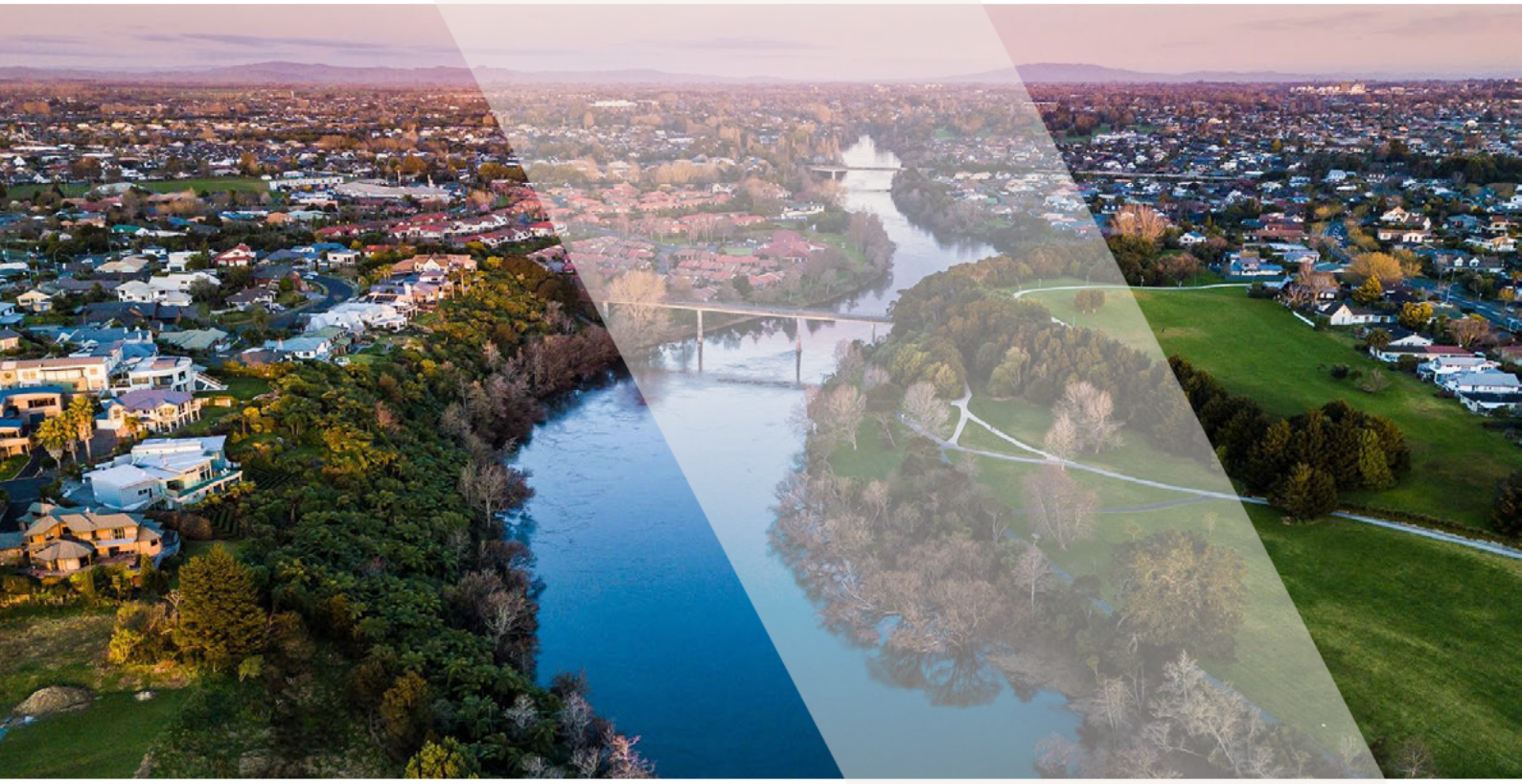
Appendix 19 – Infrastructure Report

ENGINEERING REPORT



Harlow Lifestyle Retirement Village

CIVIL ENGINEERING ▾ SURVEYING ▾ LAND DEVELOPMENT

PROJECT INFORMATION

CLIENT	Harlow Lifestyle Village Ltd
PROJECT	411001

DOCUMENT CONTROL

DATE OF ISSUE	23/10/2026
REVISION	A
AUTHOR	[redacted] Grayson Hayward Engineer
REVIEWED BY	[redacted] Sean McDonald Team Leader
APPROVED BY	[redacted] Dean Morris Director

© Maven Waikato Ltd 2026
This document is and shall remain the property of Maven Ltd. The document may only be used for the purposes for which it was commissioned and in accordance with the Terms of Engagement for the commission. Unauthorised use of this document in any form whatsoever is prohibited.

19 Kaimiro Street, Pukete
New Zealand
Phone 07 242 0601
www.maven.co.nz

Contents

1. Statement of Experience 5
2. EXECUTIVE SUMMARY 6
   - 2.1. Purpose and Context 6
   - 2.2. Design Philosophy and Targets 6
   - 2.3. Stormwater 7
   - 2.4. Wastewater 7
   - 2.5. Water supply 8
   - 2.6. Earthworks and Roading 8
   - 2.7. Staging and Delivery 8
   - 2.8. Conclusion 9
3. Introduction 10
   - 3.1. Background 10
   - 3.2. Purpose of this report 10
   - 3.3. Site Description and Project Summary 10
   - 3.4. Legal Description 11
4. Earthworks 12
   - 4.1. Geotechnical Report 12
   - 4.2. Landforms 12
   - 4.3. Ground Water 13
   - 4.4. Soakage Rates 13
   - 4.5. Earthworks Staging & Volumes 14
   - 4.6. Slope Stability 17
   - 4.7. Building Foundations 18
   - 4.8. Sediment and Erosion Control 18
5. Roading 19
   - 5.1. Design Standards 19
   - 5.2. Integrated Transportation Assessment 19
   - 5.3. Access 19
   - 5.4. Road Typology & Design 20
   - 5.5. HORIZONTAL GEOMETRICS 25
   - 5.1. VERTICAL GEOMETRICS 26
   - 5.2. Parking 27
   - 5.3. Walking and Cycling 27
   - 5.4. Lighting 28
6. Stormwater 29
   - 6.1. Design Standards 29
   - 6.2. Stormwater Management Plan (SMP) 30
   - 6.3. Stormwater System 31
   - 6.4. Overland Flow Paths (OLFPs) 31
   - 6.5. Capacity and Quality 34
   - 6.6. Flooding 34
   - 6.7. Connection Points and Staging 36
7. Wastewater 37
   - 7.1. Wastewater Network Capacity 37
   - 7.2. Servicing Options 37
   - 7.3. Proposed Network 37
   - 7.4. Pump Stations 38
   - 7.5. Proposed Wastewater Reticulation 38
   - 7.6. Staging 39
   - 7.7. Lot Connections 41
   - 7.8. Wastewater Pump Station 41
8. Water 45
   - 8.1. Design Standards 45

8.2. Existing Fire Hydrant Testing 45
8.3. Design Flows 45
8.4. Water Supply modelling 45
8.5. Strategic Water Connection 47
8.6. Proposed Reticulation 47
8.7. Lot Connections 48
8.8. Connection Points and Staging 48
8.9. Firefighting Supply 48
9. Services 50
9.1. Power 50
9.2. Communications 50
9.3. Gas 50
10. Conclusion 51
Appendix A – Engineering Plans 52
Appendix B – Integrated Transport Assessment 53
Appendix C – Water Supply Calculations 54
Appendix D – Wastewater Calculations 55
Appendix E – Geotechnical Reports 56
Appendix F – Stormwater Management Plan (SMP) 57
Appendix G – BTW/WPS Wastewater Growth Modelling Report 58
Appendix H – BTW/WPS Water Supply Growth Modelling Report 59

1. Statement of Experience

Author

My name is Grayson Hayward, and I am a civil engineer at Maven Waikato Ltd, a consulting engineering and surveying firm based in Hamilton. I have been employed with Maven since 2026.

I hold a Bachelor of Engineering Technology, graduating in 2022, and I am a member of Engineering New Zealand Te Ao Rangahau. I have worked as a civil engineer in consulting for over three years, gaining experience across a wide range of land development and infrastructure projects.

My project experience includes rural residential, residential, industrial, and commercial land development, as well as three-waters infrastructure design and three-waters management. This work involves design, modelling, assisting with consenting, construction support and supervision, and multidisciplinary coordination across various development scales.

I confirm I have read and understood the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and agree to comply with its requirements. I acknowledge that, as an expert, I have an overriding duty to assist the decision-maker impartially on matters within my area of expertise. In preparing this report, I have exercised my independent profession judgement, and the opinions for the interests of the applicant or any other party.

I confirm that the matters addressed in this report are within my area of expertise, except where I have expressly identified that I have relied upon information, assessments or opinions provided by others. I have made all reasonable enquiries and have not knowingly omitted any material facts known to me that might alter or detract from the opinions expressed in this report.

Reviewer

My name is Sean McDonald. I am a Senior Engineer and Team Leader at Maven Waikato Ltd ("Maven"), a firm of consulting engineers and surveyors based in Hamilton. I have been employed by Maven since 2025. I hold a Bachelor of Engineering Technology (Highways) which I graduated from in 2006, hold a New Zealand Certificate of Engineering (NZCE) and I am a member of Engineering New Zealand.

I have over 29 years of experience in the field of civil engineering including as a civil engineering technician, New Zealand army engineer, council engineer and consulting engineer, with over 20 years of experience post degree. Since obtaining my degree I have worked on a variety of civil projects including State Highway works, 3 Waters Infrastructure works, residential land development, commercial and industrial land development and major infrastructure projects such as the upgrade of the Port Botany (Sydney) Naphtha Storage Facilities, the Transpower upgrade and extension of the Whakamuru Substation, the Central Government Rotorua Youth Justice Facility and Contact Energy's new Tauhara Geothermal power station.

I confirm I have read and understood the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 and agree to comply with its requirements. I acknowledge that, as an expert, I have an overriding duty to assist the decision-maker impartially on matters within my area of expertise. In preparing this report, I have exercised my independent profession judgement, and the opinions for the interests of the applicant or any other party.

I confirm that the matters addressed in this report are within my area of expertise, except where I have expressly identified that I have relied upon information, assessments or opinions provided by others. I have made all reasonable enquiries and have not knowingly omitted any material facts known to me that might alter or detract from the opinions expressed in this report.

2. EXECUTIVE SUMMARY

2.1. Purpose and Context

This Infrastructure Report (IR) supports the substantive resource consent application for the Harlow Lifestyle Retirement Village (Harlow LRV or the Project) Residential Development— an approximately 25.78-hectare greenfield site on Te Awamutu's northern fringe, delivering approximately 414 residential units. The IR has been prepared in accordance with the Regional Infrastructure Technical Specifications (RITS V2, October 2025), relevant New Zealand and Austroads standards and accompanies the application under the Fast-Track Approvals Act 2024.

The site is located at 2025 Ōhaupō Road, Te Awamutu, which also serves as State Highway (SH). The site consists of two parcels totalling approximately 25.78 hectares, currently in rural use. The site contains areas of natural vegetation, existing wetlands, watercourses and an area identified as Significant Natural Area (SNA). These features form important components of the existing environment and have shaped the infrastructure strategy across all disciplines.



Figure 1: Masterplan (provided by Pacific Environment Architects 2026)

2.2. Design Philosophy and Targets

The infrastructure strategy adopts an integrated, multi-disciplinary approach with stormwater incorporating Water Sensitive Urban Design principles through a treatment train of centralised raingardens, constructed artificial wetlands and enhanced existing onsite flood attenuation basins/areas. Core design targets are:

- Stormwater: Retention of the water quality volume (1.3 mm) via centralised raingardens and soakage; WQV and EDV within the artificial wetland ponds; attenuation of the 100-year ARI climate change event (RCP 8.5, 2081–2100) to no more than 80 percent of pre-development discharge by utilising existing designated flood basins within the site constraints.
- Wastewater: Gravity reticulation to two new pump stations, each with N+1 pumped duty, enhanced storage exceeding the RITS 9-hour minimum and rising main connection to the Te Awamutu network.
- Water Supply: Staged connections achieving minimum 200 kPa at all nodes, with FW2/FW3 firefighting supply (12.5 L/s primary, 12.5 L/s secondary, 100 kPa minimum running pressure).
- Roading: State Highway intersection upgrade, collector, and local roads to Austroads and RITS, with pavement subgrade improvement layers for compressible ground.
- Earthworks: Cut-to-fill platform creation with excess cut material to be removed off site.

2.3. Stormwater

Stormwater is managed through four stages:

Stage 1 Primary Conveyance: Stormwater piped infrastructure network sized for the 10-year ARI event convey flows to the both the centralised raingardens & artificial wetlands. Overland flow within road corridors to pass the 100-year event; the maximum depth×velocity product of 0.15 m²/s is well within Austroads safety thresholds.

Stage 2 Raingardens: Five centralised raingardens located within each stormwater catchment to capture, treat and soak the initial abstraction, prior to each artificial wetland pond.

Stage 3 Treatment and Attenuation: Five (5) artificial wetlands to provide sediment capture (via forebays), water quality treatment and extended detention:

Wetland	Development Stage	Type	Area (ha)	Ownership
Wetland A	1	Road & Lot Combined	6.40	Private
Wetland B	2	Road & Lot Combined	5.42	Private
Wetland C	3	Road & Lot Combined	2.72	Private
Wetland D	4	Road & Lot Combined	1.31	Private
Wetland E	5	Road & Lot Combined	1.86	Private

Table 1: Wetland Overview

Stage 4 Controlled Discharge; Treated flows discharge at 80 percent of pre-development rates:

- Wetland A to the existing artificial farm ponds located within stage 1 extents
- Wetland B to the existing farm drainage network within the site constraints
- Wetland C to the existing farm drainage network within the site constraints
- Wetland D to the existing farm drainage network outside the site constraints; and
    - As wetland D discharges outside the sites designated flood attenuation control structures (Culverts) and flood attenuation basins/areas, wetland D has been sized to attenuate the 1in 100yr event within the wetland.
- Wetland E to the existing farm drainage network within the site constraints

2.4. Wastewater

Two wastewater pump stations (which have been designed with using the RITS) provide internal conveyance across the development. Both stations are elevated with a minimum 300 mm freeboard above the 1% AEP flood level, provide N+1 pumped duty, and are contained within either separate local purpose reserve (Pumpstation A -Lot 5005) or within the common property area (Pumpstation).

- Pump Station A (Stage 1): Serving 40 residential units and the managed care facility (186 population equivalent) with a peak design flow of 2.47 L/s. Combined storage of 13.95 m³ provides emergency redundancy and buffering capacity for off-peak discharge. Flows are pumped via a new 110mm Ø ODPE 100 SDR 13.6 PN12.5 rising main to the Greenhill Drive wastewater manhole (Manhole Asset ID 1090025) — the agreed outfall location with Waipa District Council and Waikato Waters.
- Pump Station B (Stages 2, 3, and 4): Serving 374 residential units (715 population equivalent) with a peak design flow of 11.09 L/s. Combined storage of 53.63 m³ provides emergency redundancy and buffering capacity for off-peak discharge. Flows are pumped via a new 180mm Ø ODPE 100 SDR 13.6 PN12.5 rising main to the 2025 Ōhaupō Road wastewater manhole

(Manhole Asset ID 1090999) — the agreed outfall location with Waipa District Council and Waikato Waters.

Internal reticulation uses 150mmØ uPVC SN16 gravity pipelines. The design has been based on 200l/p/d for residential in accordance with RITS V2 (October 2025) and 30l/p/d for the managed care facility in accordance with Watercare code of practice.

The existing Te Awamutu wastewater network has known capacity constraints. In response, detailed modelling was undertaken pre application, and on-going discussion with both Waipa District Council and Waikato Waters have identified the above options as having sufficient capacity within the existing network to cater for both future growth demand and the proposed development, (refer to appendix H).

2.5. Water supply

The existing Te Awamutu water supply network has known capacity constraints. In response, detailed modelling was undertaken pre application and identified the following options as being required to ensure the existing network has sufficient capacity to cater for both future growth demand and the proposed development, (refer to Appendix H).

Option 1: Upgrade the two Non-Return Valves (RVs) on Taylors Avenue: and

Option 2: Replace the current Greenhill booster pump and add a secondary “Standby Pump”.

In discussion with both Waipa District Council and Waikato Waters, it has been agreed that as part of the proposed development both Option 1 and Option 2 will be incorporated into the Harlow LRV Development water supply infrastructure design

Option 3: Additional modelling has identified the 200mmØ watermain upgrade should be upsized to a 315mmØ PE watermain – due to the inclusion of a manage care facility and associated firefighting supply requirements.

2.6. Earthworks and Roading

Earthworks encompass 20.15 ha of disturbance, 217,320m³ solid cut, 143,941m³ solid fill, and a 127,378m³ excess solid cut to waste (no allowance for bulking/compaction factors). Erosion and sediment control complies with TR2009-09. Local Road Pavement designs (30-50mm DG7 orDG10 asphaltic concrete over 350–425 mm granular pavement with 300 mm subgrade improvement layer) are based on CBR 5.0. The roading network — designed in conjunction with Boffa Miskell — delivers road connectivity via the new SH 3 roundabout, an 18-20m corridor collector road as the internal spine, and local roads with shared paths, footpaths, and indented parking. Specific NZTA pavement design will be confirmed at detailed design stage.

2.7. Staging and Delivery

The development is structured across four stages, each designed to operate independently with some stages having internal substages:

- Stage 1: Wetland A (Catchment A), Pump Station A. SW discharge to existing artificial; wastewater pumped to Ōhaupō Road wastewater manhole; water supply connection and upgrades to Taylors Hill water reservoir and valves.
    - Noting that Stage 2 earthworks are required prior to Stage 1 substages 1D and 1E progressing
    - Noting that Stage 2 Pumpstation and associated gravity main are required prior to Stage 1 substages 1D and 1E progressing

- Stage 2: Wetland B and C (Catchment B & C), Pump Station B; SW discharge to farm drainage network; wastewater pumped to Ōhaupō Road wastewater manhole; water supply extended.
- Stages 3: Wetland E (Catchment E), Pump Station B gravity network extended; SW discharge to farm drainage network; water supply extended.
- Stages 4: Wetland D (Catchment D), Pump Station B gravity network extended; SW discharge to farm drainage network; water supply extended.

Public assets (road, drainage, water mains) will be vested to Council were appropriate; private assets (road, drainage, water mains, raingardens and artificial wetland ponds) remain in private ownership. Power and telecommunications will be underground within the road reserve, coordinated with WAIPA Networks and both Chorus & Tuatahi First Fibre.

2.8. Conclusion

The Harlow LRV Development Infrastructure Report demonstrates a comprehensive, integrated servicing framework that addresses the site's defining constraints namely falling topography, existing network deficiencies, accessibility and existing wetland/drainage features. This has been achieved through a staged, gravity-preferring strategy that is compliant with RITS V2 (2025) and all relevant Council and regional standards. No engineering exceptions are proposed. Residual matters for detailed design including NZTA final roundabout design approval, council infrastructure connection confirmation and overall site accessibly to the surrounding networks do not impact the feasibility of the strategy or the suitability of the site for the proposed residential development.

3. Introduction

3.1. Background

Maven Waikato Ltd have been engaged to undertake Civil Design in support of the substantive application for the Harlow Lifestyle Retirement Village Residential Development project (Harlow LRV or the Project) under the Fast-track Approvals Act 2024 (FTAA).

3.2. Purpose of this report

The purpose of this report is to provide an infrastructure report (IR) for the Harlow LRV Development at 2025 Ōhaupō Road, Te Awamutu. The information provided herein outlines the methodology associated with the proposed earthworks and infrastructure onsite. This report is to be read in conjunction with the engineering drawings and is to accompany the Substantive Application under the FTAA.

3.3. Site Description and Project Summary

The site consists of circa 25.78 hectares of land located to the west of Ōhaupō Road (SH3) and on the northern fringe of the Te Awamutu urban boundary. The site is currently used as rural land mainly as a dry stock farm however current State Highway access restrictions prevent the site from more extension farming operations. The landholding includes two parcels held in One Record of Title, Part Lot 1 Deposited Plan 35654 and Lot 1 Deposited Plan 36696 with a combined area of circa 25.78ha.

The Project seeks approval for a substantive application under the FTAA for a comprehensively master planned and staged industrial and residential development. This includes all necessary resource consents under the Resource Management Act 1991.

The Harlow LRV Development provides for the establishment of approximately 414 units, together with infrastructure devices including centralised raingardens, artificial wetlands, 3 water drainage infrastructure and roading. The Harlow LRV Development is underpinned by a series of design principles, with a focus on creating a well-connected, legible, and diverse residential area on Te Awamutu's northern fringe. This is supported via a coordinated and connected roading and pedestrian/cycle network, and stormwater outcomes, as well as varying retirement unit sizes and types, in conjunction with the serviced apartments, managed care facility and community facilities which have been proposed to meet the growing and changing needs of the greater Waikato retirement lifestyle market.

The Harlow LRV Development includes substantial enabling works to establish developable land, including:

- bulk earthworks across the site;
- retaining structures
- roading networks
- new wastewater and water supply infrastructure including new wastewater pumpstations;
- an integrated stormwater management system consisting of conveyance systems such as piped stormwater infrastructure, as well as integrated detention/attenuation and treatment devices that will be combined into artificial wetlands.

Access and integration with the surrounding Te Awamutu area is provided through a connected transport framework, including the establishment of intersections, roading, and walking and cycle networks, along with upgrades to existing transport infrastructure. The Harlow LRV Development is proposed to be serviced via a coordinated strategy, including the construction of a new SH3 four arm

roundabout extension of the proposed pedestrian connectivity network to Green Hill Drive, and the upgrade of the Greenhill Drive/Taylors Ave footpath back to Racecourse Road, upgrades to portions of the existing water supply network and installation of appropriate infrastructure.

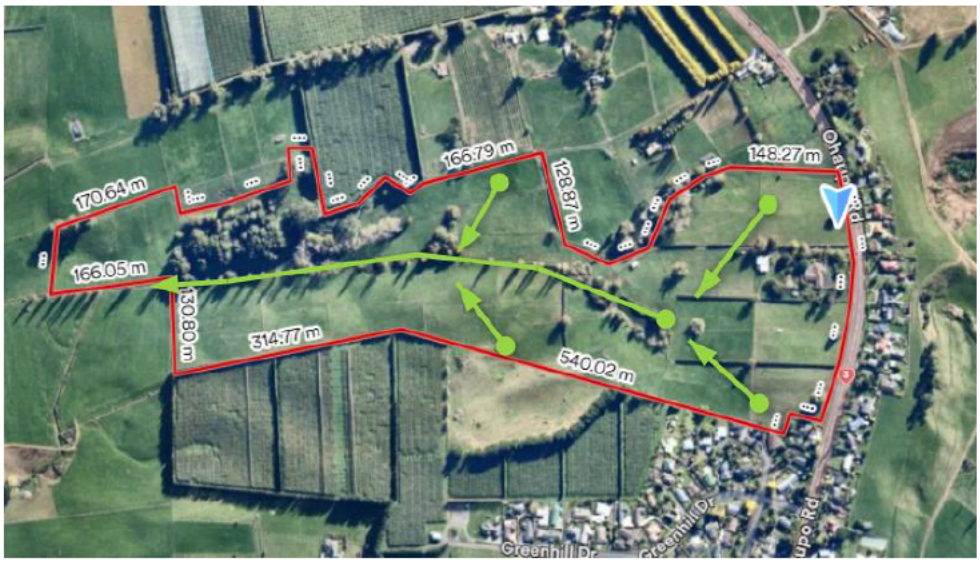


Figure 2: Subject site

3.4. Legal Description

The site consists of two parcels held in One Record of Title as shown in Table 2. The site can be accessed by State Highway 3 (Ōhaupō Road) to the east.

Address	Lot Description	Area (Ha)
2025 Ōhaupō Road, Te Awamutu 3800	PT LOT 1 DP 35654 LOT 1 DPS 36696 BKL II PUNIU SD	25.7824
Made up Off		
Parcel ID 4352868	Part Lot 1 DP 35654	16.6614
Parcel ID 4538250	Lot 1 DPS 36696	9.121
	Total Area (Ha)	**25.7824**

Table 2: Lot Description Table

4. Earthworks

Earthworks will include cut to fill operations for the construction of residential lot, apartment, recreation facilities and managed care facilities areas, excavations for pavement construction, excavations for the artificial wetland construction, excavations for the wastewater pumpstations and associated drainage reticulation, formation of building platforms. The Harlow LRV Development area will contain specific erosion and sediment control measures suitable to control the earthworks area.

4.1. Geotechnical Report

A Geotechnical Suitability Report has been prepared by Titus Consulting Engineers (Ref No: C3102) and BTW Company (Ref No: Rev A – 24/11/2022) for the Harlow Lifestyle Village. The site is characterised by rolling hill with Walton Subgroup (Hamilton Ash) and the low-lying areas consisting of Holocene swamp deposits (Alluvium) and saturated clay. The investigations confirm that compressible ground conditions are present across the low-lying portions of the site.

The geotechnical assessment showed that fill-induced settlement within the low-lying areas could be significant with preliminary predicted settlements of approximately up to 40mm without mitigation or preloading. Ground Improvement pf up to 5m of backfill is recommended for the low-lying areas of the site to avoid any surface expression of liquefaction. No ground improvement is necessary for the rolling hills.

Backfilling will be undertaken progressively within the identified soft-ground areas. Settlement monitoring will be established prior to loading and maintained throughout the earthworks, duration and removal subject to geotechnical review and agreed hold points. The observational approach allows the surcharge programme to be refined in response to measured settlement behaviour and ground conditions meaning throughout the phase a refined approach of backfill will be incorporated into the phasing of earthworks.

The general site geological composition, from greatest to least %, is as follows:

- Hamilton ash formation; stiff to very stiff clayey silt, with varying degrees of clay and silt
- Walton subgroup; weathered residual soil of very stiff silt with clay and/or fine sand and weathered pumice gravel
- Hinuera formation; low-density sandy soils over denser sand layers
- Low density colluvium at slope toes, gullies and stream channels
- Expected historic farm filling

The Hamilton ash formation soils encountered on site have been assessed for re-use suitability and are generally suitable to achieve the requirements to be re-used as engineered fill across site. For further information of the existing soil typology and suitability, see Appendix E.

4.2. Landforms

The site's landform is composed of three major geomorphologies; alluvial plains, gully landforms and low hills landform. The site slopes have been classed, in approximately 5m squares, as below:

Landscape Unit	Total %
Near level to gently undulating	19
Undulating land	42
Rolling land	35
Hilly land	3
Steep land	1

Total	100

Table 3: Landform Unit Composition

4.3. Ground Water

The preliminary geotechnical investigations were undertaken to provide full coverage of the Harlow Village development area. The investigations cover assessed ground water levels, soil soakage capacity, liquefaction potential, and lateral spreading risks, ensuring comprehensive data for stormwater management, earthworks, Roading and foundation design (refer to Appendix E for the Geotechnical reports).

1. 211365 Rev A (dated 24/11/2022)

The site is characterised as rolling low hills bisected east-to-west by a central gully that feeds two existing ponds, wetlands, and a small stream, which ultimately flows into the surrounding alluvial plains. Overall, underlain by Walton Subgroup along the low hills and the gullies, low-lying areas are underlain by the Hinuera Formation, flanked by rolling hills to the north and south, comprised of Hamilton ashes, with ground water levels varying seasonally and across different areas.

The findings provide crucial insights into the site's Geotechnical behaviour, have informed the design completed to date and will inform future design considerations. The key findings are summarised below.

Ground Water Levels

- Depths recorded 0.2m below existing ground level in low lying areas
- Groundwater was not encountered within 2m of the surface across the elevated low hills of the site.

4.4. Soakage Rates

The BTW company LTD conducted two types of field tests to determine the soakage rates at site: Falling Head tests. The results are as detailed in below.

Soakage Test Unit	Site Location	Results (mm/hr)	Soakage Rate factored by 0.5 as per RITS for conservatism (mm/hr)
CPT 1	Stage 1B	16	8
CPT 2	Stage 1A	120	60
CPT 3	Stage 2A	156	78
CPT 5	Stage 3	480	240
CPT 7	Stage 3	1,200	600
CPT 10	Stage 4	96	48

Table 4: Soakage tests Results from Falling Head

2. WGA260929-RP-HG-0010_A (dated August 2026)

The site is characterised as irregular shaped rectangle with rolling and undulating hills, with a gully running east to west which form a dendritic valley pattern. Overall, underlain by Walton Subgroup along the hills, while the stream valley is underlain by Hinuera Formation.

WGA conducted field hydraulic testing and produced hydraulic conductivity at varied locations across the site, which have been interpolated to soakage rate for a better understanding of the site.

Piezometer ID	Site Location	Hydraulic Conductivity (m/sec)	Results (mm/hr)
PZ26-01	Stage 1A	4.74×10^{-8} to 2.94×10^{-7}	0.17 to 1.06
PZ26-02	Stage 1E	2.75×10^{-7} to 1.05×10^{-6}	0.99 to 3.78
PZ26-03	Stage 3	5.76×10^{-7} to 1.34×10^{-6}	2.07 to 4.82

Table 5: Interpolated Soakage Results from Hydraulic Conductivity data

The testing results from both investigations indicate varied soakage performance across the site, with notably low rates, such as 8 mm/hr, recorded closer to the eastern boundary. Observations from soakage testing and variable groundwater depths indicate that perched layers are likely present in parts of the site.

The results in the Geotechnical report, shows soils at this site are generally suitable to provide use of SW Devices such as raingarden etc. This soakage rate has been derived from double ring infiltration test and falling head test results, the results are averaged to adopt it for design purposes and factored by 0.5 to reflect long term reduction in soakage over time.

4.5. Earthworks Staging & Volumes

The earthworks be constructed in 4 major stages with staging intended to balance the cut/fill volumes onsite with the proposed overall development master plan and associated infrastructure staging.

The expected overall "bulk volumes" will vary depending on compaction factors to achieve desired compaction strengths, unsuitable materials encountered and infrastructure construction such as pavement excavations, drainage and services trenches etc, but it is estimated to have a surplus of cut material from the balance across the development. The total expected volumes are detailed below:

TOTAL EARTHWORKS AREA	206,187	m^2
TOTAL TOPSOIL STRIP (assume 200mm)	48,253	m^3
TOTAL TOPSOIL RESPREAD (assume 150mm)	38,336	m^3
EXCESS TOPSOIL (assumed can be used onsite as landscape/contouring material)	9,917	m^3
TOTAL CUT	282,050	m^3
TOTAL FILL	143,942	m^3
TOTAL SURPLUS MATERIAL	138,108	m^3
TOTAL GRANULAR PAVEMENT MATERIAL	26,846	m^3

Table 6: Expected Earthworks Balance

Note volumes are from stripped topsoil surface (existing) to assumed design subgrade surface, with no allowance for drainage and services trenches. Volumes solid measure and unfactored.

It is noted that while the excess material offsite may seem large, several major developments located to the south of Hamilton City within the Southern Links area require large volumes of "Clean Fill" due to their geological settings, as such there is an opportunity to provide synergy between these projects that can be explored once consent is received.

4.5.1. Earthworks Stage 1

Stage 1 earthworks will include the construction of the Ōhaupō Road (SH3) roundabout, internal development stages 1A to 1D and Stage 2A (of the master plan) and include the removal of 2 existing artificial farm ponds, the construction of artificial stormwater wetland pond A1 and all associated structural retaining wall structures. No culvert crossing of the existing farm drain are to be constructed in this earthworks stage.

Drainage blankets and/or subsoil drains will be required at the base of the existing artificial ponds to continue to convey any groundwater flows through to downstream wetlands/drains prior to fill placement commencing.

Retaining walls will be required to facilitate the earthworks on the site due to the contours of Stage 1, including for the construction of artificial wetland pond A1, however some retaining structures that are deemed no essential for Stage 1A and 1B or the artificial wetland pond A1, will be left as stabilised cut batters, until such time that each master plan substage is constructed.

The site will be accessed from the existing farm entrance, and the majority of the proposed roundabout intersection will be constructed offline. Once the majority of the roundabout is constructed then the State Highway traffic can be diverted through the temporary roundabout works, as the remainder of the roundabout and the access road to the existing eastern SH3 properties (Road 02) are constructed.

Excess material from Stage 1 earthworks activities will be stockpiled in Stage 2 (refer to ESC plans) until such time as the Ōhaupō Road (SH3) roundabout is full formed and operation, as to prevent excess truck movements onto the State Highway network:

The total Stage 1 (excluding NZTA road corridor works) expected volumes are detailed below:

TOTAL EARTHWORKS AREA	89,211	m^2
TOTAL TOPSOIL STRIP (assume 200mm)	19,753	m^3
TOTAL TOPSOIL RESPREAD (assume 150mm)	19,610	m^3
EXCESS TOPSOIL (assumed can be used onsite as landscape/contouring material)	143	m^3
TOTAL CUT	153,967	m^3
TOTAL FILL	77,736	m^3
TOTAL SURPLUS MATERIAL	76,231	m^3
TOTAL GRANULAR PAVEMENT MATERIAL	7,765	m^3

Table 7: Expected Stage 1 Earthworks Balance

Note volumes are from stripped topsoil surface (existing) to assumed design subgrade surface, with no allowance for drainage and services trenches. Volumes solid measure and unfactored.

The total Stage 1 NZTA Corridor works (excluding Stage 1 road works) expected volumes are detailed below:

TOTAL EARTHWORKS AREA	8,953	m^2
TOTAL CUT	8,111	m^3
TOTAL FILL	44	m^3
TOTAL SURPLUS MATERIAL	8,067	m^3

TOTAL GRANULAR PAVEMENT MATERIAL	8,483	m^3

Table 8: Expected NZTA Corridor Earthworks Balance

Note volumes are from existing surface (non-stripped) to assumed design subgrade surface, with no allowance for drainage and services trenches. Volumes solid measure and unfactored.

4.5.2. Earthworks Stage 2

Stage 2 earthworks will include the construction the internal development stages 2A to 2C and the fill portion of Stage 4 (of the master plan), the construction of artificial stormwater wetland pond B1 and C1 and all associated structural retaining wall structures. Three (3) culverts crossing of the existing farm drain are to be constructed in this earthworks stage.

To facilitate further development after Stages 1A, 1B and 1E, the pumpstation on stage 4 will need to be constructed to allow wastewater flows to be discharged back to the Waipa District Council network. As such earthworks Stage 2 will need to be completed prior to Stage 1C, 1D of the master plan infrastructure being completed.

Retaining walls will be required to facilitate the earthworks on the site due to the contours of Stage 2, including for the construction of artificial wetland pond B1 and C1, and the construction of 3 new culverts, however some retaining structures that are deemed non-essential for Stage 2A or the artificial wetland ponds B1 & C1, or the Stage 4 wastewater pumpstation will be left as stabilised cut batters, until such time that each master plan substage is constructed.

Excess material from Stage 2 earthworks activities can be stockpiled in Stage 4 (refer to ESC plans) until such time as the it is deemed commercially viable to remove off site.

The total Stage 2 expected volumes are detailed below:

TOTAL EARTHWORKS AREA	76,471	m^2
TOTAL TOPSOIL STRIP (assume 200mm)	19,016	m^3
TOTAL TOPSOIL RESPREAD (assume 150mm)	12,813	m^3
EXCESS TOPSOIL (assumed can be used onsite as landscape/contouring material)	6,203	m^3
TOTAL CUT	72,506	m^3
TOTAL FILL	52,924	m^3
TOTAL SURPLUS MATERIAL	19,582	m^3
TOTAL GRANULAR PAVEMENT MATERIAL	8,116	m^3

Table 9: Expected Stage 2 Earthworks Balance

Note volumes are from stripped topsoil surface (existing) to assumed design subgrade surface, with no allowance for drainage and services trenches. Volumes solid measure and unfactored.

4.5.3. Earthworks Stage 3

Stage 3 earthworks will include the construction the internal development stage 3 (of the master plan), the construction of artificial stormwater wetland pond E1 and all associated structural retaining wall structures. One (1) culvert crossing of the existing farm drain are to be constructed in this earthworks stage.

Retaining walls will be required to facilitate the earthworks on the site due to the contours of Stage 3, including for the construction of artificial wetland pond E1 and internal building platform formation.

Excess material from Stage 3 earthworks activities can be stockpiled in Stage 4 (refer to ESC plans) until such time as the it is deemed commercially viable to remove off site.

The total Stage 3 expected volumes are detailed below:

TOTAL EARTHWORKS AREA	22,460	m^2
TOTAL TOPSOIL STRIP (assume 200mm)	7,269	m^3
TOTAL TOPSOIL RESPREAD (assume 150mm)	4,491	m^3
EXCESS TOPSOIL (assumed can be used onsite as landscape/contouring material)	2,778	m^3
TOTAL CUT	29,089	m^3
TOTAL FILL	9,633	m^3
TOTAL SURPLUS MATERIAL	19,456	m^3
TOTAL GRANULAR PAVEMENT MATERIAL	1,168	m^3

Table 10: Expected Stage 3 Earthworks Balance

Note volumes are from stripped topsoil surface (existing) to assumed design subgrade surface, with no allowance for drainage and services trenches. Volumes solid measure and unfactored.

4.5.4. Earthworks Stage 4

Stage 4 earthworks will include the construction the internal development stage 3 (of the master plan), the construction of artificial stormwater wetland pond E1 and all associated structural retaining wall structures. No culvert crossing of the existing farm drain are to be constructed in this earthworks stage.

Retaining walls will be required to facilitate the earthworks on the site due to the contours of Stage 4, including for the construction of artificial wetland pond D1 and internal building platform formation.

Excess material will need to be cut directly offsite during construction.

The total Stage 4 expected volumes are detailed below:

TOTAL EARTHWORKS AREA	9,092	m^2
TOTAL TOPSOIL STRIP (assume 200mm)	2,216	m^3
TOTAL TOPSOIL RESPREAD (assume 150mm)	1,422	m^3
EXCESS TOPSOIL (assumed can be used onsite as landscape/contouring material)	794	m^3
TOTAL CUT	18,376	m^3
TOTAL FILL	3604	m^3
TOTAL SURPLUS MATERIAL	14,772	m^3
TOTAL GRANULAR PAVEMENT MATERIAL	1,315	m^3

Table11: Expected Stage 4 Earthworks Balance

Note volumes are from stripped topsoil surface (existing) to assumed design subgrade surface, with no allowance for drainage and services trenches. Volumes solid measure and unfactored.

4.6. Slope Stability

Historically, the site was examined to have no obvious active or historical rotational slip or slumps observed. The site slopes are in a state of stability for the current existing land-use. Sediment and erosion controls will need to be utilized to prevent the mobilization of soils.

4.7. Building Foundations

The buildings proposed across site are composed of multi-story buildings (apartments, duplexes), a care center and normal residential buildings. These buildings have been assessed for settlement, fill suitability and liquefaction. These results can be viewed in Appendix E.

4.8. Sediment and Erosion Control

Prior to commencing earthwork operations, it is anticipated that a pre-construction meeting with the Waikato Regional Council monitor team will take place. During this meeting, the erosion and sediment control measures will be discussed and confirmed to ensure that works are to be undertaken in accordance with relevant conditions of consent and the Construction Management Plan (CMP) to ensure earthworks and erosion are effectively mitigated.

Erosion and sediment control measures will be implemented in accordance with TR2009/09 – Guidelines for Soil Disturbing Activities in the Waikato Region. The proposed controls will include clean water diversion drains to prevent runoff entering disturbed areas, sediment retention devices to capture sediment-laden runoff, stabilised construction entrances to minimise sediment tracking onto public roads.

All erosion and sediment control measures have been designed to ensure compliance with TR2009-09 - Guidelines for Soil Disturbing Activities for the Waikato Region. For a comprehensive understanding of the specific application of sediment and erosion control measures for each area of earthworks, please consult the CMP which provides detailed guidance on these measures.

Refer to the Construction Management Plan (CMP) and Earthworks Management Plan EWMP) s for more details on the proposed Sediment & Erosion controls

5. Roading

The Harlow Lifestyle Village is a site comprised of circa 25.78ha of existing Greenland, proposed to provide retirement units, a care home, apartments and a community hub. The development will be constructed over four major stages. A priority for the development's internal roading network is safety, manoeuvrability and multi-modality. These priorities help to support the future residents of the development and their integration with the wider community.

5.1. Design Standards

Boffa Miskell are the lead Transportation Engineers for the Project and have provided the technical support with the roading design. The design provided is for resource consent purpose only and final design for the engineering approval stage will be necessary.

The roading network has been designed in accordance with:

- Waikato LASS Regional Infrastructure Technical Specifications (RITS)
- NZTA pedestrian planning and design guide
- Austroads guide to road design: part 3 geometric design & part 4B roundabouts
- NZTA RTS 18: NZ on-road tracking curves for heavy motor vehicles
- NZTA traffic control devices manual

5.1.1. Engineering Exception Decisions

- No engineering exceptions are proposed for the proposed Public Roads within this Development.

5.2. Integrated Transportation Assessment

An Integrated Transportation Assessment (ITA) has been carried out by Boffa Miskell and considers the traffic and transportation effects of the proposed Lifestyle Village Development. Please refer to the Boffa Miskell Integrated Transportation Assessment Rev.1 (Appendix B) for further details.

5.3. Access

Access to the site is initially will be via the existing farm access off State Highway 3 on Ōhaupō Road, approximately 350 metres north of the Green Hill Drive intersection. A new four-arm roundabout is proposed to the south of the existing farm entrance, with 3 arms of the new roundabout able to be constructed "off-line" from the existing SH 3 road corridor with minimal interruption to the existing SH traffic flows. Once the majority of the roundabout has been constructed off-line, then SH traffic flows can be diverted on to the new alignment and the remainder of the roundabout and amendments to the old SH 3 road alignment can occur. Access to the existing properties to the east of the proposed development will need to have access maintained at all times throughout the roundabout construction period. Once the roundabout is full constructed then site access will be via the new roundabouts western access leg.

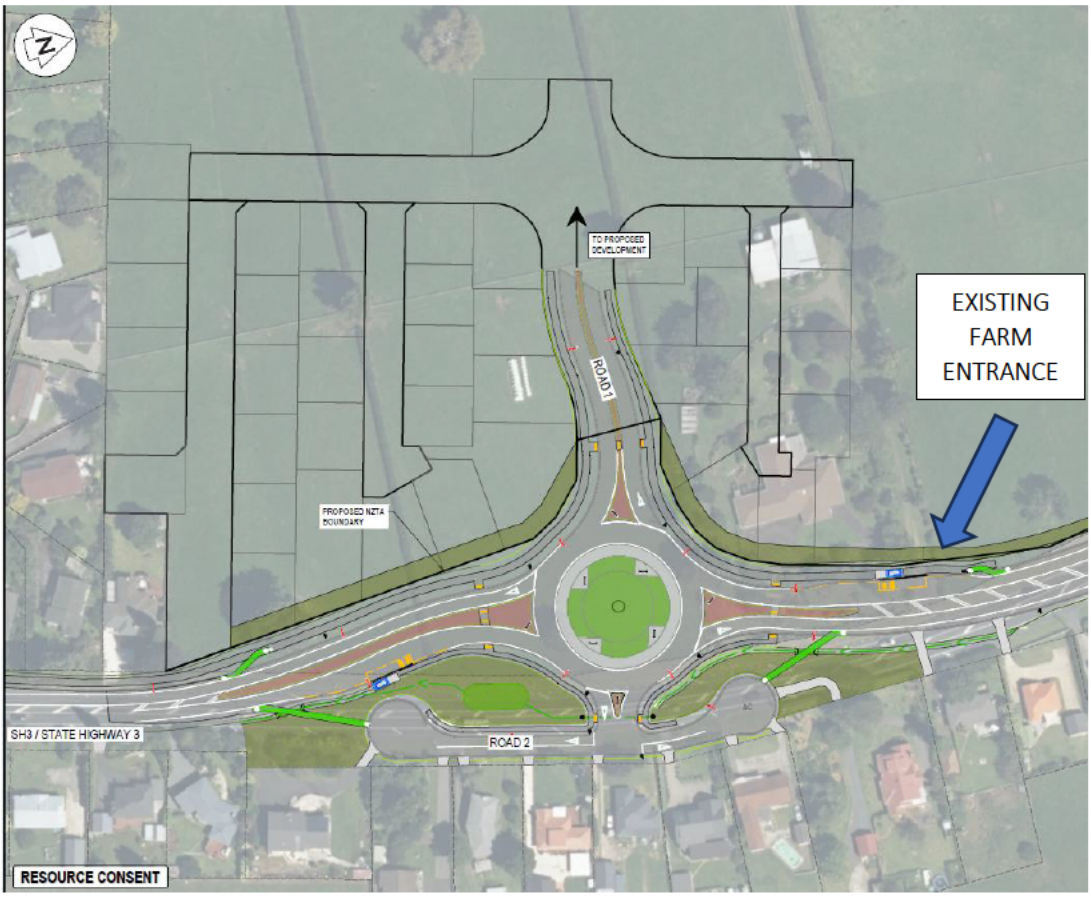


Figure 3: Proposed NZTA Four-arm Roundabout

5.4. Road Typology & Design

The road typologies are comprised of:

- Private Joint owned access lots (JOAL)
- Private Local volume roads
- Public Local volume roads
- Private Collector Road
- Public Collector Road

The public vested roads will be design to Regional Infrastructure Technical Specifications (RITS) and the Private roads and JOALs will be design to the Boffa Miskell Integrated Transportation Assessment. These are detailed in sections 5.4.1 to 5.4.6 below.

5.4.1. 20m Public Collector Road Typical Cross-section

The east-west orientated Collector Road (Public Collector Road 01) is proposed to have a 20.0m road corridor with the following features (refer to Figure 4 below):

- 8.6m carriageway width, comprised of
  - 3.3m traffic lanes, and
  - 2.0m flush median/central island
- 5.7m northern berm width, comprised of
  - 2.4m front berm
  - 1.8m footpath
  - 1.5m services berm

- 5.7m southern berm width, comprised of
    - 1.7m front berm
    - 2.5m shared path
    - 1.5m services berm

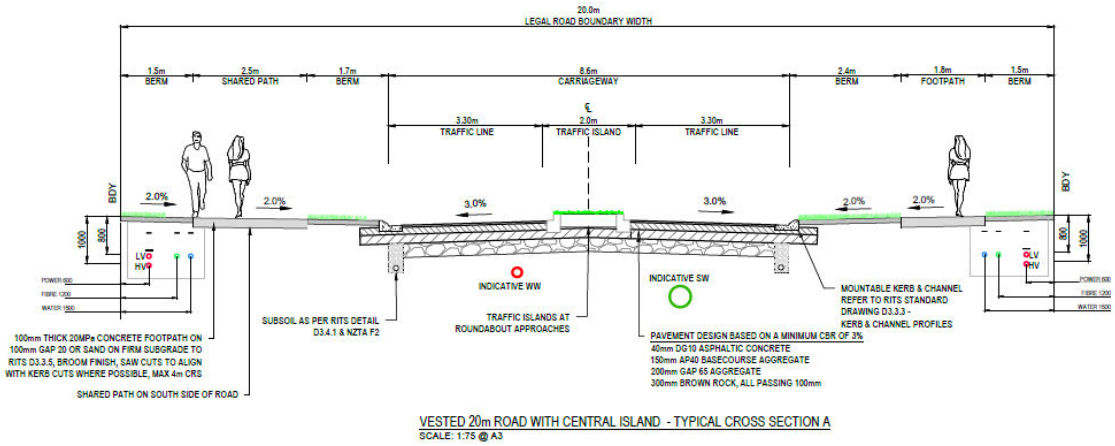


Figure 4: Public Collector Road Typical Cross-section

5.4.2. 18.0m Private Collector Road Typical Cross-section

The east-west orientated Collector Road (Private Collector Road 01) is proposed to have a 18.0m road corridor with the following features (refer to Figure 5 below):

- 6.5m carriageway width, comprised of
    - 3.25m traffic lanes, and
- 6.1m northern berm width, comprised of
    - 2.4m front berm/indented carpark bays
    - 2.5m shared footpath
    - 1.2m services berm
- 5.4m southern berm width, comprised of
    - 2.4m front berm/indented carpark bays
    - 1.8m footpath
    - 1.2m services berm

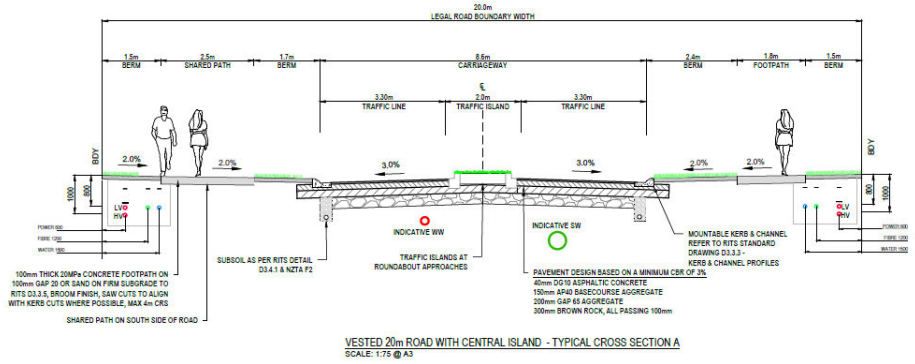


Figure 5: Private Collector Road Typical Cross-section

5.4.3. 15.0m Public Local Road Typical Cross-section

The public local road (Public Local Road 02/03) is proposed to have a 15.0m road corridor with the following features (refer to Figure 6 below):

- 6.0m carriageway width, comprised of
  - 3.0m traffic lanes
- 5.0m LHS berm width, comprised of
  - 2.2m front berm
  - 1.8m footpath
  - 1.0m services berm
- 4.0m RHS berm width, comprised of
  - 1.5m front berm
  - 1.5m shared path
  - 1.0m services berm

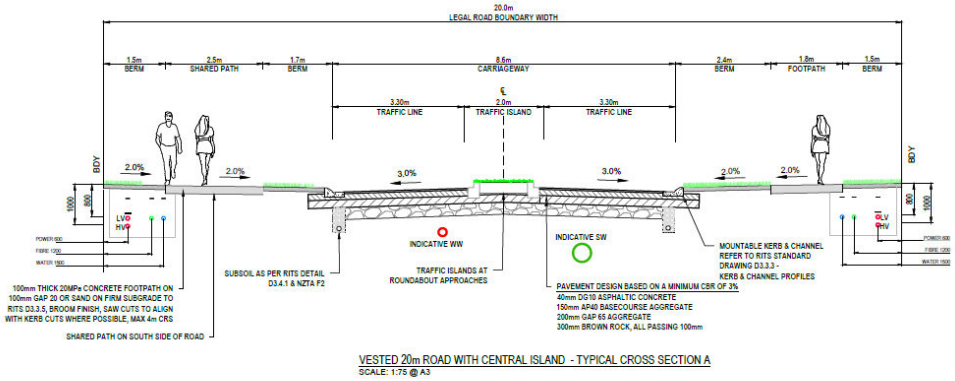


Figure 6: Public 15.0m Local Road Typical Cross-section

5.4.4. 12.5m Private Local Road Typical Cross-section

The private local road (Private Local Road 02/13) is proposed to have a 12.5m road corridor with the following features (refer to Figure 7 below):

- 6.0m carriageway width, comprised of
  - 3.0m traffic lanes
- 1.5m LHS berm width, comprised of
  - 1.5m services berm
- 5.0m RHS berm width, comprised of
  - 2.2m front berm
  - 1.8m shared path
  - 1.0m services berm

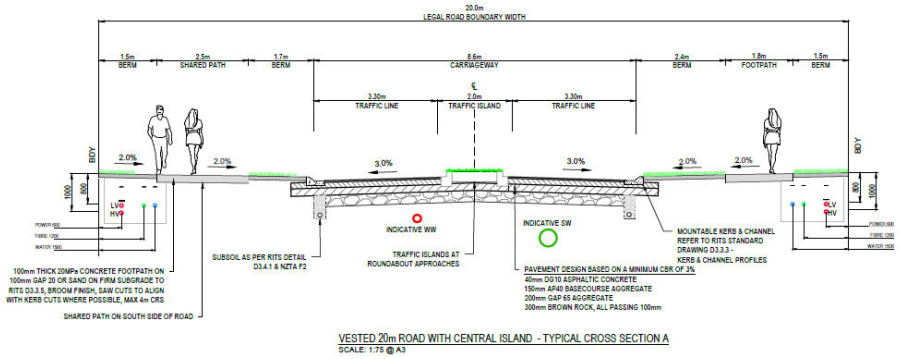


Figure 7: Private 12.5m Local Road Typical Cross-section

5.4.5. 10.0m Private Local Road Typical Cross-section

The private local road (Private Local Road 04-12, 13-17) is proposed to have a 10.0m road corridor with the following features (refer to Figure 8 below):

- 5.5m carriageway width, comprised of
  - 2.75m traffic lanes
- 1.0m LHS berm width, comprised of
  - 1.0m services berm
- 3.5m RHS berm width, comprised of
  - 1.0m front berm
  - 1.5m shared path
  - 1.0m services berm

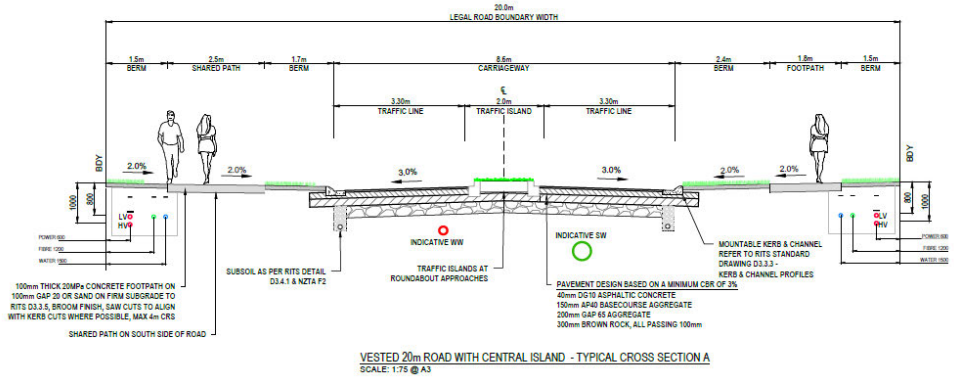


Figure 8: Private 10.0m Local Road Typical Cross-section

5.4.6. 8.5m Private JOAL Typical Cross-section

The private JOAL (Private Local Road 08) is proposed to have a 8.5m road corridor with the following features (refer to Figure 9 below):

- 5.5m carriageway width, comprised of
  - 2.75m traffic lanes
- 0.7m LHS berm width, comprised of
  - 0.7m services berm

- 2.3m RHS berm width, comprised of
    - 1.5m footpath
    - 0.8m services berm

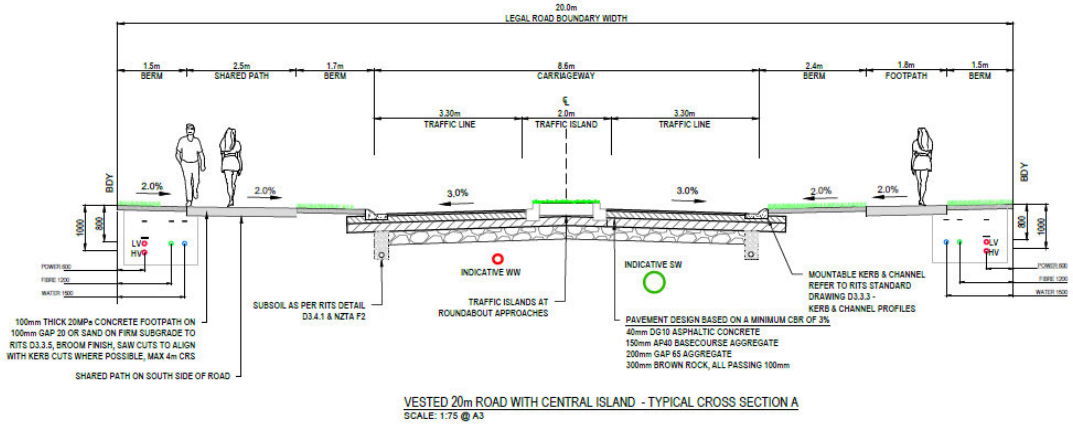


Figure 9: Private 8.5m JOAL Typical Cross-section

5.4.7. Collector/Local Road Pavement Design

The proposed council and private road designs are based on similar pavement designs undertaken in the Waipa district on similar geotechnical material within the last 10 years that have been proven to be effective and durable. Final pavement design will be undertaken at detailed design in accordance with WLASS RITS and Austroads Guide to Pavement Technology as part of engineering approval with Waipa District Council

The new public/private collector road is proposed to have the following pavement design:

- 50mm DG10 asphaltic concrete
- 425mm granular pavement of 175mm AP40 / 250mm GAP 65
- Subgrade improvement layer of 300mm grey/brown rock If required
- Design based on a subgrade strength of CBR 5.0 or greater.

The new public/private local roads and private JOALs will have the following pavement design:

- 30mm DG7 asphaltic concrete
- 350mm granular pavement of 150mm AP40 / 200mm GAP 65
- Subgrade improvement layer of 300mm grey/brown rock If required
- Design based on a subgrade strength of CBR 5.0 or greater.

5.4.8. Roundabout/SH 3 Pavement Design

The proposed SH3 and Roundabout Pavement Design are based on similar pavement designs undertaken for the Pokeno Heavy Traffic By-Pass Roundabout. Final pavement design will be undertaken at detailed design in accordance with NZTA and Austroads Guide to Pavement Technology as part of engineering approval with NZTA.

The new public/private collector road is proposed to have the following pavement design:

- 50mm AC14 asphaltic concrete
- 100mm AC20 structural asphaltic concrete basecourse
- 200mm AP40 granular sub-basecourse pavement

- 350mm GAP65 Subgrade improvement layer
- Design based on a subgrade strength of CBR 5.0 or greater.
    - 200-300 Granular additional Grey Rock granular working platform maybe required Insitu subgrade CBRs less than 5.0

5.5. HORIZONTAL GEOMETRICS

The horizontal curve geometrics for the proposed NZTA, publicly vested and private roads/JOALS are as follow are detailed below in in table 12. Road not listed below are straight alignments with no horizontal curves within their geometrics.

Road Name	Curve Number	Curve Type	Curve Radius Negative (-) LH Curve Positive RH Curve	Ownership
SH 3/ Ōhaupō Road	1	Compound Curve	Entry = -220m Exit = 140m	NZTA
	2	Compound Curve	Entry = 220m Exit = 240m	NZTA
	3	Circular Curve	-470m	NZTA
Old SH 3 / Road 02-SH 3	1		-80m	Vested WDC
Road 01	1	Compound Curve	Entry = -80m Exit = 80m	Vested WDC
	2	Circular Curve	-80m	Private
	3	Circular Curve	100m	Private
	4	Circular Curve	100m	Private
	5	Circular Curve	-30m	Private
	6	Circular Curve	80m	Private
	7	Compound Curve	Entry = 140m Exit = -180m	Private
	8	Circular Curve	-20m	Private
	9	Circular Curve	20m	Private
	10	Circular Curve	20m	Private
	11	Circular Curve	-20m	Private
Road 02	1	Compound Curve	Entry = 60m Exit = -60m	Vested WDC/Private Transition
	2	Circular Curve	-35m	Private
Road 09	1	Circular Curve	60m	Private
	2	Circular Curve	10m	Private
Road 10	1	Circular Curve	6m	Private
	2	Circular Curve	40m	Private
Road 11	1	Circular Curve	40m	Private
	2	Circular Curve	-10m	Private
	3	Circular Curve	-10m	Private
Road 13	1	Circular Curve	30m	Private
	2	Circular Curve	-30m	Private
	3	Compound Curve	Entry = -30m Exit = 40m	Private
	4	Circular Curve	-50m	Private
	5	Circular Curve	9m	Private

	6	Circular Curve	9m	Private
	7	Circular Curve	20m	Private
Road 15	1	Circular Curve	40m	Private
	2	Compound Curve	Entry = -31m Exit = 31m	Private
	3	Circular Curve	-6.5m	Private

Table 12: Horizontal Curve Geometrics

5.1. VERTICAL GEOMETRICS

The vertical geometrics for the proposed NZTA, publicly vested and private roads/JOALS are detailed in the below tables. Table 13 details the min/max gradients of each alignment and table 14 detailed the vertical sag/crest curves.

Road Name	Vertical Gradient %	
	Min	Max
SH 3/ Ōhaupō Road	0.5	6.3
Old SH 3 / Road 02-SH 3	0.5	4.5
Road 01 - Public	2.0	10
Road 01 - Private	0.5	8.5
Road 02 - Public	0.4	6.5
Road 02 - Private	0.4	5.0
Road 03 – Public	0.5	5.0
Road 04 – Private	3.0	6.5
Road 05– Private	2.0	6.5
Road 06– Private	0.5	4.5
Road 07– Private	1.0	3.0
Road 08– Private	0.4	5.0
Road 09– Private	3.0	7.5
Road 10– Private	0.4	4.5
Road 11– Private	3.0	5.0
Road 12– Private	0.5	7.5
Road 13– Private	0.5	6.5 (12.5)
Road 14– Private	3.0	5.0
Road 15– Private	2.7	12.5
Road 16– Private	1.5	3.0
Road 17– Private	2.0	3.0

Table 13: Vertical Geometric min/max gradients

Road Name	Minimum Curve K Values		DESIGN SPEED (km/hr)
	SAG	CREST	
SH 3/ Ōhaupō Road	30	35.5	80
Old SH 3 / Road 02-SH 3	3.3	4	40
Road 01 - Public	2.5	5.2	40
Road 01 - Private	2.0	5.2	40
Road 02 - Public	4.9	5.2	40
Road 02 - Private	6.7	15	40
Road 03 – Public	2.0	6.8	40
Road 04 – Private	1.3	5.7	40
Road 05– Private	2.0	4.0	40
Road 06– Private	2.5	5.2	40

Road 07– Private	3.0	6.8	40
Road 08– Private	1.9	8.6	40
Road 09– Private	1.0	3.5	40
Road 10– Private	2.0	4.0	40
Road 11– Private	2.1	3.0	40
Road 12– Private	2.2	2.9	40
Road 13– Private	1.0	2.6	40
Road 14– Private	2.0	7.5	40
Road 15– Private	1.6	3.1	40
Road 16– Private	NA	6.7	40
Road 17– Private	1.2	3.8	40

Table 14: Vertical Curve K-values

5.2. Parking

Dedicated indented car park bays will be provided on both sides of the Private Collector Road for a total of 38 indented car park bays. The location and size of the indented car park bays are detailed below:

- RHS /Northern Berm
  - 4 Car space indented car park bays = 2
  - 3 Car space indented car park bays = 3
  - 2 Car space indented car park bays = 3
  - 1 Car space indented car park bays = 2
  - Total number of car park spaces = 25
- LHS/ Southern Berm
  - 3 Car space indented car park bay = 1
  - 2 Car space indented car park bay = 2
  - 1 Car space indented car park bay = 6
  - Total number of car park spaces = 13

Private Local Road 10 will also have 11 indented 90° car park bays

Dedicated parking will be provided to the apartments, managed care facility and community recreational facility, as well on onsite parking for both the Terrance and free-standing houses,

For information regarding parking, refer to Appendix B – Integrated Transport Assessment

Appendix C – .

5.3. Walking and Cycling

The internal transport network has been designed to support a range of travel modes, including walking and cycling. Internal pedestrian and cycle routes will connect to a proposed shared path along Ōhaupō Road via Greenhill Drive and Taylors Avenue, linking to a new pedestrian crossing on Racecourse Road. This connection will provide access to existing footpaths on Racecourse Road and Ōhaupō Road SH3, as well as future shared path connections along Ōhaupō Road and Racecourse Road.

The proposal also includes two new bus stops on Ōhaupō Road adjacent to the development entrance, providing public transport connections to Te Awamutu, Hamilton and Cambridge.

Developed in consultation with NZTA and WDC, the proposal includes a new 2.5 m wide shared path along Ōhaupō Road between the development SH3 entrance and Greenhill drive. The existing footpath on Taylors Avenue will also be upgraded and widened from 1.5 m to 2.2 m, providing improved walking and cycling access for both future residents and the wider Te Awamutu community. The shared path will enhance connections to Te Awamutu and provide access to the proposed bus stops at the Ōhaupō Road roundabout.

For further information regarding walking and cycling, refer to Appendix B – Integrated Transport Assessment

Appendix C – .

5.4. Lighting

For information regarding lighting, refer to Appendix B – Integrated Transport Assessment

Appendix C – .

6. Stormwater

The Harlow LRV Development proposal has been developed in consultation with Waikato Regional Council, and other/external technical consultants. The Harlow LRV Development consists of 4 main stages, that informs the post development catchment.

This section of the report provides further details of the Harlow LRV Development, its stormwater strategies and options to meet regulatory requirements, Waikato Regional Council guidelines and Regional Infrastructure Technical Standards (RITS) as specified throughout the report and the attached Stormwater Management Plan.

Spanning approximately 25.78ha the development includes approximately 414 units across the site, designed with mix of housing typologies, duplexes, apartments, manged care centre and communal recreation facilities.

The key feature of the development is integrated stormwater treatment utilising centralised raingardens, wetlands, forebays, high flow bypass designed to convey and attenuate overflows from the road and lot areas into the existing artificial farm drain and pond network, with the overall site releasing at 80% of pre-development flows into the existing artificial farm drain to the west of the site via a series of controlled outlet structures (culverts) and existing flood attenuation basins/areas located within the proposed development site.

6.1. Design Standards

The Waipa District Council and Waikato Regional Council sets out design and construction standards for stormwater and requires all land development projects to be provided with a means of stormwater disposal.

Integrated stormwater treatment has been designed in accordance with Design guidelines and standards in the list below;

- Waipa District Council Stormwater Guidelines and standards
- Waikato Regional Council Guidelines and standards
- RITS guidelines and specifications
- National environmental standards for freshwater

6.1.1. Engineering Exception Decisions

- The proposed Primary piped stormwater network will be sized for up to the 10-year ARI climate change RCP 6 event and have been sensitivity checked for RCP 8.5.
    - Reasoning is the site contours fall to a central existing drainage network with existing flood attenuation basis, that ensure that the proposed development will not be inundated in events greater than the pipe networks capacity to accommodate, and secondary overland flow direct all flows away from habitable areas/platforms.
    - Sensitivity testing to RCP 8.5 showed surcharging out of the Double Catch Pit (DCP) Line A3-2 at the lower end of Private ROW 08. Overflows from this DCP flow directly to the existing artificial farm pond to the west, with surrounding habitable platforms a minimum of 500mm above the DCP overflow level.

6.2. Stormwater Management Plan (SMP)

Stormwater Management Plan (SMP), has been prepared to support a stormwater discharge consent application for 4 stages of the Harlow LRV Development, as outlined below in the table 15 below:

Stage	Wetland	Catchment	Area (Ha)
1	Wetland A	A	6.40
2	Wetland B	B	5.42
2	Wetland C	C	2.72
3	Wetland D	D	1.31
4	Wetland E	E	1.86
NZTA	Stormwater Basin	F	0.80
NZTA	Unmitigated	G	9.97

Table 15: SW Wetland Catchments

Each site has been assessed through hydrological and hydraulic modelling, incorporating the adopted 4°C climate-change scenario. The relevant design parameters, climate-change assumptions and modelling methodology are detailed in Section 6.1 of the SMP.

The proposed stormwater management system has been designed to provide the following:

- Water Quality Volume treatment and retention of the 1.3mm of new road seal runoff to maintain water naturality.
- Conveyance of flows up to the 100YR ARI storm events through proposed networks and overland flows within the road corridors. These will be directed to the downstream wetlands, for each of the stormwater catchments. Limit post-development peak discharges to no more than 80% of pre- development flows to the Mangapiko Stream.
- Preserve existing flow paths and discharge locations to avoid introducing new hydrological impacts.
- Account for external inflows and upstream catchments to ensure downstream impacts are not worsened under post-development conditions.
- Support future maintenance, accessibility, and ecological enhancement through features such as maintenance tracks and planting around the wetland areas.

A flood model for the 2, 10 and 100-year ARI events has been developed to assess existing and post-development conditions upstream and downstream of the Harlow LRV Development. The modelling has informed the stormwater design, including artificial wetland storage and attenuation, controlled discharge, and defined overland flow paths to manage peak flows and flood levels.

The 100-year ARI modelling demonstrates that the proposed stormwater measures effectively mitigate flood impacts. The existing drainage corridor provides distributed flood storage and attenuation, with the existing low-lying area forming the primary attenuation basin within the proposed Harlow LRV Development and the downstream culverts controlling discharge, resulting in reduced peak flows to the downstream receiving environment compared with the pre-development scenario.

The wider downstream catchment assessment identified potential increases in downstream flooding under development scenarios without mitigation. The proposed stormwater management strategy addresses these effects through wetland storage, attenuation and controlled discharge, with the assessment confirming no adverse downstream flooding effects from the Harlow LRV Development.

In addition, the model has been used to test a sensitivity scenario in which all primary stormwater devices are blocked during a 100-year event. Even under this worst-case condition the model

demonstrates that the development maintains flood immunity with only minor exceedances and expected to be mitigated through detailed design refinements. These areas will be addressed during detailed design by refining finished ground and road levels, improving secondary overland flow paths, and optimising the stormwater drainage layout to safely convey exceedance flows while maintaining flood immunity for building platforms.

The SMP aligns with the requirements of both the Waipa District Council and Waikato Regional Council.

6.3. Stormwater System

The Harlow LRV Development will be serviced by the following stormwater infrastructures.

1. Primary Network – Pipe System
2. Centralise Raingardens
3. Artificial Stormwater Wetlands
4. Enhancement of the existing onsite flood basin/areas
5. Culverts controlling discharge to receiving environment

The proposed stormwater management system for the Harlow LRV Development incorporates an integrated approach comprising a piped drainage network, and surface conveyance and treatment systems. The piped network has been designed to convey runoff from frequent storm events, while larger exceedance events are managed via defined overland flow paths to maintain flood resilience. Stormwater quality treatment and initial abstraction are achieved through the use of treatment devices, including centralised raingardens and artificial stormwater wetlands, which provide sediment removal, pollutant capture, and flow attenuation prior to discharge.

The downstream proposed artificial wetlands will function to treat road runoffs, extended detention and provide full detention for events up to the 2-year CC ARI, with the enhanced existing onsite flood basin/areas providing attenuation for the events up to the 2-year CC ARI

Please refer to the Appended Engineering Plans (Appendix A) for further details.

6.3.1. Existing Reticulation

Existing stormwater infrastructure is limited to artificial farm drains and ponds that form part of the central drainage corridor for the existing farm activities as well as multiple existing natural wetlands. Multiple culverts along the artificial farm drain were encountered/surveyed and have been incorporated in the stormwater design and modelling. There is an existing timber/earth farm/foot bridge located mid site (stage 2) but is in poor condition and likely removed at time of development.

6.3.2. Lot Connections

All lots are serviced with lot connections as per RITS guidelines and specifications.

6.4. Overland Flow Paths (OLFPs)

Additional branches of OLFPs will be created as roading corridors are formed. The following measures will be adopted to mitigate their effects of these overland flowpaths on the proposed development:

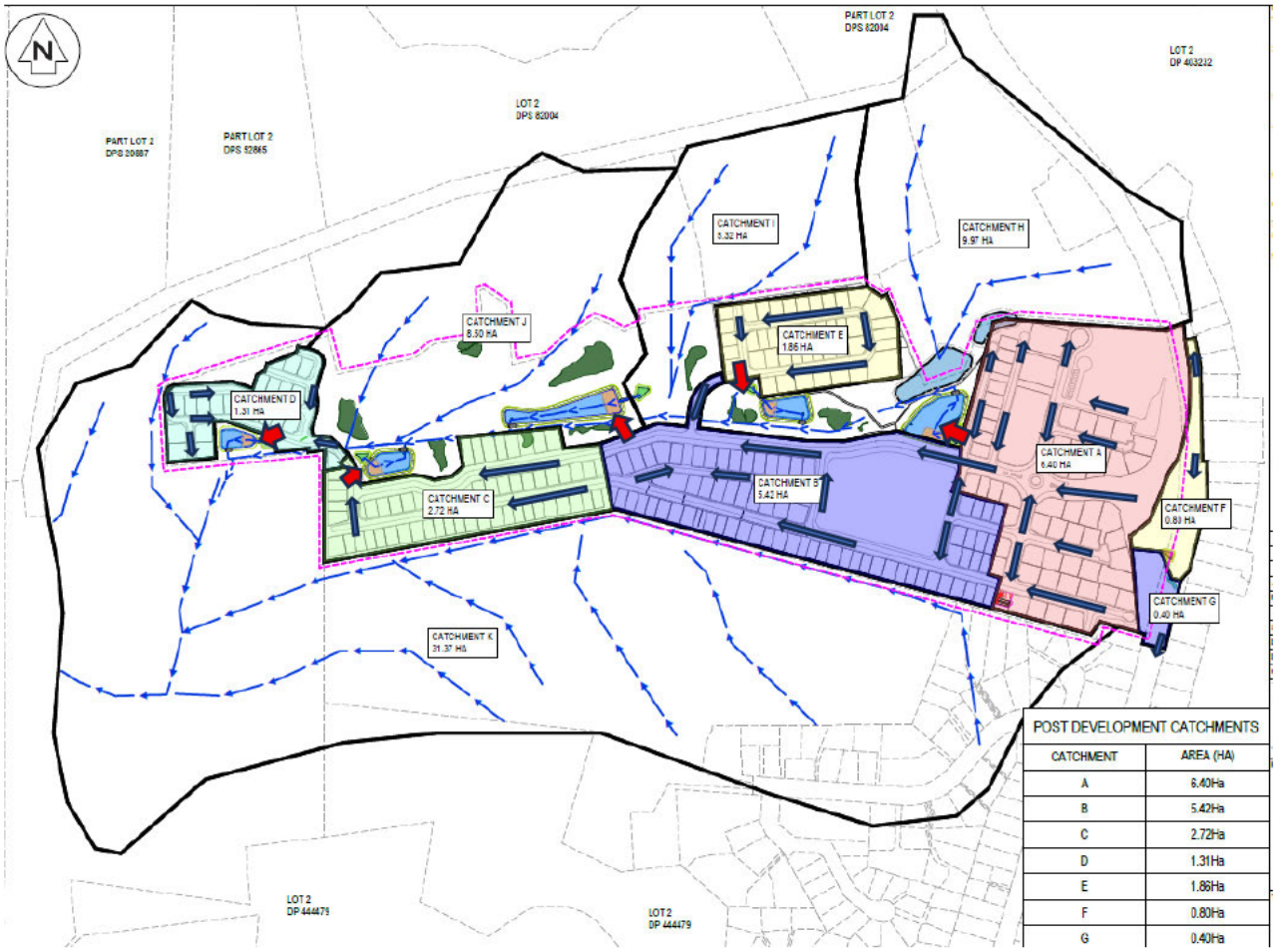


Figure 10: Proposed Stormwater Catchment Plan

- Identify and maintain existing natural overland flow paths and watercourse alignments to convey concentrated stormwater runoff from the site. Existing culverts will be retained and utilised where practicable to maintain existing discharge locations; however, additional culverts are proposed where required to provide hydraulic connectivity and accommodate the proposed development layout.
  The culverts will be designed to satisfy the required hydraulic and structural performance criteria, with fish passage provisions and any associated stream works designed.
- Identify and retain any upstream OLFPs and/or watercourses to avoid any upstream flooding.
- Ensure OLFPs are to be designed where possible within the roading network and discharge into watercourses.

The preliminary OLFP design is provided in the Stormwater Management Plan (SMP) and is summarised below.

An assessment of the post development overland flow paths (OLFPs) has been carried out to evaluate the behaviour of surface runoff in the road carriageway under the proposed stormwater management system. The design scenario is based on the 2081-2100 RCP8.5 climate change scenario, incorporating all proposed devices and the assessment is done through HEC-RAS 2D. The OLFPs represents the conveyance of surface runoff because of the proposed system during the 100-year storm event.

Flow depths and velocities were assessed at key locations throughout the development covering all the various road/accessway typologies ensuring and confirming conveyance of the OLFP is viable through proposed carriageway. See below table 16 showing results at the key locations.

CATCHMENT	SECTION	FLOW RATE FROM HEC-RAS MODEL (m^3/s)	MAX DEPTH (m)	VELOCITY (m/s)	DEPTH x VELOCITY (m^2/s)
A1	1	0.39	0.11	1.96	0.22
A1	2	0.71	0.10	2.30	0.23
B1	3	0.66	0.14	2.12	0.29
B1	4	0.96	0.17	1.00	0.17
C1	5	1.2	0.18	1.46	0.26
D1	6	0.2	0.09	1.06	0.09
E1	7	1.06	0.11	2.26	0.24

Table 16: OLFP section details

All modelled sections except Section 4 and Section 5 are within the 150mm design guideline. Depth x velocity values remain well below the critical safety thresholds defined in Austroads 2012 Part 5, which specify:

- < 0.4 m^2/s for pedestrian safety
- < 0.6 m^2/s for vehicle safety

The highest recorded value, 0.29 m^2/s is at Catchment B1 (Section 3), but confirms safe flow conveyance for both pedestrians and vehicles under design conditions. Flow is primarily routed along proposed roads and conveyed into roadside primary networks for the 10-year cc conveyance, with surcharge above this directed to the designated wetlands and/or existing channel via the OLFP.

6.4.1. Departures From Standard

A departure from standard stormwater design has been identified in Overland Flow Path (OLFP) sections 5 and 4 where the flow depth exceeds the recommended threshold of 150 mm according to RITS V2 2025 Clause 3.3.14.10. These two locations are proposed as departures and these exceedances are minor and occur in isolated sections of the residential OLFP network.

Sections exceeding the 150mm limit are:

- Catchment B1 – Section 4: Max Depth = 0.17m
- Catchment C1 – Section 5: Max Depth = 0.18m

These areas remain safely contained within their respective flow corridors and do not pose a risk to vehicles and pedestrians as Depth x Velocity (D x V) values remain well below the threshold. Further refinement of the earthworks model in conjunction with the Architectural and Landscape design will allow these maximum depths to be addressed and resolved at detailed design.

6.5. Capacity and Quality

Stormwater Strategy for Lot Areas

Roof runoff is managed using inert roofing materials, while driveway runoff is directed through a catch pit with a sump for pre-treatment before disposal to the stormwater piped infrastructure. Overflow is located in the catchpit system for flows surpassing the designed event within the lot areas. Excess flows will be diverted into the downstream system via the road carriageway.

Stormwater Strategy for Road Carriageway

The Water Quality volume (WQV) will be provided through the proposed wetland system with the initial abstraction is to be provided within centralised Raingardens.

Stormwater Strategy for proposed SW Wetlands

The proposed wetlands form critical part of the overall stormwater mitigation system. They have been sized to accommodate the 2-year event, excess flows from both the road carriageway and on lot flows exceeding the 10-year event will be directed to the high flow bypass system. Additionally, these are also designed to provide treatment and attenuation of lower flow events. Please refer to SMP Section 8.4 Artificial Wetlands and CMP Section 5 Construction Methodology and Sequence of Works for more information.

6.6. Flooding

A 2D flood model has been developed for the site using HEC-RAS. The model has been used to assess existing, design and sensitivity scenarios.

The figure below shows the contributing catchment of the site during the 100year event, delineated through QGIS/HEC HMS based on a 10m grid externally. Data used is comprised of localised site survey and LINZ data capturing the wider extent of the catchment.

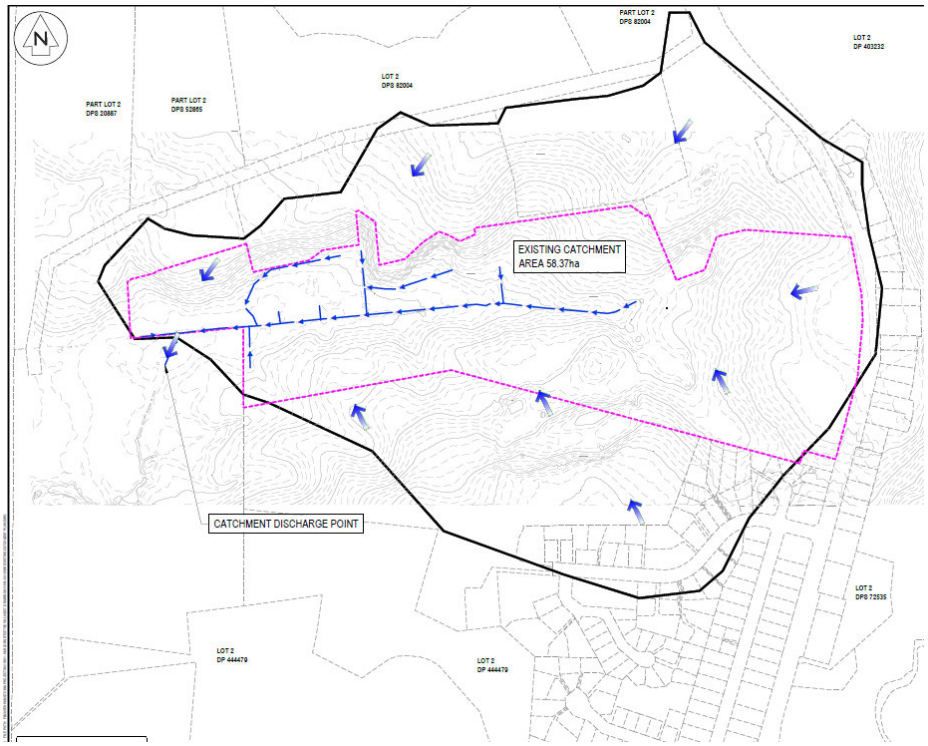


Figure 11: Existing 100YR Catchment

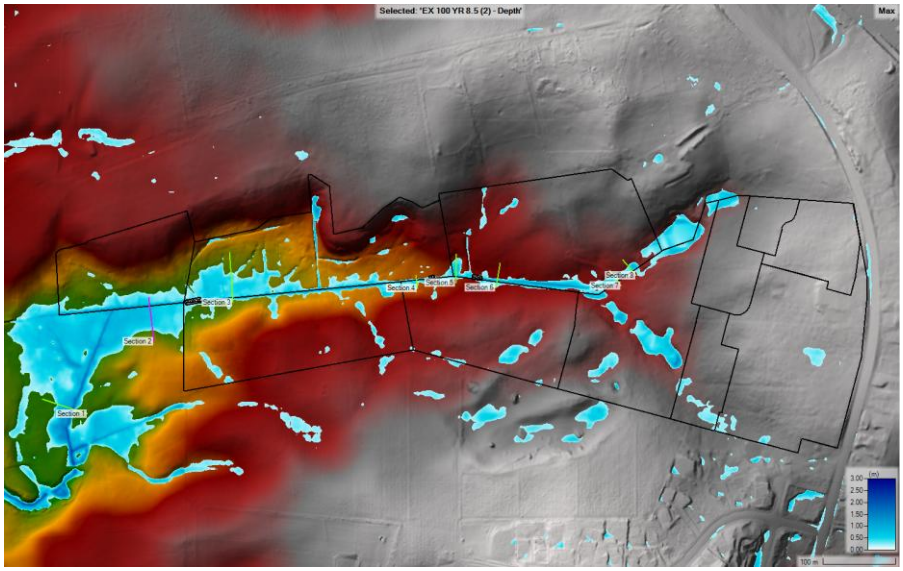


Figure 12: Pre-Development 100-year ARI C.C. Flood Map

The existing 100-year catchment (Figure 11) and existing flood extent in the 100-year event (Figure 12) demonstrates that the downstream low-lying areas just downstream from the property is flooded and act as a basin, storing significant flood volume during major storm events. Runoff from the wider catchment, including upstream residential and rural areas, is conveyed through the development area via the existing farm drains and streams before discharging into the Mangapiko Stream. As the site is influenced by significant upstream catchments, existing drainage channels, and flood-prone areas (as noted above), the proposed design seeks to maintain and improve existing hydraulic connectivity through the retention of primary drainage corridors, implementation of diversion channels, construction of stream culvert crossings, and provision of wetlands and stormwater storage areas throughout the development. The Sensitivity assessment assumed that all stormwater treatment and attenuation devices have already reached their capacity. Even under this worst-case condition, the model demonstrates that the development maintains flood immunity, with only minor exceedances expected to be mitigated through detailed design refinements as mentioned above in Section 6.4.1 Overland Flow Paths (OLFPs).

Pre- and post- development results are tabulated below, to indicate the improvement in flood behaviour over the wider catchment area.

Sections	Pre Development		Post Development	
	Q (m^3/s)	Water Surface Elevation (m)	Q (m^3/s)	Water Surface Elevation (m)
1	11.14	46.6	7.29	46.53
2	12.05	46.64	7.06	46.56
3	11.66	47.42	9.35	47.74
4	7.72	51.31	2.48	51.29
5	8.33	53.36	4.5	53.74
6	6.9	54.86	2.36	54.99
7	2.64	59.35	2.05	59.71
8	2.57	61.09	2.3	60.69

Table 17: Pre vs Post Development WSE & Flow Rate Values

6.7. Connection Points and Staging

The proposed development will be delivered in four stages, with each stage incorporating the stormwater infrastructure required to service its respective catchment. Stormwater management facilities, including centralised raingardens, artificial stormwater wetlands and piped drainage networks, will be commissioned progressively to ensure that each stage operates independently and complies with the required quantity and quality performance criteria prior to the commencement of subsequent stages.

The staging and corresponding stormwater connections are summarised below.

- Stage 1 – This stage corresponds to catchment A;
    - Which is serviced by artificial stormwater wetland A.
    - Stormwater discharges up to the 2-year ARI CC event into the adjacent existing artificial farm ponds.
    - Installation of Culvert 4 and the associated proposed Footpath embankment to the west of these existing ponds will ensure that the catchment will discharge at up to the 100-year ARI CC event at 80% predevelopment flows into existing artificial farm drain.
- Stage 2 – This stage corresponds to catchment B & C;
    - Which is serviced by artificial stormwater wetlands B & C.
    - Stormwater discharges up to the 2-year ARI CC event into the adjacent existing artificial farm drain for both wetlands
    - Installation of Culvert 1, 2 & 3 and the associated proposed road/earthworks embankments will ensure that the catchment will discharge at up to the 100-year ARI CC event at 80% predevelopment flows into existing artificial farm drain.
- Stage 3 – This stage corresponds to catchment E;
    - Which is serviced by artificial stormwater wetland E.
    - Stormwater discharges up to the 2-year ARI CC event into the adjacent existing artificial farm drain.
    - Installation of Culvert 1, 2 & 3 within stage 2 will ensure that the catchment will discharge at up to the 100-year ARI CC event at 80% predevelopment flows into existing artificial farm drain.
- Stage 4 – This stage corresponds to catchment D;
    - Which is serviced by artificial stormwater wetland D.
    - Stormwater discharges up to the 100-year ARI CC event into the adjacent existing artificial farm drain.

7. Wastewater

7.1. Wastewater Network Capacity

The client, in association with Waipa District Council, have previously undertaken a Wastewater Servicing Assessment by BTW Company Ltd. (*BTW, 2022*). A desktop assessment was undertaken to identify the preferred wastewater servicing solution for the proposed development. This included a review of the capacity of the existing Council wastewater network (as previously modelled by WSP), and modelling of future network performance, including projected growth to 2050. The full assessment is provided in Error! Reference source not found..

7.2. Servicing Options

The assessment identified two potential servicing options:

7.2.1. Option 1 - Upgrade Existing Racecourse Road WWPS Catchment

All wastewater flows generated by the development would discharge to the existing gravity network that drains to the Racecourse Road Wastewater Pump Station (WWPS). To accommodate the additional flows, upgrades would be required to sections of the downstream network, including approximately 1 km of existing 225 mm diameter gravity wastewater pipe, which would be upsized to 300 mm diameter.

7.2.2. Option 2 - Split Catchment Discharge

Approximately 80% of the development wastewater flows would be collected by a new on-site WWPS and discharged via a rising main to the Christie Avenue WWPS catchment. The remaining 20% of flows would continue to discharge by gravity to the Racecourse Road WWPS catchment. This option would require construction of a new rising main extending approximately 640 m along Ōhaupō Road, connecting to the existing wastewater network near the intersection of Ōhaupō Road and Racecourse Road.

Both options were assessed to determine the most efficient and sustainable long-term servicing strategy for the proposed development. It is noted that at time of the initial Fast Track application, Waipa District Council used this report and the above options to form the basis of their feedback on the submission.

7.3. Proposed Network

Following initial discussions with Waipa District Council and Waikato Waters, Option 2 - Split Catchment Discharge was selected as the preferred wastewater servicing approach and progressed for further design. Under this arrangement, the development catchment is divided into two wastewater servicing areas:

1. Upper wastewater catchment (approximately 20%) of the development will be conveyed to a publicly vested Wastewater Pumpstation (WWPS) and discharged to the Christie Avenue WWPS (refer to Figure 5 & 6 below)
2. Lower wastewater catchment (approximately 80%) of the development will be conveyed to a private WWPS, which will discharge to the Racecourse Road WWPS (refer to Figure 5 & 7 below)

This servicing strategy distributes wastewater flows between the two receiving catchments, reducing the demand on the existing network while providing a practical and feasible solution for the proposed development. Based on the above solution it is agreed that no additional upgrades are required to the existing Waipa District Council Network.



Figure 13: Proposed Wastewater Total Catchment Plan

7.4. Pump Stations

Calculations for the site demand & pump station's requirements are as follows:

Public Stage 1 Pump Station (Catchment A)		
PWWF=	2.47	l/s
ADWF =	37.20	m3/day
Storage Required =	13.95	m3
Private Pump Station (Catchment B)		
PWWF=	11.09	l/s
ADWF =	143.00	m3/day
Storage Required =	53.63	m3

Table 18: Post-Development Pump Station Calculations

7.5. Proposed Wastewater Reticulation

The proposed development will be serviced by two wastewater catchments, each with its own dedicated pump station and ownership arrangement. Catchment A will comprise publicly vested wastewater infrastructure, including wastewater pump station A (WWPS-A), with all assets designed and constructed in accordance with RITS requirements.

Catchment B will comprise privately owned wastewater infrastructure, including wastewater pump station B (WWPS-B), with the network designed in accordance with RITS principles and applicable Building Consent requirements.

Internal wastewater reticulation throughout the development will consist of a combination of new 150 mm diameter uPVC SN16 gravity sewer mains, conveying flows to the two pump stations located at the low points of the site. The pump stations are proposed as prefabricated package systems to be designed and supplied by a specialist pump station provider. Preliminary locations and discharge arrangements are shown on C-500 in Appendix A – Engineering Plans. Final pump station design to be confirmed during detailed design and Building Consent stages.

In accordance with RITS V2 (*2025*) requirements, both pump stations will incorporate emergency storage capable of containing 9 hours of Peak Wet Weather Flow (PWWF), providing resilience during power outages, mechanical failures, or maintenance events. Power and water supplies to the pump stations will be provided from the reticulated network, and vehicle access will be constructed to facilitate operation and maintenance activities. For publicly vested infrastructure, all-weather access and the necessary easements in favour of Waipa District Council will be established where required. Further details of the pump station design are provided in **Error! Reference source not found.**.

7.6. Staging

The proposed development will have 4 major infrastructure stages with substages include within each major stage (refer to figure Z below).

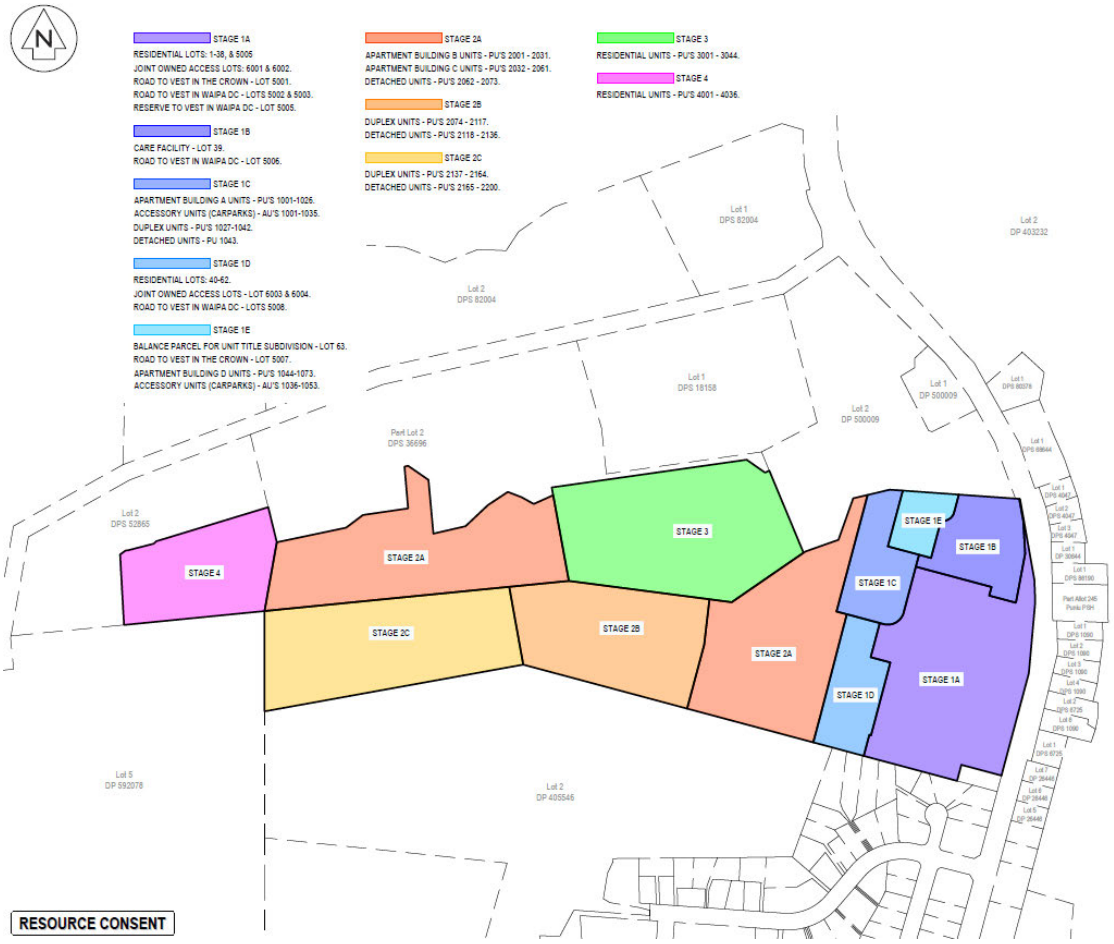


Figure 14: Proposed Infrastructure Staging

7.6.1. Pump Station A – Stage 1

Pump Station A will serve Stage 1 substages 1A & 1B which equates to approximately 20% of the proposed development flows as per the connection option agreement with Waipa District Council and Waikato Waters. Pump Station A will be constructed within a separate wastewater reserve (Lot 5005).

The outfall option for Pump Station A will be via new 110mm Ø ODPE 100 SDR 13.6 PN12.5 rising main to the Greenhill Drive wastewater manhole (Manhole Asset ID 1090025)

The proposed rising main alignment will exit the proposed wastewater reserve (Lot 5005) and have the following alignment:

- Transverse east along proposed local public road 02 within the southern berm
- Transvers south along the SH3/ Ōhaupō Road alignment within the eastern berm (beneath the new proposed 2.2m footpath)
- From Ōhaupō Road /Greenhill Drive intersection transverse east along Greenhill Drive within the northern berm (beneath the new proposed 2.2m footpath)
- From the Taylors Ave/Racecourse Road intersection transverse east along Racecourse Road within the northern berm (beneath the proposed 2.2m footpath upgrade works)
- Connect via open trench to the Greenhill Drive existing wastewater manhole (Manhole Asset ID 1090025)

The rising main is proposed to be installed beneath the proposed 2.2m footpath connecting the proposed development to Te Awamutu township.

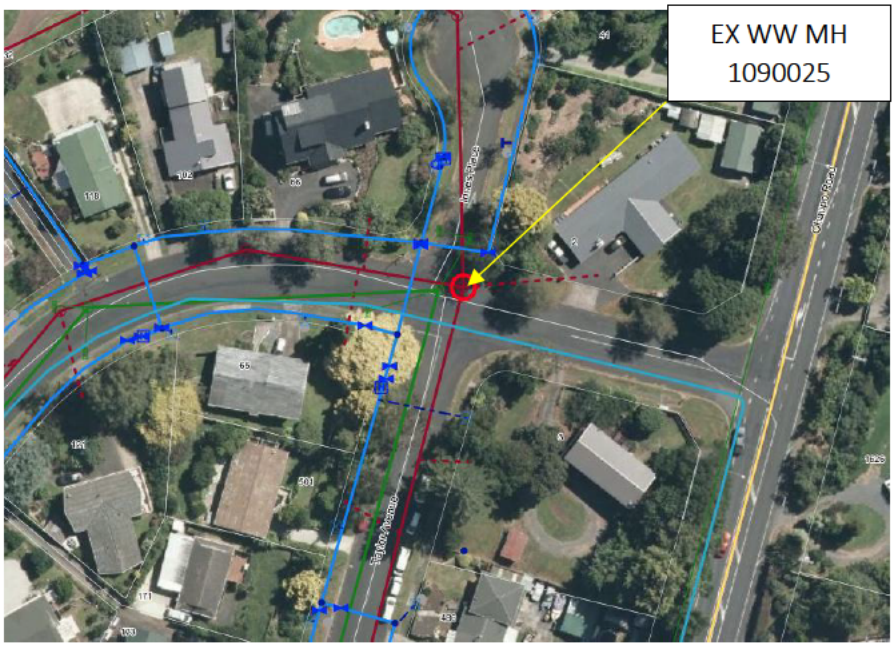


Figure 15: Wastewater Manhole ID 1090025

7.6.2. Pump Station B – Stage 1C,1D, 1E, 2, 3 & 4

Pump Station B will serve Stage 2, 3 & 4 but also Stage 1 substages 1C, 1D & 1E which equates to approximately 80% of the proposed development flows as per the connection option agreement with

Waipa District Council and Waikato Waters. Pump Station B will be constructed within a common property area (Stage 4).

As the Pump Station services Stage 1 substages them it is recommended that Stage 2 earthworks stage (refer to section 4.0) occurs prior to substages 1C,1D & 1E progresses. This will allow for the wastewater main within the proposed private collector road alignment (refer to section 5.0) and Pump Station B to be constructed. The wastewater main connecting Stage 1 to Pumpstation B has been designed to be below the stormwater infrastructure – and as such can be installed prior to Stage 2 & 4 infrastructure being constructed.

The outfall option for Pump Station B will be via new 180mm Ø ODPE 100 SDR 13.6 PN12.5 rising main to the Greenhill Drive wastewater manhole (Manhole Asset ID 1090999)

The proposed rising main alignment will exit the proposed development at the end of Local Road 02 within the same services trench as the Pump Station A rising main, with the following additions:

- From Greenhill Drive/Taylors Ave intersection transverse south along Taylors Ave within the eastern berm (beneath the proposed 2.2m footpath upgrade works)
- From the Taylors Ave/Racecourse Road intersection transverse east along Racecourse Road within the northern berm (beneath the proposed 2.2m footpath upgrade works)
- Be thrust under SH3 to the north of SH3/Racecourse Road intersection and connection existing wastewater manhole (Manhole Asset ID 1090999)

Due to the timing of Pumpstation B, the new 180mm Ø ODPE 100 SDR 13.6 PN12.5 rising main from Pump Station A to Greenhill Drive will be laid (and capped) at the same time as the Pump Station A 110mm Ø ODPE 100 SDR 13.6 PN12.5 rising main (within the same services trench).

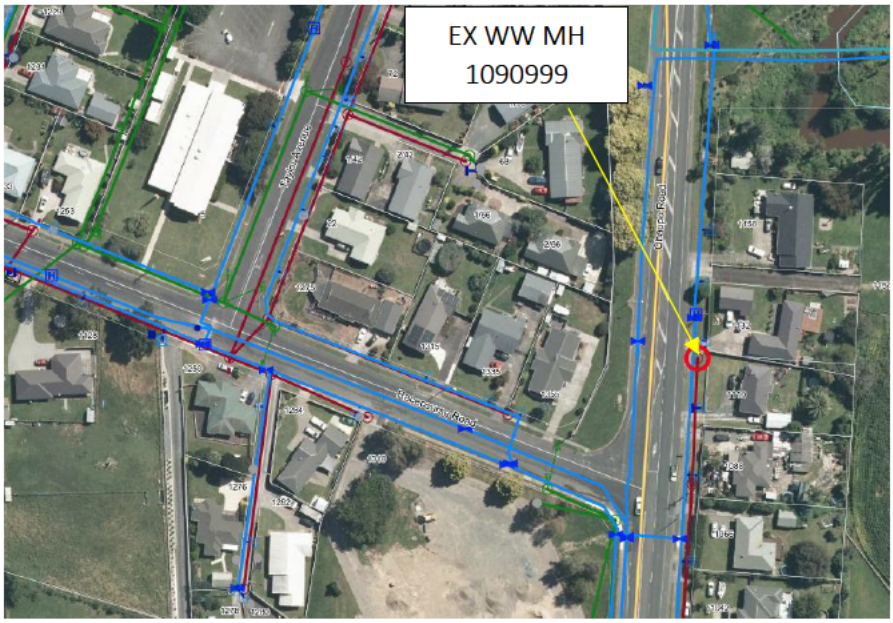


Figure 16: Wastewater Manhole ID 109099

7.7. Lot Connections

All lots/units are serviced with lot connections as per RITS guidelines and specifications. The Apartments, common amenities/facilities and managed care facility will be provided with 150mmØ connections to the wastewater main.

7.8. Wastewater Pump Station

7.8.1. RITS Wastewater Pump Station Requirements

This section of the report relates to the proposed Wastewater Pump Stations within Stage 1 and 4. The specification and design of the new wastewater pump station has been developed utilising the RITS standards. The proposed pump station will comprise of a collection manhole, inlet pipes, a wet well, control cabinet, odour biofilter and emergency storage tanks. Detailed descriptions of the pump station and compliance check in accordance with RITS is presented in Table 19 below.

Requirements	Requirements check
a) Area around the pump station shall be graded to prevent surface water flowing onto or over the pump station cover slabs.	The pump station areas have been elevated above the nearby roads, and they will grade back to the road areas.
b) Free of secondary flow paths for 1% AEP flood level, and the pump station lid levels shall be provided with a minimum freeboard of 300mm above the estimated flood level.	As the pump station will be sitting at a higher elevation than the nearby existing stream, this will ensure that the minimum freeboard will be achieved. The pump station lid level will be at least 300mm above the overland flow path on the road and 300mm above the 100yr flood level in the existing stream.
c) Pumping systems shall • Have a pumping capacity of N+1 with a minimum of two identical pedestal mounted submersible sewage pumps. • Each pump shall be capable of discharging the design peak wet weather flow rate from the catchment. • Include sufficient well volume to operate under normal conditions without surcharge to the incoming wastewater network.	The pumpstation specimen design is compliant with these conditions referring to Civil drawing (C530) for further details. (Appendix A – Engineering Plans)
d) The station shall be located to ensure that the entire design catchment can be serviced.	The eastern wastewater pumpstation (WWPS A) will manage substages 1A & 1B (approximately 20% of the total development flows). The western wastewater pumpstation (WWPS B) will manage pumped flows in Stage 1C, 1D, 1E, 2 and 4 (approximately 80% of the total development flows). The pump stations have been strategically placed in the catchment low points.
e) All stations shall be contained within a separate local purpose reserve - drainage title set out to provide safe and easy operation and maintenance of the site without impacting on public activities surrounding the site and is designated in accordance with the relevant district plan or vested as part of a subdivision.	A designated local purpose reserve for each pump station that is to be vest (WWPS A) has been proposed to allow for the operation and maintenance of the pump station. For the private pump station (WWPS B) this will be managed by the Harlow LRV body cooperate and located within the common area within Stage 4.
f) The station shall be designed to service the entire catchment area of land beyond the reach of the existing gravity system. Refer to Clause 5.1.6.	The wastewater pump stations have been designed to account for the whole Residential Development proposed in the project.

g)	In staged construction, guidance from the Council is required to ensure the correct sizing of the pump station and associated rising mains and storage facilities meet the short and long-term requirements of the catchment.	The pumpstations have been designed to cater for the short term and long-term requirements. There is allocated space to expand the underground wastewater storage if required to do so in the future.
h)	A minimum emergency storage capacity of nine hours average dry weather flow, measured between the high-level alarm and the point of overflow.	The pump station has been designed to account for a minimum emergency storage capacity of 9 hours.

Table 19: WWPS RITS Requirements

Please refer to C530-1 and C530-2 Engineering Plans (Appendix A) for the pump station specimen design. Full pump station designs will be undertaken at detailed design to confirm the specimen design.

7.8.2. Emergency Storage

The proposed pump stations are provided with storage capacity exceeding the RITS minimum of nine hours average dry weather flow (ADF). The enhanced storage volumes serve the following functions:

1. Emergency storage: Retention of wastewater in the event of pump failure or power outage, preventing uncontrolled overflow to the receiving environment.
2. Flow buffering: Retention of peak-period flows for controlled discharge to the WDC network during off-peak periods, minimising additional loading on a constrained downstream network and supporting self-cleansing velocities during off-peak discharge windows.

Storage volumes are distributed across the wet well, dedicated ancillary storage chambers, and upstream pipeline capacity. The ancillary storage chambers are connected to the collection manhole. A second water supply connection with a 24 VDC solenoid valve will be installed on the RPZ outlet to facilitate automated wash-down should storage chambers be activated.

- Pump Station A (Stage 1): 13.95m³ — exceeding the RITS 9-hour minimum by a substantial margin and providing capacity for off-peak flow management across 7 industrial lots.
- Pump Station B (Stage 1C): 53.63m³ — providing equivalent redundancy for 2 industrial lots.

7.8.3. External Services

The successful operation of the wastewater pump stations is dependent on the provision of the following external services, to be coordinated at detailed design stage:

Wash Water

A DN50 PE rider main, supplied from the proposed 63 mm OD water main in the adjacent road corridor, will provide the water supply required for pump station washdown purposes. In accordance with RITS Section 5.2.10.8(a), wastewater pump stations are classified as a High Hazard risk and require reduced pressure zone (RPZ) type backflow prevention devices installed above ground level adjacent to the electrical control cabinet.

Power

A reticulated underground power network will be designed and installed in the proposed adjacent roads to provide a dedicated point of connection for each pump station. Final electrical load requirements and connection arrangements will be confirmed with WAIPA Networks at detailed design stage.

Stormwater / Overland Flow

Each pump station platform is elevated above the surrounding ground level and situated clear of the identified overland flow paths and 1% AEP flood extent, ensuring free drainage of stormwater away from cabinet enclosures and well openings.

Electrical and Telemetry

The electrical and telemetry requirements for the proposed pump stations will be confirmed with Waikato Waters at detailed design stage. The alarm and operational data control system, including level monitoring (low level, duty/standby start, high level, critical overflow, and storage-in-use alarms), will be installed by the developer or by Waikato Waters at the developer's cost, in accordance with the control level settings prescribed in RITS Section 5.2.10.9

8. Water

8.1. Design Standards

The RITS sets out design and construction standards for water reticulation, potable water supply and firefighting supply in accordance with SNZPAS 4509:2008 (NZ Fire Service Fire Fighting Water Supply Code of Practice).

8.1.1. Engineering Exception Decisions

There are no engineering exception decisions.

8.2. Existing Fire Hydrant Testing

To determine the potable water supply available to service the proposed development, the conditions of the existing public reticulation system must be understood. WSP has undertaken a hydraulic assessment of the existing public infrastructure (refer to Appendix H for detailed model results).

8.3. Design Flows

Calculations for potable water demand indicate a calculated peak residential demand of 8.13/s. Please refer to **Error! Reference source not found.** and Fast Track Memo (August 2026) for further details and assumptions for retirement dwelling occupancy and usage. Amended design flows are based on typical retirement village occupancy, which is lower than typical urban areas.

Allowing for firefighting demand (see section 5.7) and 60% of peak residential demand, in accordance with RITS V2, we estimate the peak demand from the development to be ***29.88 L/s***.

8.4. Water Supply modelling

A Water Servicing Assessment (WSA) was previously undertaken by BTW Ltd. on behalf of the client and Waipa District Council to identify the preferred water supply servicing solution for the proposed development. The assessment included a review of the capacity of the existing Council water supply network and hydraulic modelling of future network performance, including projected growth to 2050. The full assessment is provided in Error! Reference source not found.

The assessment identified two upgrades to the existing water supply network that are required to ensure adequate levels of service for both potable water supply and firefighting demand:

8.4.1. Option 1

Taylors Avenue Network Upgrade: Upgrade the existing non-return valves (NRVs) on Taylors Avenue to improve network performance and available pressures (refer to Figure 8).

8.4.2. Option 2

Greenhill Drive Booster Pump Upgrade: Replace the existing Greenhill Drive booster pump and install a secondary standby pump to improve network resilience and maintain the required supply pressures during peak demand and firefighting events (refer to Figure 9).

At the time of the initial Fast-Track application, Waipa District Council adopted the findings of this assessment as the basis of its servicing feedback. Following discussions with Council and a cost-benefit assessment of the available options, both upgrades were agreed upon and have been incorporated into the proposed water supply servicing strategy for the development.

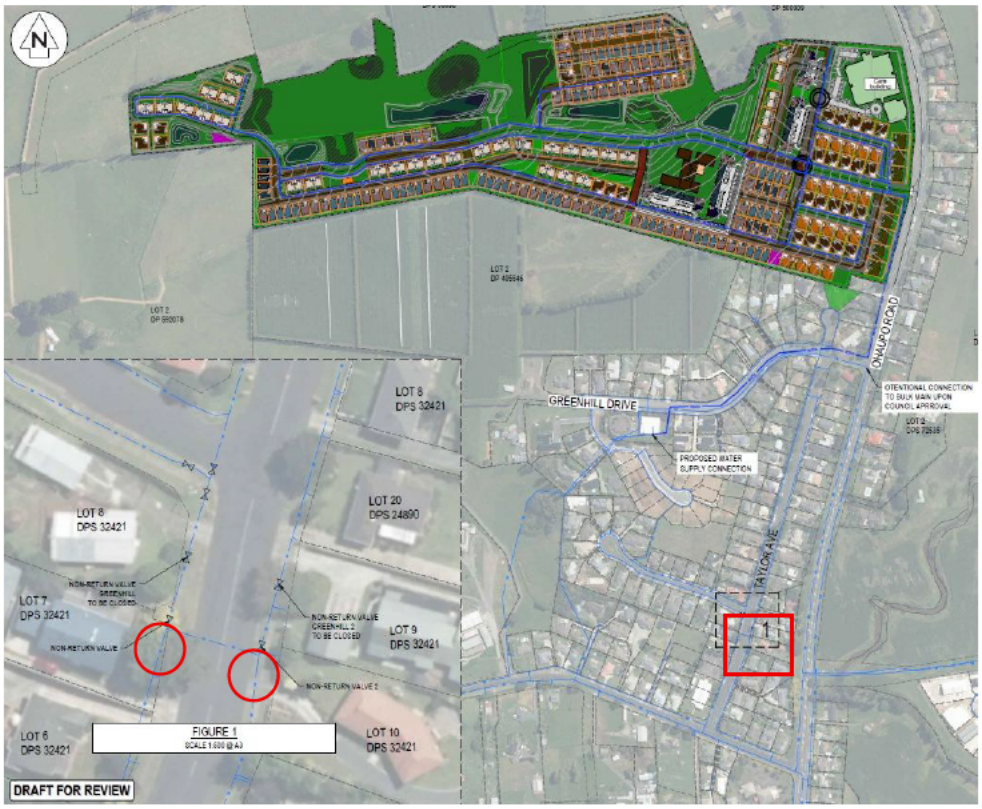


Figure 17: Proposed Water Supply Non-Return Valves



Figure 18: Proposed Water Supply Pump Upgrades

8.5. Strategic Water Connection

The proposed connection point is to an existing OD150mm watermain along Greenhill drive. The connection point and proposed alignment is indicated in figure 19 below:

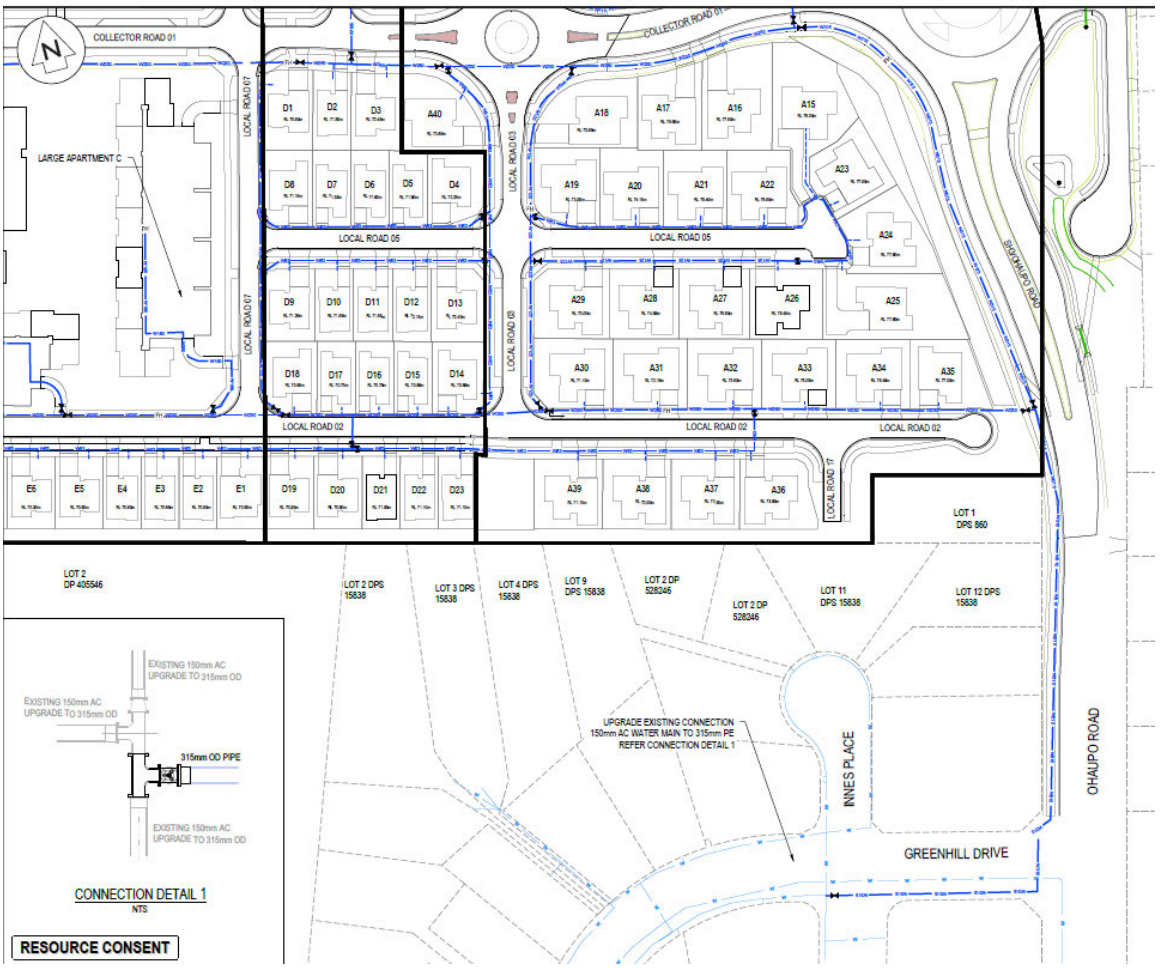


Figure 19: Proposed Water Connection

The connection will continue via a 315OD bulk main into the site via the proposed roundabout and downsize into a looped 250OD supply main throughout the site. From here a dual 125mmOD watermain and/or 63mmOD rider main network will service residential units across the wider site.

8.6. Proposed Reticulation

The proposed water supply reticulation network will comprise of:

- PE100 PN12.5 315OD, 250OD & 180OD bulk mains
- PE100 PN12.5 125OD supply mains,
- PE80B PN12.5 63OD rider mains, generally located on the opposite side of the road to the proposed watermains
- It is proposed that the Water Supply Infrastructure within stages 1A, 1B and 1E will be publicly vested
- It is proposed that the Water Supply Infrastructure with stages 1C, 1D, 2, 3 & 4 will be privately owned.

Refer to Appendix A – Engineering Plans for the water supply drawings for further details.

8.7. Lot Connections

All lots will be serviced with lot connections as per RITS guidelines and specifications.

8.8. Connection Points and Staging

Please refer to Table 20 below for further detail on the proposed servicing arrangements and Water Supply EPANET Modelling details.

Stage	Lots	Proposed Servicing	General Comment
Stage 1	78-Dwellings 56-Apartments 1 – Manged Care Facility	Construct 315OD PE main From Taylors Hill water reservoir to the proposed development terminating at the northern end for Road 03 (for future growth). Includes upgrade of existing 150mmm AC to 315OD PE main from the water reservoir to the Taylors Ave/Greenhill Drive Intersection A combination of 63OD PE, 125OD PE, DN180 PE, and DN250 PE mains to be constructed as per Maven Drawing (C600-Water Supply).	An EPANET model for the Harlow LRV development has been designed, and it is concluded that the design nodes will achieve the required 200kPa (20m head pressure) as required by RITS 6.1.3.1 On Demand Water Supply Area. **Peak Demand:** S1 – 7.85l/s S2 – 9.30l/s S3 – 1.99l/s S4 – 1.63l/s
Stage 2- 5	219-Dwellings 61-Apartments 1 – Recreation Facility	A combination of DN63 PE, DN125 PE, DN180 PE, and DN250 PE mains to be constructed as per Maven Drawing (C600-Water Supply).	

Table 20: Water Supply Connection Points & Staging

8.9. Firefighting Supply

Reticulation will be designed to provide the development area with a suitable means of potable supply and firefighting supply. The minimum firefighting supply service level for FW2 for residential areas and FW3 for apartments and the Care centre. It is anticipated that FW3 areas will meet the required flows via a combination of on street hydrants and internal sprinkler systems.

The water supply requirements for FW3 for firefighting supply are as follows:

- A primary water flow of 25 litres/sec within a radial distance of 135m.
- An additional secondary flow of 25 litres/sec within a radial distance of 270m.
- The required flow must be achieved from a maximum of three hydrants operating simultaneously.
- A minimum running pressure of 100kPa.

The water supply requirements for FW6 for firefighting supply are as follows:

- A primary water flow of 100 litres/sec within a radial distance of 135m.
- An additional secondary flow of 100 litres/sec within a radial distance of 270m.
- The required flow must be achieved from a maximum of eight hydrants operating simultaneously.
- A minimum running pressure of 100kPa.

Fire Scenario	Minimum Pipe Sizes and Pressure Head			Fire Water Classification	Result
	Trunk Main	Distribution Main	Booster Pump		
Care Centre	315mm	200mm	Required	FW6	Pass
Small Apartment Block A	250mm	200mm	Required	FW3	Pass
Small Apartment Block B	250mm	200mm	Required	FW3	Pass
Large Apartment A	250mm	200mm	Required	FW5	Pass
Large Apartment B	250mm	200mm	Required	FW5	Pass
Large Apartment C	250mm	200mm	Required	FW5	Pass
Recreation Facility	250mm	200mm	Required	FW5	Pass

Table 21: Modelled Fire Pressure Scenarios

Note: To make the fire scenario work, a booster pump, at least a 315mm trunk main, and a 200mm distribution main are required. However, for the care centre if a 200mm distribution main is used, the minimum pressure is 20.33 m. To provide sufficient margin above the minimum pressure requirement, a 250mm distribution main is recommended.

Note: That option 2 (section 8.4.2 above) requires the replacement and upgrading of the booster pump within the Taylors Hill water reservoir, discussions and co-ordination with Waikato Waters will ensure that the upgraded booster pump meets the minimum requirements for both the existing network (as modelled by WSP) and the firefighting requirements in table 21 above.

The booster pumps will either be installed by the developer or by Waikato Waters at the developer's cost, depending on final commercial arrangements with Waikato Waters.

9. Services

9.1. Power

Waipa Networks are the power service providers for this area, and they will prepare the power design for this development. Waipa Networks have confirmed that power can be extended to the proposed development with minimal upgrades. The proposed power design will be developed during the detailed design stage. The proposed power services allocation in the road corridor is shown on the proposed roading typical sections C340 series drawings. Refer to the roading and services drawings in Appendix A for further details.

It is noted that as part of the NZTA SH3/ Ōhaupō Road proposed 4-arm roundabout will require the relocation of both overhead and underground power services within the NZTA road corridor to the new proposed NZTA road alignment/corridor.

9.2. Communications

Communications are owned by Tuatahi First Fibre within the area to service the site. A multi-duct along Ōhaupō road can be detailed in the existing services plan. Discussion with TFF to ensure how to best connect to the multi-duct would be required. Otherwise, there are no known limiting factors to connect the wider site to the existing duct.

It is noted that as part of the NZTA SH3/ Ōhaupō Road proposed 4-arm roundabout will require the relocation of both Tuatahi fibre services and any older Chorus copper networks within the NZTA road corridor to the new proposed NZTA road alignment/corridor.

9.3. Gas

First Gas are the gas service provider for the Te Awamutu area and local to the proposed site area. An existing trench with a medium pressure natural gas distribution main with an operating pressure at 210-420 kPa. The main travels along Ōhaupō road within NZTA SH3/ Ōhaupō Road corridor, South of the proposed site boundary, then heads West along Greenhill drive to service the residential area. It is possible to extend this service north along the new proposed NZTA road alignment/corridor.

Discussions with First Gas service provider will occur at detailed design once internal gas demand/requirements (if any) are finalised.

10. Conclusion

Based on this engineering report we consider that the proposed development can be accommodated at the subject site, without generating any adverse effects to the existing infrastructure and the receiving environment.

Stormwater drainage can be provided for the proposed development. Various options have been considered, and we are confident the best practicable option is proposed. The treatment and disposal methods both offer a high amenity value consistent with the urban design ideals.

Wastewater drainage can be provided for the proposed development.

There is public water supply infrastructure surrounding the site, which can provide potable water and firefighting supply for this proposed development.

Utility providers will be engaged to confirm that the network have sufficient capacity in the surrounding streets to provide service or are able to undertake upgrades in a suitable timeframe.

Therefore, information gathered to-date confirms the site is suitable for the proposed development.

Appendix A – Engineering Plans

Appendix B – Integrated Transport Assessment

Appendix C – Water Supply Calculations

Maven Waikato Ltd		Job Number 411001	Sheet 1	Rev A
Job Title	Harlow Lifestyle Village	Author	Date	Checked
Calc Title	Overall - Water Demand	JPK	7/10/2026	DJM

Water Catchment

As per RITS Standard 6.2.3

	3	people per dwelling
Demand	260	l/person/day

Demand Rates

Average Demand =	260	litres/person/day
Peak Demand (5x) =	1300	litres/person/day

Population	Dwellings	People	Occupancy
Proposed Dwellings	297	3	891
Care Centre + staffs	1	120	120
Proposed Large Apartment	87	3	261
Proposed Small Apartment	30	3	90
Recreation Facility (45ppl per ha)	0.4	45	18
Total			1380

Demand	Persons	Rate l/p/day	Flow l/s
AD Water	1380	260	4.15
PD Water	1380	1300	20.76

Peak Demand	Persons	Rate l/p/day	Flow l/s
PD Water	1380	1300	**20.76**

	Maven Waikato Ltd	Job Number	Sheet	Rev
		411001	1	A
Job Title	Harlow Lifestyle Village	Author	Date	Checked
Calc Title	Stage 1 - Water Demand	JPK	7/10/2026	DJM

Water Catchment

As per RITS Standard 6.2.3

3 people per dwelling

Demand 260 l/person/day

Demand Rates

Average Demand = 260 litres/person/day

Peak Demand (5x) = 1300 litres/person/day

Population	Dwellings	People	Occupancy
Proposed Dwellings	78	3	234
Proposed Large Apartment	26	3	78
Proposed Small Apartment	30	3	90
Care Centre (100 beds + staffs)	1	120	120
Total			522

Demand	Persons	Rate l/p/day	Flow l/s
AD Water	522	260	1.57
PD Water	522	1300	7.85

Peak Demand	Persons	Rate l/p/day	Flow l/s
PD Water	522	1300	**7.85**

Maven Waikato Ltd		Job Number 411001	Sheet 1	Rev A
Job Title	Harlow Lifestyle Village	Author	Date	Checked
Calc Title	Stage 2 - Water Demand	JPK	7/10/2026	DJM

Water Catchment

As per RITS Standard 6.2.3

	3	people per dwelling
Demand	260	l/person/day

Demand Rates

Average Demand =	260	litres/person/day
Peak Demand (5x) =	1300	litres/person/day

Population	Dwellings	People	Occupancy
Proposed Dwellings	139	3	417
Proposed Large Apartment	61	3	183
Recreation Facility (45ppl per Ha)	0.4	45	18
Total			618

Demand	Persons	Rate l/p/day	Flow l/s
AD Water	618	260	1.86
PD Water	618	1300	9.30

Peak Demand	Persons	Rate l/p/day	Flow l/s
PD Water	618	1300	**9.30**

Maven Waikato Ltd		Job Number 411001	Sheet 1	Rev A
Job Title	Harlow Lifestyle Village	Author	Date	Checked
Calc Title	Stage 3 - Water Demand	JPK	7/10/2026	DJM

Water Catchment

As per RITS Standard 6.2.3

	3	people per dwelling
Demand	260	l/person/day

Demand Rates

Average Demand =	260	litres/person/day
Peak Demand (5x) =	1300	litres/person/day

Population	Dwellings	People	Occupancy
Proposed Dwellings	44	3	132
Total			132

Demand	Persons	Rate l/p/day	Flow l/s
AD Water	132	260	0.40
PD Water	132	1300	1.99

Peak Demand	Persons	Rate l/p/day	Flow l/s
PD Water	132	1300	**1.99**

Maven Waikato Ltd		Job Number 411001	Sheet 1	Rev A
Job Title	Harlow Lifestyle Village	Author	Date	Checked
Calc Title	Stage 4 - Water Demand	JPK	7/10/2026	DJM

Water Catchment

As per RITS Standard 6.2.3

	3	people per dwelling
Demand	260	l/person/day

Demand Rates

Average Demand =	260	litres/person/day
Peak Demand (5x) =	1300	litres/person/day

Population	Dwellings	People	Occupancy
Proposed Dwellings	36	3	108
Total			108

Demand	Persons	Rate l/p/day	Flow l/s
AD Water	108	260	0.33
PD Water	108	1300	1.63

Peak Demand	Persons	Rate l/p/day	Flow l/s
PD Water	108	1300	**1.63**

	Project Number	Project	Date	Author	Checked
	411001	Harlow Lifestyle Village	28-09-2026	JPK	SM

Water Supply Modelling Report

Hydraulic modelling assessment

Draft report prepared for internal review

Recommendations

To make the fire scenario work, a booster pump, at least a 315mm trunk main, and a 200mm distribution main are required. However, for the care centre if a 200mm distribution main is used, the minimum pressure is 20.33 m. To provide sufficient margin above the minimum pressure requirement, a 250mm distribution main is recommended.

Table 1: Description of modelled fire scenarios

Fire Scenario	Minimum Pipe Sizes and Pressure Head			Fire Water Classification	Result
Care Centre	Trunk Main	Distribution Main	Booster Pump	FW6	Pass
	315mm	200mm	Required		
Small Apartment Block A	250mm	200mm	Required	FW3	Pass
Small Apartment Block B	250mm	200mm	Required	FW3	Pass
Large Apartment A	250mm	200mm	Required	FW5	Pass
Large Apartment B	250mm	200mm	Required	FW5	Pass
Large Apartment C	250mm	200mm	Required	FW5	Pass
Recreation Facility	250mm	200mm	Required	FW5	Pass

Water Supply standards as per RITS,

6.1.3.1 **On Demand Water Supply Area**

The design of the network shall conform to the Code of Practice for Fire Fighting Water Supplies (SNZ PAS 4509), and shall be such that a water supply connection can be provided for each lot.

The water supply network shall achieve the following standards:

a) The residual pressure and flow at point of supply to residential lots shall be a minimum of 200 kPa (20m) and 25 L/min. Some specific areas may require a higher Level of Service – check with the relevant Council.

b) The minimum fire supply service level shall be FW2 for residential areas and FW3 for all other areas. Some specific areas may require a higher level of service.

FW3, FW5 and FW6 standards from SNZ PAS 4509:2008 – Table 2

Fire water classification	Reticulated water supply			Non-reticulated water supply	
	Required water flow within a distance of 135 m	Additional water flow within a distance of 270 m	Maximum number of fire hydrants to provide flow	Minimum water storage within a distance of 90 m (see Note 8)	
				Time (firefighting) (min)	Volume (m^3)
FW1	450 L/min (7.5 L/s) (See Note 3)	–	1	15	7
FW2	750 L/min (12.5 L/s)	750 L/min (12.5 L/s)	2	30	45
FW3	1500 L/min (25 L/s)	1500 L/min (25 L/s)	3	60	180
FW4	3000 L/min (50 L/s)	3000 L/min (50 L/s)	4	90	540
FW5	4500 L/min (75 L/s)	4500 L/min (75 L/s)	6	120	1080
FW6	6000 L/min (100 L/s)	6000 L/min (100 L/s)	8	180	2160

Table 1 – Method for determining required water supply classification

Sprinklered structures															
Category	**Water supply classification (see table 2)**														
Single family homes with a sprinkler system installed to an approved Standard	FW1														
All other structures (apart from single family homes) with a sprinkler system installed to an approved Standard	FW2														
Non-sprinklered structures															
Category	**Water supply classification (see table 2)**														
Housing; includes single family dwellings, multi-unit dwellings, but excludes multi-storey apartment blocks	FW2														
All other structures (characterised by fire hazard category[1]), examples of which are given below	**Water supply classification (see table 2)**														
	Floor area of largest firecell of the building (m^2)														
	0-199[10]	200-399	400-599	600-799	800-999	1000-1199	1200-1399	1400-1599	1600-1799	1800-1999	2000-2199	2200-2399	2400-2599	2600-2799	> 2800
FHC 1[2]	FW3	FW3	FW3	FW4	FW4	FW4	FW5	FW5	FW5	FW5	FW5	FW5	FW5	FW5	FW6
FHC 2[3]	FW3	FW3	FW4	FW5	FW5	FW5	FW6	FW6	FW6	FW7	FW7	FW7	FW7	FW7	FW7
FHC 3[4]	FW3	FW4	FW5	FW5	FW6	FW6	FW7	FW7	FW7	FW7	FW7	FW7	FW7	FW7	FW7
FHC 4[5]	FW4	FW6	FW6	FW6	FW6	FW7	FW7	FW7	FW7	FW7	FW7	FW7	FW7	FW7	FW7

Requirement: All lots to have at least 200kPa (20m) of pressure at supply point

Assumptions: The calculation is modelled based on the assumption that the existing water supply connection points will be 300kPa (30m).

Care Centre Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.79 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 251.95 L/s with a head gain of 2.77 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer's pump curve are to be confirmed during detailed design.

An FW6 classification has been adopted for the Care Centre fire scenario.

Table 3: Care Centre Fire Scenario results

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
355	250	No	Fail
400	250	No	Fail
450	250	No	Fail
250	200	Yes	Fail
315	200	Yes	Pass (20.33)
315	250	Yes	Pass (20.46)

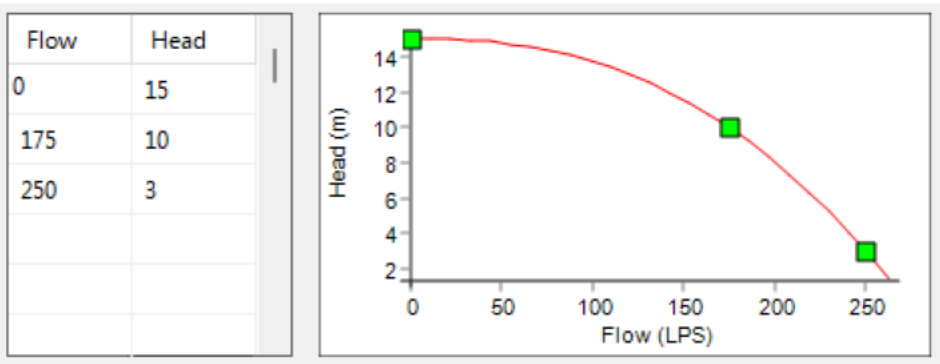


Figure 1: Care Centre Fire Scenario Pump Curve

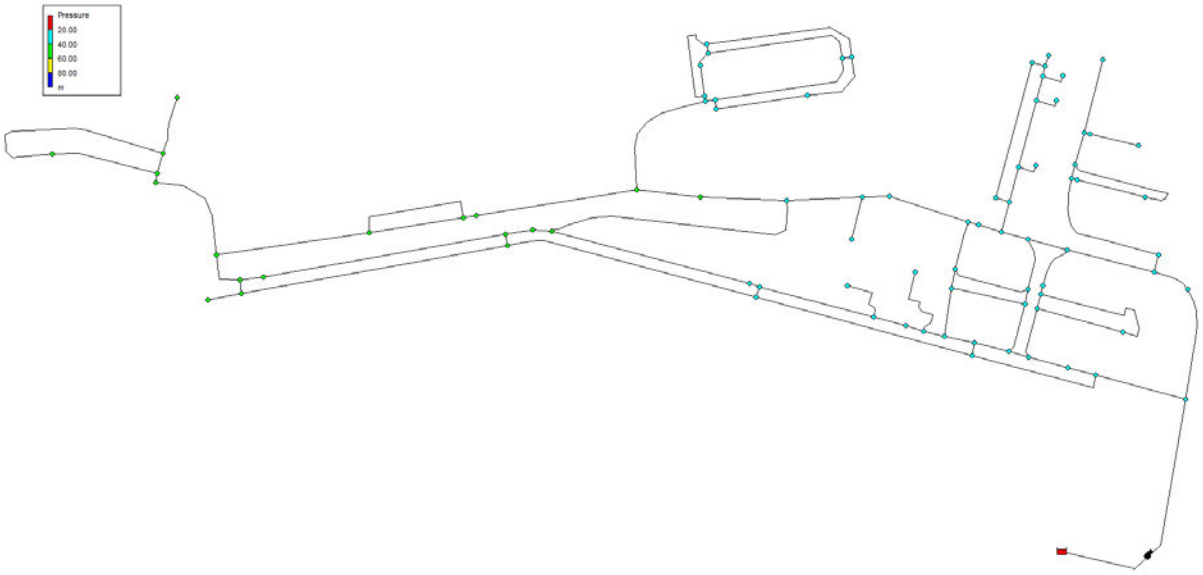


Figure 1: Care Centre Fire Scenario Network Layout

Maven Waikato Ltd		Job Number 411001	Sheet 1	Rev A
Job Title	Harlow Lifestyle Village	Author	Date	Checked
Calc Title	Overall - Water Demand	JPK	25/09/2026	DJM

Water Catchment

As per RITS Standard 6.2.3

	3	people per dwelling
Demand	260	l/person/day

Demand Rates

Average Demand =	260	litres/person/day
Peak Demand (5x) =	1300	litres/person/day

Population	Dwellings	People	Occupancy
Proposed Dwellings	297	3	891
Care Centre + staffs	1	120	120
Proposed Large Apartment	87	3	261
Proposed Small Apartment	30	3	90
Recreation Facility (45ppl per ha)	0.5	45	22.5
Total			1384.5

Demand	Persons	Rate l/p/day	Flow l/s
AD Water	1384.5	260	4.17
PD Water	1384.5	1300	20.83

Peak Demand	Persons	Rate l/p/day	Flow l/s
PD Water	1384.5	1300	20.83

Figure 2: Overall Water Demand Calculation Sheet

Care Centre Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	98.84	48.94
Junc n2	50.2	0.07	97.85	47.65
Junc n3	56.9	0	97.85	40.95
Junc n4	48.8	12.5	97.81	49.01
Junc n6	69.66	0	100.45	30.79
Junc n7	69.3	0	100.30	31.00
Junc n8	73.4	0.01	99.49	26.09
Junc n9	72.2	115	97.81	25.61
Junc n10	72.9	0	99.17	26.27
Junc n11	69.2	0	98.79	29.59
Junc n12	69.7	0	98.97	29.27
Junc n13	72.2	0.05	99.45	27.25
Junc n14	70.2	0	100.50	30.30
Junc n15	72.2	0	98.15	25.95
Junc n16	72.2	0.04	98.71	26.51
Junc n17	57.6	0	99.46	41.86
Junc n18	60.6	0.14	98.74	38.14
Junc n19	54.7	0	98.72	44.02
Junc n20	59.9	0.04	97.38	37.48
Junc n21	64.2	12.5	95.77	31.57
Junc n22	63.7	0.1	97.44	33.74
Junc n23	60.2	0.05	97.53	37.33
Junc n24	65.3	0	99.99	34.69
Junc n25	65.8	0.15	100.57	34.77
Junc n26	56.97	0.13	99.36	42.39
Junc n27	57.2	0.17	99.37	42.17
Junc n28	51.6	0.04	98.91	47.31
Junc n29	50.2	0.04	98.91	48.71
Junc n30	53.2	0.04	98.79	45.59
Junc n31	53.7	0.07	98.75	45.05
Junc n32	75.2	0	99.90	24.70
Junc n33	75.7	0	99.15	23.45
Junc n34	74.7	0.13	99.93	25.23
Junc n35	51.4	0.02	98.89	47.49
Junc n36	57.1	12.5	99.41	42.31
Junc n37	65.2	12.5	99.99	34.79
Junc n38	69.6	12.5	100.39	30.79
Junc n39	69.7	0	100.52	30.82
Junc n40	69.97	0.03	100.72	30.75
Junc n41	70.06	0	100.78	30.72
Junc n42	71.2	12.5	101.05	29.85
Junc n43	72.97	0.07	101.29	28.32
Junc n44	71.2	0.03	102.55	31.35
Junc n45	53.7	12.5	98.75	45.05
Junc n46	60.2	12.5	98.72	38.52
Junc n47	64.2	12.5	98.75	34.55
Junc n48	69.2	115	98.79	29.59
Junc n50	48.2	0.06	97.85	49.65
Junc n51	60.2	0	97.55	37.35
Junc n52	63.2	0.21	97.44	34.24
Junc n53	63.2	12.5	97.44	34.24
Junc n54	71.7	0.05	100.02	28.32
Junc n55	47.2	12.5	97.87	50.67
Junc n56	74.39	0	99.84	25.45
Junc n57	75.19	0	99.84	24.65
Junc n58	51.63	12.5	98.90	47.27
Junc n59	70.47	12.5	98.46	27.99
Junc n60	75.32	115	99.70	24.38
Junc n61	69.83	0.02	100.63	30.80
Junc n62	70.05	0.13	100.66	30.61
Junc n63	76.1	0.07	98.69	22.59
Junc n64	80.52	115	100.85	20.33
Junc n65	77.95	115	98.54	20.59
Junc n66	62.93	0	98.75	35.82
Junc n67	61.45	0.2	98.75	37.30
Junc n68	68.66	0.27	100.30	31.64
Junc n69	66.1	0.27	100.45	34.35
Junc n70	71.32	0.03	98.91	27.59
Junc n71	70.69	0	98.87	28.18
Junc n72	70.61	0	98.87	28.26

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n53	63.2	12.5	97.44	34.24
Junc n54	71.7	0.05	100.02	28.32
Junc n55	47.2	12.5	97.87	50.67
Junc n56	74.39	0	99.84	25.45
Junc n57	75.19	0	99.84	24.65
Junc n58	51.63	12.5	98.90	47.27
Junc n59	70.47	12.5	98.46	27.99
Junc n60	75.32	115	99.70	24.38
Junc n61	69.83	0.02	100.63	30.80
Junc n62	70.05	0.13	100.66	30.61
Junc n63	76.1	0.07	98.69	22.59
Junc n64	80.52	115	100.85	20.33
Junc n65	77.95	115	98.54	20.59
Junc n66	62.93	0	98.75	35.82
Junc n67	61.45	0.2	98.75	37.30
Junc n68	68.66	0.27	100.30	31.64
Junc n69	66.1	0.27	100.45	34.35
Junc n70	71.32	0.03	98.91	27.59
Junc n71	70.69	0	98.87	28.18
Junc n72	70.61	0	98.87	28.26
Junc n73	70.42	12.5	98.87	28.45
Junc n74	71.02	0	98.88	27.86
Junc n75	70.75	0	98.90	28.15
Junc n76	74.74	0.27	98.87	24.13
Junc n77	75.11	0.27	98.88	23.77
Junc n78	70.63	0.08	98.88	28.25
Junc n79	70.36	0	98.90	28.54
Junc n80	70.96	0.23	98.90	27.94
Junc n81	79.23	0.02	100.53	21.30
Junc n82	79.23	0	100.44	21.21
Junc n83	67.99	0.08	97.43	29.44
Junc n84	68.15	0	97.43	29.28
Junc n85	60.28	0.08	97.53	37.25
Junc n86	74.82	115	99.14	24.32
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	16.19	2.06	40.56	0.019
Pipe p4	44.19	100	-4.68	0.60	4.07	0.023
Pipe p5	71.43	50	-1.00	0.51	6.81	0.026
Pipe p6	236.7	50	0.65	0.33	3.04	0.028
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-3.19	1.62	58.46	0.022
Pipe p9	9.962	50	-0.30	0.16	0.75	0.031
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	0.20	0.10	0.34	0.033
Pipe p12	109.7	50	1.00	0.51	6.81	0.026
Pipe p13	24.49	200	-19.29	0.61	1.92	0.020
Pipe p14	17.45	200	-22.92	0.73	2.64	0.019
Pipe p15	2	200	-27.19	0.87	3.62	0.019
Pipe p16	29.99	200	-24.08	0.77	2.89	0.019
Pipe p17	15.84	200	-27.71	0.88	3.75	0.019
Pipe p18	20.29	200	-27.78	0.88	3.77	0.019
Pipe p19	20.52	200	-23.34	0.74	2.73	0.019
Pipe p20	36.49	200	-40.52	1.29	7.58	0.018
Pipe p21	26.03	200	-44.15	1.41	8.88	0.018
Pipe p22	83.26	200	-58.97	1.88	15.18	0.017
Pipe p23	83.3	200	8.92	0.28	0.46	0.022
Pipe p24	11.53	200	9.09	0.29	0.48	0.022
Pipe p25	146.4	200	5.47	0.17	0.19	0.024
Pipe p26	57.48	200	-1.95	0.06	0.03	0.028
Pipe p27	77.36	200	-5.57	0.18	0.19	0.024
Pipe p28	37.04	200	-43.33	1.38	8.58	0.018
Pipe p29	12.57	315	67.70	0.87	2.14	0.018
Pipe p30	242.8	50	-0.51	0.26	1.91	0.027
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.035
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	-17.19	2.19	45.29	0.019
Pipe p35	13.41	100	-16.38	2.09	41.44	0.019

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p36	7.537	100	-17.16	2.18	45.14	0.019
Pipe p37	230.3	50	-0.87	0.44	5.22	0.026
Pipe p38	2.15	100	-0.14	0.02	0.00	0.027
Pipe p39	20	50	1.49	0.76	14.32	0.024
Pipe p40	40	50	1.35	0.69	11.99	0.025
Pipe p41	100	200	-9.12	0.29	0.48	0.022
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	2.50	1.28	37.41	0.023
Pipe p46	7.6	200	3.97	0.13	0.10	0.025
Pipe p47	10	50	0.49	0.25	1.75	0.027
Pipe p48	78	50	-1.16	0.59	9.05	0.025
Pipe p49	20	50	-3.68	1.87	76.38	0.021
Pipe p50	44	150	5.04	0.28	0.65	0.023
Pipe p51	74	315	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	16.42	0.52	1.42	0.020
Pipe p53	5.3	200	-15.94	0.51	1.34	0.021
Pipe p54	237.6	200	-19.56	0.62	1.97	0.020
Pipe p55	190.9	200	-23.56	0.75	2.78	0.019
Pipe p56	1.84	150	66.70	3.77	77.43	0.016
Pipe p57	193.5	150	-4.10	0.23	0.44	0.024
Pipe p58	21.1	200	-32.82	1.04	5.13	0.018
Pipe p59	36.1	200	-22.16	0.71	2.48	0.020
Pipe p60	10	150	-10.66	0.60	2.60	0.021
Pipe p61	131.7	150	-14.80	0.84	4.76	0.020
Pipe p62	79	100	0.79	0.10	0.15	0.029
Pipe p63	127.9	50	0.77	0.39	4.25	0.027
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	315	192.97	2.48	14.92	0.015
Pipe p66	54.1	150	33.35	1.89	21.45	0.018
Pipe p67	44	150	0.06	0.00	0.00	0.058
Pipe p68	65	200	-4.96	0.16	0.16	0.024
Pipe p69	8.8	200	-5.02	0.16	0.16	0.024
Pipe p70	58.1	150	0.08	0.00	0.00	0.048

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p70	58.1	150	0.08	0.00	0.00	0.048
Pipe p71	107.2	200	-24.00	0.76	2.87	0.019
Pipe p72	81.3	150	0.08	0.00	0.00	0.051
Pipe p73	18.7	200	-37.32	1.19	6.51	0.018
Pipe p74	33.6	200	6.66	0.21	0.27	0.023
Pipe p75	8.2	200	6.58	0.21	0.26	0.023
Pipe p76	4.1	200	3.62	0.12	0.09	0.026
Pipe p77	8	50	2.96	1.51	50.94	0.022
Pipe p78	21.8	200	7.50	0.24	0.33	0.023
Pipe p79	32.1	150	0.08	0.00	0.00	0.043
Pipe p80	32.5	150	0.08	0.00	0.00	0.043
Pipe p81	28.8	315	66.70	0.86	2.09	0.018
Pipe p82	28.2	200	-44.82	1.43	9.13	0.018
Pipe p83	128.8	50	-0.67	0.34	3.23	0.027
Pipe p84	7.8	50	-0.69	0.35	3.44	0.027
Pipe p85	33.5	150	0.07	0.00	0.00	0.000
Pipe p86	16.3	200	6.81	0.22	0.28	0.023
Pipe p87	70.7	200	6.74	0.21	0.27	0.023
Pipe p88	67.5	200	-59.53	1.89	15.45	0.017
Pipe p89	30.4	315	-159.62	2.05	10.50	0.015
Pipe p90	21	315	100.09	1.28	4.42	0.017
Pipe p91	115	315	100.09	1.28	4.42	0.017
Pipe p92	93	200	-8.65	0.28	0.43	0.022
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.040
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	32.35	4.12	146.13	0.017
Pipe p102	65.2	100	-1.00	0.13	0.23	0.028
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	251.95	0.00	-2.77	0.000

Small Apartment A Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.88 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 116.43 L/s with a head gain of 13.16 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer’s pump curve are to be confirmed during detailed design.

An FW3 classification has been adopted for the Care Centre fire scenario.

Table 4: Small Apartment A Fire Scenario Results

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
355	250	No	Fail
400	250	No	Fail
450	250	No	Fail (18.99 at highest point)
250	200	Yes	Pass
315	200	Yes	Pass
315	250	Yes	Pass

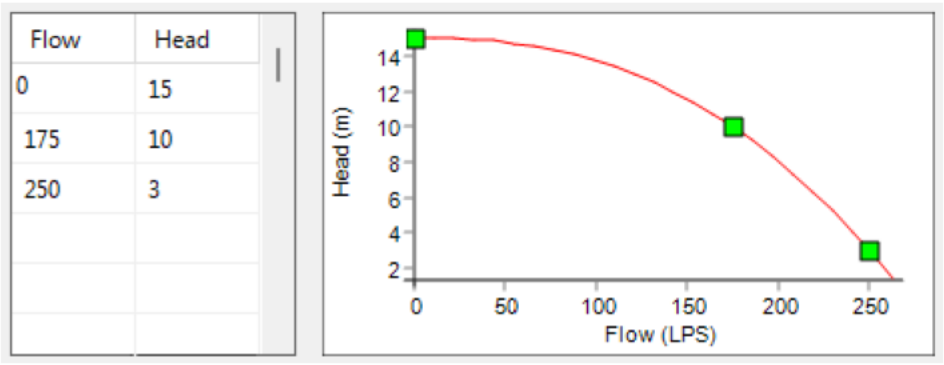


Figure 1: Small Apartment A Fire Scenario Pump Curve

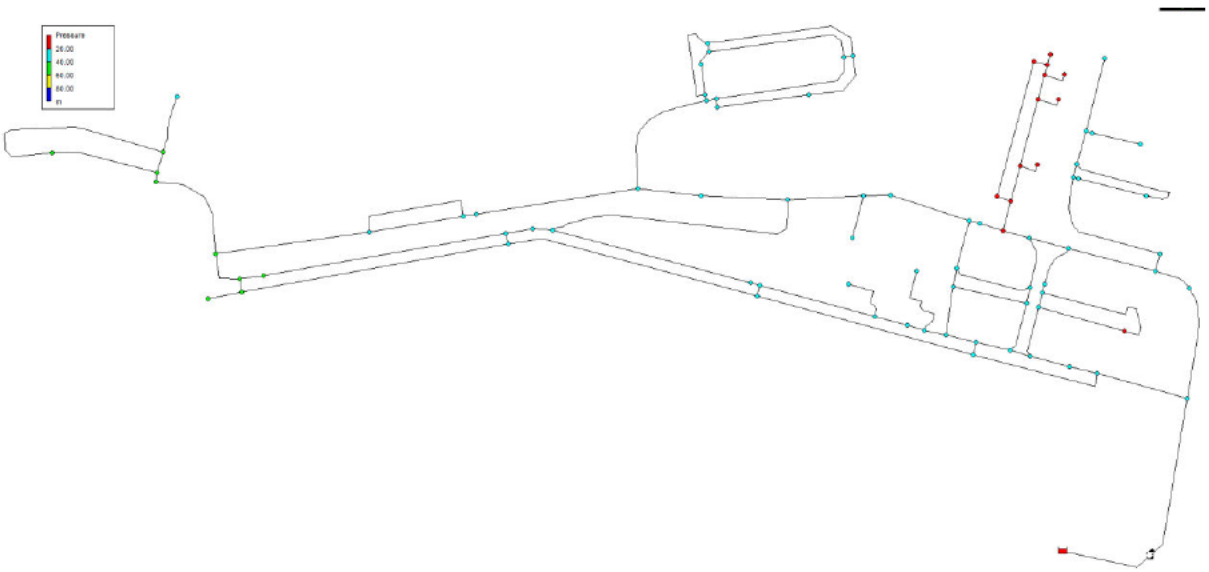


Figure 3: Small Apartment A Fire Scenario Network Layout

Small Apartment A Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m	Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	99.53	49.63	Junc n37	65.2	12.5	102.66	37.46
Junc n2	50.2	0.07	98.53	48.33	Junc n38	69.6	12.5	103.58	33.98
Junc n3	56.9	0	98.53	41.63	Junc n39	69.7	0	103.87	34.17
Junc n4	48.8	12.5	98.50	49.70	Junc n40	69.97	0.03	104.34	34.37
Junc n6	69.66	0	103.71	34.05	Junc n41	70.06	0	104.48	34.42
Junc n7	69.3	0	103.38	34.08	Junc n42	71.2	12.5	104.93	33.73
Junc n8	73.4	0.01	101.70	28.30	Junc n43	72.97	0.07	105.30	32.33
Junc n9	72.2	12.5	102.44	30.24	Junc n44	71.2	0.03	107.36	36.16
Junc n10	72.9	0	100.01	27.11	Junc n45	53.7	12.5	98.96	45.26
Junc n11	69.2	0	98.59	29.39	Junc n46	60.2	12.5	98.58	38.38
Junc n12	69.7	0	99.15	29.45	Junc n47	64.2	86.3	98.49	34.29
Junc n13	72.2	0.05	100.76	28.56	Junc n48	69.2	86.3	98.59	29.39
Junc n14	70.2	0	103.79	33.59	Junc n50	48.2	0.06	98.53	50.33
Junc n15	72.2	0	102.67	30.47	Junc n51	60.2	0	97.46	37.26
Junc n16	72.2	0.04	103.06	30.86	Junc n52	63.2	0.21	97.35	34.15
Junc n17	57.6	0	101.31	43.71	Junc n53	63.2	12.5	97.35	34.15
Junc n18	60.6	0.14	98.54	37.94	Junc n54	71.7	0.05	102.38	30.68
Junc n19	54.7	0	98.64	43.94	Junc n55	47.2	12.5	98.56	51.36
Junc n20	59.9	0.04	97.30	37.40	Junc n56	74.39	0	104.06	29.67
Junc n21	64.2	12.5	95.69	31.49	Junc n57	75.19	0	104.06	28.87
Junc n22	63.7	0.1	97.35	33.65	Junc n58	51.63	12.5	99.72	48.09
Junc n23	60.2	0.05	97.45	37.25	Junc n59	70.47	12.5	95.35	24.88
Junc n24	65.3	0	102.67	37.37	Junc n60	75.32	12.5	104.06	28.74
Junc n25	65.8	0.15	103.96	38.16	Junc n61	69.83	0.02	104.11	34.28
Junc n26	56.97	0.13	101.06	44.09	Junc n62	70.05	0.13	104.16	34.11
Junc n27	57.2	0.17	101.07	43.87	Junc n63	76.1	0.07	103.05	26.95
Junc n28	51.6	0.04	99.75	48.15	Junc n64	80.52	12.5	104.88	24.36
Junc n29	50.2	0.04	99.75	49.55	Junc n65	77.95	0.36	104.06	26.11
Junc n30	53.2	0.04	99.23	46.03	Junc n66	62.93	0	98.49	35.56
Junc n31	53.7	0.07	98.99	45.29	Junc n67	61.45	0.2	98.49	37.04
Junc n32	75.2	0	104.06	28.86	Junc n68	68.66	0.27	103.38	34.72
Junc n33	75.7	0	103.61	27.91	Junc n69	66.1	0.27	103.71	37.61
Junc n34	74.7	0.13	104.06	29.36	Junc n70	71.32	0.01	98.67	27.35
Junc n35	51.4	0.02	99.70	48.30	Junc n71	70.69	0	95.54	24.85
Junc n36	57.1	12.5	101.19	44.09	Junc n72	70.61	0	95.51	24.90

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n54	71.7	0.05	102.38	30.68
Junc n55	47.2	12.5	98.56	51.36
Junc n56	74.39	0	104.06	29.67
Junc n57	75.19	0	104.06	28.87
Junc n58	51.63	12.5	99.72	48.09
Junc n59	70.47	12.5	95.35	24.88
Junc n60	75.32	12.5	104.06	28.74
Junc n61	69.83	0.02	104.11	34.28
Junc n62	70.05	0.13	104.16	34.11
Junc n63	76.1	0.07	103.05	26.95
Junc n64	80.52	12.5	104.88	24.36
Junc n65	77.95	0.36	104.06	26.11
Junc n66	62.93	0	98.49	35.56
Junc n67	61.45	0.2	98.49	37.04
Junc n68	68.66	0.27	103.38	34.72
Junc n69	66.1	0.27	103.71	37.61
Junc n70	71.32	0.03	98.67	27.35
Junc n71	70.69	0	95.54	24.85
Junc n72	70.61	0	95.51	24.90
Junc n73	70.42	86.3	95.50	25.08
Junc n74	71.02	0	95.95	24.93
Junc n75	70.75	0	98.11	27.36
Junc n76	74.74	86.3	95.14	20.40
Junc n77	75.11	86.3	95.54	20.43
Junc n78	70.63	0.08	97.95	27.32
Junc n79	70.36	0	97.70	27.34
Junc n80	70.96	0.23	97.70	26.74
Junc n81	79.23	0.02	104.25	25.02
Junc n82	79.23	0	104.22	24.99
Junc n83	67.99	0.08	97.35	29.36
Junc n84	68.15	0	97.35	29.20
Junc n85	60.28	0.08	97.45	37.17
Junc n86	74.82	86.3	103.60	28.78
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	-10.39	1.32	17.84	0.020
Pipe p4	44.19	100	-8.62	1.10	12.61	0.021
Pipe p5	71.43	50	-1.91	0.97	22.65	0.023
Pipe p6	236.7	50	1.34	0.68	11.69	0.025
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-4.91	2.50	130.33	0.020
Pipe p9	9.962	50	-0.44	0.22	1.35	0.026
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	0.53	0.27	2.10	0.028
Pipe p12	109.7	50	0.76	0.39	4.13	0.027
Pipe p13	24.49	200	-34.08	1.08	5.50	0.018
Pipe p14	17.45	200	-37.70	1.20	6.63	0.018
Pipe p15	2	200	-42.66	1.36	8.34	0.018
Pipe p16	29.99	200	-37.83	1.20	6.67	0.018
Pipe p17	15.84	200	-41.45	1.32	7.90	0.018
Pipe p18	20.29	200	-41.53	1.32	7.93	0.018
Pipe p19	20.52	200	-38.66	1.23	6.94	0.018
Pipe p20	36.49	200	-52.71	1.68	12.33	0.017
Pipe p21	26.03	200	-56.33	1.79	13.95	0.017
Pipe p22	83.26	200	-76.78	2.44	24.75	0.016
Pipe p23	83.3	200	23.89	0.76	2.85	0.019
Pipe p24	11.53	200	24.40	0.78	2.96	0.019
Pipe p25	146.4	200	20.77	0.66	2.20	0.020
Pipe p26	57.48	200	13.36	0.43	0.97	0.021
Pipe p27	77.36	200	9.73	0.31	0.54	0.022
Pipe p28	37.04	200	-106.77	3.40	45.58	0.015
Pipe p29	12.57	250	4.49	0.09	0.04	0.026
Pipe p30	242.8	50	-0.90	0.46	5.44	0.025
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.033
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	-14.05	1.79	31.18	0.019
Pipe p35	13.41	100	-13.39	1.70	28.51	0.019

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p36	7.537	100	-14.02	1.78	31.05	0.019
Pipe p37	230.3	50	-1.39	0.71	12.57	0.025
Pipe p38	2.15	100	-0.14	0.02	0.01	0.054
Pipe p39	20	50	2.51	1.28	37.44	0.023
Pipe p40	40	50	2.42	1.23	35.20	0.023
Pipe p41	100	200	-24.43	0.78	2.97	0.019
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	4.43	2.26	107.59	0.021
Pipe p46	7.6	200	5.44	0.17	0.18	0.024
Pipe p47	10	50	0.88	0.45	5.17	0.025
Pipe p48	78	50	-2.02	1.03	25.16	0.023
Pipe p49	20	50	-6.71	3.42	232.00	0.020
Pipe p50	44	150	9.13	0.52	1.95	0.021
Pipe p51	74	250	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	31.72	1.01	4.82	0.019
Pipe p53	5.3	200	-30.85	0.98	4.57	0.019
Pipe p54	237.6	200	-34.48	1.10	5.62	0.018
Pipe p55	190.9	200	-39.04	1.24	7.07	0.018
Pipe p56	1.84	150	3.73	0.21	0.37	0.024
Pipe p57	193.5	150	-6.35	0.36	0.99	0.023
Pipe p58	21.1	200	-50.66	1.61	11.46	0.017
Pipe p59	36.1	200	-36.63	1.17	6.28	0.018
Pipe p60	10	150	-14.04	0.79	4.32	0.020
Pipe p61	131.7	150	-20.42	1.16	8.65	0.019
Pipe p62	79	100	0.65	0.08	0.11	0.030
Pipe p63	127.9	50	0.63	0.32	2.92	0.028
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	250	128.80	2.62	21.76	0.016
Pipe p66	54.1	150	0.10	0.01	0.00	0.044
Pipe p67	44	150	0.06	0.00	0.00	0.058
Pipe p68	65	200	11.03	0.35	0.68	0.022
Pipe p69	8.8	200	10.97	0.35	0.67	0.022
Pipe p70	58.1	150	0.08	0.00	0.00	0.024

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p70	58.1	150	0.08	0.00	0.00	0.024
Pipe p71	107.2	200	-37.75	1.20	6.65	0.018
Pipe p72	81.3	150	0.08	0.00	0.00	0.051
Pipe p73	18.7	200	-30.47	0.97	4.47	0.019
Pipe p74	33.6	200	51.88	1.65	11.98	0.017
Pipe p75	8.2	200	26.86	0.85	3.54	0.019
Pipe p76	4.1	200	25.03	0.80	3.10	0.019
Pipe p77	8	50	1.83	0.93	20.95	0.024
Pipe p78	21.8	200	78.80	2.51	25.97	0.016
Pipe p79	32.1	150	25.03	1.42	12.60	0.019
Pipe p80	32.5	150	25.03	1.42	12.60	0.019
Pipe p81	28.8	250	3.73	0.08	0.03	0.026
Pipe p82	28.2	200	-109.27	3.48	47.58	0.015
Pipe p83	128.8	50	-1.79	0.91	20.18	0.024
Pipe p84	7.8	50	-1.82	0.93	20.66	0.024
Pipe p85	33.5	150	0.07	0.00	0.00	0.057
Pipe p86	16.3	200	76.98	2.45	24.87	0.016
Pipe p87	70.7	200	76.91	2.45	24.83	0.016
Pipe p88	67.5	200	-96.37	3.07	37.70	0.016
Pipe p89	30.4	250	-125.17	2.55	20.64	0.016
Pipe p90	21	250	28.79	0.59	1.36	0.019
Pipe p91	115	250	28.79	0.59	1.36	0.019
Pipe p92	93	200	-14.06	0.45	1.07	0.021
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.042
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	24.26	3.09	85.78	0.018
Pipe p102	65.2	100	-0.76	0.10	0.14	0.029
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	205.58	0.00	-7.58	0.000

Small Apartment B Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.88 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 116.43 L/s with a head gain of 13.16 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer’s pump curve are to be confirmed during detailed design.

An FW3 classification has been adopted for the Care Centre fire scenario.

Table 4: Small Apartment A Fire Scenario Results

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
355	250	No	Fail
400	250	No	Fail
450	250	No	Fail (18.99 at highest point)
250	200	Yes	Pass
315	200	Yes	Pass
315	250	Yes	Pass

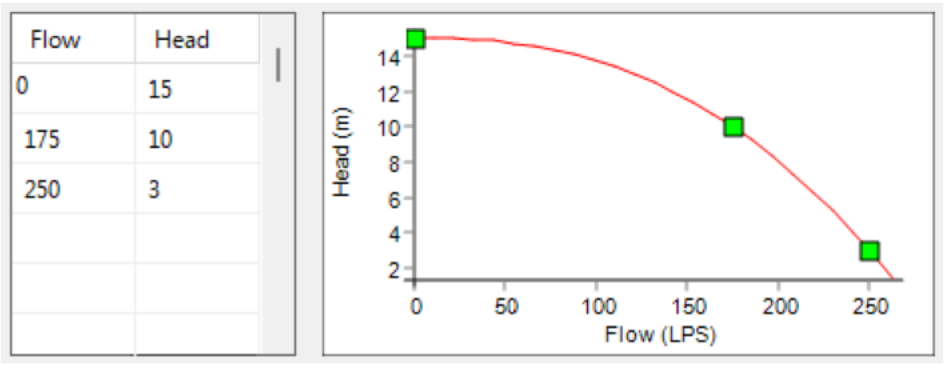


Figure 1: Small Apartment B Fire Scenario Pump Curve

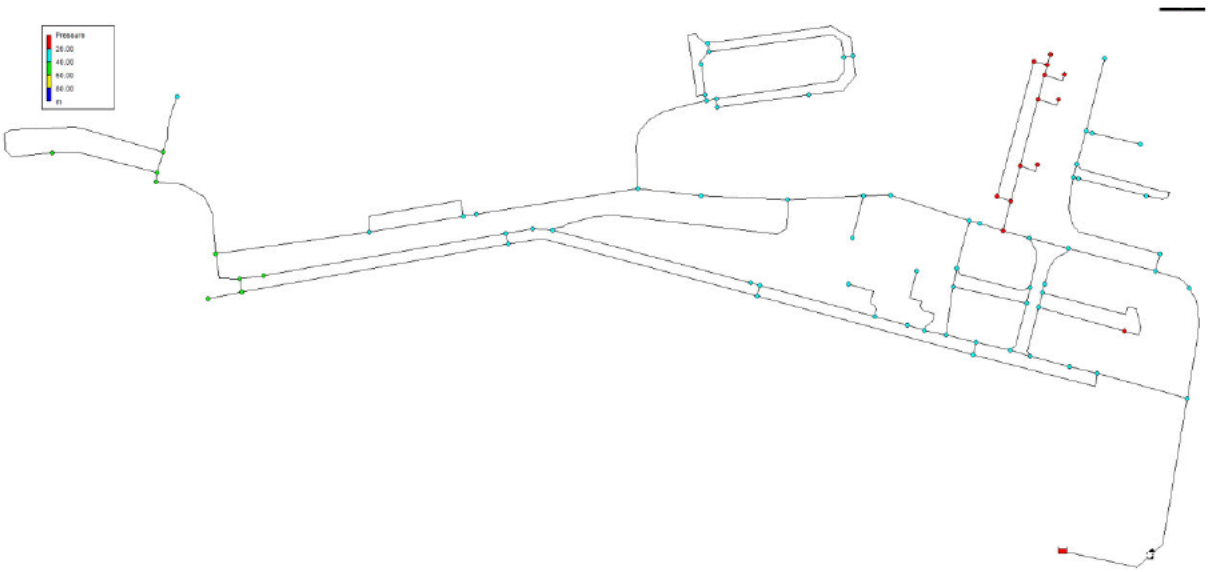


Figure 3: Small Apartment B Fire Scenario Network Layout

Small Apartment B Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m	Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	99.53	49.63	Junc n37	65.2	12.5	102.66	37.46
Junc n2	50.2	0.07	98.53	48.33	Junc n38	69.6	12.5	103.58	33.98
Junc n3	56.9	0	98.53	41.63	Junc n39	69.7	0	103.87	34.17
Junc n4	48.8	12.5	98.50	49.70	Junc n40	69.97	0.03	104.34	34.37
Junc n6	69.66	0	103.71	34.05	Junc n41	70.06	0	104.48	34.42
Junc n7	69.3	0	103.38	34.08	Junc n42	71.2	12.5	104.93	33.73
Junc n8	73.4	0.01	101.70	28.30	Junc n43	72.97	0.07	105.30	32.33
Junc n9	72.2	12.5	102.44	30.24	Junc n44	71.2	0.03	107.36	36.16
Junc n10	72.9	0	100.01	27.11	Junc n45	53.7	12.5	98.96	45.26
Junc n11	69.2	0	98.59	29.39	Junc n46	60.2	12.5	98.58	38.38
Junc n12	69.7	0	99.15	29.45	Junc n47	64.2	86.3	98.49	34.29
Junc n13	72.2	0.05	100.76	28.56	Junc n48	69.2	86.3	98.59	29.39
Junc n14	70.2	0	103.79	33.59	Junc n50	48.2	0.06	98.53	50.33
Junc n15	72.2	0	102.67	30.47	Junc n51	60.2	0	97.46	37.26
Junc n16	72.2	0.04	103.06	30.86	Junc n52	63.2	0.21	97.35	34.15
Junc n17	57.6	0	101.31	43.71	Junc n53	63.2	12.5	97.35	34.15
Junc n18	60.6	0.14	98.54	37.94	Junc n54	71.7	0.05	102.38	30.68
Junc n19	54.7	0	98.64	43.94	Junc n55	47.2	12.5	98.56	51.36
Junc n20	59.9	0.04	97.30	37.40	Junc n56	74.39	0	104.06	29.67
Junc n21	64.2	12.5	95.69	31.49	Junc n57	75.19	0	104.06	28.87
Junc n22	63.7	0.1	97.35	33.65	Junc n58	51.63	12.5	99.72	48.09
Junc n23	60.2	0.05	97.45	37.25	Junc n59	70.47	12.5	95.35	24.88
Junc n24	65.3	0	102.67	37.37	Junc n60	75.32	12.5	104.06	28.74
Junc n25	65.8	0.15	103.96	38.16	Junc n61	69.83	0.02	104.11	34.28
Junc n26	56.97	0.13	101.06	44.09	Junc n62	70.05	0.13	104.16	34.11
Junc n27	57.2	0.17	101.07	43.87	Junc n63	76.1	0.07	103.05	26.95
Junc n28	51.6	0.04	99.75	48.15	Junc n64	80.52	12.5	104.88	24.36
Junc n29	50.2	0.04	99.75	49.55	Junc n65	77.95	0.36	104.06	26.11
Junc n30	53.2	0.04	99.23	46.03	Junc n66	62.93	0	98.49	35.56
Junc n31	53.7	0.07	98.99	45.29	Junc n67	61.45	0.2	98.49	37.04
Junc n32	75.2	0	104.06	28.86	Junc n68	68.66	0.27	103.38	34.72
Junc n33	75.7	0	103.61	27.91	Junc n69	66.1	0.27	103.71	37.61
Junc n34	74.7	0.13	104.06	29.36	Junc n70	71.32	0.01	98.67	27.35
Junc n35	51.4	0.02	99.70	48.30	Junc n71	70.69	0	95.54	24.85
Junc n36	57.1	12.5	101.19	44.09	Junc n72	70.61	0	95.51	24.90

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n54	71.7	0.05	102.38	30.68
Junc n55	47.2	12.5	98.56	51.36
Junc n56	74.39	0	104.06	29.67
Junc n57	75.19	0	104.06	28.87
Junc n58	51.63	12.5	99.72	48.09
Junc n59	70.47	12.5	95.35	24.88
Junc n60	75.32	12.5	104.06	28.74
Junc n61	69.83	0.02	104.11	34.28
Junc n62	70.05	0.13	104.16	34.11
Junc n63	76.1	0.07	103.05	26.95
Junc n64	80.52	12.5	104.88	24.36
Junc n65	77.95	0.36	104.06	26.11
Junc n66	62.93	0	98.49	35.56
Junc n67	61.45	0.2	98.49	37.04
Junc n68	68.66	0.27	103.38	34.72
Junc n69	66.1	0.27	103.71	37.61
Junc n70	71.32	0.03	98.67	27.35
Junc n71	70.69	0	95.54	24.85
Junc n72	70.61	0	95.51	24.90
Junc n73	70.42	86.3	95.50	25.08
Junc n74	71.02	0	95.95	24.93
Junc n75	70.75	0	98.11	27.36
Junc n76	74.74	86.3	95.14	20.40
Junc n77	75.11	86.3	95.54	20.43
Junc n78	70.63	0.08	97.95	27.32
Junc n79	70.36	0	97.70	27.34
Junc n80	70.96	0.23	97.70	26.74
Junc n81	79.23	0.02	104.25	25.02
Junc n82	79.23	0	104.22	24.99
Junc n83	67.99	0.08	97.35	29.36
Junc n84	68.15	0	97.35	29.20
Junc n85	60.28	0.08	97.45	37.17
Junc n86	74.82	86.3	103.60	28.78
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	-10.39	1.32	17.84	0.020
Pipe p4	44.19	100	-8.62	1.10	12.61	0.021
Pipe p5	71.43	50	-1.91	0.97	22.65	0.023
Pipe p6	236.7	50	1.34	0.68	11.69	0.025
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-4.91	2.50	130.33	0.020
Pipe p9	9.962	50	-0.44	0.22	1.35	0.026
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	0.53	0.27	2.10	0.028
Pipe p12	109.7	50	0.76	0.39	4.13	0.027
Pipe p13	24.49	200	-34.08	1.08	5.50	0.018
Pipe p14	17.45	200	-37.70	1.20	6.63	0.018
Pipe p15	2	200	-42.66	1.36	8.34	0.018
Pipe p16	29.99	200	-37.83	1.20	6.67	0.018
Pipe p17	15.84	200	-41.45	1.32	7.90	0.018
Pipe p18	20.29	200	-41.53	1.32	7.93	0.018
Pipe p19	20.52	200	-38.66	1.23	6.94	0.018
Pipe p20	36.49	200	-52.71	1.68	12.33	0.017
Pipe p21	26.03	200	-56.33	1.79	13.95	0.017
Pipe p22	83.26	200	-76.78	2.44	24.75	0.016
Pipe p23	83.3	200	23.89	0.76	2.85	0.019
Pipe p24	11.53	200	24.40	0.78	2.96	0.019
Pipe p25	146.4	200	20.77	0.66	2.20	0.020
Pipe p26	57.48	200	13.36	0.43	0.97	0.021
Pipe p27	77.36	200	9.73	0.31	0.54	0.022
Pipe p28	37.04	200	-106.77	3.40	45.58	0.015
Pipe p29	12.57	250	4.49	0.09	0.04	0.026
Pipe p30	242.8	50	-0.90	0.46	5.44	0.025
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.033
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	-14.05	1.79	31.18	0.019
Pipe p35	13.41	100	-13.39	1.70	28.51	0.019

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p36	7.537	100	-14.02	1.78	31.05	0.019
Pipe p37	230.3	50	-1.39	0.71	12.57	0.025
Pipe p38	2.15	100	-0.14	0.02	0.01	0.054
Pipe p39	20	50	2.51	1.28	37.44	0.023
Pipe p40	40	50	2.42	1.23	35.20	0.023
Pipe p41	100	200	-24.43	0.78	2.97	0.019
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	4.43	2.26	107.59	0.021
Pipe p46	7.6	200	5.44	0.17	0.18	0.024
Pipe p47	10	50	0.88	0.45	5.17	0.025
Pipe p48	78	50	-2.02	1.03	25.16	0.023
Pipe p49	20	50	-6.71	3.42	232.00	0.020
Pipe p50	44	150	9.13	0.52	1.95	0.021
Pipe p51	74	250	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	31.72	1.01	4.82	0.019
Pipe p53	5.3	200	-30.85	0.98	4.57	0.019
Pipe p54	237.6	200	-34.48	1.10	5.62	0.018
Pipe p55	190.9	200	-39.04	1.24	7.07	0.018
Pipe p56	1.84	150	3.73	0.21	0.37	0.024
Pipe p57	193.5	150	-6.35	0.36	0.99	0.023
Pipe p58	21.1	200	-50.66	1.61	11.46	0.017
Pipe p59	36.1	200	-36.63	1.17	6.28	0.018
Pipe p60	10	150	-14.04	0.79	4.32	0.020
Pipe p61	131.7	150	-20.42	1.16	8.65	0.019
Pipe p62	79	100	0.65	0.08	0.11	0.030
Pipe p63	127.9	50	0.63	0.32	2.92	0.028
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	250	128.80	2.62	21.76	0.016
Pipe p66	54.1	150	0.10	0.01	0.00	0.044
Pipe p67	44	150	0.06	0.00	0.00	0.058
Pipe p68	65	200	11.03	0.35	0.68	0.022
Pipe p69	8.8	200	10.97	0.35	0.67	0.022
Pipe p70	58.1	150	0.08	0.00	0.00	0.024

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p70	58.1	150	0.08	0.00	0.00	0.024
Pipe p71	107.2	200	-37.75	1.20	6.65	0.018
Pipe p72	81.3	150	0.08	0.00	0.00	0.051
Pipe p73	18.7	200	-30.47	0.97	4.47	0.019
Pipe p74	33.6	200	51.88	1.65	11.98	0.017
Pipe p75	8.2	200	26.86	0.85	3.54	0.019
Pipe p76	4.1	200	25.03	0.80	3.10	0.019
Pipe p77	8	50	1.83	0.93	20.95	0.024
Pipe p78	21.8	200	78.80	2.51	25.97	0.016
Pipe p79	32.1	150	25.03	1.42	12.60	0.019
Pipe p80	32.5	150	25.03	1.42	12.60	0.019
Pipe p81	28.8	250	3.73	0.08	0.03	0.026
Pipe p82	28.2	200	-109.27	3.48	47.58	0.015
Pipe p83	128.8	50	-1.79	0.91	20.18	0.024
Pipe p84	7.8	50	-1.82	0.93	20.66	0.024
Pipe p85	33.5	150	0.07	0.00	0.00	0.057
Pipe p86	16.3	200	76.98	2.45	24.87	0.016
Pipe p87	70.7	200	76.91	2.45	24.83	0.016
Pipe p88	67.5	200	-96.37	3.07	37.70	0.016
Pipe p89	30.4	250	-125.17	2.55	20.64	0.016
Pipe p90	21	250	28.79	0.59	1.36	0.019
Pipe p91	115	250	28.79	0.59	1.36	0.019
Pipe p92	93	200	-14.06	0.45	1.07	0.021
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.042
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	24.26	3.09	85.78	0.018
Pipe p102	65.2	100	-0.76	0.10	0.14	0.029
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	205.58	0.00	-7.58	0.000

Large Apartment A Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.84 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 202.05 L/s with a head gain of 8.13 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer’s pump curve are to be confirmed during detailed design.

An FW5 classification has been adopted for the Care Centre fire scenario.

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
355	250	No	Fail
400	250	No	Fail
450	250	No	Fail
250	200	Yes	Pass
315	200	Yes	Pass
315	250	Yes	Pass

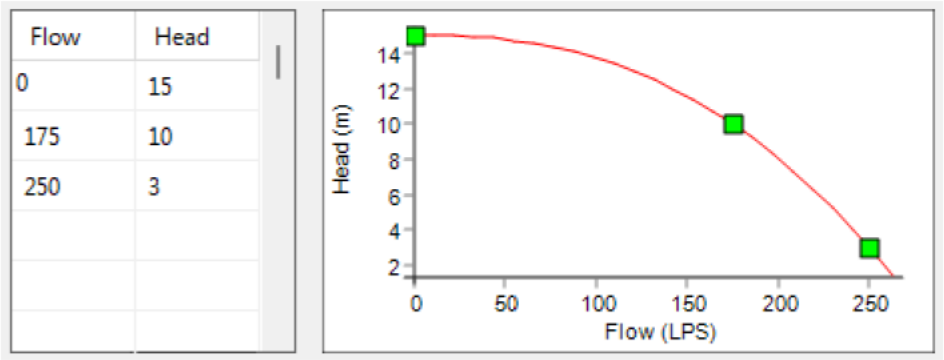


Figure 1: Large Apartment A Fire Scenario Pump Curve

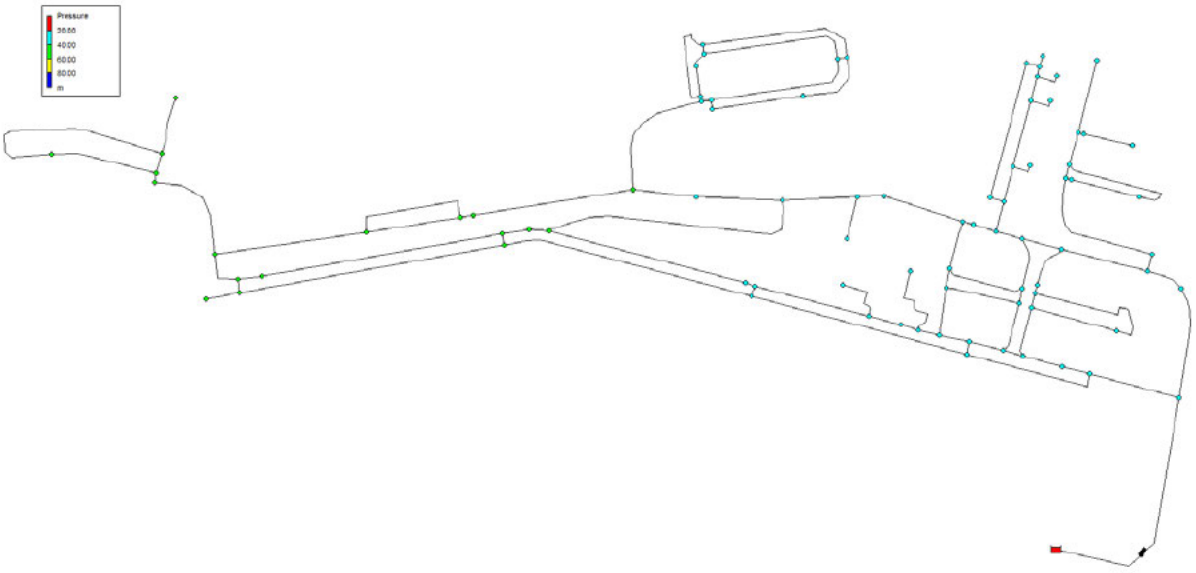


Figure 3: Large Apartment A Fire Scenario Network Layout

Large Apartment A Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	100.16	50.26
Junc n2	50.2	0.07	99.17	48.97
Junc n3	56.9	0	99.17	42.27
Junc n4	48.8	12.5	99.13	50.33
Junc n6	69.66	0	101.10	31.44
Junc n7	69.3	0	100.77	31.47
Junc n8	73.4	0.01	103.37	29.97
Junc n9	72.2	86.3	101.91	29.71
Junc n10	72.9	0	102.47	29.57
Junc n11	69.2	0	101.40	32.20
Junc n12	69.7	0	101.45	31.75
Junc n13	72.2	0.05	102.15	29.95
Junc n14	70.2	0	101.62	31.42
Junc n15	72.2	0	102.03	29.83
Junc n16	72.2	0.04	102.24	30.04
Junc n17	57.6	0	100.24	42.64
Junc n18	60.6	0.14	100.44	39.84
Junc n19	54.7	0	100.23	45.53
Junc n20	59.9	0.04	98.89	38.99
Junc n21	64.2	12.5	97.28	33.08
Junc n22	63.7	0.1	98.94	35.24
Junc n23	60.2	0.05	99.04	38.84
Junc n24	65.3	0	100.35	35.05
Junc n25	65.8	0.15	102.04	36.24
Junc n26	56.97	0.13	100.23	43.26
Junc n27	57.2	0.17	100.27	43.07
Junc n28	51.6	0.04	100.16	48.56
Junc n29	50.2	0.04	100.16	49.96
Junc n30	53.2	0.04	100.17	46.97
Junc n31	53.7	0.07	100.18	46.48
Junc n32	75.2	0	105.64	30.44
Junc n33	75.7	0	105.63	29.93
Junc n34	74.7	0.13	105.64	30.94
Junc n35	51.4	0.02	100.16	48.76
Junc n36	57.1	12.5	100.23	43.13
Junc n37	65.2	12.5	100.34	35.14
Junc n38	69.6	12.5	100.90	31.30
Junc n39	69.7	0	101.62	31.92
Junc n40	69.97	0.03	102.69	32.72
Junc n41	70.06	0	103.00	32.94
Junc n42	71.2	12.5	103.73	32.53
Junc n43	72.97	0.07	104.30	31.33
Junc n44	71.2	0.03	107.67	36.47
Junc n45	53.7	12.5	100.18	46.48
Junc n46	60.2	12.5	100.30	40.10
Junc n47	64.2	86.3	100.57	36.37
Junc n48	69.2	86.3	101.46	32.26
Junc n50	48.2	0.06	99.17	50.97
Junc n51	60.2	0	99.05	38.85
Junc n52	63.2	0.21	98.94	35.74
Junc n53	63.2	12.5	98.94	35.74
Junc n54	71.7	0.05	102.13	30.43
Junc n55	47.2	12.5	99.19	51.99
Junc n56	74.39	0	105.64	31.25
Junc n57	75.19	0	105.64	30.45
Junc n58	51.63	12.5	100.16	48.53
Junc n59	70.47	12.5	101.37	30.90
Junc n60	75.32	12.5	105.64	30.32
Junc n61	69.83	0.02	102.17	32.34
Junc n62	70.05	0.13	102.26	32.21
Junc n63	76.1	0.07	102.23	26.13
Junc n64	80.52	12.5	106.06	25.54
Junc n65	77.95	0.36	105.64	27.69
Junc n66	62.93	0	100.56	37.63
Junc n67	61.45	0.2	100.56	39.11
Junc n68	68.66	0.27	100.77	32.11
Junc n69	66.1	0.27	100.07	33.97
Junc n70	71.32	0.03	101.82	30.50
Junc n71	70.69	0	101.78	31.09
Junc n72	70.61	0	101.77	31.16

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n53	63.2	12.5	98.94	35.74
Junc n54	71.7	0.05	102.13	30.43
Junc n55	47.2	12.5	99.19	51.99
Junc n56	74.39	0	105.64	31.25
Junc n57	75.19	0	105.64	30.45
Junc n58	51.63	12.5	100.16	48.53
Junc n59	70.47	12.5	101.37	30.90
Junc n60	75.32	12.5	105.64	30.32
Junc n61	69.83	0.02	102.17	32.34
Junc n62	70.05	0.13	102.26	32.21
Junc n63	76.1	0.07	102.23	26.13
Junc n64	80.52	12.5	106.06	25.54
Junc n65	77.95	0.36	105.64	27.69
Junc n66	62.93	0	100.56	37.63
Junc n67	61.45	0.2	100.56	39.11
Junc n68	68.66	0.27	100.77	32.11
Junc n69	66.1	0.27	100.07	33.97
Junc n70	71.32	0.03	101.82	30.50
Junc n71	70.69	0	101.78	31.09
Junc n72	70.61	0	101.77	31.16
Junc n73	70.42	86.3	101.77	31.35
Junc n74	71.02	0	101.79	30.77
Junc n75	70.75	0	101.81	31.06
Junc n76	74.74	0.27	101.78	27.04
Junc n77	75.11	0.27	101.79	26.68
Junc n78	70.63	0.08	101.78	31.15
Junc n79	70.36	0	101.81	31.45
Junc n80	70.96	86.3	101.81	30.85
Junc n81	79.23	0.02	105.66	26.43
Junc n82	79.23	0	105.65	26.42
Junc n83	67.99	0.08	98.94	30.95
Junc n84	68.15	0	98.94	30.79
Junc n85	60.28	0.08	99.04	38.76
Junc n86	74.82	86.3	105.63	30.81
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	15.05	1.92	35.42	0.019
Pipe p4	44.19	100	-2.35	0.30	1.14	0.025
Pipe p5	71.43	50	-1.21	0.62	9.75	0.025
Pipe p6	236.7	50	-0.32	0.16	0.83	0.031
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-5.69	2.90	171.10	0.020
Pipe p9	9.962	50	-0.79	0.40	4.42	0.027
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	-0.09	0.05	0.07	0.035
Pipe p12	109.7	50	0.11	0.06	0.11	0.036
Pipe p13	24.49	200	-6.01	0.19	0.22	0.024
Pipe p14	17.45	200	-9.64	0.31	0.53	0.022
Pipe p15	2	200	-34.34	1.09	5.58	0.018
Pipe p16	29.99	200	-28.73	0.91	4.01	0.019
Pipe p17	15.84	200	-53.76	1.71	12.79	0.017
Pipe p18	20.29	200	-78.78	2.51	25.96	0.016
Pipe p19	20.52	200	-58.45	1.86	14.93	0.017
Pipe p20	36.49	200	-68.46	2.18	20.01	0.017
Pipe p21	26.03	200	-72.08	2.29	22.02	0.016
Pipe p22	83.26	200	-100.07	3.19	40.43	0.016
Pipe p23	83.3	200	-3.88	0.12	0.10	0.025
Pipe p24	11.53	200	-3.99	0.13	0.10	0.025
Pipe p25	146.4	200	-7.62	0.24	0.34	0.023
Pipe p26	57.48	200	-15.03	0.48	1.21	0.021
Pipe p27	77.36	200	-18.66	0.59	1.80	0.020
Pipe p28	37.04	200	-75.88	2.42	24.22	0.016
Pipe p29	12.57	250	3.84	0.08	0.03	0.026
Pipe p30	242.8	50	-0.23	0.12	0.44	0.032
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.033
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	-10.01	1.27	16.64	0.020
Pipe p35	13.41	100	-9.53	1.21	15.19	0.020

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p36	7.537	100	-9.98	1.27	16.54	0.020
Pipe p37	230.3	50	-1.07	0.54	7.69	0.026
Pipe p38	2.15	100	-0.14	0.02	0.00	0.027
Pipe p39	20	50	-1.60	0.82	16.39	0.024
Pipe p40	40	50	-1.39	0.71	12.61	0.025
Pipe p41	100	200	3.96	0.13	0.10	0.025
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	-0.38	0.19	1.13	0.030
Pipe p46	7.6	200	41.75	1.33	8.01	0.018
Pipe p47	10	50	0.20	0.10	0.36	0.033
Pipe p48	78	50	-1.03	0.52	7.20	0.026
Pipe p49	20	50	-1.14	0.58	8.76	0.025
Pipe p50	44	150	-0.25	0.01	0.00	0.035
Pipe p51	74	250	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	3.34	0.11	0.07	0.026
Pipe p53	5.3	200	-3.14	0.10	0.07	0.026
Pipe p54	237.6	200	-6.76	0.22	0.28	0.023
Pipe p55	190.9	200	-9.32	0.30	0.50	0.022
Pipe p56	1.84	150	3.73	0.21	0.37	0.024
Pipe p57	193.5	150	-6.80	0.38	1.13	0.022
Pipe p58	21.1	200	-78.54	2.50	25.81	0.016
Pipe p59	36.1	200	-57.41	1.83	14.45	0.017
Pipe p60	10	150	-21.13	1.20	9.21	0.019
Pipe p61	131.7	150	-27.97	1.58	15.48	0.018
Pipe p62	79	100	0.47	0.06	0.06	0.032
Pipe p63	127.9	50	0.45	0.23	1.56	0.029
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	250	101.96	2.08	14.11	0.016
Pipe p66	54.1	150	0.10	0.01	0.00	0.044
Pipe p67	44	150	0.06	0.00	0.00	0.000
Pipe p68	65	200	-19.02	0.61	1.87	0.020
Pipe p69	8.8	200	-19.07	0.61	1.88	0.020
Pipe p70	58.1	150	0.08	0.00	0.00	0.048

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p70	58.1	150	0.08	0.00	0.00	0.048
Pipe p71	107.2	200	-28.65	0.91	3.99	0.019
Pipe p72	81.3	150	25.03	1.42	12.60	0.019
Pipe p73	18.7	200	-66.77	2.13	19.11	0.017
Pipe p74	33.6	200	6.66	0.21	0.27	0.023
Pipe p75	8.2	200	6.58	0.21	0.26	0.023
Pipe p76	4.1	200	3.62	0.12	0.09	0.025
Pipe p77	8	50	2.96	1.51	50.94	0.022
Pipe p78	21.8	200	7.50	0.24	0.33	0.023
Pipe p79	32.1	150	0.08	0.00	0.00	0.043
Pipe p80	32.5	150	0.08	0.00	0.00	0.043
Pipe p81	28.8	250	3.73	0.08	0.03	0.026
Pipe p82	28.2	200	-74.28	2.36	23.28	0.016
Pipe p83	128.8	50	-0.67	0.34	3.23	0.027
Pipe p84	7.8	50	-0.69	0.35	3.44	0.027
Pipe p85	33.5	150	0.07	0.00	0.00	0.057
Pipe p86	16.3	200	6.81	0.22	0.28	0.023
Pipe p87	70.7	200	6.74	0.21	0.27	0.023
Pipe p88	67.5	200	-90.94	2.89	33.86	0.016
Pipe p89	30.4	250	-98.33	2.00	13.20	0.016
Pipe p90	21	250	7.39	0.15	0.11	0.024
Pipe p91	115	250	7.39	0.15	0.11	0.024
Pipe p92	93	200	-44.10	1.40	8.86	0.018
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.040
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	3.52	0.45	2.40	0.023
Pipe p102	65.2	100	-0.11	0.01	0.00	0.040
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	202.04	0.00	-7.89	0.000

Large Apartment B Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.88 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 202.05 L/s with a head gain of 7.88 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer’s pump curve are to be confirmed during detailed design.

An FW5 classification has been adopted for the Care Centre fire scenario.

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
355	250	No	Fail
400	250	No	Fail
450	250	No	Fail
250	200	Yes	Pass
315	200	Yes	Pass
315	250	Yes	Pass

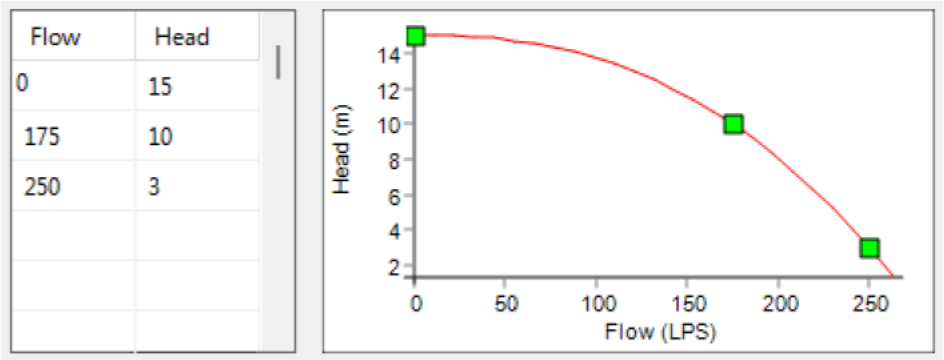


Figure 1: Large Apartment B Fire Scenario Pump Curve

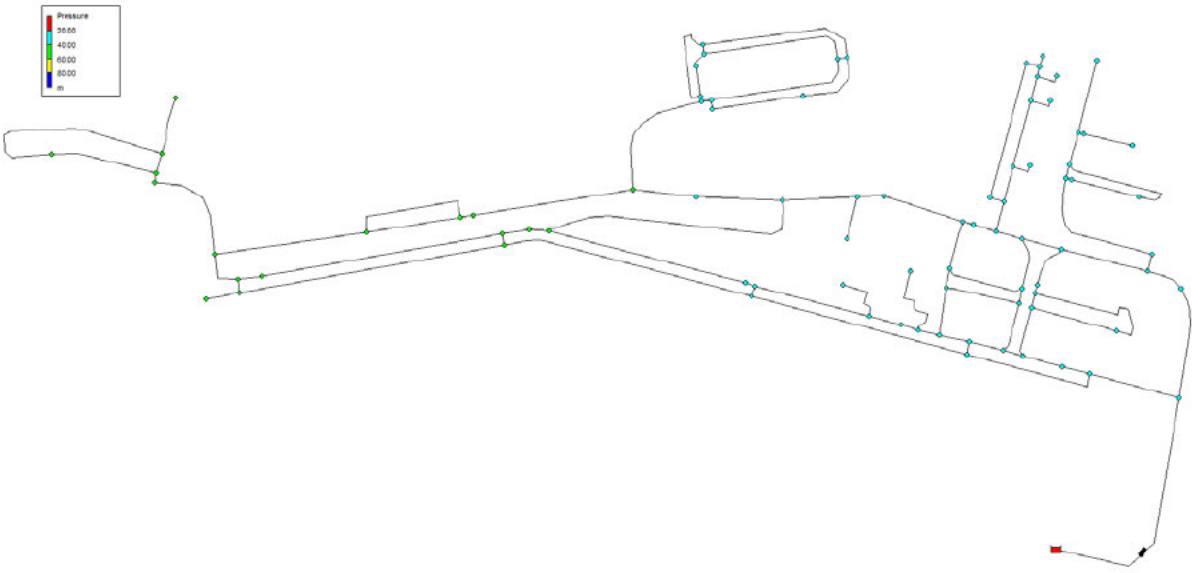


Figure 3: Large Apartment B Fire Scenario Network Layout

Large Apartment B Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	99.86	49.96
Junc n2	50.2	0.07	98.87	48.67
Junc n3	56.9	0	98.87	41.97
Junc n4	48.8	12.5	98.83	50.03
Junc n6	69.66	0	100.96	31.30
Junc n7	69.3	0	100.61	31.31
Junc n8	73.4	0.01	103.77	30.37
Junc n9	72.2	12.5	103.21	31.01
Junc n10	72.9	0	102.84	29.94
Junc n11	69.2	0	101.74	32.54
Junc n12	69.7	0	101.74	32.04
Junc n13	72.2	0.05	102.33	30.13
Junc n14	70.2	0	101.52	31.32
Junc n15	72.2	0	103.17	30.97
Junc n16	72.2	0.04	103.10	30.90
Junc n17	57.6	0	99.99	42.39
Junc n18	60.6	0.14	100.47	39.87
Junc n19	54.7	0	99.89	45.19
Junc n20	59.9	0.04	98.35	38.45
Junc n21	64.2	12.5	98.21	34.01
Junc n22	63.7	0.1	98.64	34.94
Junc n23	60.2	0.05	98.71	38.51
Junc n24	65.3	0	100.14	34.84
Junc n25	65.8	0.15	101.89	36.09
Junc n26	56.97	0.13	99.97	43.00
Junc n27	57.2	0.17	100.01	42.81
Junc n28	51.6	0.04	99.87	48.27
Junc n29	50.2	0.04	99.87	49.67
Junc n30	53.2	0.04	99.86	46.66
Junc n31	53.7	0.07	99.86	46.16
Junc n32	75.2	0	105.82	30.62
Junc n33	75.7	0	105.80	30.10
Junc n34	74.7	0.13	105.82	31.12
Junc n35	51.4	0.02	99.87	48.47

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n36	57.1	12.5	99.98	42.88
Junc n37	65.2	86.3	100.12	34.92
Junc n38	69.6	86.3	100.74	31.14
Junc n39	69.7	0	101.51	31.81
Junc n40	69.97	0.03	102.56	32.59
Junc n41	70.06	0	102.87	32.81
Junc n42	71.2	86.3	103.33	32.13
Junc n43	72.97	0.07	103.99	31.02
Junc n44	71.2	0.03	107.67	36.47
Junc n45	53.7	12.5	99.86	46.16
Junc n46	60.2	86.3	99.95	39.75
Junc n47	64.2	12.5	100.98	36.78
Junc n48	69.2	86.3	101.81	32.61
Junc n50	48.2	0.06	98.87	50.67
Junc n51	60.2	0	98.72	38.52
Junc n52	63.2	0.21	98.64	35.44
Junc n53	63.2	12.5	98.64	35.44
Junc n54	71.7	0.05	102.24	30.54
Junc n55	47.2	12.5	98.89	51.69
Junc n56	74.39	0	105.81	31.42
Junc n57	75.19	0	105.81	30.62
Junc n58	51.63	12.5	99.87	48.24
Junc n59	70.47	12.5	101.72	31.25
Junc n60	75.32	12.5	105.81	30.49
Junc n61	69.83	0.02	102.04	32.21
Junc n62	70.05	0.13	102.12	32.07
Junc n63	76.1	0.07	103.10	27.00
Junc n64	80.52	12.5	106.20	25.68
Junc n65	77.95	0.36	105.81	27.86
Junc n66	62.93	0	100.92	37.99
Junc n67	61.45	0.2	100.92	39.47
Junc n68	68.66	0.27	100.61	31.95
Junc n69	66.1	86.3	99.93	33.83
Junc n70	71.32	0.03	102.17	30.85

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n54	71.7	0.05	102.24	30.54
Junc n55	47.2	12.5	98.89	51.69
Junc n56	74.39	0	105.81	31.42
Junc n57	75.19	0	105.81	30.62
Junc n58	51.63	12.5	99.87	48.24
Junc n59	70.47	12.5	101.72	31.25
Junc n60	75.32	12.5	105.81	30.49
Junc n61	69.83	0.02	102.04	32.21
Junc n62	70.05	0.13	102.12	32.07
Junc n63	76.1	0.07	103.10	27.00
Junc n64	80.52	12.5	106.20	25.68
Junc n65	77.95	0.36	105.81	27.86
Junc n66	62.93	0	100.92	37.99
Junc n67	61.45	0.2	100.92	39.47
Junc n68	68.66	0.27	100.61	31.95
Junc n69	66.1	86.3	99.93	33.83
Junc n70	71.32	0.03	102.17	30.85
Junc n71	70.69	0	102.13	31.44
Junc n72	70.61	0	102.13	31.52
Junc n73	70.42	12.5	102.13	31.71
Junc n74	71.02	0	102.14	31.12
Junc n75	70.75	0	102.17	31.42
Junc n76	74.74	0.27	102.13	27.39
Junc n77	75.11	0.27	102.14	27.03
Junc n78	70.63	0.08	102.14	31.51
Junc n79	70.36	0	102.16	31.80
Junc n80	70.96	0.23	102.16	31.20
Junc 30	79.23	0.02	105.83	26.60
Junc 31	79.23	0	105.83	26.60
Junc 32	67.99	0.08	98.64	30.65
Junc 33	68.15	0	98.64	30.49
Junc 34	60.28	0.08	98.70	38.42
Junc 2	74.82	12.5	105.80	30.98
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	15.05	1.92	35.42	0.019
Pipe p4	44.19	100	-2.35	0.30	1.14	0.025
Pipe p5	71.43	50	-1.21	0.62	9.75	0.025
Pipe p6	236.7	50	-0.32	0.16	0.83	0.031
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-5.69	2.90	171.10	0.020
Pipe p9	9.962	50	-0.79	0.40	4.42	0.027
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	-0.09	0.05	0.07	0.035
Pipe p12	109.7	50	0.11	0.06	0.11	0.036
Pipe p13	24.49	200	-6.01	0.19	0.22	0.024
Pipe p14	17.45	200	-9.64	0.31	0.53	0.022
Pipe p15	2	200	-34.34	1.09	5.58	0.018
Pipe p16	29.99	200	-28.73	0.91	4.01	0.019
Pipe p17	15.84	200	-53.76	1.71	12.79	0.017
Pipe p18	20.29	200	-78.78	2.51	25.96	0.016
Pipe p19	20.52	200	-58.45	1.86	14.93	0.017
Pipe p20	36.49	200	-68.46	2.18	20.01	0.017
Pipe p21	26.03	200	-72.08	2.29	22.02	0.016
Pipe p22	83.26	200	-100.07	3.19	40.43	0.016
Pipe p23	83.3	200	-3.88	0.12	0.10	0.025
Pipe p24	11.53	200	-3.99	0.13	0.10	0.025
Pipe p25	146.4	200	-7.62	0.24	0.34	0.023
Pipe p26	57.48	200	-15.03	0.48	1.21	0.021
Pipe p27	77.36	200	-18.66	0.59	1.80	0.020
Pipe p28	37.04	200	-75.88	2.42	24.22	0.016
Pipe p29	12.57	250	3.84	0.08	0.03	0.026
Pipe p30	242.8	50	-0.23	0.12	0.44	0.032
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.033
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	-10.01	1.27	16.64	0.020

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p35	13.41	100	-9.53	1.21	15.19	0.020
Pipe p36	7.537	100	-9.98	1.27	16.54	0.020
Pipe p37	230.3	50	-1.07	0.54	7.69	0.026
Pipe p38	2.15	100	-0.14	0.02	0.00	0.027
Pipe p39	20	50	-1.60	0.82	16.39	0.024
Pipe p40	40	50	-1.39	0.71	12.61	0.025
Pipe p41	100	200	3.96	0.13	0.10	0.025
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	-0.38	0.19	1.13	0.030
Pipe p46	7.6	200	41.75	1.33	8.01	0.018
Pipe p47	10	50	0.20	0.10	0.36	0.033
Pipe p48	78	50	-1.03	0.52	7.20	0.026
Pipe p49	20	50	-1.14	0.58	8.76	0.025
Pipe p50	44	150	-0.25	0.01	0.00	0.035
Pipe p51	74	250	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	3.34	0.11	0.07	0.026
Pipe p53	5.3	200	-3.14	0.10	0.07	0.026
Pipe p54	237.6	200	-6.76	0.22	0.28	0.023
Pipe p55	190.9	200	-9.32	0.30	0.50	0.022
Pipe p56	1.84	150	3.73	0.21	0.37	0.024
Pipe p57	193.5	150	-6.80	0.38	1.13	0.022
Pipe p58	21.1	200	-78.54	2.50	25.81	0.016
Pipe p59	36.1	200	-57.41	1.83	14.45	0.017
Pipe p60	10	150	-21.13	1.20	9.21	0.019
Pipe p61	131.7	150	-27.97	1.58	15.48	0.018
Pipe p62	79	100	0.47	0.06	0.06	0.032
Pipe p63	127.9	50	0.45	0.23	1.56	0.029
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	250	101.96	2.08	14.11	0.016
Pipe p66	54.1	150	0.10	0.01	0.00	0.044
Pipe p67	44	150	0.06	0.00	0.00	0.000
Pipe p68	65	200	-19.02	0.61	1.87	0.020

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p69	8.8	200	-19.07	0.61	1.88	0.020
Pipe p70	58.1	150	0.08	0.00	0.00	0.048
Pipe p71	107.2	200	-28.65	0.91	3.99	0.019
Pipe p72	81.3	150	25.03	1.42	12.60	0.019
Pipe p73	18.7	200	-66.77	2.13	19.11	0.017
Pipe p74	33.6	200	6.66	0.21	0.27	0.023
Pipe p75	8.2	200	6.58	0.21	0.26	0.023
Pipe p76	4.1	200	3.62	0.12	0.09	0.025
Pipe p77	8	50	2.96	1.51	50.94	0.022
Pipe p78	21.8	200	7.50	0.24	0.33	0.023
Pipe p79	32.1	150	0.08	0.00	0.00	0.043
Pipe p80	32.5	150	0.08	0.00	0.00	0.043
Pipe p81	28.8	250	3.73	0.08	0.03	0.026
Pipe p82	28.2	200	-74.28	2.36	23.28	0.016
Pipe p83	128.8	50	-0.67	0.34	3.23	0.027
Pipe p84	7.8	50	-0.69	0.35	3.44	0.027
Pipe p85	33.5	150	0.07	0.00	0.00	0.057
Pipe p86	16.3	200	6.81	0.22	0.28	0.023
Pipe p87	70.7	200	6.74	0.21	0.27	0.023
Pipe p88	67.5	200	-90.94	2.89	33.86	0.016
Pipe p89	30.4	250	-98.33	2.00	13.20	0.016
Pipe p90	21	250	7.39	0.15	0.11	0.024
Pipe p91	115	250	7.39	0.15	0.11	0.024
Pipe p92	93	200	-44.10	1.40	8.86	0.018
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.040
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	3.52	0.45	2.40	0.023
Pipe p102	65.2	100	-0.11	0.01	0.00	0.040
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	202.04	0.00	-7.89	0.000

Large Apartment C Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.88 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 202.04 L/s with a head gain of 7.89 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer’s pump curve are to be confirmed during detailed design.

An FW5 classification has been adopted for the Care Centre fire scenario.

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
355	250	No	Fail
400	250	No	Fail
450	250	No	Fail
250	200	Yes	Pass
315	200	Yes	Pass
315	250	Yes	Pass

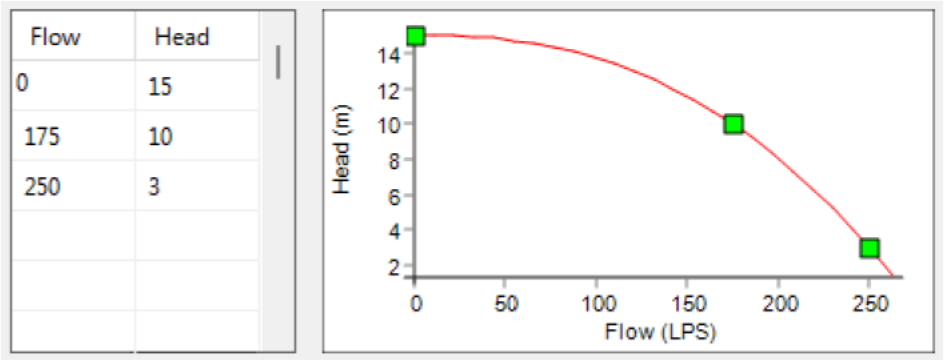


Figure 1: Large Apartment C Fire Scenario Pump Curve

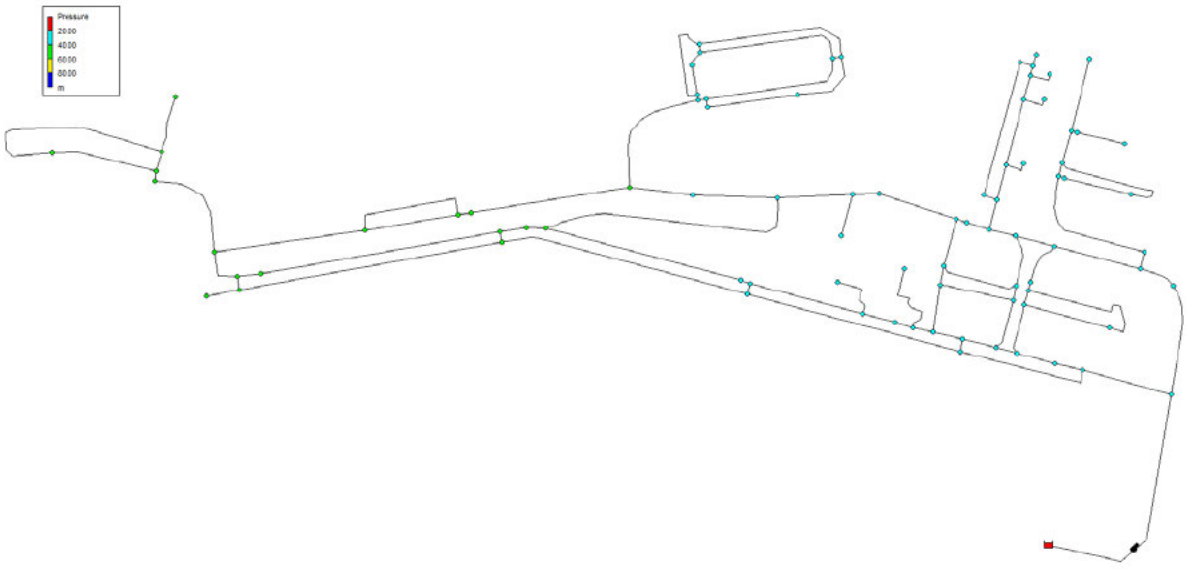


Figure 3: Large Apartment C Fire Scenario Network Layout

Large Apartment C Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	98.59	48.69
Junc n2	50.2	0.07	97.60	47.40
Junc n3	56.9	0	97.60	40.70
Junc n4	48.8	12.5	97.56	48.76
Junc n6	69.66	0	100.81	31.15
Junc n7	69.3	0	99.87	30.57
Junc n8	73.4	0.01	103.29	29.89
Junc n9	72.2	12.5	103.14	30.94
Junc n10	72.9	0	102.12	29.22
Junc n11	69.2	0	100.67	31.47
Junc n12	69.7	0	100.80	31.10
Junc n13	72.2	0.05	101.84	29.64
Junc n14	70.2	0	101.42	31.22
Junc n15	72.2	0	103.14	30.94
Junc n16	72.2	0.04	103.14	30.94
Junc n17	57.6	0	98.94	41.34
Junc n18	60.6	0.14	98.97	38.37
Junc n19	54.7	0	98.57	43.87
Junc n20	59.9	0.04	97.03	37.13
Junc n21	64.2	12.5	96.89	32.69
Junc n22	63.7	0.1	97.31	33.61
Junc n23	60.2	0.05	97.38	37.18
Junc n24	65.3	0	99.25	33.95
Junc n25	65.8	0.15	101.82	36.02
Junc n26	56.97	0.13	98.89	41.92
Junc n27	57.2	0.17	98.95	41.75
Junc n28	51.6	0.04	98.63	47.03
Junc n29	50.2	0.04	98.63	48.43
Junc n30	53.2	0.04	98.58	45.38
Junc n31	53.7	0.07	98.57	44.87
Junc n32	75.2	0	105.60	30.40
Junc n33	75.7	0	105.59	29.89
Junc n34	74.7	0.13	105.60	30.90
Junc n35	51.4	0.02	98.61	47.21
Junc n36	57.1	12.5	98.91	41.81
Junc n37	65.2	86.3	99.23	34.03
Junc n38	69.6	86.3	100.34	30.74
Junc n39	69.7	0	101.42	31.72
Junc n40	69.97	0.03	102.74	32.77
Junc n41	70.06	0	103.13	33.07
Junc n42	71.2	12.5	103.82	32.62
Junc n43	72.97	0.07	104.36	31.39
Junc n44	71.2	0.03	107.67	36.47
Junc n45	53.7	12.5	98.57	44.87
Junc n46	60.2	86.3	98.59	38.39
Junc n47	64.2	86.3	99.33	35.13
Junc n48	69.2	86.3	100.77	31.57
Junc n50	48.2	0.06	97.60	49.40
Junc n51	60.2	0	97.39	37.19
Junc n52	63.2	0.21	97.31	34.11
Junc n53	63.2	12.5	97.31	34.11
Junc n54	71.7	0.05	101.84	30.14
Junc n55	47.2	12.5	97.62	50.42
Junc n56	74.39	0	105.60	31.21
Junc n57	75.19	0	105.60	30.41
Junc n58	51.63	12.5	98.62	46.99
Junc n59	70.47	12.5	100.81	30.34
Junc n60	75.32	12.5	105.60	30.28
Junc n61	69.83	0.02	102.07	32.24
Junc n62	70.05	0.13	102.16	32.11
Junc n63	76.1	0.07	103.14	27.04
Junc n64	80.52	12.5	106.03	25.51
Junc n65	77.95	0.36	105.60	27.65
Junc n66	62.93	0	99.29	36.36
Junc n67	61.45	0.2	99.29	37.84
Junc n68	68.66	86.3	99.14	30.48
Junc n69	66.1	0.27	100.81	34.71
Junc n70	71.32	0.03	101.25	29.93

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n54	71.7	0.05	101.84	30.14
Junc n55	47.2	12.5	97.62	50.42
Junc n56	74.39	0	105.60	31.21
Junc n57	75.19	0	105.60	30.41
Junc n58	51.63	12.5	98.62	46.99
Junc n59	70.47	12.5	100.81	30.34
Junc n60	75.32	12.5	105.60	30.28
Junc n61	69.83	0.02	102.07	32.24
Junc n62	70.05	0.13	102.16	32.11
Junc n63	76.1	0.07	103.14	27.04
Junc n64	80.52	12.5	106.03	25.51
Junc n65	77.95	0.36	105.60	27.65
Junc n66	62.93	0	99.29	36.36
Junc n67	61.45	0.2	99.29	37.84
Junc n68	68.66	86.3	99.14	30.48
Junc n69	66.1	0.27	100.81	34.71
Junc n70	71.32	0.03	101.25	29.93
Junc n71	70.69	0	101.21	30.52
Junc n72	70.61	0	101.21	30.60
Junc n73	70.42	12.5	101.21	30.79
Junc n74	71.02	0	101.22	30.20
Junc n75	70.75	0	101.25	30.50
Junc n76	74.74	0.27	101.21	26.47
Junc n77	75.11	0.27	101.22	26.11
Junc n78	70.63	0.08	101.22	30.59
Junc n79	70.36	0	101.24	30.88
Junc n80	70.96	0.23	101.24	30.28
Junc 30	79.23	0.02	105.62	26.39
Junc 31	79.23	0	105.62	26.39
Junc 32	67.99	0.08	97.32	29.33
Junc 33	68.15	0	97.32	29.17
Junc 34	60.28	0.08	97.38	37.10
Junc 2	74.82	12.5	105.59	30.77
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	4.35	0.55	3.56	0.023
Pipe p4	44.19	100	-3.78	0.48	2.74	0.023
Pipe p5	71.43	50	-1.51	0.77	14.61	0.024
Pipe p6	236.7	50	-0.11	0.06	0.12	0.036
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-7.13	3.63	259.87	0.019
Pipe p9	9.962	50	-0.92	0.47	5.92	0.026
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	0.09	0.04	0.07	0.037
Pipe p12	109.7	50	0.11	0.06	0.11	0.036
Pipe p13	24.49	200	-13.64	0.43	1.01	0.021
Pipe p14	17.45	200	-17.26	0.55	1.56	0.020
Pipe p15	2	200	-42.17	1.34	8.16	0.018
Pipe p16	29.99	200	-60.07	1.91	15.71	0.017
Pipe p17	15.84	200	-85.10	2.71	29.94	0.016
Pipe p18	20.29	200	-85.17	2.71	29.99	0.016
Pipe p19	20.52	200	-66.96	2.13	19.21	0.017
Pipe p20	36.49	200	-66.27	2.11	18.84	0.017
Pipe p21	26.03	200	-69.89	2.22	20.80	0.016
Pipe p22	83.26	200	-99.08	3.15	39.69	0.016
Pipe p23	83.3	200	3.89	0.12	0.10	0.025
Pipe p24	11.53	200	3.95	0.13	0.10	0.025
Pipe p25	146.4	200	0.33	0.01	0.00	0.037
Pipe p26	57.48	200	-7.09	0.23	0.30	0.023
Pipe p27	77.36	200	-32.11	1.02	4.93	0.019
Pipe p28	37.04	200	-87.57	2.79	31.58	0.016
Pipe p29	12.57	250	3.84	0.08	0.03	0.026
Pipe p30	242.8	50	-0.41	0.21	1.32	0.029
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.033
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	0.69	0.09	0.12	0.030

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p35	13.41	100	0.66	0.08	0.11	0.030
Pipe p36	7.537	100	0.73	0.09	0.13	0.030
Pipe p37	230.3	50	-1.39	0.71	12.50	0.025
Pipe p38	2.15	100	-0.14	0.02	0.01	0.054
Pipe p39	20	50	-1.47	0.75	13.92	0.024
Pipe p40	40	50	-1.26	0.64	10.45	0.025
Pipe p41	100	200	-3.98	0.13	0.10	0.025
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	0.05	0.03	0.03	0.040
Pipe p46	7.6	200	53.57	1.71	12.71	0.017
Pipe p47	10	50	0.39	0.20	1.18	0.030
Pipe p48	78	50	-1.33	0.68	11.52	0.025
Pipe p49	20	50	-2.27	1.16	31.29	0.023
Pipe p50	44	150	1.01	0.06	0.03	0.030
Pipe p51	74	250	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	11.28	0.36	0.71	0.022
Pipe p53	5.3	200	-10.90	0.35	0.67	0.022
Pipe p54	237.6	200	-14.52	0.46	1.13	0.021
Pipe p55	190.9	200	-17.15	0.55	1.54	0.020
Pipe p56	1.84	150	3.73	0.21	0.37	0.025
Pipe p57	193.5	150	-8.56	0.48	1.73	0.022
Pipe p58	21.1	200	-86.19	2.74	30.66	0.016
Pipe p59	36.1	200	-65.63	2.09	18.51	0.017
Pipe p60	10	150	-20.57	1.16	8.76	0.019
Pipe p61	131.7	150	-29.16	1.65	16.73	0.018
Pipe p62	79	100	-0.04	0.01	0.00	0.034
Pipe p63	127.9	50	-0.06	0.03	0.01	0.011
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	250	102.95	2.10	14.37	0.016
Pipe p66	54.1	150	0.10	0.01	0.00	0.029
Pipe p67	44	150	0.06	0.00	0.00	0.000
Pipe p68	65	200	-32.27	1.03	4.97	0.018

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p69	8.8	200	-32.33	1.03	4.99	0.018
Pipe p70	58.1	150	25.03	1.42	12.60	0.019
Pipe p71	107.2	200	-35.04	1.12	5.79	0.018
Pipe p72	81.3	150	0.08	0.00	0.00	0.034
Pipe p73	18.7	200	-78.60	2.50	25.85	0.016
Pipe p74	33.6	200	6.66	0.21	0.27	0.023
Pipe p75	8.2	200	6.58	0.21	0.26	0.023
Pipe p76	4.1	200	3.62	0.12	0.09	0.025
Pipe p77	8	50	2.96	1.51	50.94	0.022
Pipe p78	21.8	200	7.50	0.24	0.33	0.023
Pipe p79	32.1	150	0.08	0.00	0.00	0.043
Pipe p80	32.5	150	0.08	0.00	0.00	0.043
Pipe p81	28.8	250	3.73	0.08	0.03	0.026
Pipe p82	28.2	200	-86.10	2.74	30.60	0.016
Pipe p83	128.8	50	-0.67	0.34	3.23	0.027
Pipe p84	7.8	50	-0.69	0.35	3.44	0.027
Pipe p85	33.5	150	0.07	0.00	0.00	0.057
Pipe p86	16.3	200	6.81	0.22	0.28	0.023
Pipe p87	70.7	200	6.74	0.21	0.27	0.023
Pipe p88	67.5	200	-91.93	2.93	34.54	0.016
Pipe p89	30.4	250	-99.32	2.02	13.45	0.016
Pipe p90	21	250	7.39	0.15	0.11	0.024
Pipe p91	115	250	7.39	0.15	0.11	0.024
Pipe p92	93	200	-57.35	1.83	14.42	0.017
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.042
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	3.52	0.45	2.40	0.023
Pipe p102	65.2	100	-0.11	0.01	0.00	0.040
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	202.04	0.00	-7.89	0.000

Recreation Facility Fire Scenario

The total lot demand calculated based on the development area is 4.17 L/s.

The total lot demand applied in the fire flow model is 3.81 L/s. Where a node represents both a lot supply point and a fire hydrant location, the lot demand at that node has not been applied in addition to the fire hydrant demand. This approach has been adopted to avoid double-counting demand at fire hydrant nodes during the firefighting scenario.

If required, additional modelling can be undertaken with the lot demand applied concurrently with the fire hydrant demand at the relevant nodes.

A booster pump is required at the connection point to satisfy the minimum pressure requirements. Based on the current fire scenario, the modelled pump operating point is approximately 202.06 L/s with a head gain of 7.88 m.

The pump curve adopted in the model is preliminary and has been used for hydraulic modelling purposes only. Final pump selection and the corresponding manufacturer’s pump curve are to be confirmed during detailed design.

An FW5 classification has been adopted for the Care Centre fire scenario.

Existing connection pressure head = 30m

Trunk Main	Distribution Main	Booster Pump	Pass/Fail
250	200	Yes	Pass
315	200	Yes	Pass
315	250	Yes	Pass

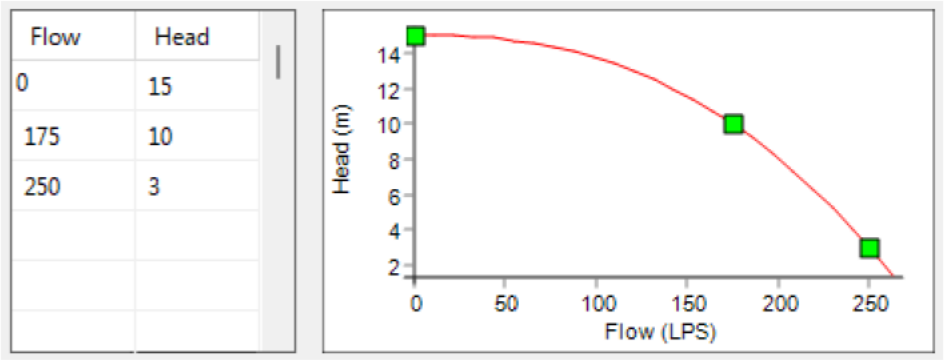


Figure 1: Recreation Facility Fire Scenario Pump Curve

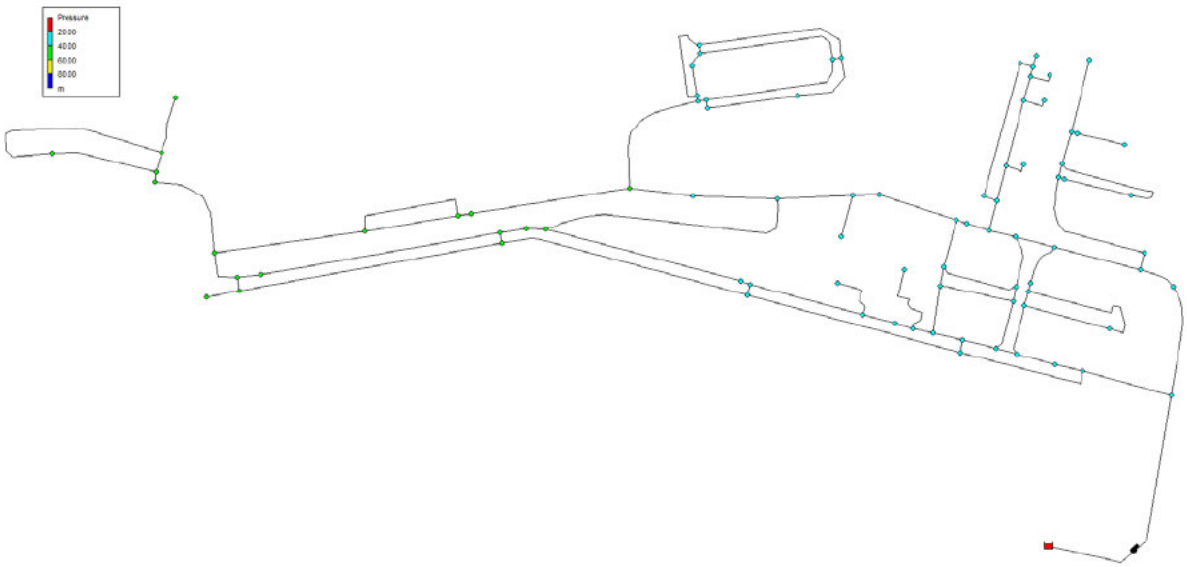


Figure 3: Recreation Facility Fire Scenario Network Layout

Recreation Facility Fire Scenario Model Results

Junctions (See attached water modelling map)

Water pressure head at supply point = Elevation + 30m (300kPa)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n1	49.9	0.03	97.89	47.99
Junc n2	50.2	0.07	96.90	46.70
Junc n3	56.9	0	96.90	40.00
Junc n4	48.8	12.5	96.87	48.07
Junc n6	69.66	0	101.52	31.86
Junc n7	69.3	0	100.86	31.56
Junc n8	73.4	0.01	103.03	29.63
Junc n9	72.2	12.5	103.05	30.85
Junc n10	72.9	0	101.63	28.73
Junc n11	69.2	0	99.83	30.63
Junc n12	69.7	0	100.10	30.40
Junc n13	72.2	0.05	101.63	29.43
Junc n14	70.2	0	101.95	31.75
Junc n15	72.2	0	103.08	30.88
Junc n16	72.2	0.04	103.14	30.94
Junc n17	57.6	0	98.93	41.33
Junc n18	60.6	0.14	97.69	37.09
Junc n19	54.7	0	97.53	42.83
Junc n20	59.9	0.04	95.99	36.09
Junc n21	64.2	12.5	95.85	31.65
Junc n22	63.7	0.1	96.27	32.57
Junc n23	60.2	0.05	96.35	36.15
Junc n24	65.3	0	99.76	34.46
Junc n25	65.8	0.15	102.24	36.44
Junc n26	56.97	0.13	98.78	41.81
Junc n27	57.2	0.17	98.82	41.62
Junc n28	51.6	0.04	98.02	46.42
Junc n29	50.2	0.04	98.02	47.82
Junc n30	53.2	0.04	97.76	44.56
Junc n31	53.7	0.07	97.66	43.96
Junc n32	75.2	0	105.49	30.29
Junc n33	75.7	0	105.48	29.78
Junc n34	74.7	0.13	105.49	30.79
Junc n35	51.4	0.02	97.99	46.59
Junc n36	57.1	12.5	98.86	41.76
Junc n37	65.2	86.3	99.74	34.53
Junc n38	69.6	86.3	101.16	31.56
Junc n39	69.7	0	101.97	32.27
Junc n40	69.97	0.03	103.04	33.07
Junc n41	70.06	0	103.36	33.30
Junc n42	71.2	12.5	104.02	32.82
Junc n43	72.97	0.07	104.54	31.57
Junc n44	71.2	0.03	107.66	36.46
Junc n45	53.7	12.5	97.65	43.95
Junc n46	60.2	86.3	97.52	37.32
Junc n47	64.2	86.3	97.90	33.70
Junc n48	69.2	86.3	99.96	30.76
Junc n50	48.2	0.06	96.90	48.70
Junc n51	60.2	0	96.36	36.16
Junc n52	63.2	0.21	96.27	33.07
Junc n53	63.2	12.5	96.27	33.07
Junc n54	71.7	0.05	101.92	30.22
Junc n55	47.2	12.5	96.92	49.72
Junc n56	74.39	0	105.49	31.10
Junc n57	75.19	0	105.49	30.30
Junc n58	51.63	12.5	98.00	46.37
Junc n59	70.47	12.5	100.12	29.65
Junc n60	75.32	12.5	105.49	30.17
Junc n61	69.83	0.02	102.50	32.67
Junc n62	70.05	0.13	102.58	32.53
Junc n63	76.1	0.07	103.14	27.04
Junc n64	80.52	12.5	105.94	25.42
Junc n65	77.95	0.36	105.49	27.54
Junc n66	62.93	0	97.82	34.89
Junc n67	61.45	86.3	97.26	35.81
Junc n68	68.66	0.27	100.86	32.20
Junc n69	66.1	0.27	101.52	35.42
Junc n70	71.32	0.03	100.57	29.25

Junctions (See attached water modelling map)

Node ID	Elevation m	Base Demand LPS	Head m	Pressure m
Junc n54	71.7	0.05	101.92	30.22
Junc n55	47.2	12.5	96.92	49.72
Junc n56	74.39	0	105.49	31.10
Junc n57	75.19	0	105.49	30.30
Junc n58	51.63	12.5	98.00	46.37
Junc n59	70.47	12.5	100.12	29.65
Junc n60	75.32	12.5	105.49	30.17
Junc n61	69.83	0.02	102.50	32.67
Junc n62	70.05	0.13	102.58	32.53
Junc n63	76.1	0.07	103.14	27.04
Junc n64	80.52	12.5	105.94	25.42
Junc n65	77.95	0.36	105.49	27.54
Junc n66	62.93	0	97.82	34.89
Junc n67	61.45	86.3	97.26	35.81
Junc n68	68.66	0.27	100.86	32.20
Junc n69	66.1	0.27	101.52	35.42
Junc n70	71.32	0.03	100.57	29.25
Junc n71	70.69	0	100.52	29.83
Junc n72	70.61	0	100.52	29.91
Junc n73	70.42	12.5	100.52	30.10
Junc n74	71.02	0	100.53	29.51
Junc n75	70.75	0	100.56	29.81
Junc n76	74.74	0.27	100.52	25.78
Junc n77	75.11	0.27	100.53	25.42
Junc n78	70.63	0.08	100.53	29.90
Junc n79	70.36	0	100.55	30.19
Junc n80	70.96	0.23	100.55	29.59
Junc 30	79.23	0.02	105.50	26.27
Junc 31	79.23	0	105.50	26.27
Junc 32	67.99	0.08	96.28	28.29
Junc 33	68.15	0	96.28	28.13
Junc 34	60.28	0.08	96.34	36.06
Junc 2	74.82	12.5	105.48	30.66
Resvr 1	99.78	#N/A	99.78	0.00

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p1	63.11	50	0.00	0.00	0.00	0.000
Pipe p2	205.4	50	0.13	0.07	0.16	0.035
Pipe p3	41.3	100	-1.46	0.19	0.47	0.027
Pipe p4	44.19	100	-5.93	0.75	6.31	0.022
Pipe p5	71.43	50	-1.85	0.94	21.31	0.024
Pipe p6	236.7	50	0.87	0.44	5.23	0.026
Pipe p7	85.87	50	-0.37	0.19	1.11	0.030
Pipe p8	9.908	50	-6.99	3.56	250.39	0.019
Pipe p9	9.962	50	-0.76	0.39	3.93	0.025
Pipe p10	30.53	50	-0.01	0.01	0.00	0.051
Pipe p11	113	50	0.34	0.17	0.91	0.030
Pipe p12	109.7	50	0.11	0.06	0.11	0.036
Pipe p13	24.49	200	-25.10	0.80	3.12	0.019
Pipe p14	17.45	200	-28.72	0.91	4.01	0.019
Pipe p15	2	200	-54.62	1.74	13.17	0.017
Pipe p16	29.99	200	-47.71	1.52	10.25	0.017
Pipe p17	15.84	200	-72.73	2.32	22.39	0.016
Pipe p18	20.29	200	-72.81	2.32	22.43	0.016
Pipe p19	20.52	200	-59.85	1.90	15.60	0.017
Pipe p20	36.49	200	-64.96	2.07	18.16	0.017
Pipe p21	26.03	200	-68.59	2.18	20.08	0.017
Pipe p22	83.26	200	-96.05	3.06	37.47	0.016
Pipe p23	83.3	200	15.23	0.48	1.24	0.021
Pipe p24	11.53	200	15.55	0.50	1.29	0.021
Pipe p25	146.4	200	11.93	0.38	0.79	0.021
Pipe p26	57.48	200	4.51	0.14	0.13	0.025
Pipe p27	77.36	200	-20.51	0.65	2.15	0.020
Pipe p28	37.04	200	-96.43	3.07	37.75	0.016
Pipe p29	12.57	250	3.84	0.08	0.03	0.026
Pipe p30	242.8	50	-0.71	0.36	3.31	0.025
Pipe p31	5.142	100	3.88	0.49	2.87	0.023
Pipe p32	8.075	100	0.28	0.04	0.02	0.035
Pipe p33	40.4	100	3.49	0.44	2.36	0.024
Pipe p34	45.73	100	-5.12	0.65	4.80	0.022

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p35	13.41	100	-4.86	0.62	4.36	0.022
Pipe p36	7.537	100	-5.08	0.65	4.75	0.022
Pipe p37	230.3	50	-1.52	0.77	14.86	0.024
Pipe p38	2.15	100	-0.14	0.02	0.00	0.027
Pipe p39	20	50	-0.09	0.04	0.07	0.036
Pipe p40	40	50	0.30	0.15	0.73	0.031
Pipe p41	100	200	-15.58	0.50	1.29	0.021
Pipe p42	105	100	7.29	0.93	9.25	0.021
Pipe p43	8.2	100	3.66	0.47	2.59	0.023
Pipe p44	5	100	0.15	0.02	0.01	0.039
Pipe p45	15	50	1.78	0.90	19.78	0.024
Pipe p46	7.6	200	63.81	2.03	17.57	0.017
Pipe p47	10	50	0.68	0.35	3.09	0.025
Pipe p48	78	50	-1.49	0.76	14.31	0.024
Pipe p49	20	50	-4.08	2.08	92.42	0.021
Pipe p50	44	150	4.38	0.25	0.50	0.024
Pipe p51	74	250	0.00	0.00	0.00	0.000
Pipe p52	35.3	200	22.88	0.73	2.63	0.019
Pipe p53	5.3	200	-22.20	0.71	2.49	0.020
Pipe p54	237.6	200	-25.83	0.82	3.29	0.019
Pipe p55	190.9	200	-29.59	0.94	4.23	0.019
Pipe p56	1.84	150	3.73	0.21	0.37	0.024
Pipe p57	193.5	150	-8.55	0.48	1.73	0.022
Pipe p58	21.1	200	-77.19	2.46	24.99	0.016
Pipe p59	36.1	200	-58.35	1.86	14.89	0.017
Pipe p60	10	150	-18.85	1.07	7.46	0.019
Pipe p61	131.7	150	-27.44	1.55	14.95	0.018
Pipe p62	79	100	0.25	0.03	0.02	0.035
Pipe p63	127.9	50	0.23	0.12	0.45	0.032
Pipe p64	100	150	-3.49	0.20	0.33	0.025
Pipe p65	114	250	106.00	2.16	15.17	0.016
Pipe p66	54.1	150	0.10	0.01	0.00	0.044
Pipe p67	44	150	25.03	1.42	12.60	0.019
Pipe p68	65	200	-19.69	0.63	1.99	0.020

Pipes (See attached water modelling map

Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor
Pipe p69	8.8	200	-44.72	1.42	9.09	0.018
Pipe p70	58.1	150	0.08	0.00	0.00	0.048
Pipe p71	107.2	200	-47.63	1.52	10.22	0.017
Pipe p72	81.3	150	0.08	0.00	0.00	0.034
Pipe p73	18.7	200	-88.84	2.83	32.43	0.016
Pipe p74	33.6	200	6.66	0.21	0.27	0.023
Pipe p75	8.2	200	6.58	0.21	0.26	0.023
Pipe p76	4.1	200	3.62	0.12	0.09	0.025
Pipe p77	8	50	2.96	1.51	50.94	0.022
Pipe p78	21.8	200	7.50	0.24	0.33	0.023
Pipe p79	32.1	150	0.08	0.00	0.00	0.043
Pipe p80	32.5	150	0.08	0.00	0.00	0.043
Pipe p81	28.8	250	3.73	0.08	0.03	0.026
Pipe p82	28.2	200	-96.35	3.07	37.68	0.016
Pipe p83	128.8	50	-0.67	0.34	3.23	0.027
Pipe p84	7.8	50	-0.69	0.35	3.44	0.027
Pipe p85	33.5	150	0.07	0.00	0.00	0.000
Pipe p86	16.3	200	6.81	0.22	0.28	0.023
Pipe p87	70.7	200	6.74	0.21	0.27	0.023
Pipe p88	67.5	200	-94.98	3.02	36.70	0.016
Pipe p89	30.4	250	-102.37	2.09	14.22	0.016
Pipe p90	21	250	7.39	0.15	0.11	0.024
Pipe p91	115	250	7.39	0.15	0.11	0.024
Pipe p92	93	200	-69.74	2.22	20.71	0.016
Pipe p93	8	100	1.85	0.24	0.73	0.026
Pipe p94	137.8	100	0.08	0.01	0.00	0.040
Pipe p95	150.4	50	0.06	0.03	0.04	0.039
Pipe p96	72.7	50	1.91	0.98	22.76	0.023
Pipe p97	123.1	100	7.41	0.94	9.54	0.021
Pipe p98	8	50	1.72	0.88	18.70	0.024
Pipe p99	5.8	100	3.54	0.45	2.42	0.023
Pipe p100	140	100	1.79	0.23	0.69	0.026
Pipe p101	5.4	100	3.52	0.45	2.40	0.023
Pipe p102	65.2	100	-0.11	0.01	0.00	0.040
Pipe p103	87	50	1.71	0.87	18.47	0.024
Pump 6	#N/A	#N/A	202.06	0.00	-7.88	0.000

Appendix D – Wastewater Calculations

Maven Associates	Job Number	Sheet	Rev
	411001	1	A

Job Title	Harlow Retirement	Author	Date	Checked
Calc Title	Total retirement village demand	BD	25/09/2026	SM

As per site specific design

Care centre staff Daily Flow= 30 *l/person/day*

Domestic Average Daily Flow= 200 *l/person/day*

2-3 bed retirement dwellings =	**381.00**	*(2ppl/dwelling)*
1 Bed Apartment dwellings =	**33.00**	*(1ppl/dwelling)*
Care centre beds =	**100.00**	*1ppl/bed)*
Care centre staff =	**30.00**	*(0.20pp/dwelling equiv)*
Community Centre facilities =	**27.00**	*0.6Ha @ 45ppl/Ha*
Residential Population	**795.0**	
Care centre population equiv	**106.0**	
Total population equiv	**928.0**	
Wastewater Peaking factor as per Watercare standard=	**6.7**	*General residential*
	5.0	*Care centre only*
Weighted value=	**6.3**	

Using a population value person (max development scenario)

Average Daily Flow (ADF)= 185,600 *litres/day*

Peak Daily Flow (PDF)= 1,171,300 *litres/day*

= **13.56** *litres/sec*

Pipe Ks (uPVC) = 0.60

Peak Daily Flow	Pipe dia	Gradient	Capacity	Velocity	Check
l/s	*m*	*%*	*l/s*	*m/s*	*OK*
13.56	0.15	0.60	**13.90**	0.79	OK

Minimum Gradient used

Maven Associates	Job Number	Sheet	Rev
	411001	1	A

Job Title	Harlow Retirement	Author	Date	Checked
Calc Title	Public catchment / Stage 1 (A&B)	BD	25/09/2026	SM

As per site specific design

Care centre staff Daily Flow=	30	*l/person/day*
Domestic Average Daily Flow=	200	*l/person/day*

2-3 bed retirement dwellings =	**40.00**	*(2ppl/dwelling)*
1 Bed Apartment dwellings =	**0.00**	*(1ppl/dwelling)*
Care centre beds =	**100.00**	*1ppl/bed)*
Care centre staff =	**30.00**	*(0.2pp/dwelling equiv)*
Residential Population	**80.0**	
Care centre population equiv	**106.0**	
Total population equiv	**186.0**	
Wastewater Peaking factor as per Watercare standard=	**6.7**	*General residential*
	5.0	*Care centre only*
Weighted value=	**5.7**	

Using a population value person (max development scenario)

Average Daily Flow (ADF)=	37200	*litres/day*
Peak Daily Flow (PDF)=	213200	*litres/day*
=	**2.47**	*litres/sec*

Pipe Ks (uPVC) = 0.60

Peak Daily Flow	Pipe dia	Gradient	Capacity	Velocity	Check
l/s	*m*	*%*	*l/s*	*m/s*	*OK*
2.47	0.15	0.55	**13.31**	0.75	OK

Minimum Gradient used

Maven Associates		Job Number 411001	Sheet 1	Rev A
Job Title	Harlow Retirement	Author	Date	Checked
Calc Title	Private catchment / stage 2	BD	25/09/2026	SM

As per site specific design

Domestic Average Daily Flow= 200 *l/person/day*

2-3 bed retirement dwellings =	**341.00**	*(2ppl/dwelling)*
1 Bed Apartment dwellings =	**33.00**	*(1ppl/dwelling)*
Residential Population	**715.0**	
Community Centre facilities =	**27.00**	*0.6Ha @ 45ppl/Ha*
Total population equiv	**742.0**	
Wastewater Peaking factor as per RITS V2.0=	**6.7**	*General residential*

Using a population value person (max development scenario)

Average Daily Flow (ADF)= 148400 *litres/day*

Peak Daily Flow (PDF)= 994280 *litres/day*

= **11.51** *litres/sec*

Pipe Ks (uPVC) = 0.60

Peak Daily Flow *l/s*	Pipe dia *m*	Gradient *%*	Capacity *l/s*	Velocity *m/s*	Check *OK*
11.51	0.15	0.55	**13.31**	0.75	OK

Minimum Gradient used

Table 1: Design Flow Volumes

Accomodation Type	occupancy	Per capita flow (l/day)
2-Bed	2	200
1 bed apartment	1	200
Care Suites	1	200
Staff		30

Maven Associates	Job Number	Sheet	Rev
	411001	1	A

Job Title	Harlow Retirement	Author	Date	Checked
Calc Title	Public catchment / Stage 1	BD	25/09/2026	SM

As per RITS: Storage sized on 9hrs ADWF

Public Stage 1 Pump Station (Catchment A)

PWWF=	2.47	l/s
ADWF =	37.20	m3/day
Storage Required =	13.95	m3

Private Pump Station (Catchment B)

PWWF=	11.51	l/s
ADWF =	148.40	m3/day
Storage Required =	55.65	m3

Appendix E – Geotechnical Reports

Appendix F – Stormwater Management Plan (SMP)

Appendix G – BTW/WPS Wastewater Growth Modelling Report

To:	Te Awamutu Developments Limited		
From:	Mathew Dickey (BTW Company Limited)		
Date:	6th December 2022	**BTW Job Number:**	211365

Subject: Wastewater Servicing Assessment – Residential Subdivision - 2025 Ōhaupō Road

EXECUTIVE SUMMARY

BTW Company Limited (BTW) have been engaged by Te Awamutu Developments Limited, to undertake a wastewater servicing assessment of the property located at 2025 Ōhaupō Road, Te Awamutu, to determine the feasibility of servicing a proposed residential lifestyle village subdivision via the existing Waipā District Council (WDC) wastewater reticulation.

The wastewater modelling for this assessment was completed by WSP consultants, with development flow information, preliminary reticulation layout, and a site Wastewater Pumpstation (WWPS) location provided by BTW to inform the modelling. The outcome of WSP's modelling assessment is attached as Appendix A, with the key outcomes and upgrades recommended from the modelling summarised below.

Modelling Conclusions and Recommendations (Adopted from WSP's Report - Appendix A):

WSP completed a wastewater modelling assessment, delivered through a two-stage modelling process in consultation with WDC and BTW. Overall, to service the wastewater flows for this development, there are two feasible options:

1. Option 1: Discharge all of the new site development flows to the existing gravity network that discharges into the Racecourse Road WWPS and upgrade an approximately 1 km length of existing 225 mm diameter wastewater gravity pipe to a 300 mm diameter size downstream of the Racecourse Road WWPS, as located in Figure 1.
2. Option 2: Discharge 80% of the development flows (from the proposed site WWPS) to the Christie Avenue WWPS conveyance route, with the remaining development flows gravity discharging to the Racecourse Road WWPS conveyance route. This would require the new site WWPS rising main to extend nominally 640 m along Ōhaupō road from the site to the existing wastewater manhole near the intersection of Ōhaupō Road and Racecourse Road (Manhole Asset ID 1090999).

Both options above result in no predicted overflows or freeboard deficiencies (for both the existing and growth network models), therefore complying with WDC's assessment criteria and presenting as feasible options to service the development wastewater flows. The modelling also concludes that the Te Awamutu Wastewater Treatment Plant (WWTP) will be able to cater for the Ōhaupō Road development flows along with the planned growth cells in the 2050 network.

Selection of the preferred option will occur in subsequent design phases. Additionally, as part of the subsequent design phases, it is recommended to conduct a detailed pumpstation capacity assessment to confirm the capacity of the downstream pumpstations (Racecourse Road WWPS and/or the Christie Avenue WWPS) and their capacity to manage for the proposed development flow.

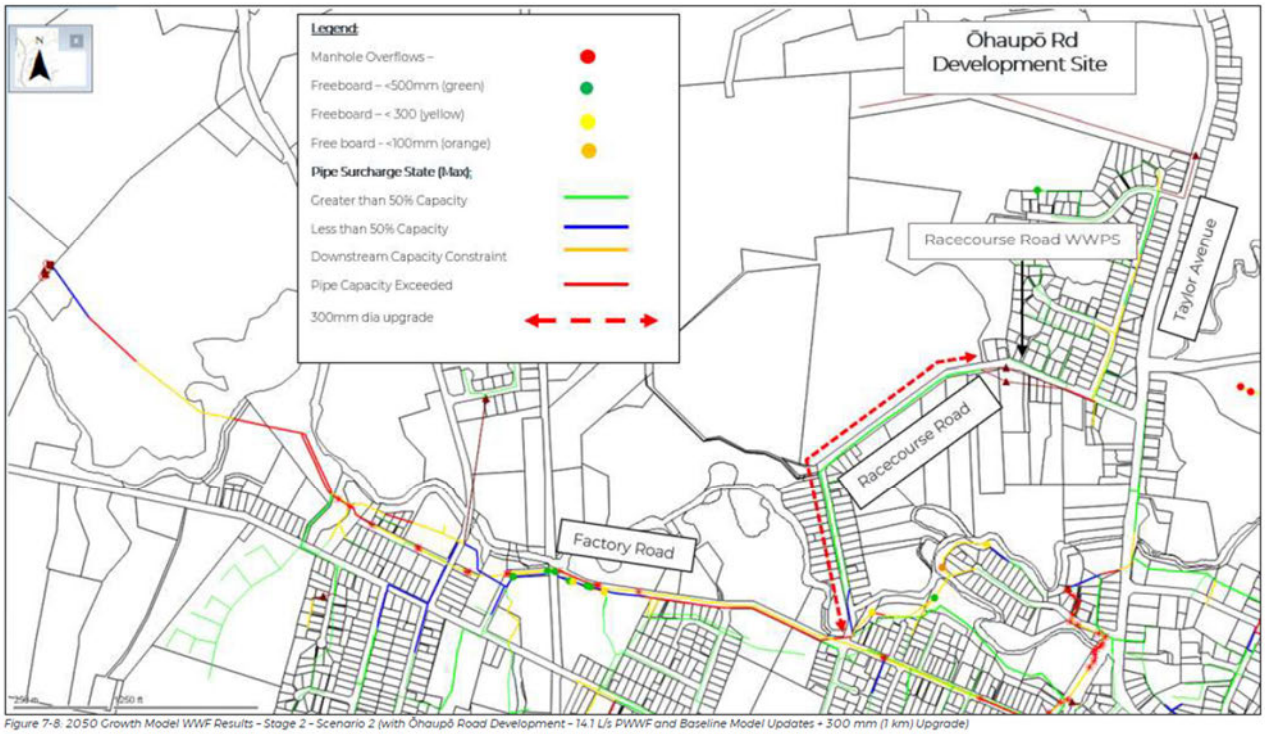


Figure 7-8: 2050 Growth Model WWF Results - Stage 2 - Scenario 2 (with Ōhaupō Road Development - 14.1 L/s PWWF and Baseline Model Updates + 300 mm (1 km) Upgrade)

Figure 1: Te Awamutu wastewater modelling output showing the section of existing wastewater pipe to upgrade for Option 1 described above (figure extract from Appendix A).

SITE LOCATION AND DEVELOPMENT INFORMATION

The site is located near the northern town boundary of the Te Awamutu township, as shown in Figure 2. The property assessed is currently in two Lots (Lot 1 DPS 36696 and Part Lot 1 DP 35654). The full site area (of both Lots) is approximately 25.78 Ha total.

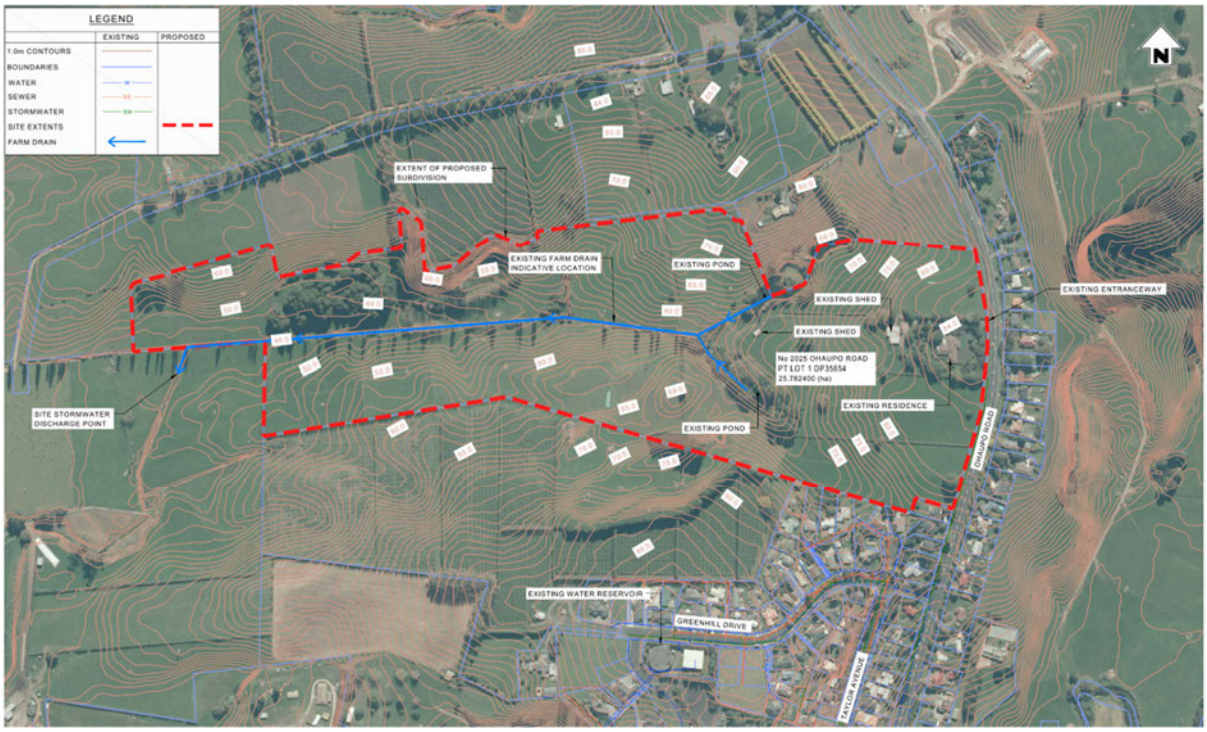


Figure 2: Site plan showing the site location and extents

The site is proposed to be developed into residential housing, with a preliminary estimate of up to 500 residential dwellings proposed by Te Awamutu Developments Limited for the purposes of this wastewater servicing assessment.

EXISTING WASTEWATER INFRASTRUCTURE AND CONVEYANCE TO THE WWTP

The existing wastewater network near the proposed development site is shown in Figure 3.The wastewater discharged from the site will likely be directed west to the manhole on Greenhill Drive (Manhole Asset ID 1090025), which is connected to a 150 mm sewer. Flows once discharged to the sewer network, will travel south along Taylor Avenue, before ultimately arriving at the Te Awamutu Wastewater Treatment Plant (WWTP) in the west of the catchment, approximately 4 km downstream of the initial discharge point. An alternative option for conveying site flows to the Christie Avenue WWPS is also shown in Figure 3.

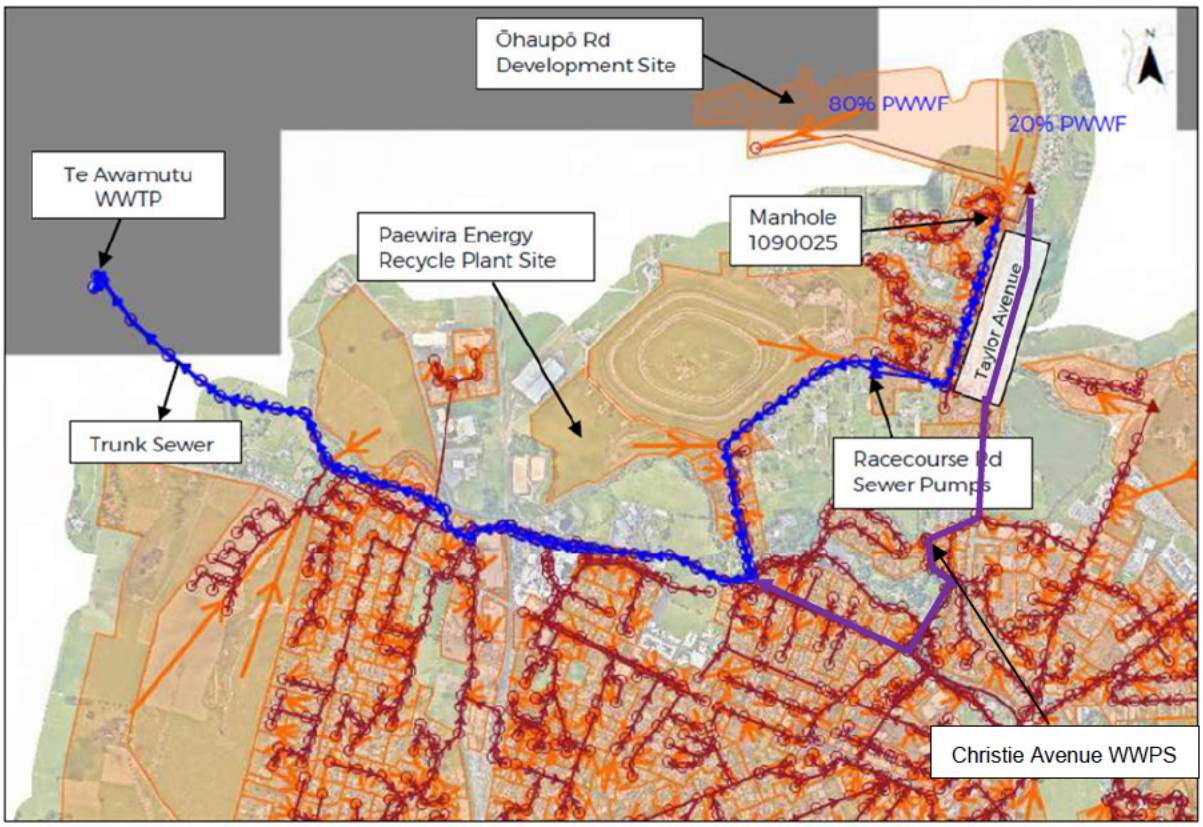


Figure 3: Catchment location plan (figure extract from Appendix A). Conveyance pathway for site wastewater flows to the WWTP shown as blue. Alternative Christie Avenue WWPS conveyance pathway shown as purple.

PRELIMINARY SITE DEVELOPMENT WASTEWATER INFRASTRUCTURE LAYOUT

A wastewater servicing schematic for the site is shown in Figure 4. The site is assessed to have an elevated area on the eastern side, which initially appears to be feasible to dispose of wastewater via gravity sewer, discharging into the existing wastewater gravity reticulation on Greenhill drive (yellow shaded area). The remainder of the site (blue shaded area) would likely have an internal gravity reticulation flowing into a new WWPS located on the western side of the site, correlating to the area with the lowest site elevation. The WWPS would pump site wastewater via a rising main, to either discharge into the existing Greenhill Drive gravity wastewater network, or alternatively via a new connection to the Christie Avenue WWPS gravity network on Ōhaupō Road.

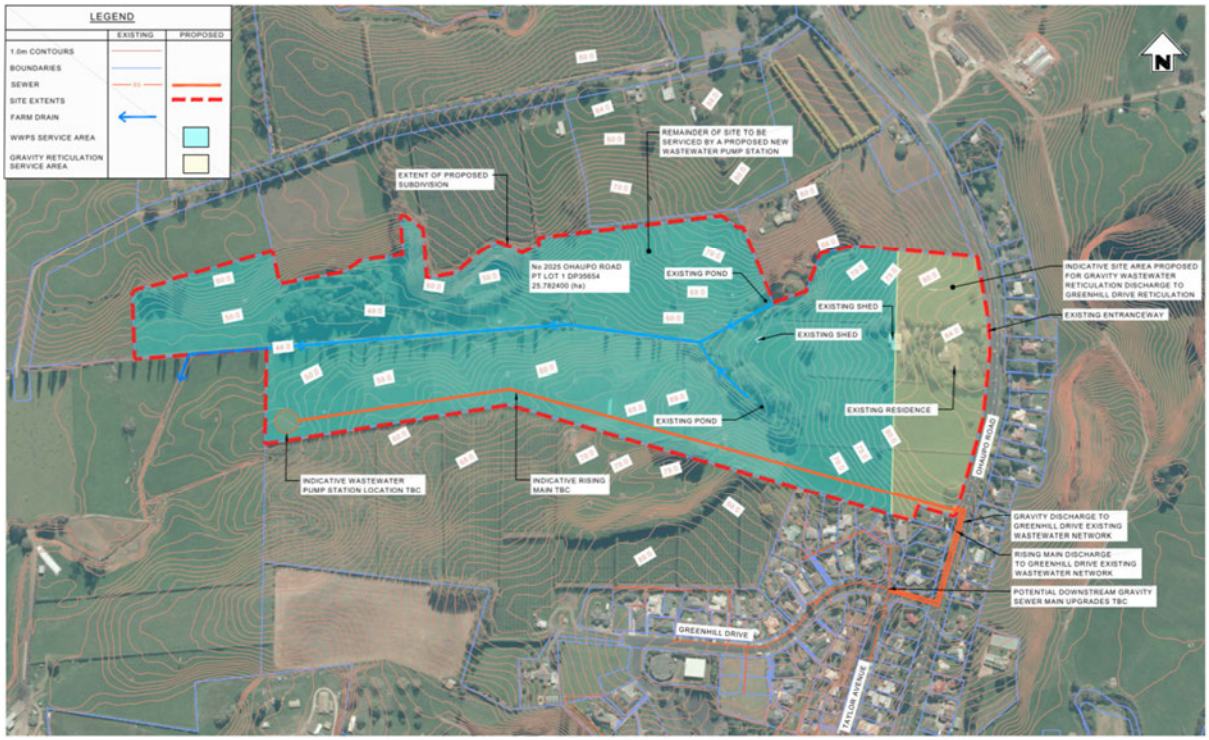


Figure 4: Wastewater disposal schematic to service the proposed site development.

WASTEWATER DESIGN PARAMETERS

All design parameters were generally taken from the WLASS RITS, Section 5 Wastewater, version May 2018. Where suitable parameters did not exist in the RITS, then parameters were taken from NZS 4404:2010 Land Development and Subdivision Infrastructure.

Table 1 describes some of the key assumed parameters used in this wastewater assessment:

Table 1: Key design parameters assumed.

Parameter	Value required by specification	Reference specification
Design life	100 years	RITS 5.2.1
Average Daily Flow (Domestic)	200 litres per person per day	RITS 5.2.4.2
Infiltration allowance	2,250 litres per hectare per day	RITS 5.2.4.2
Surface water ingress allowance	16,500 litres per hectare per day	RITS 5.2.4.2
Peaking factor	As per specific value in Table	RITS Table 5-2
Population equivalent	As per specific value in Table	RITS Table 5-3
Persons per dwelling	2.7	RITS Table 5-3 / NZS 4404:2010 5.3.5.1
ADF, PDF and PWWF	As per formulas	RITS Equations 5-1, 5-2, 5-3

WASTEWATER DISPOSAL – ESTIMATED SITE FLOWS

Wastewater flows for the Peak Wet Weather, Average Daily and Peak Daily Flow scenarios were calculated using the following formulae obtained from section 5.2.4.2 of RITS, as shown in Figure 5.

Average Daily Flow (ADF)

The Average Daily Flow is calculated as the sum of the infiltration allowance and the daily wastewater flow:

Equation 5-1: Average daily flow (ADF)

ADF (m^3/day) = (infiltration allowance × catchment area) + (water consumption × population equivalent)

Peak Daily Flow (PDF)

The system shall achieve a daily self-cleaning velocity the Peak Daily Flow.

Equation 5-2: Peak daily flow (PDF)

PDF (L/sec) = ((infiltration allowance × catchment area) + (peaking factor × water consumption × population equivalent)) ÷ 86400

Peak Wet Weather Flow (PWWF)

The system shall accommodate the design Peak Wet Weather Flow without surcharge.

Equation 5-3: Peak wet weather flow (PWWF)

PWWF (L/sec) = ((infiltration allowance × catchment area) + (surface water ingress × catchment area) + (peaking factor × water consumption × population equivalent)) ÷ 86400

Figure 5: Extract from RITS: Section 5.2.4.2

Allowing for 15% stormwater reserve area, the wastewater catchment area is estimated at 21.9 Ha. The ADF, PDF and PWWF estimated for the site is calculated as follows:

$$ADF = (2{,}250\ L/Ha/day\ x\ 21.9\ Ha) + (500\ Dwellings\ x\ 2.7\ People\ x\ \ 200\ L\ /\ person\ /\ day)$$

$$ADF = 49.3\ m^3/day + 270\ m^3/day$$

$$ADF = 319.3\ m^3/day$$

$$PDF = \frac{(2250\ L/Ha/day\ x\ 21.9\ Ha) + (3.0\ PF\ x\ 500\ Dwellings\ x\ 2.7\ People\ x\ \ 200\ L\ /\ person\ /\ day)}{86{,}400}$$

$$PDF = 9.95\ L/sec$$

$$PWWF = \frac{(2250\ L/Ha/day\ x\ 21.9) + (16{,}500\ x\ 21.9) + \ (3.0\ \ PF\ x\ 500\ Dwellings\ x\ 2.7\ People\ \ x\ \ 200\ L\ /\ person\ /\ da}{86{,}400}$$

$$PWWF = \frac{(49{,}275\ L\ /\ day) + (361{,}350\ L/day) + \ (810{,}000\ L/day)}{86{,}400}$$

$$PWWF = 14.1\ L/sec$$

WASTEWATER SUPPLY ASSESSMENT AND MODELLING RESULTS

Overall, the wastewater servicing assessment and hydraulic modelling completed has confirmed that servicing wastewater flows for the proposed residential lifestyle village development via the Te Awamutu town wastewater network (allowing for 500 residential dwellings) is feasible through either of the two options listed below:

1. Option 1: Discharge all of the new site development flows to the existing gravity network that discharges into the Racecourse Road WWPS and upgrade an approximately 1 km length of existing 225 mm diameter wastewater gravity pipe to a 300 mm diameter size downstream of the Racecourse Road WWPS, as located in Figure 1.
2. Option 2: Discharge 80% of the development flows (from the proposed site WWPS) to the Christie Avenue WWPS conveyance route, with the remaining development flows gravity discharging to the Racecourse Road WWPS conveyance route. This would require the new site WWPS rising main to extend nominally 640 m along Ōhaupō road from the site to the existing wastewater manhole near the intersection of Ōhaupō Road and Racecourse Road (Manhole Asset ID 1090999).

Both options above result in no predicted overflows or freeboard deficiencies (for both the existing and growth network models), therefore complying with WDC's assessment criteria, and presenting as feasible options to service the development wastewater flows. The modelling also concludes that the Te Awamutu Wastewater Treatment Plant (WWTP) will be able to cater for the Ōhaupō Road development flows along with the planned growth cells in the 2050 network.

Selection of the preferred option will occur in subsequent design phases. Additionally, as part of subsequent design phases, it is recommended to conduct a detailed pump station capacity assessment to confirm the capacity of the downstream pump stations (Racecourse Road WWPS and/or the Christie Avenue WWPS) and their capacity to manage for the proposed development flow.

Refer to Appendix A for a detailed report of the wastewater servicing modelling results, recommendations, and conclusions.

APPENDIX A WASTEWATER MODELLING ASSESSMENT (WSP 2022)

Memorandum

To	Mathew Dickey
Copy	James Cassidy, Charlotte Mills, Stephanie Cheevers, Melissa Allfrey, Harry Baxter
From	Haiming Li
Office	Hamilton
Date	10 November 2022
File/Ref	3-39619.00
Subject	Wastewater Hydraulic Assessment – 2025 Ōhaupō Road Development
Status	Stage 1 and 2 Report (Final)

1 Introduction

Ultimate Holdings Limited (UHL) is proposing a private plan change residential subdivision at 2025 Ōhaupō Road, Te Awamutu, as shown in Figure *1-1*. The development site is not within the current Waipa District Council's planned growth areas (not in the 2050 masterplan model).

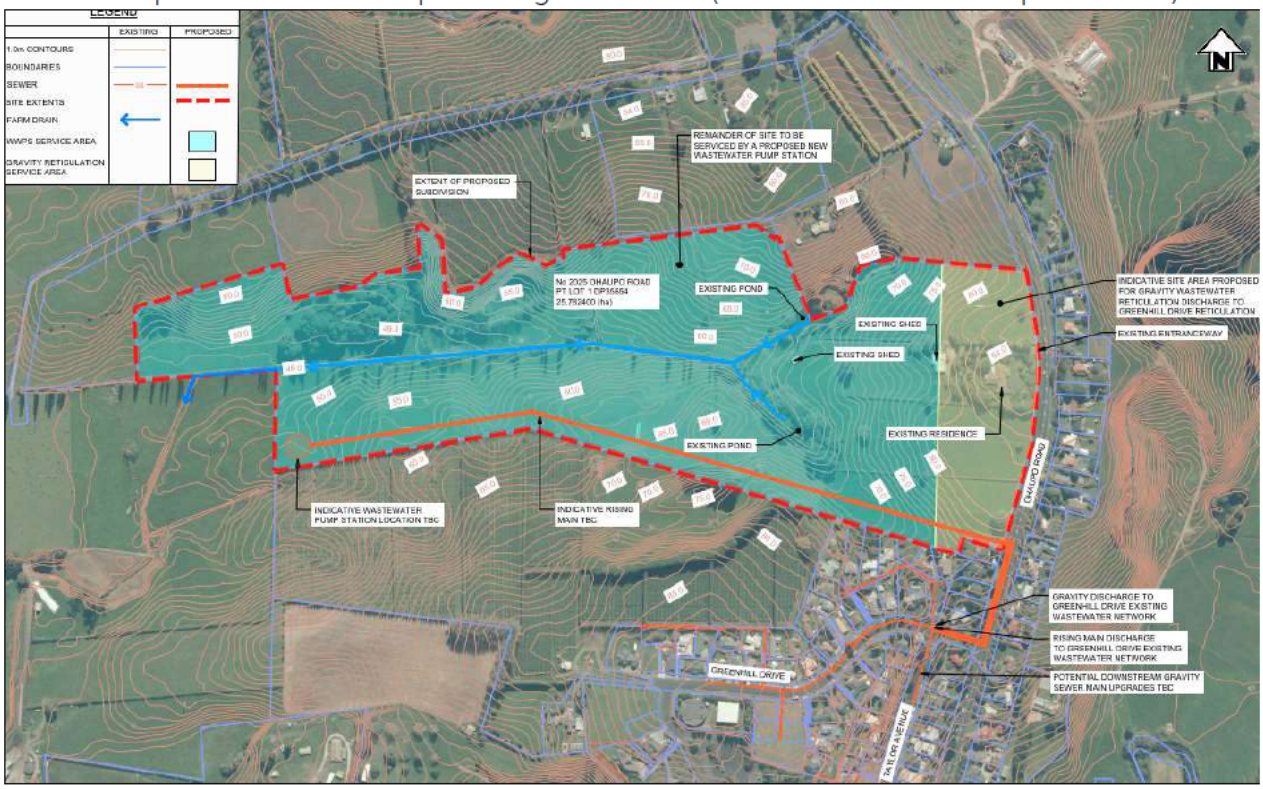


Figure 1-1 : Location of Development

There are two stages to this project. Stage 1 is to assess the impact of the proposed development on the current/future wastewater (WW) networks and Stage 2 is an options

WSP
Christchurch
12 Moorhouse Avenue
Christchurch 8011
New Zealand
+64 3 363 5400
wsp.com/nz

150 YEARS IN AOTEAROA

assessment based on the modelling results from Stage 1. This memorandum details both stages assessments and outcomes. BTW Company Ltd. (BTW) have engaged WSP on this project on behalf of the client (UHL).

1.1 Stage 1 – Impact Assessment

The objectives of Stage 1 assessment are as follows, using the Waipa District Council (WDC) 2022 wastewater base model (existing network) and 2050 wastewater future growth model (growth network), separately:

1 Scenario 1 - Assess the impact of all the 2050 growth masterplan demands on the 2022 model as a baseline scenario in the existing model network under the 5-year ARI design event. The objective is to assess the capacity of the existing pipe network to accommodate for the planned future developments.

2 Scenario 2 - Assess the impact of all the 2050 growth masterplan demands and the Ōhaupō Road development demand in the existing network under the 5-year ARI design event, using a development peak wet weather flow (PWWF) of 14.1 L/s. Flows will be discharged at manhole node 1090025. The objective is to assess the capacity of the existing pipe network to accommodate for the future developments and the Ōhaupō Road development.

3 Scenario 3 - Assess the impact of the Ōhaupō Road development on the growth network (with 2050 masterplan upgrades included) in the 5-year ARI design event, using a development peak wet weather flow (PWWF) of 14.1 L/s, discharging flows at manhole node 1090025. The objective is to assess if there is capacity in the 2050 pipe network to accommodate for the Ōhaupō Road development.

4 Conduct a capacity assessment of the wastewater treatment plant (WWTP) in the growth network (Scenario 3), using (daily) volumes and instantaneous peak flow rates only.

As informed by WDC, WSP has included the flow from the proposed Paewira energy recycle plant (at the border of T13 growth cell) in this assessment. Table 1-1 lists the PWWFs for these two developments. The Paewira discharge point is downstream of the Racecourse Road pump station.

Table 1-1: Development Flows

Development Scenario	Peak Wet Weather Flow (L/s)
2025 Ōhaupō Road Development (Scenarios 2 & 3)	14.10
Paewira Energy Recycle Plant (Scenarios 1, 2 & 3)	0.14

1.2 Stage 2 – Optioneering Assessment

Firstly, the objective of Stage 2 is to update the model with the following, based on the Stage 1 assessment results and the discussion with BTW and WDC:

- Adjust the proposed WWPS parameters as recommended by BTW as per email dated 23/08/2022.
- Update the 2022 base model and the 2050 growth model with the latest Racecourse Road WWPS information.
- Resolve the negative pipe gradings by interpolation.

The above items are discussed in more detail in Section 6.

After the above updates are made, the following optioneering scenarios will be run to access if the Te Awamutu wastewater network is able to accommodate for the Ōhaupō Road development flow without causing unplanned sewer overflows or unacceptable residual freeboard level.

- Scenario 1: 2022 base model network and 2050 growth model network both with the Ōhaupō Road development with the above three model updates (baseline).
- Scenario 2: Increase the Racecourse Road WWPS pump flow rate and upgrade the downstream pipe network if needed (in Stage 2 Scenario 1 network).
- Scenario 3: Change the Ōhaupō Road development to discharge 80% of its proposed flow to the sewer main (asset ID: 1331069) on Ōhaupō Road (upstream of Christie Avenue WWPS), with 20% draining via gravity to manhole 1090025 (the originally proposed discharge point located at the upstream of Racecourse Road WWPS) in Stage 2 Scenario 1 network.

1.3 Waipa District Council Overflow and Freeboard Criteria

WDC has advised the following assessment level of service criteria:

- no unplanned sewage overflows, and;
- no manholes with freeboard < 300 mm

2 Catchment Description

The existing wastewater network near the proposed development site is shown on Figure 2-1. The wastewater discharged from the site will be directed west to the manhole (asset ID: 1090025) on Greenhill Drive, which is connected to a 150 mm sewer. Flows once discharged to the sewer network will travel south along Taylor Avenue, before ultimately arriving at the Te Awamutu wastewater treatment (WWTP) in the west of the catchment, approximately 4 km downstream of the initial discharge point.

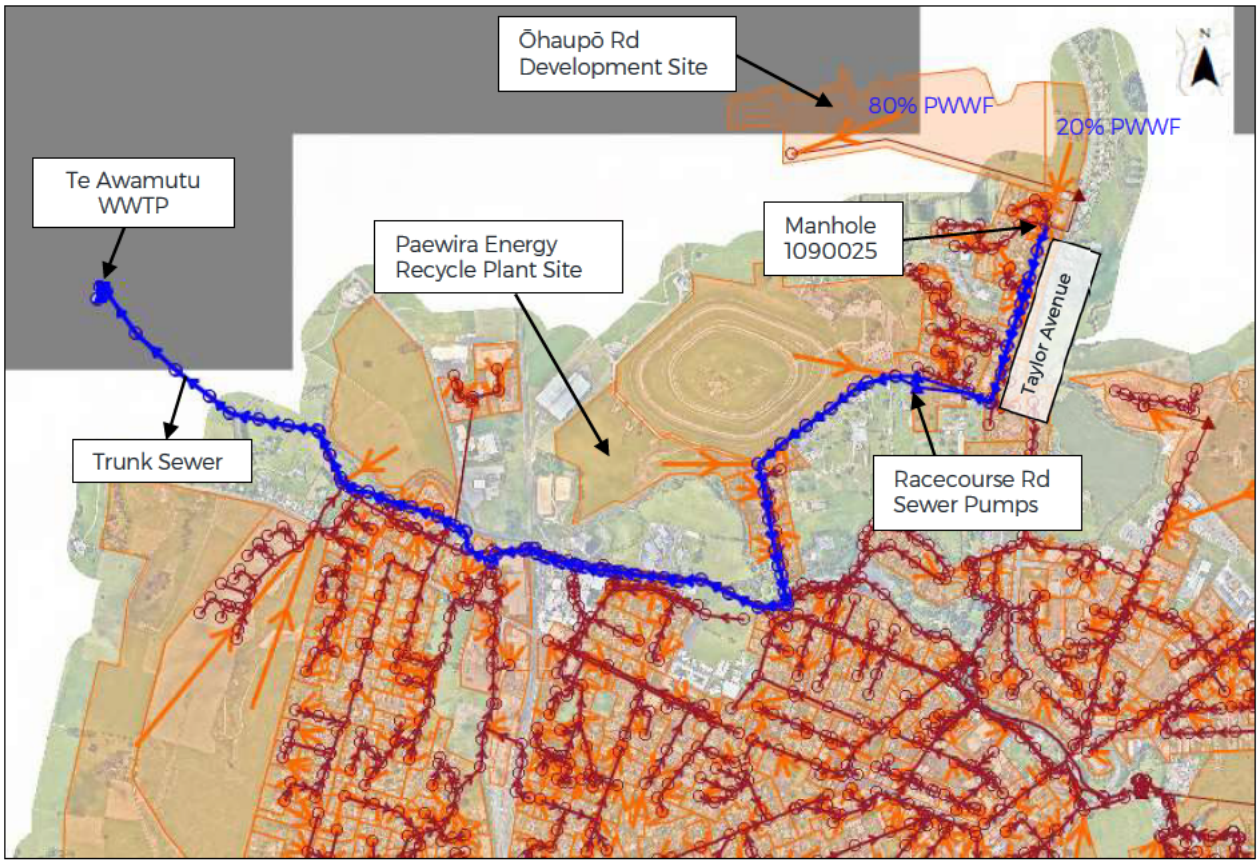


Figure 2-1 : Catchment Location Plan (downstream pipes shown in blue)

3 Assessment Methodology

The *2022 Te Awamutu* network model was applied in this investigation. This model was built and calibrated to observed flows in July 2019, with minor updates to the model pipe network completed in April 2020. There were three model updates after the model build and calibration and Table 3-1 summarises the previous model update details.

Table 3-1: Previous Model Update Details

Model Update	Scope Details
2020	WW assets constructed up to March 2020 and calibrated to 2019 network flows.
2021	WW assets constructed up to July 2021 and calibrated to 2019 network flows.
2022	WW assets constructed up to April 2022 with a calibration review of flow data between June and August 2021.

The *2050 Te Awamutu* network model was also used in this investigation and includes future growth cells and planned network upgrades up to 2050 (as part of the 2020 master planning work).

The following methodology was completed to assess the impact of the proposed development on the wastewater networks.

3.1 Stage 1 - Existing Network Assessment

- Model scenarios 1 and 2 were created in the existing *Te Awamutu 2022* model network (WSP model reference: Waipa WW Models (2019)). All modelling was completed using InfoWorks ICM version 9.0.
- The model scenarios were developed from the *base* network with all future growth cells and the Paewira energy cycle plant development being added.
- As per BTW's instruction, 80% of the PWWF provided for the development was discharged to the proposed development wastewater pumping station and then to manhole 1090025. The remaining 20% of the PWWF was loaded directly to manhole 1090025.
- Simulations for both the *base* network and network with the inclusion of flows from the development site were run to assess the impact of flows on the network for the WWF scenario only. Model simulations were run with WDC's design event for wastewater networks (2 hour 1 in 5-year average recurrence interval).

3.2 Stage 1 - Future Growth Network Assessment

- Model scenario 3 was created in the existing *Te Awamutu 2022* model network (WSP model reference: Waipa WW Models (2019)). All modelling was completed using InfoWorks ICM version 9.0.
- The model scenarios were developed from the *2050 Network and Growth* model with the Paewira energy cycle plant development, where all future growth cells and network upgrades were applied.
- As per BTW's instruction, 80% of the PWWF provided for the development was discharged to the proposed development wastewater pumping station and then to the manhole 1090025. The remaining 20% of the PWWF was loaded directly to manhole 1090025.
- Simulations for both the *2050 Network and Growth* network and the network with the inclusion of flows from the development site were run to assess the impact of flows on the network for the WWF scenario. Model simulations were run with

WDC's design event for wastewater networks (2 hour 1 in 5-year average recurrence interval).

3.3 Stage 1 - Wastewater Pumping Station Parameters

Table 3-2 summarises the sewer pump and wetwell details for the proposed development in Stage 1.

Table 3-2: Modelled Ōhaupō Sewer Pump and Wetwell Details

Wetwell		Notes:
Diameter (m):	1.8	As per the minimum diameter specified in RITS.
Depth (m):	4.0	Same as Tokanui Wetwell (WSP previous project).
Ground Level (m AD):	51.0	From the surface data provided by BTW.
Roof Level (m AD):	51.2	0.2 m above the ground level to avoid rainwater getting in and people will not reverse their car on the lid.
Floor Level (m AD):	47.2	4.0 m below the roof level.
Pump		Notes:
Pump Rate (L/s):	12.0	80% of the development PWWF.
On Level (m AD):	48.2	1.0 m above wetwell floor level.
Off Level (m AD):	47.8	0.6 m above wetwell floor level.

3.4 Analysis

The following analysis has been undertaken for Section 3.1 and 3.2;

- Potential network constraints downstream of the proposed development connection were assessed.
- The assessment of the impact of the proposed development has been undertaken by comparing the wastewater network performance pre- and post-development in terms of:
- Overflows,
- Pipe filling: when the maximum pressure (Hmax) is higher than the pipe diameter (Diam), the pipe is surcharged by downstream capacity constraints, and,
- Pipe capacity: When the maximum flow (Qmax) is higher than the maximum theoretical capacity (Qcap), the pipe capacity is exceeded.
- Pump station capacity assessment on Racecourse Road pump station by comparing the pump rate and the predicted PWWF to the pump station.
- Table 3-3 below summarises the different pipe conditions.

Table 3-3: Gravity Pipe Conditions

Pipe State	Criteria
Free Flow	(Qmax/Qcap < 0.5 AND Hmax/Diam < 0.5)
Free Flow	(Qmax/Qcap < 1 AND Hmax/Diam < 1)
Downstream Capacity Constraint	(Qmax/Qcap < 1 AND Hmax/Diam > 1)
Pipe Capacity Exceeded	(Qmax/Qcap > 1 AND Hmax/Diam > 1)

4 Assumptions

The following assumptions have been made in completing this work:

- The *2022 Te Awamutu* model represents the best available data for Te Awamutu wastewater network.
- The provided PWWF from the development site has been applied to the network as a constant flow. Flows from the development have been split into two discharges:
    - 80% of the flow has been assumed to be pumped and 20% of the flow will drain by gravity to Ōhaupō Road. The connection point from both discharges will be to the same manhole (Figure 2-1).
- This assessment was completed for existing and future growth scenarios, which include all growth cells and the Paewira Energy Recycle Plant development as per WDC request. For the future growth scenario, the model includes proposed upgrades from the Waipa Three Waters Master Plan 2020 and additional growth cells.
- As per the OOS, WSP conducted a capacity assessment of the wastewater treatment plant (WWTP), using (daily) volumes and instantaneous peak flow rates only.

5 Stage 1 - Model Results

Result maps (Figure 7-1 - Figure 7-4) are attached in Appendix A.

5.1 Stage 1 - Scenarios 1 & 2 – Existing Network (WWF)

Figure 7-1 (Appendix A) provides an overview of the model WWF performance with no flow contribution from the Ōhaupō Road development. Figure 7-2 provides an overview of the model performance with a constant flow contribution of 14.1 L/s (PWWF) from the Ōhaupō Road development. The following is a high-level overview of the model results:

- Figure 7-1 (Appendix A) shows surcharge that impacts the trunk sewer for some extents of Factory Road. However, no overflow is predicted.
- With the additional flow contribution of the 14.1 L/s from the Ōhaupō Road development, the number of pipes predicted to surcharge increases by 10. The increase in surcharge is general located on Taylor Avenue and Racecourse Road, which is downstream of the proposed development discharge point on Taylor Avenue. Refer to Figure 7-2 (Appendix A) for the locations of surcharge increases.
- The number of sewer overflows are increased by 3 in the post-development network.
- Table 5-1 provides a high-level overview of the model results discussed above.

There are some pipes with negative grades found at the downstream network (shown in Figure 7-1), which caused constraints in the network. The flag data source of the negative grade pipes were also reviewed. It was identified that the invert levels were either inferred or interpolated from the invert levels of the connected manholes or from the GIS data.

There is also a WWPS at the downstream of the proposed development (refer to Figure 2-1 for details) on Racecourse Road. The duty pump rate is 14 L/s with the current predicted PWWF of 11.5 L/s in Scenario 1 (pre-development). With the proposed development flow, the predicted PWWF of this pump is increased to be 25.6 L/s, which exceeds the current duty pump rate. The total capacity of the pump station with two pumps running is 24 L/s, which is also below the predicted PWWF, however, no overflows are predicted upstream of the pump station.

5.2 Stage 1 - Scenario 3 - Future Growth Network (WWF)

An assessment was undertaken, using the future growth model, to assess if the future network performance with the Ōhaupō Road development flow of 14.1 L/s. It is noted this assessment includes all currently proposed future masterplan developments and upgrades in the Te Awamutu catchment. The pump rate of the Te Awamutu WWTP is also increased in the growth network to accommodate for future masterplan developments. The following is a high-level overview of the model results:

- Figure 7-3 (Appendix A) shows that there is no unplanned overflows in the pre-development 2050 network.
- Figure 7-4 (Appendix A) shows that surcharge impacts the trunk sewer along Factory Road. There is slight improvement in surcharge from the existing network scenario due to the proposed upgrade to the pump at the WWTP inlet. The network on Racecourse Road is also suffers from extensive surcharge.
- Three overflows are predicted on Racecourse Road, which are at downstream of the Racecourse Road sewer pump station. The surcharge state of the trunk sewer is also increased.
- In the 2050 network, there is no masterplan upgrade to the Racecourse Road pump station and no increase in flows from its upstream network. Therefore, the predicted PWWF inflow to this pump station is still above its duty rate as per the scenario 2 result.

Table 5-1: Simulation Results Overview

Scenario	Increase in Number of Overflows (No.)	Increase in Overflow Volume (m^3)	Increase in Number. of Surcharged Pipes (No.)
2022 Network (WWF) + 14.1 L/s Ōhaupō Flows	3	45	11
2050 Network (WWF) + 14.1 L/s Ōhaupō Flows	3	45	13

5.3 Te Awamutu WWTP Capacity Review (2050 Growth Scenario)

The capacity of the Te Awamutu WWTP was reviewed, using the daily volumes and instantaneous peak flows for the 2050 growth scenario. Table 5-2 below summarises the review results. Both modelled daily volume and modelled instantaneous flows are predicted to be below the designed parameters. This suggest the WWTP will be able to cater for the Ōhaupō Road development flow along with the planned growth cells.

Table 5-2: Te Awamutu WWTP Capacity Review

	PWWF (m^3/day)	Instantaneous Peak Flow (L/s)
Designed Flows to WWTP (2050 projection)	24,256	388
Modelled Flows to WWTP with Ōhaupō Flows (scenario 3 - 2050 growth network)	14,667[1]	357
1. The 5-year 2-hour rainfall event is the critical duration for the system based on the instantaneous flow to WWTP. But it's not necessarily the worst-case volume for WWTP.		

Te Awamutu WWTP is designed for an instantaneous PWWF of 388 L/s into its inlets work. However, the treatment is limited to 21,000 m^3/day through the WWTP due to the consented maximum daily discharge to the Mangapiko stream (currently 21,000 m^3/day or 280 L/s). The

influent PWWF will be attenuated by the Wastewater Overflow Ponds, if the flow is greater than the maximum daily discharge.

6 Stage 2 – Model Results

The following section provide an overview of the model updates and optioneering work undertaken as part of Stage 2 of the project.

Stage 2 Optioneering assessment result maps (Figure 7-5 - Figure 7-10) are attached in Appendix A.

6.1 Post-Stage 1 Model Updates

A meeting was held with WDC and BTW to discuss the Stage 1 model results on 2 September 2022. During the meeting, the following model updates were agreed for the baseline model as part of the Stage 2 assessment:

- Update, where practicable, the 2022 base model and the 2050 growth model with the latest Racecourse Road WWPS operation data provided by WDC. Refer to Table 6-1 for details.
- Resolve the negative pipe grades by interpolation, between node 1092710 and node 1090126 on Racecourse Road and between node 1093690 and node 1091022 on Factory Road . Refer to *Figure 6-1* and *Figure 6-2* for their locations.

Table 6-1: Racecourse Road Pump Station Update

Pump Rate:		Notes:
Duty Pump:	27 L/s	The duty pump was previously set at 14 L/s

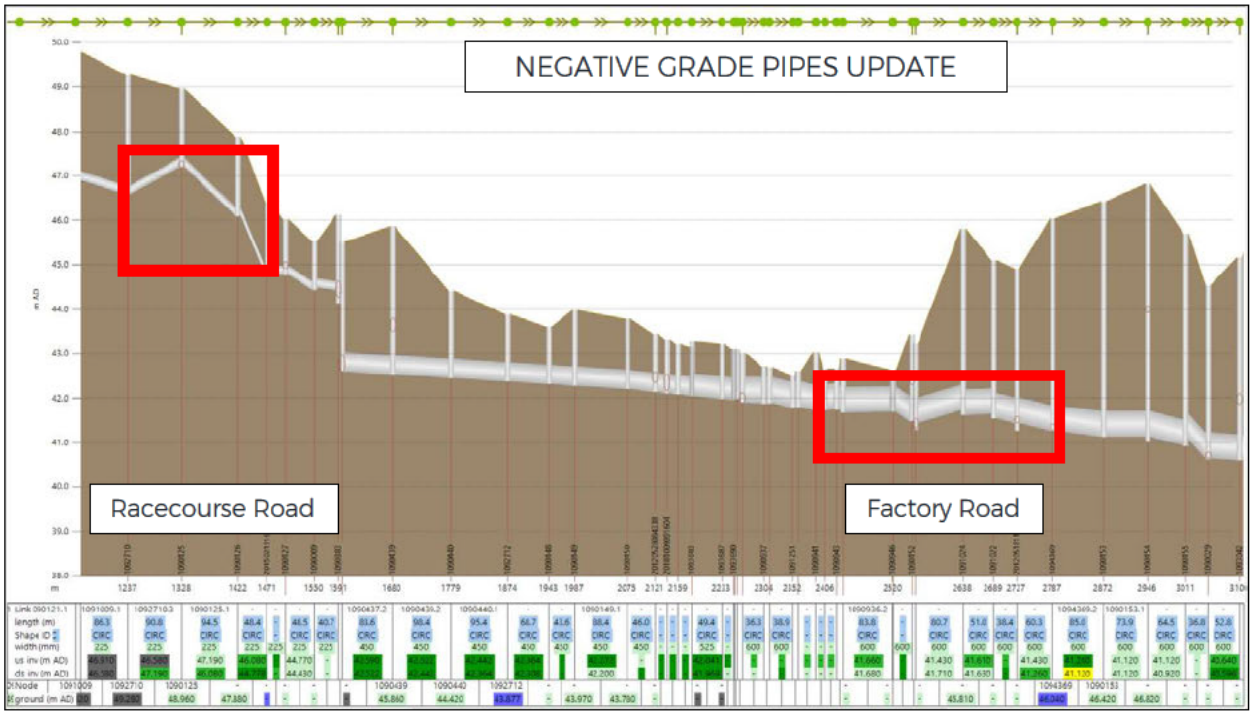


Figure 6-1: Negative Grade Pipes Downstream of the Ōhaupō Road Development Before Model Update