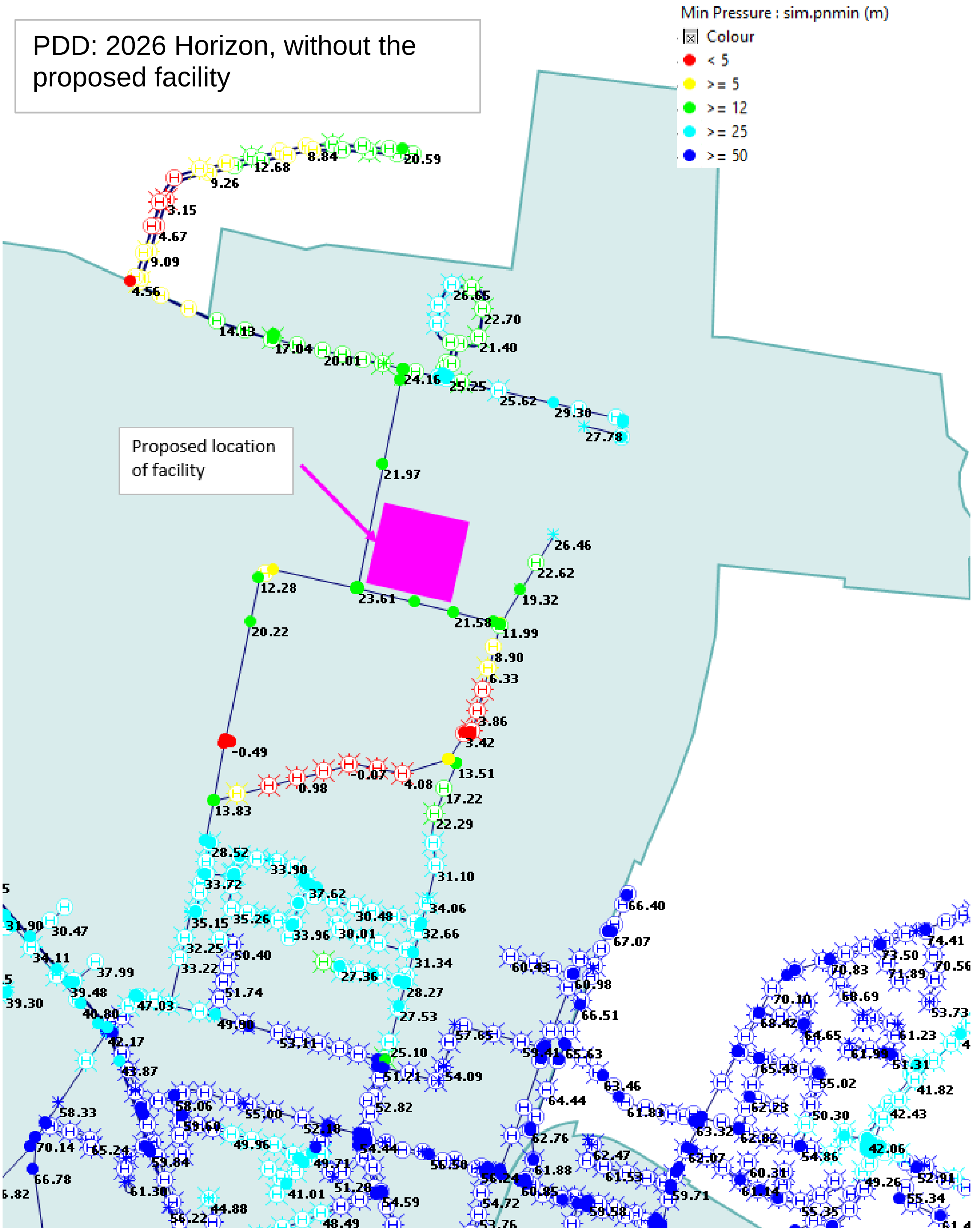



*Figure 4.11 System performance with the proposed facility, minimum pressures (PDD: 2026 horizon)*

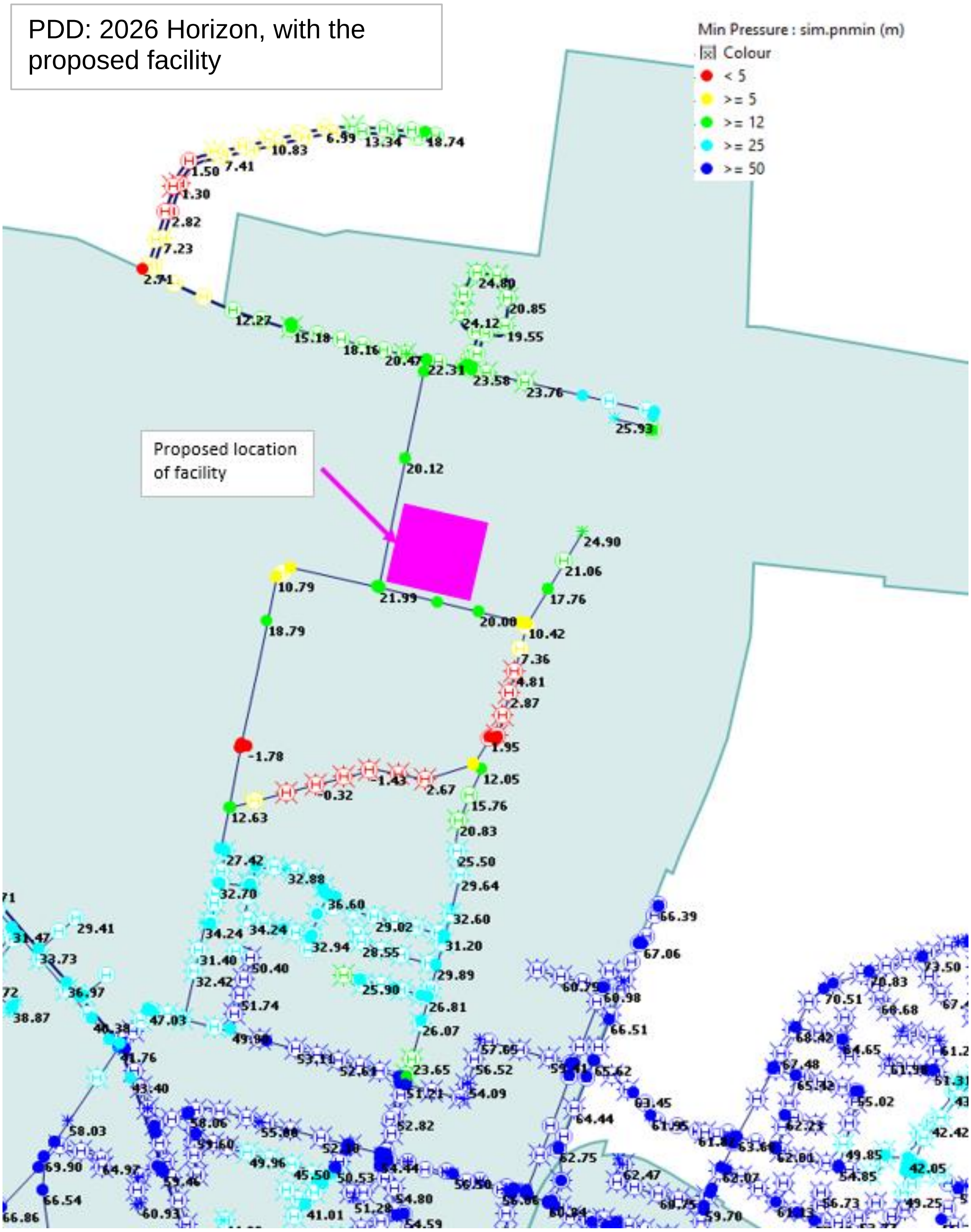



*Figure 4.12 System performance with the proposed facility, minimum pressures (PDD: 2026 horizon)*

Figure 4.11 shows that the minimum pressure for a number of customer points fall below the required 12 m of pressure in the 2026 horizon, without the proposed facility (nodes coloured yellow and red in Figure 4.12). Indicating there are supply issues for the existing network in 2026. With the addition of the proposed facility's demands in Figure 4.12, the minimum pressure for customer points decrease slightly.

Figure 4.13 shows the maximum head losses for the PDD 2026 horizon without the proposed facility. While Figure 4.14 shows the maximum head losses for the PDD 2026 horizon with the proposed facility. The figures show the maximum head losses during peak flows for each pipe.

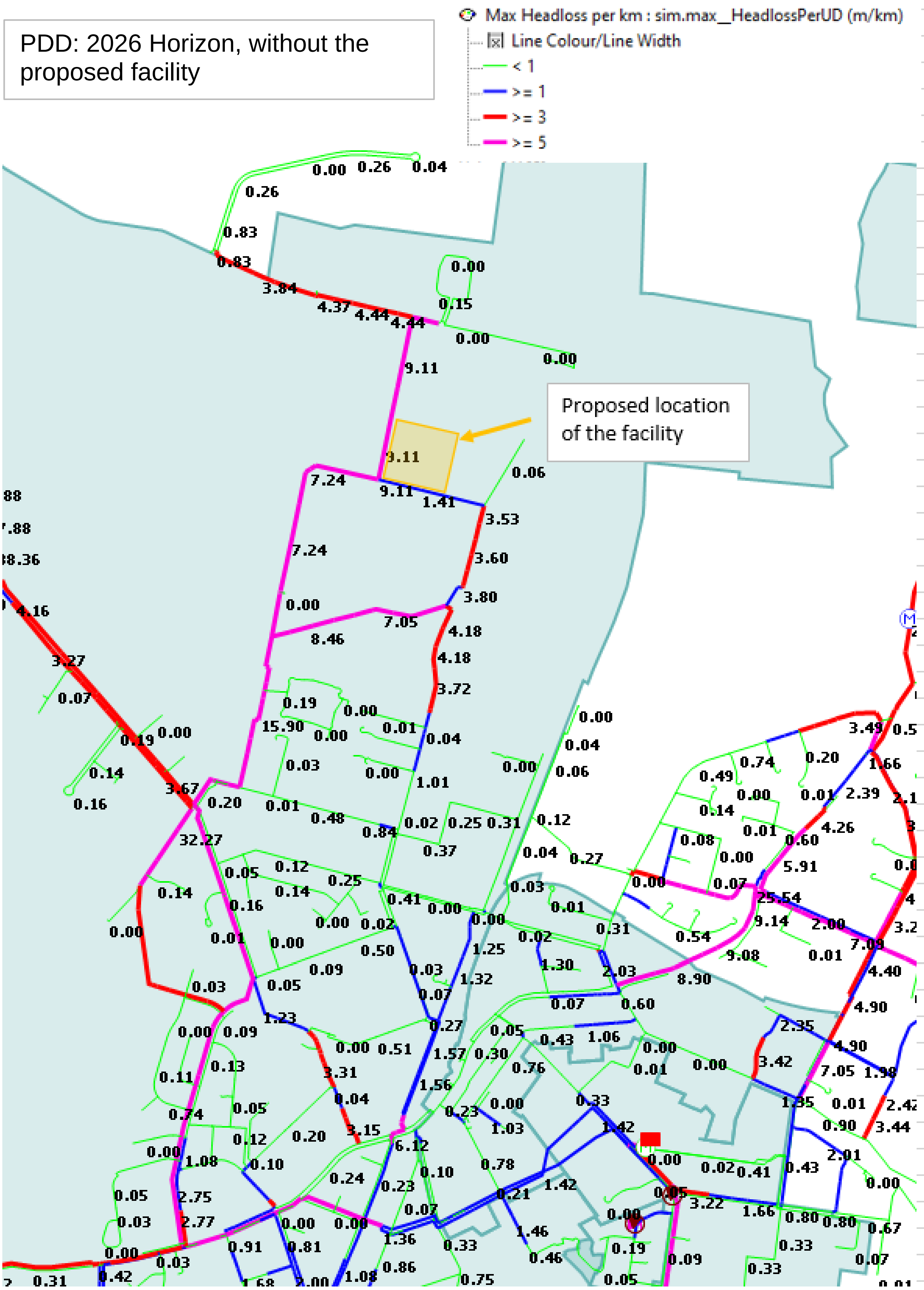



*Figure 4.13 System performance without the proposed facility, maximum head losses (PDD: 2026 horizon)*


*Figure 4.14 System performance with the proposed facility, maximum head losses (PDD: 2026 horizon)*

The additional demands have increased the maximum head losses within the system. These high head losses have caused the low minimum pressures shown in Figure 4.11 and Figure 4.12. To relieve the maximum head losses (greater than the criteria), the mains either need to be upsized or duplicated.

Figure 4.15 shows the maximum velocities for the PDD 2026 horizon without the proposed facility. While Figure 4.16 shows the maximum velocities for the PDD 2026 horizon with the proposed facility. The figures show the maximum velocities during peak flows for each pipe. From reviewing both figures, peak velocities of greater than 2 m/s are not expected to occur in the 2026 horizon.

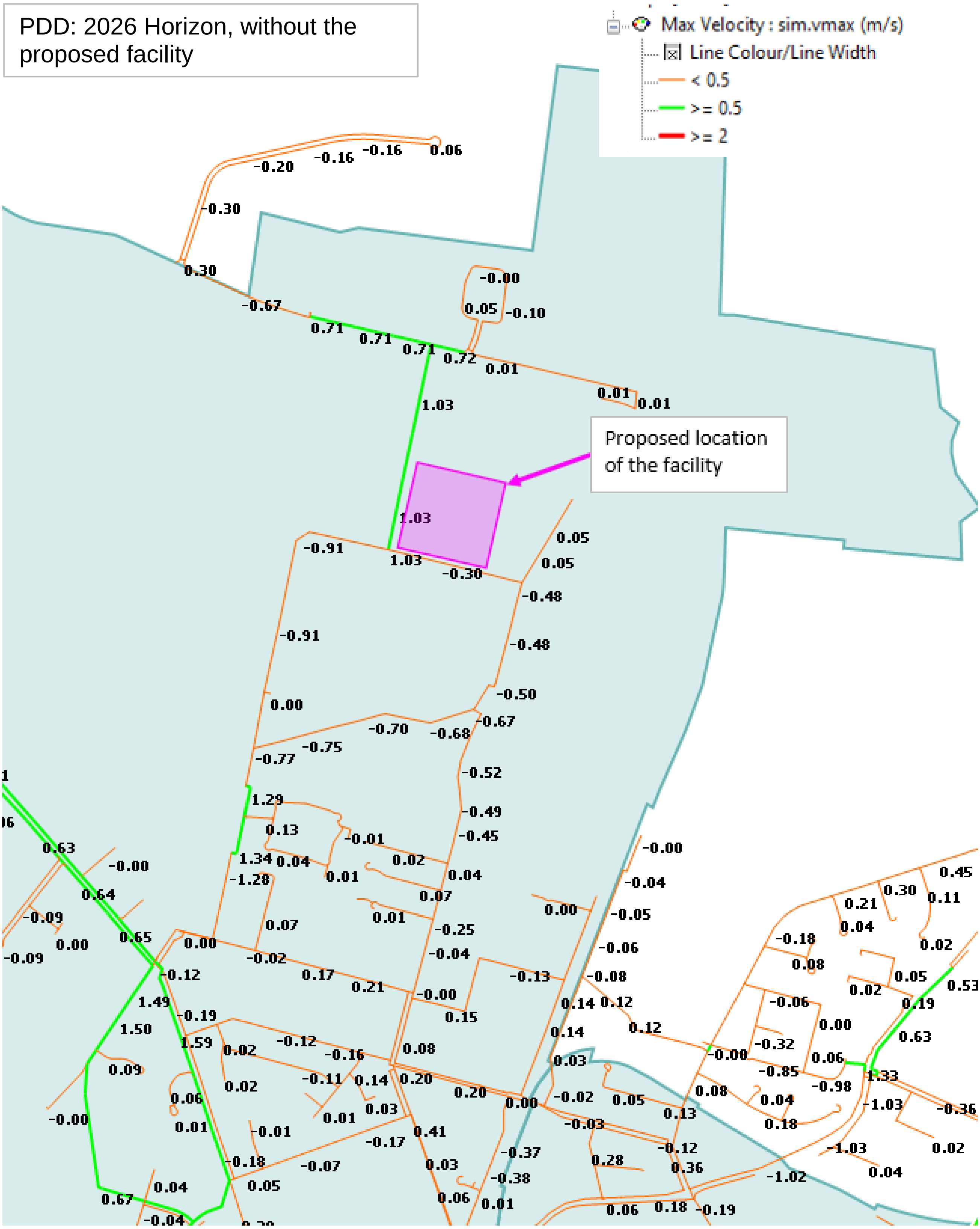



*Figure 4.15 System performance without the proposed facility, peak velocities (PDD: 2026 horizon)*

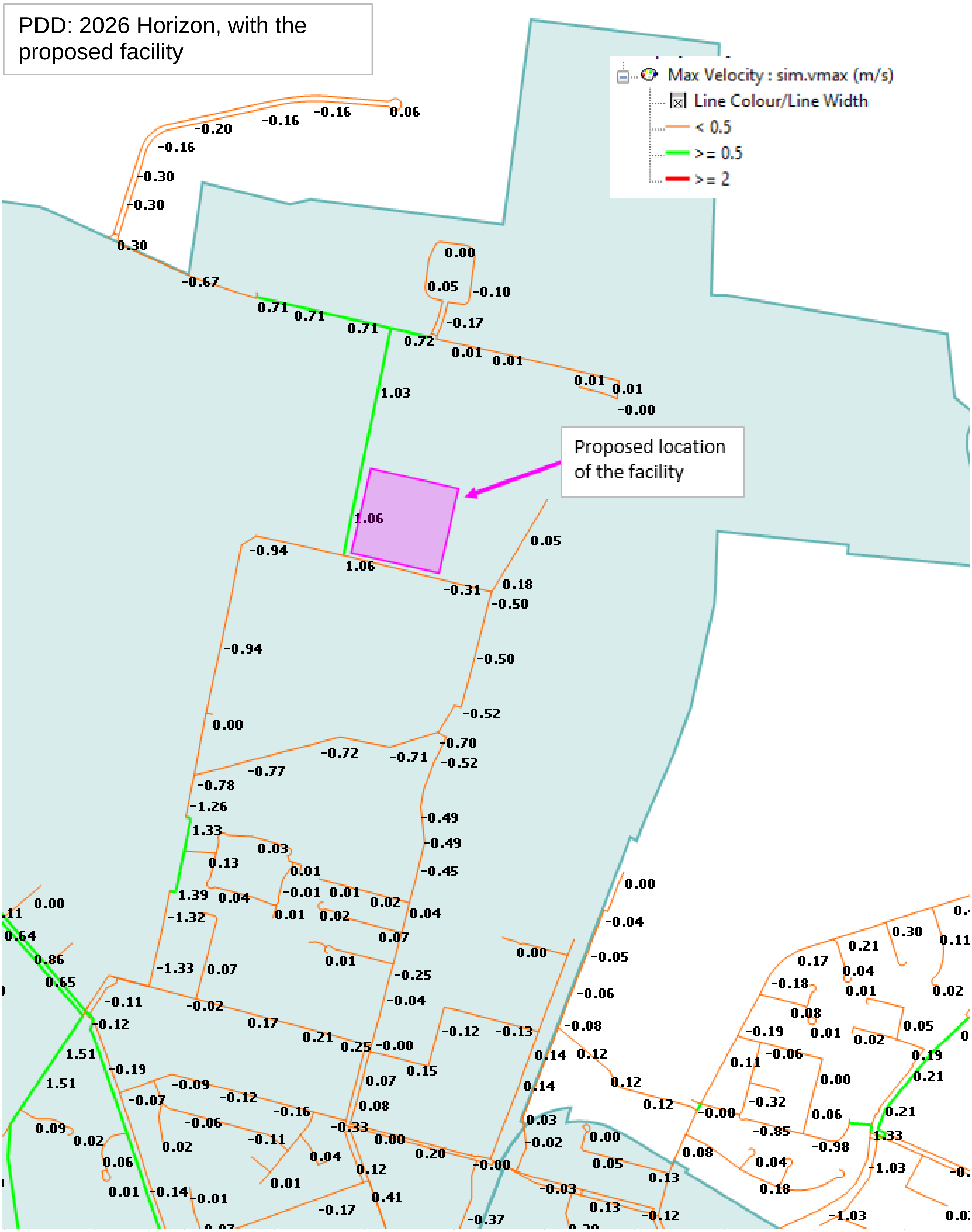



*Figure 4.16 System performance with the proposed facility, peak velocities (PDD: 2026 horizon)*

Table 4.7 and Table 4.8 show the storage performance of Hills Road Reservoir for the PDD 2026 horizon with and without the proposed facility. From Table 4.7 the reservoir does not pass the static condition for both demand cases. If the hydraulic restriction in the existing network can be resolved so that the reservoir can fill to 100 percent of its total capacity (10,000 $m^3$) it would pass the static volume condition. However, the reservoir fails the dynamic condition in Table 4.8 for both cases.

*Table 4.7 Static volume analysis (PDD: 2026 horizon)*

| Condition | 24 hr PDD ($m^3$) | Static Volume of Reservoir ($m^3$) | Pass/Fail |
|---|---|---|---|
| Without the proposed facility | 7,463 | 6,000 | Fail |
| With the proposed facility | 7,519 | 6,000 | Fail |

*Table 4.8 Dynamic volume analysis (PDD: 2026 horizon)*

| Condition | 12 hr PDD ($m^3$) | Dynamic Volume of Reservoir ($m^3$) | Pass/Fail |
|---|---|---|---|
| Without the proposed facility | 3,732 | 3,710 | Fail |
| With the proposed facility | 3,759 | 3,710 | Fail |

The model results from the sensitivity analysis for the 2026 horizon indicate that the system will be overloaded, with and without the demands from the proposed facility. By 2026 the Council plans to complete several major water projects to increase the water supply to Moss Vale area. If required additional modelling can be conducted with the planned upgrades to the network, to determine the performance of the new system.

# 5. References

Water Services Association of Australia. (2014). *WSA 02-2014-3.1, Part 1: Planning and Design, Third Edition-Version 3.1.* Water Services Association of Australia

GHD (2021). *Moss Vale Plastics Recycle and Reprocessing Facility, Fire and Incident Management Review* (Rev 0). GHD

Standards Australia. (2005). *AS2419.1-2005, Fire hydrant installations – Part 1: Systems design, installation and commissioning.* Standards Australia.

Wingecarribee Shire Council. (2020). *Water Supply Master Plan – Revision 04.* Focused Asset Services

NSW Government. (2019). *Code of Practice, Mangaing the Work Environment and Facility.* NSW Government.

ghd.com

→ The Power of Commitment