CIVIL DEVELOPMENT SOLUTIONS

RESIDENTIAL DEVELOPMENT
1-5 RAINBOW ROAD, MITTAGONG
DEVELOPMENT ASSESSMENT REPORT
WATER AND SEWER MODELLING

urbanwater solutions

JANUARY 2024

Contents Amendment Record

This report has been issued and amended as follows:

Issue	Description	Date	Prepared	Checked	Authorised
1	*For WSC Review*	*14/04/2022*	*W: JS, S: EW*	AP	AP
2	*For Issue to Client*	*14/04/2022*	AP		WSC
3	*Updated site plan*	*06/07/2022*	AP		
4	*Updated site plan*	*07/09/2022*	AP		
5 (V2.21)	*Updated site plan & calculations*	*16/01/2024*	*TP/BN*	*AP*	*AP*
6	*Updated site plan*	*18/01/2024*	AP		

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1 Introduction

Urban Water Solutions (UWS) was commissioned by Civil Development Solutions to assess the impact on the existing water and sewerage systems of a proposed **residential** development at 1-5 Rainbow Road, Mittagong.

The proposed development consists of multi-storey residential housing with a gross area of approximately 0.51 ha. The proposed development consists of 50 apartments in two 3 storey buildings.

Wingecarribee Shire Council (WSC) is the local water authority providing potable water supply and reticulated sewerage services.

This report details the impact of the proposed development on the existing WSC water and sewerage systems, incorporating this development, and is subject to approval by WSC.

1.1 Location

The development site is at 1-5 Rainbow Road, Mittagong and has a total site area of approximately 0.51 ha. It will be built over three existing residential lots.

A locality plan is presented in Figure 1-1 with a proposed site plan provided in Figure 1-2.

Figure 1-1: Development Location

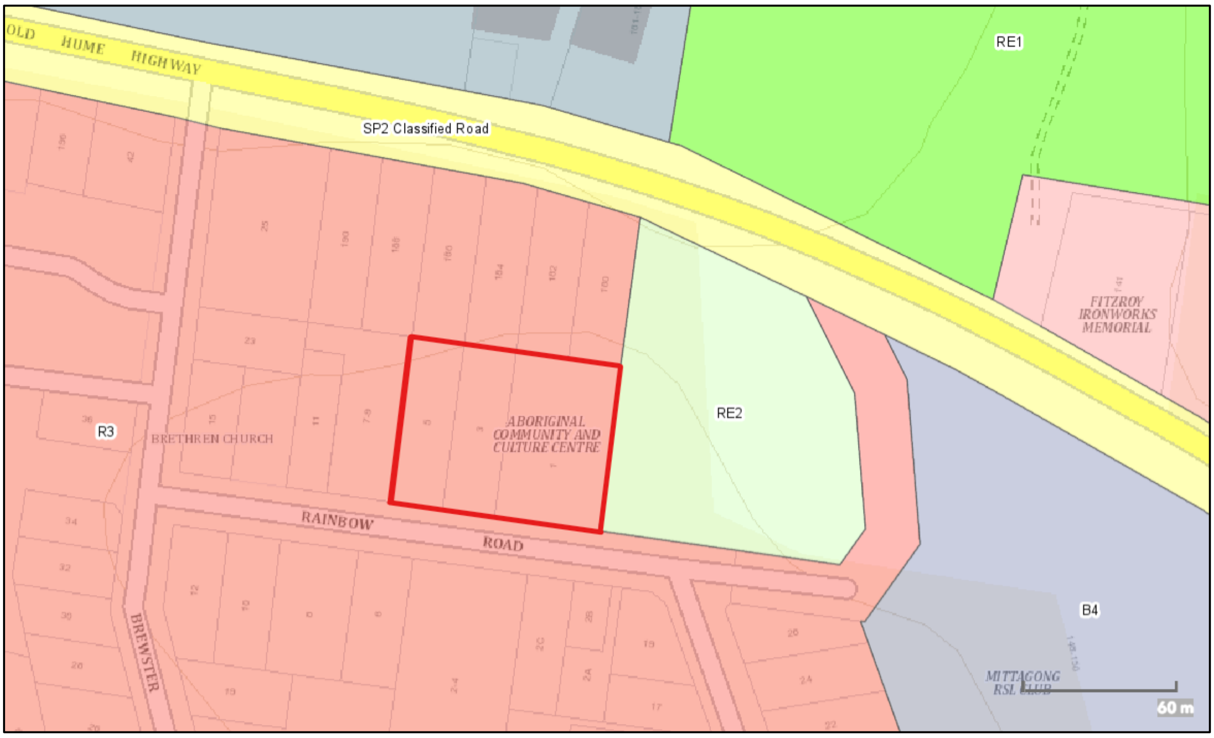
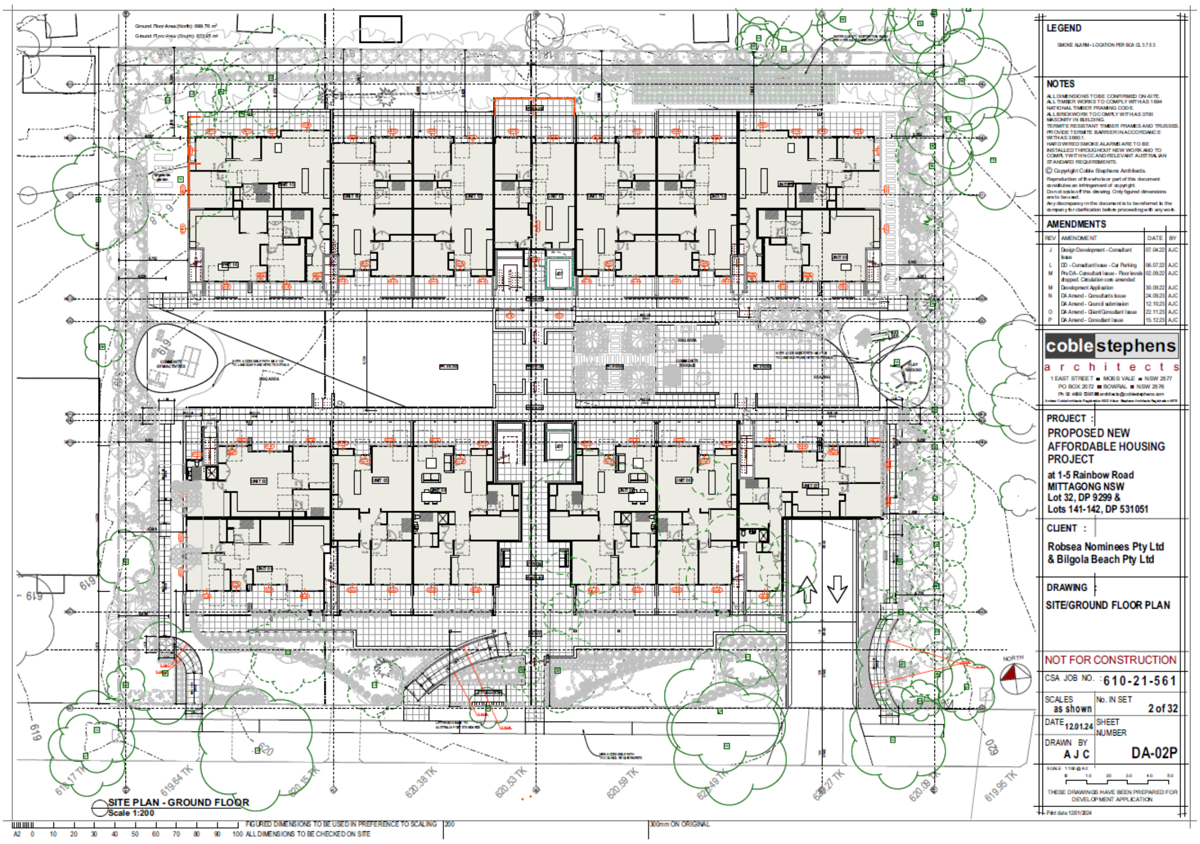


Figure 1-2: Site Plan



Source: Client (12/01/2024)

2 Sewerage System

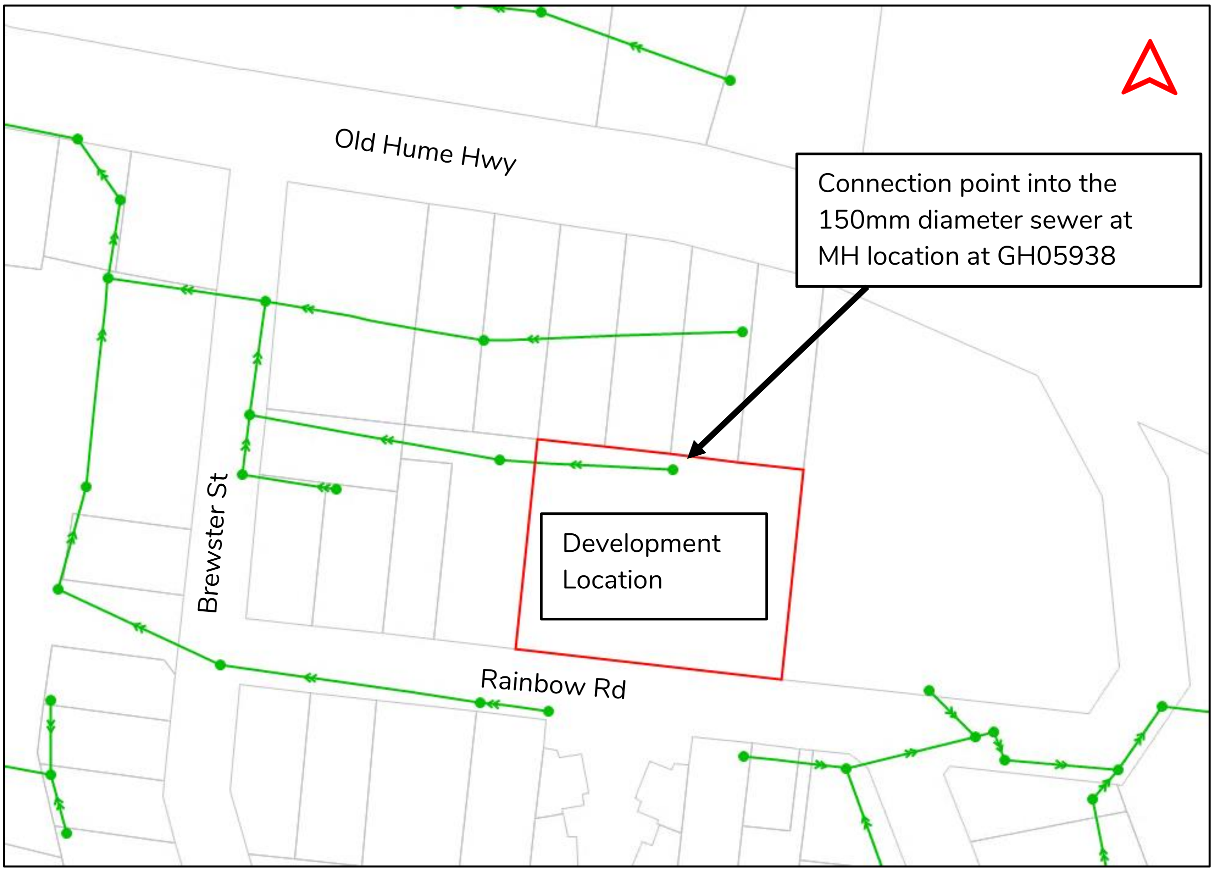
2.1 *Background*

The Mittagong InfoWorks ICM hydraulic model was used to assess the system performance and impact of flows from the development. The model was built and calibrated in 2016.

Details of the proposed development were incorporated into the hydraulic model. Figure 2-1 displays the development location and the potential connection point into the WSC sewerage system.

Flow from the development will discharge into the 150mm diameter gravity sewer at the rear of the property. Figure 2-1 displays the site boundary and the sewer loading point assigned in the model at manhole (MH) GH05938. Flows from the development gravitate to SPS-MT7, located approximately 840 m downstream, from where they are pumped to SPS-MT1 before gravitating approximately 5.2 km to SPS-WV1 at the sewage treatment plant (STP).

Figure 2-1: Development Site and Sewer Connection Point



2.2 WSC Design Standards

The Wingecarribee Shire Council (WSC) design standards applied in the assessment are as follows:

Sewer Design Standards		
Sewer Loading		
Average sewer loading	*230*	*L/person/day*
EP per Tenement	*3.5*	
Pumping Station		
Emergency storage Detention time	*8 hr ADWF as per WSA 02 code*	
Other Requirements 1. *There should be no dry weather overflow from the system* 2. *There should be no dry or wet weather overflow from a pumping station* 3. *Wet weather allowance - Inflow/infiltration (I/I): Provide 2% of the total area as a notional wet weather contribution to the sewerage system*		

Sewer Load (1-5 Rainbow Road, Mittagong)	
Dry Weather Flow	
Category – 1 Bedroom Multi Storey Apartments (10 lots x 0.50)	5.00
Category – 2 Bedroom Multi Storey Apartments (35 lots x 0.75)	26.25
Category – 3 and more Bedroom Multi-Storey Apartments (5 lots x 1)	5.00
Number of ET	36.25
Number of EP	126.9
Estimated Sewage Loading (kL/d)	29.2
Sewage Loading (L/s)	0.34
Wet Weather Flow An inflow/infiltration (I/I) allowance of 2% runoff of the development area was used to provide a notional wet weather contribution to the sewerage system. The WSC level of service standard for Mittagong is that sewerage infrastructure must have the hydraulic capacity to contain all flows associated with a 1 in 2-year rainfall event.	

2.3 *Existing WSC System Performance*

The wet weather performance of the existing system was assessed with a suite of 1 in 2-year design storms with durations ranging from 30 mins to 24 hours.

The 1 in 2-year 2-hour event was selected as the critical event.

2.3.1 Manhole Overflows

There are six manholes predicted to overflow in the catchment during 1 in 2-year design storms. These are predominantly a result of local capacity issues. The results are summarised in Table 2-1 and the locations are presented in Figure 2-2.

Figure 2-2 displays the locations of the predicted MH overflows from the 2-year 2-hour duration storm along with the location of the development site. The MH overflows are in parts of the catchment which are hydraulically independent of the development.

Table 2-1: Existing System Performance

ARI 2-year storm – Predicted Overflow Volume (kL)							
Node ID	**0.5hr**	**1.0hr**	**2.0hr**	**3.0hr**	**6.0hr**	**12.0hr**	**24.0hr**
GH06517	6.7	17.3	20.9	13.8	8.1	0.7	
GH06856		1.0	1.7				
GH06907		1.3	1.3				
GH07153			0.8				
GH07154			0.3				
GS00312		2.5	4.4	1.1			
GS00313		0.1	0.7				
GS00455		4.1	7.9	9.0	7.1		
Total	**6.7**	**26.3**	**38.0**	**23.9**	**15.2**	**0.7**	**0.0**

GH06907 is in the MT7 Frankland Street catchment however spillage is due to capacity issues in DN150 sewer between GH06725 and GH06707. This is shown in Figure 2-3.

2.3.2 Surcharge

The sewer network downstream of the development site is predicted to surcharge during the 1 in 2-year design storms (pre-development) near SPS-MT07. Figure 2-3 displays the extent of the local surcharging.

Figure 2-2: Existing Manhole Overflow Locations – 1 in 2yr 2-hour

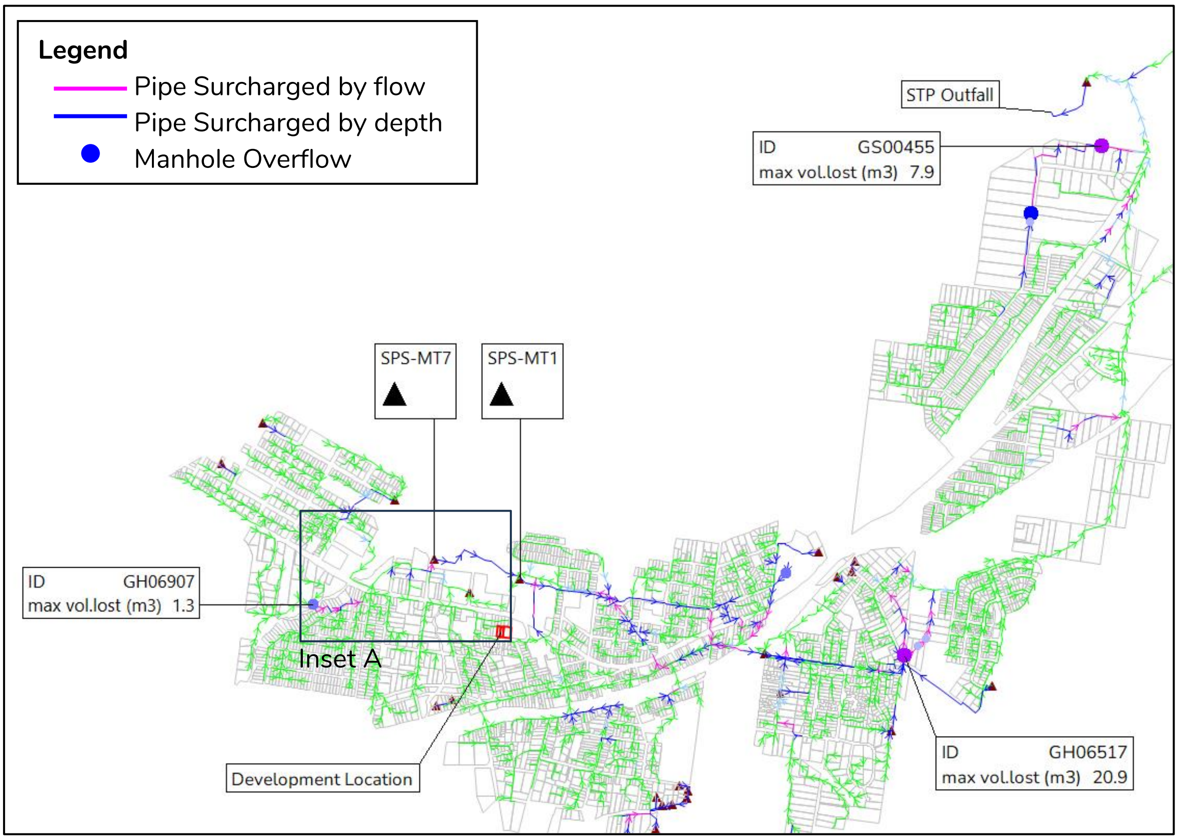
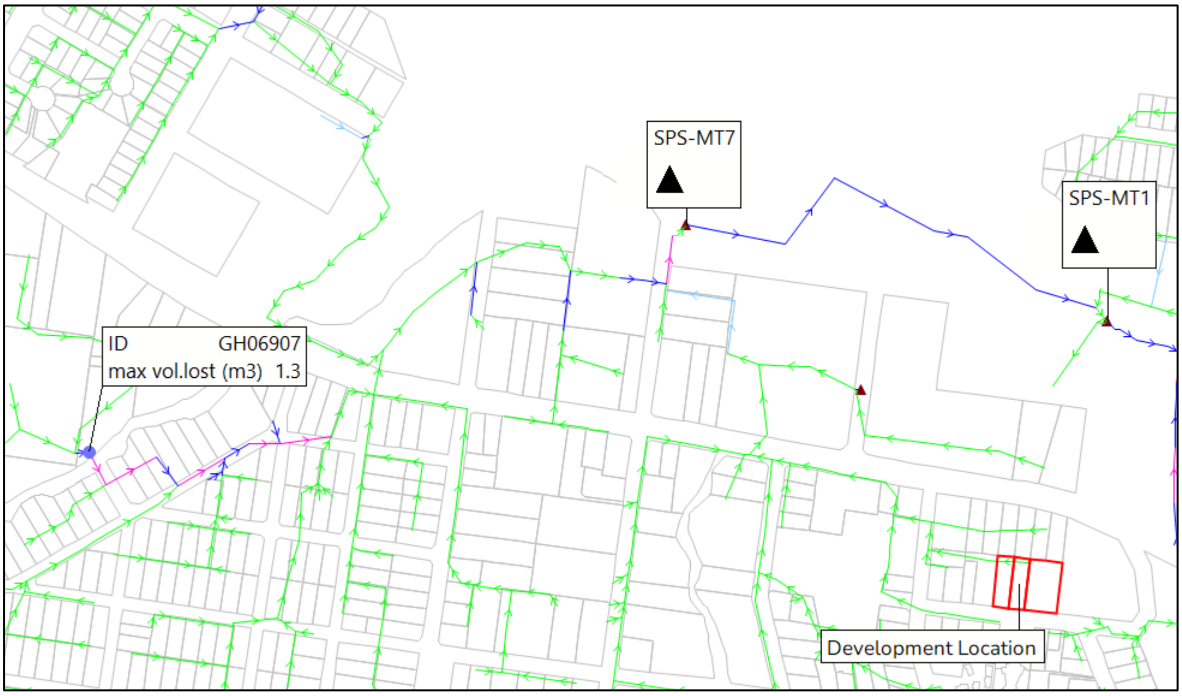


Figure 2-3: Inset A – 1 in 2yr 2-hour



2.3.3 Emergency Storage

This section summarises the emergency storage available in the downstream sewage pumping stations (SPS). The calculations of storage are available in Appendix A – Sewage Pump Station Storage Calculations.

Table 2-2: Summary of Pre-Development Storage Requirement SPS-MT7

Storage in Wet Well (kL)	174.6 kL
Additional Storage (kL)	N/A
DWF (L/s)	4.0 – 8.0 L/s
ADWF (kL/day)	405.8
8 hours ADWF (kL)	135.3
Adequate	< 174.6

Table 2-3: Summary of Pre-Development Storage Requirement SPS-MT1

Storage in Wet Well (kL)	65.2 kL
Additional Storage (kL)	734.2 kL
DWF (L/s)	7.7 – 49.5 L/s
ADWF (kL/day)	1091.8
8 hours ADWF (kL)	363.9
Adequate	< 799.4

2.4 *WSC System Performance including the Development*

The model was updated to include the proposed development and the performance of the WSC network was reassessed against the suite of 1 in 2-year design storms. The critical event remains the 2-hour duration.

2.4.1 Manhole Overflows

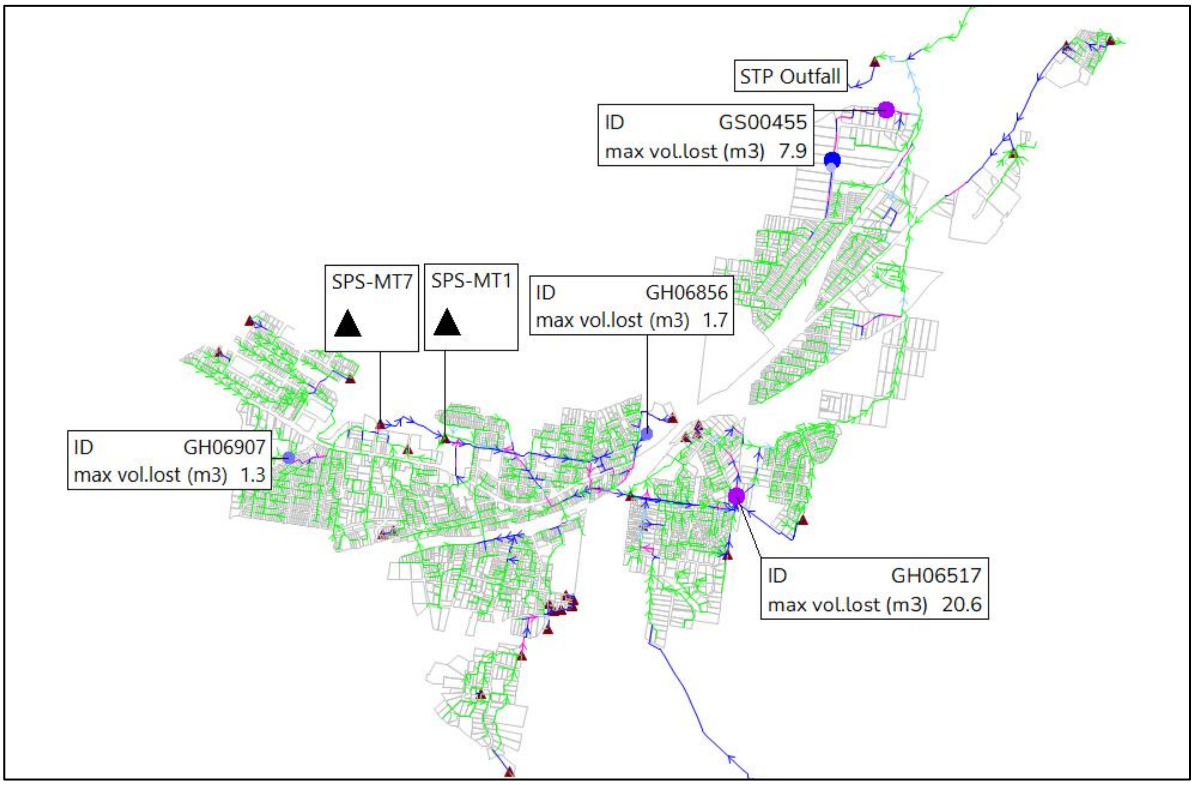
There were very minor changes to overflow volumes, across network for the design storms, with the 0.5 hr event increasing 0.5kL at MH GH06517, and all other events decreasing by 0.1-0.4kL. These changes are within simulation accuracy and cannot be directly correlated to this development.

These locations are shown in Figure 2-4.

Table 2-4: Development System Performance

ARI 2-year storm – Predicted Overflow Volume (kL)							
Node ID	**0.5hr**	**1.0hr**	**2.0hr**	**3.0hr**	**6.0hr**	**12.0hr**	**24.0hr**
GH06517	7.2	17.3	20.6	13.5	8.0	0.8	
GH06856		0.9	1.7				
GH06907		1.3	1.3				
GS00312		2.5	4.4	1.1			
GS00313		0.1	0.7				
GS00455		4.1	7.9	9.0	7.1		
Total	**7.2**	**26.2**	**36.6**	**23.6**	**15.1**	**1.7**	**0.0**

Figure 2-4: Post Development Manhole Overflow Locations – 1 in 2yr 2hr

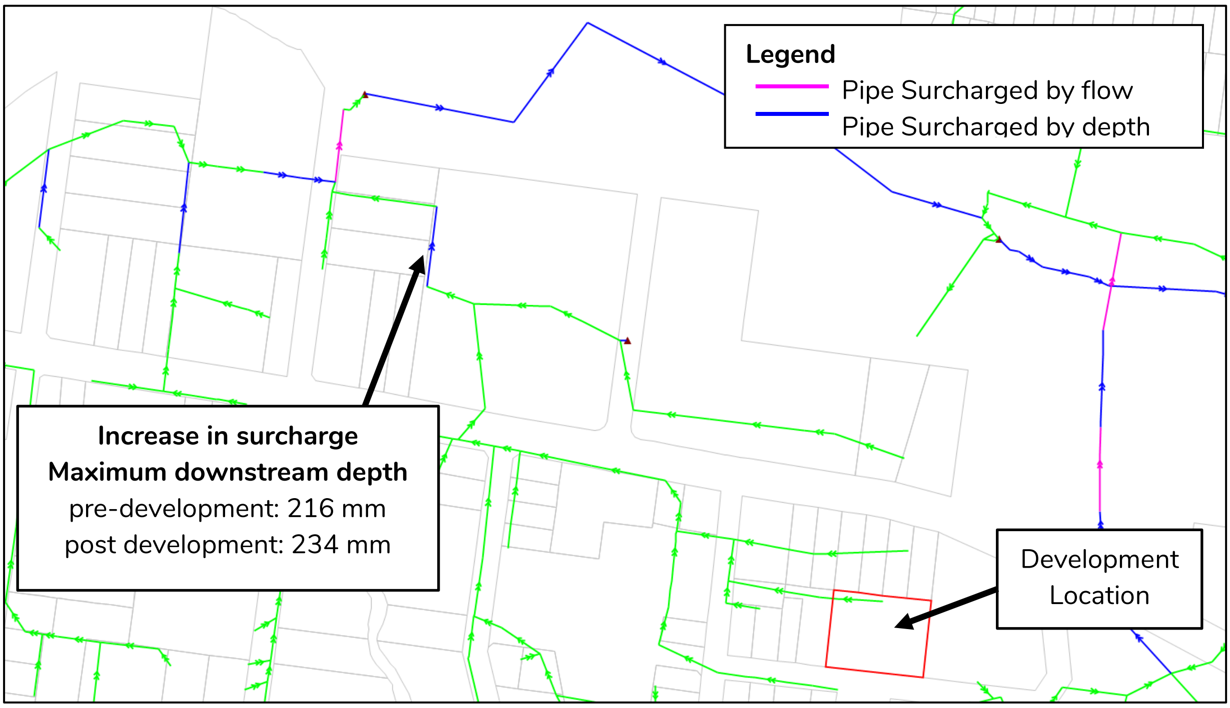


2.4.2 Surcharge

The design storm simulation results showed an increase in surcharge levels of 40 mm or less in the approximately 840m of sewer between the proposed development and SPS-MT07 due to the additional flows.

Figure 2-6 displays the long section of the main sewer downstream of the proposed development during the 1 in 2-year 2-hour design storm. The peak hydraulic grade line (HGL) under post development conditions is shown as the blue shaded area within the pipes. For comparison the peak HGL from the pre-development simulation is displayed as the red line.

Figure 2-5: Surcharging Downstream of the Development



2.4.3 Emergency Storage

This section summarises the emergency storage available in the downstream SPS with the addition of the proposed development flows.

Table 2-5: Summary of Post Development Storage Requirement SPS-MT7

Storage in Wet Well (kL)	174.6 kL
Additional Storage (kL)	N/A
DWF (L/s)	4.3 – 8.2 L/s
ADWF (kL/day)	433.9
8 hours ADWF (kL)	144.6
Adequate	< 174.6

Table 2-6: Summary of Post Development Storage Requirement SPS-MT1

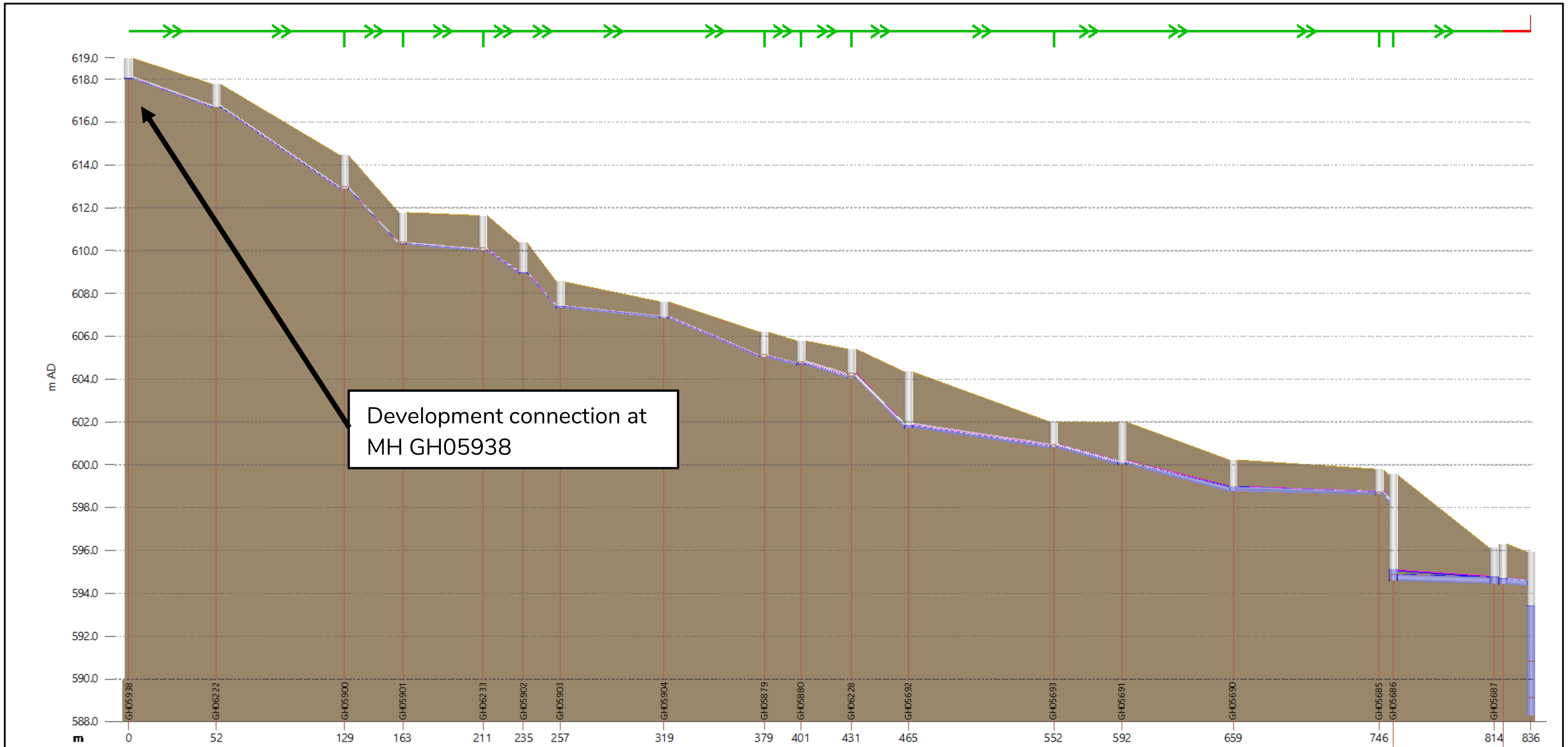
Storage in Wet Well (kL)	65.2 kL
Additional Storage (kL)	734.2 kL
DWF (L/s)	7.7 – 50.2
ADWF (kL/day)	1129.2
8 hours ADWF (kL)	376.4
Adequate	< 799.4

2.5 *Sewer Assessment Summary*

The pre-development system performance results show eight manholes are predicted to overflow during the 1 in 2yr design events. However, these are all hydraulically independent from this development.

The sewer system immediately downstream from the development location is predicted not to surcharge. Reticulation sewers near SPS-MT7 experience increased flow and surcharge conditions (maximum 40mm) but there is no increase in any of the predicted MH overflow volumes because of proposed development.

Figure 2-6: HGL Post Development during 1 in 2-year 2-hour Design Storm



Link	GH05938.1	GH06222.1	-	GH05901.1	-	-	GH05903.1	GH05904.1	-	-	-	GH05692.1	GH05693.1	GH05691.1	GH05690.1	-	GH05686.1	-
width (mm)	150	150	150	150	150	150	150	150	150	250	250	250	250	225	225	-	315	315
r.pfc (l/s)	25	35	42	12	33	42	14	27	19	85	158	64	83	63	19	-	51	75
DS flow (l/s)	1.35	1.54	1.74	2.60	4.01	4.09	4.15	4.51	4.74	12.31	21.21	21.17	25.11	24.40	22.93	-	76.42	-

Node	-	GH06222	GH05900	GH05901	GH06233	-	GH05903	GH05904	GH05879	-	-	GH05692	GH05693	GH05691	GH05690	GH05685	-	-
ground (m AD)	-	617.775	614.431	611.780	611.639	-	608.542	607.615	606.189	-	605.410	604.339	602.000	602.000	600.229	599.802	599.529	596.089

3 Water Supply System

3.1 *Introduction*

WSC provided an InfoWorks WS Pro hydraulic model to assess the impact of the additional demand from the development at 1-5 Rainbow Road, Mittagong on the water supply network.

Analysis of the existing and future network including the proposed development was undertaken for the Peak Day Demand (PDD) scenario.

3.2 *WSC Design Standards*

The following Wingecarribee Shire Council (WSC) design standards were applied in this assessment:

Water		
Demand		
Average Day Demand (ADD)	*260*	*L/person/day*
ADD (per dwelling)	*684*	*L/dwelling/day*
Peak Day Demand (PDD) (per dwelling)	*3000*	*L/dwelling/day*
Max Hour / PDD Factor	*2.76*	
Pressure		
Minimum required at the domestic meter	*12*	*m*
Maximum should be less than	*90*	*m*
Fire flow – Residential	*10 L/s*	*at 15m residual pressure in the water main*
Fire flow - Commercial	*20 L/s*	
Velocity & Headloss		
Maximum velocity in mains	*2*[1]	*m/s*
Target maximum head loss in mains	*5 m/km*[2]	*for reticulation mains*
Target maximum head loss in mains	*3 m/km*	*for trunk mains*
Reservoir		
Total storage	*24hr PDD*	*ML*
At the lowest operating range	*12hr PDD*	*ML*

[1] Velocities in the reticulation network < 2 m/s. Velocities exceeding this value should be approved by WSC. For fire fighting, velocities up to 4.0 m/s are acceptable.

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

As this is a residential development, the equivalent tenement (ET) loading was based on the number and type of dwellings under the high density multi-residential lots category. With ET loadings calculated from the number of bedrooms per apartment, it was estimated that the development would contribute a water loading of 24.2 ETs.

Water Demand Estimate	
Category – Multi-Residential Lots (High Density) 0.33 ET per 1-bedroom apartment × 10 0.50 ET per 2-bedroom apartment × 35 0.67 ET per 3-bedroom apartment × 5 Total of 49 apartments	 3.3 ET 17.5 ET 3.4 ET
Total Number of ET	*24.2 ET*
Average Day Demand (ADD) Total	16.6 kL/d 0.19 L/s
Peak Day Demand (PDD) Total	72.6 kL/d 0.84 L/s
Max Hour Demand (MHD) Total	8.35 kL/hr 2.3 L/s

Category from the Water Directorate Report: Section 64 Determinations of Equivalent Tenements Guidelines (2017)

3.3 *Hydraulic Modelling*

The development is located in the Gib North Pressure Management Area (PMA).

The Gib North PMA is mainly supplied by Welby Reservoir (RES-ME6) and receives additional water supply from the Wingecarribee WTP transfer main during peak hours through a control valve (NV04168X.NV04168Y.1).

The schematic diagram below provides a general overview of the flow directions, zones, and key assets affected by this development with an overview of the water network presented in Figure 3-2.

Figure 3-1: Water Supply Network Schematic

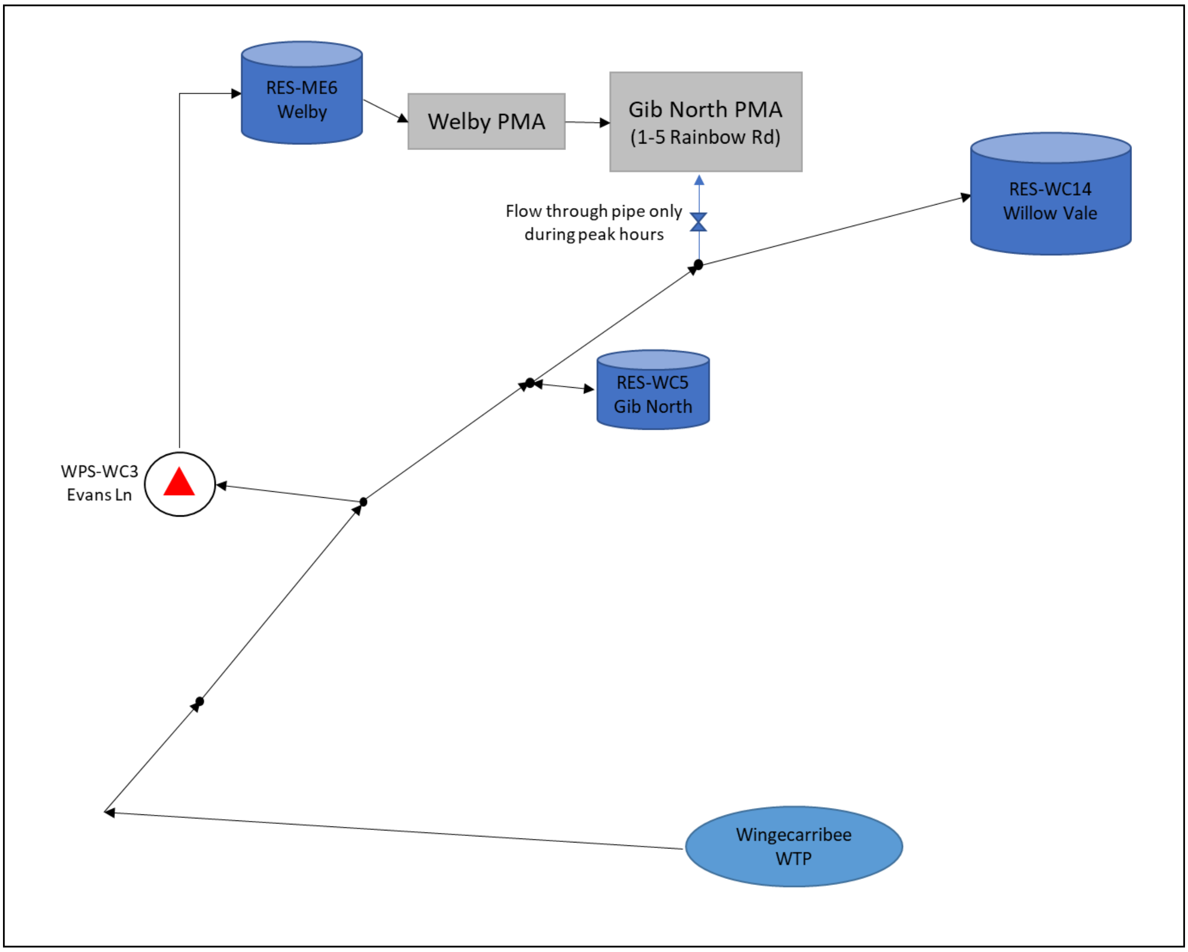
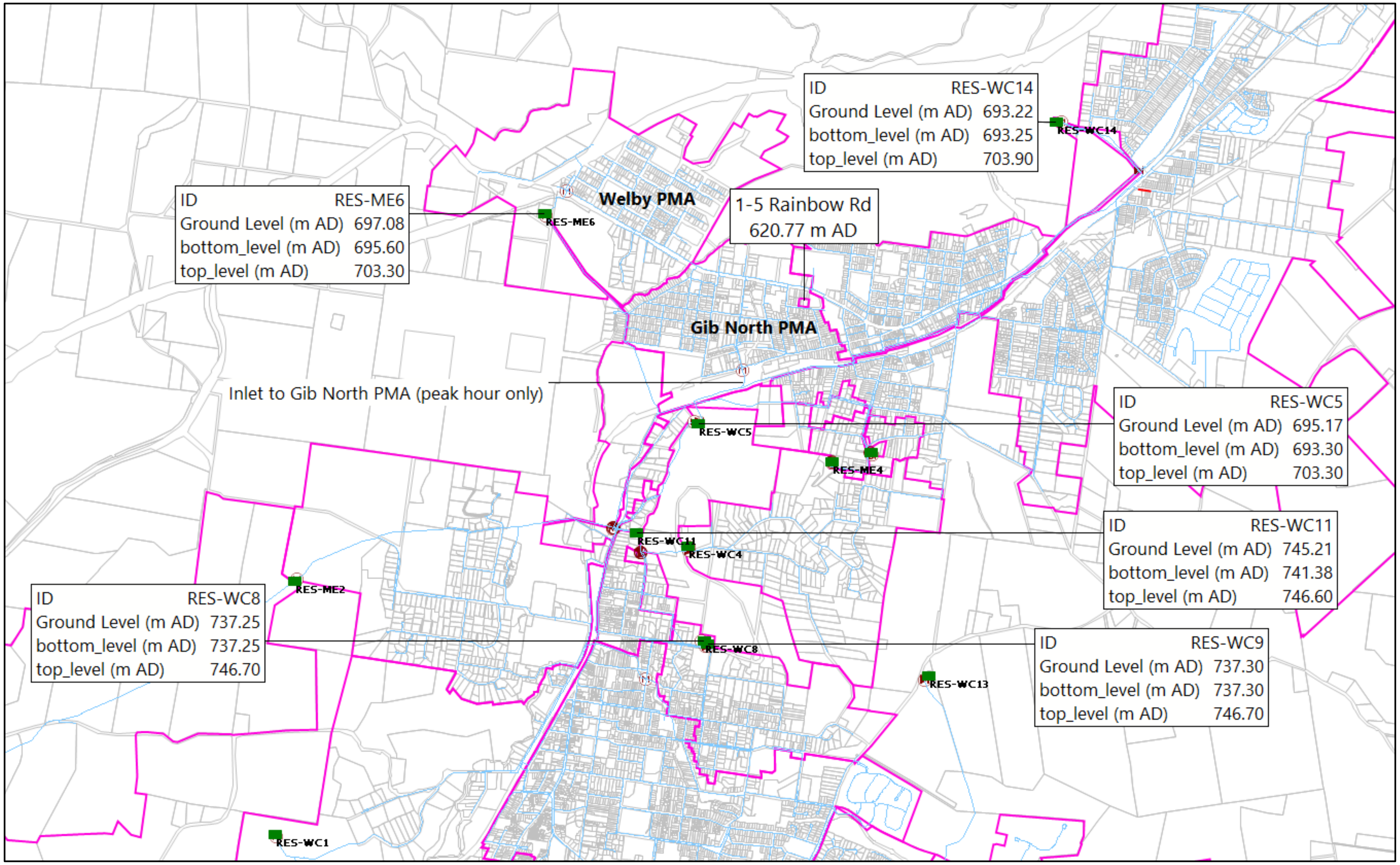


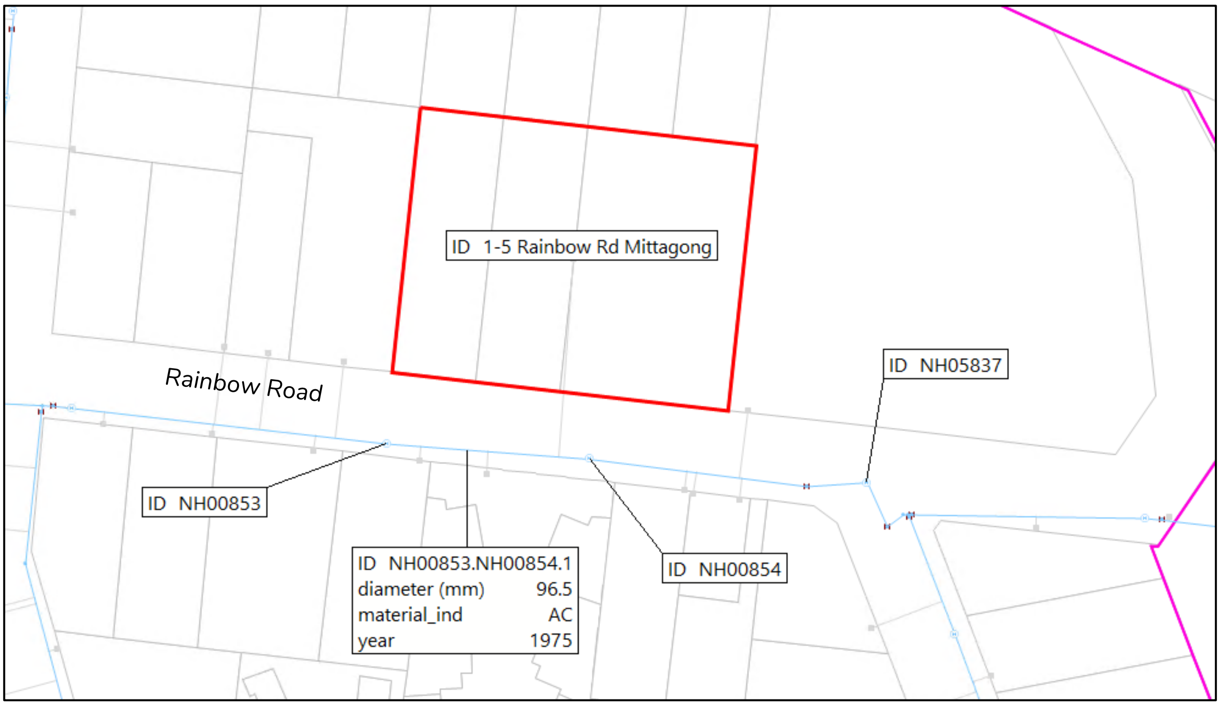
Figure 3-2: Overview of the Development and Water Supply Network



3.3.1 Internal Servicing

The development is expected to draw water from the existing DN100 AC main along Rainbow Road between fire hydrants NH00853 and NH05837.

Figure 3-3: Detailed Development layout and Water Supply Connection



3.3.2 Model Configuration

The Peak Demand Day scenario in WSC's InfoWorks WS Pro v5.0.0 model database has been updated for use in WS Pro v2021.3 and configured to conform with WSC's Development Assessment Template. A 72-hour duration simulation (representing three consecutive peak demand days) was adopted for the modelled scenarios, except where otherwise indicated.

The base model contained an allocation of non-revenue water (NRW) at this location. The NRW modelling was equally applied, without any changes, to the development assessment simulation to ensure a fair comparison between pre- and post-development network performance.

Model Items

Model items used in this assessment are:

- Network – WSC Water Supply Network CAL 0.04 SPA 0.04,
- Control – WSC Water Supply Control CAL 0.04 SPA 0.03!,
- Demand Diagram – UWS Default MDD PHF 3.0 + Growth 0.01 DevAss,
- Demand Scaling – MDD 2.0xADD Category Only 0.1 + DevAss x1, and
- Alternative Demand – Alt Dem 2020 Master Plan ADD 2021 0.2

3.4 *System Performance Results*

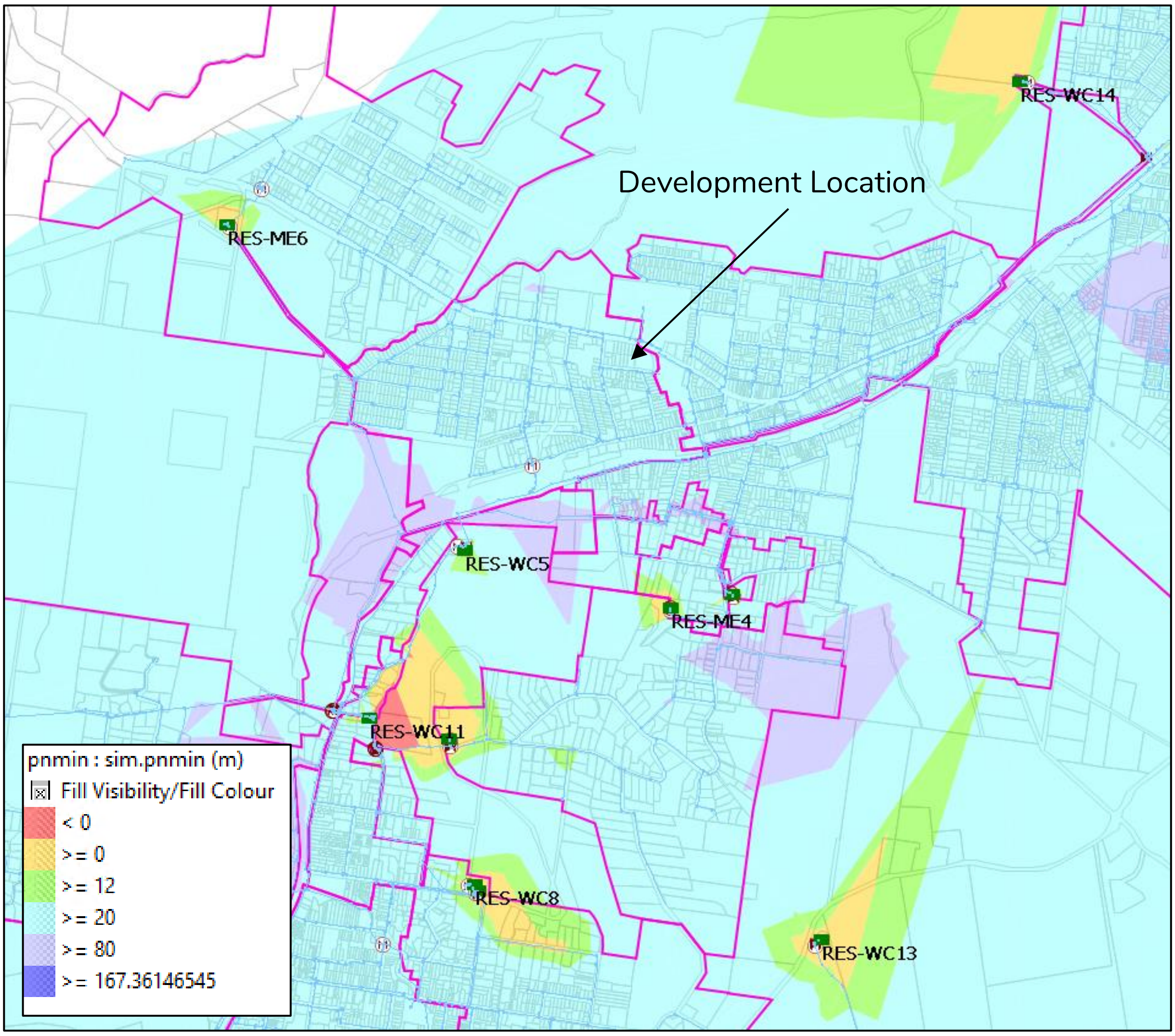
3.4.1 Base Case System Performance

Minimum Pressures

The minimum pressures within the Gib North PMA exceeds 12 m at all nodes and fire hydrants containing modelled customer demand.

Figure 3-4 shows the minimum pressure contours in the network base case.

Figure 3-4: Pressure Contours – Base Case



Maximum Pressures

Maximum pressures at three nodes with demand within the Gib North PMA exceed 90 m.

The pre-development maximum pressure at a demand node is forecasted to be 98.9 m at NV03265. This asset is located on the transfer to Willow Vale Reservoir (RES-WC14).

Reservoir Storage

The Gib North PMA is supplied from different water zones. The development will primarily draw water from Welby Reservoir (RES-ME6) and will receive additional water supply from Gib North Reservoir (RES-WC5). The Gib North Reservoir is filled from Oxley Drive Reservoir (RES-WC8) and the Wingecarribee WTP.

The Welby Reservoir (RES-ME6) has a storage volume of 1.43 ML.

The Gib North Reservoir (RES-WC5) has a storage volume of 6.0 ML.

The predicted PDD zone consumption for the Gib North PMA is 0.96 ML/day.

The predicted consumption (0.96 ML/day) is less than the reservoir storage volume (1.43 ML) and so, the WSC requirement for a minimum of 24 hours of PDD is met.

The minimum storage level at RES-ME6 of 0.64 ML (45%) is predicted to occur at 14:50 on Day 1. This exceeds 12 hours of PDD Demand (0.48 ML) so the WSC requirement is achieved.

Peak Velocity and Headloss

The peak velocity and headloss statistics of the Gib North PMA pre-development are summarised below.

Diameter (mm)	Unit Headloss (m/km)	Link Type	No. of Links Affected
≤ 300	> 5	Pipe	19
≤ 300	> 5	Valve	13
≤ 300	> 5	Meter	1
> 300	> 3	Pipe	43

None of the pipes and valves in the Gib North PMA experience velocities above 2 m/s.

3.4.2 System Performance including Development

Minimum Pressures

The minimum pressures within the Gib North PMA exceed 12 m at all nodes containing modelled customer demand.

The demand node with the lowest pressure in the Gib North PMA is node NV04752X with a post-development minimum pressure of 44.14 m and maximum pressure of 50.67 m. It services 1 customer point which is a school at 117-119 Old Bowral Road, Mittagong.

Figure 3-5 shows the minimum pressure contour with the inclusion of the additional development.

Figure 3-6 shows the difference in pressure in the water supply network due to this development. Unsurprisingly, the local reticulation area containing the proposed development experiences the largest decrease of minimum pressure. The yellow shaded region in Figure 3-6 contains demand node (hydrant) NH00854 which experiences the largest decrease of minimum pressure (-0.4 m) with a resultant post-development minimum pressure of 57.2 m.

The decrease visible in the other PMA in Figure 3-6 is due to a model instability occurring at one timestep, and is not representative of overall pressure during the PDD simulation.

Figure 3-5: Pressure Contours – Base Case + Development

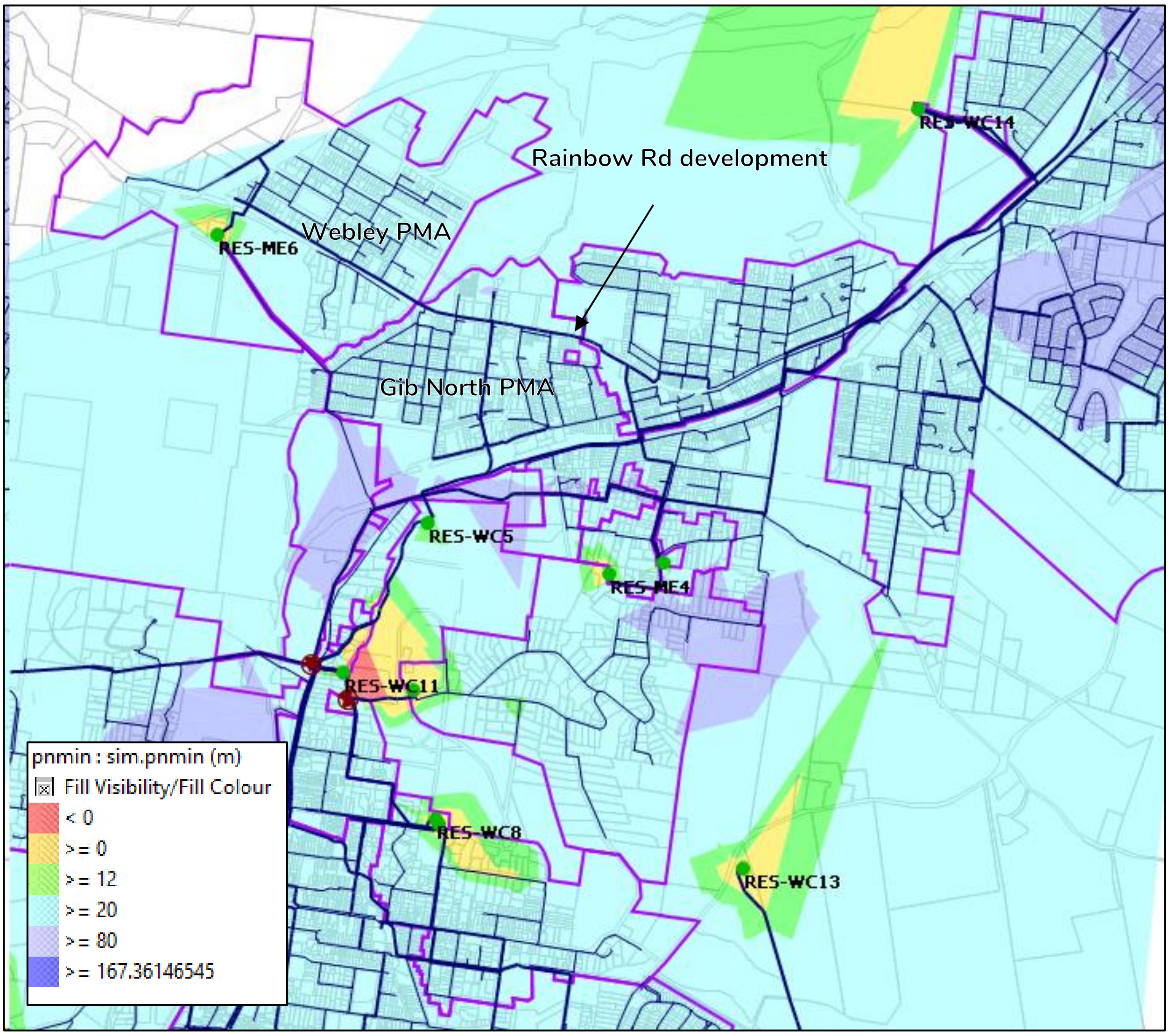


Figure 3-6: Pressure Contour Difference

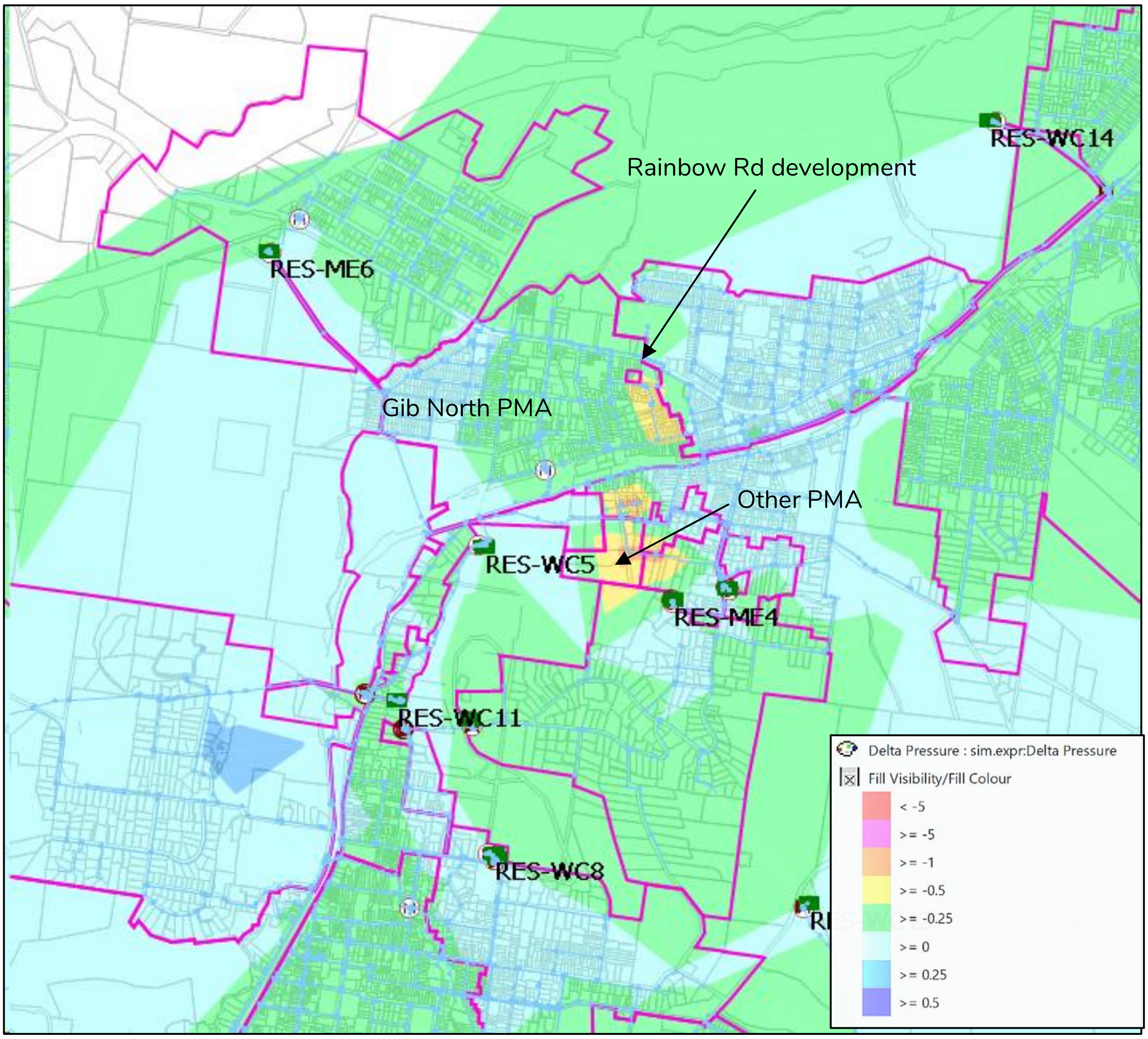
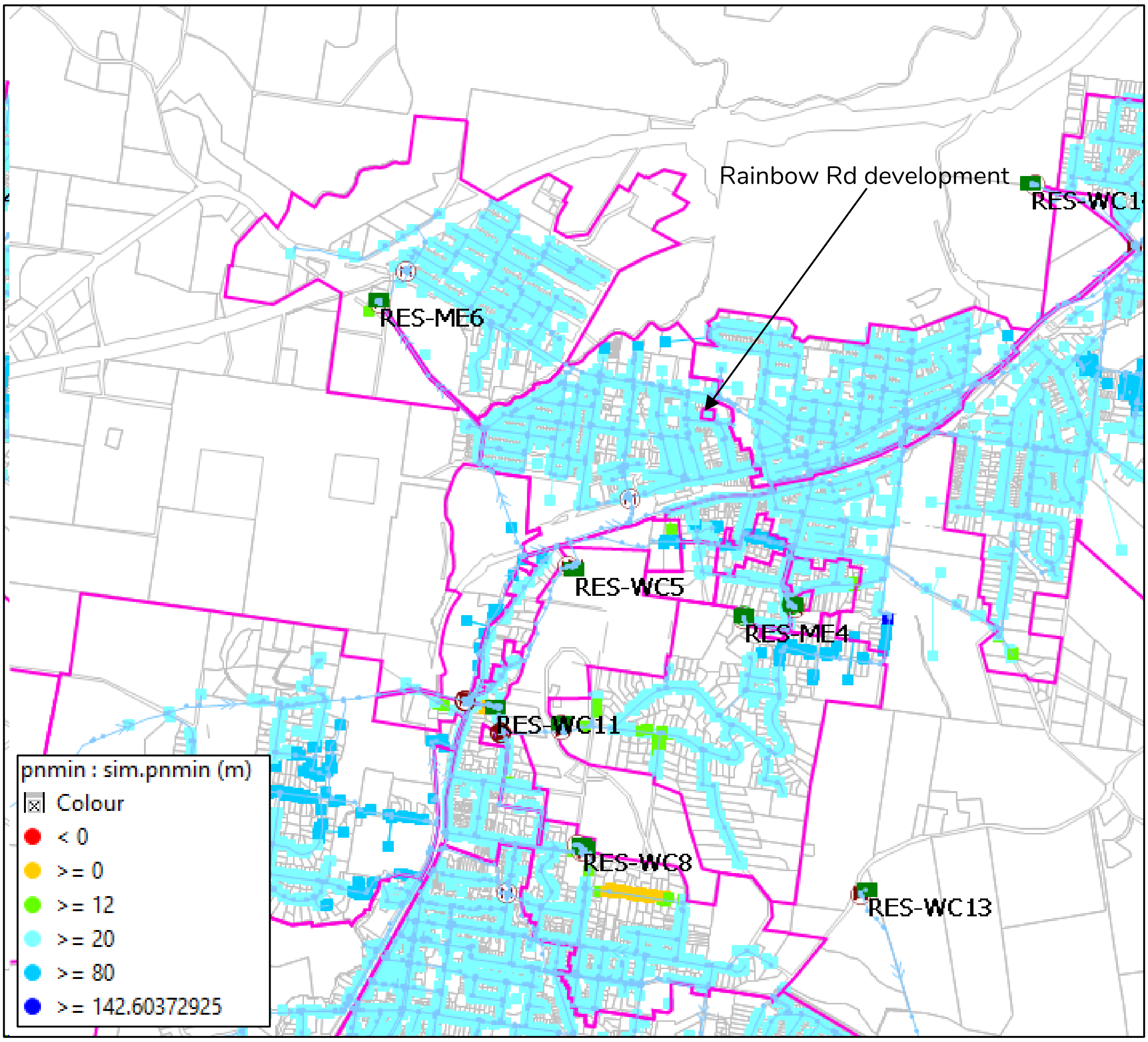


Figure 3-7: Minimum Pressure Customer Points – Base Case + Development



Maximum Pressures

Maximum pressures at nodes with demand in the Gib North PMA exceed 90 m. The post-development maximum pressure at a demand node is predicted to be 98.99 m at NV03265.

Reservoir Storage

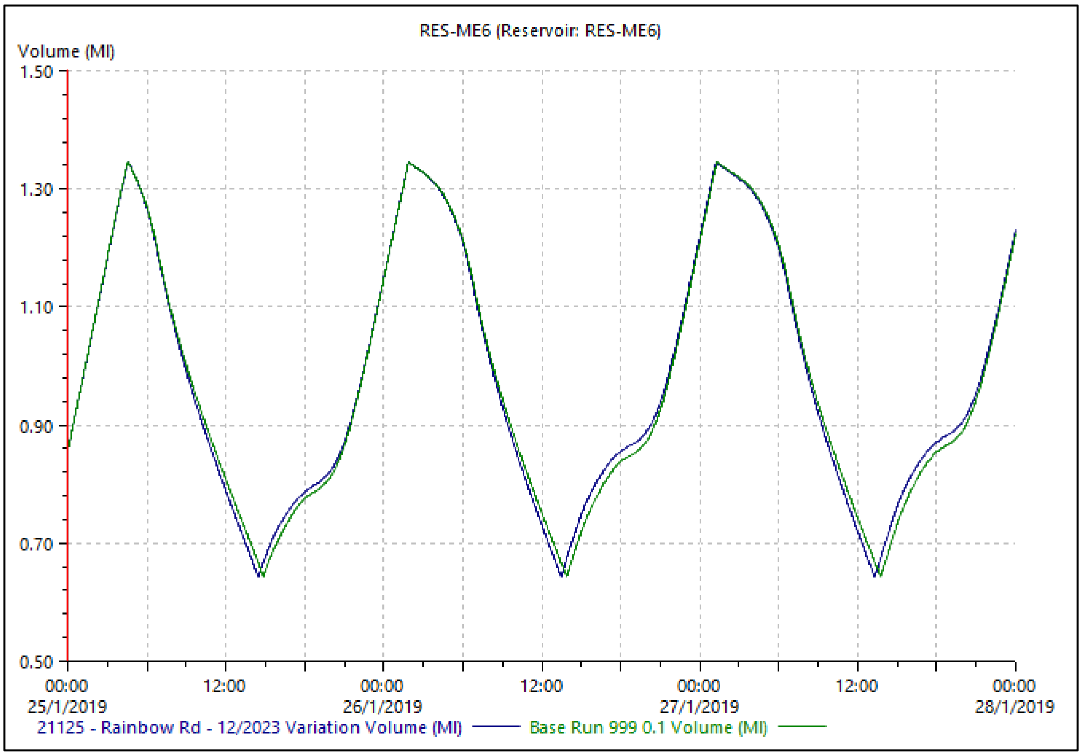
The post development PDD zone consumption for Gib North PMA is 1.04 ML/day.

The predicted consumption (1.04 ML/day) is less than the reservoir storage volume (1.43 ML) and so, the WSC requirement for a minimum of 24 hours of PDD is met.

The minimum storage level at RES-ME6 of 0.64 ML (45%) is predicted to occur at 14:30 on Day 1. This exceeds 12 hours of PDD Demand (0.52 ML) so the WSC requirement is achieved.

The development has almost no effect on RES-ME6's water levels and only shifts the start and end timings of the supply-fill cycles. The depth trend in Figure 3-8 shows that the reservoir is sufficiently replenished without any gradual deficit over the 3 days.

Figure 3-8: Comparison of Reservoir Performance



Pipe Velocity & Headloss

The following table summarizes the pipe velocity and headloss results pre- and post-development within the Gib North PMA.

Type	Before Development	After Development
No. of Pipes with Diameter ≤ 300 mm with Unit Headloss > 5 m/km		
Pipe	19	19
Valve	13	26
Meter	1	1
No. of Pipes with Diameter > 300 mm with Unit Headloss > 3 m/km		
Pipe	43	43

A more detailed summary of pipes exceeding the target maximum headless benchmarks is shown in Appendix B – Pipe Headloss Results.

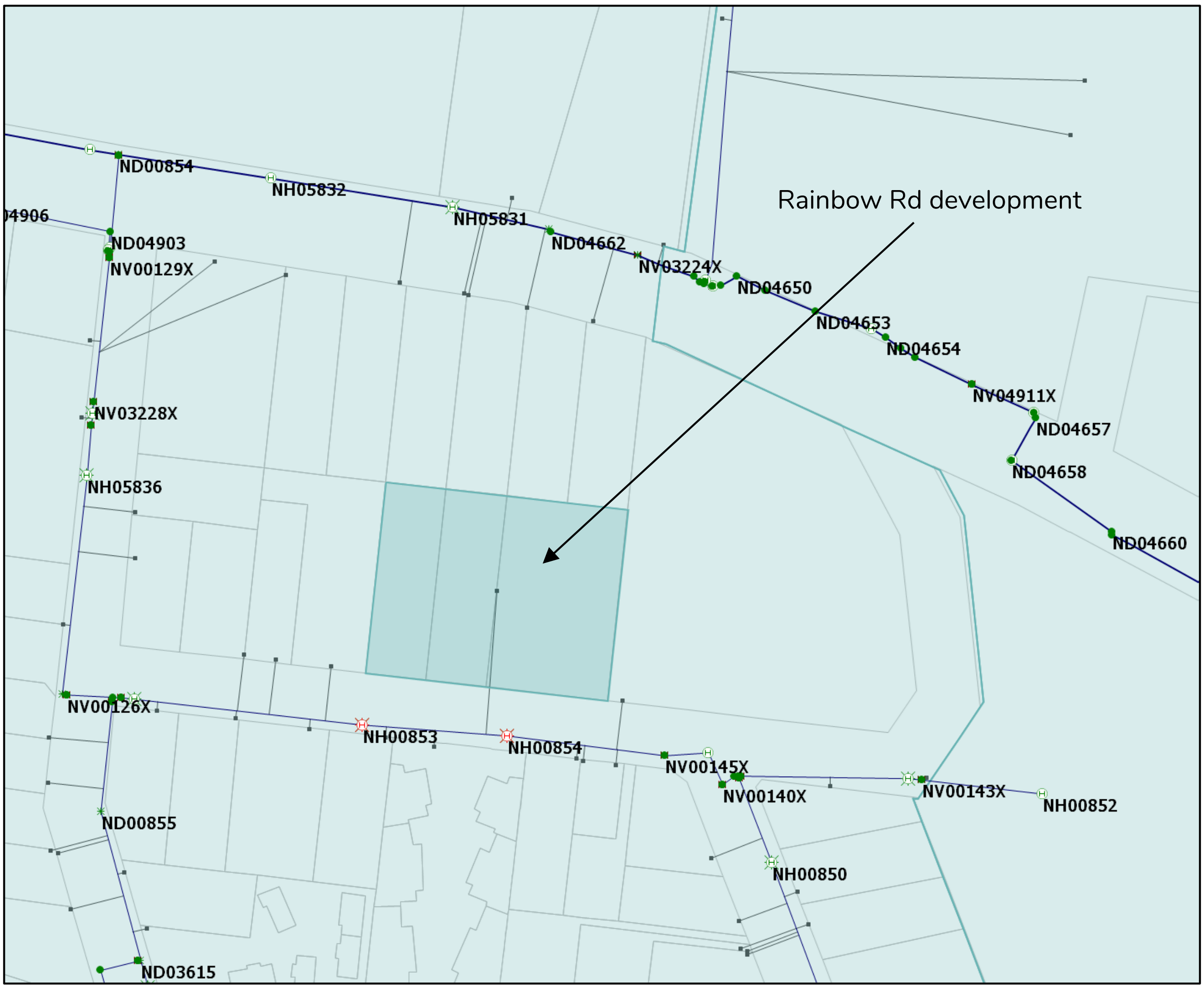
Both pre-development and post-development network models do not predict any pipes and valves to experience velocities greater than 2 m/s in the Gib North PMA.

3.5 Fire Flow

Fire flow simulations were run to determine if the reticulation around the Rainbow Road development could meet the residential lot firefighting flowrate requirement of 10 L/s while maintaining 15 m of residual pressure at 19:00 hours (peak demand time).

Fire hydrants NH00853 and NH00854 were used as the test points with the results shown in Table 3-1.

Figure 3-9: Hydrant Test Locations



Both hydrants were predicted to be able to supply the required fire flow.

Table 3-1: Fire Flow Summary Results

Node Tested	Location	Results (10 L/s and maximum hydrant flow)
NH00853	Hydrant on Rainbow Rd DN100 AC main	10 L/s with 53.8 m residual pressure 37.8 L/s with 18.3 m residual pressure
NH00854	Hydrant on Rainbow Rd DN100 AC main	10 L/s with 51.7 m residual pressure 35.1 L/s with 15.9 m residual pressure

3.6 Water Supply Assessment Summary

The analysis shows that the proposed development at 1-5 Rainbow Road, Mittagong will have negligible impact on the overall performance of the Gib North PMA water supply network. The existing reticulation is able to provide water at the flowrates and pressures as required by WSC standards during fire flow.

Appendix A – Sewage Pump Station Storage Calculations

Summary of Storage Availability SPS-MT7

Detail	Value	Source
Ground Level	RL 595.930	Hydraulic Model - LI
Overflow Level	N/A	
Wet Well Shaft Area	12.7 m²	Hydraulic Model - ADS
Wet Well Chamber Area	70.89 m²	Hydraulic Model - ADS
Storage in Wet Well above Cut In	174.6 kL	(595.930-592.730) x12.7 +(592.730-590.840) x70.89
Cut In Level	RL 590.840	Hydraulic Model - ADS
Incoming Sewer IL	RL 594.349	Hydraulic Model - IF
Cut Out Level	RL 589.120	Hydraulic Model - ADS
Manhole Floor Level	RL 588.280	Hydraulic Model - ADS
Additional Storage	N/A	

Summary of Storage Availability SPS-MT1

Detail	Value	Source
Ground Level	RL 605.00	Hydraulic Model - AB
Overflow Level	N/A	
Wet Well Shaft Plan Area	15.9 m²	Hydraulic Model - AB
Storage in Wet Well above Cut In	65.2 kL	(605-600.9) x 15.9
Cut In Level	RL 600.900	Hydraulic Model - FT
Incoming Sewer IL	RL 601.080	Hydraulic Model - FT
Wet Well Chamber Plan Area	15.9 m²	Hydraulic Model - AB
Cut Out Level	RL 599.000	Hydraulic Model - FT
Manhole Floor Level	RL 598.530	Hydraulic Model - AB
Additional Storage – GN01625	734.2 kL	(605-601.2) x 193.2
Ground Level	RL605.000	Hydraulic Model - AB
Floor Level	RL601.200	Hydraulic Model
Plan Area	193.20 m²	Hydraulic Model

Appendix B – Pipe Headloss Results

Asset ID	Model ID	Diameter (mm)	Max Headloss (m/km)	
			Pre	+ Dev
Target maximum headloss in reticulation mains (≤ 300 mm) of 5 m/km				
NV03900 (Valve)	NV03900X.NV03900Y.1	300.00	558.07	557.79
NV03902 (Valve)	NV03902X.NV03902Y.1	300.00	49.49	49.47
NV03903 (Valve)	NV03903X.NV03903Y.1	300.00	49.49	49.47
NP01262 (Pipe)	ND00804.ND00805.1	294.60	11.51	11.50
NP01262 (Pipe)	ND02851.ND00804.1	294.60	11.51	11.50
NP01262 (Pipe)	ND03733.ND04542.1	294.60	11.48	11.47
NP01262 (Pipe)	ND04542.NV03541.1	294.60	11.48	11.47
NP01262 (Pipe)	NV03265.ND02851.1	294.60	11.51	11.50
NP01262 (Pipe)	NV03459.NV03265.1	294.60	11.51	11.50
NP01262 (Pipe)	NV03460.NV03459.1	294.60	11.48	11.47
NP01262 (Pipe)	NV03541.NV03655.1	294.60	11.48	11.47
NP01262 (Pipe)	NV03655.NV03460.1	294.60	11.48	11.47
NP02301 (Pipe)	bmm000085.NV04168X.1	102.00	19.45	11.44
NP02301 (Pipe)	ND03992.bmm000086.1	102.00	19.45	11.44
NP02301 (Pipe)	ND03993.NV04166X.1	102.00	19.45	11.44
NP02301 (Pipe)	ND03994.NV04167X.1	102.00	19.45	11.44
NP02301 (Pipe)	NH06564.ND03993.1	102.00	19.45	11.44
NP02301 (Pipe)	NV04166Y.ND02965.1	102.00	19.45	11.44
NP02301 (Pipe)	NV04167Y.ND03992.1	102.00	19.45	11.44
NP02301 (Pipe)	NV04168Y.NH06564.1	102.00	19.45	11.44
NP02176 (Pipe)	ND03706.ND03715.1	101.70	22.86	22.86
NP02176 (Pipe)	ND03715.NH06304.1	101.70	22.86	22.86
NV00099 (Valve)	NV00099X.NV00099Y.1	100.00	10.04	10.17
NV00101 (Valve)	NV00101X.NV00101Y.1	100.00	6.36	6.19
NV00432 (Valve)	NV00432X.NV00432Y.1	100.00	6.48	6.32
NV00434 (Valve)	NV00434X.NV00434Y.1	100.00	10.22	10.36
NV03109 (Valve)	NV03109X.NV03109Y.1	100.00	6.47	6.28
NV03110 (Valve)	NV03110X.NV03110Y.1	100.00	6.47	6.28
NV03846 (Valve)	NV03846X.NV03846Y.1	100.00	35.50	26.13
NV04168 (Valve)	NV04168X.NV04168Y.1	100.00	102308.63	102369.98
NV04654 (Valve)	NV04654X.NV04654Y.1	100.00	13.72	10.10
NV04655 (Valve)	NV04655X.NV04655Y.1	100.00	13.72	10.10
NM00045 (Meter)	bmm000085.bmm000086.1	100.00	21.57	12.68
NV03900 (Valve)	NV03900X.NV03900Y.1	300.00	558.07	557.79

Asset ID	Model ID	Diameter (mm)	Max Headloss (m/km)	
			Pre	+ Dev
NV03902 (Valve)	NV03902X.NV03902Y.1	300.00	49.49	49.47
NV03903 (Valve)	NV03903X.NV03903Y.1	300.00	49.49	49.47
NP01262 (Pipe)	ND00804.ND00805.1	294.60	11.51	11.50
NP01262 (Pipe)	ND02851.ND00804.1	294.60	11.51	11.50
NP01262 (Pipe)	ND03733.ND04542.1	294.60	11.48	11.47
NP01262 (Pipe)	ND04542.NV03541.1	294.60	11.48	11.47
NP01262 (Pipe)	NV03265.ND02851.1	294.60	11.51	11.50
NP01262 (Pipe)	NV03459.NV03265.1	294.60	11.51	11.50
NP01262 (Pipe)	NV03460.NV03459.1	294.60	11.48	11.47
NP01262 (Pipe)	NV03541.NV03655.1	294.60	11.48	11.47
NP01262 (Pipe)	NV03655.NV03460.1	294.60	11.48	11.47
NP02301 (Pipe)	bmm000085.NV04168X.1	102.00	19.45	11.44
NP02301 (Pipe)	ND03992.bmm000086.1	102.00	19.45	11.44
Target maximum headloss in reticulation mains (> 300 mm) of 3 m/km				
NP02181 (Pipe)	bmm000188.NV03907X.1	363.30	3.88	3.88
NP02181 (Pipe)	bmm000314.NV03903X.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03667.ND03668.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03668.ND03669.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03669.ND03670.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03670.ND03711.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03707.ND03667.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03709.ND03707.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03711.bmm000188.1	363.30	3.88	3.88
NP02181 (Pipe)	ND03732.ND03733.1	363.30	3.88	3.88
NP02181 (Pipe)	NV03900X.NV03902X.1	363.30	3.88	3.88
NP02181 (Pipe)	NV03902Y.ND03709.1	363.30	3.88	3.88
NP02181 (Pipe)	NV03903Y.NV03900Y.1	363.30	3.88	3.88
NP02181 (Pipe)	NV03907Y.ND03732.1	363.30	3.88	3.88
NP01880 (Pipe)	ND00807.ND00805.1	363.20	3.89	3.89
NP01880 (Pipe)	ND00830.NV00556X.1	363.20	3.90	3.89
NP01880 (Pipe)	ND02850.NV03274.1	363.20	3.89	3.89
NP01880 (Pipe)	ND02854.NV03267.1	363.20	3.89	3.89
NP01880 (Pipe)	ND02855.ND02856.1	363.20	3.89	3.89
NP01880 (Pipe)	ND02856.NV03462.1	363.20	3.89	3.89
NP01880 (Pipe)	ND02937.NV03779.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02981.ND02983.1	363.20	3.90	3.89

Asset ID	Model ID	Diameter (mm)	Max Headloss (m/km)	
			Pre	+ Dev
NP01880 (Pipe)	ND02983.ND02850.1	363.20	3.89	3.89
NP01880 (Pipe)	ND02984.ND02985.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02985.NV03197.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02986.ND02984.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02987.ND02986.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02988.ND02987.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02994.NV03776X.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02995.NV03778.1	363.20	3.90	3.90
NP01880 (Pipe)	ND02997.NV03775.1	363.20	3.90	3.90
NP01880 (Pipe)	ND03743.ND02988.1	363.20	3.90	3.90
NP01880 (Pipe)	NV00556Y.ND02981.1	363.20	3.90	3.89
NP01880 (Pipe)	NV03196.ND00830.1	363.20	3.90	3.89
NP01880 (Pipe)	NV03197.NV03196.1	363.20	3.90	3.89
NP01880 (Pipe)	NV03267.ND02855.1	363.20	3.89	3.89
NP01880 (Pipe)	NV03274.ND02854.1	363.20	3.89	3.89
NP01880 (Pipe)	NV03462.ND00807.1	363.20	3.89	3.89
NP01880 (Pipe)	NV03775.ND02994.1	363.20	3.90	3.90
NP01880 (Pipe)	NV03776Y.ND02995.1	363.20	3.90	3.90
NP01880 (Pipe)	NV03778.ND02937.1	363.20	3.90	3.90
NP01880 (Pipe)	NV03779.NV03780.1	363.20	3.90	3.90
NP01880 (Pipe)	NV03780.ND03743.1	363.20	3.90	3.90